Table 3. Simulated volumetric budgets and flow to drains by subbasin for nonpumping and pumping conditions, Case K, West Newbury study area, Massachusetts

[NA, not applicable; ft^3/d, cubic feet per day; gal/min, gallons per minute]

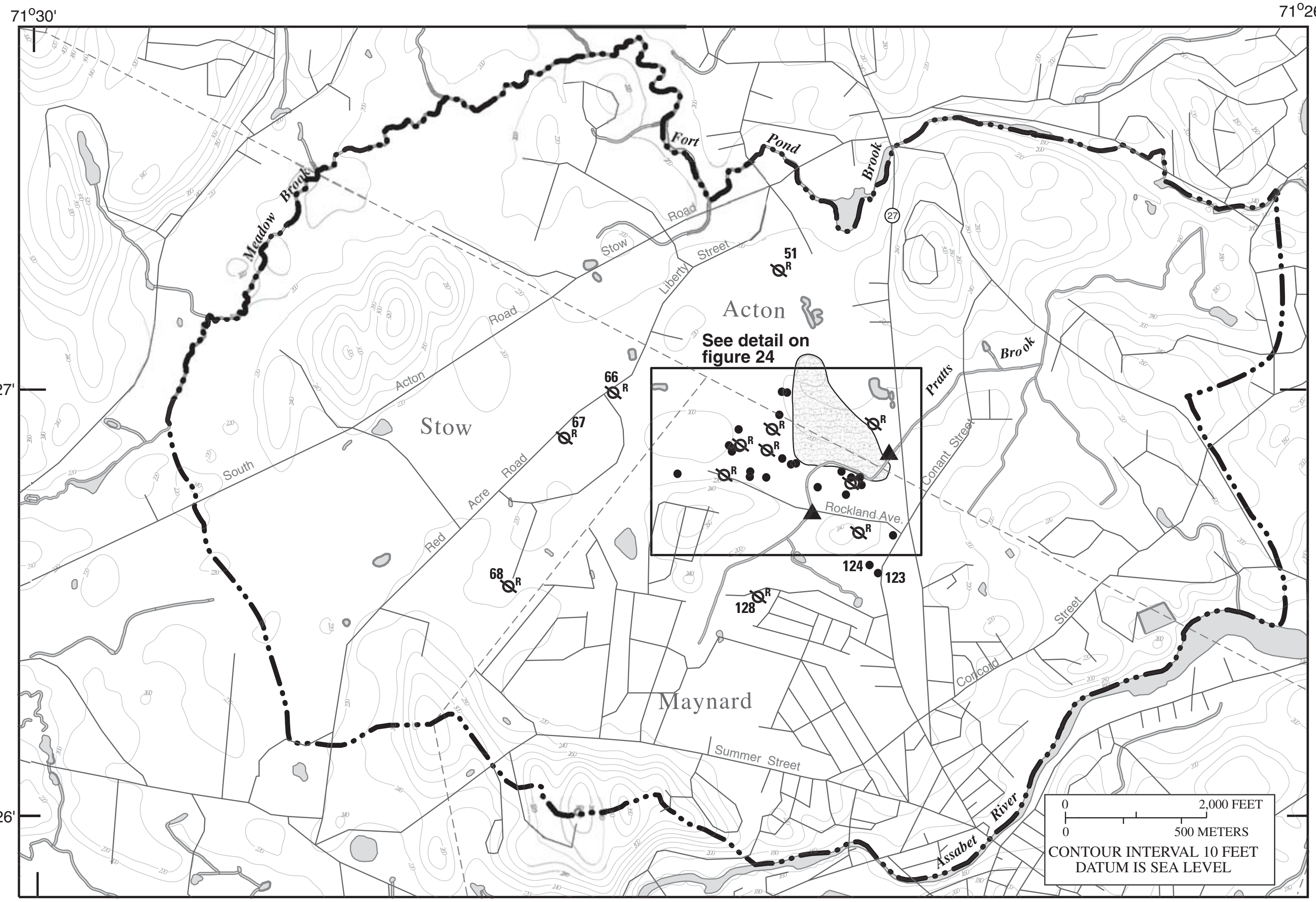
Subbasin and volumetric budget	Non-pumping (ft^3/d)	Pumping (ft^3/d)	Difference	
			ft^3/d	gal/min
Flow to drains				
Subbasin A	17,652	186	17,466	90.6
Subbasin B	18,215	15,416	2,749	14.3
Subbasin C	36,694	15,681	21,013	109.1
Subbasin D	14,302	13,418	884	4.6
Subbasin E	30,004	25,840	4,163	21.6
Subbasin F	39,202	38,165	1,037	5.4
Total	156,069	108,756	47,313	245.6
Number of active drains	321	202	NA	NA
Total model volumetric budget				
IN:				
Constant head	111	154	-43	-0.2
Wells	0	0	0	.0
Drains	0	0	0	.0
Recharge	242,563	242,563	0	.0
Total in	242,675	242,717	-43	-.2
OUT:				
Constant head	86,603	85,640	963	5.0
Wells	0	48,321	-48,321	-250.8
Drains	156,069	108,756	47,313	245.6
Recharge	0	0	0	.0
Total out	242,672	242,718	-45	-.2
IN—OUT	2	0	3	.0
Percent discrepancy	.00	.00	NA	NA

The numerical model also was used to identify wetland areas that might be affected by pumping, by identifying active drains for pumping and nonpumping conditions. Drains that become inactive while pumping (fig. 22) are interpreted as wetland areas that might be affected by pumping at the approved rates. The effects could include a permanent lowering of the water table below the land surface or a reduction in the length of time each year that water is at or near the land surface.

The model is limited in its ability to predict wetland responses to pumping in areas underlain by marine clays. Although numerous drains are shown to become inactive near the Andreas site (fig. 22), pumping may not substantially affect water levels in this wetland area, except where marine clays pinch out against thin till to the east. Where thick marine clays are present, leakage rates are likely lower than the amount of water available for recharge for much of the year. If the vertical hydraulic conductivity for marine clays is 2.7×10^{-5} ft/d (Nielsen and others, 1995) and the maximum vertical hydraulic gradient for steady pumping is 1:1, Darcy's Law yields a maximum leakage rate of 2.7×10^{-5} ft/d. This value is much lower than 5.5×10^{-3} ft/d (24 in/yr), which is assumed to be the amount of water annually available for recharge from precipitation (Lyford and Cohen, 1988). If the vertical hydraulic conductivity of clay were even an order of magnitude higher, the leakage rate would still be less than 5 percent of the water available for recharge from precipitation. The leakage rate also probably would not noticeably affect the water balance of the area underlain by marine clay, except where the clay is thin and pinches out east and northeast of the Andreas site.

ROCKLAND AVENUE WELL SITE, MAYNARD, MASSACHUSETTS

The Rockland Avenue well site is near the Acton and Stow town lines (fig. 23) in the northwestern corner of Maynard, MA. During 1999–2000, six bedrock wells were drilled north of Rockland Avenue (fig. 24) to test for aquifer yield. The area near the wells adjoins wetlands to the north, east, and west, and a till-covered bedrock hill to the south. In March 2000, the Town of Maynard tested the bedrock aquifer for 11 days to determine the sustainable yield of the bedrock aquifer as required by MADEP for permitting. The aquifer test involved pumping three of the six bedrock wells from March 13 to 24, 2000. The extended test provided an opportunity for the USGS to gather information needed to evaluate contributing areas to the wells and the effects of pumping on streamflow and wetlands. The USGS measured water levels in one well in bedrock and one in surficial materials for several months before and after the test. Before the test, the USGS installed 21 piezometers in surficial materials, 4 drive points in wetland sediments, and 2 V-notch weirs on Pratts Brook.



Base from U.S. Geological Survey Digital Elevation Model, Maynard, Massachusetts, 1:24,000, 1965, Universal Transverse Mercator, Zone 19

EXPLANATION

- APPROXIMATE EXTENT OF POND DURING MARCH 2000
- STUDY-AREA BOUNDARY
- 123 WELL OR PIEZOMETER LOCATION AND IDENTIFIER
- 68 WELL EQUIPPED WITH TRANSDUCER DURING AQUIFER TEST, AND IDENTIFIER
- STREAM-GAGING STATION

Figure 23. Location of the Rockland Avenue study area, observation wells, and stream-gaging stations, Maynard, Massachusetts.

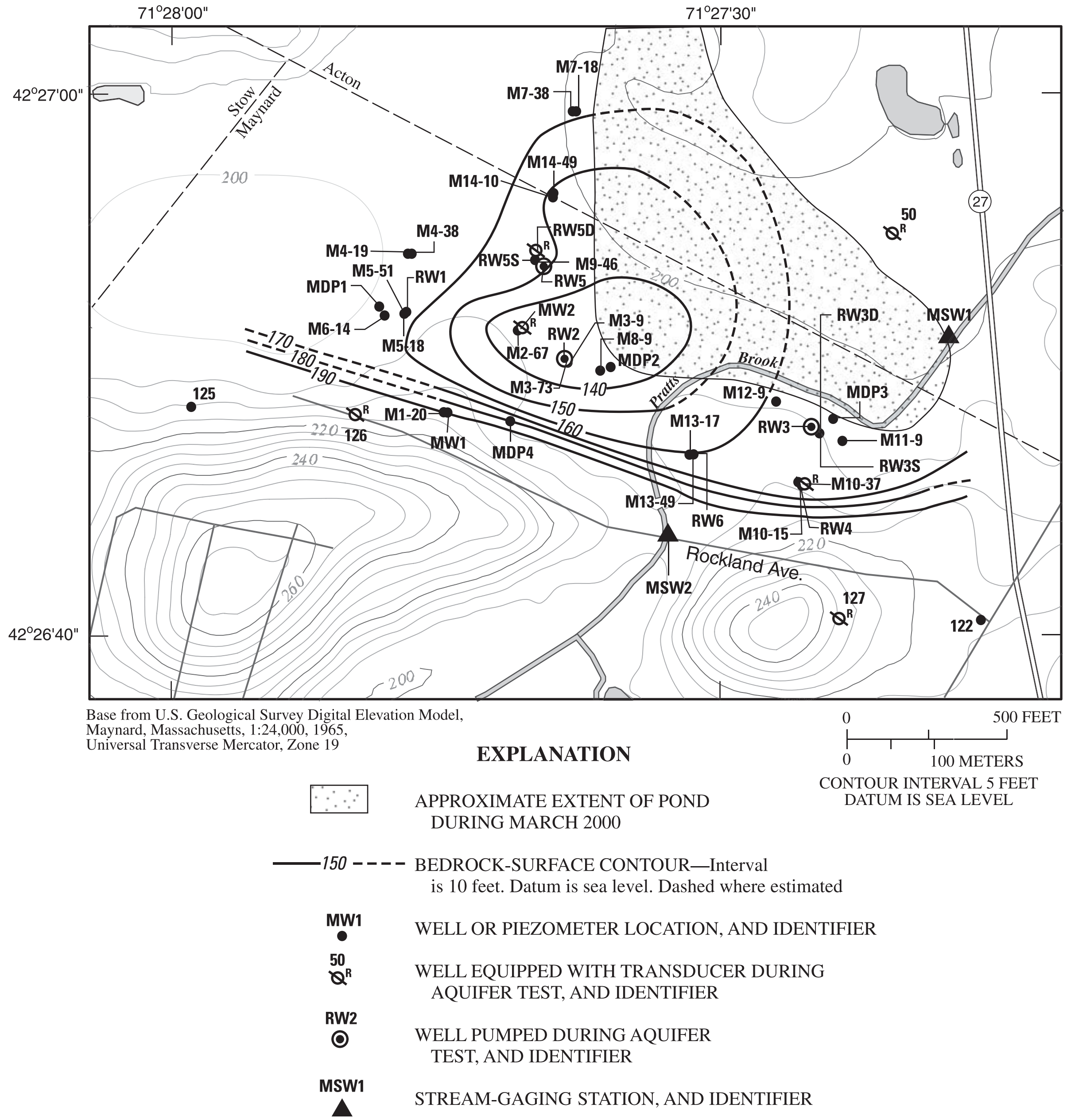


Figure 24. Locations of wells, stream-gaging stations, and bedrock-surface contours near Rockland Avenue, Maynard, Massachusetts.

