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CASE STUDIES IN WELLHEAD PROTECTION AREA DELINEATION AND MONITORING

by

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NOTICE

This report is the result of research supported by the U.S. Environmental Protection Agency's Environmental Monitoring Systems Laboratory in Las Vegas, Nevada, as part of its efforts to provide technical assistance to state, tribal, and local governments on the implementation of the Wellhead Protection Program. The specific methods and approaches contained in this document have undergone peer review but do not constitute official Agency endorsement or policy recommendations. The Office of Research and Development provides this information to help solve complex technical problems related to refined delineation and ground-water monitoring of wellhead protection areas in various hydrogeologic settings. Further assistance is available from the Environmental Monitoring Systems Laboratory in Las Vegas, the Office of Ground Water and Drinking Water in Washington, D.C., and the ground-water offices in the ten U.S. Environmental Protection Agency regions.

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EXECUTIVE SUMMARY

BACKGROUND

The increasing contaminant threat to public water supply wells has created a new political and technical awareness of ground-water protection programs. Recognizing the need for conjunctive management of contaminant sources and public water supplies to prevent, or minimize, ground-water quality degradation, Congress amended the Safe Drinking Water Act in 1986 to include Section 1428. This section mandated the development of the Wellhead Protection Program (WHPP), which established a legal framework to protect public water supply wells, wellfields, and springs from contamination.

An important technical element of WHPP implementation is wellhead protection area delineation. A wellhead protection area (WHPA) is defined as the surface and subsurface area surrounding a well, wellfield, or spring, through which contaminants may pass and reach the ground water contributing to the supply source. Criteria and methods for WHPA delineation are given in several U.S. Environmental Protection Agency (EPA) guidance documents. Ground-water monitoring is one of many management tools available for protecting public drinking water supplies. Ground-water parameters are monitored (1) to assess source-control measures, (2) to monitor compliance with drinking water standards at sites other than the wellhead, and (3) to provide advance warning of contaminants in ground water. Ground-water monitoring may enhance source characterization, WHPA delineation, and new water supply evaluation.

This technical assistance document, "Case Studies in Wellhead Protection Area Delineation and Monitoring," was prepared by Lockheed Environmental Systems & Technologies Company under contract to the U.S. Environmental Protection Agency (EPA), Environmental Monitoring Systems Laboratory at Las Vegas (EMSL-LV). The document provides technical information to municipalities, states, and the EPA Regions in their implementation of WHPPs. The primary goals of this document are to present a monitoring methodology for WHPAs and to exemplify this methodology in five unique case study settings.

MONITORING METHODOLOGY WELLHEAD PROTECTION

In 1989, EMSL-LV engaged in cooperative research with five carefully selected municipalities to develop proposed, long-term monitoring programs for their existing WHPAs. The product of the cooperative research contains two types of information:

(1) A recommended methodology for planning and implementing a wellhead protection monitoring program which emphasizes source assessment, correct WHPA delineation, and hydrogeologic characterization (Chapter 1)

(2) Five case study narratives (Chapter 1, Appendix A-1, and Chapters 2 through 5) used to exemplify the monitoring methodology for different hydrogeologic and contaminant source settings (Table 1)

TABLE 1. CHARACTERISTICS OF THE CASE STUDY RESEARCH SITES

Municipality	Hydrogeologic Setting	Characterization Tasks
Stevens Point, Wisconsin	Unconfined aquifer	Flow system modeled with FLOWPATH (two-dimensional, steady state); Point and nonpoint sources assessed
Littleton, Massachusetts	Unconfined aquifer, recharge from Spectacle Pond	Flow system modeled with FLOWPATH (two-dimensional, steady state); MODFLOW (three-dimensional, steady state); and FLOWCAD (two-dimensional, transient); Industrial and commercial point sources assessed
Sioux Falls, South Dakota	Unconfined aquifer; recharge from the Big Sioux River	Flow system modeled with FLOWPATH (two-dimensional, steady state); Big Sioux River assessed as a line source; Point and nonpoint sources present
Dover, New Hampshire	Fractured-bedrock aquifer, discrete flow system	Flow system characterized with lineament analysis, structural-control mapping, aquifer testing, dye tracing, and borehole geophysics; A few commercial, point sources, and natural sources
Springfield, Missouri	Mature karst (porous limestone) aquifer, conduit flow system	Flow system determined by watershed boundaries, dye tracing, and flow analysis; Point and nonpoint sources assessed

Chapter 1 contains a methodology which is intended to serve as a guide for WHPP implementors in establishing technically-defensible, reliable, and effective ground-water monitoring programs for wellhead protection. This methodology emphasizes saturated zone monitoring. The first four case study narratives are presented in the document (Chapters 2 through 5) in order of increasing hydrogeologic complexity (aquifer heterogeneity). The exception to this organization is the Springfield, Missouri, case study, which is presented in abbreviated form in Appendix 1-A of Chapter 1.

Basic hydrogeology concepts and equations are reviewed in Chapter 1, including: ground-water systems and flow, conceptual hydrogeologic models and flow nets, and accurate delineation and monitoring in different hydrogeologic settings. The spectrum of unconfined to confined aquifer conditions is discussed in relation to porous, granular aquifers; fractured-bedrock aquifers; and karst aquifers.

Physical and chemical parameter monitoring apply to wellhead protection. Three types of ground-water monitoring are useful in managing WHPAs: ambient trend, source assessment, and early-warning detection monitoring. Ambient trend monitoring detects the temporal and spatial trends in physical and chemical quality of the ground-water system. Source assessment monitoring evaluates the existing or potential impacts on the physical or chemical ground-water system from a proposed, active, or abandoned contaminant source. Early-warning detection monitoring is conducted upgradient from the wellhead, based on known travel times, to trigger a contingency response to prevent public exposure to contaminants. These types of monitoring are incorporated to measure or detect contaminants in aquifers, and should not be mistaken as preventative or remedial measures.

Source assessment is a critical first step in designing an effective monitoring program. Target monitoring parameters for early-warning detection and source assessment are selected from a comprehensive list of known and suspected contaminants associated with land-use activities and practices. Optimal monitoring sites may be determined, reflecting prioritization of sources. An inventory of common sources of contamination within and in proximity to WHPAs is included.

The monitoring methodology is divided into three phases:

- Phase I: WHPP Elements and Scoping Tasks
- Phase II: Research Monitoring Program
- Phase III: Wellhead Protection Monitoring Program

Phase I WHPP elements and scoping tasks include: designating roles and a management framework, preliminary WHPA delineation, and source assessment. To support the research monitoring task, an initial information base of ancillary and monitoring data should be compiled and reviewed to determine data limitations and gaps. The strategy is to maximize information content; to define monitoring objectives; and to conduct field studies with the least, but still adequate number of monitoring points. Existing monitoring sites identified in this phase can be incorporated in the long-term monitoring network. Phase I generally requires a 3- to 6-month period for completion.

Phase II is aptly named the Research Monitoring Program, or the phase of acquiring information pertaining to how the subsurface system operates and of formulating interpretations. Research monitoring is conducted to improve, or verify, elements of the hydrogeologic conceptual model. A technically-defensible conceptual flow model ensures a more protective and reliable monitoring program. Research monitoring for wellhead protection includes baseline water quality characterization, aquifer testing and characterization, refined or verified WHPA delineation, and ground-water flowpath determination to relate sources to the water supply well or spring. The product of research monitoring is a proposed long-term monitoring program that may be partly implemented in Phase II. Phase II may require 1 to 1-1/2 years for completion, depending on the complexity of the site hydrogeology and the quality of the initial information base.

The by-product of Phases I and II is a proposed wellhead protection monitoring program, Phase III. Generally, the program is submitted as a plan to be implemented in stages, as labor and financial resources become available. The plan should include an organization chart, a source assessment map and list, and a map depicting the WHPA and protective zones, as well as a

description of the delineation criteria and method(s). General and specific objectives for ambient trend, source assessment, and early-warning detection monitoring should be detailed. Each objective should justify the selection of monitoring sites, parameters, and frequencies.

The locations of existing and recommended monitoring sites in the proposed network should be shown on a map. A formal identification system with a minimum set of data elements should be used to label each site. The integrity of the design and construction of each existing site should be considered prior to inclusion in the monitoring network to ensure data quality. New sites that require installation should be described in detail, concerning completion depth, open or screened interval, schematic design, and construction materials, as well as the methods of installation, development, and testing. Physical and chemical parameters to be monitored at select frequencies should be listed and technically justified. Monitoring site information should be stored in an automated data base for convenient and safe storage, update, and retrieval. Each monitoring program should formulate a minimum set of quality assurance and control objectives to match the objectives of the wellhead protection monitoring program.

A 15-step approach for the design of a wellhead protection monitoring program is depicted as a flowchart in Figure 1. The monitoring program should be reviewed and improved in an iterative process over the life span of the WHPP. The organization of the case studies research in Chapters 2 through 5 follows the logical outline of Phases I, II, and III.

STEVENS POINT, WISCONSIN, CASE STUDY (Chapter 2)

The city of Stevens Point is located in central Wisconsin and has a population of approximately 23,000. The source of the city water supply is from the Airport and Iverson wellfields. These wellfields pump an average of 5 million gallons of water per day from a shallow, unconfined aquifer. The aquifer is composed of coarse, unconsolidated sediments deposited by meltwater during the Wisconsin glaciation. The preliminary wellhead protection zones for the combined wellfields were based on estimates of the zone of influence (ZOI), the 5-year time-of-travel (TOT) zone (analytically determined), and the recharge area. In the review process, the validity of the 5-year TOT calculation was questioned, and the WHPA was never promulgated.

An extensive, historical source assessment was conducted within the B Zone of the preliminary WHPA using aerial photographic interpretation techniques combined with conventional methods such as surveys of directories, local and state records, visual inspections, monitoring data, and so on. Point and nonpoint sources were identified, ranked, and prioritized for management and regulation. Existing contaminant sources were given highest priority. Potential sources were then prioritized based on source type, quantity, hazard, and location.

A network of 55 monitoring sites (single monitoring wells, well nests, and a multi-level well) were used to measure water levels, to sample ground water, and to conduct aquifer tests in the unconfined aquifer. Of the total network, three single wells and four well nests represent new monitoring points installed for this research. Aquifer parameter results from slug, constant-discharge, and recovery tests indicate a range of hydraulic conductivity values for three distinct geologic settings: 820 to 1,700 feet per day (ft/d) for the buried valley, 220 to 240 ft/d for outwash plains, and 2 to 3 ft/d for bedrock highs.

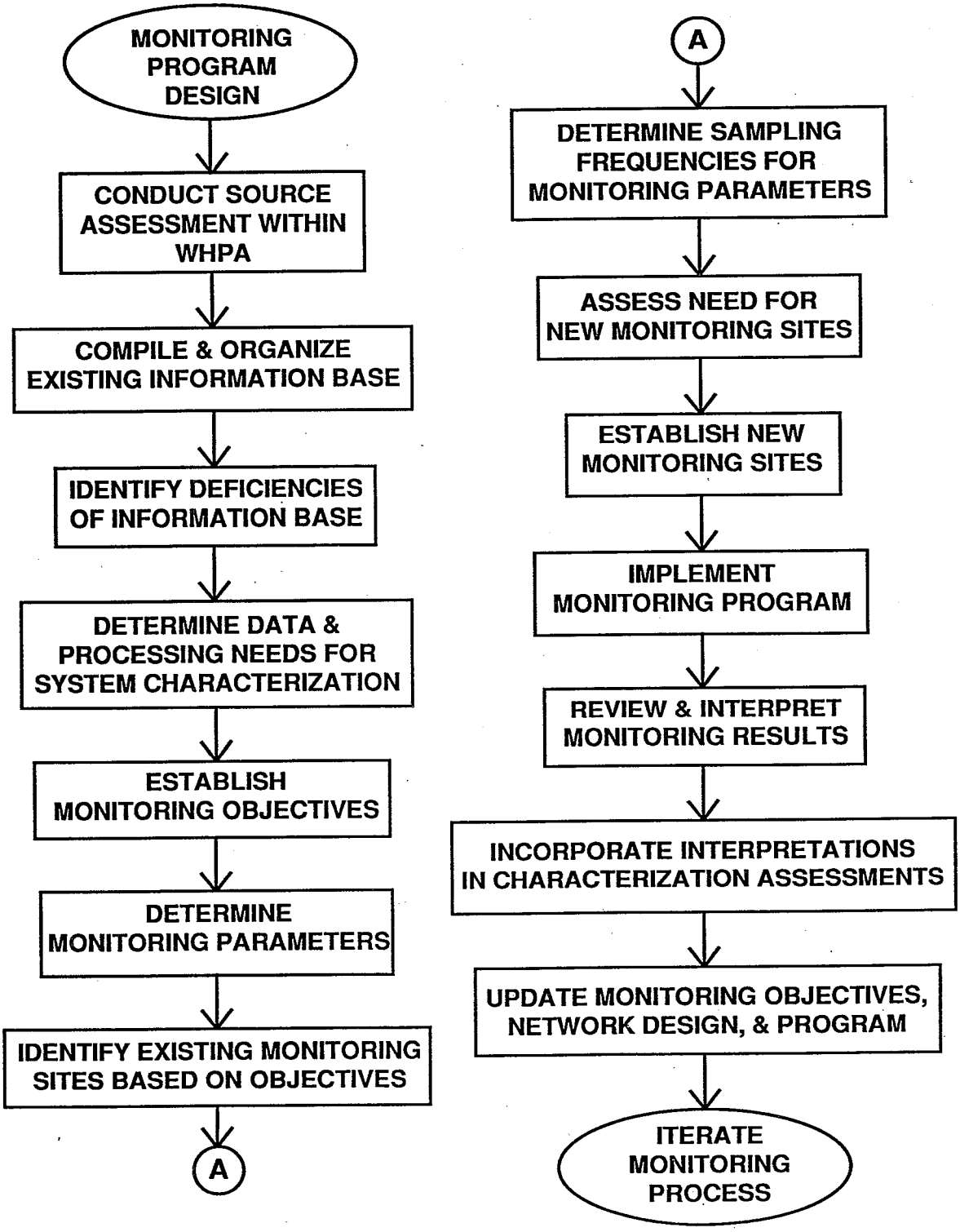

Figure 1. Flow chart of the 15-step monitoring methodology for wellhead protection areas.

Hydrochemical data indicate that nitrogen concentrations, a key indicator of contamination, have increased over time. Currently, nitrogen concentrations in the monitoring network range from less than 0.2 to 26.0 milligrams per liter. Other indicators of ground-water degradation include iron and manganese from organic-rich soils located along the Plover River, chloride in proximity to roads where de-icing occurs, and previous volatile organic compound contamination at the airport and several underground storage tanks.

A two-dimensional, ground-water flow model (FLOWPATH) was used to delineate the 5- and 10-year TOT zones for the Airport and Iverson wellfields. In comparison, the previous, analytically derived B Zone is larger; however, the 5-year TOT zone from FLOWPATH extends farther to the east due to the effects of pumping at the Iverson wellfield and the presence of bedrock highs.

A long-term ground-water monitoring network is proposed for the Airport and Iverson wellfields consisting of 34 existing and proposed wells. Nine new well locations are proposed to fill data gaps in the existing network, primarily along the boundaries of the 5- and 10-year TOT zones. Wells in the long-term monitoring network should be sampled twice a year in April and September for indicator parameters. Water levels should be recorded each time a well is sampled. Compliance monitoring networks are recommended for point sources of highest priority.

The wellhead protection contingency plan consists of three components: (1) reaction to the early-warning detection system based on preventive action and state drinking water limits, (2) spill response, and (3) new water-supply development and implementation.

LITTLETON, MASSACHUSETTS, CASE STUDY (Chapter 3)

The town of Littleton is located approximately 35 miles northwest of Boston in northeastern Massachusetts. The daily water demand for the town's population of 7,300 is from 800,000 to 1,500,000 gallons per day from four production wells. Techniques for refined delineation and long-term monitoring of the WHPA surrounding Production Well Number 5 (PW-5) are discussed in this report. Production Well Number 5 is completed at a depth of 167 feet within saturated, stratified valley-fill deposits. The aquifer is unconfined and receives significant surface-water recharge (20 to 25%) from nearby Spectacle Pond and Bennetts Brook.

Land-use activities within the WHPA cover a range of commercial, industrial, and to a lesser degree, agricultural operations. Collectively, these land-use activities pose potential contamination threats to the aquifer, including heavy metals, volatile organic compounds, pesticides, and nutrients. Baseline monitoring results indicate that ground-water quality within the capture zone of PW-5 is currently unaffected by source operations. Sodium is the only exception. Slightly elevated levels of sodium in surface water and the shallow aquifer are attributed to roadway de-icing. Manganese and iron concentrations are elevated throughout the recharge area of PW-5, primarily because of their occurrence in wetland sediments and glacial deposits. The levels of these parameters have increased at PW-5 for several years and may warrant treatment in the future.

The PW-5 WHPA consists of three protection zones delineated using a combination of numerical ground-water flow models (FLOWPATH, FLOWCAD, and MODFLOW) and hydrogeologic mapping. Zone I is the 400-foot sanitary protective radius mandated by the State of

Massachusetts. Zone II is the most critical management area and was delineated conservatively as the union of three numerical capture zone solutions. These numerical solutions incorporate two- and three-dimensional flow, as well as steady-state and transient flow conditions. Local and regional ground-water flow simulations are based on the results of short- and long-term aquifer testing.

Zone II generally represents the steady-state capture zone for PW-5 that corresponds approximately to the 400-day travel-time contour. Flowpath simulations indicate that Zone II extends to the bottom of the aquifer and is constrained by bedrock and glacial till. Within Zone II, three existing wells and two new wells are proposed for inclusion in the monitoring network for early-warning detection and source assessment purposes. These wells lie along either the 150-day or the 300-day travel-time contours. Screened intervals for the new monitoring wells were chosen based on results from MODPATH computer flow simulations. Monitoring parameter groups for these wells include general water quality, site-specific, and physical parameters. Recommended monitoring frequencies for these parameter groups vary from quarterly to annually, depending on the travel-time distance from the monitoring well to PW-5 and the monitoring well depth.

Zone III is defined as the upgradient area of the aquifer that contributes to Zone II and extends to the watershed boundary. Zone III is monitored at two surface-water locations, one at the inflow and one at the outflow of Spectacle Pond. In addition, Zone III is monitored biannually at existing compliance networks around waste management and industrial sites. Monitoring parameters for the compliance wells include general water quality, site-specific, and physical parameter groups.

The Littleton WHPP incorporates contingency planning. Catastrophic releases initiate a spill-response plan that involves many departments and agencies. In the event of contamination of PW-5 or another production well, Littleton has sited a new production well. The proposed well site is approved by the State, and protection Zones I, II, and III are delineated. The adjacent town of Boxborough shares the recharge area to the proposed well. Boxborough has adopted complementary strategies with Littleton to ensure its water quality protection.

SIOUX FALLS, SOUTH DAKOTA, CASE STUDY (Chapter 4)

The city of Sioux Falls is located in the southeast corner of South Dakota. The Big Sioux aquifer is the primary source of water for about 125,000 persons in the Sioux Falls metropolitan area. One of the municipal wells in the Big Sioux aquifer, the airport wellfield, is underlain by surficial, glacial outwash deposits. The Big Sioux River is located directly west of the airport wellfield and flows south over and through the outwash, draining approximately 4,000 square miles of upstream land.

The city's wells pump most of their water directly from the aquifer and a small quantity from the Big Sioux River. However, the river is hydraulically connected to the aquifer, and recharge from the river in the airport wellfield area is significant. In 1988, approximately 79% of the recharge to the airport wellfield aquifer was induced from the river due to wells pumping. Induced flow from the river to the aquifer is demonstrated by decreased flow in the river during low recharge periods.

This research was conducted to evaluate (1) the hydraulic connection between the Big Sioux River and the adjacent aquifer, and (2) the potential impact of the river on aquifer water quality. In the broader perspective, additional goals included refined delineation of the wellfield protection area and design of a long-term water quality monitoring program.

Drilling logs indicate that the thickness of the aquifer in the wellfield area ranges from 20 to 50 feet. Aquifer testing results yield an average hydraulic conductivity value of 800 ft/d and a transmissivity value of approximately 21,000 square feet per day for the aquifer.

Many potential point sources of contamination exist in the study area. These include: industrial and commercial areas, the South Dakota Air National Guard facility, a petroleum pipeline, the Sioux Falls Regional Airport, and a decommissioned municipal landfill. The threat of contamination from these sources is underscored by the recent history of contaminant releases in the area.

To estimate ground-water travel times in the study area, aquifer testing, dye tracing, and ground-water modeling were employed. During aquifer testing, two dye injections were made. The first dye was injected in a well approximately 40 feet north of the pumping well. Detectable dye concentrations first arrived at the pumped well after about 12 hours. The second dye was injected in a well near the edge of the river, approximately 140 feet north of the pumping well. Detectable dye concentrations from the second injection site first arrived at the pumped well in 7 to 9 days. Aquifer testing and dye-tracing results indicate that a contaminant could travel from the river to the wellfield in less than 9 days.

A two-dimensional, steady-state model (FLOWPATH) was used to generate time-related capture zones for the municipal wells and to simulate contaminant travel times. One-, two-, and five-year capture zones were calculated for each of the municipal wells in the airport wellfield. Modeling of simulated spill sites from several of the potential point-source contamination areas indicates that contaminants entering the aquifer at areas to the north and south of the well field could reach the municipal wells in 1 to 2 years.

The City of Sioux Falls and Minnehaha County have delineated wellhead protection areas by using the hydrogeologic-mapping method. Wellhead protection ordinances are designed to impose guidelines and restrictions on new land uses, or proposed changes in existing use, in order to protect the aquifer water quality.

A wellhead protection monitoring program at the airport wellfield is proposed to document ambient water quality conditions and to serve as an early-warning detection system. Line-source monitoring is proposed to monitor the Big Sioux aquifer and the diversion canal for contaminants that could potentially enter the aquifer. Point-source and nonpoint-source monitoring are proposed to monitor water quality between the airport wellfield and potential sources. The categories of parameters for monitoring are general water quality, VOCs, trace metals, pesticides, and nutrients. Sampling frequencies for each of the categories were selected as a function of the type of source to be monitored.

Contingency planning is warranted to establish emergency responses to contaminant releases at the surface of the aquifer and in the river. Alternative water supply development must also be continued as part of the contingency planning effort.

DOVER, NEW HAMPSHIRE, CASE STUDY (Chapter 5)

Dover is a city of 26,000 people located in the seacoast region of New Hampshire. To meet the increasing water supply demands of the future, the city embarked on a water exploration effort in a fractured-bedrock aquifer at the Blackwater Brook site. A test well was installed to a depth of 400 feet in the bedrock aquifer as part of the ground-water exploration program. A wellhead protection area and ground-water monitoring strategy were established for the test well. This study describes how the conceptual hydrogeologic model for the site was developed and refined.

The bedrock aquifer consists primarily of quartz monzonite and metasedimentary rocks that interfinger along a fractured, faulted contact zone trending north 60 degrees east (N60°E). A N5-10°W trending lineament and fracture zone intersects the N60°E zone at the site. The bedrock aquifer is directly overlain by Pleistocene-age sands and gravels. These sediments are overlain by low-permeability marine clay and lodgement till. It is estimated that 20% of the water produced from the bedrock aquifer is derived from overburden sediments in the watershed area.

Four overburden and bedrock well pairs constitute the present monitoring network for the test well. Two well pairs lie along the N60°E faulted contact zone, and two well pairs lie along the perpendicular N30°W trend. The test well and four of five bedrock wells airlift in excess of 150 gallons per minute. Few contaminant threats exist near the site. Baseline sampling indicates that minor, elevated levels of iron, manganese, and radon pose the only water quality problems at present.

Test drilling and borehole surveys (caliper, video camera, acoustic televiewer, thermal-pulse flowmeter, and hydrophysical logging) indicate that fracturing and ground-water flow are highly discrete. Flow occurs at isolated, definable depths rather than uniformly along the length of the borehole. Hydrophysical logging indicates that the borehole water is distinctly layered with respect to the fluid electrical conductivity parameter. Most borehole water is produced by moderately to steeply dipping fractures and fracture zones that intersect the wells.

Aquifer testing and dye-trace results indicate that the N30°W and N60°E directions have higher aquifer transmissivities relative to the surrounding bedrock matrix. Drawdown contours are elongate about the N30°W well alignment, suggesting preferred flow in this direction. Dye-trace results indicate more rapid travel of injected dye along the N30°W direction than the N60°E direction. Dye traveled 152 feet in 130 minutes (the time of first arrival of the dye) from injection in a bedrock monitoring well along the N30°W trend to the test well, which was pumped at 200 gallons per minute. This represents a velocity of 1,680 ft/d. Dye injected in a bedrock monitoring well located 596 feet from the test well arrived there in 148 hours, indicating a velocity of 96 ft/d along the N60°E direction.

Flowmeter and acoustic televiewer surveys indicate that a moderately west-dipping fracture zone provides interconnection between the test well and bedrock well R2 along the N30°W trend. Lacking discrete flow information beyond the test well and well R2, statistical fracture descriptions become good approximations of flowpaths at increasing distances from the site. Therefore, prominent

fracture peaks along the N5-10°W and N60°E trends represent the most probable flow directions within the bedrock fracture system at Blackwater Brook. The N60°E trend is substantiated by the existence of the faulted, fractured contact zone along this strike. Evidence to suggest preferred flow along the N5-10°W direction is structural and hydrogeologic. Structural control is inferred by strong expression of the lineament on several platforms of photography and in outcrop fracture trends. Enhanced transmissivity along the N30°W direction is attributed to the proximity and similar orientation of the N5-10°W fracture zone.

A quadratic equation is derived from accepted hydrogeologic relationships (Darcy's Law and the Thiem equation). In this equation, ground-water travel time (determined using the time of first arrival of dye at the test well) is directly proportional to the square root of distance from the test well. Constants of proportionality for the quadratic relationship are calculated for the N30°W and the N60°E directions based on dye-trace velocities. Distances for the 200-day and 1,000-day TOT thresholds are then calculated for the two fracture zone directions: N5-10°W and N60°E.

Three wellhead protection zones are delineated within the recharge area for the test well using a variety of criteria and methods. Zone I is the state-mandated 400-foot sanitary radius. Zone IIA consists of two 1,000-foot-wide "arms" along the N5-10°W and N60°E directions, extending to the 200-day TOT distances. Zone IIB is the area within a smooth curve connecting the outer boundaries of Zone IIA, producing an oval shape. Zone III is the upgradient area contributing to the 1,000-day TOT distance modified by hydrogeologic features. Recommended regulation of the wellhead protection zones varies from complete control and restriction of activities in Zone I to public education in Zone III.

A major component of wellhead protection program management is long-term ground-water monitoring. Under present conditions, monitoring of the test well and existing monitoring wells will focus on a moderate effort to assess ambient water quality and physical parameters. After the production well is developed, the monitoring frequency and list of monitoring parameters increases. Proposed frequencies, parameters, and new sites for monitoring derive from technical and management goals. Action levels are proposed to trigger contingency responses.

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CHAPTER 1

MONITORING METHODOLOGY FOR WELLHEAD PROTECTION

by

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ABSTRACT

A methodology for planning and implementing a wellhead protection monitoring program is formulated and demonstrated at unique case study sites. This methodology emphasizes saturated zone monitoring and is intended to serve as a guide for WHPP implementors. Careful implementation of this methodology will enable managers and scientists to establish technically defensible, reliable, and effective ground-water monitoring programs for wellhead protection.

Basic hydrogeology concepts and equations are discussed as they pertain to ground-water systems and flow, conceptual hydrogeologic models and flow nets, and accurate delineation and monitoring in different hydrogeologic settings. The spectrum of unconfined to confined aquifers is discussed in relation to porous, granular aquifers; fractured-bedrock aquifers; and karst aquifers.

Physical and chemical parameter monitoring apply to wellhead protection. Three types of ground-water monitoring are useful in managing wellhead protection areas -- ambient trend, source assessment, and early-warning detection monitoring. Ambient trend monitoring detects the temporal and spatial trends in physical and chemical quality of the ground-water system. Source assessment monitoring evaluates the existing or potential impacts on the physical or chemical ground-water system from a proposed, active, or abandoned contaminant source. Early-warning detection monitoring is conducted upgradient from the wellhead, based on known travel times, to trigger a contingency response to prevent public exposure to contaminants in aquifers; they should not be mistaken as preventative or remedial measures.

A long-term monitoring program for wellhead protection is developed in three phases. Phase I, Wellhead Protection Program Elements and Scoping Tasks, includes: determining technical roles and a management framework, preliminary wellhead protection area delineation, and source assessment. This phase may require 3 to 6 months for completion. Phase II is referred to as the Research Monitoring Program, or the phase of acquiring information pertaining to how the subsurface system operates and of formulating interpretations. Research monitoring includes baseline water quality characterization, aquifer testing and characterization, refined or verified wellhead protection area delineation, and ground-water flowpath determination to relate to the water supply, well, or spring. Phase II may require 1 to 1-1/2 years for completion.

The by-product of completing the first two phases is a proposed Wellhead Protection Monitoring Program, Phase III. Generally, the Program is submitted as a plan to be completed in stages, as labor and financial resources become available. The program should detail the roles and duties of the implementation team, a source assessment map and list, and a map depicting the wellhead protection area and protection zones. Each objective should justify the selection of monitoring sites, parameters, and frequencies.

The locations of existing and recommended monitoring sites in the proposed network should be shown on a map. A formal identification system with a minimum set of data elements should be used to label each site. The integrity of the design and construction of each existing site should be considered prior to inclusion in the monitoring network to ensure data representation. New sites that require installation should be described in detail, concerning: completion depth, open or screened interval, schematic design, and construction materials, as well as the method of installation, development, and testing. Physical and chemical parameters to be monitored at select frequencies should be listed and technically justified. Monitoring site information should be stored in an automated data base for convenient and safe storage, update, and retrieval. Each monitoring program should formulate a minimum set of quality assurance and quality control objectives to match the objectives of the wellhead protection monitoring program.

A step-wise approach for the design of a wellhead protection monitoring program is given, incorporating Phases I, II, and III. The monitoring program should be reviewed and improved over the life span of the Wellhead Protection Program as an iterative process.

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ABBREVIATIONS AND SYMBOLS

ABBREVIATIONS

BOD	biochemical oxygen demand
COD	chemcial oxygen demand
DNAPL	dense, nonaqueous phase liquid
DO	dissolved oxygen
EMSL-LV	Environmental Monitoring Systems Laboratory, Las Vegas, Nevada
EPA	U.S. Environmental Protection Agency
LNAPL	light, nonaqueous phase liquid
mg/l	milligram per liter
PCB	polychlorinated biphenyl
ppm	part per million
SDWA	Safe Drinking Water Act
SHPA	springhead protection area
TDS	total dissolved solid
TOC	total organic carbon
TSS	total suspended solid
TU	tritium unit
VOC	volatile organic compound
WHPA	wellhead protection area
WHPP	Wellhead Protection Program
ZOC	zone of contribution
ZOI	zone of influence

SYMBOLS

A	cross-sectional area
dh/dl	head loss per unit distance
Eh	redox potential
I	hydraulic gradient
K	hydraulic conductivity
n	porosity
Q	discharge per time
q	specific discharge
v	average ground-water velocity

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WELLHEAD PROTECTION AREA MONITORING

INTRODUCTION

The increasing contaminant threat to public water supply wells has created a new political and technical awareness of ground-water protection programs. Recognizing the need for conjunctive management of contaminant sources and public water supplies to prevent, or minimize, ground-water quality degradation, Congress amended the Safe Drinking Water Act (SDWA) in 1986 to include Section 1428. This section mandated the development of the Wellhead Protection Program (WHPP), which established a legal framework to protect public water supply wells, wellfields, and springs from contamination.

Elements of the WHPP include, at a minimum (U.S. EPA, 1989b):

- Specifying the roles and duties of cooperating agencies
- Delineating the wellhead protection area (WHPA) for each well, wellfield, or spring
- Identifying sources of contamination within and in proximity to each WHPA
- Developing management approaches to protect the water supply within WHPAs from contamination
- Developing contingency plans in response to well, wellfield, or spring contamination
- Siting new wells properly to maximize yield and to minimize contamination
- Ensuring public participation

An important technical element of WHPP implementation is WHPA delineation. A wellhead protection area is defined as the surface and subsurface area surrounding a well, wellfield, or spring, through which contaminants may pass and reach the ground water contributing to the supply source (Figure 1-1). Criteria and methods for WHPA delineation are discussed in the following guidance documents: U.S. EPA, 1987a; Wisconsin Geological and Natural History Survey, 1991; and Bureau of Economic Geology, Universiy of Texas-Austin, 1991.

Ground-water monitoring is one of many management tools available for protecting public drinking water supplies (U.S. EPA, 1989b). Ground-water monitoring of physical and chemical parameters may enhance source characterization, WHPA delineation, and new water supply evaluation. Ground-water parameters are monitored for the following reasons:

- To assess source-control measures
- To monitor compliance with drinking water standards at sites other than the wellhead (municipal supply monitoring already required by the SDWA)
- To provide advance warning of contaminants in ground water

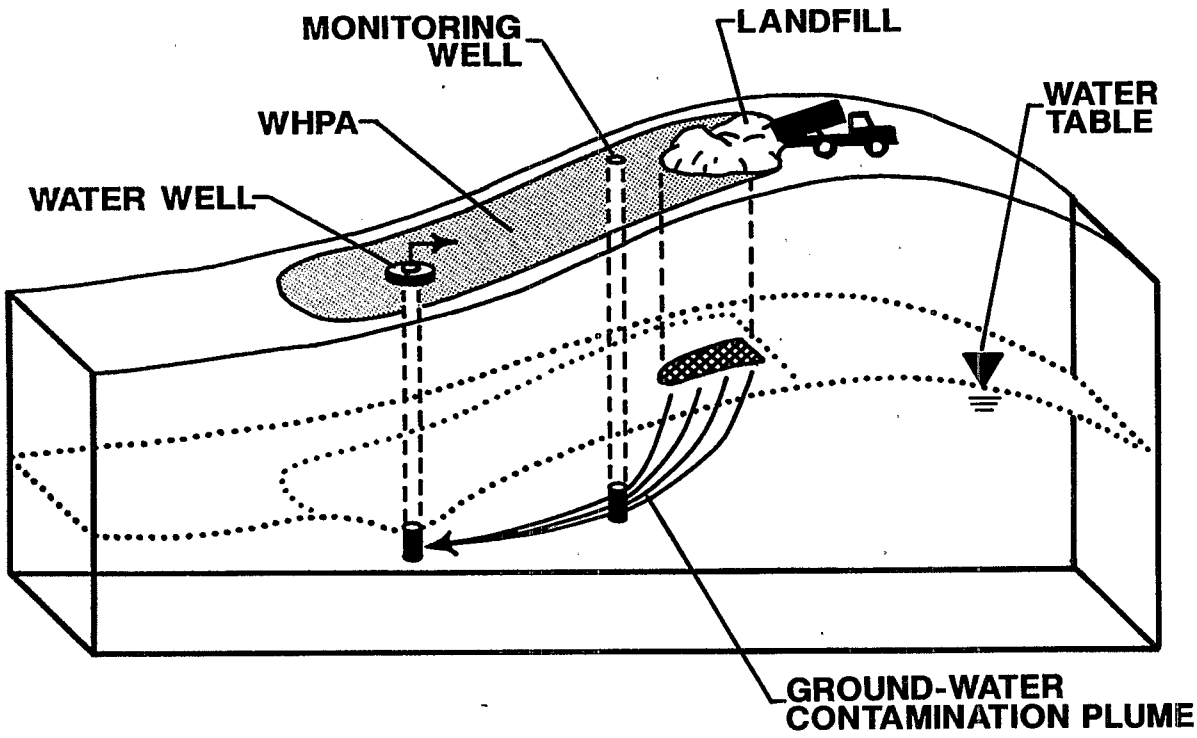

Figure 1-1. Conceptual wellhead protection area and monitoring scenario.

This technical assistance document, "Case Studies in Wellhead Protection Area Delineation and Monitoring," was developed to provide technical information to municipalities, states, and the U.S. Environmental Protection Agency (EPA) Regions in their implementation of WHPPs. The primary goals of this document are to present a monitoring methodology for WHPAs and to exemplify this methodology in different case study settings. Developing a long-term monitoring program for a WHPA involves much more than locating wells by intuition and "seat of the pants" sampling programs. Reliable and effective monitoring requires significant forethought and investigation. This document discusses monitoring objectives and recommends a sequence of steps associated with planning and implementing a long-term program, referred to as the "research monitoring phase." Correct delineation of the WHPA and a basic understanding of ground-water flowpaths to the well or wellfield are prerequisites for proper monitoring network design and sampling. Therefore, the methodology presented here emphasizes subsurface site characterization and verification of the WHPA prior to long-term monitoring program design.

PURPOSE AND SCOPE OF THE DOCUMENT

A brief description of the evolution of this document is appropriate to define its purpose and scope. The EPA's Office of Drinking Water and Ground Water (the client office) and the Environmental Monitoring Systems Laboratory in Las Vegas, Nevada (EMSL-LV), began cooperative research in 1988 to develop guidance in effective ground-water monitoring for wellhead protection.

This research was intended to produce a complementary document to "Guidelines for Delineation of Wellhead Protection Areas" (U.S. EPA, 1987a). A nation-wide survey of existing WHPPs in 1989 indicated a number of fledgling programs; most were struggling through the delineation process and legal enforcement of the WHPA. A literature survey uncovered numerous references regarding ground-water monitoring planning, methodologies, technologies, and interpretive approaches. As a result of these surveys, it was determined that general guidance on monitoring WHPAs would duplicate the existing literature and be of little use to the intended audience. By studying existing WHPPs, it became apparent that each program reflected unique municipal and state legal, economic, technical, and physical factors or constraints. In light of these findings, EMSL-LV engaged in cooperative research with five carefully selected municipalities in 1989 to conduct refined delineation of existing WHPAs and to develop long-term monitoring programs.

The product of the cooperative research is this document, which contains two types of information:

(1) A recommended methodology for planning and implementing a wellhead protection monitoring program, which emphasizes source assessment, correct WHPA delineation, and hydrogeologic characterization (Chapter 1)

(2) Five case study narratives exemplifying the application of the monitoring methodology for different hydrogeologic and contaminant source settings (Table 1-1; Figure 1-2):
- Stevens Point, Wisconsin (Chapter 2)
- Littleton, Massachusetts (Chapter 3)
- Sioux Falls, South Dakota (Chapter 4)
- Dover, New Hampshire (Chapter 5)
- Springfield, Missouri (Chapter 1, Appendix A-1)

The methodology presented in Chapter 1 serves as a guide for WHPP implementors in establishing technically-defensible, reliable, and effective ground-water monitoring programs for wellhead protection. This methodology emphasizes saturated zone monitoring, or strategies used to monitor the zone beneath the water table where interconnected openings are filled with water (Figure 1-3). References and an additional bibliography list are provided at the end of Chapter 1 regarding applicable technologies, as well as analytical and interpretive approaches used to establish ground-water monitoring programs.

Of the 66 municipal programs evaluated for participation in this research, five were chosen based on the strength of their existing WHPP, the hydrogeologic setting, the type of sources present, and the qualifications of the participating scientists to conduct research. When this research began in 1989, the WHPP was relatively well established in the eastern states, and less so in the western states. Consequently, few arid settings were evaluated as potential case study research sites, and none are included in this document. Distinguishing characteristics of the five case studies and hydrogeologic techniques used to characterize the flow systems are summarized in Table 1-1. The first four case study narratives are presented in the document (Chapters 1 through 4) in order of increasing hydrogeologic complexity. The exception to this organization is the Springfield, Missouri, case study,

TABLE 1-1. CHARACTERISTICS OF THE CASE STUDY RESEARCH SITES

Municipality	Hydrogeologic Setting	Characterization Tasks
Stevens Point, Wisconsin	Unconfined aquifer	Flow system modeled with FLOWPATH (two-dimensional, steady state); Point and nonpoint sources assessed
Littleton, Massachusetts	Unconfined aquifer, recharge from Spectacle Pond	Flow system modeled with FLOWPATH (two-dimensional, steady state); MODFLOW (three-dimensional, steady state); and FLOWCAD (two-dimensional, transient); Industrial and commercial point sources assessed
Sioux Falls, South Dakota	Unconfined aquifer; recharge from the Big Sioux River	Flow system modeled with FLOWPATH (two-dimensional, steady state); Big Sioux River assessed as a line source; Point and nonpoint sources present
Dover, New Hampshire	Fractured-bedrock aquifer, discrete flow system	Flow system characterized with lineament analysis, structural-control mapping, aquifer testing, dye tracing, and borehole geophysics; A few commercial, point sources, and natural sources
Springfield, Missouri	Mature karst (porous limestone) aquifer, conduit flow system	Flow system determined by watershed boundaries, dye tracing, and flow analysis; Point and nonpoint sources assessed

which is presented in abbreviated form in Appendix 1-A of Chapter 1. As a general rule, the hydrogeologic settings presented in this document become progressivley more heterogeneous, or incorporate additional flow factors, such as the nearby river in the case of Sioux Falls, South Dakota. This organizational approach will enable nontechnical readers to progressively increase their vocabulary and knowledge of hydrogeology and wellhead protection as they read each section.

The Stevens Point, Wisconsin, case study (Chapter 2) is typified by an unconfined, sand and gravel aquifer (modeled as a two-dimensional flow system) overlain by both urban point and nonpoint sources. The watersheds in Littleton, Massachusetts (Chapter 3), consist of stratified-drift, valley-fill deposits that are primarily under unconfined conditions and receive significant recharge from surface ponds (modeled as two- and three-dimensional flow systems). Point sources are the focus of assessment and regulation in Littleton. Sioux Falls, South Dakota (Chapter 4), derives its municipal drinking water from a surface-water and ground-water flow system consisting of induced flow from the Big Sioux River through the unconfined, sands and gravels of the Big Sioux aquifer. Three types of sources are present within the Sioux Falls municipal WHPA, listed here in order of importance: the Big Sioux River as a nearby line source to the wellfield, urban point sources, and rural nonpoint sources. Dover, New Hampshire (Chapter 5), is developing a new water supply well in a fractured, faulted zone within the metasedimentary bedrock; the bedrock is characterized by discrete flow zones.

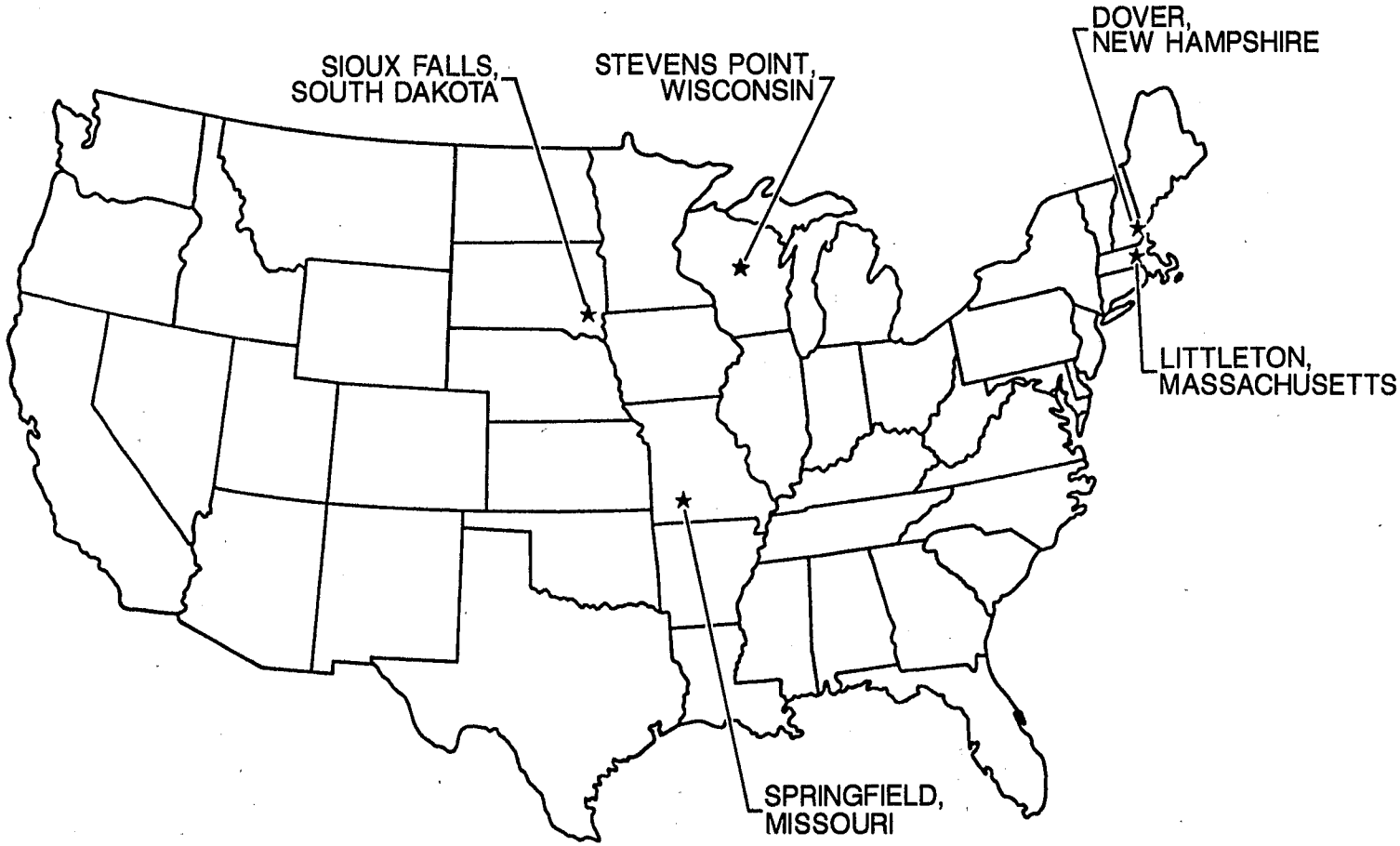

Figure 1-2. Location of the five case study research sites within the United States.

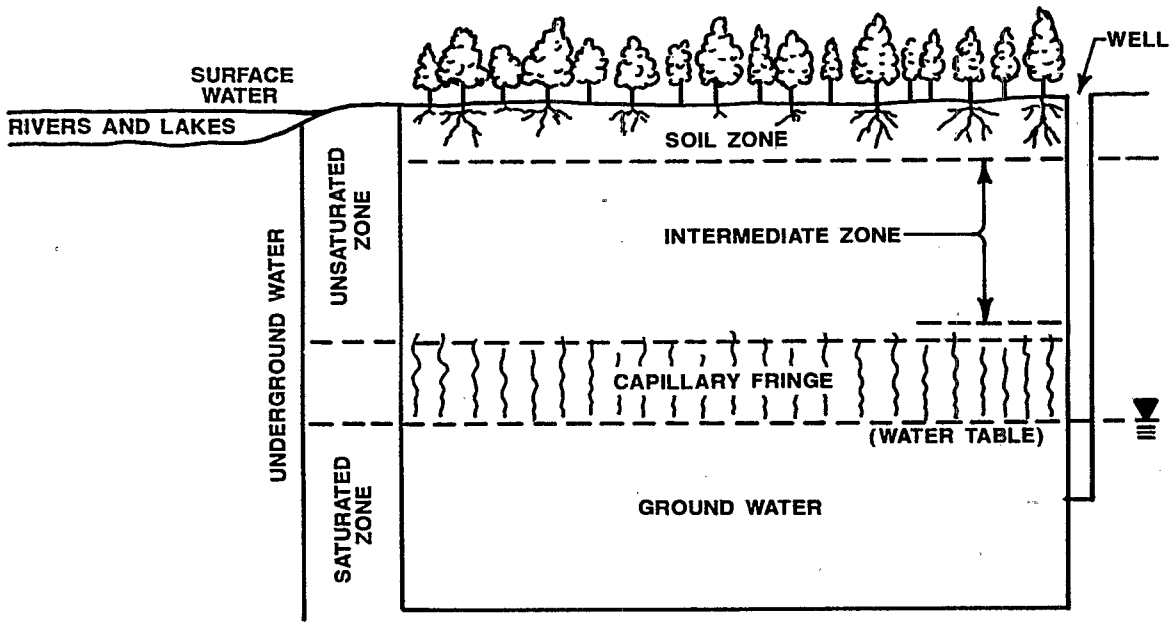

Figure 1-3. Saturated zone within the subsurface. (Modified after Pettyjohn, 1987, p. 4.)

By design, sources are rare within and in proximity to the new WHPA for Dover. However, some common point sources exist, such as gas stations and residential developments. Springfield, Missouri (Chapter 1, Appendix 1-A), is situated in a mature karst setting. The city receives 25% of its drinking water from a large capacity spring, Fulbright Spring, that issues from the base of a pure limestone formation. The springhead protection area (SHPA) contains both point and nonpoint sources of potential contamination.

ORGANIZATION OF CHAPTER 1

Chapter 1 begins with a discussion of basic hydrogeology concepts and equations pertaining to flow systems, WHPA delineation, and ground-water monitoring for different aquifer systems (unconfined, confined, fractured-bedrock, and karst aquifers). The monitoring methodology includes a discussion of general and specific objectives as well as factors to be considered in program planning. Source assessment is addressed as a critical first step in the design of an effective monitoring program. Monitoring program design and development are divided into three phases and discussed in detail, as follows:

- Phase I: WHPP Elements and Scoping Tasks
- Phase II: Research Monitoring Program
- Phase III: Wellhead Protection Monitoring Program

Chapter 1 concludes with a step-wise approach for the design of a wellhead prtoection monitoring program.

HYDROGEOLOGIC CONCEPTS

Basic hydrogeologic concepts related to ground-water flow, wellhead protection, and monitoring network design are discussed in this section. In less technical terms, an aquifer can be defined as "a natural zone below the surface that yields water in suffuciently large amounts to be important economically" (Davis and DeWiest, 1966). Aquifers may exist within unconsolidated sedimentary deposits, porous sandstone, karstic limestone, fractured bedrock, and other geologic materials. The most common subdivision of aquifers is based on stratigraphic setting and hydraulic head relationships: unconfined versus confined aquifers (U.S. EPA, 1987a). In reality, this is not a subdivision, but rather, a continuum from unconfined to semi-confined to confined conditions. Another common way to categorize aquifers is with respect to porosity and flow velocities, or potential ground-water travel times. Ground-water flow in conduits and fractures differs radically from flow in granular aquifers (Quinlan, 1989). Travel times in karst and fractured-bedrock aquifers may be as rapid as hours to days--considerably briefer than in porous, granular aquifers (U.S. EPA, 1987a). Consequently, these more susceptable aquifers should be evalutated differently regarding characterization, delineation, and monitoring methods for wellhead protection. Karst and fractured bedrock aquifers can also exist under unconfined or confined conditions. To simplify discussions in this section, aquifers will be referenced according to four basic categories: unconfined, confined, fractured-bedrock, and karst aquifers. The hydrogeologic factors governing this categorization and their effect on WHPA delineation approaches and monitoring strategies are briefly reviewed.

Ground-Water Systems

Subsurface water beneath the water table in soils and geologic formations that are fully saturated is generally referred to as ground water (Freeze and Cherry, 1979). Technically speaking, aquifers are saturated, permeable geologic units that transmit and yield water in usable quantities to a well or a spring. Ground water is an integral part of the hydrologic cycle, accounting for the movement of water between the ocean, atmosphere, and land (Figure 1-4; modified after Canter et al., 1987).

A ground-water system is comprised of the underlying aquifers and less permeable units, known as aquitards or confining beds. These units partly govern the dynamics of flow between areas where water enters (recharges) and leaves (discharges) the ground-water system (Figure 1-4). Correct identification of the recharge and discharge areas for a public water well, wellfield, or spring is critical for wellhead protection because of increasing surficial contamination and land-use for waste disposal.

A ground-water system stores and transmits water as a function of its porosity (pores and fracture openings) and hydraulic conductivity (water-transmitting capability of the geologic unit), respectively (Pettyjohn, 1987). Ground-water movement, defined in terms of flow velocity and direction, is dictated by hydraulic gradients (slope of the water table or potentiometric surface; Figure 1-4) and hydraulic conductivities.

In a ground-water system, the height to which water will rise in a well (usually given in units of elevation) is the water table, hydraulic, or potentiometric head at a specific point in the aquifer (Figure 1-4). By depicting hydraulic heads on a map (Figure 1-5A) or in cross section (Figure 1-5B)

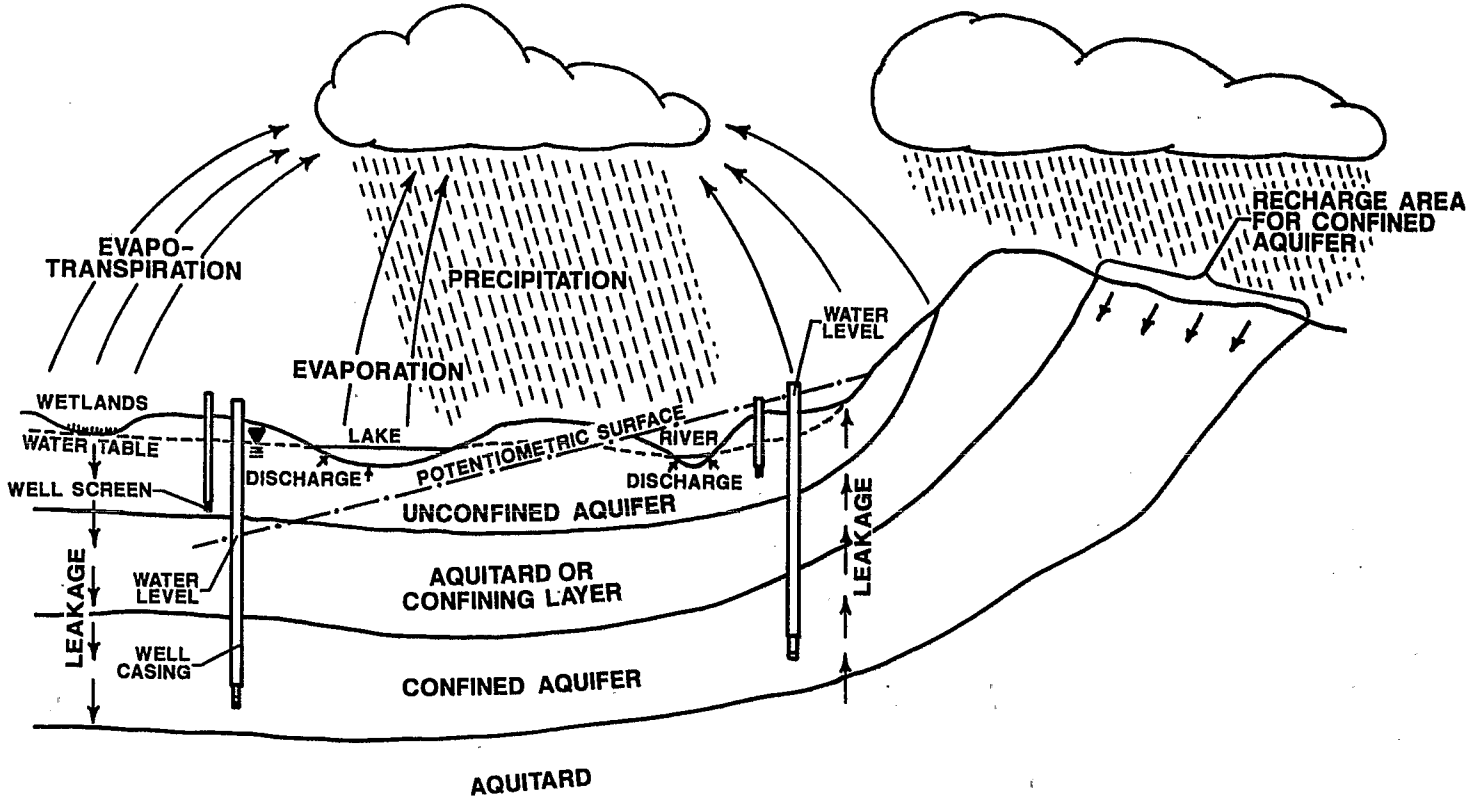

Figure 1-4. Aquifers, confining units, and the hydrologic cycle. (modified after Canter et al., 1987, p. 22.)

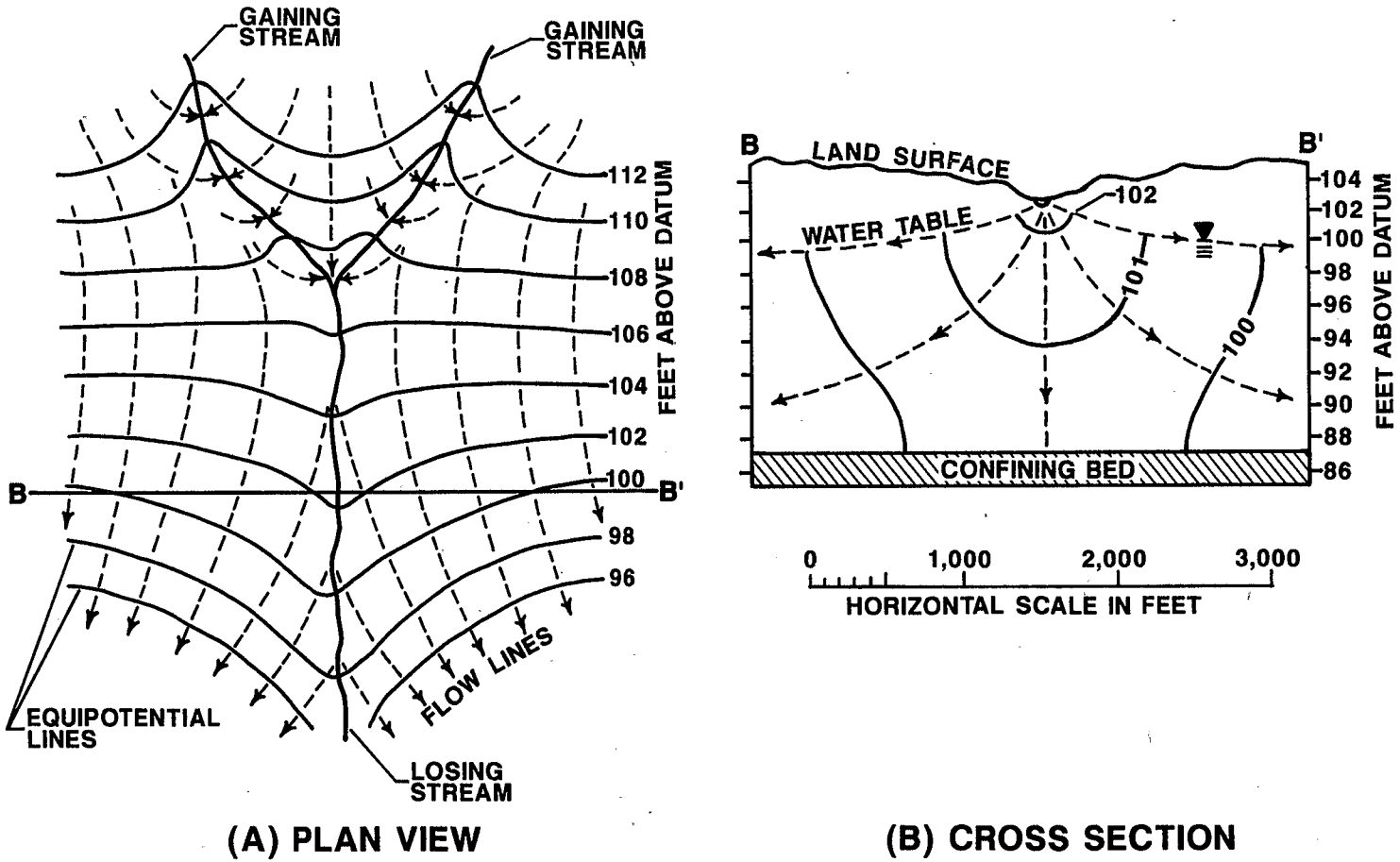

Figure 1-5. Flownet in (A) plan view and (B) cross section, depicting horizontal and vertical hydraulic gradients. (Modified after Pettyjohn, 1987, p. 19.)

for a given depth in the aquifer, the hydraulic gradient can be determined. Ground water moves generally downgradient, from recharge areas of high hydraulic head, or potential, to discharge areas of lower hydraulic head.

In general, ground-water recharge areas are much larger in areal extent than discharge areas in an unconfined system (Pettyjohn, 1987). Recharge is usually intermittent and is controlled by precipitation events. Conversely, discharge occurs as a more continuous process at wells, springs, and surface-water bodies, as well as from evaporation and transpiration (by plants) losses.

Ground-Water Flow

Ground-Water Velocity--

Henry Darcy is recognized for his contribution to present day ground-water knowledge and flow theory. In 1856, Darcy's Law (Equation 1 and Figure 1-6) was formulated relating ground-water discharge (Q, volume per time) directly to the hydraulic gradient (I or dh/dl, head loss per unit distance) and cross-sectional area (A, distance squared) through which flow occurs by a constant, hydraulic conductivity (K, velocity or distance per time):

$$Q = -KAI \qquad \text{(Equation 1)}$$

Darcy's Law assumes smooth, laminar flow through porous media, such as most granular materials and evenly-fractured rock. This relationship is sometimes incorrectly applied to describe conduit and fracture flow. Specifically, in hydrogeologic regimes where rapid, highly irregular flow occurs in large openings such as caves (karst terrains) and along fractures, joints, and bedding surfaces (discrete flow in fractured bedrock), the simplifying assumptions of Darcy's Law are often violated.

Generally, advective processes (bulk movement of flowing ground water) dominate the movement of contaminants within the capture zone of a well or wellfield (U.S. EPA, 1987a; Moltz et al., 1990). Other chemical, biological, and physical processes also affect the fate of contaminants in ground water. Retardation processes can slow the movement of a contaminant behind the centroid of the plume, whereas dispersion can accelerate the contaminant. Generally, if a contaminant is present in the subsurface as more than one phase (multi-phase flow), transport predictions are complicated by the fact that each phase can be moving in different directions and at different rates. To simplify the mathematical presentation in this section, and to develop a tractable model for those implementing WHPA delineation and monitoring, contaminant transport discussions will focus on advective processes.

The most commonly used delineation models for wellhead protection are ground-water flow models. These models do not generally incorporate the factors of solute transport. A few ground-water flow models incorporate a retardation factor. The average rate at which contaminants move through an aquifer is an important parameter to accurately determine and to integrate in the delineation of the WHPA and in the design of the monitoring system. An equation for the average ground-water particle velocity, v, can be formulated by combining Darcy's Law (Equation 1) with

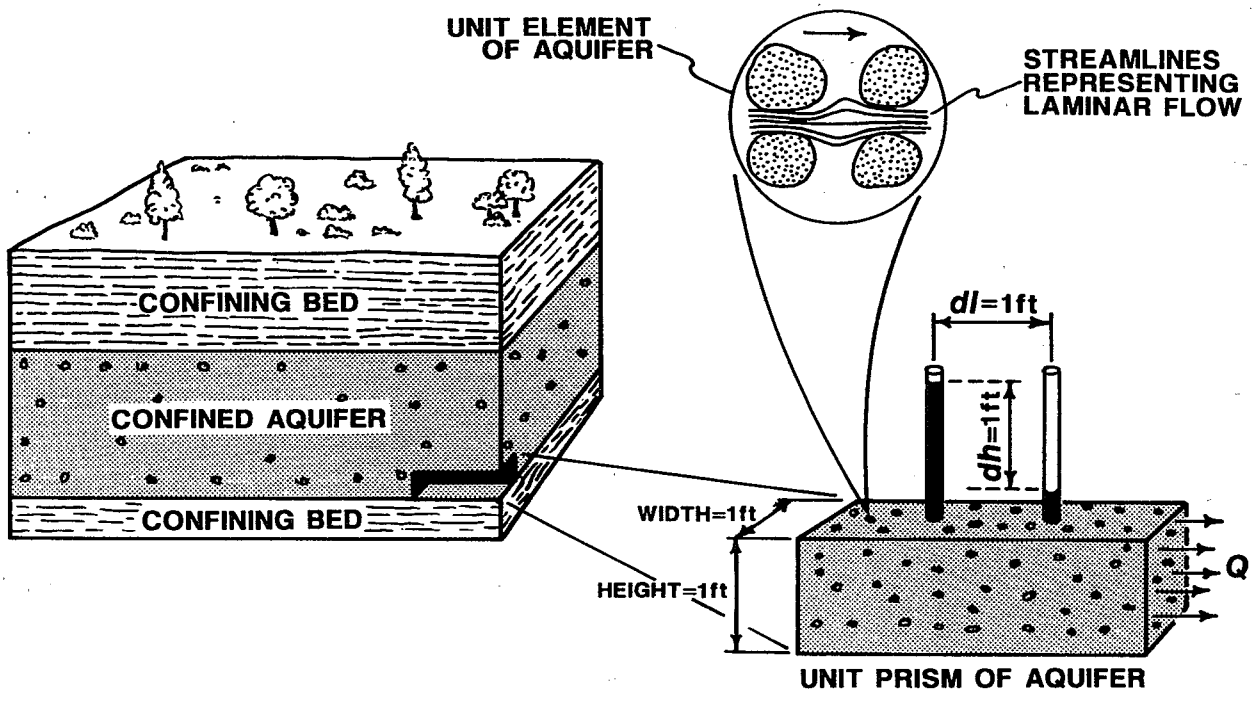

Figure 1-6. Conceptual diagram depicting Darcy's Law. (Modified after Pettyjohn, 1987, p. 10.)

Equation 2 which defines the specific discharge, q. Specific discharge is the rate of flow (discharge per time, Q) per unit area (A), and it bears the units of a velocity term, distance per time. However, specific discharge is not used as a velocity term; it is more commonly referred as to as the Darcy flux (Freeze and Cherry, 1979):

$$q = \frac{Q}{A} \qquad \text{(Equation 2)}$$

Combining terms and simplifying, Equation 3 is derived:

$$q = \frac{Q}{A} = -KI$$

$$q = -KI \qquad \text{(Equation 3)}$$

The average ground-water particle velocity through the pores, v, is found by incorporating the porosity of the geologic unit (n, unitless ratio) in the denominator of Equation 3, yielding Equation 4:

$$v = \frac{KI}{n} \qquad \text{(Equation 4)}$$

Ground-water flow velocity and travel-time calculations are used simply as average estimates of contaminant or water particle movement through an aquifer. Realizing that factors such as dispersion and retardation can complicate solute transport predictions, the actual movement of contaminants within the aquifer may be several times slower or faster than the average advective rate. In complex settings, such as mature karst, travel times for ground water and contaminants vary from hours to weeks in response to differing recharge and flood-pulse events (Quinlan, 1989; Moore et al., 1990).

Conceptual Hydrogeologic Models and Flow Nets--

The success of wellhead protection and detection monitoring is based, in part, on a good technical understanding of (1) the potential and existing sources of contamination, and (2) the hydrogeology of the ground-water supply system. Therefore, an accurate hydrogeologic conceptual model of the ground-water supply system should be developed to support wellhead protection implementation and management. The cause and effect relationships of how the aquifer system works (conceptual model) are expressed by:

- Aquifer water quality and contaminants
- Hydrogeologic units and boundaries
- Aquifer parameters and characteristics
- Ground-water flow dynamics such as recharge and discharge areas, directions, velocities, and flow-system scales (local, intermediate, and regional flow)
- Integration of the flow field created by pumping of the well, or wellfield, over a specific time period and the natural, regional flow gradient

A useful tool to depict flow dynamics is a flow net (Figure 1-5). Specifically, a flow net shows the direction of ground-water movement and can be used to estimate the quantity of ground-water in transit through the aquifer. One set of contour lines, equipotential lines, is drawn connecting lines of equal water-table, hydraulic, or potential head. The other set, flow lines, represents probable paths of ground-water flow through the aquifer.

A flow net, or similar cross-sectional representation of ground-water flow in the water-supply system, should be a prerequisite to designing and establishing a long-term monitoring network for wellhead protection. Hydrochemical characterization of strategic portions of the aquifer used for well production (for example, recharge and discharge areas) is often beneficial to corroborate physical interpretation of the flow system. Without clear definition of how potential and existing contaminant sources are physically and chemically interrelated to wells or springs, via flowpaths, ground-water supplies cannot be protected by management, regulation, or early-warning detection monitoring.

Correct delineation of the WHPA and effective monitoring are by-products of an accurate conceptual model and flow simulation, done with either analytical or numerical methods. Managers and scientists must do the necessary groundwork to ensure protection through proper well siting, representative data base development, and effective allocation of budgetary and labor resources.

Accurate Delineation and Monitoring in Different Hydrogeologic Settings

Ground water for public water supplies is derived from wells and springs. Withdrawal of ground water from a well or wellfield perturbs the natural gradient of the ground-water system creating a "cone-of-depression" in the water table around the well or wellfield (Figure 1-7). An

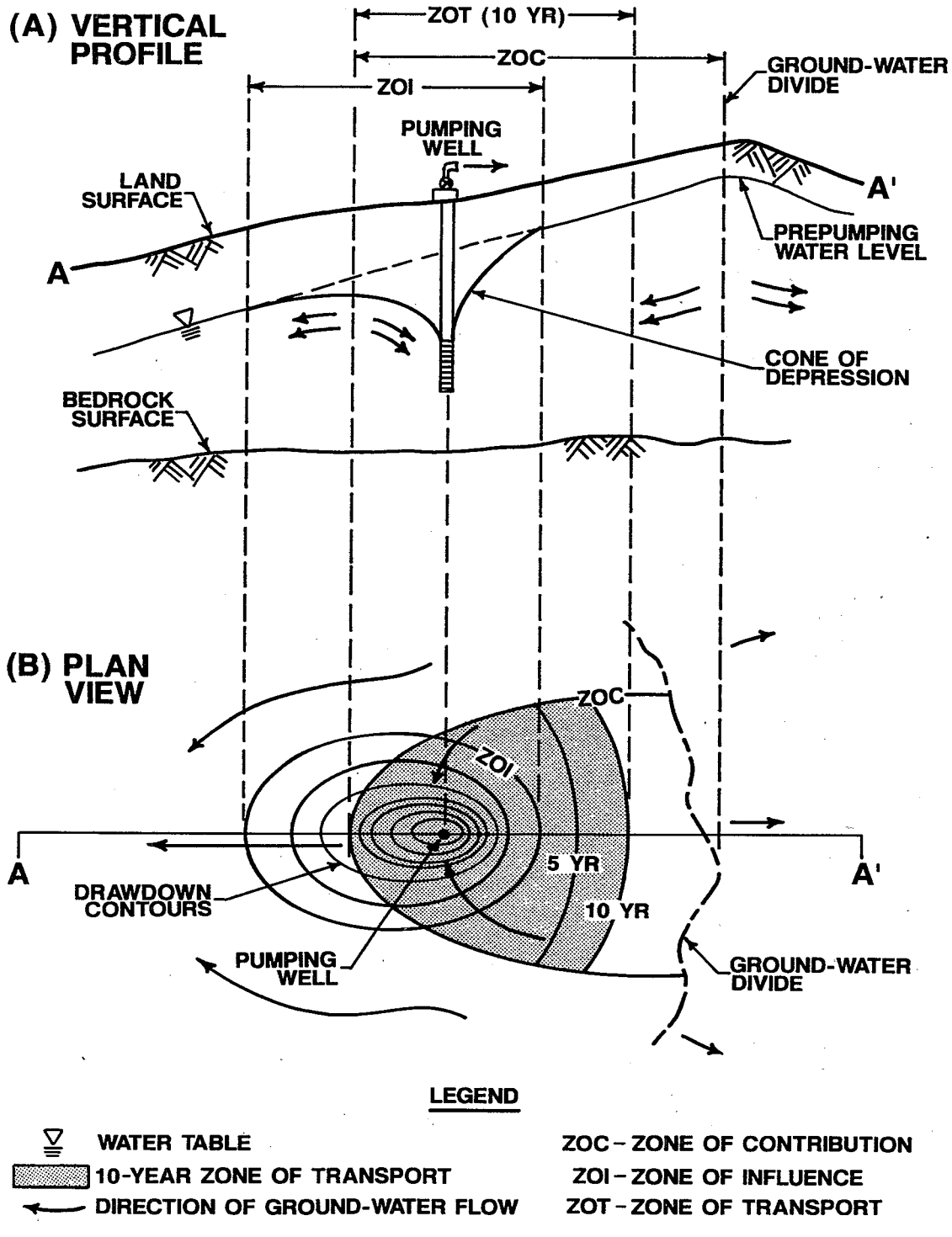

Figure 1-7. The pumping cone-of-depression and wellhead protection area terminology. (U.S. EPA, 1987b.)

unconfined aquifer is a subsurface geologic unit in which water only partly fills the aquifer and the water table is in direct contact with the atmosphere. Where an aquifer is bounded by a relatively impermeable unit, is it is said to be confined (Figure 1-4). The shape of the cone-of-depression in an aquifer of low transmissivity (the product of hydraulic conductivity and saturated aquifer thickness) is steep and localized about the well, as opposed to a flatter and more regional cone in an aquifer of high transmissivity. Transmissivity influences the shape of the cone-of-depression more than the storativity parameter (Freeze and Cherry, 1979).

The projection of the cone-of-depression at the land surface is called the zone of influence (ZOI). The entire area recharging the ZOI, from some criteria-determined (for example, time, distance, drawdown, or boundary; U.S. EPA, 1987a), upgradient point to the downgradient, inflection point (with respect to the well), is called the zone of contribution (ZOC), or capture zone. The ZOI falls within a portion of the ZOC for a sloping water table, and corresponds to the ZOC for a flat water table. It is important to determine the ZOC correctly to ensure wellhead protection because any contaminants introduced within this zone may reach the well or wellfield.

In some fractured-bedrock aquifers, ground-water flow is structurally controlled by directional fractures and faults, resulting in limited, but very productive (discrete) flow zones within the well borehole (Figure 1-8; Griswold et al., 1990). Consequently, simple analytical approaches (U.S. EPA, 1987a) for accurate ZOC determination may not apply. Specialized techniques for fracture and fault identification and flow characterization include: geologic and structure-control mapping, fracture trace analysis, surface geophysical surveying, borehole geophysical logging, and dye-trace testing. The use of these techniques to successfully delineate WHPAs was demonstrated in Lancaster, Pennsylvania (geologic and structure-control mapping and fracture trace analysis; Ogiela and Moore, 1991), and in Dover, New Hampshire (surface geophysics, borehole geophysics, and dye tracing; Griswold et al., 1990; Vernon et al., 1992, Chapter 5).

In carbonate terrains such as Springfield, Missouri (Chapter 1, Appendix A-1), where springs issue from fractures, formation boundaries, or bedding planes (Figure 1-9), the ZOC may not be so easily determined. In simple cases where contributing ground-water divides correspond to surface-water divides, the ZOC determination may be based on surface-water boundary criteria (Moore et al., 1990; U.S. EPA, 1987a). Conversely, in more complex flow regimes, contributing areas and divides may need to be determined or verified using fracture trace analysis, potentiometric-surface mapping, aquifer testing, and dye-tracing methods.

In areas typified by Darcian-flow conditions, such as in homogeneous, porous granular aquifers, standard methods and computer model evaluations for monitoring site selection or well placement may be accurate, yielding optimal sites and reliable data. However, in more heterogeneous, complex ground-water flow terrains, such as fractured-bedrock and karst aquifers, monitoring sites for pollutants should be sited or verified using more appropriate, and often sophisticated, techniques. Finally, to test the dependability of monitoring sites in these regimes, dye tracing may be used to verify interconnection to sources under a variety of flow conditions (Quinlan, 1989).

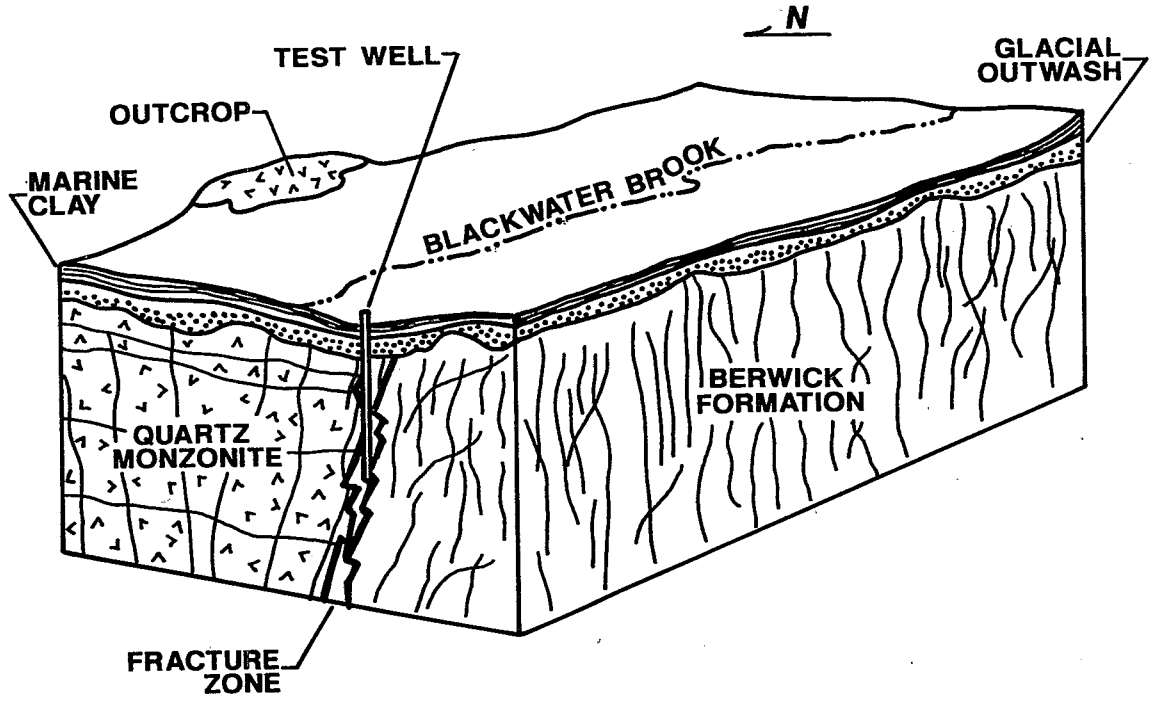

Figure 1-8. Fractured-bedrock aquifer in Dover, New Hampshire, resulting in discrete flow zones. (Griswold et al., 1990.)

WELLHEAD PROTECTION MONITORING CONSIDERATIONS AND OBJECTIVES

Physical and chemical parameter monitoring help to document ground-water trends that can improve hydrogeologic characterization and enhance protection. Todd et al. (1976) wrote: "...the objective of a monitoring program should be to collect, manage, and analyze the data on ground water quality and the sources and causes of ground water pollution, and other information - geologic, hydrologic, and econmonic - necessary to enable the EPA and the other state(s) involved to fulfil their statutory responsibilites as regards protection of ground water quality..." Historical monitoring records (long-term baseline data) provide a reference base to minimize duplication of efforts, to assess program effectiveness, and to optimize program changes. In addition, an extensive monitoring data base enables the analyst to utilize more rigorous techniques to better locate monitoring sites.

Monitoring objectives should be identified and kept in perspective for the duration of the monitoring program. Objectives should be neither too broad nor to limiting. Ideally, a complete objective should address three concerns (Canter et al., 1987). First, it is important to identify the specific reasons for monitoring or the usage of the data. For example, are monitoring data being acquired to identify unknown contaminants from a suspected source or for baseline water quality assessment? Secondly, determine the available types of monitoring access points, such as wells,

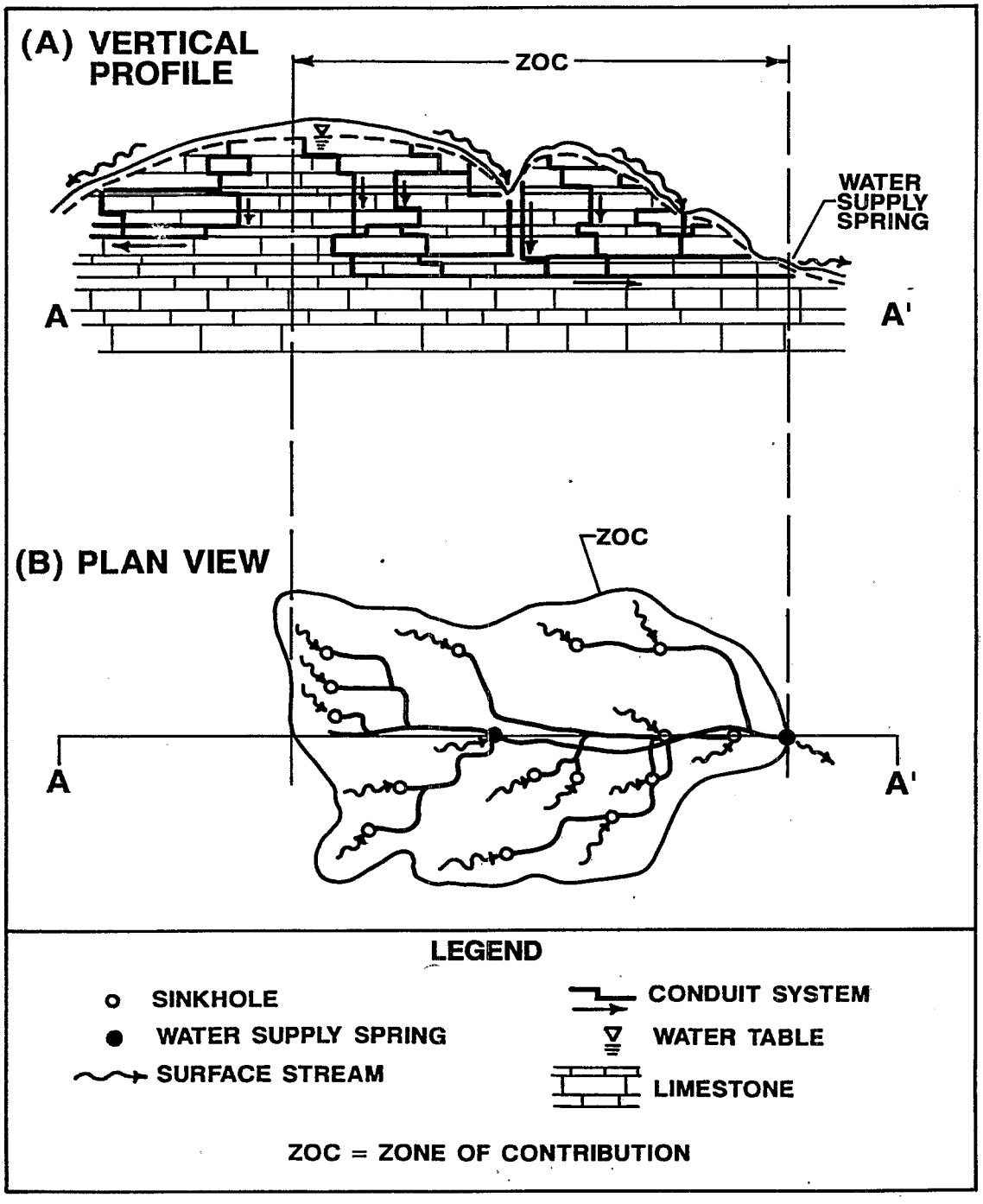

Figure 1-9. Zone of contribution for a water supply spring in a carbonate, karst terrain.

springs, surface-water bodies, and caves. Thirdly, define how the monitoring information will spatially represent the system. For example, are the data being collected to determine the type(s) and concentration(s) of pollutants in the subsurface? Alternatively, is the intent of the data to depict the spatial distribution of a contaminant plume?

Both physical and chemical parameter monitoring apply to wellhead protection. The reasons for ground-water chemical monitoring with regard to wellhead protection were brielfly listed on page 1-1. Physical monitoring examines the behavior of the aquifer over time to enhance understanding of the impact of water withdrawal, well and aquifer performance, and management.

Three types of ground-water monitoring apply to wellhead protection: ambient trend, source assessment, and early-warning detection monitoring (Canter et al., 1987). Ambient trend monitoring detects the temporal and spatial trends in physical and chemical quality of the ground-water system. Source assessment monitoring examines the existing or potential impacts on the physical or chemical ground-water system from a proposed, active, or abandoned contaminant source. Early-warning detection monitoring is conducted upgradient from the wellhead, based on known travel times, to trigger a contingency response to prevent public exposure to contaminants. These types of monitoring are planned to measure or detect contaminants in aquifers and should not be mistaken as preventative or remedial measures (U.S. EPA, 1989b).

Elements that are common to all effective monitoring systems include an accurate conceptualization of the sites and its environs, and hypotheses regarding contaminant migration. Elements of the conceptual model include the sources, pathways, and sinks (or receptors). With respect to the WHPP, the sink is the public water supply well, wellfield, or spring. During the initial stages of program implementation, sources are identified, characterized, and prioritized. Concurrently, WHPAs are delineated based on site-specific ground-water flow and contaminant transport assessment. By integrating each component of the system to form an accurate conceptual model, more efficient monitoring programs can be developed.

SOURCE ASSESSMENT

Contaminant source assessment is a critical first step in designing an effective monitoring program; therefore, it is included as a Phase I task. Target monitoring parameters for early warning detection and source assessment are best selected from a comprehensive list of known and suspected contaminants associated with land-use activities and practices. Secondly, critical or optimal monitoring sites may be determined, reflecting prioritization of sources. In this way, monitoring objectives can be directed toward immediate concerns. Optimal monitoring parameters are determined to enhance protection and characterization, to improve efficiency, and to limit the cost of the program.

An inventory of common sources of contamination within and in proximity to WHPAs is given in Table 1-2 (U.S. EPA, 1990a). General guidance for conducting inventories of, prioritizing, and characterizing sources for wellhead protection is given in the following U.S. EPA technical assistance documents:

- Managing Ground Water Contamination Sources in Wellhead Protection Areas: A Priority Setting Approach (1991a)
- A Guide for Conducting Contaminant Source Inventories for Public Drinking Water Supplies (1991b)
- A Review of Sources of Ground-Water Contamination from Light-Industry (1990b)
- Tools for Local Governments (1989b)

Table 1-3 presents a methodology for assessing contaminant point sources (U.S. EPA, 1989c) that is applicable to most sites and ground-water systems.

The source assessment process can take a qualitative approach (accomplished in days) versus more quantitative approaches (accomplished in months or years), entailing source characterization, remedial investigations, and a formal risk ranking and screening system. The approach taken is dictated by (1) the threat of contamination that sources pose to the water supply, and (2) the unique characteristics of the municipality such as WHPP objectives, financial resources, implementation timing, regulatory framework, and labor resources.

MONITORING METHODOLOGY

Long-term monitoring programs were researched and are described in Chapters 2 through 5, for four case study WHPAs. The proposed monitoring programs are expected to be implemented over a 3- to 5-year-period, as money and resources are available and as they are allocated in a prioritized manner. It is important to emphasize that wellhead protection monitoring is a long-term management option for WHPP implementation. Therefore, the long-term monitoring program should be reviewed and improved over the life span of the WHPP. As a result of the case study monitoring research, a three-phase approach for monitoring program development is recommended for implementors (Table 1-4). A flowchart is given in Figure 1-10 that outlines the 15-step monitoring methodology. Monitoring for wellhead protection is generally initiated following the source assessment, delineation, and initial regulation phases of program development.

Phase I: WHPP Elements and Scoping Tasks

Ground-water monitoring is a relatively expensive, time consuming, and technically demanding task. For these reasons, establishing a scientifically sound and functional WHPP is recommended to provide adequate protection of the public water supply. At the same time, monitoring planning and information gathering can proceed. Basic elements of WHPP implementation include: designating roles and a management framework, preliminary WHPA delineation, and source assessment. To support the research monitoring task, a preliminary information base of ancillary data (geologic and topographic maps, lithologic logs, aquifer properties and characteristics, etc.) and monitoring data

TABLE 1-2. COMMON SOURCES OF GROUND-WATER CONTAMINATION IN WELLHEAD PROTECTION AREAS *

Agricultural	Commercial	Industrial	Residential	Waste Management	Naturally Occuring
Animal burial areas	Airports	Asphalt plants	Fuel storage systems	Fire training facilities	Ground-water and surface-water interactions (iron and manganese)
Animal feedlots	Auto repair shops	Chemical manufacture, warehousing, and distribution activities	Furniture and wood strippers and refinishers	Hazardous waste management units	Natural leaching (uranium and radon gas)
Chemical application (pesticides, fungicides, fertilizers, etc.)	Boat yards	Electrical and electronic products and manufacturing	Household hazardous products	Municipal incinerators	Salt-water intrusion and brackish water upconing
Chemical storage areas	Car washes	Electroplaters and metal fabricators	Household lawns (chemical application)	Municipal landfills	
Irrigation	Cemetaries	Foundries	Septic systems, cesspools, and water softeners	Municipal wastewater and sewer lines	
Manure spreading and pits	Construction areas	Machine and metalworking shops	Sewer lines	Open burning sites	
	Dry cleaning establishments	Manufacturing and distribution sites for cleaning supplies	Swimming pools (chlorine)	Recycling reduction facilities	
	Educational institutions (labs, lawns, and chemical storage areas)	Mining and mine drainage		Stormwater drains and retention facilities	
	Gasoline stations	Petroleum products production, storage, and distribution center		Transfer stations	
	Golf courses (chemical application)	Pipelines (oil, gas, and coal slurry)			
	Jewelry and metal plating	Septic lagoons and sludge			
	Laundromats	Storage tanks			
	Medical institutions	Toxic and hazardous spills			
	Paint shops	Wells (operating and abandoned)			
	Photography establishments and printers	Wood preserving facilities			
	Railroad tracks and yard maintenance				
	Research laboratories				
	Road de-icing activities (road salt)				
	Road maintenance depots				
	Scrap and junkyards				
	Storage tanks (above and below ground)				

* U.S. EPA, 1990a.

TABLE 1-3. METHODOLOGY FOR ASSESSING POINT SOURCES OF CONTAMINATION (Modified after U.S. EPA, 1989c, Table I-1, page I-2)

Step 1	Identify all existing and potential contaminant sources within and in proximity to the wellhead protection area.
Step 2	Prioritize contaminant sources for characterization based on the degree of the threat or the need for controls.
Step 3	Delineate the three-dimensional distribution of existing contamination to accurately prioritize sources for characterization or remediation.
Step 4	Document how wastes or products are managed within the source to more accurately predict the spatial and temporal variability of contaminant releases. Important factors to consider include: • Total quantity of the waste • Locations of waste treatment, storage, and disposal • Transportation of wastes • Age of source • Spatial and temporal variability of waste management
Step 5	Document the characteristics of the wastes at the source, considering: • Waste product interactions • Chemical compatibility with the containment system material • Fate and transport within the subsurface
Step 6	Document the condition of the source to evaluate its capacity to contaminate the ground water. An important item to consider is the need for or adequacy of containment features.
Step 7	Assess the extent of contamination at the source, considering: location, horizontal and vertical distributions, plume configuration, and media effects of air, soil, surface-water, and ground-water quality.

(existing water quality, contaminant, and physical parameters) should be compiled and reviewed to determine general information limitations and data gaps. The strategy is to maximize information content; to define monitoring objectives; and to conduct field studies with the least, but still adequate number of monitoring points, which can later be integrated into the long-term monitoring program (Pfannkuch, 1982). Phase I generally requires a 3- to 6-month time period for completion.

Phase II: Research Monitoring Program

Canter et al. (1987) describe research monitoring as "...the development of information on how the subsurface system operates." The research monitoring phase of the planning process entails data acquisition and interpretation to improve, or verify, elements of the hydrogeologic conceptual model. A technically-defensible model ensures a more protective and reliable long-term monitoring program.

TABLE 1-4. WELLHEAD PROTECTION MONITORING PROGRAM DEVELOPMENT (Three-Phase Approach)

Phase I: WHPP Elements and Scoping Tasks	• WHPP implementation, roles, and management framework • Preliminary WHPA delineation: - Criteria - Methods • Source identification and prioritization • Informational case compilation, review, and limitations • Monitoring objectives: - General objectives: what, where, and how of data usage - Specific objectives: Ambient trend monitoring Source assessment monitoring Early-warning detection monitoring
Phase II: Research Monitoring Program	• Water quality characterization - Nature and fate of contaminants in the subsurface - General chemical and physical comparison - Isotope analyses and age dating - Indicator parameters • Aquifer testing and characterization methods • Refined WHPA delineation - Criteria - Methods • Flowpath determination
Phase III: Wellhead Protection Monitoring Program	• Introduction • Monitoring objectives • Optimal monitoring sites - Location - Aquifer depth • Site design, construction, and installation • Monitoring parameters and frequencies • Quality assurance and control considerations • Complimentary contingency planning - Spill-response plan - Water development site

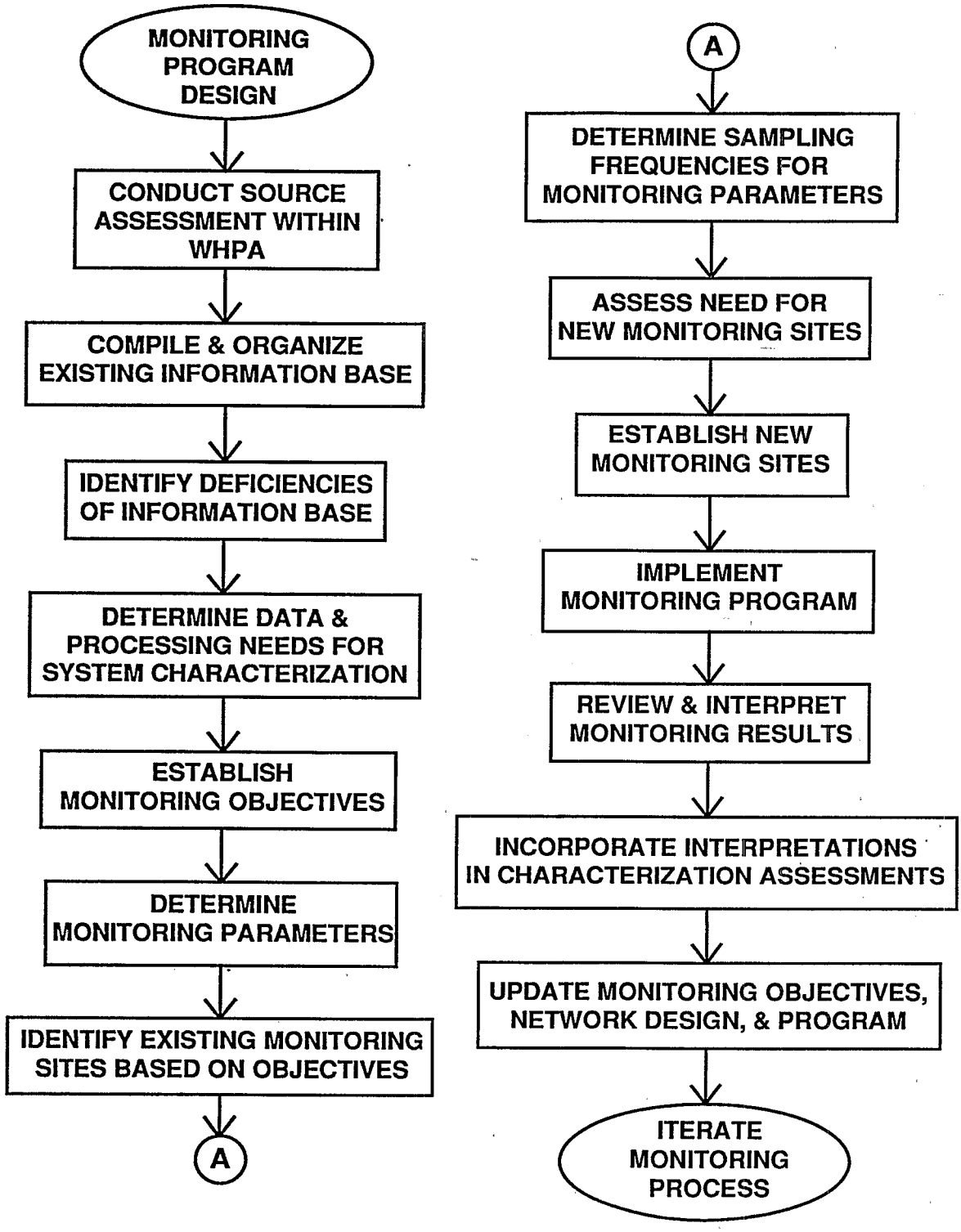

Figure 1-10. Flowchart of the 15-step monitoring methodology for wellhead protection areas.

To determine effective, long-term monitoring strategies, the nature of the contaminant, the mechanisms of contaminant propagation, and the physical properties of the site must be known and understood (Pfannkuch, 1982). Complete site characterization and monitoring require extensive knowledge of the temporal and spatial variability of the physical properties of the site.

Research monitoring for wellhead protection focuses on baseline water quality characterization, aquifer testing and characterization, refined or verified WHPA delineation, and ground-water flowpath determination to relate sources to the water supply well or spring. The product of the research monitoring is a proposed long-term monitoring program that may be partly implemented in Phase II. Case study research demonstrates that this phase may require 1 to 1-1/2 years for completion, depending on the complexity of the site hydrogeology, as well as the quality and the comprehensiveness of the informational base.

Water Quality Characterization--

Baseline water quality and contaminant parameters need to be established to document changes in the natural system due to seasonal or land-use impacts, and ultimately, degradation. Surficial or shallow aquifers are vulnerable, exhibiting rapid and dramatic changes in quality. Deeper or confined aquifers exhibit a more constant chemical quality that often reflects geochemical reactions between the ground water and the aquifer matrix.

In areas where cyclic fluctuations in water quality occur, or documentation of aquifer responses is required for characterization purposes, it may be necessary to install a network of carefully sited, nested wells (multiple well configuration with screened intervals at different depths in the aquifer) to properly monitor discrete sections of the aquifer (Pettyjohn, 1987). To establish a cyclic pattern in water quality, data may need to be acquired at a daily, weekly, or monthly frequency. Field-deployable, rapid-turnaround monitoring methods are now available to facilitate frequent monitoring, particularly ion-mobility probes, immunoassay kits, and a variety of test kits (Eschner et al., 1991). In situ, real-time monitoring devices, such as fiber optic sensors, are still being developed in the laboratory and are not readily available for field use. In addition, acquisition and use of these devices is quite costly.

Nature and Fate of Contaminants in the Subsurface--The fate of contaminants that reach the ground water is partially controlled by the chemical volatility and solubility of the contaminant in water (Figure 1-11; U.S. EPA, 1989a). Contaminants can be characterized as either water soluble (miscible), partially soluble, or insoluble (immiscible). Landfill leachates, road salts, acid-mine drainage, and liquid fertilizers are good examples of water-soluble contaminants, or "mixers." Hydrocarbons, solvents, and other organic compounds are good examples of liquids that do not readily dissolve in water and can exist as a separate phase. Generally, separate-phase fluids are subdivided into two classes: (1) those that are lighter than water, light nonaqueous phase liquids (LNAPLs or "floaters") and (2) those with a density greater than water, dense nonaqueous phase liquids (DNAPLs or "sinkers") (Figure 1-11).

The migration of water-soluble pollutants within the saturated zone is primarily dictated by local hydraulic gradients (Figure 1-11). As the contamination is transported, it will form a plume and eventually be diluted due to physical processes such as advection, dispersion, retardation, filtration, time, and distance of travel (Pettyjohn, 1987). Through advection, non-reactive contaminants are transported and diluted at the average velocity that a water particle moves through the aquifer. There is also a tendency for non-reactive contaminants to spread out from the path they would be expected

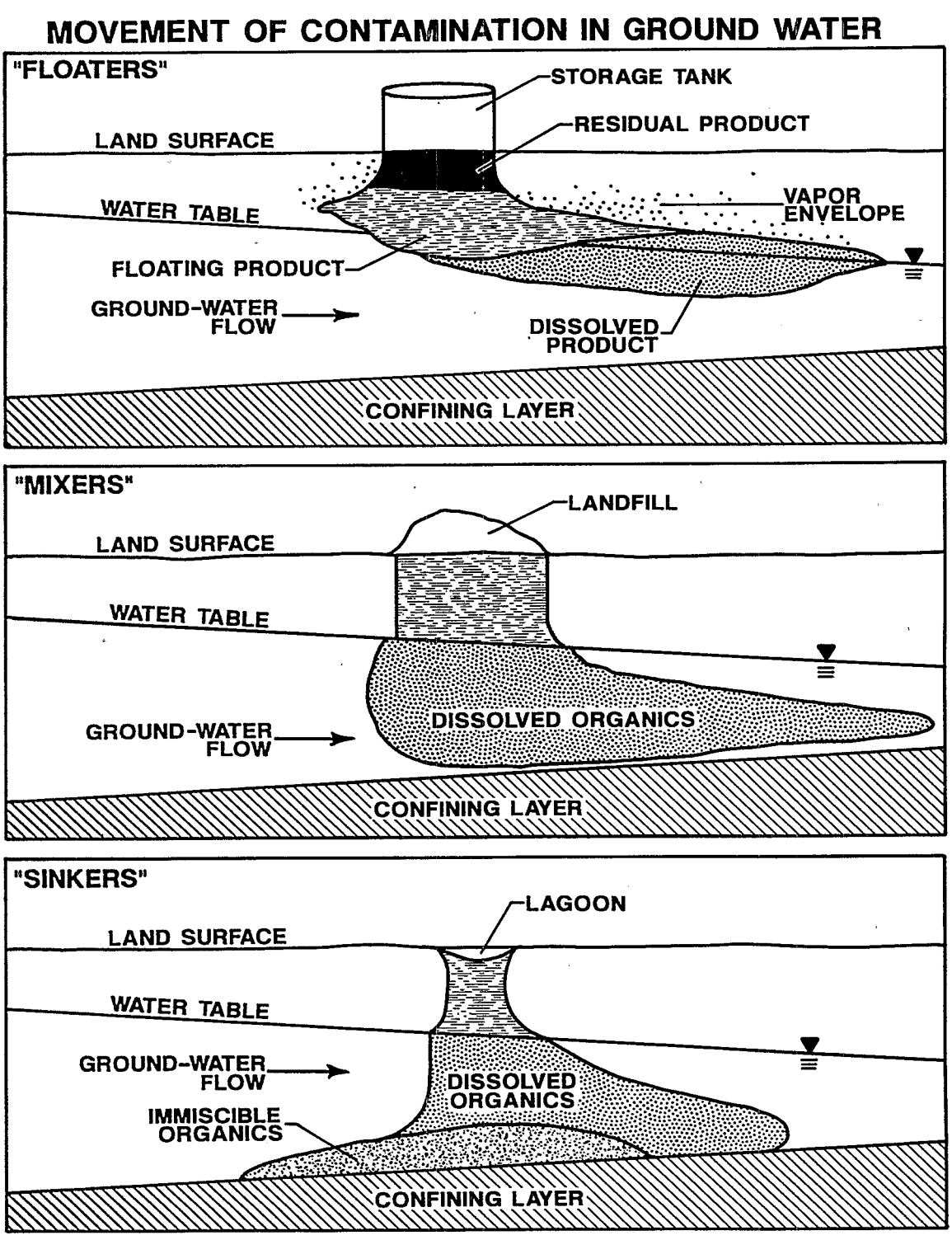

Figure 1-11. The fate of organic contaminants in the subsurface. (U.S. EPA, 1989a.)

take, as dictated by advection (Freeze and Cherry, 1979). This phenomenon is known as hydrodynamic dispersion. It causes the concentration of contaminants to decrease with increasing distance from the source as a function of velocity and the configuration of aquifer pore space. The advance of the contaminant plume can be retarded relative to the ground-water velocity and solute concentrations can be lowered if chemicals are sorbed onto or undergo chemical reactions with the surrounding matrix. Natural filtration by the aquifer matrix removes suspended particles from the water, such as iron, manganese, and other precipitates. Finally, in the case of organic compounds, microbial degradation by organisms (bacteria, algae, fungi, yeasts, and protozoans) within the aquifer can occur if the system is not overloaded by pollutants and appropriate nutrients are available (U.S. EPA, 1989a). A good example of this process is the degradation of tetrachloroethylene to more toxic vinyl chloride (Manahan, 1984).

Contaminants lighter than water (LNAPLs), such as gasoline, heating oil, kerosene, jet fuel, and aviation gas tend to form a lens on the water table and a three-phase system within the unsaturated zone: water, product, air and vapors (U.S. EPA, 1989a). Below the floating lens, dissolved constituents such as benzene, toluene, and xylene are carried away, forming the contamination plume. Water-table fluctuations can spread the floating contaminant over a greater thickness of the aquifer (Figure 1-12). In addition, if a source leaks, stops, then leaks again, the resultant floating lens may be layered with water, dissolved constituents, and the contaminant.

Insoluble contaminants denser than water (DNAPLs), such as chlorinated hydrocarbons [trichloroethane, carbon tetrachloride, chlorophenols, chlorobenzenes, tetrachloroethylene, and polychlorinated biphenyls (PCBs)], remain as a separate phase, moving under gravity through the unsaturated and saturated zones (U.S. EPA, 1989a). DNAPLs can partition into a vapor phase within the unsaturated zone and settle by gravity within the capillary fringe. Water flowing by can dissolve the residual DNAPLs or vapors, creating a dissolved constituent plume within the aquifer. If large amounts of the dense contaminant are released, residuals can penetrate the entire aquifer forming pools in depressions (U.S. EPA, 1989a). In extreme cases, these residuals can move down the slope of an impermeable boundary, spreading the contamination in directions that are independent of the ground-water flow directions and gradients (Figure 1-11). Fine-grained strata can act as effective barriers to the contaminants. Fractures and karst openings, on the other hand, can act as conduits, allowing further penetration into deeper aquifers.

The nature, fate, and transport of contaminants should be considered in the design of monitoring wells, networks, and remediation programs (1) to effectively determine the well position, screen depth, and interval; (2) to provide accurate assessments of the extent and nature of the contamination; and (3) to avoid, or minimize, spreading of the contaminant into clean areas of the aquifer.

General Chemical and Physical Composition--Ground water contains natural chemical constituents in solution. The type and quantity of these constituents depends upon the geochemical environment, migration, and source of ground water (Todd et al., 1976). The most common chemical

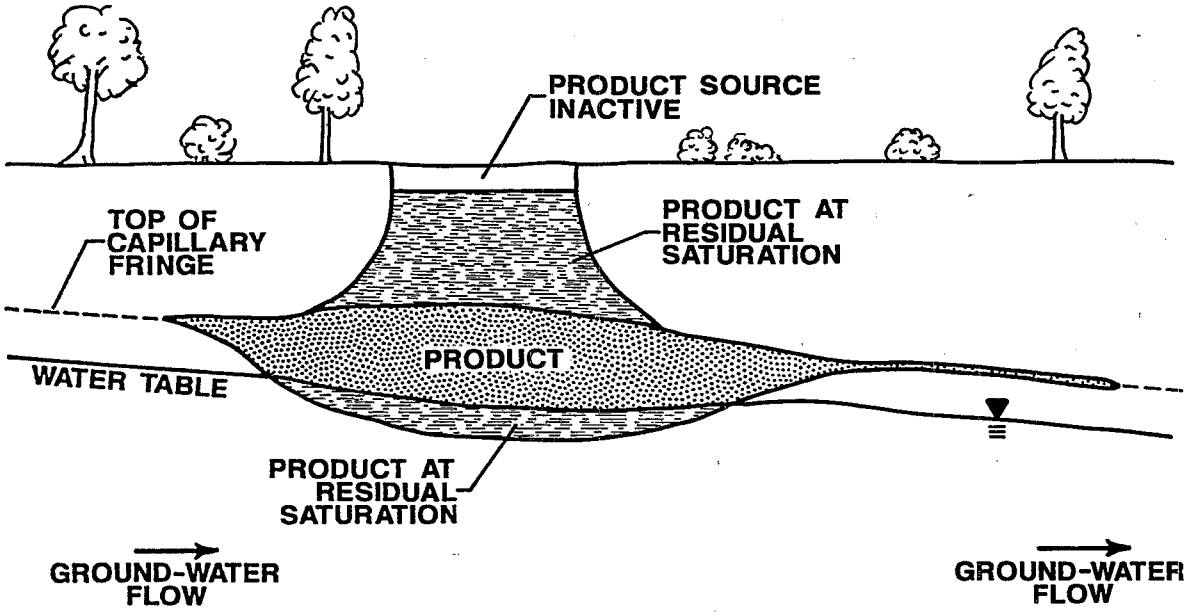

Figure 1-12. Organic contaminant spreading within the unsaturated zone because of water-table fluctuations. (U.S. EPA, 1989a.)

constituents in ground water, in the 0.01 to 1,000 parts per million (ppm) or milligrams per liter (mg/l) range, are the major cations, major anions, and undissociated constituents listed in Table 1-5 (Davis and DeWiest, 1966).

TABLE 1-5. COMMON CHEMICAL CONSTITUENTS IN GROUND WATER

Cations	Anions	Undissociated
Calcium	Carbonate	Silica
Magnesium	Bicarbonate	Strontium
Sodium	Sulfate	Boron
Potassium	Chloride	
Iron	Nitrate	
	Fluoride	

By analyzing and graphing the inorganic chemical composition of ground water, interpretations can be made concerning the classification, origin, and source of the water, ground-water flow paths, and interconnection between aquifers. Often these data are used to refute, corroborate, or refine the hydrogeologic conceptual model of the study area. For visual inspection of

water-chemistry data, several commonly used graphical methods are suggested such as bar, circle, radial, or vector graphs, Stiff diagrams, and Piper or trilinear diagrams (Davis and DeWeist, 1966; Freeze and Cherry, 1979; Hem, 1989).

Other general chemical indicators (parameters that have chemical units of measure) of water quality are alkalinity (total), biochemical oxygen demand (BOD), chemical oxygen demand (COD), dissolved oxygen (DO), hardness, pH, total organic carbon (TOC), and total dissolved solids (TDSs). Physical properties (parameters that have physical units of measure) that often reflect water quality include: color, odor, redox potential (Eh), specific conductivity, temperature, and total suspended solids (TSSs).

Data from a complete inorganic chemical analysis can be used to check the accuracy of the laboratory results by calculating the cation-anion balance (Hem, 1989). The sum of the cations, in milliequivalents per liter, should equal the sum of the anions within an error range of less than 5 to 10 percent. This check should supplement the laboratory quality assurance plan, which covers the laboratory's standard operating procedures for equipment use and maintenance, as well as operator variation. A comparison of determined versus calculated values of TDS is another procedure for checking analytical accuracy; the values should agree within a few mg/l. An approximate accuracy check is possible by determining if the TDS value is from 0.55 to 0.75 times the specific conductance. Finally, comparison of results from the same or similar sources is a good way of identifying errors of transcription or analysis.

Isotope Analysis and Age Dating--Isotopes can be used to infer much about the age, source, and movement of ground water. The radioactive isotopes tritium, carbon-14, and chlorine-36, which are used to measure water age, are of primary interest. The stable isotopes, oxygen-18 and deuterium, are used to understand the source of water and processes that have affected the water since it entered the aquifer system (Davis, 1988; Davis and Bentley, 1982; Drever, 1982).

The most common use of tritium analysis is to identify ground water which contains recent recharge water. The heavy isotope of hydrogen, tritium, has a short half-life of 12.3 years and occurs in water from both natural and man-made sources. Tritium is produced naturally in the earth's atmosphere by the interaction of cosmic-ray-produced neutrons with nitrogen (Freeze and Cherry, 1979). Tritium concentrations in precipitation ranged from 5 to 15 tritium units (TUs) prior to the 1950s. As a result of nuclear testing from 1952 to 1963, tritium levels in the atmosphere increased by three orders of magnitude relative to background levels (Schlosser et al., 1988). Consequently, the relative abundance of tritium in ground water is used to classify "pre-bomb" versus "post-bomb" water, to distinguish age zones within the recharge area of the flow system, and to identify zones of ground-water mixing. As a general rule of interpretation, a TU of less than 3 suggests water ages greater than 30 years, and a TU of greater than 20 indicates ground water originating since 1961 (Hendry, 1988; Davis and Bentley, 1982)

Carbon-14 is useful for studying ground water that has been isolated from the atmosphere for several hundred to thousands of years. Before water is recharged to the aquifer, the carbon content in the water is balanced with the carbon content in the soil atmosphere. Once the recharge water enters the ground-water system and is no longer in contact with the atmosphere, the carbon-14 content decays with time in accordance with its half-life of 5,730 years. This creates a method for age dating

ground waters up to approximately 50,000 years. Unless analytical results are corrected by known carbon-13 to carbon-12 ratios within sampled ground water and the host rock, ages determined by the carbon-14 method are relative and can only be used for comparison (Strait and Moore, 1982).

Chlorine-36 has been used for some ground-water studies (Rosenfeld, 1991). The advantages of using chlorine-36 are (1) its long half-life of 300,000 years and (2) its tendency to travel with the water and not to be significantly affected by sorption or mineralization reactions. Complications associated with chlorine-36 age dating of ground water include the following:

- Knowing the original concentrations of chlorine-36 in the recharge water
- Estimating subsurface production of chlorine-36 and chloride contributions from mineral dissolution and ground-water mixing
- Correcting for anthropogenic sources due to nuclear testing

Clorine-36 has also been used as a tracer of recent recharge water, similar to tritium, due to production by nuclear tests (Bentley et al., 1986; Davis, 1988).

The concentrations of oxygen-18 and deuterium in ground water is established during atmospheric precipitation as a result of evaporation, condensation, freezing, melting, chemical reactions, or biological processes. It is also highly dependent on temperature. Once water moves beyond the upper part of the soil zone, oxygen-18 and deuterium concentrations become a characteristic property of the ground water, enabling the source and mixing patterns of the ground water to be determined (Freeze and Cherry, 1979). Such interpretations are based on departures of isotope concentrations from relationships based on global precipitation surveys (Damsgaard, 1964).

Indicator Parameters--Indicator parameters have been the subject of much research and debate for hazardous waste site characterization. Research conducted by Plumb (1992, 1991, 1987, 1985) and Rosenfeld (1990a) lends strong evidence to the selection of volatile organic compounds (VOCs) as standard screening parameters in proximity to hazardous waste sites. Plumb and Pitchford (1985) documented that VOCs are the single most abundant class of organic contaminants in disposal-site ground water. A VOC scanning approach is recommended for moderate- to high-risk industrial and commercial sources in proximity to WHPAs. The VOC scan results would be used to establish the need for more comprehensive organic analyses for less common compounds such as semi-volatiles, pesticides, and PCBs. There are a limited number of situations where VOC monitoring may not be applicable:

- Incinerated municipal waste ash landfills, where an inorganic monitoring strategy is recommended (Plumb, 1990)
- Wood preservation and creosoting operations, where a semi-volatile monitoring strategy is recommended (Rosenfeld and Plumb, 1991)
- Single, well-defined waste disposal facilities, where customized monitoring parameters may be determined due to the presence of consistent sets of detected compounds (Rosenfeld, 1990a)

In an industry-specific survey of ground-water contaminants, Rosenfeld (1990b) found that at aerospace, metal manufacturing, and military facilities the predominant contaminants were VOCs. At chemical and oil-refining facilities, the predominant organic type varied from site to site. Rosenfeld

concluded that it may not be possible to utilize these industry-specific monitoring parameters in regional investigations due to the overlap of compound distributions at different industries.

In summary, Table 1-6 lists recommended monitoring parameters for the five categories of sources typically found in proximity to WHPAs (Table 1-2).

TABLE 1-6. MONITORING PARAMETERS FOR CATEGORIES OF SOURCES IN PROXIMITY TO WELLHEAD PROTECTION AREAS

Source Category	Monitoring Parameters
Agricultural	General water quality* Nutrients and microbiological† Pesticides‡ VOC scan (optional)§
Light, Commercial and Industrial Facilities	General water quality Heavy metals§ VOC scan Gross alpha and beta (optional)§
Residential	General water quality Nutrients and microbiological Heavy metals (optional) VOC scan (optional)
Waste Management	General water quality VOC scan/priority pollutant scan (optional)§ Heavy metals Nutrients and microbiological (optional) Gross alpha and beta (optional)
Naturally Occurring	General water quality Uranium and radon (optional) Heavy metals (optional)

* General water quality parameters include alkalinity (total), biochemical oxygen demand (BOD), chemical oxygen demand (COD), color, dissolved oxygen (DO), hardness, major cations and anions, odor, pH, redox potential (Eh), specific conductivity, temperature, total organic carbon (TOC), total dissolved solids (TDSs), and total suspended solids (TSSs).

† Nutrients and microbiological parameters include ammonia, bacteria, nitrate, nitrite, total nitrogen, and viruses.

‡ Selection of pesticides for sampling and analysis is application- and site-specific. Refer to Munch et al. (1990) for a brief discussion of pesticide parameters and methods of analysis.

§ Refer to the following for a complete list of parameters and methods of analysis:

- VOCs: U.S. EPA, 1991d; 1990c; 1990d
- Heavy Metals: U.S. EPA, 1991e; 1990c
- Gross alpha and beta: U.S. EPA, 1991e; 1990c

General water quality parameters include both the chemical and physical indicators discussed in the General Chemical and Physical Composition section. Nutrients and microbiological parameters include ammonia, bacteria, nitrate, nitrite, total nitrogen, and viruses. U.S. EPA references 1991d and 1990c provide the reader with a detailed listing of additional nutrient and microbiological parameters for monitoring and associated methods of analysis. Pesticide parameter information is referenced in Munch et al., 1990. VOC scan and parameter information is referenced in U.S. EPA (1991d, 1990c, 1990d). The heavy metal parameters consist of the inorganics excluded from the general water quality and nutrients groups such as aluminum, arsenic, asbestos, barium, cadmium, chromium, lead, mercury, selenium, silver, and so on (U.S. EPA, 1991e; 1990c). The selection of parameters from the heavy metals group for monitoring is, again, use- and site-specific. Parameters and methods of analysis for the priority pollutant and gross alpha and beta groups are referenced in U.S. EPA (1991d and 1990d) and U.S. EPA (1991e and 1990c), respectively.

Aquifer Testing and Characterization Methods--

Often, review of the existing informational base identifies gaps in hydrogeologic knowledge and data that must be addressed in order to design the monitoring program. To fill these gaps, field investigations are necessary. These include well and piezometer installation, ground-water sampling, screening samples for water quality, aquifer testing, areal water-level monitoring, structural and geologic mapping, fracture trace analysis, surface and borehole geophysical studies, and dye tracing. Hydrologic investigations generally include well or piezometer installation so that aquifer testing and ground-water sampling can be performed. Aquifer testing involves analyzing the change in water levels (hydraulic or piezometric head data), with respect to time, by ground-water withdrawal from wells. From these field investigations, conducted in amenable hydrogeologic regimes (primarily Darcian-flow conditions), the following site-specific information can be obtained (Pettyjohn, 1987):

- Position and thickness of aquifer or confining units
- Aquifer and confining unit properties such as transmissivity, storativity, and porosity
- Position and nature of aquifer boundaries
- Location and amounts of ground-water withdrawal
- Location, type, and quantity of water-quality or contaminant constituents

The reader is referred to other publications, which discuss issues in detail beyond the scope of this section. These publications address well design and installation (Aller et al., 1989; ASTM, In publication; Campbell and Lehr, 1973; Driscoll, 1986; Nielsen, 1991, U.S. EPA, 1986; U.S. EPA, 1977; U.S. EPA, 1975); ground-water sampling (Barcelona et al., 1989; Barcelona et al., 1985; Canter et al., 1987; Garrett, 1988; Nelson and Ward, 1981; Scalf et al., 1981; U.S. EPA, 1987b; U.S. EPA, 1986); as well as aquifer testing and analysis (Davis and DeWiest, 1966; Fetter, 1980; Ferris et al., 1962; Freeze and Cherry, 1979; Lohman, 1979; Kruseman and de Ridder, 1990; Stallman, 1971; Walton, 1962).

Characterizing aquifer properties and predicting the movement of contaminants in the subsurface is a more difficult undertaking in anisotropic, heterogeneous media, such as stratified drift, fracture-dominated, or karst terrains. The single most important parameter to accurately determine for characterization purposes is hydraulic conductivity, in both the lateral and vertical directions (Moltz et al., 1990). To this end, Moltz et al. (1990) describe and recommend several techniques to quantify variations in vertical hydraulic conductivity, including tracer, flowmeter, dilution, and multi-

level slug tests.

Complementary techniques that describe either borehole, secondary-porosity features (fractures, joints, and bedding planes) or borehole flow and direction include: the standard suite of borehole geophysical logs (Benson et al., 1983; Keys and MacCary, 1971); wellbore fluid-replacement logging (GZA GeoEnvironmental, Inc., 1991; Pedler et al., 1990; Pedler et al., 1989; Tsang et al., 1990, Vernon et al., 1993); acoustic televiewer logging (Vernon et al., 1993; Paillet et al., 1987); and dye tracing (Quinlan, 1989; Mull et al., 1988). To extrapolate and integrate borehole flow information at a more local or regional scale, additional techniques are recommended, such as: fracture-trace analysis (Lattman and Parizek, 1964; Ogiela and Moore, 1991; Parizek et al., 1990; Parizek, 1976), geologic and structure-control mapping, and surface geophysics (micro-gravity, electrical, magnetic, and seismic methods to locate subsurface fractures and faults). In karst regimes where conduit flow (flow through large openings and caves) occurs, fracture-trace analysis, dye tracing, and select geophysical methods are particularly helpful in defining the areal ground-water flow system.

Hydrogeologic characterization serves as the technical basis for refined WHPA delineation, flowpath identification, and ultimately, improved monitoring network and program design. It is important to remember that the characterization effort should not become an obstacle to completing the monitoring program (Todd et al., 1976). During the research monitoring phase, the informational base is incomplete, relative to the size of the resultant data base. Monitoring should be viewed as an iterative process, where data are gathered to continually augment and improve the existing informational base.

Refined Wellhead Protection Area Delineation--

Following detailed hydrogeologic characterization, existing delineation criteria and methods for the preliminary WHPA should be reviewed and improved (U.S. EPA, 1987a). Generally, increasing knowledge of the hydrogeologic situation enables a more quantitative and sophisticated delineation method to be employed, resulting in a more accurate WHPA, and often, a reduction in area. A reduction in the WHPA usually reduces management and regulatory oversight, yielding labor, time, and cost savings.

Flowpath Determination--

With more comprehensive coverage and understanding of the ground-water flow system, a flownet of the WHPA can be constructed using analytical (Freeze and Cherry, 1979; Fetter, 1980) or numerical (Anderson and Woessner, 1992; Water Science and Technology Board et al., 1990; Van der Heidje and Beljin, 1988) methods. A flownet is useful in selecting existing monitoring points or siting new ones to intersect flowpaths from known or potential contaminant sources. In addition, a more effective screen depth and interval can be determined by combining flownet interpretations with a knowledge of the fate and transport of the contaminant type (water-soluble versus insoluble contaminants) in the subsurface.

In non-Darcian flow conditions, such as fractured-bedrock or karst regimes, it may be beneficial to simulate contaminant pathways and travel times by dye tracing. Monitoring for pollutants in discrete-flow or conduit-flow systems is most reliably done at locations (such as wells, springs, caves, and streams) shown by dye tracing to be connected to sources, possibly under a variety of flow conditions (Quinlan, 1989). Conversely, ambient trend monitoring is best done in areas determined to be geochemically similar to the target area and where dye traces confirm no hydrologic

connections.

In conclusion, an accurate, optimally-sized WHPA in conjunction with flowpath assessment, improves the long-term monitoring program in the following ways:

- Effective selection and location of monitoring sites
- Effective selection of monitoring well screen depths
- More reliable monitoring data base
- Cost savings associated with a minimum number of well installations and ground-water samples
- Overall, more protective monitoring program

Phase III: Wellhead Protection Monitoring Program

The by-product of Phases I and II is a proposed wellhead protection monitoring program. Generally, the program is drafted as a plan to be implemented in stages, as labor and financial resources become available. If the technical groundwork for the monitoring program is done correctly, the plan can be drafted in two weeks to a month. A brief discussion of each program element (Table 1-4) is given as follows.

Introduction--

The plan should include a preface summarizing a brief history of the local WHPP inception and development. The purpose of the monitoring program should be integrated with respect to the management structure, designating the roles and duties of personnel responsible for implementing the monitoring program. A map depicting the location of prioritized sources should be included, accompanied by a list detailing the type of the source and suspected or known contaminants. A map depicting the WHPA (and protective zones, if applicable), as well as a description of the delineation criteria and method(s) should be given as background information to clarify objectives and the selection of monitoring sites.

Monitoring Objectives--

The plan should include general and specific objectives for ambient trend, source assessment, and early-warning detection monitoring. Each objective should be justified in terms of monitoring sites, parameters, frequencies, and quality assurance.

Monitoring Sites--

The plan should include a map depicting the location of existing and recommended monitoring sites for inclusion in the network. A formal identification system can be adopted to label monitoring sites, which may include: wells, piezometers, boreholes, springs, seeps, caves, excavations, sinkholes, and streams that can be shown to be hydraulically connected to the ground water. A minimum set of data elements should be used to identify the site, such as geographic location, political regime, source identifiers, and individual site characteristics (U.S. EPA, 1991c). Guidance and criteria for monitoring site selection based on specific objectives are given through example in Chapters 2 through 5 and in Appendix 1-A.

Site Design, Construction, and Installation--

The integrity of the design and construction of existing sites (wells, for example) should be

considered for inclusion in the monitoring network, to ensure that representative data can be obtained, such as water levels and water quality. New sites that require installation should be described in detail. Descriptions should include information on completion depth, open or screened interval, schematic design, and construction materials, as well as the method of installation, development, and testing. All monitoring site information should be stored in an automated data base for convenient and safe storage, update, and retrieval.

Monitoring Parameters and Frequencies--

Physical and chemical parameters to be monitored at select frequencies should be listed and technically justified. Specifically, monthly monitoring of water levels may be justified by cyclic water-table fluctuations. Similarly, quarterly monitoring of indicator parameters may be justified by compliance regulations. More frequent monitoring (biweekly) may be justified for statistical reasons.

Quality Assurance and Control Considerations--

Quality assurance refers to a system for ensuring that all information, data, and resulting decisions compiled under specific ground-water monitoring tasks are technically sound, statistically valid, and properly documented (Canter et al., 1987). Quality control provides the mechanisms to attain the quality assurance goals. Quality assurance and quality control apply to laboratory measurement data and field operations, such as monitoring site construction, sampling and decontamination procedures, sample custody, and equipment calibration. In general, quality assurance goals are driven by project objectives such as client concerns, compliance monitoring, court admissibility of data, statistical significance, and experimental design. Each monitoring program should formulate a minimum set of quality assurance and control objectives to match the objectives of the wellhead protection monitoring program. At a minimum, this should include laboratory standard operating procedures and a field operations plan.

Complimentary Contingency Planning--

In hydrogeologic regimes characterized by rapid ground-water velocities (.25 to 0.5 miles per hour) and travel times (hours to days in some fractured-bedrock and karst aquifers), early-warning detection monitoring is impractical and unachievable. Contaminants pass through the system essentially unaltered. In situations such as this, a spill-response plan (Quinlan, 1986) is recommended to eliminate or reduce the amount of contamination entering highly-vulnerable aquifers directly from the surface. Situational monitoring is detailed by Pfannkuch (1982) in response to accidental or unanticipated spills. Finally, contingency planning requires that a new water-development area be sited in the event that the existing water supply is rapidly diminished or contaminated.

STEP-WISE APPROACH FOR THE DESIGN OF A WELLHEAD PROTECTION MONITORING PROGRAM

In conclusion, a strategy for the design of a monitoring program for wellhead and springhead protection is presented in step-wise format (Figure 1-10). Used as a guide, this strategy may require modification to accommodate the spectrum of flow systems in many aquifers: diffuse- to conduit-flow. It synthesizes several excellent technical approaches for ground-water monitoring program design (Canter et al., 1987; Pfannkuch, 1982; Quinlan, 1989; and Todd et al., 1986).

(1) Identify, prioritize, and characterize all existing and potential contaminant sources within and in proximity to the wellhead or springhead protection area.

(2) Compile and organize all existing baseline data and ancillary information.

(3) Identify deficiencies in the existing informational base.

(4) Identify additional data acquisition and processing needs to enable system characterization.

(5) Establish monitoring objectives to improve the informational base and fill the data gaps.

(6) Determine physical, chemical, and biological parameters to be monitored.

(7) Identify existing sites for background, pollutant, and water-quality monitoring to meet specific objectives.

(8) Determine optimal sampling frequencies for monitoring parameters considering spatial, temporal, and quality assurance and control considerations.

(9) Assess the need for new monitoring points.

(10) Establish new monitoring sites.

(11) Implement the monitoring program.

(12) Review and interpret monitoring results.

(13) Integrate monitoring results to revise source, water quality, hydrogeology, and flowpath transport assessments.

(14) Modify monitoring objectives, network design, and the program approach, appropriately.

(15) Iterate the monitoring process.

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APPENDIX 1-A

STRATEGY FOR DELINEATION AND DETECTION MONITORING OF THE FULBRIGHT SPRINGHEAD PROTECTION AREA, SPRINGFIELD, MISSOURI

STRATEGY FOR DELINEATION AND DETECTION MONITORING OF THE FULBRIGHT SPRINGHEAD PROTECTION AREA, SPRINGFIELD, MISSOURI

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ABSTRACT

Reliable monitoring for contaminants within the Fulbright springhead protection area, which is situated in a mature karst terrane, is based on identification and characterization of prioritized source areas, complete delineation of the recharge area, and flow-route mapping of pollutants during variable recharge events. A combination of hydrologic, geochemical, and geologic techniques are proposed in this study to better delineate the protection area, to predict flow transit times, and to predict pulse characteristics within the recharge system to develop an effective detection monitoring system. Successful techniques include lineament analysis; spring, sinkhole, and cave surveying; geologic and potentiometric-surface mapping; dye tracing; flow-discharge quantification; and surface- and ground-water quality assessment. Potential contaminant source burdens to the upper and lower aquifers include agricultural chemicals and pesticides; septic system leachate; uncontrolled discharge to abandoned wells; storm-water runoff to sinkholes and losing streams; spills along transportation routes; and leakage from underground storage tanks, landfills, and industrial facilities. The strategy presented in this paper may be generally applicable, with necessary modifications, to other karst terranes for the successful implementation of Wellhead Protection Programs.

INTRODUCTION

The 1986 Amendments to the Safe Drinking Water Act established the Wellhead Protection Program (WHPP) to protect ground water that contributes to public water supply wells, wellfields, and springs. We focus here on one important technical and management element of WHPP implementation: ground-water monitoring. Ground-water monitoring of chemical and physical parameters may enhance source characterization, wellhead protection area delineation, and new water supply evaluation. A wellhead protection area (WHPA) is generally defined as the surface and subsurface area surrounding a well or wellfield, supplying a public water system, through which contaminants are reasonably likely to move toward and reach such a well or wellfield. Moreover, ground water is monitored to assess source-control measures, to monitor compliance with drinking water standards, and to provide advance warning of contaminants in ground water.

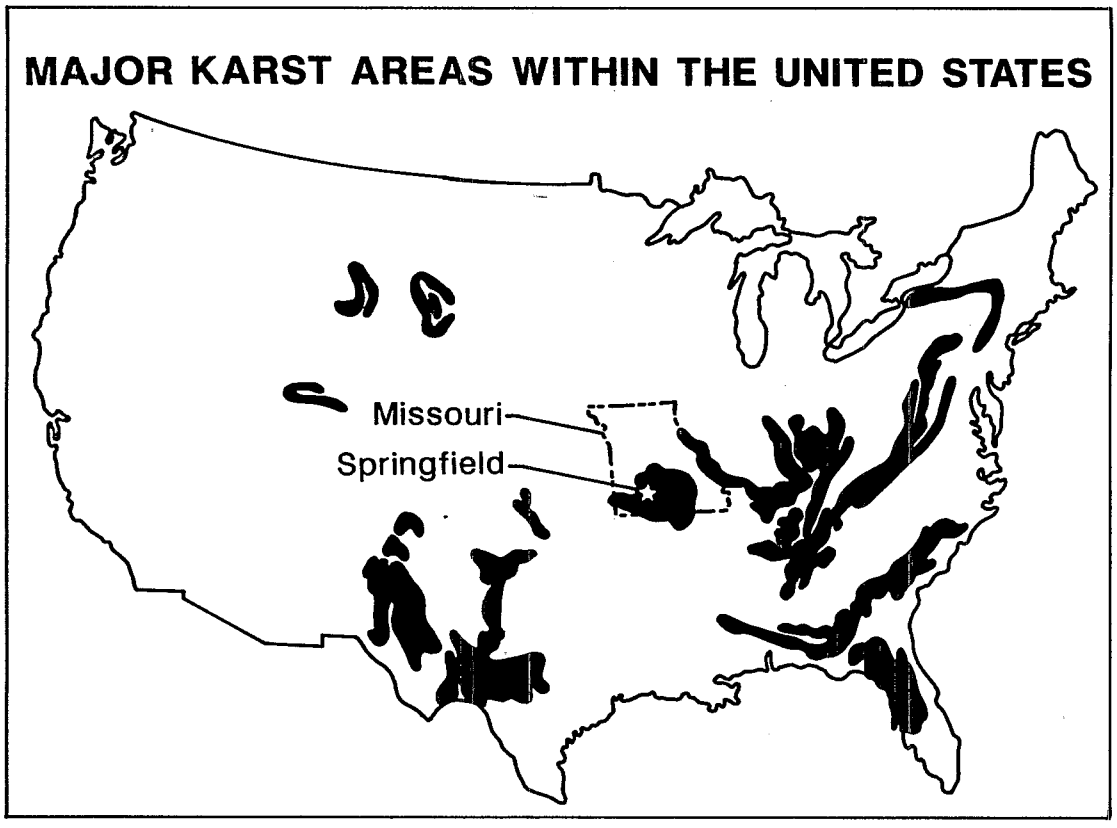

Figure 1. Regional location of Springfield, Missouri, with respect to major karst areas in the United States (After Mull et al., 1988).

This paper presents a strategy for the design of a monitoring program for the Fulbright springhead protection area (SHPA) in Springfield, Missouri. A community with a population of approximately 150,000, Springfield is located within the karst region of the Ozark Highlands of the central United States (Figure 1). The City has utilized the Spring as a significant source of its drinking water for over 80 years. Fulbright Spring contributes from 20 to 25 percent to the City's annual water supply. McDaniel Lake, Fellows Lake, and the James River, as shown in Figure 2, each contribute approximately 25 percent to the annual total. The technical approach described in this paper is being implemented by the City Utilities to protect the Fulbright Spring recharge area for present and future water supply needs.

In recent years, increasing emphasis has been placed on the complex and unusual character of karst hydrology. Karst terranes are formed by the dissolution of carbonate rock such as limestone, dolomite, and gypsum. Characteristic topographic features include karst windows, sinkholes, losing streams, caves, and springs. Some of these features are illustrated in Figure 3. Karst ground waters are highly vulnerable to contamination owing to thin soils in

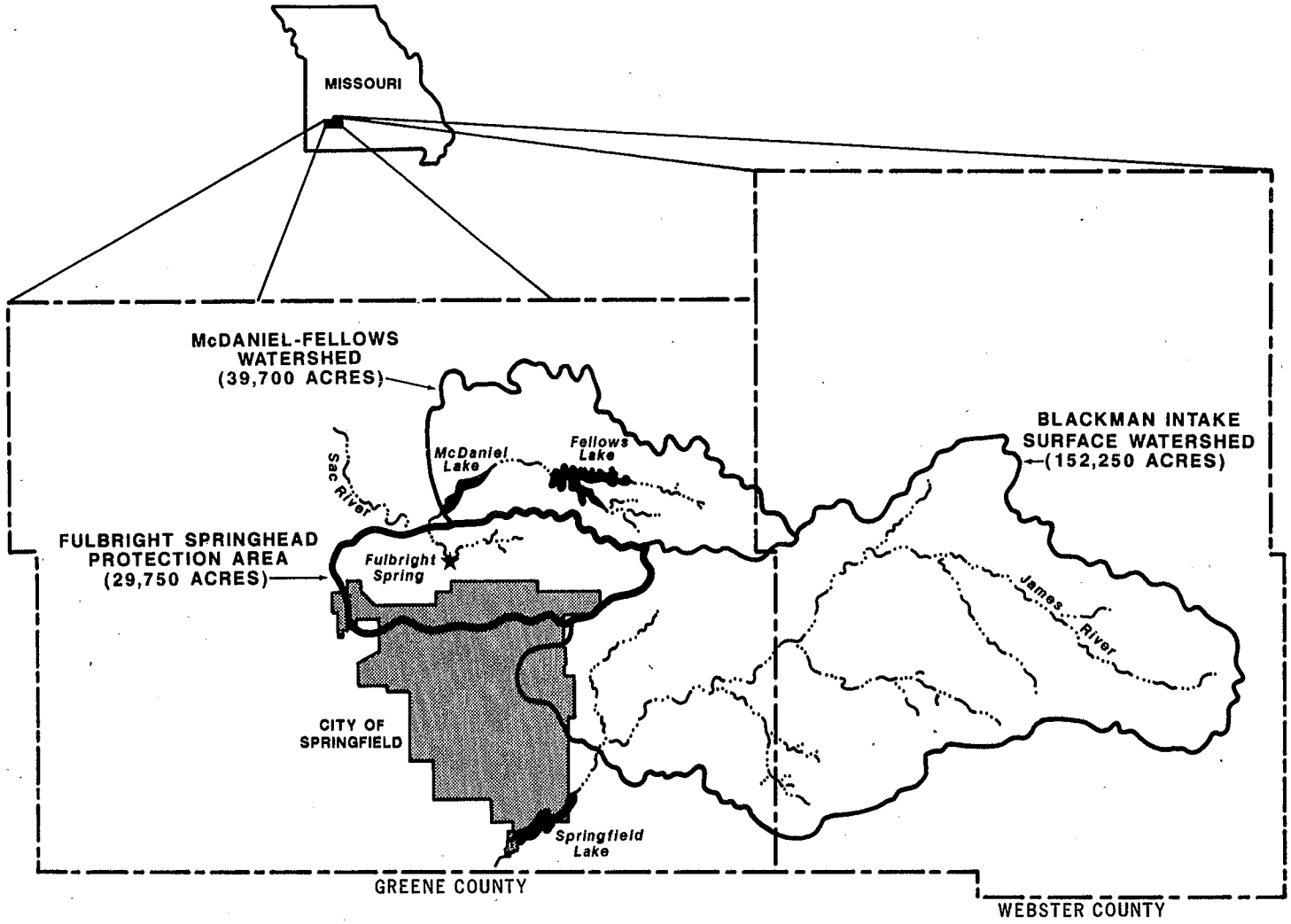

Figure 2. Location of Fulbright Spring, the springhead protection area, and adjacent watersheds near Springfield, Missouri.

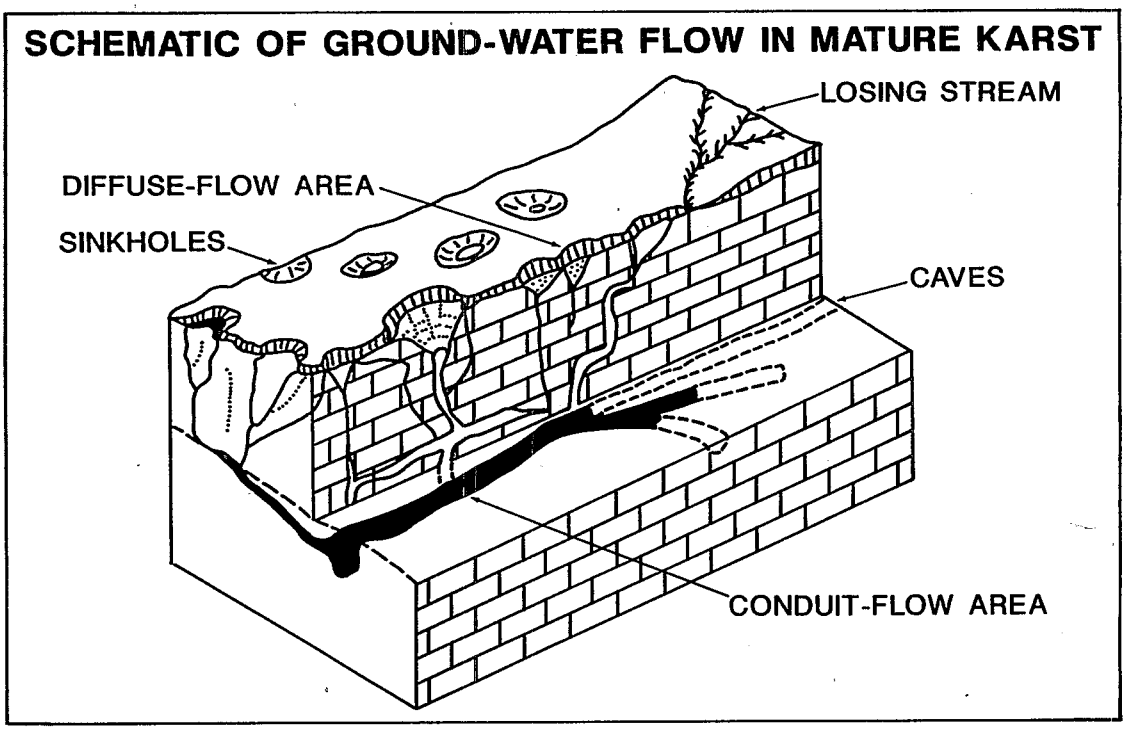

Figure 3. Schematic of some characteristic features and flow types in karst regimes (After Mull et al., 1988).

upland areas; rapid ground-water flow rates; and numerous, discrete recharge points. This vulnerability has underscored the necessity for preventative contamination measures, such as wellhead and springhead protection.

WELLHEAD PROTECTION MONITORING CONSIDERATIONS

Common to all effective monitoring systems is accurate conceptualization of the site and its environs. Elements of the conceptual model include the source, pathway, and receptor. During the initial stages of WHPP implementation, sources are identified, characterized, and prioritized. Concurrently, WHPAs are delineated based on site-specific ground-water flow and contaminant-transport assessment. The receptor corresponds to a public water supply well, wellfield, or spring. By integrating each component of the environmental system to form an accurate conceptual model, more efficient monitoring programs can be developed.

Monitoring helps to document ground-water system trends. Properly interpreted, trend analysis can improve hydrogeologic characterization and enhance protection. Three types of ground-water monitoring apply to WHPP development:

ambient-trend, source-assessment, and early-warning detection monitoring (Canter et al., 1987). Ambient-trend monitoring addresses temporal and spatial trends in the overall physical and chemical quality of a ground-water basin or subarea. Source monitoring assesses the existing or potential impacts on the physical or chemical ground-water system from a proposed, active, or abandoned contaminant source. Early-warning detection monitoring is conducted at a calculated distance from the wellhead, based on known travel times, to trigger a contingency response to prevent public exposure to pollutants. These types of monitoring are planned to measure or detect contaminants in aquifers, not as preventative or remedial measures (U.S. EPA, 1989).

SPRINGFIELD CASE STUDY

Hydrogeologic Setting

Springfield is situated on the Springfield Plateau, an erosional surface of the Mississippian Limestone in southwest Missouri (Harvey, 1980; Pendergrass, 1983; Thompson, 1986). The upland cap of the Plateau is composed of a very pure limestone, the Burlington-Keokuk Formation, which is highly solutioned. Locally, the upper aquifer exists within the Burlington-Keokuk, Elsey, and Pierson limestone formations, as depicted in Figure 4. It ranges in thickness from 0 to 300 feet in Greene County. The water table is approximately 100 feet deep in recharge areas and intersects the land surface at stream discharge areas (Imes, 1989). Above the water table, numerous dissolution channels exist where flow ranges from diffuse to conduit. Major springs, such as Fulbright Spring, issue from the base of the Burlington-Keokuk within the upper aquifer. Fulbright Spring discharges from 2 to 8 million gallons per day (gpd), whereas, smaller yields of 30,000 gpd are typical for upper aquifer wells.

Underlying the upper aquifer is the Northview Formation (5 to 80 feet thick in Greene County), a low-permeability shale and siltstone that separates the upper and lower aquifers (Figure 4). In areas beneath Springfield, the Northview Formation thins to 10 feet (Imes, 1989) and is potentially more permeable along fracture zones. The lower Ozark aquifer consists of over 1,000 feet of dolomite and sandstone formations capable of yielding up to 30 million gpd. Industrial and municipal wells completed within the lower aquifer are at depths of 1,200 to 1,700 feet.

Predevelopment regional ground-water flow gradients within the upper and lower aquifers were toward the west and southwest. Present ground-water flow directions are significantly influenced by areas of local pumpage. A downward hydraulic gradient exists between the upper and lower aquifers directly beneath the City owing to pumpage. Potentiometric levels are as much as 300 feet lower in the lower aquifer. The upper aquifer significantly recharges the lower aquifer (Harvey, 1980; Imes, 1989). Consequently, ground-water monitoring of the Fulbright springhead recharge area may help to protect the water quality of the underlying Ozark aquifer.

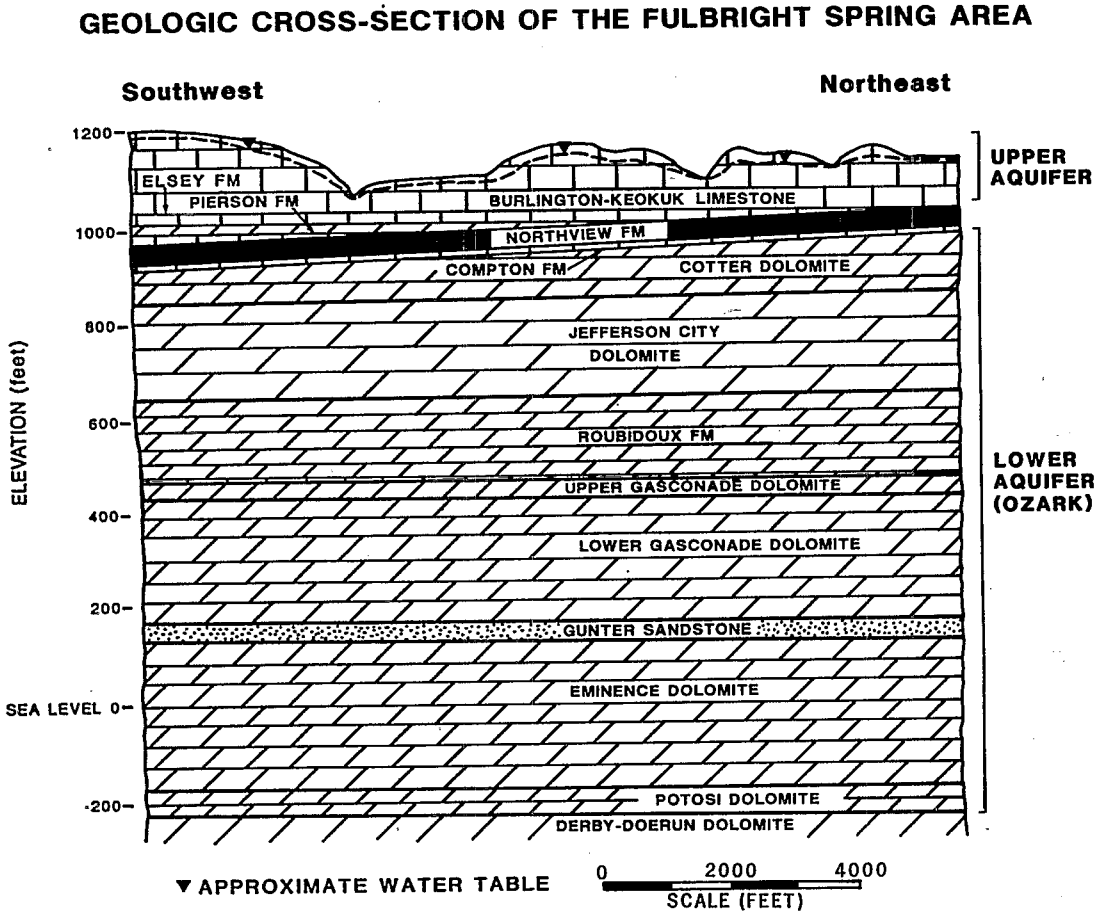

Figure 4. Typified hydrogeologic cross-section of the Fulbright springhead protection area (SCS Engineers, 1988).

Fulbright Springhead Protection Area

The inferred Fulbright SHPA is depicted in Figures 2 and 5. The SHPA boundary corresponds to the surface drainage divides of contributing watersheds and includes sinkhole plains in inferred recharge areas. The boundary is approximate owing to the complex surface-subsurface drainage system resulting from karst development and structural controls such as faults, joints, and lineaments. Ground-water flow to springs in the area often crosses topographic basin divides. Numerous dye-tracing studies and additional mapping will be required to accurately map the SHPA and to verify contributing local, intermediate, and regional flow systems.

