SIMULATED HYDROLOGIC RESPONSES OF THE QUASHNET RIVER STREAM-AQUIFER SYSTEM TO PROPOSED GROUND-WATER WITHDRAWALS, CAPE COD, MASSACHUSETTS

By Paul M. Barlow and Kathryn M. Hess

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CONVERSION FACTORS AND VERTICAL DATUM

Multiply	By	To obtain
	Length	
inch (in.)	25.4	millimeter
	2.54	centimeter
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
	Velocity	
foot per second (ft/s)	0.3048	meter per second
	Area	
acre	4,047	square meter
square mile (mi^2)	2.590	square kilometer
	Flow	
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
cubic foot per day (ft^3/d)	0.02832	cubic meter per day
cubic foot per second per square mile [(ft^3/s)/mi^2]	0.01093	cubic meter per second per square kilometer
gallon per minute (gal/min)	0.06308	liter per second
million gallons per day (Mgal/d)	0.04381	cubic meter per second
	Recharge	
inches per year (in/yr)	25.4	millimeter per year
	Specific storage	
per foot (ft^{-1})	0.3048	per meter
	Hydraulic conductivity[1]	
foot per day (ft/d)	0.3048	meter per day
	Transmissivity[1]	
foot squared per day	0.09290	meter squared per day

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

[1]In this report, the unit of hydraulic conductivity is foot per day (ft/d), the mathematically reduced form of cubic foot per day per square foot [(ft^3/d)/ft^2]. The unit of transmissivity is foot squared per day (ft^2/d), the mathematically reduced form of cubic foot per day per square foot times foot of aquifer thickness {[(ft^3/d)/ft^2]ft}.

Simulated Hydrologic Responses of the Quashnet River Stream-Aquifer System to Proposed Ground-water Withdrawals, Cape Cod, Massachusetts

By Paul M. Barlow and Kathryn M. Hess

ABSTRACT

This report describes efforts by the U.S. Geological Survey, in cooperation with the Mashpee Water District, to simulate the response of the Quashnet River stream-aquifer system, Cape Cod, Massachusetts, to ground-water withdrawals that have been proposed within 3,800 ft of the river. The study was prompted in response to concern over possible reductions in streamflow and degradation of the sea-run brown trout habitat of the river resulting from the proposed withdrawals.

A two-layer steady-state and transient finite-difference ground-water-flow model with a stream-routing component was developed to simulate the stream-aquifer system. Steady-state pumping rates of 0.5, 1.0, and 2.0 million gallons per day were simulated in five scenarios at three well sites 1,500 to 3,800 ft west of the river. No infiltration of water from the river to the aquifer was induced in any of the simulations; the river received ground-water discharge along its entire length for all withdrawal scenarios tested. Streamflow depletions in the river resulting from the simulated withdrawals range from 1.3 to 3.8 percent of the calculated, steady-state, prepumping streamflow for the site that is the upstream limit of active trout spawning, and from 3.8 to 14 percent for the gaging station, 1.6 and 0.36 miles upstream from the mouth of the river, respectively.

One withdrawal scenario was investigated further by use of the transient model to investigate the effects of pumping on seasonal fluctuations of streamflow of the river; simulated ground-water withdrawal was 1.0 million gallons per day at a site 1,500 ft west of the river. For this scenario, the calculated steady-state depletion at the streamflow gage is 7.5 percent. Monthly mean streamflow depletions calculated by use of the transient model range from 6 to 8 percent of the measured monthly mean streamflows at this site. This small range in depletions can be attributed to the relatively steady flow of the Quashnet River throughout the year. This constancy results from the large infiltration and storage capacity of the glacial outwash deposit which covers all of the Quashnet River basin and which tends to damp the effect of climatic variability on streamflow. Calculated monthly mean streamflows at the gaging station from late spring through fall meet the stream-discharge guidelines established by the Massachusetts Division of Fisheries and Wildlife. Guidelines for winter through early spring are not met for this pumping rate; however, these guideline currently are not met for existing (nonpumping) streamflow conditions.

A particle-tracking postprocessor to the flow model was used to delineate steady-state contributing areas of the river and the simulated wells. A contributing area consists of that area over which recharge entering the aquifer across the water table eventually discharges to the river or the well. The contributing area of the river for the condition of no pumping lies primarily to the

west of the river. In each withdrawal scenario, the contributing area of the pumping well also lies west of the river. Although the simulated cone of depression produced by pumping from each well extends areally across the river, the contributing area of each well does not include the river. Thus, ground-water withdrawals will reduce the flow in the river; but no river water will be diverted to pumped wells under the withdrawal scenarios investigated.

INTRODUCTION

Since 1988, the Mashpee Water District (MWD) has been responsible for the development of a public water-supply system for the town of Mashpee, Mass. on Cape Cod. In an effort to develop a high-yielding source of potable ground water, the MWD has begun a ground-water exploration program aimed at identifying sites for potential ground-water resource development. Several sites identified by the MWD are near the Quashnet River, a ground-water-fed stream that flows through the towns of Mashpee and Falmouth (fig. 1).

The Quashnet River overlies and receives discharge from the largest of the highly permeable sole-source, water-table aquifers of Cape Cod (LeBlanc and others, 1986). The river originates at Johns Pond and drains into Waquoit Bay, an estuary of Nantucket Sound (fig. 1). Waquoit Bay was identified in 1988 as a National Estuarine Research Reserve by the National Oceanic and Atmospheric Administration and the Massachusetts Department of Environmental Management (MDEM). The Quashnet River is one of the largest streams on Cape Cod and supports a naturally reproducing population of sea-run brown trout. It is recognized by the Massachusetts Division of Fisheries and Wildlife (MDFW) as one of the four most important trout fisheries of Cape Cod because of its favorable physical characteristics of stream width, length, and rate of flow, and its favorable water quality (J. D. Bergin, Massachusetts Division of Fisheries and Wildlife, oral commun., 1991). To protect the river and the underlying sole-source aquifer from the effects of increased development near the Quashnet River, the Commonwealth of Massachusetts purchased 361 acres of land along the river in 1987.

Physical characteristics of the river have not always been conducive to trout spawning and habitation. From the 19th century through the early 1950's, the floodplain was converted almost entirely to cranberry bogs. Activities associated with bog agriculture resulted in an artificial straightening of the river channel. In addition, large volumes of sand applied to the bogs flowed into the river and covered its gravel beds (McLarney, 1988). Most of these cranberry bogs were abandoned in the late 1950's.

Since 1975, the Quashnet River has been the site of an intensive restoration project by the Cape Cod chapter of Trout Unlimited, a fishing and conservation group. As a result of the restoration process, some sections of the river have been unclogged and protective cover for adult trout has been constructed. The trout population of the river has increased an estimated twenty-fold from 1975 through 1988 (McLarney, 1988). Currently (1993), four of the five sections of the river with large numbers of spawning areas are upstream from the proposed public water-supply well sites (between sites Q5 and Q7, fig. 1); the other spawning area is at a U.S. Geological Survey (USGS) streamflow-gaging station near the mouth of the river (site Q9, fig. 1).

Concern by the MDFW and local conservation groups regarding possible reductions in streamflow within the Quashnet River as a result of proposed withdrawals prompted initiation of a cooperative investigation of the Quashnet River stream-aquifer system by the USGS and MWD. In the first phase of that investigation, the physical and water-quality characteristics of the Quashnet River were described (Baevsky, 1991). In the second phase of the investigation, the hydrologic responses of the Quashnet River stream-aquifer system to proposed pumping within 3,800 ft of the river (fig. 1) were estimated. These estimated responses should assist the MWD and MDFW in managing the water and fishery resources of the Quashnet River basin.

Purpose and Scope

The purposes of this report are to describe (1) the hydrologic characteristics of the Quashnet River stream-aquifer system, and (2) the responses of the system to proposed ground-water withdrawals simulated by use of a numerical model of ground-water flow. These responses to pumping are assessed in terms of drawdowns in the altitude of the water table, streamflow depletions, and sources of water to the river and the simulated supply well. In this report, the Quashnet River stream-aquifer system is defined as that area extending approximately from the Mashpee River on the east to the Coonamessett River

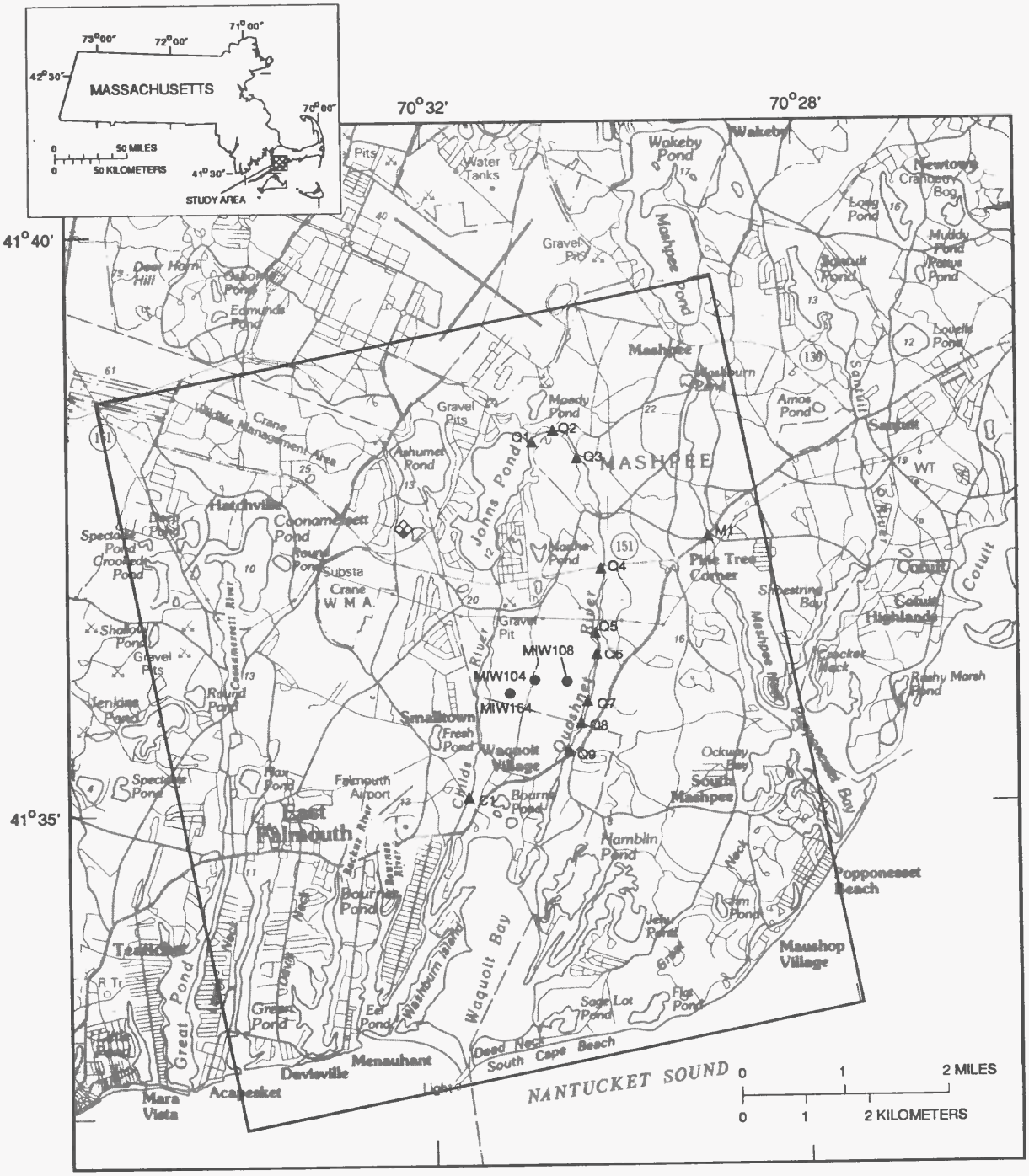


Figure 1.--Quashnet River study area, streamflow- and precipitation-gaging stations, and the extent of the modeled area.

on the west and from Nantucket Sound to approximately 1 mi north of Johns Pond (fig. 1).

This report is based on ground- and surface-water data collected during the study and compiled from engineers' reports and USGS ground- and surface-water data bases. Physical characteristics of the Quashnet and Childs Rivers (streamflow, stream velocity, width, and depth), surface altitude of Johns Pond, and hydraulic heads within the basin were measured from January 1990 through September 1991.

A two-layer ground-water-flow model with a surface-water routing component was developed during the study to assess ground-water/surface-water interactions in the basin using the computer models of McDonald and Harbaugh (1988) and Prudic (1989). Response of the system to proposed pumping was evaluated in terms of streamflow depletion in the Quashnet River and declines in the altitude of the water table. Areas that contribute water to the Quashnet River and to simulated wells were determined by means of a particle-tracking postprocessor to the ground-water-flow model (Pollock, 1989).

Acknowledgments

The authors thank several individuals who provided data or assisted in the acquisition of data during this investigation: Thomas Fudala, Mashpee Town Planner; Joseph Bergin, Massachusetts Division of Fisheries and Wildlife; Robert Sherman, Mashpee Conservation Commission; Francis Smith, Cape Cod chapter of Trout Unlimited; Linda Gordon, Association for the Preservation of Cape Cod; Christine Gault, Waquoit Bay National Estuarine Reserve; Larry Dearborn, ABB Environmental (Portland, Maine); J. Theodore Morine, D. L. Maher Company (Andover, Mass.); and Donald L. Jermyn and Ann A. McLean of Mashpee.

QUASHNET RIVER STREAM-AQUIFER SYSTEM

This section describes the ground- and surface-water hydrology and hydrogeology of the Quashnet River stream-aquifer system. The hydrology of the Cape Cod aquifer system, including the Quashnet River area, has been described previously by Strahler (1972), Ryan (1980), Guswa and LeBlanc (1985), and LeBlanc and others (1986). The geology of Cape Cod has been described previously by Oldale (1976, 1981).

Geologic Setting and Hydrogeologic Units

The Quashnet River drainage basin encompasses approximately 5 mi^2 in Mashpee and Falmouth, Mass., and includes the drainage basin of Johns Pond. The basin is underlain by glacial outwash composed of sand, gravel, silt, and clay that is part of the Mashpee Pitted Plain, a feature deposited during the Pleistocene glaciation of New England (Oldale, 1976, p. 1). The Mashpee Pitted Plain was formed by meltwater streams flowing from glacial moraines to the west and north of the study area. The surface of the plain slopes gently southward toward Nantucket Sound and is cut by dry valleys, stream channels, and kettle ponds. Four sections (figs. 3 and 4; locations shown in fig. 2), prepared from records of test borings and geophysical logs of test holes, show hydrogeologic units of the aquifer that are interpreted to be hydrologically similar.

The top part of the aquifer is composed mostly of fine to coarse sand; gravel is a locally significant component (fig. 3). The thickness of this top part ranges from less than 50 to 200 ft. Fine-grained sediments consisting of fine sand and silt from 50 to 150 ft thick underlie these coarse-grained sediments. Because few test borings have penetrated the fine sand and silt unit, the configuration of the lower boundary of the unit is generally unknown; however, most test borings that have penetrated the unit indicate that it is underlain by a heterogeneous unit of silt, clay, sand, and gravel. At site MIW 26 (fig. 3), coarse-grained sediments lie between the fine sand and silt and the lower fine-grained unit. The areal extent of this sand body is unknown; it may be part of an ice-contact deposit mapped nearby by Oldale and Barlow (1986). In general, the upper coarse-grained unit thins and the lower fine-grained units thicken from north to south and from west to east; the top of the fine-sand-and-silt unit becomes shallower from north to south. Crystalline bedrock underlies the unconsolidated units. The bedrock surface generally deepens from northwest to southeast. The altitude of the bedrock surface was compiled from test borings and seismic-refraction surveys (Oldale, 1969, p. B123, and B.D. Stone, U.S. Geological Survey, written commun., 1991).

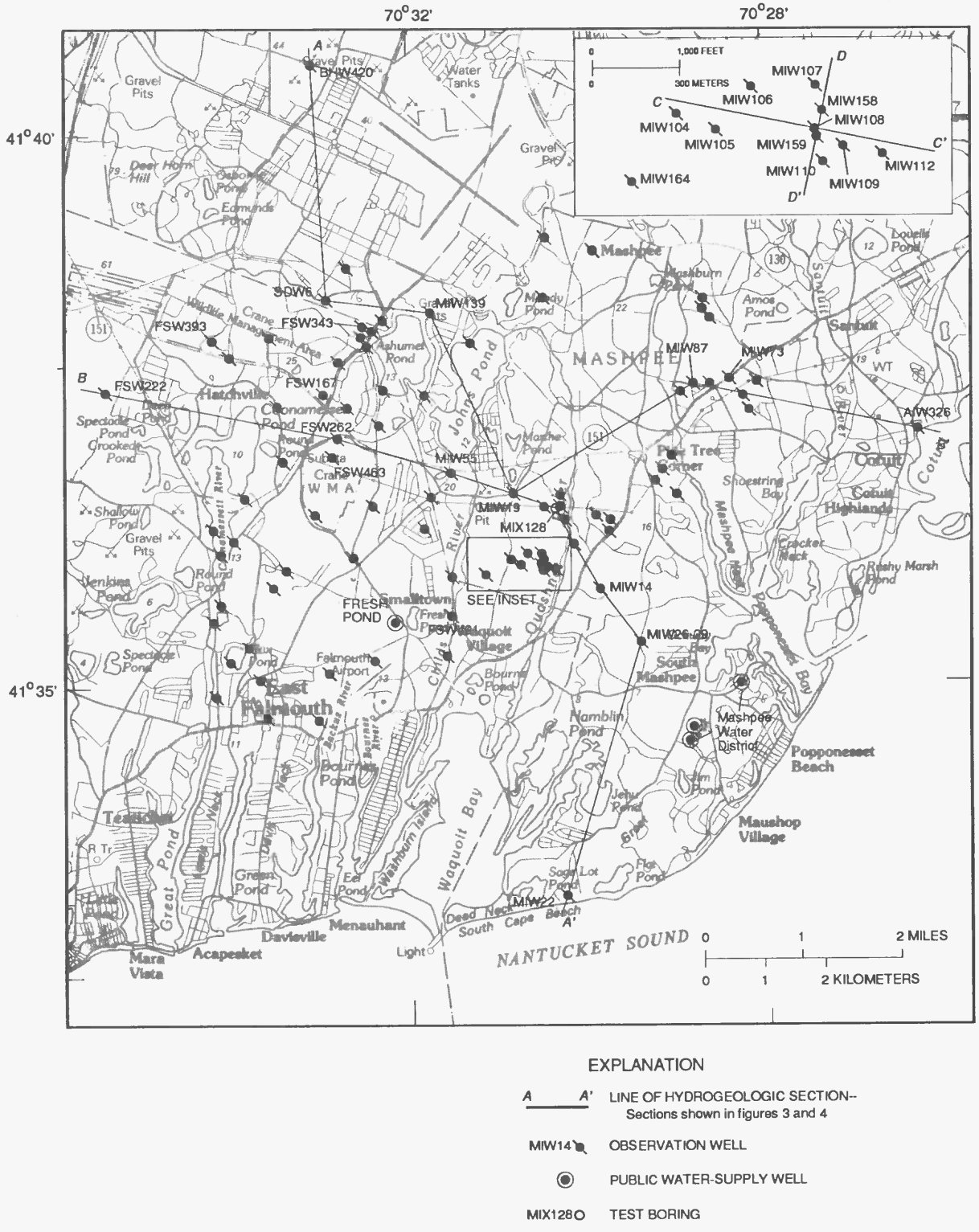


Figure 2.--Location of observation wells, public water-supply wells, and lines of hydrogeologic sections near the Quashnet River.

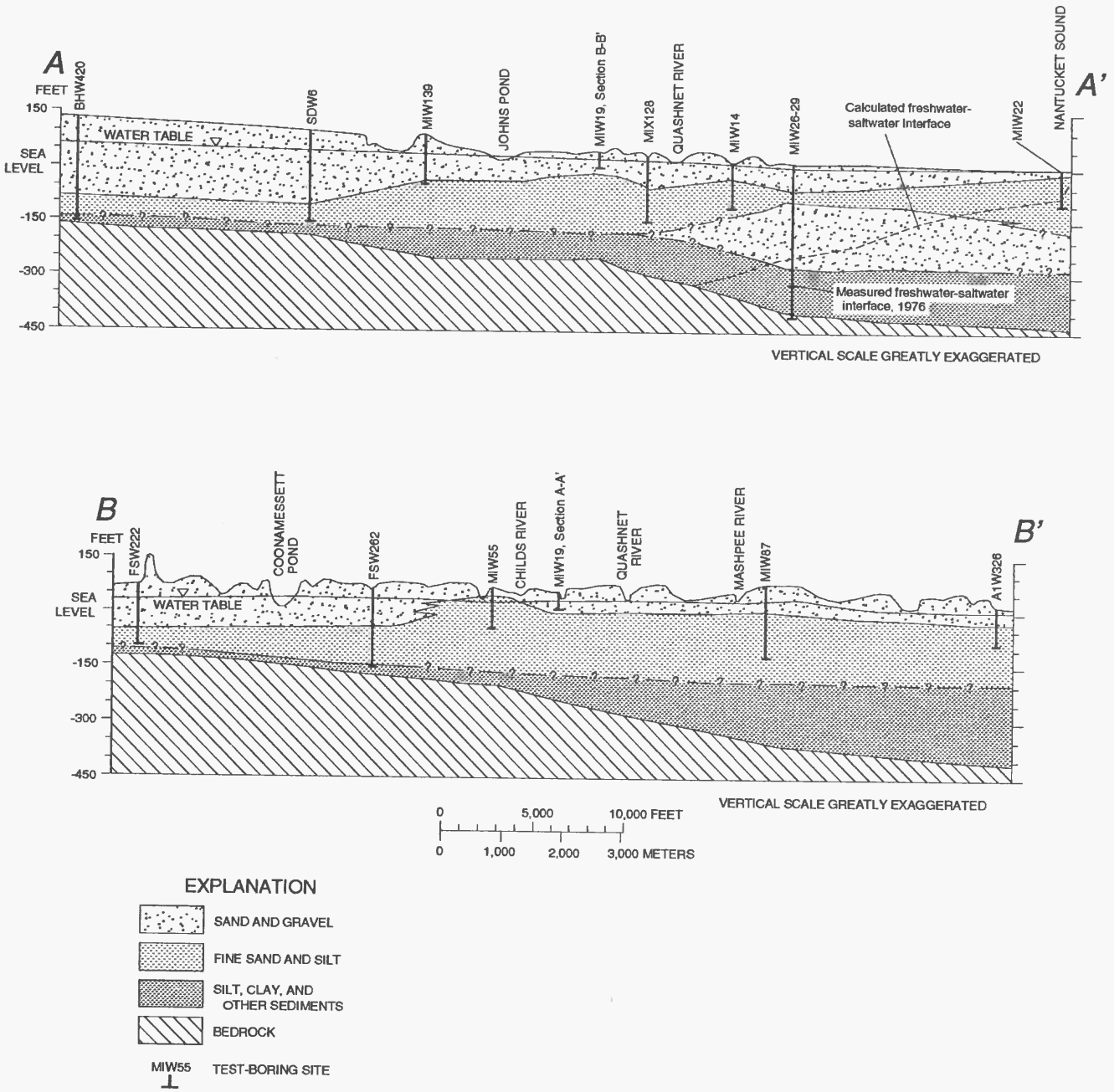


Figure 3.--Hydrogeologic sections *A-A'* and *B-B'* showing units of the Quashnet River stream-aquifer system. Calculated freshwater-saltwater interface from Guswa and LeBlanc (1985). Measured interface from LeBlanc and others (1986). Lines of sections shown in figure 2.