The USGS also operated pressure transducers and recorders in eight residential wells, two additional bedrock wells near the well site, one piezometer, and two observation wells completed in surficial materials during the period of aquifer testing. Water levels were measured manually in numerous wells during the test. Records of wells used for this study are summarized in table 11 (at back of report).

Most Maynard and Acton residents are served by public water-supply systems; Stow residents obtain their water from private wells. In July 2000, the Town of Maynard installed temporary water lines and began pumping wells RW2 and RW5 for public supply. Pumping continued from well RW3 throughout the fall and winter of 2000–01.

The study area for this investigation extends to features that also serve as boundaries in a numerical model. The study-area boundaries include the Assabet River on the southeast, Meadow Brook on the northwest, Fort Pond Brook on the north, and topographic divides on the northeast and southwest. The study area also includes several large flat areas that are shown as wetlands on topographic maps. Pratts Brook drains the area in the immediate vicinity of the wells and joins Fort Pond Brook to the northeast. The area is largely rural except for residential areas to the south (fig. 23).

Geology

Bedrock geologic mapping in June 2000 (Walsh, 2001b) expanded on previous mapping by Hansen (1956), and borehole-geophysical surveys in the autumn of 2000 provided additional information about fracture orientations in two wells. The following description of the bedrock geology is largely from Walsh (2001b). The surficial geology was mapped by Hansen (1956).

Bedrock

Bedrock in the area of the Rockland Avenue wells consists mainly of schist and biotite gneiss of the Nashoba Formation (Hansen, 1956; Zen, 1983). Near the well field, the Nashoba Formation consists of muscovite-biotite-quartz-plagioclase schist and gneiss. Sulfidic mineralization along fractures, foliation, and in the rock matrix is particularly evident in outcrops south of the well site where the rocks are weathered to a rusty color. Drillers' logs for the rock wells near Rockland Avenue describe soft weathered rock in the upper 20 to 30 ft. Pegmatite and very coarse-grained muscovite-biotite granite intrude the Nashoba Formation rocks at almost all exposures. Other lithologies in the study area include amphibolite and diorite.

Ductile structures include a first generation layer-parallel foliation (S1) and a second generation (S2) schistosity. Although bedding was not apparent in the study area, Hansen (1956) reports that the gneissic banding is approximately co-planar locally with original bedding, so the S1 foliation is approximately parallel to the original bedding.

The second-generation planar fabric is a penetrative schistosity (S2) and a foliation that ranges in character from a cleavage to gneissosity in the granitic and pegmatitic rocks. The S2 foliation is the dominant planar ductile fabric in the Nashoba Formation and consistently strikes northeast and dips steeply southeast to northwest. The average strike and dip of S2 is 243°, 89° northeast. Parting along S2 surfaces is common in all rocks where the foliation is well developed. Folds associated with the second-generation fabric plunge gently to the southwest. The intersection between S1 and S2, which is a linear feature, also plunges gently to the southwest.

Two major faults, the Spencer Brook and Assabet River faults, cross the study area (Zen, 1983) (figs. 25 and 26), but their locations are not precisely known in the study area. The area between these two faults, which includes the well site, is characterized regionally by abundant fractures. Fractures are well interconnected in bedrock outcrops.

Most fractures observed in the area are steeply dipping, but gently dipping fractures or sheeting joints are also present. A heterogeneous distribution of principal-fracture strikes is apparent in outcrops in the Nashoba Formation closest to the well field. A prominent fracture strike ranges from 34–38°. Orientations of water-bearing fractures observed on borehole geophysical logs vary widely. One somewhat prominent fracture set strikes north northwest to north and dips at about 45° east, and a second set strikes east and dips at about 45° south.

Surficial Geologic Units

Surficial materials consist of thin till over bedrock in several areas, thick drumlin till in three areas, and deposits mapped as sand and gravel, including lacustrine silts and fine sands, and ice-contact sand and gravel (fig. 25). Swamp deposits cover glacially derived sediments in many areas (fig. 25). The thickness and vertical distribution of stratified glacial materials have not been mapped in the study area. Near the well site, surficial materials mapped by Hansen (1956) as sand and gravel appear to be predomininantly silt and fine sand on the basis of drillers' logs and fine sediment produced during development of piezometers. These materials appear to fill a closed depression that is at least 70 ft deep near well RW2 (fig. 24). A small

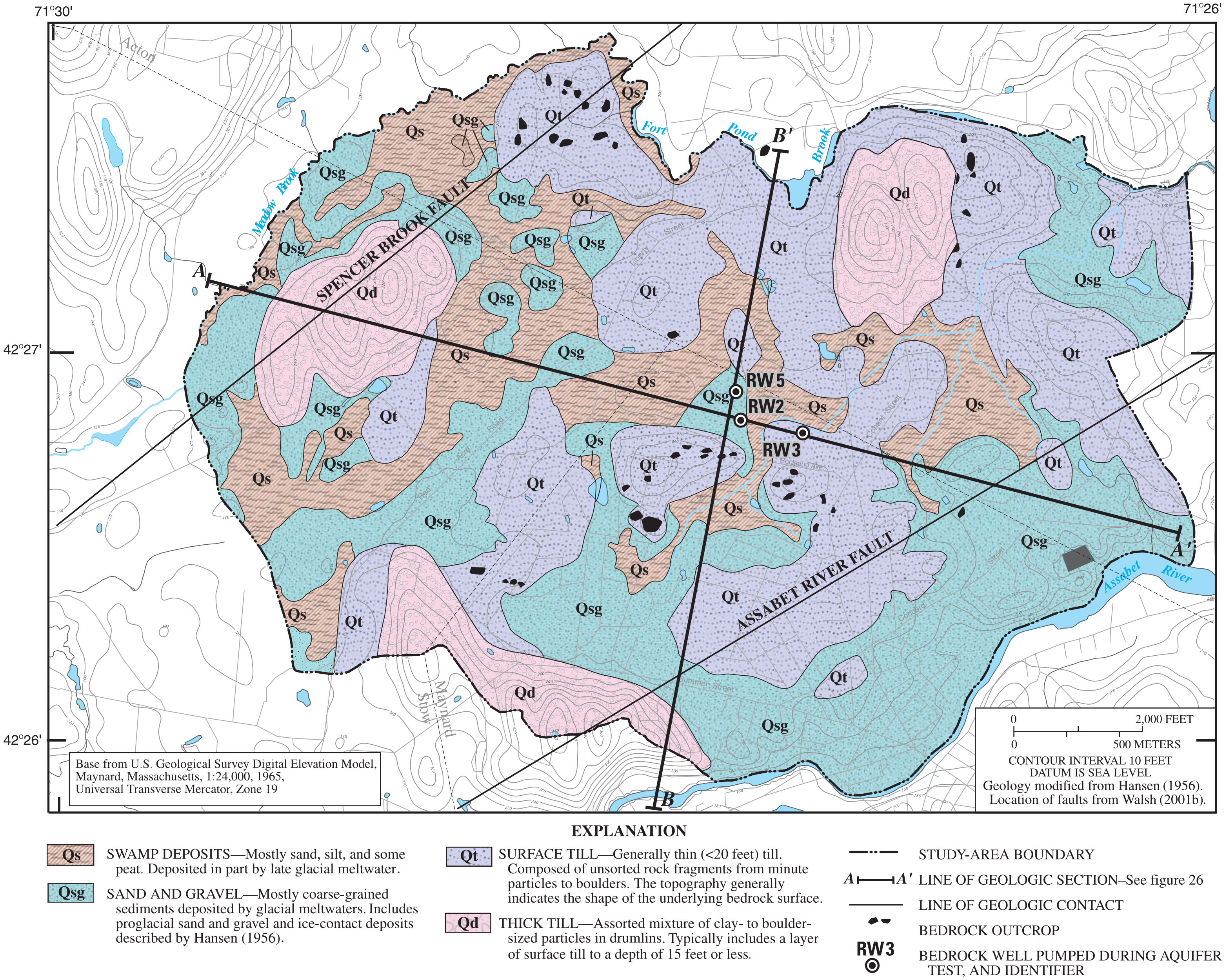


Figure 25. Surficial geologic units, bedrock outcrops, major faults, and locations of geologic cross sections, Rockland Avenue study area, Maynard, Massachusetts.

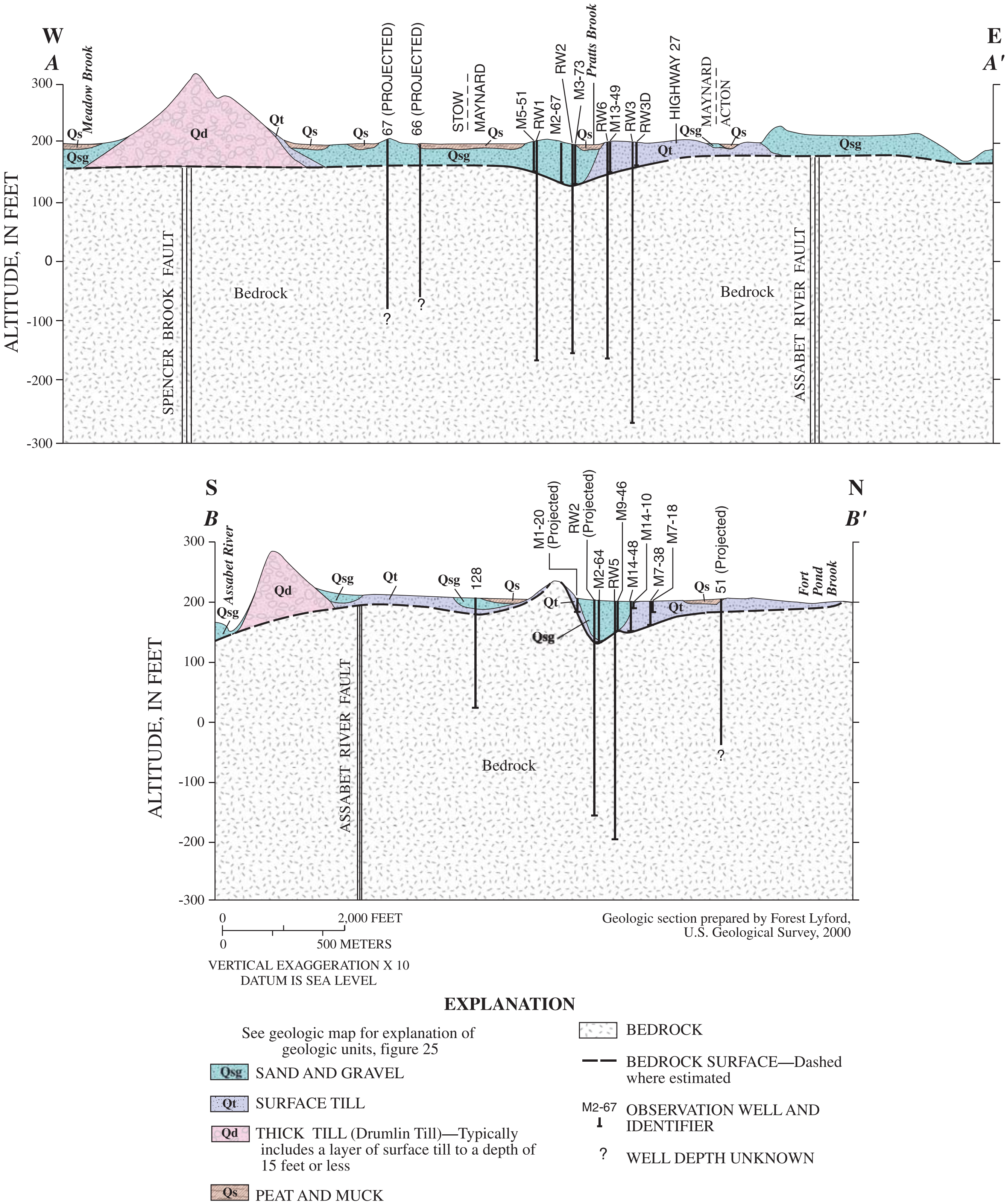


Figure 26. Generalized geologic cross sections A-A’ and B-B’, Rockland Avenue study area, Maynard, Massachusetts.

reduction in penetration rate during installation of piezometers may indicate a thin layer of till over bedrock in some locations. Generally, however, piezometers encountered little resistance during installation, even in areas mapped as till. Drillers' reports and observed water-level responses in piezometers to pumping indicate that surficial materials are generally coarser and more conductive vertically on the west side of the well site near RW1 than elsewhere. Surficial materials near observation wells M7-18 and M7-38, mapped as ground moraine (thin till) by Hansen (1956), are mostly fine silt and sand, determined from the relative ease with which piezometers were installed and the sediments removed during well development. Large stones and boulders that are scattered at the land surface in this area were not encountered below the surface during piezometer installation.