The Fulbright SHPA consists of seven separate watersheds, as shown in Figure 5. Research in this study focuses on detailed hydrogeologic characterization of the

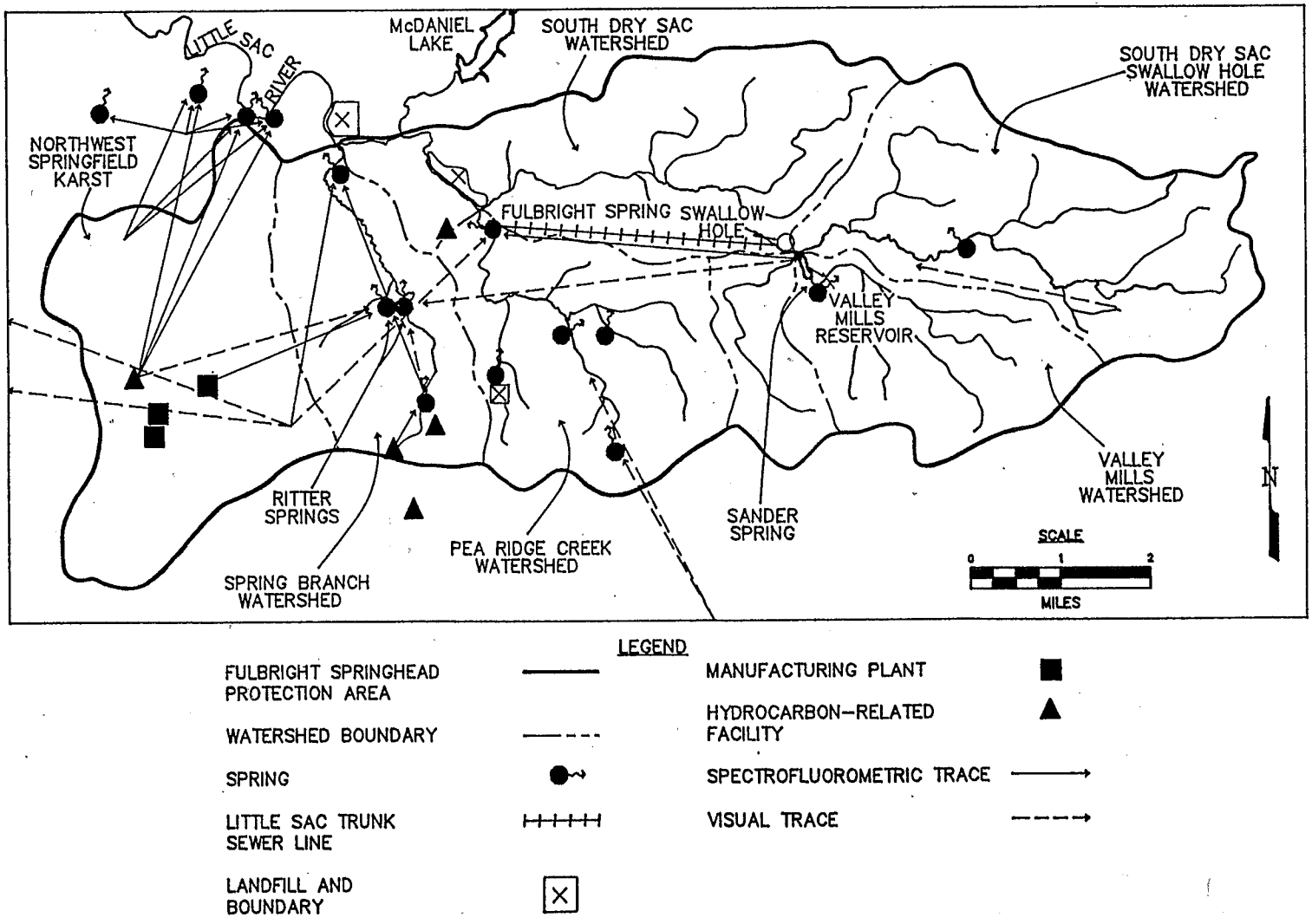

Figure 5. The Fulbright springhead protection area in northern Springfield, Missouri; significant hydrogeologic features; potential contaminant point sources; and flow directions from dye-trace results.

Fulbright Spring and Valley Mills connection and the western portion of the SHPA. Dye traces have verified interconnection between the Fulbright Spring and Valley Mills Reservoir, indicating travel times as short as three hours. Both the Valley Mills and South Dry Sac Swallow Hole Watersheds lose most of their surface flows to the Swallow Hole. Apparently, most of this water discharges at Fulbright and Ritter Springs. Hydraulic connection between Fulbright Spring and these two eastern watersheds is partly attributable to fault-controlled losing stream segments.

The Northwest Springfield Karst, as well as the Spring Branch and Pea Ridge Creek Watersheds, constitute the western portion of the protection area, as shown in Figure 5. The Northwest Springfield Karst is characterized by intense sinkhole development and near absence of surface streams. The Spring Branch and Pea Ridge Creek Watersheds define the approximate southern boundary of the SHPA.

Fulbright SHPA Monitoring Objectives

This study focuses on refined delineation and monitoring system design in two regions of the SHPA: (1) the Fulbright Spring and Valley Mills connection and (2) the central and western watersheds. Rapid development will follow installation of the Little Sac Trunk sewer line (Figure 5). Hydrogeologic characterization is required here to support land use management planning necessitated by urbanization. An early-warning detection monitoring system would probably not be appropriate owing to rapid travel times (three hours to one day) and apparent velocities (up to 0.5 mile per hour) (Pendergrass, 1983). However, source-assessment monitoring, in combination with development screening or source prohibition, may prove to be a practical management strategy for ground-water protection. Ambient-trend monitoring is necessary in this area to delineate the distributary drainage network and quantify the flow system at variable discharges. Through better evaluation of flow times and pulse characteristics, contaminant-specific monitoring may be achieved.

In the western watersheds, the need for and the feasibility of early-warning and source-assessment monitoring will be evaluated. Ambient-trend monitoring will be implemented. Early-warning detection monitoring may be relevant if ground-water velocities range from days to weeks. Coordinated with a spill-response plan (Quinlan, 1986), detection monitoring may be an effective management alternative for determining compliance and triggering emergency cleanup. Assessment monitoring of prioritized sources must be evaluated with regard to the adequacy of the site data and the degree of the contamination threat.

Source Assessment

Source assessment is being conducted as a first step in the design of an effective protection monitoring system for the Fulbright SHPA. All potential point and nonpoint sources (NPSs) were inventoried and prioritized. Characterization of prioritized sources will parallel design of the monitoring system.

As is the case in most karst aquifers, bacteria and nitrates are the most common, potential NPS pollutants in the study area. Agricultural runoff and septic system leachate contribute significantly to their presence in ground water. The dominant land use practice within the Fulbright SHPA is agriculture. Nutrient loading, primarily from animal wastes, to McDaniel Watershed has accelerated lake eutrophication and threatens ground water. Dye-trace studies conducted by Aley and Thompson (1981) have shown that many springs in the county receive effluent from septic systems.

As urbanization encroaches on the Fulbright Spring recharge area, deterioration of water quality is anticipated (Pendergrass, 1983). Three water quality problems are associated with land development in the Springfield area: (1) sewage effluent; (2) urban runoff; and (3) leaks, spills, and discharges from commercial and industrial facilities (Aley and Thompson, 1981). Expansion of the sewer system into rural areas may help prevent some NPS contamination.

Sinkholes, losing streams, and abandoned wells provide direct access to the ground-water system. One-half mile west of Fulbright Spring, a number of private water supply wells were contaminated by gasoline constituents from an unknown source. Illegal disposal into one of numerous sinkholes and an abandoned well at a gasoline station site are suspected sources. Surface pathways facilitate uncontrolled storm-water runoff to the upper and, possibly, lower aquifers. Storm-water runoff typically contains higher concentrations of metals, agricultural nutrients and pesticides, bacteria, and petrochemicals (Jenkins, 1988).

Potential point sources within and in proximity to the SHPA include landfills, manufacturing plants, and hydrocarbon-related facilities, as shown in Figure 5. Two sanitary landfills, where industrial wastes containing cyanides, heavy metals, and acids were disposed of, are located in the northern portion of the SHPA. An industrial landfill, where foundry sand was disposed of, is located within the Pea Ridge Creek Watershed. Industrial and electronics equipment are the predominant products of the manufacturing facilities located within the Northwest Springfield Karst. Hydrocarbon-related facilities include a creosote-based wood treatment plant, railroad yard, airport, asphalt plant, petroleum products distribution center, and numerous underground storage tanks (USTs).

Springhead Protection Boundary and Flow-Path Delineation

Numerous hydrogeologic studies have been conducted to characterize the shallow aquifer system around Springfield. The most significant conclusions from these studies are that "...(1) fluid flow is rapid, unrestricted, and capable of substantial lateral transport, and (2) contaminants pass through the system essentially unchanged" (Pendergrass, 1983). For karst terranes such as this, delineation can only be done by dye tracing, guided by inference from accurate potentiometric mapping (Mull et al., 1988).

Figure 5 depicts a compilation of spectrofluorometric and visual dye-trace results in proximity to the Fulbright SHPA. Interconnection to Fulbright Spring is evident from the Northwest Springfield Karst. Pathways may exist from the Spring Branch and Valley Mills Watersheds to the Spring. Recharge from the Pea Ridge Creek Watershed is poorly understood, although losing stream sections are prevalent in the upper watershed. Visually confirmed traces, originating from watersheds to the south of the SHPA, suggest that recharge also occurs from intermediate and, possibly, regional flow systems. Dye traces from the Northwest Springfield Karst indicate flow components to the north and west beyond the inferred SHPA boundary.

Inspection of Figure 5 leads to several data deficiencies that will be addressed in this study. Local flow systems in the western and central watersheds, as well as the Fulbright Spring and Valley Mills connection, must be further delineated and quantified. Through dye tracing and volumetric flow analyses at select locations (springs, sinkholes, caves, and wells) and at variable flow conditions, characteristics of the karst system can be determined, such as (Mull et al., 1988; Quinlan, 1989):

- Areas of diffuse versus conduit flow
- Flow paths, apparent velocities, and residence times
- Point-to-point connections to aid in source assessment
- Qualitative indications of contaminant fate and transport along various flow paths
- Local recharge boundaries for protection area delineation

To better characterize the Fulbright Spring and Valley Mills connection, two physical monitoring programs will be initiated: (1) flow discharges at Fulbright Spring, Sander Spring, and Valley Mills Reservoir and (2) water levels in a lower aquifer well and shallow wells along the Little Sac Trunk sewer line (Figure 5). Physical monitoring will focus on flood-pulse evaluation to establish low and intermediate flow conditions. Complementary qualitative and quantitative dye traces will be designed for recovery at the monitoring sites. A similar technical approach will be implemented for the western and central watersheds of the SHPA. However, dye-trace studies will focus on determining potential point-source connections to Fulbright Spring, as well as local flow-path assessment.

Dye-tracing results, in conjunction with potentiometric mapping, will be used to modify and improve the present conceptual model of flow within the upper aquifer. While the two techniques are complementary, accurate dye-trace results tend to be less ambiguous than interpretations based on potentiometric mapping (Quinlan, 1989). Other supporting information to refine development of the conceptual flow model include:

- Geologic unit and structural-control mapping
- Lineament analysis, both photo interpretation and ground verification
- Spring, cave, and sinkhole surveying
- Surface- and ground-water quality characterization

An iterative process of data integration and corroboration will be implemented to generate more accurate interpretations and effective monitoring systems.

Optimal location of water quality monitoring sites will be dependent on identification of diffuse- versus conduit-flow areas and distributary-flow systems. In areas of diffuse flow, standard methods and computer model evaluations for site selection or well placement may be applicable. However, monitoring for pollutants in conduit-flow systems will be done at locations shown by dye tracing to be connected to sources under base- and flood-flow conditions (Quinlan, 1989). Conversely, background monitoring will be done in areas determined to be geochemically similar to the target area and where dye-traces confirm no hydrologic connections. Knowledge of distributaries is important due to the influence that the system can have on the areal and temporal dispersion of contaminants at various recharge events (Quinlan, 1989).

Ground-Water Quality Assessment

Bacteriological data indicate that water quality has improved somewhat in many parts of the recharge area corresponding to installation of a centralized sewer system. Ozark aquifer ground water has never exhibited microbiological degradation. Analyses of the spring water for VOCs, napthalene, and cyanide indicate nondetectable to very minor concentrations. Other key indicators of upper aquifer water quality are nitrate and turbidity levels.

A major objective of this study is to document and analyze present ground-water quality trends to develop effective monitoring protection strategies. The storm-related variability in water quality at monitoring sites will be measured much more frequently than at standard intervals (for example, quarterly or semi-annually) to ensure accurate characterization (Quinlan, 1989). The maximum value of some parameters coincides with peak discharges; others culminate several days after the maximum storm event (Quinlan 1989). Consequently, sampling at 1- to 6-hour intervals may be required prior to and during peak discharge, while 4- to 24-hour intervals may be appropriate later.

An automatic sampler will be installed at Fulbright Spring to monitor the storm-related variability of most inorganic and select organic compounds. Automatic sampling will only yield qualitative estimates of VOC contamination. Continuous determination of dissolved oxygen, pH, specific conductance, and temperature will be attempted with dedicated devices. Other sites within the SHPA will be similarly monitored, but less frequently. Hydrochemical data will be analyzed to categorize water types and to establish select spatial and temporal water quality trends. Indicator parameters will be determined for early-warning detection and source-assessment monitoring.

Monitoring Parameters

The variety of potential contamination sources within the Fulbright Spring recharge area suggest the need for a broad range of monitoring parameters. Chlorinated solvents, heavy metals, volatile fuel hydrocarbons, and semi-volatile polynuclear aromatic hydrocarbons were detected in the ground water collected from springs and wells in the vicinity of some of the sources. This range of contaminants can be associated with the different types of potential sources found in the area. The chlorinated solvents and heavy metals are the predominant wastes from machining, plating, and painting activities at the manufacturing facilities. Polynuclear aromatic hydrocarbons are found in wood-treating and asphalt operations, while the lighter fuel hydrocarbons are associated with the use of gasoline and oils.

Chlorinated solvents, hydrocarbons, and heavy metals have been found in the ground-water down gradient of the municipal landfills. These compounds are thought to be related to the industrial waste disposed of at the landfills. In addition, general chemistry parameters, such as specific conductivity, chloride, sulfate, and nitrate can commonly be used to detect leakage from sanitary landfills. The possibility of nonpoint contamination from septic systems and agricultural sources suggests the need for nutrient and pesticide analyses.

In summary, contaminant detection monitoring may require complete analyses for organic priority pollutants (volatile, semi-volatile, and pesticide compounds), heavy metals, and general chemistry parameters. Comprehensive monitoring is warranted to cover the range of potential contaminant sources within the SHPA. Periodic analyses must be done at Fulbright Spring and other critical monitoring locations during the initial phases of this wellhead protection effort to determine background levels for these compounds.

CONCLUSION

This paper combines a strategy for the design of a monitoring system for wellhead and springhead protection in karst terranes with a case study application in Springfield, Missouri. Determination of target monitoring parameters requires comprehensive identification, prioritization, and characterization of potential point and nonpoint sources. Correct delineation in karst terranes can only be done by dye tracing, guided by inference from accurate potentiometric mapping (Mull et al., 1988). Complementary, and necessary, investigations include geologic and structural-control mapping; lineament analysis; spring, cave, and sinkhole mapping; and water quality characterization.

Ambient-trend, source-assessment, and early-warning detection monitoring objectives should be established to clearly support technical and management programs (Canter et al., 1987). Optimal and effective monitoring network design is based on accurate characterization of the karst flow system. This is accomplished through dye tracing and volumetric flow analysis at select locations and at variable flow rates (Quinlan, 1989). The storm variability in water quality at

monitoring sites should be measured as a basis for determining the areal and temporal dispersion of contaminants during recharge events.

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NOTICE

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CHAPTER 2

WELLHEAD PROTECTION PROGRAM AND MONITORING SYSTEM DESIGN STEVENS POINT, WISCONSIN

by

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NOTICE

This report is the result of research supported by the U.S. Environmental Protection Agency's Environmental Monitoring Systems Laboratory in Las Vegas, Nevada, as part of its efforts to provide technical assistance to state, tribal, and local governments on the implementation of the Wellhead Protection Program. The specific methods and approaches contained in this document have undergone peer review but do not constitute official Agency endorsement or policy recommendations. The Office of Research and Development provides this information to help solve complex technical problems related to refined delineation and ground-water monitoring of wellhead protection areas in various hydrogeologic settings. Further assistance is available from the Environmental Monitoring Systems Laboratory in Las Vegas, Nevada, from the Office of Ground Water and Drinking Water in Washington, D.C., and from the ground-water offices in the ten U.S. Environmental Protection Agency regions.

ABSTRACT

The Stevens Point, Wisconsin, Wellhead Protection Program is an ongoing effort by the city's Water Department to ensure a high-quality, economical, and lasting municipal water supply. The city obtains all of its water from a shallow, unconfined, sand and gravel aquifer that is highly susceptible to contamination. The major threats to ground-water quality are from urban, suburban, and agricultural land uses that create both point and nonpoint sources of contamination. Typical ground-water contaminants include nitrates from septic systems, fertilizers, and animal wastes; volatile organic compounds from leaking underground storage tanks and spills; and naturally occurring iron, manganese, and radioactivity. This study encompasses refined delineation of the Wellhead Protection Area, hydrogeologic mapping, contaminant source assessment, and monitoring system design. Completion of these tasks will serve as the basis for continued research in wellhead protection in order to increase the understanding of the hydrogeologic system and to protect the ground-water resource.

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ABBREVIATIONS, SYMBOLS, AND CONVERSION FACTORS

ABBREVIATIONS

amsl	above mean sea level
Alk	alkalinity
cfs	cubic foot per second
COD	chemical oxygen demand
Cond	specific conductivity
CW	city well
d	day
DEET	N,N-diethyl-3-methylbenzamide
DNAPL	dense, nonaqueous-phase liquid
DO	dissolved oxygen
DOW	depth of well
EMSL-LV	Environmental Monitoring Systems Laboratory, Las Vegas, Nevada
EPA	U.S. Environmental Protection Agency
ft	foot
ft/d	foot per day
ft^2/d	square foot per day
ft/s	foot per second
ft^2/s	square foot per second
gal	gallon
gpd	gallon per day
Hard	hardness
in	inch
LESAT	Lockheed Environmental Systems & Technologies Company
l	liter
min	minute
µg	microgram
µg/l	microgram per liter
µS/cm	microSiemen per centimeter
mg	milligram
Mgd	million gallon per day
mg/l	milligram per liter
MP Elev	measuring point elevation
MW	monitoring well
N/A	not applicable
ND	not detected
NSA	nonsewered area
PAL	preventive action limit

PCB	polychlorinated biphenyls
Pest	pesticide
ppm	part per million
PVC	polyvinyl chloride
SCH	schedule
sec	second
SVOA	semi-volatile organic analytes
SWL Elev	static water level elevation
TOT	time of travel
TPH	total petroleum hydrocarbon
UST	underground storage tank
USGS	U. S. Geological Survey
VOC	volatile organic compound
WGNHS	Wisconsin Geological and Natural History Survey
WHPA	wellhead protection area
WHPP	wellhead protection program
ZOI	zone of influence

SYMBOLS

Ca	calcium
Cl	chloride
Fe	iron
Mn	manganese
Na	sodium
N	nitrogen
NO_2	nitrite
NO_3	nitrate
P	phosphorus
Zn	zinc

b	aquifer thickness
d	thickness of river or lake bed
h	hydraulic head
K	hydraulic conductivity
K_v	vertical hydraulic conductivity
L	screen length
L_r	leakage factor
Q	well discharge
R	well radius
R_c	correction factor
r	distance between wells
S	specific yield for an unconfined aquifer
s	drawdown
T	transmissivity
t	time
t_0	specified time during aquifer testing

CONVERSION FACTORS

Multiply	By	To Obtain
acre-foot	1230	cubic meter
cubic foot per second	0.0283	cubic meter per second
foot	0.3048	meter
foot per second	0.3048	meter per second
gallon	3.785	liter
gallon	0.134	cubic foot
gallon	0.00379	cubic meter
gallon per day	0.000003528	cubic foot per second
gallon per day per foot	0.000207	square meter per day
gallon per day per square foot	0.0408	meter per day
inch	0.0254	meter
inch per year	25.4	millimeter per year
mile	1.609	kilometer
million gallons per day	2.629	cubic meter per minute
square foot per minute	0.0929	square meter per minute
square foot per second	0.0929	square meter per second
square mile	2.59	square kilometer

$$°F = \frac{9}{5}(°C) + 32$$

ACKNOWLEDGMENTS

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BACKGROUND FOR THE STEVENS POINT CASE STUDY

INTRODUCTION

The Stevens Point, Wisconsin, Wellhead Protection Program (WHPP) is a continuation of the city's efforts to maintain their ground-water supply and to protect their water quality. The purpose of the local WHPP is to design and implement a protection plan for the city's production wells. The four major components of the Stevens Point WHPP are a ground-water monitoring program, land-use restrictions, an education program, and a contingency plan.

This investigation focuses on the design of a monitoring program for a shallow, unconfined, highly permeable aquifer with point and nonpoint sources of ground-water contamination from urban and agricultural activities. The purpose of the ground-water monitoring system is to detect contamination before it reaches the city water supply. The monitoring system design is based on refined delineation of the Wellhead Protection Area (WHPA) and the results of the contaminant source assessment. The land-use restrictions and education program components of the WHPP may limit or modify practices that endanger the water supply, thus reducing the probability of contamination. A contingency plan is necessary to ensure that the city's water-supply needs can be met if the protection plan fails.

The objectives of this study are to design and document the elements of each of the four major components of the Stevens Point WHPP. Activities conducted in support of the project's objectives include:

- Compiling existing data
- Identifying limitations of the existing data base
- Contaminant source assessment
- Prioritization of sources
- Hydrogeologic mapping
- Installation of additional monitoring wells
- Aquifer testing
- Ground-water sampling for chemical analysis
- Flow and transport modeling
- Refined delineation of the WHPA
- Evaluation of management options

The need for a comprehensive WHPP in Stevens Point is warranted by the aquifer's sole-source classification, its vulnerability to contamination, and a history of ground-water contamination proximal to the city's well fields.

LOCATION

Stevens Point is located in central Wisconsin near the center of Portage County (Figure 2-1). Stevens Point is the largest city in the county, with a population of approximately 23,000. The city is bordered by the town of Hull to the north and east, by the town of Plover and the village of Whiting to the south, and by the towns of Linwood and Carson to the west (Figure 2-1).

In Portage County, more than 90% of all water used is obtained from ground-water supplies (Portage County Planning Department, 1988). Agricultural irrigation is the largest ground-water consumer, requiring approximately 10 trillion gallons of water per year or 67% of the total estimated ground-water withdrawals (based on the water budget for 1979). Ground water is used for all domestic and municipal drinking water supplies in the county.

Several ground-water contamination problems are common in the region, including high levels of nitrates, pesticides, and volatile organic compounds (VOCs). The village of Whiting has purchased water from the city of Stevens Point since 1979 when nitrate levels in the village wells exceeded 10 parts per million (ppm). Aldicarb, a pesticide used to control the potato beetle, was detected in 53 wells in Portage County in 1986 (Portage County, 1986). Restrictions on the application of this pesticide were enacted by the Wisconsin state legislature in March 1986. Recently the State Board of Agriculture, Trade, and Consumer Protection proposed restrictions on the application of another pesticide, atrazine, which has also been detected in drinking water from private wells in Portage County. Local contamination problems include (1) several private wells in the town of Hull contaminated with gasoline, and (2) ground-water contaminated with aviation fuel at the Stevens Point airport.

WELLHEAD PROTECTION PROGRAM OVERVIEW

Although the term "wellhead protection" was not yet conceived, the concept of wellhead protection has been applied by the city of Stevens Point since the first municipal supply well was installed in 1923. The original Iverson wellfield consisted of three wells installed between 1923 and 1938. The wells were installed near the Plover River to ensure an adequate supply of drinking water. Although the primary concern was water quantity, water quality and its preservation were also considered a priority in constructing these wells. The property surrounding the wells was designated as park land to reduce the possibility of water-supply contamination, and the Iverson wells were protected by permanent structures. Wellhead protection practices remained essentially the same for the next 58 years. In the 1960s, two wells at the Iverson wellfield and four wells at the new Airport wellfield were added to the city's water-supply system (Figure 2-2). These six wells comprise the city's existing well system.

When the site for the Airport wellfield was chosen in 1965, the city purchased property surrounding the wellfield property to provide a closely supervised buffer zone to minimize ground-water contamination (Hickok and Associates, 1965). Ground-water quality of the production wells was monitored regularly. Monitoring wells were not used until 1981. Hickok and Associates was contracted to study the potential for contamination of the city's Airport wellfield in the event of a leak in one of the airport's large underground fuel storage tanks. The Hickok and Associates (1981) study

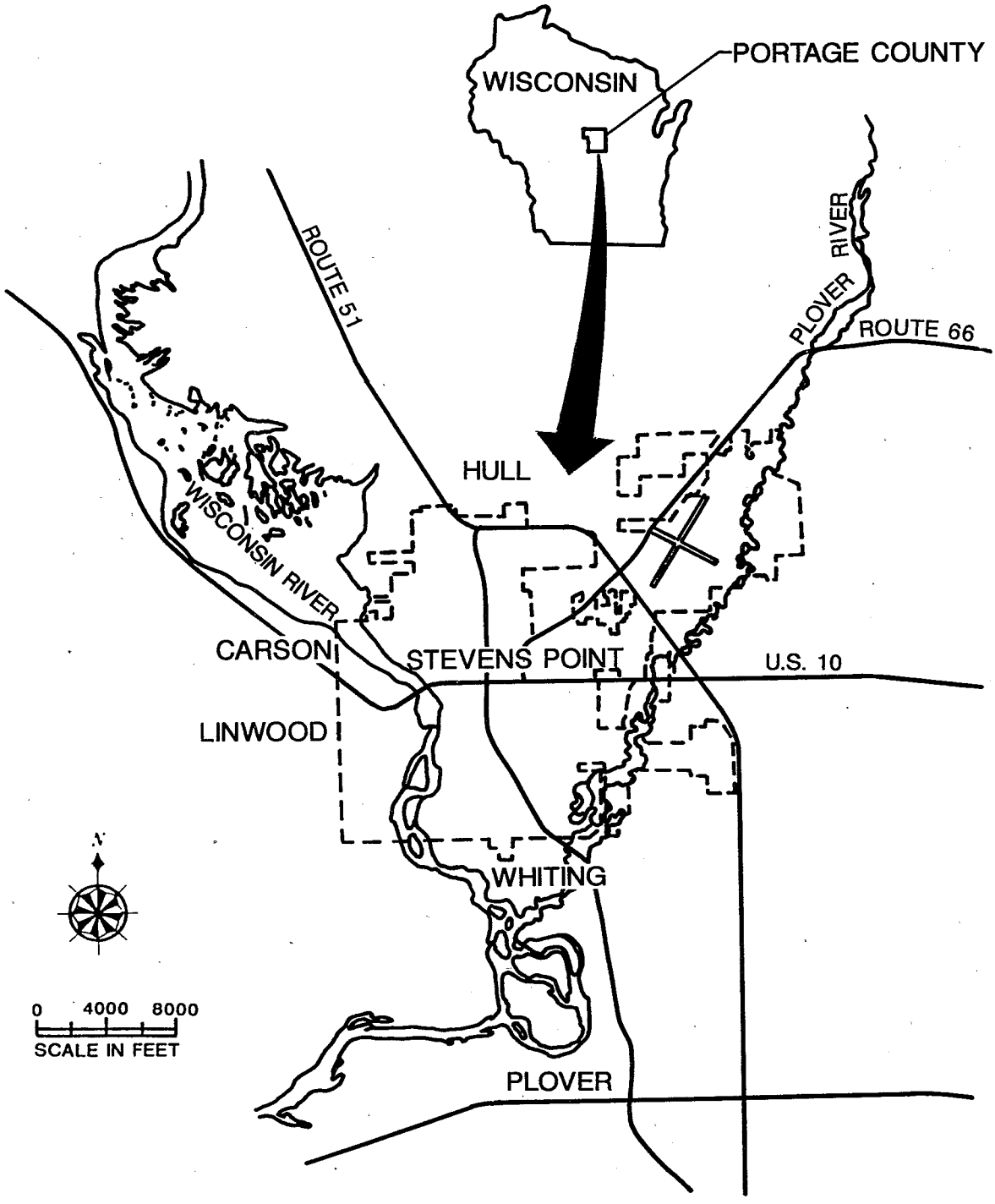


Figure 2-1. Location of the city of Stevens Point within Portage County, State of Wisconsin.

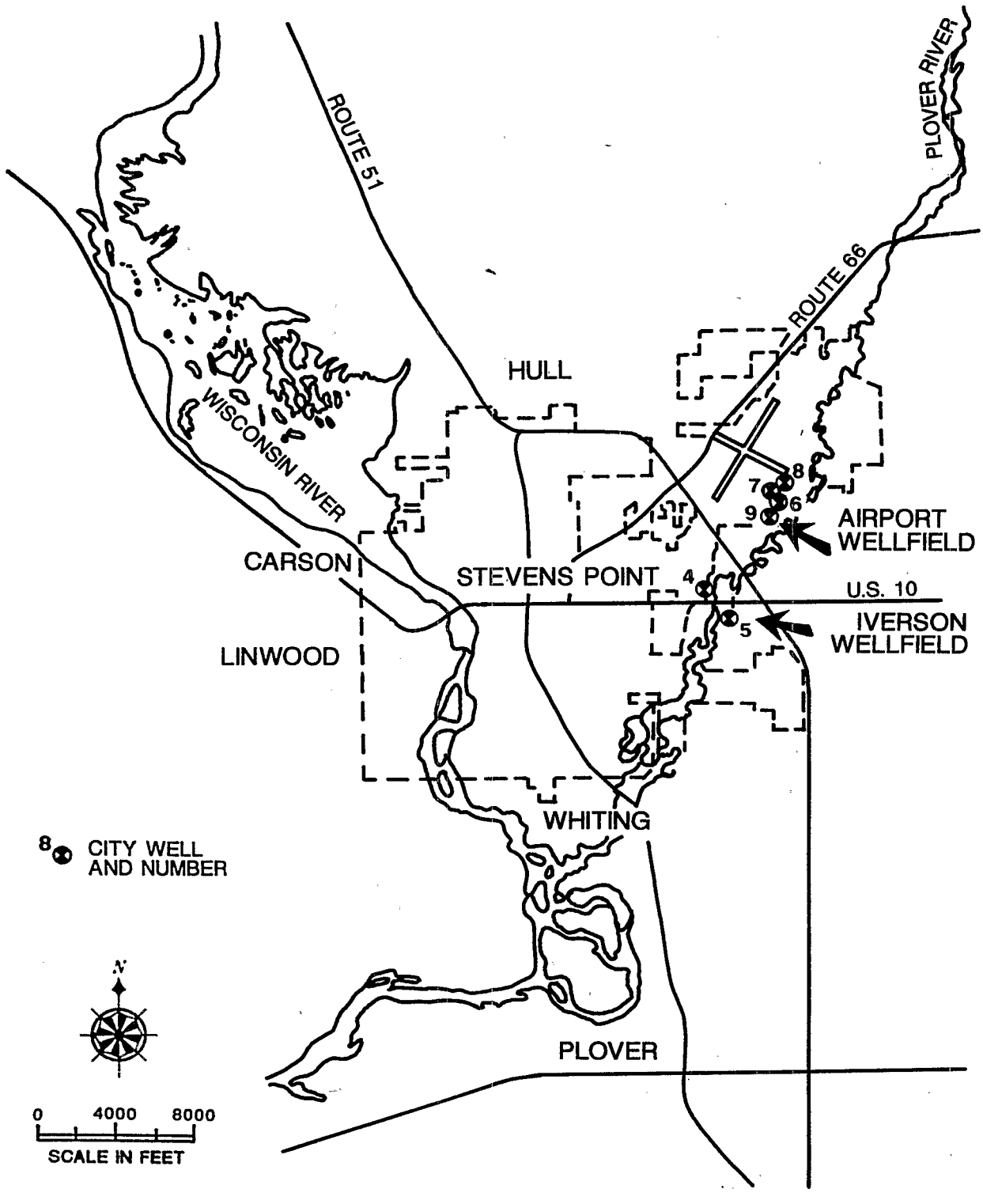


Figure 2-2. City wells at the Airport and Iverson wellfields, Stevens Point, Wisconsin.

resulted in hydrogeologic mapping of the area from the airport to the Airport wellfield and the installation of five observation wells downgradient from the airport's fuel storage tanks. Although the fuel leak at the airport in 1988 was not detected by sampling the observation wells, the estimates of contaminant transport velocities from the Hickok and Associates study prompted swift remedial action by the city.

Increased public awareness of wellhead protection and the city's and county's efforts to promote it were hastened by: (1) the closing of the town of Whiting's wellfield in 1979, (2) the discovery of aldicarb in Portage County ground water in 1980 (Chesters et al., 1982), (3) the publication of a statewide ground-water susceptibility map in 1987, and (4) the fuel leak at the Stevens Point airport in 1988. The Stevens Point Water Department began funding research through the University of Wisconsin - Stevens Point regarding recharge and water quality at both the Iverson and Airport wellfields (for example, Renaud, 1987). The Portage County Planning and Zoning Department developed a county ground-water management plan in 1988 that includes a county-wide source assessment and proposes generic land-use restrictions for the zone of influence (ZOI), the 5-year time-of-travel (TOT) zone, and the recharge area of municipal wells in the county. The Stevens Point WHPP is the next step in the evolution of the city's water management plan. This project emphasizes the importance of a technically defensible WHPA, management tools available to protect the ground-water supply, and the need for cooperation among government agencies.

The overall objectives of the WHPP project are as follows: (1) to conduct a detailed source assessment for the 5-year TOT zone, (2) to rank the sources identified in the source assessment, (3) to refine the delineation of the WHPA, (4) to design a ground-water monitoring well network, (5) to document a contingency plan, and (6) to recommend a strategy for future research to improve the WHPP.

Hydrogeologic boundaries seldom coincide with political boundaries. Consequently, cooperation among participating government agencies is essential. The groups cooperating with the Stevens Point Water Department in this study include the Stevens Point Community Development Department, the Stevens Point Public Works Department, the Portage County Planning and Zoning Department, the University of Wisconsin - Stevens Point Environmental Task Force, and the Central Wisconsin Ground-Water Center.

HYDROGEOLOGIC SETTING

Geologically, Portage County consists of unconsolidated, glacial outwash sediments that were deposited over crystalline rock, primarily granite, of Precambrian age. Scattered remnants of Cambrian sandstone exist on a few bedrock highs. A geologic map of the county (Figure 2-3) shows the location of bedrock and sandstone outcrops (Clayton 1986; Holt, 1965). In areas where the glacial sediments are thin or not present, ground water is obtained from fractured crystalline bedrock. The glacial sediments form the most productive aquifer in this region.

The city's production wells pump ground water from the sand and gravel aquifer. This aquifer is a shallow, unconfined system composed of coarse, unconsolidated sediments deposited by meltwater streams during the Wisconsin glaciation. In the study area, the glacial deposits are thickest (200 feet)

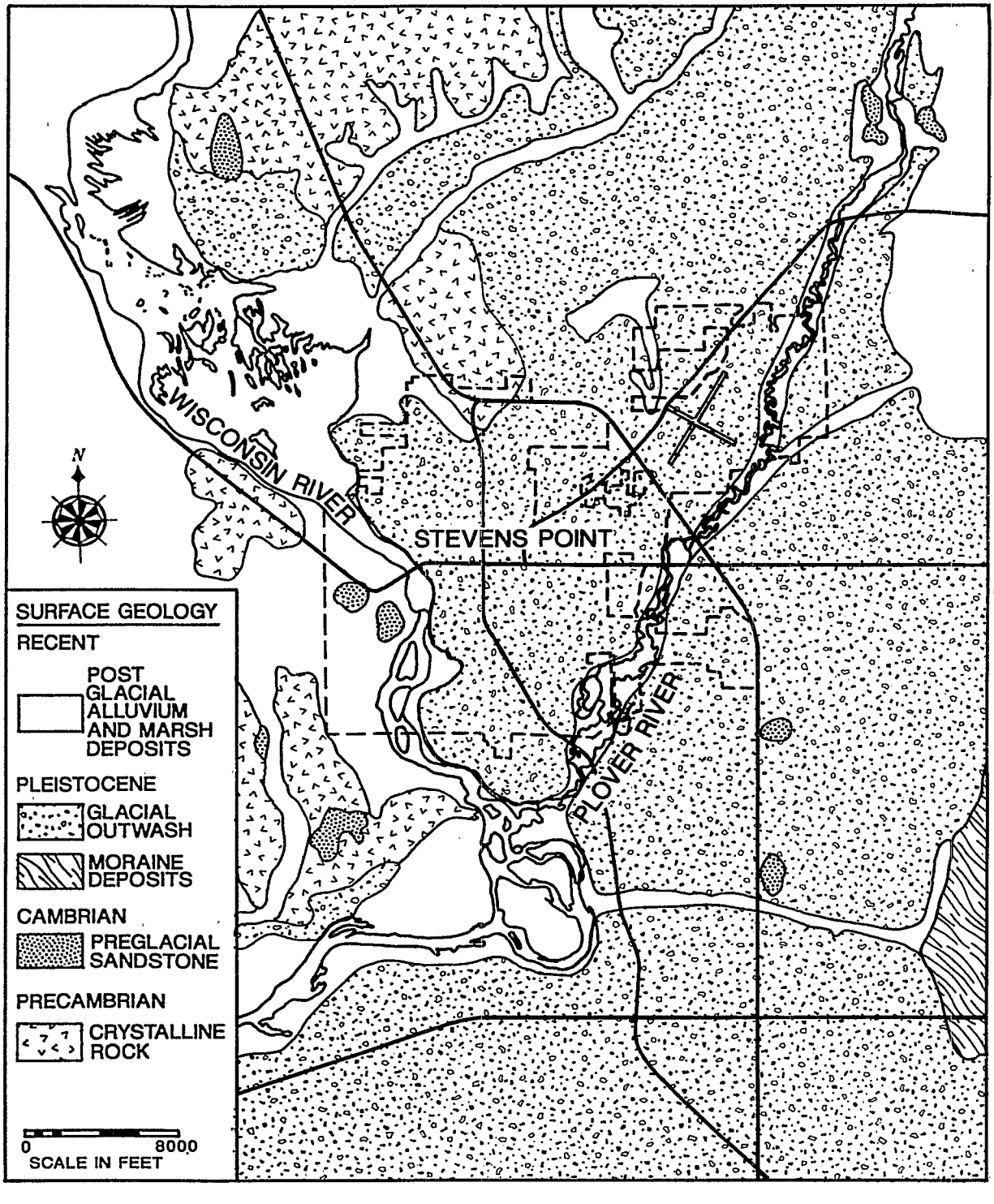


Figure 2-3. Surface geology map, Stevens Point, Wisconsin. (Modified after Holt, 1965, Plate 1.)

in the preglacial entrenched Wisconsin river valley, thinning to the west where the Precambrian age crystalline bedrock crops out. A hydrogeologic cross section of the Airport wellfield, shown in Figure 2-4, illustrates the location of the buried valley and the variability in the saturated thickness.

Regional ground-water flow is toward the Plover River (Figure 2-5). The city wellfields are within a single flow system that is delineated by the drainage basin for the Plover River. The portion of the flow system upgradient of the city's wells is 3 to 7 miles wide, extending from 1.5 miles southwest of the Airport wellfield to more than 30 miles northeast of the wellfield. Locally, the ground-water flow system is influenced by aquifer heterogeneities, changes in topography, and ground-water sources and sinks (for example, recharge events, ponds, irrigation, and production wells).

The city produces an average of 5 million gallons of water per day (Mgd) from the aquifer. The period of maximum pumping occurred in June 1988. In that month, the city wells pumped more than 300 million gallons of ground water from the sand and gravel aquifer. The city obtains water from six wells (numbers 4 through 9) in two separate wellfields (Figure 2-2). City wells 4 and 5 (CW4 and CW5) are located at the Iverson wellfield. City wells 6, 7, 8, and 9 comprise the Airport wellfield, located approximately 1 mile north-northeast of the Iverson wellfield. City well 4 is used only during peak demand periods or emergencies, because of high iron and manganese concentrations.

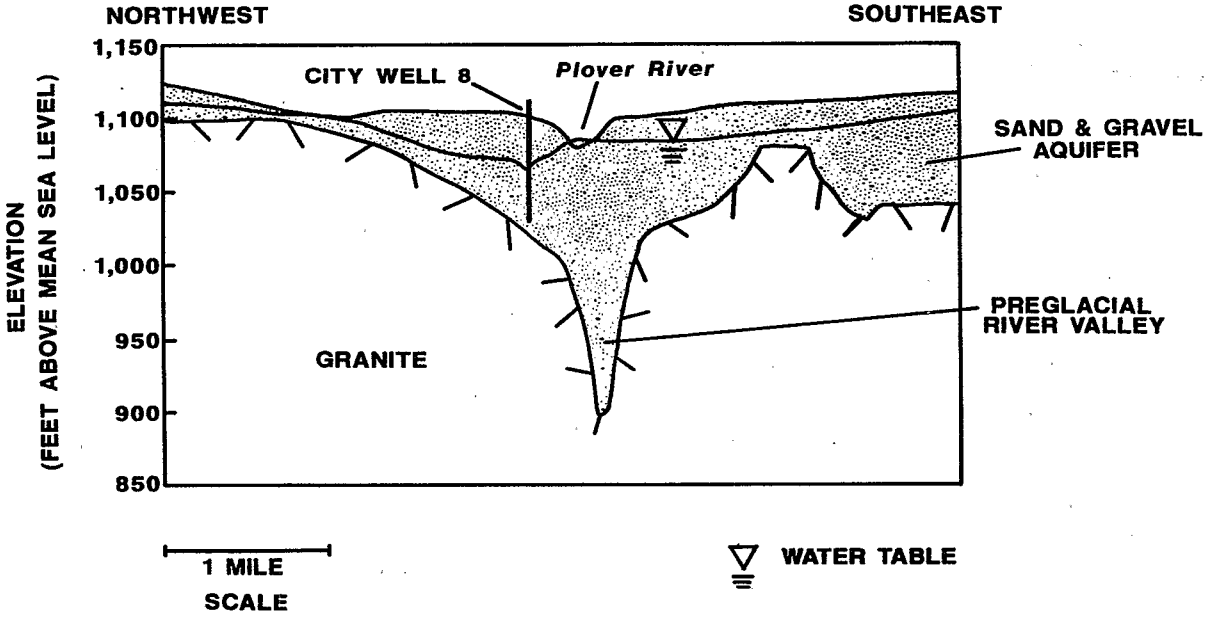


Figure 2-4. Hydrogeologic cross section northwest to southeast across the Airport wellfield, Stevens Point, Wisconsin.

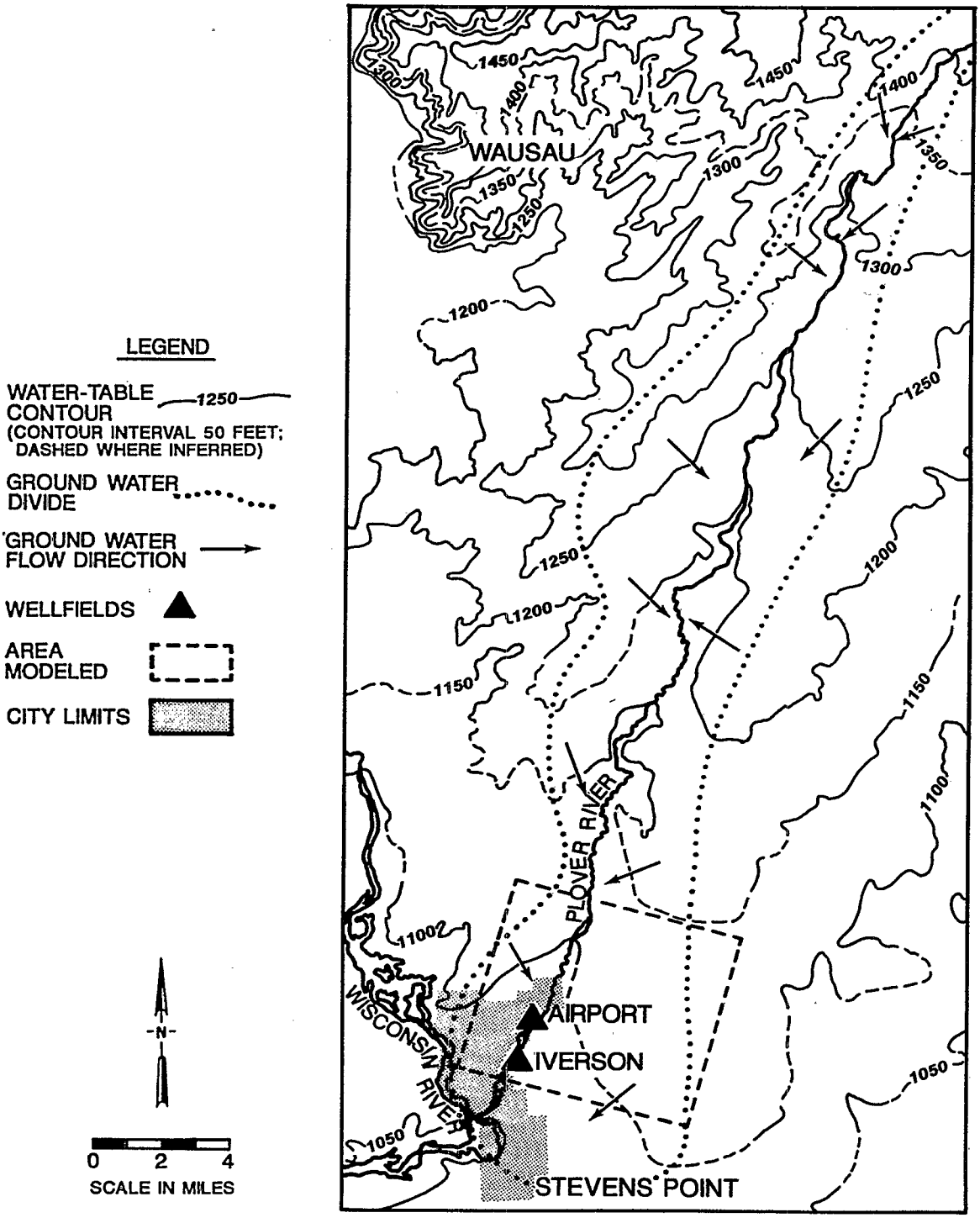


Figure 2-5. Recharge area for the Airport and Iverson wellfields, Stevens Point, Wisconsin. (Modified after Lippelt and Hennings, 1981.)

The majority of the recharge area for the city's wellfields has a DRASTIC index rating of 180 to 200, indicating a very high relative pollution potential (Aller et al., 1987; Portage County Planning Department, 1988). A statewide analysis by the Wisconsin Geological and Natural History Survey (WGNHS) indicates that the soils in the recharge area have low to moderate attenuation ability, and recommends that land-use activities in the county be carefully managed (Good and Madison, 1987). Land use in the recharge area is generally intensive, comprised of a combination of sewered and nonsewered residential, commercial, agricultural, and undeveloped property. The majority of the recharge area is beyond the city's jurisdiction. Much of the land immediately to the north and east of the Airport wellfield is in the town of Hull, a rapidly growing, high-density, nonsewered residential area.

PRELIMINARY WELLHEAD PROTECTION AREA

Wellhead protection zones proposed by the city (Figure 2-6) are based on estimates of the ZOI, the 5-year TOT zone, and the recharge area for both wellfields. The ZOI ("A" Zone) was delineated assuming a fixed radius of 1,500 feet. The 5-year TOT zone ("B" Zone) was based on a fixed radius of 2 miles, calculated using a ground-water velocity of 6 feet per day (ft/d), a hydraulic conductivity of 1,200 ft/d (calculated from aquifer-test results of the city wellfield), and a porosity of approximately 25%. The ground-water velocity was calculated based on a maximum hydraulic gradient of 25 feet per mile. The western boundary of the B Zone was modified to conform to the hydrologic boundary. The recharge area ("C" Zone) is the remainder of the flow system within the county, upgradient of the 5-year TOT zone. The C Zone boundaries were mapped hydrogeologically and correspond to the boundaries of the Plover River drainage basin (Lippelt and Hennings, 1981).

The draft Portage County Groundwater Management Plan (Portage County Planning Department, 1988) proposed that the city adopt protection zones for the Stevens Point wellfields. In the review process, the validity of the B Zone, based on analytical calculations of the 5-year TOT, was questioned regarding the representativeness of hydraulic conductivity and porosity values for the aquifer and the use of a maximum hydraulic gradient. Consequently, the Management Plan was published without determination of the preliminary WHPA and protection zones. One objective of the current study is to verify, or refine, the boundaries of the ZOI and 5-year TOT zone through further hydrogeologic characterization and ground-water modeling studies. This goal is addressed by water-level mapping, aquifer testing at the Airport wellfield and at other wells in the recharge area, and by the use of a two-dimensional numerical ground-water flow model.

SOURCE ASSESSMENT

Source Identification

A general source assessment of the recharge area for the city's wellfields was conducted during this study, and a detailed evaluation of sources in the 5-year TOT zone was made. Existing and potential sources of ground-water contamination were identified, classified, and mapped. Contaminant sources were identified by researching existing and historical land use. The research methods used

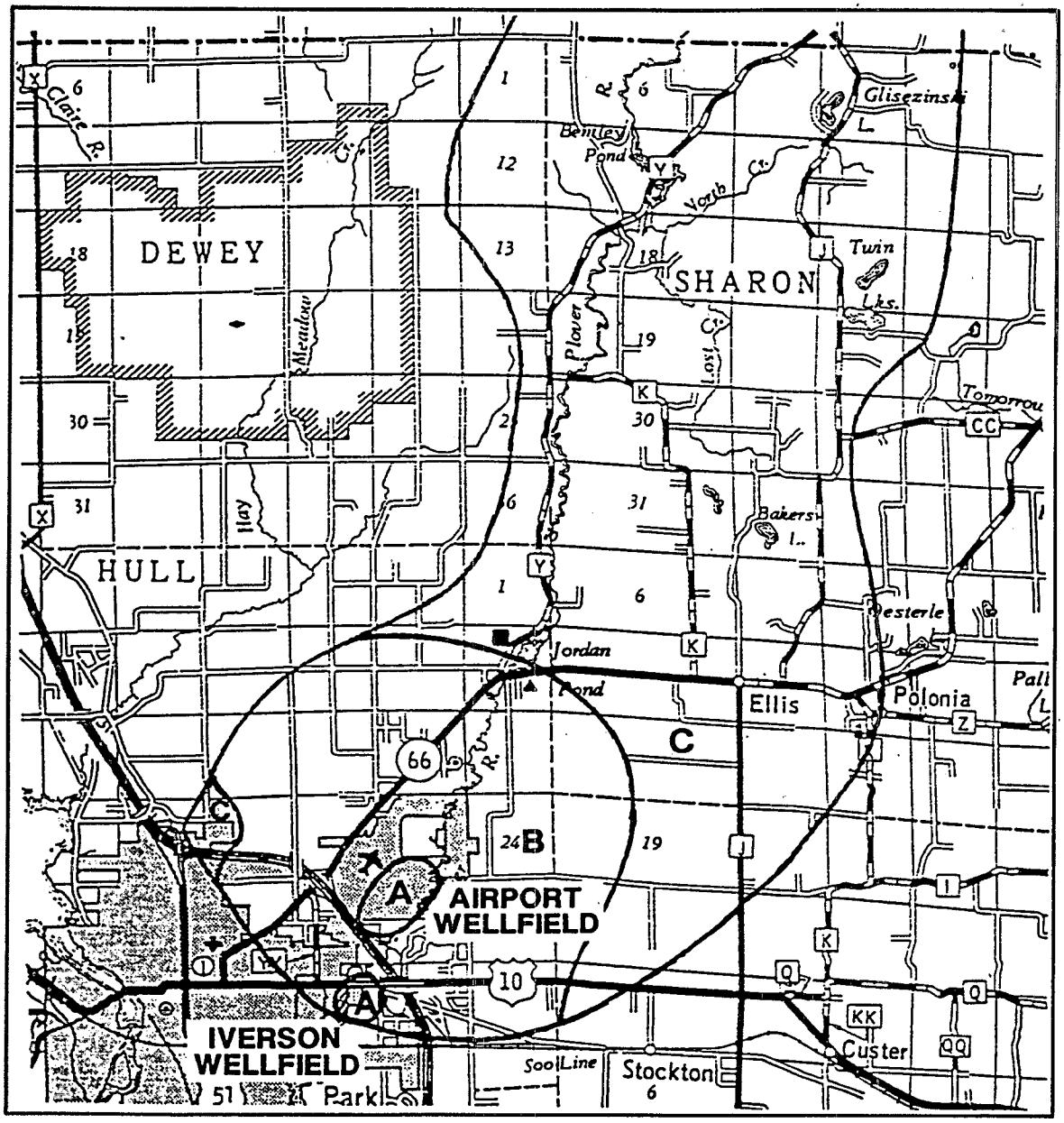


Figure 2-6. Wellhead protection zones (A, B, and C) for the Airport and Iverson wellfields, Stevens Point, Wisconsin. (Modified after Portage County Planning Department, 1988, Appendix C-8.)

included listing known ground-water contamination sources, incorporating existing data bases on potential source locations, making a visual survey of the study area for potential sources, sampling monitoring wells, and conducting an historical source assessment.

Existing ground-water contamination problems in Portage County include aviation fuel leakage at the airport in 1988; gasoline detected in three private wells in the town of Hull in 1990; nitrates in the village of Whiting's municipal wells; and aldicarb and atrazine pesticides detected in rural, private wells. Concentrations of nitrogen, manganese, and iron in ground-water samples from some of the city's monitoring wells have exceeded primary and secondary drinking water standards. Elevated levels of iron and manganese are attributed to natural sources, whereas nitrogen is anthropogenic. Sources of high nitrogen levels in the recharge area include leachates from septic systems, lawn fertilizers, crop fertilizers, and animal wastes. Gasoline contamination of ground water may occur due to leaks from underground storage tanks (USTs), spills, improper disposal of petroleum wastes, and ruptures in aboveground storage or transport vessels. Pesticide sources include applications to crops or lawns and accidental releases from storage facilities during transport or mixing operations.

Land-use practices were identified and mapped as potential sources of ground-water contamination (Table 2-1). Initially, potential sources were located based on the source assessment conducted as part of the county's ground-water management plan. During this study, a more detailed assessment within the B Zone was conducted. Businesses that store, transport, generate, or dispose of potential contaminants were located by using city directories, local emergency government files, and by visual survey. In addition, the location of USTs within the recharge area was determined from an inventory of underground petroleum-product storage tanks, maintained by the Wisconsin Department of Industry, Labor, and Human Relations. The location of hazardous material spills was identified using the Wisconsin Department of Natural Resources spills list for Portage County.

An historical source assessment was conducted in conjunction with the U.S. Environmental Protection Agency's (EPA) Environmental Monitoring Systems Laboratory in Las Vegas (EMSL-LV). Lockheed Environmental Systems & Technologies Company (LESAT), under contract to EMSL-LV, identified and documented the historical evolution of potential contaminant sources in the B Zone by analyzing aerial photographs dating back to 1938 (Finkbeiner et al., 1991). The aerial photographic source assessment was supplemented with data obtained from historical maps and city directories dating back to 1922.

Following the local source data base investigation and photographic analysis, an inventory list of existing and potential contaminant sources was compiled (Table 2-2). These sources require prioritization to focus protection and management efforts within the WHPA. Table 2-2 also lists detected and suspected contaminants associated with the point and nonpoint categories. For the purposes of this report, septic systems are categorized as both point and nonpoint sources. Individual systems are considered point sources. However, areas with a high density of nonsewered residences are classified as nonpoint sources.

TABLE 2-1. POTENTIAL CONTAMINATION AND DEGREDATION SOURCES WITHIN THE B ZONE OF THE PRELIMINARY WHPA, STEVENS POINT, WISCONSIN*

Agricultural	Commercial	Industrial
Abandoned, and poorly constructed wells	Airports	Asphalt plants
Feedlots	Cemeteries	Concrete plants
Fertilizer and pesticide application	Dry cleaners	Electroplaters
Fertilizer and pesticide storage	Exterminators	Formulators
Fuel storage	Firing ranges	Fuel and chemical storage
Pesticide mixing	Fuel and chemical storage	Machine shops
Sludge and septic spreading sites	Furniture refinishers	Pipelines
	Garages (repair shops)	Transformers (electrical)
	Gas stations	
	Golf courses	
	Junkyards	
	Laboratories	
	Landscapers	
	Paint stores and painters	
	Plumbers and welders	
	Printers	

Residential	Waste Management	Naturally Occurring
Fertilizer and pesticide application	Incinerators	Organic-rich wetland soils (iron and manganese)
Fuel and chemical storage	Injection wells	Granite bedrock (radon and radioactivity)
Septic systems	Landfills	
	Recycling facilities	
	Snow dump sites	
	Storm sewer and dry wells	
	Waste water treatment plants	

*Modified after U.S. EPA, 1990, p. 33.

Examples of potential and existing sources close to the Airport wellfield are shown in Figures 2-7 and 2-8. Typical point sources include USTs, spills, a landfill, waste and snow disposal areas, light-industrial and commercial facilities (such as gas stations, auto repair shops, and dry cleaners), dry wells (drainage wells in nonsewered areas), and perforated drainage pipes. Nonpoint sources include crop and pasture lands, septic systems, nonsewered residential areas, golf courses, and institutional lawn areas.

Farther from the wellfield, the character of the sources shift from urban and suburban to rural and agricultural. This shift is marked by the abundance of point sources in the urban and suburban area to the predominance of nonpoint sources in the rural, agricultural areas. On a regional scale, the widespread application of pesticides and fertilizers in agricultural areas is considered a threat to the city's wells. Large quantities of hazardous substances stored or handled within the recharge area may

TABLE 2-2. EXISTING AND POTENTIAL CONTAMINATION AND DEGRADATION SOURCES WITHIN THE WHPA, STEVENS POINT, WISCONSIN

	Existing Sources		Potential Sources	
	Type of Source	Detected Contaminants	Type of Source	Suspected Contaminants
Point Sources	Airport underground storage tanks	Aviation fuel	Garages and gas stations	Hydrocarbons and solvents (VOCs)
	Unknown	Gasoline	Industrial and commercial facilities	VOCs, SVOAs, and heavy metals
			Junkyards	VOCs, SVOAs, and heavy metals
			Landfills	VOCs, SVOAs, and heavy metals
			Salt and snow storage areas	Salt
			Septic systems	Nutrients and VOCs
			Spills	Hydrocarbons and solvents (VOCs)
			Storage tanks	Hydrocarbons and solvents (VOCs)
			Transformers	PCBs and hydrocarbons
			Unregulated dumpsites	VOCs, SVOAs, and heavy metals
Nonpoint Sources	Agricultural fields	Nitrate and pesticides	Feedlots	Nutrients
			Golf courses	Nutrients and pesticides
	Lawns	Nitrate	Lawns	Nutrients and pesticides
	Septic systems	Nitrate	Plover River	Nutrients and pesticides
	Organic-rich wetland soils	Iron and manganese	Road de-icing runoff	Salt
			Septic-sludge spreading	Nutrients
			Septic systems	Nutrients and VOCs
			Urban runoff	VOCs, heavy metals, and nutrients

Abbreviations: VOCs = volatile organic compounds; SVOAs = semi-volatile organic analytes; PCBs = polychlorinated biphenyls.

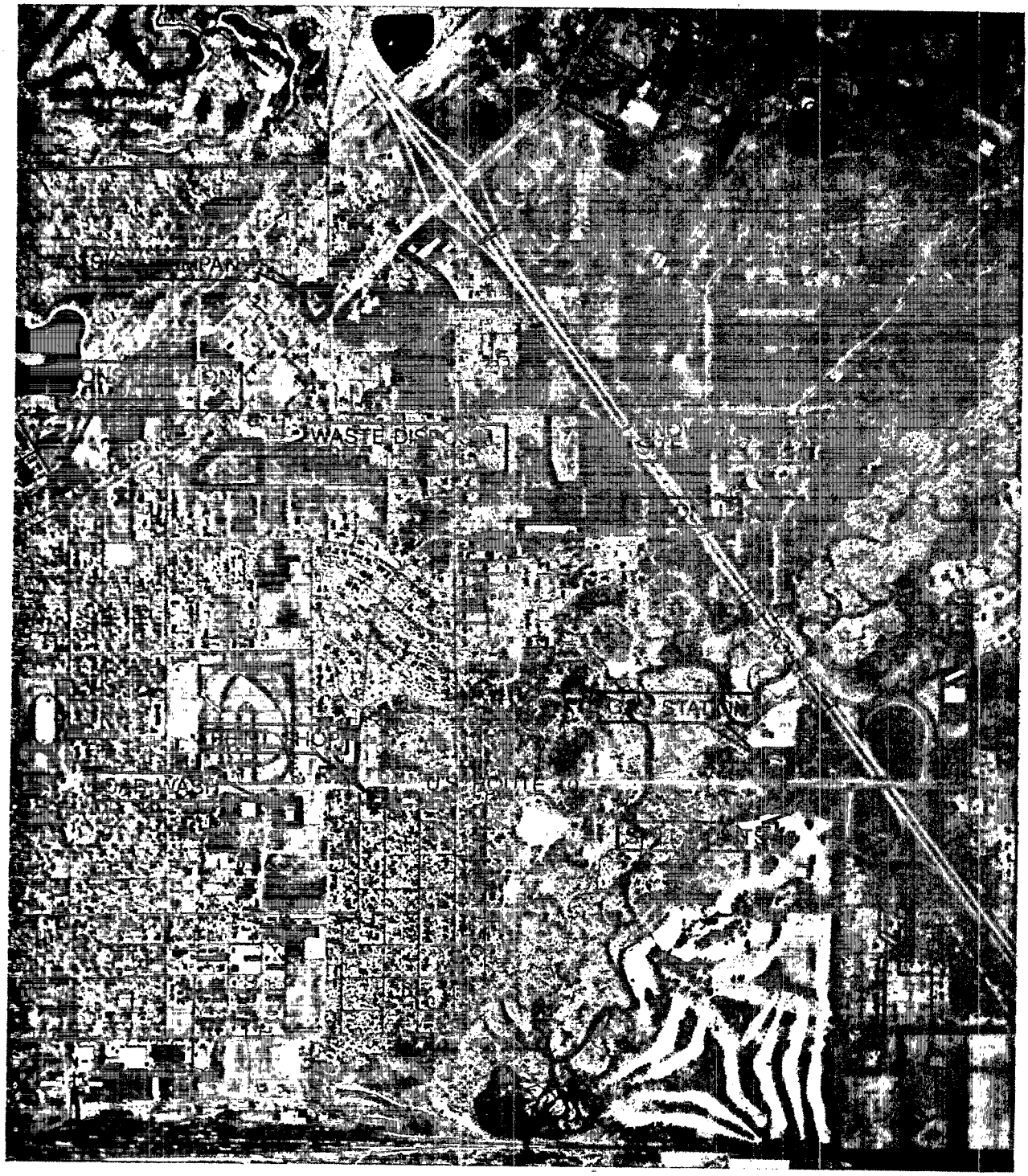


Figure 2-7. Typical point sources south of the Airport wellfield, Stevens Point, Wisconsin. (Institutional lawn area and golf course are considered as nonpoint sources for this study.)

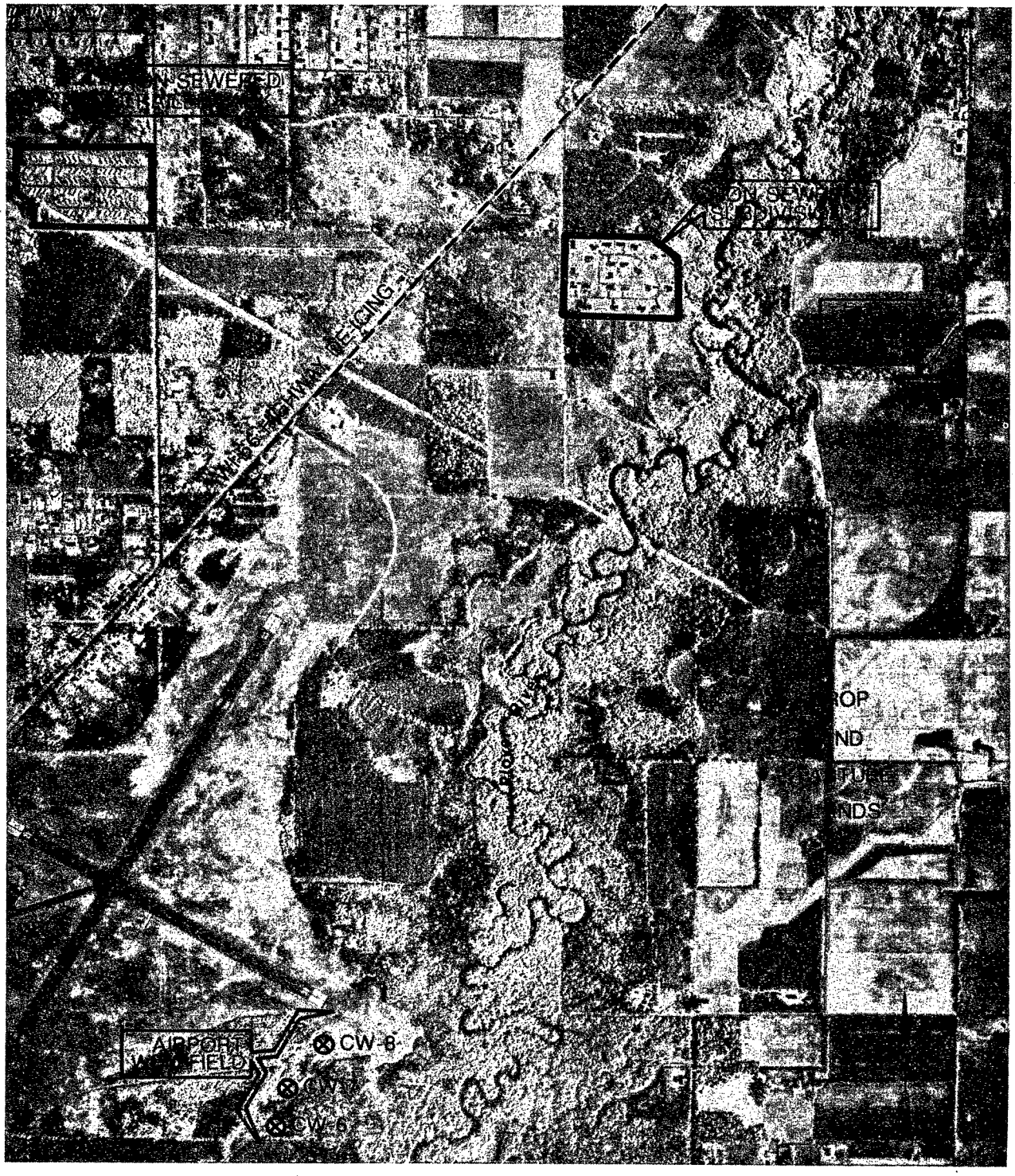


Figure 2-8. Typical nonpoint sources north of the Airport wellfield, Stevens Point, Wisconsin. (Feedlot area is considered a point source for this study.)

pose a significant threat to the city's wells. Large point sources farther from the wellfield include junkyards, municipal landfills, feedlots, a salt pile, and high-density, nonsewered residential areas (for example, trailer parks).

Source Characterization and Prioritization

Sources were prioritized on the basis of their category (point versus nonpoint), the degree of their threat (existing versus potential), their proximity to the city wellfields, and the quantity and type of hazardous or toxic substances handled. A qualitative ranking of sources and contaminants resulting from this evaluation is given in Table 2-3. Rather than assign a numeric ranking scale for each characteristic, existing and potential sources were grouped by category from greatest to least concern.

Existing contaminant sources were given highest priority. Next, potential sources were ranked based on a combination of factors: source type, quantity, hazard, and location. The highest priority contaminant sources are USTs, agricultural areas, high-density, nonsewered residential areas, landscaped areas, and feedlots. Existing VOC, hydrocarbon, nutrient, and pesticide contamination in the ground water is believed to be attributed to these sources. Large spills of hazardous substances within the 5-year TOT zone and all spills within the ZOI of the city wells are also of highest priority.

TABLE 2-3. PRIORITIZED, POTENTIAL CONTAMINATION AND DEGREDATION SOURCES WITHIN THE PRELIMINARY WHPA, STEVENS POINT, WISCONSIN

Level of Concern		Contaminant Source
Greatest	1.	Large (>500 gal) USTs at the airport, gas stations, and garages; small USTs (B Zone) from residential and commercial areas; spills (A Zone)--*existing* VOC and hydrocarbon contamination
	2.	Agriculture, residential, and commercial areas--*existing* nonpoint nutrient and pesticide contamination
Great	3.	Landfills (B Zone)
	4.	Urban runoff (B Zone)
	5.	Naturally occurring sources; granite bedrock and organic-rich wetland soils
	6.	Spills of hazardous substances (B Zone)
Moderate	7.	Residential runoff and hazardous substance disposal (B Zone NSAs)
	8.	Landfills
	9.	Storage tanks (C Zone)
	10.	Large spills (C Zone)
	11.	Junkyards (C Zone)
	12.	Road de-icing runoff and storage areas (B Zone)
Least	13.	Small spills (C Zone)
	14.	Low-density NSAs (C Zone)

Abbreviations: USTs = underground storage tanks; VOCs = volatile organic compounds; NSAs = nonsewered areas.

Iron and manganese ground-water degradation exists because of the presence of organic-rich soils high in wetland areas along the Plover River. Reducing conditions in these soils increases metal solubility, therefore increasing the concentrations of iron and manganese in ground water. These constituents are of less concern because the drinking water standards for iron and manganese are secondary, based on aesthetic considerations rather than primary health factors. Other sources of great concern within the B Zone include landfills, perforated storm pipes and dry wells that drain urban runoff, granite as a natural source, and spills. Large point sources in the C Zone [for example, fuel storage tanks larger than 1,000 gallons (gal)] and de-icing salts in the B Zone are classified as moderate threats. Point sources in the C Zone are farther from the wellfield, allowing for greater dilution and reaction time if contamination occurs. Another factor in assigning moderate priority to these sources is that the health hazards from sodium and chloride are less than those associated with VOCs and pesticides. Of least concern are small, potential sources located outside the 5-year TOT zone, such as low-density, nonsewered residential areas (less than one home per 5 acres) and small spills.

RESEARCH MONITORING PROGRAM

The ground-water monitoring program has two components: research monitoring and a proposed long-term monitoring program. The purpose of the first stage, research monitoring, is to gather reliable physical and chemical information in order to develop an accurate conceptual model of the hydrogeologic system. Specifically, research monitoring may lead to a refined WHPA delineation, flowpath assessment, or better hydrochemical characterization of the aquifer and contaminant sources. The second stage is monitoring of ground water in the WHPA for the long-term preservation of the wellfield. The design of a reliable wellhead protection monitoring system is based on: (1) thorough source assessment, (2) accurate delineation of the WHPA, and (3) hydrochemical characterization and flowpath evaluation.

DATA BASE LIMITATIONS

In the first phase of research monitoring, the existing data base was reviewed to ascertain the hydrochemical and hydrogeologic characteristics of the sand and gravel aquifer. Hydrogeologic mapping and investigations of aquifer properties in the Stevens Point area include work by Holt (1965), Hickok and Associates (1965 and 1981), Weeks and Stangland (1971), Lippelt and Hennings (1981), Renaud (1987), and Donohue and Associates (1988). In addition, a number of hydrochemical studies focused on the sand and gravel aquifer. Renaud (1987) studied the occurrence of manganese in the vicinity of the Airport wellfield. Several studies on pesticide occurrence and transport through the sand and gravel aquifer supplement the existing data base (for example, Harkin et al., 1984; Jones, 1987; and Portage County Planning Department, 1988). Other research on hydrochemistry included a local study on nitrogen contamination from feedlots (Bowen, 1987) and a county map of soil attenuation potential (Good and Madison, 1987).

All of these studies were useful in the development of a detailed conceptual model of the sand and gravel aquifer. However, none focused on the combined physical and chemical hydrologic system for the city wellfields. These studies present an incomplete description of the hydrogeologic system. They either address in detail a small portion of the aquifer or discuss the entire regional hydrogeologic system. Consequently, the existing data base requires supplementation before it can support the objectives of the wellhead protection monitoring program.

MONITORING OBJECTIVES

The second step of the research monitoring program was to refine or verify the conceptual hydrogeologic model. The goals associated with this step were to supplement and fill gaps within the existing hydrochemical and hydrogeologic data bases. Deficiencies in the data bases include information on: (1) the vertical and lateral distribution of hydraulic conductivity in the sand and

gravel aquifer; (2) the local ground-water flow patterns; (3) the extent of the ZOI for the city wellfields; and (4) the lateral, vertical, and temporal distributions of contaminants in the vicinity of the Airport wellfield.

RESEARCH MONITORING TASKS

Tasks performed in support of the monitoring objectives included:

- Installing new monitoring wells
- Sampling existing and new monitoring wells for suspected contaminants
- Measuring water-table elevations
- Conducting aquifer tests to determine hydraulic conductivities

All hydrochemical sampling during the research monitoring phase was conducted to document spatial and temporal trends in ground-water quality. Physical measurements were made to refine the WHPA, assess predominant flowpaths, and improve the conceptual hydrogeologic model.

DATA ACQUISITION

Monitoring Wells

Existing wells--

During the research monitoring stage of this project, a network of 44 single monitoring wells, 10 well nests, and one multi-level well were used to measure water levels, to sample ground water for chemical analysis, and to conduct aquifer testing (Figure 2-9). Thirty-five of the single monitoring wells, six of the well nests, and the multi-level well existed prior to this study.

The existing monitoring wells were installed for a variety of projects and reasons. Fourteen of the existing monitoring wells (MW1 through MW11, and MW25 through MW27) are shallow wells installed to measure water-table elevations and to monitor shallow hydrochemistry in the recharge area north and west of the Airport wellfield. Similarly, 18 shallow monitoring wells (MW12 through MW22, and MW29 through MW35) were installed to verify water levels and shallow hydrochemistry on the east side of the Plover River. All of these wells were drilled using a hollow-stem auger. The well construction for the single monitoring wells consists of 1.25-inch, flush-threaded, polyvinyl chloride (PVC) casing, a 3-foot screen, natural, collapsed sand filter, native sand backfill, and a 2-foot bentonite seal at the surface (Figure 2-10A).

One of the monitoring wells, MW24, is part of the monitoring system established at the airport after the aviation fuel leak in 1988. Two existing monitoring wells, MW23 and MW28, have galvanized-steel casings. One is an old test well at the Airport wellfield (MW23), and the other is an old production well for the airport (MW28).

Well nests A through F and well G were installed to monitor vertical gradients and water-level fluctuations, as well as manganese and iron concentrations between the Airport wellfield and the Plover River. Well nest G was originally installed as two wells at different depths in one borehole

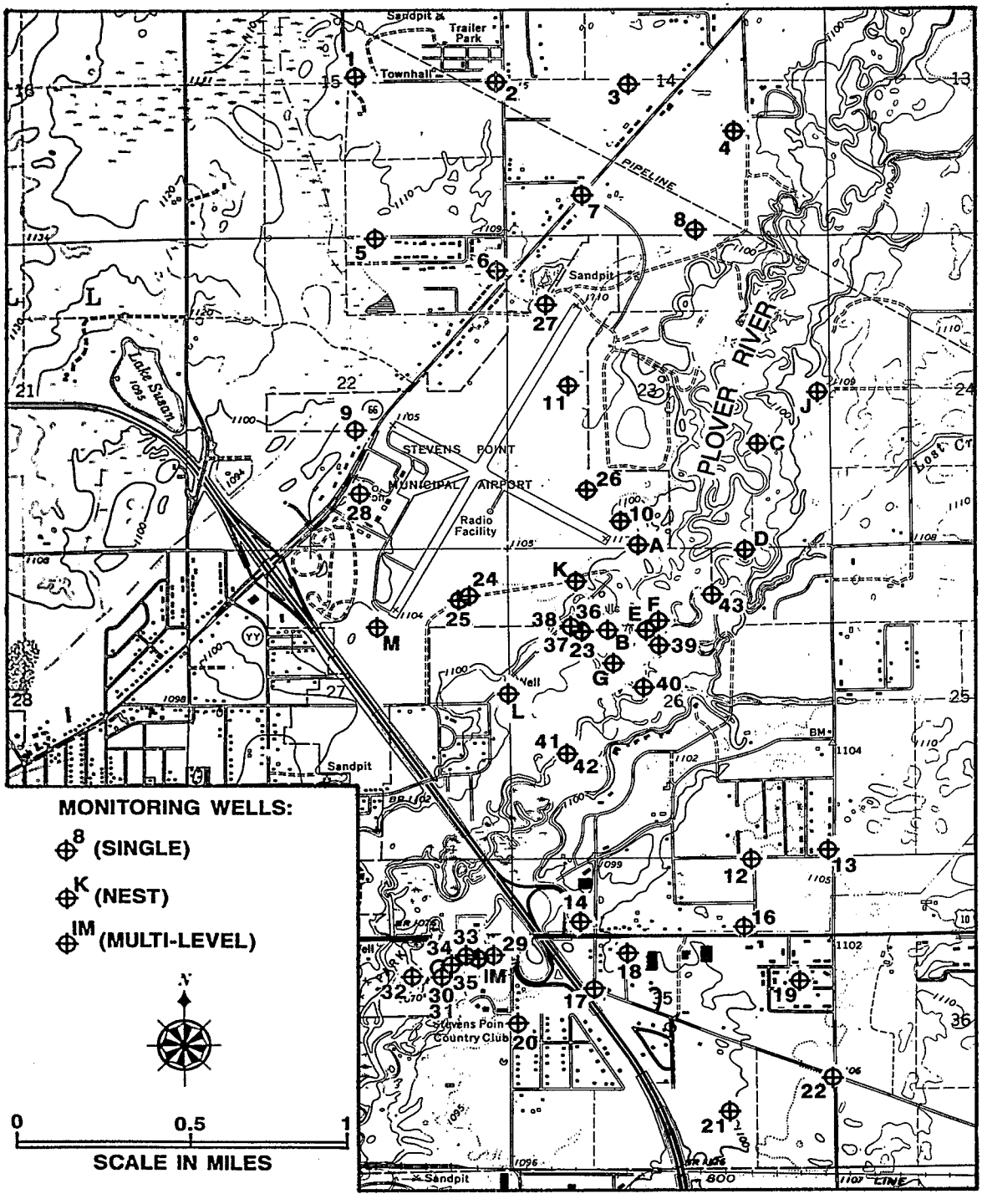


Figure 2-9. Research monitoring wells, Stevens Point, Wisconsin.

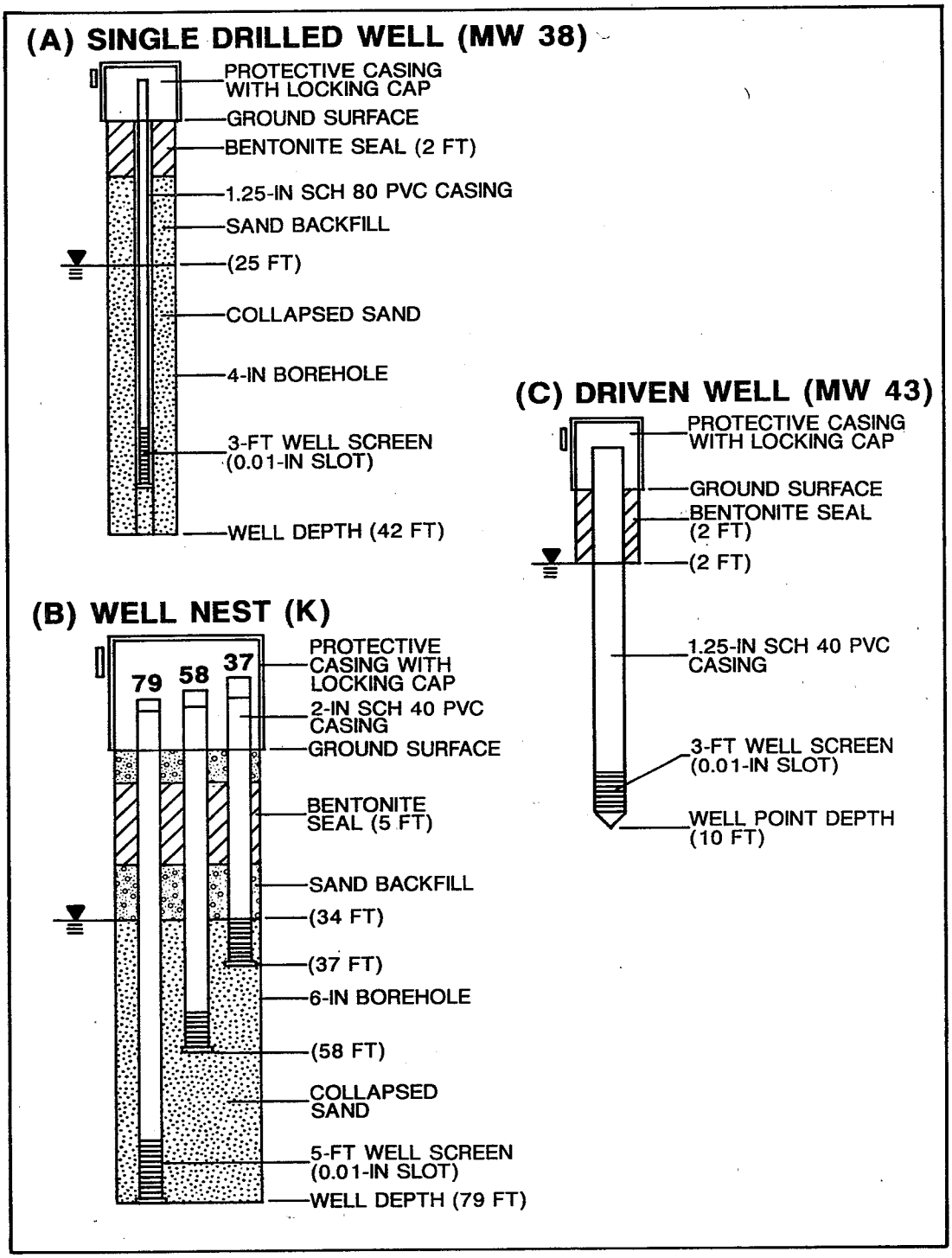


Figure 2-10. Research monitoring well construction diagrams for (A) single, drilled well; (B) well nest; and (C) hand-driven well.

(Renaud, 1987). Later, the deeper well was decommissioned, leaving a single well at shallow depth. Well nests A through F consist of two to three single wells at different depths (Figure 2-10B). These wells are constructed of 1.25-inch, flush-threaded, PVC casing, a 3-foot screen, natural, collapsed sand filter, native sand backfill, and a 5-foot bentonite seal at the surface. The multi-level well, IM1, was positioned to monitor hydrochemical variations with depth, upgradient of CW5 (Figure 2-2). The multi-level well is essentially a bundle piezometer, consisting of a series of 1.25-inch PVC main structures (supporting core) with a series of 0.25-inch polytubes attached. The monitoring well was designed to detect changes in hydrochemistry at relatively small vertical intervals.

New Wells--

New monitoring wells were installed for a variety of reasons. Three of the wells, MW36 through MW38 (Figure 2-9), were installed around CW6 (Figure 2-2) as observation wells during aquifer testing. Five monitoring wells, MW39 through MW43 (Figure 2-9), were added on the east side of the Plover River for water-table elevation measurements. Four well nests, J through M (Figure 2-9), were installed to monitor hydrochemistry at depth at identified gaps in the existing monitoring well network.

All of the water-level monitoring wells were constructed as shown in Figure 2-10A, using 1.25-inch PVC pipe, 3-foot screens with 0.01-inch slots, and a 2-foot bentonite seal at the top of the well. All the new monitoring wells, with the exception of MW42, have PVC casing. Galvanized-steel pipe was used for MW42 so that it could be driven to a greater depth to monitor vertical hydraulic gradients. Monitoring wells 39 through 43 were installed as hand-driven well points (Figure 2-10C) because the well sites were physically inaccessible to a drilling rig. The three wells installed adjacent to CW6 (MW36 through MW38) were drilled with a small, hollow-stem auger rig.

The well nests were installed with a large drilling rig by pushing 6-inch steel casing to bedrock, cleaning the casing with a bailer, installing a 2-inch PVC pipe, then pulling up the steel casing to the next depth (Figure 2-10B). At this depth, a 2-inch pipe was installed. Then, the casing was pulled up to just below the water table and the last 2-inch pipe was installed before removal of the casing from the borehole, addition of the native sand backfill, and addition of a 5-foot bentonite seal. Two-inch PVC pipe and 5-foot to 10-foot screen lengths with 0.01-inch slots were used for well-nest construction. The nests were designed with 2-inch pipe so that a submersible pump could be used to adequately develop and purge the wells.

Well nests were installed where data gaps existed, both vertically and laterally in the system. Well nest J was positioned to determine the hydrochemistry with depth near the center of the buried river valley. This nest is the deepest in the monitoring network (160 feet). Well nest K was installed to obtain more data between the airport and the Airport wellfield. Well nests L and M were positioned to determine the hydrochemistry with depth between the Airport wellfield and two nonsewered residential and commercial areas. In general, each well nest consists of three separate wells screened at variable depths: one immediately above bedrock, one just below the water table, and one halfway between the other two. Well nest J consists of six wells in two separate boreholes: one borehole with three wells extends to bedrock; the other, with three wells, extends to half the bedrock depth. The wells in the J nest boreholes were screened at various depths: one just below the water table, one immediately above bedrock, and the intermediate wells at depths based on changes in lithology.

Construction diagrams for the three types of monitoring wells and nests are shown in Figure 2-10A, B, and C. Saturated sand and gravel collapses around the well casing, creating a natural filter pack for each well near the bottom of the borehole. Native sand is then used as backfill for the open borehole up to the bentonite seal. Each monitoring well and well nest has a protective steel casing with a locking cover. After the wells were installed, they were each developed by surging and pumping until the issuing water was clear. The elevations of the mark point for each monitoring well were surveyed to the nearest ±0.01 foot.

Physical Parameters

Two physical parameters, water level and specific conductivity, were determined each time a monitoring well was sampled. Water levels were measured twice for mapping purposes in January and March 1990 at all the monitoring wells within a 24-hour period. During this 24-hour period, no precipitation occurred to recharge the aquifer system. During January, several of the wells were inaccessible because of freezing or burial. Water-level measurements were obtained using a "popper" attached to a fiberglass measuring tape. The "popper" consists of a brass weight with a concave bottom surface that makes a sharp popping sound when it encounters the free-water surface in a well. Water-level data are compiled with other well information and hydrochemical results in Table 2-4.

Hydraulic conductivity values were determined by means of slug tests (Bouwer and Rice, 1976; Horslev, 1951) at select monitoring wells and a 24-hour drawdown (constant discharge) and recovery test at CW6 (Ferris et al., 1962). Slug tests were conducted using a PVC slug, a data logger, and a pressure transducer. The slug tests were performed by "dropping" the slug into the water ("slug in") and recording the response of the aquifer as it returned to the original static water level. After the water level stabilized, the slug was removed ("slug out"), and the response of the aquifer was recorded. Both 1.25-inch and 2-inch wells were used for slug testing.

Multiple slug tests were conducted at MW36, J78, J118, J160, MW13, K58, and K79. (The number following well nest designations J and K indicates the depth of the bottom of the well screen within the nest.) These wells were selected to determine variations in hydraulic conductivity of the sand and gravel aquifer in the vicinity of the Airport wellfield. Monitoring well 36 was selected as an observation well for the drawdown and recovery test at CW6 so that comparisons could be made with the slug-test results. Test comparisons were done to determine if the buried river valley has a consistently higher hydraulic conductivity than other areas of the aquifer. Slug tests at well nest J were conducted to verify the high hydraulic conductivity value obtained during the drawdown and recovery test. Monitoring well 13 is located on a bedrock high. During sampling, it was noted that the monitoring wells completed above bedrock highs could be purged dry with a bailer and would recover slowly. Monitoring well 13 was selected for slug testing to derive the lower range of hydraulic conductivity for the aquifer. Finally, well nest K was selected for testing to verify hydraulic conductivities of the sand and gravel aquifer in areas other than the buried river valley and bedrock highs.

TABLE 2-4. RESEARCH MONITORING WELL DATA FROM 1990, STEVENS POINT, WISCONSIN

MW	Well Information				Water Quality Parameters													
							(mg/l)										(µg/l)	
	DOW (ft)	MP Elev (ft)	Date	SWL Elev (ft)	Cond (µS/cm)	pH	Na	Cl	N	Mn	Fe	Zn	DO	COD	Alk	Hard	VOC	Pest
1	16	1110.90	1/17 4/12 5/24	1100.30 1100.50 1102.01	318	6.8	8.0	41	*	0.28	16.00	0.08		15.1	96	124	ND	
2	22	1111.30	1/17 4/12 5/24 5/29	1093.80 1094.30 1094.72 1094.84	1035	7.1	86.0	215 64	26.0 15.0	* 0.04	0.12 0.44	0.03		17.0 11.5	164 248	368 280	*a	 ND
3	30	1114.40	1/18 5/24 5/29	1088.88 1089.27 1089.32	325	8.1	3.0	12	5.2	*	*	0.01		*	132	196		†
4	34	1113.42	1/18 1/30 5/24 5/29	1084.24 1084.22 1084.82 1084.78	178	7.6	2.0	3	0.8	*	*	0.03		3.4	92	92		†
5	20	1105.12	1/17 5/24	1090.76 1092.15	103	6.7	6.0	5	0.2	*	*	0.04		*	40	56		
6	30	1111.10	1/17 4/12 5/24 5/30	1086.26 1086.21 1086.41 1086.48	250 238	7.3 7.6	4.0	11 8	3.2 1.0	* *	0.03 *	0.03		37.0 7.8	92 88	116 104	ND	
7	29	1109.82	2/1 5/24	1085.91 1086.12	425	7.3	34.0	49	1.8	0.01	0.15	0.04		9.9	140	144		
8	37	1112.43	1/17 1/24 5/24 6/26	1080.23 1080.20 1080.98 1081.12	 290 390	 7.3 7.7	 6.0	 6 20	 1.0 2.0	 * *	 0.04 0.02	 0.05		 8.4 8.3	 70 144	 156 164		
9	11	1102.85	1/17 5/24 5/30	1097.17 1099.55 1099.14	100	8.1	2.0	10	0.8	0.01	0.16	0.09		4.9	28	48		‡
10	26	1091.71	1/18 5/24	1072.21 1072.80	154	8.6	0.5	1	0.2	*	*	*		8.8	72	88		

(continued)

TABLE 2-4. RESEARCH MONITORING WELL DATA FROM 1990, STEVENS POINT, WISCONSIN (continued)

MW	Well Information				Water Quality Parameters													
							(mg/l)										(µg/l)	
	DOW (ft)	MP Elev (ft)	Date	SWL Elev (ft)	Cond (µS/cm)	pH	Na	Cl	N	Mn	Fe	Zn	DO	COD	Alk	Hard	VOC	Pest
11	34	1107.05	1/18	1078.27														
			1/24	1078.25	420	7.2	6.0	15	1.0	0.01	*	0.07		12.4	96	216		
			5/24	1078.44														
			6/29	1078.99	358	7.6		21	*	*	0.04			10.4	176	196		
12	35	1103.81	1/10	1071.98														
			2/12	1071.70	435	7.1	4.0	13	6.0	0.01	0.08	0.10		17.2	152	220		
			4/13	1072.38													*b	
			5/24	1072.48														
13	33	1105.52	1/10	1076.98														
			2/12	1076.50	630	7.0	12.0	73	16.0	0.04	0.14	0.02		24.1	112	260		
			5/24	1077.27														
			6/11	1077.88	700			87	14.8	*	0.26			9.4	92	276		†
14	28	1091.55	1/10	1069.08	325	7.1	31.0	47	3.5	*	*	0.01		9.8	64	88		
			4/13	1069.43														
			5/24	1069.40														
15	34	1099.24	1/10	1070.40														
			1/23	1070.31	522	6.6	56.0	61	5.0	*	0.03	0.03		15.7	136	124		
			4/13	1070.71													ND	
			5/24	1070.78														
			5/30	1070.88	420	6.3		42	3.0	*	0.02			9.9	108	120		
16	33	1101.34	1/10	1072.98														
			1/23	1072.89	970	6.8	17.0	7	1.5	*	0.15	0.03		17.0	24	12		
			5/24	1073.75														
			6/27	1074.42	98	6.9											ND	
17	35	1098.49	1/9	1069.14	332	7.4	32.0	38	3.5	*	0.02	*		8.8	68	88		
			5/24	1069.43														
18	36	1099.45	1/10	1069.30	175	7.3	2.0	3	1.8	*	*	0.02		3.4	68	92		
			4/13	1069.49													ND	
			5/24	1069.62														
19	32	1103.90	1/10	1078.40	645	7.1	28.0	76	9.0	0.80	*	0.01		5.9	140	236		
			6/27	1079.02	697	7.1		78	6.8	0.84	0.18			8.7	140	244		

(continued)

TABLE 2-4. RESEARCH MONITORING WELL DATA FROM 1990, STEVENS POINT, WISCONSIN (continued)

MW	Well Information				Water Quality Parameters													
							(mg/l)										(µg/l)	
	DOW (ft)	MP Elev (ft)	Date	SWL Elev (ft)	Cond (µS/cm)	pH	Na	Cl	N	Mn	Fe	Zn	DO	COD	Alk	Hard	VOC	Pest
20	34	1096.65	1/9	1068.53	290	6.8	4.0	14	11.5	*	*	*		11.3	60	128		
			4/12	1068.66													ND	
			5/24	1068.75														
			5/30	1068.66														‡
			6/29	1068.83	182	8.1												
21	27	1100.94	1/10	Dry														
22	29	1103.82	1/10	1079.02	405	7.0	5.0	17	24.2	*	*	0.01		3.4	68	172		
			5/24	1078.93														
			6/27	1079.35	624	6.9		29	31.5	*	*			49.3	68	236		
23	69	1080.96	1/17	1068.91	539	7.9	3.0	13	0.2	0.01	0.05	1.00		110.0	192	240		
			4/20	1069.27													1.7c	
			5/1	1068.58														
			5/3	1068.66													543d	
24	40	1104.17	1/30		122	7.8	2.0	2	0.5	*	0.04	0.05		5.4	80	80		
			2/6	1069.17														
25	60	1105.30	1/18	1069.97														
			1/30	1069.92	159	7.5	4.0	6	1.5	0.02	0.08	0.02		3.9	72	80		
			2/6	1069.83														
	61		4/12	1069.98													ND	
			5/24	1070.89														
			6/26	1070.27	208	7.8		4	1.5	0.01	0.11			*	56	76		
26	33	1102.92	1/18	1072.74														
			2/1	1072.86	135	7.4	1.0	1	*	*	0.04	0.02		5.9	76	80		
			5/24	1073.07														
27	41	1108.54	1/17	1084.54	272	7.1	4.0	11	1.2	*	*	0.02		3.4	116	140		
			4/13	1084.01													ND	
			5/24	1084.23														
28	13	1096.31	1/19	1089.14	725	7.3	86.0	160	3.0	*	0.12	0.07		12.8	44	112		
			4/20	1088.97													ND	
			6/26	1090.79	915	6.5		160	3.0	0.40	4.66			30.2	56	116		
29	25	1087.45	1/9	1067.89	695	6.6	124.0	125	2.5	*	0.19	*		26.0	84	44		
			4/12	1067.97													ND	
			5/24	1068.32														

(continued)

TABLE 2-4. RESEARCH MONITORING WELL DATA FROM 1990, STEVENS POINT, WISCONSIN (continued)

MW	Well Information				Water Quality Parameters													
							(mg/l)										(µg/l)	
	DOW (ft)	MP Elev (ft)	Date	SWL Elev (ft)	Cond (µS/cm)	pH	Na	Cl	N	Mn	Fe	Zn	DO	COD	Alk	Hard	VOC	Pest
30	39	1093.88	1/19 1/25 5/24	1064.58 1064.38 1067.66	395	7.7	41.0	53	2.5	*	0.03	0.03		6.4	132	128		
31	48	1093.86	1/19 1/25 5/24	1064.53 1064.34 1067.64	329	8.1	16.0	53	2.5	*	0.05	0.03		5.4	68	128		
32	9	1070.86	1/25 4/13 5/24	1065.94 1066.65 1067.04	332	5.7	23.0	100	*	0.53	17.60	0.11		17.2	36	112	ND	
33	35	1095.06	1/19 5/24 6/27	1067.17 1068.49 1067.87	675 1032	7.0 6.6	76.0	110	2.2	*	0.04	0.04		7.9	116	148	 ND	
34	34	1097.16	1/19 1/31 5/24	1066.31 1066.40 1067.83	460	5.9	23.0	40	8.0	1.25	0.06	0.28		13.2	64	140		
35	38	1096.90	1/19 1/31 5/24	1065.31 1065.67 1067.65	370	7.0	48.0	52	1.2	0.02	0.12	0.08		7.8	68	64		
36	39	1090.00	4/5 5/1 5/24 5/30	1067.92 1069.35 1069.77 1068.40	154	7.6		1	0.2	*	*			4.2	72	80		
37	33	1085.86	4/9 5/1 5/24 5/30	1068.56 1069.37 1069.79 1068.64	98	7.1		1	0.2	*	*			*	48	64		
38	43	1093.16	4/11 5/1 5/24 5/30	1068.11 1069.34 1069.87 1068.82	183	7.6		1	*	0.02	0.02			2.6	100	112		
39	9	1072.69	5/31 6/5	1070.18 1070.72														

(continued)

TABLE 2-4. RESEARCH MONITORING WELL DATA FROM 1990, STEVENS POINT, WISCONSIN (continued)

MW	Well Information				Water Quality Parameters													
							(mg/l)										(µg/l)	
	DOW (ft)	MP Elev (ft)	Date	SWL Elev (ft)	Cond (µS/cm)	pH	Na	Cl	N	Mn	Fe	Zn	DO	COD	Alk	Hard	VOC	Pest
40	9	1073.10	5/31	1070.28														
			6/5	1070.36														
41	10	1071.27	5/31	1069.55														
			6/4	1069.57														
			10/22	1069.61	290	6.4	4.0	12	7.7	0.44	*			10.4	160	96		
42	20	1071.28	6/4	1069.57														
43	10	1075.44	6/1	1073.63														
A20	21	1080.27	1/17	1071.80	361	7.5	3.0	6	*	0.03	0.60	0.10		3.4	164	180		
			5/24	1072.48														
			6/6	1072.12	338	7.8		5	*	0.02	0.59		0.3	12.5	180	184		
A40	41	1080.53	1/17	1071.87	392	7.4	3.0	7	*	0.02	0.24	0.08		*	172	188		
			5/24	1072.51														
			6/6	1072.16	378	7.8		6	*	0.01	0.17		0.2	8.9	184	194		
A60	61	1080.35	1/17	1071.85	419	7.3	3.0	8	0.2	*	0.05	0.18		4.9	180	204		
			5/24	1072.45														
			6/6	1072.17	357	7.7		7	0.5	*	*		0.3	9.9	180	188		
B20	21	1082.31	1/16	1069.86	330	7.7	2.0	4	0.2	*	*	0.02		4.5	160	180		
			5/24	1070.43														
			6/5	1070.48	227	7.9		3	0.2	0.05	0.10		5.8	3.7	176	182		
B40	40	1082.05	1/16	1069.82	380	7.6	2.0	9	1.2	1.04	*	0.01		10.9	152	192		
			5/24	1069.78														
			6/5	1070.45	228	7.3		5	0.5	0.98	0.06		0.5	15.6	160	180		
B60	68	1082.12	1/16	1069.82	470	7.2	2.0	13	*	0.04	*	0.01		*	172	232		
			5/24	1070.39														
			6/5	1070.43	280	7.2		10	*	0.03	*		0.4	5.2	176	210		
C20	21	1082.20	1/16	1075.46	137	8.3	1.0	1	2.0	*	*	0.01		*	48	68		
			5/24	1076.49														
			6/11	1075.68	132			1	1.0	*	0.02			*	32	68		†
C40	40	1081.60	1/16	1075.50	407	7.3	2.0	16	7.0	*	*	0.02		9.4	152	212		
			5/24	1076.49														
			6/11	1075.69	442			14	6.8	0.02	0.08			6.2	160	212		ND

(continued)

TABLE 2-4. RESEARCH MONITORING WELL DATA FROM 1990, STEVENS POINT, WISCONSIN (continued)

MW	Well Information				Water Quality Parameters													
							(mg/l)										(µg/l)	
	DOW (ft)	MP Elev (ft)	Date	SWL Elev (ft)	Cond (µS/cm)	pH	Na	Cl	N	Mn	Fe	Zn	DO	COD	Alk	Hard	VOC	Pest
C60	59	1082.10	1/16	1075.33	490	7.3	2.0	20	8.2	*	0.03	0.03		10.3	172	260		
			5/24	1076.34														
			6/11	1075.60	559			20	8.0	0.02	0.18			18.7	176	248		†
D20	19	1083.34	1/16	1074.92	86	7.2	1.0	1	0.8	*	0.02	0.01		5.0	28	44		
			5/24	1076.06														
			6/11	1075.20	89			1	0.5	*	0.03			4.2	28	40		†
D24	24	1083.40	1/16	1074.93	372	6.9	2.0	14	5.8	0.16	*	0.03		5.4	140	196		
			5/24	1075.81														
			6/11	1074.98	410			13	4.0	0.15	0.02			9.4	152	196		‡
D60	60	1083.46	1/16	1074.93	480	6.8	2.0	16	7.2	0.08	*	0.02		6.4	140	208		
			5/24	1075.56														
			6/11	1074.67	414			16	6.5	0.08	0.09			7.8	152	208		‡
E5	7	1075.86	1/16	1072.16	269	7.4	1.0	4	1.2	0.02	0.06	0.03		36.2	120	160		
			5/24	1072.87														
			6/5	1072.78	150	6.7		3	3.0	0.02	*		0.4	20.3	80	108		
E10	12	1076.37	1/16	1072.97	300	7.5	1.0	11	*	0.05	0.04	0.11		48.5	144	176		
			5/24	1073.68														
			6/5	1073.59	140	7.4		3	*	0.01	0.06		0.2	30.2	84	106		
E37	37	1076.03	1/16	1072.95	430	7.4	2.0	13	3.0	0.06	0.02	0.08		10.9	160	220		
			5/24	1073.65														
			6/5	1073.79	289	7.4		12	4.5	0.07	0.06		0.2	5.2	176	222		
			6/29	1073.68	372	7.4												
F5	7	1075.10	1/16	1070.40	355	8.0	0.5	2	2.0	0.01	0.14	0.01		*	68	72		
			5/24	1071.40														
			6/6	1070.68	336	7.3							0.4					
F10	11	1074.14	1/16	1070.42	359	7.9	3.0	9	*	0.03	0.21	0.16		6.8	188	212		
			5/24	1071.40														
			6/6	1070.67	288	8.0							0.4					
G6	8	1075.29	1/16	1071.89	260	6.4	2.0	4	*	0.19	4.80	0.04		27.2	108	136		
			5/24	1072.81														

(continued)

TABLE 2-4. RESEARCH MONITORING WELL DATA FROM 1990, STEVENS POINT, WISCONSIN (continued)

MW	Well Information				Water Quality Parameters													
							(mg/l)										(µg/l)	
	DOW (ft)	MP Elev (ft)	Date	SWL Elev (ft)	Cond (µS/cm)	pH	Na	Cl	N	Mn	Fe	Zn	DO	COD	Alk	Hard	VOC	Pest
IM1	40	1088.14	1/9 5/24 6/27	1067.88 1068.26 1067.85	725 672	6.7 7.1	88.0	115	2.2	*	*	*		11.8	80	128		
IM1	55	1087.89	1/9 5/24	1067.93 1068.32	450	7.3	15.0	34	6.2	*	*	*		16.2	124	192		
IM1	85	1088.11	1/9 5/24 6/27	1067.89 1068.27 1067.83	425 449	7.7 7.4	7.0	17	6.5	*	*	*		4.9	140	208		
IM1	116	1087.94	1/9 5/24	1067.91 1068.24	490	7.7	11.0	20	5.8	*	*	*		4.4	168	236		
IM1	130	1087.91	1/9 6/27 6/29	1067.94 1067.81 1067.85	495 408	7.8 7.4	11.0	24	6.0	*	*	*		8.8	172	240		
IM1	155	1088.09	1/9	1067.97	322	7.8	1.0	9	3.5	*	*	0.01		7.8	120	160		
J37	37	1109.73	6/18 10/11	1077.44 1077.03	228	7.1	2.0	2	0.6	0.02	0.06		1.2	*	92	120		
J59	59	1109.72	6/18 10/11	1077.46 1077.02	74	6.3	1.0	3	1.8	*	*		2.0	*	20	32		
J78	78	1109.92	6/18 10/11	1077.47 1077.02	470	6.4	3.0	24	11.3	*	*		6.4	6.7	184	244		
J118	118	1109.49	6/20 10/11	1077.29 1076.39	318	7.1	3.0	5	0.3	0.01	0.02		6.4	5.1	188	220		
J138	138	1109.26	6/20 10/11	1077.32 1075.94	390	7.0	2.0	8	*	0.02	2.88		2.0	6.7	172	176		
J160	160	1109.54	6/20 10/11	1077.32 1077.06	330	6.4	1.5	8	*	*	3.45		2.0	7.7	168	176		
K37	37	1104.35	6/19 7/5 10/17	1070.71 1070.19 1069.64	322	7.3	2.0	9	0.3	0.30	0.06				160	164		

(continued)

TABLE 2-4. RESEARCH MONITORING WELL DATA FROM 1990, STEVENS POINT, WISCONSIN (continued)

MW	Well Information				Water Quality Parameters													
							(mg/l)										(µg/l)	
	DOW (ft)	MP Elev (ft)	Date	SWL Elev (ft)	Cond (µS/cm)	pH	Na	Cl	N	Mn	Fe	Zn	DO	COD	Alk	Hard	VOC	Pest
K58	59	1105.15	6/19 7/5 10/15	1070.65 1070.15 1070.30	215	7.6	1.0	*	1.6	*	0.24			*	108	120		
K79	79	1105.52	6/19 7/5 10/15	1070.72 1070.17 1070.30	285	7.5	12.0	33	1.8	*	0.23			*	76	112		
L30	30	1088.43	6/22 7/5 10/17	1068.64 1068.65 1069.56	340	7.3	13.0	31	2.3	0.05	0.04			4.1	92	124		
L58	58	1087.92	6/22 7/5 10/12	1068.64 1068.64 1069.46	228	7.4	8.0	18	0.7	*	0.08			*	76	92		
L96	96	1088.20	6/22 7/5 10/12	1068.64 1068.62 1069.41	235	7.4	3.0	10	*	0.02	0.06			*	96	116		
M35	35	1100.36	6/26 10/17	1070.25 1071.80	130	7.1	2.0	*	1.7	*	0.03			4.6	48	56		
M45	45	1100.46	6/26 10/15	1070.10 1071.74	97	5.5	11.0	13	0.7	* 0.01	0.32			*	4	16		
M63	63	1100.53	6/26 10/15	1070.58 1071.76	215	6.4	4.0	9	*	0.21	6.30			*	76	84		
MINIMUM DETECTION LIMITS					N/A	N/A	1.0	1	0.2	0.01	0.02	0.01	0.1	3.0	2	2	N/A	N/A

Abbreviations: MW = monitoring well; DOW = depth of well; MP Elev = measuring point elevation; SWL Elev = static water level elevation; Cond = specific conductivity; DO = dissolved oxygen; COD = chemical oxygen demand; Alk = alkalinity; Hard = hardness; VOC = volatile organic compound; Pest = pesticide; ND = not detected; N/A = not applicable.

* = below minimum detection limit.

*a = below minimum detection limit of 0.6 µg/l for chloroform.

*b = below minimum detection limit of 1.1 µg/l for 1,1,2-trichloroethane.

c = analysis for trans-1,2-dichloroethylene.

d = analysis for total petroleum hydrocarbons.

† = nitrogen- and phosphorous-based organic analysis; small unidentified peaks.

‡ = nitrogen- and phosphorous-based organic analysis; large unidentified peak, later identified as DEET (DEET introduced to sample during acquisition owing to use of insect repellant by operators).

Drawdown and recovery testing at CW6 was performed in conjunction with the University of Wisconsin - Madison. The drawdown test was performed using methods outlined by the U.S. Geological Survey (USGS, 1977). City well 6 was chosen for drawdown and recovery testing because it has the pumping capacity to produce a significant drawdown; it is located at the Airport wellfield, and it has a convenient observation well nearby, MW23. Three additional observation wells, MW36 through MW38, were installed around CW6 for this aquifer test. At the end of the drawdown portion of the test, water-level recovery rates were measured.

Chemical Parameters

Ground-water sampling for laboratory analysis was conducted using the methods established by the Wisconsin Department of Natural Resources (1987). The samples were analyzed by state-certified laboratories. The majority of samples were analyzed at the Environmental Task Force Laboratory at the University of Wisconsin - Stevens Point; a few of the organic analyses were conducted by Enviroscan in Wausau, Wisconsin.

The first set of ground-water samples was collected in January and February 1990 (Table 2-4). Ground-water samples from all the existing wells were analyzed for specific conductivity (Cond), pH, sodium (Na), chloride (Cl), nitrate and nitrite as nitrogen (N), manganese (Mn), iron (Fe), zinc (Zn), chemical oxygen demand (COD), alkalinity (Alk), and hardness (Hard). Some of these parameters were selected to screen for potential contamination problems, including Na, Cl, N, Fe, Mn and Zn. Others were selected to maintain continuity with the existing data base (COD and Hard) and to serve as indicators of general water quality (Cond, Alk, and pH).

For the second set of samples collected in May and June 1990, select wells were chosen for VOC and pesticide analyses (Pest) (Table 2-4). Dissolved oxygen (DO) was also added as a field-monitored parameter for select wells during the second sampling round. Wells were selected for sampling if they were positioned downgradient from or near a potential source. For example, MW1, MW2, MW6, MW17, and MW18 are located in areas where there are numerous, small gasoline USTs; MW12, MW15, MW16, MW25, and MW27 are downgradient from large, commercial USTs at gasoline stations or the airport; and MW16, MW20, and MW28 are located in areas with dry wells. Some wells located at the outside edge of the monitoring network, closest to the agricultural areas, were sampled and analyzed for nitrogen- and phosphorous-based pesticides. A nitrogen- and phosphorous-based pesticide analysis was chosen because many of the pesticides used in this area are based in these chemicals, including atrazine, alachlor, metolachlor, and metribuzen. Also, during the second set of tests, samples from selected wells were analyzed for the January parameter list, with the exception of zinc and sodium. Wells selected for the second round of sampling included those with high concentrations of a particular substance and those in optimal locations. Fewer wells were sampled and fewer parameters were analyzed because of project budget limitations.

For the third set of analyses, during October 1990, the new well nests were sampled for the inorganic parameters listed in the previous sample set (Table 2-4). The J nest wells were also sampled for the nitrogen- and phosphorous-based pesticide analysis. The shallow wells in the K, L, and M nests were sampled for VOCs.

DATA INTERPRETATION

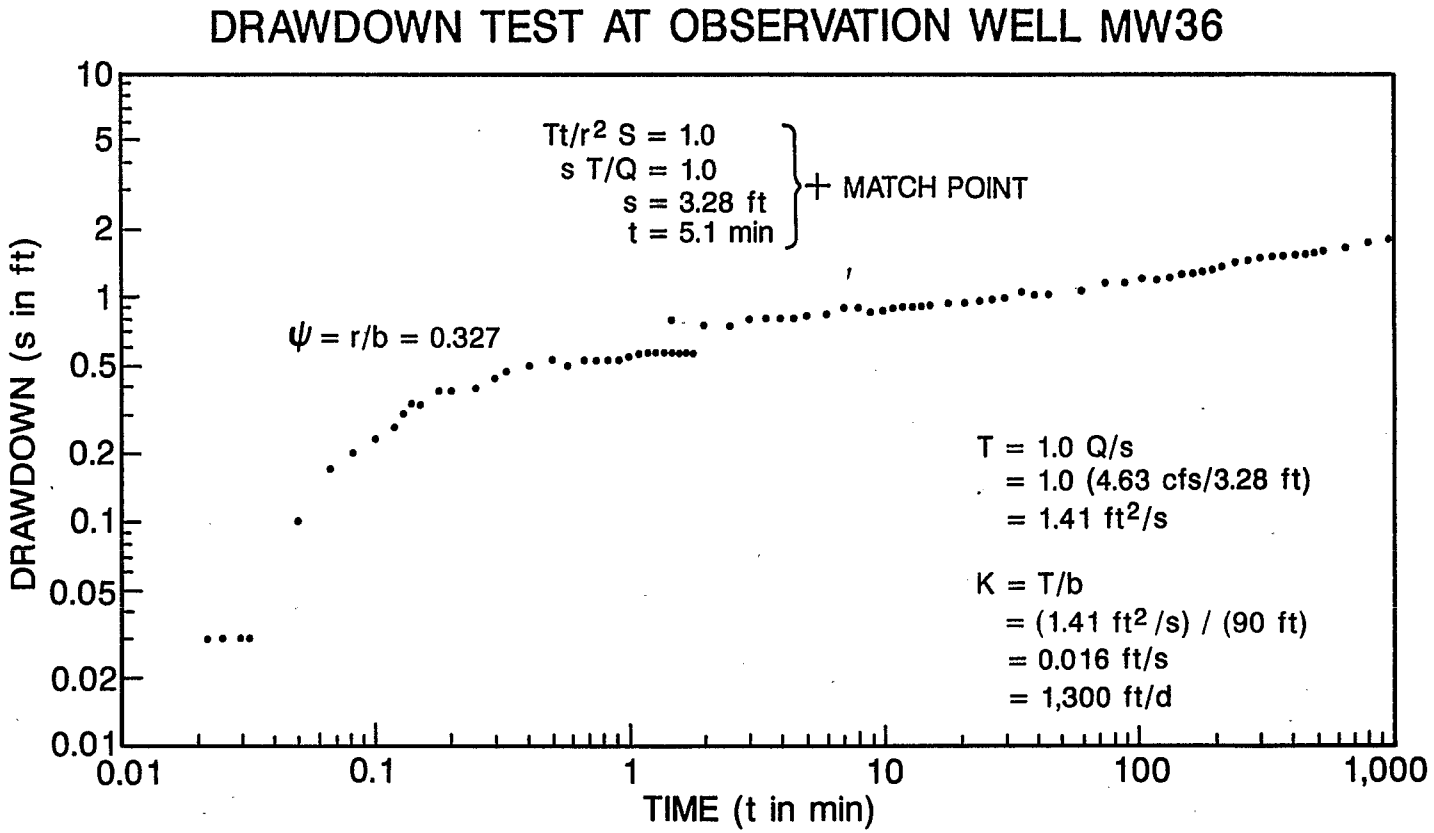
Aquifer Parameters

The drawdown test at CW6 on June 5, 1992, and observation data from MW36 confirmed the results of the aquifer test by Hickock and Associates (1965). Analysis of a logarithmic plot of drawdown versus time shown in Figure 2-11 (Neuman, 1974) indicates an aquifer transmissivity (T) of 1.41 ft^2/s (Bahr, 1990). The semi-log plot analysis of drawdown versus time for MW36 observation data shown in Figure 2-12 (Cooper and Jacob, 1946) yields a T value of 1.10 ft^2/s (Bahr, 1990). Given that the saturated aquifer thickness (b) at CW6 is 90 feet and that the hydraulic conductivity (K) is equal to T/b, then the respective K values for the logarithmic and semi-log analyses are 1,300 and 1,100 ft/d. An average K value of 1,200 ft/d was chosen to represent the buried valley aquifer at CW6 (Table 2-5).

Slug-test data were interpreted using Horslev's (1951) empirical method. Slug tests performed at MW36 yielded a K estimate of 1,100 to 1,700 ft/d (Figure 2-13; Table 2-5). The range in this estimate results from uncertainty in the results of the slug test. The high degree of uncertainty is due to the very rapid response of the aquifer to the displacement of fluid by the slug. Three sets of slug-in and slug-out tests were conducted at MW36 (Figure 2-13). Of these three sets, one slug-in test was timed correctly. The water levels in MW36 recovered completely in less than 0.04 minute, or approximately 2.4 seconds. The highest estimate of hydraulic conductivity (1,700 ft/d) is obtained from the slope of the early-time data (straight line) at a t_0 of 0.0016 minute: that is, within the first 0.01 minute of the test. The lower hydraulic conductivity (1,100 ft/d) estimate is obtained at a t_0 of 0.0025 minute using a power regression fit (curved line) to the time data points up to and including 0.07 minute. After 0.01 minute, the water-level data begin to fluctuate. Despite the uncertainty in the results, the slug test at MW36 yields a range of hydraulic conductivity values from 1,100 to 1,700 ft/d that bracket the hydraulic conductivity obtained for the drawdown test of 1,200 ft/d.

Similar problems occurred with the slug testing at well nest J. Four sets of slug tests were conducted at well J160. Two sets of tests were conducted at well J118 and two sets at well J78. Of these eight total tests, only one, a slug-in test at J160, was timed properly. In this test, water levels dropped to within 20% of the maximum displacement in less than 0.005 minute (Figure 2-14). When the slope of the earliest-time data points is used, an estimate of hydraulic conductivity of 950 ft/d is derived. From the later time data, a value of 820 ft/d is derived. This value indicates that the buried channel has a characteristically higher hydraulic conductivity than the adjacent aquifer material, as indicated by previous drawdown results. (The reader will note that well nest J is positioned near the center of the buried valley.)

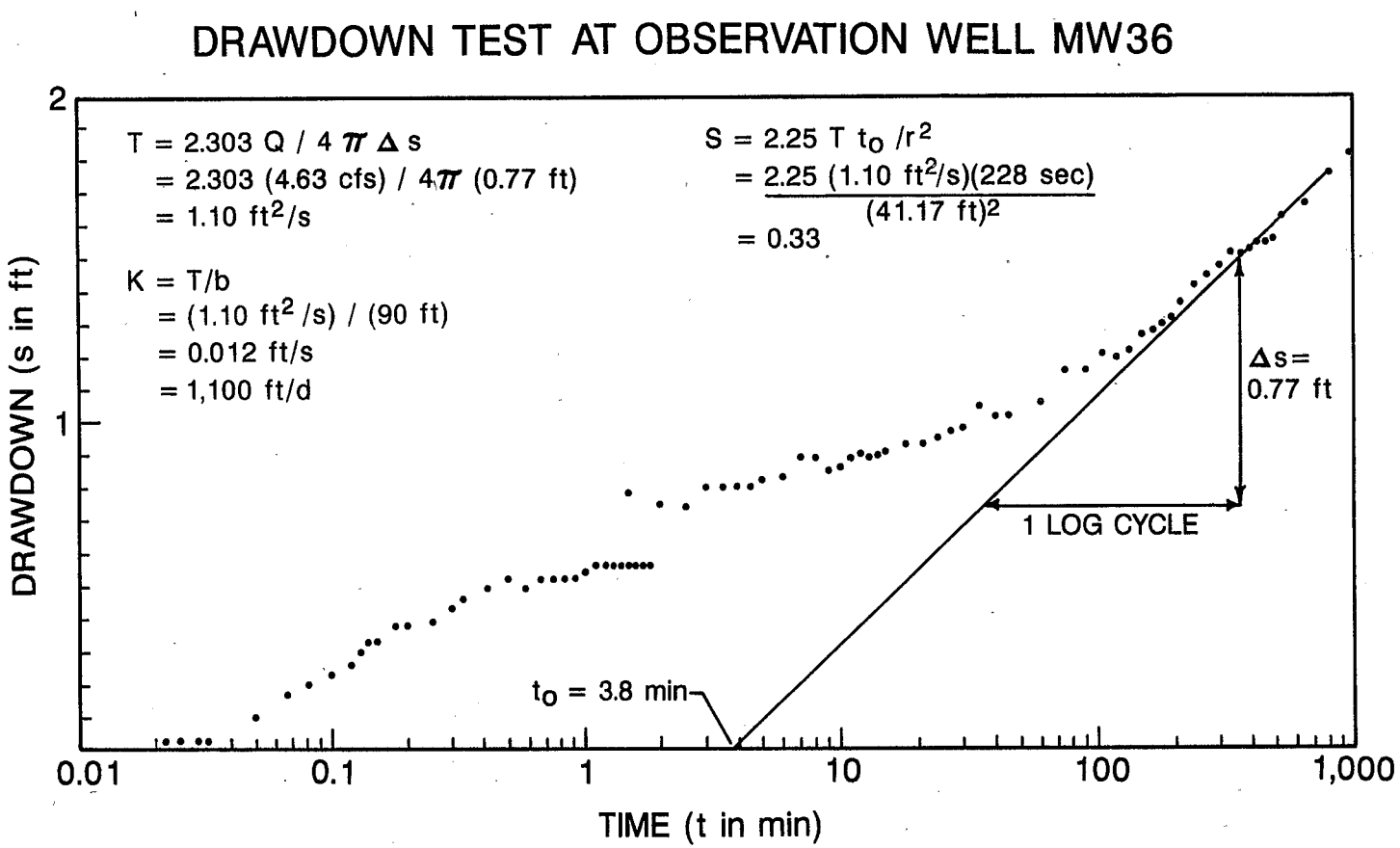
The slug-test results from well nest K (Figure 2-15) and from MW13 (Figure 2-16) are considered more reliable because the aquifer did not recover as rapidly and the data are more accurate. Slug tests at well K58 yielded a range of hydraulic conductivities from 220 to 260 ft/d for late- and early-time data, respectively. This hydraulic conductivity range (Table 2-5) is typical for the range reported for outwash deposits in other studies, from 100 to 500 ft/d (Faustini, 1985; Rothchild, 1982;



WHERE:

DISTANCE FROM CW6 TO MW36	= r = 41.17 ft
WELL DISCHARGE	= Q = 4.63 cfs
TRANSMISSIVITY	= T (ft^2/s)
SPECIFIC YIELD	= S = 0.25
HYDRAULIC CONDUCTIVITY	= K (ft/s)
AQUIFER THICKNESS	= b = 90 ft

Figure 2-11. Logarithmic plot of drawdown versus time in observation well MW36 during drawdown testing at city well 6 on June 5, 1990, Stevens Point Wisconsin (Bahr, 1990). [Observation well MW36 at Z=0.5b (45 ft); match point from type curve for well penetrating 0.3 lower portion of unconfined aquifer.]



WHERE: DISTANCE FROM CW6 TO MW36 = r = 41.17 ft
WELL DISCHARGE = Q = 4.63 cfs
TRANSMISSIVITY = T (ft^2/s)
SPECIFIC YIELD = S
HYDRAULIC CONDUCTIVITY = K (ft/s)

Figure 2-12. Semi-log plot of drawdown versus the log of time at observation well MW36 during drawdown testing at city well 6 on June 5, 1990, Stevens Point, Wisconsin (Bahr, 1990).

TABLE 2-5. AQUIFER TEST RESULTS FOR GEOLOGIC TERRAINS WITHIN THE SAND AND GRAVEL AQUIFER

Well	Terrain	Aquifer Test	K (ft/d)
CW6/MW36	Buried valley	Drawdown -- logarithmic	1,300
		Drawdown -- semi-log	1,100
		Average	1,200
MW36	Buried valley	Slug -- late time	1,100
		Slug -- early time	1,700
J160	Buried valley	Slug -- late time	820
		Slug -- early time	950
K58	Outwash	Slug -- late time	220
		Slug -- early time	260
MW13	Bedrock high	Slug -- late time	2.1
		Slug -- early time	3.5

Holt, 1965; Jones, 1987). Three slug tests at MW13 resulted in a hydraulic conductivity range of 2.1 to 3.5 ft/d for the bedrock high area (Figure 2-16). During purging of the monitoring wells, significant variability in hydraulic conductivity was noted in association with slower water-level recovery rates in some wells. Slug tests quantified these observations.

The hydraulic conductivity values calculated from the results of the drawdown and slug tests are summarized in Table 2-5. The lateral variation in hydraulic conductivity values indicates that the system is not as homogeneous as previously assumed due to the presence of distinct geologic terrains within the sand and gravel aquifer, including the buried valley, bedrock highs, and outwash plains. The aquifer-test results were incorporated in the model of ground-water flow used to refine the WHPA and to optimally site monitoring wells.

Hydrochemical Assessment

The results of the hydrochemical analyses for the monitoring wells are compiled in Table 2-4. These data provide the city with the foundation for long-term ambient trend monitoring. Because these are the initial hydrochemical data in the data base, the analysis of seasonal trends is premature and the analysis of long-term trends is unfeasible. However, the hydrochemistry of the city wells has been monitored over a long period of time, and some trends are evident.

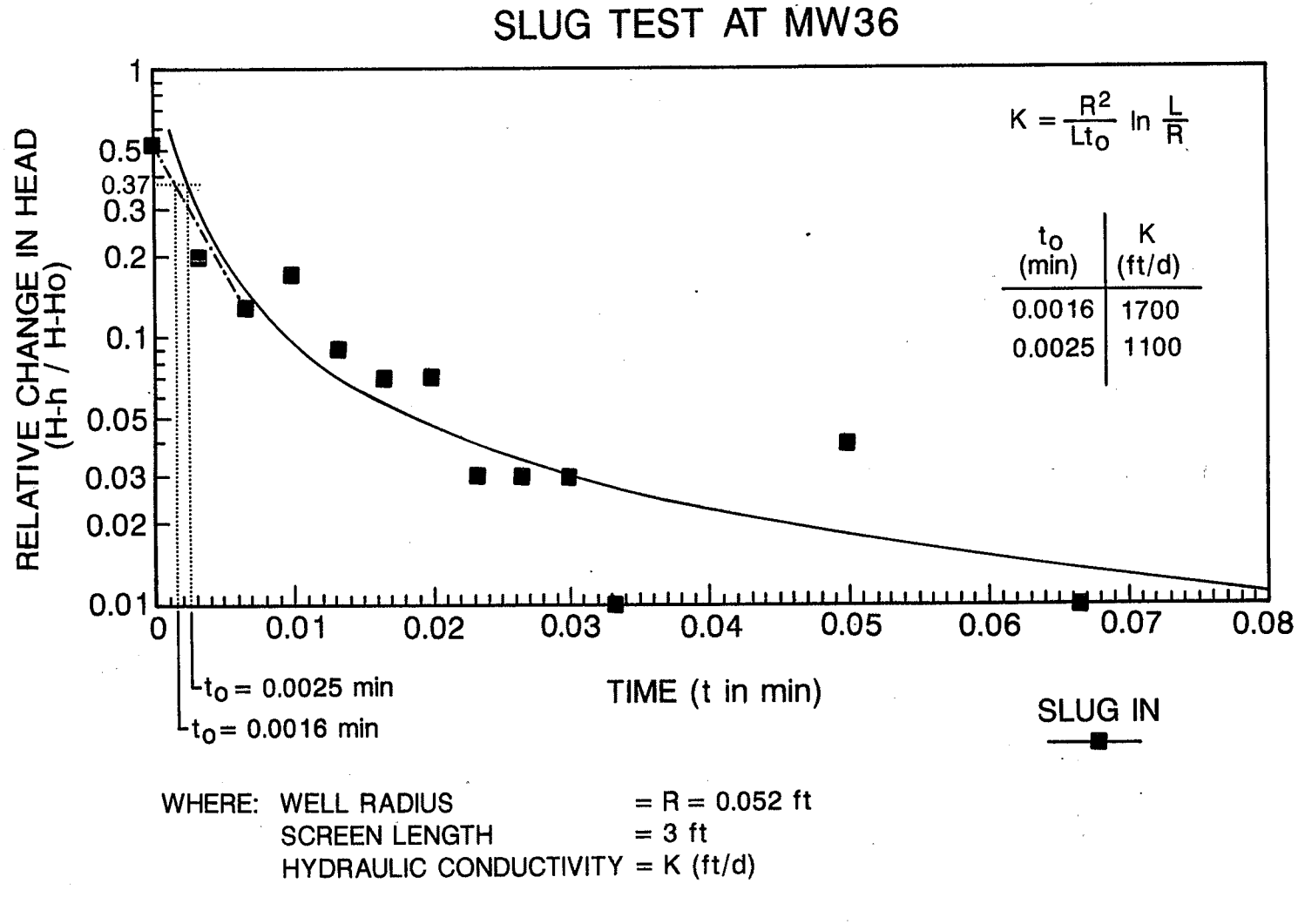


Figure 2-13. Semi-log plot of the log of the relative change in head versus time for slug testing at well MW36 on July 18, 1990, Stevens Point, Wisconsin.

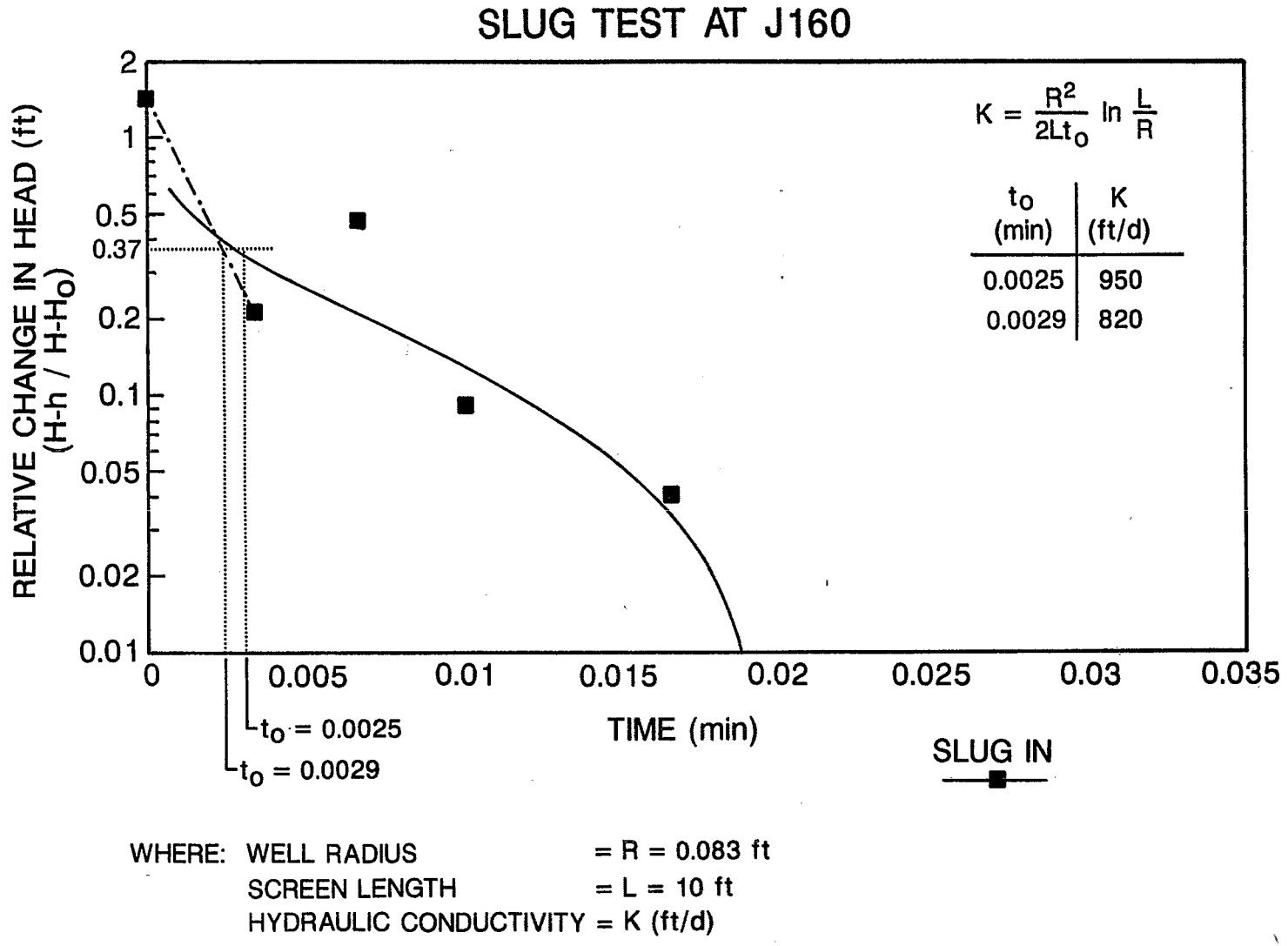


Figure 2-14. Semi-log plot of the log of the relative change in head versus time for slug testing at well J160 on July 18, 1990, Stevens Point, Wisconsin.

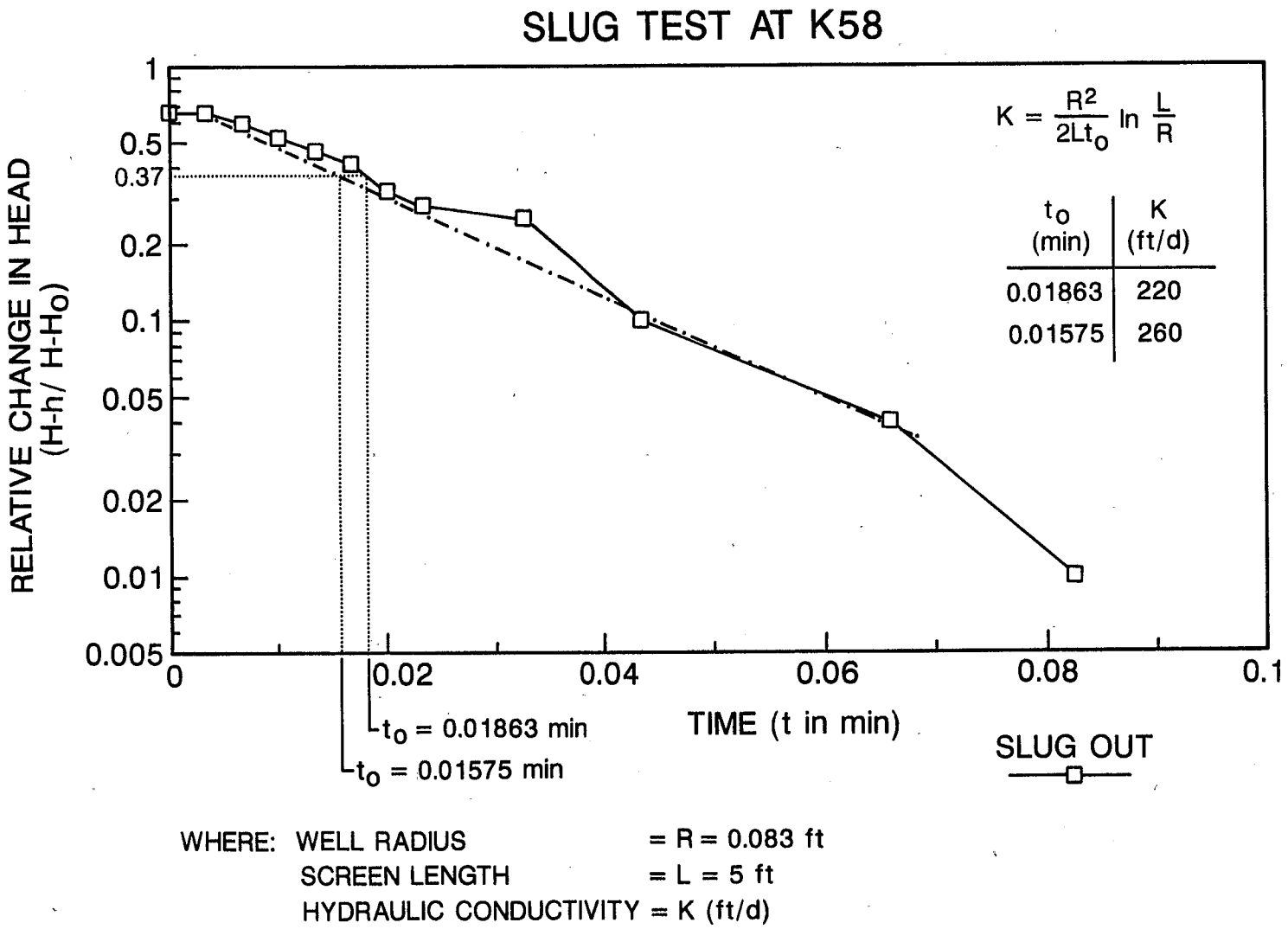
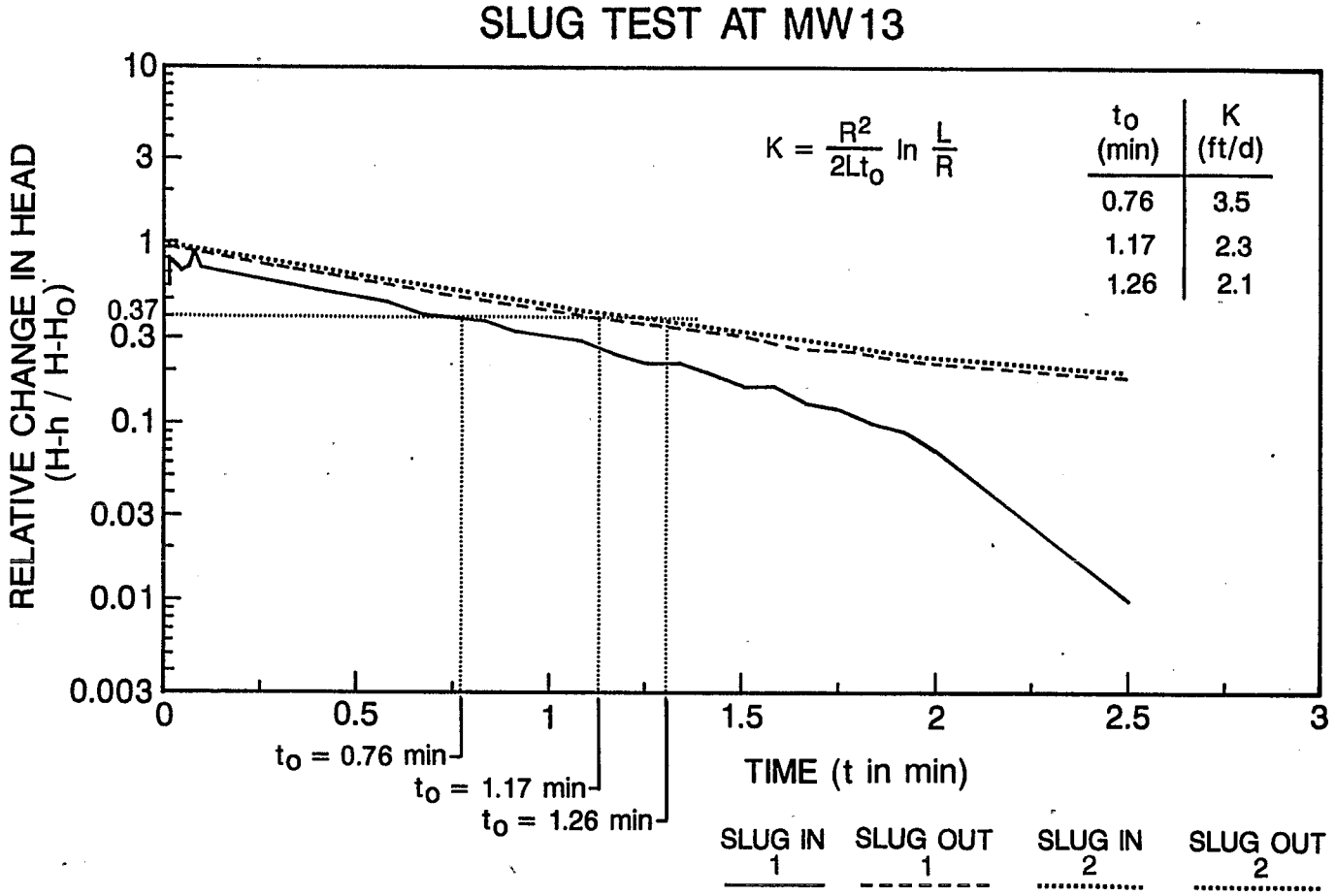


Figure 2-15. Semi-log plot of the log of the relative change in head versus time for slug testing at well K58 on July 17, 1990, Stevens Point, Wisconsin.



WHERE: WELL RADIUS = R = 0.052 ft
SCREEN LENGTH = L = 3 ft
HYDRAULIC CONDUCTIVITY = K (ft/d)

Figure 2-16. Semi-log plot of the log of the relative change in head versus time for slug testing at well MW13 on July, 18, 1990, Stevens Point, Wisconsin.

Nitrogen (nitrate and nitrite as nitrogen) concentration trends in city wells 6 through 9 from 1984 to 1990 are depicted in Figure 2-17. Although the pattern and concentration for each well differ and seasonal trends are difficult to discern without further analysis, there is a general increase in the concentration of nitrogen over time. In fact, since 1984, the peak concentrations in nitrogen have increased in all the city wells. Currently, nitrogen concentrations in the monitoring wells range from less than 0.2 mg/l, in several wells, to 26 mg/l in MW2 (Table 2-4). Monitoring well 2 is a shallow well located downgradient from a nonsewered trailer park (Figure 2-9). The location of monitoring wells in which the ground water exceeds the State Preventive Action Limit (PAL), or drinking water standard, for nitrogen are shown in Figure 2-18. All of these wells are shallow wells, either directly downgradient from high-density, nonsewered residential areas or agricultural activities.

Nitrogen concentrations also vary with depth, as shown in Figure 2-19. The monitoring wells with the highest nitrogen concentrations are shallow wells. In contrast, the deeper monitoring points in the multi-level well, with the exception of IM1-155, have nitrogen concentrations between 5 and 10 mg/l. Assuming that the hydrochemistry of the deeper ground water represents a more regional flowpath, the nitrogen in this portion of the aquifer may be derived from distant agricultural areas. The lower concentration of nitrogen in IM1-155 may be a result of dilution or alteration.

Monitoring wells with high concentrations of iron, manganese, and chloride are shown in Figure 2-18. The majority of the monitoring wells exceeding drinking water standards for iron and manganese are in wetland areas with organic-rich soils located along the Plover River. The reducing conditions associated with these environments increases metal solubility and may cause higher concentrations of iron and manganese in ground water. All of the monitoring wells with elevated chloride levels are shallow wells located near roads where de-icing salts are used during the winter.

Only one ground-water sample, from MW23, showed detectable levels of organic compounds when analyzed for VOCs. Monitoring well 23 is a hand-driven, galvanized well point--one of the test wells installed in the 1960s to investigate the Airport wellfield site. The source of the VOCs in this well is believed to be a coating that was applied to the inside of the pipe by the manufacturer. With the exception of DEET (the active ingredient in mosquito repellent used by the samplers), no nitrogen- or phosphorous-based pesticides commonly used in this area (for example, atrazine, alachlor, metolachlor, and metribuzin) were detected in the research monitoring wells.

Ground-Water Flow Modeling

The two-dimensional, ground-water flow model FLOWPATH (Franz and Guiguer, 1989) was used to refine the delineation of the 5-year TOT zone. The area of interest, shown in Figure 2-5 (dashed line), was modeled using the hydrologic boundaries to the east and west (no-flow boundaries) and constant-flux boundaries to the north and south. The constant-flux boundaries were determined by using Lippelt and Hennings' (1981) regional water-table map and by calculating the component of flow perpendicular to the boundary, based on the hydraulic gradient and the hydraulic conductivity.

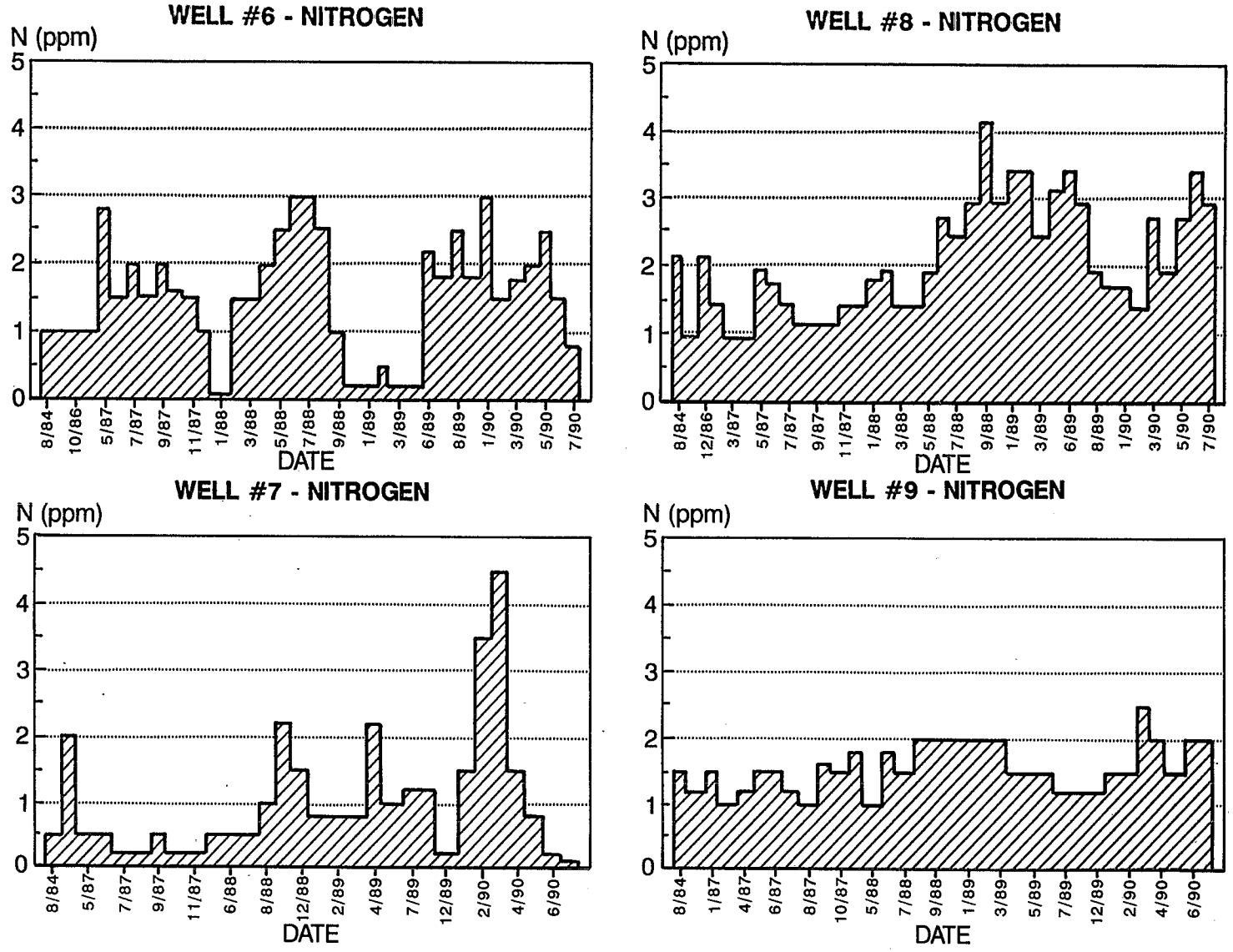


Figure 2-17. Nitrogen (nitrite and nitrate) concentrations at city wells 6 through 9 from 1984 to 1990, Stevens Point, Wisconsin.

