Cover photo:
Crew races on the lower Charles River looking upstream toward the Larz Anderson Bridge.

Courtesy of: Philip Greenspun
http://philip.greenspun.com

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

Measured and Simulated Runoff to the Lower Charles River, Massachusetts, October 1999–September 2000

By PHILLIP J. ZARRIELLO and LORA K. BARLOW

Water-Resources Investigations Report 02-4129

In cooperation with the
U.S. ENVIRONMENTAL PROTECTION AGENCY,
MASSACHUSETTS DEPARTMENT OF
ENVIRONMENTAL PROTECTION, and the
MASSACHUSETTS WATER RESOURCES AUTHORITY

Northborough, Massachusetts
2002

U.S. DEPARTMENT OF THE INTERIOR
GALE A. NORTON, Secretary

U.S. GEOLOGICAL SURVEY
Charles G. Groat, Director

For additional information write to:

Chief, Massachusetts-Rhode Island District
U.S. Geological Survey
10 Bearfoot Road
Northborough, MA 01532

or visit our Web site at
http://ma.water.usgs.gov

Copies of this report can be purchased from:

U.S. Geological Survey
Branch of Information Services
Box 25286
Denver, CO 80225-0286

CONTENTS

FIGURES

TABLES

CONVERSION FACTORS AND VERTICAL DATUM

CONVERSION FACTORS

Multiply	By	To obtain
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
foot (ft)	0.3048	meter
inch (in)	2.54	centimeter
inch (in)	25.4	millimeter
inch per hour (in/h)	0.0254	meter per hour
mile (mi)	1.609	kilometer
square mile (mi^2)	259.0	hectare
square mile (mi^2)	2.590	square kilometer

Temperature in degrees Celsius (°C) may be converted to degrees Fahrenheit (°F) as follows:
°F=1.8 °C+32

VERTICAL DATUM

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

Measured and Simulated Runoff to the Lower Charles River, Massachusetts, October 1999–September 2000

By Phillip J. Zarriello *and* Lora K. Barlow

Abstract

The lower Charles River, the water body between the Watertown Dam and the New Charles River Dam, is an important recreational resource for the Boston, Massachusetts, metropolitan area, but impaired water quality has affected its use. The goal of making this resource fishable and swimmable requires a better understanding of combined-sewer-overflow discharges, non-combined-sewer-overflow stormwater runoff, and constituent loads. This report documents the modeling effort used to calculate non-combined-sewer-overflow runoff to the lower Charles River.

During the 2000 water year, October 1, 1999–September 30, 2000, the U.S. Geological Survey collected precipitation data at Watertown Dam and compiled data from five other precipitation gages in or near the watershed. In addition, surface-water discharge data were collected at eight sites—three relatively homogenous land-use sites, four major tributary sites, and the Charles River at Watertown Dam, which is the divide between the upper and lower watersheds. The precipitation and discharge data were used to run and calibrate Stormwater Management Models developed for the three land-use subbasins (single-family, multi-family, and commercial), and the two tributary subbasins (Laundry and Faneuil Brooks). These calibrated models were used to develop a sixth model to simulate 54 ungaged outfalls to the lower Charles River. Models developed by the U.S. Geological Survey at gaged sites were calibrated with up to 24 storms. Each model was evaluated by comparing simulated discharge against measured discharge for all storms with appreciable precipitation and reliable discharge data. The model-fit statistics indicated that the models generally were well calibrated to peak discharge and runoff volumes. The model fit of the commercial land-use subbasin was not as well calibrated compared to the other models because the measured flows appear to be affected by variable conditions not represented in the model. A separate Stormwater Management Model of the Stony Brook Subbasin previously developed by others was evaluated with the newly collected data from this study; this model had a model fit comparable to the models developed by the U.S. Geological Survey.

The total annual runoff to the lower Charles River during the 2000 water year, not including contributions from combined-sewer-overflows except from the Stony Brook Subbasin, was 16,500 million cubic feet; 92 percent of the inflow was from the Charles River above Watertown Dam, 3 percent was from the Stony Brook Subbasin, 2 percent was from the Muddy River Subbasin, and less than 1 percent was from the combined inflows of Laundry and Faneuil Brooks. The remaining ungaged drainage area contributed about 2 percent of the total annual inflow to the lower Charles River. Excluding discharge from the Charles River above Watertown Dam, total annual runoff to the lower Charles River was 1,240 million cubic feet; 39 percent was from the Stony Brook Subbasin, 27 percent was from the Muddy River, which includes runoff that drains to the Muddy River conduit, 7 percent was from the Laundry Brook Subbasin, and 4 percent was from the Faneuil Brook Subbasin. Flow from the

ungaged areas composed about 23 percent of the total annual inflow to the lower Charles River, excluding discharge from the Charles River above Watertown Dam.

Runoff to the lower Charles River was calculated for two design storms representing a 3-month and a 1-year event, 1.84 and 2.79 inches of total rainfall, respectively. These simulated discharges were provided to the Massachusetts Water Resources Authority for use in a receiving-water model of the lower Charles River. Total storm runoff to the lower Charles River was 111 and 257 million cubic feet for the 3-month and 1-year storms, respectively. Excluding discharge from the Charles River above Watertown Dam, total runoff to the lower Charles River was 30 and 53 million cubic feet for the 3-month and 1-year storms, respectively. Runoff from the various tributary areas for the design storms was about in the same proportion as that for the annual runoff.

INTRODUCTION

The lower Charles River, which flows between the Watertown Dam and the New Charles River Dam, has long been recognized as an asset to the surrounding metropolitan area of Boston, Massachusetts, but impaired water quality has adversely affected this resource. As early as 1859, parts of the lower Charles River were described as "practically a cesspool" (Pritchett and others, 1903), and discussions began regarding construction of a dam to flood the tidal mud flats. In 1870, the Metropolitan Parks Commission, predecessor to the Metropolitan District Commission (MDC), was created to improve conditions in the lower Charles River. Nathan Matthews, a former Mayor of Boston, recommended creating a water park and appointed the Charles River Improvement Commission to develop this concept. In 1892, Charles Eliot, a prominent landscape architect, transformed the concept of a water park into a grander vision of a metropolitan-park system centered on the Charles River waterfront area.

Much of the land surrounding the lower Charles River was acquired in the late 1800s as part of the metropolitan-park system. In 1901, engineering studies were undertaken to investigate the extent of sewage contamination and to assess the feasibility and costs of constructing a dam to mitigate the noxious odors and adverse health effects associated with the polluted tidal flats (Pritchett and others, 1903). The Old Charles River Dam, constructed by 1908, resolved many of these problems and created a recreational resource for the Boston metropolitan area (Jobin, 1998).

The lower Charles River continues to serve as a recreational resource for thousands of boaters and many thousands more who enjoy the parks along its embankments. The largest 1-day rowing regatta in the world, the Head of the Charles, brings over 5,400 rowers and 300,000 spectators to the lower Charles River each fall. In addition, annual Fourth of July festivities attract more than 500,000 people (Massachusetts District Commission, 2000). Unfortunately, Charles River water can pose a health risk at times because of excessive levels of fecal bacteria, which can exceed Massachusetts water-quality standards, even for secondary-contact recreation (such as boating).

Just as citizens and organizations in the past have identified and resolved environmental hazards associated with the lower Charles River, work to resolve water-quality issues that affect its recreational use continues today. The U.S. Environmental Protection Agency (USEPA) has led the effort to improve water quality by setting a goal of making this resource safe for fishing and swimming by Earth Day 2005. To meet this goal, accurate assessments of inflows and constituent loads from combined-sewer overflows (CSOs) and non-CSO stormwater are needed to develop sound management plans.

The Massachusetts Water Resources Authority (MWRA) has monitored and worked toward the elimination or treatment of CSO discharges to the lower Charles River for more than a decade. These efforts have substantially improved water quality, but a better understanding of non-CSO stormwater loads would enhance efforts to improve water quality. In 1999, the USGS, in cooperation with the Massachusetts Water Resources Authority (MWRA), the USEPA, and the Massachusetts Department of Environmental Protection (MADEP), initiated a study to quantify non-CSO stormwater runoff loads to the lower Charles River. This cooperative study was designed to provide information to regulatory agencies and others on how stormwater affects water quality in the lower Charles River.

Purpose and Scope

This report describes the five StormWater Management Models (SWMM) developed to simulate runoff to the lower Charles River, Massachusetts. The modeling effort described supports a companion study conducted by the USGS to calculate non-CSO contaminant loads to the lower Charles River (Breault and others, 2002). Runoff models were used to (1) simulate discharge at ungaged sites during the 2000 water year, (2) simulate discharge at gaged sites where data are missing or suspect during the 2000 water year, (3) simulate discharge at all outfalls for the July 15–18 and the July 26–31, 2000, storms for the MWRA and (4) simulate discharge at all outfalls for two design storms. Data and results obtained from this study and the companion USGS loads study will be used by the MWRA, the USEPA, and the MADEP to address the transport and fate of contaminants within the river.

This report describes hydrologic and meteorologic data collected at the three small relatively homogeneous land-use subbasins (single-family, multi-family, and commercial), four tributary subbasins (Laundry Brook, Faneuil Brook, Muddy River, and Stony Brook), and the Charles River at Watertown Dam; the development and calibration of the SWMM at the three land-use subbasins and at two tributary subbasins (Laundry and Faneuil Brook); and the model fit at these sites. The report also describes the model (SWMM) fit to the newly collected discharge data for the Stony Brook Subbasin previously developed by Metcalf and Eddy, Inc., consultants to the MWRA.

Description of the Lower Charles River and Its Watershed

The lower Charles River is the 9-mi long water body between the Watertown Dam and the New Charles River Dam in Boston where it empties into Boston Harbor (fig. 1). This part of the river is known locally as the “basin.” The drainage area to the lower Charles River is located within Middlesex, Norfolk, and Suffolk Counties, Massachusetts. Native Americans aptly named its sinuous course “Quinobequin” for “Meandering River” (Brickford and Dymon, 1990). The drainage area to the lower river below Watertown Dam is estimated as 36.6 mi^2, but drainage divides are complicated by a highly altered, man-made drainage system. The lower Charles River Watershed is one of the oldest urban areas in the Nation; its natural drainage system has been modified for more than three centuries. Municipalities within the lower Charles River Watershed include parts of Cambridge, Boston, Brookline, Newton, and Watertown. The modeled area covers the lower Charles River Watershed, but does not include CSO drainage areas except for those in the Stony Brook Subbasin. The Charles River Watershed above the Watertown Dam drains an area of 268 mi^2; runoff from this area was measured directly or estimated from discharge records from the upstream gaging station at Waltham.

Climate

The climate of eastern Massachusetts is characterized as humid temperate (Gadoury and Wandle, 1986). Precipitation generally is distributed evenly throughout the year, although variations can be large from month to month (fig. 2) and from year to year (Trombley, 1991). Precipitation at Logan Airport in Boston averaged 41.8 in. for the 1920–2000 period. Precipitation at six gages in and near the lower Charles River Watershed averaged 42.8 in. for water year 2000, but total precipitation at each of the gages varied from 50.7 to 40.1 in. Precipitation during the 2000 water year was above the long-term average during the months of April and June (1.02) because of a single large storm in each of these months; precipitation also was above normal during July because of a greater than usual number of small storms. Precipitation from November 1999 through February 2000, and during August 2000 were below the long-term average.

Air temperature reported for Boston by the Northeast Regional Climate Center for the 80-year period 1920–2000 ranged from a low of 29.3°F in January to a high of 73.2°F in July and averaged 51.2°F annually. During the 2000 water year, the mean monthly air temperature reported at Logan Airport in Boston was similar to the long-term monthly mean air temperature; from a low of 27.5°F in January to a high of 70.3°F in August, it averaged 51.6°F in the 2000 water year. The Boston area receives an average of 42.4 in. of snow each year; the 24.9 in. of snowfall during the 2000 water year was considerably below average.

Figure 1. Principal geographic features, stream-gage stations, and precipitation stations, lower Charles River Watershed, Massachusetts.

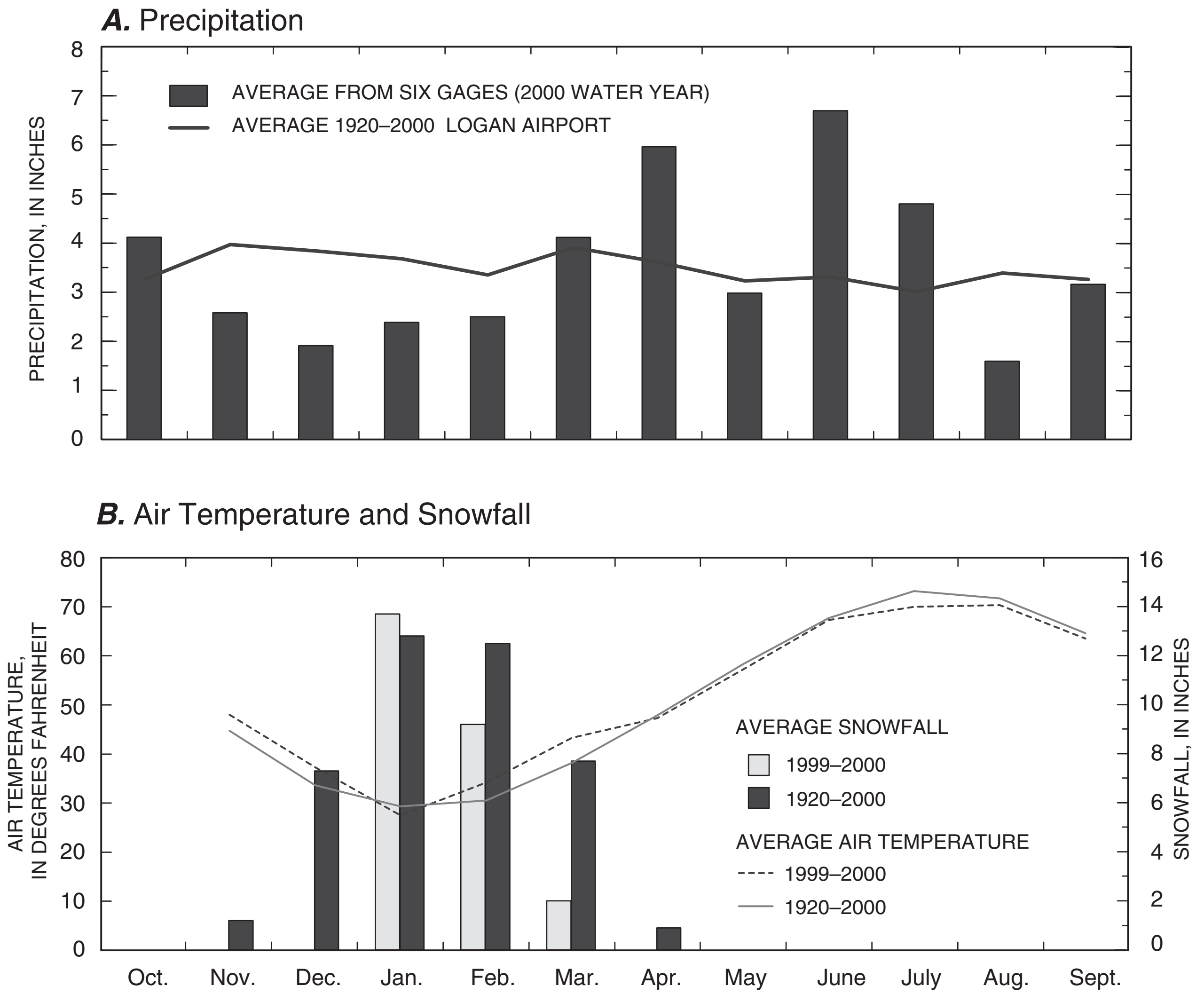


Figure 2. Climate characteristics in the lower Charles River Watershed, (*A*) total monthly precipitation averaged from six gages in and near the watershed during the 2000 water year and the average total monthly precipitation from January 1, 1920, to December 31, 2000, at Logan Airport in Boston, Massachusetts, and (*B*) average monthly air temperature and snowfall during the 2000 water year and for the 1920–2000 period at Logan Airport. [Locations are shown in fig. 1.]

Land Use

The lower Charles River Watershed is within a major metropolitan area and is highly urbanized. The majority of land use in the watershed (fig. 3) is represented by single-family residential (38 percent) and multifamily residential (13 percent) uses. Other land uses include urban open space, such as athletic fields, cemeteries, parks and institutional green space (12 percent), commercial (9 percent), forest (7 percent), open water (4 percent), transportation (3 percent), spectator recreation (3 percent), and industrial (2 percent). Other land-use types each compose less than 1 percent of the watershed area. Most new development in recent years can be classified as redevelopment; hence, the 1991 land-use conditions represented in figure 3 had not changed appreciably at the time of this study.

Soils

Soil surveys for Middlesex (U.S. Department of Agriculture, 1995), Norfolk and Suffolk Counties (Peragallo, 1989) indicate (fig. 4) that most soils in the watershed are derived from till (48 percent) or are disturbed urban land (35 percent). Soils classified as disturbed urban are found mostly near the river in areas filled to eliminate tidal marshes and mud flats. The remaining soil types are derived from glacial outwash (16 percent), post-glacial alluvium (1.4 percent) and aeolian sand and silt (0.4 percent).

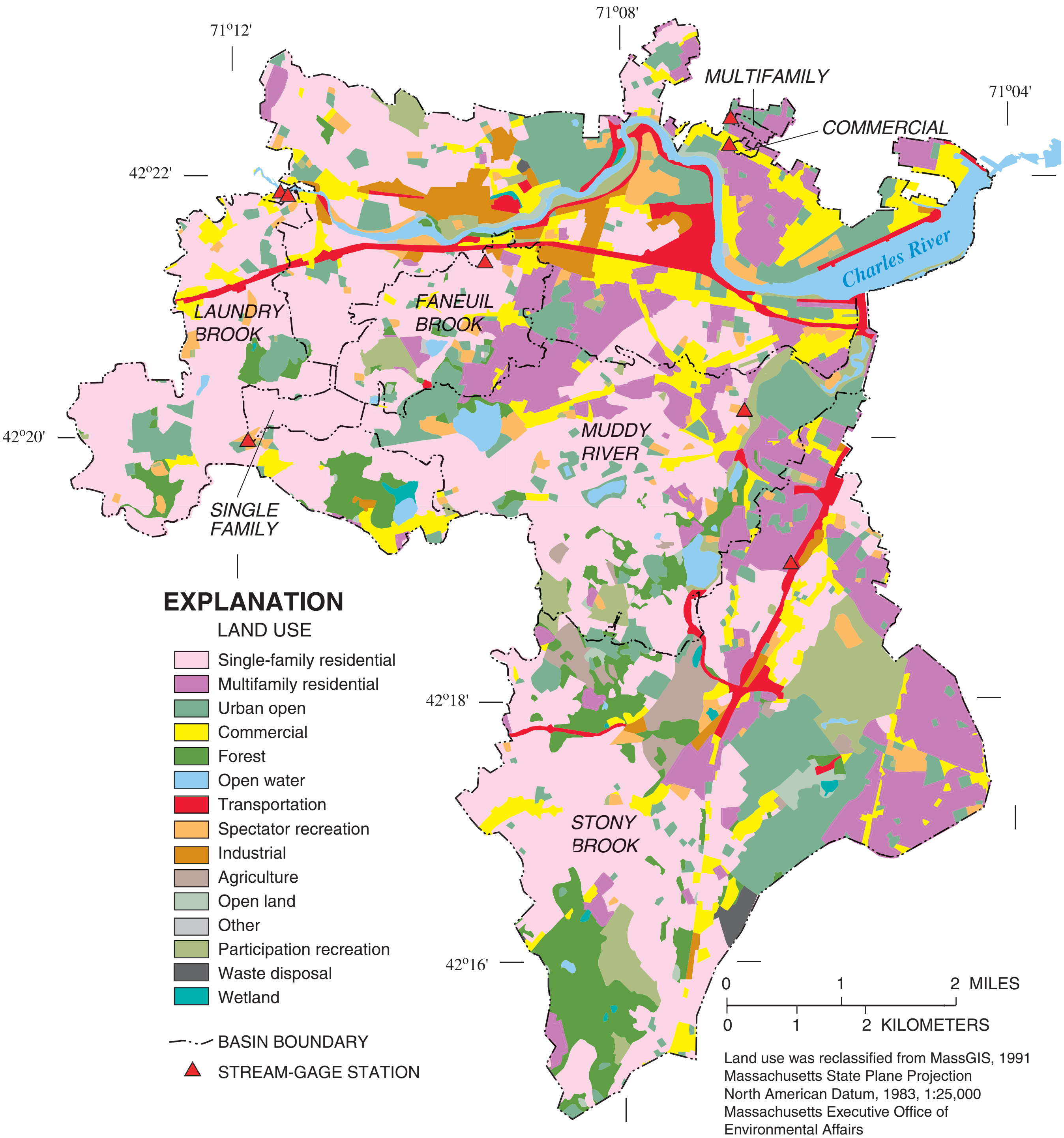


Figure 3. Generalized 1991 land use in the lower Charles River Watershed, Massachusetts.

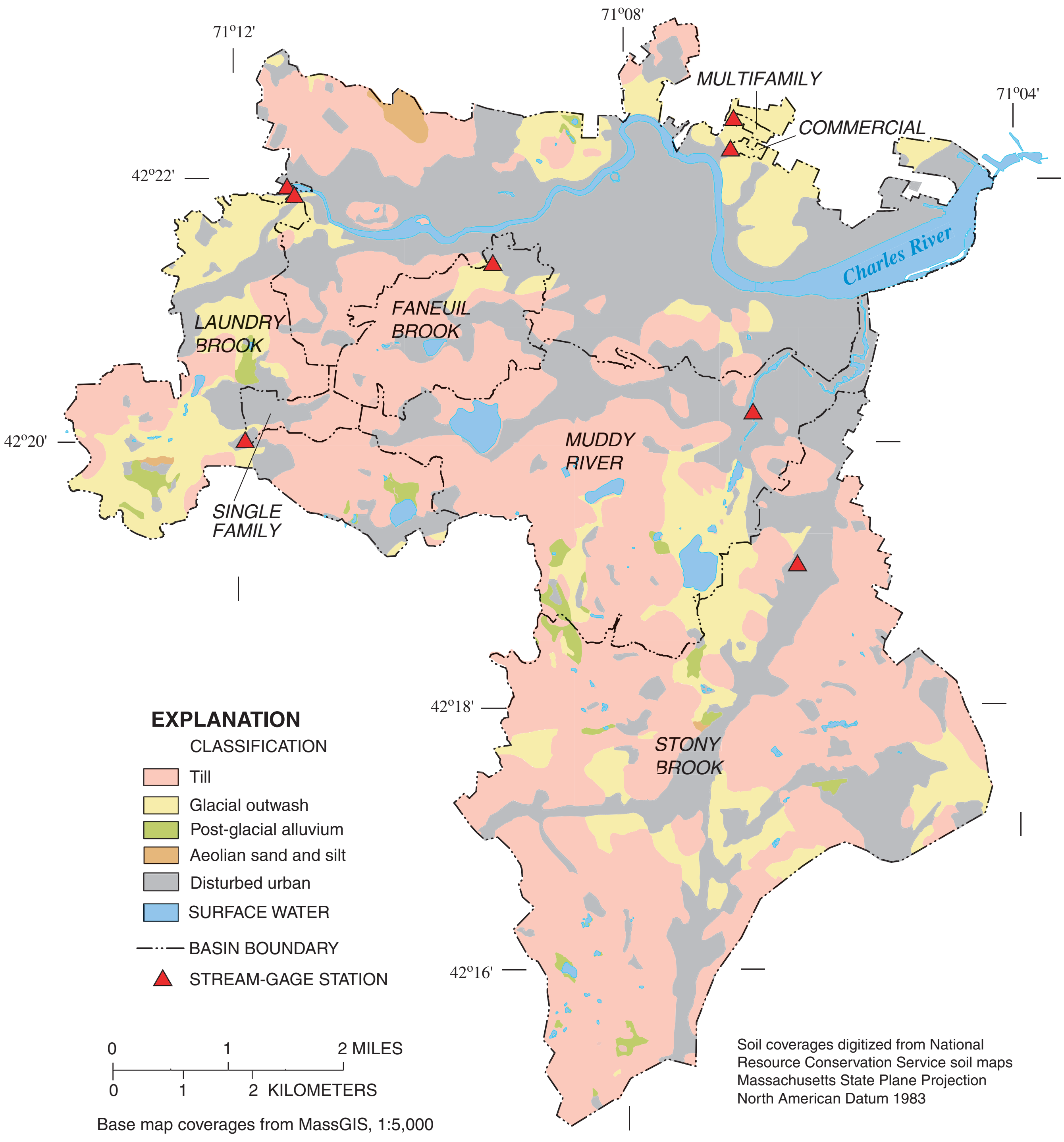


Figure 4. Generalized soil types in the lower Charles River Watershed, Massachusetts.

Soil permeability ranges from 0.6 to more than 6.0 in/h Highly permeable soils derived from well-sorted glacial outwash often exceed 2.0 in/h. Till-derived soils typically are less permeable than glacially outwash soils and range in permeability from 0.6 to 2.0 in/h. Permeability may be less in till soils that contain fragipans (U.S. Department of Agriculture, 1995; Peragallo, 1989).

Topography

Skehan (1979) describes two geomorphic districts in the watershed: the Boston Lowland, and the Needham Upland (fig. 5). Most of the watershed is in the Boston Lowland, which generally is less than 50 ft above sea level and historically comprised large areas of mudflats and tidal marsh. Tidal marshes were filled or flooded largely by 1910 when the Old Charles River Dam was completed (Seasholes, 1999). The Boston Lowland also contains small elongated hills or drumlins that range from 50 to 140 ft above sea level. The Needham Upland is in the southwestern part of the watershed and is characterized by greater relief than the Boston Lowland. Drumlins also are common in the Needham Upland and some of them exceed 300 ft above sea level (Skehan, 1979).

Hydrology

The lower Charles River Watershed has many named tributaries that appear on early maps of the area, such as the 1893 USGS 15-minute topographic map of Boston (Seasholes, 1999). The four largest tributaries to the lower Charles River, excluding the watershed above Watertown Dam (upper watershed) are Laundry Brook, Faneuil Brook, Muddy River, and Stony Brook (fig. 1). Most of the stream channels in these tributaries, with the exception of parts of the Muddy River, are enclosed in conduits. The underground storm-drain system of the watershed includes about 585 mi of conduit that varies in size, shape, and building material. The storm-drain systems were compiled into a common geographic information system (GIS) from digital engineering drawings supplied by the municipalities of Boston, Cambridge, Brookline, Newton, and Watertown.

Ninety-eight major outfalls have been identified that discharge to the lower Charles River (fig. 6); 72 outfalls are upstream of the Museum of Science. Sixteen of these outfalls are CSOs; 3 were closed and 13 were still active as of the year 2000. Discharges from the CSO outfalls are not included as part of this study, but the outfalls are shown on figure 6 for reference. Areas tributary to the CSOs of Boston and Cambridge are excluded from the study area (figs. 1 and 6); these areas are being modeled separately by the MWRA. Various small outfalls are not included in the above count; these outfalls typically drain small street or parkway areas.

Outfall identification numbers are ordered from upstream to downstream; outfall 1 is just below Watertown Dam and outfall 81 is below the Museum of Science. Areas that drain by direct sheet runoff or from small unnumbered outfalls to the lower Charles River were assigned the same outfall number as the next closest known upstream outfall with an added letter suffix. For example, three drainage areas downstream of outfall number 41 drain by sheet runoff. These drainage areas were identified as 41a, 41b, and 41c, and are labeled at the midpoint between drainage divides at the shoreline. Seventeen areas that drain by sheet runoff are identified in figure 6.

Previous Investigations

Quantification of runoff to the lower Charles River began during the engineering studies for the Old Charles River Dam in the early 1890s (Pritchett and others, 1903). Recent studies, summarized below, have focused primarily on quantifying runoff from the Muddy River and Stony Brook tributaries for the purpose of separating combined sewer areas. Measured discharge records from these studies are limited to specific events or short time periods, generally less than 2 months duration.

To assess the water quality of the Muddy River Subbasin, a model (SWMM) was developed in 1989 by Metcalf and Eddy, Inc. (M&E), for the Massachusetts Executive Office of Environmental Affairs (Metcalf and Eddy, Inc., 1989). The report describes the physical hydrologic features of the Muddy River Subbasin. Model simulations were limited to a synthetic 1-year, 6-hour design storm.

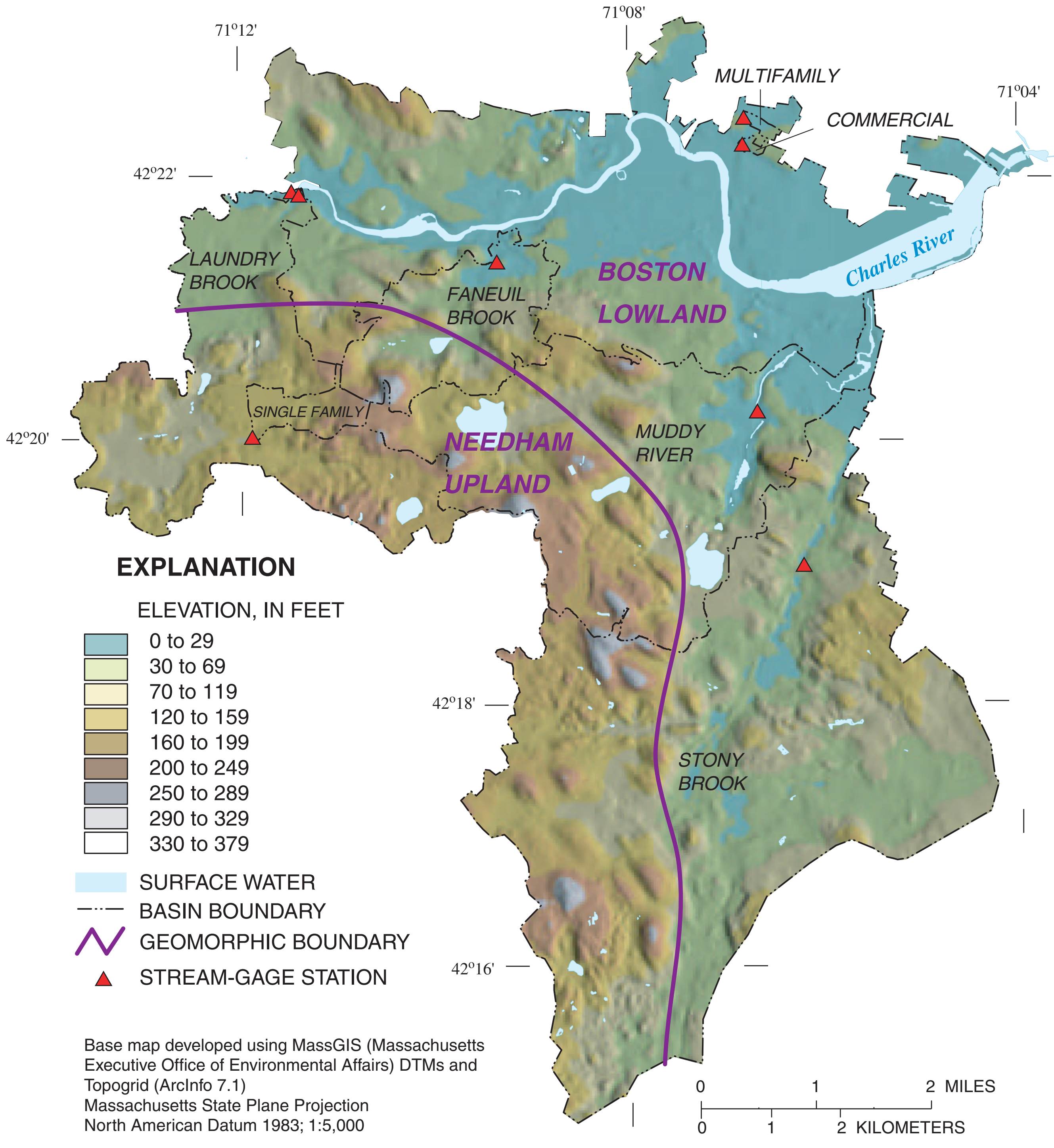


Figure 5. Geomorphic districts and land-surface elevations of the lower Charles River Watershed, Massachusetts.

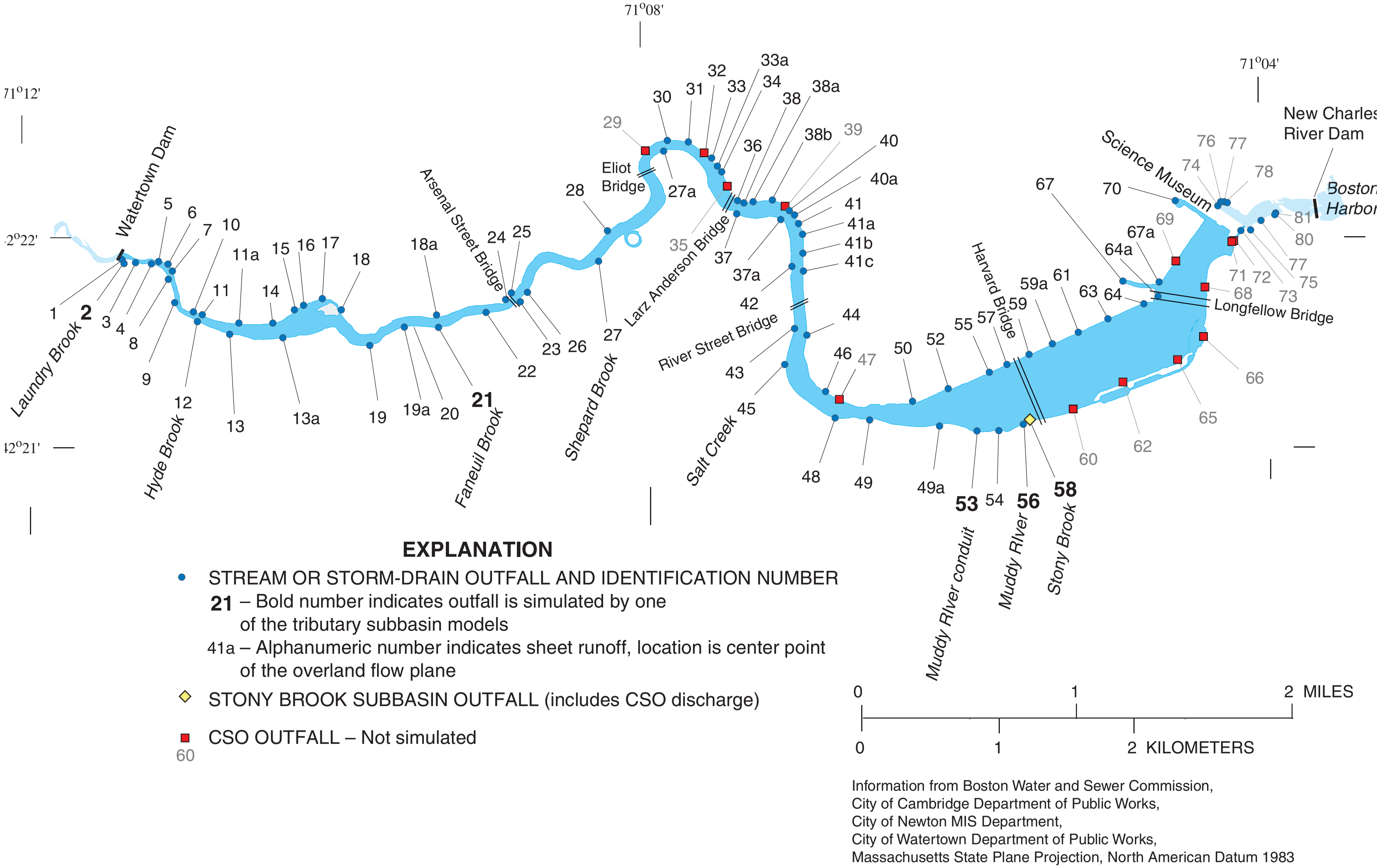


Figure 6. Outfalls to the lower Charles River, Massachusetts.

In 1992, a SWMM application was developed for the North System of the Massachusetts Water Resources Authority (MWRA) by M&E (Metcalf and Eddy, Inc., 1994). The North System includes all of the lower Charles River Watershed and portions of the Mystic and Neponset River Watersheds. The model was used to calculate CSO volumes in support of MWRA CSO facilities planning. The MWRA's recommended CSO control plan included a screening and disinfection facility to remove floatables and disinfect discharges from the Stony Brook conduit. The model was discretized coarsely to represent runoff because of the large geographic area represented and because it was intended for use as a planning tool. For example, the model represented the entire Stony Brook Subbasin as a single subcatchment with no channel routing. Models simulated (1) four actual storms that ranged from 0.73 to 0.95 in., with data collected from November 1992 to November 1993, (2) design storms for 3- and 6-month, and 1-, 2-, and 5-year storms, and (3) continuous simulation for 1992.

The U.S. Army Corps of Engineers (USACE) developed a new SWMM application of the Muddy River Subbasin following the October 1996 flood that caused damage to lower parts of the subbasin. The USACE used the SWMM RUNOFF module for subbasin runoff, SWMM TRANSPORT module for channel routing, and the USACE UNET model for unsteady-flow routing and to calculate floodwater elevations.

In 1997, MWRA provided the Boston Water and Sewer Commission (BWSC) a SWMM of the Stony Brook Subbasin. The BWSC Stony Brook model was based on the MWRA North System model and was used by Camp, Dresser, and McKee (CDM) and BWSC to develop alternatives to the MWRA plan of a screening and disinfection facility (Camp, Dresser, and McKee, 1997). From these analyses, BWSC identified sewer separation as a potential cost-effective alternative to the proposed screening and disinfection facility. MWRA and M&E conducted further evaluations that resulted in revising the CSO control plan to include separation of the 12 active regulators in the Stony Brook System. The CDM study included continuous discharge monitoring from April 22 to June 16, 1996, at 34 locations through the subbasin. During the monitoring period, discharge data were collected for eight storms that ranged in size from 0.26 to 1.72 in.; the largest was about a 3-month storm.

