Evaluation of Strategies for Balancing Water Use and Streamflow Reductions in the Upper Charles River Basin, Eastern Massachusetts

By Jack R. Eggleston

Prepared in cooperation with the
MASSACHUSETTS EXECUTIVE OFFICE OF ENVIRONMENTAL AFFAIRS,
THE MASSACHUSETTS WATERSHED INITIATIVE

Water-Resources Investigations Report 03-4330

U.S. Department of the Interior
U.S. Geological Survey

U.S. Department of the Interior
Gale A. Norton, Secretary

U.S. Geological Survey
Charles G. Groat, Director

U.S. Geological Survey, Reston, Virginia: 2004
For sale by U.S. Geological Survey, Information Services
Box 25286, Denver Federal Center
Denver, CO 80225

For more information about the USGS and its products:
Telephone: 1-888-ASK-USGS
World Wide Web: http://www.usgs.gov/

Contents

Figures

TABLES

Conversion Factors, Datums, and Abbreviations

Multiply	By	To obtain
acre	0.004047	square kilometer (km^2)
acre-foot (acre-ft)	1,233.5	cubic meter (m^3)
cubic foot per second (ft^3/s)	0.0283	cubic meter per second (cm/s)
cubic foot per second per square mile [(ft^3/s)/mi^2]	0.01093	cubic meter per second per square kilometer ($m^3/s/km^2$)
foot (ft)	0.3048	meter (m)
foot per day (ft/d)	0.3048	meter per day (m/d)
gallons per minute (gal/min)	0.0038	cubic meters per minute (m^3/min)
inch (in.)	2.54	centimeter (cm)
inch per year (in/yr)	2.54	centimeter per year (cm/yr)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)
square foot (ft^2)	0.0929	square meter (m^2)

Vertical coordinate information is referenced to the North American Vertical Datum of 1929 (NGVD 29).

Horizontal coordinate information is referenced to the North American Datum of 1983 (NAD 83).

ABF	Aquatic Base Flow
CRPCD	Charles River Pollution Control District Treatment Facility
CRWA	Charles River Watershed Association
EOEA	Massachusetts Executive Office of Environmental Affairs
MADEM	Massachusetts Department of Environmental Management
MADEP	Massachusetts Department of Environmental Protection
MOVE.1	Maintenance of Variance Extension, type 1, method of correlation
MODFLOW	Modular Three-Dimensional Finite-Difference Ground-Water Flow Model
MWRA	Massachusetts Water Resources Authority
PCG2	Preconditioned Conjugate-Gradient 2
TAC	Technical Advisory Committee
USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey
WMA	Water Management Act

Evaluation of Strategies for Balancing Water Use and Streamflow Reductions in the Upper Charles River Basin, Eastern Massachusetts

By Jack R. Eggleston

Abstract

The upper Charles River basin, located 30 miles southwest of Boston, Massachusetts, is experiencing water shortages during the summer. Towns in the basin have instituted water-conservation programs and water-use bans to reduce summertime water use. During July through October, streamflow in the Charles River and its tributaries regularly falls below 0.50 cubic foot per second per square mile, the minimum streamflow used by the U.S. Fish and Wildlife Service as its Aquatic Base Flow standard for maintaining healthy freshwater ecosystems.

To examine how human water use could be changed to mitigate these water shortages, a numerical ground-water flow model was modified and used in conjunction with response coefficients and optimization techniques. Streamflows at 10 locations on the Charles River and its tributaries were determined under various water-use scenarios and climatic conditions. A variety of engineered solutions to the water shortages were examined for their ability to increase water supplies and summertime streamflows.

Results indicate that although human water use contributes to the problem of low summertime streamflows, human water use is not the only, or even the primary, cause of low flows in the basin. The lowest summertime streamflows increase by 12 percent but remain below the Aquatic Base Flow standard when all public water-supply pumpage and wastewater flows in the basin are eliminated in a simulation under average climatic conditions. Under dry climatic conditions, the same measures increase the lowest average monthly streamflow by 95 percent but do not increase it above the Aquatic Base Flow standard.

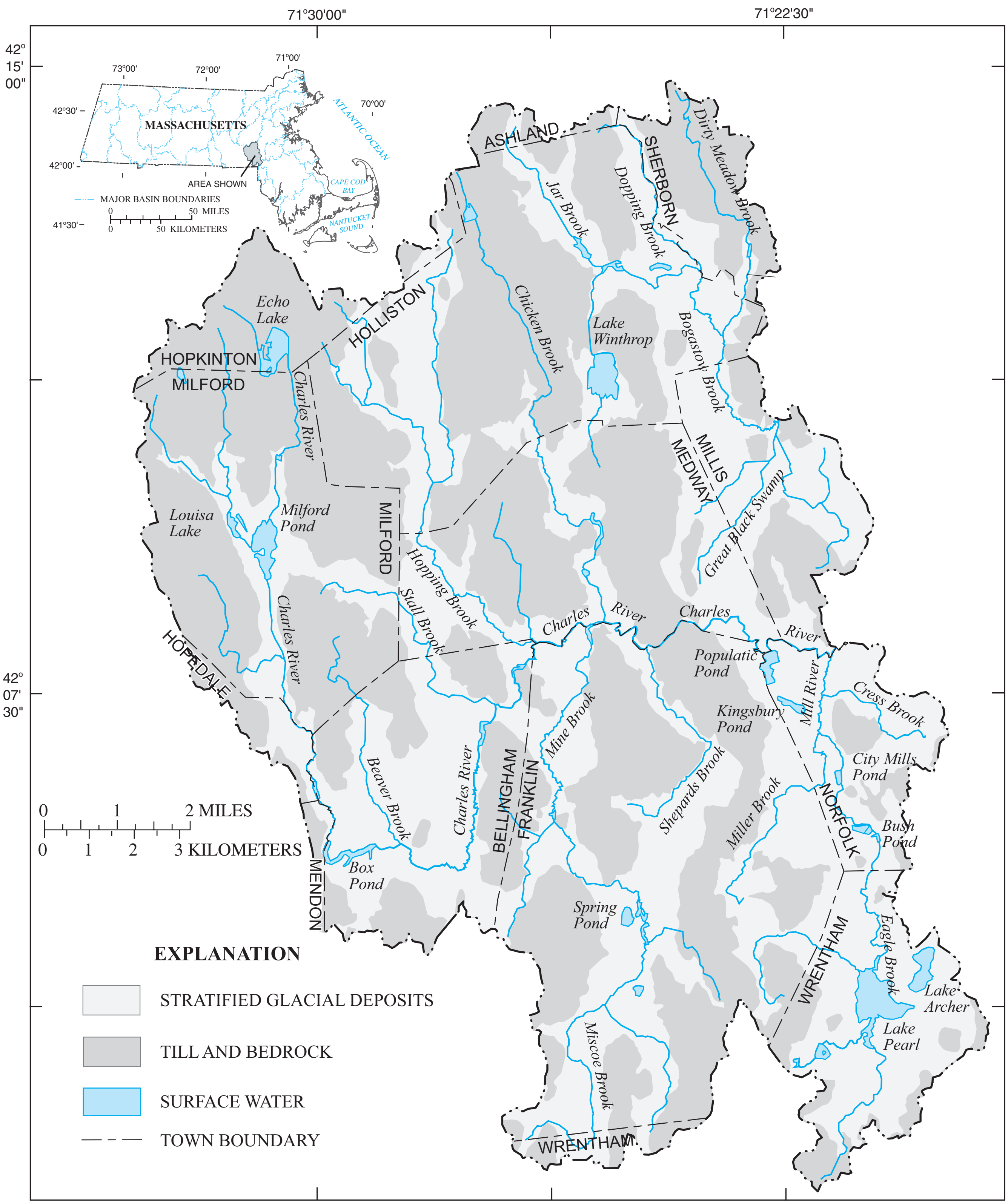
The most promising water-management strategies to increase streamflows and water supplies, based on the study results, include wastewater recharge to the aquifer, altered management of pumping well schedules, regional water-supply sharing, and water conservation. In a scenario that simulated towns sharing water supplies, streamflow in the Charles River as it exits the basin increased by 18 percent during July through September and an excess water-supply capacity of 13.4 cubic feet per second, above and beyond average use, would be available to all towns in the basin. These study results could help water suppliers and regulators evaluate strategies for balancing ground-water development and streamflow reductions in the basin.

Introduction

The upper Charles River basin, which is 30 mi southwest of Boston (fig. 1), is undergoing increasing demand on its water resources caused by population growth of 15 percent from 1990 to 2000. As populations have grown, communities in the basin have withdrawn more ground water, lowering pond levels and depleting streamflow in the upper Charles River and its tributaries (Bouck, 1998). Future water demands are likely to increase as development continues. The hydrologic system will be further altered if proposed water projects, including new supply wells, a surface-water withdrawal, and sewers, are built.

Streamflow in the summertime throughout the basin commonly falls below the Aquatic Base Flow (ABF) criteria for a healthy aquatic ecosystem (U.S. Fish and Wildlife Service, 1981). During times of low rainfall, streamflow is even lower and some smaller tributary streams go dry. Ecological concerns about low streamflow and poor water quality in the upper Charles River basin are associated with the perception that human influences, primarily ground-water pumping and land-use change, are to blame for the water problems.

Water managers for towns in the upper Charles River basin have increased pumping, imposed water restrictions, and in some instances trucked in water from outside the basin, to meet increasing water demands. Compared to water imported from outside the basin, water from local supplies is typically less expensive and is not affected by the same regulatory constraints. There currently (2003) is little sharing of drinking-water supplies between towns, and each town in the basin obtains the bulk of its water from within its town boundaries. The lack of a regional water-management approach is likely contributing to water-supply shortages in many towns, including Milford, Norfolk, Holliston, and Franklin (fig. 1).



From U.S. Geological Survey and MassGIS data sources, Geographic Projection, Spheroid GRS 1980, Datum NAD 83

Figure 1. Location and surficial geology of the study area, upper Charles River basin, eastern Massachusetts.

Purpose and Scope

This project was initiated by the U.S. Geological Survey (USGS) and the Massachusetts Executive Office of Environmental Affairs (EOEA) to improve understanding of the hydrologic system and explore strategies for mitigating water shortages in the upper Charles River basin. The study was carried out during 2002–03. This report demonstrates how optimization methods and hydrologic response coefficients can be used to solve complex water-resource management problems involving competing needs for water from multiple users and sensitive aquatic ecosystems. The methods and analysis presented should provide new hydrologic analysis tools for water-resource managers and regulators. To address the issues of water-supply shortages and low streamflow in the upper Charles basin, this report includes hydrologic analysis of regional water-management scenarios under dry climatic conditions and a variety of engineered modifications to human water use in the basin.

In this study, a numerical flow model that was developed by DeSimone and others (2002) using the USGS ground-water modeling code MODFLOW-2000 (Harbaugh and others, 2000), was updated and modified. Hydrologic response coefficients derived from the updated numerical ground-water model were used to simulate the combined stream-aquifer system and to analyze a variety of water-resource management scenarios. Optimization methods were used for more complex scenarios to determine the best management practices for meeting water demand and minimum streamflow requirements.

Previous Investigations

Bouck (1998) provides a summary of water-supply issues faced by towns in the basin. In a comprehensive hydrologic study, DeSimone and others (2002) developed a numerical model to simulate ground-water flow and stream base flow in the upper Charles River basin. The comprehensive hydrologic study included compilation and analysis of data describing climate, land use, streamflow, base flow, ground-water levels, pond levels, water withdrawals, water use, septic discharge, wastewater discharge, hydrogeology, and interactions between ground water and surface water. The numerical flow model developed in that study is modified for use in this study.

Joint simulation-optimization methods have previously been used to analyze water-resource management alternatives for shallow stream-aquifer systems (Reichard, 1995; Barlow and Dickerman, 2001; Barlow and others, 2003). An extensive body of literature describes other applications of optimization methods to water-resource management (for example, Gorelick, 1983; Mays, 1997; Heathcote, 1998; Nishikawa, 1998). The hydrologic study of the upper Charles River basin by DeSimone and others (2002) included an optimization analysis of a small portion of the study area. This study extends the optimization study to the entire basin and addresses a greater variety of water-resource management questions.

Description of Study Area

The upper Charles River basin is 30 mi southwest of Boston, MA, and has an area of 105 mi^2 consisting mostly of suburban communities (fig. 1). Thirteen towns (Ashland, Bellingham, Franklin, Holliston, Hopedale, Hopkinton, Milford, Medway, Mendon, Millis, Norfolk, Sherborn, and Wrentham) are at least partially within the study area. Seven of the towns (Bellingham, Franklin, Holliston, Medford, Medway, Norfolk, and Wrentham) withdraw ground water within the basin for municipal drinking-water supplies. The population of the study area increased 15 percent, from approximately 80,500 to 93,000, between 1990 and 2000. The basin has moderate topographic relief, with land elevations ranging from about 120 to 550 ft (NGVD29). More descriptive information about the basin can be found in DeSimone and others (2002).

Acknowledgments

The author thanks the members of this project's technical advisory committee, composed of town water officials, engineering consultants, conservation group members, state environmental officials, and USGS employees, who met regularly and provided assistance to this project. In addition, Nigel Pickering of the Charles River Watershed Association (CRWA) provided code for constructing optimization equations and assisted with analysis of land use and recharge. Town water officials, particularly Donald DiMartino of Bellingham, William Fitzgerald of Franklin, Steven Davis of Walpole, and Carolyn Dykema and Jane Pierce of Holliston, provided useful information about town water management. Leslie DeSimone of the USGS generously helped with modeling, analysis, and logistics of the project. Donald Walter of the USGS provided valuable assistance with the numerical flow model. This report was improved by the careful attention of reviewers.

Basin Hydrology

Mean monthly precipitation in the upper Charles River basin shows little seasonal variation (fig. 2) based on data from a climate station in Medway (fig. 3). From 1957 to 2000, precipitation in the basin averaged 46.6 in/yr, and daily average temperatures ranged from 24.5°F in January to 70.5°F in July, also based on Medway climate data (National Oceanic and Atmospheric Administration, 2001).

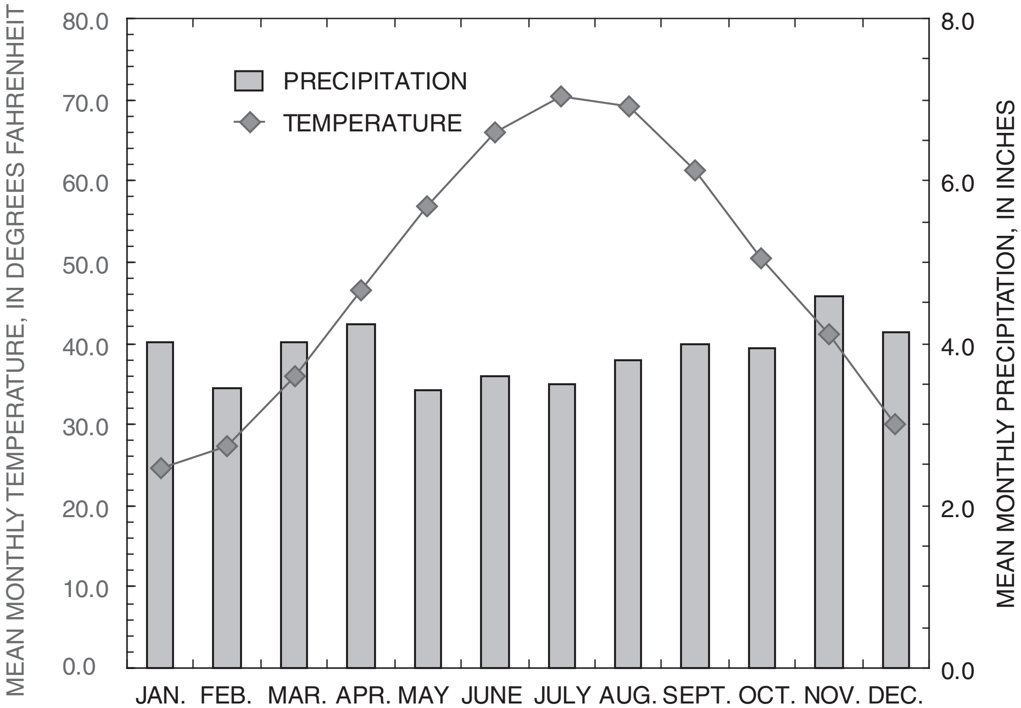


Figure 2. Monthly average precipitation (1957–2000) and temperature (National Oceanic and Atmospheric Administration, 2001) for climate station 199316 in Medway, eastern Massachusetts.

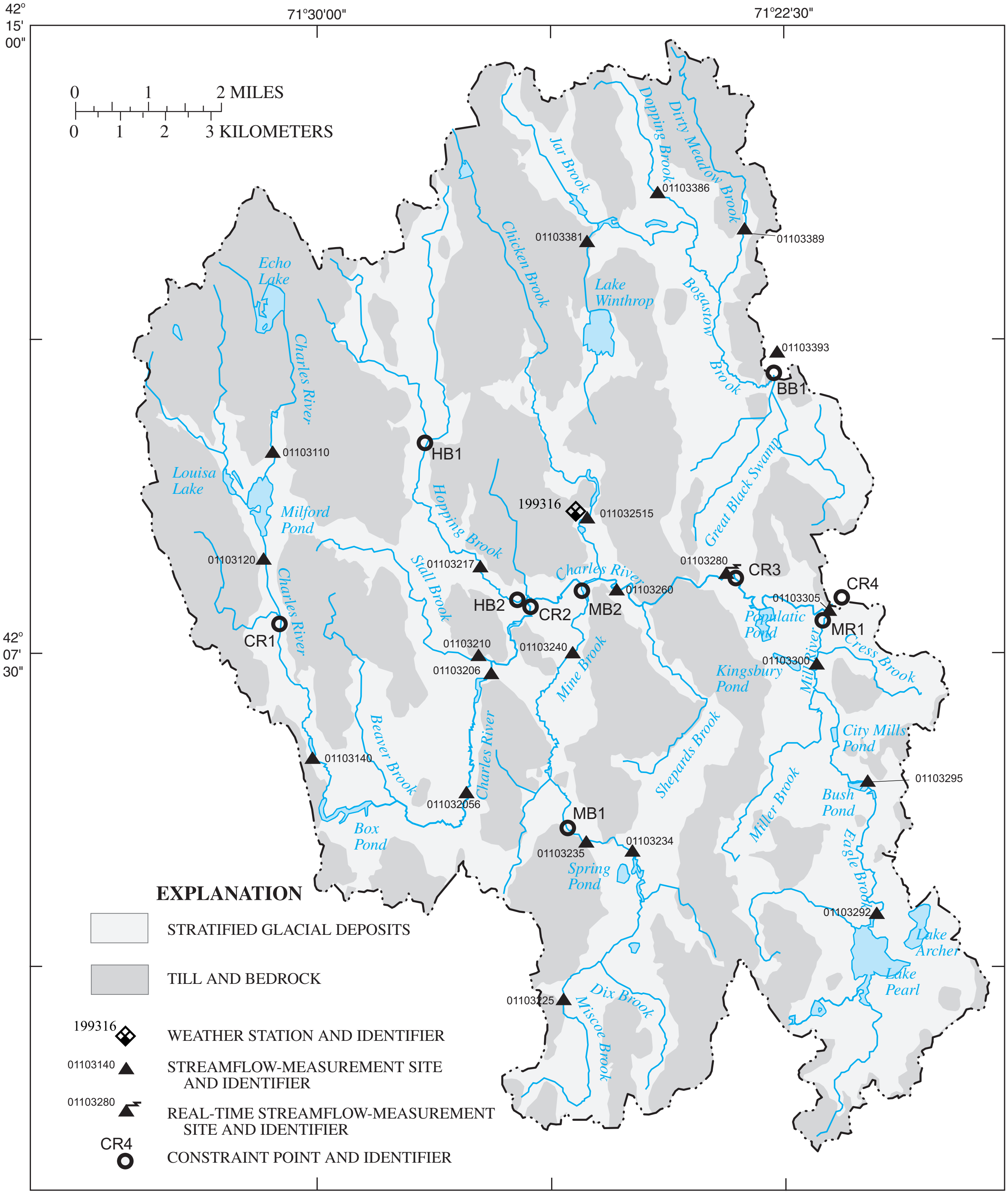
Ground Water

Ground water is the primary source of drinking water for towns in the basin. Aquifers in the study area comprise thin discontinuous sand and gravel units, primarily stratified glacial deposits (fig. 1). The aquifers, which occupy shallow valleys that trend northwest to southeast, are generally less than 70 ft thick with a maximum thickness of 130 ft. The aquifers are mostly unconfined and overlie bedrock or low-permeability glacial till. Hydraulic conductivity of the aquifers is high, ranging from 68 to 247 ft/d (DeSimone and others, 2002, table 1). The streams and underlying aquifers in the upper Charles River basin are in close connection and form a system that supplies nearly all drinking water to towns and residents. The high conductivity of aquifers and streambeds allows rapid exchange of water between aquifers and streams, and thus streamflow responds quickly to ground-water pumping. The aquifers have low storage capacity because they are thin, unconfined, highly conductive, and sparsely distributed.

Input to the ground-water system is primarily from recharging precipitation, with minor contributions from septic systems and recharging streams. A small amount of runoff infiltrates to ground water through detention basins, which are generally required in new residential developments. No wastewater recharge facilities, either infiltration basins or injection wells, are in the basin, but several such projects have been proposed. In a few areas, streams recharge to the aquifers; but overall, stream leakage into aquifers is only 5 percent of ground-water discharge to streams (DeSimone and others 2002).

Streamflow

The northeast portion of the study area is drained by Bogastow Brook and its tributaries, and the rest of the study area is drained by the Charles River and its tributaries (fig. 1). Bogastow Brook flows into the Charles River downstream of the study area. The only actively managed surface-water reservoir is Echo Lake at the headwaters of the Charles River in Milford and Hopkinton. During winter and spring, Echo Lake receives water pumped from the Charles River just upstream of Milford Pond, a dammed lake on the Charles River about 3 mi downstream from Echo Lake. During summer months water is drawn from Echo Lake for Milford’s water supplies. Other dams in the study area create impoundments that are not actively managed for water supply. An Army Corp of Engineers (1994) inventory of dams lists 17 dams in the basin that are at least 6 ft high and impound at least 50 acre-ft of water.



From U.S. Geological Survey and MassGIS data sources, Geographic Projection, Spheroid GRS 1980, Datum NAD 83

Figure 3. Streamflow measurement sites and constraint points, upper Charles River basin, eastern Massachusetts. (Modified from fig. 7, DeSimone and others, 2002.)

Table 1. Base flow under average climatic conditions and average water-use conditions (1989–98), upper Charles River basin, eastern Massachusetts.

[Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	9.6	10.4	13.3	13.5	9.0	4.1	1.7	1.8	1.7	3.9	6.2	9.4
CR2	42.5	45.0	55.3	54.0	39.3	23.5	12.4	12.6	12.5	21.8	32.3	41.2
CR3	118.8	134.4	174.1	174.9	110.4	62.0	27.9	24.2	22.7	45.5	82.6	115.6
CR4	157.6	167.4	200.4	198.1	146.4	88.4	50.7	51.7	49.8	84.1	120.3	155.1
BB1	38.6	41.1	56.5	56.4	35.4	15.4	5.9	6.2	5.8	14.5	25.3	37.4
HB1	18.3	22.6	31.3	30.5	15.6	4.1	1.0	1.1	1.1	4.1	8.7	17.7
HB2	27.0	33.4	46.3	45.2	23.1	6.1	1.5	1.7	1.7	6.0	12.9	26.2
MB1	19.1	20.9	24.0	23.7	17.9	10.4	5.8	6.0	6.0	10.1	14.0	18.8
MB2	34.6	38.6	46.0	45.7	31.9	18.0	9.2	8.6	8.4	15.6	23.8	33.6
MR1	35.4	38.0	42.5	42.3	33.5	23.2	15.0	14.2	14.0	21.0	27.7	34.6
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.85	0.93	1.18	1.20	0.80	0.36	0.15	0.16	0.15	0.35	0.55	0.84
CR2	1.67	1.77	2.17	2.13	1.55	.92	.49	.50	.49	.86	1.27	1.62
CR3	1.82	2.06	2.67	2.68	1.69	.95	.43	.37	.35	.70	1.27	1.77
CR4	1.88	2.00	2.39	2.36	1.75	1.05	.61	.62	.59	1.00	1.43	1.85
BB1	1.85	1.98	2.72	2.71	1.70	.74	.29	.30	.28	.70	1.22	1.80
HB1	2.45	3.03	4.20	4.10	2.09	.55	.14	.15	.15	.55	1.17	2.38
HB2	2.45	3.03	4.20	4.10	2.09	.55	.14	.15	.15	.55	1.17	2.38
MB1	1.90	2.08	2.39	2.37	1.78	1.03	.58	.60	.60	1.01	1.40	1.88
MB2	2.17	2.43	2.89	2.88	2.01	1.13	.58	.54	.53	.98	1.50	2.11
MR1	2.21	2.37	2.66	2.65	2.10	1.45	.94	.89	.88	1.32	1.73	2.16

Low streamflow during summer months is a problem in the study area. The months of July through September, when streamflow is typically at its lowest, are the most critical months of concern and are the focus of analysis in this study. Some tributary streams go dry during summer months and periods of low rainfall. There are no long-term surface-water records for the upper Charles River basin, but daily streamflow records are available from a streamflow-gaging station in Medway (01103280) that was installed in 1997 (fig. 3). In each of the four summers (1998–2001) for which records are available from the Medway station, streamflow dropped below not only the U.S. Fish and Wildlife Service (USFWS) ABF standard of 0.50 $ft^3/s/mi^2$ but also below the 0.21 $ft^3/s/mi^2$ low-flow standard used by the Massachusetts Department of Environmental Protection (MADEP) in wastewater-discharge and ground-water-withdrawal permits within the Charles River basin. In the summers of 1998–2001, daily flow at the Medway station had low values of 0.20, 0.03, 0.12, and 0.12 $ft^3/s/mi^2$, respectively.

Base flow, rather than streamflow, is the focus of this study because base flow is the portion of streamflow originating from ground-water discharge to streams. Base flow does not include surface runoff and quickflow, the portion of streamflow that increases quickly in response to precipitation (Hornberger and others, 1998). The numerical flow model computes base flow but does not calculate surface runoff or quickflow. Wastewater discharge to streams is included in the base-flow numbers in this study. For the water year 2000, DeSimone and others (2002) estimate that 88 percent of flow in the Charles River is base flow and, during periods of drought, the base-flow portion is 100 percent if wastewater effluent is not included.

Estimation of Base Flow

Information about average base-flow conditions is needed as a starting point for this study's analysis of water-resource management alternatives. The lack of long-term streamflow data in the basin requires that base-flow statistics be estimated. Because uncertainty associated with base-flow values produced by the numerical flow model was high, base-flow statistics were estimated by the Maintenance of Variance Extension, type 1 (MOVE.1) method (Ries and Friesz, 2000). The MOVE.1 method relates base-flow statistics at stations with relatively short-term streamflow records to relatively long-term base-flow statistics from nearby stations. Base flow at long-term stations is calculated by means of hydrograph separation (DeSimone and others, 2002). The same partial-record streamflow data, long-term data, and averaging weights were used in this study as were used by DeSimone and others (2002). Uncertainty in the MOVE.1 estimated base-flow values is low and is given in table 8 of DeSimone and others (2002).

Several options for water withdrawal and wastewater disposal are analyzed in this study to determine how they affect base flow. The response of base flow to pumping and wastewater disposal is calculated by the response coefficient method. The response coefficient method allows base-flow changes caused by pumping and other hydrologic stresses to be calculated with linear equations. This method is used throughout the study because it gives more reliable results than the numerical flow model and because it can easily be implemented in conjunction with the optimization analysis.

Two flow statistics, monthly average base flow and 90-percent duration low flow (base flow exceeded 90 percent of the time), were estimated for the streamflow-measurement sites shown in fig. 3. The same statistics were then calculated for sites where no partial-record streamflow data were available by use of equation 1 (Ries and Friesz, 2000).

$$\hat{Y}_j = \hat{Y}_i \frac{A_j}{A_i}, \qquad (1)$$

where

$\hat{Y}_j$ = base-flow statistic at constraint point;
$\hat{Y}_i$ = base-flow statistic at nearby partial-record station;
A_j = drainage area of the constraint point; and
A_i = drainage area of the partial-record station.

Equation 1 was used to estimate flow statistics at 10 constraint points (fig. 3), which are locations used to impose minimum flow standards in this study.

The values in table 1 show that, under average water use and average precipitation conditions, base flow drops to low levels during the summer in some streams. Base flow is especially low from July to September and on small tributary streams. Flow is lower in streams in the northern portion of the study area, Hopping Brook (HB1 and HB2) and Bogastow Brook (BB1), most likely because the aquifer there is more limited in extent and has less ground-water storage available to augment summertime base flow. To characterize low-flow conditions at a particular location, this study uses the lowest monthly flow out of the 12 months. The low monthly flow values at the 10 constraint points range from 0.14 $ft^3/s/mi^2$ in July on Hopping Brook at HB1 and HB2 to 0.88 $ft^3/s/mi^2$ in September on the Mill River at MR1. Taking the lowest monthly base flow value from each of the 10 constraint points and averaging gives a mean value of 0.41 $ft^3/s/mi^2$. The lowest flows at the 10 constraint points all occur either in July or September, with 4 points having the lowest flows in July and 6 points having the lowest flows in September.

The 90-percent duration monthly base-flow values represent dry climatic conditions (table 2). Monthly base flow has been lower than these values once out of every 10 months on average. The lowest average monthly base flow under average pumping and dry climatic conditions (90-percent flow duration) occurs in September at all 10 constraint points and ranges from 0.00 $ft^3/s/mi^2$ on Hopping Brook (HB1 and HB2) to 0.32 $ft^3/s/mi^2$ on the Mill River (MR1). For all 10 constraint points under dry climatic conditions, the lowest monthly flows average 0.10 $ft^3/s/mi^2$.

To examine how human activities affect low-flow conditions, base flow under "no-pumping" conditions (tables 3 and 4) is calculated. The effects of water withdrawals and wastewater and septic discharges are subtracted from average base flow to arrive at no-pumping base flow. This flow might be termed "natural" except that other human influences such as dams, land-use changes, infiltration to sewer lines, and anthropogenic climatic changes (Huff and Changnon, 1986), are still affecting flow. Overall, base flow increases under no-pumping conditions, but the change is not uniform, varying with month and stream location. The percentage change is largest during summer months and for smaller streams. For example, at location CR1 on the upper Charles River, August monthly base flow increases by 65 percent under no-pumping conditions. A few of the constraint points have less flow under no-pumping conditions. This occurs in the Mill River (MR1) and Hopping Brook (HB2) because these subbasins do not receive their normal septic discharge under no-pumping conditions.

Under no-pumping conditions and dry climatic conditions (90-percent flow duration), Hopping Brook (HB1 and HB2) is essentially dry (flow less than or equal to 0.02 ft^3/s) from July through October. Base flow falls below 0.21 $ft^3/s/mi^2$ during at least 1 month at 7 of the 10 constraint points (table 4). Removing water withdrawals and wastewater and septic discharges causes large percentage flow increases in some of the tributary streams. For example, at constraint point CR1 on the upper Charles River, August monthly flow increases by 500 percent when no-pumping conditions are imposed under 90-percent low-flow conditions.

There is ongoing discussion by state officials, town officials, and conservationists about setting minimum flow standards for streams in the upper Charles River basin. The MADEP has previously used a low-flow standard of 0.21 $ft^3/s/mi^2$ to regulate wastewater disposal and ground-water withdrawals in the upper Charles River basin (Bruce Bouck, MADEP, written commun., 2002). The USFWS uses the ABF method for New England (U.S. Fish and Wildlife Service, 1981), to set minimum streamflows for maintaining healthy streams downstream of controlled dams. The default ABF value, 0.50 $ft^3/s/mi^2$, is applied if no streamflow data are available. On the basis of streamflow data from the nearest long-term flow station on the Charles River at Dover, about 7 mi downstream from the study area, a more site-specific ABF value, calculated as the median of the monthly mean flows for August (U.S. Fish and Wildlife Service, 1981), is 0.41 $ft^3/s/mi^2$.

Table 2. Base flow under dry (90-percent low flow) climatic conditions and average water-use conditions (1989–98), upper Charles River basin, eastern Massachusetts.

[Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	3.56	4.97	9.63	6.62	4.23	1.41	0.42	0.24	0.21	0.47	1.38	2.63
CR2	21.88	28.01	41.91	31.68	23.25	11.27	4.90	3.61	3.37	5.97	13.22	19.44
CR3	33.79	47.78	80.55	53.94	36.34	14.66	4.73	3.45	3.37	6.02	15.99	25.99
CR4	81.76	104.87	156.25	120.31	86.90	44.73	20.35	14.71	13.94	22.06	49.95	72.67
BB1	13.11	19.34	37.88	25.67	15.11	5.32	1.44	.67	.72	1.71	5.86	11.49
HB1	4.23	6.95	18.95	9.33	5.12	0.66	.09	.05	.03	.13	.86	2.08
HB2	6.26	10.28	28.04	13.80	7.57	0.98	.13	.07	.05	.20	1.27	3.08
MB1	10.32	12.74	19.37	14.42	11.21	4.81	2.11	1.63	1.43	2.48	5.26	7.64
MB2	16.08	21.02	34.82	24.51	18.04	6.72	2.62	1.96	1.81	3.11	6.85	10.91
MR1	21.50	25.60	35.48	28.28	23.16	12.21	6.60	5.45	5.19	7.38	12.31	16.69
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.32	0.44	0.86	0.59	0.38	0.13	0.04	0.02	0.02	0.04	0.12	0.23
CR2	.86	1.10	1.65	1.25	.91	.44	.19	.14	.13	.23	.52	.76
CR3	.52	.73	1.24	.83	.56	.23	.07	.05	.05	.09	.25	.40
CR4	.98	1.25	1.86	1.44	1.04	.53	.24	.18	.17	.26	.60	.87
BB1	.63	.93	1.82	1.23	.73	.26	.07	.03	.03	.08	.28	.55
HB1	.57	.93	2.54	1.25	.69	.09	.01	.01	.00	.02	.12	.28
HB2	.57	.93	2.54	1.25	.69	.09	.01	.01	.00	.02	.12	.28
MB1	1.03	1.27	1.93	1.44	1.12	.48	.21	.16	.14	.25	.53	.76
MB2	1.01	1.32	2.19	1.54	1.14	.42	.16	.12	.11	.20	.43	.69
MR1	1.34	1.60	2.22	1.77	1.45	.76	.41	.34	.32	.46	.77	1.04

If the median of daily mean flows for August is used to calculate the ABF to remove the effects of infrequent storm events, a lower flow value of 0.32 $ft^3/s/mi^2$ results (Kulik, 1990; Ries and Friesz, 2000). Low-flow standards other than the ABF, such as the wetted-perimeter method or R2Cross method could also potentially be used to set regulatory flow criteria (Armstrong and others, 2001).

Current average base flow in the basin, shown in table 1, falls below even the lowest of these standards, 0.21 $ft^3/s/mi^2$, at 3 of the 10 constraint points (CR1, HB1, HB2) from July to September. Under dry climatic conditions (90-percent flow duration) base flow is at or below 0.21 $ft^3/s/mi^2$ on every stream but the Mill River (MR1) during August and September (table 2). Where streams exit the model, at constraint points CR4 and BB1, average September base flow is 0.59 and 0.28 $ft^3/s/mi^2$, respectively, and in dry years (90-percent low-flow conditions) is 0.17 and 0.03 $ft^3/s/mi^2$.

A potential option for water-resources managers of the upper Charles River basin is to maintain base flow above a standard such as the ones discussed above. The base-flow numbers in tables 1–4, however, indicate that this will require more water than is currently in the streams. For instance, under no-pumping conditions and dry climatic conditions (90-percent duration flow), base flow at CR4 during September averages 0.18 $ft^3/s/mi^2$, and in this case the towns receive no water supplies.

Table 3. Base flow under average climatic conditions and no water-supply or wastewater pumping, upper Charles River basin, eastern Massachusetts.

[Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
Base flow (ft^3/s)												
CR1	10.8	11.6	14.3	14.5	10.0	5.0	2.8	2.9	2.8	5.1	7.5	10.7
CR2	43.4	46.1	56.4	55.1	40.8	24.9	13.7	13.9	13.2	22.6	33.2	42.3
CR3	120.4	136.1	175.8	176.6	112.8	63.3	30.4	26.8	24.3	47.2	84.3	117.4
CR4	158.5	168.3	201.3	198.8	148.4	90.6	53.2	54.6	51.0	85.4	121.6	156.4
BB1	39.0	41.6	57.0	56.9	36.0	16.2	6.8	7.1	6.5	15.0	25.8	37.9
HB1	18.2	22.6	31.2	30.4	15.6	4.0	1.0	1.1	1.1	4.1	8.7	17.7
HB2	26.8	33.3	46.1	45.0	22.9	5.9	1.3	1.5	1.5	5.9	12.7	26.1
MB1	20.6	22.4	25.5	25.3	19.6	12.1	7.6	7.8	7.6	11.8	15.6	20.3
MB2	36.3	40.3	47.7	47.4	33.8	20.0	11.3	10.8	10.5	17.7	25.7	35.3
MR1	34.8	37.3	41.8	41.6	33.1	22.9	14.9	14.3	13.9	20.7	27.3	34.1
Base flow per drainage area ($ft^3/s/mi^2$)												
CR1	0.96	1.03	1.27	1.29	0.89	0.45	0.25	0.26	0.25	0.45	0.66	0.95
CR2	1.71	1.81	2.22	2.17	1.60	.98	.54	.55	.52	.89	1.31	1.66
CR3	1.85	2.09	2.70	2.71	1.73	.97	.47	.41	.37	.72	1.29	1.80
CR4	1.89	2.01	2.40	2.37	1.77	1.08	.63	.65	.61	1.02	1.45	1.87
BB1	1.87	2.00	2.74	2.73	1.73	.78	.33	.34	.31	.72	1.24	1.82
HB1	2.44	3.03	4.19	4.09	2.09	.54	.13	.15	.15	.55	1.17	2.38
HB2	2.43	3.02	4.18	4.08	2.08	.53	.12	.14	.14	.53	1.15	2.36
MB1	2.06	2.23	2.54	2.52	1.95	1.21	.76	.78	.76	1.18	1.55	2.03
MB2	2.29	2.54	3.00	2.99	2.13	1.26	.71	.68	.66	1.11	1.62	2.22
MR1	2.17	2.34	2.62	2.61	2.07	1.43	.93	.90	.87	1.30	1.71	2.14

Water Use and Budget

Water is redistributed within the basin by municipal water-supply and wastewater-disposal systems (fig. 4 and table 5). In addition, golf clubs, power plants, farms, businesses, and private households withdraw ground water from wells. There is little transfer of drinking water across the upper Charles River basin boundaries, and net transfer is estimated to be zero (DeSimone and others, 2002). Between 1989 and 1998, public-supply water withdrawals from the sources listed in table 5 increased by 8 percent from 10.4 to 11.2 ft^3/s and averaged 10.7 ft^3/s for the period. The increase in water use coincides with a population increase of 15 percent in the basin from 1990 to 2000 (DeSimone and others, 2002). Ground-water withdrawals vary seasonally, with higher withdrawals in the late spring and early summer (fig. 5). Monthly average (1989–98) rates for all stresses are given in Appendix 1. In the year 2000, 85 percent of the basin's population had public water supply, with the remaining 15 percent on private wells (table 6).