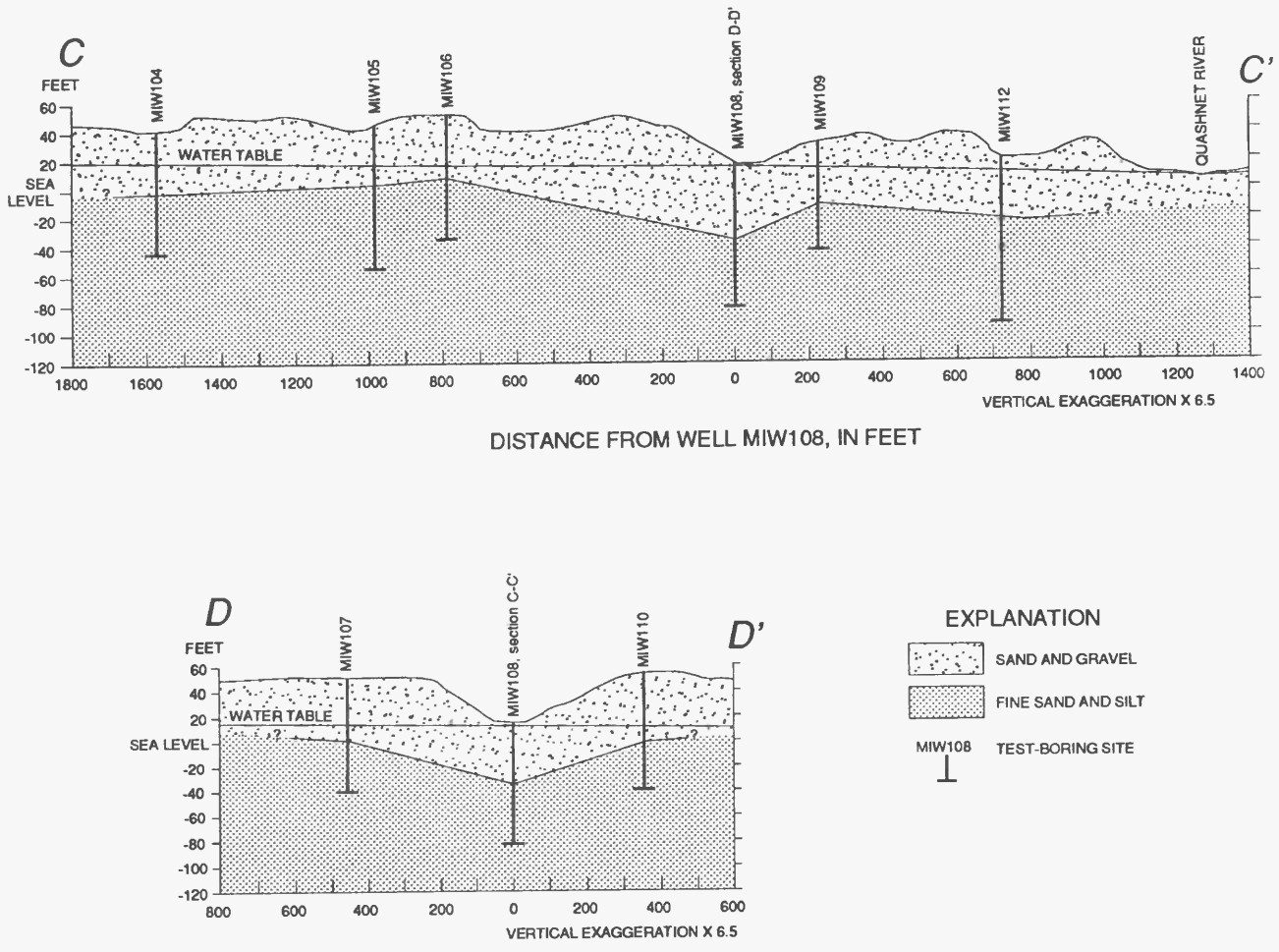


Figure 4.--Hydrogeologic sections *C-C'* and *D-D'* showing units of the Quashnet River stream-aquifer system near well MIW 108. Wells other than MIW 108 are projected onto sections. Lines of sections shown in figure 2.

The lithology of the top 120 ft of aquifer in the area of the proposed pumping reflects the regional lithology of the aquifer (fig. 4). The top 20 to 60 ft of the aquifer consists of fine to coarse sand and some gravel. Test borings indicate that the upper coarse-grained unit is locally thicker at the well site (MIW 108, fig. 4). The upper coarse-grained unit is underlain by a unit of fine sand and silt that is 25 to 80 ft thick.

Aquifer Hydraulic Properties

Hydraulic properties that need to be determined to evaluate the interaction between the Quashnet River and the underlying aquifer are the hydraulic conductivity, porosity, specific yield, and storage coefficient (or coefficient of specific storage) of the aquifer and the hydraulic conductance of the streambed sediments. The storage coefficient is the product of the coefficient of specific storage and saturated thickness of the aquifer. Estimates of horizontal and vertical hydraulic conductivity are needed to determine rates of ground-water flow and velocities of ground-water particles within the aquifer; porosity also is needed to determine the velocity of ground water within the flow system. Specific yield and storage coefficient are needed to analyze the transient response of the ground-water system to changes in the rates of ground-water pumping and recharge. Streambed conductance is needed to quantify the flow of water between the aquifer and streams.

The first part of this section summarizes estimates of aquifer hydraulic properties available from previous studies of the Quashnet River and larger Cape Cod aquifer system. The second part outlines the methodology and results of a numerical analysis of aquifer hydraulic properties that is based on an aquifer test completed at a well located 1,500 ft west of the river. This well is identified in this report by the USGS well number MIW 108; it is identified by the MWD as the P-1 site (Dufresne-Henry, 1987). Streambed conductance is discussed in the section on "Hydrologic responses of the Quashnet River stream-aquifer system to proposed ground-water withdrawals."

Previous Estimates

Several investigators have reported estimates of the hydraulic conductivity in the Quashnet River basin and adjacent areas. The average horizontal hydraulic conductivity of sediments in the area west of the Childs River (fig. 2) has been estimated by use of empirical equations relating grain-size distribution to hydraulic conductivity and by use of several types of hydraulic tests, including aquifer, borehole flowmeter, and slug tests. LeBlanc (1984, p. 22) used grain-size distributions of 12 samples of cored sediment to estimate an average hydraulic conductivity of the sand and gravel of 164 ft/d. He also estimated the hydraulic conductivity of the underlying silty sand and till to be roughly 10 to 20 times lower than that estimated for the sand and gravel. Palmer (1977, p. 45) reported an average horizontal hydraulic conductivity of 140 ft/d for four aquifer tests completed within this area. LeBlanc and others (1988, p. B7) estimated an average horizontal hydraulic conductivity of 380 ft/d and a ratio of horizontal to vertical hydraulic conductivity ranging from 2:1 to 5:1 for sand and gravel at an aquifer-test site 500 ft southwest of FSW 463 (fig. 2). J.P. Masterson (U.S. Geological Survey, written commun., 1992) estimated a horizontal hydraulic conductivity of 350 ft/d for fine to medium sand and gravel at an aquifer-test site 2,000 ft west of FSW 491 (fig. 2). Hess and others (1992) obtained a mean horizontal hydraulic conductivity of 310 ft/d for sediments near well FSW 343 (fig. 2) from field tests made with a borehole flowmeter. In the same area, Wolf (1988) estimated a mean horizontal hydraulic conductivity of 100 ft/d from permeameter tests on cores. Springer (1991) estimated a geometric mean horizontal hydraulic conductivity of 171 ft/d for sediments within the area west of the Childs River, on the basis of the analysis of 335 slug tests. In summary, previous estimates of horizontal hydraulic conductivity of sediments west of the Childs River range from about 100 to 380 ft/d.

Hydraulic conductivity and transmissivity of the aquifer east of the Childs River have been estimated by analysis of several aquifer tests. Four aquifer tests in the town of Barnstable, just east of the study area, indicate that the horizontal hydraulic conductivity of the aquifer is generally lower in the eastern part of the study area than in the western part. In 1992, P.M. Barlow (the senior author of this report) obtained a horizontal hydraulic conductivity of 160 ft/d and a ratio of horizontal to vertical hydraulic conductivity of 30:1 from a test in a fine sand portion of the aquifer. J.P. Masterson (written commun., 1992) obtained horizontal hydraulic conductivities of 145, 240, and 250 ft/d for three tests in fine to medium sand and gravel parts of the aquifer. Two aquifer tests within Mashpee, at wells MIW 108 and MIW 128 (fig. 2), have

provided estimates of the transmissivity of the aquifer in the area of proposed pumping. Transmissivity is a measure of the ability of the aquifer to transmit water and is equal to the product of horizontal hydraulic conductivity and saturated thickness of the aquifer. Average transmissivities of 7,960 ft^2/d for fine to coarse sand at well MIW 108 (Dufresne-Henry, 1987, p. 6) and 12,660 to 14,040 ft^2/d for fine to coarse sand and gravel at well MIW 128 are reported (Dufresne-Henry, 1990, p. 13 and 19). A third aquifer test in the eastern part of Mashpee, approximately 1,000 ft east of well MIW 73 (fig. 2), yielded an estimate of transmissivity of 4,680 ft^2/d for fine to coarse sand (Maher, 1988, p. 25).

Estimates of specific yield and storage coefficient have been made at only a few locations on Cape Cod. Palmer (1977, p. 45) reports a range of specific yield of 0.13 to 0.26 from four aquifer tests in the Falmouth area. Garabedian and others (1988) report a range in specific yield of 0.1 to 0.2 and a range of coefficient of specific storage of 4.4×10^{-5} to 8.7×10^{-5} ft^{-1} from the analysis of an aquifer test approximately 500 ft southwest of well FSW 463 (fig. 2). Dufresne-Henry (1990, p. 13) reports a range of specific yield of 0.13 to 0.21 for an aquifer test at well site MIW 128. Finally, J.P. Masterson (written commun., 1992) determined a range of 0.02 to 0.28 and an average of 0.16 for the specific yield of outwash on Cape Cod; these results are based on the analysis of 13 aquifer tests in sediments ranging from fine sand to coarse sand and gravel.

Porosity has been estimated for outwash near wells FSW 343 and FSW 393 (fig. 2) by means of ground-water tracer experiments and laboratory tests of cored sediments from the outwash. Estimates of porosity from the tracer tests range from 0.38 to 0.42 (Garabedian and others, 1988, p. 163; LeBlanc and others, 1988, p. B7; Barlow, 1989, p. 327), whereas that of the cored samples was 0.32 (Wolf, 1988, p. 106). The lower porosity reported by Wolf may have resulted from compaction of the cored samples. These estimates of porosity are similar to those of 0.34 to 0.38 reported for the glacial outwash on Long Island, N.Y. (Perlmutter and Lieber, 1970).

Analysis of Aquifer Test at Well MIW 108

Hydraulic properties of the aquifer in the area of proposed pumping were evaluated by analysis of a 5-day, variable-pumping-rate aquifer test at well MIW 108. The potential yield from a well at this site has been estimated at 700 gal/min (Dufresne-Henry, 1987). Horizontal and vertical hydraulic conductivities, specific yield, and coefficient of specific storage were estimated by simulating the aquifer test with a transient finite-element radial-flow numerical model (Reilly, 1984). The model simulates a vertical sector of the aquifer extending outward from the axis of the pumped well. Several simplifying assumptions were made in using the model. These assumptions (Reilly, 1984, p. 1) were that (1) ground-water flow is radially symmetric, which implies that aquifer hydraulic properties are uniform around the axis of the well; (2) no seepage face is present in the well; (3) the saturated thickness of the unconfined aquifer does not change significantly during pumping; (4) the aquifer is finite in extent; and (5) the specific yield and coefficient of specific storage are constant over the entire model grid. The finite-element model was chosen in preference to analytical methods of analysis because of the complex nature of vertical and lateral heterogeneity of the aquifer near the well and because the pumping rate varied during the aquifer test.

The finite-element-model grid was designed to simulate the two-layer geologic structure of the aquifer near the well (fig. 5). The upper zone of the model simulates the layer of fine to coarse sand and gravel near well MIW 108, whereas the lower zone simulates underlying fine sand and silt (fig. 4). The aquifer test consisted of pumping seven closely spaced 2.5-inch-diameter wells through a common suction manifold. The depths of the screened intervals of these wells ranged from 47 to 53 ft below the approximate static water level. In the model, these pumped wells are represented by a single well at the center of the cluster. The model grid extends 1,000 ft radially from the axis of the pumped well and vertically from the water table to 210 ft below the water table (fig. 5); the bottom of the grid coincides with the contact between unconsolidated sediments and bedrock.

Boundary conditions are specified on all four sides of the finite-element model. The upper boundary is a free-surface or water-table boundary, which is set to 0 ft. Although drawdown at the water table that occurred during the aquifer test is not simulated, water released from storage at the water table is simulated by the inclusion of specific yield (Reilly, 1984, p. 6). The outer-boundary side of the model is a constant-head boundary of 0 ft. This boundary is set 1,000 ft from the axis of the pumped well to minimize the influence of this boundary on simulated drawdowns. Calculated drawdowns within 225 ft of the

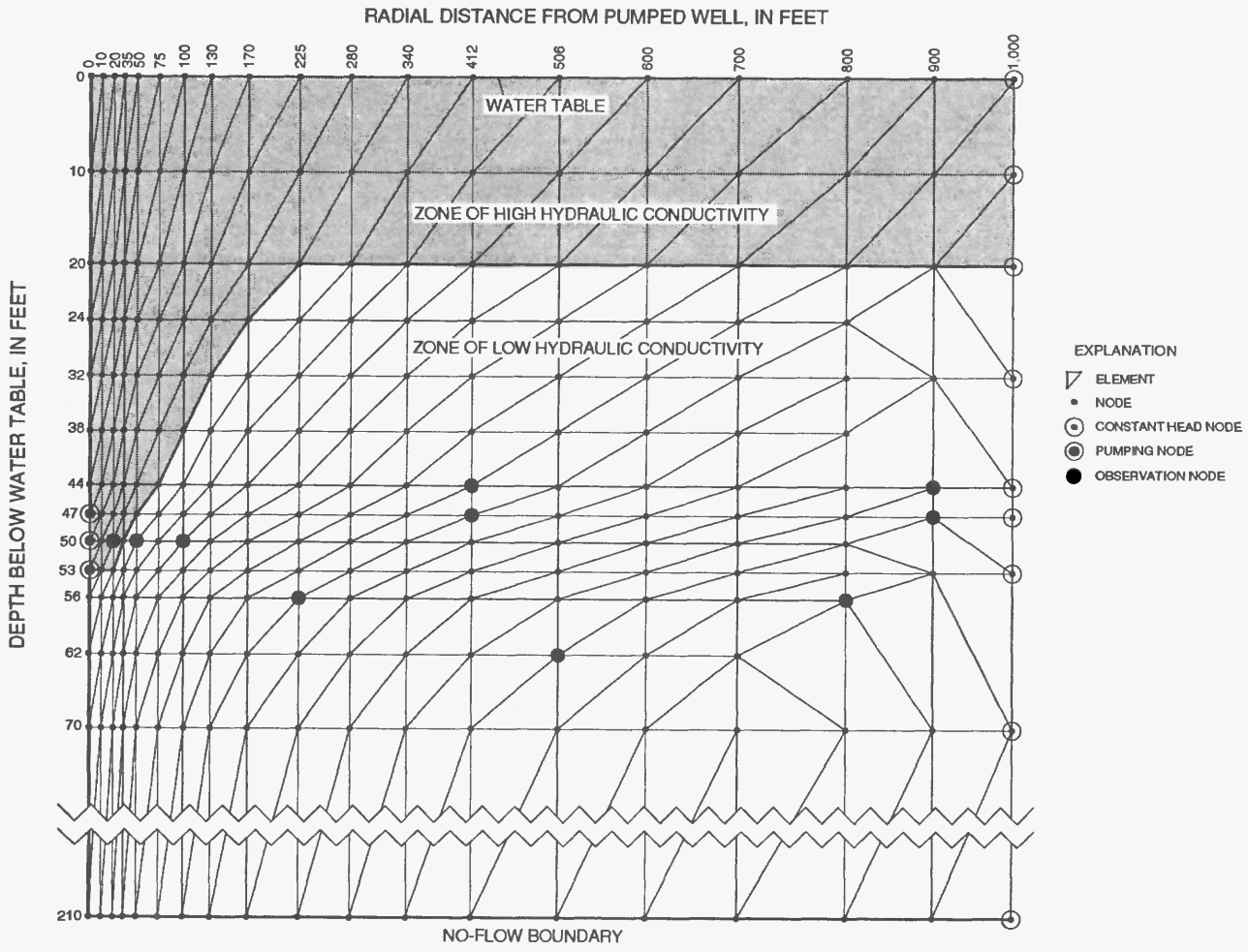


Figure 5.--Diagrammatic section of modeled aquifer showing configuration of triangular elements, boundary conditions, and distribution of hydraulic conductivity for the finite-element radial-flow model for the aquifer test at well MIW 108.

pumped well change a maximum of only 0.03 ft when the boundary is extended to 2,000 ft; thus, the location of the constant-head boundary at 1,000 ft is appropriate. The lower boundary is a no-flow boundary that coincides with the contact between unconsolidated sediments and bedrock; this boundary condition is used because bedrock is assumed to be impermeable. A streamline boundary also is specified at the center of the pumped well for all nodes except the three nodes that correspond to the location of the screened interval of the well. Pumping is simulated by use of a specified-flux boundary between these three nodes; pumping was distributed equally to the two elements bounded by the three pumping nodes.

The two pumping rates used during the aquifer test were simulated in the model. The pumping rate was 0.25 Mgal/d from 0 to 0.9 days and 0.42 Mgal/d from 0.9 to 5.0 days after pumping began. During the test (June 25-30, 1987), 2 in. of rain was measured at the Hatchville precipitation-gaging station, which is 2 mi from the test site (fig. 1); however, no discernible rise was observed in the water table at wells 900 and 1,000 ft from the pumped well. The lack of water-table rise indicates that no significant recharge to the aquifer occurred during the test.

Uniform values of horizontal and vertical hydraulic conductivity were specified for each of the two

zones of the model; uniform values of specific yield and the coefficient of specific storage were specified for the entire model. The model was calibrated by varying the hydraulic properties until calculated drawdowns at observation wells at distances of 20, 50, and 225 ft from the pumped well (wells MIW 159, 158, and 109, respectively) closely matched observed drawdowns. Observation wells beyond 225 ft were not used in the calibration because observed and calculated drawdowns in these wells were negligible.

Calculated and observed drawdowns at each of the three observation wells were compared at 16 times steps. The best match between calculated and observed drawdowns was determined by visually comparing the shapes of the two curves and by minimizing the mean percentage error between observed and calculated drawdowns at each observation well. The mean percentage error for each observation well is defined as

$$\frac{1}{n}\left(\sum_{i=1}^{n}\left|\frac{\hat{S}_i - S_i}{S_i}\right|\right)100\,, \qquad (1)$$

where $\hat{S}_i$ is the calculated drawdown at simulated time i,
S_i is the observed drawdown at actual time i, and
n is the number of time steps.

The values of hydraulic properties that resulted in the best match between observed and calculated drawdowns (fig. 6) are the following: horizontal hydraulic conductivity of 240 ft/d and a ratio of horizontal to vertical hydraulic conductivity of 3:1 for the upper zone; horizontal hydraulic conductivity of 40 ft/d and a ratio of horizontal to vertical hydraulic conductivity of 50:1 for the lower zone; specific yield of 0.25; and coefficient of specific storage of 1×10^{-6} ft^{-1}. These calibrated estimates of hydraulic conductivity, specific yield, and coefficient of specific storage compare favorably with those cited earlier for outwash of the larger Quashnet River and Cape Cod aquifer systems. If a total aquifer thickness of 210 ft near the pumped well is assumed, the calculated coefficient of specific storage corresponds to a storage coefficient of 2×10^{-4}.

In a sensitivity analysis of the radial-flow model, calculated drawdowns did not change significantly when specific yield was varied between 0.2 and 0.3, the coefficient of specific storage was set to less than 1×10^{-5} ft^{-1}, or the ratio of horizontal to vertical hydraulic conductivity of the lower zone of the aquifer was varied between 3:1 and 100:1. The model was most sensitive to increases in the horizontal hydraulic conductivity and the ratio of horizontal to vertical hydraulic conductivity of the upper zone. In addition, values of the coefficient of specific storage greater than 1×10^{-5} ft^{-1} resulted in shapes of drawdown curves that did not match the shape of observed curves.

These estimates of hydraulic properties were determined with some uncertainty because of limitations imposed by the design and underlying assumptions of the model. In addition, the insensitivity of calculated drawdowns to several of the model parameters--including specific yield, coefficient of specific storage, and ratio of horizontal to vertical hydraulic conductivity of the lower zone--indicate that these parameters were not uniquely defined by the model.

Principal assumptions made in the application of the model are that the geologic structure and hydraulic properties are symmetric about the axis of the pumped well and that the model structure accurately represents the aquifer. The hydrogeologic sections constructed through the pumped well (fig. 4) show slight deviations from radial symmetry; in general, however, the assumption of radial symmetry in hydrogeologic properties near the pumped well appears to be reasonable. A second assumption of the model is that no seepage face is present. During the aquifer test, the water table did not fall below the top of the screened interval of the pumped well; therefore, this assumption also is considered valid. A third assumption is that saturated thickness does not change significantly during the test. Observed drawdowns during the test indicate that the maximum reduction in the saturated thickness of the aquifer was only 6 percent. A fourth model assumption is that the aquifer is finite. A specified head was set at the outer boundary of the model at a radial distance of 1,000 ft, even though the aquifer extends significantly beyond this distance. As discussed above, this boundary was determined to be far enough from the pumping nodes that it did not affect simulated drawdown at the observation wells within a distance of 250 ft from the pumped well. Therefore, this lateral boundary would appear to have minimal effect on the results of the analysis. The final assumption is that the specific yield and storage coefficient are constant throughout the model. Although this is probably not the case, the variability in these storage properties is thought to be small, and mean values should adequately represent the system.

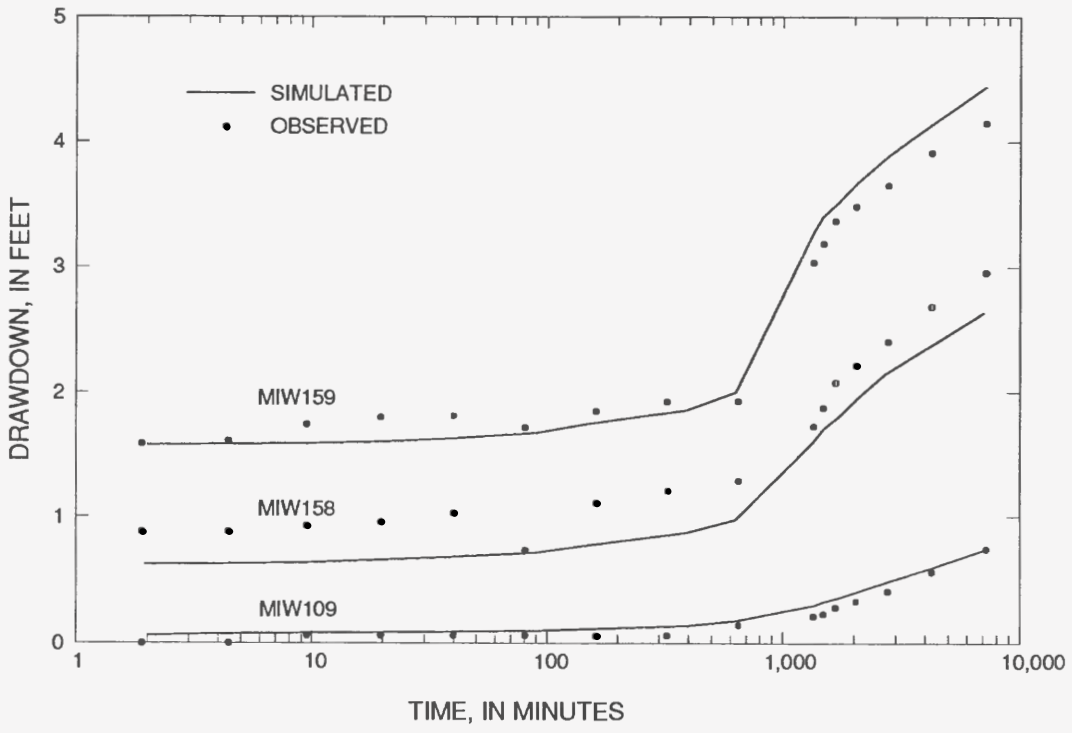


Figure 6.--Drawdowns observed during the aquifer test at well MIW 108 and simulated by use of the radial-flow model.

Uncertainty is also introduced into the simulations by the structure of the model. In the aquifer test, a cluster of seven wells was pumped; in the model, pumping is simulated for one well in the center of the cluster. This simplification may have affected the simulated drawdowns, particularly at nodes near the simulated pumped well. Simplifications also may have been introduced in grouping the aquifer system into two units. These limitations, uncertainties, and assumptions of the finite-element model, however, are considered to be small, and the estimates made of the hydraulic parameters are assumed to closely approximate those of the aquifer.

Ground-Water Hydrology

Ground water in the Quashnet River stream-aquifer system is unconfined; that is, the top of the saturated zone is the water table. Two maps showing the altitude and configuration of the water table were prepared from water levels measured during May 10-11 and November 2-3, 1990 (figs. 7 and 8). Only wells whose altitudes had been surveyed to an accuracy of at least 0.1 ft were measured. The investigation relied on observation wells that had been installed previously for the USGS, the Massachusetts Military Reservation (MMR), the MWD, and other town and private entities. In addition, steel well points with 0.6-foot-long screened intervals were driven 1 to 3 ft into the streambed sediments at the streamflow-measurement sites to determine heads immediately beneath the Quashnet River.

