GROUND-WATER RESOURCES OF THE MATTAPOISETT RIVER VALLEY, PLYMOUTH COUNTY, MASSACHUSETTS

By Julio C. Olimpio and Virginia de Lima

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 84-4043

Prepared in cooperation with the

MASSACHUSETTS WATER RESOURCES COMMISSION

Boston, Massachusetts

1984

UNITED STATES DEPARTMENT OF THE INTERIOR

WILLIAM P. CLARK, Secretary

GEOLOGICAL SURVEY

Dallas L. Peck, Director

For additional information write to:

U.S. Geological Survey
150 Causeway Street, Suite 1309
Boston, MA 02114

Copies of this report can be purchased from:

Open-File Services Section
Western Distribution Branch
U.S. Geological Survey
Box 25425, Federal Center
Denver, CO 80225
Telephone: (303) 234-5888

CONTENTS

ILLUSTRATIONS

TABLES

FACTORS FOR CONVERTING INCH-POUND UNITS TO INTERNATIONAL SYSTEM OF UNITS (SI)

For readers who prefer to use SI Units, conversion factors for terms used in this report follow:

Multiply inch-pound units	By	To obtain SI Units
inch (in)	25.40	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)
inch per year (in/yr)	25.40	millimeter per year (mm/yr)
foot per day (ft/d)	0.3048	meter per day (m/d)
foot squared per day (ft^2/d)	0.0929	meter squared per day (m^2/d)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
cubic foot per second per square mile [$(ft^3/s)/mi^2$]	0.02832	cubic meter per second per square kilometer [$(m^3/s)/km^2$]
gallon per minute (gal/min)	0.0630	liter per second (L/s)
million gallons per day (Mgal/d)	0.0438	cubic meter per second (m^3/s)
micromhos per centimeter at 25 degrees Celsius (μmhos/cm at 25°C)		microsiemens per centimeter at 25 degrees Celsius (μmhos/cm at 25°C)

NGVD of 1929 (National Geodetic Vertical Datum of 1929): A geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, called NGVD of 1929, is referred to as sea level in this report.

GROUND-WATER RESOURCES OF THE MATTAPOISETT RIVER VALLEY, PLYMOUTH COUNTY, MASSACHUSETTS

By Julio C. Olimpio and Virginia de Lima

ABSTRACT

Ground-water withdrawals by municipal wells in the Mattapoisett River valley are expected to triple in the next two decades. State and local concern about the long-term impacts of these increased withdrawals on ground-water levels and streamflow made it necessary to assess the ground-water resources of the valley and to develop a digital ground-water-flow model for management purposes.

The model was calibrated under steady-state and transient conditions and simulates ground-water withdrawals by wells, leakage through streambeds, and leakage from the bordering till. Calculated results of the model are most sensitive to decreases in the values of model parameters, particularly streambed and aquifer hydraulic conductivity. Transient-response-time tests of the model indicate that changes in long-term recharge rates would have to last at least 1 year for steady-state predictions to be realized.

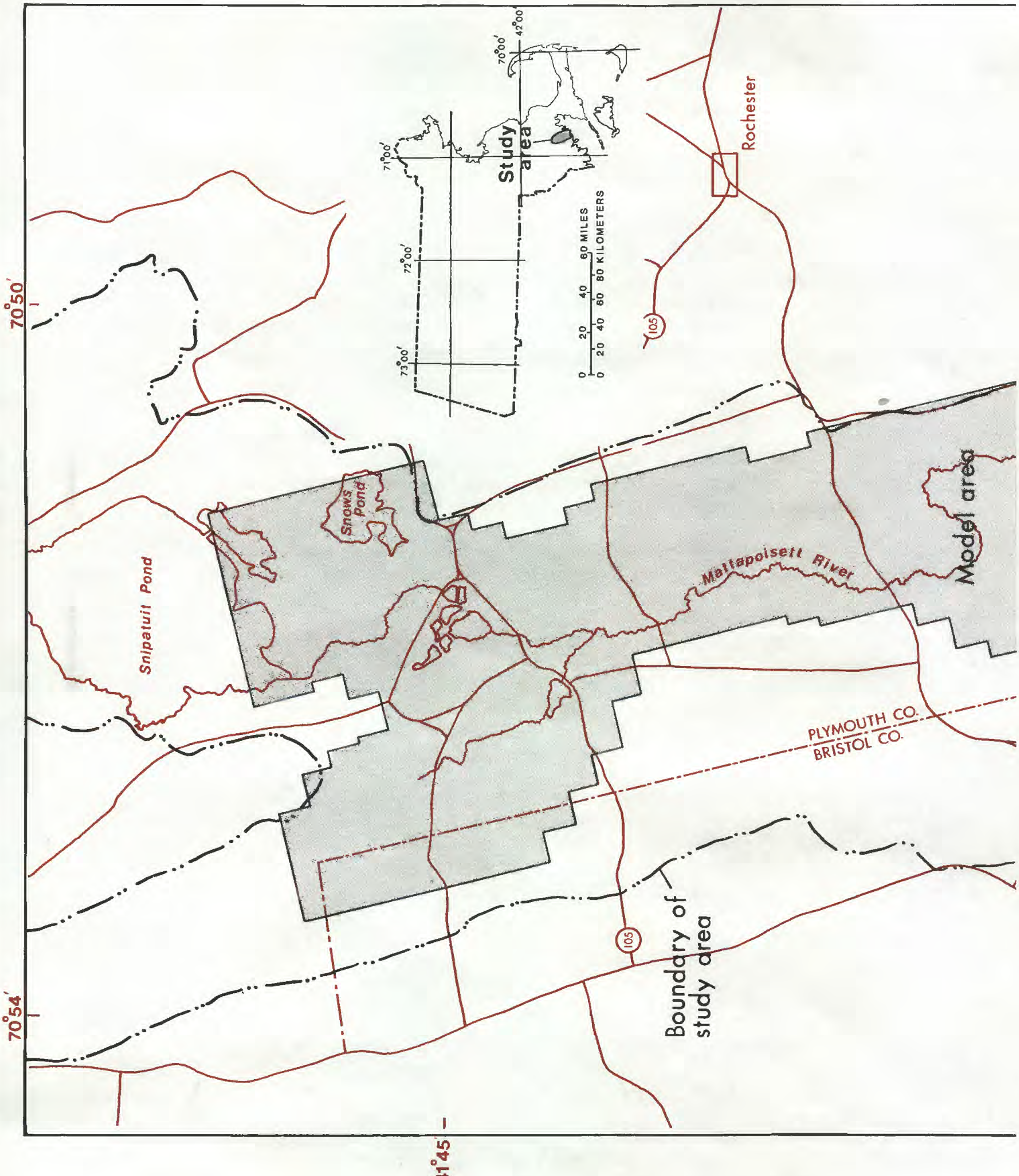
Ten pumping scenarios representing current and proposed withdrawals from the valley were simulated with conditions of reduced recharge. Under conditions simulating 1965 average annual recharge, predicted water levels in the aquifer are as much as 9 feet lower than average annual levels. At the highest withdrawal rates, the predicted drawdown in four wells exceeds the estimated available drawdown. For all pumping scenarios, at least 10 percent of the available ground water in the aquifer discharges to the Mattapoisett River. Under conditions representative of the 7-day 10-year low flow of the river, predicted water levels decline as much as 19 feet; moreover, at the highest withdrawal rates, available drawdown is exceeded in five wells. Simulated withdrawals in six scenarios use all of the available ground-water discharge. If this drought condition should occur and streamflow is not supplemented by surface water, the predictive results indicate that the downstream half of the river will stop flowing under most pumping plans.

Test drilling and seismic refraction surveys conducted to aid model development indicate that the bedrock surface generally is flat except for a deep, narrow channel in the center of the valley. Continuous stream-stage data and baseflow data for the Mattapoisett River were used to increase previous estimates of flow duration, 7-day 2-year, and 7-day 10-year low flow. Water quality in both the aquifer and river may be characterized as slightly acidic and low in dissolved solids.

INTRODUCTION

Background

The towns of Mattapoisett, Fairhaven, and Marion, Massachusetts, obtain all or part of their municipal water supply from wells located in the Mattapoisett River valley. The valley, which is located mostly in the towns of Mattapoisett and Rochester, Massachusetts (fig. 1), is underlain by an 8-mile-long sand and gravel aquifer. The aquifer is narrow (average width, 1 mile) and is as much as 100 feet thick. Ground water from the aquifer discharges to the Mattapoisett River, an important source of fresh water for both the local fishing and cranberry industries. Ground water and surface water are used conjunctively on a small scale in the sense


70°50'
70°54'
41°45'
Snipatuit Pond
Snows Pond
Mattapoisett River
Model area
PLYMOUTH CO.
BRISTOL CO.
Rochester
105
Boundary of study area
Study area
73°00'
72°00'
71°00'
70°00'
42°00'
0 20 40 60 MILES
0 20 40 60 80 KILOMETERS

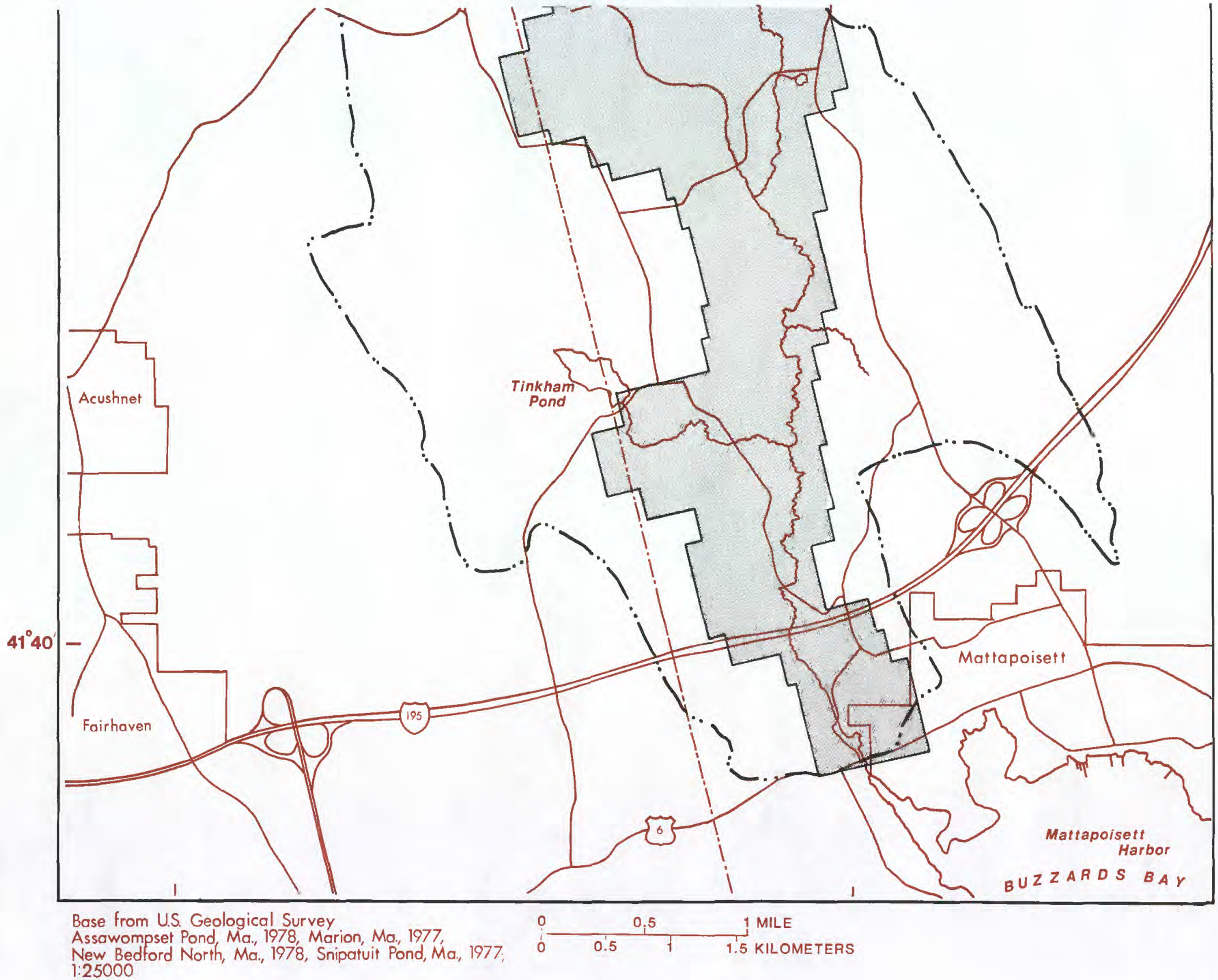


Figure 1.-- Location of study and model area.

that municipal pumpage of water from the aquifer induces infiltration from the adjacent river. A major concern is that substantial, long-term withdrawal of water from the aquifer will reduce streamflow, especially during dry periods when municipal and agricultural water demands are greatest and flow is sustained only by ground water.

Since the mid-seventies, municipal withdrawals from the aquifer have averaged about 1.0 Mgal/d. In the last 2 years, withdrawals have increased substantially; and, when a new well system installed by the town of Fairhaven begins operation in mid-1983, the average withdrawal is expected to be nearly 2.0 Mgal/d. The total pumping capacity of all the current and proposed municipal wells is estimated to exceed 7.0 Mgal/d.

Encouraged by the Massachusetts Department of Environmental Quality Engineering, the towns of Mattapoisett, Fairhaven, Marion, and Rochester have formed an association to devise and implement a valley-wide water-management program. Through this association, the towns hope to balance the use of ground water and surface water according to the availability of and the demand for water.

Purpose and Scope

This study is the first under Chapter 800 Massachusetts legislation which enables quantitative assessment of regional ground-water resources in the State. The U.S. Geological Survey, in cooperation with the Massachusetts Water Resources Commission, selected the sand and gravel aquifer in the Mattapoisett River valley for a detailed ground-water resources study and for demonstrating the hydrologic interdependence between pumping wells and streamflow in the study area. The objective of this study is to evaluate the quantity and quality of water in the Mattapoisett River valley stream-aquifer system and to provide a method for assessing impacts of alternative pumping plans by use of a digital ground-water-flow model.

The study area is the surface-water drainage basin of the Mattapoisett River and is located in southeastern Plymouth County, Massachusetts. Specifically, the data were gathered and the digital model was designed for the area of the basin south of Snipatuit Pond (fig. 1). The model area covers approximately 8 mi^2 and extends from Snipatuit Pond to Buzzards Bay.

Hydrogeologic interpretations in this report are based on data from previous Survey investigations and on new data gathered during 1980-83. The results of test drilling and aquifer tests conducted by private consultants prior to and during the course of this investigation are incorporated in the study.

This report presents the quantitative results of the study and the design, input, and predictive results of the digital model. For a concise, simplified explanation of the study and the model, readers are referred to a companion report by de Lima and Olimpio (1984).

Previous Investigations and Acknowledgments

The geology and ground-water resources of the study area have been the subject of many investigations. Among the investigations which have covered either the Mattapoisett River valley or the southeastern Massachusetts region are those by Sterling (1960), Williams and Tasker (1978), and Metcalf and Eddy, Inc. (1980). Miscellaneous geologic, surface-water, ground-water, and water-quality data that were collected by the Survey prior to this study are published in Maevsky and Drake (1963) and Williams and others (1977). Unpublished data are on file in the Massachusetts Office of the Survey.

The authors are grateful for information obtained from many sources during this study. The Massachusetts Water Resources Commission, the Division of Water Resources, and the Department of Environmental Quality Engineering provided valuable information. Many consulting firms, including Camp, Dresser and McKee, Inc.; Wright-Pierce, Inc.; DuFresne-Henry, Inc.; Metcalf and Eddy, Inc.; and Caswell, Eichler and Hill, Inc., provided, through the towns, results of detailed site studies. The authors are particularly grateful to Jeffrey Osuch and Lucien Fortin, Fairhaven, Mass.; Charles Morgan, William Nicholson, and Carolyn Perkins, Mattapoisett Mass.; and Raymond Pickles and Manuel Costa, Marion, Mass., for their continuous assistance during many discussions of water use. Finally, a special note of appreciation must be given to those landowners who granted their permission to install and monitor wells and stream gages, and to conduct geophysical surveys, on their property.

Method of Investigation

The method of investigation of this study is outlined here to explain briefly the nature and scope of the work. Available data on municipal withdrawals, land use, weather, agriculture, and recreational activities were gathered and compiled. A ground-water observation-well network was established using current wells and new wells that were drilled by the Survey (fig. 2). All the wells were leveled with reference to sea level, and water levels were measured monthly from 1981 to late 1982. The Survey also participated in three aquifer tests that were conducted by private consulting firms.

Geologic data were gathered during the drilling of the Survey wells, and core samples were collected for analysis. A seismic refraction survey was conducted along three east-west lines in the valley to obtain continuous profiles of aquifer thickness and water level across the valley.

Three stream-gaging stations were installed along the Mattapoisett River to record stream stage continuously and to measure streamflow monthly (fig. 2). In addition, streamflow measurements were conducted along the river and its tributaries during baseflow conditions in 1982 to measure streamflow gains and losses. During February 1982, flood-flow measurements were made near two of the three stream gages.

Water samples were collected in 1981 and 1982 from seven wells and at the three stream gage sites (fig. 2). The samples were analyzed for common constituents, insecticides, pesticides, and volatile organic compounds to assess water quality.

A two-dimensional digital ground-water-flow model was applied to simulate flow, discharge to streams, and municipal withdrawal. For demonstration purposes, a series of pumping scenarios was devised and tested using the model to illustrate the impacts of pumpage on water levels in the aquifer and on streamflow.

Geography

Physiography and Climate

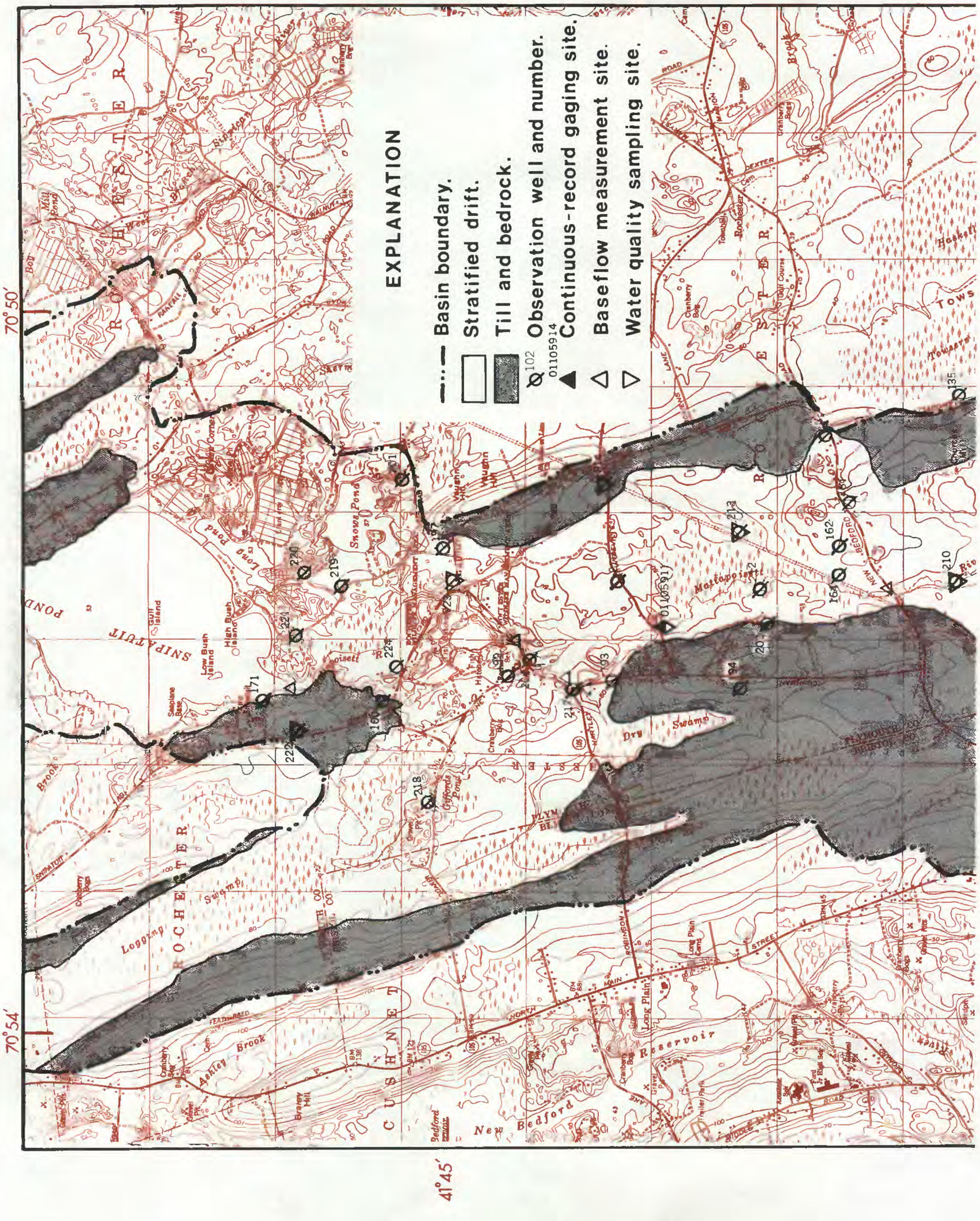
The study area is located in the Coastal Lowlands physiographic province of New England (Denny, 1982). Land-surface altitudes range from sea level to over 100 feet along the north and northwest borders of the drainage basin. Land surface is composed of smooth hills surrounded by flat, low-lying wetlands. The Mattapoisett River flows southward from Snipatuit Pond to Buzzards Bay.

The climate is characterized by warm summers and relatively mild, wet winters. Long-term (1960 to 1980) annual precipitation at Rochester, Mass., is 48.7 inches and is fairly evenly distributed throughout the year. Free-water surface evapotranspiration is approximately 28 in/yr (Farnsworth and others, 1982; Williams and Tasker, 1978), and about 75 percent of the evapotranspiration takes place during the period May to September.

Land Use and Population

In the Mattapoisett River basin, agricultural and forested land constitutes 80 percent of total area, and urban development covers the remaining 20 percent. Most of the urbanized area is located in the southern part of the basin. The projected development of land in 2020 (New England River Basins Commission, 1975) shows a 50 percent increase in urban area, primarily as urban and light-industrial development in this part of the Boston-Providence corridor.

Currently, there are no large towns in the Mattapoisett River basin, and only a small population increase of 0.8 percent is projected for 2020 (New England River Basins Commission, 1975). However, projected increases in light industry and the small projected population rise are expected to increase water usage by 100 percent.


EXPLANATION
Basin boundary.
Stratified drift.
Till and bedrock.
102 Observation well and number.
01105914 Continuous-record gaging site.
Baseflow measurement site.
Water quality sampling site.
70°50′
70°54′
41°45′
SNIPATUIT POND
Long Pond
Snows Pond
Dry Swamp
Mattapoisett
Logging Swamp
Ashley Brook
New Bedford Reservoir
Long Plain

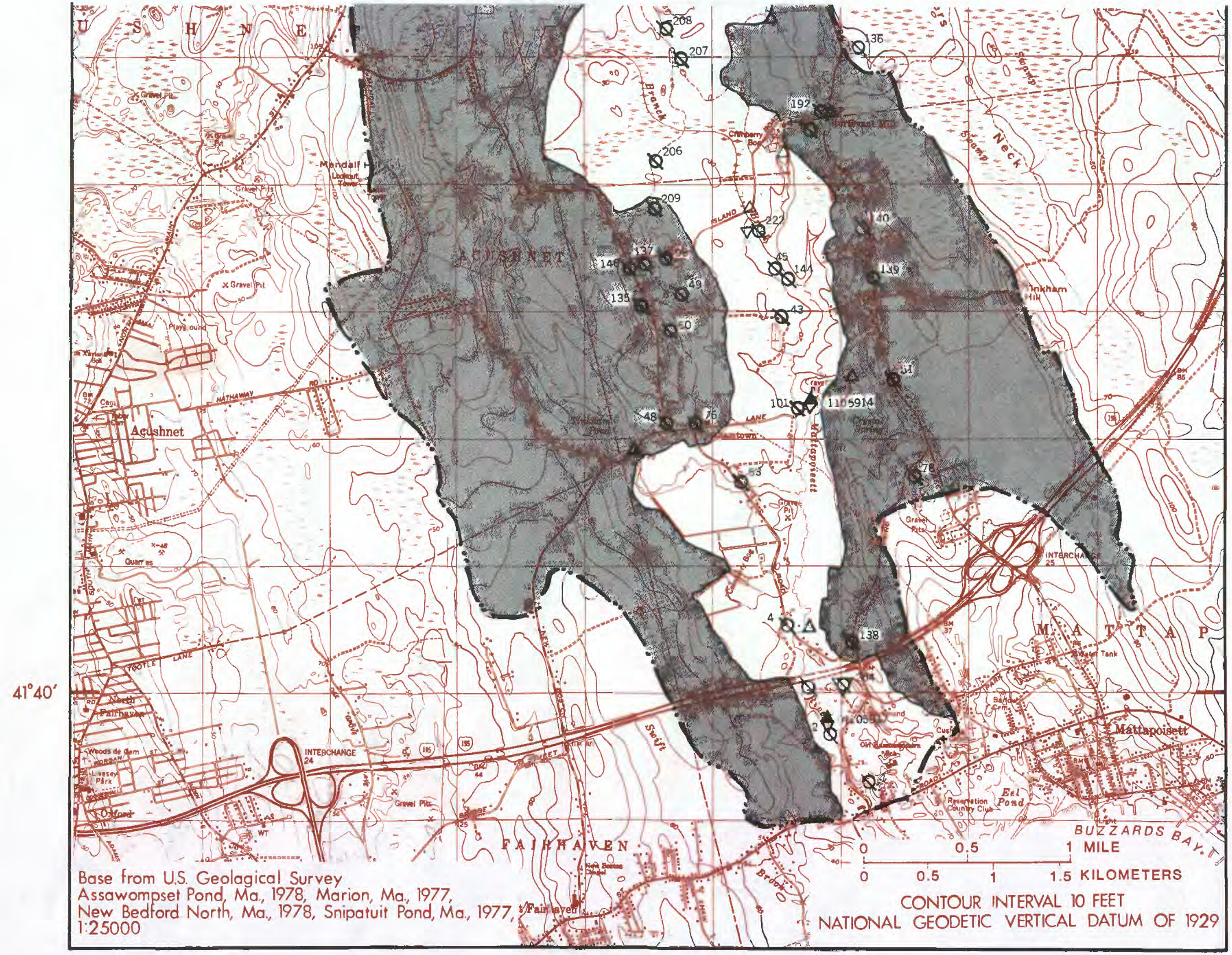


Figure 2.-- Ground-water and surface-water observation sites and surficial sediments of the Mattapoisett River drainage basin.

Geology

Bedrock

Bedrock underlying the Mattapoisett River valley is predominantly granite and granite gneiss that is moderately weathered and fractured. The bedrock is relatively impermeable compared with overlying sand and gravel. Nevertheless, many bedrock wells are used in the valley for domestic water supply.

The altitude of the bedrock surface, determined from previous studies (Sterling, 1960; pl. 1), new drilling data, and seismic-refraction surveys, is shown in figure 3. The relief of the bedrock surface is greater than that of the land surface. The principal feature is a deep, narrow, valley with an axis that coincides with the Mattapoisett River valley. The depth to bedrock from land surface ranges from 0 where bedrock crops out on hills in the middle of the valley, to 110 feet, reported at the Fairhaven test site off Tinkham Lane (Caswell, Eichler and Hill, 1983).

Surficial Sediments

The Mattapoisett River basin is composed chiefly of unconsolidated glacial sediments that overlie bedrock and form the land surface. The most common types of sediments are till and stratified drift (fig. 2).

Till is an unsorted mixture of sand, gravel, clay, and rock fragments that overlies bedrock in about 45 percent (10.8 mi^2) of the drainage basin. Till forms the upland on the east and west sides of the valley and also occurs in the center of the valley as a thin, hard, coarse-grained layer beneath the stratified drift. In the study area, the boundary between till and stratified drift is marked by a small, but distinct, downward slope toward the center of the river valley.

Stratified drift generally is composed of layers of sand and gravel with some interbedded layers of silt and clay. Drift covers about 55 percent (12.8 mi^2) of the basin, fills the north-south trending bedrock valley, and forms the lowlands of the river valley. Thickness of the drift ranges from a few feet near the aquifer boundaries to over 100 feet in depressions in the center of the bedrock valley. Despite the large variation, drift thickness commonly ranges from 30 to 70 feet.

Detailed descriptions of the lithologic and water-bearing characteristics of the stratified drift in the valley have been given by Sterling (1960) and Williams and Tasker (1978); therefore, only a brief geologic description is given here. Most of the drift in the basin consists of fine to medium sand. Exceptionally clean, well-sorted sand deposits are located in the lower part of the valley south of Tinkham Lane and in the central part of the valley between New Bedford Road and Rounseville Road. A broad sand plain extends eastward and northward of the Snows Pond-Snipatuit Pond area. Stratified silt, sand, and fine gravel deposits, up to 20 feet thick, occur along the western side of the valley from Wolf Island Road to south of Tinkham Lane. Similar deposits that are somewhat thicker and contain large boulders are located on the east side of the valley along New Bedford Road and in the northwest part of the valley between Snipatuit Pond and Quaker Lane.

The stratified drift comprises the principal aquifer of the Mattapoisett River valley. The aquifer and the river are the primary sources of water supply in the basin. The river flows over the aquifer; therefore, they are hydraulically connected and respond together to stresses such as drought and pumpage. The stream-aquifer system supplies a significant quantity of water for public and agricultural supply in this area of Massachusetts. In the following section, the hydrogeologic characteristics of the aquifer are discussed in detail.

HYDROGEOLOGY

Generalized Framework

Ground water in the stratified-drift aquifer is mostly unconfined. Recharge to the aquifer is by infiltration from precipitation and by lateral leakage from adjacent till. Discharge from the aquifer is by leakage to the Mattapoisett River and to the small ponds in the valley, by evapotranspiration in areas where ground water is near land surface, and by nonreturned pumpage. The water table marks the top of the saturated zone in the drift, and its level fluctuates continuously in response to changes in recharge and discharge.

As a general rule, the surface of the water table conforms to that of the land, but is more subdued. Gradients of the water-table surface indicate both the approximate directions of ground-water flow and the relative differences in flow rates. Ground water in the stratified-drift aquifer moves laterally from the till boundaries toward the river and ocean. Flow directly from the aquifer into the ocean is negligible in comparison to total flow through the aquifer due to very low gradients and the small size of the aquifer near the seacoast. A more detailed discussion of the ground-water system is given in the section "Occurence of Ground Water."

The Stratified-Drift Aquifer

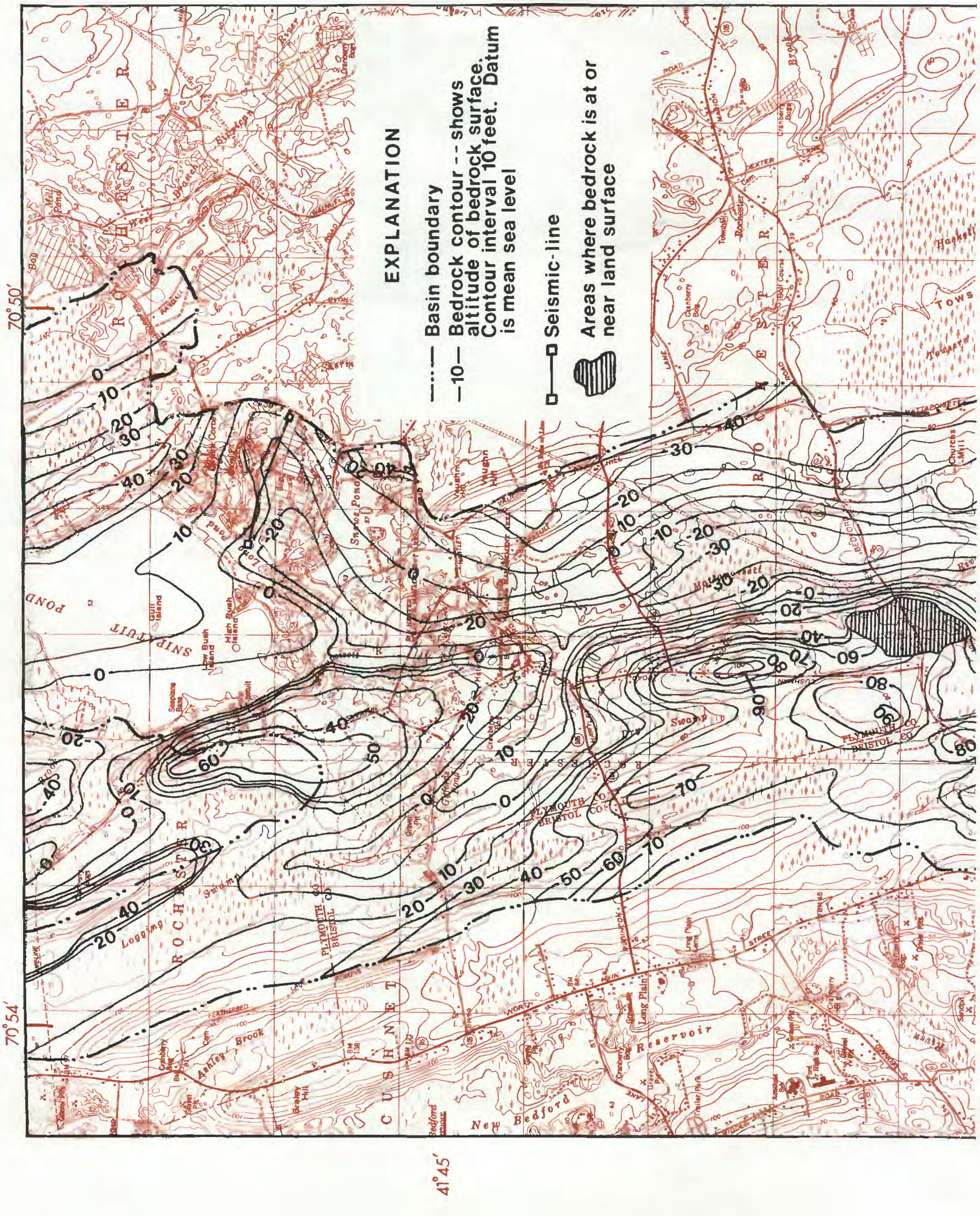
Water Table

The approximate altitude of the water table in the stratified-drift aquifer, as measured in May 1982, is shown in figure 4. The water-level data were obtained from a network of 58 observation wells. In constructing the water-level contours, the ground-water altitude adjacent to the Mattapoisett River and the nearby ponds was assumed to coincide with the surface-water altitude.

The altitude of the water table ranges from near sea level at the coast to nearly 90 feet in the northern part of the valley. Relatively steep water-table gradients occur near the till/stratified-drift boundary on the east and west sides of the valley, and relatively gentle gradients occur across the broad valley floor. Steep gradients are also common near Tinkham Lane and south of Interstate 195 where the valley is narrow, and north of Hartley Road where land elevation increases significantly. The water table commonly is a few feet below land surface in most of the low-lying areas, and 8 to 15 feet below land surface in hilly areas and in the adjacent till.

The altitude of the water table fluctuates seasonally, as illustrated by the hydrographs in figure 5. Highest water levels occur during the period from winter to early summer. Lowest water levels occur during the period from late summer to early fall. The seasonal range of water levels in wells finished in stratified drift is 3 to 5 feet (wells MJW 45, MJW 53). In contrast, the seasonal range of water levels measured in wells in till is generally 10 to 20 feet (wells MJW 51, RFW 222).