Table 4. Base flow under dry (90-percent low flow) climatic conditions and no water-supply or wastewater pumping, upper Charles River basin, eastern Massachusetts.

[Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	4.8	6.2	10.7	7.6	5.3	2.4	1.5	1.4	1.4	1.7	2.7	3.9
CR2	22.8	29.0	43.0	32.7	24.7	12.7	6.2	4.9	4.1	6.7	14.1	20.5
CR3	35.4	49.4	82.3	55.6	38.8	16.0	7.3	6.1	5.0	7.8	17.7	27.8
CR4	82.7	105.7	157.2	121.0	89.0	47.0	22.9	17.7	15.2	23.4	51.3	74.0
BB1	13.5	19.8	38.4	26.1	15.7	6.0	2.3	1.5	1.4	2.3	6.4	11.9
HB1	4.2	6.9	18.9	9.3	5.1	.6	.1	.1	.0	.1	.9	2.1
HB2	6.1	10.1	27.9	13.6	7.4	.8	.0	.0	.0	.1	1.1	2.9
MB1	11.9	14.3	20.9	16.0	12.9	6.6	3.9	3.4	3.1	4.2	6.8	9.2
MB2	17.8	22.7	36.5	26.3	19.9	8.7	4.8	4.1	3.9	5.2	8.7	12.7
MR1	20.9	25.0	34.8	27.6	22.7	11.9	6.6	5.5	5.1	7.1	11.9	16.2
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.43	0.55	0.95	0.67	0.47	0.21	0.13	0.12	0.12	0.15	0.24	0.35
CR2	.90	1.14	1.69	1.29	.97	.50	.24	.19	.16	.27	.55	.81
CR3	.54	.76	1.26	.85	.60	.24	.11	.09	.08	.12	.27	.43
CR4	.99	1.26	1.87	1.44	1.06	.56	.27	.21	.18	.28	.61	.88
BB1	.65	.95	1.84	1.26	.76	.29	.11	.07	.07	.11	.31	.57
HB1	.56	.93	2.54	1.24	.68	.08	.01	.01	.00	.02	.12	.28
HB2	.55	.92	2.53	1.23	.67	.07	.00	.00	.00	.00	.10	.27
MB1	1.18	1.42	2.08	1.59	1.28	.66	.39	.34	.31	.42	.68	.91
MB2	1.12	1.43	2.30	1.65	1.25	.55	.30	.26	.25	.33	.55	.80
MR1	1.31	1.56	2.18	1.73	1.42	.75	.41	.35	.32	.44	.74	1.01

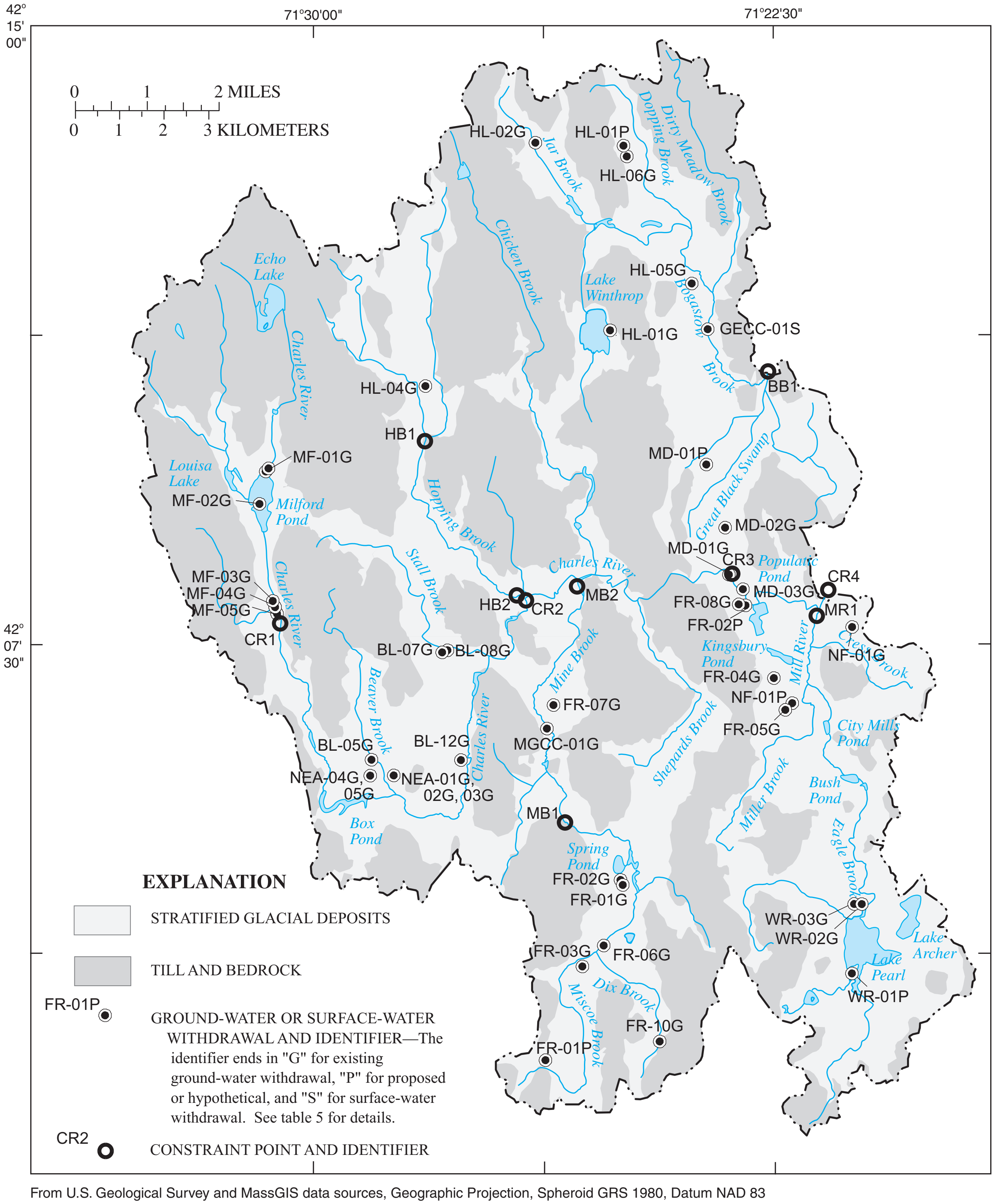


Figure 4. Ground-water and surface-water withdrawals analyzed, upper Charles River basin, eastern Massachusetts. (Modified from fig. 3, DeSimone and others, 2002.)

Table 5. Water withdrawals in the upper Charles River basin, eastern Massachusetts.

[Site locations shown in figure 4. **Optimization variables:** --, Each optimization variable represents a stress value for a particular month and is used in optimization formulations. **Type:** GW, ground water; SW, surface water. **Maximum permitted (Zone II) monthly withdrawal:** Zone II limits are State-imposed maximum withdrawal rates for individual wells. No., number; ft^3/s, cubic foot per second]

Well identifier	Town	Source name	Optimization variables	Type	Mean annual withdrawal 1989–98 (ft^3/s)	Maximum permitted (Zone II) monthly withdrawal (ft^3/s)
			Municipal Withdrawals			
BL-05G	Bellingham	Well No. 5	Q--B5	GW	0.30	0.45
BL-07G	Bellingham	Well No. 7	Q--B7	GW	.16	.65
BL-08G	Bellingham	Well No. 8	Q--B8	GW	.42	1.05
BL-12G	Bellingham	Well No. 12	Q--B9	GW	.03	.85
FR-01G	Franklin	Well No. 1	Q--F1	GW	.18	.73
FR-02G	Franklin	Well No. 2	Q--F2	GW	.18	1.11
FR-03G	Franklin	Well No. 3	Q--F3	GW	.47	.50
FR-04G	Franklin	Well No. 4	Q--F4	GW	1.03	1.42
FR-05G	Franklin	Well No. 5	Q--F5	GW	.26	.77
FR-06G	Franklin	Well No. 6	Q--F6	GW	.53	.82
FR-07G	Franklin	Well No. 7	Q--F7	GW	.44	.90
FR-08G	Franklin	Well No. 8	Q--F8	GW	.47	.40
FR-10G	Franklin	Well No. 10	Q--FT	GW	.34	.77
HL-01G	Holliston	Well No. 1 Lake Winthrop	Q--H1	GW	.08	.50
HL-02G	Holliston	Well No. 2 Maple Street	Q--H2	GW	.09	.48
HL-04G	Holliston	Well No. 4 Washington Street	Q--H4	GW	.22	.74
HL-05G	Holliston	Well No. 5 Central Street	Q--H5	GW	.69	1.10
HL-06G	Holliston	Well No. 6 Brook Street	Q--H6	GW	.70	1.33
MD-01G	Medway	Well No. 1 Populatic Street	Q--D1	GW	.64	.59
MD-02G	Medway	Well No. 2 Oakland Street	Q--D2	GW	.21	.91
MD-03G	Medway	Well No. 3 Village Street	Q--D3	GW	.52	.93
MF-01G	Milford	Dilla St. wells 1 and 2	Q--M1, Q--M2	GW	.02	1.05
MF-02G	Milford	Clark Island wellfield	Q--M6	GW	.66	1.24
MF-03G, -04G, -05G	Milford	Godfrey Brook wells	Q--M3, Q--M4, Q--M5	GW	.56	1.22
NF-01G	Norfolk	Well No. 1 Gold Street	Q--N1	GW	.38	.67
WR-02G	Wrentham	Well No. 2	Q--W2	GW	.49	1.11
WR-03G	Wrentham	Well No. 3	Q--W3	GW	.65	1.47
			Large Nonmunicipal Withdrawals			
NEA-01G, -02G, -03G	Bellingham	Northeast Energy Association wells No. 1, 2, 3	Q--B1	GW	0.83	[1]1.02
NEA-04G, -05G	Bellingham	Northeast Energy Association wells 4 and 5	Q--B4	GW	.02	[1]1.02
GECC-01S	Holliston	Glen Ellen Country Club	Q--HG	SW	.06	.25
MGCC-01G	Holliston	Maplegate Country Club well	Q--HM	GW	.05	.23

Table 5. Water withdrawals in the upper Charles River basin, eastern Massachusetts.—Continued

[Site locations shown in figure 4. **Optimization variables:** --, Each optimization variable represents a stress value for a particular month and is used in optimization formulations. **Type:** GW, ground water; SW, surface water. **Maximum permitted (Zone II) monthly withdrawal:** Zone II limits are State-imposed maximum withdrawal rates for individual wells. No., number; ft^3/s, cubic foot per second]

Well identifier	Town	Source name	Optimization variables	Type	Mean annual withdrawal 1989–98 (ft^3/s)	Maximum permitted (Zone II) monthly withdrawal (ft^3/s)
			Proposed or Hypothetical Withdrawals			
FR-01P	Franklin	Well No. 11	Q--FE	GW	--	0.72
FR-02P	Franklin	Populatic Pond well	Q--FP	GW	--	.47
HL-01P	Holliston	Well No. 7	Q--H7	GW	--	.86
MD-01P	Medway	Proposed well	Q--DP	GW	--	.43
NF-01P	Norfolk	Mill River well	Q--NP	GW	--	1.08
WR-01P	Wrentham	Proposed well	Q--W1	GW	--	1.01

[1]Maximum permitted withdrawal rate includes Northeast Energy Association wells No. 1 through 5.

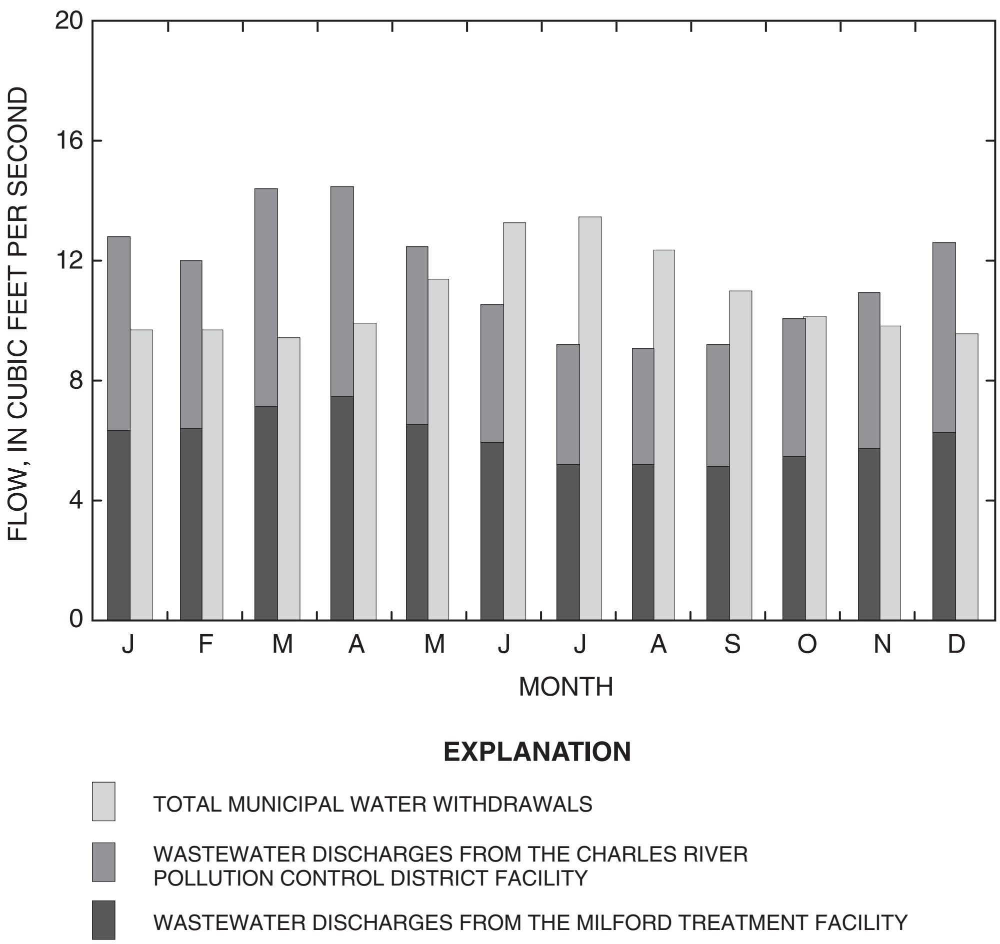


Figure 5. Average monthly municipal water withdrawals (1989–98) and wastewater discharges in the upper Charles River basin, eastern Massachusetts. (Modified from fig. 5, DeSimone and others, 2002.)

Table 6. Population characteristics of towns with greater than 10,000,000 square feet of area within the study area, upper Charles River basin, eastern Massachusetts.

[Town locations shown in figure 1. Adapted from table 3 of DeSimone and others, 2002]

Town	Proportion of town in study area (in percent)	Total population in basin		Total population on public water (in percent)		Total population on public sewer (in percent)	
		1990	2000	1990	2000	1990	2000
Bellingham	52	14,877	15,314	96	96	5	29
Franklin	91	22,095	29,560	76	80	51	61
Holliston	100	12,926	13,801	97	98	0	0
Medway	100	9,931	12,448	79	73	37	45
Milford	86	25,355	26,799	99	94	75	80
Millis	46	7,613	7,902	85	85	40	60
Norfolk	31	9,270	10,460	40	58	0	0
Wrentham	18	9,006	10,554	80	80	0	0
Total		111,073	126,838	85	85	34	43

Wastewater is discharged at sewage treatment plants (38 percent), infiltrated to aquifers through private septic discharge (52 percent), or lost to evapotranspiration (10 percent); these percentages are based on annual average estimates. Private septic systems are widely distributed in the basin. Two facilities in the basin discharge treated wastewater to the Charles River: the Milford Water Treatment Facility (MTF), which serves Milford, and the Charles River Pollution Control District facility (CRPCD), which serves Bellingham, Franklin, Medway, and Millis (fig. 6). The greatest monthly wastewater discharge is in April and the smallest is in August (fig. 5). Wastewater discharge from the two facilities averages 11.5 ft^3/s annually. At the MTF, annual wastewater flow equals 75 percent of base flow coming from upstream in the Charles River.

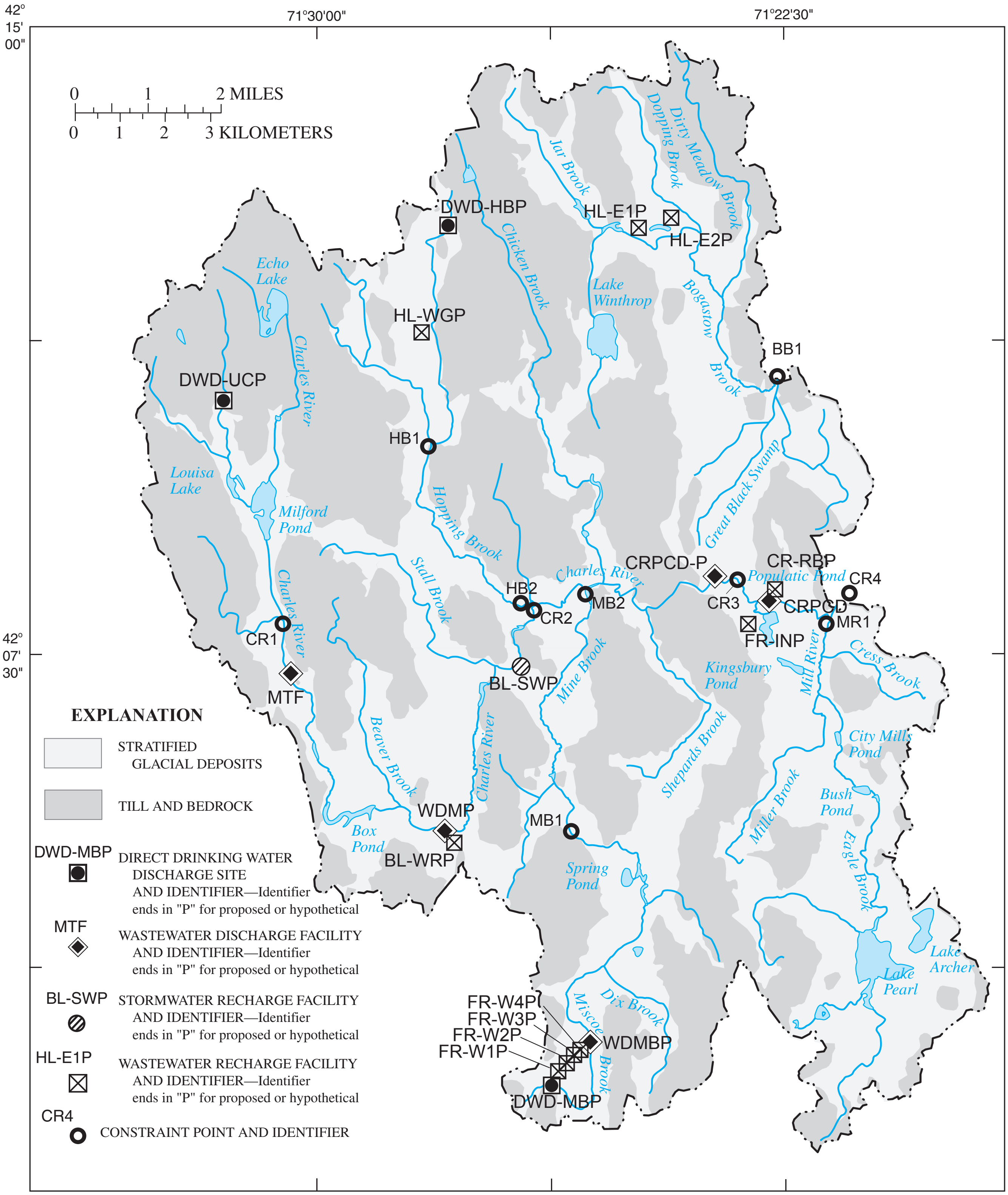
In 2000, 43 percent of the basin's population had sewer service, with the remaining 57 percent on private septic disposal (table 6). Details on wastewater returns for existing conditions and hypothetical scenarios are given in table 7.

Consumptive loss, that portion of water supply lost to evapotranspiration, averages 10 percent annually in New England (Solley and others, 1998, table 1) and is highest during summer months. Monthly consumptive use in the study area may rise as high as 42 percent in July; this estimate is based on differences between pumping in March (9.4 ft^3/s) and July (13.4 ft^3/s) (fig. 5) and an assumption that the additional pumped water is for highly consumptive uses such as filling swimming pools, washing cars, and gardening.

All drinking-water withdrawals are subject to daily maximum pumping limits established by MADEP and referred to as "Zone II" limits. Regulation also comes from the Massachusetts Watershed Management Act (WMA), which covers water withdrawals of more than 100,000 gal/d and sets maximum annual rates for combined ground-water and surface-water withdrawals. Individual limits (Zone II) on wells listed in tables 5 and 8 total 23.8 ft^3/s, whereas the separately administered WMA limits total 15.5 ft^3/s. Both of these limits are higher than average annual municipal withdrawals of 10.7 ft^3/s (table 8).

Input to the stream-aquifer system in the study area, excluding precipitation that runs off quickly to streams or is intercepted by vegetation, is as follows: precipitation, 154–200 ft^3/s (assumes 20–26 in. recharge per year); septic return flow, 4.8 ft^3/s; and wastewater discharge to streams, 11.5 ft^3/s. Outflows consist of base-flow discharge from the basin, 150.7 ft^3/s (from MOVE.1 estimates); ground-water and surface water withdrawals, 14.5 ft^3/s; and evapotranspiration from wetlands and surface water, 7.7 ft^3/s.

Total annual water use is 9 percent of annual recharge, calculated on the basis of 1989–98 average total municipal, industrial, and private water withdrawals of 14.5 ft^3/s and annual average recharge of 164.2 ft^3/s. On a monthly basis, however, this proportion can be much higher (fig. 7). In September, withdrawals are 35 percent of average recharge. High rates of ground-water withdrawal relative to short-term recharge rates are important to the hydrologic budget of the basin because the high permeability and low storage capacity of aquifers can, within a short time, lead to reduced ground-water discharge to streams.



From U.S. Geological Survey and MassGIS data sources, Geographic Projection, Spheroid GRS 1980, Datum NAD 83

Figure 6. Water-return sites analyzed, upper Charles River basin, eastern Massachusetts. Names and abbreviations for facilities are given in table 7.

Table 7. Water-discharge stresses as used in the optimization analysis for the upper Charles River basin, eastern Massachusetts.

[Site locations shown in figure 6. **Optimization variables:** --, Each optimization variable represents a stress value for a particular month and is used in optimization formulations. **Map identifier:** P, at the end of the map identifier denotes a proposed or hypothetical facility. CRPCD, Charles River Pollution Control District; ft, foot]

Map identifier	Town	Source name	Optimization variables	Status	Annual average flow (1989–98)
Wastewater Direct Discharge to Streams					
WDMP	Bellingham	Bellingham proposed wastewater plant	W--BL	--	--
CRPCD	Medway	CRPCD	W--CR	Existing	6.07
CRPCD-P	Medway	CRPCD upstream proposed discharge point	W--CP	--	--
MTF	Milford	Milford wastewater-treatment plant	W--MF	Existing	5.41
WDMBP	Franklin	Miscoe Brook hypothetical recharge site	W--FM	--	--
Drinking Water (DW) Direct Discharge to Streams					
DWD-UCP	Milford	DW discharge to upper Charles River	D--CR	--	--
DWD-HBP	Holliston	DW discharge to upper Hopping Brook	D--HB	--	--
DWD-MBP	Franklin	DW discharge to upper Miscoe Brook	D--MB	--	--
Wastewater (WW) Recharge to Aquifers (infiltration basins or injection wells)					
FR-INP	Franklin	Franklin WW injection well	Q--FI	--	--
FR-W4P	Franklin	WW recharge basin 200 ft southwest of Miscoe Brook	W--M4	--	--
FR-W3P	Franklin	WW recharge basin 894 ft southwest of Miscoe Brook	W--M3	--	--
FR-W2P	Franklin	WW recharge basin 1,720 ft southwest of Miscoe Brook	W--M2	--	--
FR-W1P	Franklin	WW recharge basin 2,608 ft southwest of Miscoe Brook	W--M1	--	--
HL-WGP	Holliston	Holliston–Gorwin Drive site	W--HG	--	--
HL-E1P	Holliston	Holliston–EHIP-1 Site	W--H1	--	--
HL-E2P	Holliston	Holliston–EHIP-2 Site	W--H2	--	--
BL-WRP	Bellingham	Bellingham proposed WW recharge facility	Q--BR	--	--
CR-RBP	Medway	WW recharge at CRPCD	W--C1	--	--
Storm Water Recharge to Aquifers					
BL-SWP	Bellingham	Bellingham Proposed Recharge Facility–Maple Street BL-02I	Q--BS	--	--
Town Wastewater Discharge (combined septic and wastewater discharge in basin)[1]					
-	Bellingham	Bellingham summer wastewater return	R--BS	Existing	0.42
-	Bellingham	Bellingham winter wastewater return	R--BW	Existing	.49
-	Franklin	Franklin summer wastewater return	R--FS	Existing	1.63
-	Franklin	Franklin winter wastewater return	R--FW	Existing	2.66
-	Holliston	Holliston summer wastewater return	R--HS	Existing	.75
-	Holliston	Holliston winter wastewater return	R--HW	Existing	1.03
-	Medway	Medway summer wastewater return	R--DS	Existing	.52
-	Medway	Medway winter wastewater return	R--DW	Existing	.85
-	Milford	Milford summer wastewater return	R--MS	Existing	.39
-	Milford	Milford winter wastewater return	R--MW	Existing	.86
-	Norfolk	Norfolk summer wastewater return	R--NS	Existing	.15
-	Norfolk	Norfolk winter wastewater return	R--NW	Existing	.23
-	Wrentham	Wrentham summer wastewater return	R--WS	Existing	.45
-	Wrentham	Wrentham winter wastewater return	R--WW	Existing	.69

[1]Summer months are May–August, winter months are September–April.

Table 8. Water withdrawals and regulatory limits by town, upper Charles River basin, eastern Massachusetts.

[Town locations shown in figure 1. **Total monthly zone II limits:** Zone II limits are State-imposed withdrawal rates for individual wells. **Annual WMA limits:** WMA, Watershed Management Act, WMA limits are State-imposed maximum withdrawal rates for a group of water sources controlled by a water supplier. ft^3/s, cubic foot per second; Mgal/d, million gallons per day]

Town	Average yearly withdrawals, 1989–98		Total monthly zone II limits		Annual WMA limits	
	(Mgal/d)	(ft^3/s)	(Mgal/d)	(ft^3/s)	(Mgal/d)	(ft^3/s)
Bellingham	0.6	0.9	1.9	3.0	1.4	2.1
Franklin	2.7	3.9	4.8	7.4	4.1	6.3
Holliston	1.2	1.8	2.7	4.1	1.1	1.8
Medway	.9	1.4	1.6	2.4	.7	1.1
Milford	.1	1.3	2.3	3.5	1.4	2.2
Norfolk	.2	.4	.4	.7	.4	.6
Wrentham	.7	1.1	1.7	2.6	.9	1.4
Totals	6.5	10.7	15.4	23.8	10.0	15.5

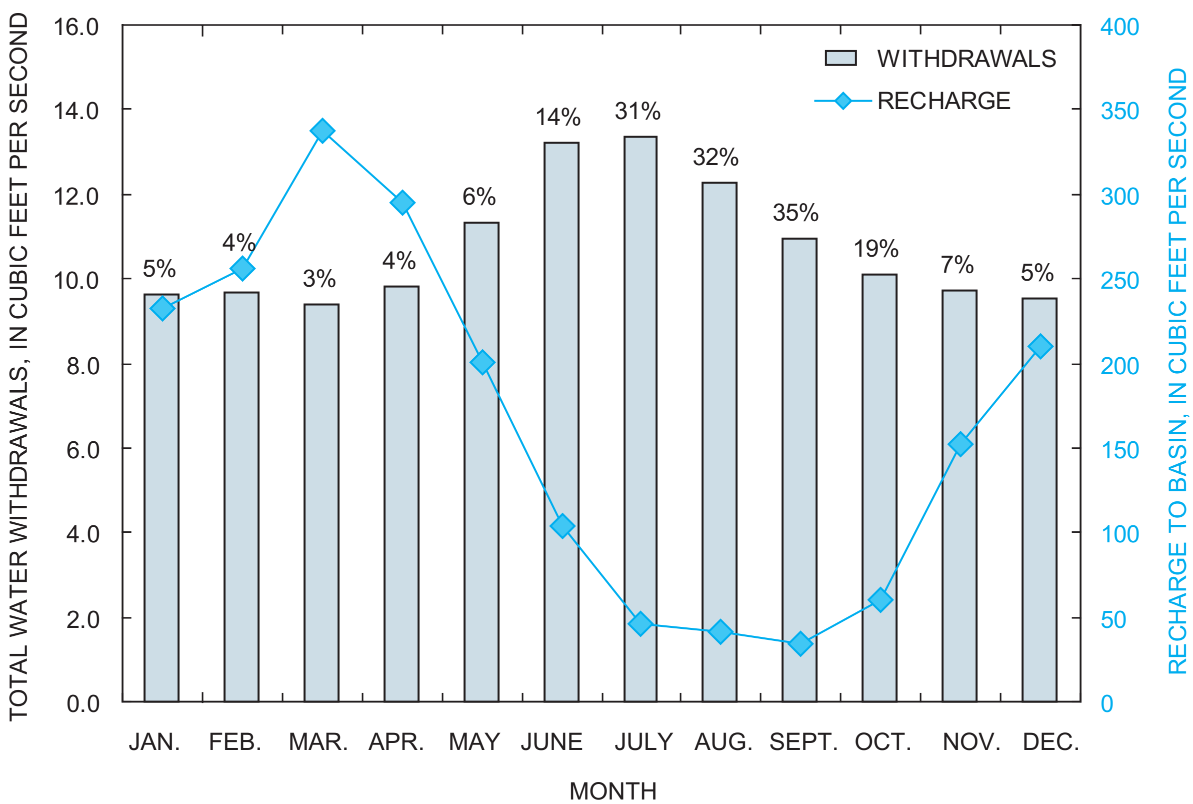


Figure 7. Monthly average recharge and withdrawals (1989–98), upper Charles River basin, eastern Massachusetts. Numbers at top of columns are total withdrawals in basin as a percentage of recharge. Recharge values are those used in the transient numerical model under average climatic conditions.

Modifications to the Original Flow Model

The numerical flow model developed by DeSimone and others (2002) was modified for use in this study. The modified numerical model has similar grid dimensions as the original model, with 2 layers, 420 rows, 325 columns, uniformly spaced cells 200 ft on a side, and grid axes coincident with north-south and east-west directions (fig. 8). The model cells are inactive over about 50 percent of the study area, where bedrock or till outcrops or where the sand and gravel deposits are thin or absent.

Modifications made to the original model include combining active cells in the east and west models to form a single model, altering the recharge array, increasing aquifer thickness in some areas, and changing aquifer and stream stresses. Once the modifications to the original model were completed, simulation results were compared to the calibration criteria documented in DeSimone and others (2002). A sensitivity analysis was not performed because changes to the model were not substantial enough to warrant it.

Combining East and West Models

The original numerical flow model (DeSimone and others, 2002) separated the upper Charles River basin into east and west models to facilitate a numerical solution to the ground-water flow equations. In this study the two models were joined to simplify analysis of regional scenarios. There is no ground-water connection between the east and west sides of the combined model, only a surface-water connection through the Charles River. In addition to combining input files, stream segments were renumbered and the Preconditioned Conjugate-Gradient 2 (PCG2) package (Hill, 1990) was specified for solving linear equation systems.

Modifications to Model Parameters and Boundaries

To address numerical instabilities, stream-stage elevations and conductance values were changed in a number of stream segments. Stream stage was modified in approximately 20 cells to reduce abrupt changes in elevation. Streambed conductance was modified in the vicinity of Lake Pearl to allow the model to converge and to improve the simulated match to observed water levels. The number of stream cells crossing Lake Pearl was increased to better simulate flow between the lake and underlying aquifer and remove numerical instabilities in that area. Other changes included adding three stream segments to simulate discharge of wastewater effluent directly to streams.

Numerical instabilities in the combined model were caused by model cells going dry, particularly near boundaries of inactive cells and during periods of low recharge. To prevent them from going dry, a large number of cells (5,421 of 19,592 active cells) were assigned greater thickness. By trial and error it was found that increasing the thickness of cells by 30 ft in layer 1 was enough to prevent cells from going dry.

Modifications to Stresses

Stress periods were modified in the transient model so that they coincided with calendar months for a total simulation time of 5 years (60 months). Monthly stresses were repeated in each year so that the model reached a state of dynamic equilibrium by the fifth year. The number of time steps within each stress period was varied from 30 to 150 with more time steps used in summer months to allow model convergence.

Slight modifications were made to recharge input. Because recharge had to be modified to address various management scenarios, recharge input to MODFLOW was calculated cell by cell by using a FORTRAN program and then was read by MODFLOW as an array of constants; the zone and parameter functions of MODFLOW2000 were no longer used to specify recharge as was done in DeSimone and others (2002). The FORTRAN program determined recharge by calculating separate contributions to recharge and then summing the contributions. Contributions were from infiltrating precipitation, septic-system discharge from publicly supplied areas that use septic fields, and infiltrating irrigation from communities on public water. Subtracted from these contributions were monthly consumptive use in areas on private wells and septic systems, withdrawals from major supply wells in upland areas, and surface-water diversions in the Echo Lake subbasin. Consumptive water loss was assumed to reduce septic recharge by 30 percent from May through August. The 30-percent figure comes from assuming that all of a 10-percent annual consumptive loss (Solley, 1998, table 1) occurs from May through August.

As in the study by DeSimone and others (2002), recharge in upland till and bedrock areas was routed to active model areas and added to recharge of the edge cells. Recharge rates from natural precipitation were assigned by land use. Monthly variations in recharge had the same percentages as in the original study. A further change was to modify only natural precipitation and not septic-system discharge during the model calibration process. Several well locations and pumping rates were updated with new information. Injection wells were added to simulate wastewater-injection wells and recharge galleries, which are used for recharging wastewater to the ground-water system.

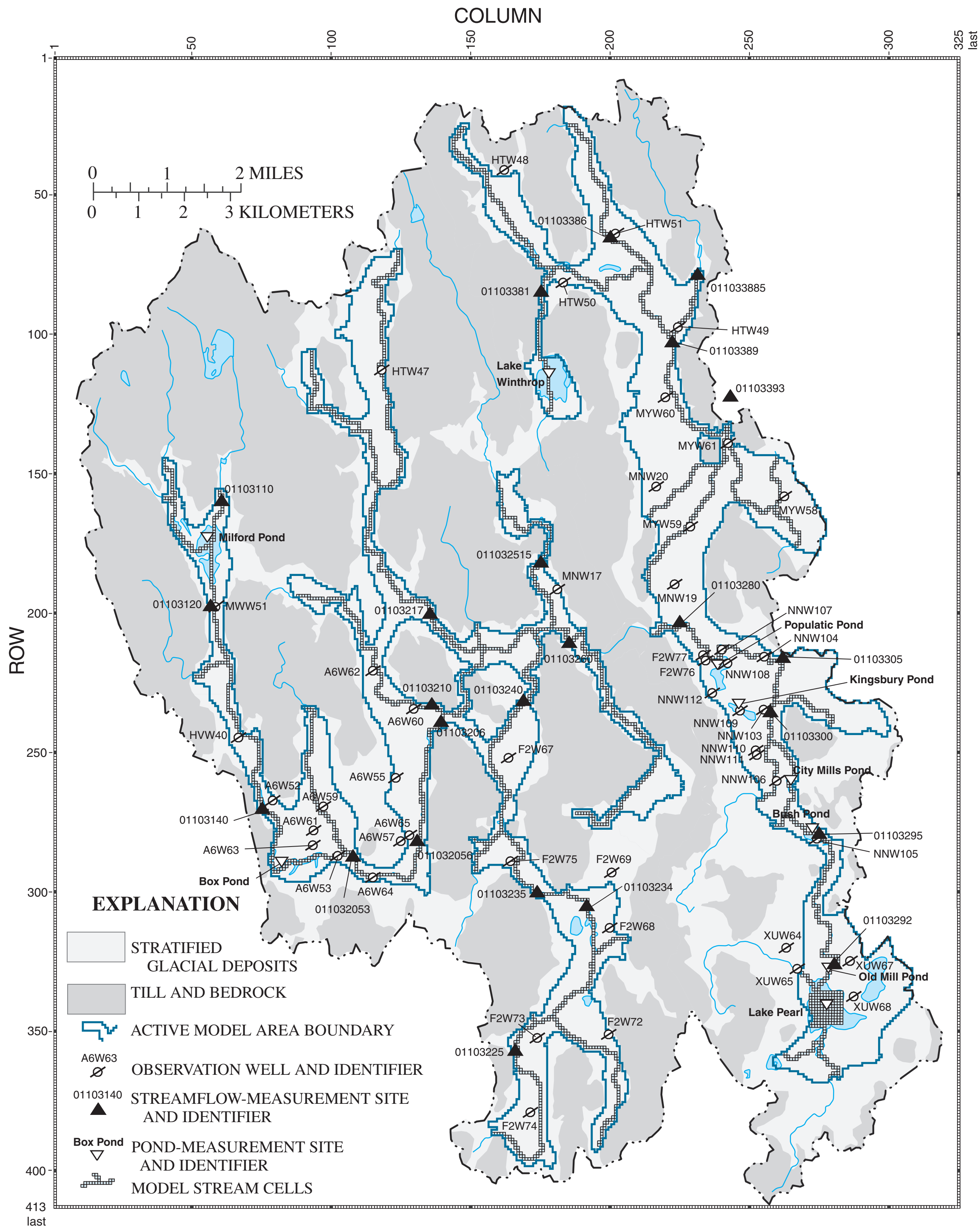


Figure 8. Model grid and locations of streamflow and water-level calibration points, upper Charles River basin, eastern Massachusetts.

