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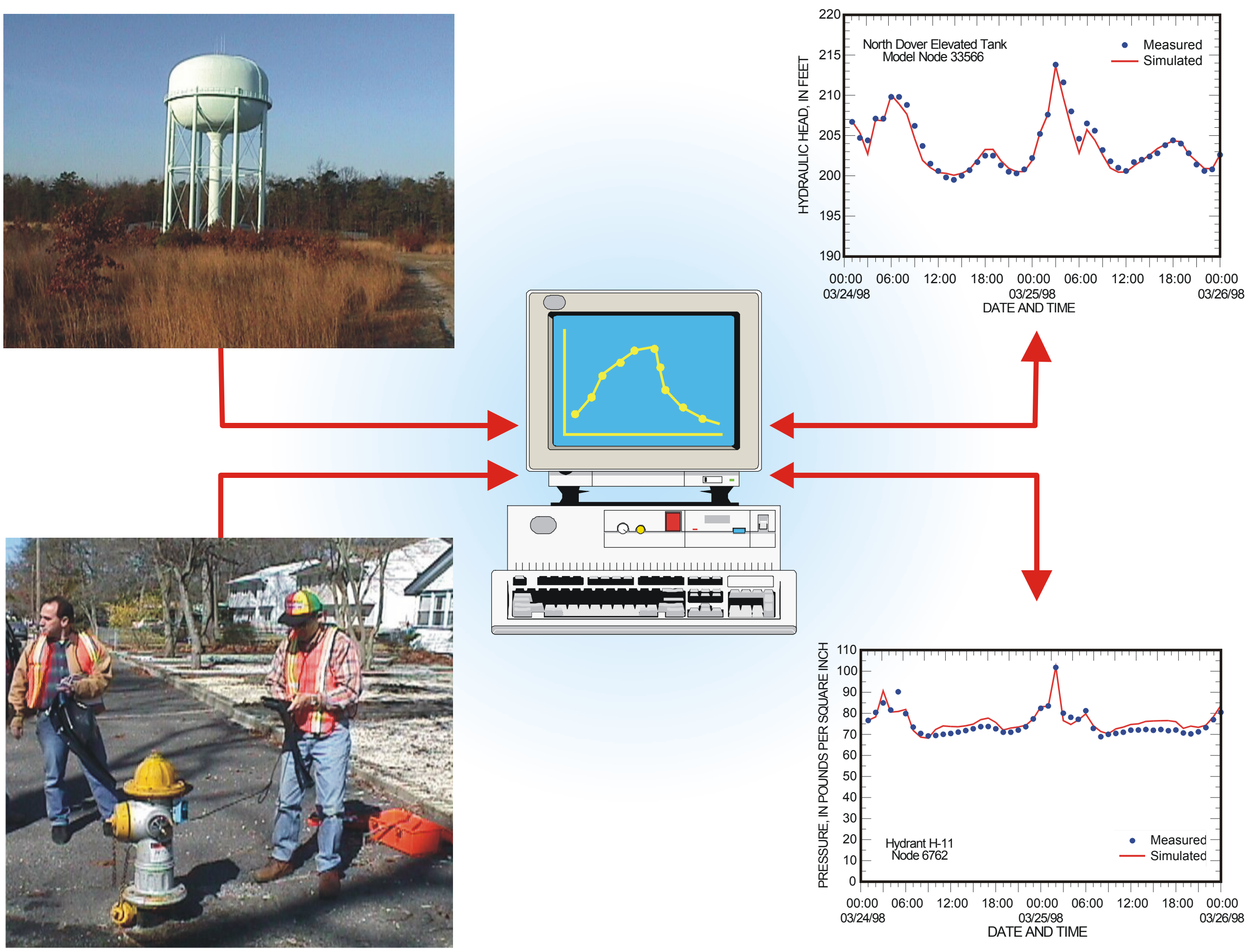
Analysis of the 1998 Water-Distribution System Serving the Dover Township Area, New Jersey:
Field-Data Collection Activities and Water-Distribution System Modeling



Agency for Toxic Substances and Disease Registry
U.S. Department of Health and Human Services
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Front cover illustration:

Diagram showing the water-distribution system modeling and calibration process

Top Left: Photograph of North Dover elevated storage tank, Dover Township, New Jersey; ***Bottom Left***: Photograph of ATSDR staff querying pressure data logger for parameters of date, time, and pressure. ***Top Right***: Graph of measured and simulated water levels for the North Dover Tank; ***Bottom Right***: Graph of measured and simulated pressures for test hydrant H-11; and ***Center***: Data on storage tank water levels and test hydrant pressures are input to the EPANET water-distribution system model. Results of model simulations from the computer program are used to produce the graphs on the right. Comparison is made between measured data (solid circles on graphs) and simulated results (solid line on graphs). After evaluating results, modifications may be made to model parameters and additional simulations are conducted to improve the match between measured and simulated values.

Analysis of the 1998 Water-Distribution System Serving the Dover Township Area, New Jersey:
Field-Data Collection Activities and Water-Distribution System Modeling

By Morris L. Maslia, Jason B. Sautner, and Mustafa M. Aral

Prepared in coordination with:

New Jersey Department of Health and Senior Services
New Jersey Department of Environmental Protection
Ocean County Health Department
Citizens Action Committee on Childhood Cancer Cluster and
United Water Toms River, Inc.

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FOREWORD

The Agency for Toxic Substances and Disease Registry (ATSDR) and the New Jersey Department of Health and Senior Services (NJDHSS) are investigating the elevated incidence of childhood cancers in Dover Township, Ocean County, New Jersey. In 1996, ATSDR and NJDHSS developed a Public Health Response Plan in cooperation with the Ocean County Health Department and the Citizen's Action Committee on Childhood Cancer Cluster. The plan outlined a series of public health activities including an updating and detailed re-evaluation of childhood cancer incidence statistics, and assessments of potential environmental exposures in the community. In 1997, ATSDR and NJDHSS determined that an epidemiologic study was warranted, and that the study would include assessments of the potential for exposure to specific drinking water sources.

ATSDR developed a workplan in February 1997 to reconstruct historical characteristics of the water-distribution system serving the Dover Township area by using water-distribution system modeling techniques. The model chosen by ATSDR for this effort, EPANET, is available in the public domain and is described in the published scientific literature. To test the reliability of model simulations, investigators need historical or present-day data with which to compare model results. Lacking such data, investigators initiated a field-data collection effort to obtain pressure measurements, storage-tank levels, and system operation schedules (the on/off cycling of pumps and wells) during winter-demand (March 1998) and peak-demand (August 1998) operating conditions. Using these data, ATSDR investigators calibrated the water-distribution system model to present-day (1998) conditions.

This report, therefore, presents and describes the following: (1) data gathered during field tests conducted in March and August 1998, (2) the development, calibration, and testing of the water-distribution system model for 1998 conditions, (3) a water-quality simulation of a naturally occurring conservative element, barium, to further test the reliability of the model calibration, and (4) the simulation of the proportionate contribution of water from points of entry to various locations throughout the distribution system for 1998 conditions.

CONTENTS

CONTENTS–CONTINUED

SUPPLEMENTAL DATA

[Supplemental data are under separate cover]

CONTENTS–CONTINUED

LIST OF ILLUSTRATIONS

[Plates are in pocket]

CONTENTS–CONTINUED

LIST OF ILLUSTRATIONS–CONTINUED

LIST OF ILLUSTRATIONS–CONTINUED

CONTENTS–CONTINUED

LIST OF TABLES

Table

CONTENTS–CONTINUED

CONVERSION FACTORS FOR UNITS

Factors for converting inch-pound units to the International System (SI) of units are given below:

Multiply	By	To obtain
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
foot (ft)	0.3048	meter (m)
gallon (gal)	3.785	liter (L)
gallon per minute (gpm)	0.06309	liter per second (L/s)
inch (in.)	2.54	centimeter (cm)
mile (mi)	1.609	kilometer (km)
million gallons (Mgal)	0.003785	cubic meter (m^3)
million gallons per day (MGD)	0.04381	cubic meter per second (m^3/s)
part per billion (ppb)	1.00	microgram per liter (μg/L)
pound per square inch (psi)	6.8948	kilopascal (kPa)

GLOSSARY OF ABBREVIATIONS

Abbreviations and their definitions used throughout this report are listed below.

Abbreviation	Definition
ATSDR	Agency for Toxic Substances and Disease Registry
CNS	Central nervous system
DEM	Digital elevation model
EPA	U.S. Environmental Protection Agency
EPANET	A water-distribution system model developed by the EPA
EPS	Extended period simulation
GPS	Global positioning system
NJDHSS	New Jersey Department of Health and Senior Services
NPL	National Priorities List
PC	Personal computer
PVC	Polyvinyl chloride (plastic) pipe
SAN	Styrene-acrylonitrile trimer
SCADA	Supervisory control and data acquisition
SVOC	Semi-volatile organic compound
TCE	Trichloroethylene
USGS	U.S. Geological Survey
VOC	Volatile organic compound
UWTR	United Water Toms River, Inc.

SEA LEVEL DEFINITION

In this report, "sea level" refers to the National Geodetic Vertical Datum of 1929 (NGVD of 1929)–a geodetic datum derived from a general adjustment of the first-order level net of both the United States and Canada, formerly called "Sea Level Datum of 1929."

ANALYSIS OF THE 1998 WATER-DISTRIBUTION SYSTEM SERVING THE DOVER TOWNSHIP AREA, NEW JERSEY: FIELD-DATA COLLECTION ACTIVITIES AND WATER-DISTRIBUTION SYSTEM MODELING

By Morris L. Maslia, Jason B. Sautner, and Mustafa M. Aral

ABSTRACT

The New Jersey Department of Health and Senior Services (NJDHSS) and the Agency for Toxic Substances and Disease Registry (ATSDR) are conducting an epidemiologic study of childhood leukemia and nervous system cancers that occurred in the period 1979 through 1996 in Dover Township, Ocean County, New Jersey. The epidemiologic study is exploring a wide variety of possible risk factors, including environmental exposures. ATSDR and NJDHSS have determined that completed human exposure pathways to groundwater contaminants have occurred in the past (through private and community water supplies) in some parts of the community. Because of this, ATSDR is developing a water-distribution model using the EPANET software to assist with environmental exposure assessment in the epidemiologic study. Results obtained from the model will be considered as one of the risk factors in the epidemiologic investigation.

As an important first step, the model was calibrated to the present-day (1998) water-distribution system characteristics. Pressure data were gathered simultaneously at 25 system hydrants using continuous pressure recording data loggers during field tests in March and August 1998. Data for storage-tank water levels, system demand, and pump and well status (on/off cycling) were also obtained. Data collected during the March 1998 test represent low, winter-time demand of 7.6 million gallons per data (MGD) and data collected during the August 1998 test represent peak-demand of 16.1 MGD. Measured pressure data and model simulations indicate that for the water-distribution system, pressures range from 40 pounds per square inch (psi) to slightly more than 100 psi for both tests.

The model network consists of 16,071 pipe segments ranging in diameter from 2 inches to 16 inches, and 14,987 junctions (nodes). The model was calibrated using the present-day (1998) data collected during winter-demand conditions of March 1998. The calibration was then tested against data collected under peak-demand conditions during August 1998. The absolute difference between measured and simulated hourly average pressures (pressure difference) for all measurement locations for March 1998 ranges 1.4-5.3 psi, and for August 1998, ranges 2.9-6.6 psi. For the March 1998 test and simulation, analysis indicates that 90% of hourly data at all test locations have an absolute pressure difference of approximately 5 psi or less. For the August 1998 test and simulation, analysis indicates that 90% of hourly data at all test locations have an absolute pressure difference of

approximately 7.5 psi or less. These small pressure differences support the assertion that the model is calibrated and an acceptable and reliable representation of water-distribution system conditions during 1998.

As further evidence of the reliability of the model calibration, a simulation of the transport of a naturally occurring conservative element, barium, was conducted and compared with data gathered at 21 schools and 6 points of entry to the water-distribution system for March and April 1996. Measured concentrations of barium ranged 13-51 micrograms per liter (μg/L). Comparison of measured and simulated barium concentrations at the 21 school locations indicates a difference ranging 0.2-12.4 μg/L, which results in a mean relative difference of 13.6% with a range of 0.6-25.6%. Additional analyses comparing measured and simulated concentrations of barium show a geometric bias of 0.93, indicating a slight under prediction by the model (1.00 indicates perfect agreement), and a correlation coefficient of 0.81, indicating a high correlation between measured concentrations and simulated values. Therefore, this water-quality simulation is further evidence that the model is reasonably calibrated and an acceptable representation of the present-day water-distribution system characteristics.

The calibrated model of the water-distribution system makes it possible to conduct trace analyses for each point of entry (well fields) to the distribution system. These analyses provide an estimate of the percentage of water that any location of interest receives from the 8 points of entry to the distribution system. The results are presented in a series of 10 maps, a graph, and a table showing the percentage of water contributed by specific wells and storage tanks to locations in the Dover Township area for 1998 conditions. Based on residence histories, the trace-simulation results will be used in an epidemiologic investigation to estimate exposure of participants to specific water sources by determining the percentage of water they may have received from each of the points of entry to the distribution system.

INTRODUCTION

The Agency for Toxic Substances and Disease Registry (ATSDR), an agency of the United States Department of Health and Human Services, is required, among several other congressional mandates, to evaluate the public health threat of hazardous waste sites using environmental characterization data, community health concerns, and health outcome data. In the spring of 1996, ATSDR and the New Jersey Department of Health and Senior Services (NJDHSS) began to investigate health concerns of the Dover Township, Ocean County, New Jersey, community. In particular, community members feared that exposure to environmental contaminants from the area's hazardous waste sites, including two National Priorities List (Superfund) sites (Plate 1) was related to the elevated incidence of childhood leukemia and brain and central nervous system (CNS) cancers.

In 1997, ATSDR and NJDHSS began designing a case-control epidemiologic study of childhood cancers that occurred in the period 1979 through 1996 (Berry and Haltmeier 1997) in Dover Township. In a case-control study, a population is delineated and cases of diseases arising in that population over a specified time period are identified. The exposure experiences of the case group are compared to the exposure experience of a sample of the non-diseased persons in the population from which the cases arose. Exposures that are more common among the cases may be considered as possible risk factors for the disease (Rothman and Greenland 1998).

The study, which began data collection in 1998, is exploring multiple possible risk factors, including environmental exposures. One of the environmental factors of community concern that is being investigated in the study is the potential for exposure to certain drinking water sources. ATSDR and NJDHSS have determined that completed human exposure pathways to groundwater contaminants have occurred in the past (through private and community water supplies) in some parts of the community (NJDHSS 1999a, b, c).

To assist with the exposure assessment component of the epidemiologic study, ATSDR is developing a water-distribution model using the EPANET software (Rossman 1994) to reconstruct historical patterns of water distribution. Given the paucity of historical contaminant-specific concentration data during the time frame relevant to the epidemiologic study, ATSDR and NJDHSS have determined that modeling would estimate the percentage of water that a study subject might have received from each of the points of entry to the water-distribution system (Plate 2). This would allow epidemiologists to assess the association between the occurrence of childhood cancers and exposure to each of the sources of potable water entering the distribution system, including ones known to have been historically contaminated.

A detailed literature review of epidemiologic investigations relating water-supply contamination with health effects is beyond the scope of this report. However, a brief review is provided below. Lagakos et al. (1986) describe an association between exposure to trichloroethylene-contaminated drinking water and increased prevalence of stillbirths and CNS defects, oral defects, and chromosomal defects. To investigate the potential reproductive health effects of long-term, low-dose exposure to waterborne chloroform, Kramer et al. (1992) conducted population-based case control analyses to study the association of trihalomethanes with low birth-weight, prematurity, and intrautarine growth retardation using state of Iowa birth certificate data. Bove et al. (1995) used environmental and birth-outcome databases for a four-county area in northern New Jersey to study the effects of public drinking water contamination on birth outcomes.

This report will focus on the four aspects of the overall exposure assessment effort, being conducted jointly by ATSDR and NJDHSS, that will eventually use a calibrated model for historical reconstruction of the hydraulic characteristics of the water-distribution system. These aspects are: (1) data gathered during field tests conducted in March and August 1998, (2) the development, calibration, and testing of the water-distribution system model for 1998 conditions, (3) a water-quality simulation of a naturally occurring conservative element, barium, to further test the reliability of the model calibration, and (4) the simulation of the proportionate contribution of water from points of entry to various locations throughout the distribution system for 1998 conditions.

BACKGROUND

Contamination of groundwater resources in Dover Township, including the contamination of water-supply wells, has been documented and ongoing since the 1960s. Water-quality analysis, conducted since the mid-1980s indicates this contamination has generally consisted of volatile organic compounds (VOCs) such as trichloroethylene (TCE) and semi-volatile organic compounds (SVOCs) such as styrene-acrylonitrile (SAN) trimer (NJDHSS 1999c). The reader is referred to the following reports for a description and analysis of contamination of groundwater resources: ATSDR (1988, 1989), Malcolm Pirnie, Inc. (1992), Pinder, et al. (1992), and NJDHSS (1999a, b, c). The source of potable water for the area is groundwater and it is withdrawn primarily from the shallow Kirkwood-Cohansey aquifer, although the deeper Piney Point and Potomac/Raritan/Magothy aquifers are also used as sources for groundwater (Table 1). Approximately 85% of the current Dover Township area residents are served by a public-supply system (as opposed to privately owned domestic wells). Therefore, the possibility exists of human exposure to these contaminants through the groundwater pathway, and an analysis of the potential distribution of contaminants through the water-distribution system was deemed necessary.

METHOD OF ANALYSIS

To reconstruct the historical flow of water from different sources into and through a system of interconnected pipelines, we have chosen to use a water-distribution system model. In a distribution system such as the one serving residents of the Dover Township area, not all public-supply wells were contaminated. Furthermore, some supply wells affect certain areas more than other wells do. Thus, at any given point in the distribution system, water may be derived from one or more sources in differing proportions, i.e., the concept of "proportionate contribution." Therefore, a water-distribution model is a useful tool to estimate the "proportionate contribution" of water sources through time.

Because the focus of the epidemiologic investigation is on children, exposure at residential locations is deemed as the most important exposure source to investigate, although other exposure sources may be present. Based on residence histories, reconstruction of historical water-distribution system characteristics can be used to estimate exposure to specific water sources by determining the percentage of water study subjects may have received from each of the points of entry (i.e., well fields) to the water-distribution system.