The BWSC Stony Brook model subsequently was expanded by CDM to include additional detail of the Stony Brook System (Camp, Dresser, and McKee, 1998). For example, the area upstream of Forest Hills, tributary to the Stony Brook conduit, which originally was modeled as a single 5,091-acre subcatchment, was discretized into 20 subcatchments. In addition, CDM further analyzed the flow capacity of the Muddy River by modifying the USACE SWMM model for the BWSC. CDM replaced the UNET model with the SWMM EXTRAN module for unsteady-flow routing and incorporated the physical characteristics of the Muddy River conduit into the model to split flow into separate outfalls to the lower Charles River (fig. 6). Model simulations of the Muddy River included hyetographs from actual storms that represent 1-, 2-, and 100-year storms and a synthetic hyetograph of a 25-year storm. CDM combined the Stony Brook and Muddy River models into one SWMM model, which is referred to herein as the Muddy River-Stony Brook model.

The Muddy River-Stony Brook model was refined further by M&E in 1998 for use in the design of the Stony Brook sewer-separation project (Metcalf and Eddy, Inc., 1999). Additional detail included an extensive representation of building and roof drains connected to the combined-sewer system to evaluate the effects of disconnecting roof drains on CSOs. This version of the Muddy River-Stony Brook model was provided to the USGS for use in this study.

Other information on storm runoff was collected in parts of the lower Charles River by other consultants. CH2M Hill (1990) collected discharge data at Electric Avenue in Brighton for two storms in May and July 1988 for the MWRA CSO facilities plan; however, it was noted that these data had a high degree of uncertainty because of instrument fouling from debris. Rizzo Associates (1993) monitored discharge at a low-density residential area in West Roxbury and at an industrial area in Brighton for nine storms from April through July 1992 for the BWSC.

Acknowledgments

The authors would like to thank the following municipalities and local agencies for allowing access to their geographic information: BWSC, Brookline Information Technologies Department, Cambridge MIS Department, Cambridge Department of Public

Works, Newton MIS Department, and Watertown Department of Public Works. Lawrence Soucie and Sharon Ho of Metcalf & Eddy, Inc., provided the latest SWMM of the Muddy River–Stony Brook Subbasins and assistance on running this model. Mark Voorhees of the USEPA, and Jay Brolin, formerly of the USEPA, provided data and reports of previous stormwater-model investigations. Peter Weiskel of the USGS coordinated field and office work, managed meetings between USGS staff and the project Technical Advisory Committee (TAC), and carefully scrutinized all aspects of the study. The project TAC included William Walsh-Rogalski and Mark Voorhees of USEPA, Lise Marx of the MWRA, Kevin Brander of the MADEP, Kathleen Baskin of the Charles River Watershed Association (CRWA), and Dominique Brocard of Metcalf and Eddy, Inc.

GAGED SUBBASINS

The gaged subbasins include three relatively homogenous land-use subbasins and the four largest tributary subbasins to the lower Charles River, and Charles River at Watertown Dam (table 1, fig. 1). The upper watershed (Charles River above Watertown Dam) represents about 88 percent of the total drainage area to the lower Charles River. The gages on the tributaries, excluding the upper watershed, collectively measured runoff from about 66 percent of the drainage area to the lower Charles River.

The land-use subbasins are 0.36 mi^2 or less in size and drain areas that predominantly are (1) single-family residential—01104630, (2) multifamily residential—01104673, and (3) commercial—01104677 (table 1). The gaged areas of the four tributary subbasins range in size from 1.42 to 11.8 mi^2. Land use in the tributary subbasins predominantly is single-family residential, but generally is less residential in the eastern subbasins than in the western subbasins (table 1). The eastern subbasins and the areas nearer to the river tend to have more commercial land use than other areas. Gaged tributary subbasins included (1) Laundry Brook—01104640, (2) Faneuil Brook—01104660, (3) Muddy River—0110483, and (4) Stony Brook—01104687.

Roads often are coincident with the storm-drainage system and have a large effect upon the hydrologic response of a drainage basin. Road density is similar among land-use subbasins and averages about 30 mi of road per square mile of drainage area; storm-drain density averages 25 mi/mi^2 and is slightly higher in the multifamily subbasin than in the commercial and single-family subbasins (table 1). Road densities are similar among the tributary subbasins, which average about 21 mi/mi^2; storm-drain densities average 18 mi/mi^2 and are slightly higher in Laundry and Faneuil Brooks than in Muddy River and Stony Brook. The characteristics of the gaged tributary subbasins are similar to the characteristics of the ungaged areas.

Discharge

Discharge data were collected during the 2000 water year (October 1, 1999, to September 30, 2000) unless otherwise noted. These data were used to compute contaminant loads described in a companion report (Breault and others, 2002), to calibrate the runoff models, and to evaluate the model fit described in this report. Specific information about the measurement and development of the discharge data collected at each station is reported in downstream order.

Charles River at Watertown Station—01104615

The upper Charles River Watershed drains an area of 268 mi^2. River stage was measured in the pool above the dam by a bubble gage and pressure sensor at 15-minute intervals. A stage-discharge relation was developed with the use of standard techniques (Carter and Davidian, 1965; Buchanan and Somers 1965; Kennedy, 1983, 1984). Four discharge measurements that ranged from 30 to 1,230 ft^3/s were used to define the rating during the water year; the computed discharge ranged from 24 to 1,350 ft^3/s during the water year. Discharge records are considered excellent (meaning that measurement error is within 5 percent) except for a short period of missing record (August 18 through September 1, 2000). The missing record estimated from the upstream USGS gage at Waltham is considered fair (the error is between 10 and 15 percent). Records for discharges below 200 ft^3/s are considered poor (measurement error is greater than 15 percent) because the Watertown Dam, which is flat and wide (190 ft), causes a large percent change in discharge for small change in stage at lower discharges. Daily mean discharge for the 2000 water year was published in the USGS MA–RI Water Resources Data Report (Socolow and others, 2001).

Table 1. Physical characteristics at gaged and ungaged subbasins in the lower Charles River Watershed, Massachusetts

[Subbasin characteristics apply to the gaged drainage areas. Location of gaged basins shown in fig. 1. **Latitude and longitude**: In °, degrees; ′, minutes; and ″, seconds. **Commercial:** Commercial also includes industrial, spectator recreation, transportation, and waste disposal. **Urban open:** Urban open also includes participation recreation, open, cemeteries, parks, institutional green space, cropland, and pasture. **Other:** Other includes open water and wetlands. mi/mi^2, miles per square mile; mi^2, square miles; --, not applicable; -, negligible amount]

Station name	Station No.	Latitude ° ′ ″	Longitude ° ′ ″	Drainage area to gage (mi^2)	Drainage area to mouth (mi^2)	Average slope (percent)	Land-use subbasins: Single-family residential	Multifamily residential	Commer-cial	Urban open	Forest	Other
Gaged												
Land-use subbasins												
Single family	01104630	42 19 59	71 11 49	0.356	--	4.3	97	-	1	2	-	-
Multifamily	01104673	42 22 28	71 06 49	.038	--	2.0	-	78	-	22	-	-
Commercial	01104677	42 22 14	71 06 53	.023	--	3.1	-	24	76	-	-	-
Tributary Subbasins												
Laundry Brook	01104640	42 21 53	71 11 21	4.70	4.70	5.4	66	-	12	9	11	2
Faneuil Brook	01104660	42 21 21	70 09 20	1.42	1.78	8.9	53	12	8	20	4	3
Muddy River	01104830	42 20 14	76 06 41	5.44	*6.32	7.2	46	14	10	16	7	7
Stony Brook	01104687	42 19 04	71 06 12	11.8	12.8	6.5	35	14	13	26	12	-
Charles River at Watertown	01104615	42 21 54	71 11 25	268	268	--	--	--	--	--	--	--
Ungaged area[1]	--	--	--	--	9.68	3.6	25	17	29	20	-	9

Station name	Road miles	Road density (mi/mi^2)	Storm drain (mi)	Storm drain density (mi/mi^2)	Soils (percent): Till	Sand and gravel	Urban	Alluvium and aeolian
Gaged								
Land-use subbasins								
Single family	9.8	28	8.6	24	51	2	47	-
Multifamily	1.1	28	1.0	27	-	75	25	-
Commercial	0.7	29	0.5	23	-	75	25	-
Tributary Subbasins								
Laundry Brook	97	20	86	18	40	33	23	4
Faneuil Brook	30	21	29	20	76	8	16	-
Muddy River	115	21	87	16	61	13	25	1
Stony Brook	252	21	191	16	62	13	23	2
Charles River at Watertown	--	--	--	--	--	--	--	--
Ungaged area[1]	290	23	192	15	26	13	60	1

*Includes the 0.54 mi^2 drainage area that drains to the Muddy River conduit.

[1]Aggregate of all ungaged areas not associated with a tributary subbasin.

Discharge at the Charles River at Watertown station for the two design storms, based on actual storms of July 20, 1982, and September 20, 1961, were calculated from discharge data available from the upstream gage at Waltham, Massachusetts (01104500). Hourly data were obtained from the original strip chart record for September 20, 1961 (1-year design storm), and from primary computation printouts for July 20, 1982 (3-month design storm). Concurrent hourly discharge data collected during the 2000 water year at the Waltham and Watertown Dam stations were related (fig. 7) by two non-linear power functions:

$$Q_{Watertown\ Dam} = 6.8097 Q_{Waltham}^{0.7334}, \quad (1)$$

when discharge is at or below 450 ft^3/s at Waltham;

$$Q_{Watertown\ Dam} = 3.6605 Q_{Waltham}^{0.8341}, \quad (2)$$

when discharge is above 450 ft^3/s at Waltham;

where

$Q_{Watertown\ Dam}$ is the discharge at Watertown Dam in cubic feet per second, and

$Q_{Waltham}$ is the discharge at Waltham in cubic feet per second.

The difference between the discharge computed from the equations and the observed discharge had a root mean square error (RMSE) of 24 ft^3/s for hourly values and 16 ft^3/s for daily values. Discharge equations (1 and 2) reflect the total drainage area to the Watertown gage. Part of the drainage area, 23.6 mi^2, usually does not contribute to the Waltham and Watertown streamgaging stations because discharge from it is usually diverted into the Cambridge water-supply reservoirs. However, during the 2000 water year, runoff from this drainage area was not diverted from the watershed because of construction in the Cambridge reservoir system.

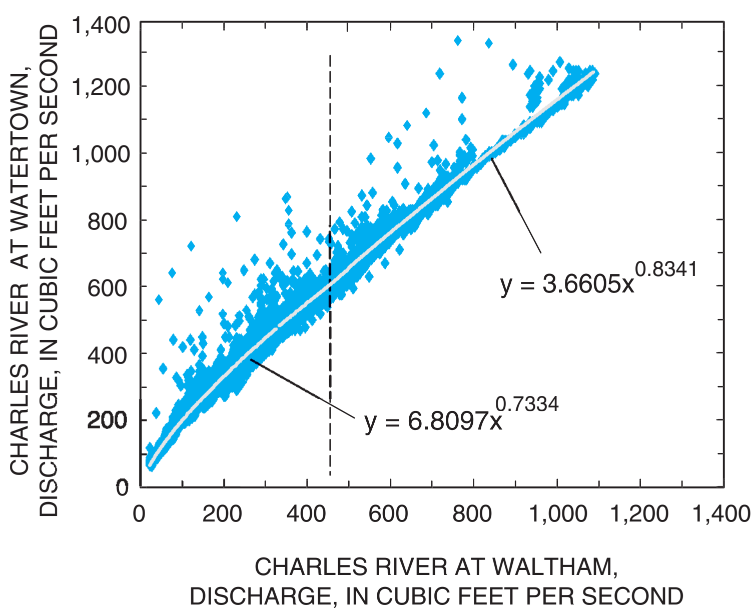


Figure 7. Relation of hourly discharge of Charles River at Waltham (01104500) to Charles River at Watertown Dam (01104615), Massachusetts, 2000 water year.

Single-Family Land-Use Station—01104630

Runoff from a single-family land-use area was measured in the headwaters of Laundry Brook Subbasin at Newton Center. Stage was measured at 2-minute intervals by a submersible pressure transducer mounted near the bottom of a brick-lined 5.5-ft diameter pipe about 0.2 ft upstream from where it discharges into an open rectangular mortared field-stone channel. A shallow, 3.32-ft long v-notch weir was used at this site for a low-water control. The bottom of the v-notch was 0.29 ft above the bottom of the pipe, and the top of the weir was 0.66 ft above the bottom of the pipe.

The stage-discharge relation was developed from eight discharge measurements and continuous velocity measurements made with an electromagnetic sensor over a period of 22 days. Combined, these data provide a good stage-discharge relation between 0.14 and 13 ft^3/s. Above 13 ft^3/s (0.53 ft), Manning's equation was used to develop a theoretical rating from measured channel slopes and roughness coefficients estimated from field observations and the literature. The roughness coefficient was adjusted slightly until the theoretical rating matched the highest measured discharges. An iron security gate at the pipe exit, which is prone to blockage by debris, has an unknown effect on the hydraulics of the channel at high discharges.

The computed 2-minute discharges ranged from 0 to 428 ft^3/s. For discharges below 13 ft^3/s, discharge is considered good except for a short period of missing record from August 14 through 23, 2000. The accuracy of the discharge above 13 ft^3/s is unknown because the theoretical stage-discharge relation could not be verified above this discharge. Therefore, the higher the computed discharge is above the verified part of the theoretical rating, the greater the uncertainty of its value. Duration analysis of the 2-minute data indicates that 13 ft^3/s is exceeded 0.31 percent of the time, 50 ft^3/s is exceeded about 0.02 percent of the time, and 100 ft^3/s is exceeded less than 0.01 percent of the time; however, flows greater than 13 ft^3/s represent about 25 percent of the measured runoff volume.

At least one high-intensity storm, on July 18, 2000 (maximum rainfall exceeded 0.5 in. in a 15-minute interval measured at several rain gages), appears to have generated full-pipe flow (pressurized flow). Full-pipe flow may have been caused by surcharging or backwater due to debris on the security grate. Debris was found lodged in the transducer mountings at the top of the pipe after the storm. The stage-discharge rating does not account for pressurized flow or backwater; thus, the peak discharge computed for this storm could be overestimated.

Laundry Brook Station—01104640

Laundry Brook streamflow was measured near the confluence with the lower Charles River. Stage was measured at 2-minute intervals by an ultrasonic transducer mounted in the roof of a 7-ft-high by 10-ft-wide concrete-box culvert about 50 ft upstream of the culvert outlet. The culvert discharges into an open channel about 50 ft from its confluence with the lower Charles River. A shallow v-notch weir about 4 ft downstream of the transducer was used for a low-water control. The bottom of the v-notch was 0.2 ft above the bottom of the culvert. The top of the weir was 1.0 ft above the bottom of the culvert.

A theoretical rating was developed by use of the HEC-RAS steady-state hydraulics model for open channels (Brunner, 2001). The friction slope was assumed to be equal to the culvert slope. Fifteen channel cross sections, positioned from 60 ft downstream of the v-notch weir to about 100 ft upstream of the weir, were used to describe the conduit geometry. Cross sections were spaced closely near the weir and near the stage sensor. A roughness coefficient of 0.011 was used for the sides of the culvert and 0.013 was used for the bottom of the culvert.

Water-surface elevations were computed at each cross section for discharges of 1, 5, 10, 50, 100, 200, 400, 500, and 600 ft^3/s. At the cross section that corresponds to the transducer location, the computed Froude number changes from 0.52 at 200 ft^3/s to 1.98 at 400 ft^3/s, which indicates a transition from critical to supercritical flow. Stage-discharge relations between 200 and 400 ft^3/s are, therefore, unstable. Five discharge measurements were made at this site that ranged from 0.41 to 75 ft^3/s. The hydraulic model rating agrees with the highest measured discharge (76 ft^3/s) to within 1 percent and generally agrees with the measured stage-discharge values at low flows. However, measured stage-discharge values were used solely for developing the rating below 20 ft^3/s.

Discharge records began October 28, 1999, and are considered good. The computed discharge ranged from 0.10 to 216 ft^3/s and likely was not high enough to become supercritical flow. Discharge at the station can be affected by the regulation of Bulloughs Pond, which at times is partially drained by the City of Newton in anticipation of large storms.

Faneuil Brook Station—01104660

Faneuil Brook streamflow was measured at Brighton about 0.3 mi upstream from its confluence with the lower Charles River; the drainage area to this point composes about 80 percent of the total Faneuil Brook drainage area. Stage and velocity were measured at 2-minute intervals by a submersible pressure transducer and electromagnetic-velocity sensor, respectively. Sensors were mounted near the bottom of a 9-ft-diameter concrete pipe. Discharge was computed directly from stage and velocity measurements; the data logger converted the stage to an area and adjusted the point velocity taken near the bottom of the pipe to a theoretical average velocity for a given stage. The computed discharge during base-flow periods (generally about 1 ft^3/s or less) was erratic because of insufficient water depth over the velocity sensor. During base flow, a stage-discharge relation was developed from velocity-stage readings and from three cross-sectional discharge measurements.

Suspect velocity measurements were observed during some storms, particularly during the recession, as indicated by an inconsistency in discharge values obtained by a stage-dischargeS relation and the velocity measurements. For example, the area-velocity discharge was consistent with the discharge computed from a stage-discharge relation during the first part of a storm on December 6, 1999 (fig. 8), but during the later part of the storm the area-velocity discharge quickly dropped while the computed stage discharge oscillated and recessed over about a 10-hour period. Fouling of the velocity sensor by debris carried during storms is believed to cause this problem. Site visits during non-storm periods indicate the presence of trash in the conduit, such as plastic bags, that could easily foul the transducer. For this reason, the stage-velocity discharge measurements were compared against the computed discharge made from the stage-discharge relation.

The computed discharge ranged from 0.33 to 179 ft^3/s. Discharge records generally are considered good except for occasional erratic readings and two short periods of missing record (October 19–27, 1999, and April 28–May 1, 2000). To the extent possible, erratic readings were edited so that they were consistent with neighboring readings. During several storms, the discharge computed from the stage-velocity reading appear inconsistent with the discharge obtained from the stage-discharge relation. This inconsistency may be a result of debris fouling the velocity sensor. For example, a plastic bag caught on the velocity sensor could disrupt or weaken the penetration of magnetic field in the column and cause an erroneously low velocity measurement. During periods when the velocity sensor appeared fouled, the stage-discharge rating was used to compute discharge.

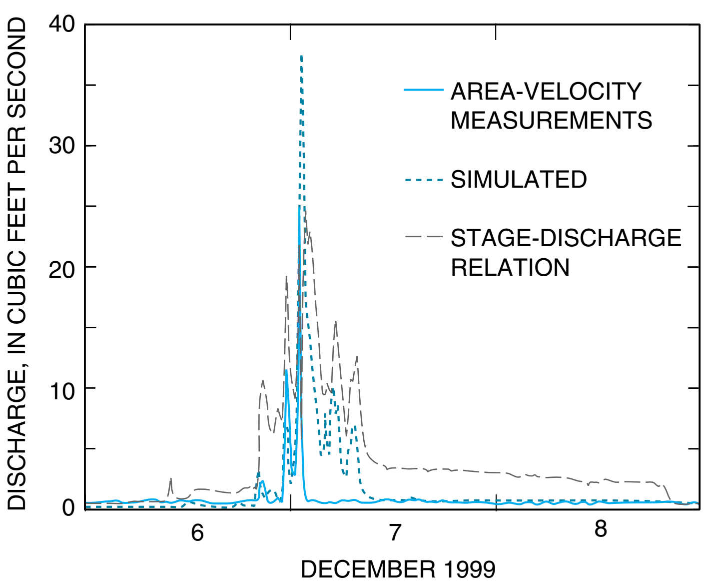


Figure 8. December 6–8, 1999, storm hydrographs at Faneuil Brook streamgaging station (01104660), lower Charles River Watershed, Massachusetts, determined by three methods (*A*) area-velocity measurements, (*B*) stage-discharge relation, and (*C*) simulated by the StormWater Management Model (SWMM).

Multifamily Land-Use Station—01104673

Runoff from a multifamily land-use area was measured at 2-minute intervals in a bricklined 2.8-ft-high oval storm drain on Broadway in Cambridge by a submersible pressure transducer mounted near the floor of the pipe. Manning's equation was used solely to develop the stage-discharge relation. The data from the stage-velocity sensor installed at this site for 56 days was erratic and could not be used. Slope was determined from invert elevations supplied by the City of Cambridge drainage network digital data layer. A roughness coefficient of 0.015 was used, which was similar to the roughness coefficient obtained when Manning's equation was applied to other brick-lined pipes in the study area.

The computed discharge ranged from 0 to 26 ft^3/s since the station became operational in November 23, 1999. The stage-discharge rating could not be verified, thus, discharge records are of unknown quality. Data are missing between February 3 and March 22, 2000. Erratic discharge was observed during storms at the beginning of April 2000; discharge would increase in response to rainfall then drop suddenly to zero. This discharge pattern would be repeated over the course of the storm several times for no apparent reason; the data are suspect during these periods.

Commercial Land-Use Station—01104677

Runoff from a commercial land-use area was measured at 2-minute intervals in a 3.0-ft-diameter concrete storm drain on Mt. Auburn Street in Cambridge by a submersible pressure transducer mounted near the bottom of the pipe. A water stage and velocity sensor temporarily installed in the storm drain for a 2-day period provided discharge data between 0.1 and 6 ft^3/s. These data provided a good stage-discharge relation below a 1.0-ft stage. A theoretical rating was developed for stages above 1.0 ft from Manning's equation. Slope was determined from invert elevations supplied by the City of Cambridge digital coverage of the drainage network. A roughness coefficient of 0.019 was determined by slightly adjusting the literature value for a smooth concrete pipe until the theoretical rating matched the highest measured discharges.

Computed discharges ranged between 0 and 25 ft^3/s since the station began operation in January 20, 2000. Stage measurements during the first 2 months of operation were noisy, but appeared to respond to storm precipitation. Discharge records are considered good below 6 ft^3/s, but above this discharge the accuracy of the theoretical rating is unknown. Hence, the larger the computed discharge is above the verified part of the theoretical rating (6 ft^3/s), the greater the uncertainty of its value. Duration analysis of the 2-minute data indicated that 6 ft^3/s was exceeded 0.42 percent of the time, which represented about 10 percent of the measured runoff volume; 10 ft^3/s was exceeded 0.02 percent of the time; and 20 ft^3/s was exceeded less than 0.004 percent of the time. Discharge records are missing from April 3 through 19, 2000.

Muddy River Station—01104683

Muddy River streamflow was measured at Brookline about 2.1 mi upstream from its confluence with the lower Charles River; the drainage area to this point composes about 91 percent of the total Muddy River drainage area. Stage was measured at 15-minute intervals by a bubble-gage transducer in an open channel. Muddy River is a low-gradient stream with heavily vegetated banks. Variable backwater can result from debris in and along the channel, and from blockage of a trash grate 0.6 mi downstream of the gage, where the channel enters a culverted section for 400–500 ft prior to its discharge point at the Charles River. In addition to backwater from these factors, stage at the gage is affected highly by the water level in the lower Charles River, which is regulated daily to maintain desired seasonal water levels. The relation between the stage at the Muddy River gage and the stage of the lower Charles River over a 3-day period is shown in figure 9. The figure also illustrates how water levels in the lower Charles River are dropped during low tide and how regulation prevents high tide from raising water levels in the lower Charles River. In addition, large pumps are used during some storms to maintain water levels in the lower Charles River even when the tidal cycle is high. As a result of these variable backwater conditions, discharge estimates for the Muddy River Subbasin are poor (more than 20-percent measurement error) and have a high degree of uncertainty.

A rating was developed for the Muddy River gage from 11 discharge measurements that ranged from 1.8 to 230 ft^3/s. The stage for each discharge measurement was adjusted to an estimated stage if the water levels in the lower Charles River were at the average low-water level. For example, if the stage during the discharge measurement was at 8.8 ft, and the water level in the lower Charles River was 0.4 ft above its average low-water level, a correction of -0.4 ft was applied to the measurement stage. The adjusted stage had less scatter around the best-fit, stage-discharge

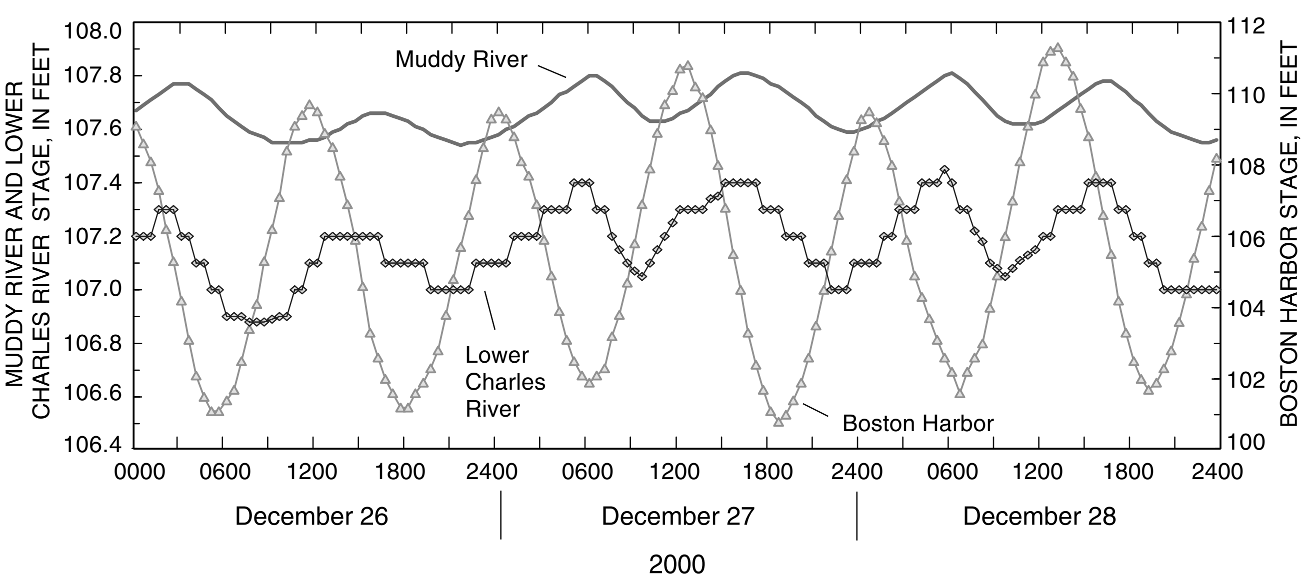


Figure 9. Relation of stage at the Muddy River streamgaging station to water levels in the lower Charles River and Boston Harbor, Massachusetts. (Stage is based on the reference to the Metropolitan District Commission datum.)

relation line than did the unadjusted stage; however, the scatter still is higher than acceptable for standard rating development. By use of the same technique, the measured storm stage was adjusted to minimize the effects of the backwater from the regulation of the lower Charles River.

Stony Brook Station—01104687

Stony Brook streamflow was measured at Jamaica Plain about 2.4 mi upstream from its confluence with the lower Charles River; the drainage area to this point composes about 90 percent of the Stony Brook drainage area. A submersible pressure transducer mounted along the bottom arch of a 17-ft-high brick-lined, horseshoe-shaped conduit recorded stage at 15-minute intervals. An inverted siphon about a mile downstream of the gage probably controls flow during high water. However, conduit geometry and slope control flow rates at lower flows. Iron grates protrude about 4 to 5 in. at the bottom of the conduit near the gage that likely affect the Manning's roughness coefficient. A small debris pile was noted above the gage that probably formed on one of the protruding grates, but, because of inaccessibility, the extent of similar obstructions downstream of the gage are unknown.

A rating was developed on the basis of Manning's equation from surveyed geometry, one cross-sectional discharge measurement made in the conduit, and seven single-point velocity measurements. The single-point velocity measurements were made by lowering a current meter through the manhole (discharge ranged from 7 to 300 ft^3/s). During storm flows, conditions are extremely hazardous and access is prohibitive. High velocities in the conduit limit a conventional point-velocity-current-meter measurement above a 2-ft stage. Slope was determined from water-surface elevations surveyed over a distance of 156 ft (the maximum distance that could be obtained by survey crews tethered to safety lines). A roughness coefficient of 0.019 was estimated by fitting the measured velocities to the computed velocities from Manning's equation.

Computed discharge ranged from 0.49 to 856 ft^3/s since the site became operational at the beginning of April 2000. A short period of record is missing from April 20 through 24, 2000. Discharge records are considered good from 6.0 to 270 ft^3/s where discharge measurements confirmed the Manning rating. The accuracy of the rating above 270 ft^3/s is uncertain because the inverted siphon could affect the rating; however, discharges exceeded 270 ft^3/s only 0.39 percent of the time; these discharges represent about 12 percent of the total measured volume.

Data Management

Stage and computed discharge values for the five tributary gaging station subbasins (Charles River at Watertown—01104615, Laundry Brook—01104640, Faneuil Brook—01104660, Muddy River—01104683, and Stony Brook—01104687), and precipitation data from the USGS-CR3, were telemetered and automatically downloaded to ASCII files on a computer at the USGS office in Northborough, Massachusetts. The computed discharges downloaded from the data loggers were based on early stage-discharge relations, except at Faneuil Brook, which computed discharge from measurements of stage-area and velocity. The preliminary computed discharge from the data loggers was for display of near-real-time data over the Internet only; final computed discharge values were used for model calibration and model-fit statistics. At the three land-use subbasins, data were transferred from on-site data loggers to field computers periodically during site visits. Data from the field computers were downloaded to a computer at the USGS office in Northborough.

The raw ASCII files were checked for erroneous data, corrected, and then processed with the use of a computer program, Device Conversion and Delivery System (DECODES), for entry into the USGS Automated Data-Processing System (ADAPS). Only stage data were entered into ADAPS for each site, except for Faneuil Brook, which also included the computed area-velocity discharge measurement. ADAPS provides permanent storage of the data in the USGS National Water Information System (NWIS). The ADAPS system converted stage measurements into discharge from a specified stage-discharge relation for the stations described previously.

The recorded data entered into NWIS had either a 2-minute or a 15-minute time step. Calibration and assessment of the SWMM were made with 15-minute data. The 2-minute time-step was transformed with the ANNIE Watershed Data Management (WDM) system (Lumb and others, 1990).

PRECIPITATION-RUNOFF MODEL

The precipitation-runoff model, SWMM (Huber and Dickinson, 1988), was used to simulate runoff in response to precipitation in the lower Charles River Watershed because it was not possible to measure discharge at all outfalls to the lower Charles River (fig. 6), and because stormwater runoff was required for two design storms that could not be measured directly. SWMM was chosen for the simulation of runoff because it was designed for use in highly urbanized areas, such as the Boston metropolitan area. The model is a well-documented public domain FORTRAN program developed by the USEPA that has been used extensively in the analysis of urban stormwater runoff.

For this study, a proprietary interface to SWMM, PCSWMM 2000, developed by Computational Hydraulics International (CHI), was used for simulation. PCSWMM 2000 is a graphical-user interface to SWMM and has additional features not available with the SWMM public-domain software, such as simplified data management, model development, and streamlined model calibration. PCSWMM 2000 can use any available SWMM engine; SWMM version 4.4 was used for this study. The SWMM models can be run independently of the PCSWMM interface with minor modifications of input files to handle data transfer between modules.

Modeling runoff to the lower Charles River required the following steps: (1) collect, compile, and process needed data, (2) build the model run files, (3) calibrate the models to selected storm events, and (4) calculate model-fit statistics for all storms. Because streamflow data were collected at multiple unrelated drainage systems, separate models were developed for separate subbasins.

Functional Description of SWMM

SWMM is divided into modules that simulate many aspects of urban runoff quantity and quality; however, only the simulation of runoff quantity was used in the study presented here. Simulation modules used in this study include RAIN, COMBINE, RUNOFF, TRANSPORT, and EXTend TRANsport (EXTRAN). The RAIN module is for "service" to the RUNOFF module; the COMBINE module "services" any of the other modules; RUNOFF, TRANSPORT, and EXTRAN are the primary computational modules for runoff and flow routing. User manuals by Huber and Dickinson (1988) and James and others (1999a; 1999b) provide comprehensive documentation of these modules.

The RAIN module formats precipitation data from 1 to 10 gages into a single binary file for input to the RUNOFF module. The RAIN module also optionally provides statistical summaries of precipitation characteristics and antecedent conditions (Woodward-Clyde Consultants, 1989). Typically, the RAIN module is used for continuous simulations so that the time-series data can be called by the RUNOFF module.

The COMBINE module allows multiple interface files (*filename*.int) generated by the process modules to be aggregated or combined. For example, large, complex drainage systems may be partitioned for simulation into smaller segments and later combined.

The RUNOFF module is one of the primary process modules of SWMM, and is used to simulate runoff from pervious and impervious areas. Runoff is generated from rainfall, snowmelt, or both, and can be simulated in either a continuous or single-event mode. Runoff from impervious areas is generated once impervious-area interception storage is satisfied. Runoff from pervious areas is generated once pervious-area interception storage is satisfied and after precipitation exceeds infiltration simulated by either the Green-Ampt or the Horton method. Infiltrated water from pervious areas is routed to inlets only when the groundwater subcatchment information is provided; otherwise, infiltrated water is included in the model hydrologic mass balance but is not routed from the subcatchment.

One or more subcatchments can be specified in the RUNOFF module to simulate a watershed. Subcatchments are simulated as idealized rectangular subcatchments with a defined width and area. The width of the subcatchment affects the time required for precipitation to reach the outflow from the subcatchment; as width increases, the time required to reach the outflow decreases and conversely, as the width decreases the time it takes for precipitation to reach the outflow increases. Surface runoff and subsurface discharge from subcatchments also are affected by the percent effective impervious area and soil variables of a subcatchment.

Runoff from a subcatchment is directed to a node or "manhole" in the RUNOFF module. Nodes can be linked together with conduits to form a network of channels or pipes in the RUNOFF, TRANSPORT or EXTRAN modules. Flow routing in the RUNOFF module is primitive compared to that available in the TRANSPORT or EXTRAN modules. For this reason, flow routing usually is done outside of the RUNOFF module.

Flow routing in the TRANSPORT module can be performed for a number of predefined conduit shapes or by a user-defined-conduit shape. The kinematic wave equations (continuity and uniform-flow equations) route flow through the conduit system, but neglect backwater effects. Unique node identification numbers used in the RUNOFF module provide the linkage to the shared node identification numbers used in the routing modules. Nodes can represent manholes (simple nodes), lift stations, flow dividers, storage units, or backwater elements that can flow to other nodes or to conduit elements (links). The assemblage of nodes and links forms the drainage network of a watershed. Model output can be specified at any node in the drainage system used to represent the watershed.

Complex drainage systems and drainage systems subject to surcharge, pressure flow, and backwater effects can be simulated with the EXTRAN module. EXTRAN is an extended version of the TRANSPORT module that solves the complete dynamic-flow equations (continuity and momentum equations), also known as the St. Venant equations. The assemblage of nodes and links described in the EXTRAN is similar to that in the TRANSPORT module; however, additional elevation information is required to solve the momentum equations. The TRANSPORT module was used to describe storage components in a drainage system such as ponds and detention basins by a volume-discharge relation.

PCSWMM run files (*filename*.dat) are linked to related data files by data-aware objects. For example, the RUNOFF modules in the lower Charles River models all are linked to the binary precipitation data file (Rain.int) created by the RAIN module. Overland runoff computed by the RUNOFF module is written to a single binary file (*filename*.int) for all subcatchments. TRANSPORT or EXTRAN modules are linked as a related object to the binary file to route subcatchment runoff to nodes in the drainage network. Executive command lines for each module run file can be substituted for the data-object linkage provided in PCSWMM to run the modules in SWMM without the PCSWMM interface.

Model Development

Separate models were prepared for the three land-use subbasins, two tributary subbasins (Laundry Brook and Faneuil Brook), and for the remaining ungaged drainage areas not included in a tributary subbasin model. Ungaged subcatchments that discharge downstream of a gaged tributary subbasin were included in the tributary subbasin models. An existing SWMM model for the Muddy River–Stony Brook Subbasins supplied by Metcalf and Eddy, Inc., was used "as is" (without modification) except for substituting hyetographs and evaporation data for the conditions under investigation. This report only describes the models developed by the USGS. Documentation of the Muddy River-Stony Brook SWMM models can be found in unpublished reports described in the "Previous Investigations" section.

Four separate models are needed to simulate runoff to the lower Charles River: (1) the Laundry Brook model, (2) the Faneuil Brook model, (3) the Muddy River-Stony Brook model, and (4) the ungaged area model. The Laundry Brook model and the ungaged area model include the land-use subbasin models that were calibrated independently; the Laundry Brook Subbasin includes the single-family residential land-use model and the ungaged-area model includes the multifamily and commercial land-use models. The ungaged area model represents about a third of the lower Charles River Watershed. The spatial coverage of the models is shown in figure 10 and detailed schematics of the model elements used in the models developed by the USGS are provided in appendix 1.

Figure 10. Areas of the StormWater Management Models (SWMM) developed for the lower Charles River Watershed, Massachusetts.

The model developed for the Muddy River and Stony Brook Subbasins differs from the models prepared for the other drainage areas. The Muddy River-Stony Brook model was prepared primarily to simulate CSOs and to aid in the engineering of infrastructure to minimize or eliminate these overflows. For this reason, the EXTRAN module was used to produce a detailed representation of the drainage system for the Muddy River-Stony Brook model, which requires simulations to run at a 5-second time-step for numerical stability. This model also required a modified SWMM engine developed by Metcalf and Eddy, Inc.

The models developed by the USGS did not require the same level of detail because these models were designed primarily to simulate stormwater volumes for calculating constituent loads. Thus, the TRANSPORT module was sufficient to provide stormflow routing through the drainage network in these models. Peak discharge, however, could be overestimated if surcharging or backwater conditions are present. All models of the lower Charles River tributaries developed by the USGS were run at a 15-minute time step.

Data

Development of a precipitation-runoff model requires spatial data to characterize the physical aspects of the watershed. Input time-series data of precipitation and evapotranspiration are necessary to run the model. Discharge time-series data previously described are needed to calibrate the model and evaluate model fit.

