Calibrating Distribution System Models with Fire-Flow Tests

by Walter M. Grayman, Morris L. Maslia, and Jason B. Sautner

Fire-flow tests, a widely used method for estimating the available fire flow from hydrants, are also frequently used in the calibration process for a hydraulic water-distribution system model to determine roughness coefficients and to find closed valves. The process is straightforward: A hydrant is opened and water is released to increase flows in the distribution system in the vicinity of the hydrant.

Figure 1. Hydrants were equipped with an analog pressure gauge (left) and a continuous recording pressure (digital) logger (right).

Because friction or head losses increase exponentially (to the power of 2 with the Darcy-Weisbach friction equation and to the power of 1.85 in the Hazen-Williams equation), higher flows intensify such losses, resulting in a pronounced lowering of the hydraulic grade line (HGL). With the HGL lowered, modeling results are more sensitive to roughness coefficients, which, in turn, allows for ease and accuracy in determining roughness coefficients.

Test Condition	Flow, *gpm*		Pressure, *psi*			
	Q_1	Q_2	P_{00}	P_{01}	P_{02}	P_{03}
Starting static case	0	0	53.1	50.7	56.2	52.6
Hydrant 1 flowed	773	0	41.4	37.3	46.8	42.9
Hydrant 1 and 2 flowed	631	579	29.7	24.5	36.5	32.7
Hydrant 2 flowed	0	747	43.9	40.7	48.1	44.1
Ending static case	0	0	53.5	51.2	56.5	52.9

Table 1. Sample results from fire-flow test. Q_1 and Q_2 refer to the discharge from the two hydrants that are being flowed. P_{00}, P_{01}, P_{02}, and P_{03} refer to the pressure measurements at the four hydrants.

The model is applied to flow and operational conditions experienced during a fire-flow test; the pressures or HGL observed in the field are compared to the model results. If significant differences are found between the model and field results, adjustments are made in model parameters to reduce the differences or "calibrate" the model. Adjustments are typically made in the roughness coefficient (e.g., Hazen-Williams *C* factor), demands, or shutoff valve positions. They can be made manually through trial and error or through automated optimization programs that systematically examine a wide range of coefficients and choose the combination that best fits the observed field data.

Walter M. Grayman is a consulting engineer in Cincinnati. Morris L. Maslia is a research environmental engineer for the Agency for Toxic Substances and Disease Registry in Atlanta. Jason B. Sautner is an environmental health scientist with the same agency.

In most traditional fire-flow tests, water is released from a single hydrant and pressure is measured at another hydrant. If the single hydrant does not sufficiently stress the system (i.e., produce enough head loss) additional hydrants can be opened simultaneously to further lower the HGL. Also, pressure and HGL measurements can be made at additional hydrants to provide more data for use in the calibration process. However, opening more hydrants and making more measurements usually require more personnel or a longer time between tests so crews can travel to the next hydrant.

Alternative Approach to Flow Testing

An alternative approach for conducting a fire-flow test was developed as part of an ongoing study at the US Marine Corps Base Camp Lejeune, N.C. The enhanced test procedure was developed to improve labor efficiency associated with conducting fire-flow tests and to collect additional data for calibration purposes.

Continuously recording pressure gauges (Figure 1) were installed at up to six hydrants in the test area. These gauges were set to record a pressure measurement at 1-min intervals. Pitot gauges were also installed on two hydrants designated as flow hydrants. An integrated Pitot tube and wire-cage diffuser was used to measure and control the discharge from the hydrant

(Figure 2). At the hydrant nearest to the flowing hydrants, an analog pressure gauge was installed in addition to the continuous-recording pressure logger so that the pressure drop could be visually monitored to be sure it dropped sufficiently, but not below 20 psi.

During the test, five different flow conditions were studied, each for a period of 3 to 4 min:

- Static conditions (no water flowed from hydrants)
- Water flowed from hydrant number 1 (Q_1)
- Water flowed from hydrant numbers 1 and 2 (Q_1 and Q_2) simultaneously
- Water flowed from hydrant number 2 (Q_2)
- Static conditions (no water flowed)

The last condition is a repeat of the first condition and is used to ensure the system returns to the conditions obtained before the test was started. Q_1 and Q_2 refer to the discharge from the two hydrants that are being flowed.

Figure 2. An integrated Pitot gauge and a wire-cage diffuser were installed on two flow hydrants.

Test Results

Fire-flow tests were performed at eight sites. Figure 3 (on page 12) shows the location of the four pressure gauges and two tested hydrants for the simultaneous test at one of the sites. The pressure gauges were placed on four hydrants upstream of the two hydrants being flowed. Based on the 1-min pressure data readings (Figure 4), an

continued on page 12

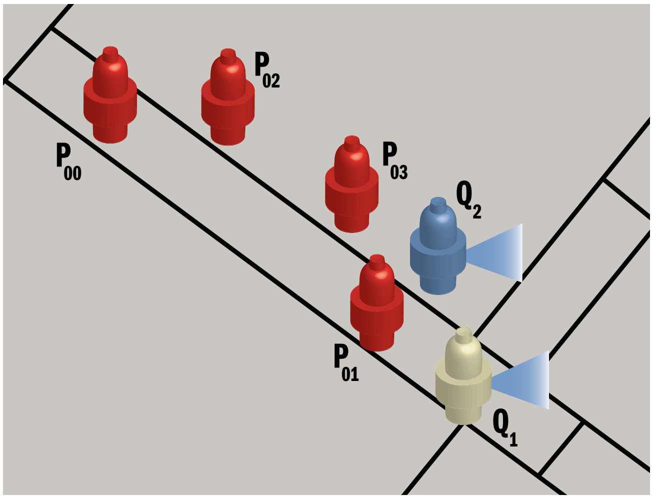


Figure 3. A schematic diagram shows the location of pressure gauges (P_{00}, P_{01}, P_{02}, and P_{03}) and flow hydrants (Q_1 and Q_2).

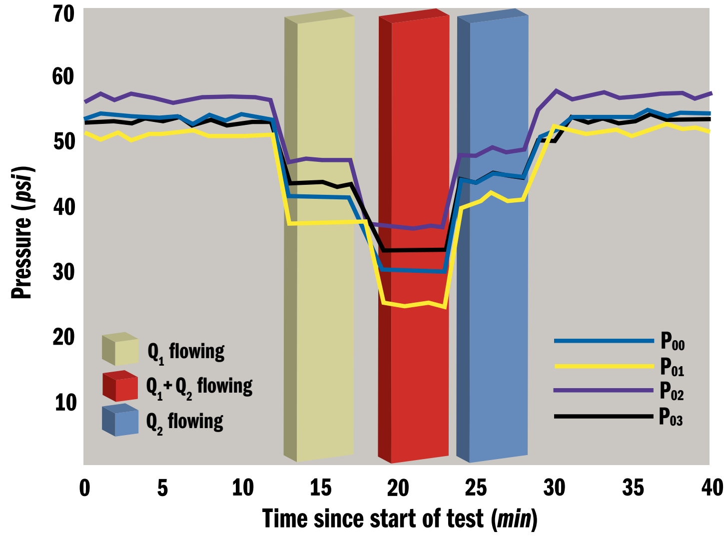


Figure 4. Hydrant pressure data and flow conditions were recorded during the fire-flow tests.

average pressure during each of the five stages of the test was computed; starting and ending static conditions for all pressure recording hydrants were within ± 0.5 psi (Table 1 on page 10). Slight variations were made at some of the test locations in terms of the number of pressure gauges that were installed or other minor variations in the protocol.

The total time to conduct this compound fire-flow test was generally less than one hour, including installation of equipment (Pitot and pressure gauges), running the test under the five conditions, and disassembling the equipment. A crew of three people can safely and quickly perform this procedure. Typically, one person is stationed at each of the flowing hydrants, and the third person reads the analog pressure gauge at a nearby hydrant. The method can be expanded if additional Pitot gauges and pressure loggers are available.

Summary

The use of continuous-recording pressure loggers and simultaneously releasing water from multiple hydrants proved to be an effective and efficient method for conducting flow tests. When compared to normal protocols for conducting flow tests, this procedure can be accomplished by smaller crews and performed faster, and can result in more data that could be used for model calibration. The cost of the equipment is relatively inexpensive, rendering this methodology quite feasible with even a limited budget.

Disclaimer: *The findings and conclusions in this article are those of the authors and do not necessarily represent the views of the Agency for Toxic Substances and Disease Registry.*

Calibration Guidelines for Water Distribution System Modeling

by
Engineering Computer Applications Committee

Hydraulic network simulation models are widely used by planners, water utility personnel, consultants, and others involved in the analysis, design, operation, and maintenance of closed-conduit hydraulic systems. Quite possibly the largest application of hydraulic network models lies with the municipal water supply industry. The results of network models have been used to assist in long-range master planning, short-term project design, fire flow studies, daily operations, emergency response, energy management, rehabilitation, troubleshooting, operator training, and water quality investigations. Clearly multi-million dollar decisions can be and have been based on the results provided by hydraulic network models. Consequently the results from the model must bear close resemblance to the actual performance of the hydraulic system. In other words, the computer model must be calibrated. The purpose of this paper is to present guidelines for network model calibration. The degree of accuracy needed for model calibration will be discussed within the context of the intended use of the model. For example, a higher degree of calibration is necessary for a model that is used to examine daily operations than is needed for a model used in long-range planning studies. The paper will also discuss calibration approaches and testing procedures that can be used to aid in the calibration effort.

Reprinted from *Proceedings of AWWA 1999 ImTech Conference*, by permission.

Calibration Guidelines for Water Distribution System Modeling

By

Engineering Computer Applications Committee

Introduction

A computer model of a water distribution system is a valuable tool which can assist engineers and planners in analyzing the hydraulic performance of water delivery systems. Computer models of water supply systems are constructed for a variety of uses including:

1) Identifying system deficiencies,
2) Analyzing the impacts of proposed development or long-term growth within the system,
3) Determining the ability of the system to deliver adequate fire flows,
4) Sizing pipes, selecting pumps, locating tanks,
5) Evaluating operating strategies,
6) Assessing pipeline rehabilitation methods,
7) Developing emergency response plans,
8) Training new operators,
9) Estimating the quality of water throughout the system.

A computer model of a water distribution system is a mathematical representation of a real physical system. Data describing the physical characteristics of the system as well as loading conditions and boundary information are supplied to a computer program that simulates the behavior of the real system. Physical data includes information such as pipe length, diameter and roughness, pump characteristics and minor loss coefficients. Loading conditions reflect demands that are placed on the system while boundary information describes reservoir and tank levels, valve settings and pump on/off status. The computer program uses this information to develop a set of equations which is then solved by the program. The solution to the set of equations provides information on pressures and flows throughout the system. Information on pressures and flows can then be used to aid in decision-making.

It is not unusual for the results provided by a computer simulation model to be used in capital projects involving several million dollars. As a result, it is imperative that the results provided by the model bear close resemblance to reality. If this is not the case, then the results provided by the model will be of limited value. Consequently, the model must be calibrated. Computer model calibration can best be defined as the process of adjusting data describing the mathematical model of the system until observed performance, typically pressures and flow rates, are in reasonable agreement with computer-predicted performance over a wide range of operating conditions.

Reprinted from *Proceedings of AWWA 1999 ImTech Conference*, by permission.

There have been a number of papers published on the subject of hydraulic network model calibration. These papers generally discuss errors that contribute to discrepancies between observed and computer predicted performance, provide detail on data collection and testing methods and provide some insight into what should be adjusted to achieve a suitable match. However a void exists in the United States network modeling community dealing with an acceptable accuracy of the calibration effort. In other words, when is the model calibrated well enough?

The purpose of this paper is to provide a little deeper background on the various sources of error that will produce differences between measured and computed system performance. Armed with a strong understanding of the errors inherent in a network model, we will briefly discuss the level of effort that can be involved in performing a comprehensive network calibration. Finally we propose some calibration guidelines and attempt to establish some criteria indicating a suitable level of calibration based upon the intended use of the model. The purpose of this paper *is not* to establish standards for model calibration. Rather members of the Engineering Computer Applications Committee hope that the proposed criteria presented here will foster meaningful discussion among the United States modeling community regarding the need and validity of calibration standards.

Sources of Error

Before discussing how accurately a computer model should be calibrated, it is valuable to closely examine why a computer model might not exactly match the field performance of a real hydraulic system. With a computer simulation model we are trying to reproduce the behavior of a real system that acts continuously over space and time. We do so by supplying data that depicts the physical characteristics of the system and by providing information that, to some degree, represents the continuous loads (system demands) placed on the system. Calibrating a hydraulic network model involves more than just adjusting pipe roughness values and nodal demands. True model calibration is achieved by adjusting whatever should be adjusted within the model until a reasonable agreement between model-predicted behavior and actual field behavior is obtained.

Errors in Input Data

In computer modeling, *any* data that is supplied to the model is a candidate for adjustment. There are two sources of error that can be directly associated with input data: 1) typographical errors and 2) measurement errors. Generally speaking, typographical errors are more easily corrected than measurement errors assuming, of course, that they can be identified. An example of a typographical error would be typing in a value of 2250 ft for a 250 ft pipe segment. A measurement error, on the other hand, might occur because of the limited precision of measuring devices combined with the scale used on system maps. For instance, all else being equal a length measurement on a map having a scale of 1"=50' would have more precision than a length measured from a map having a scale of 1"= 2000'.

A critical piece of information required of all computer simulation models is the diameter of system pipes. Many times modelers ask the question "Which pipe diameter should be used – the actual diameter or the nominal diameter?" Nominal diameters have values like 6", 8", 12", etc. The actual pipe diameter must be measured in the field. For older pipes such as unlined cast iron it is likely that the actual internal diameter will vary along the length of pipe.

Consider the pipe shown in Figure 1. Notice the build-up, called turburculation, on the interior pipe walls. The actual diameter of this pipe might be closer to 5 ½" or 5 ¼" instead of 6". Because the build-up on the pipe walls is totally random and irregular, it is highly doubtful that the actual diameter, regardless of what it is, will remain the same throughout the pipe length. Generally speaking it is usually best to use the nominal diameter in a computer model and adjust pipe roughness values to achieve calibration.

Unknown Internal Pipe Roughness Values

In the paragraph above it was stated that it is usually best to supply the nominal pipe diameter and adjust pipe roughness values until a suitable match is obtained. But what roughness values should be used? There are a number of tables in various texts that provide estimates of pipe roughness, usually Hazen-Williams C-Factors, as a function of pipe material, size and age. However pipe roughness is also a function of water quality [1]. In other words, there is no guarantee that the internal roughness of a 12" 40-year old unlined cast iron pipe in New York City will be the same as the internal roughness of a 12" 40-year old unlined cast iron pipe in Seattle.

Simulation models require a value for pipe roughness for each individual pipe segment. This can result in quite a bit of information if one considers that it is not unusual for a model of a moderately sized system to contain 500-1,000 pipes with larger systems incorporating 2,000 or more pipelines. Certainly observed pressures can point the modeler in the correct direction with regard to pipe roughness values. Nonetheless compensating errors could cause incorrect pipe roughness values to be used.

Consider the simple parallel pipe system shown in Figure 2. Suppose that pressure measurements have been taken at the node on each end of the pipe segment and the flow through the system is known. The unknowns are the internal pipe roughness values for each pipe. Table 1 shows the results of a simple analysis performed on this system. Column 1 represents an assumed internal roughness for Pipe #1. Column 2 is the resulting flow due to the measured head loss, the pipe characteristics and the assumed roughness. Column 3 is the flow in Pipe #2 assuming a total system flow of 1,350 Gpm. Finally column 4 represents the pipe roughness in Pipe 2 that is due to the head loss, the pipe characteristics and the pipe flow.

Clearly there are multiple values of pipe roughness in pipes 1 and 2 that produce the same head loss across the system. So the real question becomes which is the correct set of pipe roughness values? The only way this question can be answered is to measure the flow in one of the pipes. With the flow known the correct roughness values can be established. Frequently in the United States actual pipeline flows throughout the system are not measured in support of model calibration. Instead only system pressures at a several selected locations are measured.

The problem above illustrates the complexity for a simple two pipe system. The problem complexity is compounded in real systems. For the system shown in Figure 2, let's assume that the flow into the system is not known and that the pressure at the upstream node is also unknown. Then the number of combinations of flow, pipe roughness and upstream pressure that would produce a pressure of 47 psi at the downstream node is very large. Now consider that there are many parallel paths that water may take in real systems. With the above illustration it is clear how compensating errors can result in incorrect model parameters even though an exact match between computer predicted and observed values is obtained.

Effect of System Demands
Although simulation models require that water use be applied at a single point called a node, in reality water use occurs along the entire length of a pipe as shown in Figure 3. Water use is assigned to nodes because this approach simplifies the complexity of the modeling problem. In Figure 3 the water use associated with the eight homes closest to J-23 was assigned to this node. Likewise, the water use for the 10 homes closest to J-24 is assigned to junction node J-24.

Grouping or lumping water use at junction nodes instead of placing it at the actual location where water is withdrawn from the system will produce some differences between computer predicted and actual field performance. The differences should be relatively minor assuming, of course, that the customer demand assigned to a particular junction node is not located far away from the node. A bigger reason for the discrepancy between computer predicted and field measured results lies with the spatial and temporal nature of water use.

Water use within a home, and to a lesser extent, a commercial establishment is most likely stochastic or random. In other words, one cannot predict with absolute certainty *when* and *how much* water will be used at a particular residence. Perhaps the closest one can get to certainty in residential water use lies with those homes that have automatic lawn irrigation systems designed to operate at the same time each day. If one were to meter the instantaneous water use in a specific residence the usage might look something that shown in Figure 4. Industrial water use has a tendency to be a somewhat less variable than residential or commercial usage. Any difference between the actual water use in a distribution system and the estimates of that use supplied to a computer model will generate errors between computer predicted and field measured performance.

Errors in System Maps
Water system maps are the primary source of model data for the physical characteristics of many water distribution systems. Pipeline lengths and diameters are typically taken from water system maps. The presence and types of fittings and appurtenances that contribute to minor losses can also be found from system maps. Finally the layout of the system and the pipe/node connectivity is usually determined from the system maps.

Water system maps come in a variety of formats. Some systems may have very accurate, nicely detailed maps generated using sophisticated CAD or GIS systems. The water system maps of other utilities may consist of a set of rolled-up plans that have not been unrolled in several years even though some system improvements have taken place. For some systems, the system caretaker may be the sole individual responsible for record keeping including maintaining the system maps – that is if they even exist! Indeed, it is entirely possible that the extent of system mapping for some systems may be the knowledge that is kept in the head of the system caretaker.

As changes to a system are made then the system maps should be modified to reflect the changes that have been made. Most water utilities are very good at making these changes though some are not. Despite the best efforts of the mapping department, sometimes systems grow at a faster pace than the mapping department can make the changes. Obviously the accuracy of system maps can have a pronounced effect on the accuracy of a model calibration effort.

Node Elevations

Frequently in hydraulic network simulation ground elevations will be used to find pressures at locations throughout the hydraulic network. For most systems, pipes are buried at a uniform distance below the ground surface usually anywhere between 3-6 ft deep (1.3-2.6 psi). The pressure difference associated with using the ground level versus the actual pipe centerline elevation is usually not that great. Moreover, it is easier to gather elevation data from topographic maps than it is to scour through As-Built drawings to find the actual elevations.

When pressure measurements are taken for model calibration they are usually taken at fire hydrants or at some other direct connection to the water system. The elevation of the pressure gauge at the hydrant can be several feet higher than the ground elevation or several feel lower than the ground in the case of a meter vault. For *calibration* purposes this elevation difference can make a difference. Therefore one should always use the elevation *of the pressure gauge*, that is, the location where the pressure measurement is taken when calibrating a hydraulic network model.

Effect of Time

If the hydraulic simulation is an extended period simulation (EPS), then the calibration must also consider adjustments that should be made for time-varying conditions. Generally this will mean adjusting model parameters until there is a reasonable agreement between computer predicted and observed tank water levels. In addition to tank water levels, time-varying pressures and pipeline flows can also be compared.

Many times information on tank water levels will be obtained from circular charts such as the one shown in Figure 5. Notice that there can be fairly substantial changes in tank water level over a short period of time. This is especially true for small storage tanks such as standpipes or if the tank capacity is too small for the system it is supplying. If too large a time step is used in hydraulic calculations then the true time-varying behavior of the system between time intervals may not be captured even though a suitable agreement is obtained at individual time steps.

Model Detail

When a computer model of an existing system is constructed, quite often a simplified or skeletal version of the system will be developed instead of one which contains every pipe in the system. A skeletal version of the system is one that does not include small diameter pipes or does not include lines that have an insignificant influence the hydraulics of the overall system. A skeletal model is one which is manageable and yet accurate – when properly calibrated. Interestingly, Cesario, et al state that the economics of model construction make a strong case for including all pipes in the mathematical model [2].

Although skeletalization can have an impact on the calibration process, the effects of skeletalization on calibration are, for the most part, unknown. If one is having difficulty in achieving a reasonable calibration with a highly skeletal system, then perhaps it may be necessary to include more lines in the model. For example, omitting a dense grid of small diameter mains from a model simply because of their size may be inappropriate if, as a group, they present a sizeable hydraulic impact on the system.

Geometric Anomalies

Even if high quality information on the physical attributes of the system is available and good estimates of nodal demand is provided to the simulation model, there can still be differences between computer predicted and observed performance. Anomalies in the geometry of the system are usually to blame in these cases. For example, suppose that we have a condition where two pipes cross one another as illustrated in Figure 6. From the plan view it may appear that these two lines are connected. The cross-section view shows otherwise. Obviously the model will not be calibrated if these pipes were assumed to be connected.

Another issue related to geometric anomalies that could cause differences between observed and predicted behavior lies with system isolation valves. Most of the time in computer models gate valves, butterfly valves, etc. are assumed to be fully open. In some systems this may not be the case. If one has to use unrealistically low roughness values to obtain a suitable calibration then perhaps there are closed or partially closed isolation valves in the system.

Outdated Pump Characteristic Curves

If there are pumps in the system then it is necessary to supply information on the pump characteristics. Most hydraulic models use some type of curve fit using three or more points from the actual pump head-discharge relationship to reproduce the curve. Errors in the numerical curve fit could contribute to discrepancies. However the more likely cause of error for pumps is due to old or outdated pump curves. Over time impellers can wear and change the characteristics of the pump. It is possible that new impellers may have been installed on a pump without the pump curve being updated to reflect the change. Clearly pump curves used in hydraulic simulation should represent the in-situ pump characteristics of the unit.

Not only can inaccurate pump curves cause a discrepancy between computer predicted and observed system performance, but errors in any boundary element can cause difficulty with the calibration. These boundary elements include regulating valve settings, tank levels and pressure zone boundaries. Clearly an incorrect setting for pressure reducing valve will produce pressures in the computer model that are inconsistent with field measured values.

Poorly Calibrated Measuring Equipment
There are many ways calibration data is gathered including direct measurement using pressure gauges and flow recording devices. Many of today's water supply systems have Supervisory Control and Data Acquisition (SCADA) systems that can provide information on tank levels and possibly pressures and flows at selected locations. However in the United States pressure and flow measurement using SCADS systems is generally limited to pump stations and boundaries with other systems.

Whenever a measurement is taken we must be sure that the equipment or device that was used to take the measurement is itself calibrated. Pressure measurements that were taken with a poorly calibrated pressure gauge or pressure transducer are of limited value and will frequently be discarded. Remember not only should the equipment used to take measurement be calibrated but the pressure, flow, and tank level monitors for the SCADA system should also be calibrated.

Even if measuring and monitoring equipment is well calibrated, other problems can arise to give poor data. Suppose tank water levels is obtained from circular charts such as that shown in Figure 5. Notice how the chart indicates the tank water level at a given time. We need to make sure that the timing mechanism in the chart recorder is working. In other words, does the level on the circular chart at 6:00 a.m. really correspond to 6:00 a.m. or some other time because the timing mechanism is slow or because the chart slips or because someone installed the chart incorrectly. Issues such as these must be considered when using analog monitoring equipment. Of course the Y2K issue may create problems for digital systems that are not compliant.

Calibrating for Water Quality

In the preceding paragraphs we discussed some of the data requirements for hydraulic network models. Key among this information is internal pipe roughness values for each individual pipe segment. Providing accurate roughness coefficients for moderate to large models can be a considerable undertaking. In fact, it is possible that each individual internal pipe roughness value could be a candidate for adjustment during the calibration.

When calibrating for water quality, the level of effort required to obtain a suitable calibration can be even more imposing. Most water quality models require information on coefficients describing reactions that take place within the bulk fluid and reactions that take place between the fluid and the pipe wall. In essence this means that two additional parameters for *each* pipeline are required.

Obtaining bulk reaction coefficients is not that difficult and procedures are available to perform the necessary tests. Finding wall reaction coefficients can be much more challenging. Just as each pipeline can have a unique internal roughness, it is possible that each pipeline can have a unique wall reaction coefficient. Unlike roughness coefficients, no well-established test is available for measuring reaction coefficients. Unlike roughness coefficients, very little data is available for typical reaction coefficients. In fact, there is some question within the modeling community if "typical" reaction coefficients even exist.

In the case of water quality models, a match between some observed and predicted water quality parameter such as chlorine or fluoride concentrations is usually used for calibration purposes. However in the case of nonconservative species such as chlorine not only are system-wide concentrations dependent upon the network hydraulics, but they are a function of any reactions that take place in the system. Clearly one can see that calibrating for water quality can greatly increase the level of effort required to obtain a suitable match between observed and computer predicted performance. Interestingly as Grayman points out, a water quality model can be used to help calibrate a hydraulic model [5].

Calibration Methods

There have been a number of papers written on the subject of hydraulic network model calibration and several methods have been developed to assist with the calibration effort. However the traditional method of trial-and-error (trial-and-effort?) still seems to be the predominate method of choice. In other words, data describing the model is adjusted – sometimes randomly – until a suitable match is obtained.

The lack of calibration methods is not surprising once the full complexity of the level of effort required for a comprehensive calibration is understood. Ormsbee and Reddy provide a nice summary of the steps necessary to calibrate a hydraulic network computer model [3]. In their paper they note that direct solution techniques or optimization approaches offer some potential for simplifying the calibration effort. However for moderate to large systems calibrating a model can be a considerable undertaking.

Direct parameter calculation methods and optimization techniques are not immune to the problem of compensating errors. This problem can be addressed somewhat with optimization methods by placing constraints on the decision variables, e.g. pipe roughness values, water demand, valve setting, etc. With the direct parameter calculation methods engineering judgment is usually the filter which captures unrealistic model parameter values. Nonetheless, the problem of compensating errors may still exist.

Perhaps the best way to address the problem of compensating errors short of measuring heads and flows at numerous points in the system is to calibrate for multiple demand conditions. These demands conditions may reflect maximum day, average day and minimum day demands. Generally speaking the physical characteristics of the system will be the same between the various demand days. Only the loading and boundary conditions will change. If the same or nearly the same roughness characteristics produce a suitable match between observed and predicted pressures then chances are good that these roughness values are the correct ones. However, there is no substitute for measuring flows in throughout the system which, unfortunately, is not commonly done in the United States.

Calibration Accuracy in the US vs. the UK

It should be clear by now that the level of effort required to obtain an acceptable level of calibration can be considerable. Water quality calibrations can be even more challenging. Many modelers in the United States agree that the level of effort required to calibrate a hydraulic network model will depend upon the intended use of the model [2, 3, 4]. In other words, it may not be reasonable to expect that a model used for long-range planning purposes will have the same level of accuracy than a model intended for operations or water quality investigations. Walski provides some suggestions on a suitable level of calibration as a function of the use of the model [4].

In the United Kingdom, however, standards or performance criteria that modeler should strive for have been established. The criteria for flow and pressure are shown below. Additional criteria also exist including those for extended period simulations [6].

1. Flows agree to:
   a) 5% of measured flow when flows are more than 10% of total demand (transmission lines),
   b) 10% of measured flow when flows are less than 10% of total demand (distribution lines).
2. Pressures agree to:
   a) 0.5 m (1.6 ft) or 5% of head loss for 85% of test measurements,
   b) 0.75 m (2.31 ft) or 7.5% of head loss for 95% of test measurements,
   c) 2 m (6.2 ft) or 15% of head loss for 100% of test measurements.

The manner by which hydraulic networks are operated in the United Kingdom goes a long way to explaining why calibration criteria exist in the UK. For example, in the UK it is not unusual for water utilities to provide a lower level of fire protection than is provided in the United States. Perhaps one of the bigger differences lies in metering. In the UK most individual service connections are not metered. Rather large networks are typically divided into smaller "demand management areas" each serving about 1,000 – 2,000 connections. The demand areas are independent service areas that are metered at all inflow and outflow points. As a result, there will be more "in-line" flow metering than exists in the US. Cesario, et al provide a summary of the differences in water utility operations between the United States and the United Kingdom [2].

Calibration Accuracy

Despite the many variables that contribute to differences between observed and predicted system performance and even though calibrating a hydraulic network model can be a time consuming and sometimes frustrating task, the time is right in the United States to consider adopting some calibration criteria. The results of computer simulations are frequently used as the basis for capital projects involving many millions of dollars. Moreover, the 1997 EPA Drinking Water Needs Survey has placed the price tag over the next 20 years of bringing water distribution systems up to compliance with the Safe Drinking Water Act at nearly $140 billion [7]. It is likely therefore that in the near future computer simulation will be even more widely used for planning, design, operation and water quality investigations.

As stated earlier, the primary purpose of this paper is to suggest calibration guidelines that can be used in the United States to determine a suitable level of model calibration for a particular use of the model. The purpose of this paper *is not* to establish calibration standards. The Engineering Computer Applications Committee hopes that by providing these criteria some discussion among the network modeling community in the United States (and abroad) will ensue regarding the need for such criteria and the validity of the criteria.

Minimum calibration criteria are presented in Table 2. Note that while a hydraulic network model may have a number of different uses, four basic use categories are presented in Table 2. Most of the uses of hydraulic models should fit into one of the following categories: 1) Planning, 2) Design, 3) Operations and 4) Water Quality. For example training system operators, developing emergency response plans, or performing energy efficiency studies would all fall under the Operations category.

Six calibration criteria have been identified. The level of detail criteria relates the degree of skeletonization that a model may have while the type of time simulation reflects whether the model should be a steady-state or extended period simulation model. The number of pressure readings is expressed as a percentage of the number of nodes in the model while the number of flow readings is presented in terms of the number of pipes in the model.

Note that the criteria represent minimum criteria. For example, based on the proposed criteria a model used for long-range planning may have a low level of detail. However there is nothing to prevent a model used for long range planning from being a model with a high degree of detail. Note that the level of detail and the type of time simulation will, in large measure, depend upon the use of the model.

References

[1] Walski, Thomas M., Sharp. Wayne W. and Shields, F. Douglas Jr. 1988 "Predicting Internal Roughness in Water Mains," Miscellaneous Paper EL-88-2, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

[2] Cesario, A. L., Kroon, J.R., Grayman, W., and Wright, G. "New Perspectives on Calibration of Treated Water Distribution System Models,"

[3] Ormsbee, L.E. and Lingireddy, S. "Calibrating Hydraulic Network Models," Journal American Water Works Association, Vol. 89, No. 2, pp. 42-50, 1997.

[4] Walski, Thomas M. "Standards for Model Calibration," 1995 AWWA Computer Conference, Norfolk, VA.

[5] Grayman, Walter M. "Use of Tracer Studies and Water Quality Models to Calibrate a Network Hydraulic Mode," <u>Essential Hydraulics and Hydrology</u>, Haestad Press, Waterbury, CT., 1998.

[6] Water Authorities Association and WRc, 1989, "Network Analysis – A Code of Practice," WRc, Swindon, England.

[7] U.S. Environmental Protection Agency. 1997. "Drinking Water Infrastructure Needs Survey," EPA/012/R-97/001. Office of Water, Washington, D.C.

List of Tables

Table 1

Pipe Roughness Values and Flow for Simple Two-Pipe System

Pipe #1 Roughness	Pipe #1 Flow (Gpm)	Pipe #2 Flow (Gpm)	Pipe #2 Roughness
80	660.75	689.25	166.88
90	743.34	606.66	146.88
100	825.93	524.07	126.88
110	908.53	441.47	106.89
120	991.12	358.88	86.89
130	1073.71	276.29	66.89
140	1156.31	193.69	46.90

Table 2

Minimum Criteria for Hydraulic Network Model Calibration

Intended Use	Level of Detail	Type of Time Simulation	Number of Pressure Readings[1]	Accuracy of Pressure Readings	Number of Flow Readings	Accuracy of Flow Readings
Long-Range Planning	Low	Steady-State Or EPS	10% of Nodes	± 5 Psi for 100% of Readings	1% of Pipes	± 10%
Design	Moderate to High	Steady-State Or EPS	5% - 2% of Nodes	± 2 Psi for 90% of Readings	3% of Pipes	± 5%
Operations	Low to High	Steady-State Or EPS	10% - 2% of Nodes	± 2 Psi for 90% of Readings	2% of Pipes	± 5%
Water Quality	High	EPS	2% of Nodes	± 3 Psi for 70% of Readings	5% of Pipes	± 2%

Notes:
1. The number of pressure reading is related to the level of detail as illustrated in the table below.

Level of Detail	Number of Pressure Readings
Low	10% of Nodes
Moderate	5% of Nodes
High	2% of Nodes

List of Figures

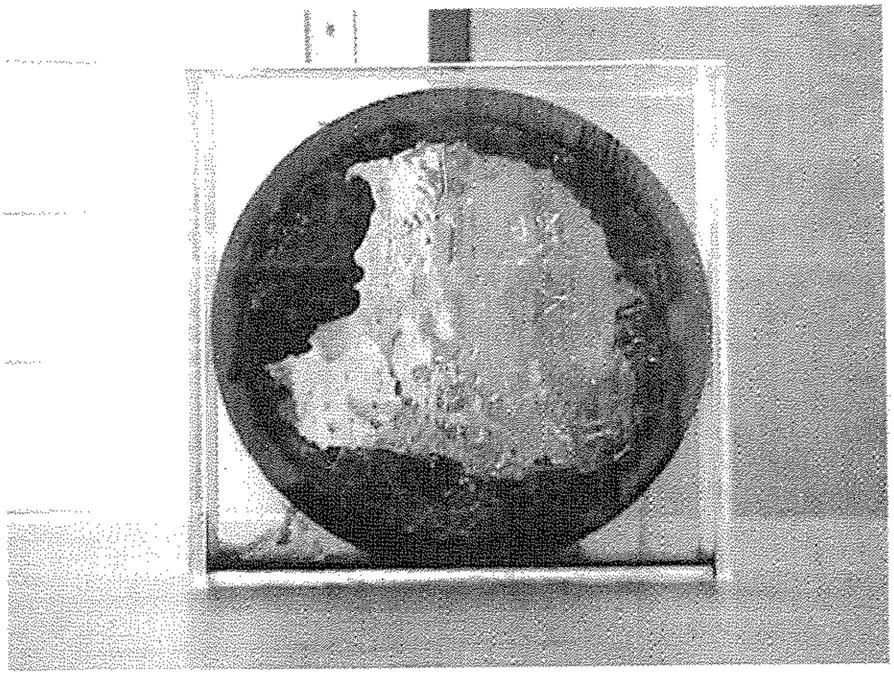


Figure 1

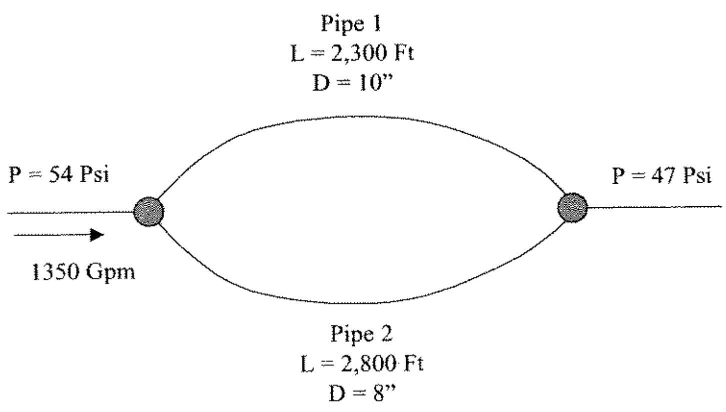


Figure 2

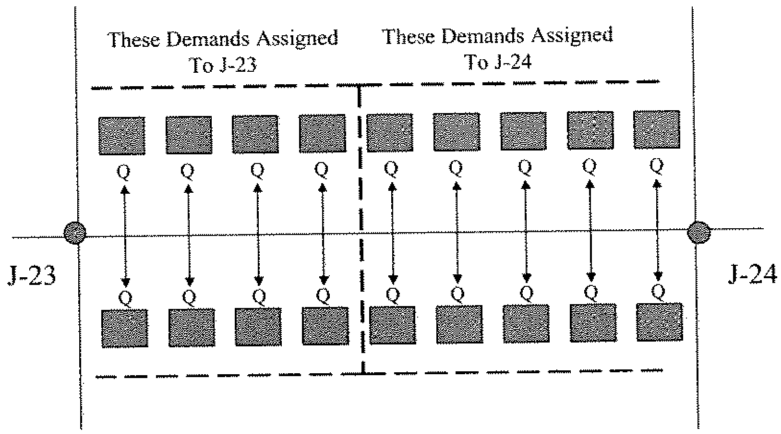


Figure 3

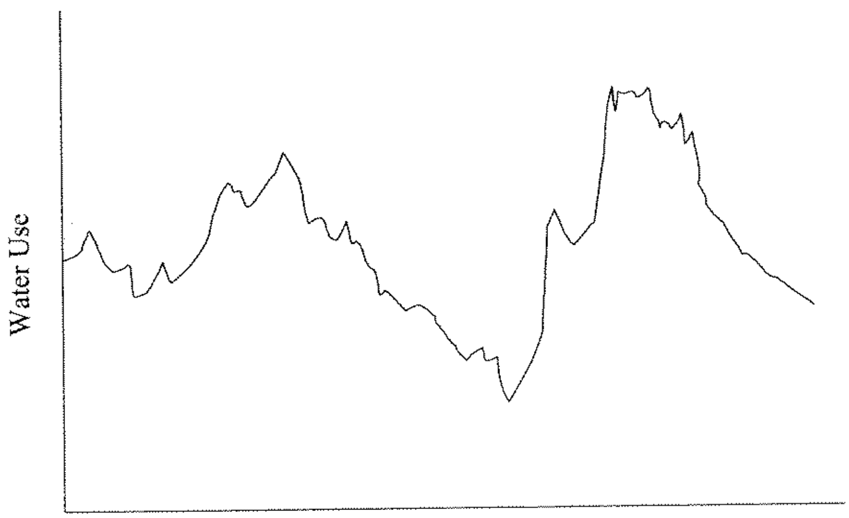


Figure 4

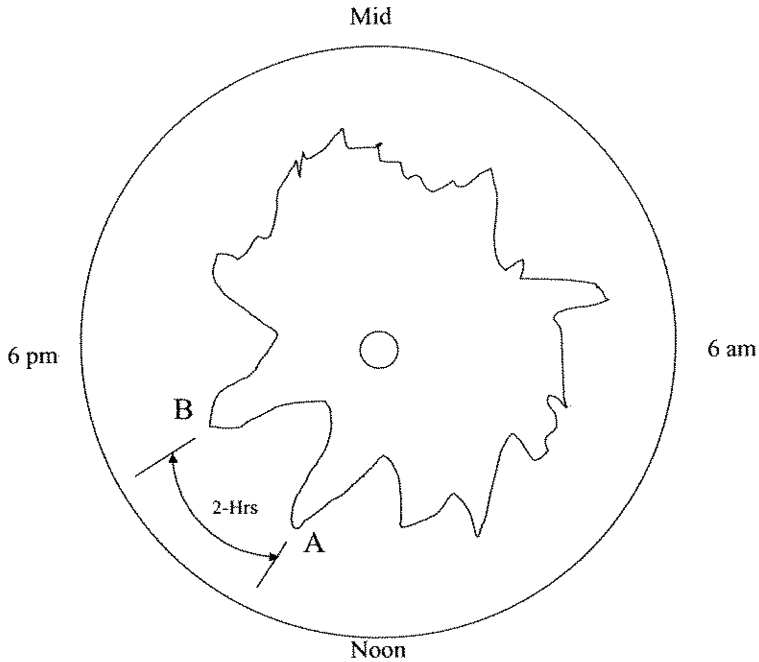


Figure 5

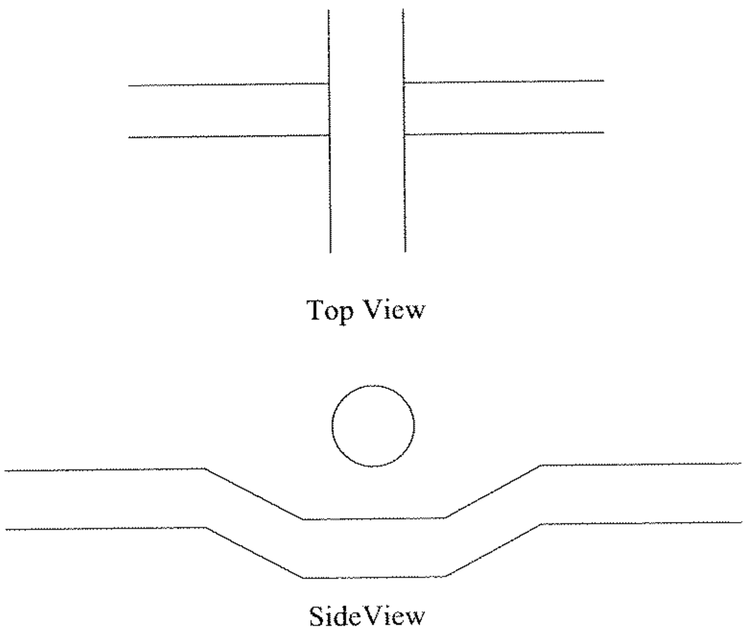


Figure 6

CHAPTER 14
CALIBRATION OF HYDRAULIC NETWORK MODELS

Lindell E. Ormsbee and Srinivasa Lingireddy
Department of Civil Engineering
University of Kentucky,
Lexington, KY

14.1 INTRODUCTION

Computer models for analyzing and designing water distribution systems have been available since the mid-1960s. Since then, however, many advances have been made with regard to the sophistication and application of this technology. A primary reason for the growth and use of computer models has been the availability and widespread use of the microcomputer. With the advent of this technology, water utilities and engineers have been able to analyze the status and operations of the existing system as well as to investigate the impacts of proposed changes (Ormsbee and Chase, 1988). The validity of these models, however, depends largely on the accuracy of the input data.

14.1.1 Network Characterization

Before an actual water distribution system can be modeled or simulated with a computer program, the physical system must be represented in a form that can be analyzed by a computer. This normally requires that the water distribution system first be represented by using node-link characterization (Fig. 14.1). In this case, the links represent individual pipe sections and the nodes represent points in the system where two or more pipes (links) join together or where water is being input or withdrawn from the system.

14.1.2 Network Data Requirements

Data associated with each link will include a pipe identification number, pipe length, pipe diameter, and pipe roughness. Data associated with each junction node will include a junction identification number, junction elevation, and junction demand. Although it is

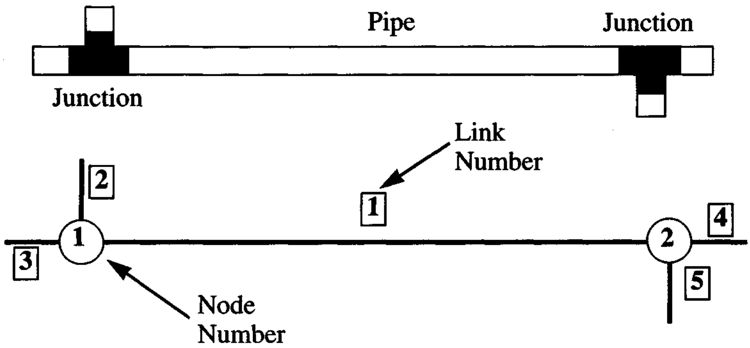


FIGURE 14.1 Node-link characterization.

recognized that water leaves the system in a time-varying fashion through various service connections along the length of a pipe segment, it is generally acceptable in modeling to lump half the demands along a line to the upstream node and the other half of the demands to the downstream node as shown in Fig. 14.2.

In addition to the network pipe and node data, physical data for use in describing all tanks, reservoirs, pumps, and valves also must be obtained. Physical data for all tanks and reservoirs normally includes information on tank geometry as well as the initial water levels. Physical data for all pumps normally include either the value of the average useful horsepower or data for use in describing the pump flow/head characteristics curve. Once this necessary data for the network model has been obtained, the data should be entered into the computer in a format compatible with the selected computer model.

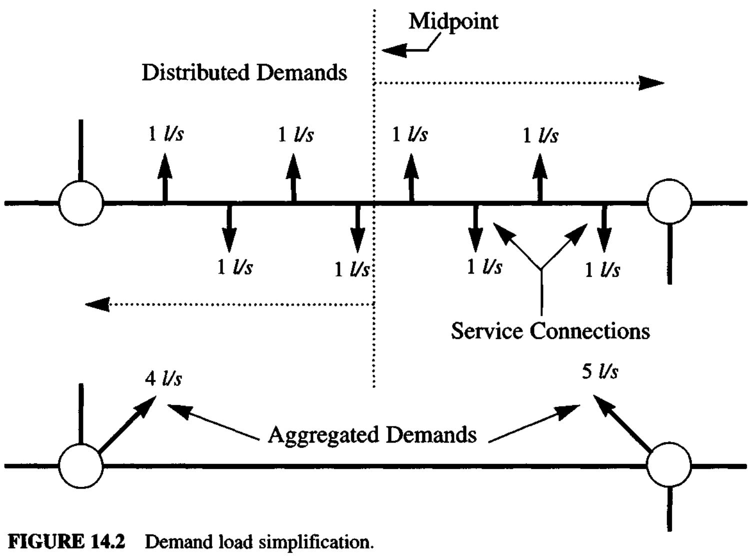


FIGURE 14.2 Demand load simplification.

14.1.3 Model Parameters

Once the data for the computer network model has been assembled and encoded, the associated model parameters should then be determined before actual application of the model. In general, the primary parameters associated with a hydraulic network model include pipe roughness and nodal demands. Because obtaining economic and reliable measurements of both parameters is difficult, final model values are normally determined through the process of model calibration. Model calibration involves the adjustment of the primary network model parameters (i.e., pipe roughness coefficients and nodal demands) until the model results closely approximate actual observed conditions, as measured from field data. In general, a network-model calibration effort should encompass the following seven basic steps (Fig. 3). Each step is discussed in detail in the following sections.

14.2 IDENTIFY THE INTENDED USE OF THE MODEL

Before calibrating a hydraulic network model, it is important to identify its intended use (e.g., pipe sizing for master planning, operational studies, design projects, rehabilitation studies, water-quality studies) and the associated type of hydraulic analysis (steady-state versus extended-period). Usually, the type of analysis is directly related to the intended use. For example, water-quality and operational studies require an extended-period analysis, whereas some planning or design studies can be performed using a study state analysis (Walski, 1995). In the latter, the model predicts system pressures and flows at an instant in time under a specific set of operating conditions and demands (e.g., average or maximum daily demands). This is analogous to photographing the system at a specific point in time. In extended-period analysis, the model predicts system pressures and flows over an extended period (typically 24 hours). This is analogous to developing a movie of the system's performance.

Both the intended use of the model and the associated type of analysis provide some guidance about the type and quality of collected field data and the desired level of agreement between observed and predicted flows and pressures (Walski, 1995). Models for steady-state applications can be calibrated using multiple static flow and pressure observations collected at different times of day under varying operating conditions. On the other hand, models for extended-period applications require field data collected over an extended period (e.g., 1.7 days).

In general, a higher level of model calibration is required for water-quality analysis or an operational study than for a general planning study. For example, determining ground evaluations using a topographic map may be adequate for one type of study, whereas another type of study may require an actual field survey. This of course may depend on the contour interval of the map used. Such considerations obviously influence the methods used to collect the necessary model data and the subsequent calibration steps. For example, if one is working in a fairly steep terrain (e.g. greater than 20 foot contour intervals), one may decided to use a GPS unit for determining key elevations other than simply interpolating between contours.

14.3 DETERMINE ESTIMATES OF THE MODEL PARAMETERS

The second step in calibrating a hydraulic network model is to determine initial estimates of the primary model parameters. Although most models will have some degree of uncertainty associated with several model parameters, the two parameters that normally have the greatest degree of uncertainty are the pipe roughness coefficients and the demands to be assigned to each junction node.

14.3.1 Pipe Roughness Values

Initial estimates of pipe-roughness values can be obtained using average values in the literature or values directly from field measurements. Various researchers and pipe manufacturers have developed tables that provide estimates of pipe roughness as a function of various pipe characteristics, such as pipe material, pipe diameter, and pipe age (Lamont, 1981). One such typical table is shown in Table 14.1 (Wood, 1991). Although such tables can be useful for new pipes, their specific applicability to older pipes decreases significantly as the pipes age as a result of the effects of such factors as tuberculation, water chemistry, and the like. As a result, initial estimates of pipe roughness for all pipes other than relatively new ones normally should come directly from field testing. Even when new pipes are being used, it is helpful to verify the roughness values in the field since the roughness coefficient used in the model actually may represent a composite of several secondary factors such as fitting losses and system skeletonization.

14.3.1.1 Chart the pipe roughness. A customized roughness nomograph for a particular water distribution system can be developed using the process illustrated in Figs. 14.4.A-C To obtain initial estimates of pipe roughness through field testing, it is best to divide the water distribution system into homogeneous zones based on the age and material of the associated pipes (Fig. 14.4A). Next, several pipes of different diameters should be tested

TABLE 14.1 Typical Hazen-William Pipe Roughness Factors

Pipe Material	*Age (years)*	*Diameter*	*C Factor*
Cast iron	New	All sizes	130
	5	>380 mm (15in)	120
		>100 mm (4in)	118
	10	>600 mm (24in)	113
		>300 mm (12in)	111
		>100 mm (4in)	107
	20	>600 mm (24in)	100
		>300 mm (12in)	96
		>100 mm (4in)	89
	30	>760 mm (30in)	90
		>400 mm (16in)	87
		>100 mm (4in)	75
	40	>760 mm (30in)	83
		>400 mm (16in)	80
		>100 mm (4in)	64
Ductile iron	New		140
Polyvinyl chloride	Average		140
Asbestos cement	Average		140
Wood stave	Average		120

Identify the intended use of the model
Determine initial estimates of the model parameters
Collect calibration data
Evaluate the model results
Perform the macro-level calibration
Perform the sensitivity analysis
Perform the micro-level calibration

FIGURE 14.3 Seven basic steps for network model calibration.

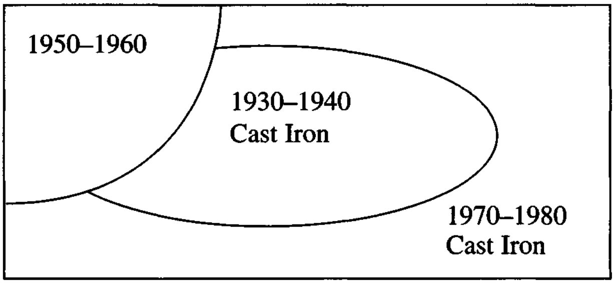


FIGURE 14.4A Subdivided network into homogeneous zones of like age and material.

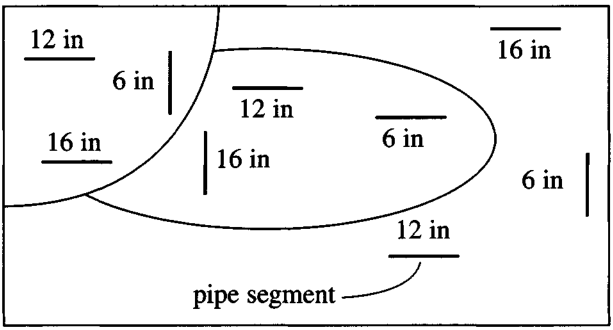


FIGURE 14.4B Select representative pipes from each zone.

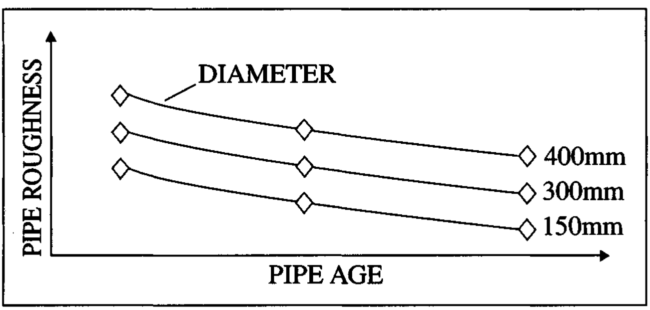


FIGURE 14.4C Plot associated roughness as a function of pipe diameter and age.

in each zone to obtain individual estimates of pipe roughness (Fig. 14.4B). Once a customized roughness nomograph is constructed (Fig. 14.4C), it can be used to assign values of pipe roughness for the rest of the pipes in the system.

14.3.1.2 Field test the pipe roughness. Pipe roughness values can be estimated in the field by selecting a straight section of pipe that contains a minimum of three fire hydrants (Figure 14.5A). When the line has been selected, pipe roughness can be estimated using one of two methods (Walski, 1984): (1) the parallel-pipe method (Fig. 14.5B) or (2) the two-hydrant method (Figure 14.5C). In each method, the length and diameter of the test pipe are determined first. Next, the test pipe is isolated, and the flow and pressure drop are measured either by using a differential-pressure gauge or two separate pressure gauges. Pipe roughness can then be approximated by a direct application of either the Hazen-Williams equation or the Darcy-Weisbach equation. In general, the parallel-pipe method

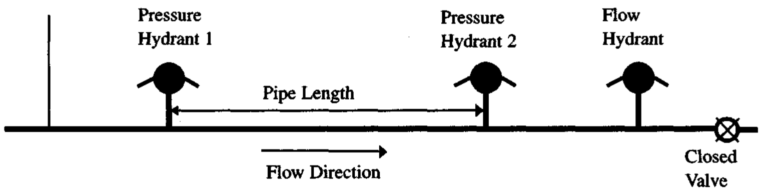


FIGURE 14.5A Pipe roughness test configuration.

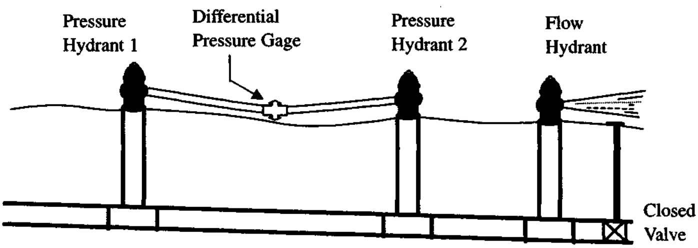


FIGURE 14.5B Parallel pipe method.

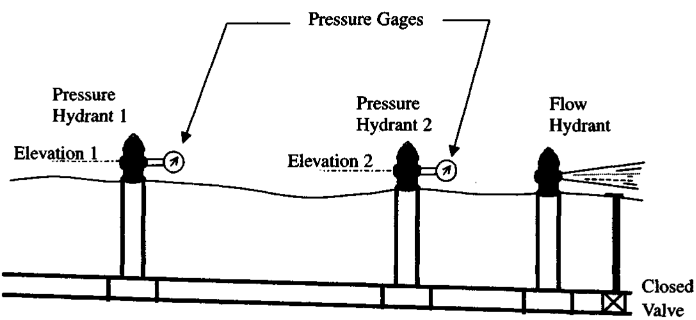


FIGURE 14.5C Two gage method.

is preferable for short runs and for determining minor losses around valves and fittings. For long runs of pipe, the two-gage method is generally preferred. Also if the water in the parallel pipe heats up or if a small leak accurs in the parallel line, it can lead to errors in the associated headloss measurements (Walski, 1985).

Parallel-pipe method. The steps involved in the application of the parallel pipe method are summarized as follows:

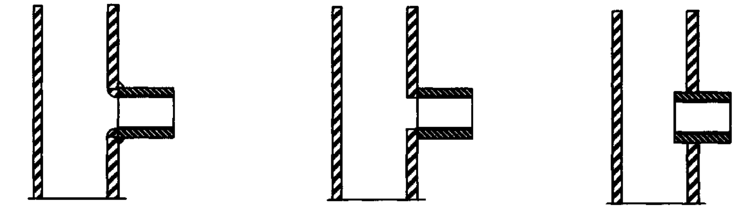
1. Measure the length of pipe between the two upstream hydrants (L_p) in meters.
2. Determine the diameter of the pipe (D_p) in millimeters. In general, this should simply be the nominal diameter of the pipe. It is recognized that the actual diameter may differ from this diameter because of variations in wall thickness or the buildup of tuberculation in the pipe. However, the normal calibration practice is to incorporate the influences of variations in pipe diameter via the roughness coefficient. It should be recognized, however, that although such an approach should not significantly influence the distribution of flow or headloss throughout the system, it may have a significant influence on pipe velocity, which in turn could influence the results of a water-quality analysis.
3. Connect the two upstream hydrants with a pair of parallel pipes, (typically a pair of fire hoses) with a differential pressure device located in between (Figure 14.5B). The differential pressure device can be a differential pressure gauge, an electronic transducer, or a manometer. Walski (1984) recommended the use of an air-filled manometer because of its simplicity, reliability, durability, and low cost. (*Note*: When connecting the two hoses to the differential pressure device, make certain that there is no flow through the hoses. If there is a leak in the hoses, the computed headloss for the pipe will be in error by an amount equal to the headloss through the hose.)
4. Open both hydrants and check all connections to ensure there are no leaks in the configuration.
5. Close the valve downstream of the last hydrant, then open the smaller nozzle on the flow hydrant to generate a constant flow through the isolated section of pipe. Make certain the discharge has reached equilibrium condition before taking flow and pressure measurements.
6. Determine the discharge Q_p (L/s) from the smaller nozzle in the downstream hydrant. This is normally accomplished by measuring the discharge pressure P_d of the stream leaving the hydrant nozzle using either a hand-held or nozzle-mounted pitot. Once the discharge pressure P_d (in kPa) is determined, it can be converted to discharge (Q_p) using the following relationship:

$$Q_p = \frac{C_d D_n^2 P_d^{0.5}}{900.3} \tag{14.1}$$

   where D_n is the nozzle diameter in millimeters and C_d is the nozzle discharge coefficient, which is a function of the type of nozzle (Fig. 14.6). (*Note*: When working with larger mains, sometimes you can't get enough water out of the smaller nozzles to get a good pressure drop. In such cases you may need to use the larger nozzle).
7. After calculating the discharge, determine the in-line flow velocity V_p (m/s) where

$$V_p = \frac{Q_p}{(\pi D_p^2/4)^2} \tag{14.2}$$

8. After the flow through the hydrant has been determined, measure the pressure drop D_p through the isolated section of pipe by reading the differential pressure gauge. Convert



Outlet Smooth and Rounded. Coefficient: 0.90

Outlet Square and Sharp. Coefficient: 0.80

Outlet Square and Projecting into Barrel. Coefficient: 0.70

FIGURE 14.6 Hydrant nozzle discharge coefficients.

the measured pressure drop in units of meters (H_p) and divide by the pipe length L_p to yield the hydraulic gradient or friction slope S_p:

$$S_p = \frac{H_p}{L_p} \tag{14.3}$$

9. Once these four measured quantities have been obtained, the HazenWilliams roughness factor (C_p) can then be determined using the HazenWilliams equation as follows:

$$C_p = \frac{218 V_p}{D_p^{0.63}\, S_p^{0.34}} \tag{14.4}$$

To calculate the actual pipe roughness e, it is necessary to calculate the friction factor f using the Darcy-Weisbach equation as follows (Walski, 1984):

$$f = \frac{g S_p D_p}{500\, V^2_p} \tag{14.5}$$

where g = gravitational acceleration constant (9.81m/s^2).

Once the friction factor has been calculated, the Reynolds number (Re) must be determined. Assuming a standard water temperature of 20°C (68°F), the Re is

$$Re = 993\, V_p D_p \tag{14.6}$$

When the friction factor f and the Re have been determined, they can be inserted into the Colebrook-White formula to give the pipe roughness e (mm) as

$$e = 3.7 D_p \left[\exp\left(-1.16\sqrt{f}\right) - \frac{2.51}{\mathrm{R}\sqrt{\mathrm{f}}}\right] \tag{14.7}$$

Two-hydrant method. The two-hydrant method is basically identical to the parallel-pipe method, with the exception that the pressure drop across the pipe is measured using a pair of static pressure gauges (Fig. 14.5C). In this case, the total headloss through the pipe is the difference between the hydraulic grades at both hydrants. To obtain the hydraulic grade at each hydrant, the observed pressure head (m) must be added to the elevation of the reference point (the hydrant nozzle). For the two-hydrant method, the headloss through the test section H_p (m) can be calculated using the following equation:

$$H_p = \frac{(P_2 - P_1)}{9.81} + (Z_2 - Z_1) \tag{14.8}$$

where P_1 is the pressure reading at the upstream gauge (kPa), Z_1 is the elevation of the upstream gauge (m), P_2 is the pressure reading at the downstream gauge (kPa), and Z_2 is the elevation of the downstream gauge (m).

The difference in elevation between the two gauges should generally be determined using a transit or a level. As a result, one should make certain to select two upstream hydrants that can be seen from a common point. This will minimize the number of turning points required to determine the differences in elevation between the nozzles of the two hydrants. As an alternative to the use of a differential survey, topographic maps can sometimes be used to obtain estimates of hydrant elevations. However, topographic maps usually should not be used to estimate the elevation differences unless the contour interval is 1 m or less. One hydraulic alternative to measuring the elevations directly is to simply measure the static pressure readings (kPa) at both hydrants before the test and convert the observed pressure difference to the associated elevation difference (m) using the relations Z1 − Z2 = [P2(static) − P1(static)]/9.81.

General suggestions. Hydrant pressures for use in pipe-roughness tests are normally measured with a Bourdon tube gauge, which can be mounted to one of the hydrant's discharge nozzles using a lightweight hydrant cap. Bourdon tube gauges come in various grades (i.e., 2A, A, and B), depending on their relative measurement error. In most cases, a grade A gauge (1 percent error) is sufficient for fire-flow tests. For maximum accuracy, one should choose a gauge graded in 5-kPa (1-psi) increments, with a maximum reading less than 20 percent above the expected maximum pressure (McEnroe et al., 1989). In addition, it is a good idea to use pressure snubbers to eliminate the transient effects in the pressure gauges. A pressure snubber is a small valve that is placed between the pressure gauge and the hydrant cap which acts as a surge inhibitor (Walski, 1984).

Before conducting a pipe roughness test, it is always a good idea to make a visual survey of the test area. When surveying the area, make certain that there is adequate drainage away from the flow hydrant. In addition, make certain that you select a hydrant nozzle that will not discharge into oncoming traffic. Also, when working with hydrants in close proximity to traffic, it is a good idea to put up traffic signs and use traffic cones to provide a measure of safety during the test. As a further safety precaution, ensure that all personnel are wearing highly visible clothing. It also is a good idea to equip testing personnel with radios or walkie-talkies to help coordinate the test.

While the methods outlined previously work fairly well with smaller lines (i.e. less than 16in in diameter), their efficiency decreases as you deal with larger lines. Normally, opening hydrants just doesn't generate enough flow for meaningful head-loss determination. For such larger lines you typically have to run conduct the headloos tests over very much longer runs of pipe and use either plant or pump station flow meters or change in tank level to determine flow (Walski, 1999).

14.3.2 Distribution of Nodal Demands

The second major parameter determined in calibration analysis is the average demand (steady-state analysis) or temporally varying demand (extended-period analysis) to be assigned to each junction node. Initial average estimates of nodal demands can be obtained by identifying a region of influence associated with each junction node, identifying the types of demand units in the service area, and multiplying the number of each type by an associated demand factor. Alternatively, the estimate can be obtained by identifying the area associated with each type of land use in the service area, then multiplying the area of each type by an associated demand factor. In either case, the sum of these products will provide an estimate of the demand at the junction node.

14.3.2.1 Spatial distribution of demands. Initial estimates of nodal demands can be developed using various approaches depending on the nature of the data each utility has on file and how precise they want to be. One way to determine such demands is by employing the following strategy.

1. Determine the total system demand for the day to be used in model calibration, (*TD*). The total system demand may be obtained by performing a mass balance analysis for the system by determining the net difference between the total volume of flow which enters the system (from both pumping stations and tanks) and the total volume that leaves the system (through pressure reducing valves (PRVs) and tanks).
2. Use meter records for the day and try to assign all major metered demands (e.g., MD_j, where j = junction node number) by distributing the observed demands among the various junction nodes serving the metered area. The remaining demand will be defined as the total residual demand (*TRD*) and can be obtained by subtracting the sum of the metered demands from the total system demand:

$$TRD = TD - \Sigma\, MD_j \tag{14.9}$$

3. Determine the demand service area associated with each junction node. The most common method of influence delineation is to simply bisect each pipe connected to the reference node, as shown in Fig. 14.7A.
4. Once the service areas associated with the remaining junction nodes have been determined, an initial estimate of the demand at each node should be made. This can be accomplished by identifying the number of different types of demand units within the service area, then multiplying the number of each type by an associated demand factor (Fig. 14.7B). Alternatively, the estimate can be obtained by identifying the area associated with each different type of land use within the service area, then multiplying the area of each type by an associated unit area demand factor (Fig. 14.7C). In either case, the sum of these products will represent an estimate of the demand at the junction node. Although in theory the first approach should be more accurate, the latter approach can be expected to be more expedient. Estimates of unit demand factors are normally available from various water resource handbooks (Cesario, 1995). Estimates of unit area demand factors can normally be constructed for different land use categories by weighted results from repeated applications of the unit demand approach.

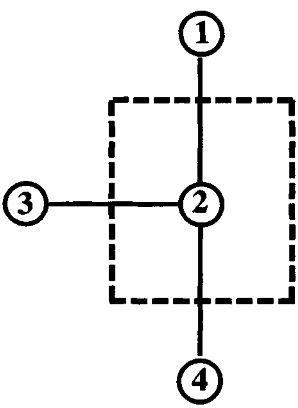
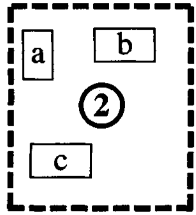
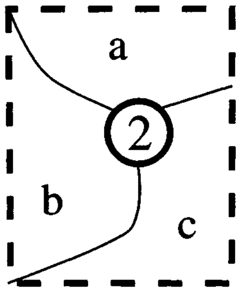


FIGURE 14.7A Delineation of region of influence for node 2.



Type of Establishment	Units	Average Annual Demand (gpd/unit)	Maximum Daily Demand (gpd/unit)
a. metered residential	gpcd	70.00	140.00
b. garden apartment	gpd/unit	213.00	272.00
c. car wash	gpd/ft^2	4.78	10.3

FIGURE 14.7B Demand assignment using individual units.



Type of Land Use	Unit Demand	Area	Total Demand
(gpd/acre)	(acres)	(acres)	(gpd)
a. metered residential	700	5	3500
b. garden apartament	600	4	2400
c. car wash	160,000	1	160000

FIGURE 14.7C Demand assignment using land use units.

5. Once an initial estimate of the demand has been obtained for each junction node j, (IED_j), a revised estimated demand (RED_j) can be obtained using the following equation:

$$RED_j = IED_j * TRD / \Sigma\, IED_j \tag{14.10}$$

6. Finally, with the revised demands obtained for each junction node, the final estimate of nodal demand can be achieved by adding together both the normalized demand and the metered demand (assuming there is one) associated with each junction node:

$$D_j = RED_j + MD_j \tag{14.11}$$

14.3.2.2 Temporal distribution of demands. Time-varying estimates of model demands for use in extended-period analysis can be made in one of two ways, depending on the structure of the hydraulic model. Some models allow the user to subdivide the demands at each junction node into different use categories, which can then be modified separately over time using demand factors for water-use categories. Other models require an aggregate-use category for each node. In the latter case, spatial-temporal variations of nodal demands are obtained by lumping nodes of a given type into separate groups, which can then be modified uniformly using nodal demand factors. Initial estimates of either water-use category demand factors or nodal-demand factors can be obtained by examining historical meter records for various water-use categories and by performing incremental mass-balance calculations for the distribution system. The resulting set of temporal demand factors can then be fine-tuned through subsequent calibration of the model.

14.4 COLLECT CALIBRATION DATA

After model parameters have been estimated, the accuracy of the model parameters can be assessed. This is done by executing the computer model using the estimated parametric values and observed boundary conditions and by comparing the model results with the results from actual field observations. Data from fire-flow tests, pump-station flowmeter readings, and tank telemetric data are used most commonly in such tests.

In collecting data for model calibration, it is very important to recognize the significant impact of measurement errors. For example, with regard to calibrating pipe roughness, the C factor may expressed as:

$$C = k(V + \text{error})/(h + \text{error})^{0.54} \tag{14.12}$$

If the magnitude of V and h are on the same order of magnitude as the associated measurement errors (for V and h) then the collected data will be essentially useless for model calibration. That is to say, virtually any value of C will provide a "reasonable" degree of model calibration (Walski, 1986). However, one can hardly expect a model to accurately predict flows and pressures for a high stress situation (i.e. large flows and velocities) if the model was calibrated using data from times when the velocities in the pipes were less than the measurement error (e.g. less than 1 ft/s). The only way to minimize this problem is to either insure that the measurement errors are reduced or the velocity or headloss values are significantly greater than the associated measurement error. This latter condition can normally be met either using data from fire flow tests or by collecting flow or pressure reading during periods of high stress (e.g., peak hour demand periods).

14.4.1 Fire-Flow Tests

Fire-flow tests are useful for collecting both discharge and pressure data for use in calibrating hydraulic network models. Such tests are normally conducted using both a normal pressure gauge (to measure both static and dynamic heads) and a pitot gauge (to calculate discharge). In performing a fire-flow test, at least two separate hydrants are selected for use in the data collection effort. One hydrant is identified as the pressure or residual hydrant and the other hydrant is identified as the flow hydrant. The general steps for performing a fire flow test can be summarized as follows (McEnroe et al., 1989):

1. Place a pressure gauge on the residual hydrant and measure the static pressure.
2. Determine which of the discharge hydrant's outlets can be flowed with the least amount of adverse impact (flooding, traffic disruption, and so on).
3. Make certain that the discharge hydrant is initially closed to avoid injury.
4. Remove the hydrant cap from the nozzle of the discharge hydrant to be flowed.
5. Measure the inside diameter of the nozzle and determine the type of nozzle (i.e, rounded, square edge, or protruding) to determine the appropriate discharge coefficient. (Fig. 14.6).
6. Take the necessary steps to minimize erosion or the impact of traffic during the test.
7. Flow the hydrant briefly to flush sediment from the its lateral and barrel.
8. If using a clamp-on pitot tube, attach the tube to the nozzle to be flowed, then slowly open the hydrant. If using a hand held pitot tube, slowly open the hydrant and then place the pitot tube in the center of the discharge stream, being careful to align it directly into the flow.
9. Once an equilibrium flow condition has been established, make simultaneous pressure readings from both the pitot tube and the pressure gauge at the residual hydrant.
10. Once the readings are completed, close the discharge hydrant, remove the equipment from both hydrants, and replace the hydrant caps.

To obtain sufficient data for an adequate model calibration, data from several fire flow tests must to collect be collected. Before conducting each test, it also is important to collect the associated system boundary condition data. This includes information on tank levels, pump status, and so forth. To obtain an adequate model calibration, it is normally desirable for the difference between the static and dynamic pressure readings measured from the residual hydrant to be at least 35 kPa (5 psi), preferably with a drop of 140 kpa (20 psi) (Walski, 1990a). In the event that the discharge hydrant does not allow sufficient discharge to cause such a drop, it may be necessary to identify, instrument, and open additional discharge hydrants.

In some instances, it may also be beneficial to use more than one residual hydrant (one near the flowed hydrant and one off the major main from the source). The information gathered from such additional hydrants can sometimes be very useful in tracking down closed valves (Walski, (1999).

14.4.2 Telemetric Data

In addition to static test data, data collected over an extended period (typically 24 h) can be useful when calibrating network models. The most common type of data will include flow-rate data, tank water-level data, and pressure data. Depending on the level of instrumentation and telemetry associated with the system, much of the data may already have been collected as part of the normal operations. For example, most systems collect and record tank levels and average pump station discharges on an hourly basis. These data are especially useful to verify the distribution of demands among the various junction nodes. If such data are available, they should be checked for accuracy before using them in the calibration effort. If such data are not readily available, the modeler may have to install temporary pressure gauges or flowmeters to obtain the data. In the absence of flowmeters in lines to tanks, inflow or discharge flow rates can be inferred from incremental readings of the tank level.

14.4.3 Water-Quality Data

In recent years, both conservative and nonconservative constituents have been used as tracers to determine the travel time through various parts of a water distribution system (Cesario, et al., 1996; Grayman, 1998; Kennedy et al., 1991). The most common type of tracer for such applications is fluoride. By controlling the injection rate at a source, typically the water treatment plant, a pulse can be induced into the flow that can then be monitored elsewhere in the system. The relative travel time from the source to the sampling point can be determined. The measured travel time thus provides another data point for use in calibrating a hydraulic network model.

Alternatively, the water distribution system can be modeled using a water-quality model such as EPANET (Rossman, 1994). In this case, the water quality-model is used to predict tracer concentrations at various points in the system. Since the result of all water-quality results depend on the underlying hydraulic results, deviations between the observed and predicted concentrations can thus provide a secondary means of evaluating the adequacy of the underlying hydraulic model. For additional insights into water-quality modeling and the use of such models in calibration, refer to Chap.9.

14.5 EVALUATE THE RESULTS OF THE MODEL

In using fire-flow data, the model is used to simulate the discharge from one or more fire hydrants by assigning the observed hydrant flows as nodal demands within the model. The flows and pressures predicted by the model are then compared with the corresponding observed values in an attempt to assess the accuracy of the model. In using telemetric data, the model is used to simulate the variation of tank water levels and system pressures by simulating the operating conditions for the day over which the field data was collected. The predicted tank water levels are then compared with the observed values in an attempt to assess the model's accuracy. In using water-quality data, the travel times (or constituent concentrations) are compared with model predictions in an attempt to assess the model's accuracy.

The accuracy of the model can be evaluated using a variety of criteria. The most common criterion is absolute pressure difference (normally measured in psi) or relative pressure difference (measured as the ratio of the absolute pressure difference to the average pressure difference across the system). In most cases, a relative pressure difference criterion is usually preferred. For extended-period simulations, comparisons are normally made between the predicted and observed tank water levels. To a certain extent, the desired level of model calibration will be related to the intended use of the model. For example, a higher level of model calibration will normally be required for analysis of water quality or an operational study rather than use of the model in a general planning study. Ultimately, the model should be calibrated to the extent that the associated application decisions will not be affected significantly. In the context of a design application, the model should normally be calibrated to such an extent that the resulting design values (e.g., pipe diameters and tank and pump sizes or locations) will be the same as if the exact parameter values were used. Determining such thresholds often requires the application of model sensitivity analysis (Walski, 1995).

Because of the issue of model application, deriving a single set of criteria for a universal model calibration is difficult. From the authors' perspective, a maximum deviation of the state variable (i.e., pressure grade, water level, flow rate) of less than 10 percent is generally satisfactory for most planning applications, whereas while a maximum deviation of less than 5 percent is highly desirable for most design, operation, or water quality applications. Although no such general set of criteria has been officially developed for the United States, a set of "Performance Criteria" has been developed by the Sewers and Water Mains Committee of the Water Authorities Association (1989) in the United Kingdom. For steady-state models, the criteria are as follows:

1. Flows agree to 5 percent of measured flow when flows are more than 10 percent of total demand, and to 10 percent of measured flow when flows are less than 10 percent of total demand.
2. Pressures agree to 0.5 m (1.6 ft) or 5 percent of headloss for 85 percents of test measurements, to 0.75m (2.31 ft) or 7.5 percent of headloss for 95 percent of test measurements, and to 2 m (6.2 ft) or 15 percent of headloss for 100 percent of test measurements.

For extended-period simulation, the criteria require that three separate steady-state calibrations must be performed for different time periods and that the average volumetric difference between measured and predicted reservoir storage must be within 5 percent. Additional details can be obtained directly from the Water Authorities Asociation's report (1989).

Deviations between the results of the model application and the field observations can be caused by several factors, including (1) erroneous model parameters (e.g. pipe-roughness values and nodal demand distribution), (2) erroneous network data (e.g. pipe-diameters or lengths), (3) incorrect network geometry (e.g. pipes connected to the wrong nodes), (4) incorrect pressure zone boundary definitions, (5) errors in boundary conditions (e.g. incorrect PRV value settings, tank water levels, pump curves), (6) errors in historical operating records (e.g. pumps starting and stopping at incorrect times), (7) measurement equipment errors (e.g. pressure gauges not properly calibrated), and (8) measurement errors (e.g. reading the wrong values from instruments). It is hoped that the last two sources of errors can be eliminated, or minimized at least, by developing and implementing a careful data-collection effort. Eliminating the remaining errors frequently requires the iterative application of the last three steps of the model calibration process—macro-level calibration, sensitivity, and micro-level calibration. Each of these steps is described in the following sections.

14.6 PERFORM A MACRO-LEVEL CALIBRATION OF THE MODEL

In the event that one or more of the measured state variable values differ from the modeled values by an amount that is deemed to be excessive (i.e., greater than 30 percent), the cause of the difference is likely to extend beyond errors in the estimates for either the pipe-roughness values or the nodal demands. Although such differences have many possible causes, they may include (1) closed or partially closed valves, (2) inaccurate pump curves or tank telemetry data, (3) incorrect pipe sizes (e.g., 6 in instead of 16 in), (4) incorrect pipe lengths, (5) incorrect network geometry, and (6) incorrect pressure zone boundaries, (Walski, 1990a).

The only way to address such errors adequately is to review the data associated with the model systematically to ensure the model's accuracy. In most cases, some data will be less reliable than others. This observation provides a logical place to begin an attempt to identify the problem. Model sensitivity analysis provides another means of identifying the source of the discrepancy. For example, if one suspects that a valve is closed, this assumption can be modeled by simply closing the line in the model and evaluating the resulting pressures. Potential errors in pump curves can sometimes be minimized by simulating the pumps with negative inflows set equal to observed pump discharges (Cruickshank and Long, 1992). This of course assumes that the error in the observed flow rates (and the induced head) are less that the errors introduced by using the pump curves. In any case, only after the model results and the observed conditions are within some reasonable degree of correlation (usually less than a 20 percent error) should the final step of micro-level calibration be attempted.

14.7 PERFORM A SENSITIVITY ANALYSIS

Before attempting a micro-level calibration, it is helpful to perform a sensitivity analysis of the model to identify the most likely source of model error. This analysis can be accomplished by varying the different model parameters by different amounts, then measuring the associated effect. For example, many current network models have as an analysis option the capability to make multiple simulations in which global adjustment factors can be applied to pipe-roughness values or nodal-demand values. By examining such results, the user can begin to identify which parameters have the most significant impact on the model results and thereby identify potential parameters for subsequent fine-tuning through micro-level calibration.

14.8 PERFORM A MICRO-LEVEL CALIBRATION OF THE MODEL

After the model results and the field observations are in reasonable agreement, a micro-level model calibration should be performed. As discussed previously, the two parameters adjusted during this final calibration phase normally will include pipe roughness and nodal demands. In many cases, it may be useful to break the micro calibration into two separate steps: steady-state calibration, and extended-period calibration. In a steady-state calibration, the model parameters are adjusted to match pressures and flow rates associated with static observations. The normal source of such data is fire-flow tests. In an extended-period calibration, the model parameters are adjusted to match time-varying pressures and flows as well as tank water-level trajectories. In most cases the steady state calibration is more sensitive to changes in pipe roughness, whereas the extended-period calibration is more sensitive to changes in the distribution of demands. As a result, one potential calibration strategy would be to fine-tune the pipe-roughness parameter values using the results from fire-flow tests and then try to fine-tune the distribution of demands using the flow-pressure-water level telemetric data.

Historically, most attempts at model calibration have typically used an empirical or a trial-and-error approach. However, such an approach can be extremely time-consuming and frustrating when dealing with most typical water systems. The level of frustration will, of course, depend to some degree on the modeler's expertise, the size of the system, and the quantity and quality of the field data. Some of the frustration can be minimized by breaking complicated systems into smaller parts and calibrating the model parameters using an incremental approach. Calibration of multitank systems can sometimes be facilitated by collecting multiple data sets with all but one of the tanks closed (Cruickshank and Long, 1992). In recent years, several researchers have proposed different algorithms for use in automatically calibrating hydraulic network models. These techniques have been based on the use of analytical equations (Walski, 1983), simulation models (Boulos and Ormsbee, 1991; Gofman and Rodeh, 1981; Ormsbee and Wood, 1986; Rahal et al., 1980) and optimization methods (Coulbeck, 1984; Lansey and Basnet, 1991; Meredith, 1983; Ormsbee, 1989; and Ormsbee et al., 1992).

14.8.1 Analytical Approaches

In general, techniques based on analytical equations require significant simplification of the network through skeletonization and the use of equivalent pipes. As a result, such techniques may only get the user close to the correct results. Conversely, both simulation and optimization approaches take advantage of using a complete model.

14.8.2 Simulation Approaches

Simulation techniques are based on the idea of solving for one or more calibration factors through the addition of one or more network equations. The additional equation or equations are used to define an additional observed boundary condition (such as fire-flow discharge head). With the addition of an extra equation, an additional unknown can be determined explicitly.

The primary disadvantage of simulation approaches is that they can handle only one set of boundary conditions at a time. For example, in applying a simulation approach to a system with three different sets of observations—all of which were obtained under different boundary conditions (e. g.) different tank levels or pump statuses-three different results can be expected. Attempts to obtain a single calibration result will require one of two application strategies: a sequential approach or an average approach. In the sequential approach, the system is subdivided into multiple zones, the number of which will correspond to the number of sets of boundary conditions. In this case, the first set of observations is used to obtain calibration factors for the first zone. These factors are then fixed, another set of factors is determined for the second zone, and so on. In the average approach, final calibration factors are obtained by averaging the calibration factors for each individual calibration application.

14.8.3 Optimization Approaches

The primary alternative to the simulation approach is an optimization approach. When an optimization approach is used, the calibration problem is formulated as a nonlinear optimization problem consisting of a nonlinear objective function subject to both linear and nonlinear equality and inequality constraints. Using standard mathematical notation, the associated optimization problem can be expressed as follows:

Minimize
$$z = f(\mathbf{X}) \tag{14.13}$$

Subject to
$$\mathbf{g}(\mathbf{X}) = \mathbf{0} \tag{14.14}$$

$$L_h \leq \boldsymbol{h}\,(\mathbf{X}) \leq U_h = \mathbf{0} \tag{14.15}$$

$$L_x \leq \mathbf{X} \leq U_x \tag{14.16}$$

where **X** is the vector of decision variables (e.g., pipe –roughness coefficients, nodal demands),

$f(\mathbf{X})$ is the nonlinear objective function,

$g(\mathbf{X})$ is a vector of implicit system constraints,

$h(\mathbf{X})$ is a vector of implicit bound constraints, and

L and U are the vectors of lower and upper bounds respectively on the explicit system constraints and the decision variables.

Normally, the objective function will be formulated in a way that minimizes the square of the differences between observed and predicted values of pressures and flows. Mathematically, this can be expressed as:

$$f(\mathbf{X}) = a \sum_{j=1}^{J} (OP_j - PP_j)^2 + b \sum_{p=1}^{P} (OQ_P - PQq)^2 \tag{14.17}$$

where OP_j = the observed pressure at junction j, PP_j = the predicted pressure at junction j, OQ_p = the observed flow in pipe p, PQ_p = the predicted flow in pipe p, and a and b are normalization weights.

The implicit bound constraints on the problem may include both pressure-bound constraints and flow rate-bound constraints. These constraints can be used to ensure that the resulting calibration does not produce unrealistic pressures or flows as a result of the model calibration process. For a given vector of junction pressures **P** these constraints can be expressed mathematically as

$$L_p \leq \mathbf{P} \leq U_P \tag{14.18}$$

Similarly, for a given vector of pipe flows **Q**, these constraints can be expressed as

$$L_Q \leq \mathbf{Q} \leq U_Q \tag{14.19}$$

The explicit bound constraints can be used to set limits on the explicit decision variables of the calibration problem. Normally, these variables will include the roughness coefficient of each pipe and the demands at each node. For a given vector of pipe-roughness coefficients **C**, these constraints can be expressed as

$$L_C \leq \mathbf{C} \leq U_C \tag{14.20}$$

Similary, for a given vector of nodal demands **D**, these constraints can be expressed as

$$L_D \leq \mathbf{D} \leq U_D \tag{14.21}$$

The implicit system constraints include nodal conservation of mass and conservation of energy. The nodal conservation of mass equation $F_c\,(Q)$ requires that the sum of flows into or out of any junction node n minus any external demand D_j must be equal to zero. For each junction node j, this may be expressed as

$$F_c\,(Q) = \sum_{n \in \{j\}}^{N_j} Q_n - D_j = 0 \tag{14.22}$$

where N_j = the number of pipes connected to junction node j and $\{j\}$ is the set of pipes connected to junction node j.

The conservation of energy constraint $F_e(Q)$ requires that the sum of the line loss (HL_n) and the minor losses (HM_n) over any path or loop k, minus any energy added to the liquid by a pump (EP_n), minus the difference in grade between two points of known energy (DE_k) is equal to zero. For any loop or path k, this may be expressed as

$$F_e(Q) = \sum_{n \in \{j\}}^{N_k} (HL_n + HM_n - EP_n) - DE_k = 0 \tag{14.23}$$

where N_k = the number of pipes associated with loop or path k, and $\{k\}$ is the set of pipes associated with loop or path k. It should be emphasized that HL_n, HM_n, and EP_n are all nonlinear functions of the pipe discharge Q.

Although both the implicit and explicit bound constraints have traditionally been incorporated directly into the nonlinear problem formulation, the implicit system constraints have been handled using one of two different approaches. In the first approach, the implicit system constraints are incorporated directly within the set of nonlinear equations and are solved using normal nonlinear programming methods. In the second approach, the equations are removed from the optimization problem and are evaluated externally using mathematical simulation; Lansey and Basnet, 1991; Ormsbee, 1989). Such an approach allows for a much smaller and more tractable optimization problem because both sets of implicit equations (which constitute linear and nonlinear equality constraints to the original problem) can now be satisfied much more efficiently using an external simulation model (Fig. 14.7). The basic idea behind the approach is to use an implicit optimization algorithm to generate a vector of decision variables, which are then passed to a lower-level simulation model for use in evaluating all implicit system constraints. Feedback from the simulation model will include numerical values for use in identifying the status of each constraint as well as numerical results for use in evaluating the associated objective function.

Regardless of which approach is chosen, the resulting mathematical formulation must then be solved using some type of nonlinear optimization method. In general, three different approaches have been proposed and used: (1) gradient-based methods, (2) pattern-search methods, and (3) genetic optimization methods.

Gradient-based methods require either first or second derivative information to produce improvements in the objective function. Traditionally, constraints are handled using either a penalty method or the Lagrange multiplier method (Edgar and Himmelblau, 1988). Pattern search methods employ a nonlinear heuristic that uses objective function values only to determine a sequential path through the region of search (Ormsbee, 1986, Ormsbee and Lingireddy, 1995). In general, when the objective function can be differentiated explicitly with respect to the decision variables, the gradient methods are preferable to search methods. When the objective function is not an explicit function of the decision variables, as normally is the case with the current problem, then the relative advantage is not as great, although the required gradient information can still be determined numerically.

Recently, several researchers have begun to investigate the use of genetic optimization to solve such complex nonlinear optimization problems (Lingireddy and Ormsbee, 1998; Lingireddy et.al., 1995; Savic and Walters, 1995). Genetic optimization offers a significant advantage over more traditional optimization approaches because it attempts to obtain an optimal solution by continuing to evaluate multiple solution vectors simultaneously (Goldberg, 1989). In addition, genetic optimization methods do not require gradient information. Finally, because these methods use probabilistic transition rules as opposed to deterministic rules, they have the advantage of insuring a robust solution methodology.

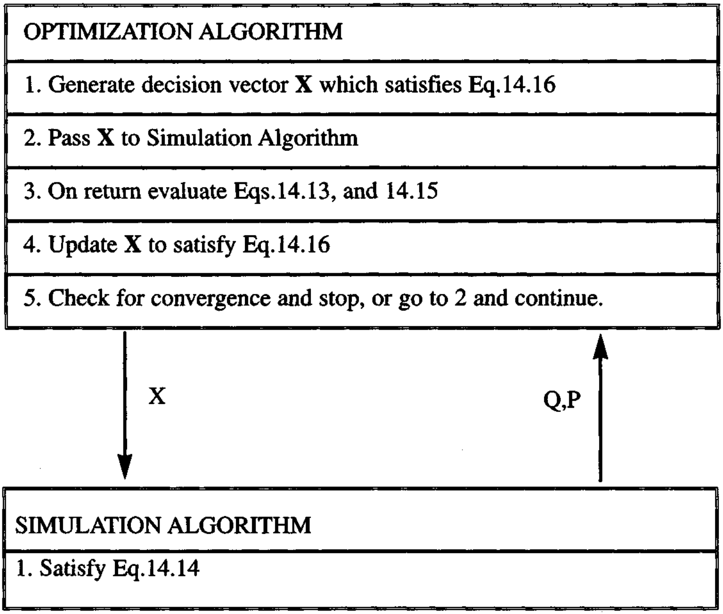


FIGURE 14.8 Bi-level computational framework.

Genetic optimization begins with an initial population of randomly generated decision vectors. For an application to network calibration, each decision vector could consist of a subset of pipe-roughness coefficients, nodal demands, and so on. The final population of decision vectors is then determined through an iterative solution method that uses three sequential steps: evaluation, selection, and reproduction. The evaluation phase involves determination of the value of a fitness function (objective function) for each element (decision vector) in the current population. On the basis of these evaluations, the algorithm then selects a subset of solutions for use in reproduction. The reproduction phase of the algorithm involves the generation of new offspring (additional decision vectors) using the selected pool of parent solutions. Reproduction is accomplished through the process of crossover in which the numerical values of the new decision vector are determined by selecting elements from two parent decision vectors. The viability of the solutions thus generated is maintained by random mutations that occasionally are introduced into the resulting vectors. The resulting algorithm is thus able to generate a whole family of optimal solutions and thereby increase the probability of obtaining a successful calibration of the model.

Although optimization in general and genetic optimization in particular offer very powerful algorithms for use in calibrations a water distribution model, the user should always recognize that the utility of the algorithms are very much dependent upon the accuracy of the input data. Such algorithms can be susceptible to convergence problems when the errors in the data are significant (e.g., headloss is on the same order of magnitude as the error in headloss). In addition, because most network model calibration problems are under-specified (i.e. roughness coefficients, junction demands) can give reasonable pressures if the system is not reasonably stressed when the data are collected.

14.9 FUTURE TRENDS

With the advent and use of nonlinear optimization, it is possible to achieve some measure of success in the area of micro-level calibration. Of course, the level of success will be highly dependent upon the degree that the sources of macro-level calibration errors have first been eliminated or at least significantly reduced. Although these sources of errors may not be identified as readily with conventional optimization techniques, it may be possible to develop prescriptive tools for these problems using expert system technology. In this case, general calibration rules could be developed from an experiential database that could be used by other modelers attempting to identify the most likely source of model error for a given set of system characteristics and operating conditions. Such a system also could be linked with a graphical interface and a network model to provide an interactive environment for use in model calibration.

In recent years, there has been a growing advocacy for the use of both geographic information systems (GIS) technology and Supervisor Control and Data Acquisition (SCADA) system databases in model calibration. GIS technology provides an efficient way to link customer's billing records with network model components for use in assigning initial estimates of nodal demands (Basford and Sevier, 1995). Such technology also provides a graphical environment for examining the network database for errors. Among the more interesting possibilities with regard to network model calibration is the development and implementation of an on-line network model through linkage of the model with an on-line SCADA system. Such a configuration provides the possibility for a continuing calibration effort in which the model is continually updated as additional data are collected through the SCADA system (Schulte and Malm, 1993).

Finally, Bush and Uber (1998) have recently developed three sensitivity-based metrics to rank potential sampling locations for use in model calibration. Although the documented sampling application was small, the approach the authors developed provides a potential basis for selecting improved sampling sites for improved model calibration. This area of research is expected to see additional activity in future years.

14.10 SUMMARY AND CONCLUSION

Network model calibration should always be performed before any network-analysis planning and design study is conducted. A seven-step methodology for network model calibration has been proposed. Historically, one difficult step in the process has been the final adjustment of pipe-roughness values and nodal demands through the process of micro-level calibration. With the advent of recent computer technology it is now possible to achieve good model calibration with a reasonable level of success. As a result, little justification remains for failing to develop good calibrated network models before conducting a network analysis. Future developments and applications of both GIS and SCADA technology as well as optimal sampling algorithms should lead to even more efficient tools.

REFERENCES

Basford, C., and C. Sevier, "Automating the Maintenance of a Hydraulic Network Model Demand Database Utilizing GIS and Customer Billing Records," *Proceedings of the 1995 AWWA Computer Conference*, Norfolk, VA, 1995, pp. 197–206.

Boulos, P., and L. Ormsbee; "Explicit Network Calibration for Multiple Loading Conditions," *Civil Engineering Systems*, 8: 153–160, 1991.

Brion, L. M., and L. W. Mays, "Methodology for Optimal Operation of Pumping Stations in Water Distribution Systems," *ASCE Journal of Hydraulic Engineering*, 117(11), 1991.

Bush, C. A., and J. G. Uber, "Sampling Design Methods for Water Distribution Model Calibration," *ASCE Journal of Water Resources Planning and Management*, 124:334–344, 1998.

Cesario, L., *Modeling, Analysis and Design of Water Distribution Systems*, American Water Works Association, Denver, CO, 1995.

Cesario, L., J. R. Kroon, W. M. Grayman, and G. Wright, "New Perspectives on Calibration of Treated Water Distribution System Models," *Proceedings of the AWWA Annual Conference*, Toronto, Canada, 1996.

Coulbeck, B., "An Application of Hierachial Optimization in Calibration of Large Scale Water Networks," *Optimal Control Applications and Methods*, 6:31–42, 1984.

Cruickshank, J. R, and S. J. Long, "Calibrating Computer Model of Distribution Systems," Proceedings of the 1992 AWWA Computer Conference, Nashville, TN, 1992.

Edgar, T. F., and D. M. Himmelblau, *Optimization of Chemical Processes*, McGraw Hill, New York, pp 334–342, 1998.

Gofman, E., and M. Rodeh, "Loop Equations with Unknown Pipe Characteristics," *ASCE Journal of the Hydraulics Division*, 107:1047–1060, 1981.

Goldberg, D. E., *Genetic Algorithms in Search, Optimization and Machine Learning*, Addison-Wesley, Reading, MA, 1989.

Grayman, W. M., "Use of Trace Studies and Water Quality Models to Calibrate a Network Hydraulic Model," in *Essential Hydraulics and Hydrology*, Haested Press, 1998.

Kennedy, M., S. Sarikelle, and K. Suravallop, "Calibrating Hydraulic Analyses of Distribution Systems Using Fluoride Tracer Studies," *Journal of the American Water Works Association*, 83(7):54–59, 1991.

Lamont, P.A., "Common Pipe Flow Formulas Compared with the Theory of Roughness," *Journal of the AWWA*, 73(5),274, 1981.

Lansey, K, and C Basnet, "Parameter Estimation for Water Distribution Networks," *ASCE Journal of Water Resources Planning and Management*, 117(1), 126-145, 1991

Lingireddy, S., L.E. Ormsbee, and D.J., Wood, User's Manual – KYCAL, Kentucky Network Model Calibration Program, Civil Engineering Software Center, University of Kentucky, Lexington, KY, 1995.

Lingireddy, S., and L. E., Ormsbee, "Neural Networks in Optimal Calibration of Water Distribution Systems," I. Flood and N. Kartam (eds.), *Artificial Neural Networks for Civil Engineers: Advanced Features and Applications*, American Society of Civil Engineers, p. 277, 1998.

McEnroe, B., D., Chase, and W. Sharp, "Field Testing Water Mains to Determine Carrying Capacity," *Technical Paper EL-89*, Environmental Laboratory of the Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS, 1998.

Meredith, D. D., "Use of Optimization in Calibrating Water Distribution Models," *Proceeding of ASCE Spring Convention*, Philadelphia, PA, 1983.

Ormsbee, L.E., "Implicit Pipe Network Calibration," *ASCE Journal of Water Resources Planning and Management*, 115(2):243–257, 1989.

Ormsbee, L. E., "A nonlinear heuristic for applied problems in water resources," *Proceedings of the Seventeenth Annual Modeling and Simulation Conference*, University of Pittsburgh, 1986, pp. 1117–1121.

Ormsbee, L. E., D.V. Chase, and W. Grayman, "Network Modeling for Small Water Distribution Systems," *Proceedings of the AWWA 1992 Computer Conference*, Nashville, TN, 1992, pp. 15–19.

Ormsbee, L., D. V. Chase, and W. Sharp, "Water Distribution Modeling", *Proceedings of the 1991 AWWA Computer Conference*, Houston, TX, April 14–17, 1991, pp. 27–35.

Ormsbee, L. E. and D. V. Chase, "Hydraulic Network Calibration Using Nonlinear Programming," *Proceedings of the International Symposium on Water Distribution Modeling*, Lexington, KY, 1988, pp. 31–44.

Ormsbee, L. E. and S. Lingireddy, "Nonlinear Heuristic for Pump Operations," ASCE *Journal of Water Resources Planning and Management*, 121 (4):302–309., 1995.

Ormsbee, L. E., and D.J. Wood, "Explicit Pipe Network Calibration," *ASCE Journal of Water Resources Planning and Management*, 112(2):166–182, 1986.

Rahal, C. M., M.J.H. Sterling, and B. Coulbeck, "Parameter tuning for Simulation Models of Water Distribution Networks," *Proceedings of the Institution of Civil Engineers*, London, UK, 69(2):751–762, 1980.

Rossman, L., EPANET User's Manual, Drinking Water Research Division, Risk Reduction Engineering Laboratory, Cincinnati, OH, 1994.

Savic, D. A., and G.A. Walters, Genetic Algorithm Techniques for Calibrating Network Models, Report No. 95/12, Center for Systems and Control, University of Exeter, UK, 1995.

Schulte, A. M., and A. P. Malm, "Integrating Hydraulic Modeling and SCADA Systems for System Planning and Control," *Journal of the American Water Works Association*, 85(7):62–66, 1993.

Walski, T. M. "Standards for model calibration," *Proceedings of the 1995 AWWA Computer Conference*, Norfolk, VA, pp. 55–64, 1995.

Walski, T. M., "Sherlock Holmes Meets Hardy Cross, or Model Calibration in Austin, Texas, *Journal of the American Water Works Association*, 82(3):34, 1990a.

Walski, T. M., *Water Distribution Systems: Simulation and Sizing, Lewis Publishers*, Chelsea, MI, 1990b.

Walski, T. M., *Analysis of Water Distribution Systems*, Van Nostrand Reinhold, New York, 1984.

Walski, T. M, "Technique for Calibrating Network Models," *ASCE Journal of Water Resources Planning and Management*, 109(4):360–372, 1983.

Water Authorities Association and WRc, *Network Analysis — A Code of Practice*, WRc, Swindon, UK, 1989.

Wood, D. J., Comprehensive Computer Modeling of Pipe Distribution Networks, Civil Engineering Software Center, College of Engineering, University of Kentucky, Lexington, KY, 1991.

DETERMINING THE ACCURACY OF AUTOMATED CALIBRATION OF PIPE NETWORK MODELS

Thomas M. Walski[1], Nicholas DeFrank[2], Thomas Voglino[2], Rick Wood[2], and Brian E. Whitman[3]
[1]Bentley Systems Inc., Senior Advisory Product Manager
3 Brian's Place, Nanticoke, PA 18634
Tom.Walski@bentley.com
[2]Former undergraduate students, Environmental Engineering and Earth Sciences Department, Wilkes University
Wilkes-Barre, Pennsylvania 18766
[3]Associate Professor, Environmental Engineering and Earth Sciences Department, Wilkes University
Wilkes-Barre, Pennsylvania 18766
brian.whitman@wilkes.edu

Abstract

Methods to automatically calibrate water distribution system models have been available for some time but it is very difficult to prove that any method is correct. Since at any one time the ability to know all the usage and flow conditions in a real system is impossible, obtaining all of the data needed in a real water distribution system to obtain an accurate and complete data set for model calibration is unrealistic. To test the ability of automated calibration methods to predict the actual conditions in a water system a laboratory scale physical model of a water distribution system was constructed and an automated water distribution model calibration program, employing genetic algorithms, was used to calibrate the model of that system.

The results indicated that the automated calibration methods worked well in estimating pipe roughness, demands and locating closed valves. More specifically, the automated calibration model exactly matched the measured flows and pressures in the system. It was able to identify whether a valve was closed and where the demands were located. If given sufficient data, it was able to identify pipe roughness. The only problems occurred when the number of unknowns greatly exceeded the number of measurements. The model worked equally well regardless of whether the head loss equation used was the Hazen-Williams, Darcy-Weisbach or Manning equation. In all, automated calibration was successful.

The paper describes how the lab data were collected, and how the calibration program matched the lab data and provides some suggestions for users of an automated water distribution calibration model.

Keywords
Water distribution modeling, calibration, genetic algorithm, optimal calibration, pipe roughness

1. BACKGROUND

Water distribution models are frequently used both for design and operation by virtually all water utilities. The value of these models is directly related to how well the model represents the real water distribution system. Utilities improve the quality of their model through a process known as model calibration in which: 1. model results are compared with field measurement of system parameters and 2. the model is adjusted to better match the real system (Herrin, 1997; Walski, et al., 2003). However, making the appropriate adjustments to the model is often difficult because there are so many parameters to be adjusted. It is often difficult to determine which parameter is causing the problem or if the problem actually lies with the field data (Walski, 1990). Problems with calibration can be traced to a large number of sources including incorrect pipe roughness, inaccurate demands, incorrect valve status, and erroneous field data.

Calibration has traditionally been a manual trial-and-error process where the modeler estimates the parameter adjustment that might bring the model into agreement with field data. These trials continue until the modeler is satisfied with calibration or cannot justify additional effort to further improve the model. Ideally, this manual calibration process can be improved by having the computer take over much of the trial-and-error work in calibration. Unfortunately it is difficult if not impossible to tell how well a model has been calibrated because it is nearly impossible to know the exact water consumption by each customer, the exact roughness for each pipe, or the correct setting of each valve at the time when the field measurements were obtained. Because the correct value of every parameter in a model is not known, it is impossible to know if any automated (or manual) calibration process is actually correct.

In this study, a laboratory-scale physical model of a distribution system was constructed and an automated calibration model was used to determine the model input parameters. The values from the automated calibration model were compared with the measured values obtained from the lab model. This paper discusses the literature on automated calibration, presents the Darwin Calibrator (the automated calibration program used in this study), describes the physical model constructed in a laboratory, and presents the results of the comparison between the physical model observations and the Darwin Calibrator results.

1.1 Literature Review

Model calibration has traditionally been a trial-and-error process. Because of the large number of potential unknowns, it is not possible to analytically solve all calibration parameters. Early methods to calibrate models (Shamir and Howard, 1968; Walski, 1983; Ormsbee and Wood 1986, Bhave, 1988) often used an approach where the number of unknowns matches the number of observations so that an explicit solution could be determined. This required a great deal of judgment to determine how to group the unknowns. This resulted in the use of models that could be erroneous if the assumptions used to group unknowns were invalid. Therefore, some form of optimization needed to be used when calibrating models so that it was not necessary to solve a number of equations equal to the number of unknowns. Meredith (1983) produced the first optimization model for calibration which was based on linear programming.

With the advance of computing technology and optimization techniques, more automated calibration methods were developed. Most methods for automated calibration relied on optimization and fall under the general category of “parameter estimation”. By using input values known as “state variables” such as measured flows and heads, the model can determine “control or decision variables” such as pipe roughness or demand. Ormsbee (1989) and Ormsbee and Chase (1988) applied optimization to calibrate models. Lansey and Basnet (1991) developed an optimization model which could match field observations but noted that sufficient quantities of high quality data were necessary to make it work well. This concurred with Walski’s observation (1986, 2000) that sufficient head loss is needed in the system for automated calibration to work, which was successfully illustrated in a sample system (Walski, 2001). Araujo and Lansey (1991) and Lansey et al. (2001) quantified how measurement errors propagated through the model calibration processes.

Methods to evaluate and optimize model calibration vary widely. Datta and Sridharan, (1994) and Reddy, Sridharan, and Rao (1996) used least squares methods to arrive at calibrated models. Greco and Del Giudice (1999) used a “sensitivity matrix” to minimize the least squares difference between observed and predicted values. Lingireddy and Ormsbee (1998) developed a method using neural networks for model calibration.

Wang (1991) was the first to apply a genetic algorithm to the calibration of a conceptual hydrology model. Wu (1994) developed a genetic algorithm approach for automatic calibration of an integrated hydrology and hydrodynamic modeling system. Savic and Walters (1995) developed a genetic algorithm method for water distribution model calibration. Genetic algorithm methods showed a great deal of promise in that they were robust and weren't hampered by local minima. Wu et al. (2002a) developed the Darwin Calibrator which used a competent genetic algorithm paradigm (Wu and Simpson 2001) which included the ability to identify correct values for demand and valve status as well as pipe roughness.

Because it is not possible to accurately know the roughness of every pipe and demands at every node in a real water system, the methods described above could only be tested against hypothetical solutions (i.e. the "correct" solution was generated by a model).

1.2 Calibration Using a Genetic Algorithm Program

The Darwin Calibrator is the genetic algorithm program used in this study. It is an add-on program to WaterCAD and WaterGEMS programs (Haestad Methods, 2006). The Darwin Calibrator uses a genetic algorithm approach developed by Wu and is described in papers by Wu et al. (2002a, 2002b).

Once the user has constructed a model of a water distribution system, the user enters field data. The user then decides on which parameters could be adjusted to achieve calibration and any boundary conditions associated with the system at the time the data were collected. Field data consists of flows and head (hydraulic grade line elevations) through the system. Boundary conditions refer to water levels in tanks and the operational status of pumps and valves.

In any real system there can be hundreds or thousands of unknowns and only a relatively small number of field observations. Wu et al. (2002b) has observed that when the number of unknowns greatly exceeds the number of useful observations, there is little confidence in the calibration results. There are too many solutions that can match the observed flows and heads. However, it is not likely that pipes with similar characteristics will have very different roughness values or nodes in a given area of the system will need large adjustments to achieve calibration. It is likely for automated calibration to be successful that pipes and nodes being adjusted be placed in "groups". This reduces the size of the problem, makes it possible to find the optimal solution and avoids issues where several identical pipes would end up with very different roughness values because of small inaccuracies in field measurement.

The correctness of the solutions obtained using genetic algorithms is quantified using what is called the "fitness" of the solutions. The fitness is based on the difference between observed and predicted values for the hydraulic grade line (HGL) and flow. There are typically three methods for calculating fitness: least squares, least absolute difference value, and least maximum error. The method used in this paper, unless otherwise stated, is least squares where the fitness is determined as

$$F = \frac{1}{w_H}\sum(H_{mod} - H_{obs})^2 + \frac{1}{w_Q}\sum(Q_{mod} - Q_{obs})^2$$

where F = fitness

H_{mod} = model value for head, L

H_{obs} = observed value for head, L

Q_{mod} = model value for flow, L^3/T

Q_{obs} = observed value for flow, L^3/T
w_H = weighting factor for head, L^2/fitness unit
w_Q = weighting factor for flow, $(L^3/T)^2$/fitness unit

Unless stated otherwise, the head weighting was 0.305 while the flow weighting was 0.631.

One of the issues with genetic algorithm type searches is that there is some uncertainty as to when to stop the solver. Three general criteria are used to stop the run:

1. Fitness within user specified tolerance: if the fitness is excellent, the solver is satisfied and stops.
2. Maximum number of iterations: if the maximum number is exceeded, the best solution(s) found thus far is shown.
3. Maximum number of non-improvement generations: if the solution stops improving after this number of generations, the solver cannot improve fitness and stops.

For initial runs, the user is encouraged to set the tolerances high and the maximum iterations low to make the solver run faster, but for final runs, the user should set the tolerance very low and the maximums very high to get the best accuracy.