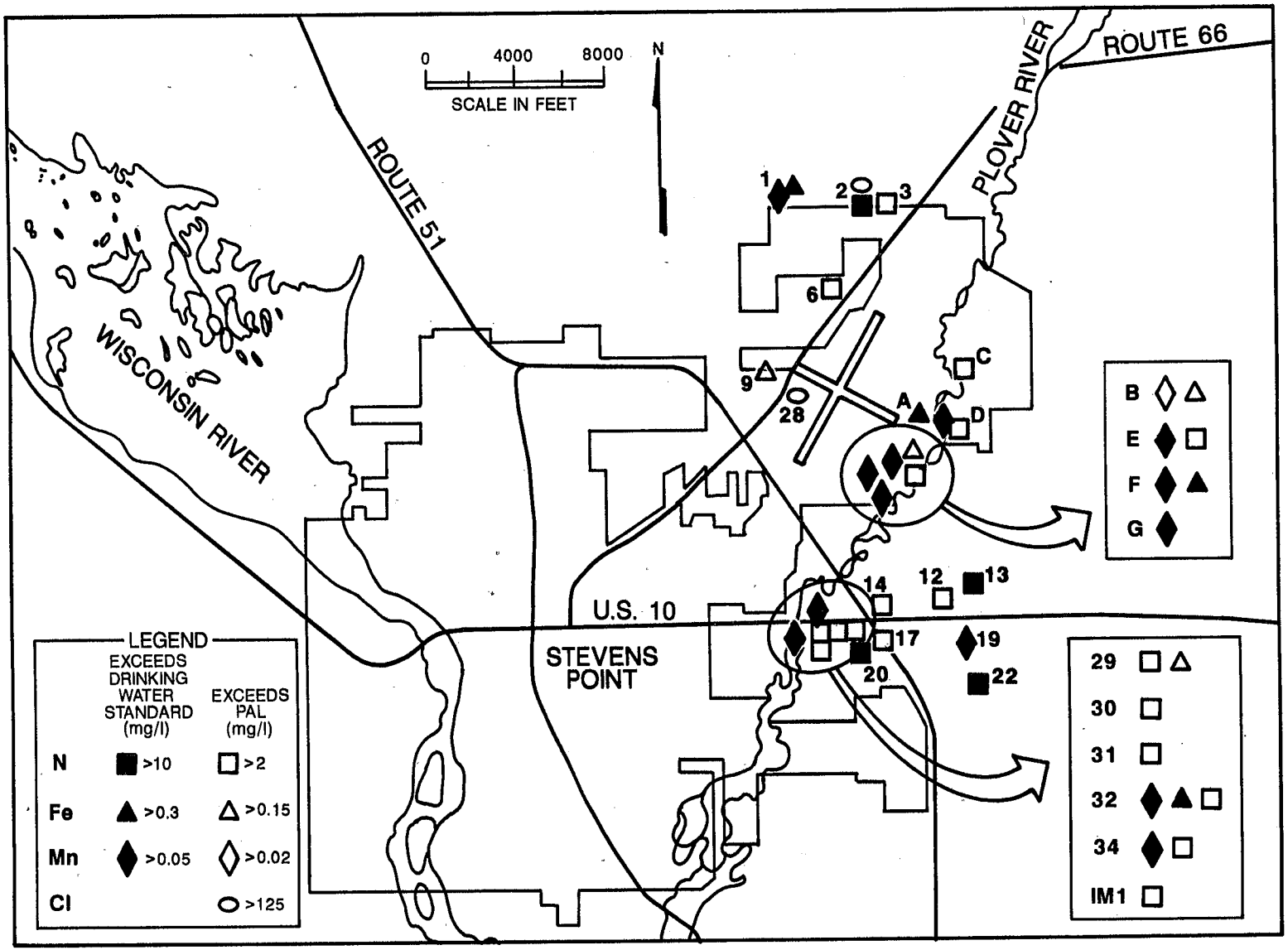


Figure 2-18. Spatial distribution of nitrogen (N), iron (Fe), manganese (Mn), and chloride (Cl) concentrations exceeding drinking water standards or Preventive Action Limits (PALs) throughout the research monitoring network, Stevens Point, Wisconsin.

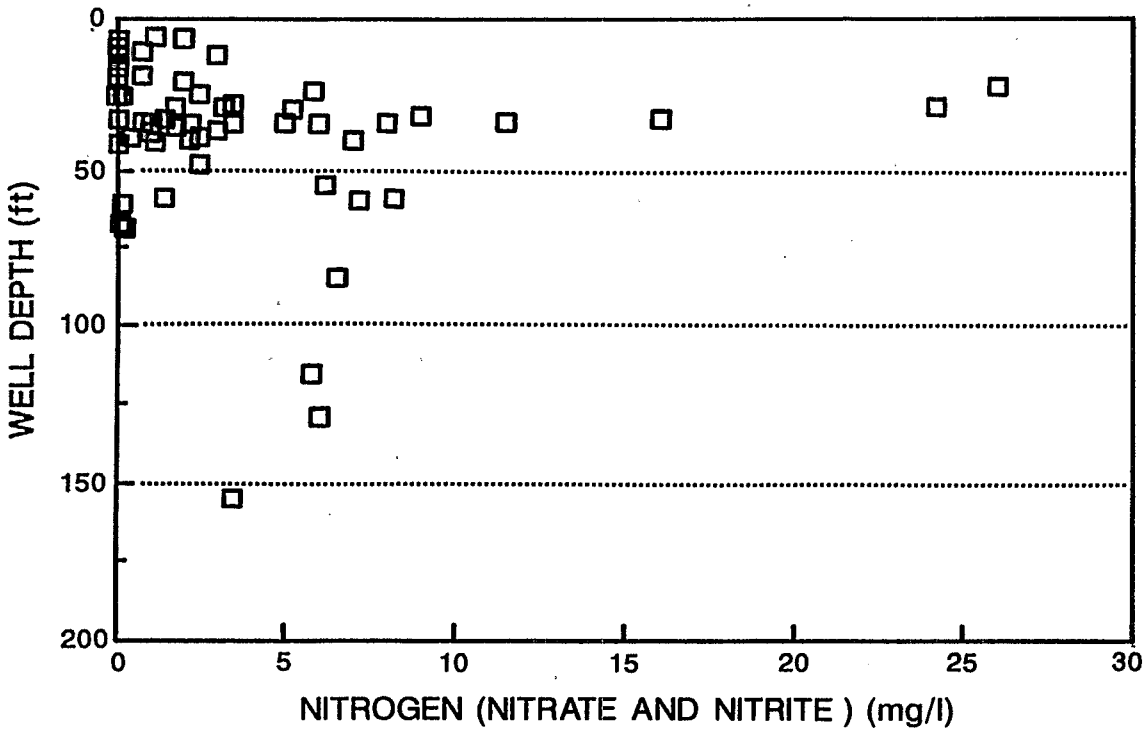


Figure 2-19. Variations in nitrogen concentration with respect to well depth throughout the research monitoring well network, Stevens Point, Wisconsin.

Variations in bedrock elevation (Figure 2-20) and hydraulic conductivity were included in the model. Bedrock elevations were based on mapped well log data and geophysical interpretations (Osborne, 1988). Hydraulic conductivity values derived from aquifer testing were used in the model. For the buried valley, the hydraulic conductivity was assumed to be 1,240 ft/d. This value assumes that the hydraulic conductivity derived from the aquifer test at CW6 is an average conductivity for the saturated thickness of the aquifer in the buried valley. The hydraulic conductivity in areas overlying bedrock highs was assigned a value of 100 ft/d. Slug-test results from MW13, representative of the bedrock high areas, indicate significantly lower hydraulic conductivities ranging from 2.1 to 3.5 ft/d (Table 2-5). The decision to assign a higher value to the bedrock high areas (100 ft/d) was based on the assumption that the slug test investigated a small portion of the aquifer with lower hydraulic conductivity. The slug test was performed in the screened interval set in decomposed granite. Realistically, the average hydraulic conductivity of the bedrock high areas is probably one to two orders of magnitude higher than the slug-test results indicate, considering the presence of sand above

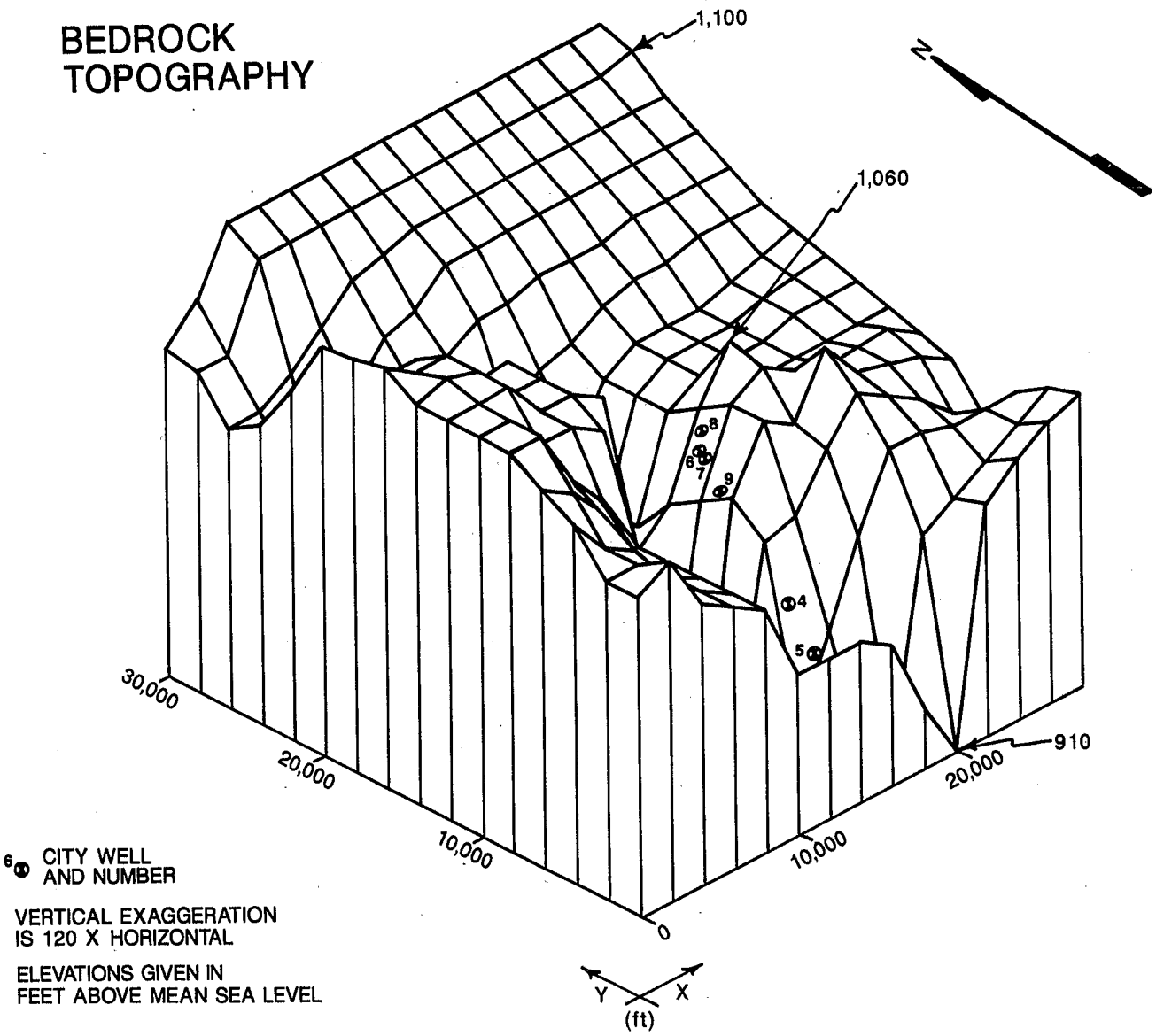


Figure 2-20. Bedrock topography map, Stevens Point, Wisconsin (Osborne, 1988).

the decomposed granite layer. The hydraulic conductivity for the remainder of the aquifer was initially assigned as 320 ft/d based on the average conductivity for the glacial outwash from previous studies (Holt, 1965; Faustini 1985; Rothchild, 1982).

Surface-water nodes were incorporated to simulate the effects of the Plover River and a small lake to the west of the Airport wellfield (Figure 2-9). Leakage rates for the river and the lake were calculated for the model on the basis of the following equation (Franz and Guiguer, 1989, p. 19):

$$L_r = \frac{K_v}{d} \times R_c$$

where,

L_r = leakage factor in $(d)^{-1}$
K_v = vertical hydraulic conductivity of the river or lake bed in ft/d
d = thickness of the river or lake bed in feet
R_c = correction factor for the ratio of surface areas of the finite-difference block and the river or lake in the block

For the Plover River, the hydraulic conductivity varied from 0.01 to 0.1 ft/d times the average hydraulic conductivity of the aquifer (320 ft/d) for calibration purposes, resulting in a K_v range of 3.2 to 32 ft/d. This range is effectively one to two orders of magnitude less than the average hydraulic conductivity of the aquifer. Leakage rates range between values that are considered reasonable for river and lake deposits (Freeze and Cherry, 1979). Similarly, leakage from the small lake was assumed to be three to four orders of magnitude less than the average hydraulic conductivity of the aquifer, 0.03 to 0.30 ft/d.

The estimated average annual recharge for the sand plain aquifer, 10 inches, was used as input to the model (Portage County Planning Department, 1988). The June, 1988, pumping rates were used as discharge values in the model. The range of pumping rates for June discharges are 1.06 Mgd to 2.48 Mgd for the Airport and Iverson wellfields. Summer discharge rates were chosen for modeling purposes, as opposed to winter discharges, to simulate maximum stresses on the aquifer.

The model was not calibrated rigorously to specific head measurements because sufficient data for comparison were lacking. The hydrogeologic mapping performed during this study covers only a portion of the area of the model. In addition, the hydrogeologic data were obtained during transient conditions--the aquifer had been stressed by periodic, high-capacity well pumping and was responding to recent recharge events. The model used for this study, FLOWPATH (Franz and Guiguer, 1989), assumes steady-state conditions. Regional water-table maps are available. However, these maps are based on data collected from a variety of sources over a number of years. The regional water-table maps are useful for determining general ground-water flow directions and regional flow boundaries; however, they do not represent steady-state hydrogeologic conditions.

Considering the project duration, model constraints, and data base limitations, the most realistic representation of the ground-water flow system is obtained by (1) using field-generated values to establish aquifer parameters; (2) incorporating known heterogeneity, where applicable; and (3) applying reasonable values for the model input parameters where gaps exist in the data base. The model was calibrated to represent regional flow paths and water-table elevations. The recharge rate was varied uniformly over the entire model area. Aquifer and surface-water body hydraulic conductivity values were varied only in regions for which there were little or no data. Values selected for hydraulic conductivity are within the range of field measurements for the sand and gravel aquifer. Recharge values for the model are lower than the average annual precipitation. The known and unknown input parameters for the ground-water flow model, as well as how these parameter values were used, or estimated, are summarized in Table 2-6.

The hydraulic head distribution map and the 5-year TOT zone generated by the model are shown in Figures 2-21A and 2-21B, respectively. Appendix A contains all input and output data pertinent to the ground-water flow modeling of the Airport and Iverson wellfields using FLOWPATH. As expected, the analytically-derived B Zone is larger than the refined 5-year TOT zone predicted by the numeric model, FLOWPATH. However, the refined 5-year TOT zone extends farther to the east than previously predicted, particularly in association with the Iverson wellfield (Figure 2-22). This is due to the spatial variability in hydraulic conductivity, coupled with the influence of the bedrock highs. As the conceptual model was refined from a homogeneous to a more heterogeneous aquifer, with an irregularly shaped bottom surface and variable hydraulic conductivity, the shape of the 5-year TOT zone became increasingly irregular.

TABLE 2-6. AQUIFER PARAMETER VALUES AND THEIR APPLICATION TO THE GROUND-WATER FLOW MODEL, FLOWPATH

Known Values	Application	Unknown Factors	Treatment in Model
Hydraulic conductivities:		Hydraulic conductivities:	
from CW6	K (buried valley) = 1,240 ft/d	with depth	Assumed an average K for each setting
		laterally	Assumed K from known values
		river bottom	Assumed K_v (river) = 3.1 - 32 ft/d
from MW13	Adjusted K (bedrock highs) = 100 ft/d	small lake	Assumed K_v (lake) = 0.03 - 0.30 ft/d
from K58 and other studies	Average K (aquifer) = 320 ft/d	Water-table elevations between/beyond monitoring wells	Averaged between wells; used regional map (Lippelt and Hennings, 1981) for calibration
Water-table elevations from monitoring wells	Calibration value		
Bedrock topography:		Bedrock topography beyond bedrock wells	Averaged between wells for bedrock surface elevation (Osborne, 1989)
from outcrops	Elevation of aquifer bottom		
from well logs	Elevation of aquifer bottom		
		Recharge rate(s)	Assumed uniform recharge from water balance (Holt, 1965)
		Three-dimensional flow	Assumed horizontal flow
		Contaminant transport	Assumed advective transport rate

Abbreviations: K = hydraulic conductivity; K_v = vertical hydraulic conductivity; CW = city well; MW = monitoring well.

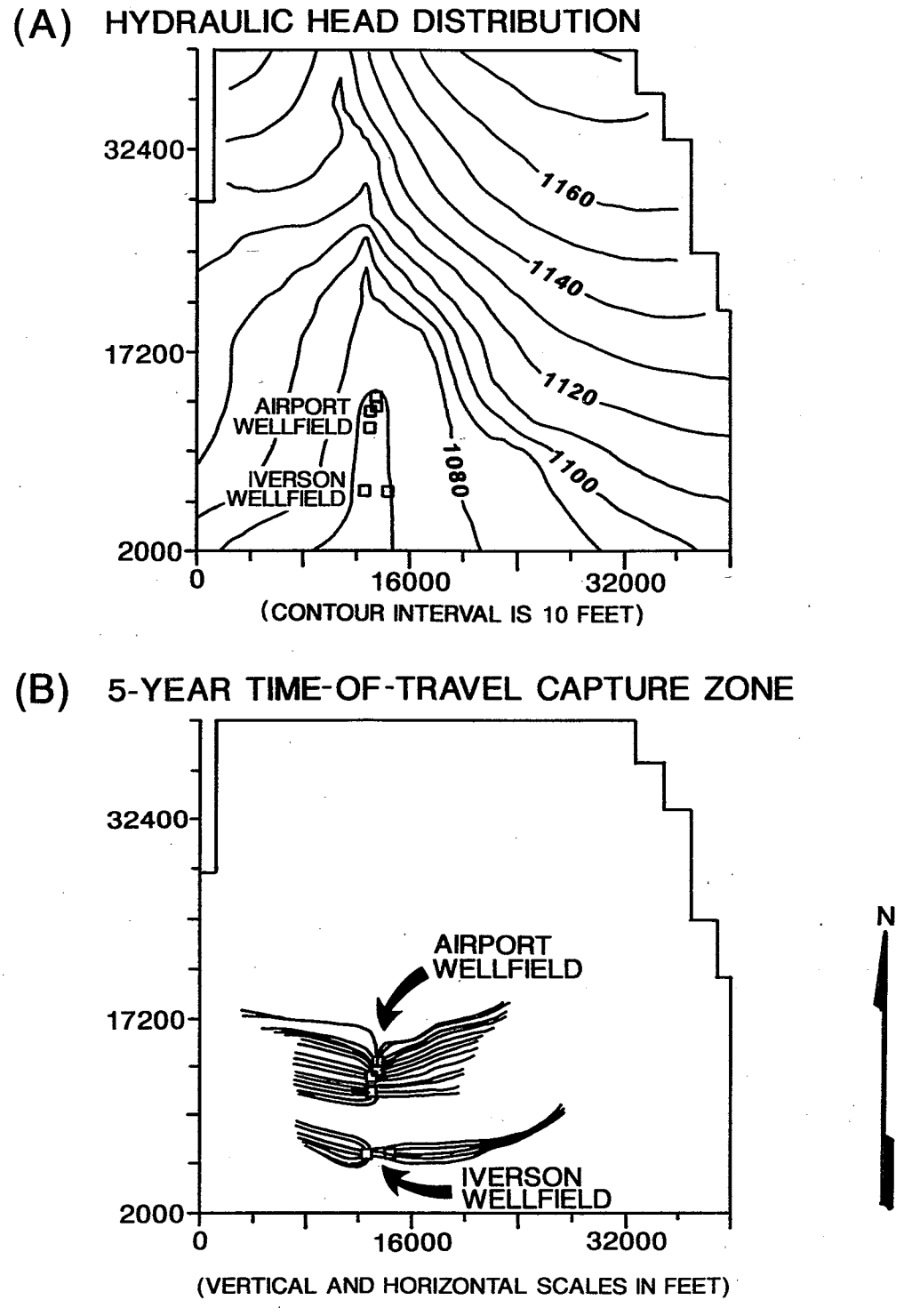


Figure 2-21. (A) Hydraulic head distribution map and (B) 5-year time-of-travel zone for the Airport and Iverson wellfields, Stevens Point, Wisconsin.

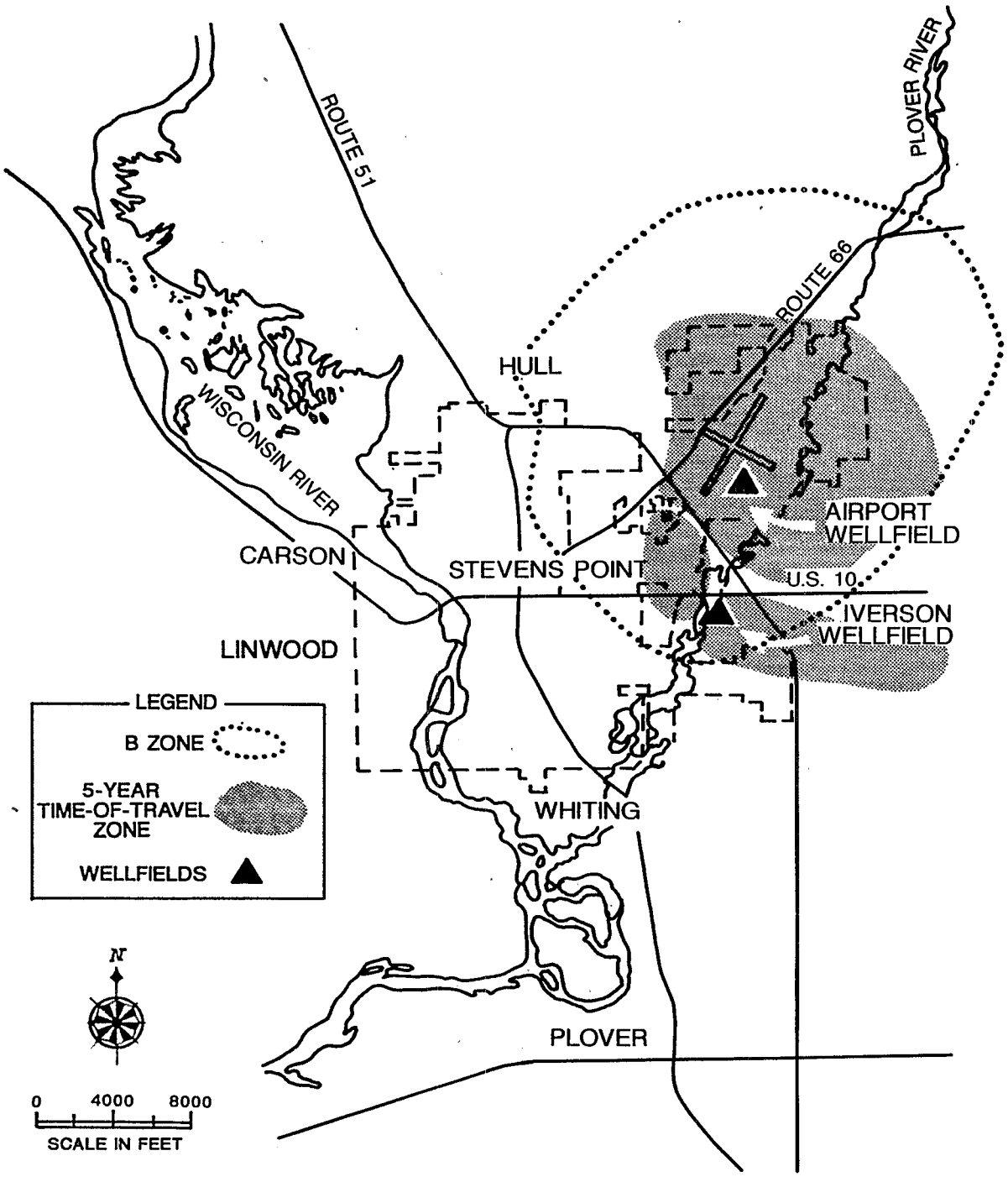


Figure 2-22. Comparison of B Zone and 5-year time-of-travel zone areas determined by analytical and modeling (FLOWPATH) methods, respectively, for the Airport and Iverson wellfields, Stevens Point, Wisconsin.

WELLHEAD PROTECTION MONITORING PROGRAM

The purpose of the proposed wellhead protection monitoring program is to preserve the quality of ground water at the Stevens Point wellfields. The monitoring system will detect changes in ground-water quality within the WHPA. The proposed program consists of two types of monitoring systems: one for ambient trend monitoring and one for compliance monitoring. The ambient-trend monitoring system will detect regional changes in ground-water quality and aid in locating contaminant sources. This system is targeted to monitor the effects of nonpoint sources and high-density point sources. The compliance monitoring system is used to (1) assess if monitoring regulations are met for a source, and (2) provide an early-warning detection system at priority point sources.

The wellhead protection monitoring program presented here gives priority to protection of the Airport wellfield. Without the Airport wellfield, the city could not meet the existing water-supply demand without severe rationing. In addition, manganese concentrations in raw ground water from CW4 (at the Iverson wellfield) exceed secondary drinking water standards. This water would require treatment for use; however, it is sufficiently diluted by addition of higher quality water from the Airport wells. If the Airport wellfield is contaminated, additional wells would need to be installed, or a treatment plant established and surface water used to meet the present water demands of the city.

AMBIENT TREND MONITORING

Monitoring Sites

The existing monitoring well system provides adequate coverage for most of the 5-year TOT zone for the Airport wellfield. Areas with insufficient coverage in this zone include the eastern portion of the zone and the deeper part of the aquifer to the north and to the west along the 5-year TOT boundary, especially in the buried valley. All of the existing monitoring wells are located within the 5-year TOT zone, leaving the city little time to respond to a contamination event.

Additional wells are proposed in the 5- and 10-year TOT zone (well nests N through V) to offer more response time. These additional wells will serve as early-warning detection indicators of contamination. Existing and proposed monitoring wells for the wellhead protection monitoring network are shown in Figure 2-23. In addition, it is recommended that dry wells in the storm-sewer system and the Plover River be sampled after storms to determine the water quality and the trends in recharge chemistry due to urban runoff.

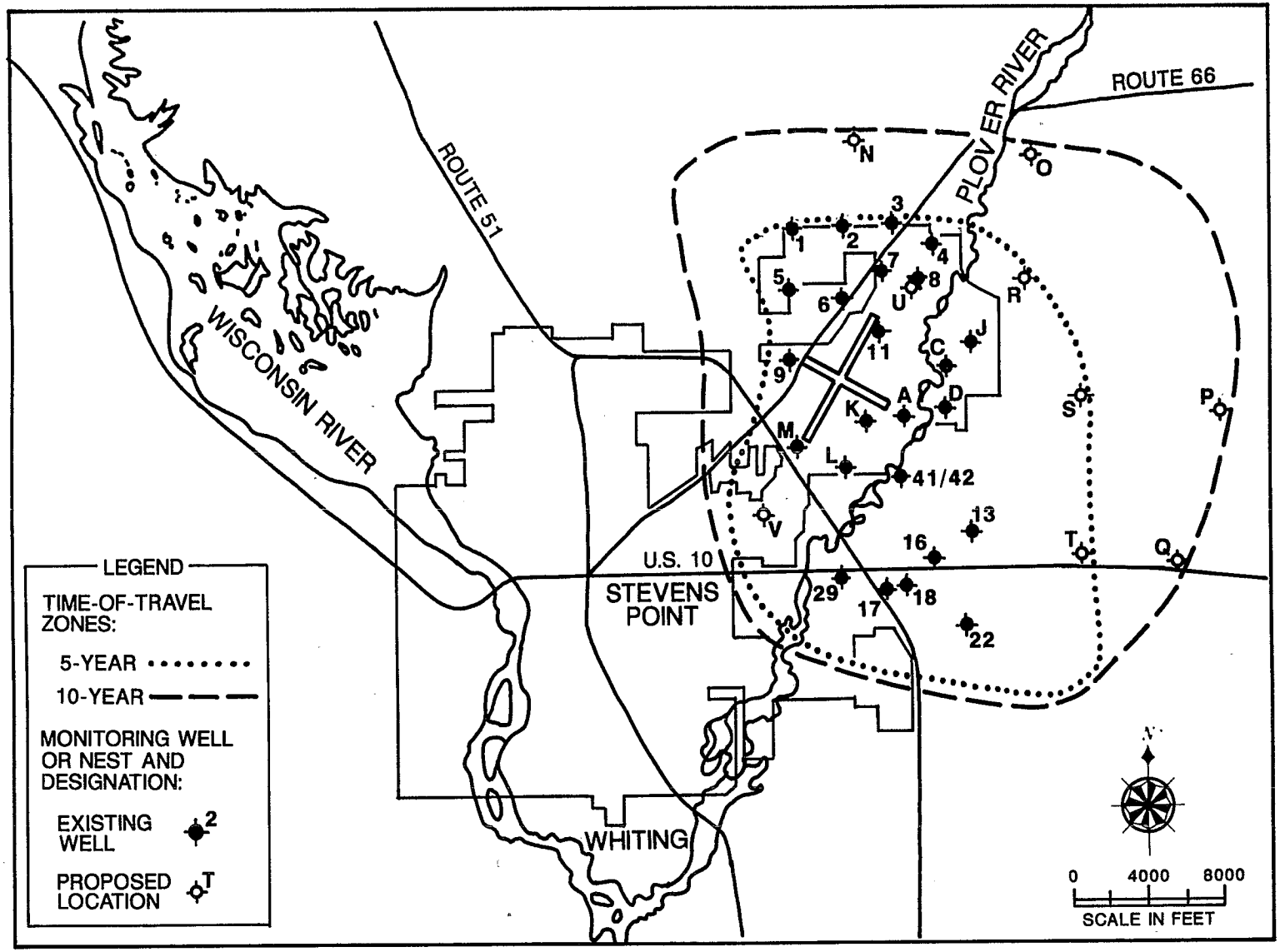


Figure 2-23. Proposed wellhead protection monitoring network for the Airport and Iverson wellfields, Stevens Point, Wisconsin.

Monitoring Well Construction

It is recommended that all new monitoring sites (N through V) be installed as well nests. These well nests should be constructed similar to well nest K (Figure 2-10B) and located on public property to guarantee accessibility. Well nests installed in rights-of-ways should be placed as far as possible from the road to prevent damage and to reduce the possibility that road construction will result in removal of the well.

Monitoring Parameters and Frequencies

The ambient-trend monitoring wells and the Plover River should be sampled twice a year. The first sampling period should occur in April after the ground has begun to thaw and after spring recharge begins. The second sampling period should begin in September after active land-use activities, such as pesticide and fertilizer application, are completed. The ambient-trend well samples should be analyzed for the indicator parameters: pH, specific conductivity, nitrogen (nitrate and nitrite), chloride, iron, manganese, nitrogen- and phosphorous-based organics (pesticides), VOCs, and total petroleum hydrocarbons (TPHs). Sampling should be conducted following standard guidelines (Wisconsin Department of Natural Resources, 1987), and water levels should be recorded each time a well is sampled.

Monitoring Data Base

The hydrogeologic and hydrochemical data should be stored on a computerized data base and updated immediately after the results of each monitoring sample set are received by the city Water Department. This data base will be used by the Water Department to monitor changes in ground-water quality and to trigger further investigation if PALs are exceeded. Further investigation may entail sampling and testing the affected well for other possible contaminants, installing a compliance monitoring system, and remediation actions. The PALs, as well as primary and secondary drinking water standards (Wisconsin Department of Natural Resources, 1988), for the ambient-trend indicator parameters are listed in Table 2-7.

COMPLIANCE MONITORING

Monitoring Sites

There is one existing compliance monitoring network in the monitoring system. This network of wells was installed to monitor the airport fuel storage tanks. Unfortunately, this network failed to detect the leak in the UST pumping system in 1988. This incident illustrates the importance of monitoring system design. In the case of USTs, careful record keeping by the owner may provide the most reliable, quickest, and least expensive leak-detection system. The first compliance monitoring wells, or network, should be installed at the highest-priority point sources (for example, USTs and large, historical spills in the 10-year TOT zone). Compliance monitoring networks at the remaining point sources should be added in order of priority, such as feedlots, golf courses, then landfills. New spills will require immediate evaluation and, possibly, remediation and monitoring.

TABLE 2-7. PREVENTIVE ACTION LIMITS AND STATE OF WISCONSIN DRINKING WATER STANDARDS*

Analysis	Parameters	PAL	Standard
		(µg/l)	
VOCs	Benzene	0.067	0.67
	1,1-Dichloroethane	85	850
	1,2-Dichloroethane	0.05	0.5
	1,2-Dichloroethylene	0.024	0.24
	Ethylbenzene	272	1,360
	Tetrachloroethylene	0.1	1
	Toluene	68.6	343
	1,1,1-Trichloroethane	40	200
	1,1,2-Trichloroethane	0.06	0.6
	Trichloroethylene	0.18	1.8
	Xylene	124	620
N- and P-based organics	Alachlor	0.05	0.5
	Atrazine	0.35	3.5
	EPTC	50	250
	Simazine	215	2,150
		(mg/l)	
NO_2 and NO_3	N	2	10
Cl	Cl	125	250†
Mn	Mn	0.025	0.05†

Abbreviations: PALs = preventive action limits; VOCs = volatile organic compounds; N = nitrogen; P = phosphorus; NO_2 = nitrite; NO_3 = nitrate; Cl = chloride; Mn = manganese.
* Wisconsin Department of Natural Resources, 1988.
† Secondary drinking water standard.

Monitoring Well Construction

The position of monitoring wells around point sources should be determined by careful investigation of the ground-water flow system in the vicinity of the potential source. The hydrogeologic mapping done for this study is more regional in nature and does not focus on the local pattern of ground-water flow at proposed compliance monitoring sites. Important factors influencing local ground-water flowpaths and contaminant transport are: (1) the effects of variable bedrock topography, (2) localized changes in the regional gradient due to pumping wells, and (3) the location and nature of the potential contaminant. Another factor influencing the design of individual compliance monitoring networks is the accessibility of the site. If monitoring wells cannot be placed near the source, the density and number of depths monitored should be increased. The exact placement of the monitoring wells will, necessarily, be site-specific. Generally, the initial monitoring

system should include wells placed between 10 and 50 feet downgradient from the potential source and at least one upgradient well placed 10 to 30 feet from the potential source. The number of downgradient wells will depend on the size of the source to be monitored; the cost of installation, sampling, and chemical analyses; and the distance from the source. No fewer than three downgradient wells should be installed (Pfannkuch, 1982; U.S. EPA, 1986).

The primary goal of compliance monitoring is to assess or to ensure that monitoring regulations are being met for a site or source. Secondly, compliance wells are used for detection monitoring. Compliance monitoring data should be acquired following strict adherence to quality assurance and quality control procedures so that data are court admissible. Each compliance monitoring well should be installed as a single well, somewhat as that depicted in Figure 2-10A and discussed in the Data Acquisition, Monitoring Wells section. Single wells are preferred to multi-level wells for compliance monitoring because boreholes are completely separate, thereby reducing the possibilities of cross-contamination and vertical interconnection. Exceptions to the recommended single well construction given in Figure 2-10A pertain to the well diameter, well material, well screen, and gravel pack.

Compliance wells may need to be completed with a minimum, 2-inch-diameter pipe and screen to accommodate regulation-type sampling and other well entry equipment. The borehole should be sufficiently wide to accommodate all well materials and tools. The WGNHS and the U.S. EPA Region V offices provide technical guidance and review of proper well construction, installation, and development procedures for compliance purposes. The well construction material should be carefully reviewed regarding its chemical compatibility and reactability to anticipated ground-water contaminants. In some cases, such as when solvents at high concentrations in ground water are the target monitoring parameter, stainless steel is recommended over PVC for well construction. All well components introduced to the borehole should be decontaminated and clean. No solvent-welded (glued) components should be used in the well construction process; all well pipe and screen should be flush-threaded.

For compliance purposes, it may be necessary to install acid-resistant, washed and graded silica sand, rather than native sand backfill for the gravel pack material. The gravel pack should be clean and free of oil, acid, or other deleterious substances. The well screen slot size should be designed with respect to the gravel pack sieve size to minimize disruption of normal flow through the well and to prevent loss of the well depth or screen length (silting-in).

Single monitoring wells may be combined in optimal spacing and depth configurations (nested) to intercept suspected contaminant flowpaths. If the compliance monitoring well network is located near a potential contaminant source where dense, nonaqueous-phase liquids (DNAPLs) have not been used at the source, then shallow monitoring wells should be installed. For DNAPL contaminant sources that tend to sink, rather than float, within the aquifer, the monitoring wells should be installed to monitor ground water in deeper portions of the aquifer, potentially to bedrock. All shallow wells should be screened in the upper 15 feet of the aquifer to access water in the shallow ground-water flow system and to allow for seasonal changes in the water level.

To fill data gaps in the compliance monitoring network relatively inexpensively, and for detection purposes only (non-regulatory), multi-level monitoring wells may be considered. The multi-level well installation method is similar to that at well nests J through M (Figure 2-10B). Casing is

pushed to the desired depth, cleaned, and a polytube bundle is installed. Then, the casing is extracted, and the installation is completed when the annular space is filled with collapsed sand, or sand backfill, and a bentonite seal is installed at the surface.

An economical method for installing multi-level wells (for detection monitoring only) should be used. For example, in areas where the water table is very shallow, surface casing with an expendable drive point can be driven into the aquifer to the desired depth, and a polytube bundle can be lowered into the casing (Sitets and Chambers, 1990). When the casing is pulled, the drive point and the polytube bundle are left in place. For deeper installations, a drilling rig can be used to push the casing, and a sand bailer can be used to clean the casing before installation of the polytube bundle. Well installation equipment and materials should be decontaminated between each well installation, using standard procedures to prevent cross contamination within the aquifer (Wisconsin Department of Natural Resources, 1988).

Monitoring Parameters and Frequencies

Each compliance monitoring network should be sampled for select, potential contaminants or a characteristic component of the monitored source. For example, if the potential source is a gasoline storage tank, the target monitoring parameters should be VOCs. The frequency of the sampling is dependent on the location of the potential source. Sources within the 5-year TOT zone should be sampled at least twice a year, and sources in the 10-year TOT should be sampled annually, at a minimum. Potential point sources and recommended monitoring parameters, as well as sampling frequencies, are listed in Table 2-8.

Monitoring Data Base

The results of the compliance monitoring program need to be kept on file. Any detection of a contaminant in a monitoring well should trigger the contingency plan.

CONTINGENCY PLAN

The contingency plan for the Stevens Point water supply system consists of three components. One component of the plan is reaction to the early-warning monitoring system. When a PAL is exceeded in an ambient-trend monitoring well, or a specific contaminant is detected in a compliance monitoring well, then the Water Department should resample the well and begin an investigation into the source and extent of the contamination. Results of that investigation will determine the nature of the remedial actions. The procedures outlined by the EPA (U.S. EPA, 1990) can be used, in part, to assess the site.

The second element of the contingency plan is to react to events that are beyond the capabilities of the monitoring system (for example, accidental spills in the 5-year TOT zone and leaks at sources that are not currently monitored). This phase of the contingency plan is currently in operation and relies on the cooperation of state emergency-response agencies. In the event of an

TABLE 2-8. COMPLIANCE MONITORING GUIDELINES

Priority Point Sources	Monitoring Parameters	Monitoring Frequency
Airport USTs (old and new)	VOCs	Quarterly
Gas stations Highway 66 and Wilshire Highway 10 and Brilowski	VOCs	Semiannually
High-density, small USTs Highway 66 and Airport Rd Highway 10, near Wildwood Highway 66, east of Highway 51	VOCs	Semiannually
Garages Highway 66 and Highway 51	VOCs	Annually
Feedlot (FL-A)	N (NO_2 and NO_3)	Semiannually
Golf courses	N (NO_2 and NO_3)	Semiannually
Abandoned landfills	VOCs; TPHs; major anions, cations, and metals	Annually

Abbreviations: USTs = underground storage tanks; VOCs = volatile organic compounds; N = nitrogen; NO_2 = nitrite; NO_3 = nitrate; TPHs = total petroleum hydrocarbons.

accident within the 10-year TOT zone involving the release of hazardous materials, the local and state emergency-response agencies (for example, the fire department, police, sheriff, highway patrol, and Department of Natural Resources) will notify Stevens Point Water Department personnel immediately, and the need for remedial action will be assessed.

If the WHPP fails and a city well is contaminated, the third component of the contingency plan is implemented: the affected wells will be shut down and other wells will supply the city's needs, the source of contamination will be investigated, and remedial actions will be taken. New wells would be required if the contamination is extensive or if the contamination cannot be remediated. One function of the ambient-trend monitoring system is to assess local ground-water quality to better identify favorable ground-water development areas. If ground-water contamination is widespread, a treatment plant may be required.

CONCLUSIONS

A study was conducted in Stevens Point, Wisconsin, to refine the preliminary wellhead protection area for the Airport and Iverson wellfields, as well as to design an early-warning detection ground-water monitoring network. The study entailed source assessment, well installation, aquifer testing, ground-water sampling, hydrochemical characterization, ground-water flow modeling, wellhead protection area delineation, and monitoring network design. Conclusions of the study are summarized as follows:

- Existing point sources within the preliminary wellhead protection area include above-ground and underground storage tanks (USTs). Nonpoint sources include agricultural fields, lawns, septic systems, and wetland soils.

- Potential point sources include garages and gas stations, industrial and commercial facilities, junkyards, landfills, salt and snow storage areas, septic systems, spills, storage tanks, transformers, and unregulated dump sites. Nonpoint sources include feedlots, golf courses, lawns, the Plover River, road de-icing runoff, septic-sludge spreading, and urban runoff.

- Existing contaminant sources were given highest priority for management. Other sources were ranked based on the source type, quantity, hazard, and location. Prioritized sources include USTs, irrigated agriculture, high-density, nonsewered residential areas, landscaped areas, feedlots, spills, landfills, perforated storm drainage pipes, and dry wells.

- A network of 44 single monitoring wells, 10 well nests, and one multi-level well were used to measure water levels, to sample ground water, and to conduct aquifer tests in the shallow, unconfined sand and gravel aquifer. Of the total network, three single wells and four well nests represent new monitoring points installed for this research.

- Aquifer parameter results from slug, constant discharge, and recovery tests indicate a range of hydraulic conductivity values for three distinct geologic settings: 820 to 1,700 ft/d for the buried valley, 220 to 240 ft/d for outwash plains, and 2 to 3 ft/d for bedrock highs.

- Hydrochemical data indicate that nitrogen concentrations, a key indicator of contamination, have increased over time. Currently, nitrogen concentrations in the monitoring network range from less than 0.2 to 26.0 mg/l. Nitrogen concentrations are generally higher in the shallow monitoring wells. Better ground water quality at depth may indicate a more regional flowpath, more distant agricultural areas, dilution of recharge water with depth, or a combination of these factors.

- Other indicators of ground-water degradation include iron and manganese from organic-rich soils located along the Plover River, chloride in proximity to roads where de-icing occurs, and previous volatile organic compound (VOC) contamination at the airport and several USTs.

- The two-dimensional, ground-water flow model FLOWPATH was used to delineate the 5- and 10-year time-of-travel (TOT) zones for the Airport and Iverson wellfields. In comparison, the previous, analytically-derived B Zone is larger; however, the 5-year TOT zone from FLOWPATH extends farther to the east due to the effects of pumping at the Iverson wellfield and the presence of bedrock highs.

- A long-term ground-water monitoring network is proposed for the Airport and Iverson wellfields consisting of 34 total existing and proposed wells. Twenty-five existing wells include 18 single wells and seven well nests. Nine new well locations are proposed to fill data gaps in the existing network, primarily along the boundaries of the 5- and 10-year TOT zones.

- Wells in the long-term monitoring network should be sampled twice a year in April and September for the indicator parameters: pH, specific conductivity, nitrogen, chloride, iron, manganese, nitrogen- and phosphorous-based organics (pesticides), VOCs, and total petroleum hydrocarbons (TPHs). Water levels should be recorded each time a well is sampled.

- It is recommended that the Plover River and dry wells in the storm-sewer system be sampled after storms to determine water quality and trends in recharge chemistry due to urban runoff.

- Compliance monitoring networks should be installed at high-priority point sources such as USTs, spill sites, feedlots, golf courses, and landfills. No fewer than one upgradient and three downgradient wells should be installed per network. Single well construction is recommended for compliance monitoring with some wells combined in nested configurations to allow detection of parameters at various depths in the aquifer.

- Select parameters are chosen for compliance monitoring based on the historical use of substances at the facility. For example, USTs, gas stations, and garages are specifically monitored for VOCs; feedlots and golf courses are monitored for nitrogen; and landfills are monitored for VOCs, TPHs, major anions, major cations, and metals. The frequency of monitoring for key parameters is site-specific, varying from quarterly to annually.

- The wellhead protection contingency plan consists of three components: (1) reaction to the early-warning detection system based on preventive action and state drinking water limits, (2) spill response, and (3) new water-supply development and implementation.

RECOMMENDATIONS

The water supply for the city of Stevens Point is currently of high quality. Compared to other cities, the potential sources of contamination are few and distant. However, the potential for contamination of the city's water supply exists. The greatest threat to the city's water supply is public unawareness of aquifer vulnerability. Most citizens are unaware of the fact that what is poured, spilled, leaked, or dumped on the land surface moves quickly into the underlying ground water. The sources of greatest concern are those that have already contaminated ground water, including petroleum from leaking storage tanks, pesticides applied to crops, nitrogen from overfertilization of lawns and crops, and nitrogen from human and animal wastes. Other possible sources of contamination that require careful monitoring include urban runoff, underground tanks that are not yet leaking, and naturally occurring sources that cause degradation (for example, iron, manganese, and radioactivity).

The long-term monitoring system proposed in this report is designed to detect potential contamination events. Its purpose is to provide early warning, to allow time for remediation, and to prevent contamination of the municipal wells. However, the most efficient method for preventing contamination is for owners and operators of potential sources to inventory the hazardous and toxic substances at their facilities and to monitor the correct use of these substances.

The next step in evaluating the potential impact of sources is to model the fate and transport of contaminants, thereby predicting the effects of sources on ground-water quality over time. Accurate predictions of contaminant transport require a detailed model of the ground-water flow system. Evolution of the ground-water flow model from a two-dimensional, steady-state model to a three-dimensional, transient model is essential to represent the flow system accurately. In addition to refining the ground-water flow model, the nature of the contaminant source must be understood. The concentration and release rate of the contaminant from each source should be determined. Field studies are necessary to verify the validity of the model. Contaminant transport models can be used to generate more effective land-use restrictions.

Public education of ground-water protection and hazardous waste disposal procedures are two additional elements of the local WHPP that need to be developed. The new Stevens Point urban area wellhead protection project, funded by the U.S. Department of Agriculture, provides an appropriate mechanism to develop these two elements. These are important additions to the city's ground-water management plan. Following installation of the long-term monitoring well network, other improvements to the system can be made, including

- Research on contaminant transport through the sand and gravel aquifer
- An increase in the number of monitoring points in the 10-year TOT zone and beyond
- Source assessment beyond the 10-year TOT
- Compliance monitoring at sources classified as a moderate risk

This project is the first stage of a continuing WHPP and is the core of an historical data base. As more information is gathered on the ground-water system, the WHPP will be refined and improved. The TOT zones and the model of the ground-water flow system will become more accurate and predictive. Ultimately, the ground-water flow system should be simulated three-dimensionally with a model that can incorporate the variable pumping rate of the city's wells and the transient nature of recharge. To accomplish this goal, a better understanding of the following hydrogeologic factors is necessary: (1) the exact location and seasonal variations of the ground-water flow boundaries, (2) the distribution of hydraulic conductivity (laterally and with depth), and (3) the variation of recharge rates (with time and location). To model the system accurately, the interaction between the Plover River and the city wells must be determined. The city's WHPA should continue to be refined and land-use controls implemented to ensure protection of the city's water supply.

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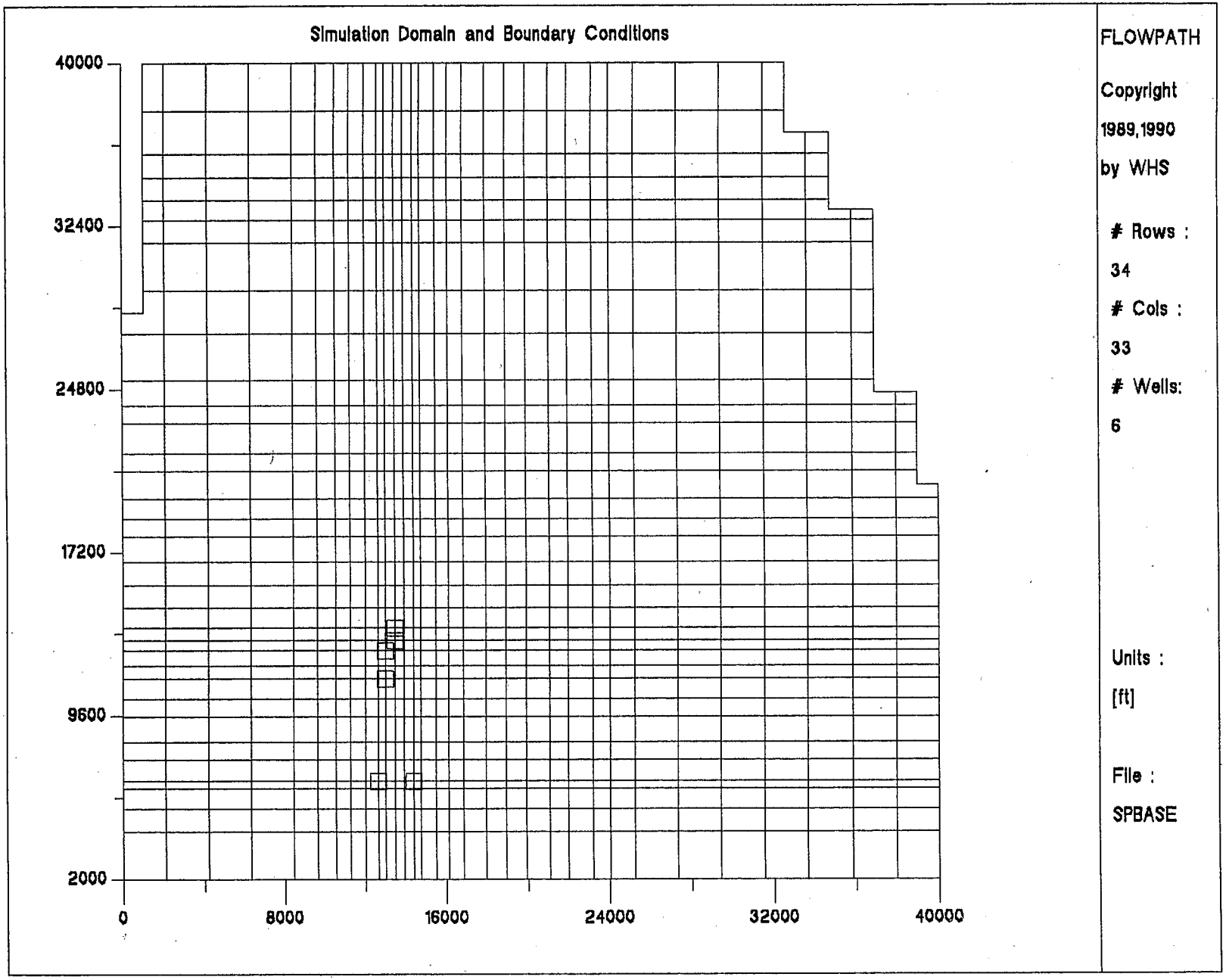
APPENDIX 2-A

FLOWPATH GROUND-WATER MODEL RESULTS FOR THE STEVENS POINT, WISCONSIN, AQUIFER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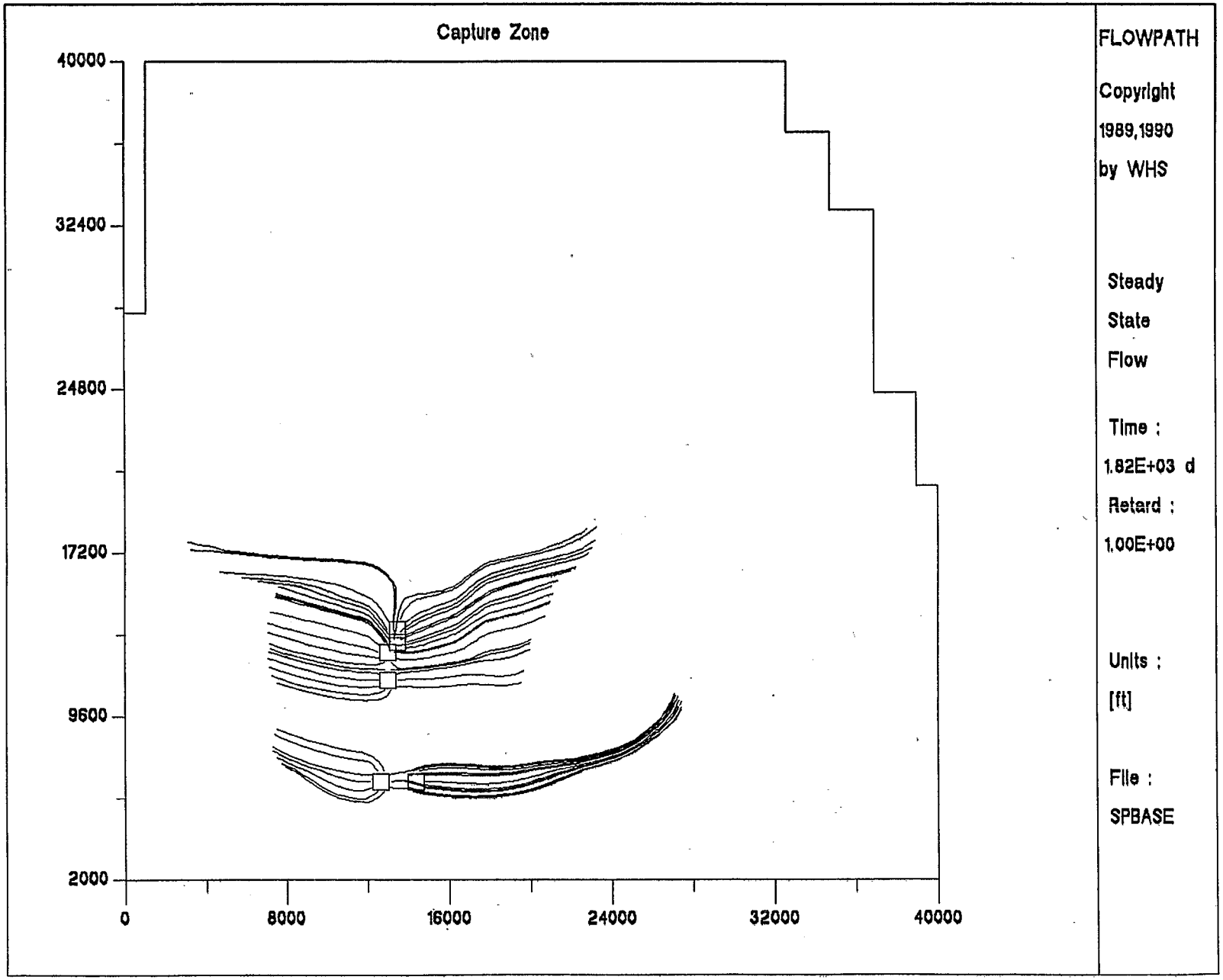
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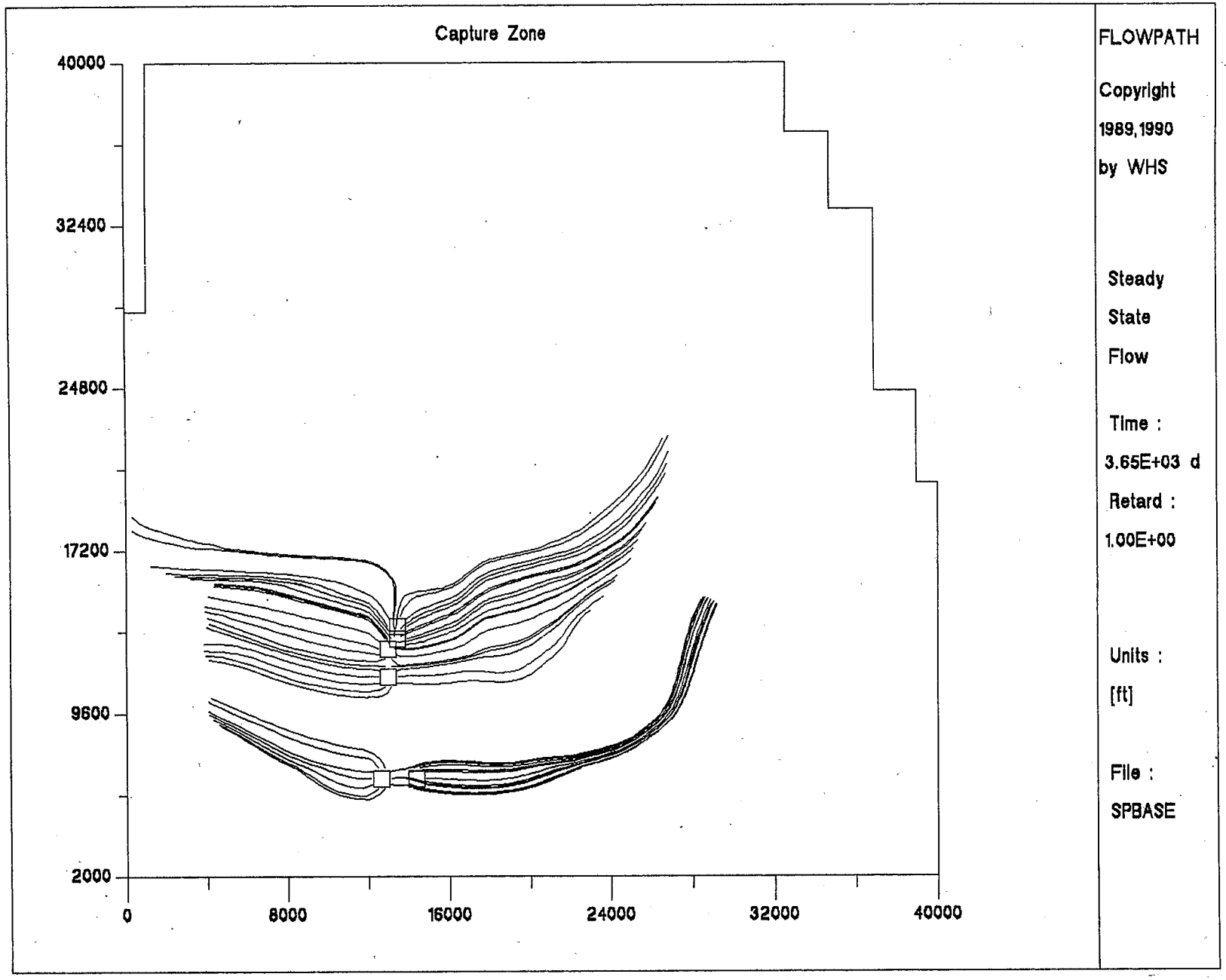
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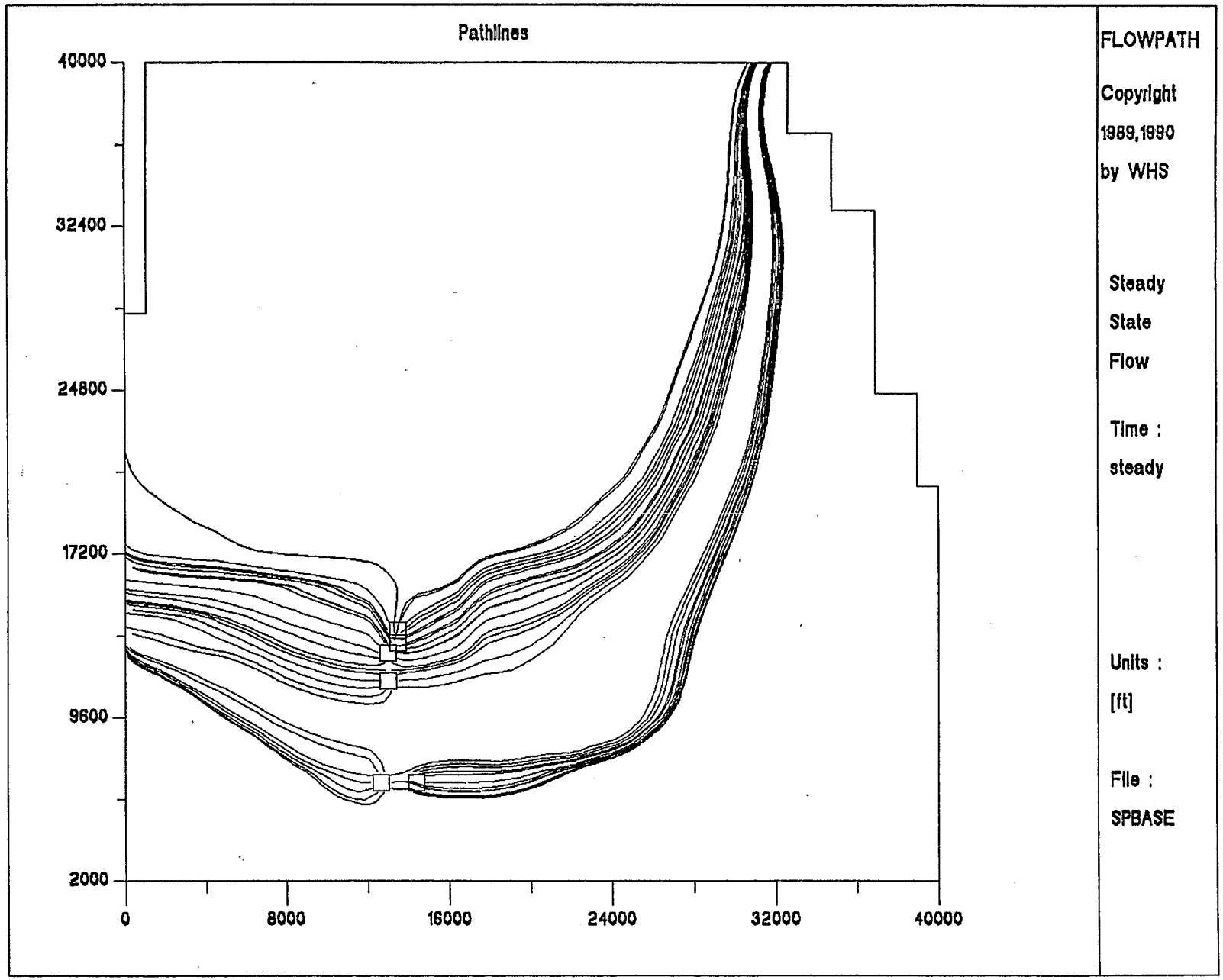
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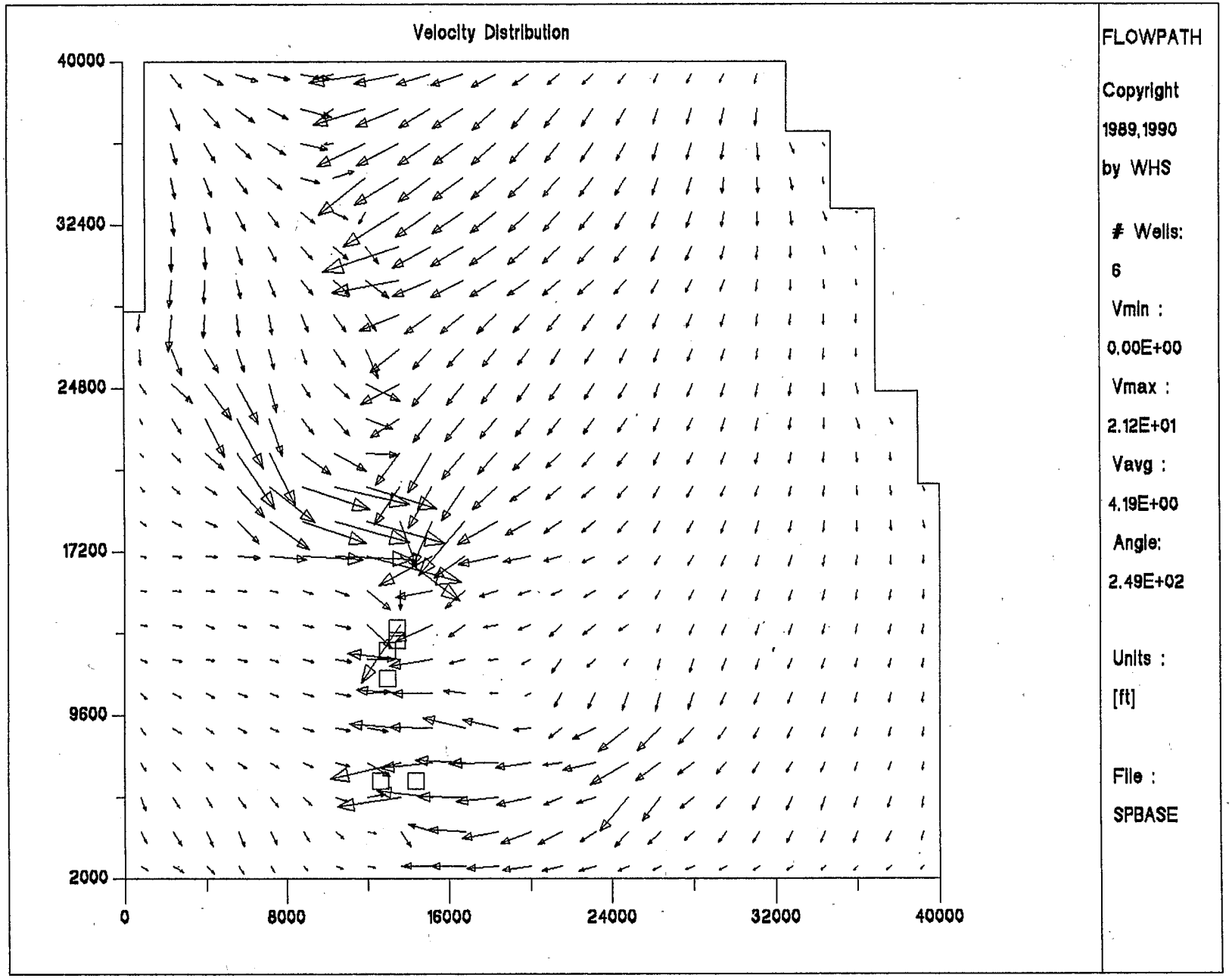
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by WHS
Wells:
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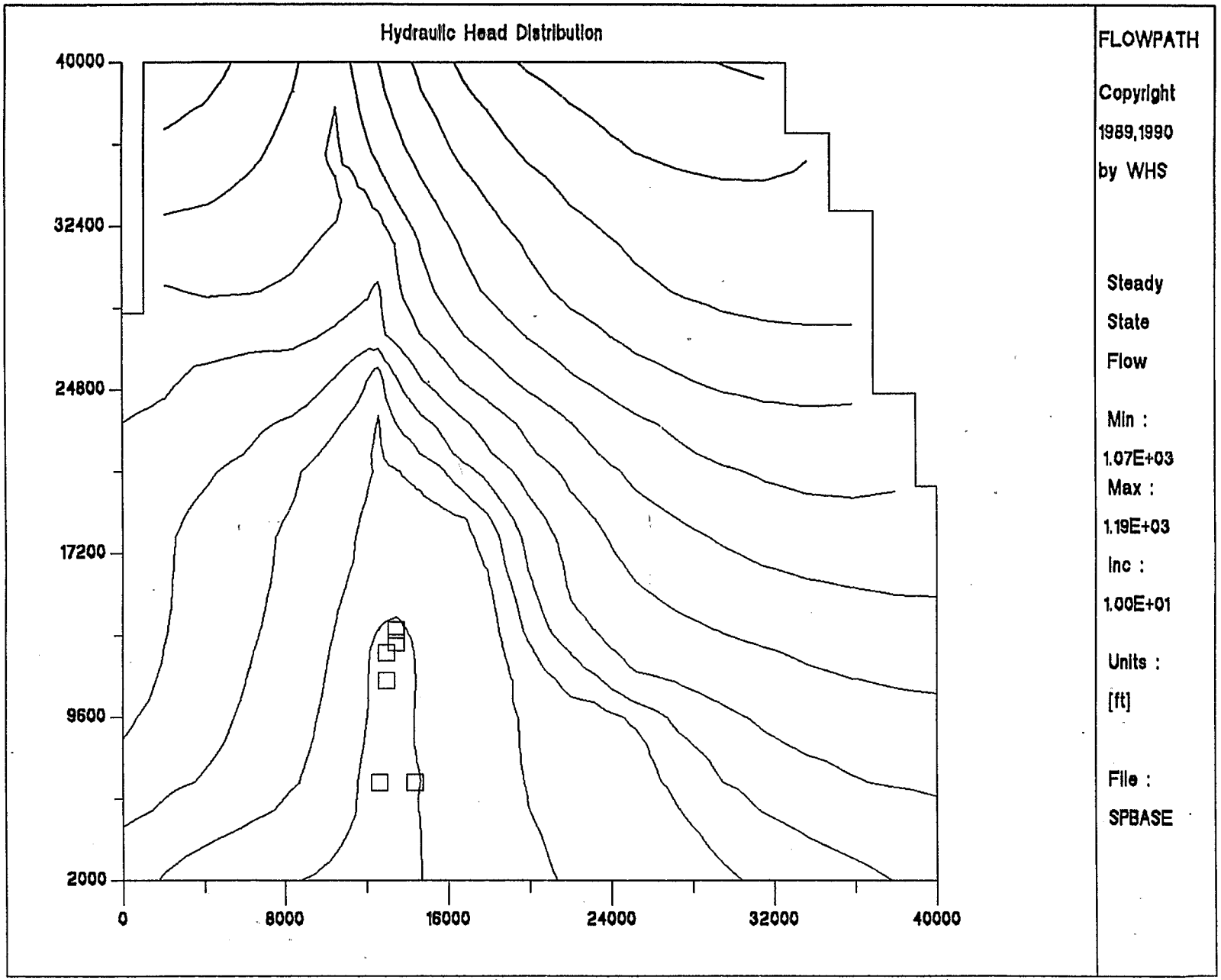
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Steady
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Flow
Min :
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Units :
[ft]
File :
SPBASE

****

F L O W P A T H
version 3.0

FLOWPATH was written by Thomas Franz and Nilson Guiguer *

****

Copyright 1989, 1990
by
Waterloo Hydrogeologic Software
113-106 Seagram Drive
Waterloo, Ontario
N2L 3B8, Canada

ph (519) 746-1798

****

FLOWPATH logbook for data set : SPBASE

Unit System : English units [ft/gal/d]

***** GRID PARAMETERS *****
Number of x-grid lines : 33
Number of y-grid lines : 34

Grid coordinates (x-grid lines) [ft] :				Grid coordinates (y-grid lines) [ft] :			
1	0.00000E+00	23	2.20418E+0	1	2.00000E+03425	23	2.32422E+04
2	2.10357E+03	24	2.32307E+0	2	4.24224E+03426	24	2.40683E+04
3	4.20714E+03	25	2.40539E+0	3	5.30435E+03427	25	2.52484E+04
4	6.31071E+03	26	2.52429E+0	4	6.24845E+03428	26	2.73727E+04
5	8.41429E+03	27	2.73464E+0	5	6.60248E+03429	27	2.93789E+04
6	9.60326E+03	28	2.94500E+0	6	7.54658E+03430	28	3.16211E+04
7	1.05179E+04	29	3.15536E+0	7	8.37267E+03431	29	3.26832E+04
8	1.12495E+04	30	3.36571E+0	8	9.55280E+03432	30	3.36273E+04
9	1.19812E+04	31	3.58522E+0	9	1.03789E+04433	31	3.46894E+04
10	1.26214E+04	32	3.79557E+0	10	1.13230E+04434	32	3.57516E+04
11	1.29873E+04	33	4.00000E+0	11	1.19130E+044	33	3.77578E+04
12	1.34446E+04			12	1.26211E+04	34	4.00000E+04
13	1.39019E+04			13	1.30932E+04		
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15	1.47250E+04			15	1.46273E+04		
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19	1.79261E+04			19	1.87578E+04		
20	1.89321E+04			20	1.97019E+044		
21	1.99382E+04			21	2.10000E+04		
22	2.11272E+04			22	2.18261E+04		

***** WELL PARAMETERS *****

Number of wells : 6

No.	i	j	X [ft]	Y [ft]	well discharge [gpd]
1	14	5	1.43592E+04	6.60248E+03	-1.71300E+06
2	11	10	1.29873E+04	1.13230E+04	-1.53300E+06
3	11	12	1.29873E+04	1.26211E+04	-2.47900E+06
4	12	13	1.34446E+04	1.30932E+04	-1.79900E+06
5	12	14	1.34446E+04	1.36832E+04	-1.57200E+06
6	10	5	1.26214E+04	6.60248E+03	-1.05600E+06

***** CONSTRAINED HEAD NODES *****

Number of constant head nodes : 1

No.	i	j	X [ft]	Y [ft]	const. head [ft]
1	14	17	1.43616E+04	1.68023E+04	1.08000E+03

***** AREAL RECHARGE *****

Number of different infiltration/evapotranspiration rates : 1

No.	infiltration [L/T]	evapotranspiration [L/T]	effective recharge [L/T]
1	7.30000E-03	3.50000E-03	3.80000E-03

***** AQUIFER TYPE *****

Unconfined aquifer

***** AQUIFER PROPERTIES *****

Number of different material properties : 5

No.	Kxx [ft/d]	Kyy [ft/d]	Porosity [-]	
1	1.00000E+02	1.00000E+02	3.20000E-01	(default)
2	2.60000E+02	2.60000E+02	3.20000E-01	
3	1.40000E+03	1.40000E+03	2.50000E-01	
4	5.20000E+02	5.20000E+02	3.20000E-01	
5	5.50000E+02	5.50000E+02	3.20000E-01	

********** DISTRIBUTION OF AQUIFER MATERIAL PROPERTIES **********

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22	2	2	3	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	*
21	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	*
20	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
19	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
18	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
17	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
16	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	2	1	1	1	2	2	2	2	2	2	2	2	2	2	2
15	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	2	1	1	1	2	2	2	2	2	2	2	2	2	2	2
14	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	2	1	1	1	2	2	2	2	2	2	2	2	2	2	2
13	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	2	1	1	1	2	1	1	1	2	2	2	2	2	2	2
12	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	2	1	1	1	2	1	1	1	2	2	2	2	2	2	2
11	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	2	1	1	1	2	1	1	1	2	2	2	2	2	2	2
10	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	2	1	1	1	2	1	1	1	2	2	2	2	2	2	2
9	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	2	1	1	1	2	1	1	1	2	2	2	2	2	2	2
8	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	3	1	1	1	2	2	2	2	2	2	2	2	2	2	2
7	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2
6	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2
5	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2
4	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2
3	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2
2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2
1	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	3	3	2	2	2	2	2	2	2

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33

***** AQUIFER BOTTOM ELEVATIONS *****

Number of different aquifer bottom elevations : 17

No.	aquifer bottom elevation [ft]	No.	aquifer bottom elevation [ft]
1	1.04000E+03 (default)	9	9.60000E+02
2	1.10000E+03	10	9.80000E+02
3	1.08000E+03	11	1.00000E+03
4	1.06000E+03	12	1.02000E+03
5	1.02000E+03	13	1.12000E+03
6	9.00000E+02	14	1.11500E+03
7	9.20000E+02	15	1.03000E+03
8	9.40000E+02	16	1.09000E+03
		17	1.07000E+03

********** DISTRIBUTION OF AQUIFER BOTTOM ELEVATIONS **********

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33
34	*	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	*	*	*	*
33	*	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	*	*	*	*
32	*	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	3	*	*	*
31	*	2	2	2	2	2	14	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	3	*	*	*
30	*	2	2	2	2	2	2	14	14	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	3	*	*	*
29	*	2	2	2	2	2	2	14	14	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	3	3	*	*
28	*	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	3	3	*	*
27	*	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	3	3	*	*
26	3	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	3	3	3	3	*	*
25	4	4	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	*	*
24	1	1	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	4	*
23	12	12	4	4	3	3	3	3	3	3	3	3	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	*
22	1	12	1	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	*
21	4	3	12	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	*
20	3	3	11	1	1	1	1	1	1	1	1	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4
19	3	3	11	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	4	4	4	4	4	4	4
18	16	3	11	10	10	10	9	9	9	9	9	9	9	9	9	8	8	8	1	1	1	1	1	1	1	1	4	4	4	4	4	4	4
17	16	3	4	4	1	1	1	1	1	1	1	1	11	11	11	11	11	7	1	1	1	1	1	1	1	1	4	4	4	4	4	4	4
16	16	3	4	4	1	1	1	1	1	1	1	1	12	12	12	12	7	1	1	1	1	1	1	1	1	4	4	4	4	4	4	4	4
15	16	3	17	4	1	1	1	1	1	1	1	1	12	12	12	12	6	1	1	4	4	4	1	1	1	1	1	1	1	4	4	4	4
14	16	16	17	4	1	1	1	1	1	1	1	1	12	12	12	12	6	1	1	4	4	4	1	1	1	1	1	1	1	4	4	4	4
13	16	16	3	4	1	1	1	1	1	1	1	1	12	12	12	12	6	1	1	4	4	4	1	1	1	1	1	1	1	1	4	4	4
12	16	16	3	4	1	1	1	1	1	1	1	1	12	12	12	12	6	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
11	16	16	3	4	1	1	1	1	1	1	1	1	12	12	12	12	6	1	1	1	1	1	1	1	1	1	1	5	5	1	1	1	
10	16	16	3	4	1	1	1	1	1	1	1	1	12	12	12	12	6	6	6	6	1	1	3	3	3	1	1	5	5	1	1	1	
9	16	16	3	4	1	1	1	1	1	1	1	1	12	12	12	12	12	12	12	6	1	1	3	3	3	1	5	5	5	1	1	1	
8	16	3	3	4	1	1	1	1	1	1	1	1	12	12	12	12	12	12	12	6	1	1	1	3	3	1	5	5	5	1	1	1	
7	16	3	17	4	1	1	1	1	1	1	1	1	12	12	12	12	12	12	12	6	6	6	1	1	1	1	5	5	5	5	5	5	
6	3	3	17	4	1	1	1	1	1	1	1	1	12	12	12	12	12	12	12	12	11	6	1	1	1	1	5	5	5	5	5	5	
5	3	3	4	4	1	1	1	1	1	1	1	1	12	12	12	12	12	12	12	12	11	6	1	1	1	5	5	5	5	5	5	5	
4	3	17	4	4	1	1	1	1	1	1	1	1	12	12	12	12	12	12	12	12	11	6	1	1	1	5	5	5	5	5	5	5	
3	17	17	4	4	1	1	1	1	1	1	1	1	12	12	12	12	12	12	12	12	11	6	6	6	1	5	5	5	5	5	5	5	
2	17	4	4	4	1	1	1	1	1	1	1	1	12	12	12	12	12	12	12	12	11	11	11	6	5	5	5	5	5	5	5	5	
1	4	4	4	4	1	1	1	1	1	1	1	1	15	15	15	15	12	12	12	12	11	11	11	6	5	5	5	5	5	5	5	5	

********** HYDRAULIC HEAD DISTRIBUTION **********

	1	2	3	4	5	6
34	*	1.1592E+03	1.1535E+03	1.1473E+03	1.1414E+03	1.1361E+03
33	*	1.1520E+03	1.1489E+03	1.1443E+03	1.1390E+03	1.1341E+03
32	*	1.1472E+03	1.1451E+03	1.1416E+03	1.1370E+03	1.1325E+03
31	*	1.1442E+03	1.1425E+03	1.1396E+03	1.1360E+03	1.1328E+03
30	*	1.1417E+03	1.1404E+03	1.1381E+03	1.1350E+03	1.1324E+03
29	*	1.1393E+03	1.1384E+03	1.1364E+03	1.1338E+03	1.1317E+03
28	*	1.1357E+03	1.1355E+03	1.1341E+03	1.1319E+03	1.1300E+03
27	*	1.1292E+03	1.1310E+03	1.1304E+03	1.1288E+03	1.1268E+03
26	1.1224E+03	1.1229E+03	1.1235E+03	1.1227E+03	1.1223E+03	1.1205E+03
25	1.1221E+03	1.1211E+03	1.1180E+03	1.1159E+03	1.1153E+03	1.1124E+03
24	1.1212E+03	1.1195E+03	1.1159E+03	1.1137E+03	1.1119E+03	1.1085E+03
23	1.1200E+03	1.1177E+03	1.1143E+03	1.1117E+03	1.1087E+03	1.1046E+03
22	1.1190E+03	1.1164E+03	1.1131E+03	1.1095E+03	1.1045E+03	1.1002E+03
21	1.1182E+03	1.1151E+03	1.1111E+03	1.1063E+03	1.1012E+03	1.0976E+03
20	1.1172E+03	1.1132E+03	1.1087E+03	1.1041E+03	1.1001E+03	1.0969E+03
19	1.1164E+03	1.1119E+03	1.1073E+03	1.1027E+03	1.0992E+03	1.0960E+03
18	1.1158E+03	1.1111E+03	1.1063E+03	1.1020E+03	1.0986E+03	1.0954E+03
17	1.1153E+03	1.1109E+03	1.1062E+03	1.1017E+03	1.0984E+03	1.0954E+03
16	1.1151E+03	1.1107E+03	1.1063E+03	1.1017E+03	1.0979E+03	1.0947E+03
15	1.1151E+03	1.1106E+03	1.1060E+03	1.1014E+03	1.0972E+03	1.0937E+03
14	1.1152E+03	1.1103E+03	1.1055E+03	1.1010E+03	1.0967E+03	1.0930E+03
13	1.1152E+03	1.1100E+03	1.1048E+03	1.1006E+03	1.0963E+03	1.0926E+03
12	1.1150E+03	1.1097E+03	1.1049E+03	1.1002E+03	1.0959E+03	1.0921E+03
11	1.1146E+03	1.1093E+03	1.1048E+03	1.0999E+03	1.0955E+03	1.0917E+03
10	1.1139E+03	1.1088E+03	1.1045E+03	1.0994E+03	1.0949E+03	1.0911E+03
9	1.1130E+03	1.1080E+03	1.1039E+03	1.0987E+03	1.0943E+03	1.0905E+03
8	1.1115E+03	1.1069E+03	1.1029E+03	1.0979E+03	1.0935E+03	1.0898E+03
7	1.1097E+03	1.1055E+03	1.1016E+03	1.0970E+03	1.0926E+03	1.0891E+03
6	1.1079E+03	1.1039E+03	1.1003E+03	1.0959E+03	1.0917E+03	1.0883E+03
5	1.1064E+03	1.1022E+03	1.0990E+03	1.0949E+03	1.0908E+03	1.0875E+03
4	1.1048E+03	1.1007E+03	1.0978E+03	1.0937E+03	1.0898E+03	1.0867E+03
3	1.1026E+03	1.0988E+03	1.0959E+03	1.0920E+03	1.0883E+03	1.0856E+03
2	1.0992E+03	1.0952E+03	1.0919E+03	1.0883E+03	1.0853E+03	1.0831E+03
1	1.0958E+03	1.0888E+03	1.0834E+03	1.0800E+03	1.0803E+03	1.0792E+03

	7	8	9	10	11	12
34	1.1316E+03	1.1397E+03	1.1457E+03	1.1500E+03	1.1529E+03	1.1555E+03
33	1.1299E+03	1.1366E+03	1.1418E+03	1.1457E+03	1.1484E+03	1.1509E+03
32	1.1277E+03	1.1332E+03	1.1379E+03	1.1415E+03	1.1442E+03	1.1466E+03
31	1.1300E+03	1.1280E+03	1.1336E+03	1.1376E+03	1.1404E+03	1.1429E+03
30	1.1307E+03	1.1291E+03	1.1276E+03	1.1337E+03	1.1367E+03	1.1395E+03
29	1.1302E+03	1.1289E+03	1.1277E+03	1.1273E+03	1.1318E+03	1.1355E+03
28	1.1285E+03	1.1272E+03	1.1261E+03	1.1254E+03	1.1251E+03	1.1302E+03
27	1.1251E+03	1.1233E+03	1.1211E+03	1.1186E+03	1.1237E+03	1.1276E+03
26	1.1187E+03	1.1169E+03	1.1153E+03	1.1175E+03	1.1200E+03	1.1225E+03
25	1.1094E+03	1.1058E+03	1.1008E+03	1.0933E+03	1.1029E+03	1.1085E+03
24	1.1053E+03	1.1017E+03	1.0971E+03	1.0910E+03	1.0986E+03	1.1032E+03
23	1.1013E+03	1.0980E+03	1.0942E+03	1.0893E+03	1.0954E+03	1.0996E+03
22	1.0972E+03	1.0944E+03	1.0914E+03	1.0880E+03	1.0908E+03	1.0933E+03
21	1.0950E+03	1.0928E+03	1.0908E+03	1.0894E+03	1.0886E+03	1.0895E+03
20	1.0943E+03	1.0921E+03	1.0900E+03	1.0882E+03	1.0881E+03	1.0882E+03
19	1.0935E+03	1.0912E+03	1.0890E+03	1.0870E+03	1.0868E+03	1.0865E+03
18	1.0929E+03	1.0907E+03	1.0885E+03	1.0866E+03	1.0860E+03	1.0854E+03
17	1.0928E+03	1.0904E+03	1.0877E+03	1.0850E+03	1.0841E+03	1.0830E+03
16	1.0918E+03	1.0889E+03	1.0856E+03	1.0830E+03	1.0823E+03	1.0822E+03
15	1.0905E+03	1.0873E+03	1.0838E+03	1.0816E+03	1.0812E+03	1.0810E+03
14	1.0897E+03	1.0862E+03	1.0824E+03	1.0799E+03	1.0792E+03	1.0785E+03
13	1.0891E+03	1.0855E+03	1.0814E+03	1.0786E+03	1.0777E+03	1.0770E+03
12	1.0887E+03	1.0850E+03	1.0808E+03	1.0777E+03	1.0760E+03	1.0774E+03
11	1.0882E+03	1.0846E+03	1.0805E+03	1.0779E+03	1.0775E+03	1.0779E+03
10	1.0877E+03	1.0842E+03	1.0803E+03	1.0778E+03	1.0769E+03	1.0777E+03
9	1.0872E+03	1.0839E+03	1.0803E+03	1.0782E+03	1.0778E+03	1.0777E+03
8	1.0866E+03	1.0834E+03	1.0801E+03	1.0781E+03	1.0778E+03	1.0776E+03
7	1.0859E+03	1.0828E+03	1.0794E+03	1.0773E+03	1.0768E+03	1.0779E+03
6	1.0852E+03	1.0821E+03	1.0787E+03	1.0766E+03	1.0762E+03	1.0775E+03
5	1.0846E+03	1.0815E+03	1.0780E+03	1.0752E+03	1.0755E+03	1.0769E+03
4	1.0840E+03	1.0812E+03	1.0781E+03	1.0760E+03	1.0757E+03	1.0769E+03
3	1.0832E+03	1.0808E+03	1.0782E+03	1.0766E+03	1.0764E+03	1.0762E+03
2	1.0813E+03	1.0796E+03	1.0779E+03	1.0766E+03	1.0757E+03	1.0747E+03
1	1.0782E+03	1.0770E+03	1.0758E+03	1.0748E+03	1.0740E+03	1.0732E+03

	13	14	15	16	17	18
34	1.1580E+03	1.1603E+03	1.1627E+03	1.1657E+03	1.1687E+03	1.1721E+03
33	1.1533E+03	1.1555E+03	1.1578E+03	1.1606E+03	1.1634E+03	1.1665E+03
32	1.1490E+03	1.1511E+03	1.1534E+03	1.1562E+03	1.1590E+03	1.1620E+03
31	1.1454E+03	1.1477E+03	1.1501E+03	1.1529E+03	1.1558E+03	1.1589E+03
30	1.1423E+03	1.1447E+03	1.1473E+03	1.1503E+03	1.1533E+03	1.1565E+03
29	1.1388E+03	1.1416E+03	1.1444E+03	1.1476E+03	1.1508E+03	1.1541E+03
28	1.1343E+03	1.1376E+03	1.1408E+03	1.1443E+03	1.1477E+03	1.1511E+03
27	1.1311E+03	1.1340E+03	1.1370E+03	1.1403E+03	1.1437E+03	1.1471E+03
26	1.1251E+03	1.1276E+03	1.1302E+03	1.1334E+03	1.1366E+03	1.1400E+03
25	1.1129E+03	1.1164E+03	1.1197E+03	1.1234E+03	1.1270E+03	1.1308E+03
24	1.1069E+03	1.1099E+03	1.1130E+03	1.1165E+03	1.1202E+03	1.1242E+03
23	1.1026E+03	1.1049E+03	1.1075E+03	1.1109E+03	1.1147E+03	1.1188E+03
22	1.0957E+03	1.0979E+03	1.1005E+03	1.1046E+03	1.1088E+03	1.1133E+03
21	1.0904E+03	1.0913E+03	1.0924E+03	1.0966E+03	1.1018E+03	1.1067E+03
20	1.0884E+03	1.0888E+03	1.0891E+03	1.0907E+03	1.0936E+03	1.0975E+03
19	1.0863E+03	1.0863E+03	1.0865E+03	1.0868E+03	1.0877E+03	1.0892E+03
18	1.0849E+03	1.0848E+03	1.0851E+03	1.0858E+03	1.0866E+03	1.0875E+03
17	1.0817E+03	1.0800E+03	1.0824E+03	1.0843E+03	1.0857E+03	1.0865E+03
16	1.0821E+03	1.0823E+03	1.0829E+03	1.0839E+03	1.0850E+03	1.0857E+03
15	1.0815E+03	1.0820E+03	1.0826E+03	1.0834E+03	1.0844E+03	1.0850E+03
14	1.0799E+03	1.0810E+03	1.0819E+03	1.0829E+03	1.0840E+03	1.0845E+03
13	1.0790E+03	1.0804E+03	1.0815E+03	1.0826E+03	1.0837E+03	1.0843E+03
12	1.0787E+03	1.0801E+03	1.0812E+03	1.0824E+03	1.0835E+03	1.0841E+03
11	1.0786E+03	1.0799E+03	1.0810E+03	1.0822E+03	1.0834E+03	1.0840E+03
10	1.0785E+03	1.0798E+03	1.0809E+03	1.0821E+03	1.0833E+03	1.0840E+03
9	1.0790E+03	1.0800E+03	1.0809E+03	1.0820E+03	1.0831E+03	1.0840E+03
8	1.0790E+03	1.0801E+03	1.0810E+03	1.0821E+03	1.0832E+03	1.0844E+03
7	1.0791E+03	1.0801E+03	1.0810E+03	1.0822E+03	1.0834E+03	1.0849E+03
6	1.0787E+03	1.0797E+03	1.0808E+03	1.0821E+03	1.0835E+03	1.0851E+03
5	1.0781E+03	1.0786E+03	1.0804E+03	1.0820E+03	1.0836E+03	1.0851E+03
4	1.0782E+03	1.0793E+03	1.0806E+03	1.0821E+03	1.0836E+03	1.0852E+03
3	1.0781E+03	1.0795E+03	1.0807E+03	1.0823E+03	1.0837E+03	1.0852E+03
2	1.0733E+03	1.0776E+03	1.0809E+03	1.0831E+03	1.0840E+03	1.0852E+03
1	1.0725E+03	1.0767E+03	1.0805E+03	1.0831E+03	1.0840E+03	1.0851E+03

	19	20	21	22	23	24
34	1.1761E+03	1.1790E+03	1.1811E+03	1.1828E+03	1.1843E+03	1.1856E+03
33	1.1699E+03	1.1728E+03	1.1753E+03	1.1775E+03	1.1793E+03	1.1808E+03
32	1.1653E+03	1.1684E+03	1.1710E+03	1.1734E+03	1.1753E+03	1.1770E+03
31	1.1623E+03	1.1654E+03	1.1681E+03	1.1706E+03	1.1726E+03	1.1743E+03
30	1.1599E+03	1.1631E+03	1.1659E+03	1.1684E+03	1.1705E+03	1.1723E+03
29	1.1577E+03	1.1609E+03	1.1638E+03	1.1663E+03	1.1685E+03	1.1704E+03
28	1.1548E+03	1.1581E+03	1.1610E+03	1.1637E+03	1.1659E+03	1.1678E+03
27	1.1508E+03	1.1542E+03	1.1572E+03	1.1600E+03	1.1623E+03	1.1644E+03
26	1.1439E+03	1.1474E+03	1.1507E+03	1.1536E+03	1.1562E+03	1.1584E+03
25	1.1350E+03	1.1389E+03	1.1425E+03	1.1458E+03	1.1487E+03	1.1513E+03
24	1.1286E+03	1.1329E+03	1.1369E+03	1.1405E+03	1.1437E+03	1.1465E+03
23	1.1236E+03	1.1282E+03	1.1325E+03	1.1364E+03	1.1399E+03	1.1428E+03
22	1.1185E+03	1.1236E+03	1.1283E+03	1.1326E+03	1.1362E+03	1.1394E+03
21	1.1127E+03	1.1185E+03	1.1239E+03	1.1286E+03	1.1326E+03	1.1360E+03
20	1.1053E+03	1.1127E+03	1.1192E+03	1.1246E+03	1.1290E+03	1.1326E+03
19	1.0978E+03	1.1078E+03	1.1154E+03	1.1214E+03	1.1261E+03	1.1299E+03
18	1.0932E+03	1.1044E+03	1.1127E+03	1.1192E+03	1.1240E+03	1.1278E+03
17	1.0907E+03	1.1018E+03	1.1106E+03	1.1174E+03	1.1222E+03	1.1258E+03
16	1.0881E+03	1.0982E+03	1.1084E+03	1.1163E+03	1.1211E+03	1.1240E+03
15	1.0867E+03	1.0949E+03	1.1051E+03	1.1140E+03	1.1192E+03	1.1222E+03
14	1.0860E+03	1.0930E+03	1.1022E+03	1.1110E+03	1.1164E+03	1.1203E+03
13	1.0856E+03	1.0918E+03	1.0999E+03	1.1081E+03	1.1131E+03	1.1179E+03
12	1.0853E+03	1.0909E+03	1.0980E+03	1.1055E+03	1.1103E+03	1.1150E+03
11	1.0852E+03	1.0898E+03	1.0955E+03	1.1028E+03	1.1074E+03	1.1122E+03
10	1.0853E+03	1.0891E+03	1.0933E+03	1.0991E+03	1.1036E+03	1.1081E+03
9	1.0855E+03	1.0892E+03	1.0925E+03	1.0960E+03	1.0992E+03	1.1007E+03
8	1.0857E+03	1.0888E+03	1.0912E+03	1.0928E+03	1.0944E+03	1.0953E+03
7	1.0867E+03	1.0891E+03	1.0907E+03	1.0917E+03	1.0926E+03	1.0937E+03
6	1.0869E+03	1.0889E+03	1.0906E+03	1.0918E+03	1.0925E+03	1.0934E+03
5	1.0869E+03	1.0887E+03	1.0904E+03	1.0917E+03	1.0924E+03	1.0931E+03
4	1.0869E+03	1.0886E+03	1.0902E+03	1.0915E+03	1.0922E+03	1.0928E+03
3	1.0868E+03	1.0884E+03	1.0899E+03	1.0913E+03	1.0921E+03	1.0926E+03
2	1.0866E+03	1.0880E+03	1.0894E+03	1.0906E+03	1.0915E+03	1.0923E+03
1	1.0863E+03	1.0875E+03	1.0888E+03	1.0898E+03	1.0907E+03	1.0915E+03

	25	26	27	28	29	30	31	32	33	
34	1.1867E+03	1.1879E+03	1.1891E+03	1.1901E+03	1.1916E+03	*	*			
33	1.1821E+03	1.1835E+03	1.1850E+03	1.1860E+03	1.1866E+03	*	*			
32	1.1784E+03	1.1799E+03	1.1815E+03	1.1825E+03	1.1824E+03	1.1803E+03	*			
31	1.1758E+03	1.1775E+03	1.1792E+03	1.1802E+03	1.1802E+03	1.1792E+03	*			
30	1.1738E+03	1.1755E+03	1.1774E+03	1.1784E+03	1.1786E+03	1.1780E+03	*			
29	1.1719E+03	1.1737E+03	1.1756E+03	1.1768E+03	1.1771E+03	1.1766E+03	1.1749E+03	*		
28	1.1695E+03	1.1714E+03	1.1734E+03	1.1747E+03	1.1752E+03	1.1751E+03	1.1743E+03	*		
27	1.1661E+03	1.1681E+03	1.1704E+03	1.1720E+03	1.1727E+03	1.1729E+03	1.1727E+03	*		
26	1.1604E+03	1.1627E+03	1.1654E+03	1.1675E+03	1.1686E+03	1.1691E+03	1.1692E+03	*		
25	1.1535E+03	1.1561E+03	1.1592E+03	1.1618E+03	1.1633E+03	1.1640E+03	1.1640E+03	*		
24	1.1489E+03	1.1517E+03	1.1549E+03	1.1577E+03	1.1593E+03	1.1601E+03	1.1597E+03	1.1563E+03	*	
23	1.1454E+03	1.1484E+03	1.1518E+03	1.1546E+03	1.1564E+03	1.1573E+03	1.1570E+03	1.1554E+03	*	
22	1.1421E+03	1.1452E+03	1.1487E+03	1.1517E+03	1.1536E+03	1.1546E+03	1.1546E+03	1.1538E+03	*	
21	1.1389E+03	1.1421E+03	1.1457E+03	1.1489E+03	1.1509E+03	1.1520E+03	1.1522E+03	1.1517E+03	*	
20	1.1356E+03	1.1389E+03	1.1426E+03	1.1460E+03	1.1482E+03	1.1494E+03	1.1498E+03	1.1493E+03	1.1469E+03	
19	1.1329E+03	1.1363E+03	1.1400E+03	1.1434E+03	1.1457E+03	1.1470E+03	1.1476E+03	1.1473E+03	1.1460E+03	
18	1.1308E+03	1.1342E+03	1.1377E+03	1.1409E+03	1.1432E+03	1.1447E+03	1.1454E+03	1.1454E+03	1.1448E+03	
17	1.1287E+03	1.1319E+03	1.1352E+03	1.1381E+03	1.1404E+03	1.1420E+03	1.1429E+03	1.1432E+03	1.1431E+03	
16	1.1266E+03	1.1296E+03	1.1327E+03	1.1352E+03	1.1374E+03	1.1390E+03	1.1402E+03	1.1409E+03	1.1410E+03	
15	1.1248E+03	1.1277E+03	1.1303E+03	1.1326E+03	1.1347E+03	1.1364E+03	1.1377E+03	1.1385E+03	1.1388E+03	
14	1.1232E+03	1.1262E+03	1.1283E+03	1.1304E+03	1.1325E+03	1.1343E+03	1.1357E+03	1.1364E+03	1.1368E+03	
13	1.1215E+03	1.1249E+03	1.1266E+03	1.1288E+03	1.1308E+03	1.1326E+03	1.1342E+03	1.1348E+03	1.1352E+03	
12	1.1192E+03	1.1230E+03	1.1250E+03	1.1272E+03	1.1293E+03	1.1312E+03	1.1328E+03	1.1336E+03	1.1340E+03	
11	1.1169E+03	1.1210E+03	1.1231E+03	1.1254E+03	1.1277E+03	1.1296E+03	1.1314E+03	1.1323E+03	1.1327E+03	
10	1.1133E+03	1.1179E+03	1.1207E+03	1.1232E+03	1.1259E+03	1.1280E+03	1.1297E+03	1.1307E+03	1.1312E+03	
9	1.1060E+03	1.1113E+03	1.1174E+03	1.1207E+03	1.1238E+03	1.1261E+03	1.1278E+03	1.1289E+03	1.1294E+03	
8	1.0976E+03	1.1020E+03	1.1130E+03	1.1179E+03	1.1213E+03	1.1238E+03	1.1255E+03	1.1267E+03	1.1273E+03	
7	1.0954E+03	1.0976E+03	1.1080E+03	1.1148E+03	1.1187E+03	1.1214E+03	1.1232E+03	1.1245E+03	1.1251E+03	
6	1.0949E+03	1.0968E+03	1.1048E+03	1.1121E+03	1.1163E+03	1.1192E+03	1.1212E+03	1.1225E+03	1.1233E+03	
5	1.0943E+03	1.0960E+03	1.1028E+03	1.1099E+03	1.1143E+03	1.1173E+03	1.1195E+03	1.1210E+03	1.1218E+03	
4	1.0937E+03	1.0953E+03	1.1015E+03	1.1082E+03	1.1127E+03	1.1158E+03	1.1180E+03	1.1196E+03	1.1206E+03	
3	1.0932E+03	1.0943E+03	1.0999E+03	1.1062E+03	1.1106E+03	1.1138E+03	1.1161E+03	1.1178E+03	1.1190E+03	
2	1.0927E+03	1.0934E+03	1.0975E+03	1.1029E+03	1.1072E+03	1.1105E+03	1.1128E+03	1.1149E+03	1.1166E+03	
1	1.0920E+03	1.0924E+03	1.0943E+03	1.0981E+03	1.1021E+03	1.1054E+03	1.1074E+03	1.1102E+03	1.1137E+03	

***** PATHLINE & PARTICLE TRACKING DATA *****

Number of forward particles : 0

Number of reverse particles : 0

Particles released at wells :

Well-No.	Particles released
1	10
2	10
3	10
4	10
5	10
6	10

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CHAPTER 3

MONITORING SYSTEM DESIGN FOR WELLHEAD PROTECTION LITTLETON, MASSACHUSETTS

by

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NOTICE

This report is the result of research supported by the U.S. Environmental Protection Agency's Environmental Monitoring Systems Laboratory in Las Vegas, Nevada, as part of its efforts to provide technical assistance to state, tribal, and local governments on the implementation of the Wellhead Protection Program. The specific methods and approaches contained in this document have undergone peer review but do not constitute official Agency endorsement or policy recommendations. The Office of Research and Development provides this information to help solve complex technical problems related to refined delineation and ground-water monitoring of wellhead protection areas in various hydrogeologic settings. Further assistance is available from the Environmental Monitoring Systems Laboratory in Las Vegas, the Office of Ground Water and Drinking Water in Washington, D.C., and the ground-water offices in the ten U.S. Environmental Protection Agency regions.

ABSTRACT

As a management approach for wellhead protection of Production Well Number 5, PW-5, the Littleton Water Department proposes source-assessment and early-warning detection monitoring. Techniques for refined delineation of the PW-5 wellhead protection area and long-term monitoring approaches are discussed in this report. Production Well Number 5 is completed at a depth of 167 feet within saturated, stratified valley-fill deposits. The aquifer is unconfined and it receives significant surface-water recharge (20% to 25%) from nearby Spectacle Pond and Bennetts Brook.

Land-use activities within the wellhead protection area cover a broad range of commercial, industrial, and to a lesser degree, agricultural operations. Collectively, these land-use activities pose potential contamination threats to the aquifer, including heavy metals, volatile organic compounds, pesticides, and nutrients. Baseline monitoring results indicate that, to date, ground-water quality within the capture zone of PW-5 is virtually unaffected by source operations. Sodium is the only exception. Slightly elevated levels of sodium in surface water and the shallow aquifer are attributed to roadway de-icing activities. Manganese and iron concentrations are elevated throughout the recharge area of PW-5, primarily because of their occurrence in wetland sediments and glacial deposits. The levels of these parameters have increased at PW-5 for several years and may warrant treatment in the future.

The PW-5 wellhead protection area consists of three protection zones delineated using a combination of numerical ground-water flow modeling (FLOWPATH, FLOWCAD, and MODFLOW) and hydrogeologic mapping. Zone I is the 400-foot sanitary protective radius mandated by the State of Massachusetts. Zone II is the most critical management area and was delineated conservatively as the union of three numerical capture zone solutions. These numerical solutions incorporate two- and three-dimensional flow, as well as steady-state and transient flow conditions. Local and regional ground-water flow simulations are based on the results of short- and long-term aquifer testing.

Zone II generally represents the steady-state capture zone for PW-5 that corresponds approximately to the 400-day travel-time contour. Flowpath simulations indicate that Zone II extends to the bottom of the aquifer and is constrained by bedrock and glacial till. Within Zone II, three existing wells and two new wells are proposed for inclusion in the long-term monitoring network for early-warning detection and source assessment purposes. These wells lie along either the 150-day or the 300-day travel-time contours. Screened intervals for the new monitoring wells were chosen based on results from MODPATH computer flow simulations. Monitoring parameter groups for these wells include general water quality parameters (cations, anions, alkalinity, etc.), site-specific parameters (total coliform, heavy metals, volatile organic compounds, etc.), and physical parameters (water level, pH, temperature, etc.). Recommended monitoring frequencies for these parameter groups vary from quarterly to annually, depending on the travel-time distance from the monitoring well to PW-5 and the monitoring well depth.

Zone III is defined as the upgradient area of the aquifer that contributes to Zone II and extends to the watershed boundary. Zone III is monitored at two surface-water locations, one at the inflow and one at the outflow of Spectacle Pond. In addition, Zone III is monitored biannually at existing compliance networks around waste management and industrial sites. Monitoring parameters for the compliance wells include general water quality, site-specific, and physical parameter groups. Monitoring results ultimately support land-use planning, risk-assessment screening of new developers, existing development compliance, and general Wellhead Protection Program management.

The Littleton Wellhead Protection Program incorporates contingency planning. Catastrophic releases initiate a spill-response plan that involves many departments and agencies. In the event of contamination of PW-5 or another production well, Littleton has sited a new production well. The proposed well site is approved by the State, and protection Zones I, II, and III are delineated. The adjacent town of Boxborough shares the recharge area to the proposed well. Boxborough has adopted complementary strategies with Littleton to ensure its water quality protection.

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ABBREVIATIONS, SYMBOLS, AND CONVERSION FACTORS

ABBREVIATIONS

2-D	two-dimensional
3-D	three-dimensional
A	annually
amsl	above mean sea level
BA	biannually
BP	before pumping
C	Cooper-Jacob corrected drawdown analysis
cfs	cubic foot per second
Cond	specific conductivity
d	day
DEP	(Massachusetts) Department of Environmental Protection
DNAPL	dense, nonaqueous phase liquid
elev	elevation
EPA	U.S. Environmenal Protection Agency
ft	foot
ft/d	foot per day
ft^2/d	square foot per day
ft^2/min	square foot per minute
GIS	Geographic Information System
gpd	gallon per day
gpd/ft	gallon per day per foot
gpf	gallon per foot
gpm	gallon per minute
GWQ	general water quality parameters
H	Hantush analysis with aquitard storage
Hard	hardness
Herb	herbicide
HM	heavy metal parameters
in	inch
in/yr	inch per year
l	liter
LF1	miscellaneous monitoring station number 1
LNAPL	light, nonaqueous phase liquid
µg/l	microgram per liter
µS/cm	microSieman per centimeter
M	monthly
mg	milligram

mg/l	milligram per liter
Mgd	millon gallon per day
min	minute
MPA	microscopic particulate analysis
MW	monitoring well
MW-25d	monitoring well, 25 feet from PW-5, deep well
MW-25s	monitoring well, 25 feet from PW-5, shallow depth well
MW-100m	monitoring well, 100 feet from PW-5, moderate depth well
MW-C	compliance monitoring well
MW-E	existing monitoring well
MW-P	proposed monitoring well
N	Neuman analysis
N/A	not applicable
ND	not detected
NR	no response
NRC	National Response Center
OD	outside diameter
OW	observation well
P	piezometer
PCB	polychlorinated biphenyl
Pest	pesticide
Pt	pumping test
PVC	polyvinyl chloride
PW	production well
PW-5	Production Well Number 5
Q	quarterly
QA	quality assurance
QC	quality control
Rt	recovery test
S1	staff guage number 1
SCH	schedule
SDWA	Safe Drinking Water Act
SS	stainless steel
SW	surface-water monitoring station
TC	total coliform
TDD	Theis drawdown versus distance analysis
TDS	total dissolved solid
Temp	temperature
Th	Theis corrected drawdown analysis
TOT	time of travel
USGS	U.S. Geological Survey
UST	underground storage tank
VOC	volatile organic compound
WHPA	wellhead protection area
WHPP	Wellhead Protection Program
WL	water level

yr	year
ZOC	zone of contribution
ZOI	zone of influence

SYMBOLS

Ag	silver
As	arsenic
Ba	barium
Ca	calcium
Cd	cadmium
Cl	chlorine
Cr	chromium
Cu	copper
°F	degree Fahrenheit
Fe	iron
HCO_3	bicarbonate
Hg	mercury
K	potassium
Mg	magnesium
Mn	manganese
Na	sodium
NH_3	ammonia
NO_2	nitrite-nitrogen
NO_3	nitratre-nitrogen
Pb	lead
Se	selenium
SO_4	sulfate

β	Neuman analysis-fitting parameter
b	saturated aquifer thickness
K	hydraulic conductivity
Kr	horizontal hydraulic conductivity
Kz	vertical hydraulic conductivity
Kr/Kz	anisotropy ratio
n	porosity
r	radius
R	distance from Production Well Number 5 to monitoring station
S	storativity
Sy	specific yield
T	transmissivity

CONVERSION FACTORS

Multiply	By	To Obtain
acre-foot	1230	cubic meter
cubic foot per second	0.0283	cubic meter per second
foot	0.3048	meter
foot per second	0.3048	meter per second
gallon	3.785	liter
gallon	0.134	cubic foot
gallon	0.00379	cubic meter
gallon per day	0.000003528	cubic foot per second
gallon per day per foot	0.000207	square meter per day
gallon per day per square foot	0.0408	meter per day
inch	0.0254	meter
inch per year	25.4	millimeter per year
mile	1.609	kilometer
million gallons per day	2.629	cubic meter per minute
square foot per minute	0.0929	square meter per minute
square foot per second	0.0929	square meter per second
square mile	2.59	square kilometer

$$°F = \frac{9}{5}(°C) + 32$$

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BACKGROUND FOR THE LITTLETON CASE STUDY

INTRODUCTION

Five years before enactment of the U.S. Environmental Protection Agency's (EPA) Wellhead Protection Program (WHPP), the town of Littleton, Massachusetts, researched and adopted a comprehensive aquifer and watershed protection program that has served as a model for state and local governments. Littleton is a semi-rural community that has experienced significant commercial and industrial development over the past 10 years. The town is located in northeast Massachusetts, approximately 35 miles northwest of Boston, at the intersection of two major transportation routes, I-495 and Route 2 (Figure 3-1). Development of the Littleton WHPP is briefly discussed in this paper, together with recent refinements in wellhead protection area (WHPA) delineation and ground-water monitoring in proximity to Production Well Number 5 (PW-5), located within the Bennetts Brook watershed (Figure 3-2).

Since the inception of Littleton's aquifer and water resource protection plan, ground-water monitoring activities have played an integral role in the overall success of the program. Because of the formidable time and expense required to conduct these monitoring activities, it became necessary to establish an optimized monitoring program. The benefits of such a program include more reliable ground-water monitoring, reduced analytical costs (otherwise misappropriated at improperly sited monitoring stations), and increased efficiency for related WHPP efforts. Optimized monitoring requires accurate WHPA delineation as a prerequisite for effective land-use planning and management policy development.

At present, the Spectacle Pond aquifer (Figure 3-2) is a primary water resource for the town of Littleton. Because of its high capacity, good water quality, and potential vulnerability to future development, this aquifer was selected for reevaluation using recently developed technologies. Specifically, computer simulation techniques were employed to delineate the WHPA with associated ground-water flowpaths and travel times accurately. The goal of this research was to create a water quality monitoring program within the refined WHPA established for PW-5. These activities will ensure the protection of this water supply in the future.

WELLHEAD PROTECTION PROGRAM OVERVIEW

Littleton's boundaries encompass 16 square miles, and the population of the town is approximately 7,300. Over two-thirds of the community are serviced by the public water supply system. The daily water demand varies from 800,000 to 1,500,000 gallons per day (gpd) and is met by four ground-water production wells located in two separate watersheds (Bennetts and Beaver Brook watersheds, Figure 3-2). The total yield of these wells is approximately 2,800,000 gpd. Residents outside of the town's distribution system generally depend on low-yielding bedrock wells to meet their

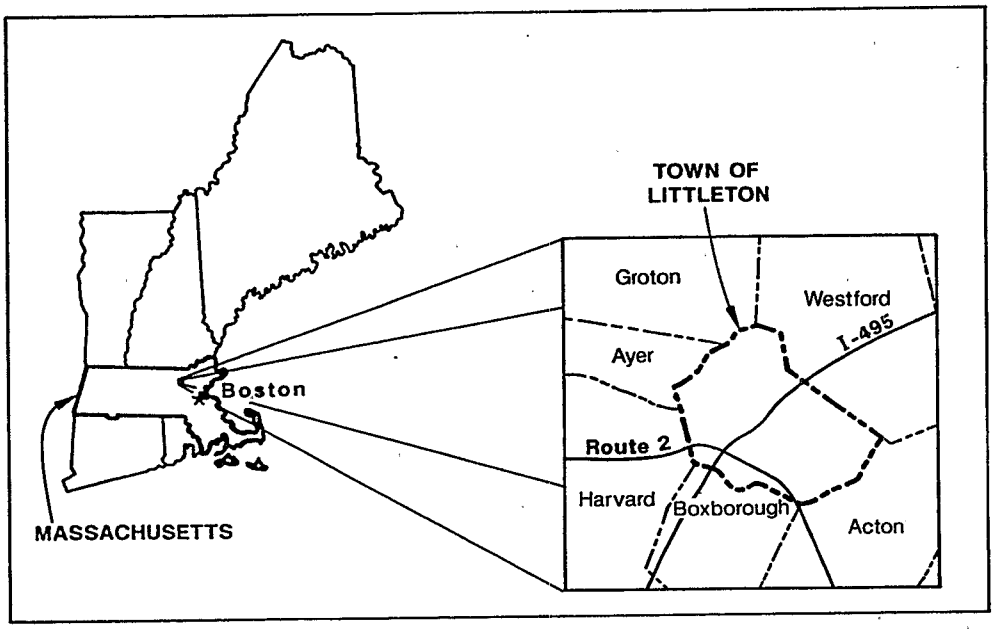

Figure 3-1. Regional location of the town of Littleton in northeastern Massachusetts. (Moore et al., 1990, p. 184)

water needs. At present, there is no public sewage collection system, and sanitary wastes are discharged into individual septic systems. Some industries discharge waste in excess of 15,000 gpd, and these industries are serviced by independent secondary or tertiary treatment facilities.

In 1979, the citizens of Littleton formed an Ad Hoc Committee for Ground-Water Protection to administer a town-wide hydrogeologic evaluation. The study was conducted in response to an estimated 70% loss of ground-water supply capacity by the town of Acton, which borders Littleton on the southeast (Figure 3-2). The loss in capacity was attributed to organic contamination from a nearby industrial development. Littleton's citizens appropriated $70,000 to hire a consultant to complete a hydrogeologic investigation, to site new ground-water development areas, and to recommend protection strategies for existing and proposed supplies. In 1981, the consultants made recommendations, which served as the foundation for the creation of zoning overlay districts, the adoption of a hazardous materials bylaw, and the adoption of other local health and wetlands protection ordinances.

Most preexisting industrial and commercial facilities that pose a contamination risk and are located within overlay districts willingly cooperate with regulatory agencies in protecting supply wells. Proposed developments within overlay districts are subject to a risk-assessment screening process. If low to moderate contaminant risk is determined and a development permit is issued, then the developer must demonstrate nondegradation of ground-water quality at the property perimeter. Compliance is provided by certified laboratory analyses and maintained through scheduled sampling of onsite monitoring wells.

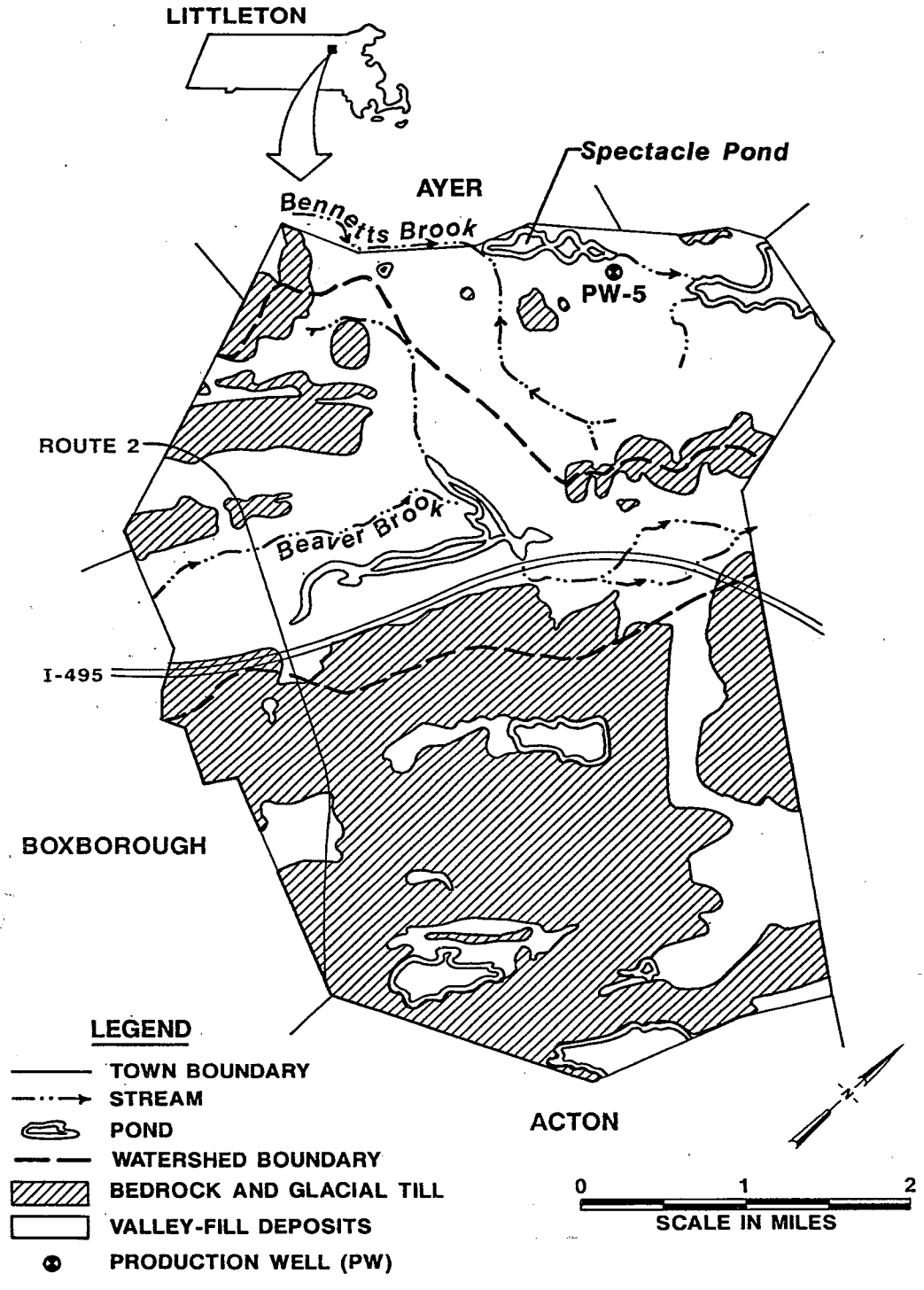

Figure 3-2. Location of PW-5, Spectacle Pond, and the distribution of valley-fill deposits within the Bennetts Brook watershed, Littleton, Massachusetts. (Modified after Moore et al., 1990, p. 185)

Town governing boards such as the Planning Board, the Board of Health, and the Conservation Commission participate in regulation of the WHPP. Daily implementation of ground-water monitoring, data collection, source assessment inspections, and overall management is performed by the Littleton Water Department. Hydrogeologic and engineering consultants are hired as needed. Development of the WHPP in Littleton is guided by the Board of Water Commissioners and the General Manager of the Water Department, who have been instrumental in the evolution of the water resource protection strategy and have encouraged its progression.

HYDROGEOLOGIC SETTING

Littleton is situated in a broad lowland belt where Pleistocene-age glacial and postglacial deposits form a discontinuous mantle over igneous and metamorphic rocks of Paleozoic age (Jahns, 1953). Bedrock underlying the Spectacle Pond study area is identified as the Lower Silurian Chelmsford facies, composed of a porphyritic biotite granite. Topographically prominent features owe their expression to resistive schists and gneisses, whereas other elongated highs are composed of glacial drift. The northeast-trending orientation of bedrock eminences reflect structural trends associated with a high-angle thrust fault, regionally known as the Clinton-Newbury fault system, and Tertiary Period drainage patterns (Zen, 1983). Drumlin, striae, and groove orientations indicate a southeast advancement and depositional environment attributed to the Wisconsin ice sheet (Jahns, 1953).

The hydrogeologic setting of public ground-water resources in Littleton is characterized as an unconfined to semi-confined, buried valley system. Surface water and ground water generally flow northeast, parallel to the strike of the fault system within the Bennetts Brook and Beaver Brook watersheds (Figure 3-2). Typically, these valley-fill deposits include outwash sands and gravels; glaciolacustrine deposits with sequences of gravel, sand, silt, and carved clay; ice contact features of boulder to sand-size fractions; and postglacial peat, muck, silt, and fine sands indicative of wetland environments.

To simplify the local hydrogeologic description, the areal extent of relatively impermeable bedrock and glacial till deposits is represented by the shaded areas in Figure 3-2. Conversely, the unshaded areas consist of highly-permeable outwash sands and gravels, as well as moderately-permeable stratified drift and recent deposits. Figure 3-2 shows the predominance of low-yield bedrock and glacial tills in the southern portion of town.

North-to-south trending gravity profiles completed across the Beaver Brook watershed and test-well depth data indicate that bedrock depths range from 0 to 140 feet (Kick, 1980). Similar profiles and test wells completed within the Bennetts Brook watershed indicate that bedrock depths range from 0 to 105 feet. Depths to water typically range from 0 to 25 feet and 0 to 35 feet within the Beaver Brook and Bennetts Brook watersheds, respectively. Other significant aquifer property ranges include hydraulic conductivities between 1,200 and 3,100 gallons per day per foot (gpd/ft); porosities between 25% and 35%; and storativities between 0.05 and 0.0003 (Goldberg-Zoino & Associates Inc., 1987; D. L. Maher Co., 1985). Aquifer studies indicate that semi-confined conditions exist in areas where peat and muck deposits are significant, particularly in the wetlands.

The Spectacle Pond study area is located in the northwest corner of Littleton within the Bennetts Brook watershed, a sub-basin of the Stony Brook watershed that feeds into the Merrimack River (Figure 3-3). The pond is one of the more prominent hydrologic features within the study area, covering more than 70 acres. Spectacle Pond consists of two interconnected lobes that are oriented southwest to northeast (Figure 3-3). The southwest lobe has a maximum depth of 33 feet, and the northeast lobe, adjacent to PW-5, has a maximum depth of 14 feet. Local topographic relief is characteristic of widespread glacial ice-contact deposits found throughout New England.

Production Well Number 5 is located on the southeast shore of the pond. This 28- X 48-inch, gravel-packed well was constructed in 1983 to a depth of 50.5 feet within the valley-fill deposits. The bottom 15 feet of the well consists of a 24-inch screen (Figure 3-4), which draws water from the unconfined aquifer. At the time of installation, the calculated safe yield of the well was 1,100 gallons per minute (gpm), with a specific yield of approximately 1,100 gallons per foot (gpf). At present, the well delivers an average of 650 gpm into the Littleton distribution system. On the northwestern shore of the pond, the bordering town of Ayer (Figure 3-2) has developed two production wells, which draw approximately 1,050 gpm from the same aquifer.

PRELIMINARY WELLHEAD PROTECTION AREA

Prior to 1981, the Massachusetts Department of Environmental Protection (DEP) established siting criteria requiring municipalities to procure and protect a 400-foot radius around public wells. This regulation was enacted to reduce the potential deleterious effects attributed to pathogenic microorganisms resulting from sewage discharge in the immediate area of a municipal production well. At a time when further water quality protection guidance was unavailable, Littleton's ground-water consultants developed a process for selecting favorable development areas for ground-water supplies and delineated WHPA's for land-use regulation enactment.

During the initial hydrogeologic data search, the consultants identified gross deficiencies in the data. Therefore, geophysical techniques, including gravity profiling and electrical resistivity, were utilized to estimate depths to bedrock and the saturated thickness of outwash deposits within proposed regulatory districts. Depth and thickness estimates were verified and calibrated through test borings, providing more detail of sediment types and stratigraphy.

Protection districts were delineated primarily based on the radius of influence of the production well determined by aquifer testing. This area was then extended to include the zones contributing to ground-water and surface-water recharge. The boundaries of protection districts were justified primarily on the basis of local hydrogeologic conditions, but were extended to include geopolitical boundaries as well. Other considerations in the delineation process included potential areas of development; existing potential sources such as road salting activities, an active landfill, service stations, and industrial development; and zoning issues relating to property boundaries.

The final recommendations for the Aquifer and Water Resources Overlay Districts were adopted as a zoning bylaw in 1981 (Town of Littleton Planning Board, 1988). The Aquifer Overlay Districts include the zone of influence (ZOI) and the upgradient, ground-water recharge area for each

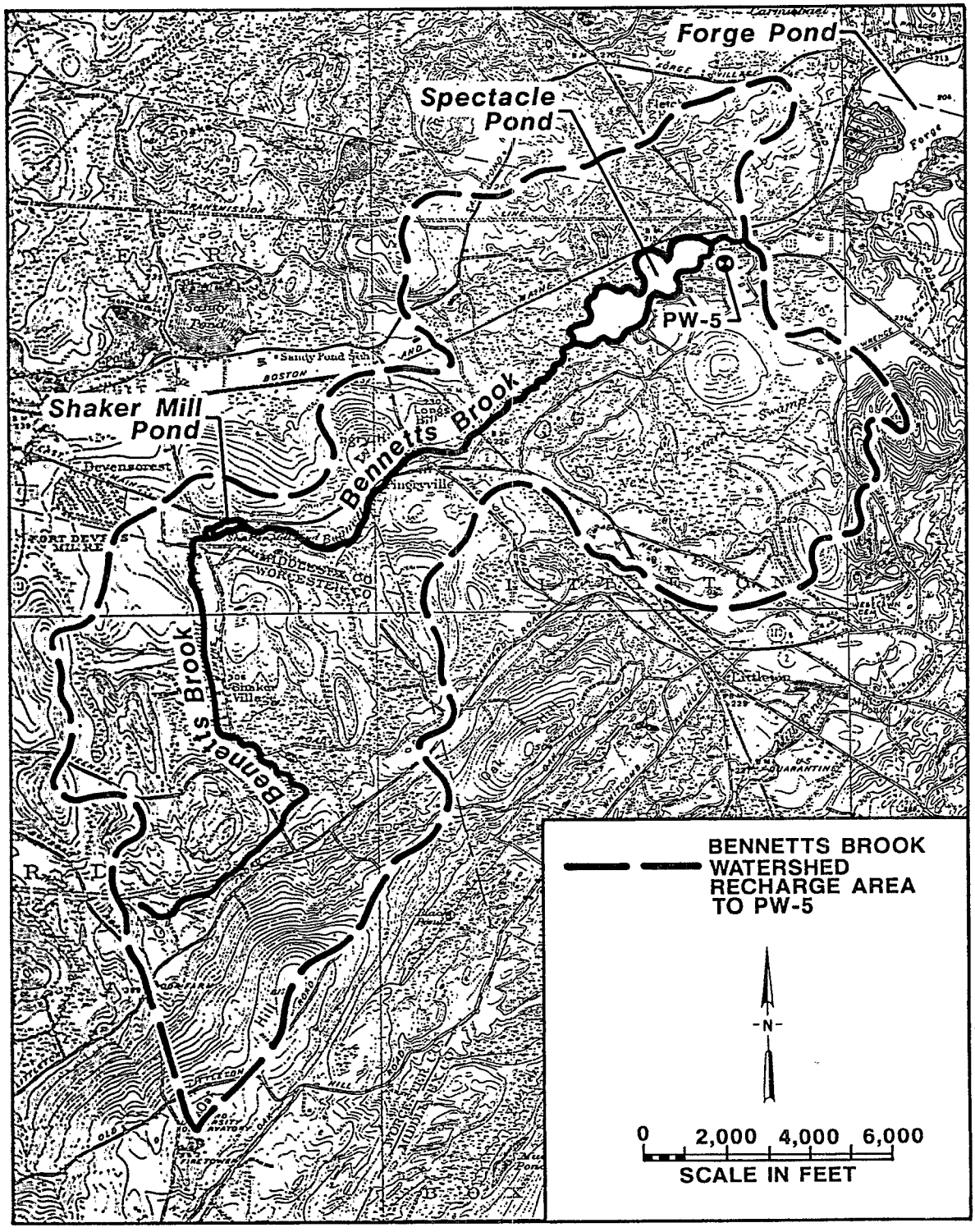

Figure 3-3. Location of Spectacle Pond and PW-5 with regional topography, Littleton, Massachusetts.

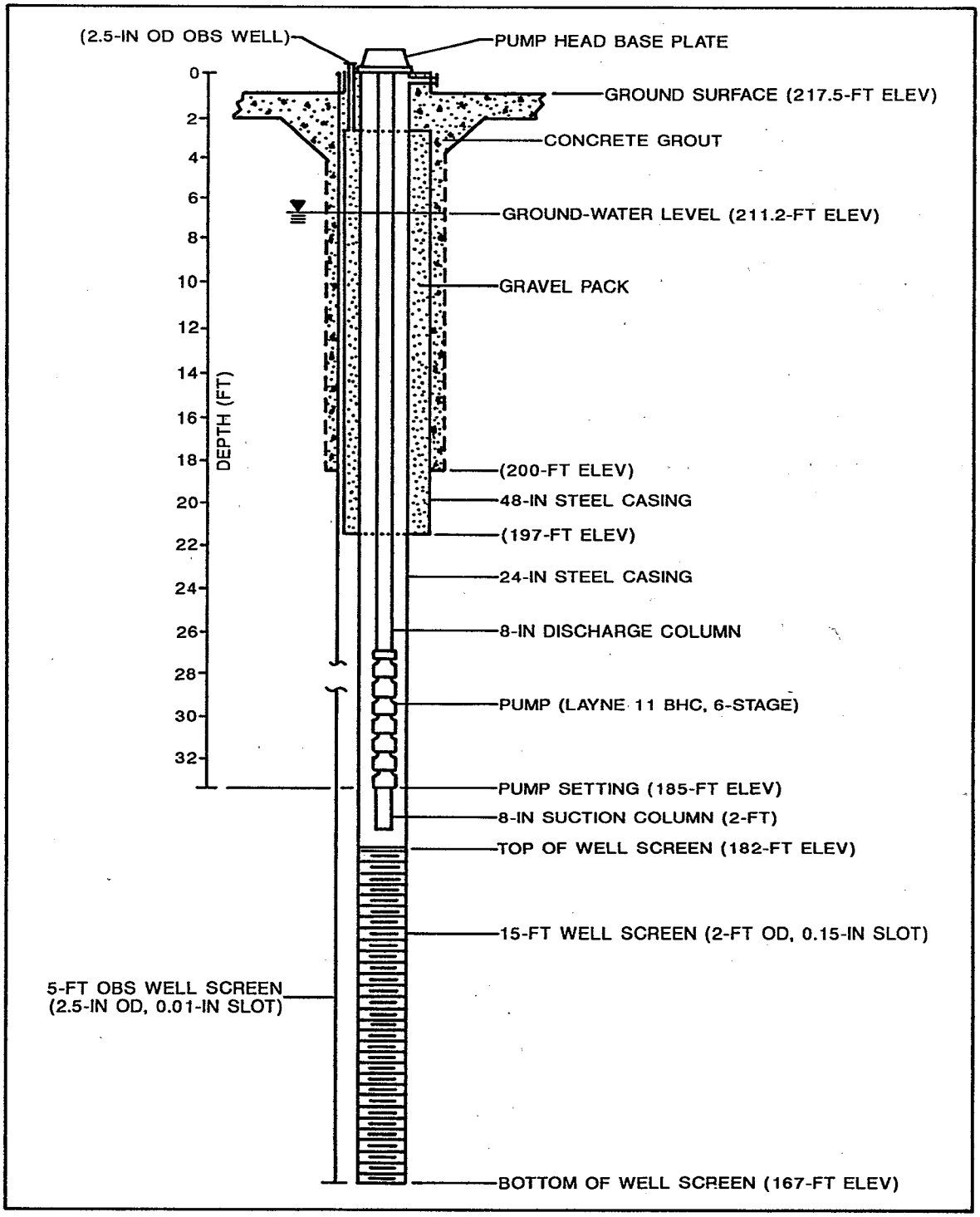

Figure 3-4. Construction diagram for PW-5, Littleton, Massachusetts. (Modified after Haley and Ward, Inc., 1991)

municipal well. Water Resources Overlay Districts were created as a less critical zone of protection and include the watershed boundaries identified within the town of Littleton (Figure 3-5). The overlay districts originally encompassed developed and undeveloped parcels of land, posing serious regulatory issues for the town. Because the town could not afford to purchase multi-million dollar land parcels and preexisting industries, performance criteria were established within the new zoning bylaw. This enabled land development to progress within these districts, while placing the responsibility of water resource protection on individual land owners. The bylaw was written to make individual owners and developers (existing and future) responsible for adverse environmental impacts within their property.

Recently, the DEP adopted new WHPA delineation and management requirements, outlined within the "Guidelines and Policies for Public Water Systems" (Massachusetts Department of Environmental Protection, 1991). To facilitate land-use management activities, the protection area is subdivided and prioritized using the following WHPA zones:

- Zone I: The protective radius required around a public water supply well or wellfield.
- Zone II: The area of an aquifer that contributes water to a well under the most severe pumping and recharge conditions that can be realistically anticipated (180 days pumping with no recharge from precipitation), extended upgradient to prevailing hydrogeologic boundaries.
- Zone III: The land area beyond the Zone II area from which surface water and ground water drain.

For practical purposes, the current state WHPA guidelines closely resemble the Littleton Overlay Districts established for the Spectacle Pond aquifer. Zone I, the 400-foot protective radius mandated by the state, is equivalent to the 14 acres surrounding PW-5 owned and controlled by the Littleton Water Department. Zones II and III, which differentiate the primary recharge area to PW-5 from farther upgradient recharge areas, are equivalent to the Aquifer and Water Resources Districts established by the town. The Aquifer District actually covers a larger area than Zone II, extending into the sands and gravels located upgradient of PW-5, but terminating at the less permeable till and bedrock contacts rather than at other recharge boundaries such as streams or ponds.

Proposed commercial and industrial developments within the overlay districts are subject to risk evaluation through the special-permit granting authority of the Littleton Planning Board. High-risk facilities, such as gasoline stations or landfills, cannot be developed within the more sensitive aquifer district. Moderate- to low-risk facilities are systematically evaluated for hazardous materials storage, handling, and disposal practices. Other considerations in this screening process include the percent of impermeable lot coverage (parking lots, buildings, etc.), nutrient loading, proximity to production wells, and other risks identified during the review process.

In addition to the Aquifer and Water Resources Overlay District Bylaw, Littleton adopted a Toxic and Hazardous Materials Bylaw (Town of Littleton Planning Board, 1981). This bylaw outlines requirements for mandatory registration of toxic and hazardous substances used or stored within all local businesses. Along with current inventories, proper containment for these substances is mandatory. Regulations regarding underground storage tanks (USTs) are also addressed within this bylaw. Because these provisions are unrelated to zoning regulations, facilities cannot be "grandfathered" or waive the onus of compliance.

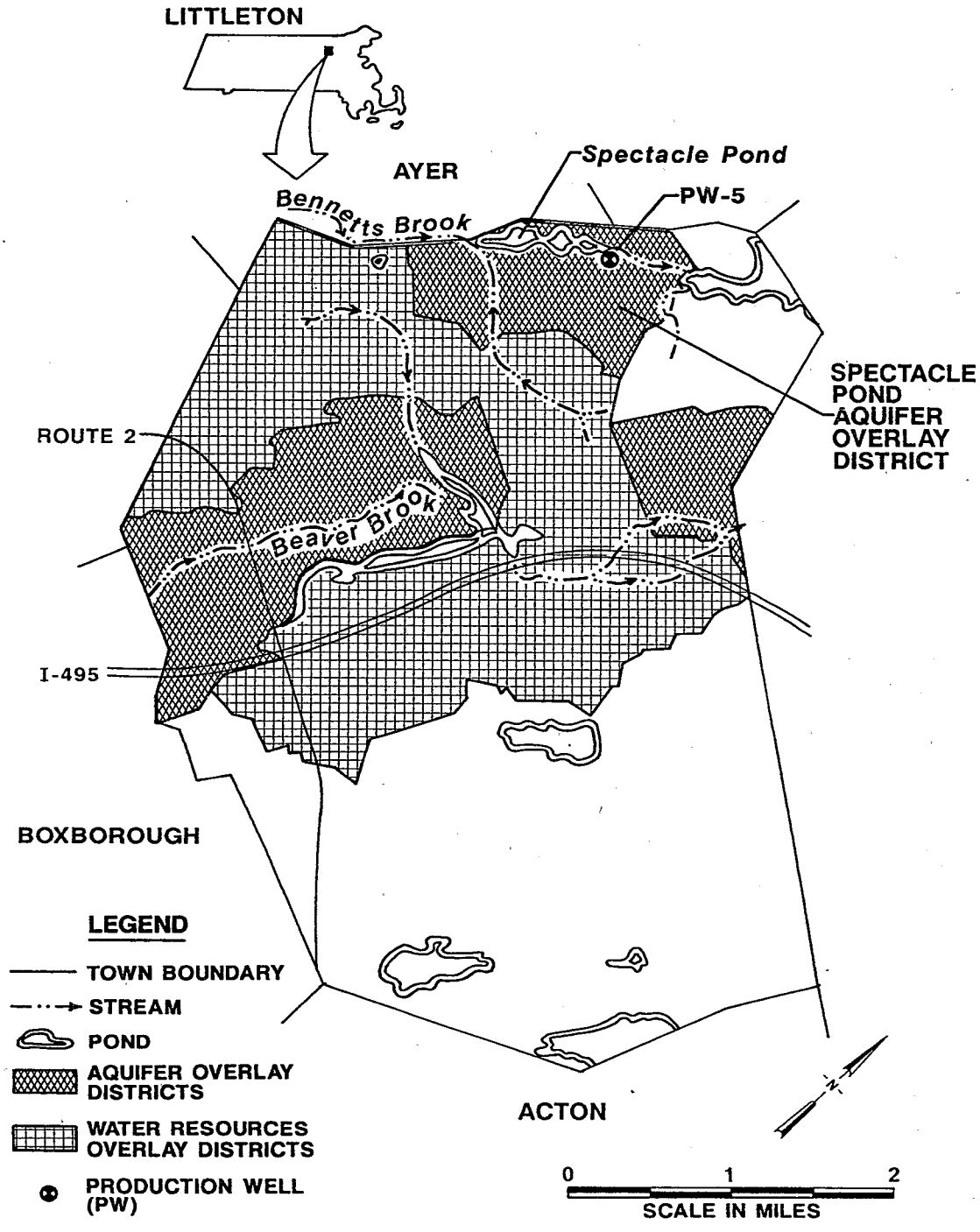

Figure 3-5. Aquifer and Water Resource Overlay Districts within the Bennetts Brook and Beaver Brook watersheds, Littleton, Massachusetts. (Modified after Moore et al., 1990, p. 188.)

Within Littleton, the greatest risks for ground-water contamination are posed by smaller, less accountable operations that were established years before water resource protection issues were realized. Larger businesses that handle significant volumes of toxic or hazardous substances on a day-to-day basis have independently learned of the environmental risk posed by their activities. To reduce their liability, many of these companies have developed substantial risk reduction programs. In many instances, the smaller operations posing the greatest risks tend to be overlooked within the regulatory framework. For this reason, Littleton has implemented very low reporting quantities of substances under the Toxic and Hazardous Materials Bylaw. The monitoring and documentation process has become a valuable educational exercise for these smaller industries.

The bylaw is administered locally through the Board of Health, the Building Inspector, and the Fire Chief. Daily compliance is monitored through scheduled audits conducted by the Water Department. These audits provide a method by which to identify potential ground-water contamination sources and to update existing records. Audits also provide an opportunity to educate commercial and industrial customers about cooperative wellhead protection practices. For some industries, potential liabilities have been minimized by source reduction practices.

SOURCE ASSESSMENT

Comprehensive source assessment is a necessary component of any effective ground-water monitoring program. Shallow sand and gravel aquifers are particularly susceptible to contamination from wastes generated by overlying impoundments (Delaney and Maevsky, 1980). Hence, proper identification, evaluation, and education of operations that pose threats of contamination within protection districts helps to reduce deleterious impacts from inattentive businesses. In Littleton, risks are minimized by regulating the storage, handling, and disposal activities associated with hazardous materials. Source reduction is the preferred method of minimizing risks, but other management strategies are employed if this option is not feasible.

Since 1981, the special permitting process (addressed within the site plan review) has enabled Littleton's Planning Board to screen and identify moderate- and high-risk operations proposed within the Aquifer and Water Resources Districts. Preexisting facilities are subject to screening if they plan an increase of onsite storage of hazardous materials or an increase in lot coverage (above 20% total property area), or when there is a change in proprietorship. Collateral risk-assessment activities are conducted by the Littleton Board of Health by means of the Toxic and Hazardous Materials Bylaw.

Source assessment activities are supplemented with site inspections by the Water Department. Initially, audits were performed at larger industries known to use or suspected of using significant quantities of toxic or hazardous materials. The priority of site audits was determined by the proximity of the facility to a municipal production well and the general outward appearance of the facility. A comprehensive list of potential point and nonpoint sources within the Bennetts Brook watershed is presented in Table 3-1.

TABLE 3-1. POTENTIAL SOURCES OF CONTAMINATION WITHIN THE BENNETTS BROOK WATERSHED, LITTLETON, MASSACHUSETTS

Agricultural	Commercial	Industrial
Animal burial areas	Auto repair shops	Fuel Storage Tanks
Animal feedlots	Car washes	Machine and metalworking shops
Chemical application	Fuel storage tanks	Toxic and hazardous spills
Chemical storage	Gasoline stations	Warehousing and distribution activities
Irrigation	Metal plating operations	Wells (operating and abandoned)
Manure spreading and pits	Paint shops	
	Railroad tracks	
	Road de-icing activities	
	Scrap and junkyards	
	Pool supply outlets	

Residential	Waste Management	Naturally Occuring
Fuel storage tanks	Unregulated agricultural and farmland disposal (farm dumps)	Iron
Household chemical and hazardous waste products	Municipal landfills	Manganese
Fertilizer and pesticide application	Recycling reduction facilities	
Sanitary septic systems	Salt storage facilities	
Sewer lines	Stormwater drains and retention basins	
Swimming pools (chlorine)	Transfer stations	

Generally, land-use activities identified during source assessments covered a broad range of commercial and industrial operations. Those identified throughout the Littleton study area are depicted in Figure 3-6. Fortunately, while no major hazardous releases have been reported to date, gasoline stations, metal plating shops, landfills, industrial developments, and residential subdivisions already exist within the watershed. Collectively, these land-use activities pose potential risks of nutrient, solvent, and heavy metal contamination to the aquifer. In addition to these anthropogenic sources, naturally occurring contamination sources are listed. Manganese and iron are detected at elevated levels in the recharge area of PW-5 and occur in wetland sediment and glacial deposits. High levels of manganese and iron in PW-5 may warrant water treatment in the future.

To better prioritize sites according to the actual ground-water contamination risks posed, site-specific information was categorized and reviewed. Information collected during site inspections was combined with information collected through bylaw-mandated reporting requirements and incorporated into a computerized source data base. The data base contains information regarding the category and quantities of hazardous materials identified for each operation and information about containment practices. The data base also includes information compiled from DEP and EPA offices,

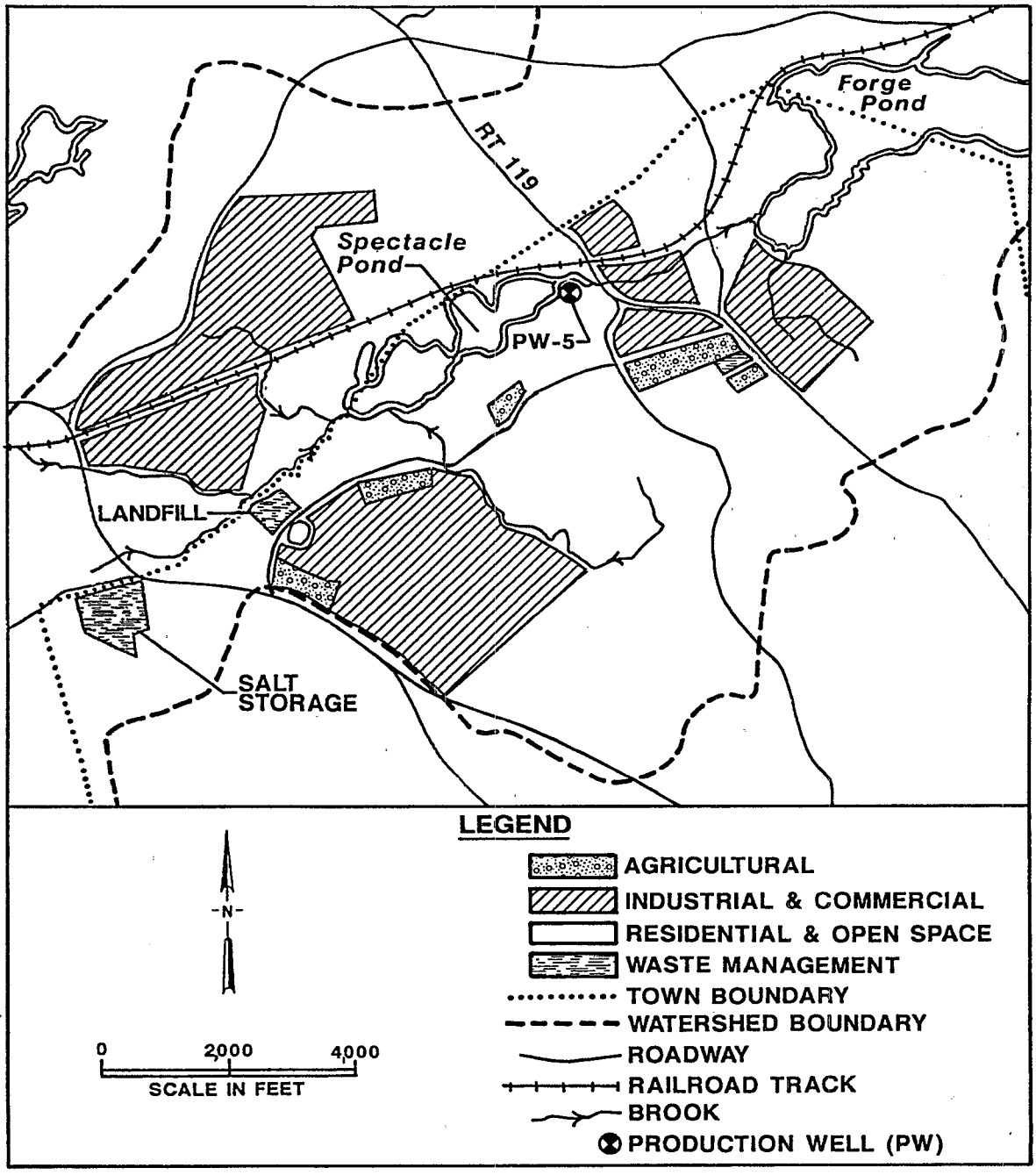

Figure 3-6. Location of existing land-use activities within the Bennetts Brook watershed, Littleton, Massachusetts.

such as hazardous waste generator identification numbers, state audit sites, disposal sites, and incidence response records. This information can be used to cross-reference information with state and federal regulatory agencies.

Ground-water contamination is often the legacy of historic land-use activities. In Littleton, economic growth between the early 1970s and the late 1980s transformed the community from a rural, agricultural base to a residential and commercial base. While existing clean developments could conceal potential contamination, it was important to identify past land-use practices, at least in a general sense. Former agricultural activities were identified throughout the community. Potential sources associated with agricultural land use include improper pesticide storage and application, rural dumps, and nitrate loading from seasonal fertilization and manure stockpiles.