Spatial Data

Spatial data were obtained to delineate subcatchments boundaries and outfall locations from soils, topography, and land-use data. These data were obtained from a variety of sources including local municipalities, consulting firms, the Massachusetts Geographic Information System (MassGIS) office of the Massachusetts Executive Office of Environmental Affairs, and the Natural Resource Conservation Service (NRCS). All spatial information was entered into ArcInfo as digital data layers.

Subcatchment boundaries were determined from the drainage network coverage that was developed from Computer Aided Design (CAD) engineering drawings at the 1 in.= 100-ft or 1 in.= 400-ft scale. Outfalls to the lower Charles River were identified from the storm-drain network. The CAD-attribute information typically included average pipe slopes, diameters, and lengths. CAD data were obtained from local municipalities and their consultants.

Topographic and land-use data were obtained from MassGIS. Digital Terrain Model (DTM) 1:5,000-scale data were converted to grids using ArcInfo GRID. GRID was used to calculate an average subcatchment slope. Land-use data were classified into 37 possible categories from interpretation of 1:25,000-scale 1991 aerial photos (MassGIS, 2001). Land use was simplified into 21 categories to calculate initial effective impervious values and to select the land-use subbasin sites.

Soils data were obtained from the NRCS 1:25,000-scale digital SSURGO (Soils Survey Geographic) data for Norfolk and Suffolk Counties. SSURGO data were unavailable for Middlesex County, which had to be manually digitized from 1: 25,000-scale county soil-survey maps. Soil units were classified by hydrologic soil groups and by soil texture.

Time-Series Data

Precipitation data were collected by tipping-bucket instrumentation at six gages (table 2, fig. 1) for the 2000 water year (October 1, 1999, through September 30, 2000). The USGS established one precipitation gage (USGS-CR3) for this study at Watertown Dam (fig. 1). The other five precipitation gages (CWD-PP1, CDPW-HS2, BWSC-CS4, MWRA-WS5, and BWSC-WS) were established previously and are operated by others listed in table 2. Data recorded at 5-minute time steps were transformed to 15-minute time-steps to develop input time-series data sets for the RUNOFF modules.

The USGS-CR3 gage did not begin operation until November 3, 1999. Rainfall was estimated at this site from October 1 to November 3, 1999, by averaging rainfall from gages BWSC-CS4, MWRA-WS5, and BWSC-WS6. The CDPW-HS2 gage was not operated from November 12, 1999, to March 15, 2000. During this period, precipitation data recorded at the BWSC-CS4 was substituted for precipitation data at CDPW-HS2 so that the assignment of precipitation gages to subcatchments in the RUNOFF module did not have to be varied at different times of the year.

The data from precipitation gages were formatted and read into the RAIN module of SWMM. The SWMM RAIN module provided information on the start hour, duration, volume, average intensity, and

Table 2. Precipitation gages in and near the lower Charles River Watershed, Massachusetts

[Locations shown in figure 1. **Latitude and longitude**: In °, degrees; ′, minutes; and ″, seconds]

Station name	Agency and location	Latitude ° ′ ″	Longitude ° ′ ″	Heated
CWD-PP1	Cambridge Water Department, Payson Park Reservoir	42 22 59	71 10 06	yes
CDPW-HS2	Cambridge Department of Public Works, Hampshire Street	42 22 17	71 05 51	no
USGS-CR3	U.S. Geological Survey, Watertown Dam	42 21 54	71 11 26	yes
BWSC-CS4	Boston Water and Sewer Commission, Cambridge Street	42 21 08	71 08 30	yes
MWRA-WS5	Massachusetts Water Resources Authority, Ward Street	42 20 09	71 05 46	yes
BWSC-WS6	Boston Water and Sewer Commission, Washington Street	42 17 12	71 07 42	yes

maximum intensity of each storm. In addition, the RAIN module reformats the data into a binary file read by the RUNOFF module.

Daily potential evapotranspiration (PET) for the 2000 water year was calculated by the Hamon method (Hamon, 1961) with the use of METCMP (Lumb and Kittle, 1995). The Hamon method was chosen because SWMM requires only a total monthly PET, and data required to compute PET by the Hamon method could be obtained readily over the Internet. Minimum and maximum daily air temperatures reported at Logan Airport in Boston were obtained from the Northeast Climate Center at Cornell University. A monthly variable coefficient (CTS) was assigned a constant value of 0.0055, as suggested by Hamon (1961). Total PET for the 2000 water year was 26.19 in. and ranged from a monthly low of 0.46 in. in January 2000 to a high of 4.75 in. in July 2000.

Representation of Overland Runoff

Drainage subbasins to each of the outfalls mapped in figure 6 were represented by one or more subcatchments in the RUNOFF module (appendix 1). Subcatchment boundaries were determined from the storm-drain maps and topography (appendix 1). Many outfalls were represented by a single subcatchment, others with large drainage areas had multiple subcatchments; the Faneuil Brook Subbasin model was represented by 19 subcatchments, the Laundry Brook Subbasin model was represented by 17 subcatchments, and the ungaged area model was represented by 90 subcatchments. Individual outfalls from the ungaged areas were represented by as many as five subcatchments, but most outfalls were represented by only one or two subcatchments. Each of the land-use subbasin models was represented by a single subcatchment.

Each subcatchment is assigned a 6-digit number in the RUNOFF module that identifies the subbasin, town, and outfall number that the subcatchment drains to in the lower Charles River. The first digit of the subcatchment number identifies the major subbasin (table 3), the second digit identifies the municipality, the third and fourth digits are sequential numbers for subcatchments within the same subbasin and town, and the fifth and sixth digits identify the outfall number that the subcatchment drains to in the lower Charles River (fig. 6). Seventeen subcatchments have no known outlet point and were assigned 00 in the outlet identification field and sequentially numbered starting from 01 in the subbasin identification field.

Assignment of Subcatchments to Precipitation Gages

Each subcatchment was assigned to a single precipitation gage. Assignments were made by intersecting Thiessen polygons of the gage network and the subcatchment coverage (fig. 10). If more than one rain gage was identified from the intersection of the two coverages, the rain gage with the largest area from the Thiessen polygon coverage was used. The CWD-PP1 rain gage (fig.1) initially was used and later omitted from model simulations because (1) there were various periods of missing record, (2) the data had to be disaggregated from an hourly time step to a 15-minute time step, and (3) only three ungaged subcatchments were assigned to this gage from the intersection with the original Thiessen polygon. The three subcatchments initially assigned to CWD-PP1 were reassigned the USGS-CR3 gage, which was the next closest gage to these drainage areas. Although the CWD-PP1 gage was

Table 3. First and second digit numbers of the six-digit subcatchment model number used to identify subbasins and municipalities in the lower Charles River Watershed, Massachusetts

First digit of the subcatchment number		Second digit of the subcatchment number	
Identifier	Subbasin name	Identifier	Municipality
1	Laundry Brook	1	Newton
2	Muddy River	2	Watertown
3	Faneuil Brook	3	Boston
4	Hyde Brook	4	Brookline
5	Stony Brook	5	Cambridge
6	Single family		
7	Multifamily		
8	Commercial		
9	Unnamed		

not used in the final model, it was left in the binary precipitation file (rain.int) because its sequential number indicates the time when the precipitation time-series data were entered into the binary file. Thus, eliminating the CWD-PP1 gage from the rain.int file would have required renumbering the rain-gage assignment of all subcatchments.

Subcatchment Parameterization

Model development requires initial variable values that are measured to the extent possible, but often are modified during the model calibration. The variables that define subcatchment characteristics in the RUNOFF module, and statements of how initial variable values were determined and whether they were modified during the model calibration are given in table 4.

Subcatchment Width, Slope, Storage, and Roughness: Subcatchment width initially was calculated as the average distance between the subcatchment boundaries perpendicular to its drainage channel; however, the widths were calibrated to obtain the best fit between the simulated and observed storm-peak discharge and time of peak. Subcatchment widths varied from 380 to 4,300 ft, and generally were larger in developed subcatchments than undeveloped subcatchments. The subcatchment slope was calculated from a 1:5,000-scale digital-elevation grid. Slopes ranged from 0 to 0.165 ft/ft. Interception storage on pervious surfaces ranged between 0.30 and 0.50 in. Roughness of the pervious surface varied by land use and ranged between 0.30 and 0.40, and was highest in open and forest areas.

Table 4. StormWater Management Model (SWMM) RUNOFF module subcatchment variables and method used to obtain their initial values in the models of the lower Charles River Watershed, Massachuscetts

[GIS indicates that a geographic information system was used to measure values electronically from various data layers; minor, the value was adjused only slightly during the model calibration. ft, foot; ft/ft, feet per foot; in., inches; in/hr, inches per hour]

Variable	Variable description	Initial value	Calibrated
WIDTH	Width of overland flow plain (ft)	GIS	Yes
AREA	Area of subcatchment (acres)	GIS	No
%IMP	Percent impervious area	Rainfall-Runoff relation	Yes
SLP	Ground slope (ft/ft)	GIS	No
IMPN	Impervious area Manning's roughness	Literature	Minor
PERVN	Pervious area Manning's roughness	Literature	Minor
IDS	Impervious area depression storage (in.)	Literature	Minor
PDS	Pervious area depression storage (in.)	Literature	Yes
Soil properties—Green-Ampt infiltration method			
SUCT	Average capillary suction (in.)	Literature	Yes
HYCON	Saturated hydraulic conductivity (in/hr)	Literature	Yes
SMDMAX	Initial soil moisture deficit (unitless fraction: volume air/volume voids)	Literature	Yes

Effective Impervious Area: Effective impervious area is the most sensitive of all model variables because the runoff from impervious surfaces, such as roads and rooftops, that drains directly to the drainage network has little opportunity for infiltration. Therefore, runoff from effective impervious area is directly proportional to rainfall minus interception and depression storage. Initial estimates of effective impervious area were assigned by land use from values of effective impervious area obtained from SWMM applications developed for the Neponset River Watershed and for Haverhill, Massachusetts (Mark Voorhees, U.S. Environmental Protection Agency, written commun., 1999), and are summarized in table 5. The effective impervious area was refined for subbasins in the lower Charles River Watershed with measured discharge by plotting the relation between rainfall and runoff in inches for small storms (rainfall volumes less than 0.5 in.) between May and September with dry antecedent conditions (fig. 11). The slope of the rainfall-runoff line indicates the fraction of effective impervious area in a basin (Jennings and Doyle, 1978). The intercept of the rainfall-runoff relation indicates the amount of rain required to produce runoff, which is a measure of the interception storage on effective impervious areas. A surface roughness coefficient of 0.02 was assigned to all impervious surfaces in the models.

Rainfall-runoff relations for the single-family land-use site indicated an effective impervious area of 18 percent (fig. 11*A*). The effective impervious area obtained from the calibrated model for this site, which appears to be better represented by rainfall-runoff values for storms less than 0.2 in., was similar (17.4 percent). The effective impervious area in this subbasin is about half that determined for single-family residential land use used in the Neponset River Basin. The depression storage for effective impervious areas indicated by the rainfall-runoff intercept (0.013 in.) is considerably less than the calibrated value (0.09 in.).

Rainfall-runoff relations for the multifamily land-use subbasin indicated an effective impervious area of 63 percent (fig. 11*B*). The calibrated effective impervious area (73 percent) was greater than that indicated by the slope of the rainfall-runoff line, but the calibrated effective impervious area is within 4 percent of the value obtained for similar land use in the Neponset River Basin. The depression storage for effective impervious areas indicated by the rainfall-runoff intercept (0.03 in.) did not change in the calibrated model.

Rainfall-runoff relations for the commercial land-use subbasin indicated an effective impervious area of 86 percent (fig. 11*C*). The rainfall-runoff plot was developed from selected low-intensity storms with rainfall-runoff coefficients less than 1.0. Possible causes for rainfall-runoff coefficients greater than 1.0 are explained later in the section “Model Fit.” The calibrated effective impervious area is the same as the slope of the rainfall-runoff line, but is about 54 percent larger than the effective impervious area for similar land use in the Neponset River Basin. The depression storage for effective impervious areas indicated by the rainfall-runoff intercept (0.03 in.) did not change in the calibrated model.

Rainfall-runoff relations for the Laundry Brook Subbasin indicated an effective impervious area of 6 percent (fig. 11*D*). The wide scatter in rainfall-runoff values likely is caused by variations in storage at Bulloughs Pond. The calibrated effective impervious area (11 percent) is about 80 percent higher than the slope of the rainfall-runoff line. The effective impervious area weighted by land-use type transferred from values used in the Neponset and Haverhill models indicated an effective impervious area of 23 percent, or about twice as large the calibrated effective impervious area for this subbasin. The depression storage for effective impervious area, indicated by the rainfall-runoff intercept (0.025 in.), was considerably less than the calibrated storage, which averaged 0.072 in. over the subbasin.

Table 5. Estimated percent effective impervious area applied to the StormWater Management Models (SWMM) of the lower Charles River Watershed, Masachusetts

[**Neponset:** Transferred from Neponset River and Haverhill SWMM for similar land uses. **Rainfall-runoff:** Calculated from rainfall-runoff plots for small summer storms. **Calibrated:** Final value after model calibration. --, value not available]

Site	Percent effective impervious		
	Neponset	Rainfall-runoff	Calibrated
Land-use subbasins			
Single family	33	18	17
Multifamily	76	63	73
Commercial	56	86	86
Tributary subbasins			
Laundry Brook	23	6	11
Faneuil Brook	36	8	14
Muddy River	30	--	42
Stony Brook	28	6	19

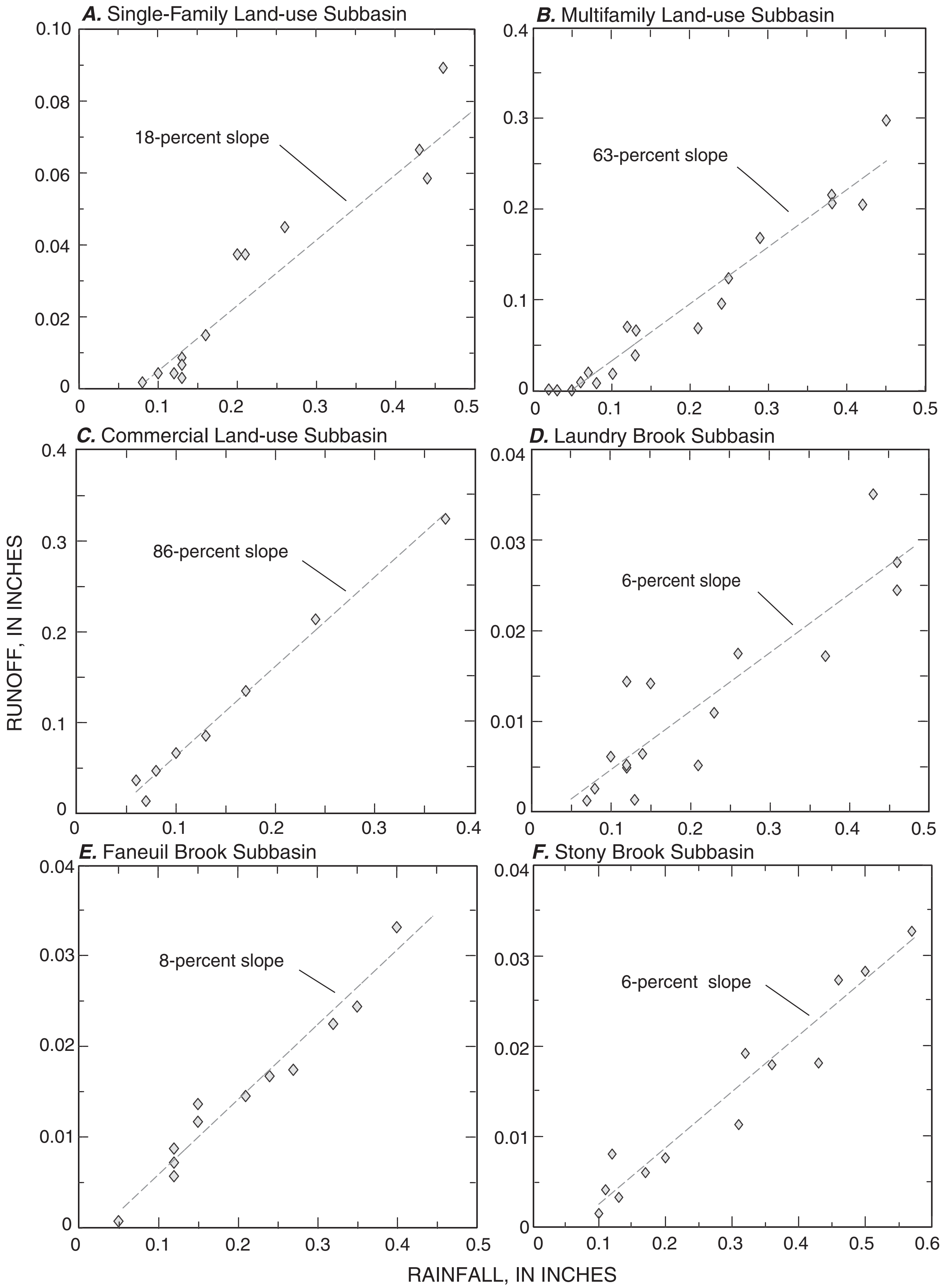


Figure 11. Rainfall-runoff relations for small storms from May through September 2000 at gaged subbasins in the lower Charles River Watershed, Massachusetts.

Rainfall-runoff relations for the Faneuil Brook Subbasin indicated an effective impervious area of 8 percent (fig. 11*E*). The calibrated effective impervious area (14 percent) is about 75 percent higher than effective impervious area indicated by the slope of the rainfall-runoff line. The effective impervious area weighted by land-use type, transferred from values used in the Neponset and Haverhill models, indicated an effective impervious area of 36 percent, or about 160 percent larger than the calibrated effective impervious area for this subbasin. The calibrated depression storage for effective impervious areas (0.066 in.) was slightly greater than that indicated by the rainfall-runoff intercept (0.05 in.).

Rainfall-runoff relations for the Stony Brook Subbasin (fig. 10*F*) indicate that the effective impervious area is 6 percent of the drainage area; this value is considerably less than the calibrated effective impervious area (19 percent) and the estimated effective impervious area for similar land uses from the Neponset River watershed (28 percent). The effective impervious area in the Muddy River model (42 percent) is somewhat higher than the estimated effective impervious area for similar land use types from the Neponset River watershed (30 percent). Although the rainfall-runoff derived value of the effective impervious area could not be calculated for the Muddy River Subbasin, the effective impervious area from the areally weighted model is high relative to the other tributary subbasins. The percent impervious values assigned in the Stony Brook–Muddy River model were not altered for this study.

Pervious Area Infiltration and Subsurface Flow: Infiltration on pervious areas was simulated by the Green-Ampt method. The Green-Ampt equation is a physically based model of infiltration that requires values of the soil's capillary suction (SUCT), saturated hydraulic conductivity (HYCON), and the initial moisture deficit (SMDMAX). Initial values for these variables were estimated from the hydrologic soil group and texture reported for the mapped soils in the Norfolk, Suffolk, and Middlesex County soil surveys (U.S. Department of Agriculture, 1995; Peragallo, 1989) and literature values for different soil textures reported by James and others (1999a). In subcatchments with more than one soil type, areally weighted averages were calculated.

Values of SUCT ranged from 4.3 to 9.7 in.; values of SUCT are lowest in sandy soils and highest in silty soils. Values of HYCON ranged between 0.01 and 0.40 in/h and values of SMDMAX ranged between 0.30 and 0.33 (unitless); values are highest in sandy soils and lowest in clayey soils.

Infiltrated water is routed to the drainage network only when ground-water cards (H2, H3, and H4) are specified in the RUNOFF module. Ground-water discharge from the saturated zone was treated as a simple linear reservoir in which water is drained at a rate defined by the coefficient A1. The coefficient A1 varied from 10^{-4} in undeveloped subcatchments down to 10^{-6} in highly developed subcatchments. The saturated zone was allowed to vary by 10 ft in all subcatchments. Some water loss from the saturated zone was allowed by deep percolation and evapotranspiration. Deep percolation was assigned a rate of 0.004 in/h for all subcatchments. The potential ground-water evapotranspiration was assigned a value of 40 percent of the potential evapotranspiration over a depth of 10 ft for all subcatchments. The saturated hydraulic conductivity variable for ground-water discharge was set to the same value as that for the soil HYCON value. Values for ground-water variables were consistent for similar soil and land-use types. Data on ground-water levels relative to storm-drain elevations during storms were not available; therefore, the ground-water variable values were calibrated to the observed hydrograph recessions. Data required for simulation of head-dependent ground-water discharge to storm drains were not available.

The saturated hydraulic conductivity for the multifamily land-use subbasin was set artificially low at 0.01 in/h to account for impervious areas that drain to pervious areas (non-effective impervious area). Lowering the saturated hydraulic conductivity, in effect, increases the response of the pervious areas to rainfall because the model does not simulate the inflow to pervious areas from non-effective impervious areas.

Representation of the Drainage Network

The drainage system was represented in the TRANSPORT module of SWMM. The actual drainage system (appendix 1) was simplified in the models to provide sufficient connectivity between subcatchments and to represent travel time from the headwater subcatchments to the outfall. For instance, the Laundry Brook Subbasin has 85.7 mi of storm drains, but the modeled drainage system includes only 6.2 mi of storm drains that represent the storm-sewer trunk lines. Pipe lengths and diameters were taken from available geographic data supplied by the municipalities.

In the TRANSPORT module, links and nodes must be assigned unique identification numbers. Node numbers identify junctions or outfalls, and link numbers identify conduits. Node numbers at junctions in the TRANSPORT module were assigned in the RUNOFF module (NGTO) for each subcatchment. Junction node numbers (ranging from 1 to 163) were assigned in ascending order from upstream to downstream in a subbasin and from upstream to downstream along the Charles Basin. Nodes that represent outfalls were assigned a 3-digit identification number, where the last two digits correspond to the outfall number shown in figure 6. The first digit of the outfall node-identification number generally begins with a 2, but the first digit was numbered sequentially for subcatchments that drain by sheet runoff and are identified by a letter suffix (fig. 6). For example, nodes representing outfalls 41 (known pipe outfall), 41a, 41b, or 41c (sheet runoff) were assigned numbers of 241, 341, 441, and 541, respectively. Links (conduits) were assigned the same identification number as the node that they drain from plus one thousand. For example, the pipe that drains from node 55 is numbered 1055.

Drainage Network Parameterization

Initial variable values for the drainage network were obtained from spatial-coverage attribute values or from literature values. The TRANSPORT module variables that describe the drainage system characteristics (E1 line) are listed in table 6. Only the length of the conduit system was changed appreciably during model calibration to match the simulated to the observed time of peak. Other drainage network variable values (in the RUNOFF module) were fixed or only slightly modified during model calibration (table 7). Only single-barrel pipe or rectangular-box conduits were simulated in the models developed by the USGS.

Table 6. StormWater Management Model (SWMM) TRANSPORT module variable values used in the lower Charles River Watershed, Massachusetts

[GIS, geographic information system—values were measured or obtained from attribute information. Minor, the values were adjusted only slightly during model calibration; ft, foot; ft/ft, feet per foot]

Variable	Variable description	Initial value	Calibrated
DIST	Length of conduit (ft)	GIS	Yes
GEOM1	Pipe diameter or rectangular conduit height (ft)	GIS	Minor[1]
SLOPE	Invert slope of conduit (ft/ft)	Measured, GIS [2]	No
ROUGH	Manning's roughness value of conduit	Rating[3]	No
GEOM2	Rectangular conduit width (ft)	GIS	Minor

[1] Applies only to pipe diameters in small ungaged subcatchments.

[2] Invert slopes at gaged locations were either measured or estimated from GIS data. Invert slopes at all other locations were estimated from the ratio of the conduit slope at the gaging station to its nearby subbasin slope and applying this ratio to the subbasin slope at ungaged locations.

[3] Manning's roughness values were determined from ratings developed at gaging stations. A characteristic roughness value of 0.012 was used at all ungaged locations.

Table 7. StormWater Management Model (SWMM) RUNOFF subcatchment variable values used in the land-use subbasins, lower Charles River Watershed, Massachusetts

["n," Manning's roughness coefficient; ft, foot; ft/ft, feet per foot; in, inch; in/hr, inch per hour]

RUNOFF variables	Single family	Multi-family	Commercial
Percent impervious area	17	73	86
Subcatchment width (ft)	2,100	1,200	2,500
Slope (ft/ft)	.043	.02	.031
Impervious area depression storage (in.)	.08	.03	.03
Pervious area depression storage (in.)	.16	.04	.03
Impervious area roughness (n)	.015	.02	.015
Pervious area roughness (n)	.040	.35	.46
Suction (in.)	8.2	8.0	5.0
Hydraulic conductivity (in/hr)	.30	.01	.06

Pond Storage

Storage nodes were used in the TRANSPORT module to simulate discharge through Bulloughs Pond in the Laundry Brook Subbasin model and Chandler Pond in Faneuil Brook Subbasin model (fig. 10). Storage nodes are defined by the depth-storage-discharge relation. Chandler Pond's storage characteristics were obtained from an unpublished report by Fugro East, Inc. (1996). The report does not include the discharge characteristics of the outlet, which was estimated by model calibration. During the 2000 water year, Chandler Pond normally was empty for restoration and was treated as a flow-through system (Timothy Smith, Massachusetts District Commission, personal commun., 2000), except when the discharge exceeded the outflow capacity of the drainpipe and was held in temporary storage.

Storage characteristics of Bulloughs Pond were estimated from the average depth (Jay Fink, Newton Public Works, written commun., 2000); the surface area of the pond was obtained from the land-use coverage. The depth-storage-discharge characteristics specified for Bulloughs Pond were estimated through model calibration.

Model Calibration

The general approach used was to calibrate models at the three land-use sites (single-family residential, multifamily residential, and commercial), then transfer the calibrated variable values from these models (table 7) to similar subcatchments in the Laundry Brook and Faneuil Brook Subbasin models. Values were transferred by a weighted average because the subcatchments in the tributary subbasin models represented mixed land use. Further adjustments of the values in tributary subbasin models were made to minimize the difference between simulated and observed discharges. These adjustments were made proportionally to the initial weighted values transferred from the land-use subbasin models. The final calibrated values for individual subcatchments in the tributary subbasin models represented a wide mix of land uses; these values were transferred to the closest subcatchment type in the ungaged subbasin model.

Model calibration focused on selected storms (up to 24) with the smallest spatial variation in total storm rainfall volume among the 6 gages in and near the study area (calibration storms are listed in appendix 3). This approach increases the likelihood that the calibrated model variables reflect the subbasin characteristics they are intended to represent. Measured discharge was not available for all selected storms for some subbasins; therefore, the number of calibration storms varied from site to site. Model calibration emphasized storm-runoff volumes, but storm-peak discharge and base flow also were considered. The model fit was evaluated for the calibration storms and for all storms of appreciable size at each site. All storms were used to test or "verify" the models. Limiting the analysis to storms not used in the model calibration would bias the fit to storms with a large spatial rainfall variability. Unequal rainfall distribution cannot be well represented by a few rainfall gages; therefore, the model fit for these storms would not be a good measure of the model's conceptualization or calibration.

Model Fit

Model fit is a measure of how well the simulated storm-peak discharge and runoff volumes match the observed values. Various measures of model fit are reported for calibration storms and for all storms including absolute measures of model error, relative measures of fit, and several supporting measures of fit. Absolute error is reported as the standard error of estimate (SE) and the RMSE, which were calculated by the SWMM calibration wizard. The relative model fit is measured by the coefficient of efficiency and the index of agreement (Legates and McCabe, 1999).

Other statistical measures and scatter plots of paired simulated and observed values are provided to assess model performance. Supporting statistics include the median, range, and standard deviation of the percent difference between simulated and observed values, and the percent of the time these values are within 10, 25, and, 50 percent. Model errors were examined to determine if there were systematic biases related to time of year or storm characteristics. Causes for the largest error between simulated and observed peak discharges and storm volumes also were examined for each model.

Land-Use Subbasin Models

The three land-use subbasins represent 60 percent of the land use in the lower Charles River Watershed. Variable values obtained from the calibrated models developed at these subbasins provided the initial values for subcatchments of similar land use in tributary subbasin models and the ungaged subbasin model. The calibrated subbasin-variable values at the three land-use subbasins are summarized in table 7.

Single-Family Residential

Calibration Storms: Twenty storms were used for model calibration; total rainfall during these storms ranged from 0.31 to 1.80 in. (mean 0.60 in.), which resulted in observed discharges that ranged from 1.6 to 59 ft^3/s (mean of 12 ft^3/s) in peak discharge and 0.027 to 0.302 in. (mean of 0.104 in.) in runoff volume. The simulated peak discharges had a SE of 7.2 ft^3/s and a RMSE of 17 ft^3/s (table 8); the difference between simulated and observed peaks ranged between -46 and 150 percent with a median difference of -19 percent and a standard deviation of 47 percent. Simulated peak discharges were within 10 percent of the observed peak 25 percent of the time, within 25 percent of the observed peak 35 percent of the time, and within 50 percent of the observed peak 90 percent of the time. The scatter plot (fig. 12*A*) and model-fit statistics (table 8) indicate that the simulated peak discharge is undersimulated.

The difference between the simulated and observed runoff volumes had a SE of 0.032 in. and a RMSE of 0.109 in. (table 8); the difference between simulated and observed runoff volumes ranged between -16 and 281 percent, with a median difference of 29 percent and a standard deviation of 67 percent. Simulated runoff volumes were within 10 percent of the observed volume 30 percent of the time, within 25 percent of the observed volume 45 percent of the time, and within 50 percent of the observed volume 70 percent of the time. Good agreement between simulated and observed runoff volume is indicated for storms with more than 0.15 in. of runoff; storms with less than 0.15 in. of runoff tend to be oversimulated (fig. 12*A*).

All Storms: Forty-nine storms were used to evaluate the overall model fit; total rainfall during these storms ranged from 0.01 to 3.64 in. (mean 0.60 in.) that resulted in observed discharges that ranged in peak discharge from 1.0 to 66 ft^3/s (mean of 14 ft^3/s) and volumes from 0.013 to 0.862 in. (mean of 0.135 in.). The simulated peak discharge had a SE of 46 ft^3/s and a RMSE of 97 ft^3/s (table 8); the difference between simulated and observed peak discharges ranged between -90 and 493 percent with a median difference of -18 percent and a standard deviation of 101 percent. Simulated peak discharges were within 10 percent of the observed peak 30 percent of the time, within 25 percent of the observed peak 45 percent of the time, and within 50 percent of the observed peak 70 percent of the time (table 8). A large variability between simulated and observed peak discharge is indicated for most storms (fig. 12*B*).

The difference between the simulated and observed storm runoff volumes for all storms had a SE of 0.121 in. and a RMSE of 0.362 in. (table 8); the difference between simulated and observed runoff volumes ranged between –52 and 502 percent with a median difference of 18 percent and a standard deviation of 95 percent. Simulated storm runoff volumes were within 10 percent of the observed volume 28 percent of the time, within 25 percent of the observed volume 51 percent of the time, and within 50 percent of the observed volume 72 percent of the time. Simulated runoff volume is in good agreement with the observed runoff volume over the entire range of storms (fig. 12*B*).

Error: Model error, for both peak discharge and runoff volume, was largest in the summer months (fig. 13*A*), which tend to be dominated by convective storms with rainfall amounts that can vary widely over a watershed. Model error was not correlated with rainfall volume, intensity, or antecedent conditions (figs. 13*B*, *C*, and *D*), as measured by the Spearman

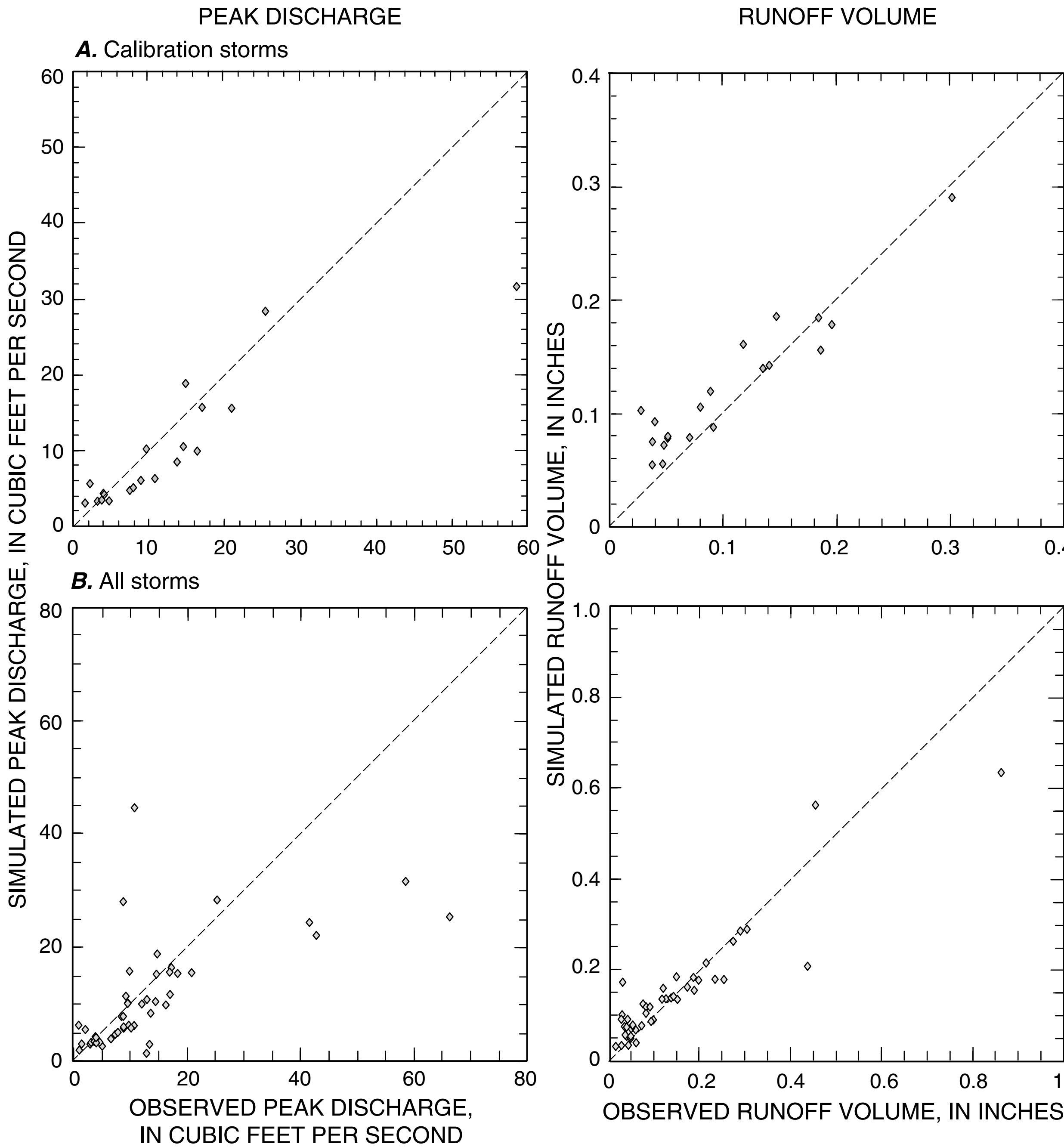


Figure 12. Simulated and observed peak discharge and runoff volume at the single-family land-use subbasin, lower Charles River Watershed, Massachusetts, for the 2000 water year: (*A*) calibration storms, and (*B*) all storms.

rank test. The tendency to undersimulate storm peak discharges and oversimulate runoff volumes indicated by the plotting position of smaller calibration storms (fig. 12*A*) suggests that subcatchment width and the interception storage could be increased; this increase was not done, however, because of the good agreement between simulated and observed runoff volumes for all storms. Undersimulated peak discharge for the larger storms (fig. 12*B*) could be a result of error in the observed discharge resulting from debris on the security gate at the pipe exit causing backwater and an overmeasurement of the actual peak discharge.

Table 8. Summary of StormWater Management Model (SWMM) fit statistics for storm-peak discharge and storm-runoff volume, lower Charles River Watershed, Massachusetts

[**No.:** Number of storms used to compute statistics. Standard error of the estimate **(SE)** and root mean square error **(RMSE)** are in units of cubic feet per second for peak discharge and inches for runoff volume. **E:** Coefficient of efficiency. **d:** Index of agreement. ft^3/s, cubic feet per second]

Site	No.	SE	RMSE	E	d	Percent time simulated value is within observed value		
						10	25	50
Storm-peak discharge								
Calibration Storms								
Land-use subbasin models								
Single family	20	7.2	17	0.69	0.88	25	35	90
Multifamily	21	2.0	6.0	.84	.96	29	62	95
Commercial	13	2.0	5.1	.81	.95	23	46	85
Tributary subbasin models								
Laundry Brook	24	15	48	.73	.94	29	58	92
Faneuil Brook	21	8.9	27	.35	.87	25	50	80
Stony Brook	17	41	120	.83	.96	18	47	71
All Storms								
Land-use subbasin models								
Single family	49	46	97	.81	.94	30	45	70
Multifamily	36	2.6	10	.71	.93	25	58	86
Commercial	31	3.0	10	.27	.83	26	52	77
Commercial[1]	10	2.5	4.8	-.01	.72	22	67	78
Tributary subbasin models								
Laundry Brook	48	22	85	.70	.94	21	60	81
Faneuil Brook	48	15	64	-.33	.76	19	44	65
Stony Brook	28	80	260	.84	.97	14	43	79
Storm-runoff volume								
Calibration Storms								
Land-use subbasin models								
Single family	20	0.032	0.109	0.81	0.94	30	45	70
Multifamily	21	.088	.303	.94	.98	43	67	95
Commercial	13	.407	.794	.66	.27	31	69	92
Tributary subbasin models								
Laundry Brook	24	.016	.061	.91	.97	46	67	87
Faneuil Brook	21	.029	.094	.59	.92	40	75	95
Stony Brook	17	.048	.139	.32	.74	29	53	76
All Storms								
Land-use subbasin models								
Single family	49	.121	.362	.83	.94	28	51	72
Multifamily	36	.220	.640	.93	.98	33	50	89
Commercial	31	.941	2.19	.40	.67	26	58	90
Commercial[1]	10	.548	1.02	.69	.87	22	67	100
Tributary subbasin models								
Laundry Brook	48	.030	.130	.90	.97	33	65	83
Faneuil Brook	48	.125	.602	.86	.97	31	69	96
Stony Brook	28	.070	.210	.90	.98	21	54	82

[1]Subset of storms with rainfall/runoff coefficients less than 1.3.