2. LABORATORY EXPERIMENTS

Because it is not possible to know the exact flows and pipe properties in a real system, a laboratory model system was constructed in the Wilkes University (Wilkes-Barre, Pennsylvania) laboratory. The dimensions of the piping system are shown in Figure 1. The piping consisted of nominal 1-inch (25 mm) and 1½-inch (38 mm) PVC pipe with actual internal diameters of 1.044 and 1.609 inches (26.5 and 40.8 mm) measured using a micrometer. Pipe sizes were selected to be small while still maintaining turbulent flow conditions during all experiments. Figure 2 shows a photograph of the actual pipe network with the manometer board in the center.

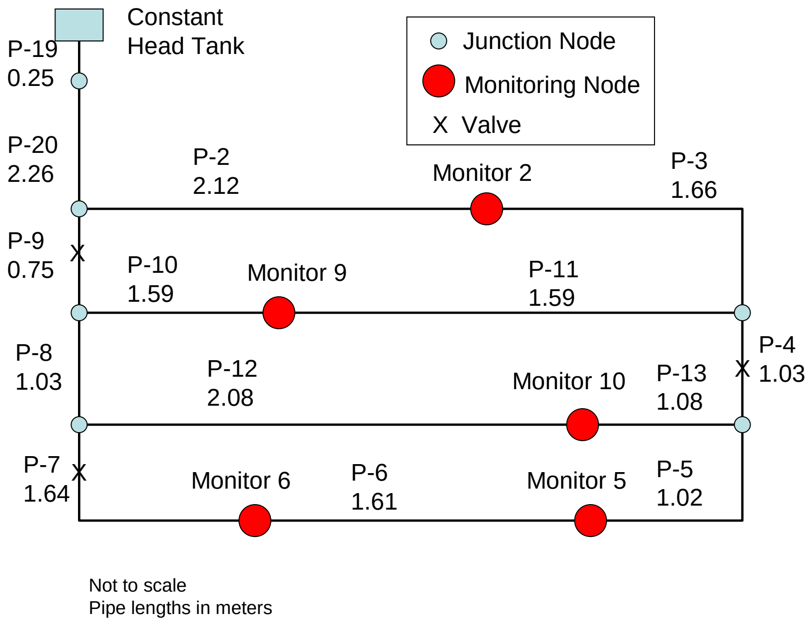


Figure 1. Schematic of original lab pipe network

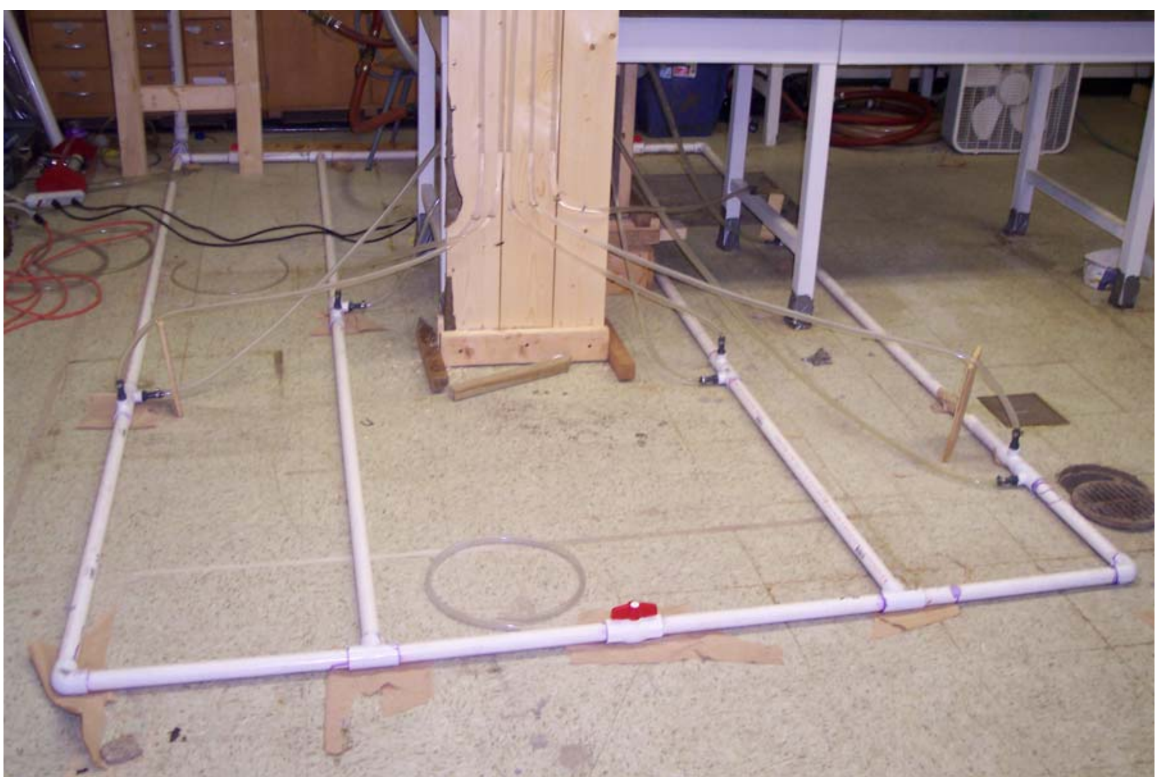


Figure 2. Pipe network used to generate data

The water in the system was supplied by a constant head tank. All water drained back to a sump and was recycled to the constant head tank. Three full port ball valves (in pipes P-4, P-7, and P-9) were initially included in the piping to make it possible to reconfigure the piping network from one run to the next and to test Darwin's ability to find closed valves.

There were five demand/monitoring nodes in the network. Each of these contained two T's as shown in Figure 3. One tap (at the side of the pipe) corresponded to the water user and was a location where flow was measured by recording the time it took to fill a container of known volume. The other T (tap on top of the pipe) was connected to a central manometer board shown in Figure 4 where the hydraulic grade line elevation could be directly measured.

Figure 3. Monitoring and water use node

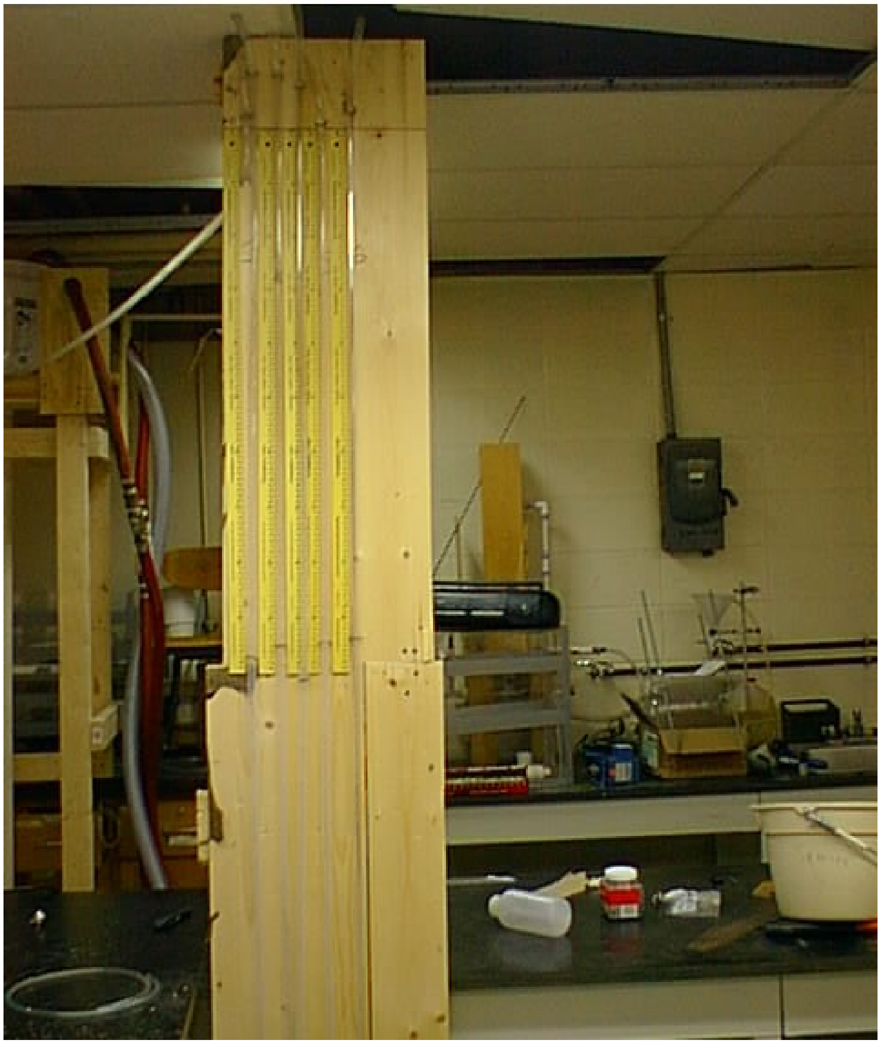


Figure 4. Manometer used to measure heads

Several experimental runs of the lab model were conducted for all combinations of valve settings. In general six sets of measurements were taken for each condition and the average value was used for this analysis. Most runs were made with all of the outlets open although some runs were made with all but one of the outlets closed. All runs corresponded to steady state conditions so there were no unsteady flow or transient effects.

3. ANALYSIS

Once the data were collected and the computer model of the system was constructed, the Darwin Calibrator was used to determine the calibration parameters. With data collected from the physical lab model, it was possible to set up numerous cases for which the Darwin Calibrator was tested. In general, there were 14 pipe roughness values, five demands and three valve settings that could be determined (i.e.

22 decision variables). Depending on the run, these values were determined using information about the three valve settings, five HGL measurements, and five measured demands. In some cases, the valve status (open/closed) and demands were treated as either known or unknown. In many of the runs, all of the pipes were placed in one group because all of the pipes should have the same internal characteristics. In other runs, 14 separate groups were set up to determine if Darwin would give similar results for each pipe. In still other runs, pipes were placed in groups depending on their location within the system.

A large number of cases were set up and solved using the Darwin Calibrator. The cases that provided the most insight into its performance for automated calibration are presented in the sections below. As expected, the Darwin Calibrator did an excellent job in matching the measured HGL values (state variables). The root mean square error (RMSE) between observed and model HGL values is usually on the order of the precision of the readings. However, because of the under constrained nature of the calibration problem, it was more difficult to match the model parameters for roughness and demand (decision variables). This is especially evident in runs with large numbers of groups.

The lab model differs from most distribution systems because the exact internal diameter is known. In real systems, the exact internal diameters are not known and nominal diameters are typically used. Only in small pipes is this difference really significant. Minor losses can usually be ignored in real water distribution systems because they account for only a small portion of the total head loss. However in this pipe network they could be significant and their impact was considered in some runs. Finally, in real systems the velocity head changes are negligible. In this system, the use of the HGL instead of energy grade line may have introduced error in some specific cases. The individual cases are presented in the sub-sections below.

3.1 Determining Roughness

The initial runs showed a range of results for the optimal value for C-factor. Because the pipes did not change between runs, this was less than ideal — one would expect a single correct C-factor. This indicated some nonuniformity in C-factors across the system and required some special work to account for this anomaly which distorted some otherwise good results.

When the pipes were placed in 14 separate groups, the pipes around node J-1 (junction of P-2, P-9, and P-20; see Figure 1) had lower C-factors. When the pipes were placed in a single group, runs with a higher flow had significantly lower C-factors. Plotting total flow from the reservoir vs. C as shown in Figure 5 showed that runs with pipe P-9 with the valve open had lower C-factors.

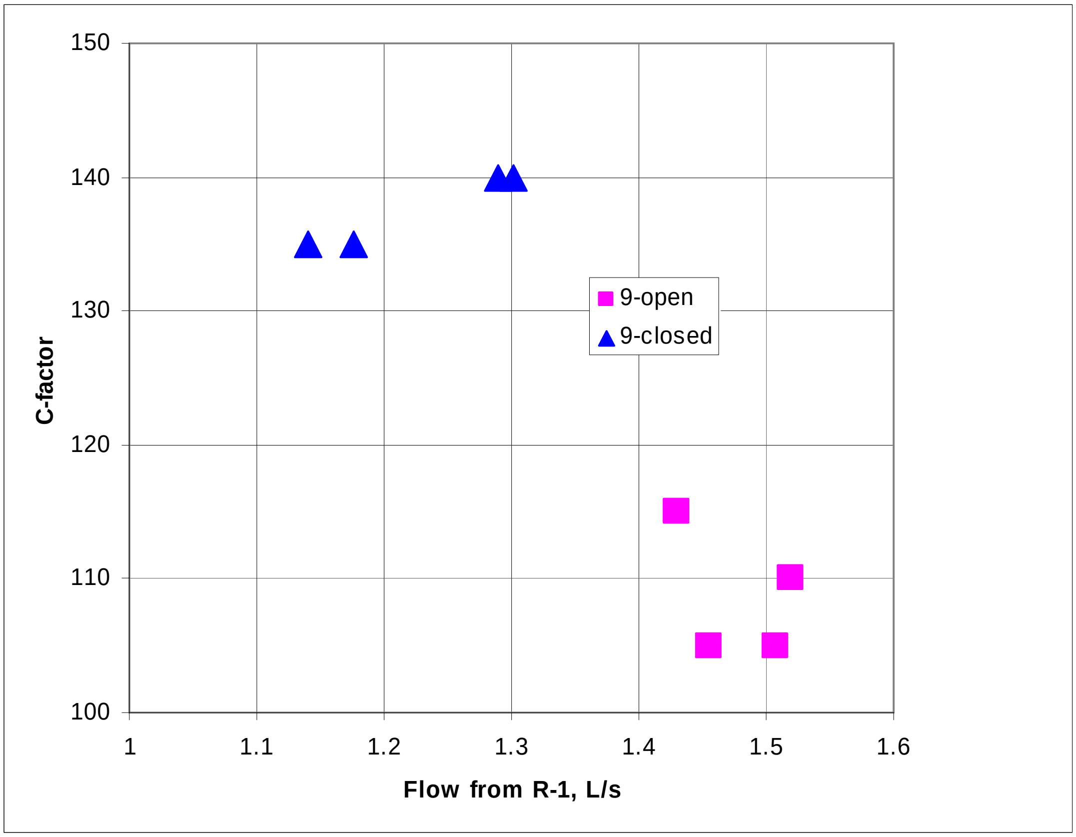


Figure 5. C-factor based on varying status for pipe P-9

This indicated that something odd was occurring in pipe P-9. Since this was a pipe with a valve in it and is from the 90-degree angle of the tee near vertical pipe from the supply tank, the supposition was that the valve was not opening completely or the minor losses due because of the tee are much higher than expected. As a follow up to the initial testing, a fourth valve was place in the system in pipe P-3 and it was closed so that all of the flow from the source passed through pipe P-9. In this case a calculated C-factor of 70 for P-9 and values on the order of 120 to 130 for all other pipes, supported the supposition that a minor loss is significant in P-9. Because the high head loss in this pipe was needed to be accounted for, a minor loss coefficient was assigned to this pipe for all later runs.

To determine the minor loss in P-9, a series of model runs were made with differing assigned minor loss K values for this pipe. A single system-wide C-factor was determined using all data sets. As shown in Figure 6, as K increased, the global C-factor had to increase to offset other losses. The fitness of the solution was at its best with a minor loss K of 4 and a C-factor of 141.

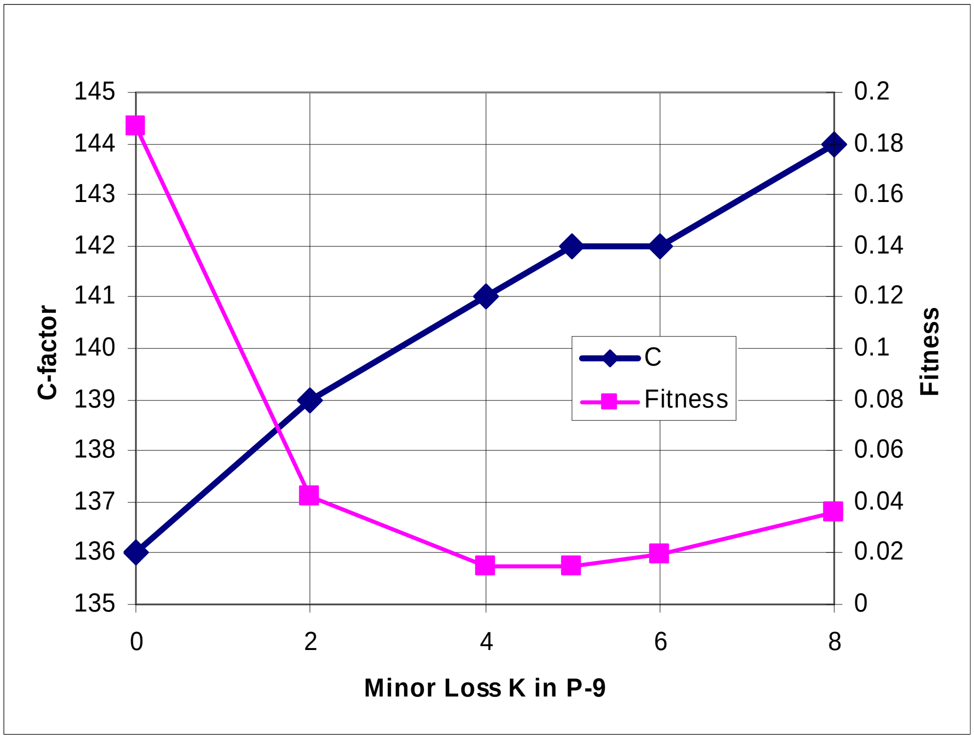


Figure 6. Effect of minor loss K on roughness and fitness

For the remaining runs a minor loss coefficient (K) value of 4 is used for P-9 unless otherwise noted. This value was required to account for the reduction from 38 to 25 mm (1.5 to 1 in.) with two right angle bends at the upstream end of P-9. Minor losses were ignored in the other bends. Based on these initial runs the "correct" value for the C-factor is 141 and other solutions will be judged with regard to how well they approach this value.

3.2 Individual Pipes vs. Spatial Grouping

In running an optimal calibration model it is tempting to set up a separate group for each pipe to allow the maximum flexibility in finding solutions. In this study there would be 14 groups. Ideally, if the correct C-factor is 141, all of the groups would end up with 141 as the predicted C-factor. However, in runs with 14 groups, while the Darwin Calibrator matched the heads exactly, there were many combinations of C-factors that worked. The program does not only save a single solution but can save many of the good solutions it determines ranked by the fitness value. The best solution and several representative solutions are shown in Table 1 below. The "solution number" refers to the rank of the solution based on the least-squared fitness value. Solution 12 has the 12th best fitness of all considered.

Table 1. C factors for several solutions

Solution Number	1	5	8	12
Fitness Value	0.00258	0.00296	0.00341	0.00407
Pipe 2	145	144	144	144
Pipe 3	152	151	152	150
Pipe 4	151	180	150	152
Pipe 5	180	147	180	180
Pipe 6	154	180	114	172
Pipe 7	115	180	175	175
Pipe 8	146	119	140	117
Pipe 9	103	116	100	116
Pipe 10	138	147	135	180
Pipe 11	112	113	110	112
Pipe 12	156	175	152	180
Pipe 13	123	125	164	101
Pipe 19	141	157	180	180
Pipe 20	131	132	130	130

While each of the solutions in the table represent good solutions and the agreement between observed and predicted heads are on the order of the precision of measurement, the table shows that there are many good solutions, some of which have unusual values for the C-factor. However, the length weighted averages of the C-factor is near 141 for all cases.

The reason for the range of C-factors is that there is no way to distinguish between pipe C-factors when there are no measurement of head between monitoring points. For example, pipes P-19, P-20 and P-2 are in series but there is no measurement between the pipes. Therefore, there are an infinite number of combinations of C-factor that will give the correct head at the end of those three pipes. The weighted average C-factor is near 141 but the software cannot distinguish between 141-131-145, 180-130-144 or 141-141-141 for P-19, P-20, and P-2 respectfully. The implication here in terms of modeling is that creating an excessive number of groups does not improve the solution if there is not sufficient data for an optimal calibrator to distinguish between the groups. Intelligent grouping is the key to using automated calibration models.

One approach to create spatial groups has been to group pipes according to the monitoring point (or flow test). In this case pipes leading up to a monitoring point would be in one group, pipes between this monitor and another monitor would be the second group, etc. The calibration process would be to determine the C-factor for the first group, then that value would be fixed and the C-factor for the second group would then be determined. For the physical lab model, groups are shown in Figure 7 and are labeled Grp2, Grp9, Grp10 and Grp56 based on the monitoring point locations. Monitoring points 5 and 6 were combined because there is very little head loss between the two points.

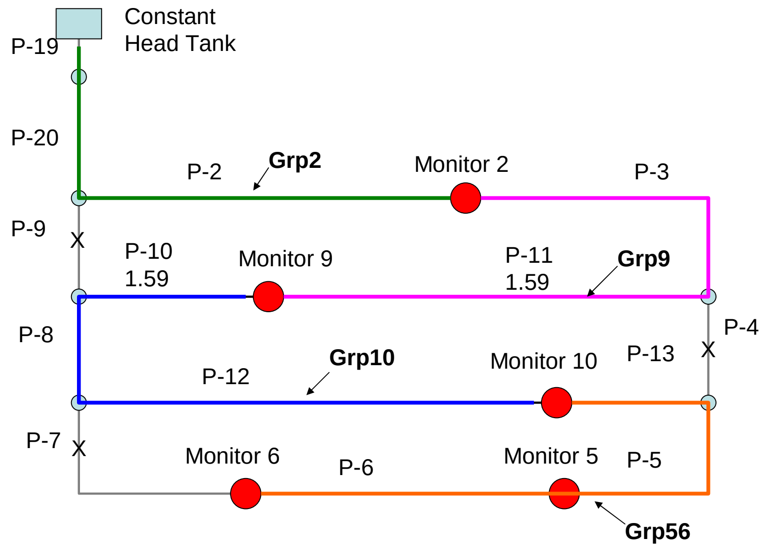


Figure 7. Spatial grouping of pipes

In this example, all data collected was when all of the flow goes to the monitoring point. For example, for monitoring point 2, only data for the case where P-9 is closed is used. When each value for C-factor is determined incrementally, the values are shown in Table 2 below.

Table 2. Solution with four groups

Group	C-factor
Grp2	143
Grp9	140
Grp10	127
Grp56	140

This showed that it is possible to move through the system and solve for roughness one area at a time. With this approach, the run times are faster because the number of groups being adjusted is small. This is logical where pipes differ spatially. For instance, if older pipes are located in the south part of town, there is no sense in adjusting their roughness using flow tests from the north part of town.

3.4 Effect of Using Discrete Increment Values

In the genetic algorithm solver, the unknowns are not continuous variables but are discrete variables. C-factors such as 140, 141, 142, etc. are determined and values such as 141.3486 are not determined unless the increment was set to 0.0001, which would result in a prohibitively large set of possible solutions. To determine the effect of the increment size on the solutions the increment was set to 1, 2, 5 and 10. Those solutions and the associated fitness values are shown in the Table 3 for the case with four spatial groups. For the case with an increment of 10, the range started at 60 (60, 70, etc.) for one run and 65 (65, 75, etc.) for the next.

Table 3. C-factors determined using different values for each increment

Increment	1	2	5	10(60)	10(65)
Grp2	146	146	145	150	145
Grp9	125	124	125	120	125
Grp10	143	144	145	150	145
Grp56	140	144	140	130	135
Fitness Value	0.000023	0.000057	0.000076	0.001382	0.000088

As one would expect, as the increment size decreased, the fitness value (agreement between model and observed values) improved. Another interesting point is that the runs with an increment of 10 yielded different solutions depending on whether the values such as 60, 70 80, etc were used instead of values like 65, 75, 85, etc. This appears to be just a quirk of this problem, but if large increments are used, the user may want to adjust the ranges to test the sensitivity of the solution to the ranges used. The lesson learned in these runs were that intelligently reducing the range of allowable roughness values will enable the solver to have a better chance to find the right value. However, if the increment size is decreased too much, the number of possible solutions greatly increases without providing a more correct value. Generally calculating a C-factor close one is unrealistic in the field so it is impractical to use an increment size less than one.

3.5 Finding Closed Valves

One of the key tests of the calibrator was the test of its ability to find closed valves in the pipe network. To test this, 8 different combinations of valves were tested in the lab as listed in Table 4 below. Calibration runs were set up to determine a single global C-factor with the three valves which each could have the status of open or closed.

Table 4. Summary of Valve Status Runs

4	7	9	Global C-factor
O	O	O	140
C	C	C	139
C	O	C	139
C	O	O	134
O	C	C	145
O	C	O	138
C	C	O	147
O	O	C	144

O = open; C = closed

Figure 8 summarizes the results of the runs to locate closed valves. They show that in all cases, the HGL predicted by the model (solid line) matched the HGL observed in the pipe network (points with same color as line). The results also show that while similar C-factors were determined for each case, there is some variation in the correct solution due to the accuracy of the measurements. The HGL in the physical lab could only be measured to a precision of approximately one centimeter due to the slight unsteady water level in the manometer. The differences between observed and predicted HGLs were at most one to two centimeters which corresponded to a variation of up to 6 C-factor units for the "correct" value of 141. The "position" (x-axis) in the figure refers to the distance from the constant head tank when all valves are closed.

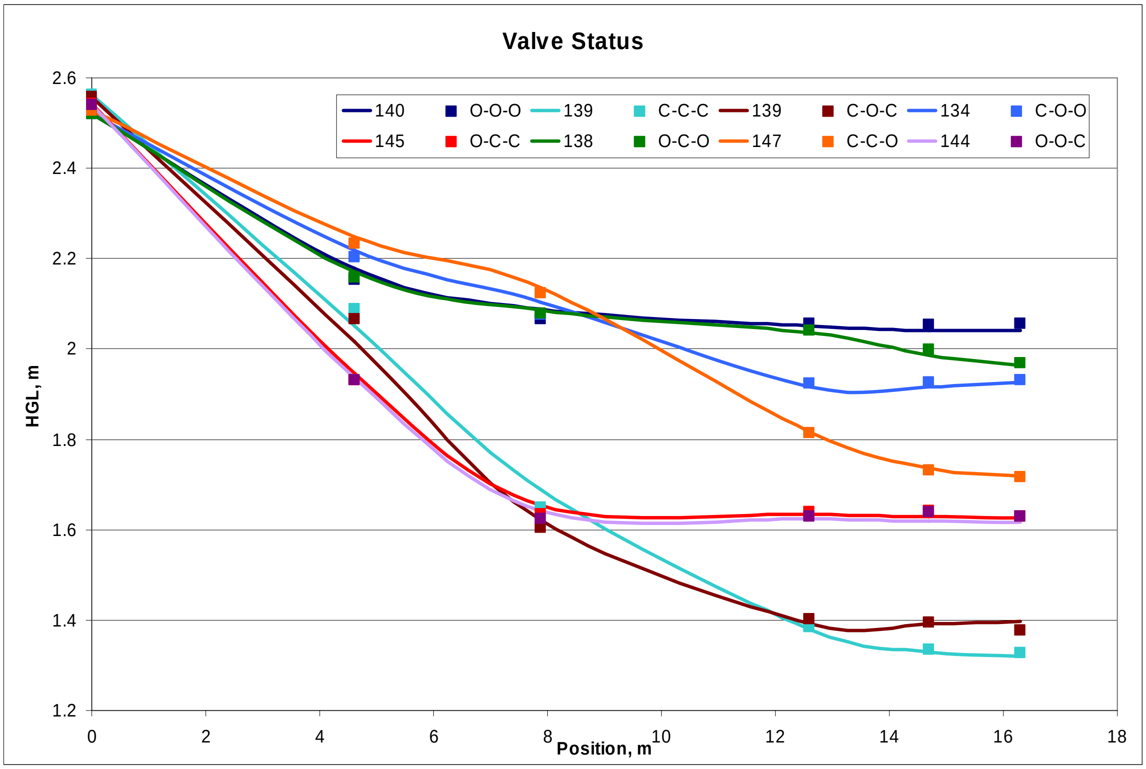


Figure 8. Correlation between observed and model HGL valves for different valve positions

Out of the eight runs, the Darwin Calibrator correctly matched the valve status in seven. The single run where the initial solution had one valve wrong involved the case were valve 4 was open and 7 and 9 were closed (O-C-C). In that run, status for valves 4 and 9 were correctly predicted but incorrectly stated that valve 7 was open. The problem was caused by the fact that with valve 7 open, virtually no flow in the pipe P-7 was observed, such that the solutions with 7 open and closed were virtually identical. The lesson here is that in order to determine the status of a valve, the flow pattern should be such that a significant amount of flow would pass through the valve if it were open.

3.6 Determining Demands

The ability of the Darwin Calibrator to determine demand flow rates were tested as shown in Table 5. With only the HGL information provided, numerous solutions were found where roughness and demands compensated for each other, i.e. a high value for the C-factor can offset a high value for the demand. Therefore, for most subsequent runs, the totalized flow was also provided. This is a reasonable piece of information since, for most systems, the total water plant or well production flow is known. Without some input as to total inflow from the source, the solver had difficulty determining the magnitude of flow because roughness errors could offset demand errors.

The table consists of pairs of columns for each test. The first with the values measured in the lab and the second the results of the model. In all there were four tests. The first used all 8 possible combinations for the valve status and all outlets were open. Only the demands for the data set where all valves are open are shown. The second data set was for the case where all the operable valves were open and all the outlets were open. The third case consisted of the data set when all three operable valves were shut. The final case consisted off all three valves shut with only Monitor 6 (the last outlet) open.

Table 5. Results of runs to evaluate the effectiveness of determining demands. (Flow rates have units of L/s.)

Open Valves	8 Combinations		4-7-9		None		None	
Open Outlets	All		All		All		Mon6	
Outlet Node	Lab	Model	Lab	Model	Lab	Model	Lab	Model
Monitor 2	0.30	0.28	0.30	0.29	0.30	0.19	0	0
Monitor 5	0.28	0.33	0.28	0.32	0.18	0.18	0	0.02
Monitor 6	0.30	0.20	0.30	0.30	0.18	0.18	0.33	0.31
Monitor 9	0.30	0.30	0.30	0.33	0.21	0.33	0	0
Monitor 10	0.33	0.40	0.33	0.27	0.27	0.26	0	0
Total Flow	1.51	1.51	1.51	1.51	1.14	1.14	0.33	0.33
C-factor		143		141		144		139

In general, the C-factor and total demand were predicted accurately. It was somewhat less accurate at assigning demands to the individual nodes. This can be expected because while the predicted HGL is sensitive to the total flow demand, it is somewhat less sensitive to the exact placement of that demand. Similarly, in a real water system, it is impossible to know exactly which customer(s) are using water when any data are collected.

3.7 Other Head Loss Equations

Thus far, the Hazen-Williams equation has been used for head loss and the Darwin Calibrator has solved for the C-factor. To examine the use of other head loss equations, the pipes were grouped into four groups as described earlier and the cases with all eight valve combinations and the single case with all valves open were run. Either the Manning or Darcy-Weisbach equations were used for head loss where Darwin solved for Manning's n or the equivalent sand grain roughness height using the Swamee-Jain formula. The results are shown in Table 6 below.

Table 6. Results for Manning's n and Darcy-Weisbach roughness height in mm.

	Manning's n		Darcy-Weisbach	
Group	8 Combinations	All valves closed	8 Combinations	All valves closed
Grp2	0.008	0.007	0.006	0.011
Grp9	0.006	0.009	0.061	0.076
Grp10	0.008	0.007	0.021	0.016
Grp56	0.006	0.010	0.001	0.006

The values for Manning's n were very low but are consistent with a small pipe with a C-factor of 140. The solutions for equivalent sand grain roughness height are reasonable for smooth small pipe but show an unexpectedly large range. However, in the range of Reynolds numbers and relative roughness for this network, it takes very large changes in roughness to make even a tiny change in friction factor. (On the Moody diagram, many lines converge in this region.) This explains the large range for roughness values. In terms of friction factor (f), all the f-values were around 0.03.

3.8 Nominal vs. Actual Diameters

In the runs thus far, the actual internal diameters were used for pipes. For many cases, the user will only know the nominal diameters. Using the same case (all valves closed—four groups—least squares fitness), the Darwin Calibrator was used to solve for the C-factor. One would expect the C-factors to be related to the ratio of diameters using the Hazen-Williams equations as

$$C_{(nom)} = C_{(actual)} \left(\frac{D_{(actual)}}{D_{(nom)}} \right)^{2.63}$$

The results shown Table 7 are consistent with what one would expect. The ratio between actual and nominal C-factors should be on the order of 1.12 based on the measured internal diameter of the pipe used in the physical model.

Table 7. Comparing solutions based on minimal and actual diameters. Ratio is determined as the Nominal value divided by the Actual value.

	Actual	Nominal	Ratio
Grp2	146	160	1.10
Grp9	125	148	1.18
Grp10	143	154	1.08
Grp56	140	164	1.17

The calculated ratio has a range of 1.08 and 1.18 which is consistent with the expected ratio of 1.12. In larger pipes, as found in real water distribution systems, the differences between nominal and actual diameters are usually much smaller than those encountered in the small pipes used in this study and the resulting ratio would be closer to 1. These results indicate that the use of the nominal pipe diameter in a real water distribution system does not lead to substantial errors.

4.0 SUMMARY

This paper showed the ability of genetic algorithms as embodied in the Darwin Calibrator to determine values for the pipe roughness, valve status, and demand for a pipe network where the flows and valve status would be known much more precisely than is likely possible for real systems. Overall, the solver worked well although several potential pitfalls in its use were identified. The most significant is trying to use more pipe groups than the data can support. In this case many possible combinations of C-factors were possible. Also when trying to determine the valve status in a case where very little flow would pass through the valve, in one case, the calibrator inaccurately predicted whether the valve was opened or closed. The Darwin Calibrator worked well regardless of whether the Hazen-Williams, Manning, or Darcy-Weisbach formulas were used. When determining demands, it is important to specify some flow, such as the total inflow to the system at sources, in order to keep the search focused in the right range of flows.

References

Araujo, J., and Lansey, K. E. (1991). ‘‘Uncertainty quantification in water distribution parameter estimation.’’ *Proc., Nat. Hydraulic Engineering Conference*, ASCE, New York, 990.

Bhave, P.R. (1988) “Calibrating Water Distribution Network Models,” *Journal of Environmental Engineering*, 114(1),120.

Datta, R.S.N. and Sridharan, (1994) “Parameter Estimation in Water Distribution Systems by Least Squares,” *Journal of Water Resources Planning and Management*, 120(4), 405.

Greco, M. and Del Guidice, G., (1999) “New Approaches to Water Distribution Network Calibration,” *Journal of Hydraulic Engineering,* 125(8), 849.

Bentley Systems, Incorported (2006) *WaterGEMS Software Manual*, Watertown, CT.

Herrin, G. (1997) “Calibrating the Model,” *Practical Guide for Hydraulics and Hydrology*, Haestad Methods, Waterbury, Conn.

Lansey, K. and Basnet, C. (1991) “Parameter Estimation for Water Distribution Networks,” *Journal of Water Resources Planning and Management*, 117(1), 126.

Lansey, K., El-Shorbagy, W., Ahmed, I., Araujo, J. and Haan, C.T., (2001) “Calibration Assessment and Data Collection for Water Distribution Networks,” *Journal of Hydraulic Engineering,* 127(4), 270.

Lingireddy, S. and Ormsbee, L. (1998) “Neural Networks in Optimal Calibration of Water Distribution Systems,” *Artificial Neural Networks for Civil Engineers: Advanced Features and Applications,* ASCE.

Meredith, D., (1983) “Use of Optimization in Calibrating Water Distribution Models,” *Proceedings of ASCE Spring Convention,* Philadelphia, Pa.

Ormsbee, L. (1989) Implicit Pipe Network Calibration,” *Journal of Water Resources Planning and Management,* 115(2), 243.

Ormsbee, L. and Chase, D.V. (1988) “Hydraulic Network Calibration using Nonlinear Programming,” *Proceedings of International Symposium on Water Distribution Modeling*, Lexington, KY.

Ormsbee, L. and Wood, D.J. (1986) “Explicit Pipe Network Calibration,” *Journal of Water Resources Planning and Management,* 112(2), 166.

Reddy, P.V.N., Sridharan, K. and Rao, P.V. (1996) “”WLS Method for Parameter Estimation in Water Distribution Networks,” *Journal of Water Resources Planning and Management,* 122(3), 751.

Savic, D. A. and Walters, G.A. (1995) “Genetic Algorithm Techniques for Calibrating Network Models,” Report No. 95/12, Center for Systems and Control Engineering, University of Exeter, UK.

Shamir, U. and Howard, C.D.D. (1968) “Water Distribution System Analysis,” *Journal of Hydraulics Division, ASCE,* 94(1), 129.

Walski, T.M. (1983) “Technique for Calibrating Network Models,” *Journal of Water Resources Planning and Management,* 109(4), 360.

Walski, T.M. (1990) “Sherlock Holmes Meets Hardy Cross or Model Calibration I Austin, Texas,” *Journal AWWA,* 82(3), 34.

Walski, T.M. (2000) “Model Calibration Data: The Good, The Bad and The Useless,” *J AWWA,* 92(1), 94.

Walski, T.M. (2001) “Understanding the adjustments for water distribution system model calibration,” *Journal of Indian Water Works Association*, XXXIII(2), 151.

Walski, T.M., Chase, D.V., Savic, D.,A., Grayman, W., Beckwith, S. and Koelle, E., (2003) *Advanced Water Distribution Modeling and Management*," Haestad Methods, Inc. Waterbury, CT.

Wang Q. J. (1991), The Genetic algorithm and its Application to Calibrating Rainfall-runoff Models, pp2467-2482, Water Resources Research, Vol 27, No. 9

Wu, Z. Y. and Simpson A. R. (2001) "Competent Genetic Algorithm Optimization of Water Distribution Systems." *J. of Computing in Civil Engineering*, ASCE, Vol 15, No. 2, pp 89-101.

Wu, Z. Y. (1994) "Automatic Model Calibration by Simulating Evolution." HH149, 150 pages, International Institute for Infrastructure, Hydraulic and Environmental Engineering, Delft, the Netherlands.

Wu, Z, Y., Walski, T.M., Mankowski, R., Herrin, G., Gurrieri,, R. and Tryby, M. (2002a), "Calibrating Water Distribution Models via Genetic Algorithms," Proceedings AWWA Information Management Technology Conference, Kansas City, Mo.

Wu, Z, Y., Walski, T.M., Mankowski, R., Herrin, G., Gurrieri,, R., Tryby, M. and Hartell, W. (2002b), "Impact of Measurement Errors on Optimal Calibration of Water Distribution Models," *Proceedings of the International Conference on Technology Automation and Control of Wastewater and Drinking Water Systems*, Gdansk, Poland.

Acknowledgement

Review comments on an early draft of the paper were provided by Wayne Hartell and Zheng Wu.

EXPLICIT PIPE NETWORK CALIBRATION

By Lindell E. Ormsbee,[1] A. M. ASCE and Don J. Wood,[2] M. ASCE

ABSTRACT: In order to improve the reliability of hydraulic network models as well as eliminate the need for trial-and-error calibration methods, an explicit calibration algorithm is proposed. The calibration algorithm is formulated in terms of headloss coefficients and is developed from a reformulation of the basic network equations. The basic network equations are solved explicitly for headloss adjustments to exactly meet one or more measured conditions of pressure or flow for given network loading and operating conditions. The adjustments determined in this manner are used to revise pipe roughnesses or defined concentrated head (minor) losses to meet the measured conditions. In order to demonstrate the feasibility of the approach, the developed algorithm is applied to an example network.

INTRODUCTION

One of the most pressing concerns in civil engineering is the integrity and reliability of the nation's infrastructure. Of principal concern is the condition of water distribution networks. During the last few years, it has become clear that many of our existing water distribution systems are going to have to be upgraded and modified if utilities are to continue to provide reliable systems for distributing water to people in urban and rural areas. Good engineering decisions based on sound analysis procedures will be required if the alterations and improvements to these systems are to be effective and economical. In light of the great need and apparent commitment to expend vast amounts of money to upgrade and improve these systems, it is imperative that sound analysis procedures are available.

During the last few decades several algorithms have been proposed for solving the basic hydraulic network equations. A general review and comparison of the various techniques has been provided by Wood and Rayes (1). Despite the availability of such simulation algorithms, the applicability of the obtained result is largely dependent on the accuracy of the input data. The two major sources of error in simulation analysis are incorrect estimates of water use and incorrect estimates of pipe carrying capacity (2). There is some disagreement in the literature as to which factor is most important. The AWWA Research Committee on Water Distribution Systems (3) states that ". . . the major source of error in simulation of contemporary performance will be in the assumed loading distributions and their variations," while Eggener and Polkowski (4) state that "the weakest piece of input information is not assumed loading condition, but the pipe friction factor." It would seem more likely that the importance of each factor will vary for different net-

[1]Asst. Prof., Dept. of Civ. Engrg., Univ. of Kentucky, Lexington, KY 40506-0046.

[2]Prof., Dept. of Civ. Engrg., Univ. of Kentucky, Lexington, KY 40506-0046.

Note.—Discussion open until September 1, 1986. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on May 7, 1985. This paper is part of the *Journal of Water Resources Planning and Management*, Vol. 112, No. 2, April, 1986. ©ASCE, ISSN 0733-9496/86/0002-0166/$01.00. Paper No. 20505.

number of pipe sections that can contain pumps and fittings such as bends and valves. The end points of the pipe sections are nodes that are identified as either junction nodes or fixed grade nodes. A junction node is a point where two or more pipe sections join and is also a point where flow can enter or leave the system. A fixed grade node is a point where a constant grade is maintained such as a connection to a storage tank or reservoir or to a constant pressure region. In addition, primary loops can be identified in a pipe network. These include all closed pipe circuits within the network that have no additional closed pipe circuits within them. Finally, additional energy paths can be identified between the various fixed grade nodes in the network. When junction nodes, fixed grade nodes, primary loops, and energy paths are identified the following relationship holds:

$$p = j + k \quad \text{(1)}$$

in which p = number of pipes; j = number of junction nodes; and $k = l + f - 1$, with l = number of loops; $f - 1$ = number of energy paths between fixed grade nodes; and f = number of fixed grade nodes. It turns out that this equation is directly related to the basic hydraulic equations that describe steady state flow in the pipe network.

Basic Equations.—The basic equations governing the flow of a fluid in a distribution network are the conservation of mass equation and the conservation of energy equation. For each junction node j the conservation of mass equation $F_m(Q)$ requires that the sum of the flows into or out of a junction node minus any external demand M_j must equal zero. This may be expressed as

$$F_m(Q) = \sum_{n \in \{j\}}^{N_j} Q_n - M_j = 0 \quad \text{(2)}$$

in which N_j = the number of pipes connected to junction j; and $\{j\}$ = the set of pipes connected to junction j. In order to satisfy the conservation of energy relationship, $F_e(Q)$, the sum of the line losses (HL) and the minor losses (HM) over any path on loop, minus any energy added to the liquid by a pump (EP), minus the difference in grade between two fixed grade nodes (ΔE) is equal to zero. For any loop or path k this may be expressed as

$$F_e(Q) = \sum_{n \in \{k\}}^{N_k} (\mathrm{HL}_n + \mathrm{HM}_n - \mathrm{EP}_n) - \Delta E_k = 0 \quad \text{(3)}$$

in which N_k = the number of pipes in loop or path k; and $\{k\}$ = the set of pipes in loop or path k. The line loss expressed in terms of the flowrate is given by

$$\mathrm{HL} = K_p Q^a \quad \text{(4)}$$

in which K_p = a pipeline constant that is a function of line length, diameter, and roughness; and a = an exponent. The values of K_p and a depend on the energy loss expression used for the analysis. Commonly used expressions for this include the Darcy-Weisbach, Hazen-Williams, and Manning equations.

$$\left[\frac{C_m}{C_e}\right]\hat{Q} = \left[\frac{\hat{M}}{\hat{E}}\right] \qquad (12)$$

For a given set of initial flowrates, $[C_e]$, and $\hat{E}$ can be determined. Using this information along with the junction demand vector, $\hat{M}$, and the node incidence matrix, $[C_m]$, an improved set of flowrates, $\hat{Q}$, can be determined by inverting the coefficient matrix of Eq. 12 and multiplying it by the right side solution vector

$$\hat{Q} = \left[\frac{C_m}{C_e}\right]^{-1}\left[\frac{\hat{M}}{\hat{E}}\right] \qquad (13)$$

This new set of flowrates can then be used to obtain new estimates of $[C_e]$, and $\hat{E}$ from which a further improved set of flowrates may be obtained. This process can be continued until the flowrate values converge and usually only 3–4 trials are required.

Calibration Algorithm

Introduction.—The previous solution technique involves the iterative solution of p simultaneous equations in terms of p unknown flowrates. Addition of another continuity or energy equation for the same network would allow the addition of another decision variable (other than flowrate) to the overall problem. One possible decision variable would be a global headloss adjustment, which will adjust the headloss in each pipe by this unknown factor. This factor could be determined in order to satisfy the conditions imposed by the extra continuity or energy equation. For example, an extra continuity equation could be added to define the flow in a pipe leading from a pumping facility. Such a condition could be used to specify both the flowrate and discharge head of a supply pump. The modified system of $p + 1$ equations could then be solved for both the flowrates and the global headloss adjustment factor that would produce the observed or specified conditions. Alternatively, an extra energy equation could be added to the system of equations. The additional energy equation could be used to specify the hydraulic grade (and thus the pressure) at a particular junction node. As before, the modified system of $p + 1$ equations could then be solved for both the flowrates and global headloss adjustment factor that would produce this condition.

Thus far, the analysis has been limited to the addition of a single operating condition as described by the addition of a single continuity or energy equation. However, many conditions and thus equations may be added if so desired. This would result in one extra decision variable for each additional equation. By the introduction of additional decision variables different headloss adjustment factors could be determined for different groups or sets of pipes. In addition some pipes could be excluded from the adjustment sets and thus left unchanged.

Instead of determining a global adjustment factor for the headloss coefficients in each pipe or group of pipes, the required headloss coefficients themselves could be determined. This would require an additional equation for each headloss coefficient to be determined. In this case, the decision variable is defined as a headloss calibration coefficient nonlinear energy equations must be linearized. Application of Newton's method to the modified energy equation, Eq. 16, yields the following:

$$G(Q, F_r, K_r) = F(Q, F_r, K_r) + \frac{\partial F(Q, F_r, K_r)(\hat{\mathbf{Q}} - Q)}{\partial Q}$$

$$+ \frac{\partial F(Q, F_r, K_r)(\hat{\mathbf{F}}_r - F_r)}{\partial F_r} + \frac{\partial F(Q, F_r, K_r)(\hat{\mathbf{K}}_r - K_r)}{\partial K_r} \qquad (17)$$

in which $$\frac{\partial F(Q, F_r, K_r)}{\partial Q} = F_r K_p Q^{a-1} + 2K_r Q + 2K_m Q - \dot{P}(Q) \qquad (18)$$

$$\frac{\partial F(Q, F_r, K_r)}{\partial F_r} = K_p Q^a \qquad (19)$$

$$\frac{\partial F(Q, F_r, K_r)}{\partial K_r} = Q^2 \qquad (20)$$

Rearranging Eq. 17 we obtain

$$\frac{\partial F(Q, F_r, K_r)\hat{\mathbf{Q}}}{\partial Q} + \frac{\partial F(Q, F_r, K_r)\hat{\mathbf{F}}_r}{\partial F_r} + \frac{\partial F(Q, F_r, K_r)\hat{\mathbf{K}}_r}{\partial K_r}$$

$$= \frac{\partial F(Q, F_r, K_r)Q}{\partial Q} + \frac{\partial F(Q, F_r, K_r)F_r}{\partial F_r} + \frac{\partial F(Q, F_r, K_r)K_r}{\partial K_r} - F(Q, F_r, K_r) \qquad (21)$$

In matrix notation Eq. 21 can be expressed as

$$[C_e]\begin{bmatrix} \hat{Q} \\ \hat{F}_r \\ \hat{K}_r \end{bmatrix} = \hat{\mathbf{E}} \qquad (22)$$

in which $[C_e]$ = the matrix of headloss coefficients made up of the three partial derivative terms on the left side of Eq. 21; $\hat{\mathbf{Q}}$ = a vector of unknown flowrates; $\hat{\mathbf{F}}_r$ = a vector of unknown headloss adjustment factors; $\hat{\mathbf{K}}_r$ = a vector of unknown headloss calibration coefficients; and $\hat{\mathbf{E}}$ = the vector of energy coefficients made up of the four terms on the right-hand side of Eq. 21.

Combining the modified energy equation, Eq. 22, with the continuity equation, Eq. 8, and adding the extra continuity and energy equations required for the determinization of the headloss adjustment factors and the headloss calibration coefficients results in the following matrix structure:

$$\begin{bmatrix} C_m \\ C_{ma} \\ C_e \\ C_{ea} \end{bmatrix}\begin{bmatrix} \hat{Q} \\ \hat{F}_r \\ \hat{K}_r \end{bmatrix} = \begin{bmatrix} \hat{M} \\ \hat{M}_a \\ \hat{E} \\ \hat{E}_a \end{bmatrix} \qquad (23)$$

in which $[C_{ma}]$ = the node incidence submatrix for the additional continuity equations; $\hat{\mathbf{M}}_a$ = the vector of junction demands for the additional continuity equations; $[C_{ea}]$ = the submatrix of headloss coefficients for

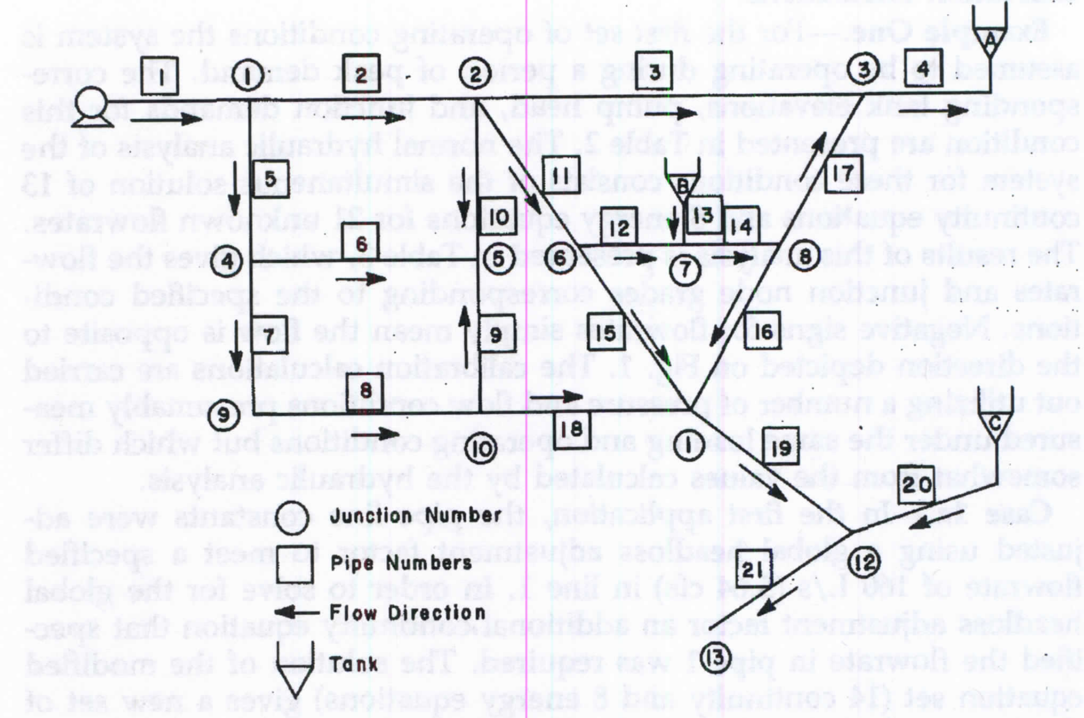


FIG. 1.—Example Network Schematic

TABLE 1.—Pipe Distribution System Characteristics

Pipe number (1)	Node #1 (2)	Node #2 (3)	Length (m) (4)	Diameter (mm) (5)	Hazen-Williams roughness coefficient (6)	Minor loss coefficient (7)
1	0	1	300.0	300.0	100.0	0.0
2	1	2	250.0	250.0	100.0	0.0
3	2	3	450.0	250.0	100.0	0.0
4	3	0	300.0	200.0	120.0	0.0
5	1	4	150.0	250.0	100.0	0.0
6	4	5	250.0	200.0	100.0	0.0
7	4	9	170.0	250.0	100.0	0.0
8	9	10	250.0	250.0	100.0	0.0
9	10	5	170.0	200.0	100.0	0.0
10	2	5	150.0	200.0	100.0	0.0
11	2	6	160.0	200.0	110.0	0.0
12	6	7	140.0	200.0	120.0	0.0
13	0	7	80.0	200.0	120.0	0.0
14	7	8	140.0	200.0	120.0	0.0
15	6	11	300.0	200.0	110.0	0.0
16	8	11	300.0	250.0	110.0	0.0
17	8	3	200.0	250.0	110.0	0.0
18	10	11	200.0	250.0	100.0	0.0
19	11	12	300.0	150.0	140.0	0.0
20	0	12	200.0	150.0	140.0	0.0
21	12	13	175.0	150.0	140.0	0.0

TABLE 2.—Operating Conditions for Example 1

Junction number (1)	Demand (L/s) (2)
2	40
3	40
6	80
8	40
9	60
11	100
13	20

Note: Elevation Tank A = 150 m; Elevation Tank B = 152 m; Elevation Tank C = 148 m; Pump discharge head = 156 m.

TABLE 3.—Results for Example 1

Pipe number (1)	Original Flow (2)	Original HW-C (3)	Case 1a Flow (4)	Case 1a HW-C (5)	Case 1b Flow (6)	Case 1b HW-C (7)	Case 1c Flow (8)	Case 1c HW-C (9)	Case 1d Flow (10)	Case 1d HW-C (11)	Case 1e Flow (12)	Case 1e HW-C (13)
1	166.7	100.0	160.0	84.5	164.7	95.6	164.4	95.4	159.7	92.8	159.1	93.2
2	78.7	100.0	75.2	84.5	77.7	95.6	77.5	95.4	75.0	92.8	73.9	93.2
3	5.2	100.0	1.4	84.5	4.1	95.6	4.1	95.4	1.7	92.8	−0.0	93.2
4	−59.7	120.0	−63.0	101.3	−60.7	114.7	−60.6	114.5	−62.1	118.3	−61.4	119.4
5	88.0	100.0	84.8	84.5	87.0	95.6	86.8	95.4	84.6	92.8	85.2	93.2
6	21.5	100.0	20.3	84.5	21.2	95.6	21.1	95.4	20.2	92.8	19.3	93.2
7	66.5	100.0	64.5	84.5	65.9	95.6	65.7	95.4	64.5	92.8	65.9	93.2
8	6.5	100.0	4.5	84.5	5.9	95.6	5.7	95.4	4.5	92.8	5.9	93.2
9	−24.9	100.0	−24.7	84.5	−24.8	95.6	−24.7	95.4	−24.8	92.8	−27.0	93.2
10	3.4	100.0	4.4	84.5	3.7	95.6	3.6	95.4	4.6	92.8	7.7	93.2
11	30.1	110.0	29.4	92.9	29.9	105.1	29.8	104.9	28.7	102.1	26.2	83.5
12	−63.8	120.0	−64.2	101.3	−63.9	114.7	−63.8	114.5	−65.6	118.3	−67.0	119.4
13	119.5	120.0	120.3	101.3	119.7	114.7	119.5	114.5	122.8	118.3	123.2	119.4
14	55.8	120.0	56.1	101.3	55.8	114.7	55.7	114.5	57.1	118.3	56.2	119.4
15	13.9	110.0	13.6	92.9	13.8	105.1	13.6	104.9	14.4	102.1	13.2	83.5
16	40.7	110.0	40.5	92.9	40.7	105.1	40.4	104.9	40.9	102.1	37.6	83.5
17	−25.0	110.0	−24.4	92.9	−24.8	105.1	−24.7	104.9	−23.8	102.1	−21.4	83.5
18	31.3	100.0	29.2	84.5	30.7	95.6	30.4	95.4	29.2	92.8	32.9	93.2
19	−14.0	140.0	−16.7	118.2	−14.8	133.8	−15.5	137.5	−15.5	137.3	−16.2	139.5
20	34.0	140.0	36.7	118.2	34.8	133.8	35.5	137.5	35.5	137.3	36.2	139.5
21	20.0	140.0	20.0	118.2	20.0	133.8	20.0	137.5	20.0	137.3	20.0	139.5
Node number	**Grade**		**Grade**		**Grade**		**Grade**		**Grade**		**Grade**	
1	147.92		145.76		147.41		147.40		147.43		147.55	
2	143.84		140.63		143.08		143.08		143.15		143.41	
3	143.79		140.62		143.05		143.04		143.15		143.41	
4	144.91		141.92		144.20		144.20		144.22		144.31	
5	143.82		140.58		143.05		143.05		143.11		143.29	
6	142.74		139.20		141.91		141.90		142.00		142.00	
7	146.02		143.72		145.47		145.47		145.54		145.61	
8	143.46		140.20		142.69		142.69		142.80		143.00	
9	142.89		139.30		142.04		142.03		142.02		142.04	
10	142.85		139.27		142.00		142.00		142.00		142.00	
11	142.25		138.56		141.38		141.39		141.40		141.26	
12	143.55		141.00		142.94		143.01		143.01		142.97	
13	142.09		139.01		141.36		141.50		141.50		141.50	

TABLE 4.—Results for Example 2

Pipe number (1)	Original		Case 2a		Case 2b		Case 2c		Case 2d	
	Flow (2)	HW-C (3)	Flow (4)	HW-C (5)	Flow (6)	HW-C (7)	Flow (8)	HW-C (9)	Flow (10)	HW-C (11)
1	110.4	100.0	100.0	90.6	100.0	98.7	100.0	100.0	100.0	92.5
2	58.0	100.0	52.5	90.6	50.5	98.7	49.8	81.9	52.5	92.5
3	35.3	100.0	32.0	90.6	36.4	98.7	30.4	81.9	31.8	92.5
4	34.6	120.0	31.3	108.7	30.0	87.0	30.0	98.2	30.0	111.0
5	52.4	100.0	47.5	90.6	49.5	98.7	50.2	89.1	47.5	92.5
6	20.8	100.0	18.8	90.6	17.7	98.7	21.3	100.0	18.8	92.5
7	31.6	100.0	28.7	90.6	31.8	98.7	28.9	89.1	28.7	92.5
8	31.6	100.0	28.7	90.6	31.8	98.7	28.9	89.1	28.7	92.5
9	−11.7	100.0	−10.6	90.6	−17.7	98.7	−12.1	100.0	−10.6	92.5
10	−9.1	100.0	−8.3	90.6	0.1	98.7	−9.2	100.0	−8.2	92.5
11	31.8	110.0	28.8	99.6	14.0	108.6	28.5	93.5	28.9	101.7
12	34.5	120.0	31.2	108.7	36.4	87.0	29.7	102.0	31.4	111.0
13	−59.9	120.0	−54.2	108.7	−55.0	87.0	−55.0	102.0	−55.0	111.0
14	−25.4	120.0	−23.0	108.7	−18.6	87.0	−25.3	120.0	−23.6	111.0
15	−2.7	110.0	−2.5	99.6	−22.4	−91.1	−1.1	110.0	−2.5	101.7
16	−24.7	110.0	−22.3	99.6	−12.2	−91.1	−24.9	110.0	−21.8	101.7
17	−0.7	110.0	−0.7	99.6	−6.4	−91.1	−0.4	110.0	−1.8	101.7
18	43.3	100.0	39.2	90.6	49.5	98.7	41.1	89.1	39.3	92.5
19	15.9	140.0	14.4	126.8	15.0	125.3	15.0	125.3	15.0	129.5
20	−15.9	140.0	−14.4	126.8	−15.0	125.3	−15.0	125.3	−15.0	129.5
21	0.0	140.0	0.0	126.8	0.0	125.3	0.0	125.3	0.0	129.5
Node number	Grade		Grade		Grade		Grade		Grade	
1	156.23		156.23		156.79		156.86		156.37	
2	153.92		153.92		154.95		154.34		154.15	
3	152.26		152.26		153.15		152.51		152.56	
4	155.08		155.08		155.73		155.54		155.26	
5	154.05		154.05		154.95		154.47		154.27	
6	152.71		152.71		154.68		153.00		152.98	
7	151.66		151.66		152.58		151.92		151.96	
8	152.26		152.26		153.18		152.51		152.56	
9	154.57		154.57		155.20		155.00		154.77	
10	153.81		153.81		154.42		154.21		154.04	
11	152.73		152.73		153.00		153.00		153.00	
12	151.09		151.09		151.20		151.20		151.31	
13	151.09		151.09		151.20		151.20		151.31	

and an additional four adjustment factors. For case 2b, one adjustment factor was assigned to each of the four groups of Hazen-Williams coefficients (140, 120, 110, and 100). The headloss adjustment factors obtained were 1.228, 1.814, −1.418, and 1.025 and the resulting flowrates, grades and new Hazen-Williams coefficients values are shown in Table 4. The negative adjustment factor for the third group (C = 110) implies that the specified conditions can be met only if the loss in these lines is negative (grade increase), which is, of course, not a feasible result.

Case 2c.—For this case, pipeline constants were again adjusted to produce the same specified flowrates and grade as in case 2b. However, in this case, the adjustments were made to groups of pipes based on their orientation in the system as opposed to their estimated initial Hazenditions with acceptable adjustments of designated Hazen-Williams roughness coefficients. Case 2b illustrated this situation very well. When the Hazen-Williams roughness coefficient adjustments were limited to groups of pipes with the same initial values, the field conditions specified can only be met by introducing a negative adjustment which is not a feasible result. This result is not totally unexpected because the groups of pipes to be adjusted are positioned such that it is difficult to influence the flow to the storage tanks with relatively small adjustments of pipe resistances for those groups. This point is further illustrated in Case 2c where groups of pipes are chosen such that the tank flows are much more sensitive to resistance adjustments for the various groups. For this example, adjustments from 12–22% of the designated Hazen-Williams roughness coefficients accomplished the required calibration.

An alternate approach to adjusting only Hazen-Williams roughness coefficients is illustrated by Case 2d where headloss calibration coefficients are introduced into critical pipes (pipes to the storage tanks). For this example, only relatively small headloss coefficients producing reasonably small energy losses had to be introduced to meet the specified field conditions and the associated global roughness adjustment is a very reasonable one. This result should be fairly representative of most situations and may offer a more acceptable means of calibrating many pipe networks since only a global roughness adjustment is required and the final Hazen-Williams roughness coefficients used will be related closely to known information on pipe age and conditions. Of course, a mix of headloss adjustments factors and headloss calibration coefficients may represent the best means of calibrating a pipe network to simultaneously meet several field conditions.

Although the algorithm can consider many different operating conditions it can only consider one loading condition at a time. This is, of course, a limitation since as pointed out by Walski (2), "A model is considered calibrated to the extent that it can predict the behavior of the water distribution system over a wide range of operating conditions and water use." The only practical way to circumvent this potential problem is to apply the algorithm for several different loading conditions. In the event that the headloss calibration coefficients vary for each case, some type of averaging process will be required.

Because the method presented provides for the direct determination of one or more calibration factors that will exactly meet the specified field conditions, increased importance should be attached to defining correct and appropriate field conditions along with a correct definition of the operating conditions under which the field measurements are made. The success of this procedure or any procedure for that matter, requires that the field conditions imposed are accurate with the operating conditions properly described. Poorly defined field conditions may lead to headloss adjustments and headloss calibration coefficients that are not reasonable. In any case, the calibrated system will meet the imposed field conditions, which, if not correct, would defeat the purpose of the calibration process.

Acknowledgments

This work was partially supported by the Kentucky Office of Water

HM = pipe minor energy loss,
j = number of junction nodes,
$\{j\}$ = set of pipes connecting junction j,
K_m = minor loss constant,
K_p = pipeline constant,
K_r = headloss calibration coefficient,
KM = minor loss coefficient,
k = number of loops and paths,
$\{k\}$ = set of pipes in loop or path k,
L = pipe length,
l = number of loops,
M = junction flowrate demand,
$\hat{\mathbf{M}}$ = vector of junction demands,
$\hat{\mathbf{M}}_a$ = vector of demand constants (for additional equations),
N_j = number of pipes connected to junction j,
N_k = number of pipes in loop or path k,
P = number of pipes, and
Q = flowrate.

Presented at ASCE/EWRI Congress 2005, May 15–19, 2005, Anchorage, Alaska, USA
Manuscript Number 8015

Field Testing of Water-Distribution Systems at U.S. Marine Corps Base, Camp Lejeune, North Carolina, in Support of an Epidemiologic Study

J. B. Sautner[1], M. L. Maslia[2], C. Valenzuela[3], W. M. Grayman[4], M. M. Aral[5], and J. W. Green, Jr. [6]

[1]Environmental Health Scientist, ATSDR, 1600 Clifton Road, Mailstop E-32, Atlanta, Georgia 30333; PH (404) 498-0496; FAX (404) 498-0420; email: jsautner@cdc.gov
[2]Research Environmental Engineer, ATSDR, 1600 Clifton Road, Mailstop E-32, Atlanta, Georgia 30333; PH (404) 498-0415; FAX (404) 498-0069; email: mmaslia@cdc.gov
[3]Post-Graduate Research Fellow, ORISE/ATSDR, 1600 Clifton Road, Mailstop E-32, Atlanta, Georgia 30333; PH (404) 498-0414; FAX (404) 498-0069; email: cvalenzuela@cdc.gov
[4]W. M. Grayman Consulting Engineer, 321 Ritchie Avenue, Cincinnati, Ohio 45215; PH (513) 761-1722; FAX (513) 761-5289; email: grayman@fuse.net
[5]Director, Multimedia Environmental Simulations Laboratory, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, Georgia 30332; PH (404) 894-2223; FAX (404) 894-5111; email: maral@ce.gatech.edu
[6]Post-Graduate Research Fellow, ORIS/ATSDR,1600 Clifton Road, Mailstop E-32, Atlanta, Georgia 30333; PH (404) 498-0413; FAX (404) 498-0069; email: jwgreen@cdc.gov

Abstract

An epidemiologic study is being conducted to determine if there is an association between exposure to contaminated drinking water and birth defects among children of women who lived at U.S. Marine Corps Base, Camp Lejeune, North Carolina while they were pregnant during 1968–1985. More than 12,000 pregnant women may have been exposed to well water contaminated with volatile organic compounds that was used for the potable water source and distributed through water-distribution systems at Camp Lejeune. Because of the paucity of historical water-distribution system operational data, information based on the operation of present-day water-distribution systems will be used for historical reconstruction. Present-day system operations will be modeled using water-distribution system models. To calibrate the models against hydraulic and water-quality parameters, field testing is being used to gather data and information on hydraulic, fate and transport, and operational characteristics of the water-distribution systems. Field activities include: (a) recording system pressures and storage tank water levels, and (b) conducting C-factor, fire-flow, tracer, and travel time tests. Because this is an ongoing and active investigation, the authors present an overview and summary of activities to date and some initial results from field-testing activities.

Introduction

When investigating childhood and rare diseases, epidemiologic studies explore a wide variety of risk factors, including environmental exposures. The Agency for Toxic Substances and Disease Registry (ATSDR) has determined that human exposure to groundwater contaminants occurred at U.S. Marine Corps Base (USMCB), Camp Lejeune, North Carolina, prior to 1985 (ATSDR, 1990; 1997). To investigate this exposure, ATSDR is conducting an epidemiologic case-control study to determine if there is an association between exposure of children in utero to groundwater contaminated with volatile organic compounds (VOCs) and elevated rates of spina bifida, anencephaly, cleft lip, cleft palate, and childhood leukemia (ATSDR, 1998). Because of the paucity of historical, contaminant-specific data, water-distribution system models are being calibrated to present-day conditions before reconstructing historical concentrations. To assemble data necessary to calibrate the models, ATSDR, in cooperation with USMCB Camp Lejeune, has initiated a field-testing program to gather hydraulic, water-quality, and operational parameter data. To date, field-testing activities at Camp Lejeune have included: (a) recording system pressures and storage tank water levels, and (b) conducting C-factor, fire-flow, tracer, and travel time tests.

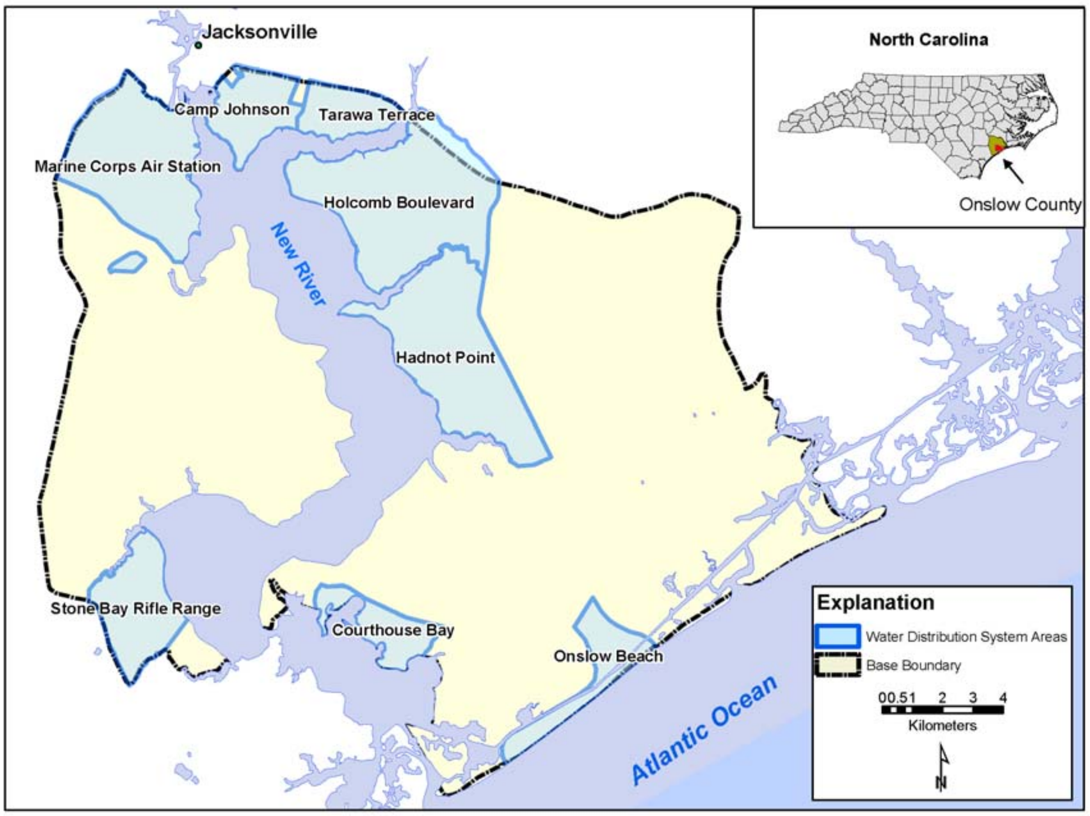


Figure 1. Water-distribution systems serving U.S. Marine Corps Base, Camp Lejeune, North Carolina

Study Area Description

USMCB, Camp Lejeune encompasses an area of about 164 mi^2 (425 km^2), and is located in Jacksonville, Onslow County, North Carolina, bordering the Atlantic Ocean. Historically, there have been eight water-distribution systems serving the base: (1) Onslow Beach, (2) Courthouse Bay, (3) Rifle Range, (4) Marine Corps Air Station, (5) Camp Johnson, (6) Tarawa Terrace, (7) Holcomb Boulevard, and (8) Hadnot Point (Figure 1). The focus of the epidemiologic study is on exposure from water-distribution systems that historically served the military base's housing—Camp Johnson, Tarawa Terrace, Holcomb Boulevard, and Hadnot Point. Presently, there are two operating water treatment plants

(WTP) that provide water for the distribution systems of interest to the epidemiologic study: (1) the Holcomb Boulevard WTP that services the Camp Johnson, Tarawa Terrace, and Holcomb Boulevard areas of the distribution system (Figure 2), and (2) the Hadnot Point WTP that services the Hadnot Point area of the distribution system (Figure 3).

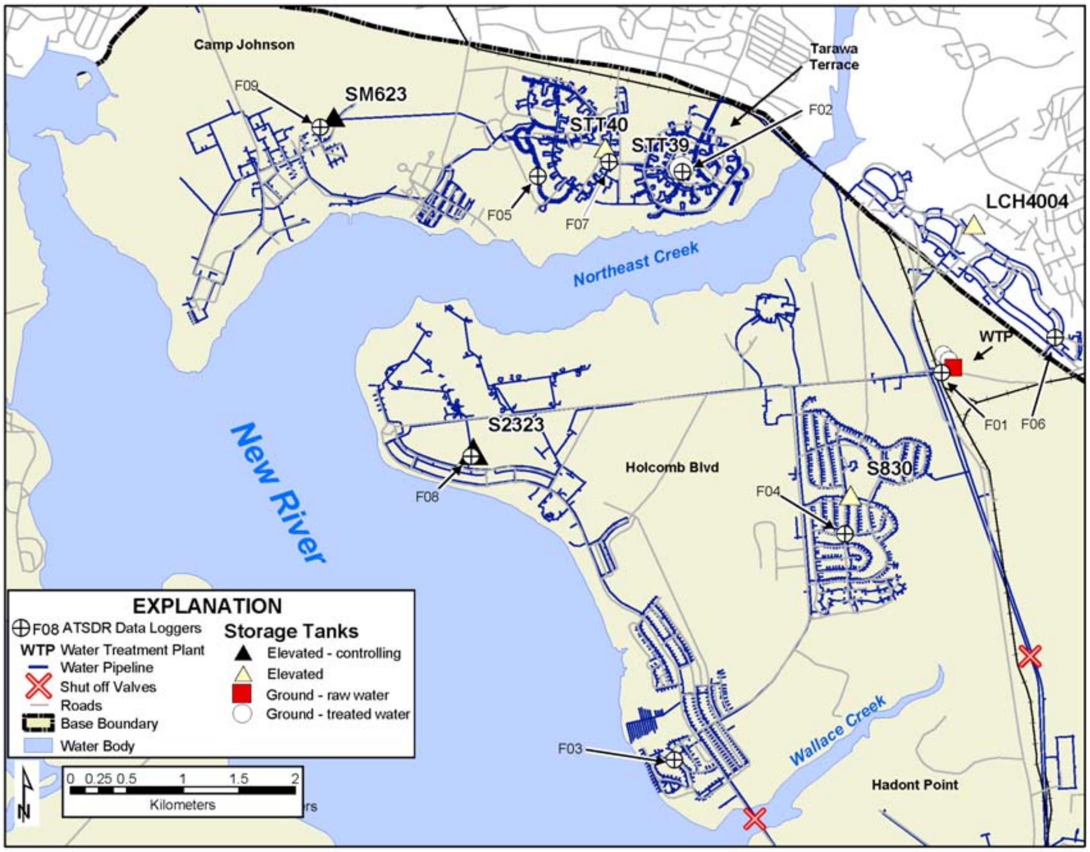


Figure 2. Holcomb Boulevard water treatment plant service area and water-distribution system

Analysis of the water-distribution systems is complex because of historical changes in system configuration and operations. Hadnot Point was the original WTP and at one time, serviced the entire base. Thus, for the ATSDR study, it will be analyzed for both present-day and historical operations. The Holcomb Boulevard WTP presently services the rest of the military housing areas. The Tarawa Terrace WTP historically serviced the Tarawa Terrace and Camp Johnson areas. After the plant was shut down, the Holcomb Boulevard plant was used to service these areas. At present, there is a treated water reservoir (ground storage tank) at Tarawa Terrace (Figure 2) that receives water directly from the Holcomb Boulevard WTP.

Shut-off valves at two locations along Wallace Creek keep Hadnot Point water isolated from Holcomb Boulevard and Tarawa Terrace water. According to base water utility mangers, valves have been opened only on very rare occasions, solely for emergency situations (S. A. Brewer, Camp Lejeune Environmental Management Division, written communication, November 19, 2004). Therefore, for purposes of the ATSDR study and water-distribution system analyses, it will be assumed that these valves are always closed.

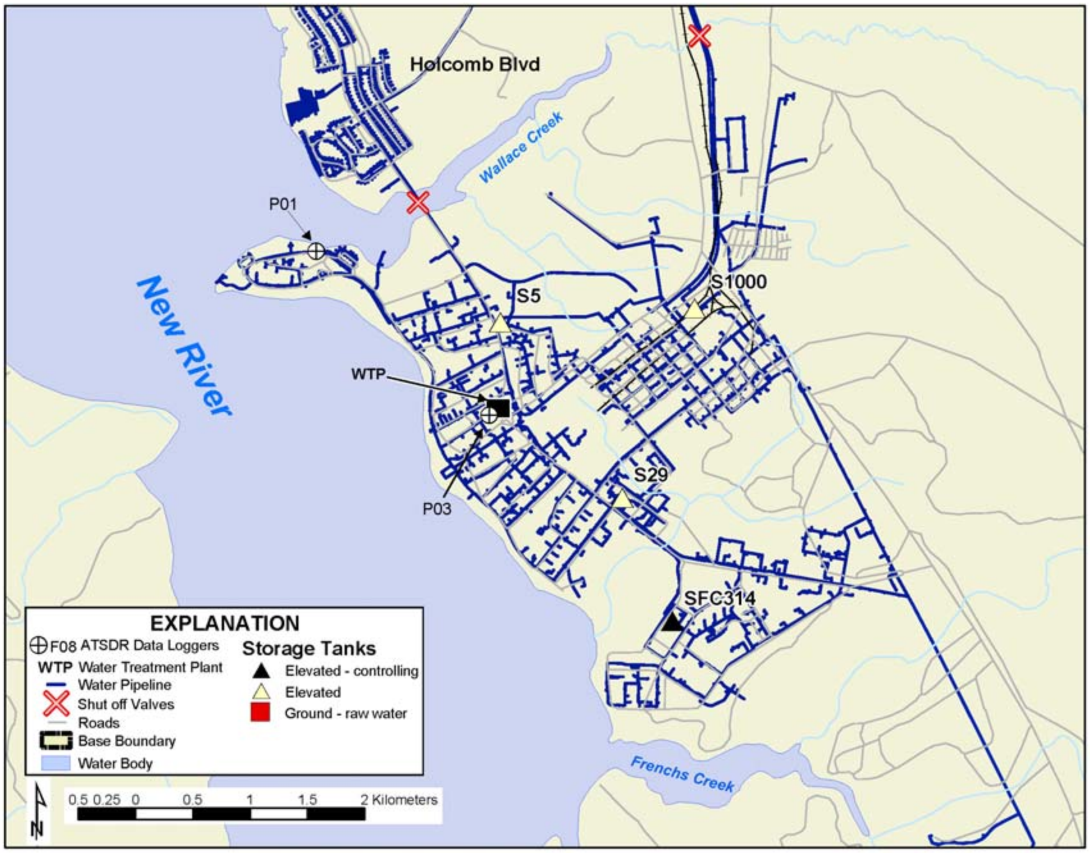


Figure 3. Hadnot Point water treatment plant service area and water-distribution system

Present-Day Operations

System pressures range from about 55–68 psi (379–469 kPa) throughout the distribution systems. As topography is very flat, ranging from sea level to less than 40 ft (12 m), hydraulic heads range 140–160 ft (43–49 m) resulting in a very mild hydraulic gradient. There are nine elevated storage tanks in the Holcomb Boulevard and Hadnot Point WTP service areas (Figures 2 and 3). The range in water level fluctuation for the elevated storage tanks is small; generally 1–6 ft (0.3–2 m) according to March 2004 data (Table 1). Three of the elevated storage tanks—SM623, S2323, and SFC314—operate as controlling tanks. When demand causes water levels in these tanks to drop below a minimum water-level mark, high-lift pumps are turned on at the WTPs or at the Tarawa Terrace treated water reservoir to fill the controlling elevated storage tanks to a maximum water level. The pumps are then shut off (Table 1).

The average annual flow for 2004 for treated water at Holcomb Boulevard WTP was 0.803 MGD (35.2 L/s). Furthermore, an additional 0.658 MGD (28.8 L/s) of treated water from the Holcomb Boulevard WTP was delivered to the Tarawa Terrace ground storage reservoir. For the Hadnot Point WTP, the average annual flow for 2004 was 2.35 MGD (103.0 L/s). Average monthly flows for 2004 for the WTPs are listed in Table 2. These data represent delivered water from the WTPs to the distribution system. Because Camp Lejeune is a military installation, the base does not require or install water consumption meters on housing units. With the exception of base power plants, other consuming entities (e.g., car washes, swimming pools, office buildings) are not metered either. Thus, quantifying the magnitude and direction of flows within the distribution system is not possible because of the absence of flow meters.

Table 1. Elevated Storage Tank Identification, Elevations, and Water Levels[*]

	Holcomb Boulevard Water Treatment Plant Area				
Storage Tank Parameter	**Camp Johnson: SM623[+]**	**Tarawa Terrace: STT40**	**Paradise Point: S2323[§]**	**Berkeley Manor: S830**	**Midway Park: LCH4004**
Elevation, Bottom of Tank, in ft (m)	82[‡] (25)[‡]	141.7 (43.2)	120.3 (36.7)	127.5 (38.9)	129.9 (39.6)
Maximum Water Level, in ft (m)	25.4 (7.7)	31.9 (9.7)	31.0 (9.4)	32.4 (9.9)	30.1 (9.2)
Minimum Water Level , in ft (m)	21.7 (6.6)	26.1 (8.0)	27.6 (8.4)	30.0 (9.1)	25.9 (7.9)
Water Level-Difference, in ft (m)	3.7 (1.1)	5.8 (1.8)	3.4 (1.0)	2.4 (0.7)	4.2 (1.3)
	Hadnot Point Water Treatment Plant Area				
Storage Tank Parameter	**S5**	**S29**	**Industrial Area: S1000**	**French Creek: SFC314[‡]**	
Elevation, Bottom of Tank, in ft (m)	126.3 (38.5)	125.3 (38.2)	127.4 (38.8)	134.8 (41.1)	
Maximum Water Level, in ft (m)	28.4 (8.6)	28.6 (8.7)	29.2 (8.9)	25.0 (7.6)	
Minimum Water Level , in ft (m)	27.5 (8.4)	27.1 (8.3)	26.8 (8.2)	20.3 (6.2)	
Water Level-Difference, in ft (m)	0.9 (0.3)	1.5 (0.4)	2.4 (0.7)	4.7 (1.4)	

[*]Data from Camp Lejeune water utility department, March 1–7, 2004
[+]Controlling tank for Tarawa Terrace treated water reservoir
[§]Controlling tank for Holcomb Boulevard WTP
[‡]Controlling tank for Handot Point WTP

With respect to water-quality parameters, all raw water is supplied from groundwater wells pumping from the Castle Hayne formation that underlies the base. Raw water concentrations of chloride and fluoride are 0.14 and 0.2 mg/L, respectively (B. T. Ashton, Camp Lejeune Environmental Management Division, electronic communication, April 6, 2004). The raw water is treated with chlorine and lime at the WTPs. As a consequence, treated water has a chloride concentration of 20 mg/L (B. T. Ashton, Camp Lejeune Environmental Management Division, electronic communication, March 31, 2004). The addition of lime causes the pH of the treated water to be high—about 8.5–9. Sodium fluoride (NaF) crystals are added to the treatment process using a gravity-feed saturator system to fluoridate the water. The concentration of fluoride in the distribution system and elevated storage tanks averages about 1 mg/L (D. E. Hill, Camp Lejeune water department, written communication, May 2004).

Table 2. Average Monthly Flows of Treated Water, 2004[*]

Month	From Holcomb Boulevard Water Treatment Plant MGD	From Holcomb Boulevard Water Treatment Plant L/s	To Tarawa Terrace Reservoir[†] MGD	To Tarawa Terrace Reservoir[†] L/s	From Hadnot Point Water Treatment Plant MGD	From Hadnot Point Water Treatment Plant L/s
January	0.824	36.1	0.728	31.9	2.570	112.6
February	0.739	32.4	0.842	36.9	2.518	110.3
March	0.699	30.6	0.761	33.3	2.431	106.5
April	0.767	33.6	0.724	31.7	2.283	100.0
May	0.889	38.9	0.788	34.5	2.334	102.2
June	0.859	37.6	0.722	31.6	2.431	106.5
July	0.784	34.4	0.655	28.7	2.371	103.9
August	0.840	36.8	0.613	26.9	2.400	105.1
September	0.954	41.8	0.401	17.6	2.202	96.5
October	0.807	35.4	0.547	24.0	2.226	97.5
November	0.732	32.1	0.568	24.9	2.285	100.1
December	0.736	32.3	0.541	23.7	2.153	94.3
Annual mean	0.803	35.2	0.658	28.8	2.350	103.0

[*]Data from Camp Lejeune water department, S. J. Whited, electronic communication, January 31, 2005.
[†]Tarawa Terrace reservoir water is treated at the Holcomb Boulevard WTP; the sum of the Holcomb Boulevard and Tarawa Terrace flows is total delivered water from Holcomb Boulevard WTP.

Field-Testing Activities

ATSDR reviewed and analyzed hydraulic and water-quality data, system operations, and a water conservation study of the base (ECG, 1999). It then began a field-testing program, in cooperation with USMCB Camp Lejeune, to obtain data necessary to calibrate present-day water-distribution models to assist the epidemiologic study. As part of this effort, ATSDR has conducted tests in the Hadnot Point and Holcomb Boulevard WTP areas, including Tarawa Terrace and Camp Johnson. These tests are considered to be “preliminary tests” that provided ATSDR with initial data to begin preliminary model simulations. The preliminary data and model simulations will be used to assist with the planning and conduct of a detailed water-distribution system test of all areas during peak-demand season in summer 2005. In what follows, the preliminary field tests are briefly described. Photographs of some of the equipment used for the field tests are shown in Figure 4A–4D. Preliminary data are presented and discussed in the “Discussion of Results” section.

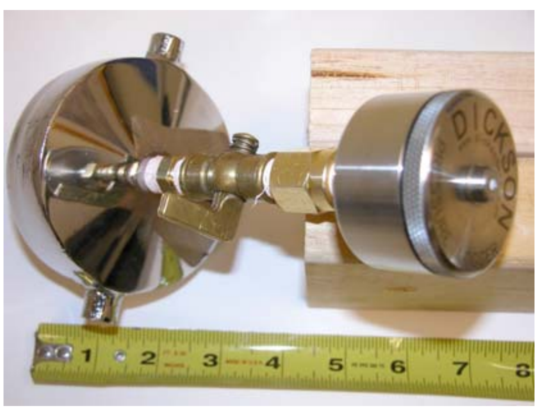
Hadnot Point WTP area, May 2004. This field test was conducted May 24–27 and consisted of three activities: (1) injecting liquid calcium chloride ($CaCl_2$), 35% by weight, into the transmission main on the distribution system side of the WTP to achieve an elevated conductance and chloride concentration, and recording conductivity and chloride concentration using continuous recording water-quality monitoring data loggers, (2) injecting a sodium fluoride solution into the transmission main to achieve an elevated fluoride concentration (before the test, the WTP fluoride was shut off so that fluoride concentrations in the distribution system pipelines approached background levels of about 0.2 mg/L), and (3) monitoring distribution system pressures with continuous recording data loggers attached to selected hydrants. In addition to continuously recording tracer concentrations and conductivity, grab samples were collected for quality assurance and quality control (QA/QC) purposes. Samples were analyzed at the Hadnot Point WTP by ATSDR staff and then also shipped to the Federal Occupational Health (FOH) laboratory in Chicago, Illinois, for analysis. Twenty-seven hydrants were selected in the Hadnot Point area as monitoring locations. For monitoring conductivity and chloride and fluoride concentrations nine hydrants were equipped with the Horiba W-23XD dual probe ion detector (Figure 4A). For monitoring conductivity, nine hydrants were equipped with the Horiba W-21XD single probe ion detector (Figure 4B), thus providing a total of 18 monitoring locations for continuously recording conductivity data. For pressure measurements, nine hydrants were equipped with continuous recording Dixon PR300 pressure data loggers (Figure 4C).

Calcium chloride solution and injection. The quantity and injection rate of $CaCl_2$ was based on: (1) using a 35% by weight liquid $CaCl_2$ solution, (2) the average flow rate of delivered water from the WTP for May 2002, and (3) assuring that the chloride concentration in the distribution system—resulting from the injection of $CaCl_2$—would not exceed the U.S. Environmental Protection Agency's secondary maximum contaminant level (MCL) for chloride of 250 mg/L. The average flow delivered by the Hadnot Point WTP for May 2002 was 3.0 MGD (131.4 L/s)—a typical year based on discussions with Camp Lejeune water utility staff. A flow-paced pump capable of injecting $CaCl_2$ at a rate of 1.0 gpm (6.3×10^{-2} L/s) with a main transmission line pressure of 55 psi (379 kPa) was used. Using this type of injection pump assured that if the flow rate of delivered water changed based on demand, the $CaCl_2$ injection rate would also change to maintain a near-constant concentration of the mixed $CaCl_2$ and treated water. A background chloride concentration of 20 mg/L and complete mixing within a short distance downstream from the injection point were also assumed. Thus, the maximum chloride concentration in the distribution system was predicted to be 164 mg/L—well below the MCL of 250 mg/L. Nine, 55-gal (208 L) drums of $CaCl_2$ were pumped into a 525-gal (1,987 L) plastic holding tank. At 0800 hours on May 25, the $CaCl_2$ was injected into the main transmission line for 6 continous hours.

Sodium fluoride solution and injection. The source of the fluoride used for the tracer injection was the NaF crystals (Solvay Fluoride) used at the Hadnot Point WTP. It comes in 50-lb (22.7 kg) bags. Based on the solubility of NaF of 42 g/L, this resulted in an equivalent of 19 g/L of F. To assure the public's health and safety, an upper limit for the fluoride concentration in the distribution system was set at 2.0 mg/L using a background fluoride concentration of 0.2 mg/L and assuming complete mixing within a short distance downstream from the injection point. The same flow and injection rates previously described for the $CaCl_2$ injection were used for the NaF injection. On the basis of mass balance calculations, 36 lb (16.4 kg) of NaF were mixed with 500 gal (1,893 L) of treated water in a 525-gal (1,987 L) plastic holding tank. Experience has shown that even when a solution is continuously stirred during an injection test, some of the solids still settle to the bottom of the tank and will not dissolve into the tracer solution (Boccelli et al., 2003). To compensate for this, the entire 50-lb (22.7 kg) bag of NaF was mixed with the 500 gal (1,893 L) of water in the tank. The treated water used to mix with the NaF crystals had a background fluoride concentration of 0.2 mg/L as the WTP fluoride was shut off the previous week in preparation for the tracer test. At 0800 hours on May 26, the NaF solution was injected into the main transmission line. Although the duration of the NaF injection was initially planned to be 6 hours long, at 1148 hours, the injection pump broke. The pump was subsequently repaired and at 1315 the injection of NaF solution was resumed. The injection was terminated at 1545 hours. Thus, unlike the $CaCl_2$ tracer test that was characterized by one continuous 6-hour long injection, the NaF tracer test was characterized by two pulses of NaF solution—the first pulse having a duration of 3 hours 48 minutes and the second pulse having a duration of 2 hours 30 minutes.

(A)

(B)



(C)

(D)

Figure 4. Photographs of selected field test equipment: (A) Horiba W-23XD dual probe ion detector inside flow cell, (B) Horiba W-21XD single probe ion detector inside flow cell, (C) Dixon PR300 continuous recording pressure logger mounted on brass shutoff valve and hydrant adapter cap, and (D) Plant PRO HFD hydrant flow tester with diffuser

Holcomb Boulevard and Hadnot Point WTP areas, August 2004. This field test was conducted August 25–27. It consisted of two activities: (1) testing different sections of pipelines of varying diameters and material types to collect hydraulic data for calculating roughness coefficients (Hazen-Williams C-factor data), and (2) applying an innovative approach for fire-flow testing (for model calibration purposes) using continuous recording pressure monitors at several fire hydrants simultaneously while different combinations of hydrants were flowed. To continuously record pressure data, the Dixon PR300 pressure data logger was used (Figure 4C). To record flow and pressure from flowed hydrants, the Plant PRO HFD hydrant flow tester with a diffuser was used (Figure 4D). C-factor tests were conducted at eight locations characterized by three different pipe materials (cast iron, polyvinyl chloride (PVC), and asbestos cement). Pipeline diameters ranged from 6–12 inches (15–30 cm). Tested pipe lengths ranged from 700–1,672 ft (213–510 m) and flows ranged from 564–1,603 gpm (35.6–67.1 L/s). Fire flow tests were conducted at 12 locations characterized by three different pipe materials (cast iron, polyvinyl chloride (PVC), and asbestos cement). Pipeline diameters ranged from 4–12 inches (10–30 cm). Tested pipe lengths ranged from 236–1,620 ft (72–494 m) and flows ranged from 773–1,120 gpm (48.8–70.7 L/s).

Holcomb Boulevard WTP, September–October 2004. This field test was conducted September 22–October 12. It consisted of monitoring fluoride decay and re-injection in the Holcomb Boulevard WTP area (including Tarawa Terrace and Camp Johnson). The purpose of this preliminary test was to (1) estimate travel time between points in the distribution system by shutting off and then restarting fluoride at the WTP, (2) to record the fill and draw characteristics at the controlling elevated storage tanks (S2323 and SM623 in Figure 2), (3) to record the sequence of when distribution-system water (with its fluoride concentration) was filling the tanks and when storage tank water (with its fluoride concentration) was being supplied to the distribution system, and (4) to conduct (QA/QC) tests on the fluoride sensors contained in the continuous recording dual probe data loggers (Figure 4C). Nine locations in the distribution system were equipped with the Horiba W-23XD continuous recording, dual probe ion detector data logger (Figure 2). Monitoring locations included the main transmission line from the WTP to the distribution system (F01 in Figure 2), the Tarawa Terrace treated water reservoir (F02), two controlling elevated storage tanks (F08 and F09), and five hydrants located throughout the housing areas (F03, F04, F05, F06, and F07). The fluoride at the Holcomb Boulevard WTP was shut off at 1600 hours on September 22. A background concentration of about 0.2 mg/L in the distribution system was reached by September 28. At 1200 hours on September 29, the fluoride was turned back on at the WTP and the test continued until loggers were removed and data downloaded on October 12. In addition to the continuous recording data loggers, split grab sample analyses were conducted for QA/QC purposes. Nine rounds of water samples were collected at each monitoring location during the test. For each round, the Holcomb Boulevard WTP water-quality lab analyzed 25 mL of the grab sample water and the FOH laboratory analyzed the remaining 225 mL of water.

Discussion of Results

Recorded pressure data confirmed that pressure throughout the Hadnot Point WTP area ranges from 55–68 psi (379–469 kPa), with a mean of about 60 psi (414 kPa). Other parts of the distribution system (Holcomb Boulevard WTP, Tarawa Terrace, etc.) are operated in a similar manner and have been operated the same way historically (1968–85). Figure 5 shows data from two continuous recording pressure data loggers recorded at 15-minute intervals during the May 2004 test. Logger P01 was located in the northwesternmost part of Hadnot Point WTP area (Figure 3). Logger P03 was located adjacent to the WTP. Because of the nearly constant pressure, flat topography, and resulting small hydraulic gradient (about 1 x 10^{-4}), as previously discussed, it would be extremely difficult to achieve a unique hydraulic calibration without using water-quality data and tracer-test results.

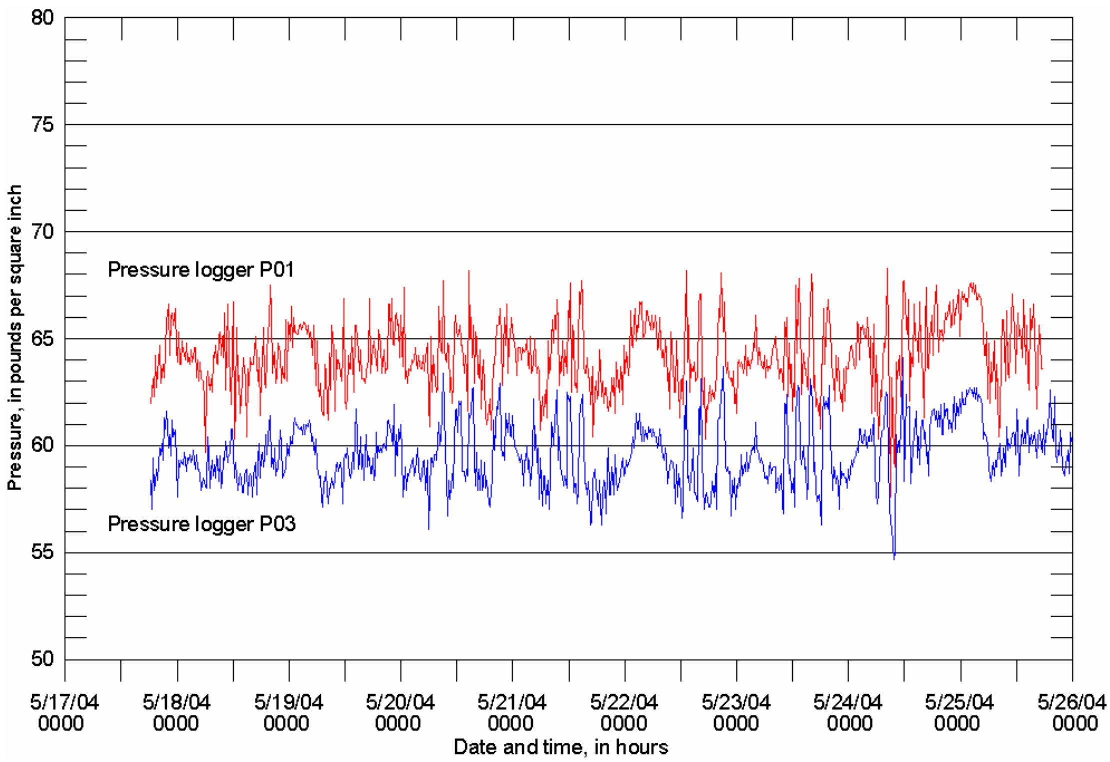


Figure 5. Graphs showing recorded pressure data in the Hadnot Point WTP area, May18–25, 2004 (refer to Figure 3 for hydrant locations)

Results from the chloride injection provide data for quantifying arrival times of the tracer at different locations throughout the Hadnot Point WTP area. Of special interest are the extremely long arrival times—in excess of 16 hours—in the northwestern part of the of the Hadnot Point WTP area (Figure 6, loggers C01, C02, and F01). Additionally comparing arrival times of the $CaCl_2$ tracer at logger location C04 with arrival times at loggers F04, C05, and F02, led investigators to suspect that there may have been undocumented closed valves in the distribution system. Post-test field verification by water utility staff confirmed the locations of closed valves, as indicated by the "•" symbol in Figure 6. Further analysis of the arrival time data shown in Figure 6 indicate that from the location of logger F08, the tracer travels east to logger C07 rather than south to loggers F06 and C06. The reason is that arrival time of the tracer at logger C07 is about 3 hours after arrival at logger F08, whereas arrival time of the tracer at loggers F06 and C06 is about 7 hours after the tracer arrives at logger F08.

Using the hydraulic data gathered during the August 2004 field-test activities, Hazen-Williams C-factors were computed for eight sections of tested pipelines (Table 3). Results presented in Table 3 show good agreement between the C-factor values determined from the field-testing activities and those published in the literature (Walski et al., 2003). The mean of the two C-factor tests conducted on PVC type pipes is 137, compared with a literature value of 147. The mean of the four C-factor tests conducted on cast iron pipes is 102, compared with literature values of 97–102. In conducting these tests, both continuous recording pressure data loggers (Dixon PR300, Figure 4C) and hand-held pressure gauges were used to record pressures. The continuous recording pressure data loggers were set to record pressure at 1-minute intervals.

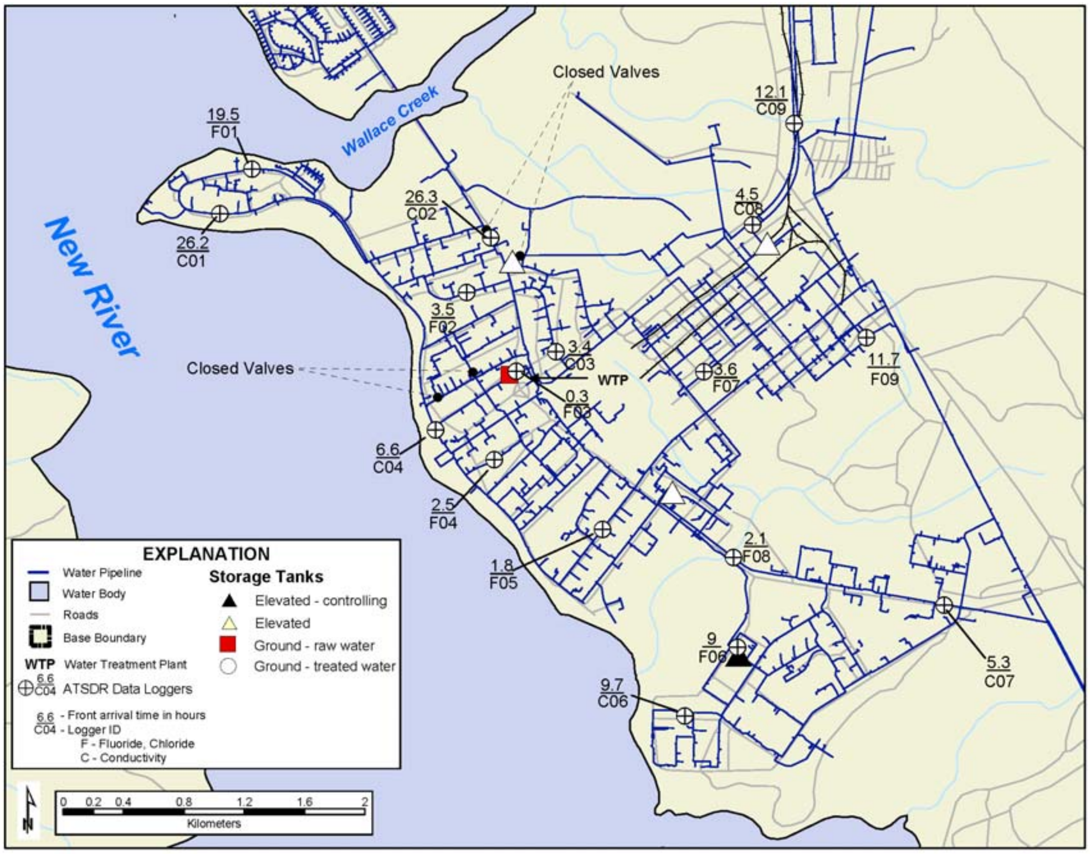


Figure 6. Arrival times of the calcium chloride tracer at monitoring locations, Hadnot Point WTP area, May 25, 2004

As discussed previously in the section on "Field-Testing Activities," the preliminary test conducted in the Holcomb Boulevard WTP area, during September–October 2004, was conducted by shutting off and then restarting the fluoride used at the Holcomb Boulevard WTP. The test was also used to conduct QA/QC on the fluoride sensors used in the continuous recording water-quality data loggers (Figure 4A). Results from the test for six selected locations (F01, F02, F03, F04, F08, and F09) are presented in Figure 7.

Logger F01 was located on the main transmission line going from the Holcomb Boulevard WTP to the distribution system (Figure 2). It can be considered as the logger that represents the source conditions for fluoride in the Holcomb Boulevard WTP service area. Agreement is very good between the continuous recording data logger (solid line) and QA/QC grab samples analyzed at the WTP water-quality laboratory and the FOH water-quality laboratory.

Logger F02 was attached to the main transmission line distributing water from the Tarawa Terrace treated water reservoir (Figure 2). The decrease and increase in fluoride concentration is significantly attenuated compared with the source logger (F01) because of the large volume of water that is contained in the reservoir. Additionally, with this logger, two fluoride sensors were used for QA/QC purposes. Comparing the two fluoride sensors (ion2 and ion3 on the graph of F02) with grab sample data indicates very consistent results. However, after about 14 days, the logger data appear to show some "drift" in the logger calibration with respect to the grab sample data.

Table 3. Hazen-Williams C-factor Values for Holcomb Boulevard and Hadnot Point Water Treatment Plant Service Areas, August 2004

Test ID	Pipe Length, ft (m)	Nominal Diameter, in.	Flow, gpm	Pipe Material	Computed C-factor	Reference C-factor*
CF-H01	848	12	1,603	PVC	161	147
CF-H02	1,181	8	590	Cast iron	102	97–102
CF-H03	793	6	564	Cast iron	93	97–102
CF-H04	1,558	8	715	Cast iron	122	97–102
CF-H05	700	10	947	Cast iron	77	97–102
CF-H06	1,416	10	835	PVC	113	147
CF-H07	1,167	8	835	Cast iron	117	97–102
CF-H08	1,672	10	920	Asbestos cement	148	150

*Data from Walski et al. (2003)
1 in. = 2.52 cm; 1 ft = 0.3048 m; 1 gpm = 0.6309 L/s

Loggers F03 and F04 were attached to hydrants located in family housing areas (Figure 2). Data from both loggers are in good agreement with grab sample data and both show logger "drift" with respect to calibration after about 10–12 days. Both loggers also show nearly identical pulses as the source logger (F01) indicating that there is probably little mixing or traveling through complex paths from the WTP to these housing areas.

Loggers F08 and F09 were monitoring elevated storage tanks. Both of these tanks are controlling tanks, so that the water level in the tank is allowed to fluctuate as demand varies (see previous discussion in section on "Present-Day Operation"). The graphs for loggers F08 and F09 clearly show the draw and fill cycles of the tanks. The graph for logger F08 shows that the fluoride concentration in the distribution system reached a near background level of about 0.2 mg/L on September 28. Even so, the elevated storage tank still has water with a fluoride concentration of about 0.8 mg/L. Data from logger F09 clearly show a more attenuated pattern than the data for logger F08.

Data from all the tests described herein are still being analyzed. These data are being used to assist with model calibration and to design a comprehensive field test that will take place during summer 2005. The hydraulic and water-quality tests to date have yielded data that investigators can use to interpret hydraulic characteristics and water-quality dynamics within the distribution systems serving family housing areas of Camp Lejeune. The pressure data shown in Figure 5 and the C-factor data listed in Table 2 represent the hydraulic characteristics of the distribution systems. Figure 6 shows arrival time data. Figure 7 presents fluoride concentration data showing the fill and draw action of elevated storage tanks. These represent water-quality data that will be used to gain insight into residence and travel times of water-distribution system dynamics. Thus, conducting field tests that yield both hydraulic and water-quality parameter data is essential to understanding parameter uncertainty and variability. The data also are essential for developing calibrated hydraulic and water-quality present-day models at Camp Lejeune to assist the epidemiologic study.