Model Calibration

Model modifications caused minor changes in simulated heads and stream base flow. A complete calibration process was not undertaken for the updated model because the hydraulic changes were minor. An exception was the southeastern part of the model around Lake Pearl, where difficulties getting the model to converge indicated that more extensive changes to stream properties were needed; consequently, more attention was paid to matching simulated and observed heads and base flow in this area. The steady-state model and the stress periods in the transient model have mass-balance errors of less than 1 percent.

Simulated heads were compared to observed heads at the same 50 observation points used to calibrate the original model (table 9). For the steady-state model, differences between observed and model calculated heads averaged -0.4 ft, and the mean absolute difference was 2.7 ft. In the original study, the mean absolute difference between observed and model calculated heads was 2.6 ft for the west model and 2.0 ft for the east model (DeSimone and others, 2002).

Water-level fluctuations over the course of the year in the transient model are compared to observed water-level fluctuations in figure 9. Water-level fluctuations are shown relative to average water levels for the period shown. The transient model heads show the same seasonal fluctuations exhibited by the original model and match the observed patterns of seasonal fluctuation reasonably well. As in the original model, the pattern of minimum water levels at some sites shows a 1- to 2-month delay relative to observed water levels (fig. 9).

Modeled steady-state base flow agrees approximately with observed base flow, as it did for the original model (table 10). At the model exits (station 01103305 on the Charles River and station 01103393 on Bogastow Brook), simulated and observed base flow differ by less than 3.0 percent (as compared to less than 4 percent in the original model). The mean absolute difference between simulated and observed base flow at 23 sites was 2.0 ft^3/s; the original model had values of 1.6 ft^3/s in the west model and 3.4 ft^3/s in the east model. At individual measurement sites, mean absolute base-flow residuals ranged from 2 to 45 percent of observed values (3 to 60 percent in the original model), with larger differences occurring at sites with low flow, as in the original model (DeSimone and others, 2002).

Seasonal fluctuations in the updated model are similar to those in the original model and agree reasonably well with observed fluctuations, with most base-flow values within the 90-percent confidence intervals for observed base-flow estimates (fig. 10). For 23 base-flow calibration points in the updated transient model, mean error was -15 percent and mean absolute error was 45 percent of observed base flow overall. At the Charles River (01103305) and Bogastow Brook (01103393) model exit points, mean error averaged 15 percent and 22 percent, respectively, and mean absolute error averaged 19 percent and 71 percent. As in the original model, errors were larger in July through October, when base flow is lowest. The monthly mean absolute residual for all 23 calibration points ranged from 3 percent in March to 126 percent in July. The large error in the late summer months was mostly due to a month delay in the modeled low-flow period. The large errors are more pronounced in low-flow months, when a comparatively small difference in volumetric flow rate can be a significant proportion of observed flow.

Model Errors and Limitations

The simulated water levels and base-flow values do not exactly match observed data. Errors in simulated base flow for the summer months are more important than for other months because summer streamflow provides a major constraint on water management and directly affects solutions to water-management problems. Three major differences between simulated and observed low base flow (fig. 10) are minimum base flow in the model is higher than observed minimum base flow; the period of minimum base flow is delayed by about 1 month in the model (August–October) as compared to observed base flow (July–September); and errors, measured in percent, are greater for small streams (low base flow) than for large streams.

Other errors may exist in the simulated results because MODFLOW is designed primarily for simulating ground-water flow and not for simulating ground-water/surface-water interactions. Model errors may result from the assumption of fixed stream stages when the actual stream level is rising and falling. Errors may also be caused by the assumption that modeled streams go dry if there is not enough ground-water discharge, whereas actual streams carry water from surface runoff.

The thin and discontinuous aquifers make the numerical model sensitive to small changes in hydraulic properties. Changing streambed conductance by a small amount (50 percent for example) can cause cells to go dry in a cascade that prevents model convergence. Base-flow values also fluctuate due to numerical instabilities. Changing the convergence parameters in the PCG2 solver package of the model input by 50 percent caused base flow to vary by as much as 0.06 ft^3/s in some cases.

The model was used only for generating response coefficients, which are described in the next section, and not for simulating base flow under the various scenarios, because model errors were large. After the ground-water model was used to generate hydrologic response coefficients, all hydrologic simulations were then performed by using the response coefficients, rather than by using the simulation model. This reduced errors and allowed optimization to be easily implemented.

Table 9. Model-calculated steady-state water levels and observed average water levels, upper Charles River basin, eastern Massachusetts.

Well or pond name	Model cell location			Average annual water level (feet relative to NGVD29)		
	Layer	Row	Column	Model-calculated	Observed	Difference (calculated minus observed)
A6W52	1	269	80	225.1	229.7	-4.58
A6W53	1	289	103	214.4	212.7	+1.70
A6W55	1	261	124	209.6	208.5	+1.11
A6W59	1	272	98	216.4	215.5	+.89
A6W60	1	236	130	200.3	205.7	-5.40
A6W61	1	280	94	219.1	219.0	+.11
A6W62	1	223	116	214.6	211.0	+3.65
A6W63	1	285	94	220.4	220.3	+.11
A6W65	1	282	129	206.2	203.2	+3.04
F2W67	1	254	164	178.1	179.0	-.86
F2W72	1	354	200	285.7	286.5	-.83
F2W73	1	355	175	249.2	254.4	-5.13
F2W74	1	381	172	261.6	265.5	-3.88
F2W75	1	291	165	190.5	188.5	+1.97
HTW47	1	114	118	240.2	238.6	+1.61
HVW40	1	247	67	232.3	238.4	-6.02
MWW51	1	200	59	251.2	248.7	+2.49
MNW17	1	193	182	173.0	183.0	-10.05
Box Pond	1	290	88	221.6	221.3	+.25
Milford Pond	1	176	57	267.4	265.7	+1.70
F2W76	1	219	235	128.8	127.1	+1.72
HTW48	1	44	163	203.0	205.2	-2.17
HTW49	1	99	225	141.9	143.8	-1.93
HTW50	1	82	184	165.2	167.1	-1.85
HTW51	1	66	202	150.4	149.0	+1.41
MNW19	1	192	224	141.2	137.7	+3.51
MNW20	1	157	217	138.5	140.7	-2.16
MYW58	1	160	263	141.5	144.7	-3.21
MYW59	1	171	229	138.4	135.6	+2.80
MYW60	1	125	221	138.9	138.5	+.37
MYW61	1	141	243	126.5	131.6	-5.14
NNW103	1	237	256	129.7	130.2	-.49
NNW104	1	218	256	125.2	128.7	-3.48
NNW105	1	283	275	173.7	173.3	+.39
NNW106	1	262	260	148.7	140.4	+8.33
NNW107	1	215	241	128.2	126.5	+1.69
NNW108	1	220	243	128.1	126.9	+1.23
NNW109	1	237	247	130.4	129.3	+1.15
NNW110	1	252	253	135.2	132.5	+2.73
NNW112	1	231	237	130.7	130.8	-.11
XUW65	1	330	268	212.1	205.7	+6.44
XUW67	1	327	287	202.2	195.3	+6.89
XUW68	1	339	288	206.8	204.2	+2.56
Bush	1	282	276	173.7	173.8	-.08
City Mills	1	263	264	149.1	148.8	+.28

Table 9. Model-calculated steady-state water levels and observed average water levels, upper Charles River basin, eastern Massachusetts.—Continued

Well or pond name	Model cell location			Average annual water level (feet relative to NGVD29)		
	Layer	Row	Column	Model-calculated	Observed	Difference (calculated minus observed)
Kingsbury	1	237	250	130.4	129.7	+.69
Pearl	1	340	278	206.8	199.6	+7.20
Winthrop	1	116	180	176.4	174.6	+1.82
Old Mill	1	330	278	203.0	194.3	+8.69
Populatic	1	219	239	128.0	127.2	+.85

Table 10. Modeled steady-state base flow and observed base flow, upper Charles River basin, eastern Massachusetts.

[Observed values are from Desimone and others, 2002. MOVE.1 flow values are estimated from streamflow records at the station in question as well as from long-term records at neighboring stations. No., number; ft^3/s, cubic foot per second]

Station No.	Model cell location			Average annual base flow (ft^3/s)				
					Observed (MOVE.1 estimated)			Difference (calculated minus observed)
						90-percent confidence interval		
	Layer	Row	Column	Model calculated	Flow	High	Low	
01103110	1	161	61	1.8	2.0	1.2	3.2	-0.1
01103120	1	197	58	7.0	5.6	3.0	10.5	+1.3
01103140	1	271	76	18.4	15.8	10.6	23.7	+2.6
011032056	1	282	131	28.3	24.2	14.7	27.5	+4.1
01103206	1	239	140	31.5	28.8	17.2	34.0	+2.6
01103260	1	210	186	88.3	86.9	21.4	38.9	+1.4
01103210	1	233	137	4.3	7.6	.8	71.7	-3.2
01103217	1	200	136	15.7	14.8	5.8	37.8	+.8
01103225	1	357	167	3.1	2.5	1.5	4.2	+.6
01103234	1	305	192	11.6	13.6	10.2	18.0	-1.9
01103235	1	302	175	13.4	15.0	7.9	28.6	-1.7
01103240	1	232	170	19.9	24.4	14.9	40.0	-4.5
011032515	1	182	176	8.4	8.1	5.5	12.1	+.3
01103280	1	204	226	96.4	91.0	59.7	139.0	+5.4
01103305	1	216	263	132.6	131.4	103.0	168.0	+1.2
01103292	1	326	281	6.5	11.3	8.0	16.0	-4.8
01103295	1	279	275	15.1	18.4	11.9	28.4	-3.3
01103300	1	236	258	23.4	26.0	18.3	37.0	-2.6
01103381	1	85	176	3.5	3.3	2.2	4.9	+.2
01103386	1	66	201	1.4	1.0	.5	2.0	+.4
011033885	1	79	232	1.6	1.6	1.1	2.2	+.0
01103389	1	103	223	18.1	12.8	8.0	20.5	+5.3
01103393	1	132	243	30.9	30.2	20.9	43.6	+.7

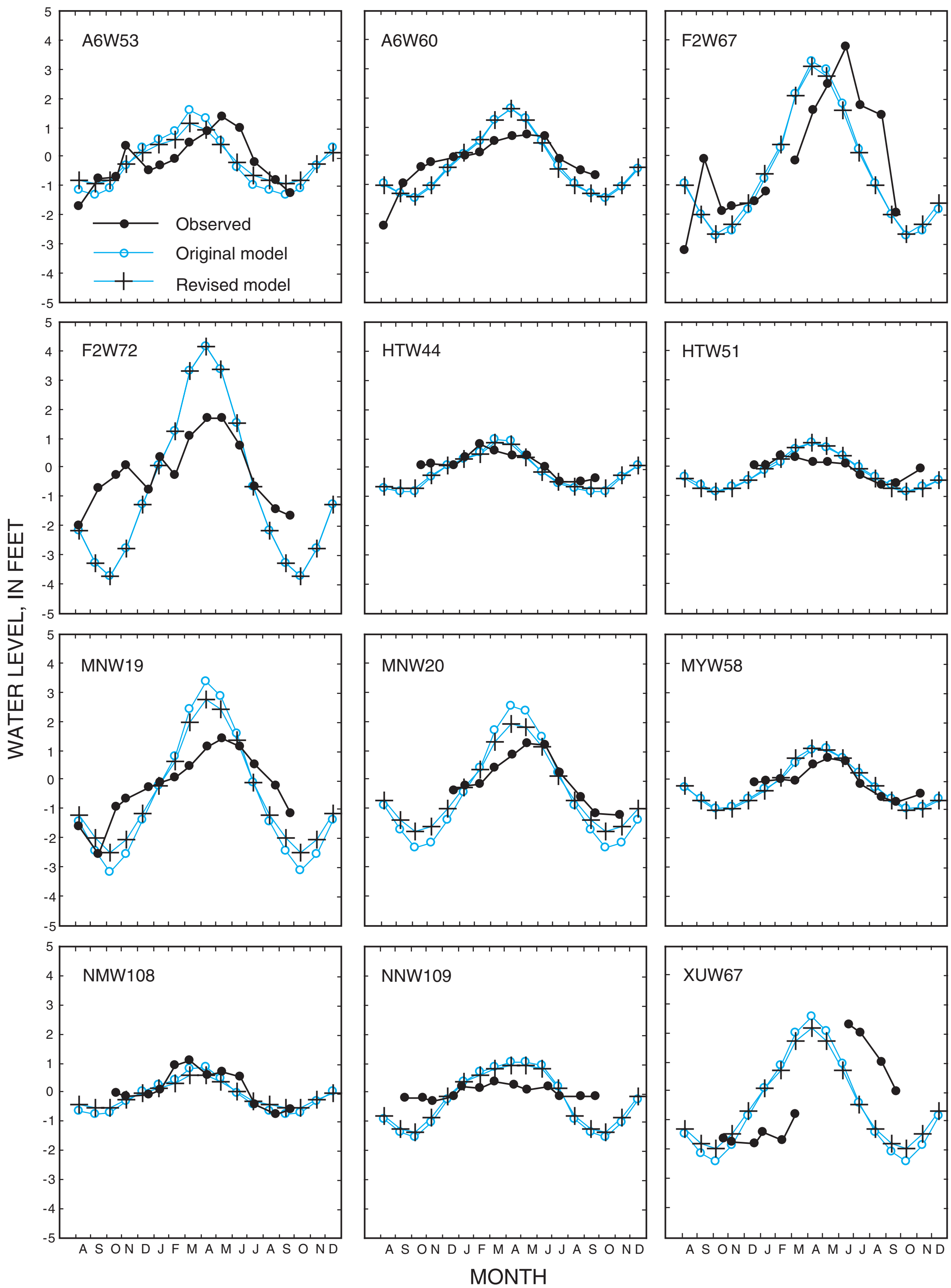


Figure 9. Simulated and observed water levels for selected observation wells in the upper Charles River basin, eastern Massachusetts. Values are deviations from average (1989–98) water levels. Original model data are from figure 20, DeSimone and others, 2002.

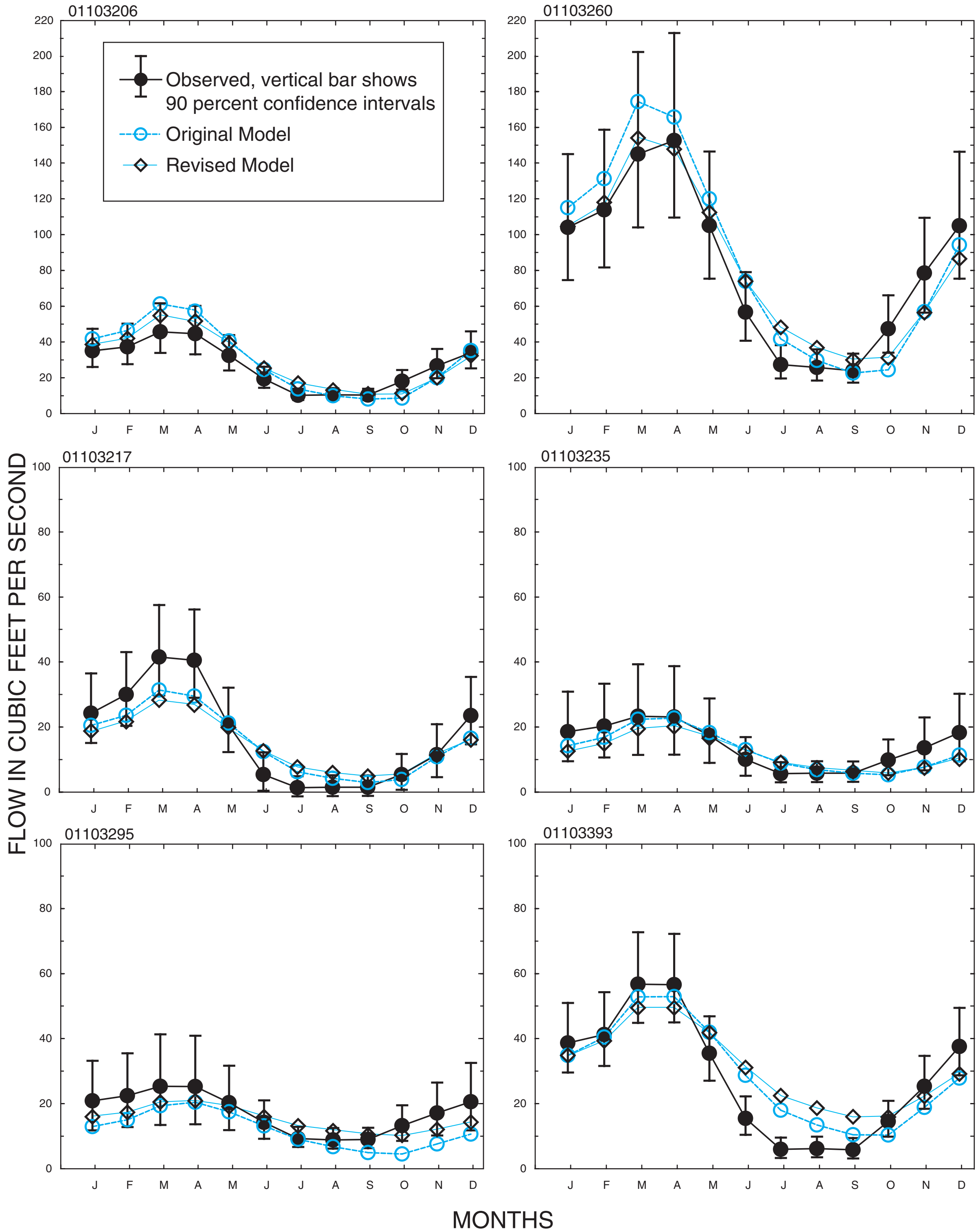


Figure 10. Simulated and observed base-flow values for selected locations in the upper Charles River basin, eastern Massachusetts. Observed data are MOVE.1 estimated flows based on partial-record observations. Original model data are from figure 21, DeSimone and others (2002). MOVE.1 flows estimated from partial streamflow records at the station and long-term streamflow records from nearby stations. Number at top of each chart is the U.S. Geological Survey site identification number. (Locations are shown in fig. 3.)

Optimization Methods

Optimization methods are a collection of mathematical techniques used to solve complex resource-management problems. Originally developed to guide management of industrial systems, optimization has been applied extensively in the field of water-resource management (for example, Gorelick, 1983; Wagner, 1995; Ahlfeld and Mulligan, 2000). When optimization methods are combined with hydrologic simulation models, the combined approach is termed "simulation-optimization."

The simulation-optimization approach used here relies on a numerical flow model to simulate the hydrologic response of a stream-aquifer system to applied stresses such as water withdrawals. Response coefficients generated from the model results are combined with optimization techniques to answer specific water-management questions. The mathematical representation of these questions is a "management model" (DeSimone and others, 2002).

A management model has three components: *decision variables, objective function*, and *constraints* (Ahlfeld and Mulligan, 2000). The *decision variables* are the quantities to be determined, for example, pumping rates at a supply well (Q_{wi}). The *objective function* expresses the management goal as a maximum or minimum. For example, maximize total pumping (Q_{wT}) at three supply wells (Maximize $Q_{wT} = Q_{w1} + Q_{w2} + Q_{w3}$). *Constraints* set limits on what values decision variables may take. For example, withdrawals must exceed a specified value ($Q_{w1} + Q_{w2} + Q_{w3} > 1$ Mgal/d). An optimization software package, LINDO (Shrage, 1997), was used to solve management models. Solutions consist of monthly stress rates for the decision variables.

Hydrologic Response Coefficients

The management model is linked to the numerical flow model through a matrix of response coefficients (Gorelick, 1983). Response coefficients quantify the hydrologic response to a unit increase in stress (such as pumping) at one site for 1 month. The stress locations are shown in figures 4 and 6. To calculate a response coefficient, a hydrologic stress is simulated in month 1, and the resulting hydrologic changes are entered in equation 2:

$$r_{i,j,k} = \frac{Qsd_{j,k}}{Qw_j}, \qquad (2)$$

where

i = index for location of hydrologic response;
j = stress location;
k = month of hydrologic response;
Qsd = hydrologic response rate (ft^3/s);
Qw = stress rate (ft^3/s); and
r = response coefficient.

The stress perturbations (Q_w) used to calculate coefficients in equation 2 were selected to represent high and low ends of the range of possible stress values. Ground-water withdrawal stresses were simulated at 25 percent and 100 percent of state Zone II maximum pumping rates.

Base-flow changes are the only hydrologic responses analyzed. Ten stream locations, called "constraint points," were selected for calculating base-flow response to hydrologic stresses (table 11 and fig. 3). Constraint points CR4 and BB1 are where the Charles River and Bogastow Brook exit the study area; all hydrologic stresses in the upper Charles River basin affect base flow at one or both of these points. Constraint points MR1, MB2, and HB2 are the downstream ends of the Mill River, Mine Brook, and Hopping Brook tributaries, respectively; base flows at these points reflect all hydrologic processes in their respective subbasins. Constraint points MB1, HB1, and CR1 represent smaller subbasins in the Charles River headwaters. The remaining constraint points, CR2 and CR3, are midway along the Charles River and allow downstream flow patterns in the main drainage basin to be determined and provide better coverage for low-flow constraints.

Response coefficients were generated for each hydrologic stress considered in this study. Tables 5 and 7 report all ground-water withdrawal and water-return stresses considered. A complete listing of all response coefficients is given in Appendix 2. Example coefficients are shown in figure 11 for streamflow depletions at CR4 (Charles River exit) in response to pumping at Wrentham well 2 (WR-02G). Because the numerical model operates on the assumption that flow in streams moves quickly, with all flow exiting the basin within each 1-month time period, all constraint points downstream of a stress will have the same response coefficients for that stress.

The response coefficient approach is based on an assumption of linear response to hydrologic stresses. Multiple stresses can be added together or subtracted under this assumption. Because linearity of hydrologic response is assumed with respect to the magnitudes and times of stress, response coefficient values do not vary under different pumping rates, recharge conditions, or times of year. The only exception is the coefficients for wastewater discharge, which are specified to take on different values in the summer to account for greater evaporative loss of water. For example, the responses to wastewater discharge by the town of Holliston during May through August are described by different response coefficients than are used during the rest of the year. Hydrologic stresses that increase base flow, such as injection wells, septic return flow, wastewater discharge, and wastewater-infiltration basins are given negative signs in equation 2, and the resulting coefficients are also negative.

Table 11. Constraint points, upper Charles River basin, eastern Massachusetts.

[Site locations are shown in figure 3. The partial record stations were used to calculate flow statistics from equation 1. No., number; mi^2, square mile]

Constraint point location	Identifier	Model cell location			Drainage area (mi^2)	Partial record station	
		Layer	Row	Column		Station No.	Area
Charles River below Godfrey Brook	CR1	1	220	64	11.3	01103120	8.3
Charles River above Hopping Brook	CR2	1	214	155	25.4	01103206	21.0
Charles River Medway	CR3	1	204	220	65.2	01103280	65.2
Charles River Exit	CR4	1	215	264	83.8	01103305	83.8
Bogastow Brook Exit	BB1	1	132	243	20.8	01103393	20.9
Hopping Brook at Beaver Brook	HB1	1	157	117	7.5	01103217	9.9
End of Hopping Brook	HB2	1	212	155	11.0	01103217	9.9
Mine Brook 1/2 way	MB1	1	296	167	10.0	01103235	9.8
End of Mine Brook	MB2	1	208	173	15.9	01103240	14.2
End of Mill River	MR1	1	220	262	16.0	01103300	13.8

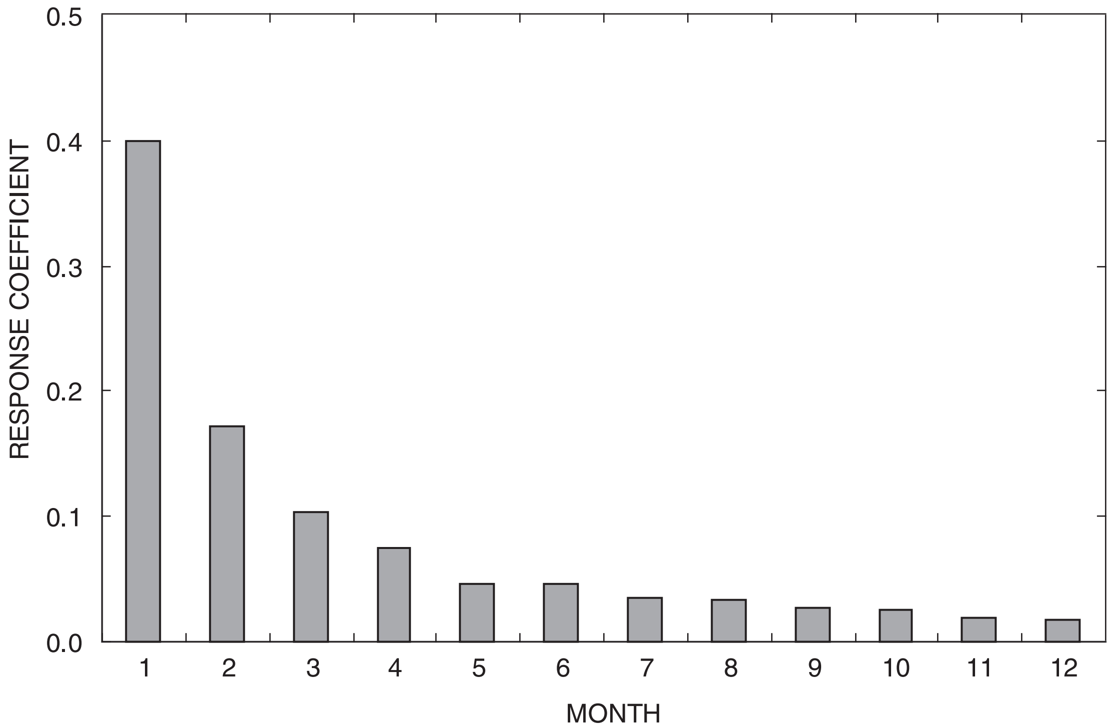


Figure 11. Response coefficients for streamflow depletions at the Charles River exit from the study area in response to pumping during month 1 at Wrentham well 2 (WR-02G in table 5 and figure 4).

Because simulated flow is in dynamic equilibrium, with monthly flows unchanging from year to year, hydrologic responses beyond month 12 were added to corresponding earlier months. For example, base-flow depletions in months 13 and 14 were added to base-flow depletions in months 1 and 2, and then equation 2 was used to calculate response coefficients for the summed depletions. Hydrologic responses with a magnitude of less than 1 percent of the stress perturbation were disregarded and not included in the response coefficient matrix. This exclusion causes small long-term responses to be ignored, but these small mass-balance errors are later corrected by scaling the coefficients.

Once the response coefficients are generated, all hydrologic simulations are then based on these coefficients rather than on the numerical model. This reduces errors and allows more efficient calculation of base-flow values under the stress conditions determined by optimization.

Wastewater disposal coefficients were set separately for each town by running a simulation that routed wastewater to septic and sewer discharge according to the proportions in table 6. Septic discharge was modeled as recharge in those areas with public-water supplies and private septic systems, with rates for each model cell set according to population density. The appropriate fraction of wastewater was discharged at the wastewater facility serving each town. Because the numerical model assumes that all stream water discharges in each stress period, any direct discharge of water to streams during month 1 has a response coefficient of -1.0 in month 1. Stream response to septic discharge typically lasts for several months because the wastewater takes time to move through aquifers to streams; this delay is expressed by the response coefficients. Water losses from supply pipes and inflow and infiltration of ground water into wastewater pipes were not explicitly modeled because insufficient data were available to quantify them.

Response coefficients for wastewater discharge were set separately according to time of year because of seasonality in consumptive losses. Changes in wastewater discharge from May through August were assumed to be only 70 percent of water withdrawal changes, following the previous use of 30-percent consumptive loss for those months. In the other 8 months of the year, it was assumed that there was no consumptive loss and that a change to water withdrawals would cause an equal change to wastewater discharge.

Water-Management Objectives and Constraints

Two types of objective function are used in this study to formulate management models: (1) maximizing water supplies to communities, and (2) maximizing water to streams. To maximize water to communities, pumping is maximized, typically only during June, July, and August when water demand is the highest (figs. 5 and 7). For example, the following objective function is used to maximize pumping by the town of Medway during June through August from the town’s three supply wells (MD-01G, MD-02G, MD-03G):

Maximize:

$$\begin{aligned}&Q06D1+Q07D1+Q08D1+Q06D2+Q07D2\\&+Q08D2+Q06D3+Q07D3+Q08D3.\end{aligned} \tag{3}$$

The variables Q--D1, Q--D2, and Q--D3 represent water withdrawals from the three Medway public-supply wells (table 5, fig.4).

To maximize water to streams, objective functions are formulated that minimize base-flow depletions during July, August, and September when streamflow falls particularly low (fig. 10). For example, to minimize total base-flow depletions at constraint point HB1 during July through September, the following objective function is used:

Minimize:

$$\begin{aligned}&- 1.0(D07HB) - 1.0(D08HB) - 1.0(D09HB)\\&+ 0.095(Q01H4) + 0.137(Q02H4) + 0.179(Q03H4)\\&+ 0.229(Q04H4) + 0.304(Q05H4) + 0.373(Q06H4)\\&+ 0.581(Q07H4) + 0.457(Q08H4) + 0.332(Q09H4)\\&+ 0.054(Q10H4) + 0.061(Q11H4) + 0.075(Q12H4)\\&- 0.014(R05HS) - 0.036(R06HS) - 0.122(R07HS)\\&- 0.108(R08HS) - 0.110(R09HW) - 0.069(W01HG)\\&- 0.080(W02HG) - 0.105(W03HG) - 0.147(W04HG)\\&- 0.207(W05HG) - 0.322(W06HG) - 0.735(W07HG)\\&- 0.658(W08HG) - 0.512(W09HG) - 0.049(W10HG)\\&- 0.055(W11HG) - 0.062(W12HG)\end{aligned} \tag{4}$$

where Q--H4 variables represent monthly water withdrawals from well HL-04G, the only well affecting flow at HB1 (table 5, fig. 4), and variables beginning with D, R, and W represent monthly water returns (table 7, fig. 6). For example, Q01H4 represents pumping at Holliston Well No. 4 in January, and R05HG represents wastewater discharge by Holliston in May. Equation 4 comprises all water stresses analyzed in the study that affect base flow at HB1 during July through September. After specifying constraints, a solution to the problem can be found that specifies values for each variable in equation 4.

The management models employ a variety of constraints. Constraints are placed to limit pumping rates so that individual wells honor State-mandated maximum pumping rates (monthly Zone II limits) and groups of wells honor maximum annual average pumping rates imposed by the WMA limits. Holliston and Medway are excluded from the WMA constraints because their annual average (1989–98) pumping rates exceed WMA limits (table 8); instead, they are limited to past average pumping rates. Constraints are placed to ensure that, for each town, mass balance of wastewater disposal against water withdrawals is achieved. Constraints are placed to ensure that each town receives a minimum amount of water, generally equal to past average monthly pumping rates.

Constraints are also placed to limit streamflow depletions. The constraint levels were chosen in consultation with the project's technical advisory committee and reflect compromises between the higher 0.50 $ft^3/s/mi^2$ value of the ABF and the lower 0.21 $ft^3/s/mi^2$ value used in some wastewater permits for the basin. As a default condition, base flow is constrained to be greater than or equal to either 0.41 $ft^3/s/mi^2$ or average flow under no-pumping conditions, whichever is less. For example, flow at CR1 in January would be constrained to be greater than or equal to 0.41 $ft^3/s/mi^2$, because 0.41 is less than the average flow of 0.96 $ft^3/s/mi^2$ at CR1 in January under average no-pumping conditions (table 3). In August, however, flow at CR1 would be constrained to be above only 0.26 $ft^3/s/mi^2$, the average no-pumping flow, because it is less than 0.41. It is important to note that these constraints require higher than average flow during the summer, because average flow (tables 1 and 2) is generally lower than no-pumping flow (tables 3 and 4). For some management models, the low-flow constraints cannot be met. In some of these cases, a lower flow was allowed so that a solution to the management model could be obtained and a better understanding of the management alternatives would ensue. The low-flow constraint levels used in this study were chosen only for purposes of analysis.

Response Coefficient Errors

Errors may be caused by linearity assumptions inherent to the response coefficient approach. For example, when a stream goes dry or when a flow boundary causes nonlinear head responses, hydrologic responses calculated with response coefficients will have errors. One potential source of error is the assumption that aquifer transmissivity remains constant as head changes; in unconfined aquifers however, it does not remain constant (Reifler and Ahlfeld, 1996). Numerical errors in simulation results, such as round-off errors, may also introduce error to the response coefficients.

Errors in the response coefficient can be classified into two types: (1) mass balance and (2) timing errors. Mass balance considerations dictate that response coefficients for a given stress sum to 1.0. For example, if 2.0 ft^3/s is withdrawn from well X in month 1, then base-flow depletions downstream of well X sum to 2.0 ft^3/s during the time it takes for ground-water storage to return to its pre-pumping state. If total base-flow depletions do not equal pumping, then the water mass is not balanced. To remove mass balance errors, all response coefficients calculated in equation 2 were scaled so that equation 3 was honored for all stress (j) and stream (i) locaion pairs:

$$\sum_{k=1}^{n} r_{i,j,k} = 1.0 \tag{5}$$

where k is the month index and n is the number of months in which base-flow depletion occurs.

In a few cases, response coefficients had the wrong sign, as for example when base flow increased in response to ground-water withdrawals. This type of response does not have a plausible physical explanation and is attributable to numerical errors in the ground-water simulation. Other errors of this type include nonmonotonic changes in base-flow depletion in response to a single stress where, for example, base-flow depletions rise, then fall, and then rise again. Because ground-water flow is described by the diffusion equation, a single prior stress of a fixed rate cannot cause this type of nonmonotonic response sequence, and the response is attributable to numerical errors. In these cases of obvious errors in the response coefficients, base-flow depletions were estimated by linear interpolation between their values before and after the month(s) with spurious values. Errors for approximately 11 percent of response coefficients were corrected in this way, with most errors in later months when responses are a small fraction of total response.

Errors in timing of response coefficients occur if base-flow depletions last for too many or too few months or if depletions are distributed incorrectly over the correct months. The uncertainty associated with these errors was quantified by measuring timing variations. For each stress (j) and stream (i) location pair, four sets of $r_{i,j,k}$ were generated, with k being the month index. The following conditions defined the four sets:

1. High stress imposed in August under average recharge.
2. Low stress (25 percent of high stress) imposed in August under average recharge.
3. High stress imposed in February under average recharge.
4. High stress imposed in August under low recharge (41 percent of average).

Each set of $r_{i,j,k}$ should be the same under the assumption of linear response. To quantify how timing of the response coefficients varied, $T_{5\%}$ was defined as the last month in which stream depletions are more than 5 percent of pumping. As an example, for the response coefficients shown in figure 11, $T_{5\%}$ = month 4. Because of simulation errors, the four sets of $r_{i,j,k}$ sometimes did not yield the same $T_{5\%}$. The mean absolute departure of $T_{5\%}$ from the group average $T_{5\%}$ was 1.5 months.

This value is the uncertainty in the timing of the coefficients, which cannot be removed. The uncertainty is increased primarily by cases 2 and 4 above. In case 2, the stress perturbation ($Qsd_{j,k}$) on the right-hand side of equation 2 is quite small. The number of significant digits in the numerical solution is thus decreased, adding to the uncertainty of $r_{i,j,k}$ (Riefler and Ahlfeld, 1996).
In case 4, low recharge causes nonlinearity because stream flow is smaller and some stream cells go dry. If other cases that imposed higher stress perturbations and precluded dry conditions were used instead of cases 2 and 4 above, the mean absolute departure of $T_{5\%}$ would decrease.

The use of response coefficients allows base-flow changes to be calculated outside of the simulation model and allows some numerical errors to be avoided. Starting with the MOVE.1 flow statistics (average and 90-percent low flow), changes from these statistical flows caused by stresses are calculated with response coefficients. Therefore, instead of unknown model errors in the base-flow values for each scenario, there are known MOVE.1 uncertainties and response coefficient errors.

Errors in the response coefficients affect solutions to water-resource management problems. Errors of mass balance have been removed from the response coefficients, but some timing errors remain, adding uncertainty to any solution based on the response coefficients. In most cases, the uncertainty is small enough to be acceptable. In small streams and under low-flow conditions, uncertainty is more important because base-flow depletions from a single pump can be a large percentage (50 percent or more) of total streamflow. Additional site-specific data collection and modeling for small streams might reduce uncertainty due to errors in response coefficients.