Ground-Water-Flow Patterns and Water-Level Fluctuations

Water-level data for wells measured in March 2000 and altitude data for streams and wetlands indicates that ground water in much of the study area flows to the northeast toward Fort Pond Brook (fig. 27). A steep gradient toward the Assabet River is likely on the southeast side of the study area. Discharge from bedrock in and near wetland areas is by upward leakage through surficial materials.

For nonpumping conditions, water levels in shallow well MW2 and bedrock well RW4 fluctuated over a range of about 4 ft from October 1999 through June 2000 (fig. 28) before production pumping began in July 2000. The shallow well responded quickly to recharge from precipitation, but responses in the bedrock well were less rapid than in the shallow well. Water levels in both wells declined during the aquifer test in March 2000, but water levels declined more in the bedrock well (RW4) than in the shallow well (MW2) in surficial materials. Water-level declines were approximately parallel in both wells after continuous pumping for public water supply began in July 2000.

Ground-Water Recharge

The amount of water available for recharge (precipitation minus evapotranspiration) totals about 24 in/yr in the study area (Lyford and Cohen, 1988; Randall, 1996). Local variations in evapotranspiration rates, such as in wetland areas, may result in either more or less than 24 in. available for recharge. Conceptually, evapotranspiration rates for wetland areas may be higher than elsewhere (Zarriello and Ries, 2000), but data were not available for the study area. Much of the water available for recharge either infiltrates or runs off saturated areas as overland flow. Additionally, intense storms may cause surface runoff from hill slopes. In the study area, much of the surface runoff from hills probably infiltrates at the base of the hill or flows into nearby channels. Recharge rates in dense residential areas are not known.

Aquifer Testing and Observed Hydrologic Responses to Pumping

The aquifer test at the Maynard site involved pumping three wells at staggered starting times for a total of 11 days during March 13–24. Streamflow and water levels in wells at the start of the test were declining after a storm of about 1.7 in. on March 11–12, 2000. On March 16–17, precipitation consisting of rain and wet snow totaled 1.1 in. Rainfall that totaled about 1.6 in. from another storm on March 28–29 accelerated recovery of water levels after pumping stopped. Precipitation amounts were measured at a USGS precipitation station about 1 mi southeast of the well site.

Well RW2 was pumped initially for 24 hours to observe water-level responses for a single pumped well. Pumping at wells RW3 and RW5 was started after the initial 24 hours. The pump in RW3, however, failed after a few hours and was restarted about 20 hours later. Water was piped to a discharge point near and downstream from stream gage MSW1 (fig. 24). Water levels were measured manually and with pressure transducers and recorders in numerous wells and piezometers during the test and for about 7 days after pumping ended. Water-level hydrographs for bedrock wells are shown in figure 29 and are shown for wells in surficial materials in figure 30. Stream stage was measured at two weirs on Pratts Brook (MSW1 and MSW2, fig. 24).

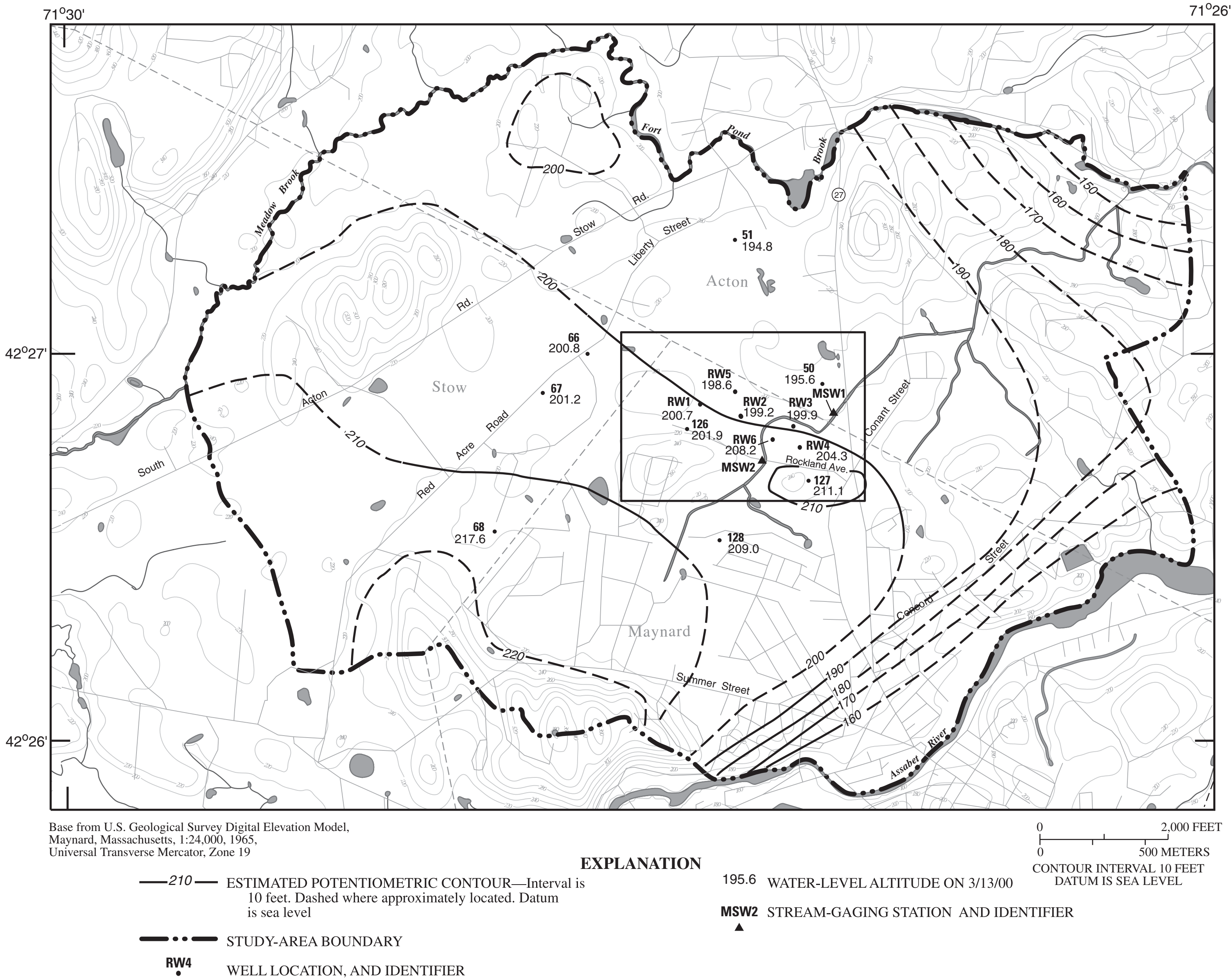


Figure 27. Potentiometric surface altitude for the bedrock aquifer, March 2000, Rockland Avenue study area, Maynard, Massachusetts.

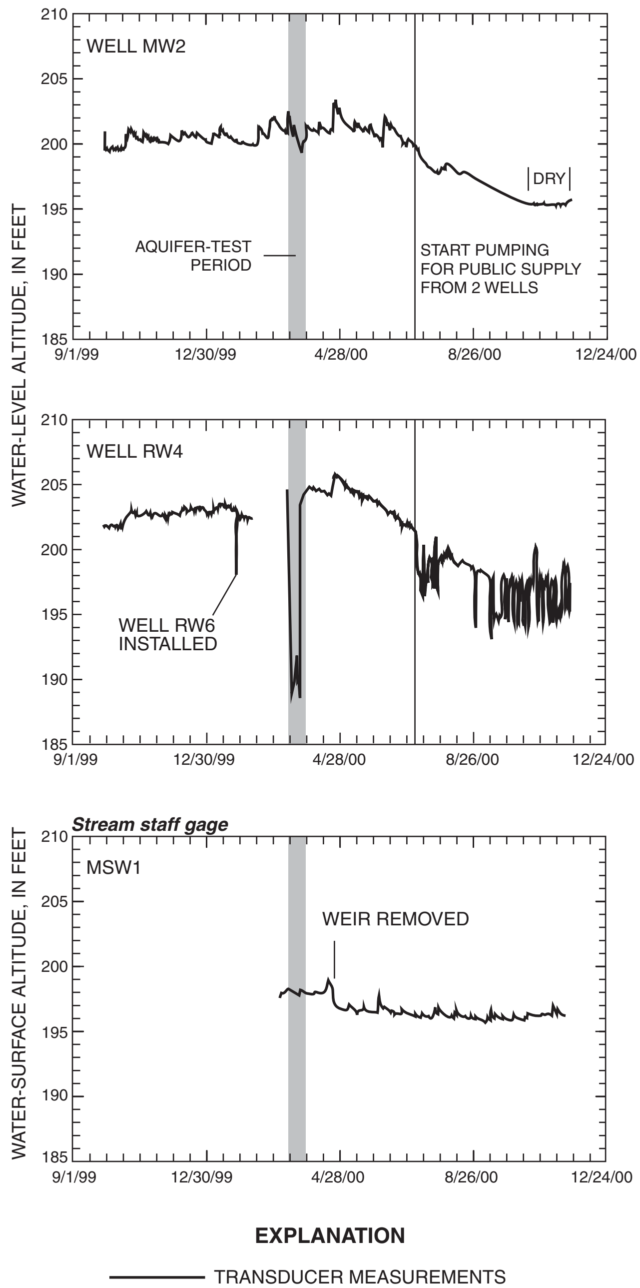
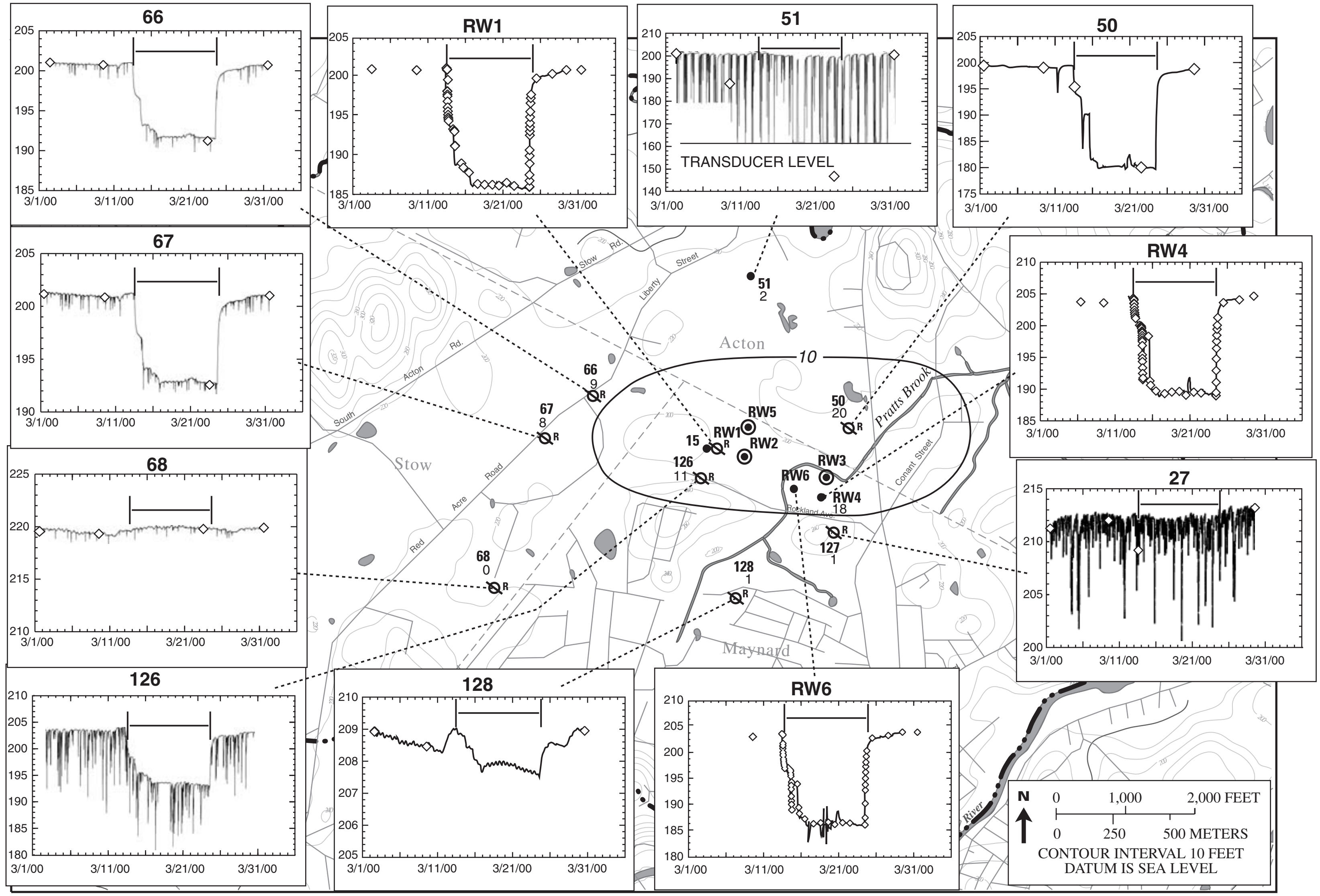


Figure 28. Water-level and stage hydrographs for wells MW2 and RW4 and streamflow station MSW1, September 1999 to November 2000, Rockland Avenue study area, Maynard, Massachusetts.