The 10-year hydrograph of a stratified-drift well, WFW 51, in nearby Wareham, Mass., shows the long-term, consistent, annual variation of water level that is common in stratified-drift aquifers in the region. No significant trends in the hydrograph are apparent, with the possible exception of that in the 1980-81 period, indicating that the annual recharge to stratified-drift aquifers in the 1972-82 period was adequate to replenish ground-water storage.


EXPLANATION
Basin boundary
—10— Bedrock contour -- shows altitude of bedrock surface. Contour interval 10 feet. Datum is mean sea level
Seismic-line
Areas where bedrock is at or near land surface
70°50'
70°54'
41°45'

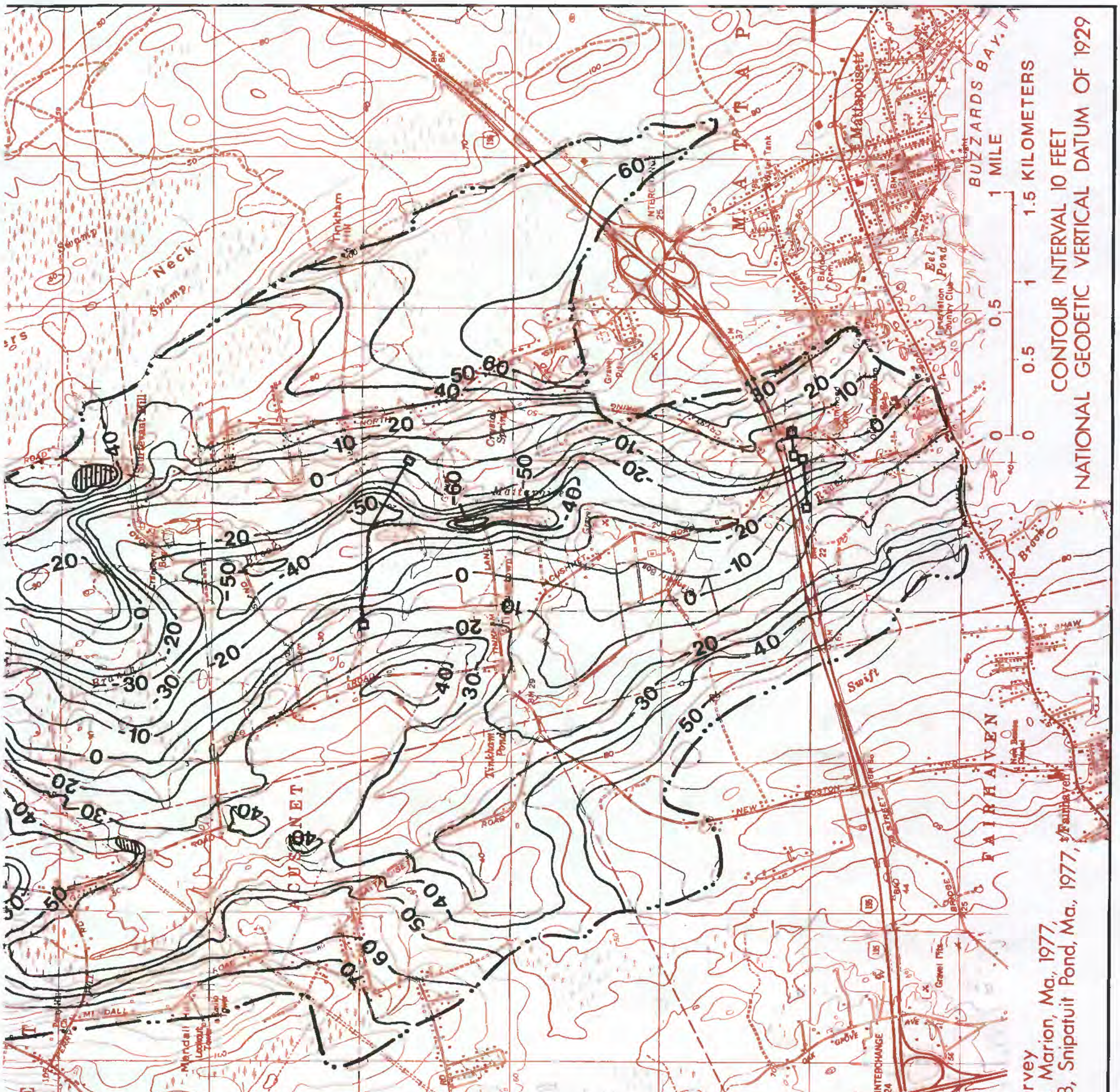
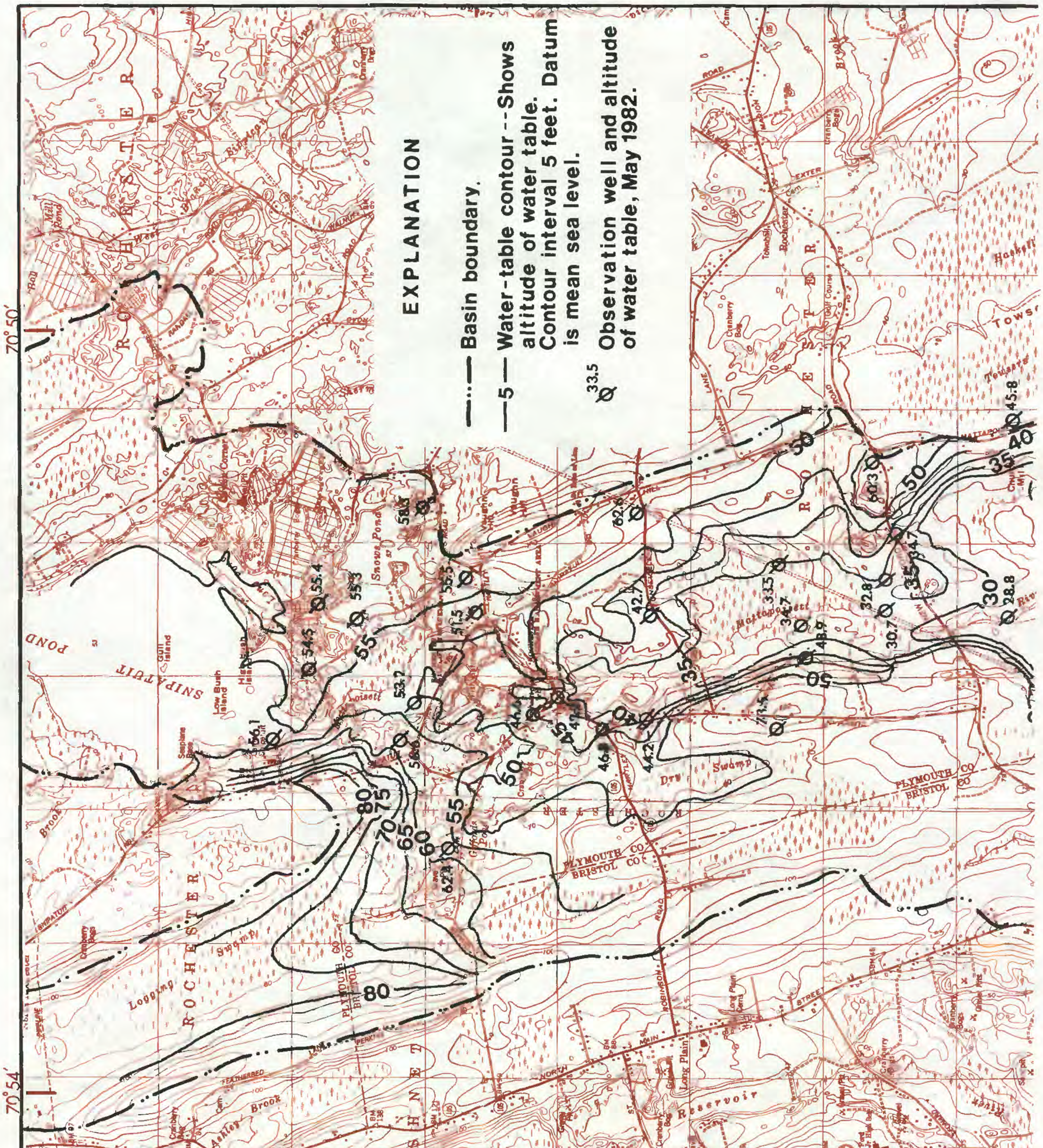


Figure 3.-- Bedrock topography.


EXPLANATION
Basin boundary.
5 — Water-table contour--Shows altitude of water table. Contour interval 5 feet. Datum is mean sea level.
33.5 Ø Observation well and altitude of water table, May 1982.
70°50'
70°54'
41°45'
SNIPATUIT POND
Snows Pond
Mattapoisett
Dry Swamp
Leonard Pond
Rochester
PLYMOUTH CO
BRISTOL CO
Long Plain
New Bedford Reservoir
Cushnet
Ashley Brook
Loggingmp Swamp

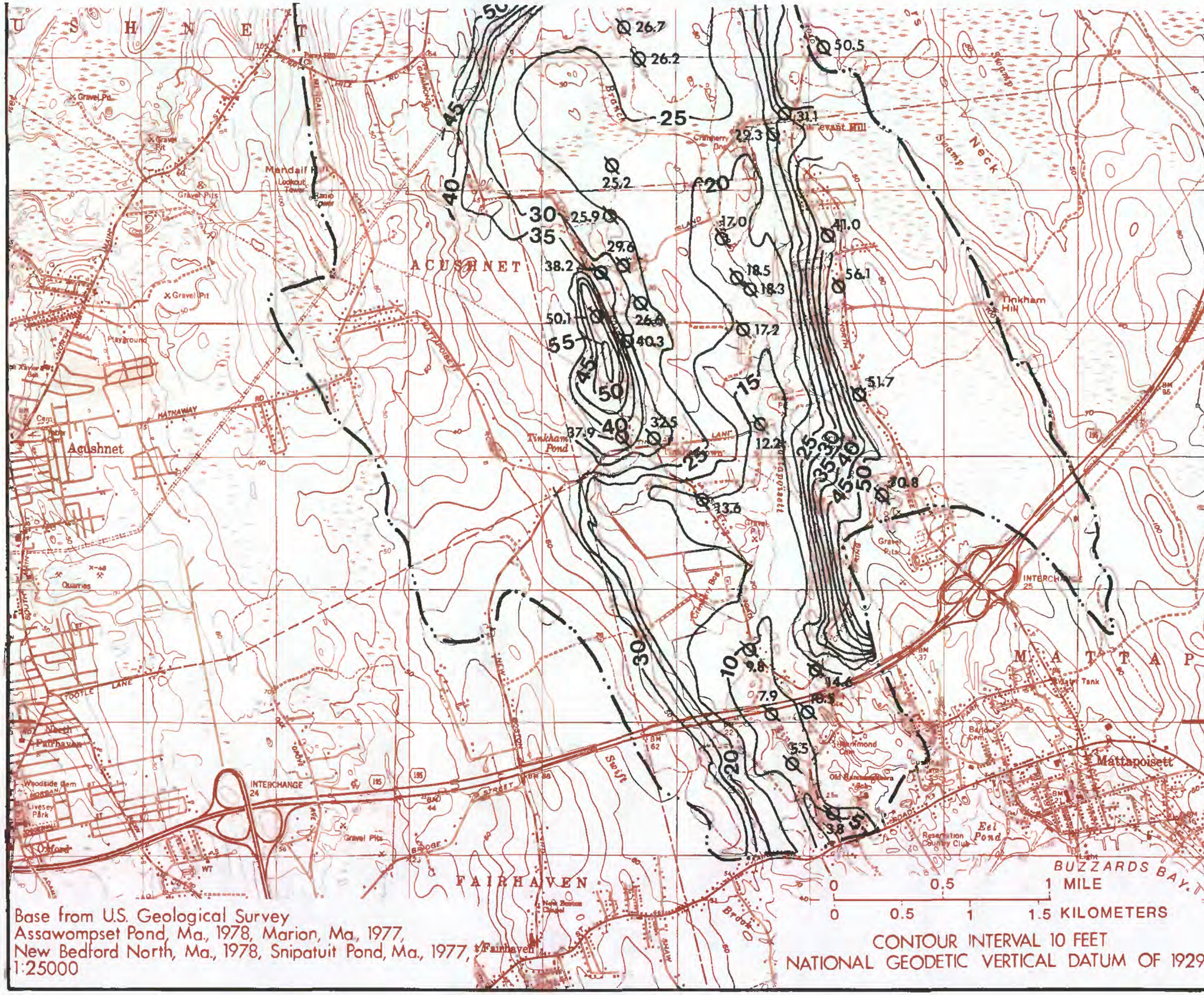


Figure 4.-- Altitude of water table, May 1982.

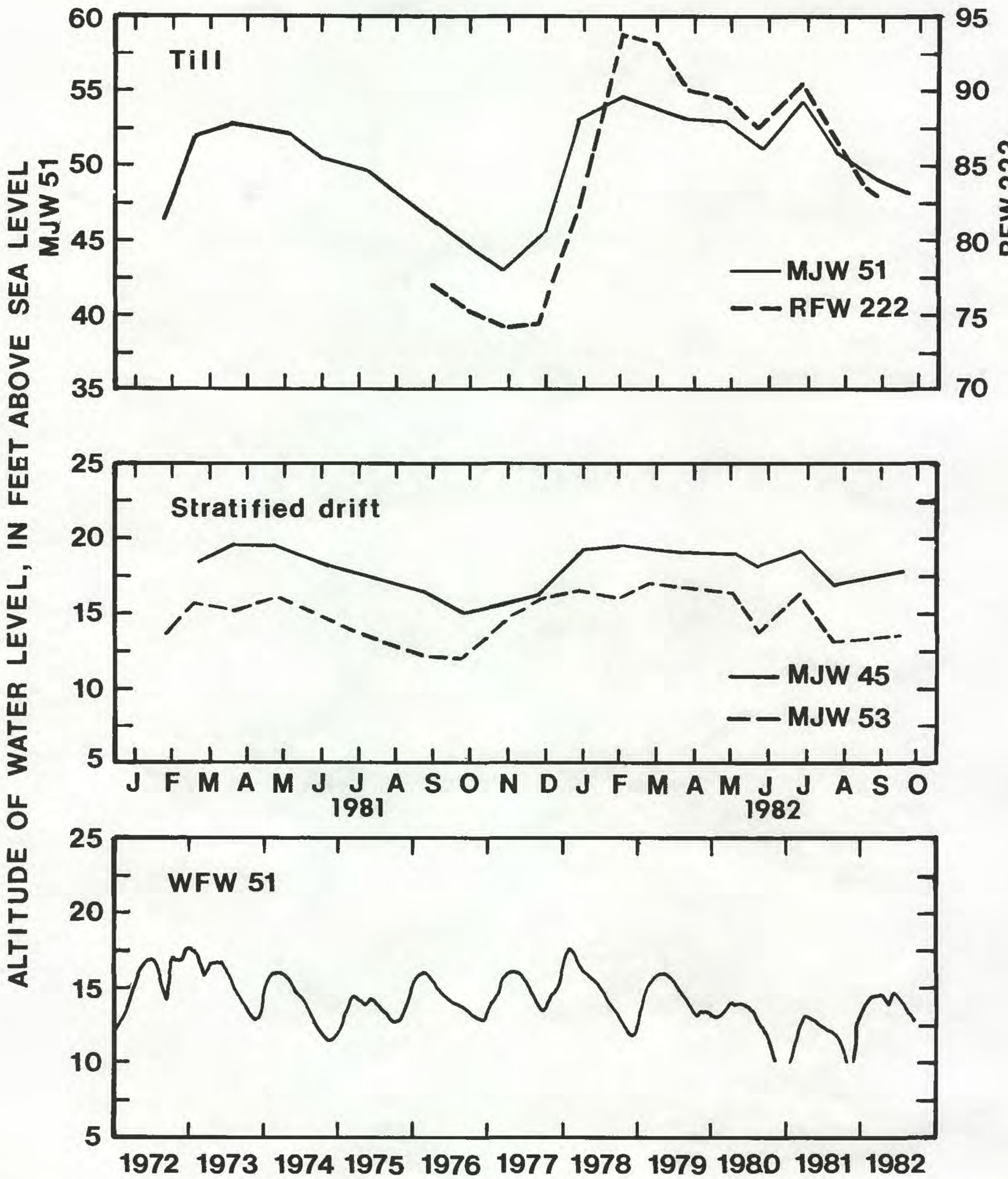


Figure 5.--Hydrographs of wells in stratified-drift and till.

Saturated Thickness

The saturated thickness of the stratified-drift aquifer is the distance from the water table to the thin till sediments that cover the underlying bedrock. Water levels in May 1982 were used to construct a saturated thickness map of the aquifer (fig. 6). The thickness contours, similiar to those illustrated by Williams and Tasker (1978; sheet 1), indicate that the greatest saturated thicknesses coincide with the deepest depressions in the bedrock surface. Also, relatively thick saturated drift (average thickness, 70 feet) occurs in the northern part of the valley where land elevation rises and the bedrock valley widens and splits into two parts (Sterling, 1960; pl. 1).

Hydraulic Conductivity and Grain Size

The hydraulic conductivity of the aquifer was estimated from lithologic data obtained at Survey well sites and from aquifer thickness and transmissivity data derived from aquifer tests at municipal well sites. Estimated values of horizontal hydraulic conductivity are based on the relation between the grain size of the stratified drift and conductivity (Masch and Denny, 1966; Krumbein and Monk, 1943; Ryder and others, 1970). Values range from 0.01 ft/d along the till/stratified-drift boundary to 320 ft/d in coarse-grained sand and gravel in the center of the valley. Average values in the drift are most commonly 50 to 150 ft/d and agree closely with values of hydraulic conductivity obtained from the aquifer tests at the municipal well sites.

The grain-size distribution of three glacial sediments typical of those in the basin, and a partial list of core samples and their estimated hydraulic conductivity, are shown in figure 7. The silt and sand samples are well sorted, and the range in grain size is rather small. On the other hand, the till is unsorted with grain sizes ranging from that of silt to gravel.

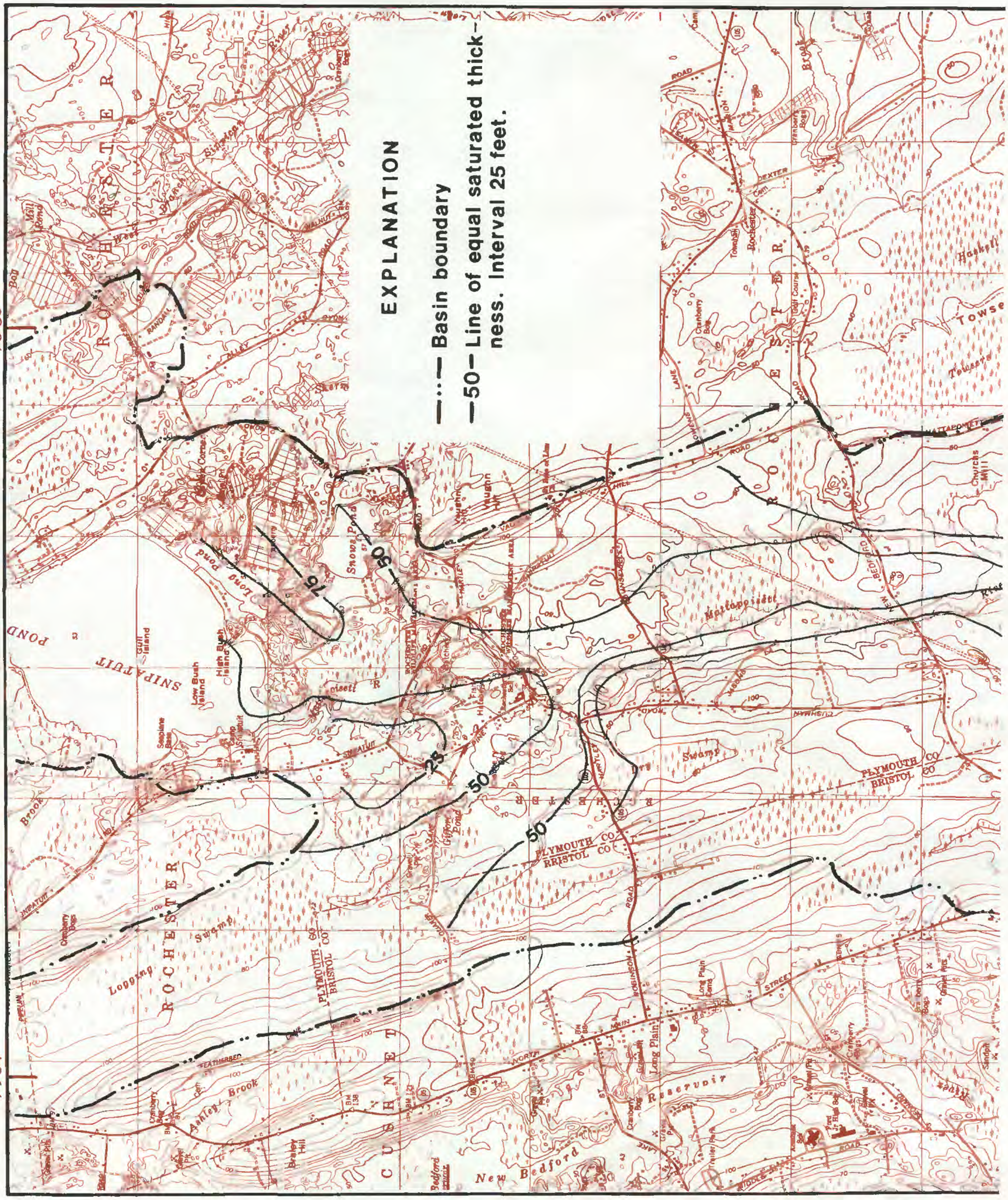
In general, the stratified drift in the basin is relatively fine grained (Williams and Tasker, 1978) and median-grain sizes often are less than 0.30 mm. Estimates of hydraulic conductivity used in this study are based on the median grain size and the degree of sorting of the sediments (Masch and Denny, 1966). Estimates are conservative in comparison to those derived from other techniques (Krumbein and Monk, 1943; Ryder and others, 1970).

Confining Bed

In the vicinity of Wolf Island Road, a relatively thick, clay- and silt-rich layer is interbedded in the stratified drift. Geologic logs of Survey wells and numerous municipal test wells indicate that the clay-silt layer ranges in thickness from 10 to 60 feet and separates an upper 10- to 20-foot-thick sand layer from a lower 12- to 20-foot-thick sand and gravel layer. Lateral extent of the clay-silt layer is relatively small. From Wolf Island Road, the layer extends northward about 1,500 feet near, but not to, wells RFW 206 and the Marion municipal supply well (fig. 8). Southward, the confining bed extends about 2,000 feet to the vicinity of the Mattapoisett well test site 11-6. The eastern and western margins of the layer are more poorly defined; available data indicate the layer is about 2,500 to 3,000 feet wide (Wright-Pierce, Inc., 1980; Wright-Pierce, Inc., 1981; Wright-Pierce, Inc., 1982; Camp, Dresser and McKee, 1971).

The clay-silt layer acts as a low-permeability confining bed that restricts ground-water movement between the upper and lower sand layers. The presence of the confining bed is particularly important in view of Fairhaven's recent completion of three large-diameter wells that are finished in the lower, confined sand layer. To help clarify the hydrogeology of this area, ground-water movement is discussed in greater detail below, and the pattern of ground-water flow is illustrated in figure 8.

Most wells finished in the confined aquifer have water levels that are either near or above the top of the overlying unconfined aquifer. In the spring, water levels in most wells, particularly in wells in the center of the confined aquifer, are above land surface. Under these conditions, hydraulic head in the confined aquifer is higher than that in the upper sand layer, and water seeps upward through the confining layer to the unconfined aquifer (fig. 8A). The rate of seepage through the clay-rich confining bed is relatively slow compared with the rate of water flow through both the underlying and overlying sand-rich aquifers. Water enters the confined aquifer chiefly by horizontal ground-water flow from the unconfined stratified-drift aquifer east, west, and north of the Wolf Island Road area.


EXPLANATION
—··— Basin boundary
—50— Line of equal saturated thickness. Interval 25 feet.
70°50′
70°54′
41°45′
75
50
25
SNIPATUIT POND
Long Pond
Snows Pond
Mattapoisett
Swamp
PLYMOUTH CO
BRISTOL CO
ROCHESTER
ACUSHNET
Logging Swamp
Ashley Brook
New Bedford Reservoir
Long Plain
Haskell
Towse
Churchs Mill

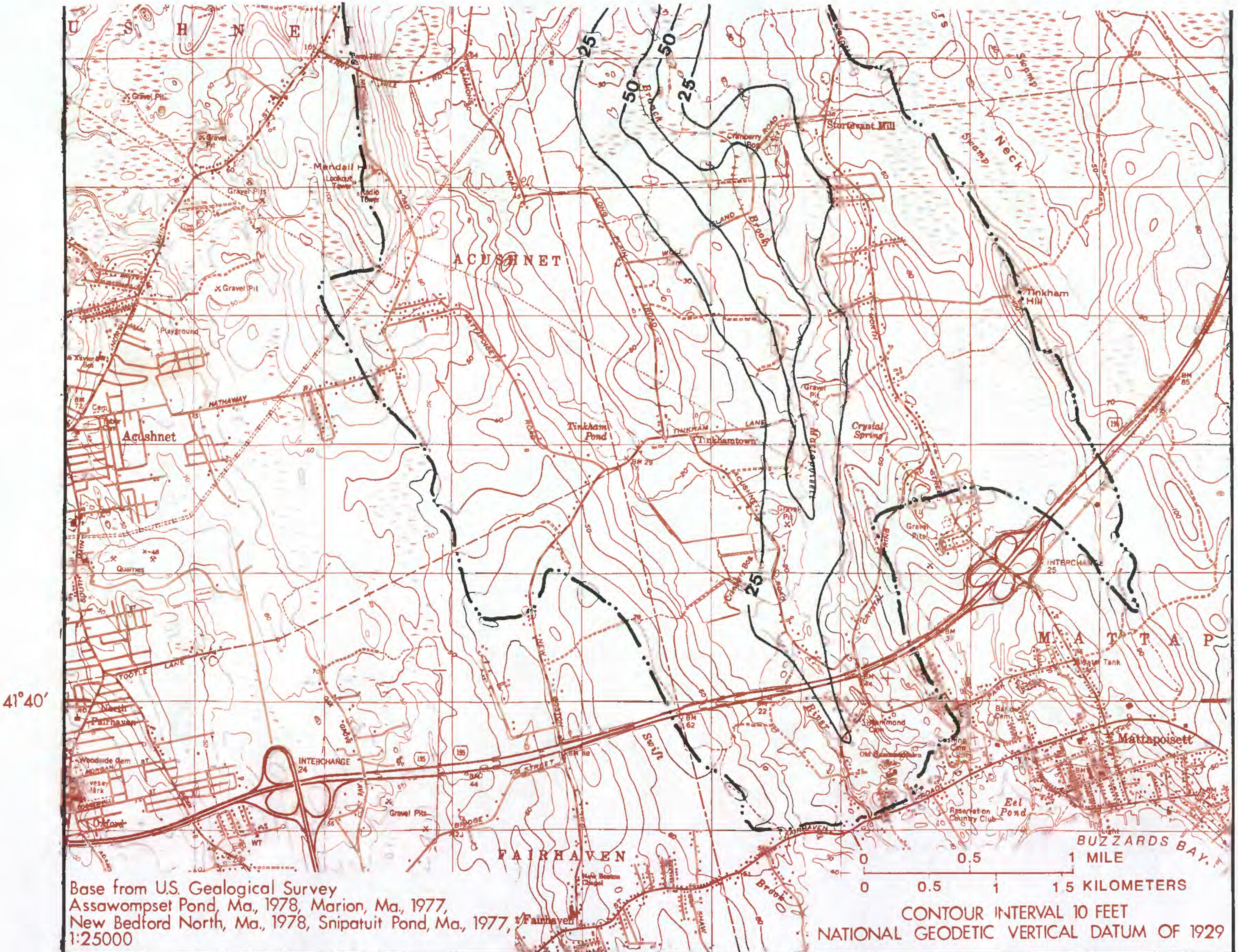
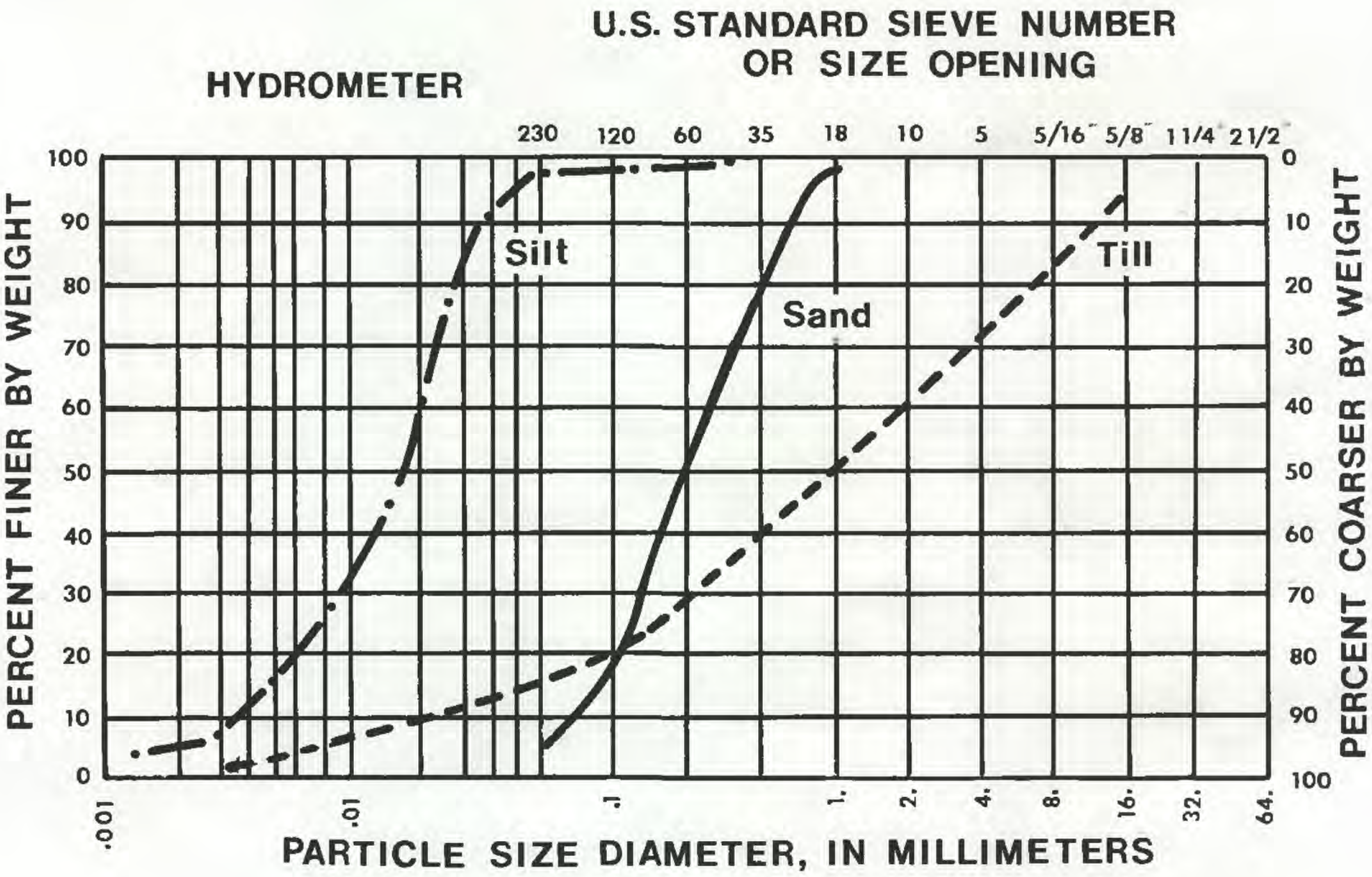


Figure 6.-- Saturated thickness of the stratified-drift aquifer.



Well	Sample number	Sample description	Median grain size (millimeters)	Hydraulic conductivity (feet per day)		
				Masch and Denny (1966)	Krumbein and Monk (1943)	Ryder and others (1970)
208	22	Sand, medium to coarse, well sorted	0.49	65	182	320
224	26	Sand, medium, loose	.28	25	29	100
213	13	Sand, fine to medium	.21	25	27	27
219	2	Sand, fine	.25	25	31	40
219	1	Sand, fine to medium	.24	23	24	40
212	19	Sand, fine to medium	.19	20	14	27
213	14	Sand, fine to medium	.15	20	15	27
206	24	Sand, fine	.13	15	5	20
214	16	Sand, fine, silty	.16	12	5	20
215	17	Sand, fine, silty	.23	10	26	27
206	23	Sand, silty	.05	10	.8	6
215	18	Silt	.04	10	.6	5
219	5	Silt	.02	10	.08	—
210	20	Silt	.01	7	.01	—

Figure 7.--Grain-size distributions of glacial sediments

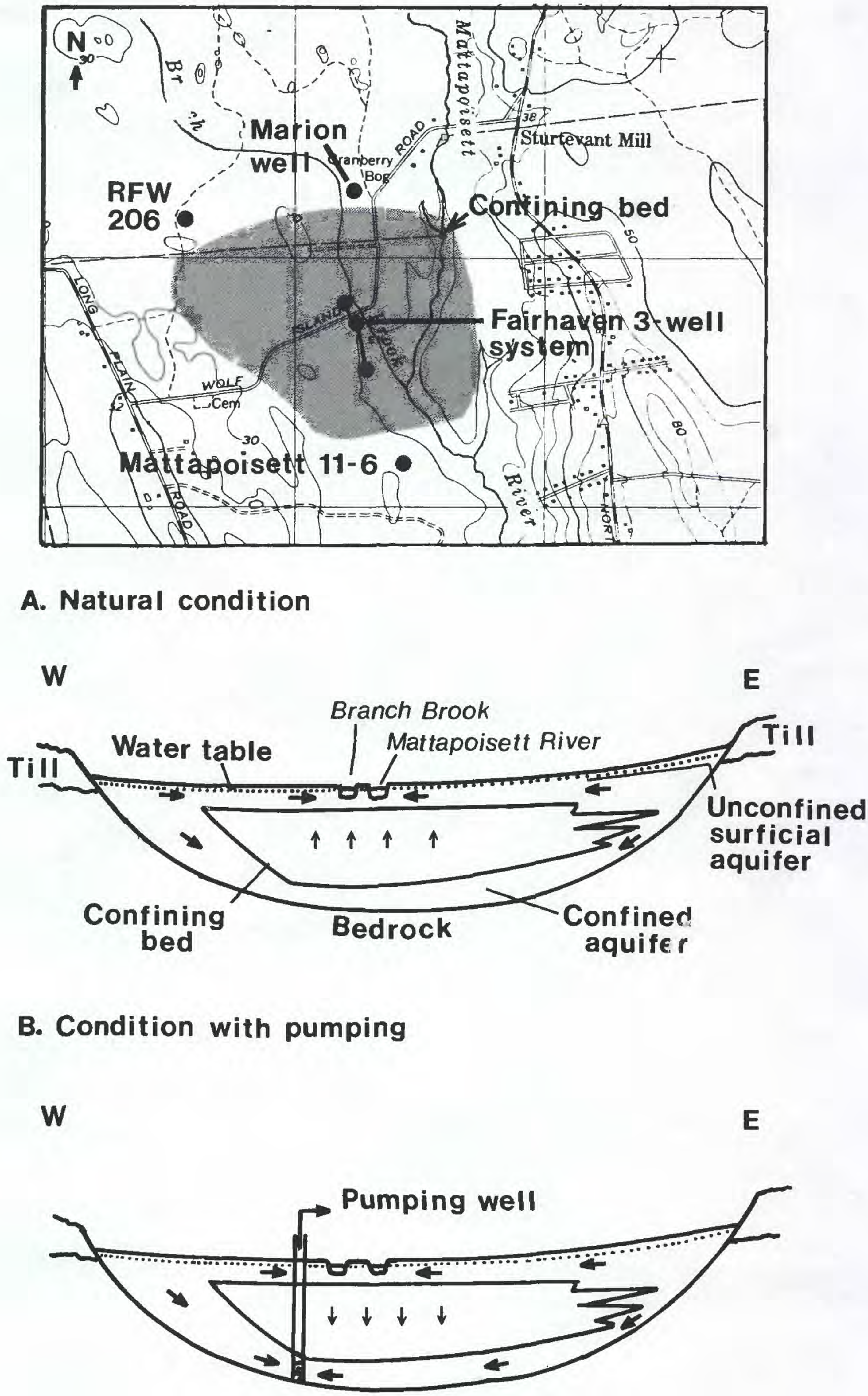


Figure 8.-- Idealized sketch of the aquifer system in the vicinty of Wolf Island Road.