The water-table map for May 1990 represents high ground-water levels, whereas the map for November 1990 represents low ground-water levels. The water table slopes from north to south at about 9 to 10 ft/mi. Ground-water inflow from adjacent parts of the aquifer occurs along the northern edge of the study area, north of the 50 ft water-table contour shown in figures 7 and 8. Water levels declined an average of 1.1 ft from May through November; the range of water-level declines was from less than 0.5 to 2.0 ft. Water levels in two observation wells measured monthly (MIW 29) or bimonthly (MIW 19) since 1975 were 0.7 to 1.0 ft above mean levels in May 1990 and

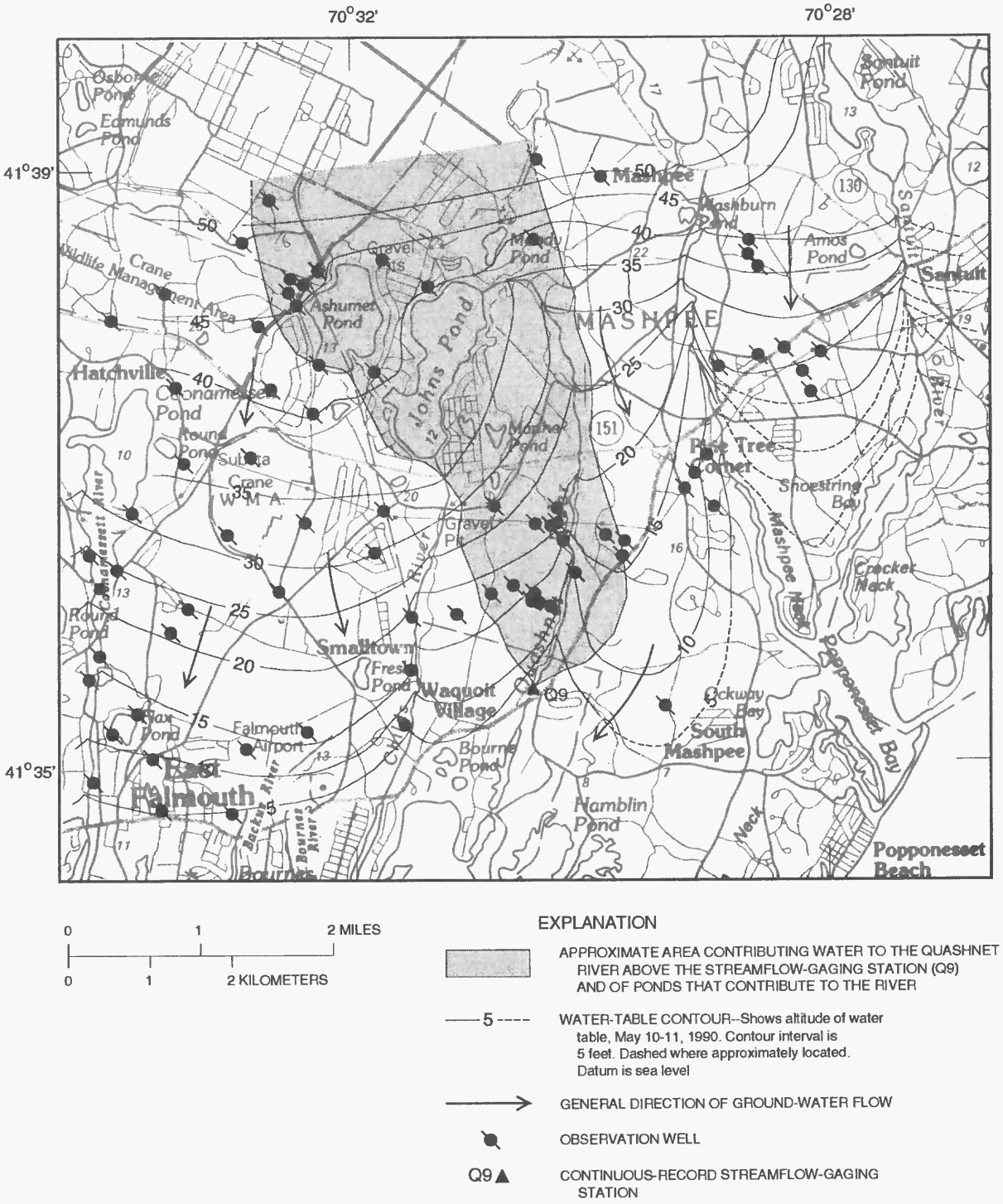


Figure 7.--Altitude and configuration of the water table and general direction of ground-water flow in the Quashnet River stream-aquifer system and approximate contributing area of the river, May 10-11, 1990.

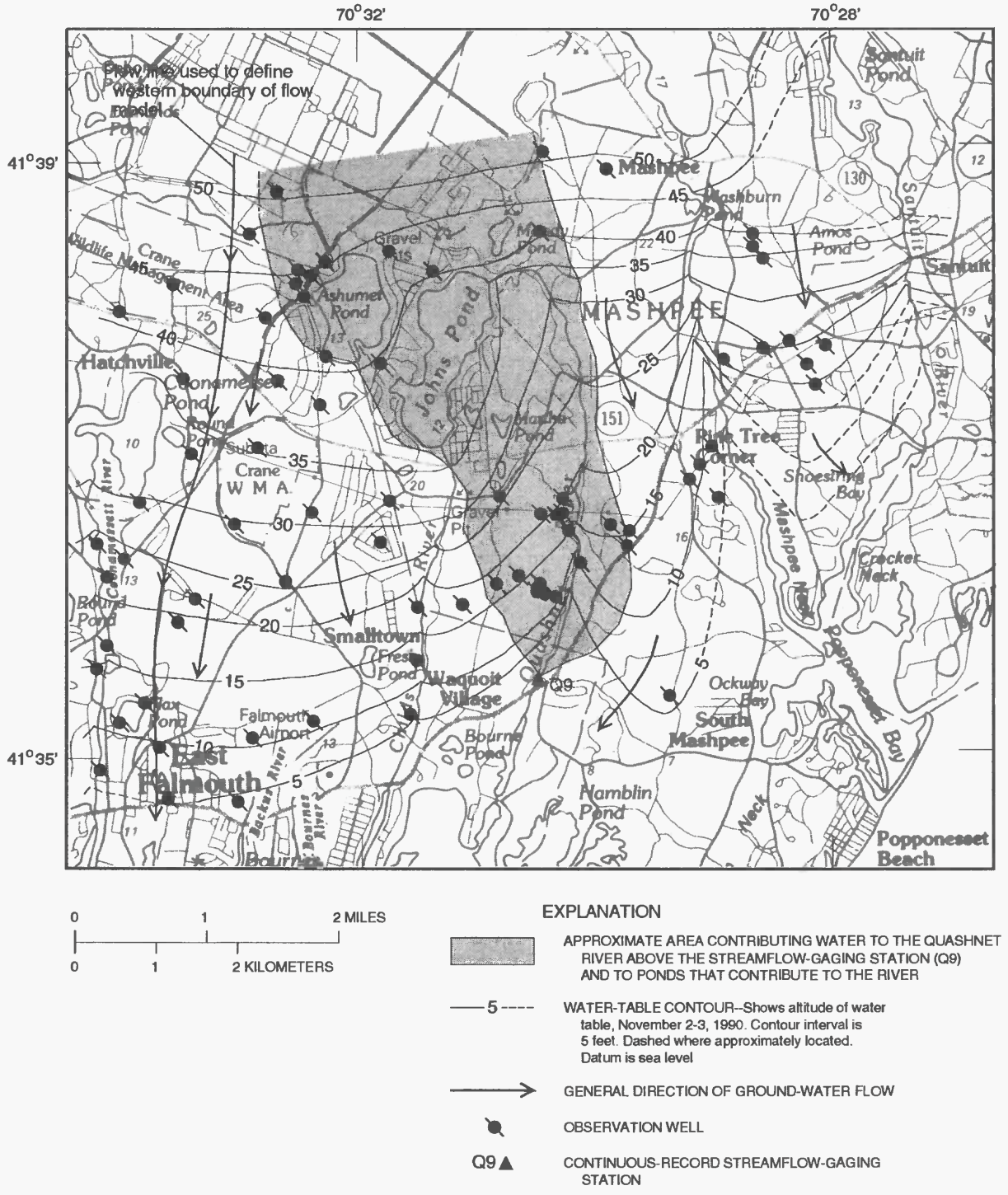


Figure 8.--Altitude and configuration of the water table and general direction of ground-water flow in the Quashnet River stream-aquifer system and approximate contributing area of the river, November 2-3, 1990.

0.7 to 0.9 ft below mean levels in November 1990 (fig. 9). Monthly mean water levels in wells were calculated for a 16-year period of record. The average of the two water-level measurements was within 0.2 ft of the annual mean level for each of these two wells. Although ground-water levels declined from May through November, the configuration of the water table and the direction of ground-water flow were nearly identical for the two measuring periods (figs. 7 and 8).

The water-table maps indicate that the Coonamessett, Childs, Quashnet, and Mashpee Rivers are gaining streams along most of their reaches (figs. 7 and 8). Water-table contours bend upgradient near the streams as ground water moves toward and discharges into the streams. Information is insufficient to determine if the Childs and Mashpee Rivers are gaining streams throughout their courses; near the stream origins at Johns and Mashpee ponds, water-table contours (figs. 7 and 8) appear to cut straight across the streams. Ground-water levels measured near site Q6 (location shown in fig. 1) on May 10, 1990, provide data on ground-water gradients near the Quashnet River. The gradient of the water table near the river, measured between the riverbank and a point 25 ft from the bank, was 0.07 ft/ft, which is significantly greater than the regional water-table gradient of about 0.0017 ft/ft. The increase in the gradient of the water table near the stream is the result of the convergence of ground-water-flow paths at the river. Thirteen vertical-head gradients measured beneath streamflow-gaging sites Q3 and Q8 (fig. 1) on February 22, May 10, and November 2, 1990, indicate a range of vertical gradients of 0.0 to 1.2 ft/ft; the mean gradient was 0.3 ft/ft.

Most ponds in the study area are connected hydraulically to the surrounding aquifer and are surface-water expressions of the ground-water system. Water-table contours bend upgradient on the upgradient ends of ponds and downgradient on the downgradient ends. This shape of the water table near the ponds directs ground-water inflow to the upgradient ends of ponds and pond-water outflow away from the downgradient ends into the aquifer; consequently, ponds are areas of ground-water throughflow. Because flow is directed toward and away from ponds, the flow rate through the aquifer near and within ponds is typically higher than in other parts of the aquifer. The ponds of Cape Cod are also areas of net ground-water recharge, because annual precipitation rates exceed annual potential evaporation rates from pond surfaces.

Water-table contours near Johns Pond indicate ground-water inflow along the northwest quadrant of the pond and pond outflow along the northeast, southeast, and southwest quadrants (figs. 7 and 8). Ground-water levels measured in a multilevel well cluster on the northern edge of the pond indicate an upward flow of ground water to the pond between an altitude of 0 and 50 ft below sea level; an upward gradient of 0.03 ft/ft is reported for the site (E.C. Jordan, 1991).

Seasonal variations in aquifer recharge cause the altitude of the water table to fluctuate 1 to 4 ft each year (LeBlanc and others, 1986, plate 2). Monthly mean water-level altitudes of observation wells FSW 167, MIW 19, and MIW 29 (for their periods of record) indicate the yearly cycle of the water table (fig. 9). The altitude of the water table is highest in the spring, typically in May, and lowest in the fall, typically in November. The yearly fluctuation of the water table tends to be least near Nantucket Sound, where the water-table altitude is nearly constant at about 0 ft. As can be seen in figure 9, ground-water levels do not fluctuate as greatly as streamflow of the Quashnet River because of the time required for water to travel through the unsaturated zone and because the saturated and unsaturated zones are able to store large quantities of water. In addition, pond levels tend to fluctuate less than ground-water levels because ponds have proportionally greater storage capacities than do aquifers. For example, during 1990 the altitude of Johns Pond fluctuated only 0.9 ft, whereas the water level in well FSW 167, which is at the same approximate altitude as Johns Pond, varied 2.5 ft (fig. 9).

The primary source of water to the aquifer is recharge by precipitation. Monthly mean recharge was estimated for the study area by use of a water-balance method that relates potential evapotranspiration empirically to climatic factors such as day length and air temperature and to the depth to which soil moisture is removed by plants (Thornthwaite and Mather, 1957). Surface runoff in the basin is negligible because of the high permeability of basin soils. LeBlanc (1984, p. 7) and LeBlanc and others (1986, plate 2) previously used this water-balance method to estimate yearly recharge rates of 17 to 22 in/yr to the Cape Cod aquifer system.

Monthly mean recharge to the aquifer was estimated by the Thornthwaite and Mather method (1957) on the basis of 60 to 80 years of climatic data (U.S. Department of Commerce, 1988-91) from the Hyannis and East Wareham weather stations located

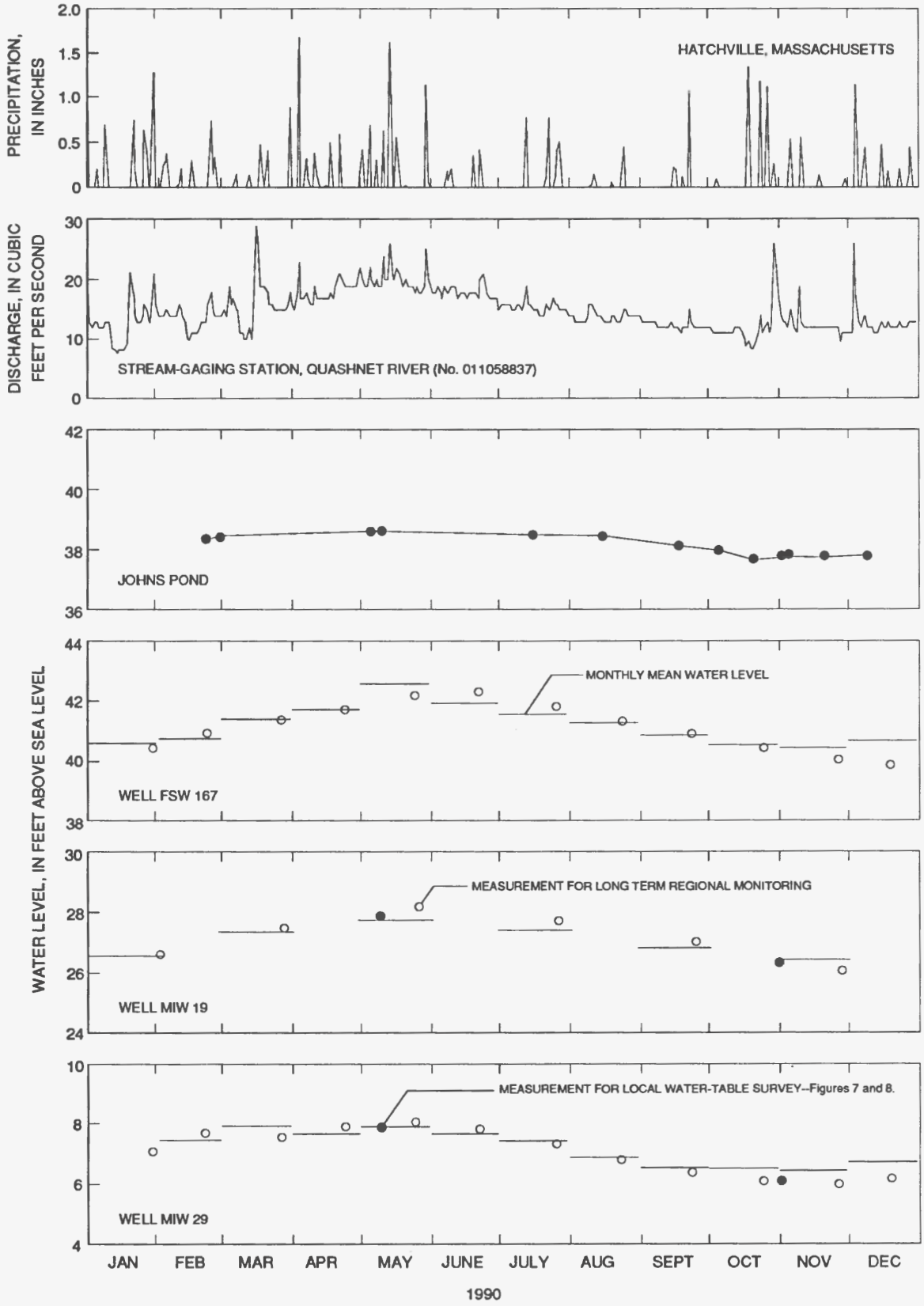


Figure 9.--Precipitation at Hatchville, Massachusetts, streamflow at the Quashnet River streamflow-gaging station (site Q9, fig. 1), and water level of Johns Pond, and in observation wells FSW 167, MIW 19, and MIW 29 in 1990. Streamflow and precipitation gaging station locations are shown in figure 1. Observation well locations are shown in figure 2.

13 mi east and 13 mi northwest of the Hatchville precipitation station (fig. 1), respectively. The calculations for East Wareham represent a period of record of 65 years. For Hyannis, 62 years of temperature data and 95 years of precipitation data were used in the calculations. Annual recharge of 20.3 and 22.9 in. was estimated for the Hyannis and East Wareham stations, respectively. These estimates are 45 to 48 percent of average annual precipitation measured at each station. As can be seen in figure 10, recharge to the aquifer occurs from November through May, and no recharge occurs from June through October. Monthly recharge estimates are consistently lower for the Hyannis station than for the East Wareham station. Because the Quashnet River stream-aquifer system lies about halfway between the two stations, an average of the two sets of estimates is considered to be a reasonable estimate of recharge in the vicinity of the Quashnet River. Therefore, an average annual recharge rate of 21.6 in. is estimated for the study area, which is consistent with those reported by LeBlanc (1984, p.7) and LeBlanc and others (1986, plate 2).

Additional sources of recharge to the aquifer in the study area are wastewater discharged to infiltration beds at the MMR wastewater-treatment facility (near well SDW 6, fig. 2) and return flows from septic systems and irrigation. The MMR wastewater-treatment facility discharged approximately 0.19 Mgal/d to the infiltration beds in 1990; nearly all of this discharge is assumed to have reached the water table. Septic-system return flow occurs in many parts of the basin; however, it is likely to be a net source of water to the aquifer only in areas that are served by public-water supplies. These areas are mostly along the populated bays and coastline of Nantucket Sound. Net recharge to the aquifer beneath areas in which there is both onsite pumping and septic-system return flow is assumed to be zero because the volume of return flow is likely to be nearly equal to the volume of ground water withdrawn. Water withdrawn for irrigation also is assumed to recharge the aquifer near where it is withdrawn; therefore, net recharge to the aquifer by irrigation return flow is assumed to be negligible.

Most ground water within the Quashnet River stream-aquifer system discharges to streams, marshes, and Nantucket Sound. The amount of ground water lost from the system as a result of evapotranspiration is assumed to be small because the water table is more than 10 ft below land surface in much of the study area. The net discharge of ground water from wells in the study area for commercial, residential, recreational, and agricultural use is small because most water is returned to the aquifer by onsite wastewater recharge and by return flows from irrigation. Three large-capacity supply wells are within the study area, however. Until recently, the town of Falmouth maintained a 0.86 Mgal/d production well near Fresh Pond (fig. 2), approximately 0.5 mi west of the Quashnet River. Currently (1993), water is not withdrawn from this well because of contamination. The MWD pumps approximately 0.36 Mgal/d from three wells 1.5 mi southeast of the Quashnet River outlet to Waquoit Bay (fig. 2).

The Quashnet River area is one of the few parts of Cape Cod where substantial streamflow occurs. The discharge of ground water to streams in this part of Cape Cod may be caused partly by the geohydrologic structure of the aquifer. The upper coarse-grained unit of sand and gravel becomes thinner and the underlying finer-grained units of lower hydraulic conductivity become thicker and shallower toward Nantucket Sound (fig. 3). This thinning of the coarse-grained unit may result in higher groundwater levels, which can, therefore, result in discharge of ground water to streams where the water table intercepts land surface, particularly in the valleys that trend roughly north to south.

Ground-water discharge from the aquifer also is controlled by the location of the interface between overlying freshwater and underlying saltwater. Freshwater is forced out of the flow system at the coast where the interface intersects sea-bottom sediments. The depth to the interface between freshwater and saltwater was measured at well MIW 26 (fig. 3) (LeBlanc and others, 1986). The interface location also has been calculated throughout the study area by use of a numerical model of the freshwater-saltwater flow system (Guswa and LeBlanc, 1985). The calculated interface along section A-A' is shown in figure 3.

Surface-Water Hydrology

There are four large kettle-hole ponds in the study area: Coonamessett, Ashumet, Johns, and Mashpee (fig. 1). These ponds range in surface area from 158 acres for Coonamessett Pond to 729 acres for the Mashpee Pond; mean depths of these ponds range from about 19 ft for Coonamessett Pond to about 26 ft for Ashumet and Mashpee Ponds (McCann, 1969). Natural or constructed streams lead from all but Ashumet Pond.

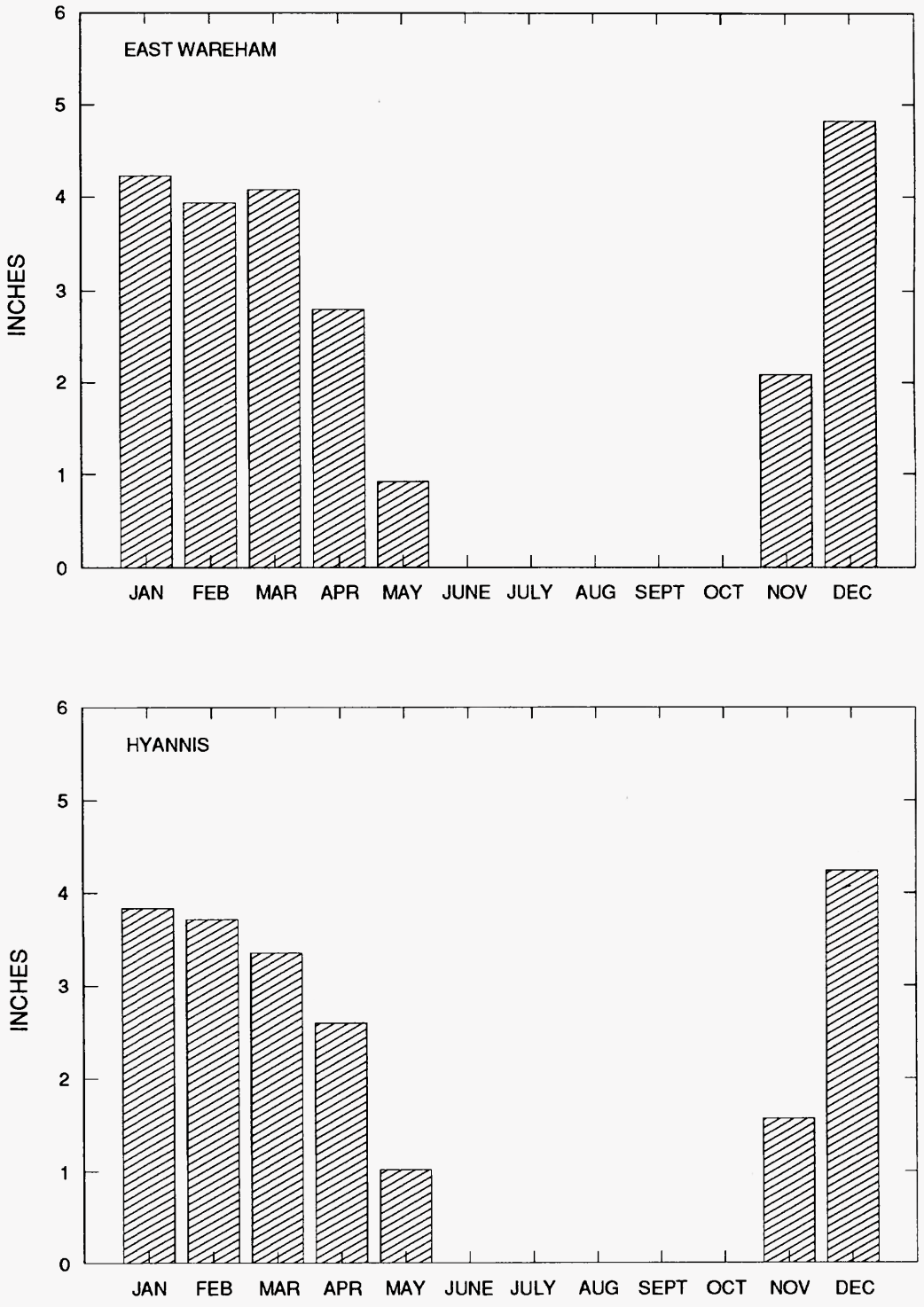


Figure 10.--Monthly mean ground-water recharge at East Wareham and Hyannis weather stations, Massachusetts, calculated by use of the method of Thornthwaite and Mather (1957).