Throughout the test, water was ponded behind the weir at MSW1. The approximate extent of the pond observed during the aquifer test is shown in figure 23. A pond was present in this area before the test because of a beaver dam at the location of MSW1. The water level was temporarily lowered during construction of the weir, but it recovered quickly during a storm about 2 days before pumping started. Stage in the pond fluctuated during the aquifer test, mainly in response to precipitation and runoff.

Water levels in most observation wells and piezometers responded to pumping (figs. 29, 30). Exceptions were bedrock wells 68, 127, and probably 128 located south of the Rockland Avenue site, and shallow wells MW1, M1-20, M12-9, 122, 123, and 124, in surficial materials (figs. 23 and 24). Of the shallow wells and piezometers in which water levels did not change, only the hydrograph for MW1-20 is included on figure 30. Water levels in piezometer M12-9 appeared to parallel water levels in the pond. The lack of response in wells and piezometers near Rockland Avenue may indicate a sharp transition to less permeable bedrock in this area. Water levels in residential well 128, completed in bedrock south of the site, may have responded slightly to pumping, but the water level declined less than 1 ft and quickly stabilized. The well also recovered about 1 ft after pumping stopped. Water levels in wells completed in bedrock east and west of the well site responded quickly to pumping but also quickly stabilized. The rapid response and stabilization of water levels probably indicates a leaky confined-aquifer condition.

Water-level declines caused by cyclic pumping for residential use are apparent on hydrographs for sev-eral residential wells. The magnitude of the declines is a general indicator of bedrock transmissivity near the wells. For example, cyclic water-level declines in wells 66 and 67 are considerably less than those in wells 51 and 127 (fig. 29). Small water-level declines during pumping cycles in well 68 indicate a relatively high transmissivity near that well.



Base from U.S. Geological Survey Digital Elevation Model,
Maynard, Massachusetts, 1:24,000, 1965,
Universal Transverse Mercator, Zone 19

EXPLANATION

VERTICAL SCALE OF GRAPHS IS WATER LEVEL ALTITUDE, IN FEET ABOVE SEA LEVEL

STUDY-AREA BOUNDARY

—10— INTERPRETED 10-FOOT-DRAWDOWN CONTOUR AT THE END OF AQUIFER TESTING.

TRANSDUCER MEASUREMENT—Individual points are manual measurements.

PUMPING PERIOD

51 WELL OR PIEZOMETER LOCATION, AND IDENTIFIER

RW5 WELL PUMPED DURING AQUIFER TEST, AND IDENTIFIER

128 1 WELL EQUIPPED WITH TRANSDUCER DURING AQUIFER TEST, IDENTIFIER AND DRAWDOWN, IN FEET AT THE END OF THE AQUIFER TEST

Figure 29. Water levels in selected observation wells in bedrock during aquifer testing , March 15–31, 2000, Rockland Avenue study area, Maynard, Massachusetts.

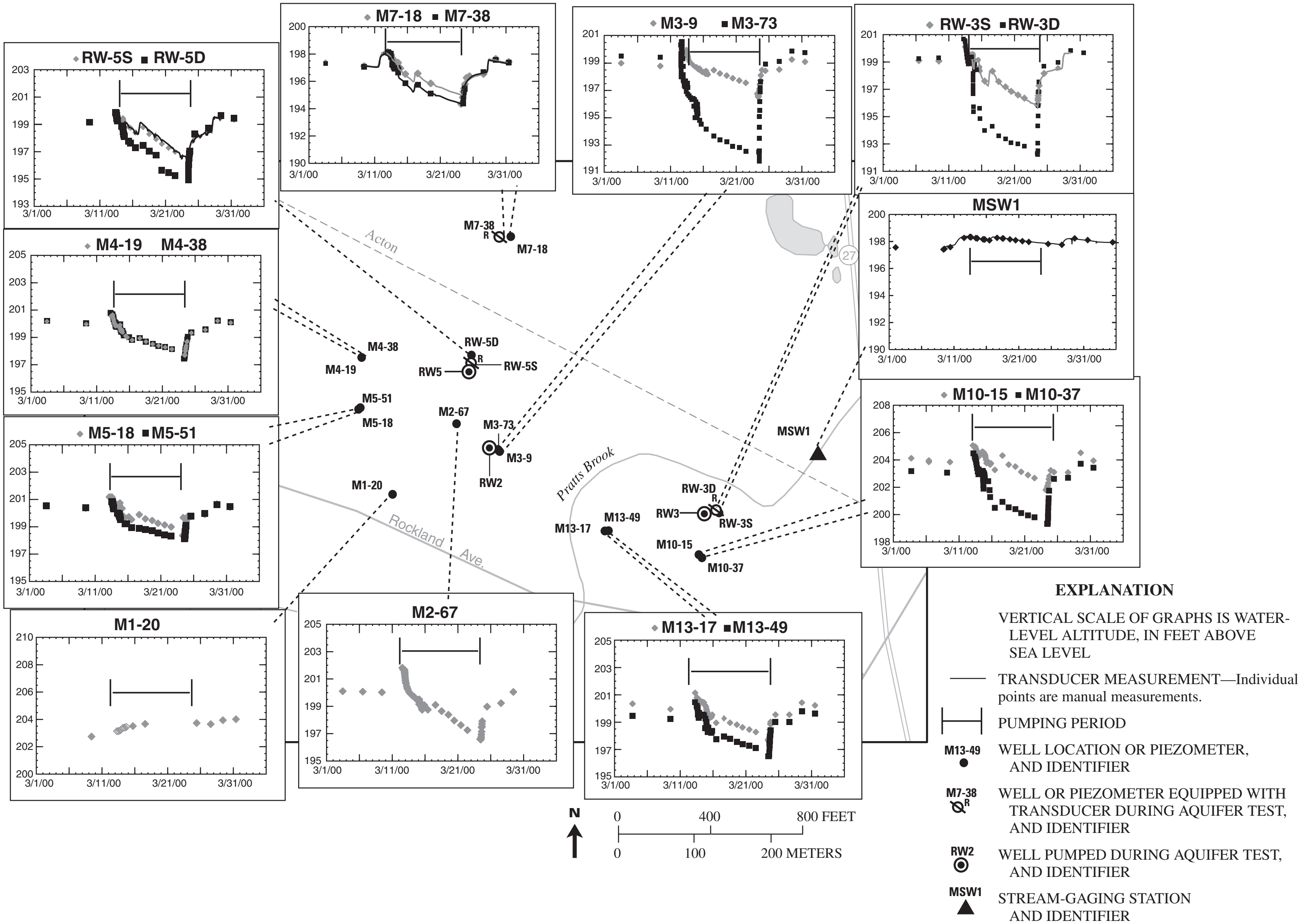


Figure 30. Water levels in selected wells in surficial material and one stream during aquifer testing, March 15–31, 2000, Rockland Avenue study area, Maynard, Massachusetts.

Water levels in piezometers that responded to pumping declined gradually throughout the test, except when recharge from a storm on March 17 caused water levels to rise (fig. 30). After the nearly instantaneous rise caused by recharge, the downward trend continued. For each pair of piezometers completed in surficial materials, drawdowns were generally greater in the deeper of the two piezometers. The largest difference was in piezometer pair M3-9 and M3-73. An exception was at piezometer pair M4-19 and M4-38 where water-level altitudes were nearly identical throughout the test, indicating a high vertical hydraulic conductance at this location. The gradual decline in water levels during the test indicated a general lowering of the water table. A rapid water-level rise in piezometers after pumping ended and before water levels in nearby bedrock wells rose above the water levels in piezometers indicated that water levels in the shallow wells were below shallower sources of water. Shallower sources of water include storage at the water table, the pond, and wetland areas.

The water level in piezometer M3-73 rose 0.20 ft during the first 3 minutes of pumping in nearby well RW2 and fell about 0.26 ft during the first 6 minutes after the pump was shut off. This reversed response is commonly observed in fine-grained confining materials and is attibuted to expansion of aquifer materials with a sudden change in pressure (Wolfe, 1970a).

The water level in drive point MDP1 (fig. 24), placed in standing water on the edge of a wetland west of the well site, dropped 0.2 ft during the aquifer test. Less ponded water was apparent in this area after the test than before the test, but the period of water-level record is not sufficient to distinguish the effects of pumping from the effects of normal recession after a storm. Water levels in drive points MDP2 and MDP3 reflect water levels in the pond upstream from the weir at MSW1. The water level in the pond rose after a storm on March 17 but was about 0.45 ft lower at the end of the test than at the beginning. The effects of pumping on streamflow at MSW1 were not readily apparent because stage was changing continuously in response to precipitation and outflow from the pond. Streamflow data are not included in this report because interpretation of pumping effects was inconclusive.

Hydraulic Properties of Geologic Units

Yield data for bedrock wells are limited to bedrock wells at the well site, where driller-reported yields ranged from 25 to 400 gal/min, and to one residential well (well 128), which has a reported yield of 30 gal/min. The initial 24 hours of pumping for well RW2 caused drawdown in five bedrock-observation wells within the well site (RW1, RW3, RW4, RW5, and RW6) and in five residential wells (50, 66, 67, 126, and 128). A rapid response to pumping and drawdowns of several feet after 24 hours of pumping in wells located 3,000 ft or more west of RW2 indicate a highly transmissive and confined aquifer. A leaky-aquifer response (Hantush and Jacob, 1955) was apparent on a logarithmic plot of drawdown and time by a deviation from the Theis (1935) type curve after 90 minutes or less of pumping (fig. 31). Bedrock hydraulic properties will be discussed in the Numerical Modeling section.

The vertical conductance between surficial materials and bedrock and the vertical hydraulic conductivity of surficial materials at the well site, which are useful for predicting long-term water-level responses to pumping, were computed from water-level data collected during the aquifer test (table 4). For these calculations, a vertical flux of water (volumetric flow per unit area in cubic feet per day per square feet, also known as specific discharge) was first calculated by multiplying the rate of steady water-level decline during a 5-day period from March 18 to 22, 2000, by a specific yield for surficial materials. An average specific yield of 0.10 was estimated from the rise of water levels in shallow piezometers caused by 1.1 in. of precipitation on March 17, 2000. The analysis method assumes that lateral flow of ground water is limited and the water-level declines resulted mainly from leakage. The method also assumes that water-level declines observed in shallow piezometers reflect declines at the water table. By this method, vertical conductance for the bedrock aquifer ranged from 0.0012 to 0.0019 day^{-1}.