Analysis of Management Alternatives Using Simulation-Optimization

A variety of scenarios was considered to assist water-resources managers in development of strategies for addressing water-supply shortages and low streamflow. All scenarios use response coefficients as previously discussed. Some scenarios additionally use optimization methods. The scenarios were selected in consultation with the project's technical advisory committee, which is composed of State environmental officials, town water officials, engineering consultants to towns, and representatives of conservation groups. The scenarios are numbered from 1A to 9A and are summarized in table 12.

Maximize Pumping

Demand for drinking water in the study area is expected to rise. If future water demands are met by increasing ground-water withdrawals, base flow will be affected. To examine this possibility, base flow is calculated for the case in which pumping at existing municipal wells increases to maximum permitted levels under monthly Zone II limits (Scenario 1A). Under these conditions, pumping increases to 24.1 ft^3/s, up from the past average annual rate of 10.0 ft^3/s. WMA limits, which cap annual average pumping at 15.5 ft^3/s, were not considered in this analysis due to uncertainty about adjusting WMA limits for towns with additional supply wells outside the basin. Pumping at all existing municipal wells was increased to maximum permitted Zone II levels for every month of the year and septic and wastewater discharge were correspondingly increased. The resulting base-flow values were calculated by use of response coefficients and are shown in table 13.

Base flow is generally lower under maximum permitted pumping than under average pumping (table 1); this result is to be expected because 30 percent of summertime pumping is lost to consumptive use and thus does not recharge the aquifer. Seven of the 10 constraint points show lower base flow in every month of the year, with the Charles River at CR1 drying up from July through September. The flow is higher in every month at only one constraint point (MR1); this higher flow is caused by increased septic discharge in the Mill River subbasin.

Under 90-percent low-flow conditions, increasing ground-water withdrawals from average (table 2) to maximum permitted rates (Scenario 1B) causes the same depletions in base flow. The depletions have greater importance, however, because flow is already so low. Under average pumping only Hopping Brook (HB1 and HB2) goes dry; but under maximum permitted pumping, Bogastow Brook (BB1), Mine Brook (MB1, MB2), and parts of the Charles River (CR1, CR3) also go dry for at least 1 month (table 14).

Low flows under average, no-pumping, and maximum pumping conditions are summarized in table 15. The numbers indicate that water use contributes to low-flow problems in the basin; if there were no water use, however, flow would still drop below the 0.50 $ft^3/s/mi^2$ ABF standard for healthy freshwater ecosystems in New England.

Table 12. Summary of water-resource management scenarios analyzed, upper Charles River basin, eastern Massachusetts.

[**Scenario:** All scenarios employ response coefficients to calculate base flow. CRPCD, Charles River Pollution Control District; =, equals; ≥, greater than or equal to; --, indicates optimization was not used]

Scenario	Climatic conditions	Management action	Optimization to calculate pumping rates		Results
			Maximize	Notable constraints	
1A	Average	Increase pumping	--	--	Decreased base flow, increased water supply
1B	Dry	Increase pumping	--	--	Decreased base flow, increased water supply
2A	Average	Maximize summer base flow	Summer base flow	Water supply ≥ existing, base flow ≥ no-pumping flow	No feasible solution
2B	Average	Maximize summer base flow	Summer base flow	Water supply ≥ existing, base flow ≥ average flow	Base flow increases
3A	Average	Proposed wells pumping at maximum permitted rates	--	--	Decreased base flow, increased water supply
3B	Average	Proposed wells allowed	Summer base flow	Water supply = existing, base flow ≥ no-pumping flow	No feasible solution
3C	Average	Proposed wells allowed	Summer base flow	Water supply = existing, base flow ≥ average flow	Base flow increases
3D	Dry	Proposed wells allowed	Summer base flow	Water supply = existing, base flow ≥ dry flow	Base flow increases
4A	Average	Holliston sewage to CRPCD	--	--	Base flow decreases
4B	Average	Holliston sewage to local recharge	--	--	Base flow unchanged
4C	Average	Bellingham sewage to recharge	--	--	Base flow unchanged
4D	Average	Choice of recharge basins at the Miscoe Brook site	Summer base flow	Direct wastewater discharge to streams is possible	Discharge directly to stream in summer, to distant recharge basins in other months
4E	Average	Recharge basin added to CRPCD	Summer base flow	Direct wastewater discharge to streams is possible	Base flow increases
5A	Average	Wastewater discharged upstream	--	--	Base flow increases
6A	Average	Stormwater recharge	--	--	Base flow increases
7A	Average	Water conservation (10 percent)	--	--	Increased base flow, increased water supply
8A	Average	Towns share drinking water	Summer base flow	Water supply = existing, base flow ≥ no-pumping flow	Increased base flow, increased water supply
8B	Dry	Towns share drinking water	Summer base flow	Water supply = existing, base flow ≥ dry flow	Increased base flow, increased water supply
9A	Average	Drinking water discharge to streams	Summer base flow	Water supply = existing	Base flow increases

Table 13. Base flow under Scenario 1A, average climatic conditions and maximum permitted pumping, upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	6.7	7.5	10.3	10.4	5.5	0.6	0.0	0.0	0.0	1.2	3.5	6.7
CR2	41.3	43.9	54.3	53.0	36.4	20.5	9.3	9.6	11.3	20.7	31.3	40.2
CR3	116.1	131.9	171.7	172.5	105.5	58.2	22.9	19.2	19.9	42.8	79.9	113.1
CR4	159.3	169.3	202.4	200.0	144.5	86.3	48.5	49.5	50.8	85.2	121.8	156.9
BB1	37.7	40.3	55.8	55.7	34.5	14.6	5.2	5.4	5.0	13.6	24.4	36.6
HB1	18.1	22.4	31.1	30.3	15.3	3.8	.8	.9	1.0	4.0	8.6	17.6
HB2	27.0	33.5	46.3	45.2	23.0	5.9	1.3	1.6	1.7	6.1	12.9	26.3
MB1	16.8	18.6	21.7	21.5	15.6	8.3	3.8	4.0	3.9	8.0	11.8	16.5
MB2	31.9	35.9	43.3	43.0	29.3	15.5	6.7	6.1	5.8	13.1	21.1	30.8
MR1	36.1	38.9	43.4	43.3	34.2	23.6	15.3	14.4	14.2	21.2	28.0	35.2
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.60	0.67	0.91	0.92	0.49	0.05	0.00	0.00	0.00	0.10	0.31	0.59
CR2	1.63	1.73	2.14	2.09	1.43	.81	.37	.38	.44	.82	1.23	1.58
CR3	1.78	2.02	2.64	2.65	1.62	.89	.35	.29	.31	.66	1.23	1.74
CR4	1.90	2.02	2.41	2.39	1.72	1.03	.58	.59	.61	1.02	1.45	1.87
BB1	1.82	1.94	2.68	2.68	1.66	.70	.25	.26	.24	.66	1.17	1.76
HB1	2.43	3.01	4.17	4.07	2.06	.51	.10	.12	.14	.53	1.15	2.36
HB2	2.45	3.03	4.20	4.09	2.08	.53	.12	.14	.16	.55	1.17	2.38
MB1	1.68	1.86	2.16	2.14	1.56	.83	.38	.40	.39	.80	1.18	1.65
MB2	2.01	2.26	2.73	2.71	1.85	.98	.42	.38	.37	.82	1.33	1.94
MR1	2.26	2.43	2.72	2.71	2.14	1.48	.96	.90	.89	1.33	1.75	2.20

Maximize Streamflow

Although current water use decreases summertime streamflow, pumping schedules could potentially be changed to increase summer flow. In Scenario 2A, streamflow is maximized by specifying an objective function that minimizes base-flow depletions at constraint points CR4 and BB1, the stream outlets from the basin:

Minimize: Total streamflow depletions at CR4 and BB1, the exit points from the basin, during July, August, and September.

Constraints:

1. Base flow must be greater than or equal to average flow under no-pumping conditions or 0.41 $ft^3/s/mi^2$, whichever is less.
2. Each town's monthly water supply is equal to average (1989–98) rates.
3. Towns do not share water with each other.
4. Withdrawals are at or below State-mandated maximum pumping limits (monthly Zone II limits and annual WMA limits, except in Holliston and Medway).

No future or hypothetical projects are included and July, August, and September all have equal weight in the objective function. Pumping rates at non-municipal wells and septic and wastewater discharges are fixed at average (1989–98) monthly rates.

There is no solution to the optimization problem because no pumping schedule can be found that allows base flow to meet all the constraints. A feasible solution can be found if constraint (1) is relaxed so that base flow can drop as low as average existing base flow, a less stringent condition for the summer months. In this case (Scenario 2B), a solution of optimal pumping rates is found (table 16). When these optimal pumping rates are imposed, the base-flow values in table 17 result. Monthly flow during July through September increases by an average of 2.6 ft^3/s (0.03 $ft^3/s/mi^2$) at CR4 and 0.4 ft^3/s (0.02 $ft^3/s/mi^2$) at BB1 over existing flow (table 1).

Table 14. Base flow under Scenario 1B, dry (90-percent low flow) climatic conditions and maximum permitted pumping, upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	0.8	2.1	6.6	3.5	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CR2	20.7	26.9	41.0	30.7	20.3	8.3	1.9	.6	2.2	4.9	12.1	18.5
CR3	31.1	45.2	78.2	51.5	31.4	10.8	.0	.0	.6	3.4	13.4	23.5
CR4	83.5	106.8	158.3	122.2	85.1	42.7	18.2	12.7	15.0	23.2	51.6	74.5
BB1	12.3	18.6	37.1	24.9	14.3	4.5	.7	.0	.0	.9	5.0	10.6
HB1	4.1	6.8	18.8	9.1	4.9	.4	.0	.0	.0	.0	.8	2.0
HB2	6.3	10.3	28.0	13.8	7.4	.8	.0	.0	.1	.3	1.3	3.1
MB1	8.1	10.5	17.1	12.2	9.0	2.8	.1	.0	.0	.4	3.1	5.4
MB2	13.4	18.3	32.1	21.9	15.4	4.2	.2	.0	.0	.5	4.2	8.2
MR1	22.3	26.6	36.4	29.2	23.8	12.7	6.9	5.6	5.4	7.5	12.7	17.3
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.07	0.19	0.59	0.31	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CR2	.81	1.06	1.61	1.21	.80	.33	.07	.02	.08	.19	.48	.73
CR3	.48	.69	1.20	.79	.48	.17	.00	.00	.01	.05	.21	.36
CR4	1.00	1.27	1.89	1.46	1.02	.51	.22	.15	.18	.28	.62	.89
BB1	.59	.89	1.79	1.20	.69	.22	.03	.00	.00	.04	.24	.51
HB1	.55	.91	2.52	1.23	.65	.05	.00	.00	.00	.00	.10	.26
HB2	.57	.93	2.54	1.25	.67	.07	.00	.00	.01	.02	.12	.28
MB1	.80	1.04	1.70	1.21	.89	.27	.01	.00	.00	.04	.30	.53
MB2	.85	1.15	2.02	1.38	.97	.27	.01	.00	.00	.03	.26	.52
MR1	1.39	1.66	2.28	1.83	1.49	.79	.43	.35	.34	.47	.79	1.08

Table 15. Low flows at the 10 constraint points under different pumping stresses, upper Charles River basin, eastern Massachusetts.

[**Average low monthly flow:** Values are the means of lowest monthly flows from each of the 10 constraint points shown in figure 3 and are based on monthly values in the tables indicated. $ft^3/s/mi^2$, cubic foot per second per square mile]

Climatic condition	Average low monthly flow ($ft^3/s/mi^2$)			Climatic condition	Average low monthly flow ($ft^3/s/mi^2$)		
	Average pumping	No pumping	Maximum permitted pumping (Zone II)		Average pumping	No pumping	Maximum permitted pumping (Zone II)
Average climatic conditions				**Dry climatic conditions (90-percent flow duration)**			
Average low flow	[a]0.41	[b]0.46	[c]0.33	Average low flow	[d]0.10	[e]0.15	[f]0.05
Lowest flow	[a g]0.14	[b g]0.12	[c h]0.00	Lowest flow	[d g]0.00	[e g].00	[f i]0.00

[a]From table 1.
[b]From table 3.
[c]From table 13.
[d]From table 2.
[e]From table 4.
[f]From table 14.
[g]Hopping Brook (HB1 and HB2).
[h]Charles River in Milford (CR1).
[i]All constraint points (CR1, CR3, BB1, HB1, HB2, MB1, and MB2) except the Mill River (MR1) and parts of the Charles River (CR2 and CR4).

Table 16. Pumping rates to maximize summertime base flow, Scenario 2B, average climatic conditions, no proposed or hypothetical stresses, upper Charles River basin, eastern Massachusetts.

[Site locations are shown in figure 4. Scenarios are described in table 12. Town water supplies and septic and wastewater discharge are equal to average rates (1989–98). Nonmunicipal withdrawals were not optimized, but set equal to average rates (1989–98). Base flow was optimized only from July through September and only at BB1 and CR4. All rates in cubic foot per second]

Withdrawal stress	Optimal monthly withdrawal rate and change from average (1989–98) rate											
	January		February		March		April		May		June	
	Rate	Change	Rate	Change	Rate	Change	Rate	Change	Rate	Change	Rate	Change
Bellingham Municipal Withdrawals												
Q--B5	0.45	+0.17		-0.30		-0.32		-0.32		-0.31		-0.30
Q--B7	.21	+.09	0.66	+.55		-.13		-.15		-.28	0.30	+.02
Q--B8		-.25		-.32	0.82	+.46	0.80	+.48	1.05	+.56	1.05	+.31
Q--B9			.10	+.07		-.01		-.02	.06	+.03		-.03
Medway Municipal Withdrawals												
Q--D1	.58		.58	-.04	.30	-.39	.26	-.35	.48	-.13	.58	-.13
Q--D2		-.15		-.15		-.10		-.16		-.26	.16	-.15
Q--D3	.65	+.14	.65	+.19	.94	+.50	.94	+.51	.94	+.39	.94	+.28
Franklin Municipal Withdrawals												
Q--F1		-.08		-.11		-.13		-.06	.49	+.36	.69	+.37
Q--F2		-.08		-.11		-.13		-.06		-.13	1.10	+.78
Q--F3		-.40		-.38	.50	+.07	.50	+.02		-.55		-.57
Q--F4	.75	-.21	.71	-.24	.24	-.63		-1.02		-1.14		-1.17
Q--F5	.77	+.52	.77	+.52	.77	+.51	.77	+.56	.77	+.49		-.30
Q--F6	.82	+.25	.82	+.25	.82	+.27	.82	+.21	.82	+.19	.82	+.19
Q--F7		-.43		-.42		-.42	.38	-.09	.89	+.41	.89	+.39
Q--F8	.41	-.07	.41		.41	-.03	.41	-.04	.41	-.09	.41	-.09
Q--FT	.78	+.48	.78	+.49	.78	+.48	.78	+.48	.78	+.45	.78	+.40
Holliston Municipal Withdrawals												
Q--H1								-.02	.49	+.39	.49	+.31
Q--H2	.48	+.47	.48	+.46	.48	+.43	.48	+.45	.48	+.36	.48	+.26
Q--H4		-.12		-.14		-.15		-.11		-.20	.43	+.05
Q--H5		-.71		-.62		-.61		-.70		-.81	1.09	+.33
Q--H6	1.07	+.36	.99	+.29	1.01	+.32	1.08	+.38	1.00	+.27		-.95
Milford Municipal Withdrawals												
Q--M1				-.01								
Q--M2	.17	+.17	.02									
Q--M3	1.23	+1.04	1.23	+1.05	.91	+.75		-.15		-.18		-.17
Q--M4		-.19		-.18		-.16	.91	+.76	.94	+.77	.96	+.79
Q--M5		-.19		-.18		-.16		-.15		-.18		-.17
Q--M6		-.81		-.67		-.42		-.45		-.42		-.46
Norfolk Municipal Withdrawals												
Q--N1	.28		.28		.33		.44		.44		.52	
Wrentham Municipal Withdrawals												
Q--W2		-.48		-.48		-.47		-.46		-.71		-.60
Q--W3	1.04	+.48	1.07	+.48	.99	+.47	1.23	+.46	1.21	+.71	1.42	+.60

Table 16. Pumping rates to maximize summertime base flow, Scenario 2B, average climatic conditions, no proposed or hypothetical stresses, upper Charles River basin, eastern Massachusetts.—Continued

[Site locations are shown in figure 4. Scenarios are described in table 12. Town water supplies and septic and wastewater discharge are equal to average rates (1989–98). Nonmunicipal withdrawals were not optimized, but set equal to average rates (1989–98). Base flow was optimized only from July through September and only at BB1 and CR4. All rates in cubic foot per second]

Withdrawal stress	Optimal monthly withdrawal rate and change from average (1989–98) rate											
	July		August		September		October		November		December	
	Rate	Change	Rate	Change	Rate	Change	Rate	Change	Rate	Change	Rate	Change
Bellingham Municipal Withdrawals												
Q--B5	0.45	+0.13		-0.32	0.45	+0.14		-0.31	0.45	+0.15	0.45	+0.16
Q--B7	.66	+.41	0.66	+.48		-.11		-.11		-.10		-.07
Q--B8		-.75		-.65		-.35		-.32		-.28		-.25
Q--B9	.28	+.21	.54	+.48	.37	+.33	0.76	+.73	.26	+.23	.19	+.16
Medway Municipal Withdrawals												
Q--D1	.58	-.08	.53	-.21		-.70	.42	-.12	.28	-.32	.35	-.21
Q--D2	.91	+.58	.91	+.66	.91	+.69		-.24		-.17		-.18
Q--D3	.18	-.50		-.45	.48		.94	+.36	.94	+.49	.94	+.38
Franklin Municipal Withdrawals												
Q--F1	.69	+.39	.69	+.40	.69	+.43		-.24		-.16		-.06
Q--F2	1.11	+.82	1.11	+.82	.98	+.71		-.24		-.16		-.06
Q--F3	.48	-.05	.22	-.29		-.50		-.43		-.40		-.43
Q--F4	1.43	+.28	1.43	+.32	1.43	+.38	1.43	+.44	1.43	+.45	1.43	+.45
Q--F5		-.32		-.29		-.24	.76	+.53	.77	+.52	.77	+.48
Q--F6		-.60		-.57		-.45	.82	+.38	.09	-.25	.06	-.30
Q--F7	.89	+.40	.89	+.45	.89	+.48		-.39		-.38		-.43
Q--F8		-.51		-.49		-.49		-.46	.41	-.04	.41	-.06
Q--FT		-.41		-.37		-.32	.78	+.42	.78	+.41	.78	+.42
Holliston Municipal Withdrawals												
Q--H1	.49	+.26	.49	+.31		-.13		-.04		-.03		
Q--H2		-.28		-.23		-.08	.48	+.45	.48	+.46	.48	+.47
Q--H4	.66	+.24	.40	+.06		-.27		-.20		-.18		-.14
Q--H5	1.09	+.35	1.09	+.40	1.09	+.46		-.64		-.66		-.72
Q--H6	.23	-.56	.14	-.55	.69	+.02	1.09	+.43	.98	+.41	1.00	+.39
Milford Municipal Withdrawals												
Q--M1	1.05	+1.03	1.05	+1.02	1.05	+1.03	.21	+.19	.24	+.21	.13	+.11
Q--M2		-.01		-.03		-.01		-.02		-.03		-.02
Q--M3		-.18		-.18		-.22		-.20		-.22		-.22
Q--M4		-.18		-.18		-.22		-.20		-.22		-.22
Q--M5		-.18		-.18		-.22		-.20		-.22		-.22
Q--M6	.28	-.47	.46	-.45	.47	-.37	1.24	+.44	1.24	+.46	1.24	+.58
Norfolk Municipal Withdrawals												
Q--N1	.48		.37		.38		.36		.35		.31	
Wrentham Municipal Withdrawals												
Q--W2	1.11	+.39	1.11	+.56	1.11	+.77		-.44		-.30		-.30
Q--W3	.39	-.39	.14	-.56	.04	-.77	.91	+.44	.90	+.30	.95	+.30

Table 17. Base flow under optimal pumping to maximize base flow, Scenario 2B, average climatic conditions, no proposed or hypothetical stresses, upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	8.9	9.8	13.0	13.3	8.9	4.1	2.3	2.3	2.1	4.1	6.3	9.4
CR2	41.7	44.5	55.0	53.8	39.3	23.5	13.0	13.2	12.9	22.0	32.3	41.1
CR3	118.0	133.9	173.6	174.5	110.2	61.7	28.8	25.1	23.4	45.4	82.6	115.7
CR4	155.8	166.0	199.3	197.4	146.1	88.6	53.5	54.5	52.2	84.1	119.6	154.2
BB1	38.1	40.8	56.2	56.2	35.1	15.5	6.5	6.7	6.0	14.2	24.8	37.0
HB1	18.4	22.7	31.4	30.6	15.8	4.2	1.0	1.1	1.2	4.2	8.8	17.8
HB2	27.1	33.6	46.4	45.3	23.2	6.1	1.5	1.7	1.8	6.1	13.0	26.3
MB1	18.8	20.6	23.5	23.2	17.5	9.9	6.1	6.4	6.3	9.8	13.9	18.8
MB2	34.3	38.4	45.7	45.3	31.7	17.5	9.4	8.9	8.6	15.2	23.7	33.6
MR1	35.3	37.4	42.1	42.2	33.6	23.7	15.6	14.7	14.3	20.8	27.1	33.9
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.79	0.87	1.15	1.18	0.79	0.36	0.21	0.20	0.19	0.37	0.56	0.84
CR2	1.64	1.75	2.16	2.12	1.55	.93	.51	.52	.51	.87	1.27	1.62
CR3	1.81	2.05	2.66	2.68	1.69	.95	.44	.39	.36	.70	1.27	1.78
CR4	1.86	1.98	2.38	2.35	1.74	1.06	.64	.65	.62	1.00	1.43	1.84
BB1	1.83	1.96	2.70	2.70	1.69	.75	.31	.32	.29	.68	1.19	1.78
HB1	2.46	3.05	4.21	4.11	2.11	.56	.14	.15	.16	.56	1.18	2.39
HB2	2.46	3.04	4.21	4.11	2.11	.56	.14	.15	.16	.55	1.18	2.39
MB1	1.87	2.05	2.34	2.31	1.75	.99	.61	.64	.63	.98	1.39	1.88
MB2	2.16	2.42	2.88	2.85	1.99	1.10	.59	.56	.54	.96	1.49	2.12
MR1	2.21	2.34	2.63	2.64	2.10	1.48	.98	.92	.89	1.30	1.70	2.12

These increases in summertime base flow are achieved by exploiting differences in timing of streamflow response to different wells. Wells close to streams with short response times are pumped preferentially from October through January; this schedule allows the resulting streamflow depletions to take place before the dry months arrive. Wells far from streams with long response times are pumped preferentially from February through September, when a portion of the resulting streamflow depletions will take place after the low-flow period has passed. This can be seen in the optimal pumping rates for Medway's wells (table 16). Well MD-02G (stress Q--D2), the Medway well with the most delayed streamflow depletion effects, is pumped during July, August, and September at the highest rate allowed by Zone II limits, 0.91 ft^3/s. Well MD-01G (stress Q--D1), which causes the most streamflow depletion of any of Medway's wells when pumped in September, is assigned to be shut down for that month. Well MD-03G (stress Q--D3) is pumped in September at a rate just high enough to meet the remainder of Medway's required water supply of 1.39 ft^3/s for that month. In October, just past the period of critically low streamflow, wells MD-03G and MD-01G meet the water-supply demand of Medway. Well MD-02G, the only Medway well whose October pumping depletes streamflow during the following July through September period, is shut down. That seasonal pumping patterns can be adjusted to augment summer base flow is also noted by Barlow and others (2003).

The pumping rates in table 16 indicate how pumping could be managed to achieve the greatest increase in base flow during the summer. The pumping rates from February through September are higher in wells that are far from streams and have long response times and, from October through January, are higher in wells that are close to streams and have short response times. Towns whose wells all produce similar streamflow response times, whether long or short, have less management flexibility, which could be addressed by installing new wells.

Proposed Wells

Several towns in the study area have applied for permits to install new municipal water-supply wells. Pumping from these proposed wells would increase withdrawals permitted under Zone II regulations from 24.1 to 31.9 ft^3/s and would alter the current hydrologic system. To estimate how this increased pumping would affect base flow, the response coefficient approach was applied (Scenario 3A). WMA limits were not included in the analysis because proposed WMA changes may or may not be issued concurrently with Zone II changes. The base-flow values shown in table 18 result when all existing and proposed wells pump at Zone II-permitted maximum rates for all months and septic, and wastewater disposal is correspondingly increased.

Differences between base flow in table 13 and table 18 indicate that increasing permitted withdrawals would cause substantial base-flow depletions if the additional permitted capacity were fully utilized. Under 90-percent low-flow conditions, the same base-flow depletions would take place and would be a larger percentage of total flow.

New wells also have the potential to increase summertime base flow, particularly if water use does not increase. Optimization was used to test how proposed wells could be managed along with existing supply wells to increase base flow during critical summer months (Scenario 3B). As with earlier scenarios, base-flow depletions were minimized at exits from the basin.

Minimize: Streamflow depletions at CR4 and BB1 during July, August, and September.

Constraints:

1. Base flow must be greater than or equal to average flow under no-pumping conditions or 0.41 $ft^3/s/mi^2$, whichever is less.
2. Each town gets at least as much water every month as the average for the period 1989–98.
3. Towns do not share water with each other.
4. Pumping from each well is at or below State-mandated maximum pumping limits (monthly Zone II limits only, no WMA limits included).

Table 18. Base flow with all wells including proposed municipal supply wells pumping at maximum permitted rates, Scenario 3A, average climatic conditions, upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	6.7	7.5	10.3	10.4	5.5	0.6	0.0	0.0	0.0	1.2	3.5	6.7
CR2	41.3	43.9	54.3	53.0	36.4	20.5	9.3	9.6	11.3	20.7	31.3	40.2
CR3	116.2	132.0	171.8	172.6	105.3	57.9	22.6	18.9	19.8	42.8	79.9	113.1
CR4	160.4	170.6	203.7	201.3	144.9	86.4	48.3	49.1	51.2	85.7	122.5	157.9
BB1	36.7	39.3	54.8	54.7	33.4	13.5	4.0	4.2	3.9	12.6	23.4	35.5
HB1	18.3	22.7	31.4	30.6	15.5	4.0	1.0	1.1	1.3	4.2	8.8	17.8
HB2	27.4	33.8	46.6	45.5	23.2	6.2	1.6	1.8	2.1	6.4	13.3	26.6
MB1	15.8	17.5	20.6	20.4	14.5	7.2	2.7	2.9	2.9	7.0	10.7	15.5
MB2	30.9	35.0	42.3	42.1	28.3	14.5	5.7	5.1	4.8	12.0	20.1	29.8
MR1	36.5	39.5	44.0	43.9	34.5	23.7	15.1	14.0	14.0	21.0	28.1	35.4
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.60	0.67	0.91	0.92	0.49	0.05	0.00	0.00	0.00	0.10	0.31	0.59
CR2	1.63	1.73	2.14	2.09	1.43	.81	.37	.38	.44	.82	1.23	1.58
CR3	1.78	2.03	2.64	2.65	1.62	.89	.35	.29	.30	.66	1.23	1.74
CR4	1.91	2.03	2.43	2.40	1.73	1.03	.58	.59	.61	1.02	1.46	1.88
BB1	1.77	1.89	2.63	2.63	1.61	.65	.19	.20	.19	.60	1.12	1.71
HB1	2.46	3.04	4.21	4.10	2.09	.54	.13	.15	.17	.56	1.19	2.39
HB2	2.48	3.07	4.23	4.13	2.11	.56	.15	.17	.19	.58	1.20	2.41
MB1	1.57	1.75	2.05	2.04	1.45	.72	.27	.29	.29	.69	1.07	1.54
MB2	1.95	2.20	2.66	2.65	1.78	.91	.36	.32	.30	.76	1.26	1.88
MR1	2.28	2.47	2.76	2.75	2.16	1.48	.94	.87	.87	1.31	1.76	2.22

The formulation does not include future projects other than proposed wells. July, August, and September all have equal weight in the objective function. Because base flow is maximized only at constraint points CR4 and BB1, flow at other constraint points may decrease.

Solution 1:

With the addition of new wells, base flow cannot be managed to meet all the constraints. So, as in Scenario 2B, constraint (1) is relaxed to allow base flow to drop as low as average existing base flow (Scenario 3C). A feasible pumping schedule is found that increases base flow during the months of July through September (table 19) by an average of 4.6 ft^3/s (0.06 $ft^3/s/mi^2$) at CR4 and by an average of 0.5 ft^3/s (0.02 $ft^3/s/mi^2$) at BB1 as compared to base flow under average pumping rates. Only about 40 percent of this flow increase (2.0 out of 4.6 ft^3/s), however, is attributable to the addition of the proposed wells, with the remainder of the increase (2.6 of 4.6 ft^3/s) attributable to optimized management of pumping schedules at existing wells as described for Scenario 2B.

Solution 2:

Under low-flow conditions (90-percent flow duration) there is no feasible solution to the optimization problem because low base flow violates constraint 1. To avoid this problem, constraint 1 is modified to allow lower base flow (Scenario 3D).

Constraints:

1. Base flow must be greater than or equal to 90-percent low flow under no-pumping conditions or 0.21 ft^3/s, whichever is less.

2–4. Same as in Scenario 3B.

The optimal solution to Scenario 3D increases base flow during the months of July through September at CR4 and BB1 by nearly the same amounts (4.7 and 0.5 ft^3/s) as under average precipitation (Scenario 3C). As in Scenario 3C, only about 40 percent of the base-flow increase is attributable to the addition of the proposed wells (table 20).

Table 19. Base flow maximized under optimal pumping with proposed wells active, Scenario 3C, average climatic conditions, towns receive average (1989–98) water supplies, upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	9.0	10.0	13.1	13.4	9.0	4.0	2.3	2.3	2.1	4.1	5.6	9.3
CR2	42.7	45.5	56.0	54.8	40.2	24.4	13.8	13.9	13.8	22.8	32.6	42.1
CR3	118.6	134.6	174.6	175.6	111.0	63.3	30.0	25.9	24.3	46.5	82.9	116.1
CR4	156.3	166.6	200.2	198.1	146.4	89.4	55.5	56.4	54.2	85.1	119.9	154.4
BB1	37.9	40.6	56.2	56.1	35.4	15.9	6.7	6.7	6.1	13.8	24.2	36.4
HB1	18.4	22.7	31.4	30.6	15.5	4.0	1.1	1.2	1.1	4.1	8.8	17.8
HB2	27.1	33.6	46.4	45.3	23.0	5.9	1.5	1.8	1.7	6.1	12.9	26.3
MB1	18.5	20.3	23.4	23.2	17.6	10.6	6.4	6.4	6.4	10.2	14.0	18.2
MB2	34.1	38.2	45.6	45.4	31.8	18.3	9.8	8.9	8.7	15.7	23.8	33.0
MR1	35.1	37.8	42.4	42.2	33.6	23.6	15.9	15.2	15.0	20.9	27.1	34.0
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.80	0.89	1.17	1.19	0.80	0.36	0.21	0.20	0.19	0.37	0.50	0.83
CR2	1.68	1.79	2.20	2.16	1.58	.96	.54	.55	.54	.90	1.28	1.66
CR3	1.82	2.07	2.68	2.70	1.70	.97	.46	.40	.37	.71	1.27	1.78
CR4	1.87	1.99	2.39	2.36	1.75	1.07	.66	.67	.65	1.01	1.43	1.84
BB1	1.82	1.95	2.70	2.70	1.70	.76	.32	.32	.29	.66	1.17	1.75
HB1	2.46	3.05	4.21	4.11	2.08	.53	.14	.16	.15	.55	1.18	2.39
HB2	2.46	3.04	4.21	4.11	2.09	.54	.14	.16	.15	.55	1.17	2.39
MB1	1.84	2.02	2.34	2.32	1.75	1.06	.64	.64	.64	1.02	1.39	1.82
MB2	2.14	2.40	2.87	2.86	2.00	1.15	.61	.56	.55	.99	1.50	2.08
MR1	2.20	2.36	2.65	2.64	2.10	1.48	1.00	.95	.94	1.31	1.69	2.13

Table 20. Base flow maximized under optimal pumping with proposed wells active, Scenario 3D, dry (90-percent low flow) conditions, upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	3.1	4.7	9.6	6.6	4.3	1.4	1.0	0.7	0.6	0.0	1.6	2.1
CR2	22.2	28.6	42.7	32.5	24.1	12.1	6.3	4.9	4.7	6.3	14.2	19.9
CR3	33.8	48.0	81.1	54.7	37.0	16.0	6.8	5.2	5.0	6.4	17.1	26.1
CR4	80.7	104.2	156.1	120.4	87.0	45.8	25.2	19.5	18.4	22.3	50.4	71.6
BB1	12.4	18.9	37.6	25.4	15.1	5.7	2.1	1.2	1.0	1.0	4.8	10.5
HB1	4.3	7.1	19.1	9.4	5.0	.6	.1	.1	.0	.2	.9	2.2
HB2	6.4	10.4	28.2	13.9	7.5	.9	.2	.2	.1	.3	1.4	3.2
MB1	9.7	12.1	18.8	13.9	10.9	5.0	2.7	2.1	1.9	2.6	5.2	7.1
MB2	15.6	20.6	34.5	24.3	17.9	7.0	3.2	2.3	2.1	3.2	6.9	10.4
MR1	21.3	25.5	35.4	28.2	23.2	12.6	7.6	6.4	6.2	7.4	11.8	16.2
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.28	0.41	0.85	0.59	0.38	0.13	0.09	0.06	0.06	0.00	0.14	0.19
CR2	.87	1.13	1.68	1.28	.95	.48	.25	.19	.18	.25	.56	.78
CR3	.52	.74	1.24	.84	.57	.25	.10	.08	.08	.10	.26	.40
CR4	.96	1.24	1.86	1.44	1.04	.55	.30	.23	.22	.27	.60	.85
BB1	.60	.91	1.81	1.22	.73	.28	.10	.06	.05	.05	.23	.50
HB1	.58	.95	2.56	1.27	.68	.07	.02	.02	.00	.03	.13	.29
HB2	.58	.94	2.55	1.26	.68	.08	.02	.01	.00	.02	.12	.29
MB1	.97	1.21	1.88	1.39	1.09	.50	.27	.21	.19	.26	.52	.70
MB2	.98	1.29	2.17	1.53	1.13	.44	.20	.15	.13	.20	.43	.65
MR1	1.33	1.59	2.21	1.76	1.45	.79	.47	.40	.39	.46	.74	1.01

New wells can improve the ability of towns to manage pumping schedules for the purpose of increasing streamflow. Towns whose wells all have similar response characteristics have less management flexibility. Towns that have only wells with short response times, like Milford, could improve management flexibility by placing new wells farther from streams and pumping them preferentially from January through September. Towns that have only wells with long response times, like Wrentham, could improve management flexibility by placing any new wells closer to streams and pumping them in the fall and winter. Towns that have only one or two wells in the basin, like Norfolk, have little management flexibility to adjust pumping schedules for the purpose of increasing streamflow in the basin.

Wastewater Management

Through strategic management of wastewater effluent, summertime base flow can be increased. Although discharging wastewater directly to streams increases streamflow immediately, recharging wastewater to an aquifer augments base flow over longer times. There are no centralized facilities in the study area that recharge wastewater to aquifers, although private septic systems provide widespread recharge of wastewater. Centralized wastewater recharge can flow through infiltration beds or injection wells (table 7), and both methods have been considered for the basin (Earth Tech, 2000; Bruce Bouck, Massachusetts Department of Environmental Protection, 2002, written commun.).

New Sewers

As towns in the basin expand their wastewater infrastructure, typically by installing new sewer lines, base-flow patterns are altered. Optimization models for proposed wastewater changes in the towns of Holliston and Bellingham under average streamflow conditions were used to analyze the altered base-flow patterns.

Holliston

The town of Holliston has considered building a sewer system (Earth Tech, 2000) to replace private septic systems, which are currently the only disposal method used in the town (table 6). The sewer system may include either:

A. transport of wastewater to the CRPCD for treatment and discharge, or

B. local treatment and recharge of wastewater at three smaller community sites.

The two options may have different effects on streamflow and are considered below.

Case A. CRPCD

For Case A (Scenario 4A), those parts of Holliston which currently receive public water and are on private septic systems had their wastewater routed to the CRPCD treatment plant for direct discharge (fig. 6). Ground-water withdrawal rates at all wells were set to average rates (table 5), and no other hypothetical or proposed projects were included. The resulting flow values (table 21) were calculated by the use of response coefficients and, when compared to existing flows (table 1), indicate that sewering reduces flow at BB1, HB1, HB2, and CR3 while increasing flow at CR4. The decrease in flow at BB1 reaches 0.05 $ft^3/s/mi^2$ in the summer months.

Case B. Local Recharge

For case B (Scenario 4B), septic-system disposal was rerouted to three wastewater recharge facilities HL-E1P, HL-E2P, and HL-WGP (fig. 6) (Scenario 4B). HL-E1P received 25 percent of monthly flow, HL-E2P received 60 percent, and HL-WGP received 15 percent, following designed flow rates (Earth Tech, 2000). Pumping rates were again set to average (1989–98) rates, and no other future or proposed projects were included. Comparison of the resulting base flow (table 22) with the existing base flow (table 1) indicates that flow differences are quite small, with BB1 having greater flow and CR3, CR4, and HB2 having lower monthly flow. The streamflow changes do not exceed ±0.02 $ft^3/s/mi^2$ at any constraint location.