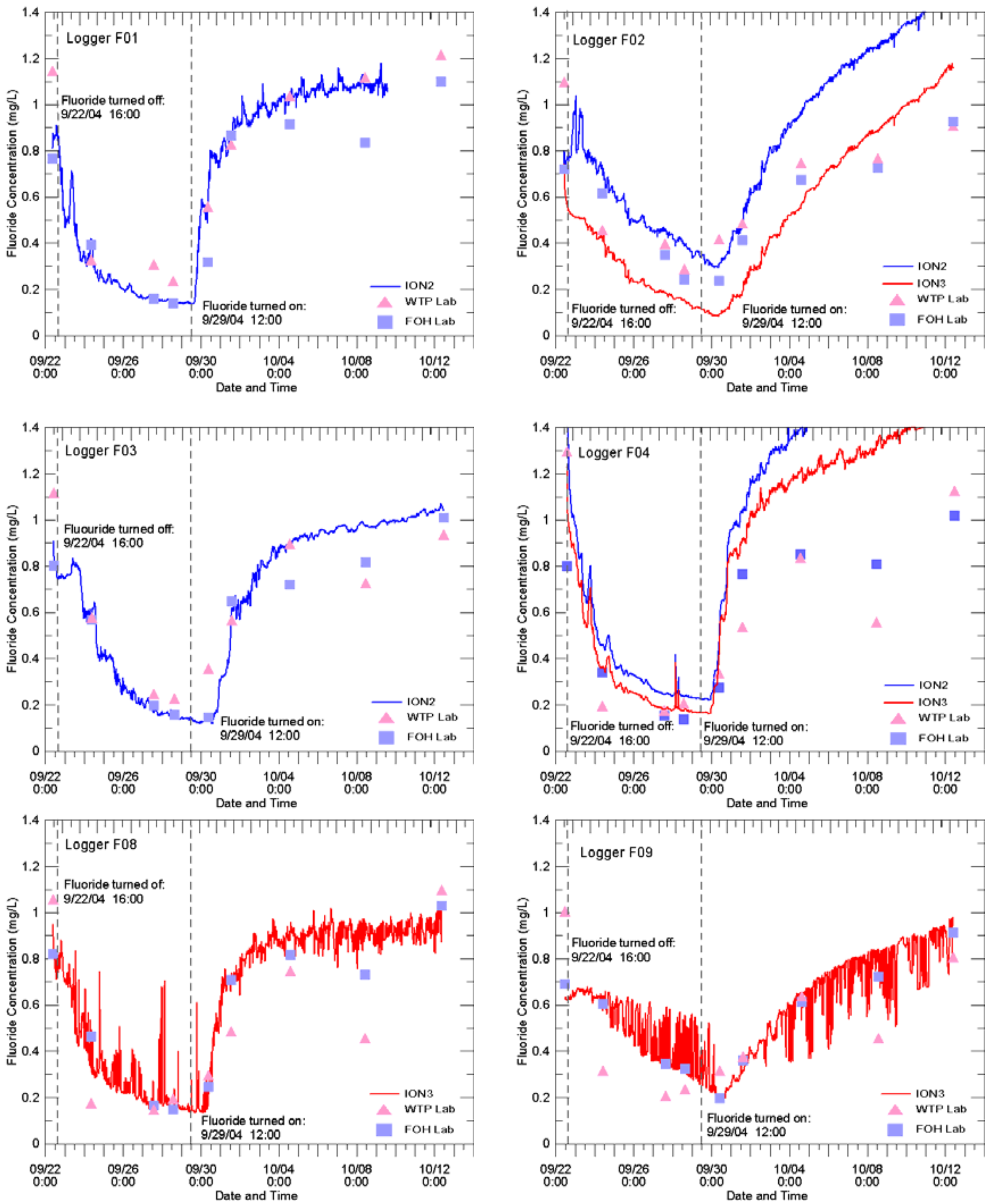


Figure 7. Graphs of fluoride concentration at logger locations F01, F02, F03, F04, F08, and F09 in the Holcomb Boulevard water treatment plant service area, September–October 2004 (refer to Figure 2 for logger locations)

Summary

This study presents results from preliminary field-test activities used to gather hydraulic and water-quality data at USMCB Camp Lejeune, North Carolina. Field tests to date have included: (a) recording system pressures and storage tank water levels, and (b) conducting C-factor, fire-flow, tracer, and travel time tests. The test data are being used to assist with hydraulic and water-quality model calibration. They also are being used to plan and carryout a more refined, detailed field test of water-distribution systems serving military base housing. These activities will assist in providing much needed model parameter data for calibrating models of the present-day water-distribution system. The present-day models are needed as a first step in reconstructing historical operations during the period 1968–1985, as part of an ongoing epidemiologic study of childhood diseases at Camp Lejeune.

Acknowledgment

The authors acknowledge the staff of the Camp Lejeune Environmental Management Division and water utility department who assisted with data requests and logistics for conducting the field tests described herein. Authors also acknowledge staff from ATSDR, National Center for Environmental Health, U.S. Geological Survey, and R.E. Faye & Associates for assistance with field testing and data collection activities.

Disclaimer

Use of trade names and commercial sources is for identification only and does not imply endorsement by the Agency for Toxic Substances and Disease Registry, the U.S. Department of Health and Human Services, the Georgia Institute of Technology, or the Oak Ridge Institute for Science and Education.

References

Agency for Toxic Substances and Disease Registry (ATSDR). (1990). *Public health assessment for ABC One Hour Cleaners, Jacksonville, Onslow County, North Carolina*, U.S. Department of Health and Human Services, Atlanta, Georgia.

Agency for Toxic Substances and Disease Registry (ATSDR). (1997). *Public health assessment for U.S. Marine Corps Base at Camp Lejeune, Military Reservation, Camp Lejeune, Onslow County, North Carolina*, U.S. Department of Health and Human Services, Atlanta, Georgia.

Agency for Toxic Substances and Disease Registry (ATSDR). (1998). *Volatile organic compounds in drinking water and adverse pregnancy outcomes, United Stated Marine Corps Base, Camp Lejeune*, U.S. Department of Health and Human Services, Atlanta, Georgia.

Boccelli, D. L., Shang, F., Uber, J. G., Orcevic, A., Moll, D., Hooper, S., Maslia, M., Sautner, J., Blount, B., and Cardinali, F. (2004) "Tracer tests for network model calibration." *Proceedings of the 2004 World Water and Environmental Resources Congress*, American Society of Civil Engineers, [CD ROM document], Salt Lake City, UT, June 27–July1.

ECG, Inc. (1999). *Marine Corps Base Camp Lejeune and Marine Corps Air Station, New River Water Conservation Analysis*, Vienna, Virginia.

Walski, T. M., Chase, D. V., Savic, D. A., Grayman, W. M., Beckwith, S., and Koelle, E. (2003). *Advanced water distribution modeling and management*, Haestad Press, Waterbury, Connecticut.

Technique for Calibrating Network Models

By Thomas M. Walski,[1] A. M. ASCE

Abstract: In calibrating a water distribution system model, the model user usually adjusts pipe roughness (e.g., Hazen-Williams C factor) or water use so that pressures and flows predicted by the model agree with values observed in the field. This paper presents formulas to assist the user in deciding whether to adjust C or water use and by how much. The key to using the formulas is to observe pressures in the system for at least two significantly different use rates. Such data are often collected during fire flow tests. A model is considered calibrated to the extent that it can predict the behavior of the water distribution system over a wide range of operating conditions and water use.

Introduction

A very important step in the development of a water distribution network model is the comparison of results predicted by the model with observations taken in the field. If the input for the model is correct, then predicted pressures and flows will match observed values. However, the data initially used to describe the network are usually not perfect, so some values must be changed in order for the predicted and observed results to agree. The question the model user must answer is, therefore: which values need to be changed and by how much?

To correct for inaccuracies in input data it is necessary to first understand the sources of these inaccuracies. These can be grouped into several categories: (1) Incorrect estimate of water use; (2) incorrect pipe carrying capacity; (3) incorrect head at constant head points (i.e., pumps, tanks, pressure reducing valves); or (4) poor representation of system in model (e.g., too many pipes removed in skeletizing the system).

In most cases it is possible to accurately determine the elevation of water in a tank or pressure at a pump during the time calibration data were collected, and unless major problems exist with the results caused by skeletizing the system, it is not standard practice to significantly change the network to be modeled by inserting additional pipes. Since it is almost impossible to accurately measure water use and pipe carrying capacity, these parameters are usually not modified to make model results agree with field observations. (Note that in this paper, the Hazen-Williams C factor is used to represent carrying capacity, although pipe roughness or Manning's n could also be used.)

If the model predicts too much head loss in a certain group of pipes, the user can either decrease the estimate of water use in that area or increase the C factor. The user is in a situation similar to a person attempting to adjust the color on a television set with two knobs. However, because of the effort and computer time involved in making a run,

[1]Research Civ. Engr., Water Resources Engrg. Group, U.S. Army Engr. Waterways Experiment Station, Vicksburg, Miss. 39180.

Note.—Discussion open until March 1, 1984. To extend the closing date one month, a written request must be filed with the ASCE Manager of Technical and Professional Publications. The manuscript for this paper was submitted for review and possible publication on July 9, 1982. This paper is part of the *Journal of Water Resources Planning and Management*, Vol. 109, No. 4, October, 1983. ©ASCE, ISSN 0733-9496/83/0004-0360/$01.00. Paper No. 18333.

the user would like to correct the data with one step rather than by trial and error.

There are several schools of thought on how to calibrate models. The AWWA Research Committee on Water Distribution Systems (5) states that ". . . the major source of error in simulation of contemporary performance will be in the assumed loadings distributions and their variations." On the other hand, Eggener and Polkowski (2) state: "the weakest piece of input information is not the assumed loadings condition, but the pipe friction factor." Cesario (1) reported that data on pump lifts, valve positions, and pressure reducing valve settings are modified first, while loadings and pipe coefficients are the last variables to be changed.

How can a user tell which parameter to change? The answer is that if the user is only trying to make the model predict pressure under one operating condition, it does not matter. The user can always force the model to fit the observations for a single set of observations. If the wrong parameter is adjusted the model, however, will give poor results if compared to observations at a different flow rate (e.g., average flow versus fire flow), since the model was calibrated using compensating errors. The key to selecting the correct parameter to change is having field observations corresponding to more than one flow rate, while knowing pump pressures, tank elevations, and valve settings corresponding to that time.

In this article, the definition of calibration is presented and formulas for determining improved values of C and water use are developed and analyzed. The data required to use the formula are described and the formulas are applied to an example problem. Some practical aspects of calibration and implications of using the formulas are also analyzed.

Definition

Shamir and Howard (4) state that calibration "consists of determining the physical and operational characteristics of an existing system and determinimg the data [that] when input to the computer model will yield realistic results." The AWWA Research Committee on Distribution (5) used the word "verified" in place of "calibrated" but described a process of calibration: "System simulation is considered verified during preliminary analysis for design when calculated pressures are satisfactorily close to observed field gage readings for given field source send-out and storage conditions. If simulation is not satisfactory, the possibility of local aberrations, such as open boundary valves, is investigated. In the absence of other expected causative factors, the assumed local arterial network loads are adjusted until computed and observed field pressures are within reasonable agreement for various levels and extremes of demand, pumping, and storage." (Note that there should be agreement over a wide range of operating conditions. Problems occur, in part, because field observations are usually not made over a wide enough range of operating conditions.)

The following, more precise, definitions are proposed. *Calibration* of a water distribution model is a two step process consisting of: (1) Comparison of pressures and flows predicted with observed pressures and flows for known operating conditions (i.e., pump operation, tank levels, pressure reducing valve settings); and (2) adjustment of the input data for the model to improve agreement between observed and predicted valves. A model is considered *calibrated* for a set of operating conditions and water uses if it can predict flows and pressures with reasonable agreement. Calibration at one set of operating conditions and water use does not necessarily imply calibration in general, although confidence in the accuracy of results from the model should increase with an increase in the range of conditions for which the model is calibrated.

Reasonable Agreement

Quantifying what is meant by "reasonable agreement" is difficult since it depends on: (1) The quality of the pressure and elevation data used; and (2) the amount of effort the model user is willing to spend fine tuning the model. While a well calibrated pressure gage is accurate to ±3 ft (1 m) of pressure head, elevation data are of widely varying quality. In the worst case, it must be read from contour maps with 20 ft (6.1 m) intervals which, with even the best interpolations, is only accurate to ±7 ft (2.1 m). In most cases, better data on hydrant elevation are available so that static head can usually be determined to an accuracy of ±5 ft (±1.5 m) to ±10 ft (±3.1 m). [Velocity head is usually on the order of 1 ft (0.31 m) and is ignored in virtually all models of water distribution systems.] The problem is also complicated by the fact that tank levels, pump operation, and water use may change dramatically during the time the data are collected. This effect can be minimized by collecting data over a short period of time, taking a snapshot of the system, during which there are no changes in pump operation and during which tank levels are recorded continuously (or at least observed frequently). Data collected over several years without regard to pump operation or tank levels are useful only for the crudest verification of a network model. Systems with many wells and pressure reducing valves and a great deal of change in hydraulic grade line elevation across the system are generally much more difficult to calibrate than small systems with only one or two pumps or tanks, and no automatic valves.

Therefore, given a good data set, a model user should be able to achieve an average difference between predicted and observed head of ±5 ft (1.5 m) with a maximum difference of ±15 ft (5.0 m). With a poor data set, an average difference of ±10 ft (3.1 m) with a maximum difference of ±30 ft (10 m) is a reasonable target.

An alternative way of looking at calibration accuracy is in terms of head loss. If the system only has 10 ft (3 m) of head loss from the source to the node with the lowest head in the system, then it should be easy to achieve an accuracy of ±5 ft (1.5 m). If there are several hundred feet of head loss, an accuracy of ±20 ft (6 m) may be quite good.

Development of Equations for Correcting C and Q

The following information may be obtained in conjunction with routine fire flow testing.

1. The hydraulic grade line elevations at a given node (the hydrant)

corresponding to some lower flow rate, Q (hydrant closed), and higher flow rate, $Q + Q_f$ (hydrant open). These elevations may be defined as h_1 and h_2, respectively.

2. The flow at the hydrant, Q_f, during the flow test.

3. The hydraulic grade line elevation, H, at some nearly constant head location (e.g., pump, tank, pressure reducing valve).

Pertinent information that will generally be unknown includes: (1) The water use, Q, corresponding to h_1; and (2) the C factor.

For this situation it is possible to develop expressions to calculate the correct C factor for pipes serving a given area and the correct water use, Q for a given group of nodes in a network model. This development is presented in the following.

To develop a simple rule for calibration, represent a section of the system from a node with known head to the test hydrant as a single equivalent pipe. The head losses between the constant head point and the node at which the fire flow test was conducted can be expressed as

$$H_1 - h_1 = K_1\left(\frac{S}{C}\right)^{1.85} \quad \text{(1a)}$$

and $$H_2 - h_2 = K_2\left(\frac{S + Q_f}{C}\right)^{1.85} \quad \text{(1b)}$$

in which Q_f = difference in flow between high and low flow condition, in gallons per minute; H_1 = hydraulic grade line elevation at known head point for low flow, in feet; H_2 = hydraulic grade line elevation at known head point for high flow, in feet; h_1 = hydraulic grade line elevation at test node for low flow, in feet; h_2 = hydraulic grade line at elevation test node for high flow, in feet; K_1 = K for equivalent pipe for low flow; K_2 = K for equivalent pipe for high flow; S = water use at nodes significantly affecting hydrant test, in gallons per minute, $S = \Sigma_{i=1}^{m} Q_i$; and m = number of nodes affecting test.

There are four unknowns in the preceding equations, K_1, K_2, C, and S. K_1 and K_2 depend upon the diameters and lengths of the complicated piping network and are equal if there is no water use between the constant head point and the test hydrant. (Note that if the known head point is a tank, a pressure reducing valve that operates in the same node for high and low flow, or a pump with a flat head-characteristic curve, then it is acceptable to let $H_1 = H_2$.)

K_1 and K_2 can be estimated utilizing the user's initial estimates of C and Q (referred to as C_e and Q_e). Given the user's values of C_e and Q_e, the model can be used to predict the hydraulic grade line elevation as h_3 for flow Q_e, and h_4 for flow $Q_e + Q_f$. Expressions similar to Eq. 1 can be used to estimate K_1 and K_2 if the user's estimates, Q_e and C_e, are not too greatly in error:

$$K_1 = (H_1 - h_3)\left(\frac{C_e}{S_e}\right)^{1.85} \quad \text{(2a)}$$

$$K_2 = (H_2 - h_4)\left(\frac{C_e}{S_e + Q_f}\right)^{1.85} \quad \text{(2b)}$$

in which h_3 = model estimate of h_1 for C_e and S_e, in feet; h_4 = model estimate of h_2 for C_e and S_e, in feet; C_e = user's estimate of C; Q_e = user's estimate of Q; and $S_e = \Sigma_{i=1}^{m} Q_{ei}$.

Inserting the values of K_1 and K_2 into Eq. 1 and solving for S and C yields

$$S = \frac{Q_f}{\frac{b}{a}\left(1 + \frac{Q_f}{S_e}\right) - 1} = AS_e \qquad (3a)$$

$$C = \frac{Q_f C_e}{b(S_e + Q_f) - a\,S_e} = BC_e \qquad (3b)$$

$$\text{in which} \quad a = \left(\frac{H_1 - h_1}{H_1 - h_3}\right)^{0.54} \qquad (3c)$$

$$\text{and} \quad b = \left(\frac{H_2 - h_2}{H_2 - h_4}\right)^{0.54} \qquad (3d)$$

$$A = \frac{Q_f}{\frac{b}{a}(S_e + Q_f) - S_e} \qquad (3e)$$

$$B = \frac{Q_f}{b(S_e + Q_f) - aS_e} \qquad (3f)$$

Eqs. 3 can be used to calculate improved values of C and Q for calibration. The coefficients a and b are useful as indicators of the magnitude and the source of error in the initial estimates. (This is analyzed in more detail later in this paper.)

To adjust estimated use rates at individual nodes use

$$Q_i = AQ_{ei}, \quad i = 1, 2, \ldots m \qquad (4)$$

Similarly, the C-factor for many pipes in an area must be adjusted, so Eq. 3b is more correctly written as

$$C_j = BC_{ej}, \quad j = 1, 2, \ldots n \qquad (5)$$

in which n = number of pipes affected by tests.

The parameters, A and B, are actually correction factors for the use and C factors, respectively. As such, an A of 1.15 means that water use should be increased by 15% over the initial estimate, while a B of 0.8 means that the C factor in the affected pipes should be reduced by 20%.

Application

The technique described earlier works best when applied to pie shaped sectors emanating from the known head node. Where the water use changes dramatically (about an order of magnitude) along one of these sectors, it is necessary to subdivide into two or more tiers and solve each tier successively from the source as a known head node. The downstream end of the first tier would become the known head node for the second, and so on.

Ideally, the data required to use Eqs. 3 already exist in the files of the water utility. Q_f, h_1, and h_2 are routinely evaluated as part of fire flow tests. Utilities with even the crudest instrumentation record water elevation in tanks or pressure at pumps, so determining H should be relatively easy. In rare instances (1), telemetry data on the entire system are available in sufficient detail for calibration.

Usually not all of the data required are available so some fire flow tests should be conducted (3). The following should be kept in mind when collecting the data: (1) Conduct the tests on the perimeter of the skeletal distribution system (i.e., not near source or on lines not included in the model); and (2) use as large a test flow at the fire hydrant as possible.

The preceding rules are necessary to insure that $H_1 - h_1$ and $H_2 - h_2$ are large. If they are not, then small errors in determining h_1 and h_2 can result in large errors in Q and C. It will also be shown in the section titled "Analysis" that if Q_f is much smaller than S, it is difficult to determine the source of error (i.e., C_e or Q_e) in the initial estimate, and errors in measuring h_1 and h_2 are accentuated.

The method for calculating C and Q developed in this paper is based on the assumption that Q_f is known. If the difference in water use between high and low flow conditions is not known, then Eqs. 3 cannot be used. The beauty of these equations lies in the fact that the data required for calibration can be collected in a single test.

Application to Network Example

Consider the skeletal network shown in Fig. 1, with the exact values for diameter, length, C, and water use given in the figure and Tables 1–3.

Suppose a model user decides to model the pipes using a C factor of 115, based on the age and type of pipe, and the user incorrectly estimates water use at nodes 20 and 30 as 100 (0.0063), and 400 (0.0252) gpm (m^3/s). If the user knows the hydraulic grade line (HGL) elevation at the tank to be 200 ft (60.9) m), the model predicts the HGL elevation as shown in row 2 of Table 1. (The measured HGL elevations based on

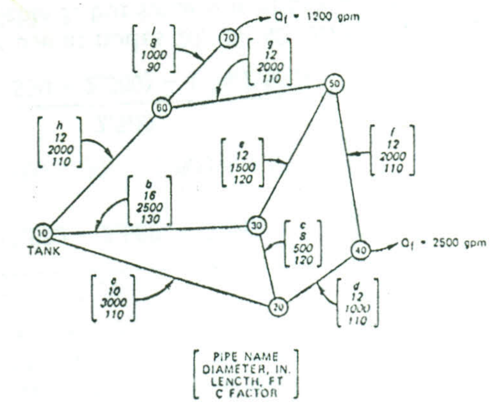


FIG. 1.—Example of Water Distribution System (gpm × 6.309 × 10^{-5} = m^3/s; in. × 25.40 = mm; and ft × 0.3048 = m)

TABLE 1.—Hydraulic Grade Line (HGL) Data for Network Example, in Feet

Water use (1)	Node 40		Node 70	
	Low flow HGL (2)	High flow (2,500 gpm) HGL (3)	Low flow HGL (4)	High flow (1,200 gpm) HGL (5)
Actual readings	181	150	173	64
Initial run	189	162	184	123
Corrected run	180	147	171	63

Note: ft × 0.3048 = m; gpm × 6.309 × 10^{-5} = m^3/s.

TABLE 2.—C-Factors for Network Example

Pipe (1)	Actual (2)	Initial (3)	Corrected (4)
a	100	115	115
b	130	115	115
c	120	115	115
d	110	115	115
e	120	115	115
f	110	115	115
g	110	115	115
h	110	115	83
i	90	115	83

TABLE 3.—Water Use for Network Example, in Gallons per Minute

Node (1)	Actual (2)	Initial (3)	Corrected (4)
20	500	150	205
30	2,000	400	548
40	500	500	685
50	1,500	1,500	2,055
60	1,000	1,000	1,000
70	400	400	400
Total	5,900	3,950	4,893

Note: gpm × 6.309 × 10^{-5} = m^3/s.

the exact data are given in row 1). With the data in rows 1–2, it is possible to calculate a and b for nodes near node 40 and 70 as

$$a(40) = \left(\frac{200 - 181}{200 - 189}\right)^{0.54} = 1.34 \qquad (6a)$$

$$a(70) = \left(\frac{200 - 173}{200 - 184}\right)^{0.54} = 1.33 \qquad (6b)$$

$$b(40) = \left(\frac{200 - 150}{200 - 162}\right)^{0.54} = 1.16 \qquad (6c)$$

$$b(70) = \left(\frac{200 - 64}{200 - 123}\right)^{0.54} = 1.36 \qquad (6d)$$

For this problem, nodes 20, 30, 40, and 50, and pipes a, b, c, d, e, and f are to be adjusted using the results from node 40. The C factor for pipe g is not adjusted since it does not significantly affect either hydrant test. For node 40, Eqs. 3 give

$$\left.\begin{aligned} A &= \frac{2{,}500}{\dfrac{1.16}{1.34}(2{,}550 + 2{,}500) - 2{,}550} = 1.37 \\ S &= 150 + 400 + 500 + 1{,}500 = 2{,}550 \end{aligned}\right\} \qquad (7a)$$

$$B = \frac{2{,}500}{1.16(2{,}550 + 2{,}500) - 1.34(2{,}550)} = 1.02 \qquad (7b)$$

The water use at nodes 20, 30, 40, and 50 are multiplied by A and are listed in Table 3, but since B is approximately one the C factors are not modified.

Next, A and B are calculated for the nodes (60 and 70) and pipes (h and i) influencing pressures around test node 70:

$$A = 0.95 \qquad (8a)$$

$$B = 0.72 \qquad (8b)$$

Since A is near one, it may not be necessary to change water use in that area (those numbers are already estimated correctly). However, the C factor for pipes h and i are multiplied by 0.72.

When the corrected values for C and Q are input to the model, the predicted pressures (Table 1 row 3) are much closer to the observed pressures (row 1). In general, the values of Q and C have improved but individual values of Q (node 50) and C (pipe h) in some cases actually become less accurate. (The total use is more accurate even though use at some individual nodes may be less accurate.)

Analysis

The parameters, a and b, as defined in Eqs. 3 are useful dimensionless indicators of the error in head loss at low and high flow conditions, respectively. Values less than one indicate that the model overestimated head loss as compared to observed head loss, while values greater than one indicate the model underestimated head loss. The model can be considered calibrated when $a \sim 1 \sim b$.

Values of a and b should generally be on the order of one. For example, if a = 2 or 0.5, the difference between observed and predicted head at the node would be 2.6 times the head loss which would indicate a very large error. The proper interpretations of a and b are summarized in Table 4.

While a and b provide insight into the nature of the error, Eqs. 3 must be used to actually determine the corrected values for the C factor and water use, Q. To give the reader an appreciation for the effect of a and

TABLE 4.—Interpretation of a and b

Value (1)	a (2)	b (3)
<1	Too much head loss predicted at low flow	Too much head loss predicted at high flow
=1	Head loss correct at low flow	Head loss correct at high flow
>1	Too little head loss predicted at low flow	Too little head loss predicted at high flow

b on the difference between the initial and corrected values of C and Q, the percent difference in C (i.e., $|C - C_e|/C_e$) and Q (i.e., $|Q - Q_e|/Q_e$) were calculated for a large array of values for a and b while letting C_e = 120, $Q_f = Q_e = S$ = 1,000 gpm (0.063 m³/s), for a one node system.

Fig. 2 shows the affect of a and b on C. Within the area titled "Accept C," the change in C is less than 10% from C_e. The dashed line corresponds to a combination of a and b for which $C = C_e$. In this case, it corresponds to the line $b = (a + 1)/2$ which was determined from Eq. 3a for $C \doteq C_e$. The region titled "Infeasible" corresponds to combinations of a and b for which the denominator in Eqs. 3 is negative.

Fig. 3 shows the affect of a and b on Q. Within the area titled "Accept Q," Q is within 10% of Q_e. The dashed line corresponds to $Q = Q_e$ and in this case is the line representing $a = b$. For $b >> a$, the water use estimate should be reduced, and for $b << a$, the water use estimate should be increased.

Figs. 2–3 were combined to yield Fig. 4 which summarizes the corrections to be made to C_e and Q_e to calibrate the model. Those who believe in achieving calibration by adjusting C are working under the assumption that $a = b$. Those who only adjust Q are assuming implicitly that

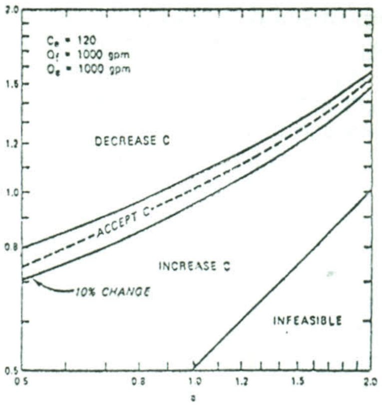


FIG. 2.—Effect of a and b on Hazen-Williams C (gpm × 6.309 × 10⁻⁵ = m³/s)

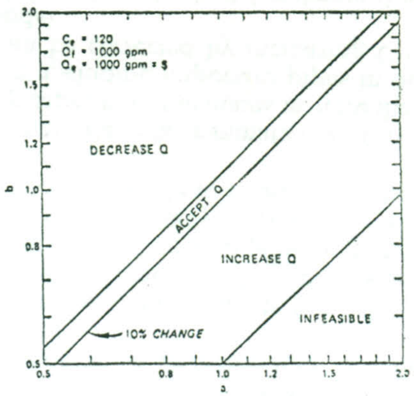


FIG. 3.—Effect of a and b on Water Use Estimate (gpm × 6.309 × 10⁻⁵ = m³/s)

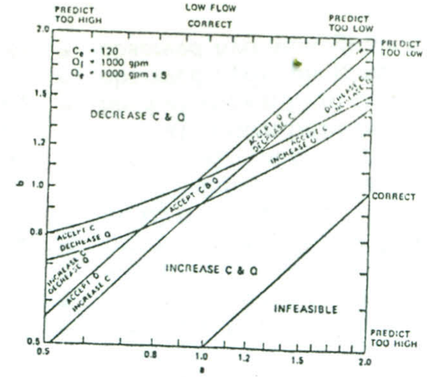


FIG. 4.—Summary of Corrections Required to Achieve Calibration (gpm × 6.309 × 10⁻⁵ = m³/s)

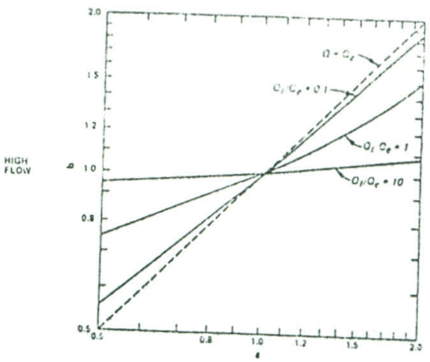


FIG. 5.—Effect of Q_f/Q_e on Lines Corresponding to $C = C_e$

$$b = \frac{aQ_e + Q_f}{Q_e + Q_f} \quad (9a)$$

or, for the data in Fig. 4:

$$b = \frac{(a + 1)}{2} \quad (9b)$$

There are a few special cases which provide insight into the adjustments needed for calibration:

1. If $a = b$ (similar error at high and low flow), adjust C.
2. If $a = 1$ and $b \neq 1$ (good calibration at low flow), adjust both Q and C by similar fraction.
3. If $a \neq 1$, $b = 1$, and $Q_f >> Q_e$ (good calibration at high flow, and low flow much less than high flow), adjust Q.

Statement 1 can be proven by setting $Q = Q_e$ in Eq. 3a and solving for b. Statement 2 can be proven by letting $a = 1$ in Eqs. 3 and showing that $(C - C_e)/C_e = (Q - Q_e)/Q_e$. Statement 3 can be proven by letting $C = C_e$ and $Q_e/Q_f = 0$ in Eq. 3b and solving for b.

The importance of insuring that Q_f is significant in comparison to Q_e (actually S when several nodes are affected) can be shown by plotting the line for $C = C_e$ for several values of Q_f/Q_e, as shown in Fig. 5. As Q_f/Q_e approaches zero, the line for $C = C_e$ approaches the $Q = Q_e$ line and it is not possible to determine if C or Q should be adjusted. In practical terms, as Q_f/Q_e becomes small, model results become increasingly sensitive to errors in measuring h_1 and h_2. Therefore, the accuracy of Eqs. 3 is greatest for large flows during fire flow tests.

Source of Errors in Initial Simulations

Ideally, the data used in the initial runs of a model are so accurate that there is no need to correct the input using Eqs. 3. In some cases, the corrections are small and are due to random error in estimating C or predicting water use at the time the calibration data were taken. When the corrections are large, the model user should not blindly change C or Q without trying to understand why the initial estimates, C_e and Q_e, were poor. Taking time to understand the source of the errors can provide valuable insights into the behavior of the water distribution system. This is shown in the following examples.

If the corrected C is much lower than the initial estimate (i.e., $C << C_e$), it is implied that the hydraulic carrying capacity of the mains is less than anticipated. While this may be due to an increase in pipe roughness with age, it may also be due to a closed or partially closed valve in a main. It is not uncommon during a modeling study to locate valves that have been mistakenly left closed.

A corrected C that is much greater than the initial estimate (i.e., $C >> C_e$) implies that the hydraulic carrying capacity of the mains is more than anticipated. This is usually due to not including important pipes in developing the skeletal model. This can be corrected by increasing C or including additional pipes in the model.

If the corrected water use is much less than the initial estimate (i.e., $Q << Q_e$), the model user should attempt to identify water users not in operation when the data were collected. For example, the field observations may have been made on a school holiday or workers at a factory near the test node may have been on strike that day.

If the corrected water use is much greater than the initial estimate (i.e., $Q >> Q_e$), the model user should look for unusual water uses. For example, was the municipal swimming pool being filled that day or was lawn watering use especially high because of a drought? The error may also be due to an illegal connection or a large main break.

Recommended Procedure

The recommended calibration procedure can be summarized as follows:

1. Prepare model data to the greatest accuracy possible.
2. Make initial run of model.
3. Measure H_1, H_2, h_1, h_2, and Q_f.
4. If error is acceptable, calibration is complete; if not, go to next step.
5. Calculate a and b.
6. Calculate corrected C and Q.
7. Rerun model.
8. If error is acceptable, calibration is complete.
9. Return to 5 (or if that is not successful, return to 3).

Summary

The formulas presented in this paper can greatly simplify the process of calibrating water distribution system models. The key to using the formulas is collecting pressure data for the system under both high and low water use conditions while recording pump and tank operation. This can be done by conducting fire flow tests.

In addition to the formulas for calculating C and Q, some qualitative guidance was developed for calibration. Given the results of the model for the initial estimates of C and Q:

1. Adjust C if there is similar error at high and low flow.
2. Adjust C and Q by similar amounts if the model is accurate at low flow but inaccurate at high flow.
3. Adjust Q if the model is accurate at high flow but inaccurate at low flow.

Acknowledgment

This paper was prepared under the Water System Operation, Maintenance, and Rehabilitation work unit of the Water Conservation and Supply Program at the U.S. Army Corps of Engineer Waterways Experiment Station. Comments on the original draft were provided by Dr. Joe Miller Morgan.

Appendix I.—References

1. Cesario, A. L., "Computer Modeling Programs: Tools for Model Operations," *Journal of the American Water Works Association*, Vol. 72, No. 9, Sept., 1980, pp. 508–513.
2. Eggener, C. L., and Polkowski, L. B., "Network Models and the Impact of Modeling Assumptions," *Journal of the American Water Works Association*, Vol. 67, No. 4, Apr., 1975, pp. 189–196.
3. "Form and Procedure for Fire Flow Test," *Journal of the American Water Works Association*, Vol. 68, No. 5, May, 1976, pp. 223–268.
4. Shamir, U., and Howard, C. D. D., "Engineering Analysis of Water Distribution System," *Journal of the American Water Works Association*, Vol. 69, No. 9, Sept., 1977, pp. 510–514.
5. "Water Distribution Research and Applied Development Needs," American Water Works Association Research Committee on Distribution Systems, *Journal of the American Water Works Association*, Vol. 66, No. 6, June, 1974, pp. 385–390.

Appendix II.—Notation

The following symbols are used in this paper:

A = correction factor for water use, $Q_f/[(b/a)(S_e + Q_f) - S_e]$;
a = indicator of accuracy of calibration at low flow, $((H_1 - h_1)/(H_1 - h_3))^{0.54}$;
B = correction factor for C factor, $Q_f/[b(S_e + Q_f) - aS_e]$;
b = indicator of accuracy of calibration at high flow, $((H_2 - h_2)/(H_2 - h_4))^{0.54}$;
C = actual Hazen-Williams C factor;
C_e = initial estimate of C;
C_{ej} = initial estimate of C for pipe j;
C_j = correct value of C for pipe j;
H_1 = head at known head point at low flow, in feet;
H_2 = head at known head point at high flow, in feet;
h_1 = observed hydraulic grade line elevation at low flow, in feet;
h_2 = observed hydraulic grade line elevation at high flow, in feet;

Proceedings of the World Water and Environmental Resources Congress, June 27–July 1, 2004, Salt Lake City, Utah

Tracer Tests for Network Model Calibration

D. L. Boccelli[*†] F. Shang[*] J. G. Uber[*] A. Orcevic[*]
D. Moll[†] S. Hooper[‡] M. Maslia[§] J. Sautner[§] B. Blount[†]
F. Cardinali[†]

Abstract

Typical distribution system network model calibration approaches adjust roughness coefficient values to match observed pressure and supervisory control and data acquisition (SCADA) equipment data assuming known user demands. Pressure data alone, however, do not contain information related to hydraulic residence time and travel path, making such data less useful for calibrating both the hydraulic and water quality portions of a distribution system network model. This study presents a network-wide dual-tracer field-scale study, coupled with water quality monitoring, to collect a rich data set for evaluating hydraulic and water quality issues. The raw data illustrate the path-specific information that can be generated beyond the use of pressure measurements alone. The observed data are used to minimally calibrate a distribution system model that is provided by the utility, and illustrates the use of tracer data for providing confidence with respect to the predictive ability of the network model. Additional considerations related to automated calibration techniques and the potential benefits of more accurate distribution system models are discussed.

1 Introduction

Model calibration requires two components: a) data collection and b) calibration technique. Typical automated calibration techniques incorporate consumer demands based on billing records and total flow data, and adjust pipe roughness coefficients to fit pressure measurements and storage levels (Walski et al., 2001) – neither of which relates directly to residence time or flow path, and thus water quality calibration. The current study has, for the first time, used network-wide dual-tracer studies (sodium chloride (NaCl) [conductivity] and fluoride) to collect data that not only relate to residence time and flow path but also to water quality.

[*]University of Cincinnati, Cincinnati, OH
[†]Centers for Disease Control and Prevention, Atlanta, GA
[‡]Emory University, Atlanta, GA
[§]Agency for Toxic Substances and Disease Registry, Atlanta, GA

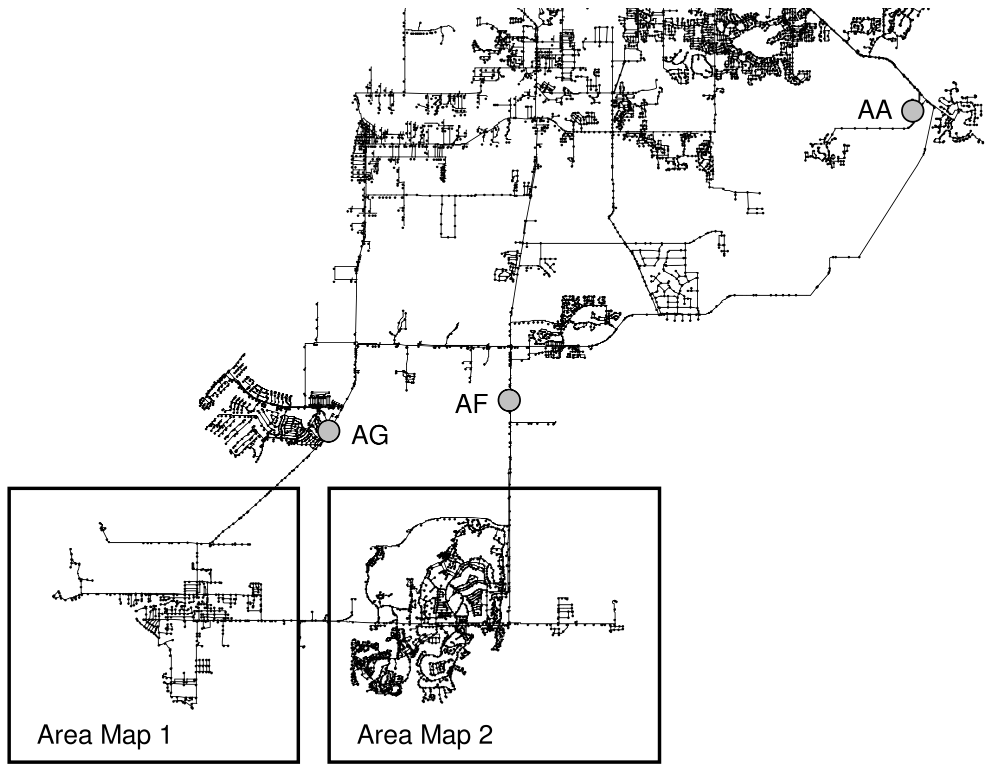


Figure 1: Network study area. Gray circles are monitoring locations. Location AA is the water treatment plant, whereas AF and AG are transmission mains that constitute alternate flow paths to the south-western study endpoint.

The rich data set collected during a recent tracer study are presented, as are the associated model-predicted results that used a minimally calibrated distribution system model. The discussion addresses future needs for more complete model calibration techniques and the potential benefits.

2 Study Area Description

A schematic of the study area piping and sampling locations are shown in Figure 1 (this area represents roughly one-half the treatment plant service area). Distribution system flow paths begin at the single treatment plant source (AA) and continue through two main transmission lines (AF and AG), leading to two residential/commercial districts (areas 1 and 2). Flow paths continue through area 2 to feed area 1, which is the ultimate output destination for the water quality study.

Areas 1 and 2 and their respective sampling locations are shown in Figure 2. Sampling locations AH and AI monitor the two main flow paths through area 2, which in turn feed area 1 (determined by prior input-output analysis). Sampling locations AB and AE monitor the two flow paths feeding area 1, and AC and AD

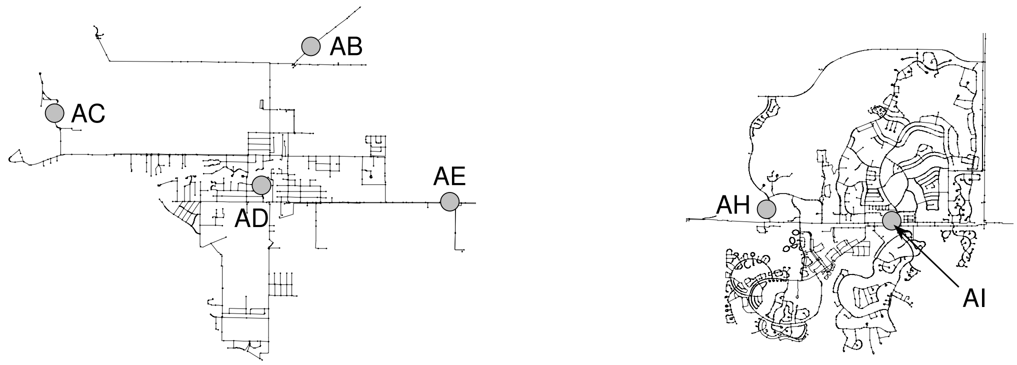


Figure 2: Area maps 1 and 2 of the main study area. Gray circles are monitoring locations. Location AA is the water treatment plant, whereas AF and AG are transmission mains that comprise alternative flow paths to the southwestern study endpoint.

are interior locations that are affected by increasingly complex sets of flow paths that have resulted from flow splitting and recombination.

Overall, the portion of the total distribution system being studied is rather unique. While most distribution systems are highly connected, several miles of main pipeline (from AA to AF or AB) produce almost plug flow conditions with little or no mixing or dispersive effects. These long runs of main pipeline lead to relatively small regions where mixing occurs to varying degrees.

3 Field Test Protocol

A saturated NaCl solution was added to the treatment plant finished water. The NaCl solution was designed to produce a series of 270 mg/L brine pulses over a 24-hour period. The 270 mg/L pulse was selected to more than double the background conductivity (420 μS/cm) and increased the NaCl concentration to 75% of the maximum allowable NaCl concentration based on applicable federal and state standards. A more detailed chronological protocol follows.

3.1 Monitoring Locations

All field sampling locations were located at fire hydrants using standard hydrant adaptors. The locations were selected using the distribution system network model provided by the utility in order to: a) track the volume of water leaving the treatment plant (AA) along the paths to areas 1 and 2; b) track the splitting and recombination of water through area 2; and c) provide a focused input-output data set in area 1, with the outputs selected to have just one input (AC from AB) and a mixture of inputs (AD from AB and AE) based on model predictions.

Each sampling location was monitored with a continuous conductivity and

Figure 3: Estimated conductivity and fluoride concentration leaving the treatment plant during the 24-hour injection period.

temperature analyzer that was housed in a security box and that contained a grab sampling port. Sample locations AC, AD, and AE also were monitored with a continuous chlorine and pH analyzer to provide greater temporal resolution at these locations for the focused study in area 1. Grab samples were collected at all sampling locations and analyzed in the field for free chlorine and pH. Additional samples were collected for laboratory fluoride ion (conducted on site) and total trihalomethane (TTHM) (conducted off site) analysis. In addition to water quality sampling, additional hydraulic data were provided by a network of nine pressure loggers co-located with conductivity meters (using separate hydrants). Five additional pressure loggers were distributed throughout area 1 to provide additional hydraulic grade line resolution to help determine flow directions.

3.2 Brine Solution and Injection

Approximately 2200 gallons of saturated NaCl solution (350 g/L) were made using food-grade salt (Morton Culinox 999) and plant finished water. The brine solution was injected a short distance upstream from a venturi meter using parallel positive displacement pumps that were flow paced to produce a constant mixed salt concentration of 300 mg/L (injection + background NaCl).

Figure 3 presents the four designed conductivity concentration pulses during the 24-hour tracer injection period (time zero in Figure 3 is the start of injection). The project team was concerned about attenuation and loss of integrity of the conductivity pulses attributed to flow splitting and recombination in the looped areas of the distribution system. Therefore, four pulses of varying duration (1, 2, 3, and 2.5 hr) were used to evaluate the effect of pulse duration on signal strength.

Simultaneous with the first salt pulse, the plant fluoride feed was shut down for 24 hours (through the final salt pulse). This change in operation was intended to produce a fluoride ion concentration drop from approximately 0.9 mg/L to the background level of 0.2 mg/L. It was discovered on site that, in addition to the plant fluoride feed, fluoride was added at the groundwater wells. Consequently, the actual change in fluoride concentration was considerably less with an observed decrease from approximately 0.8 mg/L to 0.5 mg/L.

3.3 Field Water Quality Measurements

The study was designed to measure changes in water quality of the volume of water leaving the treatment plant during the 24-hour injection period. During the NaCl injection, and for an estimated 4-5 days residence time after the injection, the conductivity pulse arrival and other water quality measurements were recorded by continuous data-logging and grab samples.

At the same time the first salt pulse started, water quality sampling for pH, temperature, free chlorine, fluoride ion, and TTHM was initiated at the treatment plant and the two nearest distribution system water quality stations (AF and AG). Grab samples continued to be taken at locations where the conductivity tracer had arrived and at locations immediately downstream of the locations with known tracer arrival. Grab sampling ceased at a location as soon as the tracer had completely passed.

3.4 Bottle Tests

In addition to field water quality measurements, two bench-scale bottle tests were conducted using finished water leaving the plant during the 24-hour injection period. Bottles were filled without head space at the start of the first pulse and approximately 12 hours afterward (midway through the injection period). The two tests were conducted to measure any changes in water quality reaction kinetics due to plant operation changes throughout the day. All bottles were stored at distribution system temperature in the dark using a water bath that was continuously flushed with finished water. Bottles were periodically harvested and sampled to measure pH, temperature, and free chlorine concentration; samples for laboratory TTHM analysis were also collected.

4 Results and Discussion

4.1 Raw Data

The tracer study produced a tremendous amount of data that can be used to interpret the hydraulic and water quality dynamics within a distribution system. Figure 4 presents the type of hydraulic (pressure, conductivity, fluoride) and water quality (temperature, pH, chlorine, TTHM) data that were collected. These data were observed at location AD; however, as previously mentioned, this amount of data was not collected at every location. The conductivity and fluoride data provide insight into the hydraulic residence time (pulse arrival) and the travel

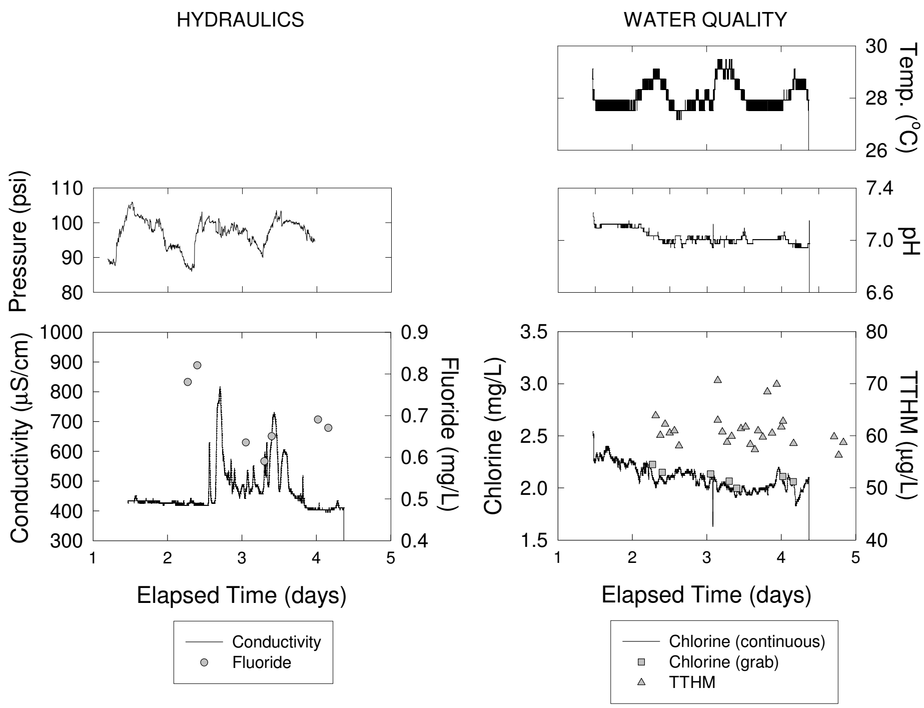


Figure 4: Raw data collected from AD. Continuous data include pressure, temperature, pH, conductivity, and chlorine. Grab samples include fluoride, chlorine, and total trihalomethanes.

path (more complex conductivity signal). The additional data provide information related to the observed variability at the specific monitoring locations, which can be important for locations that have difficulty maintaining adequate pressure or chlorine residuals or that have high disinfectant by-product concentrations.

The raw data can provide additional information by incorporating the spatial aspects of an actual distribution system. Figure 5 shows the conductivity and pressure (measured + modeled elevation) data collected along the flow path of AA-AG-AD. By simply evaluating the raw data in this context, one can observe information related to hydraulic residence time and the degree of mixing that occurs at various locations. For this flow path, there is little mixing from AA (treatment plant) to AG; however, at AD there is significant water mixing resulting from different travel paths. This type of information also illustrates the limitations of pressure data alone to represent information about residence time or flow path. This is not to suggest that pressure measurements are not useful, rather that the pressure and conductivity data should be viewed as complementary data that contribute to the overall picture.

One of the initial concerns of the research group was the magnitude of attenuation that might occur over such a long run of pipe. As shown in Figure 5, there is little attenuation in conductivity signal from AA to AG, indicating that diffusion processes have little effect on the signal strength. The conductivity signal at AD

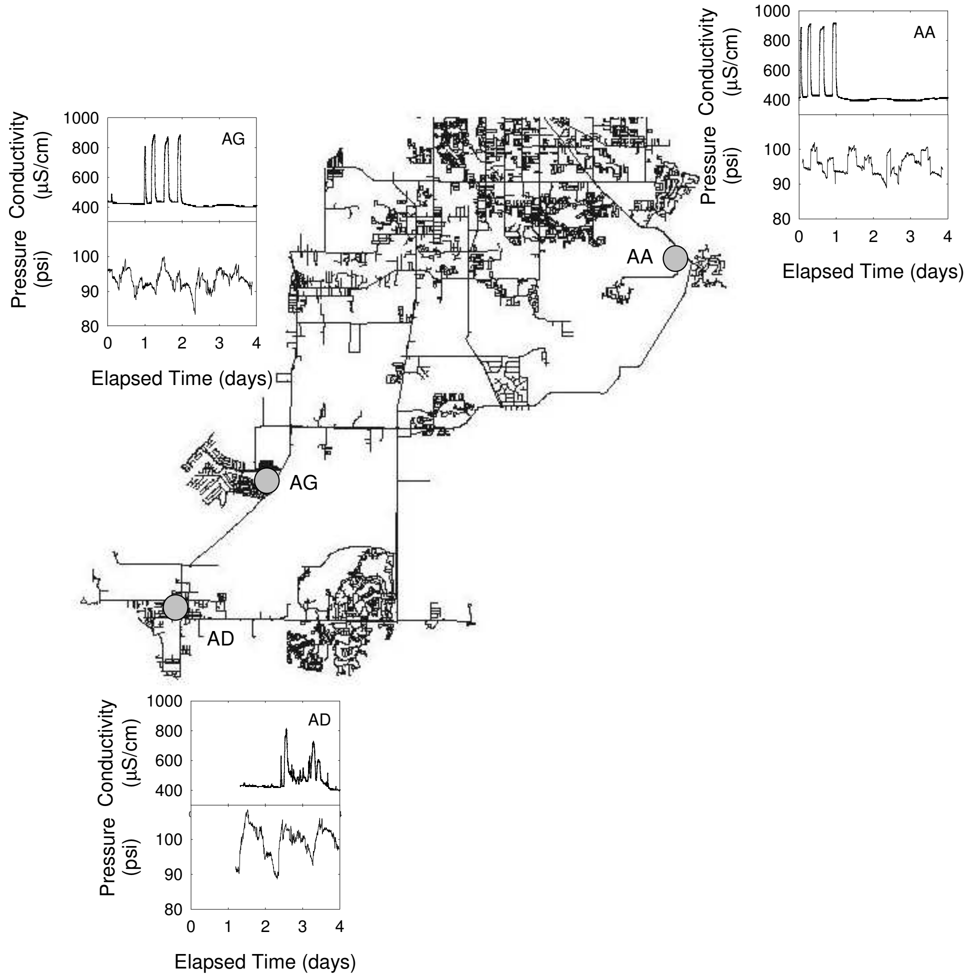


Figure 5: Conductivity and pressure data along flow path AA-AG-AD.

is quite different due to the greater degree of cross connections in area 1 and the confluence of water entering this area from AB and AE (from area 2).

4.2 Manual Calibration

The tracer and water quality data, coupled with system data, provide a powerful data set for calibrating a distribution system model. A simple, manual calibration approach was taken to update the distribution system model provided by the utility. The calibration approach proportionally adjusted the user demands to match the total system flow, and altered treatment plant pressure data to match SCADA data.

Figure 6 shows the model-predicted and observed conductivity pulses at AB and AD. The model-predicted results for AB agree well with the observed measurements. These results illustrate that the aggregate demands affect the flow through the main pipe line are well represented. The predicted and observed conductivity pulses at AD show significant differences in arrival time (∼6 hours); however, the predicted values do not capture the smaller peaks observed in the measured data. These differences are attributed to differences in the local demands that can affect the travel path of the water in the localized region as well as the fraction of water coming from the local sources (AB or AE). A more complex calibration technique would be required to reconcile the local demand differences that result in the observed conductivity pulses.

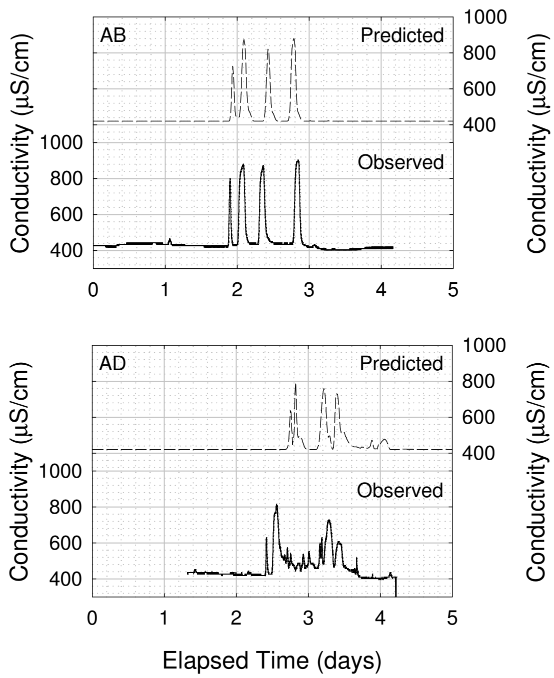


Figure 6: Observed and predicted conductivity time series at AB and AD using a minimally calibrated hydraulic model.

The ability of the distribution system model to represent water quality parameters is also of interest. Figure 7 presents the grab sample and bottle test TTHM data. The solid symbols represent the average grab sample TTHM concentration for each monitoring location using the associated average model-predicted residence time to plot the data. The bars represent the 0.05 and 0.95 quantiles (assuming the data is normally distributed) of the measured TTHM and estimated residence time values. The open symbols represent the TTHM measurements from the bottle tests. In general, the TTHM concentrations from the bottle tests agree with the concentrations that are based on the residence times predicted by the distribution system model.

The predicted conductivity and TTHM concentrations appear to be reasonable with respect to the observed results. Although the model is not perfect, the use of tracer study data can provide utilities with some degree of confidence in terms

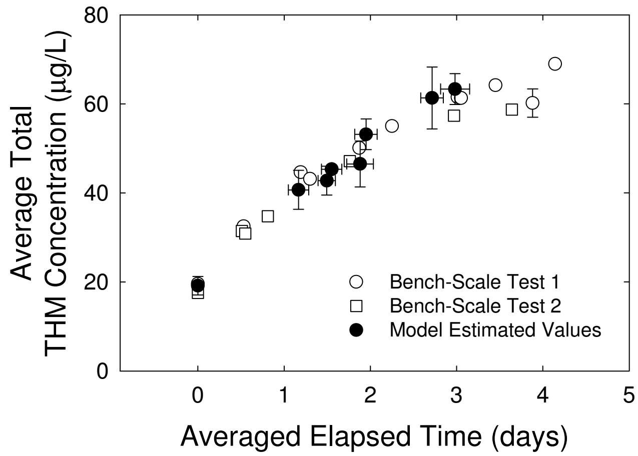


Figure 7: Experimental total trihalomethane concentrations from bench-scale tests and averaged THM concentrations based on network model residence times.

of how accurately the model represents reality (e.g., Maslia et al. (2000)). These data may also determine whether further refinements of the model are necessary.

4.3 Future Considerations

For this study, the model-predicted and observed data are reasonably matched; however, this portion of the distribution system is relatively simple with long runs of pipe that behave as plug flow reactors. Most systems are more hydraulically connected, like AD, and would be more difficult to represent with models developed by simple calibration techniques. The presented tracer study provides an approach for collecting information related to hydraulic residence times and travel paths that can be used within an automated calibration approach. Incorporating the field-scale hydraulic and water quality data into a network model requires more complex, automated calibration techniques (recent examples include Lansey et al. (2001) and Greco and Del Giudice (1999)).

Ultimately, a more accurate hydraulic representation provides opportunities for calibrating the water quality portion of a distribution system network model, which, in turn, allows disinfectant decay and by-product formation models to be evaluated at the field-scale level. In addition to extending the calibration to include water quality, the current tracer study encompasses multiple days that provide information for estimating user demand variability, not just uncertainty. The ability to incorporate residence time and path specific information within the calibration algorithm, and to generate statistical information related to user demands, which drive the hydraulics, provide a calibrated model that can be used in forward-modeling approaches to evaluate more complex issues related to consumer protection via the distribution system. Two such examples are, but not limited to, population exposure and health risks associated with carcinogenic disinfectant by-products and intentional intrusion events.

5 Summary

This study presents a network-wide dual-tracer field-scale study for collecting information related to hydraulic residence times and travel paths for a volume of water leaving the treatment plant. These tracer studies are coupled with water quality sampling to provide additional information related to disinfectant decay and by-product formation. The resulting data set is rich with distribution-specific information that can immediately provide some insight regarding distribution system dynamics. Minimal calibration of the available distribution system model to match SCADA data provides reasonable prediction of the observed results. These results provide a measure of confidence in the model predictions and illustrate promise that further calibration could improve the representation of more complex behavior. New techniques need to be developed to fully utilize the collected data set. These new techniques will provide opportunities to address more complex issues, such as population exposure to disinfection by-products or risk associated with intentional intrusion events.

6 Acknowledgements

The authors wish to gratefully acknowledge all of the enthusiasm, insight, and effort provided by the utility personnel and staff for making this tracer study a success.

References

Greco, M. and G. Del Giudice (1999). New approach to water distribution network calibration. *Journal of Hydraulic Engineering 125*(8), 849–854.

Lansey, K. E., W. El-Shorbagy, I. Ahmed, J. Araujo, and C. T. Haan (2001). Calibration assessment and data collection for water distribution networks. *Journal of Hydraulic Engineering 127*(4), 270–279.

Maslia, M. L., J. B. Sautner, M. M. Aral, J. J. Reyes, J. E. Abraham, and R. C. Williams (2000). Using water-distribution system modeling to assist epidemiologic investigations. *Journal of Water Resources Planning and Management 126*(4), 180–198.

Walski, T. M., D. V. Chase, and D. A. Savic (2001). *Water Distribution Modeling*, Chapter Calibrating Hydraulic Network Models, pp. 197–232. Waterbury, CT: Haestad Press.

Presented at ASCE/EWRI Congress 2005, May 15–19, 2005, Anchorage, Alaska, USA
Manuscript Number 8039

Use of Continuous Recording Water-Quality Monitoring Equipment for Conducting Water-Distribution System Tracer Tests: The Good, the Bad, and the Ugly

M. L. Maslia[1], J. B. Sautner[2], C. Valenzuela[3], W. M. Grayman[4], M. M. Aral[5], and J.W. Green, Jr.[6]

[1]Research Environmental Engineer, ATSDR, 1600 Clifton Road, Mailstop E-32, Atlanta, Georgia 30333; PH (404) 498-0415; FAX (404) 498-0069; email: mmaslia@cdc.gov
[2]Environmental Health Scientist, ATSDR, 1600 Clifton Road, Mailstop E-32, Atlanta, Georgia 30333; PH (404) 498-0496; FAX (404) 498-0420; email: jsautner@cdc.gov
[3]Post-Graduate Research Fellow, ORISE/ATSDR, 1600 Clifton Road, Mailstop E-32, Atlanta, Georgia 30333; PH (404) 498-0414; FAX (404) 498-0069; email: cvalenzuela@cdc.gov
[4]W. M. Grayman Consulting Engineer, 321 Ritchie Avenue, Cincinnati, Ohio 45215; PH (513) 761-1722; FAX (513) 761-5289; email: grayman@fuse.net
[5]Director, Multimedia Environmental Simulations Laboratory, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, Georgia 30332; PH (404) 894-2223; FAX (404) 894-5111; email: maral@ce.gatech.edu
[6]Post-Graduate Research Fellow, ORIS/ATSDR,1600 Clifton Road, Mailstop E-32, Atlanta, Georgia 30333; PH (404) 498-0413; FAX (404) 498-0069; email: jwgreen@cdc.gov

Abstract

An emerging and innovative technology that is a possible alternative to manual sampling is the use of continuous recording water-quality monitoring equipment (CR-WQME) for collecting multiple ion-specific tracer data. Advantages of using CR-WQME include the ability to record continuously water-quality events (including unplanned events) during a tracer test at small time intervals of 15 minutes or less. This recording provides real-time data when using hand-held logger equipment to query the CR-WQME at each sampling location. Also, the labor needed to conduct the test is reduced. Disadvantages could include the cost of multiple ion-specific sensors and units for large or complex systems, the effort required to calibrate the equipment by setting up a test-site water-quality laboratory, and the reliability of the equipment for long-term monitoring events. In this paper the authors assess the use of CR-WQME in conducting a tracer test at a military installation in North Carolina. Using results obtained from the tracer test, the authors found that CR-WQME is an emerging and innovative technology that still requires refinement and the use of some grab samples to provide quality-assurance and quality-control procedures during the tracer test. However, CR-WQME is an excellent option when designing and conducting multiple parameter tracer tests for water-distribution system model calibration activities.

Introduction

The use of water-distribution system models for analyses and assessments of contamination events—including historical, current, and future events—requires a calibrated water-quality model. Conducting tracer tests by injecting a conservative compound into the distribution system (e.g., calcium chloride) or shutting off an additive compound (e.g., sodium fluoride) and collecting concentration data at selected sampling locations can provide the information required to calibrate a water-quality model. Spatially large water distribution-system networks can be complex and could have unknown or variable operational characteristics. Collecting water samples ("grab samples") to capture a unique water-quality event, such as the passing of a tracer's peak concentration at a sampling location, can be cost and labor intensive. Also, the tracer front and peak can be missed during the test if a sudden or unplanned change in operational characteristics occurs.

Description of Tracer Study

An ongoing epidemiologic study at U.S. Marine Corps Base, Camp Lejeune, North Carolina, requires the use of calibrated water-distribution system models to assess present-day conditions and historical exposures. To obtain calibration data, a tracer test was conducted. The source of sodium fluoride (NaF) at the Holcomb Boulevard water treatment plant (WTP) was shut off at 1600 hours on 22 September 2004. The fluoride concentration in the distribution system was recorded at nine monitoring locations (Figure 1) and allowed to diminish to background levels (~ 0.2 mg/L) through dilution by non-fluoridated water. At 1200 hours on 29 September 2004 the NaF source at the WTP was turned back on and fluoride concentrations in the water-distribution system were recorded until the conclusion of the test on 12 October 2004.

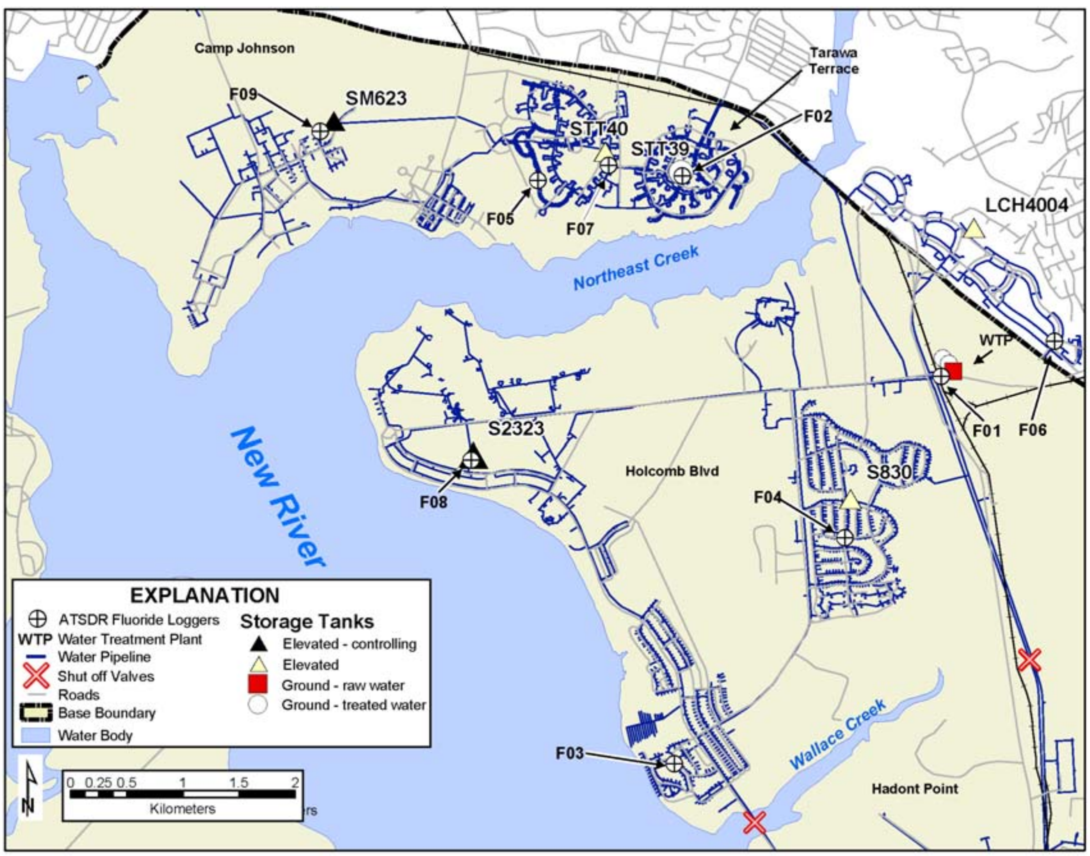


Figure 1. Holcomb Boulevard water treatment plant service area and monitoring locations (F01–F09).

Continuous recording water-quality monitoring equipment (CR-WQME) was placed on the distribution side of the WTP (logger F01, "source location"; Figure 1), at the Tarawa Terrace treated water reservoir (ground storage, logger F02), at controlling elevated storage tanks (loggers F08 and F09), and at 5 hydrants located in family housing areas in the Holcomb Boulevard WTP service area (loggers F03, F04, F05, F06, and F07). Fluoride concentrations were recorded using 15-minute intervals. In addition to the CR-WQME, nine rounds of water samples were collected at each monitoring location for quality assurance and quality control (QA/QC) purposes. For analyses, the samples were split so that 25 mL of the grab sample water were analyzed at the Holcomb Boulevard WTP water-quality lab, and the remaining 225 mL of water were analyzed by the Federal Occupational Health laboratory in Cincinnati, Ohio.

Continuous Recording Water-Quality Monitoring Equipment (CR-WQME)

To record continuously the fluoride concentration, the HORIBA W-23XD dual probe, multi-parameter, water-quality monitoring system was used. This system consisted of a dual–probe ion detector and a flow cell that fits the double probe W-23XD (Figure 2). The probe and flow cell are housed in a plastic protective container—a standard 5-gal (18.9 L) water jug (Figure 3). Distribution-system water passes through the flow cell by attaching a Dixon A7893 hydrant adapter kit to the sampling location hydrant. The adapter kit is configured with a 1/4 NPT brass "T" and two 1/4-in. (6.4 mm) ball valves on each side of the brass "T" (Figure 3). One valve was used to control flow into the flow cell, and the other valve was used to turn water on and off when obtaining grab samples from the hydrant. The complete configuration, consisting of the HORIBA W-23XD probe, flow cell, and 5-gal (18.9 L) plastic protective water jug, was secured to the hydrant by means of a chain and lock. During the test, a continuous discharge of water came from the flow cell and the plastic protective container (approximately 1–2 gpm [3.8–7.6 L/m]). To monitor and download fluoride data, the HORIBA water-quality control unit was attached to the sensor probe using a cable (Figure 4). With the configuration described above, the data logger continued to record data while real-time data values were observed using the HORIBA water-quality control unit.

The Good

The good aspect of using the CR-WQME was that it required only three people to conduct a field test that lasted 12 days. To start the test, one person calibrated the loggers while a second person deployed the loggers at the selected monitoring locations (Figure 1). A third person was used to collect grab samples for QA/QC. Once all of the loggers were installed at the selected monitoring locations and one round of grab samples was taken, two of the field test personnel returned to their office—located about 500 miles (805 km) away—while only one person was required to remain on site. The field person collected one round of QA/QC samples in the morning and one round in the afternoon. Fluoride concentrations from the grab samples from the WTP water-quality lab were reported to the two test personnel who were stationed back in the office. The field person also used the water-quality control unit (Figure 4) to obtain real-time concentration values and to assess the status of the CR-WQME. When field data (logger readings and QA/QC grab samples) indicated that fluoride concentrations had decayed to background levels, the one on-site test person communicated with WTP personnel to turn the fluoride back on. Thus, the CR-WQME provided a means by which a long-duration field test could be conducted with just three people, and only one person in the field, thereby reducing the labor costs associated with the test.

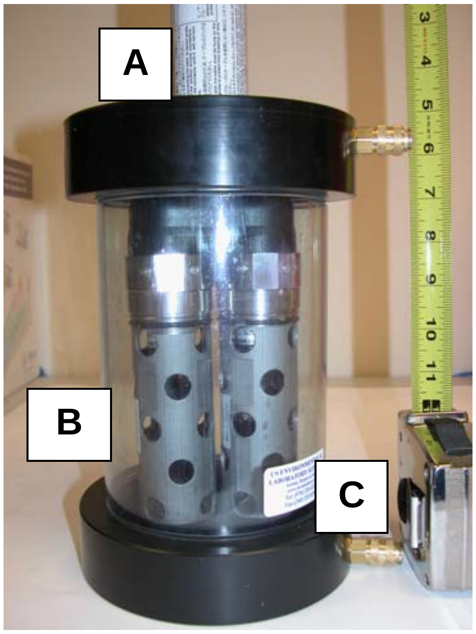


Figure 2. Photograph showing (A) HORIBA W-23XD dual probe ion detector, and (B) flow cell, and (C) Rectus 21KANNMPX, ¼ NPT brass connectors.

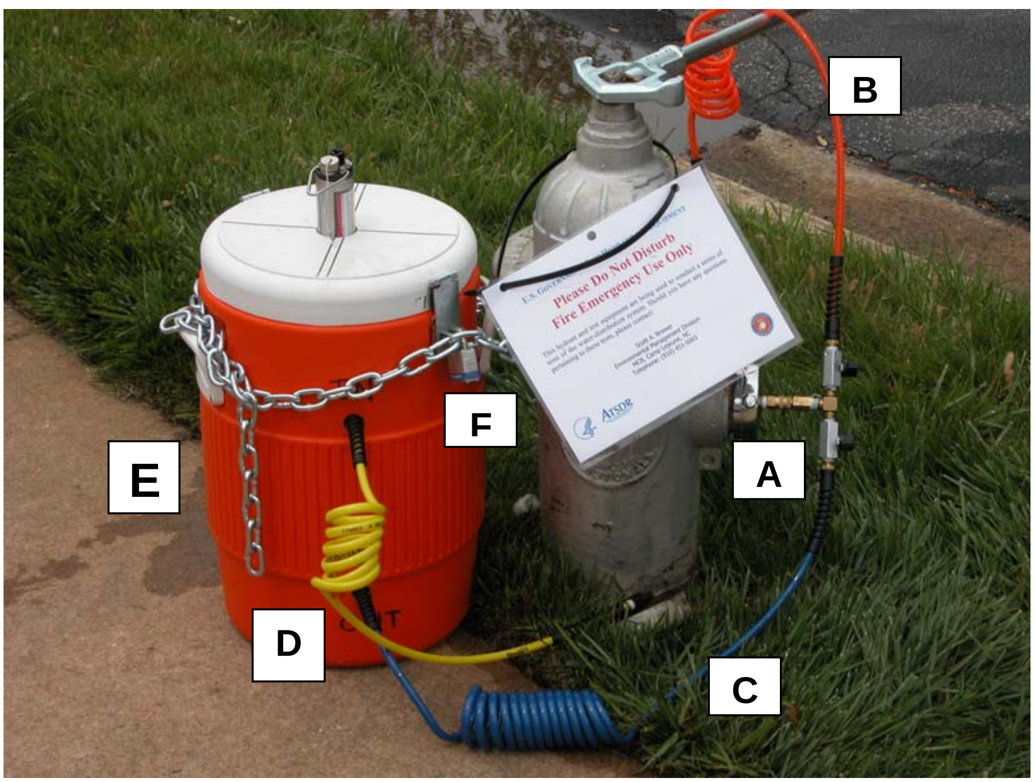
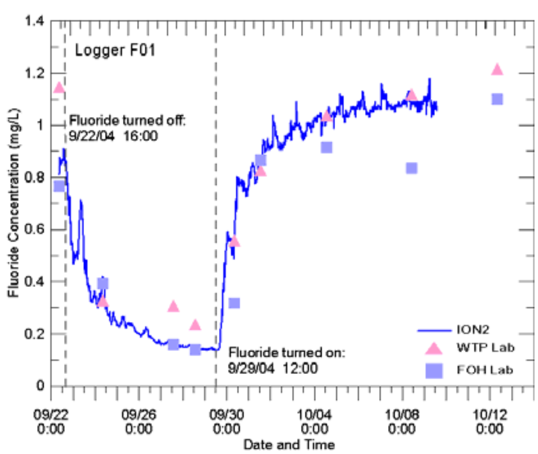


Figure 3. Photograph showing: (A) hydrant adapter kit configuration, (B) Rectus PSCH0605-16, orange hose for collecting grab sample, (C) Rectus PSCH0610-3, blue hose for supplying flow cell with hydrant water, (D) Rectus PSCH0605-5, yellow hose for discharging water from flow cell, (E) 5-gal protective plastic water jug housing the HORIBA W-23XD dual-probe ion detector and flow cell, and (F) chain and lock for securing equipment to hydrant.

Figure 4. Photograph showing HORIBA W-23XD water-quality control unit and cable attached to dual probe ion detector housed in a protective plastic 5-gal water jug

When compared with grab sample data at all but one of the monitoring locations, good results were obtained using the CR-WQME. Because of space limitations, only two graphs of data are shown (Figure 5). Logger F01 was located on the main transmission line going from the Holcomb Boulevard WTP to the distribution system (Figure 1). This logger represents the source conditions for fluoride in the Holcomb Boulevard WTP service area. The data collected by the CR-WQME (solid line), and the QA/QC grab samples analyzed at the WTP water-quality laboratory, and the FOH water-quality laboratory strongly agree. Logger F08 was stationed at a controlling elevated storage tank. Therefore, the water level in the tank fluctuates based on demand. The graph for logger F08 clearly shows the draw and fill cycle of the tank. When the fluoride concentration in the distribution system reached a near–background level of about 0.2 mg/L on September 28, the elevated storage tank still contained water with a fluoride concentration of about 0.8 mg/L. Additionally, the correlation between the continuously recorded data and the QA/QC grab sample data is also very strong.



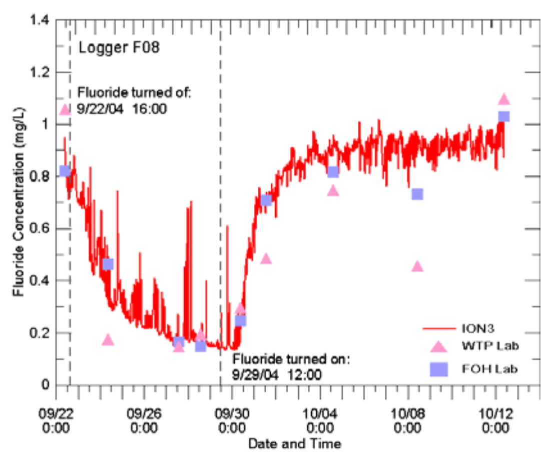


Figure 5. Graphs of fluoride concentration for loggers F01 and F08

The Bad

When using CR-WQME, two situations still require improvement and technical enhancement: the equipment-logger calibration procedures and the occurrence of calibration “drift” during the test. The authors’ experience with using the specific equipment described here is that approximately 1-hour per ion-specific parameter is required to calibrate the equipment. For one parameter this might not be an issue. However, if in addition to fluoride concentration, one wanted to gather continuous data for pH, temperature, conductivity, and chloride, it could have taken up to 5 hours to calibrate each logger. One technique the authors devised to shorten the calibration time was to calibrate each parameter for all loggers at the same time by placing all loggers in a water bath (Figure 6).

Another issue that must be confronted when using CR-WQME is the issue of logger calibration “drift.” Because the CR-WQME is connected to a hydrant, it is not possible to re-calibrate the loggers while the test progresses. If calibration drift becomes significant, the data collected by the CR-WQME may not be useful. To determine when logger calibration drift becomes significant and continuously recorded data becomes unreliable, at the present time, grab samples must still be obtained while continuously recording data. Thus, at the present time, requiring the collection of grab samples increases the cost of the test (in terms of labor and water-quality laboratory analyses) and limits the long-term usefulness of CR-WQME. Results from data logger F04 (Figure 7) clearly show logger calibration drift occurring after about 10 days. Continuously recorded data on the graph show a marked departure from the QA/QC grab sample data.

Figure 6. Photograph showing use of a water bath for calibrating multiple loggers.

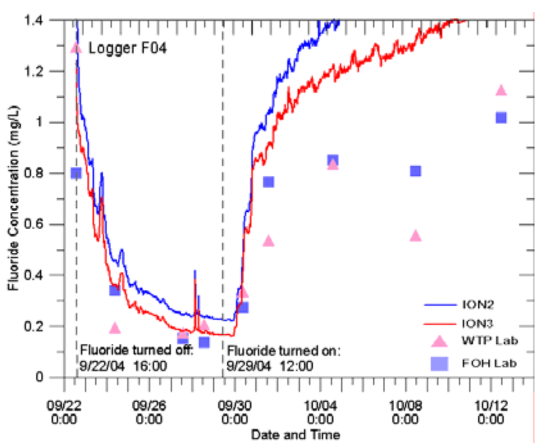


Figure 7. Graph of fluoride concentration for logger F04 showing departure of continuously recorded data (solid lines) from grab sample data (symbols) after about 10 days.

The Ugly

The most pressing and time-consuming issue pertaining to the use of CR-WQME that the authors encountered was reliability of the ion-specific sensors used to measure concentrations in the distribution system water. To test the reliability of the fluoride ion sensors, several of the loggers were equipped with two sensors (e.g., logger F04 in Figure 7). As shown in Figure 7, both sensors ("ION2" and "ION3") produced consistent and reliable results. In other loggers, however, such good results were not observed. In fact, some of the sensors were completely unreliable. Examples of this condition are shown in Figure 8 for loggers F05 and F07. Logger F05 was equipped with two fluoride sensors; the sensor identified as "ION3" shows a marked and complete departure from the sensor identified as "ION2" and from grab sample data. Results for Logger F07 show that continuously recorded data generally followed the trend of declining fluoride, as did the grab sample data; nevertheless, recorded data from this sensor also appear to be unreliable. Thus, without the use of more than one sensor or grab sample data for QA/QC, concentration data at these monitoring locations would not be reliable for analysis of the water-distribution system or for model calibration purposes.

Because the use of CR-WQME is still an emerging technology, the number of vendors supplying the equipment might be limited. As such, the cost of the equipment described here, is still high. Thus, if one conducts a test in which 10–20 locations might be required to monitor and characterize a water-distribution system properly, the purchase of that many CR-WQME could be cost prohibitive at this point in time.

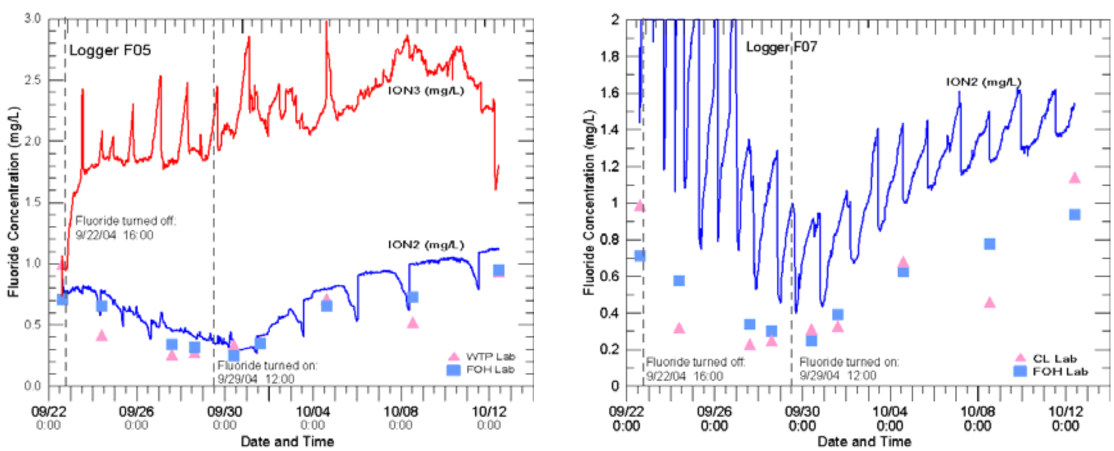


Figure 8. Graphs of fluoride concentration for loggers F05 and F07 showing marked departure of continuously recorded data (solid lines) from grab sample data (symbols).

Summary

An emerging and innovative technology that is a possible alternative to manual sampling is the use of CR-WQME at sampling locations for collecting multiple ion-specific tracer data. Advantages of using CR-WQME include the ability to record continuously water-quality events during a tracer test at short time intervals of 15 minutes and a significant reduction in labor needed to conduct the test. Disadvantages could include the cost of multiple ion-specific sensors and units for large or complex systems, the effort required to calibrate the equipment by setting up a test-site water-quality laboratory, and the reliability of the equipment for long-term monitoring events. Based on results obtained from the tracer test described herein, the use of CR-WQME is an emerging and innovative technology that still requires refinement and the use of some grab samples to properly provide QA/QC during the tracer test. However, using CR-WQME should be considered as a monitoring option when designing and conducting multiple parameter tracer tests for water-distribution system model calibration activities.

Acknowledgment

The authors would like to acknowledge the staff of the Marine Corps Base, Camp Lejeune Environmental Management Division and water utility department for their assistance with data requests and logistics while conducting the field test described here.

Disclaimer

Use of trade names and commercial sources are for identification only and do not imply endorsement by the Agency for Toxic Substances and Disease Registry, the U.S. Department of Health and Human Services, the Georgia Institute of Technology, or the Oak Ridge Institute for Science and Education.

Use of Fireflow Tests in the Calibration of a Water Distribution System Model

Tucker Southern[1], Walter Grayman[2], Leon Kjellgren[3], and Bryan Seppie[4]

[1]Nelson Engineering, P.O. Box 1599, Jackson, WY 83001, PH: (307) 733-2087, email tsouthern@nelsonengineering.net
[2]W.M. Grayman Consulting Engineer, 321 Ritchie Ave., Cincinnati, OH, 45215, PH: (513) 761-1722, email: grayman@fuse.net
[3]Nelson Engineering, P.O. Box 1599, Jackson, WY 83001, PH: (307) 733-2087, email: lkjellgren@nelsonengineering.net
[4]Joint Powers Water Board, P.O. Box 1299, Green River, WY, 82935, PH: (307) 875-4317, email: bseppie@jpwb.org

Abstract

Fireflow (or hydrant flow) tests are frequently used in the calibration process for water distribution system hydraulic models. In such a test, hydrants are opened and flowed at a high flow rate in order to cause high flows and large head losses in the pipes leading to the hydrant. When a system is stressed in this manner, the behavior of the distribution system is more sensitive to factors such as pipe roughness, closed valves, demands, and PRV settings. By comparing field measurements of flow and pressure to model results for the same situation, model parameters can be adjusted (calibrated) to better represent the system behavior. As part of a master plan and hydraulic model development for The Joint Powers Water Board (JPWB) covering the Green River/Rock Springs/Sweetwater County area in southwestern Wyoming, fireflow tests and other field techniques were used to calibrate the detailed model. Approximately 30 fireflow tests were conducted as part of the calibration process. Additional operational information on the water system (water usage, tank levels, pump operation) was collected from the SCADA system. Subsequently, the hydraulic model was applied under the test conditions and parameter adjustments made to bring the model into better agreement with the field results. The method was found to be most effective in identifying unexpected closed valves, adjusting PRV settings and behavior, and to a lesser degree in adjusting roughness coefficients. The logistics, costs, practical experience and the calibration results are discussed herein.

Keywords
Hydraulic modeling, flow tests, hydrant tests, master planning, model calibration

BACKGROUND

The Green River, Rock Springs, Sweetwater County-Joint Powers Water Board (JPWB) is a regionalized water system in Southwest Wyoming. The JPWB was formed in 1987 for the purpose of providing a joint water supply, treatment and distribution system(s) to the citizens of Green River, Rock Springs and certain unincorporated areas of Sweetwater County, Wyoming.

The JPWB's system is made up of three primary elements: the Water Treatment Facility located in Green River, the Green River Distribution System, and the Rock Springs Transmission and Distribution System. The Green River Water Treatment Facility is a 32 mgd, conventional surface water treatment plant utilizing the Green River as its source water. The Green River Distribution System is comprised of approximately 65 miles of mains, three pump stations, four reservoirs, with fourteen pressure zones. The Rock Springs System is comprised of approximately 25 miles of transmission line, 115 miles of distribution mains, five pump stations, seven reservoirs, with twelve pressure zones. In addition to the municipal customers, the JPWB serves three service districts and one large commercial customer. The topography served by the JPWB's system(s) varies from an elevation of 6080' above sea level to 6860'. The total population served by the JPWB system is just over 35,000 with an annual consumption of 3 ¼ billion gallons. Figure 1 shows a map of the Green River-Rock Springs area illustrating the major components in the water system.

In 2005, JPWB commenced a Master Plan Project (MPP) to develop a water master plan to update and replace the former plan developed in 1990. The existing steady state, hydraulic models were also in need of updating. A hydraulic model is an invaluable tool for master planning and thus updating the hydraulic model was incorporated into the ongoing MPP. To develop the master plan and the associated hydraulic model, the JPWB applied for and received approval and funding from the State of Wyoming Water Development Commission (WWDC). The WWDC is a state funded water and related resource planning program. The WWDC program was developed to facilitate, plan, design and fund water resource planning projects throughout the state. The JPWB's scope of work, RFP process and consultant selection were all administered by the WWDC. Ultimately, the MPP was awarded to Nelson Engineering of Jackson, Wyoming. The MPP's scope was broken up into the following specific tasks: (1) Future Water Supply Needs, (2) Hydraulic Modeling, (3) Transmission and Distribution Analysis, (4) Water Quality Modeling, (5) Proposed Improvements, and (6) Cost Analysis. The Hydraulic Modeling task was completed first to provide the analytical tools necessary for the remaining MPP's tasks. This paper focuses on the Hydraulic Modeling task.

The Hydraulic Modeling task of the MPP was broken into specific sub-tasks: (1) update the network representation in the current model; (2) assign updated nodal demands; (3) develop the EPS model; and (4) calibration and validation of the model. The JPWB desired a well calibrated EPS model, not only for routine hydraulic analysis, but also for water quality analyses such as chlorine decay analysis and water age calculations and for analysis of disinfection by-products.

To develop a hydraulic model to the level required of these intended uses requires a high level of accuracy and confidence. This resulted in a process of utilizing the best available data and techniques to ensure that the end product is of the highest quality available. The first subtask was to update the existing model data; GIS techniques were utilized to obtain accurate model representation and elevations. Next, geo-coded meter data was used to locate and aggregate all meter readings to the nearest node. To develop appropriate demand scenarios, archived SCADA data on the master billing meters was compiled and used in conjunction with the city's individual meter data. This data was analyzed to determine water loss, average daily demands, maximum/minimum daily demands, and the appropriate diurnal demand patterns. The next critical step was to calibrate and validate the hydraulic models. An extensive field program was performed as part of the study to calibrate and validate the model. The primary components of the field program included: (1) pump tests; (2) hydrant and PRV flow tests; and (3) tracer tests. Details of the tracer tests are described in Seppie et al (2006). The present paper describes the procedures and results of the hydrant flow tests.

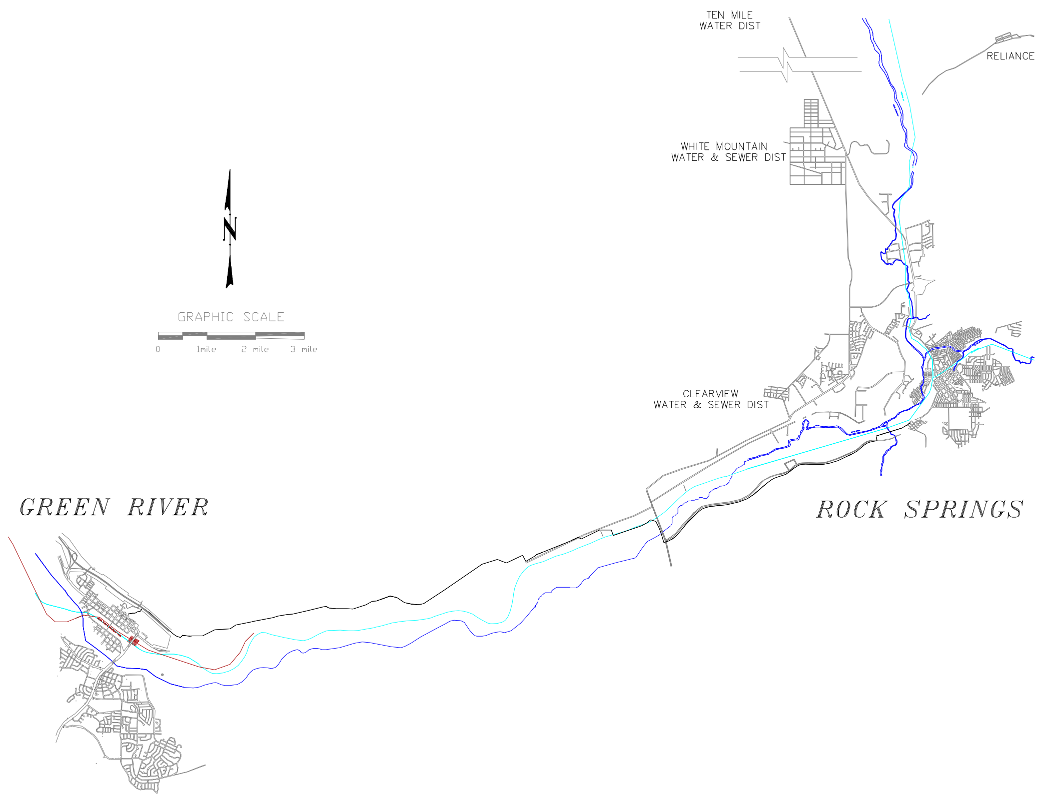


Figure 1. Green River-Rock Springs water systems

HYDRANT FLOW TESTS

Methodology

Hydrant flow tests were performed within the distribution system in order to assist in determining appropriate pipe roughness values (c-factors), to identify the presence of closed valves and to further understand the performance of the system. An advanced technique was used that extended the traditional hydrant flow test methodology to several hydrants and utilized continuous digital pressure gages (Grayman et al, 2006). For each test, two hydrants were sequentially and concurrently flowed in order to depress the hydraulic grade line. For the duration of the test; pressures were simultaneously measured at up to five hydrants using digital logging pressure gages. This procedure resulted in five flow scenarios for each test (static conditions representing normal water usage, flow hydrant # 1 open, flow hydrant # 1 and # 2 open, flow hydrant #2 open, both hydrants closed or a retest of static conditions) thus increasing the amount of field data available for calibration. It is recommended that a pressure drop of at least 10 psi is achieved (between static and both hydrants flowing) in order to ensure that the system is adequately stressed. Additional operational information on the water system (water usage, tank levels, and pump operation) was collected from the SCADA system. Subsequently, the hydraulic model was applied under the test conditions and parameter adjustments made to bring the model into better agreement with the field results.

Zones that are fed by one or more Pressure Reducing Valve(s) (PVR) require an extra step before the actual hydrant test is performed. When zones are fed by multiple PRVs, additional uncertainty is introduced into the calibration process. Each PRV feeding the zone must be tested in order to ensure it is operating correctly and to determine the exact pressure setting. This process is necessary to determine the feed into the zone as well as accurately define the headloss associated with each PRV. The first step is to isolate each PRV with a downstream hydrant to force a specific flow through the PRV. The flow from the hydrant is measured using the Pitot meter shown in Figure 2. The discharge and inlet pressures of the PRV are recorded for each flow level (or stage) throughout the PRV's operating range. The second step is to evaluate the flow and pressure results based on tables and curves provided by the PRV manufacture. From this analysis it can be determined if the PRV is operating correctly and the appropriate headloss coefficient to use in the calibration. This process must be repeated for each PRV in the system.

In the case of multiple PRVs feeding a zone, the relative location of these PRV's must be considered when designing a hydrant flow test within that zone. If the PRV's are located relatively close together, and thus the exact operation of these valves is uncertain, it is recommended that all but the dominant valve be turned off. This will guarantee the source of the flow for the hydrant test is known and from the previously conducted PRV test, the headloss will also be accurate.

A photograph of one of the hydrants being flowed is shown in Figure 2.

Figure 2. Flow measurement during a hydrant flow test

Example Hydrant Flow Test

In order to demonstrate the methodology, logistics and results of this technique, one of the flow tests is presented in detail. This test was performed in the northern part of the Green River water system. This part of the water system is largely isolated from the remainder of the Green River system and is fed from a tank. Figure 3 shows the piping in this part of the system and the placement of the two flow hydrants (Q1,Q2) and the four hydrants equipped with digital pressure gages (P1-P4).

This test was the first of the hydrant flow tests that was done as part of the model development and calibration process. A three-person crew performed this test on the afternoon of September 29, 2005. The continuous pressure measurements for the four hydrants are displayed in graphical form in Figure 4. The results of the test are summarized in Table 1.

Table 1. Measured and Modeled Hydrant Flow Test Results

North Side Zone			DATE: 9/29/05						TIME: 16:40 to 17:00					
Scenario	**Q1 (gpm)**	**Q2 (gpm)**	**Pressure Hydrant #1**			**Pressure Hydrant #2**			**Pressure Hydrant #3**			**Pressure Hydrant #4**		
			Field data	**Orig. Model**	**Adj. Model**	**Field data**	**Orig. Model**	**Adj. Model**	**Field data**	**Orig. Model**	**Adj. Model**	**Field data**	**Orig. Model**	**Adj. Model**
Static	---	---	92	90	90	95.5	95	95	106	105	105	89	91	88
1	1051	---	72.5	84	72	76	87	77	87	99	88	78	86	79
2	767	900	46.5	70	49	52.5	77	54	64	86	64	65.5	77	67
3	---	1118	71.5	83	69	75.5	88	74	86	97	84	77.5	86	78
Static	---	---	92	90	90	95.5	95	95	106	105	105	88	91	88

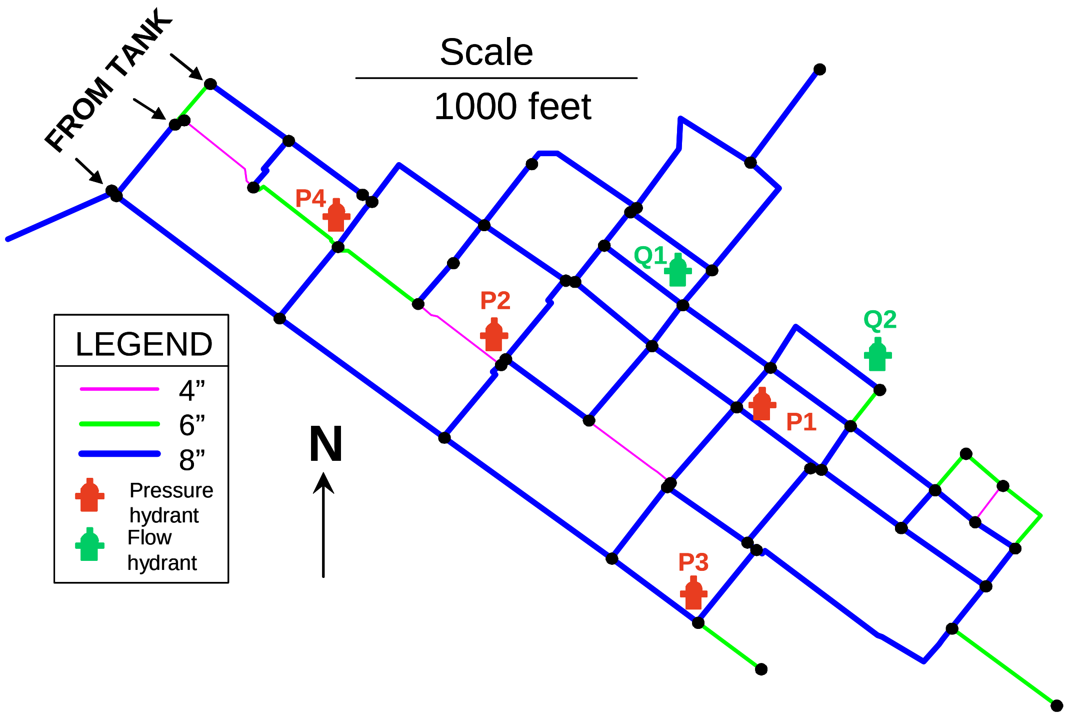


Figure 3. North Green River distribution system showing hydrants used in flow test

Following the test and the development of the hydraulic model, the model was used to simulate the conditions observed during the test. Measured and modeled pressures are shown in Table 1. During the static part of the test, the measured and modeled pressures were all within plus or minus 3 psi and the average difference was 1.5 psi. Calibration was assumed to be achieved when the maximum difference in pressures is less than 3.5 psi. However, during the part of the test when the hydrants were being flowed, there were very significant differences between measured and modeled pressures. In all cases while the hydrants were being flowed, the modeled pressures exceeded the measured pressures by significant amounts. The difference ranged from 8.0 to 24.5 psi and the average deviation was 14.0 psi.

This pattern of similar values for pressure during static conditions and significantly higher modeled pressures when the hydrants were being flowed suggested the presence of closed valves in the system. Though this type of difference could also be caused by inaccurate roughness coefficients in the model (i.e., Hazen Williams coefficients that were much too high in the model), the large magnitude of the difference suggested closed valves. Based on these observations, the City undertook an extensive valve inventory program. A total of ten closed valves were found in this and adjacent parts of the water distribution system. When these valves were closed in the model, the agreement between measured and modeled pressure was greatly improved. The pressure difference then ranged between 0 and 2.5 psi with the average of absolute differences of only 1.3 psi.

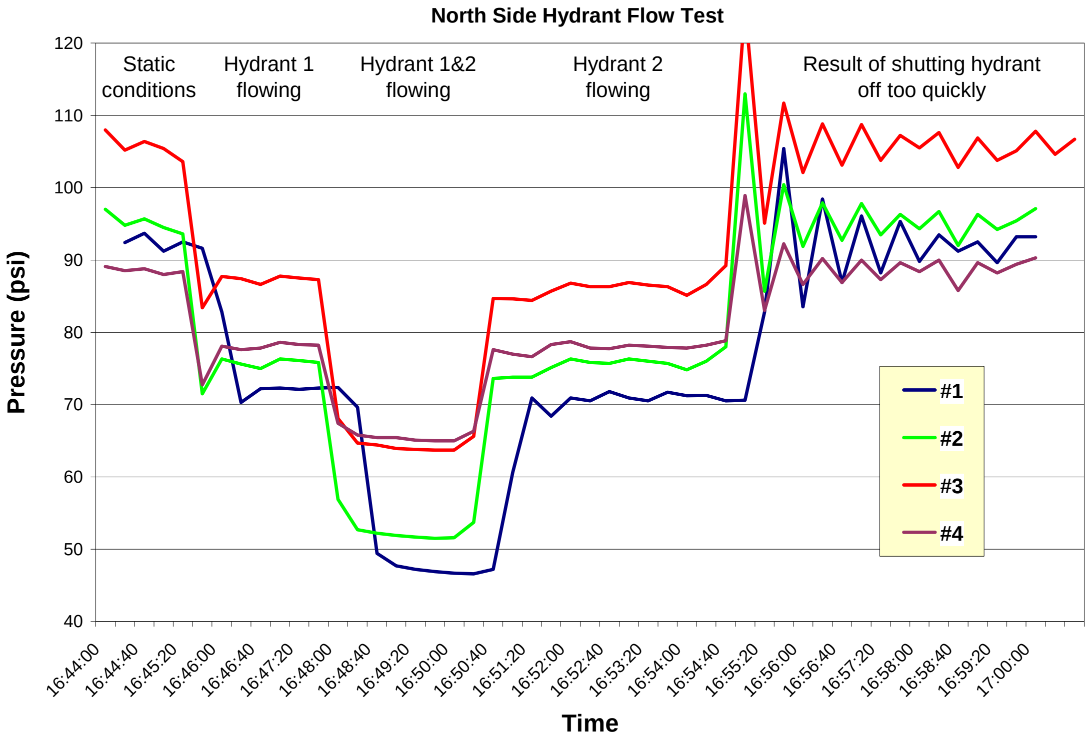


Figure 4. Pressure variations during the hydrant flow test at four hydrants

Summary of Results

Approximately 30 flow tests were run covering the entire study area. Each hydrant flow test was conducted by a 3-person crew and took approximately 1.5 hours to set up and perform. The tests resulted in the identification of many closed valves, some roughness coefficients that differed from the initially assumed literature values, and in several cases, incorrect PRV settings that significantly affected the model results. The total labor requirements for the hydrant flow tests including planning and data analysis was approximately 250 person-hours. The field equipment (five continuous pressure gages and two diffuser/pitot gages) cost approximately $4000.

In most cases, the differences between the measured and modeled pressures at the hydrants were greater than would be acceptable for a calibrated model. At that point, the modeler must act as a detective to deduce what changes may be needed in order to bring the model into calibration. Initially a trial and error process was used to identify likely needed changes. After the evaluation of several flow tests was completed, trends evolved and the level of effort to identify needed changes decreased.

SUMMARY AND CONCLUSIONS

The Green River, Rock Springs, Sweetwater County - Joint Powers Water Board (JPWB) is in the process of updating their Master Plan. An important component of this work is a significant upgrade of their hydraulic model. The resulting detailed model is intended to serve a variety of purposes including water quality analysis. In order to support these diverse applications, an extensive field investigation and calibration/validation program is being performed. The general methodologies that are being applied in the development and calibration of the hydraulic model are described in the literature. However, the field studies including the tracer tests and the multi-hydrant automated flow tests are still cutting edge technologies that are far from routine procedures. Since neither of these technologies had been applied previously in this part of the country nor had the primary consulting firm used these methodologies in the past, there was both a significant learning curve associated with their application and adaptations that were needed to apply the technologies to the local case. To overcome the lack of previous experience, a methodical engineering approach was used including detailed planning, testing and implementation procedures. This resulted in a highly successful field study that yielded no major problems. However, it also resulted in a larger degree of effort (labor hours) than had been anticipated. This can be minimized through proper implementation of a valve and PRV testing and maintenance program prior to performing the hydrant test.

The authors credit the Wyoming Water Development Commission for their foresight in funding these advanced techniques used in calibrating and validating the JPWB's system model. The experience and knowledge gained during the course of this study will benefit not only the JPWB and the project's consultants, but also future water system studies throughout the state.

REFERENCES

Grayman, W.M., Maslia, M.L., and Sautner, J.B. (2006). "Calibrating distribution system models with fire-flow test". *Opflow*, AWWA, 32(4), 10-12.

Seppie, B., Kjellgren, L., Grayman, W.M. (2006). "Detailed Distribution System Modeling For A Regional Water Master Plan". *Water Distribution System Analysis Symposium, University of Cincinnati..*

Using Continuous Monitors for Conducting Tracer Studies in Water Distribution Systems

Srinivas Panguluri[1], Radha Krishnan[2], Lucille Garner[3], Craig Patterson[4], Yeongho Lee[5], David Hartman[6], Walter Grayman[7], Robert Clark[8], Haishan Piao[9]

1. Shaw Environmental, Inc., 5050 Section Avenue, Cincinnati, OH 45212; PH (513) 782-4893; FAX (513) 782-4807; email: Srinivas.Panguluri@shawgrp.com
2. Shaw Environmental, Inc., 5050 Section Avenue, Cincinnati, OH 45212; PH (513) 782-4730; FAX (513) 782-4807; email: Radha.Krishnan@shawgrp.com
3. U. S. Environmental Protection Agency, ORD/NRMRL/WSWRD, 26 W. Martin Luther King Drive, Cincinnati, OH 45268; PH (513) 569-7417; FAX (513) 569-7185; email: Garner.Lucille@epa.gov
4. U. S. Environmental Protection Agency, ORD/NRMRL/WSWRD, 26 W. Martin Luther King Drive, Cincinnati, OH 45268; PH (513) 487-2805; FAX (513) 569-7707; email: Patterson.Craig@epa.gov
5. Greater Cincinnati Water Works, 5651 Kellogg Avenue, Cincinnati, OH 45228; PH (513) 624-5618; FAX (513) 624-5670; email: Yeongho.Lee@gcww.cincinnati-oh.gov
6. Greater Cincinnati Water Works, 5651 Kellogg Avenue, Cincinnati, OH 45228; PH (513) 624-5659; FAX (513) 624-5670; email: David.Hartman@gcww.cincinnati-oh.gov
7. WM Grayman Consulting Engineer, 321 Ritchie Avenue, Cincinnati, OH 45215; PH (513) 761-1722; FAX (513) 761-5289; email: grayman@fuse.net
8. RM Clark Consulting Engineer; 9627 Lansford Drive, Cincinnati, OH 45242; PH (513) 891-1641; FAX (513) 891-2753; email: rmclark@fuse.net
9. SBR Technologies, Inc.; c/o USEPA T&E Facility, 1600 Gest Street, Cincinnati, OH 45202; PH (513) 487-2813; FAX (513) 569-7707; email: Haishan.Piao@epa.gov

Abstract
The use of online monitors for conducting a distribution system tracer study is proving to be a helpful tool to accurately understand the flow dynamics in a distribution system. In a series of field tests sponsored by the U. S. Environmental Protection Agency (EPA) and the Greater Cincinnati Water Works (GCWW) in 2002-2003, a food-grade calcium chloride tracer was introduced into a water system network and the movement of the chemical was traced using strategically placed automated onlineconductivity meters (in conjunction with a limited grab sampling program). The benefits and results of this field testing effort are discussed in this paper.

Disclaimer
This paper has been reviewed in accordance with the EPA's peer and administrative review policies and approved for presentation and publication. The mention of trade names or commercial products in this paper does not constitute

endorsement or recommendation for use by the authors, or by their respective employers. The trade names have been included to accurately represent the equipment used for the purpose of testing and evaluation.

Introduction

Historically, tracer studies of distribution systems have been performed using grab sampling techniques. The grab sampling techniques work well in many situations, but have inherent limitations. For example, in complex looping distribution systems, the magnitude and direction of the water flow may change multiple times depending upon the instantaneous demand in the vicinity of the pipe. In a dead-end type situation, it is hard to predict and accurately monitor for tracer arrival. Besides, round-the-clock manual monitoring is expensive, unsafe, and may often result in missed tracer peaks. In heavy traffic and unsafe areas, frequent grab sampling is also difficult. Furthermore, relying solely on grab sampling may be impractical if the study area is large, the tracer front is moving rapidly, or a high frequency of sampling is desired. These phenomena can make it difficult to accurately understand the distribution system and calibrate a hydraulic and water quality model using grab sampling for certain networks and locations. In these cases, automatic continuous monitoring is the best choice, although some grab samples are recommended for confirmation purposes.

EPA and GCWW were the first large-scale users of online monitors as a central focus for distribution system tracer studies. In a series of field studies, the study team introduced calcium chloride (tracer) into a water distribution system and traced the movement of the chemical using automated conductivity meters strategically placed throughout the study area. Four separate tests were conducted representing 1) a small urbanized residential area, 2) a large urbanized residential area, 3) a small dead-end suburban residential area, and 4) a large suburban residential area pressure zone. Similar tracer studies have been subsequently conducted utilizing a combination of online monitors and grab samples by the Centers for Disease Control (CDC) using both fluoride and sodium chloride as tracers in Hillsborough County, Florida (Boccelli et al., 2004) and by the Agency for Toxic Substances and Disease Registry (ATSDR) using fluoride and calcium chloride on a large military base.

This paper presents the results from the EPA/GCWW field testing effortsat a small dead-end suburban residential area.