Vertical conductance values for surficial materials range from 0.003 day^{-1} at piezometer M3-9 to 0.035 day^{-1} at piezometer M14-10 and average 0.015 day^{-1} (table 4). Higher values are apparent at piezometer M4-19 but could not be calculated because

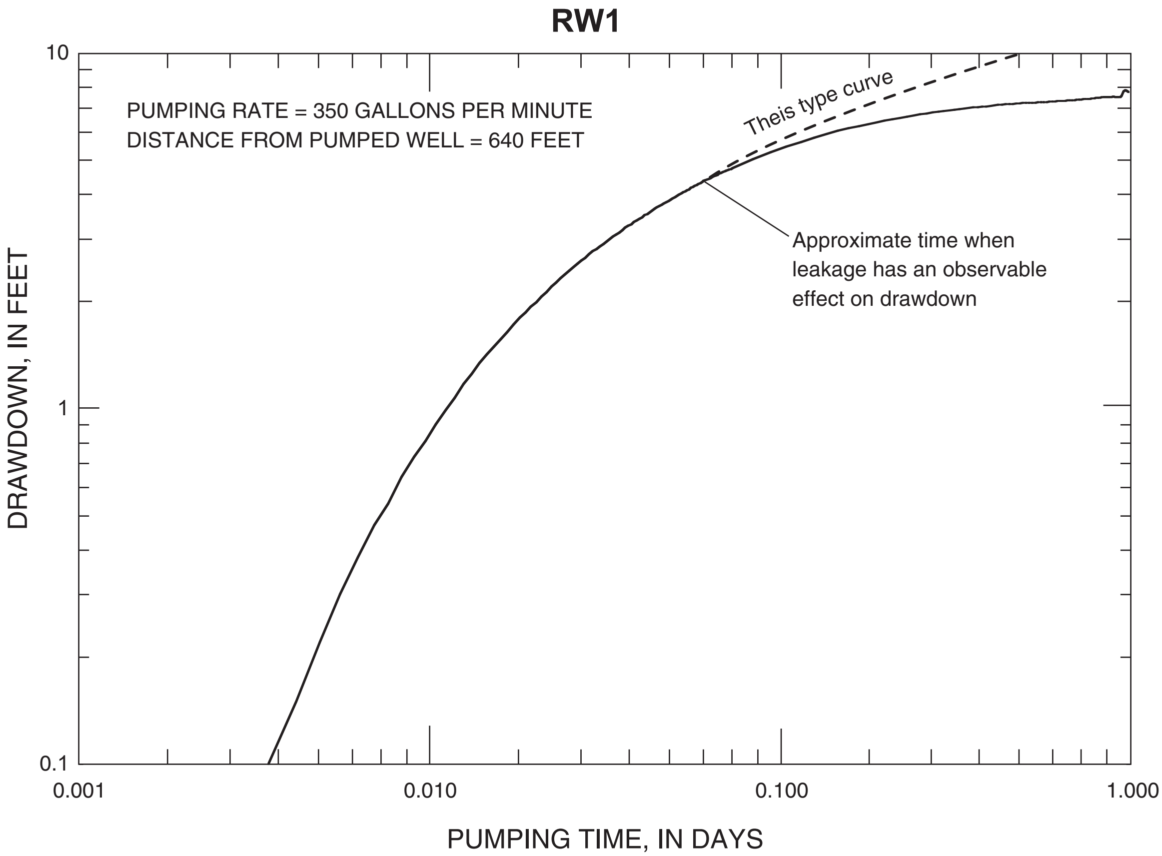


Figure 31. Logarithmic plot of drawdown with time in well RW1 compared to the Theis type curve, Rockland Avenue study area, Maynard, Massachusetts.

water-level altitudes were essentially the same in the well pair throughout the test. Vertical hydraulic-conductivity values, calculated by multiplying conductance by the distance between well screens at each piezometer pair, range from 0.13 ft/d at well RW3S to 1.35 ft/d at piezometer MW14-10 and average 0.48 ft/d. These values of vertical hydraulic conductivity are within the range that might be expected for silt and fine sand (Heath, 1989), and are consistent with a median value of 0.14 ft/d for lacustrine silt reported by Randall and others (1988). The rates of water-level decline may be overestimated because water levels also were declining in the ponded area after it filled with storm runoff on March 17. If the rate of water-level decline is corrected for a general water-level decline in the surficial aquifer caused by a lowering of pond stage during the last 5 days of the test, then the value of vertical hydraulic conductivity is decreased by about one half. The horizontal hydraulic conductivity of lacustrine silt, fine sand, and thin till has not been determined in the study area but probably ranges from 1 to 10 ft/d (Melvin and others, 1992).

Numerical Model

A numerical model was constructed to determine likely contributing areas to wells while pumping at the rate approved by MADEP. The model also was used to determine possible effects of pumping on streamflow and wetland water levels.

Head data for bedrock, surficial wells, and piezometers before aquifer testing and transient drawdown and recovery responses, were used for model calibration. Data for nearby USGS observation wells ACW-158 (Acton) and CTW-167 (Concord) indicated water levels were near the average water levels for the period of record (fig. 32). A rising water level in both wells, however, indicated that recharge rates may have exceeded average rates during this period. Stage changes in the pond were negligible compared to

Table 4. Estimates of specific yield, vertical conductance, and vertical hydraulic conductivity, Rockland Avenue wells, Maynard, Massachusetts

[**Rise on March 17, 2000:** Rise occurred after 1.01 inch (0.084 ft) of precipitation. **Rate of water-level decline:** Determined for a 5-day period of steady decline from March 18 to March 22, 2000. **Head difference:** Difference in head between the shallow and deep piezometer and between the deep piezometer and bedrock well on March 22, 2000. **Vertical conductance:** Vertical flow rate divided by head difference. **Vertical hydraulic conductivity:** vertical conductance multiplied by vertical distance. **Vertical flow rate:** Rate of water-level decline multiplied by an average specific yield of 0.10. NA, not applicable; $ft^3/d/ft^2$, cubic feet per day per square foot; day^{-1}, 1/day; ft, foot; ft/d, foot per day; --, not calculated]

Well pair	Rise on March 17, 2000 (ft)	Specific yield	Vertical distance between center of well screens (ft)	Rate of water-level decline (ft/d)	Vertical flow rate ($ft^3/d/ft^2$)	Head difference (ft)	Vertical conductance (day^{-1})	Vertical hydraulic conductivity (ft/d)
Surficial aquifer								
MW2/M2-67	1.00	0.08	[1]58.5	0.16	0.016	1.84	0.0087	0.51
M3-9/M3-73	.80	.11	64.5	.16	.016	4.97	.0032	.21
M4-19/M4-38	.50	.17	19.5	.10	.010	.00	--	--
M5-18/M5-51	.90	.09	32.5	.14	.014	.70	.0200	.65
M7-18/M7-38	.90	.09	19.0	.18	.018	.64	.0281	.53
M10-15/M10-37	1.50	.06	22.0	.20	.020	2.83	.0071	.16
M13-17/M13-49	.90	.09	32.0	.16	.016	1.12	.0143	.46
M14-10/M14-49	.70	.12	39.0	.18	.018	.52	.0346	1.35
RW3S/RW-3D	1.30	.06	25.5	.18	.018	3.55	.0051	.13
RW5S/RW-5D	.90	.09	30.5	0.18	.018	1.68	.0107	.33
Average	--	0.10	--	--	0.016	--	.0146	0.48
Bedrock								
M5-51/RW-1	NA	NA	NA	0.14	0.014	12.13	0.0012	--
M13-49/RW-6	NA	NA	NA	.16	.016	10.70	.0015	--
M10-37/RW-4	NA	NA	NA	.20	.020	10.50	.0019	--
Average	--	--	--	--	0.017	--	0.0015	--

[1]Assumed center of screen in MW2 at depth of 5 feet.

drawdown in wells and piezometers (figs. 29 and 30) and were not simulated during calibration. Model calibration was considered adequate to accomplish the goals of this study. The importance of aquifer-system uncertainties on the size and shape of contributing areas was examined by sensitivity testing.

Areal Extent and Boundary Conditions

The model area (fig. 33) coincided with the study area shown in figure 23. The Assabet River, Fort Pond Brook, and Meadow Brook, shown as perennial streams on topographic maps, were simulated as constant-head blocks in the upper layer of the model. Altitudes of the constant-head blocks were estimated from topographic contours. Elsewhere, the edge of the model extended to presumed ground-water divides, simulated as no-flow boundaries.

Horizontal and Vertical Discretization

The Maynard site finite-difference model grid consisted of uniformly spaced square blocks that were 200 ft wide on a side (fig. 33). Two layers were used to represent the ground-water system (fig. 34). The upper layer (layer 1) approximately represents the surficial materials and shallow bedrock, and a second layer represents the deeper bedrock aquifer. A uniform altitude of 160 ft was assigned for the bottom of layer 1 and the top of layer 2, except for areas in the northeast and southeast where the land surface is below this altitude.

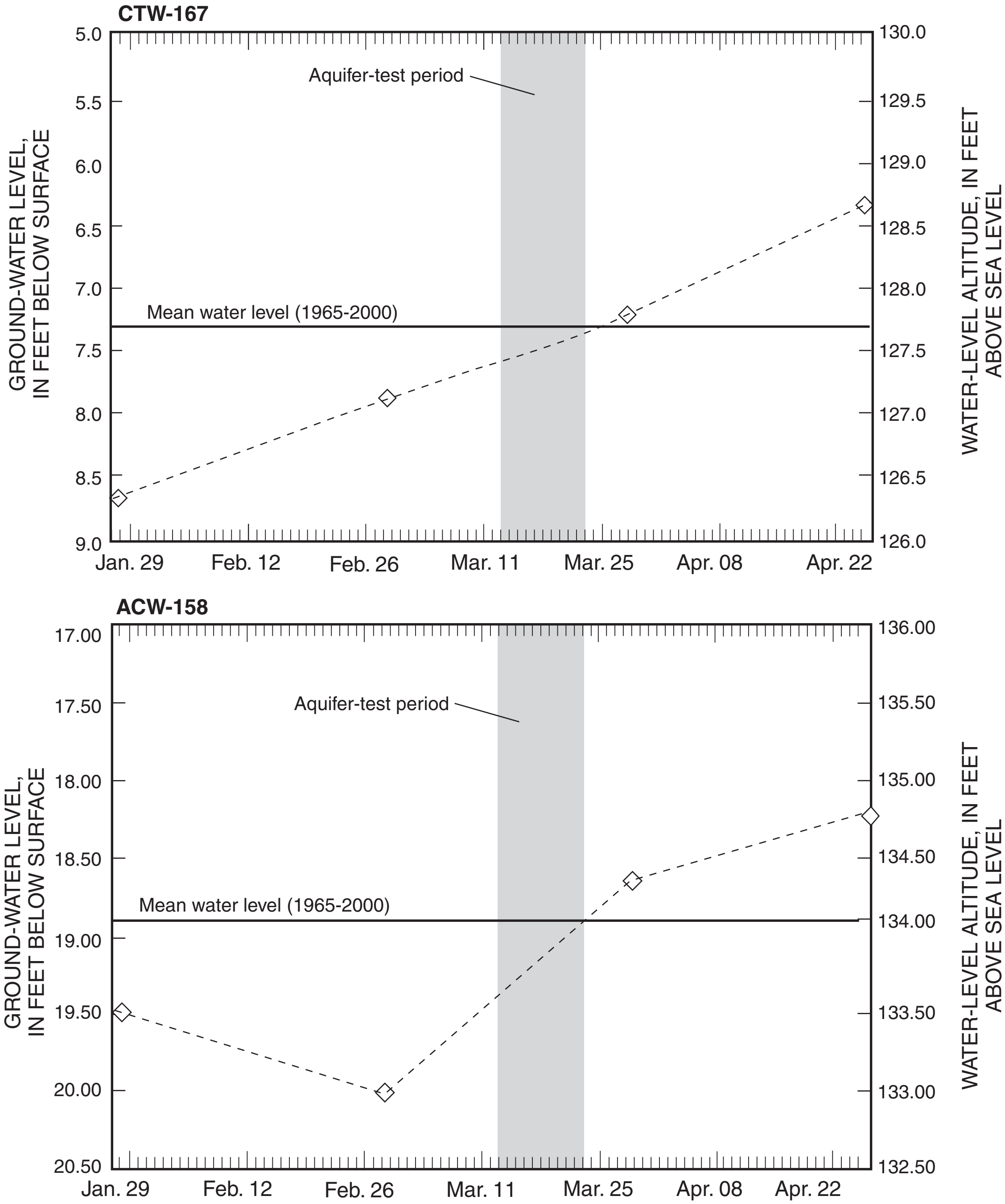


Figure 32. Water levels in U.S. Geological Survey observation wells CTW-167 and ACW-158 during January to May 2000 and mean water levels for the period of record, Acton and Concord, Massachusetts.