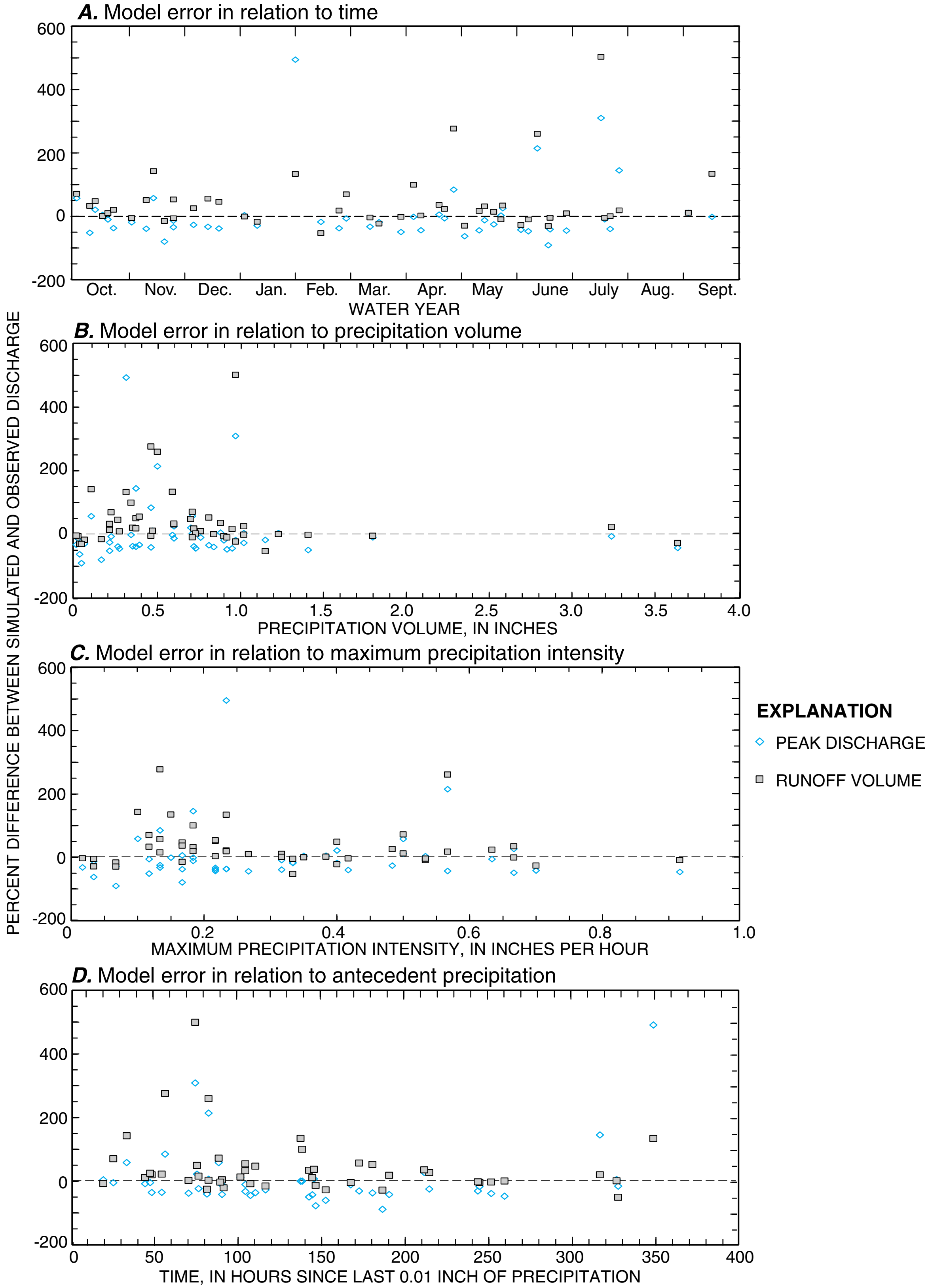


Figure 13. Percent difference between simulated and observed storm-peak discharge and runoff volume for the single-family land-use subbasin, lower Charles River Watershed, Massachusetts, for 49 storms in the 2000 water year: (*A*) by month, and in relation to (*B*) precipitation volume, (*C*) precipitation intensity, and (*D*) antecedent precipitation.

The largest model errors can be attributed to several factors. The January 31, 2000, storm had the most oversimulated peak discharge (493 percent) and one of the larger oversimulated runoff volumes (134 percent). Cold weather prior to this storm may have affected the stage readings. The storm of April 26, 2000, produced the second highest simulated runoff-volume error (276 percent); this storm followed a wet period (3.54 in. of rainfall during the previous week), which suggests that there was less available storage in the soil then the calibrated model variables allowed. The July 22, 2000, storm produced the highest simulation runoff volume error (502 percent); this storm had one of the highest maximum rainfall intensities and the second highest variability in total rainfall volume among the six gages. The June 27 storm had one of the largest negative errors in the both peak discharge and runoff volume; total rainfall volume during this storm measured at the six gages had the largest variability during the 2000 water year. The USGS-CR3 gage assigned to this subcatchment had the lowest total rainfall among the six gages. Analysis of model error indicates that much of the difference between simulated and observed discharges results from variation in rainfall; thus, further calibration would not appreciably improve the model simulations.

Some of the largest model errors, for both storm-peak discharge and runoff volume, were observed in storms with more than one pulse of rainfall resulting in multiple rises in the storm hydrograph with main peak discharge occurring during the later part of a storm. During these storms (March 28, May 10–11, June 6, and June 11–12) the second peak discharge was undersimulated, which also resulted in an undersimulation of storm volumes. SWMM resets the soil moisture deficit if there is a sufficient period without rainfall between peaks; thus, more moisture is allowed to infiltrate in pervious areas than would infiltrate if the soil moisture deficit were not reset. An option in SWMM to allow a maximum amount of infiltration during a storm likely would better represent runoff during these types of storms; however, this option currently is not available for a continuous simulation using the Green-Ampt infiltration option.

Multifamily Residential

Calibration Storms: Twenty-one storms from November 25, 1999, to January 11, 2000, and from March 27, 2000, to September 30, 2000, were used for model calibration; total precipitation during these storms ranged from 0.27 to 1.74 in. (mean 0.69 in.), which resulted in observed discharges that ranged from 1.3 to 20 ft^3/s (mean of 6.5 ft^3/s) in peak discharge and 0.142 to 1.46 in. (mean of 0.515 in.) in runoff volume. Storms of April 4 and April 8, 2000, were not used to calibrate or evaluate the model fit because the observed measurements were erratic. The simulated peak discharge had a SE of 2.0 ft^3/s and a RMSE of 6.0 ft^3/s (table 8); the difference between simulated and observed peaks ranged between -41 and 90 percent with a median difference of -0.6 percent and a standard deviation of 30 percent. Simulated peak discharges were within 10 percent of the observed peak 29 percent of the time, within 25 percent of the observed peak 62 percent of the time, and within 50 percent of the observed peak 95 percent of the time. Simulated peak discharge generally is in good agreement with the observed peak for most calibration storms (fig. 14*A*).

The difference between the simulated and observed runoff volumes had a SE of 0.088 in. and a RMSE of 0.303 in. (table 8); the difference between simulated and observed volumes ranged between -33 and 100 percent with a median difference of -1.2 percent and a standard deviation of 30 percent. Simulated runoff volumes were within 10 percent of the observed volume 43 percent of the time, within 25 percent of the observed volume 67 percent of the time, and within 50 percent of the observed volume 95 percent of the time. Simulated runoff volume is in good agreement with the observed volume for all calibrated storms (fig. 14*A*).

All Storms: Thirty-six storms were used to evaluate the overall model fit. Discharge data were unavailable prior to November 25, 1999, and from January 11, 2000, to March 27, 2000. Storms on April 4 and April 8, 2000, had erratic observed discharge and were not used in the evaluation of the model fit. Total rainfall during these storms ranged from 0.13 to 3.58 in. (mean 0.75 in.) that resulted in observed discharges that ranged from 0.9 to 20 ft^3/s (mean of 5.9 ft^3/s) in peak discharge and 0.073 to 4.44 in. (mean of 0.619 in.) in runoff volume. The simulated peak discharge for all storms had a SE of 2.6 ft^3/s and a RMSE of 10 ft^3/s (table 8); the difference between simulated and observed peaks as a percent ranged between -68 and 214 and with a median difference of 7.5 percent and a standard deviation of 47 percent. Simulated peak discharges were within 10 percent of the observed peak 25 percent of the time, within 25 percent of the observed peak 58 percent of the time, and within 50 percent of the observed peak 86 percent of the time. Simulated peak discharge generally is in good agreement with the

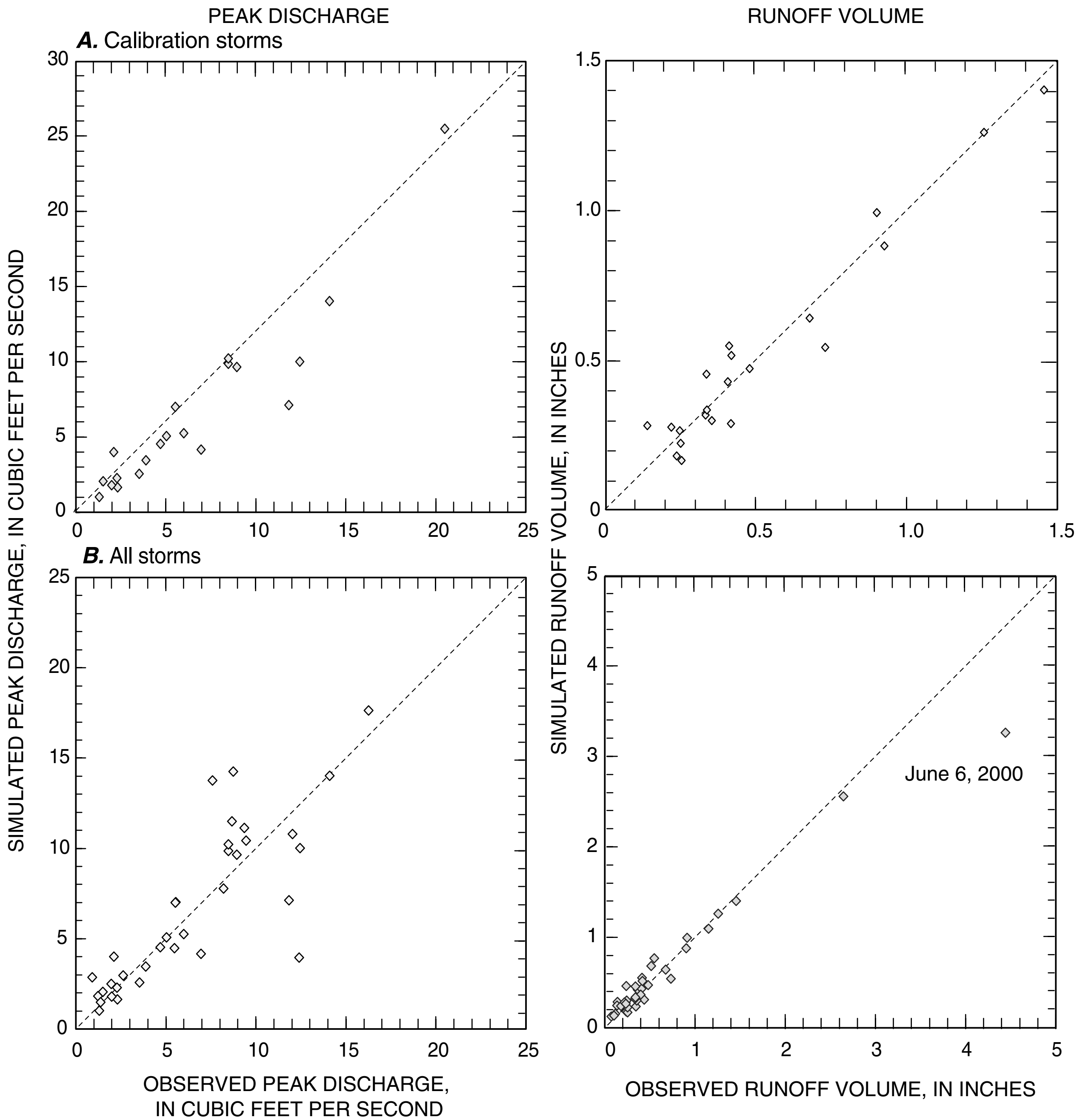


Figure 14. Simulated and observed peak discharge and runoff volume at the multifamily land-use subbasin, lower Charles River Watershed, Massachusetts, for the 2000 water year: (*A*) calibration storms, and (*B*) all storms.

observed peak (fig. 14*B*), but there is more variability between simulated and observed peak discharges than in the calibration storms (fig. 14*A*).

The difference between the simulated and observed storm-runoff volumes had a SE of 0.22 in. and a RMSE of 0.64 in. (table 8); the difference between simulated and observed volumes ranged between -33 and 100 percent with a median difference of 2.8 percent and a standard deviation of 34 percent. Simulated storm-runoff volumes were within 10 percent of the observed volume 33 percent of the time, within 25 percent of the observed volume 50 percent of the time, and within 50 percent of the observed volume 89 percent of the time. Simulated runoff volume closely matches the observed volume for most storms (fig. 14*B*).

Error: Differences between peak discharge and runoff volume tended to be largest during the summer months (fig. 15*A*). Model error was not correlated with rainfall volume, intensity, and antecedent conditions (figs. 15*B*, *C*, and *D*) as measured by the Spearman rank test.

Errors, in percent, typically were largest for small storms with rainfall volume less than 0.40 in. Most model error was attributed to rainfall variability across the watershed; for example, the storm of June 6 (the largest outlier in runoff volume shown of fig. 14*B*) had a simulated volume of 3.26 in. and observed volume of 4.44 in., and varied by ± 0.5 in. at the six rain gages. In addition, the drainage area, which has a topographic drainage divide that differs from the storm drain drainage divide (appendix 1), could have been affected by the storm magnitude. During large or high-intensity storms, such as the storm of June 6, runoff could follow topographic divides if storm drains that normally discharge out of the subbasin are blocked or are at capacity; thus, the drainage-area size could increase temporarily. The storm of January 31 had the largest oversimulated peak discharge (214 percent). Reported air temperatures at Logan Airport were below 32°F for part of this day, which likely resulted in some rainfall falling as snow and, therefore, less moisture was available for runoff.

Commercial

Rainfall-runoff (RR) relations at the commercial land-use subbasin varied widely, often yielding coefficients greater than one, which are not typical because it means that runoff exceeded the amount of rainfall. In a natural flow system, runoff rarely exceeds rainfall except in some situations where the ground-water drainage area is appreciably larger than the surface-water drainage area. Other explanations for the high RR coefficients include discharge or rainfall measurement error, or both, external sources of water that drain to the storm-sewer system, a variable drainage area not strictly defined by the subsurface drainage system, or a combination of these factors. In a highly urbanized area, such as this subbasin, external sources of water such as leakage or cross connections from the sanitary-sewer system often are an additional source of water in storm drains. The topographic drainage area is about 50 percent larger (drainage area increases from 0.023 to 0.034 mi^2) than the drainage area delineated from the storm drainage system (appendix 1). During large or high-intensity storms, drains that normally flow out of the subbasin could become blocked or at capacity; thus, runoff would follow the topographic drainage divides, which could cause the wide variation in RR coefficients. For this reason, the commercial land-use subbasin model was calibrated with preference given to storms with RR coefficients less than 1.0. Model-fit statistics are reported for all available storms regardless of the RR coefficient, but the effect of storms with high RR coefficients on the model fit are also discussed.

Observed discharge is suspect for most storms during late May through June 2000 because the hydrograph recession is uncharacteristically gradual for a highly impervious small subbasin; simulated runoff volumes averaged 39 percent less than the observed runoff volume for storms during this period. Observed storm hydrographs during other periods of the year indicated a return to base-flow conditions within about an hour following the storm peak, which is expected for this type of subbasin. Furthermore, at the commercial land-use site, a nearly constant base flow of 0.2 ft^3/s was observed, which also is unlikely given the high percentage of effective impervious area and the small drainage area (0.023 mi^2) This base flow may result because of presently unidentified discharges to storm drains upgradient of the gage site.

Calibration Storms: Thirteen storms from late April to September 2000 were used for model calibration; total rainfall during these storms ranged from 0.31 to 1.74 in. (mean 0.74 in.), which resulted in observed discharges that ranged from 1.7 to 18 ft^3/s (mean of 6.6 ft^3/s) in peak discharge and from 0.475 to 1.90 in. (mean of 0.908 in.) in runoff volume. The simulated calibration storm peaks had a SE of 2.0 ft^3/s and a RMSE of 5.1 ft^3/s (table 8); the difference between simulated and observed peaks ranged between -39 and 70 percent with a median difference of 2.0 percent and a standard deviation of 35 percent. Simulated peak discharges were within 10 percent of the observed peak 23 percent of the time, within 25 percent of the observed peak 46 percent of the time, and within 50 percent of the observed peak 85 percent of the time. Simulated peak discharge generally is in good agreement with the observed peak discharge for most storms (fig. 16*A*).

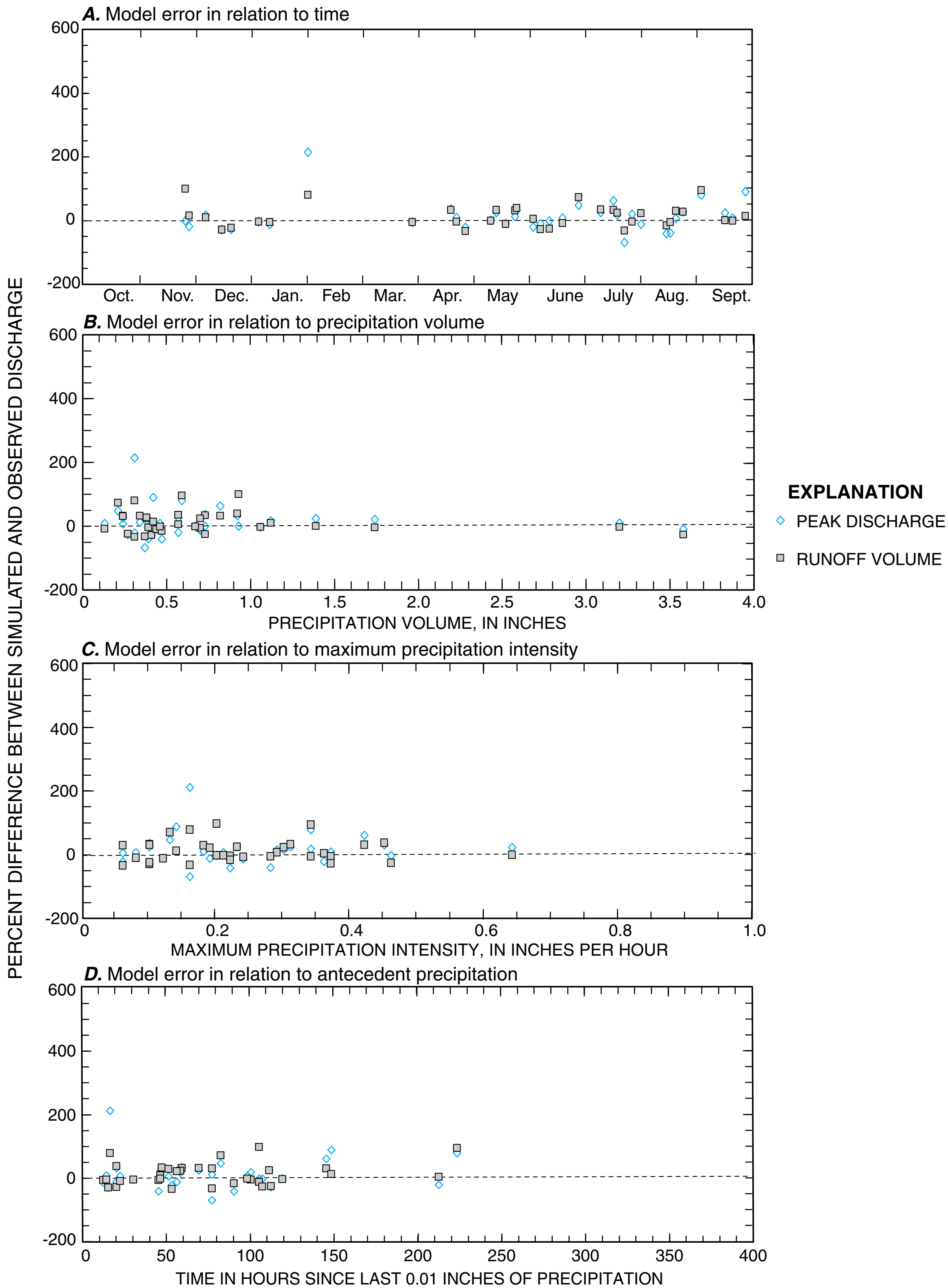


Figure 15. Percent difference between simulated and observed storm-peak discharge and runoff volume for the multifamily land-use subbasin, lower Charles River Watershed, Massachusetts, for 36 storms in the 2000 water year: (*A*) by month, and in relation to (*B*) precipitation volume, (*C*) precipitation intensity, and (*D*) antecedent precipitation.

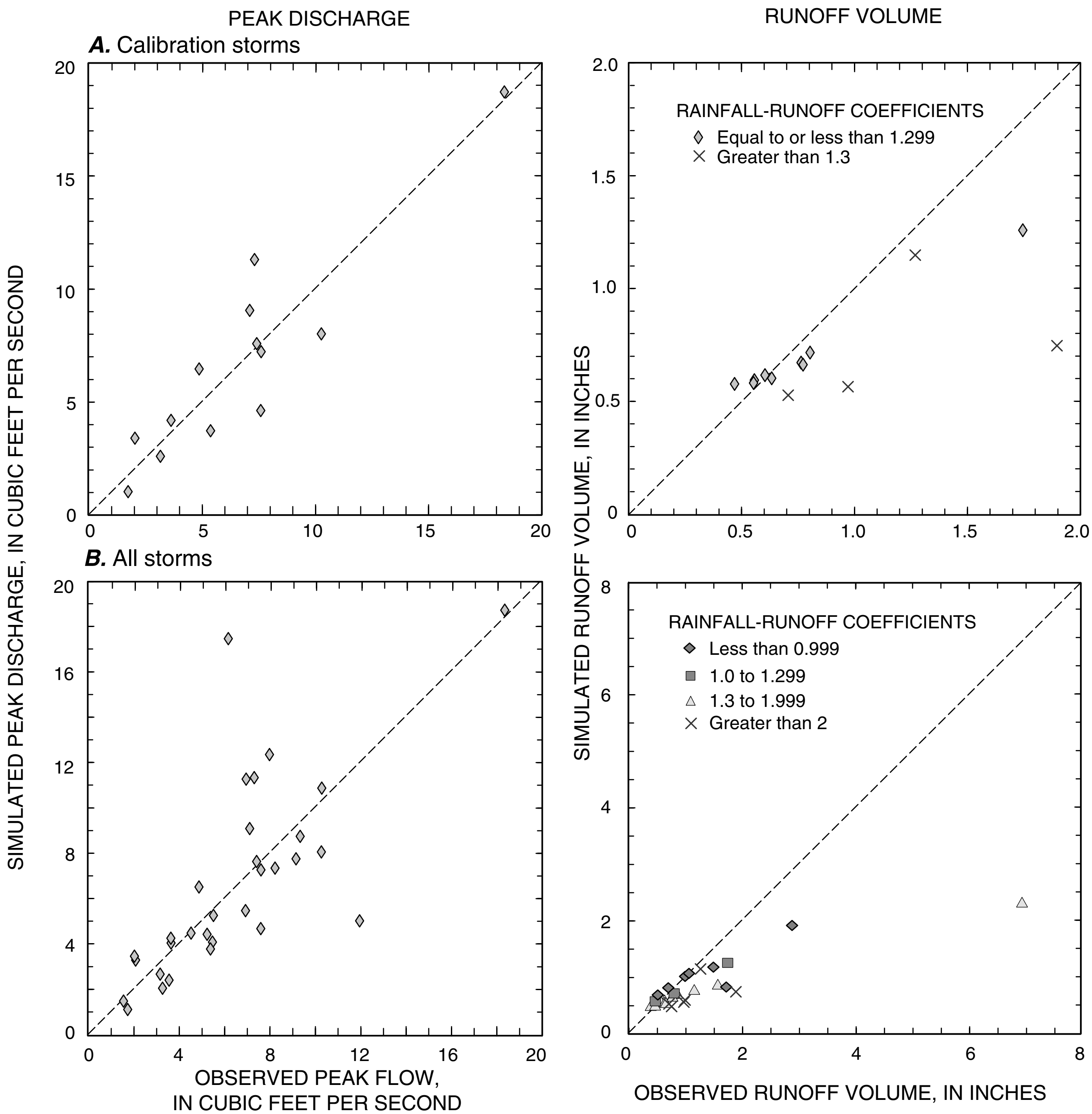


Figure 16. Simulated and observed peak discharge and runoff volume at the commercial land-use subbasin, lower Charles River Watershed, Massachusetts, for the 2000 water year; (*A*) calibration storms, and (*B*) all storms.

The difference between the simulated and observed runoff volumes had a SE of 0.407 in. and a RMSE of 0.794 in. (table 8); the difference between simulated and observed volumes ranged between -61 and 21 percent with a median difference of -11 percent and a standard deviation of 22 percent. Simulated runoff volumes were within 10 percent of the observed volume 31 percent of the time, within 25 percent of the observed volume 69 percent of the time, and within 50 percent of the observed volume 92 percent of the time. Calibration runoff volumes with RR coefficients greater than 1.3 accounted for three of the four storms with poorly fit runoff volumes (fig. 16*A*).

All Storms: Thirty-one storms between February and September 2000 were used to evaluate the overall model fit; total rainfall during these storms ranged from 0.13 to 3.58 in. (mean of 0.80 in.), which resulted in observed discharges that ranged from 1.5 to 18 ft^3/s (mean of 6.4 ft^3/s) in peak discharge and 0.392 to 6.93 in. (mean of 1.17 in.) in runoff volume. The simulated peak discharge had a SE of 3.0 ft^3/s and a RMSE of 10 ft^3/s (table 8); the difference between simulated and observed peak discharge ranged between -58 and 184 percent with a median difference of -4.6 percent and a standard deviation of 47 percent. Simulated peak discharges were within 10 percent of the observed peak 26 percent of the time, within 25 percent of the observed peak 52 percent of the time, and within 50 percent of the observed peak 77 percent of the time. The scatter plot generally indicates a good agreement between the simulated and observed peak discharge (fig. 16*B*).

The difference between the simulated and observed runoff volumes had a SE of 0.941 in. and a RMSE of 2.19 in. (table 8); the difference between simulated and observed volumes ranged from -66 and 33 percent with a median difference of -13 percent and a standard deviation of 25 percent. Simulated runoff volumes were within 10 percent of the observed volume 26 percent of the time, within 25 percent of the observed volume 58 percent of the time, and within 50 percent of the observed volume 90 percent of the time. Simulated and observed runoff volumes were in good agreement for storms with RR coefficients less than 1.3 (fig. 16*B*).

Error: Of the 31 possible storms, only 10 storms had RR coefficients less than 1.3, which were used to reevaluate the model fit. These storms ranged in rainfall volume from 0.59 to 3.20 in. (mean of 1.31 in.), which resulted in discharges that ranged from 3.6 to 8.2 ft^3/s (mean of 6.2 ft^3/s) in peak discharge and from 0.519 to 2.88 in. (mean of 1.33 in.) in runoff volume.

The differences between the simulated and observed storm peak discharge for the subset of storms declined slightly in comparison to all storms. The simulated peak discharge for the subset of storms had a SE of 2.50 ft^3/s and a RMSE of 4.75 ft^3/s (table 8); the difference between simulated and observed volumes ranged between -21 and 62 percent with a median difference of -0.89 percent and a standard deviation of 30 percent. Simulated peak discharges were within 10 percent of the observed peak 22 percent of the time, within 25 percent of the observed peak 67 percent of the time, and within 50 percent of the observed peak 78 percent of the time.

The difference between the simulated and observed runoff volumes for the subset of storms substantially improved in comparison to all storms. The simulated runoff volume had SE of 0.548 in., and a RMSE of 1.021 in., which was about half the absolute error for all storms; the difference between simulated and observed runoff volumes ranged between -33 and 33 percent with a median difference of 0.36 percent and a standard deviation of 22 percent. Simulated runoff volumes were within 10 percent of the observed volume 22 percent of the time, within 20 percent of the observed volume 67 percent of the time, and within 50 percent of the observed volume 100 percent of the time.

Differences between simulated and observed peak discharges were about equally scattered from February to September (fig. 17*A*); however, runoff volumes tended to be undersimulated in May and June when RR coefficients tended to be high. Model error was not correlated with rainfall volume, intensity, and antecedent conditions (figs. 17*B, C*, and *D*) as measured by the Spearman rank test. Model error is largely attributed to uncertainty in the observed discharge, or one or more of the other factors that could cause large RR coefficients.

Tributary Subbasin Models

The model fit is described for the tributary subbasin models at Laundry Brook and Faneuil Brook developed by the USGS and the Stony Brook model developed by others. The Muddy River portion of the Stony Brook model was not evaluated for model fit because of the high degree of uncertainty in the observed discharge data.

Laundry Brook

The model fit at this site can be affected by the dynamic regulation of Bulloughs Pond. Bulloughs Pond has a surface area of 4.7 acres and an estimated storage volume of 0.5 to 0.75 million ft^3. At times, measured discharge at the Laundry Brook gaging station was sustained and elevated prior to storms, possibly because water was being released from Bulloughs Pond. SWMM does not account for dynamic regulation of storage, thus, the model fit can be affected when the pond level is lowered in anticipation of a storm. Storms that are affected by regulation are noted, and the effect of this regulation on the model fit is described.

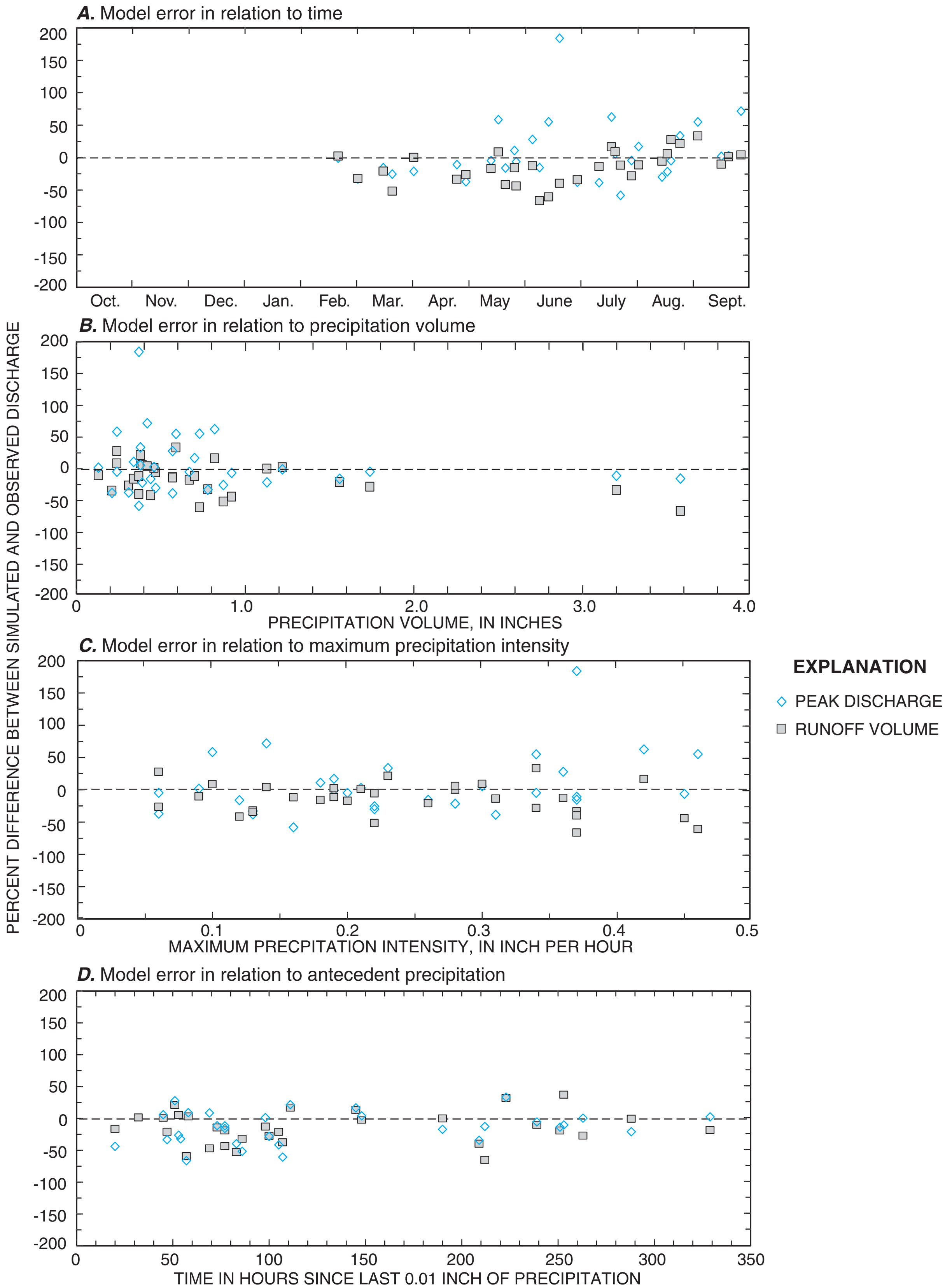


Figure 17. Percent difference between simulated and observed storm-peak discharge and runoff volume for the commercial land-use subbasin, lower Charles River Watershed, Massachusetts, for 31 storms in the 2000 water year: (*A*) by month, and in relation to (*B*) precipitation volume, (*C*) precipitation intensity, and (*D*) antecedent precipitation.

Calibration Storms: Twenty-four storms were used for model calibration; total rainfall during these storms ranged from 0.35 to 1.83 in. (mean 0.76 in.), which resulted in observed discharges that ranged from 10.9 to 100.8 ft^3/s (mean of 41.7 ft^3/s) in peak discharge and 0.028 to 0.231 in. (mean of 0.083 in.) in runoff volume. The simulated peak discharge had a SE of 15.2 ft^3/s and a RMSE of 47.5 ft^3/s (table 8); the difference between simulated and observed peak discharges ranged between –38 and 103 percent with a median difference of 14.6 percent and a standard deviation of 29 percent. Simulated peak discharges were within 10 percent of the observed peak 29 percent of the time, within 25 percent of the observed peak 58 percent of the time, and within 50 percent of the observed peak 92 percent of the time. Simulated and observed peak discharges are generally in good agreement; the largest peaks were affected by regulation of Bulloughs Pond (fig. 18*A*).

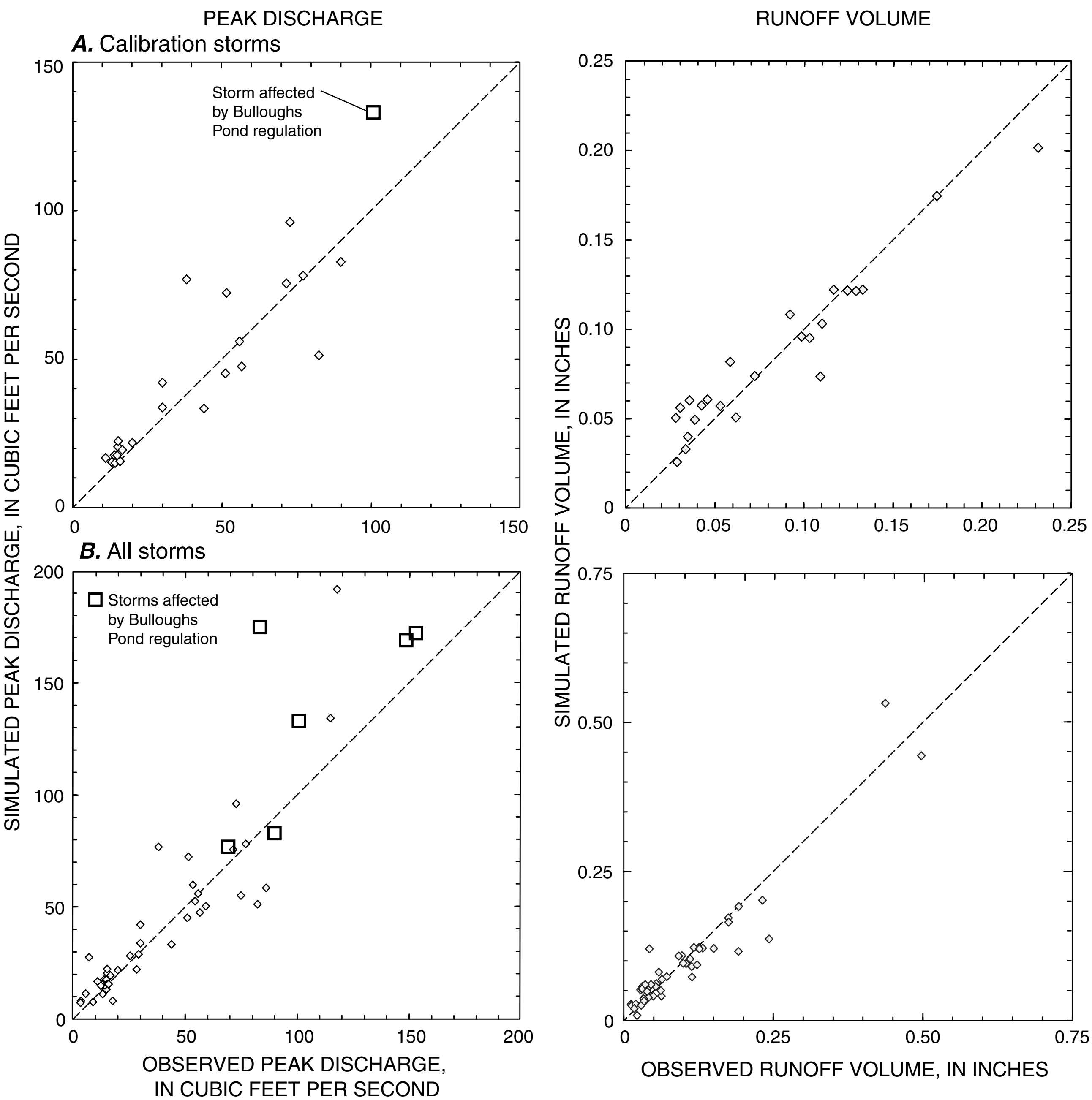


Figure 18. Simulated and observed discharge and runoff volume at the Laundry Brook Subbasin, lower Charles River Watershed, Massachusetts, for the 2000 water year: (*A*) calibration storms, and (*B*) all storms.