Table 1. *Description water-distribution system tanks, wells, and pumps, Dover Township area, New Jersey*

Plant or Facility Identification[1]	Storage Tanks				Groundwater Wells				Booster Pumps	
	Type	Diameter (ft)	Height (ft)	Volume (Mgal)	ID	[2]Depth (ft)	Capacity (gpm)	Aquifer ID[2]	ID	Capacity (gpm)
Berkeley Township stations	–[3]	–	–	–	33	102	1,000	KC	–	–
	–	–	–	–	34	105	1,000	KC	–	–
	–	–	–	–	35	105	1,000	KC	–	–
Brookside treatment plant	–	–	–	–	15	230	700	PP	–	–
	–	–	–	–	43	263	1,400	PP	–	–
Holiday City tank	ground	82.5	25	1.0	–	–	–		Pump 1	1,400
Holly Street treatment plant	ground	88	12	0.525	21	52	700	KC	Pump 1	800
	ground	88	12	0.525	30	1,875	2,100	PRM	Pump 2	1,500
	–	–	–	–	37	238	580	PP	Pump 3	3,200
Indian Head well house	–	–	–	–	20	87	450	KC	–	–
Indian Hill tank	elevated	50	37.3	0.500	–	–	–		–	–
North Dover tank	elevated	65	45	1.0	–	–	–		–	–
Parkway South station	–	–	–	–	44	131	450	KC	–	–
	–	–	–	–	45	1,345	1,000	PRM	–	–
Parkway treatment plant	ground	85	24	1.0	22	105	700	KC	Pump 1	3,200
	–	–	–	–	24	97	600	KC	Pump 2	3,200

Table 1. *Description water-distribution system tanks, wells, and pumps, Dover Township area, New Jersey–Continued*

Plant or Facility Identification[1]	Storage Tanks				Groundwater Wells				Booster Pumps	
	Type	Diameter (ft)	Height (ft)	Volume (Mgal)	ID	Depth (ft)	Capacity (gpm)	Aquifer ID[2]	ID	Capacity (gpm)
Parkway treatment plant–continued	–[3]	–	–	–	26	105	600	KC	–	–
	–	–	–	–	28	127	600	KC	–	–
	–	–	–	–	29	137	600	KC	–	–
	–	–	–	–	39	288	150	PP	–	–
	–	–	–	–	41	294	200	PP	–	–
	–	–	–	–	42	1,345	1,200	PRM	–	–
Route 37 tank (St. Catherine's)	ground	66	40	1.0	–	–	–		Pump 1	650
Route 70 well house	–	–	–	–	31	142	700	KC	–	–
South Toms River station	elevated	43.3	28	0.30	32	54	700	KC	Pump 1	500
	–	–	–	–	38	66	700	KC	Pump 2	500
Windsor Avenue plant	ground	103	24	1.5	40	318	1,900	PP	Pump 1	1,000
	–	–	–	–	–	–	–		Pump 2	1,000
	–	–	–	–	–	–	–		Pump 3	1,000

[1]Data provided by United Water Toms River, Inc.
[2]Aquifer ID: KC, Kirkwood-Cohansey; PP, Piney Point; PRM, Potomac/Raritan/Magothy
[3]Not applicable.

The use of water-distribution system modeling for estimating exposure has been described in the literature by several investigators. Murphy (1986) calculated exposure to TCE from Wells G and H in Woburn, Massachusetts, by using a water-distribution model to assess various pumping and water use configuration patterns during each month that the wells were in operation. Clark et al. (1991) and Geldreich et al. (1992) used extended period simulation hydraulic and dynamic water-quality models to investigate the distribution of occurrences of illness due to waterborne contaminants (*escherichia coli* sterotype 0157:H7) found in the Cabool, Missouri, distribution system. Clark et al. (1996a, b) used the EPANET water-distribution system model (Rossman 1994) to develop several scenarios to explain possible pathogen transport of waterborne *Salmonella typhimurium* outbreak in the Gideon, Missouri, municipal water system. Aral et al. (1996) and Aral and Maslia (1997) used the EPANET water-distribution system model in conjunction with a geographic information system (GIS) to simulate four exposure scenarios for the Southington, Connecticut, water-supply system that used groundwater contaminated with VOCs during the 1970s. For the current investigation, the EPANET water-distribution model, integrated with spatial analysis technologies, is being used to model the water-distribution system serving the residents of the Dover Township area. A description of the model is presented in a subsequent section of this report.

Water-distribution system modeling can be used in a predictive sense such as scenario testing described in Aral et al. (1996) and Aral and Maslia (1997), or as a diagnostic tool such as finding the cause for disease outbreak described by Clark et al. (1996a, b) and Geldreich et al. (1992). For the current study, we will eventually be using the model in a diagnostic mode–reconstructing historical water-distribution system characteristics. To accomplish this, it is critical that investigators understand the reliability of model generated (or simulated) results. To assess the reliability of a model, a calibration process is undertaken. That is, investigators must be able to quantify the difference between measured parameter values (e.g., pressures, storage tank water levels) and simulated parameter values. (Details of the calibration process are described in the section on "Hydraulic Model Calibration.") Thus, the first step in our investigation is to establish the reliability of the water-distribution system model for the Dover Township area by undertaking a calibration process. Once the reliability of the model has been established, then the model can be used in a diagnostic mode to examine (reconstruct) historical characteristics of the water-distribution system serving the Dover Township area.

DESCRIPTION OF THE WATER-DISTRIBUTION SYSTEM

The water-distribution system being analyzed has been operating since 1897 and is currently operated by United Water Toms River, Inc. (UWTR). It serves the residents of Dover Township, New Jersey, and communities outside of Dover Township including the borough of South Toms River and a portion of Berkeley Township (Plate 2). At the end of 1997, the water-distribution system served a population of 92,160 that consisted of 44,510 customers. The distribution system consists of 488.2 miles (mi) of mains, ranging in diameter from 2 inches (in.) to 16 in., 3 elevated and 6 ground-level storage tanks with a total rated storage volume of 7.35 million gallons (Mgal), 23 municipal groundwater wells in 8 well fields (or points of entry) with a total rated capacity of 27 million gallons per day (MGD), and 12 high service or booster pumps (Board of Public Utilities 1997). A list and description of the water-distribution system tanks, wells, and pumps serving the Dover Township area is provided in Table 1. In the distribution system as presently configured (1998), 7 of these wells pump directly into the distribution system (e.g., wells 20, 31), whereas the remaining 16 wells are used to fill storage tanks (e.g., Parkway well field ground-level storage tank) and then high service or booster pumps are used to supply the distribution system with water from the storage tanks (Plate 2, Table 1).

Demand for water in the Dover Township area is characterized by two typical demand patterns. A winter-time demand pattern, typical of data collected in March 1998 (Figure 1A), generally occurs from October through mid-May. Data collected in March 1998, show that demand was equal to 7.6 MGD (Sautner and Maslia 1998). These data were obtained from the water utility's supervisory control and data acquisition (SCADA) system and recorded by ATSDR staff stationed in the water utility's control room during the test. Peak-demand conditions, typical of field data collected in August 1998 (Figure 1B), and equal to 16.1 MGD, generally occur during the summer season from the end of May (Memorial Day) through September (Maslia and Sautner 1998b). Thus, for the water-distribution system serving the Dover Township area, average annual demand, based on data obtained during the tests from the water utility, is approximately 12 MGD.

INITIAL MODEL SIMULATION

In February 1997, ATSDR was provided with a database by the water utility (UWTR) that was used to described an equivalent pipe (or hydraulic) network of a 1993 water-distribution system network. (The 1993 distribution system was configured similarly to the present-day [1998] system, and therefore, the present-day system shown on Plate 2 can be used as a reference.) An equivalent pipe network is one in which smaller diameter pipes, bends, and valves are eliminated from the network and replaced by a simple pipe of uniform diameter. In this equivalent pipe or hydraulic network, the head loss and discharge are the same as the head loss and discharge in the multiple pipe network (Bhave 1991). Thus, equivalent pipe networks
can be used to model the generalized characteristics of a water-distribution system. The equivalent hydraulic network constructed by the water utility for the Dover Township area consisted of approximately 950 pipeline segments ranging in diameter from 6 in. to 16 in. This equivalent hydraulic network represented, according to the water utility (Flegal 1997), the salient characteristics of the 1993 water-distribution system network, and is shown in comparison with the existing 1998 water-distribution system network on Plate 3.

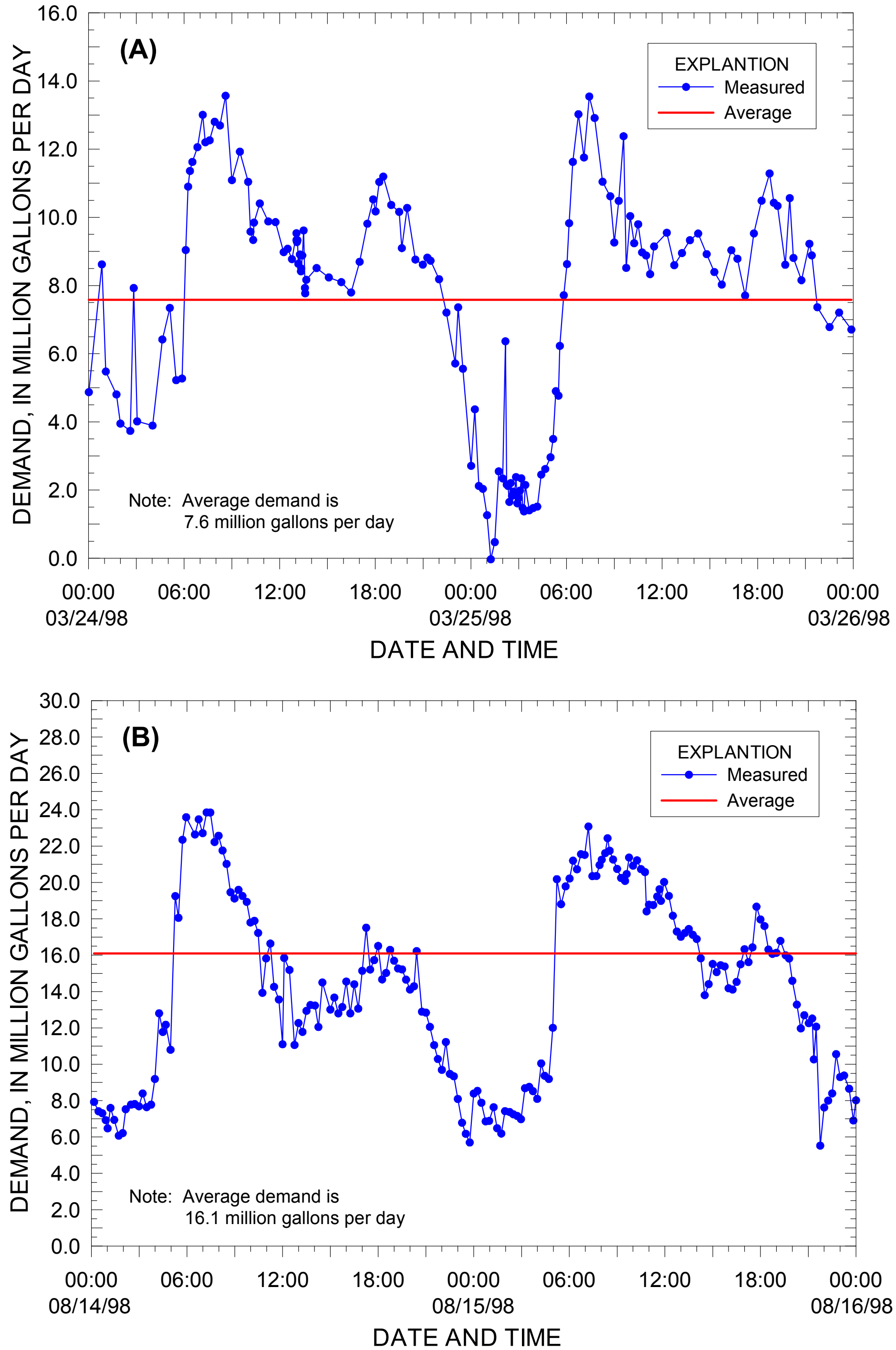

Figure 1. *Time pattern for water usage: (A) winter-time demand, March 24-25, 1998, and (B) peak-demand, August 14-15, 1998.*

Using the equivalent hydraulic network database provided by the water utility in conjunction with information on storage tanks, wells, and pumps (Table 1), initial simulations were conducted using the EPANET water-distribution system model (Rossman 1994) to simulate typical hydraulic conditions for 1993[1,2]. Evaluation of the results caused three concerns:

(1) Simulated pressure using the equivalent hydraulic network in the southern Dover Township, the Berkeley Township, and the borough of South Toms River areas appeared to be unusually high; many values were near 110 pounds per square inch (psi) and some exceeded 125 psi;

(2) The goal of the ATSDR and NJDHSS investigation is to relate study subjects to pipeline segments and sources that may have historically serviced their water-supply needs; therefore, using a skeleton of the distribution system represented by the equivalent hydraulic network could result in misclassification; and

(3) The distribution system has expanded substantially since the early 1960s; using the equivalent hydraulic network to reconstruct characteristics of historical water-distribution system networks could result in residence areas of study subjects being serviced by an unrealistically limited number of pipelines, again giving rise to the possibility of misclassification.

Because of these concerns, ATSDR investigators decided to refocus their analysis of the water-distribution system on the actual or "street-level" distribution system, which is characterized by pipelines ranging in diameter from 2 in. to 16 in. Therefore, a model would need to be developed using all "street-level" pipelines as opposed to the limited number of pipelines of the equivalent hydraulic network (see Plate 3 for a comparison of "street-level" pipelines as of 1998 with the equivalent hydraulic network pipelines). In addition, ATSDR determined that it would be necessary to obtain field data for 1998 conditions to calibrate the water-distribution system model and confirm or negate the unusually high pressures simulated by the equivalent hydraulic network model in the southern Dover Township, Berkeley Township, the borough of South Toms River areas. (A detailed discussion of model calibration is provided in the section on "Hydraulic Model Calibration.")

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[1]The reader should refer to the section on "Water-Distribution System Model Development" for detailed description of the hydraulic and water-quality simulators used in the EPANET model.

[2]The database supplied by UWTR was used by the water utility as input for a proprietary water-distribution model, Piccolo (SAFEGE Consulting Engineers 1994). ATSDR conducted simulations on the equivalent hydraulic network using both the Piccolo and EPANET models. Comparison showed results (pressures, tank levels, and flows) were nearly the same for both models.

TECHNICAL WORK GROUP AND EXPERT PANEL REVIEW

Throughout this investigation, ATSDR has sought outside technical input and expert peer review for this effort. In September 1997, ATSDR convened a technical workgroup to review the initial model simulations described above. On the basis of their discussions and review, the following recommendations were made (ATSDR 1999, p. 10):

(1) Model simulated pressures using the water utility's "skeletonized" or equivalent hydraulic network in southern Dover Township and in the South Toms River areas appear to be exceedingly high . . . thus a "reality check" is needed to either confirm or negate model simulated pressures;

(2) To use the model for simulating present-day conditions and reconstructing historical conditions, a set of spatially distributed pressure measurements, occurring under varying operating conditions, should be obtained;

(3) To use the model to simulate water-quality characteristics, a set of water-quality calibration data would be needed;

(4) The equivalent hydraulic network provided to ATSDR by the water utility should be refined down to a "street-level" network; and

(5) As much of the network as possible should be geo-referenced.

After implementing items (1), (2), (4), and (5) of the recommendations above, ATSDR held an expert panel meeting in December 1998 (ATSDR 1999). The meeting convened nine scientific and technical experts from academia, government, private consulting, and industry, who discussed the status of ATSDR's water-distribution system model and its intended use. The salient recommendations resulting from the meeting can be found in an ATSDR report (1999, p. 34) and are summarized below:

(1) ATSDR should re-calibrate the model using additional available data about pumping schedules during the March and August 1998 pressure tests and make appropriate modifications if needed;

(2) Because the UWTR system contains a large amount of polyvinyl chloride pipe, roughness coefficients are not believed to be an especially significant factor for modeling; to test this assumption, a sensitivity analysis should be performed on the effect on water flow of roughness coefficients in the UWTR system;

(3) ATSDR must fully define system operating rules, including those for filling tanks in the early morning hours; once defined, these rules must be incorporated into the model;

(4) The distribution system model should be operated for sufficient time to reach dynamic equilibrium for the various demands that characterize the system; and

(5) A tracer should be used to generate data on transport of water in the distribution system from points of entry; ideally the selected tracer would be introduced at entry points of the wells through which contaminants are suspected of entering the distribution system.

ATSDR's implementation of recommendations from the technical work group and expert panel review follows in the remaining sections of this report. Details of field collection activities, model development, and procedures used to calibrate and further test the water-distribution system model for the Dover Township area are presented below. The calibrated model is believed to be a reasonable representation of the present-day (1998) characteristics of the water-distribution system operating under winter-demand (March 1998) and peak-demand (August 1998) conditions.

FIELD-DATA COLLECTION ACTIVITIES

Three reasons exits for initiating a synoptic, system-wide collection of hydraulic and operational data. First, after conducting preliminary simulations using an equivalent network representation of the water-distribution system, results indicated higher than expected pressures, exceeding 125 psi in some locations (in the southernmost areas of Dover Township including the borough of South Toms River and Berkeley Township). Measured data were not available to either confirm or negate these initial simulation results. Second, to understand the present-day distribution of water from points of entry (sources) to locations throughout the distribution system, a calibrated model of the distribution system needed to be developed; however, a database of spatially and temporally varying data by which to characterize the distribution system was not available. Third, to reconstruct historical characteristics of the water-distribution system, a synoptic, system-wide characterization of the water-distribution system, based on measured data, was required. Neither present-day nor historical system-wide pressure measurements were available for the water-distribution system being investigated. Hence, ATSDR investigators decided to obtain present-day measurements to accurately characterize the water-distribution system.

ATSDR, in coordination with NJDHSS and the water utility, developed a protocol to collect pressure data and operational information during winter- and peak-demand periods of the year–March and August 1998, respectively. Details of the protocol are provided in a report by Maslia and Sautner (1998a) and are briefly described below.

HYDRANT SELECTION

Twenty-five hydrants (out of a system total of 2,127 in 1997) were initially selected as test hydrants (designated as H-1, H-2, etc.) on which continuous pressure recording-equipment (described below) would be installed. The number and location of the proposed test hydrants (H-1, H-2, etc., shown in Plate 4) were selected based on the following:

(1) Hydrant locations were selected to help determine the system pressures that exist in the southern Dover Township, Berkeley Township, and the borough of South Toms River areas that were initially simulated as having unusually high pressures, as described above.

(2) Hydrant locations were also selected to provide a thorough, system-wide coverage so that effects from storage tanks filling or emptying and pumps turning on or off could be characterized by pressure changes at these hydrants.

(3) ATSDR used additional hydrants for quality assurance in the event that data- collection devices failed to operate properly or that hydrants became inoperable or unusable during the test.

(4) ATSDR responds to and attempts to accommodate stakeholder input when conducting site activities. In the case of modeling the water-distribution system serving Dover Township, area residents wanted assurances that a sufficient number of measuring locations would be available to accurately characterize the distribution system. When ATSDR investigators determined that additional data-collection locations would not abrogate the scientific merits of the field test, additional monitoring locations requested by area residents were also included.