There, the bottom was placed 20 to 30 ft below the bottom of the nearest rivers and streams. The base of layer 2 was arbitrarily placed at a uniform altitude of minus 200 ft, which is the approximate altitude of the bottom of the deepest bedrock well at the well site, and a reasonable maximum depth for most ground-water circulation. Layer 1 was simulated as unconfined, and layer 2 was simulated as confined but convertible to unconfined if the simulated head drops below the top of the layer.

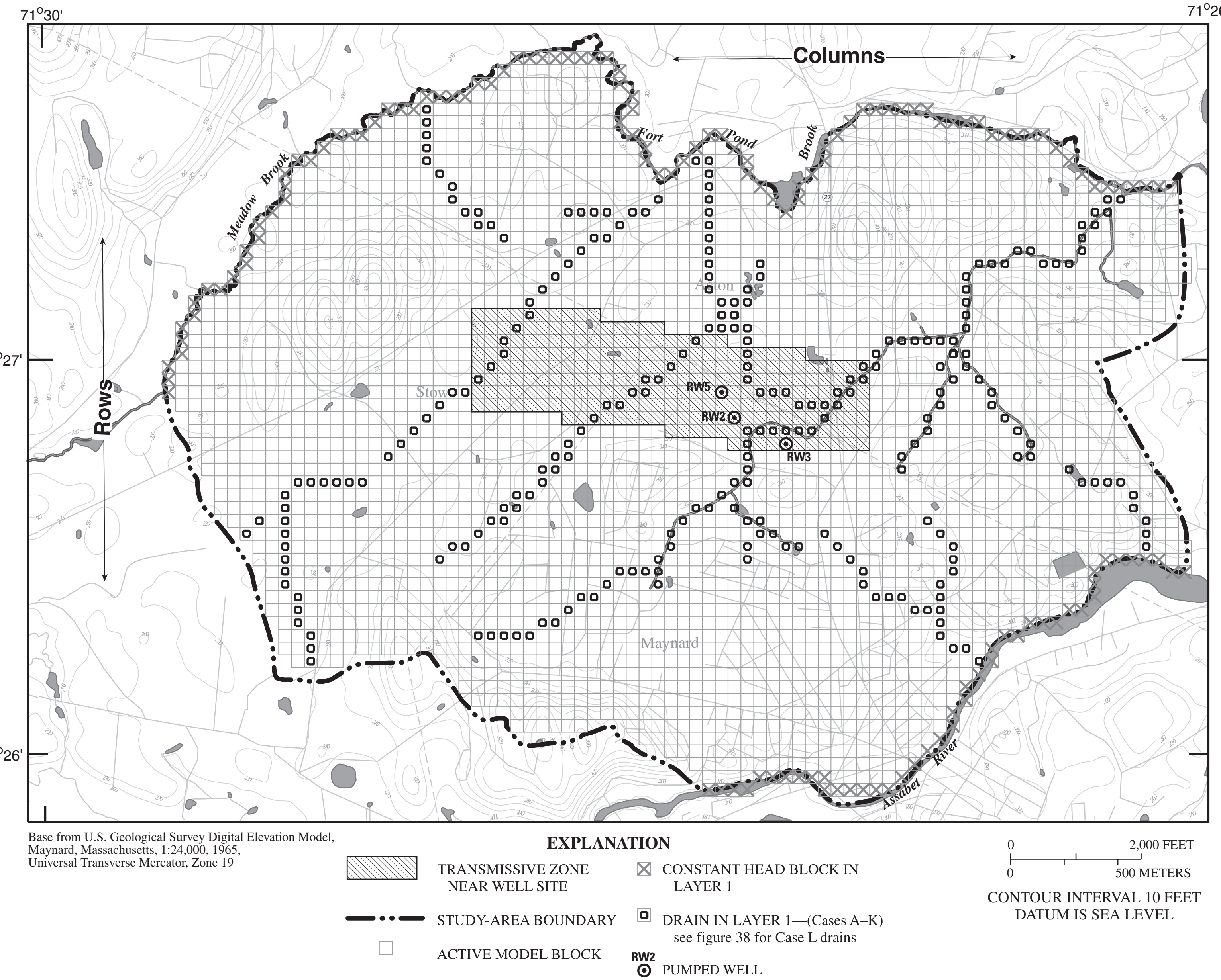


Figure 33. Model features and locations of wells pumped during aquifer testing, Rockland Avenue study area, Maynard, Massachusetts.

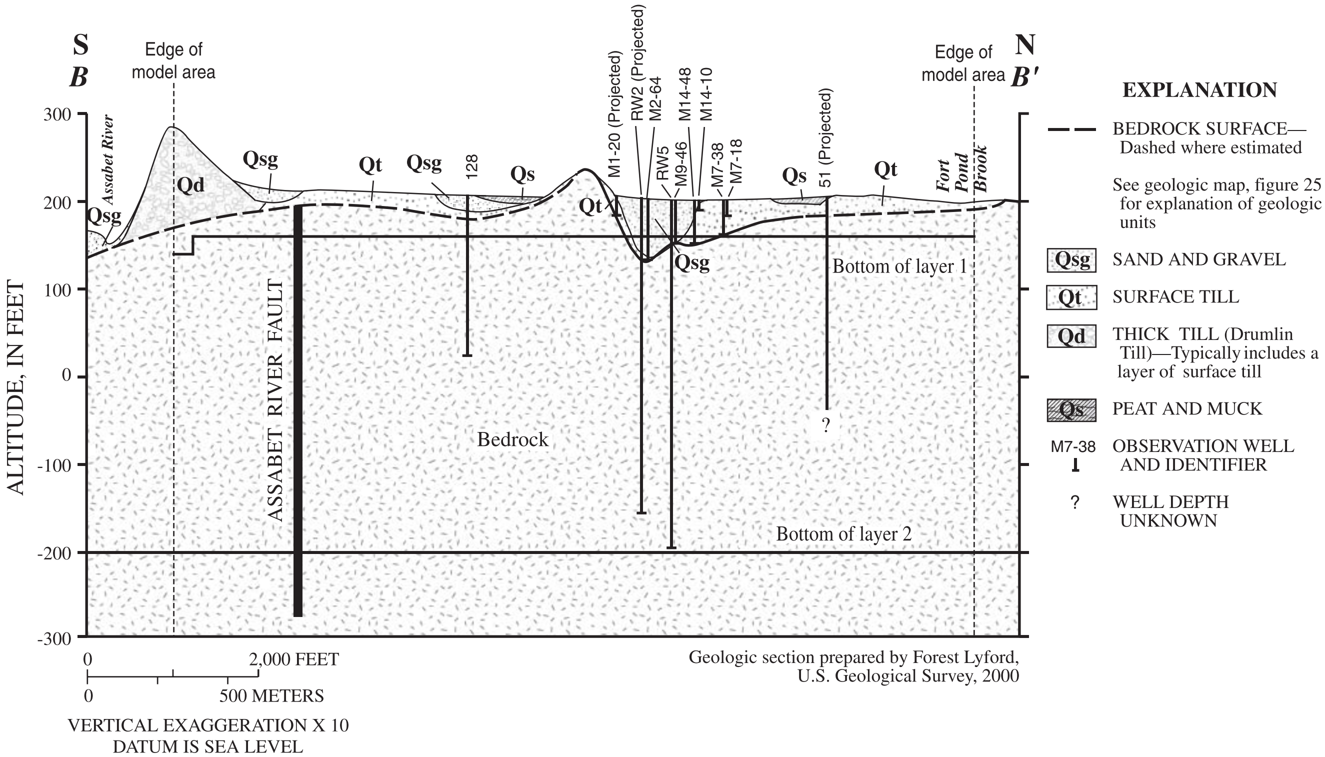


Figure 34. Geohydrologic cross section and model layers, Rockland Avenue study area, Maynard, Massachusetts.

Model Stresses

Model stresses included drains, wells, and recharge. Drains were placed at stream locations or along topographic depressions that are possible discharge areas for ground water. Drains also were placed in some wetlands where channels are not apparent on maps. These drains allow ground water to discharge in these areas when simulated heads are above the land surface. A large uniform conductance of 20,000 ft^2/d was assigned so that aquifer properties, rather than the drain conductance, constrained flow into the drains.

Wells were placed at locations of RW2, RW3, and RW5 for simulation of the aquifer test. For the aquifer test, pumping rates for the three wells were adjusted in several stress periods to approximate observed pumping rates (fig. 35). The MADEP-approved rates were used to simulate contributing areas to wells and effects of pumping on wetlands and streams. The approved rates are 322 gal/min for RW2, 199 gal/min for RW3, and 263 gal/min for RW5 (Joseph Cerutti, oral commun., 2000).

Recharge was applied uniformly at 2 ft/yr (0.0055 ft/d) throughout the model area for steady-state and transient simulations. Although recharge rates may vary where topography and the character of surficial materials causes overland flow, this flow is a small component of the water budget for the model area and, conceptually, becomes recharge at the base of slopes. The model, therefore, accounts for most of the water that is available for recharge. Where a water table at the land surface constrains recharge in real ground-water systems, drains readily remove the recharged water in the model. A somewhat higher-than-average recharge rate during the aquifer test, if accounted for in the model, would have a negligible effect on simulated water levels.

Hydraulic Properties

A uniform hydraulic conductivity value of 10 ft/d was assumed for layer 1 and was not adjusted during model calibration (table 5). This value is intended to represent the hydraulic conductivity of till and weathered rock in thin-till areas, silt and fine sand,

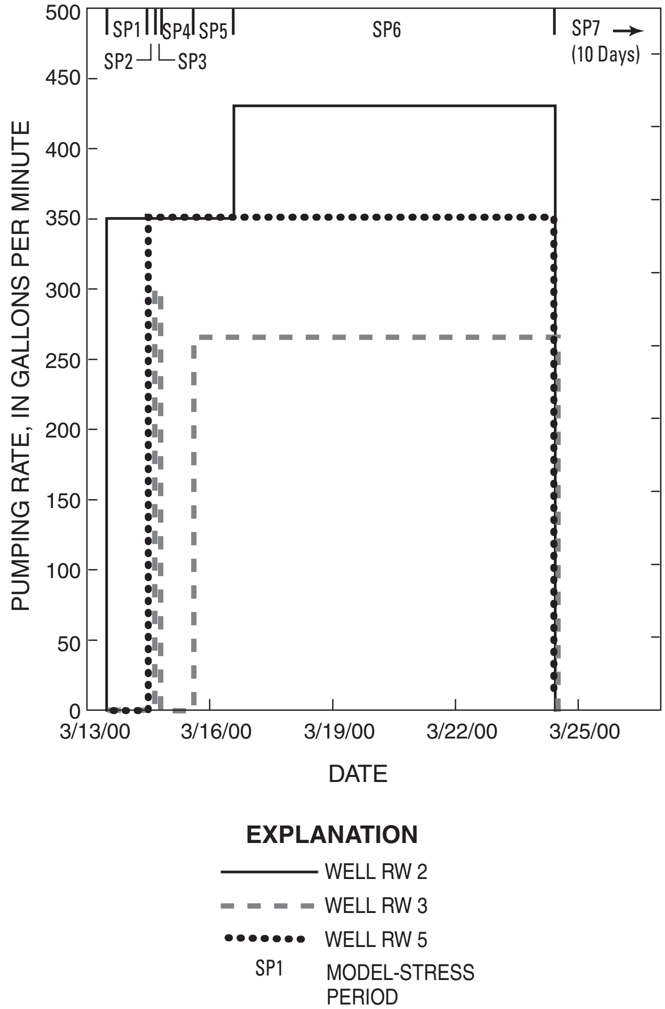


Figure 35. Observed pumping rates during aquifer testing and simulated stress periods, Rockland Avenue study area, Maynard, Massachusetts.

and wetland deposits. The effects of alternative hydraulic conductivity values for layer 1 will be addressed as part of sensitivity testing.