For potential contamination sources identified within the WHPA, a general land-use classification scheme was adopted for prioritizing each site. Because land-use classification is qualitative and, therefore, imprecise as an assessment technique, shortcomings were anticipated. A superior ranking system would utilize more accurate, site-specific, chemical inventories and containment procedures. This combined quantitative and qualitative approach would compensate for inconsistencies observed from one industry to the next.

In addition to the audit program, ground-water monitoring networks have been installed around industries that pose the greatest risk of ground-water contamination. Biannual sampling and analyses for general water quality parameters, heavy metals, and volatile organic compounds (VOCs) are performed by the Littleton Water Department. Results from these sampling activities verify if "at risk" industries are conducting operations in accordance with local bylaws, which ban the release of toxic or hazardous substances in any water resources. These historic water quality records further support our assessment of regional water quality issues within the Spectacle Pond aquifer.

RESEARCH MONITORING PROGRAM

DATA BASE LIMITATIONS

For the Spectacle Pond Aquifer and Water Resources Districts, the Littleton Water Department has compiled a data base that includes point-source water quality information, historical water-level measurements, aquifer parameters, well construction logs, borehole information, geophysical surveys, site assessment records, aerial photographs, and technical reports detailing regional investigations. Although the existing data base is extensive, aquifer testing conducted in 1981 did not provide enough detailed information for accurate development of an effective ground-water monitoring program.

In 1981, a water resource consultant conducted aquifer tests and concluded that the aquifer within the Spectacle Pond study area is hydrologically isolated from the surface-water system. This interpretation is disputable based on deficiencies of the aquifer-test methodology and subsequent analyses, as follows:

- An insufficient number of observation points were used to quantify induced infiltration from Spectacle Pond with accuracy.
- Pumped ground water was discharged 50 feet from PW-5 to a swale; therefore, recirculation of discharge water may have caused incorrect calculations of the area of influence.
- The aquifer test was conducted for 5 days, and steady-state conditions may not have been attained.
- The capture zone, ground-water flowpaths, and time of travel (TOT) to PW-5 were not determined.

In 1981, the town planners extended the protection zone to the upgradient hydrogeologic recharge boundaries determined as the glacial till and bedrock uplands, thereby somewhat compensating for these deficiencies. This delineation approach, conservative by design, resulted in overprotection. Because of escalating land values, this approach is generally unpopular in many communities. Opposition by land owners and developers and the labor and expense required to monitor these districts are only a few of the more obvious shortcomings of overprotection. Considering management issues alone, an optimum protection zone should encompass only those areas in which a contamination release will realistically pose a threat to the water supply in question.

Other deficiencies of data, which impaired the ability to design an optimum monitoring program for the study area, included:

- Lack of detailed horizontal and vertical hydrogeologic characterization
- Insufficient surface-and ground-water flow information within a one-half-mile radius of PW-5 ("near-field" scale) and to the boundaries of the aquifer (5-mile radius, "far-field" scale)
- Lack of near-field water quality data
- Inadequate aquifer testing

RESEARCH OBJECTIVES

To address deficiencies in the data base, a comprehensive investigation of the hydrogeologic properties within the Bennetts Brook watershed was conducted. The investigation included several tasks to determine aquifer properties, surface- and ground-water flow system interactions, and ambient and contaminant water quality. These tasks, coupled with source inventory and prioritization, led to the development of an optimum monitoring program for the Spectacle Pond aquifer. The monitoring program was designed to provide a relatively simple and cost-effective means of ensuring protection of PW-5. It called for monitoring sites to be installed at strategic locations based on flowpath identification; sampling frequencies to be instituted using TOT criteria; and analytical parameters to be defined by source assessment activities, including land-use identification, site inspections, and near-field water quality analyses.

RESEARCH TASKS

The most critical task performed in any hydrogeologic study is the development of a conceptual model that realistically describes the system of interest. To better understand the hydrogeologic conceptual model for the Spectacle Pond aquifer, a series of tasks was designed to qualitatively determine aquifer properties and flow system dynamics. Confirmation and modifications to the conceptual model were accomplished by a review of hydrogeologic data, installation of ground-water monitoring devices, aquifer testing, ground-water flow modeling, and comprehensive water quality analyses.

Over the years, a significant amount of hydrogeologic information was generated within the Bennetts Brook watershed through independent studies. The historic data review process helped identify and integrate hydrogeologic characterizations, aquifer testing results, and site assessment findings detailed in these previous studies. The review provided a means to collect background information and to highlight data deficiencies within the study area. Federal and state hydrogeologic investigations, as well as private site assessments, were sought during this review process.

Ground-water monitoring devices were installed to acquire hydraulic head data, stratigraphic information, and water quality characterization. Because of the range in costs and utility for various types of monitoring systems, the selection of each device for this study depended on the location of the drilling site, local hydrogeology conditions, types of pollutant(s) expected, and overall installation expense (Scalf et al., 1981). In most studies, cost is a limiting factor and often dictates the selected

number and design of monitoring devices used. Where appropriate, preexisting and newly installed ground-water monitoring wells, observation wells, "MicroWells" (Cadwgan and Swallow, 1991), and small-diameter piezometers were used for this study.

To simulate ground-water flow conditions for the near-field and far-field systems, a phased approach of aquifer testing and modeling was outlined. The approach was iterative so that computer simulations could be calibrated to generate an accurate numerical flow model of the Spectacle Pond aquifer. An initial, 3-day aquifer test, referred to as the "short-term test," was performed in May 1990. Another aquifer test, lasting 12 days and referred to as the long-term test," was performed in August 1990.

Monitoring stations were strategically located to acquire spatial aquifer parameter. Drawdown and recovery data were interpreted using four different analytical methods for distinct aquifer conditions. In addition to providing best estimates for aquifer parameters, separate analyses allowed for evaluation of ground-water flow systems in different parts of the aquifer.

Steady-state, numerical, ground-water flow models were used to simulate far-field and near-field aquifer responses under pumping and nonpumping conditions. To simulate normal hydrogeologic flow conditions, necessary for siting appropriate monitoring stations within the capture zone of PW-5, average long-term recharge and discharge input parameters were compiled. A two-dimensional (2-D), horizontal plane model, FLOWPATH (Franz and Guiguer, 1989), was used (1) to simulate far-field ground-water flow, surface- and ground-water interactions, infiltration events, and geologic controls; and (2) to determine time-related capture zones. The three-dimensional (3-D) flow model, MODFLOW (McDonald and Harbaugh, 1988), simulated vertical flow components in proximity to Spectacle Pond and PW-5. This near-field model generated capture zones, flowpaths, and travel times associated with PW-5. Near-field and far-field steady-state model results were evaluated in designing the long-term monitoring network and in determining protection zones within the WHPA.

In contrast to steady-state modeling, transient-flow modeling permits simulation of time-related responses in a hydrogeologic system with various pumping and recharge stresses. Current WHPA delineation criteria for the State of Massachusetts (Massachusetts Department of Environmental Protection, 1991) require the development of a such a model. Stress conditions of continual pumping for 180 days at a safe yield with no recharge from precipitation were imposed on the aquifer's natural, nonpumping, hydraulic head conditions. By design, this drought simulation will expand the downgradient boundary of the resulting WHPA. Using these criteria, the Spectacle Pond aquifer system was simulated with a transient-state, 2-D, numerical flow model, FLOWCAD (Franz and Guiguer, 1991). The final hydraulic head distribution was then used to delineate the capture zone for these extreme hydrologic conditions.

In addition to the aquifer testing and modeling tasks, sampling and hydrochemical analyses were performed for the following reasons:

- To characterize near-field water quality
- To corroborate or refute ground-water modeling interpretations
- To establish ambient water quality data within the Bennetts Brook watershed

DATA ACQUISITION

Hydrogeologic Data Review

Reference site-specific and regional characteristics were examined during development of the initial hydrogeologic conceptual model for the Bennetts Brook watershed. Standard references, such as U.S. Geological Survey (USGS) surficial geology and topographic quadrangle maps and the USGS Hydrological Atlas, were reviewed. Site-specific reports detailing original aquifer test analyses, soil boring logs, geophysical surveys, and historical ground-water quality data were also reviewed.

Following the literature review, a site visit was conducted by all principal investigators. This exercise was designed to orient participants involved in the study, through a realistic overview of the local geomorphology and a review of regional assumptions and interpretations. Reconnaissance site visits, such as the one conducted by the principal investigators, highlight site conditions that may not be evident from the literature review and may be significant for conceptualization by both computer modelers and field technicians.

Ground-Water Monitoring Stations

Ground-water monitoring wells, observation wells, MicroWells, and piezometers are found throughout the Bennetts Brook watershed. Each device was installed for a specific purpose, but these devices were used collectively for regional water-table mapping. Every device was field-tested to confirm whether it maintained a suitable hydraulic connection with the underlaying aquifer. The integrity of each well was confirmed with a modified falling-head recovery test. If upon adding a known quantity of water, the original water-level within the well recovered quickly, the hydraulic connection was considered acceptable. If the response was delayed, the well was redeveloped and retested. If it failed this second test, the well was not used for future monitoring activities.

Monitoring Wells--

Monitoring wells were installed where chemical and physical properties of an underlying aquifer needed to be identified. The drilling method and monitoring well design employed is dependent on the hydrogeologic environment encountered. Within the Bennetts Brook watershed, hollow-stem augers or drive-and-wash drilling methods were used successfully. After the soil borings were advanced, monitoring well casings and well screens were inserted into the boring. These materials were typically constructed of polyvinyl chloride (PVC) or stainless steel. Once the monitoring well was emplaced, the original driving casing was pulled, which allowed for representative ground-water samples to be collected from the aquifer.

Monitoring wells were advanced using a 8.25- x 4.25-inch hollow-stem auger drilling rig. The final well construction consisted of 2-inch-diameter, schedule 40, flush-joint threaded PVC pipe, coupled to 5- or 10-foot lengths of 0.010-inch slotted well screen (Figure 3-7A). Screen lengths were determined by the depth and extent of aquifer isolation required. For example, most cluster wells, installed at multiple depths within the same general area were equipped with 5-foot screen lengths. Singular wells were equipped with 10-foot screens to provide a greater depth range for monitoring.

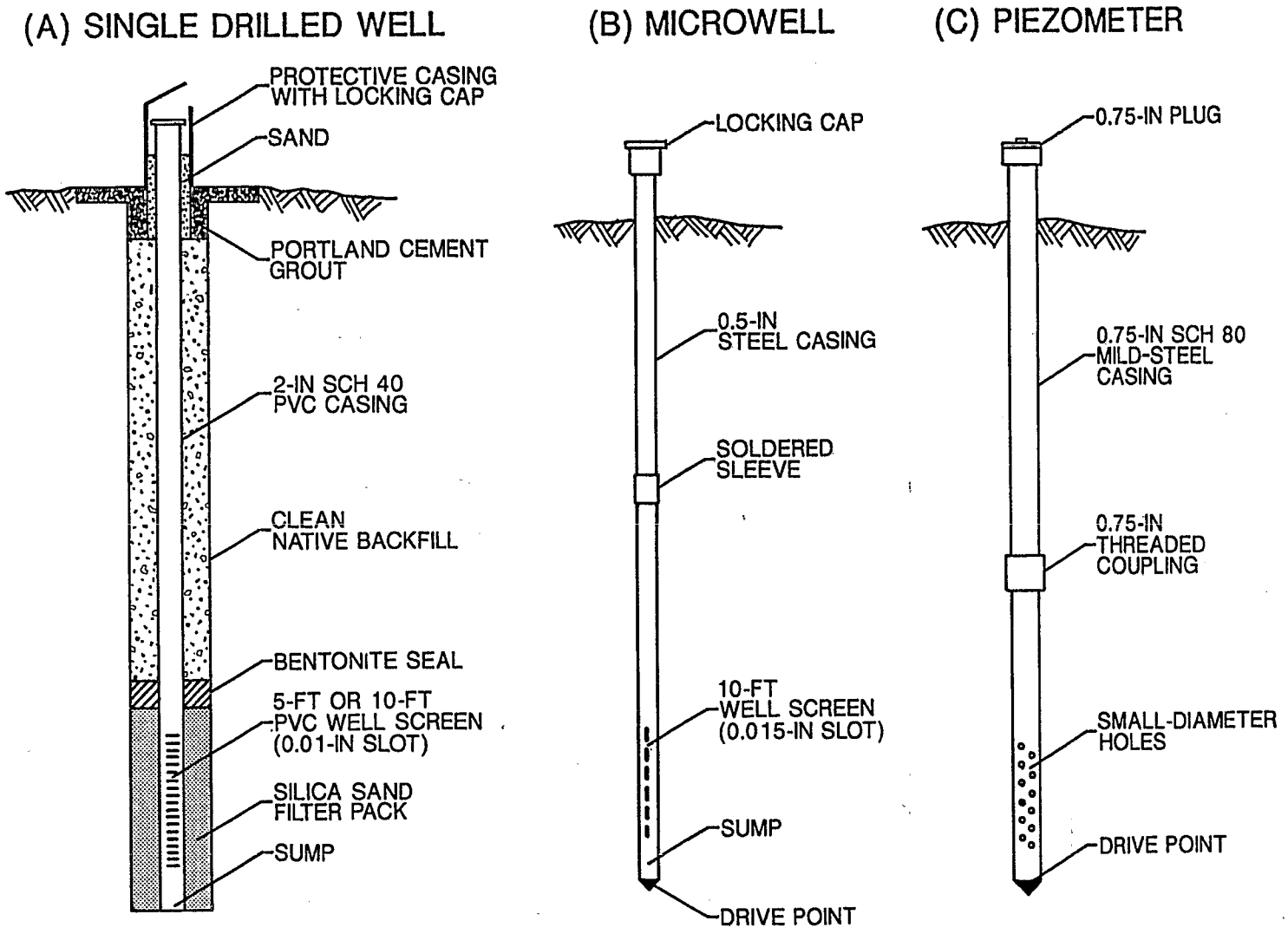

Figure 3-7. Ground-water monitoring device construction diagrams for (A) single drilled well; (B) MicroWell; and (C) piezometer.

All screens were filter-packed with clean Ottowa sand and capped with a 6-inch bentonite clay seal to minimize effects from vertical leakage. At the surface, each well was protected with a locking case and cap and sealed with 2-foot-thick cement grout. All wells were developed for several hours until a clear discharge water was observed.

Split-spoon sampling was performed at 5-foot increments to identify local stratigraphy (U.S. EPA Region 1, 1989). In areas where the larger, truck-mounted auger rig could not be taken, a portable drive-and-wash, cable tool drilling rig was employed. With the exception of the smaller-diameter PVC casing (1.5-inch), the final design of these wells is similar to the monitoring wells described above.

Observation Wells--

Observation wells were installed to determine local stratigraphy and ground-water flow conditions, without regard to ground-water quality. These wells were installed using the same drive-and-wash techniques used to drill the monitoring wells. Observation wells differ from monitoring wells in that the galvanized-steel casing used to advance each boring is left in place. Generally, the well screen, placed at the bottom of the casing, is also constructed with galvanized steel. Because this steel is not replaced with more chemically inert materials such as PVC or stainless steel, it is not possible to collect truly representative ground-water samples from these wells. However, the simplified drilling process decreases the amount of time and overall expense required for each installation. Where ground-water quality is not required, observation wells provide a cost-effective means of obtaining hydrogeologic properties.

MicroWells--

The MicroWell installation process employs a high-frequency vibratory hammer mounted on a VibraDrill all-terrain drilling machine, which can advance 0.5-inch, steam-cleaned, steel casing to depths of over 100 feet (vibracore process) (Cadwgan and Swallow, 1991). The bottom 10-foot section of the steel casing is machined with a double row of longitudinal slots 0.015-inch wide (Figure 3-7B). After the casing is driven to the desired depth, the well is naturally sealed by the surrounding aquifer and developed using a peristaltic pump attached to 0.5-inch-diameter polyethylene tubing. The well is purged until clear discharge water is observed. Although MicroWells can be used to obtain hydraulic head data and water quality samples, they are used in this study for water-level monitoring, exclusively. While the MicroWell drilling technique will reduce the time and costs associated with more traditional drilling methods, the vibracore process is effective only in sand and gravel environments. At the time of this study, the vibrocore process did not incorporate stratigraphic sampling; however, now it does.

Piezometers--

Piezometers are installed for hydraulic head measurements exclusively. Each is composed of 0.75-inch, schedule 80, mild-steel casing machined to 7- and 14-foot lengths (Figure 3-7C). Manually driven piezometers seldom reach depths greater than 30 feet but provide a useful method for collecting water-level measurements in shallow sand and gravel aquifers without significant equipment or cost. These devices can be advanced only in soft materials that are relatively free of cobbles and boulders (Roy et al., 1984). For this study, each point is hand-driven through the upper soil horizons and set into the underlying sands and gravels. The drive points are perforated with multiple small-diameter holes to provide a hydraulic connection with the aquifer. Fine-grained sediment and organic materials entrapped during the installation process are removed by air-jetting techniques.

Aquifer Testing

Hydrogeologic systems can be defined using fundamental aquifer properties such as saturated thickness (b), porosity (n), hydraulic conductivity (K), transmissivity (T), storativity (S), and specific yield (Sy). Saturated thickness is measured as the aquifer thickness saturated with ground water. The volume of water stored within the aquifer is related to porosity, defined as the ratio of aquifer void space to total aquifer volume. Ground-water flow through this void space is controlled by grain size distribution and sorting characteristics of the matrix. Hydraulic conductivity is a coefficient of proportionality, which describes the flow velocity and direction of ground-water moving through an aquifer. Often, hydraulic conductivity is separated into its horizontal and vertical flow components, Kr and Kz, respectively. The amount of water transmitted through a unit width of aquifer under a unit gradient, transmissivity, is calculated by multiplying saturated thickness by hydraulic conductivity. Finally, for water supply studies, the volume of water stored or expelled per unit area of the aquifer per unit change in pressure, storativity, is particularly important. Generally, in an unconfined system, storativity is equivalent to specific yield. Collectively, these properties, which are used to characterize an aquifer system, can be used to predict hydrologic responses to natural (recharge) or induced (pumping) stresses. The qualitative measurement for each property is determined using standard aquifer testing methods.

The short-term aquifer test was conducted at minimum cost by using existing monitoring sites in proximity to PW-5 (Figure 3-8A). To determine ground-water responses in areas where no monitoring stations exist, inexpensive piezometers were installed. Discharge water during testing was diverted from the near-field aquifer area through the existing water supply distribution network. The information obtained during the short-term aquifer test, although somewhat incomplete, proved useful in the siting of additional wells for the long-term aquifer test. The long-term aquifer test was scheduled to obtain accurate values of hydrogeologic properties within the near-field capture zone of PW-5 (Figure 3-8A).

Drawdown and recovery data obtained during the short- and long-term aquifer tests were analyzed using the following graphical methods:

- Neuman (1974) method for horizontal and vertical movement in an unconfined aquifer, modified for a partially penetrating well
- Theis (1935) nonequilibrium formula for a non-leaky, confined aquifer, modified by Kruseman and de Ridder (1990) for unconfined aquifers
- Cooper-Jacob (1946) method for graphical solution of the modified non-leaky confined formula
- Hantush (1964) modified method for a leaky, semi-confined aquifer, considering partial penetration of the well and storage in the aquitards

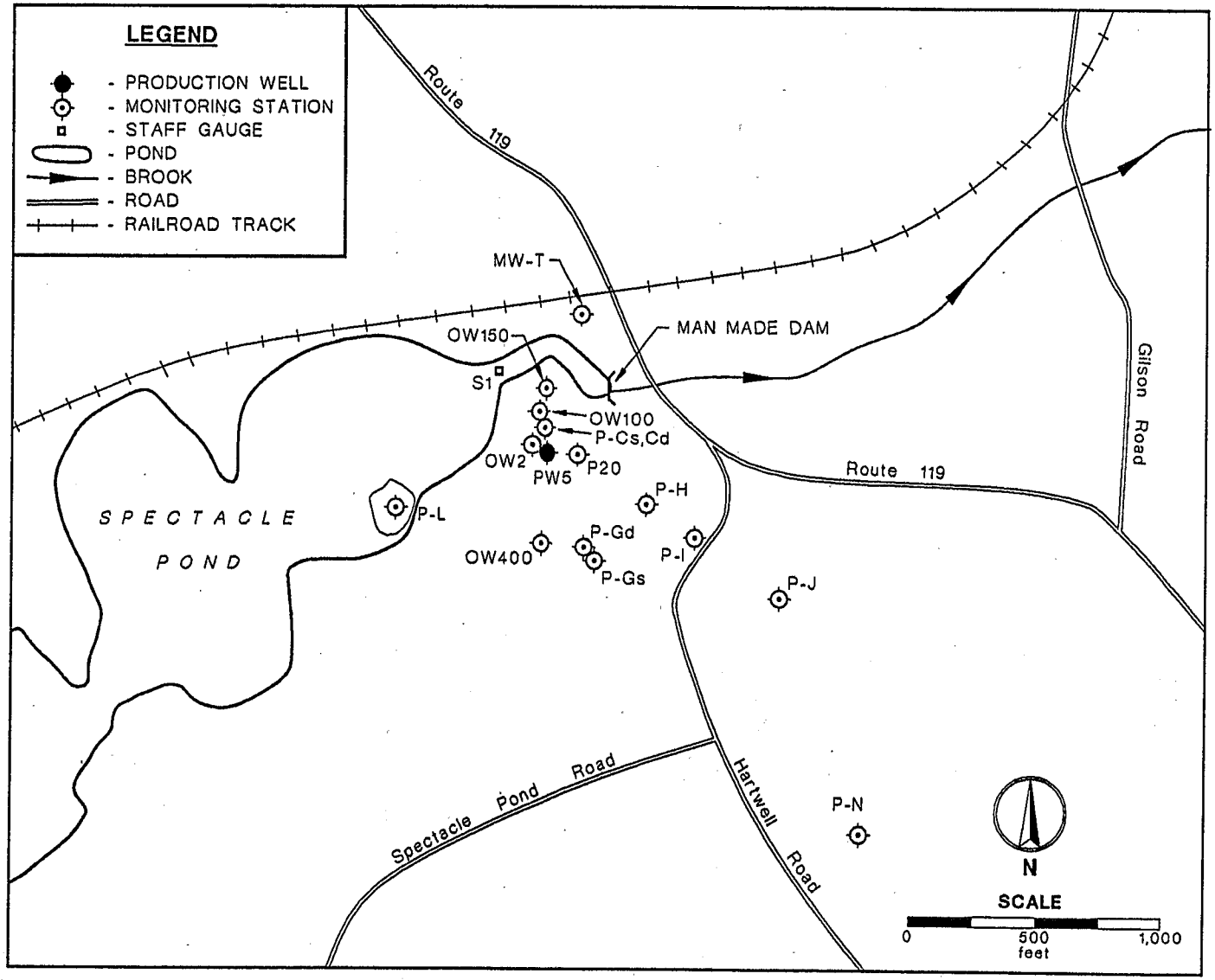

Figure 3-8A. Ground-water elevation stations for near-field studies, Bennetts Brook watershed, Littleton, Massachusetts.

Short-term Aquifer Test--

In addition to PW-5, a network of water-level monitoring stations was employed in the short-term test; these included four observation wells (OW), nine piezometers (P) and one monitoring well (MW). These observation stations were located at distances ranging from 2 to 1,393 feet from the pumping well, PW-5 (Figure 3-8A). Information regarding the distance from each monitoring station to PW-5, station depths, and screen lengths is given in Table 3-2. Prior to the short-term test, PW-5 was shut down for a period of 96 hours to determine the static, prepumping water-level condition. At 12:00 P.M. on May 22, 1990, PW-5 was turned on, and a constant pumping rate of 650 gpm was established for the duration of the test. Water-level responses were monitored in the near-field network. After 73 hours, at 1:00 P.M. on May 25, 1990, PW-5 was shut off and water-level recovery measurements were collected for the next 72 hours.

TABLE 3-2. DISTANCES FROM PW-5 TO MONITORING STATIONS, STATION DEPTHS, AND SCREEN LENGTHS FOR THE SHORT-TERM AQUIFER TEST, BENNETTS BROOK WATERSHED, LITTLETON, MASSACHUSETTS

Monitoring Station	Distance from PW-5 (ft)	Station Depth (ft)	Screen Length* (ft)
PW-5	0	50	15
OW-2	2	40	10
OW-100	100	40	10
OW-150	150	40	10
OW-400	400	40	10
P-A	20	40	10
P-Cd	50	40	10
P-Cs	50	14	3
P-Gd	441	20	5
P-Gs	441	7	3
P-H	510	7	3
P-I	993	7	3
P-J	1,393	7	3
P-L	655	7	3
MW-T	800	35	10

Abbreviations: PW-5 = Production Well Number 5; OW = observation well; P = piezometer; d = deep well; s = shallow depth well; MW = monitoring well.

* All screens are located at the bottom of each monitoring station.

Hydraulic head data at the 15 observation stations were measured with hand-held electronic water-level indicators to an accuracy of ± 0.01 foot. To address effects from regional, ambient water-level fluctuations during the aquifer test, two observation wells (OW-1a and OW-1b) located far outside the zone of influence of PW-5 were monitored (Figure 3-8B). Surface-water elevations were collected at a staff gauge, S1, located near the eastern shore of Spectacle Pond (Figure 3-8A). Other physical parameters recorded during the aquifer test included barometric pressure, rainfall, and temperature.

During analyses of the short-term test results, it was noted that several newly-installed piezometers did not respond reliably, especially those observation stations located farthest from PW-5 (P-N, P-J, and P-I). It was also decided that the number of water-level stations across the pond was insufficient. Specifically, water-level data on the other side of the pond were required to evaluate connectivity between the pond and the aquifer and to determine the recharge contribution from the pond to PW-5.

Long-Term Aquifer Test--

From the values obtained during the short-term test, a sensitivity analysis was performed using FLOWPATH varying transmissivity, leakage through the bottom of the pond, specific yield, and storage. The results of the sensitivity analyses demonstrated that the aquifer would need to be stressed for a period of at least 10 days for a water-level response to be detected in wells located on the northern shore of the pond. Based on these findings, the long-term aquifer test was conducted over a period of 287 hours (approximately 12 days).

The location of observation stations used during the long-term aquifer test are shown in Figure 3-9. Information regarding distances to monitoring stations from PW-5, station depths, and screen lengths are contained in Table 3-3. Four observation wells, OW-100, -150, -400, and -800 (formerly identified as MW-T), and two piezometers, P-20 and -50, from the short-term test were included for use in the long-term test. Observation Well-1200 and seven newly-installed monitoring stations [MW-25 (shallow and deep), MW-100 (shallow and deep), MW-150, MW-375 (shallow and deep), MW-425, MW-660 (shallow and deep), and MW-800] were also integrated into the long-term test network.

Well PW-5 was shut down for 104 hours prior to the start of the long-term test to allow for water-level recovery. Beginning on August 16, 1990, PW-5 was pumped at a constant rate of 650 gpm. After 72 hours, PW-5 was shut down and water-level recoveries were monitored for an additional 48 hours, ending on August 30, 1990.

Water-level elevations for most of the 20 monitoring stations were recorded with electronic hand-held indicators. Water-level data recorded at three cluster wells, MW-25, -100, and -375, and two single wells, MW-150 and -425, were measured using an automatic datalogger system. An automatic datalogger provides a means of recording rapid changes in hydraulic heads and temperatures anticipated at those stations suspected of exhibiting maximal response to pumping. Dramatic temperature changes provide a positive indication of surface-water and ground-water interconnection.

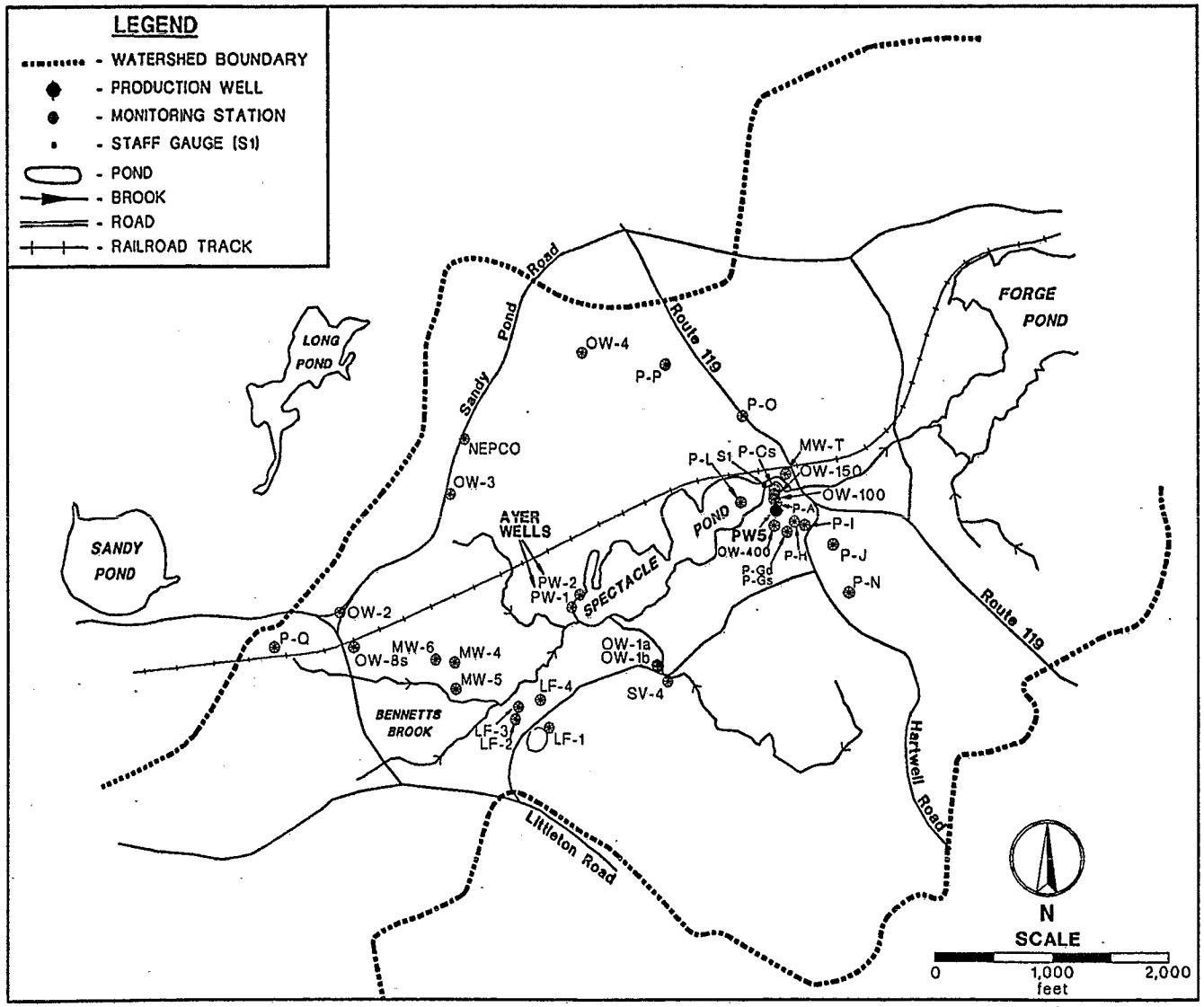

Figure 3-8B. Ground-water elevation stations for far-field studies, Bennetts Brook watershed, Littleton, Massachusetts.

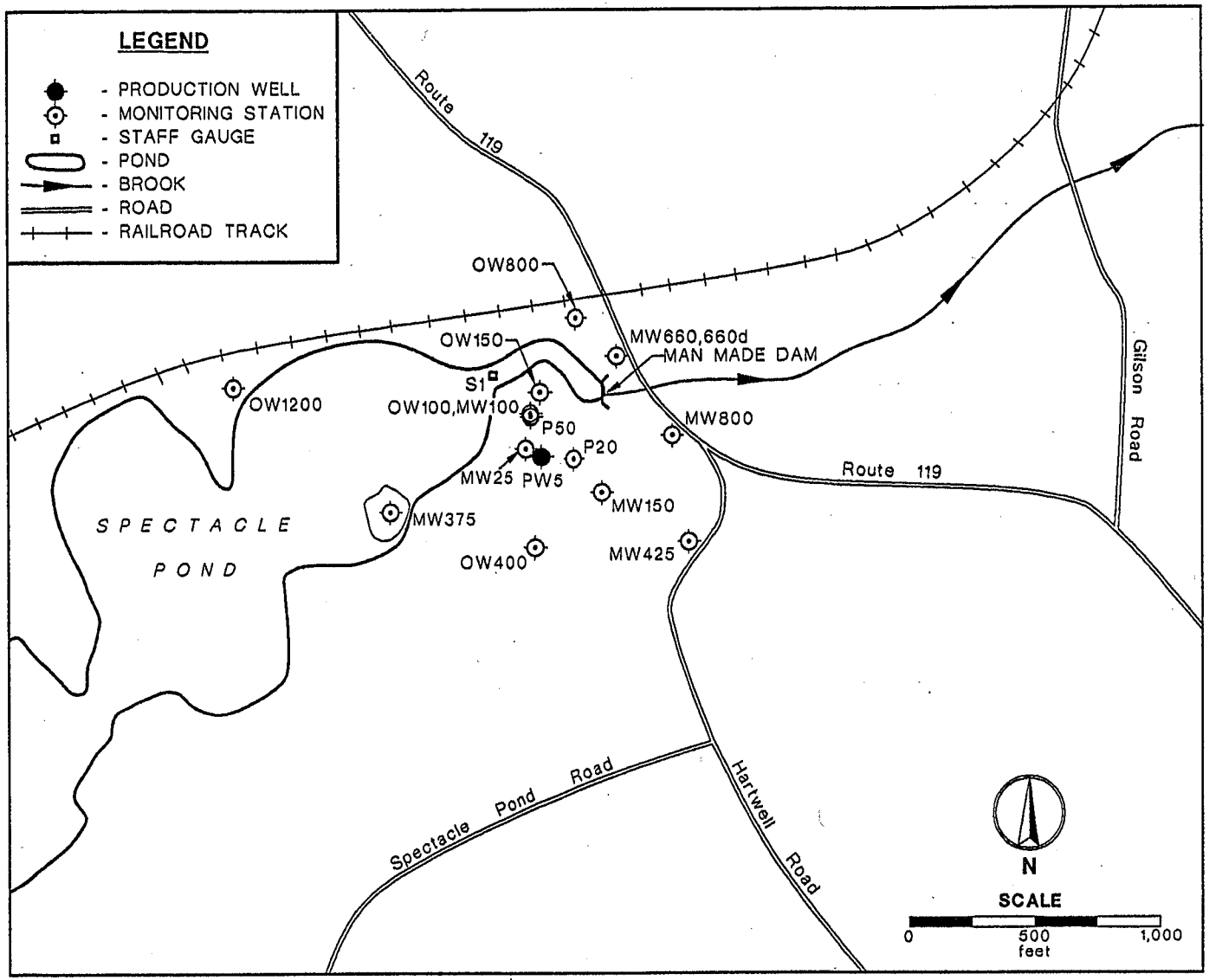

Figure 3-9. Near-field, water-level, observation and water-quality monitoring network, Bennetts Brook watershed, Littleton, Massachusetts.

TABLE 3-3. DISTANCES FROM PW-5 TO MONITORING STATIONS, STATION DEPTHS, AND SCREEN LENGTHS FOR THE LONG-TERM AQUIFER TEST, BENNETTS BROOK WATERSHED, LITTLETON, MASSACHUSETTS

Monitoring Point	Distance from PW-5 (ft)	Station Depth (ft)	Screen Length* (ft)
PW-5	0	48.0	15
P-20	20	40.0	10
MW-25s	25	18.0	5
MW-25d	25	34.0	5
P-50	50	40.0	10
MW-100s	100	18.0	5
MW-100m	100	35.0	5
OW-100d	100	40.0	10
OW-150	150	40.0	10
MW-150	150	70.0	10
MW-375s	375	10.0	5
MW-375m	375	22.0	5
MW-375d	375	40.0	5
OW-400	400	40.0	10
MW-425	425	73.0	10
MW-660s	660	37.0	10
MW-660d	660	63.5	10
MW-800	800	65.0	10
OW-800 (MW-T)	800	35.0	10
OW-1200	1,200	34.0	10

Abbreviations: PW-5 = Production Well Number 5; P = piezometer; MW = monitoring well; s = shallow depth well; d = deep well; m = moderate depth well; OW = observation well.
* All screens are located at the bottom of each monitoring station.

As in the short-term test, ambient ground-water levels were recorded at OW-1a and -1b. Surface-water elevations from S1, located within Spectacle Pond, were also monitored. In addition, regional barometric pressure, rainfall, and temperature, starting 5 days prior to and during the test, were recorded.

The water-levels recorded during the long-term aquifer test were analyzed graphically with type curves and numerically with least-square fitting for different aquifer settings. Hydraulic responses were compared to the Neuman analysis for unconfined aquifers with delayed yield, the Theis and Cooper-Jacob methods corrected for unconfined aquifers, and Hantush's solution for leaky aquifers with storage in the aquitard. Matching solutions provided estimates of hydraulic conductivity and storativity and indicated prevailing aquifer conditions at each observation station.

Peat-Probing Exercise

The areal extent and thickness of peat deposits that underlie the eastern lobe of Spectacle Pond basin may significantly affect induced flow from the pond to PW-5. To determine the configuration of the peat layer, field measurements were collected on February 6, 1990, when the ice cover on the pond supported field technicians and equipment. Peat layer thicknesses were determined by coring through the pond ice using a hand-held ice auger, then driving 10-foot-lengths of small-diameter (0.75-inch), threaded rod through the peat until refusal was encountered. The interface between peat and the sand and gravel was easily confirmed by the grinding noise made when the metal probe struck the sand and gravel layer. Peat depths and thicknesses were determined at 50-foot intervals along several transects at the eastern lobe of the pond (Figure 3-10). Peat layer thickness ranged from less than 1 foot to 35 feet.

Ground-Water Sampling and Analysis

Ground-water monitoring activities conducted within the Spectacle Pond aquifer and the capture zone of PW-5 were designed to characterize water quality for long-term monitoring planning. The scope of these monitoring activities are broadly defined by Todd et al. (1976) and include (1) ambient trend monitoring (temporal and spatial parameter trends within the watershed), and (2) source assessment monitoring (measurement of key indicators of contamination to determine the impact of a potential or known source). Sample collection is also discussed in this section.

The water quality parameters grouped in Table 3-4 were differentiated according to specific monitoring objectives: general criteria, site-specific criteria, and combined general and site-specific criteria. General water quality criteria include those ground-water constituents that are typically identified in undeveloped, pristine watersheds where anthropogenic stresses are minimal. Site-specific criteria include chemical constituents that originate from man-made stresses including point source (isolated) or nonpoint source (regional) contamination. Parameters attributed to both natural and artificial sources are listed in the combined general and site-specific category.

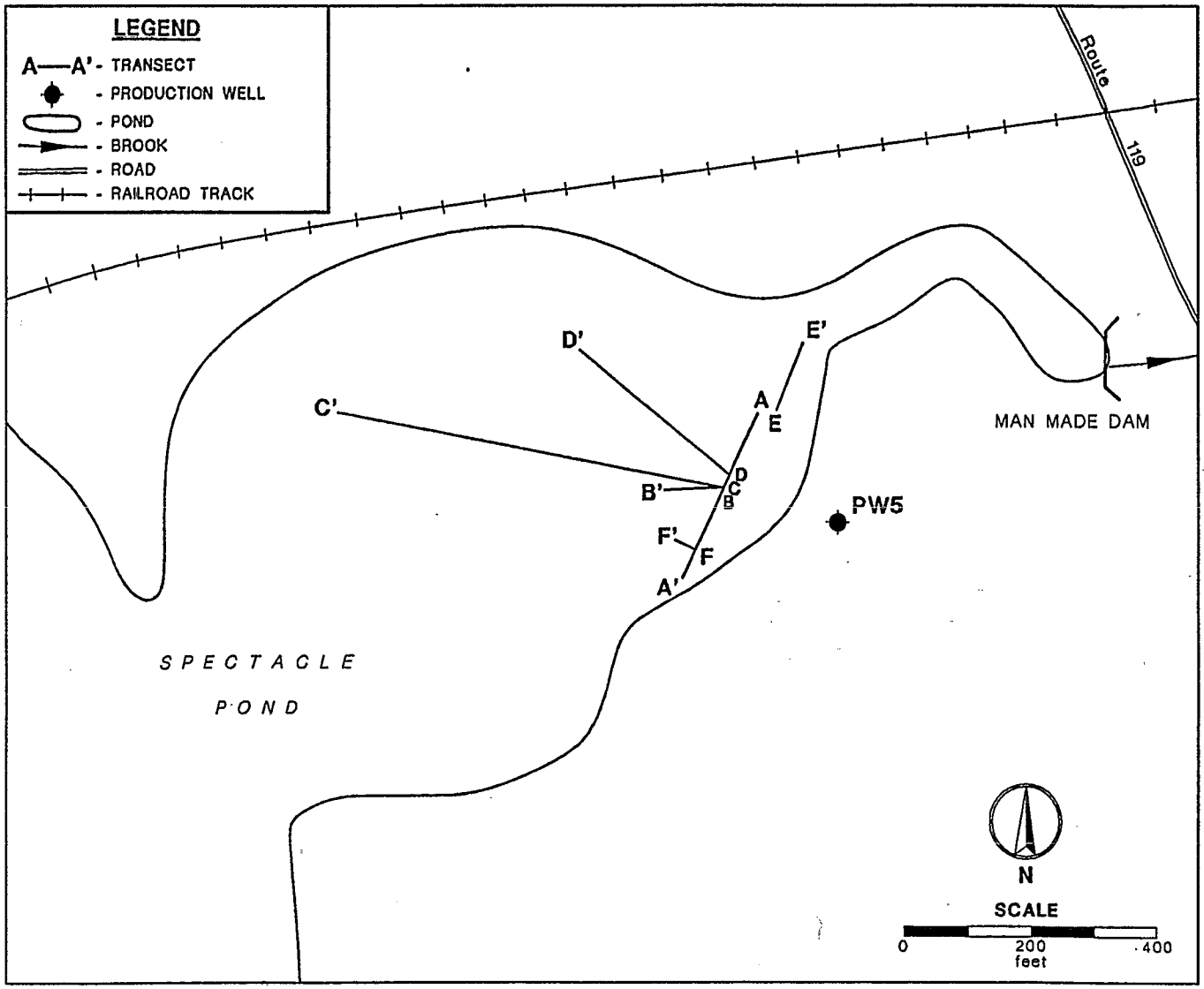

Figure 3-10. Peat-probing transects at the eastern lobe of Spectacle Pond, Bennetts Brook watershed, Littleton, Massachusetts.

TABLE 3-4. WATER QUALITY DATA FOR THE MONITORING NETWORK NEAR PW-5, LITTLETON, MASSACHUSETTS
(Part 1 of 4: Locations PW-5 through MW-375d; Parameters Temperature through Sodium)

Sample Location	Days of Pumping	Water Quality Parameters												
		General Water Quality							General and Site-Specific Water Quality					
			(mg/l)						Cond (µS/cm)	pH	(mg/l)			
		Temp (°F)	Ca	Hard	HCO_3	K	Mg	TDS			Cl	Fe	Mn	Na
PW-5	BP	57	29.4	72.0	52.8	3.5	2.8	127.0	212	6.7	32	0.18	0.62	18.4
	1	57	27.8	65.7	48.4	3.5	2.6	132.0	220	6.6	28	0.12	0.12	16.9
	6	61	24.8	68.4	58.1	3.3	2.4	142.0	237	6.7	25	0.12	0.12	17.8
	11	63	21.7	63.7	50.6	3.0	2.3	104.2	174	6.9	53	0.13	0.13	16.7
MW-25s	BP	60	16.9	45.0	41.4	6.6	4.0	130.0	218	6.2	31	8.80	0.38	24.8
	1	54	15.2	40.5	38.7	4.8	2.7	111.0	185	6.1	31	2.60	0.21	23.5
	6	66	12.7	45.0	17.6	3.5	2.2	113.4	189	6.2	27	2.60	0.21	21.5
	11	64	10.8	36.0	25.5	3.6	2.2	87.0	145	6.5	55	3.00	0.18	20.8
MW-25d	BP	54	32.3	135.0	59.4	6.7	3.8	158.4	264	7.0	31	2.00	0.83	19.2
	1	57	28.2	72.0	66.0	5.4	3.6	133.2	222	6.9	31	1.40	0.73	18.8
	6	63	23.8	72.0	52.8	4.3	3.0	151.2	252	6.8	26	1.00	0.65	19.0
	11	57	22.4	68.3	61.8	3.8	3.0	104.4	174	6.9	51	1.00	0.52	17.7
MW-100s	BP	73	17.5	108.0	140.8	26.5	16.2	126.2	210	6.7	34	76.40	1.90	33.8
	1	72	12.8	67.5	44.0	8.0	4.4	108.2	180	6.7	33	15.80	1.00	29.6
	6	73	9.9	36.0	36.0	3.6	1.7	124.2	207	6.6	28	3.40	0.73	28.4
	11	75	9.7	24.2	14.5	3.5	1.4	91.8	153	6.7	74	1.70	0.66	25.5
MW-100m	BP	64	33.8	79.2	70.0	6.5	2.5	169.2	282	7.4	32	0.44	0.57	22.3
	1	72	28.1	72.0	61.6	4.1	2.5	150.0	250	7.1	34	0.88	0.65	24.8
	6	68	25.2	67.5	66.0	3.7	2.2	160.0	267	7.3	28	0.15	0.42	25.2
	11	72	23.5	58.8	61.6	3.3	2.1	103.0	173	7.2	60	0.40	0.41	24.5
MW-375s	BP	77	13.1	43.2	21.7	3.8	2.3	126.0	210	6.5	35	0.37	0.13	26.0
	1	77	30.9	74.7	40.0	4.1	3.3	150.0	250	7.0	34	0.57	0.16	23.1
	6	77	9.3	35.1	30.8	3.2	1.5	124.8	208	6.5	28	1.40	0.18	28.0
	11	77	8.3	27.3	24.0	2.4	1.6	85.8	143	6.7	60	--	--	20.9
MW-375d	BP	59	37.0	175.5	64.7	4.3	3.3	162.0	270	8.0	33	0.37	0.13	20.0
	1	77	35.1	81.0	63.8	4.6	3.4	152.4	254	7.6	32	0.57	0.16	20.6
	6	--	--	--	--	--	--	--	--	--	--	--	--	--
	11	75	27.7	69.2	74.8	3.9	2.9	114.0	190	7.7	56	1.40	0.18	20.8

(continued)

TABLE 3-4. WATER QUALITY DATA FOR THE MONITORING NETWORK NEAR PW-5, LITTLETON, MASSACHUSETTS
(Part 2 of 4: Locations PW-5 through MW-375d; Parameters Nitrate through VOCs)

Sample Location	Days of Pumping	Water Quality Parameters (continued)														
		General and Site-Specific (continued)				Site-Specific Water Quality										
		(mg/l)				(µg/l)										
		NO_3	NO_2	NH_3	SO_4	As	Ba	Cd	Cr	Cu	Pb	Hg	Se	Ag	Pest/Herb†	VOCs
PW-5	BP	ND	ND	ND	4.0	0.006	0.134	*	*	*	*	*	*	*	ND	ND
	1	ND	ND	ND	0.0											
	6	ND	ND	ND	8.5											
	11	ND	ND	ND	4.0	0.003	0.103	*	*	*	*	*	*	*	ND	ND
MW-25s	BP	ND	ND	ND	2.0											
	1	ND	ND	ND	0.0											
	6	1.3	ND	ND	40.0											
	11	2.2	ND	ND	0.0											
MW-25d	BP	1.4	ND	ND	0.0											
	1	ND	ND	ND	7.0											
	6	2.0	ND	ND	10.5											
	11	1.7	ND	ND	7.5											
MW-100s	BP	1.3	ND	1.0	6.0											
	1	2.5	ND	ND	0.0											
	6	1.5	ND	ND	12.5											
	11	1.3	ND	ND	30.0											
MW-100d	BP	1.8	ND	ND	5.0											
	1	ND	ND	ND	0.0											
	6	3.8	ND	ND	3.0											
	11	ND	ND	ND	7.0											
MW-375s	BP	1.6	ND	ND	4.0											
	1	1.4	ND	ND	26.0											
	6	ND	ND	1.0	0.0											
	11	2.0	ND	1.0	25.0											
MW-375d	BP	1.7	ND	ND	5.0											
	1	ND	ND	ND	7.5											
	6	--	--	--	--											
	11	ND	ND	ND	5.0											

(continued)

TABLE 3-4. WATER QUALITY DATA FOR THE MONITORING NETWORK NEAR PW-5, LITTLETON, MASSACHUSETTS
(Part 3 of 4: Locations MW-425 through Spectacle Pond; Parameters Temperature through Sodium)

Sample Location	Days of Pumping	Water Quality Parameters (continued)												
		General Water Quality							General and Site-Specific Water Quality					
		Temp (°F)	(mg/l)						Cond (µS/cm)	pH	(mg/l)			
			Ca	Hard	HCO_3	K	Mg	TDS			Cl	Fe	Mn	Na
MW-425	BP	57	29.2	73.8	38.3	5.8	3.5	174.0	290	6.5	40	1.60	0.42	24.6
	1	--	37.5	72.0	52.8	5.9	3.7	183.6	306	6.8	41	0.42	0.42	25.8
	6	52	26.5	57.6	29.9	4.5	3.1	183.0	306	6.3	44	0.22	3.10	30.8
	11	52	24.4	73.3	25.5	3.8	3.0	140.0	234	6.4	85	0.09	0.14	29.2
MW-660s	BP	59	27.4	72.0	63.8	15.4	4.6	144.0	240	7.3	31	6.80	0.59	22.4
	1	59	24.4	57.6	49.3	8.2	3.2	132.2	220	6.7	33	6.40	0.58	23.5
	6	54	11.8	40.5	33.3	3.3	1.3	118.2	197	6.4	28	0.94	0.43	20.2
	11	52	17.2	54.0	33.0	6.7	2.6	92.4	154	6.5	67	5.00	0.58	25.9
MW-660d	BP	57	32.4	81.0	78.2	18.1	4.4	153.0	255	7.7	36	1.10	0.05	24.2
	1	57	27.4	71.9	92.4	14.9	3.6	120.0	200	7.3	33	0.62	0.52	21.5
	6	54	27.1	112.5	88.0	13.5	3.7	185.4	309	8.0	22	3.20	0.19	30.1
	11	54	28.2	86.5	94.6	14.3	3.8	136.2	227	7.8	43	2.40	0.26	28.5
MW-800	BP	54	26.7	62.1	42.7	8.6	3.5	136.8	228	6.5	30	3.00	0.35	16.6
	1	55	25.0	35.6	35.6	4.5	2.7	126.0	210	6.2	27	0.43	0.15	14.3
	6	50	17.6	54.0	28.6	3.5	2.0	117.0	195	6.2	20	0.27	0.16	14.0
	11	52	14.5	44.0	38.7	3.3	1.9	69.0	115	6.6	38	0.04	0.13	13.8
Spectacle Pond	BP	77	160	360	22.0	2.4	2.2	112.8	188	7.4	28	0.18	0.07	19.3
DETECTION LIMITS		N/A	0.007	1.0	1.0	0.02	0.002	1.0	N/A	N/A	1.0	0.003	0.001	0.05

(continued)

TABLE 3-4. WATER QUALITY DATA FOR THE MONITORING NETWORK NEAR PW-5, LITTLETON, MASSACHUSETTS
(Part 4 of 4: Locations MW-425 through Spectacle Pond; Parameters Nitrate through VOCs)

Sample Location	Days of Pumping	Water Quality Parameters (continued)														
		General and Site-Specific (continued)				Site-Specific Water Quality										
		(mg/l)				(µg/l)										
		NO_3	NO_2	NH_3	SO_4	As	Ba	Cd	Cr	Cu	Pb	Hg	Se	Ag	Pest/Herb	VOCs
MW-425	BP	5.5	0.25	ND	17.5	*	0.097	*	*	*	*	*	*	*	ND	ND
	1	3.5	ND	ND	16.5											
	6	3.9	ND	ND	15.0											
	11	3.4	ND	ND	20.5	*	0.093	*	*	*	*	*	*	*	ND	ND
MW-660s	BP	2.6	ND	ND	15.0	*	0.136	*	*	*	0.001	*	*	*	ND	ND
	1	1.7	ND	ND	10.0											
	6	ND	ND	ND	7.5											
	11	1.2	ND	ND	15.0	*	0.130	*	*	*	*	*	*	*	ND	ND
MW-660d	BP	ND	ND	ND	2.0	0.003	0.032	*	*	*	0.002	*	*	*	ND	ND
	1	ND	ND	ND	5.0											
	6	2.9	ND	ND	0.0											
	11	ND	ND	ND	3.0	0.005	0.043	*	*	*	*				ND	ND
MW-800	BP	2.2	ND	ND	7.5	*	0.129	*	*	*	*	*	*	*		
	1	2.4	ND	ND	11.5											
	6	2.7	ND	ND	9.5											
	11	2.0	ND	ND	7.5	*	0.093	*	*	*	*	*	*	*		
Spectacle Pond	BP	3.2	ND	ND	6.0											
	1															
	6															
	11															
DETECTION LIMITS		1.0	0.1	0.01	1.0	0.003	0.001	0.004	0.009	0.01	0.001	0.0002	0.003	0.004		0.4

Abbreviations: PW-5 = Production Well Number 5; MW = monitoring well; Ca = calcium; Hard = hardness; HCO_3 = bicarbonate; K = potassium; Mg = magnesium; TDS = total dissolved solids; Cond = specific conductivity; Cl = chloride; Fe = iron; Mn = manganese; Na = sodium; BP = before pumping; s = shallow depth well; d = deep well; m= moderate depth well; -- = not tested; NO_3 = nitrate-nitrogen; NO_2 = nitrite-nitrogen; NH_3 = ammonia; SO_4 = sulfate; As = arsenic; Ba = barium; Cd = cadmium; Cr = cromium; Cu = copper; Pb = lead; Hg = mercury; Se = selenium; Ag = silver; Pest/Herb = pesticides (EPA method 608) and herbicides (EPA method 8150); VOCs = volatile organic compounds (EPA method 524.2); * = below minimum detection limit; ND = not detected; N/A = not applicable.

† Detection limits for pesticides and herbicides are compund-specific and so are applicable for either group.

Ambient Trend Monitoring--

Ambient-trend monitoring parameters are typically influenced by the native hydrogeochemical environment. Ground water derived from unconfined aquifers in Massachusetts is typically soft to moderately hard, slightly acidic, and low in total dissolved solids (TDS) (Delaney and Maevsky, 1980). Most of the higher-yielding aquifers are found within the sand and gravel, glacial meltwater deposits where predominate constituents include sodium, calcium, magnesium, potassium, sulfate, chloride, and bicarbonate.

To evaluate spatial water quality variations within the near-field pumping area, samples were collected on August 15, 1990, throughout the monitoring well network shown in Figure 3-9 and detailed in Table 3-4. These samples were collected under static water-level conditions created by shutting down PW-5 48 hours prior to the sampling event. Temporal variations in water quality were evaluated by comparing successive samples collected from the same monitoring station over the duration of the long-term aquifer test. These sampling events were conducted on August 17, August 22, and August 27 and represent days 1, 6, and 11 of the long-term test, respectively.

Water quality data obtained during these sampling events are shown in Table 3-4. Several graphical techniques were used to identify spatial and temporal water quality variations. Particular attention was focused on hydrochemical changes within the near-field capture zone. In general, these trends are best illustrated by comparing the samples collected prior to pumping versus 11 days after pumping.

Source Assessment Monitoring--

Source assessment monitoring was conducted to identify ground-water contamination threats within the Bennetts Brook watershed. Review of source assessment records indicate that the most probable ground-water contamination sources are posed by indiscriminate dumping activities within industrial and commercial developments (U.S. EPA, 1990a). Disposal of toxic substances used within residential developments and chemical applications used for agricultural operations (historical and existing), illegal dumping, leaking USTs, and road salting operations were also considered. Naturally occurring constituents such as iron and manganese, were also evaluated. At elevated concentrations, iron and manganese will create significant aesthetic problems in water supply systems (Silvey and Johnston, 1977).

Monitoring of all the potential contaminants associated with the identified land-uses listed in Table 3-1 is beyond the scope and budget of this investigation. To maximize the amount of information obtained during the sampling program, while minimizing the analytical costs, target chemicals were identified. The "site-specific" parameters in Table 3-4 represent the more ubiquitous compounds identified in past ground-water contamination studies. In addition, monitoring wells, which were sampled for VOCs (EPA method 524.2), heavy metals (the Safe Drinking Water Act), pesticides (EPA method 608), and herbicides (EPA method 8150), were carefully selected based on upgradient source assessment activities. Although semi-volatile constituents are often included in ground-water contamination investigations, they were not included in this study as initial screening parameters because of their high analytical cost. Studies have demonstrated that, in the absence of VOCs, these compounds are seldom detected in ground-water (Rosenfeld, 1990). Analysis for VOCs is a screening technique; if detected, their presence will justify the need for more complete compound analysis.

Source-assessment monitoring stations were chosen based on the conceptual understanding of ground-water flow and with consideration of existing or historical upgradient threats. With these factors in mind, PW-5, MW-425, MW-660 (deep and shallow wells), and MW-800 were included for source assessment monitoring. In general, the Water Department attempts to site source assessment monitoring wells in groups of 3 to 4. At a minimum, one well monitors the upgradient source area, while 2 to 3 wells monitor the downgradient source area.

Sample Collection--

Representative water samples were obtained by purging each well a minimum of three times the borehole volume of water, or until field-measured temperatures and conductivities had reached stabilization. To eliminate possible errors introduced by cross contamination, each well was evacuated and sampled by a dedicated, inertial pump composed of small-diameter [0.625 inch OD] polyethylene flexible-tubing coupled to a Teflon foot valve. The effectiveness of these sampling devices for development, purging, and sampling is documented by Rannie and Nadon (1988).

To minimize quantitative errors attributed to sample collection and analysis, a water quality assurance (QA) and quality control (QC) plan was developed (Huibregtse and Moser, 1976). Storage, handling, and analytical procedures were conducted in accordance with standard EPA methods. All analyses were performed at state-certified analytical laboratories, which adhere to standard analytical protocol including chain-of-custody documentation, percent recovery and surrogate spike analysis, and use of trip and laboratory blanks. As an additional QA/QC check, only analytical results with a cation-anion balance within 20% error or less were considered acceptable for interpretation.

DATA INTERPRETATION

Conceptual Model

The Bennetts Brook watershed is composed of approximately 3.1 square miles of glacial drift, 1.1 square miles of wetlands, and 3.2 square miles of glacial till and bedrock with an estimated annual ground-water recharge rate of 3.1 million gallons per day (Mgd) (Metcalf and Eddy, Inc., 1981). The watershed is bound to the west by a ground-water divide and to the north, south, and east by glacial till and bedrock prominences. Permeable glacial outwash deposits appear to pinch out along the till and bedrock uplands. Regional topographic relief varies from 190 feet above mean sea level (amsl) at the pond to 350 feet amsl along the drumlins to the north.

Three distinct aquifers border the study area (Figure 3-11). Ground water within the Spectacle Pond aquifer originates at the headwaters of the Stony Brook subbasin and feeds into the Merrimack River basin. The Cow Brook subbasin envelopes the aquifer due north and flows northeast toward the Merrimack River. These two aquifers are separated by a surface-water divide formed by bedrock prominences. The third aquifer located west of Bennetts Brook drains into the Nashua River basin. A ground-water divide, located adjacent to the southeastern shore of Sandy Pond, separates the Nashua River basin from the Stony Brook subbasin.

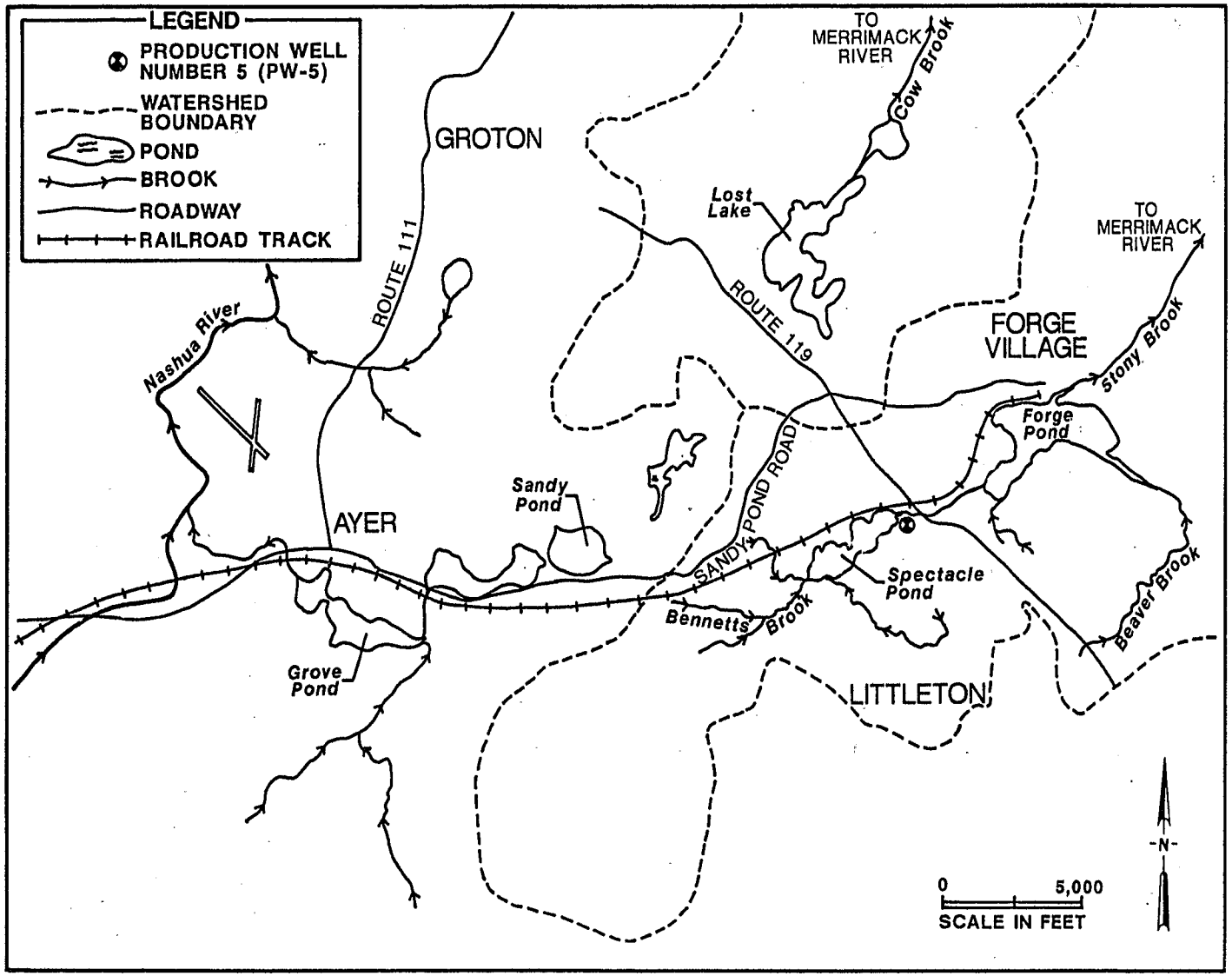

Figure 3-11. Watersheds and hydrogeologic divides near Bennetts Brook, Littleton, Massachusetts.

Regional geomorphology and aquifer-test interpretations indicate that these aquifers are predominantly unconfined. The regional ground-water table is shallow and varies in depth between 0 and 25 feet. In the immediate vicinity of PW-5, unconfined conditions exist to the east within the sands and gravels. Beneath the wetlands that extend to the west and under Spectacle Pond, the aquifer is semi-confined.

In the areas of Spectacle Pond and Forge Pond, the glacial outwash deposits have high transmissivity values of greater than 4,000 square feet per day (ft^2/d) and high potential yields of greater than 300 gpm. Zones of lower transmissivity (less than 1,350 ft^2/d) are found along the perimeter of buried valleys where the outwash materials are poorly sorted and the aquifer thickness pinches out into till and bedrock uplands (USGS, 1985).

Spectacle Pond appears to be dimictic (subject to spring and fall lake turnover) in the deeper basins but polymictic (continuous mixing) throughout the rest of the pond, with gradually sloping banks extending into the two basins. Landforms adjacent to the pond display characteristic ice-contact relief. The present shoreline may reflect impacts from a man-made weir near the outflow channel by Route 119. Seasonal vegetative growth and decay cycles contribute to the formation of peat deposits within each basin. Surface-water flows northeast and is restricted at the outflow channel by the weir. The direct correlation between drainage area size and mean annual discharge for watersheds within north-central Massachusetts show that the mean annual discharge from this watershed is approximately 12.09 cubic feet per second (cfs).

Ground-water and surface-water flow within the aquifer is influenced by constant municipal well withdrawal along the northwest and southeast shores of the pond. These production wells exaggerate the hydraulic connection between the surface- and ground-water systems. The extent of the capture zones is influenced by the rate of induced infiltration, which in turn is influenced by the areal extent, thickness, and type of peat deposits present.

Extensive organic deposits were identified in the eastern lobe of Spectacle Pond during peat-probing activities. Measured thicknesses vary from less than 1 foot to greater than 34 feet (Figures 3-12A and 3-12B). The greatest peat thicknesses were encountered near the existing outflow where the outflow stream channel appears to be sink for organic debris carried through the pond (stratigraphic sections C-C′ and D-D′). The peat encountered during field probing was loosely packed and fibrous, with variable thickness across the basin. A large, irregularly-contoured prominence, possibly a submerged ice-contact feature, was identified within the eastern basin, which may allow a hydraulic connection between the pond and the aquifer (stratigraphic sections B-B′ and C-C′).

Aquifer Testing and Analyses

Short-Term Aquifer Test--

The drawdown and recovery data for each monitoring station used during the short-term aquifer test were analyzed using four different methods to simulate distinct aquifer conditions. The computer program AQTESOLV (Duffield and Rumbaugh, 1989) was used to perform these analyses. AQTESOLV permits aquifer-test analysis using both type-curve and automatic-fitting procedures. A typical printout from AQTESOLV, which depicts the fit of the recovery data for piezometer P-20 using the Neuman analysis, is shown in Figure 3-13.

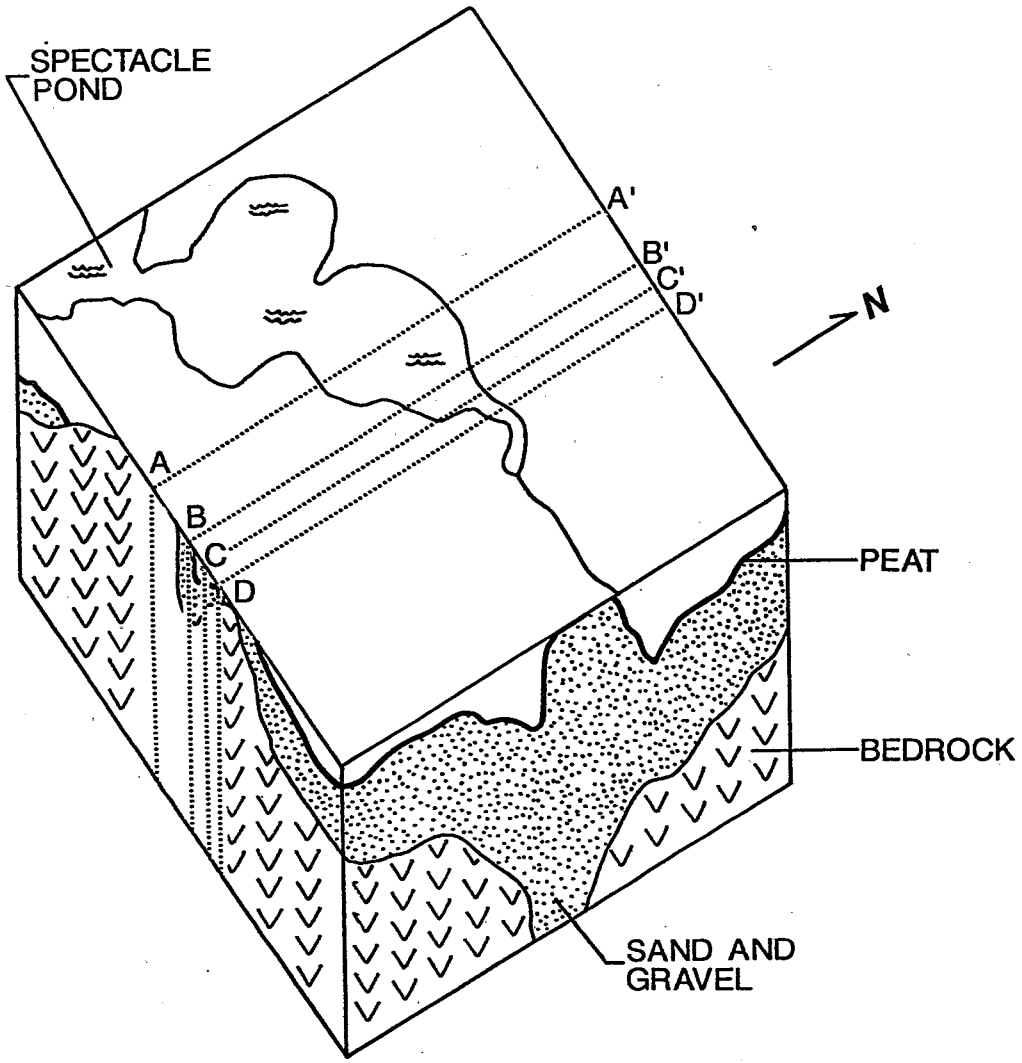

Figure 3-12A. A location map of stratigraphic sections across Spectacle Pond, Bennetts Brook watershed, Littleton, Massachusetts.

Ancillary water-level effects associated with changes in atmospheric pressure and recharge from rainfall were neglected in the data interpretation. Walton (1987) noted that water-level changes induced by fluctuations in barometric pressure are commonly negligible for unconfined aquifers. A very small variation in atmospheric pressure was measured during the aquifer test (0.4 foot of water); therefore, data were not corrected for this effect. Data were not corrected for precipitation because neither the water levels in the piezometers outside the area-of-influence of the pumping well, nor the water levels in Spectacle Pond showed a clear correlation to precipitation during the test. The municipal wells in the town of Ayer, located on the western shore of Spectacle Pond (Figure 3-2), were run at a constant rate throughout the pumping and recovery portions of the test. Therefore, no transient effects were caused by the operation of these wells.

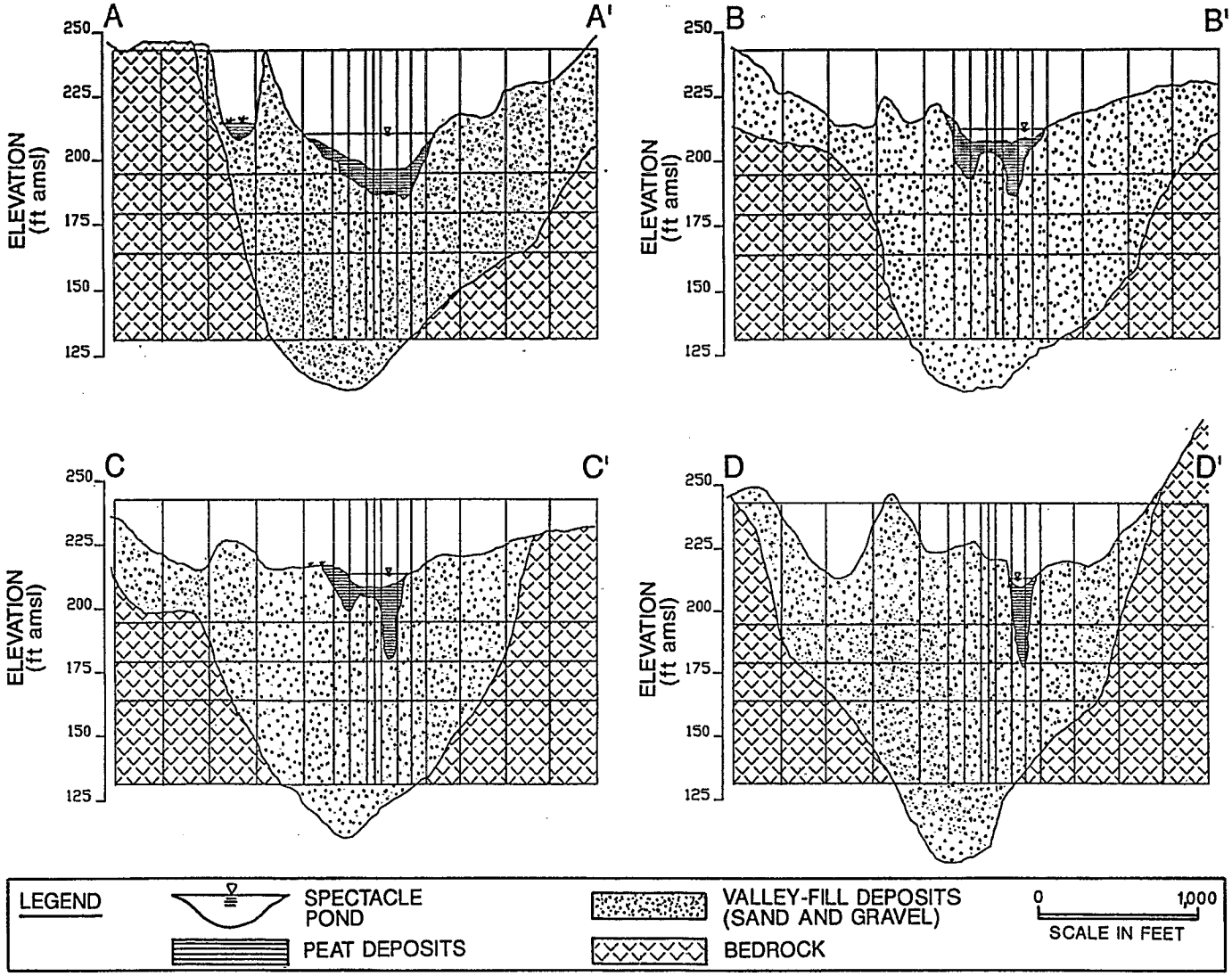

Figure 3-12B. Stratigraphic sections across Spectacle Pond, Bennetts Brook watershed, Littleton, Massachusetts.

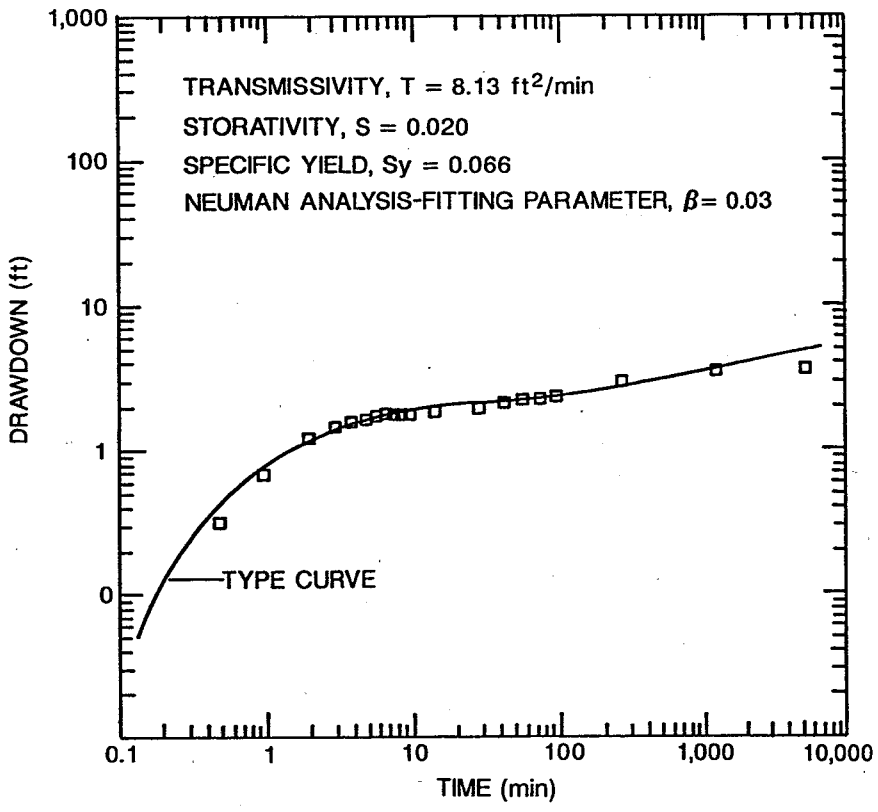

Figure 3-13. Neuman type-curve analysis of piezometer P-20 recovery data, short-term test, Bennetts Brook watershed, Littleton, Massachusetts.

The analytical results of the short-term test are shown in Table 3-5. The method that provided the best fit to the observed data is marked with an asterisk. Generally, the best fit for the drawdown and recovery curves for observation stations were obtained using the Neuman type curves for unconfined aquifers. The response for piezometer P-L, located on a near-shore island in Spectacle Pond (Figure 3-8A), resembled the Hantush type curve for leaky aquifers. This fit may reflect the semi-confining characteristics of the peat layer at the bottom of the pond.

Upon completion of the short-term aquifer test analyses, horizontal hydraulic conductivity values were found to range from 127 to 2,220 ft/d, with an average value of 560 ft/d. The representative horizontal conductivity value was estimated to be approximately 50 times greater than the vertical conductivity value.

The pumping time was too short for reliable estimates to be obtained for delayed yield at points located farther than 600 feet from PW-5. Therefore, the results from OW-800 (Figure 3-9) across the pond may be masked by this effect. Data from piezometers P-H, P-I, P-J, and P-N were

TABLE 3-5. AQUIFER PARAMETER RESULTS FOR THE SHORT-TERM AQUIFER TEST AT PW-5, LITTLETON, MASSACHUSETTS

Monitoring Station	R (ft)	Method	Aquifer Parameters T (ft^2/min)	Kr (ft/d)	S	Sy or S	β	Kr/Kz
PW-5	0	Pt-Th	24	576				
		Pt-C	24	576				
OW-2	2	Pt-N	5	127	0.01400		0.001	1.00
		Rt-N*	32	775			0.030	0.03
		Pt-Th	17	410				
		Rt-Th	26	640				
		Pt-C	25	604				
P-20	20	Pt-N	13	316	0.01100	0.0810	0.004	28.00
		Rt-N*	8	194	0.02000	0.0660	0.030	4.00
		Pt-Th	17	412		0.0150		
		Rt-Th	23	573		0.0028		
		Pt-C	19	477		0.0052		
		Rt-C	22	532		0.0032		
P-50	50	Pt-N*	21	523	0.00400	0.0300	0.010	69.00
		Rt-N	23	564	0.01600	0.0260	0.010	69.00
		Pt-Th	21	513		0.0220		
		Rt-Th	24	576		0.0250		
		Pt-C	21	508		0.0200		
		Rt-C	24	588		0.0230		
P-Cs	50	Pt-Th	18	451		0.0550		
		Pt-C*	18	451		0.0540		
OW-100	100	Pt-N*	21	511	0.00310	0.0160	0.180	21.00
		Rt-N	21	518	0.00150	0.0110	0.060	63.00
		Pt-Th	27	664		0.0030		
		Rt-Th	33	796		0.0017		
		Rt-C	29	715		0.0029		
OW-150	150	Pt-N	18	444	0.00300	0.0350	0.100	28.00
		Rt-N*	27	648	0.00630	0.0150	0.200	14.00
		Rt-Th	39	938		0.0037		
		Rt-C	35	840		0.0054		
P-150	150	Pt-Th	21	760		0.0041		
		Pt-C*	21	508		0.0041		

(continued)

TABLE 3-5. AQUIFER PARAMETER RESULTS FOR THE SHORT-TERM AQUIFER TEST AT PW-5, LITTLETON, MASSACHUSETTS (continued)

Monitoring Station	R (ft)	Method	Aquifer Parameters					
			T (ft^2/min)	Kr (ft/d)	S	Sy or S	β	Kr/Kz
OW-400	400	Pt-N*	45	1,101	0.01300	0.0670	1.500	30.00
		Pt-Th	48	1,171		0.0620		
		Rt-Th	92	2,220		0.0660		
		Pt-C	41	991		0.0810		
P-Gd	441	Pt-N*	22	544	0.00041	0.0270	0.490	109.00
		Pt-Th	26	631		0.0210		
		Pt-C	28	672		0.0180		
P-Gs	441	Pt-N*	27	664	0.00170	0.0400	0.400	135.00
		Pt-Th	26	624		0.0350		
P-H	510							
P-L	655	Pt-Th	35	842		0.1200		
		Pt-C	36	862		0.1000		
		Rt-H*	11	263		0.0086	r/β=1.800	
P-I	993	Pt-N*	33	813		0.0140	6.000	45.00
		Pt-Th	42	1,023		0.0083		
		Pt-C	42	1,010		0.0077		
P-J	1,393	Pt-Th	89	2,150		0.1000		
		Pt-C*	93	2,236		0.0780		
MW-T	800	Pt-N*	59	1,435	0.02700	0.0270	5.000	83.00
		Pt-Th	59	1,418		0.0280		
		Pt-C	62	1,495		0.0190		
All Wells	N/A	TDD	31	746		0.0340		

Abbreviations: PW-5 = Production Well Number 5; R = distance from PW-5 to monitoring station; T = transmissivity; Kr = horizontal hydraulic conductivity; S = storativity; Sy = specific yield; β = Neuman analysis-fitting parameter $[(Kz/Kr)(r/b)^2]$; Kr = horizontal hydraulic conductivity; Kz = vertical hydraulic conductivity; Kr/Kz = anisotropy ratio; Pt = pumping test; Th = Theis corrected drawdown analysis; C = Cooper-Jacob corrected drawdown analysis; OW = observation well; N = Neuman analysis; Rt = recovery test; P = piezometer; s = shallow depth well; d = deep well; H = Hantush analysis with aquitard storage; MW = monitoring well; N/A = not applicable; TDD = Theis drawdown versus distance analysis for all wells.
* = Best-fit method.

not reliable enough to be analyzed. It is possible that these piezometers were clogged by native materials. In conclusion, it was determined that additional monitoring points across the pond and a longer pumping period would be necessary in the design of the long-term aquifer test.

Long-Term Aquifer Test--

Data from the long-term aquifer test were interpreted in the same way as the short-term test data. For reasons discussed earlier, barometric pressure and pumping effects from the town of Ayer's water supply wells were not corrected in the analyses. However, rainfall data showed a clear correlation to pond water levels. Water-level data from observation wells near the pond were corrected in a linear fashion for temporal variations in the pond.

Interpretations of the long-term test data are summarized in Table 3-6. There were no water-level responses in OW-1200 located on the far side of the pond (Figure 3-9), indicating that contribution from the northwestern portion of the aquifer is drawn entirely from surface water through the bottom of the pond. Investigations of peat thickness in the area between PW-5 and OW-1200 showed the existence of a sandy ridge along the bottom of the pond, halfway between these two stations (stratigraphic sections B-B′ and C-C′, Figure 3-12B). This ridge provides direct hydraulic connection between the surface water and ground water. On the other hand, OW-800, located on the opposite side of the pond but farther east than OW-1200, showed measurable drawdown. Hence, water entering PW-5 from the north is induced flow from the pond and ground-water flow beyond the pond.

Water-level monitoring at cluster wells MW-375s, -375m, and -375d (designating shallow, moderate, and deep well depths, respectively), located on the near-shore island (Figure 3-9), reveal a downward hydraulic gradient during testing which increased in time before stabilization. This vertical gradient indicates that water from the pond is recharging the aquifer when PW-5 is operating. Immediately following the start-up of the pump, most of the flow is horizontal. Thereafter, water is withdrawn from the pond until equilibrium is established. The hydraulic conductivity values estimated from these monitoring stations range from 2,000 to 3,000 ft/d, well above the average value calculated for the other stations (approximately 520 ft/d). In conclusion, this section of the aquifer, receives approximately five to six times more water from surface water than from ground water. By averaging all ground-water fluxes reaching the well, approximately 20% to 25% of the water recharging PW-5 is believed to be derived from Spectacle Pond.

At all other monitoring stations, responses to pumping were similar to those observed during the short-term test. Again, hydraulic conductivity values ranged from about 300 to 3,600 ft/d with an average value of 520 ft/d. These values are typical for well-sorted sand and gravel outwash deposits. The Neuman analysis-fitting parameter (β), used to calculate the anisotropy ratio of Kr/Kz, indicates that regional values of vertical hydraulic conductivity (Kz) are relatively small. This suggests that the prevailing ground-water flow component is horizontal (Kr). The anisotropy ratio, determined using data collected during the short- and long-term aquifer tests, is approximately 35:1. The average specific yield, which was calculated using the best-fit analytical solution, was approximately 0.1054. Typically, specific yield is representative of porosity values within sand and gravel aquifers. Pond effects may have created overestimates of specific yield at MW-25s and MW-25d (Figure 3-9). The average storativity value of 0.0216 is one order of magnitude lower than the specific yield and is characteristic of an unconfined aquifer.

TABLE 3-6. AQUIFER PARAMETER RESULTS FOR THE LONG-TERM AQUIFER TEST AT PW-5, LITTLETON, MASSACHUSETTS

Monitoring Station	R (ft)	Method	Aquifer Parameters					
			T (ft^2/min)	Kr (ft/d)	S	Sy or S	β	Kr/Kz
P-20	20	Pt-Th*	11	270		0.100		
		Rt-Th	13	315		0.200		
MW-25s	25	Pt-C	21	507		0.340		
		Pt-N	19	463	0.0410	0.440	0.20	0.9
		Rt-N*	24	576	0.0560	0.410	0.20	0.9
MW-25d	25	Pt-Th	20	484		0.230		
		Pt-N*	16	393	0.0340	0.480	0.10	1.7
		Rt-N	19	458	0.0450	0.370	0.10	1.7
P-50	50	Pt-N	16	399	0.0056	0.051	0.06	12.0
		Rt-N*	15	361	0.0041	0.061	0.06	12.0
P-100s	100	Pt-Th	20	487		0.026		
		Pt-C	19	473		0.022		
		Pt-N*	18	438	0.0044	0.029	0.20	14.0
		Rt-N	19	471	0.0037	0.014	0.30	9.0
OW-150	150	Pt-N*	18	446	0.0057	0.130	1.50	4.0
		Rt-N	20	490	0.0120	0.150	1.50	4.0
		Rt-C	24	589		0.180		
MW-375s	375	Pt-H*	96	2,312		0.014		
		Pt-Th	110	2,640		0.014		
MW-375d	375	Pt-Th	110	2,654		0.030		
		Rt-Th*	136	3,267		0.012		
OW-400	400	Pt-C*	37	888	0.0420			
		Pt-Th	31	744	0.0370			
MW-425	425	Pt-N*	16	404	0.0031	0.120	7.00	7.0
		Rt-N	18	449	0.0023	0.087	7.00	7.0
MW-800	800	Pt-C	20	486		0.061		
		Pt-Th	15	367		0.085		

(continued)

TABLE 3-6. AQUIFER PARAMETER RESULTS FOR THE LONG-TERM AQUIFER TEST AT PW-5, LITTLETON, MASSACHUSETTS (continued)

Monitoring Well	R (ft)	Method	Aquifer Parameters T (ft^2/min)	Kr (ft/d)	S	Sy or S	β	Kr/Kz
OW-800	800	Pt-C	53	1,273			0.40	444.0
		Rt-Th	42	1,016		0.130		
OW-1200	1,200		NR	NR				
All Wells	N/A	TDD	36	801		0.034		

Abbreviations: PW-5 = Production Well Number 5; R = distance from PW-5 to monitoring station; T = transmissivity; Kr = horizontal hydraulic conductivity; S = storativity; Sy = specific yield; β = Neuman analysis-fitting parameter $[(Kz/Kr)(r/b)^2)]$; Kr = horizontal hydraulic conductivity; Kz = vertical hydraulic conductivity; Kr/Kz = anisotropy ratio; P = piezometer; Pt = pumping test; Th = Theis corrected drawdown analysis; Rt = recovery test; MW = monitoring well; s = shallow depth well; C = Cooper-Jacob corrected drawdown analysis; N = Neuman analysis; d = deep well; OW = observation well; H = Hantush analysis with aquitard storage; NR = no response; N/A = not applicable; TDD = Theis drawdown versus distance analysis for all wells.
* = Best-fit method.

Ground-Water Flow Modeling

Two-Dimensional, Steady-State, Regional Flow Model--

Model Description--FLOWPATH is a steady-state, 2-D, horizontal plane, ground-water flow and pathline analysis model (Franz and Guiguer, 1989). FLOWPATH is based on the block-centered, finite difference and the particle tracking methods. The model can simulate horizontal ground-water flow in heterogeneous aquifers, anisotropic aquifers, and in confined, unconfined, and leaky aquifers. It can simulate withdrawal and injection of water at multiple wells.

Model Assumptions and Limitations--The implementation of the 2-D, finite difference method yields a numerical approximation of the modeled system. Upon discretization of the natural system into a large number of rectangular cells, a differential water balance is written for each cell. In the block-centered, finite difference formulation, a system of equations is solved for the hydraulic head at the center of each cell. All flow parameters, including the calculated hydraulic head value, represent an average value over the entire cell volume, neglecting any small-scale variations or vertical gradients. Good approximations are usually obtained for regional-scale ground-water flow, where the horizontal extent of the aquifer is much greater than its vertical dimension.

The 2-D formulation does, however, generate inaccuracies around the partially-penetrating production well and close to surface-water bodies where 3-D flow prevails. In the context of regional-scale modeling, and especially for the purpose of siting monitoring wells at a significant

distance from the pumping well, these inaccuracies are considered minimal and are not expected to affect the validity of the modeling results. A steady-state, 2-D simulation of regional ground-water flow was conducted to provide conservative, long-term estimates of the capture zone for PW-5.

Grid Design and Boundary Conditions--The model grid design satisfies two main requirements: (1) the hydraulic gradients induced by pumping the municipal wells in Ayer and Littleton are properly represented, and (2) the variable topographic relief in the Bennetts Brook watershed, with its steep slopes, is properly represented in the numerical description of unconfined flow. Some difficulties were encountered initially in meeting the second requirement. Specifically, the rapid variation in elevation head causes large gradients in the total head, while the water depth remains small. This condition can be simulated in a 2-D model by using a very fine model grid. For these reasons, and because of the irregular geometry of the bedrock and glacial till outcrops (model boundaries), a largely uniform grid was designed (Figure 3-14). The grid is comprised of 5,418 cells, the majority of which have dimensions of approximately 150 X 100 feet. The model grid covers a total area of approximately 5 square miles (approximately 3 X 2 miles).

The domain boundary primarily reflects the bedrock and glacial till outcrops, which generally occur at an elevation of approximately 250 feet (Figure 3-15). For the regional model, it was assumed that the water table coincides with the sediment pinch-out (at outcrops) and was modeled as a fixed-head boundary. Alternatively, a flux boundary could have been chosen; however, because site-specific information about the rate of infiltration along these contacts was lacking, a fixed-head boundary was preferred. East and west of the Bennetts Brook watershed, Sandy Pond and Forge Pond were represented as constant-head (flow) boundaries. Short sections of the domain boundary adjacent to these ponds were assumed to coincide with streamlines and, therefore, were modeled as fixed-head boundary segments.

Bedrock Elevation--In an unconfined aquifer model, because the aquifer transmissivity equals the product of hydraulic conductivity and saturated thickness, it is important to quantify the elevation difference between the water table and the aquifer bottom. For this reason, the relationship between hydraulic head and transmissivity requires special attention because of complications caused by the variation in underlying bedrock topography within the model domain.

The base of the aquifer consists of relatively impermeable bedrock. Bedrock outcrops reported by the USGS (1985), gravimetry mapping along several profiles (Metcalf & Eddy, Inc., 1981), and borehole information were used to interpret the bedrock depths and configuration. The bedrock surface was generated using an interpolation algorithm commonly known as Kriging (de Marsily, 1986). This provided a nodal representation of the bedrock surface, which is considered sufficiently accurate, and is shown in Figure 3-15.

Surface-Water and Ground-Water Recharge--The interaction between surface water and ground water in the area of PW-5 was one of the major processes to be determined in the regional modeling study. Data on the bathymetry of Spectacle Pond were obtained from a previous study (GHR Engineering Associates, Inc., 1989) and supplemented by the peat-probing investigation. The peat-probing and aquifer-testing exercises indicate that where peat exists, the peat acts as a relatively impermeable layer that isolates ground water from the pond. Where the peat layer pinches out, approximately in the middle of the eastern lobe of Spectacle Pond, significant hydrologic connection between the pond and ground water was observed.

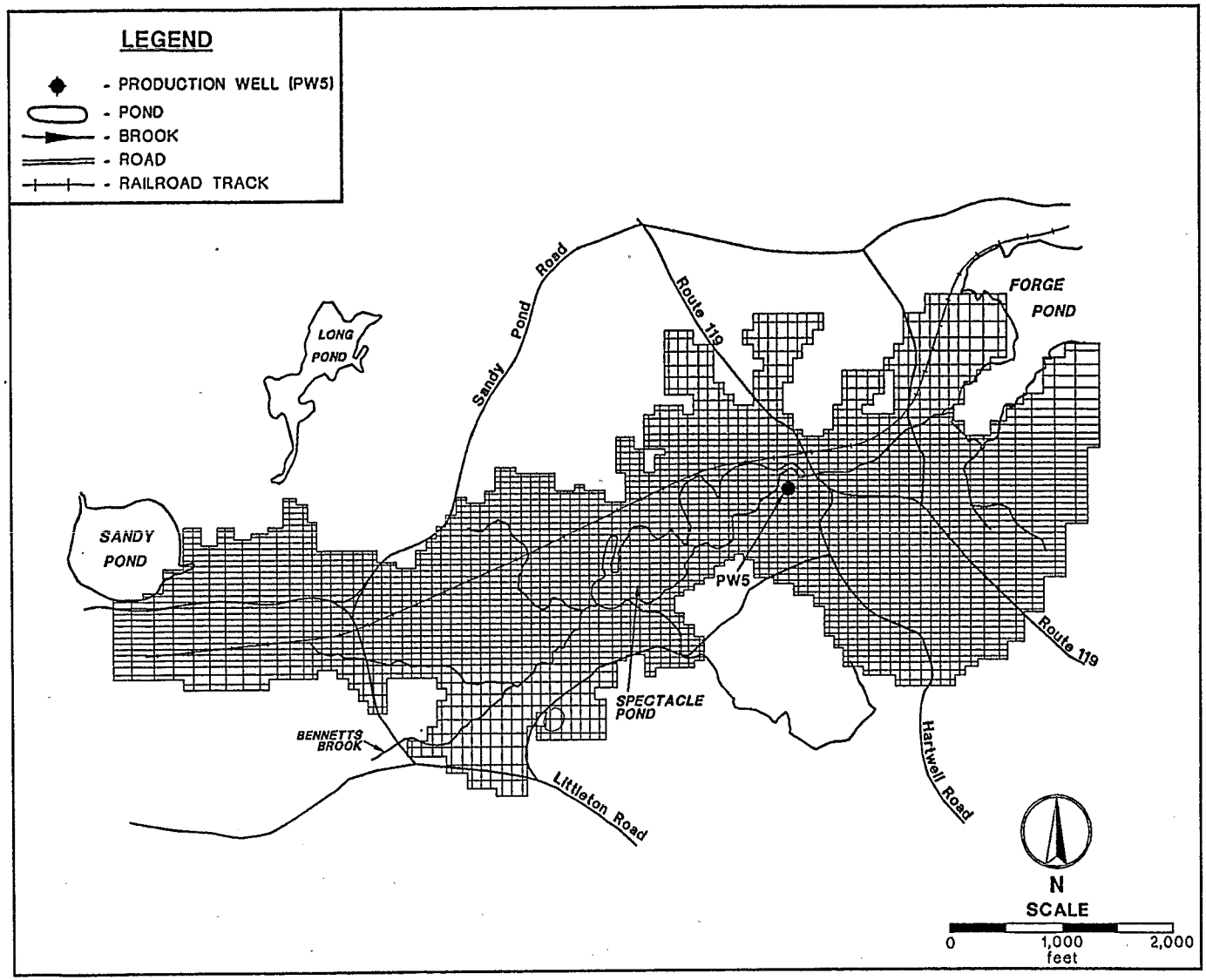

Figure 3-14. Regional two-dimensional model boundary and grid domain for the Spectacle Pond aquifer, Bennetts Brook watershed, Littleton, Massachusetts.

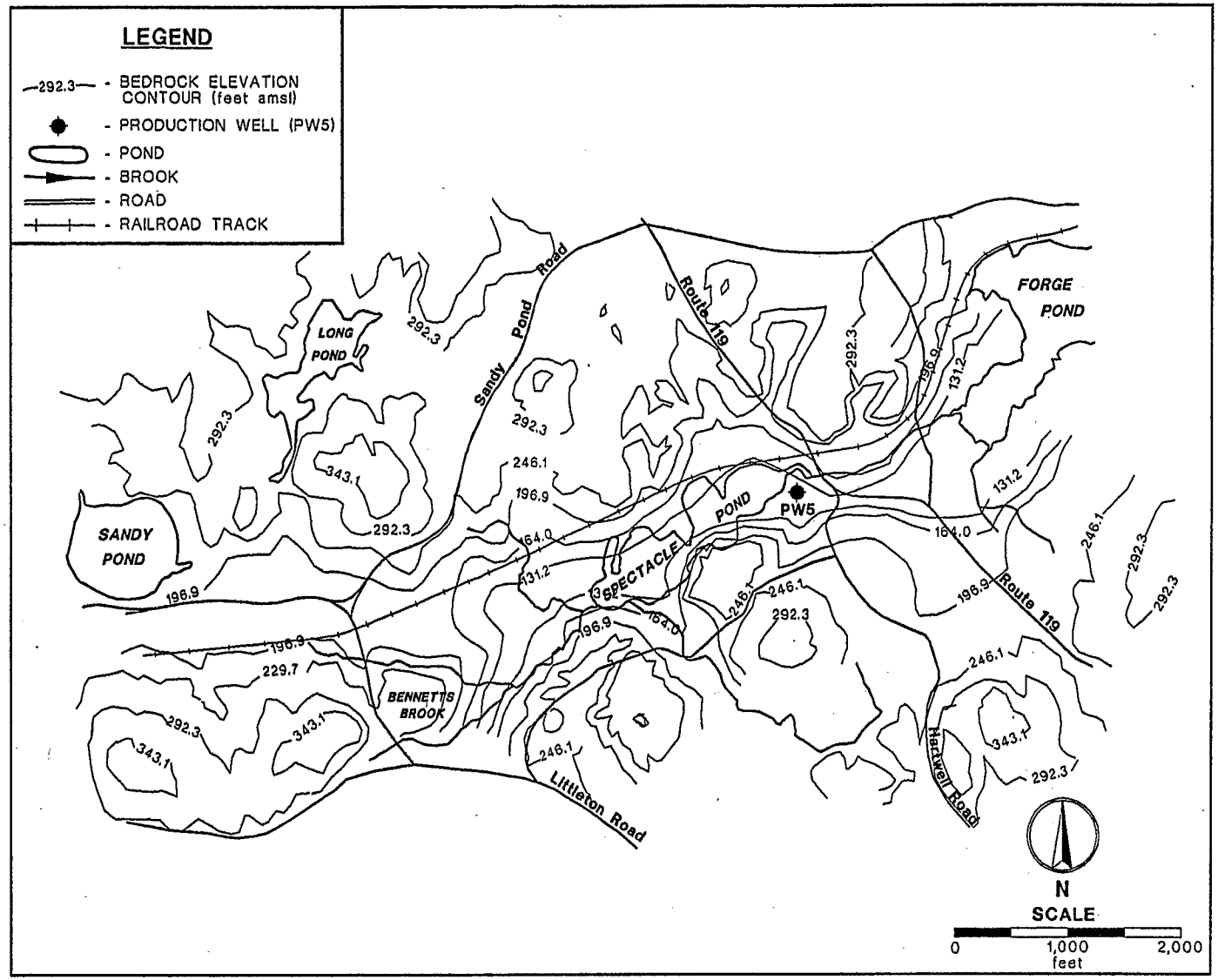

Figure 3-15. Regional bedrock elevation contour map, Bennetts Brook watershed, Littleton, Massachusetts.

In the FLOWPATH simulation model, the interaction of ground water and surface water is modeled using the principle of a leaky aquifer. With this approach, it is possible to account for the fluid flux between the pond and the aquifer, obtained from Darcy's Law. If the water-table elevation is greater than the bottom elevation of the pond, the leakage flux is proportional to the head difference in the pond and the aquifer. Otherwise, the leakage flux is proportional to the depth of water in the pond. The leakage flux is also proportional to the hydraulic conductivity of the sediments lining the surface-water body, and inversely proportional to the thickness of these sediments. The leakage factor used within the modeling exercise is based on site-specific values for peat thickness and hydraulic conductivity. For Spectacle Pond, the leakage factor was estimated to be 0.454 gpd, representing a peat layer with an average thickness of approximately 5 feet and a hydraulic conductivity of 10 ft/d. The hydraulic conductivity estimate was provided by William Nichols (USGS, Augusta, Maine, personal communication, 1990) and is based on field tests conducted in similar peat environments studied in Maine.

Flow data on other surface-water features (tributaries to Spectacle Pond, including Bennetts Brook) in the study area were limited. At the weir located at the outflow of Spectacle Pond, the water level of Gilson Brook drops approximately 8 feet. For all streams, a leakage factor of 1.32 gpd and a water depth of 2 feet were used. Monthly precipitation data from January 1975 to July 1989 were evaluated from records kept by the Littleton Water Department. The average annual precipitation rate during this time period was 48.7 inches per year (in/yr). The resulting annual ground-water recharge rate is approximately 12.2 in/yr, which represents 25% of the total precipitation rate (Frimpter et al., 1988). The remaining 75% is accounted for by evapotranspiration (50%) and runoff (25%).

The model recharge rates were chosen to be spatially variable. In areas of higher hydraulic conductivity, it was assumed that the infiltration rate is higher than average. Recharge rates in aquifer zones with lower hydraulic conductivity values were assumed to be somewhat lower than the estimated average. The model net recharge rates (infiltration minus evapotranspiration) ranged from 8.6 in/yr to 35.8 in/yr, with an average of 14.3 in/yr. Because of the uncertainties and seasonal variations inherent in the surface-water parameters, the leakage factors and recharge rates used to describe the ground-water and surface-water interaction were studied in a sensitivity analysis. The results of the sensitivity analysis are discussed along with the 2-D ground-water flow modeling results.

Hydraulic Conductivity Distribution--U.S. Geological Survey ground-water availability mapping (1985) indicates variations in transmissivity ranging from 0 to 4,000 ft^2/d in the Bennetts Brook watershed. The highest transmissivities are found around Spectacle Pond, where highly permeable outwash deposits occur. Transmissivities decrease toward the outcrop areas where these valley-fill deposits pinch-out. Somewhat lower transmissivities are found in the eastern portion of the model domain where bedrock is reported to occur at shallow depths (GHR Engineering Associates, Inc., 1989).

The hydraulic conductivity distribution was the most important calibration parameter considered during the modeling exercise. Hydraulic conductivity was assumed to be isotropic throughout the model domain, an accepted practice for horizontal, regional-scale modeling. The described transmissivity distribution was implemented qualitatively in the model hydraulic conductivity

distribution. Calibrated hydraulic conductivities range from 26 to 230 ft/d, with the lowest values occurring in the topographically-high areas within the model domain. The greatest values occur around Spectacle Pond and Forge Pond and along the Bennetts Brook valley. A hydraulic conductivity value of 90 ft/d was assigned to most of the model area.

Observed Versus Calculated Hydraulic Head Distribution--In order to calibrate the regional model, an even distribution of monitoring stations for ground-water level measurements throughout the domain was desired (Figure 3-8B). A complete survey of ground-water level measurements was conducted on May 21, 1990, after PW-5 was shut-down for 3 days. The field-measured hydraulic heads (accurate to ± 0.01 foot) are shown in Figure 3-16. Also included in Figure 3-16 is the calibrated, model-synthesized hydraulic head distribution displayed in 5-foot contour intervals. Measured and calculated hydraulic heads from observation stations located within the near-field region of PW-5 are shown in greater detail in Figure 3-17. The comparison between measured and calculated heads show reasonably good agreement in the vicinity of PW-5. Larger discrepancies occur in areas close to the domain boundary where the fixed-head boundary condition strongly constrained the model results. Despite these deficiencies, the model produced an acceptable simulation of the regional-scale ground-water flow regime, particularly in the area of greatest interest, PW-5.

Model Results--Upon calibration, FLOWPATH was used to delineate the capture zone for PW-5. The model was run with a pumping rate of 650 gpm at PW-5 to determine the hydraulic head distribution under long-term, steady-state pumping conditions. For the determination of the capture zone, the ground-water travel-time criterion was employed. The time-related criterion defines the WHPA as the aquifer volume through which ground water moves toward the well, arriving at the well within a specified time. The affected aquifer volume is called the zone of contribution (ZOC) and is also referred to as the time-related capture zone.

A second, less sophisticated criterion for WHPA delineation is drawdown. For the hydrogeologic setting of the study area, model test results from FLOWPATH indicate that the drawdown criterion produced inappropriate protection results (Figure 3-18). In other words, the ZOI, which is uncorrected for the regional gradient effect, should not be used to define the WHPA in a setting with a significant regional ground-water gradient because the area will be overprotective in the downgradient direction. The regional gradient should be subtracted from the ZOI to generate the ZOC, which should serve as the basis for the WHPA (U.S. EPA, 1987). For these reasons, and because of the more quantitative nature of the time-related criterion such as the ability to better predict contaminant transport rates, the time-related criterion is generally preferred over the drawdown criteria for delineating a WHPA.

Time-related capture zones for various intervals (50, 100, 150, 200, 300, and 400 days) are shown in Figure 3-19. The capture zones are narrow and preferentially extend toward the south. The capture zones also extend to the north across the pond, which indicates ground-water recharge from the northern portion of the aquifer to the northwest from Spectacle Pond. The effect of surface-water influences on the capture zone has major implications for wellhead protection strategies. Any pollutant entering Spectacle Pond by way of the surface water or ground water could potentially contaminate the well. Therefore, the entire Spectacle Pond watershed requires some degree of protection, with greater attention focused on the capture zone for PW-5.

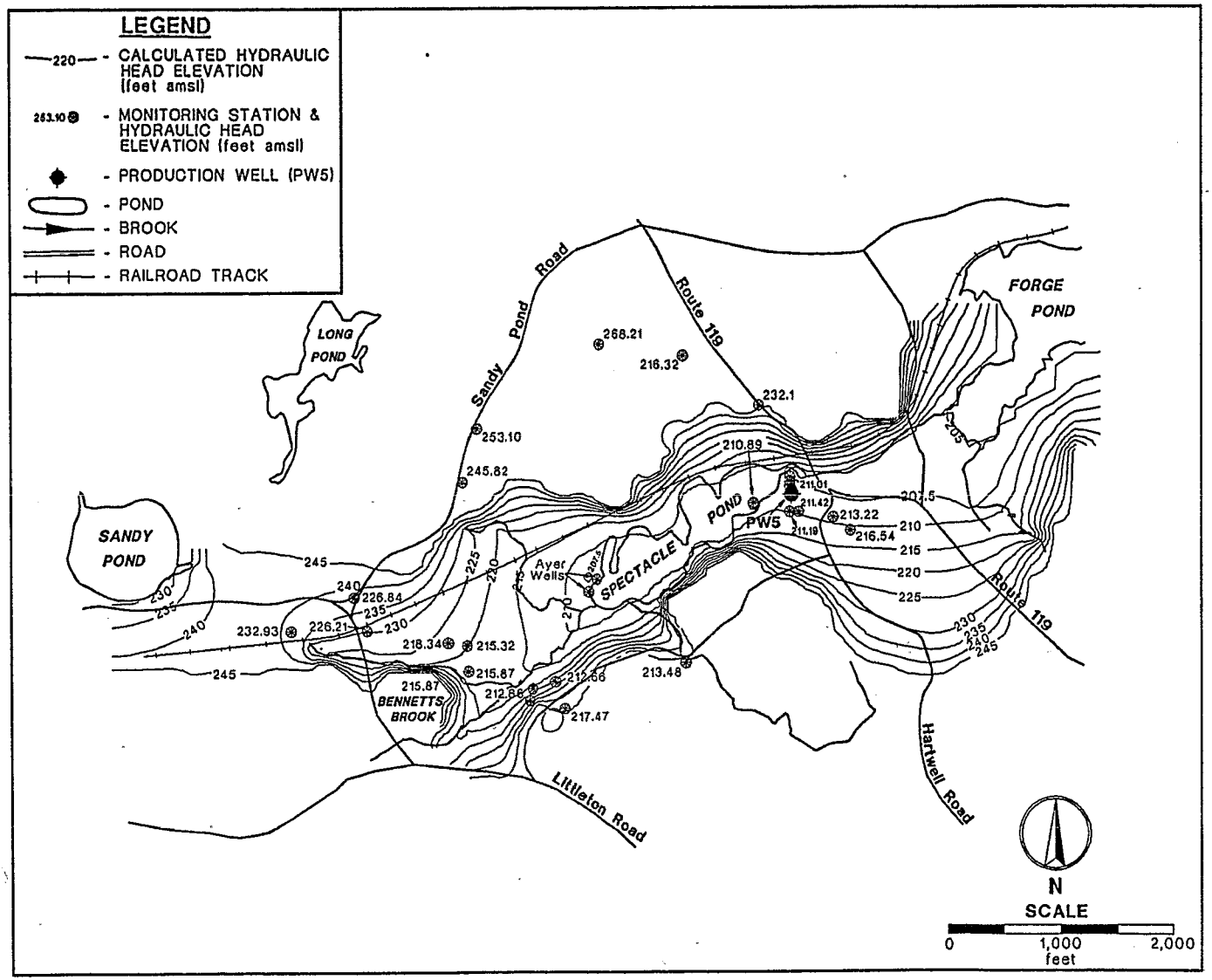

Figure 3-16. Regional hydraulic head elevations for the Spectacle Pond aquifer, Bennetts Brook watershed, Littleton, Massachusetts.

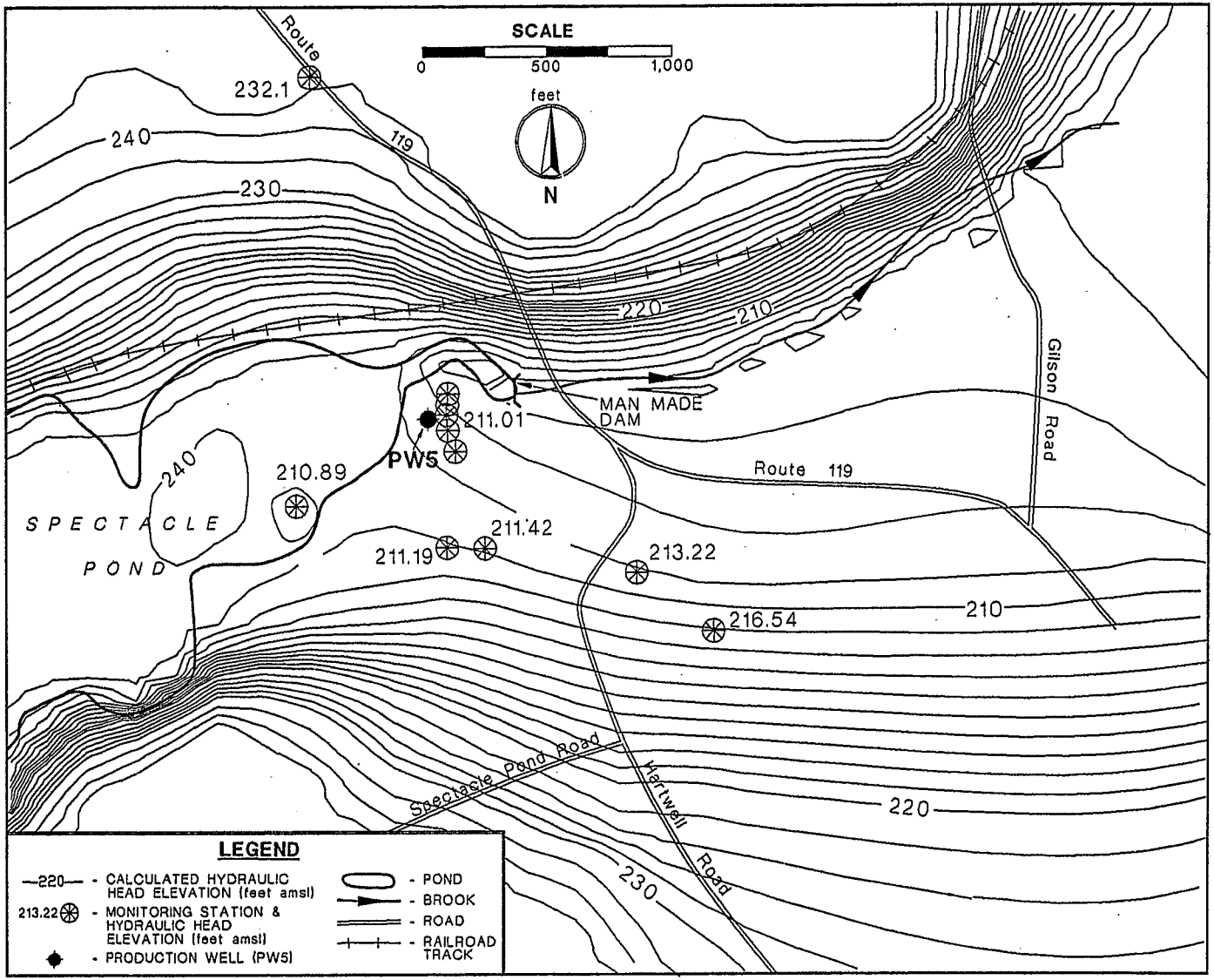

Figure 3-17. Hydraulic head elevations in the vicinity of PW-5, Bennetts Brook watershed, Littleton, Massachusetts.

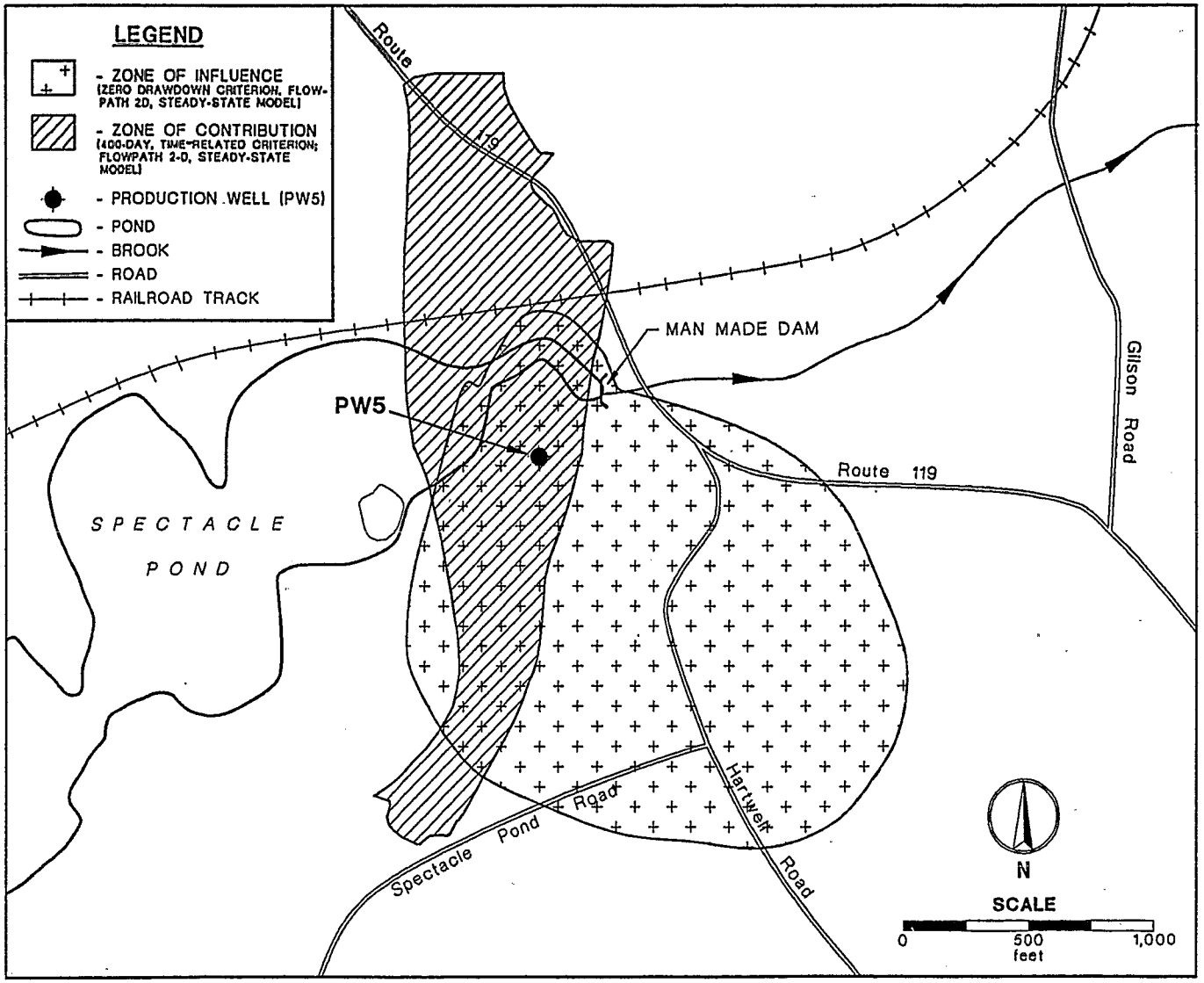

Figure 3-18. Comparison of the zone of contribution and zone of influence for PW-5 in the presence of a regional hydraulic gradient, Bennetts Brook watershed, Littleton, Massachusetts.

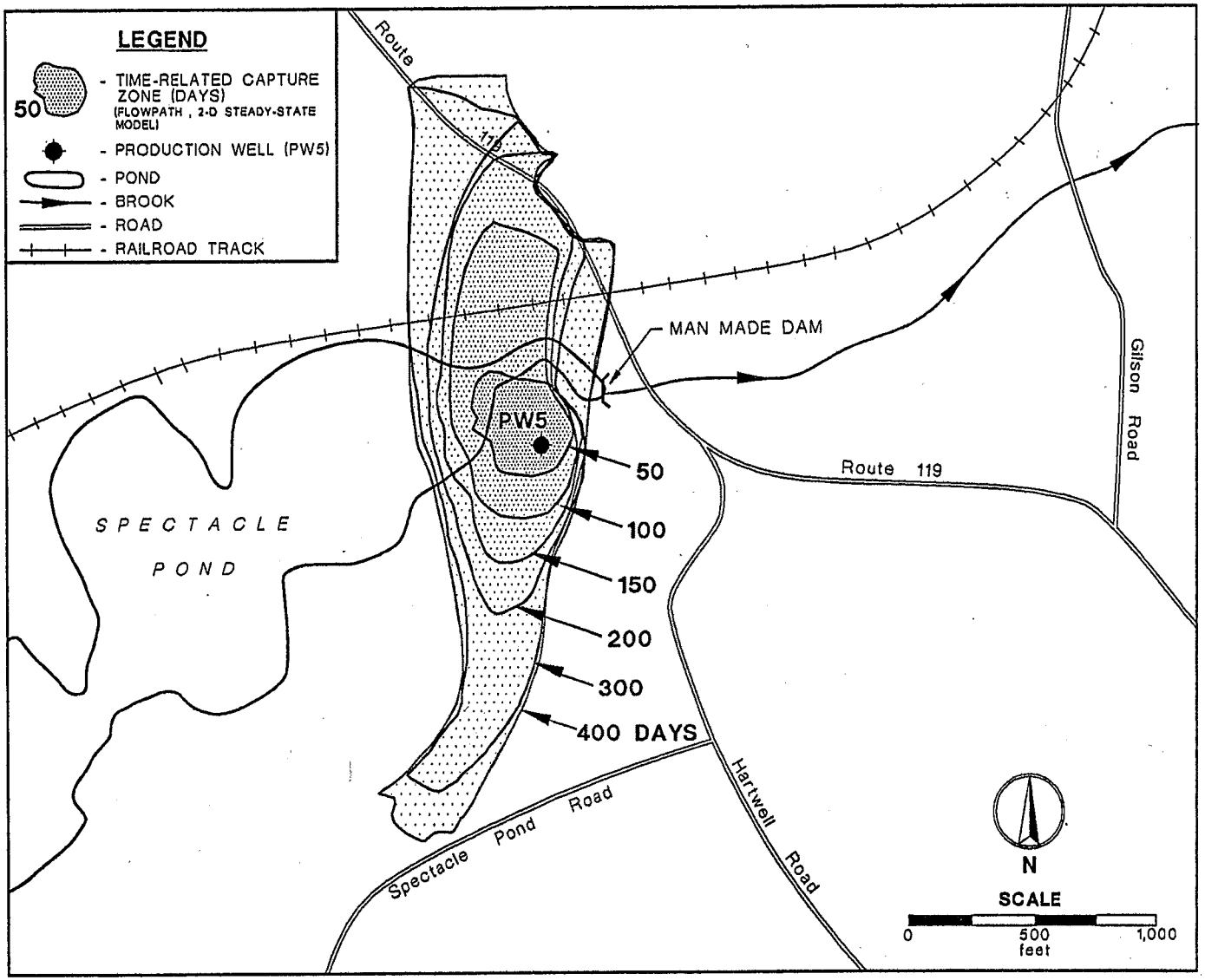

Figure 3-19. Time-related capture zones determined by two-dimensional modeling in the vicinity of PW-5, Bennetts Brook watershed, Littleton, Massachusetts.

The quantitative relationship between the well discharge and the pond leakage is addressed in the 3-D modeling study. The time values shown on the capture zones indicate the approximate time, in days, required for fluid particles located on the time contours to reach PW-5. The area of the capture zone did not increase for times greater than 400 days. This condition indicates that the amount of water withdrawn by the production well and the amount of recharge (precipitation plus leakage) occurring within the 400-day contour are in equilibrium. The 400-day capture zone, therefore, represents steady-state conditions for a discharge rate of 650 gpm at PW-5. The recharge zone would increase if the well were pumped at a higher rate.

Model Sensitivity-- A sensitivity analysis was performed to determine the effects of pond leakage, ground-water recharge, and aquifer porosity on the delineation of the WHPA for Spectacle Pond. The purpose of this analysis was to investigate values of these parameters within their uncertainty ranges. The leakage factor is given by the ratio of hydraulic conductivity and thickness of the pond bed. An increase in the leakage factor in the pond resulted in a decrease in hydraulic head in the pond area. For very high leakage factors, the ground-water level in the aquifer around the pond approached the pond water-level elevation. For low leakage factors, which correspond to low hydraulic conductivities of the pond bed, the hydraulic heads increased. This behavior indicates that the pond is recharged by ground water flowing from the topographically-higher areas through the pond bottom. Greater recharge occurs in areas of higher hydraulic conductivity, where a good connection between the ground water and the pond exists. For increased leakage factors, the production well derives more water from the pond and the upgradient areas located to the south, adjacent to the pond. For reduced leakage factors, the well derives most of its water from the south and a reduced amount from the pond.

Increased ground-water recharge causes a general reduction in the size of the time-related capture curves. The recharge area extends to the aquifer boundary, as in the original calibrated simulation. Ground water and contaminants originating at the boundary, however, reach the well within a shorter period of time. A reduction of the recharge rate has the opposite effect.

Porosity affects the magnitude of the average linear ground-water velocities, but not their directions. Travel times, therefore, decrease or increase for decreasing or increasing porosities, respectively. The shape of the time-related capture curves with the adjusted time levels remains the same. For example, the 200-day contour for the simulation shown in Figure 3-19, with a porosity of 30%, would change to a 100-day contour for a porosity of 15%.

Implications of the Regional Approach--To some degree, the limitations of a 2-D model will bias the size and shape of the predicted WHPA. The 2-D formulation neglects vertical flow gradients; therefore, it appropriately simulates regional-scale systems in which flow occurs predominantly in the horizontal plane. Three-dimensional flow conditions with significant vertical components exist in the immediate vicinity of partially-penetrating wells and shallow surface-water bodies and in strongly variable topography.

A 2-D horizontal model cannot resolve vertical layering within the aquifer. Vertical variations in aquifer parameters such as hydraulic conductivity and porosity are averaged over the aquifer thickness. In addition, the withdrawal rates of partially-penetrating wells are evenly distributed over

the entire aquifer thickness. This results in lower stress on the aquifer over a larger area than the partially-penetrating well would exert. Withdrawal from a partially penetrating well may exert very high stress over a shorter screen length and very small stress over the rest of the well bore.

These conditions all apply to PW-5. The Bennetts Brook watershed has variable topography, and the partially-penetrating production well, PW-5, is located close to a shallow pond with a semi-pervious peat layer at the pond bottom. The fine grid and the relatively extensive data base of the 2-D model suggest that the regional and local flow systems were described appropriately by FLOWPATH. For this study, a comparison of the 2-D versus the 3-D model simulations is useful to address all of the limitations of the 2-D simulation.

Three-Dimensional, Steady-State, Local Flow Model--

Modeling Purpose--One of the essential assumptions inherent in the use of the 2-D model is that the predominant direction of flow in the aquifer is horizontal. For most thin stratified-drift aquifers, this is generally true except in recharge or discharge areas. In the vicinity of the partially-penetrating PW-5, vertical flow will be greatest near the leaky stream bed of Spectacle Pond. A 3-D flow model of this section of the aquifer was constructed (1) to obtain better definition of flow patterns beneath and around the pond, and (2) to determine the effects of the partial well penetration and of vertical anisotropy. Another model, which is basically a post-processor of the flow model, was used to determine pathlines and times of travel. The numerical model used to simulate 3-D flow of ground water in this area is MODFLOW (McDonald and Harbaugh, 1988).

Model Description--MODFLOW utilizes a finite-difference numerical method to solve the 3-D flow equations. In addition to the options available in the 2-D model, the 3-D code allows simulation of vertical flow components between adjacent aquifer units. The units may or may not be separated by confining layers. Furthermore, the model simulates leakage to or from the aquifer through a leaky stream bed in any model layer. Aquifer layers may be confined, unconfined, or a combination of each, and recharge may be applied to any model layer. The post-processing of the hydraulic heads simulated by MODFLOW was completed with MODPATH (Pollock, 1989).

Model Exercise--The 3-D model for the area in the vicinity of the production well is based on the same grid used in the 2-D model. The 3-D grid uses essentially the same distribution of hydraulic properties, recharge, and discharge, except that it is divided vertically into four layers. The finite difference grid used in the 3-D simulation is shown in Figure 3-20. Layer 1 (top layer) of the model has the same areal extent as the near-field, 2-D model; however, the remaining layers are of lesser areal extent to more accurately simulate the bedrock topography (Figure 3-21). The horizontal grid spacing is approximately 300 feet, except near the well where it is 50 feet. The vertical grid spacings of 45, 15, 15, and 30 feet correspond to Layers 1 through 4, respectively (Figure 3-20).

Layer 1 in the model is simulated for unconfined conditions. Transmissivity is computed as a function of saturated thickness and hydraulic conductivity after each model iteration. Nodes where the leaky stream bed is simulated are also located in the top layer. Constant-head boundary conditions were assigned for all four sides in the top layer, using hydraulic heads generated by the steady-state 2-D simulations. The boundary conditions for the bottom three layers of the model are simulated as constant-head where there is no bedrock intercepting the layer, and otherwise, as no-flux boundaries. The bottom three layers are interactive with the layers immediately above or below.

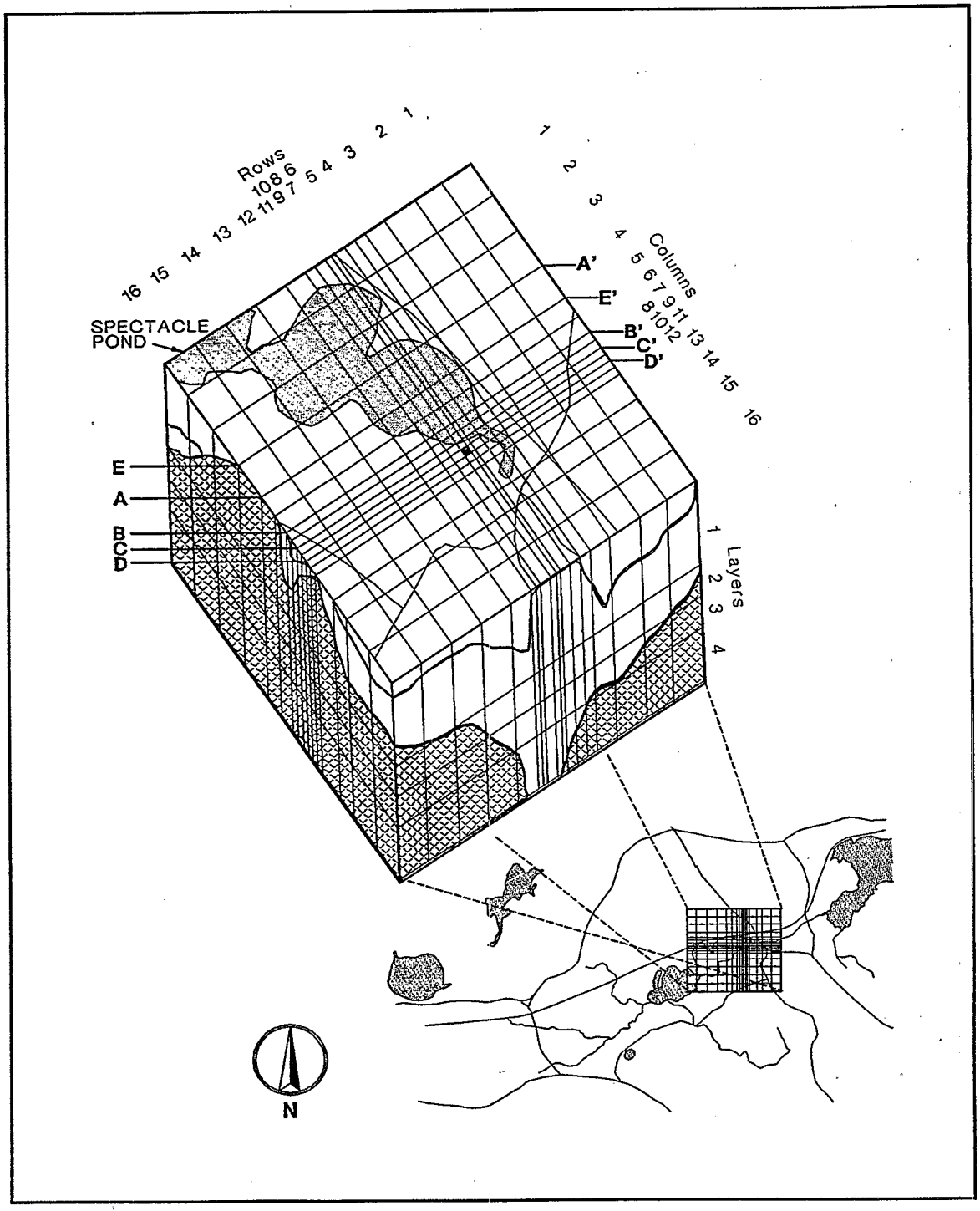

Figure 3-20. Three-dimensional model boundary, grid domain, and cross section locations in the vicinity of PW-5, Bennetts Brook watershed, Littleton, Massachusetts.

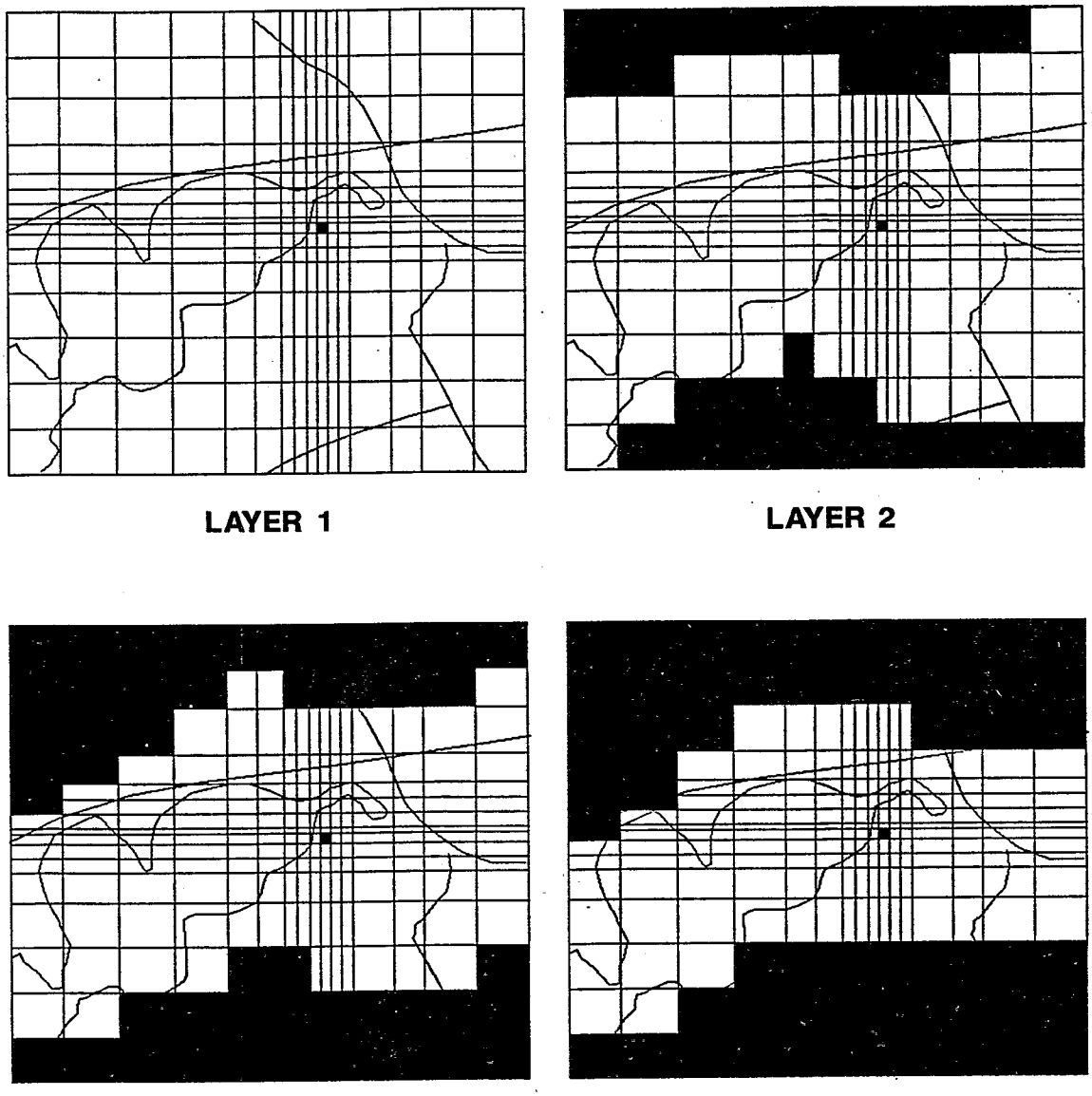

Figure 3-21. Multi-layer grid for the three-dimensional model in the vicinity of PW-5 (small black square), Bennetts Brook watershed, Littleton, Massachusetts. (Large black areas represent bedrock)

Horizontal hydraulic conductivities for the 3-D model are the same as those used in the 2-D model. The vertical hydraulic conductivity was determined as 1/35 of the horizontal values (the average ratio obtained from long-term aquifer test). A stratigraphic section was drawn for each model column to define material properties. Four examples of stratigraphic sections with the finite difference grid are shown in Figure 3-12. In finite difference blocks where two different lithologic materials were present (for example, aquifer and peat or aquifer and bedrock) the harmonic mean of the respective hydraulic conductivities was used. Recharge to the model was applied to Layer 1 in the same proportions as that used for the 2-D simulation. The production well is present in Layer 3 (Figure 3-20), thus simulating the 15-foot screen length located between the depths of 35 and 50 feet.

Model Results--As in the 2-D modeling exercise, simulations were run to steady-state conditions so that the maximum effects of pumping could be observed. At steady state, the sources of water to the production well are (1) induced infiltration from the pond, and (2) captured ground-water discharge from areal precipitation and runoff from upland till and bedrock outcrops.

Simulated hydraulic head distributions under pumping conditions at a rate of 650 gpm are shown in Figures 3-22 and 3-23. Cross section E-E′ (Figure 3-22) indicates that the pond is receiving ground water from the uplands through the bottom and is, therefore, effluent in areas far from the well. In contrast, cross section D-D′ indicates that in areas adjacent to the well, part of the production water is withdrawn from Spectacle Pond. Both cross sections in Figure 3-22 show downward vertical gradients from the valley uplands toward the center of the flow system.

Hydraulic head distributions for the four layers in the model are presented in Figure 3-23 in the plan view. Maximum drawdown occurs in Layer 3, the layer in which the well is screened. The horizontal, 2-D simulation represents an average of the heads in all layers; therefore, no direct comparison is possible. However, the layer that best matches the 2-D model simulation is Layer 2.

The surface-water contribution to the discharge of PW-5 is determined by the hydraulic conductivity and saturated thickness of the peat layer, and by the horizontal versus the vertical hydraulic conductivities of the aquifer. For average conditions, a water-budget calculation from the model indicates that 19% of the production water (approximately 120 gpm of the 650 gpm total) is induced from surface water. The value of 19% is very near the 20% to 25% range estimated from the aquifer-test analysis.

On the basis of the 3-D head distribution data, time-related capture zones were calculated for 50, 100, 200, and 400 days. The 400-day capture zone is equal to the steady-state capture zone. In 3-D, the outline of a given capture zone represents an irregularly-shaped surface. The intercept of the computed surface with the four model layers is shown in Figure 3-24. In all layers, the capture zones extend to the aquifer limits, both north and south. In Layer 3, where the well screen is completed, there is a stronger horizontal flow component than in the other layers. Therefore, the capture zone in Layer 3 extends farther to the east and west. This east-west extension is not as pronounced in Layer 1 because of the predominant vertical flow components close to the pond. The capture zone in Layer 1 represents the surficial expression of the area that should be prioritized for protection against potential contamination threats. The capture zones generated for the other layers will prove useful in designing multi-level monitoring wells for early-warning detection of ground-water contamination.

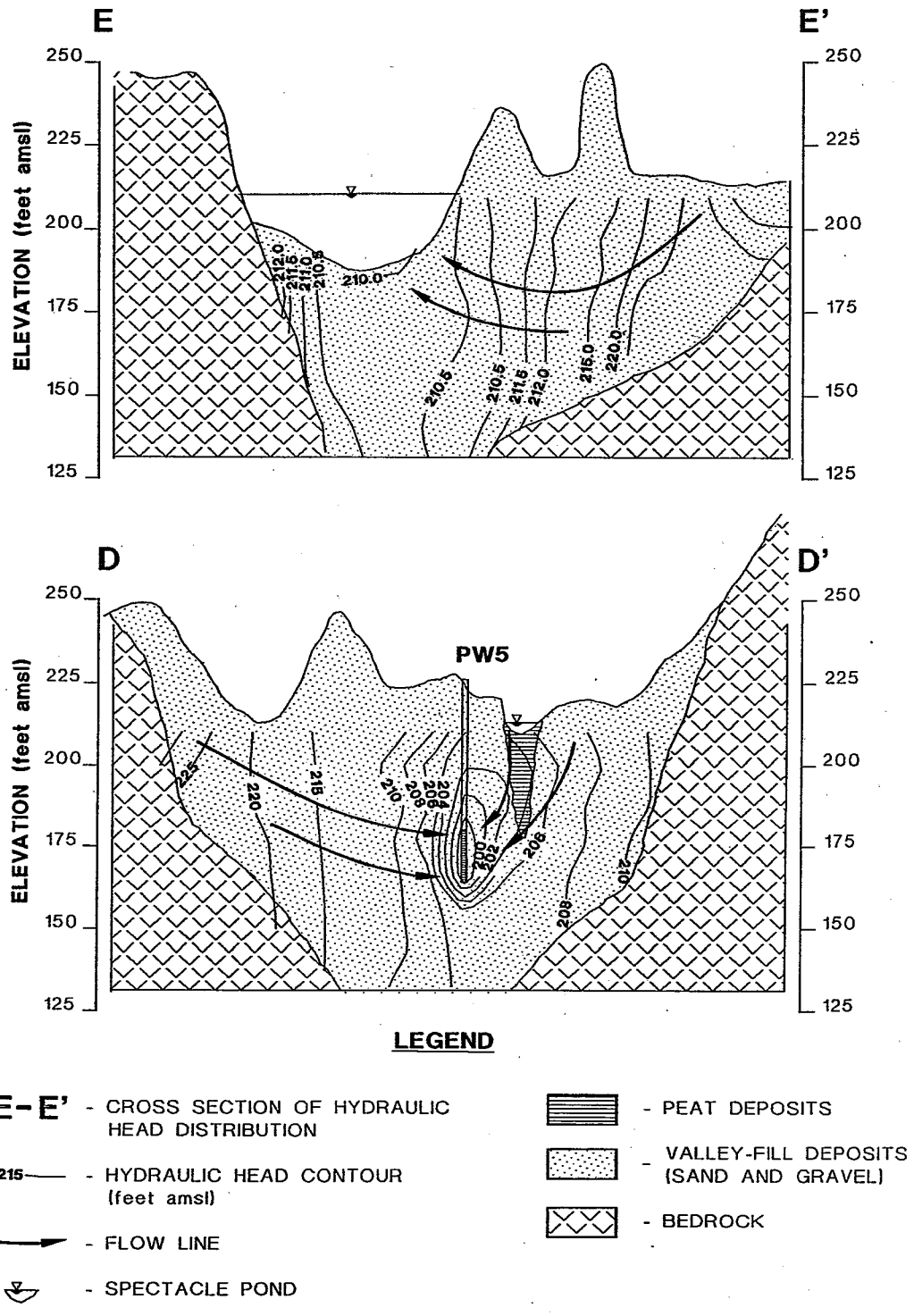

Figure 3-22. Cross sections of hydraulic head distribution, E-E′ and D-D′, determined by MODFLOW and MODPATH. (See Figure 3-20 for cross section locations relative to Spectacle Pond, Littleton, Massachusetts)

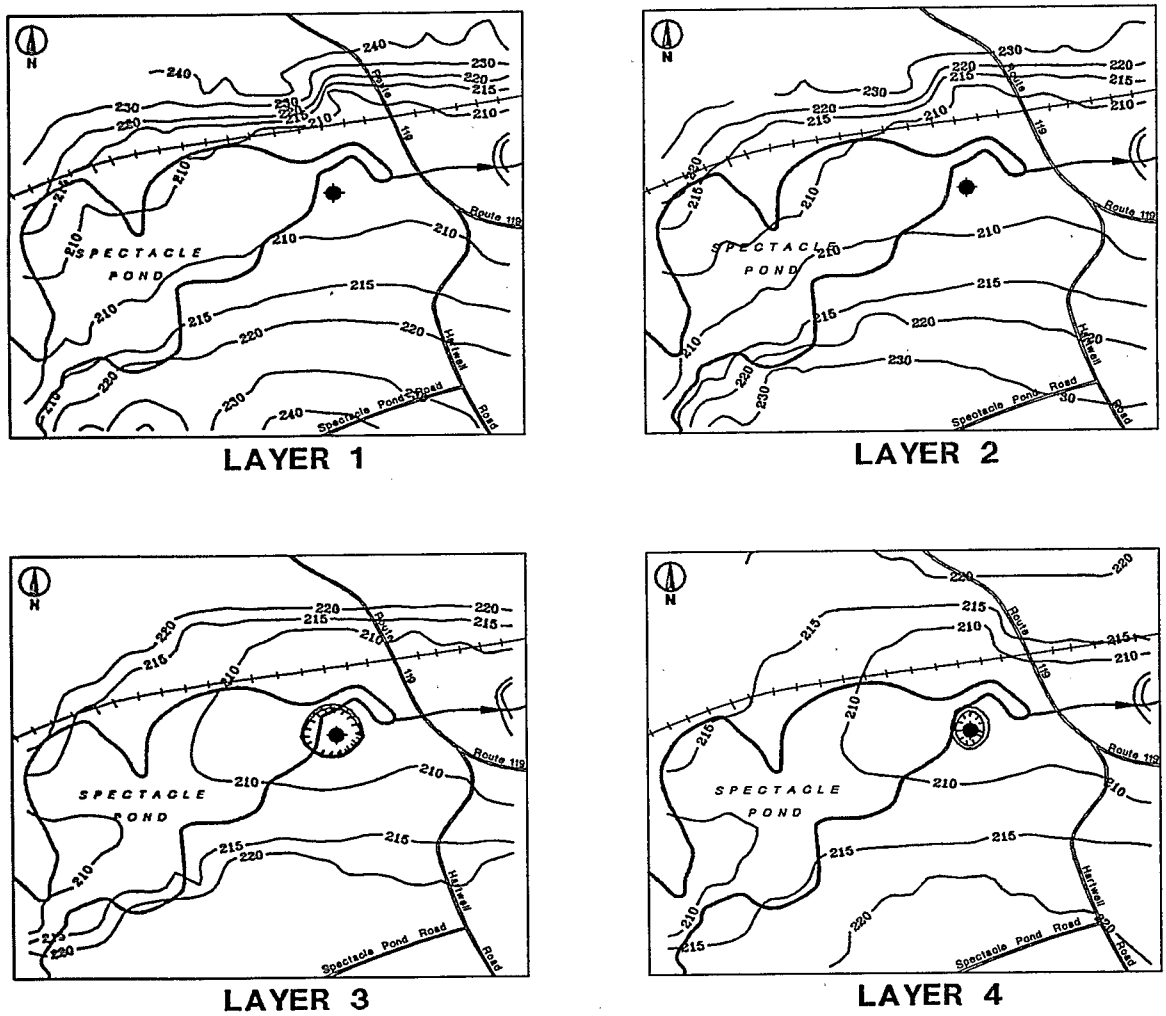

Figure 3-23. Plan view of hydraulic head distributions for each layer of the three-dimensional model, Bennetts Brook watershed, Littleton, Massachusetts.

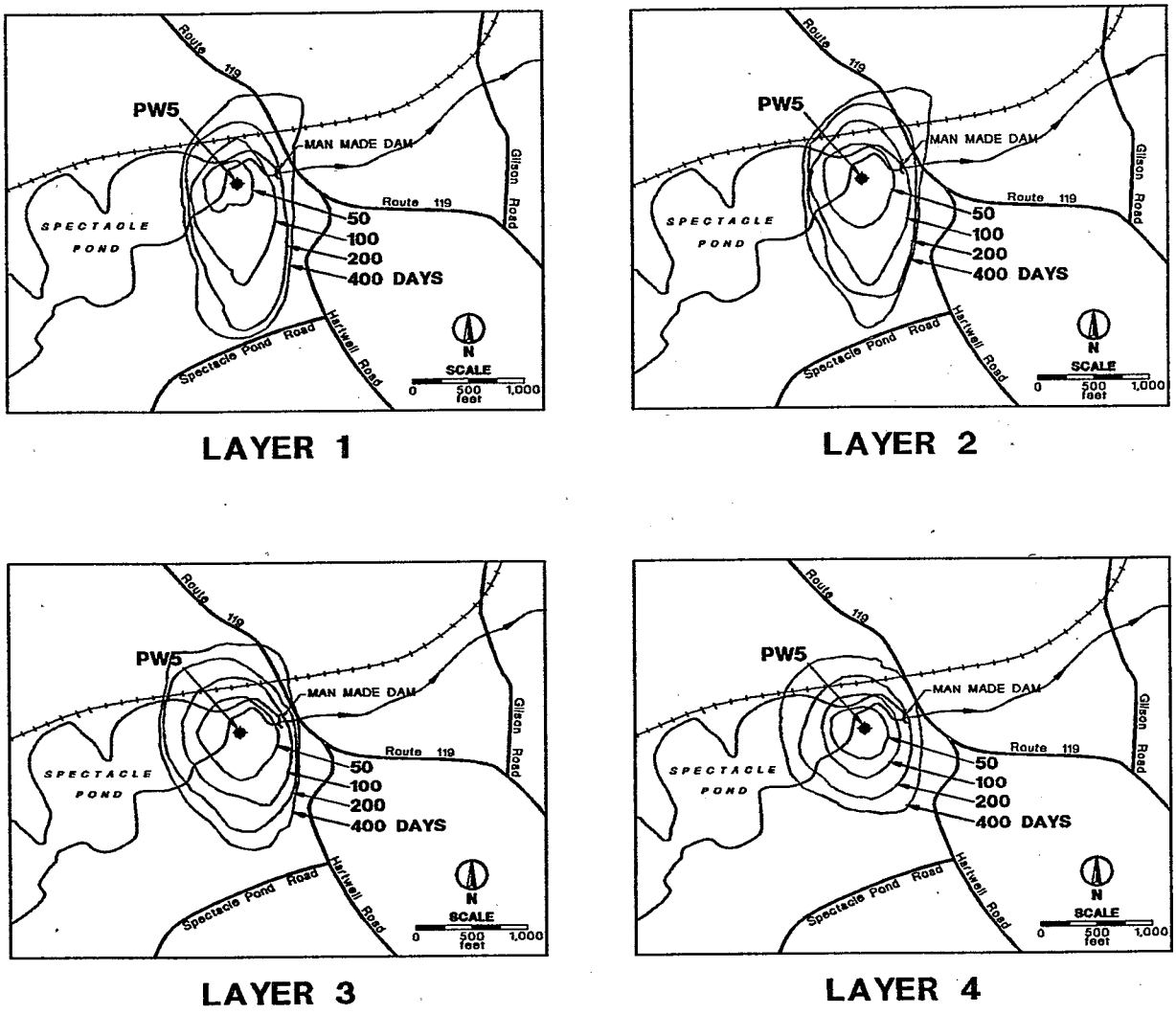

Figure 3-24. Time-related capture zones for PW-5 for each layer of the three-dimensional model, Bennetts Brook watershed, Littleton, Massachusetts.