As part of a quality assurance program, 25 hydrants were selected as alternate test hydrants (designated as AH-1, AH-2, etc.) in the event that any of the original 25 test hydrants could not be used to monitor pressure during the test periods (see Plate 5 for location of alternate test hydrants). During installation of the data-gathering equipment on the hydrants, it was determined that 5 of the designated test hydrants (H-5, H-6, H-14, H-19, and H-22; Plate 4) were not suitable for use as measuring points. Therefore, associated alternate hydrants (AH-5, AH-6, AH-14, AH-19, and AH-22; Plate 5) were used instead. The 5 hydrants (H-5, H-6, H-14, H-19, and H-22) were not used because when they were identified and turned on, investigators noticed excessive leakage flowing from the bolts joining the hydrant base to the underlying pipeline. Investigators felt that the excessive leakage would worsen with a pressure gauge or pressure recording device attached to the hydrant, resulting in erroneous pressure measurements. The

Table 2. *Identification, coordinates, and location of test hydrants used for the March and August 1998 pressure tests, Dover Township area, New Jersey*

ATSDR Hydrant ID[1]	[2]UWTR Hydrant ID	[2]Pipe Diameter (inches)	[2]Year Installed	Hydrant Location: [3]Geodetic Coordinates NAD 1927		Hydrant Location: New Jersey State Plane Coordinates, NAD 1927		Hydrant Location: Street Identification[4]	[5]Land Surface Elevation (feet)
				Latitude (decimal degrees)	Longitude (decimal degrees)	Northing (feet)	Easting (feet)		
H-1	6-075	8	1993	39.953265	-74.286958	408142.264	2106457.294	Millbrook Dr. & Westbrook Dr., NW corner	49.42
H-2	6-014	8	1983	39.977381	-74.283213	416931.503	2107469.462	Costa Mesa Dr. & Pine Valley Dr., SE corner	55.77
H-3	1-863	12	1989	39.989508	-74.265614	421370.722	2112381.845	Route 37, W. Floracraft @ no. 1600	59.91
H-4	6-173	8	1974	39.964666	-74.245314	412347.723	2118112.826	Pembroke Ln. & Fort de France Ave., SE corner	29.92
AH-5	6-227	12	1988	39.945245	-74.240738	405279.324	2119429.296	Prince Charles Dr. & Davenport Rd, NE corner	51.60
AH-6	1-156	6	1960	39.971423	-74.221675	414841.283	2124726.787	Oakside Dr. & Shady Nook Dr., NW corner	44.14
H-7	1-304	12	1966	39.957666	-74.212299	409843.256	2127380.239	Lakehurst Rd. & Edgewood Dr., SW corner	32.86
H-8	5-012	10	1988	39.950151	-74.199608	407124.136	2130952.345	South Main St. & Flint Rd., SE corner	6.92
H-9	1-061	6	1991	39.952620	-74.193779	408032.128	2132581.850	Washington St. & Hooper Ave., NE corner	32.68
H-10	1-078	12	1976	39.953508	-74.170042	408391.758	2139234.937	Washington St., across from Pine St.	35.68
H-11	1-731	8	1988	39.947713	-74.145932	406319.347	2146006.636	Elizabeth Ave. & Berkeley Ave., NW corner	13.91
H-12	1-702	8	1988	39.944769	-74.121910	405287.147	2152748.462	Minturn Rd. & Bay Shore Dr., NW corner	4.26
H-13	1-665	12	1988	39.952430	-74.118217	408084.170	2153766.796	Marshall Rd. across from Maritime Dr.	6.56
AH-14	1-762	8	1985	39.958156	-74.146045	410123.244	2145952.755	Windsor Ave. & Huckleberry Ln., SW corner	28.27
H-15	1-591	12	1986	39.968498	-74.122447	413930.025	2152545.164	Bay Ave. & Bermuda Dr., SE corner	6.18
H-16	1-973	8	1996	39.973560	-74.206719	415640.998	2128914.673	S. Dakota Ave. & N. Carolina Ave., NE corner	38.66

Table 2. *Identification, coordinates, and location of test hydrants used for the March and August 1998 pressure tests, Dover Township area, New Jersey–Continued*

ATSDR Hydrant ID[1]	UWTR Hydrant ID [2]	[2]Pipe Diameter (inches)	[2]Year Installed	Hydrant Location					[5]Land Surface Elevation (feet)
				[3]Geodetic Coordinates NAD 1927		New Jersey State Plane Coordinates, NAD 1927		Street Identification[4]	
				Latitude (decimal degrees)	Longitude (decimal degrees)	Northing (feet)	Easting (feet)		
H-17	2-332	6	1987	39.992107	-74.206319	422397.734	2128991.902	Indian Head Rd. & Hill Grass Ct., NW corner	89.30
H-18	2-124	12	1984	39.999978	-74.228648	425233.402	2122721.279	Whitesville Rd. & Clayton Ave., N corner	46.76
AH-19	2-348	12	1987	40.020870	-74.220673	432854.892	2124917.511	Route 9 Hwy. & Riverwood Dr., NW corner	71.78
H-20	2-720	12	1996	40.050572	-74.250934	443633.608	2116391.508	Whitesville Rd. & Clear Lake Blvd., NW corner	82.68
H-21	2-171	12	1974	40.009504	-74.215593	428721.714	2126361.277	Church Rd. & Rte 9 , NE corner	71.24
AH-22	2-025	8	1965	40.018729	-74.154675	432174.358	2143406.735	Hovsons Blvd. & Adirondack Pl., NE corner	14.17
H-23	2-093	12	1963	40.003062	-74.154607	426467.378	2143458.575	Fisher Blvd. & Hooper Ave., SE corner	10.50
H-24	1-400	12	1971	39.992985	-74.173007	422767.509	2138324.083	Indian Hill Rd. & 300 ft South of Hooper Ave.	88.89
H-25	1-308	8	1988	39.975514	-74.176163	416398.421	2137474.800	Brookside Dr. & Bay Ave., SW corner	42.81

[1]Refer to Plate 6 for hydrant locations.
[2]Data obtained from United Water Toms River, Inc., November 1997.
[3]Geodetic data obtained by ATSDR staff using global positioning system (GPS) equipment, January 1998; refer to Sautner, et al. (1998).
[4]Refer to Appendix A for photographs showing hydrant location referenced to street corners.
[5]Land surface elevation, referenced to sea level datum, determined by use of GPS equipment. For hydrants H-2, AH-6, H-11, H-12, and H-18, GPS data were determined to be in error. Therefore, elevations were determined from U.S. Geological Survey digital elevation model (DEM), 7 ½-minute quadrangles for the Dover Township area.

final set of test hydrants used for both the March and August 1998 tests is shown in Plate 6. These 25 hydrants are connected to network pipelines that: (1) were installed between 1963 and 1996, (2) are constructed of asbestos cement (14 hydrants), plastic (PVC, 10 hydrants), and ductile iron (1 hydrant) materials, and (3) range in diameter size from 6 in. to 12 in.

The location of the final set of test hydrants, described by: (1) geodetic coordinates, (2) New Jersey state plane coordinates, and (3) street location, is listed in Table 2. Photographs showing the location of the test hydrants with reference to street corners are provided in Appendix A. The geodetic locations were determined using global positioning system (GPS) equipment[3]. Details pertaining to determining hydrant locations are provided in Sautner, et al. (1998). Land surface elevation at the hydrant locations was also determined using GPS equipment by placing the top of the receiving antenna on the ground next to the hydrant (see photograph of hydrant H-8 in Appendix A), and subtracting the length of the antenna (1.83 feet [ft]) from the recorded GPS elevation. Because elevations of the measuring points will be used in the calibration process to assess the reliability of the model (see section on "Hydraulic Model Calibration"), elevations determined by use of the GPS equipment were further verified by using land surface elevations obtained from 7½-minute U.S. Geological Survey (USGS) digital elevation model (DEM) quadrangles for the Dover Township area. For six hydrants (H-2, AH-6, H-11, H-12, and H-18; Table 2) the GPS elevations were found to be in error, although the cause of the error was unknown. Therefore, elevations for these hydrants were derived using the DEM data instead. (For these hydrants, field verification of land surface elevations indicated that the GPS-determined elevations were in error.)

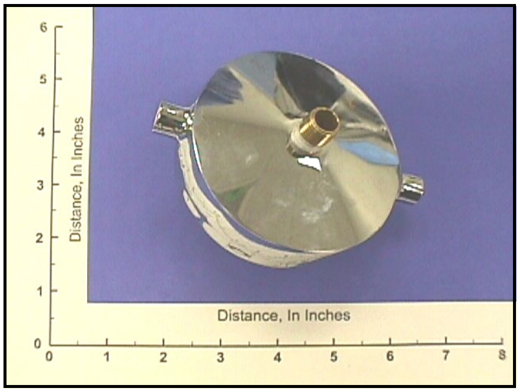
DATA RECORDING EQUIPMENT

To record system pressures continuously over the 48-hour duration of the tests, each test hydrant was equipped with a RADCOM Lolog LL™ continuous recording pressure data logger that sampled pressures at one-minute intervals. Components that made up the hydrant pressure measurement configuration were (Figures 2 and 3): (a) a hydrant adapter kit, (b) a brass lever handle shut-off valve, (c) a coiled pressure hose with a quick release coupling, and (d) the RADCOM Lolog LL™ pressure data logger. The pressure data recording equipment was chosen because of:

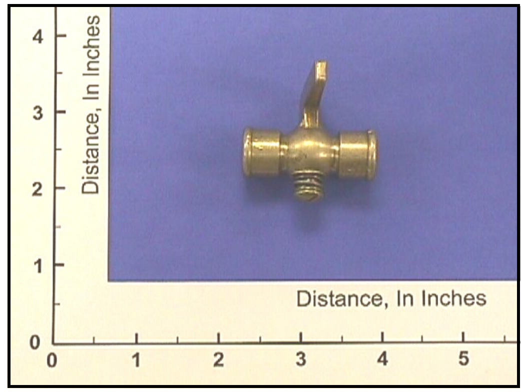
(1) a factory calibration to 150 psi (see Appendix B);

(2) a pressure range of 0 psi to 300 psi and accuracy of +/- 0.2% of the pressure range;

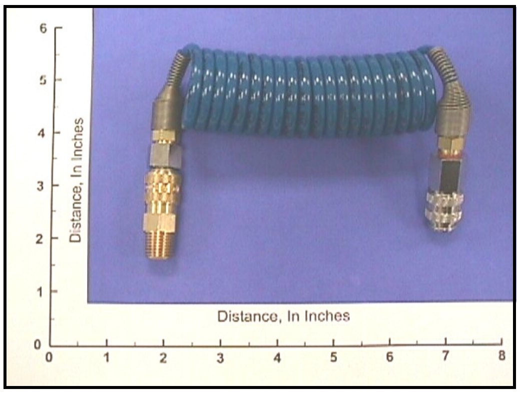
[3] Information relating to the use of GPS equipment including post analysis and differential correction of data is provided in an ATSDR report by Maslia et al. (1997).



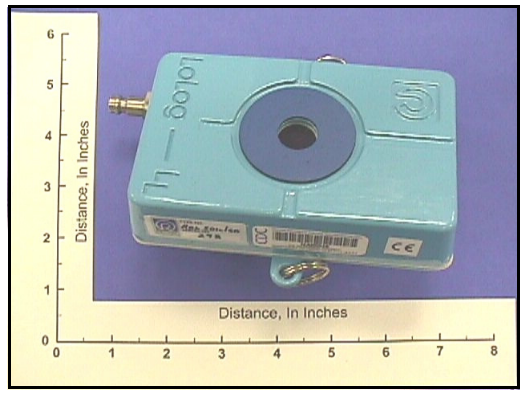
(A) Dickson A7983 hydrant adapter kit



(B) Dickson brass lever handle shut-off valve, 1/4 NPTF x 1/4 NPTF, shown in “off” position



(C) Coiled pressure hose with a quick release coupling



(D) RADCOM Lolog LL™ pressure data logger

Figure 2. *Components of hydrant pressure measurement configuration.*

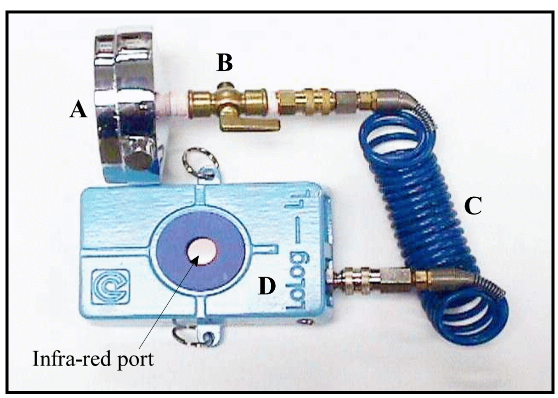


Figure 3. *Assembly of hydrant pressure measurement configuration: (A) hydrant adapter kit, (B) brass shut-off valve in “on” position, (C) coiled pressure hose with quick release coupling, and (D) RADCOM LoLog LL™ pressure data logger.*

(3) the variable sampling rate of 1, 5, 15, 30, and 60 minutes;

(4) the compact size of the device (5.5 in. x 3.3 in. x 1.3 in.) that would allow its attachment directly to a hydrant; and

(5) the reliability of the device with respect to environmental factors such as precipitation and cold because the device is constructed of die-cast aluminum and IP68 sealing (fully submersible).

The complete assembly of the hydrant pressure-measurement configuration is shown in Figure 3. At the conclusion of the test, data from each pressure logger was retrieved by downloading the data to a Psion HC-120™ hand-held computer through the use of an RS232C cable that connects to the infra-red port on the logger (Figure 3) to the Psion HC-120™ hand-held computer in the manner shown in Figure 4.

TEST PROTOCOL

ATSDR established a test protocol or workplan (Maslia and Sautner 1998a) to ensure that successful data-gathering would occur during the test and to establish quality-assurance procedures that would be followed for each hydrant monitored. Because a large number of hydrants were to be monitored on a continuous basis, two installation teams were used. As part of the test protocol and quality-assurance procedure, each logger was identified by a hydrant number (H-1, H-2, etc.) and a logger serial number. Table 3 lists hydrant identifications and the associated pressure data logger serial numbers used for the March and August 1998 tests. Before conducting the tests, the loggers were factory calibrated to 150 psi, and, therefore, did not need field calibration (a letter from the manufacturer indicating the loggers were calibrated is provided in Appendix B). However, as part of the quality-assurance procedure, logger pressure during installation was verified with a manual pressure gauge, as described below. Installation teams were composed of ATSDR, NJDHSS, and water-utility staff. The following 7-step procedure was used to install and quality-assure the pressure data logger installation on each of the 25 hydrants:

(1) Water utility staff opened a hydrant and flushed it to remove debris (Figure 5); the water utility staff determined the length of time required to flush each hydrant so that clear running water would be observed. At that point, the hydrant was shut off.

(2) ATSDR and NJDHSS staff installed a hydrant adapter kit, brass lever handle shut-off valve, and Ashcroft Duralife industrial pressure gauge on the test hydrant (Figure 6).

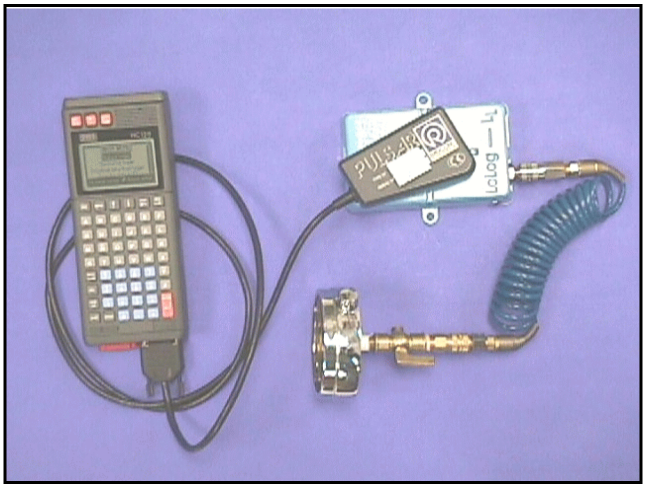


Figure 4. *Configuration of test equipment showing method of downloading data from pressure logger to a Psion HC-120™ hand-held computer using an RS232C cable attached to pressure logger's infra-red port.*

Table 3. *Hydrant identification and corresponding data logger serial numbers used for the March and August 1998 pressure tests, Dover Township area, New Jersey*

Hydrant Identification	Data Logger Serial Number	Hydrant Identification	Data Logger Serial Number
H-1	294	AH-14	307
H-2	295	H-15	308
H-3	296	H-16	309
H-4	297	H-17	310
AH-5	298	H-18	311
AH-6	299	AH-19	312
H-7	300	H-20	313
H-8	301	H-21	[1]314, 222
H-9	302	AH-22	315
H-10	303	H-23	316
H-11	304	H-24	317
H-12	305	H-25	318
H-13	306		

[1] Logger 314 was determined to be defective during installation of the March 1998 test. The manufacturer supplied a replacement logger (number 222) that was installed on hydrant H-21at approximately 10:34 hours on March 24, 1998 and used for the remainder of the March 1998 test and for the August 1998 test.

(3) With the shut-off valve in the "off" position, the hydrant was reopened, the shut-off valve was moved to the "on" position, and a check was made for any leaks occurring around the pressure gauge connection and the hydrant adapter kit. At this point, a pressure reading was obtained from the pressure gauge (Figure 6) and recorded in a field book. An example of the entry form used to record installation information for hydrant H-1 is provided in Appendix C. After the pressure was recorded, the shut-off valve was turned to the "off" position, and the pressure gauge was disconnected from the shut-off valve and hydrant adapter kit.

(4) The RADCOM Lolog LL™ pressure data logger (Figure 2D) was now attached to the hydrant by means of a nylon cable tie (Figure 7). A coiled pressure hose with a quick release coupling (Figure 3) was then attached to the data logger and the shut-off valve. The shut-off valve was then placed in the "on" position, and the data logger started recording water pressure at the hydrant. A piece of duct tape was wrapped around the shut-off valve handle to ensure that it remained in the "on" position. A photograph showing the attachment of the pressure data logger installation to test hydrant H-23 is provided in Figure 7.

(5) Because the logger is a continuous recording device, it was "zeroed" and the date and time initialized during installation. For a quality assurance check, an instantaneous pressure reading from the pressure data logger was obtained (Figure 8), and this was compared with the manual gauge pressure recorded earlier (see field book sheet in Appendix C). The pressure from the data logger was obtained within about one minute of installation by using the Psion HC-120 ™ hand-held computer to query the data logger by attaching an RS232C cable from the infra-red port on the data logger to the hand-held computer (Figure 8).

(6) After the installation was completed, the hydrant was enclosed in a black, heavy-duty plastic bag (0.003 in.) to indicate that the hydrant was out of service, as required by local ordinance (Figure 9).

(7) ATSDR stationed staff in the water utility's operations control room for the duration of the tests. The staff recorded the operational history of wells and booster pumps. They also recorded instantaneous system-production data and storage tank water levels registered by the water utility's SCADA system.