Table 21. Base flow with Holliston wastewater routed to the Charles River Pollution Control District treatment plant, Scenario 4A, average climatic conditions, average withdrawals (1989–98) and return stresses (except for Holliston wastewater), upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile; --, indicates flow unchanged from average conditions (table 1)]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	--	--	--	--	--	--	--	--	--	--	--	--
CR2	--	--	--	--	--	--	--	--	--	--	--	--
CR3	118.1	133.8	173.4	174.2	109.9	61.4	27.3	23.6	22.0	44.8	81.9	114.9
CR4	158.4	168.2	201.2	199.0	147.2	89.5	51.8	52.5	50.9	85.0	121.1	155.9
BB1	37.7	40.3	55.7	55.6	34.5	14.5	5.0	5.3	4.9	13.6	24.4	36.6
HB1	18.0	22.4	31.1	30.3	15.4	3.8	.7	.9	.9	3.8	8.5	17.5
HB2	26.7	33.2	46.0	44.9	22.8	5.7	1.1	1.3	1.3	5.7	12.6	25.9
MB1	--	--	--	--	--	--	--	--	--	--	--	--
MB2	--	--	--	--	--	--	--	--	--	--	--	--
MR1	--	--	--	--	--	--	--	--	--	--	--	--
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	--	--	--	--	--	--	--	--	--	--	--	--
CR2	--	--	--	--	--	--	--	--	--	--	--	--
CR3	1.81	2.05	2.66	2.67	1.69	0.94	0.42	0.36	0.34	0.69	1.26	1.76
CR4	1.89	2.01	2.40	2.37	1.76	1.07	.62	.63	.61	1.01	1.44	1.86
BB1	1.81	1.94	2.68	2.67	1.66	.70	.24	.25	.23	.65	1.17	1.76
HB1	2.42	3.00	4.17	4.07	2.06	.51	.10	.12	.12	.51	1.14	2.35
HB2	2.42	3.01	4.17	4.07	2.07	.52	.10	.12	.12	.52	1.14	2.35
MB1	--	--	--	--	--	--	--	--	--	--	--	--
MB2	--	--	--	--	--	--	--	--	--	--	--	--
MR1	--	--	--	--	--	--	--	--	--	--	--	--

Table 22. Base flow with Holliston wastewater routed to local recharge basins, Scenario 4B, average climatic conditions, average withdrawals (1989–98) and return stresses (except for Holliston wastewater), upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile; --, indicates flow unchanged from average conditions (table 1)]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	--	--	--	--	--	--	--	--	--	--	--	--
CR2	--	--	--	--	--	--	--	--	--	--	--	--
CR3	118.4	134.0	173.6	174.5	110.1	61.7	27.6	23.9	22.2	45.0	82.1	115.2
CR4	157.1	166.9	199.9	197.7	146.0	87.9	50.2	51.2	49.2	83.6	119.8	154.6
BB1	38.8	41.3	56.7	56.6	35.6	15.5	6.1	6.4	6.0	14.7	25.5	37.7
HB1	18.3	22.6	31.3	30.5	15.6	4.1	1.0	1.1	1.1	4.1	8.7	17.7
HB2	27.0	33.4	46.2	45.1	23.0	5.9	1.4	1.6	1.6	6.0	12.8	26.2
MB1	--	--	--	--	--	--	--	--	--	--	--	--
MB2	--	--	--	--	--	--	--	--	--	--	--	--
MR1	--	--	--	--	--	--	--	--	--	--	--	--
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	--	--	--	--	--	--	--	--	--	--	--	--
CR2	--	--	--	--	--	--	--	--	--	--	--	--
CR3	1.82	2.06	2.66	2.68	1.69	0.95	0.42	0.37	0.34	0.69	1.26	1.77
CR4	1.87	1.99	2.38	2.36	1.74	1.05	.60	.61	.59	1.00	1.43	1.84
BB1	1.86	1.99	2.73	2.72	1.71	.75	.29	.31	.29	.71	1.23	1.81
HB1	2.45	3.03	4.20	4.10	2.09	.54	.13	.15	.15	.54	1.17	2.38
HB2	2.45	3.03	4.19	4.09	2.09	.54	.12	.14	.14	.54	1.16	2.37
MB1	--	--	--	--	--	--	--	--	--	--	--	--
MB2	--	--	--	--	--	--	--	--	--	--	--	--
MR1	--	--	--	--	--	--	--	--	--	--	--	--

Bellingham

Bellingham has considered building a wastewater treatment plant (WDMP) with lagoons for wastewater recharge (BL-WRP) adjacent to the Charles River (figs. 6 and 12). Areas in Bellingham that receive public water contribute wastewater either to the CRPCD treatment facility (29 percent) or to local septic-system discharge (71 percent) (table 6). To determine how the proposed facility could affect streamflow, it was assumed that all Bellingham wastewater would be recharged to the aquifer at the BL-WRP recharge facility (Scenario 4C).

Response coefficients were used to calculate flows, and the resulting flow changes at CR3 and CR4 are small, as can be seen by comparing flows in table 23 to flows under average conditions in table 1. The changes are small because Bellingham is only 29 percent sewered (table 6).

Siting of Recharge Basins

Timing of streamflow response to wastewater recharge is important in the design and management of recharge basins. As formulated in the ground-water model, timing of streamflow depletions depends particularly on downgradient distance of the stream from the recharge basin, aquifer transmissivity and storativity, hydraulic gradient, and width and conductance of the streambed, all of which are summarized in the response coefficients. Scenario 4D shows how the siting of a recharge basin affects the timing of streamflow depletions. A series of hypothetical recharge basins at various distances west of Miscoe Brook is considered for disposing wastewater from the town of Franklin (fig. 13). Response coefficients for the four hypothetical basins reflect their distances from Miscoe Brook (table 24).

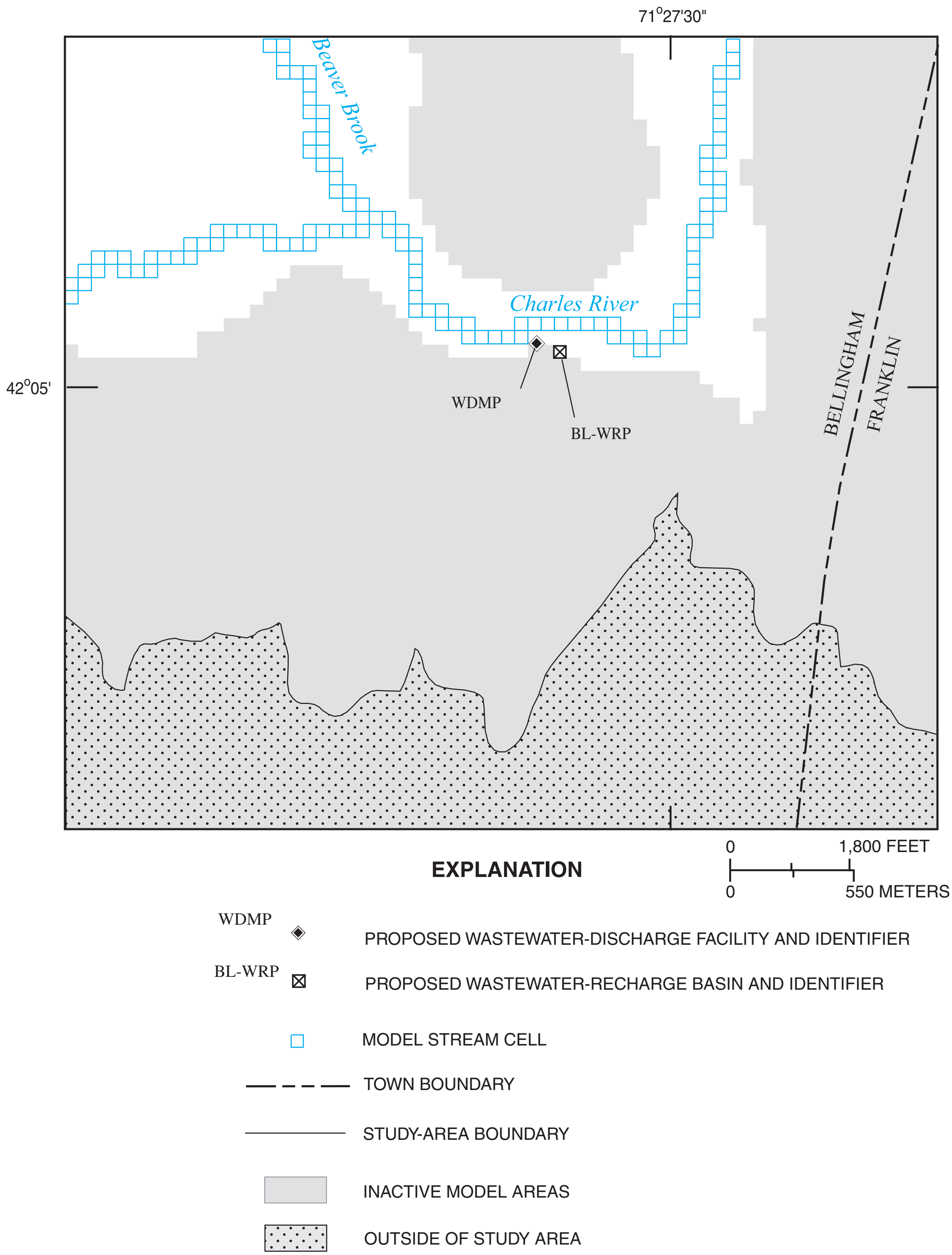


Figure 12. Proposed Bellingham wastewater facility location, upper Charles River basin, eastern Massachusetts. See figure 6 for view of entire study area.

Table 23. Base flow with Bellingham wastewater routed to BL-WRP recharge basin, Scenario 4C, average climatic conditions, average withdrawals (1989–98) and return stresses except for Bellingham wastewater, upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile; --, indicates flow unchanged from average conditions (table 1)]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
							Base flow (ft^3/s)					
CR1	--	--	--	--	--	--	--	--	--	--	--	--
CR2	--	--	--	--	--	--	--	--	--	--	--	--
CR3	118.8	134.5	174.1	175.0	110.4	62.1	27.9	24.1	22.6	45.4	82.5	115.5
CR4	157.6	167.4	200.4	198.1	146.4	88.5	50.7	51.5	49.7	84.0	120.1	155.0
BB1	--	--	--	--	--	--	--	--	--	--	--	--
HB1	--	--	--	--	--	--	--	--	--	--	--	--
HB2	--	--	--	--	--	--	--	--	--	--	--	--
MB1	--	--	--	--	--	--	--	--	--	--	--	--
MB2	--	--	--	--	--	--	--	--	--	--	--	--
MR1	--	--	--	--	--	--	--	--	--	--	--	--
							Base flow per drainage area ($ft^3/s/mi^2$)					
CR1	--	--	--	--	--	--	--	--	--	--	--	--
CR2	--	--	--	--	--	--	--	--	--	--	--	--
CR3	1.82	2.06	2.67	2.69	1.69	0.95	0.43	0.37	0.35	0.70	1.27	1.77
CR4	1.88	2.00	2.39	2.36	1.75	1.06	.60	.61	.59	1.00	1.43	1.85
BB1	--	--	--	--	--	--	--	--	--	--	--	--
HB1	--	--	--	--	--	--	--	--	--	--	--	--
HB2	--	--	--	--	--	--	--	--	--	--	--	--
MB1	--	--	--	--	--	--	--	--	--	--	--	--
MB2	--	--	--	--	--	--	--	--	--	--	--	--
MR1	--	--	--	--	--	--	--	--	--	--	--	--

To allow for greater management control, wastewater is allowed to discharge in the simulation either entirely to Miscoe Brook (WDMPB) or to any of the four basins (FR-W1P, FR-W2P, FR-W3P, FR-W4P), in any combination. Franklin's other wastewater disposals (R--FS and R--FW) are set to zero under the assumption that all households on municipal water will send effluent to the hypothetical facility. The best wastewater-disposal solution for increasing summertime base flow at MB1 is found as follows:

Minimize: Streamflow depletions at MB1 during July, August, and September.

Constraints:

1. Monthly pumping rates at wells are equal to monthly average (1989–98) rates.
2. Total discharge (to Miscoe Brook and to recharge basins) equals total Franklin pumping in each month.

Not all of the five possible disposal options are used in the optimal wastewater-disposal solution (table 25). Direct discharge to Miscoe Brook and the most distant recharge basins are used, whereas the two basins closest to the stream are not used. This result can be explained by the response coefficients (table 24). During July through September, the greatest streamflow results when all wastewater is discharged to the stream (WDMBP). For all other months except June, the greatest increase in summertime streamflow is obtained by using the most distant basin because it has the highest response coefficients for months 4–12. To build a recharge basin even farther from the stream is impractical because of impermeable bedrock outcrops. If a recharge basin were far from a stream, however, it would have response coefficients for every month close to -1/12 (-0.083). In table 24, the response coefficients of basin FR-W1P are closest to this value because FR-W1P is the most distant basin. A distant recharge basin with response coefficients of -0.083 would provide additional benefit to streamflow when recharging during the October–January period as compared to basin FR-W1P.

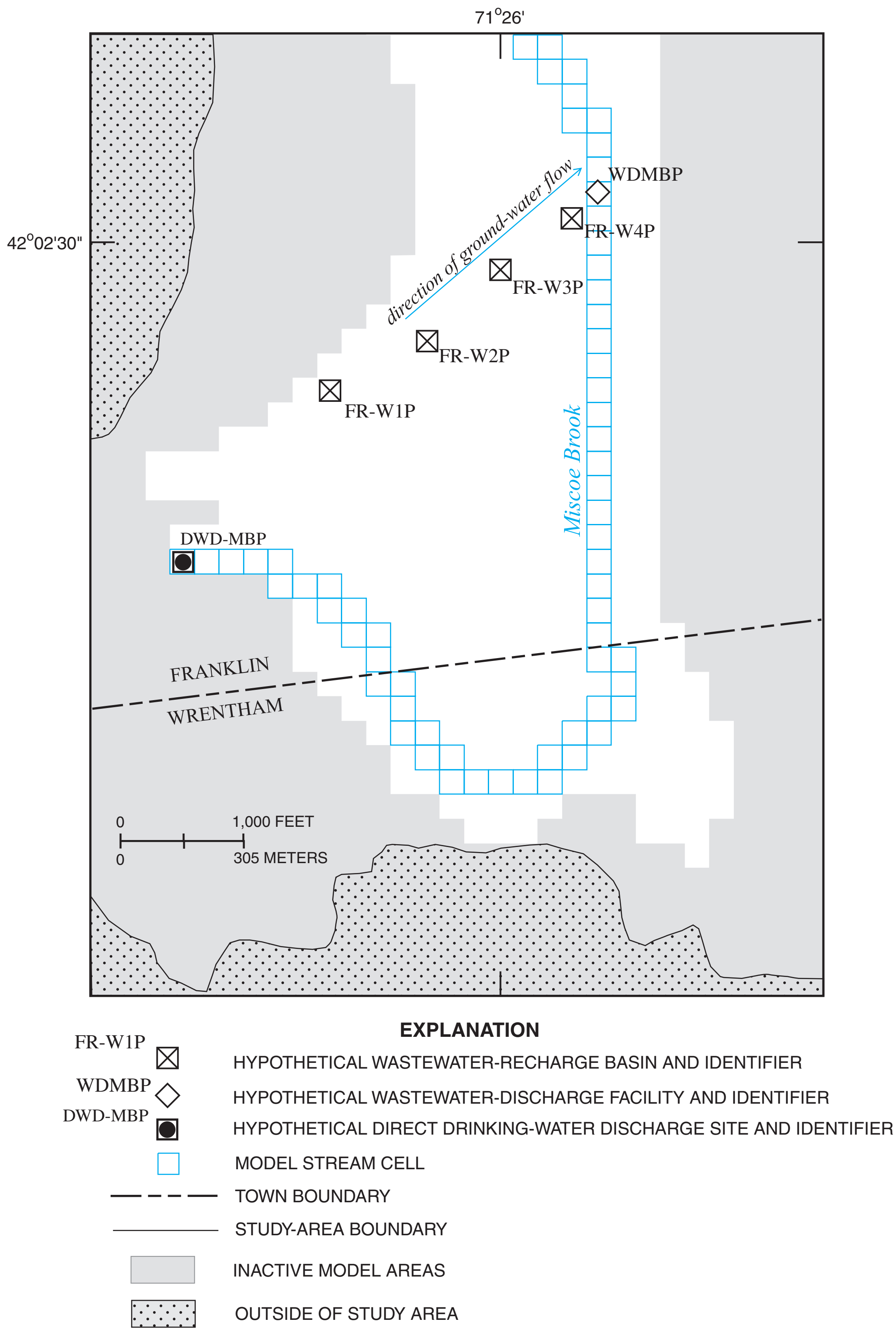


Figure 13. Hypothetical wastewater-recharge and discharge facility, Franklin, upper Charles River basin, eastern Massachusetts. See figure 6 for view of entire study area.

Table 24. Response coefficients for hypothetical recharge basins west of Miscoe Brook in Franklin, upper Charles River basin, eastern Massachusetts.

[**Response coefficients:** Negative values indicate water returns. Type: DP, discharge pipe; RB, recharge basin. ID, identifier; ft, foot; --, zero]

Recharge stress ID	Type	Downgradient distance to stream (ft)	Response coefficient by month												Total
			1	2	3	4	5	6	7	8	9	10	11	12	
WDMBP	DP	0	-1.000	--	--	--	--	--	--	--	--	--	--	--	-1.000
FR-W4P	RB	200	-.319	-0.163	-0.119	-0.097	-.0080	-0.064	-.0056	-0.044	-0.030	-0.014	-0.014	--	-1.000
FR-W3P	RB	894	-.214	-.164	-.131	-.110	-.095	-.075	-.068	-.054	-.043	-.020	-.014	-0.011	-1.000
FR-W2P	RB	1,720	-.115	-.148	-.137	-.123	-.108	-.095	-.082	-.068	-.052	-.032	-.023	-.017	-1.000
FR-W1P	RB	2,608	-.090	-.144	-.134	-.125	-.113	-.099	-.088	-.071	-.059	-.035	-.023	-.020	-1.000

Table 25. Optimal wastewater-flow schedule, Miscoe Brook hypothetical recharge-discharge facility, Scenario 4D, Franklin, upper Charles River basin, eastern Massachusetts.

[Facility location is shown in figure 12. Scenarios are described in table 12. ft^3/s, cubic foot per second; --, zero]

Stress	Distance to stream (foot)	Monthly flow rate (ft^3/s)											
		January	February	March	April	May	June	July	August	September	October	November	December
Wastewater recharge in basins													
FR-W4P	200	--	--	--	--	--	--	--	--	--	--	--	--
FR-W3P	894	--	--	--	--	--	--	--	--	--	--	--	--
FR-W2P	1,720	--	--	--	--	--	3.22	--	--	--	--	--	--
FR-W1P	2,608	3.53	3.49	3.52	3.66	4.16	--	--	--	--	3.79	3.47	3.45
Wastewater discharge to stream													
WDMBP	0	--	--	--	--	--	--	3.04	2.79	3.99	--	--	--

If a single recharge basin receives all wastewater and direct discharge of wastewater is not allowed, then siting of basins to maximize summertime base flow requires a different approach. The optimal solution in this case would be to site the recharge basin so that the travel time from the basin to the nearby stream matches the time between peak wastewater discharge and the low-flow period of July through September. March and April are the typical months of peak wastewater discharge from the CRPCD and MTF plants (fig. 5). Based on this scenario, a basin constantly receiving all available wastewater will cause the greatest increase in summertime streamflow if there is a 4 to 5-month ground-water travel time from the basin to the downgradient receiving stream reach.

Adding Wastewater Recharge to an Existing Wastewater Facility

To illustrate how adding recharge basins to an existing wastewater facility might improve management control over base flow, a hypothetical recharge basin (not shown on map) was modeled as available for recharging discharge from the CRPCD facility by means of optimization methods (Scenario 4E). The management objective is to increase base flow at constraint point CR4 just downstream of the CRPCD outflow. It is assumed that the hypothetical recharge basin is distant enough from streams to have response coefficients of -0.0833 in all 12 months at constraint point CR4.

Minimize: Streamflow depletions at constraint point CR4 during July, August, and September

Constraints:

1. All withdrawal and return stresses (except for CRPCD flow) are at monthly average (1989–98) rates.
2. Total monthly discharge at the CRPCD (direct discharge plus recharge to the basin) equals average (1989–98) monthly discharges.

The resulting optimal management solution is shown in table 26, and the resulting base-flow values are shown in table 27. As in the previous example, the optimal solution is to recharge wastewater every month of the year except those months in which it is desired to maximize base flow. In the optimal case, base flow at CR4 increases compared to average flow (table 1) by an average of 4.77 ft^3/s (0.06 $ft^3/s/mi^2$) during July through September, which is 79 percent of annual average CRPCD discharge. Base flow in the other months of the year decreases by an average of 1.56 ft^3/s (0.02 $ft^3/s/mi^2$) compared to average flows (table 1). The large increase in summertime flow is not achieved if wastewater is recharged in all 12 months of the year; this schedule gives an increase in flow of only 0.9 ft^3/s during July through September.

Injection Wells

Injection wells are formulated the same way as recharge basins in the numerical model because the aquifer is thin and horizontal discretization is limited to 200 ft by the model grid. Therefore, no separate analysis of injection wells was performed. Injection wells are expected to have the same effects on base flow as recharge basins with the same flow rates.

Moving Wastewater Effluent Upstream

Another management option that can increase streamflow is moving wastewater discharges upstream. Such projects increase streamflow in the reach between the old and new discharge locations by an amount equal to the wastewater-discharge rate. As an example, a hypothetical case was analyzed in which CRPCD wastewater effluent was moved 1,200 ft upstream on the Charles River to CRPCD-P (fig. 14), and the resulting streamflow changes at constraint point CR3 were calculated with the response coefficient approach (Scenario 5A). The relocation increases flow between the current and hypothetical discharge points by the rate of CRPCD discharge. Table 28 shows flow changes at CR3, the only constraint point lying between CRPCD and CRPCD-P; at all other locations, streamflow is unchanged.

Stormwater Recharge

Recharging stormwater runoff is another management practice with potential to increase summertime streamflow. Stormwater basins catch and recharge runoff, which otherwise would flow quickly to streams. A hypothetical case, based on a proposed project in Bellingham, serves as an example of how stormwater basins can affect base flow (fig. 15). The proposed project involves retrofitting an existing residential development with a stormwater catchment system.

Table 26. Pumping rates for wastewater discharge and recharge at Charles River Pollution Control District to optimize base flow, Scenario 4E, upper Charles River basin, eastern Massachusetts.

[The sum of discharge and recharge equals average (1989–98) CRPCD discharge rates. Scenarios are described in table 12. Variable locations are shown in figure 6. CRPCD, Charles River Pollution Control District. ft^3/s, cubic foot per second; --, zero]

Variable identifier	Description	Optimal pumping rate (ft^3/s)											
		January	February	March	April	May	June	July	August	September	October	November	December
CRPCD	Wastewater discharge to stream	--	--	--	--	--	--	5.23	5.17	5.12	--	--	--
CR-RBP	Wastewater recharge to basin	6.35	6.40	7.15	7.48	6.55	5.93	--	--	--	5.47	5.71	6.29

Table 27. Base-flow changes at outflow constraint point CR4 resulting from the addition of a hypothetical recharge basin to the Charles River Pollution Control District, Scenario 4E, upper Charles River basin, eastern Massachusetts.

[Pumping rates are given in table 26. Changes from average were calculated using average base flows shown in table 1. Scenarios are described in table 12. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Flow	Month											
	January	February	March	April	May	June	July	August	September	October	November	December
Base flow (ft^3/s)												
Flow	156.0	165.8	198.0	195.4	144.6	87.3	55.5	56.5	54.6	83.5	119.3	153.6
Change from average	-1.57	-1.62	-2.37	-2.71	-1.78	-1.15	+4.77	+4.77	+4.77	-.69	-.94	-1.52
Base flow per drainage area ($ft^3/s/mi^2$)												
Flow	1.86	1.98	2.36	2.33	1.72	1.04	.66	.67	.65	1.00	1.42	1.83
Change from average	-.02	-.02	-.03	-.03	-.02	-.01	+.06	+.06	+.06	-.01	-.01	-.02

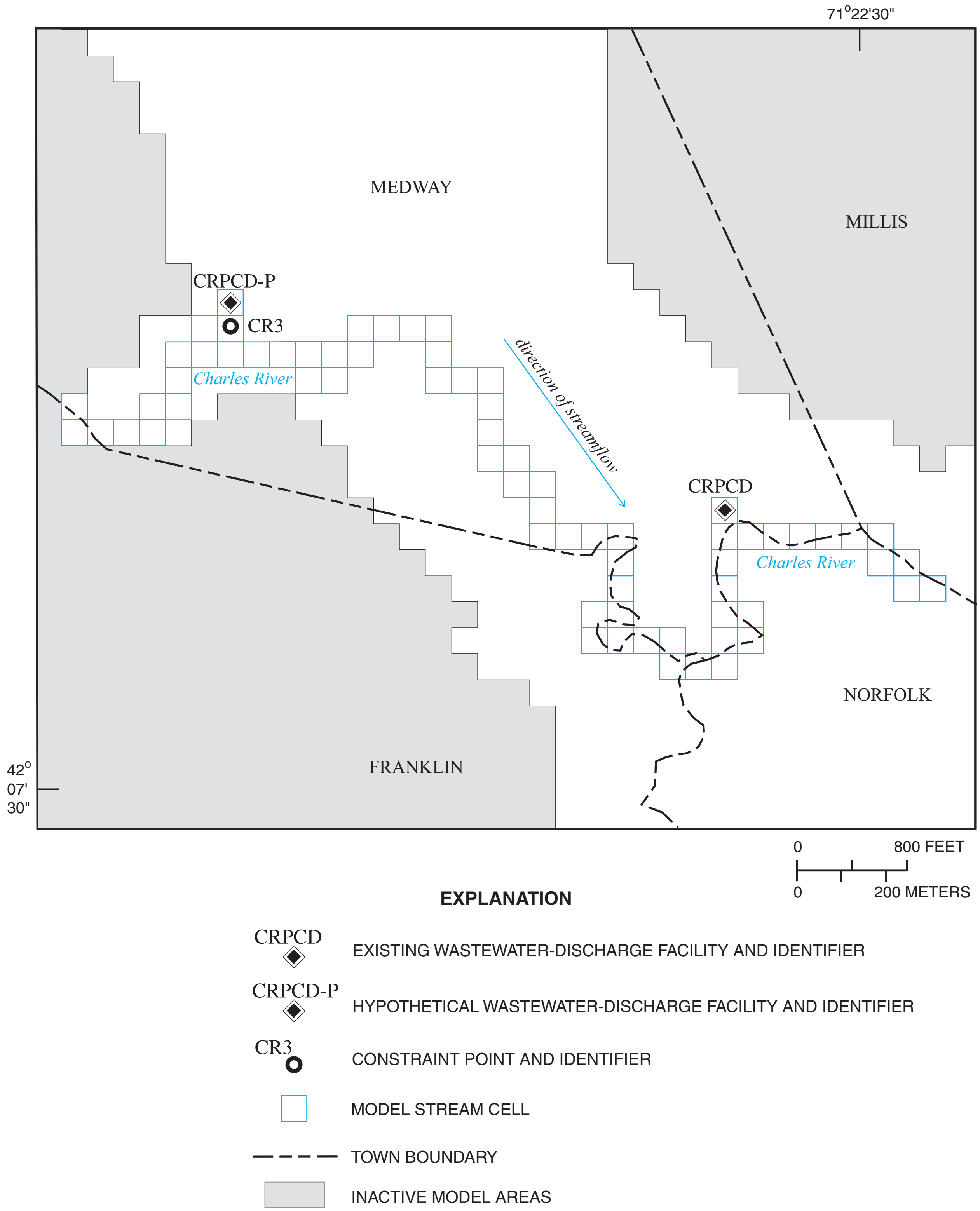


Figure 14. The existing Charles River Pollution Control District discharge location and a hypothetical upstream discharge location (CRPCD-P), upper Charles River basin, eastern Massachusetts. See figure 6 for view of entire study area.

Table 28. Base-flow changes at CR3 resulting from relocating Charles River Pollution Control District effluent upstream, Scenario 5A, upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Flow at constraint points other than CR3 is unchanged. Changes from average were calculated by using average base flows shown in table 1. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Flow	Month											
	January	February	March	April	May	June	July	August	September	October	November	December
	Base flow (ft^3/s)											
Flow	125.2	140.8	181.2	182.4	116.9	67.9	33.1	29.3	27.8	51.0	88.3	121.9
Change from average	6.3	6.4	7.1	7.5	6.6	5.9	5.2	5.2	5.1	5.5	5.7	6.3
	Base flow per drainage area ($ft^3/s/mi^2$)											
Flow	1.92	2.16	2.78	2.80	1.79	1.04	.51	.45	.43	.78	1.35	1.87
Change from average	+.10	+.10	+.11	+.11	+.10	+.09	+.08	+.08	+.08	+.08	0.09	+.10

It is assumed that the retention basin will capture runoff from a 500,000-ft^2 area that currently routes runoff directly to the Charles River. The proposed basin has the capacity to hold runoff from a rainstorm of 0.5 in. An analysis of storm records (storms separated by dry periods of more than 12 hours) from Logan Airport in Boston during the period 1970–95 indicates that a basin in this area with a capacity for 0.5 in. storms would retain 55 percent of annual runoff. Based on 55 percent of total rainfall (47 in/yr at the site) and the assumptions that all rainfall makes it to the retention basin and that the recharge rate is constant, the annual average recharge rate for the basin is 0.034 ft^3/s.

Response coefficients are used in calculating changes to base flow that result from the hypothetical recharge basin (Scenario 6A). No reduction in average streamflow is made to account for runoff no longer reaching the stream due to installation of the stormwater basin because in this study it is assumed that surface runoff is not a large component of streamflow during low-flow periods. In reality, stormwater basins would reduce streamflow during and just after intense rains because surface runoff from the catchment area would no longer reach streams. In Scenario 6A, because the recharge rate is the same each month, all constraint points downstream of the stormwater basin (CR2, CR3, CR4) have monthly increases in flow equal to the recharge rate of 0.034 ft^3/s.

During dry periods (90-percent low-flow conditions), the small increases in base flow from the proposed stormwater basin are comparatively more important. In the month of October under average climatic conditions, base flow at CR2 increases by 0.07 percent due to the proposed stormwater basin; under 90-percent low-flow conditions, base flow increases by 0.25 percent. A larger drainage area or multiple recharge basins would, however, produce correspondingly greater base-flow increases.

It is important to consider typical management practices when deciding where future stormwater recharge projects will be located. Stormwater basins are usually left to operate with little management other than vegetation control and debris cleaning. Although monthly rainfall is fairly consistent throughout the year, runoff is likely to be highest from December to March when frozen ground and lower evapotranspiration lead to higher runoff. To achieve the highest increases in base flow during the critical months of July through September, stormwater basins need to have high response coefficients in response months 5–10. The best stormwater basin location would therefore have a ground-water travel time from basin to stream of about 6 months.

Recharge of Flood Waters

During floods, water can be withdrawn from streams and infiltrated to aquifers to augment base flow in the summer months. Such systems require an intake pipe at the stream, pumps, controls, and either injection wells or infiltration basins to recharge water to the aquifer. The system would ideally have several infiltration basins at different distances from the stream and with different response coefficients. Water could be infiltrated at the basins in accordance with a schedule giving the greatest streamflow increases during July through September. The effects of such a project on streamflow are analogous to the effects from wastewater and stormwater basins. Flood waters can also be diverted to surface-water storage for later summertime release or water supply, as is done by Milford with Echo Lake.

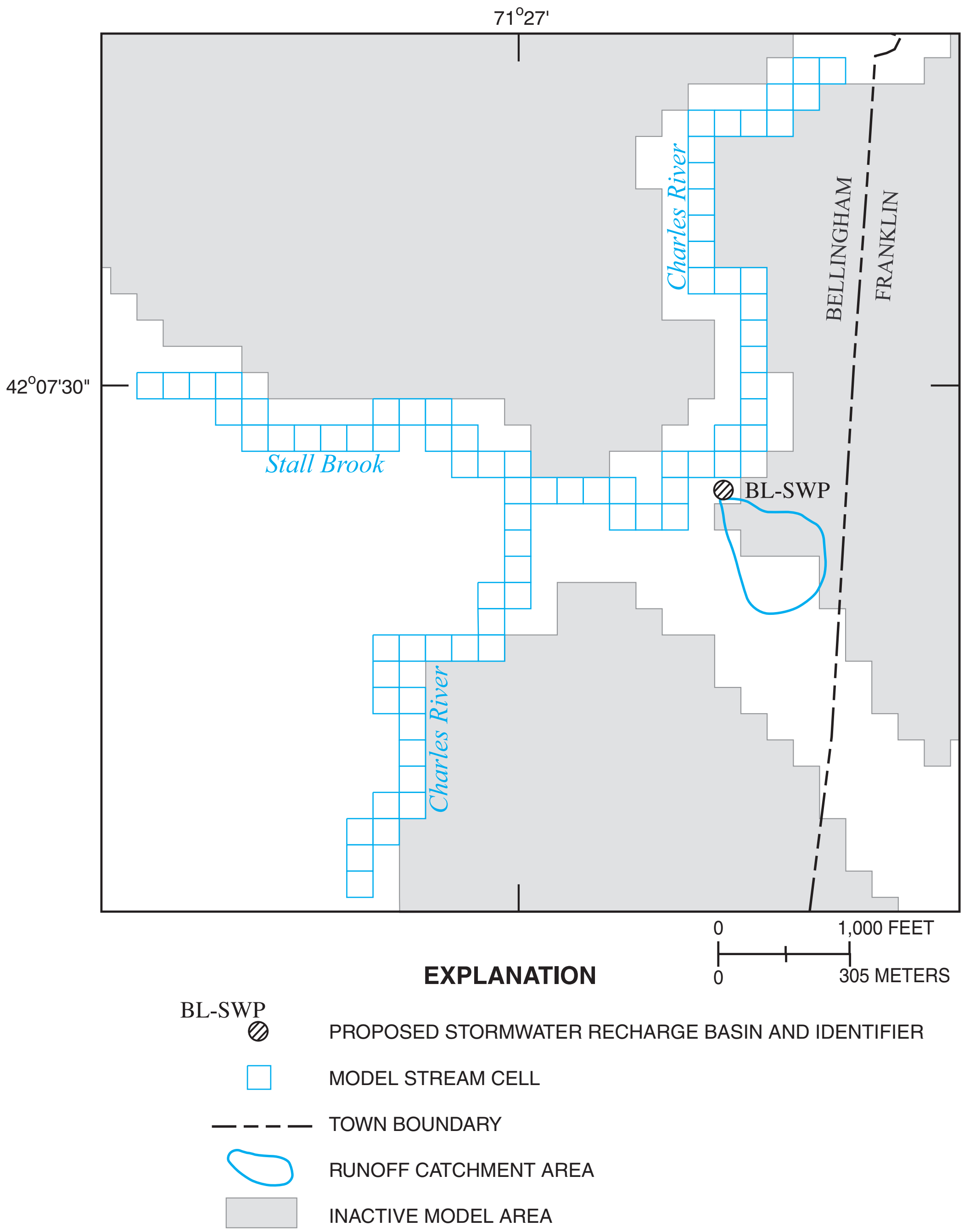


Figure 15. Proposed detention basin (BL-SWP), Bellingham, eastern Massachusetts. See figure 6 for entire study area.

Water Conservation

In response to water-supply shortages, many towns in the basin have instituted water-conservation measures. These conservation measures commonly impose restrictions on outdoor water use during the summer. An example is provided by Walpole, a town 5 mi east of the study area. From 1999 to 2002, Walpole restricted water use from mid-May to mid-September, limiting outdoor water use to every other day and banning automatic sprinkling devices. Over 3 years, average water demand from June through September dropped from 3.38 to 3.07 Mgal/d, a 9.2-percent decline which has helped the town avoid costly infrastructure investment (Steven Davis, Walpole Water Commission, written commun., 2002).

Hypothetical water conservation was simulated across the entire study area by reducing pumping rates to 90 percent of average during June through September (Scenario 7A). Other months of the year were assigned 100 percent of the average pumping rate. The 10-percent summer decrease was applied to all municipal and country club wells, but not to power plant wells (Bellingham NEA wells # 1, 2, 3, 4, and 5) or private wells. No proposed wells or other future projects were included and average streamflows were assumed. Wastewater disposal was not decreased, despite the decrease in pumping, because it was assumed that the reduction in pumping would only affect consumptive use and that wastewater production and hence wastewater disposal would remain constant. The resulting base flow was calculated with the response coefficient approach and is shown in table 29. Flow increases during July through September by an average of 0.17 ft^3/s at CR4 and 0.03 ft^3/s at BB1. Under 90-percent low-flow conditions, these base-flow increases are the same, but the increases are proportionally greater. In addition to increasing base flow, water conservation has the additional benefit to towns of reducing water demand on days of peak use during the summer when the largest stress is placed on water-supply capacity.

Table 29. Base flow under water conservation measures, Scenario 7A, municipal water use limited to 90 percent of average during June through September, average climatic conditions (1989–98), upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	9.6	10.4	13.3	13.5	9.0	4.1	1.8	1.9	1.8	4.0	6.2	9.4
CR2	42.5	45.1	55.3	54.0	39.3	23.6	12.6	12.8	12.7	21.9	32.4	41.2
CR3	118.9	134.5	174.1	175.0	110.4	62.2	28.2	24.6	23.1	45.7	82.7	115.7
CR4	157.7	167.5	200.4	198.2	146.4	88.9	51.3	52.3	50.6	84.5	120.5	155.3
BB1	38.6	41.1	56.5	56.4	35.4	15.5	6.1	6.3	5.9	14.6	25.3	37.5
HB1	18.3	22.6	31.3	30.5	15.6	4.1	1.0	1.1	1.2	4.1	8.7	17.7
HB2	27.0	33.5	46.3	45.2	23.1	6.1	1.5	1.7	1.7	6.0	12.9	26.2
MB1	19.1	20.9	24.0	23.8	17.9	10.5	6.0	6.1	6.1	10.2	14.1	18.9
MB2	34.6	38.7	46.0	45.7	32.0	18.1	9.3	8.8	8.6	15.7	23.8	33.6
MR1	35.3	38.0	42.5	42.4	33.6	23.3	15.1	14.4	14.2	21.2	27.8	34.7
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.85	0.93	1.18	1.20	0.80	0.36	0.16	0.17	0.16	0.35	0.55	0.84
CR2	1.67	1.77	2.17	2.13	1.55	.93	.49	.50	.50	.86	1.27	1.62
CR3	1.82	2.06	2.67	2.69	1.69	.95	.43	.38	.35	.70	1.27	1.78
CR4	1.88	2.00	2.39	2.36	1.75	1.06	.61	.62	.60	1.01	1.44	1.85
BB1	1.86	1.98	2.72	2.71	1.70	.75	.29	.30	.29	.70	1.22	1.80
HB1	2.45	3.03	4.20	4.10	2.09	.55	.14	.15	.16	.55	1.17	2.38
HB2	2.45	3.03	4.20	4.10	2.09	.55	.14	.15	.15	.55	1.17	2.38
MB1	1.91	2.09	2.39	2.37	1.79	1.05	.59	.61	.61	1.02	1.40	1.88
MB2	2.18	2.43	2.90	2.88	2.01	1.14	.59	.55	.54	.99	1.50	2.12
MR1	2.21	2.38	2.66	2.65	2.10	1.46	.95	.90	.89	1.32	1.74	2.17

Drinking-Water Transfers Between Towns

Transfer of drinking water between towns in the study area is quite limited and typically done only during emergency water shortages. Municipal drinking-water systems are managed on a town-by-town basis and state regulations are imposed on towns or on individual wells. Towns with excess water-supply capacity may be reluctant to sign long-term contracts for selling water to neighboring towns because they may anticipate future increases in their own water demand and difficulties obtaining permits for new withdrawals. Pipe connections are in place in many areas, however, to allow neighboring towns to share water supplies during emergency situations. To examine the possibility of regional cooperation in more detail, an optimization problem was formulated in which towns share water (Scenario 8A).