Two hydraulic conductivity zones were assumed for layer 2 (fig. 33, table 5). A transmissive zone approximately 1,600 ft wide and 6,200 ft long was assumed for the area that encompasses the well site and residential wells to the east and west that responded strongly to pumping. This transmissive zone is not clearly related to mapped geologic features. The transmissivity of this zone determined from aquifer responses to pumping averaged about 2,400 ft^2/day. This value of transmissivity, when divided by a thickness for layer 2 of 360 ft, yields a hydraulic conductivity of about 7 ft/d. This value was increased to 14 ft/d during model calibration. A hydraulic conductivity value of 0.4 ft/d was assumed for bedrock elsewhere, which was the regional layer 1 value for the West Newbury model. This value was not adjusted during model calibration. Small water-level declines caused by cyclic pumping for residential use in well 68 indicated that high-transmissivity zones may be in other parts of the model area. Data were not available, however, to define dimensions of other transmissive zones, if present.

A uniform VCONT value was assigned throughout the model area, and was adjusted somewhat during model calibration to 0.0015 day^{-1} (table 5). This value is similar to values determined from leakage rates and head differences given in table 4. For transient simulations, a specific yield of 0.1 was assigned to layer 1 and a primary storage coefficient of $2x10^{-4}$ was assigned to layer 2. A secondary storage coefficient of 0.1 assigned to layer 2 is higher than a specific-yield value of 0.005 reported by Gburek and others (1999) for highly fractured rock but was inconsequential because the simulated head was never lowered below the top of the layer for any of the alternative models tested.

Alternative Models for Uncertainty Analysis

Initially, Case A adequately represented aquifer-system properties to estimate contributing areas and effects of pumping on streams and wetlands. During sensitivity tests, however, drains were added to more closely simulate heads in areally extensive wetlands. Case L, which includes additional drains in wetland areas, is a better representation of the ground-water system than Case A. Table 5 summarizes the properties used for model Cases A and L. Sensitivity tests demonstrated how alternative representations of the aquifer system affect model results. Table 6 summarizes the adjustments from Case A conditions that were made during sensitivity testing, the rationale for those adjustments, the effects on simulation of the aquifer test, and the effects on the contributing area to wells.

Model-Simulated Heads

For model Case L, simulated heads for the period of the aquifer test approximately match observed water levels at wells RW1, RW4,

Table 5. Summary of properties for model Cases A and L, Rockland Avenue study area, Maynard, Massachusetts

[ft, foot; ft/d, foot per day; ft^3/d, cubic feet per day; ft^2/d, square foot per day; gal/min, gallons per minute; day^{-1}, 1/day]

Model property	Cases A and L
Top and bottom of layers	
Layer 1	Bottom at 160 ft except northeast and southeast corners of model area where it was placed at 20 to 30 ft below nearest stream.
Layer 2	Bottom at altitude of -200 ft. Top at altitude of the bottom of layer 1.
Hydraulic conductivity, layer 1	10 ft/day
Hydraulic conductivity, layer 2	
Area	0.4 ft/d
High-transmissivity zone	14 ft/d
Vertical conductance	0.0015 day^{-1}
Storage coefficient, layer 1 (Specific yield)	0.1
Storage coefficient, layer 2	
Primary	0.0002
Secondary	0.1
Recharge	0.0055 ft/day (2 ft/yr)
Drain conductance	20,000 ft^2/d
Pumping rates	
Well RW2	62,090 ft^3/d (322 gal/min)
Well RW3	38,260 ft^3/d (199 gal/min)
Well RW5	50,530 ft^3/d (263 gal/min)

(fig. 36) and RW6 (not shown on fig. 36) in the area of the pumped wells. Starting heads representing average recharge conditions are somewhat lower than observed water levels at wells RW4 and RW6, possibly indicating a lower actual transmissivity than simulated transmissivity in that area. Similarly, a lower transmissivity near well 127 might have yielded higher simulated water levels and less response to pumping. At wells 66 and 67, simulated drawdowns that were less than observed drawdowns may be attributable to lateral variations in vertical hydraulic conductivity. During model calibration, a model-wide reduction of VCONT resulted in better head matches at these wells, but drawdowns were too great near the pumped wells. Further refinement of the model was not necessary for simulation of contributing areas because the recharge rate, rather than VCONT, constrained recharge to bedrock for the range of VCONT values that were reasonable for this aquifer system.

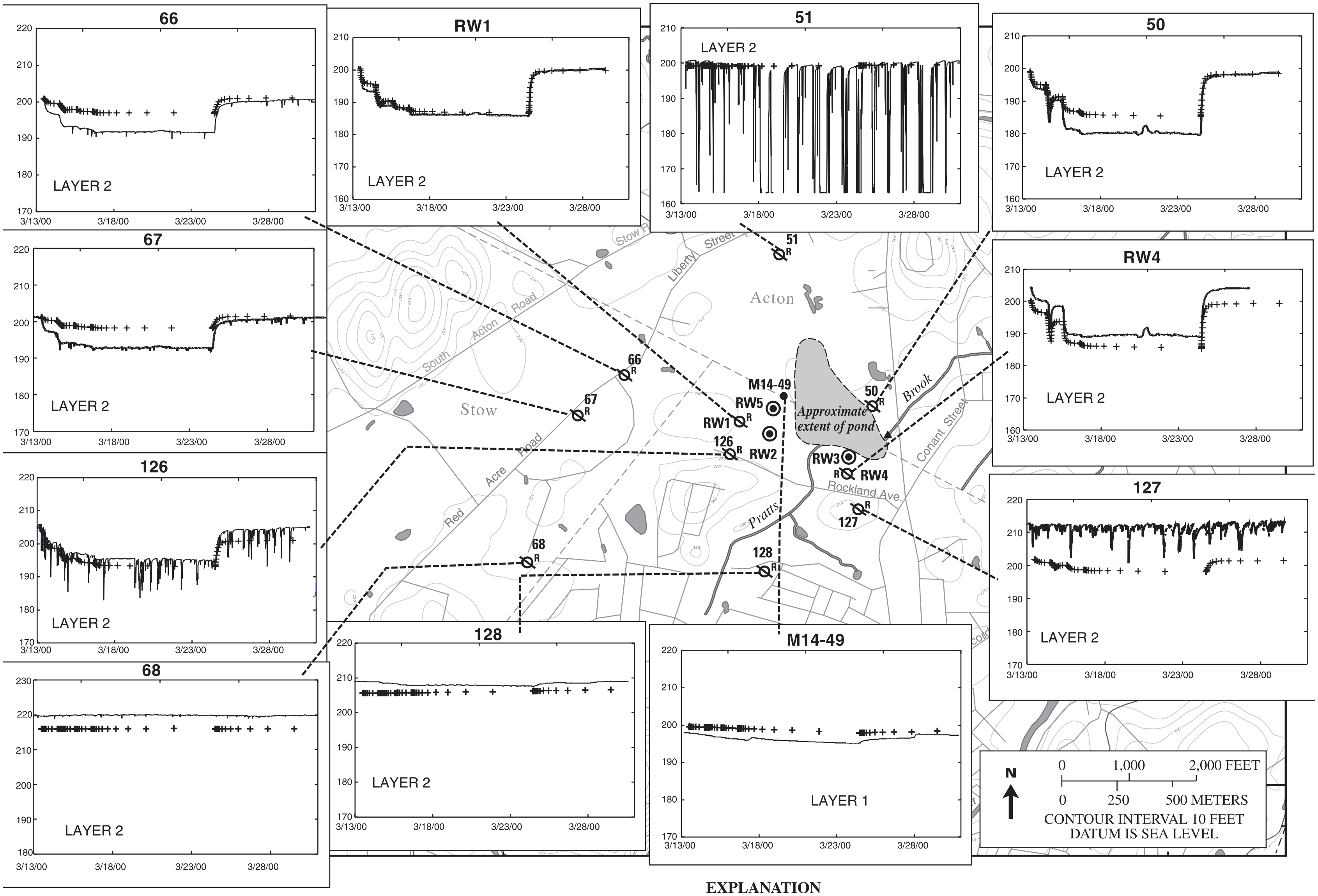
Simulated Contributing Areas to Wells

The simulated source area for Case L included much of the area within approximately 2,500 ft of the pumped wells (fig. 37) and an extended area about 4,000 ft to the west. Narrow branches extend to ground-water divides south and southwest of the pumped wells. The extension of the source area to the west may result partly from the extension of the transmissive zone in that direction. The contributing area, which is about 1.8 mi^2, includes areas of underflow and extends somewhat beyond the source area in places. The contributing area for Case A, which had fewer drains in wetland areas, is nearly the same as for Case L. If upstream discharges to drains were routed to downstream recharge areas, the size of the contributing area probably would decrease somewhat, but this condition was not simulated.

Table 6. Variations of model characteristics for alternative numerical models, rationale, and assessment, Rockland Avenue study area, Maynard, Massachusetts

[**Variation from Case A:** VCONT, vertical conductance or hydraulic conductivity divided by thickness. ft, foot; ft/d, foot per day; in/yr, inches per year]

Model alternative	Variation from Case A (see table 5 for Case A properties)	Rationale	Assessment
Case B	Reduce recharge to 18 in/yr.	Conceptually, the size of the contributing area is strongly controlled by recharge rates, and recharge rates are poorly constrained by available data.	Simulation of aquifer test was not affected. Contributing area was similar to Case A because flow paths were similar.
Case C	Reduce the hydraulic conductivity of layer 2 by 1/2 throughout the model area.	Available data indicated that a lower hydraulic conductivity value was possible.	Simulated drawdowns during aquifer test were not appreciably affected, except at well RW6 where the drawdown was too large. The contributing area was nearly the same as for Case A.
Case D	Reduce VCONT by 1/2.	Available data indicated that a lower value was possible, and would actually improve simulation of the aquifer test in some areas.	Drawdowns in layer 2 for simulated aquifer test generally were too large in the area of the pumped wells. Drawdowns at bedrock observation wells along the high-transmissivity zone were improved. Contributing area was not affected.
Case G	Reduce recharge to 12 in/yr.	This recharge rate was considered to be a lower limit.	Simulated drawdowns for the aquifer test were not affected. Starting heads generally were lower. Contributing area was appreciably enlarged.
Case H	Redirect drain inflow to recharge some wetland areas	Water that discharges to wetlands under pumping conditions may be available for recharge in areas near pumping wells.	No discernable effect on drawdown and contributing area was observed.
Case I	Increase the hydraulic conductivity of layer 1 by 10 times to 100 ft/d.	This was considered an upper limit value for sand and gravel and reflects conditions if the model area was covered by sand and gravel.	Starting heads generally were too low. Simulated drawdowns were not affected. The contributing area extended farther to the southwest and was narrower east to west.
Case J	Increase recharge to 30 in/yr.	This was considered an upper limit of possible recharge rates.	Starting heads and simulated drawdowns were not affected appreciably. The size of the source area was reduced but flow lines passed through much of the contributing area for Case A.
Case K	Delineate two hydraulic conductivity zones for layer 1: till = 3 ft/d; sand and gravel and swamp deposits = 30 ft/d.	The character and associated hydraulic properties of surficial materials may affect the contributing area, as shown by Case I.	Starting heads and drawdowns were not affected appreciably. The contributing area was nearly the same as for Case A.
Case L	Add drains to wetland areas.	Simulated head was too high in area between drains and some wetland areas. Additional drains should more closely simulate heads in wetland areas.	Simulated heads were closer to measured heads near wetlands. Contributing area was similar to Case A.



Base from U.S. Geological Survey Digital Elevation Model, Maynard, Massachusetts, 1:24,000, 1965, Universal Transverse Mercator, Zone 19

EXPLANATION

VERTICAL SCALE OF GRAPH IS WATER-LEVEL ALTITUDE, IN FEET ABOVE SEA LEVEL

— MEASURED HEAD

\+ MODEL-SIMULATED HEAD

M14-49 ● WELL OR PIEZOMETER LOCATION AND IDENTIFIER

128 WELL OR PIEZOMETER EQUIPPED WITH TRANSDUCER DURING AQUIFER TEST, AND IDENTIFIER

RW5 ◉ WELL PUMPED DURING AQUIFER TEST, AND IDENTIFIER

Figure 36. Observed and simulated heads at selected wells during aquifer testing for Case L conditions, Rockland Avenue study area, Maynard, Massachusetts.