Two-Dimensional, Transient, Regional Flow Model--

Modeling Purpose--The 2-D, transient model, FLOWCAD (Franz and Guiguer, 1992), was used to delineate capture zones for extreme hydrologic conditions given by the Massachusetts Department of Environmental Protection (1991). Under the DEP criteria, the hydrogeologic simulation is controlled by stressed recharge conditions (no precipitation) for a specific time period (180 days). These delineation criteria result in conservative capture zones and associated Water Resource Protection Areas. Zone I is set by the State at a fixed radius of 400 feet; Zone II is determined using the following procedure:

1. Construct a water-table contour map representative of the predeveloped, long-term average conditions in the aquifer.

2. Predict drawdowns by imposing Zone II criteria (180 days of pumping at safe yield rates with no recharge from precipitation), using an appropriate analytical or numerical model.

3. Determine the Zone II water-table contours by subtracting the predicted drawdowns from the long-term average water-table contours.

4. Construct a flow net based on the resulting Zone II water-table contours.

5. Identify the ground-water divide, induced by pumping, which separates the area of contributing water to the well (Zone II) for the aquifer outside of Zone II.

6. Extend the ground-water divide upgradient to its point of intersection with prevailing hydrogeologic boundaries.

7. Delineate Zone II for the production well as the area determined by this procedure.

Zone II was delineated for PW-5 using FLOWCAD to meet DEP criteria. The results of the transient-state modeling are compared to the steady-state, 2-D modeling and the 3-D, near-field model in the following section, titled, "Comparison of Model Results."

Model Description--FLOWCAD is an extension of FLOWPATH (2-D, numerical model), except that it performs transient-state modeling. Specifically, FLOWCAD simulates time-related responses of the hydraulic head distribution to recharge and discharge stresses.

Model Exercise--The calibrated hydraulic heads simulated with FLOWPATH, representing average recharge conditions under nonpumping conditions, were used to construct the regional water-table contour map (Procedure 1). FLOWCAD was used to predict production well drawdowns (Procedure 2). The same model grid, bedrock elevations, boundary conditions, and aquifer property distributions used in the steady-state, regional FLOWPATH simulation were used for the FLOWCAD simulation. All model assumptions and limitations discussed in the regional FLOWPATH simulation apply to the transient exercise.

The regional water-table contour elevations previously determined were used for the initial hydraulic head distribution. The production well was then pumped at a rate of 650 gpm, and recharge from precipitation was assigned a zero value. The model simulated 180 days of continuous pumping, and the resulting hydraulic head distribution was used to predict the Zone II water-table contour. Using this hydraulic head distribution, a pathline analysis was performed to determine the ZOC for PW-5.

Model Results--The resulting capture zone is shown in Figure 3-25. Because the capture zone is controlled primarily by the limited recharge condition rather than the time-related constraint, it extends to the hydrogeologic boundaries, thus satisfying the Zone II criteria outlined earlier (Procedure 6). While hydrologic boundary conditions were reached to the north and south (bedrock and till), and to the west (Spectacle Pond), the transient-state capture zone expands toward the east and west to compensate for the loss of recharge from precipitation. Generally, as recharge from precipitation decreases, the areal extent of the ZOC increases, if hydrogeologic boundaries are not encountered.

Comparison of Model Results--

A graphical comparison of the steady-state 2-D (regional scale), steady-state 3-D (local scale), and transient 2-D (regional scale) capture zones is shown in Figure 3-26. The variations in size and shape of the capture zones are directly related to the assumptions and limitations of the initial model. Spatial factors in model design are best exemplified by comparing the 2-D and 3-D steady-state solutions. Effects of temporal and recharge conditions are best represented when comparing the 2-D steady-state results with the 2-D transient results. Depending on the predetermined modeling and wellhead protection objectives, each can be used effectively to delineate a WHPA.

The 2-D and 3-D steady-state models generated slightly different capture zones. Compared to the 3-D model, the 2-D solution underestimates the eastern and western directions and overestimates the contributing areas to the north and south. The reason for these differences is the 3-D model, which accounts for variations in aquifer depth and the resultant vertical flow field, making generation of a more realistic 3-D ZOC possible. For the relatively thin aquifer considered in these simulations, the 3-D ZOC extends throughout the entire depth of the aquifer. The enveloping surface of the contributing area is close to vertical everywhere, unless constrained by the bedrock. In general, the 3-D ZOC is egg-shaped at depth (Figure 3-24); the deeper boundaries (Layers 3 and 4) are constrained by the bedrock valley.

Neither the 2-D steady-state nor the 2-D transient model accounts for the vertical flow field and variations in aquifer depth. Consequently, these model limitations cannot be a factor in the comparison of the simulated capture zones. Interestingly, their respective capture zones are similar in shape. The capture zone created by the transient flow model extends well beyond the downgradient (eastern) boundary of the steady-state equivalent and slightly beyond the western boundary. In both cases, the aquifer boundaries are reached to the north and south. The wider capture zone generated by the 2-D transient model underscores the influence of limited recharge during the 180-day simulation. The effects of temporal variations, steady-state versus 180 days of pumping, are not as significant as the recharge (precipitation) effect.

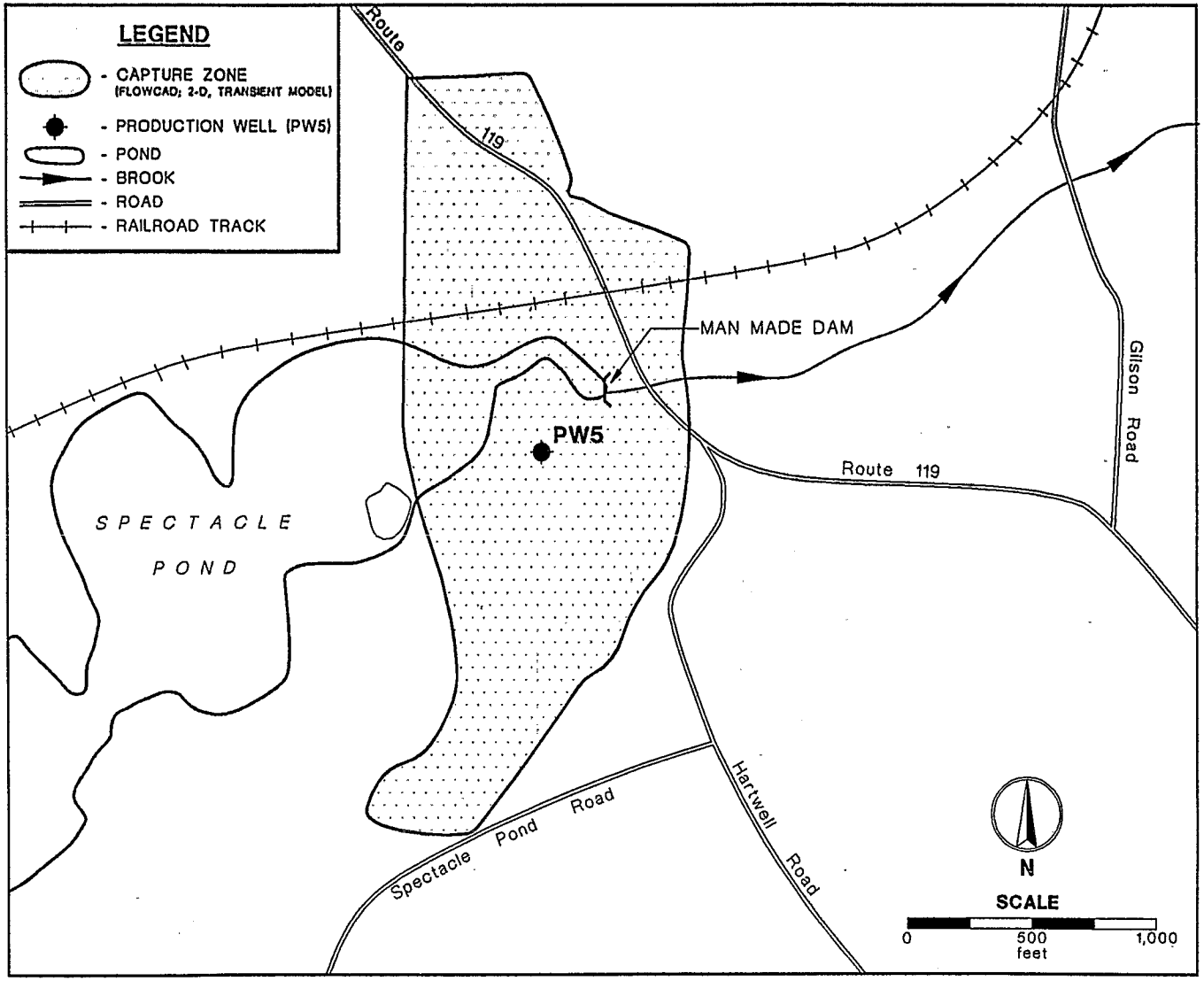

Figure 3-25. Capture zone for PW-5 determined by FLOWCAD using the 1991 Massachusetts Department of Environmental Protection (1991) delineation criteria, Bennetts Brook watershed, Littleton, Massachusetts.

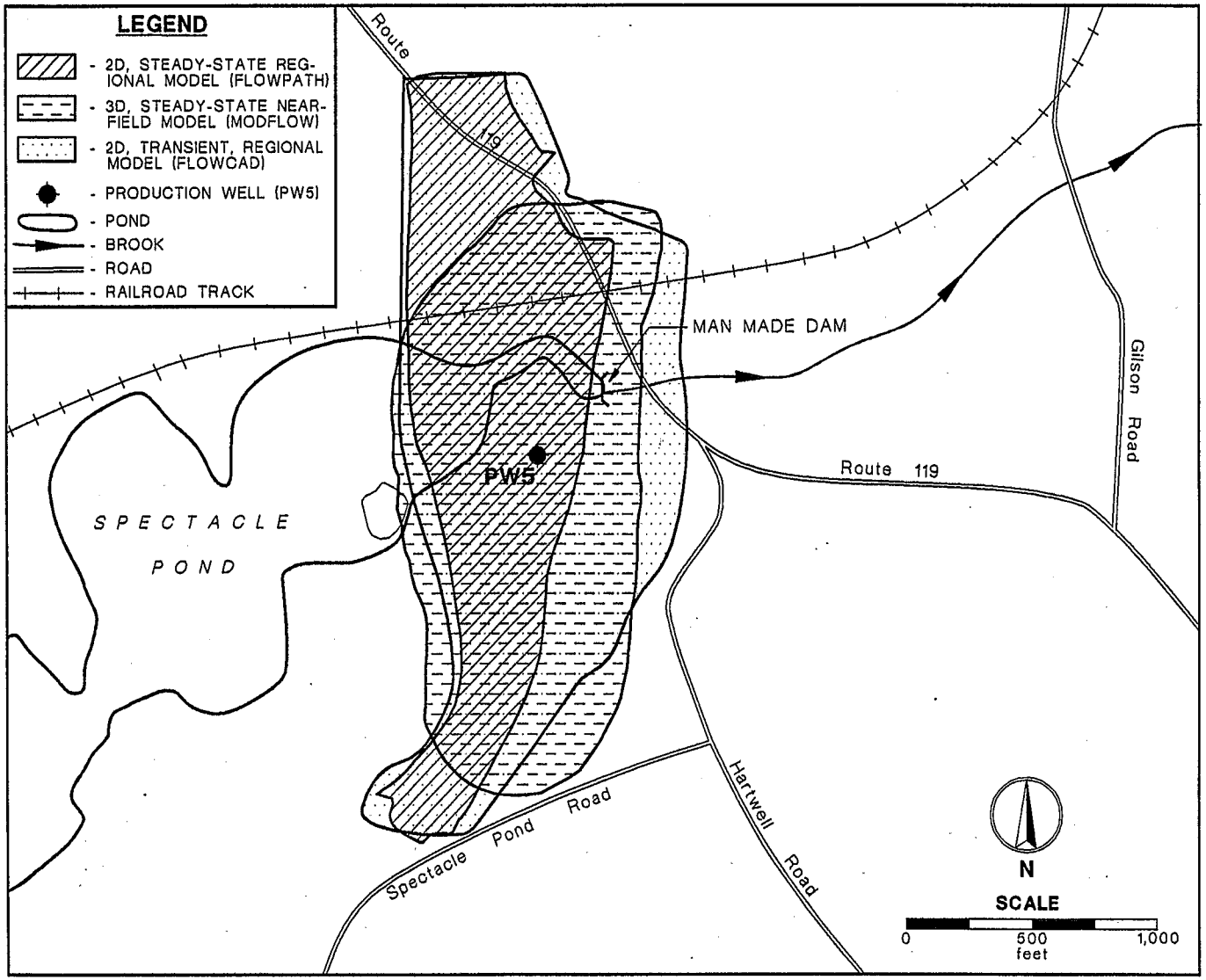

Figure 3-26. Comparison of the capture zones for PW-5 generated by the 3-D steady-state, 2-D steady-state, and 2-D transient flow models, Bennetts Brook watershed, Littleton, Massachusetts.

Water Quality Monitoring

Ambient Trend Monitoring--

The chemical composition of ground water within glaciofluvial deposits can be classified into several different water quality types depending on the relative abundance of anions and cations. Using the classification system described by Freeze and Cherry (1979), the water quality within the Bennetts Brook watershed has been characterized as *Type I Water*. This is defined as slightly acidic and very fresh [total dissolved solids (TDS) <100 mg/l], in which sodium, calcium, or magnesium are the predominant cations, and bicarbonate is the predominant anion (Table 3-4). More detailed chemical analyses may be conducted graphically using various techniques such as Piper, Stiff, or radial plotting (Hem, 1989). These analyses are created by plotting the percent milliequivalents of the major anions against the percent milliequivalents of major cations. For water quality data collected within the Spectacle Pond aquifer, trilinear Piper diagrams were used to evaluate these chemical relationships. This graphical representation helped identify two distinct hydrochemical facies that exist within the study area (Figure 3-27). A sodium-bicarbonate water type was identified above 25 feet in depth, and a calcium-bicarbonate water type was identified below 25 feet in depth.

To evaluate ambient water quality changes over the duration of the long-term aquifer test, a series of samples were collected from the near-field monitoring well network. The results of these tests indicate that the water quality at all stations, including the shallow and deep cluster wells as well as PW-5, shifted toward more saline (sodium chloride) conditions as the aquifer test progressed (Figure 3-28). Although the concentration of sodium is low, its increase in concentration is proportionately higher than the other cations measured (such as calcium and magnesium). It appears that the proportionately higher sodium concentrations observed within the surface and shallow ground waters are migrating downward within the area-of-influence of PW-5, thereby changing the water chemistry throughout the monitoring well network. This temporal trend in chemistry is supported by modeling results, which indicate a downward flow gradient within the capture zone under pumping conditions.

Source Assessment Monitoring--

Chemical Parameters--Target chemical parameters were analyzed to evaluate anthropogenic and natural impacts on water quality. Anthropogenic impacts are traced to specific land-use activities. In the Spectacle Pond study area, potential sources of heavy metals, VOCs, pesticides, herbicides, and nutrients were identified. Source-assessment monitoring locations were carefully selected based on their downgradient position with respect to existing and potential threats. If the source is believed to issue chronic releases (steadily over time with little variation in concentration), then only one sample was collected to provide an accurate representation of ground-water quality (Achinger and Shigehara, 1968). This procedure was adopted to establish baseline water quality characterization within the capture zone. As a QC measure, and to monitor possible errors incurred during the initial sampling event, the monitoring network was sampled on two separate occasions, once on August 15, 1990, and again on August 27, 1990. It is important to emphasize that for compliance monitoring or for more stringent QA/QC objectives, additional data would be required.

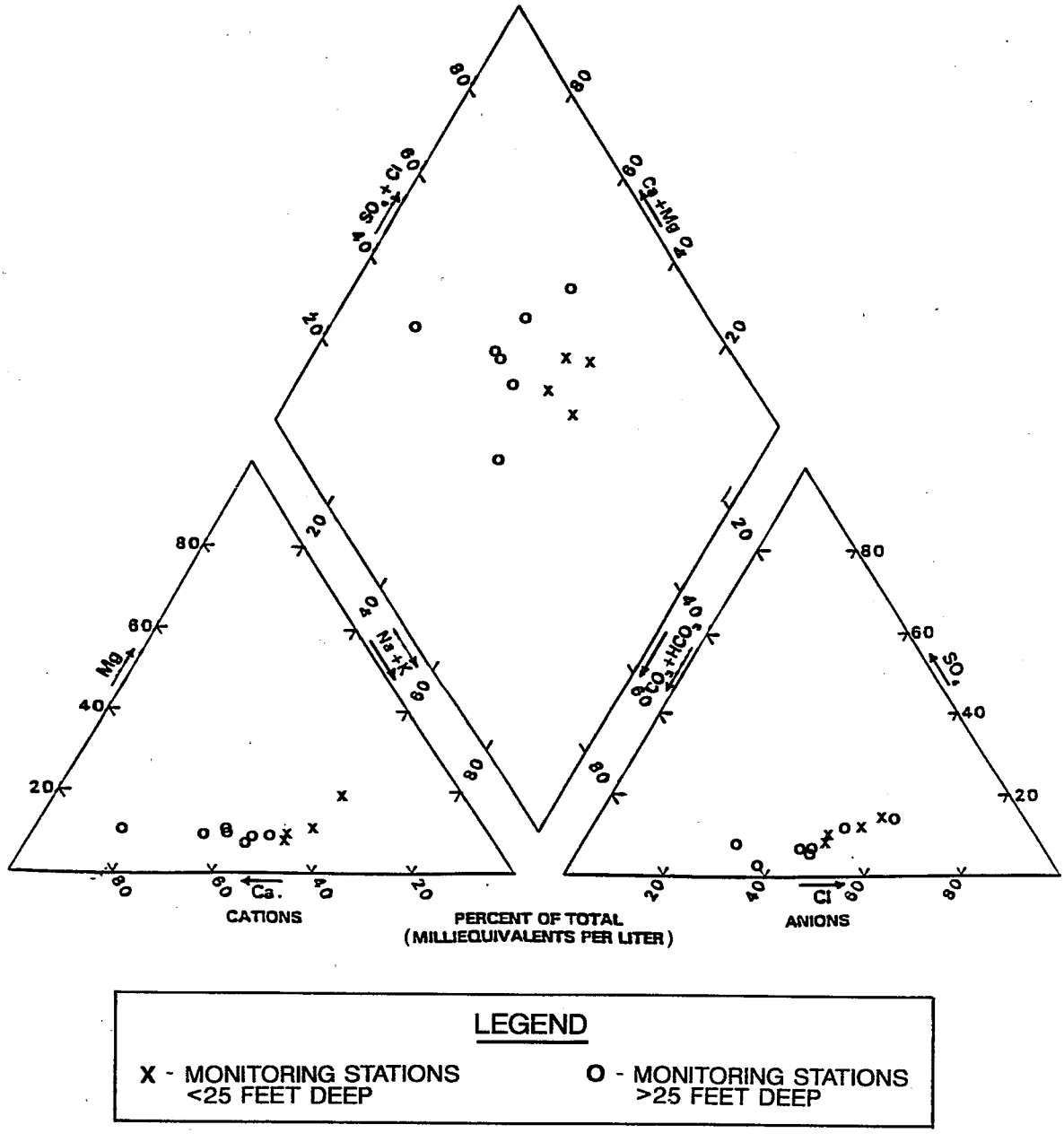

Figure 3-27. Piper diagram comparison of average water quality in shallow and deep ground-water zones near PW-5, Bennetts Brook watershed, Littleton, Massachusetts.

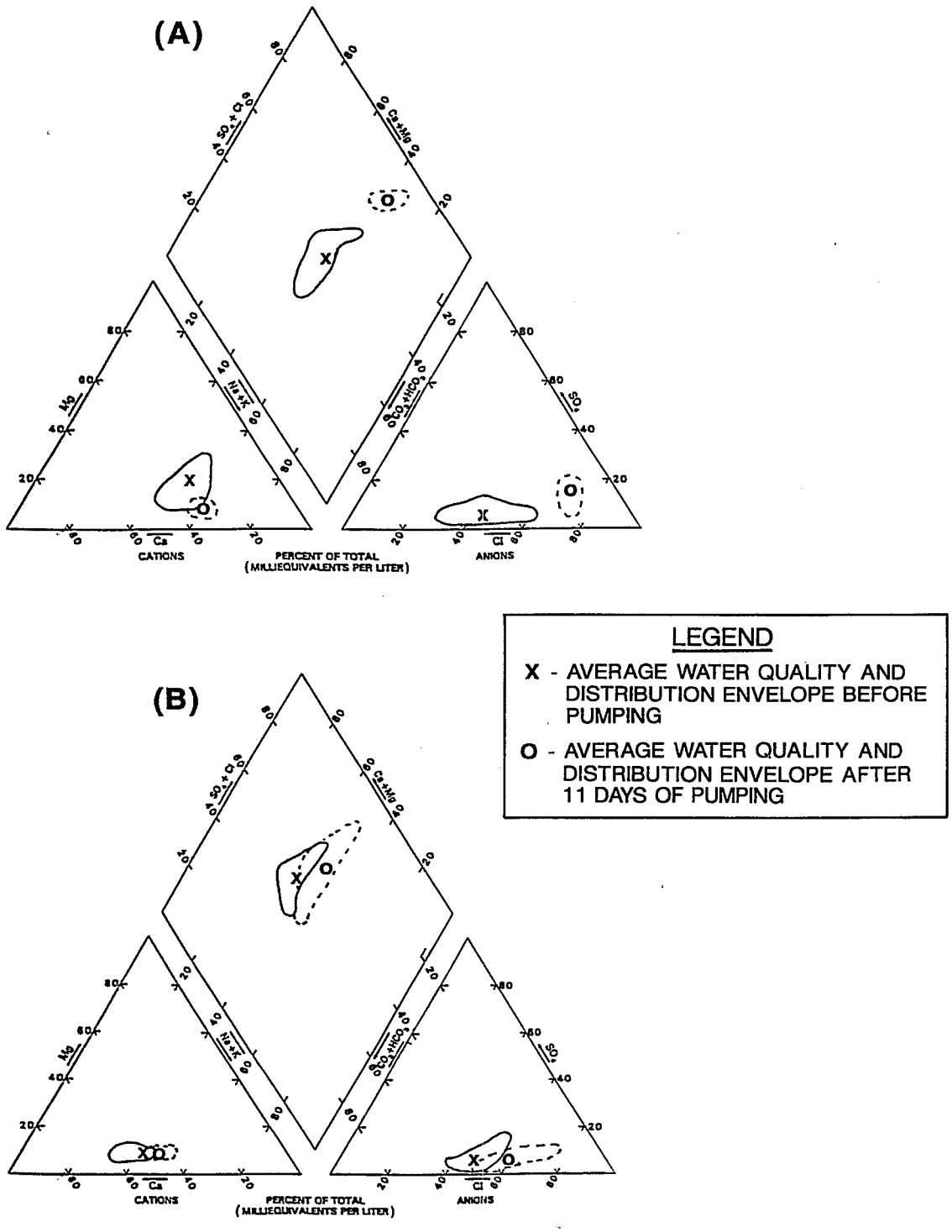

Figure 3-28. Piper diagram comparison of ambient water quality changes during the long-term aquifer test in the (A) shallow and (B) deep aquifer systems, Bennetts Brook watershed, Littleton, Massachusetts.

Results from these analyses, detailed in Table 3-4, show that, with the exception of sodium, ground-water quality within the capture zone of PW-5 is unaffected by land-use activities. The higher levels of sodium detected within the surface water and the shallow aquifer may result from roadway de-icing activities.

Naturally occurring iron and manganese have been detected at elevated concentrations within the Spectacle Pond watershed (Bray et al., 1990). Over the past several years, iron and manganese levels within the PW-5 have been gradually increasing (Figure 3-29). Locally, total iron and total manganese concentrations are elevated throughout the recharge area of PW-5 (Figures 3-30A and 3-30B, respectively). The average concentrations of iron and manganese were 2.03 mg/l (range, 0.13 to 6.9 mg/l) and 0.48 mg/l (range, 0.06 to 1.07 mg/l), respectively. These ground-water concentrations were significantly higher than the levels detected in either the surface water or PW-5 (0.18 mg/l and 0.08 mg/l for iron and manganese, respectively). This discrepancy may be caused, in part, by the high amounts of suspended clay particulates found within the monitoring well network during the sampling events. Negatively charged clay particulates can preferentially attract iron and manganese cations. Hem (1989) has suggested that colloids smaller than 0.45 micron can pass through most standard filter membranes. If the water sample contains a high percentage of these colloids, total iron and manganese analyses performed on the resulting filtrate may result in erroneously elevated values.

A temporal trend analysis, generated by plotting the concentrations of iron and manganese over the duration of the long-term aquifer test, was also performed. Other than a very slight decrease in manganese concentration, no significant trends were observed (Table 3-4).

Physical Parameters--To verify hydraulic connection between the surface-water and ground-water systems, physical parameters were measured and compared between the pond and the monitoring network. The spatial variation in temperature measurements was assessed horizontally and vertically, under nonpumping water-table conditions. Temporal changes were measured throughout the long-term aquifer test for 11 days.

On August 15, 1990, the day before the long-term aquifer test began, baseline temperatures were measured throughout the monitoring network. The overall temperature range was 52°F to 77°F. These values are typical for late-summer conditions within this region (Wetzel, 1975). The pond was slightly stratified, as indicated by a temperature of 77°F near the surface (epilimnion) and 73°F near the bottom of the pond (hypolimnion). The greatest vertical temperature gradient was observed within cluster well MW-375, where a 10°F difference was measured between the shallow well (8 feet) and the deep well (47 feet). Temperature ranges measured in wells, set at a minimum depth of 20 feet, indicate a less pronounced, lateral range of 59°F at MW-375d to 54°F at MW-800.

Automatic temperature probes installed in several of the near-field monitoring wells were used to record rapid, early-time and late-time temperature changes during the startup and shutdown stages of the long-term aquifer test. These recordings, in addition to periodic measurements taken as part of the water quality sampling events, highlighted two significant trends. First, monitoring stations located between Spectacle Pond and the production well (MW-25, MW-100, and MW-375; Figure 3-9) showed a noticeable warming trend as the aquifer test progressed. Other monitoring stations (MW-425, MW-660, and MW-800; Figure 3-9) located downgradient of PW-5 showed a less

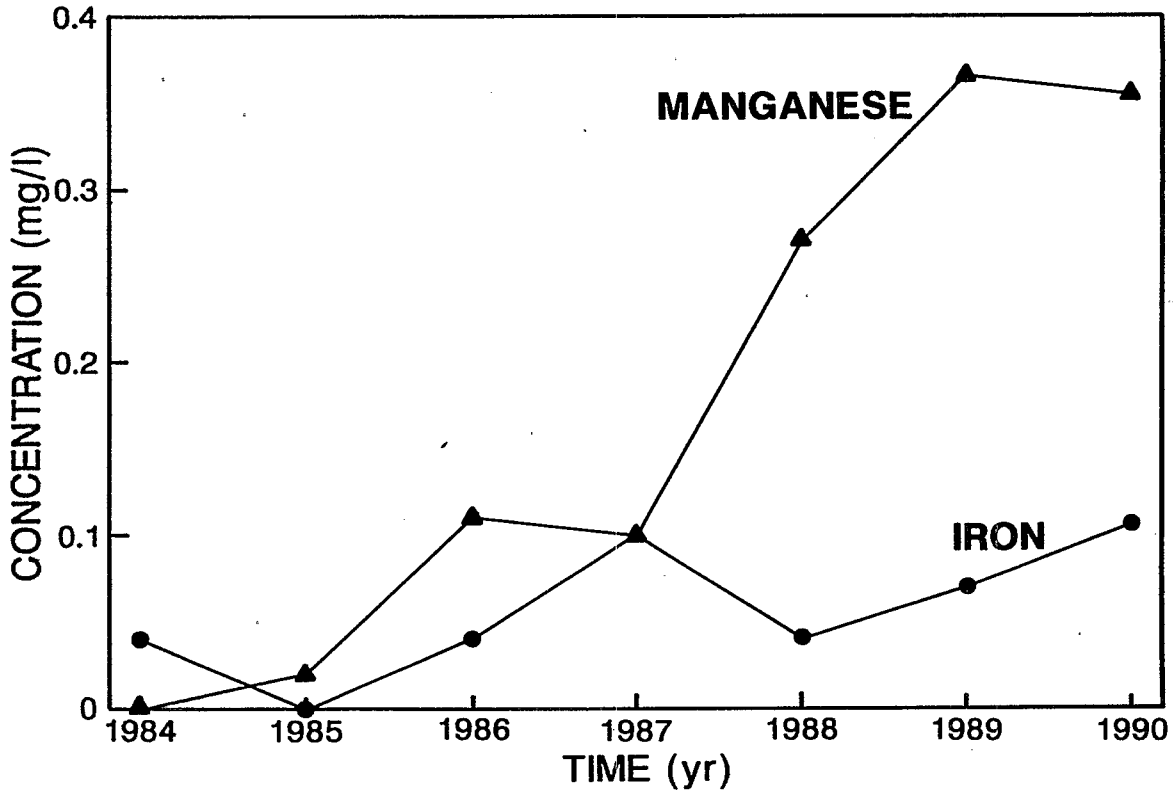

Figure 3-29. Increasing concentrations of iron (Fe) and manganese (Mn) in PW-5 from 1984 through 1990, Bennetts Brook watershed, Littleton, Massachusetts.

pronounced cooling trend (Figure 3-31). Secondly, at cluster wells where warming trends were observed, both the shallow and deep wells responded similarly to overall temperature changes (Figure 3-32).

In general, ground-water temperatures increase, both vertically and laterally, toward Spectacle Pond. As the aquifer was stressed during pumping, ground water between the pond and production well displayed a significant warming trend at all depths. These measurements demonstrate the significant hydraulic connection that exists between warmer surface water and cooler ground water within the capture zone of PW-5.

Using a technique described by Lapham (1989), quantitative estimates were made for downward flow velocities at MW-600s and -600d, MW-25s and -25d, and MW-375s and -375d. This technique establishes one relationship between vertical temperature profiles in sediment beneath a surface-water body and vertical ground-water flow. Then, a secondary relationship is established between these factors and the effective vertical hydraulic conductivity of the sediment. Using this relationship, vertical hydraulic conductivities were measured from 0.01 ft/d, under nonpumping

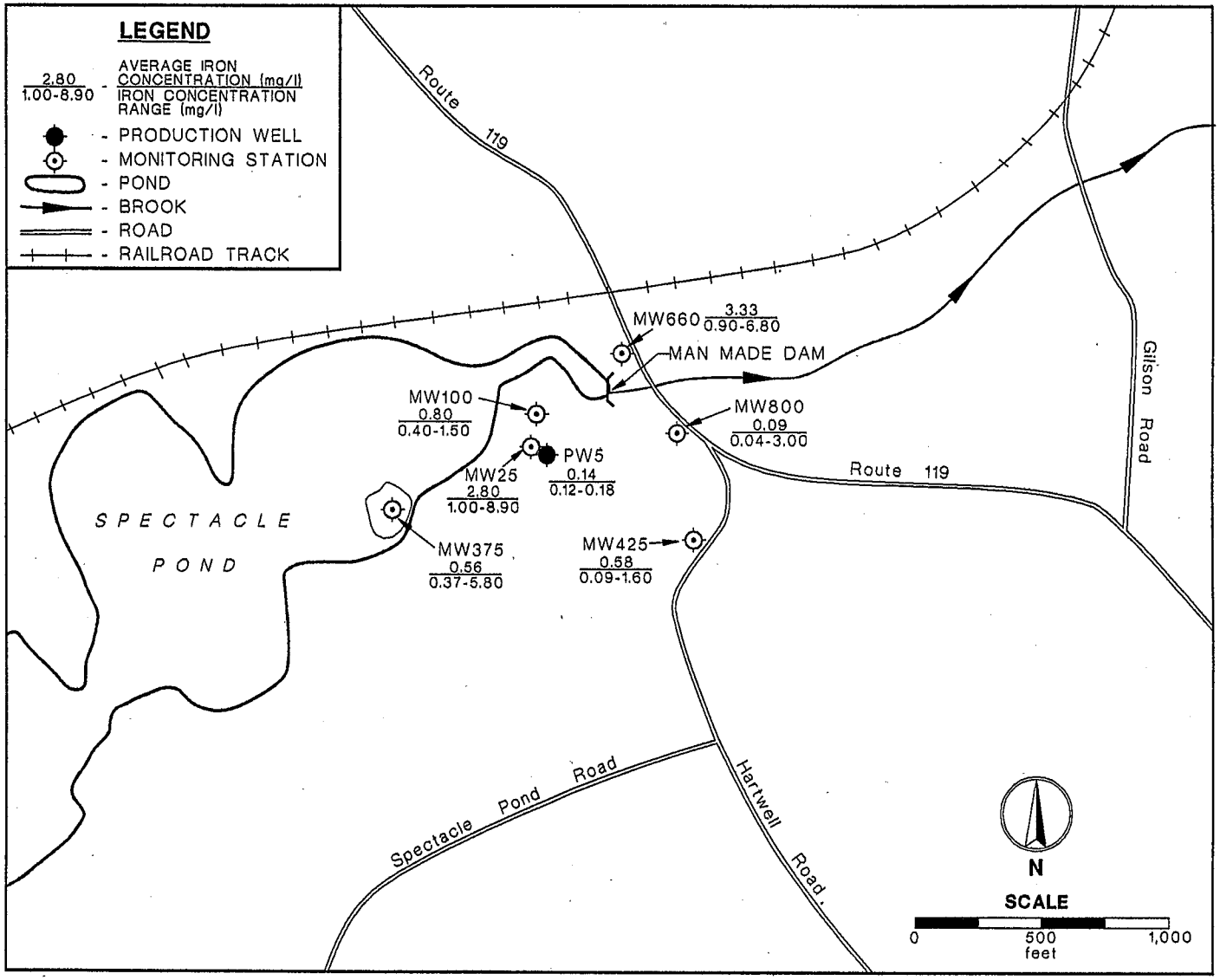

Figure 3-30A. Spatial distribution of iron concentrations near PW-5, Bennetts Brook watershed, Littleton, Massachusetts.

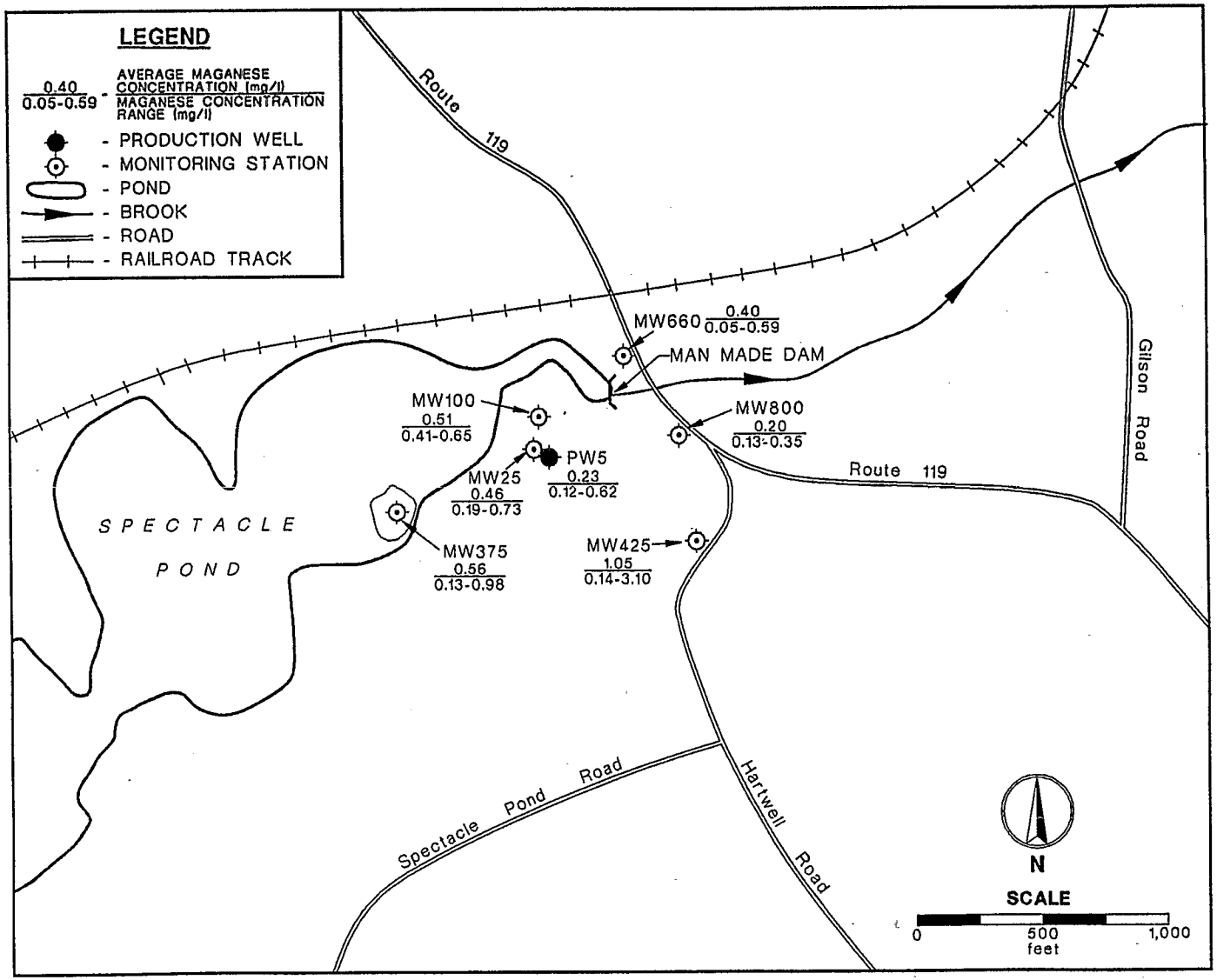

Figure 3-30B. Spatial distribution of manganese concentrations near PW-5, Bennetts Brook watershed, Littleton, Massachusetts.

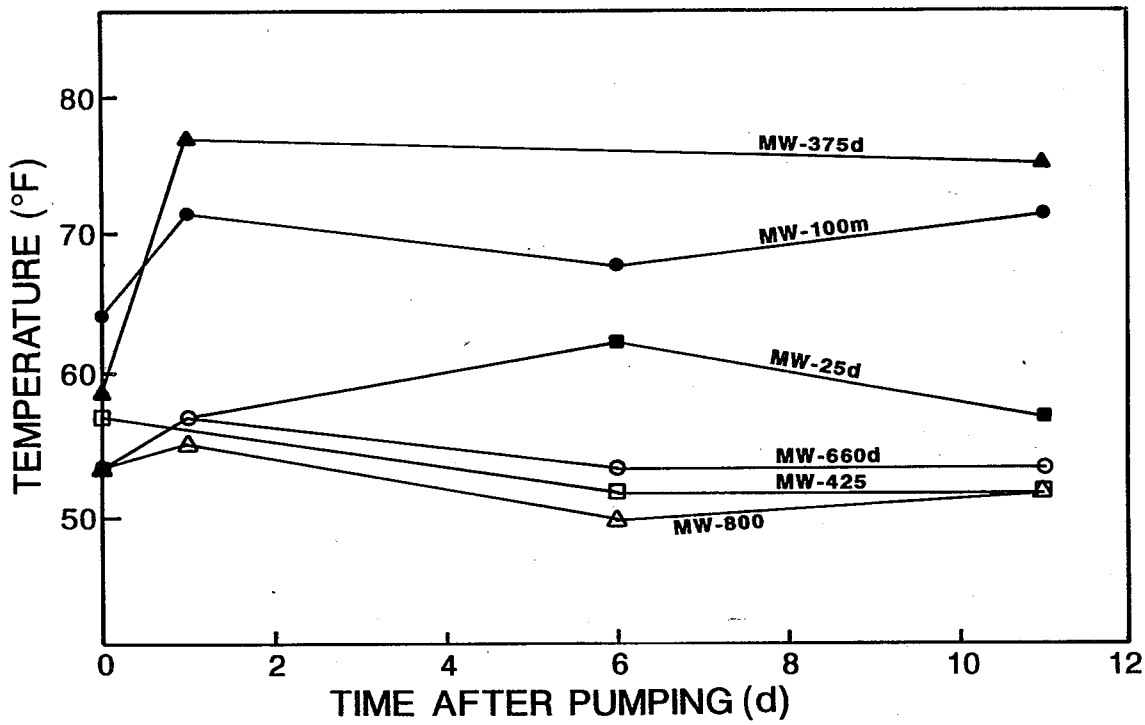

Figure 3-31. Water temperature changes observed at depths greater than 25 feet near PW-5 during the long-term aquifer test, Bennetts Brook watershed, Littleton, Massachusetts.

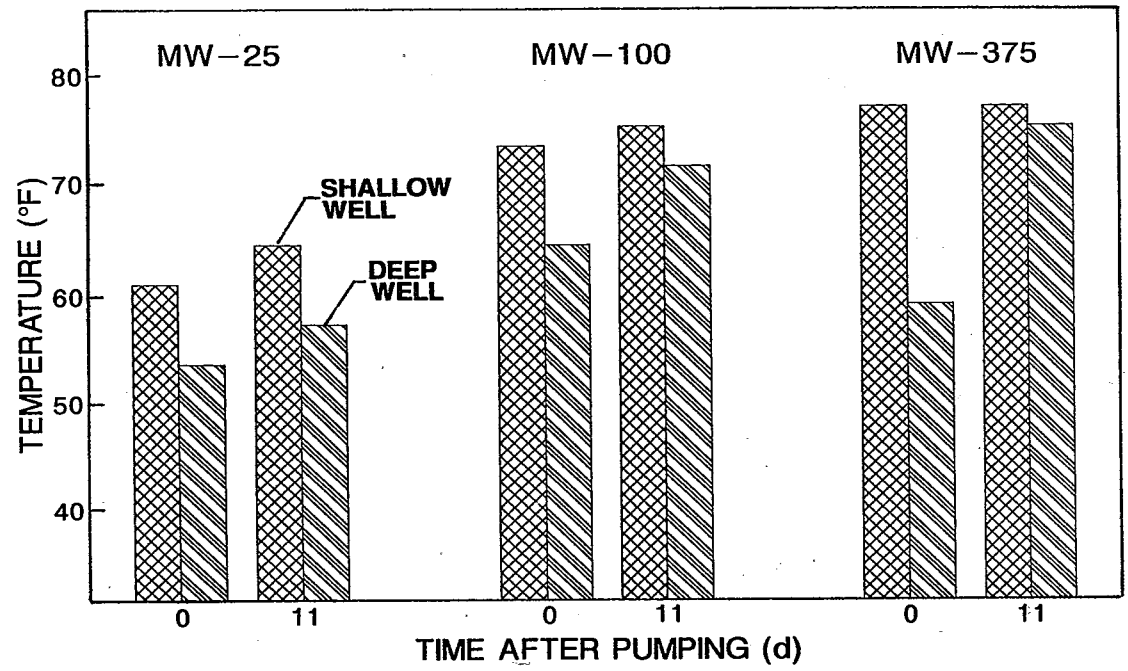

Figure 3-32. Comparison of temperatures at deep and shallow cluster wells exhibiting warming trends during the long-term aquifer test, Bennetts Brook watershed, Littleton, Massachusetts.

conditions, to greater than 1.0 ft/d under pumping conditions. These values, although somewhat lower than those obtained during the short- and long-term aquifer tests, provide additional evidence of the downward hydraulic gradient detected within the capture zone of PW-5.

REFINED WELLHEAD PROTECTION AREAS

To facilitate ground-water monitoring activities and land-use management efforts, the Spectacle Pond WHPA has been sub-divided into three protection zones: Zones I, II, and III (Figure 3-33). Each zone encompasses an area where contamination threats represent a specified level of risk to the water quality at PW-5. Consequently, each zone requires different levels of risk abatement as the potential for drinking water contamination decreases from Zone I to Zone III. Establishing protection zones within the WHPA provides WHPP managers with a method of determining variable land-use control measures and water quality monitoring activities.

Zone I is defined as the land surrounding PW-5 that is currently owned and controlled by the Littleton Water Department. This 14-acre area encompasses the 400-foot protective radius mandated by the Massachusetts DEP and applies to all public water supplies with planned yields of 100,000 gpd or greater. The fixed-radius criteria of Zone I is designed to reduce potential sources of pathogenic contamination introduced by septic discharge. Septic effluent is one of the more common source(s) of pathogenic contamination. Although the threat is greatest in surface-water systems subject to direct contamination from point-source sewage discharge or abundant native fauna, ground-water supplies can be contaminated indirectly by induced surface-water contributions. Pathogenic organisms that are subject to tortuose ground-water transport, especially where the travel times are 30 to 90 days or more, have a fairly poor survival rate. While this process provides some measure of protection, it is not entirely effective. Because of the proximity and short travel times calculated between Spectacle Pond and PW-5 (approximately 50 days), land-use controls within Zone I may not be completely effective in eliminating pathogenic contamination at PW-5.

Using the most conservative approach, Zone II was delineated as the union of the capture zones, which were established using the 3-D, steady-state criteria (MODFLOW simulation) and the 2-D, transient criteria (FLOCAD simulation) employed by the Massachusetts DEP (1991). The 2-D, steady-state ZOC (FLOWPATH simulation) is encompassed within this union. Ground-water flowpaths and TOTs, obtained during the 3-D, steady state modeling exercise, were used to develop the long-term optimized monitoring program for Zone II. The ground-water monitoring program will provide detailed, baseline information regarding local water quality impacts.

Zone III is defined as the upgradient area of the Bennetts Brook watershed, which recharges Zone II. Because regional impacts on water quality at PW-5 are buffered by Spectacle Pond (dilution effect), land-use regulations and continued source assessment monitoring will provide sufficient water resource protection within Zone III.

Zones I, II, and III are created to facilitate cost-effective and successful WHPP management. Each zone requires a specific level of protection. Zone I must be owned and controlled by the local water department or town. Strict land-use controls and ground-water monitoring activities are

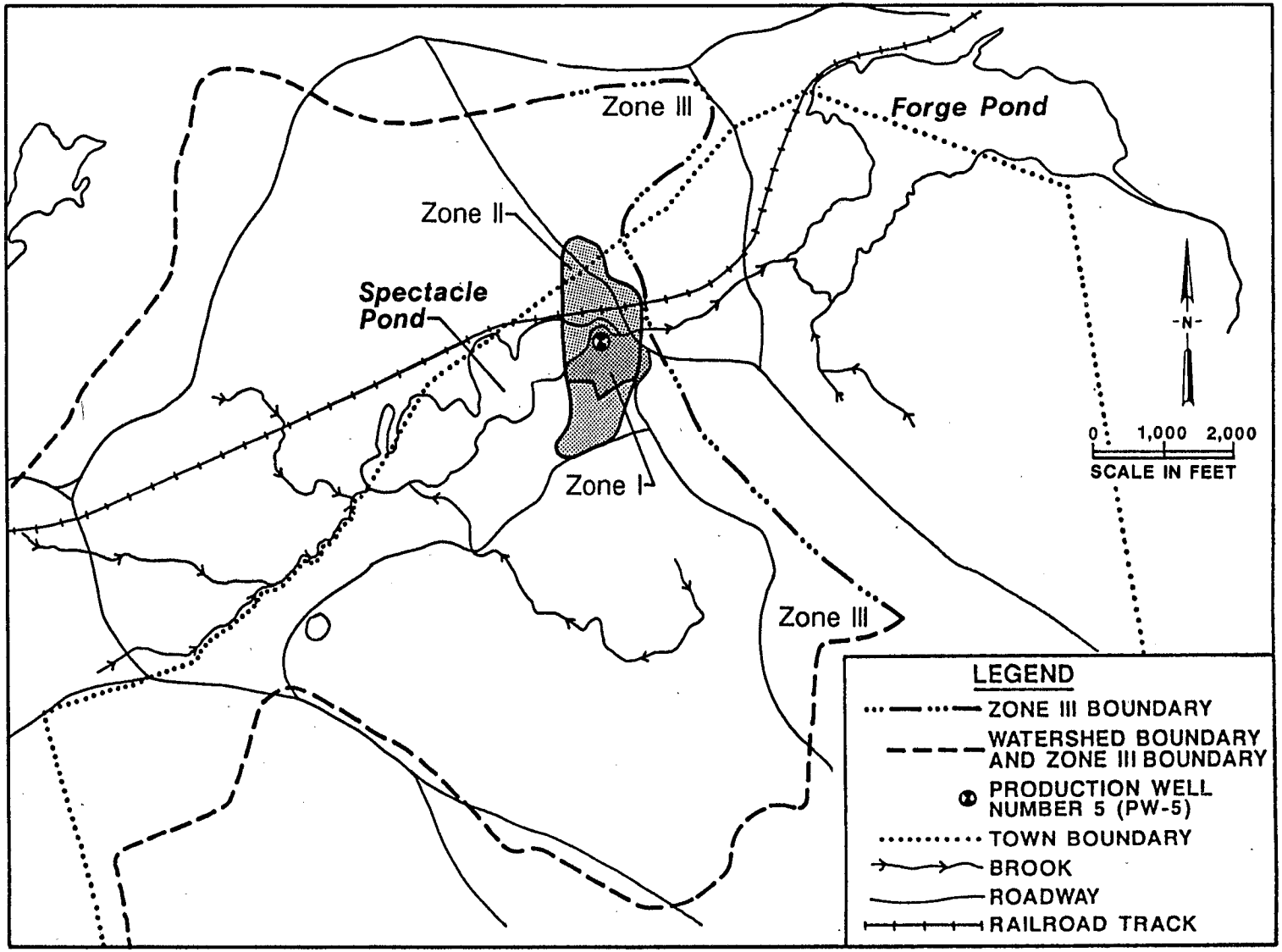

Figure 3-33. Proposed wellhead protection zones for PW-5, Bennetts Brook watershed, Littleton, Massachusetts.

required within Zone II. Zone III necessitates practical, yet comprehensive, land-use regulations with additional ground-water monitoring activities performed around high-risk, land-use activities. Without differential zoning of the WHPA, administrators may not have sufficient information regarding the ground-water flow dynamics to properly allocate monies and management efforts needed to maintain WHPP goals.

WELLHEAD PROTECTION MONITORING PROGRAM

MONITORING OBJECTIVES

The existing ground-water monitoring program for PW-5 incorporates water quality testing to satisfy state and federal drinking water regulations and source assessment monitoring to satisfy local land-use requirements. Ground-water flowpaths and TOTs recently identified as a result of this research will be used to develop a more comprehensive, long-term program. While monitoring program design within Zone II of PW-5 is emphasized, the upgradient recharge zone, Zone III, will also be evaluated. To meet long-term monitoring objectives, existing and new monitoring wells (identified as MWs) and surface-water stations (identified as SWs) are proposed. Each station is sited for a specific monitoring purpose. Monitoring parameters and sampling frequencies are also proposed for each station. Specific chemical and physical parameters were selected based on baseline water quality information and potential contamination sources. Monitoring frequencies were determined from hydrogeologic properties and associated ground-water travel times.

Ultimately, the success and reliability of the monitoring program is dependent on the accuracy of the capture zone delineation, ground-water flow paths and travel times, and baseline water quality determination and source assessment activities. Source audits will continue to be performed to identify and prioritize land-use risks because Zones II and III are subject to regional development. Dramatic changes in land-use activities may result in the addition of new monitoring stations.

MONITORING SITES

Although dilution effects from Spectacle Pond may reduce contamination burdens originating in Zone III, ground-water compliance monitoring activities will continue to be conducted for source-assessment purposes. Within Zone II, a monitoring well network will be established to evaluate water quality within the capture zone of PW-5. In addition to ground-water monitoring wells, surface-water stations will be sampled to evaluate effects from induced recharge identified between Spectacle Pond and PW-5.

Ground-Water Stations

All ground-water monitoring stations will conform to accepted EPA structural designs, as described in the Ground-Water Monitoring Stations section. Within the unconsolidated sediments of Zones I, II, and III, all monitoring wells will be constructed with 2-inch-diameter, schedule 40, flush-threaded PVC casing and 0.01-foot slotted well screens. The annular space between the drill boring and PVC screen will be backfilled with clean silica sand, and sealed 1 foot above the screen with bentonite clay. Protective casing and security locks will be placed over exposed risers and

cemented in place 2 feet below ground level (Figure 3-7A). Sampling protocol for surface- and ground-water monitoring stations will conform to accepted EPA methodologies, as previously described and referenced.

Existing Ground-Water Stations--

Compliance monitoring wells, which were installed around the perimeter of existing ground-water sources, will continue to be used for source-assessment water quality evaluations. Compliance monitoring stations (abbreviated as MW-C; shown as a well symbol in Figure 3-34) are sited around the municipal landfill and transfer station and a large industrial and commercial complex within Zone III.

By designation, Zones I and II are more stringently regulated than Zone III; therefore, all land-use activities identified within these regions will be strictly monitored. Development is prohibited within Zone I. Zone II contains residential and commercial developments, which pose direct ground-water contamination threats to PW-5. To meet the monitoring needs within this area, existing monitoring wells (identified as MW-Es) and proposed monitoring wells (identified as MW-Ps) were incorporated into the optimized monitoring network (Figure 3-34). Flowpath and travel-time results were consulted in determining well sites.

Monitoring well 660s and MW-660d (renamed MW-E1s and MW-E1d, respectively) will be used to evaluate impacts from an agricultural supply warehouse located within the northeast quadrant of Zone II. Pesticides and solvents are sold at this facility, and past storage practices are not well documented. These two wells will also be used to monitor Route 119 and railroad activities immediately upgradient. The cluster arrangement of deep and shallow well screens will permit monitoring of both light, nonaqueous-phase liquids (LNAPLs such as petroleum distillates) and dense, nonaqueous-phase liquids (DNAPLs such as industrial-strength solvents and pesticides). The locations of MW-E1s and MW-E1d correspond to the 150-day and 300-day travel-time contours, respectively (Figure 3-34). Assuming contaminant transport through adhesive processes, this well pair is expected to provide between 150 to 300 days of early warning detection.

Monitoring well 800 (renamed MW-E2 in Figure 3-34) is located along the eastern edge of Zone II near a dense commercial district. This well will be used to monitor the commercial development as a potential source. It will be monitored for site-specific parameters in the event of a severe regional drought and for regional water quality parameters. Located along the Zone II boundary, MW-E2 corresponds to approximately the 400-day travel-time contour. However, this aspect of the well location is not as critical because MW-E2 is a source-assessment monitoring well rather than one for early warning detection.

Recommended Ground-Water Stations--

To complete the ground-water monitoring network within Zone II, two proposed monitoring wells (identified as MW-P1 and MW-P2) were sited in the northern and southern portions of the zone, respectively. These wells will be installed for early warning monitoring. Because the aquifer pinches-out to the north and south, the well screens will be set at shallow depths of less than 30 feet. Monitoring well P1 and MW-P2 are situated to monitor nonpoint source activities attributed to the residential development. Both wells will be fully screened to allow for the detection of both LNAPLs and DNAPLs. The locations of MW-P1 and MW-P2 correspond to the 300-day travel-time contour (Figure 3-34). These wells are expected to provide approximately 300 days of early warning detection.

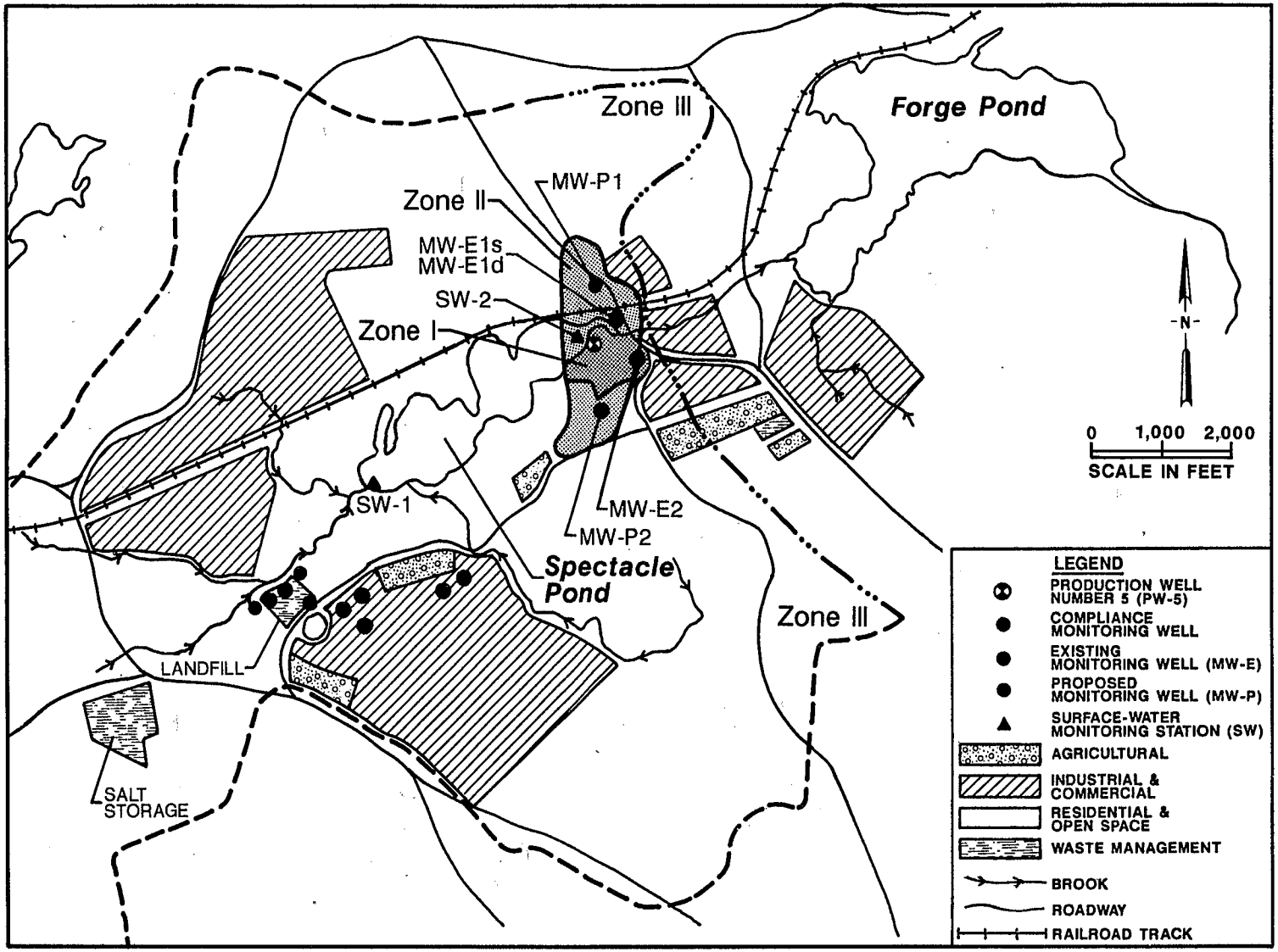

Figure 3-34. Optimized wellhead protection monitoring stations for PW-5, Bennetts Brook watershed, Littleton, Massachusetts.

Although the intent of the long-term monitoring program is to maintain the clean ground-water resources available to PW-5, samples will still be collected from PW-5 to comply with the SDWA. These samples will also demonstrate the effectiveness of the wellhead protection monitoring program. If water quality problems are encountered at PW-5 and not at other monitoring stations in Zone II, then an investigation, which will utilize all available monitoring stations within Zone II, will follow to determine the origin of the contamination.

As additional wells are incorporated in the long-term monitoring network over time; criteria should conform to standard installation and design protocols and guidelines. Well placement and design specifications will be determined by the type of contamination identified and local flowpath and travel-time characteristics. At present, PVC and stainless steel are acceptable materials for routine ground-water quality monitoring stations. If ground-water contamination is identified, MicroWells may be employed for plume delineation.

Surface-Water Stations

Spectacle Pond is a discharge area for regional ground-water flow and it receives surface water from the upstream reaches within the Bennetts Brook watershed. For these reasons, the water quality within Spectacle Pond reflects that of the entire watershed. Spectacle Pond contributes 20% to 25% of the total recharge to PW-5; therefore, the surface-water quality of the pond must be considered in the long-term monitoring program. The water quality evaluation of Spectacle Pond provides a method of determining regional land-use impacts from the towns of Littleton and Ayer. Ayer has not yet adopted a comprehensive WHPP.

Even though the travel time between Spectacle Pond and PW-5 is approximately 50 days, significant changes in surface-water quality associated with land-use activities will occur. It is anticipated that these changes will be gradual because of ground-water mixing and surface-water dilution. To evaluate surface-water quality changes, the inflow (SW-1) and outflow (SW-2) regions of Spectacle Pond will be monitored regularly (Figure 3-34). To evaluate impacts from historic and potentially chronic sources, sediment samples will be collected from the bottom of each lobe of Spectacle Pond. The parameters of interest for analysis of the sediment samples are heavy metals and PCBs.

MONITORING PARAMETERS AND FREQUENCIES

Ambient-trend, source-assessment, and early-warning monitoring objectives were integrated in the long-term monitoring strategy. Ambient trend monitoring is done to determine regional water quality variations and changes over time. Source assessment monitoring is done to evaluate source-specific water quality degradation. Early warning monitoring is done to decrease potential risks for public exposure to ground-water contamination within Zone II. Each goal will be met by analyzing samples from monitoring stations for target parameters at scheduled frequencies.

A complete list of chemical and physical monitoring parameters for the WHPA is given in Table 3-7. Chemical parameters are divided into general water quality (GWQ) and site-specific parameters. General water quality parameters are included to monitor long-term, basin-wide fluctuations in the hydrochemistry. Site-specifc parameters constitute a short list of indicator

TABLE 3-7. LIST OF CHEMICAL AND PHYSICAL MONITORING PARAMETERS FOR THE PW-5 WELLHEAD PROTECTION AREA, LITTLETON, MASSACHUSETTS.

Chemical Parameters		
General Water Quality	Site Specific	Heavy Metals
Alkalinity (total)	Microscopic particulate analysis (MPA)	Arsenic
Ammonia	Total coliform (TC)	Barium
Bicarbonate	Heavy metals (HM)	Cadmium
Calcium	Volatile organic compounds (VOCs)	Chromium
Chloride	Pesticides (Pest)	Copper
Hardness	Herbicides (Herb)	Lead
Iron		Mercury
Magnesium		Selenium
Manganese		Silver
Nitrate-nitrogen		
Nitrite-nitrogen		
Potassium		
Sodium		
Total dissolved solids (TDS)		
Physical Parameters		
Water Level (WL)	Temperature (Temp)	pH
Specific Conductivity (Cond)		

Abbreviation: PW-5 = Production Well Number 5.

chemicals that are likely to be associated with prioritized sources. Physical tests will be conducted at all monitoring stations for water quality and flow-field characterization. Details of the long-term monitoring program for the PW-5 WHPA are presented in Table 3-8. Sample frequencies determined by perceived risks of contamination from upgradient land uses and travel times are also listed in Table 3-8.

Ground-Water Stations

Compliance monitoring within Zone III will continue to be performed biannually (BA). Samples are collected in the spring and fall when the regional water table is typically highest, and lowest, respectively. Because the compliance monitoring wells are located around known or suspected contamination sources (Figure 3-34), site-specific criteria and general water quality testing criteria are required. The number and location of compliance monitoring wells are directly related to land-use activities within Zone III.

TABLE 3-8. LONG-TERM MONITORING PROGRAM FOR THE PW-5 WELLHEAD PROTECTION AREA, BENNETTS BROOK WATERSHED, LITTLETON, MASSACHUSETTS

Monitoring Station		Monitoring Station Information				Monitoring Parameters and Frequencies										
						Chemical							Physical			
						GWQ*	Site-Specific Water Quality									
		Well Depth (ft)	Screened Interval (ft amsl)	Well Material	TOT (d)		MPA	TC	HM†	VOC‡	Pest§	Herb§	WL	Temp	pH	Cond
Existing Ground-Water Wells	PW-5	50	167-182	SS	N/A	BA	BA	M	A	A	A	A				
	MW-C	Variable	Variable	PVC	N/A	BA			BA	BA		BA	BA	BA	BA	
	MW-E1s	37	185-195	PVC	150	Q			Q	Q	Q	Q	Q	Q	Q	Q
	MW-E1d	64	156-166	PVC	300	BA			BA	BA	BA	BA	Q	Q	Q	Q
	MW-E2	65	175-185	PVC	N/A	BA			A	A			BA	BA	BA	BA
Proposed Ground-Water Wells	MW-P1	~30	± 200-225	PVC	300	BA			A	BA	A	A	BA	BA	BA	BA
	MW-P2	~30	± 200-225	PVC	300	BA			A	BA	A	A	BA	BA	BA	BA
Proposed Surface Water Stations	SW-1	N/A	N/A	N/A	N/A	BA	BA		BA	BA	A	A	N/A	BA	BA	BA
	SW-2	N/A	N/A	N/A	N/A	BA	BA		BA	BA	A	A	N/A	BA	BA	BA

Abbreviations: PW-5 = Production Well Number 5; ft = feet; ft amsl = feet above mean sea level; TOT = time of travel; d = days; GWQ = ground-water quality parameters; MPA = microscopic particulate analysis; TC = total coliform; HM = heavy metals; VOC = volatile organic compounds; Pest = pesticides; Herb = herbicides; WL = water level; Temp = temperature; Cond = specific conductivity; SS = stainless steel; N/A = not applicable; BA = biannually; M = monthly; A = annually; MW-C = compliance monitoring well; PVC = polyvinyl chloride; MW-E = existing monitoring well; s = shallow depth well; Q = quarterly; d = deep well; MW-P = proposed monitoring well; ~ = approximately; SW = surface-water monitoring station.

* General water quality parameters include alkalinity (total), ammonia, bicarbonate, calcium, chloride, hardness, iron, magnesium, manganese, nitrate-nitrogen, nitrite-nitrogen, potassium, sodium, sulfate, and total dissolved solids.

† Heavy metal parameters include arsenic, barium, cadmium, chromium, copper, lead, mercury, selenium, and silver.

‡ VOCs analyzed using EPA method 524.2.

§ Pesticides analyzed using EPA method 608; herbicides analyzed using EPA method 8150.

The primary objective for all monitoring activities in Zone II is early warning detection of ground-water and surface-water contamination. Source-assessment and ambient-trend monitoring goals are also met by sampling ground water. The period of advanced warning for each monitoring station is dependent on travel times based on modeling results. Although MW-E1s and MW-E1d are equidistant from PW-5, ground-water travel times from each well to PW-5 are 150 days and 300 days, respectively (Figure 3-34). The variation in ground-water velocity at depth indicates that shallow ground-water contamination will arrive at PW-5 in less time than along deeper ground-water flowpaths. Contaminant transport rates are also controlled by the chemical properties of the contamination plume. Ground-water travel times are calculated conservatively so that no retardation effects are taken into account. For a known chemical plume, the retardation effects can be considered in determining more accurate travel times.

Production Well Number 5 will be sampled at appropriate frequencies to comply with SDWA requirements. The mandated SDWA parameters conform to the combined general- and site-specific water quality parameters listed in Table 3-5. Discharge water from PW-5 represents a composite of the water quality detected among the ground-water and surface-water monitoring stations.

Surface-Water Stations

To meet state drinking water compliance criteria, total coliform (TC) bacteria tests are performed monthly at PW-5. Coliform bacteria comprise a specific group of organisms, including aerobic and faculative anaerobic bacteria, both gram-negative rods from fecal and non-fecal origins. As an indicator bacteria of sanitary quality, the fecal group includes the organisms present in the gut and feces of warm-blooded animals. Because the indicator group may become stressed or injured in natural waters, a substantial portion (from 10% to 90%) are incapable of growth under standard laboratory conditions and may not be detected (McFeters et al., 1986). Because the presence of coliform bacteria may infer surface-water intrusion and sanitary contamination, the absence of this group is only indicative of sanitary quality and cannot, by itself, be used to indicate the absence of surface-water influence.

The use of heterotrophic bacteria plate counts and microscopic particulate analyses (MPA) may provide more useful information in assessing the transport mechanism for microbiological contamination originating from Spectacle Pond. It is recommended that heterotrophic plate counts, in conjunction with total coliform analyses, be performed monthly, and MPAs be performed biannually. Heterotróphic plate counts provide an estimate of the total numbers of live heterotrophic bacteria in the sample. Heterotrophic bacteria constitutes the major bacterial group and include organisms that derive their nutrition from both living and dead organic (carbon-based) material.

Microscopic particulate analyses are used to identify Giardia cysts, Cryptosporidium oocyst, diatoms, rotifers, coccidia, and insect parts exceeding 1 micrometer in size (Rapacz and Stephens, 1991). These organisms as a group, or in part, are found in surface-water ecosystems, and their presence is a good indicator of surface-water influences. In addition, this method can be used to enumerate green, blue-green, and other chloroplast-containing algae, which are further indicators of surface-water influence. Recently, EPA accepted the MPA method for determining the natural filtration capabilities of sand and gravel aquifers that receive recharge from surface water. Results from two separate MPAs performed on water from Spectacle Pond and PW-5 demonstrate that the sand and gravel deposits are effectively inhibiting intrusion of surface-water micro-organisms.

Until more information is available regarding transport rates and dilution effects within Spectacle Pond, surface-water testing parameters will include general and site-specific water quality groups. Sampling will be performed biannually for general water quality parameters. Because of the buffering capacity of Spectacle Pond, the site-specific parameters will be tested on an annual basis. The detection of any site-specific parameters will initiate more thorough source assessment auditing and the installation of ground-water monitoring devices, if the source is identified.

QUALITY ASSURANCE AND QUALITY CONTROL

Water quality sampling for wellhead protection will be conducted in strict adherence to the following guidelines. Ground-water monitoring wells will be purged, at a minimum, three times the standing volume of water, or until temperature and conductivity have stabilized. Cross contamination will be avoided or minimized by placing a dedicated, inertial pump, composed of small-diameter (0.625-inch OD) polyethylene flexible-tubing coupled to a Teflon foot valve within each well. To minimize quantitative errors attributed to sample collection and analysis, a QA/QC plan has been developed. Storage, handling, and analytical procedures will be conducted in accordance with standard EPA guidelines. Emphasis will be placed on appropriate well development, sample filtration, and preservation techniques. Chemical analyses will be performed at state-certified laboratories that adhere to standard analytical protocols, including chain-of-custody documentation, percent recovery and surrogate spike analysis, and use of trip and laboratory blanks.

MONITORING DATA BASE STORAGE, UPDATE, AND RETRIEVAL SYSTEMS

A data base management system is required to store, update, and retrieve information associated with source audits and ground-water monitoring. Information collected from these activities is currently in hard copy in a filing system. A number of computer software programs are available to provide a more effective and efficient means of data storage, update, and retrieval. Current options range from standard, off-the-shelf, data base management packages to highly technical geographic information systems (GISs). The associated costs and complexities of each vary widely.

The appropriate management system will provide many functional benefits to persons involved in WHPA management. By design, the system should eliminate tedious data entry processes, while providing ease in data retrieval, manipulation, and evaluation. Communication with analytical laboratories using computer modems would allow the transfer of results electronically. This mode of transfer will reduce processing time and data-entry errors. For any computer function, it is important to consider the specific applications needed to perform management tasks. Sophisticated software and hardware often require matched technical expertise. Whatever system is selected, the technical abilities of the WHPP staff and the constraints of the project budget must be considered.

CONTINGENCY PLANNING

Historically, ground-water contamination threats in Littleton have been addressed as they occur, with the level of response commensurate to the perceived threat. A catastrophic event such as the accidental release of a large volume of hazardous material initiates an emergency response involving many departments and agencies. Responses are coordinated through the Littleton Fire Chief and Water Department. The detection of ground-water contamination at downgradient monitoring wells at facilities initiates a more methodical, site-specific investigation and remediation.

Catastrophic releases initiate a spill-response plan involving the Police, Fire, and Water Departments (Figure 3-35). Generally, the notification of a catastrophic release is made to the Fire or Police Department. If the incident occurs near a water supply well, the spill-response team contacts the Water Department immediately. Containment and diversion methods are implemented first, and where appropriate, additional support is sought from the Highway Department, Civil Defense Authority, or a professional incidence-response contractor. In addition to these measures, the Water Department shuts down the affected production well and utilizes other wells outside the area of concern to meet water demands. The well is not put into production again until the threat abates.

In the event of contamination of PW-5 or another production well, Littleton's contingency plan contains an element for new ground-water supply development. The town's future well site is located in the southwest corner of the Beaver Brook Watershed (Figure 3-36). The proposed well site has been approved by the State, and protection Zones I, II and III have been delineated. A pumping station has not yet been constructed at the proposed well.

The adjacent town of Boxborough shares the recharge area to the proposed well and its protection zones. Boxborough and Littleton formed an environmental support program in 1986 to protect their shared resources. The towns will implement effective protection strategies together, including development screening, source regulation, and ground-water monitoring.

Environmental damage can be avoided or minimized through contingency planning and action. The primary objective of the spill-response team is to contain or divert released hazardous substance(s) from the water supply well, under the direction of the Water Department. Hay bales, oil booms, absorptive pads, leak repair kits and other spill containment equipment are stored in emergency vehicles and commonly utilized by the response teams.

The refined capture zone for PW-5 and the ground-water flowpaths and travel times generated by modeling provide spill-response coordinators with baseline information to better assess contamination risks posed to PW-5. The necessary response actions are dictated by the concentration, chemistry, and location of the identified release with respect to the water supply well.

Priority contaminants include those parameters detected above state or federal primary drinking water standards that have been identified within Zone II. If contamination is identified, a pumping management strategy would be designed to reduce the likelihood of drawing the contaminant toward the production well. Concurrently, resampling, plume mapping, site-specific monitoring, and site-inspection activities, would be initiated. These activities would be followed by remediation.

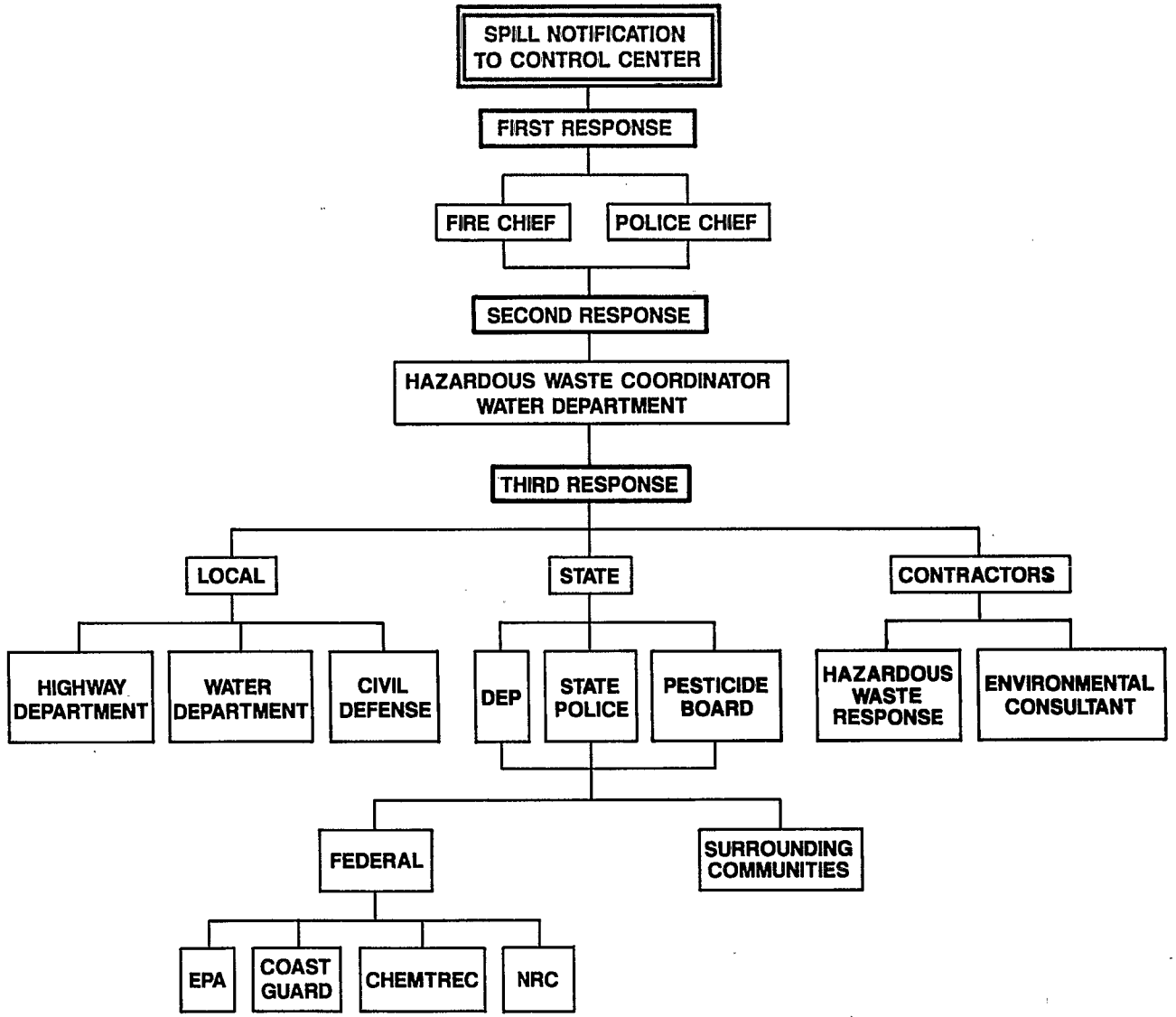

Figure 3-35. Spill-response hierarchy chart, Littleton, Massachusetts. (DEP = Massachusetts Department of Environmental Protection; EPA = U.S. Environmental Protection Agency; Chemtrec = Chemical Transportation Emergency Center; NRC = National Response Center)

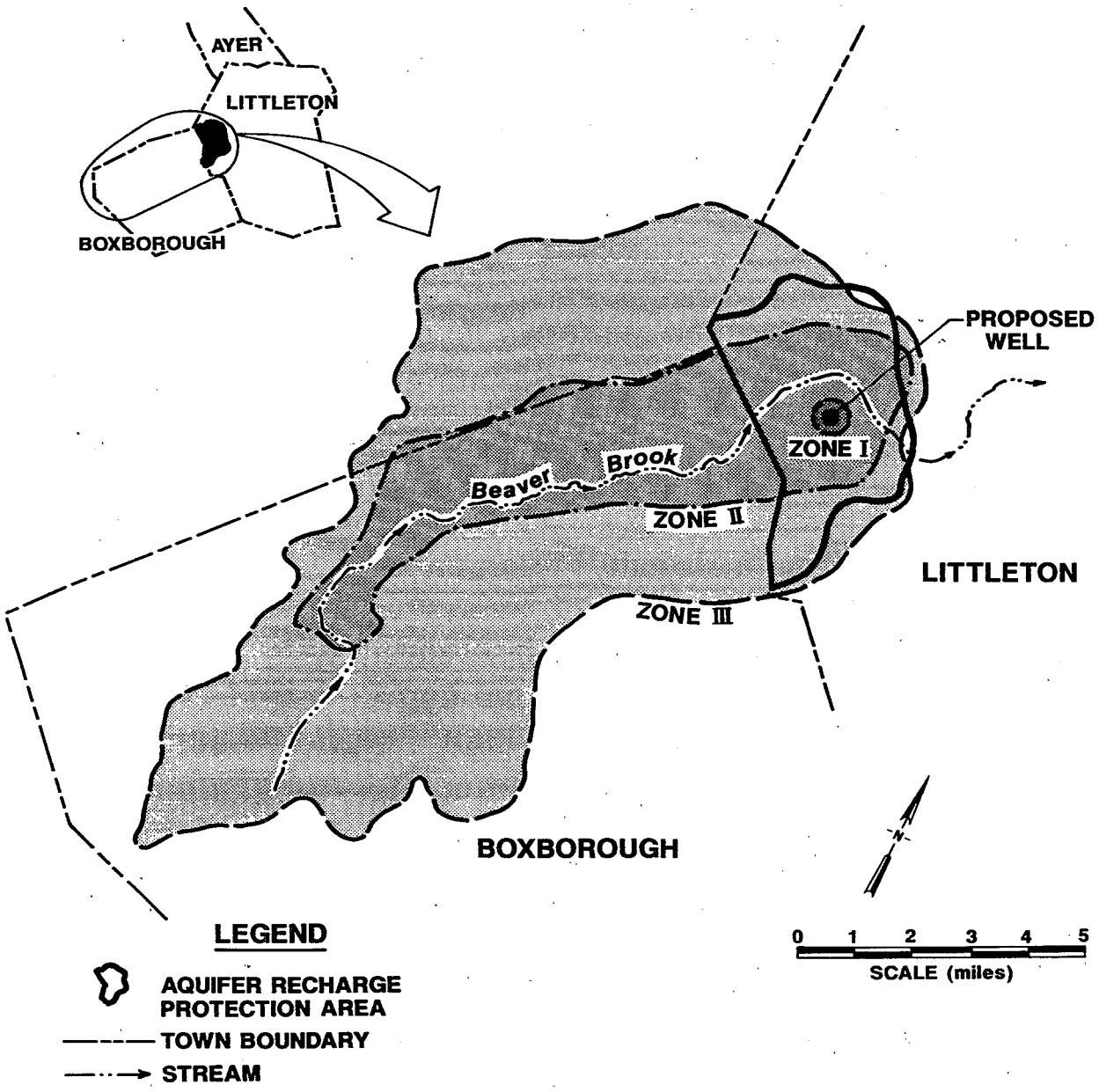

Figure 3-36. Proposed well site and protection zones, Bennetts Brook watershed, Littleton, Massachusetts.

An alternative pumping regime to redirect the plume migration away from the production well could be engineered using capture wells to alter the regional hydraulic gradient. This action could greatly reduce or even eliminate migration of the contamination toward the well.

Nonpriority contaminants are characterized as low-level contamination issues within Zone II, or elevated concentration levels within Zone III. In both cases, monitoring well(s) in which contamination is detected are resampled to ensure the integrity of the original test. If positively confirmed, a source assessment investigation is initiated. The intent of the investigation is to determine the source and volume of the contamination. In addition to detailed source audits, groundwater monitoring frequencies may be increased to allow sufficient early warning for the production well.

CONCLUSIONS

In 1980, the Littleton Water Department began researching and implementing a comprehensive aquifer and watershed protection program for the town of Littleton, Massachusetts, after the bordering town of Acton lost 70% of their municipal ground-water supply capacity due to contamination from an industrial facility. This incident initiated development of Littleton's existing wellhead protection program (WHPP) that now serves as a model for state and local agencies. In a continuing effort to improve the WHPP, the Water Department delineated a new capture zone for Production Well Number 5 (PW-5) and designed an optimized monitoring program for its long-term protection. Ground-water monitoring has played an integral role in the overall success of Littleton's program.

Several tasks were performed to develop an optimized ground-water and surface-water monitoring program for the new WHPA for PW-5 in the Bennetts Brook watershed. Existing hydrogeologic reports were used to develop a conceptual hydrogeologic model accounting for aquifer properties, boundaries, flow dynamics, and water quality. To validate the conceptual flow model, short- and long-term aquifer testing, ground-water flow simulations, and water quality analyses were performed. A thorough source assessment was conducted to augment the water quality characterization.

Regional geomorphology and aquifer testing demonstrate that the Spectacle Pond aquifer is generally an unconfined, buried valley system. The regional water-table depth varies from 0 to 25 feet and ground water flows northeast, parallel to the strike of the fault system within the Bennetts Brook watershed. Most drawdown and recovery curves generated during the aquifer tests were matched with Neuman type curves established for unconfined aquifers. Exceptions to the unconfined ground-water response better approximated the Hantush type curve for leaky aquifers. The locations of the monitoring stations that exhibit this response coincide with semi-confining peat deposits identified at the perimeter and along the bottom of Spectacle Pond, immediately adjacent to PW-5.

Locally, the glacial outwash deposits have very high transmissivity values of greater than 4,000 feet per day (ft^2/d) and high potential well yields of greater than 300 gallons per minute (gpm). Zones of lower transmissivity (less than 1,350 ft^2/d) are encountered along the perimeter of the buried valleys, where the outwash materials are poorly sorted and the aquifer pinches-out into till and bedrock uplands. Results from the short-term aquifer test indicate horizontal hydraulic conductivities ranging from 127 to 2,220 ft/d, with an average value of 560 ft/d. The ratio of horizontal to vertical hydraulic conductivities (the anisotropy ratio) was estimated to be 50:1.

Interpretations of the long-term aquifer test revealed a downward hydraulic gradient between PW-5 and Spectacle Pond. As PW-5 was pumped, the hydraulic gradient increased until it finally reached stabilization. The presence of the vertical hydraulic gradient, confirmed by a vertical temperature gradient, demonstrates that the pond is recharging the aquifer when PW-5 is in

operation. The long-term test also demonstrated that the northwest portion of the aquifer receives approximately five to six times more water from the pond than from ground-water recharge. This translates to approximately 20% to 25% of the total recharge to PW-5. Throughout the rest of the aquifer, responses were similar to those observed during the short-term test. Again, hydraulic conductivities ranged from 300 to 3,600 ft/d with an average value of 520 ft/d. Regionally, the prevailing ground-water flow component is horizontal, with an anisotropy ratio of approximately 35:1. The average specific yield value is approximately 0.11, which is typical for an unconfined aquifer.

The capture zone for PW-5 was delineated by combining results from several ground-water flow models:

- FLOWPATH, a steady-state, two dimensional (2-D), horizontal plane, ground-water flow and pathline analysis model
- MODFLOW, a steady-state or transient, three-dimensional (3-D) model which utilizes a finite-difference numerical method to solve 3-D flow equations in combination with MODPATH
- MODPATH, a pathline analyses post processor for MODFLOW
- FLOWCAD, an extension of FLOWPATH, used to simulate transient hydraulic head distributions required by the Massachusetts Department of Environmental Protection

Depending on the modeling objective, each model can be used effectively for delineating WHPAs.

FLOWPATH provided the regional (far-field) approximation of steady-state flow dynamics for the Spectacle Pond aquifer. MODFLOW was used to address near-field, vertical flow components and variations in aquifer characteristics (hydraulic conductivity and depth) in proximity to PW-5. FLOWCAD was employed to generate the transient, Zone II boundary, as required under current state WHPA delineation regulations. Each simulation utilized a time-related criterion to define the zone of contribution, also referred to as the time-related capture zone. A second and less sophisticated criterion, drawdown, generated inappropriate protection results because of uncorrected regional ground-water flow gradients.

Baseline ground-water monitoring was done to characterize water quality for long-term program planning. Monitoring goals were differentiated into ambient trend and source assessment activities. These categories were further sub-divided by analyses of general water quality parameters, site-specific water quality parameters, and a combination of both parameters.

The water quality of the Spectacle Pond aquifer is slightly acidic and very fresh: a *Type I Water*. Sodium and calcium are the most abundant cations in the ground water; bicarbonate is the most abundant anion. Trilinear Piper diagrams were used to differentiate two hydrochemical facies within the study area. Above a depth of 25 feet, a sodium-bicarbonate water type predominates; a calcium-bicarbonate water type predominates below 25 feet.

The land uses identified during the source assessment include underground fuel storage tanks, metal plating shops, a municipal landfill, agricultural operations, industrial and commercial developments, and residential subdivisions. Collectively, these sources pose potential ground-water contamination threats of heavy metals, VOCs, pesticides, and nutrients. Results of source-assessment monitoring demonstrate that, to date, ground-water quality within the capture zone of PW-5 has been

virtually unaffected by source operations. Sodium is the only exception. Moderate sodium concentrations detected within the surface water and shallow aquifer are probably related to highway de-icing activities. Despite these otherwise favorable analyses, naturally occurring manganese and iron were detected at elevated levels in proximity to PW-5.

To maintain the drinking water quality of PW-5, the current WHPA will be modified to include the new Zone I, II, and III criteria determined through the ground-water modeling effort. Zone I is the 14-acre parcel (400-foot protective radius) adjacent to PW-5, owned by the Littleton Water Department. Future land use in this area will be restricted to open space, exclusively. Zone II is established as the union of the capture zones generated using the 3-D steady-state (MODFLOW) criteria and the 2-D transient (FLOWCAD) criteria required by the state. Ideally, total land-use restrictions would also be proposed within Zone II. However, because of existing zoning regulations and associated property values, this option is impractical. Instead, moderate- and high-risk land-use activities will be prohibited, while other land-use activities such as residential development and existing commercial operations will be allowed. All activities within Zone II will continue to be closely monitored through ground-water analysis and periodic site inspections, where appropriate.

Zone III includes the upgradient portion of the Bennetts Brook watershed, which contributes recharge to Zone II and PW-5. Protection measures within Zone III will be less restrictive. Land-use regulations, coupled with source assessment monitoring, will provide sufficient protection against regional ground-water quality degradation. Although effective land-use controls already exist as bylaws, the newly-delineated protection zones provide a template of ground-water and surface-water flow dynamics necessary to design an optimized monitoring program.

A long-term, water quality monitoring program is proposed to provide an accurate and cost-effective means of ensuring protection of PW-5 in the future. Monitoring sites are strategically located to provide early warning detection within Zone II and compliance monitoring within Zone III. The use of ground-water flow paths and travel times was critical in designing the long-term monitoring network for the WHPA of PW-5. To allow for sufficient early-warning response time, a minimum 300-day TOT criterion has been established for all new monitoring locations. This criterion will allow biannual sampling events to be conducted, while providing sufficient response time between initial detection of ground-water contamination and the execution of necessary contingency measures.

Flowpath studies have demonstrated the importance of surface-water monitoring in the overall WHPA monitoring program. Despite Littleton's commitment to wellhead protection, it can neither eliminate nor regulate ground-water or surface-water contamination originating from Zone III areas that fall within the jurisdiction of neighboring towns. Fortunately, regional flowpath analyses indicate that scheduled monitoring of Spectacle Pond will provide an effective means of monitoring these politically unaccessible areas. Land-use activities in bordering towns are generally unregulated.

Hydrogeologic boundaries do not often correspond with geopolitical boundaries. Inevitably, the mode of implementation and technical sophistication of WHPPs from one municipality to the next will vary considerably. Mutual water quality issues need to be identified and cooperation fostered toward resolution of these issues, so that a cooperative relationship can be attained. In most instances, the exchange of hydrogeologic evaluations will greatly enhance the accuracy and effectiveness of a regional WHPP. To address the present WHPP discrepancies between Littleton and its neighboring communities, educational outreach programs will be initiated. Specifically, the benefits

of ground-water protection will be discussed among appropriate town boards and planning committees. If neighboring communities fail to recognize the benefits of such a program, state or federal assistance may be sought. Regardless of the approach, the need for the implementation of an effective, joint WHPP should be emphasized.

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CHAPTER 4

WELLHEAD PROTECTION AND MONITORING OPTIONS FOR THE SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA

by

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NOTICE

This report is the result of research supported by the U.S. Environmental Protection Agency's Environmental Monitoring Systems Laboratory in Las Vegas, Nevada, as part of its efforts to provide technical assistance to state, tribal, and local governments on the implementation of the Wellhead Protection Program. The specific methods and approaches contained in this document have undergone peer review but do not constitute official Agency endorsement or policy recommendations. The Office of Research and Development provides this information to help solve complex technical problems related to refined delineation and ground-water monitoring of wellhead protection areas in various hydrogeologic settings. Further assistance is available from the Environmental Monitoring Systems Laboratory in Las Vegas, from the Office of Ground Water and Drinking Water in Washington, D.C., and from the ground-water offices in the ten Environmental Protection Agency regions.

ABSTRACT

The city of Sioux Falls is located in the southeast corner of South Dakota. The Big Sioux aquifer is the primary source of water for about 125,000 persons in the Sioux Falls metropolitan area. One of the municipal wells in the Big Sioux aquifer, the airport wellfield, is underlain by surficial, glacial outwash deposits. The Big Sioux River is located directly west of the airport wellfield and flows south over and through the outwash, draining approximately 4,000 square miles of upstream land.

The city's wells pump most of their water directly from the aquifer and a small quantity from the Big Sioux River. However, the river is hydraulically connected to the aquifer, and recharge from the river in the airport wellfield area is significant. In 1988, approximately 79% of the recharge to the airport wellfield aquifer was induced from the river due to wells pumping. Induced flow from the river to the aquifer is demonstrated by decreased flow in the river during low recharge periods.

This research was conducted to evaluate (1) the hydraulic connection between the Big Sioux River and the adjacent aquifer, and (2) the potential impact of the river on aquifer water quality. In the broader perspective, additional goals included refined delineation of the wellfield protection area and design of a long-term water quality monitoring program.

Drilling logs indicate that the thickness of the aquifer in the wellfield area ranges from 20 to 50 feet. Aquifer testing results yield an average hydraulic conductivity value of 800 feet per day and a transmissivity value of approximately 21,000 square feet per day for the aquifer.

Many potential point sources of contamination exist in the study area. These include: industrial and commercial areas, the South Dakota Air National Guard facility, a petroleum pipeline, the Sioux Falls Regional Airport, and a decommissioned municipal landfill. The threat of contamination from these sources is underscored by the recent history of contaminant releases in the area.

To estimate ground-water travel times in the study area, aquifer testing, dye tracing, and ground-water modeling were employed. During aquifer testing, two dye injections were made. The first dye was injected in a well approximately 40 feet north of the pumping well. Detectable dye concentrations first arrived at the pumped well after about 12 hours. The second dye was injected in a well near the edge of the river, approximately 140 feet north of the pumping well. Detectable dye concentrations from the second injection site first arrived at the pumped well in 7 to 9 days. Aquifer testing and dye-tracing results indicate that a contaminant could travel from the river to the wellfield in less than 9 days.

A two-dimensional, steady-state model (FLOWPATH) was used to generate time-related capture zones for the municipal wells and to simulate contaminant travel times. One-, two-, and five-year capture zones were calculated for each of the municipal wells in the airport wellfield. Modeling

of simulated spill sites from several of the potential point-source contamination areas indicates that contaminants entering the aquifer at areas to the north and south of the well field could reach the municipal wells in 1 to 2 years.

The City of Sioux Falls and Minnehaha County have delineated wellhead protection areas by using the hydrogeologic-mapping method. Wellhead protection ordinances are designed to impose guidelines and restrictions on new land uses, or proposed changes in existing use, in order to protect the aquifer water quality.

A wellhead protection monitoring program at the airport wellfield is proposed to document ambient water quality conditions and to serve as an early-warning detection system. Line-source monitoring is proposed to monitor the Big Sioux aquifer and the diversion canal for contaminants that could potentially enter the aquifer. Point-source and nonpoint-source monitoring are proposed to monitor water quality between the airport wellfield and potential sources. The categories of parameters for monitoring are general water quality, volatile organic compounds, trace metals, pesticides, and nutrients. Sampling frequencies for each of the categories were selected as a function of the type of source to be monitored.

Contingency planning is warranted to establish emergency responses to contaminant releases at the surface of the aquifer and in the river. Alternative water supply development must also be continued as part of the contingency planning effort.

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ABBREVIATIONS, SYMBOLS, AND CONVERSION FACTORS

ABBREVIATIONS

acre-ft	acre-foot
amsl	above mean sea level
ASTM	American Society for Testing and Materials
d	day
DNAPL	dense, nonaqueous phase liquid
DOT	Department of Transportation
EMSL-LV	Environmental Monitoring Systems Laboratory, Las Vegas, Nevada
EPA	U.S. Environmental Protection Agency
ft	foot
ft/d	foot per day
ft^2/d	square foot per day
ft^2/min	square foot per minute
GIS	Geographic Information System
gpm	gallon per minute
hr	hour
HW	horizontal well
in	inch
in/yr	inch per year
l	liter
mg	milligram
mg/l	milligram per liter
Mgd	million gallon per day
min	minute
ml	milliliter
MW	monitoring well
µg/l	microgram per liter
µS/cm	microSiemen per centimeter
N/A	not applicable
ND	none detected
PA	phenolphthalein alkalinity
psi	pound per square inch
PVC	polyvinyl chloride
QA/QC	quality assurance and quality control
R49W	Range 49 West
SC	specific conductivity
SCH	schedule
SDDENR	South Dakota Department of Environment and Natural Resources

SDDOT	South Dakota Department of Transportation
SDGS	South Dakota Geological Survey
SFE	Sioux Falls Elmwood Landfill
T101N	Township 101 North
TA	total alkalinity
TDS	total dissolved solid
TOT	time of travel
USGS	United States Geological Survey
VOC	volatile organic compound
VRW	vertical well in the river
WHPA	wellhead protection area
WHPP	Wellhead Protection Program
ZOC	zone of contribution

SYMBOLS

2,4-D	2,4-dichlorophenoxy acetic acid
As	arsenic
Ca	calcium
Cl	chloride
CO_3	carbonate
Eh	oxidation-reduction potential
F	fluoride
Fe	iron
HCO_3	bicarbonate
K	potassium
Mg	magnesium
Mn	manganese
Na	sodium
N	nitrogen
NO_2	nitrite
NO_3	nitrate
Se	selenium
SO_4	sulfate
u	well function
Q	pumping rate
r	radial distance from pumping well to monitoring well
S	storativity
s	drawdown
T	transmissivity
t	time
W	well function

CONVERSION FACTORS

Multiply	By	To Obtain
acre-foot	1230	cubic meter
cubic foot per second	0.0283	cubic meter per second
foot	0.3048	meter
foot per second	0.3048	meter per second
gallon	3.785	liter
gallon	0.134	cubic foot
gallon	0.00379	cubic meter
gallon per day	0.000003528	cubic foot per second
gallon per day per foot	0.000207	square meter per day
gallon per day per square foot	0.0408	meter per day
inch	0.0254	meter
inch per year	25.4	millimeter per year
mile	1.609	kilometer
million gallons per day	2.629	cubic meter per minute
square foot per minute	0.0929	square meter per minute
square foot per second	0.0929	square meter per second
square mile	2.59	square kilometer

$$°F = \frac{9}{5}(°C) + 32$$

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BACKGROUND FOR THE SIOUX FALLS CASE STUDY

INTRODUCTION

In support of the Wellhead Protection Program (WHPP) for the city of Sioux Falls, South Dakota (Figure 4-1), the South Dakota Geological Survey (SDGS) conducted research in cooperation with the U.S. Environmental Protection Agency (EPA) Environmental Monitoring Systems Laboratory in Las Vegas, Nevada (EMSL-LV), to design a long-term monitoring program for the Sioux Falls municipal (airport) wellfield. The research focuses on characterization of the hydraulic relationship between the Big Sioux River and the unconfined aquifer providing potable water to the city of Sioux Falls (Figure 4-2). Potential sources of aquifer contamination in proximity to the wellfield and the river as a line source were assessed. The study area is located in the upper midwest region of the United States in the city of Sioux Falls, Minnehaha County, South Dakota (Figure 4-1). The region is semiarid, and the average annual precipitation is approximately 26 inches. The evapotranspiration rate is approximately 36 inches per year.

Ground water is the primary source of water used for municipal, domestic, and agricultural purposes in the region. The predominant aquifer in the area is the Big Sioux aquifer (Figure 4-1). The use of other aquifers in the area as water supply sources for the city is limited because of inferior water quality, insufficient water quantity, or distance of the aquifer from the city of Sioux Falls. The Big Sioux aquifer provides most of the water used by the city. It is the source of drinking water for about 125,000 persons in the Sioux Falls metropolitan area. In 1988, only about 7% of the water used by the city was pumped directly from the Big Sioux River; 93% was provided by the aquifer. Recently, however, the city has increased the amount of water taken from the river after installation of a 44 million-gallon-per-day (Mgd) surface-water intake structure.

The Big Sioux aquifer is composed of surficial outwash deposits that typify the Big Sioux River valley. The Big Sioux River flows southward over and through the outwash, draining approximately 4,000 square miles of land upstream from the Sioux Falls municipal wellfield. One of the Sioux Falls municipal wellfields in the Big Sioux aquifer is the focus of study for this research--the airport wellfield (Figure 4-2). The airport wellfield consists of 21 of the city's 51 water supply wells.

The land upstream from the Sioux Falls municipal wellfield is used generally for agricultural purposes and is heavily irrigated in some areas. Several cities upstream from Sioux Falls may also have an impact on river water quality. From a hydrologic perspective, the Big Sioux River acts as a central drain for all potential sources of pollution, including surface runoff in the drainage basin and all point and nonpoint sources of pollution overlying the Big Sioux aquifer.

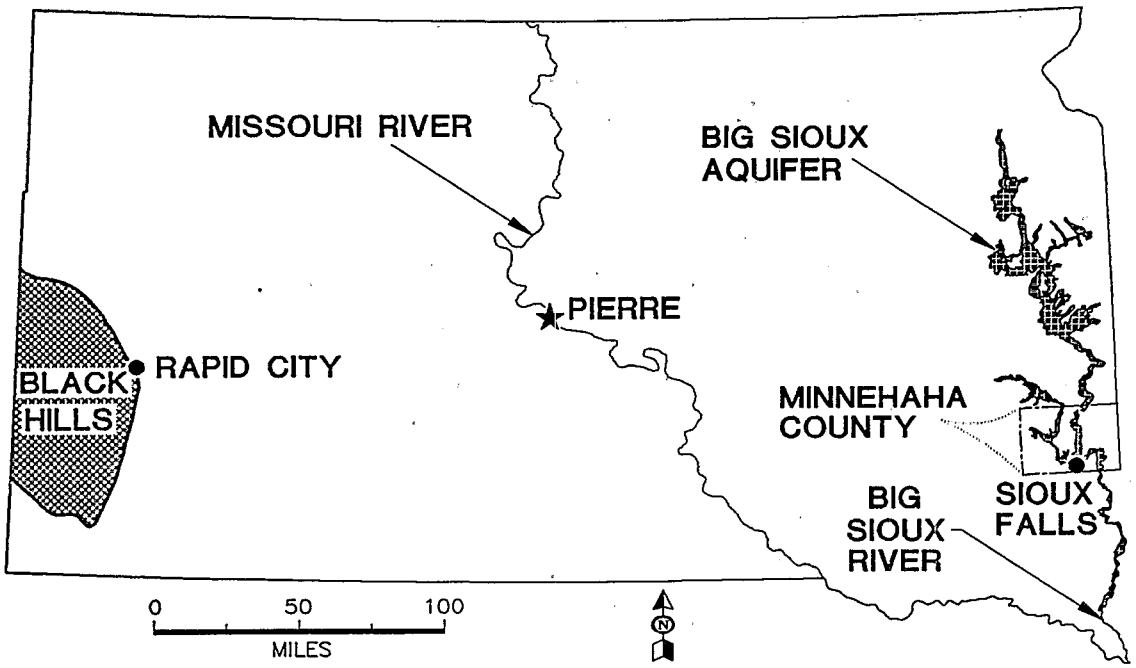


Figure 4-1. Regional location of the city of Sioux Falls within the Big Sioux aquifer and Minnehaha County, South Dakota.

Discharge from the municipal wells in the airport wellfield induces river water flow into the adjacent aquifer because of good hydraulic interconnection and because discharge from pumping exceeds natural recharge rates to the aquifer in the wellfield area. Surface-water inducement is extensive and is demonstrated by the cessation of river flow in the vicinity of the municipal wellfield during low-flow periods. Thus, any significant degradation of river water quality will impact the quality of water pumped from the Sioux Falls municipal wellfield. Therefore, Sioux Falls should be cognizant of potential pollutants that may enter the river and the aquifer upstream and upgradient from the city wellfield.

The importance of recharge induced from the river into the aquifer cannot be overstated. In 1988, an estimated 79% of recharge to the airport wellfield was from the river. Although the amount of recharge from the river varies for different years because of variations in the wellfield pumping rate and the river flow, this percentage reflects the importance of the river recharge component.

Surface water generally moves at a much faster rate than ground water because of its unrestricted flow at the surface, as opposed to flow through porous media. Consequently, contaminants introduced upstream have the potential to reach the wellfield in a relatively short time

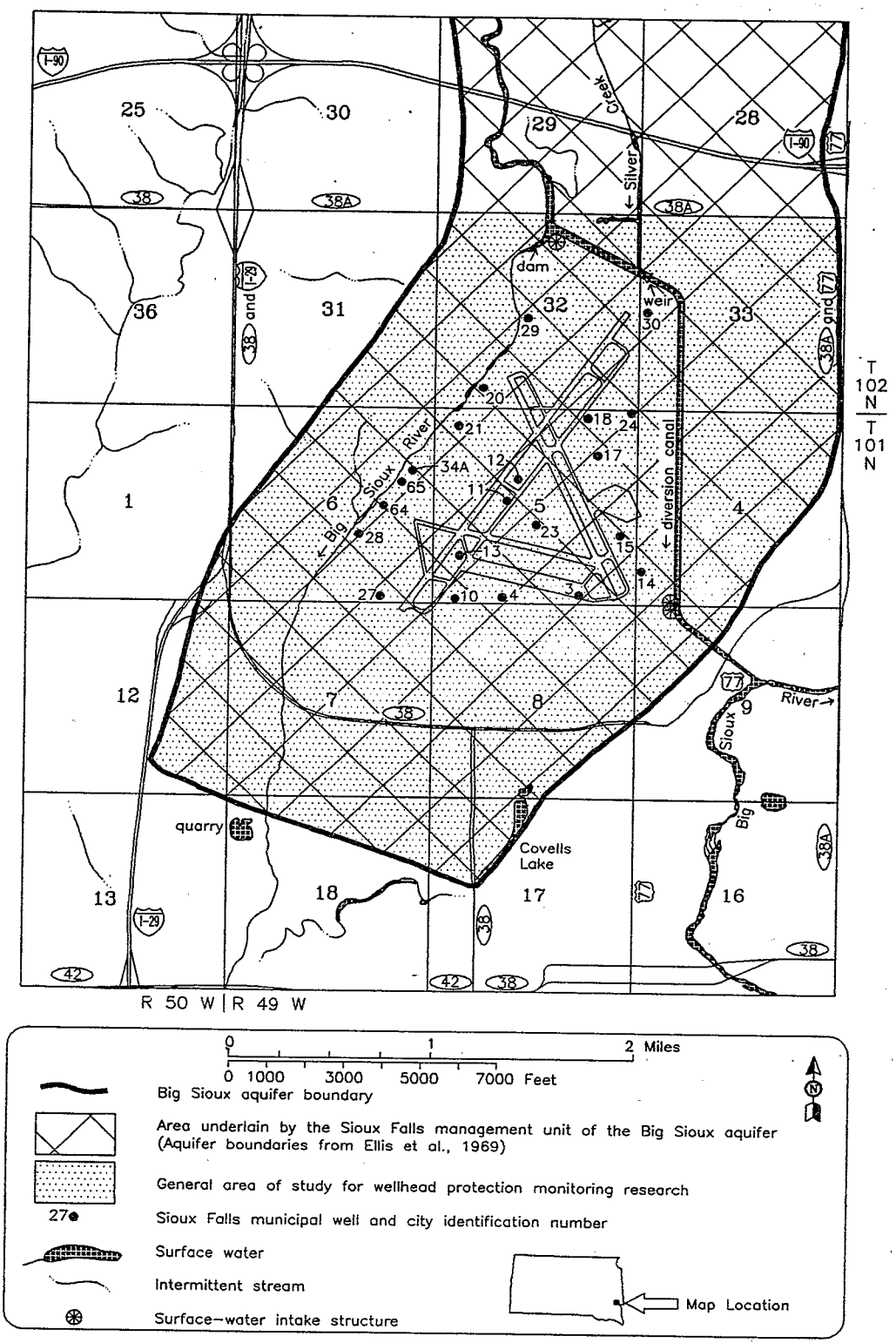


Figure 4-2. Location of the Sioux Falls airport wellfield and study area, South Dakota.

period. It is very difficult to delineate an effective time-related wellhead protection area (WHPA) for a specific time period (for example, 1 month) without considering the impact of the river. Therefore, protection of water quality in the river must be included in an effective WHPP for the city.

In addition to the threat of contamination from the Big Sioux River, there are several potential point-source areas of contamination around the wellfield. In particular, contamination has been documented for the industrial and commercial areas south and east of the wellfield, as recorded in the South Dakota Department of Environment and Natural Resources (SDDENR) files. Nonpoint sources are also present in the vicinity of the wellfield. These sources must also be considered in the development of an effective wellhead protection program.

WELLHEAD PROTECTION PROGRAM OVERVIEW

Program Inception and Development

The 1986 amendments to the Federal Safe Drinking Water Act (SDWA) require states to develop and implement protection measures for municipal and community ground-water sources. As a result of this Act and the increasing awareness of the susceptibility of the Big Sioux aquifer to contamination, the Sioux Falls City Commission adopted multijurisdictional ordinances in April 1990 to establish Water Source Protection Overlay Districts. Minnehaha County adopted a similar ordinance in September 1990.

City ordinances were developed in a complementary fashion with the county ordinance to provide the desired areal coverage. Both city and county ordinances were needed to provide protection because some municipal water supply wells located outside the city boundaries fall under county jurisdiction. Both city and county ordinances are applicable to parts of the study area.

Wellhead Protection Program Objectives

The overall objective of the WHPP developed by Sioux Falls and Minnehaha County is to establish a comprehensive water resource protection plan that will prevent and reduce risks to the public water supply. Resources to be protected include the Big Sioux aquifer system, the Skunk Creek aquifer system, and associated drainage basins (City Ordinance 15.43.088). The ordinance establishes procedures and criteria for reviewing and restricting land uses having the potential to pollute water sources in designated Water Source Protection Overlay Districts.

The intent of the county's efforts is essentially the same as that of the city (County Ordinance MC16-90). Restrictions apply to land-use activities having the potential to contaminate aquifers and wellhead sites currently in use and those of potential future use as a public water supply. In addition, a Water Source Protection Overlay District has been formed. The overlay district is superimposed on all districts established by the county ordinance. If the Water Source Protection Overlay District imposes a greater restriction than the underlying zoning district regulations, the more stringent regulations are to be followed.

Implementation of Wellhead Protection

The WHPP is implemented through use of the Water Source Protection Overlay Districts (zoning ordinances). The city's ordinances apply within municipal boundaries; the county's ordinance applies outside municipal boundaries. However, the zoning authority in Minnehaha County allows for a joint-jurisdictional area 3 miles outside the municipal boundaries, in which both the city and county ordinances apply. The authority of the city versus that of the county is clear in the joint-jurisdictional area because ordinances were developed to complement one another.

The city's ordinances apply if new uses or changes in existing uses are proposed for a property within the Water Source Protection Overlay District. A Screening Committee composed of the City Planning Director, the City Public Health Director, and the Water and Wastewater Manager must review the proposed new or changed use prior to its onset. Residential land uses that are connected to the municipal sanitary sewer system are exempt from the screening process. At a minimum, the following information is considered:

- Type of new or changed use, development, or operation
- Quality, quantity, and concentration of a contaminant or hazardous material, and its physical, chemical, or infectious characteristics
- Storage or use of contaminants or hazardous materials
- Leak or spill prevention methods
- Contingency plans
- Adverse effects of contaminants or hazardous materials, including the probability that any leak or spill would contaminate ground water
- Compliance with other city, state, or federal regulations

The review by the Screening Committee can result in a recommendation of approval or denial to the City Planning Commission. Or the proposed new or changed use can be referred by the Screening Committee to another review process by an Environmental Impact Committee.

The Environmental Impact Committee must review all proposed new or changed uses that are not exempted from further review by the Screening Committee. Members of the Environmental Impact Committee are appointed by the mayor and city commissioners, and include representatives from the Health, Planning and Zoning, Engineering, Fire, Attorney, and Utilities Departments. Factors considered in this review are, at a minimum, the same as those examined by the Screening Committee. The Environmental Impact Committee will then recommend approval or denial of the proposed use to the City Planning Commission.

The county's ordinance becomes a factor if new uses or changes in existing uses are proposed for property within the Water Source Protection Overlay District outside the municipal boundary. A conditional use permit must be obtained from the County Planning Office. The county review process considers factors similar to the city review process.

By addressing only new uses or changes in use of property, the city and county are permitting reasonable and environmentally responsible growth, yet imposing reasonable and adequate safeguards on land use within the Water Source Protection Overlay Districts. All existing businesses and activities are allowed to continue in their present form if in conformance with existing federal, state,

and municipal regulations. Thus, any existing use of property within the Water Source Protection Overlay District that poses a potential threat to the municipal water supply, in spite of existing regulations, will continue to be a threat under the enacted ordinances. Any existing threats must be dealt with by enforcement of existing regulations pertaining to the storage, use, and disposal of potential contaminants and by monitoring as part of the wellhead protection efforts.

HYDROGEOLOGIC SETTING

The Sioux Falls airport wellfield is underlain by surficial glacial outwash deposits (sand and gravel) that are part of the Big Sioux aquifer. This aquifer occurs primarily in the Big Sioux River valley and some of its tributaries. The areal extent of the aquifer is about 770 square miles, including parts of northeastern South Dakota and extending to the southeastern corner of the state (Figure 4-1). The Sioux Falls airport wellfield lies within the Sioux Falls management unit of the Big Sioux aquifer. This management unit occurs in Minnehaha County and encompasses about 34 square miles. Ground water in this part of the aquifer occurs generally under unconfined conditions.

Outwash deposits associated with the Big Sioux aquifer occur primarily at or near land surface and are commonly mantled by only a thin layer of soil or fine-grained, alluvial sediments. In the study area, lithologic logs indicate that the aquifer is overlain by up to 12 feet of alluvial clay (average thickness, 6.7 feet). The approximate thickness of the outwash in the wellfield area ranges from 20 to 50 feet. The greatest thickness of saturated outwash is reported to be 48 feet near the weir on the diversion canal (Koch, 1982). Ground-water levels in the study area are about 10 feet below land surface. In the study area, the Big Sioux River is incised into the Big Sioux aquifer, but does not fully penetrate the aquifer.

The hydrogeologic relationship between the aquifer and the river in proximity to municipal well 34A is shown in Figure 4-3. There is direct contact between the Big Sioux River and the aquifer under generally unconfined conditions, even though the sand and gravel are overlain by a layer of alluvial clay. The hydraulic relationship between the river and the aquifer depicted in Figure 4-3 is representative of the entire study area. However, the thickness and extent of silt and clay on the river bottom and sides are not depicted because their occurrence is uncertain.

Recharge to the Big Sioux aquifer is from the infiltration of precipitation and from the inducement of river water. In calibrating a digital ground-water model for a large area of the Big Sioux aquifer, Koch (1982) assumed 6.9 inches per year (in/yr) of recharge from precipitation. Hedges et al. (1985) estimated the natural recharge rate to be 6.5 in/yr. Using a value of 6.5 for natural recharge and the actual pumping rate from the Sioux Falls management unit, Barari et al. (1989) showed that, on average, approximately 59% of the recharge to the entire management unit was from precipitation and 41% was due to induced recharge from the river. Because a clay layer overlies the aquifer in the wellfield, it is expected that this section of the aquifer receives less than 6.5 in/yr of natural recharge. Considering this situation, 6 in/yr of recharge was assumed for this research.

The significance of recharge from the Big Sioux river to the aquifer can be demonstrated. The area of the aquifer bounded by the diversion canal to the north and east, the Big Sioux River to the west, and the aquifer boundary to the south (Figure 4-2), is 4.3 square miles. If an annual recharge rate of 6 inches is assumed, then this part of the aquifer receives approximately 1,400 acre-feet

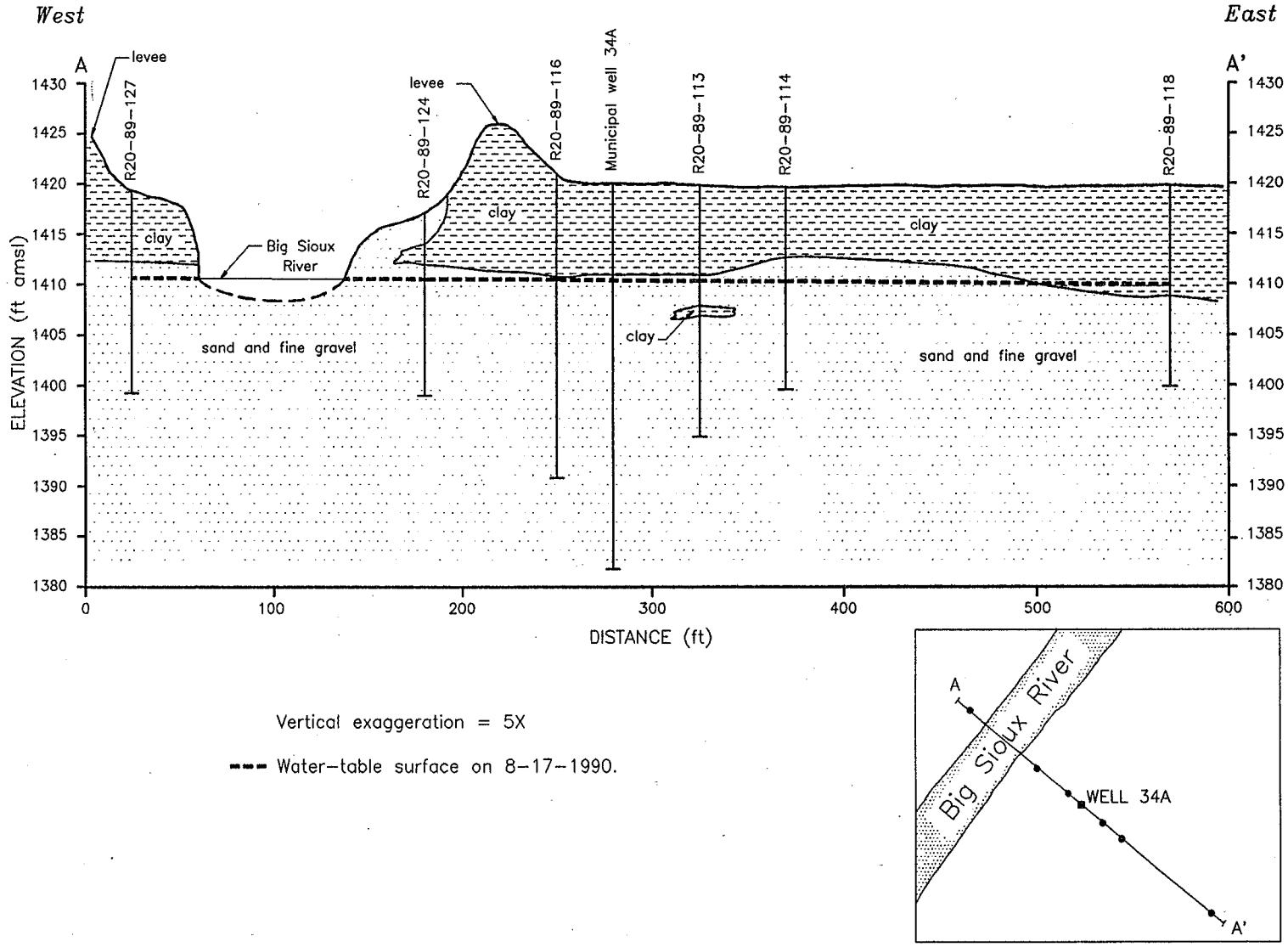


Figure 4-3. Hydrogeologic cross section near Sioux Falls municipal well 34A, South Dakota.

(acre-ft) of water from precipitation annually. However, in 1988, the city pumped approximately 6,700 acre-ft (2.2×10^9 gallons) of water from the airport wellfield, which was about 32% of the total water pumped by the city. No continuous decline in water levels was observed in this wellfield in 1988 or in 1989; therefore, recharge appears to have been equal to the discharge. In order to balance the water budget, it is assumed that this portion of the aquifer received an additional 5,900 acre-ft of water from sources other than the direct infiltration of precipitation. While some of this additional water may have come from portions of the aquifer outside of the 4.3-square-mile area, most probably came from the Big Sioux River and the diversion canal. Because the source of water in the drainage canal is the Big Sioux River, it was concluded that induced recharge from the river was 3.8 times the natural recharge in this portion of the aquifer. Consequently, in 1988, approximately 79% of recharge to the airport wellfield was from the river. This high recharge percentage demonstrates the significant contribution of the Big Sioux River to this wellfield. Loss of water from the river to the shallow ground-water system is also demonstrated by the fact that flow in the river has been observed to cease in the vicinity of the municipal wellfield during periods of low flow. The most recent occurrence of this was in July 1988.

Data on stream bed infiltration rates were presented by Jorgensen and Ackroyd (1973). On the basis of three aquifer tests, they estimated the infiltration rate to range from 0.5 to 1.0 foot per day (ft/d). The infiltration rate varies depending on the presence of silts and clays, which accumulate preferentially on the upstream side from the dam and weir across the Big Sioux River and the diversion canal.

PRELIMINARY WELLHEAD PROTECTION AREA

The WHPAs, or Water Source Protection Overlay Districts defined in the city and county ordinances, were delineated using the criterion of aquifer boundary and the method of hydrogeologic mapping (U.S. EPA, 1987). Specifically, the ordinances (City Ordinance 15.43.088; County Ordinance MC16-90) define an overlay district as "a geographical area overlying a geologic formation, group of formations, or part of a formation capable of yielding, storing, or transmitting a usable amount of ground water to wells or springs for domestic or animal use." This definition is appropriate for the city of Sioux Falls, where the aquifer occurs at, or very near, land surface. The Big Sioux River is included in the Water Source Protection Overlay District because of its presence within the described geographic area. One objective of this research is to better define the river-aquifer relationship and to determine its effect on the preliminary WHPA.

The boundaries of the Water Source Protection Overlay Districts within the study area do not correspond exactly to the geologic boundaries of the surficial outwash deposits shown in Figure 4-3. Rather, overlay district boundaries correspond to slightly larger political boundaries, which include the geologic features of concern. Most of the aquifer within the study area is included within the overlay district boundaries, with the exceptions of small portions of Sections 17 and 31 (Figure 4-4). For a 3-mile distance beyond the municipal boundaries, the overlay district falls under joint jurisdiction between the city and the county. The overlay district authorized by the county (not shown on Figure 4-4), which is outside the 3-mile joint-jurisdictional area follows geologic boundaries rather than political boundaries.

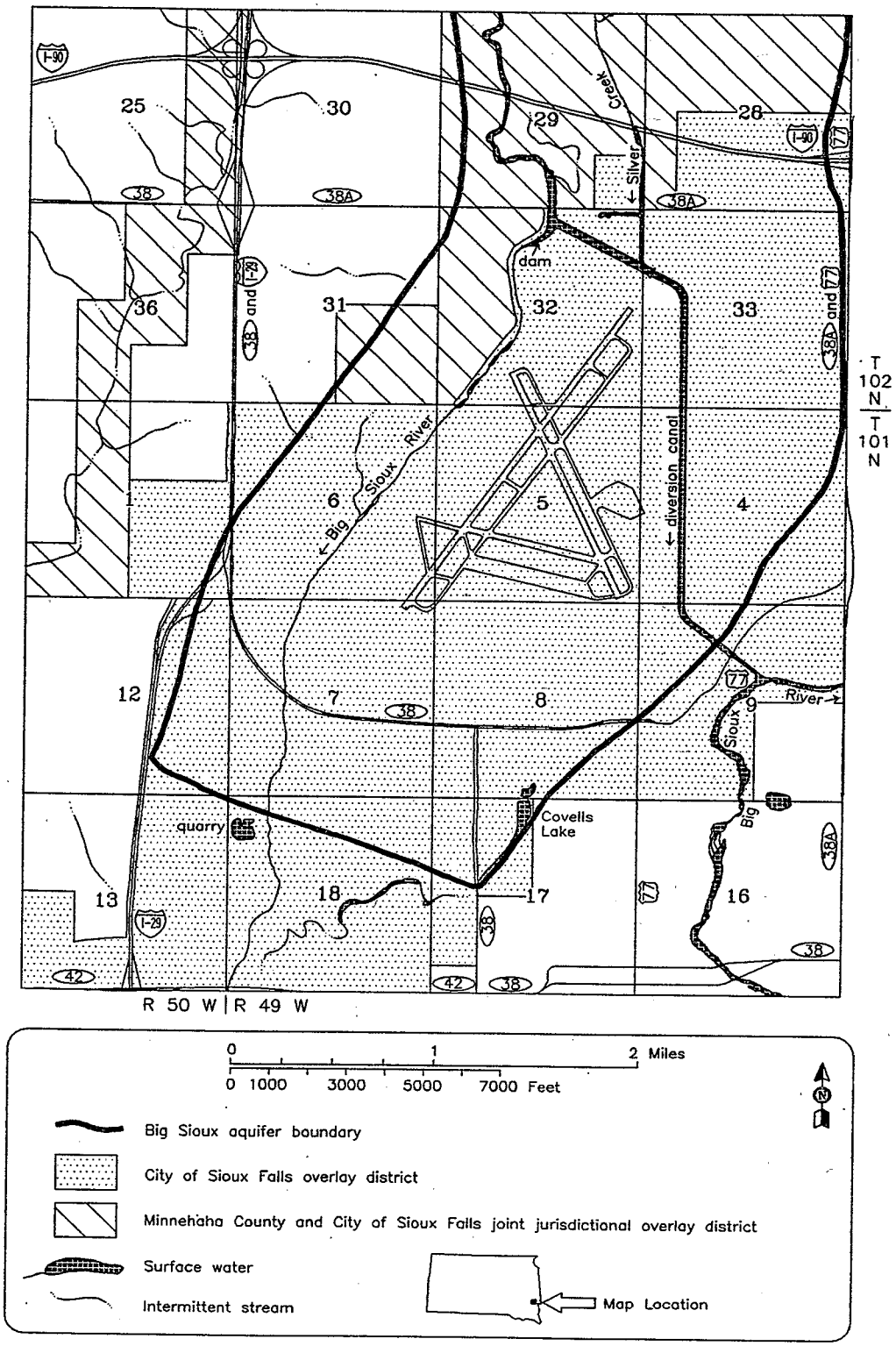


Figure 4-4. Water Source Protection Overlay Districts in and near the Sioux Falls airport wellfield, South Dakota.

SOURCE ASSESSMENT

Line Sources

From a hydrologic perspective, the Big Sioux River acts as a central drain for all known and potential sources of pollution in the aquifer, including surface runoff in the drainage basin and discharges to the river. The relationship of potential line sources of contamination to the airport wellfield is one important focus of this research. Both the Big Sioux River and the diversion canal act as potential line sources of contamination.

The Big Sioux River drains agricultural land on which fertilizers and pesticides are commonly applied; these chemicals are potential contaminants. Accidental chemical spills or illegal dumping of hazardous and toxic substances into the river are other types of potential pollution. Because of significant river infiltration to the aquifer, production wells at the airport wellfield are directly impacted by river water quality. As a result, the city of Sioux Falls needs to be aware of potential pollution that may enter the river upstream from the city.

Point Sources

Despite the presence of many potential point sources of contamination in proximity to the airport wellfield, the level of detail for a complete source assessment of each entity is not currently available. The threat of contamination from these sources is evident from the recent history of contaminant releases in the area. Several chemical releases involving both underground and above-ground leaks have occurred and are on record with several agencies including the Sioux Falls Fire Department, the local Emergency Planning Commission, and the SDDENR. The potential point sources adjacent to the airport wellfield (Figure 4-5) indicate the varied contaminants present:

- Industrial and commercial areas
- The South Dakota Air National Guard facility
- A petroleum pipeline belonging to the Williams Pipe Line Company
- The Sioux Falls Regional Airport
- A sand and salt storage area
- A decommissioned, municipal landfill
- A fire training center

The industrial and commercial areas south of the airport have existed for at least 40 years; those to the east of the diversion canal have existed for 15 to 20 years. Potential sources of groundwater contamination within the commercial and industrial areas in the vicinity of the airport wellfield are listed by facility type in Table 4-1. Many underground and aboveground storage tanks are located in these areas and contain petroleum-based products. A complete listing of the types of chemicals used at these facilities is beyond the scope of this research. However, for the types of facilities listed in Table 4-1, research completed by Rosenfeld (1990) indicates that volatile organic compounds (VOCs) are likely to be potential, predominant contaminants. In addition, the SDDENR has documented inorganic contamination of near-surface soils (primarily heavy metals) at one industrial facility.

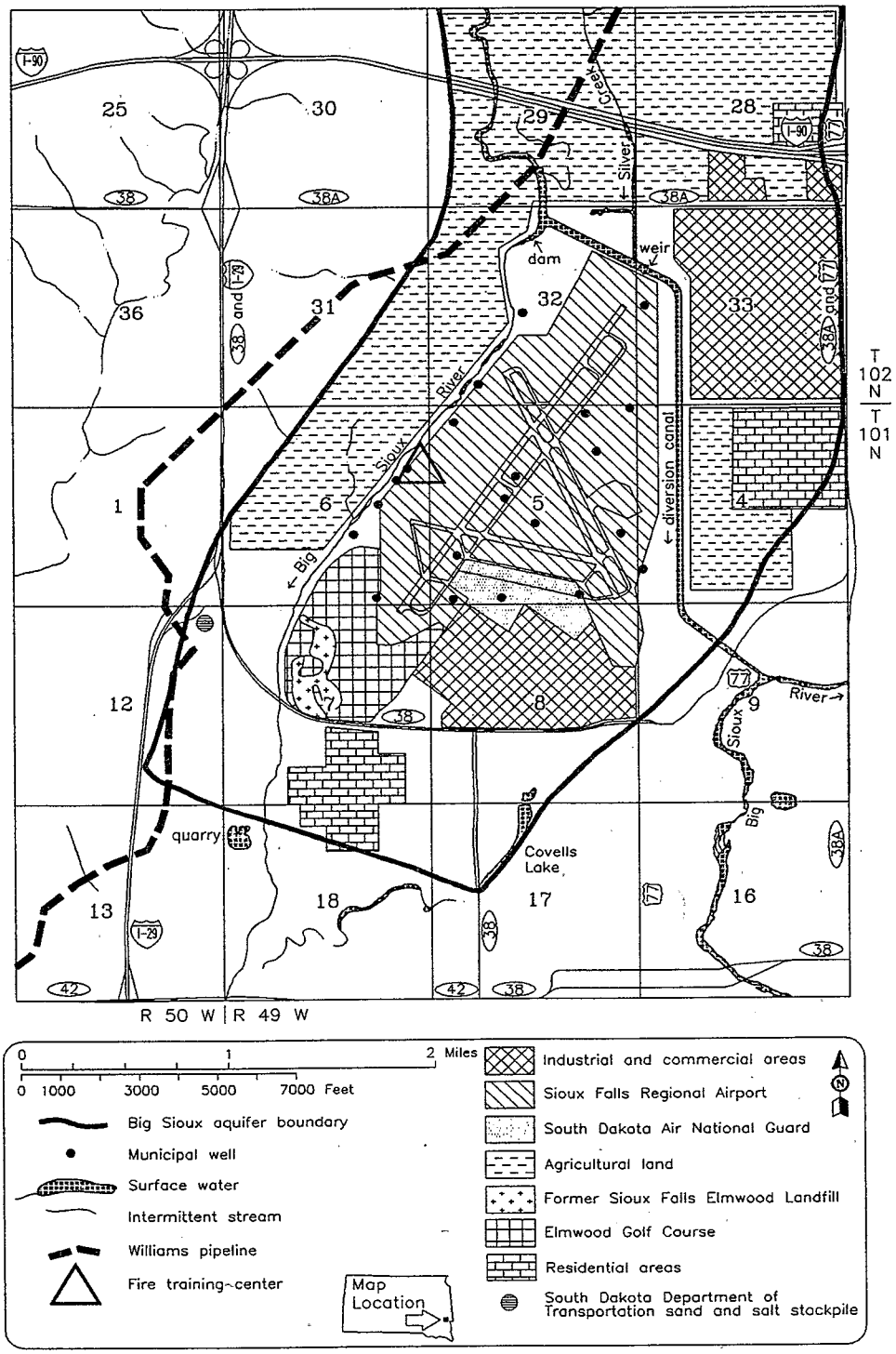


Figure 4-5. Areas of potential point- and nonpoint-source ground-water contamination by land-use type, Sioux Falls airport wellfield, South Dakota.

TABLE 4-1. COMMON SOURCES OF GROUND-WATER CONTAMINATION IN INDUSTRIAL AND COMMERCIAL AREAS NEAR THE SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA*

Commercial	Industrial
Agricultural equipment dealer	Chemical manufacture, warehousing and distribution activities
Airport	Electrical and electronic products and manufacturing
Auto repair shops	Electroplaters and metal fabricators
Car washes	Fiberglass and plastic ware manufacturing
Cemeteries	Fire retardant and foam products manufacturing
Construction areas	Jewelry and metal plating
Gas stations	Manufacturing and distribution sites for cleaning supplies
Golf courses (chemical applications)	Petroleum products production, storage, and distribution centers
Laundromats	Pipelines (such as oil and gas)
Lawn care services	Quartzite quarry
Machine and metalworking shops	Railroad tracks and yards
Paint shops	Storage tanks and pipes (both aboveground and underground)
Photography and printing shops	Toxic and hazardous spills
Road de-icing operations	
Road maintenance depots	
Toxic and hazardous spills	
Wells, operating and abandoned (such as water supply, injection, monitoring, and exploration)	

* Modified after Eschner et al., 1991.

The South Dakota Air National Guard facility, adjacent to the south edge of the airport (Figure 4-5), has been in operation since September 1946. Potential contaminants in use at this facility include aviation fuel (JP4), automobile fuels (diesel and unleaded gasolines), solvents, and degreasers. Trichloroethylene is a common constituent in the solvents and degreasers, and perchloroethylene is contained in a commonly-used cleaner.

The Williams Pipe Line Company owns and operates an 8-inch-diameter pipeline used solely to transport petroleum products. In the past, some agricultural chemicals were transported through this pipeline; however, that practice was discontinued. The pipeline crosses the aquifer and the Big Sioux River above the dam (Figure 4-5) and has been in operation since about 1946. It is buried about 3 to 5 feet below land surface, but is above ground where it crosses the Big Sioux River upstream from the airport wellfield and the city's surface-water intake structures. About 1 Mgd of gasoline, diesel fuel, fuel oil, and aviation fuel are currently transported through this line.

In January 1992, a small leak in the Williams pipeline was detected when petroleum product was found in the glacial till at a site approximately 3 miles northeast of the center of the study area. To date, about 170,000 gallons of product has been recovered; the total spill is estimated to range

between 200,000 to 400,000 gallons. This nearby contamination incident occurred in the glacial till adjacent to the aquifer, rather than in the aquifer. Nonetheless, it underscores the vulnerability of the aquifer and the need to implement source management practices to protect the airport wellfield.

The Sioux Falls Regional Airport opened in the mid-1930s and was greatly expanded during World War II. The primary chemicals utilized there are aviation fuels (Jet-A and 100-octane), runway de-icer (ethylene glycol), and herbicides [Round-up (mono-isopropylamine salt) and 2,4-D (2,4-dichlorophenoxy acetic acid)]. The herbicides are used to control weeds and to prevent growth of herbage around the airport fence.

The sand and salt storage area is owned and operated by the South Dakota Department of Transportation (SDDOT) and is located near the west edge of the aquifer (Figure 4-5). A sand and salt mixture used for road de-icing in the winter has been stockpiled in this location for many years. Historically, the salt and sand mixture was stored uncovered and unlined. Around 1987, runoff from the stockpile area endangered nearby trees because of uptake through the root system. Concurrently, higher levels of total dissolved solids (TDS) were detected in the ground water.

The Sioux Falls Elmwood Landfill was in operation in proximity to the airport wellfield from 1955 through 1958 (Figure 4-5). At that time, there were few restrictions regarding type and quantity of materials that could be disposed of in landfills. The operators reportedly accepted a variety of refuse ranging from municipal solid waste to scrap metal.

The Sioux Falls Fire Department maintains and operates a regional training center about 250 feet east of municipal well 34A (Figure 4-5) for departmental personnel. The main chemicals used at this facility are aviation fuels, to ignite some of the fires, and a fire-fighting foam (FC-203CE Light Water Brand Aqueous Film Forming Foam), to extinguish some of the fires.

Nonpoint Sources

Three nonpoint sources of potential contamination include residential areas, agricultural land, and the Elmwood Golf Course (Figure 4-5). Potential ground-water contaminants in the residential and agricultural areas are nitrogen-based fertilizers and pesticides used for control of weeds and insects.

The Elmwood Golf Course now covers the old Elmwood Landfill and includes additional land (Figure 4-5). The golf course and landfill existed concurrently from 1955 through 1958. This golf course was opened in 1923 and was redesigned to include more area in 1959 after the landfill was closed. Chemicals or products known to be applied to the golf course area are nitrogen-based fertilizers, a fungicide, Round-up, and 2,4-D.

References for Source Identification

A complete source assessment has not yet been completed by the city for the airport wellfield. However, many of the potential categories of sources discussed in this section were identified from the following information centers:

- Data from the Sioux Falls Fire Department. The Department maintains a comprehensive, computerized data base of addresses where hazardous materials are stored. This data base contains information such as type and quantity of hazardous, onsite materials.
- Records of the Sioux Falls Health Department
- Records of the Minnehaha County Civil Defense Office
- Aerial photographs at the Minnehaha County Office of Planning and Zoning
- Reports of spills and leaks on file at the SDDENR and in the files of local and state law enforcement departments
- Records of injection wells on file at the SDDENR
- Records of permitted discharges to either ground water or surface water on file at the SDDENR

RESEARCH MONITORING PROGRAM

DATA BASE LIMITATIONS

The types of background information available for this study include lithologic logs; recorded water levels; laboratory analytical data of general water chemistry; pumping records for municipal wells; aquifer maps; and regional, digital ground-water models that include the study area. The following technical reports pertaining to the area of study were consulted: Ellis and Adolphson (1969), Ellis et al. (1969), Iles (1992), Jorgensen and Ackroyd (1973), Koch (1982), Koch (1983), Lindgren and Niehus (1992), Steece (1959a, 1959b), Tomhave (In Preparation), and Vaughn and Ackroyd (1968).

Within the aquifer boundaries (Figure 4-2), there are approximately 118 test holes or wells for which lithologic logs are available. In addition, there are 21 municipal wells within the airport wellfield. Lithologic logs, water-chemistry data, and water-level data are stored in computer data bases controlled and maintained by the SDGS and the SDDENR. Other related data are found in reports and files of the city of Sioux Falls. The number and configuration of existing wells in the study area did not facilitate aquifer testing and sampling in proximity to the river. For this and other reasons, the existing monitoring network was considered inadequate for the research objectives.

Water-level records are available from chart recorders for each municipal well in the airport wellfield. These water levels are accurate to the nearest ±0.1 foot and are recorded relative to mean sea level. However, the accuracy of measuring-point elevations for some critical wells in the research monitoring network are questionable. Therefore, measuring-point elevations were resurveyed for select wells.

The quality of water entering the Sioux Falls water-purification facility is representative of the quality of water in the aquifer and in the Big Sioux River. However, data are not available for individual production or monitoring wells that demonstrate the spatial or temporal variability in water quality in proximity to the airport wellfield.

Pumping records in gallons per month are available for each of the municipal wells in the wellfield. These records are kept on file at the water-purification plant for the city of Sioux Falls. More detailed records of well discharges, accurate to gallons per minute, are required for ground-water flow modeling studies.

Surficial maps and cross sections showing the areal extent and thickness of the aquifer have been prepared by various investigators (Ellis et al., 1969; Jorgensen and Ackroyd, 1973; Steece, 1959a, 1959b), based on surficial geologic mapping, numerous lithologic logs, and water-level measurements. These maps are regional in scale and are, therefore, deficient in the detail necessary to meet the objectives of this site-specific research.

The resolution and predictive capabilities of existing digital ground-water models for the Sioux Falls management unit of the Big Sioux aquifer are regional in scale. The regional models are invaluable in demonstrating the overall interaction between the Big Sioux River and the ground-water system, but utilize too large a grid spacing to simulate detailed flow patterns within the study area.

MONITORING OBJECTIVES

The specific objectives of the research monitoring program are the following:

- To quantify the contribution of stream infiltration to water pumped from the municipal wells in the airport wellfield
- To calculate and field-verify the time of travel (TOT) between the Big Sioux River and municipal well 34A
- To estimate the TOT from the Big Sioux River and the diversion canal to selected municipal wells, under pumping conditions similar to conditions prevailing in 1988
- To estimate the TOT between selected potential point sources of contamination and some of the municipal wells under pumping conditions similar to those in 1988
- To address monitoring options (ambient trend, source assessment, and early warning detection) regarding ground-water contamination from the Big Sioux River, as well as general point source areas in the study area
- To examine the limitations of current wellhead protection guidelines in areas where there is extensive interaction between surface water and ground water
- To refine the delineation of the existing WHPP, if warranted

RESEARCH MONITORING TASKS

The following tasks were performed to satisfy the specific objectives of the research monitoring program:

- Installation and development of monitoring wells
- Surveying the elevations of monitoring wells, the Big Sioux River, and municipal well 34A
- Sampling and analysis for the following constituents:
  - Inorganic parameters from 10 monitoring wells, biannually, and from nine of these same wells a third time
  - Pesticides and VOCs from five monitoring wells, biannually
  - Inorganic parameters from the Big Sioux River, three times per year
  - Pesticides and VOCs from the Big Sioux River, biannually
- Measurement of water levels
- Aquifer testing using municipal well 34A as the pumping well
- Dye tracing at selected monitoring wells between the river and pumping municipal well 34A
- Ground-water flow modeling in the vicinity of the airport
- Ground-water monitoring program design
- Research documentation

DATA ACQUISITION

Monitoring Wells

New Monitoring Wells--

Vertical wells--Twenty-one vertical monitoring wells were constructed for this study at the locations shown on Figures 4-6 (five wells marked R-20-90) and 4-7 [16 wells, excluding 34A, the horizontal well (HW), and the vertical well in the river (VRW)]. Selected wells were used to monitor water levels. Others were used to monitor both water levels and ground-water chemistry. Water levels were monitored to determine spatial and temporal trends, as well as to calibrate the ground-water model.

Sixteen of the 21 monitoring wells were installed near municipal well 34A (Figure 4-7). These wells were used to accurately establish the configuration of the water-table surface in the immediate vicinity of the municipal well. All 16 wells were monitored during the aquifer and tracer tests that were conducted by pumping municipal well 34A. Ground-water elevations were carefully determined, at a detailed scale, between the Big Sioux River and municipal well 34A.

Two monitoring wells, R20-89-119 and R20-89-122 (Figure 4-7), are located approximately 1,000 feet northeast and southwest of municipal well 34A, respectively. These wells were used to obtain water-level data from an area of the aquifer that was not anticipated to be affected by pumping municipal well 34A.

Three wells, R20-89-126, -127, and -128 (Figure 4-7), located on the west side of the Big Sioux River near municipal well 34A, were used to document the gaining-versus-losing condition of the river under nonpumping conditions. Additionally, these wells were used to monitor water-level responses on the west side during the pumping of municipal well 34A.

Five monitoring wells, R20-90-1, -2, -3, -4, and -5 (Figure 4-6), were installed outside the immediate wellfield area for general characterization of water levels and the ground-water chemistry near the perimeter of the aquifer. Water-elevation data were needed in these areas to determine ground-water gradients and flow directions, and to support the modeling effort.

The 21 vertical wells were drilled using the hollow-stem auger method. The outside diameter of the auger was 8.5 inches. A plug was present in the lead auger to prevent native sediment from entering the hollow auger. Cuttings brought to the surface along the outside of the auger were examined and described. Lithologic logs and well-construction data are stored in the SDGS computerized data base. The geographical location and elevations of the top of the casing, the screened interval, and the ground surface for each well are listed in Table 4-2.

Prior to drilling, the work area of the drill rig, augers, and other downhole tools and equipment were cleaned with high-pressure hot water. This decontamination procedure was repeated between the drilling of each hole to minimize the possibility of cross-contamination between drill holes.

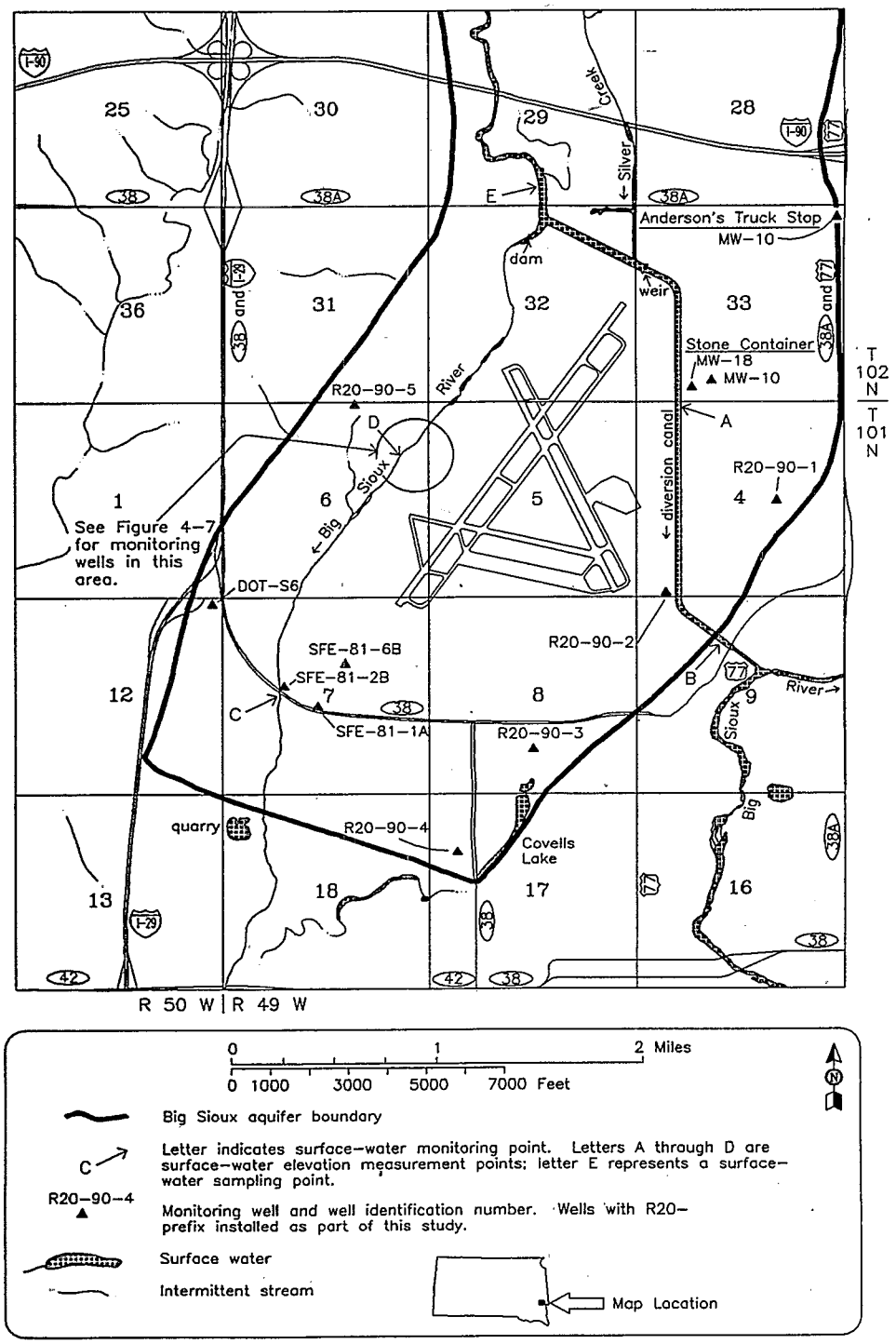


Figure 4-6. Locations of monitoring wells and surface-water monitoring points, Sioux Falls airport wellfield, South Dakota.