When wells that are screened in the confined aquifer are pumped heavily, hydraulic head in the confined aquifer is lowered below the elevation of the water table in the upper sand layer. Under these conditions, the natural upward seepage through the confining bed is reversed (fig. 8B), and water seeps downward to replenish the confined aquifer. Aquifer tests have resulted in very large drawdowns in the pumping wells due in large part to the small thickness of the confined aquifer. Specific-capacity data reveal the differences in the performances of wells that are finished in the confined and unconfined aquifers. The specific capacities of wells in the confined aquifer are commonly 5 to 9 (gal/min)/ft. The capacities of wells in the unconfined aquifer are as much as 65 (gal/min)/ft.

Occurence of Ground Water

Recharge

The primary source of recharge to the stratified-drift aquifer is infiltration from precipitation. The long-term (National Oceanic and Atmospheric Administration, 1960-83) average water year precipitation at Rochester, Mass. (National Oceanic and Atmospheric Administration station 6938), was 48.7 inches. Despite several recent years of abnormally high and low precipitation, 1982 water-year precipitation at Rochester was nearly normal at 47.6 inches.

Based on the assumption that near-normal precipitation results in near-normal basin runoff, total-runoff data of the Mattapoisett River basin for the 1982 water year (October 1, 1981, through September 30, 1982) were used to estimate an average annual recharge rate. The total annual runoff measured at the River Road stream gage near Mattapoisett was 24.1 inches. Applying the relations of total runoff, basin geology, pumpage, ground-water runoff, and recharge (Cervione and others, 1972; Mazzaferro and others, 1979), the recharge to the stratified-drift aquifer in the 1982 water year was estimated to be 15.9 inches.

In addition to infiltration of precipitation, a second source of water to the aquifer is leakage from till. Leakage, which consists of ground-water runoff from the till-covered uplands along the east and west sides of the river valley, is assumed to equal the recharge to the till. In most areas, this assumption is valid because water losses in the till due to evapotranspiration are relatively small and pumpage is light.

Few data are available on the ground-water runoff of till-covered land in either southern Massachusetts or New England, and no data were collected in the study area. A study by Morrissey (1983) indicates that the mean annual ground-water runoff from till is approximately 0.5 $(ft^3/s)/mi^2$ of drainage area. Using 6.3 mi^2 as the area of till-covered land southwest of Snipatuit Pond that contributes directly to the stratified-drift aquifer (fig. 9), total ground-water runoff from till is estimated to be approximately 6.8 inches. This is about 14 percent of total precipitation for the 1982 water year.

Direction of flow

Generalized directions of ground-water flow in the stratified-drift aquifer are shown in the water-table map of May 1982 (fig. 9). As indicated by the arrows, regional flow is from areas of high water-table altitude to areas of low water-table altitude.

The flow pattern illustrated in figure 9 shows the widespread, horizontal movement of water from the sides of the valley to the river. Not shown are areas where vertical water flow may occur; for example, along the till/stratified-drift boundary, within and adjacent to the confining layer at Wolf Island Road, in the immediate vicinity of pumping wells, and beneath the streambed of the Mattapoisett River. Moreover, the water-table contours and flow lines indicate the extent of the ground-water-flow system in the Mattapoisett River basin. Water from recharge within the basin flows through the stratified drift and discharges primarily to the river and its tributaries. Little ground water discharges directly to the ocean at the southern end of the valley because the stratified drift is thin, narrow, and unsaturated in places, particularly near the boundary with the till. In effect, the ground-water-flow system of the stratified-drift aquifer in the drainage basin is self-contained and largely separate from any regional ground-water-flow system, excluding leakage to and from the underlying bedrock.

Discharge

The Mattapoisett River and its tributaries are the principal areas of ground-water discharge from the stratified-drift aquifer. Three stream gaging stations were constructed to record stream stage continuously and to measure discharge monthly (Rounseville Road, station number 01105911; Tinkham Lane, station number 01105914; and River Road, station number 01105917; fig. 2). Streamflow and discharge data collected during this study that are not included in this report are on file in the Massachusetts Office of the Survey.

In addition to the monthly discharge measurements, five baseflow measurements were conducted in 1982 to determine streamflow gains and losses (table 1). Baseflows were measured at up to eight locations along the trunk of the river, at five locations where tributaries enter the river, and at two locations where surface-water inflow from outside the basin occurs. The measurements were conducted during long periods of little or no precipitation when surface-water runoff was low. The baseflow data shown in table 1 represent the cumulative ground-water discharge to the river and its tributaries at various flow durations.

Table 1.—Summary of ground-water discharge, flow duration, and low-flow data for the Mattapoisett River

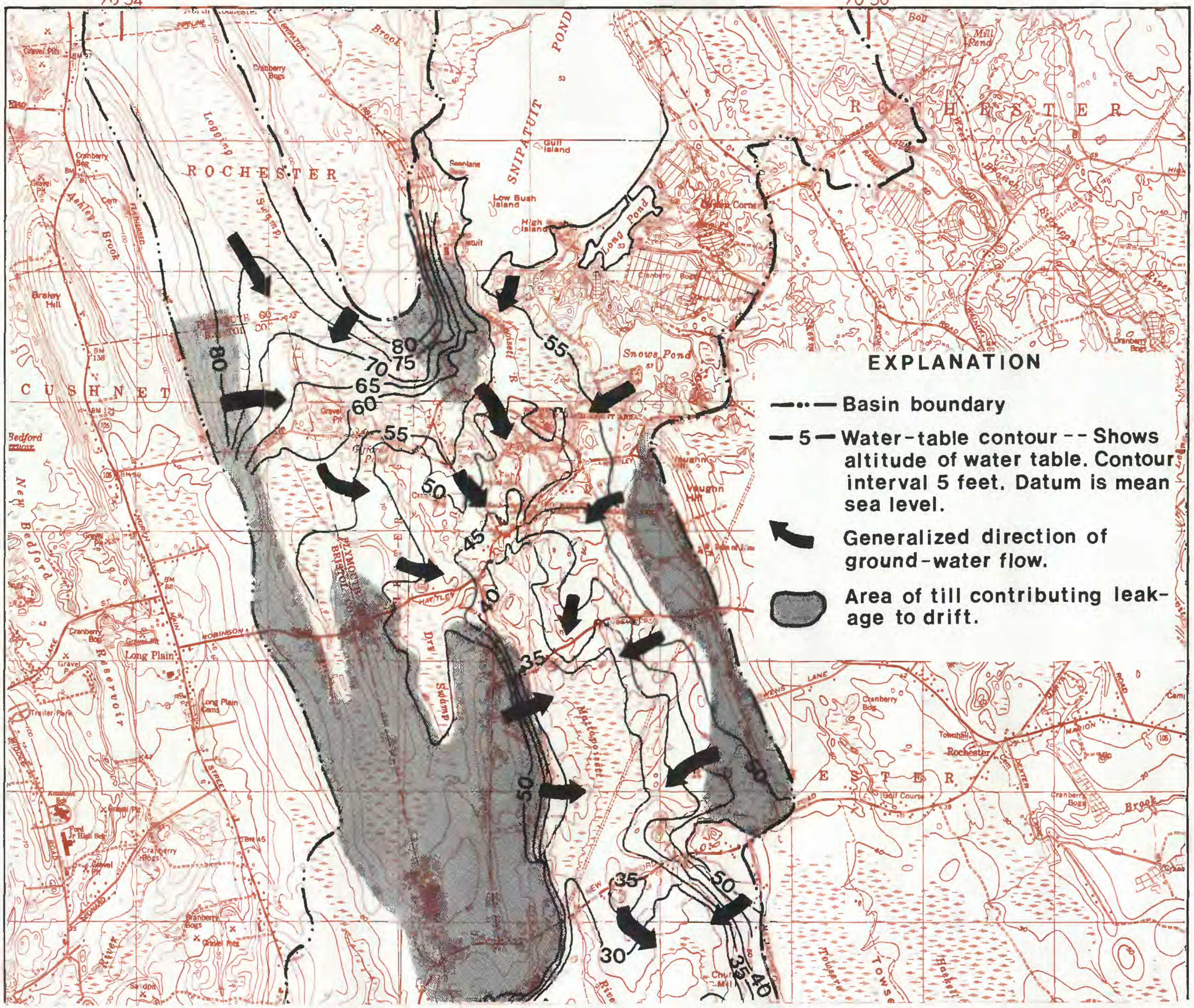
(Discharge and low flow, in cubic feet per second)

Location[1]	Cumulative discharge: Date of seepage run (1982)					Flow duration percent[2]		7-day, 2-year low flow		7-day, 10-year low flow	
	3/22	5/18	7/13	8/30	9/15	90	95	This study	Williams and Tasker[3]	This study	Williams and Tasker[3]
Hartley Road	—	—	0.8	1.0	0.4	—	—	—	—	—	—
Hartley Pond	3.2	1.0	.6	—	—	—	—	—	—	—	—
Cushman Road	1.3	.5	.7	—	—	—	—	—	—	—	—
Rounseville Road	19.3	5.1	2.6	3.0	1.7	2.0	1.5	1.5	1.5	0.8	1.0
New Bedford Road	—	—	4.0	3.9	4.1	—	—	—	—	—	—
Churchs Mill	—	—	3.7	4.7	3.2	—	—	—	—	—	—
Sturtevant Mill	1.0	.2	.1	—	—	—	—	—	—	—	—
Wolf Island Road	—	—	4.8	3.5	4.1	—	—	—	—	—	—
Branch Brook	1.5	.4	—	—	—	—	—	—	—	—	—
Crystal Spring	—	.1	.1	—	—	—	—	—	—	—	—
Tinkham Lane	34.1	9.1	6.3	3.2	4.4	3.9	3.1	3.2	3.5	2.0	2.5
Acushnet Road	—	—	5.8	5.4	6.5	—	—	—	—	—	—
River Road	43.4	11.1	6.8	5.4	7.7	5.0	4.0	4.1	.5	2.5	.2

[1] Point of measurement. Surface-water inflows at two locations have been subtracted from discharge values. Tributaries are indented, and a dash indicates that no data are available. Accuracy of baseflow measurements is ±5 percent.

[2] Flow duration based on discharge measurements from 8/14/72 to 9/15/82 using the flow-duration curve from index station Wading River (01109000) for period of record 1926-81.

[3] Williams and Tasker (1978).


70°54′
70°50′
41°45′
EXPLANATION
Basin boundary
5 Water-table contour -- Shows altitude of water table. Contour interval 5 feet. Datum is mean sea level.
Generalized direction of ground-water flow.
Area of till contributing leakage to drift.
ROCHESTER
CUSHNET
SNIPATUIT POND
Snows Pond
Long Pond
Gull Island
Low Bush Island
High Island
Logging Swamp
Ashley Brook
Braley Hill
New Bedford Reservoir
Long Plain
Dry Swamp
Mattapoisett
Vaughn Hill
PLYMOUTH BRISTOL
80
75
70
65
60
55
50
45
40
35
30

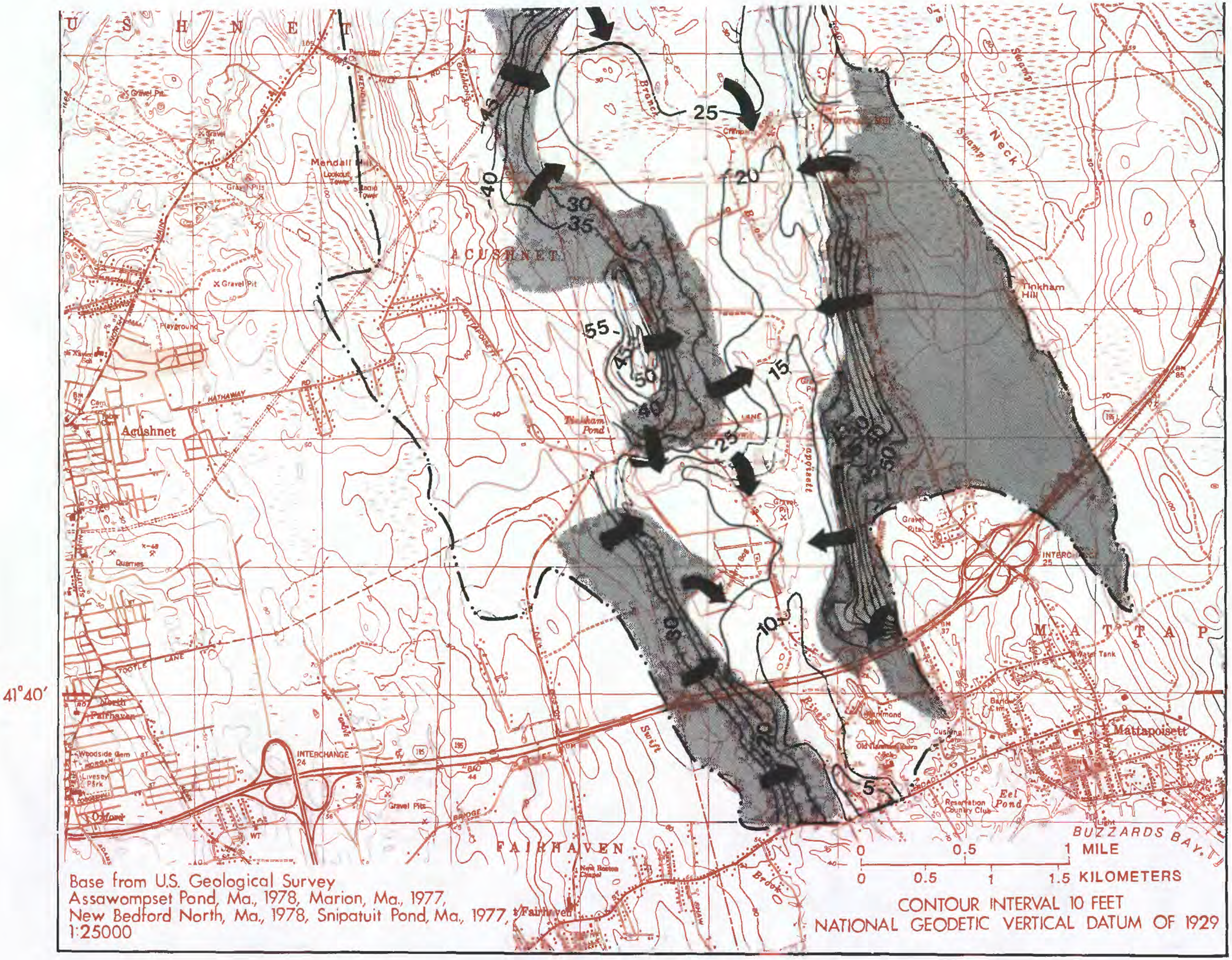


Figure 9.--Direction of ground-water flow in the stratified-drift aquifer.

At this point, it may be helpful to review several important factors which play a role in the collection and interpretation of baseflow data. Baseflow is composed chiefly of ground-water discharge, but may include surface runoff delayed by slow passage through lakes and wetlands. Baseflow is not constant because ground-water discharge and surface-water runoff properties are transient. Ground-water discharge may vary because of time-dependent changes in recharge to the aquifer. Streamflow may vary during baseflow primarily because of time- and weather-dependent changes in the amounts of diversion either into or out of the stream. Finally, the error inherent in the actual measurement of baseflow is ± 5 percent. For the Mattapoisett River, surface inflow takes place at Snipatuit Pond and at Tinkham Pond. Diversion from the river includes seasonal pumpage for agricultural purposes and induced infiltration through the streambed into the aquifer in response to pumpage at nearby municipal wells.

Despite the above limitations in gathering and interpreting baseflow data, the gains and losses determined from the baseflow measurements shown in table 1 seem to be real. On March 22, 1982, and again on May 18, 1982, streamflow steadily gained from Snipatuit Pond to River Road. However, during the summer and fall, measurements indicated both gains and losses in streamflow. On July 13, 1982, small streamflow losses were measured between New Bedford Road and Churches Mill (-0.22 ft^3/s) and between Tinkham Lane and the Acushnet Road bridge (-0.52 ft^3/s). There are no pumping wells along the river upstream of Churches Mill. If measurement errors are assumed to be small, the loss was probably because of evapotranspiration along the swampy stream reach. The relatively larger streamflow loss downstream of Tinkham Lane was because of evapotranspiration and induced infiltration from the normally high July pumpage at wells located nearby.

The average total gain in streamflow for the three summer baseflow measurements ranged from 0.75 to 0.37 $(ft^3/s)/mi^2$ of drainage area. The contributing drainage area, 14.7 mi^2, includes all areas of the drainage basin south of Snipatuit Pond except the drainage area for Tinkham Pond and the small drainage area south of the River Road stream gage. As mentioned above, the data shown in table 1 do not include surface inflows from Snipatuit and Tinkham Ponds. For the 1982 measurements, the total surface inflow ranged from 9.5 ft^3/s (3/22/82) to 1.5 ft^3/s (7/13/82).

For comparison, table 1 also includes estimates of flow duration and low flow at the three gaging stations. The estimates, which are derived from baseflow data gathered during this and previous studies (Sterling, 1960; Williams and Tasker, 1978; unpublished Survey data), include the effects of surface inflows. In addition, estimates of low flow at each gage published by Williams and Tasker (1978) have been refined and updated. The low-flow estimate for the River Road gage has been increased primarily because the estimate by Williams and Tasker (1978) included some data on unusually large streamflow losses between Tinkham Lane and River Road.

Summarizing the streamflow data gathered during this study, the 1982 baseflow measurements, the flow-duration estimates, and the low-flow estimates show that streamflow increases from Snipatuit Pond to River Road.

GROUND-WATER WITHDRAWALS

Ground water from the stratified-drift aquifer is the major source of water supply in the basin. Since the mid-1970's withdrawals from the aquifer have averaged about 1 Mgal/d. In 1981-82, withdrawals increased over 60 percent and a further substantial increase is expected in 1983. According to current plans, withdrawal rates for municipal supplies could more than triple in the next two decades.

Municipal Supplies

The towns of Mattapoisett, Fairhaven, and Marion depend all or in part on ground water from the stratified-drift aquifer. Table 2 lists daily municipal withdrawal rates for the period 1977-1982 which are based on average annual withdrawals, and projected 1990 withdrawal rates. Presently, the town of Rochester neither operates nor is constructing municipal wells in the valley.

Table 2—Ground-water withdrawal rates for municipal supplies, 1977-82, 1990

(Withdrawal rates, in million gallons per day)

Municipal supply[1]	1977	1978	1979	1980	1981	1982[2]	1990[3]
Fairhaven	0.33	0.29	0.29	0.26	0.31	0.37	0.87
Mattapoisett	.43	.44	.41	.47	.42	.43	.62
Marion	.31	.13	.08	.18	.26	.86	1.36
Total	1.07	0.86	0.78	0.91	0.99	1.66	2.85

[1] Data from town records.
[2] Through Oct. 31, 1982.
[3] Estimates from Massachusetts Water Resources Commission (written commun., Jan. 1983).

Major present and proposed ground-water withdrawal sites are shown in figure 10. With the exception of the Marion test-well site off New Bedford Road, all the present and proposed well sites are in the southern half of the river valley. The older sites, the Fairhaven River Road sites, and Mattapoisett 2, are well fields consisting of numerous, 2 1/2-inch-diameter wells. Newer sites are single wells up to 24 inches in diameter. All the withdrawal sites in the valley are located within a few hundred feet of the Mattapoisett River or its tributary, Branch Brook. With the exception of the Fairhaven three-well site on Wolf Island Road, the present and proposed well systems are completed in the stratified-drift aquifer, and are thus hydraulically connected with the Mattapoisett River. The Fairhaven wells on Wolf Island Road are completed in the confined aquifer and do not seem to be in direct hydraulic connection with either Branch Brook or the Mattapoisett River. Although withdrawals from these wells have relatively less impact on streamflow in their immediate vicinity, the total impact on streamflow downstream of the wells remains the same.

Total ground-water-withdrawal rates vary seasonally depending on availability and demand. The average daily withdrawal rate for the summer 1982 was 1.22 Mgal/d and the average rate for the highest month, July, was 1.46 Mgal/d. Pumping capacity of the present wells is about 3.5 Mgal/d and the total pumping capacity of present and proposed wells is estimated to exceed 7.0 Mgal/d. Withdrawal rates in May 1982, that were used in the ground-water model described below, are representative of the present average annual withdrawal rates.

Municipal Well Site Tests Through 1982

In recent years, a great deal of private study and an extensive amount of construction, testing, and production at present and proposed municipal well sites has taken place. Because there is a large amount of available information and because knowledge of these well sites is an important part of this study, a brief review of past stratified-drift aquifer-test results in the study area is presented here. Two conclusions are clear and well documented:

1. Ground-water withdrawals from the wells adjacent to the Mattapoisett River reduce streamflow.
2. High rates of withdrawal from the relatively closely spaced wells in the vicinity of Wolf Island Road cause measurable interference effects throughout that area.

The baseflow data and low-flow estimates described in the previous section indicate the approximate amount of streamflow reduction due to present withdrawals. Further reduction of streamflow due to withdrawals by additional wells should be considered as cumulative; thus, the amount of potential water supply for most of the present and proposed wells depends, in part, on the amount of available streamflow.

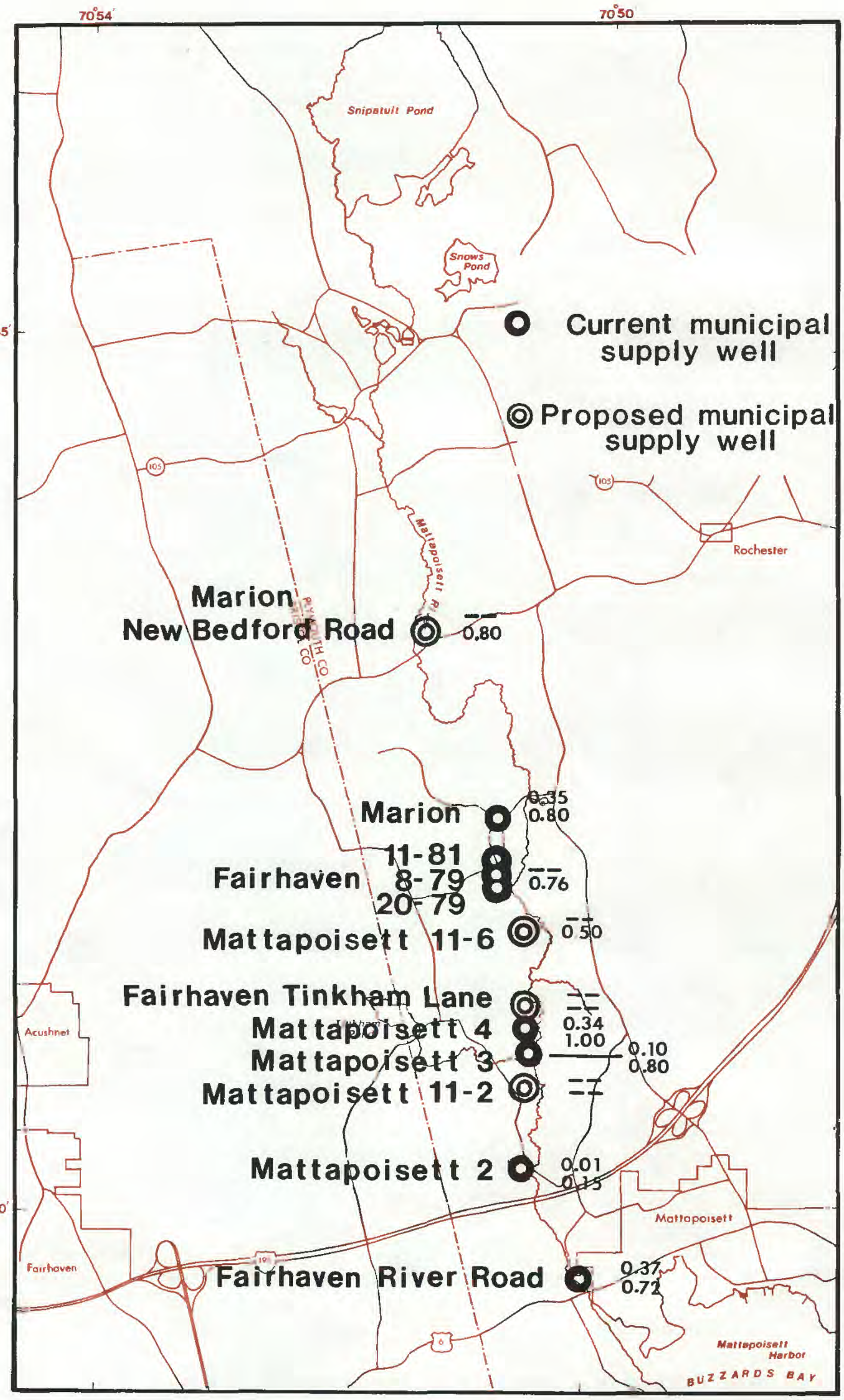


Figure 10.--Current and proposed municipal wells, pumping rates (top number), and pumping capacities (bottom number) in the Mattapoisett River Valley.

According to the results of aquifer tests in the Wolf Island Road area, it seems that well interference also plays an important role affecting the amount of water supply at present and proposed wells. In a 5-day test of the Fairhaven three-well site in November 1982 (fig. 10), the interference effect of each of the three wells ranged from 8.1 to 9.8 feet (Wright-Pierce, Inc., 1982; fig. 11). The interference of the Marion well on the Fairhaven wells ranged from 2.2 to 3.8 feet. Moreover, previous tests of the Mattapoisett test well 11-6 show that withdrawals from this well causes an additional average interference of 2 feet on the Fairhaven wells (Dufresne-Henry, Inc., 1981).

Conversely, withdrawals from the Fairhaven three-well site caused significant interference at both the Marion and Mattapoisett wells. Drawdown at the Marion well exceeded normal levels and did not stabilize completely, and interference at the Mattapoisett site was 3 to 4 feet.

Total well interference at the Fairhaven site ranges from 15 to 25 percent of the saturated thickness of the stratified drift. The range of interference is equal to, and in some places greater than, the thickness of the confined aquifer in which the wells are finished. On the basis of the experience gathered through 1982, it is clear that all wells in the Wolf Island Road area will be affected by the new wells. Well yields cannot be predicted accurately; however, it is certain that the actual yields of wells in the area will depend on the amount of available drawdown, which, in turn will depend in large part on the amount of well interference that is allowed to take place. The amount of well interference will be controlled chiefly by the rate, duration, and timing of water withdrawals from each well.

Agricultural Supplies

Irrigation of cranberry bogs in the basin requires large quantities of water throughout the year. In 1982, ten cranberry bogs consumed approximately 110 Mgal (Richard Thibedeau, Massachusetts Division of Water Resources, written commun., 1983). There are no data on the amount of ground water used by irrigation; however, observations at the cranberry bogs confirm that very little ground water is used. Nearly all the bogs, particularly the large bogs on Acushnet Road and southeast of Snipatuit Pond, either pump water from nearby ponds or from the Mattapoisett River. The effect of cranberry bog operation in the basin is to increase ground-water recharge in the bog areas and locally divert surface water from, and release of water to, the river.

QUALITY OF GROUND WATER

Water-quality samples were collected at 10 sites in the Mattapoisett River basin to evaluate the quality of the ground water and surface water baseflow. The sites include seven wells and three surface-water gaging stations (fig. 2). Samples, collected in August 1981 and July 1982, were analyzed for major constituents, insecticides, pesticides, and volatile organic compounds. Not enough samples were collected to identify temporal or areal chemical trends, and the samples were scanned only for chemical compounds.

Major Constituents

A summary of the major chemical constituents in surface- and ground-water samples collected during this study is listed in table 3. In general, the quality of water of both the river (during baseflow conditions) and the aquifer can be classified as soft, slightly acidic (pH 5.5-6.4), and low in dissolved solids. Dissolved iron and manganese concentration levels are relatively high, which is common in stratified-drift aquifers in the State. The predominant cations are calcium and sodium, and the predominant anion is bicarbonate.

Unusually high specific conductances were measured in wells MJW 204 (1981) and RFW 214 (1982). In MJW 204, the calcium level also was high suggesting that the water contained dissolved particles of the concrete that was used to construct the well. In RFW 214, sodium and chloride levels were high showing that some contamination by salt at the roadside well had occurred. Highest iron-concentration levels were observed in MJW 204, a backfilled area adjacent to a highway; in MJW 120, an unused 24-inch test well; and in RFW 214, a roadside well.

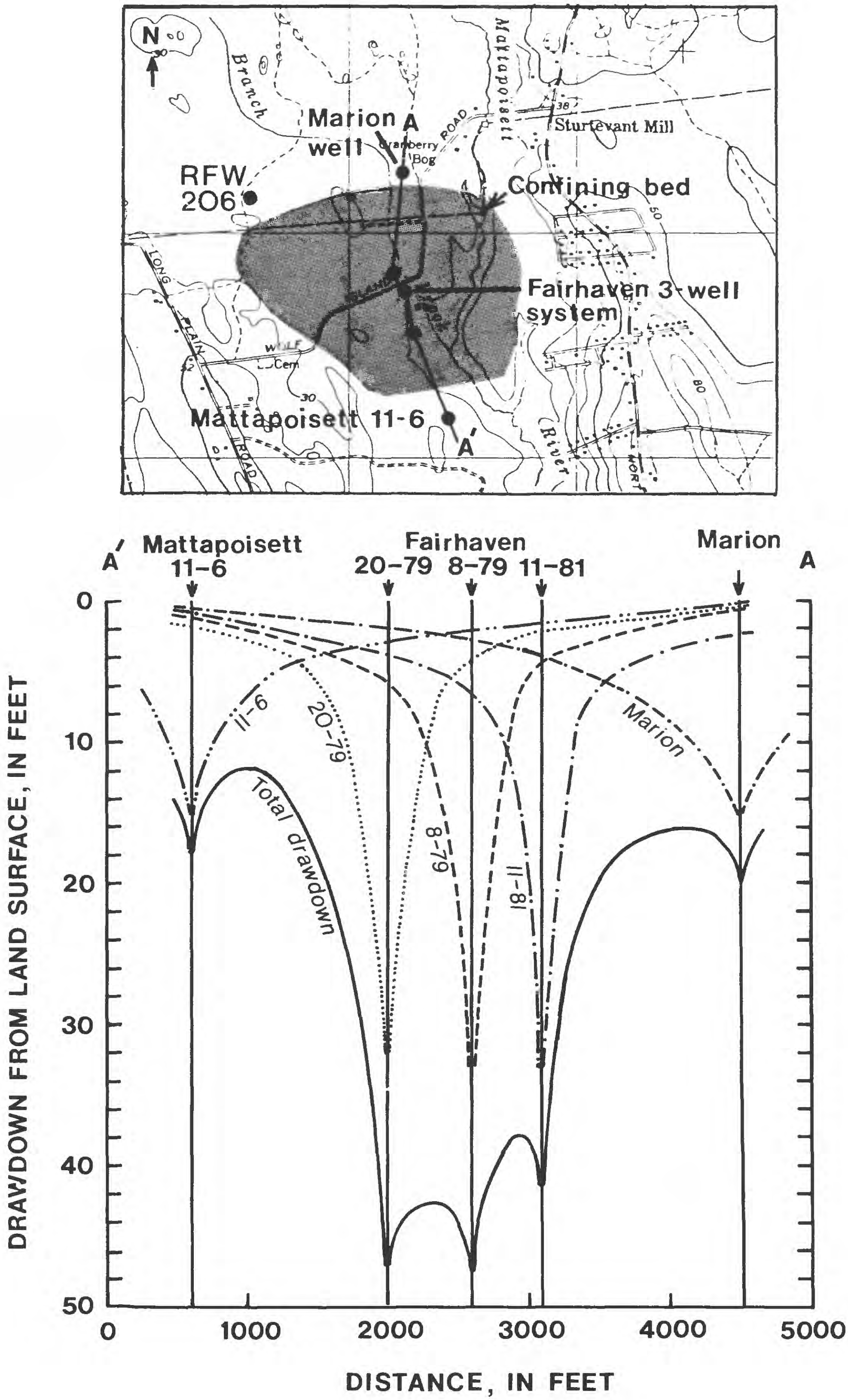


Figure 11.--Well interference in the Wolf Island Road area.