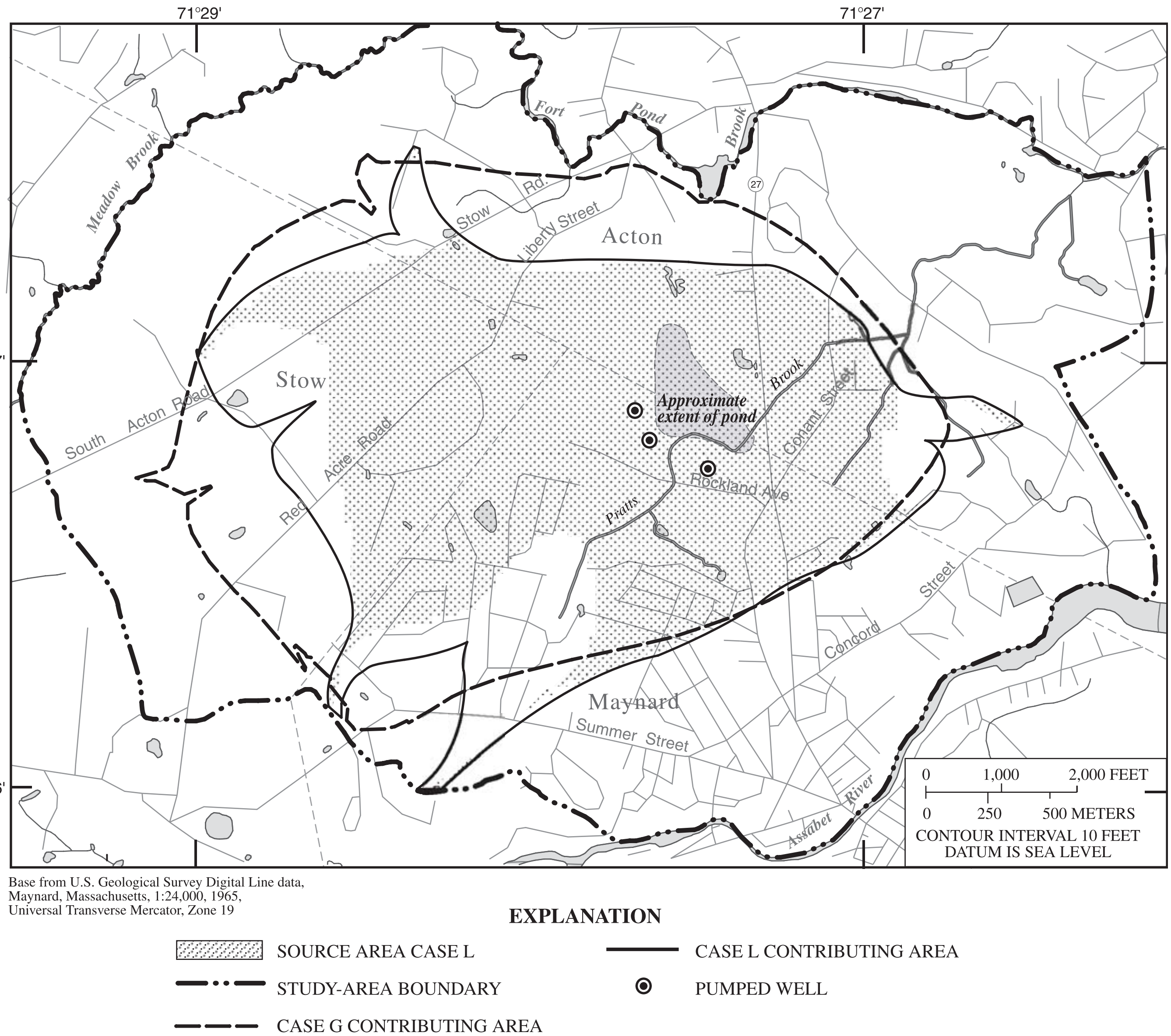


Figure 37. Simulated contributing areas to wells for model Cases L and G, Rockland Avenue study area, Maynard, Massachusetts.

Model Case G illustrates the effects of reducing recharge from 24 in/yr to 12 in/yr, and was simulated to illustrate a maximum likely contributing area (fig. 37). For this case, the contributing area shown on figure 37 expanded to about 2.3 mi^2 and was approximately the same as the source area. Case G had fewer drain nodes than Case L for wetland areas, but most of the drains for steady-state pumping were inactive, so that feature of the model did not appreciably affect results.

Model Case J (table 6) was selected to illustrate the effects of increasing recharge from 24 to 30 in/yr. This alternative was simulated to represent an upper plausible limit for the recharge rate. The source area was considerably smaller, but the contributing area is similar to Case L.

Case K was designed to test the effects of variable hydraulic-conductivity values for surficial materials. Hydraulic conductivity values of 3 ft/d were assigned for surface till and thick-till areas and 30 ft/d for sand and gravel and swamp areas (fig. 25). This change did not appreciably change contributing areas from Case A and Case L conditions. Because of uncertainties in the character of materials in the subsurface, and because the model appeared to be fairly insensitive to lateral variations in hydraulic properties, further refinement of the model to account for variable characteristics of surficial materials was unnecessary to accomplish the goals of this study.

Simulated Effects of Pumping on Streamflow and Wetlands

Pumping the three wells would reduce total streamflow for the model area by about the same amount pumped. Conceivably, some water that would be lost to evapotranspiration may be captured by lowering heads in the wetlands. The quantity of captured evapotranspiration, however, would be negligible and small relative to total diversions by pumping.

Model Case L was used to determine possible effects of pumping on streamflow for subbasins within the model area by examining simulated flow reductions to drains caused by pumping (fig. 38). Table 7 summarizes simulated flows to drains by subbasin and changes caused by pumping. Also included in table 7 are volumetric budgets for nonpumping and pumping conditions. Of the 784 gal/min pumped from the three wells, about 451 gal/min are diverted from subbasin C, which encompasses the wells. Most of the remaining 333 gal/min are diverted from subbasins B, D, and E. A small amount (11 gal/min) is diverted from constant head nodes on the model boundary.

Wetlands that might be affected by pumping were identified by comparing active drains for pumping and nonpumping conditions. Drains that become inactive for pumping conditions reflect wetlands that might be affected by pumping. The effects could include a permanent lowering of the water table below the land surface or a reduction in the length of time each year that water is at or near the land surface.

Most model drains within a radius of about 2,000 ft of the wells become inactive for steady-state pumping. Routing water that discharges to drains in upstream areas to downstream recharge areas could reduce the number of drains that become inactive. The water that discharges to drains in subbasin D for pumping conditions is potentially available for recharge in subbasin C along Pratts Brook. The amount of water actually lost to infiltration along the channel where it crosses area C probably would be less than the 260 gal/min that discharges to subbasin D for pumping conditions. The amount can be estimated from Darcy's Law. For example, if the stream is 10 ft wide and 3,000 ft long in a losing reach over silt and fine sand that has a vertical hydraulic conductivity of 0.5 ft/d, the maximum leakage rate, assuming a vertical hydraulic gradient of 1:1, is about 15,000 ft^3/d (78 gal/min). If the streambed sediments are less permeable than the aquifer materials, which is a likely condition, leakage rates would be reduced proportionally.

Table 7. Simulated volumetric budgets and flow to drains by subbasin for nonpumping and pumping conditions, Case L, Rockland Avenue study area, Maynard, Massachusetts

[NA, not applicable; ft^3/d, cubic feet per day; gal/min, gallons per minute]

Subbasin and volumetric budget	Non-pumping (ft^3/d)	Pumping (ft^3/d)	Difference	
			ft^3/d	gal/min
Flow to drains				
Subbasin A	57,667	57,543	124	0.6
Subbasin B	161,558	133,306	28,252	146.7
Subbasin C	103,116	16,266	86,849	451.1
Subbasin D	68,437	50,057	18,380	95.5
Subbasin E	82,394	67,706	14,688	76.3
Subbasin F	24,295	23,813	482	2.5
Total	497,466	348,691	148,776	772.7
Number of active drains	692	513	NA	NA
Total model volumetric budget				
IN:				
Constant head	844	865	-21	-0.1
Wells	0	0	0	.0
Drains	0	0	0	.0
Recharge	687,060	687,060	0	.0
Total in	687,904	687,925	-21	-.1
OUT:				
Constant head	190,435	188,358	2,077	10.8
Wells	0	150,880	-150,880	-783.7
Drains	497,466	348,691	148,776	772.8
Recharge	0	0	0	.0
Total out	687,901	687,929	-28	-.1
IN–OUT	3	-4	7	.0
Percent discrepancy	.00	.00	NA	NA

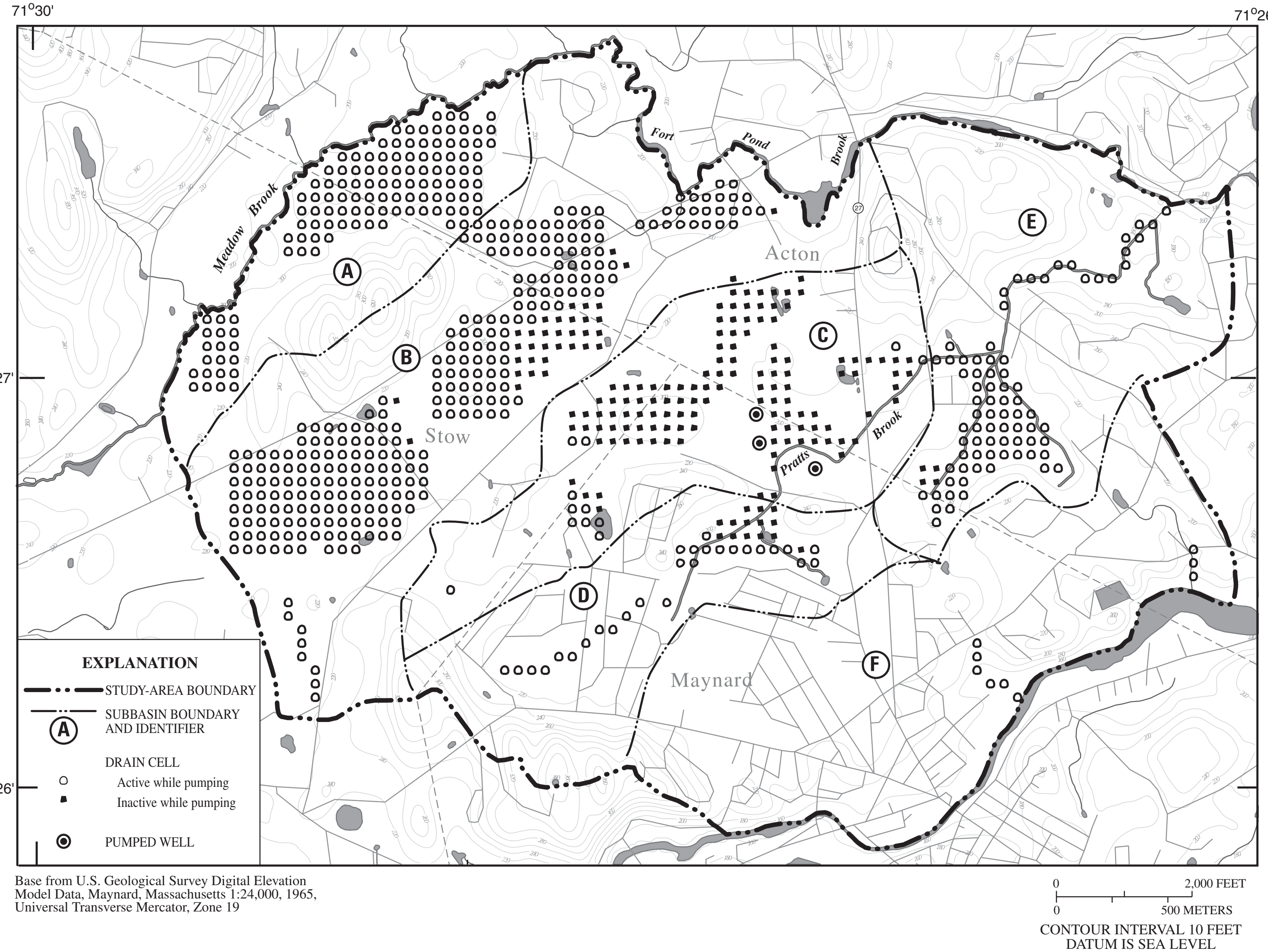


Figure 38. Drain cells inactivated by pumping for model Case L and subbasin boundaries, Rockland Avenue study area, Maynard, Massachusetts.