Figure 5. *Water utility staff flushing test hydrant H-9 in preparation for pressure data logger installation.*

(A)

(B)

Figure 6. *(A) hydrant adapter kit, brass lever handle shut-off valve in "off" position, and Ashcroft Duralife industrial pressure gauge, and (B) installation of equipment on test hydrant H-9.*

Figure 7. *Attachment of pressure data logger installation to test hydrant H-23.*

Figure 8. *ATSDR staff querying pressure data logger attached to test hydrant H-11 for parameters of date, time, and pressure using the Psion HC-120™ hand-held computer.*

Figure 9. *ATSDR staff covering test hydrant H-11 with a heavy-duty (0.003 in.) plastic bag to indicate "out-of-service" condition.*

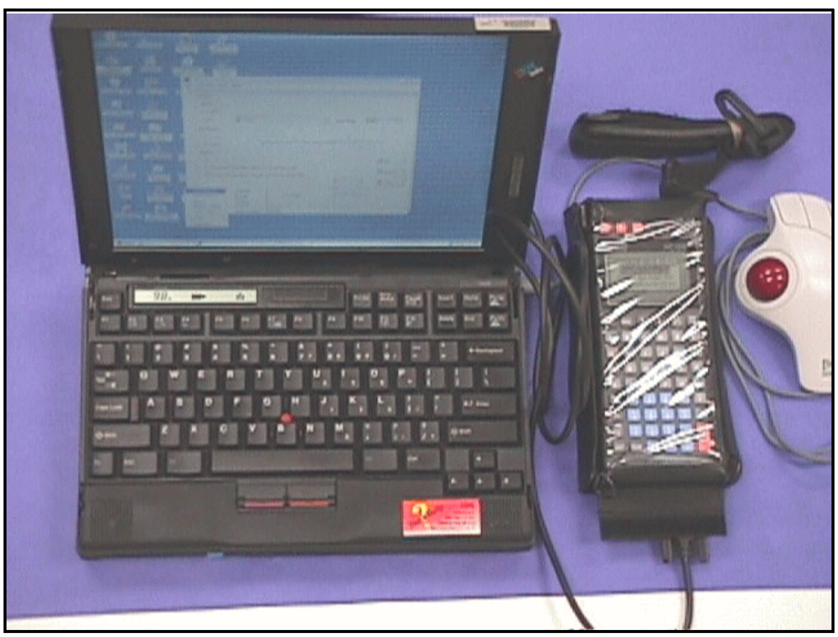


Figure 10. *Data being downloaded from the Psion HC-120™ hand-held computer to a laptop computer using the RS232 ports on both computers.*

Review of installation pressure data indicated that all data logger and gauge pressures were within 3 psi for both the March and August 1998 tests (e.g., Appendix C) with the exception of the logger installed on hydrant H-21 (Plate 6; Table 3) during the March 1998 test. Because of the quality-assurance procedures in place, the logger at this test hydrant location, was determined to be defective and a new logger was flown in overnight and installed the next morning. Therefore, the first 10 hours of data for the March 1998 test were not recorded for hydrant H-21 (see pressure graph for test hydrant H-21 in Appendix D). For the duration of the March and August 1998 tests, ATSDR and NJDHSS staff routinely checked each test hydrant and pressure data logger and, using the Psion HC-120™ hand-held computer to observe instantaneous pressure readings at each hydrant, assured that the loggers were functioning properly (e.g., Appendix C). Additionally, routine checks of each test hydrant were made to assure that no significant water leaks, potentially affecting the pressure measurements, had developed. After the tests were concluded, loggers were removed in a process that was the reverse of the one described in steps 1-6 above. The recorded data in the data loggers were downloaded to and stored in the Psion HC-120™ hand-held computer (Figure 4) for post-test analysis. After all data loggers were removed from the test hydrants, data in the Psion HC-120™ hand-held computer were downloaded, as an additional quality assurance step, to a laptop computer in the manner shown in Figure 10.

ANALYSIS OF PRESSURE DATA

Because data from the tests were gathered at one-minute sampling intervals, the pressure graphs show transients or "spikes" in the data (Appendix D and E). To use the measured pressure data as a reference for calibrating a model, the data should be averaged over one-hour time periods--the smallest time step for which the water-distribution system model is valid (see section on "Requirements for Model Input"). Therefore, the measured pressure data from the tests, shown in Appendix D for the March 1998 test and in Appendix E for the August 1998 test, were averaged over one-hour time periods for use with model calibration. From this point in the report, all pressure data will be referenced to the one-hour average values of the measured pressure data. Graphs of the one-hour average of measured pressure values along with graphs of measured storage tank levels and well and pump flows will be presented and discussed later in the report in the section on "Hydraulic Model Calibration."

Analysis of pressure data from both tests indicates that pressures throughout the water-distribution system generally range from a minimum of approximately 40 psi (test hydrant H-24, Table 4) to a maximum of about 108 psi (test hydrant H-12, Table 4). Because of higher demand conditions existing in August 1998 (Figure 1), maximum one-hour averages of measured pressures for the August test are lower than that those for the March test. For example, at test hydrant H-12, the maximum one-hour average measured pressure of 93.0 psi for the August test is more than 15 psi lower than the maximum one-hour average measured pressure of 108.2 psi for the March test. In the southern Dover Township, Berkeley Township, and the borough of

Table 4. *Comparison of maximum and minimum pressures obtained from the March and August 1998 pressure tests, Dover Township area, New Jersey*

Hydrant Identification[1]	[2]Pressure, in Pounds Per Square Inch			
	March 1998 Test		August 1998 Test	
	Maximum	Minimum	Maximum	Minimum
H-1	92.1	55.8	79.1	48.1
H-2	90.7	50.0	80.0	50.4
H-3	87.3	51.8	76.5	47.6
H-4	103.4	66.0	93.1	63.1
AH-5	98.6	58.7	87.8	55.4
AH-6	97.8	61.6	80.1	55.8
H-7	102.0	66.4	89.8	66.5
H-8	103.8	74.4	92.4	76.2
H-9	99.0	65.4	82.2	63.4
H-10	90.5	59.2	78.7	62.9
H-11	101.8	68.9	88.2	70.6
H-12	108.2	71.3	93.0	72.8
H-13	107.6	71.6	91.4	73.7
AH-14	94.3	62.4	82.9	66.8
H-15	103.1	70.9	90.0	74.0
H-16	91.9	62.7	78.9	62.1
H-17	72.0	45.6	61.6	44.3
H-18	87.9	62.8	76.3	59.5
AH-19	71.6	52.7	61.7	46.2
H-20	61.9	50.4	57.3	42.8
H-21	72.8	50.2	63.6	47.2
AH-22	97.4	68.9	80.1	63.0
H-23	102.5	71.1	84.3	70.5
H-24	68.9	39.6	53.8	39.8
H-25	89.0	58.5	74.1	59.1

[1]See Plate 6 for hydrant locations.
[2]Pressure values are hourly averages derived from one-minute sampling measurements.

South Toms River areas, where initial model simulation indicated areas of questionably high pressure (some in excess of 125 psi--see section on “Initial Model Simulation”), test results indicate that for March 1998, hourly averages of measured pressures ranged from about 50 psi to 104 psi (test hydrants H-1 - H-10, Table 4 and Plate 6) and for August 1998, pressures ranged from 48 psi to 93 psi (for test hydrants H-1 - H-10). Specific details for the tests and results for each test are provided in reports by Sautner and Maslia (1998) and Maslia and Sautner (1998b).

WATER-DISTRIBUTION SYSTEM MODEL DEVELOPMENT

HISTORICAL BACKGROUND

Mathematical modeling has been used for more than 60 years to analyze flow in water-distribution system networks since the concept was proposed by Cross (1936). Using computers for conducting analyses of flow in pipe networks originated in the early 1960s and was greatly expanded during the ensuing decade of the 1970s with the advent of enhanced solution algorithms (Epp and Fowler 1970, Wood and Charles 1972) and the implementation of modeling techniques for devices such as pumps and valves (Jeppson and Davis 1976). In the late 1970s, single-time-period simulations were advanced to extended period simulations with techniques developed by Rao and Bree (1977). Hydraulic models can be used to analyze systems where demand and operating conditions are static or are time varying. The former type of model is a ‘steady-state’ model, and the latter is referred to as an ‘extended period simulation’ or EPS model.

Modeling the spatial distribution of water quality in pipelines first began with a steady-state modeling approach as suggested by Wood (1980) who studied slurry flow. Other researchers developing steady-state water-quality models in the 1980s and early 1990s include Chun and Selznick (1985), Metzger (1985), Males et al. (1985), Clark et al. (1988), Grayman et al. (1988a), Wood and Ormsbee (1989), and Clark (1993). The representation of temporally varying conditions for contaminant movement in a distribution system or ‘dynamic’ water-quality models began to be used in the mid-1980s. Investigators developing such models include Clark et al. (1986), Liou and Kroon (1986), Grayman et al. (1988b), and Hart (1991). With the widespread use and relatively low cost of personal computers and desktop workstations during the mid-1980s and 1990s, many models, both proprietary and public domain, can now be used to conduct hydraulic and water-quality analyses. Two such models in use today are the proprietary model Piccolo (SAFEGE Consulting Engineers 1994) and the public domain model, EPANET (Rossman 1994, Rossman et al. 1994) developed by the U.S. Environmental Protection Agency. The reader is referred to Rossman (1999) and Clark (1999) for a thorough discussion on the evolution and development of hydraulic and water-quality models.

HYDRAULIC SIMULATION MODEL

Hydraulic modeling of water-distribution systems can be conducted by solving mathematical equations that characterize the pipe network of the distribution system. The EPANET water-distribution system model was chosen to conduct an extended period simulation of the hydraulic behavior within the water-distribution system. EPANET solves the following set of equations for each storage node *s* (tank or reservoir) in the system (Rossman 1994, Rossman et al. 1994):

$$\frac{\partial y_s}{\partial t} = \frac{q_s}{A_s} \quad \textbf{(1)}$$

$$h_S = Z_S + y_S \quad \textbf{(2)}$$

$$q_S = \sum_i q_{is} - \sum_j q_{sj} \quad \textbf{(3)}$$

and the following equations for each link (between nodes *i* and *j*) and each node *k*:

$$h_i - h_j = f(q_{ij}) \quad \textbf{(4)}$$

$$\sum_i q_{ik} - \sum_j q_{kj} - Q_k = 0 \quad \textbf{(5)}$$

where the unknown quantities are:

y_s	=	height of water stored at node *s,* (L);
q_s	=	flow into storage node *s*, (L^3 T^{-1});
q_{ij}	=	flow in link connecting nodes *i* and *j,* (L^3 T^{-1});
h_i	=	hydraulic grade line elevation at node *i,* (L); and

the known constants are:

A_s	=	cross-sectional area of storage node *s,* (L^2);
Z_s	=	elevation of node *s,* (L);
Q_k	=	flow consumed (+) or supplied (-) at node *k,* (L^3 T^{-1}); and
$f(\cdot)$	=	functional relation between head loss and flow in a link.

Equation (1) expresses conservation of water volume at a storage node while equations (3) and (5) do the same for pipe junctions. Equation (4) represents the energy loss or gain due to flow within a link. For known initial storage node levels y_s at time zero, Equations (4) and (5) are solved for all flows q_{ij} and heads h_i using equation (2) as a boundary condition. The system of equations is solved using a technique known as the gradient method, and the reader is referred to Todini and Pilati (1987), Salgado et al. (1988), and Rossman (1994) for details.

WATER-QUALITY SIMULATION MODEL

The fate of a dissolved substance flowing through a distribution network over time is tracked by EPANET's dynamic water-quality simulator. To model water quality of a distribution system, EPANET uses flow information computed from the hydraulic simulation as input to the water-quality model. The water-quality model uses the computed flows to solve the equation for conservation of mass for a substance within each link connecting nodes i and j, such that:

$$\frac{\partial c_{ij}}{\partial t} = \frac{q_{ij}}{A_{ij}} \frac{\partial c_{ij}}{\partial x_{ij}} + \theta\left(c_{ij}\right) \qquad \textbf{(6)}$$

where:

c_{ij}	=	concentration of a substance in link i, j as a function of distance and time (i.e., $c_{ij} = c_{ij}(x_{ij}, t)$), ($ML^{-3}$);
x_{ij}	=	distance along link i, j, (L);
q_{ij}	=	flow rate in link i, j at time t, (L^3T^{-1});
A_{ij}	=	cross-sectional area of link i, j, (L^2); and
$\theta(c_{ij})$	=	rate of reaction of constituents within link i, j, ($ML^{-3}T^{-1}$).

Equation 6 must be solved with a known initial condition at time zero and the following boundary condition at the beginning of a link (i.e, at node i) where $x_{ij} = 0$:

$$c_{ij}\left(0, t\right) = \frac{\sum_k q_{ki} c_{ki}\Big|_{L_{ki}, t} + M_i}{\sum_k q_{ki} + Q_{si}} \qquad \textbf{(7)}$$

where:

$c_{ki}\big|_{L_{ki},t}$ = concentration at end node of link k, i, of length L_{ki}, (ML^{-3});

L_{ki} = the length of link k, i, (L);

M_i = the mass of a substance introduced by any external source at node i, (M); and

Q_{si} = the flow rate of the source, (L^3T^{-1}).

The summations are made over all links k, i that have flow into the head node i, of link i, j. Note, the boundary condition for link i, j depends on the end node concentrations of all links k, i that deliver flow to link i, j. Hence, Equations (6) and (7) form a coupled set of differential and algebraic equations over all the links in the network. These equations are solved using a numerical method called the Discrete Volume-Element Method; for details the reader is referred to Rossman et al. (1993) and Rossman (1994).

The EPANET water-quality simulator provides a mechanism to account for the gain or loss of a substance by considering its reaction as it travels through the distribution system ($\theta(c_{ij})$ in Equation (6)). For the intended use of the present study, however, the transport of conservative constituents (no reaction, transformation, or decay) will be analyzed. Therefore, the reaction rate, $\theta(c_{ij})$, is set equal to zero for all simulations.

Identifying the source of delivered water in a distribution system has become a necessity when trying to determine the location of a source that may supply water that exceeds a given level of a chemical or biologic constituent. Wood and Ormsbee (1989) developed an explicit method to calculate the percentage of flow, under steady flow conditions, originating at various source points at a specific location in a distribution system. EPANET also has the ability to track the percentage of water reaching any point in the distribution network over time from a specified location (source) in the network (i.e., the "proportionate contribution" of water from a specified source). In this case, the value of c_{ij} in Equation (7) is set at 100 percent for the source location and the value of c_{ij} in Equation (6) becomes the percentage of flow the source location has contributed to the location of interest. Given the multiple number of points of entry to the water-distribution system serving the residents of Dover Township, the ability to track the percentage of water originating from a point of entry becomes a very useful analysis tool for this investigation.

HYDRAULIC MODEL CALIBRATION

OVERVIEW

With a computer model of the water-distribution system, we are trying to reproduce the behavior of a "real-world" hydraulic system as closely as feasible in terms of spatial and temporal characteristics. The collection of field data (previously described) provides an opportunity to understand the operation of the real system at a specified number of locations and times. Such efforts are consistent with the findings of the American Water Works Association Engineering Computer Applications Committee which indicate that "true model calibration is achieved by adjusting whatever parameter values need adjusting until a reasonable agreement is achieved between model-predicted behavior and actual field behavior" (AWWA Engineering Computer Applications Committee 1999). Once a model is considered to be calibrated, it can then be used to, among other purposes, estimate hydraulic characteristics of the real-world system at locations where measured data are unavailable or unknown, spatially and temporally.

In the United States, definitive standards to assess the accuracy of model calibration have yet to be agreed upon or established. However, the following calibration criteria have been suggested:

(1) An average pressure difference of ±2.2 psi with a maximum difference of ±7.3 psi for a "good" data set, and an average pressure difference of ±4.3 psi with a maximum difference of ±14.2 psi for a "poor" data set (Walski 1983); and

(2) The difference between measured and simulated values should be ±5 psi to ±10 psi (Cesario and Davis 1984).

We have used these criteria as general guidelines and have taken into account the availability and accuracy of the data for the water-distribution system serving the Dover Township area. Therefore, we selected a pressure difference at the test-hydrant locations (difference between measured and simulated pressure) of ±5 psi to ±7.5 psi as the calibration criteria for the model of the Dover Township area water-distribution system.

According to the AWWA Engineering Computer Applications Committee (1999), 10 sources of possible error could cause poor agreement between simulated model values and measured field values. These sources of error, which provide a potential list of factors that can be adjusted during the model-calibration process, are: (1) errors in input data (measured and typographic), (2) unknown pipe roughness values (i.e., Hazen-Williams "C-Factors"), (3) effects of system demands (distributing consumption along a pipe to a single node), (4) errors in data derived from network maps, (5) node elevation errors, (6) errors introduced by time variance of parameter values such as storage tank water levels and pressures, (7) errors introduced by a skeletal representation of the network as opposed to modeling all small-diameter pipes, (8) errors introduced by geometric anomalies or partially closed valves, (9) outdated or unknown pump-characteristic curves, and (10) poorly calibrated measuring equipment including data loggers, tank water-level monitors, and SCADA systems. We will discuss these sources of possible model error as they relate to calibrating the model of the distribution system serving the residents of Dover Township in a subsequent section of this report (see section on "Calibration Procedures").

REQUIREMENTS FOR MODEL INPUT

The EPANET water-distribution system model (Rossman 1994) was used in conjunction with the collected 1998 field-test data (previously described in the section on "Field-Data Collection Activities") to develop and calibrate a model of the present-day (1998) water-distribution system serving the Dover Township area. Information required to conduct a simulation using EPANET include data describing pipeline characteristics, booster pump characteristics, groundwater-well pumping factors, consumption and diurnal demand patterns, tank geometries and initial water levels, and simulation time parameters. Table 5 describes the set of input data properties needed to model components of the water-distribution system serving the Dover Township area using EPANET. Specific requirements for the creation of a data input file and the necessary data formats are provided in the EPANET Users Manual (Rossman 1994). (The manual is available on the Internet at the following address: http://www.epa.gov/ORD/NRMRL/wswrd/epanet.html. The data for developing the physical network attributes (e.g., pipe lengths, diameters) required for the EPANET input file for simulating conditions of March and August 1998 were obtained from databases supplied by the water utility (Flegal, 1997), and from the field data ATSDR collected (refer to section on "Field-Data Collection Activities"). Sources for the data and model parameter values are listed in Table 6 and are described below.

JUNCTION DATA

EPANET identifies junctions (or nodes) as the beginning and ending points associated with each pipe or pipe segment in the model network. Each junction is assigned an alpha-numeric identification label, an elevation, a demand (or consumption) value, and a demand pattern number (Table 5). Because the goal of the investigation is to conduct a population-based assessment, geo-spatial location information for pipe junctions, pipelines, and network facilities is required. Geographic coordinates of the model network (in decimal degrees and New Jersey State Plane coordinates) were determined using GPS equipment to obtain locations of the 50 test and alternate hydrants described above, in conjunction with TIGER/Line™ files (1992) for the Dover Township area (coordinate values of the hydrants are listed in Table 2). These known coordinates were used to geo-reference all model nodes (and links) in the distribution-system

Table 5. *Set of input data properties required by EPANET to model the water-distribution system serving the Dover Township area, New Jersey*[1]

Component	Properties
Junction	Identification label Elevation Demand Demand pattern
Tanks	Identification label Bottom elevation Initial water level Minimum allowable water level Maximum allowable water level Tank diameter
Pipes	Identification label Start node label End node label Length Diameter Roughness coefficient Status (Open / Closed)
Pumps	Identification label Start node label End node label Head-discharge curve
Pump Controls	Pump identification label Control Type (Time, Low Level, High Level) Pump Setting (Open or Closed) Control Setting (Time or Tank ID / Level)
Patterns	Identification label Multiplication factors
Time Parameters	Duration Hydraulic Time Step Pattern Time Step

[1]From Rossman (1994, 1999).

Table 6. *Sources for data and model parameter values used to construct the water-distribution system model, Dover Township area, New Jersey*

Data or Model Parameter	Source for Data or Model Parameter	Modified During Calibration	Comments
Physical Data			
Network and pipeline geometry	UWTR electronic data files	No	Network pipelines range from 2 inches to16 inches in diameter
Test hydrant locations	UWTR; ATSDR	No	Horizontal and vertical control of hydrants determined by ATSDR
Hydraulic Data			
Pressure data from test hydrants	ATSDR-supplied pressure data loggers	No	1-minute sampling data averaged to hourly values for model
Ground and elevated storage	UWTR SCADA output to data file	No	5-minute output; value for each hour of test used for model
Ground-water well production	UWTR SCADA output to computer screen and total	No	ATSDR staff recorded well production from screen output
High service and booster pump flows	UWTR SCADA output to data file	No	15-minute output; average value over each hour of test used for
Model Data			
Pipe roughness ("C-factor")	EPANET Users Manual (Rossman 1994)	No	See Table 8 for details
Pump rating curves	UWTR data	Yes	Data obtained in early 1990s
System demand factors	UWTR SCADA production data output to	Yes	Factors derived from instantaneous production data
Nodal demand	UWTR metered consumption data	No	Quarterly data for October 1997 - April 1998 by meter location;

network. The model of the 1998 water-distribution system has 14,987 junctions or nodes.