The difference between the simulated and observed storm runoff volume had a SE of 0.016 in. and a RMSE of 0.061 in. (table 8); the difference between simulated and observed volumes ranged from –33 to 81 percent, with a median difference of 1.0 percent and a standard deviation of 30 percent. Simulated runoff volumes were within 10 percent of the observed volume 46 percent of the time, within 25 percent of the observed volume 67 percent of the time, and within 50 percent of the observed volume 87 percent of the time. Simulated runoff volume is generally in good agreement with the observed volume for most storms (fig. 18*A*).

All Storms: Forty-eight storms were used to evaluate the overall model fit; total rainfall during these storms ranged from 0.10 to 3.64 in. (mean 0.73 in.), which resulted in observed discharges that ranged from 3.3 to 153 ft^3/s (mean of 53.7 ft^3/s) in peak discharge and 0.012 to 0.437 in. (mean of 0.081 in.) in runoff volume. The simulated peak discharge had a SE of 21.9 ft^3/s and a RMSE of 85.1 ft^3/s (table 8); the difference between simulated and observed peaks ranged from –54 to 287 percent with a median difference of 11 percent and a standard deviation of 56 percent. Simulated peak discharges were within 10 percent of the observed peak 21 percent of the time, within 25 percent of the observed peak 60 percent of the time, and within 50 percent of the observed peak 81 percent of the time. Simulated peak discharge generally is in good agreement with the observed peak, but several peaks were oversimulated as a result of regulation of Bulloughs Pond (fig. 18*B*).

The difference between the simulated and observed runoff volumes had a SE of 0.030 in. and a RMSE of 0.130 in. (table 8); the difference between simulated and observed volumes ranged from -61 to 181 percent, with a median difference of -0.9 percent and a standard deviation of 44 percent. Simulated storm volumes were within 10 percent of the observed volume 33 percent of the time, within 25 percent of the observed volume 65 percent of the time, and within 50 percent of the observed volume 83 percent of the time. Simulated runoff volume is generally in good agreement with the observed storm volume (fig. 18).

Error: Model performance as a function of time of year, rainfall volume, intensity, and antecedent conditions did not indicate systematic bias in the simulation results (fig. 19). The Spearman rank test indicates that the model error is not correlated with rainfall characteristics or antecedent conditions.

The dates when storms discharge appeared to be affected by regulation of Bulloughs Pond include the March 16–17, April 21–26, July 18–19, July 22 and 26–28, and September 15–16, 2000. The available storage in the pond will affect storm-peak discharges more than it effects storm-runoff volume if the water drained from the pond is included in the runoff volume. For example, if water drained from the pond immediately prior to a storm is included in the total storm volume, the stormwater remaining in the pond at the end of a storm is about equivalent to what was drained; thus, the total storm volume is unchanged. Conversely, storm volume would appear to be oversimulated by the volume of water drained from storage if this volume is not included in the measured runoff volume of the storm. The measured peak discharge is likely always affected when the pond is lowered prior to a storm to allow stormwater to be retained in storage.

Rainfall variability affected the model fit during the July 22 storm, which had one of the largest oversimulated peak discharges (110 percent) and the largest oversimulated runoff volume (181 percent). Measured rainfall at the USGS-CR3 was 0.97 in., but rainfall measured at other gages varied from 0.12 to 1.32 in. Mixed precipitation as rain and snow can also affect the model fit, which may have caused the January 31 storm to have the largest oversimulated peak discharge (287 percent).

Faneuil Brook

The model fit at this site was assessed with discharge data computed from stage-velocity measurements and by a stage-discharge relation during periods with erratic or suspect velocity measurements, as explained previously. If the stage-velocity discharge appeared erroneously low, the computed discharge determined by the stage-discharge relation was used to evaluate the model fit. The reported model-fit statistics do not include the storm of June 6, with a total rainfall of more than 4 in.; the measured volume of runoff during this storm (regardless of the discharge method used) was comparable to runoff for storms that ranged between 0.7 and 1.6 in. of rainfall. Therefore, this storm was excluded from the model fit-statistics because it exerted a large effect on them, and the volume and peak discharge appear to be undermeasured.

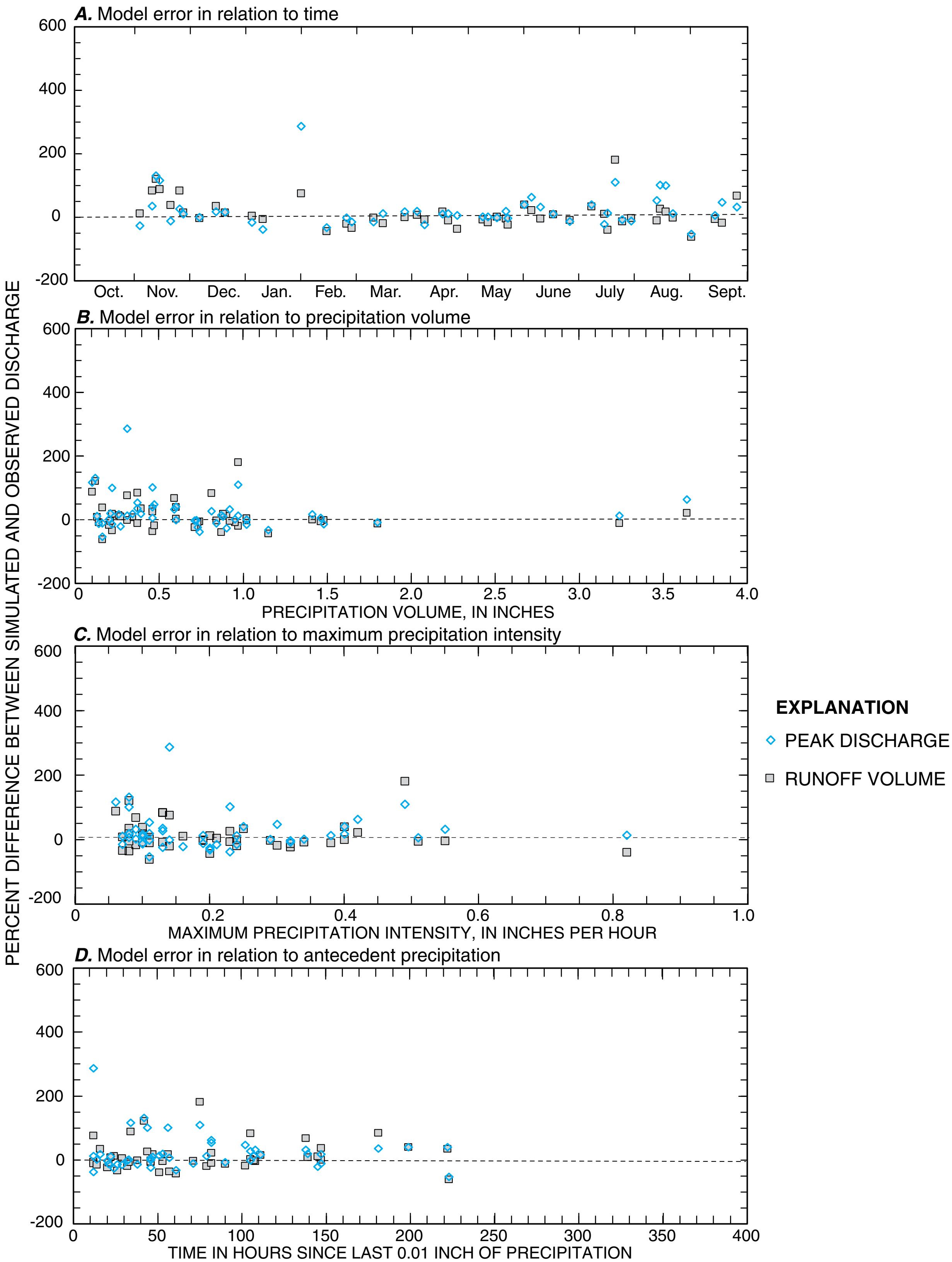


Figure 19. Percent difference between simulated and observed storm-peak discharge and runoff volume for the Laundry Brook Subbasin, lower Charles River Watershed, Massachusetts, for 48 storms in the 2000 water year: (*A*) by month, and in relation to (*B*) precipitation volume, (*C*) precipitation intensity, and (*D*) antecedent precipitation.

Calibration Storms: Twenty-one storms were used for model calibration with total rainfall ranging from 0.27 to 2.10 in. (mean 0.67 in.), which resulted in discharges that ranged from 6.5 to 39 ft^3/s (mean of 19 ft^3/s) in peak discharge and from 0.047 to 0.207 in. (mean of 0.091 in.) in runoff volume. The simulated storm peaks had a SE of 8.9 ft^3/s and a RMSE of 27 ft^3/s (table 8); the difference between simulated and observed peaks ranged between –33 and 89 percent with a median difference of 5 percent and a standard deviation of 22 percent. Simulated peak discharges were within 10 percent of the observed peak 25 percent of the time, within 25 percent of the observed peak 50 percent of the time, and within 50 percent of the observed peak 80 percent of the time. The simulated peak discharge is generally in good agreement with the observed when peaks are less than 20 ft^3/s, but peaks above 20 ft^3/s tended to be more variable (fig. 20*A*).

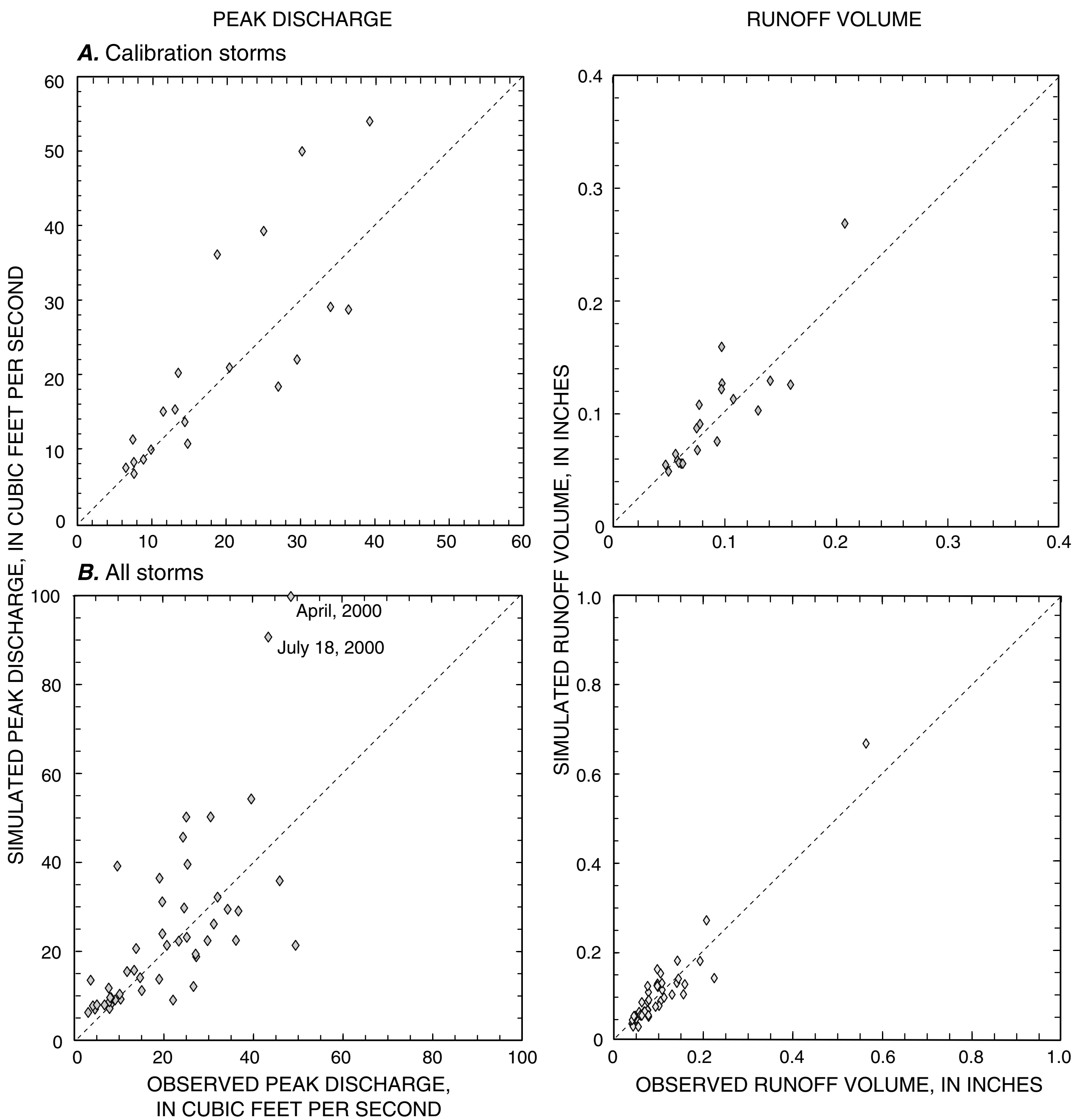


Figure 20. Simulated and observed peak discharge and runoff volume at Faneuil Brook Subbasin, lower Charles River Watershed, Massachusetts, for the 2000 water year: (*A*) calibration storms, and (*B*) all storms.

The difference between the simulated and observed runoff volumes had a SE of 0.029 in. and a RMSE of 0.094 in. (table 8); the difference between simulated and observed storm volumes ranged between -21 and 64 percent with a median difference of 4 percent and a standard deviation of 14 percent. Simulated storm volumes were within 10 percent of the observed volume 40 percent of the time, within 25 percent of the observed volume 75 percent of the time, and within 50 percent of the observed volume 95 percent of the time. Simulated runoff volume in relation to the observed volume indicates that the model fit did not vary with the size of the storm (fig. 20*A*).

All Storms: Forty-eight storms were used to evaluate the overall model fit; total rainfall during these storms ranged from 0.10 to 3.39 in. (mean 0.69 in.), which resulted in observed discharges that ranged from 2.8 to 49 ft^3/s (mean of 20 ft^3/s) in peak discharge and 0.040 to 0.566 in. (mean of 0.102 in.) in runoff volume. The simulated peak discharge had a SE of 15 ft^3/s and a RMSE of 64 ft^3/s (table 8); the difference between simulated and observed peak discharges ranged between –61 and 314 percent with a median difference of 11 percent and a standard deviation of 73 percent. Simulated peak discharges were within 10 percent of the observed peak 19 percent of the time, within 25 percent of the observed peak 44 percent of the time, and within 50 percent of the observed peak 65 percent of the time. Simulated peaks varied considerably from the observed peaks, but, in general, the model fit did not vary with the peak discharge size (fig. 20*B*).

The difference between the simulated and observed runoff volumes had a SE of 0.125 in. and a RMSE of 0.602 in. (table 8); the difference between simulated and observed volumes ranged between -42 and 65 percent with a median difference of -3 percent and a standard deviation of 24 percent. Simulated runoff volumes were within 10 percent of the observed volume 31 percent of the time, within 25 percent of the observed volume 69 percent of the time, and within 50 percent of the observed volume 96 percent of the time. Simulated runoff volume is generally in good agreement with the observed volume for most storms (fig. 20*B*).

Error: The June and July storms tended to have larger error than the storms during other times of the year (fig. 21*A*); the storms during this period also tended to have the greatest spatial rainfall variability. Model error was not correlated with rainfall volumes, intensity, and antecedent conditions (figs. 21*B*, *C*, and *D*), except for a weak correlation (31 percent of the model error is explained) with rainfall intensity as measured by the Spearman rank test.

Storms of April 22 and the July 18 had the largest differences between simulated and observed peak discharge. The April 22 storm had the second largest rainfall volume (3.4 in.) and the July 18 storm had the highest rainfall intensity (0.55 in/h) recorded during the 2000 water year. Upstream surcharging of the drainage system could be a factor in limiting the observed peak discharge during these storms.

Stony Brook

The Stony Brook model has undergone several transformations. The model fit described is for the version of the model that was supplied by M&E consultants (Lawrence Soucie, Metcalf and Eddy, Inc., personal commun., February 28, 2001) who have made the most recent revisions to the model. The model-conduit number that corresponds to the gaging station is 3544.

Calibration Storms: Seventeen storms were used for model calibration; total precipitation during these storms ranged from 0.35 to 1.83 in. (mean 0.76 in.), which resulted in observed discharges that ranged from 16 to 350 ft^3/s (mean of 126 ft^3/s) in peak discharge and 0.058 to 0.207 in. (mean of 0.118 in.) in runoff volume. The simulated peak discharge had a SE of 41 ft^3/s and a RMSE of 120 ft^3/s (table 8); the difference between simulated and observed peaks ranged between -72 and 100 percent with a median difference of -22 percent and a standard deviation of 44 percent. Simulated peak discharges were within 10 percent of the observed peak 18 percent of the time, within 25 percent of the observed peak 47 percent of the time, and within 50 percent of the observed peak 71 percent of the time. The simulated peak discharge is not affected by the peak discharge (fig. 22*A*).

The difference between the simulated and observed runoff volumes had a SE of 0.048 in. and a RMSE of 0.139 in. (table 8); the difference between simulated and observed volumes ranged between -58 and 81percent with a median difference of 5.3 percent and a standard deviation of 35 percent. Simulated runoff volumes were within 10 percent of the observed volume 29 percent of the time, within 25 percent of the observed volume 53 percent of the time, and within 50 percent of the observed peak 76 percent of the time. Simulated runoff volume tends to agree less with observed runoff volume than simulated with observed peak discharge (fig. 22*A*).

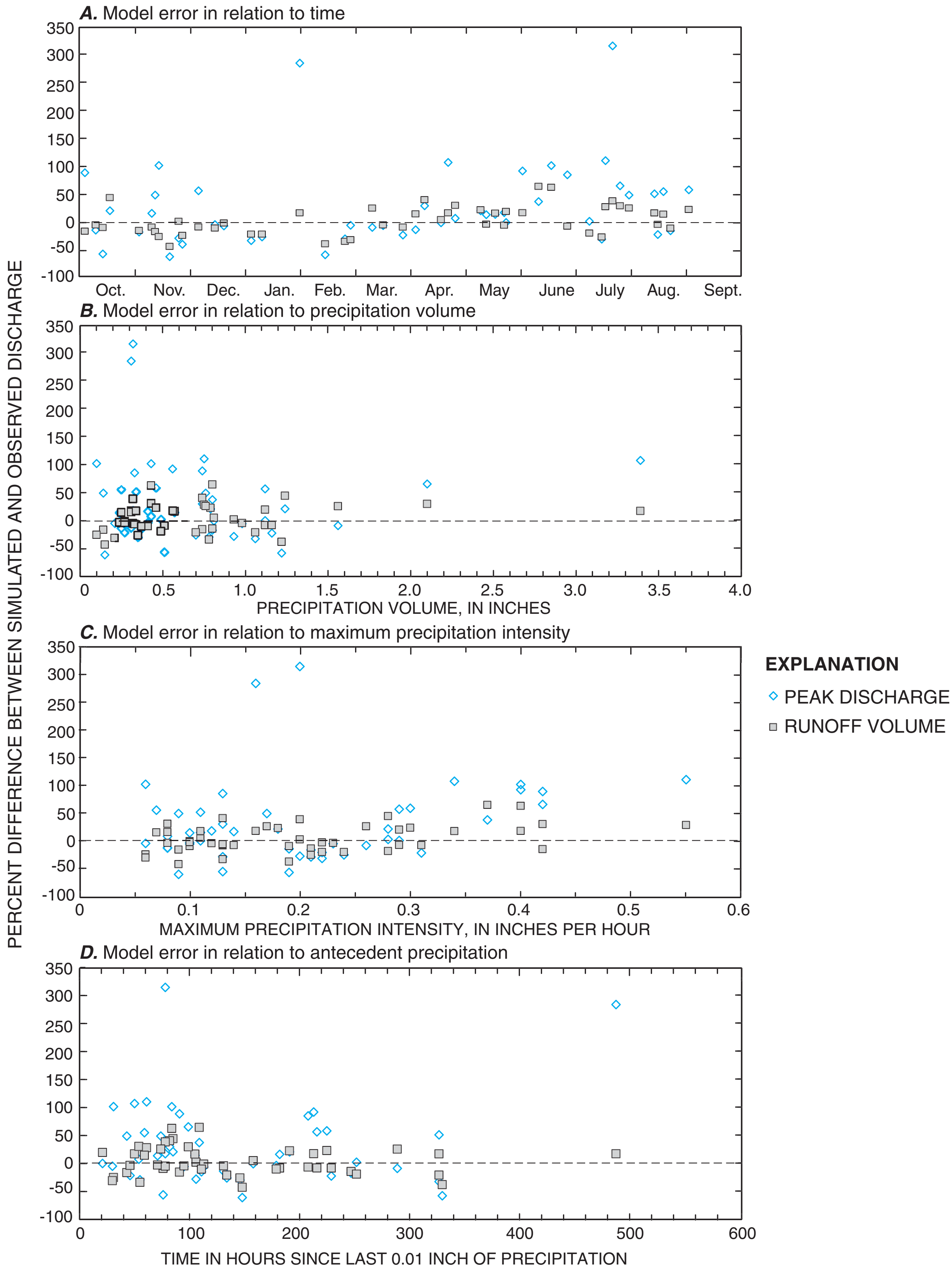


Figure 21. Percent difference between simulated and observed storm-peak discharge and runoff volume for the Faneuil Brook Subbasin, lower Charles River Watershed, Massachusetts, for 48 storms in the 2000 water year: (*A*) by month, and in relation to (*B*) precipitation volume, (*C*) precipitation intensity, and (*D*) antecedent precipitation.

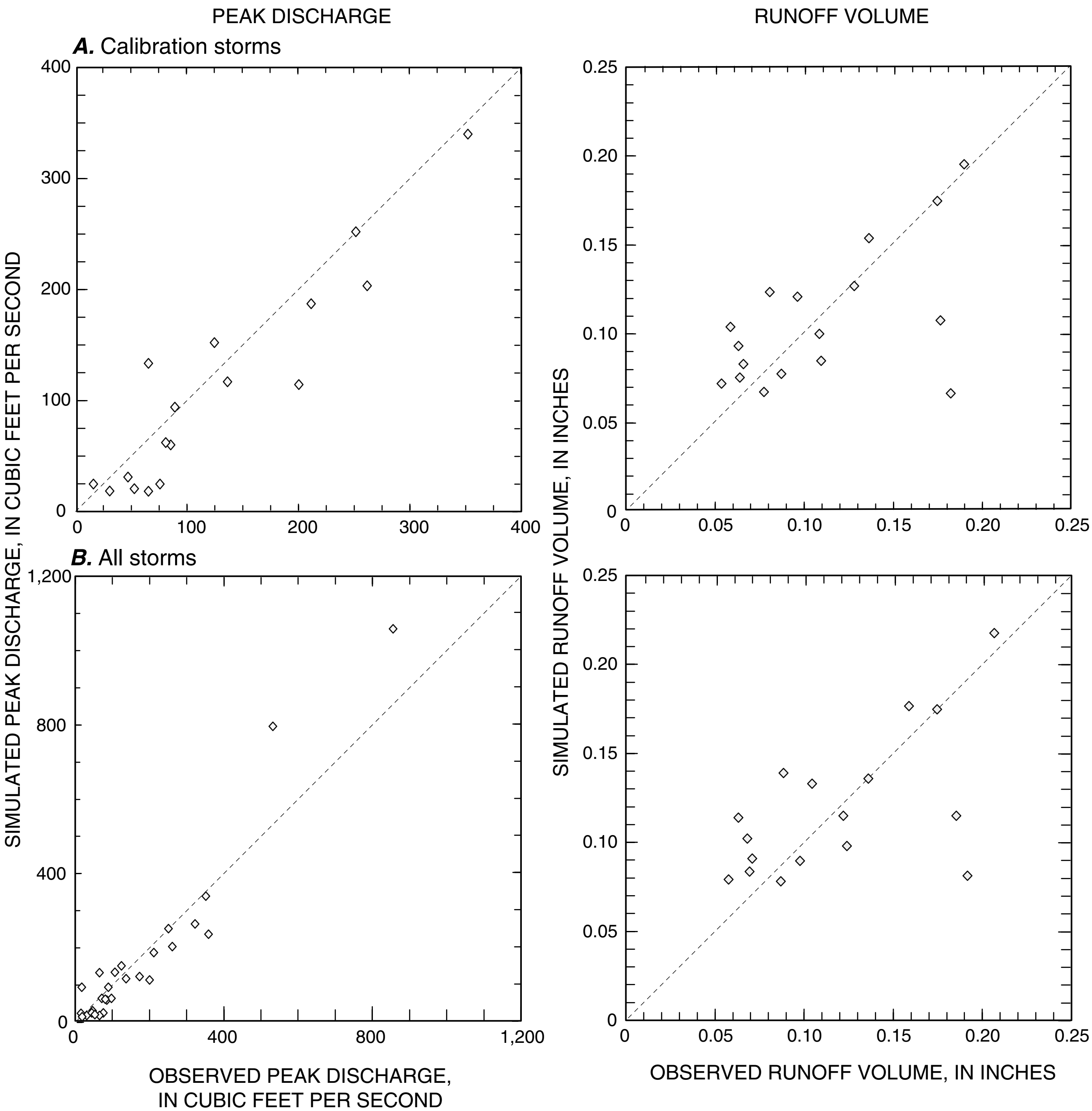


Figure 22. Simulated and observed peak discharge and runoff volume at Stony Brook Subbasin, lower Charles River Watershed, Massachusetts, for the 2000 water year: (*A*) calibration storms, and (*B*) all storms.

All Storms: Twenty-eight storms were used to evaluate the overall model fit; discharge data were not available prior to April 1, 2000. The April 21 storm was not used to assess the model fit because of a meter malfunction. Total precipitation during these storms ranged from 0.10 to 4.75 in. (mean 0.87 in.), which resulted in observed discharges that ranged from 16 to 860 ft^3/s (mean of 120 ft^3/s) in peak discharge and 0.048 to 1.24 in. (mean of 0.156 in.) in runoff volume. The simulated peak discharge had a SE of 80 ft^3/s and a RMSE of 260 ft^3/s (table 8); the difference between simulated and observed peaks ranged between -72 and 430 percent with a median difference of -20 percent and a standard deviation of 93 percent. Simulated peak discharges were within 10 percent of the observed peak 14 percent of the time, within 25 percent of the observed peak 43 percent of the time, and within 50 percent of the observed peak 79 percent of the time. Simulated peak discharge is in good agreement with the observed peak for most storms, except for the two largest, which are oversimulated (fig. 22*B*).

The difference between the simulated and observed runoff volume had a SE of 0.070 in. and a RMSE of 0.210 in. (table 8); the difference between simulated and observed volumes ranged between –58 and 117 percent with a median difference of 18 percent and a standard deviation of 37 percent. Simulated runoff volumes were within 10 percent of the observed volume 21 percent of the time, within 25 percent of the observed volume 54 percent of the time, and within 50 percent of the observed volume 82 percent of the time. Simulated runoff volume is generally in good agreement with observed runoff volume, but simulated and observed volumes tend to agree less well than simulated and observed peak discharges (fig. 22*B*).

Error: Plots of model error in relation to time of year, precipitation volume, intensity, and antecedent conditions did not indicate any systematic bias (fig. 23). The storm of July 22 had the largest oversimulated peak discharge (434 percent) and the largest oversimulated runoff volume (117 percent). The model error for this storm is largely attributed to areal variation in precipitation that ranged from 0.12 to 1.32 in. at the six rain gages.

In general, the Stony Brook model fit is comparable to that of the other models developed for the lower Charles River by the USGS. The Stony Brook model was developed primarily to assist in design of sewer-separation projects; therefore, development of the model focused on peak-discharge calibration. The models developed by the USGS are intended to calculate stormwater runoff for calculating constituent loads, and, thus, design and calibration of these models focused primarily on stormwater volume. For this reason, the USGS models tend to fit stormwater volumes slightly better than the Stony Brook model does, and, conversely, the Stony Brook model tends to fit storm peak discharges slightly better than the USGS models.

Relative Model Fit

The relative model fit of the models developed for the lower Charles River are quantified by the index of agreement and the coefficient of efficiency. Values of relative model fit can range from 0 to 1.0 for the index of agreement and from minus infinity to 1.0, for the coefficient of efficiency. Coefficient of efficiency values less than zero indicate that the observed mean provides a better predictor than the model; values greater than zero indicate that the model is a better predictor than the observed mean. Interpretation of the value of the index of agreement is similar to the coefficient of determination (R^2) in a regression equation—the closer the R^2 value is to 1.0, the better the agreement between simulated and observed values. Unlike the coefficient of determination, the index of agreement does not provide a measure of the total variance explained by the model. Both relative measures of model fit are sensitive to extreme values. The relative model fit for storm peak discharge and runoff volume calculated by the index of agreement and the coefficient of efficiency for each of the models is shown in figure 24. James and Burgess (1982) report that an excellent calibration is obtained if the coefficient of efficiency exceeds 0.97.

Peak discharge: The index of agreement for calibration storms ranged from 0.87 to 0.96 and averaged 0.93; the coefficient of efficiency ranged from 0.35 to 0.84 and averaged 0.71. The index of agreement for all storms ranged from 0.72 to 0.97 and averaged 0.86; the coefficient of efficiency ranged between -0.33 and 0.84 and averaged 0.43 (table 8). The coefficient of efficiency at the Faneuil Brook site indicates that the model is in less agreement with the observations than the mean value for peak discharge. Possible explanations for the relatively poor peak discharge simulation at Faneuil Brook are: (1) the storage-discharge characteristics of Chandler Pond are not well defined, (2) Chandler Pond is dynamically regulated, and (3) surcharging in the storm-drain system could cause an oversimulated peak discharge. In general, the relative model fit for peak discharge was better for calibration storms than for all storms because the precipitation variability was less in the calibration storms than in the other storms.

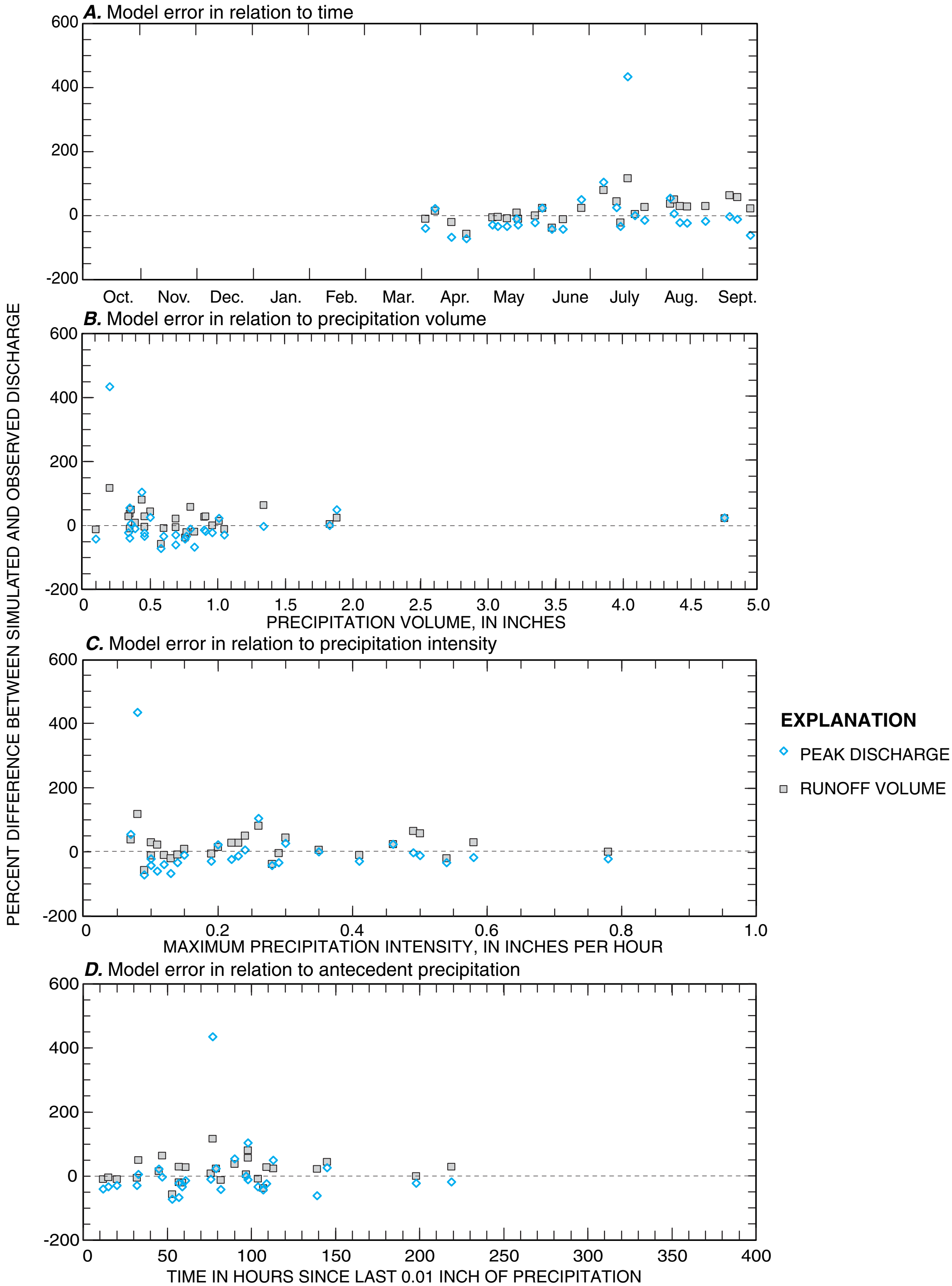


Figure 23. Percent difference between simulated and observed storm-peak discharge and runoff volume for the Stony Brook Subbasin, lower Charles River Watershed, Massachusetts, for 28 storms in the 2000 water year: (*A*) by month, and in relation to (*B*) precipitation volume, (*C*) precipitation intensity, and (*D*) antecedent precipitation.

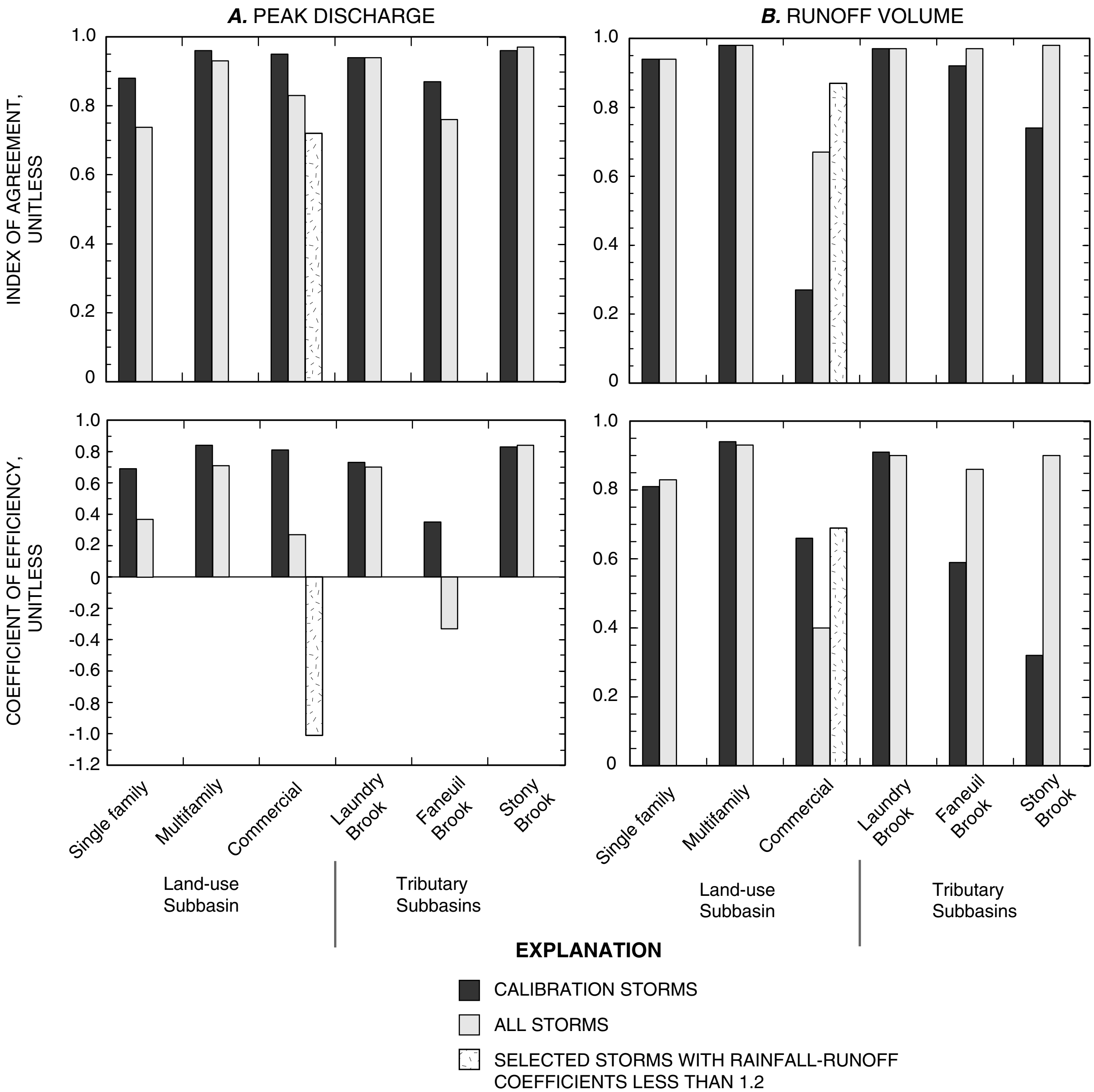


Figure 24. Relative measures of model fit calculated by the index of agreement and coefficient of efficiency at three land-use and three tributary subbasin models in the lower Charles River Watershed, Massachusetts, for the (*A*) peak discharge and (*B*) runoff volumes.