Table 3.--Summary of major constituents in surface-water and ground-water samples

DATE	SPECIFIC CON- DUCTANCE (FIELD) (µS/cm)	SPECIFIC CON- DUCTANCE (LAB) (µS/cm)	PH (FIELD) (UNITS)	PH (LAB) (UNITS)	TEMPER- ATURE (°C)	HARD- NESS (MG/L AS $CACO_3$)	HARDNESS NONCAR- BONATE (MG/L AS $CACO_3$)	ACIDITY (MG/L AS $CACO_3$)
01105911 - MATTAPOISETT RIVER AT ROUNSEVILLE ROAD NEAR ROCHESTER, MASS. (LAT 41 44 10 LONG 070 51 46)								
8-31-81	--	77	--	6.8	--	12	0	10
7-26-82	60	68	6.3	6.4	22.1	14	7	5.0
01105914 - MATTAPOISETT RIVER AT TINKHAM LANE NEAR MATTAPOISETT, MASS. (LAT 41 41 06 LONG 070 50 29)								
8-31-81	--	77	--	7.1	--	15	5	10
7-27-82	118	81	6.4	6.4	22.9	16	8	--
01105917 - MATTAPOISETT RIVER ABOVE RIVER ROAD NEAR MATTAPOISETT, MASS. (LAT 41 39 45 LONG 070 50 20)								
8-31-81	--	83	--	7.1	--	17	7	10
7-27-82	77	79	6.4	6.3	21.9	16	8	5.0
MJW 204 (LAT 41 39 55 LONG 070 50 16)								
8-28-81	--	250	--	8.0	--	92	0	25
7-26-82	46	52	5.8	6.0	15.6	10	4	--
MJW 120 (LAT 41 41 49 LONG 070 50 49)								
8-28-81	--	77	--	8.0	--	13	0	5.0
7-27-82	81	90	6.1	6.4	13.3	23	4	15
RFW 210 (LAT 41 42 52 LONG 070 51 28)								
8-28-81	--	69	--	7.3	--	23	11	5.0
7-26-82	47	53	5.5	6.3	12.1	19	16	--
RFW 213 (LAT 41 43 49 LONG 070 51 13)								
8-28-81	--	39	--	7.1	--	4	0	10
7-26-82	24	33	6.0	6.8	13.6	7	4	--
RFW 214 (LAT 41 44 24 LONG 070 50 58)								
8-28-81	--	104	--	7.1	--	26	0	15
7-26-82	633	674	6.3	6.8	19.8	54	23	--
RFW 223 (LAT 41 45 02 LONG 070 51 32)								
8-28-81	--	--	--	--	--	--	--	--
7-26-82	140	152	6.1	6.9	13.3	51	36	--
RFW 222 (LAT 41 45 42 LONG 070 52 25)								
8-28-81	--	138	--	7.0	--	39	3	10
7-26-82	143	144	5.7	6.3	16.7	29	19	--

Table 3.--Summary of major constituents in surface-water and ground-water samples (Continued)

DATE	CALCIUM DIS-SOLVED (MG/L)	MAGNE-SIUM DIS-SOLVED (MG/L)	SODIUM, DIS-SOLVED (MG/L)	POTAS-SIUM DIS-SOLVED (MG/L)	SUL-FATE DIS-SOLVED (MG/L)	CHLO-RIDE, DIS-SOLVED (MG/L)	FLUO-RIDE, DIS-SOLVED (MG/L)	SILICA, DIS-SOLVED (MG/L)	SOLIDS, RESIDUE AT 180°C DISSOLVED (MG/L)
01105911 - MATTAPOISETT RIVER AT ROUNSEVILLE ROAD NEAR ROCHESTER, MASS. (LAT 41 44 10 LONG 070 51 46)									
8-31-81	2.8	1.2	6.8	1	8.8	10	<0.1	4.6	68
7-26-82	3.4	1.3	6.3	.3	6.0	9.2	<.1	4.6	68
01105914 - MATTAPOISETT RIVER AT TINKHAM LANE NEAR MATTAPOISETT, MASS. (LAT 41 41 06 LONG 070 50 29)									
8-31-81	3.6	1.4	7.3	1	9.1	10	<.1	5.0	59
7-27-82	3.9	1.4	6.8	.7	7.0	11	.1	6.3	74
01105917 - MATTAPOISETT RIVER ABOVE RIVER ROAD NEAR MATTAPOISETT, MASS. (LAT 41 39 45 LONG 070 50 20)									
8-31-81	4.0	1.6	7.7	1	9.2	11	<.1	4.7	62
7-27-82	4.1	1.5	6.9	.7	7.0	11	.1	6.9	76
MJW 204 (LAT 41 39 55 LONG 070 50 16)									
8-28-81	24	7.8	10	6	1.6	8.0	.1	14	164
7-26-82	2.6	.9	3.8	1	4.0	5.0	<.1	7.9	34
MJW 120 (LAT 41 41 49 LONG 070 50 49)									
8-28-81	3.8	.8	8.0	2	.1	9.6	.2	1.9	42
7-27-82	5.7	2.2	6.0	1	9.0	9.0	.1	13	68
RFW 210 (LAT 41 42 52 LONG 070 51 28)									
8-28-81	5.0	2.5	2.0	1	17	2.8	<.1	6.9	50
7-26-82	4.4	1.9	1.6	.6	14	2.6	<.1	5.1	40
RFW 213 (LAT 41 43 49 LONG 070 51 13)									
8-28-81	1.4	.1	2.8	.8	4.7	3.1	<.1	7.8	29
7-26-82	1.7	.6	2.3	.5	4.0	3.0	<.1	6.0	28
RFW 214 (LAT 41 44 24 LONG 070 50 58)									
8-28-81	7.1	2.0	7.4	2	6.7	4.7	<.1	10	51
7-26-82	9.3	7.4	110	5	15	180	.1	33	363
RFW 223 (LAT 41 45 02 LONG 070 51 32)									
8-28-81	--	--	--	--	--	--	--	--	--
7-26-82	14	3.9	5.4	2	6.0	9.6	<.1	4.2	125
RFW 222 (LAT 41 45 42 LONG 070 52 25)									
8-28-81	10	3.3	10	3	11	12	<.1	13	93
7-26-82	7.3	2.5	13	2	9.0	27	<.1	12	113

Table 3.--Summary of major constituents in surface-water and ground-water samples (Continued)

DATE	NITROGEN, NO_2+NO_3, DISSOLVED (MG/L AS N)	PHOS-PHORUS, ORTHO, TOTAL (MG/L AS P)	ALUMINUM, TOTAL RECOV-ERABLE (μG/L)	ALUM-INUM DIS-SOLVED (μG/L)	IRON, TOTAL RECOV-ERABLE (μG/L)	IRON, DIS-SOLVED (μG/L)	MANGANESE, TOTAL RECOV-ERABLE (μG/L)	MAN-GANESE, DIS-SOLVED (μG/L)
01105911 - MATTAPOISETT RIVER AT ROUNSEVILLE ROAD NEAR ROCHESTER, MASS. (LAT 41 44 10 LONG 070 51 46)								
8-31-81	0.22	<0.01	140	70	690	280	600	30
7-26-82	.29	.03	400	100	1,800	320	50	42
01105914 - MATTAPOISETT RIVER AT TINKHAM LANE NEAR MATTAPOISETT, MASS. (LAT 41 41 06 LONG 070 50 29)								
8-31-81	.27	.02	120	50	2,300	190	10	10
7-27-82	.37	.04	400	120	1,500	670	20	20
01105917 - MATTAPOISETT RIVER ABOVE RIVER ROAD NEAR MATTAPOISETT, MASS. (LAT 41 39 45 LONG 070 50 20)								
8-31-81	.22	.02	130	40	410	210	10	10
7-27-82	.35	.04	500	130	1,400	800	20	19
MJW 204 (LAT 41 39 55 LONG 070 50 16)								
8-28-81	.08	.01	1,200	20	92,000	2,900	7,400	4,300
7-26-82	<.10	.18	4,800	10	7,700	14	2,300	110
MJW 120 (LAT 41 41 49 LONG 070 50 49)								
8-28-81	.04	<.01	30	20	50,000	80	270	70
7-27-82	.25	.01	500	70	3,600	1,100	320	59
RFW 210 (LAT 41 42 52 LONG 070 51 28)								
8-28-81	.60	<.01	18,000	350	34,000	1,400	600	190
7-26-82	.55	.07	--	20	5,500	16	900	21
RFW 213 (LAT 41 43 49 LONG 070 51 13)								
8-28-81	.84	<.01	1,100	20	2,400	150	100	80
7-26-82	.67	.02	200	60	3,500	61	90	31
RFW 214 (LAT 41 44 24 LONG 070 50 58)								
8-28-81	.11	.09	--	3,500	180,000	4,100	4,500	1,200
7-26-82	.14	.09	9,600	22,000	2,200	20,000	920	920
RFW 223 (LAT 41 45 02 LONG 070 51 32)								
8-28-81	--	--	--	--	--	--	--	--
7-26-82	8.9	.02	1,800	--	2,200	8	60	9.0
RFW 222 (LAT 41 45 42 LONG 070 52 25)								
8-28-81	.40	<.01	3,400	450	6,400	2,600	240	1,200
7-26-82	1.0	.16	2,400	<10	3,200	260	190	380

Pesticides and Volatile Organic Compounds

Water samples were collected at the 10 sampling sites in 1981-82 for analysis of 55 pesticides and volatile organic compounds (table 4). In both years, all samples had concentration levels of every compound listed that were less than the lower limit of detection for the analytical technique. Therefore, results of the analyses are not listed here. Only the following compounds were identified, and only at concentration levels either at or slightly above the detection level indicated on table 4:

1. Diazinon was detected in both water samples collected from the Mattapoisett River at the River Road stream gage. In 1981 and 1982, the concentration level was 0.05 and 0.12 µg/L, respectively. The detection of this insecticide was not unexpected in view of the agricultural activity in the valley.
2. 2,4-D (0.03 µg/L), a broadleaf herbicide, was detected in 1981 in MJW 204.
3. The solvents benzene (10 µg/L), 1,2-dichloroethane (7 µg/L), and methylene chloride (30 µg/L) were detected in 1981 in RFW 213.
4. Benzene (6 µg/L) and the insecticides DDT (0.32 µg/L) and dieldrin (0.12 µg/L) were detected in 1981 in roadside well RFW 214.
5. Benzene (4 µg/L), and 2,4-D (0.04 µg/L) were detected in 1981 in RFW 222.

Concentrations of the solvents benzene and methylene chloride, and of the herbicide 2,4-D, were below the maximum contaminant levels established by the U.S. Environmental Protection Agency (1977). Maximum levels have not been set for the solvent 1,2-dichloroethane or the insecticides DDT, diazinon, and dieldrin.

Table 4.--Pesticides and volatile organic compounds analyzed in water samples (Detection levels are given in micrograms per liter.)

Name	Detection level	Name	Detection level
Aldrin, dissolved	0.01	Ethion, dissolved	0.01
Benzene, total	3.0	Ethylbenzene, total	3.0
Bromoform, total	3.0	Heptachlor, dissolved	.01
Carbon tetrachloride, total	3.0	Lindane, dissolved	.01
Chlordane, dissolved	.10	Malathion, dissolved	3.0
Chlorobenzene, total	3.0	Methylbromide, total	3.0
Chlorodibromomethane, total	3.0	Methylene chloride,total	3.0
Chloroethane, total	3.0	Methoxychlor, dissolved	.01
2-Chloroethyl vinyl ether, total	3.0	Methyl parathion, dissolved	.01
Chloroethylene, total	3.0	Methyl trithion, dissolved	.01
Chloroform, total	3.0	Mirex, dissolved	.01
2,4-D, total (comb)	.01	Nitrogen, dissolved	
DDD, dissolved	.01	NO_2+NO_3 as N	1.00
DDE, dissolved	.01	Parathion, dissolved	.01
DDT, dissolved	.01	PCB, dissolved	.10
Diazinon, dissolved	.01	PCN, dissolved	.10
Dichlorobromomethane, total	3.0	Perthane, dissolved	.01
Dichlorodifluoromethane, total	3.0	2,4,5-T, total (comb)	.01
1,1-Dichloroethane, total	3.0	1,1,2,2-Tetrachloroethane, total	3.0
1,2-Dichloroethane, total	3.0	Tetrachloroethylene, total	3.0
1,1-Dichloroethylene, total	3.0	Toluene, total	3.0
1,2-trans-Dichloroethylene, total	3.0	Toxaphene, dissolved	3.0
2,4-Dichlorophenol, total (comb)	.01	Trichloroethylene, total	3.0
1,2-Dichloropropane, total	3.0	1,1,1-Trichloroethane, total	3.0
1,3-Dichloropropene, total	3.0	1,1,2-Trichloroethane, total	3.0
Dieldrin, dissolved	.01	Trichlorofluoromethane,total	3.0
Endosulfan, dissolved	.01	Trithion, dissolved	.01
Endrin, dissolved	.01	Vinyl chloride, total	3.0

SIMULATION OF AQUIFER RESPONSE TO GROUND-WATER WITHDRAWALS BY MUNICIPAL WELLS

Description of Digital Model and Conceptual Model

A digital model was used to simulate the response of hydraulic head to ground-water discharge from the stratified-drift aquifer under natural and manmade stress conditions. The model uses a two-dimensional, finite-difference method in which differential equations that describe ground-water flow are solved numerically. The equations require definition of the geologic and hydrologic properties of the area modeled, the boundaries, and the stresses. The model can simulate an aquifer that may be confined or unconfined, or a combination of both. Also, the aquifer may be heterogeneous and anisotropic and may have irregularly shaped boundaries. The model, which permits constant recharge, well discharge, and leakage from a confining bed, was described by Trescott and others (1976).

A rectangular finite-difference grid was superimposed on a map of the valley (fig. 12). Rectangle sizes in the grid range from 208 X 208 feet to 832 X 832 feet. A variable grid was designed to provide greater detail in areas of special interest, particularly along the river and near the pumping wells, and to provide greater accuracy in areas of large hydraulic-head gradient. The center of each rectangle is referred to herein as a node, and the position of each node is designated by row and column numbers; for example, the node at row 15, column 12 is expressed as 15,12. Each node represents a block of the stratified-drift aquifer.

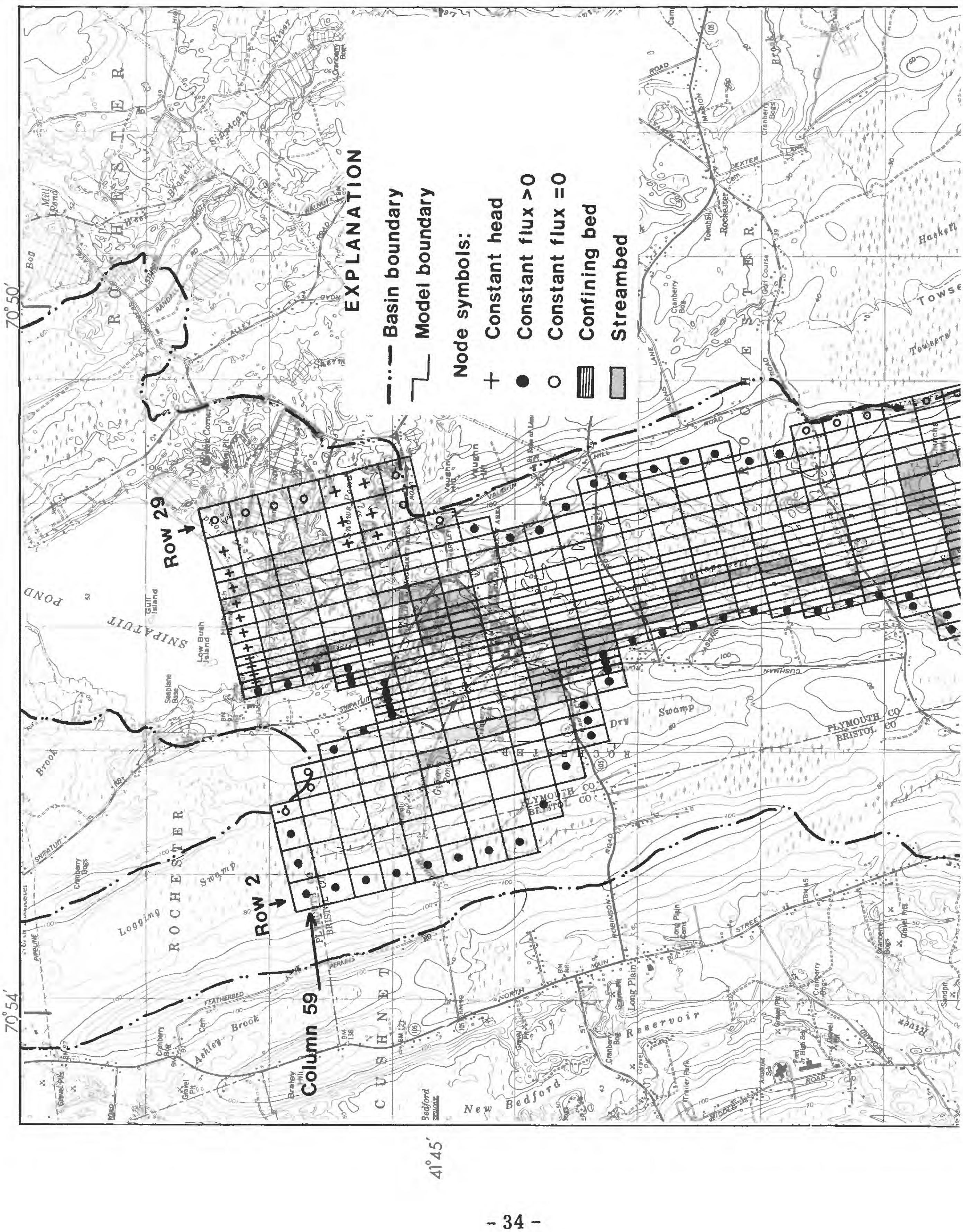
Within the grid, model boundaries were selected that delineate the active model area, or the area of the grid over which ground-water flow is simulated. On the east and west sides of the valley, the boundaries coincide as closely as possible with the till/stratified-drift boundary. The northern model boundary is an east-west line parallel to the southern shoreline of Snipatuit Pond. The southern boundary corresponds to the surface drainage divide that follows Fairhaven Road (U.S. Route 6). The active model area covers 8.0 mi^2 and consists of 1068 nodes.

To develop the digital model of the stratified-drift aquifer, a conceptual model was used to describe, but simplify, the complexity of the real system. The conceptual model consists of a set of general ideas of how the ground-water system works and a set of assumptions which are useful and necessary for model analysis. Principal assumptions of the conceptual model follow:

1. Ground-water flow in the aquifer is horizontal. In the confining bed in the Wolf Island Road area, flow (leakage) is vertical. There is no ground-water flow either to or from the underlying bedrock. Ground-water flow in the stratified drift is not strictly horizontal; however, the assumption applies reasonably well to most of the model area. The importance of this assumption is that simulated hydraulic heads are not accurate in areas with significant vertical water flow, such as in the recharge areas in the basin. There are no bedrock leakage data to verify the assumption of an underlying impermeable boundary, but comparisons of recharge estimates and runoff measurements for the basin suggest that there is little flow either to or from the bedrock.

2. Recharge to the aquifer is distributed uniformly over the active model area. Effects of evapotranspiration are included in estimates of recharge. As a result of this assumption, simulated hydraulic heads will tend to be lower than real heads in upland areas where there is more recharge and less evapotranspiration, and will tend to be higher than real heads in lowlands where there is less recharge and more evapotranspiration.

3. The elevation of surface water in ponds and streams remains constant with time. Pumpage near streams does not significantly lower stream stage. Stream stages and pond levels usually reflect the altitude of the water table nearby. The result of this assumption is that simulated leakage through streambeds and pond bottoms will be slightly higher than real leakage during falling water-table conditions, and will be slightly lower than real leakage during rising water-table conditions.


EXPLANATION
Basin boundary
Model boundary
Node symbols:
Constant head
Constant flux >0
Constant flux =0
Confining bed
Streambed
Row 29
Row 2
Column 59
70°50′
70°54′
41°45′

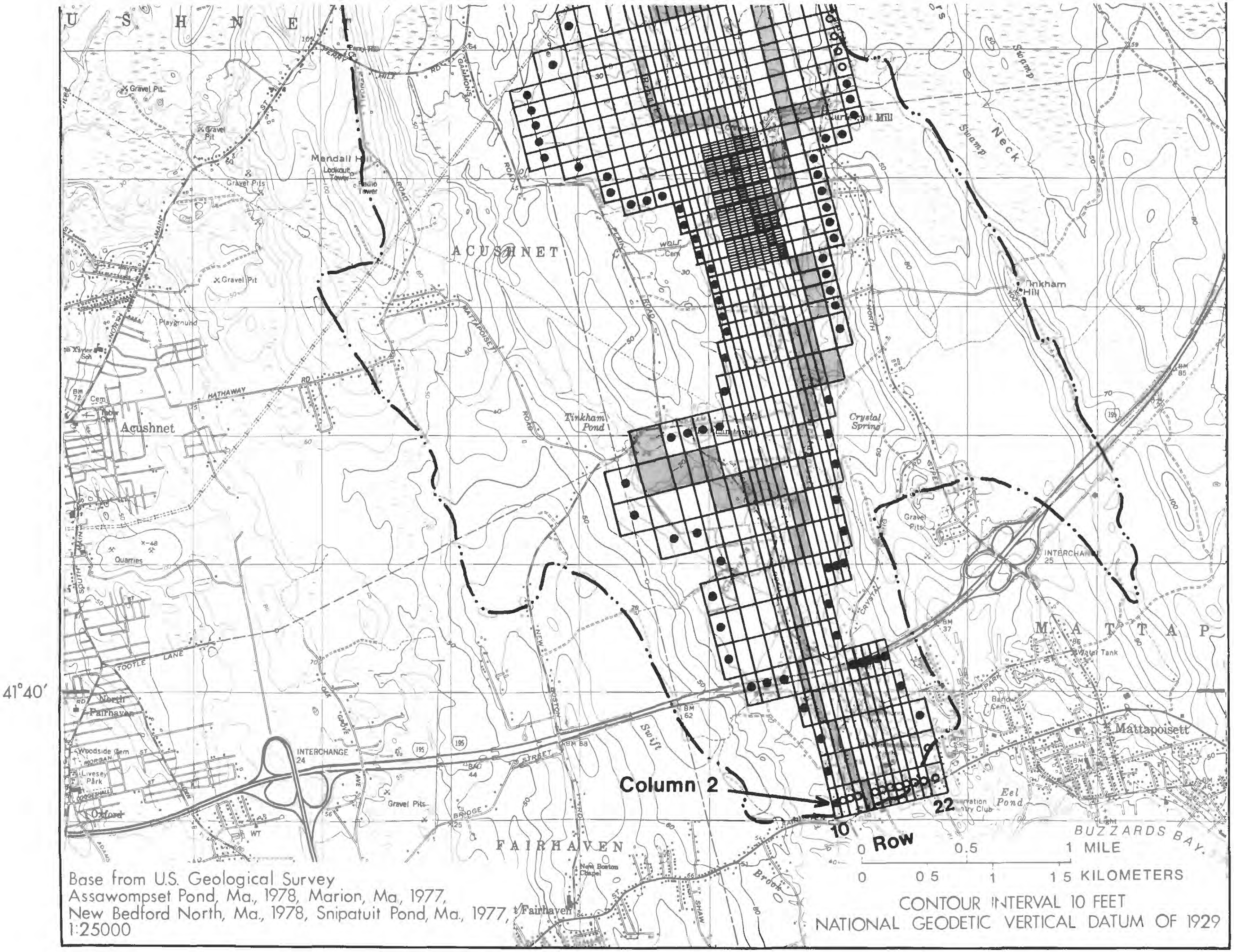


Figure 12.--Finite-difference grid for the stratified-drift aquifer.

4. Ground-water discharge from the aquifer to the Mattapoisett River and to its tributaries is through a leaky streambed. Streambeds have constant thickness, vary in width, and are composed of sediments that have lower vertical hydraulic conductivity than that of the aquifer. Small-scale variations in the physical properties of the streambed are not simulated. The stage of the river is set to a constant value in each river node, and this may lead to inaccurate simulation of hydraulic head in the aquifer in cases where computed ground-water discharge is insufficient to maintain positive streamflow. The river, in effect, goes dry.

5. There is no ground-water discharge either to the till or to the ocean. As described above, there may be a small amount of real discharge to the ocean, but current evidence indicates that there is little, if any, ground-water flow to the ocean through the thin, narrow southern end of the aquifer.

6. The leakage rate from the till to the stratified drift is constant in each border node. In actuality, the leakage rate may vary spatially and temporally, especially in view of the seasonal fluctuations in hydraulic head in the till (Frimpter, 1981).

The conceptual model and corresponding digital model of the ground-water-flow system are shown schematically in figure 13. The digital model simulates many elements of the real system including the unconfined and confined parts of the aquifer, till boundaries, stream boundaries, recharge, and wells. The confining bed and the confined part of the aquifer in the Wolf Island Road area is represented by 56 nodes which permit both upward and downward vertical leakage. Numerical values for geologic and hydrologic properties of the aquifer are assigned to each node in the grid. Physical properties representing each node are assumed to be constant over the area of the node and represent an average value throughout the vertical thickness of the block. In the horizontal flow model, the initial and computed hydraulic heads in each node are assumed to equal the altitude of the water table.

Careful interpretation of model results is important because the digital model is an approximation of a complex, real system. Inaccuracy must be considered in view of the coarseness of the approximation, the assumptions of the conceptual model, and the inadequacy of the model to simulate several natural processes. Considering these limitations, the conceptual model is appropriate and accurate, in view of the objective of the study, the size and nature of the problem, and the current knowledge of the physical system.

Selection of Initial Conditions and Input Parameters

The first step in the modeling procedure was to select a time period during this study when recharge to the aquifer, water-table altitude, and ground-water discharge represented long-term, average conditions. Precipitation, water-level, and ground-water discharge data gathered in 1981-82 indicated that long-term average conditions were most closely reached in May 1982. As a result, initial conditions of the model were set to observed May 1982 water-table and streamflow conditions. The long-term average conditions were assumed to represent steady-state aquifer conditions.

Input parameters to the steady-state model included measured and estimated values of water-table altitude, aquifer hydraulic conductivity, bedrock altitude, confining bed thickness, and municipal withdrawals. The model grid was overlain on maps of each parameter, such as water-table altitude (fig. 4) and bedrock altitude (fig. 2), and average numerical values were selected for each node in the active model area.

Aquifer hydraulic conductivity used in the model is shown in figure 14. The initial model input values of hydraulic conductivity derived from aquifer tests and grain-size analyses were modified during the modeling process. Summarizing the modifications, the most notable difference between the original data and the estimated values are in areas along the east and west sides of the model area. In these areas, original values ranging from 5 to 50 ft/d were modified to range from 25 to 100 ft/d. Figure 14 is a final product of the modeling process.

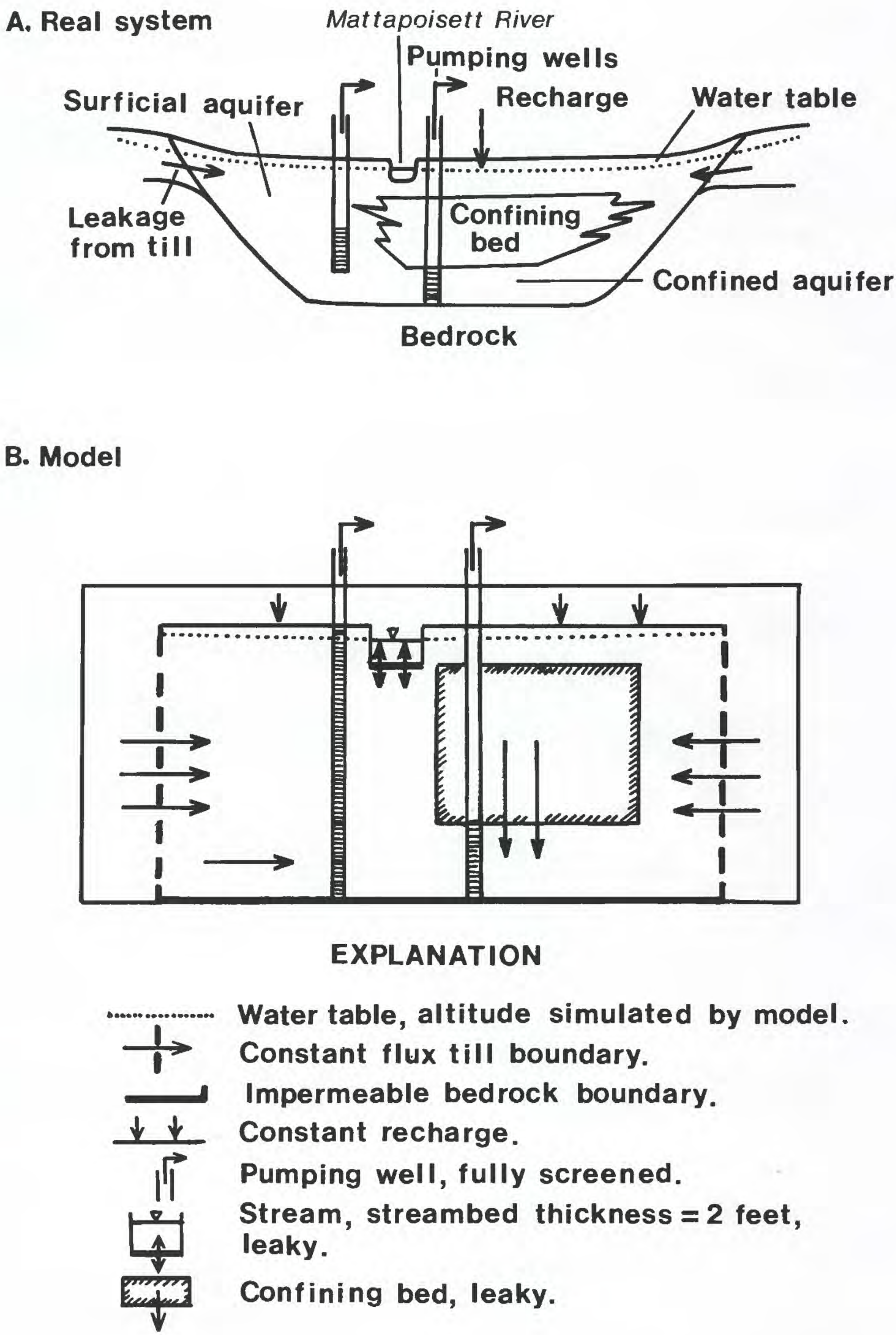
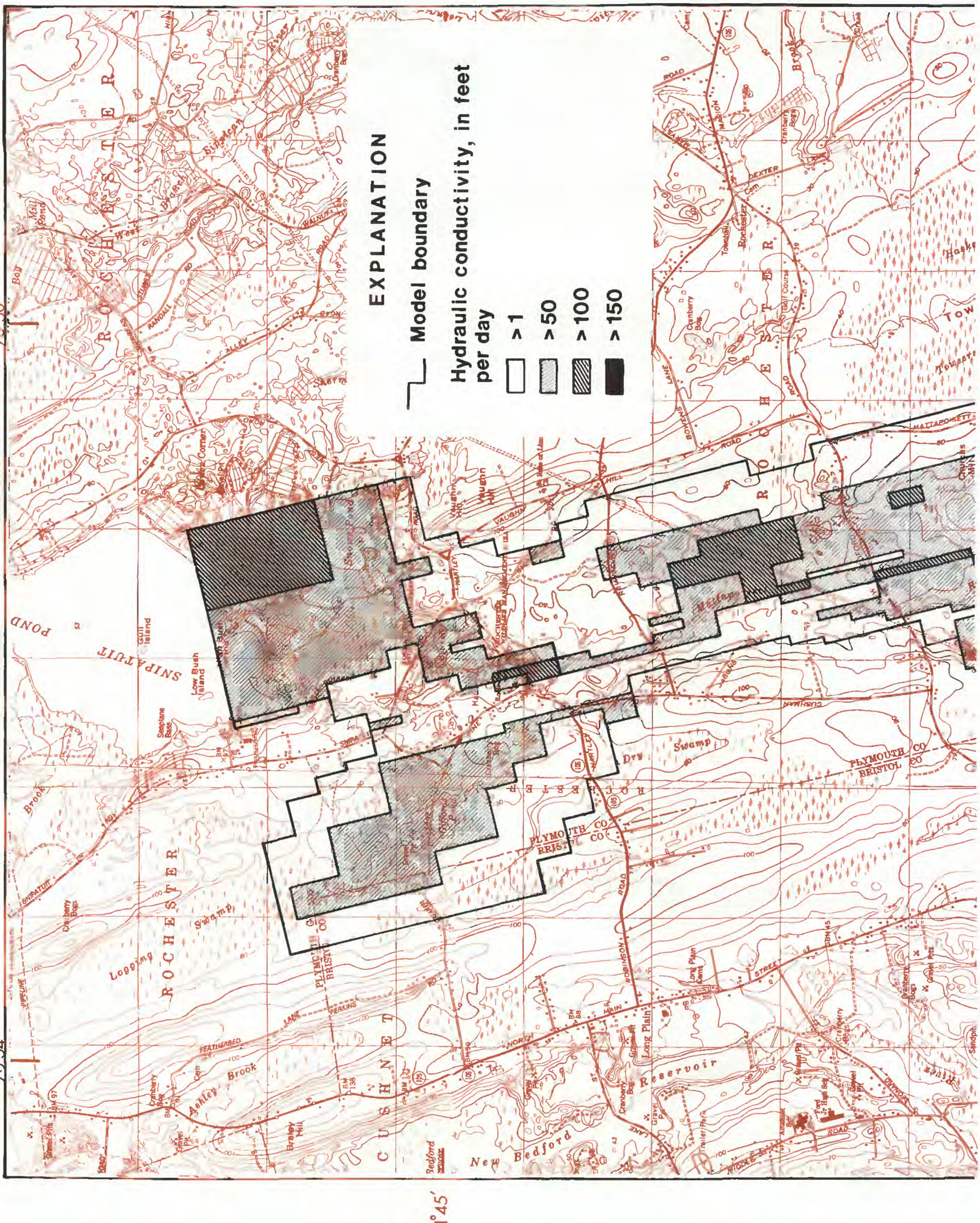


Figure 13. --Idealized conceptual model of steady-state ground-water flow.


EXPLANATION
Model boundary
Hydraulic conductivity, in feet per day
>1
>50
>100
>150
70°50'
70°54'
41°45'
ROCHESTER
CUSHNET
Snipatuit Pond
Dry Swamp
Long Plain
New Bedford Reservoir
PLYMOUTH CO
BRISTOL CO

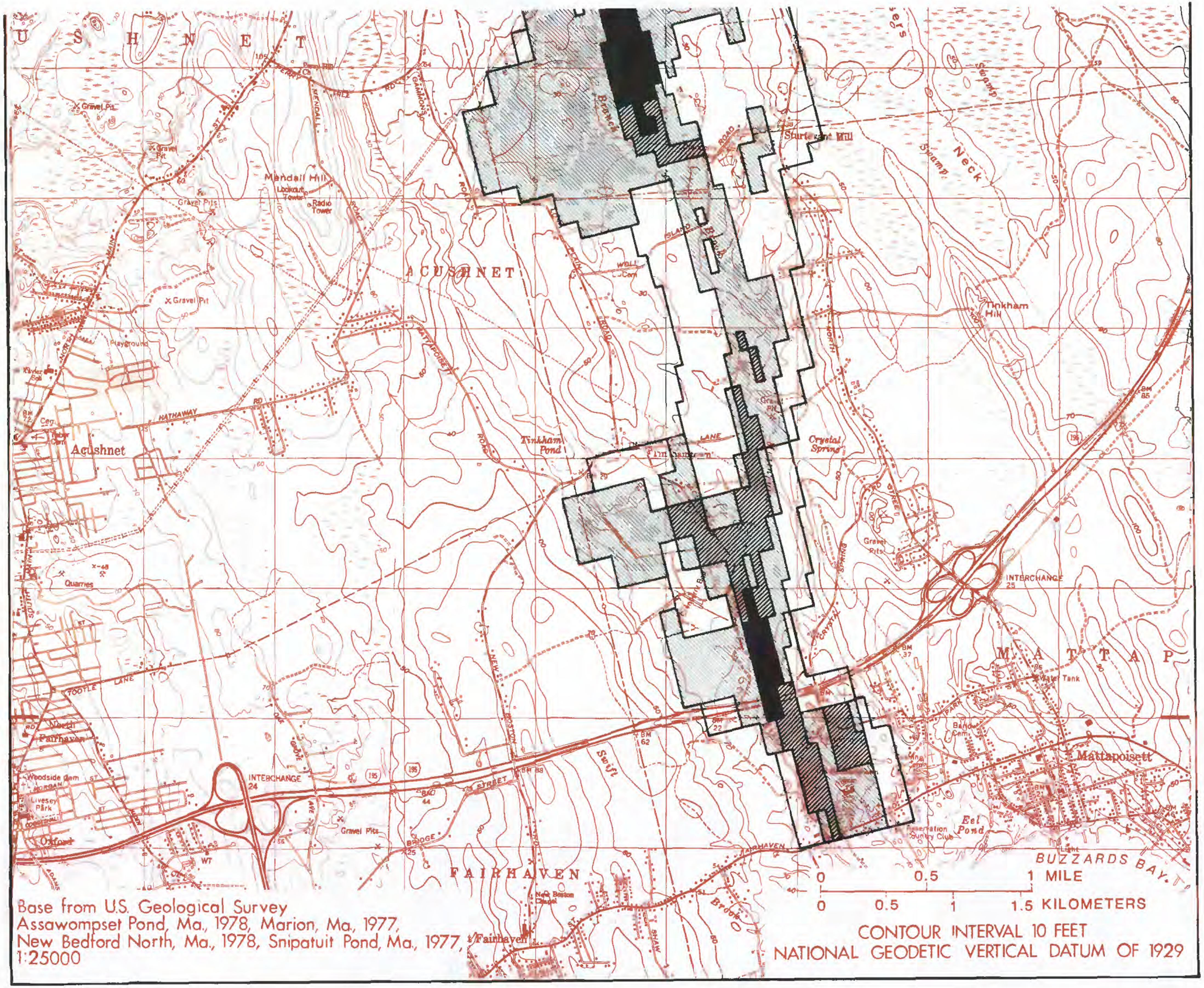


Figure 14.--Hydraulic conductivity of the stratified-drift aquifer as used in the digital model.