For the model of the Dover Township area, nodal values of elevation were derived by relating the geo-spatial locations of the pipe junctions to elevation data derived from the USGS 7½-minute DEM quadrangles that cover the Dover Township area using GIS software. For modeling, pipelines were assumed to be located at land surface and not buried below ground level. For junctions assigned to test-hydrant locations, elevations for land surface at the hydrant locations were determined using GPS equipment and verified using DEM data, as previously explained (See section on "Hydrant Selection").

Demand or consumption was assigned to model nodes based on data provided by the water utility. For the Dover Township area, metered consumption data were available for the area serviced by the water utility solely on a quarterly basis. Thus, each meter is read four times per year; however, all meters are not read at the same time or even within a few days of each other. Quarterly consumption data for October 1997 through April 1998, representing the distribution of consumption in the Dover township area, were used for both the March and August 1998 simulations. Metered data specifically representing the distribution of consumption for the August 14-16, 1998, test, or for peak-demand conditions, were not available to investigators for simulating the August 1998 test. To use the model to simulate network configurations that differ from the present-day (1998) network (i.e., historical network configurations), data on total demand and the nodal distribution of consumption should be obtained.

The quarterly consumption data for October 1997 through April 1998 obtained from the water utility for all meter locations in the distribution system were averaged and allocated to model nodes. This was accomplished by using an address-matching technique and GIS software to locate the water meter address provided by the water utility with the closest model node. If more than one meter address was adjacent to a model node, then the consumption for the node was the sum of all metered consumption adjacent to the node. The spatial distribution of average consumption assigned to model nodes is shown in Plate 7. Values range from 0.001 gallons per minute (gpm) to about 6.1 gpm with a mean of about 0.4 gpm. (In EPANET, a positive demand or consumption value indicates outflow from the network; a negative demand value indicates inflow or supply to the network.)

Supply of water to the distribution system can be input to EPANET by assigning a negative demand value. Using this approach, groundwater wells supplying either storage tanks or pumping directly into the distribution system (Table 1) were simulated by assigning the wells to a model node and specifying a negative demand value for the node. The demand for the node was set equal to the rated capacity of the well (Table 1) for individual wells (e.g., well 21), or the combined rated capacity for a group of wells in a well field (e.g., Parkway well field wells). The actual amount of water supplied by the wells on an hourly basis when they were pumping was varied by use of a demand pattern (discussed below) associated with the well node.

The last parameter associated with junction data is the demand pattern. With this parameter, EPANET has the ability to modify the nodal demand data based on the demand pattern. For example, if the water utility serviced residential, commercial, and industrial users, each group of water users might have a different diurnal demand pattern and therefore, nodal demand data would need to be modified depending on the type of use. This is accomplished in EPANET by assigning a demand pattern number to each junction. To enter demand patterns for the Dover Township area, the diurnal demand pattern specific to each type of user, derived from diurnal demand data (e.g., Figure 1), is required. However, investigators did not have this information. Rather, the only demand data available was the data obtained from the water utility's SCADA system for the supply of water to the overall system (Figure 1). Therefore, for the Dover Township area, all model nodes that were assigned a positive demand value (indicating outflow from the system) used the same diurnal demand pattern--the same demand pattern number was assigned to each model junction identified as having a positive consumption value. Each node representing an individual groundwater well or a combination of wells in a well field (thus having a negative demand value) was assigned a unique demand pattern number. (The hourly demand factors associated with the demand pattern numbers are described below in the section on "Pattern Data.")

TANK GEOMETRY AND INITIAL WATER-LEVEL DATA

Ground-level and elevated storage tanks (Plate 2) are associated with model junctions in EPANET. The parameters used to describe storage tanks in EPANET are listed in Table 5; specific features for each storage tank are listed in Table 7. For this investigation, all storage tanks were modeled as having cylindrical geometries. The initial water level for each tank was determined from data collected by ATSDR staff monitoring the water utility's SCADA system during the March and August 1998 tests. For the ground-level storage tanks, the elevation of the bottom of the tank was determined by using GPS equipment and verified using the USGS 7½-minute DEM data for the Dover Township area. For the elevated storage tanks, the elevation of the bottom of the tank was obtained from data supplied by the water utility (Flegal 1997).

PIPELINE DATA

Data pertaining to the pipeline characteristics constituting the distribution system network were retrieved from electronic computer-aided-design files supplied by the water utility. Parameters required by EPANET to describe pipes include (Table 5): a pipe identification label, starting and ending node labels, length, diameter, roughness coefficient, and the status of the pipe (open or closed). The model network consists of 16,071 pipe segments or links. Table 8 lists the material type for pipes composing the network as of the end of December 1997, the range of pipe diameters for specific material types of pipe, and estimated values for the Hazen-Williams "C-factors" (or roughness coefficient) assigned to pipes for use in model calibration

Table 7. *Storage tank identification, hydraulic data, and dimensions used for EPANET simulations of the water-distribution system serving the Dover Township area, New Jersey*

Tank Identification	EPANET Identification	Bottom Elevation (ft)[1]	Initial Water Level, (ft)		Minimum Allowable Water Level (ft)	Maximum Allowable Water Level (ft)	Tank Diameter (ft)
			March 1998 Simulation	August 1998 Simulation			
Holly Plant ground level	33443-HTA	6.52	8.87	8.80	0.0	20.0	[2]130.0
Holiday City ground level	33530-HCTA	87.12	13.30	13.97	0.0	24.0	82.5
Indian Hill elevated	33564-IHTA	160.0	35.11	37.85	0.0	42.0	48.0
North Dover elevated	33566-NDTA	170.0	36.77	41.13	0.0	51.0	61.5
Windsor ground level	33673-WATA	9.84	23.26	19.39	0.0	24.0	103.0
Route 37 (St. Catherine's) ground level	33684-R37TA	42.93	33.90	35.91	0.0	40.0	71.0
South Toms River elevated	33708-STRTA	166.0	25.37	21.01	0.0	28.0	42.0
Parkway Well Field ground level	33714-PTANK	82.74	10.43	21.44	0.0	24.0	83.0

[1]Datum is sea level.

[2]Effective diameter for two tanks simulated as one tank in EPANET.

Table 8. *Pipeline characteristics of the water-distribution system, Dover Township area, New Jersey*[1]

Material Type[2]	ID	Year First Installed	Year Last Installed	Number of Pipe Segments in Model	Length of Pipe Segments in Model (miles)	Range of Pipe Diameters (inches)	Values of Hazen-Williams "C" Used in Model[3]
Asbestos cement	AC	1950	1981	9,512	289.89	4, 6, 8, 10, 12, 16	120
Cast iron	CI	1950	1975	78	2.01	2, 4, 6, 8, 10, 12	130
Copper	CP	1950	1950	3	0.07	2	130
Ductile iron	DI	1950	1994	194	6.32	6, 8, 12, 16	130
Galvanized	GA	1950	1962	45	1.43	2	120
Plastic	PVC	1950	1997	5,949	177.05	2, 4, 6, 8, 12, 16	140
Plastic	PE	1973	1983	280	5.98	2	140
Plastic	IPS	1981	1981	10	0.15	2	140
Total number of pipe segments (links) in model: 16,071 Total number of pipe junctions (nodes) in model: 14,987 Total length of pipe segments (links) in model: 482.99 miles							

[1]Data for water-distribution system network pipelines as of December 1997.
[2]Data for material type, year first installed, year last installed, and range of pipe diameters from Flegal (1997).
[3]Values for Hazen-Williams "C" from Rossman (1994, Table 2.2).

(see section on "Calibration Parameters" for a discussion of Hazen-Williams "C-factors"). Spatial distribution of the network pipes classified by diameter and by roughness coefficient are shown on Plates 8 and 9, respectively. As shown on Plate 8 and listed in Table 8, the model network of the distribution system is composed of pipes ranging in diameter from 2 in. to 16 in. Additionally, data in Table 8 (column 6) show that most of the network is composed of asbestos cement (60%) and plastic (PVC, 37%) pipes.

PUMP DATA

Booster pumps are used in water-distribution systems to raise the hydraulic head of water and increase the pressure in certain portions of a system. In EPANET, pumps are modeled as separate links. As described in Table 5, each pump in the model is identified by a numeric identification label (Table 9), a start and end node label, and a head-discharge or pump- characteristic curve. The pump-characteristic curve describes the relationship between the hydraulic head imparted to the fluid (water) as a function of the flow rate of the fluid through the pump (Table 9; Appendix F). In EPANET, the pump-characteristic curve is represented as a function with the form of:

$$h_G = h_0 - aq^b \quad \textbf{(8)}$$

where:

h_G	=	head gain imparted by the pump (L),
h_0	=	shutoff head (L),
q	=	flow through the pump (L^3T^{-1}),
a	=	a resistance coefficient, and
b	=	a flow exponent.

EPANET requires a minimum of 3 points--the shutoff head, h_0, and two additional points on the pump characteristic curve (Table 9; Appendix F) to estimate values for coefficients *a* and *b*. Initial pump-characteristic curve data for every booster in the water-distribution system were provided to ATSDR by the water utility (Flegal 1997). These data are listed in Table 9 and shown graphically in Appendix F. The initial pump-characteristic curve data were modified during the calibration process (see section on "Calibration Parameters").

PUMP CONTROL DATA

To control the on/off cycling of booster pumps, EPANET uses pump control data. These data include (Table 5): a pump identification label, the type of control (time or level), the pump setting (open or closed), and the control setting (time or tank water level). When a

Table 9. *Pump identification, characteristic data, calibrated values, and status during March and August 1998 pressure tests, Dover Township area, New Jersey*

Pump Location and Description	EPANET Pump Curve ID	EPANET Pump Id	Initial Pump Characteristic Data[1]		Calibrated Pump Characteristic Data used in EPANET		Pump Status During Test	
			Flow (gpm)	Head (ft)	Flow (gpm)	Head (ft)	March 1998	August 1998
Holly Plant Booster Pump 1	C3-HOLLY1	20003	800.0	285.4	0.0	315.0	OFF	ON
			900.0	270.4	810.0	243.0		
			1000.0	245.4	990.0	180.0		
			1100.0	200.3				
Holly Plant Booster Pump 2	C4-HOLLY2	20004	700.0	290.4	0.0	328.3	OFF	ON
			800.0	280.4	784.0	274.0		
			900.0	265.4	1470.0	196.0		
			1000.0	255.4				
			1100.0	245.4				
			1200.0	235.3				
			1500.0	200.3				
Holly Plant Booster Pump 3	C5-HOLLY3	20005	1900.0	295.4	0.0	306.0	OFF	ON
			2300.0	282.4	2430.0	238.5		
			2700.0	265.4	3150.0	189.0		
			3000.0	250.4				
			3300.0	233.4				
			3500.0	220.3				
Parkway Booster Pump 1	C22-PKWYB1	20022	1500.0	175.3	0.0	209.4	ON	ON
			1800.0	167.2	1742.4	187.4		
			2000.0	158.2	3000.0	135.0		
			2200.0	146.2	3920.0	0.0		
			2400.0	132.2				
			2600.0	115.0				
			2800.0	103.0				
Parkway Booster Pump 2	C23-PKWYB2	20023	3100.0	209.3	0.0	240.0	OFF	ON
			3600.0	202.3	3600.0	202.3		
			3900.0	193.3	4300.0	172.5		
			4300.0	173.2				
Holiday City Booster Pump	C11-HCBP	20011	500.0	134.0	0.0	220.0	ON	ON
			700.0	130.0	900.0	200.0		
			900.0	126.2	1500.0	175.0		
			1200.0	119.0				
			1300.0	116.0				
			1500.0	107.0				

Table 9. *Pump identification, characteristic data, calibrated values, and status during March and August 1998 pressure tests, Dover Township area, New Jersey–Continued*

Pump Location and Description	EPANET Pump Curve ID	EPANET Pump Id	Initial Pump Characteristic Data[1]		Calibrated Pump Characteristic Data used in EPANET		Pump Status During Test	
			Flow (gpm)	Head (ft)	Flow (gpm)	Head (ft)	March 1998	August 1998
Route37 (St. Catherine's) Booster Pump	C13-R37B	20013	450.0	210.0	0.0	240.0	OFF	OFF
			500.0	206.0	500.0	206.0		
			600.0	195.8	700.0	184.0		
			700.0	183.3	2200.0	0.0		
South Toms River Booster Pump 1	C24-STRB1	20024	0.0	320.4	0.0	320.4	OFF	OFF
			100.0	315.5	300.0	290.0		
			200.0	305.4	600.0	210.0		
			300.0	290.4	975.0	0.0		
			400.0	275.4				
			500.0	250.4				
			600.0	210.2				
South Toms River Booster Pump 2	C25-STRB2	20025	0.0	320.4	0.0	320.4	OFF	OFF
			100.0	315.5	400.0	275.0		
			200.0	305.4	600.0	215.0		
			300.0	290.4	975.0	0.0		
			400.0	275.4				
			500.0	250.4				
			600.0	210.2				
Windsor Booster Pump 1	C19-WA1	20019	750.0	245.0	0.0	228.4	OFF	ON
			850.0	242.3	739.5	210.5		
			950.0	236.3	957.0	194.9		
			1100.0	224.3				
Windsor Booster Pump 2	C20-WA2	20020	750.0	245.0	0.0	223.1	OFF	ON
			850.0	242.3	705.5	200.9		
			950.0	236.3	880.0	179.2		
			1100.0	224.3				
Windsor Booster Pump 3	C21-WA3	20021	750.0	245.0	0.0	228.4	OFF	ON
			850.0	242.3	739.5	210.5		
			950.0	236.3	957.0	194.9		
			1100.0	224.3				

[1]From Flegal (1997).

"time" control is used, pumps are cycled on and off using the time of day as the controlling criterion. When a "level" control is used, pumps are cycled on and off using storage tank water-level as the criterion. Input data used to simulate the March and August 1998 tests were obtained from field data ATSDR staff collected monitoring the water utility's SCADA system. During the test periods, the water utility used a time-of-day control setting (rather than a tank water-level control setting) to cycle booster pumps on and off (Table 9) .

PATTERN DATA

EPANET allows for varying of demand values by using a demand pattern number. Pattern data are entered into EPANET by specifying an alpha-numeric pattern identification label, and then supplying factors by which the nodal demand value is to be multiplied. The 48 hourly demand factors for water consumption used in the simulations are listed and presented graphically in Appendices G (March 1998) and H (August 1998). These factors, when multiplied by the nodal consumption, represent the diurnal demand that occurred during the time of the tests in March and August 1998. The 48 hourly demand factors were derived from the water utility's SCADA system demand data recorded by ATSDR staff during the March and August 1998 tests (Figure 1). The demand factors were obtained by using the demand data recorded by ATSDR staff for March 1998 (Figure 1A) and August 1998 (Figure 1B) averaged over a period of one hour and dividing the values by the average demand of 7.6 MGD for March 1998 or 16.1 MGD for August 1998 (compare Figure 1A with demand factors for March 1998 shown in Appendix G and Figure 1B with demand factors for August 1998 shown in Appendix H).

As described above in the section on "Junction Data," supply of water from groundwater wells to the distribution system and to storage tanks was represented by using a negative base demand equal to the rated capacity of a well or well field (Table 1) at the node corresponding to the well or well-field location. The base demand was modified by multiplying a pumping factor for each hour of the simulation by the base demand (the rated capacity of the well). In this manner, supply was provided to the distribution system equaling the hourly amount metered by the water utility's SCADA system and recorded by ATSDR staff during the tests. Tables listing and graphs showing the pumping factors for wells in operation during the tests are provided in Appendices G (March 1998) and H (August 1998).

TIME PARAMETER DATA

EPANET assumes that consumption values, supply rates, and concentrations at source nodes remain constant over a fixed period of time. However, these parameter values can change from one time period to another. To conduct an extended period hydraulic simulation, EPANET requires three time parameters: (1) the duration of the simulation, (2) the hydraulic time-step size, and (3) the pattern time-step size. For the Dover Township simulations, the duration of the simulation was set equal to the duration of the tests–48 hours. The hydraulic and pattern time-step sizes were set equal to 1 hour, which is the default time-step size used by EPANET.

CALIBRATION PROCEDURES

As described in the "Overview" section, model calibration entails adjusting model parameter values until an acceptable match is achieved between measured data and model-simulated values (i.e., pressures at the test hydrants, water levels in the storage tanks, flows from booster pumps, and pumpage from groundwater wells). The Dover Township water-distribution system model was calibrated to the hydraulic and operational data collected during the March 1998 test and then further tested against data collected during the August 1998 test. The model was run as an extended period simulation (EPS) using one-hour hydraulic time steps and demand-pattern factors derived from the control-room data collected during the test. The 10 sources of possible error that could lead to model simulated values not agreeing with measured values discussed previously are listed in Table 10. These sources of error also provide a list of potential model parameters that can be modified during the calibration process. To decide which parameters might require more, less, or no modification, investigators evaluated each parameter as to the qualitative magnitude of error (high, moderate, or low) that could result from uncertainty and variability of the parameter. These evaluations are listed in Table 10. Three of the sources of possible error were evaluated as having a qualitatively high or moderate error magnitude: (1) unknown pipe roughness (Hazen-Williams "C-factor") values, (2) effects of system demands and consumption, and (3) outdated or unknown pump-characteristic curve data. The initial estimates for these three parameters where subjected to possible variation during the calibration process and will be discussed below. The remaining 7 sources of possible error are believed to introduce minor to insignificant errors to model simulations, and therefore, were not modified during the calibration process.

For model calibration (March 1998 test) and testing (August 1998 test), four data comparisons (measured data versus simulated values) were made during the calibration and testing process and these comparisons are summarized in Table 11 and presented in the accompanying appendices. The data comparisons are for (Table 11): (1) pressure at each of the 25 test hydrants (Appendix I [March 1998] and Appendix J [August 1998]), (2) storage tank hydraulic head at each storage tank that was operational during the tests (Appendix K [March 1998] and Appendix L [August 1998]), (3) booster pump flows for each booster pump that was operated during the tests (Appendix M [March 1998] and Appendix N [August 1998]), and (4) groundwater well pumpage for each well that was operated during the tests (Appendix O [March 1998] and Appendix P [August 1998]). In each of these appendices (I-O), graphs and tables that compare measured data with model simulated results are provided.