Study Objectives and Site Selection

The small dead-end suburban residential study area is part of a larger pressure zone. It was selected because of the relatively compact size (approximately 1 mile long) of the water distribution network, fed by a single feeder pipe with no additional storage. As a result, the movement of the tracer was relatively rapid through the system and it could be monitored in great detail with continuous meters placed at several locations in the system. The schematic layout of this subsystem, the location of the injection site and the monitoring locations for this field test are shown in Figure 1. The movement of the tracer was monitored using 20 continuous conductivity meters (to measure specific conductance) located throughout the study area (as shown in Figure 1). The continuous monitoring effort was complemented with periodic manual grab sampling. Additionally, four ultrasonic flow meters were installed in the study area to provide continuous water flow rate measurements at key locations.

The primary objective of this field-site test was to study the effect of the level of EPANET model refinement (e.g., skeletal vs. full pipe, demand allocation, simulation time-step) and calibration of the study area specific distribution system model on the ability of the EPANET model to accurately represent the system. EPANET is a public domain computer program distributed by EPA. EPANET performs extended period simulation of hydraulic and water-quality behavior within pressurized pipe networks.

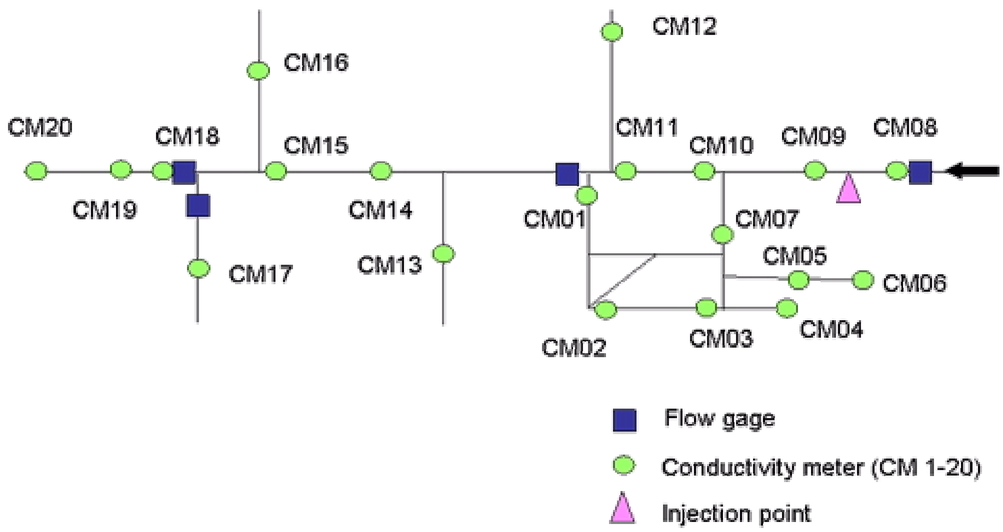


Figure 1. Schematic Layout of the Small Dead-End Suburban Residential Study Area

Field Test Overview

Calcium chloride (tracer) was introduced as two separate pulsesduring the field study. The first two hour pulse (target chloride concentration of 120 mg/L) was followed by a 2.5-hour period of no tracer introduction and then followed by a second pulse (target chloride concentration of 190 mg/L) for 2 hours duration. Thetracer injection rate was calculated based on the expected water flow rate in the pipe and the tracer concentration in the calcium chloride stock solution. The resulting concentration of the tracer in the distribution system (just downstream of the injection point) was carefully monitored to insure that the resulting chloride concentration did not exceed the secondary maximum concentration limit (MCL) of 250 mg/L for chloride. The downstream chloride concentration was monitored at the first continuous monitoring location shown in Figure 1 (CM09, approximately 100 feet downstream of the injection point), with an expected travel time of approximately 10 minutes. However, unexpected variations in flow through the main pipe and, tracer travel time delays resulted in the chloride values slightly exceeding the target level for a very short period before the injection rate could be adjusted.

For the purposes of analysis and calibration, the conductivity readings at various continuous monitor (CM) locations were converted to chloride concentrations using a relationship developed from conductivity and chloride measurements performed on several samples in the laboratory. Figure 2 shows the relationship between conductivity and chloride; both the best-fit linear and polynomial relationships are shown. This conversion was necessary because conductivity is not always a completely linear parameter and, as a result, cannot be exactly simulated in a water distribution system model. The converted continuous chloride concentrations were then compared to the manually collected data for quality control purposes.

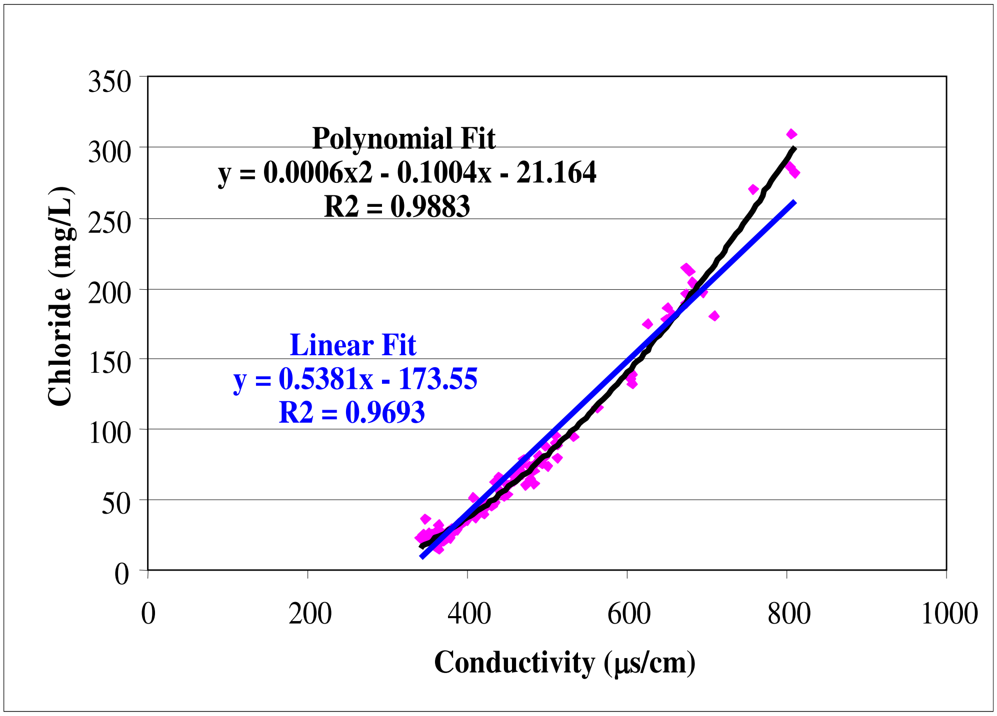


Figure 2. Conductivity vs. Chloride Plot

The continuous monitoring instruments provided round-the-clock minute-by-minute data and the grab sampling data allowed for corrections and adjustments of continuous monitoring data (in situations where some of the automated conductivity meters failed to log time correctly).

Study Results

The preliminary results indicated some discrepancy between the EPANET-model predicted values and the actual field-verified values, indicating the need for model refinement and re-calibration to improve the prediction capability of the EPANET model. Therefore, EPANET modeling was performed to evaluate the following four levels of model refinements:

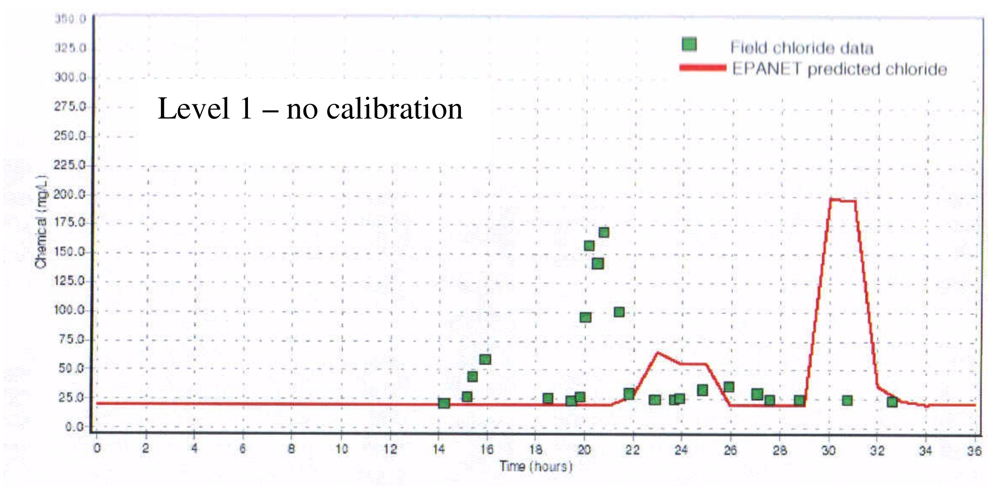
Level 1 (prior to calibration): A skeletonized EPANET model was used with the original hourly demand pattern and a time step injection pattern of 60 minutes.

Level 2: The same as Level 1, but a refined 10-minute time step pattern for injection was used along with the conversion of the original hourly demand patterns to 10-minute patterns. Additional demand nodes were added to represent the water usage at the continuous monitoring stations.

Level 3: The same as Level 2, with a refined demand pattern for each node using the field-measured flow data, adjustment for a large industrial user of water in the study area (based on data obtained during the study) and the available residential water billing information.

Level 4: The same as Level 3, with a detailed all-pipe (non-skeletonized) EPANET model.

The results of the four-stage model refinement and calibration process are shown in Figure 3, for a continuous monitoring location (CM-18 in Figure 1) located on the main feeder pipe. As illustrated, the improvements in the demand estimates and inclusion of the system details in the all-pipe model resulted in a vast improvement in the model's prediction ability for that monitoring location. Similar improvements (but, to a lesser extent) were found for most monitoring locations on the main pipe. The improvements in the case of CM18 are greater because the large industrial user was located close to CM18.



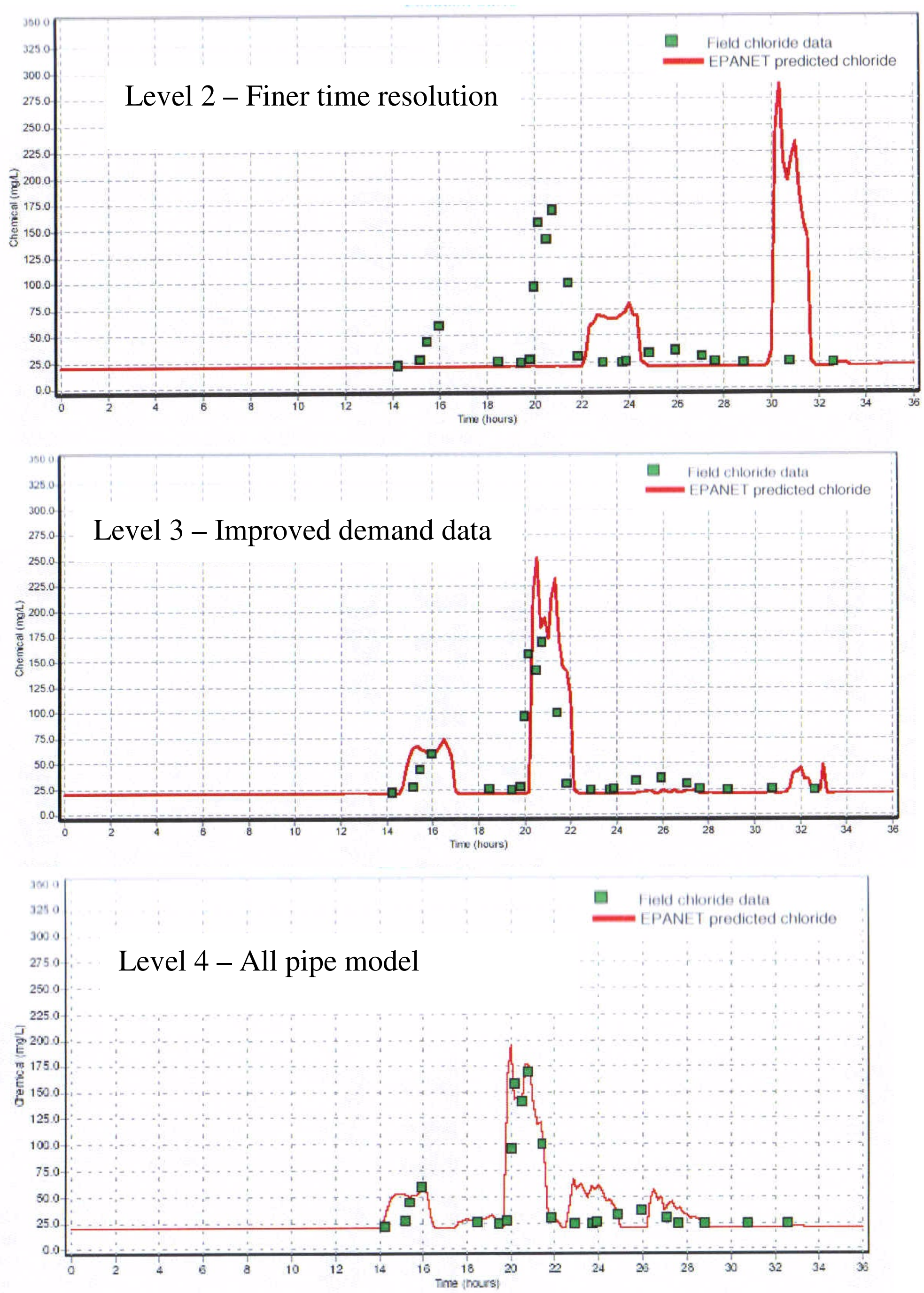


Figure 3. **Comparison of Model Vs. Field Results for Continuous Monitor Location CM-18 at Various Calibration and Refinement Stages**

The calibration of the "looped" portion (referring to the portion of the network on the bottom right hand side of Figure 1) of the water distribution network proved to be more difficult and the results for some monitoring locations on the looped piping were less satisfactory. The most problematic were continuous monitoring locations CM-02 and CM-04 shown in Figure 1. Monitoring station CM-02 was located near the confluence of two separate loops, with the actual monitored connection being slightly offset from the junction node. Examination of the model results showed that flow reached that junction from both directions, and small variations in the amount of flow in each of the loops resulted in very different travel times. The complex travel pattern, associated with the offset location of the monitoring station, resulted in poor prediction of travel time to that station. Also, monitoring station CM-04 is located at the end of a dead-end pipe section and travel to this node is strongly influenced by demands at the very far end of the dead-end section. It is postulated that dispersion and laminar flow, which are not represented in the EPANET model, may have had an influence on the peak concentration due to the very low velocities in the dead end pipe. However, for monitoring location CM-03 located in the main part of the looping system, agreement between the model-predicted and field data was quite good.

Conclusions

The study results illustrate that continuous monitoring dataplayed a key role in the successful calibration of the model. The EPANET modeling time step adjustments and the quick reaction to the tracer concentration would not have been possible without the use of continuous monitors. The continuous monitoring of the flow allowed for the adjustment in water demand inputs which also played a key role in successful calibration and refinement of the model. The results also illustrate that, depending upon the level of refinement (and calibration), there is a significant variation in the capability of the EPANET model to accurately represent the system. In general, the parts of the network that are configured as trees (main stem with branches) are more easily calibrated by making adjustments in demands. For looping parts of the system and at dead-ends, results are very sensitive to small variations in demands and system configuration, leading to the possibility of significant prediction errors at some locations.

Recent technology developments and application of online monitoring technology has greatly improved the level of information captured during a tracer study event. The results from this study and the ongoing improvements in online monitoring technology provide a promise for similar applications in the future. With increased availability of these technologies, costs associated with continuous monitoring are expected to decrease so that larger utilities can afford to purchase and routinely use the equipment, and consulting engineers can affordably offer these services to smaller utilities. The application of this technology has the potential for providing new insights on how water distribution systems may be operated and designed to improve water quality.

The use of continuous monitoring results in a large amountof data document ing minute-by-minute changes in water quality at various points of a network. Therefore, these systems require a relatively high level of sophistication in terms of data management, including the capability to generate real-time reports, graphical and visual representation of information and compliance reports for meeting drinking water standards.

References

Boccelli, D.L., F. Shang, J. G. Uber, A. Orcevic, D. Moll, S. Hooper, M. Maslia, J. Sautner, B. Blount, and F. Cardinali. “Tracer Tests for Network Model Calibration.” Proceedings, ASCE-EWRI Annual Conference. 2004.

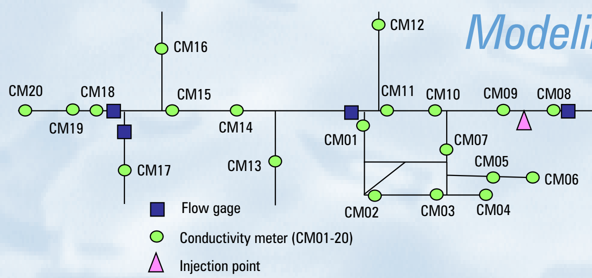
United States
Environmental Protection
Agency

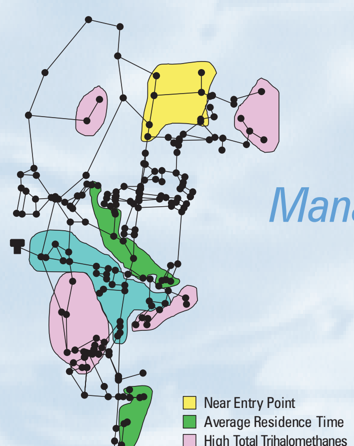
Water Distribution System Analysis: Field Studies, Modeling and Management

A Reference Guide for Utilities

Field Studies

Modeling





Management

EPA/600/R-06/028
December 2005

Water Distribution System Analysis: Field Studies, Modeling and Management

A Reference Guide for Utilities

U. S. Environmental Protection Agency

Office of Research and Development
National Risk Management Research Laboratory
Water Supply and Water Resources Division
Cincinnati, Ohio

Recycled/Recyclable
Printed with vegetable-based ink on
paper that contains a minimum of
50% post-consumer fiber content
processed chlorine free

Notice

Foreword

The U.S. Environmental Protection Agency (EPA) is charged by Congress with protecting the Nation's land, air, and water resources. Under a mandate of national environmental laws, the Agency strives to formulate and implement actions leading to a compatible balance between human activities and the ability of natural systems to support and nurture life. To meet this mandate, EPA's research program is providing data and technical support for solving environmental problems today and building a science knowledge base necessary to manage our ecological resources wisely, understand how pollutants affect our health, and prevent or reduce environmental risks in the future.

The National Risk Management Research Laboratory (NRMRL) is the Agency's center for investigation of technological and management approaches for preventing and reducing risks from pollution that threaten human health and the environment. The focus of the Laboratory's research program is on methods and their cost-effectiveness for prevention and control of pollution to air, land, water, and subsurface resources; protection of water quality in public water systems; remediation of contaminated sites, sediments and groundwater; prevention and control of indoor air pollution; and restoration of ecosystems. NRMRL collaborates with both public and private sector partners to foster technologies that reduce the cost of compliance and to anticipate emerging problems. NRMRL's research provides solutions to environmental problems by: developing and promoting technologies that protect and improve the environment; advancing scientific and engineering information to support regulatory and policy decisions; and providing the technical support and information transfer to ensure implementation of environmental regulations and strategies at the national, state, and community levels.

This publication has been produced as part of the Laboratory's strategic long-term research plan. It is published and made available by EPA's Office of Research and Development to assist the user community and to link researchers with their clients.

Sally Gutierrez, Director
National Risk Management Research Laboratory

Table of Contents Overview

Table of Contents

List of Tables

List of Figures

Acronyms and Abbreviations

AC	Alternating Current
ACCNSCM	Arsenic and Clarifications to Compliance and New Source Contaminant Monitoring
ADAPT	Areal Design and Planning Tool
ADEQ	Arizona Department of Environmental Quality
$Al_2(SO_4)_3$	Aluminum Sulfate
AM/FM	Automated Mapping (or Asset Management)/Facilities Management
AMR	Automated Meter Reading
ASCE	American Society of Civil Engineers
ATSDR	Agency for Toxic Substances and Disease Registry
AWWA	American Water Works Association
AwwaRF	Awwa Research Foundation
C	Coefficient of Roughness
ClO_4	Perchlorate Anion
$CaCl_2$	Calcium Chloride
CAD	Computer-Aided Design
CADD	Computer-Aided Design and Drafting
CDC	Centers for Disease Control and Prevention
CFD	Computational Fluid Dynamics
CIS	Customer Information System
CM	Continuous Monitoring
COGO	Coordinated Geometry
CRT	Cathode Ray Tube
CWS	Contamination Warning System
DBP	Disinfection By-Products
DBPR1	Disinfectant By-Product Rule - Stage 1
DBPR2	Disinfectant By-Product Rule - Stage 2
DC	Direct Current
D.C.	District of Columbia
DEM	Digital Elevation Model
DIME	Dual Independent Map Encoding
DLG	Digital Line Graph
DSOP	Distribution System Water Quality Optimization Plan
DSS	Distribution System Simulator
DTM	Digital Terrain Model
DWQM	Dynamic Water Quality Model
EBMUD	East Bay Municipal Utility District
EDM	Electronic Distance Measurement
EMPACT	Environmental Monitoring for Public Access and Community Tracking
EOSAT	Earth Observation Satellite
EPA	U.S. Environmental Protection Agency
EPS	Extended Period Simulation
ESSA	Environmental Science Services Administration
EWS	Environmental Warning System
FC	Fecal Coliform
$FeCl_3$	Ferric Chloride
FOH	Federal Occupational Health
GA	Genetic Algorithm
gal	Gallon
GBF	Geographic Base File
GC	Gas Chromatograph
GCWW	Greater Cincinnati Water Works
GIS	Geographic Information System
GPD	Gallons Per Day
gpm	Gallons Per Minute
GPS	Global Positioning System
GRASS	Geographic Resources Analysis Support System
GUI	Graphical User Interface
GWR	Ground Water Rule
HAA	Haloacetic Acid
HAA5	The five Haloacetic Acids
HACCP	Hazard Analysis Critical Control Point
HGL	Hydraulic Grade Line
HSPP	Health and Safety Project Plan
ICR	Information Collection Rule
IDSE	Initial Distribution System Evaluation
IESWTR	Interim Enhanced Surface Water Treatment Rule
ILSI	International Life Sciences Institute
I/O	Input/Output
ISE	Ion Selective Electrode
ISO	Insurance Services Office
LCR	Lead and Copper Rule

LIFO	Last In/First Out
LIMS	Laboratory Information Management System
LIS	Land Information System
LT1ESWTR	Long Term 1 Enhanced Surface Water Treatment Rule
LT2ESWTR	Long Term 2 Enhanced Surface Water Treatment Rule
LVVWD	Las Vegas Valley Water District
ma	Milli-Amperes
MCL	Maximum Contaminant Level
MCLG	Maximum Contaminant Level Goal
MDL	Minimum Detection Limit
MDNR	Missouri Department of Natural Resources
MDOH	Missouri Department of Health
MGD	Million Gallons per Day
mg/L	milligrams per liter
MIT	Massachusetts Institute of Technology
MOC	Master Operating Criteria
MRDLG	Maximum Residual Disinfectant Level Goals
MS	Mass Spectrometer
MSU	Montana State University
NaCl	Sodium Chloride
NAD27	North American Datum of 1927
NAD83	North American Datum of 1983
NAPP	National Aerial Photography Program
NASA	National Aeronautics and Space Administration
NFPA	National Fire Protection Association
NHAP	National High Altitude Photography
NIPDWR	National Interim Primary Drinking Water Regulations
NJDHSS	New Jersey Department of Health and Senior Services
NMWD	North Marin Water District
NPL	National Priorities List
NPWA	North Penn Water Authority
NOM	Naturally Occurring Organic (and/or Inorganic) Matter
NRC	National Research Council
O&M	Operations and Maintenance
OCMS	Online Contaminant Monitoring System
ODBC	Open Database Connectivity
ORD	Office of Research and Development
ORP	Oxidation Reduction Potential
PAB3D	A Three-Dimensional Computational Fluid Dynamics Model developed by Analytical Services & Materials, Inc.
PC	Personal Computer
PDD	Presidential Decision Directive
PHRP	Public Health Response Plan
PL	Public Law
POE	Point of Entry
POGA	Progressive Optimality Genetic Algorithm
psi	Pounds Per Square Inch
PVC	Polyvinyl Chloride
PWS	Public Water System
QA	Quality Assurance
QAPP	Quality Assurance Project Plan
QC	Quality Control
RDBMS	Relational Database Management Systems
RDWR	Radon in Drinking Water Rule
SAN	Styrene Acrylonitrile
SCADA	Supervisory Control and Data Acquisition
SCCRWA	South Central Connecticut Regional Water Authority
SDMS	Spatial Database Management System
SDWA	Safe Drinking Water Act
SDWAA	Safe Drinking Water Act Amendments
SMP	Standard Monitoring Program
SNL	Supply Node Link
SOP	Standard Operating Procedure
SPC	State Plane Coordinates
SSS	System Specific Study
SVOC	Semivolatile Organic Compound
SWTR	Surface Water Treatment Rule
SYMAP	Synagraphic Mapping
T&E	Test and Evaluation
TCE	Trichloroethylene
TCR	Total Coliform Rule
TEVA	Threat Ensemble Vulnerability Assessment
THM	Trihalomethane
TIGER	Topologically Integrated Geographic Encoding and Referencing

TIN	Triangulated Irregular Network
TIROS1	Television and Infrared Observation Satellite 1
TOC	Total Organic Carbon
TT	Treatment Technique
TTHM	Total Trihalomethane
TV	Television
U.S.	United States
UF	Ultrafiltration
UHF	Ultra High Frequency
UV	Ultraviolet
UV-Vis	Ultraviolet-Visible
USGS	United States Geological Survey
UTM	Universal Transverse Mercator
VHF	Very High Frequency
WASA	Water and Sewer Authority
WATERS	Water Awareness Technology Evaluation Research and Security
WQP	Water Quality Parameter
WRC	Water Research Centre
WSSM	Water Supply Simulation Model
WSTP	Wells, Storage Tanks, and Pumps
WTP	Water Treatment Plant

Acknowledgments

EPA would like to acknowledge the principal authors of the document, "Water Distribution System Analysis: Field Studies, Modeling, and Management," a Reference Guide for Utilities. The authors were Mr. Srinivas Panguluri, P.E. (Shaw Environmental, Inc.), Dr. Walter M. Grayman, P.E. (W.M. Grayman Consulting Engineer), and Dr. Robert M. Clark, P.E., D.E.E. (Environmental Engineering and Public Health Consultant). The work was performed under EPA Contract No. EP-C-04-034, Work Assignment No. 0-08 and 1-08, with Shaw Environmental, Inc.

EPA acknowledges the significant contributions from Mr. David J. Hartman and Dr. Yeongho Lee, P.E., of the Greater Cincinnati Water Works. Mr. Hartman and Dr. Lee served as external reviewers and directly contributed to the development of portions of this document. Greater Cincinnati Water Works also participated significantly in the field tracer tests that served as a foundation for the development of this document.

EPA acknowledges the peer reviews by the following utility personnel: Mr. William R. Kirkpatrick and Mr. Ronald B. Hunsinger of East Bay Municipal Utility District, and Mr. Arnold Strasser, P.E., of Denver Water.

EPA also acknowledges the peer reviews provided for this document by Dr. Fran A. DiGiano, P.E., of University of North Carolina, Mr. Morris L. Maslia, P.E., D.E.E., of the Agency for Toxic Substances and Disease Registry, Dr. Edward A. McBean, P.E., of University of Guelph, Dr. Vanessa L. Speight, P.E., of Malcolm Pirnie, Ms. L. Michelle Moore of National Drinking Water Clearinghouse, and Mr. E. Radha Krishnan, P.E., of Shaw Environmental, Inc.

EPA also gratefully acknowledges the following contributors to this document: Mr. Daniel R. Quintanar of Tucson Water, Ms. Laura B. Jacobson, P.E., Mr. Sridhar Kamojjala, P.E., and Mr. Mao Fang, P.E., of Las Vegas Valley Water District, Mr. Morris L. Maslia, P.E., D.E.E., of the Agency for Toxic Substances and Disease Registry, Dr. William B. Samuels, P.E., of SAIC, Dr. Mark W. LeChevallier and Dr. Kala K. Fleming of American Water.

EPA Contributors

Ms. Shirley J. Gibson functioned as Project Officer of EPA Contract No. EP-C-04-034, Ms. Lucille M. Garner served as Work Assignment Manager and Mr. Craig L. Patterson, P.E., served as the Alternate Work Assignment Manager and technical reviewer for this document. Mr. Roy C. Haught served as EPA Technical Advisor. Mr. Jonathan G. Herrmann, P.E., Mr. Kim R. Fox, P.E., D.E.E., and Dr. James A. Goodrich performed technical reviews of the document. Mr. Stephen M. Harmon was the EPA Quality Assurance Manager, and was responsible for the quality assurance review of the document. The document was also reviewed by many other EPA reviewers including the following: Mr. Blake L. Atkins, Mr. William H. Davis, Ms. Jean E. Dye, Mr. Robert J. Janke, Mr. Bruce A. Macler, Ms. Jill R. Neal, Dr. Lewis A. Rossman, P.E., Mr. Kenneth H. Rotert, Dr. Irwin J. Silverstein, P.E., D.E.E., and Ms. Elin A. Warn.

Chapter 1
Introduction

Drinking water utilities in the United States (U.S.) and throughout the world face the challenge of providing water of good quality to their consumers. Frequently, the water supply is derived from surface water or groundwater sources that may be subject to naturally occurring or accidentally introduced contamination (ILSI, 1999; Gullick et al., 2003). In other cases, routine upstream waste discharges or purposeful contamination of the water can diminish the quality of the water. The treated water may be transmitted through a network of corroded or deteriorating pipes. All of these factors can result in degradation in the quality of the water delivered to customers.

In the U.S., drinking water quality has to comply with federal, state, and local regulations. This is based on selected physical, chemical, and biological characteristics of the water. The U.S. Environmental Protection Agency (EPA) has promulgated many drinking water standards under the Safe Drinking Water Act (SDWA) of 1974. These rules and regulations require that public water systems (PWSs) meet specific guidelines and/or numeric standards for water quality. The SDWA defines a PWS as a system that serves piped water to at least 25 persons or 15 service connections for at least 60 days each year. For the purposes of this reference guide, PWSs are referred to as utilities.

The SDWA has established two types of numeric standards. The first type of standard is enforceable and referred to as a maximum contaminant level (MCL). The other non-enforceable standard is referred to as a maximum contaminant level goal (MCLG). MCLGs are set at a level at which no known or anticipated adverse human health effects occur. Where it is not economically or technologically feasible to determine the level of a contaminant, a treatment technique (TT) is prescribed by EPA in lieu of establishing an MCL. For example, *Giardia* is a microbial contaminant that is difficult to measure. To ensure proper removal, experimental work has established optimum treatment conditions for the water at a specified pH, temperature, and chlorine concentration for a specified length of time to achieve a fixed level of inactivation.

Compliance with MCL and TT requirements is typically ensured by requiring that water utilities periodically monitor various characteristics of the treated water. In summary, the EPA Guidelines and Standards are designed to ensure that drinking water is adequately treated and managed by water utilities to support public safety, protect public health, and promote economic growth (Clark and Feige, 1993).

Removing contaminants from drinking water can be expensive. Depending upon the type and level of contaminant(s) present in the source water, utilities can choose from a variety of treatment processes. These individual processes can be arranged in a "treatment train" (a series of processes applied in a sequence). The most commonly used treatment processes include coagulation/flocculation, sedimentation, filtration, and disinfection. Some water systems also use ion exchange, membrane separation, ozonation, or carbon adsorption for treatment. The basic treatment options are briefly discussed later in this chapter. As an example, Figure 1-1 depicts the water treatment process implemented by the Greater Cincinnati Water Works (GCWW) at the Miller Plant on the Ohio River.

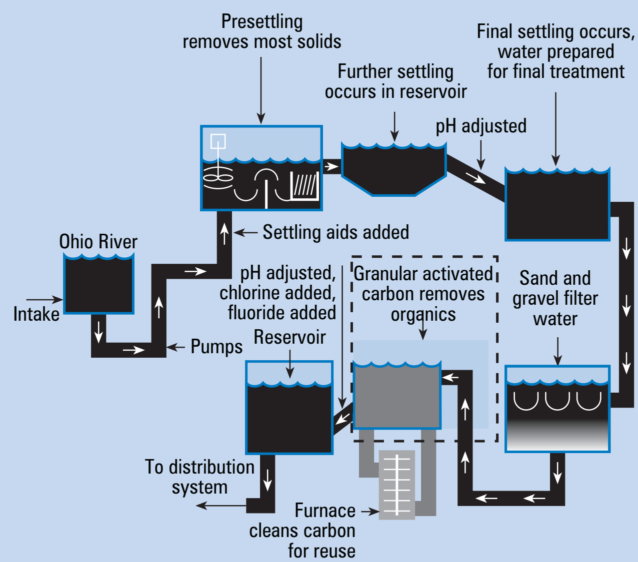


Figure 1-1. Water Treatment Process at the Miller Plant on the Ohio River (Adapted from: GCWW 2005).

Disinfection of drinking water is considered to be one of the major public health advances of the 20th century. The successful application of chlorine as a disinfectant was first demonstrated in England. In 1908, Jersey City (NJ) initiated the use of chlorine for water disinfection in the U.S. This approach subsequently spread to other locations, and soon the rates of common epidemics such as typhoid and cholera dropped dramatically. Today, disinfection is an essential part of a drinking water treatment train. Chlorine, chlorine dioxide, and chloramines are most

While disinfectants are effective in controlling many microorganisms, they can react with naturally occurring organic (and/or inorganic) matter (NOM) in the treated and/or distributed water to form potentially harmful disinfection byproducts (DBPs). Many of these DBPs are suspected of causing cancer, reproductive, and developmental problems in humans. To minimize the formation of DBPs, EPA has promulgated regulations that specify maximum residual disinfectant level goals (MRDLGs) for chlorine (4 milligrams per liter [mg/L] as chlorine), chloramines (4 mg/L as chlorine), and chlorine dioxide (0.8 mg/L as chlorine dioxide). In addition, MCLs for the DBPs total trihalomethanes (TTHMs) and haloacetic acids (HAA5) have been established as 0.080 and 0.060 mg/L, respectively. The TTHMs include chloroform, bromodichloromethane, dibromochloromethane and bromoform. The HAA5 include monochloroacetic acid, dichloroacetic acid, trichloroacetic acid, monobromoacetic acid and dibromoacetic acid. In order to meet these requirements, utilities may need to remove the DBP precursor material from the water prior to disinfection by applying appropriate treatment techniques or modify their disinfection process.

often used because they are very effective disinfectants, and residual concentrations can be maintained in the water distribution system. Some utilities (in the U.S. and Europe) use ozone and chlorine dioxide as oxidizing agents for primary disinfection prior to the addition of chlorine or chlorine dioxide for residual disinfection. The Netherlands identifies ozone as the primary disinfectant, as well as common use of chlorine dioxide, but typically uses no chlorine or other disinfectant residual in the distribution system (Connell, 1998).

Prior to the passage of the SDWA of 1974, most drinking water utilities focused on meeting drinking water standards at the treatment plant, even though it had long been recognized that water quality can deteriorate in a distribution system. The SDWA introduced a number of MCLs that must be measured at various monitoring points in the distribution system. Consequently, water quality in the distribution system became a focus of regulatory action and of major interest to drinking water utilities. Subsequently, utilities worked with various research organizations (including EPA) to understand the impact of the distribution system on water quality. The collective knowledge from this research has been applied to improve the quality of water delivered to the consumer (Clark and Grayman, 1998).

Some important distribution system water quality concerns are: maintenance of proper disinfectant levels; minimization of DBP formation; turbidity, taste, color, and odor issues; distribution tank mixing and utilization; main repair and pressure stabilization; flow management; cross-connection control and back-flow prevention.

Some water quality goals are contradictory. For example, an important goal is to maintain a positive disinfectant residual in order to protect against microbial contamination. However, DBPs (TTHMs) will increase as water moves through the network as long as disinfectant residual and NOM is available. Other DBPs (HAA5) are degraded biologically when free chlorine or chloramines are nearly absent.

Prior to September 11, 2001 (9/11), few water utilities were using online monitors in a distribution system as a means of ensuring that water quality was being maintained and addressed in cases of deviation from established ranges. Now the enhanced focus on water security has led EPA and water utilities to collectively evaluate commercial technologies to remotely monitor the distribution system water quality in real-time. As a part of an evolutionary process, in the future, these monitoring technologies are expected to be integrated with computer modeling and geospatial technologies. This evolution of monitoring and modeling technologies can potentially minimize the risks from drinking water contaminants in distribution systems.

This reference guide has been prepared to provide information to drinking water utilities and researchers on the state of the art for distribution system management and modeling. Guidance is provided on the application of advanced modeling tools that can enhance a utility's ability to better manage distribution system water quality. This introductory chapter provides the basic concepts, which include:

- Distribution system – infrastructure design and operation (definitions and overview).
- Water quality problems and issues (a brief review).
- Regulatory framework (an overview).
- Assessment and management of water quality (current practices).
- Advanced tools for water quality management (in distribution systems).

Subsequent chapters will provide more details on related concepts and tools.

1.1 Distribution System - Infrastructure Design and Operation

Distribution system infrastructure is a major asset of a water utility, even though most of the components are either buried or located inconspicuously. Drinking water distribution systems are designed to deliver water from a source (usually a treatment facility) in the required quantity, quality, and at satisfactory pressure to individual consumers in a utility's service area. In general, to continuously and reliably move water between a source and a customer, the system would require storage reservoirs/tanks, and a network of pipes, pumps, valves, and other appurtenances. This infrastructure is collectively referred to as the drinking water distribution system (Walski et al., 2003).

1.1.1 Key Infrastructure Components

A detailed description of the various distribution system infrastructure components is readily available from other sources and beyond the scope of this document. However, for the purposes of establishing the basics, this section includes a brief discussion of the uses of the major components, their characteristics, general maintenance requirements, and desirable features.

1.1.1.1 Storage Tanks/Reservoirs

Tanks and reservoirs are used to provide storage capacity to meet fluctuations in demand, to provide reserves for fire-fighting use and other emergency situations, and to equalize pressures in the distribution system. The most frequently used type of storage facility is the elevated tank, but other types of tanks and reservoirs include in-ground tanks and open or closed reservoirs. Materials of construction include concrete and steel. An issue that has drawn a great deal of interest is the problem of water turnover within storage facilities. Much of the water volume in storage tanks is dedicated to fire protection. Unless utilities make a deliberate effort to exercise (fill and draw) their tanks, or to downsize the tanks when the opportunity presents itself, there can be both water aging and water mixing problems. The latter can lead to stratification and/or large stagnant zones within the water volume. Some of these issues will be discussed later in this document.

1.1.1.2 Pipe Network

The system of pipes or "mains" that carry water from the source (such as a treatment plant) to the consumer is often categorized as transmission/trunk, distribution, and service mains. Transmission/trunk mains usually convey large amounts of water over long distances, such as from a treatment facility to a storage tank within the distribution system. Distribution mains are typically smaller in diameter than the transmission mains and generally follow city streets. Service mains are pipes that carry water from the distribution main to the building or property being served. Service lines can be of any size, depending on how much water is required to serve a particular customer, and are sized so that the utility's design pressure is maintained at the customer's property for the desired flows. The most commonly used pipes today for water mains are ductile iron, pre-stressed concrete, polyvinyl chloride (PVC), reinforced plastic, and steel. In the past, unlined cast iron pipe and asbestos-cement pipes were frequently used. Even a medium-sized water utility may have thousands of miles of pipes constructed from various types of materials, ranging from new, lined or plastic pipes to unlined pipes that are more than 50 years old. Over time, biofilms and tubercles attached to pipe walls can result in both loss of carrying capacity and a significant loss of disinfectant residual, thereby adversely affecting water quality (Clark and Tippen, 1990). Figure 1-2 depicts the various distribution system interactions that may adversely affect water quality.

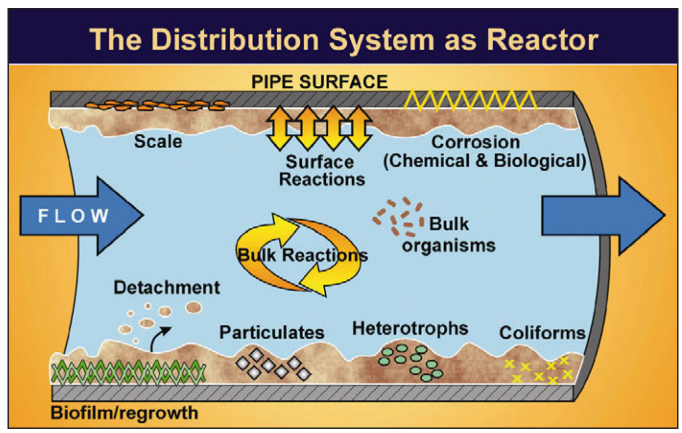


Figure 1-2. Distribution System Interactions that Affect Water Quality (Adapted from: MSU, 2005).

The mains should be placed in areas along the public right of way, which provides for ease of access, installation, repair, and maintenance. Broken or leaking water mains should be repaired as soon as possible to minimize property damage and loss of water. In the past, it has been standard practice to maintain the carrying capacity of the pipe in the distribution system as high as possible to provide the design flow and keep pumping costs as low as possible. However, there has been recent concern that excess capacity can lead to long residence times and thus contribute to deterioration in water quality.

1.1.1.3 Valves

There are two general types of valves in a distribution system: isolation valves and control valves. Isolation valves are used in the distribution system to isolate sections for maintenance and repair and are typically located in a system so that the areas isolated will cause a minimum of inconvenience to other service areas. Maintenance of the valves is one of the major activities carried out by a utility. Many utilities have a regular valve-turning program in which a percentage of the valves are opened and closed on a regular basis. It is desirable to turn each valve in the system at least once per year. In large systems, this may or may not be practical, but periodic exercise and checking of valve operations should occur. This practice minimizes the likelihood that valves will become inoperable due to corrosion. The implementation of such a program ensures that, especially during an emergency, water can be shut off or diverted and that valves have not been inadvertently closed.

Control valves are used to regulate the flow or pressure in a distribution system. Typical types of control valves include pressure-reducing valves, pressure-sustaining valves, flow-rate control valves, throttling valves, and check valves.

1.1.1.4 Pumps

Pumps are used to impart energy to the water in order to boost it to higher elevations or to increase pressure. Routine maintenance, proper design and operation, and testing are required to insure that they will meet their specific objectives. Pump tests are typically run every five to ten years to check the head-discharge relationship for the pump. Many system designers recommend two smaller pumps instead of one large pump to ensure redundancy.

1.1.1.5 Hydrants and Other Appurtenances

Hydrants are primarily a part of the fire-fighting infrastructure of a water system. Although water utilities usually have no legal responsibility for fire flow, developmental requirements often include fire flows, and thus, distribution systems are designed to support needed fire flows where practical (AWWA, 1998). Proper design, spacing, and maintenance are needed to insure an adequate flow to satisfy fire-fighting requirements. Fire hydrants are typically exercised and tested periodically by water utility or fire department personnel. Fire-flow tests are conducted periodically to satisfy the requirements of the Insurance Services Office (ISO, 2003) or as part of a water distribution system calibration program. Other appurtenances in a water distribution system include blow-off valves and air release valves.

1.1.2 Basic Design and Operation Philosophy

A detailed understanding of "how water is used" is critical to understanding water distribution system design and operation. Almost universally, the manner in which industrial and residential customers use water drives the overall design and operation of a water distribution system. Generally, water use varies both spatially and temporally. Besides customer consumption, a major function of most distribution systems is to provide adequate standby fire-flow capacity (Fair and Geyer, 1971). For this purpose, fire hydrants are installed in areas that are easily accessible by fire fighters and are not obstacles to pedestrians and vehicles. The ready-to-serve requirements for fire fighting are governed by the National Fire Protection Association (NFPA) that establishes standards for fire-fighting capacity of distribution systems (NFPA, 2003). In order to satisfy this need for adequate standby capacity and pressure (as mentioned earlier), most distribution systems use standpipes, elevated tanks, and large storage reservoirs. Additionally, most distribution systems are "zoned." Zones are areas or sections of a distribution system of relatively constant elevation. Zones can be used to maintain relatively constant pressures in the system over a range of ground elevations. Sometimes, zone development occurs as a result of the manner in which the system has expanded.

The effect of designing and operating a system to maintain adequate fire flow and redundant capacity can result in long travel times for water between the

> Conservative design philosophies, aging water supply infrastructure, and increasingly stringent drinking water standards have resulted in concerns over the viability of drinking water systems in the U.S. Questions have been raised over the structural integrity of these systems as well as their ability to maintain water quality from the treatment plant to the consumer. The Clean Water and Drinking Water Infrastructure Gap Analysis (EPA 2002), which identified potential funding gaps between projected needs and spending from 2000 through 2019, estimated a potential 20-year funding gap for drinking water capital, and operations and maintenance, ranging from $45 billion to $263 billion, depending on spending levels. Based on current spending levels, the U.S. faces a shortfall of $11 billion annually to replace aging facilities and comply with safe drinking water regulations. Federal funding for drinking water in 2005 remained level at $850 million—less than 10% of the total national requirement (ASCE, 2005). Parts of many systems are approaching or exceed 100 years old, and an estimated 26 percent of the distribution system pipe in this country is unlined cast iron and steel in poor condition. At current replacement rates for distribution system components, it is projected that a utility will replace a pipe every 200 years (Kirmeyer et al., 1994). Grigg, NS, 2005, provides comprehensive guidance to utilities on how to assess options for distribution system renewal. Grigg's report contains a knowledge base on condition assessment, planning and prioritization, and renewal methods.

Non-potable waters (e.g., sea, river, and lake water) without adequate treatment have been used for fire protection for many years, often with disastrous results. However, reclaimed wastewater (in cases where its quality is better managed than the aforementioned unregulated sources) has been effectively used for providing fire protection (AwwaRF, 2002). St. Petersburg, FL, has been operating such a system to bolster fire-protection capacity since 1976. The reclaimed water hydrants are distinguished from potable water hydrants by color and their special valves. If the reclaimed water system is designed for fire protection, the potable water piping can have a very small diameter and investments can be made in higher quality pipe materials, which, with much shorter residence time in the system, would vastly improve the quality of the water at the tap. With this in mind, where retrofitting one of the two systems is necessary, it might be wiser to use the existing potable water system for the reclaimed water and retrofit with new, high-quality, smaller, potable water lines (Okun, D., 1996).

treatment plant and the consumer. These long travel times and low velocities may be detrimental to meeting the drinking water MCLs. Long residence times may lead to formation of DBPs, loss of disinfectant residuals, bacterial growth, and formation of biofilm.

1.1.2.1 Pipe-Network Configurations

The branch and grid/loop are the two basic configurations for most water distribution systems. A branch system is similar to that of a tree branch with smaller pipes branching off larger pipes throughout the service area. This type of system is most frequently used in rural areas, and the water has only one possible pathway from the source to the consumer. A grid/loop system is the most widely used configuration in large municipal systems and consists of interconnected pipe loops throughout the area to be served. In this type of system, there are several pathways that the water can follow from the source to the consumer. Transmission mains are typically 20 to 24 inches in diameter or larger. Dual-service mains that serve both transmission and distribution purposes are normally 12 to 20 inches in diameter. Distribution mains are usually 6 to 12 inches in diameter in every street. Service lines are typically 1 inch in diameter. Specific pipe sizes can vary depending on the extent of the distribution system and the magnitude of demand. Looped systems provide a high degree of reliability should a line break occur, because the break can be isolated with little impact on consumers outside the immediate area (Clark and Tippen, 1990; Clark et al., 2004).

1.1.2.2 Multiple Source Configuration

Many systems serve communities with multiple sources of supply, such as a combination of wells and/or surface sources. In a grid/looped system, this configuration will influence water quality in a distribution system due to the effect of mixing of water from these different sources. These interactions are a function of complex system hydraulics (Clark et al., 1988; Clark et al., 1991a). Water quality models can be very useful in defining mixing and blending zones within water utility distribution networks. Mixing of water in a network can result in taste and odor problems or other water quality problems and can influence maintenance, repair, and rehabilitation procedures.

1.1.2.3 Impact of System Design and Operation on Water Quality

Based on the design and configuration of a particular system, there are many opportunities for water quality to change as water moves between the treatment plant and the consumer. These unwanted changes may occur due to various reasons including: failures at the treatment barrier, transformations in the bulk phase, corrosion and leaching of pipe material, biofilm formation, and mixing between different sources of water. Many researchers have investigated the factors that influence water quality deterioration once it enters the distribution system. It has been well documented that bacteriological growth can cause taste-and-odor problems, discoloration, slime buildup, and economic problems, including corrosion of pipes and bio-deterioration of materials (Water Research Centre, 1976). Bacterial numbers tend to increase during distribution and are influenced by several factors, including bacterial quality of the finished water entering the system, temperature, residence time, presence or absence of a disinfectant residual, construction materials, and availability of nutrients for growth (Geldreich et al., 1972; LeChevallier et al., 1987; Maul et al., 1985a and b; Zhang and DiGiano, 2002; Camper et al., 2003).

It is difficult and expensive to study the problems caused by system design and configuration in full-scale systems. For example, one approach to studying residual chlorine levels in dead-end or low-flow situations would be to construct a pilot-scale pipe system to simulate the phenomena. Another approach would be to use mathematical hydraulic and water quality models for simulation. For either of these approaches to work, they must be properly configured and/or calibrated to closely simulate a full-scale system. A combination of these approaches may be used to assess various operational and design decisions, to determine the impacts resulting from the inadvertent or deliberate introduction of a contaminant into the distribution system, and to assist in the design of systems to improve water quality.

In pipes, it has been found that chlorine can be lost through both the interaction with NOM in the bulk phase and with pipe walls themselves in transporting finished water. This mechanism for loss of chlorine may be even more serious than long residence times in tanks. The pipe wall demand, possibly due to biofilm and tubercles, may use up the chlorine very rapidly in a distribution system. Maintaining adequate levels of disinfectant residual may require routine cleaning/ replacement of pipes and intensive treatment (Clark et al., 1993a).

1.2 Water Quality Problems and Issues

Drinking water treatment in the U. S. has played a major role in reducing waterborne disease. For example, the typhoid death rate for a particular year in the 1880s was 158 per 100,000 in Pittsburgh, PA, compared with 5 per 100,000 in 1935. Such dramatic reductions in waterborne disease outbreaks were brought about by the application of drinking water standards and engineering "multiple barriers" of protection. The multiple-barrier concept includes the use of conventional treatment (e.g., sand filtration) in combination with disinfection to provide safe and aesthetically acceptable drinking water. The residual disinfectant levels served to protect the water quality within the distribution system prior to its delivery to the consumer (Clark et al., 1991b).

Despite the passage of the SDWA, waterborne outbreaks still occur. Two extensively studied examples of waterborne disease in the U.S. were an *Escherichia coli O157:H7 (E. coli)* outbreak in Cabool, Missouri, in 1989 and a *Salmonella* outbreak in Gideon, Missouri, in 1993. These two examples, discussed later in Chapter 7, illustrate the importance of the multiple-barrier concept. In both cases, the water source was un-disinfected groundwater and the utility's infrastructure was breached, allowing contaminants to enter the system. This contamination resulted in major waterborne outbreaks. Water quality modeling was used in both cases to identify the source of the outbreaks and to study the propagation of the outbreak through the distribution network (Clark et al., 1993a and b).

One useful outcome of the outbreaks in Missouri is that the ensuing investigative studies have typically led to the development and enhancement of scientific analysis techniques. For example, the Gideon *Salmonella* outbreak conclusions were based on statistical studies performed by Centers for Disease Control and Prevention (CDC) and corroborated by water quality modeling performed by EPA. The study provides an example of how tools such as water quality models can be used to reliably study contaminant propagation in a distribution system (Clark et al., 1996). Both the Gideon and Cabool incidents were associated with source water contamination, inadequate treatment, and breeches in the distribution system.

These types of problems are not just isolated incidents of infrastructure breakdowns. In fact, several problems with drinking water systems in the U. S. have been identified by researchers. The National Research Council (NRC, 2005) examined the causes of waterborne outbreaks reported by various investigators between 1971 and 2004. Figure 1-3 presents the total number and proportion of waterborne diseases associated with distribution system deficiencies

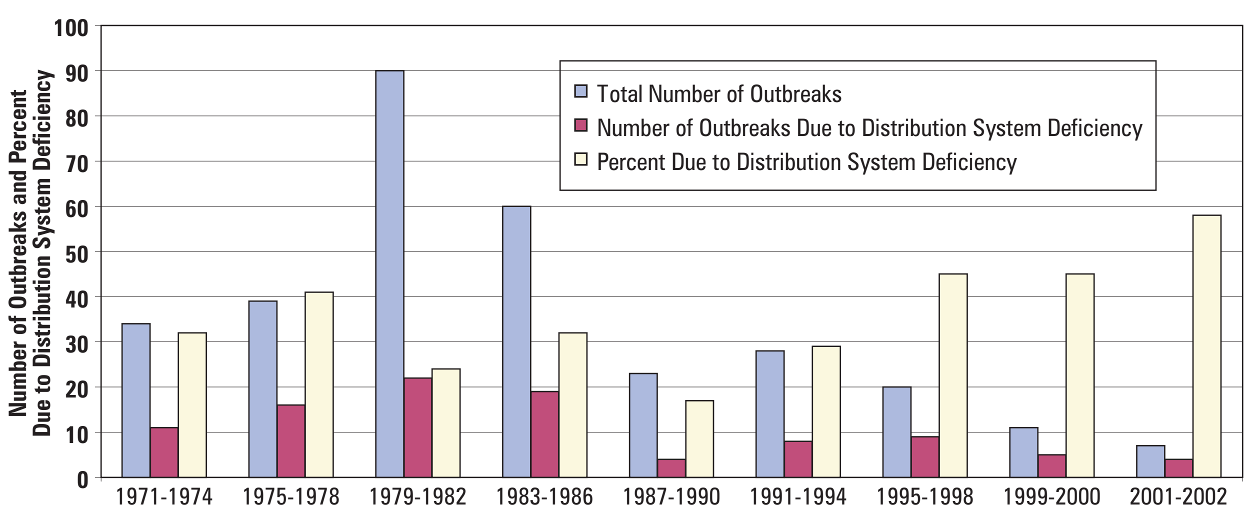


Figure 1-3. Total Number and Proportion of U.S. Waterborne Diseases Associated with Water Distribution System Deficiencies.

On December 16, 1974, the U. S. Congress passed the SDWA, which authorized the EPA to promulgate regulations which would "protect health to the extent feasible, using technology, treatment techniques, and other means, which the Administrator determines are generally available (taking costs into consideration)…"(SDWA, 1974). As a result, a set of regulations was promulgated in 1975 which became effective June 24, 1977. These were known as the National Interim Primary Drinking Water Regulations (NIPDWR). The NIPDWR established MCLs for 10 inorganic contaminants, six organic contaminants, turbidity, coliform, radium-226, radium-228, gross alpha activity, and man-made radionuclides. The NIPDWR also established monitoring and analytical requirements for determining compliance.

(extracted from the NRC report). As the figure reveals, overall there is a general decrease in the total number of waterborne disease outbreaks during the reported period. However, there is a general increase in the percentage of outbreaks that are associated with distribution system deficiencies. The NRC report attributes this increase in percentage of outbreaks (attributable to distribution system deficiencies) to the lack of historical regulatory focus on distribution systems.

1.3 Regulatory Framework

Concerns about waterborne disease and uncontrolled water pollution resulted in federal water quality legislation starting in 1893 with the passage of the Interstate Quarantine Act and continuing to 1970 under the stewardship of the U.S. Public Health Service (AWWA, 1999). Even though significant advances were made to eliminate waterborne disease outbreaks during that period, the focus of drinking water concerns began to change with the formation of the EPA in late 1970. By the 1970s, more than 12,000 chemical compounds were known to be in commercial use and many more were being added each year. Many of these chemicals cause contamination of groundwater and surface water, and are known to be carcinogenic and/or toxic. The passage of the SDWA of 1974 was a reflection of concerns about chemical contamination. In this section, a brief overview of the regulatory framework is presented. A detailed history of the evolution of the federal drinking water regulations is beyond the scope of this document.

Early in the history of the SDWA, the major focus of EPA was to implement the Act and to initiate the regulatory process. The first MCL established under the SDWA was the TTHM Rule in 1979. However, after several years of developing regulations, it became obvious that the rulemaking process must extend beyond a focus on MCLs at the treatment plant and into the distribution system. Many water utilities in the U.S. using surface supplies were experiencing waterborne outbreaks, especially from *Giardia*. The 1986 SDWA Amendments laid the groundwork for the promulgation of the Total Coliform Rule (TCR) and the Surface Water Treatment Rule (SWTR) in 1989. The 1986 SDWA Amendments also set forth an aggressive plan to eliminate lead from PWSs and resulted in the promulgation of the Lead and Copper Rule (LCR) in 1991. These actions therefore extended the SDWA beyond its focus on the treatment plant and into the distribution system (Owens, 2001).

A summary of the evolution of federal drinking water regulation since the passage of the SDWA in 1974 is presented in Figure 1-4. In addition to the rules and regulations promulgated under the SDWA, security has recently become an issue for the water utility industry. Security of water systems is not a new issue. The potential for natural, accidental, and purposeful contamination of water supplies has been the subject of many studies. For example, in May 1998, President Clinton issued Presidential Decision Directive (PDD) 63 that outlined a policy on critical infrastructure protection, including our nation's water supplies. However, it was not until after September 11, 2001, that the water industry focused on the vulnerability of the nation's water supplies to security threats. In recognition of these issues, President George W. Bush signed the Public Health Security and Bioterrorism Preparedness and Response Act of 2002 (Bioterrorism Act) into law in June 2002 (PL107-188). Under the requirements of the Bioterrorism Act, community water systems (CWSs) serving more than 3,300 people are required to prepare vulnerability assessments and emergency response plans. CWSs are PWSs that supply water to the same population throughout the year.

Table 1-1 summarizes the key requirements of the regulations presented in Figure 1-4 from a distribution system compliance perspective.

Many of the tools and techniques discussed in this reference guide can assist in complying with the rules and regulations and security issues discussed above. Water quality modeling techniques can be used to identify points in the distribution system that experience long retention times, which can in turn represent locations in the system that may experience chlorine residual loss, excessive formation of DBPs, and the formation of biofilms. Chlorine residual loss, in conjunction with biofilm

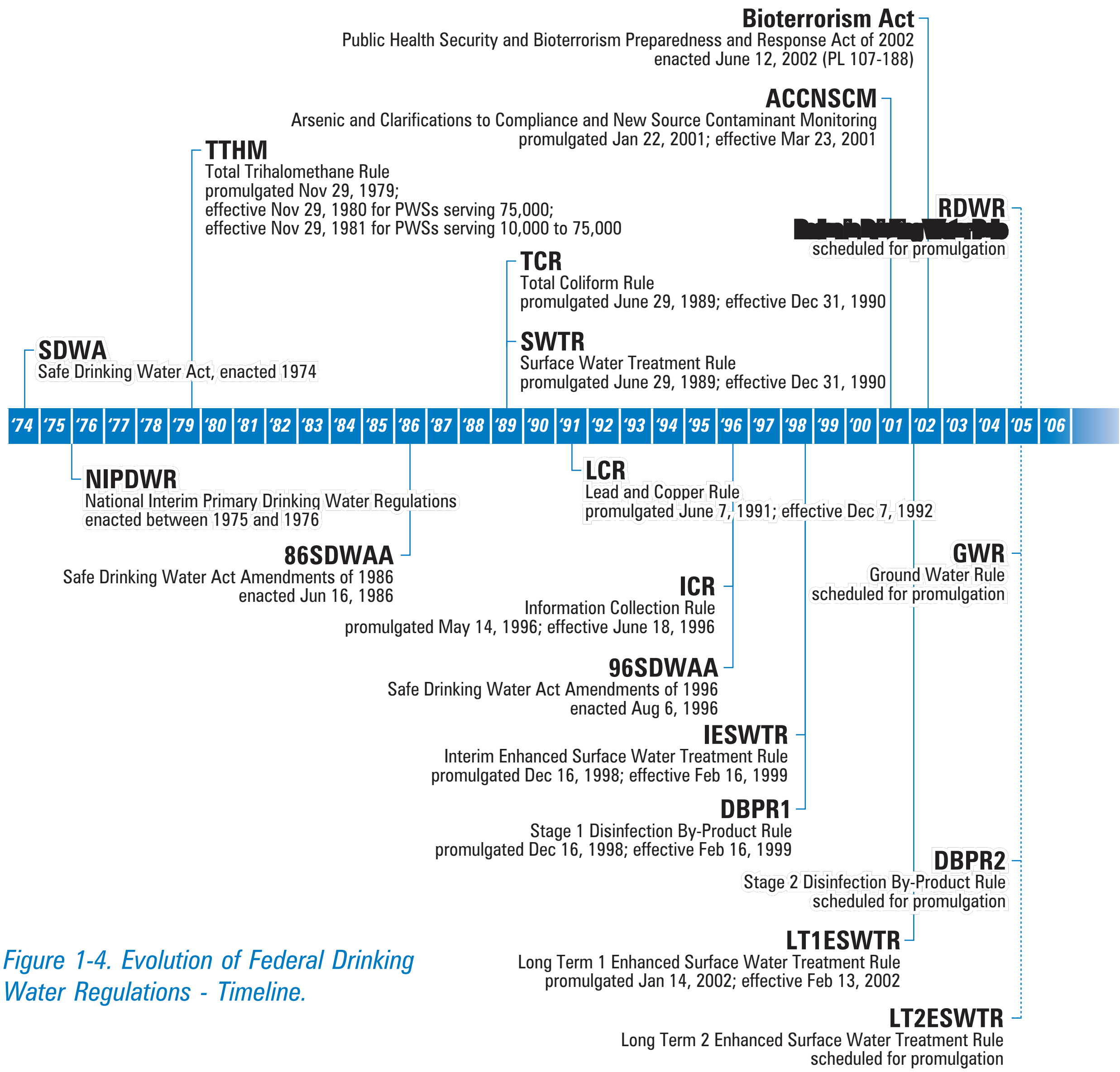


Figure 1-4. Evolution of Federal Drinking Water Regulations - Timeline.

Meeting and balancing the requirements of the various regulations can provide a significant challenge to water utilities. In some cases, regulations provide guidance or requirements that could result in contradictory actions. For example, the SWTR requires the use of chlorine or some other disinfectant. However, chlorine or other disinfectants interact with NOM in treated water to form DBPs. Similarly, raising the pH of treated water will assist in controlling corrosion but may increase the formation of TTHMs. Various analytical tools, such as water quality models, can provide the utility with information and an understanding that helps the utility in balancing the contradictory requirements of some regulations.

formation, may result in the sporadic occurrence of coliforms ("indicator" organisms associated with bacteriologically polluted water). Models can be used to define mixing zones where blending water from two or more sources results in water quality problems. Specifically, water quality modeling tools may assist utilities in complying with the TCR, SWTR, IESWTR, LT1ESWTR, and LCR. Modeling can assist in identifying parts of the system with high TTHM formation potential (DBPR1) and meeting the Initial Distribution System Evaluation (IDSE) requirements of the DBPR2 (see the IDSE Case Study in Chapter 7). In addition, modeling techniques can assist in tracking contamination from cross-connections and other accidental or deliberate contamination events such as a waterborne outbreak.

1.4 Assessment and Management of Water Quality

Water utilities treat nearly 34 billion gallons of water every day (EPA, 1999). Generally, surface water systems require more treatment than groundwater

Table 1-1. Selected Rules and Regulations Dealing with Distribution Systems (Not Inclusive)

Regulation	Key Distribution System Requirements
SDWA	Gives EPA the authority to establish national primary and secondary drinking water regulations (MCLs and MCLGs).
NIPDWR	The NIPDWR which was adopted at the passage of the SDWA required that representative coliform samples be collected throughout the distribution system.
TTHM	Established a standard for TTHMs as 0.1 mg/L.
86SDWAA	Established the MCLG concept.
TCR	Regulates coliform bacteria which are used as surrogate organisms to indicate whether or not treatment is effective and system contamination is occurring.
SWTR	Requires using chlorine or some other disinfectant.
LCR	Monitoring for compliance with the LCR is based entirely on samples taken at the consumer's tap.
ICR	Provided data to support the interim and long-term enhanced SWTR, and Stage 2 DBP rule.
96SDWAA	Has many provisions dealing with distribution systems, including the role that surface water quality can play in influencing the quality of distributed water.
IESWTR	Provisions to enhance protection from pathogens, including *Cryptosporidium*, and intended to prevent increases in microbial risk while large systems comply with the DBPR1.
DBPR1	Has lowered the standard for TTHMs from 0.1 mg/L to 0.08 mg/L. This standard applies to all community water supplies in the U. S. and requires monitoring and compliance at selected points in the distribution system.
LT1ESWTR	Provisions to enhance protection from pathogens, including *Cryptosporidium*, and prevent increases in microbial risk for systems serving less than 10,000 people while they comply with the DBPR1.

systems because they are directly exposed to the atmosphere, runoff from rain and melting snow, and other industrial sources of contamination. Water utilities use a variety of treatment processes to remove contaminants from drinking water prior to distribution. The selected treatment combination is based on the contaminants found in the source water of that particular system. The general techniques include:

- Coagulation/Flocculation: This process removes dirt and other particles suspended in the water. In this process, alum, iron salts, and/or synthetic organic polymers are added to the water to form sticky particles called "floc," which attract the suspended particles.

- Sedimentation: In this process, the flocculated particles are gravity-settled and removed from the water.

- Filtration: Many water treatment facilities use filtration to remove the smaller particles from the water. These particles include: clays and silts, natural organic matter, precipitates from other treatment processes in the facility, iron and manganese, and microorganisms. Filtration clarifies the water and enhances the effectiveness of disinfection.

- Disinfection: Water is disinfected at the water treatment plant (or at the entry to the distribution system) to ensure that microbial contaminants are inactivated. Secondary disinfection is practiced in order to maintain a residual in the distribution system.

Once the treated water enters the distribution system, a number of processes may occur that can adversely impact the water quality delivered to consumers. As the water enters a network of buried pipes, valves, joints, meters, and service lines, it is subject to disruptions such as water hammer (transient pressure shock wave), aging (at dead ends and large tanks), corrosion, cross-connections, leaching of toxic chemicals, intrusion of pathogens, and pipeline breaks. Some of these events may be regular occurrences, such as water aging, loss of chlorine residual in dead ends, or deposition of sedimentation in stagnant areas. Others may be rare or unusual events. Any of these events can cause the water quality to deteriorate and pose a potential public health risk. Some routine distribution system design changes and maintenance or operational procedures that can help to prevent or reduce the effects of such events include the following:

- Tank Mixing: Inadequate mixing in a tank can lead to stagnant areas containing older water

Maintaining water quality in a drinking water distribution system while assuring adequate disinfection and reducing DBPs is a significant challenge for many drinking water utilities. This challenge will be even greater under the more stringent requirements of the LT2ESWTR and the DBPR2. Utilities that use chorine as their primary disinfectant and that have elevated organic levels in their treated water, long detention times, and/or warm water may have difficulty in meeting these regulations. The Las Vegas Valley Water District is conducting research to explore the feasibility of employing "targeted" distribution system treatment systems. This type of targeted system (or systems) would utilize small-scale water treatment technology to reduce the concentration of disinfection byproducts in those areas that might exceed the SDWA MCLs established under the LT2ESWTR and DBPR2. These systems are intended to be designed and operated in conjunction with a water quality/hydraulic model which would be used to predict where these decentralized treatment systems should be located. If the treatment technology is relatively mobile, it could be moved based on model predictions to locations where MCL violations are likely to occur. In addition, these types of systems would be valuable should a security threat arise.

that has lost its disinfectant residual. Changes in operations (e.g., exercising the tank) or modifications to inlet-outlet configurations can improve mixing.

- Re-chlorination: Some parts of a distribution system may experience long travel times from the treatment plant resulting in loss of chlorine residual. Installation of booster chlorination facilities at these locations can sometimes be an effective means of insuring an adequate residual in these areas.
- Conventional Flushing: This procedure generally involves opening hydrants in an area until the water visibly runs clear. The object of this action would be to quickly remove contaminated water; however, it would not likely be effective in removal of contaminants that become attached to the pipe surfaces. Flushing only provides a short-term remedy.
- Unidirectional Flushing: This procedure involves the closure of valves and opening of hydrants to concentrate the flow in a limited number of pipes. Flow velocities are maximized so that shear velocity near the pipe wall is maximized. It is intended to be done in a progressive fashion, proceeding outward from the source of water in the system so that flushing water is drawn from previously flushed

Federal and state drinking water regulations are designed to provide a water supply to consumers that meets minimum health-based requirements. However, water utilities may choose to implement programs that go beyond current federal, state, and local regulatory requirements to increase the water quality and reduce the potential for contamination in water systems. There are several methods and guidelines that have been designed to assist utilities in providing water of a quality that exceeds the minimum requirements. These methods include: Hazard Analysis Critical Control Point (HACCP), source water optimization, and distribution system water quality optimization plans (DSOP).

DSOP is one example of a framework for evaluating and improving programs that affect distribution system water quality (Friedman et al., 2005). Aspects of the DSOP include evaluation of conditions within the distribution system, creation of improved documentation, and enhancement of communication between the various utility functions that impact water quality in the distribution system. DSOPs address both regulatory/compliance issues and customer issues related to aesthetic properties of drinking water. The DSOP approach was piloted at three water utilities and a general template was developed that can be used by small, medium, and large utilities. The following ten steps are identified as part of the development of a DSOP:

1. Formation of a committee to discuss distribution system issues of interest/concern and to guide the process of DSOP development.
2. Identification of water quality and operating goals.
3. Completion of a distribution system audit.
4. Comparison of audit results to industry best management practices.
5. Development of a list of utility needs for optimizing distribution system water quality.
6. Prioritization of DSOP elements based on relative contribution towards improving water quality and precluding water quality degradation or contamination.
7. For each priority DSOP element, compilation of applicable standard operating procedures (SOPs) and ongoing programs that provide information related to the condition of the distribution system and water quality.
8. Development and implementation of priority programs.
9. Periodic review of programs and goals developed as part of the DSOP.
10. Development of revised SOPs that describe the optimized approach.

DSOP and other aforementioned methodologies are still in their early stages of application in the water supply industry and will require further evaluation to determine their effectiveness in meeting the goals to improve water quality in drinking water systems.

reaches. No special equipment is required; however, some planning time is required to determine the flushing zones, the valves and hydrants to be operated, and the duration of the flushing exercise for each zone.

- Valve Exercising Program: A routine program to exercise isolation valves can have several positive effects. These include identifying (and repairing) malfunctioning valves and identifying valves that are in an inappropriate setting (e.g., closed valves that are expected to be open).
- Cross-Connection Control Program: An inspection program intended to ensure no interconnection(s) between the drinking water and wastewater systems in homes and buildings.

Examples of routine maintenance and operation procedures for pipe cleaning include the following (AwwaRF, 2004):

- Air Scouring, Swabbing and Abrasive pigging: Air scouring, swabbing, and abrasive pigging are progressively more aggressive cleanup techniques that involve more specialized equipment and skills. A few water utilities have implemented these methods using their own staff; typically, these methods are contracted to specialty firms. Implementation of these methods would require installation of new pipeline appurtenances (e.g., pig launching and receiving stations; pigging is not recommended for cast iron pipes).
- Chemical/Mechanical Cleaning and Lining: Chemical cleaning involves the recirculation in an isolated pipe section of proprietary acids and surfactants to remove scale and deposits, while mechanical cleaning is accomplished by dragged scrapers. These techniques are typically applied in the rehabilitation of older unlined cast iron pipe which, over time, have become scaled and tuberculated. These cleaning operations are typically followed by an in-situ application of a thin cement mortar or epoxy lining to ensure lasting protection.

If the symptoms persist after the application of these techniques, the pipes are usually replaced.

1.5 Advanced Tools for Water Quality Management

Recent advancements in computation and instrumentation technologies have led to the availability of advanced tools that are already beginning to improve a utility's ability to effectively manage water quality in distribution systems. These computational advancements have led to the development of software models that can simulate the behavior of distribution system networks. Water distribution system models (such as EPANET) have become widely accepted both within the water utility industry and the general research arena for simulating both hydraulic and water quality behavior in water distribution systems. The advancements in instrumentation and Supervisory Control and Data Acquisition (SCADA) systems now enable the utilities to monitor and control various water quality parameters from a remote location in real-time within a distribution system network. Furthermore, recent advances in Geographic Information Systems (GIS) technology have led to the integration of remote monitoring network models with GIS layers. This combination provides utilities a visual tool to efficiently manage both water quality and distribution system assets such as pipes, pumps, and valves.

1.6 Report Organization

Various chapters of this reference guide will describe modeling and monitoring tools for effectively managing water quality in drinking water distribution systems. Examples and protocols for effectively applying water quality models for understanding and resolving water quality issues in networks will be presented. Another important aspect of effectively applying water quality models is to ensure that they are properly and periodically calibrated. Tracer tests are one of the most effective techniques for calibrating a water quality model. Modeling techniques, when combined with advanced monitoring and geospatial technologies, can play a vital role in managing water quality in distribution systems. Chapter 2 provides an overview on modeling of distribution systems for water quality. Chapter 3 describes techniques for conducting tracer studies in distribution systems. Chapter 4 presents data analysis techniques for effectively calibrating a distribution system model using tracer or other field data. Chapter 5 provides an overview of monitoring techniques and technologies available for monitoring water quality. Chapter 6 introduces geospatial technology and its relation to water distribution systems. Finally, Chapter 7 is a compilation of real-world applications of water quality modeling and monitoring for planning, analysis and simulation of historical events.

1.7 Summary

Distribution system infrastructure is a major asset of most water utilities. It serves many important functions in a community, such as promoting eco-

The information presented in this reference guide is intended for a general technical audience. The various chapters provide an overview of the state-of-art techniques for managing water quality in distribution systems. For a more comprehensive case-specific solution, the reader should refer to text books in specific subject areas and/or consult with water quality professionals. The following is a brief listing of recommended books (listed in alphabetical order by title):

1. Advanced Water Distribution Modeling and Management. T.M. Walski, D.V. Chase, D.A. Savic, W. Grayman, S. Beckwith, and E. Koelle. Haestad Press, Waterbury, CT. 2003.
2. Comprehensive Water Distribution Systems Analysis Handbook. P.B. Boulos, K.E. Lansey, and B.W. Karney. MWHSOFT, Inc., Pasadena, CA. 2004.
3. Computer Modeling of Water Distribution Systems (M32), AWWA. 2004.
4. GIS Applications for Water, Wastewater, and Stormwater Systems. U. Shamsi. CRC Press. 2005.
5. Hydraulics of Pipeline Systems. B.E. Larock, R.W. Jeppson, G.Z. Watters. CRC Press. 1999.
6. Microbial Quality of Water Supply in Distribution Systems. Edwin E. Geldreich. CRC Press. 1996
7. Modeling, Analysis and Design of Water Distribution Systems. L. Cesario. AWWA. 1995.
8. Modeling Water Quality in Drinking Water Distribution Systems. R.M. Clark and W.M. Grayman. AWWA. 1998.
9. Online Monitoring for Drinking Water Utilities. Edited by E. Hargesheimer, O. Conio, and J. Popovicova. AwwaRF – CRS ProAqua. 2002.
10. Safe Drinking Water: Lessons from Recent Outbreaks in Affluent Nations. S.E. Hrudey and E.J. Hrudey, IWA Publishing. 2004.
11. Water Distribution Systems Handbook. Edited by L.W. Mays, McGraw Hill. 2000.
12. Water Supply Systems Security. Edited by L.W. Mays, McGraw Hill. 2004.

nomic growth, supporting public safety, and protecting public health. In order for a community to grow and prosper, it must have the physical infrastructure to provide basic services such as water supply. In addition to the economic implications of adequate water supply, water systems play a critical role in supporting public safety through the provision of fire protection capacity. Frequently, insurance rates in a community are tied to the fire protection capability of the water system. Water systems play a key role in protecting a community's public health by providing safe drinking water to water consumers.

References

American Society of Civil Engineers (ASCE). "Report Card for America's Infrastructure, Drinking Water." ASCE, Reston, VA. 2005.

American Water Works Association (AWWA). *AWWA Manual M31, Distribution System Requirements for Fire Protection*, AWWA, Denver, CO. 1998.

AWWA. *Water Quality & Treatment – A Handbook of Community Water Supplies.* Edited by R.D. Letterman, McGraw-Hill, NY. 1999.

American Water Works Association Research Foundation (AwwaRF). *Standard Operating Procedures for Decontamination of Water Infrastructure.* March, 2004.

AwwaRF. *Impacts of Fire Flow on Distribution System Water Quality, Design, and Operation.* 2002.

Camper, A.K., K. Brastrup, A. Sandvig, J. Clement, C. Spencer, and A.J. Capuzzi. "Impact of distribution system materials on bacterial regrowth." *Journal of AWWA,* 95(7):107-121. 2003.

Clark, R.M., W.M. Grayman, and R.M. Males. "Contaminant Propagation in Distribution Systems," *Journal of Environmental Engineering*, ASCE, 114:(2). 1988.

Clark, R.M., W.M. Grayman, and J.A. Goodrich. "Water Quality Modeling: Its Regulatory Implications." Proceedings, AwwaRF/EPA Conference on Water Quality Modeling in Distribution Systems, Cincinnati, OH. 1991a.

Clark, R.M., D.J. Ehreth, and J.J. Convery. "Water Legislation in the US: An Overview of the Safe Drinking Water Act," *Toxicology and Industrial Health*, 7(516):43-52. 1991b.

Clark, R.M., and W.A. Feige. "Meeting the Requirements of the Safe Drinking Water Act," in *Strategies and Technologies for Meeting the Requirements of the SDWA.* Edited by Robert M. Clark and R. Scott Summers, Technomics, Publishing Company, Inc., Lancaster, PA. 1993.

Clark, R.M., W.M. Grayman, R.M. Males, and A.F. Hess, 1993a. "Modeling Contaminant Propagation in Drinking Water Distribution Systems," *Journal of Environmental Engineering*, ASCE, 119(2):349-364. 1993.

Clark, R.M., J.A. Goodrich, and L.J. Wymer. "Effect of the Distribution System on Drinking Water Quality," *Journal of Water Supply Research and Technology - Aqua*, 42(1):30-38. 1993b.

Clark, R.M., E.E. Geldreich, K.R. Fox, E.W. Rice, C.H. Johnson, J.A. Goodrich, J.A. Barnick, and F. Abdesaken. "Tracking a *Salmonella* serovar Typhimurium Outbreak in Gideon, Missouri: Role of Contaminant Propagation Modeling," *Journal of Water Supply Research and Technology - Aqua*, 45(4):171-183. 1996.

Clark, R.M., and W.M. Grayman. *Modeling Water Quality in Drinking Water Distribution Systems.* AWWA, Denver, CO. 1998.

Clark, R.M., W.M. Grayman, S.G. Buchberger, Y. Lee, and D.J. Hartman. "Drinking Water Distribution Systems: An Overview," in *Water Supply Systems Security*. Edited by Larry W. Mays, McGraw-Hill, NY. pp 4.1- 4.49. 2004

Clark, R.M., and D. L. Tippen. "Water Supply" in *Standard Handbook of Environmental Engineering.* Edited by Robert A. Corbitt, McGraw-Hill, NY. pp 5.173-5.220. 1990.

Connell, G.F. "European Water Disinfection Practices Parallel U.S. Treatment Methods," *Drinking Water and Health Quarterly*. August 1, 1998. http://www.clo2.com/reading/waternews/european.html

EPA. *The Clean Water and Drinking Water Infrastructure Gap Analysis.* EPA - Office of Water, Washington DC. September 2002.

EPA. *25 Years of the Safe Drinking Water Act: History & Trends.* EPA 816-R-99-007. December 1999.

Fair, G.M., and J.C. Geyer. "Water Supply and Waste-Water Disposal," John Wiley and Sons, Inc., NY. 1971.

Friedman, M., G. Kirmeyer, G. Pierson, S. Harrison, K. Martel, A. Sandvig, and A. Hanson. *Development of Distribution System Water Quality Optimization Plans.* AwwaRF/AWWA. Denver, CO. 2005.

Geldreich, E.E., H.D. Nash, D.J. Reasoner, and R.H. Taylor. "The Necessity of Controlling Bacterial Populations in Potable Water: Community Water Supply," *Journal of AWWA,* 64:596-602. 1972.

Greater Cincinnati Water Works. "Water Treatment," http://www.cincinnati-oh.gov/water/pages/-3283-/. 2005.

Grigg, NS. *Assessment and Renewal of Water Distribution Systems.* AwwaRF Report # 91025F, June 2005.

Gullick, R.W., W.M. Grayman, R.A. Deininger, and R.M. Males. "Design of Early Warning Monitoring Systems for Source Waters," *Journal of AWWA*, 95(11):58-72. 2003.

Insurance Services Office (ISO). *Fire Suppression Rating Schedule.* Insurance Services Office, NY. 2003.

International Life Sciences Institute. *Early Warning Monitoring to Detect Hazardous Events in Water Supplies.* Edited by T.M. Brosnan, Washington, D.C. 1999.

Kirmeyer, G.J., W. Richards, and C. Dery-Smith. *An Assessment of Water Distribution Systems and Associated Needs*. AwwaRF. 1994.

LeChevallier, M.W., T.M. Babcock, and R.G. Lee. "Examination and Characterization of Distribution System Biofilms," *Applied and Environmental Microbiology*, 53:2714-2724. 1987.

Maul, A., A.H. El-Shaarawi, and J.C. Block. "Heterotrophic Bacteria in Water Distribution Systems - I. Spatial and Temporal Variation." *The Science of the Total Environment*, 44:201-214. 1985a.

Maul, A., A.H. El-Shaarawi, and J.C. Block. "Heterotrophic Bacteria in Water Distribution Systems - II. Sampling Design for Monitoring." *The Science of the Total Environment*, 44:215-224. 1985b.

Montana State University (MSU) Center for Biofilm Engineering. Image provided by Pat Dirckx. 2005.

National Fire Protection Association (NFPA). *Fire Protection Handbook, 19th edition.* Edited by A.E. Cote, National Fire Protection Association, Quincy, MA. 2003.

NRC. *Public Water Supply Distribution Systems: Assessing and Reducing Risks – First Report.* The National Academies Press, Washington, DC. 2005.

Okun, D. "Distributing reclaimed water through dual systems," *Journal of AWWA,* 89 (11), pp. 52-64. 1996.

Owens, J. *A Review of Federal Drinking Water Regulations in the U.S., in Controlling Disinfection By-Products and Microbial Contaminants in Drinking Water.* Edited by Robert M. Clark and Brenda Boutin, EPA-600-R-01-110. pp 2-1 to 2-14. December 2001.

PDD 63. *Critical Infrastructure Protection*. The White House, Washington D.C. May 22, 1998.

PL107-188. Public Health Security and Bioterrorism Preparedness and Response Act of 2002.

Safe Drinking Water Act (SDWA), 1974, Public Law 93-523.

Walski, T. M, D.V. Chase, D.A. Savic, W.M. Grayman, S. Beckwith, and E. Koelle. *Advanced Water Distribution Modeling and Management.* Haestad Press, Waterbury, CT. pp 1-4. 2003.

Water Research Centre. "Deterioration of Bacteriological Quality of Water During Distribution," Notes on Water Research No. 6. 1976.

Zhang, W., and F.A. DiGiano. "Comparison of Bacterial Regrowth in Distribution Systems Using Free Chlorine and Chloramine: A Statistical Study of Causative Factors." *Water Research*, 36:6:1469-1482. 2002.

Chapter 2
Modeling Water Quality in Drinking Water Distribution Systems

This chapter covers the use of models to simulate the flow and water quality conditions in a distribution system network. Models are mathematical or physical approximations of a real-world system and can be used to study the behavior of actual system(s). A variety of computer software modeling tools are now available to perform these simulations. These tools are now commonly used by trained engineers and scientists to study and improve water distribution system network design and operation.

Water distribution system models have become widely accepted within the water utility industry as a mechanism for simulating the hydraulic and water quality behavior in water distribution system networks. Current water distribution modeling software is powerful, sophisticated and user-friendly. Many software packages are integrated with GIS and Computer Aided Design (CAD) technology in order to facilitate model construction and storage and display of model results. Early network models simulated only steady-state hydraulic behavior. In the 1970s, modeling capability was expanded to include Extended Period Simulation (EPS) models that could accommodate time-varying demand and operations. Subsequently, in the early 1980s, investigators began introducing the concept of water quality modeling. Most water distribution system modeling software packages now routinely incorporate water quality simulation capability. More recently, transient models for simulating water hammer (a transient phenomenon) and tank mixing/aging models have either been incorporated into or integrated with water distribution system models. Algorithms have been developed that enable users to optimize water system design and operation, assist in model calibration, and perform probabilistic analyses. Each of these model types are briefly described later in this chapter.

Water distribution system models are more commonly being used to replicate the behavior of a real or proposed system for a variety of purposes including: capital investment decisions, development of master plans, estimation of fire protection capacity, design of new systems and extension or rehabilitation of existing systems, energy management, water quality studies, various event simulations and analysis, optimal placement of sensors, and daily operations. The costs associated with constructing and maintaining a distribution system model may be more easily justified if it is used for a variety of applications by a water utility (Grayman, 2000).

2.1 Distribution System Network Hydraulic Modeling

The network hydraulic model provides the foundation for modeling water quality in distribution systems. This subsection provides a brief history of hydraulic modeling, an overview of theoretical concepts, basic model inputs, and general criteria for selection and application.

2.1.1 History of Hydraulic Modeling

Hardy Cross first proposed the use of mathematical methods for calculating flows in complex networks (Cross, 1936). This manual, iterative procedure was used throughout the water industry for almost 40 years. With the advent of computers and computer-based modeling, improved solution methods were developed for utilizing the Hardy Cross methodology. The improved implementations of this method were in widespread use by the 1980s (Wood, 1980a).

Also, in the early 1980s, the concept of modeling water quality in distribution system networks was developed based on steady-state formulations (Clark et al., 1986). By the mid-1980s, water quality models were developed that incorporated the dynamic behavior of water networks (Grayman et al., 1988). The usability of these models was greatly improved in the 1990s with the introduction of the public domain EPANET model (Rossman, 2000) and other Windows-based commercial water distribution system models.

Initially, hydraulic models simulated flow and pressures in a distribution system under steady-state conditions where all demands and operations remained constant. Since system demands (and consequently the flows in the water distribution network) vary over the course of a day, EPS models were developed to simulate distribution system behavior under time-varying demand and operational conditions. These models have now become ubiquitous within the water industry and are an integral part of most water system design, master planning, and fire flow analyses.

2.1.2 Overview of Theoretical Concepts

The theory and application of hydraulic models is thoroughly explained in many widely available references (Walski et al., 2003; American Water Works Association, 2004; Larock et al., 2000). Essentially, three basic relations are used to calculate fluid flow in a pipe network. These relationships are:

- Conservation of Mass: This principle requires that the sum of the mass flows in all pipes entering a junction must equal the sum of all mass flows leaving the junction. Because water is essentially an incompressible fluid, conservation of mass is equivalent to conservation of volume.
- Conservation of Energy: There are three types of energy in a hydraulic system: kinetic energy associated with the movement of the fluid, potential energy associated with the elevation, and pressure energy. In water distribution networks, energy is referred to as "head" and energy losses (or headlosses) within a network are associated primarily with friction along pipe walls and turbulence.
- Pipe Friction Headloss: A key factor in evaluating the flow through pipe networks is the ability to calculate friction headloss (Jeppson, 1976). Three empirical equations commonly used are the Darcy-Weisbach, the Hazen-Williams, and the Manning equations. All three equations relate head or friction loss in pipes to the velocity, length of pipe, pipe diameter, and pipe roughness. A fundamental relationship that is important for hydraulic analysis is the Reynolds number, which is a function of the kinematic viscosity of water (resistance to flow), velocity, and pipe diameter. The most widely used headloss equation in the U.S. is the Hazen-Williams equation. Though the Darcy Weisbach equation is generally considered to be theoretically more rigorous, the differences between the use of these two equations is typically insignificant under most circumstances.

A distribution system is represented in a hydraulic model as a series of links and nodes. Links represent pipes whereas nodes represent junctions, sources, tanks, and reservoirs. Valves and pumps are represented as either nodes or links depending on the specific software package. Figure 2-1 illustrates a simple link-node representation of a water distribution system.

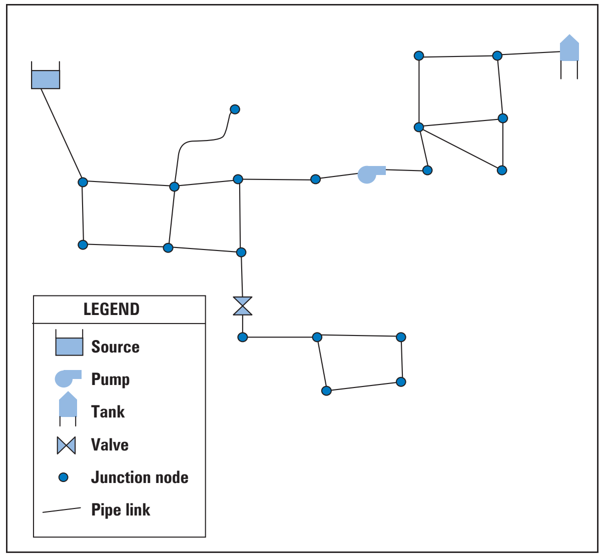


Figure 2-1. Simple Link-Node Representation of a Water Distribution System.

As mentioned previously, there are two types of analyses that may be conducted on drinking water distribution systems: steady-state and EPS. In a steady-state analysis, all demands and operations are treated as constant over time and a single solution is generated. In the EPS mode, variations in demand, tank water levels, and other operational conditions are simulated by a series of steady-state analyses that are linked together. Each steady-state solution in the EPS mode involves a separate solution of the set of non-linear equations. EPS is used as the basis for

Hydraulic models represent the basic underlying equations (conservation of mass and conservation of energy) as a series of linear and non-linear equations. Because of the non-linearity, iterative solution methods are commonly used to numerically solve the set of equations. The most common numerical method utilized is the Newton-Raphson method.

Conservation of Mass: The conservation of mass principle for hydraulic analysis requires that the sum of the mass flow in all pipes entering a junction must equal the sum of all mass flows leaving the junction. In EPS, if storage is involved, a term for describing the accumulation of water at those nodes is included. Mathematically, the principle can be represented as follows:

$$\sum_{i=1}^{n}(Q_i - U_i) - \frac{dS}{dt} = 0 \quad \text{(Equation 2-1)}$$

where

Q_i = inflow to node in i-th pipe in ft^3/sec (m^3/sec)

U_i = water used or leaving at the i-th node in ft^3/sec (m^3/sec)

$\frac{dS}{dt}$ = change in storage in ft^3/sec (m^3/sec) water quality modeling. Though the EPS solution does introduce some approximations and ignores the transient phenomena resulting from sudden changes (e.g., a pump being turned on), these more refined assumptions are generally not considered significant for most distribution system studies.

Conservation of Energy: The conservation of energy principle requires that the difference in energy between two points in a network must be the same regardless of flow path. For hydraulic analysis, this principle can be represented in terms of head as follows:

$$Z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + \sum h_P = Z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + \sum h_L + \sum h_M \quad \text{(Equation 2-2)}$$

where

$Z_{1\text{ and }2}$ = elevation at points 1 and 2, respectively, in ft (m)

$P_{1\text{ and }2}$ = pressure at points 1 and 2, respectively, in lb/ft^2 (N/m^2)

γ = fluid (water) specific weight, in lb/ft^3 (N/m^3)

$V_{1\text{ and }2}$ = velocity at points 1 and 2, respectively, in ft/s (m/s)

g = acceleration due to gravity, in ft/sec^2 (m/sec^2)

h_P = pumping head gain, in ft (m)

h_L = head loss in pipes, in ft (m)

h_M = head loss due to minor losses, in ft (m)

Pipe-friction headloss: The equation most commonly used in modeling software for computation of pipe-friction headloss is the Hazen-Williams equation represented as follows:

$$h_L = \frac{C_f L}{C^{1.85} D^{4.87}} Q^{1.85} \quad \text{(Equation 2-3)}$$

where

h_L = head loss due to friction, in ft (m)

C_f = Unit conversion factor
(4.73 in British units; 10.7 in Metric units)

D = pipe diameter, in ft (m)

L = length of pipe, in ft (m)

Q = pipe flow rate, in ft^3/sec (m^3/sec)

C = Hazen-Williams coefficient (dimensionless)

2.1.3 Basic Hydraulic Model Input Characterization

Building a network model, particularly if a large number of pipes are involved, is a complex process. The following categories of information are needed to construct a hydraulic model:

- Characteristics of the pipe network components (pipes, pumps, tanks, valves).
- Water use (demands) assigned to nodes (temporal variations required in EPS).
- Topographic information (elevations assigned to nodes).
- Control information that describes how the system is operated (e.g., mode of pump operation).
- Solution parameters (e.g., time steps, tolerances as required by the solution techniques).

Commonly used methods for these inputs are briefly described in the following subsections.

2.1.3.1 Pipe Network Inputs

Construction of the pipe network and its characteristics may be done manually or through use of existing spatial databases stored in GIS or CAD packages. Most commonly, GIS or CAD packages are used in this process and are described in more detail in Chapter 6. The initial step in constructing a network model is to identify pipes to be included in the model. Nodes are usually placed at pipe junctions, or at major facilities (tanks, pumps, control valves), or where pipe characteristics change in diameter, "C"-value (roughness), or material of construction. Nodes may also be placed at locations of known pressure or at sampling locations or at locations where water is used (demand nodes). The required pipe network component information includes the following:

- pipes (length, diameter, roughness factor),
- pumps (pump curve),
- valves (settings), and
- tanks (cross section information, minimum and maximum water levels).

2.1.3.2 Water Demand Inputs

Water consumption or water demand is the driving force behind the operation of a water distribution system. Any location at which water leaves the system can be characterized as a demand on the system. The water demands are aggregated and assigned to nodes, which represents an obvious simplification of real-world situations in which individual house taps are distributed along a pipe rather than at junction nodes. It is important to be able to determine the amount of water being used, where it is being used, and how this usage varies with time (Walski et al., 2003). Demand may be estimated by a count of structures of different types using a representative consumption per structure, meter readings and the assignment of each meter to a node, and to general land use. A universal adjustment factor should be used to account for losses and other unaccounted water usage so that total usage in the

Early software packages limited the number of pipes that could be included due to computer storage restrictions. This led to the concept of "skeletonizing" a network or including only those pipes that were considered to be the most important. The degree of skeletonization that is acceptable should depend upon the ultimate use of the model. For example, master plans and energy studies might be based on the use of skeletonized networks. Other applications, such as water quality modeling and designing flushing programs, require a model that includes more pipes. Though there is no national standard for skeletonization, the EPA draft guidance issued for modeling to support the IDSE under DBPR2 suggests inclusion of (EPA, 2003):

- At least 50 percent of total pipe length in the distribution system.
- At least 75 percent of the pipe volume in the distribution system.
- All 12-inch diameter and larger pipes.
- All 8-inch and larger pipes that connect pressure zones, influence zones from different sources, storage facilities, major demand areas, pumps, and control valves, or are known or expected to be significant conveyors of water.
- All 6-inch and larger pipes that connect remote areas of a distribution system to the main portion of the system.
- All storage facilities with controls or settings applied to govern the open/closed status of the facility that reflect standard operations.
- All active pump stations with realistic controls or settings applied to govern their on/off status that reflect standard operations.
- All active control valves or other system features that could significantly affect the flow of water through the distribution system (e.g., interconnections with other systems, valving between pressure zones).

A case study presented in Section 7.3.1 illustrates the use of models in support of IDSE.

Most modern software packages support an unlimited number of pipes; however, skeletonization is still frequently used in order to reduce the modeling effort. A minimal skeletonization should include all pipes and features of major concern.

model corresponds to total production.

In order to use a model in the EPS mode, information on temporal variations in water usage over the period being modeled are required. Spatially different temporal patterns can be applied to the individual network nodes. The best available information should be used for developing temporal patterns in order to make EPS most effective. For example, some users may have continuous water metering data, while others may use literature values as a first approximation for estimating residential temporal patterns. Temporal patterns also vary with climate. For example, lawn watering in summer months will cause a spike in usage of water during that time period. In some cases, information from SCADA systems can be used to estimate system-wide temporal patterns.

A typical hierarchy for assigning demands includes the following:

- Baseline Demands: Baseline demands usually correspond to consumer demands and unaccounted-for-water associated with average day conditions. This information is often acquired from a water utility's existing records, such as customer meter and billing records. Although the spatial assignment of these demands is extremely important and should include the assignment of customer classes such as industrial, residential, and commercial use, actual metering data should be used when available.
- Seasonal Variation: Water use typically varies over the course of the year with higher demands occurring in warmer months. When developing a steady-state model, the baseline (average day) demand can be modified by multipliers in order to reflect other conditions such as maximum day demand, peak-hour demand, and minimum day demand.
- Fire Demands: Water provided for fire services can be the most important consideration in developing design standards for water systems. Typically, a system is modeled corresponding to maximum-use conditions, with needed fire-flow added to a single node at a time. It is not uncommon for a requirement that multiple hydrants be flowing simultaneously.
- Diurnal Variation: All water systems are unsteady due to continuously varying demands. It is important to account for these variations in order to achieve an adequate hydraulic model. Diurnal varying demand curves should be developed for each major consumer class or geographic zones within a service area. For example, diurnal demand curves might be developed for industrial establishments, commercial establishments, and residences. Large users such as manufacturing facilities may have unique usage patterns.

Future water use: For design and planning purposes, a water system must be examined under future conditions. In situations where a system is largely currently built out, future demands may be estimated by developing global or regional multipliers that are applied to current demands. However, in new or developing areas, existing water use does not provide a useful basis for estimating future demands. Alternative approaches use population-based projections, socioeconomic modeling, and land-use methods (Johnson and Loux, 2004).

In estimating future demands for use in a network model, the most appropriate method is generally the land-use method. The land-use method is based on mapping land uses and then applying a water-use factor to each land-use category. When applied to existing situations or in historical reconstruction of water systems, aerial photographs are most commonly used as the base map for identifying land-use categories. For development of future demands, land–use maps can be obtained from planners. The land-use methodology is depicted in Figure 2-2.

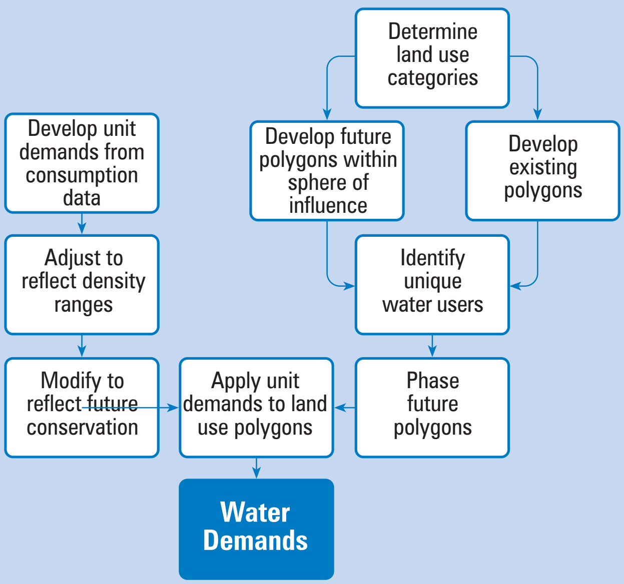


Figure 2-2. A Flow Chart for Estimating Future Water Demand Based on Land-Use Methodology.

Land-use unit demands or water-use factors are typically developed in units of gallons per day (GPD) per acre from local historical consumption data or from available regional information. GIS technology is frequently used as a means of developing and manipulating the land-use polygons and assigning the calculated demands to the model nodes.

2.1.3.3 Topographical Inputs

Hydraulic models use elevation data to convert heads to pressure. Actual pipe elevations should be used to establish the correct hydraulic gradeline. Elevations are assigned to each node in a network where pressure information is required. Various techniques are used to determine elevation information including the following:

- Topographical maps: Paper topographical maps produced by the United States Geological Survey (USGS) or other local agencies may be used to manually interpolate elevations for nodes. The relative accuracy depends upon the degree of topography in the area, the contour elevations on the map, and the manual takeoff methods used.
- Digital elevation models (DEM): USGS and other agencies produce digital files containing topographical information. When used with various software tools, elevation information can be directly interpolated and assigned to nodes based on the coordinates of the nodes. The accuracy of this process depends upon the degree of detail in the DEM.
- Global Positioning Systems (GPS) or other field survey methods: Standard field surveying techniques or modern surveying methods using a GPS satellite can be used to measure elevations at nodes. The modern GPS units can calculate elevation by using four or more satellites. However, elevation is the most difficult calculation for a GPS unit, and depending upon the location surveyed, it may be prone to significant error.

2.1.3.4 Model Control Inputs

In order to apply an EPS model, it is necessary to define a set of rules that tells the model how the water system operates. This may be as simple as specifying that a particular pump operates from 7:00 AM to 10:00 AM each day. Alternatively, it may be a set of complex "logical controls" in which operations such as pump off/on, pump speed, or valve status are controlled using Boolean operators (including if-then-else logic) for factors such as tank water levels, node pressures, system demand, and time of day (Grayman and Rossman, 1994). For water systems that operate automatically based on a set of rules, determination of these rules are quite straightforward. For manual systems, the rules must be determined by interviews with system operators.

2.1.3.5 Extended Period Simulation (EPS) Solution Parameters

Solution techniques used to iteratively solve the set of non-linear equations typically have various global parameters that control the solution technique. These parameters may be time-step lengths for EPS runs or tolerance factors that tell the model when a solution is considered to have converged. The user must specify the values for the solution parameters, or (as is frequently done) accept the default values that are built into the software products. The specific solution parameters vary between solution techniques and specific software products.

2.1.4 General Criteria for Model Selection and Application

The initial step in modeling is to define the basic scope and needs of the modeling process and to select an appropriate software package that will satisfy both the specific needs of the current project and likely future needs. Factors that may enter into the selection of a software package include:

- technical features,
- training/support and manuals,
- user interface,
- integration with other software (such as GIS, CAD),
- compatibility with EPANET,
- cost, and
- response from existing users.

A summary of major available hydraulic-water quality modeling software is provided in Section 2.3.2. Once a suitable model has been selected, the following steps should be followed in applying network models (Clark and Grayman, 1998):

- Develop the basic network model.
- Calibrate and validate the model.
- Establish clear objectives and apply the model in a manner to meet the objectives.
- Analyze and display the results.

2.1.4.1 Developing a Basic Network Model

The basic network model inputs should be first characterized using the techniques described in Section 2.1.3. The model should be developed based on accurate, up-to-date information. Information should be entered carefully and checked frequently. Following the entry of the data, an initial run of the model should be made to check for reasonableness.

2.1.4.2 Model Calibration and Validation

Calibration is an integral aspect of the art of modeling water distribution systems. Model calibration is the process of adjusting model input data (or, in some cases, model structure) so that the simulated hydraulic and water quality output sufficiently mirrors observed field data. Depending on the degree of accuracy desired, calibration can be difficult, costly, and time-consuming. The extent and difficulty of calibration are minimized by developing an accurate set of basic inputs that provide a good representation of the real network and its components.

A traditional technique for calibration is the use of "fire-flow" tests. In a fire-flow test, the system is stressed by opening hydrants to increase flows in small parts of the system. This results in increased headloss in pipes in the vicinity of the test. Pressures and flow are then measured in the field. Model parameters, such as roughness factors (C), demands, and valve positions, are adjusted so that the model adequately reflects the field data. Another common calibration technique is to measure predicted tank/reservoir levels derived from computer simulations against actual tank levels during a given period of record. For example, using water level, pressure, or flow data from SCADA systems or from on-line pressure and tank-level recorders, model parameters (such as roughness, water demands, and pump controls) can be adjusted in the simulation model until the model results match the actual tank level and other continuous information for the defined criteria. The resulting optimal parameter values should be checked to ensure that the values are realistic. Sophisticated commercial hydraulic models, such as those listed in Section 2.4, may also incorporate optimization components that aid the user in selecting system parameters resulting in the best match between observed system performance and model results (Walski, 2003).

Model validation is the step that follows calibration and uses an independent field data set to verify that the model is well calibrated. In the validation step, the calibrated model is run under conditions differing from those used for calibration and the results compared to field data. If the model results closely approximate the field results (visually) for an appropriate time period, the calibrated model is considered to be validated. Significant deviations indicate that further calibration is required. A variety of calibration and validation techniques suitable to both large and small water utilities are discussed in Chapter 4 of this document.

> Another rigorous methodology for calibration and validation is the use of tracers. Concentrations of naturally occurring materials or added chemical tracers may be measured in the field and the results used to calibrate hydraulic and water quality models. This methodology is further described in Chapter 3 of this document.

2.1.4.3 Establishing Objectives and Model Application

Prior to applying the model, the specific modeling objectives should be clearly established. The objectives may include specification of particular water demand and operational modes. Based on these specifications, a series of scenarios can be defined and the model applied appropriately. Some software products contain a scenario manager that helps the user to define and manage a large number of specific model runs. Additional scenarios can be developed in order to test the sensitivity of the system to variations in model parameters that are not known with certainty.

2.1.4.4 Analysis and Display of Results

Water distribution system models generate a large amount of output. The amount of calculated information increases with increasing model size and, for EPS, the duration of the model run. Modern water distribution system analysis software typically provides a range of graphical and tabular displays that help the user wade through the large amount of output data so that it may be efficiently analyzed. Figures 2-3, 2-4, and 2-5 contain examples of various graphical and tabular outputs generated by the EPANET software. These outputs represent a small subset of types of graphics generated by most modeling software. The output should be analyzed to ensure that the model is operating properly and to extract the information required in order to analyze the specific problem being studied.

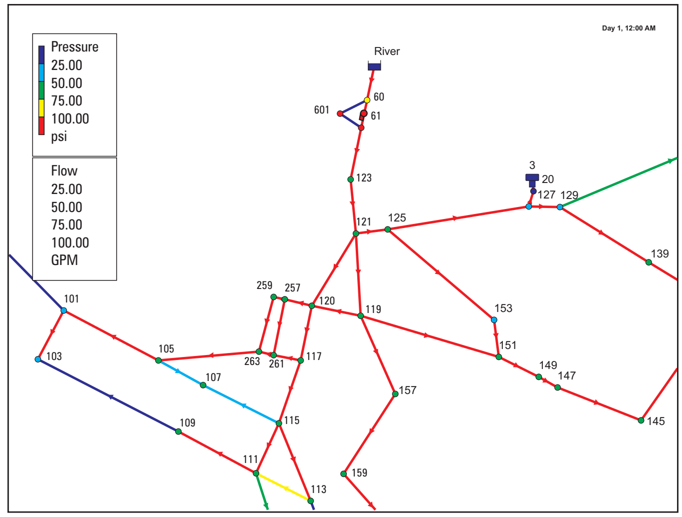


Figure 2-3. EPANET Graphical Output Showing Flow and Pressure.

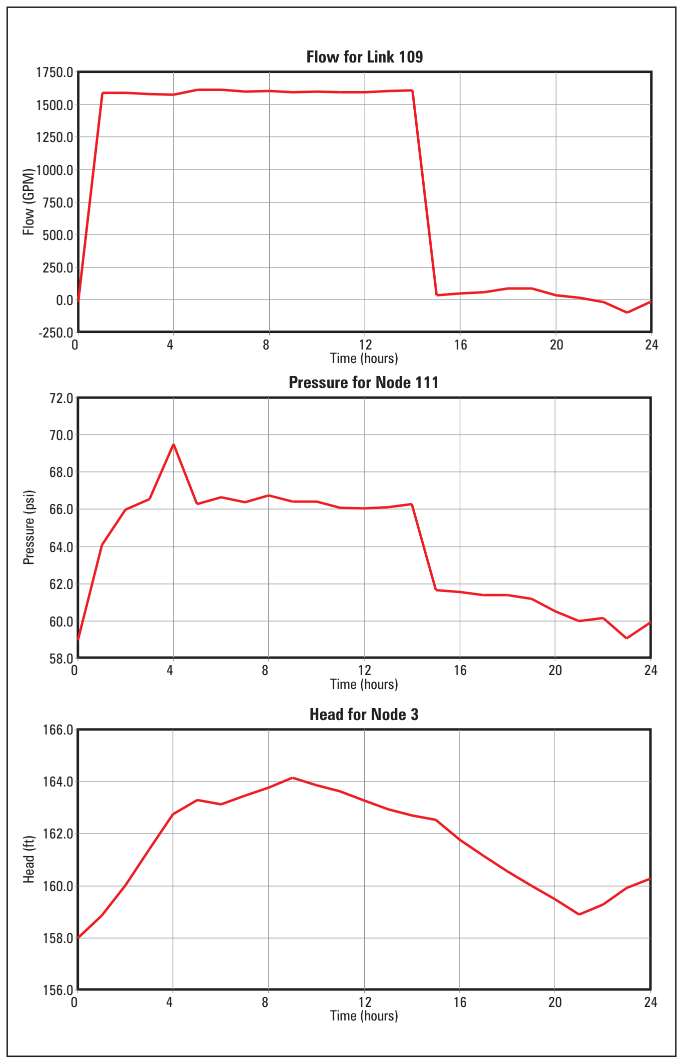


Figure 2-4. Sample EPANET Time Series Plots of Flow, Pressure, and Tank Water Level.