The recharge rate to the aquifer was set at 15.9 in/yr and was applied uniformly over the active model area. The May 1982 water-table map was used to estimate hydraulic head at each node. Where there were no observation-well data, the water table was assumed to be a few feet or less below land surface in flat swampy areas and 5 to 10 feet below land surface in hilly areas. Surface-water elevations of ponds and streams were estimated from Survey topographic quadrangle maps. The thickness and lateral extent of the confining bed was defined in the digital model based on lithologic data from test wells. The vertical hydraulic conductivity of the confining bed was set at 3 ft/d.

Ground-water withdrawals by the municipal wells were set at May 1982 pumping rates. The withdrawals, which total 1.16 Mgal/d (1.80 ft^3/s), include those by the Fairhaven, Mattapoisett, and Marion wells. Withdrawal of ground water by domestic wells and by irrigation wells is not significant in the study area and, therefore, was not included in the simulation.

Boundary Conditions

A constant-flux boundary was used to simulate leakage from till to the active model area. The method used to estimate the amount of leakage from till at each node consisted of two steps. First, the area of till between the model boundary and the surface-drainage divide, that is not drained by a major stream, was determined. Then, the recharge rate to each node was calculated using the till area and previously described leakage rate of 0.5 $(ft^3/s)/mi^2$. Thus, the flux rate varies along the model boundary depending on the size of the contributing area of till.

No-flow boundaries (constant flux = 0) were placed in the northeast part of the model area between Snipatuit Pond and Snows Pond by drawing the boundary approximately parallel to the known water-table gradient. On hilltops, it was assumed that surface-water and ground-water divides coincide. Therefore, no-flow boundaries also were located along surface-drainage divides north of Vaughn Hill, along Mattapoisett Road north of Sturtevant Mill, and along Fairhaven Road at the southern end of the basin. The Mattapoisett Road boundary is unusual because it is located in till east of the till/stratified-drift boundary. The active model area between Churchs Mill and Sturtevant Mill includes a small portion of till because of the location of the river.

Constant-head nodes were placed where the boundary coincides with Snipatuit Pond and Snows Pond. Heads in each pond were set at 53 and 55 feet, respectively. Nodes in the model that correspond to the location of Mattapoisett River and its major tributaries were assigned parameters describing the hydrologic properties of the streambed and the river. Stream node properties simulated in the model were hydraulic conductivity, thickness, width, and river-stage elevation. Streambed thickness was estimated at 2 feet along the entire river and hydraulic conductivity of the mostly muddy streambed was estimated to be 1.5 ft/d. In all cases, the area of the real streambed was less than that of the corresponding model node; therefore, the ratio of hydraulic conductivity to streambed thickness was reduced proportionally to simulate leakage more accurately. The head in the stream nodes was set equal to the measured and estimated elevation of the river surface on May 18, 1982.

Parameter Estimation Procedure

Parameter estimation refers to the process of adjusting input parameters of the model until differences between the model simulations and field observations are within acceptable limits. Such a procedure was followed to calibrate the model. Acceptability was determined by comparing simulated and observed heads and ground-water discharge, and the procedure was used for both steady-state and transient conditions.

The procedure was a repetitive process of adjustment and re-adjustment. Changes were made only to those parameters where little field data were available; principally, aquifer hydraulic conductivity and confining-bed hydraulic conductivity. Changes were made on an areal rather than node-by-node basis and within a reasonable range of parameter values.

The match between the computed and observed values of hydraulic head and ground-water discharge was improved and the conceptual model was modified during the estimation process. Although final, accepted, computed heads and discharges did not match observed data precisely, the differences can generally be accounted for by the range of the input model parameters. An acceptable difference between simulated and observed values depended on several factors, including the amount and range of each data value, the reasonableness of the computed result based on knowledge of the real system, and the potential for further improvement considering the assumptions and limitations of the model.

The parameter estimation procedure was followed during transient-flow runs as a further check on the accuracy and reasonableness of selected input parameters. In the transient model, computed hydraulic head depended on starting conditions and length of the simulation time. Therefore, storage properties of the aquifers and the confining bed were included in the model. A long-term, area-wide simulation of the active model area was not conducted because of lack of data. The transient-flow parameter estimation procedure consisted of six short-term, site-specific simulations of the aquifer tests conducted at the major municipal water-supply sites.

Following a trial and error procedure similar to that used for the steady-state model, the computed heads and well drawdowns at each site were compared with the aquifer-test data. Adjustments of the transient model were made only to values of aquifer hydraulic conductivity at nodes in the vicinity of each well.

The entire parameter estimation procedure to calibrate the model required approximately 60 simulations. The digital model was completed and accepted for use when the computed results of both the steady-state and transient models were within acceptable limits.

Steady-State Conditions

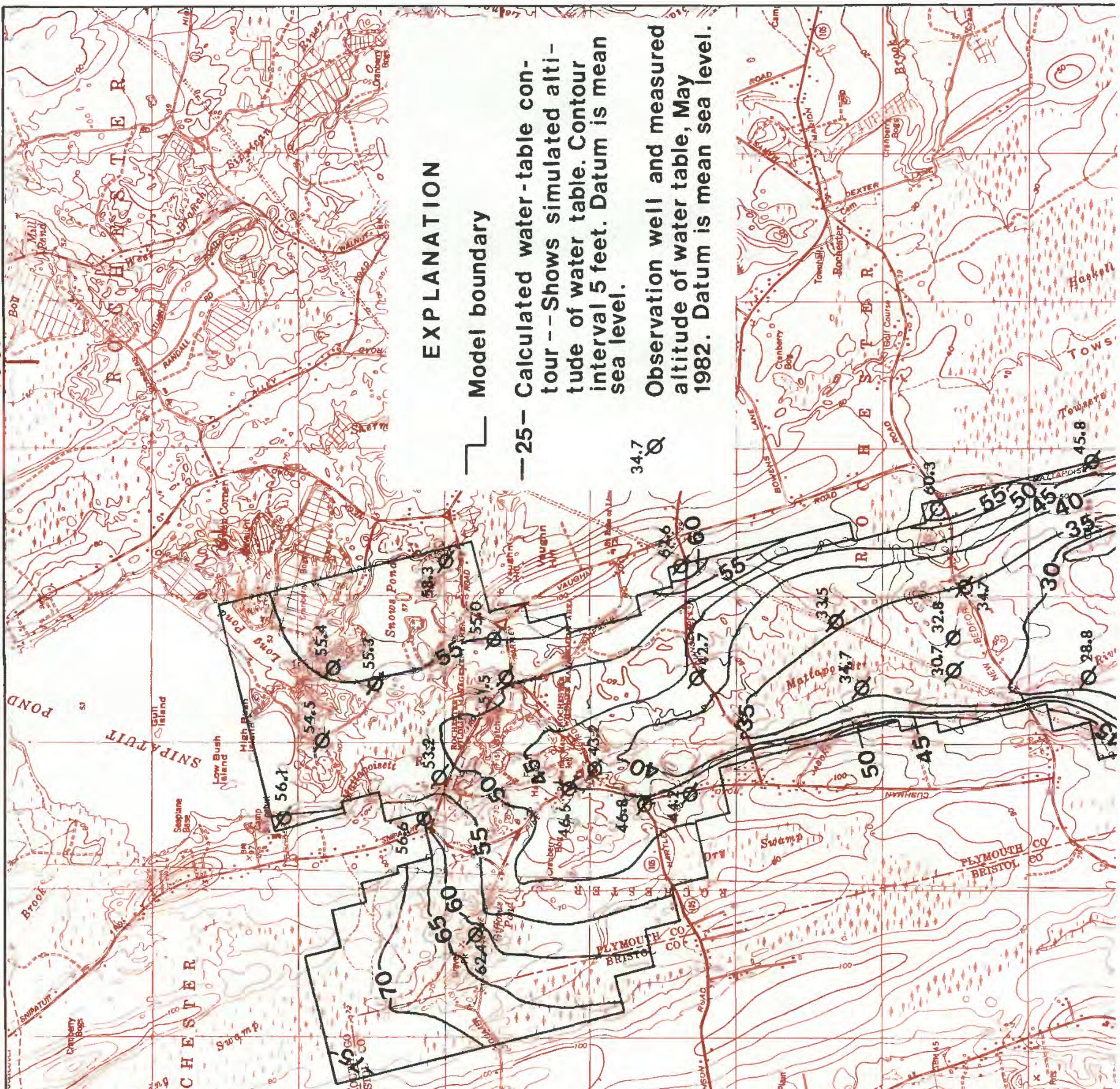
Simulation of May 1982 conditions

The computed steady-state water table of the stratified-drift aquifer for May 1982 is shown in figure 15. Also illustrated are the Survey observation wells and water-level altitudes measured on May 26, 1982. The map of the computed water table may be compared with the map based on observed data shown in figure 9. Differences between computed and observed heads at 41 observation-well nodes ranged from -3.5 to +5.2 feet. The difference was less than 1 foot at 80 percent of the observation-well nodes and less than 5 feet at all but one well node. For all active model nodes, the absolute difference between computed and observed heads ranged from 0.0 to 10.5 feet. The differences were less than 4 feet in 90 percent of the model area, and this was considered to be an acceptable model simulation.

Most differences greater than 4 feet were on the east and west sides of the valley, particularly in the vicinity of Tinkham Pond, upper Branch Brook, Giffords Pond, and Sturtevant Mill. The differences occurred in nodes where the head gradient was relatively steep, in areas adjacent to boundaries, and in corners of the active model. In all these areas, a poorer fit of up to 10 feet was accepted because of the sparsity of water-level data and the limitations of the horizontal flow model.

An equally important check during the parameter estimation procedure was the comparision of computed ground-water discharge to streams with the observed baseflow measured on May 18, 1982. Comparisions were made along seven stream reaches, as shown in table 5. The computed discharge values compared favorably with the observed data and fell well within the range of observed 1982 baseflow values (table 1). Model values were expected to differ slightly from observed values because of the range of error in the baseflow data, the relatively coarse finite-difference approximation in the northern part of the model area, and the conceptual model limitation that restricted simulation of distributed evapotranspiration effects.

The computed steady-state ground-water budget of the stratified-drift aquifer for May 1982 is given in table 6. The total ground-water inflow rate for the active model area was 13.4 ft^3/s. The outflow was distributed between the Mattapoisett River and its tributaries (11.2 ft^3/s), nonreturned pumpage (1.8 ft^3/s), and leakage from Snows Pond to the eastern edge of Snipatuit Pond (0.3 ft^3/s). Outflow rate matched inflow rate to within 0.02 percent indicating an acceptable computed balance of water inflows and outflows.


EXPLANATION
Model boundary
—25— Calculated water-table contour--Shows simulated altitude of water table. Contour interval 5 feet. Datum is mean sea level.
34.7 Ø Observation well and measured altitude of water table, May 1982. Datum is mean sea level.
70°50'
70°54'
41°45'
ROCHESTER
SNIPATUIT POND
Long Pond
Snows Pond
Mattapoisett River
PLYMOUTH CO
BRISTOL CO
Swamp
New Bedford Reservoir

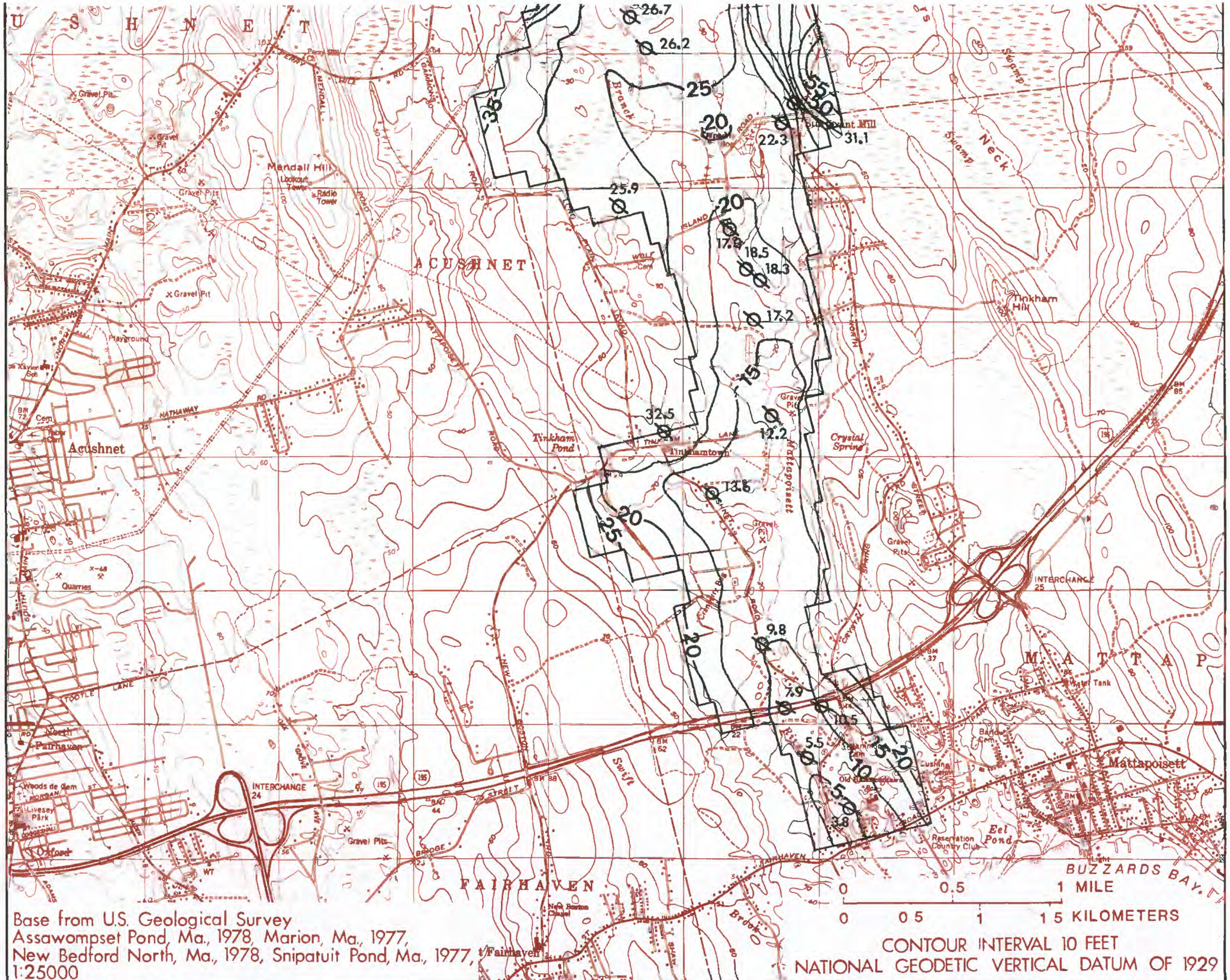


Figure 15.--Simulated steady-state water table of the stratified-drift aquifer, May 1982.

Table 5.—Observed and computed ground-water discharge from the stratified-drift aquifer
(Discharge in cubic feet per second)

Reach	Observed discharge May 18, 1982	Computed discharge
Snipatuit Pond-Rounseville Road	3.60	2.45
Tributary: Hartley Pond and Cushman Road	1.53	1.88
Rounseville Road-Tinkham Lane	3.24	4.28
Tributary: Branch Brook	.41	.68
Tributary: Sturtevant Mill	.22	.06
Tributary: Crystal Spring	.13	.07
Tinkham Lane-River Road (including Tinkham Brook)	1.92	1.72
Total ground-water discharge at River Road	11.05	11.14

Table 6.—Steady-state ground-water budget of the stratified-drift aquifer, May 18, 1982
(Rates, in cubic feet per second)

Inflow rate		Outflow rate	
Recharge from precipitation	9.7	Ground-water discharge to streams	11.2
Leakage from till	3.2	Pumpage	1.8
Leakage from ponds	.5	Leakage to pond	.3
Total inflow	13.4	Total outflow	13.3

Sensitivity Analysis

A sensitivity analysis of the steady-state model was conducted to assess the the limitations of the conceptual model and the uncertainty in the selection of input data values and boundary conditions. The analysis was made to determine if the differences between computed and observed data values could be accounted for by the likely range of each value. The analysis provided a measure of the sensitivity of the model results to changes in the values of key parameters, and thus a check on the reasonableness of the steady-state model.

Throughout the model area, the principal input parameters, aquifer, and streambed hydraulic conductivity were each independently increased and decreased by a constant factor, while other parameters were left unchanged. Differences between computed and observed values of head and ground-water discharge were used to evaluate model sensitivity. The amount of adjustment of each parameter differed according to the likely range of each parameter. Other parameters, such as bedrock altitude and withdrawal rates, were not adjusted because their ranges of values are relatively small.

The results of the analysis of each change in parameter value, described in detail below, are shown along three profiles in figure 16. The profiles, taken north-south along row 15 and east-west along columns 31 and 53, show the differences between observed May 1982 heads and heads that were computed using increased and decreased values of input parameters. The computed results of the steady-state model are included for comparison.

1. Aquifer hydraulic conductivity times 2.0 (2xKa): Doubling the hydraulic conductivity of the aquifer results in a 3- to 5-foot decrease in head on the east and west sides of the model area and up to a 10 foot decrease in head in the northwest part of the model area. The effect was equivalent to partial desaturation of the aquifer. A small increase (3 percent) in the total amount of ground-water discharge occurs due to additional contribution of water from the constant-head boundaries in the northeast part of the model area. Raising the hydraulic conductivity by a factor of 2 increased model errors significantly, causing nodes to go dry and numerical solution problems to occur.
2. Aquifer hydraulic conductivity times 0.5 (0.5xKa): Halving the hydraulic conductivity of the aquifer had an effect on the model opposite to that of doubling the value. The aquifer was made relatively impermeable, and head increases up to 20 feet occurred; particularly in the most sensitive areas of the model where steady-state values of hydraulic conductivity were low initially. Total ground-water discharge, through the constant-head boundaries, decreases 3 percent. Decreasing hydraulic conductivity introduced larger errors than those caused by increasing hydraulic conductivity indicating the model is more sensitive to lower rather than higher values.
3. Aquifer hydraulic conductivity set at 150 ft/d: Hydraulic conductivity throughout the aquifer was set at a uniform value of 150 ft/d. The sensitivity analysis was not completed due to large numerical oscillation and convergence problems. However, interim results of simulations that were terminated several iterations before simulation failure indicated that computed heads and ground-water discharge were similar to those in the steady-state model. This suggests that a uniform model value of hydraulic conductivity may be reasonable, time-saving, and sufficient for some modeling applications if the numerical problems can be overcome.
4. Streambed hydraulic conductivity times 10.0 (10xKs): Increasing the hydraulic conductivity of the streambed had little or no effect on computed heads and ground-water discharge. As shown in the profiles in figure 16, the computed (10xKs) heads were almost identical to those of the steady-state model. Differences between computed (10xKs) and observed heads at the 41 observation wells were actually smaller than those of the steady-state model indicating a relatively better fit. The insensitivity of the model shows that the streambed in the steady-state model does not control ground-water discharge to the river. This was confirmed by further analysis as described in (6) below.
5. Streambed hydraulic conductivity times 0.1 (0.1xKs): The greatest sensitivity of model results to changes in input parameters were produced when streambed hydraulic conductivity values were decreased by a factor of 10. Ground-water discharge to the river decreased by 7 percent, and heads throughout the model area increased by an average of 3 feet. As in previous simulations, the computed heads in areas of low hydraulic conductivity were most sensitive to the change in the value of the parameter. Ground-water discharge rates at streambed nodes differed from those in the steady-state model by up to 50 percent.
6. Leaky streambed boundary changed to constant-head boundary: The effect of changing the river from a leaky boundary to a constant-head boundary also was evaluated. The change had either little or no effect on computed heads and ground-water discharge and the results matched the steady-state model results very closely. The insensitivity of the model suggests, as in (4) above, that the leaky boundary in the steady-state model effectively is a constant-head boundary. The result of this determination is that a constant-head boundary condition along the river may be sufficient for some modeling applications in which stresses far from the river are evaluated. In cases where the impacts of pumping stresses are evaluated at wells near the river, a constant-head boundary condition will result in significant errors in simulated aquifer heads and well drawdowns.

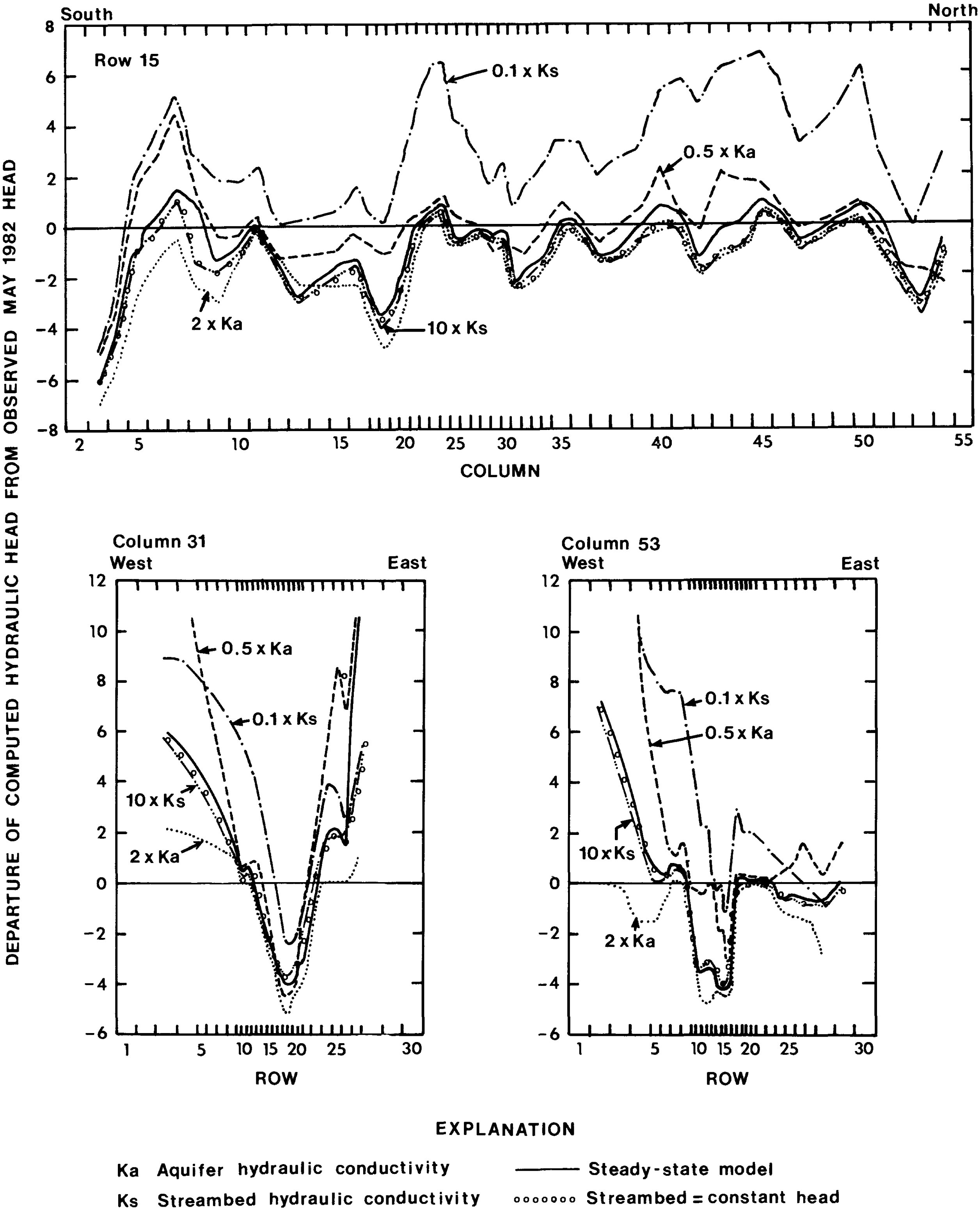


Figure 16. -- Effects of varying aquifer and streambed input values on the results of the steady-state model.

Summarizing the results of the steady-state sensitivity analysis, it is clear from figure 16 that the departure of the steady-state computed heads from the May 1982 observed heads could be reduced by increasing and decreasing one or more of the parameters in different areas, within the ranges shown. A better fit of computed and observed heads could be obtained with more adjustments in various parts of the model area. The close spacing of the profiles along row 15 and in the middle of columns 31 and 53 show that computed heads in the center of the valley are relatively insensitive to changes in the values of input parameters. This is consistent with the general conclusion that model results are most sensitive to lowered input parameter values. Progressively larger errors are introduced in the model when a progressively less permeable aquifer and (or) streambed are simulated.

Transient Conditions

Simulation of aquifer tests (1952-82)

Since 1952, aquifer tests of Mattapoisett, Fairhaven, and Marion well sites have been conducted using large-diameter wells. Tests at six sites have lasted as long as 7 days and have yielded numerous data on the hydraulic properties of the stratified-drift aquifer. Because of the lack of long-term, water-level data for the model area, a parameter estimation procedure was followed using the pumping-well and observation-well data gathered during aquifer tests at each site. The objective of the procedure was to improve model capability to simulate aquifer behavior in response to transient pumping stress.

Aquifer tests at each site were simulated individually. The simulations were done in the chronological order of the tests and only the well at the test site, and wells elsewhere in the valley existing at the time of the test, were modeled. The initial conditions were May 1982 observed head, recharge, and constant-flux boundary conditions. The specific yield of the unconfined aquifer was set at a constant value of 0.30, which is representative of medium-grained sand materials (Todd, 1980; p. 38). This value was checked with an analytical model by computing drawdowns at observation wells at aquifer test sites using different values of specific yield, and comparing those drawdowns with the observed drawdowns. The 0.30 value most closely represents the lithology of the aquifer in those areas of clean, medium-grained sand, such as north of the Marion test-well site near New Bedford Road, and in the Snows Pond-Snipatuit Pond area. The value is high for other areas which contain coarser and finer grained sediments.

The storage coefficient of the confined part of the aquifer in the Wolf Island Road area was set at 3.0×10^{-5}. This value was revised during transient model runs from an initial value of 5.0×10^{-5} determined from aquifer tests at the Fairhaven three-well site (Wright-Pierce, Inc., 1981; Wright-Pierce, Inc., 1982). The storage coefficient of the leaky confining bed was set during model runs at 1.0×10^{-3}.

Computed heads in the well, in the well node, and in the adjacent nodes were compared with the aquifer-test data. The computed head for a well node represents an average hydraulic head for the block and is not the head in the well. In the model, an equation is used to extrapolate from the average head in the node to the head in a well at a known well radius. Thus, model input requires data on municipal well radii and model output consists of hydraulic head in both the well and the well node. In the case of withdrawal, the head in the well is always lower than the head in the node.

The results of the parameter estimation procedure for the transient model are shown in table 7. Adjustments to the model were made only to values of aquifer hydraulic conductivity in well nodes and adjacent nodes. For clarity and simplicity, detailed results of the transient simulations are not shown in table 7, and only the computed and observed drawdowns in the wells and well nodes are compared. The following discussion summarizes the results of the relatively long and complex modeling procedure.

Table 7.--Aquifer-test sites, model conditions, and simulated drawdowns in pumping wells; transient model

Mattapoisett well 3 11/29/52-12/2/52		Mattapoisett well 4 8/24/70-9/1/70	
Model node	15,12	Model node	16,13
Well diameter	0.50 foot	Well diameter	0.67 foot
Saturated thickness	60.00 feet	Saturated thickness	63.00 feet
Pumping rate	variable	Pumping rate	variable[1]
Hydraulic conductivity:		Hydraulic conductivity:	
Aquifer test	170 ft/d	Aquifer test	200 ft/d
Model	150 ft/d	Model	125 ft/d
Observation well total drawdown:		Observation well total drawdown:	
At 4 feet	5.50 feet	At 2 feet	6.50 feet
At 100 feet	3.10 feet	At 80 feet	2.50 feet
Drawdown in node[2]	2.2 feet	Drawdown in node[2]	2.2 feet

Time since pumping began, in minutes	Pumping rate, in gallons per minute	Drawdown in well, in feet		Time since pumping began, in minutes	Pumping rate, in gallons per minute	Drawdown in well, in feet	
		Observed[3]	Model			Observed[1]	Model
7.2	440	7.00	8.69	30.8	305	5.30	6.39
23.5	440	7.78	8.72	50.0	305	5.52	6.41
60.0	440	8.40	8.78	110.0	350	6.74	7.47
148.6	465	9.52	9.46	190.8	402	8.06	8.73
347.9	465	10.11	9.73	430.0	402	8.53	8.95
796.4	465	10.69	10.19	1200.0	402	9.13	9.38
1200.0	465	10.98	10.48	2159.7	402	9.47	9.69
2300.0	457	11.24	10.86	5000.0	402	9.95	10.24
3007.1	476	11.90	11.56	9200.0	402	10.31	10.82
4108.6	476	12.13	11.86	10,920.0	430	11.13	11.78
5100.0	476	12.28	12.07				

[1] Mattapoisett well 3, located 1,200 feet south, was alternately on and off.
[2] Nodes 15,12 and 16,13 are 208 X 832 feet.
[3] Pumping well water-level data not available; drawdown computed from field data under Theis nonequilibrium aquifer conditions (Walton, 1970) and verified with available observation-well data.

Table 7.--Aquifer-test sites, model conditions, and simulated drawdowns in pumping wells; transient model (continued)

Marion well Wolf Island Road 7/24/75		Marion test well New Bedford Road 11/5-10/80	
Model node	19,29	Model node	15,40
Well diameter	2.00 feet	Well diameter	0.66 feet
Saturated thickness	59.00 feet	Saturated thickness	60.70 feet
Pumping rate	constant	Pumping rate	variable
Hydraulic conductivity:		Hydraulic conductivity:	
Aquifer test	195 ft/d	Aquifer test	160 ft/d
Model	130 ft/d	Model	35 ft/d
Observation well total drawdown		Observation well total drawdown:	
At 3 feet	15.20 feet	At 2 feet	20.00 feet
At 100 feet	10.00 feet	At 125 feet	2.20 feet
Drawdown in node[4]	3.60 feet	Drawdown in node[4]	1.50 feet

Time since pumping began, in minutes	Pumping rate, in gallons per minute	Drawdown in well, in feet		Time since pumping began, in minutes	Pumping rate, in gallons per minute	Drawdown in well, in feet	
		Observed[5]	Model			Observed[6]	Model
6.2	710	7.80	10.90	5.0	200	17.00	17.08
20.1	710	9.40	10.97	32.0	225	43.00	19.72
51.4	710	10.35	11.12	430.0	243	44.00	21.93
122.0	710	11.75	11.44	530.0	180	--	15.29
280.5	710	12.95	12.10	1127.6	180	--	15.41
637.3	710	14.31	13.34	1500.0	180	--	15.49
1440.0	710	15.17	15.34	2424.2	220	42.00	19.95
				3000.0	220	42.00	20.14
				7290.0	239	46.00	24.14

[4] Node 19, 29 is 208 X 416 feet; node 15, 40 is 208 X 832 feet.
[5] R. E. Chapman Co., unpublished data, written commun., 1975.
[6] Camp, Dresser and McKee, Inc., 1980. Water-level measurement problems occurred throughout the test, broken air line and poor well efficiency.

Table 7.--Aquifer-test sites, model conditions, and simulated drawdowns in pumping wells; transient model (continued)

Mattapoisett test well 11-6 11/5-10/80	
Model node	19,29
Well diameter	0.66 foot
Saturated thickness	70.00 feet
Pumping rate	variable
Hydraulic conductivity:	
Aquifer test	150 ft/d
Model	100 ft/d
Observation well total drawdown:	
At 2 feet	11.00 feet
At 50 feet	5.00 feet
Drawdown in node[7]	4.50 feet

Time since pumping began, in minutes	Pumping rate in gallons per minute	Drawdown in well, in feet			
		Well 11-6		Marion well[8]	
		Observed[9]	Model	Observed[10]	Model
5.0	195	4.00	4.08	--	--
32.0	195	4.00	4.04	--	--
430.4	195	5.50	4.28	--	--
530.0	0	--	.20	--	--
1127.6	421	14.00	10.22	--	--
1500.0	421	14.00	10.69	--	--
2424.2	421	14.00	11.62	13.20	12.59
3000.0	421	14.00	12.07	14.70	13.84
7290.0	412	15.00	13.94	--	.52
7350.0	412	15.00	13.97	12.20	10.70
7530.0	412	15.00	14.04	14.70	11.30

[7] Node 19,29 and 17,23 are 208 X 416 feet.
[8] Marion well alternately on (650 gal/min) and off.
[9] Dufresne-Henry, Inc., 1981, data are approximate, well shut down temporarily at 430.0 minutes.
[10] Unpublished data in the files of the Massachusetts Office of the Survey.

Table 7.--Aquifer-test sites, model conditions, and simulated drawdowns in pumping wells; transient model (continued)

Fairhaven test well 20-79 5/25-31/81	
Model node	17,23
Well diameter	2.00 feet
Saturated thickness	68.00 feet
Pumping rate	constant
Hydraulic conductivity:	
Aquifer test	41 ft/d
Model	50 ft/d
Observation well total drawdown:	
At 3 feet	18.50 feet
At 100 feet	13.00 feet
Drawdown in node[7]	19.90 feet

Time since pumping began, in minutes	Pumping rate in gallons per minute	Drawdown in well, in feet: Well 20-79		Drawdown in well, in feet: Marion well[8]	
		Observed[11]	Model	Observed[12]	Model
8.5	300	31.15	44.21	--	--
51.7	300	47.41	47.54	--	--
270.0	300	49.65	51.20	--	--
390.4	300	51.17	54.60	13.00	10.28
1000.0	300	53.08	56.42	13.00	12.15
1700.0	300	52.03	58.22	--	1.30
2350.0	300	53.57	59.00	13.00	13.20
3100.0	300	52.33	59.42	--	1.95
3210.0	300	52.47	59.53	13.00	12.53
3765.0	300	53.53	59.64	13.00	13.89
5685.0	300	52.03	59.84	--	1.57

[7] Node 19,29 and 17,23 are 208 X 416 feet.
[8] Marion well alternately on (650 gal/min) and off.
[11] Wright-Pierce, Inc., 1981.
[12] Estimate from previous, similar pumping conditions.

Table 7.--Aquifer test sites, model conditions, and simulated drawdowns in pumping wells; transient model (continued)

Fairhaven-Marion Test, Wolf Island Road
10/13-18/82

	Fairhaven wells[13]			Marion well[14]
	20-79	8-79	11-81	
Model node	17,23	17,24	17,25	19,29
Well diameter	2.0 feet	1.5 feet	1.5 feet	1.5 feet
Saturated thickness	68.5 feet	67.5 feet	59.0 feet	59.0 feet
Pumping rate	constant	variable	constant	constant
Hydraulic conductivity:				
Aquifer test	41.0 ft/d	80.0 ft/d	91.0 ft/d	195 ft/d
Model	50.0 ft/d	80.0 ft/d	40.0 ft/d	130 ft/d
	18.0 ft/d	20.0 ft/d	15.0 ft/d	--
Observed drawdown at 100 feet	--	--	--	--
Drawdown in node[15]	15.6 feet	17.5 feet	12.2 feet	8.0 feet

Time since pumping began, in minutes	Well 20-79			Well 8-79			Well 11-81			Marion		
	Pumping rate	Drawdown		Pumping rate	Drawdown		Pumping rate	Drawdown		Pumping rate	Drawdown	
		Observed	Model		Observed	Model		Observed	Model		Observed	Model
4.3	239	31.11	33.09	278	26.38	34.89	180	14.89	31.27	650	--	9.88
26.3	239	33.95	34.54	278	38.20	36.55	180	30.00	32.16	650	--	9.96
40.0	239	36.88	35.02	278	39.24	37.09	180	31.42	32.47	650	--	10.02
144.7	239	39.55	38.31	282	43.10	41.34	180	33.99	34.62	650	--	10.41
210.0	239	40.57	38.92	282	43.64	42.03	180	34.79	35.06	650	11.00	10.65
731.6	239	42.63	42.30	285	46.49	47.37	180	37.05	37.54	650	14.80	12.28
1410.0	239	43.46	43.31	285	47.38	47.57	180	37.99	38.36	650	16.90	13.87
1893.9	239	43.81	44.45	282	47.75	48.50	180	38.56	39.25	650	17.25	14.79
4343.6	239	44.55	45.40	282	48.80	49.61	180	39.67	40.07	650	18.50	17.72
5870.0	239	45.14	45.65	282	49.07	49.91	180	40.04	40.30	650	19.00	18.95
6429.1	239	45.20	44.78	275	48.37	49.04	180	40.10	40.41	650	19.30	19.37
7300.0	239	45.37	45.83	275	48.51	49.06	180	40.33	40.45	650	19.42	19.97

[13] Wright-Pierce, Inc, 1982.
[14] Ray Pickles, Marion Water Department, written commun., 1983.
[15] Nodes 17,23, 17,24, 17,25, and 19,29 are 208 X 416 feet.