Table 10. *Qualitative evaluation of sources for model error, water-distribution system model, Dover Township area, New Jersey*

Error Type[1]	Qualitative Estimate of Error			Notes
	High	Moderate	Low	
1. Input data			X	Measurement and typographical
2. Unknown pipe roughness values	X			Hazen-Williams "C-Factors"–no measured
3. Effects of system demands	X	X		Metered consumption data available for
4. Data derived from network maps			X	Data from UWTR databases (Flegal, 1997);
5. Node elevation data			X	Data obtained from USGS DEM
6. Time variance of pressures and water levels			X	Pressures monitored with continuous-recording data loggers; tank water-level data from SCADA, verified by ATSDR staff
7. Skeletal representation of network			X	Not applicable--"street-level" network used
8. Geometric anomalies or partially closed			X	Areas of suspected partially closed valves
9. Outdated or unknown pump-	X			Curves obtained from UWTR (Flegal,
10. Poorly calibrated measuring equipment			X	Data loggers factory calibrated for each test;

[1]List of error sources from AWWA Engineering Computer Applications Committee (1999)

Table 11. *Summary of comparisons between test data and model simulations for the March and August 1998 pressure tests described in report, Dover Township area, New Jersey*

Source of Data	Number of Measurement Locations		Measurement Parameter	Measurement Unit	Location of Data Comparison in Report	
	March 1998	August 1998			March 1998	August 1998
Test hydrants	25	25	Pressure	Pounds per square inch	Appendix I	Appendix J
Storage tanks	5	6	Hydraulic head[1]	Feet	Appendix K	Appendix L
Booster pumps	2	4	Flow	Gallons per minute	Appendix M	Appendix N
Groundwater wells	4	5	Pumpage	Gallons per minute	Appendix O	Appendix P

[1]Water levels measured for storage tanks; hydraulic head is derived by adding elevation of bottom of tank to measured water level

Calibration Parameters

Discussions with the water utility indicated that the network pipes were believed to be very clean, and inspections had shown very little debris. In addition, as shown in Table 8, more than one-third of the pipes (in quantity and lengthwise) are made of PVC where the variation in "C-factor" is negligible. Therefore, initial estimates for "C-factor", obtained from published tabular values (Rossman 1994) and listed in Table 8 for every pipe material type, were not varied during the calibration process. A sensitivity analysis, subsequent to model calibration, (see section on "Sensitivity Analysis") confirms that for this distribution system, variation in "C-factor" has little influence on system pressures and flow directions.

ATSDR staff obtained initial estimates of system demand factors by recording demand data (water production by the utility) during the tests from the control-room SCADA output. During the calibration process, individual hourly factors were modified. Factors for the March test differ from factors for the August test because the conditions of the each test represented different demand conditions–winter-time demand for the March test and peak demand for the August test. The rational used for modifying the individual hourly demand factors for each test follows in the discussion below.

Initial estimates for the demand factors were derived from the instantaneous output of the water utility's SCADA system (Figure 1). These data were recorded at random time intervals and then averaged over a period of one hour. As can be seen in Figure 1, these data are quite variable in times of significant system-demand change. Therefore, initial estimates of demand factors, based on an average over a period of one hour, may not have accurately reflected or have been representative of the system's demand. Therefore, investigators felt justified in modifying the initial estimates of the demand factors. It is important to note, however, that although individual hourly factors were modified, the total system-wide demand, 7.6 MGD for March 1998 and 16.1 MGD for August 1998, was not modified during the calibration process.

Calibrated values for the hourly demand factors for the tests are provided in Appendix G (March 1998) and H (August 1998) are in general agreement with the individual system demand patterns obtained from the water utility's SCADA system (compare Figure 1A with the demand factors shown in Appendix G and Figure 1B with the demand factors shown in Appendix H). Use of system-demand factors in conjunction with measured and recorded hydraulic device operations (pump on/off status, groundwater well status, etc.) helped investigators calibrate the model to conditions of March 1998 and further test the calibration to conditions of August 1998.

The last model parameter adjusted during the calibration process was the booster pump-characteristic curves. The water utility provided initial characteristic-curve data to ATSDR (Flegal 1997). These data are listed in Table 9 and shown graphically in Appendix F. However, the source of these data (e.g., manufacturer's data, field testing) could not be determined. Therefore, investigators believed this was a key parameter that could be modified during the calibration process. Modifications to the original data were made to increase or decrease the characteristic curve so that system operations, as observed during the two tests, could be duplicated as closely as possible. These modifications, however, still provide typical and reasonable head-discharge curve relationships for the water-distribution system. The calibrated characteristic-curve data along with the status of the pumps (on/off) are listed in Table 9; they are shown graphically in Appendix F for the March and August 1998 tests. Of the 11 system pumps listed in Table 9, 3 pumps (Holly booster 2, Parkway booster 2, and Route 37) required very little or no modification during the calibration process. Two pumps (South Toms River booster 1 and 2) were not operated during either the March or August 1998 tests.

CALIBRATION STATISTICS

In an effort to assess the overall quality and reliability of the model calibration, analyses were conducted using calibration statistics. Because measured data for both March and August 1998 were collected in terms of pressure at the test hydrant locations, analysis of calibration statistics will be presented in terms of measured and simulated pressures. The calibration statistic used for the analyses, referred to as "mean absolute pressure difference (δ_P)," is defined as the mean of the absolute value of the difference between measured pressure values and simulated pressure values in pounds per square inch (psi) over the 48-hour duration of the test. This calibration statistic is defined mathematically as:

$$\delta_P = \frac{\sum_{i=1}^{NHM} \left| P_{m_i} - P_{s_i} \right|}{NHM} \qquad \textbf{(9)}$$

where:

δ_P	=	mean absolute pressure difference, (psi),
NHM	=	number of hourly measurements,
P_{m_i}	=	measured pressure at hour *i*, (psi),
P_{s_i}	=	simulated pressure at hour *i*, (psi), and
$\left| P_{m_i} - P_{s_i} \right|$	=	the absolute value of pressure difference, (psi).

Results of the calibration analysis are presented as a series of illustrations (Figures 11, 12, and 13; Plates 10 and 11) and are described below.

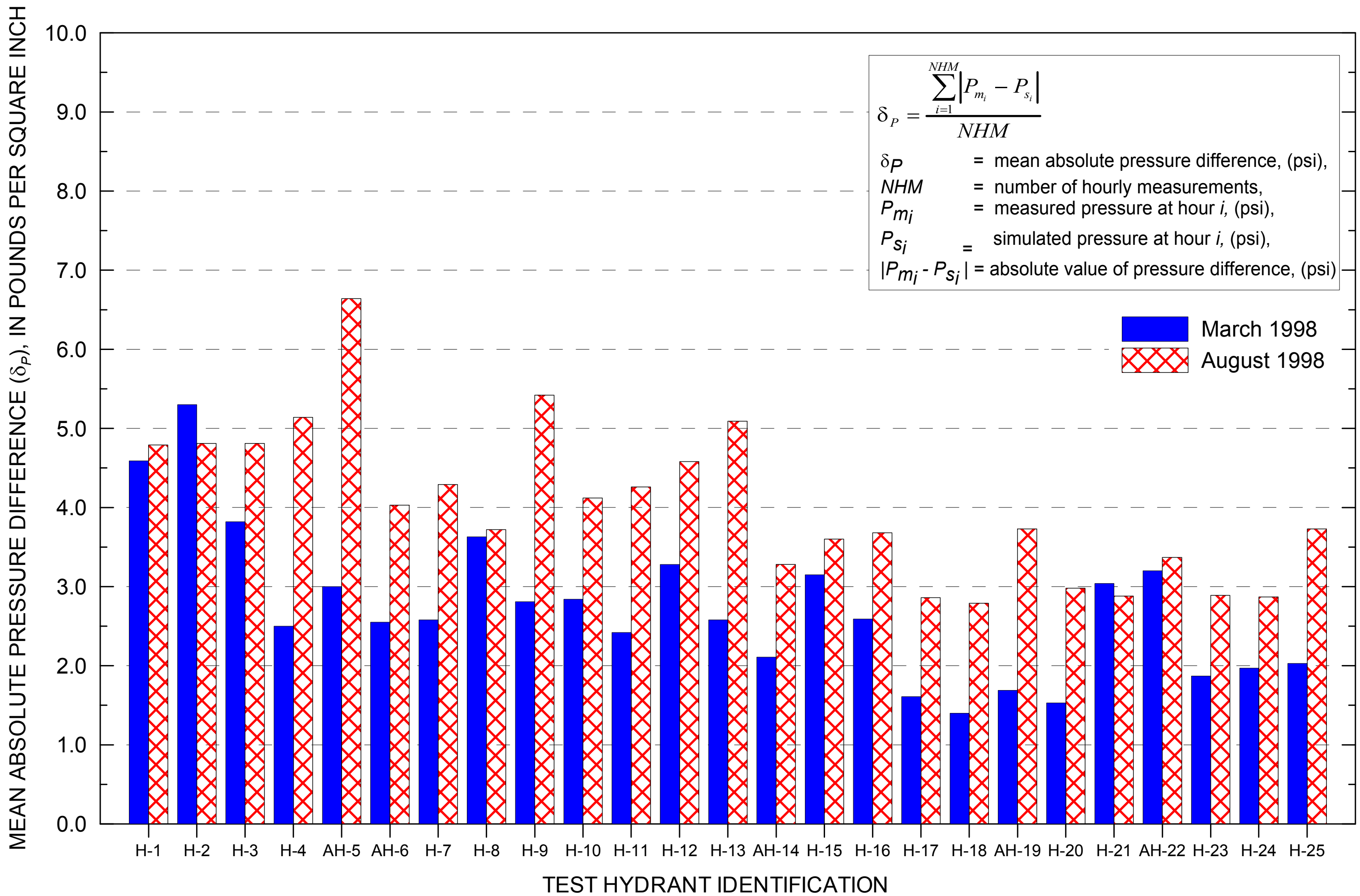


Figure 11. *Comparison of mean absolute pressure difference (δ_P) for test hydrants, March and August 1998 simulations.*

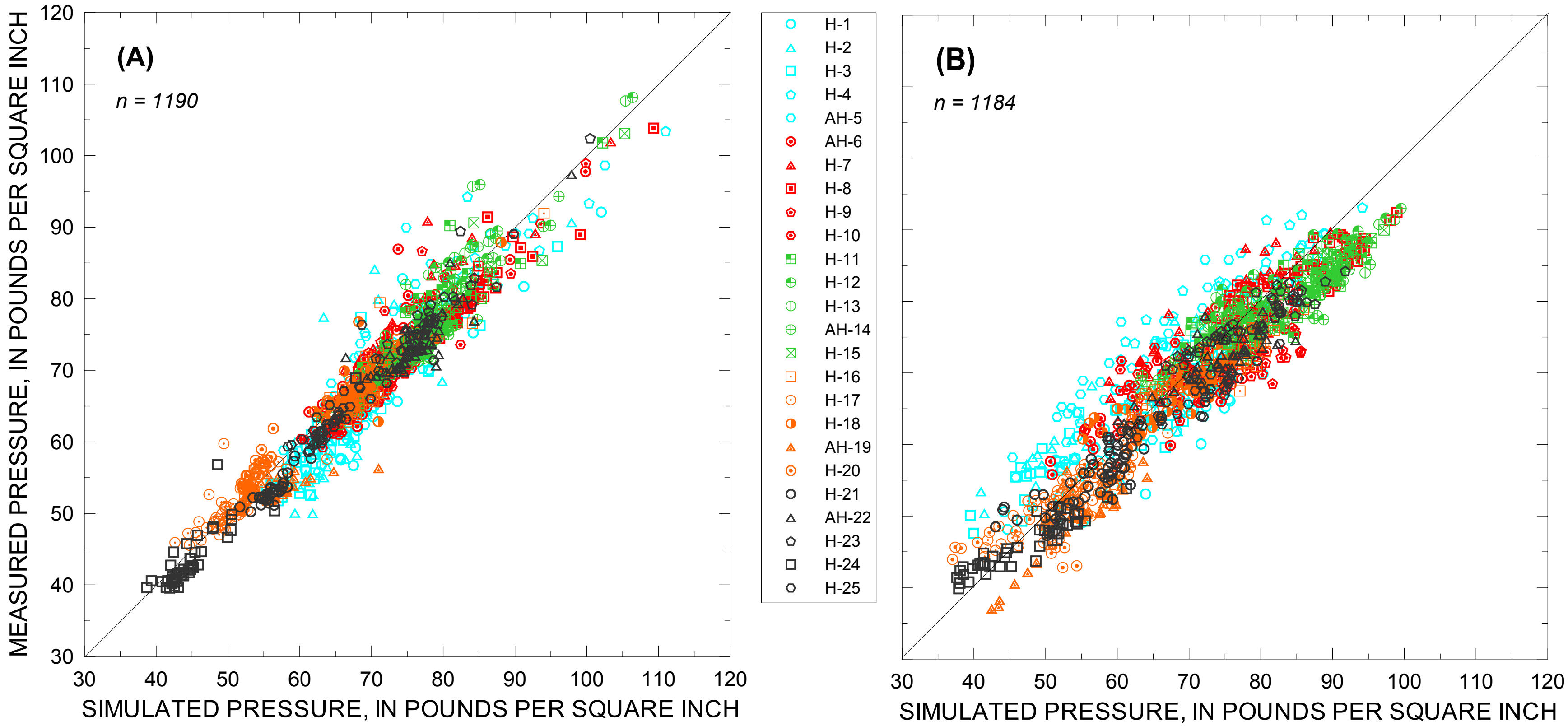


Figure 12. *Comparison of measured and simulated hourly pressure data: (A) March 24-25, 1998; and (B) August 14-15, 1998.*

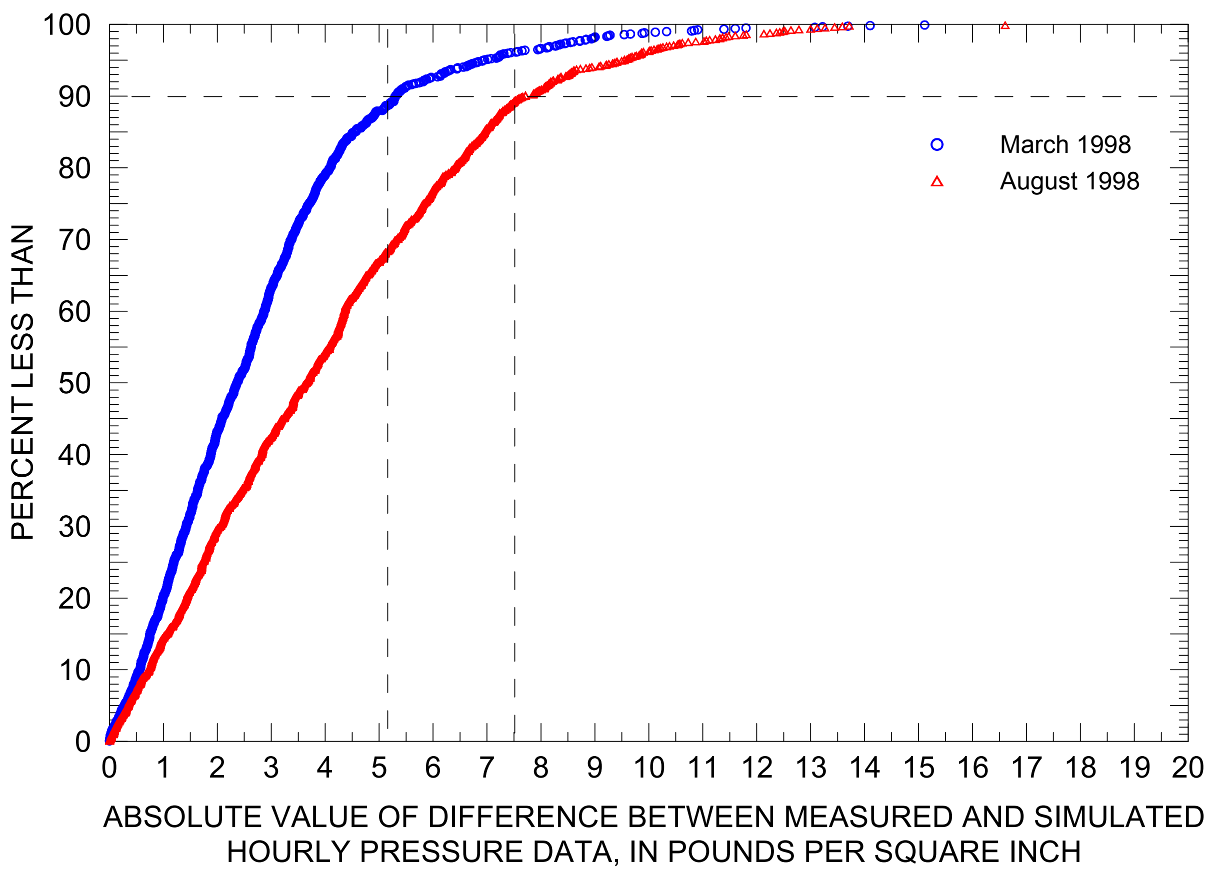
A comparison of δ_P by hydrant (or measuring point) location is presented in Figure 11. The bars on this graph were computed by applying Equation (9) over the 48-hour test period for each location. The data used for the graph indicate that for March 1998, δ_P for all locations ranges from 1.4 psi to 5.3 psi, and for August 1998, δ_P for all locations ranges from 2.9 psi to 6.6 psi. (The absolute pressure differences and δ_P for all hydrants for the tests are also provided in Appendices I [March 1998] and J [August 1998].) System-wide δ_P for March 1998 is lower than for August 1998, and with the exception of hydrant H-2, δ_P for March 1998 is less that 5 psi. For August 1998, system-wide δ_P is also less than 5 psi except for hydrants H-4, AH-5, H-9, and H-13.

To assess the goodness-of-fit of the calibrated model to field conditions, a comparison of measured hourly pressure values with simulated hourly pressure values is presented in Figure 12A (March 1998) and 12B (August 1998). A different plotting symbol is used for each test hydrant location. Ideally, if no difference between measured and simulated pressure data occurred, each hourly data point would coincide with the lines of equality shown in Figure 12. Because a difference exists between measured data and model simulated results, the data scatter about the lines of equality. For the model simulations, the August 1998 data have more scatter than do the March 1998 data, indicating a larger difference between measured and simulated pressure data.

The reliability of the calibrated model can also be judged in terms of the frequency of absolute pressure difference (absolute value of difference between measured and simulated hourly pressure data). Figure 13 is a plot of the frequency of absolute pressure difference for both March and August 1998. The graph indicates that for the March 1998 simulation, 90% or more of the simulated hourly values result in an absolute pressure difference of about 5 psi or less. For the August 1998 simulation, data presented in Figure 13 indicate that 90% or more of the simulated hourly values result in an absolute pressure difference of about 7.5 psi or less.

To view the areal distribution of δ_P in terms of the spatial locations of the test hydrants, δ_P for each test hydrant is plotted using the test hydrant location map from Plate 6. The δ_P for each test hydrant is shown on Plates 10 (March 1998) and 11 (August 1998). Comparison of Plates 10 and 11 indicates that the largest δ_P occur in the southern Dover Township and Berkeley Township areas, and that overall, the March 1998 simulation has a smaller δ_P. Primarily we attribute this characterization of the difference in pressure to the lack of metered consumption data available specifically during the test periods. As discussed above, metered consumption data were available for the Dover Township area serviced by the water-distribution system solely on a quarterly basis; thus, each meter was read four times per year. However, all meters were not read at the same time nor within a few days of each other. For the March 1998 simulation, modelers had quarterly consumption data available for October 1997 through April 1998. These same data were used for the August 1998 simulation because metered data representing consumption conditions for August 14-16, 1998, or for the peak-demand season were not available to investigators for the August 1998 simulation. To further reduce δ_P, metered consumption data (and consequently, nodal consumption data) during the time of the test, and specifically during the peak-demand test, would be required.

Figure 13. *Frequency of absolute pressure difference between March and August 1998 test data and simulations.*



Overall, δ_P for the March and August 1998 simulations are within the calibration criteria of ±5 psi to ±7.5 psi for pressure difference selected as a calibration target by investigators, and within the criteria suggested by Cesario and Davis (1984) and Walski (1983) previously discussed (see "Overview" section above). The March and August 1998 model simulations are accurate characterizations of the water-distribution system for winter-demand and peak-demand conditions. In addition to the analysis of pressure difference (Figures 11-13; Plates 10 and 11), comparison of measured and simulated pressures for the 25 hydrants presented in this report (Appendices I and J), the comparison of measured and simulated hydraulic head at ground-level and elevated storage tanks (Appendices K and L), comparison of measured and simulated booster pump flows (Appendices M and N), and comparison of measured and simulated groundwater well flows (Appendices O and P) support the assertion that the model presented herein is an acceptable and reliable representation of water-distribution system conditions existing during 1998.