Johns Pond, which is the source of the Quashnet and Childs Rivers, has a surface area of 323 acres and a mean depth of 19.4 ft (McVoy, 1982, p. 26). Johns Pond was formed from two kettle holes with maximum depths of 30 and 62 ft. The pond is fed primarily by ground-water inflow; the pond is fed also by surface water through a small stream that enters from the northeast. Surface-water outflow from Johns Pond is only to the Quashnet and Childs Rivers, and both outflow points are controlled. Outflow to the Quashnet River is regulated by a gate-controlled spillway that consists of a channel, a flume, and a sluiceway capable of drawing down the level of the pond to approximately 4 ft below its average altitude (McVoy, 1982, p. 24). The spillway was constructed to regulate the flow of water to cranberry bogs 0.4 mi downstream from the pond; the water is used for irrigation, frost protection, and harvesting. Outflow from Johns Pond to the Childs River is controlled through a 30-inch-diameter pipe and stop-plank structure; this outlet also is capable of drawing down the altitude of the pond by approximately 4 ft below its average altitude (McVoy, 1982, p.24).

The surface altitude of Johns Pond was measured periodically during 1990 (fig. 9). During this period, the altitude of the pond fluctuated between 37.7 and 38.6 ft, and its average altitude was 38.2 ft. Ground-water levels and streamflow within the Quashnet River stream-aquifer system were near average conditions in August and September 1990 (fig. 9). During these two months, the measured altitudes of Johns Pond were 38.5 ft (August 15) and 38.2 ft (September 17), respectively. These data indicate that the average altitude of the pond measured during this investigation (38.2 ft) is likely to be near the actual average altitude of the pond.

Streamflow from Johns Pond and pond-surface altitude measurements made during the investigation indicate that, typically, there is no discharge from the pond to the Quashnet River. Surface-water discharge from the pond to the river was observed only once during the investigation (May 10, 1990). Streamflow on that date was 2.3 ft^3/s at a point approximately 300 ft downstream from the spillway. The altitude of the pond on that date was 38.6 ft, or about 0.4 ft above the average altitude measured during the study. No discharge from the pond to the river was observed on February 22, 1990, at which time the altitude of the pond was 38.4 ft, slightly greater than the average altitude measured during this study. According to Donald L. Jermyn (oral commun., 1990 and 1992), operator of the spillway and the downstream cranberry bog, the pond typically does not flow to the Quashnet River from approximately June to March, but may flow to the river when pond altitudes are high in the spring.

There are six rivers within the study area. From west to east, these are the Coonamessett, Backus, Bournes, Childs, Quashnet, and Mashpee Rivers (fig. 1). Of these six rivers, only the Quashnet River has been gaged; the gaging station is approximately 1,900 ft upstream from the mouth at Waquoit Bay (site Q9, fig. 1). Streamflow at the gaging station has been measured continuously since 1988; the mean streamflow was 13.8 ft^3/s for October 1, 1988, through September 30, 1991 (U.S. Geological Survey, 1991). This mean streamflow is likely close to the long-term average flow at this site because precipitation in the study area and ground-water levels at observation wells FSW 167, MIW 19, and MIW 29 (locations shown in fig. 2) were close to their period-of-record mean values during these 3 years (see fig. 9 for 1990 data).

Monthly mean streamflow in the Quashnet River for the 3-year period was above the mean flow during March through June and below the mean flow during July through February (fig. 11). Maximum monthly mean streamflow for the 3-year period occurred in May (17.3 ft^3/s), and minimum monthly mean streamflow occurred in October (11.8 ft^3/s).

Comparison of the precipitation record measured at the Hatchville weather station (location shown in fig. 1) (U.S. Department of Commerce, 1988-91) to the streamflow record from the Quashnet River gaging station (site Q9, fig. 1) for 1990 indicates that most high flows in the river were caused by precipitation (fig. 9). For example, the high flows of January, April, and December are clearly related to precipitation. The high flows of late October through early November, however, were caused both by precipitation and by the rapid release on October 27, 1990, of water that had been stored in the cranberry bog for frost protection of the cranberry plants and for harvesting. Baevsky (1991, p. 2) notes that the autumn 1988 spawning run of the sea-run brown trout coincided with the release of water from the cranberry bog after harvest and that peak spawning activity coincided with a period of peak discharge within spawning reaches between sites Q5 and Q6 (fig. 1). The spawning run of the brown trout in autumn 1990, however, was observed to precede the release of water stored in the cranberry bog (Francis Smith, Trout Unlimited, oral commun., 1990).

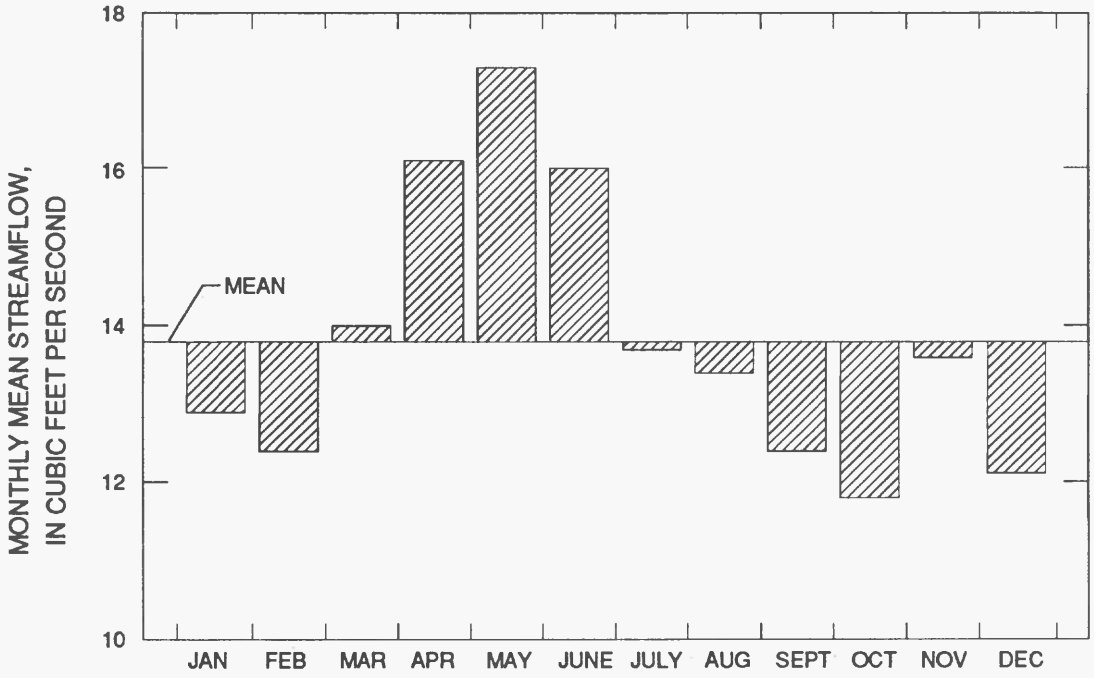


Figure 11.--Deviation of the monthly mean streamflow from the mean streamflow (13.8 cubic feet per second) measured at the Quashnet River gaging station, October 1988-September 1991.

The relation between streamflow and percentage of time that flow is equaled or exceeded for the Quashnet River at the streamflow-gaging station (site Q9, fig. 1) for October 1988 through September 1991 (water years 1989-91) is shown in figure 12. Also shown in figure 12 is the relation between streamflow and percentage of time that flow is equaled or exceeded for the Sevenmile River near Spencer, Worcester County, Mass., for water years 1989-91 and for water years 1961-91 (period of record). The Sevenmile River is considered reasonably representative of unregulated streams in eastern Massachusetts (R.A. Gadoury, U.S. Geological Survey, oral commun., 1992). Two curves are shown for the Sevenmile River to indicate that the range of streamflow during water years 1989-91 is representative of typical streamflow conditions of the river. Mean streamflow of the Sevenmile River for 1961-91 was 14.8 ft^3/s, which is similar to the mean streamflow of the Quashnet River (13.8 ft^3/s) for 1989-91. The standard deviation of daily streamflows, however, was 18.9 ft^3/s for the Sevenmile River and only 3.6 ft^3/s for the Quashnet River. The three curves highlight the contrast between the relatively constant streamflow of the Quashnet River, which is assumed to be typical of streams on the broad outwash plains of Cape Cod, and the more variable streamflow of rivers in valley aquifer settings typical of New England.

The drainage basins of the Sevenmile River and the Quashnet River differ in the percentages of outwash and till covering each of them. The drainage area of the Sevenmile River streamflow-gaging station is 8.7 mi^2, of which 87 percent is till and 13 percent is outwash (K.G. Ries, U.S. Geological Survey, oral commun., 1992). The drainage area of the Quashnet River at the streamflow-gaging station is 5.0 mi^2, of which 100 percent is outwash. Several variables might affect the rate of ground-water discharge to a river, including geohydrologic characteristics of the aquifer sediments and underlying bedrock, depth to the water table, area of ponds and wetlands, rate of evapotranspiration, slope of land surface, vegetative type and extent of coverage, climate, and the percentage of outwash covering a drainage basin (Lapham, 1988, p.12); the percentage of outwash is a particularly important control on streamflow rates and duration in Massachusetts. The relative stability of streamflow in the Quashnet River is likely the result of the large percentage of outwash in the river

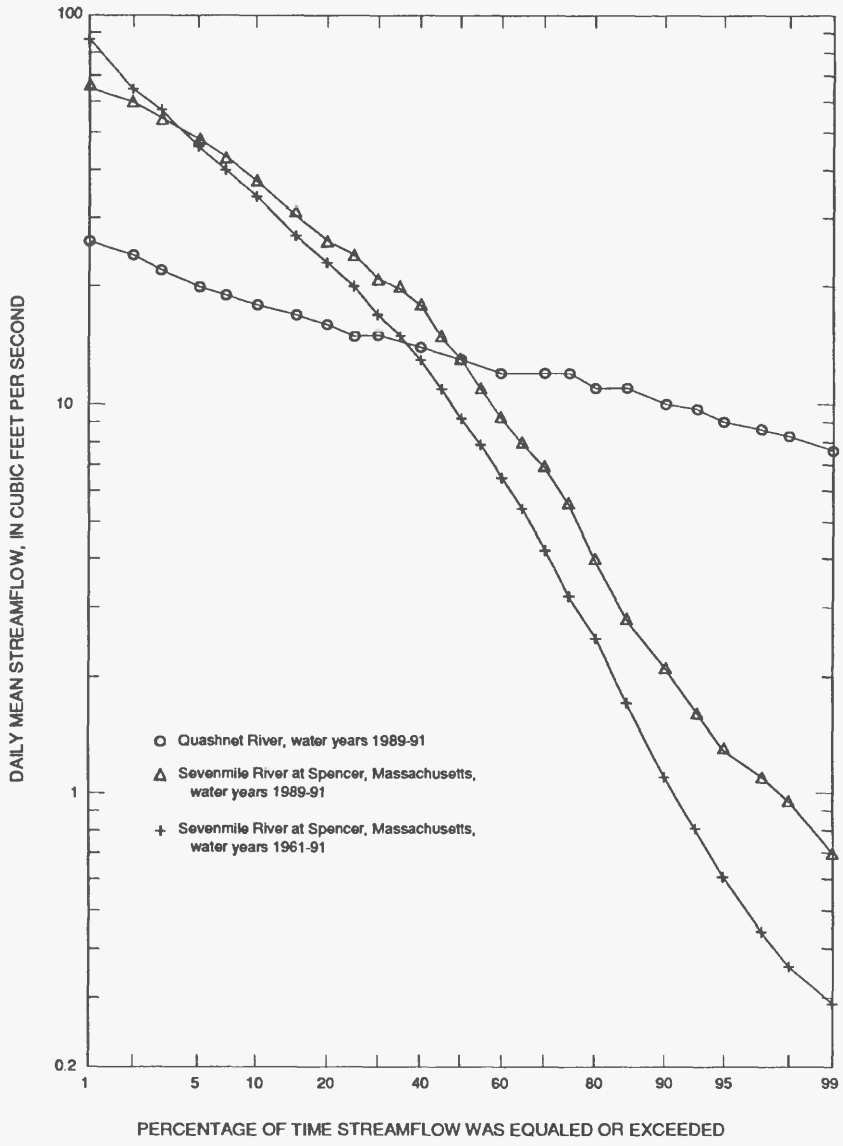


Figure 12.--Relation between streamflow and percentage of time the indicated flow was equaled or exceeded for the Quashnet and Sevenmile Rivers, Massachusetts, for water years 1989-91, and for the Sevenmile River for water years 1961-91.

basin. The reduction in streamflow variability results from the capacity of the outwash to store large volumes of water and to release that water slowly during periods of little to no recharge. During heavy rains, drainage basins with large percentages of till produce high rates of surface runoff because the till is unable to absorb and store the excess precipitation; consequently, streams in basins covered largely by till are characterized by high streamflows. By contrast, most rainfall on basins with large percentages of outwash infiltrates and is stored in the outwash; consequently, streamflows in basins covered mostly by outwash do not increase as dramatically in response to rain as do streamflows in basins covered mostly by till.

During periods of drought, extremely low streamflows are produced in drainage basins with large percentages of till because little water is released from storage within the till; however, streams in drainage basins with large percentages of outwash, such as the Quashnet River, are able to maintain higher streamflows because of the large volume of

water released from storage in the outwash. In summary, streamflow on Cape Cod remains relatively constant in comparison to streamflow in most other areas of New England because of the large infiltration and storage capacities of the outwash deposits, which tend to damp the effect of climatic variability on streamflow.

Measurements of streamflow were made on February 22, May 10, and November 2, 1990, and July 17, 1991, (fig. 13 and table 1) at as many as eight sites on the Quashnet River, from its origin at Johns Pond to the streamflow-gaging station. These sites are shown in figure 1 as Q1 through Q9. Measurements were made by conventional current-meter methods at 20 or more stations across each section. These measurements were made to determine rates of ground-water inflow along different reaches of the river for high, low, and average streamflow conditions. Measurements also were made to determine stream width, depth, and streamflow velocity for various streamflow conditions. Highest streamflows were measured on May 10, 1990, and lowest streamflows were measured on July 17, 1991 (table 1). Streamflow at the gaging station (site Q9) was near its mean of 13.8 ft^3/s on February 22 and November 2, 1990. Measurements made on November 2, 1990, however, likely were affected by the release of stored water from the cranberry bog 6 days earlier, on October 27.

There are two sources of ground water to the Quashnet River. The first is direct seepage to the river through streambed sediments. The second source is inflow from small rivulets that begin as springs flowing from the embankments that bound the floodplain; the springs form where the land surface intersects the water table. These rivulets provide feeding areas and protection from predation for developing juvenile fish (J.D. Bergin, oral commun., 1991). No distinction was made during this investigation between the streamflow of the Quashnet River derived from seepage through the streambed and that derived from spring discharge.

The rate of ground-water inflow to the Quashnet River varies along the river's length. This variable rate is illustrated by changes in the slope of the streamflow-distance relations plotted in figure 13. Highest rates of ground-water inflow occur between Johns Pond and site Q3 and between sites Q5 and Q8. Part of the area of high rates of inflow between sites Q5 and Q8 coincides with the section of the river that supports the largest number of spawning sites (Q5 to Q7). Controls on the location and volume of inflow to the river include (1) differences in hydraulic conductivity of the streambed sediments, (2) variability in the streambed altitude relative to the altitude of the water table, and (3) changes in the water-table gradient near the stream.

The stream width, average stream depth, cross-sectional area, average stream velocity, and streamflow measured at the eight sites along the Quashnet River on the four dates are listed in table 1. Although no specific determinations have been made by the Massachusetts Division of Fisheries and Wildlife regarding requirements of the sea-run brown trout, general guidelines have been developed and used to evaluate habitats (J.D. Bergin, written commun., 1992). First, stream width should not decrease to less than 50 percent of the full width of the stream from bank to bank. This requirement is met for all widths measured at sites downstream from the fish ladder (sites Q5-Q9), which is the furthest point upstream where active spawning has been observed, if the widths measured during the highest flows (May 10, 1990) are assumed to represent full width conditions. Second, stream depth should not be less than 0.5 ft; the data listed in table 1 indicate that the average depth of the stream for all four measurements at all sites downstream from the fish ladder is greater than 0.5 ft. The third general guideline is that stream velocity should be near 1 ft/s in spawning areas because, at this velocity, the streambed tends to be composed of sand and gravel, which is needed for the construction of fish-egg nests (redds); the average velocity at all sites downstream from the fish ladder for all measurements except those of July 17, 1991, was greater than 1 ft/s, and all but one measurement taken on July 17, 1991 (Q7) were approximately 1 ft/s (table 1).

Finally, streamflow per surface-drainage area should meet the U.S. Fish and Wildlife Service Aquatic Base Flow guidelines (J.D. Bergin, written commun., 1992): 1.0, 4.0, and 0.5 $(ft^3/s)/mi^2$ during the fall (September-December), winter and early spring (January-March), and late spring and summer (April-August), respectively. Because the drainage area of the Quashnet River basin at the streamflow-gaging station is 5.0 mi^2, these guidelines translate to streamflows at the gaging station of 5, 20, and 2.5 ft^3/s during the fall, winter and early spring, and late spring and summer, respectively. Although these guidelines are met on the Quashnet River during the fall and late spring and summer, they are not met under current streamflow conditions during the winter and early spring (fig. 11), even though the river

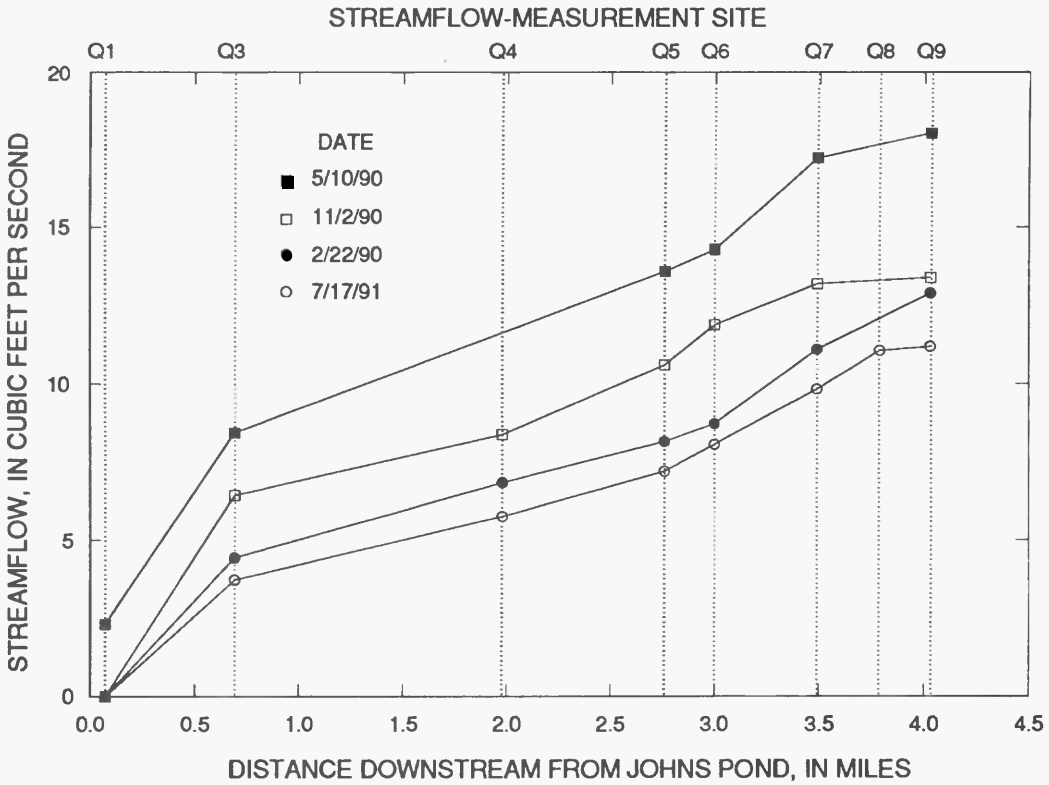


Figure 13.--Streamflow measured at selected sites along the Quashnet River, 1990-91. Location of sites shown on figure 1.

supports a highly productive trout habitat. A more detailed understanding of the relation between streamflow conditions and trout ecology of the river is required before the response of the trout habitat to changing streamflow conditions can be fully described.

Streamflow measurements on the remaining five streams in the study area have been made intermittently. Measurements were made on four of the five rivers within 2,100 ft of the mouth of each river on August 9, 1989, when the Quashnet River streamflow and ground-water levels in observation wells FSW 167, MIW 19, and MIW 29 were near their average conditions. Streamflows were 8.2, 2.3, 1.5, and 6.0 ft^3/s for the Coonamessett, Backus, Bournes, and Childs Rivers, respectively, on this date (U.S. Geological Survey, 1991, p. 178-179). Only one streamflow measurement is recorded for the Mashpee River; this measurement was made 1,250 ft upstream from the mouth of the river on August 4, 1978, and was 15.5 ft^3/s (U.S. Geological Survey, 1991, p. 180). Streamflow measured on this date at the location of the present gaging station on the Quashnet River, 19.3 ft^3/s, was exceeded only 7 percent of the time (fig. 12) during the period of record for the station (1989-91). Therefore, the recorded streamflow for the Mashpee River is probably significantly greater than its mean streamflow. If yearly cycles of discharge to the streams in the study area are similar and periods of high and low streamflow are consistent among all six rivers, then these measurements indicate that the Quashnet River has a greater streamflow than other streams in the study area.

HYDROLOGIC RESPONSES OF THE QUASHNET RIVER STREAM-AQUIFER SYSTEM TO PROPOSED GROUND-WATER WITHDRAWALS

A three-dimensional finite-difference model was used to simulate ground-water flow in the Quashnet River stream-aquifer system for current hydrologic conditions and to evaluate the hydrologic responses of the system to proposed ground-water withdrawals. A

Table 1.--*Channel and flow characteristics at selected sites along the Quashnet River, 1990-91*

[--, no data; mi, mile; ft, foot; ft^2, square foot; ft/s, foot per second; ft^3/s, cubic foot per second]

Date	River mile[1] (mi)	Width (ft)	Average Depth (ft)	Area (ft^2)	Average Velocity (ft/s)	Streamflow (ft^3/s)
Site Q1--Outlet from Johns Pond						
02-22-90	0.0	--	--	--	--	No flow
05-10-90		6.5	0.43	2.8	0.82	2.3
11-02-90		--	--	--	--	No flow
07-17-91		--	--	--	--	No flow
Site Q3--Within Cranberry Bog						
02-22-90	.6	11.9	1.22	14.5	.30	4.4
05-10-90		20.1	1.71	34.3	.25	8.4
11-02-90		17.6	1.45	25.6	.25	6.4
07-17-91		19.7	.96	19.0	.20	3.7
Site Q4--Downstream from the culvert under Route 151						
02-22-90	2.0	6.7	.99	6.6	1.03	6.8
11-02-90		17.1	.82	14.1	.60	8.4
07-17-91		11.5	.70	8.0	.71	5.7
Site Q5--Downstream from the fish ladder, the upper boundary of active trout spawning						
02-22-90	2.7	8.2	.78	6.4	1.27	8.1
05-10-90		10.0	.73	7.3	1.86	13.6
11-02-90		8.3	.81	6.7	1.58	10.6
07-17-91		8.2	.90	7.4	.97	7.2
Site Q6--Site 2, within active spawning reach						
02-22-90	3.0	8.8	.74	6.5	1.34	8.7
05-10-90		10.9	.89	9.7	1.47	14.3
11-02-90		9.7	.90	8.7	1.37	11.9
07-17-91		7.8	1.10	8.6	.93	8.0
Site Q7--Site C, within active spawning reach						
02-22-90	3.5	13.4	.60	8.0	1.39	11.1
05-10-90		17.2	.74	12.7	1.35	17.2
11-02-90		14.2	.70	9.9	1.33	13.2
07-17-91		14.4	1.01	14.6	.67	9.8
Site Q8--New site, downstream of primary spawning reach						
07-17-91	3.8	14.9	.85	12.7	.87	11.1
Site Q9--Continuous streamflow-gaging station, with some spawning activity						
02-22-90	4.0	9.0	1.04	9.4	1.37	12.9
05-10-90		9.9	1.14	11.3	1.59	18.0
11-02-90		10.0	.96	9.6	1.40	13.4
07-17-91		9.5	.63	6.0	1.87	11.2

[1]River mile refers to the number of miles downstream from the river's origin at Johns Pond.

three-dimensional model was used to simulate the system because of the lithology and three-dimensional nature of ground-water flow within the aquifer. Analytical methods and two-dimensional numerical models would not have provided either an accurate representation of the stream-aquifer dynamics within reaches of the Quashnet River that support trout spawning or an accurate determination of the areas contributing water to simulated wells or the river.

A finite-difference model is a computer program that calculates hydraulic heads on the basis of the hydraulic properties, boundary conditions, and hydraulic stresses within the aquifer. The modular finite-difference ground-water-flow modeling code (MODFLOW) developed by McDonald and Harbaugh (1988) was used. This model was chosen because it (1) can be used to simulate three-dimensional transient ground-water flow, (2) contains several options for the specification of boundary conditions, (3) has a streamflow-accounting subroutine (Prudic, 1989), (4) is well documented and widely applied in ground-water investigations, and (5) has a particle-tracking postprocessor (Pollock, 1988 and 1989). The particle-tracking program (MODPATH), which tracks water particles through the simulated flow system, was used to estimate steady-state contributing areas of the Quashnet River and simulated wells in the area of proposed pumping.