Minimize: Streamflow depletions at CR4 and BB1 during July, August, and September.

Constraints:

1. Base flow must be greater than or equal to average flow under no-pumping conditions or 0.41 ft^3/s, whichever is less.
2. Each town's monthly water supply is equal to average (1989–98) rates.
3. Towns share water (any town can draw from any municipal well in the basin).
4. Pumping from each well is at or below State-mandated maximum pumping rates (monthly Zone II and annual WMA limits are imposed).

No future projects are included other than drinking-water transfers. July, August, and September have equal weight in the objective function. Base flow is maximized only at constraint points CR4 and BB1; limiting the maximization to these two constraint points may cause flow at other constraint points to be lower.

The optimal pumping schedule increases base flow during July through September by an average of 3.4 ft^3/s (0.04 $ft^3/s/mi^2$) at CR4 and by an average of 0.9 ft^3/s (0.04 $ft^3/s/mi^2$) at BB1 (table 30), as compared to base flow under average pumping conditions (table 1). The base-flow increases are achieved by preferentially pumping wells that are close to the river during the winter and wells that are far from the river during the summer, as was done in Scenario 2B. The base-flow increases are greater than in Scenario 2B (31 percent greater at CR4 and 124 percent at BB1) because allowing any well to supply any town gives greater management flexibility and allows differences in response coefficients between wells to be exploited. Norfolk and Franklin, which have wells with long response times, pump more than average in the summer and less than average in the winter. Holliston, Medway, and Milford, which have wells with mostly short response times, pump less than average in the summer and more than average in the winter. Wrentham, whose wells have long response times, pumps more than average in every month; and Bellingham, which has wells with 8- to 10-month response times, pumps less than average in every month except May.

When 90-percent low-flow conditions are in effect (Scenario 8B), constraint 1 must be modified to allow flow to drop as low as 0.21 $ft^3/s/mi^2$ or to its 90-percent low-flow value under no-pumping conditions, whichever is less. The optimal pumping schedule increases base flow (table 31) during the July through September period by an average of 2.9 ft^3/s (0.03 $ft^3/s/mi^2$) at CR4 and by an average of 0.9 ft^3/s (0.04 $ft^3/s/mi^2$) at BB1 as compared to base flow under average pumping and 90-percent low-flow conditions (table 2).

Drinking-Water Discharge to Streams

Another management technique with potential to increase summertime streamflow is pumping drinking water directly to streams. This action is analogous to releasing water from a surface-water reservoir to increase streamflow, but with the aquifer acting as the reservoir. In scenario 9A, drinking water can be discharged to Hopping Brook (DWD-HBP), Mine Brook (DWD-MBP), and the Charles River upstream of Milford (DWD-UCP) as shown in figure 6. All stream reaches downstream of a discharge point have an increase in flow in the same month as the discharges. Further assumptions are made that water from any municipal well can be piped to any drinking-water discharge point without passing through septic or wastewater systems.

Optimization methods are used to determine pumping rates that most improve streamflow in the summer months. It is assumed that towns receive average drinking-water supplies and any excess pumping goes to discharge at DWD-UCP, DWD-HBP, or DWD-MBP (fig. 6). It is required that all discharge points have equal discharge rates.

Minimize: Streamflow depletions at CR1, MB1, and HB1 from July through September.

Constraints:

1. Base flow must be greater than or equal to average flow under no-pumping conditions or 0.41 ft^3/s, whichever is less.
2. Each town's monthly water supply is equal to average (1989–98) rates.
3. Towns share water (any town can draw from any municipal well in the basin).
4. Pumping from each well is at or below State-mandated maximum pumping rates (monthly Zone II and annual WMA limits are imposed).
5. In each month, flows at DWD-UCP, DWD-HBP, and DWD-MBP are equal.

Table 30. Base flow at constraint points under basin-wide water-supply sharing, Scenario 8A, average climatic conditions, each town receives its average water supplies (1989–98), upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	8.5	9.5	12.4	12.8	9.3	4.8	2.8	2.9	2.8	4.6	5.6	8.0
CR2	42.4	45.5	55.7	54.2	39.9	24.7	13.9	14.6	14.2	22.9	32.7	41.0
CR3	118.7	134.8	174.5	175.3	111.1	63.1	29.9	26.8	24.5	46.3	83.1	115.7
CR4	156.9	167.0	199.9	197.9	146.8	89.8	54.2	55.2	53.0	84.8	120.0	154.6
BB1	38.0	40.6	56.0	55.9	35.5	16.2	7.1	7.1	6.5	14.4	24.8	36.9
HB1	18.4	22.7	31.4	30.6	15.8	4.3	1.2	1.3	1.1	4.2	8.8	17.8
HB2	27.1	33.6	46.4	45.3	23.2	6.2	1.6	1.9	1.7	6.1	13.0	26.3
MB1	18.8	20.5	23.6	23.5	17.8	10.0	6.1	6.4	6.1	9.7	13.9	18.8
MB2	34.4	38.4	45.9	45.7	32.0	17.7	9.6	9.0	8.5	15.2	23.8	33.7
MR1	35.8	37.7	42.0	42.2	33.2	23.1	14.6	13.3	13.6	20.8	27.4	34.6
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.75	0.84	1.10	1.13	0.83	0.43	0.25	0.26	0.25	0.41	0.50	0.71
CR2	1.67	1.79	2.19	2.13	1.57	.97	.55	.58	.56	.90	1.29	1.61
CR3	1.82	2.07	2.68	2.69	1.71	.97	.46	.41	.38	.71	1.28	1.78
CR4	1.87	1.99	2.38	2.36	1.75	1.07	.65	.66	.63	1.01	1.43	1.84
BB1	1.83	1.95	2.69	2.69	1.71	.78	.34	.34	.31	.69	1.19	1.78
HB1	2.46	3.05	4.21	4.11	2.11	.57	.15	.18	.15	.56	1.18	2.39
HB2	2.46	3.04	4.21	4.11	2.11	.57	.15	.17	.15	.55	1.17	2.39
MB1	1.87	2.05	2.36	2.34	1.77	1.00	.61	.63	.61	.97	1.39	1.88
MB2	2.16	2.42	2.89	2.87	2.01	1.11	.60	.56	.53	.96	1.50	2.12
MR1	2.24	2.36	2.63	2.64	2.07	1.45	.91	.83	.85	1.30	1.71	2.17

Base flow increases by an average of 194 percent during July through September at CR1, HB1, and MB1 under the optimal pumping schedule. Clean water is discharged to each of the three stream locations in July, August, and September at respective rates of 3.8, 4.2, and 4.6 ft^3/s. These flow rates are the same at each discharge location as required by constraint 5 and no water is discharged in other months of the year. The resulting base flows at the 10 constraint points are given in table 32.

Direct discharge of drinking water would be of less benefit to streams if done on a town-by-town basis. If towns were not sharing drinking water in Scenario 9A, then only those towns with excess supply capacity during summer months would have water available to discharge to streams.

Other Management Strategies

Other strategies for addressing water shortages include leaky pipe detection and repair, land-use change, increased use of surface-water reservoirs, and importing water supplies from outside the basin. Loss of water from distribution pipes can be large. For example, investigations by the Massachusetts Water Resources Authority (MWRA), the water suppliers for Boston and many nearby towns, have detected leaks in their water-distribution system totaling 30 Mgal/d (Yeo and Estes-Smargiassi, 2000). Repairs to the pipes make more water available for use but also reduce the amount of leaking water that recharges ground water. Land-use changes, such as reduction of impermeable surface area, removal of ditching, and revegetation, can increase aquifer recharge by converting runoff to infiltration. New surface reservoirs, which help to alleviate the naturally low storage capacity of aquifers in the basin, can be managed either to increase municipal water supplies or to augment summertime streamflow. Connecting to outside water-supply systems, most notably the MWRA system, is an option that other suburban Boston towns have chosen. Water imported to the basin increases municipal water supplies and the resulting wastewater, if discharged within the basin, increases summertime streamflow. These other strategies were not analyzed in this study because a lack of supporting data would require major assumptions to be made and would increase the uncertainty of the results.

The results of this study indicate that several water-resources management options can increase municipal water supplies and summertime streamflows. In table 33, benefits to water supplies and summertime streamflows are summarized based on the results of this study.

Table 31. Base flow at constraint points under dry conditions (90-percent low flow) and basin-wide water-supply sharing, Scenario 8B, each town receives its average water supplies (1989–98), upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	3.5	4.8	9.4	6.9	5.1	2.4	1.5	1.4	1.4	1.7	2.4	2.7
CR2	22.2	28.9	42.7	32.8	24.1	12.2	6.1	4.9	4.0	6.8	14.3	19.9
CR3	34.4	48.9	81.6	55.4	37.6	16.0	7.2	6.1	5.6	7.7	17.5	26.6
CR4	81.3	104.7	156.1	120.9	87.5	45.9	23.4	17.8	16.4	22.3	50.0	72.1
BB1	12.5	18.7	37.3	25.1	15.2	6.0	2.5	1.5	1.4	2.3	5.7	11.0
HB1	4.3	7.1	19.1	9.4	5.3	.9	.1	.1	.0	.2	.9	2.2
HB2	6.4	10.4	28.2	13.9	7.7	1.2	.2	.1	.0	.3	1.3	3.2
MB1	10.2	12.6	19.2	14.3	11.2	5.0	3.0	2.8	2.7	3.0	5.3	7.5
MB2	16.2	21.2	35.0	24.7	18.4	7.3	3.8	3.3	3.3	3.9	7.2	11.0
MR1	22.0	25.0	34.8	27.9	22.5	11.4	5.6	4.1	3.8	6.2	11.6	16.1
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.31	0.43	0.83	0.62	0.45	0.21	0.13	0.12	0.12	0.15	0.21	0.24
CR2	.87	1.14	1.68	1.29	.95	.48	.24	.19	.16	.27	.56	.78
CR3	.53	.75	1.25	.85	.58	.25	.11	.09	.09	.12	.27	.41
CR4	.97	1.25	1.86	1.44	1.04	.55	.28	.21	.20	.27	.60	.86
BB1	.60	.90	1.79	1.21	.73	.29	.12	.07	.07	.11	.27	.53
HB1	.58	.95	2.56	1.27	.70	.12	.02	.01	.00	.03	.13	.29
HB2	.58	.94	2.55	1.26	.70	.11	.01	.01	.00	.02	.12	.29
MB1	1.02	1.25	1.91	1.42	1.11	.50	.30	.28	.27	.30	.53	.75
MB2	1.02	1.33	2.20	1.56	1.16	.46	.24	.21	.21	.25	.45	.70
MR1	1.38	1.56	2.18	1.74	1.41	.71	.35	.26	.24	.39	.72	1.01

Limitations of this Analysis

Although the analyses in this study focus on monthly average flow rates, there can be substantial streamflow variation within a month. A comparison of daily flow variability to monthly mean flow at the Medway streamflow-gaging station (01103280, fig. 3) in the eastern part of the study area is shown in figure 16. A statistical analysis was done on daily streamflow data collected from November 1997 to December 2000 at the Medway station. Daily mean flow values are below the monthly mean flow value more than half the month because of short-duration high stormwater flows (fig. 17). During July and September at the same station, daily flow values are less than monthly mean flow more than 75 percent of the time.

Awareness of how flow statistics vary with length of averaging period is important in setting minimum flow standards for water-resource management goals. Because streamflow data are characterized by a lack of negative values and infrequent extreme high values, longer averaging periods will generally result in higher mean values. Therefore, the mean monthly flow values used in this study will be higher than mean daily flow values more than half the time. If a regulatory program has the goal of protecting freshwater aquatic ecology, then the duration of low flow that is critical to maintaining ecology is important. For instance, if streamflow below 0.05 $ft^3/s/mi^2$ is fatal to fish when it lasts for more than a day, then a management process that uses at least daily average streamflow values to set management actions will be most effective at protecting fish. The results of this study, quantified as monthly average streamflow values, can be at least partially applied to averaging periods of less than a month by comparing monthly average flow-duration curves to shorter term (such as daily) flow-duration curves.

The results presented here are affected by errors in the response coefficients. These errors can be important in the analysis of low flow, particularly in small streams, because errors are a higher percentage of total streamflow than for average and high flows. Other errors due to model

Table 32. Base flow under direct drinking-water discharge to streams, Scenario 9A, each town receives its average water supplies (1989–98), average climatic conditions, upper Charles River basin, eastern Massachusetts.

[Scenarios are described in table 12. Grey shading indicates July–September months with flow less than 0.5 $ft^3/s/mi^2$. ft^3/s, cubic foot per second; $ft^3/s/mi^2$, cubic foot per second per square mile]

Constraint point	January	February	March	April	May	June	July	August	September	October	November	December
						Base flow (ft^3/s)						
CR1	8.1	9.4	12.5	14.0	9.8	5.0	4.7	4.4	4.3	1.4	4.0	7.5
CR2	40.8	44.0	54.2	54.4	39.8	23.9	14.7	14.5	14.1	19.5	30.0	39.1
CR3	117.0	133.4	173.7	176.2	112.0	63.5	37.6	33.6	32.1	42.3	79.8	113.2
CR4	155.1	166.2	200.0	199.1	147.6	89.2	58.9	59.1	56.9	80.3	117.4	152.8
BB1	39.2	41.3	56.7	56.4	35.4	15.6	5.4	5.2	4.4	14.0	25.3	37.9
HB1	18.3	22.7	31.4	30.6	15.7	4.3	4.8	5.2	5.6	4.0	8.7	17.7
HB2	27.1	33.5	46.4	45.3	23.2	6.2	5.3	5.8	6.1	6.0	12.9	26.2
MB1	19.3	21.2	24.9	24.9	19.3	12.0	9.8	9.7	9.6	9.6	13.9	18.9
MB2	34.4	38.6	46.5	46.5	32.9	19.3	12.8	12.0	11.7	14.7	23.3	33.2
MR1	34.8	37.4	42.2	41.4	32.2	22.0	13.5	12.6	12.2	20.0	27.0	33.8
						Base flow per drainage area ($ft^3/s/mi^2$)						
CR1	0.72	0.84	1.11	1.25	0.87	0.44	0.42	0.39	0.38	0.13	0.36	0.67
CR2	1.61	1.73	2.13	2.14	1.57	.94	.58	.57	.55	.77	1.18	1.54
CR3	1.80	2.05	2.67	2.70	1.72	.97	.58	.52	.49	.65	1.23	1.74
CR4	1.85	1.98	2.39	2.38	1.76	1.06	.70	.71	.68	.96	1.40	1.82
BB1	1.88	1.99	2.73	2.71	1.70	.75	.26	.25	.21	.67	1.22	1.82
HB1	2.46	3.04	4.21	4.11	2.11	.57	.65	.70	.75	.54	1.17	2.38
HB2	2.45	3.04	4.20	4.10	2.11	.57	.48	.52	.55	.54	1.17	2.38
MB1	1.93	2.12	2.48	2.49	1.92	1.20	.98	.97	.96	.95	1.39	1.88
MB2	2.17	2.43	2.93	2.93	2.07	1.21	.81	.76	.74	.93	1.47	2.09
MR1	2.18	2.34	2.64	2.59	2.02	1.37	.84	.79	.76	1.25	1.69	2.11

insufficiencies may be present in the results. These error sources include inadequate representation of surface topography and surface-water features. For example, if a stream that is close to a well is absent in the numerical flow model, the actual stream response times may be much faster than indicated by the model and derived response coefficients will contain inherent inaccuracies.

This analysis only considers flow at the 10 constraint points shown in figure 3. In other stream reaches, particularly upstream of constraint points, normalized flow (cubic foot per second per square mile) may be lower. Water quality was not considered in this study. Many of the scenarios considered involve management of variably treated wastewater, which may affect surface-water quality. Changing the pumping schedules of wells may also be difficult due to water-quality deterioration associated with high pumping rates.

In some cases, gaps in the data used to describe pumping wells may have affected results. For example, mechanical limits to pumping rates for some wells were not included in the analysis because reliable data about such limits were not available.

Table 33. Benefits of potential water-management actions, upper Charles River basin, eastern Massachusetts.

Management action	Benefits: Increased water supply	Benefits: Increased base flow
New wells	✓	[1]✓
Stormwater recharge		✓
Treated wastewater recharge		✓
Inter-town water transfers (intra-basin)	✓	[1]✓
New surface-water reservoirs	✓	[1]✓
Leaky pipe repair	✓	
Pumping schedule management	✓	✓
Direct discharge of drinking water to streams		✓
Import water from outside basin	✓	✓
Water-use restrictions	✓	✓
Land-use management		✓
Flood skimming	[2]✓	✓
Moving wastewater effluent upstream		✓

[1]If actively managed to increase base flow.

[2]Only if combined with a surface-water reservoir.

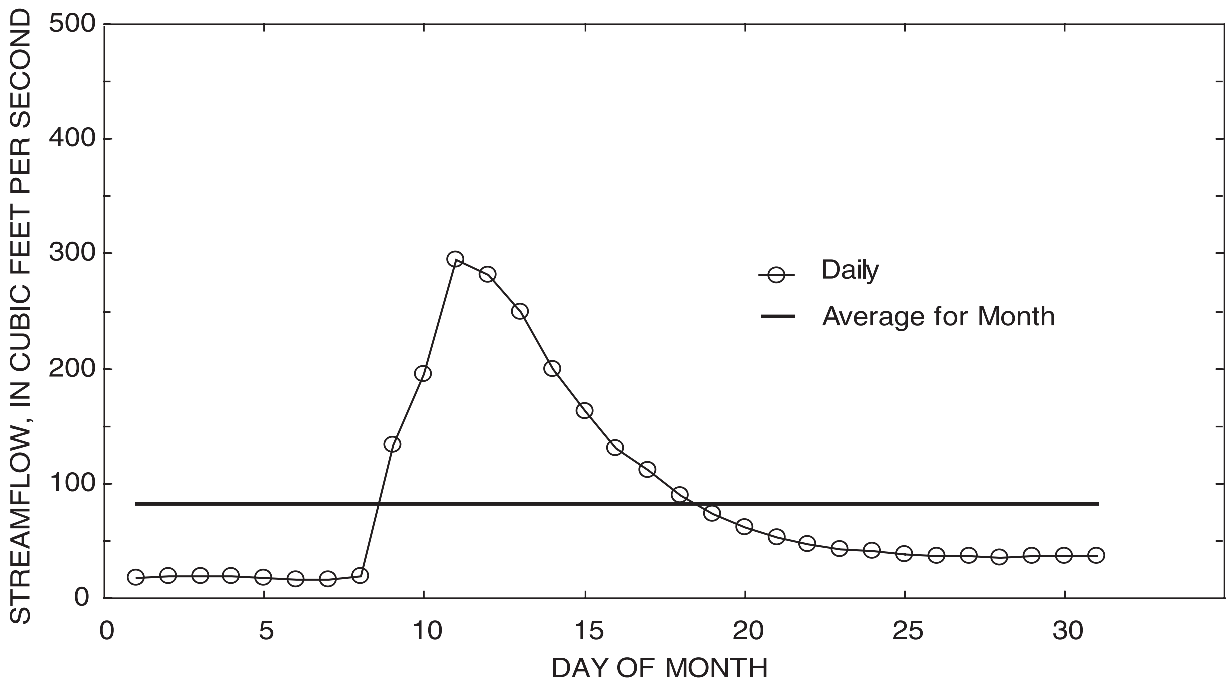


Figure 16. Daily and average streamflow for October 1998 at the Medway streamflow-gaging station (01103280), eastern Massachusetts.

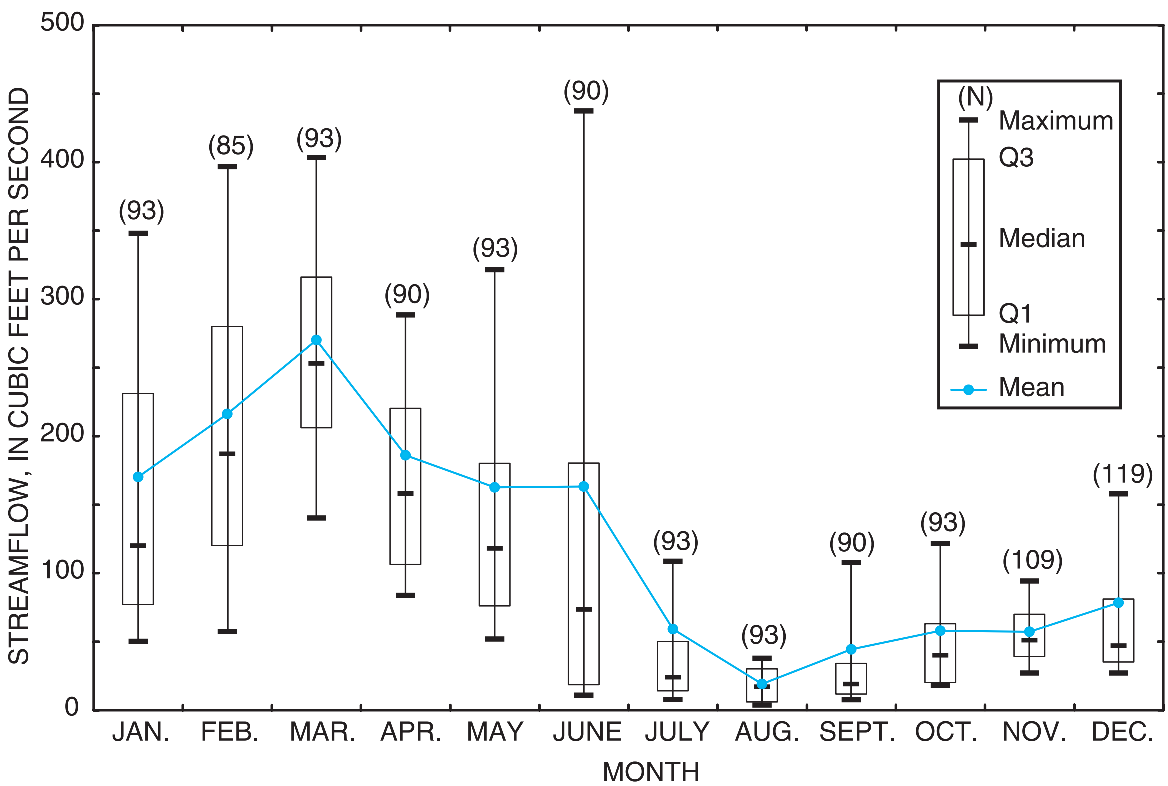


Figure 17. Daily streamflow variability by month at the Medway streamflow-gaging station (01103280), eastern Massachusetts. The period of record is November 1997 through December 2000. N is the number of daily streamflow values used to calculate the statistics. Q1 is the 25th percentile, and Q3 is the 75th percentile.

Summary and Conclusions

Water shortages in the upper Charles River basin are evident from strained municipal water supplies and average stream base flow that falls below 0.50 $ft^3/s/mi^2$, the ABF low-flow value recommended to "sustain and perpetuate indigenous aquatic fauna" (U.S. Fish and Wildlife Service, 1981). Under average climatic and pumping conditions, 6 of 10 constraint points analyzed (BB1, HB1, HB2, CR1, CR2, CR3) have mean monthly flows below 0.50 $ft^3/s/mi^2$ in summer, with the lowest monthly flow at the 10 constraint points averaging 0.41 $ft^3/s/mi^2$. Under dry climatic conditions (defined as periods of 90-percent flow duration) and average pumping, base flow drops below 0.50 $ft^3/s/mi^2$ at every constraint point, average low monthly flow at the 10 constraint points is 0.10 $ft^3/s/mi^2$, and Hopping Brook (at constraint points HB1 and HB2) essentially goes dry. Dry climatic conditions magnify the effects of water-management actions because human-controlled flow then constitutes a much larger percentage of streamflow.

Although human water use exacerbates summertime low flow, study results indicate that human water use is not the only, or even the primary, cause of low flow in the basin. When no-pumping conditions are simulated, flow at 5 of the 10 constraint points (BB1, HB1, HB2, CR1, CR3) still falls below 0.50 $ft^3/s/mi^2$ in the summertime. Under no-pumping conditions, the lowest monthly streamflow at the 10 constraint points increases slightly from an average of 0.41 $ft^3/s/mi^2$ to an average of 0.46 $ft^3/s/mi^2$, due to the cessation of human induced evaporative losses.

New ground-water wells can certainly increase the water supply available to towns. If handled carefully, new wells can also increase the ability of towns to manage withdrawals so that they cause less streamflow depletion during the summer. To increase streamflow, however, pump locations must be chosen, and pumping schedules must be actively managed with that explicit goal.

Because the two wastewater facilities contribute such large flows to the Charles River, management actions at these facilities have important consequences for water supply and streamflow in the basin. The addition of a recharge basin at the CRPCD facility, in combination with optimal effluent management, can increase flow in the Charles River at its outlet point CR4 by 4.77 ft^3/s (0.06 $ft^3/s/mi^2$) during the July-September period; this increase is more than twice the flow increase of 2.2 ft^3/s resulting from shutting off all pumps in the basin.

Sewering projects, which replace distributed septic systems with sewers and centralized wastewater treatment, have the potential to decrease summertime base flow. This is evident from the results of Scenario 4A, in which Holliston's wastewater is routed to the CRPCD. The use of localized recharge facilities in Holliston negates this streamflow decrease.

Aquifer recharge from construction of a storm-water basin could increase base flow during the summer (Scenario 6A). If the size of the basin and its collecting watershed in Scenario 6A were increased, the summertime base-flow would also increase. However, other limitations on the size of the basin, such as availability of property in an existing neighborhood without a stormwater recharge system, were not considered in this hydrologic analysis.

The use of response coefficients to describe how streamflow responds to water withdrawals and wastewater disposals can provide useful information to water-resources managers about the timing of streamflow changes relative to the timing of ground-water pumping or recharge. The results of this study show that considerable enhancement of summertime streamflow can be realized from towns strategically managing their pumping well schedules. The greatest benefit can be realized by towns that have wells with a variety of streamflow response times. On the basis of this study, wells that have short response times will cause less summertime flow depletion if they are pumped preferentially from October to January; wells that have long response times, because they are far from the river, can be pumped preferentially from February to September.

If recharge basins are built for the purpose of increasing ground-water storage and base flow, the optimal location of a basin depends on how it is to be managed. If wastewater or stormwater continually flow at an approximately constant rate to the basin, then, on the basis of this study, a basin sited so that ground-water travel time from basin to stream equals the time between peak wastewater discharge and the low streamflow period will give the greatest increase to summertime flow. In the Charles River basin, wastewater discharge and stormwater flow often reach their peaks during March and April, 5 to 6 months before the months of lowest streamflow. Based on this study, recharge basins receiving continuous flow will therefore achieve the greatest increase in August and September streamflow by being located so that ground-water travel-time from the basin to the downgradient stream is 5 to 6 months. However, if a recharge basin is paired with a pipe that discharges directly to a stream, the greatest benefit to summertime streamflow can be achieved by locating the basin as far as possible from the stream, discharging water directly to the stream during periods of low flow, and discharging to the recharge basin the rest of the year. Greater streamflow increases result from a basin sited far from a stream because the response coefficients are more likely to be uniform across all months; this uniformity of response yields the greatest ground-water discharge to the stream in the summertime. Even greater benefit to summertime streamflow can be realized with multiple recharge basins managed in conjunction with a pipe discharging directly to the stream. Optimization methods are useful for choosing optimal basin locations and discharge schedules for such facilities.

Response coefficients can also help in the siting of locations for new ground-water wells. If a town plans to manage its well system to minimize summertime streamflow depletions, then the results of this study indicate that new well locations should be chosen to have response times unlike those of existing wells. A collection of wells with a variety of response times provides the greatest management flexibility for reducing pumping effects on streams. For example, if it is known that a new well will only be used in the summer to meet peak demands, then the well can be sited as far as possible from streams to delay the resulting streamflow depletions.

Water-conservation measures have been implemented by some towns in the basin to reduce peak water use in the summer. The results of this study indicate that water conservation also has the desired effect of increasing summertime base flow in the upper Charles River and its tributaries. The benefits of water conservation for water supply and streamflow will be greater than in Scenario 7A if more than a 10-percent reduction in water use is achieved.

If basin-wide management of water resources were instituted in the upper Charles River basin, both towns and streams could benefit. The results of Scenarios 8A and 8B indicate that if drinking-water transfers between towns were practiced and coordinated, summertime flow could be increased at the basin exit points by 2.9 ft^3/s at constraint point CR4 and by 0.9 ft^3/s at constraint point BB1, which corresponds to 18 percent and 95 percent of flow, respectively, under dry climatic conditions. If water sharing were practiced, an excess water-supply capacity of 13.4 ft^3/s would be available to any town because Zone II limits allow 24.1 ft^3/s of pumping as compared to 10.7 ft^3/s for current average municipal water use. This study focused on hydrologic analyses and did not consider factors such as the economic effects of current water-sharing practices on the availability and cost of public-water supply in the basin.

Pumping clean water directly to streams has the potential to raise summertime streamflow and improve the quality of the water. The water can be pumped either from ground-water storage (aquifers) or a surface-water reservoir. The benefit to streams is much greater if towns share water supplies, so that the water can be delivered to the most stressed streams, even if they are in a town that does not have excess drinking-water supply in the summer.

Low streamflow and public water-supply shortages during the summer in the upper Charles River basin have caused concern about water supplies in the basin. In response to this concern, the USGS and the Massachusetts EOEA cooperated to prepare this study of potential alternatives for water-resources managers to address water shortages in the basin. A technical advisory committee composed of town water officials, USGS staff, state environmental regulators, and environmental advocates helped define the questions to be addressed.

The results of this study indicate that response coefficients and simulation-optimization methods are well suited for analyzing stressed stream-aquifer systems and are useful for analyzing water-resource management strategies to increase water supply and streamflow. The following strategies have potential benefit for addressing water shortages in the basin:

- Water conservation measures
- Strategic scheduling of ground-water pumping
- Direct discharge of water to streams
- Towns sharing drinking water
- Use of wastewater recharge basins and injection wells
- Stormwater basins
- New wells

Each of these strategies can, when properly managed, both increase summertime flow in streams and increase summertime water supply for human use. The results of this study will be useful to managers, planners, and regulators of water resources in the upper Charles River basin and in similar hydrogeologic settings where human water use affects shallow stream-aquifer systems.

References Cited

Ahlfeld, D.P., and Mulligan, A.E., 2000, Optimal management of flow in ground-water systems: Academic Press, New York, 185 p.

Armstrong, D.S., Richards, T.A., and Parker, G.W., 2001, Assessment of habitat, fish communities, and streamflow requirements for habitat protection, Ipswich River, Massachusetts: U.S. Geological Survey, Water-Resources Investigations Report 01-4161, 72 p.

Army Corp of Engineers, 1994, The National inventory of dams 1993–94: Troy, NY, Venture Computing Group, Inc., CD-ROM.

Barlow, P.M., Ahlfeld, D.P., and Dickerman, D.C., 2003, Conjunctive-management models for sustained yield of stream-aquifer systems: Journal of Water Resources Planning and Management, v. 129. no. 1, p. 35–48.

Barlow, P.M., and Dickerman, D.C., 2001, Numerical-simulation and conjunctive-management models of the Hunt–Annaquatucket–Pettaquamscutt stream-aquifer system, Rhode Island: U.S. Geological Survey Professional Paper 1636, 88 p.

Bouck, Bruce, 1998, The Upper Charles River dilemma—A growing basin: Boston, MA, Massachusetts Department of Environmental Protection, 15 p.

DeSimone, L.A., Walter, D.A., Eggleston, J.A., and Nimiroski, M.T., 2002, Simulation of ground-water flow and evaluation of water-management alternatives in the Upper Charles River basin, eastern Massachusetts: U.S. Geological Survey Water-Resources Investigations Report 02-4234, 94 p.

Earth Tech, 2000, Comprehensive wastewater management plan and final environmental impact report, Phase III document, Holliston, Massachusetts: Concord, MA, Earth Tech Inc., variously paged.

Gorelick, S.M., 1983, A review of distributed parameter ground-water management modeling methods: Water Resources Research, v. 19, no. 2, p. 305–319.

Harbaugh, A.W., Banta, E.R., Hill, M.C., and McDonald, M.G., 2000, MODFLOW-2000, the U.S. Geological Survey modular ground-water model—User guide to modularization concepts and the ground-water flow process: U.S. Geological Survey Open-File Report 00-92, 121 p.

Heathcote, I.W., 1998, Integrated watershed management; principles and practice: New York, John Wiley & Sons, 414 p.

Hill, M.C., 1990, Preconditioned conjugate-gradient 2 (PCG2), A computer program for solving ground-water flow equations: U.S. Geological Survey, Water-Resources Investigations Report 90-4048, 43 p.

Hornberger, G.M., Raffensperger, J.P., Wiberg, P.L., Eshleman, K.N., 1998, Elements of physical hydrology: Baltimore, MD, Johns Hopkins Press, 302 p.

Huff, F.A., and Changnon, S.A., 1986, Potential urban effects on precipitation in the winter and transition seasons at St. Louis, Missouri: Journal of Climate and Applied Meteorology, v. 25, no. 12, p. 1887–1907.

Kulik, B.H., 1990, A method to refine the New England Aquatic Base Flow Policy: Rivers, v.1, no. 1, p. 8–22.

Mays, L.W., 1997, Optimal control of hydrosystems: New York, Marcel Dekker, 372 p.

National Oceanic and Atmospheric Administration, 2001, National Virtual Data System, Climate data, Daily surface data, accessed May 12, 2001, at URL http://www.ncdc.noaa.gov/climate/stationlocator.html

Nishikawa, Tracy, 1998, Water-resources optimization model for Santa Barbara, California: Journal of Water Resources Planning and Management, v. 124, no. 5, p. 252–263.

Reichard, E.G., 1995, Ground-water-surface water management with stochastic surface water supplies—A simulation optimization approach: Water Resources Research, v. 31, no. 11, p. 2845–2865.

Riefler, R.G., and Ahlfeld, D.P., 1996, The impact of numerical precision on the solution of confined and unconfined optimal hydraulic control problems: Hazardous Waste and Hazardous Materials, v. 13, no. 2, p. 167–176.

Ries, K.G., and Friesz, P.J., 2000, Methods for estimating low-flow statistics for Massachusetts streams: U.S. Geological Survey Water-Resources Investigations Report 00-4135, 81 p.

Schrage, Linus, 1997, Optimization modeling with LINDO (5th ed): Pacific Grove, CA, Duxbury Press, 470 p.

Solley, W.B., Pierce, R.R., and Perlman, H.A., 1998, Estimated use of water in the United States in 1995: U.S. Geological Survey Circular 1200, 71 p.

U.S. Fish and Wildlife Service, 1981, Interim regional policy for New England streamflow recommendations: Newton Corner, MA, U.S. Fish and Wildlife Service, 3 p.

Wagner, B.J., 1995, Recent advances in simulation-optimization groundwater management modeling: Reviews of Geophysics, Supplement, v. 33, p. 1021–1028.

Yeo, J.L., and Estes-Smargiassi, S.A., 2000, Summary report of MWRA demand management program Fiscal Year 2000: Boston, MA, Water Resources Authority, Report MS-061, 23 p.

Appendix 1

Appendix 1. Historic average monthly stress rates (1989–98), upper Charles River basin, eastern Massachusetts.