2.2 Modeling Water Quality In Distribution System Networks

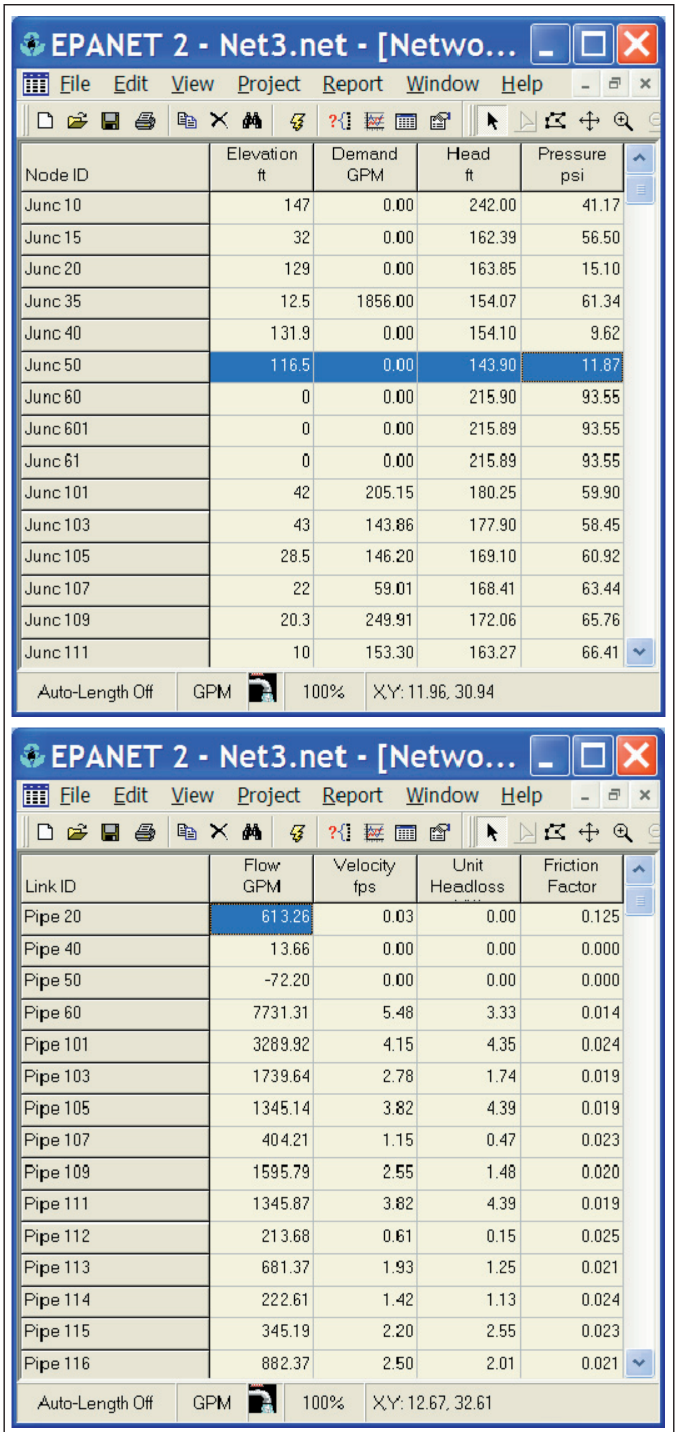
Water quality models use the output of hydraulic models in conjunction with additional inputs (described later in this section) to predict the temporal and spatial distribution of a variety of constituents within a distribution system. These constituents include:

- The fraction of water originating from a particular source.
- The age of water (e.g., duration since leaving the source).
- The concentration of a non-reactive constituent or tracer compound either added to or removed from the system (e.g., chloride or fluoride).
- The concentration of a reactive compound including the concentration of a secondary disinfectant with additional input of its loss rate (e.g., chlorine or chloramines) and the concentration of disinfection by-products with their growth rate (e.g., THMs).

The following subsection provides a brief history of water quality modeling, an overview of theoretical concepts related to water quality modeling, basic model inputs, and model application.

2.2.1 History of Water Quality Modeling

The use of models to determine the spatial pattern of



Node ID	Elevation ft	Demand GPM	Head ft	Pressure psi
Junc 10	147	0.00	242.00	41.17
Junc 15	32	0.00	162.39	56.50
Junc 20	129	0.00	163.85	15.10
Junc 35	12.5	1856.00	154.07	61.34
Junc 40	131.9	0.00	154.10	9.62
Junc 50	116.5	0.00	143.90	11.87
Junc 60	0	0.00	215.90	93.55
Junc 601	0	0.00	215.89	93.55
Junc 61	0	0.00	215.89	93.55
Junc 101	42	205.15	180.25	59.90
Junc 103	43	143.86	177.90	58.45
Junc 105	28.5	146.20	169.10	60.92
Junc 107	22	59.01	168.41	63.44
Junc 109	20.3	249.91	172.06	65.76
Junc 111	10	153.30	163.27	66.41

Link ID	Flow GPM	Velocity fps	Unit Headloss	Friction Factor
Pipe 20	613.26	0.03	0.00	0.125
Pipe 40	13.66	0.00	0.00	0.000
Pipe 50	-72.20	0.00	0.00	0.000
Pipe 60	7731.31	5.48	3.33	0.014
Pipe 101	3289.92	4.15	4.35	0.024
Pipe 103	1739.64	2.78	1.74	0.019
Pipe 105	1345.14	3.82	4.39	0.019
Pipe 107	404.21	1.15	0.47	0.023
Pipe 109	1595.79	2.55	1.48	0.020
Pipe 111	1345.87	3.82	4.39	0.019
Pipe 112	213.68	0.61	0.15	0.025
Pipe 113	681.37	1.93	1.25	0.021
Pipe 114	222.61	1.42	1.13	0.024
Pipe 115	345.19	2.20	2.55	0.023
Pipe 116	882.37	2.50	2.01	0.021

Figure 2-5. EPANET Sample Tabular Outputs (at time 10.00 hrs).

water quality in a distribution system resulting from sources of differing quality was suggested by Wood (1980b) in a study of slurry flow in a pipe network. The steady-state hydraulic model was extended by solving a series of simultaneous equations at each node. In a generalization of this formulation, Males et al., (1985) used simultaneous equations to calculate the spatial distribution of variables that could be associated with links and nodes such as concentration, travel times, costs, and others. This model, called SOLVER, was a component of the Water Supply Simulation Model (WSSM), an integrated data base management, modeling, and display system that was used to model water quality in networks (Clark and Males, 1986). A more general "marching out" solution was proposed by Males et al., (1988). Although steady-state water quality models provided some general understanding of water quality behavior in distribution systems, the need for models that would represent contaminant dynamics was recognized. This resulted in the introduction of three such dynamic models in the mid-1980s (Clark et al., 1986; Liou and Kroon, 1986; and Hart et al., 1986).

The ability to model the transport and fate of the water constituents in a distribution system can help utility managers perform a variety of water quality studies. Examples include:

- Locating and sizing storage tanks and modifying system operation to reduce water age.
- Modifying system design and operation to provide a desired blend of waters from different sources.
- Finding the best combination of: i) pipe replacement, relining, and cleaning; ii) reduction in storage holding time; iii) location and injection rate of booster stations to maintain desired disinfectant levels throughout the system.
- Assessing and minimizing the risk of consumer exposure to disinfectant by-products.
- Assessing system vulnerability to incidents of external contamination.
- Designing a cost-efficient, routine monitoring program to identify water quality variations and potential problems.

The history and proliferation of water quality modeling in distribution systems can be traced back to two expert workshops that were convened in 1991 and in 2003. The results of these workshops are presented in AWWARF/USEPA (1991) and Powell et al., (2004). Figure 2-6 illustrates the evolution of hydraulic and water quality models since the 1930s.

2.2.2 Theoretical Concepts for Water Quality Modeling

Various water quality processes are occurring in water distribution systems that can lead to introduction of contaminants and water quality transformations (see Figure 1-2, presented earlier in Chapter 1) as water moves through the distribution system. Cross connections, failures at the treatment barrier, and

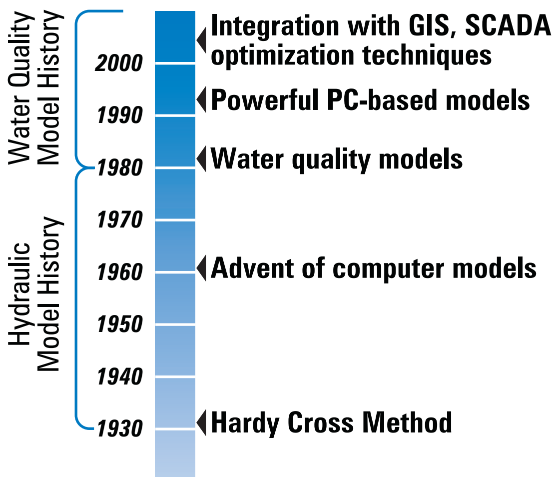


Figure 2-6. Illustration of the Evolution of Hydraulic and Water Quality Models.

transformations in the bulk phase can all degrade water quality. Corrosion, leaching of pipe material, biofilm formation, and scour can occur at the pipe wall to degrade water quality. Bacteriological quality changes may cause aesthetic problems involving taste and odor development, discolored water, and other adverse impacts.

In addition to the basic hydraulic modeling equations presented earlier in this chapter, the water quality models utilize various mathematical equations that are based on conservation of constituent mass. These models represent the following phenomena occurring in a distribution system (Rossman et al., 2000):

- Advective transport of mass within pipes: A dissolved substance will travel down the length of a pipe with the same average velocity as the carrier fluid while at the same time reacting (either growing or decaying) at some given rate. Longitudinal dispersion is not an important transport mechanism in turbulent flow, which is normal inside transmission mains under most operating conditions. It may, however, be an important factor in dead-end pipes or in low and intermittent flow scenarios.
- Mixing of mass at pipe junctions: All water quality models assume that, at junctions receiving inflow from two or more pipes, the mixing of fluid is complete and instantaneous. Thus, the concentration of a substance in water leaving the junction is simply the flow-weighted sum of the concentrations in the inflowing pipes.
- Mixing of mass within storage tanks: Most water quality models assume that the contents of storage tanks are completely mixed. See the discussion in Section 2.4.1 for further details and alternative representations.
- Reactions within pipes and storage tanks: While a substance moves down a pipe or resides in storage, it can undergo reaction. The rate of reaction, measured in mass reacted per volume of water per unit of time, will depend on the type of water quality constituent being modeled. Some constituents, such as fluoride, do not react and are termed "conservative." Other constituents, such as chlorine residual, decay with time; while the generation of DBPs, such as THMs, may increase over time. Some constituents, such as chlorine, will react with materials both in the bulk liquid phase and at the liquid-pipe wall boundary.

Water quality models represent these phenomena (transport within pipes, mixing at junctions and storage tanks, and reaction kinetics in the bulk liquid phase and at the liquid-pipe wall boundary) with a set of mathematical equations. These equations are then solved under an appropriate set of boundary and initial conditions to predict the variation of water quality throughout the distribution system.

Several solution methods are available for dynamic water quality models (Rossman and Boulos, 1996). All of these methods require that a hydraulic analysis be run first to determine how flow quantities and directions change from one time period to another throughout the pipe network. The water quality constituent is subsequently routed through each pipe link and then mixed at downstream nodes with other inflows into the node. For non-conservative substances, concentrations are continuously adjusted to accommodate the decay or growth of the constituent with time. This concentration is then released from the node into its out-flowing pipes. This process continues for all pipes and for the duration of the simulation.

The methods described above are also applied when modeling water age and source-tracing in water quality models. Water age is equivalent to modeling a reactive constituent that ages and combines linearly. For example, for every hour that a "packet" of water spends in a tank, its age will increase by one hour. Additionally, combining a volume of water that is four days old with a similar volume of water that is eight days old will result in an average age of six days. When modeling the fraction of water coming from a designated source (source tracing), this parameter is modeled as a conservative substance and is linearly combined. For example, combining a volume of water that is entirely from the designated source with a similar volume of water from a different

Modeling the movement of a contaminant within the distribution systems as it moves through the system from various points of entry (e.g., wells or treatment plants) to water users is based on three principles:

- Conservation of mass within differential lengths of pipe.
- Complete and instantaneous mixing of the water entering pipe junctions.
- Appropriate kinetic expressions for the growth or decay of the substance as it flows through pipes and storage facilities.

This change in concentration can be expressed by the following differential equation:

$$\frac{dC_{ij}}{dt} = -v_{ij}\frac{\partial C_{ij}}{\partial x} + k_{ij}C_{ij} \qquad \text{(Equation 2-5)}$$

where

C_{ij} = substance concentration (mg/L) at position x and time t in the link between nodes i and j.

v_{ij} = flow velocity in the link (equal to the link's flow rate divided by its cross-sectional area) (m/sec)

k_{ij} = rate at which the substance reacts within the link (sec^{-1})

According to Equation 2-5, the rate at which the mass of material changes within a small section of pipe equals the difference in mass flow into and out of the section plus the rate of reaction within the section. It is assumed that the velocities in the links are known beforehand from the solution to a hydraulic model of the network. In order to solve Equation 2-5, one needs to know C_{ij} at x=0 for all times (a boundary condition) and a value for k_{ij}.

Equation 2-6 represents the concentration of material leaving the junction and entering a pipe:

$$C_{ij@x=0} = \frac{\sum_k Q_{ki}C_{kj@x=L}}{\sum_k Q_{kj}} \qquad \text{(Equation 2-6)}$$

where

$C_{ij@x=0}$ = the concentration at the start of the link connecting node i to node j in mg/L (i.e.,where x=0)

$C_{kj@x=L}$ = the concentration at the end of a link, in mg/L

Q_{kj} = flow from k to i

Equation 2-6 states that the concentration leaving a junction equals the total mass of a substance flowing into the junction divided by the total flow into the junction.

source will provide a mixed volume calculated as 50 percent from the designated source.

2.2.3 Water Quality Model Inputs and Application

In addition to the basic hydraulic model inputs described in Section 2.1.3, the water quality models require the following data elements to simulate the behavior in a distribution system:

- Water Quality Boundary Conditions - A water quality model requires the quality of all external inflows to the network and the water quality throughout the network be specified at the start of the simulation. Data on external inflows can be obtained from existing source monitoring records when simulating existing operations or could be set to desired values to investigate operational changes. Initial water quality values can be estimated based on field data. Alternatively, best estimates can be made for initial conditions. Then the model is run for a sufficiently long period of time under a repeating pattern of source and demand inputs so that the initial conditions, especially in storage tanks, do not influence the water quality predictions in the distribution system. The water age and source tracing options only require input from the hydraulic model.

- Reaction Rate Data – For non-conservative substances, information is needed on how the constituents decay or grow over time. Modeling the fate of a residual disinfectant is one of the most common applications of network water quality models. The two most frequently used disinfectants in distribution systems are chlorine and chloramines (a reactant of chlorine and ammonia). Free chlorine is more reactive than chloramine and its reaction kinetics have been studied more extensively. Studies have shown that there are two separate reaction mechanisms for chlorine decay, one involving reactions within the bulk fluid and another involving reactions with material on or released from the pipe wall (Vasconcelos et al., 1997). Bulk decay is typically represented as a first order exponential decay function with a single decay coefficient specified to represent the decay over time. In some circumstances, this function does not adequately represent the observed decay characteristics, and more complex formulations may be used to describe the decay. Wall reaction represents the disinfectant decay due to contact with oxidizeable substances at the pipe wall, such as corrosion products or biofilm. The most widely used approach for representing wall demand considers two interacting processes – transport

Storage tanks are usually modeled as completely mixed, variable volume reactors in which the changes in volume and concentration over time are as follows:

$$\frac{dV_s}{dt} = \sum_k Q_{ks} - \sum_i Q_{sj} \qquad \text{(Equation 2-7)}$$

$$\frac{dV_s C_s}{dt} = \sum_k Q_{ks} C_{ks@x=L} - \sum_i Q_{sj} C_s + k_{ij}(C_s) \qquad \text{(Equation 2-8)}$$

where

C_s = the concentration for tanks, in mg/L

dt = change in time, in seconds

Q_{ks} = flow from node k to s, in ft^3/sec (m^3/sec)

Q_{sj} = flow from node s to j, in ft^3/sec (m^3/sec)

dV_s = change in volume of tank at nodes, in ft^3(m^3)

V = volume of tank at nodes, in ft^3(m^3)

C_{ks} = concentration of contaminant in link ks, in mg/ft^3 (mg/m^3)

k_{il} = decay coeficient between nodes i and j, in sec^{-1}

Many algorithms and methods exist for the numerical solution of fluid flows described by the Navier-Stokes equations. These algorithms can be classified as Eulerian or Lagrangian and as either time-driven or event-driven. In a Eulerian method, the movement of the fluid is viewed from a stationary grid as the water moves through the system. On the contrary, in a Lagrangian method, the analysis is viewed from a framework that is moving with the flow. Time-driven methods assess the system at fixed time steps. Event-driven methods evaluate the system only when there is a discrete change in water quality such as a pulse of water with different concentrations entering or leaving a pipe. Various methodologies combine either Eulerian or Lagrangian solutions (or hybrid combinations of these two cases) with either time-driven or event-driven procedures.

of the disinfectant from the bulk flow to the wall and interaction with the wall (Rossman et al., 1994). Recent studies have suggested that this formulation may not adequately represent the actual wall demand processes and that further research is needed (Clark et al., 2005; Grayman et al., 2002; DiGiano and Zhang, 2004). There has been little study on the nature of the wall reaction in chloraminated systems. A limited amount of modeling of the growth of DBPs (most notably THMs) has been performed assuming an exponential growth approaching a maximum value corresponding to the THM formation potential. Both the formation potential and the growth rate constant must be specified in this type of model (Clark et al., 1996). There has been extensive research on biofilm formation in distribution systems and this has led to the development of several theoretical models of this phenomenon (Powell et al., 2004). However, these models are generally quite complex involving many parameters that are difficult to determine, and thus are not ready for inclusion in a general water distribution system model.

The following section provides an overview of available software for hydraulic and water quality modeling.

Distribution system water quality models are generally limited to tracking the dynamics of a single component (e.g., chlorine, water age) at a time when the selected component is transported throughout the network of pipes and storage tanks. Such models do not consider interactions between different components in the flowing water or complex reactions between components that are transported with the water and surface components that are fixed to the pipe wall. This can be a significant limitation when modeling reactive components, for example when chlorine residual is modeled for a case where there are multiple sources with significant differences in water quality characteristics. Another more complex example that is not adequately represented by the single-species model is modeling of DBP formation. A solution to this deficiency is a general-purpose, multi-species capability that is being added to EPANET (Uber et al., 2004). This addition will allow users to program their own chemical/physical/biological reactions in EPANET with almost unlimited interaction capability between multiple species.

2.3 Hydraulic and Water Quality Modeling Software

A variety of software packages are available to perform hydraulic and water quality modeling. A majority of these packages utilize the EPANET formulation as the basic computation engine. A full discussion of individual software packages is beyond the scope of this document. The following subsections briefly describe the EPANET model and summarize the features of other available software.

2.3.1 EPANET Software

EPANET was initially developed in 1993 as a distribution system hydraulic-water quality model to support research efforts at EPA (Rossman et al., 1994). The development of the EPANET software has also satisfied the need for a comprehensive public-sector model and has served as the hydraulic and water quality "engine" for many commercial models.

EPANET can be used for both steady-state and EPS hydraulic simulations. In addition, it is designed to be a research tool for modeling the movement and fate of drinking water constituents within distribution systems. EPANET can be operated in the SI (metric) or British systems of measurement.

The water quality routines in EPANET can be used to model concentrations of reactive and conservative substances, changes in age of water and travel time to a node, and the percentage of water reaching any node from any other node. Outputs from EPANET include:

- color-coded network maps,
- time series plots, and
- tabular reports.

Example outputs from EPANET were previously presented in Figures 2-3, 2-4, and 2-5.

2.3.2 Commercial Hydraulic-Water Quality Modeling Software

In addition to EPANET, there are several commercial software packages that are widely used in the U.S. and internationally. Most of these packages are based on the EPANET formulation and include value-added components as parts of GUI that increase the capability of the software. Examples of such value-added components that are part of one or more of the commercially available software packages include:

- Scenario manager: Manage inputs and outputs of a group of model runs.
- Calibration optimization: Utilize genetic algorithm optimization technique to determine model parameters that best fit a set of field data.
- Design optimization: Utilize genetic algorithm optimization techniques to select pipe sizes that minimize costs or other selected objectives.
- Integration with GIS or CAD: Water distribution model directly integrates with GIS or CAD to assist in constructing or updating model and display results.
- Flexible output graphics: Provides convenient ways to modify parameters for graphical displays of output data.
- Energy management: Calculates energy use for a selected alternative.
- Automated fire-flow analysis: Assesses the availability of fire flow at a range of nodes and determines whether a system meets fire-flow requirements.
- Water security and vulnerability assessment methods, skeletonization, and demand allocation tools.

Table 2-1 provides a summary listing of major commercial software and a Web link where additional details may be obtained on specific features and current version availability/pricing.

> In addition to the standard use of EPANET in a Windows environment using the graphical user interface (GUI), the functionality of EPANET can be accessed through the EPANET toolkit. The toolkit is a series of open source routines available in both Visual Basic and C (programming language) that can be used as is or modified and accessed from a user's own computer program. This powerful capability has been widely used throughout the world to support both research and specific applications in the field of water distribution system analysis.

2.4 Additional Modeling Tools

In addition to standard hydraulic and water quality modeling of distribution systems, there are several other related types of models that can be used to assess hydraulic and water quality behavior in distribution systems. These include: storage modeling tools, transient (water hammer) modeling tools, optimization tools, and probabilistic models. Each of these types of models are briefly described and demonstrated in the following sections.

2.4.1 Storage Modeling Tools

An important aspect of water quality and contaminant propagation in drinking water distribution systems is the effect of system storage. Most utilities use some type of ground or elevated storage system to process water during time periods when treatment facilities would otherwise be idle. It is then possible to distribute and store water at one or more locations in the service area closest to the user.

The principal advantage of distribution storage is that it equalizes demands on supply sources, production works, and transmission and distribution mains. As a result, the sizes or capacities of these elements may be minimized and peak power tariff periods for pumping can often be avoided. Additionally, system flows and pressures are improved and stabilized to better serve the customers throughout the service area. Finally, reserve supplies are provided in the distribution system for emergencies, such as fire fighting and power outages.

In most municipal water systems, less than 25 percent of the volume of the storage in tanks is actively used (on a daily basis) under routine conditions. As the

Table 2-1. Available Hydraulic and Water Quality Network Modeling Software Packages

Network Modeling Software	Company	EPANET Based	Website
AQUIS	Seven Technologies		www.7t.dk/aquis
EPANET	EPA	X	www.epa.gov/ord/nrmrl/wswrd/epanet.html
InfoWater H2ONET/H2OMAP	MWHSoft	X	www.mwhsoft.com
InfoWorks WS	Wallingford Software		www.wallingfordsoftware.com
MikeNet	DHI, Boss International	X	www.dhisoftware.com/mikenet
Pipe2000	University of Kentucky		www.kypipe.com
PipelineNet	SAIC, TSWG	X	www.tswg.gov/tswg/ip/pipelinenettb.htm
SynerGEE Water	Advantica		www.advantica.biz
WaterCAD/WaterGEMS	Haestad Methods	X	www.haestad.com
STANET	Fisher-Uhrig Engineering		www.stanet.net
Wadiso	GLS Eng. Software	X	www.wadiso.com

water level drops, tank controls require high-service pumps to start in order to satisfy demand and refilling of the tanks. The remaining water in the tanks (70 to 75 percent) is normally held in reserve as dedicated fire or emergency storage. This water tends to be stagnant and may cause water quality problems.

Storage tanks and reservoirs are the most visible components of a water distribution system, but are often the least understood in terms of their effect on water quality. Although these facilities can play a major role in providing hydraulic reliability for fire fighting needs and in providing reliable service, they may also serve as vessels for unwanted complex chemical and biological changes that may result in the deterioration of water quality. These storage tanks and reservoirs also contribute to increased residence time in drinking water systems. This increased residence time can contribute to the loss of disinfectant residuals and cause subsequent growth of microorganisms. Modeling can provide information on what will happen in existing, modified or proposed distribution system tanks and reservoirs under a range of operating situations (Grayman et al., 2004a).

Three primary types of models are used for representing storage tanks and reservoirs: computational fluid dynamics (CFD) models, compartment models, and physical scale models. In mathematical models, equations are written to simulate the behavior of water in a tank or reservoir. These models range from detailed representations of the hydraulic mixing phenomena in the facility called CFD models to simplified conceptual representations of the mixing behavior called compartment or systems models. Physical scale models are constructed from materials such as wood or plastic. Dyes or chemicals are used to trace the movement of water through the model.

2.4.1.1 CFD Models

CFD models use mathematical equations to simulate flow patterns, heat transfer, and chemical reactions. Partial differential equations representing conservation of mass, momentum, and energy are solved numerically for a two- or three-dimensional grid that approximates the geometry of the tank. CFD modeling has been used widely in the chemical, nuclear, and mechanical engineering fields, and in recent years has emerged as a modeling tool in the drinking water industry (Grayman and Arnold, 2003). CFD models can be used to simulate temperature variations, unsteady hydraulic and water quality conditions, and decay of constituents in storage facilities. Significant experience is required to apply CFD models, and model run times of many hours, days, or even weeks are required for complex situations. Figure 2-7 depicts a graphical output from a CFD model showing the concentration throughout a tank at a snapshot in time resulting from a tracer that has been injected into the inflow.

Many generalized CFD software packages are available that can be used to construct CFD models of tanks. Examples of such packages are listed in Table 2-2. These packages vary in terms of capabilities, solution methods, ease of use, and support. Prior to selection of a package, the specific needs and capabilities of the user should be carefully evaluated.

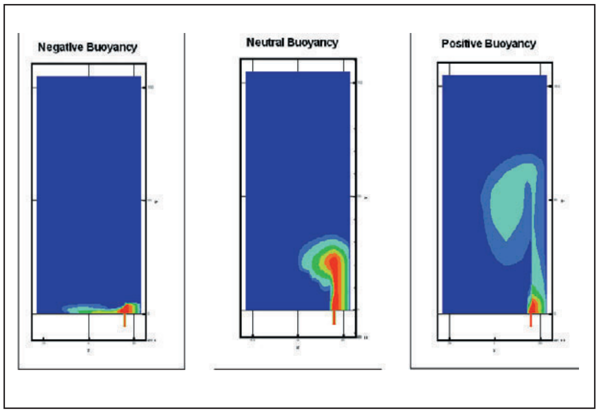


Figure 2-7. Graphical Output from a CFD Model Showing Tracer Concentration in a Tank.

Generally the purchase or lease of these packages is significant (typically on the order of $25,000 per year) and significant training/expertise is required to effectively apply them.

2.4.1.2 Compartment Models

Compartment models are a class of models in which physical processes (i.e., the mixing phenomena in the tank or reservoir) are represented by highly conceptual, empirical relationships. This type of model is also referred to as a black box model, or input-output model. Since such models do not use detailed mathematical equations to describe the movement of water within the tank, they rely on engineering judgment or upon field data and past experience to define the parameters that control the behavior of the model. Compartment models are used in water distribution network models to represent mixing in tanks and reservoirs. Various assumptions can be made in these models about the mixing behavior in tanks including complete and instantaneous mixing, plug flow, last-in/first-out (LIFO) behavior, and multi-compartment models. Both conservative substances and substances that decay according to a first-order decay function may be simulated in addition to simulation of water age. Compartment models are relatively easy to use and run in seconds as opposed to the long run times of CFD models.

Table 2-2. Example CFD Modeling Software Packages

CFD Package	Company	Website
CFD-ACE	CFD Research Corp.	www.cfdrc.com
Cfdesign	Blue Ridge Numerics	www.brni.com
CFX	Ansys, Inc.	www.software.aeat.com/cfx
FLOW-3D	Flow Science, Inc.	www.flow3d.com
Fluent	Fluent, Inc.	www.fluent.com
Phoenics	CHAM	www.cham.co.uk
SWIFT	AVL	www.avl.com
Sinda/Fluint	C&R Technologies	www.crtech.com
PAB3D	Analytical Services & Materials	www.asm-usa.com

Compartment models of tanks are available as part of most water distribution system models. EPANET and several of its derivative commercial models allow the user to select from four options – a complete mix model, a plug flow first-in/first-out (FIFO) model, a LIFO (short circuiting) model and a two-compartment model. A stand-alone model called CompTank provides a wide range of alternatives and allows the user to model water age and reactive or conservative substances over a long period of time (Grayman et al., 2000). This model uses tank inflow and outflow information that is generally available from SCADA records as its primary input.

2.4.1.3 Physical Scale Models

Physical scale models provide a relatively inexpensive mechanism for studying the mixing characteristics of tanks. In a physical scale model, a tracer chemical is added to the inflow (or internally within the model) and the movement of the tracer is monitored during the experiment (Grayman et al., 2000). Tracer substances include visible dyes, which are appropriate for developing a qualitative understanding of mixing behavior, and chemicals (e.g., calcium chloride) that can be measured by sensors in the tanks and used for quantitative assessments. Use of tracers of different density or careful control of temperature of the tracer can be used to study the impacts of thermal variations on mixing. Laws of similitude in hydraulics must be followed in order to account for the scaling effects. Scale models can vary in size and complexity from small tabletop models to large-scale models built in hydraulics laboratories. Figure 2-8 depicts such a large-scale model.

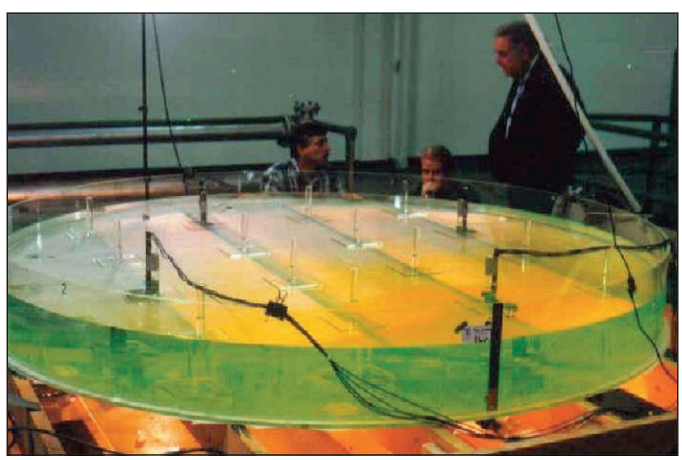


Figure 2-8. A Large Physical Model of a Tank (Source: Bureau of Reclamation Laboratory).

In an advanced technology form of physical scale modeling, three-dimensional laser induced fluorescence is being used to provide detailed measurements of mixing in tanks (Roberts and Tian, 2002). Figure 2-9 shows an illustration of output from this technology.

2.4.2 Transient Analysis Software

A hydraulic transient is a rapid change in pressure associated with a pressure wave that moves rapidly through a piping system. A transient can be caused by a variety of events, such as rapid operation of a valve (including fire hydrants) or rapid pump starts and stops. If the magnitude of the resulting pressure wave is large enough and adequate transient control measures are not in place, a transient can cause a water hammer leading to failure of hydraulic components. It can also lead to instantaneous low or negative pressures that can result in intrusion of untreated water into the pipe, potentially resulting in contamination. Transient events are highly dynamic and sophisticated. Mathematical models are required to analyze their movement in a distribution system.

Several commercial software packages for performing transient analysis in water distribution systems are available. Examples of such software are listed in Table 2-3. The technical capabilities, user interface, solution methods, graphical display, and technical support and training vary considerably among the packages.

2.4.3 Optimization Tools

Optimization tools allow the user to evaluate a large number of options and to select the specific alternative that gives the best results in terms of predefined objective functions. In the area of water distribution system analysis, optimization models are used for calibration, design, and operational purposes. These applications are briefly described in the following subsections.

2.4.3.1 Optimizing Calibration

Calibration of a water distribution system model involves adjustments in various model parameters so that the model agrees with field measurements of flow and pressure. Such a tool is used most frequently with flow and pressure measurements taken during flow (hydrant) tests to stress the system. Parameters that are typically adjusted include roughness factors, demands, and status of isolation valves.

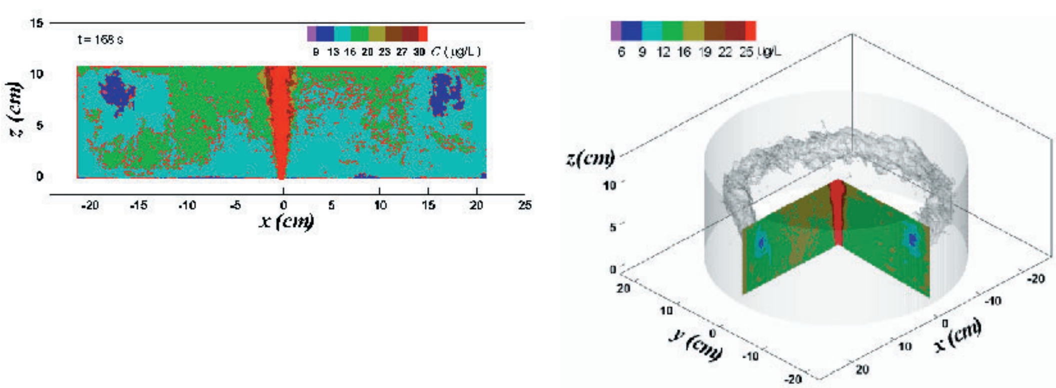


Figure 2-9. Graphical Output Based on 3-D Laser Induced Fluorescence with a Physical Scale Model Showing Mixing in Tank (Source: Georgia Tech).

Table 2-3. Example Transient Modeling Software Packages

Transient Modeling Software	Company	Website
AQUIS Surge	Seven Technologies	www.7t.dk/aquis
HAMMER	Haestad Methods	www.haestad.com
Hytran v3.0	Hytran Solutions	www.hytran.net
Impulse	Applied Flow Technology	www.aft.com/products/impulse
InfoSurge, H2OSurge	MWHSoft	www.mwhsoft.com

The production of transient low-and negative-pressures in otherwise pressurized drinking water supply distribution systems creates the opportunity for contaminated water to enter the pipe from outside. Such events may be caused by the sudden shutdown of pumps or by other operational events such as flushing, hydrant use, and main breaks. Figure 2-10 illustrates an event that results in a negative pressure transient for 22 seconds caused by a power outage associated with a lightning strike.

In a series of research projects (LeChevallier et al., 2003; Gullick et al., 2004), the frequency and location of low-and negative-pressures in representative distribution systems were measured under normal operating conditions and during specific operational events. These investigators also confirmed that fecal indicators and culturable human viruses were present in the soil and water exterior to the distribution system pipes. Their research shows that a well-calibrated hydraulic surge model can be used to simulate the occurrence of pressure transients under a variety of operational scenarios, and a model can also be used to determine optimal mitigation measures.

Although there are insufficient data to indicate whether pressure transients pose a substantial risk to water quality in the distribution system, mitigation techniques can be implemented. These techniques include the maintenance of an effective disinfectant residual throughout the distribution system, leak control, redesign of air relief venting, installation of hydro-pneumatic tanks, and more rigorous application of existing engineering standards.

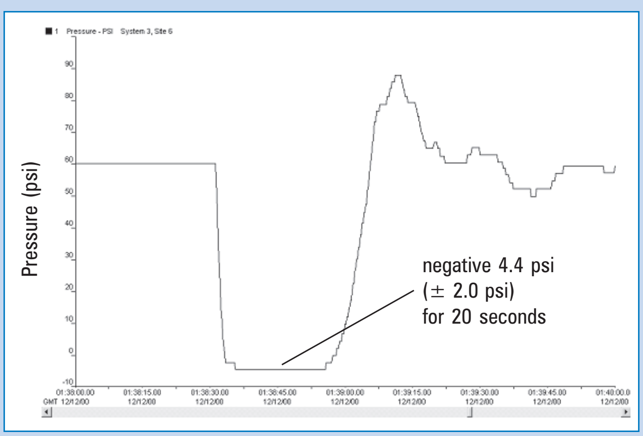


Figure 2-10. Negative Pressure Transient Associated with a Power Outage.

Use of manual adjustment techniques may involve many tedious runs of a distribution system model until the resulting predicted flows and pressures approximate the values observed in the field. When an optimization model is applied, the user defines an objective function, such as minimizing the square of the difference between observed and predicted values (for pressure and flow). The optimization algorithm then uses some type of controlled search method to identify the set of model parameters that will result in the best results (i.e., minimize the error). The user will generally set constraints on parameters so that the resulting values are reasonable. For example, the user may specify that the allowable range for the roughness factor for a certain set of ductile iron pipes range between 90 and 120.

Over the past 40 years, various techniques have been applied as part of automated calibration methods (Rahal et al., 1980; Walski et al., 2003). The most common optimization technique in use today couples a hydraulic model with an optimization routine using genetic algorithms. Genetic algorithms are based on the theory of genetics in which successive population trials are generated with the "fittest" ones surviving to breed and evolve into increasingly desirable offspring solutions. The fitness of a solution is based on the objective functions that were previously described. Genetic algorithm-based calibration tools are available as optional components of several water distribution system analysis software packages.

2.4.3.2 Design Optimization

In a manner analogous to the calibration optimization technique described above, design optimization techniques evaluate a large number of distribution system design options and select the one that provides the best solution (Lansey, 2000). Schaake and Lai (1969) first proposed such an approach and applied it to the design of major transmission lines providing water to New York City. Since that time, numerous papers have been written on the subject (Walski et al., 2003) and have included a variety of techniques such as linear programming, dynamic programming, mixed integer programming, heuristic algorithms, gradient search methods, enumeration methods, genetic algorithms, and simulated annealing. In recent years, genetic algorithm methods have been favored for this problem and have been widely used in a variety of situations and are included in several commercial software packages. The user should, however, be aware that genetic algorithms do not guarantee optimality. These algorithms must be run several times to ensure near optimal solutions.

Typically, design optimization tools limit a user to choose from designated piping options and to size the pipes to meet present and future demands. Cost minimization is the most common objective function.

Additionally, some researchers have incorporated reliability and capacity considerations (Mays, 1989).

2.4.3.3 Optimization of Operation

Models can also be used to optimize operations of a distribution system (Goldman et al., 2000). The most common areas of operation where such models have been applied are in energy management and water quality. Chase et al. (1994) describe a computer program to control energy costs that incorporates a hydraulic model, a pump optimization program, and an interface. In the water quality area, Uber et al. (2003) used optimization techniques to determine optimal location and operation of chlorine booster stations. Jentgen et al. (2003) implemented a prototype energy and water quality management system at Colorado Springs Utilities. This system combines a simplified distribution system model and an optimization routine to adjust operation of the water system and power generation system in near real-time.

2.4.4 Probabilistic Models

Hydraulic and water quality models of distribution systems are deterministic models. For a set of network parameters and specific operations and demands, the model produces a single set of resulting flows and pressures. However, there is uncertainty in many of the aspects of these models including parameters such as roughness, demands, actual inside diameter of pipes, valve settings, and system controls. This uncertainty is generally due to both imperfect knowledge and natural variability. An emerging procedure is to embed a deterministic network model within a probabilistic framework and to examine the effect of uncertainty on the results.

The most common approach to incorporating uncertainty in models is the use of a Monte Carlo simulation (Vose, 2000). In this method, probability distributions are assigned to model parameters to represent the uncertainty associated with each parameter. The distribution system model is then run many times with parameter values being randomly drawn from the probability distributions. The results of many iterations are combined to determine the most likely result and a distribution of results. This approach has been used in legal cases where historical contamination events have been reconstructed (Grayman et al., 2004b), in evaluation of the impacts of purposeful contamination (Murray et al., 2004) and modeling bacterial regrowth in distribution systems (DiGiano and Zhang, 2004).

2.5 Summary and Conclusions

Acquiring and utilizing the proper data is very important for implementing water distribution system models. The key inputs include the characterization of the pipe network (e.g., pipes, pumps, tanks, and valves), water-demand information (temporal variations are required in EPS), topographic information (elevations assigned to nodes), control information that describes how the system is operated, and EPS solution parameters (e.g., time steps, tolerances as required by the solution techniques). Periodic calibration and validation of a model is important to achieve optimum results.

Models have become widely accepted within the water utility industry as a mechanism to simulate the hydraulic and water quality behavior of a real or proposed distribution system. They are routinely used for a number of tasks including capital investment decisions, master plan development, and fire protection capacity design. Furthermore, these models have become very sophisticated and typically simulate both hydraulic and water quality behavior. Many modeling packages are integrated with GIS or CAD. Some software packages incorporate water hammers and tank mixing. EPANET is a public sector hydraulic/water quality model developed by EPA. EPANET also serves as the computation engine for many of the commercial models used by water utilities throughout the country. In addition to EPANET and EPANET-based water distribution system models, there are several other tools available to users for studying specific needs, such as transient analysis and optimization analysis.

To successfully apply a model to study a problem, one should clearly define the objectives and select an appropriate tool. Thereafter, understanding the accuracy of the input data and limitations of the model will enable the user to better interpret the results of the analysis and develop appropriate solutions.

Many of the assumptions and methodologies in use today in water distribution system modeling date back to the early work of Hardy Cross (1936). With the monumental increase in computational power and improvements in the ability to measure flow in experimental distribution systems, it is natural that some of the basic assumptions are being examined and challenged. Three notable examples of active research areas include the following:

- Distribution system water quality models currently assume advective flow that results in water quality pulses moving through a pipe without spreading out longitudinally. Lee and Buchberger (2001) have studied pipe flow and found that dispersion has a significant effect on concentration profiles, especially in cases of intermittent laminar flow. Lee (2004) developed an analytical equation which describes the unsteady dispersion of changing flow velocity in pipes based on the classic one-dimensional advection-dispersion equation by Taylor (1953). Tzatchkov et al., (2002) have developed an extension to the standard EPANET model that includes dispersion.

- In distribution system models, deterministic demands are assigned to nodes. Buchberger et al., (2003) monitored water use at the individual home and neighborhood level and found that there are significant short-term variations in water use. They have developed a model that represents water use as a series of pulses which can be simulated using a Poisson Rectangular Pulse model to capture the natural variability associated with water use.

- Distribution system models currently assume complete mixing at a junction. As a result, if there are two pipes with flow entering the junction and two pipes through which the flow exits, the chemical content of the water in the two exiting pipes will be identical and represent an average of the characteristics of the two entering pipes. Van Bloemen Waanders et al., (2005) have tested this assumption using both laboratory analysis and CFD modeling. Figure 2-11a depicts the velocity field at a junction. Figure 2-11b presents the corresponding tracer concentrations at that junction. The figures indicate that the complete mix assumption would lead to some inaccuracy in computing chemical transport in a distribution system.

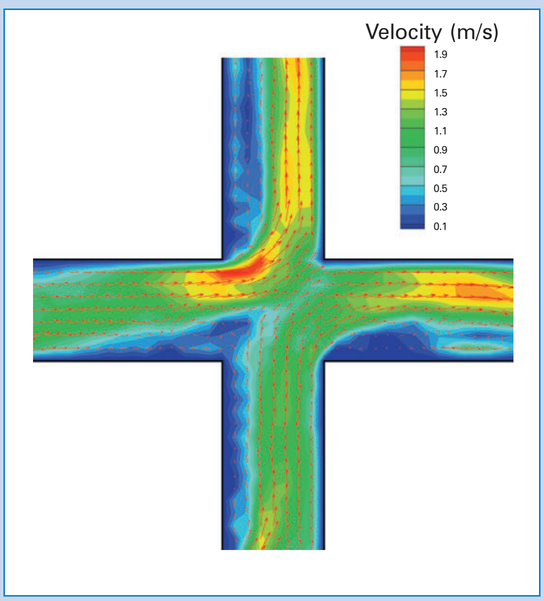


Figure 2-11a. Velocity Field at a Junction.

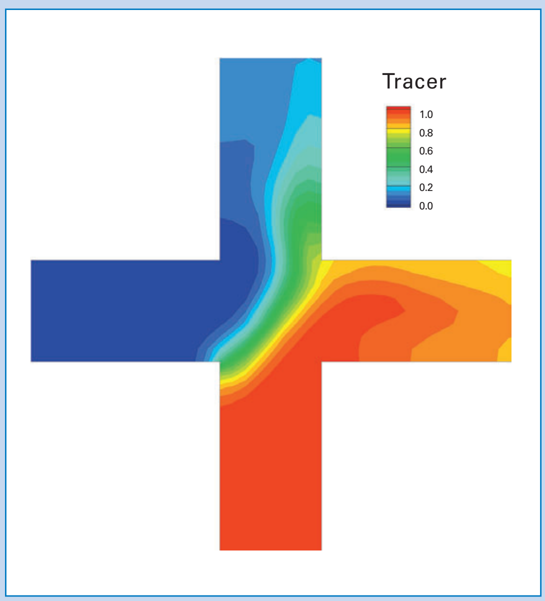


Figure 2-11b. Tracer Concentration at a Junction.

References

American Water Works Association (AWWA). *Computer Modeling of Water Distribution Systems, AWWA Manual M-32*, Denver, CO. 2004.

AwwaRF/U.S. EPA. *Water Quality Modeling in Distribution Systems.* Edited by W.M. Grayman, R.M. Clark, and R.M. Males. AwwaRF-AWWA. Denver. CO. 1991.

Buchberger, S.G., J.T. Carter, Y.H. Lee, and T.G. Schade. "Random Demands, Travel Times, and Water Quality in Deadends." AwwaRF/AWWA, Denver, CO. 2003.

Bureau of Reclamation Laboratory. Photo provided by Bates Engineering, Inc., 2004.

Chase, D.V., D. Guastella, K.L Lai, and L.E. Ormsbee. "Energy Management for Large Scale Water Supply Systems." AWWA Computer Conference, Los Angeles, CA. 1994.

Clark, R.M., W.M. Grayman, R.M. Males, and J.A. Coyle. "Predicting Water Quality in Distribution Systems." Proceedings, AWWA Distribution System Symposium, Minneapolis, MN. 1986.

Clark, R.M., and R.M. Males. "Developing and Applying the Water Supply Simulation Model," *Journal of AWWA*, 78(8). 1986.

Clark, R.M., and W.M. Grayman. *Modeling Water Quality in Drinking Water Systems.* AWWA. 1998.

Clark R.M., R.C. Haught, and W.M. Grayman. "Predicting Chlorine Residuals in Unlined Metallic Pipes." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

Clark, R.M., H. Pourmaghaddas, L.G. Wymer, and R.C. Dressman. "Modeling the Kinetics of Chlorination By-Product Formation: The Effects of Bromide," *Journal Water Supply Research and Technology - Aqua*, 45(1):1-8. 1996.

Cross, H. "Analysis of Flow in Networks of Conduits or Conductors," *University of Illinois Engineering Experiment Station Bulletin 286*, Urbana, IL. 1936.

DiGiano, F. A., and W. Zhang. "Uncertainty Analysis in a Mechanistic Model of Bacterial Regrowth in Distribution System." *Environmental Science and Technology* 38:22:5925-5931. 2004.

EPA. *Stage 2 Disinfectants and Disinfection Byproducts Initial Distribution System Evaluation Guidance Manual, Draft*. 2003.

Georgia Tech. Photo provided by Dr. Philip J. W. Roberts, School of Civil Engineering, Georgia Institute of Technology, 2004.

Goldman, F.E., A.B.A. Sakarya, L.E. Ormsbee, J.G. Uber, and L.W. Mays. "Optimization Models for Operation." Chapter in *Water Distribution System Handbook*. Edited by Larry Mays for McGraw-Hill, NY. 2000.

Grayman, W.M., R.M. Clark, and R.M. Males. "Modeling Distribution System Water Quality Dynamic Approach," *Journal of Water Resources Planning and Management*, ASCE, 114(3). 1988.

Grayman, W.M. "Put Your Hydraulic Model to Work – How That Costly Model Can Earn its Keep by Making Operations More Effective." Proceedings, AWWA Annual Conference, Denver, CO. 2000.

Grayman, W.M., L.A. Rossman, M.A. Gill, Y. Li, and D.E. Guastella. "Measuring and Modeling Disinfectant Wall Demand in Metallic Pipes." Proceedings, EWRI Conference on Water Resources Planning and Management, Reston VA. ASCE. 2002.

Grayman, W.M., L.A. Rossman, R.A. Deininger, C.D. Smith., C.N. Arnold., and J.F. Smith. "Mixing and Aging of Water in Distribution System Storage Facilities." *Journal of AWWA*, 96(9):70-80. 2004a.

Grayman W.M., R.M. Clark, B.L. Harding, M. Maslia, and J. Aramini. "Reconstructing Historical Contamination Events," Chapter in *Water Supply Systems Security*. Edited by Larry Mays, McGraw-Hill, NY. 2004b.

Grayman, W.M., and C. N. Arnold. "Overview of CFD Methods in Analysis of Distribution System Tanks and Reservoirs." Proceedings, AWWA Annual Conference, AWWA, Denver, CO. 2003.

Grayman, W.M., and L.A. Rossman. "Integrating a Network Model with an Expert System." Proceedings, AWWA Computer Conference, AWWA, Denver, CO. 1994.

Grayman, W.M., L.A. Rossman, C.N. Arnold, R.A. Deininger, C.D. Smith, J.F. Smith, and R. Schnipke. *Water Quality Modeling of Distribution System Storage Facilities.* AwwaRF and AWWA. 2000.

Gullick, R.W., M.W. LeChevallier, R.C. Svindland, and M. Friedman. "Occurrence of Transient Low and Negative Pressures in Distribution Systems." *Journal of AWWA*. 96(11): 52-66. November 2004.

Hart, F.L., J.L. Meader, and S.N. Chiang. "CLNET A Simulation Model for Tracing Chlorine Residuals in a Potable Water Distribution Network." Proceedings, AWWA Distribution System Symposium, Minneapolis, MN. 1986.

Jentgen, L., S. Conrad, R. Riddle, E. Von Sacken, K. Stone, W.M. Grayman, and S. Ranade. *Implementing a Prototype Energy and Water Quality Management System.* AwwaRF, Denver, CO. 2003.

Jeppson, R.W. *Analysis of Flow in Pipe Networks.* Ann Arbor Science, Ann Arbor, MI. 1976.

Johnson K.E., and J. Loux. *Water and Land Use: Planning Wisely For California's Future.* Solano Press, Point Arena, CA. 2004.

Lansey, K.E. "Optimal Design of Water Distribution Systems." Chapter in *Water Distribution System Handbook.* Edited by Larry Mays for McGraw-Hill, NY. 2000.

Larock, B.E., R.W. Jeppson, and G.Z. Watters. *Hydraulics of Pipeline Systems.* CRC Press, LLC, Boca Raton, FL. pp 12-26. 2000.

LeChevallier, M.W., R.W. Gullick, M.R. Karim, M. Friedman and J.E. Funk. "The Potential for Health Risks from Intrusion of Contaminants into the Distribution System from Pressure Transients," *Journal of Water and Health,* IWA Publishing. pp. 3-14. January 2003.

Lee, Y. and S.G. Buchberger. "Estimation of dispersion in unsteady random flow conditions in dead-end pipes of water distribution systems." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Orlando, FL. 2001.

Lee, Y. Mass Dispersion in Intermittent Laminar Flow. Ph.D. Dissertation in Civil and Environmental Engineering, University of Cincinnati, Cincinnati, Ohio. 2004.

Liou, C.P., and J.R. Kroon. "Propagation and Distribution of Waterborne Substances in Networks," Proceedings, AWWA Distribution System Symposium, Minneapolis, MN. 1986.

Males, R.M., R.M. Clark, P.J. Wehrman, and W.E. Gates. "Algorithm for Mixing Problems in Water Systems," *Journal Hydraulic Engineering*, ASCE, 111(2). 1985.

Males, R.M., W.M. Grayman, R.M. Clark. "Modeling Water Quality in Distribution Systems," *Journal Water Resources Planning and Management*, ASCE, 114(2). 1988.

Mays, L.W., Editor. *Reliability Analysis of Water Distribution Systems.* ASCE. NY. 1989.

Murray, R., R. Janke, and J. Uber. "The Threat Ensemble Vulnerability Assessment (TEVA) Program for Drinking Water Distribution System Security." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Salt Lake City, UT. 2004.

Powell, J., J. Clement, M. Brandt, R. Casey, D. Holt, W. Grayman, and M. LeChevallier. "Predictive Models for Water Quality in Distribution Systems." AwwaRF, Denver, CO. 2004.

Rahal, C.M., M.J.H. Sterling, and B. Coulbeck. "Parameter Tuning for Simulation Methods of Water Distribution Networks." Proceedings, Institute of Civil Engineers, Part 269, 751. 1980.

Roberts, P.J.W., and X. Tian. "Application of Three-Dimensional Laser-Induced Fluorescence to Stratified Turbulent Mixing Processes." Proceedings, ASCE Hydraulic Measurements and Experiments Conference. 2002.

Rossman, L.A., and P.F. Boulos. "Numerical Methods for Modeling Water Quality in Distribution Systems: A Comparison." *Journal Water Resources Planning and Management*, ASCE, 122(2):137-146. 1996.

Rossman, L.A. *EPANET Version 2 Users Manual*, Drinking Water Research Division, EPA, Cincinnati, OH. 2000.

Rossman, L.A., R.M. Clark, and W.M. Grayman. "Modeling Chlorine Residuals in Drinking Water Distribution Systems." *Journal Environmental Engineering,* 120(4):803-820. 1994.

Schaake, J., and D. Lai. *Linear Programming and Dynamic Programming Applications to Water Distribution Network Design - Report 116.* Department of Civil Engineering, MIT, Cambridge, MA. 1969.

Taylor, G.I. "Dispersion of soluble matter in solvent flowing slowly through a tube," *Journal of the Royal Society of London Series A*, The Royal Society, London, UK. 219:186-203. 1953.

Tzatchkov, V.G., A.A. Aldama, and F.I. Arreguin. "Advection-dispersion Reaction Modeling in Water Distribution Networks." *Journal Water Resources Planning & Management*, 128(5):334-342. 2002.

Uber, J., F. Shang, M. Polycarpou, and Z.Wang. "Feedback Control of Booster Chlorination Systems." AwwaRF, Denver, CO. 2003.

Uber, J., F. Shang, and L. Rossman. "Extensions to EPANET for Fate and Transport of Multiple Interacting Chemical or Biological Components." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Salt Lake City, UT. 2004.

Van Bloemen Waanders, B., G. Hammond, J. Shadid, S. Collis, and R. Murray. "A Comparison of Navier-Stokes and Network Models to Predict Chemical Transport in Municipal Water Distribution Systems." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

Vasconcelos, J.J., L.A. Rossman, W.M. Grayman, P.F. Boulos, and R.M. Clark. "Kinetics of Chlorine Decay," *Journal of AWWA*, 89(7):54-65. 1997.

Vose, D. *Risk Analysis – A Quantitative Guide.* John Wiley & Sons. Chichester, UK. 2000.

Walski, T.M., D.V. Chase, D.A. Savic, W. Grayman, S. Beckwith, and E. Koelle. *Advanced Water Distribution Modeling and Management*. Haestad Press, Waterbury, CT. 2003.

Wood, D.J. *Computer Analysis of Flow in Pipe Networks.* Department of Civil Engineering, University of Kentucky, Lexington, KY. 1980a.

Wood, D.J. "Slurry Flow in Pipe Networks." *Journal of Hydraulics*, ASCE, 106(1). May 1980b.

Chapter 3
Tracer Studies for Distribution System Evaluation

Tracers have been used for decades to determine flow, travel time, and dispersion in surface waters and groundwater. Tracers can be of various types, ranging from a physical object that can be visually detected in a stream or river to dyes or other chemicals whose concentrations can be monitored using special instrumentation. Fluorescent dyes have been used for many years to measure velocity and tidal movement in streams and estuaries. Use of tracers to understand the hydraulic movement in drinking water treatment unit processes or distribution systems is a more recent development. When tracers are used in drinking water, care must be taken to ensure that they will have no adverse health effects and that their use does not result in any violations of primary and/or secondary drinking water MCLs.

Tracers have been used in drinking water to estimate the travel time through various water treatment unit processes including clearwells (Teefy and Singer, 1990; Teefy, 1996; DiGiano et al., 2005). Tracer studies have also been conducted in distribution system tanks and reservoirs in an attempt to understand their mixing characteristics (Grayman et al., 1996; Boulos et al., 1996). They have also been used in water distribution networks to provide insight into the complex movement of water in a distribution system, to determine travel times, and to assist in calibration of distribution system hydraulic models (Clark et al., 1993; DeGiano et al., 2005; Vasconcelos et al., 1997; Grayman, 2001). For example, Boccelli et al. (2004) and Sautner et al. (2005) have used dual tracers injected into water distribution systems to assess travel time and characterize flow patterns in support of epidemiological investigations. With the recent interest in homeland security issues, tracers are being used to simulate the movement and impacts of accidental or intentional contamination of water distribution systems (Panguluri et al., 2005).

Conducting a distribution system tracer study involves (1) injecting the tracer into a pipe upstream of the area to be studied, (2) shutting off or reducing a continuous chemical feed at the water treatment plant, or (3) use of a naturally occurring substance in source water. The concentration is measured over time at various locations in the water distribution network as it moves through the study area. To be successful, a tracer study requires careful planning and implementation. This chapter provides information and guidance on planning and conducting tracer studies in drinking water distribution systems.

Tracer studies in distribution systems may provide a wide variety of useful information, including the following:

- Calculating travel time, residence time, or water age in a network.
- Calibrating a hydraulic model.
- Defining zones in a network served by a particular source and/or assessing the degree of blending with water from other sources.
- Determining the impacts of accidental or intentional contamination.
- Identifying appropriate sampling locations within the water distribution network.

Tracer studies may also assist water utilities in complying with various regulatory requirements. For example, the DBPR2 IDSE draft Guidance Manual (EPA, 2003a) recognizes the use of tracers as a means of calibrating models and predicting residence time as a partial substitute for required field monitoring. Several rules and regulations (both existing and proposed) are currently being reviewed, such as the TCR and a proposed distribution system rule. Water quality modeling and model calibration are likely to play a role in the development and/or promulgation of these rules.

The scope of a tracer study may vary considerably depending upon the study needs, size, and complexity of the distribution network being evaluated. A study area may consist of a single stretch of pipe, an entire neighborhood, a portion of a large distribution system, a pressure zone, or in some cases, the entire distribution network. The resources required to conduct a tracer study will vary with the extent, complexity of the study, and the test equipment used. Careful planning and implementation are critical in all cases to ensure meaningful results. Section 3.1 of this chapter contains information that can be used during the planning phases of a tracer study. Section 3.2 provides a summary of the tasks associated with executing a tracer study. Section 3.3 presents typical costs associated with conducting a tracer study. Finally, Section 3.4 presents a summary, conclusions, and recommendations for this chapter. The use of tracer study data for model calibration/validation is described in Chapter 4.

3.1 Planning and Designing a Distribution System Tracer Study

The initial step in any tracer study is a planning and design phase during which study-specific logistical details are identified and addressed. These details should be presented in a comprehensive manner in a planning document or work plan that can be reviewed and commented on by parties that may have an interest in the tracer study (e.g., team members, water utility staff and managers, and state regulatory officials). Planning and design-phase elements may include the following:

- Establishing study objectives and timeline.
- Forming a study team.
- Defining study area characteristics.
- Selecting tracer material.
- Selecting field equipment and procedures.
- Developing a detailed study design.
- Addressing agency and public notification.

The details of each of these tasks are described in the following sub-sections.

3.1.1 Establishing Study Objectives and Time-Line

A clear statement of the study objectives should be developed, even before logistical planning begins. For example, an objective statement might read "determine travel times from the Lincoln Water Treatment Plant to key locations (transmission mains and representative local mains) in the Washington Pressure Zone under typical summer operation." Such a statement provides a clear understanding of the study's overall goals and objectives. A study objective may also be more specific and define additional key elements such as tracer material, dosage, and injection duration.

Depending upon the objective, an approximate timeline (schedule) for the study should be formulated. Frequently, external constraints such as weather, system operation, and availability of personnel/equipment may influence this timeline. In other cases, the project timeline may depend upon the specific objective of the study. For example, if the maximum community exposure to a contamination event is being studied, the timeline should be consistent with the season and time during which the event is likely to occur. If the study is intended to identify locations in the system where the lowest chlorine residuals are found, the study should be conducted during a period when minimum chlorine residuals occur. However, it is not always possible to conduct a tracer study to match system conditions that coincide with the study time-frame. Therefore, a reasonable alternative is to use the tracer to calibrate a study-area-specific network model, under a given set of conditions, that can be used to simulate other critical events under different conditions.

> In mid-western U. S., October-November is the best time-frame to conduct a tracer study in a residential area. During this time, the utility has greater operational flexibility because it is not stressed by high demands, weather is conducive to outdoor activity, and cold weather pipe breaks are minimal.

3.1.2 Forming a Study Team

A "tracer study team" should be formed at the beginning of the project. Depending on the size and scope of the study, the size of the team may vary from as few as three members to a sizable group of as many as twenty members. However, the range of functions and responsibilities that must be considered are approximately the same in all types of studies. The team makeup must include members with expertise for planning and carrying out the following activities and functions: understanding study area distribution system and treatment operations; conducting preliminary modeling studies; selecting, acquiring, and installing field equipment; managing and organizing field crews; performing field sampling; conducting laboratory analysis; analyzing and reporting results; and performing communications and notifications.

Study teams may be made up of water utility personnel, consulting engineering firm personnel, contractor staff, students from universities, and in some cases, federal or state governmental agency employees. Specific responsibilities and roles should be assigned to each team member. It is recommended that the study team meet on a regular basis to ensure that the task deadlines are met and the study objectives are achievable. If the tracer study includes new or never-before used equipment, training sessions for study team members should be included as part of study timeline and activities.

3.1.3 Defining Study Area Characteristics

After the study team is formed, perhaps the first task to be undertaken is to identify the key characteristics of the study area. These characteristics include: the piping system network, pumping and storage operations, inflow and outflow through study area boundaries, temporal and spatial variations in water consumption, presence of large water users that may significantly impact water use patterns, and the

When planning a tracer study, the effects of distribution system tanks and reservoirs should be considered (Grayman at al., 2004). When a tracer enters a tank in the inflow, it mixes with the distributed water and then exits the tank at a different concentration during the subsequent draw cycles. Mixing in the tank may be rapid and complete or there may be short-circuiting or plug flow behavior that affects the concentration in the effluent. Various mathematical tools such as CFD models may be applied to estimate the mixing characteristics of a tank and the effects on tracer concentration during discharge periods (Grayman et al., 2004). Distribution system models such as EPANET allow the user to simulate mixing in tanks by several alternative conceptual and simplified models such as completely and instantaneously mixed, short circuiting, plug flow, and multiple compartment mixing. The effects of tanks can impact the needed tracer dosage rate and injection duration and the subsequent sampling frequency and duration in parts of the distribution system impacted by the tank. During the tracer study, the impacts of mixing in the tank can be determined by sampling in the inflow and outflow lines, and in some cases, internally within the tank.

geography and local features associated with the study area that could potentially constrain field activities.

A large commercial user such as a golf course in the neighborhood may impact the study events.

There are several tools and procedures that can be applied to improve the team's understanding of the target water distribution system area prior to conducting the tracer study. If a hydraulic model of the distribution system (under study) is available, it would be very helpful to use the model to simulate the tracer study under expected conditions. Examination of documents, such as master plans or operational reports, can also shed light on how the water system behaves. The study team or key members of the study team should also tour the study site with as-built pipe drawings to identify potential locations for safely installing field injection equipment, as well as flow and tracer monitoring equipment.

3.1.4 Selecting Tracer Material

Criteria that can influence the selection of a particular tracer include:

- regulatory requirements,
- analytical methods and instruments available for measuring tracer concentration,
- injection and storage requirements,
- chemical reactivity,
- chemical composition of the finished water,
- overall cost, and
- public perception.

Ideally, a tracer should be inexpensive, nonreactive with both water and distribution system materials, safe to drink when dissolved in water, easily dispersed in water, aesthetically acceptable to customers, able to meet all drinking water regulations, and inexpensively and accurately monitored in the field by manual and automated methods. There is no one tracer that will meet all of these criteria for a given study. Frequently, there are tradeoffs among the criteria listed above that must be assessed when selecting a tracer. The tracer to be used in the study should be determined early in the planning stage, and approval for its use received from the water utility and state regulatory agencies.

Tracers may fall into three broad categories: a chemical that is normally added to the water during the treatment process and that may be temporarily shut off during the study; a chemical that is added to the water by the team during the study; or a naturally occurring substance in the source water that may be adjusted in some manner to create a tracer.

The most commonly used tracers are fluoride, calcium chloride, and sodium chloride.

3.1.4.1 Fluoride

Fluoride is frequently added to water supplies because of its health benefits, but can be turned off for short periods, thereby making the non-fluoridated water a tracer in the system. When fluoridation is not practiced, fluoride can be added to the water system and used as a tracer by injection. It is especially popular with utilities that routinely add fluoride as part of the treatment process, because little effort is required to turn the fluoride off and on. When the fluoride feed is shut off, a front of low-fluoride water (or no fluoride if there is no natural background concentration) becomes the tracer. A second tracer test (or a continuation of the initial test) can be performed when the fluoride feed

Fluoride can interact with coagulants that have been added during treatment and in some circumstances can interact with pipe walls leading to non-conservative behavior. Thus, when used in systems that do not generally fluoridate, a field test should be performed to determine possible interactions with pipes.

is turned back on, thus making it possible to generate two sets of tracer data in one study.

The MCL for fluoride is 4 mg/L. However, if the secondary MCL of 2 mg/L is exceeded, customers must be notified. Background levels of fluoride can vary significantly and actually exceed the secondary MCL in some geographic areas.

In cases where a utility is not permitted to completely shut off the fluoride feed, it may be feasible to increase the fluoride feed prior to the tracer study and to reduce the fluoride feed during the test. Care should be exercised to avoid exceeding the secondary MCL. However, there must be a sufficient change in the fluoride concentration feed in order to trace the change through the system. Thus, for example, a decrease in feed concentration from 1.2 mg/L to 0.8 mg/L may not be sufficient, but a decrease in concentration from 1.5 mg/L to 0.5 mg/L may be adequate. A change in fluoride dosage may have to be pre-approved by state regulators. Depending upon the duration of the study, the state agency may choose to allow a temporary shutoff or set a specific lowest allowable-fluoride-concentration requirement.

In most treatment plants, fluoride is injected prior to a final clearwell. As a result, when the feed is shut off as a part of the tracer study, there is both a time delay and a gradual change in concentration in the clearwell discharge as the non-fluoridated and fluoridated water mix. Therefore, wherever and whenever possible, the clearwell should be operated at minimum water levels during the tracer test in order to achieve a relatively sharp front of non-fluoridated water leaving the treatment plant. It is also important to evaluate the impact of travel through finished water storage reservoirs on the concentration of tracer during the study. An alternative is to inject fluoride solution (e.g., sodium fluoride) at a point in the main transmission line downstream of the clearwell where both flow and injection rate can be simultaneously monitored and measured.

In a study conducted in the Cheshire service area of the South Central Connecticut Regional Water Authority (SCCRWA) in 1989, the fluoride feed was turned off to provide a tracer to validate a hydraulic and water quality model of their water distribution system (Clark et al., 1991). This study was among the first applications of water quality models in the world. SCCRWA normally added fluoride at a level of approximately 1 mg/L. For purposes of the model validation study, the fluoride feed was turned off for a period of 7 days and then turned back on with sampling occurring for an additional 7 days. This approach yielded, in effect, two tracer fronts. During the study, grab samples were taken every few hours at 16 hydrants, two well fields, one tank, four continuous analyzer sites, and daily at 19 "deadend" sites. Additionally, experimental units were installed at a few sheltered sites to automatically measure fluoride concentrations and to take discrete samples for later analysis. A total of 2,150 fluoride grab samples were taken during the study and analyzed in the laboratory.

Ion-selective electrodes (ISE) can be used in conjunction with data loggers to provide continuous monitoring capability. At present, however, these instruments are relatively expensive (approximately $5,000 to $10,000 each) and have only been used extensively in large-scale tracer studies (Maslia et al., 2005; Sautner et al., 2005). Generally, grab samples are taken and analysis is performed manually in the field or laboratory.

Under some circumstances, fluoride is not a fully conservative chemical. In one study (Vasconcelos et al., 1996) in a system that did not normally fluoridate, a 13-hour pulse (step input of limited duration) of fluoride was injected into the feed line of a pressure zone. Field measurements of fluoride concentrations in the zone during the study indicated a significant loss of fluoride. It was postulated that some of the fluoride was deposited on the pipe wall. In a followup study, this problem was virtually eliminated by injecting fluoride over a period of several days prior to the actual study in order to pre-condition the pipes.

3.1.4.2 Calcium Chloride

Calcium chloride ($CaCl_2$) has been used in many tracer studies throughout the U.S. It is considered to be safe and relatively easy to handle. Generally, a food grade substance is required. It can be purchased as a liquid (typically a 30 to 35% solution) or as a powder that can be mixed with water to form a solution.

If calcium chloride is chosen as a tracer, the study personnel should be aware of the secondary drinking water MCL for chloride (250 mg/L). A target that is less than the secondary MCL should be set in order to provide a safety factor. Where chloride levels are high, calcium chloride may not be an appropriate choice for a tracer.

Grayman et al., (2000) utilized calcium chloride as a tracer in two studies of mixing in distribution system tanks. In both studies, the chemical was injected into the inflow pipe of the tank during the fill cycles, and conductivity and chloride were measured at locations within the tank. Calcium chloride has recently been used in several distribution system studies (Panguluri et al., 2005; Maslia et al., 2005; and Sautner et al., 2005).

Calcium chloride can be monitored by measuring conductivity, or by measuring the calcium or chloride ion (Standard Methods, 1998). Conductivity is typically the easiest of these parameters to measure and is most amenable to inexpensive continuous monitors. However, conductivity is not a truly linear parameter (i.e., if a beaker of water of conductivity 100 mS/cm is combined with a like volume of water with a conductivity of 300 mS/cm, the conductivity of the resulting solution will not be exactly 200 mS/cm). As a result, distribution system models (that all assume linearity) can only approximately represent conductivity. Therefore, when using conductivity as the measured parameter, the options are to accept the linear approximation or convert conductivity to a true linear parameter such as chloride or calcium. If the former option is chosen, the amount of resulting error should be established in laboratory tests of waters of varying conductivity. If the latter option is chosen, the relationship between conductivity and chloride (or calcium) must be established in the laboratory. It should also be noted that most field devices are set up to measure specific conductance instead of conductivity (conductivity is temperature sensitive, whereas specific conductance is referenced to 25°C). For the purposes of this document, conductivity is assumed to represent specific conductance.

3.1.4.3 Sodium Chloride

Sodium chloride (NaCl) can be used as a tracer and has many characteristics similar to calcium chloride in that it can be traced by monitoring for conductivity or for the concentration of the chloride or sodium ion. The allowable concentration for sodium chloride is also limited by the secondary MCL for chloride and the potential health impacts of elevated sodium levels. EPA reports that taste thresholds for sodium vary significantly among individuals, ranging from 30 to 460 mg/L (EPA, 2003b).

In a recent tracer study in Hillsborough County, Florida, two separate tracer chemicals were used to study the movement of water in a large distribution system (Boccelli et al., 2004). Approximately 2,200 gallons of a saturated NaCl solution was injected into the finished water of a treatment plant as a series of four pulses ranging in duration from 1 to 3 hours over a 24-hour period. Simultaneously, the normal fluoride feed was shut off at the plant. Continuous conductivity monitors were installed at 14 locations in the distribution system to monitor for the NaCl tracer. Grab samples were taken to monitor the low fluoride front as it moved through the system and to evaluate water quality changes. The resulting extensive hydraulic and water quality database is being used to calibrate a hydraulic and water quality model of the system (Boccelli and Uber, 2005).

3.1.4.4 Other Chemicals That May be Added as Tracers

Other chemicals added as part of a tracer study include lithium chloride and chlorine. Lithium chloride is a popular tracer in the United Kingdom but is used less frequently in the U.S., partly because of the public perception of lithium as a medical pharmaceutical. There are no field techniques for measuring lithium, and it is not easily amenable to automated continuous measurement. Samples must be collected and lithium concentrations measured in the laboratory.

Chlorine is commonly used as a disinfectant in many water systems. Because chlorine is reactive, it will decay over time. Under some circumstances, however, it can be used effectively as a tracer. It is most effective in a water where chlorine is not highly reactive (low decay rate) with either the water or distribution system material, and where the concentration levels can be increased above the normal level to create a front of water with a high chlorine concentration propagating through the system. However, in no case should the chlorine or chloramine be decreased to a level that may affect the disinfection process (Ferguson and DiGiano, 2005). Again, any tracer study should first be approved by the state regulators.

3.1.4.5 Naturally or Normally Occurring Tracers

Perhaps the most difficult part of conducting a tracer study is obtaining permission to add a chemical and then injecting the tracer into the system at a concentration consistent with regulations. Much of this effort can be avoided if there is a natural tracer available. Natural tracers are generally site-specific, but many options do exist and should be explored. The most common situation is the existence of multiple sources of water with different chemical signatures or if a change is planned in the chemical signature at a single source. Examples of these situations are described below.

Some of the chemical signatures that may be used to differentiate between sources include THM concentrations, hardness, conductivity, and treatment coagulant. Sampling in the distribution system for these “tracers” will provide information on zones served by each of the sources and the extent and variation of the mixing that takes place in these zones over time. Alternatively, if one water source can be turned off for a period of time until the other source has reached chemical equilibrium throughout the system, the original source can be turned back on and used as a tracer as it propagates through the system. One of the first uses of natural tracers was in the North

Making a major change in the incoming water supply such as a change in source water or modifying treatment may provide an opportunity to conduct a tracer test. The increased use of chloramines as a secondary disinfectant, to reduce the formation of DBPs, introduces another potential tracer opportunity. When a water utility switches from chlorine to chloramines (or vice versa), the chemical signature of the water changes and can be monitored by measuring both free and total chlorine. Namely, with chloramination, total chlorine is typically much higher than free chlorine, while with free chlorination, free and total chlorine will typically be very similar. A tracer study can be conducted when a system first adopts chloramination. Alternatively, many water utilities routinely switch back from chloramination to chlorine (e.g., annually for a month) in order to kill ammonia-oxidizing bacteria and thus reduce the chances of nitrification. This provides a recurring opportunity to conduct such a tracer study.

Penn Water Authority (NPWA) located in Lansdale, PA (Clark and Coyle, 1990). A field research project was conducted by EPA and NPWA that resulted in the development of a series of models that were used to study contaminant propagation in the water distribution system. The utility used a combination of groundwater with high levels of hardness and surface water containing higher levels of THMs. This resulted in two sources of water with very different quality characteristics. By monitoring changes in water quality that occurred at selected sampling points in the utility network, it was possible to use hardness and THM concentrations as tracers to validate the model.

Another case occurred in the North Marin Water District (NMWD) in northern California (Clark et al., 1994) where natural differences in water characteristics were used to serve as a tracer for validation of a water distribution system model. In this EPA-sponsored study, the utility used two sources of water with dramatically different water quality characteristics. The first source, Stafford Lake, has a very high humic content and thus has a very high THM formation potential. The other source is the North Marin Aqueduct with a very low humic content and thus a very low THM formation potential. The model was further validated by predicting chlorine residual losses at various points in the network. In a follow-up study supported by AwwaRF (Vasconcelos et al., 1997), the investigators used sodium as a tracer to validate the model.

DiGiano and Carter (2001) and DiGiano et al. (2005) traced the flow from two separate treatment plant sources at the same time by simultaneously reducing the fluoride feed at one plant while changing the coagulant added at the other plant. Normally, ferric chloride ($FeCl_3$) was used as a coagulant at both plants. During the tracer study, the coagulant at one plant was changed to aluminum sulfate [$Al_2(SO_4)_3$]. Fluoride, sulfate, and chloride were measured throughout the distribution system.

Water utilities should carefully examine their particular system to determine if a natural tracer is available or if source-chemical signatures may be modified to be used as a tracer.

Sweetwater Authority in southern California took advantage of a normal changeover in source water quality to perform a tracer study in their distribution system (Hatcher et al., 2004). In this case, the utility semi-annually changes the primary source of their water supply from local Sweetwater Reservoir raw water to water provided by the California Aqueduct. These two sources have very different chemical characteristics; most significantly, the organic carbon content (i.e., humic and fulvic acids) of Sweetwater Lake water is much higher compared to the raw aqueduct water. The measurement of molecular organic carbon absorbance at 254 nanometers, utilizing an ultra-violet-visible (UV-VIS) spectrophotometer, is a surrogate measurement for the organic carbon content in water. UV-254 measurements were taken from grab samples at the treatment plant and at 28 sites within the distribution system over the five-day changeover period. The distribution system sites included most of the TCR sampling sites in addition to selected tanks. The resulting database was used to assess the movement of water in the system, the travel time throughout the system, boundary zones in the distribution system between areas served by the surface water plant and secondary sources, and calibration/validation of the distribution system model.

3.1.4.6 Comparison of Tracers

Teefy (1996) investigated tracer alternatives for use in studies of residence time in clearwells and described the chemical characteristics of the individual tracers. Table 3-1 summarizes various chemical characteristics identified in that report.

There are advantages and disadvantages associated with each of the general types of tracers: conservative (non reactive) tracers, reactive tracers, chemicals that are normally added to the water but can be turned off, and natural chemical signatures in the finished water. Conservative tracers are more easily modeled than non-conservative tracers. Natural tracers or chemicals that can be turned off are easier to use than injected chemicals. Certain chemicals are more amenable to continuous monitors. These and other factors should all be considered when selecting a tracer for a study.

Table 3-1. Tracer Characteristics (adapted from Teefy, 1996)

	Fluoride	Calcium	Sodium	Lithium	Chloride
Commonly available forms	H_2SiF_6 NaF Na_2SiF_6	$CaCl_2$	NaCl	dry LiCl	$CaCl_2$ NaCl KCl
Analytical methods	IC, ISE, SPADNS method	AA, IC, ICP, EDTA titration Conductivity	AA, IC, ICP, FEP Conductivity	AA, IC, ICP, FEP	IC, ISE, $AgNO_3$ titration, $Hg(NO_3)_2$ titration
Typical chemical cost	Food-grade H_2SiF_6 $7.6/100 lb - 23.97% liquid[1] $140/55 gallons - 49% liquid[2]	Food-grade $CaCl_2$ $150/55 gallons - 35% liquid[3]	Food-grade NaCl $12/50 lb[4] $6/50 lb[5]	Lab-grade LiCl[6] $22 - $48/500g[7]	Food-grade NaCl $12/50 lb[4] $6/50 lb[5]
Typical analytical cost per sample	$18[8] (IC) $16[10] (IC) $12[11] (ISE) $25[12] (IC)	$10[8] (ICP) $12[10] (ICPMS) $5[11] (ICP)	$10[8] (ICP) $12[10] (ICPMS) $5[11] (ICP)	$12[8] (ICP[9]) $12[10] (ICPMS) $6[11] (AA[13])	$18[8] (IC) $16[10] (IC) $12[10] (EPA 325.3) $12[11] (IC)
Typical background levels in water distribution systems	0-4 mg/L	Varies greatly (1-300 mg/L), use only when low	Varies greatly (1-500 mg/L)	Usually below 5 mg/L	Varies greatly (1-250 mg/L)
Regulatory limits	4 mg/L SDWA MCL, 2 mg/L secondary MCL	None known. See limits for chloride.	20 mg/L for restricted diet (EPA recommendation)	None known. See limits for chloride.	250 mg/L secondary standard

[1] Provided by Lucier Chemical Industries (LCI), Ltd., http://www.lciltd.com
[2] Provided by Bonded Chemicals, Inc., http://www.chemgroup.com/bci.htm
[3] Provided by Benbow Chemical Packaging, Inc., http://www.benbowchemical.com
[4] Provided by Skidmore Sales and Distributing Company, Inc., http://www.skidmore-sales.com
[5] Provided by Ulrich Chemical, Inc., http://www.ulrichchem.com
[6] Food grade LiCl is not available.
[7] Provided by Science Kit & Boreal Laboratories, http://www.sciencekit.com
[8] Provided by Severn Trent Laboratories (STL) North Canton, Ohio, http://www.stl-inc.com. Prices are based on a large sample volume (> 500 samples).
[9] STL North Canton Laboratory is not certified for Lithium test in Ohio.
[10] Provided by SPL Laboratories, Inc., http://www.spl-inc.com Prices are based on a large sample volume (> 500 samples).
[11] Provided by Environmental Enterprises, Inc., http://www.eeienv.com Prices are based on a large sample volume (> 500 samples).
[12] Provided by FOH Environmental Laboratory for the CDC study at Camp Lejeune, NC. http://www.foh.dhhs.gov/. The analytical cost per sample includes cost for providing a sample bottle and report.
[13] Environmental Enterprises, Inc. is not certified for Lithium test.

Note: Tracers

$CaCl_2$	calcium chloride
H_2SiF_6	hydrofluosilicic acid
KCl	potassium chloride
LiCl	lithium chloride
NaF	sodium fluoride
Na_2SiF_6	sodium silicofluoride
NaCl	sodium chloride

Analytical methods

AA	atomic absorption spectrometry
$AgNO_3$	silver nitrate
EDTA	ethylenediaminetetraacetic acid
FEP	flame emission photometric method
$Hg(NO_3)_2$	mercuric nitrate
IC	ion chromatography
ICP	inductively coupled plasma
ISE	ion selective electrode
SPADNS	Trisodium (4,5-Dihydroxy-3-[(p-sulfophenyl)-2,7-) naphthalene disulfonic acid

After investigating tracer options and selecting the most appropriate tracer, the governing state drinking water agency should be contacted. The agency should be provided with the specifics regarding the proposed study including location(s), proposed timeline(s) and selected tracer material. Once agreement has been reached and consent is received, the study team can then proceed with the next steps in the planning process.

3.1.5 Selecting Field Equipment and Procedures

Once a tracer has been selected and approval has been received from the appropriate water utility managers and regulatory agencies, specialized equipment must be identified and procured, including injection pumps, temporary tracer storage tanks, and various flow and tracer monitoring equipment (e.g., tracer chemical, reagents, and/or sample bottles). Vendors should be contacted for technical information, equipment availability, and cost quotations for the required field equipment and analytical instrumentation. The major decisions to be made and the items to be purchased prior to the execution of the study are discussed in the following subsections.

3.1.5.1 Injection Pump(s)

Pumps that are typically used in drinking water applications can be broadly classified as centrifugal pumps or positive displacement pumps. The centrifugal pumps produce a head and a flow by increasing the velocity of the liquid with the help of a rotating vane impeller. The positive displacement pumps operate by alternating between filling a cavity and displacing the volume of liquid in the cavity. The positive displacement pumps deliver a constant volume of liquid (for a given speed) against varying discharge pressure or head. By design, the positive displacement pumps are better suited to serve as an injection pump for a tracer study. Examples of positive displacement pumps include: rotary lobe, progressing cavity, rotary gear, piston, diaphragm, screw, and chemical metering pumps (e.g., bellows, diaphragm, piston, and traveling cylinder).

Selection of the most appropriate positive displacement pump depends upon the injection rate, the pressure in the receiving system, the chemical characteristics of the tracer, and local experience and preferences. Two types of positive displacement pumps have generally been used in tracer studies: gear pumps and metering pumps. The final selection depends upon viscosity of the tracer material, variability of pressure in the main, dosage accuracy needs, and other local factors. Furthermore, to control the drive speed (i.e., dosage), these pumps are equipped with alternating current (AC) or direct current (DC) motor. If a pump has an AC motor, frequency is adjusted; if it is equipped with a DC motor, voltage is adjusted to control speed.

EPA has used gear pumps equipped with variable frequency drives in the past with success for conducting tracer studies. Other studies have reported success with metering pumps with variable speed or variable stroke controllers. The pump should be sized in accordance with the anticipated tracer dosage (for more details, see Tracer Dosage and Injection Duration Section 3.2.3) and pressure range in the main pipe for the selected injection location(s) in the study area. Depending upon the location and dosage requirements, more than one size of pump may be needed (excluding backup pumps).

3.1.5.2 Tracer Storage and Dosage Rate Measurement

Tracers are available in dry or liquid form. If purchased as a powder, provisions for mixing the powder with water must be made. If the tracer is purchased in liquid form, it typically comes in either 55-gallon drums or in larger containers such as a 330-gallon tote. If only a small amount of tracer is needed, a single 55-gallon drum will typically suffice. For greater accuracy, it is recommended that the tracer be transferred from 55-gallon drums to a suitably sized day tank with a sight glass (used to periodically monitor the total tracer volume dosed). It is easiest to pump the tracer from a single container rather than having to switch the pump from container to container during the injection process. Details on tracer dosage calculations are presented in Section 3.2.3.

If a metering pump is purchased, care must be taken so that the pump flow rate is calibrated for the specific tracer solution (by the vendor). Furthermore, the

During a tracer study when a tracer chemical is being injected into the system, in order to meet water quality regulations and to simplify the modeling, it may be desirable to maintain a constant tracer concentration in the receiving pipe. This can be accomplished by monitoring the resulting concentration in the receiving pipe and manually adjusting the tracer injection rate or through the use of a closed-loop system for automatically controlling the injection rate based on flow in the receiving pipe. The automated process is most effective at a location where the flow in the pipe is varying relatively slowly and where a flow meter exists. A typical situation is the use of an existing venturi meter that generates a 4-20 milli-ampere (ma) signal. This signal can be used as input to a controller that has been calibrated and programmed to control the stroke or speed of a variable stroke or speed injection pump. If the flow in the receiving pipe is varying rapidly over a large flow range, it is difficult for the closed-loop system to respond quickly.

variable area flow meters (rotameters - with floats contained in an upright conical tube) are relatively inaccurate for measuring tracer dosage even after adjustments are made for density and viscosity. Figure 3-1 shows a "flow tube" that can easily be custom fabricated and calibrated to accurately measure the rate of tracer injection. It is recommended that the supply tank also be marked to keep track of the tracer fluid level. Times should be noted at each mark so that it is possible to create a mass balance for the tracer injected during the study.

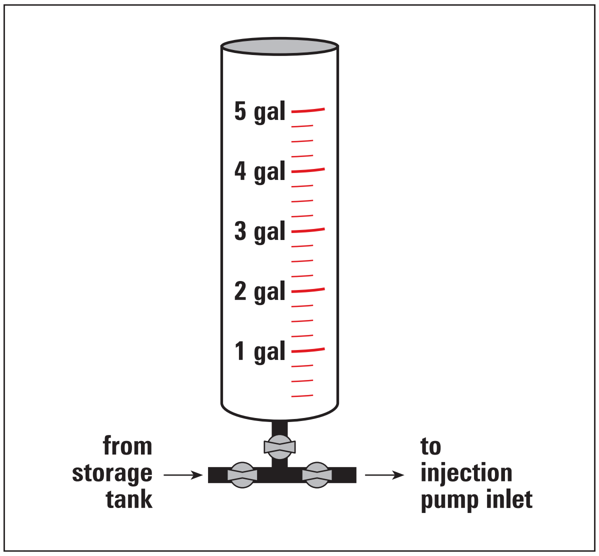


Figure 3-1. Flow Calibration Tube.

3.1.5.3 Distribution System Flow Rate Measurement

In order to calculate the concentration of the tracer in the receiving pipe, it is necessary to know the flow rate in the pipe, the injection rate of the tracer, the injected concentration of the tracer, and the background concentration in the water before tracer is added. Flow rate should be measured continuously, because variations in pipe flow rate can affect tracer concentration. These fluctuations in flow can be accommodated by manually adjusting the tracer injection rate in the field or through the use of a flow-paced injection pump that responds to the flow in the receiving pipe.

Placement of additional flow meters or other flow measuring devices at various points in the system is recommended. This information will be very useful during the post-tracer modeling studies and is invaluable in calibrating a network hydraulic model. If the existing system does not have an adequate number of flow meters for purposes of a tracer study, installation of additional meters is recommended.

Various types of flow meters may be used to measure flow in pipes. They are categorized as either non-intrusive or intrusive meters. Portable ultrasonic flow meters are non-intrusive and provide reasonably accurate data if the pipe material is conductive and relatively non-tuberculated. The ultrasonic flow meter requires suitable upstream/downstream straight runs of pipe. Insertion flow meters are also an option for measuring pipe flow rates. Insertion meters are intrusive, and may be magnetic (magmeters) that are flange coupled to the pipe or have propellers that must be inserted through a hole in the pipe. All meters require that the receiving main pipe be exposed (via excavation) or that an existing vault be used. If the injection location is in the vicinity of a reservoir/tank and the water level changes are available in real time, it may, in some instances, serve as a rough surrogate for in-pipe flow measurement. The selected method of flow measurement must be field tested.

> Depending upon the size of the reservoir/tank and the local demand, the reservoir level changes may not be fast or accurate and precise enough to determine the flow rate in real time.

3.1.5.4 Field Measurement of Tracer Concentration

Tracer concentration may be measured in the field using either automated monitors that analyze a sample at a preset frequency, by collecting "grab" samples, or a combination of both. Grab samples can be manually analyzed in the field or in the laboratory.

If grab sampling is used during a tracer study, the sampling team will generally traverse a circuit of several sampling locations. Using such an approach will generally yield a sampling frequency of one sample per station every one to three hours for an average-sized residential neighborhood (unless multiple crews are used). Some of the factors that will influence sampling frequency include the speed at which the tracer is moving within the distribution system, the number of sampling crews participating in the study, the number of sampling sites selected, the time of the day, and the distance between sampling sites. Equipment requirements for grab sampling are minimal and may include the following: coolers, ice, labeled sample bottles, log books, and temperature blanks. If samples are to be analyzed in the field, the sampling teams will need the appropriate analytical equipment. If samples are to be analyzed in the laboratory, the team will need the means to properly store and transport samples to a central laboratory. The quality assurance project plan (QAPP) may require duplicate or split samples for some or all of the primary samples. When taking a grab sample, care must be taken to flush the tap for a sufficient time to ensure that the sample is representative of the distribution main rather than the service lines.

Reliance solely on grab sampling may be impractical if the study area is large, the tracer front is moving rapidly, or a high frequency of sampling is desired. In these cases, continuous automated monitoring may be the best choice although some grab samples for quality assurance and quality control are recommended. If calcium chloride or sodium chloride is the tracer selected, an online specific-conductivity meter equipped with an associated data logger is recommended. Automated monitors are available if chlorine residual is used as a tracer. There are also automated monitors available if fluoride is used as a tracer, but there has been relatively limited use under field conditions. Since most automated monitors require a continuous side stream (rather than being inserted directly into a main), the drainage flow from the monitor must be discharged into a sewer, into the street and subsequently into a storm drain, or into a pervious area. This discharge can be an added complication during cold weather when it may freeze. Since this discharge stream is generally chlorinated or chloraminated, regulations may control discharge into natural water courses. Additionally, this discharge flow may have to be accounted for if the data set is being used to calibrate a distribution system model, and the quantity of discharge through a particular meter is significant relative to the demand in the vicinity of the meter. If the total drainage discharge is significant for the purposes of modeling, provisions for continuously or manually measuring the amount of flow being bypassed are needed.

Potential grab and online sampling sites include: dedicated sampling taps, hydrants, pump stations, tank inlet-outlet lines, and faucets located inside or outside of buildings. Figure 3-2 depicts an automated monitoring station used by EPA. This figure illustrates the case where the sampling tap is allowed to run continuously throughout the study with the water going to a drain. The flow rate to or through the sampling tap must be sufficient to minimize the travel time from the main to the monitor.

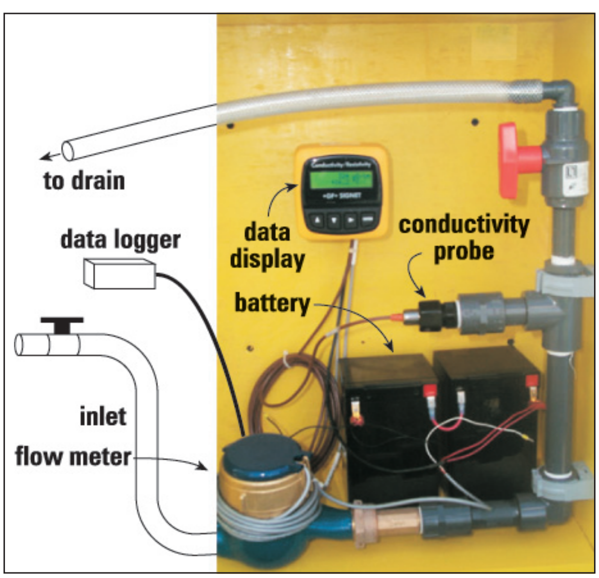


Figure 3-2. Automated Monitoring Station.

EPA and GCWW have pioneered the use of online monitors as a central focus for distribution system tracer studies. In a series of field tests, EPA and GCWW injected calcium chloride tracer into the water system and followed the movement of the tracer using automated conductivity meters strategically placed throughout the study area. Three separate studies were conducted in a large water system representing a small highly urbanized area, a small dead-end suburban area, and a large suburban pressure zone. Based on the success of these studies, similar tracer studies have been conducted utilizing a combination of online monitors and grab samples by the CDC using both fluoride and sodium chloride as tracers in Hillsborough County, Florida (Boccelli et al., 2004) and by the Agency for Toxic Substances and Disease Registry (ATSDR) using fluoride and calcium chloride at a large military base in North Carolina (Maslia et al., 2005; Sautner et al., 2005).

Online, automated sampling programs should be complemented with a grab sampling program to add a degree of confidence in measured data and to supplement field data at additional locations or at the automated monitor stations if they fail to record correctly.

3.1.6 Developing a Detailed Study Design

A key element in planning and designing a tracer study is the preparation of a study design document. This document serves as the overall plan for conducting a tracer study and thus, the roadmap for execution of the study. Three important study-specific parts of the design plan that may be required before the execution phase are a QAPP, a Health and Safety Project Plan (HSPP), and a contingency plan. The contingency plan describes the actions to be taken if unexpected events occur; for example, if distribution system concentrations of the tracer exceed the MCL for chloride or fluoride. The HSPP should at a minimum define the job hazards that might be encountered and the controls, protective equipment, sample handling and work practices, safety review procedures, and emergency procedures to be employed during the study.

The QAPP should clearly define the project objectives, organization, experimental approach, sampling procedures, analytical methods, protocols, instrument calibration requirements, data reporting, data reduction, and data verification procedures.

3.1.7 Addressing Agency and Public Notification

Appropriate agencies, including fire and police departments, should be notified prior to the commencement of field activities. With heightened awareness of security, all people participating should have a valid identification and contact information. A standard statement concerning the study should be developed and provided to all team members in case they receive inquires at the study site. This same statement should be used by utility personnel to answer any telephone inquires that might be received.

> A summary information card may be provided to the study participants that could be handed out to the public during the study (if requested). This minimizes the risks of mis-communication.

If the injection site or installation of meters requires excavation, the study team must obtain the necessary permits and approvals. This is especially important if any of the sites are in a residential neighborhood or near a busy street or road. Care should be taken in all cases to provide adequate traffic control. Safety is of paramount consideration.

3.2 Executing a Tracer Study

The team should first become familiar with the detailed study design documents discussed in Section 3.1.6. Based on these documents, there are several tasks that need to be completed during the execution phase of a tracer study. These tasks include:

- Procurement, setup, testing, and disinfection of study equipment (including pumps, storage tanks, chemicals, reagents, tubing, connectors, and continuous tracer monitoring stations).
- Installation of field equipment and testing (both flow and tracer monitoring equipment to confirm study-specific distribution system operation and flow stability).
- Tracer dosage and injection duration calculations.
- “Dry runs” and planned tracer injection events.
- Real-time field assessments, sampling, and analysis.
- Equipment demobilization, initiation of data collection, reduction, and verification process.

These specific execution subtasks are further discussed in the following sub-sections.

> A tailgate safety meeting before commencement of any field work is the best method to increase awareness.

3.2.1 Procurement, Setup, Testing and Disinfection of Study Equipment

Field equipment identified under Section 3.1.5 and its subsections should be procured on a timeline such that the items arrive several weeks before the planned study date, especially the monitoring and injection equipment that may require assembly. An early arrival will ensure that the equipment can be properly configured and tested before field use.

Unless pre-calibrated flow-paced injection equipment is purchased (or if the study does not require injection equipment – as in the case of using naturally/normally occurring tracers), the study team should obtain an appropriate injection pump setup. Figure 3-3 shows a picture of a tracer injection system used by EPA for field tests. This setup should be calibrated in the lab to compute the speed-specific dosage rate using the tracer solution. If appropriate, a flow-calibration tube should also be fabricated to confirm the flow in the field. Figure 3-3 also depicts a flow-tube used by EPA.

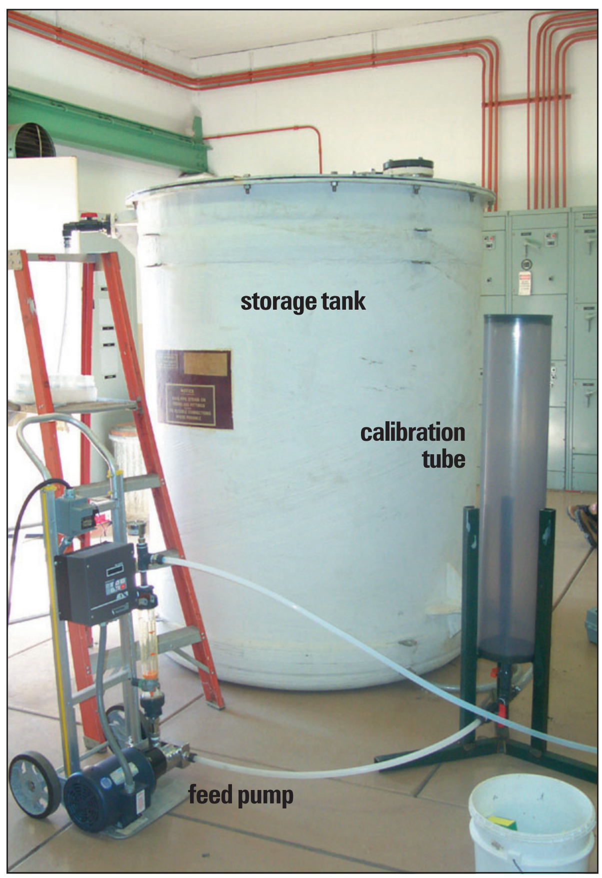


Figure 3-3. Tracer Injection Setup (Storage Tank, Calibration Tube and Feed Pump).

Concurrently, if applicable, the team should initiate the fabrication of the automated tracer monitoring stations. These stations are typically equipped with a probe for measuring the tracer (or a surrogate parameter such as conductivity), associated data logger, and batteries (for powering the probe and the data logger). If accurate measurement of flow through the automated monitoring station is needed, it should be augmented with a household-style water meter and logger. The equipment should be housed in a secure lock box to protect it during the field study. Figure 3-2 shows an automated monitoring station used by EPA and GCWW to conduct a tracer study. The entire setup should be tested in the lab to ensure proper operation and battery capacity to maintain uninterrupted operation.

The grab sampling, laboratory equipment, tracer storage tanks, transportation equipment, and arrangements should be procured and set up. The field equipment hookup, including interconnections between the tracer storage tank, injection pump, and flow-tube, should be leak tested. The equipment used for injection should be properly disinfected and tested prior to field deployment to ensure that no microbiological contamination results from the field tests.

If ultrasonic flow meters are procured for field deployment, the equipment should be set up in a lab environment to confirm the individual component operation and approximate battery life. The existing flow and data acquisition systems to be used in the field study should be sampled for data accuracy and field communication.

During the lab testing phase of the field equipment, the entire field (and backup) crew should familiarize themselves with proper operating procedures for the equipment they are designated to operate.

One procedure for equipment disinfection is to prepare approximately 50 gallons of 50 ppm chlorine disinfectant solution. This solution is then re-circulated through the injection pump setup for about 15 minutes. Thereafter, continuously flush the injection pump using de-ionized water for about 15 minutes. Collect a water sample at the end of the flush cycle and send it for bacteriological analysis (Coliform and E. coli) to insure that the disinfection procedure was successful. For the purposes of sampling, use sterile sample bottles with a de-chlorinating agent (e.g., sodium thiosulfate). The de-chlorinating agent is added to remove any residual chlorine or other halogen that may continue the disinfection process in the sample and yield incorrect test results.

3.2.2 Installation of Field Equipment and Testing

Prior to the commencement of field activity, a brief “tailgate” health and safety meeting should be conducted at the beginning of each day to remind the crew of potential job hazards. Mobilization of field equipment for excavations (if required – for installing main flow meters) should be initiated to allow for the flow monitoring devices to be installed prior to the scheduled injection event(s). This time lag will vary according to the needs of the specific study and could range from several days to several weeks. The early installation of flow meters will allow the study team to capture actual field flow data for performing any revisions to tracer dosage computations and preliminary hydraulic modeling analysis. The flow meter installation location should meet the manufacturer’s recommendations for upstream and downstream straight lengths of undisturbed pipe. The excavations should be performed in accordance with the HSPP. Appropriate drainage for the excavated pits should be arranged in case rain is forecast during the study period.

The measured field flow data should be utilized to confirm the stability and range of flow at the injection location and other major branches of the system where flow is monitored. It may be necessary to operate the distribution system under specified conditions in order to achieve optimum results during the study. The operational changes that may be required include: scheduled cycling of tank levels, pumps, and valves. Time required for the deployment of the automated monitoring stations prior to the start of the tracer tests is dependent upon several factors, including the number of monitoring stations, the distances between stations, the ease of attaching the stations to the sampling hydrants, and the effort required to calibrate the monitoring equipment. If feasible and consistent with normal operating policies, the system should be operated to avoid frequent abrupt changes in flow such as would be associated with a pump that was cycling on and off very rapidly.

A day or two prior to the execution of the tracer injection event, the study team should fully deploy the continuous monitoring stations (if used). These stations should be hooked up at the designated sampling locations and data logs should be checked to ensure data are being collected. Flow through a monitoring station should be sufficient to minimize the time delay in detecting the injected tracer between the main and the sampling location. Experience has shown that 1 to 2 gallons per minute (gpm) is usually sufficient. The field crew should also test the coverage and reliability of field communication devices (such as cellular phones) in the designated study area.

3.2.3 Tracer Dosage and Injection Duration Calculations

Factors affecting the amount of tracer required for the study include the duration of the injection, the flow rate in the receiving pipe, and the target concentration in the distributed water. This target concentration should be consistent with drinking water standards. For example, if fluoride is being injected (into a system that does not fluoridate) with a secondary MCL of 2 mg/L, a reasonable target concentration level is 80% of the MCL, i.e., 1.6 mg/L. The injection rate should be set to meet that goal.

Using the principle of material balance, the resulting tracer concentration in a receiving pipe downstream of the point of injection can be calculated as follows:

$$Q_D = Q_U + Q_T \quad \text{(Equation 3-1)}$$

$$C_D = \frac{(C_B \bullet Q_U) + (C_T \bullet Q_T)}{Q_D} \quad \text{(Equation 3-2)}$$

Where

Q_D = flow downstream of injection point, L^3/T

Q_U = flow upstream of injection point, L^3/T

Q_T = flow of tracer solution, L^3/T

C_D = concentration of tracer material downstream of injection point, M/L^3

C_B = background concentration of tracer material in distributed water, M/L^3

C_T = tracer concentration, M/L^3

Equation 3-1 represents continuity and Equation 3-2 represents conservation of mass. As written, these equations are independent of units for mass (M), length (L), and time (T) as long as consistent units are used for computations. However, when tracer concentrations, injection rates, and injection duration are used to calculate the required volume of tracer material purchased, units for flow, concentration, and time must be commensurate or appropriate conversion factors must be employed.

For some tracers, the allowable concentration in the distributed water may be controlled by one of the dissolved ions that are part of the tracer. For example, if calcium chloride is the selected tracer, the concentration of the chloride ion in the distributed water controls the amount of tracer that may be injected.

Injection duration depends upon the size and complexity of the distribution system, and the modeling objectives of the study. A typical duration can range from one hour in a small or branched system, to eight hours or more in a larger, looped system. Some studies have reported success with a series of pulses. However, if the duration of the injection is too short or the series of pulses too close together in time, it is difficult to separate the tracer fronts as they traverse different paths at different velocities through the looped systems. The presence of tanks can also impact the needed tracer duration since active filling and drawing can dampen the resulting tracer concentration as it moves through the system.

The injection equipment should be located close to the main in order to minimize the tracer travel time to the main. Alternatively, the travel time should be compensated for during the appropriate phases of the study evaluation.

3.2.4 Dry Runs and Planned Tracer Injection Event(s)

Before the planned full-scale tracer injection event is actually carried out, the project team should consider conducting a smaller duration dry run injection to confirm the system operation and expected levels of tracer concentration. If continuous monitors are to be used in the study, then during the dry run some or all of the monitors should be installed and tested. The timing and duration of the dry run should be such that the injected pulse should be short and clear the system well before the actual event is initiated.

The dry run serves as a final systems check and provides the study team an opportunity to make any necessary last minute changes prior to the actual study. Thereafter, the actual full-scale injection event should be conducted as planned.

3.2.5 Real Time Field Assessments, Sampling, and Analysis

While the injection event is ongoing, the study team should carefully monitor the tracer concentration at the immediate downstream location of the injection to ensure that there are no significant deviations in the expected versus observed concentrations in the field. Field crews should communicate directly with the system operations. It is critical that the field personnel are aware of any changes in system operations that may affect the study. Unanticipated changes in water demand may cause the tracer concentration to exceed target concentration levels. In such an event, the field crew should be trained to take measures to minimize any adverse effects. The preventive measures may include lowering (or stopping) the injection rate, or achieving appropriate dilution by means of rerouting water through the distribution system (as appropriate). Furthermore, any such tracer concentration exceedances should be confirmed by performing field grab sample analysis to make sure that the exceedance is real and not an instrument anomaly. Until the results are confirmed, it is best to err on the safe side and take preventive measures to maintain water quality.

Periodically, the field crew should take grab samples and inspect the continuous monitoring stations to ensure that the equipment is operating properly. The grab samples should be appropriately handled and analyzed in the field or transported to the laboratory for further analysis. The sampling and monitoring effort should continue well past the conclusion of the injection event until the tracer is expected (and observed) to have moved out of the system. This may take a period of 24 to 48 hours or more after completion of the injection event.

During the course of the sampling event, it is very useful to examine and assess the field data on a near real-time basis. Questions that should be asked include "Are the results reasonable?" "Is the tracer moving through the system at a speed consistent with predictions?" Based on this assessment, modifications may be made in terms of injection rate, grab sampling frequency, or study duration.

3.2.6 Equipment De-Mobilization, Initiation of Data Collection, Reduction, and Verification Process

After the scheduled injection event(s) are completed, the field crew should download the data (including flow and tracer concentrations) from the various monitoring devices. The data should be spot checked against field grab sampling data to ensure that there are no time anomalies or gaps in the data log and the readings match relatively well.

After the field sampling events are completed, the crew should de-mobilize the equipment, remove the automated monitoring stations, refill any excavations, and restore the system operations to their normal conditions.

Downloaded data from the field should be processed according to the QAPP and used for further modeling and analysis. The use of field data in calibration and validation of hydraulic and water quality models is discussed further in Chapter 4.

3.3 Tracer Study Costs

In general, the cost of conducting a tracer study is proportional to the study area size, number of monitoring sites, study duration, sophistication and amount of equipment, and complexity of post-study analysis. If a study incorporates an injected tracer and the use of continuous monitors, it can be much more expensive initially than a study using a natural tracer and grab samples. However, the injection equipment and continuous monitoring equipment can be reused at various locations. These are the cost tradeoffs between purchase of automated monitoring equipment and labor associated with grab sampling. In some cases, a larger dataset derived from an automated monitor is necessary for a detailed analysis. Cost data presented in this section are intended to provide the basis for this type of analysis. For the purposes of this chapter, the overall costs have been broken down into two distinct categories: equipment and labor. Material costs are only a fraction of the total, and therefore, have been combined and included with equipment costs for simplicity.