Simulation of the stream-aquifer system required several simplifying assumptions. The three major assumptions follow:

1. The aquifer is composed of unconsolidated glacial deposits and is underlain by an impermeable crystalline bedrock. The assumption that bedrock is impermeable is based on the very low hydraulic conductivity reported for crystalline rocks (Davis and De Weist, 1966, p. 164 and 320; Freeze and Cherry, 1979, p. 29; Marsily, 1986, p. 78).

2. The aquifer can be represented by a two-layer model, in which a top layer of sand and gravel overlies a bottom layer of fine sand, silt, and clay; borehole logs indicate that the glacial deposits may be characterized by these two major lithologies (figs. 3 and 4).

3. Hydraulic head may differ between the upper and lower layers; however, there are no vertical gradients within each layer. This assumption presumes horizontal flow within each layer, which is consistent with nearly horizontal flow observed within the upper sand and gravel deposits. Although vertical flow occurs within each lithologic layer near streams, ponds, wells, and the coast, the influence of these hydrologic features on ground-water flow is generally localized.

Description of Model

The ground-water-flow model of the Quashnet River stream-aquifer system extends from just west of the Coonamessett River to the Mashpee River and from Nantucket Sound to approximately 1 mi north of Ashumet and Johns Ponds (figs. 1 and 14). The lateral extent of the model was chosen to include an area large enough to minimize the effect of model boundary conditions on heads calculated near the proposed well sites and the Quashnet River.

Model Grid

The modeled area, 44.4 mi^2, was subdivided into a grid of rectangular cells arranged in 82 rows and 81 columns. Grid cells range from a minimum of 200 by 200 ft to a maximum of 660 by 1,220 ft. Grid cells are smallest near site MIW 108 and the Quashnet River, so that the aquifer and river are represented in detail sufficient to determine their response to proposed pumping. The model is aligned with regional models of the Cape Cod flow system developed by the USGS (Guswa and LeBlanc, 1985; J.P. Masterson, written commun., 1992).

Grid cells in each of the two model layers are of variable thickness. Cells in the top layer extend from the water table to the contact between coarse-grained and underlying fine-grained deposits. The altitude of the water table ranges from 55 ft near the northern boundary of the model to 0 ft at the coasts; the altitude of the contact between coarse-grained and underlying fine-grained deposits ranges from 5 to -95 ft. Total thickness of cells in the top layer ranges from 10 to 146 ft. Cells in the bottom layer extend from the contact between coarse-grained and underlying fine-grained deposits to either the contact with underlying bedrock or the contact between freshwater and underlying saltwater as calculated by Guswa and LeBlanc (1985). Cells in the bottom layer extend to altitudes ranging from -10 to -350 ft. Total thickness of cells in the lower layer ranges from 20 to 345 ft.

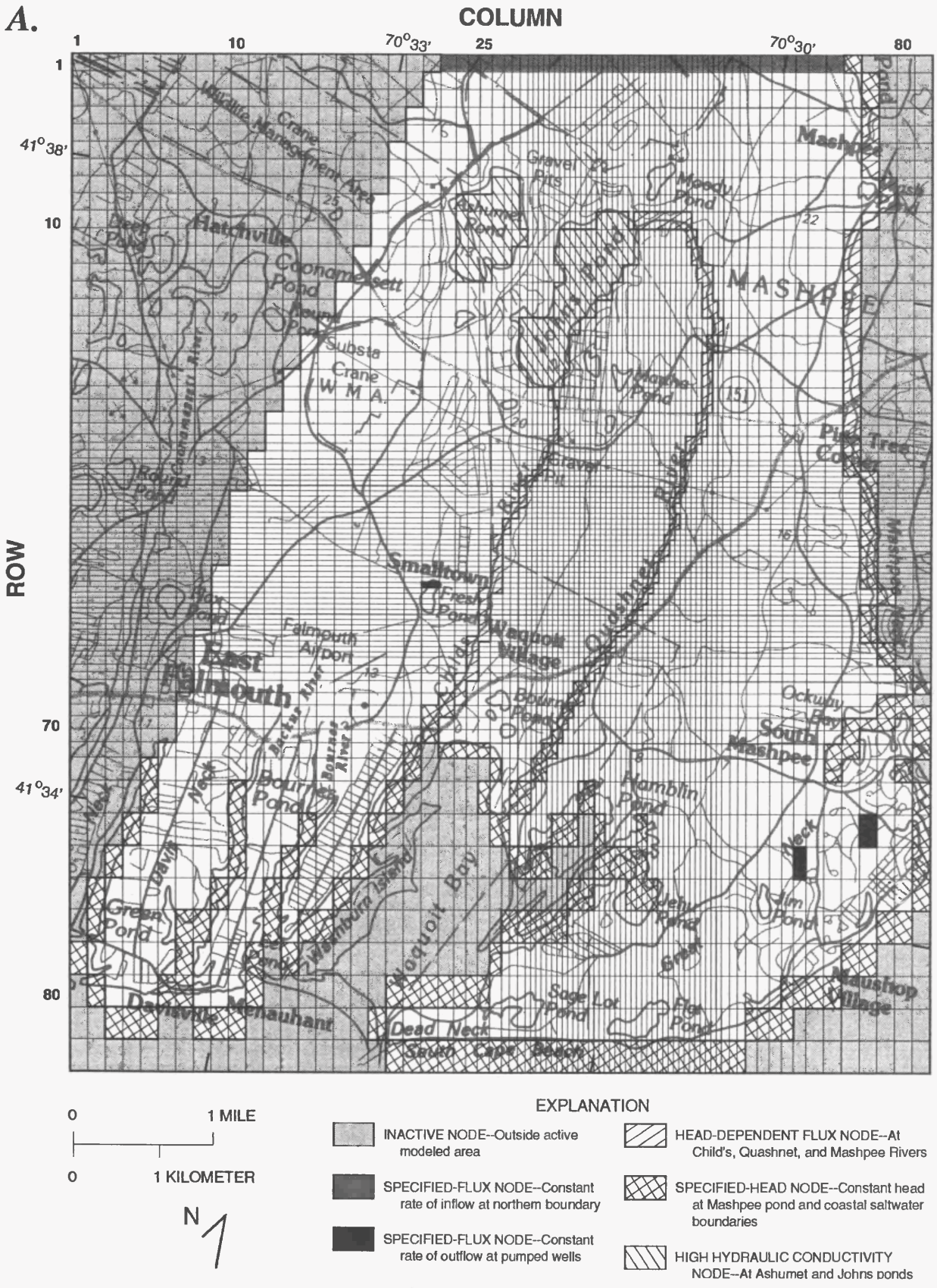


Figure 14.--Finite-difference grid and boundary conditions used to model the Quashnet River stream-aquifer system: (*A*) model layer 1, and (*B*) model layer 2.

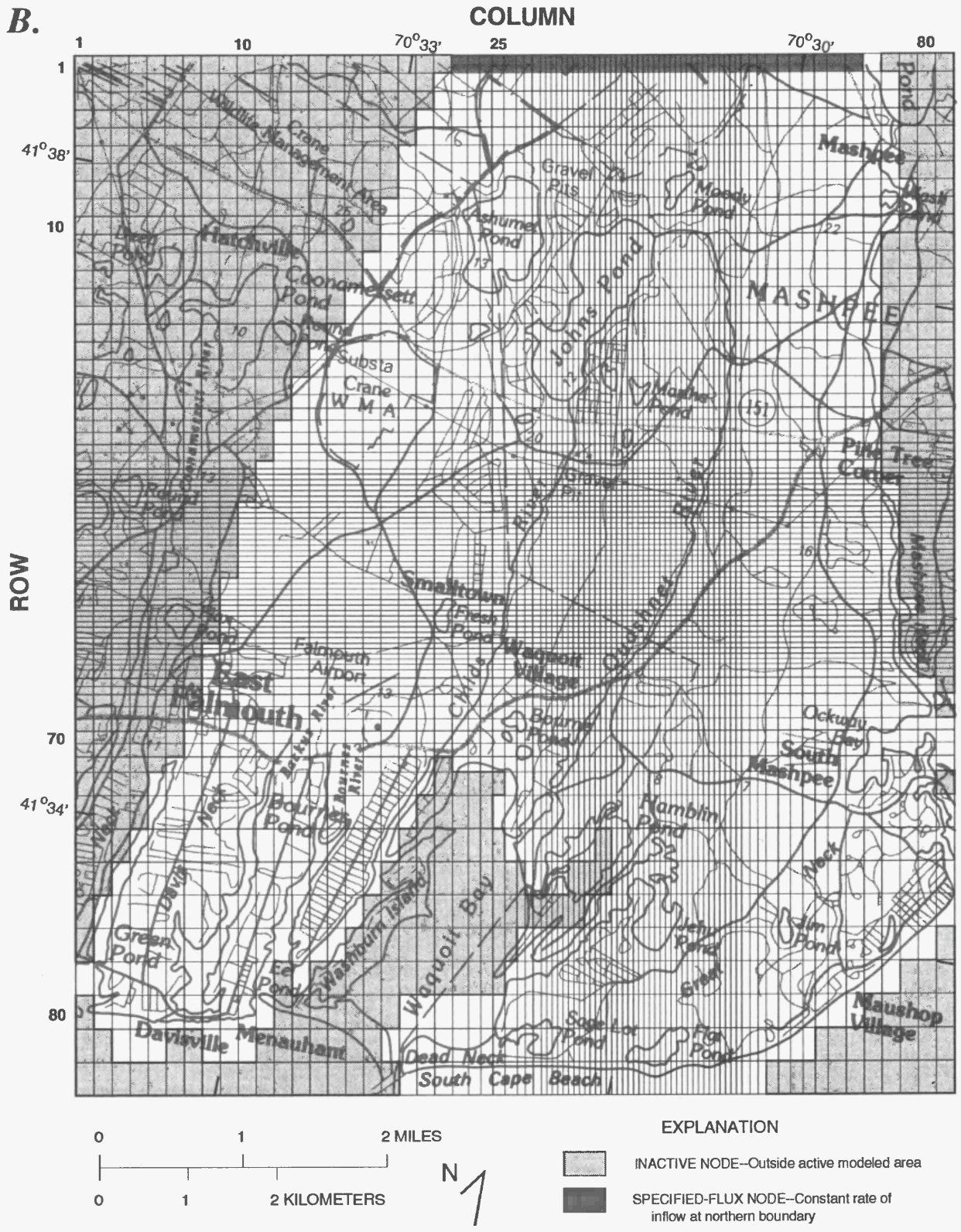


Figure 14.--Finite-difference grid and boundary conditions used to model the Quashnet River stream-aquifer system: (*A*) model layer 1, and (*B*) model layer 2--Continued.

Hydraulic Conductivity and Vertical Conductance

Horizontal hydraulic conductivities reported for coarse-grained sand and gravel west of the Childs River are generally higher than those reported for sediments east of the Childs River. Therefore, uniform values of horizontal hydraulic conductivity of the upper layer of the model of 300 and 240 ft/d were specified for areas west and east of the Childs River, respectively. The value used for the area east of the Childs River, 240 ft/d, is equal to that determined for the upper, coarse-grained zone of the finite-element radial-flow model used to simulate the aquifer test at well MIW 108. Grid cells in the upper layer that underlie Ashumet and Johns Ponds were assigned a very high horizontal hydraulic conductivity of 50,000 ft/d to simulate their damping effect on the flow system. A uniform value of horizontal hydraulic conductivity of 40 ft/d was specified for the lower layer of the model; this value is equal to that determined for the lower, fine-grained zone of the finite-element radial-flow model.

Vertical conductance was specified between vertically adjacent nodes of the model (McDonald and Harbaugh, 1988, p. 5-11) according to the following equation:

$$\frac{1}{\frac{(h_1 - b_1)}{2K_{v_1}} + \frac{(b_1 - b_2)}{2K_{v_2}}}, \qquad (2)$$

where K_{v_1} is the vertical hydraulic conductivity of the upper layer of the model (feet per day),

K_{v_2} is the vertical hydraulic conductivity of the lower layer of the model (feet per day),

h_1 is the head calculated for the upper layer of the model (feet),

b_1 is the altitude of the bottom of the upper layer of the model (feet), and

b_2 is the altitude of the bottom of the lower layer of the model (feet).

Vertical hydraulic conductivities used in equation 2 were based on ratios of horizontal to vertical hydraulic conductivity obtained through calibration of the finite-element radial-flow model. Uniform ratios of 3:1 and 50:1 were used for the upper and lower layers of the model, respectively. Therefore, vertical hydraulic conductivities of 100 and 80 ft/d were specified for the upper layer for areas west and east of the Childs River, respectively. A uniform value of vertical hydraulic conductivity of 0.8 ft/d was specified for the lower layer of the model. Vertical conductance and h_1 were calculated iteratively during model calibration. An initial estimate of h_1 was determined from water-table maps of the study area, from which an initial estimate of vertical conductance was made. A second estimate of h_1 was then determined from results of the first model run, and vertical conductance was recalculated. The values of h_1 and vertical conductance were adjusted iteratively in subsequent calibration runs until the difference between calculated heads for consecutive model runs was small. Vertical conductance in the calibrated model ranged from 4.6×10^{-3} to 1.6×10^{-1} day^{-1}. Vertical conductance was then held constant for all subsequent model simulations.

Boundary Conditions and Stresses

Hydraulic boundaries of the Quashnet River stream-aquifer system include ground-water inflow from adjacent areas of the aquifer, streams, ponds, wells, coastal saltwater bodies, the water table, the contact between unconsolidated sediments and underlying bedrock, and the interface between freshwater and underlying saltwater. The northern boundary of the model is an area of ground-water inflow from adjacent, upgradient parts of the aquifer (fig. 14). A specified-flux boundary condition was used to simulate the inflow of ground water across the boundary. Inflow across the boundary was calculated by three methods. First, an inflow rate of 7.7 ft^3/s was obtained by multiplying the area between the northern boundary of the model and the water-table divide by the estimated areal recharge rate for the study area (21.6 in/yr). The location of the water-table divide was obtained from a water-table map of the Cape Cod flow system (LeBlanc and others, 1986). Second, an inflow rate of 12.7 ft^3/s was obtained from Darcy's law by use of observed hydraulic gradients along the northern boundary, estimates of horizontal hydraulic conductivity of the upper and lower layers of the model, and the cross-sectional area of the upgradient face of each cell along the boundary. Finally, an inflow rate of 6.4 ft^3/s was obtained from Darcy's law by use of calculated hydraulic gradients and model-derived estimates of horizontal hydraulic conductivity from the regional-scale ground-water-flow model developed for the study area by Guswa and LeBlanc (1985) and the cross-sectional area of the upgradient face of each cell along the northern boundary. The average of the

three estimated rates (8.9 ft^3/s) was distributed to the active 110 cells along the northern boundary in proportion to the cross-sectional area and horizontal hydraulic conductivity of each cell.

Coastal saltwater bodies along the southern and eastern boundaries of the model, such as Nantucket Sound, Waquoit Bay, and Popponesset Bay (fig. 1), are areas of nearly constant ground-water levels. These boundaries were simulated by means of a specified-head boundary condition in the top layer of the model. Because the boundaries consist of saltwater, the saltwater heads were converted to equivalent freshwater heads by dividing the thickness of the saltwater body within each specified-head cell by 40.0, the ratio of the specific weight of freshwater (1.000 gm/cm^3) to the difference between the specific weights of saltwater (1.025 gm/cm^3) and freshwater. The thickness of the saltwater body for each cell was obtained from bathymetric maps. Only saltwater areas along the coastline were simulated (fig. 14) because all discharge to the saltwater bodies was assumed to occur at the coast. This assumption is based on the location of the interface between freshwater and saltwater in the study area as reported by LeBlanc and others (1986, plate 3) who indicate that most freshwater discharges at the coastline.

The western extent of the active model area was defined by a ground-water-flow line drawn perpendicular to water-table contours within the study area for November 2-3, 1990 (fig. 8). The flow line was simulated as a streamline, or no-flow, boundary across which ground water cannot flow. In the natural system, however, ground-water-flow lines may shift in response to changes in ground-water levels.

The bottom of the model was defined by the contact between either unconsolidated deposits and underlying crystalline bedrock or freshwater and underlying saltwater. Flow across these contacts was assumed to be insignificant and a no-flow boundary condition was used in the model to simulate the contacts. The location of this no-flow boundary was obtained from a map showing the altitude of the bedrock surface for the study area (B.D. Stone, written commun., 1991) and from the location of the freshwater-saltwater interface calculated by use of a regional flow model (Guswa and LeBlanc, 1985).

The northeast corner of the model intersects Mashpee Pond, a large pond whose altitude is assumed to remain constant for the purpose of this study. Although the altitude of the pond fluctuates seasonally, the effects of this fluctuation on flow in the Quashnet River and ground-water levels near the proposed well sites is likely negligible. A specified-head boundary condition equal to 55 ft was used to simulate Mashpee Pond.

Along the eastern boundary of the model is the Mashpee River, which receives ground-water discharge. The volume of discharge to the river depends on the hydraulic gradient between the aquifer and the river and the hydraulic conductance of the glacial and postglacial deposits that compose the streambed. The river was simulated by means of a head-dependent flux boundary, in which the altitude of the streambed and the hydraulic conductance of the streambed deposits were specified for each cell in which the river is located. The river was simulated such that it could only receive ground-water discharge; there was no flow of water between the river and aquifer if the calculated head in the aquifer fell below the streambed altitude. The streambed altitude was determined from topographic maps. The hydraulic conductance of the streambed deposits (C_S) was calculated for each cell according to the relation:

$$C_S = \frac{KWL}{M}, \qquad (3)$$

where K is the vertical hydraulic conductivity of streambed deposits (feet per day),
W is the width of the river in the cell (feet),
L is the length of the river in the cell (feet), and
M is the distance over which the head loss between aquifer and river occurs (feet).

The hydraulic conductivity of streambed deposits was assumed to be 80 ft/d, which is equal to the vertical hydraulic conductivity of the upper layer of the model east of the Childs River. This value was used because it was assumed that most ground-water discharge to the river occurs in the vertical direction. The width of the river was assumed to equal 5 ft. The assumed distance over which the head loss between aquifer and river occurs, M, was equal to one-half the thickness of the top layer, which was calculated as the distance between the bottom of the upper layer of the model and the streambed altitude in each grid cell. This assumption was made so that calculated head losses between the aquifer and river were consistent with the block-centered finite-difference scheme of the model.

To determine if proposed ground-water withdrawals will cause induced infiltration from the Childs and Quashnet Rivers, the rivers were simulated as head-dependent flux boundaries that can either gain or lose water to the surrounding aquifer, by use of the stream-aquifer computer module developed by Prudic (1989). Streambed altitude and thickness, river stage, and hydraulic conductance of streambed deposits were specified for each cell in which the rivers are located.

Streambed altitudes for the Childs and Quashnet Rivers were obtained from topographic maps and field surveys. Two sites were surveyed on the Childs River and nine sites were surveyed along the length of the Quashnet River (sites Q1 to Q9, fig. 1). A streambed thickness of 1 ft was assumed for both rivers. A uniform river stage of 0.5 ft was assumed for the Childs River, but the stage was varied for the Quashnet River; river-stage estimates for the Quashnet River were obtained from field measurements. Hydraulic conductance of streambed deposits underlying the Childs River was obtained for each cell by use of the same method that was used for the Mashpee River. Estimates of hydraulic conductance for the Quashnet River, however, were determined by use of field-measured characteristics where possible. The leakage between the river and the underlying aquifer between two streamflow-measurement sites is obtained from Darcy's Law (Prudic, 1989, p. 7):

$$Q_l = \frac{KWL_r\,(H_s - H_a)}{M}, \qquad (4)$$

where Q_l is leakage to or from the aquifer between measurement sites (cubic feet per day),

K is mean hydraulic conductivity of streambed deposits between measurement sites (feet per day),

W is mean width of stream between measurement sites (feet),

L_r is distance between measurement sites (feet),

H_s is mean head of the stream between measurement sites (feet),

H_a is mean head in the aquifer between measurement sites (feet), and

M is mean distance over which head loss occurs from aquifer to stream between measurement sites (feet).

Substituting $(H_s-H_a)/M$ by I, the hydraulic gradient between the stream and aquifer, and rearranging equation 4 gives:

$$KW = \frac{Q_l}{L_r I}. \qquad (5)$$

The product KW was estimated for individual reaches of the Quashnet River by use of measurements of the streamflow and the hydraulic gradient between the aquifer and river. Average values of Q_l and I were obtained from one to three measurements at each of the nine streamflow-measurement stations during the investigation. Average vertical hydraulic gradients near the stream ranged from approximately 0.05 to 0.41 ft/ft; discharge measurements are shown in table 1. The distances between streamflow-measurement sites were obtained from maps. Estimated values of KW range from 100 to 2,200 ft^2/d and are greatest between Johns Pond and the site within the cranberry bog (Q3). A streambed hydraulic conductance was then obtained for each cell of the model in which the Quashnet River is located (by use of eq. 3).

The top boundary of the aquifer, which is the water table, was simulated by use of a free-surface boundary condition that receives spatially variable rates of recharge. The altitude of the water table is calculated by the model for the upper layer of grid cells. The rate of recharge from precipitation specified for steady-state model simulations, 21.6 in/yr, was obtained by use of the method of Thornthwaite and Mather (1957). At Ashumet and Johns Ponds, the steady-state rate of recharge was reduced to account for evaporation from the pond surfaces; a value of 18 in/yr was obtained by subtracting the estimated rate of free-water-surface potential evaporation from the ponds, 28 in/yr (Farnsworth and others, 1982, plate 1), from the average rate of precipitation in the study area, 46 in/yr, which was obtained from the East Wareham and Hyannis weather stations. Changes to these recharge rates made for the transient simulations are discussed under "Temporal variation of recharge."

Artificial recharge from septic systems was estimated for each cell of the upper layer of the model from maps of public-water-supply distribution lines and from the average daily rate of water supplied to unsewered areas by water companies in 1990. These estimates were obtained for each grid cell by means of the following equation:

$$R_r = \left(\frac{Q_{wd}\ L_{cell}}{A_{cell}\ L_{total}}\right) 0.9\ , \qquad (6)$$

where R_r is return-flow recharge (feet per day),
Q_{wd} is average daily rate of water distributed to unsewered areas by the water supplier (cubic feet per day),
A_{cell} is the area of the grid cell (square feet),
L_{cell} is the length of roads within unsewered areas in the cell that are served by the water supplier (feet), and
L_{total} is the total length of roads in unsewered areas that are served by the water supplier (feet).

It was assumed that 0.9 is the fraction of the water supplied to unsewered areas that returns to the aquifer as recharge. Ten percent of the supplied water is lost by consumptive use and evapotranspiration. No net recharge or discharge was simulated in areas serviced by both shallow domestic wells and onsite disposal systems or in irrigated areas. Artificial recharge at the MMR wastewater treatment facility, equal to 0.19 Mgal/d, was distributed evenly over the grid cells underlying the facility's infiltration beds.

Ground water is pumped from three wells screened within the aquifer; pumpage was simulated by means of a specified-flux boundary condition. Pumping rates of 0.86 Mgal/d for the Fresh Pond well in Falmouth and 0.36 Mgal/d for each of the two wells operated by the MWD were simulated in the upper layer of the model. Although the Fresh Pond well is currently not in operation (1993), it was included in the simulation because the field measurements against which the model was calibrated were taken when the well was in operation and because of the possibility that the well might resume operation in the future.

Analysis of Steady-State System

The response of the Quashnet River stream-aquifer system to proposed ground-water withdrawals was investigated primarily by use of steady-state model simulations. Steady-state analyses assume that ground-water levels, hydraulic gradients, and the velocity distribution of ground-water flow do not change with time. Although ground-water levels in the aquifer fluctuate 1 to 3 ft seasonally, they fluctuate about long-term averages and the state of the aquifer approximates a dynamic equilibrium.

Model Calibration

The first step in the modeling analysis of the Quashnet River stream-aquifer system is calibration of the model by use of available hydrologic data. The calibration process is completed to assure that the model is able to simulate current hydrologic conditions of the system, within the accuracy of the data used and assumptions made for model development. The initial estimates of model parameters, boundary conditions, and stresses, discussed in preceding sections, were modified during the process of model calibration such that calculated ground-water heads and discharge rates to the Quashnet River compared reasonably well with observed heads and discharge measurements.

The flow model was calibrated to the average of water levels measured in 49 observation wells and at Johns Pond during May 10 and 11 and November 2 and 3, 1990. These two sets of dates were chosen because ground-water levels in the three long-term observation wells in the study area (FSW 167, MIW 19, and MIW 29) were from 0.7 to 1.0 ft greater than the average in May 1990 and from 0.7 to 0.9 ft less than the average in November 1990. Therefore, the average of the water-table altitudes measured on these two sets of dates should be near long-term average conditions. The model also was calibrated to streamflow measurements made along the Quashnet River on February 22, 1990, because streamflow at the gaging station on this date was near its average daily value for the 3-year period of record. It was assumed that streamflow also was near average conditions at other sites along the river on this date.