[CRPCD, Charles River Pollution Control District; NEA, Northeast Energy Association; --, zero]

Stress description	Historic average stresses, in cubic foot per second												
	January	February	March	April	May	June	July	August	September	October	November	December	Annual
					Pumping								
Bellingham NEA Wells 1, 2, and 3	0.802	0.851	0.825	0.865	0.705	0.768	0.753	0.805	0.817	0.851	0.892	0.986	0.827
Bellingham NEA Wells 4 and 5	.023	.026	.023	.027	.028	.018	.016	.015	.015	.014	.016	.002	.019
Bellingham Well Number 12	.003	.028	.011	.016	.026	.032	.072	.057	.039	.029	.029	.026	.031
Bellingham Well Number 5	.280	.296	.320	.319	.313	.299	.317	.316	.306	.311	.294	.283	.304
Bellingham Well Number 7	.120	.111	.128	.148	.281	.271	.248	.172	.115	.106	.104	.072	.156
Bellingham Well Number 8	.253	.319	.356	.316	.484	.739	.746	.652	.354	.316	.279	.251	.422
Franklin Populatic Injection Well	--	--	--	--	--	--	--	--	--	--	--	--	--
Franklin Populatic Pond Well	--	--	--	--	--	--	--	--	--	--	--	--	--
Franklin Proposed Well Number 11	--	--	--	--	--	--	--	--	--	--	--	--	--
Franklin Well Number 1	.075	.108	.125	.056	.129	.322	.298	.289	.265	.241	.155	.064	.177
Franklin Well Number 10	.299	.289	.295	.301	.331	.375	.407	.372	.318	.365	.366	.362	.340
Franklin Well Number 2	.075	.108	.125	.056	.129	.322	.298	.289	.265	.241	.155	.064	.177
Franklin Well Number 3	.401	.375	.430	.481	.546	.572	.529	.504	.499	.435	.405	.427	.467
Franklin Well Number 4	.961	.950	.871	1.019	1.136	1.172	1.149	1.107	1.045	.984	.977	.980	1.029
Franklin Well Number 5	.249	.250	.260	.208	.276	.304	.320	.286	.237	.228	.244	.285	.262
Franklin Well Number 6	.571	.575	.555	.618	.633	.636	.596	.565	.451	.447	.339	.360	.529
Franklin Well Number 7	.427	.418	.417	.470	.480	.504	.496	.436	.414	.386	.379	.433	.438
Franklin Well Number 8	.475	.415	.438	.446	.500	.493	.511	.491	.493	.462	.450	.472	.470
Holliston Glen Ellen Country Club	--	--	--	.004	.060	.156	.189	.146	.078	.039	--	--	.056
Holliston Maplegate Country Club	--	--	--	.016	.031	.064	.111	.111	.115	.077	.032	--	.046
Holliston Proposed Well Number 7	--	--	--	--	--	--	--	--	--	--	--	--	--
Holliston Well Number 1 Lake Winthrop	--	--	--	.018	.101	.183	.234	.176	.133	.043	.026	--	.076
Holliston Well Number 2 Maple Street	.007	.011	.041	.029	.119	.219	.280	.234	.079	.022	.019	.003	.089
Holliston Well Number 4 Washington Street	.123	.138	.147	.111	.203	.382	.421	.333	.266	.200	.181	.143	.221
Holliston Well Number 5 Central Street	.707	.617	.612	.700	.811	.757	.746	.688	.637	.644	.661	.715	.691
Holliston Well Number 6 Brook Street	.704	.701	.687	.697	.733	.947	.794	.686	.665	.652	.567	.612	.704
Medway Proposed Well	--	--	--	--	--	--	--	--	--	--	--	--	--
Medway Well Number 1 Populatic Street	.581	.625	.694	.608	.616	.713	.665	.736	.696	.538	.597	.558	.636
Medway Well Number 2 Oakland Street	.147	.147	.104	.157	.260	.301	.336	.255	.221	.240	.171	.177	.210
Medway Well Number 3 Village Street	.507	.464	.437	.428	.545	.661	.677	.447	.477	.579	.443	.553	.518

Appendix 1. Historic average monthly stress rates (1989–98), upper Charles River basin, eastern Massachusetts.—Continued

[CRPCD, Charles River Pollution Control District; NEA, Northeast Energy Association; --, zero]

Stress description	Historic average stresses, in cubic foot per second												
	January	February	March	April	May	June	July	August	September	October	November	December	Annual
						Pumping—*Continued*							
Milford Clark Island Wellfield	0.807	0.672	0.419	0.449	0.416	0.458	0.749	0.908	0.836	0.802	0.776	0.664	0.663
Milford Dilla Street Wells Number 1	.008	.011	.004	--	--	--	.014	.027	.014	.021	.025	.017	.012
Milford Dilla Street Wells Number 2	.008	.011	.004	--	--	--	.014	.027	.014	.021	.025	.017	.012
Milford Godfrey Brook Well 3	.194	.184	.162	.153	.176	.167	.181	.182	.215	.201	.217	.224	.188
Milford Godfrey Brook Well 4	.194	.184	.162	.153	.176	.167	.181	.182	.215	.201	.217	.224	.188
Milford Godfrey Brook Well 5	.194	.184	.162	.153	.176	.167	.181	.182	.215	.201	.217	.224	.188
Norfolk Proposed Mill River Well	--	--	--	--	--	--	--	--	--	--	--	--	--
Norfolk Well Number 1 Gold Street	.276	.279	.333	.441	.436	.524	.481	.365	.375	.356	.353	.311	.378
Wrentham Proposed Well	--	--	--	--	--	--	--	--	--	--	--	--	--
Wrentham Well Number 2	.477	.477	.474	.457	.714	.598	.727	.556	.343	.443	.302	.300	.489
Wrentham Well Number 3	.559	.594	.514	.776	.499	.825	.773	.697	.806	.463	.601	.650	.646
						Stormwater Recharge							
Bellingham SW Recharge	--	--	--	--	--	--	--	--	--	--	--	--	--
						Wastewater Recharge							
Bellingham Proposed Facility	--	--	--	--	--	--	--	--	--	--	--	--	--
Holliston–Gorwin Drive Site	--	--	--	--	--	--	--	--	--	--	--	--	--
Holliston–EHIP-1 Site	--	--	--	--	--	--	--	--	--	--	--	--	--
Holliston–EHIP-2 Site	--	--	--	--	--	--	--	--	--	--	--	--	--
Recharge 2,608 ft from Miscoe Brook	--	--	--	--	--	--	--	--	--	--	--	--	--
Recharge 1,720 ft from Miscoe Boork	--	--	--	--	--	--	--	--	--	--	--	--	--
Recharge 894 ft from Miscoe Brook	--	--	--	--	--	--	--	--	--	--	--	--	--
Recharge 200 ft from Miscoe Brook	--	--	--	--	--	--	--	--	--	--	--	--	--
Hypothetical recharge at CRPCD	--	--	--	--	--	--	--	--	--	--	--	--	--
						Wastewater Discharge							
Bellingham Proposed	--	--	--	--	--	--	--	--	--	--	--	--	--
CRPCD	6.348	6.395	7.145	7.484	6.550	5.926	5.226	5.173	5.116	5.466	5.712	6.290	6.069
CRPCD Upstream Discharge	--	--	--	--	--	--	--	--	--	--	--	--	--
Milford	6.440	5.611	7.244	7.001	5.908	4.591	4.005	3.872	4.076	4.576	5.247	6.332	5.408
Miscoe Brook Hypothetical	--	--	--	--	--	--	--	--	--	--	--	--	--

Appendix 1. Historic average monthly stress rates (1989–98), upper Charles River basin, eastern Massachusetts.—Continued

[CRPCD, Charles River Pollution Control District; NEA, Northeast Energy Association; --, zero]

Stress description	Historic average stresses, in cubic foot per second												
	January	February	March	April	May	June	July	August	September	October	November	December	Annual
Combined Wastewater Discharge and Septic Discharge													
Bellingham Summer	--	--	--	--	1.104	1.342	1.382	1.198	--	--	--	--	0.419
Bellingham Winter	0.656	0.754	0.815	0.799	--	--	--	--	0.813	0.762	0.706	0.631	.495
Franklin Summer	--	--	--	--	4.555	5.129	5.069	4.788	--	--	--	--	1.628
Franklin Winter	3.857	3.837	3.836	3.997	--	--	--	--	4.417	4.226	3.901	3.859	2.661
Holliston Summer	--	--	--	--	1.967	2.488	2.475	2.117	--	--	--	--	.754
Holliston Winter	1.542	1.467	1.487	1.555	--	--	--	--	1.779	1.561	1.455	1.473	1.027
Medway Summer	--	--	--	--	1.420	1.675	1.678	1.438	--	--	--	--	.518
Medway Winter	1.235	1.236	1.236	1.193	--	--	--	--	1.394	1.357	1.211	1.288	.846
Milford Summer	--	--	--	--	.942	.960	1.320	1.508	--	--	--	--	.394
Milford Winter	1.404	1.246	.912	.909	--	--	--	--	1.510	1.449	1.478	1.368	.856
Norfolk Summer	--	--	--	--	.436	.524	.481	.365	--	--	--	--	.151
Norfolk Winter	.276	.279	.333	.441	--	--	--	--	.375	.356	.353	.311	.227
Wrentham Summer	--	--	--	--	1.213	1.423	1.500	1.253	--	--	--	--	.449
Wrentham Winter	1.036	1.071	.987	1.233	--	--	--	--	1.149	.905	.903	.950	.686
Direct Drinking-Water Discharge to Streams													
Drinking Water Discharge to Charles River	--	--	--	--	--	--	--	--	--	--	--	--	--
Drinking Water Discharge to Hopping Brook	--	--	--	--	--	--	--	--	--	--	--	--	--
Drinking Water Discharge to Mine Brook	--	--	--	--	--	--	--	--	--	--	--	--	--

Appendix 2

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Bellingham Northeast Energy Association wells 1, 2, and 3	Q01B1	1	—	0.574	0.574	0.574	—	—	—	—	—	—
	Q02B1	2	—	.199	.199	.199	—	—	—	—	—	—
	Q03B1	3	—	.092	.092	.092	—	—	—	—	—	—
	Q04B1	4	—	.063	.063	.063	—	—	—	—	—	—
	Q05B1	5	—	.036	.036	.036	—	—	—	—	—	—
	Q06B1	6	—	.025	.025	.025	—	—	—	—	—	—
	Q07B1	7	—	.010	.010	.010	—	—	—	—	—	—
	Q08B1	8	—	—	—	—	—	—	—	—	—	—
	Q09B1	9	—	—	—	—	—	—	—	—	—	—
	Q10B1	10	—	—	—	—	—	—	—	—	—	—
	Q11B1	11	—	—	—	—	—	—	—	—	—	—
	Q12B1	12	—	—	—	—	—	—	—	—	—	—
Bellingham Northeast Energy Association wells 4 and 5	Q01B4	1	—	.485	.485	.485	—	—	—	—	—	—
	Q02B4	2	—	.214	.214	.214	—	—	—	—	—	—
	Q03B4	3	—	.119	.119	.119	—	—	—	—	—	—
	Q04B4	4	—	.083	.083	.083	—	—	—	—	—	—
	Q05B4	5	—	.043	.043	.043	—	—	—	—	—	—
	Q06B4	6	—	.034	.034	.034	—	—	—	—	—	—
	Q07B4	7	—	.013	.013	.013	—	—	—	—	—	—
	Q08B4	8	—	.010	.010	.010	—	—	—	—	—	—
	Q09B4	9	—	—	—	—	—	—	—	—	—	—
	Q10B4	10	—	—	—	—	—	—	—	—	—	—
	Q11B4	11	—	—	—	—	—	—	—	—	—	—
	Q12B4	12	—	—	—	—	—	—	—	—	—	—
Bellingham proposed wastewater recharge facility	Q01BR	1	—	-.910	-.910	-.910	—	—	—	—	—	—
	Q02BR	2	—	-.077	-.077	-.077	—	—	—	—	—	—
	Q03BR	3	—	-.013	-.013	-.013	—	—	—	—	—	—
	Q04BR	4	—	—	—	—	—	—	—	—	—	—
	Q05BR	5	—	—	—	—	—	—	—	—	—	—
	Q06BR	6	—	—	—	—	—	—	—	—	—	—
	Q07BR	7	—	—	—	—	—	—	—	—	—	—
	Q08BR	8	—	—	—	—	—	—	—	—	—	—
	Q09BR	9	—	—	—	—	—	—	—	—	—	—
	Q10BR	10	—	—	—	—	—	—	—	—	—	—
	Q11BR	11	—	—	—	—	—	—	—	—	—	—
	Q12BR	12	—	—	—	—	—	—	—	—	—	—
Bellingham proposed stormwater recharge facility	Q01BS	1	—	-.754	-.754	-.754	—	—	—	—	—	—
	Q02BS	2	—	-.170	-.170	-.170	—	—	—	—	—	—
	Q03BS	3	—	-.042	-.042	-.042	—	—	—	—	—	—
	Q04BS	4	—	-.020	-.020	-.020	—	—	—	—	—	—
	Q05BS	5	—	-.014	-.014	-.014	—	—	—	—	—	—
	Q06BS	6	—	—	—	—	—	—	—	—	—	—
	Q07BS	7	—	—	—	—	—	—	—	—	—	—
	Q08BS	8	—	—	—	—	—	—	—	—	—	—
	Q09BS	9	—	—	—	—	—	—	—	—	—	—
	Q10BS	10	—	—	—	—	—	—	—	—	—	—
	Q11BS	11	—	—	—	—	—	—	—	—	—	—
	Q12BS	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Bellingham well 12	Q01B9	1	—	0.368	0.368	0.368	—	—	—	—	—	—
	Q02B9	2	—	.235	.235	.235	—	—	—	—	—	—
	Q03B9	3	—	.139	.139	.139	—	—	—	—	—	—
	Q04B9	4	—	.090	.090	.090	—	—	—	—	—	—
	Q05B9	5	—	.060	.060	.060	—	—	—	—	—	—
	Q06B9	6	—	.038	.038	.038	—	—	—	—	—	—
	Q07B9	7	—	.029	.029	.029	—	—	—	—	—	—
	Q08B9	8	—	.025	.025	.025	—	—	—	—	—	—
	Q09B9	9	—	.016	.016	.016	—	—	—	—	—	—
	Q10B9	10	—	—	—	—	—	—	—	—	—	—
	Q11B9	11	—	—	—	—	—	—	—	—	—	—
	Q12B9	12	—	—	—	—	—	—	—	—	—	—
Bellingham well 5	Q01B5	1	—	.613	.613	.613	—	—	—	—	—	—
	Q02B5	2	—	.141	.141	.141	—	—	—	—	—	—
	Q03B5	3	—	.073	.073	.073	—	—	—	—	—	—
	Q04B5	4	—	.086	.086	.086	—	—	—	—	—	—
	Q05B5	5	—	.021	.021	.021	—	—	—	—	—	—
	Q06B5	6	—	.031	.031	.031	—	—	—	—	—	—
	Q07B5	7	—	.021	.021	.021	—	—	—	—	—	—
	Q08B5	8	—	.013	.013	.013	—	—	—	—	—	—
	Q09B5	9	—	—	—	—	—	—	—	—	—	—
	Q10B5	10	—	—	—	—	—	—	—	—	—	—
	Q11B5	11	—	—	—	—	—	—	—	—	—	—
	Q12B5	12	—	—	—	—	—	—	—	—	—	—
Bellingham well 7	Q01B7	1	—	.406	.406	.406	—	—	—	—	—	—
	Q02B7	2	—	.188	.188	.188	—	—	—	—	—	—
	Q03B7	3	—	.125	.125	.125	—	—	—	—	—	—
	Q04B7	4	—	.104	.104	.104	—	—	—	—	—	—
	Q05B7	5	—	.061	.061	.061	—	—	—	—	—	—
	Q06B7	6	—	.039	.039	.039	—	—	—	—	—	—
	Q07B7	7	—	.029	.029	.029	—	—	—	—	—	—
	Q08B7	8	—	.021	.021	.021	—	—	—	—	—	—
	Q09B7	9	—	.016	.016	.016	—	—	—	—	—	—
	Q10B7	10	—	.012	.012	.012	—	—	—	—	—	—
	Q11B7	11	—	—	—	—	—	—	—	—	—	—
	Q12B7	12	—	—	—	—	—	—	—	—	—	—
Bellingham well 8	Q01B8	1	—	.480	.480	.480	—	—	—	—	—	—
	Q02B8	2	—	.175	.175	.175	—	—	—	—	—	—
	Q03B8	3	—	.099	.099	.099	—	—	—	—	—	—
	Q04B8	4	—	.067	.067	.067	—	—	—	—	—	—
	Q05B8	5	—	.047	.047	.047	—	—	—	—	—	—
	Q06B8	6	—	.044	.044	.044	—	—	—	—	—	—
	Q07B8	7	—	.027	.027	.027	—	—	—	—	—	—
	Q08B8	8	—	.024	.024	.024	—	—	—	—	—	—
	Q09B8	9	—	.020	.020	.020	—	—	—	—	—	—
	Q10B8	10	—	.016	.016	.016	—	—	—	—	—	—
	Q11B8	11	—	—	—	—	—	—	—	—	—	—
	Q12B8	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Bellingham wastewater return May–August	R01BS	1	—	-0.387	-0.387	-0.387	—	—	—	—	—	—
	R02BS	2	—	-.133	-.133	-.133	—	—	—	—	—	—
	R03BS	3	—	-.070	-.070	-.070	—	—	—	—	—	—
	R04BS	4	—	-.037	-.037	-.037	—	—	—	—	—	—
	R05BS	5	—	-.037	-.037	-.037	—	—	—	—	—	—
	R06BS	6	—	-.023	-.023	-.023	—	—	—	—	—	—
	R07BS	7	—	-.013	-.013	-.013	—	—	—	—	—	—
	R08BS	8	—	—	—	—	—	—	—	—	—	—
	R09BS	9	—	—	—	—	—	—	—	—	—	—
	R10BS	10	—	—	—	—	—	—	—	—	—	—
	R11BS	11	—	—	—	—	—	—	—	—	—	—
	R12BS	12	—	—	—	—	—	—	—	—	—	—
Bellingham wastewater return September–April	R01BW	1	—	-.597	-.597	-.597	—	—	—	—	—	—
	R02BW	2	—	-.159	-.159	-.159	—	—	—	—	—	—
	R03BW	3	—	-.083	-.083	-.083	—	—	—	—	—	—
	R04BW	4	—	-.047	-.047	-.047	—	—	—	—	—	—
	R05BW	5	—	-.034	-.034	-.034	—	—	—	—	—	—
	R06BW	6	—	-.032	-.032	-.032	—	—	—	—	—	—
	R07BW	7	—	-.029	-.029	-.029	—	—	—	—	—	—
	R08BW	8	—	-.018	-.018	-.018	—	—	—	—	—	—
	R09BW	9	—	—	—	—	—	—	—	—	—	—
	R10BW	10	—	—	—	—	—	—	—	—	—	—
	R11BW	11	—	—	—	—	—	—	—	—	—	—
	R12BW	12	—	—	—	—	—	—	—	—	—	—
Franklin wastewater return May–August	R01FS	1	—	—	-.031	-.584	—	—	—	—	-0.013	-0.125
	R02FS	2	—	—	-.023	-.050	—	—	—	-0.013	-.016	-.052
	R03FS	3	—	—	-.011	-.024	—	—	—	—	—	-.044
	R04FS	4	—	—	-.010	-.015	—	—	—	—	—	-.037
	R05FS	5	—	—	—	-.015	—	—	—	—	—	-.033
	R06FS	6	—	—	—	-.013	—	—	—	—	—	-.033
	R07FS	7	—	—	—	—	—	—	—	—	—	—
	R08FS	8	—	—	—	—	—	—	—	—	—	—
	R09FS	9	—	—	—	—	—	—	—	—	—	—
	R10FS	10	—	—	—	—	—	—	—	—	—	—
	R11FS	11	—	—	—	—	—	—	—	—	—	—
	R12FS	12	—	—	—	—	—	—	—	—	—	—
Franklin wastewater return September–April	R01FW	1	—	—	-.045	-.788	—	—	—	-.016	-.019	-.178
	R02FW	2	—	—	-.033	-.067	—	—	—	—	-.023	-.074
	R03FW	3	—	—	-.016	-.053	—	—	—	—	-.011	-.063
	R04FW	4	—	—	-.015	-.035	—	—	—	—	-.011	-.053
	R05FW	5	—	—	-.013	-.029	—	—	—	—	-.011	-.047
	R06FW	6	—	—	-.011	-.028	—	—	—	—	—	-.047
	R07FW	7	—	—	—	—	—	—	—	—	—	—
	R08FW	8	—	—	—	—	—	—	—	—	—	—
	R09FW	9	—	—	—	—	—	—	—	—	—	—
	R10FW	10	—	—	—	—	—	—	—	—	—	—
	R11FW	11	—	—	—	—	—	—	—	—	—	—
	R12FW	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Holliston wastewater return May–August	R01HS	1	—	—	-0.129	-0.166	-0.202	-0.085	-0.110	—	—	—
	R02HS	2	—	—	-.043	-.043	-.065	-.022	-.027	—	—	—
	R03HS	3	—	—	-.025	-.025	-.047	-.014	-.015	—	—	—
	R04HS	4	—	—	-.015	-.015	-.035	—	—	—	—	—
	R05HS	5	—	—	-.012	-.012	-.025	—	—	—	—	—
	R06HS	6	—	—	-.010	-.010	-.021	—	—	—	—	—
	R07HS	7	—	—	—	—	-.014	—	—	—	—	—
	R08HS	8	—	—	—	—	-.011	—	—	—	—	—
	R09HS	9	—	—	—	—	-.008	—	—	—	—	—
	R10HS	10	—	—	—	—	—	—	—	—	—	—
	R11HS	11	—	—	—	—	—	—	—	—	—	—
	R12HS	12	—	—	—	—	—	—	—	—	—	—
Holliston wastewater return September–April	R01HW	1	—	—	-.280	-.283	-.249	-.110	-.139	—	—	—
	R02HW	2	—	—	-.061	-.061	-.089	-.023	-.027	—	—	—
	R03HW	3	—	—	-.038	-.038	-.063	-.012	-.014	—	—	—
	R04HW	4	—	—	-.027	-.027	-.046	—	—	—	—	—
	R05HW	5	—	—	-.022	-.022	-.034	—	—	—	—	—
	R06HW	6	—	—	-.026	-.026	-.026	—	—	—	—	—
	R07HW	7	—	—	—	—	-.018	—	—	—	—	—
	R08HW	8	—	—	—	—	-.011	—	—	—	—	—
	R09HW	9	—	—	—	—	-.008	—	—	—	—	—
	R10HW	10	—	—	—	—	—	—	—	—	—	—
	R11HW	11	—	—	—	—	—	—	—	—	—	—
	R12HW	12	—	—	—	—	—	—	—	—	—	—
Medway wastewater return May–August	R01DS	1	—	—	-.127	-.478	-.056	—	-.029	—	—	—
	R02DS	2	—	—	-.051	-.055	-.021	—	-.014	—	—	—
	R03DS	3	—	—	-.027	-.030	-.014	—	—	—	—	—
	R04DS	4	—	—	-.017	-.018	-.011	—	—	—	—	—
	R05DS	5	—	—	-.009	-.009	-.008	—	—	—	—	—
	R06DS	6	—	—	—	—	—	—	—	—	—	—
	R07DS	7	—	—	—	—	—	—	—	—	—	—
	R08DS	8	—	—	—	—	—	—	—	—	—	—
	R09DS	9	—	—	—	—	—	—	—	—	—	—
	R10DS	10	—	—	—	—	—	—	—	—	—	—
	R11DS	11	—	—	—	—	—	—	—	—	—	—
	R12DS	12	—	—	—	—	—	—	—	—	—	—
Medway wastewater return September–April	R01DW	1	—	—	-.211	-.721	-.082	—	-.052	—	—	—
	R02DW	2	—	—	-.046	-.048	-.019	—	-.016	—	—	—
	R03DW	3	—	—	-.029	-.033	-.016	—	—	—	—	—
	R04DW	4	—	—	-.021	-.027	-.016	—	—	—	—	—
	R05DW	5	—	—	-.020	-.022	-.014	—	—	—	—	—
	R06DW	6	—	—	—	—	—	—	—	—	—	—
	R07DW	7	—	—	—	—	—	—	—	—	—	—
	R08DW	8	—	—	—	—	—	—	—	—	—	—
	R09DW	9	—	—	—	—	—	—	—	—	—	—
	R10DW	10	—	—	—	—	—	—	—	—	—	—
	R11DW	11	—	—	—	—	—	—	—	—	—	—
	R12DW	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Milford wastewater return May–August	R01MS	1	-0.015	-0.626	-0.626	-0.626	—	—	—	—	—	—
	R02MS	2	-.013	-.017	-.017	-.017	—	—	—	—	—	—
	R03MS	3	-.010	-.016	-.016	-.016	—	—	—	—	—	—
	R04MS	4	-.009	-.014	-.014	-.014	—	—	—	—	—	—
	R05MS	5	-.009	-.014	-.014	-.014	—	—	—	—	—	—
	R06MS	6	—	-.012	-.012	-.012	—	—	—	—	—	—
	R07MS	7	—	—	—	—	—	—	—	—	—	—
	R08MS	8	—	—	—	—	—	—	—	—	—	—
	R09MS	9	—	—	—	—	—	—	—	—	—	—
	R10MS	10	—	—	—	—	—	—	—	—	—	—
	R11MS	11	—	—	—	—	—	—	—	—	—	—
	R12MS	12	—	—	—	—	—	—	—	—	—	—
Milford wastewater return September–April	R01MW	1	-.107	-1.000	-1.000	-1.000	—	—	—	—	—	—
	R02MW	2	—	—	—	—	—	—	—	—	—	—
	R03MW	3	—	—	—	—	—	—	—	—	—	—
	R04MW	4	—	—	—	—	—	—	—	—	—	—
	R05MW	5	—	—	—	—	—	—	—	—	—	—
	R06MW	6	—	—	—	—	—	—	—	—	—	—
	R07MW	7	—	—	—	—	—	—	—	—	—	—
	R08MW	8	—	—	—	—	—	—	—	—	—	—
	R09MW	9	—	—	—	—	—	—	—	—	—	—
	R10MW	10	—	—	—	—	—	—	—	—	—	—
	R11MW	11	—	—	—	—	—	—	—	—	—	—
	R12MW	12	—	—	—	—	—	—	—	—	—	—
Norfolk wastewater return May–August	R01NS	1	—	—	—	-.412	—	—	—	—	—	-0.302
	R02NS	2	—	—	—	-.104	—	—	—	—	—	-.067
	R03NS	3	—	—	—	-.057	—	—	—	—	—	-.041
	R04NS	4	—	—	—	-.037	—	—	—	—	—	-.028
	R05NS	5	—	—	—	-.026	—	—	—	—	—	-.021
	R06NS	6	—	—	—	-.019	—	—	—	—	—	-.016
	R07NS	7	—	—	—	-.013	—	—	—	—	—	-.012
	R08NS	8	—	—	—	-.011	—	—	—	—	—	-.011
	R09NS	9	—	—	—	-.011	—	—	—	—	—	-.009
	R10NS	10	—	—	—	-.009	—	—	—	—	—	--
	R11NS	11	—	—	—	—	—	—	—	—	—	--
	R12NS	12	—	—	—	—	—	—	—	—	—	--
Norfolk wastewater return September–April	R01NW	1	—	—	—	-.534	—	—	—	—	—	-.393
	R02NW	2	—	—	—	-.150	—	—	—	—	—	-.137
	R03NW	3	—	—	—	-.098	—	—	—	—	—	-.103
	R04NW	4	—	—	—	-.065	—	—	—	—	—	-.081
	R05NW	5	—	—	—	-.056	—	—	—	—	—	-.068
	R06NW	6	—	—	—	-.046	—	—	—	—	—	-.060
	R07NW	7	—	—	—	-.019	—	—	—	—	—	-.014
	R08NW	8	—	—	—	-.013	—	—	—	—	—	-.011
	R09NW	9	—	—	—	-.011	—	—	—	—	—	-.009
	R10NW	10	—	—	—	-.010	—	—	—	—	—	—
	R11NW	11	—	—	—	—	—	—	—	—	—	—
	R12NW	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Wrentham wastewater return May–August	R01WS	1	—	—	—	-0.179	—	—	—	—	—	-0.179
	R02WS	2	—	—	—	-.083	—	—	—	—	—	-.083
	R03WS	3	—	—	—	-.073	—	—	—	—	—	-.073
	R04WS	4	—	—	—	-.067	—	—	—	—	—	-.067
	R05WS	5	—	—	—	-.061	—	—	—	—	—	-.061
	R06WS	6	—	—	—	-.055	—	—	—	—	—	-.055
	R07WS	7	—	—	—	-.038	—	—	—	—	—	-.038
	R08WS	8	—	—	—	-.035	—	—	—	—	—	-.035
	R09WS	9	—	—	—	-.032	—	—	—	—	—	-.032
	R10WS	10	—	—	—	-.029	—	—	—	—	—	-.029
	R11WS	11	—	—	—	-.024	—	—	—	—	—	-.024
	R12WS	12	—	—	—	-.025	—	—	—	—	—	-.025
Wrentham wastewater return September–April	R01WW	1	—	—	—	-.204	—	—	—	—	—	-.204
	R02WW	2	—	—	—	-.139	—	—	—	—	—	-.139
	R03WW	3	—	—	—	-.125	—	—	—	—	—	-.125
	R04WW	4	—	—	—	-.113	—	—	—	—	—	-.113
	R05WW	5	—	—	—	-.101	—	—	—	—	—	-.101
	R06WW	6	—	—	—	-.096	—	—	—	—	—	-.096
	R07WW	7	—	—	—	-.051	—	—	—	—	—	-.051
	R08WW	8	—	—	—	-.046	—	—	—	—	—	-.046
	R09WW	9	—	—	—	-.041	—	—	—	—	—	-.041
	R10WW	10	—	—	—	-.037	—	—	—	—	—	-.037
	R11WW	11	—	—	—	-.026	—	—	—	—	—	-.026
	R12WW	12	—	—	—	-.022	—	—	—	—	—	-.022
Franklin hypothetical wastewater injection well	Q01FI	1	—	—	—	-.755	—	—	—	—	—	—
	Q02FI	2	—	—	—	-.176	—	—	—	—	—	—
	Q03FI	3	—	—	—	-.048	—	—	—	—	—	—
	Q04FI	4	—	—	—	-.021	—	—	—	—	—	—
	Q05FI	5	—	—	—	—	—	—	—	—	—	—
	Q06FI	6	—	—	—	—	—	—	—	—	—	—
	Q07FI	7	—	—	—	—	—	—	—	—	—	—
	Q08FI	8	—	—	—	—	—	—	—	—	—	—
	Q09FI	9	—	—	—	—	—	—	—	—	—	—
	Q10FI	10	—	—	—	—	—	—	—	—	—	—
	Q11FI	11	—	—	—	—	—	—	—	—	—	—
	Q12FI	12	—	—	—	—	—	—	—	—	—	—
Franklin proposed Populatic Pond well	Q01FP	1	—	—	—	.776	—	—	—	—	—	—
	Q02FP	2	—	—	—	.145	—	—	—	—	—	—
	Q03FP	3	—	—	—	.036	—	—	—	—	—	—
	Q04FP	4	—	—	—	.011	—	—	—	—	—	—
	Q05FP	5	—	—	—	.016	—	—	—	—	—	—
	Q06FP	6	—	—	—	.016	—	—	—	—	—	—
	Q07FP	7	—	—	—	—	—	—	—	—	—	—
	Q08FP	8	—	—	—	—	—	—	—	—	—	—
	Q09FP	9	—	—	—	—	—	—	—	—	—	—
	Q10FP	10	—	—	—	—	—	—	—	—	—	—
	Q11FP	11	—	—	—	—	—	—	—	—	—	—
	Q12FP	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Franklin proposed well 11	Q01FE	1	—	—	0.117	0.117	—	—	—	0.117	0.117	—
	Q02FE	2	—	—	.110	.110	—	—	—	.110	.110	—
	Q03FE	3	—	—	.105	.105	—	—	—	.105	.105	—
	Q04FE	4	—	—	.120	.120	—	—	—	.120	.120	—
	Q05FE	5	—	—	.111	.111	—	—	—	.111	.111	—
	Q06FE	6	—	—	.105	.105	—	—	—	.105	.105	—
	Q07FE	7	—	—	.086	.086	—	—	—	.086	.086	—
	Q08FE	8	—	—	.080	.080	—	—	—	.080	.080	—
	Q09FE	9	—	—	.066	.066	—	—	—	.066	.066	—
	Q10FE	10	—	—	.046	.046	—	—	—	.046	.046	—
	Q11FE	11	—	—	.033	.033	—	—	—	.033	.033	—
	Q12FE	12	—	—	.021	.021	—	—	—	.021	.021	—
Franklin well 1	Q01F1	1	—	—	.148	.148	—	—	—	.148	.148	—
	Q02F1	2	—	—	.143	.143	—	—	—	.143	.143	—
	Q03F1	3	—	—	.110	.110	—	—	—	.110	.110	—
	Q04F1	4	—	—	.098	.098	—	—	—	.098	.098	—
	Q05F1	5	—	—	.093	.093	—	—	—	.093	.093	—
	Q06F1	6	—	—	.076	.076	—	—	—	.076	.076	—
	Q07F1	7	—	—	.081	.081	—	—	—	.081	.081	—
	Q08F1	8	—	—	.075	.075	—	—	—	.075	.075	—
	Q09F1	9	—	—	.072	.072	—	—	—	.072	.072	—
	Q10F1	10	—	—	.054	.054	—	—	—	.054	.054	—
	Q11F1	11	—	—	.050	.050	—	—	—	.050	.050	—
	Q12F1	12	—	—	--	--	—	—	—	—	—	—
Franklin well 10	Q01FT	1	—	—	.654	.654	—	—	—	.654	.654	—
	Q02FT	2	—	—	.222	.222	—	—	—	.222	.222	—
	Q03FT	3	—	—	.074	.074	—	—	—	.074	.074	—
	Q04FT	4	—	—	.034	.034	—	—	—	.034	.034	—
	Q05FT	5	—	—	.016	.016	—	—	—	.016	.016	—
	Q06FT	6	—	—	—	—	—	—	—	—	—	—
	Q07FT	7	—	—	—	—	—	—	—	—	—	—
	Q08FT	8	—	—	—	—	—	—	—	—	—	—
	Q09FT	9	—	—	—	—	—	—	—	—	—	—
	Q10FT	10	—	—	—	—	—	—	—	—	—	—
	Q11FT	11	—	—	—	—	—	—	—	—	—	—
	Q12FT	12	—	—	—	—	—	—	—	—	—	—
Franklin well 2	Q01F2	1	—	—	.273	.273	—	—	—	.273	.273	—
	Q02F2	2	—	—	.180	.180	—	—	—	.180	.180	—
	Q03F2	3	—	—	.115	.115	—	—	—	.115	.115	—
	Q04F2	4	—	—	.102	.102	—	—	—	.102	.102	—
	Q05F2	5	—	—	.091	.091	—	—	—	.091	.091	—
	Q06F2	6	—	—	.070	.070	—	—	—	.070	.070	—
	Q07F2	7	—	—	.041	.041	—	—	—	.041	.041	—
	Q08F2	8	—	—	.036	.036	—	—	—	.036	.036	—
	Q09F2	9	—	—	.031	.031	—	—	—	.031	.031	—
	Q10F2	10	—	—	.031	.031	—	—	—	.031	.031	—
	Q11F2	11	—	—	.029	.029	—	—	—	.029	.029	—
	Q12F2	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Franklin well 3	Q01F3	1	—	—	0.335	0.335	—	—	—	0.335	0.335	—
	Q02F3	2	—	—	.212	.212	—	—	—	.212	.212	—
	Q03F3	3	—	—	.138	.138	—	—	—	.138	.138	—
	Q04F3	4	—	—	.098	.098	—	—	—	.098	.098	—
	Q05F3	5	—	—	.069	.069	—	—	—	.069	.069	—
	Q06F3	6	—	—	.050	.050	—	—	—	.050	.050	—
	Q07F3	7	—	—	.035	.035	—	—	—	.035	.035	—
	Q08F3	8	—	—	.028	.028	—	—	—	.028	.028	—
	Q09F3	9	—	—	.021	.021	—	—	—	.021	.021	—
	Q10F3	10	—	—	.013	.013	—	—	—	.013	.013	—
	Q11F3	11	—	—	—	—	—	—	—	—	—	—
	Q12F3	12	—	—	—	—	—	—	—	—	—	—
Franklin well 4	Q01F4	1	—	—	—	.186	—	—	—	—	—	0.191
	Q02F4	2	—	—	—	.284	—	—	—	—	—	.236
	Q03F4	3	—	—	—	.205	—	—	—	—	—	.149
	Q04F4	4	—	—	—	.136	—	—	—	—	—	.088
	Q05F4	5	—	—	—	.097	—	—	—	—	—	.050
	Q06F4	6	—	—	—	.044	—	—	—	—	—	.031
	Q07F4	7	—	—	—	.030	—	—	—	—	—	.019
	Q08F4	8	—	—	—	.019	—	—	—	—	—	—
	Q09F4	9	—	—	—	—	—	—	—	—	—	—
	Q10F4	10	—	—	—	—	—	—	—	—	—	—
	Q11F4	11	—	—	—	—	—	—	—	—	—	—
	Q12F4	12	—	—	—	—	—	—	—	—	—	—
Franklin well 5	Q01F5	1	—	—	—	.373	—	—	—	—	—	.373
	Q02F5	2	—	—	—	.303	—	—	—	—	—	.303
	Q03F5	3	—	—	—	.154	—	—	—	—	—	.154
	Q04F5	4	—	—	—	.079	—	—	—	—	—	.079
	Q05F5	5	—	—	—	.045	—	—	—	—	—	.045
	Q06F5	6	—	—	—	.032	—	—	—	—	—	.032
	Q07F5	7	—	—	—	.014	—	—	—	—	—	.014
	Q08F5	8	—	—	—	—	—	—	—	—	—	—
	Q09F5	9	—	—	—	—	—	—	—	—	—	—
	Q10F5	10	—	—	—	—	—	—	—	—	—	—
	Q11F5	11	—	—	—	—	—	—	—	—	—	—
	Q12F5	12	—	—	—	—	—	—	—	—	—	—
Franklin well 6	Q01F6	1	—	—	.622	.622	—	—	—	.622	.622	—
	Q02F6	2	—	—	.146	.146	—	—	—	.146	.146	—
	Q03F6	3	—	—	.078	.078	—	—	—	.078	.078	—
	Q04F6	4	—	—	.054	.054	—	—	—	.054	.054	—
	Q05F6	5	—	—	.035	.035	—	—	—	.035	.035	—
	Q06F6	6	—	—	.022	.022	—	—	—	.022	.022	—
	Q07F6	7	—	—	.016	.016	—	—	—	.016	.016	—
	Q08F6	8	—	—	.014	.014	—	—	—	.014	.014	—
	Q09F6	9	—	—	.011	.011	—	—	—	.011	.011	—
	Q10F6	10	—	—	—	—	—	—	—	—	—	—
	Q11F6	11	—	—	—	—	—	—	—	—	—	—
	Q12F6	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Franklin well 7	Q01F7	1	—	—	0.165	0.165	—	—	—	—	0.165	—
	Q02F7	2	—	—	.151	.151	—	—	—	—	.151	—
	Q03F7	3	—	—	.106	.106	—	—	—	—	.106	—
	Q04F7	4	—	—	.095	.095	—	—	—	—	.095	—
	Q05F7	5	—	—	.084	.084	—	—	—	—	.084	—
	Q06F7	6	—	—	.079	.079	—	—	—	—	.079	—
	Q07F7	7	—	—	.063	.063	—	—	—	—	.063	—
	Q08F7	8	—	—	.053	.053	—	—	—	—	.053	—
	Q09F7	9	—	—	.042	.042	—	—	—	—	.042	—
	Q10F7	10	—	—	.040	.040	—	—	—	—	.040	—
	Q11F7	11	—	—	.021	.021	—	—	—	—	.021	—
	Q12F7	12	—	—	.017	.017	—	—	—	—	.017	—
Franklin well 8	Q01F8	1	—	—	—	.732	—	—	—	—	—	—
	Q02F8	2	—	—	—	.192	—	—	—	—	—	—
	Q03F8	3	—	—	—	.056	—	—	—	—	—	—
	Q04F8	4	—	—	—	.020	—	—	—	—	—	—
	Q05F8	5	—	—	—	—	—	—	—	—	—	—
	Q06F8	6	—	—	—	—	—	—	—	—	—	—
	Q07F8	7	—	—	—	—	—	—	—	—	—	—
	Q08F8	8	—	—	—	—	—	—	—	—	—	—
	Q09F8	9	—	—	—	—	—	—	—	—	—	—
	Q10F8	10	—	—	—	—	—	—	—	—	—	—
	Q11F8	11	—	—	—	—	—	—	—	—	—	—
	Q12F8	12	—	—	—	—	—	—	—	—	—	—
Holliston Glen Ellen Country Club well	Q01HG	1	—	—	—	—	0.900	—	—	—	—	—
	Q02HG	2	—	—	—	—	.083	—	—	—	—	—
	Q03HG	3	—	—	—	—	.017	—	—	—	—	—
	Q04HG	4	—	—	—	—	—	—	—	—	—	—
	Q05HG	5	—	—	—	—	—	—	—	—	—	—
	Q06HG	6	—	—	—	—	—	—	—	—	—	—
	Q07HG	7	—	—	—	—	—	—	—	—	—	—
	Q08HG	8	—	—	—	—	—	—	—	—	—	—
	Q09HG	9	—	—	—	—	—	—	—	—	—	—
	Q10HG	10	—	—	—	—	—	—	—	—	—	—
	Q11HG	11	—	—	—	—	—	—	—	—	—	—
	Q12HG	12	—	—	—	—	—	—	—	—	—	—
Holliston EHIP-1 wastewater recharge site	W01H1	1	—	—	—	—	-.191	—	—	—	—	—
	W02H1	2	—	—	—	—	-.194	—	—	—	—	—
	W03H1	3	—	—	—	—	-.133	—	—	—	—	—
	W04H1	4	—	—	—	—	-.106	—	—	—	—	—
	W05H1	5	—	—	—	—	-.081	—	—	—	—	—
	W06H1	6	—	—	—	—	-.073	—	—	—	—	—
	W07H1	7	—	—	—	—	-.062	—	—	—	—	—
	W08H1	8	—	—	—	—	-.048	—	—	—	—	—
	W09H1	9	—	—	—	—	-.042	—	—	—	—	—
	W10H1	10	—	—	—	—	-.034	—	—	—	—	—
	W11H1	11	—	—	—	—	-.020	—	—	—	—	—
	W12H1	12	—	—	—	—	-.017	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Holliston EHIP-2 wastewater recharge site	W01H2	1	—	—	—	—	-0.186	—	—	—	—	—
	W02H2	2	—	—	—	—	-.223	—	—	—	—	—
	W03H2	3	—	—	—	—	-.177	—	—	—	—	—
	W04H2	4	—	—	—	—	-.120	—	—	—	—	—
	W05H2	5	—	—	—	—	-.093	—	—	—	—	—
	W06H2	6	—	—	—	—	-.062	—	—	—	—	—
	W07H2	7	—	—	—	—	-.048	—	—	—	—	—
	W08H2	8	—	—	—	—	-.038	—	—	—	—	—
	W09H2	9	—	—	—	—	-.027	—	—	—	—	—
	W10H2	10	—	—	—	—	-.014	—	—	—	—	—
	W11H2	11	—	—	—	—	-.010	—	—	—	—	—
	W12H2	12	—	—	—	—	—	—	—	—	—	—
Holliston Gorwin Drive wastewater recharge site	W01HG	1	—	—	-0.480	-0.480	—	-0.480	-0.480	—	—	—
	W02HG	2	—	—	-.162	-.162	—	-.162	-.162	—	—	—
	W03HG	3	—	—	-.093	-.093	—	-.093	-.093	—	—	—
	W04HG	4	—	—	-.067	-.067	—	-.067	-.067	—	—	—
	W05HG	5	—	—	-.047	-.047	—	-.047	-.047	—	—	—
	W06HG	6	—	—	-.034	-.034	—	-.034	-.034	—	—	—
	W07HG	7	—	—	-.020	-.020	—	-.020	-.020	—	—	—
	W08HG	8	—	—	-.022	-.022	—	-.022	-.022	—	—	—
	W09HG	9	—	—	-.022	-.022	—	-.022	-.022	—	—	—
	W10HG	10	—	—	-.017	-.017	—	-.017	-.017	—	—	—
	W11HG	11	—	—	-.016	-.016	—	-.016	-.016	—	—	—
	W12HG	12	—	—	-.016	-.016	—	-.016	-.016	—	—	—
Holliston Maplegate Country Club well	Q01HM	1	—	—	.347	.347	—	—	—	—	0.347	—
	Q02HM	2	—	—	.155	.155	—	—	—	—	.155	—
	Q03HM	3	—	—	.109	.109	—	—	—	—	.109	—
	Q04HM	4	—	—	.090	.090	—	—	—	—	.090	—
	Q05HM	5	—	—	.072	.072	—	—	—	—	.072	—
	Q06HM	6	—	—	.057	.057	—	—	—	—	.057	—
	Q07HM	7	—	—	.050	.050	—	—	—	—	.050	—
	Q08HM	8	—	—	.034	.034	—	—	—	—	.034	—
	Q09HM	9	—	—	.028	.028	—	—	—	—	.028	—
	Q10HM	10	—	—	.023	.023	—	—	—	—	.023	—
	Q11HM	11	—	—	.019	.019	—	—	—	—	.019	—
	Q12HM	12	—	—	.016	.016	—	—	—	—	.016	—
Holliston proposed well 7	Q01H7	1	—	—	—	—	.047	—	—	—	—	—
	Q02H7	2	—	—	—	—	.102	—	—	—	—	—
	Q03H7	3	—	—	—	—	.161	—	—	—	—	—
	Q04H7	4	—	—	—	—	.339	—	—	—	—	—
	Q05H7	5	—	—	—	—	.166	—	—	—	—	—
	Q06H7	6	—	—	—	—	.087	—	—	—	—	—
	Q07H7	7	—	—	—	—	.047	—	—	—	—	—
	Q08H7	8	—	—	—	—	.025	—	—	—	—	—
	Q09H7	9	—	—	—	—	.015	—	—	—	—	—
	Q10H7	10	—	—	—	—	.011	—	—	—	—	—
	Q11H7	11	—	—	—	—	—	—	—	—	—	—
	Q12H7	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Holliston well 1	Q01H1	1	—	—	—	—	0.343	—	—	—	—	—
	Q02H1	2	—	—	—	—	.220	—	—	—	—	—
	Q03H1	3	—	—	—	—	.144	—	—	—	—	—
	Q04H1	4	—	—	—	—	.104	—	—	—	—	—
	Q05H1	5	—	—	—	—	.054	—	—	—	—	—
	Q06H1	6	—	—	—	—	.047	—	—	—	—	—
	Q07H1	7	—	—	—	—	.033	—	—	—	—	—
	Q08H1	8	—	—	—	—	.026	—	—	—	—	—
	Q09H1	9	—	—	—	—	.017	—	—	—	—	—
	Q10H1	10	—	—	—	—	.012	—	—	—	—	—
	Q11H1	11	—	—	—	—	--	—	—	—	—	—
	Q12H1	12	—	—	—	—	--	—	—	—	—	—
Holliston well 2	Q01H2	1	—	—	—	—	.760	—	—	—	—	—
	Q02H2	2	—	—	—	—	.123	—	—	—	—	—
	Q03H2	3	—	—	—	—	.046	—	—	—	—	—
	Q04H2	4	—	—	—	—	.031	—	—	—	—	—
	Q05H2	5	—	—	—	—	.026	—	—	—	—	—
	Q06H2	6	—	—	—	—	.015	—	—	—	—	—
	Q07H2	7	—	—	—	—	—	—	—	—	—	—
	Q08H2	8	—	—	—	—	—	—	—	—	—	—
	Q09H2	9	—	—	—	—	—	—	—	—	—	—
	Q10H2	10	—	—	—	—	—	—	—	—	—	—
	Q11H2	11	—	—	—	—	—	—	—	—	—	—
	Q12H2	12	—	—	—	—	—	—	—	—	—	—
Holliston well 4	Q01H4	1	—	—	0.300	0.300	—	0.300	0.300	—	—	—
	Q02H4	2	—	—	.141	.141	—	.141	.141	—	—	—
	Q03H4	3	—	—	.140	.140	—	.140	.140	—	—	—
	Q04H4	4	—	—	.092	.092	—	.092	.092	—	—	—
	Q05H4	5	—	—	.072	.072	—	.072	.072	—	—	—
	Q06H4	6	—	—	.066	.066	—	.066	.066	—	—	—
	Q07H4	7	—	—	.042	.042	—	.042	.042	—	—	—
	Q08H4	8	—	—	.030	.030	—	.030	.030	—	—	—
	Q09H4	9	—	—	.023	.023	—	.023	.023	—	—	—
	Q10H4	10	—	—	.022	.022	—	.022	.022	—	—	—
	Q11H4	11	—	—	.016	.016	—	.016	.016	—	—	—
	Q12H4	12	—	—	.016	.016	—	.016	.016	—	—	—
Holliston well 5	Q01H5	1	—	—	—	—	.221	—	—	—	—	—
	Q02H5	2	—	—	—	—	.193	—	—	—	—	—
	Q03H5	3	—	—	—	—	.149	—	—	—	—	—
	Q04H5	4	—	—	—	—	.113	—	—	—	—	—
	Q05H5	5	—	—	—	—	.084	—	—	—	—	—
	Q06H5	6	—	—	—	—	.069	—	—	—	—	—
	Q07H5	7	—	—	—	—	.045	—	—	—	—	—
	Q08H5	8	—	—	—	—	.038	—	—	—	—	—
	Q09H5	9	—	—	—	—	.029	—	—	—	—	—
	Q10H5	10	—	—	—	—	.022	—	—	—	—	—
	Q11H5	11	—	—	—	—	.013	—	—	—	—	—
	Q12H5	12	—	—	—	—	.012	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Holliston well 6	Q01H6	1	—	—	—	—	0.323	—	—	—	—	—
	Q02H6	2	—	—	—	—	.324	—	—	—	—	—
	Q03H6	3	—	—	—	—	.177	—	—	—	—	—
	Q04H6	4	—	—	—	—	.098	—	—	—	—	—
	Q05H6	5	—	—	—	—	.041	—	—	—	—	—
	Q06H6	6	—	—	—	—	.024	—	—	—	—	—
	Q07H6	7	—	—	—	—	.013	—	—	—	—	—
	Q08H6	8	—	—	—	—	—	—	—	—	—	—
	Q09H6	9	—	—	—	—	—	—	—	—	—	—
	Q10H6	10	—	—	—	—	—	—	—	—	—	—
	Q11H6	11	—	—	—	—	—	—	—	—	—	—
	Q12H6	12	—	—	—	—	—	—	—	—	—	—
Franklin hypothetical recharge basin 200 ft from Miscoe Brook	W01M4	1	—	—	-0.319	-0.319	—	—	—	-0.319	-0.319	—
	W02M4	2	—	—	-.163	-.163	—	—	—	-.163	-.163	—
	W03M4	3	—	—	-.119	-.119	—	—	—	-.119	-.119	—
	W04M4	4	—	—	-.097	-.097	—	—	—	-.097	-.097	—
	W05M4	5	—	—	-.080	-.080	—	—	—	-.080	-.080	—
	W06M4	6	—	—	-.064	-.064	—	—	—	-.064	-.064	—
	W07M4	7	—	—	-.056	-.056	—	—	—	-.056	-.056	—
	W08M4	8	—	—	-.044	-.044	—	—	—	-.044	-.044	—
	W09M4	9	—	—	-.030	-.030	—	—	—	-.030	-.030	—
	W10M4	10	—	—	-.014	-.014	—	—	—	-.014	-.014	—
	W11M4	11	—	—	-.014	-.014	—	—	—	-.014	-.014	—
	W12M4	12	—	—	--	--	—	—	—	--	--	—
Franklin hypothetical recharge basin 2,608 ft from Miscoe Brook	W01M1	1	—	—	-.090	-.090	—	—	—	-.090	-.090	—
	W02M1	2	—	—	-.144	-.144	—	—	—	-.144	-.144	—
	W03M1	3	—	—	-.134	-.134	—	—	—	-.134	-.134	—
	W04M1	4	—	—	-.125	-.125	—	—	—	-.125	-.125	—
	W05M1	5	—	—	-.113	-.113	—	—	—	-.113	-.113	—
	W06M1	6	—	—	-.099	-.099	—	—	—	-.099	-.099	—
	W07M1	7	—	—	-.088	-.088	—	—	—	-.088	-.088	—
	W08M1	8	—	—	-.071	-.071	—	—	—	-.071	-.071	—
	W09M1	9	—	—	-.059	-.059	—	—	—	-.059	-.059	—
	W10M1	10	—	—	-.035	-.035	—	—	—	-.035	-.035	—
	W11M1	11	—	—	-.023	-.023	—	—	—	-.023	-.023	—
	W12M1	12	—	—	-.020	-.020	—	—	—	-.020	-.020	—
Franklin hypothetical recharge basin 1,720 ft from Miscoe Brook	W01M2	1	—	—	-.115	-.115	—	—	—	-.115	-.115	—
	W02M2	2	—	—	-.148	-.148	—	—	—	-.148	-.148	—
	W03M2	3	—	—	-.137	-.137	—	—	—	-.137	-.137	—
	W04M2	4	—	—	-.123	-.123	—	—	—	-.123	-.123	—
	W05M2	5	—	—	-.108	-.108	—	—	—	-.108	-.108	—
	W06M2	6	—	—	-.095	-.095	—	—	—	-.095	-.095	—
	W07M2	7	—	—	-.082	-.082	—	—	—	-.082	-.082	—
	W08M2	8	—	—	-.068	-.068	—	—	—	-.068	-.068	—
	W09M2	9	—	—	-.052	-.052	—	—	—	-.052	-.052	—
	W10M2	10	—	—	-.032	-.032	—	—	—	-.032	-.032	—
	W11M2	11	—	—	-.023	-.023	—	—	—	-.023	-.023	—
	W12M2	12	—	—	-.017	-.017	—	—	—	-.017	-.017	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Franklin hypothetical recharge basin 894 ft from Miscoe Brook	W01M3	1	—	—	-0.214	-0.214	—	—	—	-0.214	-0.214	—
	W02M3	2	—	—	-.164	-.164	—	—	—	-.164	-.164	—
	W03M3	3	—	—	-.131	-.131	—	—	—	-.131	-.131	—
	W04M3	4	—	—	-.110	-.110	—	—	—	-.110	-.110	—
	W05M3	5	—	—	-.095	-.095	—	—	—	-.095	-.095	—
	W06M3	6	—	—	-.075	-.075	—	—	—	-.075	-.075	—
	W07M3	7	—	—	-.068	-.068	—	—	—	-.068	-.068	—
	W08M3	8	—	—	-.054	-.054	—	—	—	-.054	-.054	—
	W09M3	9	—	—	-.043	-.043	—	—	—	-.043	-.043	—
	W10M3	10	—	—	-.020	-.020	—	—	—	-.020	-.020	—
	W11M3	11	—	—	-.014	-.014	—	—	—	-.014	-.014	—
	W12M3	12	—	—	-.011	-.011	—	—	—	-.011	-.011	—
Medway proposed well	Q01DP	1	—	—	—	—	0.406	—	—	—	—	—
	Q02DP	2	—	—	—	—	.247	—	—	—	—	—
	Q03DP	3	—	—	—	—	.124	—	—	—	—	—
	Q04DP	4	—	—	—	—	.071	—	—	—	—	—
	Q05DP	5	—	—	—	—	.046	—	—	—	—	—
	Q06DP	6	—	—	—	—	.043	—	—	—	—	—
	Q07DP	7	—	—	—	—	.020	—	—	—	—	—
	Q08DP	8	—	—	—	—	.017	—	—	—	—	—
	Q09DP	9	—	—	—	—	.015	—	—	—	—	—
	Q10DP	10	—	—	—	—	.010	—	—	—	—	—
	Q11DP	11	—	—	—	—	—	—	—	—	—	—
	Q12DP	12	—	—	—	—	—	—	—	—	—	—
Medway well 1	Q01D1	1	—	—	.010	.801	—	—	—	—	—	—
	Q02D1	2	—	—	—	.132	—	—	—	—	—	—
	Q03D1	3	—	—	—	.045	—	—	—	—	—	—
	Q04D1	4	—	—	—	.022	—	—	—	—	—	—
	Q05D1	5	—	—	—	—	—	—	—	—	—	—
	Q06D1	6	—	—	—	—	—	—	—	—	—	—
	Q07D1	7	—	—	—	—	—	—	—	—	—	—
	Q08D1	8	—	—	—	—	—	—	—	—	—	—
	Q09D1	9	—	—	—	—	—	—	—	—	—	—
	Q10D1	10	—	—	—	—	—	—	—	—	—	—
	Q11D1	11	—	—	—	—	—	—	—	—	—	—
	Q12D1	12	—	—	—	—	—	—	—	—	—	—
Medway well 2	Q01D2	1	—	—	.000	.050	.024	—	—	—	—	—
	Q02D2	2	—	—	.010	.107	.040	—	—	—	—	—
	Q03D2	3	—	—	—	.094	.070	—	—	—	—	—
	Q04D2	4	—	—	—	.093	.065	—	—	—	—	—
	Q05D2	5	—	—	—	.065	.061	—	—	—	—	—
	Q06D2	6	—	—	—	.053	.049	—	—	—	—	—
	Q07D2	7	—	—	—	.026	.041	—	—	—	—	—
	Q08D2	8	—	—	—	.026	.036	—	—	—	—	—
	Q09D2	9	—	—	—	.013	.032	—	—	—	—	—
	Q10D2	10	—	—	—	—	.028	—	—	—	—	—
	Q11D2	11	—	—	—	—	.016	—	—	—	—	—
	Q12D2	12	—	—	—	—	.012	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Medway well 3	Q01D3	1	—	—	—	0.782	—	—	—	—	—	—
	Q02D3	2	—	—	—	.156	—	—	—	—	—	—
	Q03D3	3	—	—	—	.044	—	—	—	—	—	—
	Q04D3	4	—	—	—	.018	—	—	—	—	—	—
	Q05D3	5	—	—	—	—	—	—	—	—	—	—
	Q06D3	6	—	—	—	—	—	—	—	—	—	—
	Q07D3	7	—	—	—	—	—	—	—	—	—	—
	Q08D3	8	—	—	—	—	—	—	—	—	—	—
	Q09D3	9	—	—	—	—	—	—	—	—	—	—
	Q10D3	10	—	—	—	—	—	—	—	—	—	—
	Q11D3	11	—	—	—	—	—	—	—	—	—	—
	Q12D3	12	—	—	—	—	—	—	—	—	—	—
Milford Clark Island wellfield	Q01M6	1	0.303	0.303	0.303	.303	—	—	—	—	—	—
	Q02M6	2	.198	.198	.198	.198	—	—	—	—	—	—
	Q03M6	3	.154	.154	.154	.154	—	—	—	—	—	—
	Q04M6	4	.136	.136	.136	.136	—	—	—	—	—	—
	Q05M6	5	.121	.121	.121	.121	—	—	—	—	—	—
	Q06M6	6	.065	.065	.065	.065	—	—	—	—	—	—
	Q07M6	7	.023	.023	.023	.023	—	—	—	—	—	—
	Q08M6	8	—	—	—	—	—	—	—	—	—	—
	Q09M6	9	—	—	—	—	—	—	—	—	—	—
	Q10M6	10	—	—	—	—	—	—	—	—	—	—
	Q11M6	11	—	—	—	—	—	—	—	—	—	—
	Q12M6	12	—	—	—	—	—	—	—	—	—	—
Milford Dilla Street well 1	Q01M1	1	.240	.240	.240	.240	—	—	—	—	—	—
	Q02M1	2	.199	.199	.199	.199	—	—	—	—	—	—
	Q03M1	3	.164	.164	.164	.164	—	—	—	—	—	—
	Q04M1	4	.163	.163	.163	.163	—	—	—	—	—	—
	Q05M1	5	.135	.135	.135	.135	—	—	—	—	—	—
	Q06M1	6	.073	.073	.073	.073	—	—	—	—	—	—
	Q07M1	7	.026	.026	.026	.026	—	—	—	—	—	—
	Q08M1	8	—	—	—	—	—	—	—	—	—	—
	Q09M1	9	—	—	—	—	—	—	—	—	—	—
	Q10M1	10	—	—	—	—	—	—	—	—	—	—
	Q11M1	11	—	—	—	—	—	—	—	—	—	—
	Q12M1	12	—	—	—	—	—	—	—	—	—	—
Milford Dilla Street well 2	Q01M2	1	.311	.311	.311	.311	—	—	—	—	—	—
	Q02M2	2	.201	.201	.201	.201	—	—	—	—	—	—
	Q03M2	3	.146	.146	.146	.146	—	—	—	—	—	—
	Q04M2	4	.136	.136	.136	.136	—	—	—	—	—	—
	Q05M2	5	.119	.119	.119	.119	—	—	—	—	—	—
	Q06M2	6	.064	.064	.064	.064	—	—	—	—	—	—
	Q07M2	7	.023	.023	.023	.023	—	—	—	—	—	—
	Q08M2	8	—	—	—	—	—	—	—	—	—	—
	Q09M2	9	—	—	—	—	—	—	—	—	—	—
	Q10M2	10	—	—	—	—	—	—	—	—	—	—
	Q11M2	11	—	—	—	—	—	—	—	—	—	—
	Q12M2	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Milford Godfrey Brook well 5	Q01M5	1	0.813	0.813	0.813	0.813	—	—	—	—	—	—
	Q02M5	2	.110	.110	.110	.110	—	—	—	—	—	—
	Q03M5	3	.048	.048	.048	.048	—	—	—	—	—	—
	Q04M5	4	.029	.029	.029	.029	—	—	—	—	—	—
	Q05M5	5	—	—	—	—	—	—	—	—	—	—
	Q06M5	6	—	—	—	—	—	—	—	—	—	—
	Q07M5	7	—	—	—	—	—	—	—	—	—	—
	Q08M5	8	—	—	—	—	—	—	—	—	—	—
	Q09M5	9	—	—	—	—	—	—	—	—	—	—
	Q10M5	10	—	—	—	—	—	—	—	—	—	—
	Q11M5	11	—	—	—	—	—	—	—	—	—	—
	Q12M5	12	—	—	—	—	—	—	—	—	—	—
Milford Godfrey Brook well 4	Q01M4	1	.829	.829	.829	.829	—	—	—	—	—	—
	Q02M4	2	.096	.096	.096	.096	—	—	—	—	—	—
	Q03M4	3	.052	.052	.052	.052	—	—	—	—	—	—
	Q04M4	4	.022	.022	.022	.022	—	—	—	—	—	—
	Q05M4	5	—	—	—	—	—	—	—	—	—	—
	Q06M4	6	—	—	—	—	—	—	—	—	—	—
	Q07M4	7	—	—	—	—	—	—	—	—	—	—
	Q08M4	8	—	—	—	—	—	—	—	—	—	—
	Q09M4	9	—	—	—	—	—	—	—	—	—	—
	Q10M4	10	—	—	—	—	—	—	—	—	—	—
	Q11M4	11	—	—	—	—	—	—	—	—	—	—
	Q12M4	12	—	—	—	—	—	—	—	—	—	—
Milford Godfrey Brook well 3	Q01M3	1	.820	.820	.820	.820	—	—	—	—	—	—
	Q02M3	2	.099	.099	.099	.099	—	—	—	—	—	—
	Q03M3	3	.050	.050	.050	.050	—	—	—	—	—	—
	Q04M3	4	.025	.025	.025	.025	—	—	—	—	—	—
	Q05M3	5	—	—	—	—	—	—	—	—	—	—
	Q06M3	6	—	—	—	—	—	—	—	—	—	—
	Q07M3	7	—	—	—	—	—	—	—	—	—	—
	Q08M3	8	—	—	—	—	—	—	—	—	—	—
	Q09M3	9	—	—	—	—	—	—	—	—	—	—
	Q10M3	10	—	—	—	—	—	—	—	—	—	—
	Q11M3	11	—	—	—	—	—	—	—	—	—	—
	Q12M3	12	—	—	—	—	—	—	—	—	—	—
Norfolk proposed Mill River well	Q01NP	1	—	—	—	.512	—	—	—	—	—	0.512
	Q02NP	2	—	—	—	.237	—	—	—	—	—	.237
	Q03NP	3	—	—	—	.122	—	—	—	—	—	.122
	Q04NP	4	—	—	—	.064	—	—	—	—	—	.064
	Q05NP	5	—	—	—	.033	—	—	—	—	—	.033
	Q06NP	6	—	—	—	.022	—	—	—	—	—	.022
	Q07NP	7	—	—	—	.010	—	—	—	—	—	.010
	Q08NP	8	—	—	—	—	—	—	—	—	—	—
	Q09NP	9	—	—	—	—	—	—	—	—	—	—
	Q10NP	10	—	—	—	—	—	—	—	—	—	—
	Q11NP	11	—	—	—	—	—	—	—	—	—	—
	Q12NP	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Norfolk well 1	Q01N1	1	—	—	—	0.082	—	—	—	—	—	0.046
	Q02N1	2	—	—	—	.145	—	—	—	—	—	.091
	Q03N1	3	—	—	—	.157	—	—	—	—	—	.096
	Q04N1	4	—	—	—	.161	—	—	—	—	—	.113
	Q05N1	5	—	—	—	.153	—	—	—	—	—	.117
	Q06N1	6	—	—	—	.141	—	—	—	—	—	.078
	Q07N1	7	—	—	—	.054	—	—	—	—	—	.046
	Q08N1	8	—	—	—	.042	—	—	—	—	—	.034
	Q09N1	9	—	—	—	.033	—	—	—	—	—	.027
	Q10N1	10	—	—	—	.017	—	—	—	—	—	.019
	Q11N1	11	—	—	—	.015	—	—	—	—	—	.015
	Q12N1	12	—	—	—	--	—	—	—	—	—	--
Wrentham proposed well	Q01W1	1	—	—	—	.159	—	—	—	—	—	.159
	Q02W1	2	—	—	—	.147	—	—	—	—	—	.147
	Q03W1	3	—	—	—	.128	—	—	—	—	—	.128
	Q04W1	4	—	—	—	.131	—	—	—	—	—	.131
	Q05W1	5	—	—	—	.096	—	—	—	—	—	.096
	Q06W1	6	—	—	—	.071	—	—	—	—	—	.071
	Q07W1	7	—	—	—	.059	—	—	—	—	—	.059
	Q08W1	8	—	—	—	.052	—	—	—	—	—	.052
	Q09W1	9	—	—	—	.046	—	—	—	—	—	.046
	Q10W1	10	—	—	—	.044	—	—	—	—	—	.044
	Q11W1	11	—	—	—	.035	—	—	—	—	—	.035
	Q12W1	12	—	—	—	.031	—	—	—	—	—	.031
Wrentham well 2	Q01W2	1	—	—	—	.400	—	—	—	—	—	.400
	Q02W2	2	—	—	—	.173	—	—	—	—	—	.173
	Q03W2	3	—	—	—	.103	—	—	—	—	—	.103
	Q04W2	4	—	—	—	.074	—	—	—	—	—	.074
	Q05W2	5	—	—	—	.047	—	—	—	—	—	.047
	Q06W2	6	—	—	—	.046	—	—	—	—	—	.046
	Q07W2	7	—	—	—	.035	—	—	—	—	—	.035
	Q08W2	8	—	—	—	.033	—	—	—	—	—	.033
	Q09W2	9	—	—	—	.027	—	—	—	—	—	.027
	Q10W2	10	—	—	—	.025	—	—	—	—	—	.025
	Q11W2	11	—	—	—	.019	—	—	—	—	—	.019
	Q12W2	12	—	—	—	.017	—	—	—	—	—	.017
Wrentham well 3	Q01W3	1	—	—	—	.467	—	—	—	—	—	.467
	Q02W3	2	—	—	—	.142	—	—	—	—	—	.142
	Q03W3	3	—	—	—	.089	—	—	—	—	—	.089
	Q04W3	4	—	—	—	.065	—	—	—	—	—	.065
	Q05W3	5	—	—	—	.052	—	—	—	—	—	.052
	Q06W3	6	—	—	—	.036	—	—	—	—	—	.036
	Q07W3	7	—	—	—	.030	—	—	—	—	—	.030
	Q08W3	8	—	—	—	.028	—	—	—	—	—	.028
	Q09W3	9	—	—	—	.026	—	—	—	—	—	.026
	Q10W3	10	—	—	—	.025	—	—	—	—	—	.025
	Q11W3	11	—	—	—	.023	—	—	—	—	—	.023
	Q12W3	12	—	—	—	.018	—	—	—	—	—	.018