Runoff Volume: The index of agreement for calibration storms ranged from 0.27 to 0.98 and averaged 0.80; the coefficient of efficiency ranged from 0.32 to 0.94 and averaged 0.71. The index of agreement for all storms ranged from 0.67 to 0.98 and averaged 0.92; the coefficient of efficiency ranged between 0.40 and 0.93 and averaged 0.80 (table 8). The index of agreement and the coefficient of efficiency were relatively low at the commercial land-use subbasin for reasons previously described. The relative model fit improved substantially for runoff volume at the commercial land-use subbasin for storms with rainfall-runoff coefficients less than 1.2. In general, the values for the coefficient of efficiency indicated a model fit that was good to excellent for all storms, and the values for the index of agreement indicated a model fit that was excellent for most sites for all storms.

Design Storms

The model fit for the design storms was evaluated as a surrogate measure by the collective model fit at the three tributary subbasin models, Laundry Brook, Faneuil Brook, and Stony Brook, for measured storms with precipitation volumes roughly equivalent to the MWRA 3-month and 1-year design storms. Collectively, these three tributary models represent 49 percent of the total drainage area to the lower Charles River below Watertown Dam.

Three storms with measured discharge and total precipitation within ± 20 percent of the 3-month and 1-year design storms (1.84 and 2.79 in., respectively) were available. Storms with precipitation volumes roughly equivalent to the 3-month storm occurred on March 12, and July 27, 2000, with precipitation totals of 1.56 and 2.10 in., respectively, at BWSC-CS4 gage (fig. 10). Only one storm, April 22, 2000, had a total precipitation volume roughly equivalent to the 1-year storm—3.39 in. measured at BWSC-CS4 gage. Measured discharge was not available for the March 12 and April 22 storms at the Stony Brook, thus, a combined total of seven storms were used in this analysis.

The simulated storm peak discharges had a SE of 27 ft^3/s and a RMSE of 41 ft^3/s; the difference between simulated and observed peaks ranged between -15 and 107 percent with a median difference of 0.2 percent and a standard deviation of 46 percent. Simulated peak discharges were within 10 percent of the observed peak 43 percent of the time, within 25 percent of the observed peak 71 percent of the time. Simulated peak discharge (fig. 25*A*) is generally in good agreement with observed peak discharge, except for one storm that was oversimulated (April 22 storm at Faneuil Brook).

The difference between the simulated and observed runoff volumes had a SE of 0.061 in. and a RMSE of 0.110 in.; the difference between simulated and observed volumes ranged between -13 and 30 percent with a median difference of 5 percent and a standard deviation of 17 percent. Simulated runoff volumes were within 10 percent of the observed volume 29 percent of the time, within 25 percent of the observed volume 71 percent of the time, and within 50 percent of the observed volume 100 percent of the time. Simulated runoff volume agrees well with observed runoff volume (fig. 25*B*).

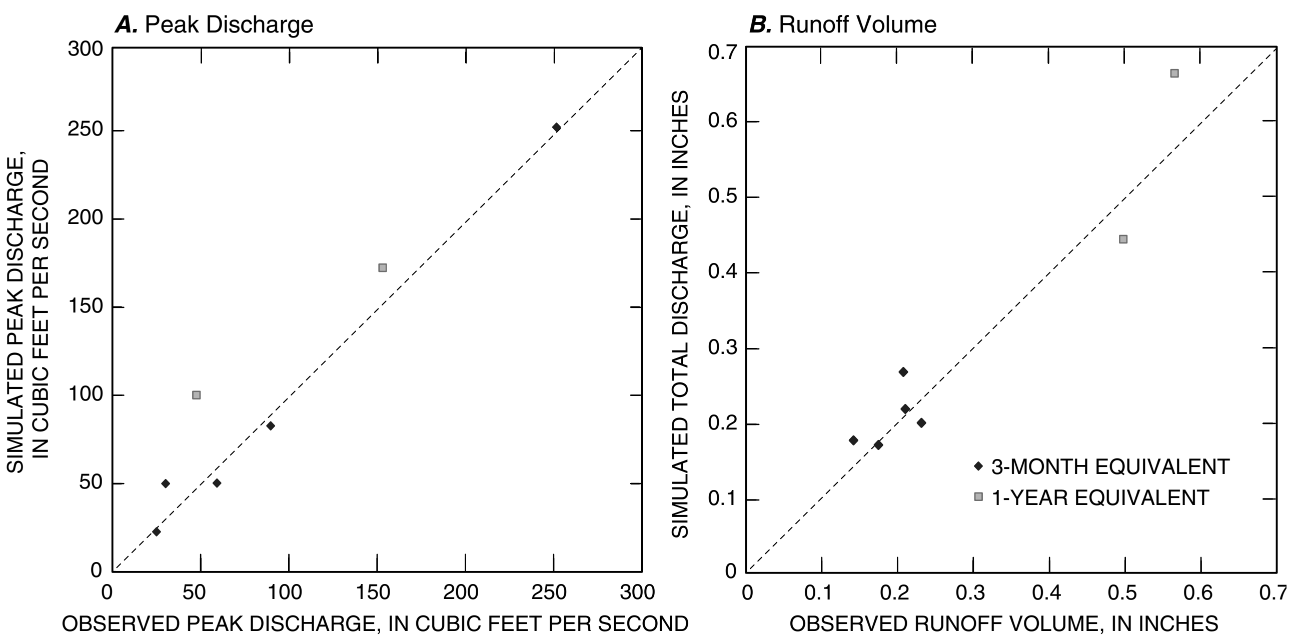


Figure 25. Simulated and observed (*A*) peak discharge and (*B*) runoff volume at Laundry Brook, Faneuil Brook, and Stony Brook, lower Charles River Watershed, Massachusetts, for 2000 water-year storms with total precipitation within 20 percent of the 3-month and 1-year design-storm precipitation volume.

Model-fit statistics and scatter plots for storms comparable to the 3-month and 1-year design storms at the tributary subbasins indicate that these models provide a reliable simulation of the peak discharge and runoff volume for the design storms. Although the ungaged area model fit is unknown, the variable values for the ungaged model are derived from the tributary variable values for similar land use. Thus, a reliable simulation model for the ungaged area can be inferred from the model fit of the tributary subbasin models.

Sensitivity Analysis

Sensitivity analysis measures the response of the model-simulated discharge to a change in a variable value that represents watershed properties. Typically, a sensitivity analysis is an iterative process whereby the value of a given variable is changed while all other variables are held constant thus indicating the degree to which that variable can affect simulation results. The process of varying a model variable and measuring the effect on the simulation results is automated in the PCSWMM 2000 Sensitivity Wizard. Variables that were tested for sensitivity include: subcatchment width, percent effective impervious area, impervious area roughness, pervious area roughness, impervious area depression storage, pervious area depression storage, capillary suction, saturated hydraulic conductivity, and the initial moisture deficit.

The Sensitivity Wizard tests the sensitivity of a model variable to storms of different durations and intensities—long, medium, and short duration (10-hour, 1-hour, and 20-minute storms, respectively), and high, medium, and low intensity (varied from 3.0 to 0.1 in/h, dependent on the duration). Sensitivity analysis was performed on peak discharge and runoff volume with the use of hyetographs that represent each of the different permutations of duration and intensity (only long and short durations are presented in figs. 26 and 27) for the single-family land-use subbasin. The single-family land-use subbasin was chosen for the sensitivity analysis because (1) it represents the dominant land-use type in the lower Charles River Watershed, and (2) it is more likely to demonstrate the sensitivity of pervious area values than the multifamily and commercial subbasins, which have a higher percentage of effective impervious area. Variables were allowed to change by ±30 percent during the sensitivity testing.

Results of the sensitivity analysis (figs. 26 and 27) indicate that the percent effective impervious area was the most sensitive variable affecting runoff volumes and peak discharge for most storm types. For storms with precipitation totals of 1 in. or more, the value of effective impervious area was about 50 times more sensitive than the next closest subcatchment variable—typically impervious and pervious area depression storage for runoff volume and subcatchment width for peak discharge. The sensitivity gradients of effective impervious area are proportional to the value of storm-peak discharge or runoff volume. Decreases in effective impervious area result in decreases in peak discharge and runoff volume and, conversely, increases in effective impervious area result in increases in peak discharge and runoff volume.

Decreases in impervious and pervious area depression storage were the most sensitive variables affecting peak discharge and runoff volume for medium-duration, low-intensity storms. Total precipitation for this type of storm was only 0.1 in., which is near the interception storage starting values; thus, decreasing the values of interception storage has a large effect on both peak discharge and runoff volume for small storms. Other subcatchment variables have the greatest effect on peak discharge and runoff volume for short-duration storms (20 minutes) and medium-duration storms (1 hour) of low intensity. Peak discharge is affected mostly by the value of the impervious area roughness for the short-duration, low-intensity storm (total precipitation of 0.03 in.).

Model Limitations

SWMM was chosen to simulate runoff to the lower Charles River because its numerical solutions are well suited to this type of watershed. Still, a number of simplifications are made in the mathematical model representation of runoff process and in the structural representation (discretization) of the hydrologic system. The models developed by the USGS for the lower Charles River were conceptualized and calibrated to represent runoff to compute stormwater loads. For example, these models do not include the entire drainage network, but only a sufficient portion of the drainage system to provide subcatchment linkage, which allows water to be routed by the kinematic-wave method to its outfall. Thus, the models developed by the USGS are not suitable for storm-sewer design or other applications that require robust analysis of flow hydraulics of the storm-drain system.

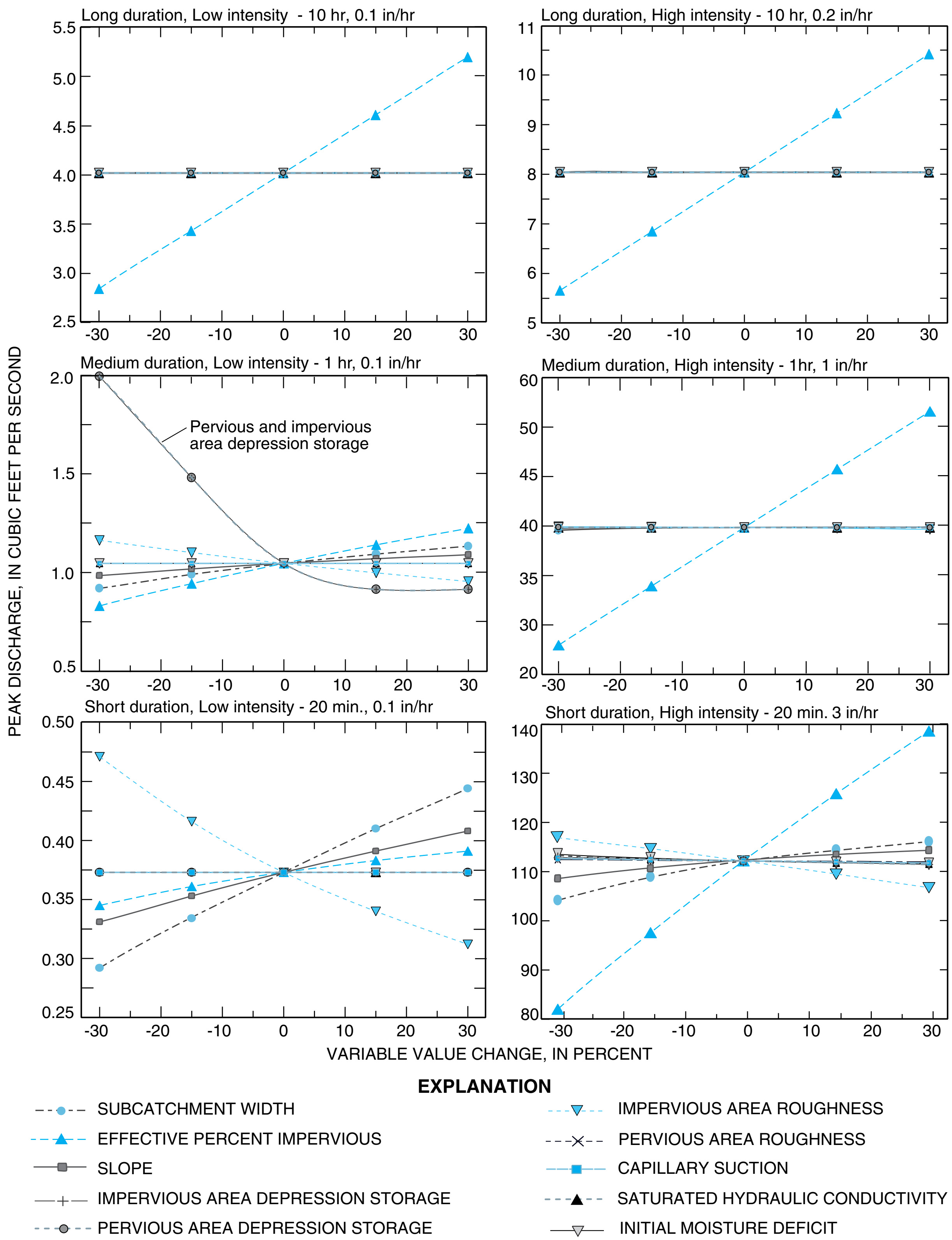


Figure 26. Peak-discharge sensitivity gradients for hypothetical storms of varying duration and intensity for selected subcatchment variable values, lower Charles River Watershed, Massachusetts.

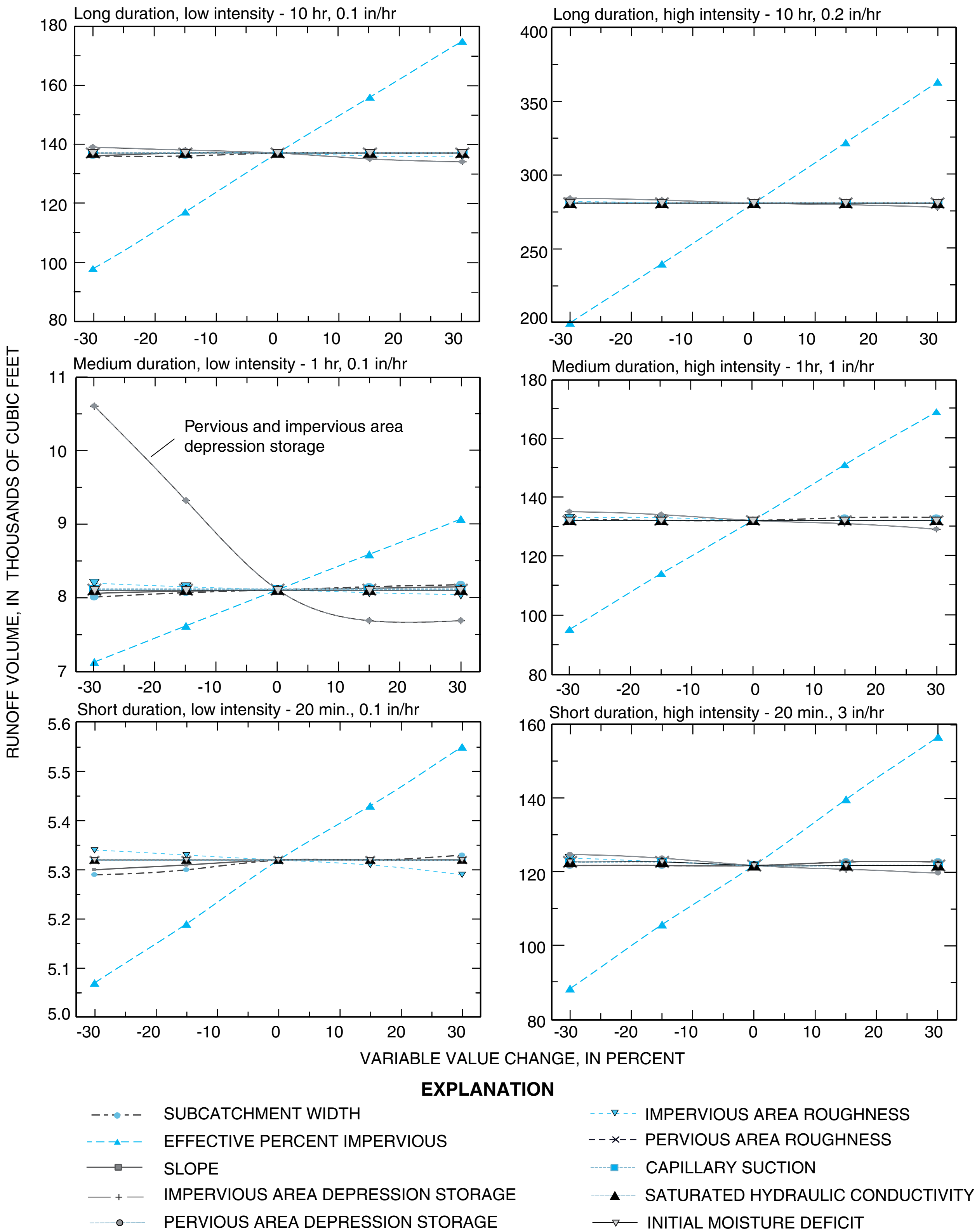


Figure 27. Runoff-volume sensitivity gradients for hypothetical storms of varying duration and intensity for selected subcatchment variable values, lower Charles River Watershed, Massachusetts. [hr, hours; in, inch; min, minute]

SWMM requires a single set of variable values to represent a subcatchment (lumped variables), but the physical properties of the tributary subbasin model subcatchments are spatially heterogeneous. Although simulation results obtained from the tributary subbasin models at the gage locations (calibration points) were good, the model performance of individual subcatchments is unknown. The ungaged subbasin model has an even larger degree of uncertainty than the gaged model because the variable values were obtained by transferring each value from subcatchments with similar land-use characteristics in the tributary subbasin models. The model performance for the aggregate of these subcatchments could not be evaluated as it was for the gaged subbasins. The sensitivity analysis indicates that even small changes in the effective imperviousness can produce large changes in the storm peak discharge and runoff volume. Ungaged areas, however, represent only about 4 percent of the total drainage area to the lower Charles River.

The performance of the models outside of the range of storms evaluated in the calibration and model fit analysis is unknown. The models were calibrated and tested over a wide range of storms (0.10 to 4.4 in.), but the majority of these storms had less than 1 in. of total precipitation. The largest storm included in the evaluation of the model fit, June 6, 2000, had a total precipitation of 4.4 in. and an estimated recurrence interval of 2 to10 years. The simulated runoff volume error ranged between -27 and 23 percent, excluding the model performance at the commercial land-use subbasin and Faneuil Brook Subbasin, which had questionable observed discharge data as previously described. The median simulated runoff volume error ranged between –13 and 18 percent; thus, the simulated runoff volume for storms of this magnitude or larger is potentially greater than that reported for all storms. Flow conditions other than those evaluated for this study could affect the model results and, therefore, simulation results outside the range of conditions used to assess the model should be used with caution.

RUNOFF TO THE LOWER CHARLES RIVER

The calibrated models were used to simulate runoff to the lower Charles River from the watershed below Watertown Dam for (1) the 2000 water year, October 1, 1999, through September 30, 2000, (2) a 3-month design storm, and (3) a 1-year design storm. SWMM results provided annual and design-storm hydrographs for the calculation of constituent loads to the lower Charles River presented in a companion report by Breault and others (2002). The annual (water year) runoff to the lower Charles River is summarized in table 9.

Annual and Monthly Water Budget

For each of the gaged subbasins, simulated flow components are described for the annual water budget and the simulated monthly runoff volumes are compared to the measured monthly volumes. Differences between the simulated and the observed monthly runoff relative to the previously described stormwater-volume error indicate the potential error associated with the calculation of constituent loads.

Single-Family Land-Use Subbasin

Annual precipitation simulated on the single-family land-use subbasin was 42.0 in.—simulations indicate about 80 percent infiltrated the ground, 14 percent became surface runoff, and 5 percent evaporated

Table 9. Annual runoff observed at Charles River at Watertown Dam, and simulated at land-use and tributary subbasins to the lower Charles River, Massachusetts, Water Year 2000, October 1, 1999 through September 30, 2000

[WY, water year; --, not applicable]

Site	Annual WY 2000 volume (Millions of cubic feet)	
	To stream gage	To outfall
Land-use subbasins		
Single family	9.51	--
Multifamily	3.04	--
Commercial	8.11	--
Tributaries and ungaged subbasins		
Charles River at Watertown	15,300	--
Laundry Brook	82.3	82.3
Faneuil Brook	38.6	49.1
Muddy River	202	157
Muddy River conduit	--	183
Stony Brook	478	489
Aggregate of ungaged subbasins	--	284

from surface storage. Of the water that infiltrated the ground, 46 percent entered deep ground-water storage and is considered lost from the subbasin, 41 percent was lost to evapotranspiration, and 14 percent entered the drainage systems as ground-water discharge. The simulated runoff volume for the 2000 water year was 9.51 million ft^3 (table 9), the average discharge was 0.30 ft^3/s, the minimum discharge was 0.001 ft^3/s, and the maximum discharge was 79 ft^3/s.

Simulated monthly runoff volumes (fig. 28*A*) differed from the observed runoff volume by -37 to 18 percent. Monthly runoff was generally undersimulated by about 12 percent, but storm-runoff volumes are oversimulated by 18 percent on average. Hence, the difference in the monthly runoff is largely attributed to differences in base flow. The observed base flow generally was between 0.02 and 0.05 ft^3/s, whereas simulated base flow was typically zero. Therefore, constituent loads calculated from the simulated discharge values can be expected to underestimate base-flow loads by an average of 12 percent and overestimate storm loads by about 18 percent on average.

Multifamily Land-Use Subbasin

Annual precipitation simulated on the multifamily land-use subbasin was 40.95 in.—simulations indicate about 13 percent infiltrated the ground, 77 percent became surface runoff, and 10 percent evaporated from surface storage. Of the 6.66 in. of water that infiltrated the ground or was lost from the subsurface storage over the simulation period, 60 percent entered deep ground-water storage (considered lost from the subbasin), 39 percent was lost to evapotranspiration, and 1 percent entered the drainage systems as ground-water discharge. Total runoff volume for the 2000 water year was 3.04 million ft^3, the average discharge was 0.10 ft^3/s, the minimum discharge was zero, and the maximum discharge was 25 ft^3/s.

Simulated monthly runoff volumes differed from the observed volumes by -24 to 39 percent (fig. 28*B*). The model slightly undersimulated runoff in December and January, and slightly oversimulated runoff in May, July, August, and September. The month of June was undersimulated by 24 percent largely because of the undersimulation of the June 6 storm, but storm-runoff volumes were oversimulated by 3 percent on average. On average, base-flow constituent loads can be expected to be underestimated slightly and stormwater constituent loads can be expected to be oversimulated by 3 percent.

Commercial Land-Use Subbasin

Annual precipitation simulated on the commercial land-use subbasin was 40.95 in.—simulations indicate about 80 percent became surface runoff, about 11 percent infiltrated the ground, and about 9 percent evaporated from surface storage. Of the 5.83 in. of water that infiltrated into the ground or drained from subsurface storage over the simulation period, about 24 percent was lost to evapotranspiration, and most of the rest (about 76 percent) entered deep ground-water storage. The simulated total runoff volume for the 2000 water year was 8.11 million ft^3, the average discharge was 0.26 ft^3/s, the minimum discharge was zero, and the maximum discharge was 19 ft^3/s.

The constant base flow of 0.20 ft^3/s that was added to the TRANSPORT module is not included in the water budget above. A base flow of this magnitude would require a ground-water discharge of 71 in/yr, or about twice the amount of annual precipitation over the subbasin. A small portion of the base flow (about 5 percent) could be gained by eliminating the loss to the deep ground-water system; however, this rate of loss is consistent with the calibrated ground-water loss used in other subbasins models.

Simulated monthly runoff volumes differed from the observed runoff by -51 to 25 percent (fig. 28*C*); the month of June (-51 percent error) was appreciably undersimulated primarily because measured discharge during the June 6 storm had a prolonged elevated recession. This type of recession is atypical for this type of subbasin and is likely caused by measurement error or the possible conditions that caused high RR coefficients in this subbasin as previously explained. After the station was made operational in mid-January, the measured discharge was noisy and undersimulated compared to the observed discharge until early April. Measured discharge is missing from April 2 to 20. Therefore, constituent loads calculated from simulated discharges are expected to overestimate base-flow loads during the late summer and overestimate storm loads by -13 percent, on average; however, the error in the load estimate is expected to be minor if runoff is unaffected by the variable source area or the cross connections.

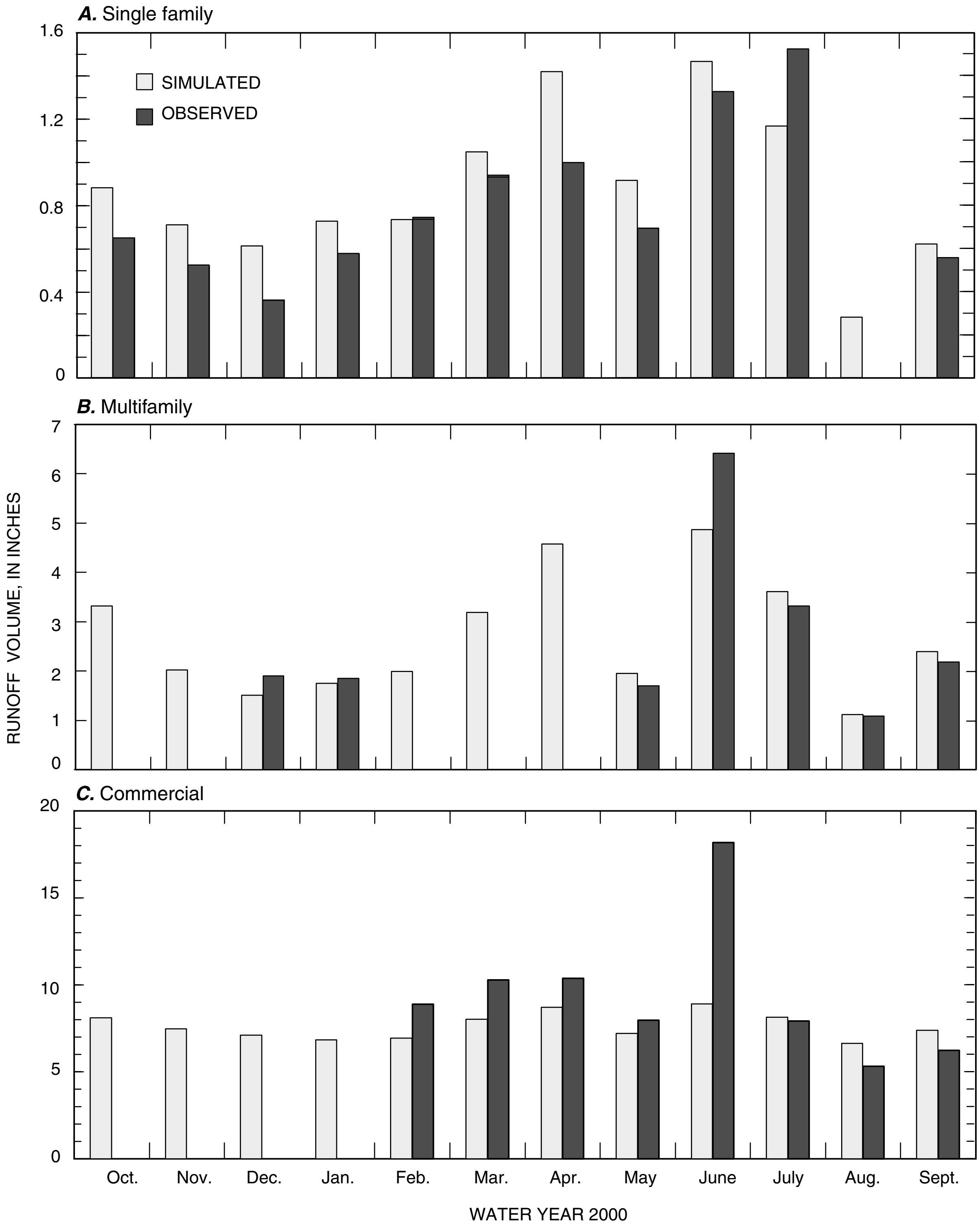


Figure 28. Simulated and observed monthly runoff volume at the three land-use subbasin sites in the lower Charles River Watershed, Massachusetts, 2000 water year: (*A*) single family, (*B*) multifamily, and (*C*) commercial. [Observed monthly runoff volumes are not shown for months with missing data.]

Laundry Brook Subbasin

Annual precipitation simulated on the Laundry Brook Subbasin was 42.05 in.—simulations indicate about 86 percent infiltrated into ground, about 10 percent became surface runoff, and about 4 percent evaporated from surface storage. Of the 36.06 in. of water that infiltrated into the ground, 47 percent entered deep ground-water storage, 42 percent was lost to evapotranspiration, 10 percent entered the drainage systems as ground-water discharge, and 1 percent was added to subsurface storage over the simulation period. Total runoff volume for the 2000 water year was 82.3 million ft^3, the average discharge was 2.60 ft^3/s, the minimum discharge was 0.36 ft^3/s, and the maximum discharge was 194 ft^3/s.

Simulated monthly runoff volumes differed from the observed volumes by -25 to 52 percent (fig. 29*A*). Monthly runoff was undersimulated by about 1.5 percent on average and storm runoff was undersimulated by 0.9 percent, on average. The model undersimulates runoff in February, March, April, May, August, and September; and oversimulates runoff in November, December, January, June, and July. Slightly underestimated annual loads and storm loads are expected when constituent loads are calculated from simulated discharges.

Faneuil Brook Subbasin

Annual precipitation simulated on the Faneuil Brook Subbasin was 42.2 in.—simulations indicate that about 81 percent infiltrated the ground, 15 percent was surface runoff, and 4 percent evaporated from surface storage. Of the 34.13 in. of water that infiltrated the ground, 42 percent entered deep ground-water storage, 40 percent was lost to evapotranspiration, 17 percent entered the drainage systems as ground-water discharge, and 1 percent was added to the subsurface storage over the simulation period. Total annual-runoff volume to the gaging station for the 2000 water year was 38.6 million ft^3, the average discharge was 1.22 ft^3/s, the minimum discharge was zero, and the maximum discharge was 136 ft^3/s. Total runoff volume from Faneuil Brook to the lower Charles River for the 2000 water year was 49.1 million ft^3, the average discharge was 1.55 ft^3/s, the minimum discharge was zero, and the maximum discharge was 171 ft^3/s.

Simulated monthly runoff volumes differed from the observed volumes by -57 to 24 percent (fig. 29*B*); the largest error is in June largely because the storm volume on June 6 was oversimulated. Runoff volumes during November through March were undersimulated; storm volumes were undersimulated by 3 percent on average. Base flow during the winter and spring generally was undersimulated because the ground water was treated as a linear reservoir that drained to a fixed elevation in the conduit. Therefore, constituent loads calculated from simulated discharges are expected to underestimate base-flow loads during the winter and spring and slightly underestimate storm loads.

Stony Brook Subbasin

Annual precipitation simulated on the Stony Brook Subbasin was 42.36 in.—simulations indicate that about 67 percent infiltrated the ground, about 28 percent became surface runoff, and about 6 percent evaporated from surface storage. Ground water was not simulated in the Stony Brook model; therefore, infiltrated ground water in the subbasin is not routed to the stream. The total simulated runoff volume to the gage for the 2000 water year was 478 million ft^3, the average discharge was 15.1 ft^3/s, the minimum discharge was 10.1 ft^3/s, and the maximum discharge was 1,059 ft^3/s. The simulated annual runoff volume to the mouth for the 2000 water year was 489 million ft^3, the average discharge was 15.5 ft^3/s, the minimum discharge was 10.3 ft^3/s, and the maximum discharge was 685 ft^3/s.

Simulated monthly runoff volumes differed from the observed volumes by -3 to 63 percent (fig. 29*C*). The model undersimulates runoff in June, July, August, and September and oversimulates runoff in May; on average, monthly runoff is undersimulated by 24 percent. Storm-runoff volumes were oversimulated by 18 percent, on average. Underestimated annual loads and overestimated storm loads are expected from constituent loads calculated from simulated discharges.

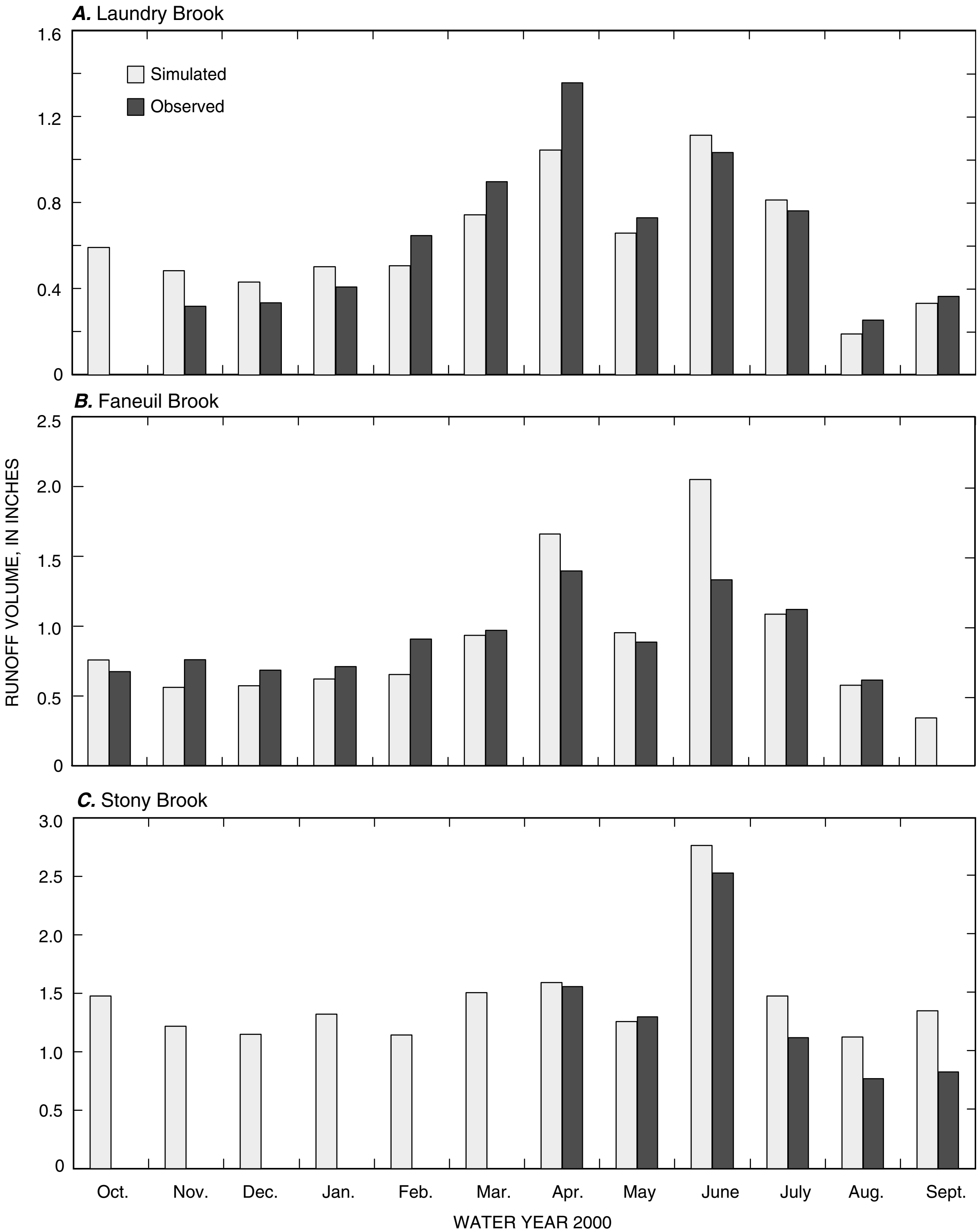


Figure 29. Simulated and observed monthly runoff volume at three tributary subbasin sites in the lower Charles River Watershed, Massachusetts, 2000 water year: (*A*) Laundry Brook, (*B*) Faneuil Brook, and (*C*) Stony Brook. [Observed monthly runoff volumes are not shown for months with missing data.]

Total Inflows to the Lower Charles River

Total annual inflow to the lower Charles River was 16,500 million ft^3; about 92 percent of the annual inflow was from the upper basin, 3 percent was from the Stony Brook Subbasin, 2 percent was from the Muddy River Subbasin, and less than 1 percent was from the combined inflows of Laundry and Faneuil Brooks (table 9). These inflows do not include CSO discharges to the river, except those entering Stony Brook prior to its discharge to the lower Charles River. The remaining ungaged drainage area contributed about 2 percent of the total annual inflow to the lower Charles River. Excluding inflows from the upper basin (Charles River above Watertown Dam), the total annual inflow to the lower Charles River was 1,320 million ft^3; about 39 percent was from the Stony Brook Subbasin, 27 percent was from the Muddy River Subbasin, which includes the area that drains to the Muddy River conduit, 7 percent was from the Laundry Brook Subbasin, 4 percent was from the Faneuil Brook Subbasin, and the ungaged areas not included in the tributary subbasin models contributed about 23 percent.