Several model parameters were modified during calibration. The most significant changes were increases made to the initial estimates of hydraulic conductance of streambed sediments. The largest increases, nearly one order of magnitude, were made for the Quashnet River. These increases are not inconsistent with the error associated with the initial estimates of streambed conductance; in particular, the measured vertical gradient near the streambed and the assumed thickness over which head loss between aquifer and streambed occurs are uncertain. The streambed altitudes of the Mashpee and Childs Rivers also were reduced by a maximum of 1 and 2 ft, respectively. Finally, the rate of recharge from precipitation was increased by 5 percent and the rate of inflow along the northern boundary was increased by 10 percent. Given the uncertainty of the estimates of

recharge, inflow across the northern boundary, streambed altitudes, and hydraulic conductance of streambed sediments, the changes made to the input data during calibration of the model are considered reasonable. An analysis of the uncertainty associated with these parameters and their effect on calculated streamflow are discussed in the section "Sensitivity of Predicted Streamflow in the Quashnet River to Changes in Model Parameters".

Calculated water-table altitudes for the upper layer of the model are shown in figure 15. The magnitude and direction of simulated gradients are similar to those measured in May and November 1990 (figs. 7 and 8). The average absolute difference between calculated and observed heads at the 50 measurement points, 1.45 ft (table 2), represents less than 3 percent of the total head range in the study area. Calculated heads were generally lower than observed heads; the average difference between calculated and observed heads was -0.58 ft. The largest errors between observed and calculated heads (greater than 4 ft) were near the southwestern boundary of the model; these errors indicate that either the streamline used to define the western boundary or the hydraulic properties specified for the model were not adequately defined for that area. However, because this area is outside the contributing areas of the Quashnet River and the area of proposed ground-water withdrawal (as discussed in sections that follow), these errors are assumed to be insignificant for the purpose of this model.

Components of the calculated hydrologic budget of the modeled area are given in table 3. Total inflow to the modeled area is 57.7 ft^3/s, of which 82 percent is recharge from precipitation and wastewater return flow and 15 percent is inflow of ground water across the northern boundary of the model. Total outflow to the Childs, Quashnet, and Mashpee Rivers from the modeled area is 23.6 ft^3/s, or 41 percent of total outflow. It should be noted that outflow from the Mashpee River represents only contributions from the modeled area and cannot be compared directly to field measurements of total flow in the stream.

Calculated streamflow at the gaging station on the Quashnet River (site Q9) is 13.3 ft^3/s, which is 0.5 ft^3/s less than the 3-year period-of-record mean streamflow of 13.8 ft^3/s. The shape of the curve of calculated streamflows along the length of the river from Johns Pond to the gaging station (fig. 16) closely matches that of observed streamflows for the four measurement dates, indicating that the model accurately simulates the high rates of inflow from the aquifer between Johns Pond and Q3 and between sites Q5 and Q9. Calculated streamflow for the Childs River is 3.3 ft^3/s, or about 2.7 ft^3/s less than that measured during a period of near-average hydrologic conditions in the system.

Contributing Area of the Quashnet River for Current Hydrologic Conditions

A recent advance in the ability to analyze ground-water-flow systems quantitatively has been the development of particle-tracking algorithms that determine fluid-particle pathlines from the results of numerical flow models. Particle tracking allows for delineation of the contributing areas of streams and wells because particles can be tracked from areas of ground-water recharge to areas of ground-water discharge, thereby identifying the area at the water table that is the source of water for these hydrologic boundaries. The only parameter in addition to those used in the flow model that is required for the particle-tracking analysis is porosity, which, on the basis of values of porosity of the aquifer cited previously, was set at 0.39. Porosity does not affect the shape, location, or size of contributing areas of the Quashnet River or pumped wells, only the time required for a particle to travel through the system.

Particles were tracked in the forward direction to delineate the contributing area of the Quashnet River upstream of the streamflow-gaging station (site Q9). Particles were tracked from the water table and from cell faces forming the northern boundary of the model to the head-dependent flux boundaries simulating the Quashnet River. Some particles originating at the cell faces forming the northern boundary of the lower layer discharged into the river along its northernmost reaches. The remainder of these particles traveled below the Quashnet River southeastward toward the Mashpee River.

The steady-state contributing area delineated for the Quashnet River, shown in figure 17, is similar in size, shape, and location to contributing areas drawn on the water-table maps (figs. 7 and 8). The particle-tracking analysis indicates that streamflow in the river is sustained both by water recharged to the aquifer within the modeled area and by ground-water inflow along the northern boundary of the model; consequently, the contributing area of the river extends upgradient from the modeled area. The contrib-

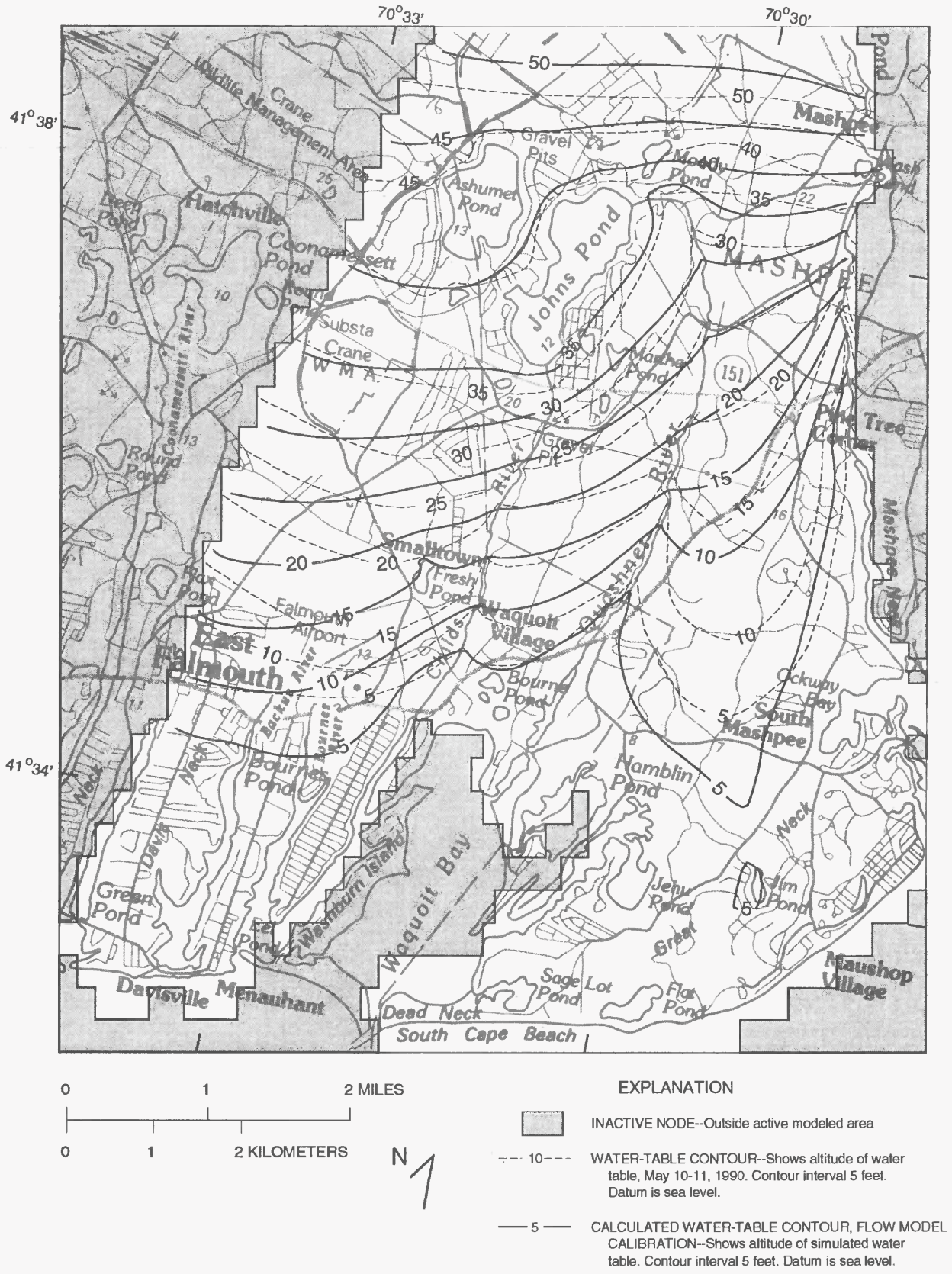


Figure 15.--Altitude and configuration of the water table from water levels observed May 10-11, 1990, and from levels calculated by use of the calibrated steady-state model. Location of modeled area is shown in figure 1.

Table 2.--*Average observed heads in the Quashnet River study area and heads calculated by use of the calibrated steady-state model*

[In the USGS well identifier, the first three letters identify the location (FSW, Falmouth; MIW, Mashpee; SDW, Sandwich), the first three digits are a unique well cluster number, and the second three digits are the approximate well depth. The town identifier and the well cluster number only are shown on report figures]

USGS well identifier	Model node location			Calculated head (feet above sea level)	Average observed head (feet above sea level)	Difference (feet)
	layer	row	column			
FSW 237-088	1	10	19	43.16	43.52	-0.36
FSW 279-061	1	16	21	39.32	39.54	-.22
FSW 343-036	1	7	21	45.12	46.05	-.93
FSW 348-021	1	13	22	42.20	42.30	-.10
FSW 375-015	1	34	11	26.41	23.51	2.91
FSW 382-032	1	8	21	44.29	45.19	-.90
FSW 411-036	1	14	19	40.73	40.42	.31
FSW 412-042	1	23	19	32.47	32.56	-.09
FSW 414-036	1	8	22	43.97	45.26	-1.29
FSW 420-036	1	9	22	43.11	44.44	-1.33
FSW 436-036	1	38	10	24.84	20.35	4.49
FSW 438-035	1	34	17	27.03	27.12	-.09
FSW 463-020	1	18	17	36.79	36.40	.39
FSW 488-047	1	23	25	31.91	33.07	-1.16
FSW 489-043	1	30	24	27.36	29.24	-1.88
FSW 491-038	1	54	25	12.39	14.63	-2.24
FSW 492-007	1	70	12	9.34	4.67	4.67
FSW 493-008	1	63	13	14.75	12.14	2.61
FSW 495-010	1	44	26	17.92	19.50	-1.58
FSW 497-035	1	68	8	10.18	5.43	4.75
FSW 498-040	1	62	17	14.01	17.52	-3.51
FSW 499-018	1	64	23	7.12	6.63	.49
MIW 19-046	1	26	44	25.03	27.14	-2.11
MIW 29-040	1	68	69	6.34	6.98	-.64
MIW 68-044	1	14	27	40.35	40.48	-.13
MIW 98-030	1	37	68	13.85	16.35	-2.50
MIW 99-050	1	35	65	15.26	16.67	-1.41
MIW 100-016	1	42	58	11.56	11.12	.44
MIW 101-050	1	40	67	13.01	14.98	-1.97
MIW 104-069	1	43	40	16.95	19.82	-2.87
MIW 106-085	2	42	45	15.74	15.96	-.22
MIW 108-054	1	45	48	13.19	13.35	-.16
MIW 109-075	2	46	49	12.42	12.67	-0.25
MIW 112-014	1	47	51	9.89	10.92	-1.03
MIW 112-076	2	47	51	11.48	10.94	.54
MIW 113-020	1	25	76	6.65	7.35	-.70
MIW 114-060	1	27	75	9.67	10.90	-1.23
MIW 115-069	2	34	76	7.09	8.35	-1.26
MIW 116-015	1	33	76	5.84	7.90	-2.06
MIW 117-060	1	29	75	9.20	13.17	-3.97

Table 2.--*Average observed heads in the Quashnet River study area and heads calculated by use of the calibrated steady-state model*--Continued

USGS well identifier	Model node location layer	row	column	Calculated head (feet above sea level)	Average observed head (feet above sea level)	Difference (feet)
MIW 120-060	1	35	56	16.46	17.76	-1.30
MIW 121-073	1	31	56	18.61	19.74	-1.13
MIW 122-068	1	30	52	20.42	21.11	-.69
MIW 127-074	1	28	56	20.05	20.39	-.34
MIW 136-088	1	3	66	49.74	51.70	-1.96
MIW 136-125	2	3	66	49.31	51.77	-2.46
MIW 164	1	45	33	16.97	17.59	-.62
SDW 313-020	1	7	24	44.56	45.83	-1.27
SDW 315-061	1	2	21	49.29	51.20	-1.91
Johns Pond	1	15	39	37.31	38.20	-.89
Average difference						-.58
Absolute average difference						1.45

uting area defined by particle tracking lies primarily to the west of the river.

The particle-tracking analysis also indicates that ground-water inflow to Ashumet and Johns Ponds contributes water to the river because the contributing area of the river overlaps the ponds (fig. 17). Because the ponds are represented in the model as zones of high hydraulic conductivity, particle pathlines are traced through the ponds. In the natural system, however, water within the ponds becomes mixed before discharging back into the aquifer. Consequently, the entire contributing areas of the ponds must be included in the contributing areas of the river. An additional particle-tracking run was made to identify the entire contributing area of each pond; these areas are added to the contributing area of the Quashnet River and are shown in figure 17 but not on subsequent contributing-area maps.

System Responses to Proposed Pumping

The MWD has determined that a well developed between the Quashnet and Childs Rivers in the area of observation wells MIW 108, MIW 104, and MIW 164 (fig. 1) would provide a high yield of potable ground water. Five pumping scenarios were chosen to show the effect of selected ground-water-development alternatives on the location and volume of streamflow depletion and on the location and size of the contrib-

Table 3.--*Hydrologic budget for the Quashnet River stream-aquifer system calculated by use of the calibrated steady-state model*

Budget component	Rate of flow: Cubic feet per second	Rate of flow: Million gallons per day
Inflow:		
Recharge		
Precipitation	41.2	26.6
Septic system	6.0	3.9
Sewage treatment facility	.3	.2
Across northern model boundary	8.9	5.8
Leakage from Mashpee Pond	1.3	.8
Total inflow	57.7	37.3
Outflow:		
Pumpage from wells	1.9	1.2
Discharge to streams		
Childs	3.3	2.2
Quashnet	14.2	9.2
Mashpee	6.1	3.9
Discharge to saltwater boundaries	32.6	21.0
Total outflow	58.1	37.5
Inflow minus outflow (numerical error)	-0.4	-0.2

uting areas of the simulated river and the simulated supply wells. The five scenarios are: pumping rates of (1) 0.5 Mgal/d, (2) 1.0 Mgal/d, and (3) 2.0 Mgal/d at well MIW 108, and pumping rates of 1.0 Mgal/d at (4) well MIW 104 and (5) well MIW 164. All pumping was simulated in the top layer of the model. Alternative pumping scenarios at these and other nearby sites could be simulated with the model.

Contributing areas of the simulated river and wells for the five pumping scenarios are shown in figures 18-22. Also shown in these figures are drawdowns produced by the pumped wells. Drawdowns are the decline from calculated pre-ground-water development steady-state heads produced by each ground-water development scenario, and aid in the interpretation of the effect of ground-water withdrawals on the stream-aquifer system.

The contributing areas of the three simulated wells do not include the Quashnet River in any of the five scenarios, indicating that there is no induced infiltration of streamflow from the river to the aquifer. The simulated river actually receives ground-water discharge along its entire length for all pumping scenarios (fig. 23), even though drawdown produced by the pumped wells extends beneath the river. Drawdowns produced by the pumped wells decrease the volume of discharge to the river but are insufficient to reverse the direction of ground-water flow near the river; the pumped wells capture water that would have discharged to the river but that has been diverted to the wells instead.

As indicated in figure 23, the depletion in streamflow caused by pumping at well MIW 108 increases along the length of the river, from the smallest depletions at site Q4 to the largest depletions at site Q9 (the Streamflow-Gaging Station). At a pumping rate of 1.0 Mgal/d, streamflow depletion at the streamflow-gaging station decreases as the simulated location of pumping is moved progressively westward from MIW 108 to MIW 104 to MIW 164 (table 4), as would be expected; however, the volume of water captured by the pumped well that would have discharged to the Childs River increases from 0.1 to

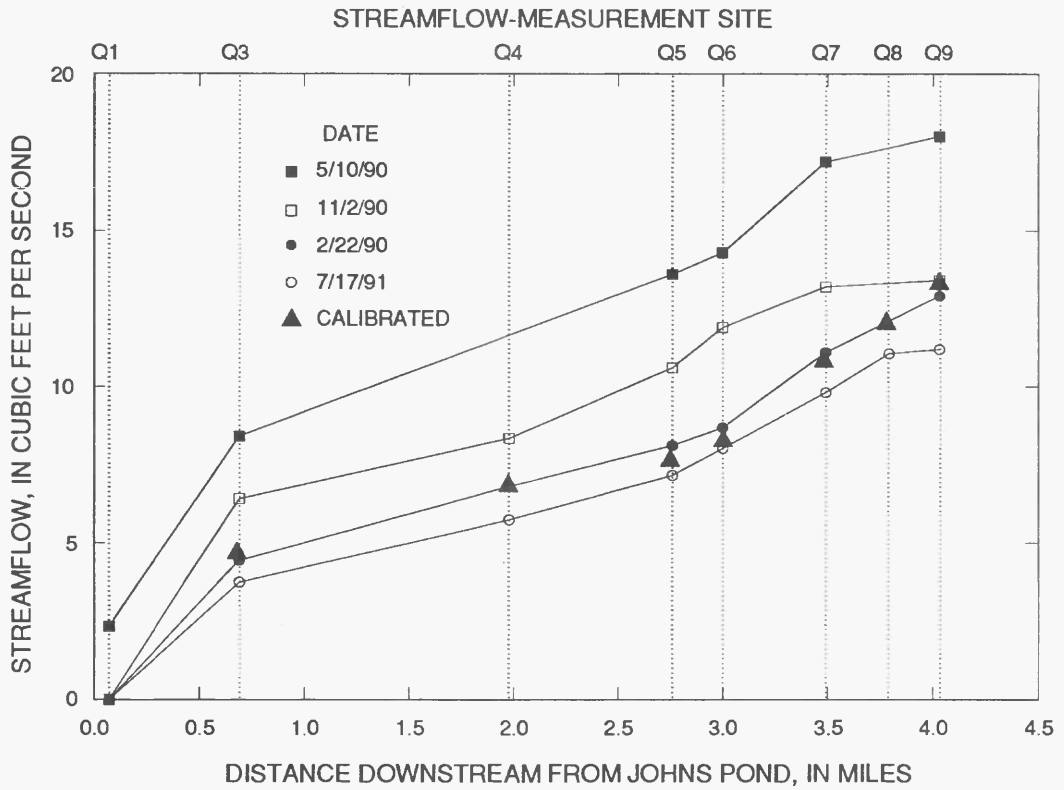


Figure 16.--Streamflow measured at selected sites along the Quashnet river, 1990-91, and streamflow calculated by use of the calibrated steady-state model.

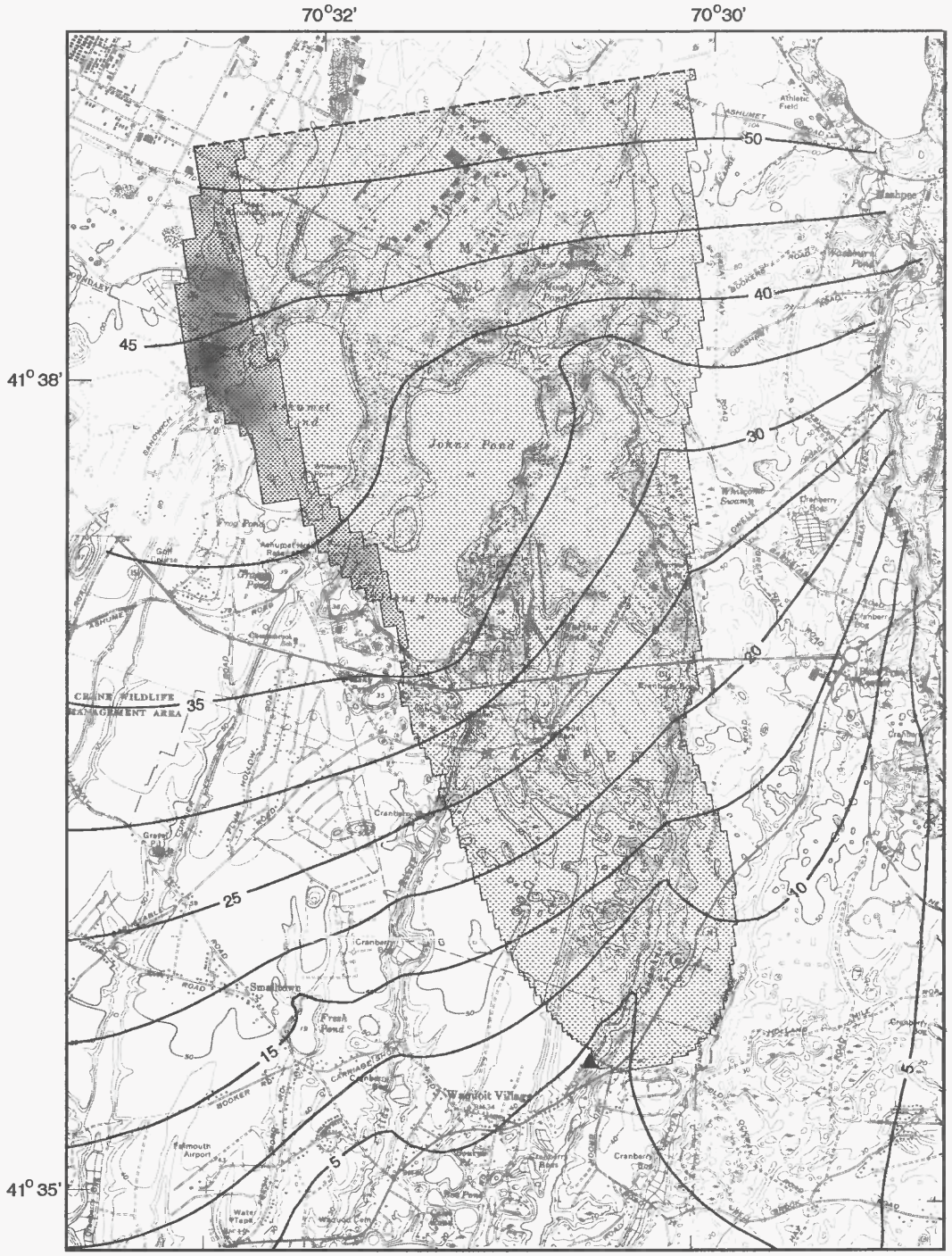


Figure 17.--Contributing areas of the Quashnet River and Ashumet and Johns Ponds, calculated by use of the calibrated steady-state model.

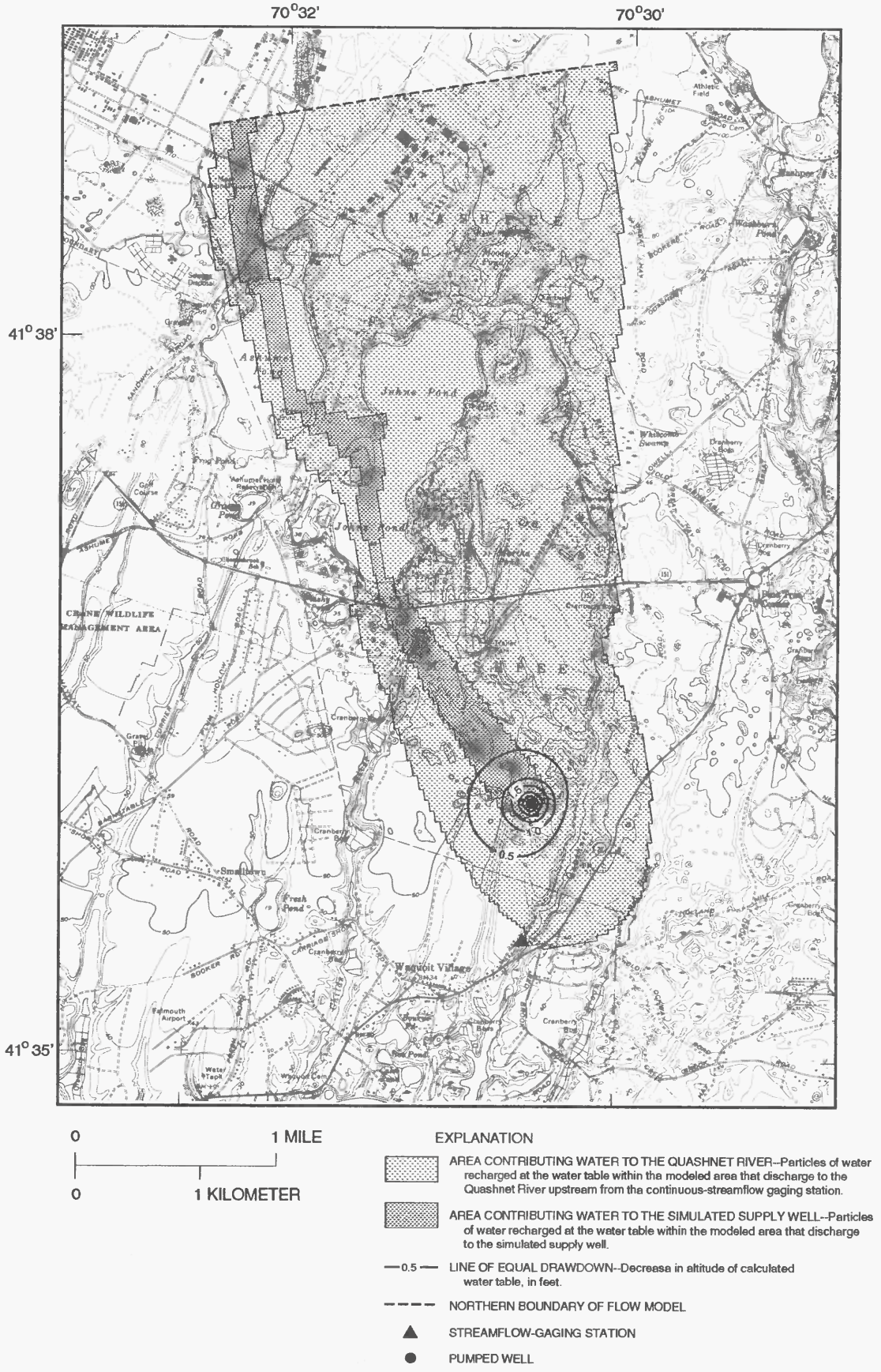


Figure 18.--Contributing areas of the Quashnet River and simulated supply well at MIW 108, pumped at a rate of 0.5 million gallons per day, and simulated drawdown.