SENSITIVITY ANALYSIS

Model analyses give rise to parameter uncertainty because the models use parameters for which data values may be incomplete, missing, unknown, or contain errors. As previously discussed, up to 10 sources of error may be introduced when calibrating a water distribution system model (AWWA Engineering Computer Applications Committee 1999). Therefore, it is important to understand and acknowledge which parameters may have a substantial effect on model output results if the parameter value is varied. Four methods exist for quantifying parameter uncertainty (EPA 1997): (1) sensitivity analysis, (2) analytical uncertainty propagation, (3) probabilistic uncertainty analysis (Monte Carlo simulation), and (4) classical statistical methods. The sensitivity analysis method will be used for the current study.

For the sensitivity analysis method, a model parameter value is varied from the calibrated value, and the resulting model output is then compared with the calibrated values of model output to assess the sensitivity of the model to that particular parameter. As shown in Table 10, the parameters estimated to have high or moderate sources for error, and thus presumably the parameters with the greatest uncertainty, are pipe roughness values (Hazen-Williams "C-factor"), effects of system demands (variation of consumption), and outdated or unknown pump-characteristic (head-discharge) curves.

Of the three parameters that are presumed to have the greatest source for error, two parameters, the effects of system demand and outdated or unknown pump-characteristic curve data, are dependent on one another. That is, if a modification is made to a pump-characteristic curve so that more or less water is supplied for a given head, a change must also be made to system demands (consumption) to increase or decrease demand, respectively. This is because in water-distribution system models such as EPANET, supply must equal demand–if supply is increased, demand must be increased. Thus, it is not possible to change one of these parameters without changing the other parameter. Because of this, it is not possible to determine the sensitivity of the model to a variation in just one of these parameters.

The third of the model parameters with the greatest uncertainty for this investigation is pipe roughness (Hazen-Williams "C-factor") values. The values chosen for this parameter will cause higher or lower friction losses for water flowing through network pipes. Using the Hazen-Williams roughness coefficient to express the head loss due to friction associated with flow through a pipe, the following head loss formula is used in EPANET:

$$h_L = (4.72 C^{-1.85} d^{-4.87} L) q^{1.85} \qquad \textbf{(10)}$$

where:

h_L = the head loss, (ft),
C = Hazen-Williams roughness coefficient,

d	=	pipe diameter, (ft),
L	=	pipe length (ft), and
q	=	flow in the pipe, (ft^3/s).

Thus, as the value assigned to C decreases, the head loss due to friction increases. As shown in Equation (10), pipe diameter could have an effect on head loss similar to varying C. This uncertainty is introduced into the model by using nominal pipe diameters rather than the manufacturer's specified internal pipe diameter.

Table 12. *Variation of parameter values used to conduct sensitivity analyses using the calibrated water-distribution system model, Dover Township area, New Jersey*

Simulation Identification	Pipe Material	Calibrated Value	Sensitivity Analysis Value	Number of Pipe Segments
Pipe Diameter Variation, in inches				
[2, 3]SENS1	Asbestos cement (AC)	4	[1]3.9927	276
	Asbestos cement (AC)	6	5.85039	3,894
	Asbestos cement (AC)	8	7.85039	3,092
	Plastic (PVC)	4	4.23622	234
	Plastic (PVC)	6	6.08661	1,407
	Plastic (PVC)	8	7.98425	2,851
	Plastic (PVC)	12	11.6457	1,169
Roughness Coefficient (Hazen-Williams "C") Variation				
SENS2	Asbestos cement (AC)	120	140	9,506
SENS3	Cast iron (CI) Ductile iron (DI)	130	110	272
SENS4	Plastic (PE, IPS, PVC)	140	150	6,233
[4]SENS5	AC, CI, DI, PE, IPS, PVC	120, 130, 140	140, 110, 150	16,011

[1]Internal diameter pipe data obtained from United Water Toms River, Inc., February 1999.
[2]The internal diameter for 10-inch and 12-inch AC pipe is the same as the nominal diameter and was not varied.
[3]There were no 10-inch PVC pipes in the distribution-system network as of the end of December 1997.
[4]Simulation SENS5 is a combination of simulations SENS2, SENS3, and SENS4.

For the sensitivity analysis, values assigned for roughness coefficient, C, and pipe diameter, d, were varied from the calibrated values. Internal pipe diameter data were supplied by the water utility. Values for roughness coefficient were obtained from the range of C values generally associated with a particular type of pipe material (Rossman 1994, Walski 1984). Table 12 lists the parameter value variations for *C* and *d* and the simulations conducted as part of the sensitivity analysis. For pipe diameter variation, one sensitivity simulation was conducted for all varied diameters (SENS1 in Table 12). For roughness coefficient variation, a sensitivity simulation was conducted for each pipe material type (SENS2, SENS3, and SENS4, in Table 12). A simulation was also conducted that included all the variations in roughness coefficient in one simulation (SENS5 in Table 12). The sensitivity simulations identified in Table 12 were conducted for both the March and August 1998 test conditions.

To compare sensitivity simulation results with calibrated model results (March and August 1998), two statistics were computed for each simulation. The statistics, the root-mean square (*RMS*) and the relative simulation error (*RSE*), are defined as follows:

$$RMS = \sqrt{\frac{\sum_{i=1}^{NH}\sum_{j=1}^{NTS}\left(P_{m_{i,j}} - P_{s_{i,j}}\right)^2}{NH \times NTS}} \qquad \textbf{(11)}$$

and

$$RSE = \frac{\sum_{i=1}^{NH}\sum_{j=1}^{NTS}\left|P_{m_{i,j}} - P_{s_{i,j}}\right|}{P_{m_{i,j}}} \times 100\% \qquad \textbf{(12)}$$

where:

RMS	=	root-mean-square of pressure difference (psi),
RSE	=	relative simulation error (percent),
NH	=	number of test hydrants (25),
NTS	=	number of hourly time steps in the simulation (48),
$P_{m_{i,j}}$	=	measured pressure data for hydrant *i*, at hour *j*, (psi),
$P_{s_{i,j}}$	=	simulated pressure for hydrant *i*, at hour *j*, (psi), and
$\left\|P_{m_{i,j}} - P_{s_{i,j}}\right\|$	=	the absolute value of pressure difference (psi).

These two statistics were calculated for the simulations using the calibrated parameter values (March and August 1998) and for each sensitivity simulation where parameter values were varied. Results are shown in Table 13. For both the March and August 1998 conditions, results in Table 13 clearly indicate that the model of the distribution system is insensitive to variations in diameter and roughness coefficient. The calculated statistics for the two demand conditions (March and August 1998) vary in tenths of psi for the *RMS* and tenths of a percent for the *RSE*. Thus, obtaining additional information on roughness coefficient (through additional field testing) or using actual pipe diameters (instead of nominal diameters) would not significantly change or reduce parameter uncertainty for our situation.

Table 13. *Comparison of simulation statistics for the water-distribution system model using calibrated and varied parameters, Dover Township area, New Jersey*

Simulation Identification	March 1998 Conditions		August 1998 Conditions	
	[1]Root-mean-square, *RMS* (psi)	[2]Relative simulation error, *RSE* (percent)	Root-mean-square, *RMS* (psi)	Relative simulation error, *RSE* (percent)
Calibration	3.7	4.3	4.8	5.8
[3]SENS1	4.1	4.8	4.9	6.0
SENS2	4.4	5.1	4.6	5.5
SENS3	4.2	5.0	4.8	5.8
SENS4	4.2	4.9	4.8	5.6
SENS5	4.4	5.1	4.6	5.5

[1]Refer to Equation (11) in text for definition of *RMS*
[2]Refer to Equation (12) in text for definition of *RSE*
[3]Refer to Table 12 for definition of sensitivity simulations SENS1 - SENS5

WATER-QUALITY SIMULATION

The panel of expert peers who reviewed the ATSDR modeling effort in December 1998 (ATSDR 1999), recommended that ATSDR obtain a set of water-quality calibration data to assess the reliability of the model calibration for fate and transport simulations. There are two methods that could be used to accomplish this:

(1) Introducing a tracer into the water-distribution system and collecting data simultaneously for pressure, flow, and concentration of the tracer at different sampling locations; or

(2) Collecting water samples at different sampling locations and analyzing the samples for a naturally occurring tracer already present in the water throughout the distribution system.

We chose the second method because: (1) it was less intrusive, (2) the data were readily available, and (3) at the time the pressure data were collected (March and August 1998), it was not possible to collect flow and water-quality data because of institutional, operational, and budgetary constraints. ATSDR investigators obtained water-quality data from sampling events that occurred on March 28, April 4, and April 24, 1996 (NJDHSS 1999c). These data are used to compare with results of a water-quality simulation described below.

On March 28, 1996, NJDHSS collected water samples from taps at 21 schools located throughout Dover Township and serviced by the water utility (Plate 12, Table 14). The samples were analyzed for, among other constituents, the naturally occurring element, barium. Naturally occurring barium is a conservative constituent in groundwater and as such, the concentration will not vary significantly over time. Therefore, a one-time collection of water samples is sufficient for our purposes of testing the reliability of the model calibration for use in fate and transport simulations. On April 4, 1996, NJDHSS also collected water samples from 5 points of entry to the distribution system (the assumed sources for the barium). These points of entry were (Plate 12; Table 14): well 32 (South Toms River), well 20 (Indian Head), well 31 (Route 70), and the Parkway ground-level storage tank. An additional point of entry, the Holly Plant ground-level storage tank, was sampled on April 24, 1996. This tank was sampled at a later date than were the other points of entry because all wells supplying the tank (wells 21, 30, and 37) were off-line on the day the other points of entry were sampled (April 4, 1996), but well 30 was on-line when the tap samples were obtained at the 21 schools (March 28, 1996). These points of entry were the only sources of water to the distribution system during this time period. The measured concentrations of the barium samples at the 21 school locations and the 6 points of entry are shown on Plate 12 and listed in Table 14.

To simulate the spatial distribution of barium, information on the operating schedule for pumps and wells (cycling on/of schedules) and on tank levels for March 28, 1996, was obtained from the water utility. In addition, data for the daily production for March 28 were also obtained. The system-demand factors based on the March 1998 field-test data were modified so that the average daily production for March 28, 1996, (which was greater than the calibration period of March 24-25, 1998) could be distributed on an hourly basis. The concentrations for barium measured at the points of entry were used as the source concentration and assigned to the model node associated with a specific point of entry (Table 14). For the EPANET simulation, the hydraulic time step was set at 1 hour and the water-quality time step was set at 5 minutes. Initial conditions must be "flushed out" of the distribution system before retrieving

Table 14. *Identification of barium sampling locations and comparison of measured and simulated barium concentration values, Dover Township area, New Jersey*

Sample Location[1]	Model Node	Concentration, in µg/L		Difference		[5]Model Bias
		[2]Measured	[3]Simulated	µg/L	[4]%	
Distribution System Sampling Locations (schools)[6]						
Silver Bay Elementary	9507	50.9	43.1	7.9	15.4	0.85
North Dover Elementary	5134	31.5	34.7	-3.2	10.2	1.10
Ocean County College	14429	49.1	40.2	8.9	18.1	0.82
Intermediate East	14409	48.1	40.7	7.4	15.4	0.85
Hooper Avenue Elementary	8668	52.6	40.7	11.9	22.6	0.77
Intermediate West	4564	45.1	36.2	8.9	19.7	0.80
Toms River High School North	4799	40.0	42.5	-2.5	6.3	1.06
East Dover Elementary	7089	48.3	36.0	12.4	25.6	0.75
Cedar Grove Elementary	13446	50.7	42.0	8.7	17.2	0.83
Saint Joseph Elementary	3087	39.0	38.8	0.2	0.6	0.99
Toms River High School South	12131	35.7	40.7	-5.0	13.9	1.14
West Dover Elementary	2106	23.3	22.7	0.6	2.6	0.97
Walnut Street Elementary	3139	37.5	43.8	-6.3	16.8	1.17
Ocean County Votech	12863	50.3	41.8	8.5	17.0	0.83
Toms River High School East	13877	47.5	39.6	7.9	16.7	0.83
Ambassador Christian	2894	37.5	43.0	-5.5	14.7	1.15
Mnsgr. Donovan High School.	3089	41.5	39.1	2.5	5.9	0.94
South Toms River Elementary	2388	13.0	12.6	0.4	13.1	0.97
Washington Street Elementary	5828	32.6	39.2	-6.6	20.3	1.20
Toms River Special Education	14125	45.2	35.3	9.9	21.9	0.78
Alternate Learning Center	1191	12.8	12.6	0.2	1.6	0.98
Distribution System Point of Entry Sampling (sources)[7]						
Berkeley wells (#33)	1351	23.0				
Holly Plant storage tank	33443-HTA	43.9				
Indian Head well (#20)	44230	49.0				
Parkway well field storage tank	33714-PTANK	51.0				
Route 70 well (#31)	44322	35.0				
South Toms River well (#32)	16198	13.0				

[1]See Plate 12 for sampling and point of entry locations
[2]Data from NJDHSS (1999c)
[3]Simulation results for 08:00 hours on March 28, 1996
[4]Relative difference in percent, computed using Equation (14). Mean relative difference for the 21 school samples is 13.6%
[5]Model bias = simulated value / measured value
[6]Date of sampling is March 28, 1996
[7]Date of sampling is April 4, 1996 except for Holly Plant storage tank whose sampling date is April 24, 1996

fate and transport simulation results. Information from the calibrated model of the distribution system indicates that approximately 1,000 hours (42 days) are needed for the entire system to reach a dynamic equilibrium condition in terms of the mixing of a chemical constituent (such as barium) in the storage tanks (e.g., Holiday City ground-level storage tank shown in Appendix Q). Graphs showing the concentration of barium reaching a dynamic equilibrium condition in the storage tanks operating during this time are provided in Appendix Q. Once dynamic equilibrium was reached, EPANET was run for an additional 24 hours. The concentration at model nodes corresponding to the 21 school locations is reported for 08:00 hours, the approximate time that sample collection was completed. Comparison of measured and simulated barium concentrations is presented in Table 14 and shown spatially on Plate 12.

To assess the accuracy of the water-quality simulation with respect to the transport of barium, 5 statistics are computed: (1) the concentration difference (column 5 in Table 14); (2) the percent absolute difference (column 6 in Table 14); (3) the model bias (column 7 in Table 14); (4) the geometric bias; and (5) the correlation coefficient. The mathematical definitions for these statistics are provided below:

$$\Delta C = C_m - C_s \tag{13}$$

where:

ΔC = concentration difference, (µg/L),
C_m = measured concentration, (µg/L), and
C_s = simulated concentration,(µg/L);

$$\delta C = \left| \frac{C_m - C_s}{C_m} \right| x100\% \tag{14}$$

where:

δC = the relative difference, (percent), and
$|\cdot|$ = the absolute value of a function;

$$B_m = C_s / C_m \tag{15}$$

where:

B_M = model bias, (dimensionless);

$$B_G = \exp\left(\frac{\sum_{i=1}^{NS} \ln\left(C_{s_i} / C_{m_i}\right)}{NS}\right) \quad \textbf{(16)}$$

where:

B_G	=	geometric bias, (dimensionless),
$exp\ (\cdot)$	=	the exponential of a function,
$ln\ (\cdot)$	=	the natural logarithm of a function,
C_{m_i}	=	measured concentration of the *i*th sample, (μg/L),
C_{s_i}	=	simulated concentration of the *i*th sample, (μg/L), and
NS	=	the number of samples (21); and

$$r = \frac{\sum_{i=1}^{NS}\left[\left(C_{m_i} - \mu_m\right) \times \left(C_{s_i} - \mu_s\right)\right]}{\sqrt{\left(\sum_{i=1}^{NS}\left(C_{m_i} - \mu_m\right)^2\right) \times \left(\sum_{i=1}^{NS}\left(C_{s_i} - \mu_s\right)^2\right)}}, \quad \textbf{(17)}$$

$$\mu_m = \frac{\sum_{i=1}^{NS} C_{m_i}}{NS}, \text{ and} \quad \textbf{(18)}$$

$$\mu_s = \frac{\sum_{i=1}^{NS} C_{s_i}}{NS} \quad \textbf{(19)}$$

where:

r	=	correlation coefficient, (dimensionless),
μ_m	=	arithmetic mean of measured concentration values, (μg/L), and
μ_s	=	arithmetic mean of simulated concentration values, (μg/L).

The values in column 5 of Table 14 were computed using Equation (13) and indicate a difference between measured and simulated barium concentrations ranging from -6.6 micrograms per liter (µg/L) to 12.4 µg/L and an absolute difference ranging from 0.2 µg/L to 12.4 µg/L. Using Equation (14) to compute the relative difference (column 6 in Table 14) yields a range of 0.6% to 25.6 % with a mean relative difference of 13.6%. This analysis indicates that model-simulated values are acceptable when compared with measured values. However, a more rigorous analysis of evaluating the accuracy of the model with respect to the water-quality simulation can be conducted using the concept of model bias and the correlation coefficient as described below.

Model bias allows us to test the accuracy of the model with respect to the transport of barium by expressing the bias in terms of a simulated-to-measured ratio (Rogers et al. 1999). This ratio (C_s/C_m), computed using Equation (15) and listed in Table 14 (column 6), has the following properties:

when $C_s/C_m < 1$, there is under prediction by the model,
when $C_s/C_m = 1$, there is exact agreement, and
when $C_s/C_m > 1$, there is over prediction by the model.

The data we have available, which are spatially disparate, are best suited for an analysis using the geometric bias. The geometric bias is the geometric mean of the individual C_s/C_m ratios, and is computed using Equation (16). The geometric mean is used because the distribution of C_s/C_m ratios is skewed like a lognormal distribution. That is, the values are restricted for under prediction (0-1), but are unrestricted for over prediction (anything greater than 1). For the data presented in Table 14, the geometric bias is 0.93, which indicates that the model slightly under predicts the measured values. To assess the correlation of simulated values with the measured data, the correlation coefficient was computed using Equations (17-19). For the data in Table 14, the correlation coefficient is 0.81, indicating a high degree of correlation. These statistical analyses (model bias, geometric bias, and correlation coefficient) provide further evidence that the model of the water-distribution system presented herein (both hydraulic and water quality components): (1) is reasonably calibrated, and (2) provides an acceptable representation of the 1998 water-distribution system characteristics for our intended use.