Design Storms

The MWRA selected two design storms for their river model simulations of bacterial transport and fate in the lower Charles River. The design storms represent a 3-month storm, based on an actual storm of July 20, 1982, and a 1-year storm, based on an actual storm of September 20, 1961. The 3-month storm lasted 30 hrs, had 1.84 in. total rainfall, 0.06 in/h average rainfall intensity, and 0.4 in/h maximum rainfall intensity. The 1-year storm lasted 26 hrs, had 2.79 in. total rainfall, 0.11 in/h average rainfall intensity, and 0.65 in/h maximum rainfall intensity. Discharge from the upper basin (Charles River above Watertown Dam) was estimated from discharge recorded at the Waltham, Massachusetts, gage (01104500) as previously described. The models developed by the USGS and MWRA were used to calculate runoff produced by these storms from all other areas to the lower Charles River, excluding CSO discharges except those to Stony Brook.

Total storm runoff for the 3-month and 1-year storms was 111 and 257 million ft^3, respectively. The upper basin drainage area, the largest source of water to the lower Charles River, produced 21 and 46 percent of the total inflow for the 3-month and 1-year design storms, respectively (table 10). Typically, after a large storm the discharge at Charles River at Watertown Dam takes several days to return to base-flow conditions; the prolonged recession is attributed to water storage in upstream riparian wetlands. For this reason, the storm volumes for both design storms were truncated after about 3 days because discharges were affected by later storms. Muddy River, Stony Brook, and the ungaged subbasins not included in tributary subbasin models each make up about 5 to 7 percent of the total runoff volume for the 3-month storm and about 3 to 6 percent of the runoff volume for the 1-year storm. The Laundry Brook and Faneuil Brook Subbasins each made up less than 2 percent of the total runoff volume for both the 3-month and 1-year storms.

Excluding inflow from the upper basin the runoff to the lower Charles River was as follows. Total storm runoff for the 3-month and 1-year design storms was 30 and 53 million ft^3, respectively. The largest source was Muddy River Subbasin, which contributed 34 percent of the runoff for the 3-month storm and 35 percent of the runoff for the 1-year storm. The Stony Brook Subbasin and the ungaged subbasins, not included in the tributary subbasin models, contribute about equal proportions for both storms (about 26 to 30 percent of the total runoff, respectively). Combined, the Muddy

Table 10. Storm-runoff volumes simulated for the 3-month and 1-year design storms to the lower Charles River, Massachusetts

Site	Design storm volume (millions of cubic feet)	
	3 month	1 year
Land-use subbasins		
Single family	0.278	0.443
Multifamily	.120	.208
Commercial	.104	.160
Tributaries and ungaged subbasins		
Charles River at Watertown	80.2	204
Laundry Brook	2.15	3.53
Faneuil Brook	.968	1.64
Muddy River conduit	5.16	8.77
Muddy River	4.91	9.54
Stony Brook	7.91	14.9
Aggregate of ungaged subbasins	8.99	14.3
Total	111	257

River and Stony Brook Subbasins contributed about 60 percent of the design-storm runoff; these inputs are significant because they enter the lower Charles River in a prime recreational area. Laundry Brook contributes about 7 percent and Faneuil Brook contributes about 3 percent of the total runoff for these design storms.

SUMMARY

The lower Charles River is an important recreational resource for the Boston metropolitan area, but contaminated stormwater and combined-sewer overflows (CSOs) have impaired its use because at times the river is unfit for secondary contact recreation (boating). The U.S. Environmental Protection Agency (USEPA) has set a goal of making the lower Charles River fishable and swimmable by Earth Day 2005. To meet this goal, a better understanding of the non-CSO stormwater discharge and constituent loads to the lower Charles River is needed. The U.S. Geological Survey (USGS), in cooperation with the USEPA, the Massachusetts Water Resources Authority (MWRA), and the Massachusetts Department of Environmental Protection (MADEP), developed and calibrated Storm-Water Management Models (SWMM) to quantify non-CSO stormwater loads to the lower Charles River. This report documents the stormwater-modeling procedures used to calculate runoff to the lower Charles River. Constituent loads are described in a companion report.

During the 2000 water year (September 30, 1999, through October 1, 2000), surface-water discharge data were collected at eight sites—three relatively homogenous land-use sites, four major tributaries downstream of the Watertown Dam, and the Charles River where it enters the lower Charles River at Watertown Dam. The three land-use sites represent the largest land-use categories in the lower Charles River Watershed: (1) single-family residential, (2) multifamily residential, and (3) commercial. The major tributary sites included all of the drainage area of Laundry Brook, and 80, 84, and 90 percent of the Faneuil Brook, Muddy River, and Stony Brook drainage areas, respectively. The USGS collected precipitation data at Watertown Dam and compiled precipitation data maintained by other agencies at five gages in or near the lower Charles River Watershed.

Precipitation and discharge data were used to run and calibrate models developed for the three land-use subbasins, and two tributary subbasins, Laundry and Faneuil Brook. Precipitation data were also used in a model that simulates discharge to the lower Charles River from numerous ungaged outfalls not included in the tributary subbasin models. The three homogeneous land-use subbasin models were later incorporated into other models; the residential land-use subbasin model was part of the Laundry Brook model, the multifamily subbasin and the commercial subbasin models were part of the ungaged area model. The Laundry, Faneuil, and ungaged area models include 126 subcatchments and 116 conduits.

The SWMM of the Stony Brook Subbasin was developed prior to this study by others, but evaluated with the newly collected data from this study. The Stony Brook model also included the Muddy River Subbasin, but variable backwater conditions at the Muddy River stream gage resulted in poor quality discharge records; therefore, the Muddy River Subbasin portion of the model was not evaluated. To simulate runoff from the lower Charles River Watershed requires four separate models—(1) Laundry Brook model, (2) Faneuil Brook model, (3) Stony Brook model, and (4) the ungaged area model.

Model variable values were calculated from relations of rainfall to runoff, available spatial data, field measurements, and literature values. The land-use subbasin models were calibrated first and the variable values obtained from these models were used as initial values for similar land-use types in the tributary and ungaged subbasin models. Variable values were adjusted during model calibration to minimize the difference between simulated and observed discharges; particularly the value of the effective impervious area, which largely determines the storm-peak discharge and runoff volume. Models developed by the USGS were calibrated with up to 24 storms. This number depended on the availability of measured discharge data; these storms were selected because they had the least variability in precipitation measured over the six rain gages.

The model fit was evaluated for each model for the calibration storms and for all storms with appreciable precipitation and reliable discharge data. Statistics reported to evaluate the model fit included the standard error of estimate (SE), the root mean square error (RMSE), the coefficient of efficiency (E), and the index of agreement (d) for storm-peak discharge and runoff volume. The first two statistics are absolute measures of the model fit, whereas the last two statistics are relative measures of the model fit. Model calibration

emphasized minimizing the differences between simulated and observed storm-runoff volumes rather than storm-peak discharge, although peak discharge was also considered in the model calibration.

Twenty storms, on average, were used for model calibration; the number of calibration storms varied between 13 and 24. The overall runoff volume model fit for the models developed by the USGS for these storms had SE values that ranged from 0.016 to 0.407 in. (average of 0.114 in.), RMSE that ranged from 0.061 to 0.794 in. (average of 0.272 in.), E that ranged from 0.32 to 0.94 (average of 0.78), and that ranged from 0.27 to 0.98 (average of 0.82). For the USGS models overall, the simulated runoff volume was within 10 percent of the observed volume from 30 to 46 percent of the time (average 38 percent), within 25 percent of the observed volume from 45 to 75 percent of the time (average 65 percent), and within 50 percent of the observed volume from 70 to 95 percent of the time (average 88 percent).

Forty-two storms, on average, were used to test the model fit; the number of storms ranged between 31 and 48. For the USGS models, the runoff-volume model fit for these storms had SE that ranged from 0.030 to 0.941 in. (average of 0.209 in.), RMSE that ranged from 0.130 and 2.19 in. (average of 0.551 in.), E that ranged from 0.40 to 0.93 (average of 0.84), and that ranged from 0.67 to 0.98 (average of 0.95). The simulated storm-runoff volume was within 10 percent of the observed volume between 26 and 33 percent of the time (average 29 percent), within 25 percent of the observed volume between 51 and 69 percent of the time (average 60 percent), and within 50 percent of the observed volume between 72 and 96 percent of the time (average 88 percent). The SE was at least four times larger and RMSE was at least three times larger for the commercial land-use subbasin model than for the other models, because measured discharge appears to be affected by variable conditions that are not considered in the model.

Over all storms, the USGS model runoff volumes were, on average, oversimulated by 18 percent by the single-family land-use subbasin model, undersimulated by 3 percent by the multifamily land-use subbasin model, oversimulated by 0.4 percent by the commercial land-use subbasin model, undersimulated by 1 percent by the Laundry Brook Subbasin model, and undersimulated by 3 percent by the Faneuil Brook Subbasin model. Model-fit statistics for storm-peak discharges generally were comparable to the statistics for runoff volume.

The Stony Brook Subbasin model generally had a model fit comparable to that of the USGS models. Twenty-eight storms were used to evaluate the model fit. The simulated storm-runoff volume had a SE of 0.070 in., RMSE of 0.210 in., E of 0.90, and d of 0.98. The simulated storm-runoff volume was within 10 percent of the observed volume 21 percent of the time, within 25 percent of the observed volume 54 percent of the time, and within 50 percent of the observed volume 82 percent of the time. Simulated storm runoff volumes at Stony Brook were oversimulated by 18 percent, on average. In general, the USGS models tend to fit stormwater volumes slightly better than the Stony Brook model, and, conversely, the Stony Brook model tends to fit storm-peak discharges slightly better than the USGS models, reflecting the different purposes for which the models were developed.

The total annual runoff to the lower Charles River, not including CSOs (except those entering Stony Brook), during the 2000 water year was 16,500 million ft^3; 92 percent of the inflow was from the upper basin, 3 percent was from the Stony Brook Subbasin, 2 percent was from the Muddy River Subbasin, which includes runoff that drains to the Muddy River conduit, and less than 1 percent was from the combined inflows of Laundry and Faneuil Brooks. The ungaged drainage area, not included in the tributary models, contributed about 2 percent of the total annual inflow to the lower Charles River. Total annual runoff to the lower Charles River, which excludes runoff above Watertown Dam, was 1,320 million ft^3; 39 percent was from the Stony Brook Subbasin, 27 percent was from the Muddy River Subbasin, which includes runoff that drains to the Muddy River conduit, 7 percent was from the Laundry Brook Subbasin, and 4 percent was from the Faneuil Brook Subbasin. The ungaged areas, not included in the tributary subbasin models, contributed about 23 percent of the total annual inflow to the lower Charles River, excluding runoff above Watertown Dam.

Runoff to the lower Charles River was calculated for two design storms that represent a 3-month and a 1-year event. The MWRA used these storms in a receiving water model to simulate the transport and fate of bacteria in the lower Charles River. These are actual storms on July 20, 1982 (3-month storm), and September 20, 1961 (1-year storm), with total rainfall volumes of 1.84 and 2.79 in., respectively. The models were used to simulate runoff for these storms from the lower Charles River Watershed; a relation between the Charles River Watertown station and the long-term Waltham station was developed to calculate inflow from the upper basin. Total storm runoff to the lower Charles River was 111 and 257 million ft^3 for the 3-month and 1-year storms, respectively. Total storm runoff to the lower Charles River excluding that from the upper basin was 30 and 53 million ft^3 for the 3-month and 1-year storms, respectively. Runoff from the various tributary areas for the design storms occurred in about the same proportion as the annual runoff.

Models developed and calibrated by the USGS were designed to estimate stormwater runoff to the lower Charles River. These models provide a planning tool for calculating stormwater runoff, for example, simulating runoff to compute constituent loads. The drainage system network, however, was not represented in sufficient detail for use in engineering design. Although the models are well calibrated to observed data for the 2000 water year, simulation results for storms larger than about 4.5 in. are untested and care should be exercised in the use of simulated discharges for storms larger than this amount.

REFERENCES CITED

Breault, R.F., Sorenson, J.R., and Weiskel, P.K., 2002, Streamflow, water quality, and contaminant loads in the lower Charles River Watershed, Massachusetts, 1999–2000: U.S. Geological Survey Water-Resource Investigations Report 02-4137, 131 p.

Brickford, W.E. and Dymon, U.J., 1990, An atlas of Massachusetts River systems—Environmental designs for the future: Amherst, University of Massachusetts Press, 87 p.

Buchanan, T.J., and Somers, W.T., 1965, Stage measurement at gaging stations, in Hydraulic measurement and computation: U.S. Geological Survey Surface Water Techniques, book 1, chap. A10, p. 1–26.

Brunner, G.W, 2001, HEC-RAS, River analysis system hydraulics reference manual: U.S. Army Corps of Engineers report CDP-69, 262 p.

Camp, Dresser and McKee, Inc., 1997a, Stony Brook Sewer System Study: Unpublished report on file at Camp, Dresser and McKee, Inc., Cambridge, MA, variously paged.

_____1997b, Technical Memorandum Number 3, Hydraulic Model of the Stony Brook System: Unpublished report on file at Camp, Dresser and McKee, Inc, Cambridge, MA, variously paged.

Carter R.W., and Davidian, J. 1965, General procedure for gaging stations, *in* Hydraulic measurement and computation: U.S. Geological Survey Surface Water Techniques, book 1, chap. 9, p. 1–13.

CH2M Hill, 1990, Final combined sewer overflow facilities plan and final environmental impact statement, Appendix E—Summaries of technical memoranda and errata for the draft CSO facilities plan: Boston, MA, CH2M Hill, Inc., variously paged.

Computational Hydraulics International, 2000, PCSWMM 2000: accessed August 2000 at URL http://www.chi.on.ca./pscwmm.html.

Fugro East, Inc., unpublished, Hydrologic evaluation and preliminary investigation of Chandler Pond: Boston, MA, 61 p.

Gadoury, R.A., and Wandle, W.S., 1986, Massachusetts surface-water resources, *in* National Water Summary 1985—Hydrologic Events and surface-water resources: U.S. Geological Survey Water-Supply Paper 2300, p. 271–276.

Hamon, W.R. 1961, Estimating potential evapotranspiration: Proceedings of the American Society of Civil Engineers, Journal of the Hydraulic Division, v. 87, no. HY3, p. 107–120.

Huber, W.C., and Dickinson, R.E., 1988, Storm water management model user's manual, version 4.0: U.S. Environmental Protection Agency, EPA/600/3-88/001a, 569 p.

James, L.D., and Burgress, S.L., 1982, Selection, calibration, and testing of hydrologic models, *in* Hydrologic Modeling of Small Watersheds: American Society of Agriculture Engineers, p. 466.

James, William, Huber, W.C., Dickinson, R.E., and James, R.C., 1999a, Water systems models, hydraulics, A guide to rain, temperature and runoff models of the USEPA SWMM 4: Guelph, Ontario, Canada, Computational Hydraulics International, 283 p.

James, William, Huber, W.C., Dickinson, R.E., and James, R.C., 1999b, Water systems models, Hydraulics, A guide to the Extran, Transport and Storage Treatment modules of the USEPA SWMM4: Guelph, Ontario, Canada, Computational Hydraulics, 218 p.

Jennings, M.E., and Doyle, W.H., Jr., 1978, Deterministic modeling of urban storm water processes, Broward County, Florida, *in* Proceedings International Symposium on Urban Storm Water Management: Lexington, University of Kentucky, July 1978, p. 275–281.

Jobin, W. 1998, Sustainable management for dams and waters: Boca Raton, FL, Lewis Publishers, 265 p.

Kennedy, E.J., 1983, Computation of continuous records of streamflow: U.S. Geological Survey Techniques of Water-Resources Investigations Report, book 3, chap. 13, 53 p.

_____1984, Discharge ratings at gaging stations: U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chap. 16, 59 p.

Legates, D.H., and McCabe, G.J., Jr., 1999, Evaluating the use of "goodness-of-fit" measures in hydrologic and hydroclimatic model validation: Water Resources Research, v. 35, no. 1, p. 233–241.

Lumb, A.M., and Kittle, J.L., 1995, Users manual for METCMP, a computer program for the interactive computation of meteorologic time series: Unpublished report on file at Office of Surface Water, Reston, VA, U.S. Geological Survey, 88 p.

Lumb, A.M., Kittle, J.L., and Flynn, K.M., 1990, Users manual for ANNIE, a computer program for the interactive hydrologic analysis and data management: U.S. Geological Survey Water-Resources Investigation Report 89-4080, 236 p.

Massachusetts District Commission, 2000, Charles River Basin—the second century: Boston, MA, 1 pl.

MassGIS, 2001, MassGIS Datalayer—Description and guide to user service: Boston, MA, Massachusetts Executive Office of Environmental Affairs, 222 p.

Metcalf and Eddy, Inc., 1989, First interim report on the Muddy River water quality improvement plan, January 1989: Unpublished report on file at Metcalf and Eddy, Inc., Woburn, MA, variously paged.

_____1994, Sub-task 2.5.5. Final technical memorandum, estimation of stormwater flows and loads: Unpublished report on file at Metcalf and Eddy, Inc., Woburn, MA, variously paged.

_____1999, Preliminary design report, Boston Water and Sewer Commission contract No. 98-206-003: Unpublished report on file at Metcalf and Eddy, Inc., Woburn, MA, variously paged.

Peragallo, T.A., 1989, Soil Survey of Norfolk and Suffolk Counties, Massachusetts: U.S. Department of Agriculture, Soil Conservation Service, p. 194.

Pritchett, H.S., Mansfield, S.M., and Dana, R.H., 1903, Report of the Committee on Charles River Dam: Boston, MA., Resolves of 1901, chap. 105, Wright and Potter Printing, 579 p.

Rizzo Associates, 1993, NPDES stormwater permit application, part 2: Unpublished report on file at Rizzo and Associates, Inc,, Framingham, MA, variously paged.

Seasholes, N.S., 1999, Gaining ground—Boston's topographical development in maps, *in* Krieger, Alex, Cobb, David, and Turner, Amy, Mapping Boston: Cambridge, MA, MIT Press, p. 118–145.

Skehan, J.W., 1979, Puddingstone, drumlins, and ancient volcanoes—A field guide along historic trails of Greater Boston: Dedham, MA., WesStone Press, 63 p.

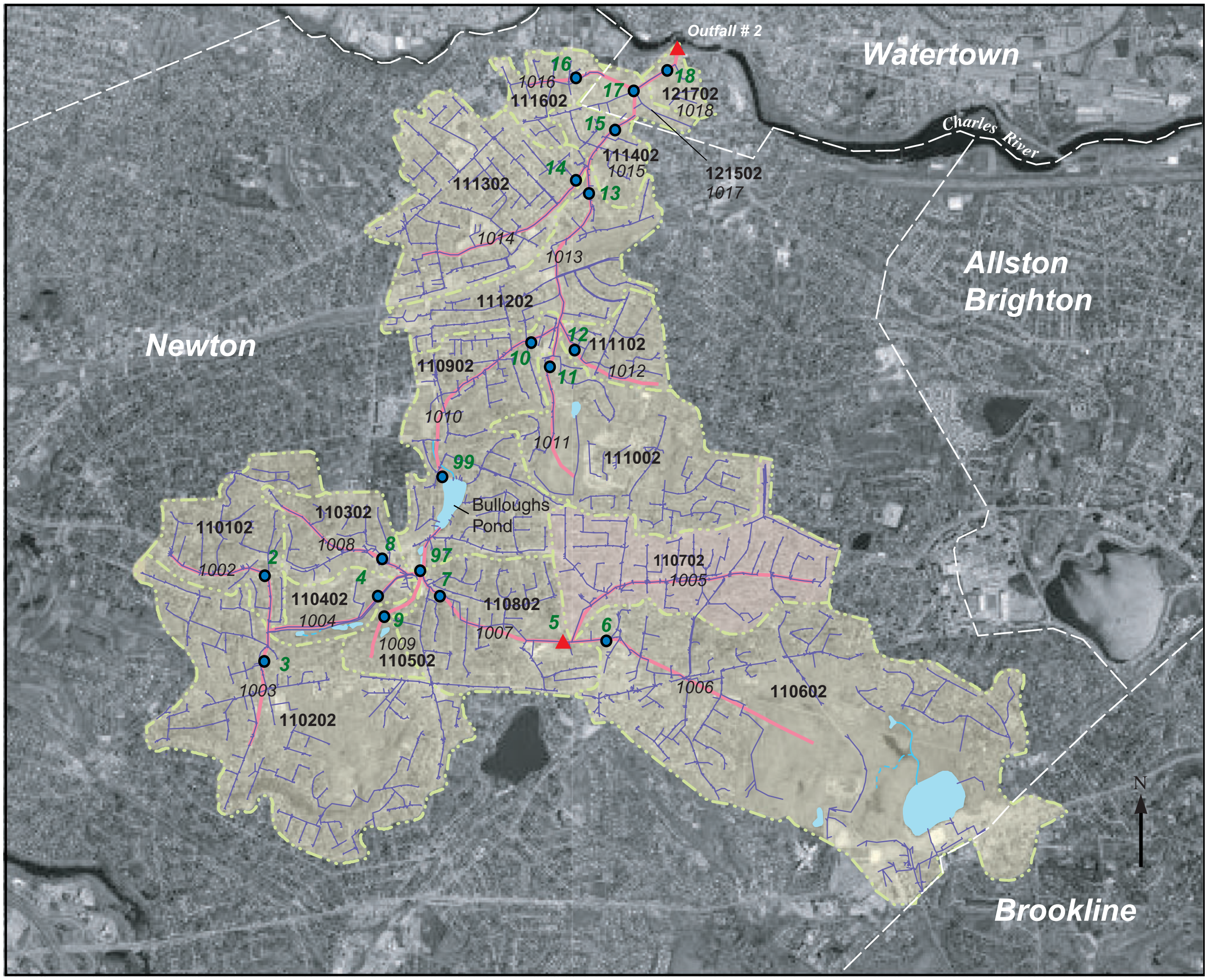
Socolow, R.S., Whitley, J.S., Murino, Domenic, Jr., and Ramsbey, L.R., 2001, Water Resources Data Report, Massachusetts and Rhode Island, Water Year 2000; U.S. Geological Survey Water-Data Report MA-RI-00-1, 459 p.

Trombley, T.J., 1991, Quality of water from public-supply wells in Massachusetts, 1975–86: U.S. Geological Survey Water-Resources Investigations Report 91-4129, 63 p.

Woodward-Clyde Consultants, 1989, Synoptic analysis of selected rainfall gages throughout the United States: Oakland, CA, Report to U.S. Environmental Protection Agency, Woodward-Clyde Consultants, variously paged.

U.S. Department of Agriculture, 1995, Soil Survey Geographic (SSURGO) Data Base, Data Use Information: Natural Resources Conservation Service, National Soil Survey Center, Miscellaneous Publication Number 1527, variously paged.

Appendix 1: Model areas and schematics of the StormWater Management Model (SWMM) elements used to represent the model areas



EXPLANATION

SURFACE WATER

MODEL BASIN

DRAINAGE AREA OF SINGLE FAMILY LAND-USE SUBBASIN

SUBCATCHMENT BOUNDARIES

SIMULATED DRAINAGE NETWORK

STORM-DRAIN NETWORK

119 NODE AND IDENTIFICATION NUMBER

1118 CONDUIT NUMBER

850040 SUBCATCHMENT NUMBER

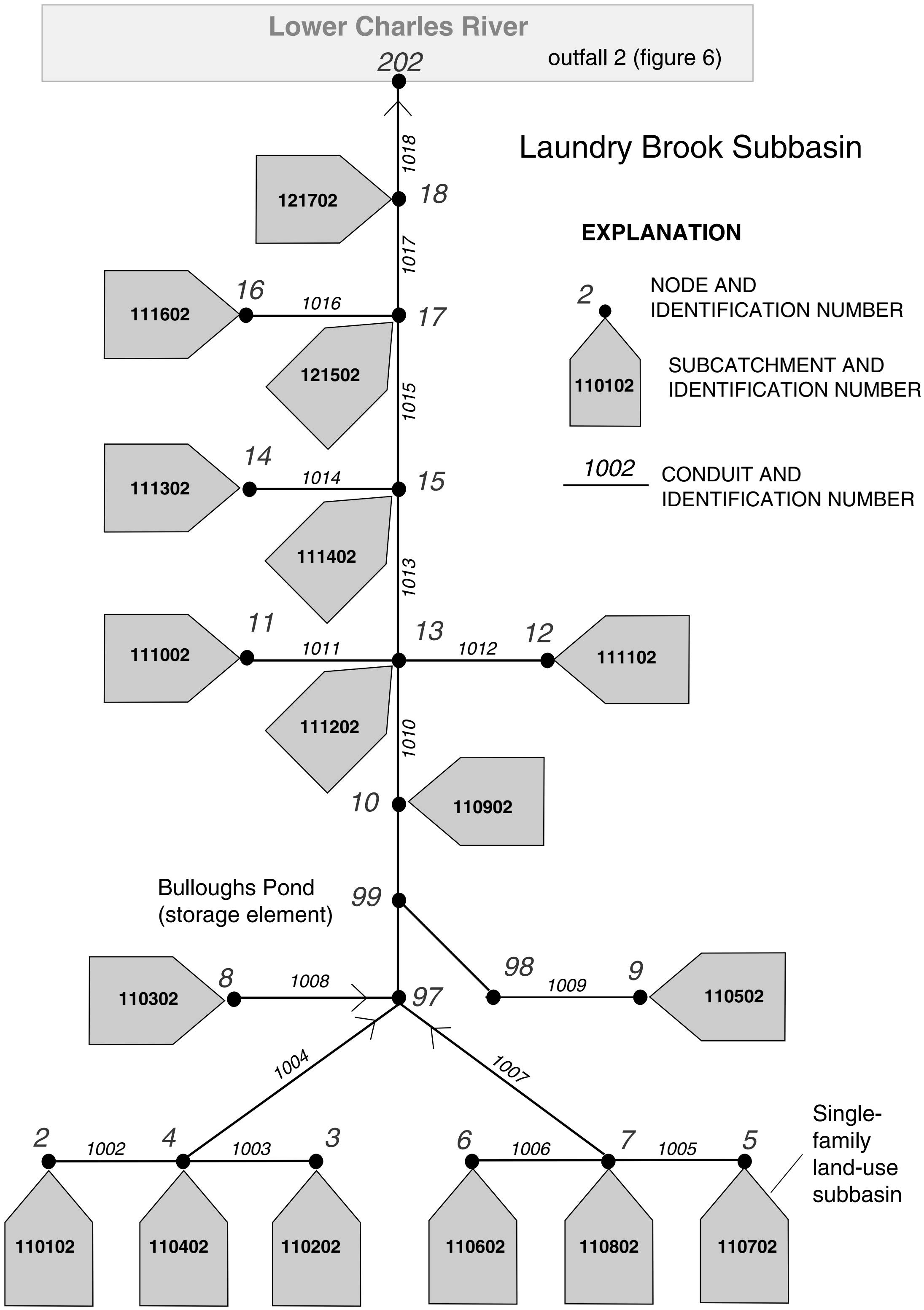
STREAM-GAGE STATION

0 1 2 MILES

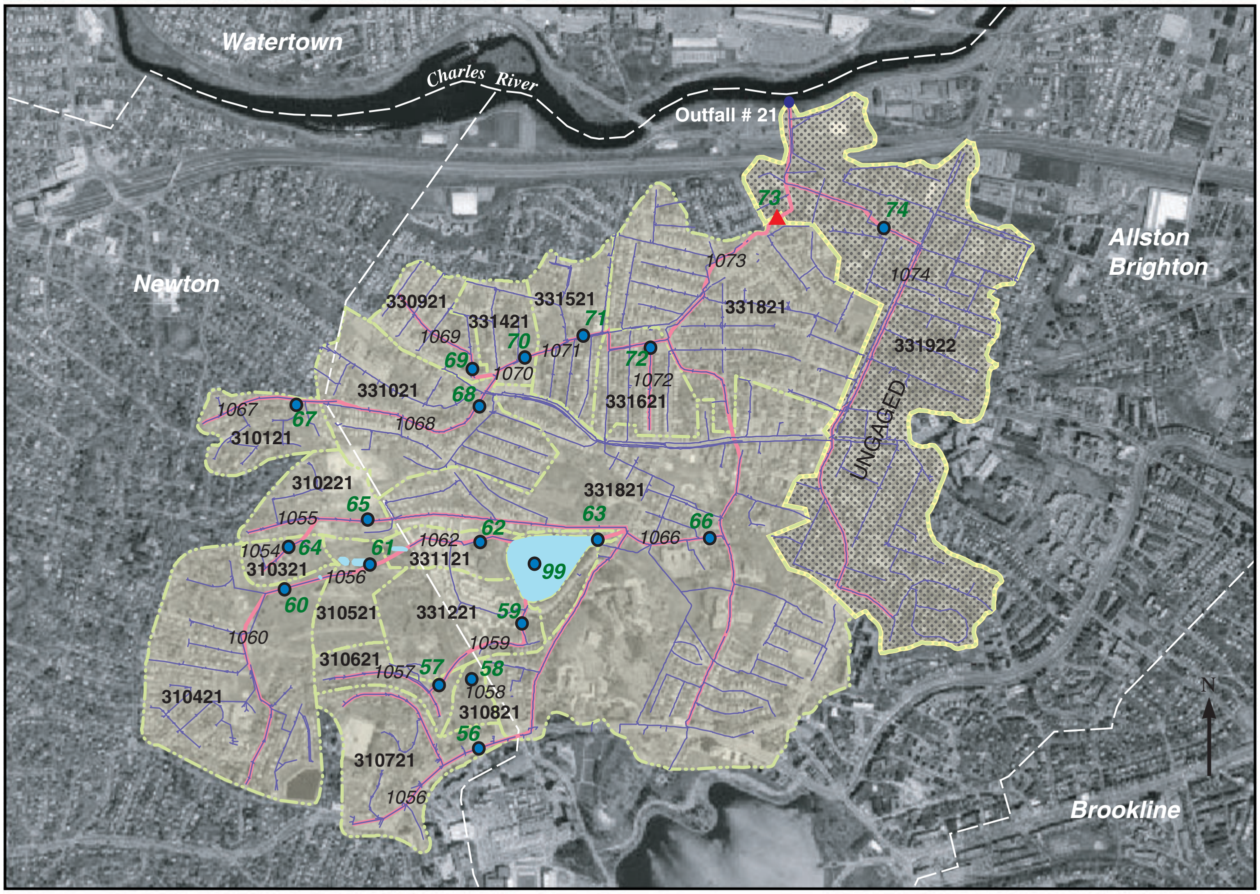
0 1 2 KILOMETERS

Information from City of Newton MIS Department and
City of Watertown Department of Public Works
Massachusetts State Plane Projection
North American Datum 1983

Appendix 1A. Laundry Brook Subbasin, lower Charles River Watershed, Massachusetts.



Appendix 1B. Schematic of StormWater Management Model (SWMM) elements used to represent Laundry Brook Subbasin, lower Charles River Watershed, Massachusetts.



0 1 MILE

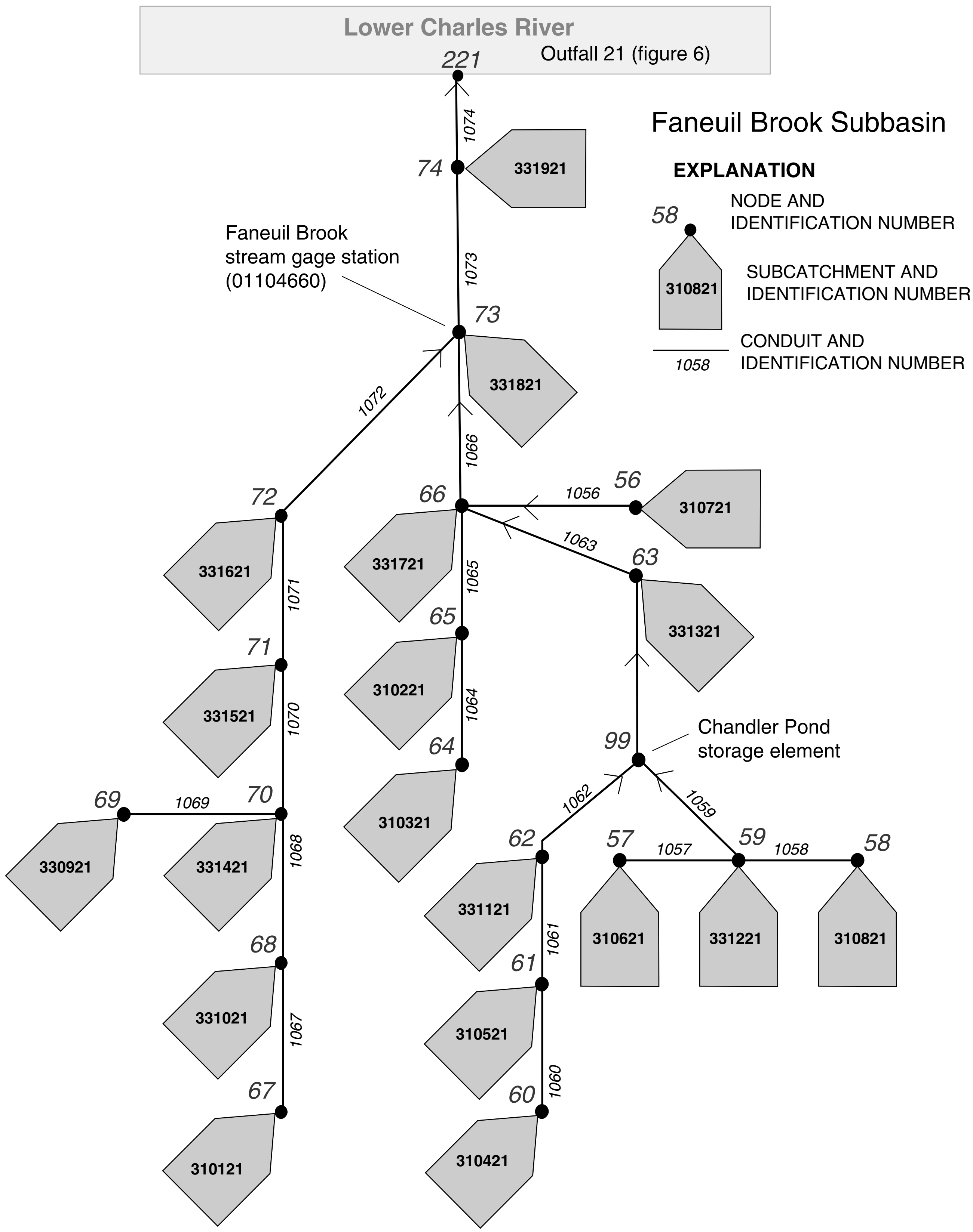
0 1 KILOMETER

EXPLANATION

- SURFACE WATER
- UNGAGED AREA AND BOUNDARY
- MODEL BASIN
- SUBCATCHMENT BOUNDARIES
- SIMULATED DRAINAGE NETWORK
- STORMDRAIN NETWORK
- 119 NODE AND IDENTIFICATION NUMBER
- *1118* CONDUIT NUMBER
- **850040** SUBCATCHMENT NUMBER
- STREAM-GAGE STATION

Information from City of Newton MIS Department and
Boston Water and Sewer Commission
Massachusetts State Plane Projection
North American Datum 1983

Appendix 1C. Faneuil Brook Subbasin, lower Charles River Watershed, Massachusetts.



Appendix 1D. Schematic of StormWater Management Model (SWMM) elements used to represent Faneuil Brook Subbasin, lower Charles River Watershed, Massachusetts.

Information from City of Cambridge MIS Department and
City of Cambridge Department of Public Works
Massachusetts State Plane Projection
North American Datum 1983

0 1,000 2,000 FEET
0 250 500 METERS

EXPLANATION

MULTIFAMILY LAND-USE SUBBASIN

OTHER DRAINAGE AREA TO OUTFALL 40

COMMERCIAL LAND-USE SUBBASIN

ADDITIONAL DRAINAGE AREA TO COMMERCIAL LAND-USE SUBBASIN THAT COULD CONTRIBUTE TO COMMERCIAL LAND-USE SUBBASIN BASED ON TOPOGRAPHIC DIVIDE

SIMULATED STORM-DRAIN NETWORK

STORM-DRAIN NETWORK

119 NODE AND IDENTIFICATION NUMBER

1118 CONDUIT NUMBER

850040 SUBCATCHMENT NUMBER

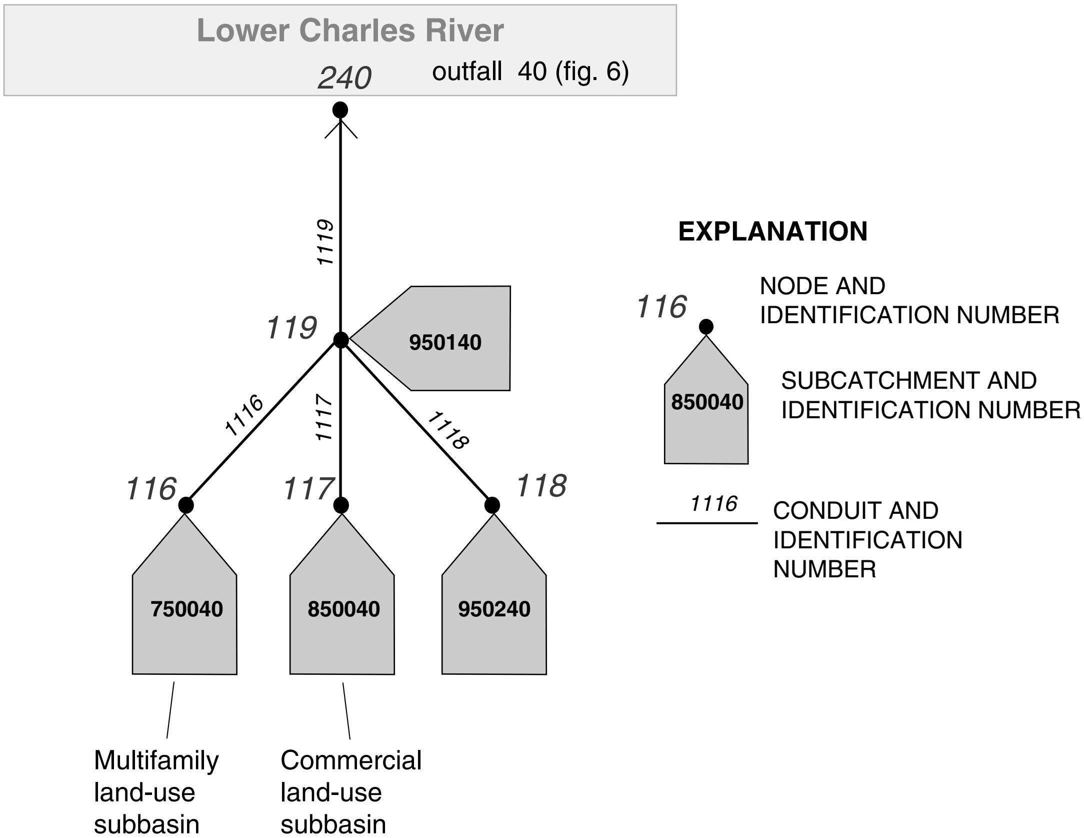
TOPOGRAPHIC CONTOUR—In 10-foot increments. Datum is sea level. (NAD 1983)

BOUNDARY OF DRAINAGES TO OUTFALL 40

SUBBASIN BOUNDARY

STREAM-GAGE STATION

Appendix 1E. Multifamily and commercial land-use subbasins, and drainage area to outfall 40 (fig. 6), lower Charles River Watershed, Massachusetts.