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Bellingham proposed wastewater discharge	W01BL	1	—	-1.000	-1.000	-1.000	—	—	—	—	—	—
	W02BL	2	—	—	—	—	—	—	—	—	—	—
	W03BL	3	—	—	—	—	—	—	—	—	—	—
	W04BL	4	—	—	—	—	—	—	—	—	—	—
	W05BL	5	—	—	—	—	—	—	—	—	—	—
	W06BL	6	—	—	—	—	—	—	—	—	—	—
	W07BL	7	—	—	—	—	—	—	—	—	—	—
	W08BL	8	—	—	—	—	—	—	—	—	—	—
	W09BL	9	—	—	—	—	—	—	—	—	—	—
	W10BL	10	—	—	—	—	—	—	—	—	—	—
	W11BL	11	—	—	—	—	—	—	—	—	—	—
	W12BL	12	—	—	—	—	—	—	—	—	—	—
Charles River Pollution Control District discharge	W01CR	1	—	—	—	-1.000	—	—	—	—	—	—
	W02CR	2	—	—	—	—	—	—	—	—	—	—
	W03CR	3	—	—	—	—	—	—	—	—	—	—
	W04CR	4	—	—	—	—	—	—	—	—	—	—
	W05CR	5	—	—	—	—	—	—	—	—	—	—
	W06CR	6	—	—	—	—	—	—	—	—	—	—
	W07CR	7	—	—	—	—	—	—	—	—	—	—
	W08CR	8	—	—	—	—	—	—	—	—	—	—
	W09CR	9	—	—	—	—	—	—	—	—	—	—
	W10CR	10	—	—	—	—	—	—	—	—	—	—
	W11CR	11	—	—	—	—	—	—	—	—	—	—
	W12CR	12	—	—	—	—	—	—	—	—	—	—
Charles River Pollution Control District discharge hypothetical upstream location	W01CP	1	—	—	-1.000	-1.000	—	—	—	—	—	—
	W02CP	2	—	—	—	—	—	—	—	—	—	—
	W03CP	3	—	—	—	—	—	—	—	—	—	—
	W04CP	4	—	—	—	—	—	—	—	—	—	—
	W05CP	5	—	—	—	—	—	—	—	—	—	—
	W06CP	6	—	—	—	—	—	—	—	—	—	—
	W07CP	7	—	—	—	—	—	—	—	—	—	—
	W08CP	8	—	—	—	—	—	—	—	—	—	—
	W09CP	9	—	—	—	—	—	—	—	—	—	—
	W10CP	10	—	—	—	—	—	—	—	—	—	—
	W11CP	11	—	—	—	—	—	—	—	—	—	—
	W12CP	12	—	—	—	—	—	—	—	—	—	—
Milford wastewater treatment plant	W01MF	1	—	-1.000	-1.000	-1.000	—	—	—	—	—	—
	W02MF	2	—	—	—	—	—	—	—	—	—	—
	W03MF	3	—	—	—	—	—	—	—	—	—	—
	W04MF	4	—	—	—	—	—	—	—	—	—	—
	W05MF	5	—	—	—	—	—	—	—	—	—	—
	W06MF	6	—	—	—	—	—	—	—	—	—	—
	W07MF	7	—	—	—	—	—	—	—	—	—	—
	W08MF	8	—	—	—	—	—	—	—	—	—	—
	W09MF	9	—	—	—	—	—	—	—	—	—	—
	W10MF	10	—	—	—	—	—	—	—	—	—	—
	W11MF	11	—	—	—	—	—	—	—	—	—	—
	W12MF	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimization variable	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Miscoe Brook hypothetical wastewater discharge site	W01FM	1	—	—	-1.000	-1.000	—	—	—	-1.000	-1.000	—
	W02FM	2	—	—	—	—	—	—	—	—	—	—
	W03FM	3	—	—	—	—	—	—	—	—	—	—
	W04FM	4	—	—	—	—	—	—	—	—	—	—
	W05FM	5	—	—	—	—	—	—	—	—	—	—
	W06FM	6	—	—	—	—	—	—	—	—	—	—
	W07FM	7	—	—	—	—	—	—	—	—	—	—
	W08FM	8	—	—	—	—	—	—	—	—	—	—
	W09FM	9	—	—	—	—	—	—	—	—	—	—
	W10FM	10	—	—	—	—	—	—	—	—	—	—
	W11FM	11	—	—	—	—	—	—	—	—	—	—
	W12FM	12	—	—	—	—	—	—	—	—	—	—
Drinking water direct discharge to Charles River	D01CR	1	-1.000	-1.000	-1.000	-1.000	—	—	—	—	—	—
	D02CR	2	—	—	—	—	—	—	—	—	—	—
	D03CR	3	—	—	—	—	—	—	—	—	—	—
	D04CR	4	—	—	—	—	—	—	—	—	—	—
	D05CR	5	—	—	—	—	—	—	—	—	—	—
	D06CR	6	—	—	—	—	—	—	—	—	—	—
	D07CR	7	—	—	—	—	—	—	—	—	—	—
	D08CR	8	—	—	—	—	—	—	—	—	—	—
	D09CR	9	—	—	—	—	—	—	—	—	—	—
	D10CR	10	—	—	—	—	—	—	—	—	—	—
	D11CR	11	—	—	—	—	—	—	—	—	—	—
	D12CR	12	—	—	—	—	—	—	—	—	—	—
Drinking water direct discharge to Hopping Brook	D01HB	1	—	—	-1.000	-1.000	—	-1.000	-1.000	—	—	—
	D02HB	2	—	—	—	—	—	—	—	—	—	—
	D03HB	3	—	—	—	—	—	—	—	—	—	—
	D04HB	4	—	—	—	—	—	—	—	—	—	—
	D05HB	5	—	—	—	—	—	—	—	—	—	—
	D06HB	6	—	—	—	—	—	—	—	—	—	—
	D07HB	7	—	—	—	—	—	—	—	—	—	—
	D08HB	8	—	—	—	—	—	—	—	—	—	—
	D09HB	9	—	—	—	—	—	—	—	—	—	—
	D10HB	10	—	—	—	—	—	—	—	—	—	—
	D11HB	11	—	—	—	—	—	—	—	—	—	—
	D12HB	12	—	—	—	—	—	—	—	—	—	—
Drinking water direct discharge to Miscoe Brook	D01MB	1	—	—	-1.000	-1.000	—	—	—	-1.000	-1.000	—
	D02MB	2	—	—	—	—	—	—	—	—	—	—
	D03MB	3	—	—	—	—	—	—	—	—	—	—
	D04MB	4	—	—	—	—	—	—	—	—	—	—
	D05MB	5	—	—	—	—	—	—	—	—	—	—
	D06MB	6	—	—	—	—	—	—	—	—	—	—
	D07MB	7	—	—	—	—	—	—	—	—	—	—
	D08MB	8	—	—	—	—	—	—	—	—	—	—
	D09MB	9	—	—	—	—	—	—	—	—	—	—
	D10MB	10	—	—	—	—	—	—	—	—	—	—
	D11MB	11	—	—	—	—	—	—	—	—	—	—
	D12MB	12	—	—	—	—	—	—	—	—	—	—

Appendix 2. Hydrologic response coefficients for withdrawal and return stresses analyzed, upper Charles River basin, eastern Massachusetts.—Continued

[See tables 5 and 7 for more information on stress variables. ft, foot; — zero]

Stress causing streamflow change	Optimi-zation vari-able	Month	Hydrologic response coefficients for constraint-point locations									
			CR1	CR2	CR3	CR4	BB1	HB1	HB2	MB1	MB2	MR1
Charles River Pollution Control District facility hypothetical recharge	W01C1	1	—	—	—	-0.083	—	—	—	—	—	—
	W02C1	2	—	—	—	-.083	—	—	—	—	—	—
	W03C1	3	—	—	—	-.083	—	—	—	—	—	—
	W04C1	4	—	—	—	-.083	—	—	—	—	—	—
	W05C1	5	—	—	—	-.083	—	—	—	—	—	—
	W06C1	6	—	—	—	-.083	—	—	—	—	—	—
	W07C1	7	—	—	—	-.083	—	—	—	—	—	—
	W08C1	8	—	—	—	-.083	—	—	—	—	—	—
	W09C1	9	—	—	—	-.083	—	—	—	—	—	—
	W10C1	10	—	—	—	-.083	—	—	—	—	—	—
	W11C1	11	—	—	—	-.083	—	—	—	—	—	—
	W12C1	12	—	—	—	-.083	—	—	—	—	—	—