Table 3-2 lists typical equipment and material costs for those items that may be used in tracer studies. The unit costs can be easily scaled to the needs of a specific study. Chemical tracer costs, including analytical costs, were provided earlier in Table 3-1.

Costs may vary widely among studies. For example, if it is necessary to purchase or rent a storage tank or a

Table 3-2. Equipment Costs

Equipment & Material	Unit Cost ($)
Injection pump	$1,000 - $5,000
Flow meter (ultrasonic meter for main pipes)	$7,000 - $9,000
Excavation, rigging and backfill (equipment rental per site)	$1,500
Lab chemicals, batteries and plumbing supplies (lump sum*)	$1,000 - $5,000
Automated monitoring box (self constructed)	< $200
Online conductivity ISE, meter and logger	$800 - $1,500
Automated monitoring station water flow meter	$600 - $800
Online fluoride meter	$5,000 - $10,000
Safety equipment (e.g., vests, first aid kits, rain gear, and flashlights)	$500 - $1,000
Communication equipment (e.g., radios and GPS)	$500 - $1,000
Hydrant equipment (e.g., wrenches, caps, and hoses)	$1,000 - $2,000
Transportation (e.g., rental vehicles)	$500 - $2,000
Tracer storage tanks (depending upon volume and material)	$500 - $1,000

truck, costs will be higher if these types of items are not readily available. If the study team elects to analyze samples in-house rather than using an outside laboratory, the team should balance the cost of labor, and the cost of additional reagents and chemicals against the cost of performing the analyses at an outside commercial laboratory. Labor costs may be even more variable than equipment and material costs and are a function of the size and complexity of the study. In order to provide an easy basis for comparison, the labor costs are presented in labor hours (Table 3-3) and include a combination of engineers and technicians. Labor hours have been estimated for low, medium, and high-end studies. These estimates are obtained from actual field studies, as described below. This approach should allow utilities to make site-specific cost estimates.

Table 3-3. Representative Labor Hours for a Range of Studies

Activity	Low-End	Medium	High-End
Planning	27	274	480
Setup	-	150	520
Field study	51	604	370
Laboratory analysis	8	160	120
Post-study assessment	24	212	740
Total	110	1,400	2,230

A typical example of a low-end tracer study is provided by the Sweetwater Authority distribution system in Southern California (see second sidebar in Section 3.1.4.5, page 3-6). The Sweetwater system covers a service area of 28 square miles. The utility was able to take advantage of a naturally occurring tracer and used grab samples taken at 28 existing dedicated sampling sites over a period of 5 days. A study performed in the 21-square-mile Cheshire service area of the South Central Connecticut Regional Water Authority in 1989 (see second sidebar in Section 3.1.4.1 on page 3-4) provides an example of a medium-level tracer study. In this case, the normal fluoride feed was shut off for a period of 7 days (and then turned back on) and grab samples were taken at intervals of a few hours at 23 sites over a period of 14 days. An example of a high-end study is provided by a two-phased field investigation conducted in two suburban areas of GCWW. The first area is a small (<1 square mile) dead-end system, and the second area, a 12-square-mile pressure zone. A calcium chloride tracer was injected and monitored using a combination of automated conductivity meters and grab samples. In the smaller area, 20 meters were used and monitoring was conducted over a 24-hour period. In the second area, 33 meters were used and two separate tracer injections were conducted over a period of 5 days. Including both studies, a total of 725 grab samples were taken and analyzed for conductivity, chloride, and calcium. Flow was monitored at four locations using ultrasonic flow meters.

Table 3-3 presents estimated labor hours for these types of studies. They are divided into the planning phase (as described in Section 3.1); setup, field work, and laboratory analysis that together make up the execution phase (see Section 3.2); and the post-study modeling, assessment, and report phase. As illustrated in this table, there is a significant variation in the labor hours required to conduct a tracer study. For example, the low-end labor costs resulted due to the following study characteristics: naturally occurring tracer was used, no new equipment was purchased, existing routine monitoring sites were used, and only a limited post-study assessment was made. The medium-sized study included the following characteristics: a chemical that was routinely added (fluoride) to the water distribution system was used as the tracer (by shutting it off), the study required a much longer period to complete, and since it was the first major tracer study in the distribution system, it required significant planning. The high-end study included the following characteristics: it was the first major tracer study employing wide-scale use of continuous monitors; a non-naturally occurring, non-routinely added chemical was injected as a tracer; and significant time was required for acquiring and installing the equipment. For purposes of this study, a very detailed post-study data assessment involving processing of tracer study data, pipe network model calibration and report preparation required significant labor expenditures. Examples of model calibration efforts associated with tracer studies are presented in Chapter 4.

3.4 Summary, Conclusions and Recommendations

Tracers and tracing techniques have been used for many years in a number of engineering applications to estimate stream velocity and retention time in water and water supply unit processes. More recently, tracers have been used for calibrating drinking water distribution system hydraulic and water quality models. For the purposes of this document, it is assumed that tracer studies are used to calibrate and validate network models. The calibrated and validated network models are then used to estimate other parameters such as water age and travel times. However, the data from a tracer study can be directly used to estimate some specific parameters such as water age (DiGiano et al., 2005). A comprehensive summary of potential uses and regulatory applications for tracer studies is provided in the first subsection of this chapter. Drinking water tracers might include chemicals that are injected into a water distribution pipe, the temporary shutoff of a chemical additive currently being added to treated water (such as fluoride), or significant changes in concentration of disinfectants, DBPs, or natural compounds. The tracer methodology selected would significantly impact the overall costs of the study. Probably, the most expensive option would be to inject a chemical tracer, monitor it using leased or purchased online instrumentation, and conduct the study using contractor staff. The least expensive approach would be to take advantage of a natural tracer, monitor the progress of the tracer by grab sampling, and conduct the study using primarily in-house staff. Once a tracer "injection" methodology has been selected, careful planning and execution will ensure the success of the study.

When planning a tracer study, if the specific steps outlined in this chapter are followed, they should greatly increase the potential for a successful study. These steps include: establishing clear study objectives, forming a study team, defining the study area characteristics, carefully selecting an appropriate tracer, selecting the proper field equipment, developing key planning documents, and ensuring that the public and affected agencies are notified. Application of a distribution and water quality model during the planning stage is highly recommended to simulate the approximate behavior that will be expected during the actual tracer event.

During the execution phase of the study, the following issues should be addressed: procurement of equipment and materials; setup, testing and disinfection of the procured equipment; availability of analytical instrumentation and laboratory facilities; and, finally, the installation, testing, and operation of field equipment. During the execution phase, it is important to review and understand how tracer dosages and injection duration are to be implemented. Dry runs are highly recommended as a means of debugging the procedures prior to a full study.

Distribution system tracer studies have been conducted for over 15 years, but recent technology developments have improved the efficiency of these studies and provide promise for greatly expanded applications in the future. Specific components that will fuel this expanded use include the following: continuous monitors that can be easily adapted for use in distribution systems are being developed and tested, in part in response to water security concerns; automated meter reading (AMR) equipment is being installed by many utilities and could provide more detailed temporal and spatial consumption data for hydraulic models; advanced analysis software is evolving that will facilitate the use of large amounts of continuous data in calibrating distribution system models; and with increased availability of these technologies, costs are expected to decrease so that larger utilities can afford to purchase and routinely use the equipment, and consulting engineers can affordably offer these services to smaller utilities.

During the field study, it is important that the study team be able to assess the progress of the tracer, in real time, as it propagates through the system. Concise and consistent communications between tracer study team members, test coordinator, and water utility staff, is critical al all times during the test.

In the future, it is highly likely that advances currently on the horizon will result in significant increased use of both online tracer (or water quality) monitors and flow monitoring instrumentation. As the on-line technology becomes more widely used in drinking water, the use of network water quality models will also be more widely accepted. Online monitoring in conjunction with water quality modeling will provide an in-depth understanding of the manner in which water quality changes can be monitored in a drinking water distribution system. Also, given the current climate of concern over distribution water quality from both a regulatory and security viewpoint, it is reasonable to assume that there will be increased interest in applying this type of technology in the water industry.

References

Boccelli, D.L., F. Shang, J. G. Uber, A. Orcevic, D. Moll, S. Hooper, M. Maslia, J. Sautner, B. Blount, and F. Cardinali. "Tracer Tests for Network Model Calibration." Proceedings, ASCE-EWRI Annual Conference. 2004.

Boccelli, D., and J. Uber. "Incorporating Spatial Correlation in a Markov Chain Monte Carlo Approach for Network Model Calibration." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

Boulos, P.F., W.M. Grayman, R.W. Bowcock, J.W. Clapp, L.A. Rossman, R.M. Clark, R.A. Deininger, and A.K. Dhingra. "Hydraulic Mixing and Free Chlorine Residuals in Reservoirs," *Journal of AWWA*, 88(7):48-59. 1996.

Clark, R.M., W.M. Grayman, R. Males, and A. F. Hess. "Modeling Contaminant Propagation in DrinkingWater Distribution Systems," *Journal of Environmental Engineering, ASCE*, 119(2):349-364. 1993.

Clark, R.M., W.M. Grayman, J.A. Goodrich, R.A. Deininger, and A.F. Hess. "Field Testing Distribution of Water Quality Models," *Journal of AWWA*, 83(7):67-75. 1991.

Clark, R.M., G. Smalley, J.A. Goodrich, R. Tull, L.A. Rossman, J.T. Vasconcelos, and P.F. Boulos. "Managing Water Quality in Distribution Systems: Simulating TTHM and Chlorine Residual Propagation," *Journal of Water Supply Research and Technology – Aqua*, 43(4):182-191. 1994.

Clark, R.M., and J.A. Coyle. "Measuring and Modeling Variations in Distribution System Water Quality," *Journal of AWWA*, 82(8):46-53. 1990.

DiGiano, F.A., and G. Carter. "Tracer Studies to Measure Water Residence Time in a Distribution System Supplied by Two Water Treatment Plants." Proceedings, AWWA Annual Conference. 2001.

DiGiano, F.A., W. Zhang, and A. Travaglia. "Development of the mean residence time from tracer studies in distribution systems." *Journal of Water Supply: Research and Technology - Aqua* 54:1-14. 2005.

EPA. *The Stage 2 DBPR Initial Distribution System Evaluation Guidance Manual.* 2003a.

EPA. *Drinking Water Advisory: Consumer Acceptability Advice and Health Effects Analysis on Sodium.* Available at: http://www.epa.gov/safewater/ccl/pdfs/reg_determine1/support_cc1_sodium_dwreport.pdf. 2003b.

Ferguson, B.A. and F.A. DiGiano. "Impact of temporary switches from monochloramine to free chlorine on water quality in distribution systems." Proceedings, AWWA Annual Conference. 2005.

Grayman, W.M., R.A. Deininger, A. Green, P.F. Boulos, R.W. Bowcock, and C.C. Godwin. "Water Quality and Mixing Models for Tanks and Reservoirs," *Journal of AWWA*, 88(7):60-73. 1996.

Grayman, W.M., L.A. Rossman, C. Arnold, R.A. Deininger, C. Smith, J.F. Smith, and R. Schnipke. *Water Quality Modeling of Distribution System Storage Facilities.* AwwaRF and AWWA. Denver, CO. 2000.

Grayman, W.M. "Use of Tracer Studies and Water Quality Models to Calibrate a Network Hydraulic Model," *Current Methods*, 1(1):38-42, Haestad Press, Waterbury, CT. 2001.

Grayman, W.M., L. A. Rossman, R. A. Deininger, C. D. Smith, C. N. Arnold, and J. F. Smith. "Mixing and Aging of Water in Distribution System Storage Facilities," *Journal of AWWA*, 96(9):70-80. 2004.

Hatcher, M.D., W. M. Grayman, C. D. Smith, and M. Mann. "Monitoring and Modeling of the Sweetwater Authority Distribution System to Assess Water Quality." Proceedings, AWWA Annual Conference. 2004.

Maslia, M.L., J.B. Sautner, C. Valenzuela, W.M. Grayman, M.M. Aral, and J.W. Green, Jr. "Use of Continuous Recording Water-Quality Monitoring Equipment for Conducting Water-Distribution System Tracer Tests: The Good, the Bad and the Ugly." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

Panguluri, S., R. Krishnan, L. Garner, C. Patterson, Y. Lee, D. Hartman, W. Grayman, R. Clark and H. Piao. "Using Continuous Monitors for Conducting Tracer Studies in Water Distribution Systems." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

Sautner, J.B., M.L. Maslia, C. Valenzuela, W.M. Grayman, M.M. Aral, and J.W. Green, Jr. "Field Testing of Water Distribution Systems at U.S. Marine Corps Base, Camp Lejeune, North Carolina, in Support of an Epidemiological Study." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

AWWA. *Standard Methods for the Examination of Water and Wastewater, 20th Edition.* Edited by: Lenore S. Clrsceri, Arnold E. Greenberg, and Andrew D. Eaton, American Pubic Health Association/AWWA/ Water Environment Federation, pp 2-44 to 2-45. 1998.

Teefy, S.M., and P.C. Singer. "Performance Testing and Analysis of Tracer Tests to Determine Compliance of a Disinfection Scheme with the SWTR." *Journal of AWWA*, 82(12):88-98. 1990.

Teefy, S.M. *Tracer Studies in Water Treatment Facilities: A Protocol and Case Studies.* AwwaRF and AWWA, Denver, CO. 1996.

Vasconcelos, J.J., L.A. Rossman, W.M. Grayman, P.F. Boulos, and R.M. Clark. "Kinetics of Chlorine Decay," *Journal of AWWA*, 89(7):54-65. 1997.

Vasconcelos, J., P. Boulos, W. Grayman, L. Kiene, O. Wable, P. Biswas, A. Bahri, L. Rossman, R. Clark, and J. Goodrich. *Characterization and Modeling of Chlorine Decay in Distribution Systems.* AwwaRF. 1996.

Chapter 4
Calibration of Distribution System Models

Water distribution system models can be used in a wide variety of applications to support design, planning, and analysis tasks. Since these tasks may result in engineering decisions involving significant investments, it is important that the model used be an acceptable representation of the "real world" and that the modeler have confidence in the model predictions. In order to determine whether a model represents the real world, it is customary to measure various system values (e.g., pressure, flow, storage tank water levels, and chlorine residuals) during field studies and then compare the field results to model predictions. If the model adequately predicts the field measurements under a range of conditions for an extended period of time, the model is considered to be calibrated. If there are significant discrepancies between the measured and modeled data, further calibration is needed. There are no general standards for defining what is adequate or what is a significant discrepancy. However, it is recognized that the level of calibration required will depend on the use of the model. A greater degree of calibration is required for models that are used for detailed analysis, such as design and water quality predictions, than for models used for more general planning purposes (e.g., master planning).

All models are approximations of the actual systems that are being represented. In a network model, both the mathematical equations used in the model and the specific model parameters are only numerical approximations. For example, the Hazen-Williams equation used to describe friction headloss is an empirical relationship that was derived based on laboratory experiments (Williams and Hazen, 1920). Furthermore, the roughness parameter (C-factor) used in the Hazen-Williams equation that modelers assign to each pipe is not known with total certainty because it is not feasible to examine and test every pipe in the system. The goal in calibration is to reduce uncertainty in model parameters to a level such that the accuracy of the model is commensurate with the type of decisions that will be made based on model predictions.

The types of model calibration associated with water distribution system analysis can be categorized in several ways. The nomenclature depends upon the adjusted parameters and the technique employed. In general, calibration can be categorized (or referenced) as follows:

- Hydraulic and water quality model calibration.

The concept of calibration can be compared to fine tuning an old fashioned television (TV) set. One knob on the TV is used for tuning the channel while other knobs are adjusted to improve color, sharpness, contrast, and hue. However, in calibrating a network model, there are far more knobs to adjust as illustrated in Figure 4-1.

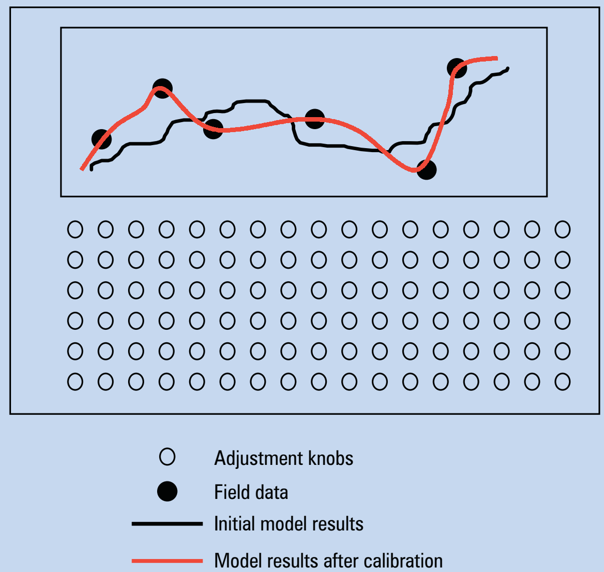


Figure 4-1. Conceptual Representation of Calibration.

Some of the knobs may be used to adjust roughness coefficients for pipes, other knobs to adjust demands assigned to nodes, while still other knobs may control valve positions, pump curves, or other parameters that are not known with complete certainty. Calibrating a model is an arduous task because there are many knobs that can be adjusted. Finding the combination of parameters that results in the best agreement between measured and modeled results is difficult. This process is complicated by the fact that there may not be a single best set of parameters. Extending the TV analogy, the knobs may be adjusted in order to get the best reception for one channel. However, when the channel is changed, the knobs may need to be adjusted to improve the reception for the new channel. Similarly, with a network model, a set of parameters may give the best match for one set of data while other parameters may give better results for another set of data. Therefore, it is recommended that a modeler first calibrate the model using one or more sets of field data and then validate it with an independent set of field data.

- Static (steady state) or dynamic (extended period simulation) calibration.
- Manual or automated calibration.

Hydraulic calibration refers to the process of adjusting the parameters that control the hydraulic behavior of the model. Similarly, water quality calibration relates to the process of adjusting parameters used in the water quality portion of the model. Static or steady-state calibration relates to calibration of a model that does not vary over time, or using data that is collected representing a snapshot in time. Dynamic or EPS calibration uses time-varying data in the calibration process. Manual calibration relies upon the user to investigate the effects of a range of possible parameter values. Automated calibration employs optimization techniques to find the set of parameters that results in the "best" match between measured and modeled results.

It should be noted that the specific application method and availability of some of these techniques will vary depending upon the software used for modeling and the available network model information. Therefore, only the general techniques employed in each of these types of calibration are discussed in the following sections. Then, some example case studies are presented to illustrate their use. The final section in this chapter discusses future trends in calibration and the possibility of general calibration standards.

4.1 Hydraulic and Water Quality Model Calibration

Hydraulic calibration is essential for any model simulation to be meaningful. Furthermore, the distribution system water quality models work in concert with the hydraulic model and utilize the flow and velocity information calculated by the hydraulic model. Thus, if the hydraulic model is not properly calibrated and results in inaccurate flow and velocity estimates, the water quality model will not perform correctly. In fact, water quality modeling is very sensitive to the underlying hydraulic model. Frequently, a hydraulic model that has been calibrated sufficiently for applications such as master planning may require additional calibration before it is appropriate for use in water quality modeling. The following subsections describe the parameters and techniques employed for hydraulic and water quality model calibration.

4.1.1 Hydraulic Model Calibration

Hydraulic behavior refers to flow conditions in pipes, valves and pumps, and pressure/head levels at junctions and tanks. Parameters that are typically set and adjusted include pipe roughness factors, minor losses, demands at nodes, the position of isolation valves (closed or open), control valve settings, pump curves, and demand patterns. When intially establishing and adjusting these parameters, care should be taken to keep the values for the parameters within reasonable bounds. For example, if local experience shows that the roughness factor for a 20-year old ductile iron pipe typically falls within a range from 100 to 130, a value that is not within or close to that range should not be used just to improve the agreement between the measured and modeled data. Use of unreasonable values may lead to a better match for one set of data, but will typically not provide a robust set of parameters that would apply in other situations.

Proper calibration requires that adjustments be made to the correct parameters. A common mistake occurs when adjustments are incorrectly made in one set of parameters in order to match the field results while the parameters that are actually incorrect are left untouched. This process is referred to as "compensating errors" and should obviously be avoided. Field verification of suspect parameters (e.g., open or closed valves) can reduce confusion created by compensating errors.

> An example of compensating errors is an adjustment in roughness factors in order to compensate for a closed isolation valve in the system that is represented as open, or partially open, in the model. In this case, unreasonably low values for the Hazen-Williams roughness coefficients are typically introduced in order to force a large headloss in the pipes that are actually closed. Though this may result in approximating the pressure measurements made in the field, it will introduce other errors in flow and velocity calculations. Compensating errors may also result from incorrectly adjusting demands or other parameters.

4.1.2 Water Quality Model Calibration

Subsequent to the proper calibration of a hydraulic model, additional calibration of parameters in a water quality model may be required. The following parameters are used by water quality models that may require some degree of calibration:

- Initial Conditions: Defines the water quality parameter (concentration) at all locations in the distribution system at the start of the simulation.
- Reaction Coefficients: Describes how water quality may vary over time due to chemical, biological or physical reactions occurring in the distribution system.
- Source Quality: Defines the water quality characteristics of the water source over the time period being simulated.

The details of calibration depend upon the type and application of the water quality model. Calibration requirements for each type of modeling are described below and summarized in Table 4-1.

Table 4-1. Calibration/Input Requirements for Water Quality Models

Model Application	Initial Conditions	Reaction Coefficients	Source Quality
Water age	YES	NO	Usually NO
Source tracing	YES	NO	Usually NO
Conservative constituent	YES	NO	YES
Reactive constituent	YES	YES	YES

- Water age: No explicit water quality calibration can be performed because there are no reaction coefficients. Estimates of initial water age in tanks and reservoirs are desirable in order to shorten the length of the simulation. Source water age is usually set to zero for all sources. Water age can be especially sensitive to inflow-outflow rates for tanks, mixing characteristics of tanks, and travel times in dead-end pipes.

When modeling a tank, an important parameter is the initial age of the water in the tank at the start of the simulation. This value cannot be measured in the field but can be estimated by dividing the tank volume by the volume of water that is exchanged each day. Frequently, modelers will just assume that the initial age is zero and run the model for a long period until it has reached a dynamic equilibrium. This occurs when the initial water in the tank has been flushed out entirely through the fill and draw process. The following figure (Figure 4-2) shows the effects of the initial water age on the modeled results. As illustrated, a good initial estimate for water age (120 hours in this case) results in a much shorter time period until the dynamic equilibrium is reached. In fact, in this case when the initial age was input as zero hours, the model did not even come close to reaching dynamic equilibrium during the simulation period and would have required a much longer run duration to reach the same point.

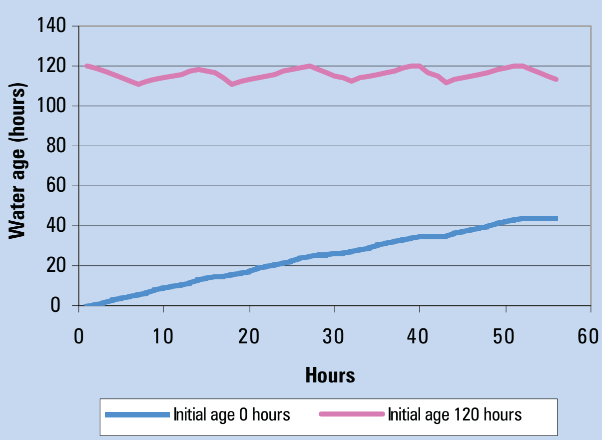


Figure 4-2. Effects of the Initial Water Age on the Modeled Results.

- Source tracing: No explicit water quality calibration can be performed because there are no reaction coefficients. Estimates of initial conditions in tanks for percentage of water coming from a source are desirable in order to shorten the length of the simulation. Values for sources are usually set to zero for all sources except for the specific source being traced.

- Conservative constituents: No explicit water quality calibration can be performed because there are no reaction coefficients. Estimates of initial conditions in tanks for concentrations of the conservative constituents can usually be determined from field data and are desirable in order to shorten the length of the simulation. Values for sources are set to the typical concentrations found in the source.

- Reactive constituents: For reactive constituents, both the form of the reaction equation and the reaction coefficients must be provided. When modeling chlorine or chloramine decay, the most common formulation is a first order decay equation including both bulk and wall decay coefficients. Values for these coefficients typically require laboratory and field analysis and calibration in order to match model results to the concentrations measured in the field. Correspondingly, THMs, a group of DBPs formed when water is chlorinated or chloraminated, generally increase in concentration with time (Vasconcelos et al., 1996). This process is frequently represented as a first order growth function that asymptotically approaches a limiting value representative of maximum concentration reached when all of the NOM has reacted or all of the chlorine has been consumed. Both the limiting value and the rate of growth must be determined in this case.

Water quality modeling is very sensitive to the hydraulic representation of the system. To reiterate, hydraulic calibration that may be sufficient for some hydraulic simulation may require additional calibration when used as a basis for water quality modeling.

4.2 Static Calibration and Dynamic Calibration

Just as water distribution system models can be run in a steady-state or an extended period mode, calibration can be performed in either a static mode using a steady-state model or in a dynamic mode using an extended period model. A common approach is to perform a static calibration first followed by EPS, to enhance the static calibration through a dynamic calibration. The options and procedures for these two types of calibration are described below.

4.2.1 Steady-State Calibration Methods

The two most common approaches used in calibrating a steady-state hydraulic model are C-factor tests and fire-flow tests. For water quality models of chlorine/chloramines, a test procedure for estimating bulk and wall demand may be employed. In all of these cases, field data is collected under controlled conditions and then applied to determine the model parameters that result in the best fit of the model to the field data.

4.2.1.1 C-Factor Tests

C-factor tests (sometimes called head loss tests) are performed to estimate the appropriate C-factors to be used in a hydraulic model. The C-factor represents the roughness of the pipe in the widely used Hazen-Williams friction equation. Typically, such tests are performed on a set of pipes that are representative of the range of pipe materials, pipe age, and pipe diameters found in the water system that is being studied. The results of the tests are then used to assign C-factors for other pipes of similar characteristics.

In a field test, a homogeneous section of pipe between 400 and 1,200 feet long is initially isolated. Subsequently, flow, pipe length, and head loss are measured in the field. Typically, nominal pipe diameters are

The underlying concept for a C-factor test is that all factors in the Hazen-Williams friction equation can be measured in the field and the equation can then be solved for the unknown C-factor. It can also be used to account for minor losses that occur through distribution system components (e.g., valves, fittings). The following equation is the Hazen-Williams equation (Equation 2-3) arranged to solve for roughness.

$$C = 8.71\ V\ D^{-0.63}\ (H/L)^{-0.54} \qquad \text{(Equation 4-1)}$$

where
C = roughness factor
V = velocity in feet per second
D = pipe diameter in inches
H = head loss in feet
L = pipe length in feet

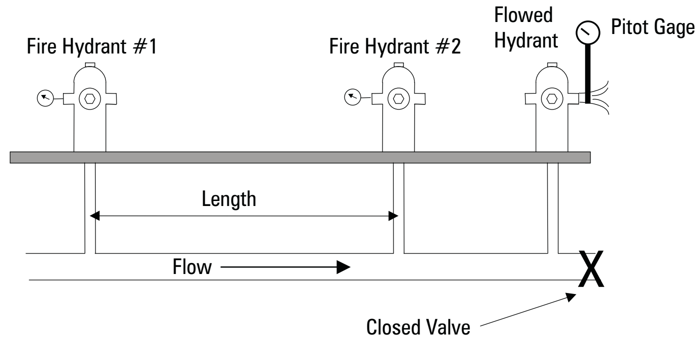


Figure 4-3. Schematic of Standard Two-Gage C-Factor Test Setup.

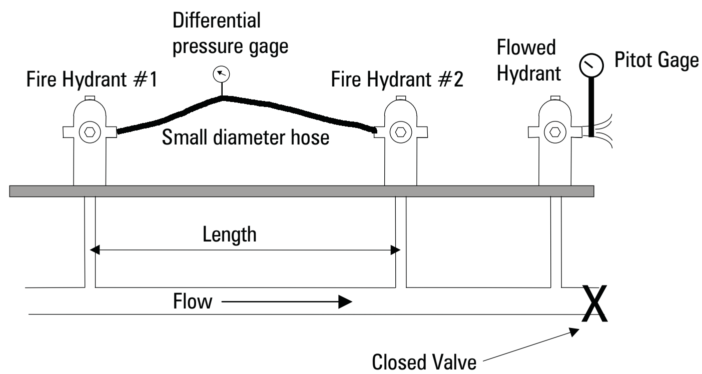


Figure 4-4. Schematic of Parallel Hose C-Factor Test Setup.

taken from system maps and these values are used along with flow rate to calculate velocity. There are two alternative methods for determining head loss in the pipe section: a two-gage method (Figure 4-3) and a parallel hose method (Figure 4-4). With the two-gage method, pressure is read at hydrants located at the upstream and downstream end of the section and used along with elevation difference between the ends to calculate head loss. With the parallel hose method, a small-diameter hose is used to connect the two hydrants to a differential pressure gage to directly measure the difference in pressure. The two end hydrants should be spaced far enough apart and there should be sufficient flow so that there is a pressure drop of at least 15 pounds per square inch (psi) for a two-gage test or a 3-psi pressure drop for a parallel hose test (McEnroe et al., 1989). In both cases, a hydrant downstream of the test section is opened to induce flow and a sufficient pressure drop. Multiple downstream hydrants may also be employed to induce a greater flow and larger pressure drop. Typically, a pitot gage (as shown in Figures 4-3 and 4-4) is attached to the flowing hydrants to measure the flow rate. It is important to ensure that all flow between hydrants is accounted for (i.e., any connections that may bleed water into or out of the test section). The two-gage method is the more commonly used approach. The parallel hose method requires more specialized equipment, but is inherently more accurate and may be used when a large pressure drop cannot be achieved. Note that the valve is closed downstream of the flowing hydrant.

As noted above, an assumption is made that the pipe diameter has not diminished from its original nominal diameter due to tuberculation on the pipe walls. If that assumption is not valid, the calculated C-factors will be lower than expected. If very low C-factors are calculated based on a field C-factor test, it is recommended that further actions be taken in order to determine the effective diameter of representative pipes. These actions could include direct inspection of sample pipes or use of calipers inserted into the pipe to measure the effective pipe diameter.

4.2.1.2 Fire-Flow Tests

Fire-flow tests are routinely performed by water utilities to determine the ability of the system to deliver large flows needed to fight fires. In such a test, fire hydrants are opened, the flow through the hydrants measured and pressures measured at adjacent hydrants (see Figure 4-5). The high demands caused by the open hydrants lead to high flows and increased head loss in pipes in the area around the hydrants. Under these conditions, the system is stressed and the capacity of the system to deliver these flows is very sensitive to the roughness of the pipes.

These fire-flow tests can also be very effective as a calibration methodology. In this case, in addition to the standard information routinely collected as part of a fire-flow test (flows and pressures), information is collected on the general state of the system such as pump and valve operation, tank water levels, and general system demand. The distribution system model is then run under the system conditions observed during the test and adjustments made in roughness factors (or other parameters) so that the model adequately represents the data measured in the field.

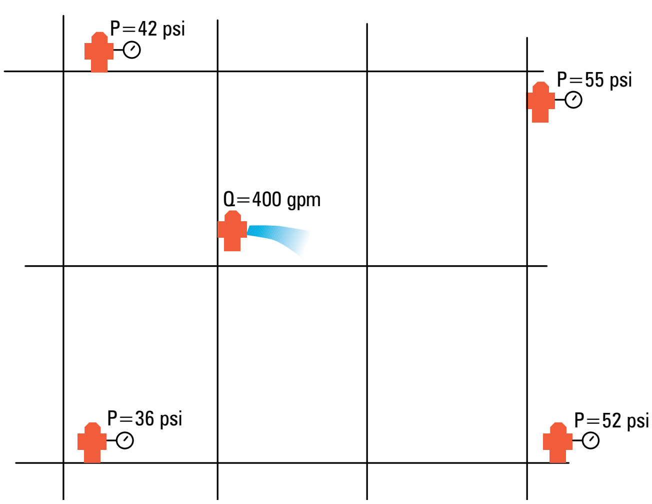


Figure 4-5. Fire-Flow Test Setup.

Figure 4-6. A Hydrant Being Flowed with a Diffuser as Part of a Fire-Flow Test.

Figure 4-6 illustrates an example setup for a fire-flow test. The diffuser attached to the hydrant in the figure includes a pitot gage used to measure the flow. The cage diffuses the flow and prevents any objects in the stream from being projected out at high speed.
In the case shown in Figure 4-5, only a single hydrant is opened, with the flow measured at that hydrant and pressure measurements made at four hydrants. Additional hydrants may be flowed and monitored as part of a fire-flow test for calibration (see Case Study in Section 7.7).

4.2.1.3 Chlorine Decay Tests

Chlorine bulk reaction and wall reaction (or demand) testing procedures can be used to determine the reaction parameters used in water quality models. Bottle tests measure the rate of chlorine reaction that occurs in the bulk flow independent of wall effects. This procedure is performed by first measuring the chlorine at a representative location such as in the effluent from a water treatment plant. Then several bottles are filled with the same water and kept at a constant temperature. Separate bottles are subsequently opened at intervals of several hours (or days) and the chlorine content is measured. The resulting record of chlorine at different times is used to estimate the bulk reaction rate. See AWWA (2004) for a more complete protocol for this test.

The purpose of the chlorine decay field testing procedure is to estimate the chlorine wall demand coefficient for representative pipes in the distribution system. The method described here involves the measurement of chlorine concentrations in a pipe segment under controlled flow conditions and use of the resulting chlorine measurements to determine the wall reaction rate for that pipe segment. The method is designed to be complementary with C-factor testing so that it can be conducted in conjunction with a C-factor test. The method is considered to be experimental and feasible only for pipes that are expected to have relatively high wall reaction values, such as smaller diameter unlined cast iron pipes. For the smaller diameter unlined cast iron pipes, pipe sections with a length in the range of 1,500 to 2,000 feet will be required to estimate wall demand. For other types of pipes that typically have low wall decay factors (e.g., plastic and new pipes), the required length of the pipe may be so long as to make this test impractical. Other factors that should be considered in selecting sites include the following:

- Ability to measure flow in the pipe.
- Ability to valve off the pipe segments.
- Presence of a reasonable chlorine residual (preferably > 0.4 mg/L) at the upstream end of the pipe segment.
- Ability to vary flow in the pipe over a reasonable flow range (e.g., for a 6" pipe, a range of flows of 100 to 500 gpm would be desirable).
- Ability to estimate the actual pipe diameter for the pipe segment.

For the selected pipe segment, major lateral(s) and downstream segments should be valved off to control flow in the pipe. Two or three sampling points should be established along the segment of interest (upstream, downstream, and an optional midpoint). Typically, these would be taps on fire hydrants. Prior to the testing, the taps should be run for several minutes to clean out the line. The approximate travel time through the pipe should be calculated and chlorine measurements taken from upstream to downstream so that approximately the same parcel of water is sampled at each station. Flow measurements can be made at any location within the segment.

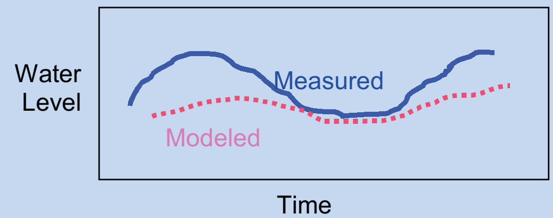
The test should be repeated for three flow values: a low flow rate, a medium flow rate, and a high flow rate. During each flow test, chlorine residual should be measured at each of the two or three sampling points. Since relatively small variations in chlorine concentration are expected, a good quality field chlorine meter should be employed and three replicates should be taken at each sampling point for each flow test. Following the field analysis, a spreadsheet can be used to back calculate the resulting wall reaction coefficients, or a water distribution model can be used to determine the wall reaction coefficient through trial and error.

4.2.2 Dynamic Calibration Methods

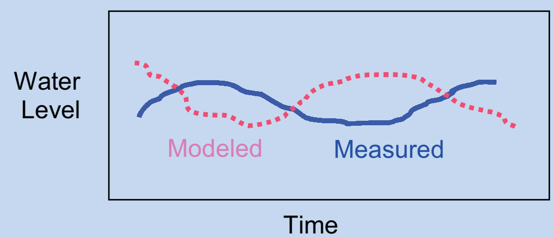
Dynamic calibration methods are associated with the use of an EPS model. The dynamic calibration methods include: (1) comparison of modeled results to measurements made in the field over time, and (2) tracer studies. In both cases, model parameters are adjusted so that the model adequately reproduces the observed behavior in the field. Tracer studies are discussed in detail in Chapter 3.

Comparison of modeled and measured data can be used for calibration of both hydraulic and water quality models. The most commonly measured hydraulic data are tank water levels, flows, and pressures. Frequently, this information is routinely reported through SCADA systems to a database and can be extracted. In other cases, continuous flow meters or pressure gages must be installed to collect data during a test period. Generally, tank water level data and flow measurements are the most useful form of data for calibrating an extended period model. Under average water use conditions, temporal variations in pressure measurements typically vary over a relatively small range and then only in response to variations in tank water levels. As a result, they are less useful in calibrating model parameters. If pressure measurements are going to be used for

If measured and modeled records of tank water levels do not agree well, the relationship between the two traces can provide clues as to the potential problems. In the example depicted below, the timing of the fill and draw cycles in the measured and modeled results are quite close but the modeled and measured depth of the fill cycles vary significantly. This suggests that the system demands may be in error, resulting in an incorrect amount of flow entering the tank.



In the second example illustrated below, the magnitude of the change in water level is quite close in the modeled and measured results, but the timing of the fill and draw cycles differ. This is typically caused by errors in the pumping controls in the model, resulting in pumps being turned on and off at the wrong time.



dynamic calibration, the system must be stressed by conducting fire-flow tests during the testing period. The primary model parameters that are adjusted during dynamic calibration are: demand patterns, pump schedules and pump curves, control valve settings, and the position (open or closed) of isolation valves.

Dynamic calibration procedures using tracer study data is discussed via a case study in Section 4.4 of this chapter. Dynamic calibration can also be used for calibrating water quality parameters, such as the wall demand coefficient for computing chlorine residuals. Generally, water quality field studies are performed in conjunction with field hydraulic studies or with a tracer study. For chlorine models, measurements of chlorine are taken at frequent intervals in the field at representative sites. These may include dedicated sampling taps, hydrants, tank inlet/outlets, or other accessible sites. Continuous chlorine meters may also be used. During the model calibration process, the model is first calibrated for hydraulic parameters, and water quality coefficients are subsequently adjusted so that the model results match the field data.

4.3 Manual Calibration and Automated Calibration

The aforementioned process of adjusting model parameters so that the model reproduces the hydraulic and/or water quality results measured in the field can involve a significant amount of effort in large or complex systems. As discussed earlier in this chapter, there are many parameters that can be adjusted in the model and the combinations of possible parameter values can sometimes appear to be quite overwhelming. Typically, a manual trial and error approach is used. The most influential parameters can be identified based on sensitivity analysis and then adjusted to see if they improve the results. This process is continued until an acceptable level of calibration is achieved or until budgetary constraints dictate closure. It is not unusual for many (dozens or even hundreds) separate model runs to be made in this process.

An extension to the manual calibration process is an automated approach that allows the computer to search through different combinations of model parameters (with a realm of realistic values) and to select the set of parameters that results in the best match between measured and modeled results. The development of this type of program has been the topic of many studies over the past 25 years (Walski et al., 2003).

Automated methods require a formal definition of an objective function for measuring how good a particular solution is. Generally, the value of a solution is measured by a statistic that reflects the deviation between measured and modeled results in flow and pressure. A commonly used objective function is minimization of the square root of the weighted summation of the squares of the differences between observed and predicted values. The weighting is used to establish a relationship between the errors associated with flow and pressure. For example, the user may choose a 1-psi error in pressure prediction to be equivalent in value to a 10-gpm error in flow.

In most automated methods, the user also groups pipes by common characteristics, such as age, material, and nodes, into common demand characteristics such as residential or commercial. The user then specifies a range of allowable values for pipe roughness factors or a range of multipliers applied to the existing roughness factors. Similarly, a range of allowable demand multipliers is also specified, as are potential pipes where an existing isolation valve may be closed. The optimization routine is then applied and the roughness, demands, and isolation valve positions are selected that result in the minimum error.

Though manual calibration still remains the predominant methodology, automated calibration methods are becoming more available in commercial modeling packages. It is likely that as the automated calibration methods are refined, the technology will expand for routine use with EPS hydraulic and water quality models.

4.4 Case Studies

In order to illustrate some of the calibration methods described earlier in this chapter, two case studies are presented in this subsection. The two case studies are similar in general methodology but differ in the overall scale and specifics of the study area. In both cases, the distribution system model that was used as a starting point for the calibration exercise was part of a skeletonized model extracted from unspecified portions of the GCWW distribution system.

Most larger urban water systems generally have at least a skeletonized model of their distribution system. It should be noted that (as discussed in Chapters 2 and 3 of this report) a skeletonized model denotes a model that includes only a major subset of actual pipes rather than all pipes in the distribution system. The extracted system model was modified and converted to EPANET format for use in this project. The modifications included: addition of key pipes, updates to consumer demand data, and an interconnection between the case study area and the full system by a fixed grade node (reservoir). These portions of the base model had been previously calibrated using various dynamic calibration methods and were used for routine water utility work. For the purposes of calibration, separate field studies were conducted in each study area.

In both field studies, a food-grade conservative tracer (calcium chloride) was introduced into the system and its movement through the system was monitored by both grab sampling and continuous monitoring (CM) stations installed at key locations in the distribution systems. The CM stations were installed at hydrants which were left partly open for the duration of the study to minimize travel time between the main and sampling location. Each open hydrant was added as a new demand node in the EPANET network model. Additionally, several ultrasonic flow meters were installed to provide continuous flow measurements at key locations. The general procedures, methodology, and instrumentation used in these field studies are consistent with those presented in Chapter 3.

4.4.1 Case 1 - Small-Suburban, Dead-End System

This system is part of a larger pressure zone. It was selected because of the relatively compact size and simple structure, fed by a single supply pipe with no additional storage. As a result, the movement of the tracer was relatively rapid through the system and it could be monitored with continuous meters placed at several locations. The general layout of this subsystem, the location of the injection site, and the monitoring locations for this study are shown in Figure 4-7.

The calcium chloride tracer was injected as two pulses, a two-hour pulse followed by a 2.5-hour period of no injection and then followed by a higher concentration pulse of two hours duration. The injection rate and the resulting concentration of the tracer in the distribution system just downstream of the injection point were carefully monitored to ensure that the resulting chloride concentration did not exceed the secondary maximum contaminant level (MCL) of 250 mg/L for chloride.

Compliance with state and federal regulations during a tracer study is obviously quite important. In order to ensure that the tracer will not exceed allowable levels, it is necessary to monitor information such as the rate of injection of the tracer, the flow in the receiving pipe, and the resulting concentration in the receiving pipe. Frequently, a safety factor for the injection rate is included to account for uncertainty. In this field study, the tracer injection rate was very low and the flow meter on the injection pump provided approximate values. Chloride concentrations were monitored at a suitable location approximately 100 feet downstream of the injection point with a travel time of approximately 10 minutes. Due to unexpected variations in flow through the pipe, delay in measurements, and related computations (associated with tracer travel time), chloride values exceeding the target level were experienced for a brief period before the injection rate was adjusted.

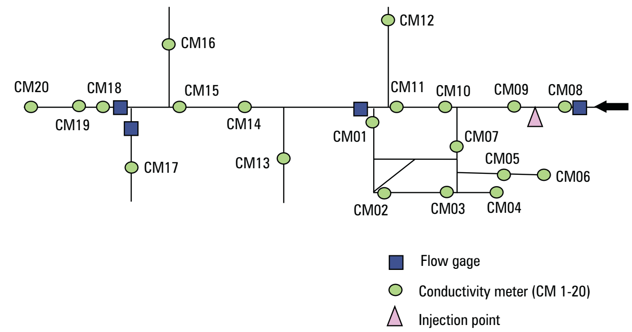


Figure 4-7. Schematic Representation of Small-Suburban Dead-End System.

The movements of the tracer pulses were monitored by using both manual sampling and continuous conductivity meters located throughout the distribution system. Additionally, four ultrasonic flow meters were installed in the study area to provide continuous flow measurements at key locations within the distribution system.

In preparation for the calibration process, the conductivity readings were converted to chloride concentrations using a relationship developed in the laboratory. Figure 4-8 shows the relationship between conductivity and chloride and the best-fit linear and polynomial relationships between them. This conversion was necessary because conductivity is not a truly linear parameter and, as a result, cannot be simulated exactly in a water distribution system model. The converted continuous concentration readings were then compared to the manually collected data for quality control purposes. Figure 4-9 shows the resulting chloride data set that was used at one location as a basis for evaluating model predictions as part of the calibration process.

The preliminary results indicated some discrepancy between the EPANET-model predicted values and the

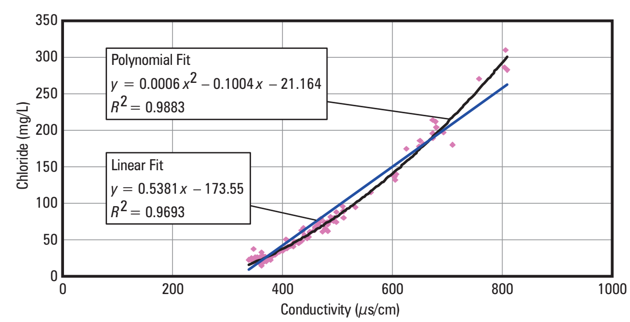


Figure 4-8. Empirical Relationship Between Chloride and Conductivity.

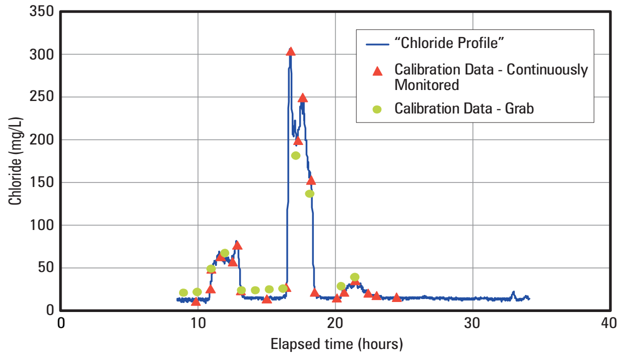


Figure 4-9. Sample Chloride Data Used at One Station for Calibration.

actual field-measured values, indicating the need for model refinement and re-calibration to improve the prediction capability of the EPANET model. Therefore, EPANET modeling was performed to evaluate the following four levels of model refinements:

- Level 1 (prior to calibration): A skeletonized EPANET model was used with the original hourly demand pattern provided by GCWW and a time-step injection pattern of 60 minutes.
- Level 2: The same as Level 1, but a refined 10-minute time-step pattern for injection was used along with the conversion of the original hourly demand patterns to 10-minute patterns.
- Level 3: The same as Level 2 with a refined demand pattern for each node using the field-measured flow data, addition of demand nodes representing water demand of the partially open hydrants, adjustment for a large industrial user of water in the study area (based on data obtained during the study), and the residential water billing information provided by GCWW.
- Level 4: The same as Level 3 with a detailed all-pipe (non-skeletonized) EPANET model.

The results of the four-stage model refinement and calibration process are shown in Figure 4-10 for a continuous monitoring location (CM-18) located on the main feeder pipe. As illustrated, the improvements in the demand estimates and inclusion of the system details in the all-pipe model resulted in a vast improvement in the model's prediction ability for that monitoring location. Similar results were found for most monitoring locations on the main pipe.

During the calibration and refinement process, various model inputs such as flow, demand, and pipe characteristics were adjusted to improve the model prediction. The EPANET model was considered to be calibrated for the area when the field data matched the model-predicted output to an acceptable degree based on visual observation. Depending upon the location of the junction (where the model predictions are compared with the field values), both concentration and predicted time of tracer arrival might not be in perfect agreement due to local variation in demands, local flow velocities, and dilution impacts. The sharp tracer fronts observed in this field study made it difficult to employ quantitative statistical measures (e.g., mean error, standard deviation, root mean square error). Therefore, a graphical (visual) approach was considered to be more suitable for model calibration

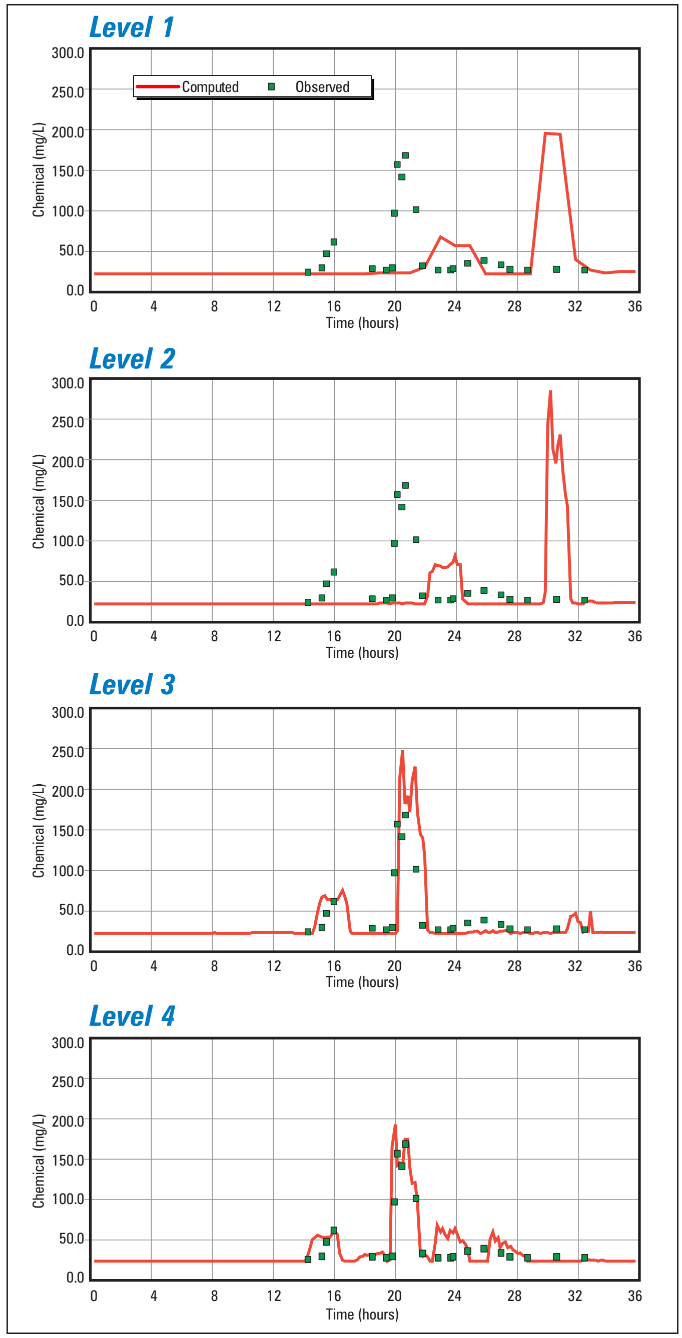


Figure 4-10. Comparison of Model Versus Field Results for Continuous Monitor Location CM-18 at Various Calibration Stages.

in this application. For example, if the prediction of the arrival time for the tracer differs by even a few minutes from the observed arrival time, use of these standard measures of error could result in a high number, even though the prediction could be viewed graphically as very good.

The calibration of the "looped" portion (referring to the portion of the network on the bottom right hand side of Figure 4-7) of this network proved to be more difficult and the results for some monitoring locations on the looped piping were less satisfactory. The most problematic were continuous monitoring locations CM-02 and CM-04. Monitoring station CM-02 was located near the confluence of two separate loops, with the actual monitored connection being slightly offset from the junction node. Examination of the model results showed that flow reached that junction from both directions and small variations in the amount of flow in each of the loops resulted in very different travel times. As illustrated in Figure 4-11, this complex travel pattern along with the offset location of the monitoring station resulted in poor

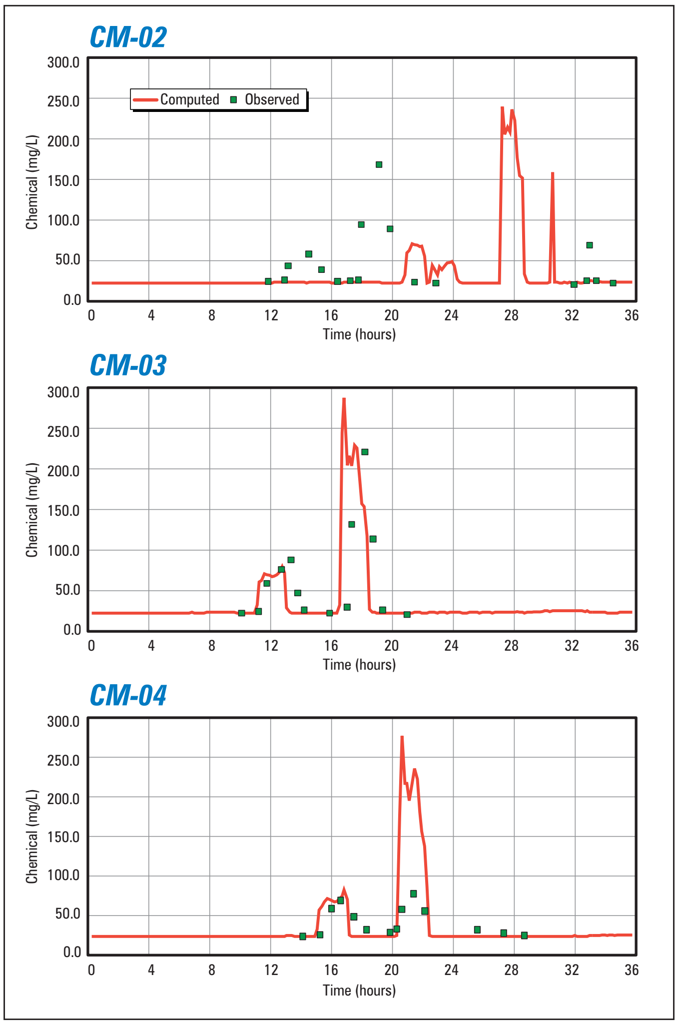


Figure 4-11. Calibration of "Looped Portion."

prediction of travel time to that station. Also, monitoring station CM-04 is located at the end of a dead-end pipe section and travel to this node is strongly influenced by demands at the very far end of the dead-end section. As illustrated in Figure 4-11, this resulted in a poor match of the peak concentration during the second pulse. It is also postulated that dispersion, which is not represented in EPANET, may have had an influence on the peak concentration due to the very low velocities in the dead end pipe. In some cases, this could also be caused by inaccurate C-factors as applied to the distribution system. However (as illustrated in Figure 4-11), for monitoring location CM-03 located in the main part of the looping system, the model and field agreement was quite good.

Case 1 data illustrates that, depending upon the level of refinement and calibration, there is a significant variation in the capability of a model to accurately represent the system. In general, the parts of the network that are configured as trees (main stem with branches) are more easily calibrated by making adjustments in demands. For looping parts of the system and at dead-ends, results are very sensitive to small variations in demands and system configuration, leading to the possibility of significant prediction errors at some locations. Uncertainty in demand estimates can be a major source of error in the model estimates.

4.4.2 Case 2 - Large-Suburban Pressure Zone

Similar to Case 1, a field study and calibration exercise was carried out in a large-suburban, pressure zone. This area was selected in order to demonstrate the application of tracer studies and calibration techniques in a more complex area. The selected area contained multiple pumps and tanks. The selected distribution system area is representative of relatively complex, well-gridded systems found in many larger water systems. The layout of the system, the location of the injection site, and the monitoring locations are shown in Figure 4-12.

Two separate tracer studies were performed in this zone. The first study was used to further calibrate the skeletonized model received from the water utility. The second study served as a validation event to test the veracity of the calibrated model. In the calibration event, the tracer was introduced directly into the main feed line servicing the entire area (characterized by higher flow/higher pressure). In the validation study, the tracer was pulsed. A total of 34 continuous conductivity meters were installed in the system. Four flow meters were temporarily installed to provide flow measurements at key locations.

During the calibration study, the calcium chloride tracer was injected into the main feed line serving the

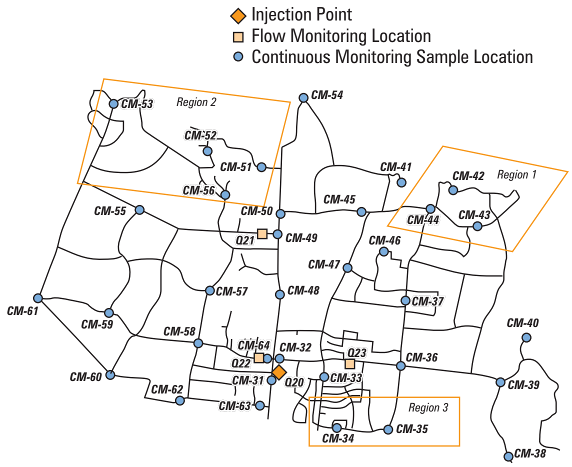


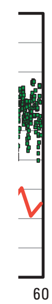
Figure 4-12. Schematic Representation of Case 2 Study Location.

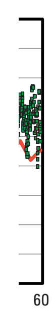
area for a period of 6 hours. In the validation study, the tracer was pulsed by fill and draw cycles in a storage tank at the same location. In both cases, a target chloride concentration of 190 mg/L or lower was set in order to safely not exceed the 250 mg/L secondary MCL for chloride.

During the calibration process, initial EPANET model simulations were reviewed in detail to determine the flow patterns around various monitoring locations and to attempt to identify causes for discrepancies in the observed and predicted values. A careful examination of the areas of significant discrepancies indicated that these were primarily limited to three geographic sub-regions within the skeletonized network. In addition to these three sub-regions, there were a few isolated locations where the predicted tracer pattern did not match the observed tracer pattern from the field study. The modeling team carefully examined each of these regions and addressed the zonal issues accordingly. The three sub-regions are shown in Figure 4-12.

In Region 1 (CM42, CM43, and CM44), the field data indicated that the tracer arrived at these continuous monitoring locations several hours before the model's prediction. On closer inspection, it was found that a potential flow path existed which was not included in the skeletonized model. While the pipe diameter was small, it significantly altered the hydraulic water flow path to that region. This missing pipe-link was added to the model, using the appropriate pipe parameters. Furthermore, the modeling team investigated the GIS database to see if there were any substantial changes in these areas since the time when the original water demand patterns were developed five years ago. The updated GIS information indicated a presence of recent housing development in that region. Therefore, additional demand nodes were entered into the EPANET model to accommodate for this development. Another possibility for the discrepancy was that the demand in this region was significantly higher than the average residential demand modeled in the area. To simulate this possibility, a sensitivity analysis was performed in which the modeled demand in this region was doubled. The model-predicted results improved significantly for this region based on these three adjustments.

In Region 2 (CM52, CM53, CM55 and CM56), an opposite phenomenon to that in Region 1 was observed. The field data indicated that the tracer arrived several hours after the model's prediction. One possible explanation was that this region had lower demand than the average residential demand modeled in this area. The flow meter data upstream of this location supported this theory as the EPANET predicted flow in this pipe was much higher than the field observed flow (see Figure 4-13a). To simulate this possibility, the local demand in this region was reduced by 30 percent in the model. The resultant flow matched the flow meter data (see Figure 4-13b). Also, similar to Region 1, it was found that a potential flow path had again been left out due to skeletonization of the model, which affected CM52. This pipe link was added to the model using the appropriate pipe parameters. Distribution mains between CM55 and CM53 were also found to have been upgraded since the EPANET network model was





developed for this area. The EPANET model pipes for this location were updated using the newer information. The model-predicted results improved significantly for this region based on these three adjustments.

In Region 3 (CM34 and CM35), the field data indicated that the tracer arrived at locations CM34 and CM35 several hours after the model's predicted arrival time. However, the field-verified tracer arrival time matched the predicted tracer arrival time at location CM33 which is slightly upstream of these locations. Also, a review of the water flow pattern in this region indicated that the water traveled from CM33 towards CM34 and CM35 (at all times). Based on the demands in the EPANET model, the pipe lengths, and the regional water flow information, the delay in tracer arrival at CM34 and CM35 could not be explained. A closer inspection of the region revealed a complex grid of interconnected pipes in this region, which were skeletonized as two parallel pipes. This skeletonization eliminated a number of different possible hydraulic flow paths between CM 33 and CM34/CM35. Also, in the EPANET model inputs, it appeared that the demand close to CM34 and CM35 was set artificially higher (to account for the overall demand in the skeletonization process). This model setup resulted in the predicted faster tracer arrival times at CM34/CM35 than those observed in the field. To account for this anomaly, a few pipe segments from the master plan were added to the skeletonized model of this region to better simulate the actual grid demands near CM34 and CM35. This model adjustment resulted in better prediction of the tracer arrival times.

During the calibration process, as demands were adjusted, a mass balance was performed for each hour to ensure that the net water demand in the study area remained the same, i.e., the increase in the demand at certain nodes was balanced by the reduced demand at other nodes to eliminate any net impact on water demand. In the final refinements, a multiplier of 2.0 was used for the base demand in Region 1, and a multiplier of 0.7 for the base demand in Region 2. These refinements showed some improvement in the model's ability to correctly predict the tracer arrival time and concentration. These calibration efforts resulted in a relatively well-calibrated network model. However, some local problems remained, especially in looped areas and areas that were branched off from the main lines.

The substantial changes made to the EPANET skeletonized model representing the large area necessitated a validation process. Therefore, the calibrated EPANET model input file from the first event was used to validate the model's capability to predict the results during the subsequent tracer addition. For the purposes of this validation, the data

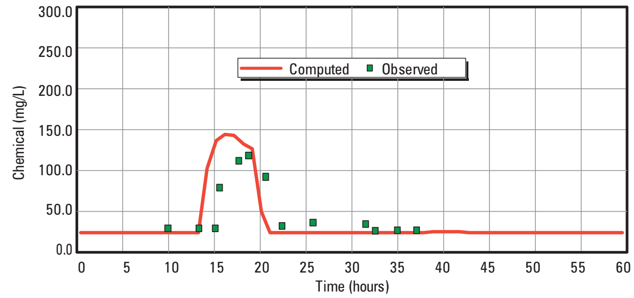


Figure 4-14a. Chloride Concentration for Calibration Event at Continuous Monitor Location CM-59.

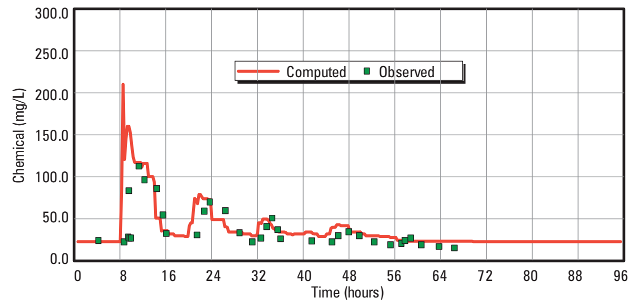


Figure 4-14b. Chloride Concentration for Validation Event at Continuous Monitor Location CM-59.

from the second set of pulsed injections was modeled using the calibrated EPANET network model for the study area to see how the predicted results compared with the continuous monitoring data collected during this event. The modeled and measured concentrations are compared in Figure 4-14a for the EPANET calibration. A similar comparison is shown in Figure 4-14b for the validation study.

Additionally for the purposes of this analysis, the EPANET predictions from the validation event were compared with the field results for each monitoring site and each site was given a grade as follows:

- Very good match (within ±20 percent of the actual concentration and within ±1 hour of the actual tracer arrival time)
- Moderate match (within ±30 percent of the actual concentration and within ±5 hours of the actual tracer arrival time)
- Poor match (greater than ±30 percent of the actual concentration or greater than ±5 hours of the actual tracer arrival time).

Of the 34 monitoring sites in this study area for the validation event, 15 received a grade of very good match, 14 were in the moderate match category, and 5 received the lowest grade of poor match. In general, it was found that better matches occurred on larger pipes serving large populations, while the poorest matches occurred in more localized loops serving fewer customers. These results are, in general, quite similar to the results obtained for the calibration event, and most problems repeatedly occurred at the same locations for both events. The validation event results confirm the fact that the calibrated EPANET network model can now be used to predict the outcome of a separate event to the same degree of accuracy.

4.5 Future of Model Calibration

Calibration continues to be a major focus of most modeling efforts. It can provide a model that may be used with greater confidence and produce results that are commensurate with the important decisions that are made based on the application of the model. However, there is significant room for improvements in calibration methodologies and in developing a standardized set of calibration protocols. This has led to an active research program in this area that is expected to continue into the future.

4.5.1 Calibration Standards

The following issues are raised frequently in the field of distribution system modeling:

- extent of calibration needed for various applications, and
- standards for calibration.

Though these are very reasonable questions, straight forward answers are usually not readily available.

There is general agreement in the modeling profession that the amount and degree of calibration required for a model should depend upon the intended use of the model (Engineering Computer Applications Committee [ECAC], 1999). Some applications such as design and water quality analysis typically require a high degree of calibration, while other uses, such as master planning, can be performed with a model that has not been calibrated to such a high standard. However, there are no universally accepted standards.

In the United Kingdom, there are performance criteria for modeling distribution systems (Water Authorities Association and WRc, 1989). These are expressed in terms of the ability to reproduce field-measured flows and pressures within the model, as shown below.

Flow

1. ±5 percent of measured flow when flows are more than ±10 percent of total demand (transmission lines).
2. ±10 percent of measured flow when flows are less than ±10 percent of total demand (distribution lines).

Pressure

1. 0.5 m (1.6 ft) or 5 percent of head loss for 85 percent of test measurements.
2. 0.75 m (2.31 ft) or 7.5 percent of head loss for 95 percent of test measurements.
3. 2 m (6.2 ft) or 15 percent of head loss for 100 percent of test measurements.

In 1999, the AWWA Engineering Computer Applications Committee developed and published a set of draft criteria for modeling. These were not intended as true calibration standards, but rather as a starting point for discussion on modeling needs. These criteria are summarized in the following table (Table 4-2).

Table 4-2. Draft Calibration Criteria for Modeling (based on ECAC, 1999)

Intended Use	Level of Detail	Type of Simulation	Number of Pressure Readings[1]	Accuracy of Pressure Readings	Number of Flow Readings	Accuracy of Flow Readings
Long-Range Planning	Low	Steady-State or EPS	10% of Nodes	±5 psi for 100% Readings	1% of Pipes	± 10%
Design	Moderate to High	Steady-State or EPS	5% - 2% of Nodes	±2 psi for 90% Readings	3% of Pipes	± 5%
Operations	Low to High	Steady-State or EPS	10% - 2% of Nodes	±2 psi for 90% Readings	2% of Pipes	± 5%
Water Quality	High	EPS	2% of Nodes	±3 psi for 70% Readings	5% of Pipes	± 2%

[1] The number of pressure readings is related to the level of detail as illustrated in the table below.

Level of Detail	Number of Pressure Readings
Low	10% of Nodes
Moderate	5% of Nodes
High	2% of Nodes

At this point, there is no clear movement toward establishing calibration standards. However, it is likely that the need for further guidance in this area will increase as the extent and sophistication of modeling continues to expand.

4.5.2 Technological Advances

Research is continuing in two areas that strongly influence the likelihood of improved calibration of water distribution systems models: monitoring technology and optimization techniques. The available optimization techniques (and those under development) have been briefly discussed in this chapter and in Chapter 2. Active research and development areas include optimization techniques for water quality calibration, EPS models, and use of tracer data. Areas of research, development, and experimental applications in monitoring technology include less expensive meters that can be inserted into pipes in the distribution system and automated monitoring for use in conjunction with tracer studies (as discussed in Chapter 3).

4.6 Summary and Conclusions

Water distribution system models can be used for a number of purposes. Many of these uses result in engineering decisions that involve significant investments. It is therefore important that the model represent the "real world." Calibration techniques can be used to ensure that the mathematical representation of the system, or model, adequately simulates the system.

Calibrating a model is a difficult task because there are many parameters that can be adjusted and finding the combination of parameters that result in the best agreement between measured and modeled results is often challenging. It is recommended that the model be calibrated using one set or more of field data and subsequently validated with an independent set of field data.

Calibration of water distribution system models can be viewed in many dimensions. Hydraulic calibration is used to adjust the parameters associated with hydraulic simulations, while water quality calibration is applied to reaction rates and other parameters that control the water quality simulation. Static or steady-state calibration methods are used with steady-state models and data collected at instantaneous snapshots in time, while dynamic calibration is conducted with extended-period simulation models and time-series data. Manual calibration techniques involve manual application of models in a trial-and-error mode, while automated calibration uses the power of the computer to search a wide range of solutions and to select the set of parameters that best achieve a stated objective. Automated methods can reduce much of the tedium associated with calibration but require the modeler to formally define a quantitative objective function for measuring how well the model matches the field data. Such automated methods are becoming more available in commercial modeling packages.

> During the calibration process, it is important to eliminate various sources of errors in modeling. As a first pass, a modeler should check for typographical errors, accuracy of affected piping layout and material, general system flow, velocity values, and distribution system demands. Thereafter, one should look into other sources of errors such as skeletonization, valve position, geometric node placement anomalies, SCADA data errors, and pump performance.

Two case studies are presented in this chapter. The case studies differ in terms of the overall scale of the study area. In both cases, the distribution system model that was used as a starting point for the calibration exercise was part of a skeletonized model. The results demonstrate the need for adequate model calibration.

The extent of calibration and calibration techniques are a major issue in most modeling efforts. There is significant potential for improvements in calibration methodologies and in standardization of calibration. This has led to an active and continuing research program in this important area.

References

AWWA. *Computer Modeling of Water Distribution Systems (M32).* AWWA, Denver, CO. 2004.

ECAC. "Calibration Guidelines for Water Distribution System Modeling." Proceedings, AWWA ImTech Conference, New Orleans, LA. April 18-21, 1999.

McEnroe, B.M., D.V. Chase, and W.W. Sharp. "Field Testing Water Mains to Determine Carrying Capacity." Miscellaneous Paper EL-89, U.S. Army Engineer Waterways Experiment Station. Vicksburg, MS. 1989.

Vasconcelos, J.J., L.A. Rossman, W.M. Grayman, P.F. Boulos, and R.M. Clark. *Characterization and Modeling of Chlorine Decay in Distribution Systems.* AWWA and AwwaRF, Denver, CO. 1996.

Walski, T. M, D.V. Chase, D.A. Savic, W.M. Grayman, S. Beckwith, and E. Koelle. *Advanced Water Distribution Modeling and Management.* Haestad Press, Waterbury, CT. pp 268-278. 2003.

Water Authorities Association and WRc. *Network Analysis – A Code of Practice.* WRc, Swindon, UK. 1989.

Williams, G.S., and A. Hazen. *Hydraulic Tables.* John Wiley & Sons, NY. 1920.

Chapter 5
Monitoring Distribution System Water Quality

Monitoring a water supply system and its various components facilitates the gathering of data about the state of the system (physical, operational and water quality). If the state of the system has minimal changes in time or space, a simple monitoring system may be sufficient to define and manage the system characteristics. However, if there is potential for significant variation in the state of the system, the monitoring system must be adequately designed to capture that variability. Thereafter, depending upon the type and magnitude of variability, an appropriate response can be provided to restore the "normal" system state. This chapter will focus on monitoring water quality-related parameters in a distribution system.

In a distribution system, water quality may vary due to factors such as normal patterns in water consumption, seasonal variations, source water quality, components of the distribution system, operation of the system, retention time in storage, travel time in the piping system, or the condition of the system itself. Variability may also result from unusual occurrences, such as intentional/accidental intrusions of contaminants, or chemical processes such as nitrification. Design of a water quality monitoring program must take into account both the nature of the variability and the manner in which monitoring data will be used. In other words, the objective of the monitoring program must be defined along with appropriate output or reporting requirements.

In general, monitoring systems can be defined based on the uses or needs of the monitoring program, the general type of monitoring to be performed (manual grab sampling and/or continuous automated online monitoring), or the specific monitoring equipment characteristics. It is important to first establish a clear objective(s) for monitoring. Thereafter, depending upon the availability of funding, need, and expertise, one should select the appropriate sampling technique(s) and monitoring equipment. Once an appropriate monitoring system has been selected and implemented, it is important to operate and maintain the program to achieve optimal results and benefits. However, the system should be flexible enough so that it can be modified in case it does not meet the original objective(s).

This chapter discusses the various drivers or objectives for monitoring followed by a summary of available monitoring techniques. An overview of monitoring equipment is presented followed by guidelines for establishing monitoring requirements (e.g., selection of parameters, number and locations of monitors, and monitor characteristics). Some guidance for engineering and evaluating remote monitoring systems is also presented, along with some EPA-sponsored monitoring case studies. The chapter concludes with a summary and a listing of references.

The recent studies involving the use of online continuous monitoring systems have resulted in large streams of data that document the minute-by-minute changes in water quality that exist at various points in the water networks. The application of this technology has the potential for providing new insights as to how water distribution systems may be operated and designed to improve water quality. However, these systems will require a relatively high level of sophistication in terms of data management, including the capability to generate real-time reports, graphical and visual representation of information, and compliance reports for meeting drinking water standards. Some of these data streams may well reveal excursions in water quality that constitute violations of current or future drinking water standards, or a security-related incident. This type of information may put pressure on drinking water utilities and regulatory agencies to take remedial action, possibly on an emergency basis, even when such actions may not be fully justified (or warranted). However, careful planning and negotiations with appropriate regulatory authorities to define these potential "excursions" and the proper corrective action to be taken would prevent any misunderstandings and minimize or eliminate the potential for unjustified enforcement or response actions.

5.1 Establishing Monitoring Objective(s)

In order to define and implement an effective monitoring plan, clear objectives must be established. Collecting data just for the sake of accumulating information is not cost effective. In drinking water systems, there are several specific reasons to collect data and, typically, the monitoring system is tailored to meet one or more of these needs. The objectives of monitoring distribution systems can be broadly classified into the following five uses:

- regulatory driven monitoring,
- security related monitoring,
- process control related monitoring,

Table 5-1. Federal Distribution System Water Quality Monitoring Requirements

Regulation	Monitoring Requirement(s)
TCR	Samples must be collected at sites that are representative of the water quality throughout the distribution system based on a site plan that is subject to review by the primacy regulatory agency. The minimum number of samples that must be collected per month depends on the population served by the system. For each positive total coliform sample, there are repeat sampling requirements, additional analyses, and an increased number of routine samples.
SWTR and IESWTR	Disinfectant residuals must be measured at TCR monitoring sites.
LCR	All systems serving a population > 50,000 people must do water quality parameter (WQP) monitoring. The number of sample sites for Pb/Cu and WQP monitoring is based on system size.
DBPR2	The IDSE requirement of DBPR2 in turn requires the establishment of a Standard Monitoring Program (SMP). The SMP will require one year of data on THMs and Haloacetic Acids (HAAs). The number of sampling locations is based on utility size and source characteristics. Modeling can reduce sampling requirements.

- water quality characterization (e.g., general, baseline, or other research-related monitoring), and
- multi-purpose (a combination of above) use of monitoring data.

The following subsections present the overall scope of each of these five objectives.

5.1.1 Regulatory Driven Monitoring

Various federal, state, or other governmental agencies have regulations that specify distribution system monitoring requirements. An overall review of federal regulations impacting distribution systems was presented in Chapter 1. The specific federal distribution system monitoring requirements (existing and proposed) are summarized in Table 5-1. In some cases, states have imposed more stringent criteria and monitoring requirements.

5.1.2 Security Related Monitoring

Assessments performed by utilities and various research studies have identified that water distribution systems are vulnerable to intentional (or accidental) contamination. In addition to "hardening" systems in order to deter intentional contamination, monitoring as part of an early warning system (EWS) has emerged as a logical approach to cope with potential contamination events. There are no existing or proposed standards for such monitoring. However, it is well recognized that monitors will need to be sufficiently sensitive to a broad range of potential contaminants and appropriately located to detect a contamination event within a reasonable time. Additionally, as detailed in EPA's Response Protocol Toolbox (EPA, 2003-2004), monitors must be an integral part of an emergency response management plan in order to be effective. Extensive research and development is underway on monitor development, calibration, and placement in response to the perceived security monitoring needs.

Currently, EPA has an ongoing test program to evaluate the potential of sensors monitoring routine online water quality parameters, such as pH, oxidation reduction potential (ORP), free chlorine, total organic carbon (TOC), conductivity, and turbidity, to serve as rapid detection devices for detecting contamination events in distribution systems. Online monitors were selected because response time is critical for achieving the objective of providing early warning. Both bench- and pilot-scale studies are being conducted at the Water Awareness Technology Evaluation Research and Security (WATERS) Center within the EPA's Test and Evaluation (T&E) Facility in Cincinnati, Ohio. The bench-scale runs are designed to identify the detection threshold of each sensor for specific contaminants. The pilot-scale runs are designed to evaluate overall response of the selected sensors by injecting known quantities of potential contaminants into the distribution system simulator (DSS). For this purpose, several pilot-scale DSSs have been fabricated and used for these test runs. The sensor data are collected continuously and archived electronically to establish stable baseline conditions and to also record sensor responses to injected contaminants. Grab samples are collected periodically before and after injection of contaminants to confirm the sensor results.

5.1.3 Process Control-Related Monitoring

Monitors can also be used in a distribution system to provide real-time or near real-time information on water quality that can then be used to control treatment processes at a treatment plant or in the distribution system. The use of continuous chlorine monitors in the distribution system to control disinfectant feed rates at the plant or at in-distribution system booster chlorination stations are examples of this type of monitoring (Uber et al., 2003).

5.1.4 Water Quality Characterization

Information from long-term monitoring of distribution systems can be used to develop baseline trends in water quality for that system. Such information is useful in evaluating a water supply system and for planning upgrades or modifications to system design or operation.

Additionally, if this information is appropriately distributed, it builds consumer confidence and helps to keep customers up to date about the water quality so that they can use this information to make decisions about protecting their health. Currently, there are no standards or guidelines for this type of monitoring. However, for this information to be useful and cost-effective, a regular program for examining and analyzing the collected information is essential.

5.1.5 Multi-Purpose Use of Monitoring Data

Monitoring can be an expensive undertaking in terms of capital costs, as well as operation and maintenance (O&M) costs, including labor. Costs include the purchase and upkeep of equipment, laboratory analysis, labor, and consumable supplies. The investment in monitoring and automated monitoring systems is justifiable if the resulting data are used for more than one objective. For example, if data collected for security purposes can also be used for process control, it should be easy to justify potentially large investments in automated monitoring equipment. Monitoring systems should be properly designed in order to meet multi-purpose requirements.

5.2 Monitoring Techniques

The two major factors in designing and implementing an effective monitoring program are sampling techniques and equipment selection. This section focuses on available monitoring techniques. Samples can be collected and analyzed in two ways: grab samples and/or by automated online monitoring. Automated monitors (continuous or discrete) are sometimes supplemented with automated samplers that can collect both discrete and composite water samples for further analysis at a later date/time. Grab samples are collected manually and analyzed in the field or in the laboratory. Grab samples are labor-intensive in comparison to automated sampling and provide snapshot information about the system at the time of sample collection. Automated monitoring uses online instrumentation, and data is collected by means of sensors and automated data loggers. They can also be tied to a SCADA System. High-end monitors require a higher capital expense for the purchase and maintenance of sensors, data acquisition, data communication, data storage, and data-processing hardware and software. However, this type of monitoring provides a continuous time-series profile of changes in water quality. Both automated and grab sampling can be incorporated into a comprehensive monitoring plan. These techniques are further discussed in the following subsections.

5.2.1 Manual Grab Sampling

Historically, routine water quality monitoring in distribution systems has been carried out through manual grab samples followed by analysis in the field or in the laboratory. Essentially, all regulatory monitoring is still carried out by this method. For example, samples required for large community water supply systems under the SWTR are manually

The equipment routinely required in a manual grab sampling program includes field sampling equipment (e.g., chlorine meter), safety equipment (vests, rain gear, and flashlights), and laboratory equipment. Consumable supplies include sampling containers, reagents, and marking pens. One should identify the needs and availability of equipment and supplies and investigate various sources for equipment. Because equipment malfunction or loss is possible, some redundancy in equipment is appropriate. Some important functions to consider when establishing a field sampling program include the following:

- Establish a systematic and organized method for all sampling and data recording. Take notes to document all aspects of the process.
- Provide training to sampling crews and specify these training requirements in the sampling program plan.
- Contingency planning is important; therefore, consider the potential for equipment malfunction, illness of crew members, communication problems, severe weather, malfunction, and customer complaints.
- Establish a communications protocol to coordinate actions. A means of communication is needed to respond to unexpected events. Alternatives include radios, cellular phones, walkie-talkies, or a coordinator in a vehicle to circulate among field crews.
- Calibrate field analytical equipment before and during the sampling activity.

collected at sites within the distribution system and tested for disinfectant levels in the field. Samples taken to satisfy the requirements of the TCR are also manually collected in the field and subsequently analyzed in the laboratory. Manual sampling is labor-intensive and the number of samples that can be collected is limited by availability of personnel and analysis costs. However, they are specified by some regulations. Potentially important events that may occur between the routine grab samples may be lost (e.g., process upset). Also, there is a potential for dismissing unusual grab sampling results as some type of manual monitoring error (Hargesheimer et al., 2002).

5.2.2 Automated/Online Monitoring

As stated in the report, "Online Monitoring for Drinking Water Utilities" (Hargesheimer et al., 2002), "There is an evolution from grab-sample monitoring to online monitoring as sampling, analysis, data processing, and control functions become more automated." Online monitoring requires a mechanism for moving the sample water from the distribution system to an instrument, appropriate instrumentation for analyzing the water, a mechanism for communicating the results, and a means of assessing the results of the monitoring. Additionally, the instrumentation must be periodically calibrated and maintained for quality control/quality assurance.

In the past, distribution system online monitors were typically housed in a controlled environment with sample lines from the distribution system to the instrument. This resulted in most instrumentation being located at facilities such as tanks and pump stations. The instrumentation was sometimes connected to a SCADA system so that results could be communicated to a central office. More recently, some instrumentation is available that is designed for installation in manholes or for direct insertion into water mains.

The American Society of Civil Engineers (ASCE), in concert with other leading organizations, entered into a cooperative agreement with the EPA to develop standards documents and guidance aimed at enhancing the physical security of the nation's water and wastewater/stormwater systems. Under this agreement, ASCE is leading the effort to develop guidelines for designing an online contaminant monitoring system (OCMS). The Interim Voluntary Guidelines for Designing an OCMS were published in December 2004 (ASCE, 2004). This document provides comprehensive information on several topics, including rationale for OCMS and system design basics, selection and siting of instruments, data analysis, and use of distribution system models.

5.3 Monitoring Equipment Overview

In general, monitors can be categorized by the types of parameters (contaminants, agents, and characteristics) that the monitor is used to measure. For establishing water quality, the monitors are designed to measure one or more parameters that represent physical, chemical, and/or biological characteristics of the system. Typically, in manual grab sampling programs, hand-held physical and/or chemical parameter measuring devices are used. These hand-held devices are carried to the sampling location along with appropriate containers to collect water samples for performing more complex chemical and biological analyses in a laboratory. The online sampling devices are more complex devices that are designed to automatically measure, record, and display specific physical, chemical, or biological parameters. A brief overview of these devices is presented in the following subsections.

5.3.1 Physical Monitors

Physical monitors are used to measure the physical characteristics of the water in a distribution system. They include a variety of instrumentation that measures various macro characteristics, such as flow, velocity, water level, pressure, and other intrinsic physical characteristics. Examples of intrinsic physical characteristics include pH, turbidity, color, conductivity, hardness, alkalinity, radioactivity, temperature, fluorescence, UV254, and ORP. In general, physical monitors tend to be relatively inexpensive, quite durable, and readily available.

5.3.2 Chemical Monitors

Chemical monitors are used to detect and measure inorganic or organic chemicals that may be present in the water. A wide range of chemicals may be of interest, and a large variety of technologies can be used. A specific technology or multiple technologies must be properly selected for a particular chemical or a group of chemicals of interest. Examples of chemical monitors include, but are not limited to residual chlorine monitor, TOC analyzer, and gas chromatograph/mass spectrometer (GC/MS). Typically, the same general type of technology may be available in either automated online monitoring capability or to support manual grab sample analysis.

5.3.3 Biological Monitors

Biological monitors (bio monitors) include bio-sensors and bio-sentinels. Bio-sensors detect the presence of biological species of concern, such as some forms of algae or pathogens. The general operating principles of bio-sensors may include photometry, enzymatic, and/or some form of biochemical reaction. The bio-sentinels use biological organisms as sentinels to determine the likely presence of chemical toxicity in a water sample.

In general, bio-sentinels cannot be used to identify the presence of a specific toxic contaminant – rather only that there is some form of toxic contaminant present. Most bio-sentinels operate by observing the behavior of selected organisms. Examples of such organisms include: fish, mussels, daphnia, heterotrophic bacteria, and algae. When the sentinel organism senses the presence of toxicity, it reacts in some unusual manner. Bio-sentinel instruments respond to these reactions and note that an unusual event is occurring. This application is somewhat analogous to the use of indicator organisms (e.g., total coliforms) to indicate the water quality in the distribution system.

While bio-sensors can be directly applied in distribution systems without pretreatment of the sample, the bio-sentinels are typically used in source waters. This is because most organisms are sensitive to the presence of chlorine (or other disinfectants) in the water. Therefore, if a bio-sentinel is proposed to be used for distribution system monitoring, the water must be de-chlorinated prior to entering the bio-sentinel instrument. Dechlorination may affect detection reliability and the chemical characteristics of the water. Also, the bio-sentinels require a protected housing environment along with some sort of nutritional supply to keep the sentinel organism alive and healthy.

5.4 Establishing Monitoring Requirements

Selection of the types, numbers, and locations of monitors is dependent on the nature of the monitoring program desired. These requirements depend upon the overall monitoring objectives and the distribution system site-specific requirements. For example, a monitor used for regulatory purposes may need to monitor different constituents than one used as part of a process control or security system. Similarly, a different monitor may be needed for a utility that uses chlorine as the disinfectant compared to one that uses chloramine. The site-specific monitoring requirements can be evaluated and represented in the following terms:

- monitoring parameters,
- number and location of monitors,
- nonitor characteristics (e.g., detection limits, sampling frequency, cost, false negatives/false positives), and
- amenability to remote monitoring and SCADA integration.

These requirements are further discussed in the following subsections.

5.4.1 Monitoring Parameters

The parameters to be monitored depend strongly upon the specific use of the monitor and upon utility-specific situations. For regulatory purposes, the regulations typically specify the minimum set of parameters that must be sampled. For each system, the regulating authority typically also specifies the monitoring locations and frequency. A utility may choose to analyze the water for additional parameters and/or increase the frequency of monitoring in order to address other water quality concerns.

For security monitoring, there are no regulations or standards. Utilities can choose whether or not they want to perform such monitoring and select the parameters they will monitor. Generally, such monitoring will be limited by budgets and by technology. Research and development is being conducted on security monitoring systems, in conjunction with event detection platforms, that measure standard parameters, such as TOC, pH, turbidity, conductivity, chlorine, ORP and temperature. For both process control and security-related monitoring, instrument response time is critical. Therefore, online monitors are typically used in these types of applications. The parameters monitored vary widely depending upon the type of process and/or security monitoring.

The goal of online monitoring for security purposes is to automatically analyze the data to determine (1) whether there is an indication of unusual contamination in the sample; and (2) what the likely contaminant is, based on the water quality signature of these parameters.

5.4.2 Number and Location of Monitors

For selecting monitoring locations in distribution systems, there are two related decisions: (1) how many monitors to place in the system, and (2) where to place them. The number of monitors is generally controlled by the monitoring objective (e.g., regulatory requirement) or by budgetary factors, while the location of monitors is a more complex issue that can be addressed in many ways. For example, for compliance with the TCR and the SWTR, there are specific requirements as to the number of samples that must be taken. For most other uses, the number of sampling points (or the number of monitors installed) is controlled by budgetary and financial constraints and through comparison to the benefits associated with the monitors. The following subsections summarize an approach that can be used when the established objectives do not clearly define the number and location of monitors.

EPA's research in contamination warning systems (CWSs) at the T&E Facility is developing data based on bench- and pilot-scale experiments that reveal how traditional water quality parameters, if monitored online, can serve as triggers for contamination events. Figure 5-1 shows the response of several instruments to the injection of secondary wastewater into a DSS.

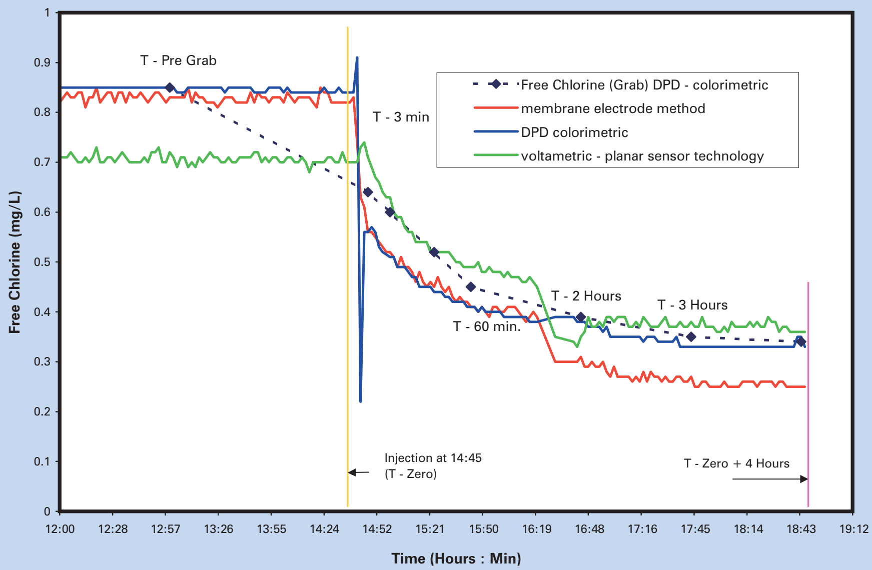


Figure 5-1. Wastewater Injection: Free Chlorine and Associated Grab Sample Results.

5.4.2.1 Number of Monitors

To select the optimum number of monitors for a distribution system, theoretically one can perform a simple cost-to-benefit analysis. If the overall life cycle benefits of each monitor exceed its life cycle costs, analysis would suggest that the monitor is justified. Life cycle costs represent both the capital and operational costs for the monitors. Depending upon the location-specific requirements, as the number of monitors increase, there may be economies of scale or the unit cost may actually increase disproportionately. The unit costs increase when the additional monitors are placed in less convenient locations where servicing and/or data communication costs are higher. Frequently, budgetary constraints may also limit the number of monitors that can be deployed, even if benefits justify their costs.

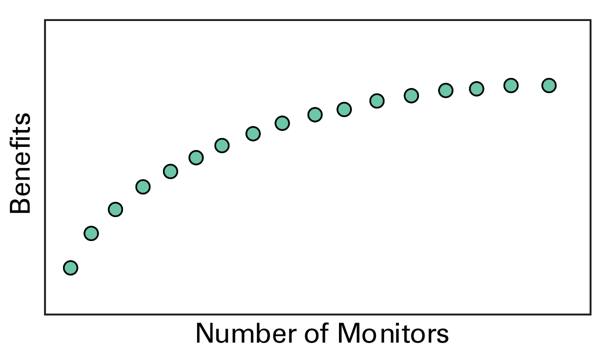


Figure 5-2. Theoretical Example of Benefits from Monitors.

Figure 5-2 is a graphical representation of benefits associated with increasing the number of monitors in a distribution system. This graph illustrates that typically after a basic network of monitors has been established for a distribution system, the incremental benefits gained by installing additional monitors follow the law of diminishing returns. The actual development of such a graph is difficult because of the need to explicitly quantify benefits. In the case of water security-related monitoring, one could measure the value based on population or sensitive facilities (e.g., hospitals) protected by use of online monitors. For other types of monitoring situations, quantification of benefits is more difficult. Though a formal cost-benefit analysis may not be feasible, this discussion provides a general framework that can informally guide the design of a monitoring network.

5.4.2.2 Optimal Monitor Locations

Historically, monitors/sensors have been placed in distribution systems to meet regulatory requirements. Their locations have been determined based on ease of access and a general intuitive assessment of representative locations. Lee et al. (1991) proposed a method for locating monitors, based on the concept of coverage, which is defined as the percentage of total demand that is sampled by a set of monitors. Various other researchers further addressed this issue using alternative mathematical methods (Kessler et al., 1998). Though widely cited, these methodologies have rarely been applied in actual practice. However, following the attacks of September 11, 2001, there has been a renewed interest in the development of monitoring technology and placement of monitors in the distribution system as a mechanism for detecting intentional contamination of distribution systems.

Many current studies are applying optimization techniques to determine the optimal placement for monitors in distribution systems based on a defined objective function. Ostfeld (2004) and Ostfeld and Salomons (2004) provide reviews of past work in this area and present example mathematical formulations

A 17th century Italian economist, Vilfredo Pareto, developed a method for comparing alternatives. Based on his work, a situation is defined as being Pareto-optimal if by reallocation you cannot make someone better off without making someone else worse off. This can be applied to evaluating monitors by examining the diagram (Figure 5-3) where various monitoring options are compared in terms of their cost and some measure of effectiveness. Just looking at alternatives A and B, we can say that A is better than B because it costs less and is more effective. By comparing all potential alternatives, we can define a Pareto front. All alternatives located on that front are better than alternatives located to the right and below the front. This provides a useful conceptual mechanism for evaluating alternative monitoring schemes. For additional information on the work of Pareto, see Johansson (1991). For more details on the application of Pareto's concepts in the area of optimization related to water distribution system analysis, see Walski et al., 2003).

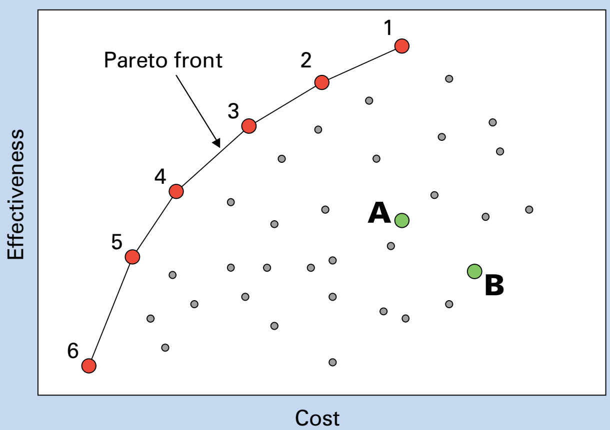


Figure 5-3. Pareto-Optimal Cost Effectiveness Diagram.

using genetic algorithm solution techniques. Their methodology finds an optimal layout of an early warning detection system comprised of a set of monitoring stations aimed at capturing contamination from external sources, nodes, or tanks under EPS conditions. Berry et al. (2004) developed an optimization program that considers the maximum volume of contaminated water exposure at a concentration higher than a defined safe level. The method uses an integer programming optimization technique to place a limited number of "perfect" sensors in the pipes or junctions of a water network so as to minimize the expected amount of exposure to the public before detection, assuming the attack occurs on a typical day.

Watson et al. (2004) use mixed-integer linear programming models for sensor placement over a range of design objectives. Using two case studies, they

Bahadur et al. (2003) describes an approach using PipelineNet in which GIS data and hydraulic model results are used to guide the manual placement of monitors in order to fulfill some general criteria. In a case study conducted with personnel at a water utility, 25 potential monitoring sites were identified and subsequently reduced to two best sites using the GIS/PipelineNet framework. This approach is more closely related to the traditional methods for locating monitors compared to the optimization techniques described in this section.

showed that optimal solutions with respect to one design objective (e.g., population exposed) are typically highly sub-optimal with respect to other design objectives (e.g., time for detection). The implication is that robust algorithms for the sensor placement problem must carefully and simultaneously consider multiple, disparate design objectives.

In general, the optimization methods described above are experimental approaches that have been applied only to hypothetical or small water systems and are based on assumptions about the availability of monitoring technology, ability to define explicit objective functions, and limited incorporation of the variability of water system operation. Further research and development is needed before this technology is ready for routine use.

5.4.3 Monitor Characteristics

The following characteristics of monitors must be evaluated prior to selecting an appropriate device:

- Minimum detection limit (MDL) – The minimum detection limit is the lowest concentration or value at which the monitor can dependably detect the constituent of interest. The MDL can vary for different constituents, different technologies, or for different implementations of the same technology and constituent.

- False negatives/false positives – Two forms of errors associated with a monitor are false positives and false negatives. A false positive exists when a monitor reports, incorrectly, that it has detected a constituent where none exists in reality. A false negative exists when a monitor reports, incorrectly, that a constituent was not detected when, in fact, it was present. False positives can lead to unneeded responses, and repeated false positives will lead to a lack of confidence in the instrument. Lack of detection associated with a false negative results in no response to a real contamination event and can expose consumers to contaminants in the system.

The heightened level of concern over the need to protect water distribution systems has led to the initiation of research into the development of CWSs for both source and finished waters (Clark et al., 2004a). CWSs are intended to reliably identify low probability/ high impact contamination events in source or distributed water. The International Life Sciences Institute (ILSI) developed a report (ILSI, 1999) focused on the development of environmental warning systems (EWSs) for source water. The same development principles apply to distribution systems. EWSs applied to distribution systems are commonly referred to as CWSs. The following design requirements for EWSs were identified by ILSI in their report:

- provides warning in sufficient time to respond to a contamination event and prevent exposure of the public to the contaminant,
- capable of detecting all potential contamination threats,
- remotely operable,
- identifies the point at which the contaminant was introduced,
- generates a low rate of false positive and false negative results,
- provides continuous, year-round surveillance,
- produces results with acceptable accuracy and precision,
- requires low skill and training, and
- be affordable to the majority of public water systems.

A key aspect of an effective EWS will be the need for it to operate in a remote monitoring and reporting mode.

- Sampling frequency – The rate at which a monitor analyzes and reports a value is the sampling frequency. This may vary from a few seconds or less for an instrument such as a pressure gage to an hour or more for instruments that take longer periods to perform the analysis such as a gas chromatograph. For grab sampling, this delay may be even higher. Some instruments can be set for different sampling frequencies. More frequent sampling may result in higher operating costs, shorter battery life, increased data storage requirements, or increased communication needs.

- Amenability to SCADA integration – The monitor's ability to be online and integrated into some sort of SCADA or remote data acquisition system is critical if multiple remote locations are monitored simultaneously. Most current online monitors have analog (e.g., 4-20 mA, 1-20 V) or digital signal (e.g., RS232, RS485) outputs that provide the ability to remotely collect and store data at a central location for analysis.

- Operation and maintenance requirements – The operational requirements of monitors can vary significantly and may strongly impact the selection process. Issues include the electrical needs, expendable material needs (e.g., reagents, wear related components), temperature and humidity requirements, needs to handle waste streams from the monitor, and other factors related to the housing of the monitor. Similarly, the maintenance requirements of the monitors will also impact the selection process. Issues such as how frequently a technician must service the monitor in the field and the level of expertise required to service the device are important considerations when evaluating monitors.

- Combinations of monitors – The ability of a monitoring system to reliably detect a contamination event generally increases with multiple monitors working in tandem. For example, a single monitor that reports a signal slightly above the noise level may easily be dismissed. However, if multiple monitors at several locations in close proximity or several instruments at the same location monitoring for different parameters all detect a potential event, a more forceful and rapid response is likely. An ongoing area of research is the development of data mining algorithms that can differentiate or detect a signal above background levels that are not normally observed in the monitored system.

- Costs – The cost of monitoring systems can vary over several orders of magnitude. A single simple instrument monitoring for a physical parameter such as conductivity may cost less than $1,000. The cost of a multi-parameter physical monitor is typically in the neighborhood of $10,000. More complex instruments such as a TOC monitor or a GC/MS cost in the range of $25,000 to $90,000. The cost of more complex instruments or a monitoring station containing multiple instruments can easily exceed $100,000 in capital cost. Installation and ongoing maintenance costs are frequently site-specific and vary according to environmental conditions.

5.4.4 Amenability to Remote Monitoring and SCADA Integration

For a comprehensive network-wide water quality remote monitoring program, it is essential to ensure that the system and its monitored components are amenable to remote monitoring and SCADA integration. The SCADA component adds the element of control to the monitored network. Most utilities have some sort of SCADA functionality to automate and monitor the key water treatment and/or distribution operations. The control logic is typically triggered based on a specified time and/or event. For example, the pumps may be set to fill a distribution system tank at midnight and when the tank level monitor detects that the tank is full (an event reported through the SCADA system), the control logic to turn off the pumps is initiated. This type of control logic can be enhanced to perform control functions based on detection of water quality change in the distribution system. However, to achieve this functionality, one needs to understand the following three major components of a remote monitoring and/or control system (or SCADA):

- online sampling instruments (e.g., pH, ORP) and/or control devices (e.g., pump, valves),
- SCADA or remote monitoring network, and
- field wiring and communications media.

These components are discussed briefly in the following subsections.

Electric power is generally required for operating these components. If electric power is not readily available at the desired location where a monitor is to be installed, consider the costs for installing a suitable power apparatus (e.g., a solar panel, battery pack).

5.4.4.1 Online Sampling/Control Devices

Online sampling/control devices can be the most expensive component of a SCADA system. The sensors, switches, monitors, and controllers used in a SCADA system may vary widely, depending upon the parameters that need to be controlled and/or observed. The cost for online sampling devices can range from a few hundred dollars to over $100,000. Control units such as sample feed pumps or shut-off valves are less expensive (Panguluri et al., 1999). Costs associated with maintenance and calibration of the online sensors when planning the acquisition and implementation of a remote monitoring network should also be considered.

5.4.4.2 SCADA or Remote Monitoring Network

Larger utilities typically use some type of SCADA system for water distribution system control that can easily be integrated to include online sampling instrumentation in a cost-effective manner. Also, recent advances in electronic hardware and software technologies have resulted in several cost-effective SCADA alternatives for smaller systems. A microprocessor-based "smart" SCADA system can be used in remote locations by small system operators where direct online communication is expensive. Smart systems have higher initial costs, but overall costs are reduced since the communication costs (e.g. long-distance phone costs) are negligible because most of the burden is transferred from the main computer to the individual SCADA unit at the remote site (Panguluri et al., 1999). Newer SCADA units are fairly inexpensive, with capital costs ranging between $500 (PC card-based units and remote data collection nodes) and $5,000 (independent PC-based full SCADA units).

Sensors and Transducers: A sensor responds to a physical and/or chemical stimulus, such as thermal energy, flow, light, chemical, pressure, magnetism, or motion. A transducer takes the measured physical and/or chemical phenomenon (e.g., pressure, temperature, humidity, and flow) and converts it to an electrical signal. In each case, the electrical signals produced are proportional to a physical and/or chemical quantity being measured based on a pre-defined relationship. The electrical signals generated by transducers often require "conditioning." Depending upon the transducer, a signal conditioner can be used to perform one or more conditioning functions, such as noise filtration, amplification, linearization, isolation, and excitation.

The data acquisition hardware processes the digital and analog inputs/outputs from various online sampling and control devices. For monitoring systems, the hardware typically processes the analog data measured from various instruments and transfers it to a computer system for display, storage, and analysis. In a monitoring/control system (SCADA) scenario, the hardware would process both analog and digital inputs (typically from a field device) and outputs (to perform control functionality). The application software provides the operator the display, control, and analysis (trends and reports) of collected data.

5.4.4.3 Field Wiring and Communication Media

Depending upon availability, cost, user preference, and the relative location of the sensors to the data acquisition system, the communication media can be either wired (e.g., direct, phone line) or wireless (e.g., radio, cellular). In field environments, distributed input/output (I/O) is typically employed. A remote data acquisition hardware unit employed at the field location performs the appropriate signal conditioning and transmits the data to a central hub. More recently, mesh or grid computing systems are used in remote locations to add redundancy in cases of link failures. The field wiring between the sensor and the remote data acquisition hardware unit is usually direct wire.

Depending upon the area covered and availability, in some cases it may be preferable to use some form of radio communication devices. The available radio communication devices operate mostly in the very high frequency (VHF) or ultra high frequency (UHF) range. The VHF frequencies range between 30-300 megahertz (MHZ) and the UHF frequencies range between 300-1,000 MHZ. In U.S., most of the available VHF/UHF radio frequencies are licensed. The unlicensed bands available include the industrial, scientific and medical device channels with frequency ranges between 902 - 928 and 2,400 - 2,484 MHZ. The unlicensed bands do not have any connection or monthly fee requirements.

Typically, direct wire and phone line (including cellular) communication media are inexpensive. The primary limitations associated with selecting the communication media include installation and operating costs, which can vary between $200 (for a simple telephone or cellular modem) and several hundred dollars for a satellite-based system per location. Ongoing monthly operating costs can range from $25 for a phone line to approximately $200 per month for satellite-based services within the U. S (per monitored location).

5.5 Engineering and Evaluating a Remote Monitoring System

Once all of the basic requirements have been established (e.g., objectives, parameters, location) as outlined in the previous section (Section 5.4) and the requirements indicate a need for a system-wide remote monitoring program for water quality, the following additional site-specific needs should be evaluated for water quality monitoring in a distribution system (Panguluri et al., 1999):

- What are the complexities of the distribution system (size, location)?
- What locations are best suited for sampling and/or control system installation?
- Is sufficient flow and water pressure available for online instruments?
- Is there an existing SCADA system available?
- What types of communication media are available at the selected locations?
- How many parameters are going to be monitored and/or controlled at each location?
- What other site-specific information (e.g., availability of power, access, security) will be needed?

Additonal factors to be considered are (Haught and Panguluri 1998):

- system features (e.g., ease of operation, customization, networkability, operator security),
- cost (initial, training, service agreements, and operation and maintenance), and
- vendor support (hardware and software upgrades and remote diagnosis).

It is important that each site is evaluated individually for appropriate SCADA system selection. The cost of SCADA software has plummeted over the past few years. For example, the cost of one commercially available graphical (Windows-based) SCADA software package has dropped from $30,000 in the early 1990s to $2,000 today.

Prior to selecting and implementing a remote monitoring network, one should evaluate the options carefully. Engineering a remote monitoring system is a difficult task that typically involves many factors: multi-dimensional objectives, changing needs, rapid

Besides the aforementioned immediate needs (e.g., ease of operation, customization, networkability), SCADA system features include:

- Scalability: This allows for future growth with respect to addition of I/O blocks with more channels or advance capabilities. These I/O channels are used to communicate with various field monitoring instruments (sensors) and control devices.
- Local Memory: The SCADA hardware must also contain sufficient local memory to store the monitored data for extended periods of time in case of communication failures.
- Remote operation and diagnosis: In the event of brownouts or blackouts, the field SCADA units should normally self-boot upon resumption of power supply. The field SCADA units should also allow for remote diagnosis.
- Call-out feature: This feature allows the system's software to notify appropriate personnel if problems develop with a treatment system or water quality. This feature can greatly enhance operator response in emergency situations and prevent costly shutdowns and loss of water and/or water quality.
- Open Database Connectivity (ODBC): This feature allows for open communication with other databases and tools that can be integrated to provide additional features. The data then can also be used for network modeling.

technological change, conflicting technical claims, and budgetary constraints. The following subsection presents general methods for evaluating and assessing alternatives followed by a set of specific criteria for evaluating alternative monitoring systems.

5.5.1 Remote Monitoring System Evaluation

In order to justify a remote monitoring system and to select the best monitoring system, it would be ideal if one could evaluate the benefits derived from monitoring and compare them to costs and choose the system that maximizes net benefits subject to budgetary constraints. Depending on the uses of the monitoring data, monitoring benefits may be associated with:

- reduced risks from an intentional or accidental contamination event,
- improved understanding of the variation in water quality of a distribution system,
- enhanced operation if the data are used as part of a process control system, and
- increased compliance if the information is used for regulatory purposes.

5.6 Monitoring Case Studies

EPA has conducted research into the use of remote monitoring and control technology alternatives for many years. These projects have involved both water treatment systems and water distribution (Clark et al., 2004b). The agency's first research project that incorporated real-time monitoring at a remote location was conducted at the T&E Facility. The initial research was focused on evaluating SCADA systems for small drinking water package plants. The goal was to demonstrate that SCADA systems could be used to monitor and control several small plants remotely from a centralized location at one time (Haught and Panguluri, 1998). The following case studies represent some of the highlights of the research and collaboration with different water utilities.

5.6.1 Rural Community Application

In May 1991, EPA provided funding to support a research project titled "Alternative Low Maintenance Technologies for Small Water Systems in Rural Communities" (Goodrich et al., 1993). This project involved the installation of a small drinking water treatment package plant in a rural location in West Virginia. The primary objective of this study was to evaluate the cost-effectiveness of package plant technology in removing and disinfecting microbiological contaminants. The secondary objectives of this project included: remote monitoring and automation of the system to minimize the O&M costs, assessment of the community's acceptance of such a system, ability to pay, and the effect of the distribution system on water quality at the tap. The following is a brief summary of the overall project.