Appendix 1F. Schematic of StormWater Management Model (SWMM) elements used to represent multifamily and commercial land-use subbasins, lower Charles River Watershed, Massachusetts.

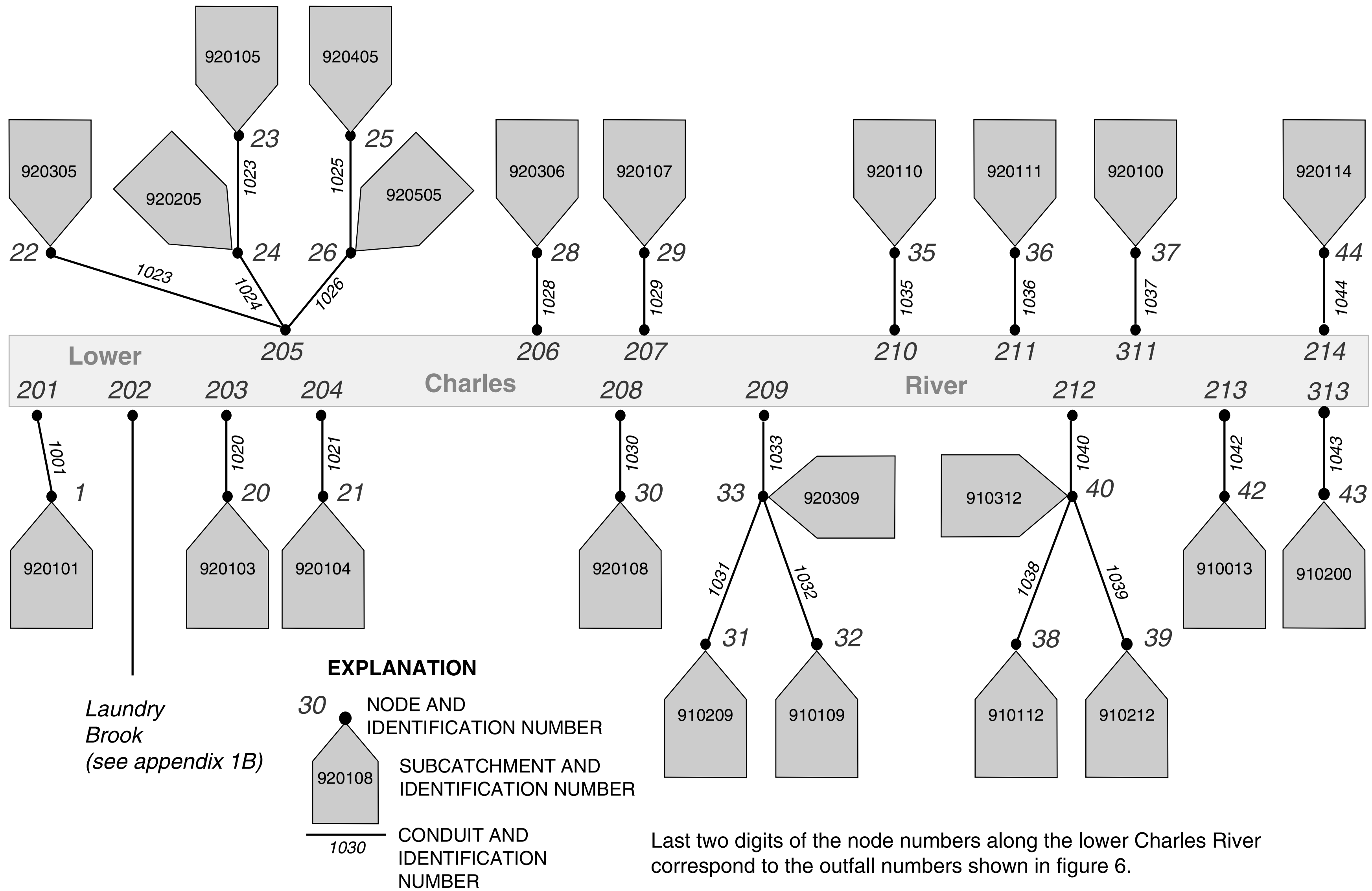


Figure 1G. Schematic of StormWater Management Model (SWMM) elements used to represent ungaged subbasin (outfalls 1 through 14, fig. 6), lower Charles River Watershed, Massachusetts.

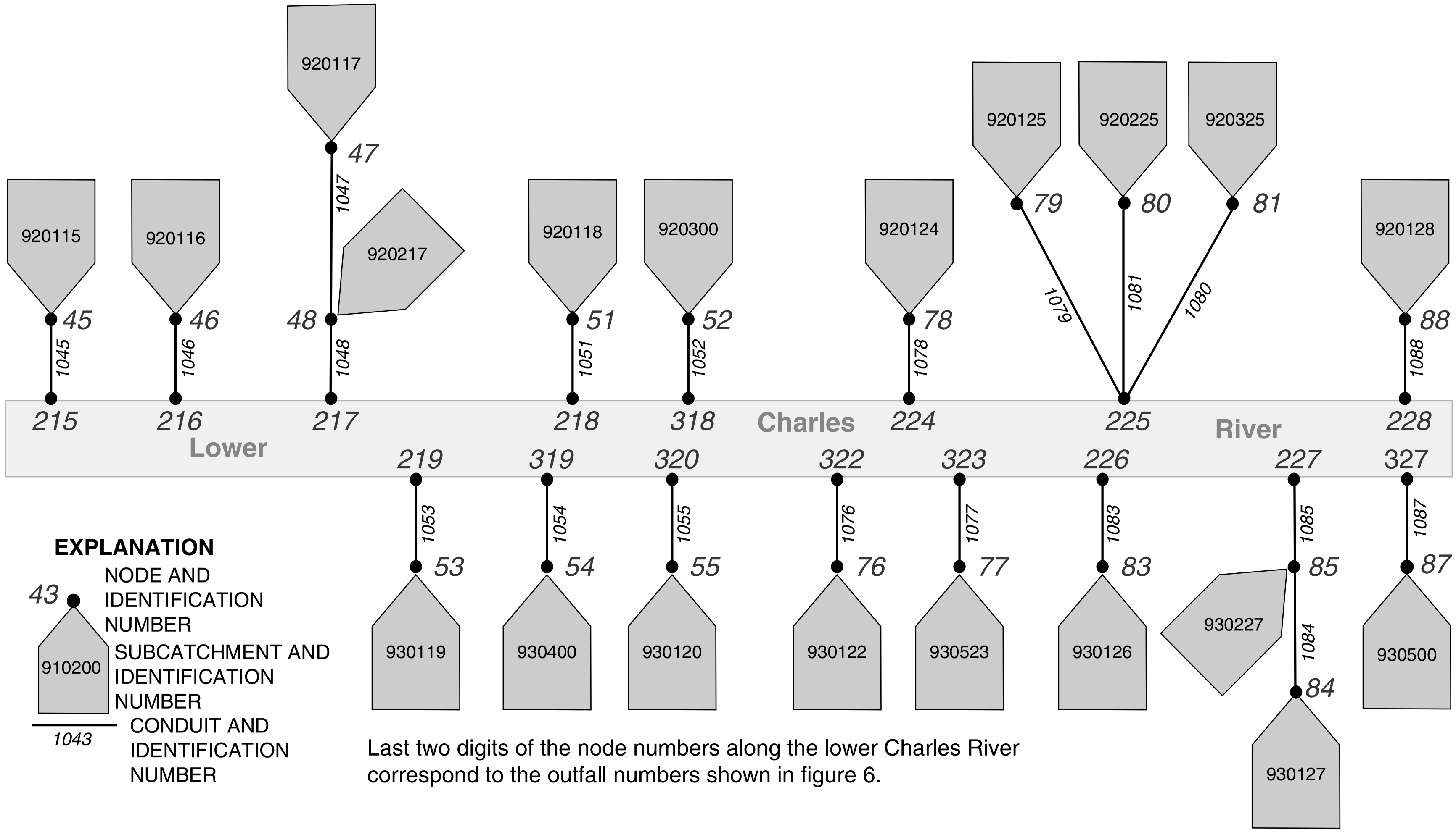


Figure 1G. Schematic of StormWater Management Model (SWMM) elements used to represent ungaged subbasin (outfalls 15 through 28, fig. 6), lower Charles River Watershed, Massachusetts—*Continued.*

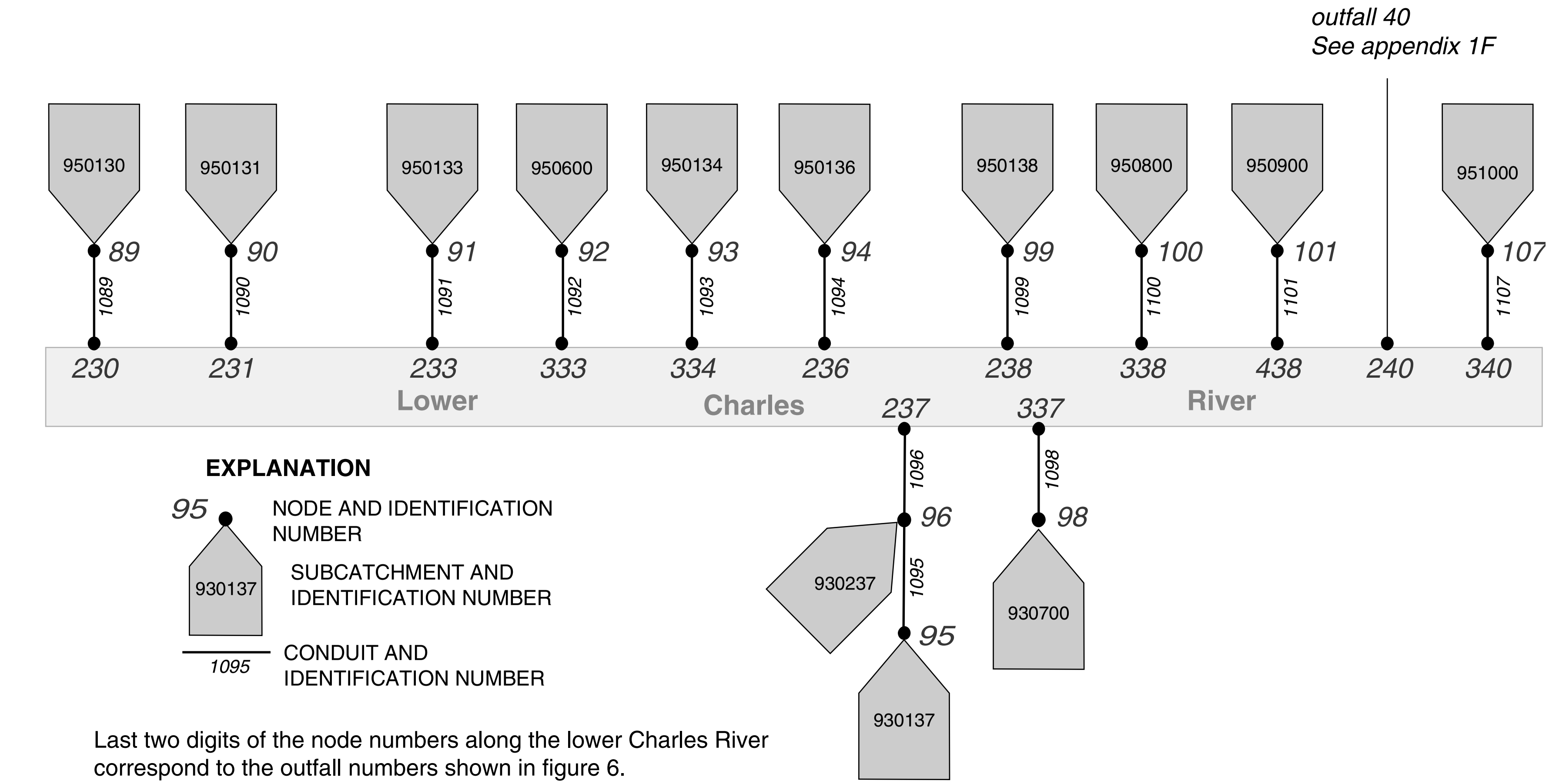


Figure 1G. Schematic of StormWater Management Model (SWMM) elements used to represent ungaged subbasin (outfalls 30 through 40, fig. 6), lower Charles River Watershed, Massachusetts—*Continued.*

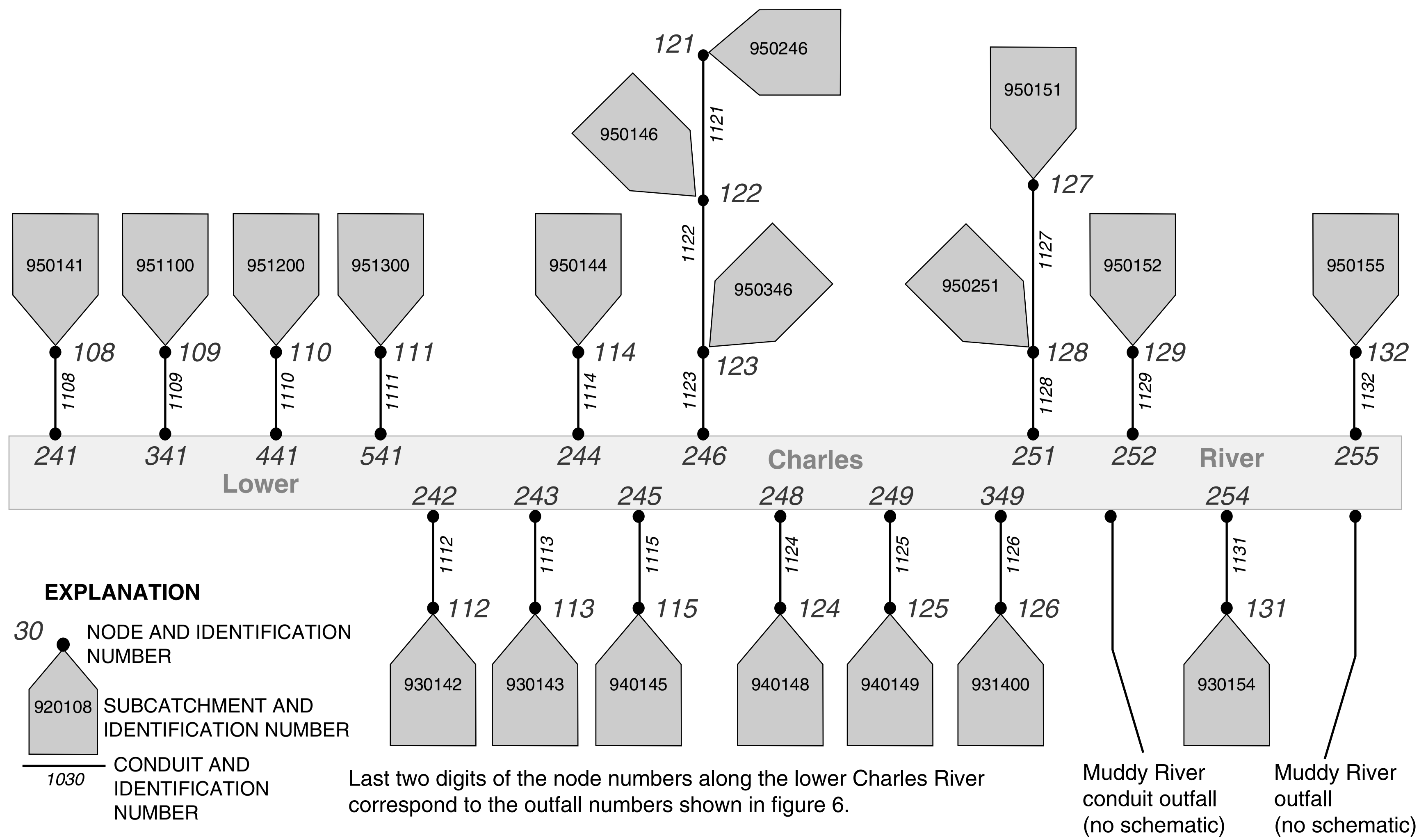


Figure 1G. Schematic of StormWater Management Model (SWMM) elements used to represent ungaged subbasin (outfalls 41 through 55, fig. 6), lower Charles River Watershed, Massachusetts—*Continued.*

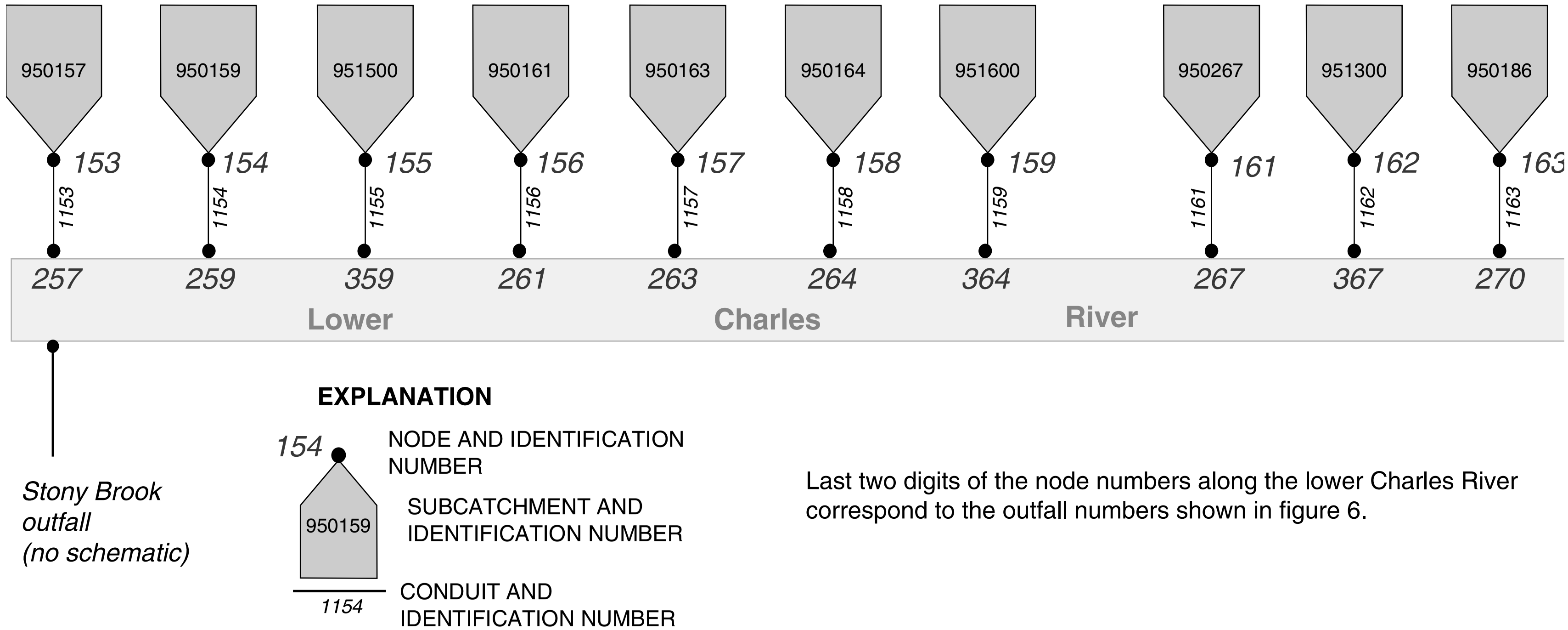


Figure 1G. Schematic of StormWater Management Model (SWMM) elements used to represent ungaged subbasin (outfalls 57 through 70, fig. 6), lower Charles River Watershed, Massachusetts—*Continued.*

Appendix 2: Rainfall characteristics of storms at BWSC-CS4, lower Charles River Watershed, 2000 water year

Appendix 2. Precipitation characteristics of storms at BWSC-CS4 in the lower Charles River Watershed, Massachusetts, 2000 water year

[Gage location shown in fig. 10. Shaded cells indicate storm used for model calibration**. Range:** Range was calculated from the 5 precipitation gages in or near the basin: CDPW-HS2, USGS-CR3, BWSC-CS4, MWRA-WS5, and BWSC-WS6. hrs, hours; **>,** actual value is greater than value shown]

Date	Dura-tion (hrs)	Volume (inches)		Intensity (inches per hour)			Time since specified precipitation totals (hours)					Antecedent precipitation amounts since specified time in hours (inches)			
		Total	Range	Average	Maxi-mum	Range	>0.01	>0.1	>0.2	>0.5	>1.0	24	48	72	168
10-04-1999	19	0.74	0.67–0.82	0.04	0.42	0.30–0.47	90	90	90	407	407	0	0	0	0.45
10-10-1999	7	.25	0.21–0.26	.04	.08	0.06–0.09	28	130	130	130	556	0	.06	.06	.80
10-13-1999	13	.51	0.51–1.01	.04	.13	0.13–0.58	75	75	75	212	638	0	0	0	.31
10-17-1999	10	1.24	1.14–1.35	.12	.28	0.22–0.28	84	84	84	84	735	0	0	0	.51
10-20-1999	17	.71	0.69–0.90	.04	.18	0.15–0.24	44	44	44	44	44	0	.23	1.24	1.75
10-23-1999	5	.35	0.35–0.41	.07	.15	0.14–0.18	52	52	52	52	113	0	0	.71	1.95
11-02-1999	18	.80	0.74–1.27	.04	.21	0.15–0.29	246	246	246	303	364	0	0	0	0
11-10-1999	7	.41	0.37–0.41	.06	.14	0.12–0.14	181	181	181	181	563	0	0	0	0
11-12-1999	12	.14	0.12–0.14	.01	.09	0.08–0.09	42	42	42	230	612	0	.37	.41	.41
11-14-1999	3	.10	0.09–0.12	.03	.06	0.06–0.08	30	30	84	272	654	0	.14	.14	.55
11-20-1999	2	.15	0.13–0.20	.08	.09	0.09–0.11	147	147	234	422	804	0	0	0	.10
11-25-1999	52	.93	0.81–0.93	.02	.20	0.13–0.20	105	105	341	529	911	0	0	0	.15
11-27-1999	11	.52	0.40–0.52	.05	.19	0.17–0.21	4	23	31	148	387	.10	.19	.41	.56
12-06-1999	21	1.12	1.02–1.33	.05	.29	0.28–0.29	46	215	215	215	1178	0	.02	.02	.06
12-14-1999	34	.41	0.36–0.45	.01	.10	0.07–0.10	15	178	178	178	178	.08	.08	.08	.10
12-20-1999	7	.27	0.24–0.32	.04	.10	0.10–0.15	112	112	112	324	324	0	0	0	.49
1-03-2000	33	1.06	1.02–1.30	.03	.22	0.20–0.27	30	326	326	657	657	0	.03	.03	.03
1-10-2000	6	.70	0.70–0.83	.12	.24	0.23–0.28	12	133	133	133	133	.06	.06	.06	1.12
1-31-2000	3	.31	0.29–0.64	.10	.16	0.12–0.19	16	487	487	487	626	.02	.02	.02	.02
2-14-2000	12	1.22	1.12–1.32	.10	.19	0.19–0.22	59	329	329	819	958	0	0	.07	.07
2-24-2000	50	.78	0.67–0.85	.02	.13	0.11–0.14	54	54	247	247	247	0	0	.16	.16
2-28-2000	11	.21	0.21–0.26	.02	.06	0.06–0.08	29	29	29	29	326	0	.10	.73	.94
3-11-2000	31	1.56	1.48–1.75	.05	.26	0.20–0.28	38	288	288	328	625	0	.01	.01	.06
3-16-2000	12	.98	0.87–1.01	.08	.23	0.21–0.24	94	94	94	94	94	0	0	0	1.57
3-28-2000	6	1.16	1.10–1.53	.19	.31	0.28–0.40	147	228	262	262	368	0	0	0	.01

Appendix 2. Precipitation characteristics of storms at BWSC-CS4 in the lower Charles River Watershed, Massachusetts, 2000 water year—*Continued*

Date	Dura-tion (hrs)	Volume (inches)		Intensity (inches per hour)			Time since specified precipitation totals (hours)					Antecedent precipitation amounts since specified time in hours (inches)			
		Total	Range	Average	Maxi-mum	Range	>0.01	>0.1	>0.2	>0.5	>1.0	24	48	72	168
4-04-2000	32	0.31	0.16–0.35	0.01	0.08	0.07–0.12	138	139	160	164	166	0	0	0	1.35
4-08-2000	14	.74	0.62–1.01	.05	.13	0.13–0.20	35	81	81	97	97	0	.04	.04	.35
4-18-2000	16	.81	0.73–0.88	.05	.11	0.10–0.13	59	157	225	225	336	0	0	.08	.18
4-21-2000	58	3.39	3.20–3.85	.06	.34	0.34–0.45	13	49	49	49	401	.01	.01	.82	.90
4-26-2000	23	.43	0.31–0.58	.02	.08	0.06–0.09	53	53	53	53	53	0	0	.40	3.64
5-10-2000	17	.79	0.62–0.95	.05	.18	0.18–0.34	33	190	318	394	394	0	.05	.05	.11
5-13-2000	4	.24	0.20–0.46	.06	.10	0.09–0.29	70	70	70	70	481	0	0	.21	.84
5-18-2000	31	.57	0.44–0.60	.02	.11	0.08–0.14	104	104	104	178	589	0	0	0	.24
5-23-2000	5	.32	0.21–0.39	.06	.12	0.08–0.20	77	77	77	77	697	0	0	0	.57
5-24-2000	22	.80	0.71–1.05	.04	.29	0.29–0.45	20	20	20	102	722	.30	.32	.32	.89
6-02-2000	4	.56	0.30–0.96	.14	.40	0.14–0.78	199	212	212	212	956	0	0	0	0
6-06-2000	26	4.40	3.58–4.75	.17	.50	0.37–0.53	20	82	82	82	1042	.04	.04	.04	.60
6-11-2000	12	.80	0.73–0.92	.07	.37	0.28–0.55	108	108	108	108	108	0	0	0	4.44
6-18-2000	9	.14	0.13–0.22	.02	.10	0.07–0.14	20	20	20	148	268	.43	.43	.43	1.36
6-27-2000	5	.21	0.14–1.88	.04	.13	0.10–1.43	20	207	236	364	484	.02	.02	.02	.05
7-09-2000	4	.49	0.42–0.57	.12	.28	0.21–0.31	81	251	288	657	777	0	0	0	.04
7-16-2000	3	.35	0.27–0.82	.12	.21	0.16–0.42	145	145	145	806	926	0	0	0	.49
7-18-2000	3	.75	0.38–0.94	.25	.55	0.30–0.82	60	60	60	869	989	0	0	.35	.35
7-22-2000	7	.32	0.12–0.97	.05	.20	0.06–0.49	77	77	77	77	1069	0	0	0	1.10
7-26-2000	30	2.10	1.74–2.10	.07	.42	0.32–0.42	98	98	98	182	1174	0	0	0	.32
7-30-2000	18	.76	0.70–0.90	.04	.17	0.17–0.23	73	73	73	73	73	0	0	0	2.10
8-14-2000	12	.34	0.34–0.47	.03	.11	0.07–0.22	90	326	326	326	417	0	0	0	.06
8-16-2000	2	.27	0.27–0.46	.14	.22	0.22–0.42	32	45	45	383	474	0	.12	.35	.40
8-18-2000	8	.25	0.22–0.34	.03	.07	0.06–0.10	58	58	58	443	534	0	0	.27	.62
8-23-2000	3	.37	0.31–0.46	.12	.19	0.19–0.23	110	110	110	561	652	0	0	0	.25
9-02-2000	9	.46	0.16–0.91	.05	.30	0.11–0.58	224	224	224	788	879	0	0	0	0
9-15-2000	9	1.50	1.34–1.50	.17	.75	0.49–0.75	47	47	304	1101	1192	0	.02	.12	.12
9-19-2000	13	.52	0.46–0.80	.04	.32	0.21–0.50	99	99	99	99	99	0	0	0	1.62
9-26-2000	19	.42	0.42–0.69	.02	.08	0.08–0.14	141	141	141	141	253	0	0	0	.52

Appendix 3: Observed and simulated runoff volume and peak discharge for calibration storms

Appendix 3A. Observed and simulated runoff volume and peak discharge for calibration storms at the single-family land-use subbasin, lower Charles River Watershed, Massachusetts

[ft^3/s, cubic feet per second]

Date	Runoff volume (inch)			Peak discharge (ft^3/s)		
	Observed	Simulated	Percent difference	Observed	Simulated	Percent difference
11-10-1999	0.048	0.072	50	7.5	4.6	-39
11-25-1999	.051	.078	53	9.0	6.0	-33
12-06-1999	.147	.185	26	21.0	16.0	-24
12-15-1999	.051	.080	57	4.8	3.2	-33
12-20-1999	.037	.055	49	8.0	5.0	-38
1-04-2000	.184	.184	0	9.7	10.0	3.1
1-10-2000	.186	.156	-16	15.0	10.0	-33
4-04-2000	.038	.075	97	3.2	3.2	0
4-08-2000	.135	.140	3.7	11.0	6.2	-44
4-18-2000	.118	.161	36	4.0	4.3	7.5
4-26-2000	.027	.103	281	1.6	3.0	88
5-18-2000	.080	.106	33	3.8	3.4	-11
6-02-2000	.089	.119	34	15.0	19.0	27
6-11-2000	.196	.178	-9.2	59.0	32.0	-46
7-09-2000	.091	.088	-3.3	16.0	9.8	-39
7-26-2000	.302	.290	-4.0	17.0	16.0	-5.9
7-31-2000	.141	.142	.7	14.0	8.4	-40
8-14-2000	.047	.056	19	2.2	5.5	150
9-19-2000	.070	.079	13	25.0	28.0	12
9-26-2000	.040	.093	133	4.1	4.1	0

Appendix 3B. Observed and simulated runoff volume and peak discharge for calibration storms at the multifamily land-use subbasin, lower Charles River Watershed, Massachusetts

[ft^3/s, cubic feet per second]

Date	Runoff volume (inch)			Peak discharge (ft^3/s)		
	Observed	Simulated	Percent difference	Observed	Simulated	Percent difference
11-25-1999	0.142	0.284	100	2.3	2.3	0
12-06-1999	.910	.994	9.2	8.5	9.9	16
12-14-1999	.436	.313	-28	2.3	1.6	-30
12-20-1999	.239	.183	-23	3.5	2.6	-26
1-04-2000	.904	.877	-3.0	4.7	4.5	-4.3
1-10-2000	.677	.642	-5.2	6.0	5.3	-12
4-18-2000	.412	.551	34	1.5	2.1	40
4-26-2000	.253	.169	-33	1.3	1.0	-23
5-10-2000	.481	.475	-1.2	5.0	5.1	2.0
5-18-2000	.252	.226	-10	2.0	1.8	-10
6-02-2000	.408	.431	5.6	12.0	10.0	-17
6-11-2000	.732	.545	-26	14.0	14.0	0
7-09-2000	.338	.455	35	5.5	7.0	27
7-26-2000	1.456	1.402	-3.7	8.5	10.0	18
7-31-2000	.419	.518	24	3.9	3.4	-13
8-14-2000	.353	.300	-15	7.0	4.2	-40
8-16-2000	.334	.319	-4.5	12.0	7.1	-41
8-23-2000	.221	.280	27	5.5	7.0	27
9-15-2000	1.258	1.261	.2	20.0	25.0	25
9-19-2000	.338	.336	-.6	8.9	9.6	7.9
9-26-2000	.235	.268	14	2.1	4.0	90

Appendix 3C. Observed and simulated runoff volume and peak discharge for calibration storms at the commercial land-use subbasin, lower Charles River Watershed, Massachusetts

[ft^3/s, cubic feet per second]

Date	Runoff volume (inch)			Peak discharge (ft^3/s)		
	Observed	Simulated	Percent difference	Observed	Simulated	Percent difference
4-26-2000	0.713	0.525	-26	1.7	1.1	-35
5-18-2000	.972	.568	-42	3.1	2.6	-16
6-02-2000	.769	.672	-13	7.1	9.1	28
6-11-2000	1.896	.746	-61	7.3	11.0	51
7-09-2000	.775	.668	-14	7.6	4.6	-39
7-27-2000	1.749	1.256	-28	7.6	7.2	-5.3
7-31-2000	.809	.717	-11	3.6	4.2	17
8-13-2000	.639	.603	-5.6	5.3	3.7	-30
8-16-2000	.562	.595	5.9	10.0	8.0	-20
8-23-2000	.475	.577	21	4.8	6.5	35
9-15-2000	1.275	1.145	-10	18.0	19.0	5.6
9-19-2000	.609	.616	1.1	7.4	7.6	2.7
9-26-2000	.559	.582	4.1	2.0	3.4	70

Appendix 3D. Observed and simulated runoff volume and peak discharge for calibration storms at the Laundry Brook Subbasin, lower Charles River Watershed, Massachusetts

[ft^3/s, cubic feet per second]

Date	Runoff volume (inch)			Peak discharge (ft^3/s)		
	Observed	Simulated	Percent difference	Observed	Simulated	Percent difference
11-10-1999	0.028	0.050	79	15	20	33
11-25-1999	.031	.056	81	14	18	29
12-06-1999	.124	.122	-1.6	56	56	0
12-15-1999	.043	.057	33	13	15	15
12-20-1999	.035	.040	14	17	19	12
1-04-2000	.117	.122	4.3	57	48	-16
1-10-2000	.110	.103	-6.4	82	51	-38
4-04-2000	.053	.057	7.5	15	18	20
4-08-2000	.103	.095	-7.8	44	33	-25
4-18-2000	.092	.108	17	20	22	10
4-26-2000	.109	.073	-33	14	15	7.1
5-10-2000	.133	.122	-8.3	77	78	1.3
5-18-2000	.072	.074	2.8	16	16	0
6-02-2000	.059	.082	39	52	72	38
6-11-2000	.129	.121	-6.2	73	96	32
7-09-2000	.046	.061	33	30	42	40
7-26-2000	.231	.202	-13	90	83	-7.8
7-31-2000	.099	.096	-3.0	51	45	-12
8-13-2000	.029	.026	-10	11	17	55
8-16-2000	.039	.049	26	38	77	103
8-23-2000	.034	.033	-2.9	30	34	13
9-15-2000	.175	.175	0	100	130	30
9-19-2000	.062	.051	-18	72	76	5.6
9-26-2000	.036	.060	67	15	22	47

Appendix 3E. Observed and simulated runoff volume and peak discharge for calibration storms at the Faneuil Brook Subbasin, lower Charles River Watershed, Massachusetts

[ft^3/s, cubic feet per second; in, inch]

Date	Runoff volume (inch)			Peak discharge (ft^3/s)		
	Observed	Simulated	Percent difference	Observed	Simulated	Percent difference
11-10-1999	0.061	0.056	-8	13.0	15.0	15
11-25-1999	.057	.059	4	15.0	11.0	-27
12-06-1999	.141	.129	-9	25.0	39.0	56
12-15-1999	.075	.068	-9	8.9	8.6	-3
12-20-1999	.049	.049	0	14.0	14.0	0
1-04-2000	.159	.126	-21	27.0	18.0	-33
1-10-2000	.130	.103	-21	30.0	22.0	-27
4-04-2000	.056	.064	14	7.6	6.6	-13
4-09-2000	.077	.108	40	12.0	15.0	25
4-18-2000	.107	.113	6	9.9	9.9	0
4-26-2000	.097	.127	31	7.6	8.2	8
5-18-2000	.075	.087	16	6.5	7.5	15
6-02-2000	.077	.091	18	19.0	36.0	89
6-11-2000	.097	.159	64	39.0	54.0	38
7-09-2000	.093	.076	-18	20.0	21.0	5
7-26-2000	.207	.269	30	30.0	50.0	67
7-31-2000	.097	.122	26	14.0	20.0	43
8-14-2000	.047	.055	17	7.4	11.0	49
8-16-2000	.059	.057	-3	36.0	29.0	-19
8-23-2000	.062	.056	-10	34.0	29.0	-15

Appendix 3F. Observed and simulated runoff volume and peak discharge for calibration storms at the Stony Brook Subbasin, lower Charles River Watershed, Massachusetts

[ft^3/s, cubic feet per second]

Date	Runoff volume (inch)			Peak discharge (ft^3/s)		
	Observed	Simulated	Percent difference	Observed	Simulated	Percent difference
4-04-2000	0.087	0.078	-10	31	18	-42
4-09-2000	.159	.177	11	120	150	25
4-18-2000	.124	.098	-21	76	25	-67
4-26-2000	.192	.081	-58	65	18	-72
5-10-2000	.122	.115	-5.7	85	60	-29
5-18-2000	.098	.090	-8.2	47	31	-34
6-02-2000	.136	.136	0	260	200	-23
6-11-2000	.185	.115	-38	200	110	-45
7-09-2000	.063	.114	81	65	130	100
7-26-2000	.207	.218	5.3	250	250	0
7-31-2000	.104	.133	28	140	120	-14
8-13-2000	.058	.079	36	16	25	56
8-16-2000	.068	.102	50	89	94	5.6
8-23-2000	.071	.091	28	81	62	-23
9-15-2000	.175	.175	0	350	340	-2.9
9-19-2000	.088	.139	58	210	190	-10
9-26-2000	.069	.084	22	53	21	-60