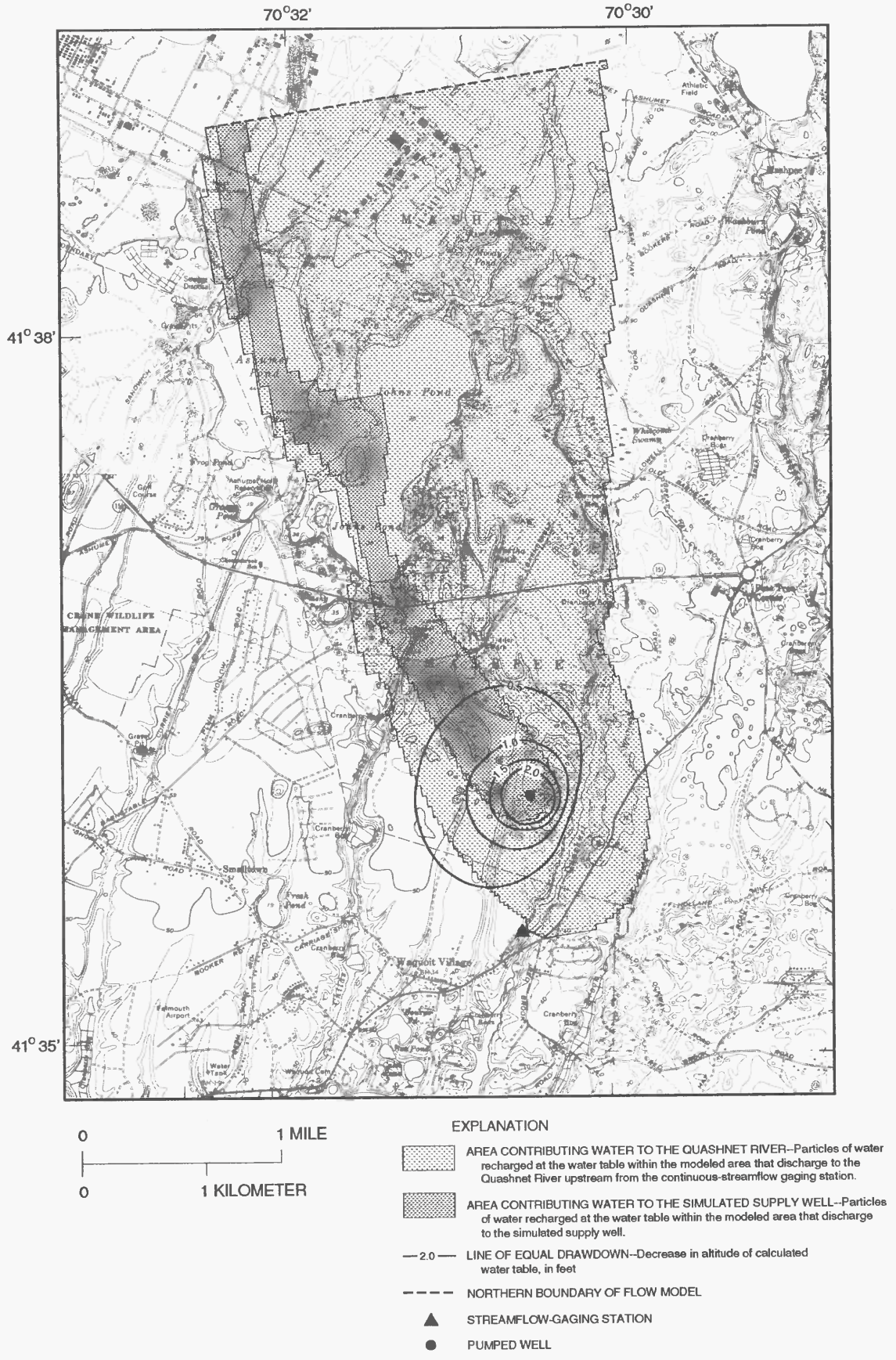


Figure 19.--Contributing areas of the Quashnet River and simulated supply well at MIW 108, pumped at a rate of 1.0 million gallons per day, and simulated drawdown.

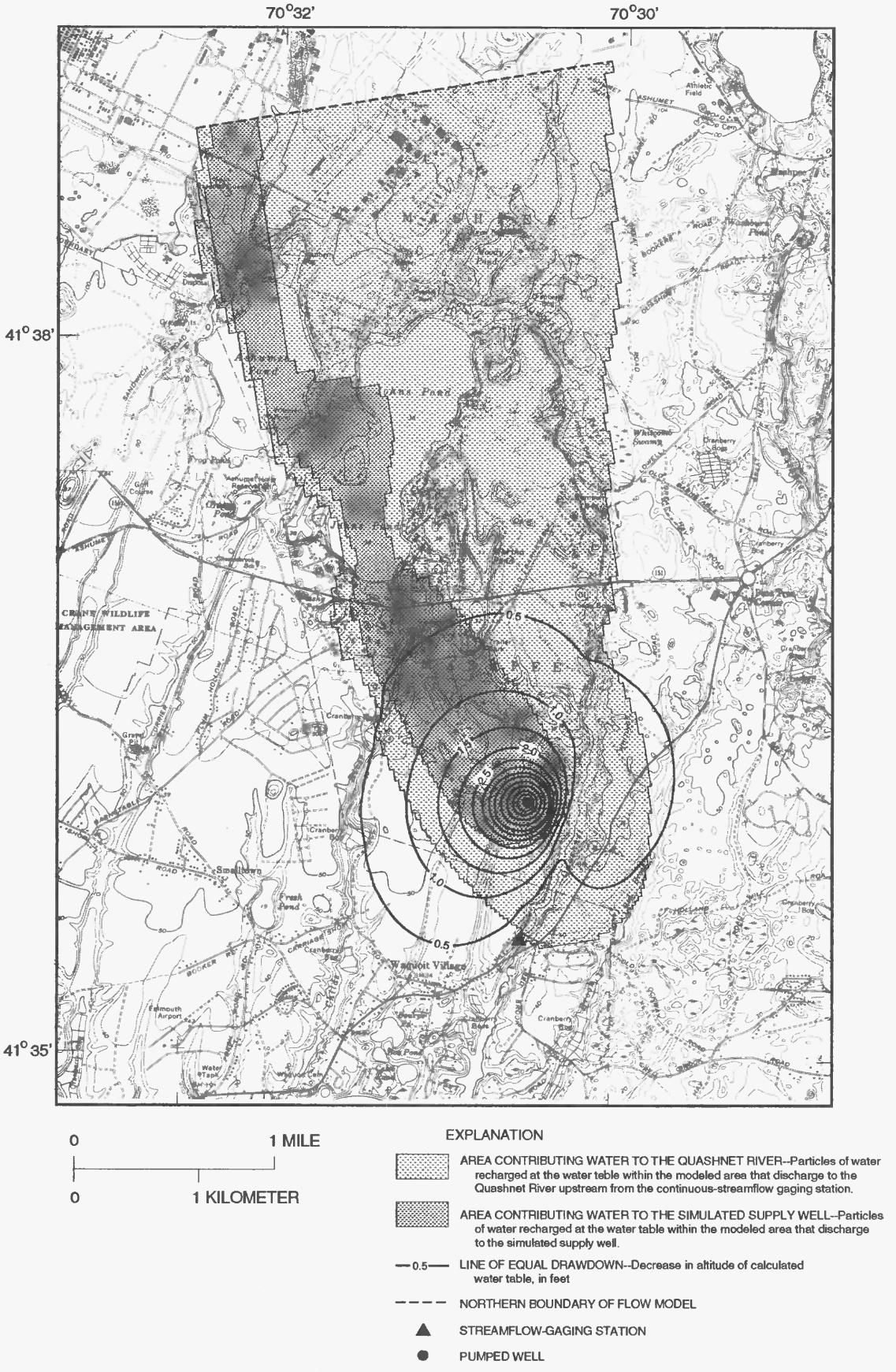


Figure 20.--Contributing areas of the Quashnet River and simulated supply well at MIW 108, pumped at a rate of 2.0 million gallons per day, and simulated drawdown.

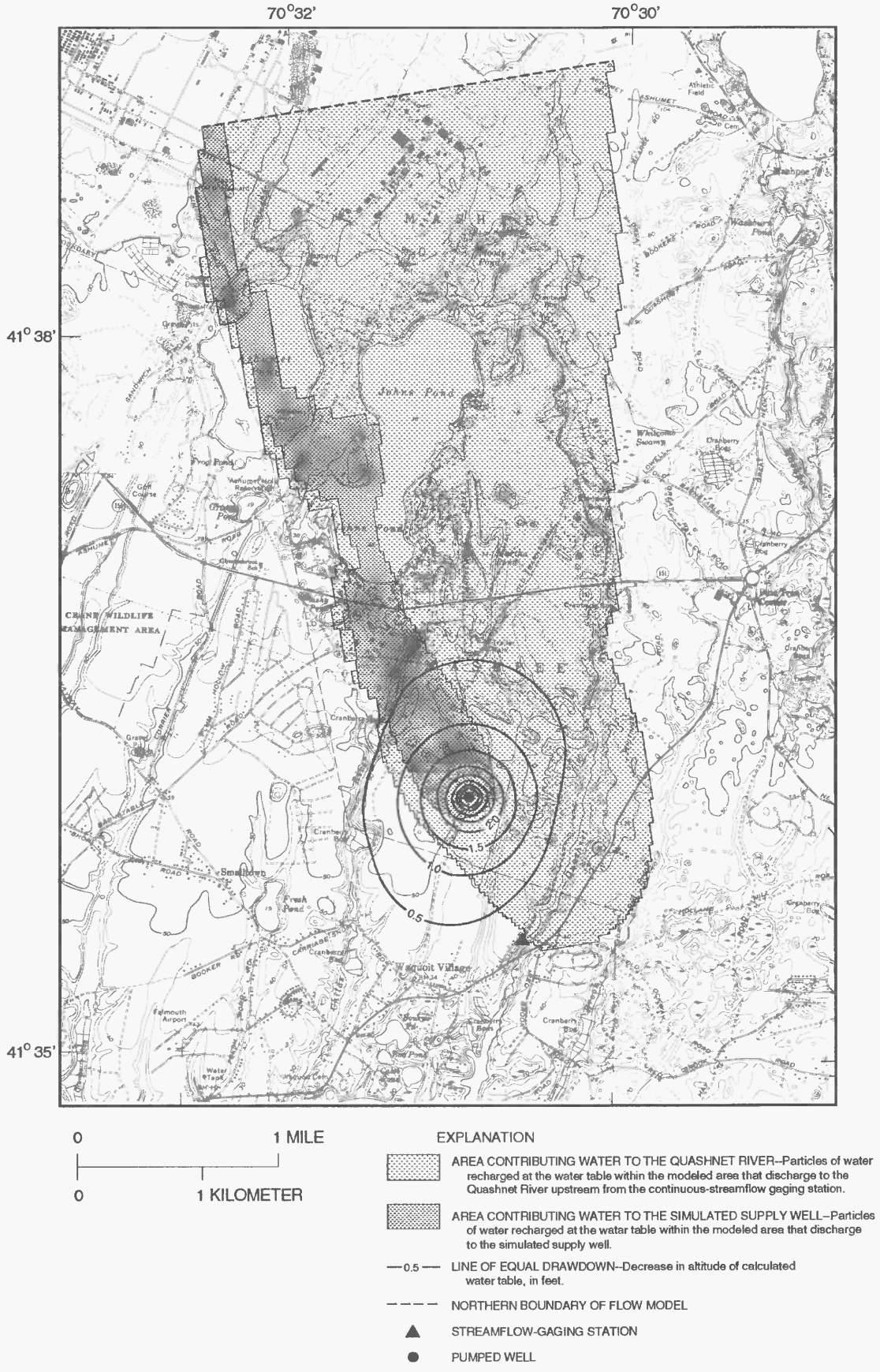


Figure 21.--Contributing areas of the Quashnet River and simulated supply well at MIW 104, pumped at a rate of 1.0 million gallons per day, and simulated drawdown.

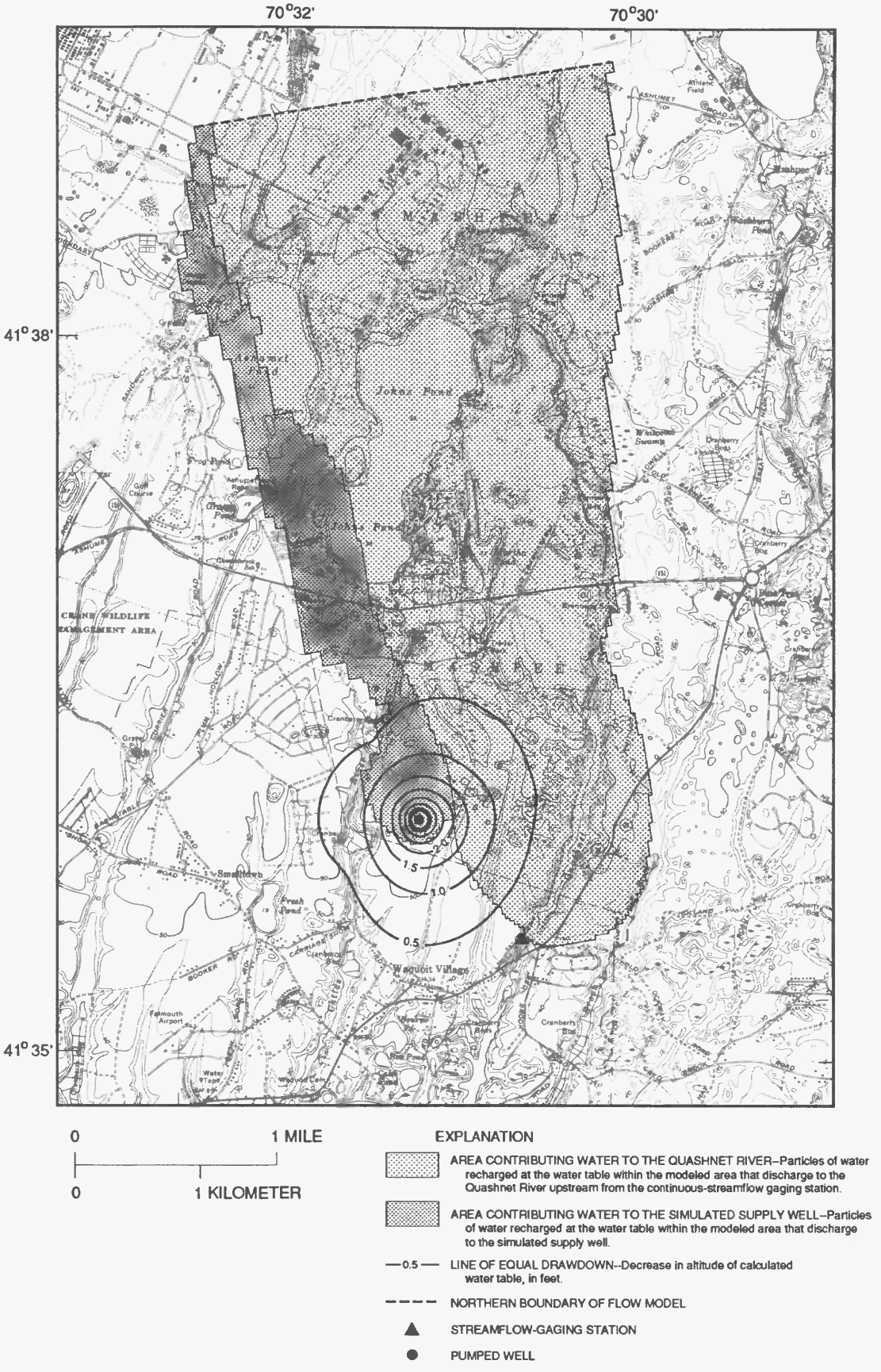


Figure 22.--Contributing areas of the Quashnet River and simulated supply well at MIW 164, pumped at a rate of 1.0 million gallons per day, and simulated drawdown.

0.4 Mgal/d as the location is moved westward. The streamflow depletion at the Fish Ladder site (Q5) is less sensitive to the location of the pumped well; each 1.0-Mgal/d pumping scenario produced about the same streamflow depletion at this site.

The depletion in streamflow ranges from 0.1 to 0.3 ft^3/s at the Fish Ladder site (Q5) and from 0.5 to 1.9 ft^3/s at the streamflow-gaging station (Q9) for all five scenarios (table 4). These depletions are small when viewed on the basis of percentage of total flow. For pumping rates less than 2.0 Mgal/d, the maximum percentage depletion (calculated as the reduction in streamflow divided by the streamflow estimated for conditions of no pumping at any of the three wells) is 2.6 percent at the Fish Ladder site (Q5) and 7.5 percent at the streamflow-gaging station (Q9). The rating curve of gage height as a function of streamflow for the Quashnet River at the gaging-station site, which relates the stream-surface altitude to streamflow of the river, can be used to estimate changes in stream depths that would result from these pumping scenarios. Maximum declines in the stream-surface altitude of about 0.10 and 0.15 ft would be expected for decreases in streamflow resulting from pumping at rates of 1.0 and 2.0 Mgal/d, respectively, at well MIW 108. On the basis of 29 streamflow-discharge measurements made at the gaging station from October 1988 through September 1991, the mean depth of the stream at this site is 0.96 ft. Therefore, a drop of 0.10 ft would result in about a 10-percent decline in the total depth of the stream at the site, and a drop of 0.15 ft would result in about a 16-percent decline in the total depth of the stream at the site. Although the decline in stream-surface elevation resulting from pumping would vary along the length of the river, the small decline at the gaging station indicates that these declines likely would be small.

Hydrologic budgets calculated for the calibrated model with no ground-water withdrawals other than those currently being made within the modeled area (fig. 2) and with an additional withdrawal rate of 1.0 Mgal/d at well MIW 108 are listed in Table 5. The table indicates that 70 percent of the water captured by the well at the 1.0-Mgal/d pumping rate consists of

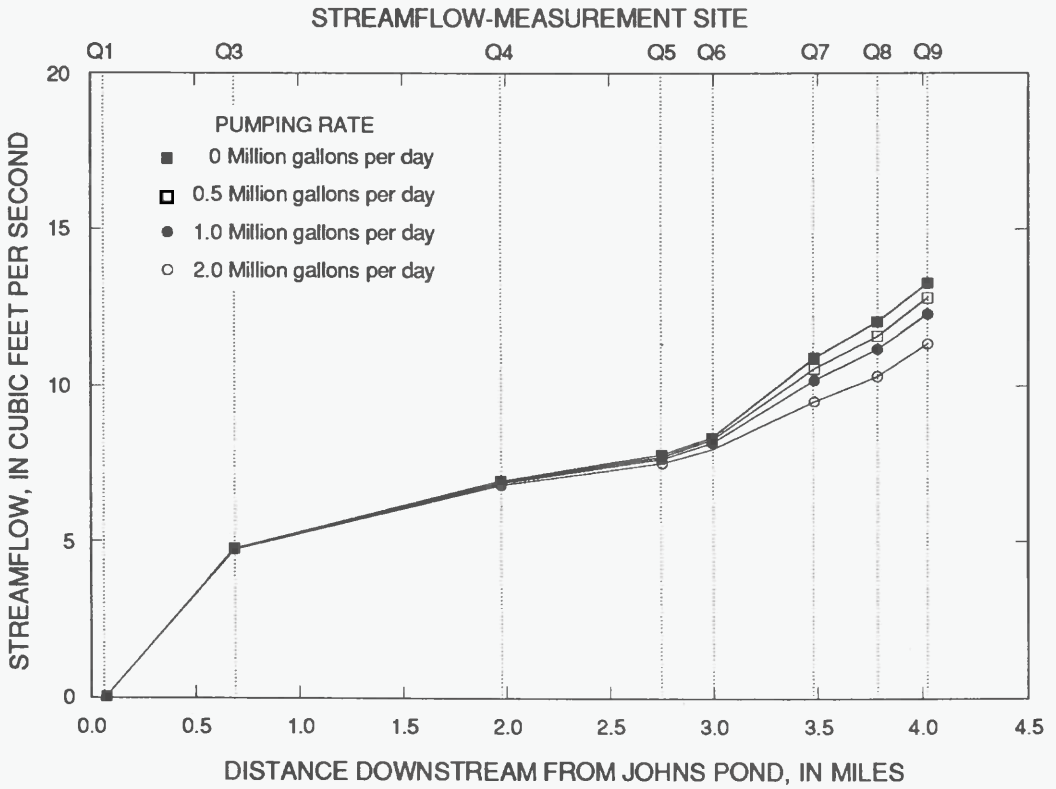


Figure 23.--Streamflow along the Quashnet River calculated by use of the steady-state model for pumping rates of 0, 0.5, 1.0, and 2.0 million gallons per day from the simulated supply well at MIW 108.

Table 4.--*Streamflow of Quashnet River for five withdrawal scenarios, calculated by use of the steady-state model*

[Percent reduction is decrease in streamflow from that in the calibrated model divided by streamflow in the calibrated model; ft^3/s, cubic foot per second; Mgal/d, million gallons per day; --, not applicable]

			Fish ladder (site Q5)		Gaging station (site Q9)	
Scenario	Pumping rate (Mgal/d)	Site of pumping	Streamflow (ft^3/s)	Percent reduction	Streamflow (ft^3/s)	Percent reduction
	0	(Calibrated model)	7.8	--	13.3	--
1	.5	MIW 108	7.7	1.3	12.8	3.8
2	1.0	MIW 108	7.6	2.6	12.3	7.5
3	2.0	MIW 108	7.5	3.8	11.4	14
4	1.0	MIW 104	7.6	2.6	12.7	4.5
5	1.0	MIW 164	7.7	1.3	12.8	3.8

ground water that would have otherwise discharged to the Quashnet River. The well also captures ground water that would have discharged to the Childs and Mashpee Rivers and to the coastal saltwater boundary. Model results indicate that 40 percent and 65 percent of the water captured by the pumped well at this location would have otherwise discharged to the Quashnet River for simulated pumping rates of 0.5 Mgal/d and 2.0 Mgal/d, respectively, whereas 50 and 30 percent of well discharge would have discharged to the river for a pumping rate of 1.0 Mgal/d at MIW 104 and MIW 164, respectively.

Inflow of ground water along the northern boundary of the model is not a source of water to the simulated supply well at MIW 108 for a pumping rate of 0.5 Mgal/d, although the contributing area of the well extends to the boundary (fig. 18). The volume of water obtained by the pumped wells from the boundary is less than 0.1 Mgal/d, or 10 percent of total pumping, for all three scenarios of pumping at 1.0 Mgal/d; this volume increases to 0.4 Mgal/d, or 20 percent of total pumping, when the pumping rate is 2.0 Mgal/d at site MIW 108. Because water flowing into the modeled area along the northern boundary is a source of water to the wells, the contributing areas of the wells extend upgradient from the northern model boundary (fig. 19-22). Some of the recharge and ground-water inflow to Ashumet and Johns Ponds also reaches the pumped well in all pumping scenarios (figs. 18-22).