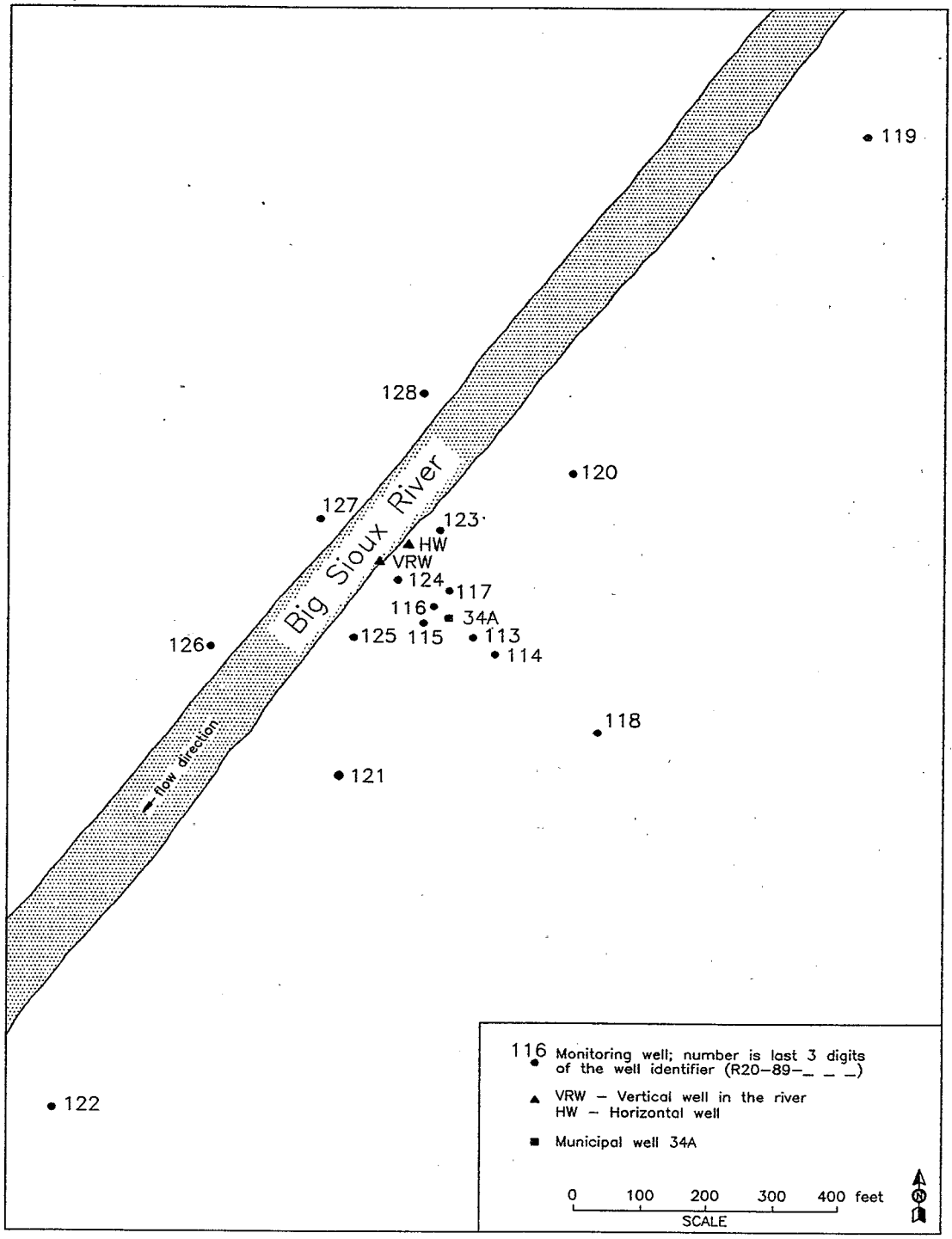


Figure 4-7. Locations of monitoring wells near municipal well 34A, Sioux Falls airport wellfield, South Dakota.

TABLE 4-2. MONITORING WELL LOCATIONS AND ASSOCIATED ELEVATION DATA, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA

Well	Latitude	Longitude	Top of Casing Elevation*	Screened Interval Elevations*	Ground Elevation*
New Wells					
R20-89-113	43°34'58"	96°45'10"	1421.94	1396.94-1411.94	1420.02
R20-89-114	43°34'58"	96°45'10"	1421.64	1401.64-1411.64	1419.78
R20-89-115	43°34'59"	96°45'11"	1421.59	1391.59-1411.59	1419.84
R20-89-116	43°34'59"	96°45'11"	1422.44	1392.44-1412.44	1420.80
R20-89-117	43°34'59"	96°45'11"	1422.18	1392.18-1412.18	1420.37
R20-89-118	43°34'57"	96°45'08"	1421.78	1401.78-1411.78	1419.96
R20-89-119	43°35'06"	96°45'02"	1424.26	1404.26-1414.26	1422.43
R20-89-120	43°35'01"	96°45'08"	1423.16	1403.16-1413.16	1420.17
R20-89-121	43°34'56"	96°45'13"	1421.26	1401.26-1411.26	1419.42
R20-89-122	43°34'51"	96°45'19"	1423.57	1403.57-1413.57	1421.67
R20-89-123	43°35'00"	96°45'11"	1418.57	1401.37-1411.37	1417.30
R20-89-124	43°34'59"	96°45'12"	1418.90	1401.00-1411.00	1417.01
R20-89-125	43°34'58"	96°45'13"	1418.60	1400.80-1410.80	1416.58
R20-89-126	43°34'58"	96°45'15"	1421.56	1401.56-1411.56	1419.57
R20-89-127	43°35'00"	96°45'13"	1421.26	1401.26-1411.26	1419.29
R20-89-128	43°35'02"	96°45'11"	1421.89	1401.89-1411.89	1420.00
R20-90-1	43°34'49"	96°43'04"	1422.65	1402.65-1412.65	1421.00
R20-90-2	43°34'24"	96°43'42"	1423.92	1403.92-1413.92	1421.87
R20-90-3	43°33'43"	96°44'27"	1421.67	1404.17-1414.17	1418.44
R20-90-4	43°33'15"	96°44'54"	1420.92	1403.32-1413.32	1418.95
R20-90-5	43°35'14"	96°45'29"	1429.16	1404.16-1414.16	1427.00
Vertical river well	43°34'59"	96°45'13"	1414.17	1409.00-1411.50	N/A†
Horizontal well	43°35'00"	96°45'12"	1417.93	1409.73-1409.90	1414.00

(continued)

TABLE 4-2. MONITORING WELL LOCATIONS AND ASSOCIATED ELEVATION DATA, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA (continued)

Well	Latitude	Longitude	Top of Casing Elevation*	Screened Interval Elevations*	Ground Elevation*
Existing Wells					
SFE-81-1A	43°33'54"	96°45'42"	1418.21	1370.71-1375.71	1418.00
SFE-81-2B	43°34'07"	96°45'53"	1417.27	1399.07-1404.07	1417.00
SFE-81-6B	43°34'03"	96°45'33"	1418.29	1399.70-1404.70	1418.00
DOT-S6	43°34'22"	96°46'16"	1431.43	1403.23-1418.23	1427.65
MW-10 (Anderson's Truck Stop well)	43°36'04"	96°42'47"	1431.20	1408.70-1418.70	1431.49
MW-10	43°35'21"	96°43'25"	1427.56	1404.06-1414.06	1425.60
MW-18 (Stone Container wells)	43°35'18"	96°43'33"	1422.89	1400.89-1410.89	1420.79

* All elevations are presented in feet above mean sea level.
† Vertical well located in the river; ground elevation is not applicable (N/A).

A schematic diagram of the vertical well design is shown in Figure 4-8. The wells were constructed with 2-inch-diameter, schedule 40, flush-threaded (ASTM F480-88A) polyvinyl chloride (PVC) casing and screen. All PVC parts were cleaned with high-pressure hot water and were wrapped in plastic prior to transport to the drill site.

These wells were installed by placing the well screen and casing through the center of the hollow-stem auger after the hole was drilled to the desired depth and the plug was removed from the auger. The auger was then removed from the drill hole, and the native sediment (sand and gravel) was allowed to collapse around the screen. A clean filter pack (washed and sorted; predominantly quartz sand) was added to the annular space. The top of the filter pack ranged from 0.9 to 2 feet above the top of the well screen. A layer of granular bentonite ranging in thickness from 0.9 to 2 feet was placed on top of the filter pack. Neat-cement grout placed on top of the granular bentonite extended up to the land surface. Concurrent with the placement of the cement grout, a locking steel well protector was installed over the well casing.

New monitoring wells were developed to ensure that all wells would respond physically and reproducibly to ground-water fluctuations and that they would yield representative water samples. New vertical monitoring wells were developed at about 10 gallons per minute (gpm) using a jet pump. The suction pipe for the pump was equipped with a check valve at the bottom to prevent backflow of water into the well. The suction pipe and check valve consisted of PVC material. Suction pipe and check valves were cleaned with high-pressure hot water, and were wrapped prior to transport to the site. A clean check valve and clean suction pipe were used for each well. The suction pipe was equipped with a valve to allow suction to be broken at the top of the well in order to purge water between the well and the point of water discharge. This prevented water that had come in contact with the pump from returning to the well.

A minimum of about 120 well volumes of water was removed from each well during the development process. Development continued, if necessary, until the temperature and electrical conductivity of the water had stabilized in three successive measurements taken at 5-minute intervals.

Horizontal well--A very shallow, horizontal monitoring well (Figure 4-9) was installed adjacent to the Big Sioux River (designated as HW in Figure 4-7) to monitor the river water level without actually placing the monitoring point in the river. However, clays and silts deposited on the bottom and sides of the river greatly reduced the hydraulic interconnection between the river and the horizontal well screen, thereby making this well a ground-water, rather than a surface-water, monitoring point. The horizontal well was, therefore, used as a monitoring point for the ground-water levels adjacent to the river bed.

The excavation for the horizontal well was made using a backhoe to a depth of about 2 feet below the river, or water-table level. Deeper excavation was prevented because of caving of the sand and gravel in the trench. After excavation of the trench, the horizontal well was placed on top of the native sand and gravel. A filter of sand and gravel was placed over the horizontal portion of the well. The remaining portion of the trench was filled with excavated material. The well was constructed of 2-inch-diameter galvanized steel with a 1-foot-long manufactured well screen at the end of the well adjacent to the river.

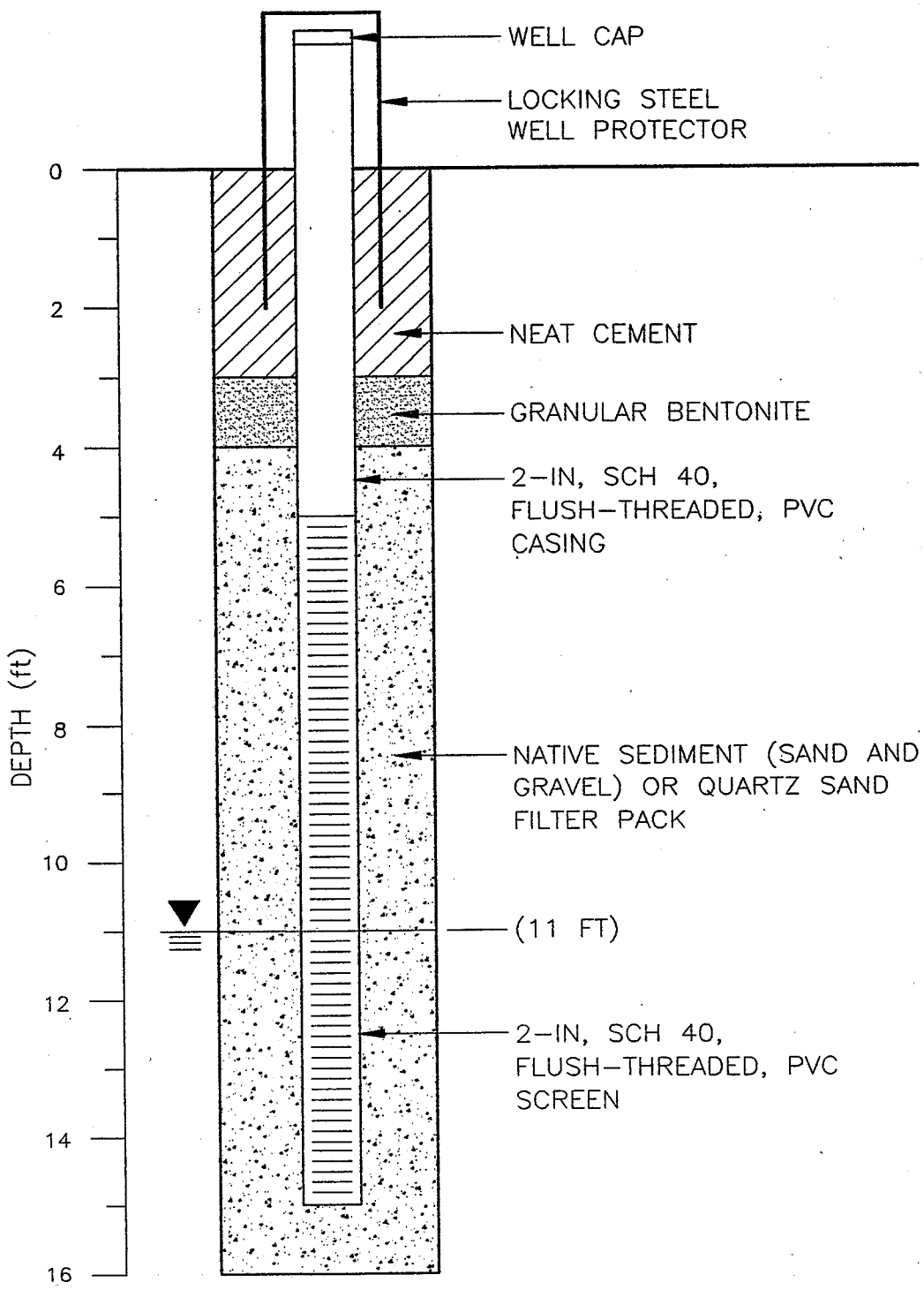


Figure 4-8. Generalized diagram of vertical monitoring well construction, Sioux Falls airport wellfield, South Dakota.

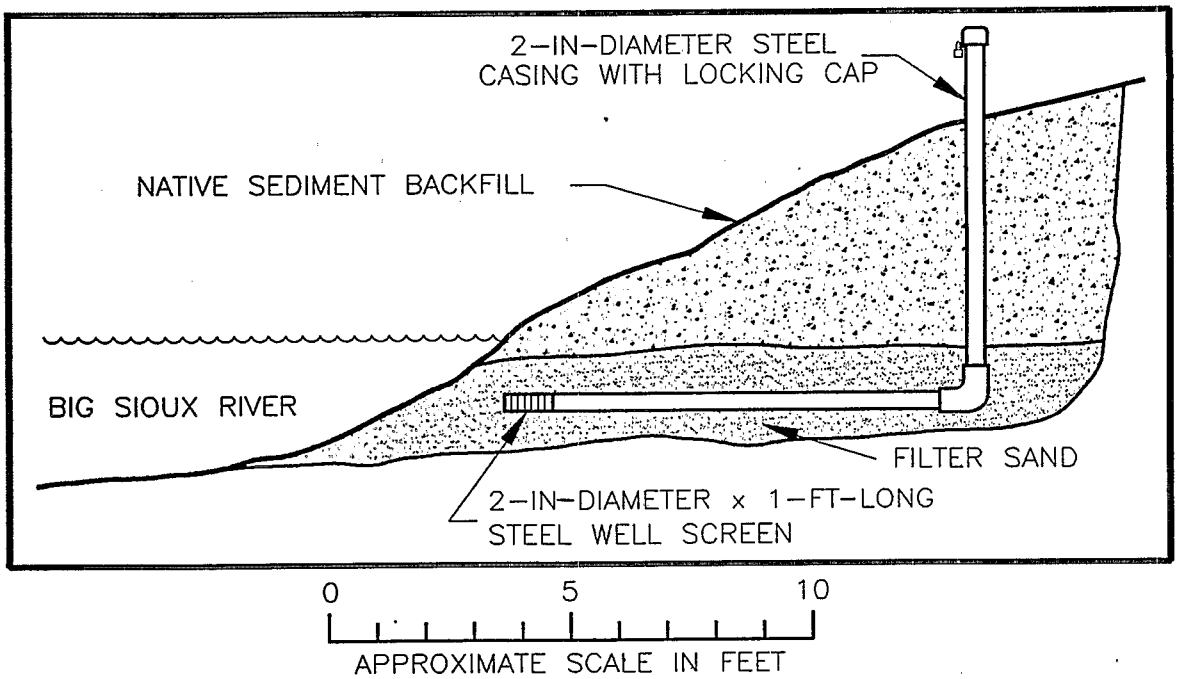


Figure 4-9. Generalized diagram of horizontal monitoring well construction, Sioux Falls airport wellfield, South Dakota.

Water levels in the river and aquifer were higher during installation of the horizontal well than later, when the aquifer test was conducted. At the beginning of the aquifer test, water levels had fallen to the point where the horizontal well had no water in it at the bottom of the vertical-well riser. This necessitated the installation of an alternative surface-water measuring point, which consisted of a vertical monitoring well in the river.

Vertical well in the river--The vertical well (designated as VRW in Figure 4-7) was installed near the river's edge when water levels had fallen to the point that the horizontal monitoring well was dry. The well in the river consisted of 1.25-inch-diameter, galvanized-steel casing and a 2.5-foot-long manufactured well screen. The screen, attached to the casing, was driven into the river-bottom sediments with a post driver until it was stable, and some of the well screen was exposed to the surface water. This well position allowed free entry of surface water into the casing.

Existing Monitoring Wells--

Six existing monitoring wells, installed for previous ground-water investigations, were used to obtain water-level data outside the immediate area of the airport wellfield. A seventh existing monitoring well was used to obtain water-level and chemistry data. The locations of these wells are shown in Figure 4-6 and are indicated by the prefixes DOT, SFE, and MW. Information on these wells is given in Table 4-2.

Three of these seven wells--Stone Container MW-10 and -18 and Anderson's Truck Stop MW-10--are located northeast and east of the airport. These wells were installed in 1986 and 1990 to investigate petroleum-contamination sites. The MW identifier was designated by the consultant who installed the wells. Three other wells--SFE-81-1A, -2B, and -6B--are located southwest of the airport. These wells were installed by the SDGS in 1981 to examine water levels around the old Sioux Falls Elmwood Landfill (SFE identifier), now occupied by the Elmwood Golf Course. The well to the west-southwest of the airport identified as DOT-S6 (Figure 4-6) was installed in 1987 by the SDDOT as part of a water-contamination problem involving incorrect storage of road de-icing salt. Of these seven wells, only well DOT-S6 was used for water sampling. The others were not sampled, either because they were at known contamination sites or because the wells were not of acceptable construction for the collection of representative water quality samples.

Water Quality Sampling

During the study period, 33 water samples were collected for analysis of inorganic constituents, and 12 water samples were collected for analysis of pesticides and VOCs. With one exception, water samples for inorganic analysis were collected during three sampling events in 1990: January 15 through 17, April 9 through 11, and July 9 and 10. Water samples for pesticide and VOC analyses were collected during two sampling events in 1990: January 15 through 17 and July 9 and 10. These sampling dates were chosen to allow characterization of the ground-water chemistry during various seasons and human-related activities.

Chemical data from analytical results were used to: (1) characterize the general ground-water quality, (2) document any natural changes in ground-water quality, and (3) examine induced flow of river water to the aquifer. Analytical results are stored in the SDGS computer data base.

Sampling Procedures--

Water samples were collected and handled in accordance with the *South Dakota Geological Survey Water Sampling Manual* (Coker et al., 1988) and according to the following procedures. The manual is an unpublished document intended for in-house use at the SDGS.

Prior to sampling, a minimum of 38 well volumes of water were purged from the monitoring wells concurrently with the measurement of water temperature and electrical conductivity. Purging of water continued, if necessary, until the water temperature and electrical conductivity had stabilized in three successive measurements taken at 5-minute intervals. The pH parameter was not monitored during the purging process because of the large volume of water that was removed from each well concurrently with the monitoring of water temperature and electrical conductivity. An exception to this purging procedure occurred at well R20-89-122 during the July sampling event. In this instance, equipment failure required that a Teflon bailer be used, and only 3.5 well volumes were removed prior to sampling.

Monitoring wells were purged by means of a jet pump. The suction pipe for the pump was equipped with a check valve at the bottom to prevent backflow of water into the well. The suction pipe was also equipped with a valve to allow suction to be broken at the top of the well in order to purge water between the wellhead and the point of water discharge. This prevented water that had

come in contact with the pump from returning to the well. Suction pipe and check valves were constructed of PVC, and were cleaned with high-pressure hot water and wrapped prior to transport to the site.

Water samples for inorganic analysis were collected either from the discharge point on the jet pump or from laboratory-cleaned Teflon bailers. All water samples to be analyzed for pesticides and VOCs, and some samples for inorganic analysis, were collected using bailers. The bailers were attached to stainless-steel wire used to raise and lower them. The bailers and stainless-steel wire were handled with clean gloves during sampling activities to minimize the possibility of introducing contamination to the water sample. Before connection of the stainless-steel wire to the bailer and lowering of the bailer down the well, the wire was rinsed with distilled water. The bailer was then attached to the wire and lowered. While the bailer and wire were being lowered into the well for the first time, the wire was wiped with a clean towelette wetted with distilled water. The first bailer of well water was dumped to waste, thereby providing a rinse of the laboratory-cleaned bailer. Subsequent water withdrawn from the well with the bailer was used for sampling purposes.

Water samples collected for analysis of pesticides and other VOCs were analyzed by the University of Iowa Hygienic Laboratory, Iowa City, Iowa. Samples were placed in containers supplied by the laboratory and were shipped in accordance with requirements of the laboratory.

Laboratory Quality Assurance and Quality Control--

The SDGS Basic and Analytical Studies Laboratory analyzed all water samples for inorganic parameters. The University of Iowa Hygienic Laboratory analyzed water samples for selected pesticides and VOCs and is certified by EPA for all methods used in the study. The SDGS Basic and Analytical Studies Laboratory is certified by EPA for the analysis of arsenic, fluoride, nitrate and nitrite as nitrogen, and selenium. The SDGS Basic and Analytical Studies Laboratory participates in the U.S. Geological Survey's (USGS) Analytical Evaluation Program twice per year for the analytical methods used in this study. Both laboratories adhere to strict quality assurance and quality control (QA/QC) procedures.

Sampling for Inorganic Analysis--

The sampling plan for inorganic constituents was designed (1) to examine spatial and temporal trends in ground-water and surface-water quality, and (2) to provide baseline data for long-term monitoring. Ten monitoring wells and a surface-water sample site were chosen for sampling of inorganic constituents. Five of these wells (R20-89-118, -119, -122, -124, and -127) are in proximity to municipal well 34A, which is the focal point of the study (Figure 4-7). Five other wells (R20-90-1, -2, -3, -4, and -5) were located outside of the airport wellfield (Figure 4-6). The surface-water sample site was located immediately upstream from the divergence of the Big Sioux River and the diversion canal (Figure 4-6).

The sampling plan included three events during the study period. The sampling events occurred in 1990 on January 15 through 17, April 9 through 11, and July 9 through 10. The network was successfully sampled during the first two sampling events. However, after the April sampling event, well R20-90-124 was struck by construction equipment and sufficiently damaged so that it could not be used for future sampling. Well R20-90-125 (Figure 4-7) was substituted for the damaged well

in the July sampling event. No water sample was taken from well R20-90-1 during the July sampling event. Other than these two differences during the July sampling event, each water-sampling point was sampled three times for inorganic constituents.

Analyses for inorganic parameters were performed for the parameters listed in Table 4-3. These parameters can be categorized as major cations and anions, minor constituents, and water quality indicators such as specific conductivity, oxidation-reduction potential (Eh), laboratory pH, and so on. The major cations and anions listed in Table 4-3 were chosen for analysis because they comprise the bulk of the constituents in the water and can, therefore, be used for water typing. The minor constituents were chosen for analysis because they are important in characterization of ambient water quality. Selenium and arsenic are of particular concern because they are naturally occurring, sometimes at significant concentrations, in glacial outwash in eastern South Dakota. The origin of the selenium and arsenic is believed to be from Cretaceous rocks within the glacial outwash.

TABLE 4-3. INORGANIC PARAMETERS ANALYZED IN WATER SAMPLES, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA

Major Cations and Anions	Minor Constituents	Water Quality Indicators
Bicarbonate	Arsenic	Specific conductivity
Calcium	Fluoride	Eh
Carbonate	Iron	Field pH
Chloride	Manganese	Hardness
Magnesium	Nitrate and nitrite as nitrogen	Laboratory pH
Potassium	Selenium	Total alkalinity
Sodium		Total dissolved solids
Sulfate		

The list of parameters given in Table 4-3 represents a baseline water quality characterization suite. Potential contaminant sources identified in the assessment process did not greatly influence the choices of parameters for analysis. Key indicators of contamination will be generated for the long-term monitoring program from the baseline water quality data and from more detailed source assessment information. These parameters are selectively chosen from long-term water quality data and tailored to monitor potential, or existing, sources of greatest threat.

Sampling for Pesticides and Volatile Organic Compounds--

The sampling plan for pesticides and VOCs was designed in a manner similar to that for the inorganic constituents. Five monitoring wells and a river site were chosen for sampling organic constituents. The five wells selected are a subset of the 10 wells sampled for inorganic constituents. Wells R20-89-119, -124, and -127 were chosen to provide detailed information near municipal well 34A (Figure 4-7). Wells R20-90-2 and -3 were chosen to provide information from more distant parts of the aquifer (Figure 4-6). The river sampling site is the same as that used for sampling the inorganic constituents (Figure 4-6).

The sampling plan designated two sampling events. Because of budget constraints, five fewer wells and one less sampling round were scheduled for the pesticides and VOCs than for inorganic constituents. Sampling occurred on January 15 through 17, 1990, and July 9 and 10, 1990, concurrently with the first and last sampling events for inorganic constituents. As with the sampling for inorganic constituents, well R20-89-125 was substituted for damaged well R20-89-124 in the July sampling event.

Analyses were performed for the pesticides and VOCs listed in Table 4-4. The pesticides and VOCs were chosen to provide background information on water quality. Pesticide selection was based primarily on known usage and because some of these pesticides are part of an analytical scan for a particular chemical class. Volatile organic compound analysis is commonly employed as a screening technique in association with point-source areas.

Water-Level Measurement

Monitoring-point elevations were established for all stations (land, surface water, and wells) used in this study, based on existing benchmarks established by the Coast and Geodetic Survey and the USGS. Elevations were reduced to a common datum to allow direct comparison of lithology and water levels at various locations. All elevations are relative to mean sea level, thus are directly comparable with published topographic maps. Reference elevations are accurate to the nearest ±0.1 foot. This accuracy is less than that of the water-level measurements (±0.01 foot) due to an accumulated error in surveying.

Water levels were measured to define the configuration of the water-table surface relative to the river elevation during different seasons and under different pumping conditions. These data are integral to understanding the dynamics of the surface-water and ground-water systems in the wellfield area.

Water levels were measured with an electric water-level indicator to an accuracy of ±0.01 foot. Prior to measurement, the probe on the end of the water-level indicator was rinsed with distilled water. A clean towelette, wetted with distilled water, was then used to wipe the electrical tape as it was lowered into the monitoring well. The same process was repeated with a clean towelette as the tape was drawn out of the well.

Ground-water levels were measured in some wells by using a pressure transducer and data logger. The pressure transducer has a pressure range of either 0 to 15 pounds per square inch (psi) or 0 to 5 psi. Both were utilized in the project. The transducers have a linearity of 0.05%, with a resolution of better than 0.1%. They are compatible with the data logger, and automatically compensate for barometric changes on the water column. Each data logger has 32 kilobytes of memory and is capable of storing approximately 16,000 water-level data points. Pressure transducers and attached cables were rinsed with distilled water and wiped with a clean towelette before being placed in a well.

During aquifer testing, data loggers nearest the pumping well (municipal well 34A) were set to collect data on a logarithmic time scale in the early stages of the test. Loggers more distant from the pumping well collected data on a linear time scale throughout the test. Data collection at these times

TABLE 4-4. PESTICIDES AND VOLATILE ORGANIC COMPOUNDS ANALYZED IN WATER SAMPLES, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA

Pesticides		Volatile Organic Compounds
Trade Name	Chemical Name	
Atrazine	Atrazine	Chloromethane
Bladex	Cyanazine	Bromomethane
Dual	Metolachlor	Vinyl chloride
Lasso	Alachlor	Chloroethane
Sencor	Metribuzin	Methylene chloride
Sutan	Butylate	Acetone
Treflan	Trifluralin	Carbon disulfide
Counter	Terbufos	1,1-Dichloroethene
Dyfonate	Fonfos	1,1-Dichloroethane
Lorsban	Chlorpyrifos	1,2-Dichloroethene (total)
Mocap	Ethoprop	Chloroform
Thimat	Phorate	1,2-Dichloroethane
Furadan	Carbofuran	2-Butanone (methyl ethyl ketone)
2,4-D	2,4-D	1,1,1-Trichloroethane
Silvex*	2,4,5-TP	Carbon tetrachloride
Amiben*	Chloramben	Vinyl acetate
Banvel*	Dicamba	Bromodichloromethane
		1,2-Dichloropropane
		cis-1,3-Dichloropropene
		Trichloroethene
		Dibromochloromethane
		1,1,2-Trichloroethane
		Benzene
		trans-1,3-Dichloropropene
		Bromoform
		4-methyl-2-Pentanone (MIBK)
		2-Hexanone
		Tetrachloroethane
		1,1,2,2-Tetrachloroethane
		Toluene
		Chlorobenzene
		Ethylbenzene
		Styrene
		Xylene (total)
		p-Dichlorobenzene

* Analyzed in July sampling event only.

provided for acquisition of the most usable data. Data loggers and pressure transducers proved to be very beneficial in this study by eliminating much of the staff time required for water-level measurements.

Aquifer Test

An aquifer test was conducted using municipal well 34A as the pumping well. Aquifer testing was conducted to provide information regarding (1) the configuration of the water-table surface within the area of influence of municipal well 34A under pumping conditions, (2) the characteristics of the aquifer, and (3) the interaction of surface water and ground water. These data were used to calculate ground-water flow velocities between the river and the pumping well.

The pumping portion of the aquifer test was conducted from noon on May 15, 1990 through 2:00 p.m. on June 7, 1990 (23 days and 2 hours). The test was scheduled to last 3 days, but was greatly extended to accommodate dye-tracing tests. Specifically, it was extended to accommodate a second injection of dye during a tracer test conducted concurrently with the aquifer test. The recovery phase of the aquifer test lasted for 72 hours after shutdown of the well on June 7, 1990.

Data obtained for calculation of aquifer transmissivity and storativity values were limited to the first 23 hours of the test. This is because the initial pumping rate of 396 gpm was lowered to 388 gpm to prevent the water level in the pumping well from reaching the top of the well screen and because the river level began to rise and impact ground-water levels. Sioux Falls city officials required lowering of the pumping rate because of their concerns that the screen would become aerated, causing well maintenance problems.

The depth to water in the monitoring wells and municipal well 34A (pumping well), and the surface elevation of the Big Sioux River were also measured during the test. Pressure transducers and data loggers were used on wells R20-89-113 through 118 and for wells R20-89-120, -123, -125, and -128 (Figure 4-7). Well R20-89-117 was measured manually for a portion of the test because this well was used for the first dye injection during the tracer test. Wells R20-89-119, -121, -122, -124, -126, and -127 were measured manually during the aquifer test. Monitoring wells outside of the area shown on Figure 4-7 were also measured during the test to provide background information on aquifer conditions during the test.

Dye-Trace Test

A tracer test was conducted in conjunction with the aquifer test to determine ground-water travel times between the river and municipal well 34A. This information was used to estimate travel times from the river to other municipal wells.

The first step in preparing the tracer test was to select the appropriate tracer(s) to use in the hydrogeologic setting of the study area. The main criteria were:

1. *Toxicity of the tracer.* It was necessary to select a nontoxic, nonradioactive trace material, even though the tracer would be significantly diluted before reaching the water-purification plant. The tracer was injected into an area of the aquifer used as a principal wellfield for the city and exited the aquifer through municipal well 34A, which discharged to the city's water purification plant.

2. *Adsorptivity of the tracer.* It was desirable to choose a tracer that would not readily adsorb to the aquifer materials. Although the aquifer does not have a relatively high clay content, the percentage of silt and clay present could enhance adsorption. Thus, a relatively conservative tracer was desired.

3. *Ability to analyze for the tracer onsite.* Field determination of tracer concentrations was needed so that sampling frequencies could be adjusted to maximize the amount of useful information. For example, sampling frequency increases at the time a tracer concentration begins to peak in order to define the shape and duration of the concentration curve. Additionally, because the SDGS owns a spectrofluorometer, it was highly advantageous to select a fluorescent tracer.

4. *Cost of tracer materials and analyses.* Some tracer materials are extremely expensive, while others are relatively inexpensive. Some tracer materials cost more to analyze than others. Because a relatively large dilution factor was expected between the time the tracer was injected and the time it arrived at municipal well 34A, a considerable amount of tracer needed to be injected. Thus, the unit cost of the tracer material needed to be minimized. Additionally, analyses of tracer concentration would be performed by a contractual laboratory to check the reliability of the field-determined concentrations. Because a fairly large number of analyses were anticipated, the unit cost per analysis for the selected tracer had to be reasonable.

5. *Availability of the tracer.* The tracer had to be readily available from a known source in the event that more was needed in unanticipated circumstances.

These five criteria were best met by the use of fluorescein dye. Fluorescein dye was chosen because of the following unique characteristics:

- Low toxicity (Gaspar, 1987)
- Very low adsorptivity property (Gaspar, 1987)
- Fluorescence content, which makes it suitable for onsite analysis with the spectrofluorometric equipment available from the SDGS
- Inexpensive cost per unit of material
- Inexpensive analytical cost
- Availability from the laboratory contracted to perform the analyses for tracer concentration

In addition, fluorescein dyes can be detected in the nanograms per liter range. This was also a desirable property because dilution was expected to be significant between the points of dye injection and detection.

Determination of the injection point(s) for the dye was the second step in preparing for the tracer test. Because the objective of the tracer test was to determine ground-water travel times from the river to the well, the dye had to be injected as close to the river as possible. The most logical injection points were the monitoring wells between the river and municipal well 34A. Two injection points were chosen. The first injection occurred in monitoring well R20-89-117, located approximately 40 feet north of municipal well 34A (Figure 4-7). Injection of dye close to the pumping well permitted rapid detection and an opportunity to field-test sampling frequencies and analytical equipment. The second injection point was monitoring well R20-89-123, located approximately 140 feet north of municipal well 34A. This monitoring well is very close to the river and provides a representative point by which to determine the travel time from the river to the pumping well, 34A (Figure 4-7).

For the tracer tests, 1 pound of fluorescein dye was obtained in powder form. The dye was divided into 0.25-pound allotments, and each was mixed in approximately 0.5 gallon of distilled water and stored in a polyethylene container in the dark to limit degradation of the dye. After pumping began at municipal well 34A, water levels were monitored in the two injection wells to ensure that the cone of influence had reached these wells. Approximately 3 hours after pumping began, one of the 0.5-gallon containers of dye was injected into monitoring well R20-89-117. A length of Tygon tubing just long enough to reach into the screened area of the monitoring well and remain below the water table was attached to a plastic funnel. The dye was poured into the funnel and was flushed with approximately five well volumes of water to help surge the dye from the well casing into the aquifer. Approximately 72 hours after the first dye injection, field testing with a spectrofluorometer showed that the majority of the dye pulse had passed through municipal well 34A. At that point, the remaining three 0.5-gallon containers of dye were injected into monitoring well R20-89-123 in the same manner as in monitoring well R20-89-117.

Ground-water samples were collected at variable frequencies that coincided with the anticipated pulse of dye moving through the aquifer. The first arrival time of the dye was estimated roughly using the pumping rate of the well (the volume of water that would be removed from the aquifer) and an estimated porosity of the aquifer. A garden hose attached to a spigot on the discharge line of municipal well 34A provided access to the water for sampling.

Two methods of sampling were used. A Y-adaptor was placed at the end of the hose. Attached to one end of the adaptor was a PVC coupler in which a charcoal filter pack could be stored. As ground water passed through the coupler, dye was adsorbed onto the charcoal pack. The purpose of the charcoal pack is to adsorb and accumulate dye concentrations. Without use of the charcoal packs, the dye concentrations may be below the detection limit of a spectrofluorometer. After the charcoal pack was removed from the PVC coupler, it was sealed in a plastic bag, labeled with the date and time of sampling, and stored on ice in a cooler (in the dark) until it could be shipped to a laboratory for analysis. Although no specific holding times were specified for either the water samples or the charcoal packs, it was advisable that analyses be performed within 2 weeks of the sampling time (Tom Aley, Ozark Underground Laboratory, Protem, Missouri, personal communication, 1990). It is also very important to keep the samples in a cool, dark environment to minimize dye degradation.

The other opening on the Y-adaptor was used to fill a 60-milliliter (ml) polyethylene bottle with discharge water from the well. Ground-water samples were obtained by rinsing the 60-ml bottle

several times with discharge water and then filling the bottle completely. The sample bottle was labeled with the date and time the sample was collected, and stored on ice in a cooler until it could be shipped to a laboratory. The same procedure was performed for a second 60-ml bottle, which was analyzed onsite by SDGS personnel.

Sampling began immediately after municipal well 34A was turned on, and prior to injection of the dye, to determine background fluorescence concentrations in the aquifer. Background data are necessary to document the presence of chemicals that can contribute to fluorescence in ground water (for example, laundry detergent). Water samples were taken at 30-minute intervals, and charcoal packs were changed at 60-minute intervals. After the first dye injection, water samples were taken approximately every 60 minutes, and charcoal packs were changed every 60 minutes. After onsite analysis of water samples confirmed that the dye pulse had arrived, sampling frequency was increased to better define the dye-concentration curve. After several hours it became apparent that the second dye pulse would take a few days to reach municipal well 34A. Thus, the sampling frequency was decreased without compromising the data resolution necessary to accurately define the dye-concentration curve.

Analysis of the discharge water from municipal well 34A for dye was performed in three different ways.

1. *Onsite analysis of samples by the SDGS.* Ground-water samples were analyzed onsite by SDGS personnel with a fluorescence spectrofluorometer. The fluorometer was set up inside the well house at municipal well 34A. It was calibrated, initially, with fluorescein standards that bracketed the anticipated concentrations and was periodically recalibrated to correct for instrument drift. The instrument was also recalibrated when the concentration of the dye in the water approached the concentration of the upper standard. The detection limit using this instrument was about 2 micrograms per liter (µg/l). The field-analytical setup proved to be invaluable in adjusting sampling frequencies and determining the duration of the tracer test.

2. *Laboratory analysis of samples by the Ozark Underground Laboratory.* Some duplicates samples were sent to the Ozark Underground Laboratory, primarily as a check against the validity of onsite analyses. This laboratory used a spectrofluorophotometer that scanned both excitation and emission spectra. The detection limit for the fluorescein in the water samples was about 0.0005 µg/l.

3. *Laboratory analysis of charcoal packs by the Ozark Underground Laboratory.* Charcoal pack samples were also sent to the Ozark Underground Laboratory for analysis. Upon receipt, the packs were placed in a beaker, and an eluting solution consisting of 5% aqua ammonia and 95% isopropyl alcohol was added. After the charcoal pack was soaked in the eluting solution for 1 hour, the elutant was poured off into a separate container. Approximately 3 ml of the elutant was then pipetted into a cuvette. The cuvette was placed in a spectrofluorophotometer, where it was synchronously scanned for both the excitation and emission spectra, from 417 to 667 nanometers. The charcoal pack data were very valuable in determining time of first arrival of the dye because the concentrations were initially too low to be detected in the water samples.

DATA INTERPRETATION

Water Levels

The temporal and spatial variability of the water-table surface within the study area are partially depicted in the following four illustrations. Two of the figures portray the entire wellfield area (Figures 4-10 and 4-11), and two provide close-up views of the municipal well 34A area (Figures 4-12 and 4-13). Ground-water and surface-water elevations used in constructing these figures are presented in Table 4-5. Water levels depicted in Figures 4-10 and 4-12 were obtained on August 2, 1990; those for Figures 4-11 and 4-13 were obtained on August 17, 1990. Examination of the water-table contour maps for these dates reveals that ground-water flow directions can change significantly over a short period of time (say, 2 weeks) in some areas while remaining essentially unchanged in other areas.

Surface-water recharge from the Big Sioux River and the diversion canal to the Big Sioux aquifer in the airport wellfield area remained essentially unchanged during this period (Figures 4-10 and 4-11). Also, the ground-water flow direction is generally to the north in the area just south of the wellfield and to the west in the area just east of the wellfield (Figures 4-10 and 4-11). Industrial and commercial facilities exist in these parts of the aquifer (Figure 4-5). Ground-water recharge areas to the wellfield are coincident with areas of industrial and commercial development. This situation underscores the potential contamination threat of these areas to the public water supply and, therefore, the need for adequate regulation and monitoring of sources.

The relationship between the river and the ground water on the northwest side of the airport differed between August 2 and August 17, 1990 (Figures 4-10 and 4-11). On August 2, 1990, relatively high flow conditions existed in the river, so that the river level was higher than the ground water on either side. In contrast, on August 17, 1990, ground water was recharging the river on the northwest side and the river was recharging the ground water on the southeast side. This relationship is more easily seen in Figures 4-12 and 4-13, which depict close-up views of the well 34A area.

When water levels were measured concurrently in the vertical well in the river and the horizontal well near municipal well 34A, differences were discovered between elevations in the two wells. Instead of measuring the river level, the horizontal well was providing data on the ground water level immediately adjacent to the river bed. For example, a head difference of 1.32 feet was measured on August 2, 1990, with the river level being higher than that of the ground water. These measurements were taken during pumping of municipal well 34A; they illustrate the amount of head loss that was occurring across an approximately 1-foot-thick layer of alluvial silt and clay on the river bottom. In spite of the less-than-perfect hydraulic continuity between the river and the ground water, the ground-water system responded rapidly to changes in river level. The relatively abrupt rises in the river level on May 18 and 23, 1990 (Figure 4-14), caused nearly equivalent rises in monitoring wells R20-89-113, -119, and -127 (Figures 4-15 through 4-17), located southeast, northeast, and west,

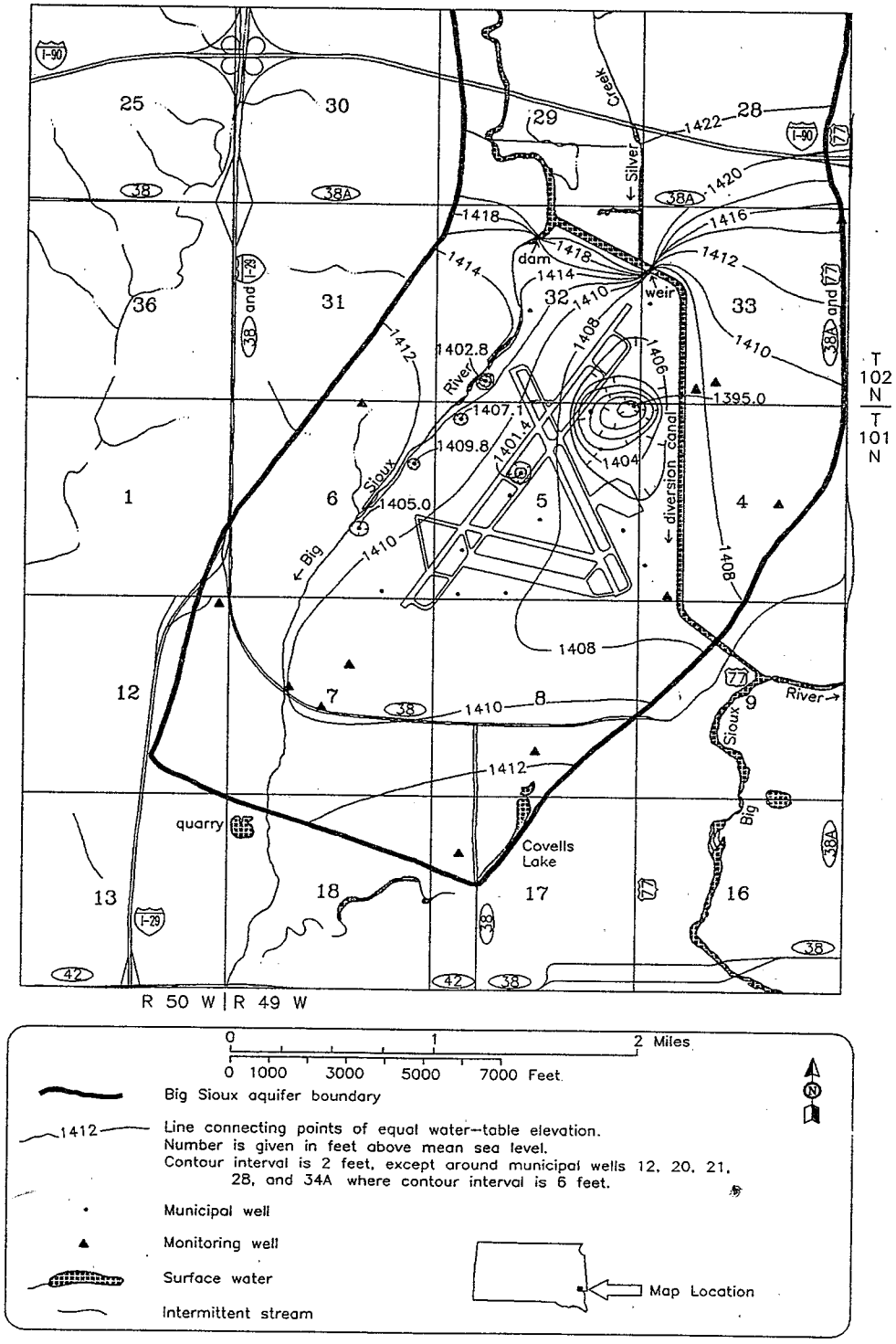


Figure 4-10. Water-table elevations for the study area on August 2, 1990, Sioux Falls airport wellfield, South Dakota.

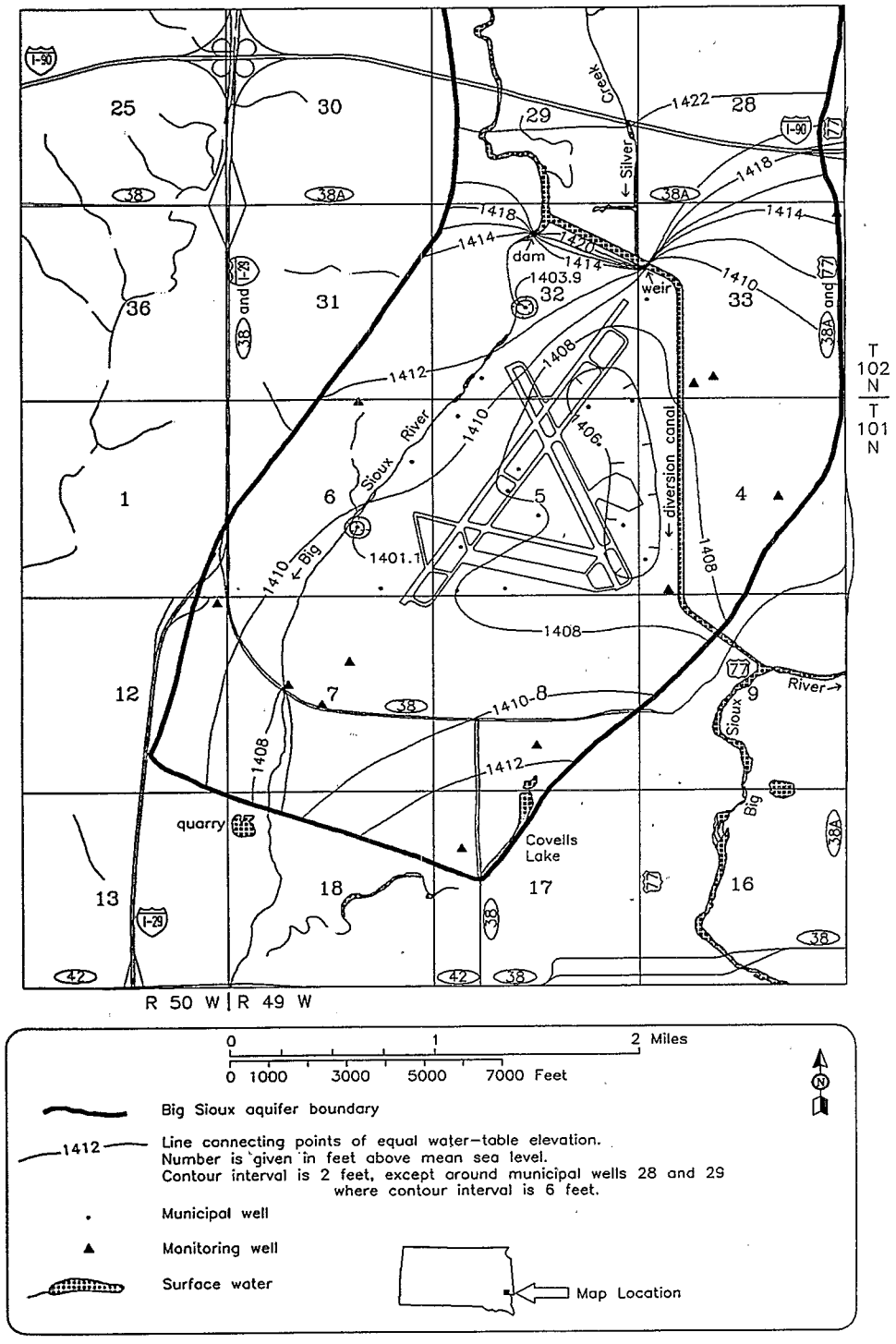


Figure 4-11. Water-table elevations for the study area on August 17, 1990, Sioux Falls airport wellfield, South Dakota.

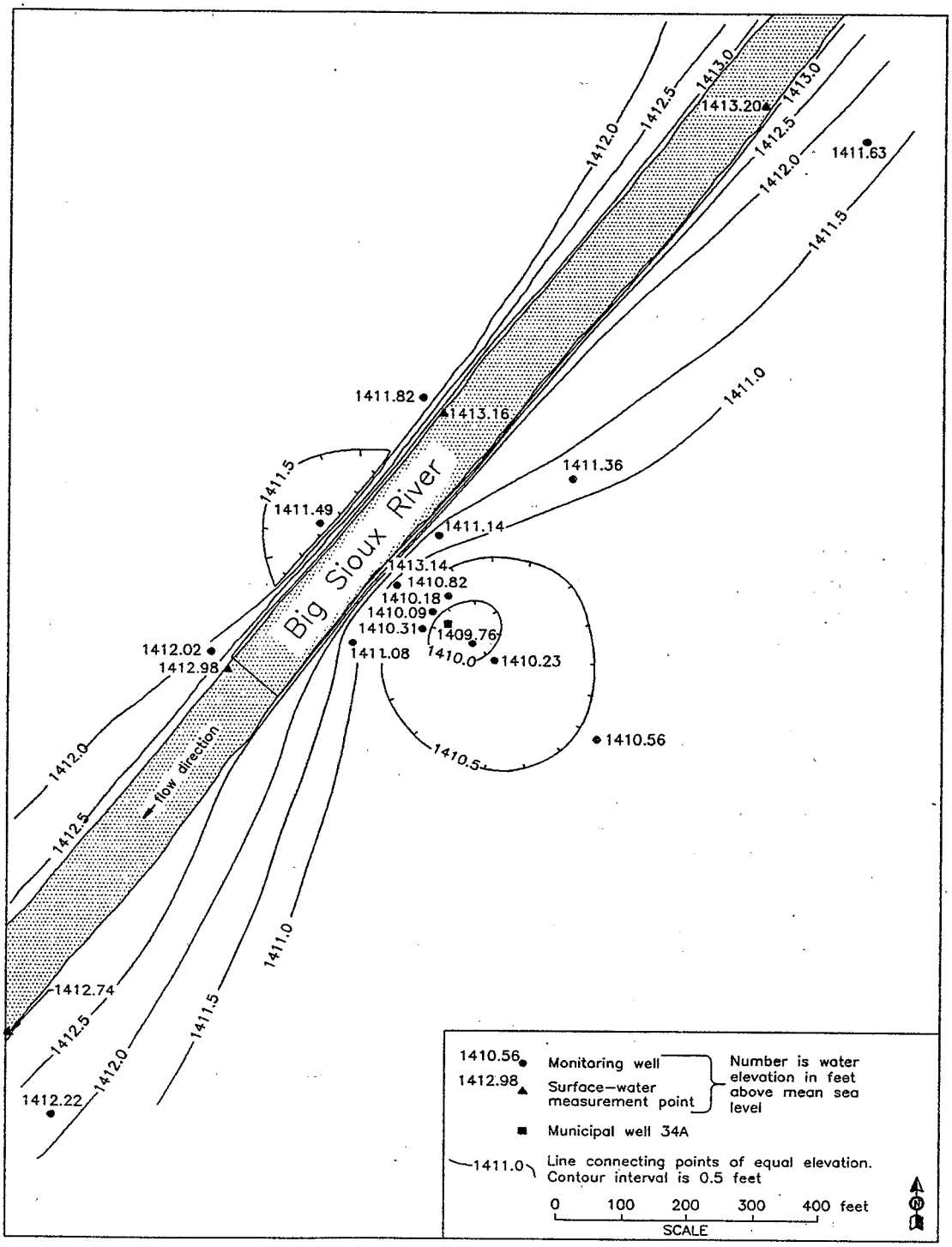


Figure 4-12. Water-table elevations near municipal well 34A under pumping conditions on August 2, 1990, Sioux Falls airport wellfield, South Dakota.

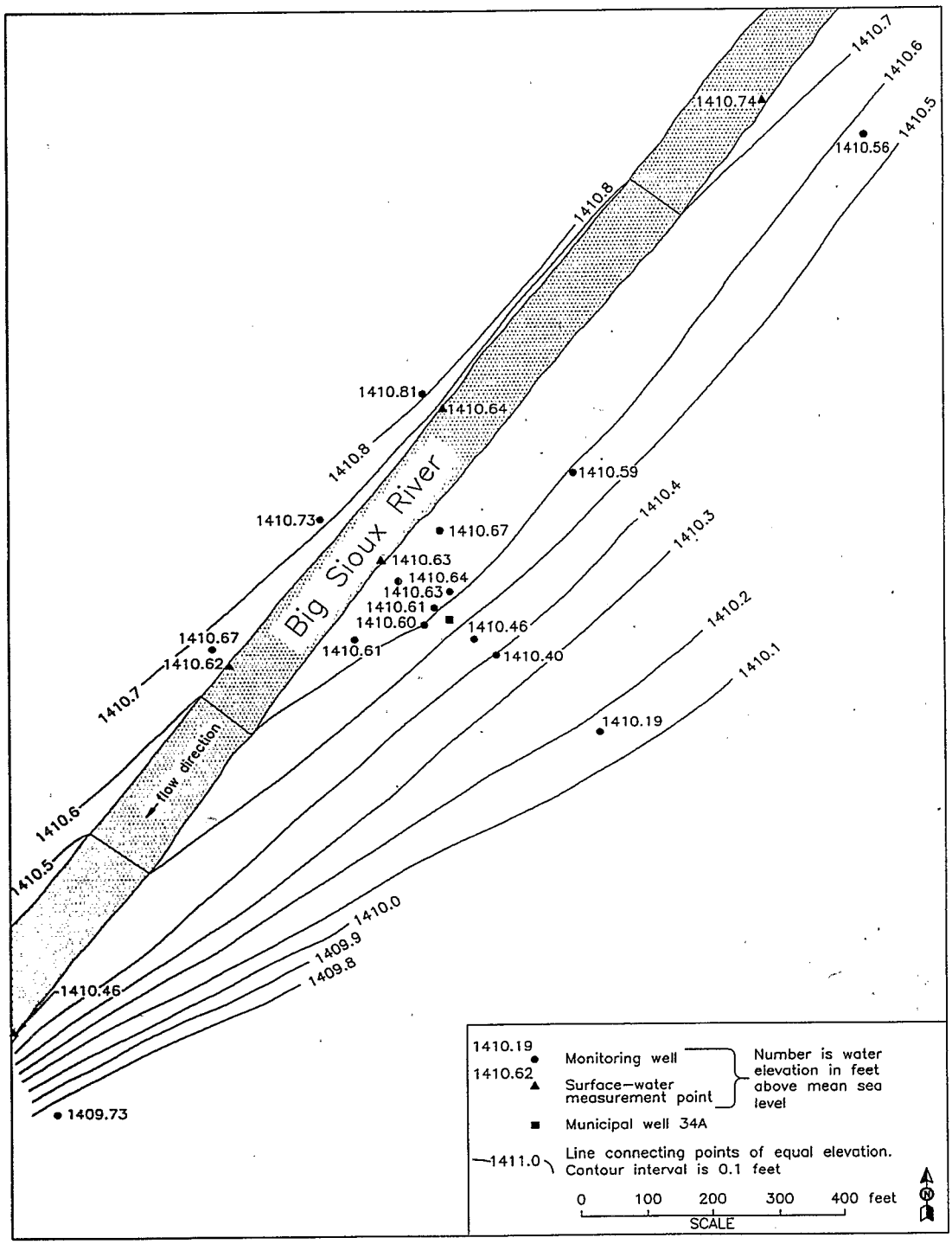


Figure 4-13. Water-table elevations near municipal well 34A under static conditions on August 17, 1990, Sioux Falls airport wellfield, South Dakota.

TABLE 4-5. WATER ELEVATIONS AT GROUND-WATER AND SURFACE-WATER MONITORING POINTS, AUGUST 2 AND 17, 1990, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA

Monitoring Point	Water Elevation* 08/02/90	Water Elevation* 08/17/90	Monitoring Point	Water Elevation* 08/02/90	Water Elevation* 08/17/90
Monitoring well†			Municipal well†		
R20-90-1	1409.19	1409.07	4	1408.7	1406.9
R20-90-2	1407.92	1407.68	10	1408.3	1408.0
R20-90-3	1411.59	1411.27	11	1409.2	1407.6
R20-90-4	1413.46	1413.20	12	1401.4‡	1406.9
R20-90-5	1411.75	1411.81	13	1409.0	1408.8
R20-89-113	1409.76	1410.46	14	1406.2	1405.4
R20-89-114	1410.23	1410.40	15	1407.7	1405.2
R20-89-115	1410.31	1410.60	17	1402.7‡	1406.7
R20-89-116	1410.09	1410.61	18	1400.3‡	1404.7
R20-89-117	1410.18	1410.63	20	1402.8‡	1411.9
R20-89-118	1410.56	1410.19	21	1407.1‡	1410.7
R20-89-119	1411.63	1410.56	23	1409.0	1409.7
R20-89-120	1411.36	1410.59	24	1395.1	1405.0
R20-89-122	1412.22	1409.73	27	1408.8	1408.8
R20-89-123	1411.14	1410.67	28	1405.0‡	1401.1‡
R20-89-124	1410.82	1410.64	29	1410.8	1403.9†
R20-89-125	1411.08	1410.61	30	1406.4	1408.4
R20-89-126	1412.02	1410.67			
R20-89-127	1411.49	1410.73			
R20-89-128	1411.82	1410.81			
SFE-81-1A	1409.19	1408.21			
SFE-81-2B	1409.99	1408.13			
SFE-81-6B	1409.02	1408.61			
DOT-S6	1411.31	1411.29			
Vertical river well	1413.14	1410.63	Surface Water†		
Horizontal well	1411.82	1410.75	A	1407.14	1407.99
Anderson's Truck Stop MW-10	1414.70	1414.62	B	1407.08	1407.94
Stone Container MW-10	1408.96	1408.62	C	1410.11	1407.90
Stone Container MW-18	1407.78	1407.76	D	1413.16	1410.69

* Elevations are in feet above mean sea level.

† See Figures 4-6 and 4-7 for well locations; see Figure 4-6 for locations of surface-water monitoring points.

‡ Indicates that the municipal well was pumping at time of measurement.

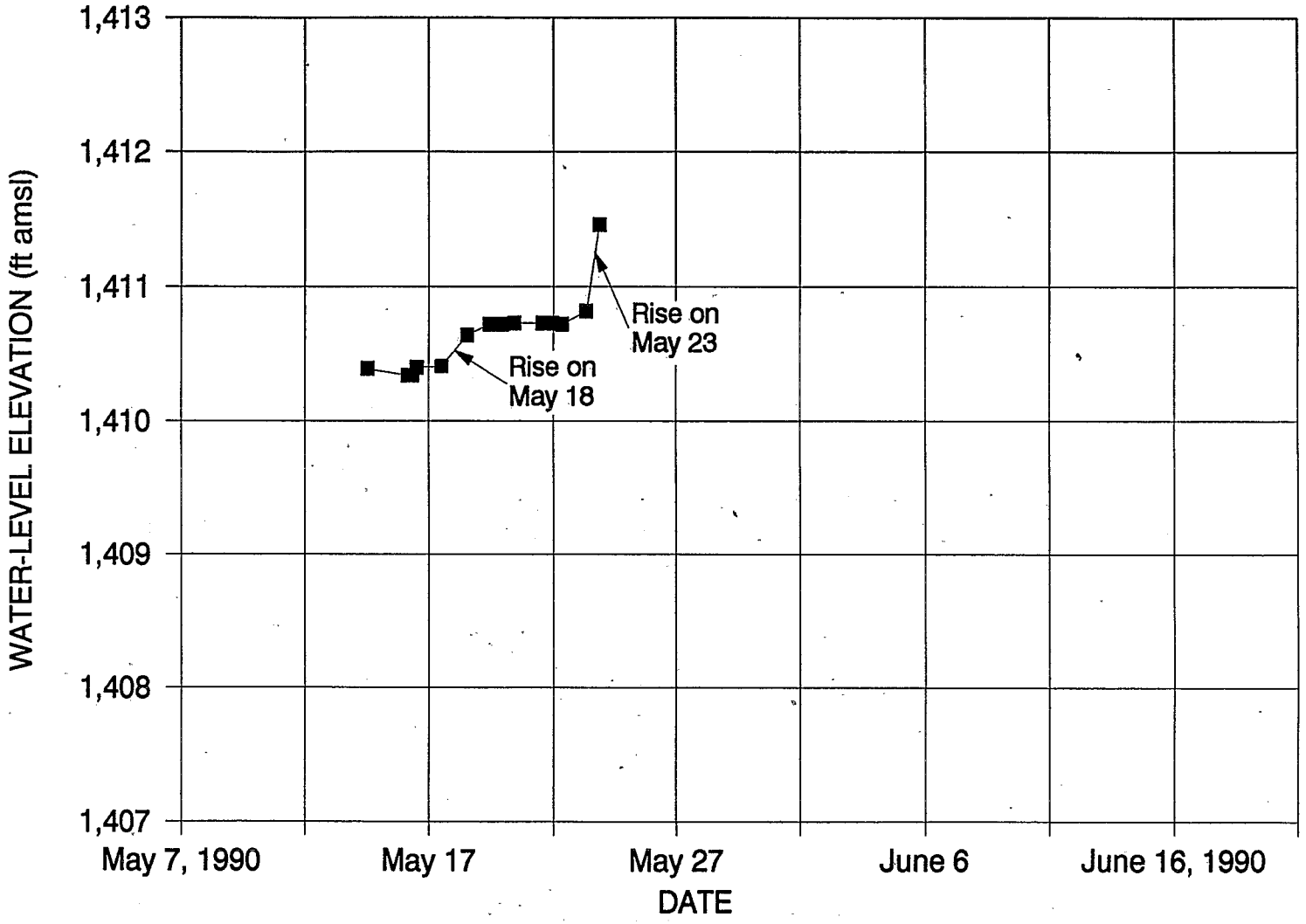


Figure 4-14. Hydrograph of the vertical well in the river, May 16 through 23, 1990, Sioux Falls airport wellfield, South Dakota.

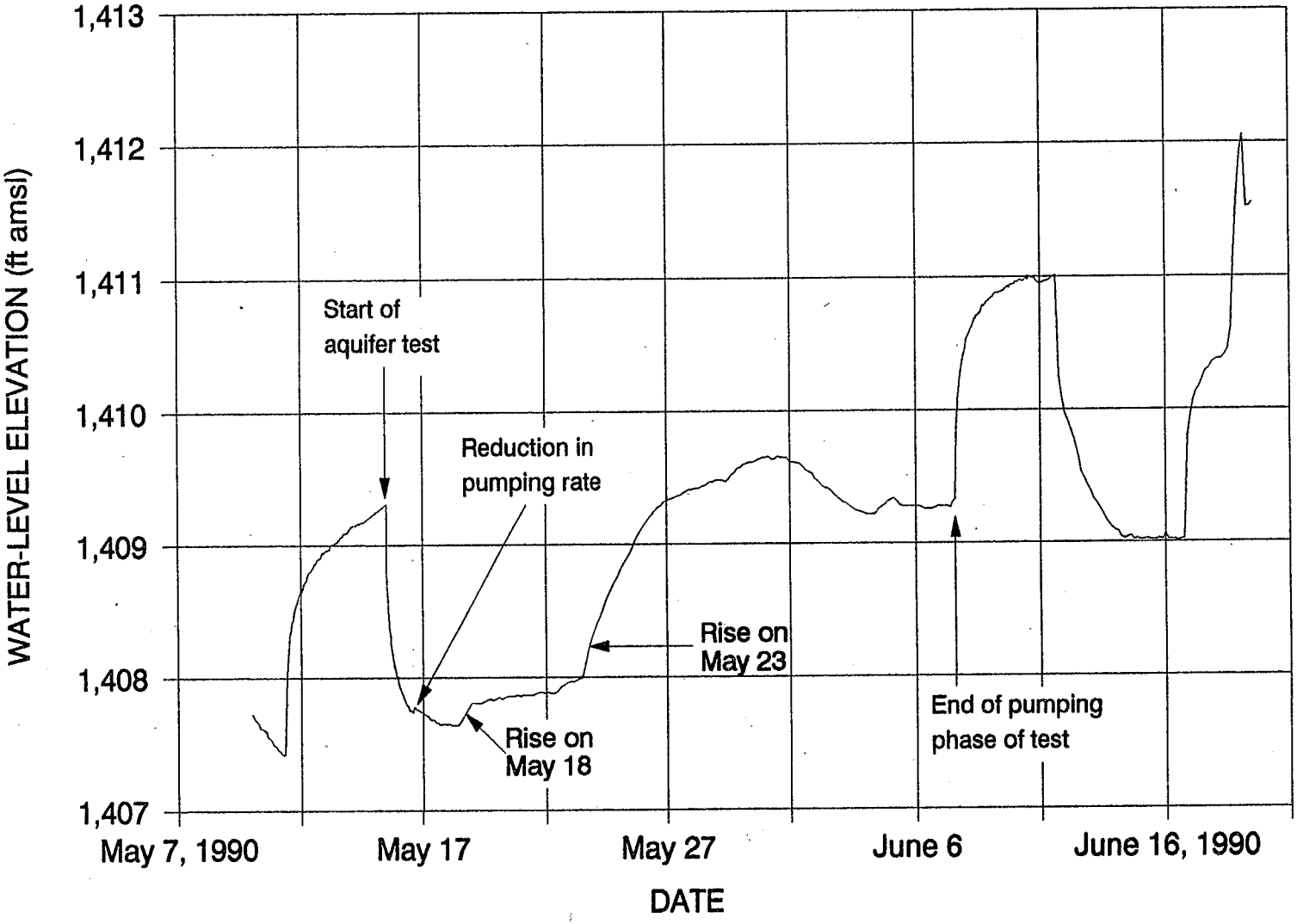


Figure 4-15. Hydrograph of well R20-89-113, May 10 through June 19, 1990, Sioux Falls airport wellfield, South Dakota.

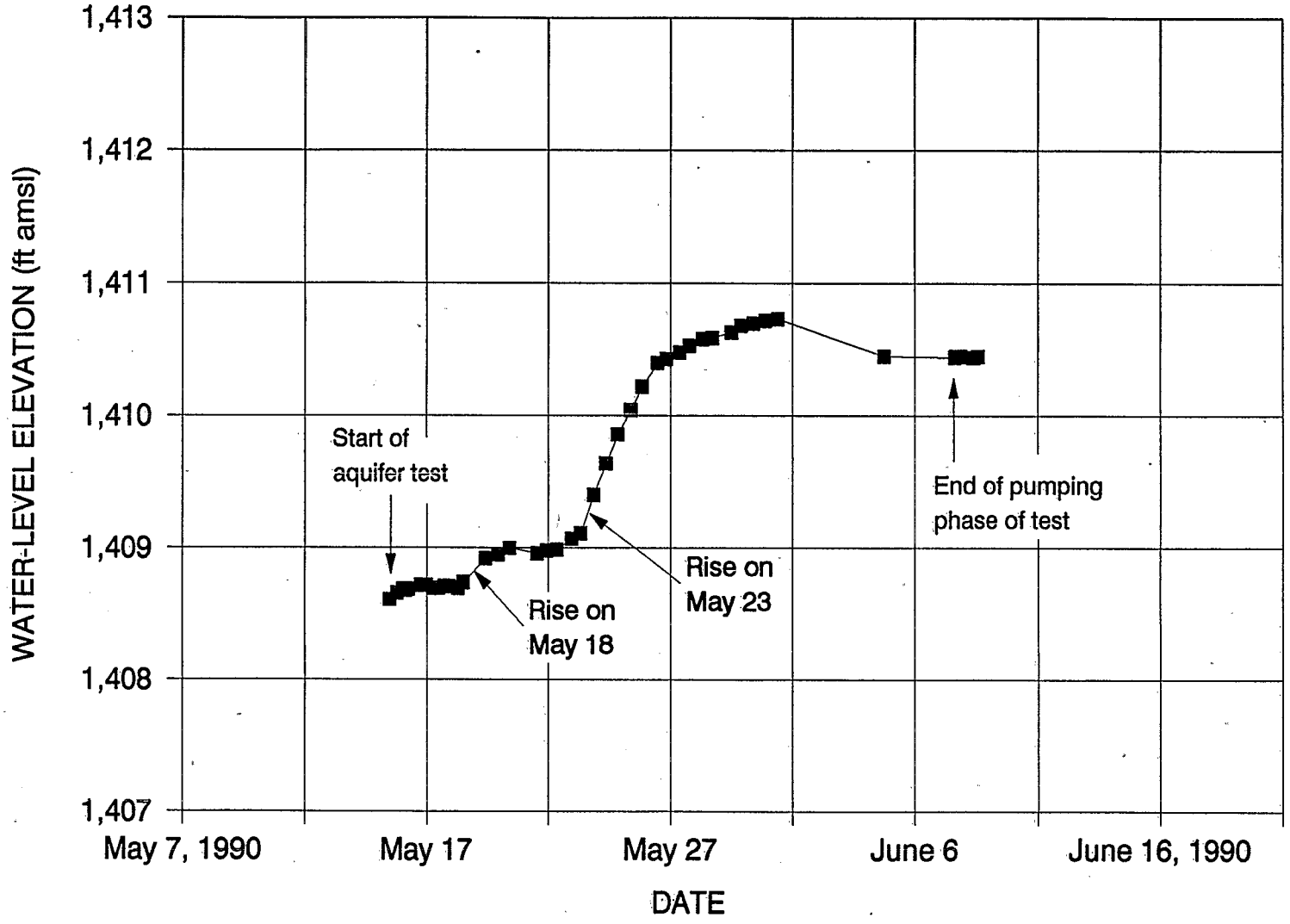


Figure 4-16. Hydrograph of well R20-89-119, May 15 through June 8, 1990, Sioux Falls airport wellfield, South Dakota.

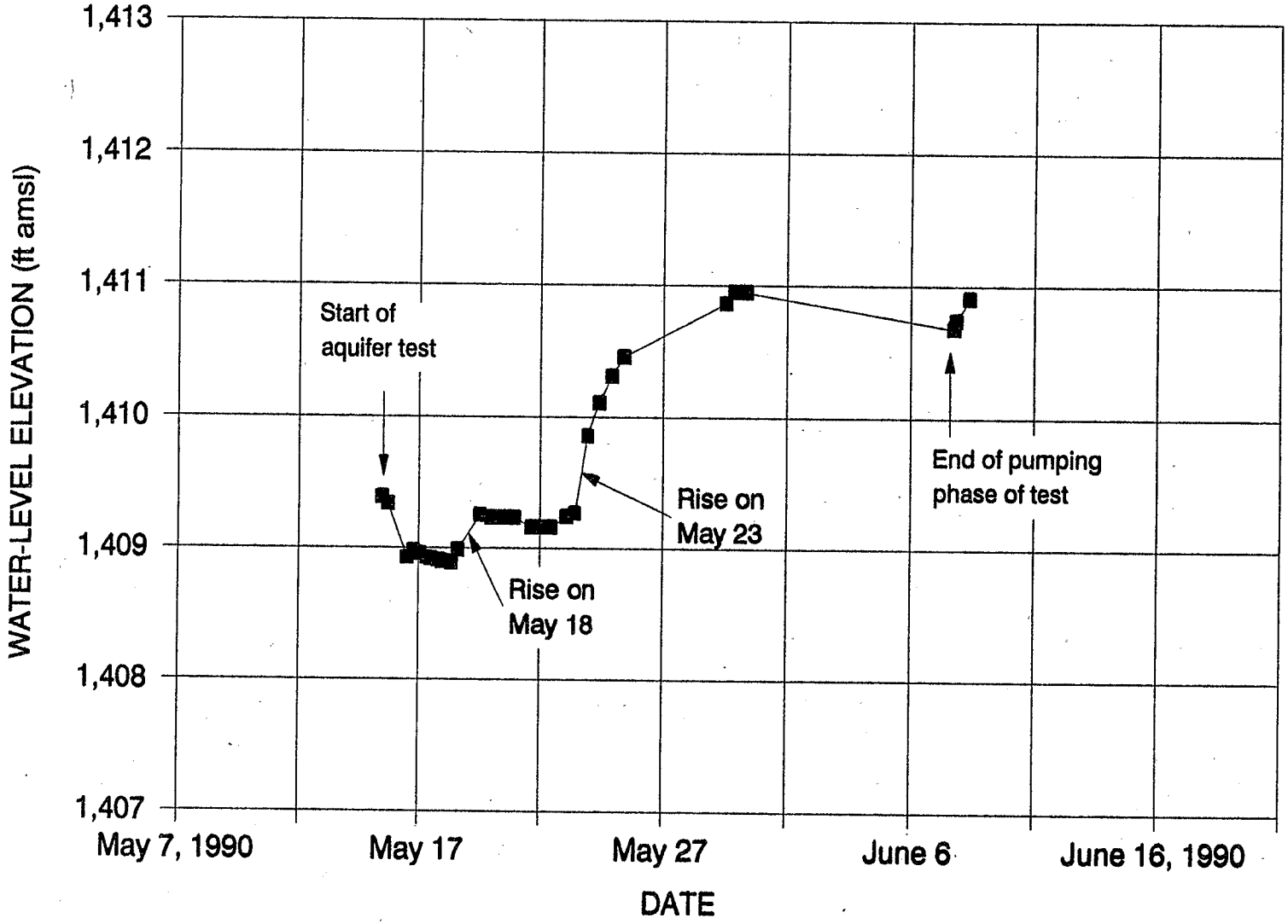


Figure 4-17. Hydrograph of well R20-89-127, May 15 through June 8, 1990, Sioux Falls airport wellfield, South Dakota.

respectively, of the horizontal well (Figure 4-7). Most rises in the ground-water levels on these hydrographs coincided with a rise in river level. The rapid response of ground-water fluctuations to changes in river levels confirm good hydraulic interconnection.

Water Quality and Chemistry

Results of analyses for dissolved inorganic parameters for samples collected during January, April, and July 1990 are presented in Table 4-6. Analyses for arsenic and selenium were performed only on the April and July samples because the January samples were inadvertently discarded before analysis.

Trilinear plots of these water quality data, presented in Figures 4-18 through 4-20, indicate that the ground-water chemistry remains relatively constant throughout the year. The dominant cation in ground water is calcium, and the dominant anion is bicarbonate. The anion sulfate shows a significant variation in the ground-water sample analyses, which results in a calculated mean concentration that is slightly less than the concentration in the surface water (Table 4-6).

Results from analyses of surface water collected in January and April show that the dominant cation was magnesium, whereas calcium was dominant in July. The dominant anion in surface water was sulfate in January and April. However, as shown in Figure 4-20, carbonate plus bicarbonate, and sulfate, were present in equal amounts in July.

Analyses for arsenic show concentrations ranging from less than 1 µg/l to 19 µg/l in ground water and from 3 to 8 µg/l in surface water. Only one ground-water sampling location, well R20-89-122, showed an arsenic concentration greater than 5 µg/l. All concentrations of arsenic are below the EPA primary drinking water standard of 50 µg/l.

Analyses for selenium show concentrations ranging from less than 1 µg/l to 30 µg/l in ground water, while concentrations were less than or equal to 1 µg/l in surface water (Table 4-6). Only two (R20-89-127 and R20-90-2) of the 10 ground-water sampling locations showed concentrations greater than 5 µg/l. Only one location, well R20-90-2, showed a selenium concentration consistently greater than 5 µg/l. All concentrations of selenium are below the EPA primary drinking water standard of 50 µg/l. Based on the results listed in Table 4-6, arsenic and selenium in the ground and surface waters are not currently present at levels of concern to public health.

None of the VOCs listed in Table 4-4 were detected in analyses except acetone, which was found in nine of the 12 total water samples. Acetone was used as a cleaning solvent for the Teflon bailers and probably was introduced to the water during sampling. This conclusion is supported by the results of analyses of field blanks collected using the Teflon bailers and a shipping blank supplied by the University of Iowa Hygienic Laboratory that accompanied the sample bottles. Acetone was detected in one of the two field blanks but not in the shipping blank. Acetone is not believed to be present in the aquifer.

TABLE 4-6. INORGANIC ANALYTES FOR WATER SAMPLES, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA
(Part 1 of 4: Wells R20-89-118 through R20-90-5; Parameters Field pH through Iron)

Monitoring Point	Date Sampled	Field pH	SC (µS/cm)	Parameter and Concentration (mg/l)							
				Alkalinity							
				TA	PA	HCO_3	CO_3	Ca	Cl	F	Fe*
Ground Water											
R20-89-118	1/16/90	7.26	1,280	307	0	374	0	158	35.0	0.25	1.68
	4/11/90	7.03	1,140	270	0	329	0	136	33.0	0.20	1.32
	7/10/90	7.19	1,310	291	0	355	0	169	35.0	0.21	3.75
R20-59-119	1/16/90	7.30	908	239	0	291	0	104	34.0	0.36	<0.05
	4/10/90	7.13	1,090	205	0	250	0	118	58.0	0.33	<0.05
	7/10/90	7.54	884	232	0	283	0	90	30.0	0.32	<0.05
R20-89-122	1/16/90	7.06	921	241	0	294	0	100	26.0	0.36	0.27
	4/11/90	6.76	1,060	224	0	273	0	122	44.0	0.45	0.21
	7/10/90	7.21	1240	295	0	360	0	180	33.0	0.30	0.68
R20-69-124	1/16/90	7.49	1,190	322	0	393	0	138	11.0	0.26	<0.05
	4/10/90	7.53	680	183	0	223	0	58	35.0	0.36	<0.05
R20-89-125	7/10/90	7.55	768	236	0	288	0	86	14.0	0.22	<0.05
R20-89-127	1/16/90	7.08	1,220	315	0	384	0	211	3.4	0.36	0.13
	4/10/90	7.04	1,150	316	0	385	0	200	3.4	0.40	<0.05
	7/10/90	7.11	1,220	320	0	390	0	207	3.7	0.48	<0.05
R20-90-1	1/15/90	7.44	648	237	0	289	0	94	14.0	0.33	<0.05
	4/09/90	7.20	784	268	0	327	0	117	20.0	0.27	0.48
R20-90-2	1/15/90	7.17	1,050	344	0	419	0	150	26.0	0.40	<0.05
	4/09/90	6.95	1,110	347	0	423	0	154	36.0	0.35	<0.05
	7/07/90	7.04	1,100	355	0	433	0	155	37.0	0.39	<0.05
R20-90-3	1/15/90	7.24	764	326	0	397	0	109	21.0	0.30	0.28
	4/10/90	7.06	758	275	0	335	0	111	54.0	0.25	0.49
	7/09/90	7.18	634	271	0	330	0	104	6.6	0.25	2.05
R20-90-5	1/16/90	7.44	890	303	0	369	0	131	3.1	0.29	<0.05
	4/09/90	7.23	929	339	0	413	0	136	1.9	0.30	<0.05
	7/09/90	7.46	804	289	0	352	0	119	3.6	0.29	<0.05

(continued)

TABLE 4-6. INORGANIC ANALYTES FOR WATER SAMPLES, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA
(Part 2 of 4: Wells R20-89-118 through R20-90-5; Parameters Potassium through Arsenic)

Monitoring Point	Date Sampled	Parameter and Concentration (mg/l) K	Mg	Mn*	Na	NO_3-N + NO_2-N*	SO_4	TDS	Hardness	(µg/l) Se*†	As*†
Ground Water											
R20-89-118	1/16/90	7.4	60	1.87	36	<=0.04	370	916	642	--	--
	4/11/90	6.7	54	1.79	40	0.15	317	824	562	<1	5
	7/10/90	7.0	61	2.34	40	<0.04	397	980	673	<1	4
R20-59-119	1/16/90	6.5	40	<0.05	35	0.04	211	602	424	--	--
	4/10/90	6.3	48	<0.05	46	0.76	300	748	492	2	<1
	7/10/90	4.8	39	0.07	45	0.80	204	582	385	2	<1
R20-89-122	1/16/90	8.6	43	1.26	36	<0.04	224	612	427	--	--
	4/11/90	8.5	43	1.55	47	0.27	290	750	482	<1	19
	7/10/90	6.4	44	1.80	39	<0.04	350	918	631	<1	6
R20-69-124	1/16/90	5.4	72	1.78	28	<0.04	346	860	641	--	--
	4/10/90	3.9	33	0.05	38	0.20	125	416	281	1	2
R20-89-125	7/10/90	8.0	39	0.56	18	<0.04	184	523	375	<1	5
R20-89-127	1/16/90	2.7	40	2.06	19	0.06	400	892	692	--	--
	4/10/90	2.7	38	1.71	13	0.04	350	818	656	4	1
	7/10/90	2.5	43	0.94	12	0.17	380	920	694	30	<1
R20-90-1	1/15/90	3.3	19	2.02	17	0.06	94	398	313	--	--
	4/09/90	3.4	25	2.43	14	<0.04	132	521	395	<1	1
R20-90-2	1/15/90	4.0	33	2.12	38	1.12	204	700	510	--	--
	4/09/90	4.2	34	2.28	44	1.29	211	758	525	12	<1
	7/07/90	4.4	34	2.54	44	1.57	200	740	527	14	<1
R20-90-3	1/15/90	2.2	15	1.83	39	0.29	63	486	334	--	--
	4/10/90	2.2	15	1.80	34	<0.04	54	470	339	<1	5
	7/09/90	1.8	14	1.72	15	<0.04	79	409	317	<1	2
R20-90-5	1/16/90	2.5	37	<0.05	15	0.45	187	600	479	--	--
	4/09/90	3.3	39	<0.05	15	0.77	180	664	500	2	1
	7/09/90	2.7	34	<0.05	9.6	0.84	152	560	437	1	1

(continued)

TABLE 4-6. INORGANIC ANALYTES FOR WATER SAMPLES, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA
(Part 3 of 4: Well DOT-S6 through Surface Water; Parameters Field pH through Iron)

Monitoring Point	Date Sampled	Field pH	SC (µS/cm)	Parameter and Concentration (mg/l)							
				Alkalinity							
				TA	PA	HCO_3	CO_3	Ca	Cl	F	Fe*
DOT-S6	1/17/90	7.34	797	289	0	352	0	93	8.0	0.51	<0.05
	4/09/90	7.29	686	284	0	346	0	90	7.3	0.46	<0.05
	7/09/90	7.47	702	288	0	351	0	94	9.5	0.42	<0.05
	Mean	7.23	956	283	0	345	0	129	22.3	0.33	0.41
	Maximum	7.55	1,310	355	0	433	0	211	58.0	0.51	3.75
	Minimum	6.76	634	183	0	223	0	58	1.9	0.20	<0.05
	Standard deviation	0.19	210	44	0	53	0	38	16.0	0.08	0.81
Surface Water (Big Sioux River)											
	1/17/90	8.11	919	140	0	171	0	58	53.0	0.28	<0.05
	4/11/90	8.45	674	120	7	129	8	46	44.0	0.26	<0.05
	7/10/90	8.39	854	235	4	278	4	99	20.0	0.27	<0.05
	Mean	8.32	816	165	4	193	4	68	39.0	0.27	0.03
	Maximum	8.45	919	235	7	278	8	99	53.0	0.28	<0.05
	Minimum	8.11	674	120	0	129	0	46	20.0	0.26	<0.05
	Standard deviation	0.15	104	50	3	63	3	23	14.0	0.01	0.00

(continued)

TABLE 4-6. INORGANIC ANALYTES FOR WATER SAMPLES, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA
(Part 4 of 4: Well DOT-S6 through Surface Water; Parameters Potassium through Arsenic)

Monitoring Point	Date Sampled	Parameter and Concentration (mg/l) K	Mg	Mn*	Na	NO_3-N + NO_2-N*	SO_4	TDS	Hardness	(µg/l) Se*	As*
DOT-S6	1/17/90	2.8	29	<0.05	20	3.27	73	438	352	--	--
	4/09/90	2.7	29	<0.05	16	3.33	71	466	344	5	1
	7/09/90	2.7	29	<0.05	15	3.82	70	452	354	5	<1
	Mean	4.5	37	1.23	29	0.67	214	656	475		
	Maximum	8.6	72	3.28	47	3.82	400	980	694		
	Minimum	1.8	14	<0.05	10	<0.04	54	398	281		
	Standard deviation	2.1	13	0.99	13	1.04	112	179	127		
Surface Water (Big Sioux River)											
	1/17/90	6.6	52	0.06	54	0.04	263	592	359	--	--
	4/11/90	5.8	34	<0.05	42	<0.04	170	426	255	1	3
	7/10/90	7.2	43	<0.05	30	<0.04	220	610	424	<1	8
	Mean	6.5	43	0.04	42	0.03	218	543	346		
	Maximum	7.2	52	0.06	54	0.04	263	610	424		
	Minimum	5.8	34	<0.05	30	<0.04	170	426	255		
	Standard deviation	0.6	7	0.01	10	0.01	38	83	70		

Abbreviations: SC = specific conductivity; TA = total alkalinity; PA = phenolphthalein alkalinity; HCO_3 = bicarbonate; CO_3 = carbonate; Ca = calcium; Cl = chloride; F = fluoride; Fe = iron; K = potassium; Mg = magnesium; Mn = manganese; Na = sodium; NO_3-N + NO_2-N = nitrate nitrogen plus nitrite nitrogen; SO_4 = sulfate; TDS = total dissolved solids; Hardness = hardness as calcium carbonate; Se = selenium; As = arsenic; < = less than or equal to; -- = no data; < less than.

* For parameters with less than (<) values for concentrations, one-half of the less than value was used to calculate the mean and standard deviation.

† No data for January samples.

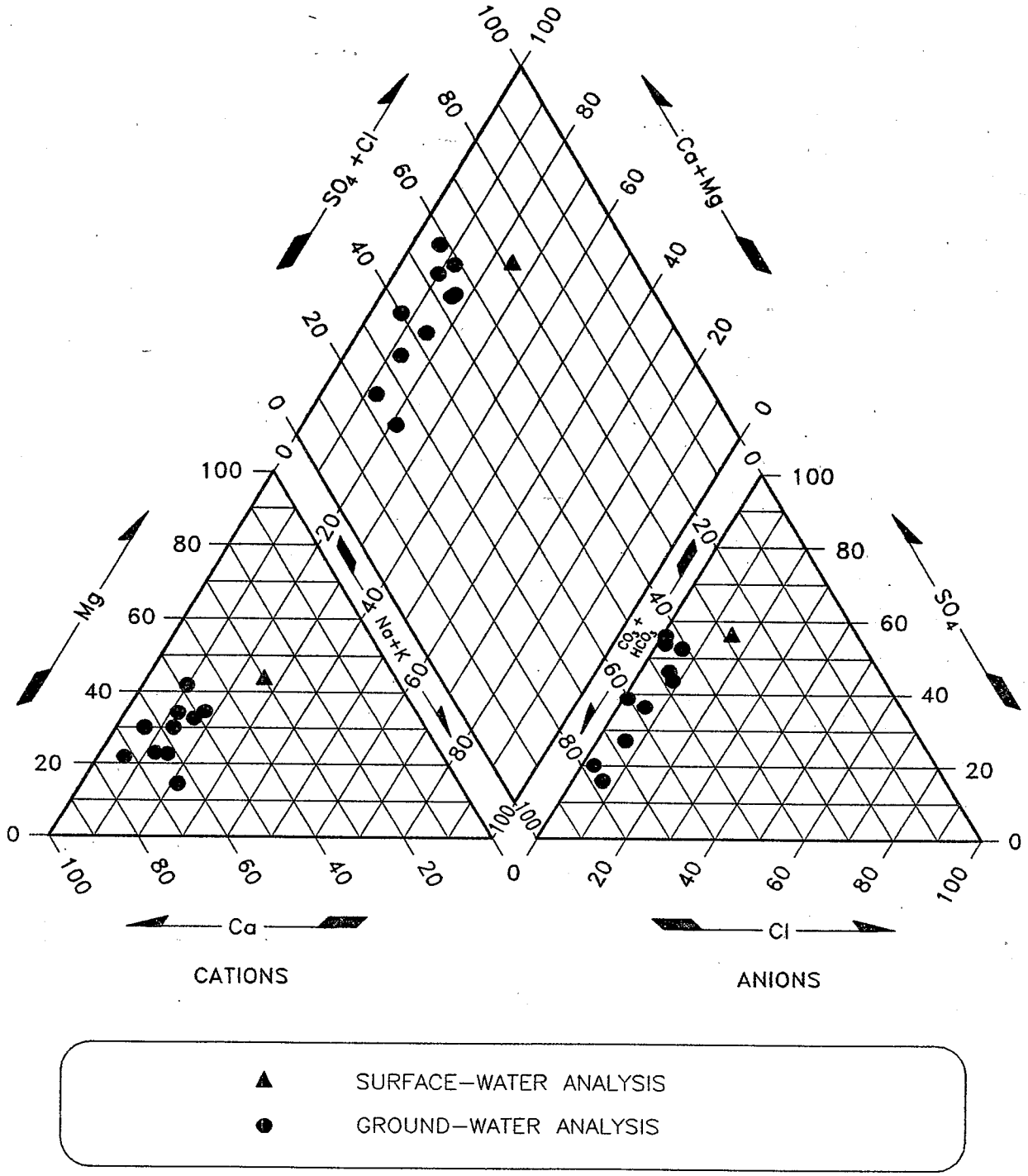


Figure 4-18. Trilinear diagram of major cations and anions from January 1990 water samples, Sioux Falls airport wellfield, South Dakota.

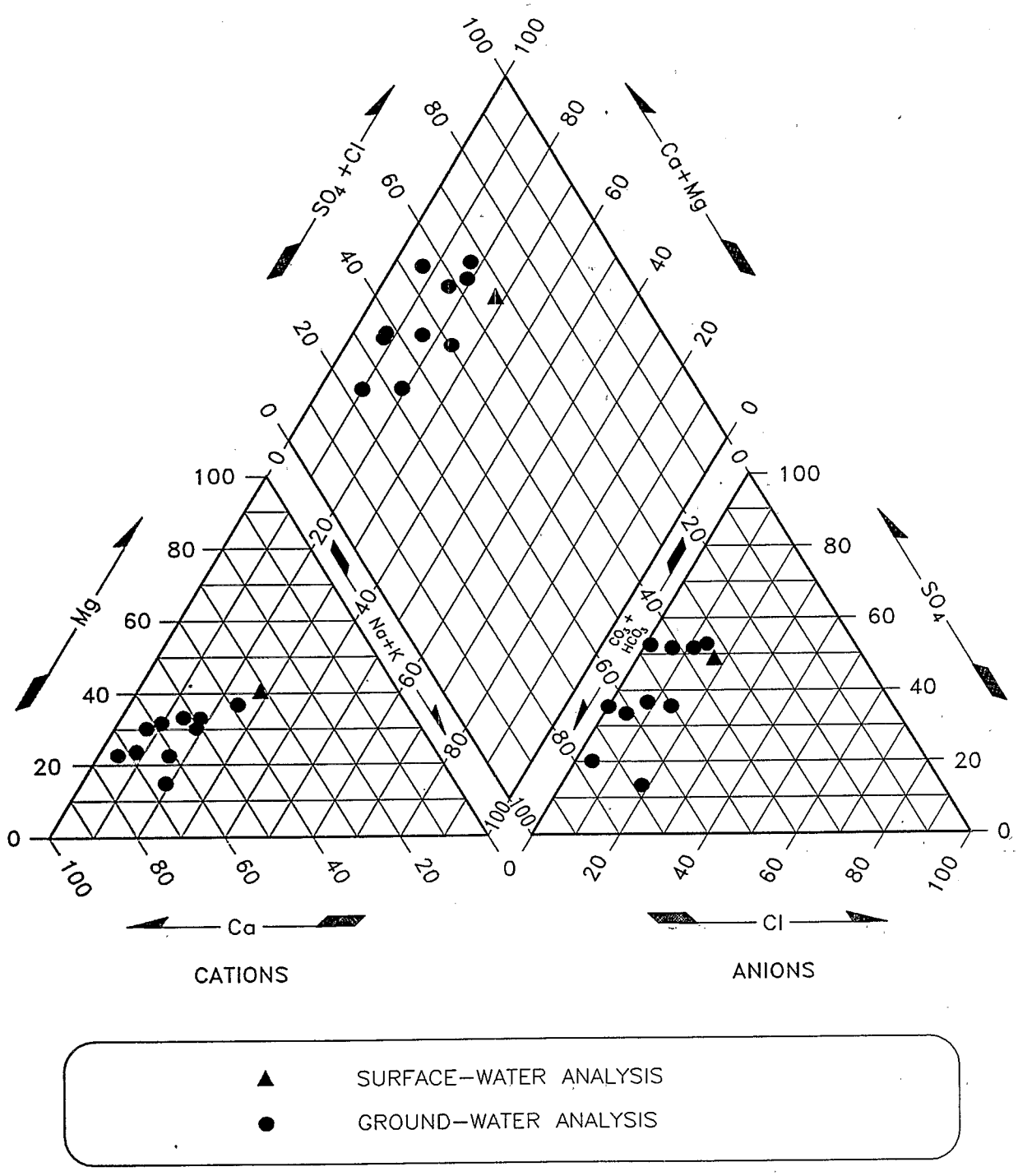


Figure 4-19. Trilinear diagram of major cations and anions from April 1990 water samples, Sioux Falls airport wellfield, South Dakota.

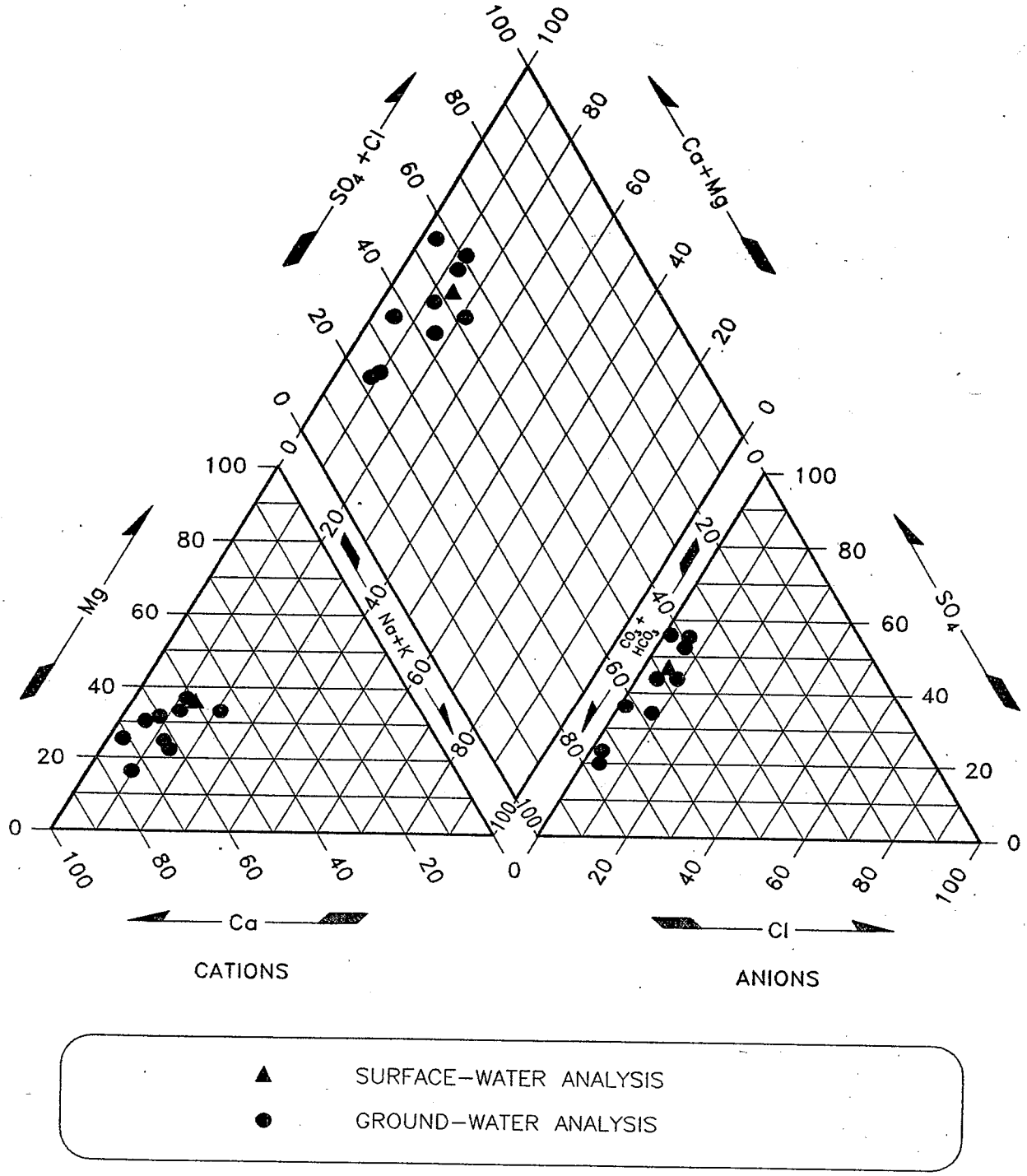


Figure 4-20. Trilinear diagram of major cations and anions from July 1990 water samples, Sioux Falls airport wellfield, South Dakota.

Of the pesticides listed in Table 4-4, atrazine, cyanazine (Bladex), 2,4-D, and dicamba (Banvel) were detected in ground water or in the river. The sample collection points and pesticide concentrations are listed in Table 4-7. Analyses for 2,4,5-TP (Silvex), chloramben (Amiben), and dicamba (Banvel) were not requested; however, analytical results were provided for these parameters in the results from the July water samples. Thus, no comparison is available for these pesticides with respect to the January water samples.

TABLE 4-7. PESTICIDES DETECTED IN JANUARY AND JULY 1990 WATER SAMPLES, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA

Sample Collection Point	Date of Sample Collection	Concentration of Pesticide (µg/l)*			
		Atrazine	Cyanazine (Bladex)	2,4-D	Dicamba (Banvel)
Well R20-89-119	01/16/90	0.1	ND	ND	—
Well R20-89-125	07/10/90	2.2	0.89	ND	ND
Big Sioux River	07/10/90	2.0	0.39	0.2	0.69

Abbreviations: ND = none detected; -- = no analysis performed.

* Dicamba was not analyzed in January water samples. Atrazine, cyanazine, and 2,4-D were analyzed in both the January and July water samples.

Wells R20-90-119 and R20-90-125 are very close to the Big Sioux River at the airport wellfield (Figure 4-7). Pesticides in these wells may be attributed to induced movement of contaminants from the river into the aquifer. The EPA primary drinking water standards for atrazine and 2,4-D are 3 µg/l and 70 µg/l, respectively. No primary standards currently exist for cyanazine or dicamba. All pesticide detections are below available EPA standards, although the level of atrazine detected is only about 1 µg/l lower than that of the primary drinking water standard.

Aquifer-Test Analysis

Pumping and recovery phases of an aquifer test at municipal well 34A were conducted from noon on May 15 through 2:00 p.m. on June 10, 1990. The test was run under less than ideal conditions because of rainfall and, therefore, rises in both surface-water and ground-water levels. However, the test provided adequate data for estimation of aquifer properties and evidence of the rapid response of water levels in the aquifer owing to changes in river level. It also allowed a qualitative examination of head loss across the river-bottom sediment.

To gain an understanding of the relationship between the river and the aquifer near municipal well 34A, the water-table surface was mapped under pumping and nonpumping conditions. Figures 4-12 and 4-13 depict times when water-level measurements were available for the entire airport wellfield area. The water-table surface under nonpumping conditions on August 17, 1990 (Figure 4-13), indicates that the river was gaining ground water from the northwest and losing water on the southeast side. No cone of depression existed around municipal well 34A. For the same area of the aquifer under pumping conditions on August 2, 1990 (Figure 4-12), the river level was higher than the ground water on either side of the river. The cone of depression around municipal well 34A, including a slight depression in water levels on the northwest side of the river due to pumping, is depicted in Figure 4-12. Figures 4-12 and 4-13 complement Figures 4-10 and 4-11, which portray the water-table configurations for the entire airport wellfield. These figures are significant because they show that the river is not a complete hydraulic barrier to ground-water movement, under either pumping or nonpumping conditions.

Values measured with pressure transducers and data loggers indicate that water levels were rising during the four-day period prior to the aquifer test, as illustrated in the hydrograph of well R20-89-113 (Figure 4-15). Prior to the test, pressure transducers and data loggers recorded water levels in wells R20-89-113, -114, -115, -116, -118, -120, -123, -125, and -128. The water-level rise in these wells ranged from 0.52 foot in well R20-89-120 to 1.97 feet in R20-89-116, which is the nearest monitoring well to municipal well 34A (Figure 4-7). Rising, or recovering, water levels resulted from the shutdown of municipal well 34A four days prior to the test. Further evidence of recovering water levels are from well R20-89-118; records indicate a 0.02-foot rise during the first half hour of the aquifer test, before drawdown around municipal well 34A began to influence the well.

The configuration of the water-table surface just prior to the aquifer test is shown in Figure 4-21. The river was higher than the ground water on either side, and the ground-water gradients were to the northwest and southeast, away from the river. The difference in the two ground-water elevations shown on the northwest side of the river in Figure 4-21 may indicate a residual cone of depression due to pumping of municipal well 34A, similar to the cone of depression shown on Figure 4-12.

All wells shown in Figure 4-21 responded to pumping of municipal well 34A, except for wells R20-89-119 and R20-89-122, located approximately 1,000 feet to the northeast and southwest, respectively, of municipal well 34A (Figure 4-7). The hydrograph record of well R20-89-119 (Figure 4-16) typifies water-level behaviors in areas outside the influence of municipal well 34A. The hydrograph shows relatively constant water levels during the first 72 hours of the aquifer test, followed by a small rise on May 18 and 19, 1990; a significant rise beginning on May 23, 1990; and ultimately, a slight decline and leveling off near the end of the aquifer test.

Hydrographs of wells R20-89-113 and R20-89-127 (Figures 4-15 and 4-17) illustrate the influence of municipal well 34A on the southeast and northwest sides of the river, respectively. The hydrograph of well R20-89-113 (Figure 4-15) shows the change from a rising to a declining water level on the southeast side of the river after the onset of pumping conditions. The hydrograph of well R20-89-127 (Figure 4-17) illustrates a similar decline in water level on the northwest side of the river as a result of pumping. Declines in the water level on the northwest side of the river are attributed to expansion of the cone of depression from municipal well 34A beyond the hydraulic boundary of the river.

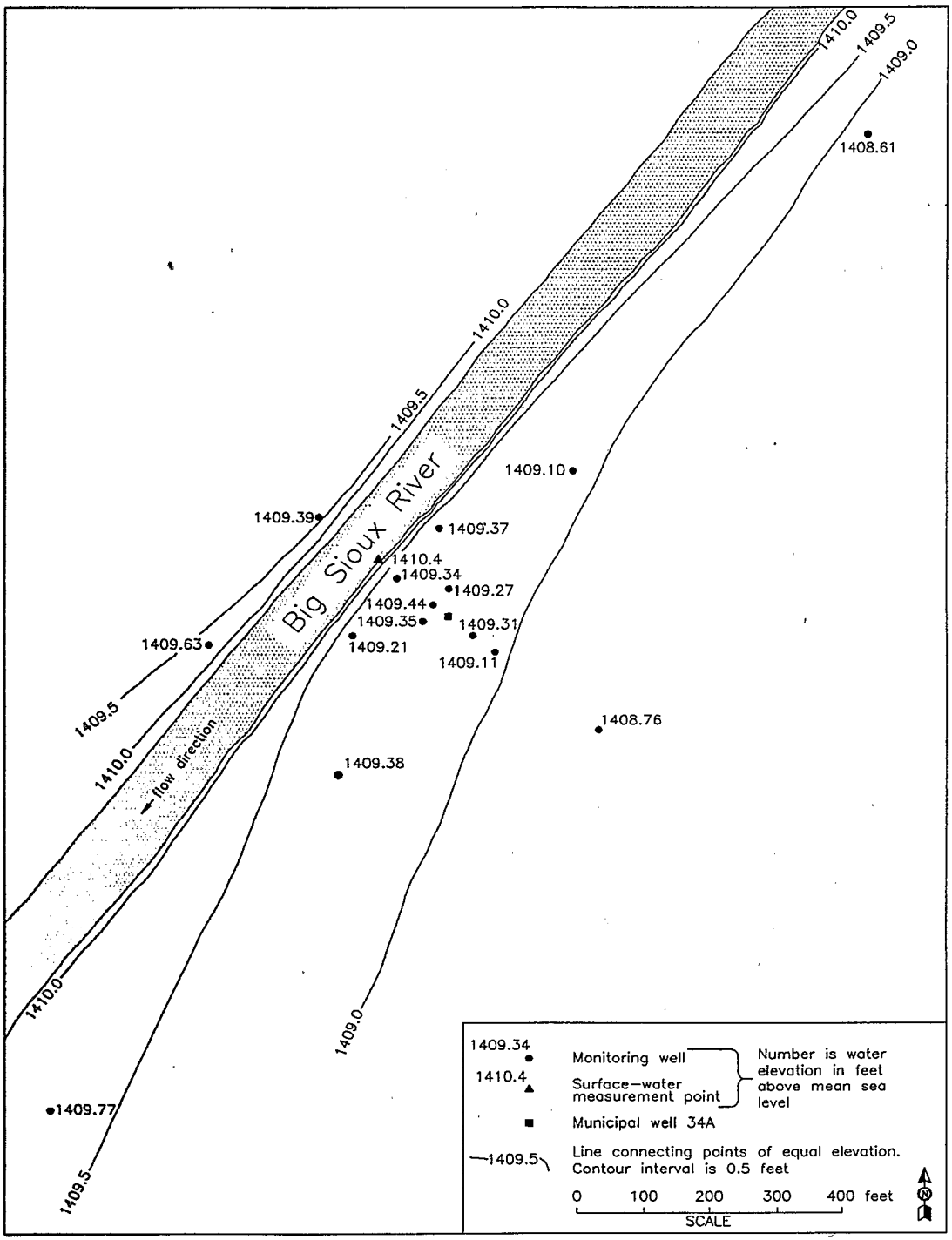


Figure 4-21. Water-table elevations near municipal well 34A prior to aquifer testing on May 15, 1990, Sioux Falls airport wellfield, South Dakota.

Data used for calculation of the transmissivity and storativity parameters of the aquifer were limited to the first 23 hours of the test, after which the pumping rate in municipal well 34A was reduced from 396 to 388 gpm. Also, at about the time the pumping rate was lowered, the river level began to rise and impact water levels in the aquifer. The reduction in pumping was reflected by a small rise in water levels in nearby wells before water levels continued declining (Figure 4-15). During the first 23 hours of the test, the river level remained essentially unchanged.

The Theis and Cooper-Jacob methods of data analysis were employed in examining aquifer-test results. Drawdown data were analyzed using AQTESOLV software (Geraghty & Miller, Inc., 1991). Output from the AQTESOLV software and some example computations are illustrated in Figures 4-22 and 4-23. Values from manual, pressure transducer, and data-logger measurements were used. Water levels were recovering up to the start of the aquifer test; therefore, the measured drawdowns are minimal. This situation resulted in higher calculated transmissivity values than would have been derived had the aquifer been at static conditions prior to the test. This fact is supported by the use of lower hydraulic conductivity values for calibration of the ground-water flow model than what would have been computed using average transmissivity values (Table 4-8).

Aquifer-test data yielded average values of aquifer transmissivity (T) and storativity (S), as follows (Table 4-8):

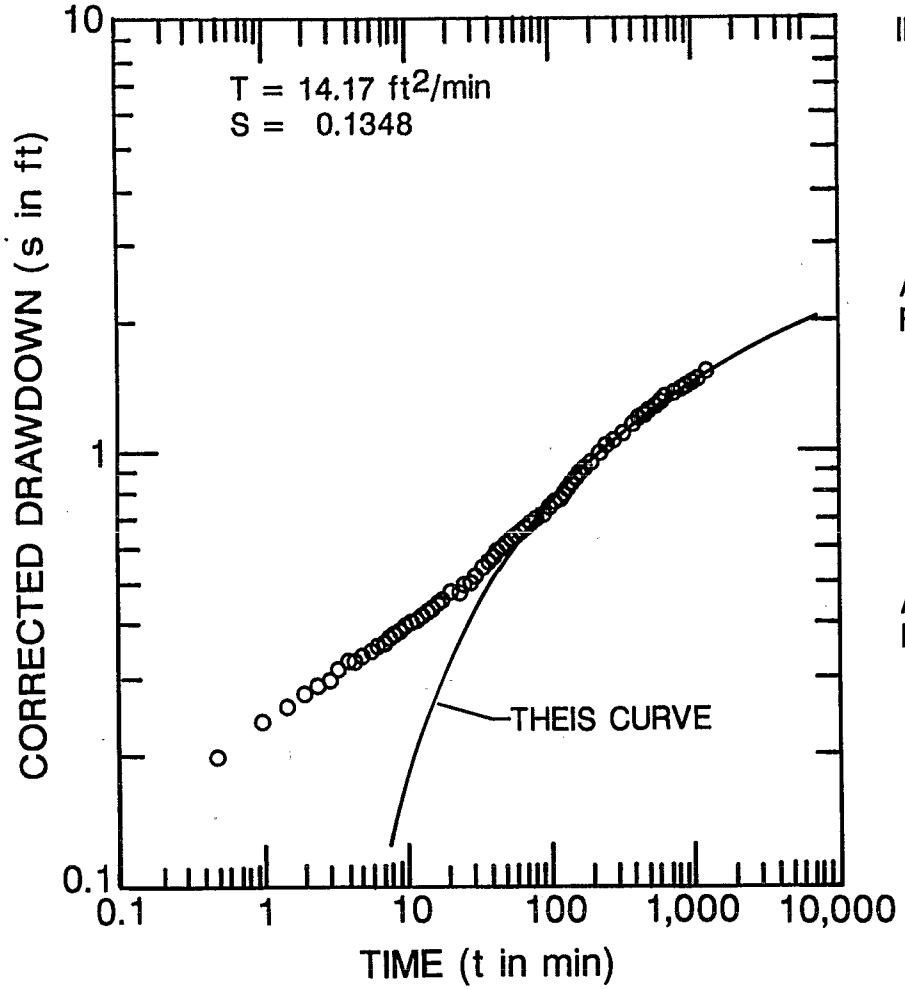
- Using the Theis method, T = 14.51 ft^2/min (20,900 ft^2/d) and S = 0.1931
- Using the Cooper-Jacob method, T = 14.59 ft^2/min (21,000 ft^2/d) and S = 0.1856

The transmissivity averages derived from the Theis and Cooper-Jacob methods, 20,900 ft^2/d and 21,000 ft^2/d respectively, are typical for the Big Sioux aquifer, where transmissivity values normally exceed 13,400 ft^2/d. Corresponding storativity values are 0.1931 and 0.1856; these values fall within the usual range for unconfined aquifers, 0.01 to 0.30 (Freeze and Cherry, 1979). The average saturated thickness of the aquifer in the area of the airport wellfield is 26 feet. Using an average transmissivity value of 20,950 ft^2/d from these two methods yields an average hydraulic conductivity of 800 ft/d.

The data from wells R20-89-118, -120, and -121 were not used to calculate the averages presented in Table 4-8 for the following reasons:

- There was less than 1 foot of drawdown in these wells over the duration of the usable data because of their distance from the pumping well.
- There was an overall trend of slightly rising water levels in the aquifer during the test.
- The calculated transmissivity values from these wells are significantly higher than values for other wells.

The termination of the pumping phase of the aquifer test is clearly shown on Figures 4-15 and 4-17 by the abrupt rises in ground-water levels on June 7, 1990. These figures show a general rise in water level throughout much of the pumping phase of the test. Rainfall in the drainage basin during portions of the aquifer test caused the river level to rise. The general rise in ground-water levels is the result of the rising river level and illustrates the sensitivity of the surface-water and ground-water system.



INPUT PARAMETERS TO AQTESOLV:

Q = PUMPING RATE = 52.9 ft^3/min

r = DISTANCE FROM PUMPING WELL TO WELL R20-89-113

= 45 ft

AQTESOLV-DETERMINED PARAMETERS FROM CURVE MATCHING:

W(u) = WELL FUNCTION OF u

u = WELL FUNCTION FACTOR

s = DRAWDOWN (ft)

t = TIME SINCE PUMPING STARTED (min)

AQTESOLV-CALCULATED PARAMETERS FROM EQUATIONS:

T = TRANSMISSIVITY

$$= \frac{1\ Q\ W(u)}{4\pi s}$$

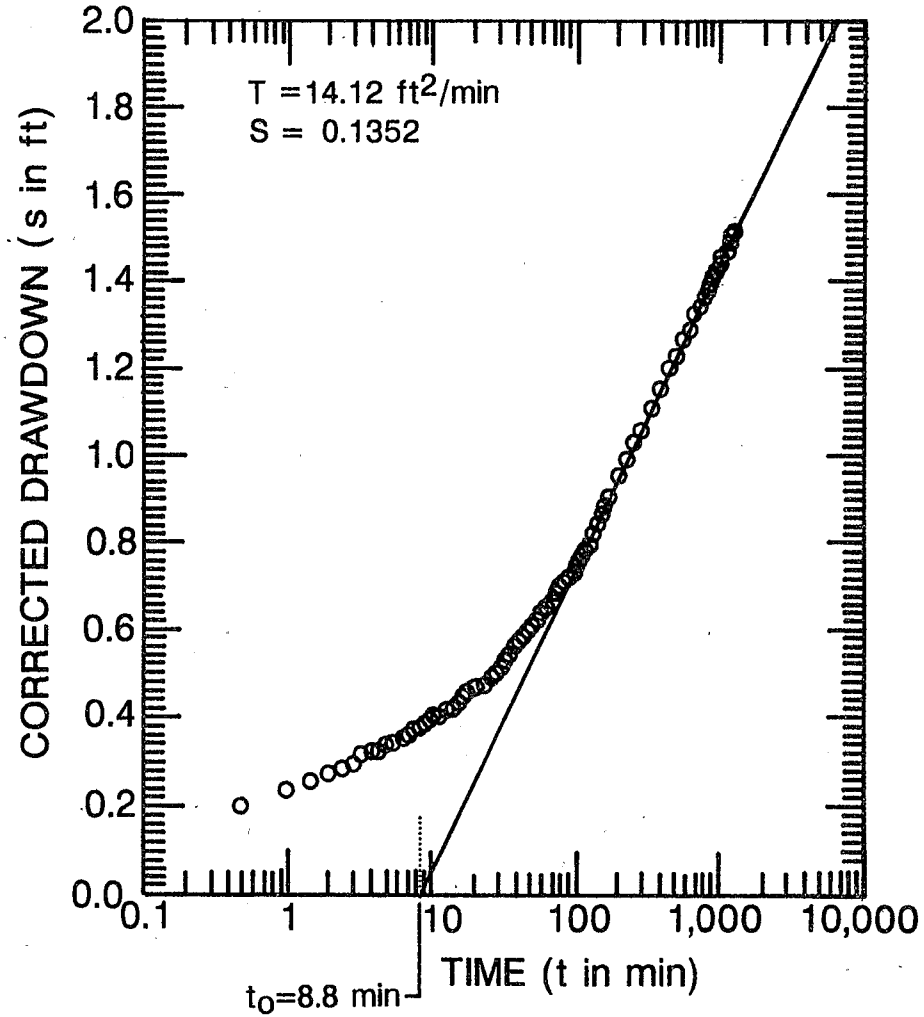
= 14.17 ft^2/min

S = STORATIVITY

$$= \frac{4uTt}{r^2}$$

= 0.1348

Figure 4-22. Theis analysis of the logarithmic plot of time versus drawdown data from well R20-89-113, Sioux Falls airport wellfield, South Dakota.



INPUT PARAMETERS TO AQTESOLV:

Q = PUMPING RATE = 52.9 ft^3/min

r = DISTANCE FROM PUMPING WELL TO WELL R20-89-113

= 45 ft

AQTESOLV- DETERMINED PARAMETERS FROM CURVE MATCHING:

W(u) = WELL FUNCTION OF u

u = WELL FUNCTION FACTOR

Δs = CHANGE IN DRAWDOWN PER LOG CYCLE (ft)

t_o = INTERCEPT OF STRAIGHT LINE AT ZERO DRAWDOWN

= 8.8 min

AQTESOLV-CALCULATED PARAMETERS FROM EQUATIONS:

T = TRANSMIISSIVITY

$$= \frac{2.3\,Q}{4\pi\Delta s}$$

= 14.12 ft^2/min

S = STORATIVITY

$$= \frac{2.25\,T\,t_o}{r^2}$$

= 0.1352

Figure 4-23. Cooper-Jacob analysis of drawdown data from well R20-89-113 versus the logarithmic plot of time, Sioux Falls airport wellfield, South Dakota.

TABLE 4-8. AQUIFER TEST SUMMARY RESULTS,
SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA

Well	Theis Method		Cooper-Jacob Method	
	Transmissivity (ft^2/min)	Storativity (unitless)	Transmissivity (ft^2/min)	Storativity (unitless)
R20-89-113	14.17	0.1348	14.12	0.1352
R20-89-114	14.89	0.1685	14.94	0.1617
R20-89-115	15.01	0.2616	15.16	0.2501
R20-89-116	14.32	0.2686	14.23	0.2632
R20-89-117	12.84	0.2767	12.87	0.2704
R20-89-118*	18.94	0.1510	--	--
R20-89-120*	18.93	0.1994	--	--
R20-89-121*	18.90	0.0878	--	--
R20-89-123	14.34	0.1728	14.92	0.1558
R20-89-124	14.86	0.1656	14.87	0.1570
R20-89-125	15.66	0.0961	15.58	0.0915
Average	14.51	0.1931	14.59	0.1856

Abbreviation: -- = no data.

* Transmissivity and storativity values for these wells were not used in calculating average values. The Cooper-Jacob method of analysis was not valid for these wells because u (the well function factor) is not less than 0.05.

Dye-Trace Analysis

Dye-trace tests were conducted to determine ground-water travel times between the river and municipal well 34A. Dye concentrations were determined (1) from onsite analysis of water by the SDGS, (2) from laboratory analysis of water by the Ozark Underground Laboratory, and (3) from laboratory analysis of charcoal packs by the Ozark Underground Laboratory (shown in Figures 4-24 through 4-26, respectively). All dye concentrations were corrected for background fluorescence in the aquifer. Background fluorescence concentrations ranged from 18 to 33 µg/l in the samples analyzed onsite.

The first dye injection occurred at well R20-89-117, approximately 40 feet north of pumping well 34A (Figure 4-7). Detectable dye concentrations first arrived at municipal well 34A after about 12 hours; peaked about 1.3 days after injection, and had moved through the aquifer after 6 days. The second dye injection occurred at well R20-89-123, approximately 140 feet north of well 34A (Figure 4-7), 3 days after the first dye injection. Detectable dye concentrations from the second injection first arrived at municipal well 34A after 7 to 9 days; peaked 16 or 17 days after injection; and were present, but declining in concentration, after 20 days when the test was terminated. Results of the three sets of analyses are discussed as follows:

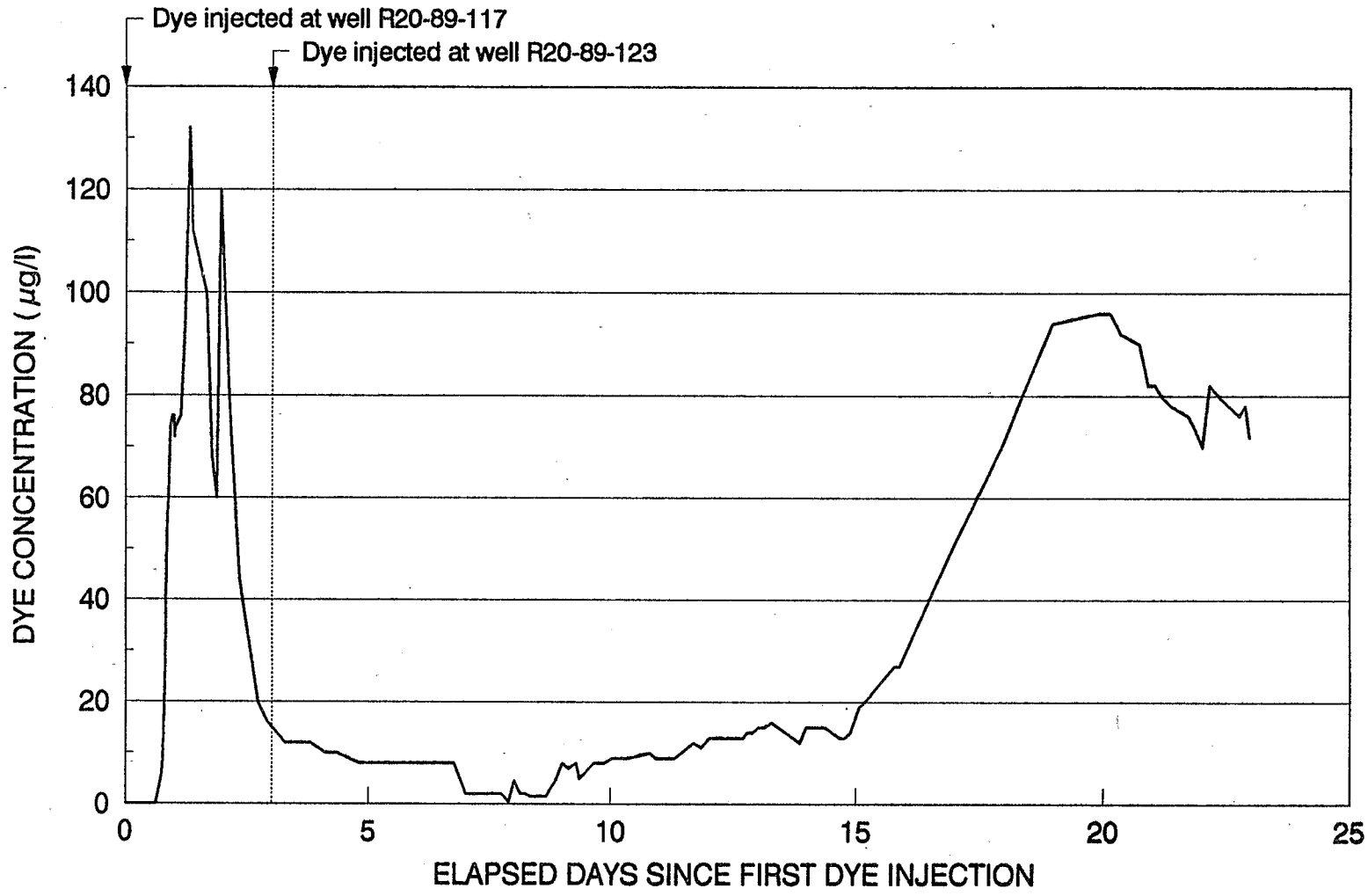


Figure 4-24. Dye concentration versus time for samples analyzed onsite from municipal well 34A, Sioux Falls airport wellfield, South Dakota.

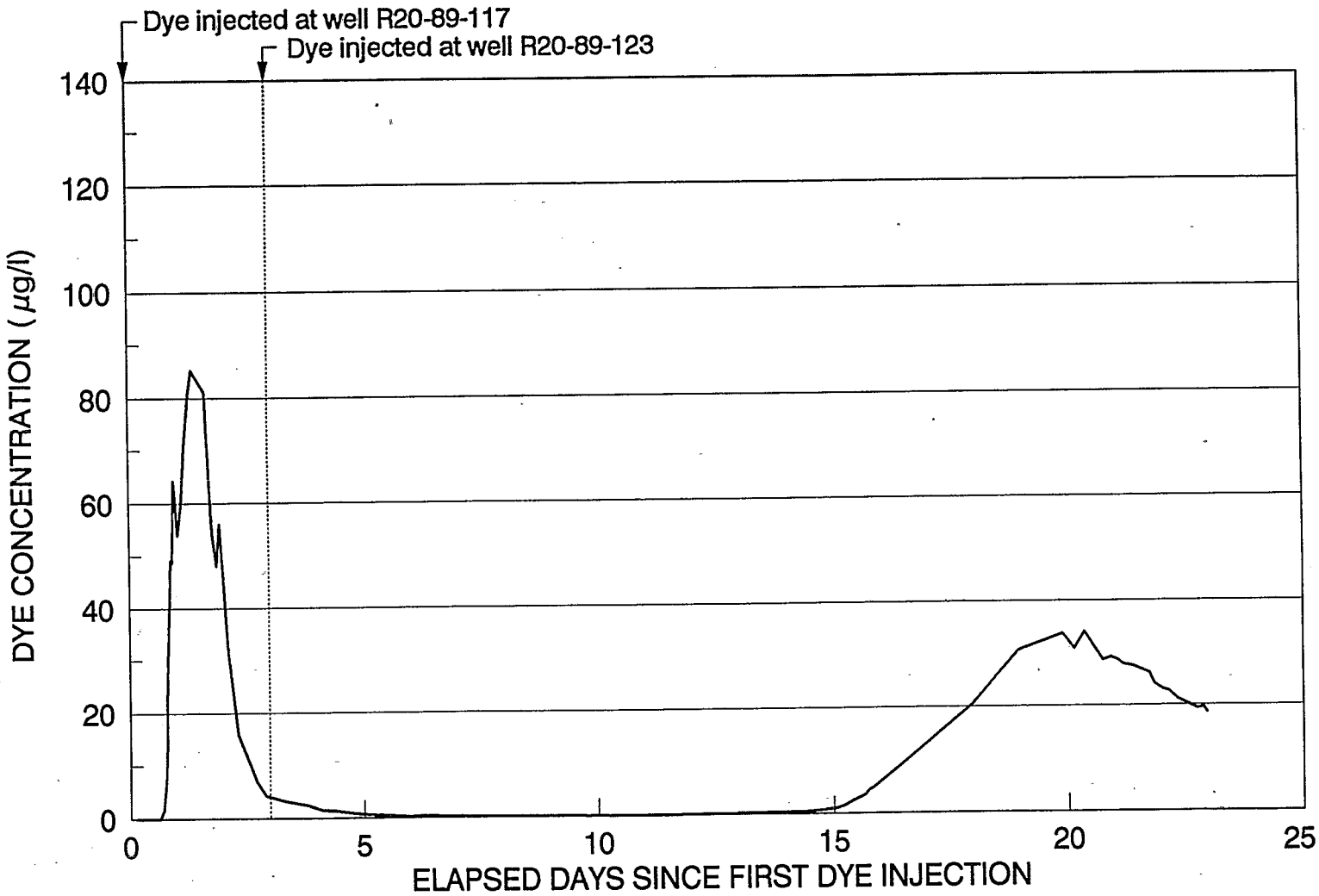


Figure 4-25. Dye concentration versus time for samples analyzed by the Ozark Underground Laboratory from municipal well 34A, Sioux Falls airport wellfield, South Dakota.

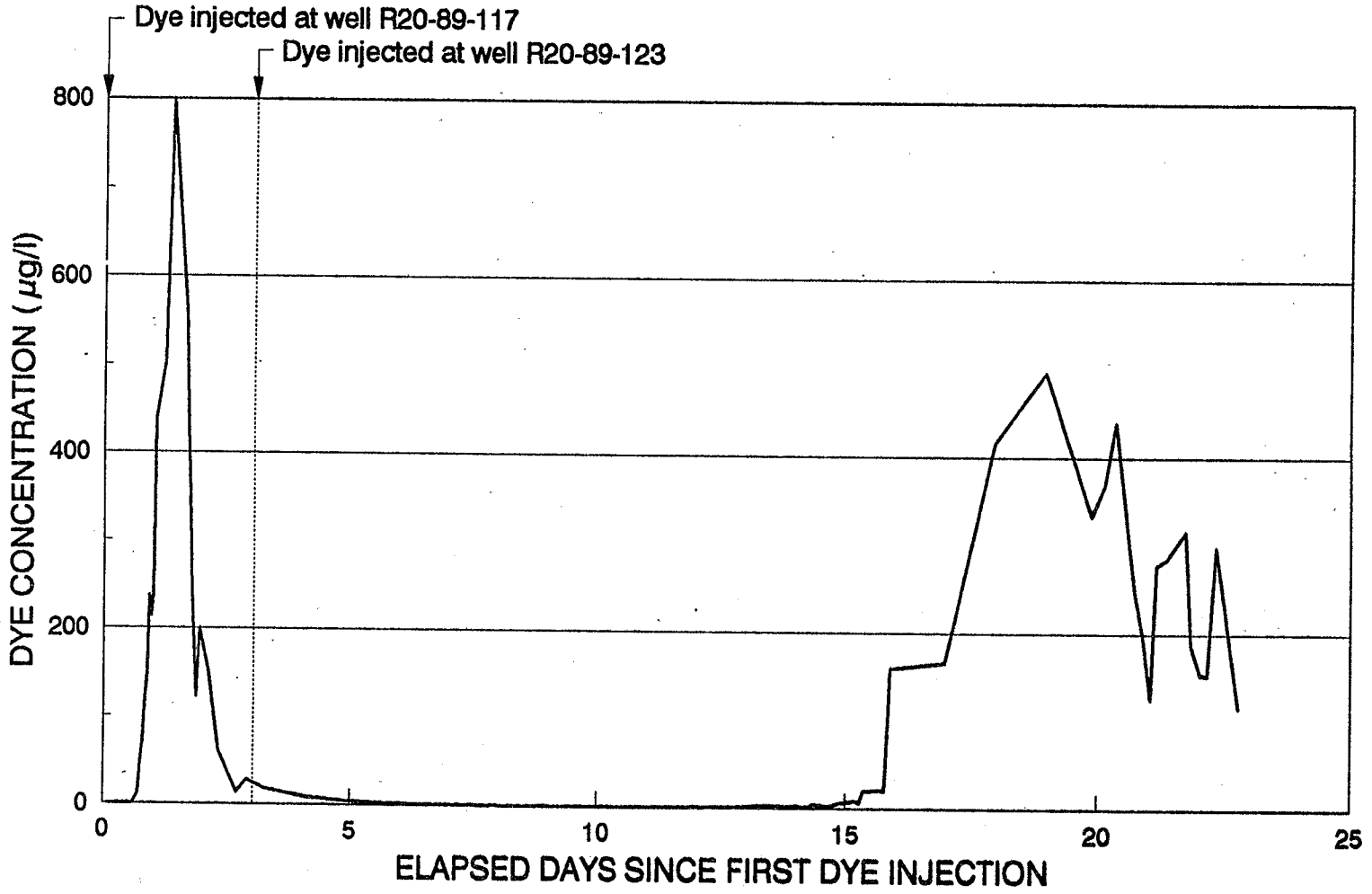


Figure 4-26. Dye concentration versus time from charcoal packs analyzed by the Ozark Underground Laboratory from municipal well 34A, Sioux Falls airport wellfield, South Dakota.

1. *Onsite analysis of samples by the SDGS.* Results indicate that the first dye began arriving at well 34A about 18 hours after it was injected at monitoring well R20-89-117 (Figure 4-24). The dye concentration peaked at 132 µg/l about 31 hours after injection, and then steeply declined. About 47 hours after injection, a second dye peak occurred at a concentration of 120 µg/l. The concentration then began to decline rapidly until it reached near-background levels approximately 7 days after injection. The reason for the second peak at 47 hours is unclear. However, it may have been caused by a lens of lower permeability sediment between the injection well and municipal well 34A. The subsurface lens could have caused the dye plume to split, with one part of the plume taking a longer flowpath to the discharge point. The dye injected at well R20-89-123 first arrived at about 7 to 9 days, peaked at about 17 days, and then gradually declined. Only an estimate can be made for the time of first arrival of the second dye pulse because of erratic instrument response prior to its arrival at well 34A.

2. *Laboratory analysis of samples by the Ozark Underground Laboratory.* Results indicate that detection of the dye at well 34A began about 15 hours after injection at monitoring well R20-89-117 (Figure 4-25). Earlier detection by the Ozark Underground Laboratory, as compared with the onsite analyses, can be attributed to a lower detection limit of the instrumentation at the laboratory. The dye concentration peaked at 85 µg/l at about 33 hours, and then began a fairly steep decline. The laboratory analyses did not show a second major peak at 47 hours as the onsite analyses did, but a small rise in dye concentration did occur at this time. The lack of correspondence between onsite and laboratory data may be a result of degradation of the dye between the time the sample was taken and the time it was analyzed. Dye decay is enhanced by light and warmer temperatures. The dye concentrations shown in Figure 4-25 reaffirm the hypothesis that at least two different peaks resulted from the first injection. The second dye injection, which occurred at well R20-89-123, first appeared about 9 days after injection, peaked at about 17 days, and then began a gradual decline in concentration.

3. *Laboratory analysis of the charcoal packs by the Ozark Underground Laboratory.* Results indicate that the dye arrived at municipal well 34A about 12 hours after injection at monitoring well R20-89-117 (Figure-26). Earlier detection by analyses of the charcoal packs, as compared with water-sample analyses, can be attributed to the accumulator effect of the charcoal packs. A charcoal pack allows otherwise nondetectable dye concentrations to accumulate to a detectable level. The dye concentration peaked at about 33 hours, and then began a fairly steep decline. Similar to the laboratory water analyses, the charcoal packs did not show a second major peak at 47 hours, as the onsite water analyses did. However, a small rise in dye concentration did occur at this time. The dye injected at well R20-89-123 first appeared about 8 days after injection and peaked at about 16 days, at which time the concentration began to decline.

Dye concentrations from the second injection, determined by analysis of the charcoal packs, are erratic; the charcoal packs were left in the discharge line for inconsistent time intervals. Because of the accumulator effect of the charcoal packs, use of inconsistent sampling intervals had a sporadic effect on the resulting concentration. Erratic trends do not occur in the concentration curve for the first dye pulse because

the charcoal packs were left in the discharge line for consistent time intervals. The amount of dye accumulated in the charcoal pack is not a direct function of the concentration and sampling interval. Although the exact function is not known, it is more closely related to concentration and the square root of the sampling interval (Tom Aley, Ozark Underground Laboratory, personal communication, 1990).

Dye concentrations determined from charcoal-pack analyses, divided by the square root of the sampling interval, are depicted in Figure 4-27. As expected, the peaks and concentrations for the first dye pulse in Figures 4-26 and 4-27 are similar, because the sampling intervals were quite similar during this phase. The major difference in the dye concentrations for the second dye injection is that the peak in Figure 4-26 occurs at about 16 days, whereas the peak in Figure 4-27 occurs at about 17 days.

The fluorescein concentrations determined by the laboratory analysis are consistently below those determined by the onsite analyses. Although the fluorescein concentrations differ, the relative peaks and troughs essentially correspond. Thus, interpretations of the time of first arrival of the dye and the time required for the dye pulse to move through municipal well 34A are similar in both cases. The dye traveled from well R20-89-117 to well 34A in 12 hours, and from well R20-89-123 to well 34A in about 8 days. In this test, the velocity of the dye in the aquifer was dependent on its location with respect to the cone of depression created by well 34A. As the dye moved from the outer to the inner part of the cone of depression, its velocity increased substantially, due to the increase in the water-table gradient.

The travel time determined from the second dye injection should be close to the travel time between the river and well 34A because the distances are similar. Specifically, the distance from well R20-89-123 (second dye injection well) to well 34A is 140 feet, and the shortest distance from the river to well 34A is 135 feet. Thus, it can be concluded that a contaminant entering the aquifer from the river would reach municipal well 34A in approximately 8 days, under similar water-level and pumping conditions. These calculations assume that contaminant transport is primarily advective. Dispersive processes of contaminant flow, in addition to advective processes, would reduce the travel time from the river to the well.

The city of Sioux Falls has many other municipal wells located along the Big Sioux River; this study focuses on the area in proximity to well 34A and the river. Most of these other wells are high-capacity Bragstad or Ranney wells. In some cases, the laterals extending from the Ranney wells are closer to the river than municipal well 34A. Ranney wells typically pump at rates of 1,500 gpm or more, as opposed to the 400-gpm discharge pumped from municipal well 34A during testing. The travel time for a contaminant entering the aquifer from the river (through primarily advective processes) near a high-capacity well is expected to be significantly less than 8 days, and may well be on the order of hours.

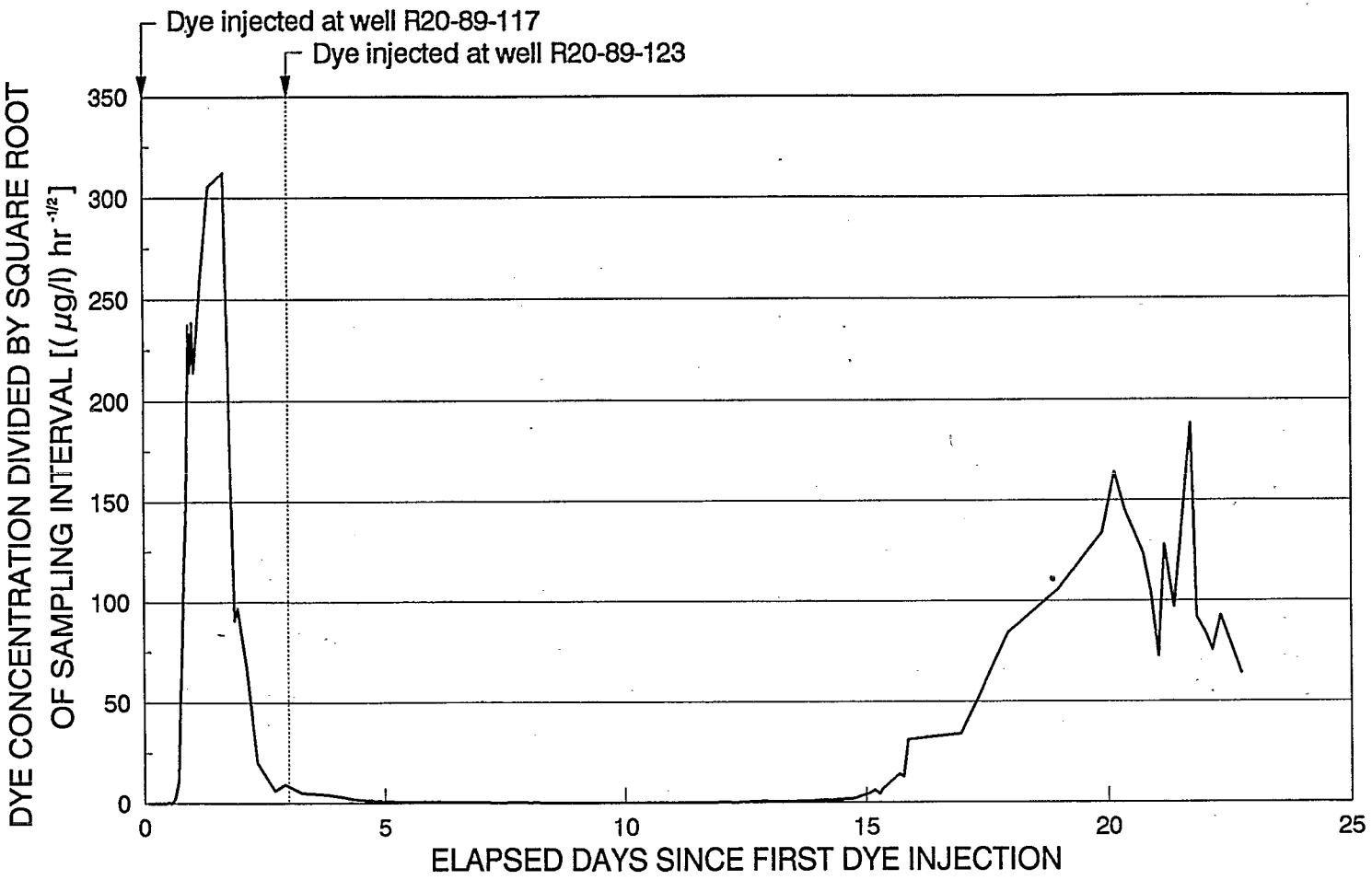


Figure 4-27. Dye concentration divided by the square root of the sampling interval versus time, for charcoal packs analyzed by the Ozark Underground Laboratory from municipal well 34A, Sioux Falls airport wellfield, South Dakota.

Ground-Water Flow Model and Flowpath Assessment

A two-dimensional, steady-state flow model of the study area was developed to further evaluate surface-water and ground-water relationships, to delineate capture zones, and to enable particle tracking from simulated spill sites. The FLOWPATH modeling code (Waterloo Hydrogeologic Software, 1989) was used to develop this model. FLOWPATH is a flow and particle-tracking numerical model designed for an IBM personal computer that allows calculation of steady-state, hydraulic-head distributions; ground-water velocities; pathlines; travel times; and capture zones. Some of the features of the FLOWPATH model are as follows:

- It allows a maximum of 10,000 grid cells.
- It handles confined, unconfined, and leaky confined conditions.
- It accommodates heterogeneous aquifer properties.
- It accepts spatial variations in ground-water recharge from infiltration.
- It accounts for surface-water interaction with the aquifer.
- It allows for multiple pumping and injection wells.
- It uses the finite-difference method to calculate ground-water flow, and the particle-tracking method to calculate pathlines and travel times.

Grid and Boundary Design--

A 64- by 74-node, rectangular grid with a variable-grid spacing was developed for the model (Figure 4-28). The grid spacing was more dense in areas where there were pumping wells. A tighter grid spacing was incorporated in these areas to accommodate exact geographical locations of wells. The 19 municipal wells used in the wellfield during 1988 were assigned representative locations in the grid. Average annual pumping rates for 1988 were included as an attribute with each well. Boundary conditions for the model were defined by nodes coinciding with the east, west, and south aquifer boundaries. For the initial design of the model, these boundaries were considered impermeable. The northern boundary of the aquifer is hydraulically connected to the Big Sioux aquifer. To simulate this condition, a constant-head boundary of 1,423 feet above mean sea level was incorporated in the design of the model. Surface-water nodes were placed along each node in the model where the Big Sioux River or the diversion canal was present. In all, 119 surface-water nodes were incorporated in the model. Representative surface-water elevations and river-bottom elevations were included as attributes of the surface-water nodes.

Aquifer Properties--

An initial hydraulic conductivity of 800 ft/d was assigned uniformly to the aquifer, based on results of the aquifer test. The average transmissivity calculated from the aquifer test data was 14.6 ft^2/min (Table 4-8), or 21,000 ft^2/d. The average saturated thickness in the area of the aquifer test is 26 feet. This yields an average hydraulic conductivity of about 800 ft/d. The actual hydraulic conductivity may vary considerably, but 800 ft/d but was used as a starting point in calibrating the model and was modified as necessary during model calibration. A hydraulic conductivity of 0.75 ft/d was assigned to the river bed. This represents an average of the infiltration rate of 0.5 to 1.0 ft/d, as reported by Jorgensen and Ackroyd (1973). Results from the aquifer test yielded an average storativity of 0.19. In an unconfined aquifer, the storativity is approximately the same as the effective porosity. Thus, a porosity of 0.2 was used for the modeled area.

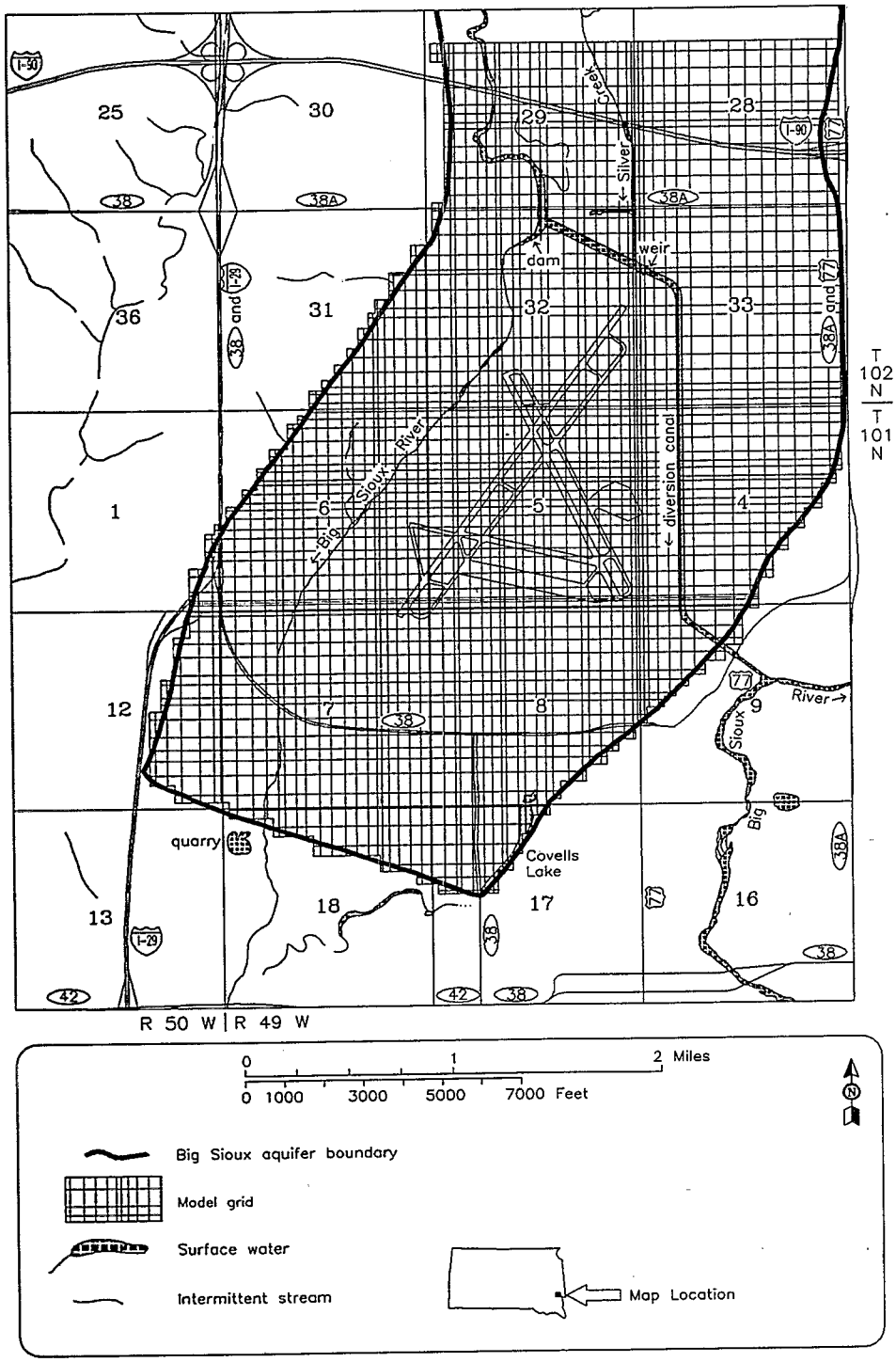


Figure 4-28. Variable-spaced grid for the ground-water flow model, Sioux Falls airport wellfield, South Dakota.