USE OF THE MODEL FOR EPIDEMIOLOGIC INVESTIGATIONS

In the next phase of this investigation, ATSDR will provide to NJDHSS epidemiologists information on the percentage of water that residences of study subjects may have received from each of the points of entry (wells or well fields) to the water-distribution system–the concept of "proportionate contribution" previously discussed in the "Method of Analysis" section. The application of this methodology can be demonstrated using the trace analysis option available within EPANET in conjunction with the characterization of the present-day (1998) water-distribution system. A trace analysis was conducted for each water source point of entry to the water-distribution system operating during the time of interest (Table 15). Once dynamic equilibrium was reached (as described in the section on "Water-Quality Simulation"), a 24-hour trace analysis simulation was conducted by

Table 15. *Simulated proportionate contribution of water for 1998 conditions from points of entry to selected locations, Dover Township area, New Jersey*

Point of Entry to Water-Distribution System	Model Node Number	Percentage of Water from Point of Entry (24-Hour Average)[1]				
		Selected Location and Model Node Number[2]				
		H-1	H-11	H-17	H-20	AH-22
		6	6762	4551	5329	8932
Winter-Demand Conditions – March 1998						
Berkeley wells 33, 34, 35	1351	99.9	42.7	4.5	0.0	7.0
South Toms River wells 32, 38	16198	0.0	49.0	0.0	0.0	0.0
Brookside well 43	16711	–[3]	–	–	–	–
Indian Head well 20	44230	–	–	–	–	–
Route 70 well 31	44322	0.0	0.0	0.0	100.0	26.2
Parkway ground storage tank	33714-PTANK	0.0	8.1	95.4	0.0	66.5
Holly plant ground storage tank	33443-HTA	–	–	–	–	–
Windsor ground storage tank	33673-WATA	–	–	–	–	–
Sum from all points of entry		99.9	99.8	99.9	100.0	99.7
Peak-Demand Conditions – August 1998						
Berkeley wells 33, 34, 35	1351	99.9	0.0	0.0	0.0	0.2
South Toms River wells 32, 38	16198	–	–	–	–	–
Brookside well 43	16711	0.0	0.0	0.0	0.0	8.4
Indian Head well 20	44230	–	–	–	–	–
Route 70 well 31	44322	0.0	0.0	0.0	100.0	1.4
Parkway ground storage tank	33714-PTANK	0.0	0.0	99.8	0.0	83.1
Holly plant ground storage tank	33443-HTA	0.0	0.0	0.1	0.0	1.2
Windsor ground storage tank	33673-WATA	0.0	100.0	0.0	0.0	5.7
Sum of all points of entry		99.9	100.0	99.9	100.0	100.0

[1]Based on calibrated model of the water-distribution system reaching dynamic equilibrium.
[2]Refer to Plate 6 for locations.
[3]Well(s) supplying distribution system or storage tanks not in operation during this time period.

assigning a trace node to coincide with a known and operating point of entry. Locations of the points of entry for the present-day distribution system are listed in Table 15. Results of the trace analysis were obtained in terms of the percentage of water that any location of interest receives from the trace node or location (water source) as an average over a 24-hour time period. These results are presented as a series of maps displaying the areal distribution of simulated proportionate contribution of water from points of entry to model nodes in the distribution-system network for March 1998 winter-demand conditions (Plates 13-16) and August 1998 peak-demand conditions (Plates 17-22). In addition, results are presented in terms of the proportionate contribution of water to 5 selected nodes (Figure 14; Table 15) coinciding with: (1) test hydrant H-1 representing, the southwestern portion of the study area; (2) test hydrant H-11, representing the south-central portion of the study area; (3) test hydrant H-17, representing the central portion of the study area; (4) test hydrant H-20 representing the northwestern portion of the study area; and (5) test hydrant AH-22, representing the northeastern portion of the study area.

PROPORTIONATE CONTRIBUTION OF WATER FOR THE 1998 SYSTEM

AREAL DISTRIBUTIONS, DOVER TOWNSHIP AREA, NEW JERSEY

Simulated results obtained from the trace analysis for each point of entry to the water-distribution system, operating under winder-demand (March 1998) and peak-demand (August 1998) conditions, are now discussed in terms of generalized areal distributions of the proportionate contribution of water to locations throughout the Dover Township area. Plate 13 shows the areal distribution of the simulated proportionate contribution of water from the Berkeley wells (33, 34, and 35) for March 1998 conditions. Under these conditions, the wells supply 90% to100% of the demanded water to the southwestern portion of the study area, 25% to 50% of the water to the southeastern portion, and from 1% to 10% to the central and northeastern portions of the study area. As a comparison, the simulated proportionate contribution of water from the Berkeley wells under peak-demand conditions for August 1998 (Plate 17) indicates a supply of water ranging from 50% to 100% for the southwestern portion of the study area and a 1% to 10% contribution of water to the south-central area. The simulations for the Berkeley wells indicate that they supply 1% to 10% of the water for the borough of South Toms River in August but do not supply any water in March (compare Plates 13 and 17), and they do not supply any water to the central, southeastern, north-central, and northern portions of the study area under the August 1998 conditions (Plate 17).

The reason the Berkeley wells do not supply water to the borough of South Toms River under March 1998 conditions is evident by comparing Plates 13 and 14. Plate 14 presents the areal distribution of the simulated proportionate contribution of water from the South Toms River well 32 under March 1998 conditions. This simulation indicates that well 32 supplies from 90% to 100% of the water demanded in the borough of South Toms River during March; however, for the August 1998 conditions, well 32 was not operating. Therefore, part of the water demand for the borough of South Toms River was supplied by the Berkeley wells (Plate 17).

As discussed above, under the simulated August 1998 conditions, the Berkeley wells do not supply any water to the southeastern, eastern, and northeastern portions of the Dover Township area (Plate 17) when compared with simulated March 1998 conditions (Plate 13). The reason for this is clear by reviewing Plate 18 (Brookside well 43) and Plate 22 (Windsor ground-level storage tank). Under simulated August 1998 conditions, these two points of entry are operating, and therefore, they supply the water demand in the southeastern, eastern, and northeastern portions of the study area. Specifically, the Windsor ground-level storage tank (Plate 22) supplies 90% to 100% of the water demand in the southeastern portion of the area, from 1% to 75% of the water in the eastern portion of the area, and 1% to 25% of the water in the northeastern portion of the study area. Brookside well 43, under August 1998 simulated conditions (Plate 18), supplies from 1% to 50% of the water in the northeastern portion of the study area and from 1% to 100% of the demand for water in an elliptical north-to-south oriented area (Plate 18).

The areal distribution of the simulated proportionate contribution of water from the Parkway well field tank is shown on Plate 16 for winter-time conditions (March 1998) and on Plate 20 for peak-demand conditions. Comparison of these two maps shows that the amount of water contributed by the Parkway ground-level storage tank in August 1998 to the southeastern and eastern parts of the distribution system (Plate 20) is considerably reduced or eliminated when compared with the March 1998 conditions (Plate 16). This is a result of well 40 and the Windsor ground-level tank being in operation for the peak-demand conditions existing in August 1998 (Plate 22). As a consequence of this operation, the Windsor tank meets the demand for water in the southeastern and eastern parts of Dover Township serviced by the distribution system. The trace analysis also shows the contribution of water from the Parkway well field tank to the northern area of Dover Township increases from 50% to 75% in March (Plate 16) to 75% to 100% in August (Plate 20). In addition, in the northern area of Dover Township between the Holiday City ground-level tank and the North Dover elevated tank, the Parkway ground-level tank does not supply any water in March 1998 (Plate 16), but supplies between 10% and 50% of the water demand in August 1998 (Plate 20).

The South Toms River well 32 supplies 90% to 100% of the water demand for March 1998 conditions for the borough of South Toms River (Plate 14). For August 1998 conditions, however, well 32 was not operating. Instead, water was obtained from the Holly Plant ground-level tank, supplied by wells 21, 30, and 37 (Plate 21). Results of the August 1998 analysis show that the Holly Plant tank supplied 10% to 100% of the water demand in a fan-shaped region in the south-central portion of the study area, and from 1% to 75% of the water in a narrow band along the southwestern portion of the area. In addition, Holly Plant tank supplied from 1% to 10% of the water demand for the remainder of the Dover Township area serviced by the distribution system with the exception of the southeastern and extreme northwestern portions of the study area.

The final point of entry for which a trace analysis was conducted is the Route 70 well 31 and results are shown for March 1998 (Plate 15) and August 1998 (Plate 19) conditions. Under March 1998 conditions, the well supplies from 90% to 100% of the water demand for the northwestern portion of the study area and from 10% to 50% of the water demand for the north-central and northeastern portions of the study area (Plate 15). This contrasts with August 1998 conditions (Plate 19) where the Route 70 well 31 supplies 75% to 100% of the water demand in the northwestern portion of the study area, 75% to 90% in the north-central portion, and 1% to 10% percent in the northeastern portion of the study area. The difference in supply from the March to August 1998 conditions (compare Plates 15 and 19) is made up by supply from the Parkway ground-level storage tank for the north-central portion of the study area (Plate 20) and the Windsor ground-level storage tank for the northeastern portion of the study area (Plate 22).

SELECTED LOCATIONS, DOVER TOWNSHIP AREA, NEW JERSEY

Five test hydrant locations are chosen as examples of the proportionate contribution of water to selected points of interest in the Dover township area and these results are now presented. The test hydrants represent the southwest (test hydrant H-1), south-central (test hydrant H-11), central (test hydrant H-17), northwest (test hydrant H-20), and northeast (test hydrant AH-22) portions of the study area. (See Plate 6 for hydrant locations; see Figure 14 and Table 15 for the ensuing discussion.) At the location represented by test hydrant AH-22 (northeast area of Dover Township), under winter-demand conditions (March 1998), the Berkeley wells (33, 34, and 35) supply 7% of the water, the Route 70 well 31 supplies 26% of the water, and the Parkway well field ground-level storage tank supplies 66% of the water. Under peak-demand conditions (August 1998), the Berkeley wells (33, 34, and 35) supply less than 1% of the water, Route 70 well 31 and the Holly plant ground-level storage tank supply about 1% of the water, Brookside well 43 supplies about 8% of the water, the Parkway well field ground-level storage tank supplies 83% of the water, and the Windsor ground-level storage tank supplies 6% of the water to the area of test hydrant AH-22. Thus, in terms of an exposure assessment, persons residing in an area represented by test hydrant AH-22 would receive, over an average day, more than 80% of their potable water from the Parkway well field tank during summer time (peak-demand) conditions, and about 66% of their potable water from the Parkway well field tank during winter-demand conditions. Similar observations can be made from results presented in Figure 14 and Table 15 for persons residing in areas represented by the other test hydrant locations.

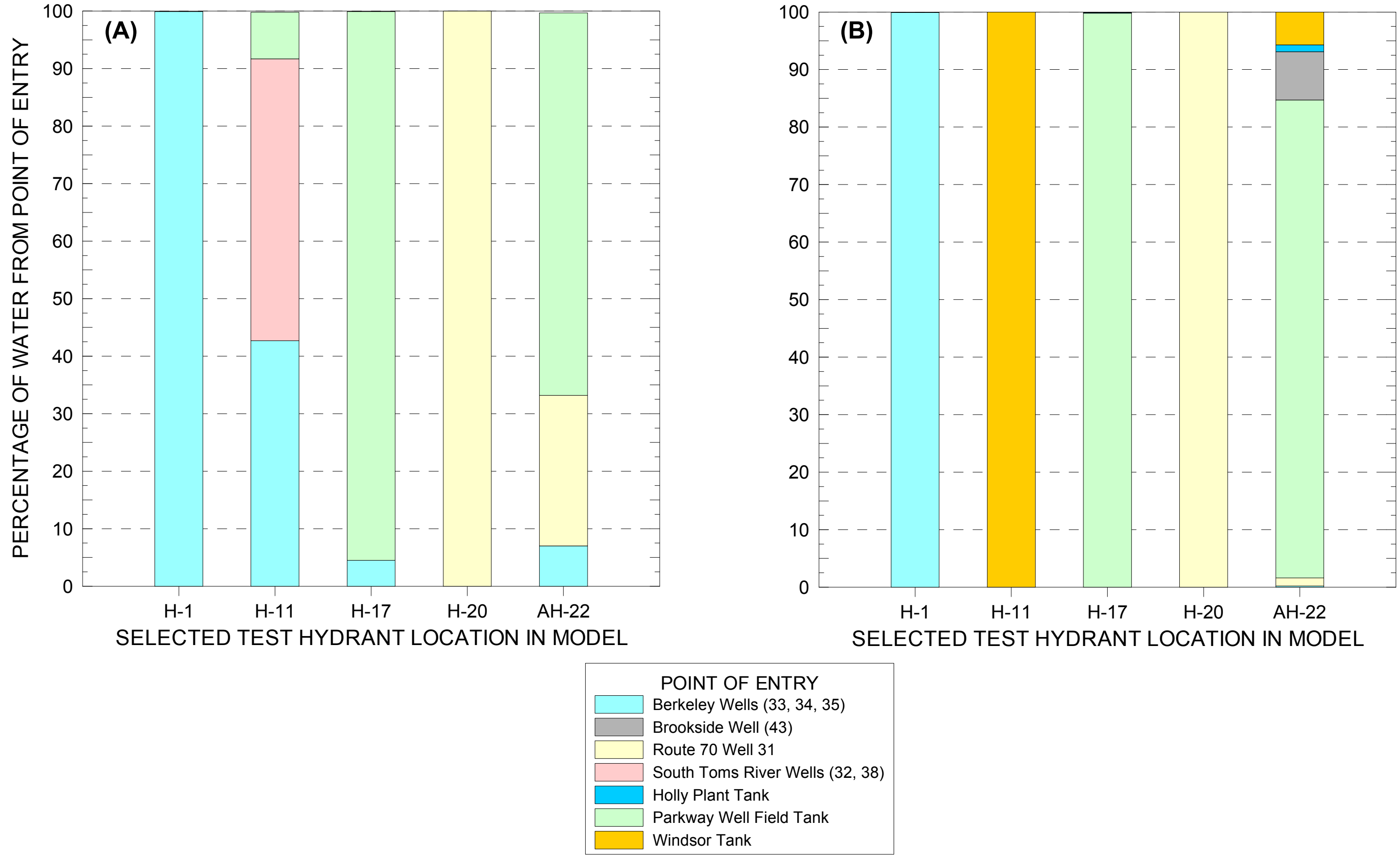


Figure 14. *Comparison of simulated percentage of water from points of entry to selected test hydrant locations, Dover Township area, New Jersey: (A) March 1998 conditions, and (B) August 1998 conditions. (Refer to Plate 6 for a map of locations.)*

Generalized Approach for Historical Reconstruction

To help NJDHSS with the case-control investigation, water-distribution system networks representing the location of pipelines from 1962 through 1996, based on historical information, will be derived. Using historical information on: (1) system demand conditions, (2) locations of wells, storage tanks, and booster pumps, and (3) system operations, trace analyses will be conducted for each month from January 1962 through December 1996. This will enable ATSDR investigators to provide information to epidemiologists and health scientists that can be used to relate study subject addresses to areas historically served by the water-distribution system using spatial analysis and address-matching techniques. ATSDR investigators will be blinded to case and control status of study participants. In the next phase of this investigation, historical model trace-simulation results, based on residence histories, will be used to estimate exposure to specific water sources by determining the percentage of water both cases and controls may have received from each of the points of entry (i.e., well fields) to the water-distribution system.

Summary

ATSDR and NJDHSS have initiated an exposure assessment for use in an epidemiologic study of childhood leukemia and brain and CNS cancers that occurred in the period 1979 through 1996 in Dover Township, New Jersey. Because groundwater contamination has been documented historically in public- and private-supply wells in the Dover Township area, human exposure through this pathway is possible. The Dover Township area has been primarily served by public water supply that relies solely on groundwater; therefore, ATSDR has developed a protocol for using a water-distribution model, (e.g., EPANET) as a tool to assist the exposure assessment component of the epidemiologic investigation. This report has presented the following aspects of the overall exposure assessment effort: (1) data gathered during field tests conducted in March and August 1998, (2) the development, calibration, and testing of the water-distribution system model for 1998 conditions, (3) a water-quality simulation of a naturally occurring conservative element, barium, to further test the reliability of the model calibration, and (4) the simulation of the proportionate contribution of water from points of entry (i.e., well fields) to various locations throughout the distribution system for 1998 conditions.

The present-day (1998) water-distribution system has 23 municipal wells distributed at eight points of entry (wells or well fields). In 1997, it serviced a population of 92,160. Preliminary simulations using an equivalent network representation of the water-distribution system (pipe diameters ranging from 6 in. to 16 in.) indicated higher-than-expected pressures–exceeding 125 psi in the southernmost areas of Dover Township, in the borough of South Toms River, and in Berkeley Township. Measured data were not available to either confirm or negate these initial simulation results. Therefore, system operation and pressure data were gathered during 48-hour tests in March and August 1998. Data were gathered simultaneously at 25 hydrants using continuous- pressure-recording data loggers with one-minute sampling rates. Data for storage tank water-levels, system demand, and pump and well cycling status were also obtained. Results of the tests indicated that system-wide pressures ranged from a low of about 40 psi to a maximum of slightly more than 100 psi.

For this investigation, the water-distribution system is being modeled as a network consisting of 16,071 pipe segments (or links) ranging in diameter from 2 in. to 16 in., six ground-level and three elevated storage tanks, and 17 high service or booster pumps. The model network also consists of 14,987 junctions or nodes. The model has been calibrated against winter-demand conditions for March 1998 and further tested against data gathered in August 1998 under peak-demand conditions. The reliability of the calibrated mode was judged in terms of the frequency of the absolute pressure difference (absolute value of difference between measured and simulated hourly pressure data). For the March 1998 simulation, 90% or more of the simulated hourly values for all hydrant locations resulted in an absolute pressure difference of approximately 5 psi or less. For the August 1998 simulation, 90% or more of the simulated hourly for all hydrant locations resulted in an absolute pressure difference of approximately 7.5 psi or less. These results are within the calibration limits established by ATSDR investigators at the outset of modeling activities and are within the general calibration guidelines suggested by Cesario and Davis (1983) and Walski (1984). Comparison of: (1) measured and simulated pressures for the 25 hydrants (Appendices I and J), (2) measured and simulated hydraulic head at ground-level and elevated storage tanks (Appendices K and L), (3) measured and simulated booster pump flows (Appendices M and N), and (4) measured and simulated groundwater well flows (Appendices O and P) support the assertion that the model presented herein is calibrated and an acceptable and reliable representation of water-distribution system conditions existing during 1998.

As further evidence of the reliability of the calibrated model, a simulation of the transport of a naturally occurring conservative element, barium, was conducted and compared with data gathered at 21 schools and 6 points of entry to the water-distribution system for March and April 1996. Measured concentrations of barium ranged from 13 μg/L to 51 μg/L. Comparison of measured and simulated barium concentrations at the 21 school locations indicates a difference ranging from 0.2 μg/L to 12.4 μg/L, which results in a mean relative difference of 13.6% with a range of 0.6% to 25.6%. Additional analyses comparing measured and simulated concentrations of barium show a geometric bias of 0.93, indicating a slight under prediction by the model (1.00 indicates perfect agreement), and a correlation coefficient of 0.81, indicating a high correlation between measured concentrations and simulated values. Therefore, this water-quality simulation is further evidence that the model is reasonably calibrated and an acceptable representation of the present-day water-distribution system characteristics.

To demonstrate the concept of "proportionate contribution" of water, the trace analysis option available within EPANET was used in conjunction with characterization of the present-day (1998) water-distribution system. For each point of entry to the water-distribution system (well or well fields) operating during March and August 1998, a trace analysis was conducted. These analyses provide an estimate of the percentage of water that any location of interest receives from the 8 points of entry to the distribution system. The results are presented in a series of 10 maps, a graph, and a table showing the percentage of water contributed by specific wells and storage tanks to locations in the Dover Township area for 1998 conditions. Based on residence histories, the trace-simulation results will be used in an epidemiologic investigation to estimate exposure of participants to specific water sources by determining the percentage of water they may have received from each of the points of entry to the distribution system serving the Dover Township area.

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DISCLAIMER

AVAILABILITY OF MODEL INPUT DATA FILES

Readers wishing to obtain model input data files for the March or August 1998 simulations described herein, should contact the senior author of the report at the following address:

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