Only minor adjustments were made to the steady-state model values of hydraulic conductivity. At well nodes, values were decreased from 0 to 20 percent, and at adjacent nodes smaller changes were made. The average difference between final, accepted model values and initial input values estimated by private consultants is 33 percent, which is approximately the probable range of error in the hydraulic conductivity data for these types of sediments. One exception was the Marion test-well site near New Bedford Road. The difference between the model value and the estimated value of hydraulic conductivity exceeded the likely range of error for that parameter. Equipment problems and poor well efficiency plagued the aquifer test (Camp, Dresser and McKee, 1980) and an extremely large drawdown in the pumping well occurred. These problems, together with related information, suggest that the observed drawdowns were excessive and that the departure of model hydraulic conductivity and computed drawdown from estimated hydraulic conductivity and observed drawdown may be due more to aquifer-test problems than to a defect in the model.

Pumping rates varied during most aquifer tests, and the model was designed accordingly to simulate multiple pumping periods and variable pumping rates as accurately as possible. Computed maximum drawdowns in pumping wells finished in the unconfined aquifer and located next to the river (Mattapoisett wells 3,4,11-6) were within 1 foot of the observed drawdown. As a general rule at these sites, aquifer tests that last 5 to 7 days with pumping rates in the range 400 to 500 gal/min resulted in pumping-well drawdowns ranging from 11 to 15 feet. Observed drawdowns in both the pumping well and the observation wells that are located near the river were small relative to the drawdowns in those wells at sites located far from the river. The small drawdowns occurred because of induced infiltration from the river in the later stages of the aquifer tests.

In 1981, the first tests of the confined aquifer in the Wolf Island Road area, using large-diameter wells, were conducted. A 5-day test of Fairhaven well 20-79 caused a maximum drawdown in the pumping well of over 50 feet. The Marion well, located 2,500 feet north of well 20-79, was pumped 6 to 12 hours daily during the aquifer test and imposed up to 2.2 feet of drawdown at the Fairhaven well (Wright-Pierce, Inc., 1981). In the model, the computed drawdown in well 20-79 was 10 to 15 percent greater than the observed drawdown, and the rising and falling of water levels during the test, recording the on-off cycle of the Marion well, was simulated satisfactorily but not precisely. Another indication of how closely the model simulated aquifer behavior was the difference between computed and observed interference effects. For example, at 400 feet north of well 20-79 the observed interference was 3.0 feet; the computed interference was 4.2 feet.

Because of great concern about potential interference and lowered sustained yields of wells in the confined aquifer, a 5-day, joint aquifer test was conducted in November 1982 at the Fairhaven and Marion well sites. During the test, the three Fairhaven wells (20-79, 8-79, and 11-81) were pumped at relatively low rates, and the Marion well was pumped at its normal operating rate. Drawdown in the Fairhaven wells ranged from 40 to 49 feet, and drawdown in the Marion well exceeded normal drawdown levels (13 to 15 feet) by several feet. The model simulated the well drawdowns accurately: Differences between computed and observed total drawdowns ranged from 0.1 to 0.5 foot. Computed drawdowns in the well nodes ranged from 8.0 feet at the Marion site to 17.5 feet at the site of well 8-79 and compared closely with the drawdowns measured in observation wells located within 100 feet of each pumping well. The computed interference between the Fairhaven and Marion well sites ranged from 0.5 to 2.0 feet.

Transient response time of the stratified-drift aquifer

Although the aquifer responds to seasonal and short-term variations of recharge, the response time is not instantaneous. In the case of long-term reduced recharge due to a drought, storage changes in a stratified-drift aquifer may take from either weeks to months (Wilson and Scheiber, 1982). Knowledge of how fast an aquifer responds to a change in recharge is important, particularly in modeling studies, because it indicates how long the real conditions would have to last to equal the results of the steady-state model. Therefore, to use a steady-state model for prediction purposes, it is necessary to test aquifer-response time.

Aquifer response to reduced recharge can be measured by evaluating change in storage. Ideally, the time it takes for the aquifer to re-equilibrate to the reduced recharge condition, and thus storage changes to cease, is the aquifer-response time. For practical purposes, a reasonable estimate of response time is the time it takes for 75 percent of the total storage change to occur. The transient model was used to test aquifer-response time by simulating a long-term drought. Precipitation data from 1965 were used to estimate a reduced recharge rate and a corresponding reduced rate of till leakage. Results of the model simulation showing the computed change in storage during long-term conditions of reduced recharged are illustrated in figure 17.

Approximately 75 percent of the total storage decline takes place in 360 days, and further decreases in storage take place, but at progressively smaller rates. The response time of the aquifer is significantly slower than the 45-day response time estimated for the aquifer on Cape Cod (Wilson and Scheiber, 1982), and this result may be interpreted in one of two ways. Either the response accurately reflects the relatively lower permeability of the stratified drift in the Mattapoisett River valley, or the predicted response is too slow because the average value of the specific yield of the aquifer used in the model (0.30) is too high. Should the latter interpretation be correct, a lower average value for specific yield would result in a response time which more closely compares with the Wilson and Scheiber value. In either case, the 360-day value may be considered a reasonable first estimate of the response of the stratified-drift aquifer which should be further defined as data become available. Aquifer response varies over the model area, as indicated by the curves of head decline in figure 17. Heads in the aquifer respond more quickly in areas of high hydraulic conductivity (node 4,12; K = 90 ft/d) than in areas of low hydraulic conductivity (node 27,37; K = 10 ft/d).

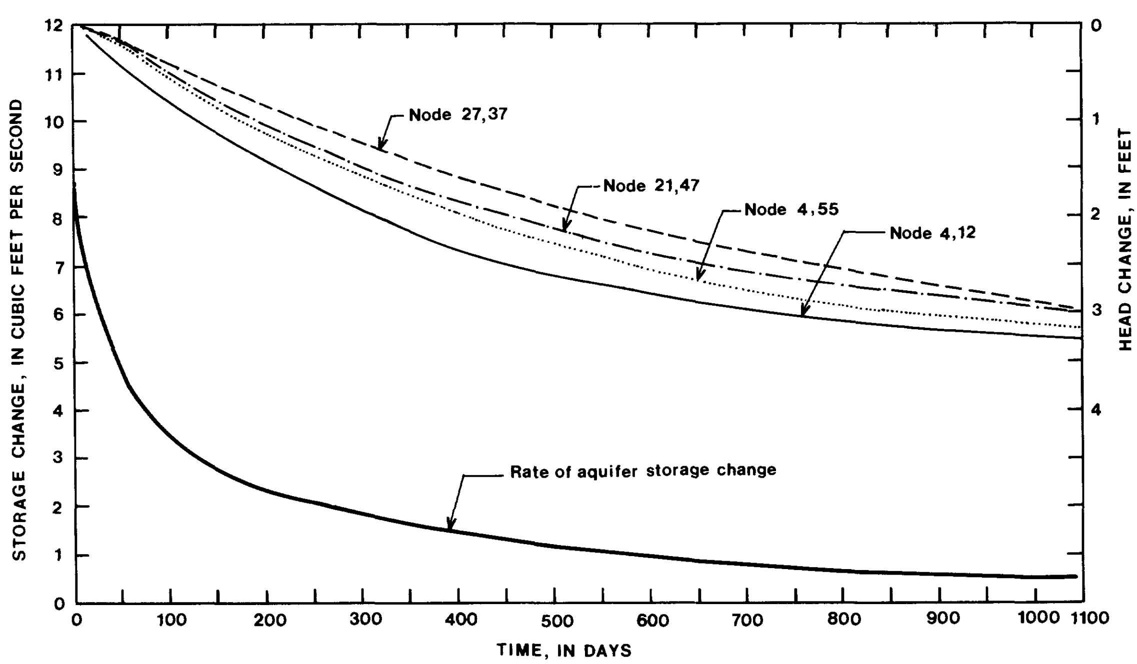


Figure 17.--Rate of aquifer storage change and head decline under reduced recharge conditions.

The purpose of this analysis is to show that steady-state model predictions must be carefully evaluated when monthly, weekly, and especially daily variations in recharge conditions are simulated. Assuming that model values of specific yield and storage are representative of the aquifer, real, long-term, reduced recharge conditions would have to last at least 1 year for most of the total storage change to occur, and thus for steady-state model predictions to be realized. In 1 month of significantly reduced recharge, the aquifer will respond rapidly, but not completely, to the change in stress, and effects due to withdrawal may be partly hidden by transient aquifer response. As a result, steady-state model predictions are conservative and are applicable chiefly to seasonal and annual recharge conditions.

Predicted Effects of Ground-Water Withdrawals During Drought

The digital model was used to simulate drawdown in the vicinity of wells and decreased ground-water discharge to the Mattapoisett River resulting from projected withdrawals by municipal wells. In addition, the simulations were used to assess the impact of withdrawals under reduced recharge conditions. Estimates of projected withdrawals for the period 1983-90+ were incorporated into the model, and the effects of withdrawals were considered both separately and in combination. Surface-water withdrawals by cranberry bog operators were assumed to affect streamflow only, and the impacts of these users were not simulated.

The model was designed and tested under average annual 1982 recharge conditions; however, of most concern is the impact of withdrawals during drought conditions. To use the model to predict impacts under drought conditions, it was necessary to find answers to several important questions: (1) What are the drought conditions under which pumping impacts are to be evaluated? (2) What model adjustments are needed to simulate drought conditions? and (3) What are the projected pumping scenarios that are to be simulated?

Model Adjustments to Simulate Drought Conditions

Two reduced recharge conditions were chosen for simulating drought:

1. A "dry" condition representative of 1965 average annual recharge, in which streamflow of the Mattapoisett River equals the 85 percent duration of flow;
2. A "severely dry" condition representative of the driest period of 1965, in which streamflow of the Mattapoisett River equals the 99.7 percent duration of flow.

For the dry condition, an average annual recharge rate of 9.2 inches was used in model simulations. This rate was estimated from the 1965 total precipitation at Rochester, Massachusetts. The dry condition is assumed to reflect a long-term recharge rate that is intermediate between average and zero recharge.

For the severely dry condition, a recharge rate that is equivalent to an annual rate of 5.6 inches was used in the model simulations. Although there were periods of zero recharge in 1965, a model recharge rate greater than zero was used because it was assumed that some recharge occurred during the year. In terms of discharge to the Mattapoisett River, the recharge rate is equivalent to 2.0 ft^3/s which is the estimated 7-day, 10-year low flow at the River Road stream gage (table 1).

Lowering the recharge rate was not the only model adjustment required to simulate drought conditions. Several other adjustments were made, in part because it was necessary to change the conceptual model to more accurately define the aquifer under reduced recharge conditions, and in part because of limitations in the way in which the digital model computes hydraulic head at constant-head and at leaky confining-bed nodes. The model adjustments were as follows:

1. The hydraulic head in the stream nodes representing the Mattapoisett River and its tributaries was lowered by an average of 0.8 foot below that on May 18, 1982. The lowered head values correspond to stream stage at 7-day, 10-year low flow. Although small tributary streams would actually dry up during a drought, this change closely represents the decline in stage in the main stem of the river.

2. The hydraulic head in the confining bed beneath the Wolf Island Road area was lowered 2 feet. This change is relatively small considering the range of average annual water-table fluctuation in the area. The effect of this change is to lower water levels and decrease the vertical head gradient across the confining bed which, in turn, decreases the leakage to the confined part of the aquifer.

3. The hydraulic head of the constant head nodes representing Snipatuit Pond and Snows Pond was lowered 1 foot. Pond levels are regulated, but a decline during drought conditions is assumed.

4. Leakage rates along the till boundary were lowered. For the dry condition, the leakage rate was lowered 45 percent. For the severely dry condition, it was assumed that a very low water table in the till effectively stops leakage. This assumption is based on the general experience gained in till-covered areas in Massachusetts during the 1965 drought when very large well drawdowns were observed and streams and wetlands went dry. Accordingly, the leakage rate was set to zero.

Municipal Well Pumping Scenarios and the Modeling Procedure

As explained earlier in this report, three of the four towns in the study area either have tested or propose to test new well sites to meet these demands. To determine the impact of these pumping wells on both the aquifer and the river, the projected increased pumping rates for the current and the proposed wells were formulated in a series of pumping scenarios and were incorporated into the model (table 8). Because of the importance of evaluating maximum demands under driest conditions, three levels of pumping demand were estimated, corresponding to the 3-month summer average, the high-month average, and the maximum daily consumption. The projected pumping rates listed in table 8 were obtained from the towns; the Massachusetts Department of Environmental Management, Division of Water Resources; and information on proposed well sites supplied by the towns' consultants.

Pumping scenarios 1 to 3 represent the demands of the current, pre-1983 wells. Scenarios 4 to 6 represent the demands of the current wells plus the demands of the Fairhaven three-well system which is proposed to begin operation in mid-1983. Additional proposed wells are not scheduled to begin operation until after 1990. Therefore, scenarios 7 to 10 are designed to simulate the demands of the current wells plus the progressive start-up of the proposed wells. The proposed wells in scenarios 7 to 10 are arranged in order of their current stage of testing and development.

The modeling procedure consisted of two steps:

1. The transient model was adjusted for drought conditions according to the changes described above. Starting from the computed May 1982 steady-state heads, and using a reduced recharge rate, the transient model was run until head and storage changes equilibrated under the new recharge condition and steady-state was reached.

2. Using the reduced recharge rates, each of the 10 scenarios was simulated using the new steady-state model, and the computed results were evaluated.

Steps 1 and 2 were followed for both the dry and the severely dry reduced recharge conditions. Results of the modeling runs are presented primarily in tables showing the predicted drawdown in the pumping wells and the resulting total ground-water discharge of the Mattapoisett River as measured at its outfall to the ocean. Also provided are contour maps showing simulated changes in hydraulic head as compared with average annual conditions. For illustration purposes in this report, the results of Scenarios 2, 5, and 10 are compared.

Table 8.--Ten steady-state pumping scenarios

(Pumping rates, in cubic feet per second)

Well	Current withdrawal, 1982			Current and proposed withdrawal 1983-90[1]			Current and proposed withdrawal, 1990+[2]			
	Summer average[3]	High month[4]	Maximum daily[5]	Summer average	High month	Maximum daily	High month	High month	High month	High month
Scenario number	1	2	3	4	5	6	7	8	9	10
Fairhaven:										
River Road	0.50	0.56	1.11	0.50	0.56	1.11	0.56	0.56	0.56	0.56
Mattapoisett 2	.00	.00	.23	.00	.00	.23	.00	.00	.00	.00
Mattapoisett 3	.23	.23	1.24	.23	.23	1.24	.23	.23	.23	.23
Mattapoisett 4	.54	.68	1.55	.54	.68	1.55	.68	.68	.68	.68
Marion:										
Wolf Island Road	.62	.81	1.24	.62	.81	1.24	.81	.81	.81	.81
Fairhaven 20-79[6]	--	--	--	[7].33	[8].41	[9].57	.41	.41	.41	.41
Fairhaven 8-79	--	--	--	.37	.46	.64	.46	.46	.46	.46
Fairhaven 11-81	--	--	--	.24	.31	.43	.31	.31	.31	.31
Mattapoisett 11-6	--	--	--	--	--	--	[8].77	.77	.77	.77
Marion:										
New Bedford Road	--	--	--	--	--	--	--	[10].60	.60	.60
Fairhaven:										
Tinkham Lane	--	--	--	--	--	--	--	--	[10].77	.77
Mattapoisett 11-2	--	--	--	--	--	--	--	--	--	[10].77
Total	1.89	2.28	5.37	2.83	3.46	7.01	4.23	4.83	5.60	6.37

[1] Assume 1982 withdrawals at current wells.
[2] 1982 pumping rates for high month; proposed withdrawals in order of current stage of development.
[3] June, July, August.
[4] July.
[5] Maximum daily withdrawal rate assumed equal to pumping capacity.
[6] Percent of total withdrawal by each well: 20-79, 35 percent; 8-79, 39 percent; 11-81, 26 percent.
[7] 80 percent of 1982 approved pumping limit (Massachusetts Department of Environmental Quality Engineering, written commun., 1982).
[8] 1982 approved pumping limit (Massachusetts Department of Environmental Quality Engineering, written commun., 1982).
[9] 90 day safe discharge (Wright-Pierce, Inc., 1982; p. 15).
[10] Approximately 50 percent of the proposed pumping capacity.

Dry Conditions

Table 9 shows the combined effects of both reduced recharge under dry conditions and projected withdrawals. For each scenario, the predicted drawdown in each well is listed along with the total withdrawal rate and the total ground-water discharge from the aquifer. For comparison, the estimated available drawdown at each well is also listed.

With the exception of several wells in Scenarios 3 and 6, the predicted drawdown in each well, in all of the pumping scenarios, is less than the estimated available drawdown. Scenarios 3 and 6 simulate extremely high withdrawal rates and in each case the predicted drawdowns at a number of wells exceed the available drawdowns (indicated in parentheses). In Scenario 3, predicted drawdowns are exceeded in Mattapoisett well 4. In Scenario 6, predicted drawdowns are exceeded in this well and in each of the three Fairhaven wells that are located on Wolf Island Road. Furthermore, the available drawdown limit is nearly reached in the Marion well. The drawdowns for the wells in Scenarios 7 to 10 show the progressive impact on aquifer head and ground-water discharge as each well is turned on. For example, turning on the proposed Fairhaven well off Tinkham Lane (Scenario 9) results in a predicted change in drawdown in the nearby Mattapoisett well 4 from 10.49 to 11.80 feet and a decrease in flow of the Mattapoisett River, as measured at the ocean outfall, from 2.92 to 2.15 ft^3/s.

Reviewing all the data, total withdrawal removes from 25 percent (Scenario 1) to 90 percent (Scenario 6) of the available ground-water discharge of the aquifer. Under these dry conditions, the predicted results show that some ground-water discharge is maintained in the Mattapoisett River, even during the heaviest municipal withdrawals.

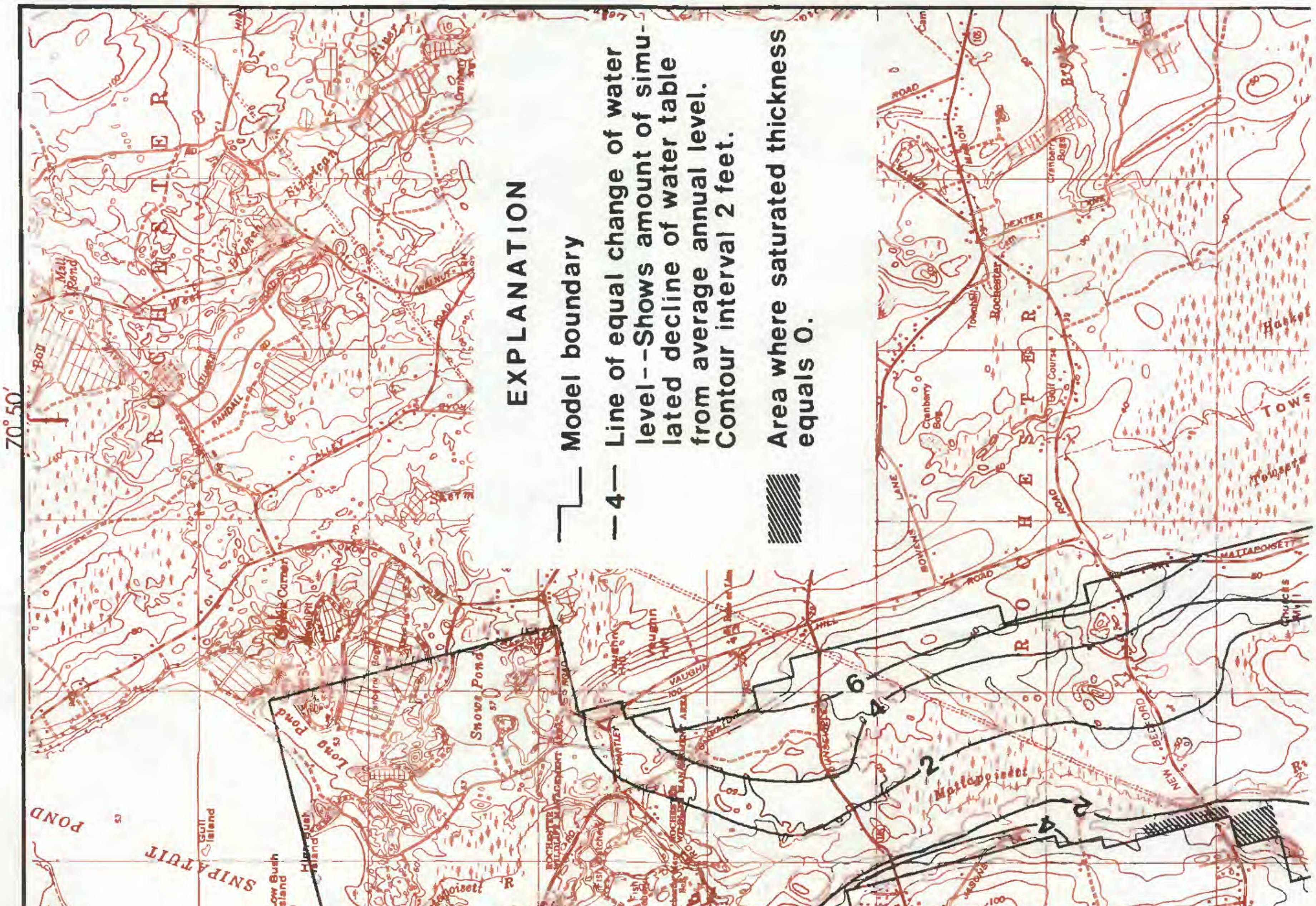
The combined effects of reduced recharge and municipal withdrawal are clearly illustrated in figures 18 to 20. The effects of reduced recharge are especially evident along the east and west boundaries and in the northern part of the model area where simulated water levels are more than 9 feet lower than average levels. Also, the saturated thickness of the aquifer decreases to zero in a few widely scattered areas where the aquifer is thin. In general, the predicted water-level declines are within the range of observed water levels and seem to be representative of dry recharge conditions. As for the additional stress due to withdrawal, the impacts of the current wells (Scenario 2, fig. 18), the current wells plus the new Fairhaven wells (Scenario 5, fig. 19), and all the current and proposed wells (Scenario 10, fig. 20) are shown by the increased water-level declines in the vicinity of each well. The broadest and deepest cones of depression form at wells constructed beneath and adjacent to the confining bed at Wolf Island Road. As seen from real experiences, wells that are located in the unconfined aquifer adjacent to the Mattapoisett River, but have construction designs and withdrawal rates that are similar to those of wells in the Wolf Island Road area, create much smaller cones of depression.

Table 9.--Steady-state model results--dry conditions: Computed well drawdown and total ground-water discharge

(Drawdown, in feet; pumpage rates and ground-water discharge, in cubic feet per second. Values enclosed in parentheses indicates that the predicted drawdown exceeds the estimated available drawdown.)

Well	Available drawdown[1]	Drawdown from current pumpage, 1982			Drawdown from current and proposed pumpage, 1982-90			Drawdown from current and proposed pumpage 1990+			
		Summer average	High month	Maximum daily	Summer average	High month	Maximum daily	High month	High month	High month	High month
Scenario number		1	2	3	4	5	6	7	8	9	10
Fairhaven:											
River Road[2]	[3]30.0	4.70	5.40	7.70	4.70	5.40	7.70	5.40	5.40	5.40	5.40
Mattapoisett 2[2]	[3]21.5	.00	.00	3.70	.00	.00	3.70	.00	.00	.00	.00
Mattapoisett 3	43.0	3.94	4.04	23.61	3.94	4.04	23.61	4.04	4.04	4.25	4.46
Mattapoisett 4	[4]30.0	8.68	10.49	(38.79)	8.68	10.49	(38.79)	10.49	10.49	11.80	11.94
Marion:											
Wolf Island Road	40.0	14.96	20.59	38.70	15.05	20.72	39.05	20.72	20.72	20.72	20.72
Fairhaven 20-79	43.2	--	--	--	27.91	35.06	(49.35)	35.27	35.27	35.27	35.27
Fairhaven 8-79	41.7	--	--	--	29.26	36.75	(51.68)	36.82	36.82	36.82	36.82
Fairhaven 11-81	33.0	--	--	--	23.57	31.02	(43.92)	31.04	31.04	31.04	31.04
Mattapoisett 11-6	44.0	--	--	--	--	--	--	18.85	18.85	18.89	18.89
Marion:											
New Bedford Road	52.0	--	--	--	--	--	--	--	30.62	30.62	30.62
Fairhaven:											
Tinkham Lane	95.0	--	--	--	--	--	--	--	--	11.44	11.48
Mattapoisett 11-2	[5]33.0	--	--	--	--	--	--	--	--	--	20.89
Total pumpage from aquifer[6]		1.89	2.28	5.37	2.83	3.46	7.01	4.23	4.83	5.60	6.37
Total ground-water discharge of the Mattapoisett River[6]		5.86	5.48	2.37	4.92	4.29	0.74	3.53	2.92	2.15	1.38

[1]Depth from average level of water table to 5 feet above the top of the current/proposed well screen; interference effects of Marion, Fairhaven, and Mattapoisett wells located in the Wolf Island Road area included. Estimated available drawdown values are obtained from reports of aquifer tests conducted by town consultants.
[2]Drawdown in the node representing a well field composed of numerous 2-inch diameter wells.
[3]Estimated average available drawdown of well field.
[4]Upper screen estimate; drawdown does not include interference from Fairhaven Tinkham Lane well.
[5]Estimated; no field data.
[6]Under "no pumping" conditions, total ground-water discharge of the Mattapoisett River is 7.75 cubic feet per second.


EXPLANATION
Model boundary
—4— Line of equal change of water level--Shows amount of simulated decline of water table from average annual level. Contour interval 2 feet.
Area where saturated thickness equals 0.
70°50′
70°54′
41°45′
ROCHESTER
CUSHNET
SNIPATUIT POND
Long Pond
Snows Pond
Mattapoisett
Dry Swamp
Logging Swamp
Long Plain
New Bedford Reservoir
PLYMOUTH CO
BRISTOL CO

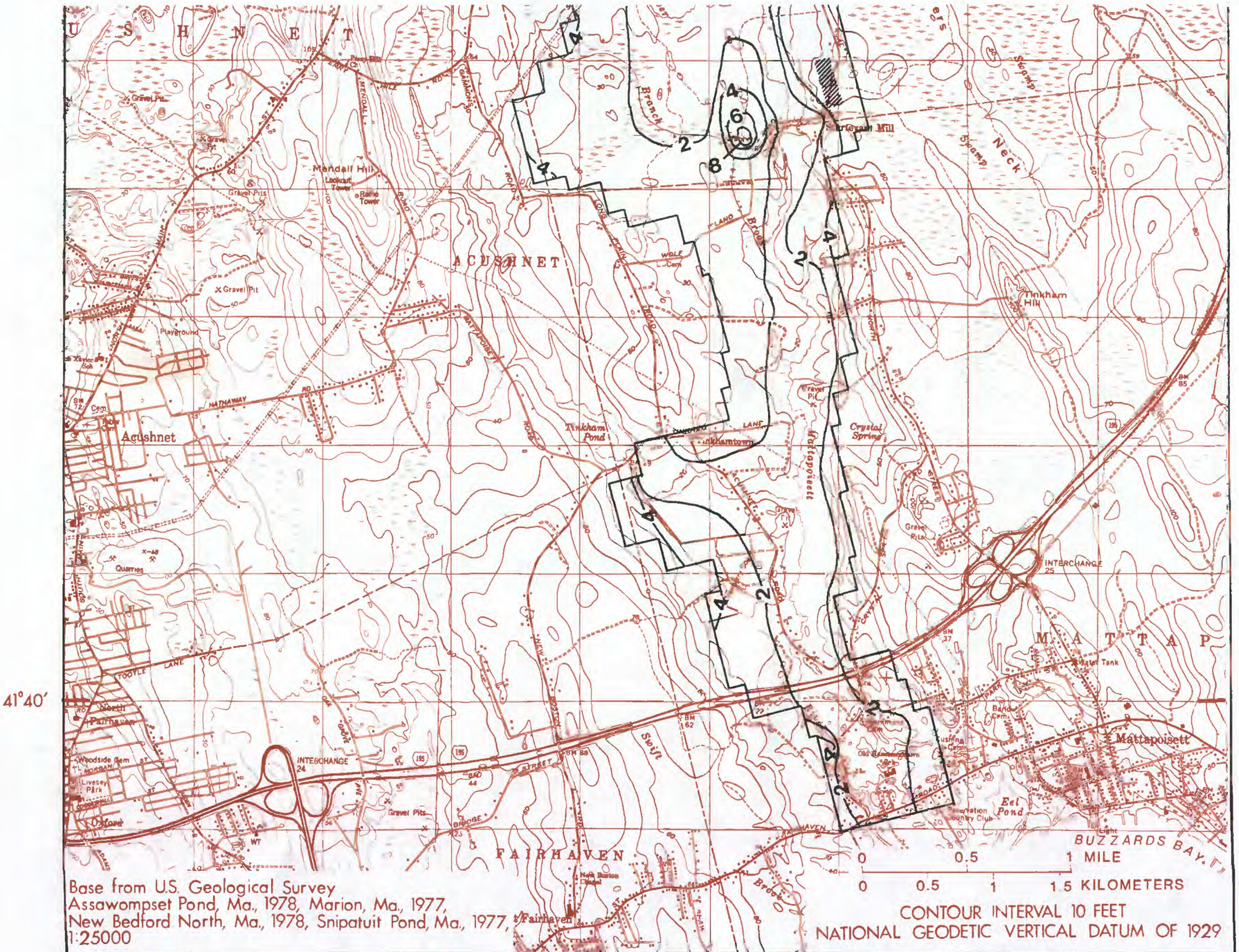
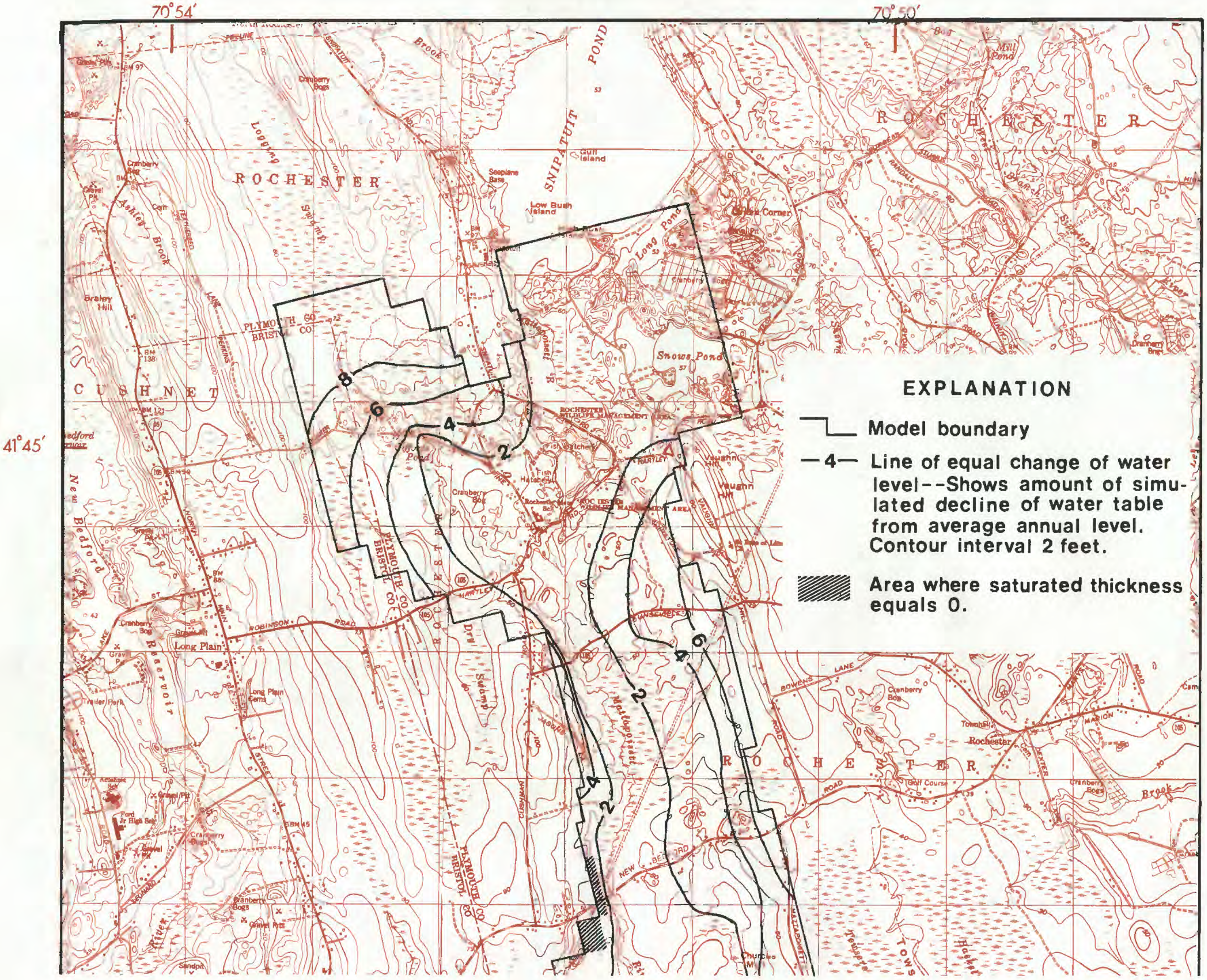


Figure 18.--Simulated decline in the altitude of the water table: Scenario 2, dry conditions.