Koch (1982) used an average annual recharge value of 6.9 inches per year (in/yr) in calibrating a digital model for a larger area of the Big Sioux aquifer. Hedges et al. (1985) estimated the natural recharge rate to the entire Big Sioux aquifer to be 6.5 in/yr. A clay layer overlies the aquifer in the wellfield. Therefore, an average annual recharge of 6 in/yr was assumed for this study. A recharge amount of 6 in/yr permitted a good fit of the model to observed water levels. Bottom elevations of the aquifer were determined from test holes in the area and incorporated into the model. The model calculated aquifer transmissivities at each node by using the test-hole elevation data.

Model Calibration--

The primary objective of model calibration is to minimize the error of the resultant output by comparing generated head values with observed head values to improve interpretation of ground-water flow. The calibration was conducted by modifying the variables of the aquifer properties until the model generates a set of hydraulic heads similar to observed hydraulic heads. The model was calibrated to hydraulic conditions observed in 1990 by Waterloo Hydrogeologic Software (designer of the FLOWPATH software code) according to the following steps:

1. Flux nodes were added to the west, east, and south boundaries of the aquifer to account for surface-water runoff from topographic highs surrounding the domain. Runoff becomes ground-water recharge because of infiltration along the domain boundary. The flux nodes on the west and east were assigned values of 0.07 ft/d; flux nodes on the south were assigned values of 0.03 ft/d.

2. Hydraulic conductivity values were modified from the default value of 800 ft/d, which was initially assigned to all nodes except impermeable boundary nodes. The default hydraulic conductivity value was changed to 650 ft/d. Additional sets of nodes near the center of the aquifer were modified to values of 400, 500, and 900 ft/d.

3. The leakage factor was increased fivefold to a value of 0.3, based on the following rationale. The leakage factor at surface-water nodes is reflective of how well the surface water is hydraulically interconnected to the adjacent aquifer. The larger the number, the better the interconnection. The leakage factor is derived from three terms: the hydraulic conductivity of the stream bed, the thickness of the stream bed, and the relative area of the stream to the area of the node that it occupies. A leakage factor of 0.06 applies to most surface-water nodes in this area based on estimated stream infiltration rates, stream bed thicknesses, and the area of the river relative to the surface-water node. During initial calibration of the model, the river did not significantly influence the adjacent water-table elevations in the aquifer. Based on observed water levels, changes in river level directly affect ground-water levels. To simulate this effect in the model, the leakage factor was increased fivefold to a value of 0.3. This value created the desired effect on the hydraulic heads in the aquifer.

Following calibration of the model, a final simulation was run. The hydraulic heads calculated by the model for each node were quite similar to observed hydraulic heads, reflecting an acceptable calibration. Water-table contour lines generated by the model are shown in Figure 4-29. These contour lines compare favorably with the observed hydraulic conditions depicted by the contour lines in Figure 4-11. In most cases, the difference between observed and calculated heads is no more than 0.5 foot, although there are some areas where the difference approaches 1 foot. In these areas, the

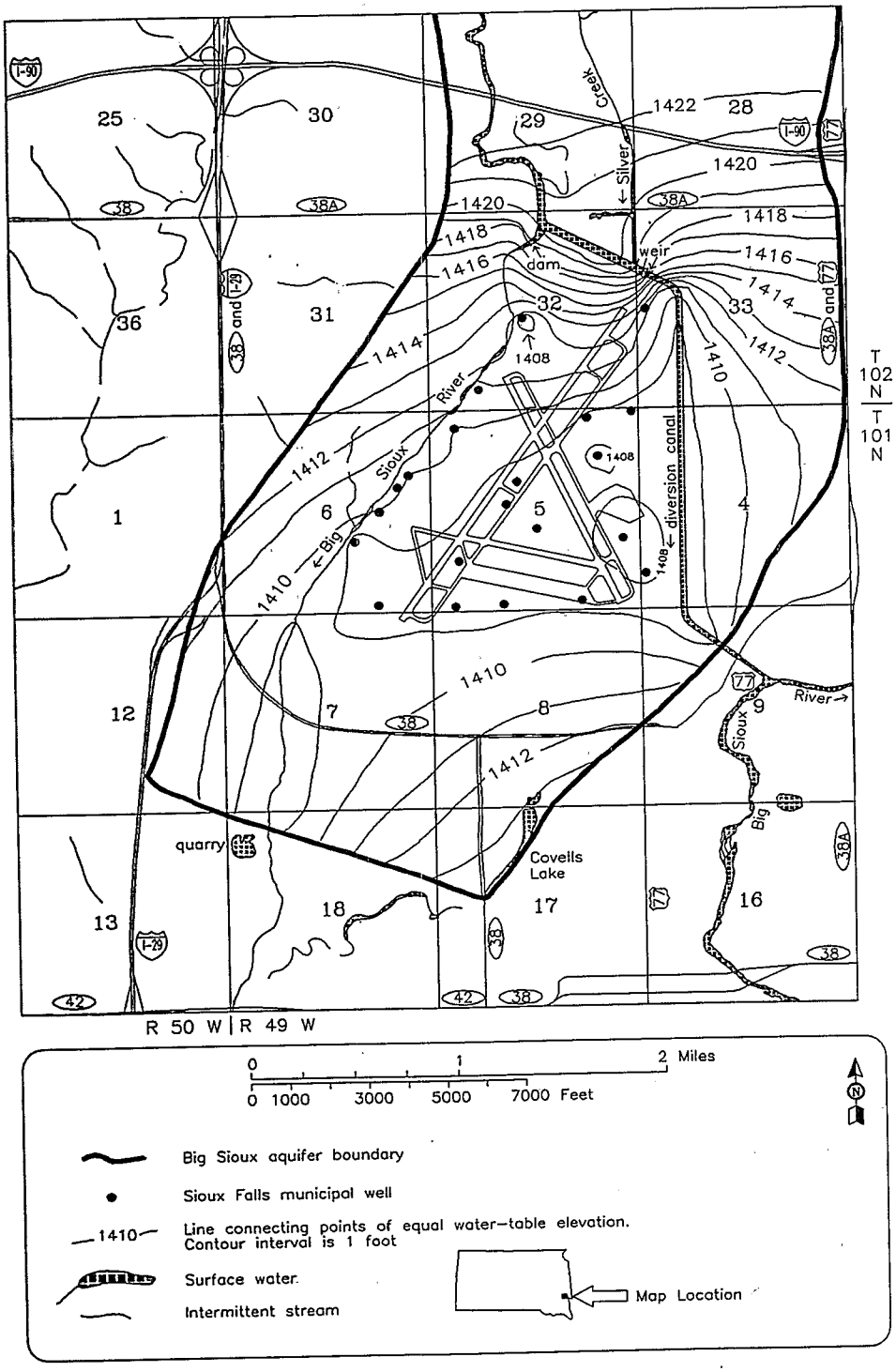


Figure 4-29. Water-table configuration map generated by the calibrated ground-water flow model, Sioux Falls airport wellfield, South Dakota.

calibration is considered acceptable because the degree of accuracy of observed water levels is ±0.5 to 1 foot. Calibrated hydraulic head values show that the Big Sioux River is gaining ground water (a gaining river) to the north of the wellfield. The river begins losing water to the aquifer (a losing river) as it flows adjacent to the wellfield. The wellfield induces a significant amount of recharge from the surface water. After the river flows to the south of the wellfield, it once again becomes a gaining river. The diversion canal reflects a similar hydrologic relationship to the aquifer; it changes from a gaining mode to a losing mode, and then back to a gaining mode.

Model Output--

Following successful calibration, the model was used to generate time-related capture zones and perform particle tracking from simulated spill sites. Three separate travel-time criteria were selected to depict enlargement of capture zones: 1, 2, and 5 years. The Big Sioux aquifer has characteristically high transmissivity values (greater than 13,400 ft^2/d), so 1- and 2-year travel times were used to depict FLOWPATH results to facilitate monitoring site selection in relation to priority sources. A 5-year travel-time criteria was selected to assess the validity of the preliminary WHPA. The current delineation criteria and method are recharge boundary and hydrogeologic mapping, respectively.

Time-related capture zones--One of the major objectives in developing an accurate ground-water flow model was to calculate capture zones for each well in the wellfield. Each municipal well was assigned 100 particles to be released from a radius of 150 feet. The model then tracked the particles upgradient for a specified time period. The velocity of the particle tracking is dependent on several factors, including hydraulic conductivity, hydraulic gradient, effective porosity, and retardation factor. The hydraulic conductivity and hydraulic gradient factors were determined during the model calibration. Two effective porosities, 0.15 and 0.25, were used in the model simulations. These numbers represent lower and upper values of effective aquifer porosities based, in part, on the average storativity value of 0.19 obtained from the aquifer-test analysis. By using different effective porosities, the sensitivity of the model to this parameter can be demonstrated. A retardation factor of 1 was used in all the model runs. The retardation factor is a variable that can be used to account for the adsorption of particles to the aquifer materials. Because these simulations are based on the predicted behavior of a generally conservative contaminant, no attempt was made to determine specific retardation factors. A retardation factor of 1 indicates that no adsorption occurs. FLOWPATH assumes that contaminant transport is advective. Therefore, no effects of contaminant dispersion are reflected in the model results.

One-year TOT capture zones are shown for the municipal wells in the study area in Figures 4-30 and 4-31 for aquifer porosities of 15% and 25%, respectively. Two-year TOT capture zones (Figures 4-32 and 4-33) and 5-year TOT capture zones (Figures 4-34 and 4-35) for aquifer porosities of 15% and 25%, respectively, are also shown. The sensitivity of the model to porosity is apparent in this set of figures. Significantly larger capture zones result from the 15% porosity than from the 25% porosity.

The 1-year TOT capture zones of many of the wells intercept either the surface-water bodies (river or diversion canal) or the industrial and commercial area south of the wellfield. In reality, the capture zones of many of the wells along the river will intercept the Big Sioux River in much less than a year. As the results of dye-tracing indicate, capture zones from some of the wells will intercept the river in a matter of hours or days.

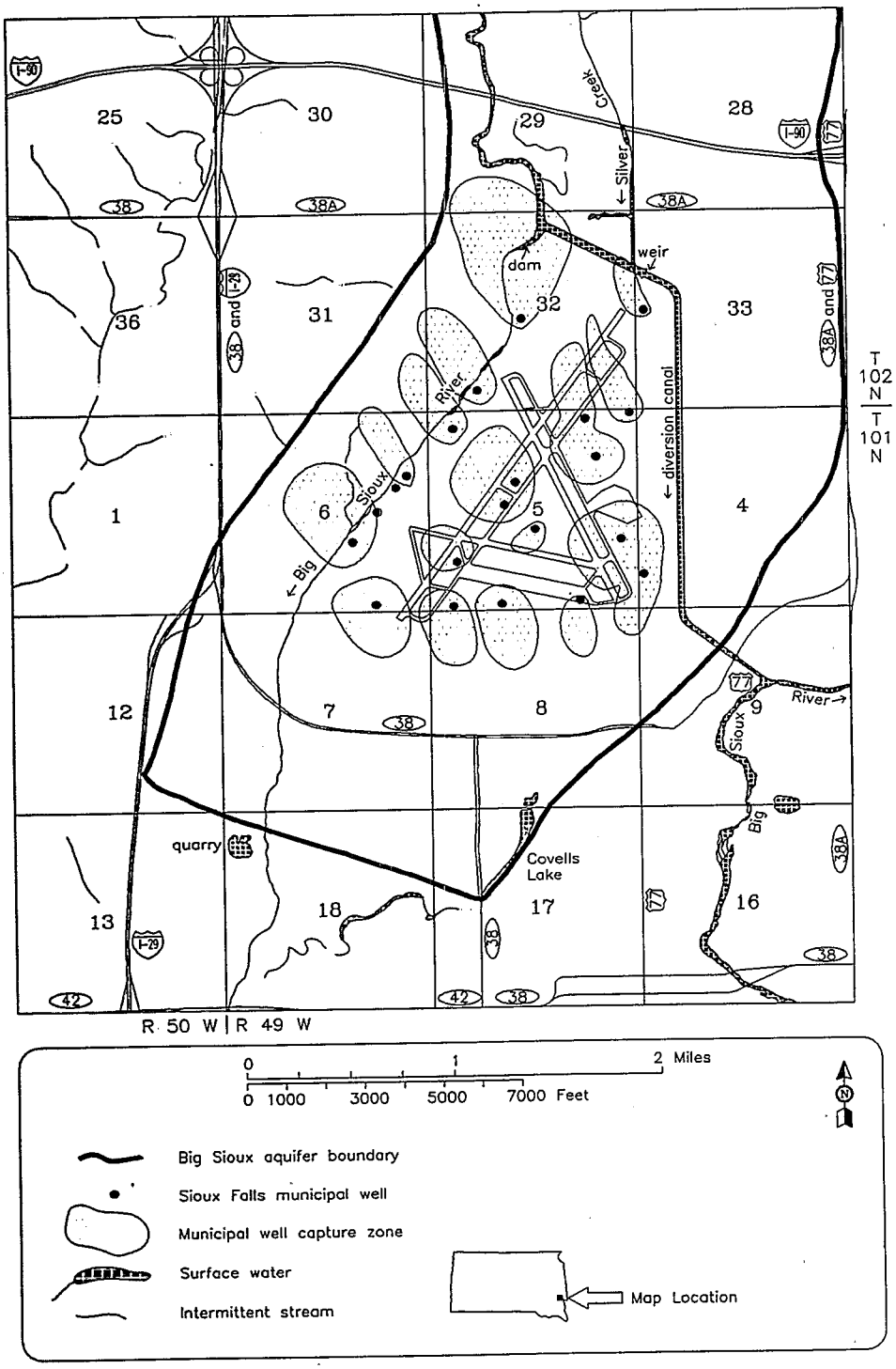


Figure 4-30. One-year TOT capture zones at a porosity of 15% for the Sioux Falls airport wellfield, South Dakota.

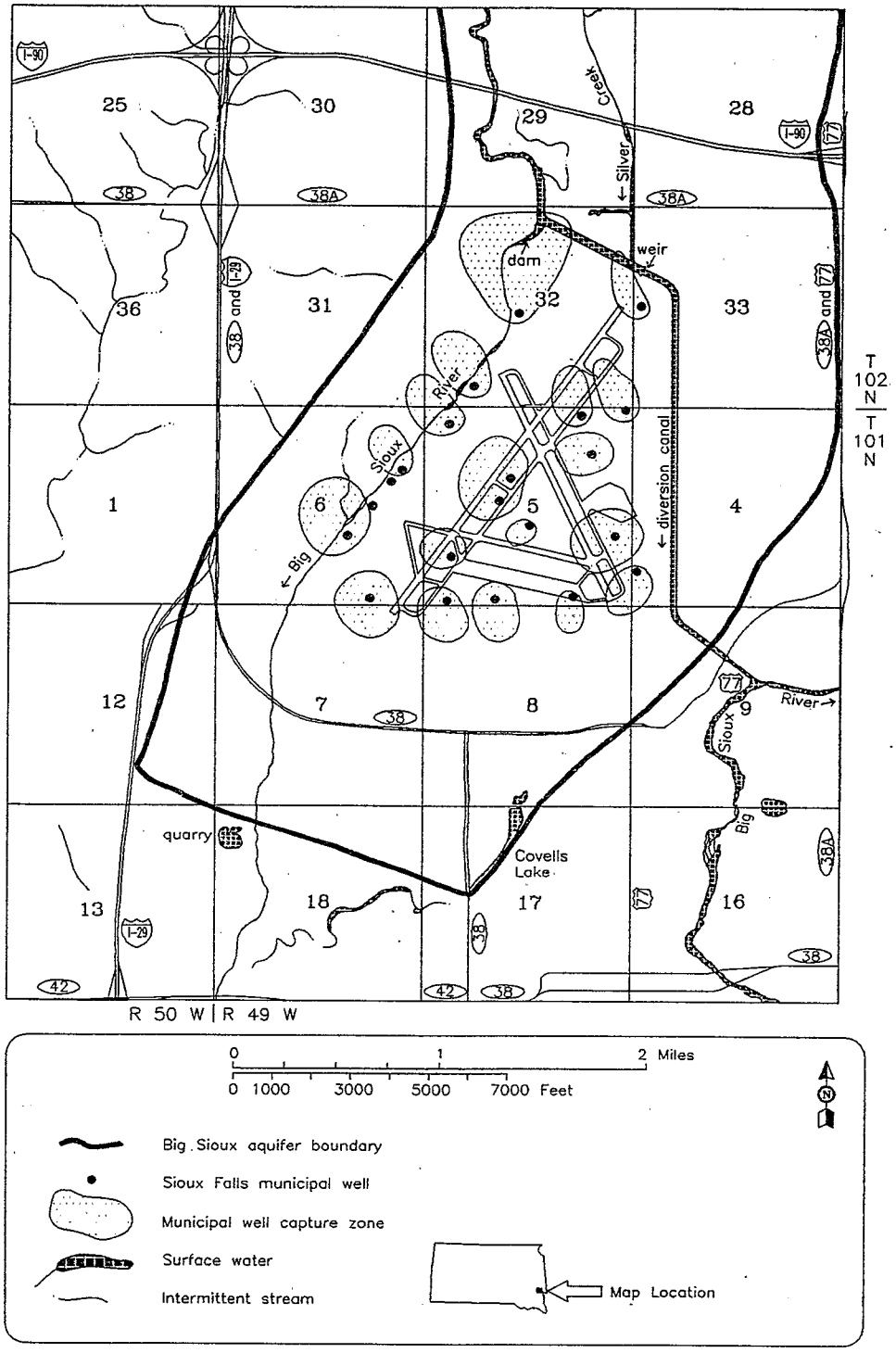


Figure 4-31. One-year TOT capture zones at a porosity of 25% for the Sioux Falls airport wellfield, South Dakota.

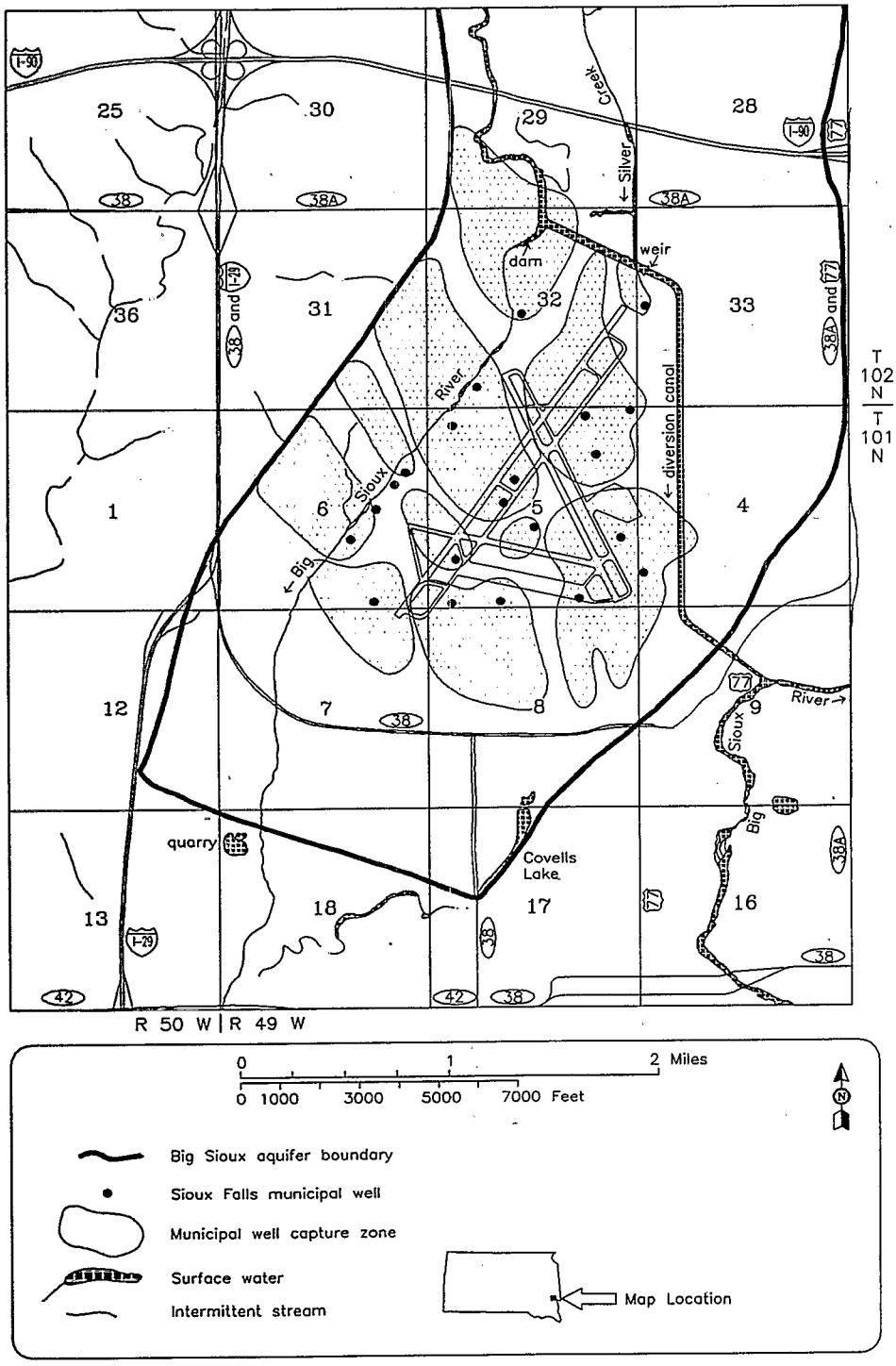


Figure 4-32. Two-year TOT capture zones at a porosity of 15% for the Sioux Falls airport wellfield, South Dakota.

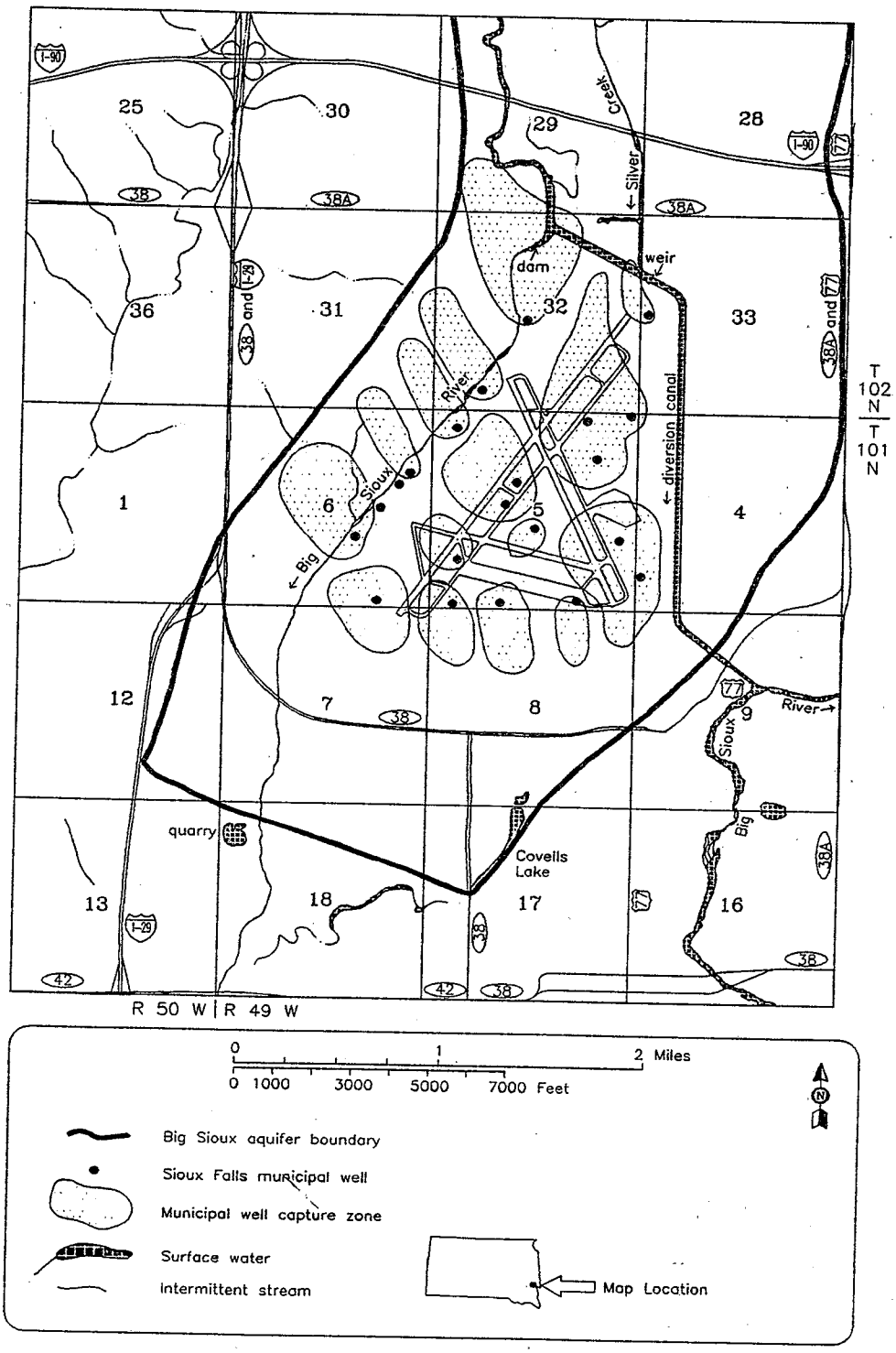


Figure 4-33. Two-year TOT capture zones at a porosity of 25% for the Sioux Falls airport wellfield, South Dakota.

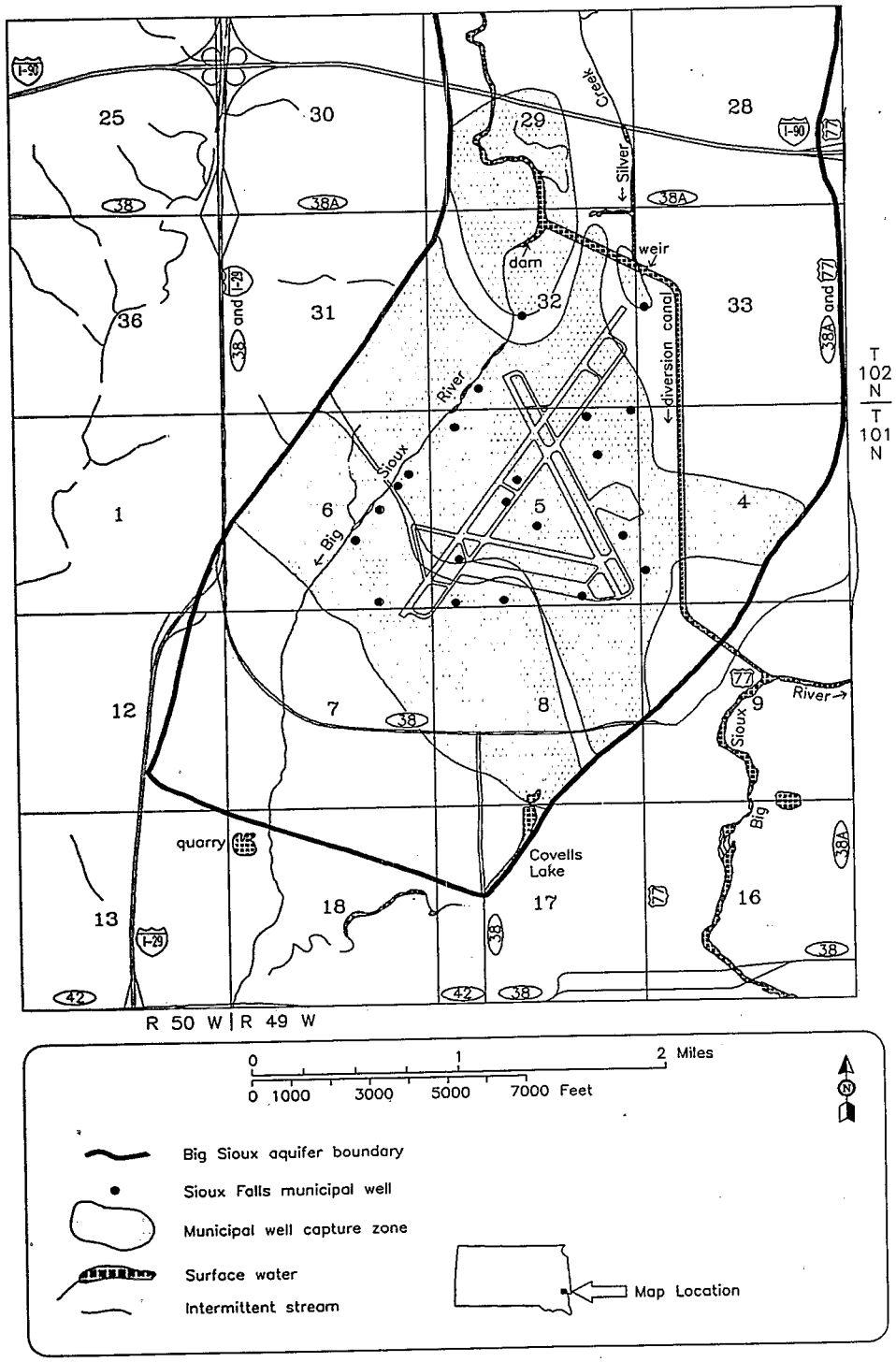


Figure 4-34. Five-year TOT capture zones at a porosity of 15% for the Sioux Falls airport wellfield, South Dakota.

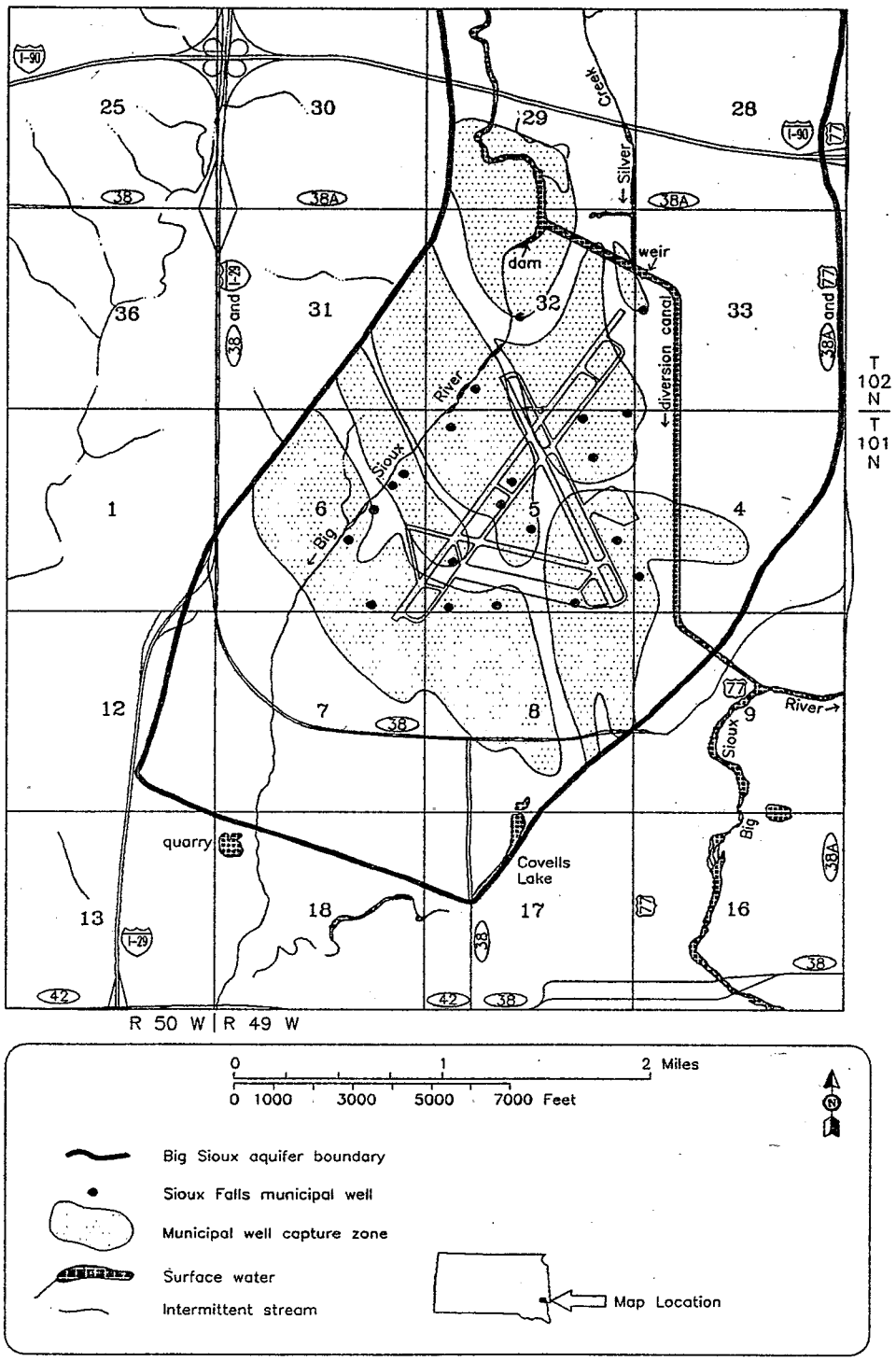


Figure 4-35. Five-year TOT capture zones at a porosity of 25% for the Sioux Falls airport wellfield, South Dakota.

When the travel time is increased to 2 years, some of the capture zones of wells in the center of the wellfield intercept the surface water. It can be seen from Figure 4-32 that, at a porosity of 15%, after 2 years many of the capture zones reach the aquifer boundary.

After 5 years (Figures 4-34 and 4-35), more surface water and a greater length of the aquifer boundary are intercepted by the capture zones. A capture zone approaching an impermeable boundary, such as the glacial till, will begin to bend upgradient and away from the boundary. For this reason, the exact configuration of flow lines near the boundary has a greater error of uncertainty. In addition, the accuracy and quantity of water-level elevation data near the aquifer boundaries are insufficient to define water-table contours and, consequently, flow lines. Therefore, capture zones that approach the boundaries of the aquifer should be considered approximate.

The capture zones presented here are reflective only of ground-water travel times. Because of the close interaction between the river and aquifer, surface water must also be considered as a mode of contaminant transport into the wellfield. Surface-water velocities are much greater than ground-water velocities. If the zone of contribution (ZOC) is viewed in terms of both surface water and ground water, which it must be, then the zone becomes much larger than that produced by the model results. The ZOC actually includes all upstream portions of the Big Sioux River and its tributaries. Because the river and its tributaries act as a central drain for shallow ground water in the basin, all upgradient portions of the Big Sioux aquifer are also in the ZOC to the airport wellfield. Thus, protection of the quality of water in the river and upgradient portions of the aquifer is warranted, over as large an area as is practical.

Simulated spill sites--The model was used to simulate spills with particle tracking for four scenarios: (1) from the industrial and commercial areas south and east of the wellfield, (2) from the old Elmwood Landfill southwest of the wellfield, (3) from a sand and storage area west of the landfill, and (4) from a location at the north end of the wellfield. The threat of a potential point-source spill is greatest in the industrial and commercial areas (Figure 4-36), therefore, the model was used as an aid in understanding the impact of an unremediated spill in these areas. Figure 4-36 shows the particle tracking from these simulated spills into the wellfield after 2 years. A porosity of 15% and a retardation factor of 1 were assumed to simulate a worst-case scenario. (Larger capture zones result and, therefore, overprotection occurs.) The travel times calculated by the FLOWPATH model assume advective transport.

Results from several particle tracking runs indicate that contamination north and south of the wellfield would readily migrate to the municipal wells in 1 to 2 years. Contamination east of the wellfield would move west toward the diversion canal and then be diverted south along the canal. At this point, some of the contamination may be discharged into the diversion canal. This could threaten the city's surface-water intake structures near the southern end of the diversion canal. It is not likely that contamination from this area would enter any of the municipal wells after 1 year. However, one of the 2-year municipal well capture zones and two 5-year capture zones intercept the diversion canal downstream from the simulated spill site in Section 33 (Figures 4-32, 4-34, and 4-35). Therefore, it is possible that contaminants could enter the wellfield by way of the diversion canal. Simulated spill sites in the vicinity of the old Elmwood Landfill and the sand and salt storage area indicate that contamination from these areas would flow into the Big Sioux River downstream from the wellfield, and thus would not affect the wellfield.

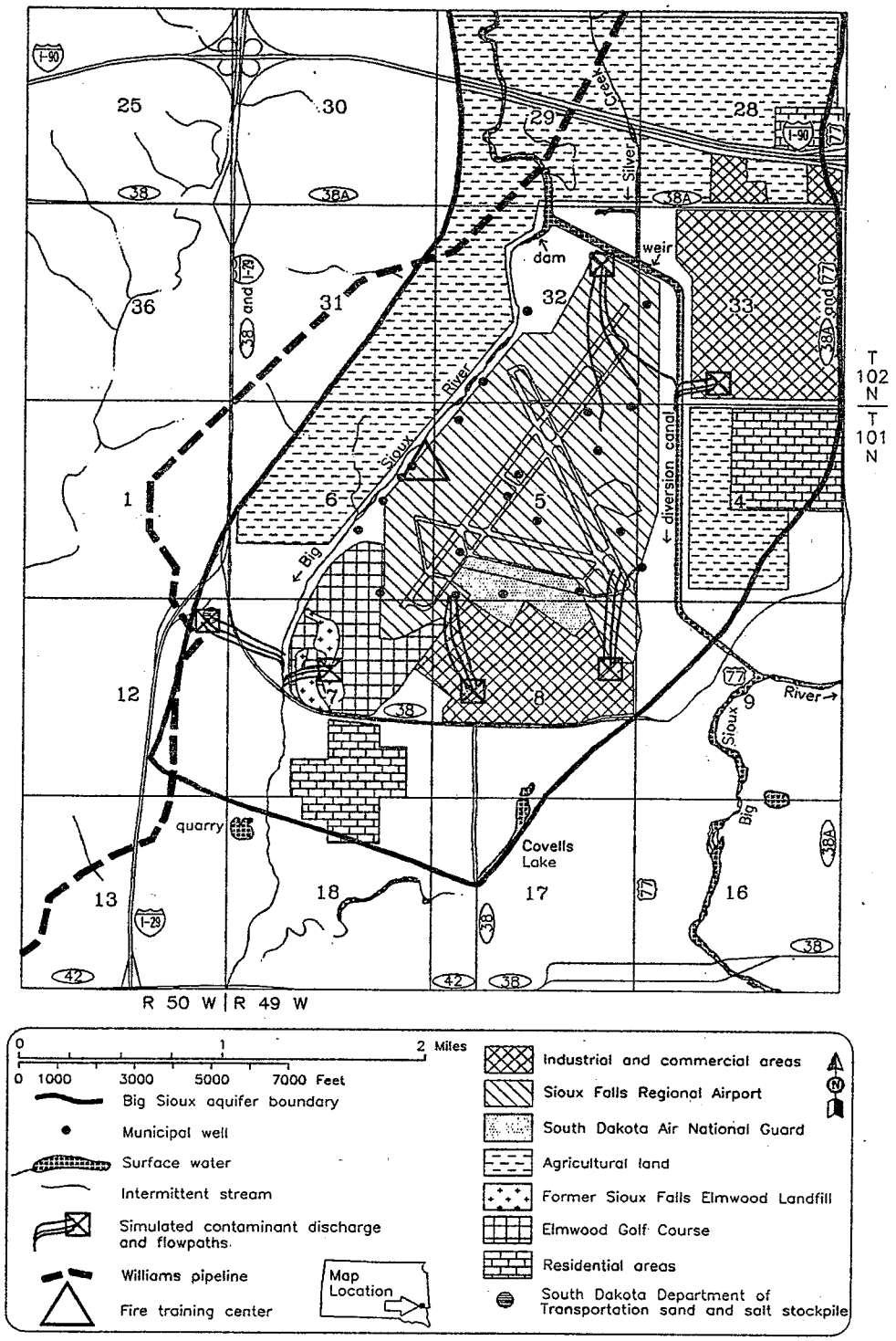


Figure 4-36. Simulated contaminant flowpaths from discharge sites for a 2-year TOT, Sioux Falls airport wellfield, South Dakota.

Modeling results aided in understanding the complex surface-water and ground-water interaction in the wellfield area. These results will also be used as a guide in the development of a wellfield management scheme and in the formulation of wellhead protection monitoring options. Modeling results are based on the calibrated model, which, in turn, are based on observed hydraulic conditions depicted in Figure 4-11 and average annual pumping rates in 1988. The wellfield and aquifer are a very dynamic system in which pumping rates and hydraulic heads are continuously changing. Therefore, model results should be used only as a tool in understanding the flow system. Ground-water flow directions and velocities vary throughout the year in response to pumping regimes.

Refined Wellhead Protection Area

The city and the county chose the hydrogeologic mapping method to delineate the preliminary WHPAs in Minnehaha County (Figure 4-4); the corresponding delineation criteria is recharge boundary. This method of delineation resulted in WHPAs that are actually aquifer-protection zones rather than subunits of the aquifer. Initially, WHPA delineation using the calculated, fixed-radius method, showed that the majority of the aquifer was included for protection by the areas calculated for each well. Individual areas calculated by fixed radius either overlapped, were close to one another, or intersected an aquifer boundary. The decision was made to use the hydrogeologic mapping method because:

- The areas determined using the fixed-radius method are underprotective because the method does not account for well interference.
- The actual size of the fixed-radius areas is difficult to calculate accurately because the calculation is dependent on variable pumping rates and times for each well, as well as river recharge to the aquifer.
- Implementation and management of wellhead protection are easier for the entire aquifer than for discrete areas. This approach is also more technically defensible, if challenged in court.
- Inclusion of the entire aquifer as the WHPA supports effective, long-term protection of the ground-water supply, considering the complex interconnection between the river and the aquifer.

The Aquifer Protection Overlay Districts in the study area that were established by the city and the county prior to this research are illustrated in Figure 4-4.

The results of the modeling exercise also indicate that the capture zones of individual wells (for 2- and 5-year criteria) cover the majority of the aquifer in the vicinity of the airport wellfield (as did the fixed-radius method). These results justify the use of the hydrogeologic mapping method for delineation. There are small portions of the aquifer which occur outside of the current WHPA that should be included to ensure water supply protection. These excluded areas are the result of using political boundaries rather than strictly geologic boundaries for delineation. The areas are located in Section 31 [Township 102 North (T102N), Range 49 West (R49W)] and near the center of the north edge of Section 17 (T101N, R49W; Figure 4-4). Other than these two inclusions, no modifications to the WHPA are recommended. Consideration should be given to adding buffer zones along the aquifer boundaries to regulate activities in areas that contribute surface runoff directly to the aquifer.

Research results demonstrate that the Big Sioux River is a major source of recharge to the aquifer. Thus, the river must be considered a potential line source of contamination to the ground-water supply. Protection of the river from contamination must be an integral part of an effective WHPP. Agreements with political entities upstream from the wellfield should be negotiated to maintain or improve the present river water quality (1) because of the river's contribution to the ground water in the airport wellfield, and (2) because the city of Sioux Falls also uses surface water directly from the river.

WELLHEAD PROTECTION MONITORING PROGRAM

OBJECTIVES

The objectives of the Sioux Falls wellhead protection monitoring program at the airport wellfield are:

- To document ambient water quality conditions during different seasons and precipitation conditions
- To serve as an early-warning detection system for contamination

Both objectives pertain to ground water in or near the wellfield and to surface water around and upstream from the wellfield. The degree to which these objectives are achieved is dependent on economic considerations, the level of protection that is acceptable to the city and enforceable under current regulations, and the limitations of current technology. Other ideas and examples beyond those presented in this report which may be of use in formulating a monitoring program, monitoring network, and data-management system can be found in Todd et al. (1976), Tinlin (1976), and Hampton (1976).

MONITORING TYPES AND GENERAL LOCATIONS

Three general types of monitoring pertain to the airport wellfield:

- Line-source monitoring
- Point-source monitoring
- Nonpoint-source monitoring

Line-source monitoring should be a combination of surface-water and ground-water monitoring. Recommended point-source and nonpoint-source monitoring will consist only of ground-water monitoring.

Line-Source Monitoring

The purpose of line-source monitoring is to examine the water quality of the river, the diversion canal, and the aquifer. The line sources to be monitored include the Big Sioux River and the diversion canal, which lie on the west, north, and east sides of the airport wellfield. Contaminants introduced to the river travel rapidly downstream and may enter the aquifer at the airport wellfield from three sides in a very short time.

Currently, the dangers to the water supply exceed any protection afforded by line-source monitoring strategies and technologies. For example, the time required to collect water samples and to conduct laboratory analyses exceeds the travel time necessary for water to flow from a line source to some of the municipal wells. In situ, real-time, analytical capabilities are needed to provide a relatively continuous and current assessment of water quality for this dynamic flow system. The line-source monitoring sites recommended here are of limited use for early warning detection of contaminants in the airport wellfield area. However, frequent line-source monitoring will provide baseline water quality data. With the development and implementation of newer, in situ technologies, line-source monitoring sites may become a more important component of an early warning system.

The few water samples taken from the river for the purposes of this research reveal no current violations of state and federal drinking water standards. Thus, the present river water quality does not adversely affect the ground-water quality of the airport wellfield. Because there is the possibility of accidental or illegal discharges of hazardous and toxic substances to the river, and because of the significant hydraulic interconnection between the river and the aquifer, the river must be considered as a potential source of contamination to the airport wellfield.

Two surface-water line-source monitoring sites are recommended. One site should be located upstream from the divergence of the Big Sioux River and the diversion canal along the border of Sections 29 and 32, as shown in Figure 4-37. A second site should be located at the diversion canal on the east side of the airport wellfield (Section 4, Figure 4-37); this location is downstream from the industrial and commercial area, yet upstream from the southern surface-water intake structures. These two monitoring sites will provide water quality data representative of surface water that is induced to the aquifer or pumped into the intake structures. Data from these monitoring sites, therefore, may trigger contingency responses for the wellfield.

Nine line-source sites for ground-water monitoring are recommended at the locations shown in Figure 4-37 (Sections 32, 5, 6, and 4), between the river or diversion canal and some of the municipal wells. These locations were selected based on the criterion that the municipal well, or group of wells, have a capture zone that intercepts either the river or diversion canal (Figure 4-32). Monitoring-site selection is based on model predictions for a worst-case scenario, assuming a 2-year travel time, a porosity of 15%, and a retardation factor of 1 (Figure 4-32). Data from ground-water, line-source monitoring will provide an early warning detection ranging from hours to more than 1 year. Ground-water, line-source monitoring will function primarily to provide data on the ambient trend conditions of the surface water that is induced into the ground water. As in the case of surface-water, line-source monitoring, in situ, real-time devices are required to permit the ground-water component to function as an effective early warning system.

Point-Source Monitoring

The purpose of the point-source monitoring is to examine the water quality between key potential point sources and both the municipal wells and the surface-water intake structures. Only those point sources identified in the source assessment process were considered in siting monitoring stations. Of the point sources identified, the sand and salt storage area and the former Elmwood Landfill (Figure 4-37) were not considered a threat to the airport wellfield, based on simulated spill

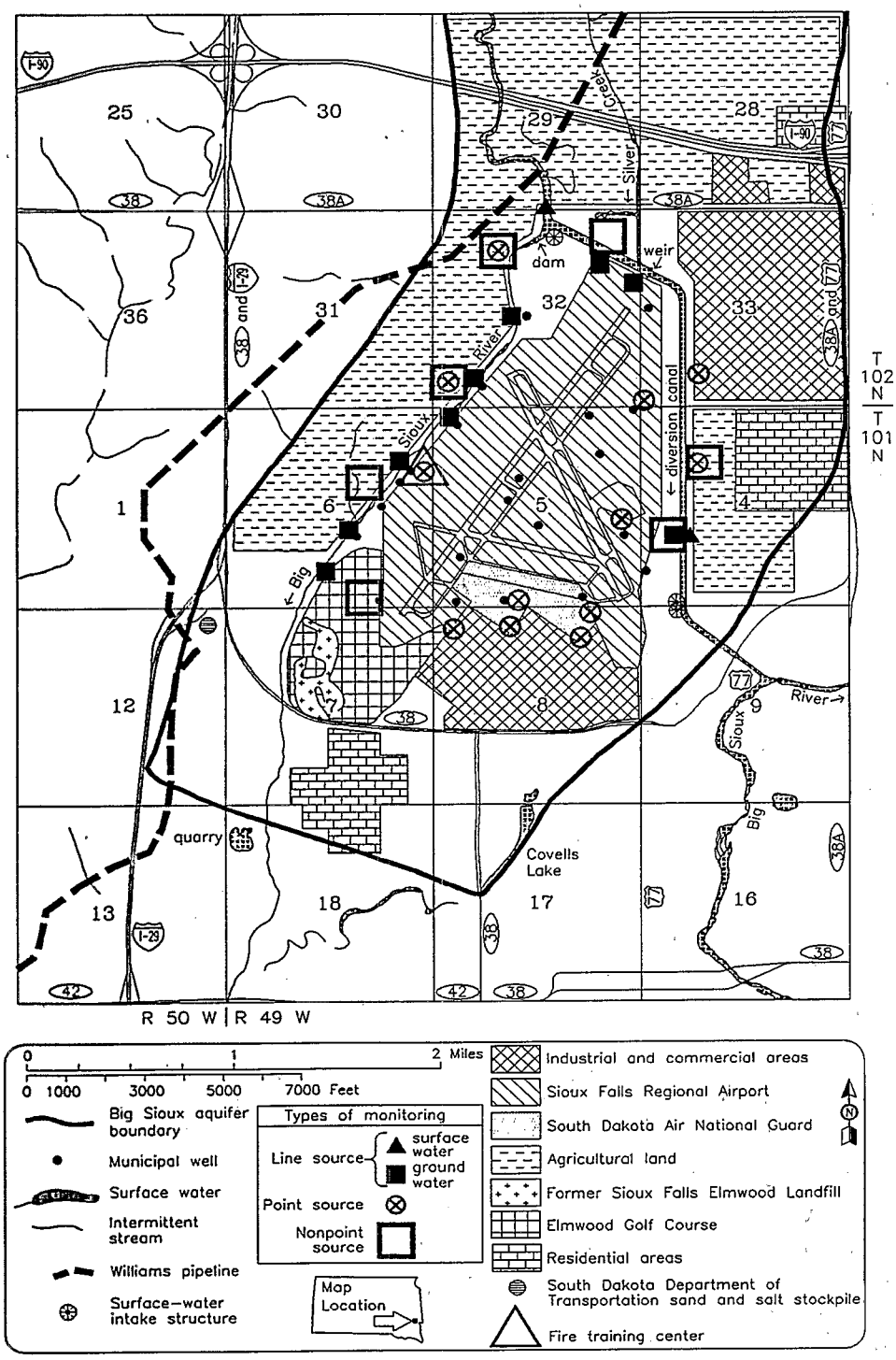


Figure 4-37. Recommended monitoring locations for line, point, and nonpoint sources, Sioux Falls airport wellfield, South Dakota.

results (Figure 4-36). However, monitoring is recommended downgradient of the South Dakota Air National Guard facility, the Williams pipeline, the industrial and commercial areas, the fire training center, and the Sioux Falls Regional Airport.

Ground water beneath the South Dakota Air National Guard facility (Figure 4-37) is drawn into the municipal water distribution system. The Air National Guard is currently remediating known petroleum ground-water contamination and is investigating other potential sources at their facility (Kim Kurtenbach, SDDENR, personal communication, 1992). At present, monitoring is recommended for two contaminant sources on this property. Upon completion of the Air National Guard's site-specific investigation, the city may want to evaluate the need for a complementary monitoring program for compliance purposes.

As previously discussed, a soil contamination (petroleum product) incident occurred northeast of the study area in the glacial till because of a leak in the Williams pipeline in 1992. If petroleum product from the pipeline (Figure 4-37) reaches the ground water, contaminants could easily enter the airport wellfield, the Big Sioux River, or the diversion canal. If a leak occurs where the pipeline crosses the Big Sioux River, the river would transport contaminants to three sides of the wellfield. Contamination would not only threaten the ground water in the wellfield because of induced infiltration, but would also require immediate shutdown of pumping at the surface-water intake structures. Therefore, monitoring is necessary to address potential contamination from the pipeline.

Ground water beneath the industrial and commercial area northeast of the wellfield (Figure 4-37) may discharge to the diversion canal and be carried downstream to the southern intake structures. From the diversion canal, this water may recharge ground water on the west side of the canal due to pumping from the municipal wells. Alternatively, ground water may move under the diversion canal toward wells in the airport wellfield. For these reasons, monitoring of the industrial and commercial areas is warranted.

The remaining point sources of concern include the industrial and commercial area to the south, a fire training center near the west edge of the wellfield, and the airport facility itself (Figure 4-37). The fire training center and the regional airport present a challenge to early-warning detection monitoring. These sources are within the wellfield boundaries, rather than some distance from it, and ground-water travel times are short. The fire training center is only about 250 feet east of municipal well 34A, and most of the municipal wells in the airport wellfield are inside of the boundaries of the regional airport.

Twelve locations for point-source monitoring sites are shown in Figure 4-37. Monitoring for point-source contamination will equally serve to document ambient water quality and to act as early warning detection for contamination.

Two sources of contamination at the South Dakota Air National Guard facility are recommended for monitoring: (1) an area near the south edge of Section 5 (Figure 4-37), for contamination leaked from a heating fuel tank; and (2) a site along the east edge of the Air National Guard property in Section 8, for contamination leaked from an old underground fuel storage facility. These two monitoring locations were chosen based on the capture zones shown in Figure 4-30.

Two locations are recommended for monitoring the Williams pipeline. Both are on the northwest side of the airport and river in Section 32 (Figure 4-37). These sites were chosen based on the capture zones shown in Figure 4-32 and serve the dual purpose of point-source and nonpoint-source monitoring. A third possible location for pipeline monitoring is the surface-water, line-source monitoring point downstream from where the pipeline crosses the Big Sioux River (Section 29 and 32 border; Figure 4-37).

The industrial and commercial area south of the airport is treated as one point source, rather than a number of smaller, discrete sources. Three monitoring locations in Section 8 are recommended along the northern edge of the area (Figure 4-37). These three locations were chosen based on the capture zones delineated in Figure 4-30.

Two locations are recommended for monitoring the east edge of the airport (Sections 33 and 5; Figure 4-37). These locations are based on the capture zones depicted in Figure 4-30 as they relate to (1) the airport fuel storage area, and (2) the area near the airport terminal where de-icing of commercial aircraft occurs. Upon completion of a thorough assessment of the use of de-icing chemicals at the airport, additional monitoring may be warranted for a larger area.

Another point-source monitoring location is recommended in Section 6, at the west edge of the airport. The purpose of this location is to examine ground-water quality between the fire training center and municipal well 34A.

The two point-source monitoring locations east of the airport and the diversion canal (Sections 33 and 4; Figure 4-37) are intended to examine the commercial and industrial area northeast of the airport. The rationale for these locations is based on the present understanding of ground-water flow directions. These sites will permit characterization of the water moving under the industrial and commercial area (1) before it discharges to the diversion canal and is pumped into the south surface-water intake structures, or (2) before it enters the wellfield on the west side of the diversion canal. The surface-water, line-source monitoring point in Section 4, just downstream from these two point-source monitoring locations, may also be used to monitor the northeastern industrial and commercial areas.

Nonpoint-Source Monitoring

The purpose of nonpoint-source monitoring is the same as that of point-source monitoring. All three types of nonpoint sources, namely, residential areas, agricultural areas, and the Elmwood Golf Course (Figure 4-37), were considered in choosing monitoring sites.

The residential area southwest of the airport (Sections 7 and 18; Figure 4-37) is not considered to be a threat to the water quality at the wellfield, based on ground-water modeling results. However, the residential area east of the airport could possibly impact the ground-water quality. Ground water beneath the residential area probably discharges to the east side of the diversion canal, and then partially recharges the aquifer on the west side of the canal. Some ground water may flow into the wellfield under the canal.

Agricultural land occupies considerable area on the west, north, and east sides of the wellfield (Figure 4-37). The contamination threat from agricultural land is considered to be minimal at this time. However, this short-term perception should not result in ignoring the long-term adverse impacts of activities on the agricultural land. A major, unanticipated spill of a hazardous or toxic substance could cause adverse water quality effects in a very short period of time.

The Elmwood Golf Course includes the area of the former Elmwood Landfill (Figure 4-37). Although the former landfill is not considered a potential source of contamination based on the modeling of simulated spills (Figure 4-36), the golf course is considered a potential contaminant source. Two-year TOT capture zones depicted in Figure 4-32 indicate that the golf course area contributes recharge water to the wellfield.

Seven locations are recommended for nonpoint-source monitoring and are depicted in Figure 4-37. Few monitoring locations are sited relative to the total area of the nonpoint sources. This reflects the perception that the nonpoint sources are considered to be a minimal short-term threat. The nonpoint-source monitoring site located on the east side of the diversion canal in Section 4 will serve a dual purpose: to monitor the residential area in Section 4, and to monitor the industrial and commercial area in Section 33. The ground-water, line-source point in Section 4 may also be used to monitor agricultural land east of the airport. Both of the surface-water, line-source points (one at the border of Sections 29 and 32; one in Section 4) may be used to monitor nonpoint sources. Nonpoint-source monitoring data may prove more useful in determining ambient water quality trends than serving as an early warning of imminent and serious contamination problems.

Future Studies

Hydrologic conditions at the wellfield are very dynamic, particularly ground-water flow directions and gradients. For this reason, it is recommended that transient-state, ground-water modeling be performed to determine flowpaths and gradients under variable pumping conditions. A thorough inventory of potential point sources of contamination is also warranted. This additional information would aid in determining final monitoring locations. However, the monitoring strategies proposed as a result of this research should be used to implement the program. Refinements in the program can be incorporated later, based on transient-state model studies and a detailed point-source assessment.

PROPOSED MONITORING WELLS

Proposed wells for the monitoring system should have a well screen that intersects the water table and that will accommodate water-level fluctuations resulting from pumping and from seasonal effects. The screen length should be sufficient to monitor the upper 5 to 10 feet of the saturated sand and gravel. Monitoring wells with this construction (Figure 4-8) may not be suitable for examining contaminants of the dense, nonaqueous phase liquid (DNAPL) type, which would likely be at or near the base of the aquifer. If DNAPL-type contaminants become the source of investigation within the wellfield area, then the typified well construction diagram (Figure 4-8) will need to be modified. Wells would need to penetrate the saturated thickness of the aquifer, with screen lengths and depths situated to intercept the DNAPL at, or toward, the base of the aquifer.

MONITORING PARAMETERS AND FREQUENCIES

Five general categories of parameters are recommended for monitoring line, point, and nonpoint sources, as listed in Table 4-9. A tailored list of key water quality parameters and optimum sampling frequencies cannot be generated from the limited data base of this research. Selection of key parameters for monitoring and associated frequencies must be based on a quantitative assessment of potential point and nonpoint sources, as well as a risk analysis.

TABLE 4-9. CATEGORIES OF MONITORING PARAMETERS AND FREQUENCIES, SIOUX FALLS AIRPORT WELLFIELD, SOUTH DAKOTA

	Source Type and Monitoring Frequency (sampling events per year)		
Monitoring Categories	Line Source	Point Source	Nonpoint Source
General water quality	4	4	4
VOCs	2	2	0
Trace metals	2*	2	2*
Pesticides	4*	0	4*
Nutrients	4*	0	4*

Abbreviation: VOCs = volatile organic compounds.
* The timing of sampling events is dependent on the seasonal usage of agricultural chemicals.

The five categories of monitoring parameters listed in Table 4-9--general water quality, VOCs, trace metals, pesticides, and nutrients--will be used as a starting point for assessment of water quality in the wellfield monitoring system. The general water quality parameters include those listed in Table 4-3, except for arsenic and selenium. The VOCs include the chemicals listed in Table 4-4. Analysis for trace metals include arsenic, barium, cadmium, chromium, lead, mercury, selenium, and silver. These trace metals are examined by the South Dakota Department of Health when evaluating the suitability of water for human consumption. The pesticide compounds include those listed in Table 4-4. The nutrients include the parameters of nitrate plus nitrite, potassium, sulfate, iron, phosphorous, and zinc. This list of nutrients is recommended by the South Dakota Department of Agriculture, Division of Regulatory Services (Bruce Jacobson, personal communication, 1992). Four of these nutrients--nitrate plus nitrite, potassium, sulfate, and iron--are included in the list of general water quality parameters for completeness, if the situation arises that one category is analyzed, but the other is not.

Sampling frequencies for the three source types (line, point, and nonpoint sources) are proposed in Table 4-9. The minimum recommended sampling frequencies for the five parameter categories are:

- Quarterly for general water quality parameters for all three source types
- Biannually for VOC parameters for line and point sources
- Biannually for trace metal parameters for line, point, and nonpoint sources
- Quarterly for pesticide and nutrient parameters for line and nonpoint sources

Monitoring at these frequencies should continue until in situ monitoring technology is commercially available and affordable, or until other information warrants modification of these frequencies.

Quarterly monitoring is conducted to document water quality trends attributed to recharge events and to the seasonal use of certain chemicals, such as pesticides. For example, in the Big Sioux aquifer, quarterly monitoring of pesticides and nutrients is necessary, at a minimum, to understand the chemical transport rate through the aquifer after field application. Biannual monitoring of metals and VOCs, which are costly to analyze, is determined primarily as a function of budget limitations. Where quarterly monitoring is preferred to yield a more complete data base, biannual monitoring represents a compromise between technical objectives and funding constraints. Analysis for these parameters is chiefly for early-warning detection monitoring.

QUALITY ASSURANCE AND QUALITY CONTROL CONSIDERATIONS

Quality assurance and quality control considerations are an integral part of a successful monitoring program and are a function of program objectives. These considerations must be consistent with the ultimate intent of using the data for legal enforcement of wellhead protection ordinances. At a minimum, the QA/QC procedures developed for field operations and laboratory analyses for the research monitoring program should be continued in the long-term monitoring program. In addition, QA/QC requirements should be developed for data storage and validation.

Field operations QA/QC must address proper well design (Figure 4-8), construction materials, installation, and development (discussed in the section Data Acquisition, Vertical Wells) to ensure that cross-contamination is avoided and that environmental data are representative of the subsurface area from which they were acquired. After wells are installed, measuring-point elevations must be determined accurately (preferably to ± 0.01 foot) to determine ground-water flow directions for map construction and modeling purposes (discussed in the section Data Acquisition, Water-Level Measurement). Finally, samples must be acquired following strict adherence to established guidelines (Coker et al., 1988) to avoid inadvertent contamination and to ensure representativeness and reproducibility. Field sampling QA/QC should include an appropriate number of blanks, spikes, duplicates, and splits to establish the validity of the hydrochemical data (CDM Federal Programs Corporation, 1987; Canter et al., 1988).

Analytical laboratories used for determination of water chemistry parameters should be have a rigorous QC program to guarantee that measurements generate precise, accurate, and reliable data. These laboratories should be certified by the State or the EPA; certification often aids in lending credibility to data that may be court-admissible. Laboratories should have a documented, internal QA/QC program with verifiable effectiveness, often available to the client in report format.

Data base QA/QC involves methods of data reporting accuracy, storage (computer storage versus paper files), retrieval, manipulation, and access. Data quality and accuracy should be addressed by the person who is closest to the level of its acquisition (for example, the driller or field personnel), before it is entered into the data base. Thereafter, a controller should be responsible for checking the electronic data base for accuracy, as well as doing backups and inventories. Error check programs that search for inconsistencies and common errors can be created and run; this is commonly done following keyboard entry procedures (Rowe and Dulaney, 1991).

Data should be readily available for use in temporal-contaminant studies and be in a form compatible with anticipated computer software applications. Access to sensitive or compliance monitoring data should be restricted to select personnel, perhaps as "data retrieval only," to protect the integrity of the data base.

MONITORING DATA BASE STORAGE, UPDATE, AND RETRIEVAL

The WHPP monitoring program is very data extensive. Large quantities of information are gathered on a continual basis and need to be kept in an organized format that will allow ready access. The most applicable format for this task is a Geographic Information System (GIS), together with complementary relational data bases and electronic spreadsheets.

A GIS allows information to be transferred either electronically or manually. Both modes will be used for the type of information derived from the monitoring system. Examples of the types of information that will be generated and incorporated in the GIS are: (1) locations of monitoring wells, municipal wells, streams, and other surface water; (2) locations and characteristics of potential sources of contamination; (3) continuous water-level measurements; (4) inorganic and organic analyte data from water samples; and (5) daily meteorological data.

Much of the tabular data for values such as water levels and water chemistry will be placed in electronic spreadsheets and relational data bases. This will facilitate an appropriate means for updates and queries. In addition, this information can be electronically interfaced with GIS software. Nontabular information, such as location maps and water-table contours, is digitized into the GIS in the form of a map. Specific attributes such as well depth, water levels, and parameter concentrations can be related to the entities on the maps.

Developing a GIS for wellhead protection will greatly enhance the capabilities of the monitoring system. The effectiveness of the system is dependent on the ability to organize and interpret the data derived from it. Therefore, it is important that appropriate technologies are used to process the information produced by the wellhead protection monitoring system in a timely and effective manner.

CONTINGENCY PLANNING

The data presented in this report indicate that there is significant recharge from the Big Sioux River to the Big Sioux aquifer in proximity to the airport wellfield and that contaminants in the river can reach the municipal wells in a matter of hours to days. Contaminants in the aquifer, in or near

the airport wellfield, also can reach the municipal wells in a relatively short time period. Therefore, contingency planning is warranted in the event that contamination of a well, or wells, occurs. Contingency plans to respond to a contamination event must address the potential for a release on the surface of the aquifer and in the river, or the diversion canal.

Contaminants released on the surface of the aquifer will probably be localized in extent and may affect only one or a few wells. However, depending on where the release occurs and the ground-water flow direction, the surface-water intake structures may also be threatened by ground-water discharge to the river north of the wellfield or to the diversion canal.

If contaminants are released at the surface, the wells or surface-water intake structures that will be impacted may be identified using the research modeling results. However, it is recommended that a transient-state, ground-water flow and transport model be developed for this wellfield to design optimum pumping rates and durations of the individual municipal wells in order to minimize the inducement of contamination into the wellfield. The installation of monitoring wells near the contaminant release point will assist in defining the extent of contamination and the potential threat to the municipal water supply. These same monitoring wells can also be used to track the progress of remediation efforts and to help determine when resumption of normal wellfield operations is appropriate.

Contaminants released to the Big Sioux River or to the diversion canal will receive some dilution from the surface water, and then be induced to the aquifer due to pumping of the municipal wells. The surface-water intake structures are most vulnerable to contamination in the event of a release to the river north of the wellfield, or to the canal. In addition, a number of wells could also be contaminated.

The development of a transient-state, ground-water flow and transport model may show that under certain conditions it is beneficial to pump wells near the river and discharge the water to the river or another suitable location. In this way, the local hydraulic gradient may be modified, to eliminate or minimize degradation of the ground-water drinking supply. The pumping of existing, and possibly additional, wells near the river could be used to create a hydraulic barrier between contaminated river water and municipal wells in the center of the wellfield. This may be a realistic contingency plan because data presented in this report (Ground-Water Flow Model and Flowpath Assessment section) indicate that there is usually a gradient from the river and diversion canal toward the wellfield. The installation of monitoring wells near the perimeter of the wellfield will assist in defining the extent of ground-water contamination and the potential threat to the municipal water supply. These monitoring wells may also be used to track the progress of remediation efforts and to help determine when the resumption of normal wellfield operation is appropriate.

Another option to deal with contamination of the Big Sioux River or the diversion canal may be to shut off, or significantly reduce pumping from, select wells in the wellfield. Drinking water would then need to be obtained from other wellfields in the Big Sioux aquifer, or from other aquifers. Other aquifers that may serve as supplemental sources of water during remediation efforts are the Split Rock Creek aquifer (east of Sioux Falls) and the Skunk Creek aquifer (west and northwest of Sioux Falls).

CONCLUSIONS

The South Dakota Geological Survey (SDGS) conducted research at the Sioux Falls municipal, airport wellfield to design a long-term monitoring program for wellhead protection. The research focused on characterization of the hydraulic relationship between the Big Sioux River and the surrounding unconfined aquifer which supplies potable ground water to the city. Potential sources of contamination in the wellfield recharge area were assessed, particularly the river as a line source. Research monitoring tasks included qualitative source assessment, monitoring well installation and development, ground-water and surface-water sampling, water-level measurement, aquifer testing, dye tracing, ground-water flow and simulated spill modeling, and monitoring program design. Major summary points and conclusions are as follows:

- The qualitative source assessment was used to categorize potential sources within the recharge area into three types: line, point, and nonpoint sources. The Big Sioux River and the diversion canal act as potential line sources of contamination, draining fertilizers and pesticides from agricultural land, as well as hazardous and toxic substances from spills or discharges to the river.

- Potential point sources include industrial and commercial areas, the South Dakota Air National Guard facility, the Williams pipeline, the Sioux Falls Regional Airport, a sand and salt storage area, the former Elmwood Landfill, and a fire training center.

- Potential nonpoint sources include residential areas, agricultural land, and the Elmwood Golf Course.

- A network of 30 ground-water monitoring points was established to measure water levels, to collect samples, to conduct aquifer testing, and to perform dye tracing. These 30 points consisted of seven existing and 21 new vertical (single) wells, one new horizontal well, and one new vertical well located in the river. A surface-water sample site was located in the Big Sioux River upstream from the diversion canal.

- Inorganic analytes were determined for ground-water samples taken from 10 monitoring wells biannually, and for nine of these wells a third time. Pesticide and volatile organic compound (VOC) analyses were performed biannually on ground-water samples taken from five wells. Inorganic analytes were determined for one river monitoring location which was sampled three sample taken three times during the study year. Pesticide and VOC analyses were performed for this same river location twice in the year.

- Water-level measurements indicate that ground-water flow is generally to the north in the area south of the wellfield and to the west in the area east of the wellfield. Ground-water recharge areas are coincident with the areas of industrial and commercial development. This situation underscores the need to regulate and monitor these source areas.

- Rises in ground-water levels often coincide with rises in river level, indicating good hydraulic interconnection between the river and the aquifer.

- Water quality analyses indicate a calcium-bicarbonate ground-water type that is relatively constant throughout the year. The surface-water quality exhibits more yearly variation; in January and April, the dominant water type was magnesium-sulfate and in July the predominant water type was calcium-bicarbonate/sulfate.

- Concentration levels for arsenic and selenium in ground-water and surface-water are below the primary drinking water standard of 50 µg/l and, therefore, are not presently a public health concern. Volatile organic compounds (VOCs) were not detected in any water samples, with the exception of acetone. The presence of acetone in the samples is attributed to the use of this solvent as a cleaning agent for the Teflon bailers. The pesticides atrazine, cyanazine, dicamba, and 2,4-D were detected in samples from wells very close to the river, or from the river. Detection levels for atrazine and 2,4-D are below the EPA primary drinking water standards.

- Aquifer-test data were analyzed with the Theis and Cooper-Jacob analytical methods to yield average values of aquifer transmissivity (T) and storativity (S). These values for the Theis method are T = 20,900 ft^2/d and S = 0.1931; for the Cooper-Jacob method they are T = 21,000 ft^2/d and S = 0.1856. The aquifer has an average saturated thickness of 26 feet in the area of the airport wellfield, yielding an average hydraulic conductivity of 800ft/d.

- Dye tracing was conducted between the river and municipal well 34A to determine ground-water travel times. Dye from well R20-89-117 (40 feet north) reached the 34A in 12 hours, based on the first time of arrival; dye from well R20-89-123 (140 feet north) reached 34A in approximately 8 days. The travel time from well R20-89-123 is considered to be most representative of the travel time between the river and municipal well 34A because the separation distances are similar. For a conservative contaminant entering the aquifer from the river, an estimate of several hours to 8 days is given as the travel time range to municipal well 34A.

- A two-dimensional, steady-state flow model of the airport wellfield was developed using FLOWPATH to evaluate surface-water and ground-water relationships, to delineate capture zones, and to simulate contaminant spills from potential sources.

- One-year and 2-year time-of-travel (TOT) capture zones were generated for aquifer porosities of 15% and 25%. A worst-case scenario of a 2-year TOT capture zone, using a 15% porosity, was prepared to assess the relative threat of existing and potential source of contamination to the capture zones of wells within the airport wellfield.

- Simulated spill modeling indicated that the threat of contamination from point sources is greatest in the industrial and commercial areas east and south of the airport. Spills from the salt storage area and old Elmwood Landfill would flow into the Big Sioux River downstream from the wellfield and, therefore, are considered a minor threat. Contamination north and south of the wellfield could easily migrate to the municipal wells in 1 to 2 years.

- A 5-year TOT capture zone map was generated to compare to the existing wellhead protection area (WHPA--the area within the aquifer boundaries) so that the present delineation criteria (recharge boundary) and method (hydrogeologic mapping) could be verified or improved. The 5-year TOT capture zones for individual wells cover the majority of the aquifer in proximity to the wellfield. These results verify the use of the recharge boundary criteria and hydrogeologic mapping method to delineate the WHPA.

- A long-term monitoring program is proposed (1) to document ambient water quality conditions during different seasons and recharge events, and (2) to serve as an early-warning detection system for contamination. Line-source monitoring consists of both surface- and ground-water monitoring; point- and nonpoint-source monitoring will consist primarily of ground-water monitoring with wells.

- Two surface-water line-source monitoring sites are recommended: one upstream from the divergence of the Big Sioux River and the diversion canal, and one in the diversion canal on the east side of the airport wellfield. Nine ground-water line-source monitoring sites are recommended between the river, or diversion canal, and some of the municipal wells.

- Twelve monitoring sites are recommended downgradient from five point sources of concern; that is, between five point sources and downgradient municipal wells. The five point sources targeted for monitoring include the South Dakota Air National Guard facility, the Williams pipeline, a fire training center, the industrial and commercial areas (south and east), and the Sioux Falls Regional Airport. The sand and salt storage area and the former Elmwood Landfill are considered minimal threats to the wellfield based on simulated spill modeling.

- Nonpoint-source monitoring focuses on three areas: the residential area east of the airport; agricultural land on the west, north, and east sides of the airport; and the Elmwood Golf Course. Seven sites are recommended for well installation.

- Proposed wells for ground-water monitoring should be designed primarily to sample water-soluble or partially-soluble contaminants in the upper 5 to 10 feet of the aquifer. Well screens should be long enough to accommodate water-level fluctuations due to recharge events or due to pumping. Wells of this construction are not suitable for monitoring dense, nonaqueous-phase liquid contaminants, which are considered of least importance for monitoring at the present time.

- Five categories of parameters are recommended for monitoring line, point, and nonpoint sources: general water quality, VOCs, trace metals, pesticides, and nutrients.

- The minimum recommended sampling frequencies for the five parameter categories are:
  - Quarterly for general water quality parameters for all three source types
  - Biannually for VOC parameters for line and point sources
  - Biannually for trace metal parameters for line, point, and nonpoint sources
  - Quarterly for pesticide and nutrient parameters for line and nonpoint sources

- Quality assurance and quality control (QA/QC) objectives and procedures were implemented for the research monitoring program for field operations, laboratory analyses, data interpretation, and report review. At a minimum, QA/QC procedures for field operations and laboratory analyses should be maintained for the long-term monitoring program. Additional QA/QC should be developed to address data accuracy, precision, and reproducibility if compliance monitoring emerges as an integral part of the long-term program.

- Monitoring data should be stored in a Geographic Information System (GIS), to accommodate anticipated volume and diversity and a QA/QC program should be established to account for data accuracy and completeness. A GIS will allow electronic and manual information storage and transfer, data analysis through interface with spreadsheets and other software, and enhanced cartographic capabilities.

- Contingency planning is warranted to establish emergency responses to contaminant releases at the surface of the aquifer and in the river, or the diversion canal. It is recommended that a transient-state, ground-water flow and transport model be developed for the airport wellfield to predict optimum pumping rates and durations, and to minimize the contamination threat to individual wells.

- New water supply development and wellhead protection should be initiated for the Split Rock Creek and Skunk Creek aquifers as part of the contingency planning effort.

RECOMMENDATIONS

The byproducts of the research monitoring program are (1) validation of the present WHPA delineation criteria (recharge boundary) and method (hydrogeologic mapping), and (2) a proposed long-term, surface-water and ground-water monitoring program. In a continuing effort to improve the Sioux Falls Wellhead Protection Program, the following recommendations are offered:

- Protection of the Big Sioux River from upstream contamination should be implemented to the extent that is technically practical and financially feasible. This would include such actions as:
  - Establishing a buffer zone along both sides of the river to reduce direct runoff of contaminants from adjacent lands, particularly pesticides and nutrients from agricultural areas
  - Posting wellhead protection warning signs at transportation route and river junctions to stress that the Big Sioux River contributes significantly to the Sioux Falls drinking water supply

- A detailed assessment of potential point sources in proximity to the wellfield should be conducted by the city. This assessment should include a tabulation of the type of industry or activity at each location, the type and quantity of chemicals used or stored at each location, and the precise location and manner or storage of each chemical. Some of this information can be derived from the computerized data base maintained by the Sioux Falls Fire Department.

- At a minimum, early warning and ambient trend monitoring should be implemented as described in the wellhead protection monitoring program. Items associated with this effort are:
  - Educating the public about the importance and benefits of wellhead protection so that the South Dakota Department of Environment and Natural Resources and the city of Sioux Falls are informed at the earliest possible time about accidental chemical spills near or in the river. In this way, surface remediation may be expedited to reduce or minimize aquifer contamination.
  - Establishing permanent and protected surface-water and ground-water sample-collection sites. A network of monitoring wells should be established along the river and diversion canal, as well as between the airport wellfield and potential sources of contamination to the south. Frequent, manual sampling should be conducted until in situ, real-time monitoring devices are commercially available and affordable.

- Contingency plans should be adopted in the event that surface-water intake structures or the municipal wells become contaminated, or if it is determined that contaminants are moving toward the wells, requiring them to be shut down. At present, the Big Sioux aquifer and the river are the primary sources of drinking water for the city. The SDGS and the U.S. Geological Survey are conducting new water supply investigations of the Split Rock Creek aquifer. Another potential water development area for the city is the Skunk Creek aquifer. If it is determined that these aquifers can sustain further development, it is recommended that they be used as contingency water supplies and that wellhead protection be evaluated for these aquifers, also.

- It is recommended that a transient-state, ground-water flow and transport model be developed for the airport wellfield for the following reasons:
  - To determine flowpaths and gradients under variable pumping conditions so that the location of monitoring sites can be refined or verified
  - To determine pumping times, rates, and durations of municipal wells that may provide water in a contamination event

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CHAPTER 5

WELLHEAD PROTECTION AREA DELINEATION AND MONITORING STRATEGIES FOR A FRACTURED BEDROCK AQUIFER DOVER, NEW HAMPSHIRE

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NOTICE

This report is the result of research supported by the U.S. Environmental Protection Agency's Environmental Monitoring Systems Laboratory in Las Vegas, Nevada, as part of its efforts to provide technical assistance to state, tribal, and local governments on the implementation of the Wellhead Protection Program. The specific methods and approaches contained in this document have undergone peer review but do not constitute official Agency endorsement or policy recommendations. The Office of Research and Development provides this information to help solve complex technical problems related to refined delineation and ground-water monitoring of wellhead protection areas in various hydrogeologic settings. Further assistance is available from the Environmental Monitoring Systems Laboratory in Las Vegas, from the Office of Ground Water and Drinking Water in Washington, D.C., and from the ground-water offices in the ten U.S. Environmental Protection Agency regions.

ABSTRACT

A wellhead protection area and ground-water monitoring strategy were established for a fractured bedrock aquifer in Dover, New Hampshire. The delineation and monitoring studies proceeded in tandem with an exploration project to expand the municipal ground-water supply. This study describes how the conceptual hydrogeologic model for the site was developed and refined. Techniques used to conceptualize the site hydrogeology include a survey of existing geologic and hydrogeologic information, lineament analysis, surficial and bedrock geologic mapping, surface geophysical surveys, test well drilling, overburden and bedrock monitoring well installation, borehole geophysics and other surveys, chemical sampling of surface and ground water, aquifer testing, dye tracing, and data analysis.

The bedrock aquifer consists primarily of quartz monzonite and metasedimentary rocks that interfinger along a fractured, faulted contact zone trending north 60 degrees east (N60°E). A N5-10°W trending lineament and fracture zone intersects the N60°E zone at the site. The bedrock aquifer is directly overlain by Pleistocene-age sands and gravels. These sediments are overlain by low-permeability marine clay and lodgement till. It is estimated that 20% of the water produced from the bedrock aquifer is derived from overburden sediments in the watershed area.

A test well was installed to a depth of 400 feet in the bedrock aquifer as part of the ground-water exploration program. Four overburden and bedrock well pairs constitute the present monitoring network for the test well. Two well pairs lie along the N60°E faulted contact zone, and two well pairs lie along the perpendicular N30°W trend. The test well and four of five bedrock wells airlift in excess of 150 gallons per minute. Few contaminant threats exist near the site. Baseline sampling indicates that minor, elevated levels of iron, manganese, and radon pose the only water quality problems at present.

Test drilling and borehole surveys (caliper, video camera, acoustic televiewer, thermal-pulse flowmeter, and hydrophysical logging) indicate that fracturing and ground-water flow are highly discrete. Flow occurs at isolated, definable depths rather than uniformly along the length of the borehole. Hydrophysical logging indicates that the borehole water is distinctly layered with respect to the fluid electrical conductivity parameter. Most borehole water is produced by moderately- to steeply-dipping fractures and fracture zones that intersect the wells.

Aquifer testing and dye-trace results indicate that the N30°W and N60°E directions have higher aquifer transmissivities relative to the surrounding bedrock matrix. Drawdown contours are elongate about the N30°W well alignment, suggesting preferred flow in this direction. Dye-trace results indicate more rapid travel of injected dye along the N30°W direction than the N60°E direction. Dye traveled 152 feet in 130 minutes (the time of first arrival of the dye) from injection in a bedrock monitoring well along the N30°W trend to the test well, which was pumped at 200 gallons per minute.

This represents a velocity of 1,680 feet per day. Dye injected in a bedrock monitoring well located 596 feet from the test well arrived there in 148 hours, indicating a velocity of 96 feet per day along the N60°E direction.

Flowmeter and acoustic televiewer surveys indicate that a moderately west-dipping fracture zone provides interconnection between the test well and bedrock well R2 along the N30°W trend. Lacking discrete flow information beyond the test well and well R2, statistical fracture descriptions become good approximations of flowpaths at increasing distances from the site. Therefore, prominent fracture peaks along the N5-10°W and N60°E trends represent the most probable flow directions within the bedrock fracture system at Blackwater Brook. The N60°E trend is substantiated by the existence of the faulted, fractured contact zone along this strike. Evidence to suggest preferred flow along the N5-10°W direction is structural and hydrogeologic. Structural control is inferred by strong expression of the lineament on several platforms of photography and in outcrop fracture trends. Enhanced transmissivity along the N30°W direction is attributed to the proximity and similar orientation of the N5-10°W fracture zone.

A quadratic equation is derived from accepted hydrogeologic relationships (Darcy's Law and the Thiem equation). In this equation, ground-water travel time (determined using the time of first arrival of dye at the test well) is directly proportional to the square root of distance from the test well. Constants of proportionality for the quadratic relationship are calculated for the N30°W and the N60°E directions based on dye-trace velocities. Distances for the 200-day and 1,000-day time-of-travel thresholds were then calculated for the two fracture zone directions: N5-10°W and N60°E.

Three wellhead protection zones are delineated within the recharge area for the test well using a variety of criteria and methods. Zone I is the state-mandated 400-foot sanitary radius. Zone IIA consists of two 1,000-foot-wide "arms" along the N5-10°W and N60°E directions, extending to the 200-day time-of-travel distances. Zone IIB is the area within a smooth curve connecting the outer boundaries of Zone IIA, producing an oval shape. Zone III is the upgradient area contributing to the 1,000-day time-of-travel distance modified by hydrogeologic features. Recommended regulation of the wellhead protection zones varies from complete control and restriction of activities in Zone I to public education in Zone III.

A major component of wellhead protection program management is long-term ground-water monitoring. Under present conditions, monitoring of the test well and existing monitoring wells will focus on a moderate effort to assess ambient water quality and physical parameters. After the production well is developed, the monitoring frequency and list of monitoring parameters increases. Proposed frequencies, parameters, and new sites for monitoring derive from technical and management goals. Action levels are proposed to trigger contingency responses.

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ABBREVIATIONS, SYMBOLS, AND CONVERSION FACTORS

ABBREVIATIONS

amsl	above mean sea level
cfs	cubic foot per second
cps	count per second
d	day
D1	domestic well number 1
EPA	U.S. Environmental Protection Agency
FEC	fluid electrical conductivity
ft	foot
ft/d	foot per day
$ft/d^{1/2}$	foot per square root of day
ft^2/s	square foot per second
gpd/ft	gallon per day per foot
gpm	gallon per minute
gpm/ft	gallon per minute per foot
gr	gram
hr	hour
l	liter
m	meter
m^2/d	square meter per day
MCLG	Maximum Contaminant Level Goals
µS/cm	microSiemen per centimeter
Mft^2/d	million square feet per day
mg	milligram
mg/l	milligram per liter
mi	mile
min	minute
N/A	not applicable
N5-10°W	north 5 to 10 degrees west
N60°E	north 60 degrees east
NHDES	New Hampshire Department of Environmental Services
NHSGS	New Hampshire State Geological Survey
PCB	polychlorinated biphenyl
pCi/l	picocurie per liter
ppb	part per billion
O1	overburden well number 1
QA	quality assurance
QC	quality control
R1	bedrock well number 1

SDWA	Safe Drinking Water Act
TDS	total dissolved solids
TOT	time of travel
TW	test well
USGS	U.S. Geological Survey
UST	underground storage tank
VOC	volatile organic compound
WHPA	wellhead protection area
WHPP	Wellhead Protection Program
ZOC	zone of contribution

SYMBOLS

Ca	calcium
Cl	chloride
CO_3	carbonate
Fe	iron
HCO_3	bicarbonate
K	potassium
Mn	manganese
Na	sodium
SO_4	sulfate

C	constant term or coefficient
D	hydraulic diffusivity
$dh/d\iota$	hydraulic gradient
Eh	oxidation-reduction potential
F	well function
γ	gamma
h	hydraulic head
K	hydraulic conductivity
ι	distance between the injection well and the test well
ϕ	porosity
q	Darcy velocity
Q	discharge
r	radius or radial distance
s	drawdown
S	storage coefficient
t	time of first arrival of dye
t_o	initial time
T	transmissivity
V	average velocity
x	distance from a given well to the linear extended well

CONVERSION FACTORS

Multiply	By	To Obtain
acre-foot	1230	cubic meter
cubic foot per second	0.0283	cubic meter per second
foot	0.3048	meter
foot per second	0.3048	meter per second
gallon	3.785	liter
gallon	0.134	cubic foot
gallon	0.00379	cubic meter
gallon per day	0.000003528	cubic foot per second
gallon per day per foot	0.000207	square meter per day
gallon per day per square foot	0.0408	meter per day
inch	0.0254	meter
inch per year	25.4	millimeter per year
mile	1.609	kilometer
million gallons per day	2.629	cubic meter per minute
square foot per minute	0.0929	square meter per minute
square foot per second	0.0929	square meter per second
square mile	2.59	square kilometer

$$°F = \frac{9}{5}(°C) + 32$$

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BACKGROUND FOR THE DOVER CASE STUDY

INTRODUCTION

As part of an effort to provide assistance to the states for wellhead protection, and recognizing the different problems posed by varying geologic and hydrologic conditions that exist throughout the United States, the U.S. Environmental Protection Agency (EPA) provided several grants to municipalities to examine the issue of developing monitoring strategies in a number of hydrogeologic environments. This document reports the results of the EPA grant to the city of Dover, New Hampshire, to develop wellhead protection monitoring strategies in a fractured bedrock aquifer at Blackwater Brook in the northern part of the city (Figure 5-1).

The report first examines some of the steps Dover has taken to protect its ground-water resources, including the development of temporary protection zones for areas yet to be explored and developed for ground-water supplies. A brief description of the regional hydrogeologic setting of the city is presented. The report focuses on the Blackwater Brook site, examining the process of delineating an appropriate wellhead protection area (WHPA) and of developing monitoring strategies for the fractured bedrock aquifer. Contaminant sources are examined, characterized, and prioritized, both for existing threats and for potential future threats.

Details of the steps taken to characterize the hydrogeology of the Blackwater Brook site are presented, including results from test drilling, surface and borehole geophysics, aquifer testing, and dye tracing. Based on this information, a refined WHPA and protection zones are described for the fractured bedrock site. Monitoring strategies proposed for the WHPA include additional monitoring sites, monitoring parameters, sampling frequencies, and contingency planning. The report concludes with recommendations both for the delineation process and for the development of further monitoring strategies.

WELLHEAD PROTECTION PROGRAM OVERVIEW

Dover, New Hampshire, a city of 26,000 people located in the seacoast region of New Hampshire (Figure 5-1), depends entirely on ground water for its municipal water supply. A 2.5-million-gallon-per-day withdrawal comes from seven gravel-packed wells located in kame and esker deposits. Dover has grown substantially during the past 5 years, and additional water supplies are needed. Also, the seacoast area of New Hampshire is undergoing substantial changes in its economic character. The closing of Pease Air Force Base in 1991 in nearby Newington has affected the entire region profoundly. Many experts predict that following the current economic recession, the regional

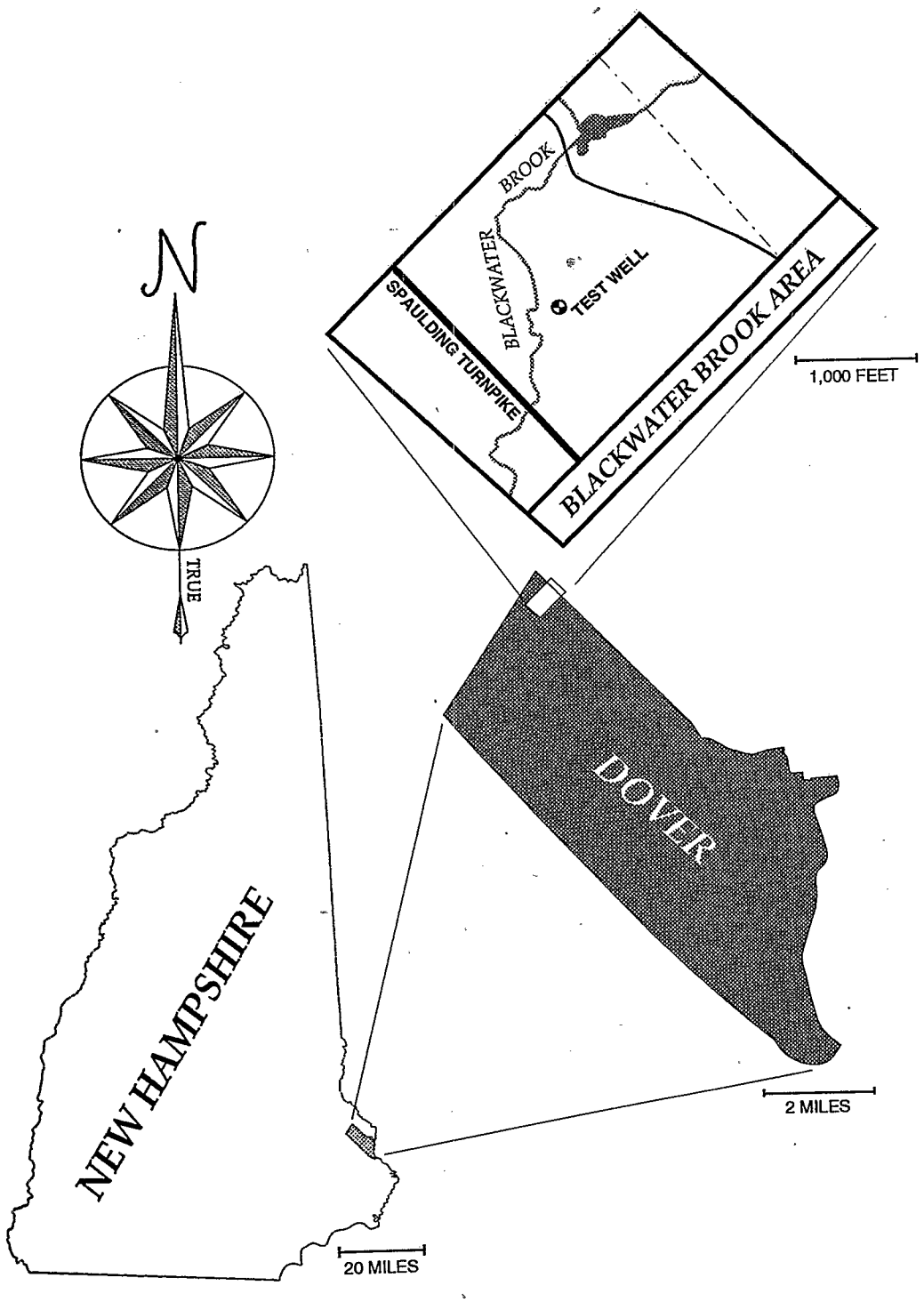


Figure 5-1. Location of the Blackwater Brook ground-water development site, Dover, New Hampshire.

economy will expand as the land and base facilities redevelop in the private sector. To meet the anticipated growth, the city of Dover has embarked on a number of options to augment its water supplies, including artificial recharge of several of its existing well sites and water exploration in fractured bedrock aquifers.

The city passed a ground-water protection ordinance in 1988 that stipulates permitted and nonpermitted activities within primary and secondary protection areas around the Dover wells. The ordinance offers a means of minimizing contaminant threats to sensitive hydrogeologic areas that might adversely affect the quality of water in the Dover municipal well system. A corollary objective is to monitor ground-water quantity and quality to provide a solid management basis for this resource.

The State of New Hampshire had not yet implemented a Wellhead Protection Program (WHPP) when the city of Dover passed its ground-water protection ordinance. Primary and secondary wellhead protection zones were therefore delineated for the city wells using as sophisticated a method as existing hydrogeologic data would support. The wellhead protection zones were viewed as a preliminary effort, subject to additional hydrogeologic information that might lead to modification or improvement of the boundaries.

The primary protection zone is defined as the area encompassed by a 400-foot radius around a well. Municipal ownership or control of this land is mandated by New Hampshire state law, which prohibits all activities other than recreational activities and maintenance of the well facilities. Secondary protection zones for the existing wells were established on the criterion of the effective radius from a 180-day, no-recharge pumping scenario or the distance to the 1,000-day time-of-travel contour, depending on the level of information available. The zones were drawn and modified on the basis of analytical modeling of available aquifer test data, hydrogeologic mapping, and a mapping effort for contaminant threats. The ordinance restricts land-use activities, specifically focusing on secondary protection zones. For example, solid and toxic waste disposal is prohibited, as is the establishment of salvage yards, motor vehicle repair shops, salt storage, animal feedlots, and commercial storage of fertilizer, manure, and pesticides.

The development and implementation of Dover's Protection Ordinance and the WHPP involves several key agencies. The city's Department of Public Works is responsible for the construction, maintenance, and production of water from Dover's well system. The Department has primary responsibility for both the physical and hydrochemical monitoring of the wells. The Department of Planning developed the protection ordinance language; this department reviews and governs challenges to the ordinance. The Planning Department also controls zoning regulations for the city. The city's Fire and Police Departments have responsibility for the public safety elements of wellhead protection contingency plans.

HYDROGEOLOGIC SETTING

The 28 square miles that constitute the city of Dover (Figure 5-1) lie in a regional hydrogeologic environment of Pleistocene-age glacial sediments ranging from high-permeability outwash sands and gravels (eskers and kames) to low-permeability marine clays and lodgement tills. The surficial glacial and marine sediments overlie fractured Precambrian and Paleozoic metasedimentary rocks intruded by younger Paleozoic granites and diorites (Figure 5-2).

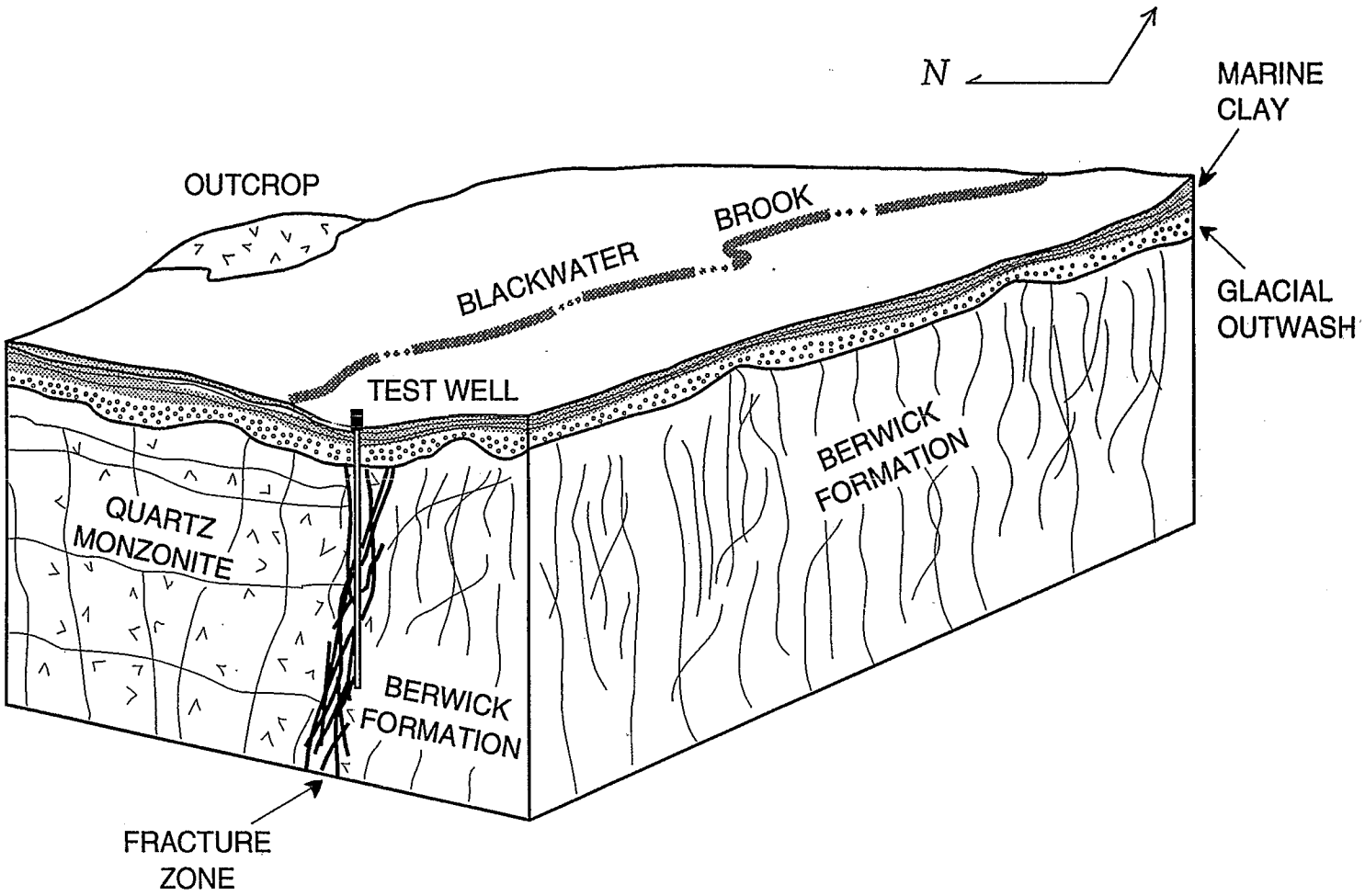


Figure 5-2. Conceptual hydrogeologic block diagram, Blackwater Brook site, Dover, New Hampshire.

Younger intrusions of gabbro and possibly other mafic rocks occur in the area. The bedrock geology was mapped and described by Novotny (1968), with updates of the mapping by Lyons et al. (1986). Bradley (1964) mapped the surficial geology and provided details of the hydrogeology of the region. The surficial mapping is currently being updated by the New Hampshire State Geological Survey (NHSGS) and the U.S. Geological Survey (USGS).

The Blackwater Brook site was selected for water supply development in bedrock based on a suite of criteria including photolineaments, mapped geologic features, bedrock outcrop fractures, surface geophysical surveys, and a contaminant threats assessment. Strong photolineaments striking N5-10°W as well as northeast-trending photolineaments, occur in the area (Figure 5-3). These photolineaments are present on several platforms of imagery and suggest structural bedrock control. Surface geophysical surveys at the Blackwater Brook site reveal no geophysical signature for the N5-10°W lineaments, but do show the trace of a strong magnetic anomaly coincident with a northeast-trending lineament (Figure 5-3). Together, the lineaments and magnetic anomaly correlate with an inferred geologic contact at the site. This northeast-trending feature is expressed as a geologic contact (Figure 5-4), photolineament, and a magnetic anomaly. The new municipal water supply well (the test well shown in Figures 5-1 and 5-2) was sited along this northeast feature.

The Blackwater Brook site has a 5-square-mile watershed, containing primarily surficial marine geologic deposits ranging from clays to fine- and medium-grained beach sands and outwash gravels (Figure 5-4). Drilling data support a depositional model of Pleistocene marine transgression. Marine clays, up to 30-feet-thick at the site of the new supply well, are underlain by 4 to 10 feet of permeable sand and gravel. Highland areas are either rock-cored drumlins covered with till or bedrock exposures where till has been eroded. Ground-water flow in both the overburden and bedrock at the site moves generally from the northeast to the southwest.

Ground-water recharge to the Blackwater Brook site occurs as precipitation, which averages 42 inches per year. Recharge to the bedrock aquifer occurs where open bedrock fractures are in contact with overlying weathered bedrock and permeable sand and gravel. Recharge of the overburden occurs in areas where the low-permeability clays at the site pinch-out and where permeable sand, gravel, or bedrock outcrop to receive direct precipitation. Based on estimates of recharge rates for similar geologic and climatic environments in central Connecticut (Randall et al., 1966), the total potential recharge to the watershed ranges from 380 to 520 million gallons per year.

According to initial conceptual models, the bedrock well initially receives water from overlying sediments during pumping. However, storage within the surficial deposits in the immediate area probably cannot sustain long-term municipal-level production requirements. Sustainable recharge to the well comes from leakage from overburden deposits, which travels along water-bearing fractures in a zone of contribution (ZOC), reflecting directional transmissivities.

PRELIMINARY WELLHEAD PROTECTION AREA DELINEATION

To provide temporary protection of the ground-water development site, a WHPA (Figure 5-5) was delineated at Blackwater Brook. At the time of the preliminary delineation, the test well was sited along the N5-10°W lineament (Figure 5-3) that exhibited strong surface expression. Early in the study, this lineament was identified as a potential, high transmissivity zone within the bedrock aquifer.

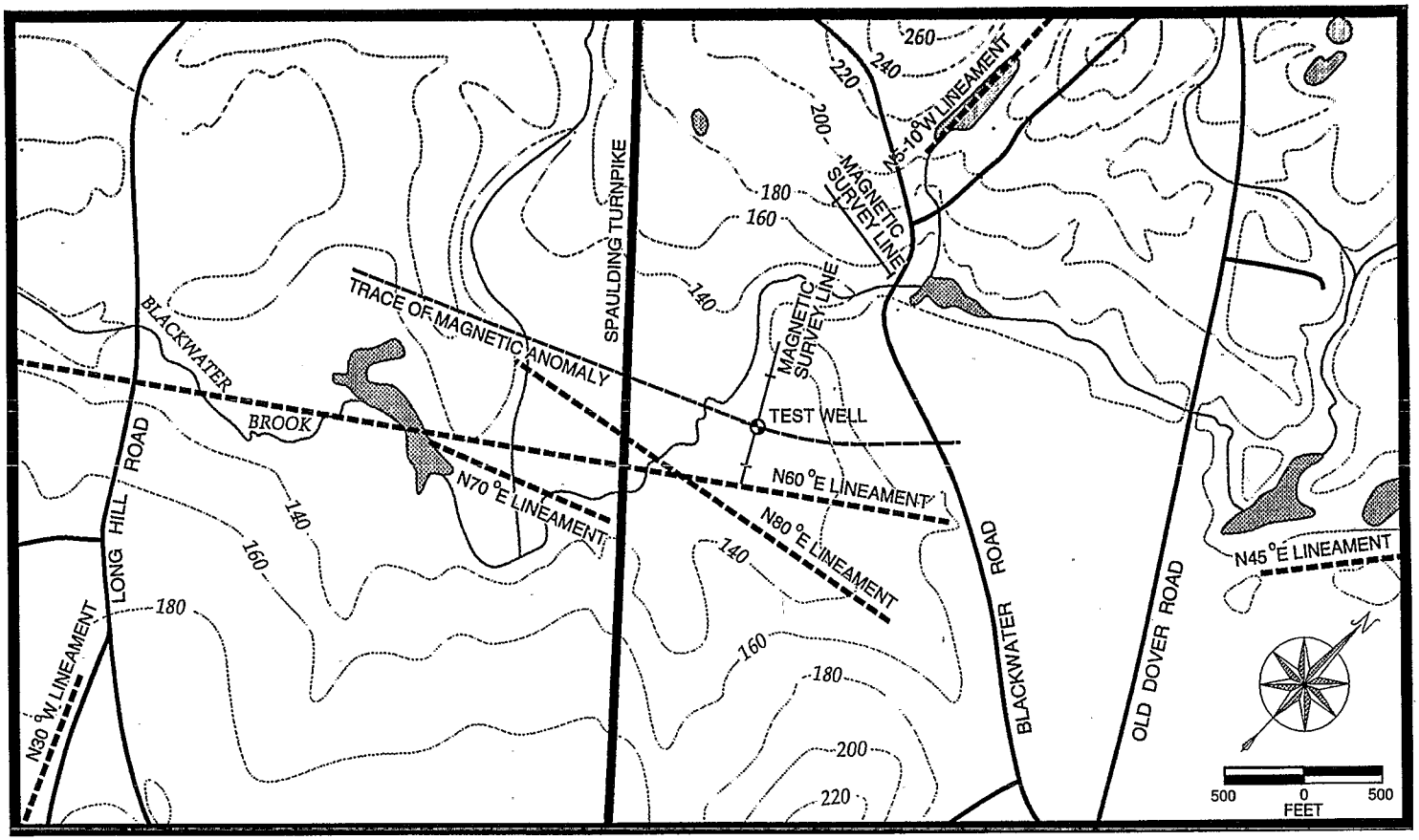


Figure 5-3. Lineament map, Blackwater Brook site, Dover, New Hampshire. (Contour interval is 20 feet.)

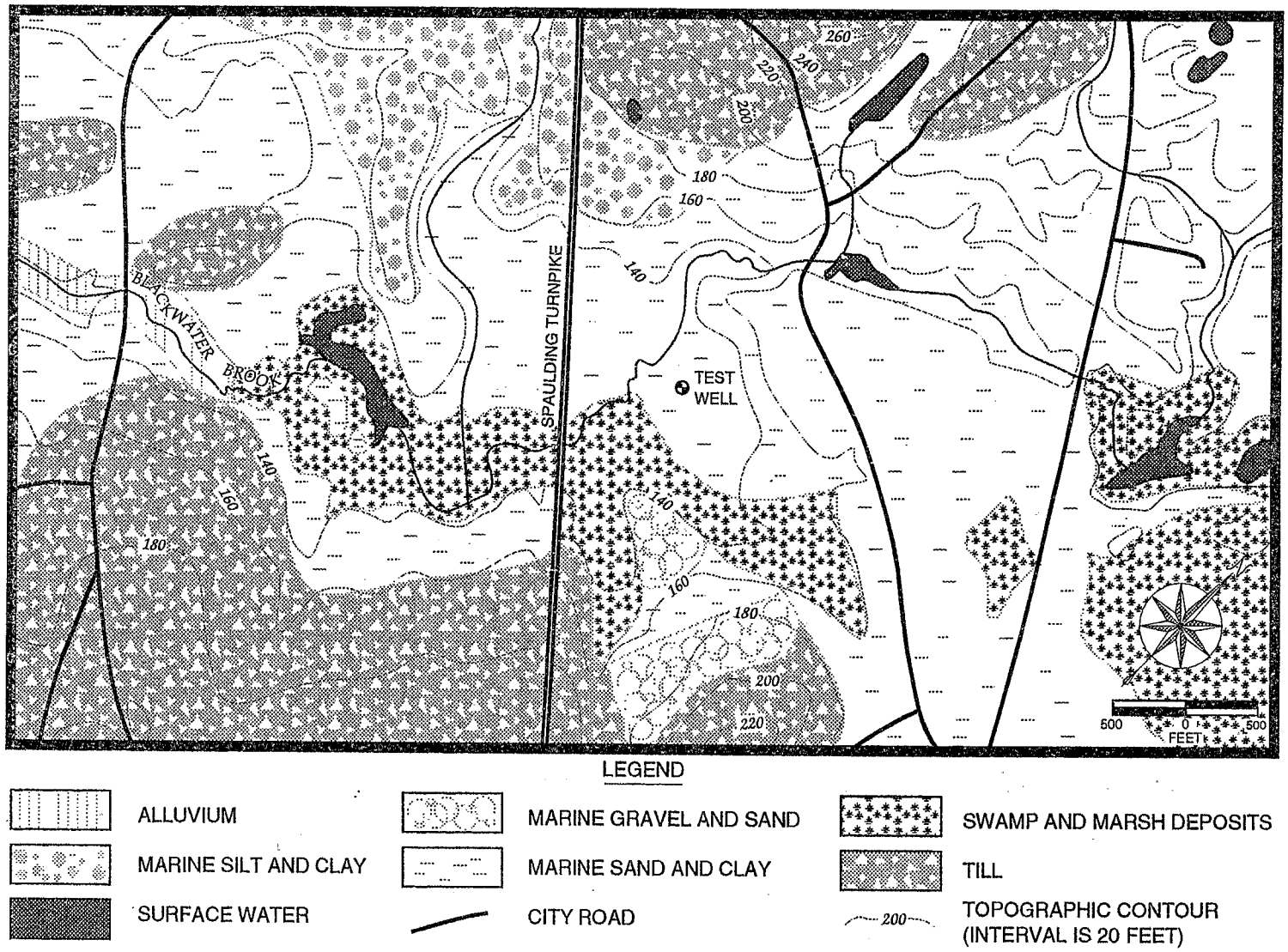


Figure 5-4. Surficial geology map, Blackwater Brook site, Dover, New Hampshire. (Modified after Mack and Lawlor, 1991.)

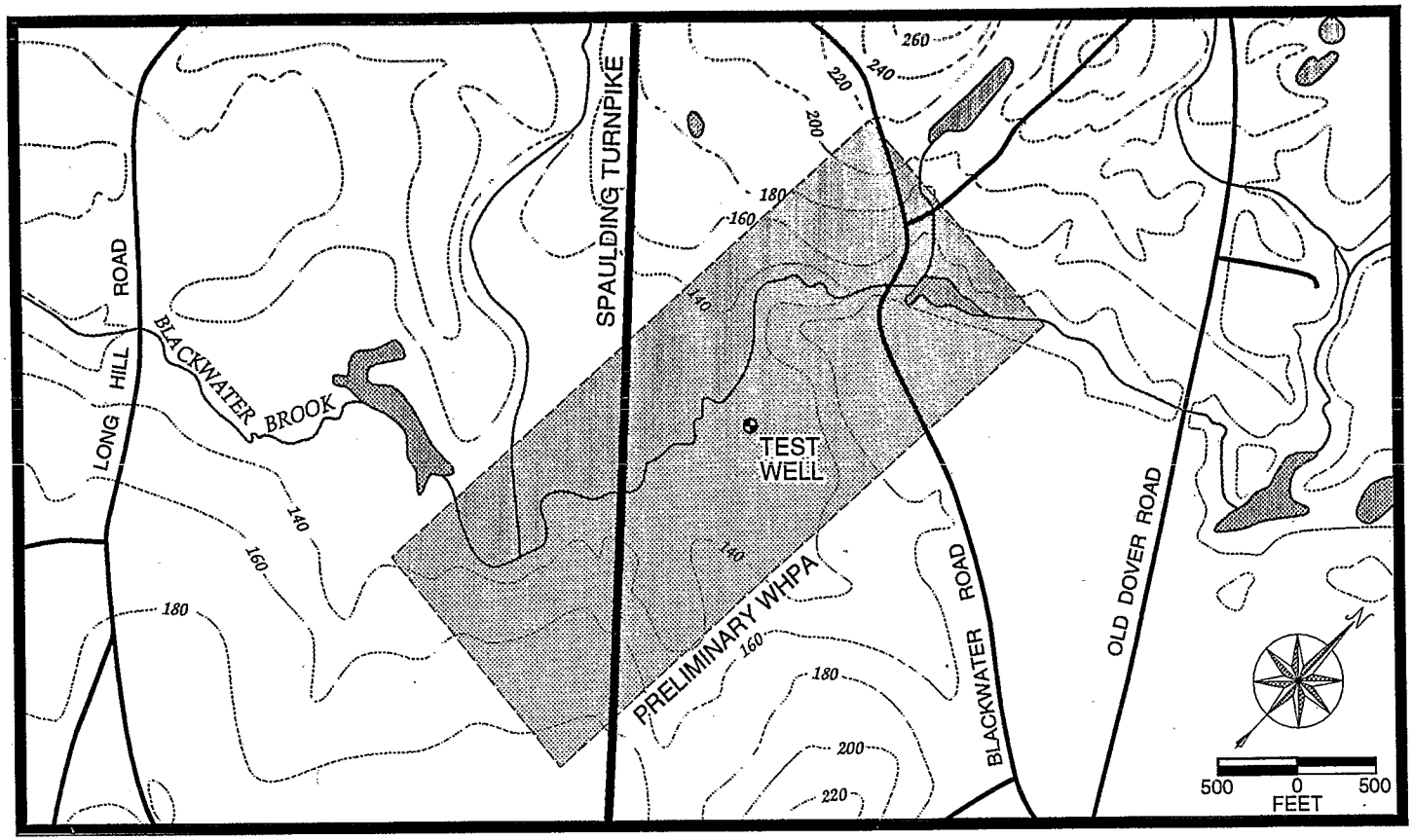


Figure 5-5. Preliminary wellhead protection area, Blackwater Brook site, Dover, New Hampshire. (Contour interval is 20 feet.)

To represent the directional properties of flow in the aquifer, the Cooper-Jacob equation (Cooper and Jacob, 1946) was applied along the potentially transmissive N5-10°W lineament and the perpendicular direction. Representative transmissivities derived for similar hydrogeologic regimes were assigned for the N5-10°W fracture zone and the perpendicular direction of 10,000 gallons per day per foot (gpd/ft) and 1,000 gpd/ft, respectively. It was assumed that the N5-10°W fracture zone may have as much as a 10-fold greater transmissivity than the perpendicular direction. The Cooper-Jacob solution used a discharge of 200 gallons per minute (gpm), a storage coefficient of 0.01 (a combined value for the fractured bedrock and overburden storage capacities), and a drawdown criterion of 0.10 foot. The resulting protection zone formed an oval extending 2,200 feet on either side of the test well along the N5-10°W fracture zone and 700 feet on either side of the test well perpendicular to the zone.

At the time of the preliminary delineation, the exact location of the test well was undetermined. Therefore, a 300-foot-wide zone corresponding to the potential width of the lineament (the fractured, faulted contact zone in Figure 5-2) was chosen as the center of the WHPA with the test well at its center. To either side of this zone, 700 feet of protection was added (calculated using the Cooper-Jacob solution), resulting in a 1,700-foot-wide protection zone (Figure 5-5). The 2,200-foot-long protection zone on either side of the test well along the N5-10°W fracture zone was modified for regulatory reasons. The north-northwest side of the WHPA was shortened from 2,200 feet to 1,800 feet because it extended approximately 400 feet beyond the town boundary. Since the city of Dover has no regulation authority in the adjacent town, the preliminary WHPA boundary coincided with the northern town boundary. The preliminary WHPA consisted of a rectangle with dimensions of 4,000 feet (north-northwest) by 1,700 feet.

CONTAMINANT SOURCE ASSESSMENT

Source Inventory List

A number of organizations including the EPA Region I office in Boston, the New Hampshire Department of Environmental Services (NHDES), and the New Hampshire State Department of Agriculture provided information relating to possible ground-water contamination sources in the region. Additionally, other sources of information pertaining to possible ground-water pollution were identified from contacts with Dover authorities (for example, fire department and public works officials), local landowners, and construction site personnel to obtain information on potentially contaminated sites. At the NHDES, the Waste Management Division, the Underground Storage Tank (UST) Program, and the Ground Water Protection Bureau provided sources of information. After a detailed review of available information had been completed, known sites of potential or actual contamination were checked by means of field surveys. This study identified 80 sites throughout the city (excluding the downtown, developed areas) as potentially hazardous. These include landfills, salt storage areas, gasoline stations, petrochemical storage facilities, agricultural concerns, and industrial parks.

Few contaminant sources pose an immediate threat to a future production well at the Blackwater Brook site (Figure 5-6). This is one of the primary reasons for the site selection. A listing of the existing and potential sources of ground-water contamination in proximity to the site (Table 5-1) includes existing road de-icing operations, spills, and illegal disposal associated with major

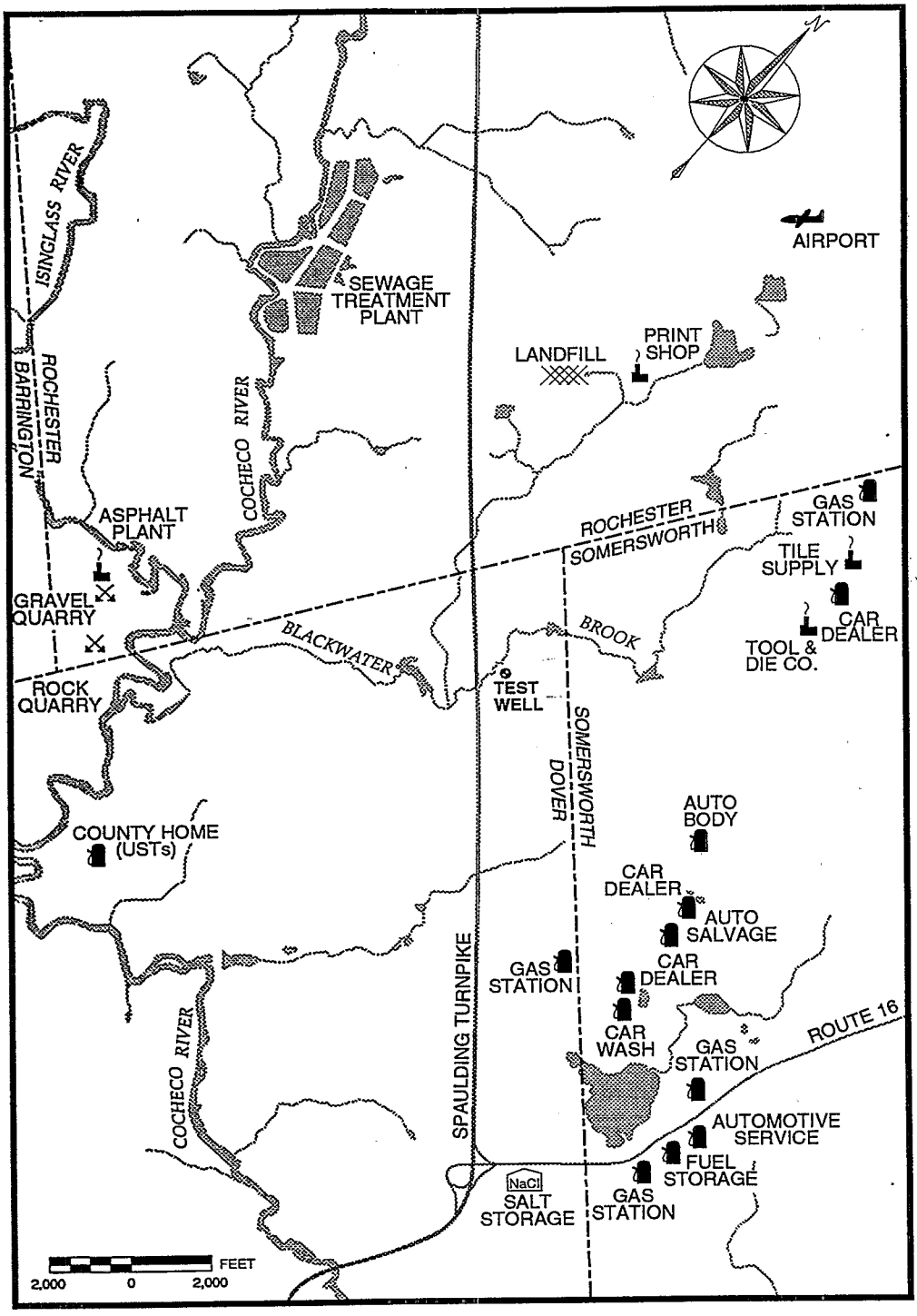


Figure 5-6. Contaminant source assessment map, Blackwater Brook site, Dover, New Hampshire. (USTs = underground storage tanks)

TABLE 5-1. POTENTIAL SOURCES OF GROUND-WATER CONTAMINATION AND DEGRADATION IN PROXIMITY TO THE BLACKWATER BROOK SITE, DOVER, NEW HAMPSHIRE*

Source Type	Existing Sources	Potential Sources
Commercial and Industrial	Airport Automotive services Landfill Quarry Road de-icing operations Unanticipated toxic and hazardous spills	Auto repair, paint, and machine shops Car washes Dry cleaners Gas stations Golf courses
Residential and Waste Management	Household hazardous products Septic systems Storage tanks Wells (operating and abandoned)	Household lawns (chemical applications) Sewer lines Storm water drains and retention basins Swimming pools (chlorine) Transfer stations Wastewater treatment facilities
Naturally Occurring	Iron and manganese (Berwick Formation source) Radon (quartz monzonite source) Sodium and chloride (trapped sea water or a marine clay source) Uranium (quartz monzonite source)	

*Modified after U.S. EPA, 1990, p. 330.

transportation routes. A landfill exists about 3 miles northwest of the site, and a rock quarry lies about 2 miles to the southwest. A few residences currently exist near the site. The primary residential threats include storage and disposal of household products (for example, heating oil), septic system leachate, and abandoned or operating wells. Naturally occurring iron, manganese, radon, sodium, and chloride are concerns for municipal water treatment.

It is anticipated that a significant amount of land near the well site will be zoned for residential use. Lawn chemicals, chlorine from swimming pools, and leakage from sewer lines, storm water drains, retention basins, and municipal waste transfer stations (Table 5-1) may add to existing residential threats. Light-commercial and light-industrial facilities that support residential developments such as auto repair shops, laundromats, car washes, golf courses, medical institutions,

dry cleaners, and warehouses represent other possible future threats to the well site. However, the city of Dover's ground-water protection ordinance ensures that controls against hazardous and toxic sources will be implemented to protect the new water supply well at the Blackwater Brook site. The advantage of implementing wellhead protection prior to urbanization is that the city can specify growth locations and development procedures that will minimize later problems with ground-water quality (U.S. EPA, 1990a).

Source Characterization and Prioritization

The degree of the perceived threat and the need for controls determined the prioritization of existing contaminant sources at the Blackwater Brook site. The highest priority threat is a spill of hazardous substances along the Spaulding Turnpike, a heavily traveled, north-south thoroughfare (Figure 5-6). Another threat is salt contamination from highway de-icing activities. Because the area is rural and undeveloped, illegal dumping in open fields along the numerous access roads to remote areas (for example, old logging trails) and possibly along the Turnpike is also a priority concern.

A quartz monzonite formation mapped in the area (Novotny, 1968) is known to contain pockets of water with high radon levels from elsewhere in the region. The other major bedrock unit is the Berwick Formation, a "rusty" schist characteristically high in iron and manganese. Sodium and chloride levels are unusually high in the bedrock aquifer and are believed to be from trapped sea water. It is anticipated that these levels will decrease in time as water is discharged through production. These naturally occurring constituents will be the targets of ambient trend monitoring to determine if water treatment is necessary.

The landfill located approximately 3 miles to the northwest (Figure 5-6) is not considered to be a major threat. It does not occur along the identified hydraulically conductive fracture zones in the bedrock aquifer. Contaminants from a 3-mile distance are unlikely to impact a well at Blackwater Brook unless their source occurs along a preferentially conductive flowpath.

At present, the nearest septic leachate field, situated 1,000 feet from the test well, would probably contribute negligible amounts of nitrate to the future well. However, as development encroaches on the WHPA, residential threats will increasingly take on more importance as a result of density impacts. Permeable sand and gravel deposits approximately 1,000 feet to the northeast and east of the test well, in areas where residential development could occur, could provide a direct avenue for contamination to the bedrock aquifer.

RESEARCH MONITORING PROGRAM

DATA BASE LIMITATIONS

Prior to exploration work at the Blackwater Brook site, comparatively little data existed regarding aquifer parameters, flowpath directions, and flow rates in the area. However, the region was reasonably well described in terms of its geologic and geophysical characteristics. Information is available concerning the surficial geology (mapped in detail at the 1:24,000 scale) and the bedrock and structural geology, including an extensive data base of fracture fabric analysis, photolineament analysis, and structural interpretations developed as part of the ground-water exploration process. Published well logs on file with the USGS, the NHSGS, and the NHDES were compiled, along with files of well logs from drillers. From this data base, the objectives and tasks for the research monitoring effort took form.

MONITORING OBJECTIVES

The research monitoring program leading to long-term monitoring of the well site area had two basic objectives. The first objective was to characterize the hydrogeology of the area, including a detailed examination of physical and chemical parameters. This information-gathering phase resulted in the delineation of a wellhead protection zone based on knowledge about the hydrogeology of the region and site.

The second objective of the research monitoring program was the development of a technically defensible and reliable long-term monitoring plan to protect the public water supply. Both physical and hydrochemical monitoring were used to achieve this objective. Physical monitoring examines the behavior of the aquifer over time to acquire a better understanding of the impact of water withdrawal as well as ground-water flow dynamics. Information collected from physical monitoring provides valuable data to support well and aquifer performance and management.

Hydrochemical monitoring focuses on the chemical character of the ground water. Ambient or background hydrochemical monitoring examines the system on a regional basis, recording changes in ground-water chemistry over comparatively large intervals of time and space. Source-assessment hydrochemical monitoring concentrates on impacts to ground-water quality from an identified potential pollution source near the wellhead. An early-warning hydrochemical monitoring system provides a mechanism for initiating contingency plans to protect the public from dangerous exposure to pollutants in the well water if a contamination event should occur (Canter et al., 1987).

Some of the work undertaken in the Dover project is not commonly identified as "monitoring" per se. In particular, the hydrogeologic characterization (information-gathering) phase is frequently not considered part of an overall monitoring effort. However, to determine effective strategies for

field monitoring, the rate of propagation of both anticipated and unanticipated contaminants must be understood and the physical properties of the site known. An effective research monitoring program requires both hydrogeologic characterization and the development of long-term monitoring strategies (Pfannkuch, 1982).

RESEARCH MONITORING TASKS

The process of ground-water exploration is one of developing and refining a conceptual hydrogeologic model of a designated site and the surrounding area. The exploration for a test well site at Blackwater Brook employed photolineament analysis, fracture fabric mapping, and surficial geologic and geomorphic mapping. Geophysical surveys helped further isolate the structural anomalies that form the major conduits of flow. Wellhead protection area delineation involves essentially the same process of building and revising a conceptual hydrogeologic model. Accordingly, the scope of work for this research program involved similar tasks, including:

- Lineament analysis of high- and low-altitude photography
- Geologic mapping, including fracture fabric analysis of bedrock and geomorphologic analysis of watersheds
- Surface geophysics to isolate suspected structural anomalies
- Test drilling
- Observation well and piezometer installation
- Borehole geophysics
- Chemical sampling of surface and ground water
- Aquifer testing
- Dye tracing
- Data compilation and interpretation

DATA ACQUISITION AND INTERPRETATION

Lineament Analysis

Extensive use of aerial photography and images was employed at the Blackwater Brook site, with a number of scales and image types for lineament analysis ranging from high-altitude, 1:126,000 color infrared images to 1:7,200 black-and-white photographs. These were examined with a mirror stereoscope, and lineaments were drawn on mylar overlays attached to the photographs. The image analyst discriminated among strong, moderate, and weak lineaments.

Two lineament orientations predominate in the area (Figures 5-3 and 5-7A). One trends approximately N5-10°W and was the focus of the original exploration efforts. Part of its intensity and visibility may be due to glacial enhancement of the original fracture pattern. The other predominant trend, N60°E, is much more subdued but appears to be significantly larger in extent. Additional photolineaments trend N45°E (parallel to the regional foliation), N30°W (perpendicular to the N60°E trend), N70°E, and N80°E (Figures 5-3 and 5-7A).

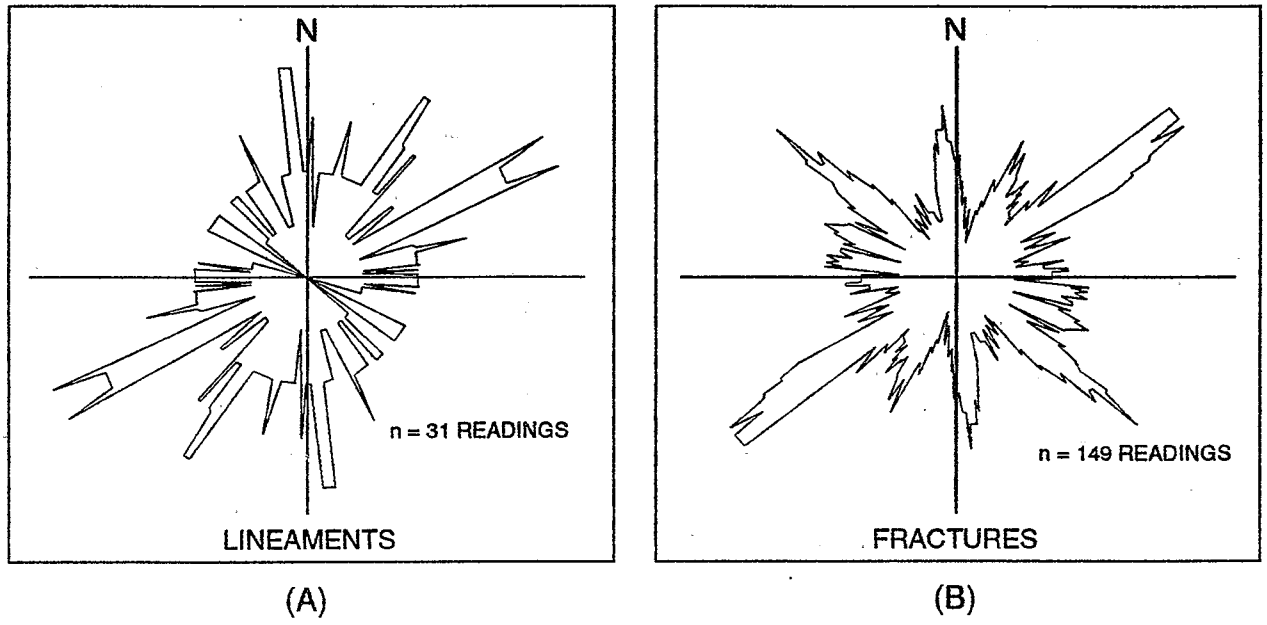


Figure 5-7. Rose diagrams of (A) lineament mapping, and (B) fracture fabric of bedrock, Blackwater Brook site, Dover, New Hampshire. (N = north; n = total number of observations.)

Geologic Mapping

Existing geologic maps and publications (Lyons et al., 1986; Lyons et al., 1982; Novotny, 1968; Bradley, 1964; and Billings, 1956) provide a general description of bedrock geology and structural features. To better define the relationship between lineaments seen on the photographs and mappable structural conditions on the ground, rock fractures were measured at several outcrops near the proposed wellfield. Fractures observed at outcrops were measured with a pocket transit, and the data were reduced and compared with lineament results (Figures 5-3, 5-7A, and 5-7B).

The fracture fabric rose diagram (Figure 5-7B), a statistical display of fracture strikes, shows that the N5-10°W trend is well represented. Strong northeasterly trends are also present. With the knowledge of a fault outcrop in a quarry 2 miles southwest of the site with a similar strike (Figure 5-6), attention focused on the N60°E lineament as a potential water-bearing zone. The quarry fault has a N54°E strike and cuts a quartz monzonite formation intruded by a variety of pegmatite and quartz veins. The N60°E photolineament coincides approximately with the interpreted fault projected northeastward from the quarry. Extensive alteration of the rocks, particularly along fractures observed at the quarry outcrop and in drill cuttings at the Blackwater Brook site, indicates weathering and possibly hydrothermal alteration of the rocks. Quartz monzonite at the quarry contains quartz veins with pyrite and extensive iron oxide staining along fractures.

Rather than one discrete fault connecting the quarry with the Blackwater Brook site, it is likely that both the quarry and the well site lie within a fault zone. The interpreted fault underlying the Blackwater Brook site may be one of a system or network of northeast-trending faults linking the Norumbega Fault System in Maine with the Clinton-Newbury Fault System in Massachusetts (Lyons et al., 1982; Cotten, 1988). As such, the interpreted fault and contact zone could be a previously unmapped link in a major fault system trending approximately northeast in eastern New England. If this fault is part of a regional fault system, the zone probably contains a number of individual faults and a high fracture density that would promote ground-water flow.

The lineament and interpreted fault zone also coincide with a geologic contact between the Devonian quartz monzonite and the Precambrian or Cambrian Berwick Formation (gray phyllite, biotite schist, gneiss, quartz-mica schist, and calcareous schist). The contact, as mapped by Novotny (1968), trends about N46°E. However, the contact is uncertain and probably exists as a zone in which the two rock types interfinger and are cut into a melange of fault and fracture-bounded slivers (Figure 5-2).

In addition to the lineament and fracture fabric data collection and analysis, the area was analyzed for local drainage basin size, soil type, and recharge potential. Surficial geologic mapping done by the NHSGS and the USGS aided in the geologic characterization (Mack and Lawlor, 1991). A variety of surficial deposits occur at the site and include glacial till, swamp deposits, and marine silt, sand, and clay (Figure 5-4). Drilling logs from the test well indicate that marine clay is underlain by permeable sands and gravels (marine and glacial outwash deposits). These permeable deposits, which outcrop along the fringes of the Blackwater Brook watershed (Figure 5-4), are capable of storing water and recharging the bedrock aquifer. With the potential for drilling into a major structure (a fault) that cuts across a number of small surface-water drainage basins, work focused on assessing the surficial deposits that could potentially recharge the fault zone.

Surface Geophysics

Geophysical surveys initially targeted the N5-10°W and N60°E lineament features (Figure 5-3) using instruments that measure microgravity, magnetics, and electromagnetics. The location of the geophysical survey lines is shown in Figure 5-6. Attention centered on the N60°E-trending lineament when it was recognized that this lineament coincides with a pronounced magnetic high (Figures 5-3 and 5-8). From a lower base level representative of the less magnetic quartz monzonite, the magnetic signature increases approximately 600-gammas (γ) before stabilizing at a higher base level characteristic of the iron-rich Berwick Formation (Figure 5-8). While the broad north-to-south increase in magnetic field can be explained by the change in rock formations, the explanation for the 600-γ peak is less certain. Possible explanations include a steeply-dipping mafic dike (86° to the southeast) along the contact zone (Figure 5-8); a horizontal or subhorizontal mafic dike that terminates at the contact zone; or, in part, alteration of chlorite to magnetite along the faulted contact zone. The N5-10°W fracture zone showed no geophysical signature on the surveys performed. The results of the surface geophysics, coupled with the evidence from lineament and fracture fabric analysis, warranted test drilling to determine whether the site could produce sufficient quantities of water for municipal needs.

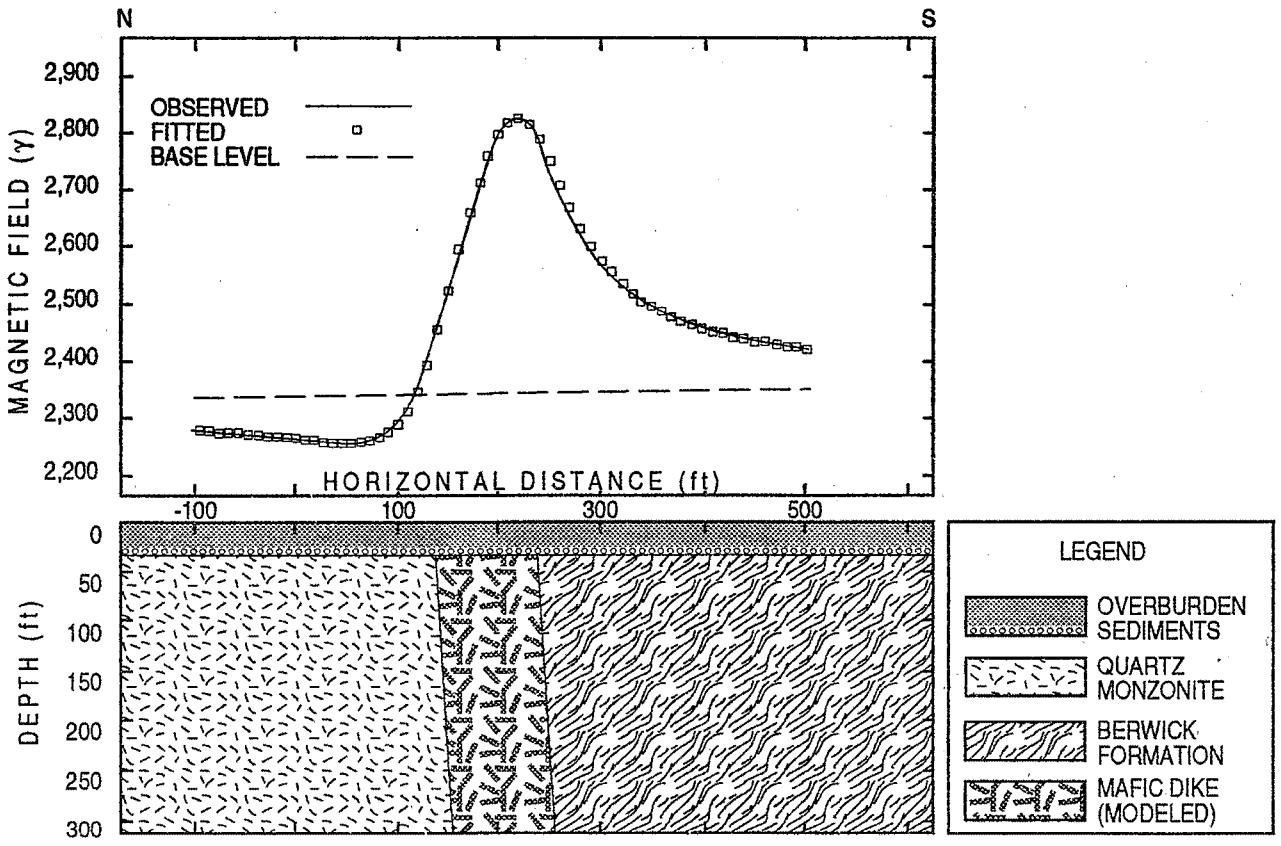


Figure 5-8. Plot of magnetic geophysical survey data with fitted model and mafic dike hypothesis, Blackwater Brook site, Dover, New Hampshire.

Test Drilling

A test well (TW) was drilled near the N60°E lineament, near the peak of the magnetic anomaly (Figure 5-3). The well penetrated 32 feet of clay and 6 feet of permeable sand and gravel overburden (Figure 5-9), which were cased off. The initial bedrock encountered at 40 feet was the quartz monzonite; next, the Berwick Formation was logged at a depth of 107 feet. The upper portion of the Berwick Formation encountered is differentiated as the phyllitic Gonic Member of the Berwick Formation. Significant water-bearing fractures were encountered in the Berwick Formation at 220 feet and at 342 feet. Evidence of these producing fractures can be seen as greater-diameter openings in the caliper log and as dipping fractures recorded in the video camera log (Figure 5-9). The well was developed with airlift techniques for approximately 12 hours, removing cuttings that had fallen into the fracture zones. Final airlift yields exceeded 300 gpm from the 6-inch borehole.

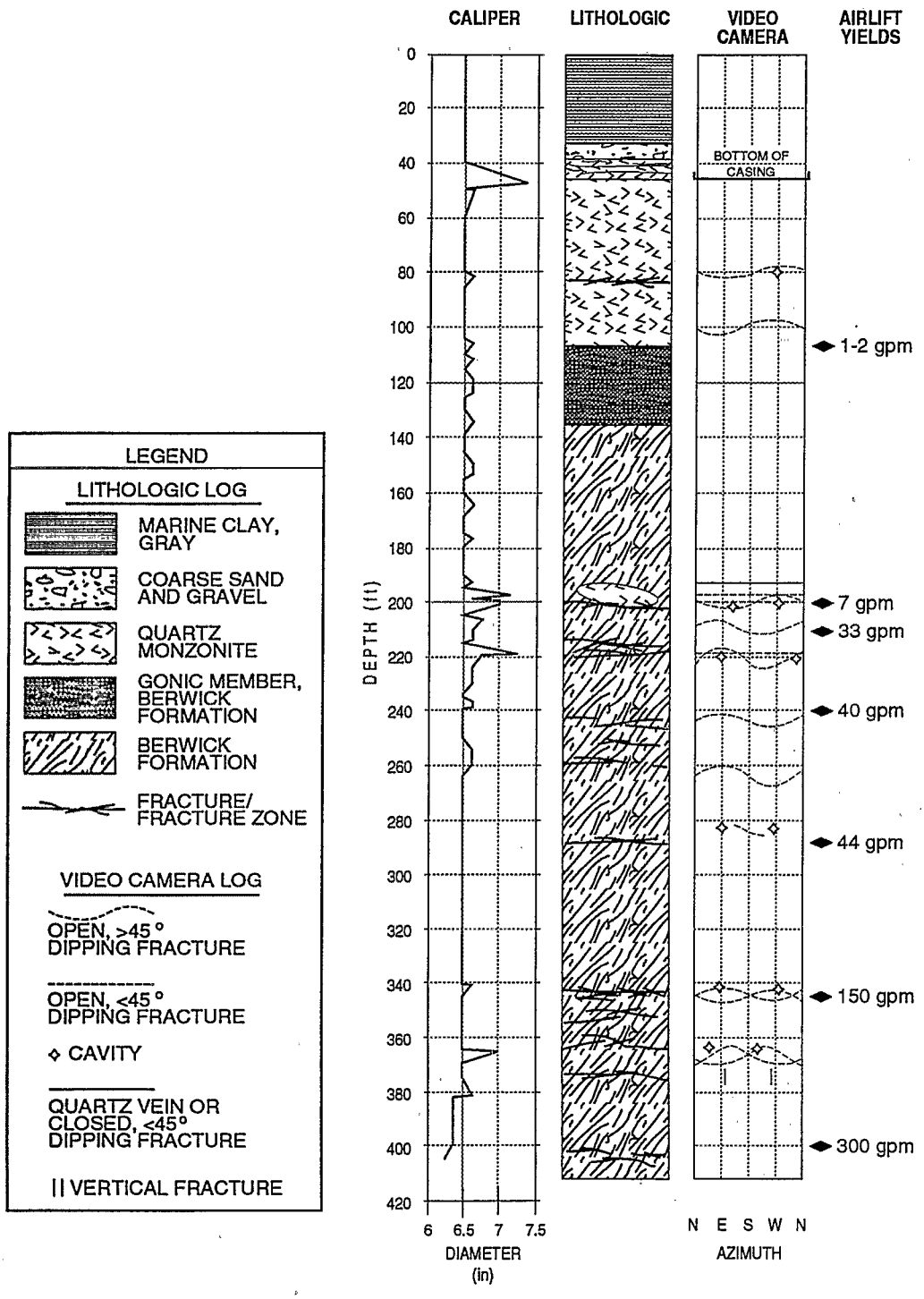


Figure 5-9. Diagram of caliper, lithologic, and borehole video camera logs of the test well, Blackwater Brook site, Dover, New Hampshire.

No mafic bedrock was encountered during drilling. It is possible that a steeply dipping mafic dike, wide enough to cause the magnetic anomaly but narrow enough to be missed by drilling, exists at the site. Recent blasting at the quarry 2 miles southwest of the site (Figure 5-6) has exposed a large body of gabbro (mafic rock). It may be that the sharp magnetic anomaly reflects, in part, shallow-depth alteration of chlorite to magnetite along the faulted contact. Examination of drill cuttings from the wells show ample evidence of iron-bearing mineralization (pyrite), as well as the magnetic minerals pyrrhotite and magnetite in the sections of borehole corresponding to the Berwick Formation. No magnetic minerals were found in the cuttings from the quartz monzonite. Alternatively, the step-like anomaly may be a deeper-seated feature. The geologic significance of the magnetic increase remains speculative.

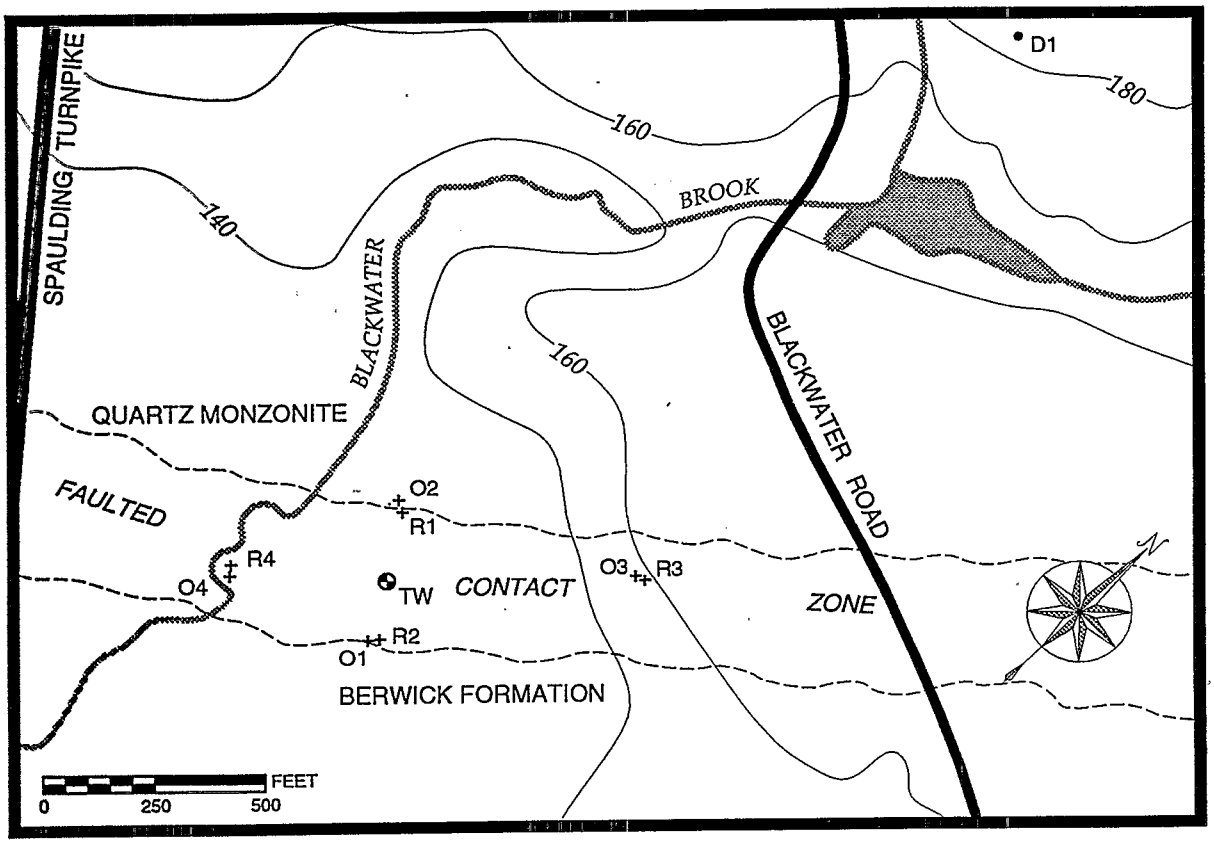
Well and Piezometer Installation

Bedrock Observation Wells--

With the success of the test drilling, the city elected to pursue the Blackwater Brook site as a future municipal water source. Four observation wells were drilled into bedrock: two, R3 and R4, along the N60°E trend and two, R1 and R2, in an orthogonal orientation along N30°W (Figure 5-10). All of these wells were drilled to approximately 400 feet (Table 5-2), and well construction was performed according to standard operating procedures outlined in the American Water Works Association publication "Standard for Water Wells" (1988) (Figure 5-11). The domestic well, D1, located about 2,000 feet north of TW (Figure 5-10), was used for water-level monitoring during aquifer testing. It penetrates about 10 feet of overburden and then enters bedrock. The well was drilled to about 350 feet and is estimated by drillers to yield 10 gpm.