The treatment system was located in rural Coalwood (McDowell County), WV, approximately 12 miles from the McDowell County Public Services Division office. Prior to 1994, an aerator combined with a slow sand filter was being used for water treatment at this site. This combined unit had been operational for over 30 years and needed substantial repairs. The water flowed by gravity from an abandoned coal mine to an aerator built over a six-foot-diameter slow sand filter. A hypochlorinator provided disinfection of the treated water, and the water flowed by gravity through the distribution system to the consumer. The volume of water from the mine was considered sufficient for the small rural community.

Based on a review of existing technology, EPA determined that a packaged ultrafiltration (UF) system would be ideally suited for this location. In 1992, a UF unit was purchased and installed at this site. In 1996, EPA developed, installed, and tested a remote monitoring system at the site. The system used commercially available hardware along with EPA-developed software. The software was not user-friendly and the overall cost of ownership was very high. Therefore, in 1998, EPA updated the SCADA system with a scalable commercially available off-the-shelf, user-friendly SCADA system. The total cost (including instrumentation, technical support, training, and set-up) was approximately $33,000. EPA installed similar SCADA systems at Bartley and Berwind sites in McDowell County, WV, for remote monitoring of water quality.

5.6.2 Washington D.C. Remote Monitoring Network

Following a number of coliform violations, EPA's Region 3 office directed the Washington D.C. Water and Sewer Authority (WASA) to implement a number of corrective actions for its water distribution system (Clark et al., 1999). Remote monitoring of water quality parameters within the distribution system was identified as being one possible method for identifying water quality problems. Consequently in 1997, EPA initiated a study to install a remote network at various locations in Washington D.C. to monitor water quality within the distribution system (Meckes et al., 1998). The WASA staff teamed with EPA to select appropriate locations within the distribution system for installation of online sampling stations. Following are some of the study objectives:

- development of methods to monitor real-time water quality at various locations within the distribution system,

- field evaluation of sensors and remote monitoring technologies for inclusion in the network,
- development of effective methods to publish real-time data that enhanced consumer confidence,
- evaluation of costs associated with implementing such systems, and
- identification of potential problems and suggestions for remedial actions when implementing remote monitoring networks.

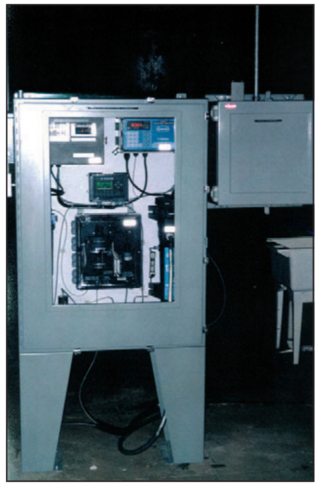


Figure 5-4. Fort Reno #2 Remote Sampling System.

Free chlorine, pH, temperature, and turbidity were selected as the monitored parameters based on the availability of online sensor technologies. The selection was based on the premise that these were parameters which could be reliably monitored continuously and the selected instruments required limited maintenance. Additionally, WASA used their SCADA system to track various operating parameters within the distribution system. During the evaluation, it was clear that use of the existing SCADA system to manage the monitored data provided clear advantages over other available systems. Using the existing SCADA system minimized long-term on-site support costs.

After suitable location(s) were identified, customized sampling and monitoring systems were built. The remote monitoring system in Washington D.C. was implemented in three phases. In the first phase, a remote monitoring system was installed at the Fort Reno #2 tank (Figure 5-4), which provided security and easy access to the distribution system. Subsequently, based on initial success at this location, two other sites (Bryant Street and Blue Plains) were selected and added to the remote monitoring network in the second phase. The third phase involved the development of a Web-based application to publish the real-time data in order to enhance consumer confidence.

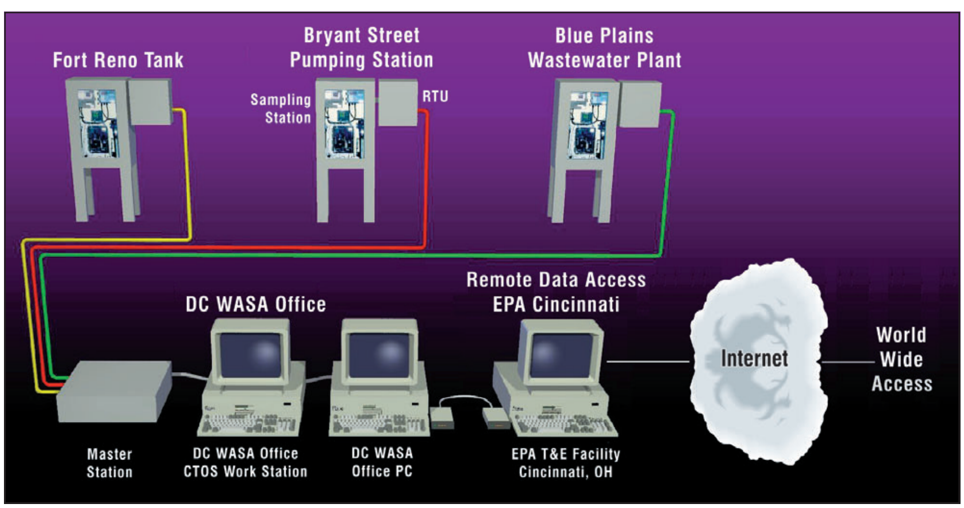


Figure 5-5. WASA Remote Monitoring System Layout and Data Transmission Scheme.

Figure 5-5 shows the relationship between the SCADA system and the transmission of the data. Although WASA's SCADA system used a proprietary operating system, it provided a personal computer (PC) link which was used to dump data into a regular PC for further processing. The hardware-based feature enabled tight security; an authorized end user could only copy the relevant data published on the PC and could not directly access the SCADA system. This feature also eliminated any potential interference between the sampling system data and other distribution system operations data. Unfortunately, the EPA funding for this study was terminated and, as a result, the systems and the Website are currently not operational. The overall project, however, did demonstrate that such systems could be developed and operated. Figure 5-6 shows some of the output data for the Fort Reno tank which indicates the loss of disinfectant chlorine levels at night. Clearly, this type of information can be used to improve system operations to better maintain the water quality.

5.6.3 Tucson Water Monitoring Network

Based on a grant received from the EPA's Environmental Monitoring for Public Access and Community

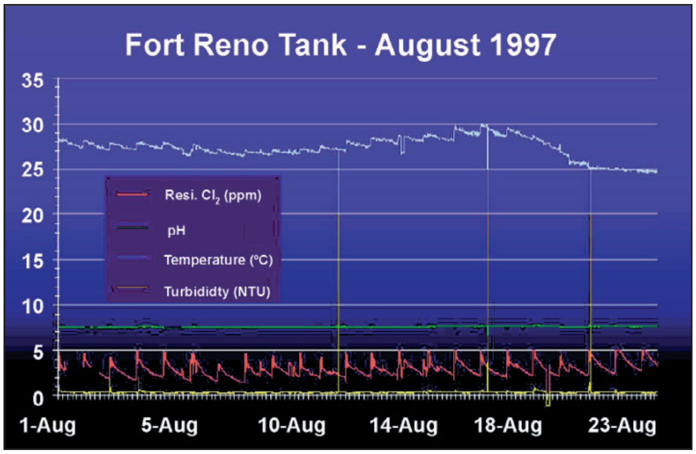


Figure 5-6. Monitoring Data for Fort Reno Tank.

Tracking (EMPACT) Program, the city of Tucson implemented a comprehensive water quality monitoring program. The city's EMPACT goals included the following: implementing enhanced monitoring of the utility's potable distribution system, providing the community with near real-time water quality information on Tucson Water's Website (www.cityoftucson.org/water), and creating community partnerships to better inform water consumers about water quality and resource issues. The water quality monitoring and data collection tools provided through EMPACT also enables the utility to track and respond to real-time changes in system water quality.

Tucson Water's distribution system consists of one central drinking water distribution system that serves the majority of the customers and ten isolated drinking water distribution systems. All eleven drinking water distribution systems cover a service area of 300 square miles and serve 680,000 customers in the Tucson metropolitan area. The two types of source water that supply the central distribution system are native groundwater and renewable recharged surface water from the Colorado River. The source water that supplies the ten isolated distribution systems is groundwater.

For the purposes of monitoring, the central distribution system is divided into ten water quality zones and each isolated distribution system is considered an individual water quality zone. Figure 5-7 shows the zone map. A water quality zone is defined as an area of the distribution system that is similar in water quality characteristics, water pressure, geographical, and political boundaries. Each water quality zone has a set number of dedicated sampling stations and points-of-entry (POE). The dedicated sampling stations monitor the quality of the drinking water in the distribution system before delivery to the customer. The POEs are usually individual wells that represent the water quality of a single well or in a few cases, combined POE systems that represent the collective blended water quality from a group of wells that directly supply Tucson's drinking water.

In total, there are 262 dedicated sampling stations and approximately 154 active POEs located within the multiple distribution systems. In addition, 22 online water quality stations (for monitoring: chlorine residual, total dissolved solids, pH, and temperature) are located throughout the central distribution system at strategic locations, such as reservoirs, well sites, and booster stations, as one of the primary objectives of the EMPACT program.

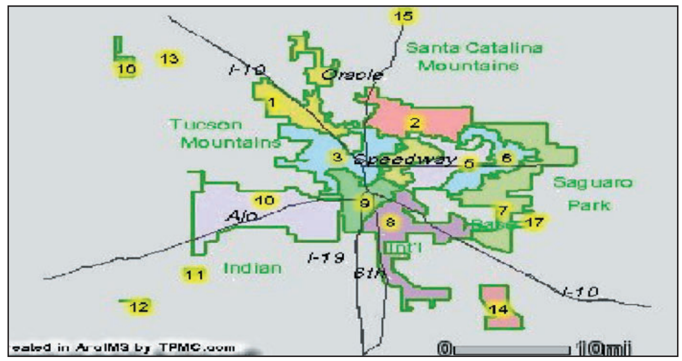


Figure 5-7. City of Tucson Water Quality Zone Map.

Figure 5-8 depicts a continuous water quality monitoring station. The monitoring frequency ranges from tri-annually (for grab sample locations) to every 60 seconds (for continuous monitoring stations), depending on the location and specific monitoring program that is being utilized for that location.

The comprehensive water quality monitoring program encompasses the entire distribution system. Source waters are monitored and sampled according to the Arizona Department of Water Resources and the

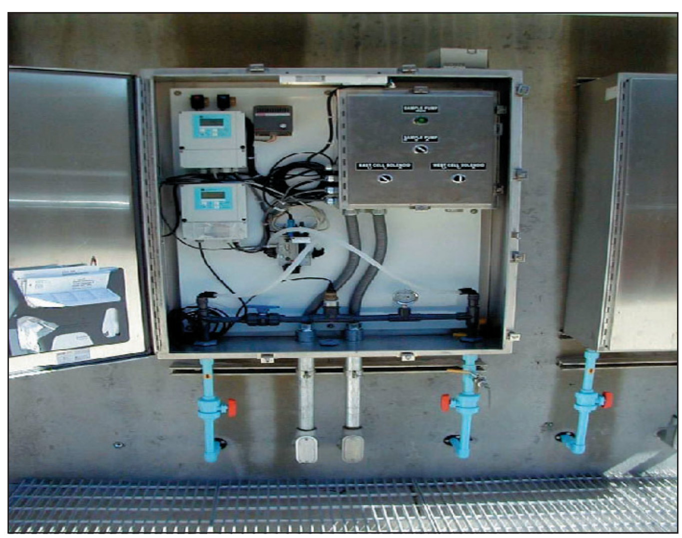


Figure 5-8. Continuous Water Quality Monitoring Station.

Arizona Department of Environmental Quality (ADEQ) regulations, while the drinking water is monitored according to EPA and ADEQ regulations and drinking water standards. Drinking water is also evaluated against a set of consumer-established water quality goals. Special-purpose samples are taken to characterize and track changing trends in water quality for both source water and drinking water. All data sets are utilized to track and monitor changes in water quality to learn the baseline water quality operating parameter levels and also to be able to identify and react appropriately when a contamination event occurs. Most of the analysis is conducted by the utility's water quality laboratory and all the results are tracked through the Water Quality Management Division.

All 262 dedicated sampling locations are monitored at least once each month for total coliform and chlorine residual, while 26 other parameters are monitored once every three months. Based on the water quality measurements collected each month from these 262 sampling locations, the trends in water quality conditions are determined for each water quality zone and for the distribution system as a whole. This information can be found on the aforementioned Web site in the Water Quality section under Tucson's Water Quality and Water Quality in My Neighborhood links. The water quality information displayed on two interactive maps shows data charts and tables for each location that is sampled under the Water Quality program. In addition, the information provided to all Tucson water customers in the annual water quality report or consumer confidence report is based on POE monitoring data.

5.7 Summary and Conclusions

Distribution system monitoring is intended to identify the spatial and temporal variations in water quality that take place in a drinking water system. Monitoring data can be used to satisfy various objectives, such as regulatory requirements, security requirements, or process control requirements. The costs of implementing such a system can best be justified if the resulting data can be used for more than one of the aforementioned objectives.

A monitoring program can implement either routine grab sampling or continuous monitoring. A combined approach, utilizing both continuous and grab sampling data, may prove to be very effective as the basis for a comprehensive system-wide monitoring plan. In the past, distribution system online monitors were typically housed in a controlled environment with sample lines from the distribution system to the instrument. This resulted in most instrumentation being located at facilities such as tanks and pump stations. The instrumentation was sometimes connected to a SCADA system, so that results could be communicated to a central office. Recently, some instrumentation has been designed for installation in manholes or for direct insertion into water distribution system pipes.

Vulnerability assessments performed by utilities and various research studies have identified that water distribution systems are vulnerable to intentional or accidental contamination. In addition to hardening systems to make it more difficult to contaminate a system, monitoring as part of a CWS has emerged as a logical approach to cope with potential contamination events. Monitors can also be used in a distribution system to provide real-time or near real-time information on water quality. The data can then be used to control treatment processes at a treatment plant or in the water distribution system. However, this type of program may not be practical for small systems.

SCADA is widely used in industrial environments and by larger water utilities to control and monitor their individual facility operations. However, water utilities typically do not use available SCADA systems for conventional water quality monitoring. Water utilities typically monitor water quality parameters by performing grab sampling on a scheduled or random basis that provides a periodic snapshot of the overall system. Current drinking water regulations require all public water systems to implement water quality monitoring for total coliform to ensure that good quality water is provided to consumers (EPA, 1996). Since the regulations do not clearly specify that real-time monitoring of water quality is required, utilities have been reluctant to install and operate such devices.

After the events of 9/11, utilities have become more interested in the potential for continuous water quality monitoring. SCADA systems can assist in this function by constantly monitoring water quality within drinking water distribution systems. These systems can potentially reduce the risk of security related threats or even non-security related threats, and detect undesirable water quality changes within a system (Meckes et al., 1998).

Users should evaluate monitoring data appropriately for errors and inconsistencies before commencing actions based on acquired data. Each component in a monitoring system is a potential source of error. For example, a remote monitoring system could have data errors for one or more of the following reasons: instrument errors and spikes, SCADA data errors related to system failure, backfilling due to communication failure, timing errors, or missing data. It is important to validate data and understand routine changes in water quality due to system-specific operations.

Monitoring equipment should be chosen appropriately after establishing the monitoring requirements. The individual monitor characteristics, costs, and amenability to SCADA integration are key to effective implementation. Each system should be individually examined and engineered for implementation.

The monitoring case studies presented in this chapter demonstrate the manner in which effective monitoring systems can be implemented in small, medium, and large distribution systems. If the data are used for responding to a contamination threat, it is important to understand the movement of water in the system.

References

ASCE. *Interim Voluntary Guidelines for Designing an Online Contaminant Monitoring System.* Published by American Society of Civil Engineers, Reston, VA. 2004.

Bahadur, R., W. Samuels, W. M. Grayman, D. Amstutz, and J. Pickus. "PipelineNet: A Model for Monitoring Introduced Contaminants in a Distribution System." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Philadelphia, PA. 2003.

Berry, J., W. Hart, C. Phillips, and J. Uber. "A General Integer-Programming-Based Framework for Sensor Placement in Municipal Water Networks." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Salt Lake City, UT. 2004.

Clark R.M., W.M. Grayman, S.G. Buchberger, Y. Lee, and D.J. Hartman. "Drinking Water Distribution Systems: An Overview" in *Water Supply Systems Security*. Edited by L. W. Mays, McGraw-Hill, NY, pp 4.1-4.49. 2004a.

Clark, R.M., S. Panguluri, and R.C. Haught. "Remote Monitoring and Network Models: Their Potential For Protecting U.S. Water Supplies," in *Water Supply Systems Security*, edited by L. W. Mays, McGraw-Hill, NY, pp 14.1-14.22. 2004b.

Clark, R.M., G.S. Rizzo, J.A. Belknap, C. Cochrane. "Water Quality and the Replacement and Repair of Drinking Water Infrastructure: The Washington, DC Case Study." *Journal of Water Supply Research and Technology - Aqua*, 48(3):106-114. 1999.

EPA. *Drinking Water Regulations and Health Advisories.* Office of Water, EPA 822-B-B-96-002. October, 1996

EPA. "Response Protocol Toolbox." Overview and Modules 1 through 6. Can be downloaded at: http://cfpub.epa.gov/safewater/watersecurity/home.cfm?program_id=8#response_toolbox. 2003-2004.

Goodrich, J., J. Adams, and B. Lykins, Jr. "Ultrafiltration Membrane Application for Small System." EPA National Risk Management Research Laboratory. 1993.

Haught, R. C., and S. Panguluri. "Selection and Management of Remote Telemetry Systems for Monitoring and Operation of Small Drinking Water Treatment Plants." Proceedings, First International Symposium on Safe Drinking Water in Small Systems, May 10-13, 1998, Washington, D.C. USA. 1998.

Hargesheimer, E., O. Conio, and J. Popovicova (Editors). *Online Monitoring for Drinking Water Utilities.* AwwaRF – CRS ProAqua. 2002.

International Life Sciences Institute (ILSI). *Early Warning Monitoring to Detect Hazardous Events in Water Supplies.* ILSI Press, Washington D.C. 1999.

Johansson, P. *An Introduction to Modern Welfare Economics.* Cambridge University Press, Cambridge, UK. 1991.

Kessler, A., A. Ostfeld, and G. Sinai. "Detecting Accidental Contaminations in Municipal Water Networks." *Journal of Water Resources Planning and Management*, ASCE. 124(4):192-198. 1998.

Lee, B., R. Deininger, and R. Clark. "Locating Monitoring Stations in Water Distribution Systems." *Journal of AWWA*, 83(7):60-66. July 1991.

Meckes, M. C., J. S. Mattingly, G. J. Papadopoulos, M. Dosani, and S. Panguluri. "Real Time Water Quality Monitoring of a Water Distribution System Using Remote Telemetry." Proceedings, AWWA Distribution System Symposium, Austin, Texas. September 20-22, 1998.

Ostfeld, A. "Optimal Monitoring Stations Allocations for Water Distribution System Security" in *Water Supply Systems Security*, edited by L. W. Mays, McGraw-Hill, NY, pp 16.1-16.15. 2004.

Ostfeld, A., and E. Salomons. "Optimal Layout of Early Warning Detection Stations for Water Distribution Systems Security." *Journal of Water Planning and Resources Management*, ASCE. 130(5):377-385. 2004.

Panguluri, S., R. C. Haught, M. C. Meckes, and M. Dosani, 1999. "Remote water quality monitoring of drinking water treatment systems." Proceedings, AWWA Water Quality Technology Conference, Denver, CO. November 1999.

Uber, J., F. Shang, M. Ploycarpou, and Z. Wang. "Feedback Control of Booster Chlorination Systems." AwwaRF, Denver, CO. 2003.

Walski, T.M., D.V. Chase, D.A. Savic, W. Grayman, S. Beckwith, and E. Koelle. *Advanced Water Distribution Modeling and Management*. Haestad Press, Waterbury, CT. 2003.

Watson, J., H. J. Greenberg, W. E. Hart. "A Multiple-Objective Analysis of Sensor Placement Optimization in Water Networks." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Salt Lake City, UT. 2004.

Chapter 6
Geospatial Technology for Water Distribution Systems

Section 6.1 provides a brief summary of the history and development of geospatial data management based on information extracted from many sources. This history is included in order to provide a context for the current geospatial data management methodologies in use today by utilities. Readers who are already familiar with the history may choose to skip Section 6.1 of this chapter.

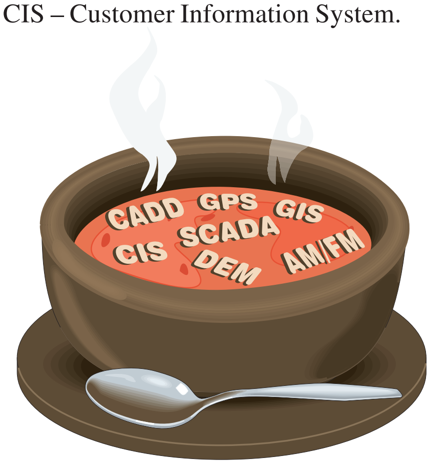
Geospatial data identifies the geographic location and characteristics of natural or constructed features and boundaries on the earth. This information may be derived from various sources of data, including remote sensing, mapping, and surveying technologies. More simply, geospatial data is any information in or on the earth that has a "where" component. This can be a house address, a street intersection on a map, a pump station with a coordinate location stored in a facilities list, or the location of the sampling tap on a diagram of a pump station. Thus, every object has a geospatial data component based on its location.

Geospatial data provides a mechanism for incorporating geographic locations of various functions and facilities in a distribution systems analysis. The cost of incorporating map data into the water distribution systems discipline is decreasing, which enables a wider audience of users to perform powerful spatial analyses over time, such as master plan development, pipe break analysis, and locational information on sensitive subpopulations (e.g., nursing homes, schools). As these tools and datasets become more commonly used and shared among engineers, new efficiencies will be realized that will have a positive impact on water distribution system management.

Water systems are by nature quite geographically extensive and the location of a particular component or feature may significantly affect its performance. Source watersheds can cover hundreds or thousands of square miles. Similarly, distribution systems can cover vast areas. The operation of a water system entails moving water from one location to another. Elevation (Z), the third dimension of location (along with the X and Y dimensions of a Cartesian coordinate system), is an important factor in designing and operating a water system. This illustrates that the management of a water system is inherently a geospatial issue.

Because of the spatial nature of water systems, many aspects of managing a water system consist of using, managing, and displaying geospatial data. This has led to a variety of mechanisms ranging from maps and plans to sophisticated, computerized database management systems. The following is a list of some of the computerized data systems that water utilities typically use for managing their spatial data. These systems will be discussed in greater detail later in this chapter.

- GIS – Geographic Information System.
- CADD – Computer-Aided Design and Drafting.
- AM/FM – Automated Mapping (or Asset Management)/Facilities Management.
- CIS – Customer Information System.



- DEM – Digital Elevation Model.
- GPS – Global Positioning System.
- SCADA – Supervisory Control And Data Acquisition.
- LIMS – Laboratory Information Management System.
- LIS – Land Information System.
- RDBMS – Relational Database Management System.
- SDMS – Spatial Data Management System.

6.1 History of Geospatial Data Management

Geospatial data is of interest in many professional fields and each of these fields has approached the issue of using and managing geospatial data in a different manner. Examples include the following:

- Cartographers concentrate on making maps.
- Surveyors emphasize accurate capture of locational information on natural and manmade land features.
- Engineers use spatial data to draw construction plans and, more recently, use it as input for various types of models.
- Planners use maps and spatial data to assess growth and to determine the suitability of land to support a particular type of development.
- People in the public works area are concerned with managing assets such as streets, sewers, and water lines, which all have a spatial component.
- Fields such as the military, engineering, mining, and hydrology are interested in topographic (elevation) data.

As a result of these varying interests in "spatially arrayed data," the tools and methodologies for managing these data have evolved from many directions and recently there has been a significant move towards integrating the basic concepts. A brief history of geospatial data management is presented below, organized by the various disciplines that have influenced this field. The needs of the water industry in the spatial arena cross each of these areas. Geospatial data management in the water industry will be discussed in greater detail in a later section of this chapter.

6.1.1 Mapping, Surveying, and Remote Sensing

Mapping is the oldest of the geospatial disciplines. Examples of maps date back many millenniums. A wall painting, dating back to around 6200 B.C. in Turkey, depicts the positions of the streets and houses of the town together with surrounding features such as the volcano close to the town. The Babylonians produced clay tablets containing maps that date back to around 1000 B.C. Other early maps were prepared by the Egyptians and Chinese (O'Connor and Robertson, undated).

Many of the advances in map making are attributed to the Greeks. Around 350 B.C., Aristotle argued that the earth was a sphere and around 250 B.C., Eratosthenes accurately calculated the circumference of the earth. In 140 A.D., Ptolemy's eight-volume *Guide to Geography* was written and provided the basic principles of cartography. It introduced the concept of map projections and attempted to map the known world, giving coordinates of the major places in a system akin to the present day latitude-longitude system. This document served as the definitive reference on geography for over a thousand years and was later translated into Latin and printed in 1475.

The 16th century saw the introduction of globes and many improvements in the mathematical basis of cartography. Gerardus Mercator developed a wall map of the world in 1569 on 18 separate sheets (see Figure 6-1). In the "Mercator projection," lines of longitude, lines of latitude, and rhomb lines all appear as straight lines on the map. This projection was a great aid to navigators and is still in use today. With the basis of cartography well in hand almost 500 years ago, the cartographic methodologies continued to evolve. Additionally, emphasis was placed on methods of accurately establishing the coordinates of places of interest. This led to the field of surveying and, more recently, the field of remote sensing.

Figure 6-1. Mercator's Map of the World in 1569 (Whitfield, 1994).

"Surveying is the science and art of measuring distances and angles on or near the surface of the earth. It is an orderly process of acquiring data relating to the physical characteristics of the earth and in particular the relative position of points and the magnitude of areas. Evidence of surveying and recorded information exists from five thousand years ago in places such as China, India, Babylon, and Egypt" (Queensland Government, undated). Some key inventions in the area of surveying include the following:

- Knotted ropes – Measuring device developed by the Egyptians and used in construction of the pyramids.
- Levels – Mechanism developed by the Egyptians composed of a hanging "plumb bob" used to establish a level surface.

- Magnetic compass – Device for determining north direction. Developed by Chinese circa 200 B.C. and composed of a magnetic “lodestone.” Later used by Chinese for navigation.
- Theodolite - An instrument, graduated in 360 degrees, used in the mid-1500s by an Englishman, Leonard Digges, to measure angles.
- Alidade and Plane table - Sighting mechanism and flat table developed in 1590 for mapping the surface features of the earth. Attributed to Jean Praetorius.
- Quadrant and Sextant - The quadrant is an apparatus developed in 1730 by John Hadley for measuring angles of celestial bodies. This led to the development of the sextant, which is a precision instrument made from brass or aluminum that is used for ocean navigation where celestial observations are taken to plot a ship’s position.
- Transit - The transit is used to measure vertical and horizontal angles and may also be used for leveling; its chief elements are a telescope that can be rotated (transited) about a horizontal and about a vertical axis, spirit levels, and graduated circles supplemented by Vernier scales. Attributed to W.J. Young in Philadelphia in 1831.
- Electronic Distance Measurement (EDM) – Starting in the 1950s, electronic distance measuring instruments were developed and have now largely replaced traditional methods for measuring distance. Horizontal distances are measured using a variety of instruments that employ a laser beam aimed at a reflector station. Low-cost instruments that employ sound waves or infrared beams are also available.
- GPS - The use of GPS in surveying procedures is the most recent and revolutionary change to influence land measurement. GPS was designed and built and is operated and maintained by the U.S. Department of Defense. Originally called the Navstar GPS, it was first brainstormed at the Pentagon in 1973. In 1978, the first operational GPS satellite was launched; by the mid-1990s, the system was fully operational with 24 satellites. The basic principle behind GPS is the measurement of distance between satellites and the receiver. The distance to at least 3 satellites must be known in order to find out a position. Satellites and receivers generate duplicate radio signals at exactly the same time. As satellite signals travel at the speed of light (186,000 miles per second), they only take a few hundredths of a second to reach the GPS receiver. This difference and the speed at which the signal travels is used in the equation to find out the distance between the GPS receiver and the satellite (Radio Shack, 2004). GPS is now also being used to provide information on elevations.

Local governments frequently store survey information on parcels in an LIS. This information can include property ownership, construction date, land assessment, and land taxation. This information may be linked to a computerized database system for storing the geographic coordinates of the parcels.

Remote sensing refers to imagery from airplanes or satellites. Some early examples of remote sensing include: aerial photography from a balloon in 1859 by Gaspard Felix Tournachon in an attempt to conduct a land survey; use of light cameras attached to pigeons in Bavaria in 1903 to monitor troop positions; photographs of San Francisco following the 1906 earthquake by George Lawrence from a kite; and the work of a photographer who accompanied Wilbur Wright on one of his first demonstration flights in 1909. More serious aerial photography was conducted during World War I and II and during the Cold War period.

Aerial photography has become a staple item in the development of maps and documentation of land use changes. The National Aerial Photography Program (NAPP) is an interagency Federal effort coordinated by the USGS, which uses NAPP products to revise maps. Other agencies have varied uses for these photographs, which are taken on a 5- to 7-year cycle and produced to rigorous specifications. The NAPP effort encompasses the entire lower 48 states and Hawaii. The photos are acquired from airplanes flying at an altitude of 20,000 feet using a 6-inch focal length camera resulting in a scale of 1:40,000. Each 9-inch by 9-inch photo (without enlargement) covers an area of slightly more than 5 miles on a side. The NAPP effort began in 1987 and replaced the National High Altitude Photography (NHAP) program which was initiated in 1980. Strict specifications regarding sun angle, cloud cover, minimal haze, stereoscopic coverage, and image inspection were followed and all NAPP photography is cloud-free (USGS, undated).

Satellite remote sensing can be traced to the early days of the space age (National Aeronautics and Space Administration [NASA], undated). On April 1, 1960, the Television and Infrared Observation Satellite (TIROS 1) was launched, which proved that satellites could observe Earth’s weather patterns. In 1966, the Environmental Science Services Administration (ESSA) Satellites I and II gave the United States its first global weather satellite system. In 1972, NASA began the Landsat series with the launch of the Earth Resources Technology Satellite 1, which was later renamed Landsat 1 by NASA. Figure 6-2 illustrates an image derived from Landsat. Subsequently, U.S. governmental satellites, such as Landsat 7, are still gathering consistently calibrated imagery of the earth under the Earth Observation Satellite (EOSAT) program. Satellites of other governments (SPOT-France) and private satellites (GE, Digital Globe) have expanded the routine availability of imagery, and further enhanced the resolution of the data collected.

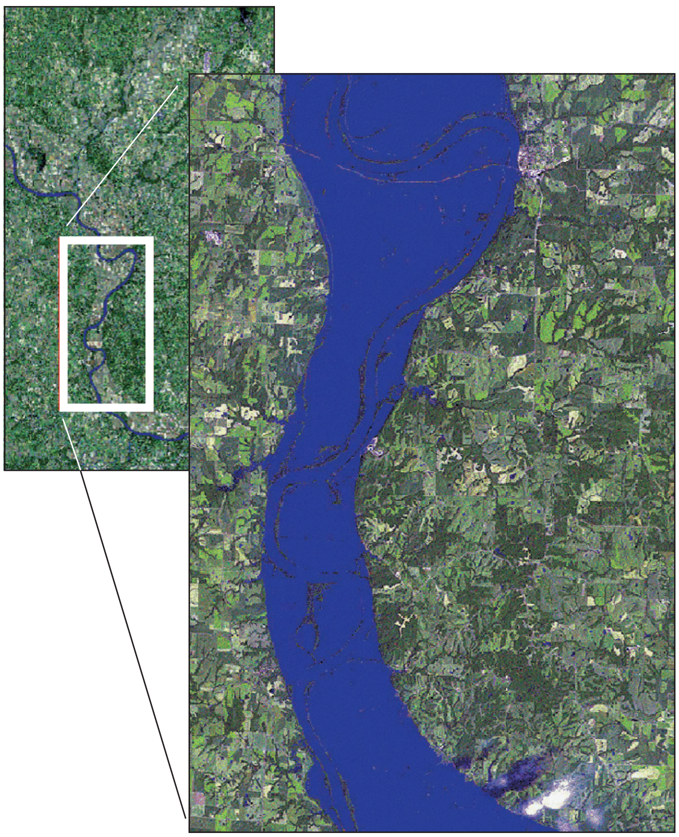


Figure 6-2. Landsat Thematic Mapper™ Images of the Missouri River Floodplain Near Glasgow, Missouri. (USGS, 1993).

6.1.2 CADD

Over the past quarter of a century, CADD has revolutionized the way in which engineers and architects perform their work. The basis for CADD was laid by Ivan Sutherland's 1963 Ph.D. thesis at Massachusetts Institute of Technology (MIT) titled, "Sketchpad: A Man-machine Graphical Communications System" (Sutherland, 2003). Sutherland used a lightpen to create engineering drawings directly on the Cathode Ray Tube (CRT). His thesis laid out virtually all of the graphical human interface issues. Sketchpad pioneered the concepts of graphical computing, including memory structures to store objects, rubber-banding of lines, the ability to zoom in and out on the display, and the ability to make perfect lines, corners, and joints. This was the first GUI long before the term was coined.

> The acronyms CAD, CADD, CAM, and CAE refer to "computer aided" methodologies used in various fields of engineering. CAD can stand for "computer aided drafting" or "computer aided design". CADD can mean "computer aided design and drafting" or "computer aided drafting and design". CAE refers to computer aided engineering and CAM refers to "computer aided manufacturing". The fields of CAD, CAM and CAE overlap and are frequently lumped into a single field of CAD/CAM/CAE.

In the late 1960s and early 1970s, several companies were founded that developed and commercialized the concepts of CADD. In the 1980s, Autodesk (maker of AutoCAD) and Bentley Systems (Microstation) were founded and led to the wider availability of CADD on personal computers. Later in that decade, Parametric Technology Corp. produced a 3-dimensional design system.

6.1.3 GIS

GIS represents computerized systems for the storage, retrieval, manipulation, analysis, and display of geographically referenced data (Mark, 1997a). Though the term GIS was first coined by Roger Tomlinson, director of the Canada GIS in the early 1960s, many of the concepts of GIS lie in the earlier fields of mapping and cartography. There are several fields and institutions that contributed to the GIS area in a non-linear manner over the past 40 years, resulting in the very powerful and widespread use of GIS today. Figure 6-3 shows the typical inputs and results of current GIS packages.

The development of the Geographic Base File/Dual Independent Map Encoding (GBF-DIME) files by the U.S. Census Bureau in the 1960s was the first large-scale use of digital mapping by the government. This system led to the production of the Census Topologically Integrated Geographic Encoding and Referencing (TIGER) files. Important geographic work was also being done at universities throughout the 1950s and 1960s. A grid-based mapping program called Synagraphic Mapping (SYMAP), developed at the Laboratory for Computer Graphics and Spatial Analysis at the Harvard Graduate School of Design in 1966, was widely distributed and served as a model for later systems. Output from SYMAP was on a line printer. A companion program called SYMVU allowed for mapping of topographic and other data using a pen plotter.

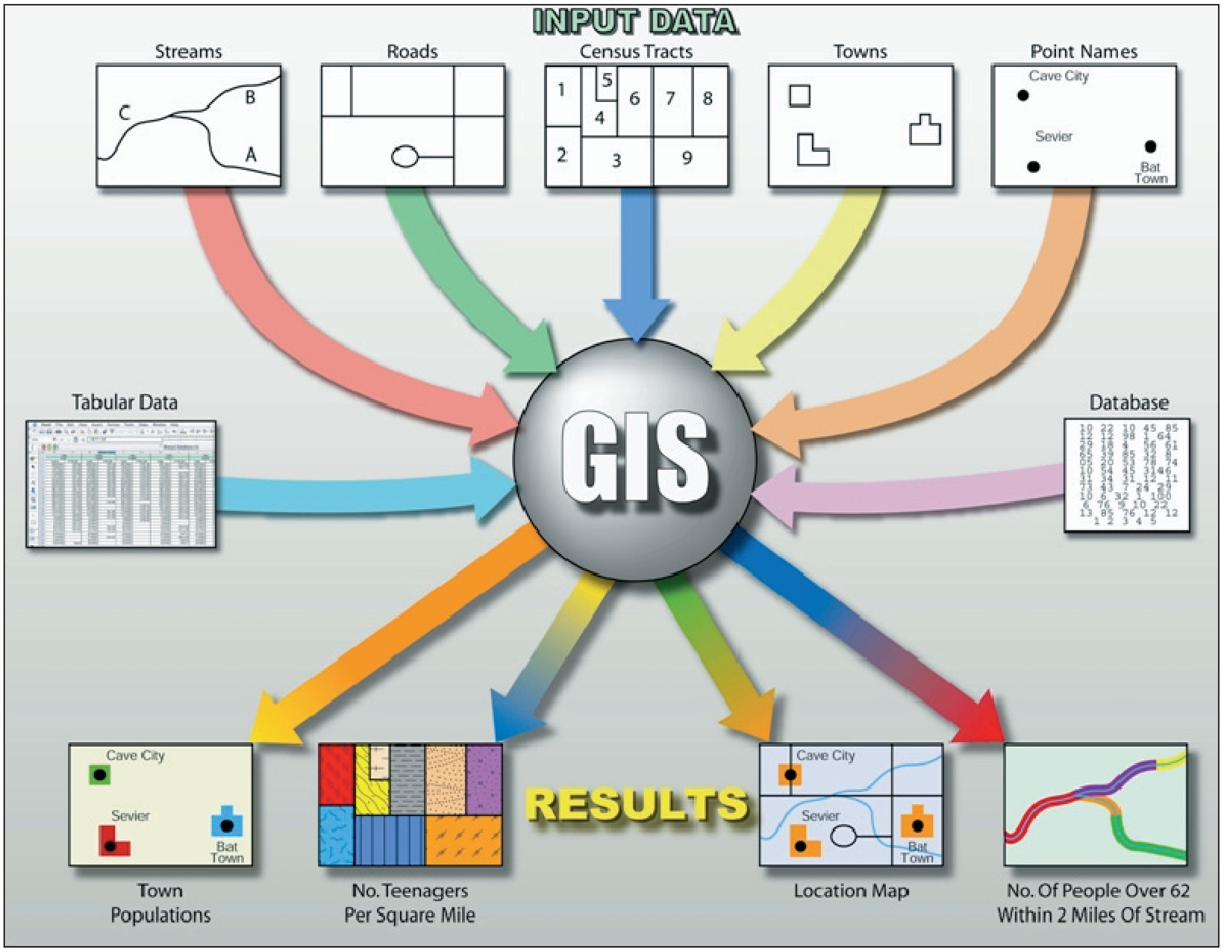


Figure 6-3. Typical Inputs and Results of Current GIS Packages.

From an application viewpoint, much of GIS modeling technology, as it is used today, is largely an outgrowth of planning approaches that are based on the work of Ian McHarg, as articulated in his book *Design with Nature* (McHarg, 1969). His manual methods involved overlaying a grid on the area to be studied, comparing and combining values for different types of attributes in a grid cell to determine the suitability of each grid cell for various uses. Attributes could include characteristics such as land slope and soil attributes.

He demonstrated this process in his book by creating maps of different attributes on transparencies with the darkness proportional to the degree to which that attribute would support a particular use. For example, significantly sloping land would be represented as dark areas because it is difficult to build under these circumstances. Then the reader could physically overlay the transparencies and select the lighter areas which were most appropriate for development. When computerized, this became the basis of the common overlay analysis of GIS technology, which served as the basic modeling technology of GIS for many years.

Another source for GIS technology was computerized photogrammetry and surveying. In the early 1960s, the desire to manipulate spatial data in a computer led to such well-known civil engineering programs as Coordinated Geometry (COGO), developed at MIT for calculating surveying analyses on coordinate data. At the same time, the concept of the Digital Terrain Model (DTM) was developed, in which the computer would be used to store a digital database representing the earth's surface (see the next subsection for more details on DTM).

In the past decade, development within the GIS community has been primarily associated with commercial enterprises that develop and market GIS software. Companies such as ESRI, Smallworld, Intergraph, Bentley, MapInfo, and AutoCAD dominate the GIS field today. Early important public domain or academic GIS packages such as Geographic Resources Analysis Support System (GRASS), developed by the U.S. Army Corps of Engineers and Clark University, have been largely replaced by the commercial software packages.

6.1.4 DEMs

DEMs and DTMs refer to representation of ground surfaces in a computer. Over the past 40 years, they have been used to support applications such as highway design, sewer design, hydrologic analysis, mining calculations, and military applications (such as part of a guidance system for missiles). Various methods utilizing regular/irregular grids and triangulated irregular networks (TIN) (Mark, 1997b) have been employed to provide efficient representations of surfaces. DEMs have become a regular feature of today's GIS packages. DEM databases are readily available for most of the U.S. from USGS and other sources. Figure 6-4 depicts a DTM output.

Figure 6-4. Digital Terrain Model of Mount St. Helens after Eruption in 1980 (R. Horne, 2004).

6.1.5 Database Management Systems

Spatial data is composed of two forms of information: geographic coordinates and attribute information. As an example, in order to represent a water main, geographic information describes the coordinates of the start and end of the pipe and any curves or bends in the pipe. Attribute information may include the pipe diameter, length, material, age, and other data of interest. Typically, attribute data are stored in a relational database system that may be part of a GIS or an asset management system. The Relational Database model for database design was invented by Dr. E.F. Cobb in 1969 and published in *Computer World* in 1985. A relational database system is composed of a series of tables that are related through keywords. This model is considered to be highly efficient and minimizes errors.

6.1.6 Facility Management

Facilities management (or asset management as it is frequently called today) pertains to use of computer database and mapping technology to store and manage information related to physical assets in a water system. In the 1980s, the term AM/FM was used to describe the automation of mapping and the management of facilities represented on those maps. This typically involved the integration of CAD technology and database management technology. In addition to the water industry, AM/FM was used by the electric and gas industry, telecommunications industry, and other industries that maintained physical networks. In the 1990s, the focus of facilities management both shifted and expanded to encompass a broader management of geographic spatially arrayed data that are used and maintained by the various types of utilities. This shift brought a closer interdependence to the GIS field and frequently this broader area is now referred to as AM/FM/GIS (Cesario, 1995) or even more broadly as Geospatial Information Technology.

6.2 GIS Principles

Understanding the basic principles behind geographic information systems is difficult because of the breadth of the field, the rapid change in technology, and the lack of standardization for terminology. This section provides a general overview of the most significant GIS principles.

6.2.1 GIS Features

A GIS is composed of a group of objects or features that have both a locational description and a description of their characteristics or attributes. For example, a water tank can be identified by its location in terms of latitude-longitude or other coordinate systems and its characteristics, such as diameter, height, and type of construction. Similarly, a pipe can be described by its route, diameter, length, material, and age. More importantly, these attributes can be stored, updated, and analyzed in a database over time.

Geographic features are stored in three general ways: vector, raster, or TIN. Though a geographic feature can frequently be stored in more than one way (e.g., as a vector or as raster), typically there is a preferred way to store each piece of information. Under the general area of vector representation, features can be stored as points, lines, or polygons. Figure 6-5 illustrates the three types of features in an example map of a water

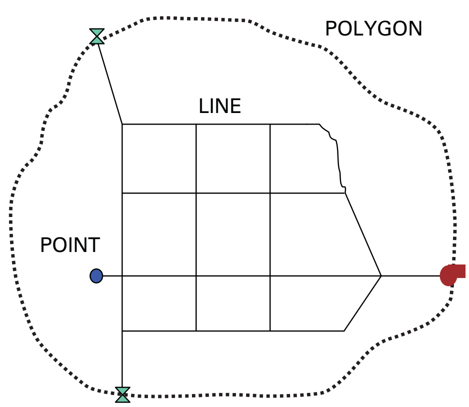


Figure 6-5. Map of Pressure Zone Showing Three Types of GIS Vector Data.

distribution system pressure zone. Points are geographically represented as a single set of coordinates in two or three dimensions. A line can represent either a single, straight-line segment, identified by the coordinates of the two end points, or a series of connected line segments to represent a curved line. A polygon is defined by a closed set of line segments and identifies the area contained within the defined outer boundary.

Raster data most commonly refers to a set of data that has been defined in terms of a regular square or rectangular grid system that is tied to a geographical coordinate system. Each grid cell can have one or more characteristics assigned to it. As an example of a raster data set, Figure 6-6 illustrates land use information derived from a satellite that is represented as a raster database. Other methodologies for storing raster data include scan lines and other regular grid cell configurations.

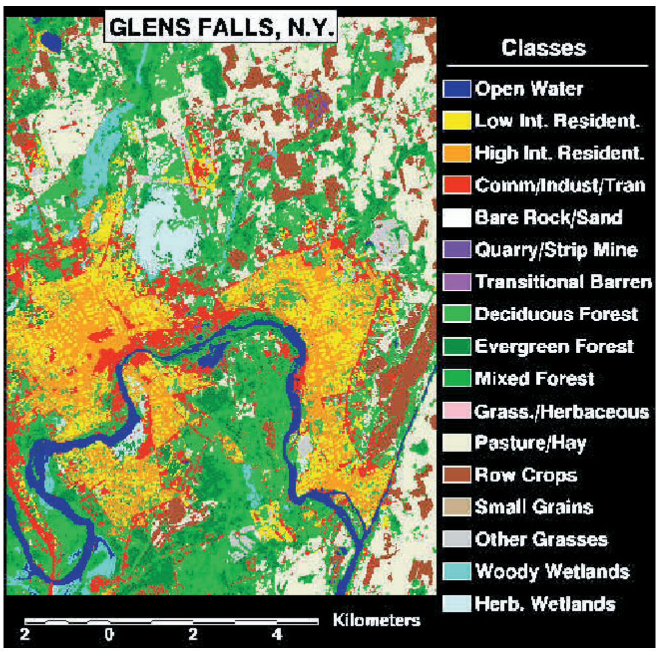


Figure 6-6. Regional Land Cover Characterization as a Raster Database (USGS, 1992).

The third general type of GIS feature is the TIN structure. As the name implies, a TIN structure is composed of a series of irregularly-sized triangular cells. TIN is most frequently used as a mechanism for storing topographical information, though it can also be used to store other discrete or continuous spatial data fields. The applicability of the TIN structure for storing topographical information lies in the simple geometric axiom that three points define a plane, as illustrated in Figure 6-7. As shown, a continuous surface can be represented by a faceted set of triangles with the sides of triangles representing topographical elements such as streams, ridges, and drainage divides (Grayman et al., 1975).

The resolution and accuracy of a TIN database generally depends upon the size of the triangles relative to the degree of detail in the surface being represented. Various mathematical techniques can be used to construct a TIN representation with the most commonly used method for constructing a TIN from a series of points known as Delaunay triangulation (named after a Russian mathematician who invented the procedure in 1934).

Within a GIS, features are organized as separate layers in a manner analogous to the original concept developed by Ian McHarg over 35 years ago. When viewing GIS data, layers can be turned on or off or moved forward or backward in order to better understand or view the spatial relationships.

6.2.2 Topology

An important characteristic of GIS is the concept of topology. Topology may be described as the locational interrelationship between features. Terms such as adjacency, intersection, and connectivity are all topological characteristics that describe how individual features interact. When we look at a map, our eyes and mind construct the topological linkages between features. Observations such as the Mississippi River forming the boundary between Illinois and Missouri, the Monongahela River and Allegheny River intersecting to form the Ohio River, or that a highway intersection between Interstate 95 and Interstate 10 is completely contained within the state of Florida are all statements of topology. GIS

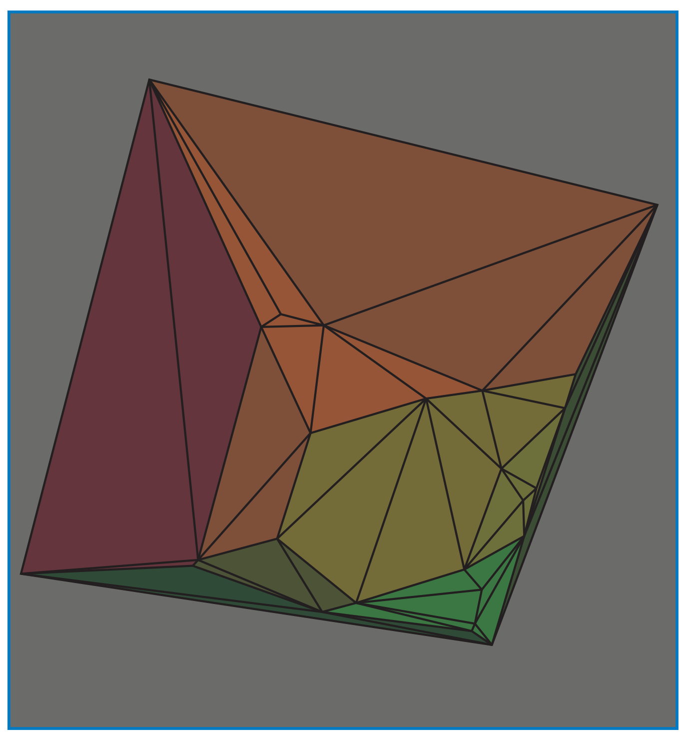


Figure 6-7. Triangulation of Elevation (Z) Data.

constructs the topological relationship between individual features, and this capability is used in various analyses and modeling tasks.

6.2.3 Map Projections, Datum, and Coordinate Systems

Map projections, datum, and coordinate systems provide the mechanism for establishing a unique geographic location for a point on the earth's surface. As scientists have known for centuries (or even millennia), the earth is approximately a spheroid, or in reality, an ellipsoid. However, when we are viewing maps or utilizing spatial data in a GIS, the earth is represented as a planar (flat) system. For maps or plans covering smaller areas, the distortion introduced by ignoring the curvature in the earth's surface is generally insignificant. However, for maps and plans covering larger areas, this distortion would be unacceptable. With the wide availability of regional or national GIS databases, the necessity for accurately determining coordinates is paramount.

The mechanism for converting a location on the earth surface to a flat surface is performed using a map projection. A map projection is a mathematical relationship for performing this conversion. There are many projections or relationships that can be used to make this conversion. Some of the more commonly used projections include:

- State Plane Coordinates (SPC),
- Universal Transverse Mercator (UTM),
- Albers Equal Area,
- Lambert Conformal Conic, and
- Space Oblique Mercator.

In each projection, the earth is divided into a series of zones. A best-fit, separate, planar coordinate system is established for each zone. When examining an area that straddles multiple zones, such as use of the SPC system with a metropolitan area that is in multiple states, coordinate conversions are needed in order to view the entire area in a single, consistent coordinate system.

A final important issue in understanding projections and coordinate systems is the concept of a datum. Because of the complexity of the shape of the earth and the inability to exactly describe it mathematically, the earth has been historically modeled by a best-fit ellipsoid. The parameters of that ellipsoid are defined by key datum points located on or in the earth. For many years, the North American Datum, developed in 1927 (NAD27) that uses a point on the earth's surface in Meade's Ranch, Kansas, as an anchor, was the major standard. Most projections for North America used this datum.

With improved mathematics and measurements of the earth, the North American Datum of 1983 (NAD83) was developed with a datum located within the earth. This has become the new standard for projections. As a result, a point having a particular coordinate using the NAD27 datum may be shifted by tens or hundreds of feet from a point with the same coordinate using the NAD83 datum. If this is not properly accounted for in a GIS system, a map that used the NAD27 datum would not properly overlay on a map using the NAD83 datum. There are several GIS utilities available that will properly convert datasets from one projection and datum into another, as well as some newer GIS programs that re-project datasets with different coordinate systems "on-the-fly." In either case, it is the responsibility of the GIS user to know the projection and datum associated with each data source and to make the appropriate definition.

6.2.4 GIS Database Design

GIS concepts and software provide an opportunity and a platform for utilizing spatial data. However, in order to effectively store, analyze, and display the data, they must be arranged in an organized manner. Factors that affect database design include: the goals of the GIS implementation, the short- and long-term plans for the GIS, the type and number of users for the particular application, any existing industry-wide standards, and other application-specific factors. Zeiler (1999) discusses the issues associated with database design. Various industry groups are attempting to define generic data structures for a particular industry (such as the water utility industry) in order to facilitate data transfer and common usage of a GIS in that industry (Grise et al., 2000).

6.2.5 Management of GIS

In the early days of GIS development, the GIS was typically developed, managed, and used by a central core of a few people at a governmental or private organization. With the growth and acceptance of GIS, there are now frequently many GIS stations using a specific GIS at an agency, utility or consortium of utilities. Management of such a system and controls on the manner in which changes in the GIS are made are very important issues.

GIS installations may be classified as a personal (or local) system or an enterprise system. In a personal system, the GIS is used and managed by an individual or a small group. On the other hand, an enterprise system may be used by dozens of users distributed throughout an agency and many locations. Though many of the management issues may be similar in these two scales of operation, the enterprise system presents a more challenging situation in terms of managing the system. Issues such as personnel (user) assignment for changing data or backing up the system and interconnectivity between stations and users must be carefully spelled out in order to insure the integrity of the system.

A common management model for large systems involving multiple users and locations is the combination of a central enterprise GIS with multiple local GIS installations. A central core of managers, who control any modifications to the database and maintain its integrity, maintains the central system. Local stations can either access the enterprise system on a read-only basis or can download copies of all or part of the database for their local use and modification. Specific protocols are then used if changes made at the local level are to be incorporated into the enterprise system. These protocols may include assigned responsibility at the local level to selected layers within the enterprise system. This form of management is relatively common with a county-wide or multi-utility agency managing an enterprise GIS and a water utility maintaining responsibility for the water system layers within the GIS.

6.3 Geospatial Data Management in the Water Industry

Because of the spatial extent and nature of water supply systems, management of geospatial data is an important task. This is accomplished through a series of systems under the overall umbrella of SDMS used to collect, store, and employ these spatial data. In some cases, these various systems are integrated; in other cases, they are independent systems.

6.3.1 CADD

CADD systems have long served as the basis for designing water distribution systems and facilities and for managing maps of the water system. Most utilities and consulting engineers use commercial packages such as AutoCAD, Intergraph or MicroStation. The CADD system may be organized around a collection of maps or plans with a local coordinate system for each plan or may utilize a regional coordinate system such as SPC. Many water utilities use water distribution system models that are integrated with CADD packages.

6.3.2 GIS

GIS has made significant inroads in supplementing or replacing CADD packages at many water utilities. GIS capabilities to store, access and map data are leading to increased usage of GIS in areas such as planning, facilities management, and management of customer and water quality data. Some water utilities share a GIS database with other entities, such as city or county governments, and other utilities, such as gas, electric, and telephone. At many utilities, GIS technology has also subsumed the capabilities that were formerly classified as AM/FM systems. Similarly, GIS systems may include an LIS as a means of storing land property, parcel, and ownership information and geographic descriptions. DEMs are also a regular feature in GIS packages. They provide a mechanism for storing topographical information. In the past few years, integration of GIS with water distribution system models has been a significant area of research and development in the water industry.

6.3.3 CIS

CIS provides a mechanism for storing and using information on water consumption by customers. The geographic component in a CIS is an address and/or a geographic coordinate. AMR systems facilitate collection of consumption data that can be stored in databases. Standard GIS "address matching" capabilities facilitate conversion of addresses to geographic coordinates. A geographically enabled CIS provides an excellent mechanism for automatically recording current consumption data to be used in water distribution system models.

6.3.4 SCADA

SCADA systems typically include capabilities to remotely access information on the state of the water system, to manually or automatically control components such as pumps and valves, and to store and display current or historical time-series data about system operation. A wide range of commercial SCADA hardware/software systems is available and can be tailored to the specific needs of the water utility. Each component that is referenced in a SCADA system can have a unique geographic identifier that can be used as a linkage to a GIS or other spatial data management systems. Research and development is underway related to integrating SCADA systems and hydraulic/water quality distribution system models so that these models can be used in real-time operation and emergencies.

6.3.5 LIMS

LIMS are computerized systems for managing samples in a laboratory. Such systems typically include a mechanism for storing, managing, displaying, and tracking samples. Since the origin of a sample must be identified both spatially and temporally, this information provides a means of associating LIMS data with other spatial database management systems.

6.3.6 Support Technology

Other technological advances related to spatial database management that are used by water utilities include GPS and RDBMS. GPS is a widely used technology in surveying and can be used for tagging field data with a geographic coordinate. RDBMS is a general methodology for efficiently storing information as a series of related 2-dimensional tables. Most modern database management systems associated with GIS, LIMS, and other systems utilize the RDBMS structure.

6.4 Integration of Geospatial Data Management and Modeling

The concept of integrating water distribution system modeling with geospatial database management systems has been evolving over the past quarter of a century and continues to be a major focus of development in the water industry today. Early water distribution system models were standalone entities. In the very early models, input was provided by punch cards and output was in the form of printed tabular information. This cumbersome I/O gave way to input via terminals in the 1980s and GUIs in the 1990s. The 1990s also saw the first commercial integration of water distribution system models with CADD followed by integration with GIS in the 2000s.

The basis for integrating water distribution system models with geospatial data can be traced back to an early study that interfaced a planning level sewer design model with a TIN-based GIS called ADAPT (Areal Design And Planning Tool) (Grayman et. al., 1975). This approach was called "geo-based modeling" and was subsequently applied to various other water engineering situations such as hydrologic modeling (Males and Gates, 1979). In these systems, a geo-based network representing sewer lines or streams was integrated with GIS elevation, land use, and soil data. This network directly interfaced with design and simulation models.

In the 1980s, the same geo-based modeling concept was applied to water distribution system analysis through a series of EPA research projects. The Water Supply Simulation Model (WSSM) integrated a geo-based, link-node system to several models including a hydraulic model, a steady-state water quality and cost allocation model, and various display and editing routines (Clark and Males, 1985). Subsequently, WSSM was expanded to include an interface to GIS files using AutoCAD. USGS digital line graph (DLG) files of road networks and DEMs were used within AutoCAD to create a detailed representation of the water distribution system. The resulting database was used to generate an input file for the Wadiso hydraulic model whose engine worked as a prototype for EPANET.

In the past 10 years, commercial vendors of network modeling software working in conjunction with CADD and GIS vendors have led the integration of modeling software and spatial database technology. In the mid 1990s, hydraulic/water quality models were built to operate within AutoCAD. More recently, commercial modeling systems have been released as a version that are fully integrated and operate within the GIS environment. Commercial products include WaterGEMS (Haestad Methods/Bentley Systems) and InfoWater (MWHSoft).

6.4.1 Model Integration Taxonomy

The term "integration" can refer to a wide range of capabilities related to use of network models in conjunction with a spatial database system. Shamsi (2001) provides a taxonomy of three levels for model-database integration. These are described as interchange, interface, and integration.

Interchange provides a mechanism for transferring data between a spatial database such as GIS and a model. With interchange, there is no direct linkage between the two systems. Rather, they are run separately and information is extracted from one system and stored in an intermediate file that is subsequently accessed by the other system. In the direction of database to model, information stored in a GIS is used to generate a complete or partial dataset that is used as input to the model. In the other direction, output from a model is used as input into a GIS in order to display the results of the model application. Most commercial water distribution modeling packages can interchange data with CADD and GIS platforms.

An interface involves a direct connection between the database and the model in order to transfer information in either direction. As is the case in interchange, the two systems still operate independently, but in this case, there is a direct linkage so that intermediate files are not necessary. Protocols and structures must be established and compatible within the two systems in order to support this interface. Current trends are directed towards open architecture in which information on the data structures for models and GIS are designed to data structures. For example, H2OMap and WaterCAD are standalone software packages which can directly interface with data in CADD and GIS platforms.

True integration is the most sophisticated of the three methods. Ideally, the two systems work together seamlessly as a single entity. In such integration, either the model can operate within the spatial database software or the spatial database capability can be part of the model. For example, WaterCAD and H2ONET software packages are integrated and operate within AutoCAD. InfoWater and WaterGEMS software packages are integrated to operate within ArcGIS.

6.4.2 Issues in Integrating GIS and Water Distribution System Models

As an evolving technology, there are still issues in truly integrating GIS technology with water distribution system models. These issues primarily revolve around the level of detail required in the two systems and the procedures for updating the model and the database.

In most cases, a water utility GIS is used for many purposes including mapping, facility management, planning, and modeling support. As a result, there may be a great deal of detail in the GIS. For example, it may include hydrants, shutoff valves, water meters and household connections, air release valves, and other appurtenances. On the other hand, typically, water distribution system models do not explicitly include many of these components. This is shown graphically in Figure 6-8. In this case, only junction nodes and pipes are included in the model representation. As a result, the GIS representation includes 17 links and 11 nodes, and the model representation includes 6 links and 3 nodes. The disparity between the two representations can increase by another order of magnitude if water meters and customer connections are included in the GIS. Various approaches are taken to deal with this situation.

- A very detailed model is constructed that includes all of the elements in the GIS. This solution can result in a very large model with an excessive number of nodes and links.
- The GIS representation goes through a consolidation (skeletonization) procedure to eliminate unneeded nodes and to aggregate the resulting links in order to construct the model representation. Though this results in a more appropriate model, it adds an intermediate step between the GIS and the model. Additionally, after the consolidation process, there is no longer a one-to-one correspondence between GIS and model features. This lack of correspondence leads to issues related to storing model output in the GIS and updating the model.
- Multiple representations are maintained within the GIS for different uses. The detailed representation is the complete, base case and used for facility management while the skeletonized version is used for modeling. This approach has the limitation that requires changes in information to be made in multiple databases.
- The basic link-node network (as used in the model) is maintained as the base case in the GIS and associates other components (such as hydrants) with links rather than structurally embedding them in the network.

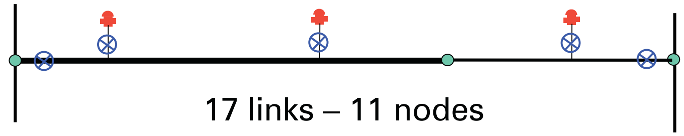


Figure 6-8a. Typical Representation of a Pipe Section in GIS.

Figure 6-8b. Typical Representation of a Pipe Section in a Network Model.

In any of the options described above, procedures for updating the GIS are essential. There are many issues associated with GIS updating such as authorization of specific users to make changes, nature of the changes as permanent or part of a 'what-if' modeling scenario, and frequency of replication of the two databases if separate model and GIS files are maintained. All of these should be carefully spelled out prior to designing and implementing a GIS.

6.5 Use of GIS in Water Utilities - Case Studies

This section presents the potential uses of GIS in the water utility industry. Case studies from the Las Vegas Valley Water District (LVVWD) and Denver Water are presented.

6.5.1 Use of GIS at LVVWD

Between 1989 and 2004, Las Vegas grew faster than any other metropolitan area in the U.S. As a result, LVVWD has more than doubled its service area population during this period. In 1989, the service area population was 558,000 and in 2004 it rose to 1,209,000, representing an increase of 651,000 people serviced by LVVWD (Jacobsen and Kamojjala, 2005). Figure 6-9 is a GIS representation of the LVVWD distribution system growth between 1989 and 2004.

To address a variety of issues related to this rapid growth, LVVWD integrated the functions of master planning, operational planning, and development review by integrating its GIS data with modeling, SCADA, and enterprise data (such as CIS, AM/FM and LIMS). Figure 6-10 presents the conceptual relationship model of these functions and potential integration benefits (Jacobsen et al., 2005).

During the process of integration, LVVWD developed a one-to-one relationship between the GIS spatial data and its network model (Jacobsen and Kamojjala, 2005). An example of this one-to-one relationship is

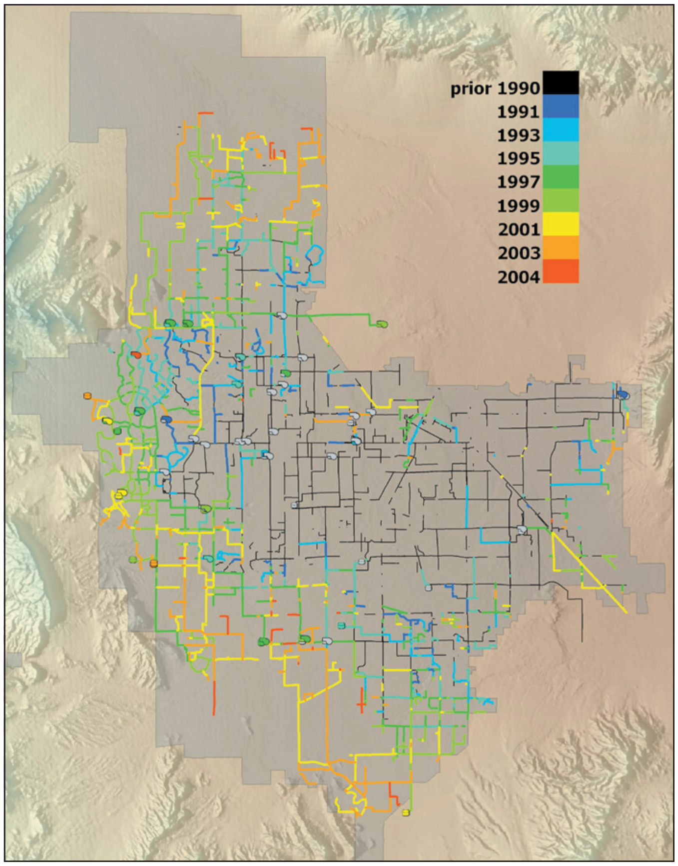


Figure 6-9. LVVWD Distribution System Growth.

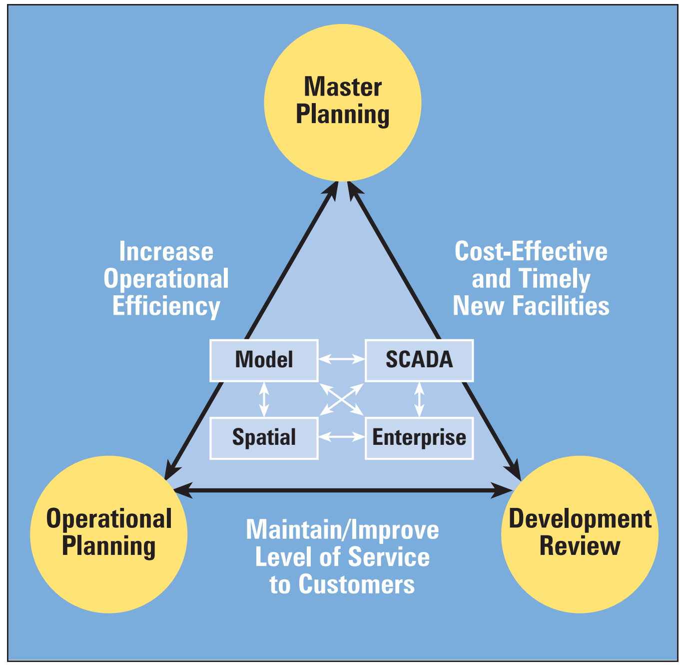


Figure 6-10. Conceptual Relationship Model for Integration.

shown in Figure 6-11. Depending on the size of the network, developing such a relationship and subsequent data integration has both advantages and disadvantages.

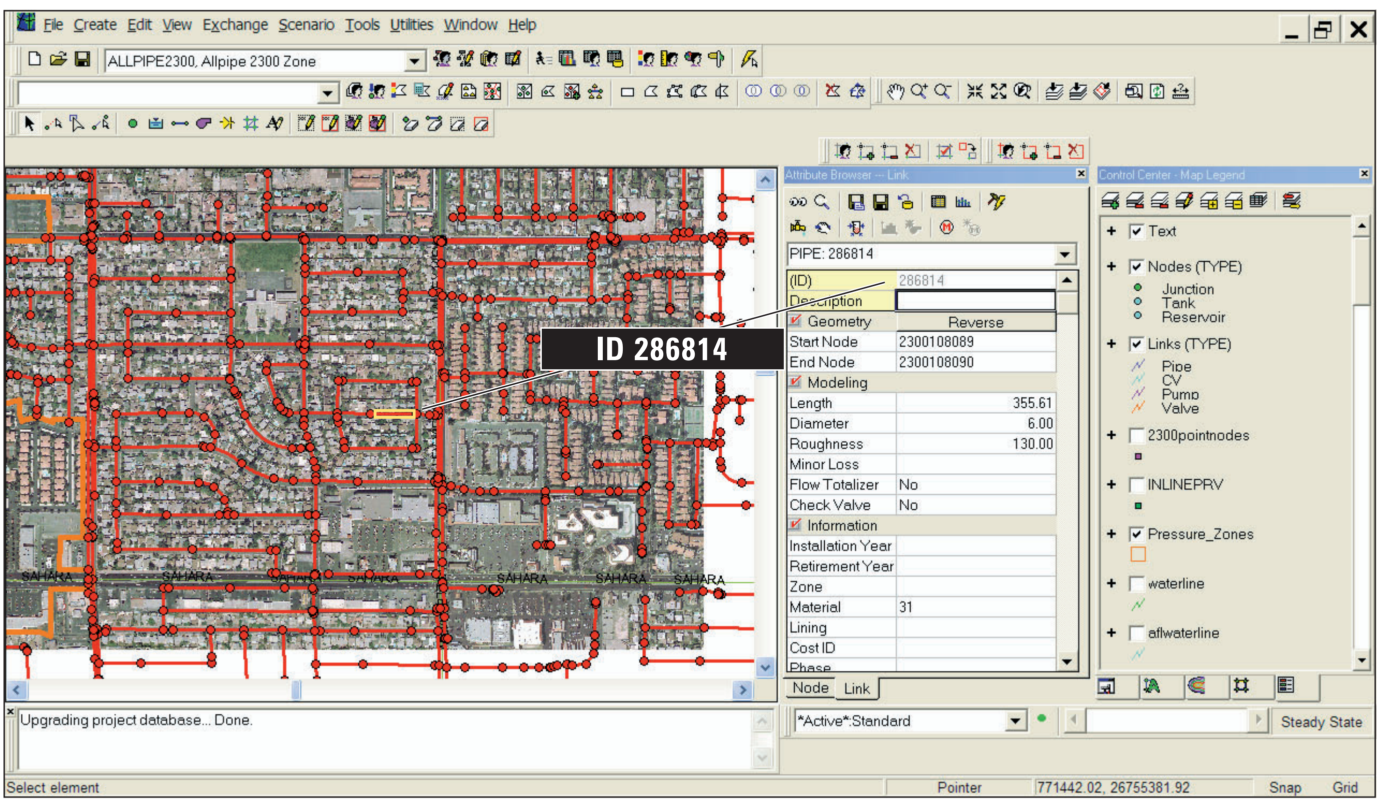


Figure 6-11. One-to-One Relationship Between GIS and Network Modeling Data.

The advantages include ease of search and retrieval with other data/applications, and ease of importation, development, and maintenance. For a large network model, the disadvantages include an increase in the runtime of the network model due to the addition of detailed components and relatively slow water quality simulations. To minimize this, LVVWD has taken an "all-pipes capable" approach where the distribution system is divided according to existing pressure zones and attached to an operational backbone network (skeletonized). Each of the zone models can be attached seamlessly to the backbone network for detailed hydraulic and water quality modeling. Examples of how GIS data are used by LVVWD on a day-to-day basis are presented below.

6.5.1.1 Pressure Complaint Resolution

Once a pressure complaint is received from a customer, the GIS data is searched for parcel and account information, together with modeled and measured pressure data in the vicinity of the complaint. Figure 6-12 shows an example search window. Depending on the results of the analyses, a crew may be dispatched to trace field pressures and abnormal conditions from the water supply source to the customer location, and to install hydrant pressure recorders to capture dynamic variations. Upon retrieval of field information, model and field results are compared to identify possible problems. Figures 6-13 and 6-14 present examples resulting from this search (Jacobsen and Kamojjala, 2005).

6.5.1.2 Water Main Break Analysis

During a water main break, it is critical to quickly identify the distribution system valves that must be closed in order to minimize water loss, potential flooding, or possible contamination. Repairs must be performed quickly so that service can be resumed. Figure 6-15 illustrates the procedure for rapidly identifying the valves to be isolated utilizing the GIS and modeling tools. A list of affected customers is generated for appropriate notification (Figure 6-16). An analysis is performed to evaluate the impact on existing services. Figure 6-17 shows a comparison of pressures after a shutdown and identifies a lower-pressure area after the shutdown. Response to emergencies, such as main breaks, can be provided quickly and accurately using integrated GIS tools (Jacobsen and Kamojjala, 2005).

6.5.2 Geo-coding for Demand Forecasting and Allocation at Denver Water

Between January 1997 and December 2000, Denver Water conducted a treated water study to evaluate the transmission, pumping and storage system for capacity and the need for new facilities for capital planning and operations. Denver Water made extensive use of GIS tools for demand forecasting and

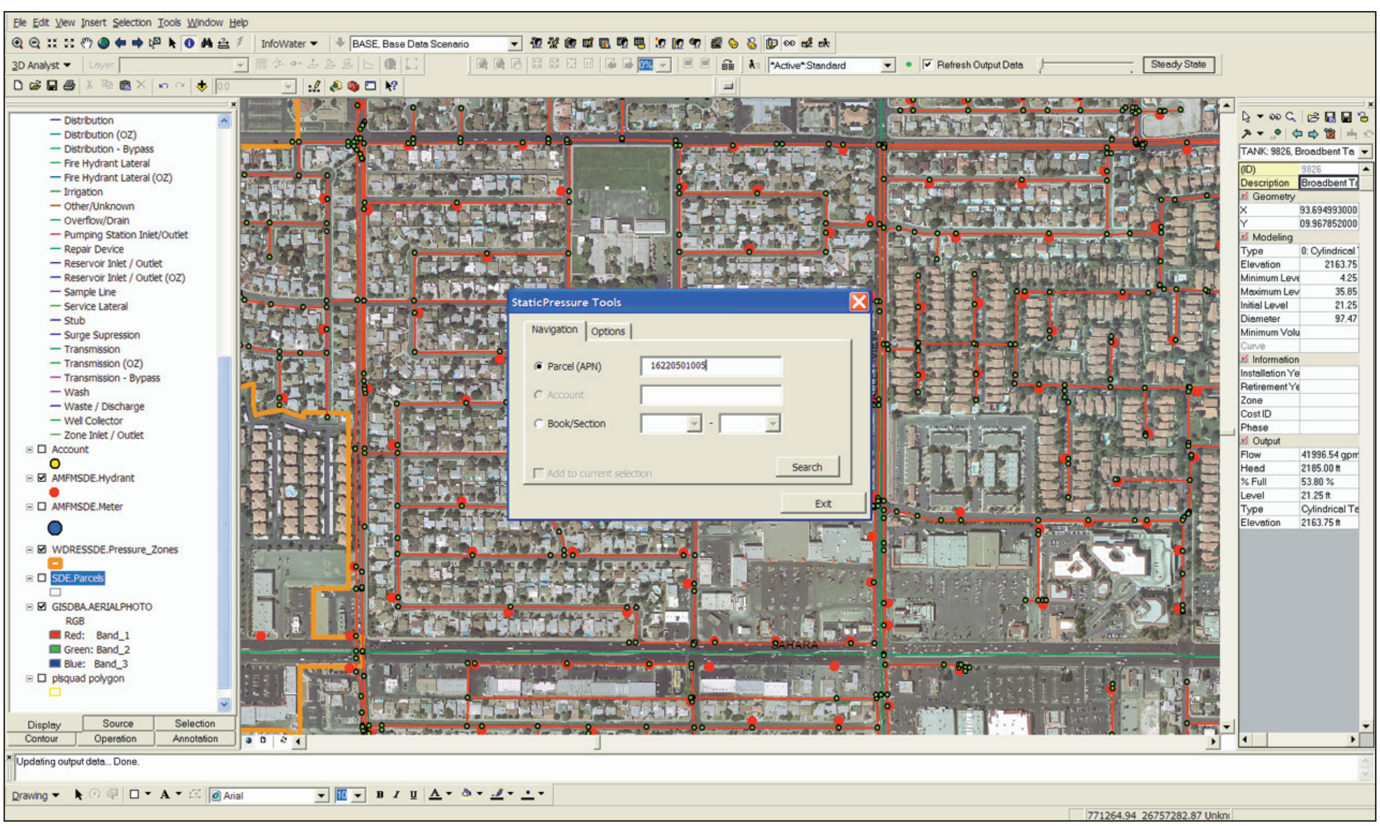


Figure 6-12. Pressure Complaint Resolution – GIS Parcel/Account Search Window.

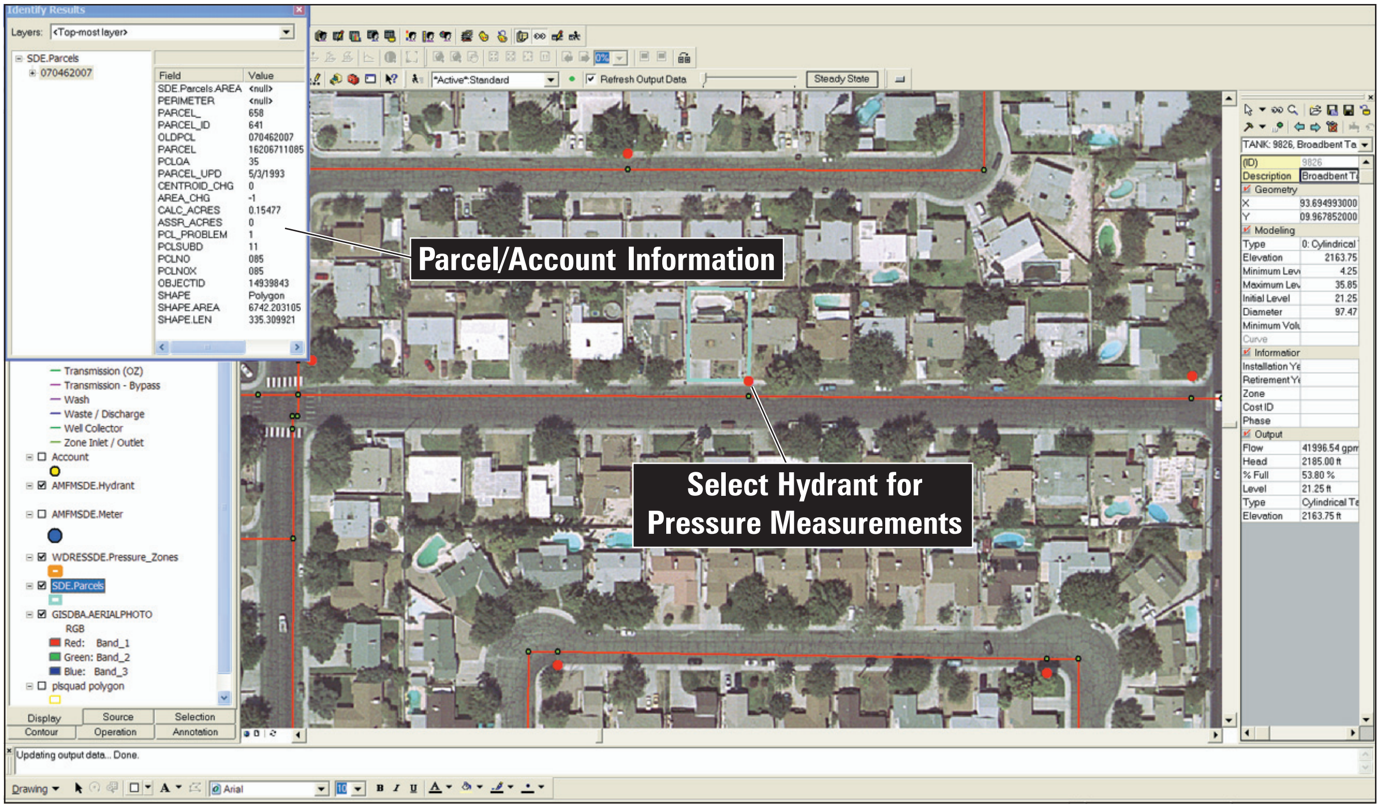


Figure 6-13. Pressure Complaint Resolution – Parcel and Hydrant Location.

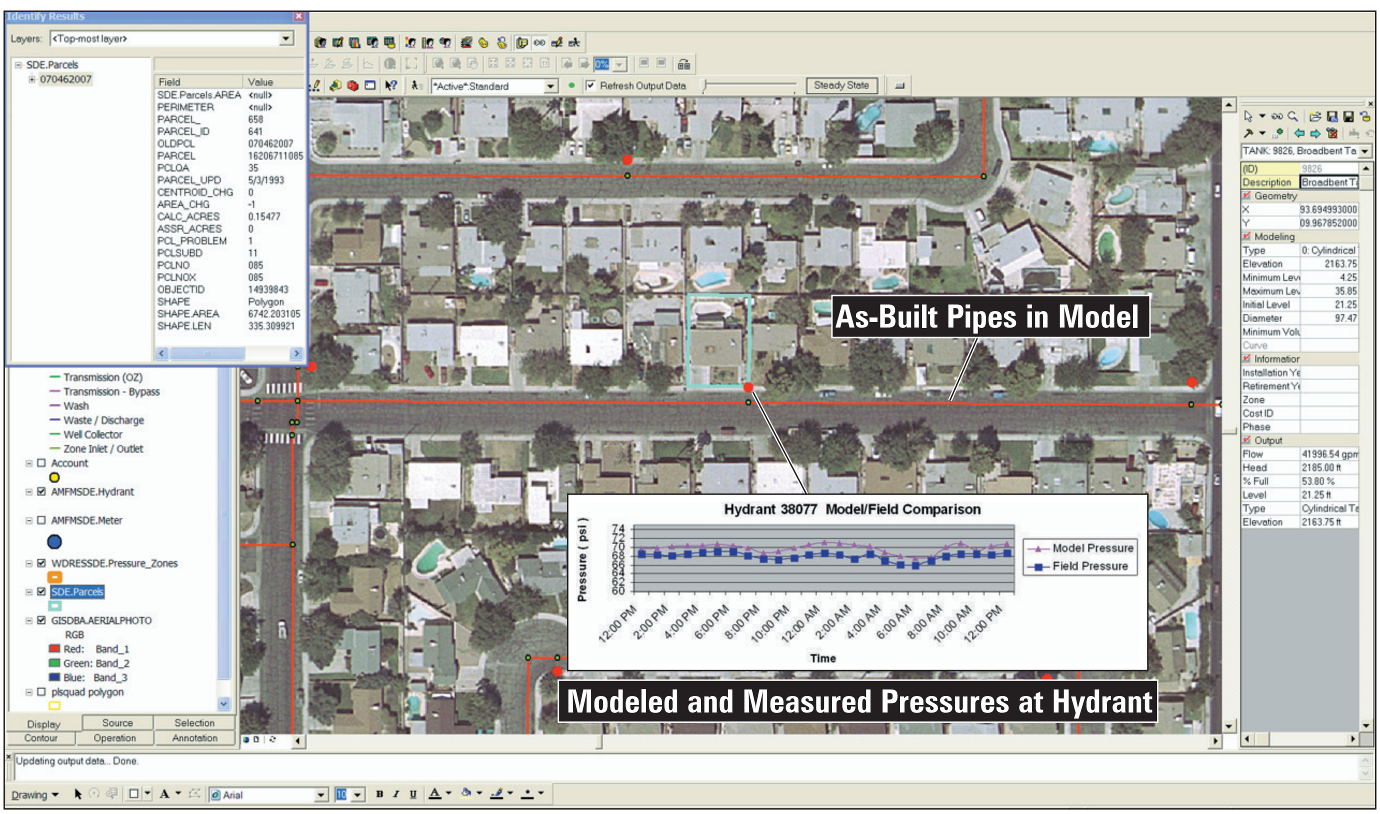
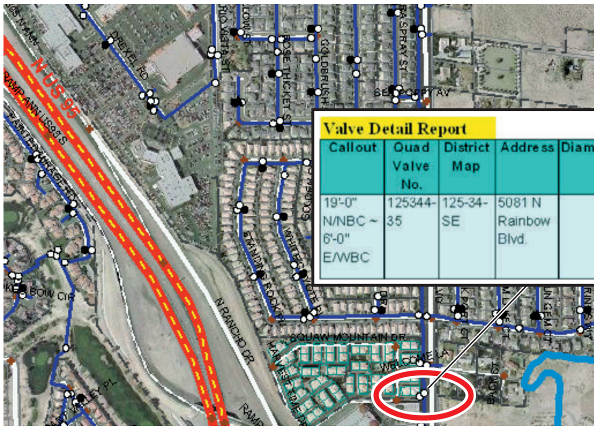


Figure 6-14. Pressure Complaint Resolution – Model and Field Pressure Comparison.



Valve Detail Report

Callout	Quad Valve No.	District Map	Address	Diameter	Drawing	Normal Position	Current Position	Status	Valve Type	Jurisdiction	QA Map	Install date	Critical Service ?	Vault	Manufacturer
19'-0" N/NBC ~ 6'-0" E/WBC	125344-35	125-34-SE	5081 N Rainbow Blvd.	16	XC-4462-006	O	O	Existing / Active	Butterfly West	County - Clark	Survey Grade GPS Data	4/19/95	No	No Vault	Mueller

Emergency Work Order

Task: Close in-line valve

Date: 11/11/1234

Valve No. 125344-35

Address: 5081 N Rainbow Blvd.

Drawing: XC-4462-006 (attached)

ID Feature: 353622

Size: 16-inch

Vault: No vault

Comments:
Urgent task and require completion by 5:30PM on 11/11/1234.

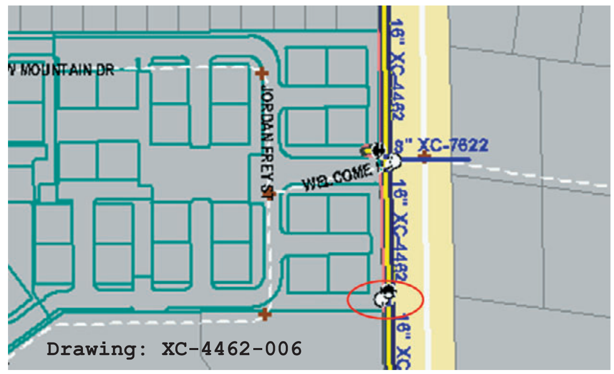


Figure 6-15. Water Main Break Analysis – Valve Isolation.

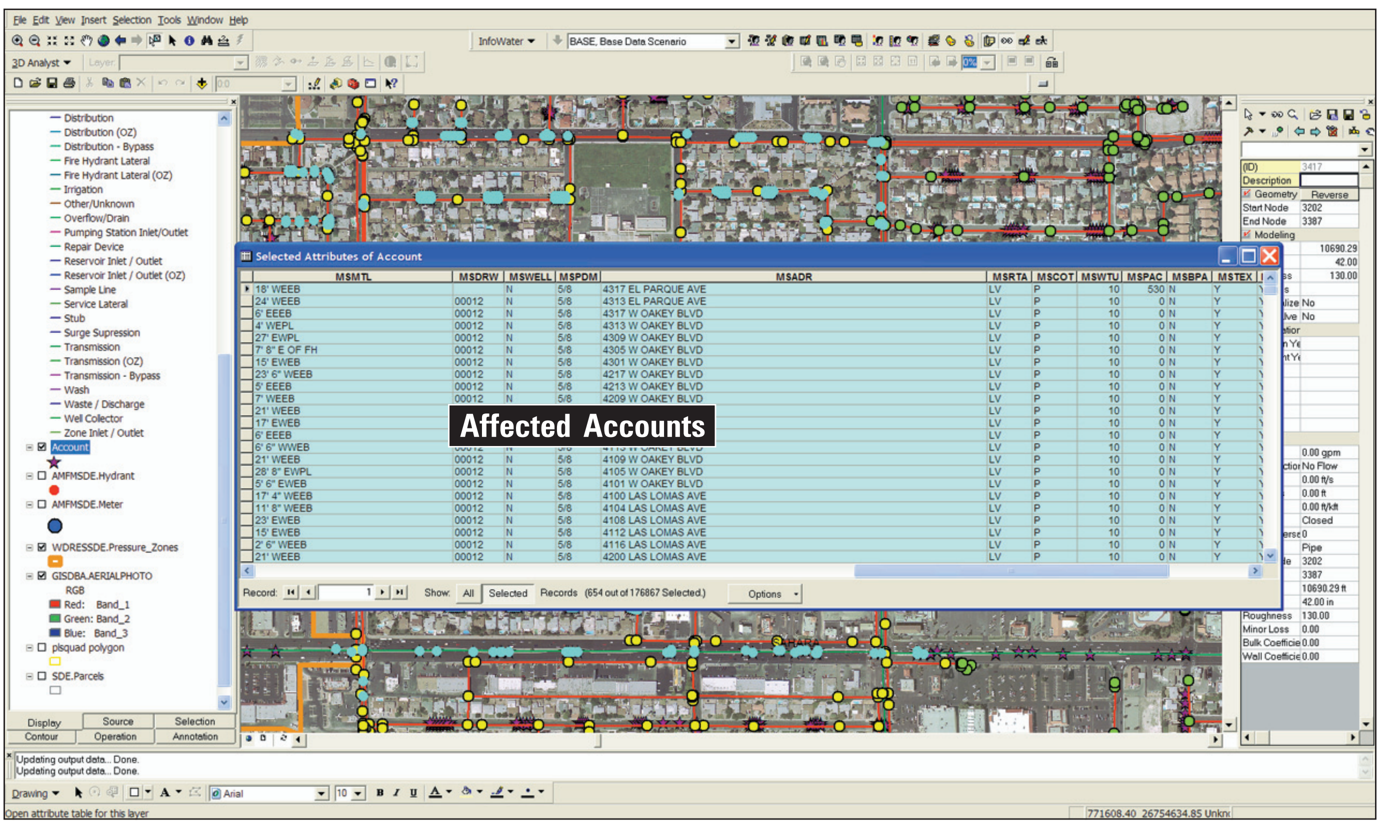
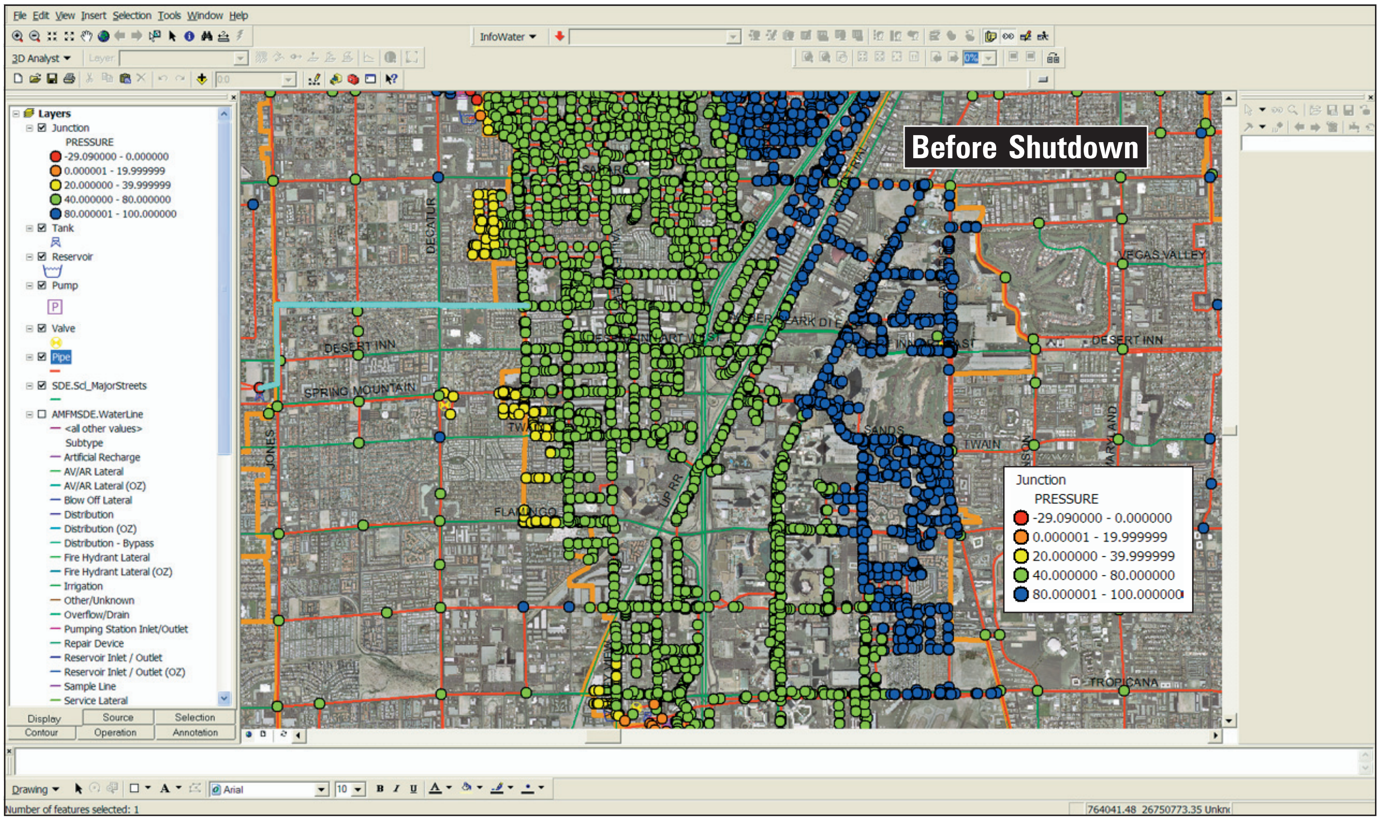


Figure 6-16. Water Main Break Analysis – Impacted Customer List.



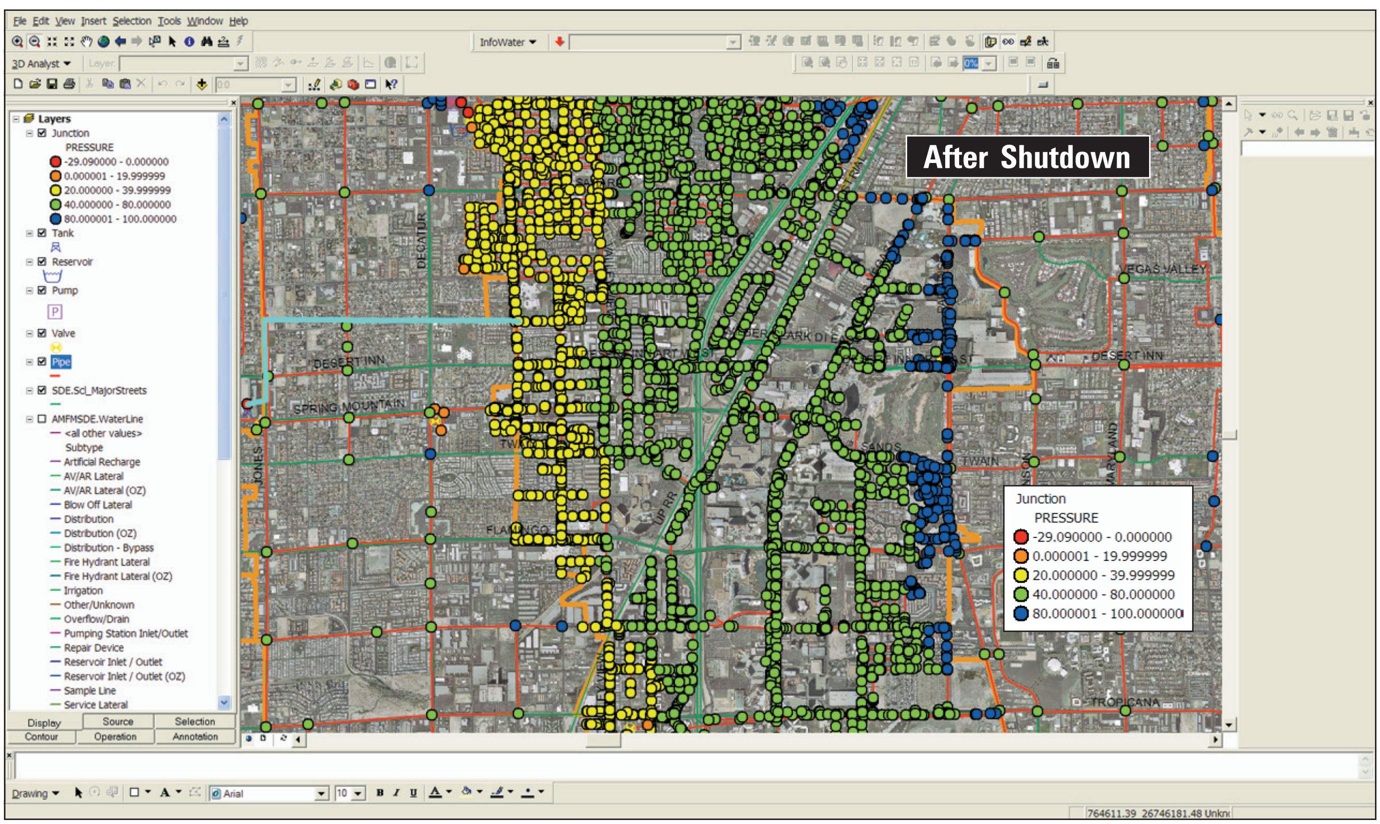


Figure 6-17. Water Main Break Analysis – Comparison of Junction Pressures - Before and After Shutdown.

allocation as part of this analysis. Specifically, existing demands were spatially allocated using address geo-coding (Strasser et al., 2000).

Denver Water has been a completely metered system since 1992. Historic consumption data is available from 1993 to the present. Consumption data was extracted from the billing system, and imported into an MS Access database for use in various tasks including the treated water study. The extracted data included both customer address and customer class information (e.g., single family, multi-family, commercial, industrial, and public). The customer class information allowed the demand information to be aggregated by customer class. Because the service area was large (over 250 square miles), it was important to identify the location of the consumption demand points. This is where the use of GIS became very important (Strasser et al., 2000).

Denver Water used the "address geo-coding" feature available within ArcInfo which allowed for each customer or demand point to be identified on a base map. Using this process, a dot is placed on the base map representing each customer that could be positively geo-coded. The match rate was over 93 percent. Those accounts that could not be geo-coded, mostly large accounts representing master meter accounts, and wholesale customers, were entered in the system manually. Results from the geo-coding process for one pressure zone are illustrated in Figure 6-18. A quality control check was performed on the results of this geo-coding process by reconciling consumer demands with Denver Water's annual statistical report (Strasser et al., 2000).

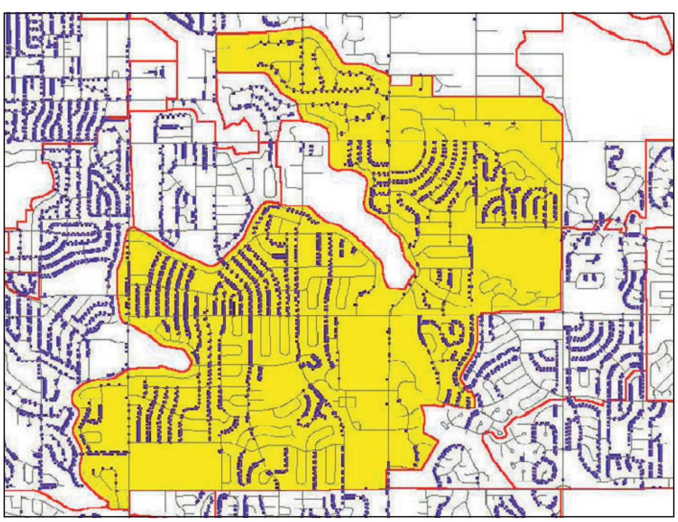


Figure 6-18. GIS Geo-coding - Metered Sales Demand Allocation Procedure.

6.6 Summary

Use and management of geospatial data is an important aspect of the design and operation of water systems. This can be accomplished through a range of systems under the overall umbrella of SDMS utilized to collect, store, and use the spatial data. This umbrella covers not only the broad topics of GIS and CADD that are widely recognized as geospatial data systems, but also systems such as SCADA and LIMS that have a spatial component associated with all data.

The area of spatial database management is continuing to evolve within the water industry. Just as the capabilities of the various individual components within the SDMS umbrella continue to expand, the integration of the various systems is an active area of development. Water distribution system analysis is a significant beneficiary of these improvements and integration. As a result, models can be built more quickly and in greater detail. Information on facilities and demands can be routinely updated. The results of a model application can be rapidly displayed and viewed along with other spatial data. The prospect of real-time application of models to assist in system operation under routine conditions or under emergency conditions is getting closer.

References

Clark, R.M., and R.M. Males. "Simulating Cost and Quality in Water Distribution." *Journal Water Resources Planning and Management*, ASCE. 111(4):454-466. 1985.

Cesario, A.L. *Modeling, Analysis, and Design of Water Distribution Systems.* AWWA, Denver, CO. 1995.

Grayman, W.M., R.M. Males, W.E. Gates, and A.W. Hadder. "LandBased Modeling System for Water Quality Management Studies." *Journal of Hydraulics,* ASCE, HY5. May 1975.

Grise, S., E. Idolyantes, E. Brinton, B. Booth, and M. Zeiler. "Water Utilities ArcGIS Data Models." ESRI Press, Redlands, CA. 2000.

Horne, R. "Gallery of 3DEM Terrain Images." http://www.visualizationsoftware.com/3dem/gallery.html. 2004.

Jacobsen L., and S. Kamojjala. "Full System Models and GIS Integration." AWWA Annual Conference and Exposition, San Francisco, June 2005.

Jacobsen L., S. Kamojjala, and M. Fang. "Integrating Hydraulic and Water Quality Models with Other Utility Systems: A Case Study." AWWA Information Management and Technology Conference, Denver, April 2005.

Males, R.M., and W.E. Gates. "ADAPT: A Digital Terrain Model-Based Geographic Information System." Symposium, Laboratory for Computer Graphics & Spatial Analysis, Harvard University, Cambridge, MA. 1979.

Mark, D.M., N. Chrisman, A. U. Frank, P. H. McHaffie, and J. Pickles. "The GIS History Project." UCGIS Summer Assembly, Bar Harbor, ME. http://www.geog.buffalo.edu/ncgia/gishist/bar_harbor.html. 1997a

Mark, D.M. "The History of Geographic Information Systems: Invention and Re-Invention of Triangulated Irregular Networks (TINs)." Proceedings, GIS/LIS '97. http://www.geog.buffalo.edu/ncgia/gishist/GISLIS97.html. 1997b.

McHarg, I.L. *Design with Nature.* Published for the American Museum of Natural History by the Natural History Press. 1969.

NASA. "Earth Observatory – Remote Sensing." http://earthobservatory.nasa.gov/Library/RemoteSensing/remote.html. Undated.

O'Connor, J.J., and E. F. Robertson. "The History of Maps." http://geogdata.csun.edu/geogcourses/history_of_maps.html. Undated.

Queensland Government. "Maps and Compasses – The History of Surveying." http://education.qld.gov.au/curriculum/area/maths/compass/html/surveying/suhis.html. Undated.

Radio Shack. "A Guide to the Global Positioning System (GPS)." http://support.radioshack.com/support_tutorials/gps/gps_main.htm. 2004.

Shamsi, U.M. "GIS and Modeling Integration." *CE News*, 13(6). 2001.

Strasser, A., N. Diallo, and E.J. Koval. "Development & Calibration of Denver Water's Hydraulic Models for a Treated Water Study." AWWA National Conference, Denver, Colorado, 2000.

Sutherland, I.E. "Sketchpad: A Man-machine Graphical Communications System." Reprinted version of original 1963 thesis issued as a report by Cambridge University. UCAM-CL-TR574. Cambridge, UK. http://www.cl.cam.ac.uk/TechReports/UCAM-CL-TR-574.pdf . 2003.

USGS. "National Aerial Photography Program (NAPP)". http://edc.usgs.gov/guides/napp.html. Undated.

USGS. "Scientific Assessment and Strategy Team (SAST) Study of the Moberly Quadrangle." http://edc.usgs.gov/sast/moberly.html. 1993.

USGS. "National Land Cover Characterization (NLCD) Database Program." http://landcover.usgs.gov/images/glensfalls_web.jpg. 1992.

Whitfield, P. *The Image of the World: 20 Centuries of World Maps.* Pomegranate Artbooks, San Francisco, CA, 144 p. 1994.

Zeiler, M. *Modeling Our World.* ESRI Press, Redlands, CA. 1999.

Chapter 7 Real-World Applications – Planning, Analysis, and Modeling Case Studies

The previous chapters of this reference guide showcased several tools for analyzing water quality in drinking water distribution systems. Some of these chapters also have relevant case studies that relate to the individual topic of discussion for that chapter. This chapter focuses on the broader application of multiple tools to analyze real-world situations. The types of applications presented here include: reconstruction of historical contamination events, analysis of waterborne outbreaks of infectious diseases, regulatory compliance, monitoring systems location, and security. Each real-world application is presented as an individual case study and the portions related to water quality and analysis have been highlighted along with some specifics on techniques used in the analysis.

7.1 Analysis of Waterborne Outbreak – Gideon, Missouri

This case study is focused on evaluating the distribution of microbiologically contaminated water in a distribution system. The supporting investigations for this case study were primarily sponsored by the EPA, CDC, and the State of Missouri.

Key Phrases to Characterize Case Study: water borne outbreak analysis, *salmonella*, tank contamination, hydraulic and water quality modeling, exposure modeling, contamination assessment, and flushing.

7.1.1 Gideon Case Study Overview

From November 1993 through January 1994, the Missouri Department of Health (MDOH) had identified 31 cases of laboratory-confirmed salmonellosis infections associated with a waterborne outbreak in Gideon, Missouri (Clark et al., 1996). The State Public Health Laboratories identified 21 of these isolates as dulcitol negative *Salmonella* serovar *Typhimurium*. *Salmonella* is a pathogenic bacterium that has been classified into several serotypes (common set of antigens). *Salmonella* serovar *Typhimurium* is among the most common *Salmonella* serovars causing salmonellosis in the U. S. Fifteen of the 31 laboratory culture-confirmed patients were hospitalized (including two patients hospitalized for other causes and who developed diarrhea while in the hospital). These 15 patients were admitted to 10 different hospitals. Seven nursing home residents exhibiting diarrheal illness died; four of these patients were culture confirmed (the other three were not cultured). Two of the patients had positive blood cultures. Interviews conducted by the MDOH during this period suggested that there were no food exposures common to a majority of the patients. However, all of the ill persons, including the culture-confirmed patients, had consumed municipal water which supported the association. The MDOH reported their suspicion to the Missouri Department of Natural Resources (MDNR).

7.1.2 The Gideon Water System Setup

The Gideon municipal water system was originally constructed in the mid-1930s and obtained water from two adjacent, 1,300 ft deep wells. The well waters were not disinfected at the time of the outbreak. After the outbreak emergency, chlorination was initiated, and later a permanent chlorination system was installed. The distribution system consisted primarily of small-diameter (2-, 4-, and 6-inch) unlined, steel and cast iron pipe. Tuberculation and corrosion were major problems in the distribution pipes. Raw water temperatures were unusually high for a groundwater supply system (58°F), because the system overlies a geologically active fault. Under low flow or static conditions, the water pressure was close to 50 psi. However, under high flow or flushing conditions the pressure dropped dramatically. These sharp pressure drops were evidence of major problems in the Gideon distribution system. The municipal system had two elevated tanks. One tank was a 50,000 gallon (gal) tank (referred to as small tank) and the other was a 100,000 gal tank (referred to as large tank).

Initially, another 100,000 gal privately owned tank was suspected to be the cause of the outbreak (as it was in a state of disrepair) and connected to the city water system. However, subsequent investigations revealed that this private tank was connected via a backflow prevention valve to the city water system that was later confirmed to be functional. Furthermore, the *Salmonella* found in a sample collected at a hydrant matched the serovar of the patient isolate when analyzed by the CDC laboratory (comparing deoxyribonucleic acid [DNA] fragments using pulse field gel electrophoresis). Although the samples from the private tank sediment also contained *Salmonella* serovar *Typhimurium* dulcitol negative organisms, the isolate did not provide an exact DNA match with the other two isolates. No *Salmonella* isolates were found elsewhere in the system. Therefore, the subsequent EPA field investigations and modeling efforts focused on the two municipal tanks as the source of contamination.

7.1.3 EPA Field Study

On January 14, 1994, an EPA field team, in conjunction with the CDC and the State of Missouri, initiated a field investigation that included a sanitary survey and microbiological analyses of samples collected on site. A system evaluation was also conducted in which EPANET was used to develop various scenarios to explain possible contaminant transport in the Gideon system. Prior to the Gideon outbreak, a similar waterborne disease outbreak in Cabool, Missouri, and subsequent advancements in water quality modeling firmly established the use of water quality models to analyze such events.