As the simulated pumping rate at site MIW 108 increases from 0.5 to 2.0 Mgal/d, the 0.5-ft drawdown contour produced by the pumping extends farther outward from the well (figs. 18-20). The shape of the cone of depression that is defined by the drawdown contours is not symmetric about the well because of the proximity of the Quashnet and Childs Rivers. As the cone of depression extends outward from the wells, it intersects and becomes parallel to the rivers because the rivers function as specified-head boundaries. At the higher pumping rate of 2.0 Mgal/d (fig. 20), the 0.5-ft contour extends east of the Quashnet River and parallels the Childs River to the west. When pumping is simulated at MIW 104 (fig. 21), the 0.5-ft contour parallels both rivers, because the well is approximately midway between the two rivers. Finally, when pumping is simulated at MIW 164 (fig. 22), the 0.5-ft drawdown contour extends further in the direction of the Quashnet River and crosses the Childs River.

The drawdowns produced by each of the pumped wells and the contributing areas of each of the wells, as shown in figures 18-22, provide clear examples of how the cone of depression (or zone of influence) of a pumped well is not coincident with its contributing area. As discussed by Brown (1963), the cone of depression formed by a pumped well will expand in all directions to the natural boundaries of the aquifer, whereas the contributing area of a well will expand only until the well has captured enough water to balance the pumping rate of the well. Although the cone of depression extends in all directions around each pumped well, the contributing area of each pumped well consists only of that area in which ground-water flow has been diverted to and captured by the well. The modeling results presented here illustrate that a pumped well can decrease ground-

Table 5.--*Hydrologic budget for the Quashnet River stream-aquifer system for ground-water withdrawals of 0 and 1.0 million gallons per day from the simulated supply well at MIW 108, calculated by use of the calibrated steady-state model*

[All values are Mgal/d (million gallons per day)]

Budget component	Withdrawal rate		Change in flow rate
	0 Mgal/d	1.0 Mgal/d	
Inflow:			
Recharge			
Precipitation	26.6	26.6	0
Septic system	3.9	3.9	0
Sewage treatment facility	.2	.2	0
Across northern model boundary	5.8	5.8	0
Leakage from Mashpee Pond	.8	.8	0
Total inflow	37.3	37.3	0
Outflow:			
Pumpage from wells	1.2	2.2	1.0
Discharge to streams			
Childs	2.2	2.0	-.2
Quashnet	9.2	8.5	-.7
Mashpee	3.9	3.9	0
Discharge to saltwater boundaries	21.0	20.9	-.1
Total outflow	37.5	37.5	0
Inflow minus outflow (numerical error)	-0.2	-0.2	0

water heads near and beneath a stream and reduce the volume of discharge to the stream, but the pumping does not necessarily induce infiltration of stream water to the aquifer.

Sensitivity of Predicted Streamflow in the Quashnet River to Changes in Model Parameters

A sensitivity analysis of the model was completed to determine which parameters in the model most affect streamflow in the Quashnet River. Values of several parameters were changed in a series of simulations to evaluate the sensitivity of calculated streamflow in the Quashnet River to uncertainty in these parameters. Because the purpose of this investigation is to evaluate the response of the hydrologic system to proposed ground-water withdrawal, pumping at a rate of 1.0 Mgal/d from well MIW 108 was included in all simulations. Parameters were changed within ranges consistent with the relative uncertainty associated with each parameter. For example, whereas recharge and horizontal hydraulic conductivity of the top layer of the model were changed by a maximum of only 25 percent, the vertical conductance between the layers and the streambed hydraulic conductance of the Quashnet River were changed by one order of magnitude. The following simulations were made (fig. 24): (a) inflow along the northern boundary of the model was increased and decreased by 25 percent; (b) streambed hydraulic conductance of the Quashnet River was increased and decreased by one order of magnitude; (c) horizontal hydraulic conductivity of the top layer of the model was increased and decreased by 25 percent; (d) horizontal hydraulic conductivity of the bottom layer of the model was increased and decreased by 50 percent; (e) vertical conductance between the model layers was increased and decreased by one order of magnitude; and (f) recharge to the model was increased and decreased by 15 percent.

Results of the sensitivity analysis indicate that calculated streamflow is most sensitive to changes in the streambed hydraulic conductance of the Quashnet River (fig. 24*B*) because this parameter is the strongest control on the volume of water moving between the stream and aquifer. Streamflow is also highly sensitive to the recharge rate to the aquifer (fig. 24*F*) because the recharge rate affects the volume of water moving through the upper layer of the model that can intercept the stream. Calculated streamflow is less sensitive to the rate of inflow along the northern boundary of the model (fig. 24*A*) because the total inflow at this boundary is a small percentage of the total inflow to the model. Calculated streamflow also is relatively insensitive to changes in horizontal hydraulic conductivity of the upper and lower layers of the model (figs. 24*C* and *D*) because calculated heads near the river are not substantially affected by changes made to these parameters within this range of uncertainty. Calculated streamflow is least sensitive to changes made in the vertical conductance between model layers (fig. 24*E*) because the volume of water moving to the river is largely unaffected by

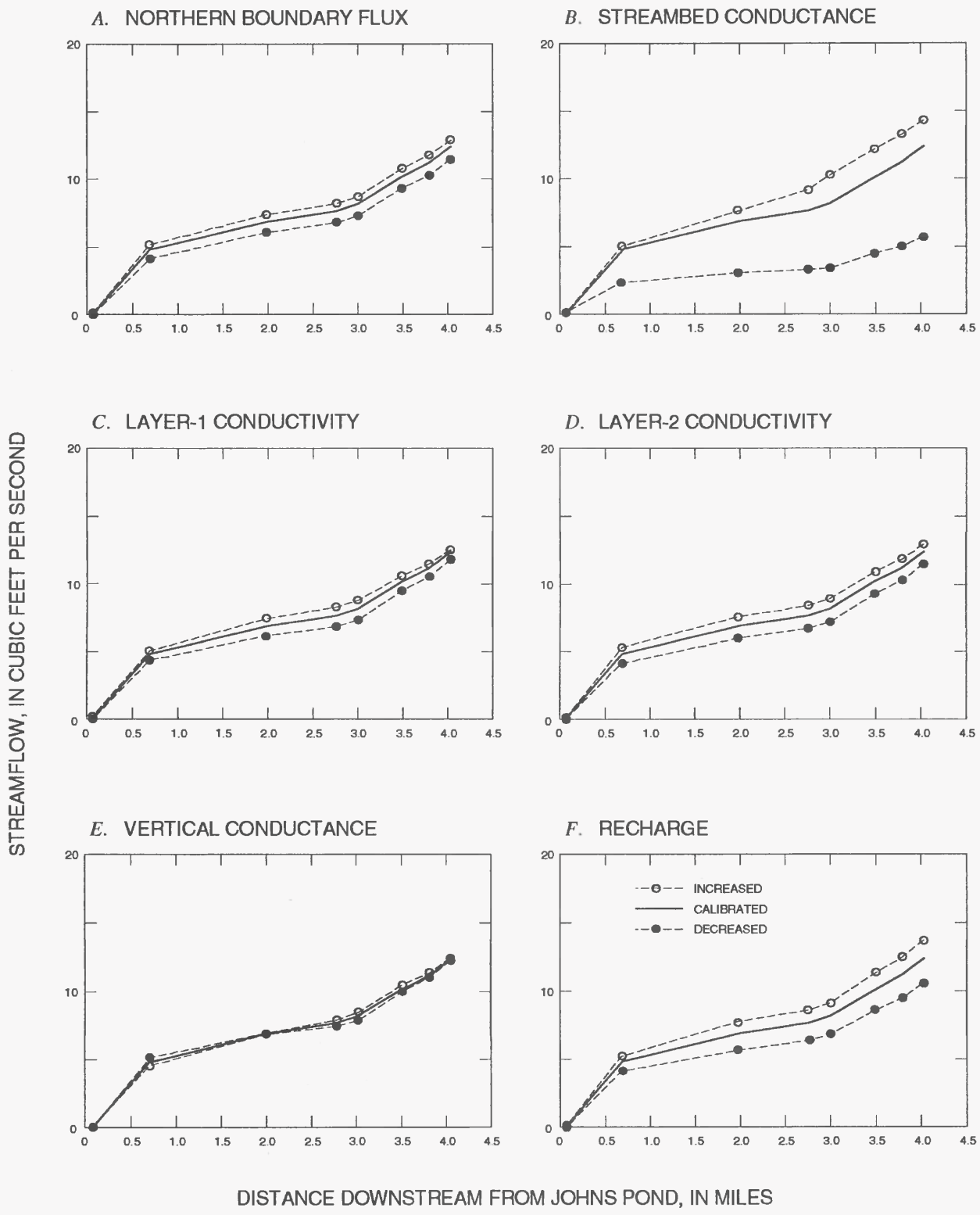


Figure 24.--Sensitivity of calculated streamflow in the Quashnet River to variations in model parameters: (A) northern model boundary flux, (B) Quashnet River streambed conductance, horizontal hydraulic conductivity of (C) layer-1, and (D) layer-2, (E) vertical conductance between layers 1 and 2, and (F) areal recharge rate.

changes in the volume of water moving between the layers.

The results of these simulations indicate the parameters to which calculated streamflows are most sensitive. The results cannot be used alone, however, to determine the range of streamflows resulting from uncertainty because parameters were not varied simultaneously nor were other parameters adjusted to achieve an adequate match between simulated and observed heads. For example, although the sensitivity analysis indicates that streamflow would be greatly reduced by an order-of-magnitude reduction in streambed hydraulic conductance, this condition is inconsistent with the stream-aquifer system simulated by the calibrated model. A simulation was made in which the calibrated model for current hydrologic conditions was rerun with the order-of-magnitude reduction in streambed hydraulic conductance to verify that these simulation conditions cause calculated ground-water altitudes and streamflow to deviate significantly from calibrated values. The discharge at the gaging station resulting from this simulation was only 6.8 ft^3/s, whereas the flow at the gaging station calculated by the calibrated steady-state model is 13.3 ft^3/s. Also, the mean absolute difference between average observed heads and calculated heads for this simulation is 3.18 ft, whereas it is 1.45 ft for the calibrated steady-state model. This indicates that an order-of-magnitude decrease in streambed hydraulic conductance is not consistent with observed streamflow and head data.

Analysis of Transient System

The calibrated steady-state model was modified to evaluate the transient response of the system to a yearly cycle of monthly varying rates of ground-water recharge. All hydraulic parameters and boundary conditions of the steady-state model except recharge were held constant for the transient analysis, including specified-flux boundary conditions at existing pumped wells and along the northern boundary of the model. It was assumed that the variation in ground-water inflow along the northern boundary resulting from monthly variations in ground-water heads is small and that errors produced by a constant rate of inflow along the boundary would be negligible. Specific yield and storage coefficient also were specified for the transient model; values of 0.25 for specific yield and 2×10^{-4} for the storage coefficient were specified on the basis of the results of the finite-element radial-flow model developed to analyze the results of the aquifer test at MIW 108. A specific yield of 0.15 also was used in a simulation of current conditions to evaluate the response of the system to this lower specific yield.

Temporal Variation of Recharge

The total volume of recharge to the calibrated steady-state model was distributed over a 12-month period of simulation such that the total volumes of recharge for the steady-state and transient models were equal. Monthly rates of recharge from precipitation for the transient model were obtained from the average of the monthly rates determined for the East Wareham and Hyannis weather stations (fig. 10). These monthly rates were increased by 5 percent to be consistent with the increase to the recharge rate made during calibration of the steady-state model. Figure 25*A* shows the distribution of average monthly recharge from precipitation specified for the transient model. Recharge rates from septic systems and the MMR Wastewater Treatment Facility were maintained at their average annual rates.

Initial Conditions and Temporal Model Discretization

Because ground-water heads and streamflows calculated by the steady-state models are representative of average conditions in the system, steady-state heads were used as initial conditions for the transient model. As indicated in figure 11, the monthly mean streamflow of the Quashnet River at the streamflow-gaging station is close to the period-of-record mean streamflow during March and July. For this analysis, it was assumed that steady-state calculated heads and streamflows were representative of heads and streamflows within the natural system that occur in mid-July; therefore, transient simulations were initiated on July 15. Transient simulations for current hydrologic conditions used heads calculated by the calibrated steady-state model as initial conditions; the transient simulation in which pumping was simulated from well MIW 108 at a pumping rate of 1.0 Mgal/d used heads calculated by the steady-state model for scenario 2 (table 4), in which this well was pumped at a rate of 1.0 Mgal/d.

The model was divided into 13 stress periods; the 1st and 13th stress periods were each 15.5 days; the

lengths of the remaining 11 stress periods were equal to the number of days in each respective month (August-June). Each stress period was subdivided into eight time steps, and monthly mean streamflows calculated for the streamflow-gaging station were determined by averaging the eight values calculated for each stress period.

Analysis of Current Hydrologic Conditions and System Response to Proposed Pumping

Results of the first two simulations for current hydrologic conditions for values of specific yield of 0.15 and 0.25 are shown in figure 25*B*. As can be seen in the figure, a specific yield of 0.15 causes the calculated maximum and minimum streamflow to overestimate and underestimate, respectively, the maximum and minimum monthly mean streamflow rates at the gaging station. The maximum and minimum calculated streamflows are closer to the observed flows when a specific yield of 0.25 is used. Nevertheless, the peak streamflow calculated by the transient model when a specific yield of 0.25 is used occurs 1 month earlier than the peak streamflow of the natural system, possibly because the model does not simulate movement and storage of water through the unsaturated zone. Also, the minimum streamflow calculated by the model, 9.6 ft^3/s, is about 2.2 ft^3/s less than the minimum monthly mean streamflow of the natural system. The mean error of the absolute difference between observed and calculated monthly streamflows is 1.73 ft^3/s, or about 13 percent of the calculated steady-state streamflow at the gaging station, 13.3 ft^3/s. Discrepancies between calculated and observed streamflows are the result of uncertainty in several parameters and boundary conditions that affect the modeled system, including monthly recharge rates; hydraulic conductance of the streambed sediments; and hydraulic conductivity, specific yield, and storage coefficient of the aquifer.

Monthly streamflow depletions at the gaging station resulting from a constant pumping rate of 1.0 Mgal/d at the location of observation well MIW 108 range from 0.9 to 1.0 ft^3/s (fig. 25*C*); the mean depletion for all months is 1.0 ft^3/s, which is equal to the depletion at the gaging station for steady-state pumping conditions of 1.0 Mgal/d at the site (scenario 2, table 4). Monthly depletions range from 6 to 8 percent of measured monthly mean streamflows at the gaging station for this simulation and are greatest in October and least in May. In all transient simulations, the net change in storage for the 13 stress periods was less than 3 percent of the total change in storage for all stress periods, indicating that this 1-year cycle nearly simulates a dynamic equilibrium of the system.

Calculated streamflows at the gaging station from late spring through the fall meet the stream-discharge guidelines established by the MDFW; that is, streamflows are greater than 5 ft^3/s during the fall and greater than 2.5 ft^3/s during late spring and summer. Stream-discharge guidelines for the winter and early spring (20 ft^3/s) are not met at the gaging station for a simulated pumping rate of 1.0 Mgal/d. As discussed earlier, however, these guidelines also are not met during winter and early spring for existing streamflow conditions of the river.

SUMMARY

Concern of the Massachusetts Division of Fisheries and Wildlife and local conservation groups over possible reductions in streamflow and degradation of the sea-run brown trout habitat of the Quashnet River, Mashpee and Falmouth, Mass., as a result of proposed ground-water withdrawal prompted an investigation of the Quashnet River stream-aquifer system. This report describes (1) the ground- and surface-water hydrology of the Quashnet River stream-aquifer system and (2) the simulated response of the system to proposed pumping by use of a numerical model of ground-water flow.

Unconsolidated glacial outwash deposits constitute an aquifer underlying the Quashnet River. These deposits are underlain by crystalline bedrock. Analysis of a 5-day, variable-pumping-rate aquifer test by means of a finite-element radial-flow model provided estimates of hydraulic properties of the aquifer in the area of proposed pumping. These properties are horizontal hydraulic conductivities of the upper coarse-grained unit equal to 240 ft/d and of the lower fine-grained unit of 40 ft/d, ratios of horizontal to vertical hydraulic conductivity of 3:1 and 50:1 for the upper and lower units, respectively, specific yield of 0.25, coefficient of specific storage of 1×10^{-6} ft^{-1}, and storage coefficient of 2×10^{-4}.

Two maps showing water-table altitude and configuration indicate a regional horizontal gradient from north to south of about 9 to 10 ft/mi; horizontal gradients steepen sharply near the river, to 380 ft/mi, as a result of convergence of ground-water flow paths at

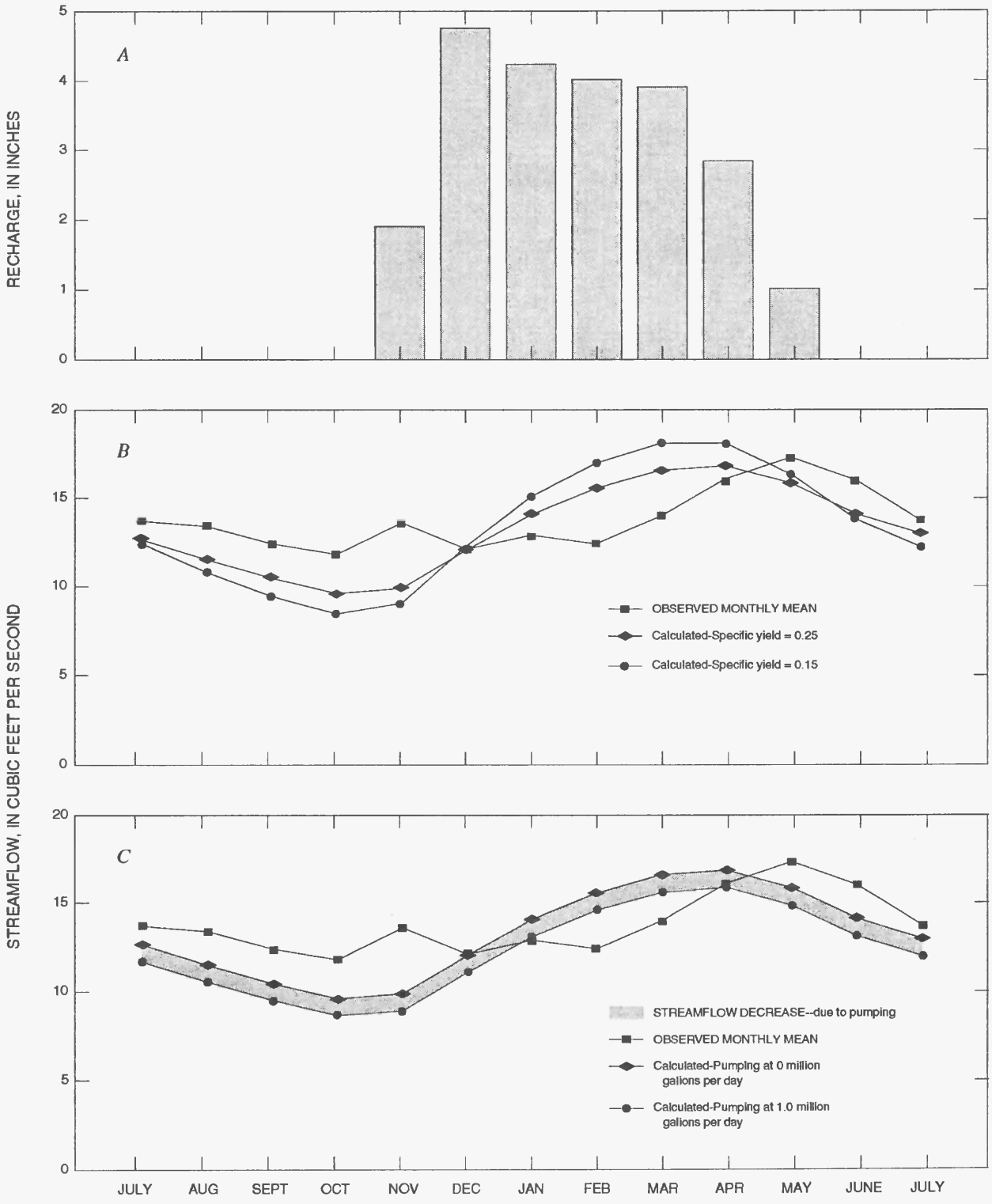


Figure 25.--(A) Monthly recharge specified for the transient model, (B) average monthly streamflow observed and calculated by use of the transient model for the Quashnet River at the streamflow-gaging station for specific yield equal to 0.25 and 0.15 with no pumping and (C) for a specific yield equal to 0.25 with pumping at rates of 0 and 1.0 million gallons per day at a simulated supply well at MIW 108.

the river discharge boundary. The average annual recharge rate to the aquifer from precipitation was estimated to be 21.6 in. Additional sources of recharge to the aquifer are septic-system and irrigation return flow and effluent from the MMR Wastewater Treatment Facility.

The drainage basin of the Quashnet River at a USGS streamflow-gaging station near the mouth of the river at Waquoit Bay is 5.0 mi^2. The drainage basin includes drainage to Johns Pond, from which the river originates. Streamflow of the river at the gaging station 0.3 mi upstream from the river outlet into Waquoit Bay, for the 3-year period October 1988 through September 1991, is 13.8 ft^3/s. Minimum monthly mean streamflow for the 3-year period occurred in October (11.8 ft^3/s), and maximum monthly mean streamflow occurred in May (17.3 ft^3/s).

Streamflow in the Quashnet River meets Massachusetts Division of Fisheries and Wildlife stream-discharge guidelines for sea-run brown trout during most of the year. Mean discharge falls below the recommended levels in winter and early spring, even though the river sustains a highly productive trout habitat throughout the year. The discrepancy between streamflow guidelines and the existing trout habitat indicate that a more detailed understanding of the relation between streamflow conditions and trout ecology of the river is required before the response of the habitat to changing streamflow conditions can be fully described.

A two-layer finite-difference ground-water-flow model was developed to simulate the stream-aquifer system. The steady-state model was calibrated to ground-water heads measured at 50 sites on two separate dates and to streamflow measured along the Quashnet River during a period of near-average streamflow. Mean error between observed and calculated ground-water heads, 1.45 ft, is within 3 percent of the 50 ft of total head loss in the system. Calculated steady-state streamflow at the gaging station is 13.3 ft^3/s, which is similar to the mean flow, 13.8 ft^3/s, for the 3-year period of record.

Steady-state pumping at 0.5, 1.0, and 2.0 Mgal/d was simulated in five scenarios at three hypothetical well sites 1,500 to 3,800 ft west of the Quashnet River. Streamflow depletions in the river resulting from the simulated pumping range from 1.3 to 3.8 percent of calculated steady-state prepumping streamflow at the most upstream active trout-spawning areas, 1.6 mi upstream from the mouth of the river; streamflow depletions range from 3.8 to 14 percent of calculated steady-state prepumping streamflow at the streamflow-gaging station 0.36 mi upstream from the mouth of the river. There was no induced infiltration of streamflow from the river to the aquifer; the river received ground-water discharge along its entire length for all pumping scenarios tested. Streamflow reduction decreases as the location of the simulated supply well is moved further away from the Quashnet River. For the five simulations tested, a maximum of 70 percent of the discharge captured by simulated wells is water that would have otherwise discharged to the Quashnet River. The reduction in streamflow produced by the pumping rate of 1.0 Mgal/d would result in a maximum decline in the average depth of the river at the gaging station of about 0.10 ft, or about 10 percent of the total depth; at a pumping rate of 2.0 Mgal/d a maximum decline of about 0.15 ft, or 16 percent of the average river depth at the gaging station, would result.

A particle-tracking postprocessor to the flow model was used to delineate steady-state contributing areas of the river and to the simulated wells. The contributing area of the river for conditions of no pumping at the proposed well sites lies primarily to the west of the river. The source of water to the river is recharge at the water table within the modeled area and ground-water inflow along the northern boundary of the model. Contributing areas to all three simulated wells for the pumping scenarios tested lie west of the river. Recharge at the water table within the modeled area accounts for 80 to 100 percent of the water captured by the simulated wells. Ground-water inflow along the northern boundary of the model constitutes a maximum of 10 percent of well discharge for simulations in which each of the three wells is pumped separately at 1.0 Mgal/d and 20 percent of well discharge for a simulation in which the well 1,500 ft west of the river is pumped at 2.0 Mgal/d. Although the simulated cone of depression produced by pumping each of the wells extends to the river, the contributing area of each well, consisting of that area in which ground-water flow has been diverted to and captured by the wells, does not include the river. Thus, pumping will reduce flow in the river, but no river water will be diverted to the pumped wells under the pumping scenarios investigated.

Monthly mean streamflow depletions at the gaging station determined by use of the transient model for a constant pumping rate of 1.0 Mgal/d at the simulated pumping well 1,500 ft west of the river range from 0.9 to 1.0 ft^3/s, with a mean monthly streamflow depletion over 12 months of about 1.0 ft^3/s.

Calculated streamflows at the stream gaging station from late spring through the fall meet the stream-discharge guidelines established by the Massachusetts Division of Fisheries and Wildlife. Stream-discharge guidelines for the winter and early spring are not met at the gaging station for this pumping rate; however, these guidelines also are not met for existing (non-pumping) streamflow during winter and early spring.

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