EXPLANATION
Model boundary
—4— Line of equal change of water level--Shows amount of simulated decline of water table from average annual level. Contour interval 2 feet.
Area where saturated thickness equals 0.
70°54′
70°50′
41°45′
ROCHESTER
CUSHNET
SNIPATUIT POND
Long Pond
Snows Pond
Long Plain
Reservoir
PLYMOUTH CO
BRISTOL CO
8
6
4
2

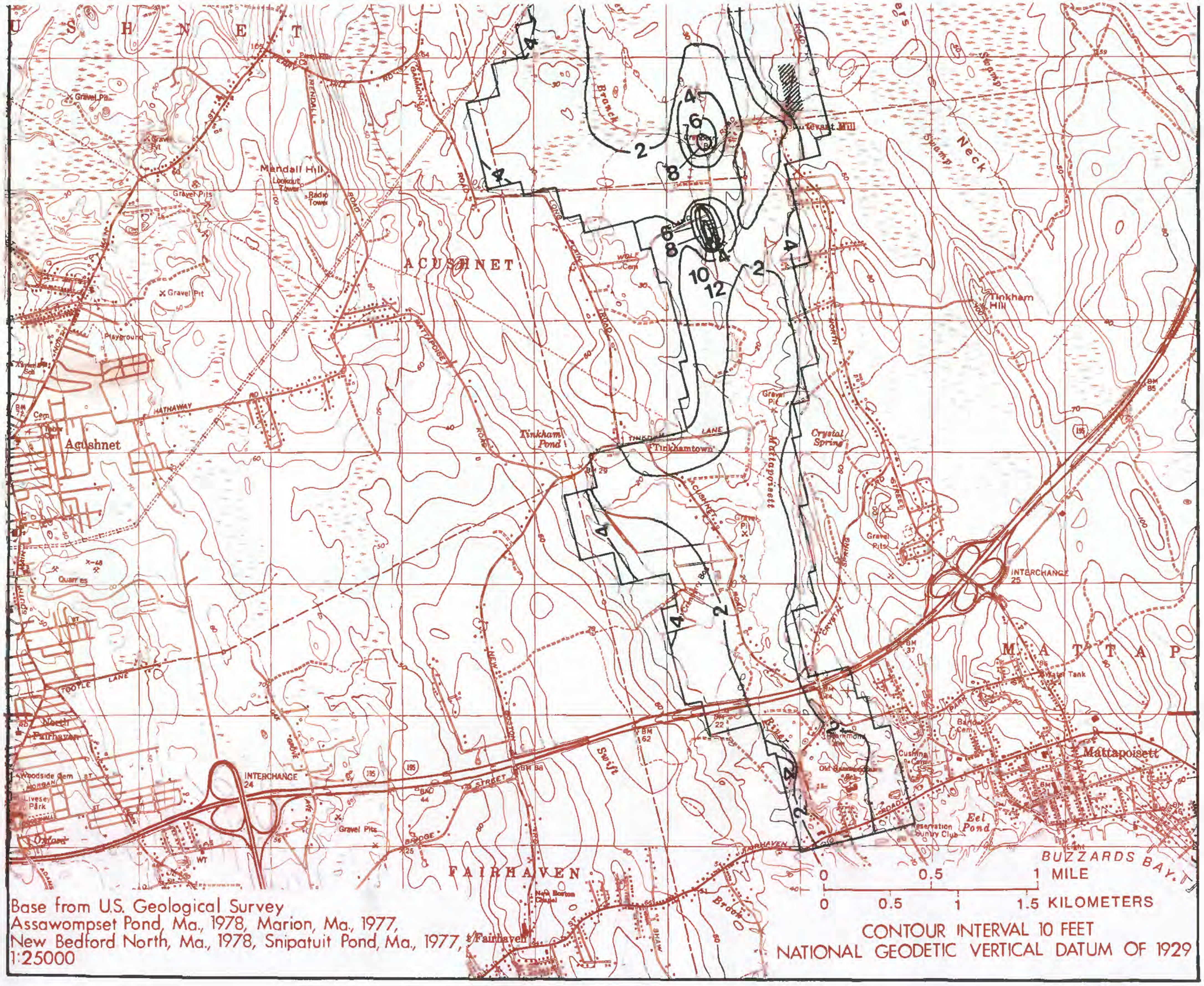
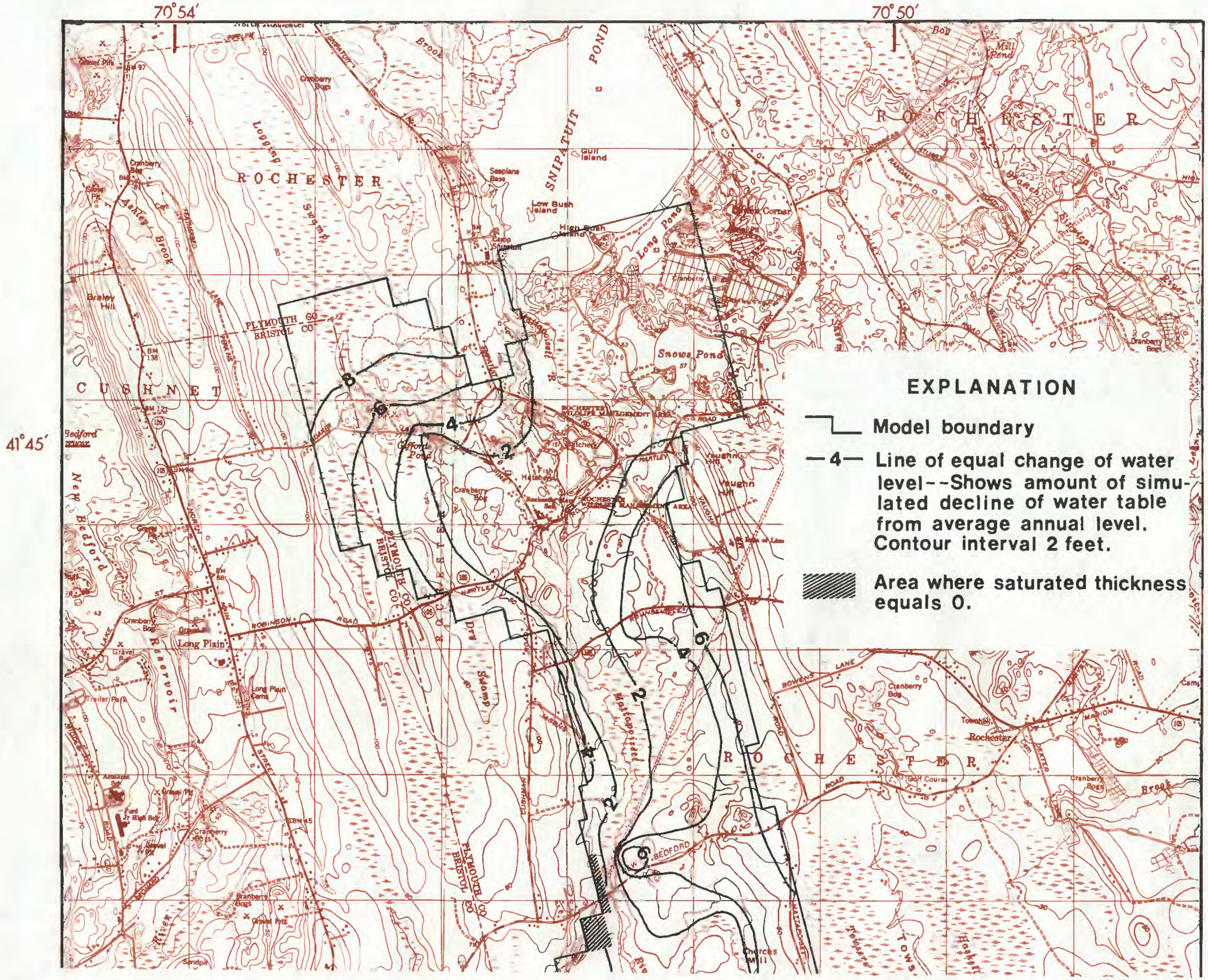


Figure 19.-- Simulated decline in the altitude of the water table: Scenario 5, dry conditions.



70°54′
70°50′
41°45′
EXPLANATION
Model boundary
—4— Line of equal change of water level--Shows amount of simulated decline of water table from average annual level. Contour interval 2 feet.
Area where saturated thickness equals 0.
ROCHESTER
CUSHNET
SNIPATUIT POND
Long Pond
Snows Pond
Logging Swamp
Dry Swamp
Mattapoisett
Long Plain
Reservoir
Ashley Brook
New Bedford
PLYMOUTH CO
BRISTOL CO
Gull Island
Low Bush Island
High Bush Island
Seaplane Base
Cranberry Bogs
Rochester

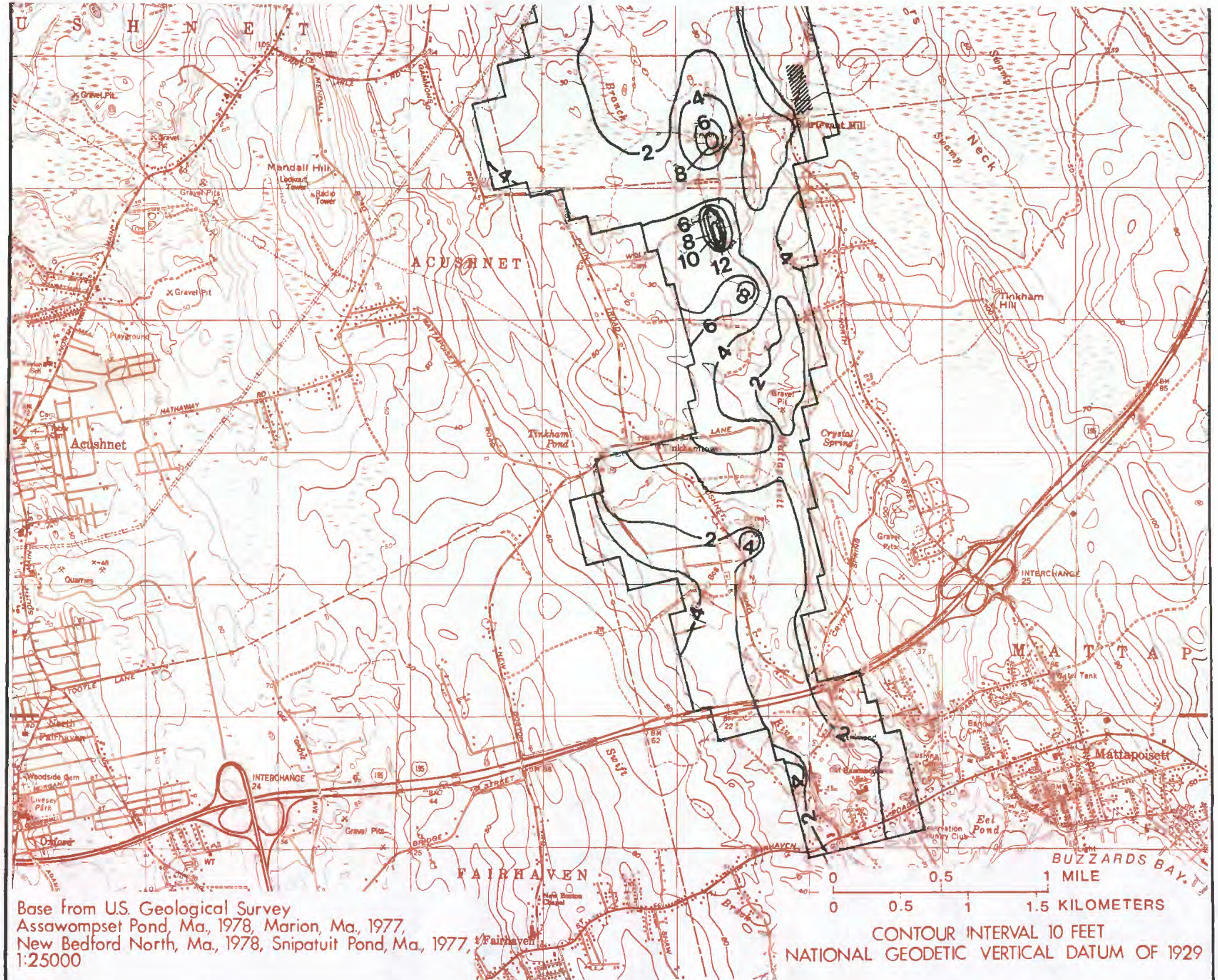


Figure 20.--Simulated decline in the altitude of the water table: Scenario 10, dry conditions.

Severely Dry Conditions

Table 10 shows the combined effects of both reduced recharge under severely dry conditions and projected withdrawal. Given the amounts of well drawdown and ground-water discharge previously simulated under dry conditions, the relatively large drawdowns and small streamflows that were predicted under severely dry conditions were expected.

Over the central part of the model area, the predicted decline of water levels under severely dry conditions is approximately 1 foot more than that simulated under dry conditions. In Scenarios 3 and 6, the estimated available drawdowns are exceeded in Mattapoisett well 4 and in the Marion and Fairhaven wells located on Wolf Island Road. The model results further show that withdrawals of Scenarios 3, 6, 7, 8, 9, and 10 intercept all the ground-water discharge from the aquifer. Table 10 also shows the amount of potentially available streamflow that is necessary to make up the deficit and to satisfy the demands of the pumping wells.

The predicted drawdowns of the current and proposed wells are conservative because a positive rate of leakage from the stream to the aquifer is maintained by the model. Although a very low stream stage representative of drought conditions is accurately simulated, it is assumed that this streamflow is available from a source such as Snipatuit Pond. However, should streamflow not be available under real conditions, there will be no leakage through the streambed, and the pumping demands will not be fully satisfied. In this event, the predicted drawdowns in table 10 will be too small. Real drawdowns will be greater than those in the table and will exceed available limits in most cases.

The combined effects of severely dry recharge conditions and the projected withdrawals of Scenarios 2, 5, and 10 are illustrated in figures 21 to 23. The impact of reduced recharge is greatest along the western model boundary near Cushman Road, along the eastern model boundary near Vaughn Hill Road, and in the northeast model area near Giffords Pond where simulated head declines are as much as 19 feet, and the relatively thin, border areas of the aquifer are almost completely desaturated. Numerical model problems were encountered during model runs due to the model nodes which went dry during the iterative solution process.

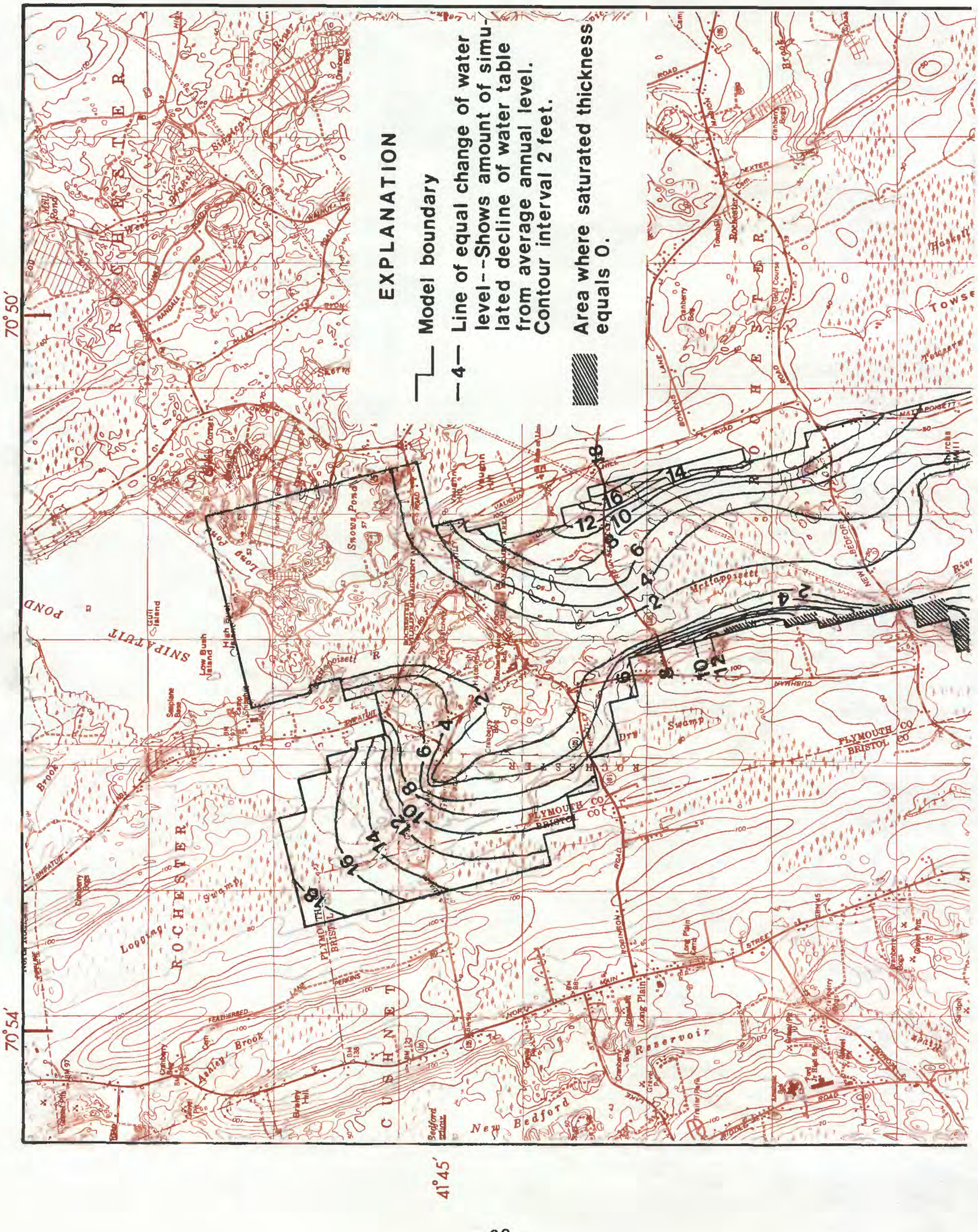
Cones of depression for the pumping wells are broader and deeper than those predicted under dry conditions. The deepest depressions surround the Fairhaven wells at Wolf Island Road (Scenario 5, fig. 22; Scenario 10, fig. 23) in which predicted drawdowns reach to within a few feet of the estimated available drawdown. The average simulated head decline in the Wolf Island Road area exceeds 5 feet in most places. Of further note is that the cone of depression of the Marion well on Wolf Island Road is shifted northward, and that of the proposed Mattapoisett well 11-6 is shifted southward. These shifts occur because the specific capacities of the wells finished in the confined part of the aquifer are not sufficient to satisfy the pumping demands of the wells, and water is obtained from the unconfined aquifer nearby.

Table 10.--Steady-state model results--severely dry conditions: Computed well drawdown and total ground-water discharge

(Drawdown, in feet; pumpage rates and ground-water discharge, in cubic feet per second. Values enclosed in parentheses indicates that the predicted drawdown exceeds the estimated available drawdown.)

Well	Available drawdown[1]	Drawdown from current pumpage, 1982			Drawdown from current and proposed pumpage, 1982-90			Drawdown from current and proposed pumpage 1990+			
		Summer average	High month	Maximum daily	Summer average	High month	Maximum daily	High month	High month	High month	High month
Scenario number		1	2	3	4	5	6	7	8	9	10
Fairhaven:											
River Road[2]	[3]30.0	5.80	6.70	7.70	5.80	6.70	7.20	6.70	6.70	6.70	6.70
Mattapoisett 2[2]	[3]21.5	.00	.00	3.80	.00	.00	3.80	.00	.00	.00	.00
Mattapoisett 3	43.0	4.13	4.22	25.40	4.13	4.22	23.40	4.22	4.22	4.51	4.73
Mattapoisett 4	[4]30.0	8.95	10.80	(42.67)	8.95	10.80	(42.67)	10.80	10.80	12.63	12.77
Marion:											
Wolf Island Road	40.0	15.47	21.19	39.93	15.56	21.32	(40.33)	21.32	21.32	21.32	21.32
Fairhaven 20-79	43.2	--	--	--	28.10	35.26	(49.55)	35.47	35.47	35.47	35.47
Fairhaven 8-79	41.7	--	--	--	29.44	36.94	(51.87)	37.01	37.01	37.01	37.01
Fairhaven 11-81	33.0	--	--	--	23.74	31.19	(44.09)	31.21	31.21	31.21	31.21
Mattapoisett 11-6	44.0	--	--	--	--	--	--	19.51	19.51	19.51	19.51
Marion:											
New Bedford Road	52.0	--	--	--	--	--	--	--	31.74	31.74	31.74
Fairhaven:											
Tinkham Lane	95.0	--	--	--	--	--	--	--	--	11.84	11.87
Mattapoisett 11-2	[5]33.0	--	--	--	--	--	--	--	--	--	21.35
Total withdrawal from aquifer[6]		1.89	2.28	5.37	2.83	3.46	7.01	4.23	4.83	5.60	6.37
Total ground-water discharge of the Mattapoisett River[6,7]		1.88	1.48	-1.61	0.93	0.30	-3.25	-0.47	-1.07	-1.84	-2.61

[1] Depth from average level of the water table to 5 feet above the top of the current/proposed well screen; interference effects of Marion, Fairhaven, and Mattapoisett wells located in the Wolf Island Road area included. Estimated available drawdown values are obtained from reports of aquifer tests conducted by town consultants.
[2] Drawdown in the node representing a well field composed of numerous 2-inch diameter wells.
[3] Estimated average available drawdown of well field.
[4] Upper screen estimate; drawdown does not include interference from Fairhaven Tinkham Lane well.
[5] Estimated; no field data.
[6] Under "no pumping" conditions, total ground-water discharge of Mattapoisett River is 3.76 cubic feet per second.
[7] Negative values indicate amount of additional streamflow that would be necessary to satisfy the demands of the pumping wells.


EXPLANATION
Model boundary
—4— Line of equal change of water level--Shows amount of simulated decline of water table from average annual level. Contour interval 2 feet.
Area where saturated thickness equals 0.
70°50′
70°54′
41°45′

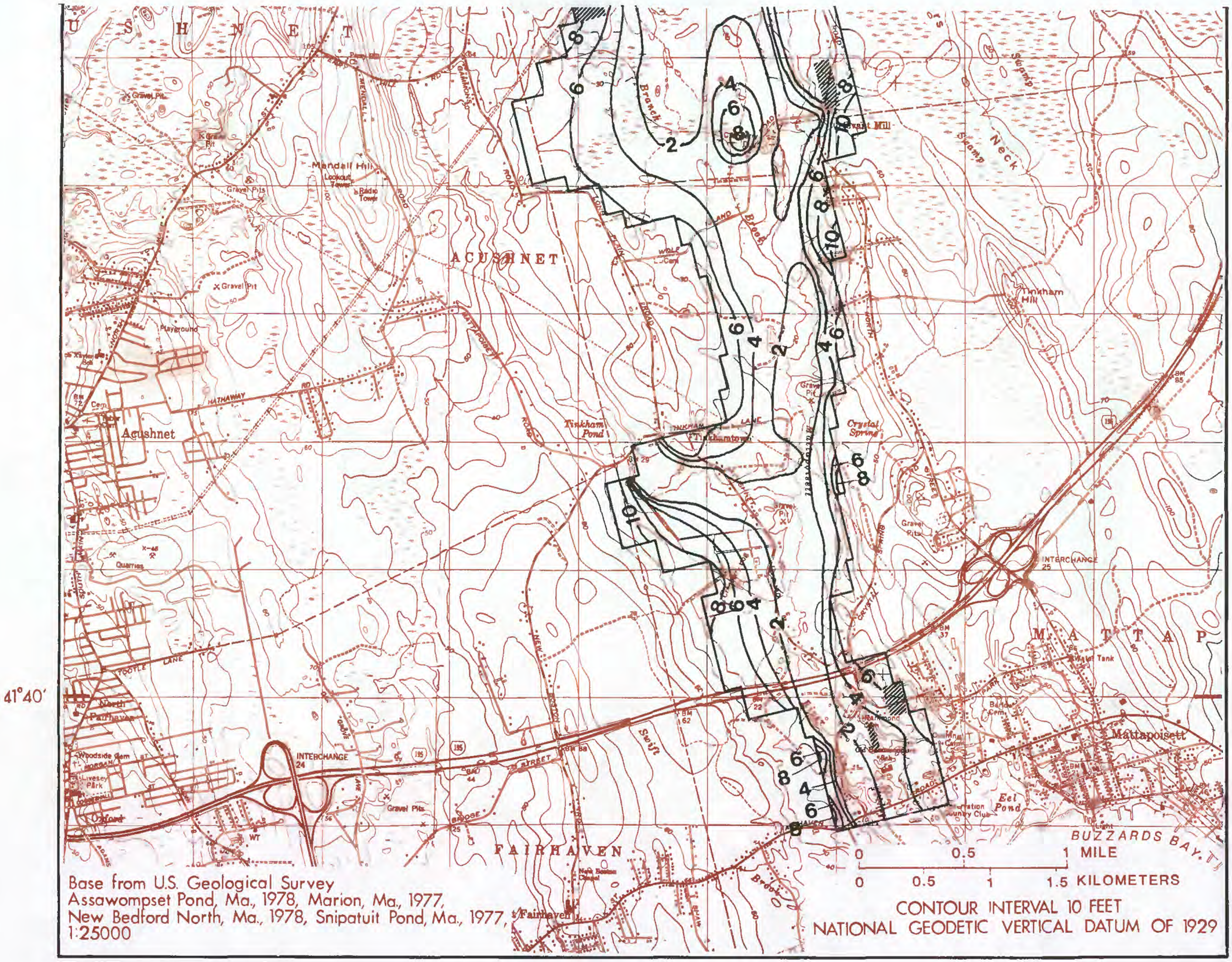
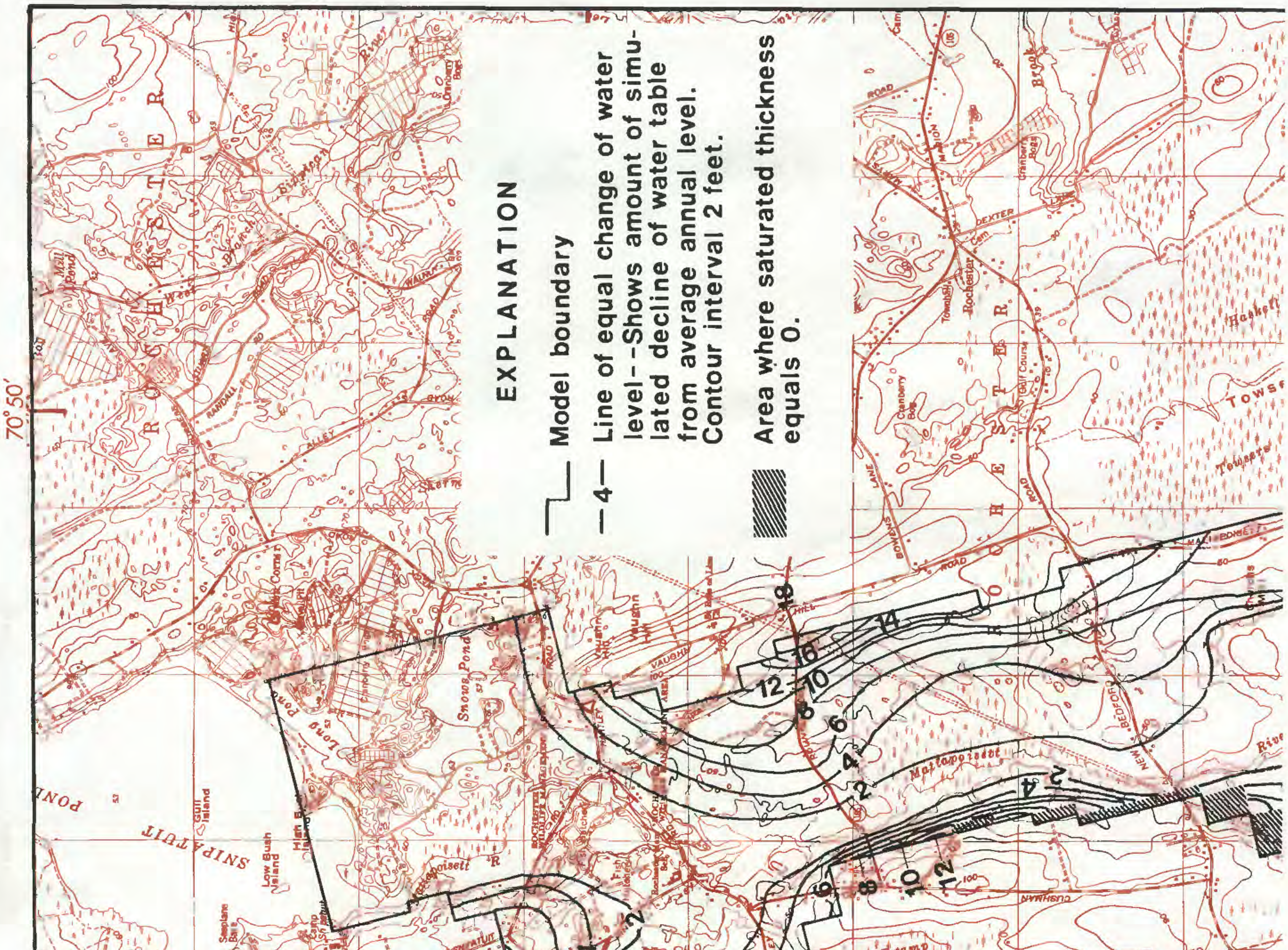


Figure 21.--Simulated decline in the altitude of the water table: Scenario 2, severely dry conditions.


70°50′
70°54′
41°45′
EXPLANATION
Model boundary
—4— Line of equal change of water level--Shows amount of simulated decline of water table from average annual level. Contour interval 2 feet.
Area where saturated thickness equals 0.

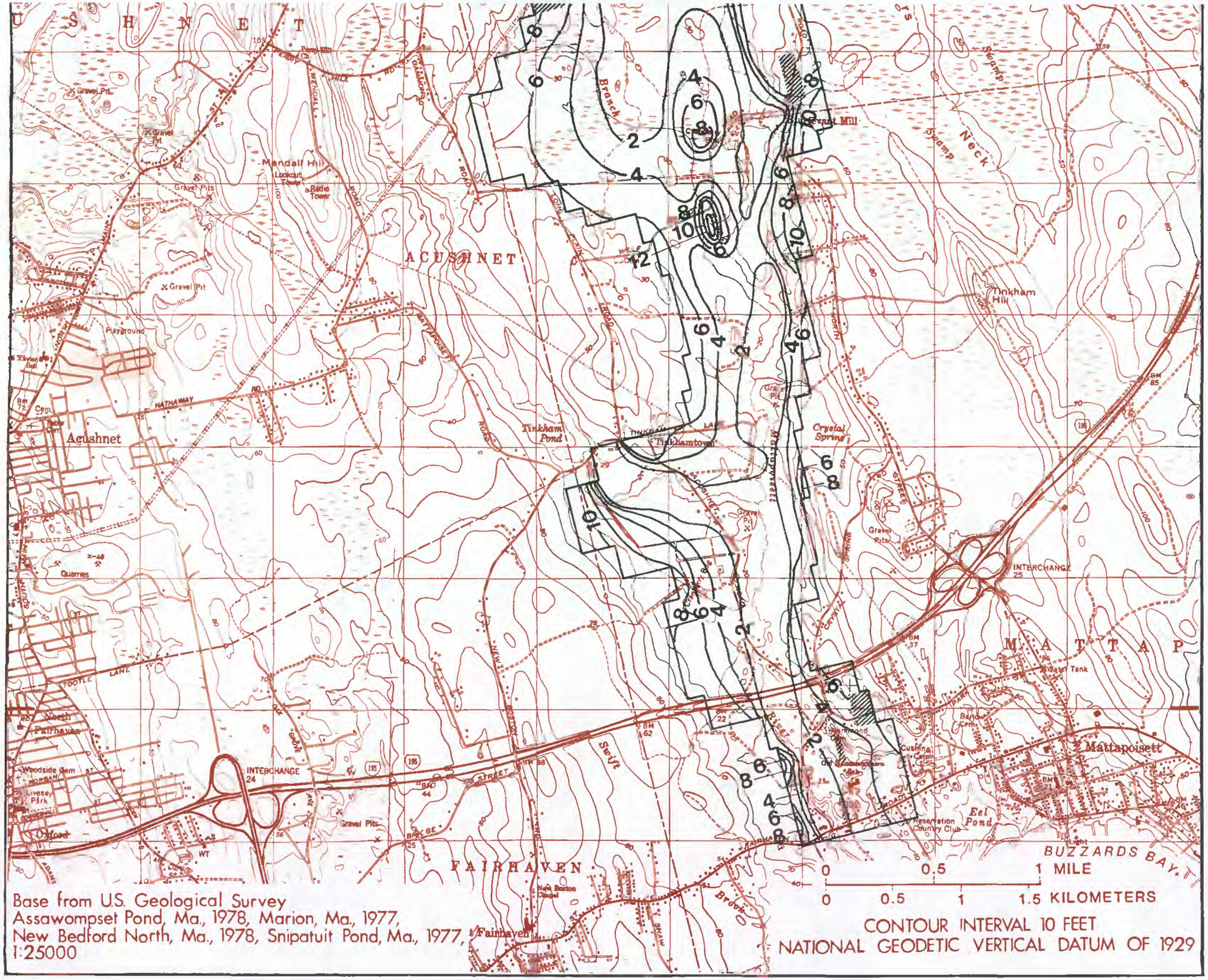
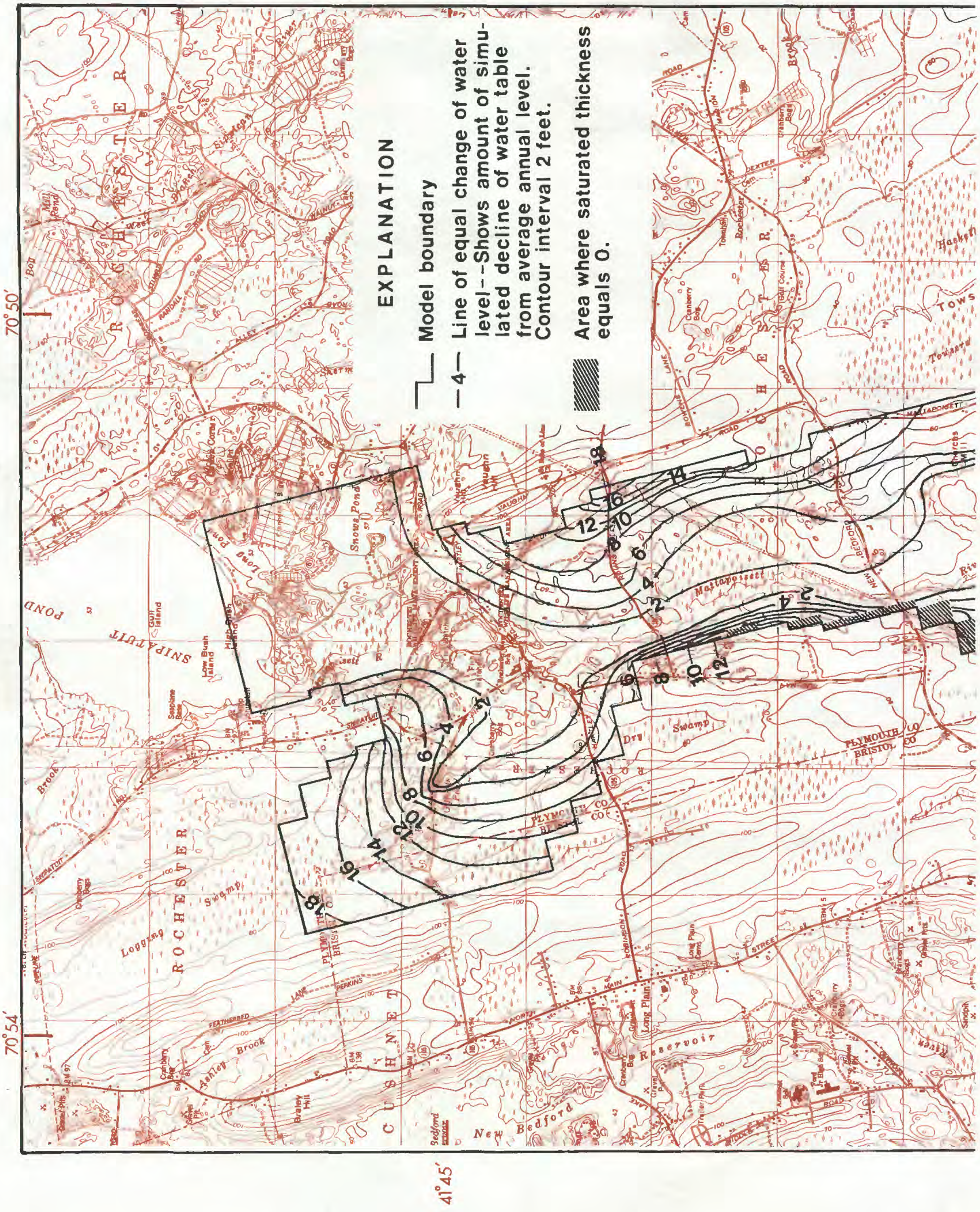


Figure 22.--Simulated decline in the altitude of the water table: Scenario 5, severely dry conditions.