The key analysis was focused on a flushing program conducted earlier by the utility in response to taste and odor complaints. A sequential flushing program was conducted on November 10, 1993, involving all 50 hydrants in the system. The flushing program was started in the morning and continued through the entire day. Each hydrant was flushed for 15 minutes at an approximate rate of 750 gallons per minute (gpm). It was observed that the pump at one of the wells was operating at full capacity during the flushing program (approximately 12 hours), which would indicate that the municipal tanks were discharging during this period.

During the evaluation, it was hypothesized that the taste and odor problems may have resulted from a thermal inversion that had taken place due to a sharp temperature drop prior to the day of the complaint. If stagnant or contaminated water were floating on the top of a tank, a thermal inversion could have caused this water to be mixed throughout the tank and to be discharged into the system resulting in taste and odor complaints (Fennel et al., 1974). As a consequence, the utility initiated the aforementioned city-wide flushing program. Turbulence in the tank from the flushing program could have stirred up the tank sediments that were subsequently transported into the distribution system. It is likely that the bulk water and/or the sediments were contaminated with *Salmonella* serovar *Typhimurium*. During the EPA field visit, a large number of pigeons (bird droppings are known to contain *Salmonella*) were observed roosting on the roof of the 100,000 gal municipal tank.

7.1.4 Distribution System Evaluation

The EPA study team evaluated the effects of distribution system design and operation, demand, and hydraulic characteristics on the possible propagation of contaminants in the system. Given the evidence from the lab samples and the results from the valve inspection of the private tank, it was concluded that the most likely contamination source was bird droppings in the large municipal tank. Therefore, the analysis concentrated on propagation of water from

In 1991, a joint workshop sponsored by the EPA and AwwaRF recommended the application of water quality modeling techniques to evaluate waterborne disease outbreaks. The first opportunity to attempt this type of application arose as a result of an outbreak that occurred between December 15, 1989, and January 20, 1990, in Cabool, Missouri, population 2,090 (Geldreich et al., 1992). During the outbreak, residents and visitors to Cabool experienced 243 cases of diarrhea (85 bloody) and six deaths. The illness and deaths were attributed to the pathogenic agent *E. coli.* serotype O157:H7. At the time of the outbreak, the water source was untreated groundwater. Shortly after the outbreak was identified, EPA was invited to send a team to conduct a research study with the goal of determining the underlying cause of the outbreak.

Exceptionally cold weather prior to the outbreak contributed to two major water system line breaks and 43 water meter replacements throughout the city area. The sewage collection lines in Cabool were located (for the most part) away from the drinking water distribution lines but did cross or were near to water lines in several locations. At the time of the outbreak, stormwater drained via open ditches along the sides of the streets and roads. During heavy rainfalls, sewage was observed to overflow manhole covers, and to overflow streets in several locations, parking lots and residential foundations.

The Dynamic Water Quality Model (DWQM), developed by EPA, was applied to examine the movement of water and contaminants in the system (Grayman et al., 1988). Steady-state scenarios were examined, and a dynamic analysis of the movement of water and contaminants associated with meter replacement and the line breaks was conducted. Typical demand patterns were developed from available meter usage for each service connection, and it was found that the water demand was 65 percent of the average well production, indicating inaccurate meters, un-metered uses, and a high water loss in the system.

The modeling effort revealed the pattern of illness occurrence was consistent with water movement patterns in the distribution system assuming two water line breaks. It was concluded that some disturbance in the system, possibly the two line breaks or 43 meter replacements, allowed contamination to enter the water system. Analysis showed the simulated contaminant movement covered 85 percent of the infected population.

The application of DWQM proved to be a vital step in completing the analysis of the outbreak. The next opportunity to apply water modeling techniques occurred in 1994 as a result of a waterborne outbreak in Gideon, Missouri (Clark et al., 1996). In the intervening period, EPA had developed EPANET and Gideon provided an opportunity to test its application.

the large municipal tank in conjunction with the flushing program. Other possible sources of contamination, such as cross connections were also studied.

The system layout, demand information, pump characteristic curves, tank geometry, flushing program, and other information needed for the modeling effort were obtained from maps and demographic information and numerous discussions with consulting engineers and city and MDNR officials. EPANET was used to conduct the contaminant propagation study (Rossman et al., 1994).

The EPANET network model was calibrated by simulating flushing at the hydrants assuming a discharge of 750 gpm for 15 minutes. The “C” factors (pipe roughness – see Chapter 4) were adjusted until the head loss in the model matched head losses observed in the field. After the calibration, the hydraulic model was simulated for 48 hours. Thereafter, the flushing program was simulated starting at 8 a.m. on day 3, by sequentially imposing a 750 gpm demand on each hydrant for 15 minutes. Utilizing the TRACE option in EPANET, the percentages of water from both municipal tanks were calculated at each node over a period of 72 hours.

During the simulation of the flushing program, the pump at one of the wells was operated (as previously observed) at full capacity, which was over 800 gpm, and then reverted to cyclic operation. The simulation results showed that the tank elevation fluctuated for both municipal tanks, and both the tanks discharged during the flushing program. At the end of the flushing period, nearly 25 percent of the water from the large municipal tank passed through the small municipal tank where it was again discharged into the system. The model predicted dramatic pressure drops during the flushing program. Based on the information available, it was felt that these modeling results replicated the conditions that existed during the flushing program closely enough to provide a basis for an analysis of water movement in the system.

Data from the simulation study, the microbiological surveillance data, and the outbreak data were utilized to provide insight into the nature of both general contamination problems in the system and the outbreak itself. The water movement patterns showed the majority of the collected samples that were total coliform and fecal coliform (FC)-positive occurred at points within the zone of influence of the small and large tanks. During both the flushing program and for large parts of normal operation, these areas were predominately served by tank water, which confirmed the belief that the tanks are the source of the fecal contamination since there were positive FC samples prior to chlorination. Figure 7-1 shows the comparison of early confirmed cases of *Salmonella* positive sample versus the estimated distribution of tank water during the first six hours of the flushing program.

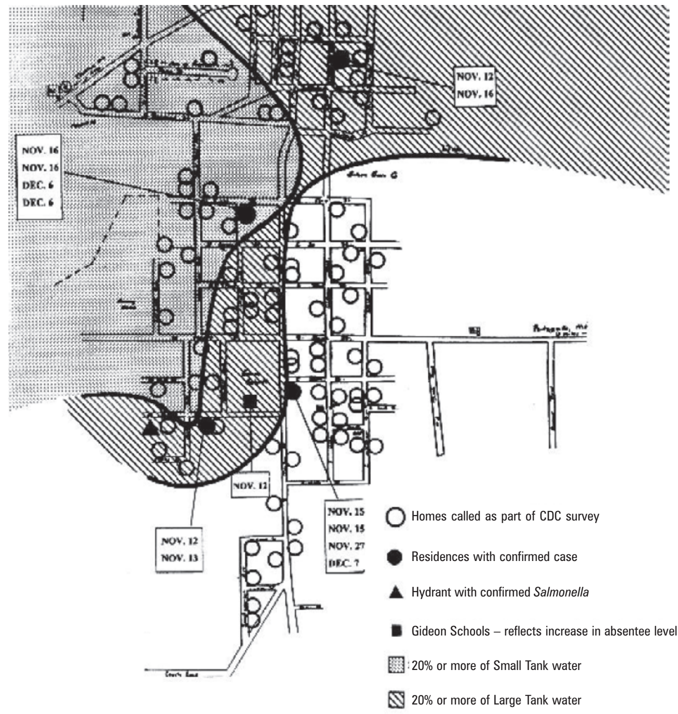


Figure 7-1. Comparison of Early Confirmed Cases of Salmonella Positive Sample Versus the Estimated Distribution of Tank Water During the First 6 Hours of the Flushing Program.

7.1.5 Case Study Summary and Conclusions

Data from the CDC survey of the outbreak, in combination with the EPANET simulated water movement, were utilized to establish the possible source of contamination. An overlay of the CDC data on the water movement simulations showed that the areas served by the small and large tanks (during the first six hours of the flushing period) coincided with the earliest recorded infectious cases. Furthermore, the earliest recorded cases and the positive *Salmonella* hydrant sample were found in the area that was primarily served by the large tank, but outside the small tank’s area of influence.

The investigators concluded that during the first six hours of the flushing period, the water that reached an infected resident and the Gideon School (the earliest reported infections) was almost totally from the large tank. Based on the results of the study, it appeared that the contamination had been occurring over a period of time, which is consistent with the possibility of bird contamination. It is likely that the contaminant was pulled through the system during the flushing program. The application of EPANET to the outbreak proved to be a vital part of the study.

7.2 Reconstructing Historical Contamination Events - Dover Township (Toms River), NJ

This case study is focused on evaluating the distribution of chemically contaminated source water in a distribution system. The supporting investigations for this case study were primarily sponsored by the ATSDR. The investigations involved several other organizations. The major contributors included the New Jersey Department of Health and Senior Services (NJDHSS), the Multimedia Environmental Simulations Laboratory at the Georgia Institute of Technology, EPA's National Risk Management Research Laboratory, and the U.S. Geological Survey.

Key Phrases to Characterize Case Study: historical reconstruction, hydraulic and water quality modeling, exposure modeling, contamination assessment, source tracing, source contribution, model calibration, sensitivity analysis, genetic algorithm.

7.2.1 Case Study Overview

In August 1995, responding to an evaluation requested by the ATSDR, the New Jersey Department of Health (now NJDHSS) determined that the childhood cancer incidence rate in Dover Township (and the Toms River section) was higher than expected for all malignant cancers combined (brain and central nervous system cancer, and leukemia, Berry, 1995). In March 1996, NJDHSS and ATSDR developed a Public Health Response Plan (PHRP) describing actions these agencies would take to investigate the unexpected increase in childhood cancers and environmental concerns in Dover Township (NJDHSS and ATSDR, 1996). The PHRP included a list of several evaluations. One of the key evaluations was to identify potential environmental exposure pathways relative to two National Priorities List (NPL) sites in Dover Township (Figure 7-2) – Ciba-Geigy and Reich Farm. Figure 7-2 also shows the two public water supply well fields (Parkway and Holly) that were identified as potential routes of exposure. These well fields are not only located in the vicinity of the aforementioned NPL sites, but are also in areas where the statistically higher childhood cancer rates were established.

The ensuing evaluations revealed the presence of a previously unidentified compound, styrene acrylonitrile (SAN), in the groundwater from the Parkway wellfield that could be traced to the Reich Farm NPL site.

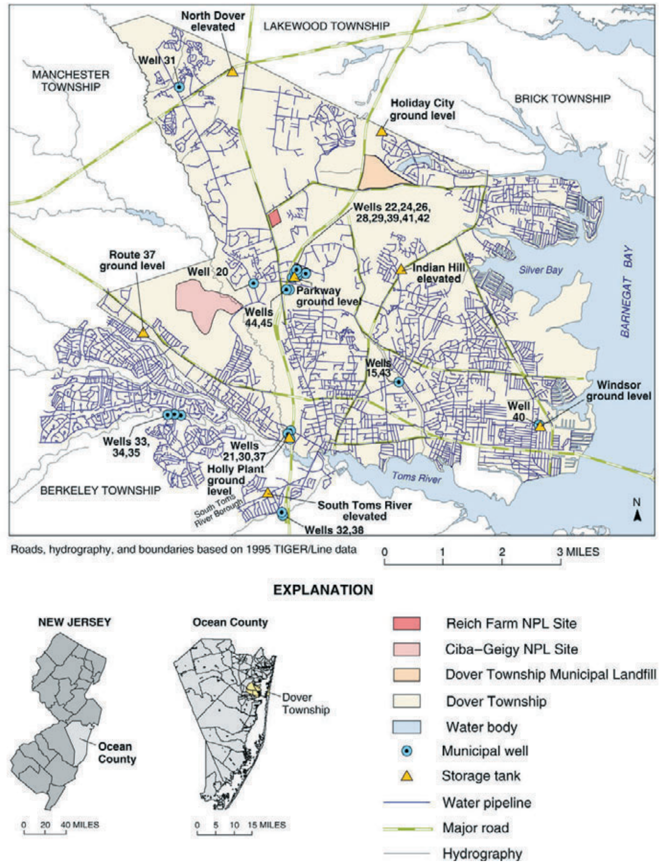


Figure 7-2. Investigation Area, Dover Township, Ocean County, NJ (modified from Maslia et al., 2001).

Similarly, a search of historical records revealed contamination (primarily semivolatile organics [SVOCs]) of the Holly well fields that could be traced to the Ciba-Geigy NPL site. Furthermore, one of the hypotheses for the epidemiologic case-control study was that the higher cancer incident rate was related to the higher exposure to public water supplies with documented contamination (the Parkway and Holly well fields). To assist NJDHSS with the contaminated drinking water exposure assessment component of the epidemiologic study, ATSDR developed a water distribution model for the study area using the EPANET software. This network model was used to simulate historical characteristics of the water distribution system serving Dover Township from 1962–1996. Because there was a lack of historical contaminant-specific data during most of the period relevant to the epidemiologic study, the modeling effort focused on estimating the percentage of water that a study subject might have received from each well that supplied water to the impacted area. The following subsections present a brief overview of the water distribution modeling effort (both hydraulic and water quality) followed by a summary of findings and conclusions.

Prior to the ATSDR's analysis of well field contamination in Dover Township and the potential linkages to childhood diseases, another study in Woburn, Massachusetts, heralded the era of such analyses. Though both the model and graphical presentations are primitive by today's standards, they were effective in providing a quantitative basis for assessing the spread of contaminants in the distribution system. The following is a brief description of the Woburn analysis.

In May 1979, the Massachusetts Department of Environmental Quality Engineering discovered that two wells (Wells G & H in the B Zone – See Figure 7-3) in Woburn, Massachusetts were contaminated with toxic chemicals. Subsequent analysis showed that parts of the city experienced elevated levels of childhood leukemia and other illnesses attributed to drinking water derived from these wells. This event resulted in legal action, a diverse set of scientific studies that are still ongoing, and the publication of a book entitled *A Civil Action* (Harr, 1995). Early steady-state distribution system hydraulic and water quality models were also used as a means to track the movement of the contaminated water in the distribution system under a range of operating and demand conditions (Murphy, 1986). The accompanying figure is one example of a plot resulting from this early modeling effort. As shown in Figure 7-3, based on the modeling, the city was divided into three zones for each scenario – the A zone that received no water from the contaminated wells, the B zone that received all of its water from the contaminated wells, and the C zone that received some of its water from the contaminated wells.

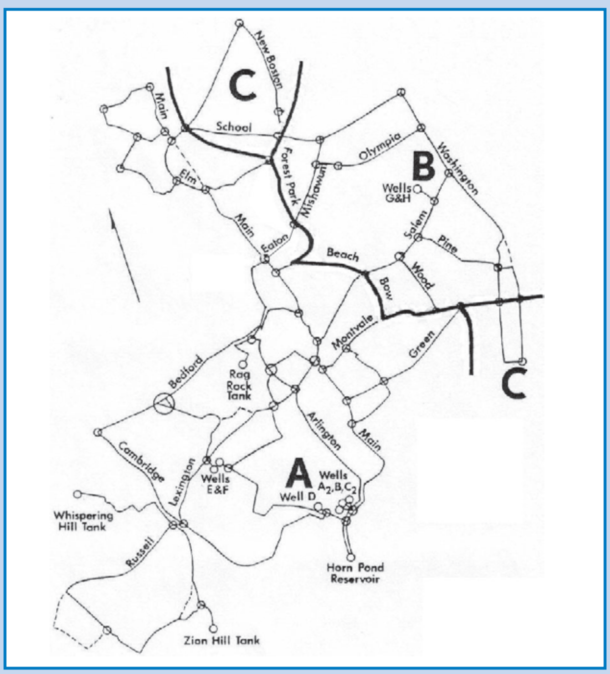


Figure 7-3. Distribution System Zones – Woburn, MA (May 1969).

7.2.2 Overall Modeling Approach

Because of the lack of historical hydraulic and water quality information, the water distribution system was characterized using data gathered during an extensive field investigation in 1998. The 1998 field investigation consisted of two components: (1) determining spatial locations of distribution system facilities (wells, tanks, pump, and hydrants) and (2) equipping hydrants with continuous-recording digital data loggers and monitoring supply sources (wells, pumps, and tanks) to measure system responses during winter demand (March) and summer demand (August) periods. Twenty-five hydrants located throughout the distribution system were equipped with data loggers to simultaneously collect information on system response (Maslia et al., 2000). The collected response data included on-off cycling of groundwater wells, high service and booster pump operations, pressure variations, storage tank water-level fluctuations, and total production.

A detailed "all-pipe" hydraulic network model was developed and calibrated to present-day conditions (1998) using the field investigation results. The reliability of the calibrated model was successfully demonstrated through a water quality simulation of the transport of a naturally occurring conservative element (barium) and a comparison of the results with data collected in March and April 1996 at 21 schools and 6 points of entry to the water distribution system. Thereafter, to describe the historical distribution system networks specific to the Dover Township area, databases were developed from diverse sources of information. These data sources included water utility pipeline installation records, quarterly billing records, NJDHSS groundwater well records, and annual water utility reports to the state board of public utilities. These data were applied to EPANET and simulations were conducted for each month of the historical period—January 1962 through December 1996 (420 simulations). After completing those 35-year/420-month analyses, source-trace analysis simulations were conducted to determine the percentage of water contributed by each well or well field operating during each month for all study subject locations.

A review of the historical network configuration revealed that the water distribution system complexity increased significantly during this period. The model inputs were appropriately adjusted to account for these historical changes. For example, the 1962 water distribution system was represented with an approximate peak production of 1.3 million gallons per day (MGD) produced from three wells that served nearly 4,300 customers (population ~17,200). By contrast, in 1996, the water distribution system had an approximate peak production of 13.9 MGD produced from 20 wells that served nearly 44,000 customers (population ~ 89,300). Appropriate adjustments were made to modeled pipe segments, storage reservoirs, and operational details. Grayman et al. (2004) present a more detailed account of the EPANET model input adjustments. Production data for the 420-month historical period is graphically represented in Figure 7-4.

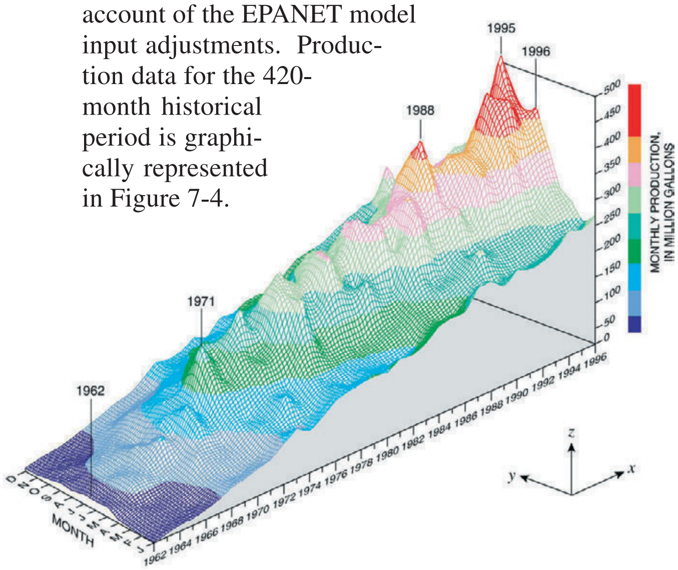


Figure 7-4. Three-Dimensional Representation of Monthly Production of Water, Dover Township Area, NJ (from Maslia et al., 2001).

To perform an extended period simulation (EPS) of the distribution of water for each of the 420 months of the historical period, information was required on network configuration, demand, and operational data. However, operational data prior to 1978 were unavailable, requiring the development of system operation parameters—designated as "master operating criteria (MOC)." The MOC is based on hydraulic engineering principles necessary to successfully operate distribution systems similar to the one serving the Dover Township area (Table 7-1). From 1978 forward, for selected years, operators of the water utility provided information on the generalized operating practices for a typical peak-demand (summer) and non-peak demand (fall) day. These guidelines were used in conjunction with the MOC to simulate a typical 24-hour daily operation of the water distribution system for each month of the historical period.

The model parameter of interest from the epidemiologic study perspective was the proportionate contribution of water from wells and well fields to locations throughout the historical pipeline networks. Thus, the distribution of water delivered to pipeline locations was the item of interest rather than the specific operations of the wells, storage tanks, and pumps (WSTP) that delivered the water. Normally, detailed WSTP operational inputs would be required for EPANET simulation. However, to simplify the simulation methodology and reduce data requirements, a "supply-node-link" (SNL) method of idealizing the WSTP combination was developed. In the SNL simulation method, an equivalent amount of water is supplied to the distribution system (based on estimated monthly demands and the typical daily operation of the systems). To demonstrate that the surrogate SNL simulation method supplies the distribution system with an equivalent amount of water when compared to the real-world WSTP simulation method, both simulation methods were applied to the present-day (1998) water distribution system for conditions existing in August 1998. The results obtained from these simulations produced nearly identical flows in the modeled system.

7.2.3 Simulation Techniques

Using the EPANET network model developed for the Dover Township area, hydraulic modeling was conducted whereby average network conditions were simulated for every month of the historical period

Table 7-1. Master Operating Criteria Used to Develop Operating Schedules for the Historical Water Distribution System, Dover Township Area, NJ (from Maslia et al., 2001)

Parameter	Criteria
Pressure[1]	Minimum of 15 psi; maximum of 110 psi at pipeline locations, including network end points
Water level	Minimum of 3 ft above bottom elevation of tank; maximum equal to elevation of top of tank; ending water level should equal the starting water level
Hydraulic device on-line date	June 1 of year installed to meet maximum-demand conditions
On-and-off cycling: Manual operation	Wells and high-service and booster pumps cannot be cycled on-and-off from 2200 to 0600 hours
On-and-off cycling: Automatic operation	Wells and high-service and booster pumps can be cycled on-and-off at any hour
Operating hours	Wells should be operated continuously for the total number of production hours, based on production data[2]

[1]Generally, for residential demand, minimum recommended pressure is about 20 psi. However, for some locations in the Dover Township area (mostly in areas near the end of distribution lines), lower pressures were simulated.
[2]See Maslia et al. (2001) for production data (Appendix B) and hours of operation (Appendix D)

(420 simulations). These simulations were completed under balanced flow conditions that utilized hydraulic engineering principles and conformed to the MOC (Table 7-1). Thereafter, using the results of the monthly network hydraulic simulations, water quality simulations (source-trace analysis) were conducted for each water source (point of entry) of the network in order to determine the monthly proportionate contribution of source water at all locations in the Dover Township area serviced by the water distribution system.

EPANET is a dynamic water quality model that has the ability to compute the percentage of water reaching any point in the distribution system over time from a specified location (source) in the network. To estimate this proportionate contribution of water, a source location is assigned a value of 100 percent. The resulting solution provided by the water quality simulator in EPANET then becomes the percentage of flow at any location in the distribution system network (for example, a demand node) contributed by the source location of interest. For the purposes of this analysis, a source-trace analysis was conducted for every month of the historical period. Source nodes were assigned a value of 100 percent in order to estimate the proportionate contribution of water to locations in the historical distribution system networks. These initial conditions were fully propagated through most of the distribution system before retrieving the proportionate contribution results (Maslia et al., 2000). Accordingly, for each monthly historical network model, 24-hour demand and operational patterns were defined and these patterns were repeated for approximately 1,200 hours to reach a state of stationary water-quality dynamics (dynamic equilibrium). For most of the analyses, hydraulic time steps of 1 hour and water-quality time steps of 5 minutes were used within EPANET. For some monthly simulations, the water-quality time steps were reduced to 1 minute to ensure that the mass balance summed to ~100 percent (range of 98 to 101 percent due to numerical approximations).

With respect to the scheduling of groundwater well operations, the EPANET model was set to utilize pattern factors corresponding to the hourly operations of supply wells. These pattern factors along with the operational extremes of storage tank water levels were manually adjusted during each of the 420 monthly network simulations to achieve balanced flow conditions. This approach to simulation was designated as the manual adjustment process. A second simulation approach designated as the genetic algorithm (GA) approach was also utilized to achieve balanced flow conditions for each of the 420 monthly networks of the historical period. This approach required the development of an innovative methodology known as the progressive optimality genetic algorithm (POGA), which is an automated objective simulation technique (Aral et al., 2004a, b). The GA simulations utilized the balanced flow conditions obtained by the manual adjustment process as starting conditions. The GA technique was used to address the following key questions:

- If a balanced flow operating condition was achieved using the manual adjustment process, was the resulting operating condition the only way the system could have been successfully operated?
- Could alternative or additional operating conditions be defined such that system operations would also be satisfactory or even optimal?

Thus, the POGA methodology was used in conjunction with EPANET to simulate alternative and possibly optimal water distribution system operations and to assess the effects of variations in system operations on the results of the proportionate contribution simulations.

7.2.4 Simulation Results and Conclusions

Figure 7-5 shows the aerial distribution of simulated proportionate contribution results for all model nodes (pipeline junctions) for the month of July 1988, using the Parkway well field as the point of entry (source point). The simulated proportionate contribution results are divided into six intervals (1 to 10 percent, 10 to 25 percent, 25 to 50 percent, 50 to 75 percent, 75 to 90 percent, and 90 to 100 percent) and a color is assigned to all nodes within each interval (results are not shown for negligible proportionate contributions of less than 1 percent).

Simulated proportionate contribution results can also be viewed in terms of selected pipeline locations and the combination of wells or well fields that contribute water to that location. Five geographically distinct pipeline locations are selected from the historical networks to represent the spatial distribution of proportionate contribution results. These locations are identified on Figure 7-5 as locations A, B, C, D, and E. The simulated proportionate contribution of water for July 1988 corresponding to each pipeline location is shown graphically on Figure 7-6. The simulation results demonstrated that the contribution of water from wells and well fields varied by time and location. However, the results also showed that certain wells provided the predominant amount of water to locations throughout the Dover Township area. Additionally, although the pattern factors for some hours of operations showed marked differences, the simulated proportionate contributions of water using

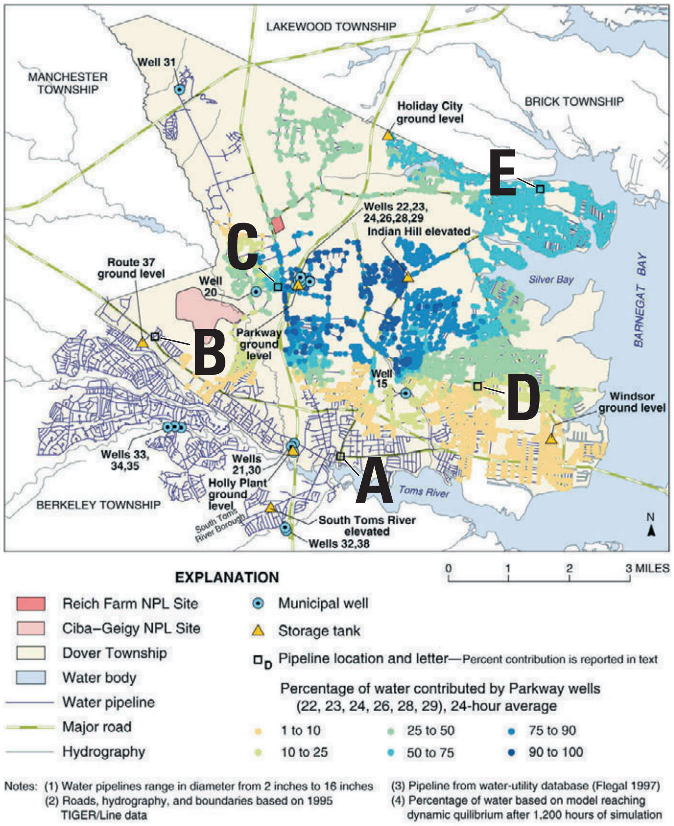


Figure 7-5. Areal Distribution of Simulated Proportionate Contribution of Water from the Parkway Wells (22, 23, 24, 26, 28, 29) to Locations in the Dover Township Area, NJ, July 1988 Conditions (from Maslia et al., 2001).

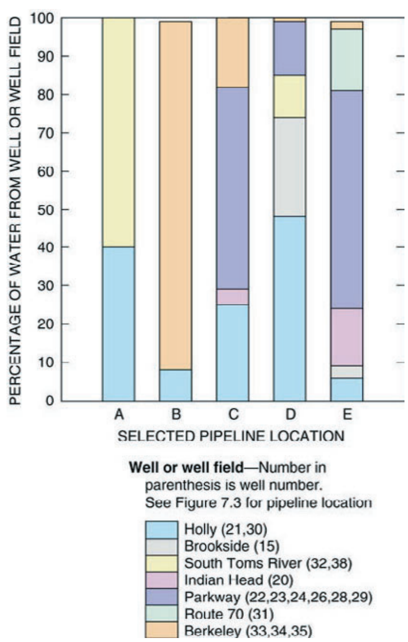


Figure 7-6. Simulated Proportionate Contribution of Water from Wells and Well Fields to Selected Locations, Dover Township Area, NJ, July 1988 Conditions (from Maslia et al., 2001).

pattern factors derived from the application of the POGA methodology showed little difference throughout the Dover Township area when compared to corresponding proportionate contribution of water simulated using the manual adjustment process. The results of sensitivity analyses conducted using the historical reconstruction process indicated the following:

- There was a narrow range within which the historical water distribution systems could have successfully operated and still satisfy hydraulic engineering principles and the MOC.
- Daily operational variations over a month did not appreciably change the proportionate contribution of water from specific sources.

Therefore, the reconstructed historical water distribution systems were determined to be the most plausible and realistic scenarios under which the 1962–1996 historical water distribution systems were operated. The health scientists conducting the case-control epidemiologic study used the resulting percentage of water derived from the different sources to derive exposure indices for each study subject.

The results from the case-control study showed that there was an association between prenatal exposure to contaminated community water and leukemia in female children (NJDHSS, 2003). For example, female leukemia cases were 5 times more likely to have occurred when exposed during the prenatal period to a high percentage of Parkway well water than were control children. The control children are those living in the study area, but were not exposed to the water from the contaminated well fields. These findings would not have been possible without the results derived from the innovative water distribution system modeling efforts. These efforts have led to developing new methods for evaluating the accuracy of modeling results and exposure classification techniques that are critical components of epidemiologic studies. Some of the innovations documented by the Dover Township historical reconstruction analysis are:

- A new approach, proportionate contribution analysis, was developed that utilized water distribution system modeling and source tracing to quantify exposure on a monthly basis for all locations historically served by the distribution system.
- Through the use of an innovative genetic algorithm approach (POGA), historical water distribution system operating schedules were

The city of Redlands lies in the San Bernardino valley of California, approximately 60 miles east of central Los Angeles. In 1981, a routine analysis for chlorination byproducts revealed the presence of trichloroethylene (TCE) in a sample of water from the Redlands water system. Subsequent water quality analyses revealed that a number of wells supplying the city were contaminated with TCE. In 1997, the perchlorate anion (ClO_4) was also detected in several wells. In 1996, the first of a series of lawsuits was filed in California State Court alleging that the source of these contaminants was a manufacturing facility located up-gradient from the most seriously contaminated wells.

One of these lawsuits claimed that plaintiffs were harmed by exposure to toxic chemicals that were improperly disposed of at the manufacturing site and found their way into groundwater that was subsequently extracted through the city's wells and delivered to water customers, including the plaintiffs. The plaintiffs' burden of proof requires them to establish, among other things, that they were actually exposed to contaminated water at their homes, places of work, or other locations and that the amounts of contaminants that entered their bodies as the result of these exposures were sufficient to cause harm to them.

To establish this proof, experts for the plaintiffs reconstructed the historical conditions in the water distribution system of the City of Redlands over a period from the mid 1950s to the late 1990s using the EPANET model. As part of litigation, several forensic reconstructions of water quality in the Redlands water distribution system were performed. The reconstruction involved estimates of both human exposure to toxic contaminants and whole-body intakes of these chemicals. Estimates of exposures and intakes were expressed as credibility intervals, which were calculated using Monte Carlo simulation techniques. As an example output from the analysis, Figure 7-7 illustrates the estimated upper 97.5 percent credibility limit of one plaintiff's exposure to perchlorate. Similar information was developed to describe the lower 2.5 percent credibility limit for each of the test plaintiffs in the case (Grayman et al., 2004).

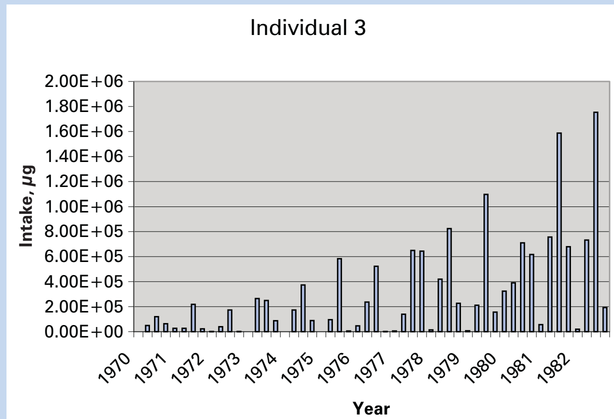


Figure 7-7. Estimated Upper 97.5 Percent Credibility Limit for Annual Perchlorate Intake by One Plaintiff (Grayman, 2004).

synthesized. Sensitivity analyses indicated that operating system changes did not appreciably change the proportionate contribution of water to Dover Township locations.

- The association between exposure and disease would not have been possible without developing the integrated approach using environmental science, engineering evaluations, and epidemiologic analyses.

Historical reconstruction of environmental exposure is not an easy task. The procedures and results summarized herein (and the detailed analyses in Maslia et al., [2001]) represent one of the most comprehensive, well-documented, and quality-controlled studies of its kind. Another example of public exposure assessment using modeling is related to the City of Redlands, California.

7.3 Application of Water Distribution System Modeling in Support of a Regulatory Requirement

The new DBPR2 regulation that is proposed for promulgation in the near future requires all water utilities that have a disinfectant residual in the distribution system to perform an IDSE unless they obtain a small-system or "40/30" waiver (EPA, 2003). Systems that can certify TTHM and HAA5 compliance data to be less than or equal to 40 µg/L for TTHM and 30 µg/L for HAA5 are not required to perform an IDSE. The goal of the IDSE is to identify compliance monitoring sites that may have high DBP levels in distribution systems. Utilities may choose to perform an SMP that involves extensive monitoring. Alternatively they may choose to perform a system-specific study (SSS) that uses historical data, distribution system models, or other analyses combined with minimal monitoring to evaluate TTHM and HAA5 levels throughout the distribution system as the basis to select future compliance monitoring sites. This case study demonstrates how a hydraulic/water quality distribution system model can be applied to satisfy the IDSE requirements of an SSS.

Key Phrases to Characterize Case Study: Regulatory modeling, IDSE, water quality modeling.

7.3.1 IDSE Requirements Overview

The IDSE guidance manual spells out a series of suggested minimum requirements for the use of a calibrated water distribution system hydraulic model to perform an SSS. In general, the water distribution system model should be more comprehensive for the purpose of an SSS than models typically used for long-range capital improvement program analysis (e.g., master planning). A calibrated hydraulic model intended for detailed distribution system design (e.g., for system improvements) or operational studies is likely to be adequate. Because systems are always changing (e.g., population growth, industry development in network area, aging of mains), it is important that the model generally reflect system conditions and demand at the time of the IDSE SSS. A model that has not been updated or calibrated in the last 5 to 10 years is unlikely to be adequate for an SSS. The guidelines provided in the draft guidance manual are summarized below:

- EPS model that has been recently calibrated using generally accepted methods.
- An all-pipe model or skeletonized model that includes (a) at least 50 percent of total pipe length in the distribution system, (b) at least 75 percent of the pipe volume in the distribution system, (c) all 12-inch-diameter and larger pipes, (d) all 8-inch and larger pipes that connect major facilities, (e) all 6-inch and larger pipes that connect remote areas of a distribution system, and (f) all active control valves or other system features that could significantly affect the flow of water through the distribution system.
- Water demand should be allocated among the nodes of the model in a manner that reflects the actual spatial distribution of such demand throughout the system.
- A system-specific, diurnal (24-hour) demand pattern should be applied to the overall system demand.
- The model should accurately simulate seasonal system configurations and operational changes.

Once the model is established, it is then run in EPS mode until a consistent, repeating temporal pattern of water age is established at all nodes of the model. Generally, the model should be run under high DBP formation conditions (typically summer months) to estimate residence times. Based on the modeled water age results, preliminary monitoring sites are identified near locations that satisfy the sampling site requirements. Sampling sites are selected to represent:

- High-TTHM Sites: High TTHM values are expected at high-residence-time locations. These locations can be identified by reviewing the modeled water age at each node in the model. These sites are generally downstream of storage facilities and in remote locations. However, the regulation does not require extremes or non-representative sites to be sampled.
- High-HAA5 Sites: The criteria and procedure for selecting high HAA5 sites using a hydraulic model are generally the same as those described above for selecting high-TTHM sites with one important difference: the sites chosen to represent high HAA5 should have a disinfectant residual sufficient to suppress bacteria which can degrade HAAs.
- Average-Residence-Time Sites*:* Average-residence-time sites can be selected from sites with residence times close to the flow-weighted mean of all nodal residence times.
- Near-Entry-Point Sites: Modeled water age can be used to identify locations in the near vicinity to entry points into the water system.

Requirements for the number of monitoring sites have not yet been finalized. As a result, this case study demonstrates the general usage of models for IDSE and relies upon the 2003 draft guidance issued by EPA to illustrate the usage of models.

7.3.2 Example Application of Modeling in the IDSE Process

The following example is a hypothetical case study based in large part on an actual water distribution system. The system purchases disinfected groundwater and serves approximately 15,000 people. Water enters the distribution system from two separate interconnections to a wholesale utility. The average demand is 2.2 MGD. A north interconnection operates intermittently and provides approximately 80 percent of the demand, while the south interconnection operates at all times and provides the remaining 20 percent of the system demand. There is a 1.5-million-gallon storage tank. In order to comply with the Stage 2 requirements, the draft proposed DBPR2 states that a total of six sites are required for a utility of this size using groundwater: one representing a site near to the predominant entry, one representing a site with average residence time, two sites representing high TTHM conditions, and two representing high HAA5 conditions.

The water utility water distribution system model has been used extensively in the past for both hydraulic and water-quality studies. It is a skeletonized model that includes all 8-inch-diameter and larger pipes and all major facilities. The pipes in the model represent 74 percent of the total length of pipe in the system and 86 percent of the total volume. The model has been previously calibrated based on two tracer studies and has been shown to have excellent agreement with observed field results. Demands have been assigned to nodes based on actual meter readings, and information from the SCADA system has been used to construct a typical diurnal water use pattern. The model is being operated in an EPS mode to simulate a 12-day period. The model has also been calibrated for use in simulating chlorine residual in the distribution system.

A series of runs of the model were performed to help understand the movement of water and water quality transformations in the system. Specific simulations included water age and chlorine residual. The results of the water-age model run are shown in Figure 7-8. This plot shows the average water age throughout the distribution system over the last 24 hours of the 2-week simulation. This period was selected to avoid the uncertainty associated with assigning initial water age in the system. The plot illustrates the nodes in the vicinity of the predominant northern interconnection that receive water with an average age of less than 2 hours. As also shown, the average water age increases significantly for areas that are further from the interconnections. The demand flow-weighted average water age for delivered water was calculated to be 27 hours. However, water age can also vary quite significantly over the course of a day in water systems, largely due to the impacts of storage tanks. This is illustrated in Figure 7-9, which depicts the minimum water age at each node over the same 24-hour period. This is also shown in the plot in Figure 7-10 for Node J-456 in the vicinity of the tank. The IDSE guidance does not require utilities to explicitly consider the effects of tanks on diurnal variations in water age, and thus on the formation of DBPs. If Node J-456 was selected as representative of high DBP because of its high water age, it would be expected that the DBPs would only be high during the part of the day when water is being discharged from the tank.

The model was also used to determine the chlorine residual throughout the system. Figure 7-11 contains a plot of the minimum chlorine residual throughout the system. It is important to note areas with high

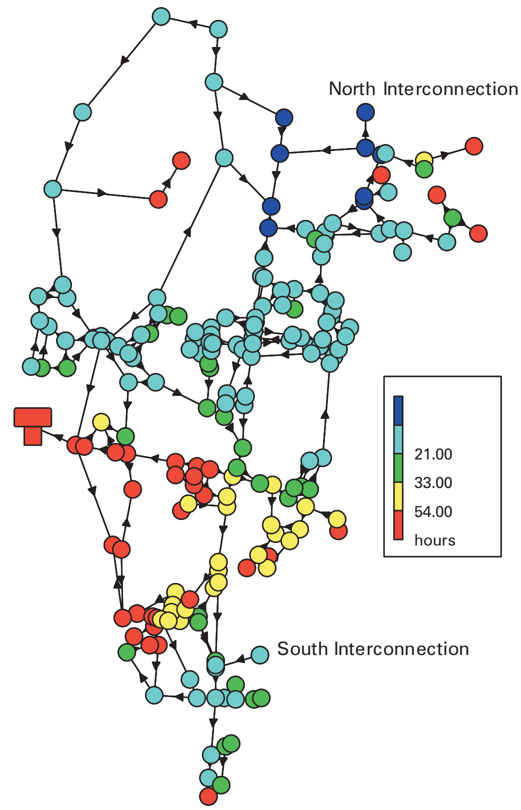


Figure 7-8. Average Water Age in the Distribution System Over Last 24 Hours of a 2-Week Simulation.

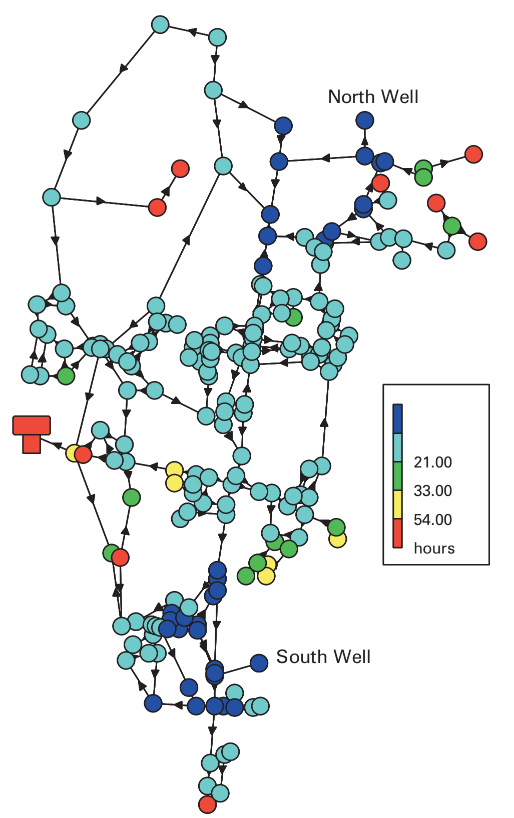


Figure 7-9. Minimum Water Age in the Distribution System Over Last 24 Hours of a 2-Week Simulation.

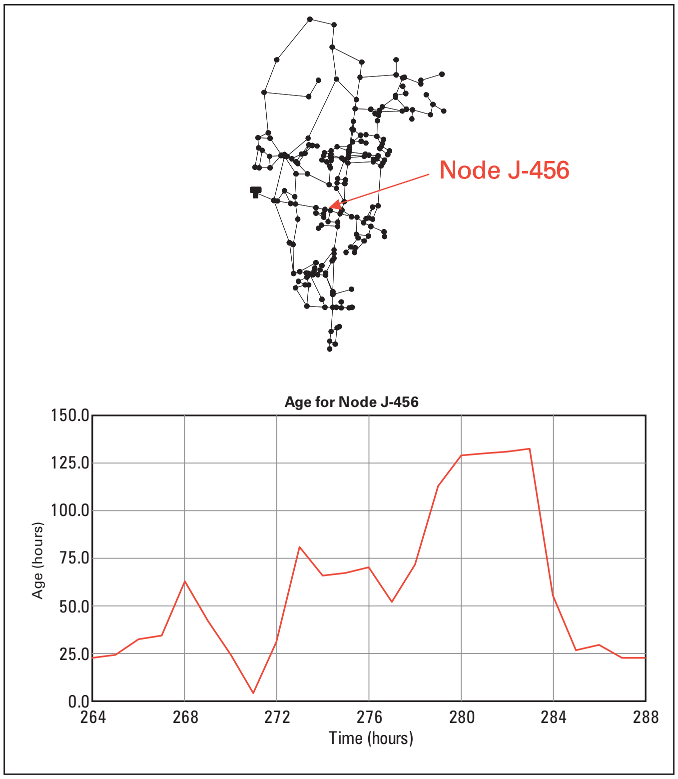


Figure 7-10. Diurnal Water Age at Node J-456.

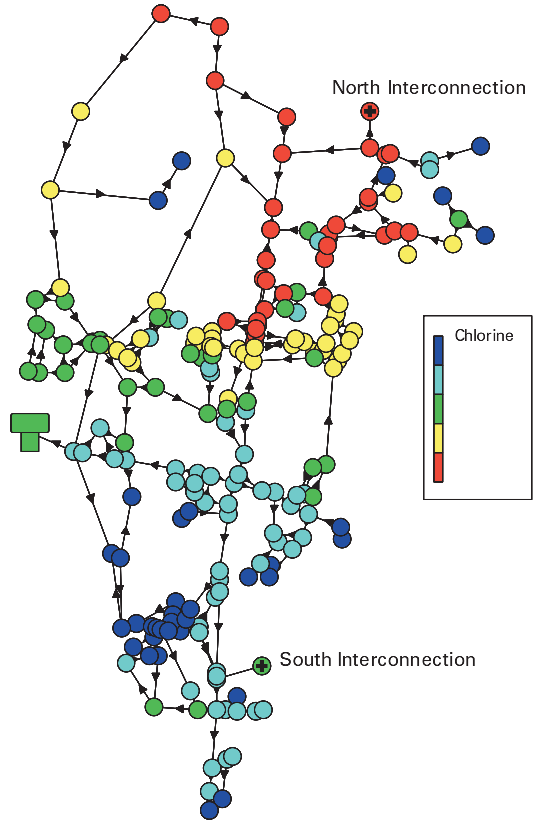


Figure 7-11. Minimum Chlorine Residual in Distribution System Over Last 24 Hours of a 2-Week Simulation.

residence time and low residual for high-TTHM sites and high residence time and high residual for high-HAA5 sites. This information can be used to avoid selecting monitoring sites that are to be used as representative of high HAA5 concentrations. The current ability to accurately model HAA5 in a distribution system is limited. However, research has shown that depressed chlorine residual can result in biodegradation of HAA5, thus lowering the HAA5 concentrations even for older water. Figure 7-11 also shows a small area in the southwestern portion of the system that the model predicts to potentially experience chlorine residuals less than 0.2 mg/L of chlorine.

Based on the modeling results, various zones were defined in the distribution system representing areas that are appropriate for different types of compliance monitoring requirements (Figure 7-12). In actual use, information generated by the model would be supplemented by historical field data. The zones shown in the plot include:

1. Nodes in the vicinity of the predominant north connection with water age less than 2 hours representative of near entry locations;

2. Nodes with average water age in the range of 21 to 33 hours that represent locations that approximate the average residence time of 27 hours;

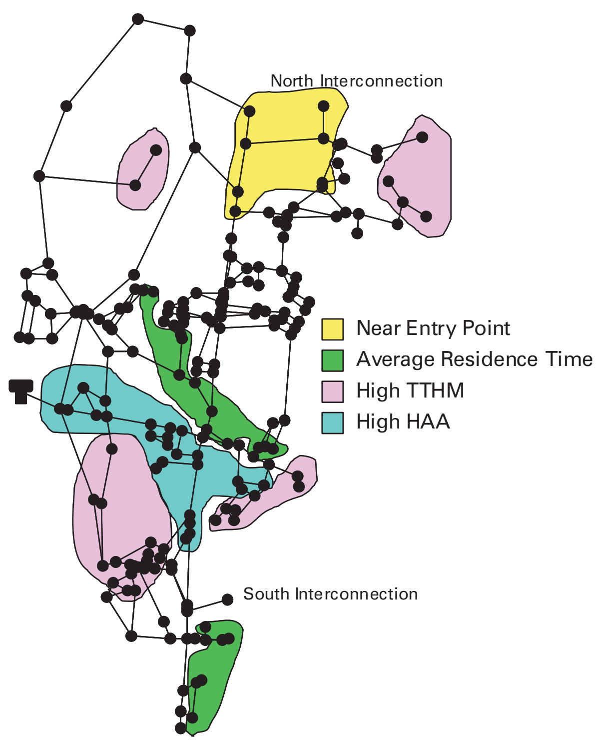


Figure 7-12. Zones Representing Potential Monitoring Locations for IDSE Based on Modeling.

3. Nodes with residence times that exceed twice the average water age (> 54 hours) and have a minimum chlorine residual exceeding 0.20 mg/L representing potential high-HAA5 sites; and

4. Other nodes with residence times that exceed twice the average water age (> 54 hours) representing potential high-TTHM sites.

As illustrated by the case example, if a detailed, calibrated EPS model is available, the model represents an efficient means of defining the compliance monitor locations as required under the forthcoming regulation.

7.4 Use of Water Distribution System Models in the Placement of Monitors to Detect Intentional Contamination

The increasing concern over the potential for intentional contamination of a water distribution system has led to interest in the placement of monitors to detect contamination and to serve as part of a rapid detection system. Design of such monitoring systems must include decisions on the type, number and location for the monitors. Water distribution system models can play a significant role in the decision making by providing a quantitative mechanism for determining the movement of a contaminant through the distribution system and testing the effectiveness of a monitoring system design.

To illustrate this application, a red team-blue team concept is used (Grayman et al., 2005). The red team-blue team concept is part of "war gaming" that is widely used today as a mechanism for training and development and testing of security plans. The red team acts as the aggressor and the blue team acts as the defenders. Each team has different types and amounts of information available to them and different rules or constraints that they must follow. In this case study, network models are used in two modes to assist in evaluating monitoring networks:

1. As part of a red team-blue team exercise to demonstrate the effectiveness of manual selection of location of monitors as part of a CWS.

2. As part of an optimization model to select the best locations for monitors based on a stated metric for measuring the effectiveness of the monitoring system.

Key Phrases to Characterize Case Study: Water security, contamination, optimization, monitor placement

7.4.1 Red Team-Blue Team Exercise

In this simulated exercise, the red team attacks a water distribution system by adding a harmful chemical to the water. The red team is provided with limited information on the distribution system, a number of potential locations where they can inject a contaminant, and predetermined information on the characteristics of the contaminant (quantity and lethality of the contaminant). The blue team represents the water utility and attempts to protect the water system by installing three monitors as part of a CWS that detects contaminants. It is assumed that they have extensive information on the design and operation of the distribution system but no firm information on where the attackers may choose to introduce the contaminant or the nature of the contamination scenario.

The water distribution system network used in this exercise is a skeletonized version of a major pressure zone of a water distribution system in California approximating the conditions (design and operation) in the mid 1990s. The system is fed by two sources; one that operates continuously and one that operates only during the day. There are three storage facilities located in the network. This network is one of the example networks provided as part of the EPANET model. The simulation performed in the exercise is a 24-hour EPS starting at 7AM. The model representation of the network is shown in Figure 7-13. This figure also illustrates the relative nodal demands, and the typical flow directions and magnitude during the day. This plot is given only to the blue team to provide them with information on the design and operation of the system. The red team is provided

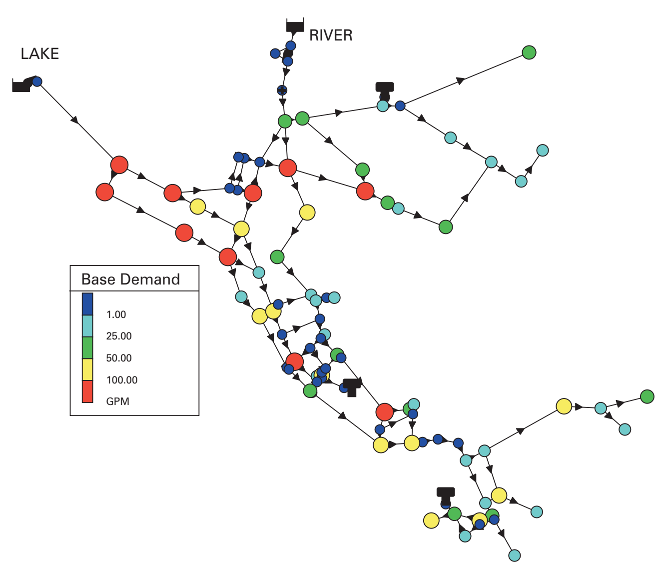


Figure 7-13. Water Distribution System Characteristics.

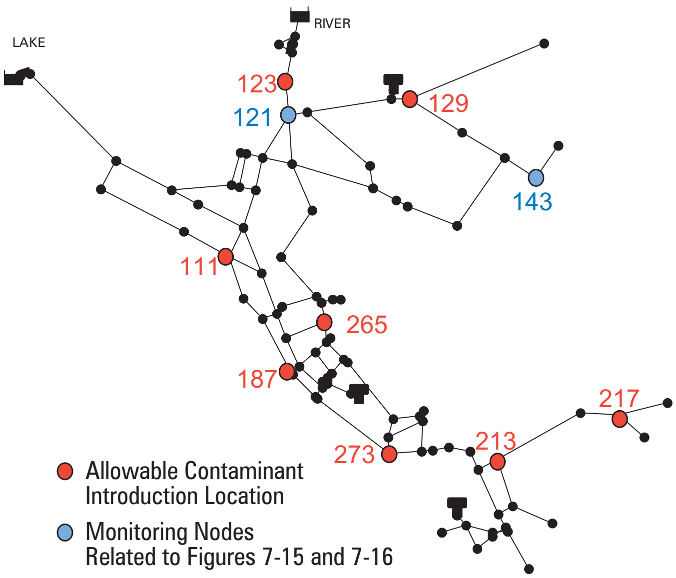


Figure 7-14. Allowable Contaminant Introduction Locations.

only with a map of the distribution system showing eight potential sites that can be used to introduce a contaminant (Figure 7-14).

Following the selection of points of attack by the red team and selection of monitor locations by the blue team, the contaminant introduction is simulated using the EPANET model. The movement and concentration of the contaminants are then viewed graphically by animating the movement of the contaminant in the distribution system in the EPANET model. The time history of contaminant concentrations is also viewed at selected nodes in the distribution system. The effectiveness of monitors is illustrated by graphing the concentrations of the contaminants at monitoring nodes and assessing whether (and how quickly) the monitors will serve their purpose of detecting the contaminant. Figures 7-15 and 7-16 show the concentrations resulting from 8 hours of contamination at node 123 starting at 11 a.m.

In Figure 7-15, the resulting contamination at a node located 1500 feet immediately south of the injection point is shown. As expected, the contaminant moved very rapidly and reached this node in less than an hour after the injection. If a monitor was located at this node, and rapid analysis and response occurred, it could be very effective as an early warning for most of the distribution system. Figure 7-16 illustrates concentrations resulting from the same contaminant introduction location. This node is located near the eastern edge of the distribution system approximately 2.5 miles downstream of the contaminant introduction location. As illustrated, the concentration of the contaminant remained about the same but the travel time to this point was approximately 7 hours. For this injection scenario, a monitor located at this point would be relatively ineffective as a warning device for most of the distribution system because of the significant time lag.

In the exercise, most red team members tend to select contamination introduction locations that they perceive would result in the most widespread impacts. The most often selected sites were those near to the water sources. Little attention is generally given to the amount of dilution that would result at a particular location. Blue team members tend to select monitoring locations that cover a wide range of locations. Frequently, the three allowable monitors will be located in the north, central, and southern portions of the distribution system.

7.4.2 Application of Optimization Model

The optimization model used in this demonstration is a methodology developed by Ostfeld and Salomons (2004). The model links EPANET and a genetic algorithm in an overall framework for optimally allocating monitoring stations, aimed at detecting deliberate external contamination into water distribution system nodes. The model operates under extended period (unsteady) hydraulics and water quality conditions. The optimization routine determines the monitor placement to detect contaminants in order to minimize the exposure of

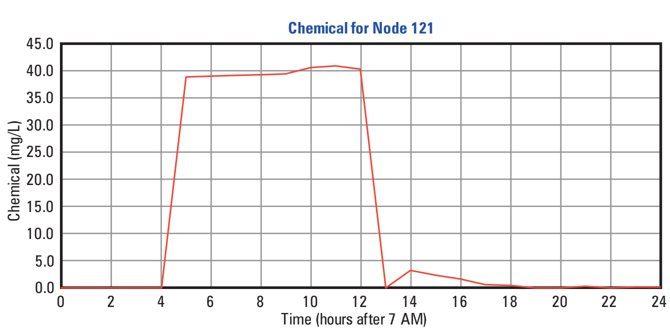


Figure 7-15. Contaminant Concentration Just Downstream of Contaminant Introduction Location (Node 121).

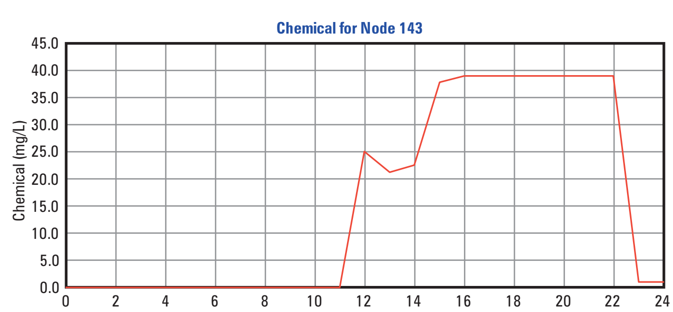


Figure 7-16. Contaminant Concentration Far Downstream of Contaminant Introduction Location (Node 143).

customers above an allowable minimum concentration. The algorithm can be used to study contamination of fixed duration, quantity, or location or can simulate contamination under stochastic conditions. There are several model parameters that can be specified to control the number of monitors, the allowable contaminant introduction locations, the characteristics of the event, and whether the event characteristics and demands are to be considered as stochastic variables.

In one application of the model, it was assumed that the pollutant could be introduced at any single node of the distribution system at any time, all with the same injection probability. The following additional assumptions were made:

- The maximum contamination exposure volume to the public above which an alarm signal of the monitoring stations is required is 25 gallons.
- The water is considered contaminated above 1 mg/L.
- The pollutant flow discharge is 2 kg/min for 5 minutes (i.e., a total of 10 kg of a solution of 100 percent is introduced within a total of 5 minutes).
- The pollutant flow discharge of the contaminant introduced and the consumer demands are deterministic.
- The monitoring stations are providing real-time data and detection alarms.
- All monitoring stations have a detection sensitivity of 1 mg/L.
- 3 monitors are to be placed.

The model suggests placing monitors at nodes 143, 181, and 213 with a detection likelihood of 0.4354 (i.e., there is a probability of about 44 percent that the contaminant will be detected prior to the consumption of more than 25 gallons at a concentration higher than 1 mg/L). The location of the monitors is shown in Figure 7-17. As illustrated, the selected monitor locations were relatively evenly spaced around the network.

Other evaluated scenarios looked at a different number of allowable monitors, the allowable contaminant introduction locations, the critical exposure threshold, and representation of contaminant quantity and nodal demands as stochastic variables. Though the exact "optimal" locations varied slightly between the different runs, typically the monitors were placed throughout the network. However, the effectiveness of the monitoring network, as measured by the detection likelihood does vary considerably between scenarios.

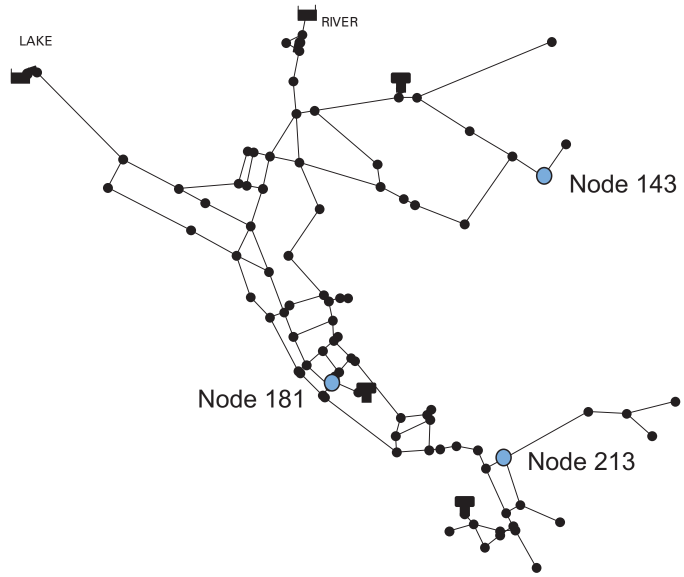


Figure 7-17. Monitoring Locations Selected by the Optimization Model.

7.4.3 Case Summary

The red team – blue team exercise serves as a good mechanism for demonstrating both the dynamics of contaminant movement in the distribution system and the potential effectiveness of monitors. Application of the optimization model, both as a demonstration procedure and as a practical tool, provides a method that moves the monitor placement from a purely intuitive process to a quantitative procedure. Both the exercise and the optimization tool show the importance in minimizing delays in the detection, notification, and response process. Even an added delay of an hour or two can lead to a significant increase in the number of customers that would be impacted by a contamination event.

7.5 Case Study – Use of PipelineNet Model

This case study focuses on the application of the PipelineNet model, which incorporates both GIS and the EPANET model discussed in the previous chapters of this reference guide. The supporting investigations were primarily sponsored by the Awwa Research Foundation (AwwaRF) and EPA with work performed by a consulting firm (SAIC) and assistance from water utility personnel (Ron Hunsinger, Bill Kirkpatrick, Dave Rehnstrom) working at the East Bay Municipal Utility District (EBMUD), Oakland, CA. The text and figures are adapted from AwwaRF report 2922 prepared by Bahadur et al. (2003).

Key Phrases to Characterize Case Study: hydraulic and water quality modeling, placement of monitors, exposure modeling, contamination assessment, contamination response tools, geospatial analysis.

7.5.1 Overview

PipelineNet is an EPANET/ArcView[1] -based model, and uses the same hydraulic engine as EPANET. The EPANET portion of the model can simulate the fate and transport of potentially introduced contaminants in a water distribution system. The ArcView (or the GIS layer) portion of the model can relate the geospatial components of the simulated impact. The GIS layer allows for geo-features and map display with an overlay of model output. This feature is particularly useful in applications such as emergency response, determining optimal placement of sampling, and monitoring instruments.

AwwaRF and EPA jointly funded a project to develop techniques to locate monitoring points in a distribution system, determine appropriate timing and frequency of monitoring, and establish monitoring techniques and relevant water quality parameters. For this purpose, a fully calibrated extended period simulation (EPS) network model hypothetically representing a portion of EBMUD was developed using PipelineNet. This study area network model represents 16 of the 123 pressure zones in the overall EBMUD distribution system. The study area contained 27 tanks, 748 miles of pipes, 62 pumps and 17,997 pipe segments with diameters equal to or greater than 2 inches. Figure 7-18 shows a partial view of the hypothetical network of pipelines.

7.5.2 Model Calibration

The network model was calibrated by comparing the observed (SCADA data) and simulated (PipelineNet model) water level in 25 tanks located in the study area. The primary focus of the calibration was to match the shape of the observed water level in the tanks. The model calibration was performed for a 24-hour time interval using data measured on July 1, 2001. To further enhance calibration, the pump characteristic curves were used to achieve a good comparison between simulated and observed tank levels. The flow value of the characteristic curve was changed as necessary to reflect field conditions. Each pump was operated with time controls.

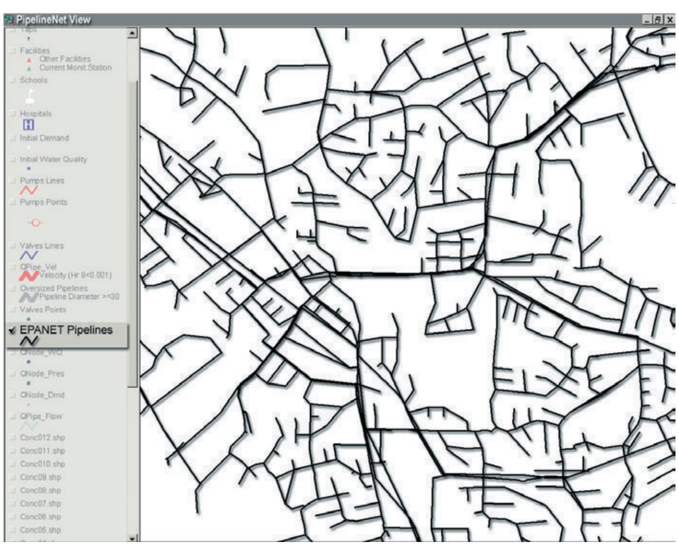


Figure 7-18. Hypothetical Water Distribution System Showing Pipelines.

7.5.3 Monitoring Site Location Methodology

A hierarchical selection process was developed to locate monitoring stations in the distribution system. A three-step approach was employed based on model inputs, outputs, and GIS layers (see Figure 7-19).

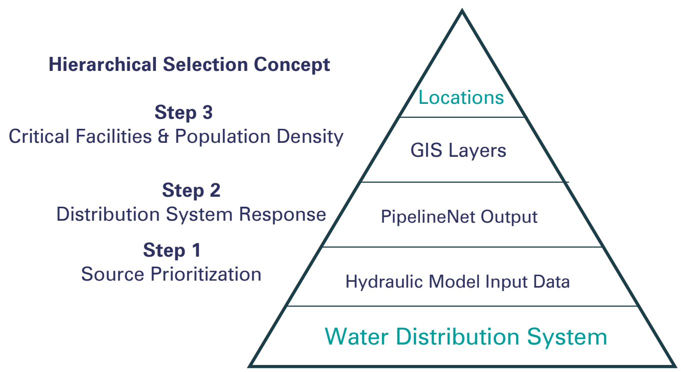


Figure 7-19. Conceptual Diagram Showing the Ranking and Prioritization Methodology.

In the first step (source prioritization factor), all the elements of the water distribution system are assumed to be available for monitoring. This universe is then reduced to a smaller set based on accessibility (location) and amenability (e.g., eliminating dead ends, crosses, tees, junctions with different pipe material) to monitoring. Initially, all nodes are considered available for monitoring and are assigned a score of 1. Subsequently, all the nodes, which are either not amenable or not accessible, are assigned a score of 0. This reduced the number of pipes available for monitoring from 17,997 to 14,938. Therefore, only the 14,938 nodes with a score equal to 1 are considered for Step 2 (described below).

In the second step (distribution system response factor), the PipelineNet model is run to quantify the distribution system response in terms of flow, velocity, and pressure. Concentration of water quality parameters could also be considered in this ranking procedure but was not included as a factor in this case study. Each system response parameter has equal

[1]Registered Trademark of ESRI

weighting and is assigned an initial score of 1 for every pipe. Thereafter, based on the run, the scores are re-assigned values ranging between 1 and 10, where a score of 10 would indicate a higher level of concern. For any given parameter, the user can determine the distribution of scores over the parameter range. For example, a score range of 10 to 1 could be distributed over a flow range of 0.001 to 100 gpm. The PipelineNet model and Bahadur et al. (2003) provide some guidance for assigning scores, but the user can select any score based on the requirements of the analysis.

In the third step (critical facilities and population density factor), user defined buffer zones (polygons) are created around critical facility locations. In addition, areas of low-, medium-, and high-population density are delineated by the creation of polygons. Pipes closest to the critical facilities and/or near high population density areas are assigned a score of 10.

The total score for each pipe based on Steps 2 and 3 are computed. These final scores are linked to the GIS pipeline layer. The user can identify areas where monitoring stations should be placed based on the display of pipes with high scores. Figure 7-20 shows the pipes in the hypothetical network with scores greater than 27 overlaid with critical facility locations. The methodology outlined above for selecting the location of monitoring stations is a subjective procedure that requires input and judgment from water utility personnel. It would not result in a common solution for all distribution systems but can incorporate the specific needs and objectives of the participating water utility.

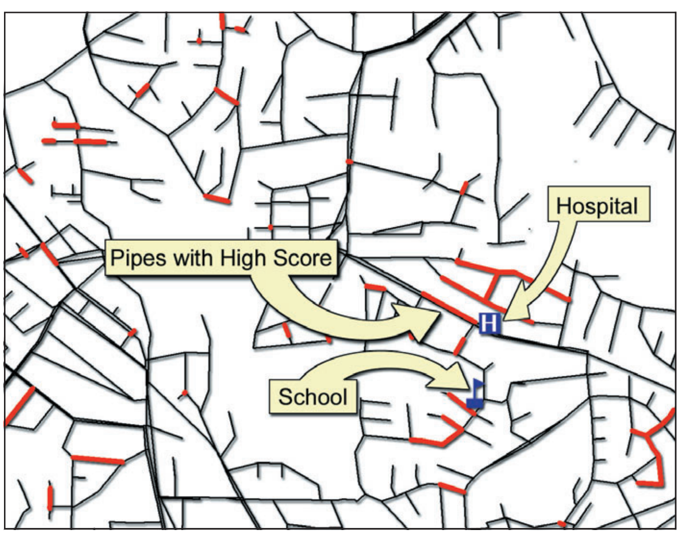


Figure 7-20. Hypothetical System Showing High Score Areas (>27) Overlain with Hospitals and Schools.

7.5.4 Response and Mitigation Tools

In addition to the monitoring site location methodology, three additional tools were developed as part of this case study to enhance the capability of PipelineNet in the areas of emergency response, mitigation, and normal operations. These three tools are briefly described in the following subsections.

7.5.4.1 Consequence Assessment Tool

The consequence assessment tool of PipelineNet provides the ability to quickly identify and quantify the population, infrastructure, and resources at risk from a contaminant event. For a defined contaminated area, this tool can calculate:

- total population at risk,
- number of taps contaminated,
- miles of pipe contaminated,
- total number of hospitals and beds for each hospital, and
- total number of schools and student population.

7.5.4.2 Isolation Tool

The isolation tool of PipelineNet provides the ability to change the status (open or closed) of any pipe in the distribution system. After completing a water quality simulation and examining the contaminant distribution from the event, this tool could be used to close off one or more pipes to control the flow of water. The model would then be re-run, reflecting these new hydraulic conditions, and the output examined to determine if this mitigation step was successful in limiting the area of contamination.

7.5.4.3 Spatial Database Display Tool

PipelineNet's spatial database display tool can overlay the EPANET model output with various geospatial properties. The spatial database display tool of PipelineNet has nineteen criteria to choose from. The users can select any combination(s) of the available criteria. For illustration purposes, Figure 7-21 shows the display of three criteria: oversized pipes (diameter >30 inches), current monitoring locations, and low velocity (velocity < 0.001 FPS) pipes.

7.5.5 Case Summary

The case study demonstrates that the PipelineNet model can be used to perform a variety of practical analyses to locate monitoring systems. Three additional tools are available that enhance PipelineNet's capability in the areas of emergency response, mitigation, and normal operations. However, to effectively utilize the PipelineNet model, the utility must have a calibrated EPS EPANET-based network model and utility-specific GIS data. At the

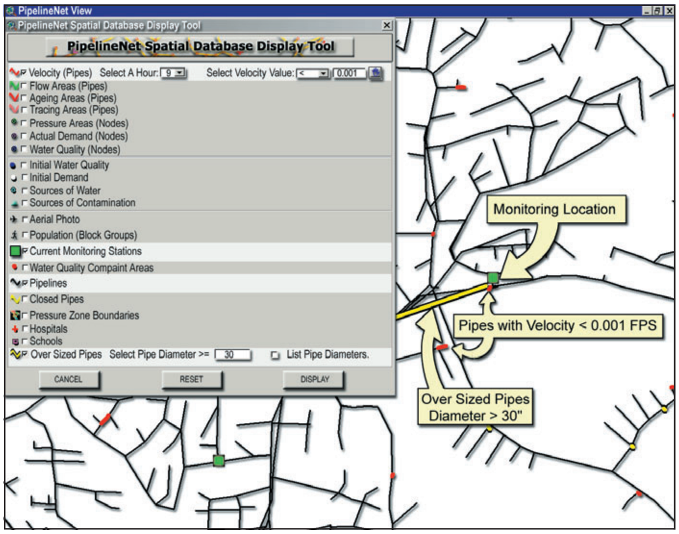


Figure 7-21. Display of Low-Velocity Pipes, Oversized Pipes, and Current Monitoring Stations Using the Spatial Database Display Tool.

time this reference guide was being written, the model is slated to undergo additional enhancements to improve the following: contaminant database, consequence assessment, inclusion of time of travel, conversion from ArcView 3.2 to ArcGIS, and establishment of links to SCADA data.

7.6 Use of Threat Ensemble Vulnerability Assessment (TEVA) Program for Drinking Water Distribution System Security

In response to the increased focus on the vulnerability of drinking water systems to the intentional introduction of chemical, biological, or radiological contaminants, EPA is developing the Threat Ensemble Vulnerability Assessment (TEVA) Program. TEVA, when completed, will be capable of analyzing the vulnerabilities of drinking water distribution systems, measure public health and economic impacts, and design and evaluate threat mitigation and response strategies. TEVA is a probabilistic framework for assessing the vulnerability of a water utility to a variety of contamination attacks. Monte Carlo simulations generate ensembles of scenarios, and statistics are analyzed to explore the feasibility of scenarios, identify vulnerable areas of the water distribution network, and analyze the sensitivity of the model to various parameters.

The TEVA team includes several individuals from various organizations. The key EPA TEVA leads are: Regan Murray, Robert Janke, and Jim Uber.

Key Phrases to Characterize Case Study: hydraulic and water quality modeling, placement of monitors, vulnerability assessment, exposure modeling, contamination assessment, contamination response tools, probabilistic analysis, economic impact assessment, threat mitigation strategies.

7.6.1 TEVA Overview

TEVA incorporates a probabilistic framework for analyzing the vulnerability of drinking water distribution systems. Figure 7-22 outlines the major components of the framework: the stochastic modeling of scenarios, the analysis of potential impacts, and the assessment of threat mitigation strategies. Together, these three components present an integrated view of the vulnerability of a unique distribution system to a wide variety of contamination threats and the potential for a water utility to decrease this vulnerability through a set of mitigation strategies. Preliminary design and implementation has been completed for a core set of components. A longer-term research effort is being planned for the other components.

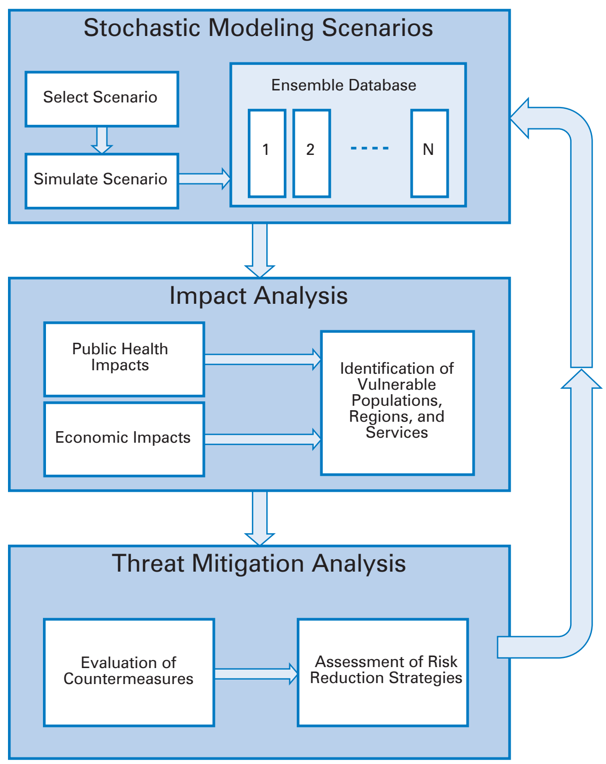


Figure 7-22. Threat Ensemble Vulnerability Assessment Framework.

Without specific intelligence information, one cannot predict exactly how a terrorist group might sabotage a water system. Therefore, TEVA is based on a probabilistic analysis of a large number of likely threat scenarios. While the number of possible variations on scenarios is nearly infinite, the vulnerability of the system can be assessed by selecting a “large enough” set of likely scenarios. TEVA creates a threat ensemble, or a set of contamination scenarios, based on varying the type of contaminant, the amount and concentration of the contaminant, the location of the contaminant introduction into the distribution system, and the duration of the contamination event. The vulnerability of the system is based on an assessment of the entire threat ensemble. The following subsections present an overview of the aforementioned three key modeling elements.

7.6.1.1 Stochastic Modeling

The stochastic modeling element involves three steps: selection of the threat ensemble, simulation of the ensemble, and storage of the output in the ensemble database. The threat ensemble is a collection of scenarios that will be simulated. One scenario may represent, for example, the injection of a 55-gallon drum containing a biotoxin mixture into one node of a particular distribution system with 1,000 nodes. This scenario can be repeated for each of the 1,000 nodes, generating a threat ensemble of 1,000 scenarios. One could vary other parameters, such as contaminant type, quantity, concentration, location, or duration, to generate other threat ensembles. Current work is determining how to best select a large enough threat ensemble in order to accurately assess vulnerability. While a larger number of scenarios will allow for the consideration of more threats, constraints on computation time require that the number of scenarios be minimized.

Next, each scenario in the threat ensemble is simulated using a network hydraulic and water quality model. In TEVA, an EPANET based network model is generated with all necessary data for running the simulations. The EPANET model currently includes first order decay of constituents. Soon to be completed upgrades to EPANET will allow modeling of the fate and transport of multiple dissolved constituents in distribution systems (Uber et al., 2004a). These upgrades will permit the modeling of reactions at the pipe wall and in the bulk flow and enable the inclusion of chemical reaction products, thereby resulting in more accurate estimates of human exposure and health risk. The results of the stochastic modeling of the threat ensemble are stored in the ensemble database, allowing for later analysis of the data in the other components of TEVA.

7.6.1.2 Impact Analysis

The Impact Analysis element uses the data stored in the ensemble database to estimate likely public health impacts and economic impacts. Public health impacts include injuries, disease, illness, and deaths. People can be exposed to contaminants from ingestion of water, inhalation of volatilized chemicals or particles, and/or contact with the skin. Depending on the contaminant, specific dose-response models can be employed to estimate the various health endpoints. For many threat agents, reliable data for such models are lacking, and the ensuing uncertainty in the results must be understood. For contagious diseases, dynamic models of disease transmission also must be included in order to accurately assess health impacts.

Economic impacts include restoration costs (cleanup, treatment, remediation, and decontamination), denial of service costs (providing alternative sources of water), and other costs, such as medical costs (hospitalization, vaccines). Psychological costs related to consumers’ loss of trust in the water supply system are very difficult to estimate. The distribution of impacts will be calculated from the ensemble database, thereby providing an estimate of the expected impacts for the ensemble of threat scenarios.

7.6.1.3 Threat Mitigation Analysis

The Threat Mitigation element explores various mitigation strategies such as the use of early warning systems (sensors and data analysis tools), operational approaches (chlorine boosters, back-up equipment), and emergency response methods (isolation of part of the system, public notification). A range of mitigation strategies can be evaluated with the TEVA simulations using health risk and economic impact analyses to rank and select the best alternative for a set of scenarios (Uber et al., 2004b). This will enable a quantitative analysis of the benefits of implementing various strategies.

7.6.2 Application of TEVA to a Water Distribution System for Optimal Monitoring

The TEVA computational framework (Murray et al., 2004, Uber et al., 2004b, Murray et al., 2005) has been applied to three sizes of distribution systems, each differing in population by approximately one order of magnitude. The results shown for this case study illustrate that TEVA has the potential to help water utilities assess the contaminants to which they are most vulnerable, identify the most vulnerable regions of their distribution systems, and select the most appropriate mitigation strategies for their system.

Many different forms of contamination monitoring systems have been proposed, using water quality sensors, composite or grab sampling, and various numbers and locations of sensors. Any contamination monitoring and surveillance program will be budget constrained. Optimizing the placement of a fixed number of monitoring stations plays an important role in the design of the monitoring system. Selecting the best locations for conducting a routine sampling program to serve as a monitoring and surveillance program for detection of intentional contamination can be considered an integer linear programming problem in which a quantity is optimized subject to a set of constraints (Berry 2005). Formal optimization methods and heuristic methods have been applied to solve such problems. In this case study, a Greedy heuristic algorithm is used to locate monitoring stations, given a defined budget or number of monitoring stations, in order to minimize health impacts. The following is an example of using TEVA to evaluate and optimize (in a limited fashion) the design of a contamination monitoring system for a water distribution system.

7.6.2.1 Simulation Overview

For the purposes of this analysis, an "all-pipes" EPANET network model for the sample distribution system was generated. There are 1,062 miles of pipe represented in this network model. This system contains approximately 12,000 nodes, has an average daily demand of approximately 20 million gallons, and an estimated population of 130,000. Approximately 6,000 potential sampling locations were selected randomly from the nearly 12,000 nodes by considering that each node had a 50 percent probability of inclusion. This pruning was used to make the problem less computationally intensive and emphasize that an optimal placement of monitoring stations will likely be difficult given legal, financial, or logistical constraints for placing and managing monitoring stations.

To simulate a contamination scenario, many parameters must be specified, including characteristics of the contaminant, the contaminant-introduction scenario, and the consumption patterns of the population. In order to represent the range of possible parameter values, the TEVA computational framework uses simulation to vary parameters, such as contaminant type, quantity, concentration, location, rate, or duration, to generate threat ensembles (collections of many threat scenarios) which collectively can be analyzed for health impact statistics. All nodes (a grouping of service connections) in the distribution system are considered equally likely introduction points. Biological and chemical contaminants can be considered, and contaminant introduction can last from a few minutes to hours to days. For the purposes of this analysis, a biological agent was considered as the contaminant and the introduction duration was 24 hours, at a rate of 8.675 liters per hour, and a mass rate of 1.45×10^{11} organisms per minute.

The health impacts are affected by factors such as dose-response relationships, lethal doses, time-to-onset of symptoms, time for effective medical intervention, and the time delay for transporting and analyzing samples in laboratories. Health impacts to a population will increase with an increase in the time required to implement an effective response for a known contamination event. Considering these factors, modeling and simulation analyses, such as those presented here, must be performed on a contaminant specific basis. The health impact statistics can include mean infections/illnesses or mean fatalities. Infection/illness is a function of the dose of organisms or contaminant ingested and the probability of illness caused by that dose, as determined from the contaminant dose-response curve (in this case, *Salmonella*). Mean infections/illnesses are statistically determined from the probabilistic analysis of all threat scenarios. Maximum infections resulted from introduction at the node delivering the maximum health impacts. Although there was one worst case node, there were approximately 60 threat scenarios (contaminant introduction locations), which delivered at least 50 percent of the maximum lethality. The maximum number of lethalities was approximately 13,000.

In this TEVA-simulated analysis, the benefits of two composite grab sampling programs were evaluated (daily and every 48 hours) and compared to the benefits provided by a system of real-time (inline, contaminant specific) sensors. The benefits of the sampling or monitoring programs are measured by the hypothetical reduction in public health impacts. In this analysis, sample location designs are based on minimizing the mean number of fatalities for 2 sampling frequencies: 24 hours and 48 hours. Six sampling/sensor station placement scenarios were evaluated in this analysis: 5, 10, 15, 20, 30, and 40 locations for each program. The Greedy algorithm used in this analysis will provide an optimal solution for minimizing public health impacts.

For the purposes of this analysis, the grab samples are considered to be filtered samples. Filtered samples represent composited samples that have been collected and concentrated through a filtration device to better enable the collection and analysis of biological organisms. Also, real-time water quality monitors are assumed to detect chemical contaminants or biological organisms through the change in water quality, such as determined by the reduction of chlorine residual or increase in turbidity. These real-time and sampling-based analyses are considered ideal, meaning that detection limits were zero and the biological contaminant was always detected.

7.6.2.2 TEVA Analysis Approach

This analysis considers attacks at every non-zero demand node, totaling approximately 10,000. Only non-zero demand nodes are considered because they represent service connections that are using water from the distribution system on a regular basis and, therefore, could possibly be used for contaminant introduction. Statistically analyzing the approximately 10,000 threat scenarios provides an estimate of the hypothetical health impacts in terms of average health impacts (e.g., fatalities or illnesses) and maximum impacts. Average impacts could be expected to result if a saboteur had no knowledge of where best to attack and simply randomly chose a node location for contaminant introduction. Maximum health impacts correspond to a relatively small set of contaminant introduction node locations (threat scenarios) that maximize health impacts to the associated receptors.

The contaminants were modeled as tracers, i.e., free of hydrolysis, chlorination, pipe wall, or biofilm reactions, which would largely decrease the contaminant's effectiveness in causing harm to public health (an extended version of EPANET is undergoing testing to allow multi-species modeling of contaminants). Contaminants are modeled using a mass injection rate, zero volume added, which consequently does not influence the hydraulic properties of the network, i.e., flow increase, decrease, or reversal of flow.

Hydraulic and water quality simulations were run for 192 hours. The disease-causing agent or contaminant was considered to be a hypothetical, biological contaminant that is expected to cause infection (50 percent of the time) in an adult when 10,000 or more of the organisms are ingested. The incubation period was assumed to be 24 hours, and the time for effective treatment was 48 hours after the onset of symptoms. After 72 hours, people either recovered or died. A sigmoidal dose-response curve was assumed for the ingestion of organisms with the untreated fatality rate at 16 percent of those infected.

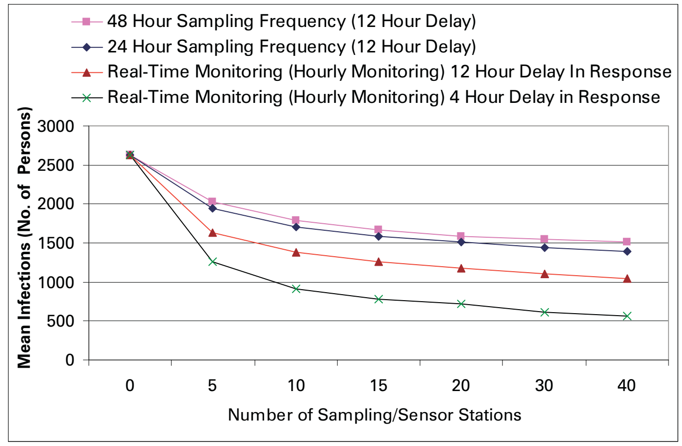


Figure 7-23. Comparison of 24-Hour, 48-Hour, and Real-Time, Continuous Contamination Monitoring Systems for the Reduction in Mean Infections for a 24-Hour Contaminant Attack.

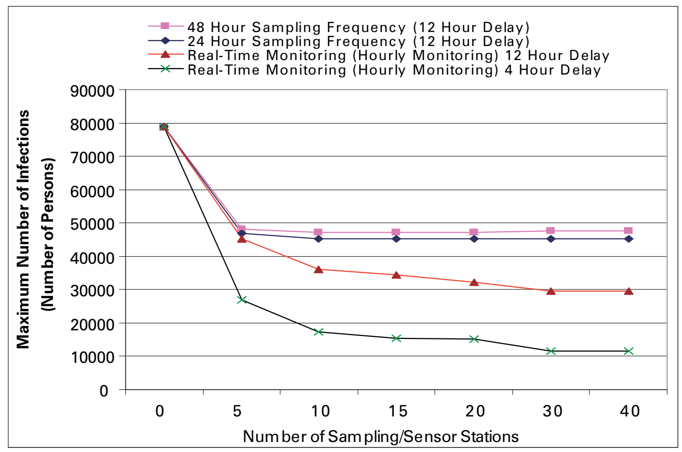


Figure 7-24. Comparison of 24-Hour, 48-Hour, and Real-Time, Continuous Contamination Monitoring Systems for the Reduction in the Maximum Number of Infections for a 24-Hour Contaminant Attack.

7.6.2.3 TEVA Analysis Results

Figure 7-23 compares the reduction in mean infections provided by a routine 24-hour filtered sampling program, a routine 48-hour filtered sampling program, and a real-time, continuous, monitoring program. Similarly, Figure 7-24 compares the reduction in the maximum number of infections of the same 3 monitoring programs. Again, this scenario assumes contamination by an individual who understands distribution systems and has the knowledge and resources to determine the maximum impact location(s).

It is assumed that the computer simulations and the monitoring/surveillance programs are successful in reducing public health impacts by preventing further consumption after detection. The results show that there is not a significant difference between daily (24-hour) sampling and 48-hour sampling in terms of reducing health impacts. As expected, the continuous monitoring program detects the incident much earlier than the daily sampling program. Figure 7-23 shows an 80 percent reduction in mean infections is achieved using 40 real-time monitors, as compared to having zero monitors. The results show that it is important that the real-time program be integrated with a response protocol. A comparison of the two real-time, continuous monitoring cases (12-hour delay versus 4-hour delay in notifying the public) illustrates the importance of response time. Shortening the time needed to implement an effective response to reduce further exposure is critical for reducing the number of additional infections.

The results of this case study also illustrate that, for this distribution system, strategically placing just 5 or 10 sampling/sensor stations as part of a monitoring and surveillance system can have a significant effect on reducing potential health impacts from intentional contamination.

7.6.3 Case Summary

TEVA is an integrated system intended to provide the capability for analyzing the vulnerabilities of drinking water distribution systems and to measure the public health and economic impacts. TEVA can be used to design and evaluate threat mitigation and response strategies related to events of intentional introduction of chemical, biological, or radiological contaminants into drinking water networks. Monte-Carlo simulations generate ensembles of threat scenarios to identify vulnerable areas of the water distribution network. TEVA is based on the use of an all pipes EPANET network model.

7.7 Field Testing of Water-Distribution Systems in Support of an Epidemiologic Study

This case study is focused on the use of information collected as part of field studies to assist in the calibration of a hydraulic and water quality model of a distribution system. The information presented in this section is based on an ongoing investigation by the ATSDR at the U.S. Marine Corps Base, Camp Lejeune, NC (Camp Lejeune). These data are being collected to support an ongoing epidemiologic study at Camp Lejeune. The resulting calibrated model is needed to perform a historical reconstruction of the water system for the period of interest. This case study highlights the field methodologies employed to generate the information proposed for use in the calibration of the model.

Key Phrases to Characterize Case Study: field studies, historical reconstruction, hydraulic and water quality modeling, model calibration.

7.7.1 Case Study Overview

ATSDR is conducting an epidemiologic study to determine if there is an association between exposure to contaminated drinking water and birth defects among children of women who lived at Camp Lejeune while they were pregnant between 1968 and 1985. Because of the paucity of historical water distribution system operational data, information based on the operation of present-day water distribution systems will be used for historical reconstruction. Present-day system operations will be modeled using water-distribution system models. To calibrate the models against hydraulic and water quality parameters, field testing is being performed to gather data and information on hydraulic, fate and transport, and operational characteristics of the water distribution systems (Maslia et al., 2005; Sautner et al., 2005). The specific field activities are discussed later in Section 7.7.2.

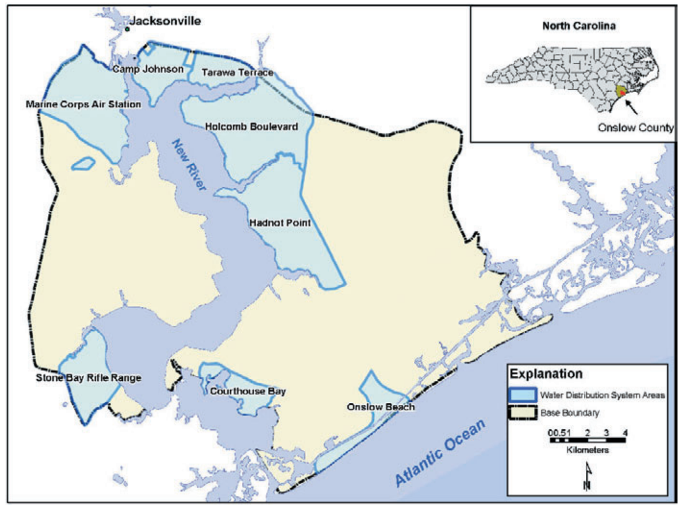


Figure 7-25. Water Distribution Systems Serving U.S. Marine Corps Base, Camp Lejeune, NC.

Camp Lejeune encompasses an area of about 164 square miles, and is located in Jacksonville, Onslow County, North Carolina, bordering the Atlantic Ocean. The focus of the epidemiologic study is on exposure from water-distribution systems that historically served the military base's housing—Camp Johnson, Tarawa Terrace, Holcomb Boulevard, and Hadnot Point (see Figure 7-25). Presently, there are two operating water treatment plants (WTPs) that provide water for the distribution systems of interest to the epidemiologic study: (1) the Holcomb Boulevard WTP that services the Camp Johnson, Tarawa Terrace, and Holcomb Boulevard areas of the distribution system, and (2) the Hadnot Point WTP that services the Hadnot Point area of the distribution system. Hadnot Point was the original WTP and at one time, serviced the entire base. The Holcomb Boulevard WTP presently services the rest of the military housing areas. A third plant, the Tarawa Terrace WTP, historically serviced the Tarawa Terrace and Camp Johnson areas, but this plant was shut down and replaced by a ground storage tank at Tarawa Terrace that receives water directly from the Holcomb Boulevard WTP.

System pressures range from about 55–68 psi throughout the distribution systems. As topography is very flat, ranging from sea level to less than 40 ft, hydraulic heads range from 140–160 ft resulting in a very mild hydraulic gradient. There are nine elevated storage tanks in the Holcomb Boulevard and Hadnot Point WTP service areas. The range in water level fluctuation for the elevated storage tanks is small; generally 1–6 ft. Excellent mapping information and a detailed GIS provide good information on the location and characteristics of the water system facilities. SCADA data are available that provide continuous

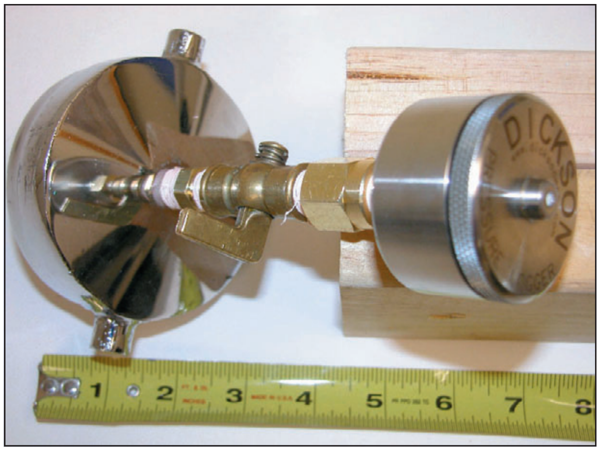


Figure 7-26. Continuous Recording Pressure Logger Mounted on Brass Shutoff Valve and Hydrant Adapter Cap Used for Fire-Flow and C-Factor Tests.

information on plant discharges and tank water levels. However, individual buildings and residences are not metered.

7.7.2 Field Work

A variety of field activities are being performed to provide a better understanding of the operation of the water system and to provide information that can be used to calibrate a detailed water distribution system model. To date, these activities have included:

- conducting C-factor and fire-flow tests,
- recording system pressures and storage tank water levels over time,
- tracer and associated travel time tests, and
- recording continuous flow information at key locations.

Several of these field activities were performed in tandem in order to provide an integrated understanding of the system operation and performance.

7.7.2.1 C-Factor and Fire-Flow Tests

C-factor tests and fire-flow tests were conducted in August 2004 at various sites at Camp Lejeune. Continuous pressure loggers (Figure 7-26) set to record pressure at 1-minute intervals were attached to hydrants. Standard analog pressure gages were also used as backup during the tests. Hydrant flows were measured using pitot gages installed on hydrants that were flowed during the tests. One of the pitot gages was integrated with a diffuser and cage to both diffuse the flow from the hydrant and to trap any solids to prevent damage from the flow (see Figure 4-6 in Chapter 4). The other pitot gage was a standard gage attached to the hydrant. Standard C-factor testing procedures were used to measure data needed to calculate the Hazen-Williams C-factors for eight sections representing a variety of pipe materials and diameters.

Fire-flow tests are frequently used in the process of calibrating a hydraulic water distribution system model. One or more hydrants are opened and flowed to increase flows in the distribution system in the vicinity of the hydrants. Since friction losses increase exponentially, the higher flows can result in a significant lowering of the hydraulic grade line (HGL). In calibrating the model, the model is applied under the flow and operational conditions experienced during the fire-flow test and the pressures or hydraulic grade line observed in the field are compared to the model results. If there are significant differences between the model and field results, adjustments are made in model parameters in order to reduce the differences or calibrate the model. In the simplest configuration, a single hydrant is flowed and pressure read at another single hydrant. An alternative approach was used at Camp Lejeune to improve the labor efficiency and to collect more data. Continuous recording pressure gages were installed at up to six hydrants in the area being tested. Additionally, pitot gages were installed on two hydrants that were designated as hydrants to be flowed. Pressure was measured under several conditions: (1) static conditions at start of testing, (2) while each of the two hydrants was flowed separately, (3) while both hydrants were flowed together, and (4) static conditions at the end of testing. The results of such a test at one site are shown in Figure 7-27 and Table 7-2.

7.7.2.2 Tracer Test and Continuous Measurements

A field test was conducted May 24–27, 2004, in the Hadnot Point (Camp Lejeune) distribution system consisting of three activities: (1) injecting liquid $CaCl_2$, 35 percent by weight, into the transmission main on the distribution system side of the WTP to achieve an elevated conductance and chloride concentration, and recording conductivity and chloride concentration using continuous recording

Figure 7-27. Location of Fire Hydrants Used in Fire-Flow Test at Site H02.

Table 7-2. Field Data Collected During Fire-Flow Test at Site H02

	FF-H02-P0 Pressure, Psi	FF-H02-P1 Pressure, Psi	FF-H02-P2 Pressure, Psi	FF-H02-P3 Pressure, Psi	FF-H02-Q1 Flow, gpm	FF-H02-Q2 Flow, gpm
Static case (start)	53.1	50.7	56.2	52.6	0	0
Hydrant 1 flowed	41.4	37.3	46.8	42.9	773	0
Hydrant 1+2 flowed	29.7	24.5	36.5	32.7	631	579
Hydrant 2 flowed	43.9	40.7	48.1	44.1	0	747
Static case (end)	53.5	51.2	56.5	52.9	0	0

1.0 psi = 6.8948 kPa; 1 gpm = 0.0639 L/S

water-quality monitoring data loggers, (2) injecting a sodium fluoride solution into the transmission main to achieve an elevated fluoride concentration and monitoring fluoride concentration in the distribution system, and (3) monitoring distribution system pressures with continuous recording data loggers attached to selected hydrants and flows and tank water levels from SCADA data. In addition to continuously recording tracer concentrations and conductivity, grab samples were collected for quality assurance and quality control (QA/QC) purposes. Samples were analyzed at the Hadnot Point WTP by ATSDR staff and then also shipped to the Federal Occupational Health (FOH) environmental laboratory in Chicago, Illinois, for analysis. Twenty-seven hydrants were selected in the Hadnot Point area as monitoring locations. For monitoring conductivity and chloride and fluoride concentrations, nine hydrants were equipped with the Horiba W-23XD dual probe ion detector (Figure 7-28). For monitoring conductivity, nine hydrants were equipped with the Horiba W-21XD single probe ion detector, thus providing a total of 18 monitoring locations for continuously recording conductivity data. For pressure measurements, nine hydrants were equipped with continuous recording Dixon PR300 pressure data loggers (Figure 7-26).

Results from the chloride injection were used to estimate arrival times of the tracer at different locations throughout the Hadnot Point WTP area. Of special interest are the extremely long arrival times—in excess of 16 hours—in the northwestern part of the of the Hadnot Point WTP area (Figure 7-29, loggers C01, C02, and F01). Additionally, a comparison of arrival times of the calcium chloride tracer at logger location C04 with arrival times at loggers F04, F05, and F02, led investigators to consider that there may have been undocumented closed valves in the distribution system (the closed valves did not affect C-factor measurements). Post-test field verification by water utility staff confirmed the locations of closed valves, as indicated by the "•" symbol in Figure 7-29.

7.7.3 Additional Test Procedures

A second tracer test was conducted in the Holcomb Boulevard WTP area in September-October 2004. In this test, the normal fluoride feed was turned off for a period of a week and then turned back on and

Figure 7-28. Horiba W-23XD Dual Probe Ion Detector Inside Flow Cell.

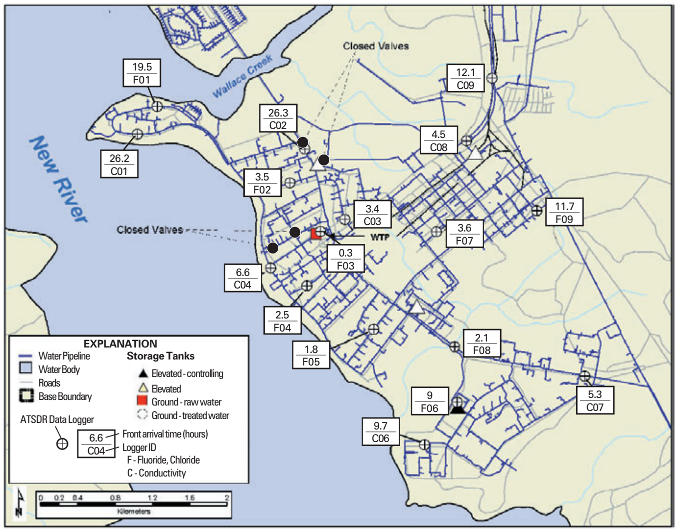


Figure 7-29. Arrival Times of the Calcium Chloride Tracer at Monitoring Locations in Hadnot Point WTP Area, May 25, 2004.

monitored for another week. Nine locations in the distribution system were equipped with the Horiba W-23XD continuous recording, dual probe ion detector data logger. Minimal labor was required in support of this test. Additionally, 16 magnetic flow meters have been installed throughout the system and will be used in conjunction with future tracer tests to provide additional calibration information.

7.7.4 Case Study Summary

This case study presents results from preliminary field-test activities used to gather hydraulic and water quality data at Camp Lejeune. Field tests to date have included: (a) recording system pressures and storage tank water levels, and (b) conducting C-factor, fire-flow, tracer, and travel time tests. The test data are being used to assist with hydraulic and water quality model calibration. They are also being used to plan and carry out a more refined, detailed field test of water distribution systems serving military base housing. These activities will assist in providing much-needed model parameter data for calibrating models of the present-day water distribution system. The present-day models are needed as a first step in reconstructing historical operations during the period between 1968 and 1985, as part of an ongoing epidemiologic study of childhood diseases at Camp Lejeune.

7.8 Chapter Summary

The case studies presented in this chapter illustrate the various ways in which the tools presented in this reference guide can be used. The case studies also demonstrate that, for each application, a specific analysis methodology needs to be developed depending upon the study objectives and available data.

References

Aral, M. M., J. Guan, M.L. Maslia, J.B. Sautner, R.E. Gilig, J.J. Reyes, and R.C. Williams, "Optimal Reconstruction of Historical Water Supply to a Distribution System: A. Methodology." *Journal of Water and Health*, 2(3):123-–136, 2004a.

Aral, M. M., J. Guan, M.L. Maslia, J.B. Sautner, R.E. Gilig, J.J. Reyes, and R.C. Williams. Optimal Reconstruction of Historical Water Supply to a Distribution System: B. Applications." *Journal of Water and Health*, 2(3):137–156, 2004b.

Bahadur, R., W.B. Samuels, and J. Pickus. *Case Study for a Distribution System Emergency Response Tool.* Published by AWWA Research Foundation. 2003.

Berry, M. Letter (unpublished) to Steve Jones, ATSDR Region 2 from Michael Berry, Environmental Health Service, New Jersey Department of Health. August 31, 1995.

Berry, J., L. Fleischer, W. E. Hart, C. A. Phillips, and J. P. Watson. "Sensor Placement in Municipal Water Networks." *Journal of Water Planning and Resources Management*. ASCE, vol. 131. 2005.

Clark, R.M., E.E. Geldreich, K.R. Fox, E.W. Rice, C.H. Johnson, J.A. Goodrich, J.A. Banik, F. Abdesaken. "Tracking a *Salmonella* serovar typhimurium outbreak in Gideon, Missouri: Role of Contaminant Propagation Modeling." *Journal of Water Supply Research and Technology – Aqua*. Vol.45, No. 4, pp 171-183. 1996.

EPA. *Stage 2 Disinfectants and Disinfection Byproducts Rule, Initial Distribution System Evaluation. Draft Guidance.* 2003.

Fennel, H., D.B. James, and J. Morris. "Pollution of a Storage Reservoir by Roosting Gulls." *Journal of Society of Water Treatment Exam*. 23:5-24. 1974.

Geldreich, E.E., K.R. Fox, J.A. Goodrich, E.W. Rice, R.M. Clark, and D.L. Swerdlow. "Searching for a Water Supply Connection in the Cabool, Missouri Disease Outbreak of Escherichia coli 0157:H7." *Water Research* 84:49-55. 1992.

Grayman, W.M., R.M. Clark, and R.M. Males. "Modeling Distribution System Water Quality: Dynamic Approach." *Journal of Water Resources Planning and Management*, ASCE, 114(3). May 1988.

Grayman, W.M., R.M. Clark, B.L. Harding, M.L. Maslia, and J. Aramini. "Reconstructing Historical Contamination Events." *Water Supply Systems Security*. Editor L. Mays. Published by McGraw-Hill, NY. pp10.1-10.55. 2004.

Grayman, W.M., A. Ostfeld, and E. Salamons. "Red Team-Blue Team Exercise for Locating Monitors in Distribution Systems." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

Harr, J. *A Civil Action.* Published by Random House. NY. 1995.

Maslia, M. L., J. B. Sautner, and M. M. Aral. "Analysis of the 1998 Water-Distribution System Serving the Dover Township Area, New Jersey: Field-Data Collection Activities and Water-Distribution System Modeling." Agency for Toxic Substances and Disease Registry, Atlanta, GA. 2000.

Maslia, M. L., J. B. Sautner, M. M. Aral, R. E. Gillig, J. J. Reyes, and R. C. Williams. "Historical Reconstruction of the Water-Distribution System Serving the Dover Township Area, New Jersey: January 1962–December 1996." Agency for Toxic Substances and Disease Registry, Atlanta, GA. 2001.

Maslia, M. L., J. B. Sautner, C. Valenzuela, W. M. Grayman, M. M. Aral, and J.W. Green, Jr. "Use of Continuous Recording Water-Quality Monitoring Equipment for Conducting Water-Distribution System Tracer Tests: The Good, the Bad, and the Ugly." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

Murphy, P.J. "Water Distribution in Woburn, Massachusetts." The Environmental Institute, University of Massachusetts. Amherst, MA. 1986.

Murray, R., R. Janke, and J. Uber. "The Threat Ensemble Vulnerability Assessment Program for Drinking Water Distribution System Security." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Salt Lake City, UT. 2004.

Murray, R., J. Uber, and R. Janke. "A Model for Estimating the Acute Health Impacts Resulting from Consumption of Contaminated Drinking Water." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

NJDHSS. "Case-Control Study of Childhood Cancers in Dover Township (Ocean County), New Jersey." NJDHSS, Division of Epidemiology, Environmental and Occupational Health. Trenton, NJ. 2003.

Ostfeld A., and E. Salomons. "A Stochastic Early Warning Detection System Model for Drinking Water Distribution Systems Security." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Salt Lake City, UT. 2004.

Rossman, L.A., R.M. Clark, and W.M. Grayman. "Modeling Chlorine Residuals in Drinking Water Distribution Systems." *Journal of Environmental Engineering*, 120(4):803-820. July/August 1994.

Sautner, J. B., M. L. Maslia, C. Valenzuela, W. M. Grayman, M. M. Aral, and J. W. Green, Jr. "Field Testing of Water-Distribution Systems at U.S. Marine Corps Base, Camp Lejeune, North Carolina, in Support of an Epidemiologic Study." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Anchorage, AK. 2005.

Uber, J., L. Rossman, and F. Shang. "Extensions to EPANET for the Fate and Transport of Multiple Interacting Chemical or Biological Components." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Salt Lake City, UT. 2004a.

Uber, J, R. Janke, R. Murray, and P. Meyer. "A Greedy Heuristics Model for Locating Water Quality Sensors in a Water Distribution System." Proceedings, ASCE-EWRI World Water & Environmental Resources Congress, Salt Lake City, UT. 2004b.

Walski, T.M., D.V. Chase, D.A. Savic, W. Grayman, S. Beckwith, and E. Koelle. *Advanced Water Distribution Modeling and Management.* Haestad Press, Waterbury, CT. 2003.

United States
Environmental Protection
Agency

Office of Research and Development
National Risk Management
Research Laboratory
Cincinnati, OH 45268

EPA/600/R-06/028
December 2005
www.epa.gov