Lithologic logs, obtained during drilling and by subsequent description of drill cuttings at a 5-foot interval, reveal that all the bedrock wells penetrate both the Berwick Formation and the quartz monzonite. The mapped contact between the two formations (Lyons et al., 1986; Novotny, 1968; Billings, 1956) coincides approximately with the N60°E lineament and is evident in all of the bedrock wells (Figure 5-10). Wells R1 and R2 along the N30°W direction, penetrate fracture zones that bear water. Wells R1 and R2 airlifted 170 and 300 gpm, respectively. Thus, the contact between the Berwick Formation and the quartz monzonite is actually a zone at least 300 feet wide (the distance between wells R1 and R2, transverse to the N60°E lineament). This zone contains an interfingering, or a melange, of the two rock types illustrated in the lithologic log of TW (Figure 5-9). In some wells (TW, R1, and R4) the Gonic Member of the Berwick Formation is distinguishable, and 20- to 40-foot-thick pegmatites are present in wells R3 and R4. In all the bedrock wells, quartz veins and monzonite intrusions from a few inches to a few feet thick are present.

Similar to TW, producing fracture zones in the bedrock observation wells are highly discrete. During drilling, the presence of larger cuttings and drill stem drops indicated fractures or fracture zones at one or more depths in all the bedrock wells. In all wells except R4, significant water production during drilling and airlifting indicated flow from one or more discrete fracture zones. Caliper logs, exemplified by Figure 5-9, showed increases in wellbore diameter at major fracture zones. These fracture zones correlate with fracture zones and water-producing zones indicated on lithologic logs.



LEGEND

TW = TEST WELL

D1 = DOMESTIC WELL AND NUMBER

O1 = OVERBURDEN WELL AND NUMBER

R1 = BEDROCK WELL AND NUMBER

—140— TOPOGRAPHIC CONTOUR (INTERVAL IS 20 FEET)

Figure 5-10. Locations of the test well and monitoring wells, Blackwater Brook site, Dover, New Hampshire.

TABLE 5-2. INFORMATION REGARDING BEDROCK AND OVERBURDEN MONITORING WELL CONSTRUCTION, LITHOLOGY, AND TESTING, BLACKWATER BROOK SITE, DOVER, NEW HAMPSHIRE

Well	Total Depth (ft)	Maximum Airlift Yield (gpm)	Geologic Material	Distance from Test Well (ft)	Static Water Level* (ft amsl)	Maxiumum Drawdown at 200 gpm Pumping (ft)
TW	412.0	300.0	Bedrock	N/A	>136 (flowing)	74
R1	400.0	170.0	Bedrock	153	135.92	59
R2	398.0	300.0	Bedrock	152	136.00	59
R3	425.0	150.0	Bedrock	596	140.73	32
R4	426.5	1.5	Bedrock	365	135.77	54
O1	35.0	N/A	30 ft clay, 6 ft sand and gravel	153	135.55	34
O2	41.0	N/A	36 ft clay, 5 ft sand	158	136.17	34
O3	21.0	N/A	11 ft clay, 6 ft rocks and boulders	590	142.80	14
O4	191.0	N/A	5 ft clay, 12 ft sand and gravel	369	135.37	19
D1	365.0†	10.0†	N/A	1,989	185†	None

Abbreviations: gpm = gallons per minute; ft amsl = feet above mean sea level; N/A = not applicable.
* Static water levels obtained in July 1990.
† Estimated values due to incomplete well records.

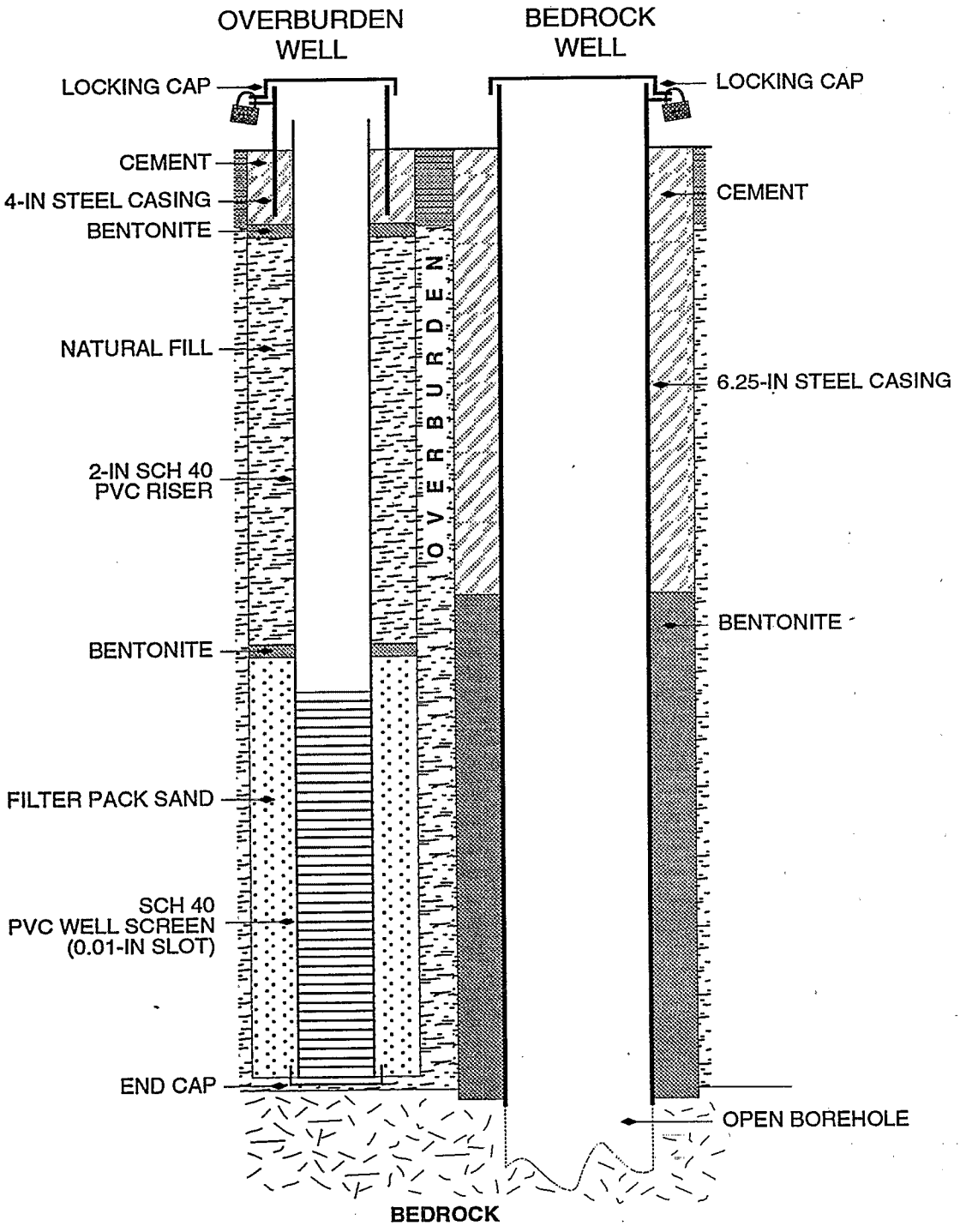


Figure 5-11. Schematic diagram of overburden and bedrock monitoring well designs, Blackwater Brook site, Dover, New Hampshire. (ANSI/AWWA A100-84; American Water Works Association, 1988.)

The importance of surface geophysical results in this environment was particularly apparent during the drilling of the observation wells along the N60°E zone. One of the two bedrock wells located along the N60°E zone, R3, was sited approximately 600 feet N60°E of TW, based on results of the magnetic survey. Well R3 airlifted more than 150 gpm and demonstrated interconnection to TW when the water level in TW dropped approximately 15 feet from its normal flowing-artesian condition during development of well R3. The second well, R4, was sited approximately 365 feet west of TW without rigorous use of the geophysical results. It is 425 feet deep, with an airlift yield of only 1.5 gpm. A fence diagram of the site geology is presented in Figure 5-12.

Overburden Observation Wells--

Overburden wells (O1, O2, O3, and O4) were placed next to bedrock observation wells (Figure 5-10). The surficial geologic mapping conducted by the USGS, along with field checks during the exploration process, resulted in a high degree of confidence in the conceptual model of surficial material deposition and distribution. Overburden wells were installed primarily to provide water-level measurements in the bedrock and overlying aquifers at the same locations. A secondary goal was to refine the model for surficial material deposition and distribution. All well construction (Figure 5-11) was performed according to standard operating procedures outlined by the American Water Works Association (1988).

The overburden material was sampled using a split-spoon device. Overburden thicknesses range between 19 and 45 feet (Table 5-2) and include a diversity of material: coarse sand and gravel, marine clay, weathered bedrock, fine sand, topsoil, rocks, and boulders. The marine clays are 30 or more feet thick in wells TW, O1, and O2, and thin to the east and the west (15 feet in well O3; 5 feet in well O4) (Figure 5-12).

Borehole Geophysics

Fractures provide the most important pathways for ground-water flow in bedrock aquifers. An understanding of the character of bedrock fractures (for example, depth, orientation, and morphology) is vital to conceptualizing the nature of flow in such aquifers. The primary means of obtaining information on fractures is through borehole logging, particularly with geophysical tools. Borehole geophysics can then be correlated with other hydrogeologic investigations, such as aquifer testing and dye tracing, to better understand the flow system. The suite of borehole logs and geophysical surveys acquired in this study include the following:

- Video camera logs
- Flowmeter surveys
- Hydrophysical logs
- Borehole acoustic televiewer logs
- Gamma logs

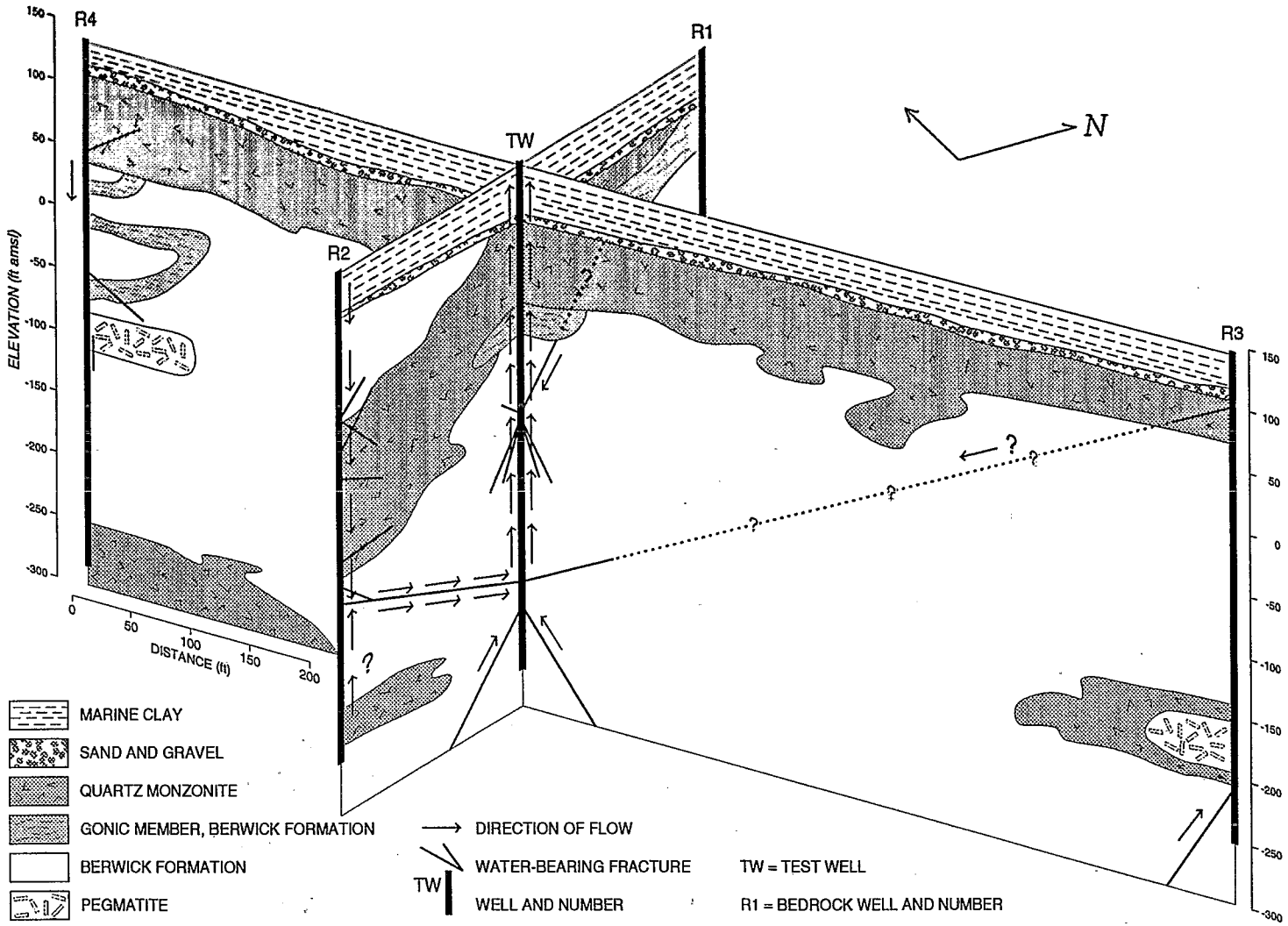


Figure 5-12. Geologic fence diagram, Blackwater Brook site, Dover, New Hampshire.

Video Camera Logging--

All bedrock wells were examined with a 4.75-inch-diameter borehole video camera. The camera has two lenses: one capable of looking directly downhole, and one that rotates to examine the sides of the borehole. Low-rate pumping (5 to 10 gpm) of the well during logging ensured a clear picture by minimizing turbid water. Another advantage of pumping is the ability to see particles of rock (for example, drill cuttings) moving into the borehole from large fracture zones. The camera is oriented by suspending a rope down the magnetic north side of the wellbore before the tool begins to descend. Visual inspection of the videotapes suggests that the camera generally did not rotate with respect to the wellbore. If the camera did spin, it did so slowly. Such spinning would invalidate interpretations of fracture orientation, but not of fracture depth or morphology.

The video camera revealed discrete water-bearing fractures and complex zones of intense, intersecting vertical and dipping fractures (Figure 5-9). Major increases in water production, determined during drilling with airlift tests, occurred where the borehole intersected high-angle water-bearing fractures (Figure 5-9, at 220 and 342 feet). Low-angle fractures do not appear to be major flow conduits in TW.

Flowmeter Surveys--

Two heat-pulse flowmeter surveys were conducted in the bedrock wells at the Blackwater Brook sites: one using a commercially available tool and, later, one supplied and operated by the USGS, Denver office. Analysis of the data attest to some of the complexities of dealing with flow through fractured bedrock. Water enters and exits the borehole at very discrete intervals rather than uniformly along its length. Portions of a borehole may show downward flow, while other sections exhibit upward flow. Correlation of ground-water movement between boreholes is complicated because of the large distance involved and convoluted flowpaths. Flow directions within the fracture system, as determined from the combination of the two flowmeter surveys, are graphically depicted in Figure 5-12.

Results were obtained with commercially available vertical and horizontal flowmeter equipment for well R2 during the pumping of TW at about 30 gpm. From the surface to about 270 feet, flow in well R2 is downward. Downward flow generally increases from less than 0.10 gpm at the surface to more than 0.75 gpm at 265 feet, as fractures at several discrete depths contribute water to downward flow in the wellbore. From 280 to 300 feet, upward flow occurs at about 0.5 gpm. Data were not obtained below 300 feet. A net outflow of over 1.3 gpm must occur between 270 and 280 feet. Drilling and caliper logs indicate a major water-bearing fracture zone at this depth, and the horizontal flowmeter indicates flow in a northwesterly direction.

The USGS flowmeter, equipped with inflatable packers, was used to survey wells TW, R2, R3, and R4. This tool can only be used to perform vertical-flow surveys. Preliminary results indicate that the TW has upward flow from the main producing fracture at 342 feet. Nearly stagnant conditions prevail below 342 feet. Under static conditions (no pumping, with the natural, 5-gpm artesian flow in TW), well R2 receives water from several fractures between 100 and 270 feet. Downward flow occurs in this well to 275 feet. Pumping TW at 16.5 gpm intensifies the same effects. Under static conditions, flow in well R4 is undetectable. When TW was pumped at 16 gpm, a barely detectable downward flow of 0.01 gpm was measured between 0 and 200 feet in well R4. When well R4 was

pumped at 1.5 gpm, the water level dropped rapidly, but upward flow was detected with inflow at 90 to 100 feet. Inflow to well R3 occurs between 350 and 380 feet; water exits the well between 260 and 300 feet, with weak upward flow between these two zones (Vernon et al., 1993)

The major direction of flow appears to occur along the well R2 and TW alignment (N30°W), with lesser amounts along the N60°E trend. Well R2 probably contributes water directly to TW from the outflow in R2 at 275 feet (Figure 5-12). Flow moves down the borehole from the surface and up the borehole from the bottom to the 275-foot level in well R2. From this information, coupled with analysis of the acoustic televiewer logs, a direct connection is seen to exist between wells R2 and TW, with water entering TW at the major fracture at 342 feet (Figure 5-12). Wells R3 and R4 also show interconnection with TW, but precisely which fractures or system of fractures intersect is difficult to ascertain.

Hydrophysical Logging--

Hydrophysical logging involves replacing the standing column of formation water with uniformly deionized water, and then profiling the physical and chemical changes of the replacement water. These changes occur when contrasting formation water is drawn back into the wellbore by continuous, low flow rate pumping (Pedler, 1991). Downhole wireline water quality instrumentation, which simultaneously measures fluid electrical conductivity (FEC), temperature, pH, and oxidation-reduction potential (Eh), is used to log the physical and chemical changes of the replacement water. The FEC logs obtained from these wellbores are used to evaluate the location, rate of production (hydraulic conductivity), and FEC of the formation water for each of the identified hydraulically conductive intervals (Pedler, 1991).

The wellbores tested (TW, R2, R3, and R4) display numerous, discrete producing intervals (Table 5-3). Vertical flow within two of the wellbores, TW and R2, was observed under ambient conditions. The TW displayed very strong upward flow (up to 6 gpm), with the major inflow at 345 feet. Well R2 displayed moderate downward vertical flow (as much as 0.5 gpm), with fluids exiting the wellbore at the 274-foot interval (Pedler, 1991).

Three types of formation water were observed at the Blackwater Brook site based on the results of the temperature-corrected FEC logs. Type I water is of low FEC, with values of 100 to 150 microSiemens per centimeter (μS/cm); Type II water is of moderate FEC, with values of 450 to 600 μS/cm; and Type III water is of high FEC, with values of greater than 800 μS/cm. In general, a trend of increasing FEC with depth was observed in each wellbore (Table 5-3) with abrupt, not continuous increases.

The abrupt changes in FEC with depth may be attributed to a combination of local, intermediate, and regional flow systems interacting with the discrete character of flow within the bedrock fracture system (Toth, 1963). The water quality of the local system more closely reflects that of rain water with its low conductivities. For example, the fracture at 40 feet in well R3 lies just below the end of the well casing. Well R3 lies near permeable overburden that receives direct precipitation, and its low FEC reading probably reflects recent recharge. The local system extends to as deep as 200 feet in TW (Table 5-3), but does not appear in well R2 or in well R4, indicating that flow within the fracture system is extremely discrete. The intermediate flow system, interpreted as Type II water, also demonstrates isolated flow within the bedrock aquifer. Type II water is seen at 345 feet in TW

TABLE 5-3. FLOW RATES AND FLUID ELECTRICAL CONDUCTIVITIES AT DISCRETE ZONES WITHIN BOREHOLES FROM HYDROPHYSICAL LOGGING, BLACKWATER BROOK SITE, DOVER, NEW HAMPSHIRE

Well	Depth (ft)	Flow Rate (gpm)	FEC* (µS/cm)
TW	194	0.17	130
	199	0.17	130
	218	0.17	130
	345	6.65	580
	362	0.84	780
R2	106	0.21	500
	128	0.26	500
	156	0.05	500
	172	0.16	500
	230	0.22	500
	265	0.33	500†
	274	0.52	500†
R3	40	0.07	150
	358	3.43	460
R4	100	0.10	1,000
	198	0.10	600
	245	0.19	1,250
	365	0.03	900

Abbreviations: FEC = fluid electrical conductivity; gpm = gallon per minute; µS/cm = microSiemens per centimeter.
* Flow rate and FEC values are depth-specific.
† FEC increases from 5 to 500 µS/cm during pumping, then stabilizes at 500 µS/cm.

and at 358 feet in well R3. It comprises the entire borehole in well R2, but is completely absent in well R4. The regional flow system is reflected by the Type III water, and may represent older, very slow moving water recharged at greater distances than the local or intermediate flow systems. The higher FEC readings may also be influenced by the presence of trapped sea water, vestiges of periods of marine inundation during the Pleistocene era. High readings of FEC throughout the length of well R4 reflect the lack of interconnection of fractures that could bring in younger, more recently recharged water of lower conductivity.

Borehole Acoustic Televiewer Logging--

The borehole acoustic televiewer is a wireline tool that emits an acoustic signal and receives the signal's reflection from the wellbore. The presence, attitude, and, sometimes, morphology (fracture width and degree of filling) of fractures can be determined using the televiewer (Paillet et al., 1987; Plumb et al., 1985; Zemanek et al., 1970).

The televiewer logs show fewer fractures than those shown on the video camera logs. However, the fracture orientations can generally be determined with more confidence using the televiewer, because the instrument is oriented and controlled, whereas slow spinning of the video camera cannot be ruled out. In addition, the televiewer tends to select the more prominent fractures. The televiewer showed a number of distinct fractures and fracture sets in each well surveyed (TW, R2, R3, and R4). Analysis of televiewer logs indicates good correspondence with the occurrence of fractures as interpreted by other means, such as drilling, caliper logging, video camera logging, and flowmeter surveying.

The televiewer logs reveal major fractures in TW and well R2 at 342 and 275 feet, respectively. In both cases, the fractures strike approximately N20°E and dip about 45°W. According to flowmeter results, 342 feet is the main depth of inflow to TW, and 275 feet is the main depth of outflow from well R2. Thus, it is likely that one fracture or set of fractures directly connects well R2 and TW, intersecting well R2 at 275 feet and TW at 342 feet. Because the fractures appear to be dipping to the west on the televiewer logs and TW lies west of well R2 (Figures 5-10 and 5-12), the fracture may intersect TW at greater depth. If the estimated N20°E strike and 45°W dip for this fracture or fracture set is correct, the fracture (or fracture set) would intersect TW at about 350 feet--the approximate depth of the main producing fracture (Figure 5-12). The discrepancy in the exact depth is within a ±20° precision error of fracture orientation determined from the televiewer logs (Vernon et al., 1993)

The televiewer log for well R4 shows a prominent fracture at 356 feet, but no water is produced from this fracture. While this fracture appears to have a large aperture, it does not show evidence of alteration or reflective infilling minerals, as do the producing fractures in TW and well R2. Also, the fracture at 356 feet in well R4 occurs as an isolated fracture rather than a fracture set. If the water-bearing fracture that connects TW and well R2 has a strike of N20°W and a dip of 40°W, its projection would intersect well R4 below its completion depth.

The televiewer log for well R3 suggests the existence of a weathered fracture zone between 350 and 380 feet in depth, which is the major water producing zone for the well (Figure 5-12). This zone also appears to contain fractures that dip to the west (Vernon et al., 1993)

Gamma Logging--

Natural gamma logs depict gamma-ray emissions from different depths in the wellbore. Different rock types produce characteristic gamma-ray counts. In the Blackwater Brook wells, the Berwick Formation generally produces about 300 counts per second (cps), and the quartz monzonite generally produces about 600 cps. Sharp changes in gamma-ray counts generally accompany formation changes.

The gamma-ray logs tend to confirm the depths of bedrock contacts in the lithologic logs. A gamma-ray count of several thousand per second at 225 feet in well R4 corresponds to the presence of a low-grade uranium ore in pegmatite logged at that depth. This uranium ore is not considered to be a water quality threat to a future production well that would be sited next to TW. Nevertheless, periodic monitoring of radioactive constituents at the production well may be required under the Safe Drinking Water Act (SDWA).

Aquifer Testing and Characterization

Ambient Water-Level Monitoring--

Ambient water-level data for the bedrock and overburden wells were gathered before aquifer testing. The purpose of this task was to document water-level changes under nonstress conditions. Water levels vary for several reasons, including recharge events, barometric pressure differences, and diurnal variation. Aquifer-test data should be corrected for natural fluctuations in water level prior to analysis. Water-level trends were recorded before and during aquifer testing in well D1, topographically upslope from TW, and in wells R1, R2, R3, O1, O2, and O3 (Figure 5-10). Transducers were not available to place in wells R4 and O3 to monitor ambient water levels. Water-level monitoring at 3-hour intervals began 16 days before constant-rate pumping, changing to hourly intervals 2 days before testing. Water levels in well D1 were monitored hourly throughout pumping and recovery.

Water levels in well D1 fluctuated substantially, as might be expected for a low-yield (10 gpm) domestic bedrock well (Figure 5-13). The specific capacity of a low-yield bedrock well can be exceeded during periods of frequent, maximum usage resulting in large drawdowns and fluctuations such as those typified in Figure 5-13. A slight drop in the overall trend of the water levels from the beginning of monitoring to the start of aquifer testing can be explained by the fact that the property changed ownership. The single owner sold the property to a couple, and their higher level of water use may be reflected in the amplitude of the curve. However, neither the onset of aquifer testing at Blackwater Brook nor its cessation produced meaningful changes in the overall trend of water levels in well D1. Water-level data in Figure 5-13 indicate that the 15-day pumping and recovery test at TW showed no response in well D1.

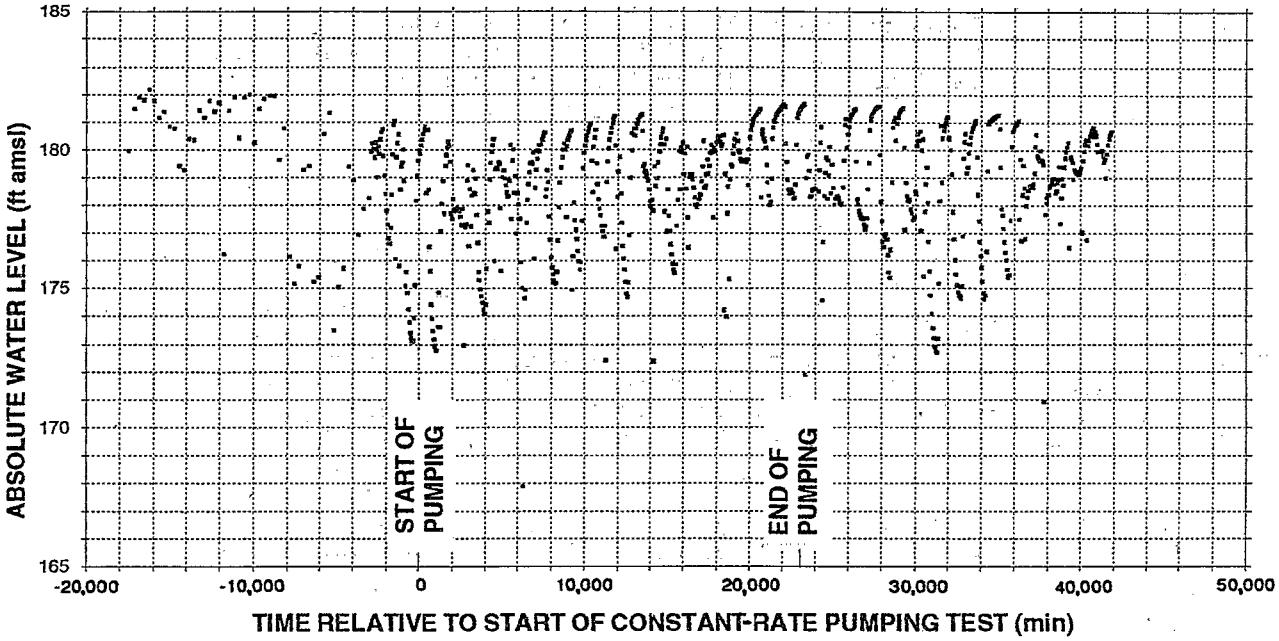


Figure 5-13. Plot of water-level data from domestic well D1, Blackwater Brook site, Dover, New Hampshire.

A plot of water-level trends in the overburden and bedrock wells monitored 2 weeks prior to pumping is given in Figure 5-14. Most of the water levels cluster around the 136-foot level. The higher water levels in O3 and R3 reflect the fact that these wells are upgradient and about 600 feet away from the others. Diurnal variations are seen in the water-level record of well R3 (denoted as a solid diamond in Figure 5-14), particularly during the time just prior to the beginning of pumping when measurements were taken every hour. The slight depression in water levels at approximately 11,000 and 8,500 minutes before pumping was caused by some minor pumping required to perform flowmeter testing at the site. However, the data show no discernible trend prior to aquifer testing (pumping phase). As a result, no correction for ambient trends was made to the pumping data gathered during the test.

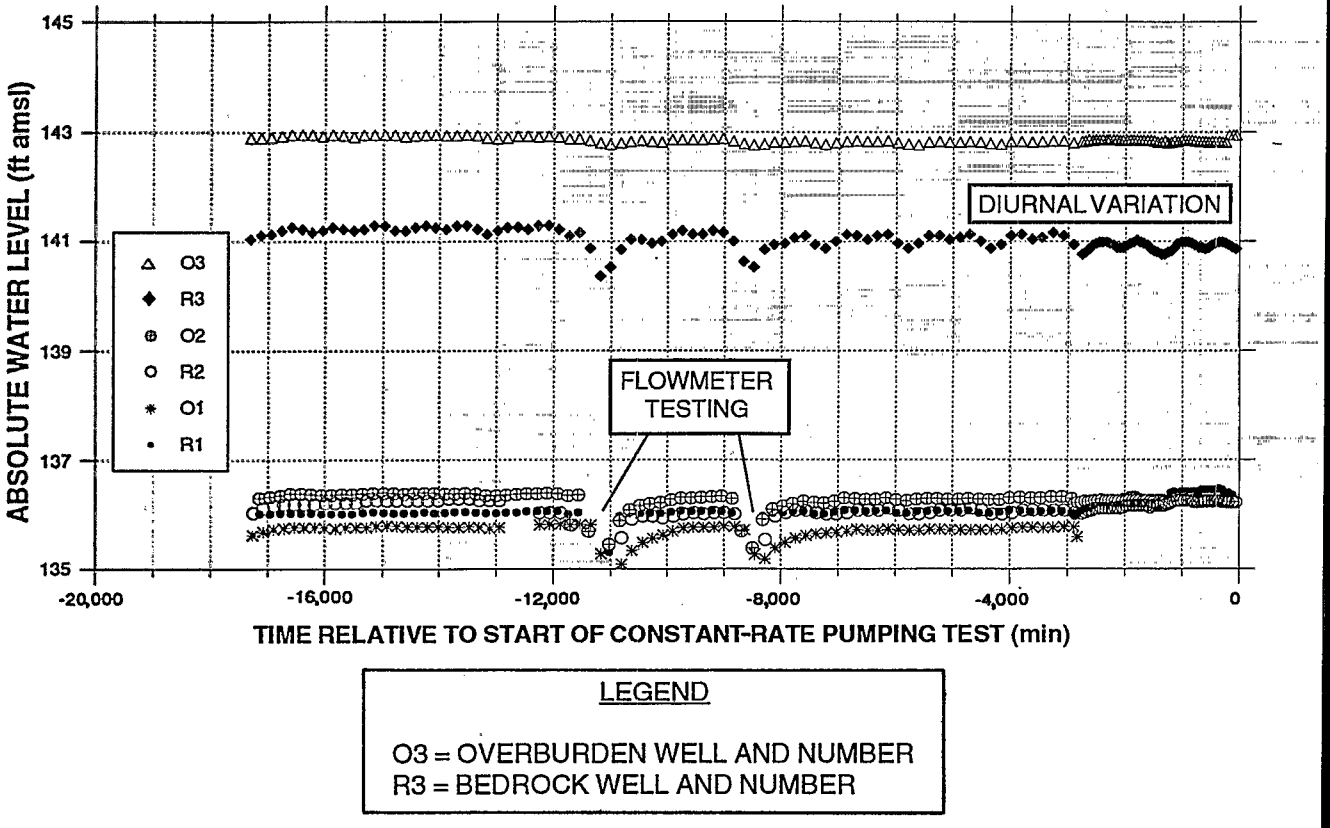


Figure 5-14. Plot of ambient water levels in overburden and bedrock monitoring wells, Blackwater Brook site, Dover, New Hampshire.

Ambient ground-water flow in the bedrock and overburden aquifers at the Blackwater Brook site is from the northeast to the southwest (Figures 5-15 and 5-16, respectively; Table 5-2), generally along the trend of the N60°E fracture zone. Blackwater Brook is perched above the overburden marine clay layer (Figure 5-2) and is not believed to be hydraulically connected to either the overburden or the bedrock aquifers. Northeast of TW, the hydraulic gradient in the bedrock aquifer is approximately 0.008, while that in the overburden aquifer (the sands and gravels beneath the clay layer) is 0.005. Southwest of TW, however, the gradient drops to 0.0005 in the bedrock aquifer and to 0.001 in the overburden aquifer. The decrease in ground-water gradient coincides approximately with the intersection of the N5-10°W and N60°E fracture zones (Figure 5-3). The ambient water-level contouring results suggest that the N5-10°W fracture zone is a partial barrier to ground-water flow. Assuming that the flux of water (the Darcy velocity, q) across that plane is constant, then the difference in hydraulic gradient is explained by a 16-fold increase in hydraulic conductivity in the bedrock aquifer and a fivefold increase in the overburden aquifer west of the N5-10°W fracture zone. The cross-cutting N5-10°W fracture zone may act as a partial barrier boundary restricting flow along the N60°E trend.

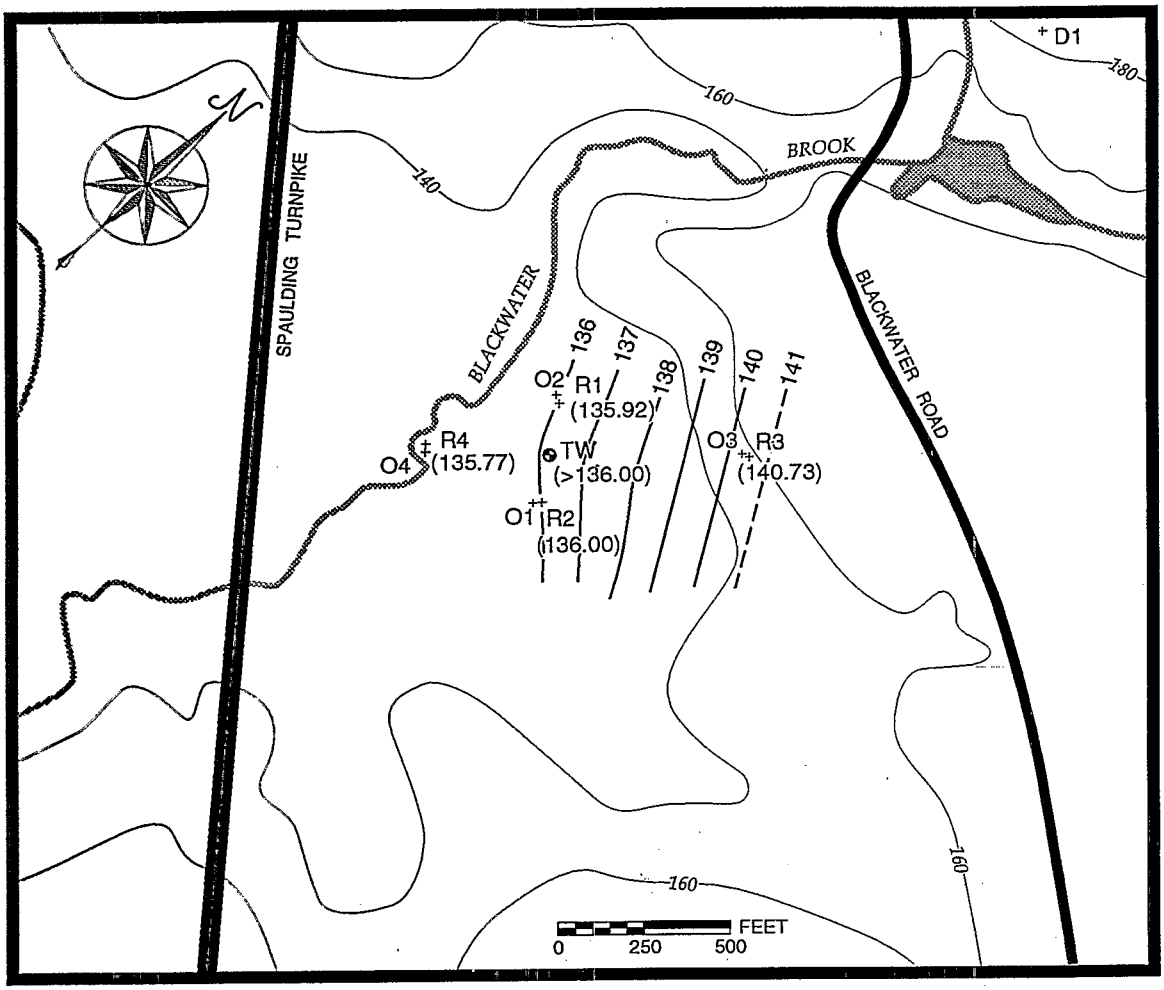
Constant-Rate Aquifer Testing--

The test well was pumped at a constant rate of 200 gpm for 15 days. Water was discharged to a marshy area downgradient from the monitoring wells that drains to Blackwater Brook. With 20 to 30 feet of marine clay overlying the gravel and bedrock in this area, it is unlikely that the discharge water recharged the aquifer during testing. Pumping was suspended briefly four times between 6,500 and 7,200 minutes (4.5 to 5 days into pumping) because of generator malfunction (a total of about 2 hours shutdown). Drawdown levels returned to expected levels after resumption of continuous pumping (Figure 5-17). Discharge rates were monitored throughout the test using both an inline flowmeter with totalizer and an orifice weir. A total of 4,285,000 gallons of water was discharged from TW over the course of the test, averaging 199.5 gpm (correcting for 2 hours of shutdown).

Water levels in six of the Blackwater Brook Site wells (TW, R1, R2, R3, O2, and O3) were monitored with automatic data loggers. The remaining three wells, R4, O1, and O4, were monitored manually with dedicated electronic water-level probes. All readings were collected to tolerances of ±0.01 foot. Readings were taken on a logarithmic schedule appropriate for aquifer-test analysis. This schedule was followed for 15 days of pumping, and repeated for 14 days of recovery.

Diagnostic plots (logarithmic and semi-log) of time-drawdown and time-recovery were created for all observation points. An example of the time-drawdown semi-log plots for well R4 is shown in Figure 5-17. The rise in water level at approximately 7,500 minutes is due to the 2-hour pump shutdown resulting from generator problems. Barrier-boundary conditions could not be detected on these plots, possibly because the pumping well is so close to the boundary represented by the N5-10°W fracture zone and its impact was felt immediately at the pumping well.

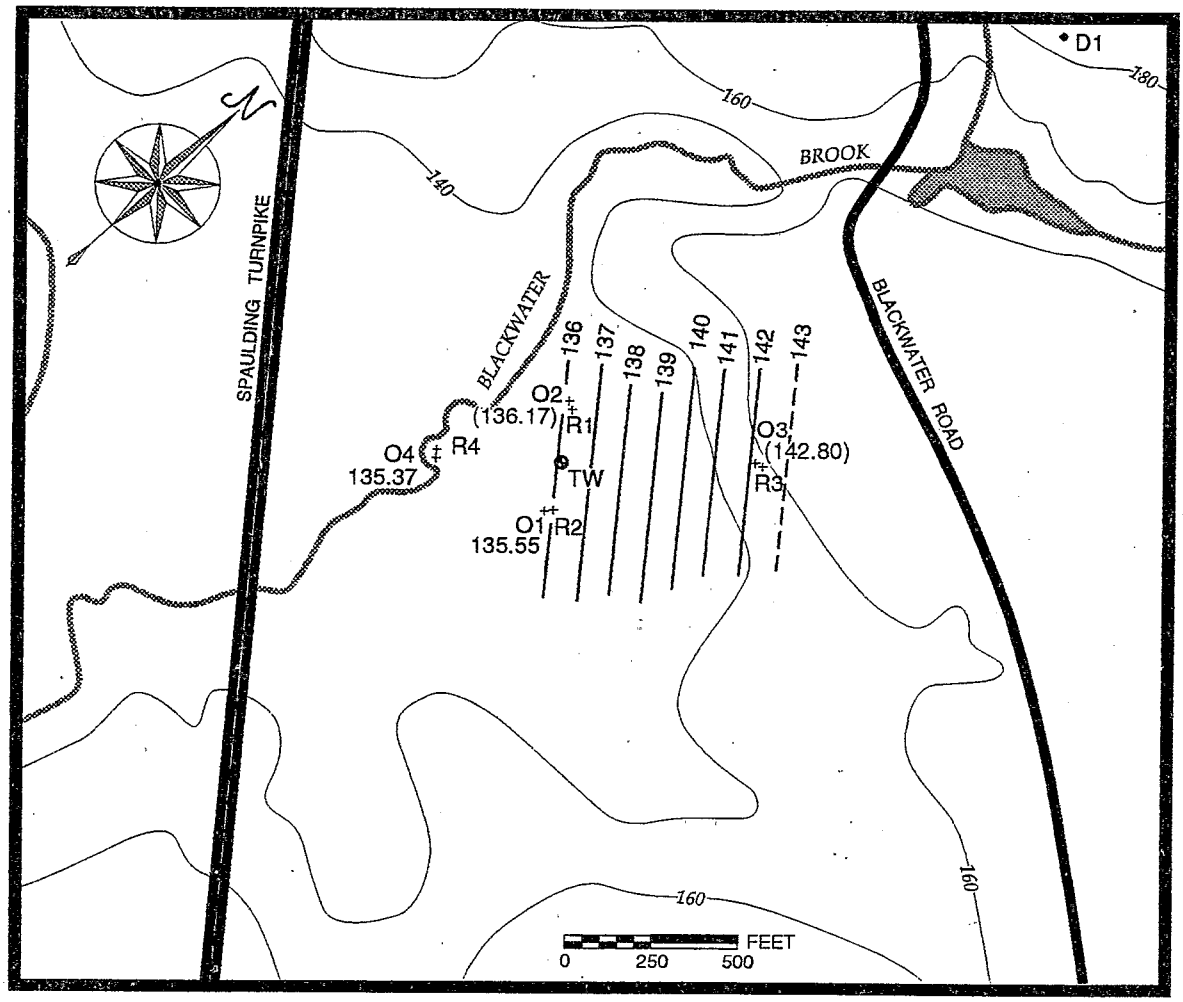
The pumping well (TW) stabilized at 74 feet of drawdown (Table 5-2) approximately 10,000 minutes (7 days) after pumping began, for a specific capacity of 2.7 gallons per minute per foot (gpm/ft). The two bedrock wells nearest TW, R1 and R2 (located 150 feet away), responded immediately to pumping. Wells R4 and R3 (at 365 and 596 feet from TW, respectively) began to



LEGEND

TW = TEST WELL	O1 = OVERBURDEN WELL AND NUMBER
D1 = DOMESTIC WELL AND NUMBER	R1 = BEDROCK WELL AND NUMBER
138	WATER-LEVEL CONTOUR (INTERVAL 1 FOOT; DASHED WHERE INFERRED)
140	TOPOGRAPHIC CONTOUR (INTERVAL 20 FEET)
(135.77)	WATER LEVEL IN BEDROCK WELL (FEET AMSL)

Figure 5-15. Ambient hydraulic gradient and water-level data for the bedrock aquifer, July 1990, Blackwater Brook site, Dover, New Hampshire.



LEGEND

TW = TEST WELL

O1 = OVERBURDEN WELL AND NUMBER

D1 = DOMESTIC WELL AND NUMBER

R1 = BEDROCK WELL AND NUMBER

138 WATER-LEVEL CONTOUR (INTERVAL 1 FOOT; DASHED WHERE INFERRED)

140 TOPOGRAPHIC CONTOUR (INTERVAL 20 FEET)

(135.37) WATER LEVEL IN OVERBURDEN WELL (FEET AMSL)

Figure 5-16. Ambient hydraulic gradient and water-level data for the overburden aquifer, July 1990, Blackwater Brook site, Dover, New Hampshire.

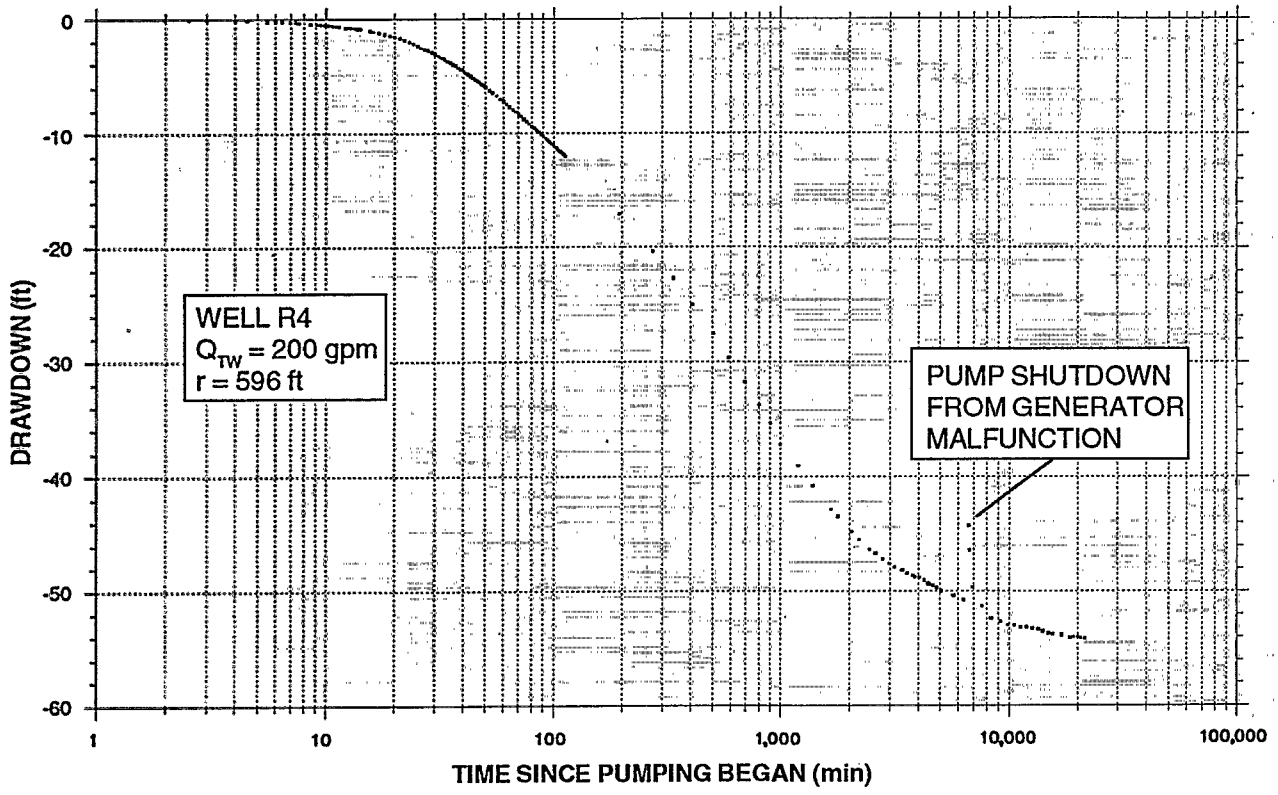


Figure 5-17. Semi-log plot of drawdown versus the log of time for observation well R4 when the test well is pumping, Blackwater Brook site, Dover, New Hampshire.

draw down after 8 and 10 minutes of pumping, respectively. All bedrock wells reached steady-state or pseudo steady-state conditions (a constant change in head with time; McWhorter and Sunada, 1977) by about 10,000 minutes (7 days) of pumping. Recovery was rapid; 90% of drawdown in TW recovered after 10,000 minutes (7 days) following pumping shutdown.

Overburden wells also responded quickly to the pumping of TW and exhibited substantial drawdown (Table 5-2). These wells also stabilized approximately 10,000 minutes (7 days) into the test. One overburden well, O4, dewatered completely after approximately 2,450 minutes (1.7 days) of pumping. The lithologic log from well O4 indicated the thinnest section of permeable sands and gravels under the marine clays, which is interpreted as a pinch-out of a beach facies.

Water-level contours noted under pumping conditions at TW suggest the presence of a partial barrier boundary between TW and well R4 (Figure 5-18). The TW could not draw the water it needed from the northeast; hence, the cone of depression expanded in an oval shape. The resulting oval potentiometric surface typifies the 15-day constant-rate test. Contouring of overburden well water levels generally mirrored the patterns seen in the bedrock aquifer. Water-level data from well D1, (approximately 2,000 feet from TW; Figure 5-10) showed no response during the test. The ambient-trend monitoring data indicate that D1 was in use during the test, but there was no significant trend of dropping or rising water levels associated with the stress due to the pumping of TW (Figure 5-13).

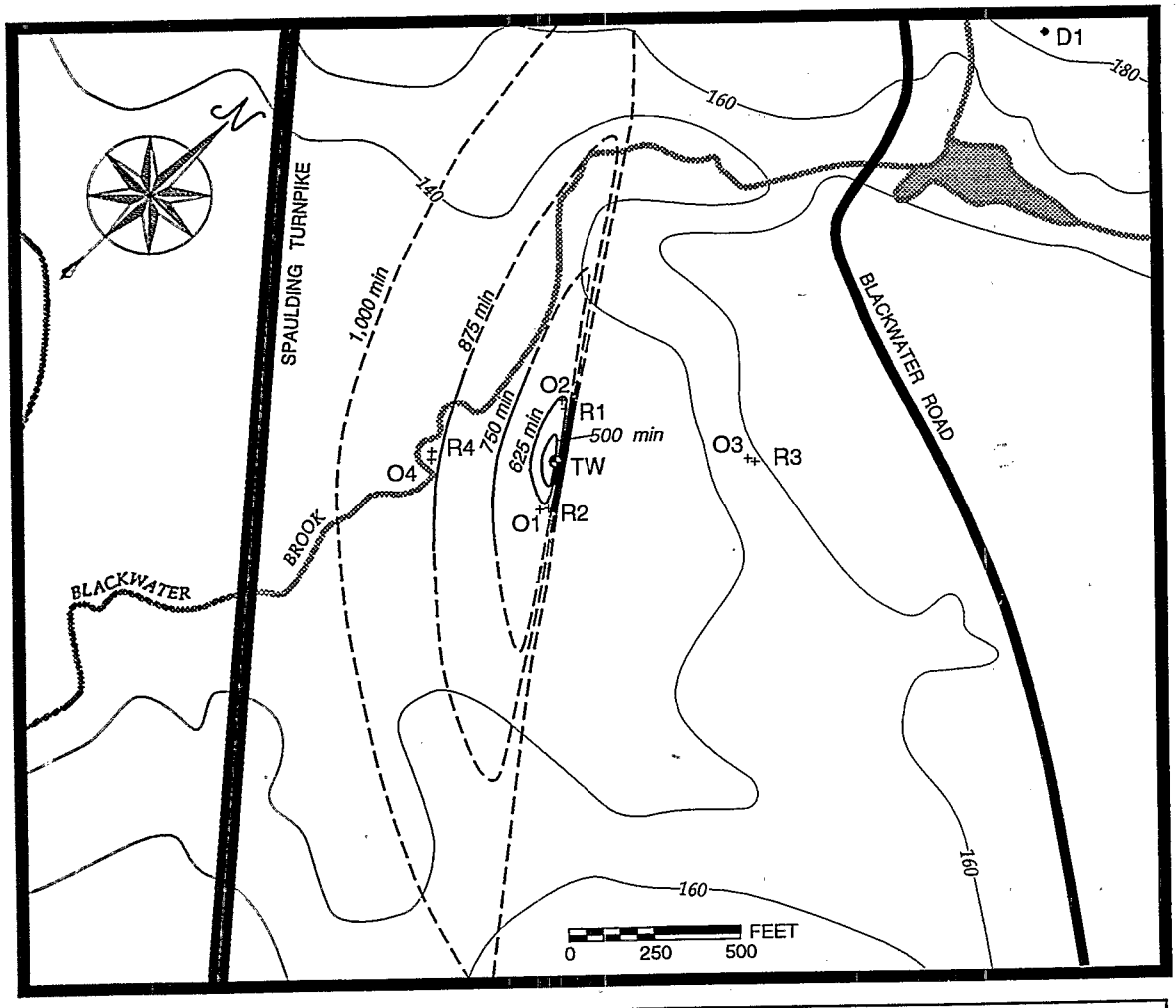
An estimate of the percentage of discharge from TW contributed by the sands and gravels above the bedrock during constant-rate aquifer testing was calculated volumetrically. The elevations of the water-level surface during pumping are subtracted from the elevations of the ambient water-level surface. The resulting volume between pre- and post-pumping conditions in the overburden was multiplied by a calculated storage coefficient (S) of 5×10^{-5} for those areas that remained confined, and by an apparent specific yield of 0.15 (Morris and Johnson, 1967) where water levels drew down below the clay. From this analysis, approximately 20% of the discharge came from the permeable sands and gravels beneath the confining clay in the immediate vicinity of TW.

Aquifer Coefficient Calculations--

Analytical models provide powerful tools for determining aquifer properties and for predicting aquifer behavior under differing hydrogeologic conditions. The principal challenge in using analytical models correctly lies in matching the proper analytical model with the conceptual model of the system. That is, it is important to find an analytical model whose attributes and assumptions most closely match conditions in the field.

The results of the hydrogeologic investigation prior to aquifer testing demonstrate the complex character of the system. Rather than an equivalent porous medium, the aquifer at Blackwater Brook is a regime of separated flow systems within the fractured bedrock. Discrete flow is reflected in the isolated fractures noted in well logs as well as in video and acoustic televiewer analyses; it is also corroborated by distinct hydrochemical characteristics seen in the hydrophysical logging. Application of radial flow models such as the Theis (1935) or the Cooper-Jacob (1946) solutions would not appropriately match the conceptual model for the site. Accordingly, several other analytical approaches were used to try to quantify the aquifer coefficients and to serve as predictive tools for WHPA delineation and ground-water monitoring design.

Initially, the time-drawdown curve for the pumping well (TW) was analyzed by using an analytical model conceived by Gringarten, Ramey, and Ranghaven for a single-plane, vertical fracture in an otherwise homogeneous, isotropic, confined aquifer (Kruseman and de Ridder, 1989). The bulk transmissivity calculated for the aquifer is 2,200 gpd/ft without taking into consideration directional variables associated with fracturing and faulting.



LEGEND

TW = TEST WELL

O1 = OVERBURDEN WELL AND NUMBER

D1 = DOMESTIC WELL AND NUMBER

R1 = BEDROCK WELL AND NUMBER

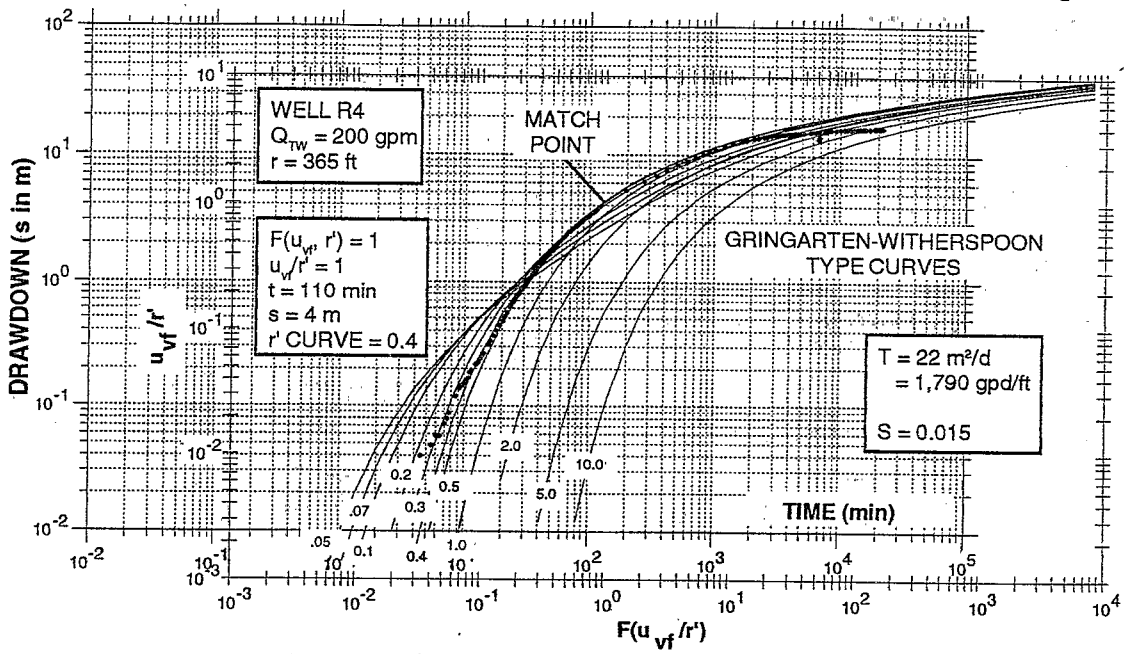
140 TOPOGRAPHIC CONTOUR (INTERVAL 20 FEET)

875 min EXPANSION OF 100-FOOT WATER-LEVEL CONTOUR IN TIME (DASHED WHERE INFERRED)

Figure 5-18. Contour map of the expansion of the 100-foot water-level contour in time during constant-rate aquifer testing, Blackwater Brook site, Dover, New Hampshire.

An analytical model by Gringarten and Witherspoon also assumes a single-plane, vertical fracture, but uses observation data to calculate a storage coefficient and directional transmissivities (Kruseman and de Ridder, 1989). Different type curves are used to calculate directional hydraulic conductivities for the more permeable and less permeable directions. An example calculation is provided in Figure 5-19 for well R4. Applying these curves resulted in transmissivity values of 4,700 gpd/ft for well R1 and 4,100 gpd/ft for well R2 along the N30°W well alignment. The transmissivity is 1,800 gpd/ft in the less permeable direction along the N60°E trend (wells R3 and R4). Storage values calculated using data from wells R3 and R4 are 0.05 and 0.015, respectively. Water levels in both wells were drawn down below the confining layer of clay into bedrock, so the high storage values (greater than 0.01 for semi-confined to confined aquifers) probably reflect a combination of confined and unconfined conditions.

Storage values calculated for wells R1 and R2 using the Gringarten-Witherspoon method are 0.56 and 0.77, respectively. A storage coefficient in the traditional sense cannot be determined for this setting. Wells R1 and R2 are probably completed in the same fracture system as TW, in the manner of an extended well, suggesting the applicability of the linear flow model by Jenkins and Prentice (1982). This method provides values of hydraulic diffusivity (D), where D = T/S, for the aquifer, given good knowledge of the structural conditions that produce the linear flow patterns in the aquifer.



WHERE:	BEDROCK WELL	= R4
	TEST WELL DISCHARGE	= Q_{TW} (gpm)
	DISTANCE FROM R4 TO TEST WELL	= r (ft)
	WELL FUNCTION AND FACTORS	= F (u_{vf}, r')
	TIME	= t (min)
	TRANSMISSIVITY	= T (gpd/ft)
	STORAGE COEFFICIENT	= S

Figure 5-19. Logarithmic plot of time versus drawdown and Gringarten-Witherspoon type curves (Kruseman and de Ridder, 1989) for observation well R4 when the test well is pumping, Blackwater Brook site, Dover, New Hampshire.

The Jenkins-Prentice method could not be applied to data from wells R1 or R2. Automatic data loggers indicated drawdown impact at both wells at 0.001 minute following the beginning of pumping. While plots of drawdown versus the square root of time for wells R1 and R2 show linear flow patterns at an early time (a straight-line plot), the extrapolated time of zero drawdown necessary to perform the calculations is impossible to determine.

The linear flow model was applied to the data for wells R3 and R4 using the radial distance from the pumping well as the distance from the fracture (the extended well). Values for D were approximately 3.4 million feet squared per day (Mft^2/d) for well R4 and 8.5 Mft^2/d for well R3 (Figure 5-20) using early- to middle-time data. The method does not allow independent calculation of a storage coefficient. However, because the linear flow portion of the curve occurred before unconfined conditions were obtained, a low storage coefficient can be assumed. Assuming a calculated value of 5×10^{-5} for S, the resulting transmissivity values are 1,300 gpd/ft for well R4 and 3,200 gpd/ft for well R3.

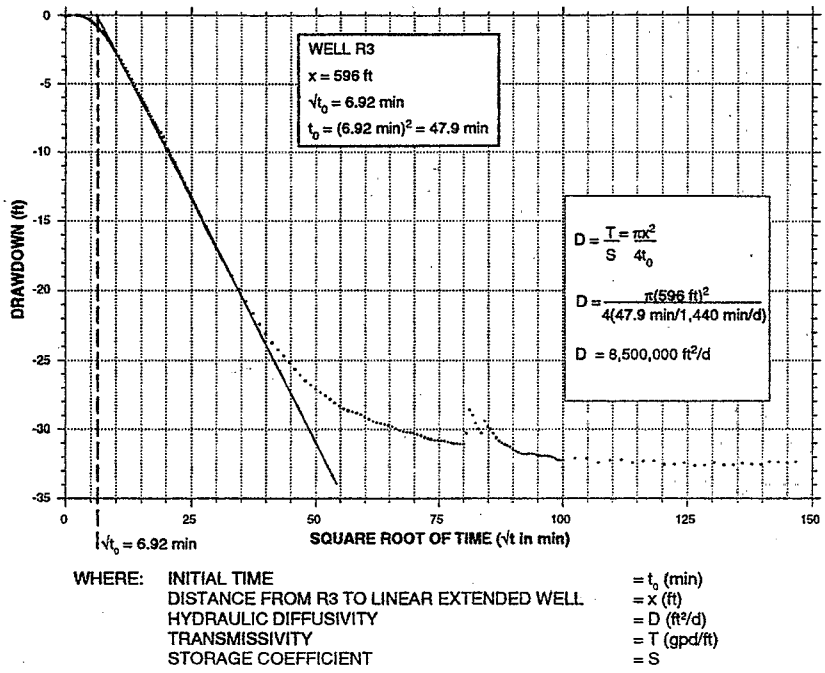
The results of analytical modeling are ambiguous, and the exercise is subject to various interpretations. None of the analytical approaches adequately accounts for the complexities of the system. Perhaps for an aquifer of this type, purely analytical solutions are inadequate for the task of describing the system. Other analytical models (for example, Papadopolus, 1965) should be tried as other possible approaches and may be applicable in less complicated systems.

The results of the analytical modeling indicate that it is not practical to use such models as a means of quantifying the aquifer properties and then employ the model to predict a particular criterion (such as drawdown or time of travel) for a wellhead protection zone. To develop a predictive tool for WHPA delineation and monitoring at the Blackwater Brook site, a more direct approach was taken. Dye tracing was used to determine seepage velocities in the aquifer empirically from selected observation wells to TW.

Dye Tracing

Two dye traces were conducted, with dye injected as a slug into the major water-bearing fracture in two bedrock observation wells known to have hydraulic connection to TW (wells R2 and R3, which are 152 and 596 feet from TW, respectively; Figure 5-12). The dyes chosen, rhodamine-WT and fluorescein, are the most conservative of the available fluorescent dyes for tracing (Mull et al., 1988). They are readily adsorbed onto charcoal samples and have high detectibility. Analysis for these two dyes is reliable and inexpensive.

Two gallons of rhodamine-WT solution (20% solution) were injected into well R2 through a 280-foot garden hose. The 280-foot-length hose was chosen to emplace the dye as a slug at a depth of 275 feet, corresponding to the major water-bearing fracture identified in well R2 (Figure 5-12). The volume of the 280-foot, 5/8-inch-diameter hose is about 4.5 gallons. Thus, the 2 gallons of dye solution were followed by 2 gallons of water rinsed from the dye-solution jugs and 3 gallons of deionized water to completely replace the hose volume. Rhodamine-WT dye emplacement began 4.5 hours after initiation of test pumping, to avoid disturbing water levels during the early period of the aquifer test. The dye injection in well R2 was completed in approximately 12 minutes.



WHERE:

INITIAL TIME	= t_0 (min)
DISTANCE FROM R3 TO LINEAR EXTENDED WELL	= x (ft)
HYDRAULIC DIFFUSIVITY	= D (ft^2/d)
TRANSMISSIVITY	= T (gpd/ft)
STORAGE COEFFICIENT	= S

Figure 5-20. Plot of drawdown versus the square root of time and Jenkins-Prentice (1982) linear-flow model calculations for monitoring well R3, Blackwater Brook site, Dover, New Hampshire.

Fluorescein dye solution (75% solution) was injected into well R3 in a similar manner, using a sufficient length of clean hose to emplace the slug of dye near the main water-bearing fracture at a depth of 357 feet (Figure 5-12). Because this well required a longer length of hose, the hose was flushed with 6 gallons (the approximate hose volume) of water (including the jug rinse) following the 2 gallons of dye solution. Fluorescein dye injection into well R3 began 6 hours after test pumping began and was completed in approximately 13 minutes.

Monitoring for the presence and the level of dye at TW under pumping conditions was accomplished in two ways. Individual water samples were taken at prescribed intervals during pumping. The sampling interval was frequent in the early phase of the test (every 15 to 30 minutes) to detect fast arrivals from the near injection well, R2, and was less frequent later (every 4 hours). Additionally, cumulative samples of the dyes were taken in charcoal samplers for quality assurance purposes. A small amount of discharge water was continuously diverted into a clean, plastic bucket which housed the charcoal samplers. These samplers were changed every 4 hours during pumping. Analysis of the dye samples was performed by the Ozark Underground Laboratory (Protem, Missouri) using a spectrofluorophotometer. A complete description of the sampling and analysis methodology has been provided by Aley (1990).

Rhodamine-WT arrived at TW 130 minutes (2.2 hours) after injection in well R2 and was detected at a concentration of 0.64 ppb (Figure 5-21, top). This indicates a seepage velocity of approximately 1,680 ft/d over the 152-foot distance with a gradient between R2 and TW of 0.11 (along the N30°W well alignment) under pumping conditions of 200 gpm. The rhodamine-WT breakthrough curve resembles a typical dye-trace curve, with a steep rising limb, a relatively sharp peak, and a shallow falling limb (Figure 5-21, top). The jagged character of the curve may reflect the irregular

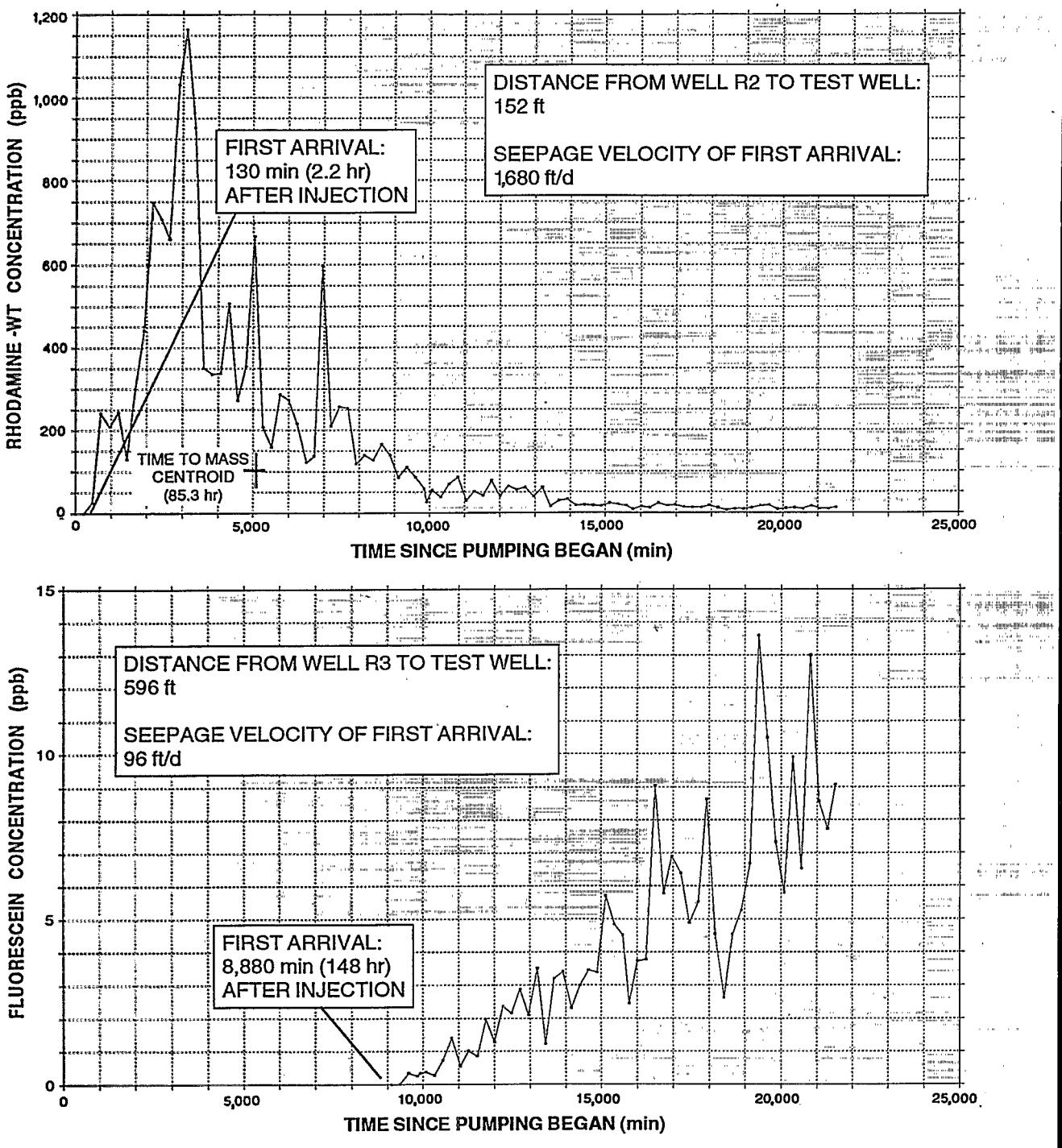


Figure 5-21. Dye-trace breakthrough curves of rhodamine-WT injection (top) at well R2 to the test well, and fluorescein injection (bottom) at well R3 to the test well, Blackwater Brook site, Dover, New Hampshire.

character of the bedrock aquifer; fractures widen and narrow over the length of the flowpath. However, mass balance analysis of the rhodamine-WT curve (Table 5-4) indicates comparatively little dispersion of the dye [0.02 foot squared per second (ft^2/s)] and very direct connection between the boreholes. The statistical analysis of the breakthrough curve given in Table 5-4 was performed using the BASIC code provided by Mull et al. (1988).

TABLE 5-4. STATISTICAL ANALYSIS OF RHODAMINE-WT DYE TRACE FROM WELL R2 TO THE TEST WELL, BLACKWATER BROOK SITE, DOVER, NEW HAMPSHIRE

Distance from well R2 to TW, ft	152
Total dye injected, gr	1,800
Total dye recovered, gr	506
Mean discharge, cfs	0.446
Time of first arrival at TW, hr	2.2
Velocity (from first arrival), ft/d	1,680
Time to centroid of mass, hr	85.3
Time to peak, hr	42.9
Standard deviation of time to peak, hr	64.3
Dispersion coefficient, cfs	0.02
Skewness coefficient	1.41

Abbreviations: TW = test well; ft = foot; ft/d = foot per day; gr = grams; cfs = cubic foot per second; hr = hour.

The fluorescein dye placed in well R3 first arrived at TW approximately 148 hours (6.2 days) after injection, yielding a seepage velocity of 96 ft/d over a distance of 596 feet under pumping conditions of 200 gpm (Figure 5-21, bottom). The gradient in water level at that time between the two wells was 0.07. The fluorescein dye trace shows only the rising limb of the breakthrough curve, which bears little resemblance to the rhodamine-WT curve. Where the rise in rhodamine-WT concentrations was sharp, that of the fluorescein was slow, moving from the detection limit to slightly less than 1.2 ppb over a period of approximately 7 days. No peak was detected, and as a result, statistical analysis of the fluorescein curve was not possible. It appears that the dispersion factor in the N60°E direction is much greater than that of the N30°W direction. This is probably due to the longer flowpath and the lower transmissivity of the aquifer in the N60°E direction. In addition, there may not be direct connection between well R3 and TW. Fluctuations in the fluorescein curve seen in Figure 5-21 (bottom) may indicate arrival of dye from different flowpaths between well R3 and TW.

Water Quality Analysis

Hydrochemical monitoring occurred during several phases during the project. Several months before aquifer testing, water from TW was sampled for general chemistry parameters [cations, anions, coliform bacteria, alkalinity, hardness, total dissolved solids (TDS), sediment, turbidity, color, odor, pH, and specific conductivity] along with pesticides, radon, volatile organic compounds (VOCs), and heavy metals. Hydrochemical analyses served as a screening tool early in the exploration process to

determine acceptable water quality. Later, all wells, several nearby springs, and Blackwater Brook were sampled for basic cations and anions. These analyses were done as part of the ambient-trend monitoring effort to characterize water types.

During aquifer testing, water samples were collected from the discharge line of TW three times during the course of pumping in order to monitor variations in water quality parameters at the beginning, middle, and end of the test. The final sample also served as the state-required sampling for new source approval of a municipal supply well. The first sample was taken 20 minutes after the test began and was analyzed for general chemistry parameters, nitrates, arsenic, and lead. The second sample was collected at approximately the midpoint of pumping (at 9,981 minutes or 6.9 days), and was analyzed for general chemistry parameters and nitrates. For the second sample, arsenic and lead were not analyzed because these constituents were detected at nonsignificant levels in the first sample round. The final sample was collected just prior to shutdown of the pump (at 21,550 minutes or 15 days), and was analyzed for all of the previously mentioned constituents plus VOCs, insecticides, polychlorinated biphenyls (PCBs), and radon.

Grab samples were collected periodically during pumping and field-analyzed for temperature, specific conductivity, pH, chloride, and sodium concentrations to monitor variations in water quality over time. The grab samples were obtained at a higher frequency than the laboratory samples. More frequent sampling was conducted as part of the ambient-trend monitoring effort to establish the chemical character of the water. The instruments used for analysis were a specific conductivity and temperature meter, a chloride titration kit, and sodium and pH pens. These instruments do not guarantee a high degree of accuracy or resolution; however, they do permit frequent testing and rapid turnaround of results. During the first 2 days of testing, these analyses were conducted at 1.5-hour intervals. During the next 13 days, the sampling interval was decreased to approximately every 4 hours to reduce labor and budget requirements.

Analytical results show that the bedrock ground water differs in character from the surface water and ground water in the overburden material. Total dissolved solid concentrations average 268 milligrams per liter (mg/l) in the bedrock water and 113 mg/l in the overburden water. Average sodium (Na) and chloride (Cl) concentrations are much higher in the bedrock water, 86.8 mg/l Na and 107 mg/l Cl, compared with an average of 22.4 mg/l Na and 9.67 mg/l Cl in the overburden water. Iron (Fe) and manganese (Mn) levels are higher than EPA secondary standards, averaging 1.40 mg/l Fe and 0.06 mg/l Mn in bedrock water and 3.91 mg/l Fe and 0.11 mg/l Mn in the overburden and surface water. All wells contain water that is slightly to strongly alkaline (pH = 7.9 to 9.2). Two samples taken showed slightly acidic water, one from Blackwater Brook (pH = 6.5) and one from D1 (pH = 6.7). Specific conductivity is generally lower in overburden water (average of 198 µS/cm) than in bedrock water (average of 445 µS/cm).

No VOCs, pesticides, or heavy metal compounds were detected in any surface- or ground-water samples. No detections for these compounds were anticipated, as few contaminant sources exist to threaten the site. Radon levels in TW were 3,200 picocuries per liter (pCi/l) after the 15-day aquifer test, compared with 2,080 pCi/l following 2 days of pumping several months earlier. The rise in radon levels indicates that significant amounts of water may have been drawn from fractures in the quartz monzonite during the 15-day aquifer test. The EPA has not yet established a maximum limit for radon, but is considering options in the range of 200 to 2,000 pCi/l (U.S. EPA, 1990a). In view of this possible requirement, water treatment for radon may be required at the Blackwater Brook site.

A Piper trilinear diagram (Piper, 1944) displaying the variation of the major inorganic ground-water constituents allows characterization of and distinction between the shallow ground water in the overburden wells and the older, deeper ground water within the bedrock wells (Figure 5-22). Envelopes around the bedrock (dashed line) and overburden (solid line) inorganic constituent data indicate water types of sodium chloride and sodium bicarbonate, respectively. Levels of sodium, chloride, and specific conductivity tend to be higher immediately following drilling of bedrock wells and to decrease within a few days of well completion. In general, levels of these parameters increase from north to south in the bedrock aquifer. The variation in water quality among samples taken from TW when it was first drilled (September 22, 1989), in the middle (July 11, 1990), and at the end (July 26, 1990) of the pumping test shows a trend of declining sodium and chloride and improving water quality. During 15 days of pumping, sodium levels in TW dropped from 116 mg/l to 81.7 mg/l. This evidence indicates that the bedrock system is connected to active recharge from overburden sands and gravels.

Overburden marine clays may contribute sodium to the ground water; sodium would be desorbed by acid precipitation as it slowly infiltrates through the soil. The overburden sand and gravels are quickly flushed of residual salts as water recharges the system. The recharge area of these gravels, above the 190-foot elevation of maximum marine transgression, has fairly thick forest soils that probably contain abundant carbon dioxide-producing bacteria. Consequently, these waters tend to have high concentrations of bicarbonate alkalinity. An exception to the overburden water type is well O3, where the dominant cation is calcium rather than sodium. This well is positioned closest to the deposit of sand and gravel to the north and east. The lower levels of sodium probably reflect more active recharge from water percolating through the sands and gravels and a less significant component of water passing through the sodium-rich marine clay.

REFINED CONCEPTUAL HYDROGEOLOGIC MODEL

In the earliest conceptual model for the Blackwater Brook site, the structural influence of the N5-10°W fracture zone was seen as the most likely conduit for ground-water flow. Based solely on lineament analysis, the N5-10°W feature was protected under the city of Dover's ground-water protection ordinance (Figure 5-5) until further work could be done to refine the model. Later the focus shifted. According to the revised model, the primary flowpath for ground water in the bedrock aquifer coincided with a photolineament, contact zone, and magnetic anomaly, all trending roughly N60°E (Figures 5-3 and 5-4). Analysis of data gathered from imagery, fracture mapping, and surface geophysics, coupled with drilling of a high-yield test well, supported the interpretation that the northeast-trending feature is the significant water-bearing zone.

Subsequent work revealed that the structural and hydrogeologic pictures are much more complex than originally suggested. As opposed to the earlier model of a discrete contact in which two bedrock formations are juxtaposed along a single plane, the N60°E contact is conceptualized as a somewhat irregular, or possibly curvilinear, zone. Along this contact, there is extensive interfingering of the quartz monzonite and the Berwick Formation metasedimentary rocks. Instead of one preferred direction of fracturing, there are two: one trending N60°E and one trending N5-10°W. The Blackwater Brook site lies at the intersection of these two fracture zones. It follows that ground water flows preferentially along both fracture zones with primary flow along the N5-10°W trend.

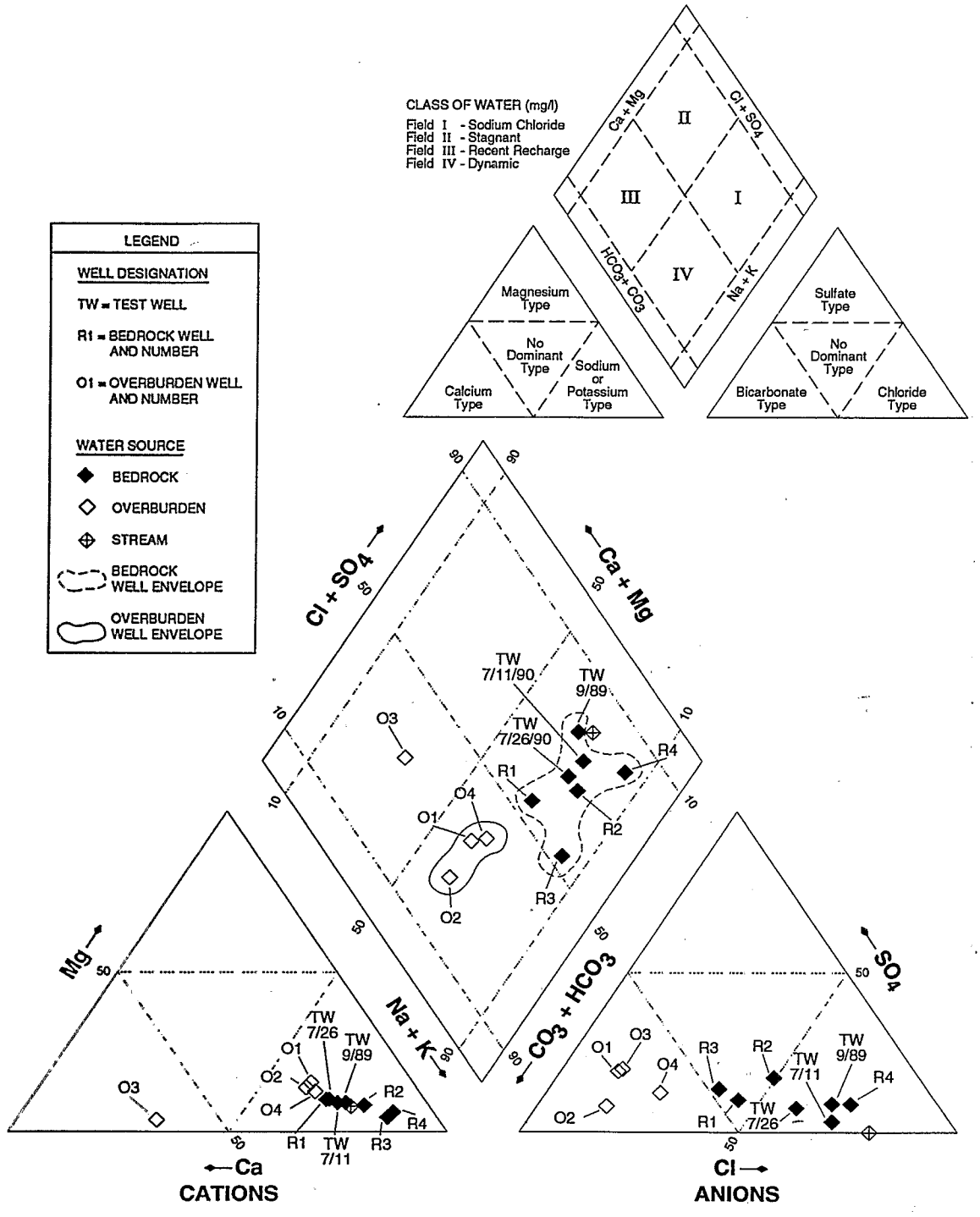


Figure 5-22. Trilinear diagram of ground-water types from overburden and bedrock monitoring wells, Blackwater Brook site, Dover, New Hampshire.

The flow field is anisotropic, and ground-water flow is highly discrete within the bedrock fracture system. Water enters and exits boreholes at distinct intervals rather than uniformly along the length of the borehole. Portions of a borehole may show downward flow, while other sections of the same borehole may have upward flow. While cross correlations of water entry and exit points between boreholes are complicated by the large distances involved, the major direction of flow is N30°W with lesser flow along the N60°E trend. Wells TW, R1, and R2 are contained within the intersection of the N5-10°W and N60°E fracture zones. Wells TW, R3, and R4 lie approximately along the N60°E fracture zone.

Analysis of aquifer-test data yielded values of low to moderate hydraulic conductivity, with the N30°W trend exhibiting higher values than the N60°E trend. The zone of highest transmissivity is in the N30°W direction with moderate transmissivities in the range of 4,000 to 5,000 gpd/ft, while the N60°E trend has transmissivity values of 1,000 to 3,000 gpd/ft. Storage is difficult to quantify for two reasons: (1) the change from confined to unconfined conditions during pumping, and (2) the quantification of storage within a fractured aquifer rather than a porous one. From dye-trace results, the movement of water due to pumping is rapid, with flow velocities greater than 1,600 ft/d along the zone of highest transmissivity (N30°W) and greater than 90 ft/d along the N60°E trend at prevailing hydraulic gradients within the well's zone of influence.

Extrapolation of ground-water flow characterization beyond TW and well R2 is difficult. Lacking discrete flow information beyond TW and well R2, a valid interpretive approach is to assume that statistical fracture descriptions become increasingly good approximations of flowpaths at increasing distances from the site. Therefore, prominent fracture peaks along the N60°E and N5-10°W trends (Figure 5-7) represent the most probable flow directions within the bedrock fracture system at Blackwater Brook (Vernon et al., 1993). The N60°E trend is substantiated by the existence of the faulted, fractured contact zone along this strike. Evidence to suggest preferred flow along the N5-10°W direction is both structural and hydrogeologic. Structural control is inferred by strong expression of the N5-10°W lineament on several platforms of photography and in outcrop fracture trends. Greater aquifer transmissivity in the N30°W direction is demonstrated by analytical calculations, faster dye arrival times, and elongation of drawdown contours in this general direction. Enhanced transmissivity along the N30°W direction is attributed to the proximity and similar orientation of the N5-10°W fracture zone.

The original conceptual model suggested that overburden materials in the vicinity of the well would not be able to provide sufficient recharge to sustain large volume production. Analysis of the drawdown data in the overburden wells, however, indicates that a large percentage, 20%, of the water discharged from TW during pumping is derived from these sediments. Moderate drawdowns in the overburden wells and establishment of steady-state or pseudo steady-state conditions during aquifer testing indicate that appreciable recharge comes directly from overlying sediments. The future production well (near TW) will produce intersecting zones of contribution extending preferentially along the two trends of highest transmissivity. The ZOC for the production well may not be as extensive as previously believed, due to recharge from the overburden sediments.

The primary water-bearing fractures at the Blackwater Brook site are steeply dipping. In general, the deeper the flow system, the longer the time required for water to move through the system in a manner analogous to Toth's (1963) local, intermediate, and deep flow systems. Vertical profiles of FEC and pH measured during the hydrophysical logging procedure indicate that water in the

boreholes is distinctly layered. Generally, FEC increases with depth, but not in a continuous manner. Rather, sharp gradients in FEC and pH occur at discrete depths in the bedrock wells, supporting the conceptual model of isolated, discrete flow zones. The deeper portions of fracture systems impacted by the marine transgression may have higher concentrations of sodium and chloride than the more shallow fractures.

Ground-water quality at the Blackwater Brook site is dependent on the mineralogy of the bedrock and overburden, as well as the history of marine transgression, human activities, and biological processes. Chemical dissolution of minerals within the bedrock by infiltrating water contributes potassium, calcium, magnesium, and sodium to the chemical makeup of the ground water. The large concentrations of sodium and chloride found in several of the bedrock wells may be attributed to connate water within deeper bedrock fractures as a result of Pleistocene marine transgression.

Water quality sampling results (Figure 5-22) showed distinct drops in the levels of sodium and chloride at TW during the 15-day aquifer test. This and other evidence of active recharge from the overburden to the bedrock system suggest that the water quality within the bedrock system will evolve over time in the following manner:

- The dissolved iron and manganese concentrations will increase in the bedrock well water.
- The sodium, chloride, and TDS concentrations will decrease as a result of recharge of the bedrock system by infiltrating overburden water.
- Hardness should decrease because of recharge of the bedrock system by infiltrating overburden water.
- Radon will probably increase slightly, but should stabilize or decrease with well usage. If radon remains at its present level, water treatment may be necessary.

With the exception of iron, manganese, and possibly radon, the bedrock water quality should improve over time as long as the WHPA is enforced. This protection should eliminate, minimize, or regulate potential sources of contamination related to human activity, ensuring maintenance or improvement of water quality.

REFINED WELLHEAD PROTECTION AREA DELINEATION

Existing Wellhead Protection Programs

The New Hampshire State WHPP was approved by the EPA in September 1990. This program defines a WHPA as a wellfield management area that includes all of the contributing area to a well under expected conditions of recharge and well pumping. The criteria for establishing WHPAs are (1) a calculated, fixed radius based on the size of the well for the primary protection zone (Zone I) up to a 400-foot radius; and (2) ground-water flow boundaries for the secondary protection zone (Zone II). The NHDES is the lead agency in wellhead protection for the state and is responsible for implementation of the program.

The NHDES has established an interim protection policy, establishing specified radial distances around wells until final WHPAs can be delineated and the program fully established. Specified radial distances for the interim policy period are 2,500 feet for major community wells, 1,500 feet for small community wells, and 1,000 feet for noncommunity wells. The state program will identify potential contaminant sources, inform owners and operators of potential sources and their location within a WHPA, and use periodic inspections to ensure compliance with best management practice rules. These regulations cover existing supplies of water and treat bedrock and sand-and-gravel wells in a similar manner. Regulations are still under consideration for new source approval. It is likely that overburden and bedrock supply wells will not be approved under precisely the same criteria (P. Currier, NHDES, personal communication, April 1991).

The adjoining New England states also have WHPPs in place. For instance, Massachusetts requires a 400-foot radius for Zone I; a 180-day, no-recharge ZOC for Zone II; and the upgradient contributing area to Zone II as Zone III. The State of Maine proposes to use a 200-day time of travel (TOT) for Zone I, and a 2,500-day TOT boundary for Zone II for unconsolidated aquifers. Zone III in the Maine system is also the contributing area to Zone II. For fractured bedrock aquifers in Maine, Zone I is a 300-foot protective radius, Zone II a 1,000-foot protective radius, and Zone III the remainder of the upgradient watershed.

Delineation of the Wellhead Protection Area for the Blackwater Brook Site

Criteria and Methods for Wellhead Protection Area Delineation--

An approach often used to determine the ZOC in a porous medium involves a numerical flow model calibrated to steady-state and to transient conditions with data from an aquifer test. The calibrated model is then extrapolated to identify the ZOC for an extended time period of pumping. For a fractured bedrock aquifer, the ZOC for a particular time frame is extremely difficult to establish. Specifically, the theoretical difficulties of modeling flow through fractured media using numerical models designed for porous media make such efforts inappropriate in many situations.

The approach used to delineate the WHPA at the Blackwater Brook site employs TOT rather than the ZOC as the primary criterion for delineation. The criteria and threshold levels chosen for the Blackwater Brook WHPA and protection zones (Table 5-5) reflect the efforts of the EPA, the NHDES, and the Maine Department of Environmental Services to establish statewide criteria for bedrock wells, coupled with information gained in this research monitoring project.

The TOT criterion and analytical solution for the delineation utilize the seepage velocity data gathered empirically from dye tracing. The time and distance the dye traveled from the injection wells (R2 and R3) to TW are used to determine the constant in the analytical solution, which varies with direction. An analytical equation was derived (Appendix 5-A) and was used to calculate the distances for the 200-day and 1,000-day TOT thresholds. Corresponding distances for these thresholds were calculated for the two dye trace directions: N30°W (R2 to TW) and N60°E (R3 to TW). The calculated distances were then used to establish protective distances along the two preferred flow directions (two fracture zones): N5-10°W and N60°E. The resultant WHPA and protection zone boundaries were finalized using the hydrogeologic mapping method drawing upon the conceptual flow model and characterization information from aquifer testing, geologic mapping, photolineament and fabric fracture analysis, and surface and borehole geophysical surveys (Vernon et al., 1993).

TABLE 5-5. WELLHEAD PROTECTION ZONES AT THE BLACKWATER BROOK SITE, DOVER, NEW HAMPSHIRE

Zone	Description	Criterion	Purpose
I	Area encompassed by a 400-foot radius around the well	Distance	State-mandated sanitary protection area
IIA	Area covered by the 200-day TOT limit along directions of highest bedrock transmissivity	TOT and physical features for protection against pathogens in ground water	Protection against pathogens in ground water
IIB	Area between the directions of highest transmissivity extending to the 200-day TOT limit	Same as IIA	Same as IIA
III	Area contributing to the 1,000-day TOT contour	TOT and physical features to ensure adequate time for remediation	Ensure adequate time for remediation

Abbreviation: TOT = time of travel.

The theoretical approach to determine the TOT distances for the protection zones utilizes a quadratic relationship in which the time of first arrival of the dye, t, is proportional to the square of the distance between the injection well, R2 or R3, and the test well, ι (Appendix 5-A):

$$\iota = Cxt^{\frac{1}{2}}$$

This relationship is derived from Darcy's Law (Equation 1, Appendix 5-A) and the Thiem Equation (Equation 2, Appendix 5-A). It represents a simple analytical expression to interpret the dye-trace and aquifer test information. The constant, C, varies with direction.

The value for C in the N30°W direction was determined from the empirical results of the dye tracing at well R2, where t=130 min (0.09 d) and ι=152 ft (Figure 5-21). Solving for C, a value of 506 feet per day$^{1/2}$ (ft/d$^{1/2}$) results. The equation for the TOT distance in the N60°E direction is determined in a similar manner, where t=148 hr (6.2 d) and ι=596 ft (R3 to TW), resulting in a value for C of 240 ft/d$^{1/2}$. Calculated distances corresponding to the 200-day and 1,000-day TOT thresholds for the N30°W and N60°W directions are given in Table 5-6.

TABLE 5-6. TIME-OF-TRAVEL DISTANCES ALONG DYE-TRACE DIRECTIONS, BLACKWATER BROOK SITE, DOVER, NEW HAMPSHIRE

TOT Criteria (d)	Calculated Distances (ft) N30°W (R2 to TW)	N60°E (R3 to TW)
200	7,200	3,400
1,000	16,000	7,600

Abbreviations: TOT = time of travel; TW=test well.

Delineation of Zones I, II, and III--

Protection zones within the WHPA (Zones I, II, and III) were established by incorporating the State of New Hampshire delineation criteria, as well as factors inherent to the conceptual flow model (Figure 5-23). The TOT distances for the 200-day and 1,000-day thresholds along the two dye-trace directions, N30°W and N60°E (Table 5-6), were applied to the two principal flow directions, N5-10°W and N60°E, respectively. (A complete discussion of the selection of the two principal flow directions is given in the Refined Hydrogeologic Conceptual Model section of Chapter 5.) The TOT distances form the basis for the Zone II and III delineations. The final Zone III boundary was determined using the hydrogeologic mapping method.

Zone I--The method of delineation for Zone I is a simple matter of drawing a circle on the map. The State of New Hampshire proposes a calculated fixed radius for the sanitary protection area based on well production rate. However, 400 feet is the largest radius used.

Zones IIA and IIB--Zone IIA is the area within the 200-day TOT limit along the directions of highest bedrock transmissivity: N5-10°W and N60°E (Figure 5-23). The two "arms" of Zone IIA are 1,000 feet wide and radiate from TW, or the future production well. The choice of a 1,000-foot width was based on facts unique to this site. The N5-10°W lineament passes between two bedrock ridges approximately 3,000 feet northwest of TW. The distance between the bedrock ridges is approximately 1,000 feet.

The domestic well, D1, located east of the ridges (Figure 5-10), shows no response to pumping during testing (Figure 5-13), so it may be assumed that the ridges represent relatively unfractured bedrock on either side of the N5-10°W fracture zone. From lithologic and borehole geophysical logging results, the faulted contact along the N60°E trend is actually a zone of interfingering rock types and numerous fracture sets at least 300 feet wide (the distance between wells R1 and R2; Figure 5-10). Considering the 1,000-foot distance between ridge tops along the N5-10° W trend, the larger of the two widths (1,000 versus 300 feet) was selected for the width of the 200-day TOT protection zone in both directions. Zone IIA is subject to the zoning requirements of the Dover Municipal Aquifer Protection Ordinance. This ordinance takes a somewhat more restrictive approach to aquifer protection than do the proposed New Hampshire State regulations for a secondary protection zone.

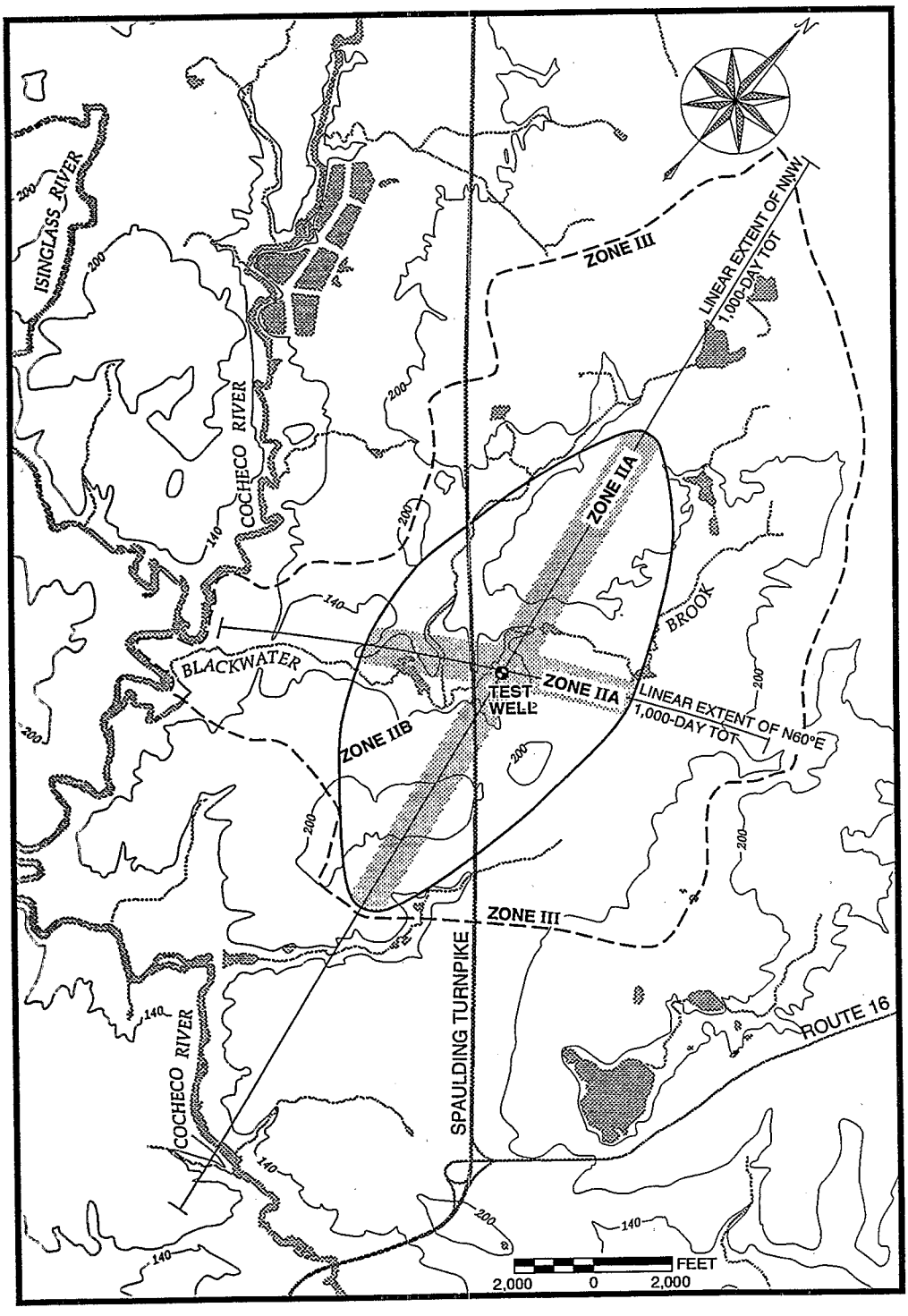


Figure 5-23. Wellhead protection area and zones, Blackwater Brook site, Dover, New Hampshire. (Contour interval is 60 feet.)

The calculated TOT distance values determined from the derived equation and the dye-trace results used in determining the secondary protection zone should not be construed as absolute. Bedrock aquifers are extremely complex. The addition of a surrounding zone of protection reflects a healthy respect for the intricacies of the system and attempts to account for some of the limitations of this approach. Consequently, Zone IIB was introduced (Figure 5-23) as a smooth curve connecting the "arms" of the Zone IIA. By incorporating the oval as another zone of protection, consideration is given for other possible directions of flow within the bedrock system which may not yet be resolved, as well as for inhomogeneities along the directions of higher transmissivity already identified, the N5-10°W and N60°E trends. This site zone would be regulated by the secondary protection regulations proposed by the State of New Hampshire.

Implementation of the 200-day TOT criterion represents a conservative approach to wellhead protection. Both TOT calculations and dye tracing were employed to predict contaminant transport directly through the bedrock aquifer. As an example of this scenario, at a rock outcrop, a spill or illegally dumped substance may penetrate quickly into the bedrock aquifer. In other situations, however, some time would elapse before a contaminant released at the surface passed through the overburden into the bedrock aquifer.

Zone III--Zone III is defined as the area contributing to the 1,000-day TOT distances, calculated to be approximately 16,000 and 7,600 feet (Table 5-6) in the two principal directions of flow, N5-10°W and N60°E. The contributing area was mapped using surface drainage divides corresponding closely to the TOT distances (Figure 5-23). The downgradient limit of Zone III was placed at the southern end of Zone IIB rather than extending the protection zone further. This step was taken to account for the ground-water gradient and the stagnation point downgradient from TW. Extending the protection zone an additional 8,400 feet downgradient is not realistic hydrogeologically, and would overprotect the resource. Protection strategies within Zone III should focus on public education and awareness for the production well at Blackwater Brook.

WELLHEAD PROTECTION MONITORING PROGRAM

MONITORING OBJECTIVES

Once the assessment of contaminant sources has been made, flowpaths have been adequately defined, and WHPAs have been delineated (Figure 5-24), monitoring in the traditional sense of the term takes place. The monitoring program can be divided into two primary facets: physical monitoring and hydrochemical monitoring.

Monitoring of physical parameters such as water levels, flow volume, and temperature play a continuing role in the overall monitoring strategy in several ways. A prime objective of the Dover ground-water monitoring program is to discern the long-term impact of water production on water levels within the fractured bedrock and the overburden aquifers. At the Blackwater Brook site, the surficial materials have moderate storage and transmissivity capabilities because of the presence of marine clays and silts. The ability of the N60°E faulted contact zone and the intersecting N5-10°W fracture zone to draw water from outside the immediate drainage basin to TW remains unknown, but can be assessed by physical monitoring. Once a production well is brought on line, steady-state ground-water flow conditions may no longer prevail because of the varying pumping rates. Variation in pumping rates may expand the ZOC to include previously excluded contaminant threats. A physical monitoring program could provide the data needed to predict expansion of the ZOC and possibly indicate a need for increased hydrochemical monitoring near a potential contaminant source.

Physical monitoring results may justify hydrochemical monitoring objectives and parameters. For example, techniques such as flowmeter and hydrophysical logging, used to develop a conceptual model for delineation, indicate that ground-water flow at the Blackwater Brook site is three-dimensional. Water flows up or down in boreholes, following convoluted pathways as it moves through bedrock fractures. Observation of vertical flow, as well as FEC and pH stratification, indicates that vertical averaging of physical and chemical parameters in a monitoring well will not adequately describe either the ambient trends or the characteristics of a possible contaminant plume. If a contaminant plume is small, or if a plume enters the aquifer with a vertical flow component, the monitoring network should be optimally designed to detect the contaminant concentration and its associated flowpath. These objectives may be compromised if vertical averaging in the monitoring well occurs.

Hydrochemical monitoring serves to determine ambient water quality trends, identify sources of contamination, and provide early warning detection of pollutants. Hydrochemical monitoring focuses management, remediation, and prevention efforts in municipal and adjacent water districts. Location strategies for hydrochemical monitoring that apply to the Blackwater Brook site include monitoring at the wellhead, at WHPA or zone boundaries, and at point sources.

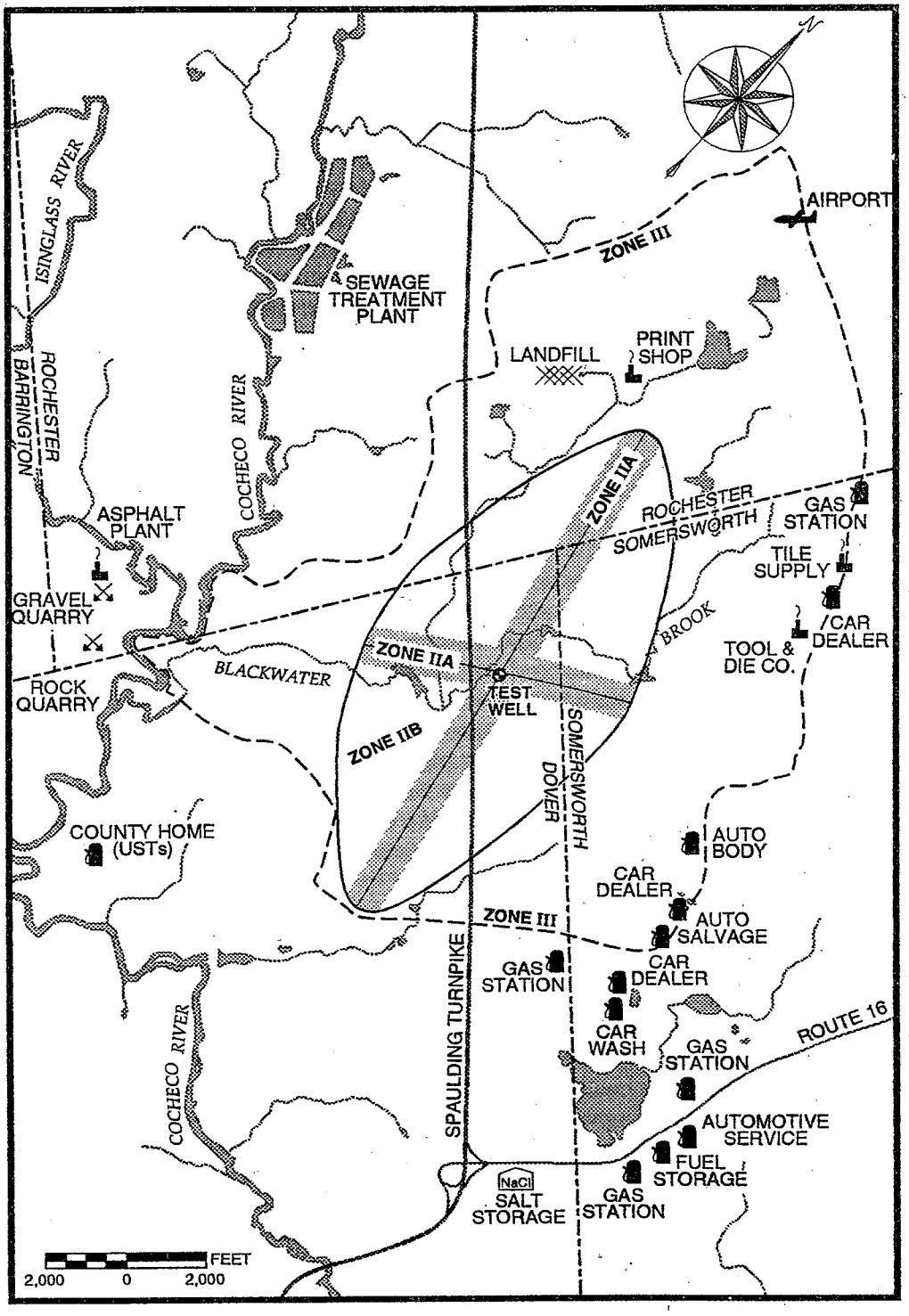


Figure 5-24. Potential contaminant sources in proximity to wellhead protection zones, Blackwater Brook site, Dover, New Hampshire.

At Blackwater Brook, the objective of early warning monitoring is difficult to achieve because of the high average flow velocities within the fracture system. Despite strong evidence that the primary ground-water flow directions have been identified and can be monitored, early warning detection of unanticipated contamination with a monitoring well system is extremely difficult, given the rapid and highly discrete character of water movement in the bedrock aquifer. The focus of hydrochemical monitoring in this hydrogeologic environment is on source assessment and ambient conditions along with contingency planning to achieve the overall objective of ground-water protection. To address these concerns, the following strategy for monitoring has been designed, dealing specifically with monitoring sites, parameters, frequencies, quality assurance and quality control (QA/QC) considerations, and the data base.

MONITORING SITES

Existing sites

All bedrock and overburden wells installed in the research monitoring program will serve in a physical and a hydrochemical monitoring program at the site (Figure 5-25). Sampling parameters and frequencies for these wells are discussed in the subsection titled "Monitoring Parameters and Frequencies."

Recommended Sites

Additional wells for physical monitoring should, first, be placed at a greater distance from TW than the existing wells. Optimally, new wells should be completed in bedrock along the N60°E lineament and along the N5-10°W trend as much as a mile from TW to better characterize the fracture system and to provide distant monitoring points for water levels. At a minimum, two recommended bedrock well sites are R5 and R6 (Figure 5-25). Of secondary, but significant, importance is locating some new monitoring wells in areas where data gaps exist: that is, in other directions from TW than along the N5-10°W and N60°E trends. Additional wells placed in other directions could be used to verify water levels, drawdown, and the TOT contours. Domestic bedrock wells such as D1 (Figure 5-25), or others drilled in the future, should be used for physical monitoring.

New overburden wells should be placed in areas known or suspected to be sources of direct recharge to the bedrock system. Marine clay in the overburden thins from 32 feet thick at TW to only 6 feet in well O3. The clay deposit thins eastward and pinches out. Marine sand deposits northeast, east, and uphill of the site constitute a major recharge area for TW. The estimated extent of marine clay in the vicinity of the site is shown in Figure 5-25. Consequently, the hillslope and hilltop northeast and east of the site are appropriate locations for one or more overburden wells for physical monitoring (for example, well O8 as shown on Figure 5-25).

The site is reasonably well monitored for hydrochemical parameters by the existing four bedrock and four overburden wells (Figure 5-25). However, the location of another overburden well, well O5, nested with well R5 just east of the Spaulding Turnpike would improve source assessment monitoring (Figure 5-25). Wells O5 and R5 would serve to monitor unanticipated spills along the Spaulding Turnpike.

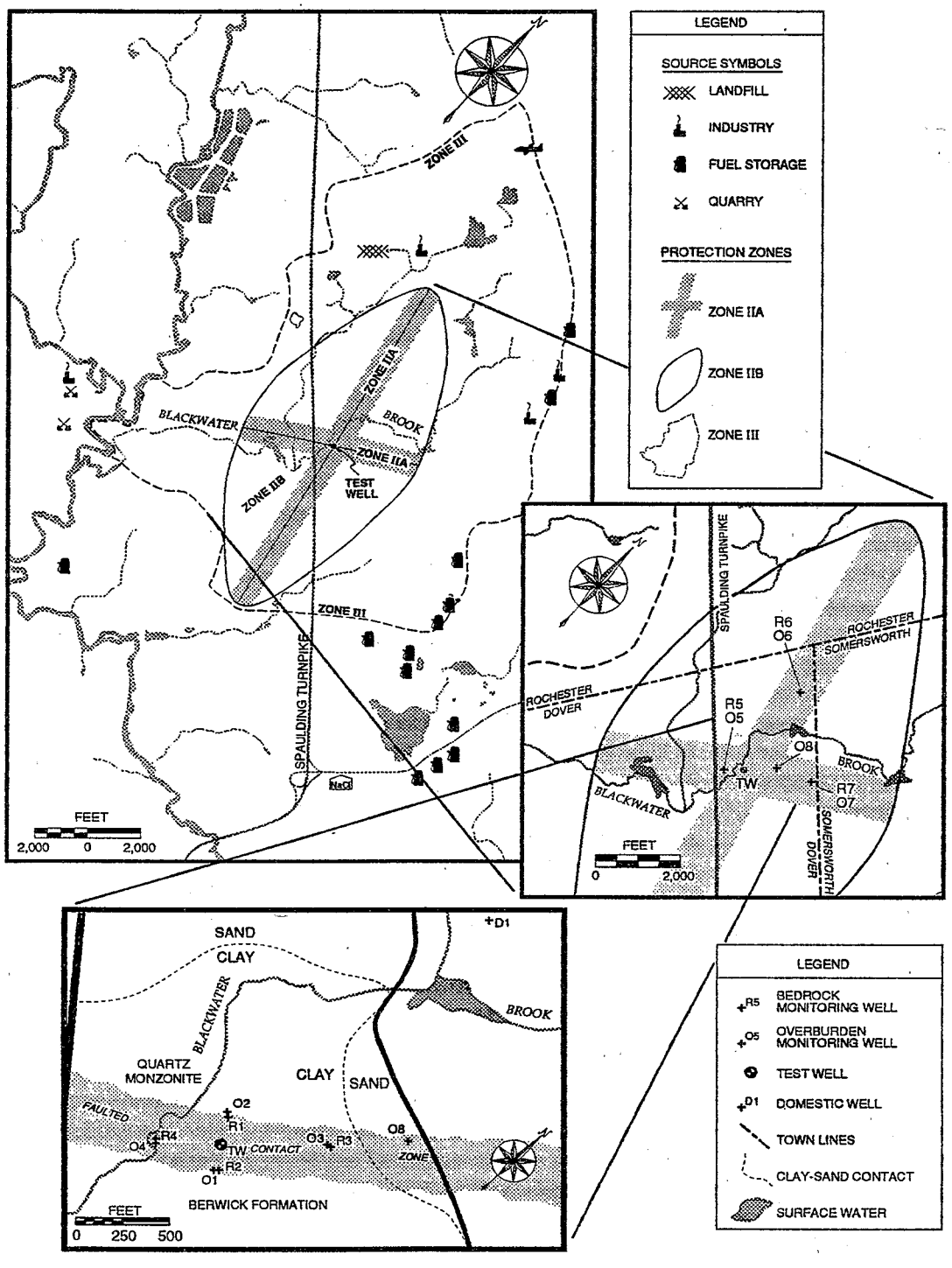


Figure 5-25. Existing and proposed monitoring sites, Blackwater Brook site, Dover, New Hampshire.

To monitor the threat from septic systems and domestic storage tanks, single monitoring wells should be installed in the overburden directly downgradient (approximately 300 feet) from the homes and upgradient of the future production well. One or more overburden wells on the hillslope or hilltop northeast and east of the site (for example, well O8 shown in Figure 5-25) would be very useful. Permeable marine sand deposits in these areas provide significant recharge to the system. Such overburden wells would be necessary to monitor septic systems if the area is residentially developed.

A landfill 3 miles northwest of the site lies within Zone III (Figures 5-24 and 5-25). It may be prudent to monitor this landfill as a potential contamination source as a secondary objective of the program. To monitor a potential point source, one monitoring well should be placed upgradient of the source and two or more monitoring wells downgradient from the source. The upgradient well should be located and sampled to obtain background levels for potential contaminants from the source (U.S EPA, 1986).

In many circumstances, WHPAs cross political boundaries. Optimally, monitoring and compliance with protection regulations can be coordinated and implemented by adjoining towns. In some cases it may be necessary to monitor the political boundaries. To ensure wellhead protection, monitoring wells should be placed where Zone II, and possibly Zone III, are truncated by town boundaries. Zone II is truncated by the town of Somersworth and the city of Rochester boundaries along the N5-10°W trend and by the town of Somersworth boundary along the N60°E trend (Figure 5-25). If Zone II is truncated politically, at least one overburden and one bedrock monitoring well should be located adjacent to the city limits at about the center of each arm of Zone IIA. Wells O6 and R6 should be positioned at the center of the N5-10°W trend, and wells O7 and R7 at the center of the N60°E trend (Figure 5-25). New wells along the N5-10°W trend should be located as close as possible to the intersection of a line drawn from TW to the city limits. Along the N60°E trend, it is recommended that surface geophysical surveys be used to locate new monitoring wells in the same manner that TW was sited.

MONITORING PARAMETERS AND FREQUENCIES

A production well has not yet been established at the Blackwater Brook site. Only the test well and the monitoring wells installed as part of the research monitoring program are currently in place. Accordingly, two monitoring programs, consisting of detailed parameters and frequencies, have been developed to reflect the current scenario and to reflect conditions of an on-line production well. Table 5-7 presents a summary of monitoring parameters and frequencies recommended for existing conditions. Recommendations for monitoring a water-supply production system at the site are given in Table 5-8.

Recommended physical monitoring parameters (parameters that bear physical units of measure) include water level, temperature, and specific conductivity. A chemical parameter often categorized with these physical parameters is pH. Together, these four parameters constitute a field-screening procedure for water quality changes. Automated instrumentation and data recorders should be considered for acquiring physical and chemical parameter data, where possible. The initial high cost of field-dedicated equipment is usually offset by the increase in frequency and quality of the data, and the rapid turnaround of results.

TABLE 5-7. SAMPLING PARAMETERS AND FREQUENCIES FOR THE TEST WELL AND ASSOCIATED MONITORING WELLS, BLACKWATER BROOK SITE, DOVER, NEW HAMPSHIRE

Well(s)	Parameters	Frequency
Test well	Water level Temperature Specific conductivity pH	Monthly
	Cation and anion suite Bacteria	Biannually
	VOC scan Heavy metals	Annually
Overburden wells	Water level Temperature Specific conductivity pH	Monthly
	Cation and anion suite Bacteria	Biannually
	VOC scan Heavy metals	Annually
Bedrock wells	Water level Temperature Specific conductivity pH	Monthly
	Cation and anion suite Bacteria	Biannually
	VOC scan Heavy metals Radon Uranium	Annually

Abbreviation: VOC = volatile organic compound.

TABLE 5-8. SAMPLING PARAMETERS AND FREQUENCIES FOR THE PROPOSED PRODUCTION WELL AND ASSOCIATED MONITORING WELLS, BLACKWATER BROOK SITE, DOVER, NEW HAMPSHIRE

Well	Parameters	Frequency
Production well	Water level	Daily*
	Applicable state, local, and SDWA requirements Iron Manganese	Weekly
Overburden wells	Water level Temperature Specific conductivity pH	Monthly
	Sodium Chloride Cation and anion suite Bacteria	Biannually
	VOC scan Heavy metals	Annually
Bedrock wells	Water level Temperature pH Specific conductivity	Monthly
	Sodium Chloride Iron Manganese Radon Uranium Cation and anion suite Bacteria	Biannually
	VOC scan Heavy metals	Annually

Abbreviations: SDWA = Safe Drinking Water Act; VOC = volatile organic compound.
* The city of Dover conducts daily water-level monitoring of all production wells.

Monitoring of the test well should include monthly water-level measurements to establish ambient ground-water conditions (Table 5-7). Water levels for all on-line production wells in the city system are monitored every day by the Dover Department of Public Works. The production well at Blackwater Brook should also be measured daily (Table 5-8).

Temperature, specific conductivity, and pH should be monitored monthly at the test well, at bedrock and overburden wells used in the research phase, and at any new wells installed for long-term monitoring (Table 5-7). After the production well is in service, the bedrock and overburden monitoring wells should continue to be monitored monthly for water level, pH, temperature, and specific conductivity.

Under present conditions, existing monitoring locations and the test well should be examined biannually for general chemical character, including cations, anions, and bacteria, to establish the natural hydrochemistry of the area. Overburden and bedrock wells should undergo annual scanning for total VOCs and for heavy metals. Additionally, bedrock wells and the test well should be sampled annually for radon and uranium.

Once the production well is on line, hydrochemical monitoring of the production well will be dictated by applicable state and local stipulations, as well as the SDWA requirements (Table 5-8). Because of the documented high levels of iron and manganese at the test well, it may be prudent to sample the production well weekly for these elements. Overburden monitoring wells should be sampled biannually for sodium and chloride, while bedrock wells should be sampled for sodium, chloride, iron, manganese, radon, and uranium (Table 5-8). Monitoring for sodium and chloride is important for two reasons: (1) to determine water quality degradation from road salting in both aquifers, and (2) to track the movement of the deeper bedrock flow regime, which has high levels of chlorine from recharge water induced by pumping. Results from the gamma borehole geophysical logging indicate that uranium is present in well R4. While the threat of water quality degradation from uranium is considered to be minimal, cautious monitoring is recommended. Radon levels are elevated, and treatment may be considered when the production well is developed. Spatial and temporal trends in radon levels may provide important information in determining future water treatment requirements. All monitoring wells should be sampled biannually for cations, anions, and bacteria and should be scanned annually for VOCs and heavy metals (Table 5-8).

QUALITY ASSURANCE/QUALITY CONTROL CONSIDERATIONS

Samples taken during monitoring are subject to acceptable QA/QC protocol. City personnel will receive specific training in sampling and monitoring methodologies. Some analysis can be done in the field. Field kits can be used to measure chemical parameters such as chloride, sodium, iron, and manganese in the monitoring wells. When the production well is brought on line, sampling and subsequent laboratory analysis for SDWA parameters will occur. To complement this regulatory monitoring program, laboratory analysis of select chemical parameters from the monitoring wells is recommended. In this way, field analysis of chemical parameters can be verified and improved.

Particular attention must be paid to proper ground-water sampling methodology, which includes purging of monitoring wells, filtering of samples, proper calibration of instruments, and collection of duplicate samples (Garrett, 1988). The duplicate sample should be refrigerated and kept

as backup until successful analysis of the first sample is completed. At least one full water quality sample from the production well should be analyzed by an EPA-certified laboratory each year. Laboratory results should be reviewed by performing a mass balance calculation to compare total milliequivalents per liter of cations and anions. Agreement should be within 10%. Chain-of-custody for sample preparation and tracking will be the responsibility of the city of Dover Department of Public Works.

MONITORING DATA BASE

Collection of data is not an end in itself. To be of use, the data should be compiled and stored in an efficient and meaningful manner for later analysis. A great deal of information has been and will continue to be generated by the research and long-term monitoring programs, including an extensive data base on well installation and completion, aquifer testing, physical parameters, and hydrochemical parameters. To store, manipulate, retrieve, and output such information easily requires the use of a microcomputer with spreadsheet and interactive graphics or interpretive software. The field data from the long-term monitoring conducted by the Department of Public Works personnel should be entered into a spreadsheet data base on the day the data are collected. Hydrochemical data received from the laboratory should also be entered into the spreadsheet to facilitate analysis, with the original documentation stored in files. Review of these data should be undertaken periodically by Public Works personnel to identify spatial and temporal trends, to pinpoint possible water quantity or quality problems that can be discerned from the information, and to perform necessary QA/QC checks. This may be accomplished in association with consultants.

CONTINGENCY PLANNING

Because ground-water flow travel times are so rapid and the flowpaths are so highly discrete within the fractured bedrock aquifer, installment of an early-warning detection network of monitoring wells is probably inappropriate. Based on the dye-trace results, ground water is predicted to travel roughly 16,000 feet and 7,600 feet in 1,000 days along the N5-10°W and N60°E directions, respectively. A WHPA of this size extends into adjoining towns (Figure 5-25). Therefore, it may be politically and legally quite difficult to protect a bedrock water supply well with Dover's present zoning and regulatory structure. Therefore, beyond identifying and monitoring known sources of potential contamination, the optimal approach to protecting the production well from unanticipated contamination is through public education and contingency planning. Contaminants may threaten the wellhead from a major accident along the Spaulding Turnpike, from illegal dumping of toxic or hazardous materials along a roadside into permeable sands and gravels that recharge the well, or from an unknown "preregulation" source.

Monitoring Action Levels

Action levels are needed to determine when to put a contingency plan into effect. The contingency plan for the Blackwater Brook site contains a three-tiered system of action levels for water quality parameters based on the Maximum Contaminant Level Goals (MCLGs) set by EPA (U.S. EPA, 1990b and 1990c), along with the SDWA standards. The focus of the action levels is sampling results from the monitoring wells, because SDWA requirements regulate the production well.

- Level A: Samples show secondary standard MCLGs exceeded. If a secondary standard MCLG is exceeded at a monitoring well anywhere in the WHPA, sampling frequencies increase and field or in-house analyses are supplemented by mandatory analyses at certified laboratories that can provide court-admissible data.

- Level B: Samples show primary standard MCLGs exceeded. If a primary standard MCLG is exceeded in a monitoring well anywhere in the WHPA, a contamination event is suspected and a site investigation and monitoring strategy may be initiated, possibly with drilling of new boreholes for monitoring.

- Level C: Samples show SDWA standards exceeded. If SDWA standards are exceeded in a monitoring well anywhere in the WHPA or if the Level B action level is reached at the pumping well, the production well is immediately taken off line.

Thus, a Level A situation occurs when a water sample from a monitoring well shows values greater than the EPA MCLG for a secondary drinking water standard. Two actions are then initiated: monitoring frequency increases appropriately (for example, from yearly to quarterly), and analysis by a certified laboratory is initiated in conjunction with field or in-house sampling. If an MCLG is exceeded for a primary drinking water standard in a monitoring well (Level B), a contamination event such as a spill or UST leak is suspected. A site investigation and a new, intensified monitoring strategy are initiated (Pfannkuch, 1982). If Level C action occurs at a monitoring well, or if Level B action occurs at the production well, the introduction of contaminants near the production well or within the WHPA is assumed. Level C action, in which the SDWA standards at any monitoring well are exceeded, will trigger immediate shutdown of the production well until the problem is understood and remedied. Any level of violation may necessitate review and selection of additional or different monitoring parameters tailored to the character of the contamination incident.

Contaminant Incident Response

Contingency planning begins with assignment of responsibility for specified actions in the event of an emergency. If a spill or major accident with the chance of pollution takes place, police will notify the Director of Public Works regarding the time, place, and severity of the accident, giving a preliminary assessment of the character of the spilled material. The Director will, in turn, order immediate shutdown of the production well. Public Works personnel will begin water quality sampling at monitoring wells that are between the spill and the production well. These samples will then be analyzed for VOCs, both in-house (if a gas chromatograph is available) and through a certified laboratory. The rationale to screen for aquifer contamination with VOC analysis is based on the unique source assessment and prioritization that is conducted in proximity to the test well. Monitoring should proceed, with frequent samples taken in the early phases, perhaps every hour or 2 hours for the first 24 hours following the incident. The frequency of continued sampling should be based on the presence or absence of VOCs in the wells.

Unanticipated contamination events may require rapid emplacement of a source-assessment monitoring program to determine the contaminant(s), extent of contamination, and effect on the ground water. Such a monitoring program would include the following steps (Pfannkuch, 1982):

- Rapid preliminary survey
- Phased drilling of observation boreholes
- Phased monitoring of boreholes.

For the rapid preliminary survey, topographic maps would be used and a site visit conducted for qualitative assessment of local surface drainage, topography (probable water-table configuration), water levels in nearby wells, soil characteristics, and background levels of the contaminant. Representative water samples for these assessments would be taken from upgradient or unaffected wells.

Borehole testing and monitoring well installation would then begin. The first phase of borehole drilling involves placing a minimum of three boreholes in a triangular pattern around the spill site or contaminant source. Initial monitoring of these boreholes should provide data on ambient levels of the contaminant (from the upgradient borehole) and the transport direction of the contaminant. Monitoring results would aid in determining locations for drilling the next boreholes: drilling downstream on the flowline, with triangulation determined from information obtained from the first wells. Transport velocities in the downgradient direction can be obtained at this phase of the assessment (Pfannkuch, 1982). Thus, initial monitoring and drilling proceed in an iterative manner, with monitoring results determining the decisions for drilling locations and whether or not to construct a permanent monitoring well at any specific drill hole. Also, the drilling and initial monitoring allow refinement of the conceptual hydrogeologic model, which in turn allows (1) design of the main monitoring program to track the evolution of the contaminant plume, (2) possible numerical simulation of contaminant flow and transport, and (3) establishment of a clean-up plan.

Long-term and final monitoring for certain wells at selected and increasing time intervals should track the rehabilitation and recovery (or lack thereof) of the aquifer (Pfannkuch, 1982). The contaminated well should remain out of use until the surface spill is cleaned up and information from exploratory wells or test pits is analyzed to determine the location and movement of contaminants. Water supplies from other city wells will be used until the site is pronounced safe by city officials.

CONCLUSIONS

Fractured bedrock aquifers pose unique challenges to the tasks of delineating and monitoring wellhead protection areas (WHPAs). Some aquifers that possess sufficient secondary porosity and permeability warrant conceptualization as equivalent porous media, and a number of analytical and numerical models are applicable to simulate ground-water flow in these systems. Frequently, igneous and metamorphic bedrock aquifers do not exhibit hydraulic behavior consistent with that of a porous medium. For these types of aquifers, other approaches must be sought to characterize the physical flow system.

This document offers an approach to delineating wellhead protection zones and establishing monitoring strategies for a fractured bedrock aquifer in Dover, New Hampshire. A combination of different geologic and hydrogeologic tools was used in the process of developing, testing, and refining the conceptual model for the Blackwater Brook well site in Dover. The characterization approach taken at the Blackwater Brook site may be used elsewhere, with modification, to reflect different geologic environments in other fractured bedrock aquifers.

The aquifer at the Blackwater Brook site is confined by up to 30 feet of marine clay in the overburden and consists of both unconsolidated overburden and fractured crystalline bedrock. Permeable sands and gravels, above the bedrock but beneath the clay, provide recharge to the fractured bedrock aquifer. Two directions of relatively high bedrock transmissivity exist at the Blackwater Brook site: one trending north 60 degrees east (N60°E) and one trending north 5 to 10 degrees west (N5-10°W). The N60°E trend is a fractured, faulted contact zone between the quartz monzonite and the Berwick Formation metasedimentary rocks. This zone is expressed as a photolineament and as a magnetic anomaly. The intersecting N5-10°W fracture zone is expressed with strong signature on several platforms of aerial photography and is present in an outcrop area north of the site.

Aquifer testing indicates that the direction of highest transmissivity within the bedrock is north 30 degrees west (N30°W) along wells aligned perpendicular to the N60°E trend. A seepage velocity of 1,680 feet per day (ft/d) was demonstrated with dye-trace results in this direction. Drawdown contours are elongate in this direction during pumping of the test well, indicating preferred flow along N30°W. Enhanced flow along this direction is attributed to the proximity and similar orientation of the N5-10°W fracture zone. The N5-10°W fracture zone is believed to act as a conduit for flow along the trend, but may be a partial barrier boundary to flow moving east across the trend. Dye-trace results indicate a seepage velocity of 96 ft/d along the N60°E trend.

The hydrochemistry at the Blackwater Brook site is characterized by stratification of water types; pH and fluid electrical conductivity increase at abrupt intervals with depth. These characteristics reflect the presence of discrete flow zones at depth within the bedrock aquifer. The ground water at the site is free of contamination from bacteria, volatile organic compounds (VOCs),

pesticides, and heavy metals. The majority of water quality parameter levels are within state and federal standards, with only a few exceptions. Iron and manganese levels exceed secondary standards in both the bedrock and overburden aquifers due to naturally occurring sources in the subsurface. Additionally, elevated levels of sodium and chloride exist in the bedrock aquifer, probably from trapped sea water. These levels are expected to drop as the production well is developed and used. The only serious contaminant threat is naturally occurring radon, which may require treatment. The potential for contamination at the site is low, but future threats include spills along the Spaulding Turnpike, highway de-icing, illegal dumping, and domestic septic system leachate.

Time-of-travel (TOT) contours for the delineation process were obtained, in part, by analysis of aquifer-test results, including test pumping and dye tracing. A quadratic equation was derived relating the time of first arrival of the dye at the pumping well, t, to the distance from the injection well to the pumping well, ι:

$$\iota = Ct^{\frac{1}{2}}$$

the constant , C, in the equation was determined from dye-trace results along the two directions of higher transmissivity in the bedrock aquifer, N5-10°W and N60°E. The constant, C, varies with direction.

The 200-day and 1,000-day TOT thresholds applied at the Blackwater Brook site represent a conservative approach to WHPA delineation in this fractured bedrock aquifer. Time-of-travel calculations assume that a contaminant is already in the bedrock aquifer system, as was the injected dye. At the bedrock outcrop, a spill or illegally-dispersed substance could penetrate very quickly into the bedrock aquifer. In another scenario, however, a contaminant released through the overburden could take significant time to reach the bedrock aquifer.

The criterion for delineating the Zone I protection area is the state-mandated 400-foot protective radius. The area encompassed by the 200-day TOT contour along the direction of highest bedrock transmissivity is defined as Zone IIA. This zone will be protected under the City of Dover Ground Water Protection Ordinance. Zone IIB was delineated to compensate for the technical limitations and difficulties of precise definition of Zone IIA. Zone IIB is the area defined by a smooth curve connecting the outer boundaries of Zone IIA. This zone will be protected under the State of New Hampshire Wellhead Protection Program regulations, which are somewhat less restrictive than the City of Dover Ordinance. Zone III is the upgradient contributing area to Zones IIA and IIB and the 1,000-day TOT distance measured along the zones of highest transmissivity, using surface drainage contribution areas to delineate the zone. The downgradient limit of Zone III was placed at the southern end of Zone IIB, rather than extending the zone to the nearest surface-water divide. This step was taken to account for the ground-water gradient (toward the southwest) and the stagnation point downgradient from the test well.

Recommendations for long-term monitoring of the WHPA include locations for new monitoring wells, physical and hydrochemical parameters, and sampling frequencies. All existing overburden and bedrock wells installed as part of the research monitoring program will serve as long-term monitoring stations. Three pairs of overburden and bedrock monitoring wells should be

installed at a greater distance from the test well, each coinciding with an identified subsurface fracture zone. These well pairs will serve (1) to better characterize the fracture system, (2) to provide more distant monitoring points for water levels and select hydrochemical parameters, and (3) to monitor upgradient activities in bordering towns. One new overburden well is recommended for installation downgradient from a residential area to monitor septic systems and domestic storage tanks.

Two programs, with detailed sampling parameters and frequencies, are recommended: (1) to monitor the current test well and associated observation wells, and (2) to monitor the future production well and associated observation wells. Monitoring of the test well and existing observation wells focuses on gathering ambient-water quality and physical parameter data. Hydrochemical parameters of interest include the cation and anion suite, pH, bacteria, VOCs and heavy metal scans, radon, and uranium. Key physical monitoring parameters include water level, temperature, and specific conductivity. Sampling frequencies vary for categories of parameters from monthly to annually.

After the production well is operational, hydrochemical monitoring will be dictated by applicable state and local stipulations, including the Safe Drinking Water Act (SDWA) requirements. Levels of iron and manganese should be monitored weekly at the production well for decisions regarding water treatment. Levels of sodium and chloride should be monitored in the overburden and bedrock wells biannually to assess the impact of road-salting and to connate water movement. Levels of iron, manganese, radon, and uranium should be monitored biannually in the bedrock wells to determine potential water quality degradation caused by natural bedrock sources. All overburden and bedrock wells should be sampled biannually for cations, anions, and bacteria, as well as undergo annual VOC and heavy metal scans.

Early-warning detection monitoring is inappropriate in the fractured bedrock aquifer because ground-water velocities are rapid and flowpaths are discrete. The optimum approach to protecting the production well from unanticipated contamination is through public education and contingency planning. A three-tiered system of action levels is proposed for water quality parameters to trigger select contingency responses. Action levels are based on the Maximum Contaminant Level Goals set by the EPA, in conjunction with SDWA requirements.

- Level A violations trigger an increase in sampling frequency for target parameters, or additional parameters if warranted. All analyses are procured through certified laboratories.

- Level B violations signify a potential contamination event and justify a site-specific remedial investigation.

- Level C violations indicate that SDWA standards are exceeded, and the production well is shut down.

It is important to emphasize that the unique TOT distance values determined for Zones IIA and IIB from the quadratic equation and the dye-trace results should not be construed as absolute. Bedrock aquifers are immensely complex. The calculated TOT distances for both the 200-day and 1,000-day thresholds are large, and the hydrogeologic conceptual flow model of the test well area becomes much less reliable at greater distances. Fractures recharging the test well may, or may not, be

regionally continuous. Water flow through the fractured bedrock aquifer is likely to be contributed from additional, unmapped structures. However, employing the TOT criterion in conjunction with the dye-trace method, then modifying the protection zones based on a sound conceptual model of the aquifer system, provides a defensible approach to wellhead protection in this fractured bedrock regime.

RECOMMENDATIONS

As the city of Dover develops new water supply sources, the test well at the Blackwater Brook site will be replaced with a production well. The city purchased 50 acres of property surrounding the test well and plans to promulgate secondary and tertiary protection zones for this parcel. Other steps that may be considered as part of the city's ongoing efforts to protect their water supply include:

- Expansion of the recommended physical and hydrochemical monitoring program
- Joint implementation of wellhead protection management and regulation with adjacent municipalities
- More complete source assessment, possibly including door-to-door surveys, screening, and risk ranking as the area is further developed

The recommendations made in this report for long-term monitoring should be implemented. Of particular concern is the sand unit mapped to the north and northeast of the well, a deposit that may supply considerable recharge to the bedrock aquifer. The area should be monitored, in part, to evaluate the role of the sand unit in the recharge cycle.

The best scientific attempt to characterize the site hydrogeology and potential contaminant pathways can be verified or improved only through monitoring of the hydrologic cycle in an iterative procedure. The long-term monitoring program and its resultant data base will verify or refute the technical reliability of the WHPA delineation.

Zones II and III of the WHPA extend into the jurisdiction of adjoining municipalities. For this reason, negotiations between municipalities should occur to ensure that governing bodies enforce protection of shared water supplies. The optimum approach is for all municipalities to implement wellhead protection in sensitive areas. If critical differences in implementation strategy cannot be resolved through negotiation, then the city of Dover should install and monitor wells at municipal boundaries, as previously described.

As the area to the north of Dover continues to develop, improvements in the preliminary source assessment will be required. This area is currently unsewered, and developments will utilize onsite septic systems until sewer lines are extended into the rural areas. As residential areas develop, particularly on land identified as important recharge to the well, additional source assessment monitoring for nitrates may be needed. Another important target for improved source assessment is the landfill to the northwest. The present hydrogeologic conceptual model indicates that flow from the landfill toward the test well is unlikely -- the landfill is outside of Zone II. Nevertheless, source monitoring is appropriate for baseline characterization until a future time when monitoring data verify the correctness of the Zone boundaries.

To improve the conceptual model of the site, numerical simulation of ground-water flow may be considered, taking advantage of the extensive data base generated in the research monitoring program. The Blackwater Brook flow system should not be modeled with conventional numerical codes such as MODFLOW or AQUIFEM because they employ a mathematical foundation for porous, rather than fractured, media. However, recently developed model codes that simulate dual porous media hold promise for understanding ground-water flow in complex, fractured bedrock aquifers.

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APPENDIX 5-A

DERIVATION OF THE TIME-OF-TRAVEL EQUATION, BLACKWATER BROOK SITE, DOVER NEW HAMPSHIRE

DERIVATION OF THE TIME-OF-TRAVEL EQUATION

When using dye-trace breakthrough curve results for volume measurements of a hydraulic system, the time to the peak or the time to the mass of the centroid of the trace generally provides the best estimate of the mean travel time (Mull et al., 1988). However, assuming that a contaminant travels principally by way of advection in a fractured bedrock aquifer, the most useful aspect of the dye-trace results for purposes of protecting the well is the time of first arrival of the dye. Additionally, utilizing the time of first arrival aids in data analysis and interpretation and reduces costs.

The first arrival times of rhodamine-WT and fluorescein (2.2 and 148 hours, respectively; see Figure 5-21) reflect variations in the direction and distance of travel from the injection wells to the test well (TW, the detection well) where samples were taken. In order to contour the time-of-travel (TOT) from the injection wells to TW on a map, the effects of direction and distance must be separated. The theoretical approach to determining TOT distances presented here justifies a quadratic relationship in which the time of first arrival (or the TOT) is proportional to the square of the distance between the injection and detection wells. The validity of the quadratic relationship depends upon the validity of the following assumptions:

- Darcy's Law holds
- The Thiem equation holds
- Steady-state conditions prevail
- Discharge, transmissivity, and porosity are constant in space and time

Any mathematical description or model of a natural system must be based on simplifying assumptions. The applicability of the model depends on the appropriateness of the assumptions and on the recognition or prediction of circumstances under which the assumptions will be valid. Darcy's Law applies to ground-water flow except in areas of rock with large openings or in areas of steep hydraulic gradients, such as the vicinity of the pumped test well. Flow in fractured bedrock is often different from that in a porous medium. Fracture openings may be large with turbulent water flow; Darcy's Law assumes laminar flow. Darcy's Law may be violated close to the pumping well at the Blackwater Brook site where hydraulic gradients are steeper. Flow in the fracture directly connecting TW to well R2, for example, is probably non-Darcian. However, over the larger distances to be considered in the wellhead protection area (WHPA) delineation process, the bulk rock properties and flow may be nearly approximated by Darcy's Law. At large distances, radial flow patterns may predominate even when pumping analysis indicates a linear flow system near the well (Jenkins and Prentice, 1982).

The derived equation is taken from standard, accepted hydrogeologic relationships (Darcy's Law and the Thiem equation). It represents the most simple analytical relationship to interpret the hydrogeologic data. Graphical plots of distance versus drawdown from pumping TW at 200 gpm indicate that the well stabilized about 7 days (10,000 minutes) after pumping began. Therefore, the steady-state assumption required to use the Thiem equation is satisfied. Discharge was constant at 200 gallons per minute throughout the 15 days of pumping. Hydraulic conductivity and porosity should not vary appreciably in time, but may vary in space as a result of local heterogeneities in the bedrock. However, the use of empirically determined seepage velocities from the dye-trace results eliminates the need to estimate the parameters of hydraulic conductivity (K) and porosity (ϕ) required

to calculate seepage velocities independently of the average velocity, V, where $V = (K\, dh/d\iota)/\phi$, and $dh/d\iota$ is the hydraulic gradient.

The results represent an intermediate approach between TOT distances calculated using pipe-flow assumptions and those determined using an equivalent porous medium approach. The Thiem equation assumes radial flow, which is not appropriate in the case of the Blackwater Brook bedrock aquifer. The aquifer is best characterized as a linear flow system along the N5-10°W fracture zone, which justifies an intermediate approach in the analytical velocity calculation. However, this problem is solved to some degree by presenting separate solutions and using different constants in the TOT equation for the two dye-trace directions, N30°W and N60°E. Additionally, uncertainties of the exercise are mitigated by using the results of other research technologies, such as borehole geophysics and geologic mapping, to modify the calculated TOT distances to create wellhead protection zones for the well.

Assuming that Darcy's Law holds,

$$V = \frac{-K}{\phi}\frac{dh}{d\iota} \tag{1}$$

where V is average velocity; K is hydraulic conductivity; ϕ is porosity; and $dh/d\iota$ is the hydraulic gradient, the change in head (h) with respect to distance (ι) in a radial direction. We also know that

$$Q = \frac{2\pi T\,(s_p - s_{mw})}{\ln\,(\iota/r_p)} \qquad \text{(Thiem equation)} \tag{2}$$

where s_p and s_{mw} are changes in head in pumping and monitoring wells, respectively; Q is discharge; T is transmissivity; and r_w is the radius of the pumping well. Rearranging yields

$$(s_p - s_{mw}) = \frac{Q}{2\pi T}\ln\,(\iota/r_w) \quad \textit{and}$$

$$(s_p - s_{mw}) = \frac{Q}{2\pi T}[\ln\,(\iota) - \ln\,(r_w)]. \tag{3}$$

Because head difference, s, equals change in head $(h_0 - h_1)$,

$$(s_p - s_{mw}) = [(h_{p0} - h_{p1}) - (h_{mw0} - h_{mw1})].$$

Under steady-state conditions, all of these parameters (except ι and h_{mw}) are constant with respect to distance. Substituting the $(h_0 - h_1)$ for s values and differentiating with respect to distance (ι) yields

$$\frac{d}{d\iota}[(h_{p0} - h_{p1}) - (h_{mw0} - h_{mw1})] = \frac{d}{d\iota}\left[\frac{Q}{2\pi T}\ln\,(\iota) - \ln(r_w)\right],$$

which yields

$$\frac{dh_{mwl}}{d\iota} = \frac{Q}{2\pi T}\frac{d}{d\iota}[\ln(\iota) - \ln(r_w)]$$
$$= \frac{Q}{2\pi T}[\frac{d}{d\iota}(\ln(\iota)) - \frac{d}{d\iota}(\ln(r_w))]$$
$$= \frac{Q}{2\pi T}[\frac{1}{\iota}], \textit{ since } r_w \textit{ is constant.}$$

Letting $\frac{Q}{2\pi T} = -C_1$, we obtain

$$\frac{dh}{d\iota} = \frac{-C_1}{\iota}. \quad (4)$$

Substituting the $\frac{-C_1}{\iota}$ term from Equation 4 as $\frac{dh}{d\iota}$ in Darcy's Law (Equation 1) yields

$$V = \frac{-K}{\phi}(\frac{-C_1}{\iota}) = \frac{KC_1}{\phi\iota}. \quad (5)$$

Since the average velocity, V, equals $\frac{d\iota}{dt}$, where dt is the change in time, then

$$\frac{d\iota}{dt} = \frac{KC_1}{\phi\iota}.$$

Separating variables,

$$\iota d\iota = \frac{KC_1}{\phi} dt.$$

Integrating $\int \iota d\iota = \int \frac{K}{\phi} C_1 dt$ yields

$$\frac{\iota^2}{2} + (C_2) = \frac{K}{\phi} C_1 t. \quad (6)$$

The dye travels no distance at a time equal to zero, therefore, substituting $\iota=0$ and $t=0$ into Equation 6 indicates that $C_2=0$, and

$$\frac{\iota^2}{2} = \frac{K}{\phi} C_1 t \,. \tag{7}$$

Rearranging yields

$$\iota^2 = \frac{2KC_1}{\phi} t \,.$$

Setting $\frac{2KC_1}{\phi} = C_3$ yields the following quadratic equation relating the time of first arrival of the dye (t) to the distance from the injection well to TW (ι). The constant, C_3, varies with direction.

$$\iota = C_3 t^{\frac{1}{2}} \tag{8}$$

For simplicity, Equation 8 can be expressed as

$$\boxed{\iota = C t^{\frac{1}{2}}} \tag{9}$$

Values for C in Equation 9 were determined from the empirical results of the dye tracing for the N30°W and N60°E directions at Blackwater. For the N30°W direction,

$$t = 130 \; min \; (0.09 \; d) \quad and$$

$$\iota = 152 \; ft \; (well \; R2 \; to \; TW).$$

Given that $\iota = Ct^{\frac{1}{2}}$ (Equation 9), and replacing,

$$152 \; ft = C \; (0.09)^{\frac{1}{2}} \; (d)^{\frac{1}{2}}$$

$$C = (\frac{152}{0.3}) \; ft/d^{\frac{1}{2}}$$

$$C = 506 \; ft/d^{\frac{1}{2}}.$$

The equation to determine TOT distances in the N30°W direction is therefore

$$\iota = 506\, t^{\frac{1}{2}}.$$

The equation for the TOT distance in the N60°E direction is determined in a similar manner, using $t^{1/2}$ = 148 hr (6.2 d) and ι = 596 ft, resulting in a value for C of 240 ft/$d^{1/2}$. The equation in the N60°E direction is

$$\iota = 240\, t^{\frac{1}{2}}.$$

Calculated distances corresponding to the 200-day and 1,000-day TOT for each direction are presented in Table A5-1.

TABLE A5-1. TIME-OF-TRAVEL DISTANCES ALONG DYE-TRACE DIRECTIONS, BLACKWATER BROOK SITE, DOVER, NEW HAMPSHIRE

TOT Criteria (d)	Calculated Distances (ft)	
	N30°W (R2 to TW)	N60°E (R3 to TW)
200	7,200	3,400
1,000	16,000	7,600

Abbreviations: TOT = time of travel; TW = test well.

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