EXPLANATION
Model boundary
—4— Line of equal change of water level--Shows amount of simulated decline of water table from average annual level. Contour interval 2 feet.
Area where saturated thickness equals 0.
70°50'
70°54'
41°45'
ROCHESTER
SNIPATUIT POND
Long Pond
Snows Pond
Mattapoisett
Logging Swamp
Reservoir
New Bedford
PLYMOUTH CO
BRISTOL CO

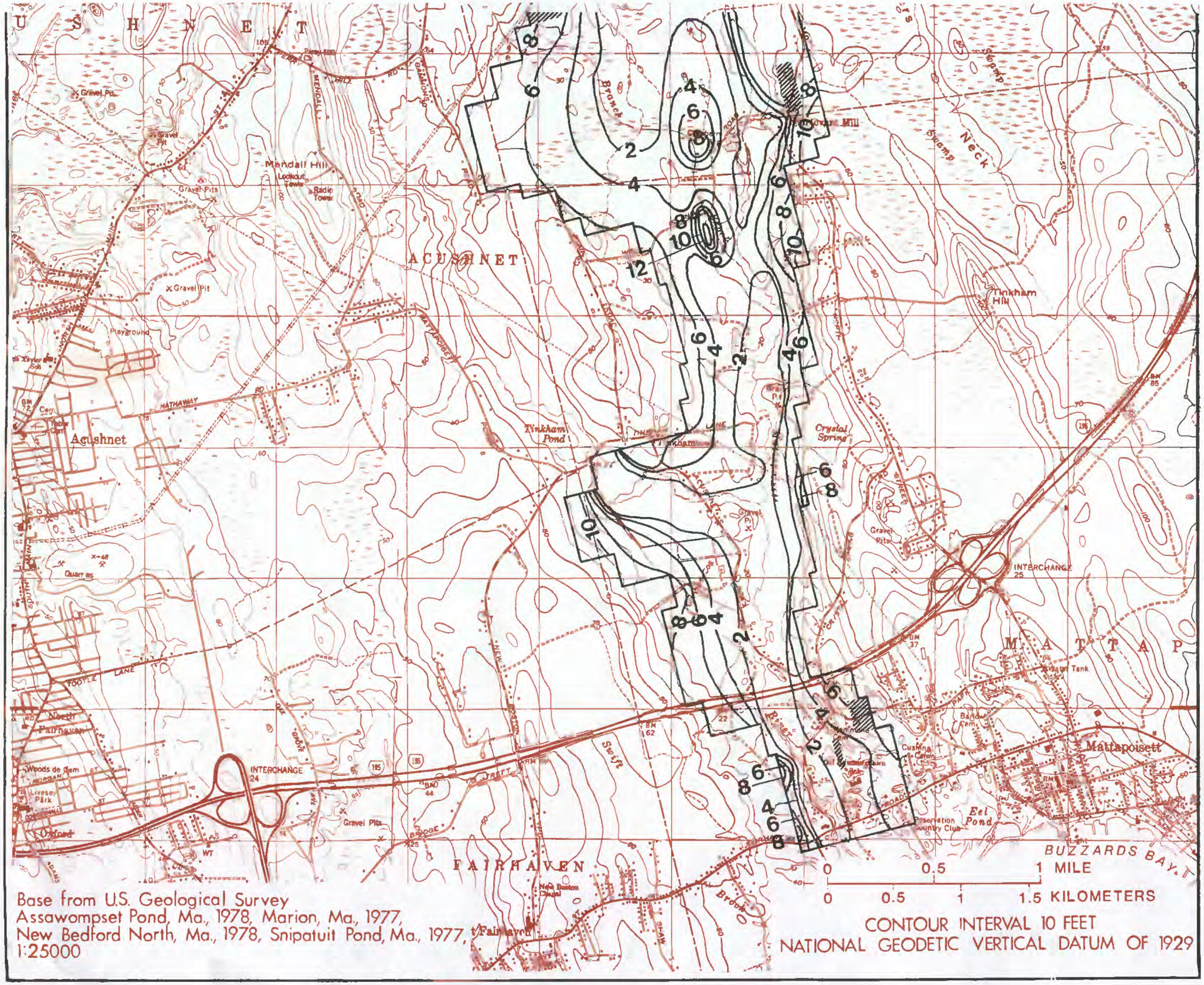


Figure 23.--Simulated decline in the altitude of the water table: Scenario 10, severely dry conditions.

Effects of Pumping on Streamflow

An important objective of this study is to describe the impact of pumping on streamflow in the Mattapoisett River. Specifically, what portion of withdrawal from wells is intercepted ground-water discharge from the aquifer and what portion is induced infiltration from the river? With induced infiltration occurring, what is the remaining streamflow? Answers to these questions are demonstrated by the results of Scenarios 2, 5, and 10, listed in table 11.

The streamflow and pumping data for 10 river locations are listed in downstream order, from left to right (see also fig. 10). With the exception of the Mattapoisett River-Branch Brook junction, the locations are adjacent to municipal well sites. The Marion and Fairhaven wells on Wolf Island Road are located adjacent to Branch Brook; therefore, the impact of these wells on the flow of the Mattapoisett River is assessed at the junction downstream. For comparison, streamflows at each location are listed under both the dry and severely dry recharge conditions.

Table 11 shows the amount of induced infiltration that takes place at each well site and how infiltration affects streamflow downstream of the site. The results of Scenario 10 under dry conditions are described in detail below to explain the impacts in the southern half of the valley as the river flows by the wells, and to provide one example of how the data listed in table 11 may be interpreted.

The 85-percent duration of the flow of the Mattapoisett River at the midpoint of the valley, where withdrawals from wells begin, is 3.69 ft^3/s. Withdrawal by the proposed Marion well at this location is 0.60 ft^3/s. The model results indicate 0.27 ft^3/s would be derived from induced infiltration and 0.33 ft^3/s from intercepted ground-water discharge, leaving 3.42 ft^3/s in the river. Downstream at the Mattapoisett River-Branch Brook junction, streamflow increases to 4.26 ft^3/s. The combined withdrawal of the Marion and Fairhaven wells is 1.99 ft^3/s of which 1.49 ft^3/s is derived from induced infiltration along both Branch Brook and the Mattapoisett River and 0.50 ft^3/s is from intercepted ground-water discharge. As a result, streamflow is reduced to 2.77 ft^3/s. Simulated withdrawal at the next well site, the proposed Mattapoisett well 11-6, is 0.77 ft^3/s. The model results indicate that a relatively small amount of induced infiltration occurs because the well site is located several hundred feet from the river. Proceeding downstream, the river arrives at Tinkham Lane with a streamflow of 2.70 ft^3/s.

The river reach between Tinkham Lane and Acushnet Road Bridge is dotted with four major current and proposed municipal wells that are located along the river bank. The model results show that the source of most of the pumped water from these wells is the river. The amount of withdrawal derived from induced infiltration ranges from 50 percent at Mattapoisett well 4 (0.68 ft^3/s withdrawn; 0.35 ft^3/s induced) to over 90 percent at Mattapoisett well 3 (0.23 ft^3/s withdrawn; 0.21 ft^3/s induced). The remaining streamflow in the river below the Mattapoisett wells is 1.11 ft^3/s. Moving southward, streamflow increases to 1.64 ft^3/s before reaching the last set of wells next to the river, the Fairhaven well field. The simulated withdrawal of the northernmost section of the well field is 0.37 ft^3/s, and that of the southernmost section of the well field is 0.19 ft^3/s. According to the model results, approximately one half of the withdrawal from these wells is derived from induced infiltration. Consequently, the remaining streamflow in the river at its outfall to the ocean is 1.38 ft^3/s.

The model simulations of severely dry conditions are particularly important because the amount of streamflow available for induced infiltration is reduced to zero by the municipal withdrawals in several of the scenarios. Using Scenario 10 under severely dry conditions, for example (table 11), the model results show that by the time the river arrives at the Mattapoisett River-Branch Brook junction, streamflow is only 1.87 ft^3/s. Combined withdrawal of the Marion and Fairhaven wells induces 1.81 ft^3/s from both Branch Brook and Mattapoisett River which is nearly the entire amount of available streamflow in the river. At the proposed Mattapoisett well 11-6, the river stops flowing, and no streamflow is available for induced infiltration. Proceeding downstream, the amount of remaining ground-water discharge from the aquifer is not enough to satisfy the withdrawals from the other wells and re-establish flow in the river.

Table 11.--Steady-state model results--induced infiltration and intercepted ground-water discharge by wells adjacent to the Mattapoisett River

(Withdrawal rates and streamflow, in cubic feet per second)

	Marion test site New Bedford Road	Junction with Branch Brook	Matta-poisett test site 11-6	Fairhaven test site Tinkham Lane	Mattapoisett wells 4	Mattapoisett wells 3	Mattapoisett wells 11-2	Mattapoisett wells 2	Fairhaven River Road (North well field)	Fairhaven River Road (South well field)
DRY CONDITIONS (85 PERCENT FLOW DURATION)										
Scenario 2										
Streamflow upstream of well field	3.69	5.41	4.87	5.30	5.30	5.01	4.91	5.43	5.75	5.57
Total withdrawal	--	.81	--	--	.68	.23	--	--	.37	.19
Withdrawal derived from induced infiltration	--	.57	--	--	.29	.13	--	--	.18	.09
Withdrawal derived from intercepted ground-water discharge	--	.24	--	--	.39	.10	--	--	.19	.10
Streamflow remaining downstream of well field	3.69	4.84	4.87	5.30	5.01	4.88	4.94	5.43	5.57	5.48
Scenario 5										
Streamflow upstream of well field	3.69	4.88	3.66	4.10	4.10	3.81	3.74	4.23	4.55	4.38
Total withdrawal	--	1.99	--	--	.68	.23	--	--	.37	.19
Withdrawal derived from induced infiltration	--	1.24	--	--	.29	.13	--	--	.17	.09
Withdrawal derived from intercepted ground-water discharge	--	.75	--	--	.39	.10	--	--	.20	.10
Streamflow remaining downstream of well field	3.69	3.64	3.66	4.10	3.81	3.68	3.74	4.23	4.38	4.29
Scenario 10										
Streamflow upstream of well field	3.69	4.26	2.77	2.70	2.10	1.75	1.54	1.32	1.64	1.47
Total withdrawal	.60	1.99	.77	.77	.68	.23	.77	--	.37	.19
Withdrawal derived from induced infiltration	.27	1.49	.09	.60	.35	.21	.43	--	.17	.09
Withdrawal derived from intercepted ground-water discharge	.33	.50	.68	.17	.33	.02	.34	--	.20	.10
Streamflow remaining downstream of well field	3.42	2.77	2.68	2.10	1.75	1.54	1.11	1.32	1.47	1.38

Table 11.--Steady-state model results--induced infiltration and intercepted ground-water discharge by wells adjacent to the Mattapoisett River (continued)

	Marion test site New Bedford Road	Junction with Branch Brook	Matta-poisett test site 11-6	Fairhaven test site Tinkham Lane	Mattapoisett wells				Fairhaven River Road	
					4	3	11-2	2	(North well field)	(South well field)
SEVERELY DRY CONDITIONS (99 PERCENT FLOW DURATION)										
Scenario 2										
Streamflow upstream of well field	1.97	2.66	2.11	2.20	2.20	1.85	1.67	1.83	1.88	1.61
Total withdrawal	--	.81	--	--	.68	.23	--	--	.37	.19
Withdrawal derived from induced infiltration	--	.57	--	--	.35	.18	[1].01	[1].09	.27	.13
Withdrawal derived from intercepted ground-water discharge	--	.24	--	--	.33	.05	--	--	.10	.06
Streamflow remaining downstream of well field	1.97	2.09	2.11	2.20	1.85	1.67	1.66	1.74	1.61	1.48
Scenario 5										
Streamflow upstream of well field	1.97	2.48	0.94	1.02	1.02	0.67	0.49	0.65	0.70	0.42
Total withdrawal	--	1.99	--	--	.68	.23	--	--	.37	.19
Withdrawal derived from induced infiltration	--	1.55	--	--	.35	.18	[1].01	[1].09	.28	.12
Withdrawal derived from intercepted ground-water discharge	--	.44	--	--	.33	.05	--	--	.09	.07
Streamflow remaining downstream of well field	1.97	.93	.94	1.02	.67	.49	.48	.56	.42	.30
Scenario 10 (negative values indicate streamflow deficit)										
Streamflow upstream of well field	1.97	1.87	0.06	-0.31	-1.00	-1.35	-1.63	-2.07	-2.21	-2.48
Total withdrawal	.60	1.99	.77	.77	.68	.23	.77	--	.37	.19
Withdrawal derived from induced infiltration	.27	1.81	.06	.00	.00	.00	.00	--	.00	.00
Withdrawal derived from intercepted ground-water discharge	.33	.18	.68	.08	.33	--	.26	--	.10	.06
Streamflow remaining downstream of well field	1.70	0.06	-0.03	-1.00	-1.35	-1.63	-2.14	-2.35	-2.48	-2.61

[1]Natural leakage from stream.

For Scenario 10, table 11 also includes the amounts of deficit streamflow necessary to maintain the simulated withdrawals in the remaining downstream wells. The deficit values are minimum streamflows: if the deficit at a well site is made up by some other source of water, the withdrawal of that well site will be fully satisfied. However, streamflow will still be zero and only an amount of made-up water greater than the deficit will result in positive streamflow. If the amount of streamflow upstream of the current and proposed wells is reduced for any reason, streamflow deficits indicated in table 11 will increase by a proportional amount. If, under real conditions, the streamflow deficits are not made up by another source of water, withdrawals will have to be reduced to decrease the amount of induced infiltration.

To summarize the predicted streamflow of the Mattapoisett River under dry and severely dry conditions, the streamflows according to Scenarios 2, 5, and 10 are illustrated as discharge profiles in figure 24. The discharge profiles show predicted streamflows along the entire length of the river for both drought conditions, given the withdrawals of each scenario. Also shown, for comparison, is the streamflow of the river under no-pumping conditions. By way of brief review, the model conditions include high-month withdrawal rates and no surface-water inflows.

From a water management point of view, the discharge profile under no-pumping conditions may be considered a "best case" alternative because it represents the total amount of available ground-water discharge from the Mattapoisett River valley aquifer. Similarly, the discharge profile under Scenario 10 pumping conditions may be considered a "worst case" alternative in that it represents the available streamflow that remains after all the current and proposed wells have removed water from the river and from the aquifer. Any proposed distributed pumping plan for water management will cause impacts on streamflow that fall between the best case and worst case alternatives if all the assumptions upon which these scenarios are based remain valid. If, on the other hand, one or more of the assumptions are invalidated, for example, higher withdrawals (such as increased withdrawal rates and more wells), then withdrawals will cause greater impacts than those illustrated in figure 24.

Appraisal of Model Results

The model simulations are based on available information about current and proposed municipal withdrawals. The pumping scenarios (table 8) were not designed to conform with any water-level or streamflow criteria. The results provide new information for those who wish to evaluate their own comprehensive water resources management plans for the region and provide an example for those who wish to use the model to test their own pumping plans. In view of the large amount of information given in this report, a brief appraisal may help to clarify the results and limitations of the model.

The modeling work represents an initial effort to integrate all the geologic and hydrologic characteristics of the aquifer that affect water levels and streamflow and to determine the net effects of municipal withdrawals on a regional scale. Precise simulation of well interference on a small scale was not an objective of the modeling work; consequently, the model was designed to assess regional rather than local processes.

The withdrawal rates simulated in the pumping scenarios cover an extensive range of well development and give a relatively complete picture of the proposed withdrawal of water from the stream-aquifer system in the study area. For the purposes of this study, these scenarios were used to: (1) Explain the operation of the digital model, and (2) provide illustrative examples of the cause-and-effect relations of municipal withdrawal, streamflow, and hydraulic head.

The real condition which the model was adjusted to simulate was a long drought (2 to 3 years) in which water levels, ground-water discharge, and streamflow are much less than average, but are assumed to be at steady-state. The pumping scenarios were run under steady-state conditions to allow direct comparisons of the pumping impacts. Proper interpretation of the results of this modeling approach requires knowledge of the response time of the aquifer, and the transient model runs of this study show that real recharge and withdrawals would have to operate continuously for at least 1 year to realize the model predictions. An alternate modeling approach would have been to simulate the pumping scenarios in a series of transient model runs in which recharge and pumping stresses are simulated over a time period

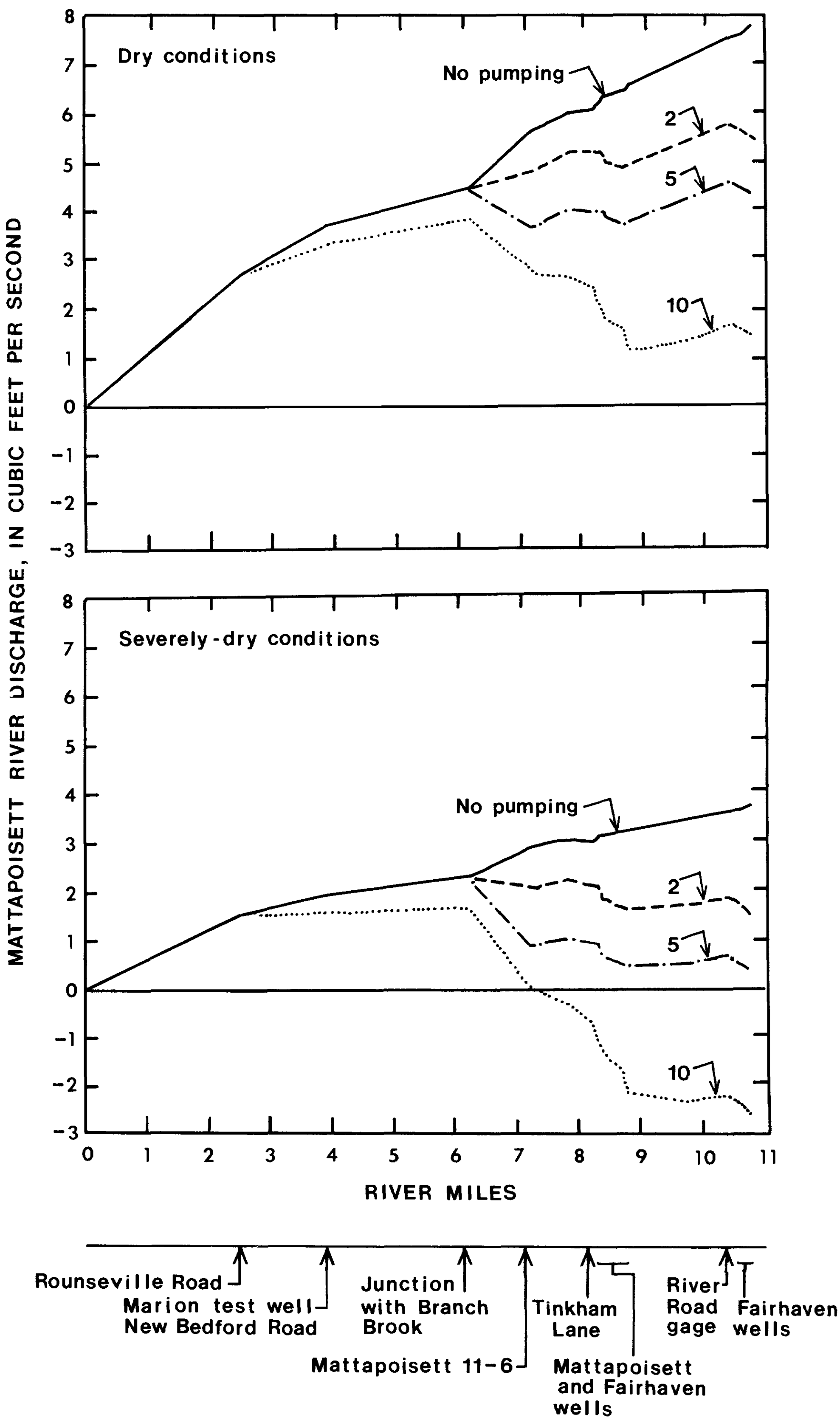


Figure 24.--Predicted discharge profiles of the Mattapoisett River: dry and severely dry conditions.

beginning at a known point (May 1982 average conditions) and ending at some point representative of steady-state (drought) conditions. This approach is an appropriate and often preferred modeling method; however, it was not followed in this study because of the difficulty of clearly separating impacts due to pumping from impacts due to the change in recharge. Regardless of the approach, it should be emphasized also that starting conditions for drought simulations were May 1982 average annual conditions and these conditions may not be completely representative of worst-case situations. A real drought may be preceded by a year or more of relatively dry conditions which makes starting conditions worse, as was the case in 1965.

The potential model user should remain aware of other limitations of the current model. For example, values for aquifer hydraulic conductivity at model nodes were adjusted during steady-state calibration, and a good match between computed and observed hydraulic heads was obtained in most, but not all, of the model area. A poor match was obtained in some areas along the model border, partly because these are areas in which there often is significant vertical flow which the model cannot simulate, and partly because these areas are in corners of the active model which cause unique computational problems. Further attempts to refine the calibration in these areas may not significantly improve the model results.

The storage coefficient values were adjusted slightly during transient model calibration, and only constant values for the aquifer and confining bed were used. No attempt was made to assign and adjust individual storage values to the nodes near wells to improve the match between calculated and observed drawdowns (table 7). Although the calculated and observed drawdowns match very well indicating accurate calibration of hydraulic conductivity and storage coefficient values, the match only indicates that this combination of values produces an acceptable answer, not necessarily the correct answer. A prerequisite of future transient modeling analysis is the addition to the model of distributed values for aquifer specific yield and storage and a sensitivity test of those values.

In cases where there is little ground-water discharge from the aquifer to the stream, the stream effectively goes dry. A major limitation of the model arises when pumping wells near the "dry" stream are simulated. The user may adjust the stage of stream in the current model to a level which accurately represents either low flow or no flow. In actuality, no streamflow is available for induced infiltration by the wells, and leakage through the streambed is zero. However, the model does not account for a dry streambed and maintains a positive rate of leakage, based on the assigned stage of the river. As a result of this leakage, simulated hydraulic head in the aquifer and drawdowns in the wells will be incorrect; real aquifer heads will be lower, and well drawdowns will be greater than the predicted values. The impact of this limitation on model results will be greatest in situations where wells with shallow screens are located next to a stream and a major portion of the yield of the well is derived from induced infiltration. Correction for this model limitation requires revision of the computer program to stop leakage through the streambed when calculated heads fall below the bottom of the bed.

Estimates of ground-water flow from the till to the stratified drift should be refined when baseflow data from till-covered basins in the region become available. Under severely dry conditions, the flux rate across the till/stratified-drift boundary may be greater than the zero rate used in model simulations. Many sections of the model boundary, particularly along the east and northwest sides of the study area, are in wetland areas. In reality, the fluxes across these boundaries are head-dependent in the sense that the rate of flux across the boundary depends on the average head in the wetland on the other side of the boundary. Potential model users may also want to add this model capability to the computer program.

Experience has shown that pumping impacts at current and proposed wells do not extend laterally to the till/stratified-drift boundary. For wells finished in the unconfined parts of the aquifer, drawdowns are generally small (approximately 15 feet), and the influence of the wells extends hundreds, not thousands, of feet. In almost every case, the yields of the wells are related to local factors, such as the hydraulic conductivity of both the aquifer and the nearby streambed. For wells finished in the confined part of the aquifer near Wolf Island Road, drawdowns tend to be large (approximately 40 feet) and the influence of the well may extend thousands of feet. Some evidence for an influence of this magnitude was gathered during this study when the on-off cycle of a pumping well was clearly recorded by water-level-observation equipment attached to a well located 3,600 feet away. If new wells are added to future model simulations, care should be taken not to place the wells near the border of the active model area where their influence may impinge on the model boundary. Simulated drawdowns in wells near the boundaries will be inaccurate, reflecting the limitations of the constant-flux boundary conditions and the level of model calibration in that area.

Finally, the post-1990 simulated withdrawal rates are rough estimates at best. None of the wells proposed by the towns have been constructed, and only estimates of proposed town water needs are currently available. Nevertheless, the model results can be used to obtain a sense of the magnitude of changes in water levels and streamflow that could be expected on a regional basis if the proposed withdrawals take place. The impacts of Rochester water supply needs will also have to be analyzed in future model simulations if a municipal well system is constructed.

SUMMARY

An area of 8 mi^2 in the Mattapoisett River drainage basin, Plymouth County, Massachusetts, was modeled to simulate changes from 1982 to 1990+ in water levels and streamflow due to proposed increases in municipal well withdrawals. The ground-water-flow model was constructed and calibrated with hydrologic data gathered during this study and data from previous Survey and private studies.

Ground water in the study area occurs in stratified-drift sediments which form a high-water-yielding, unconfined aquifer in the Mattapoisett River valley. The unconfined aquifer is composed mostly of sand and gravel, is underlain by bedrock, and is bordered by till. Ground water derived from infiltration of precipitation to the aquifer and from leakage from adjacent till moves through the aquifer and discharges to the Mattapoisett River.

Thickness of the sand and gravel sediments ranges from zero at the till/stratified-drift boundary to over 110 feet at one location in the center of the valley. The altitude of the water table ranges from near zero at the seacoast at the south end of the valley to over 90 feet in the upland areas at the north end of the valley. The average seasonal water-table fluctuation is 3 to 5 feet. Aquifer thickness generally ranges from 25 to 75 feet and hydraulic conductivity ranges from about 2 to 350 ft/d. In the center of the valley, a small confining layer composed of clay and silt is interbedded with the stratified drift and separates the aquifer into an upper unconfined part and a lower confined part.

Total recharge to the aquifer simulated in the model, based on 1982 water year data, was 15.9 inches. Leakage from till was approximately 6.8 inches and was determined by applying a rate of 0.5 $(ft^3/s)/mi^2$ of contributing drainage area. Total baseflow of Mattapoisett River, as determined during five measurements in 1982, ranged from 5.4 to 43.4 ft^3/s. Withdrawal rate from wells in May 1982 was 1.80 ft^3/s.

Water quality at seven well sites and at three stream-gage sites is soft, slightly acidic, and low in dissolved solids. High iron and manganese concentrations are common, and some contamination by salt was detected at one roadside well. The water samples collected in 1981-82 also were analyzed for 55 insecticide, pesticide, and volatile organic compounds. Some solvents, herbicides, and insecticides were detected in a few wells.

A digital model of two-dimensional ground-water flow was used to compute water-table elevations, rates of induced stream infiltration, and rates of ground-water discharge to the Mattapoisett River. The till/stratified-drift boundary was modeled as a constant-flux boundary, and the Mattapoisett River and its tributaries were modeled as leaky boundaries. The active model area consists of 1,068 nodes and includes a 56-node confining bed. A parameter estimation procedure, which was used to refine the model, was based on data from May 1982 steady-state hydrologic conditions and from transient conditions that existed during six short-term aquifer tests conducted at the major municipal well sites.

The difference between computed and observed May 1982 water levels was less than 4 feet at 80 percent of the model nodes; the maximum difference was 10.5 feet. A poorer fit between computed and observed values was accepted in some model areas, particularly in areas adjacent to the boundaries, because of the sparsity of the observed data, the coarseness of the grid, and the model limitations concerning horizontal flow. The transient simulations of the aquifer tests showed that the model is capable of computing well drawdowns accurately over time.

A sensitivity analysis of the model indicated that the departure between computed and observed heads could be reduced by increasing and decreasing values of both aquifer and streambed hydraulic conductivity. Further adjustments do not significantly improve the model. The analysis further showed that the leaky, streambed of the Mattapoisett River acts as a constant-head boundary and does not effectively retard leakage. The model results are most sensitive to decreases in the hydraulic conductivity values of the aquifer and the streambed.

The response of the aquifer to changes in recharge is not instantaneous. According to the results of transient model runs simulating 1965 reduced recharge conditions, approximately 75 percent of the total expected storage decline in the aquifer takes place in the first year, and further decreases in storage take place at progressively slower rates. The estimate is conservative, and may either reflect the lithology of the aquifer materials or the optimistic value of specific yield used in transient model simulations. Knowledge of the aquifer response time is particularly important in steady-state model analysis using variable recharge rates because it indicates how long the real stress conditions must last for the model results to be realized.

Steady-state model runs were used to assess the impacts of present and proposed withdrawals on water levels and streamflow. Ten pumping scenarios, which consist of 1982, 1982-90, and post-1990 projected withdrawal rates, were devised for simulation. The scenarios were run in the order of the expected startup of the wells. Impacts of the withdrawals were evaluated under two reduced recharge conditions that simulate drought:

1. A "dry" condition representing 1965 average annual recharge; and

2. A "severely dry" condition representing 7-day, 10-year low flow of the Mattapoisett River.

Under dry conditions, the predicted impacts on water levels due to reduced recharge are greatest along the till/stratified-drift boundaries and in the northern part of the model area where simulated water levels are more than 9 feet lower than average levels. For most of the scenarios, the predicted drawdowns in present and proposed wells are within the estimated available drawdown. In two scenarios which simulate the highest withdrawal rates, the available drawdown is exceeded in four wells. Even at the highest withdrawal rates, at least 10 percent of the total ground-water discharge from the aquifer enters the Mattapoisett River.

Under severely dry conditions, the predicted water levels in the aquifer declined up to 19 feet, and several relatively thin areas of the aquifer became desaturated. The model results show that the estimated available drawdown is exceeded in five wells in two scenarios which simulate withdrawal rates, and that the withdrawal rates of six out of the ten scenarios intercept all the ground-water discharge from the aquifer. The predicted results vary depending on the distribution and rate of simulated withdrawal. Nevertheless, the results indicate that there is no net increase in streamflow due to ground-water discharge in the southern half of the valley under most pumping plans.

The predicted impacts of other pumping plans should fall between the best case and worst case alternatives simulated by these scenarios if the assumptions of scenarios remain valid. If new pumping plans are devised which invalidate the model assumptions, then the impacts likely will be greater than those predicted by the present model.

SELECTED REFERENCES

Camp, Dresser and McKee, Inc., 1971, Marion, Massachusetts report on additional water supply: Boston, Massachusetts, September 1971, 14 p.

_____1980, Town of Marion, Massachusetts, New Bedford Road, Rochester well exploration program: Boston, Massachusetts, December 1980, 7 p.

Caswell, Eichler and Hill, Inc., 1983, Water supply exploration and development work for the town of Fairhaven, Tinkham Lane and Pomfret sites: Portsmouth, New Hampshire, March 1983, 15 p.

Cervione, M. A., Jr., Mazzaferro, D. L., and Melvin, R. L., 1972, Water resources inventory of Connecticut, part 6, upper Housatonic River basin: Connecticut Water Resources Bulletin no. 21, 84 p.

de Lima, Virginia, and Olimpio, J. C., 1984, Ground-water resources of the Mattapoisett River aquifer, Plymouth County, Massachusetts: Summary for water-resource managers: U.S. Geological Survey Water-Resources Investigations Report 84-4023.

Denny, C. S., 1982, Geomorphology of New England: U.S. Geological Survey Professional Paper 1208, 18 p.

Dufresne-Henry, Inc., 1981, Report on continuous pumping test, site: test well no. 6: Concord, Massachusetts, February 1981, 11 p.

Farnsworth, R. K., Thompson, E. S., and Peck, E. L., 1982, Evaporation atlas of the contiguous 48 United States: National Oceanic and Atmosphere Administration Technical Report NWS33, 26 p.

Frimpter, M. H., 1981, Probable high ground-water levels in Massachusetts: U.S. Geological Survey Water-Resources Investigations/Open File Report 80-1205, 19 p.

Krumbein, W. C., and Monk, G. D., 1943, Permeability as a function of the size parameters of unconsolidated sand: Transactions of the American Institute of Mineralogical and Metallurgical Engineers, v. 151, p. 153-163.

Maevsky, Anthony, and Drake, J. A., 1963, Southeastern Massachusetts: U.S. Geological Survey open-file report, Massachusetts Basic-Data Report 7, 55 p.

Masch, F. O., and Denny, K. J., 1966, Grain-size distribution and its effect on the permeability of unconsolidated sands: Water Resources Research, v. 2, p. 665-667.

Mazzaferro, D. L., Handman, E. H., and Thomas, M. P., 1979, Water resources investigations of Connecticut, part 8, Quinnipiac River basin: Connecticut Water Resources Bulletin no. 27, 88 p.

Metcalf and Eddy, Inc., 1980, Letter report to Mattapoisett River Water Supply Protection Advisory Committee on evaluation of the aquifer safe yield of the Mattapoisett River basin, Massachusetts: Boston, Massachusetts, July 1980, 11 p.

Morrissey, D. J., 1983, Hydrology of the Little Androscoggin River valley aquifer, Oxford County, Maine: U.S. Geological Survey Water-Resources Investigations Report 83-4018, 55 p.

National Oceanic and Atmospheric Administration, 1960-83, Hourly precipitation data: Asheville, North Carolina.

New England River Basins Commission, 1975, Report of the Southeastern New England Study: Boston, Massachusetts, December 1975.

Ryder, R. B., Cervione, M. A., Jr., and Thomas, C. E., Jr., 1970, Water resources inventory of Connecticut, part 4, southwestern coastal river basins: Connecticut Water Resources Bulletin no. 17, 54 p.

Sterling, C. I., Jr., 1960, Special report on the ground-water resources in the Mattapoisett River valley: Massachusetts Water Resources Commission Bulletin no. W.R. 1, 28 p.

Todd, D. K., 1980, Groundwater hydrology: New York, John Wiley and Sons, 535 p.

Trescott, P. C., Pinder, G. F., and Larson. S. P., 1976, Finite-difference model for aquifer simulation in two dimensions with results of numerical experiments: U.S. Geological Survey Techniques of Water-Resources Investigations, book 7, chap. C1, 116 p.

U.S. Environmental Protection Agency, 1977, National interim primary drinking water regulations: U.S. Environmental Protection Agency, EPA-570/9-76-003, 159 p.

Walton, W. C., 1970, Groundwater resource evaluation: New York, McGraw-Hill, 664 p.

Williams, J. R., and Tasker, G. D., 1974, Water resources of the coastal drainage basins of southeastern Massachusetts, Plymouth, to Weweantic River, Wareham: U.S. Geological Survey Hydrologic Investigations Atlas HA-507.

_____1978, Water resources of the coastal drainage basins of southeastern Massachusetts, northwest shore of Buzzards Bay: U.S. Geological Survey Hydrologic Investigations Atlas HA-560.

Williams, J. R., Tasker, G. D., and Willey, R. E., 1977, Hydrologic data of the coastal drainage basins of southeastern Massachusetts, Plymouth to Wewantic River, Wareham: U.S. Geological Survey Open-File Report 77-186, Massachusetts Hydrologic-Data Report 18, 31 p.

Williams, J. R., Willey, R. E., and Tasker, G. D., 1977, Hydrologic data of the coastal drainage basins of southeastern Massachusetts, northwest shore of Buzzards Bay: U.S. Geological Survey Open-File Report 80-583, Massachusetts Hydrologic-Data Report 20, 30 p.

Wilson, J. L., and Scheiber, R. T., 1982, Analysis of ground-water development of the Truro aquifer: Camp, Dresser and McKee, Inc., Consulting Engineers, Boston, Massachusetts

Wright-Pierce, Inc., 1980, Report on ground-water development at the Wolf Island Road site for the town of Fairhaven, Massachusetts: Lowell, Massachusetts, June 1980, 3 p.

_____1981, Evaluation of 24 X 36 inch gravel-packed well at site 20-79 Wolf Island Road site: Lowell, Massachusetts, July 1981, 15 p.

_____1982, Report on evaluation of the Fairhaven Wolf Island Road well site: Lowell, Massachusetts, November 1982, 19 p.

-NOTES-