COST-EFFECTIVENESS OF THE U.S. GEOLOGICAL SURVEY'S STREAM-GAGING PROGRAMS IN MASSACHUSETTS AND RHODE ISLAND

By R. A. Gadoury, J. A. Smath, and R. A. Fontaine

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CONVERSION FACTORS

The following factors may be used to convert inch-pound units to the International System of Units (SI).

Multiply inch-pound units	By	To obtain SI Units
	Length	
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
	Area	
square mile (mi^2)	2.59	square kilometer (km^2)
	Volume	
cubic foot (ft^3)	0.2832	cubic decimeter (dm^3)
	Flow	
cubic foot per second (ft^3/s)	0.2832	cubic decimeter per second (dm^3/s)

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ABSTRACT

This report documents the results of a study of the cost-effectiveness of the U.S. Geological Survey's continuous-record stream-gaging programs in Massachusetts and Rhode Island. Data uses and funding sources were identified for 91 gaging stations being operated in Massachusetts and Rhode Island. Cost-effectiveness analyses were performed on 63 continuous-record gaging stations in Massachusetts and 15 stations in Rhode Island, at budgets of $353,000 and $60,500, respectively. Current operations policies result in average standard errors per station of 12.3 percent in Massachusetts and 9.7 percent in Rhode Island. Minimum possible budgets to maintain the present numbers of gaging stations in the two States are estimated to be $340,000 and $59,000, with average standard errors per station of 12.8 percent and 10.0 percent, respectively. If the present budget levels were doubled, average standard errors per station would decrease to 8.1 percent and 4.2 percent, respectively. Further budget increases would not improve the standard errors significantly. Three gaging stations in Massachusetts are being operated to provide data for two special purpose hydrologic studies, and they are planned to be discontinued at the conclusion of the studies. Nine gaging stations were identified for discontinuance because of reduced cooperator funding.

INTRODUCTION

The U.S. Geological Survey is the principal Federal agency collecting surface-water data in the Nation. The collection of these data is a major activity of the Water Resources Division of the Survey. The data are collected in cooperation with State and local governments and other Federal agencies. The Survey is presently (1983) operating approximately 8,000 continuous-record gaging stations throughout the Nation. Some of these records extend back to the turn of the century. Any activity of long standing, such as the collection of surface-water data, should be re-examined at intervals, if not continuously, because of changes in objectives, technology, or external constraints. The last systematic nationwide evaluation of the streamflow information program was completed in 1970 and is documented by Benson and Carter (1973). The Survey is presently (1983) undertaking another nationwide analysis of the stream-gaging program that will be completed over a 5-year period, with 20 percent of the program being analyzed each year. The objective of this analysis is to define and document the most cost-effective means of furnishing continuous-record streamflow information.

Two kinds of streamflow stations are operated—partial record or continuous record. Included in partial-record stations are: (1) Peak-flow data (records of highest stream level or discharge); (2) base-flow data (seepage runs); and (3) miscellaneous discharge measurements (for specific purposes such as calibration of stage-discharge ratings, site investigations, or discharge at the time a water quality sample is collected). Partial-record stations can contribute specific information at a large number of sites.

Continuous-record gaging stations, on the other hand, provide a large amount of information at a specific site. The Survey's mission is to collect data that serves national as well as local needs. National needs can, and often do, exceed the immediate needs of a cooperating local agency. Records collected at continuous-record gaging stations are valuable for:

1. Analysis of present and past floods;

2. analysis of present and past low-flow periods, with local and regional application;

3. determining daily and seasonal flow trends and changes to those trends;

4. providing communities that are dependent upon ground water for water supplies with base-flow records for managing their limited resources;

5. providing long-term record used in the analysis of short-term (partial-record) data;

6. providing the complete range of data needed in the development and management of hydropower; and

7. providing data in real time to serve management needs, from controling floods to restricting water use during droughts.

Record collection is primarily the responsibility of the U.S. Geological Survey. Cooperators support most stations through 50 percent of the funding or equivalent services and often contribute to the data-collection phase by providing supplemental records, such as reservoir releases, diversions, telemetry, or observer readings.

Purpose and Scope

Continuous-record gaging stations cost more to operate than partial-record stations; hence the greater need to periodically review their continued operation. This report presents an analysis of the U.S. Geological Survey's continuous-record stream-gaging program.

For every continuous-record gaging station, the analysis identifies the principal uses of the data and relates these uses to the sources of funding. Gaged sites for which data are no longer needed are identified, as are deficient or unmet data demands. In addition, gaging stations are categorized as to whether the data are available to users in a real-time sense, on a provisional basis, or at the end of the water year.

The second part of the analysis identifies less costly alternative methods of furnishing the needed information; among these are flow-routing models and statistical methods. The stream-gaging activity is not considered a network of observation points, but rather an integrated information system in which data are provided by observation and synthesis.

The final part of the analysis involves the use of Kalman-filtering and mathematical programing techniques to define strategies for operation of the stations. Kalman-filtering techniques are used to compute uncertainty functions (relating the standard errors of computation or estimation of streamflow records to the frequencies of visits to the stream gages) for all stations in the analysis. A steepest descent optimization program uses these uncertainty functions, information on practical stream-gaging routes, the various costs associated with stream gaging, and the total operating budget to identify the visit frequency for each station that will minimize the overall uncertainty in the streamflow. The stream-gaging program that results from this analysis will meet the expressed water-data needs in the most cost-effective manner.

The standard errors of estimate given in the report are those that would occur if daily discharges were computed through the use of methods described in this study. No attempt has been made to estimate standard errors for discharges that are computed by other means. Such errors could differ from the errors computed in the report. The magnitude and direction of the differences would be a function of methods used to account for shifting controls and for estimating discharges during periods of missing record.

This report is organized into five sections, the first being an introduction to the stream-gaging activities in Massachusetts and Rhode Island and to the study itself. The middle three sections contain discussions of individual steps of the analysis. Because of the sequential nature of the steps and the dependence of subsequent steps on the previous results, conclusions are made at the end of each of the middle three sections. The study, including all conclusions, is summarized in the final section.

The Stream-Gaging Programs in Massachusetts and Rhode Island

Daily mean discharge, the principal output of the continuous-record stream-gaging program, has been collected at 132 stations on 102 Massachusetts streams. Data were collected at one station in 1900, at 95 stations (the highest number in any given year) in 1970, at 76 stations in 1983, and will be collected at possibly as few as 67 stations in 1984. Abrupt increases in the number of stations occurred in 1939, following the floods of 1936 and 1938, and in 1962, when a study of the characteristics of peak flows on streams with drainage areas of less than 10 square miles was started (Johnson and Tasker, 1974; Wandle, 1983). Since 1970, the number of stations has gradually declined, as additional data at particular stations were no longer needed. Even as this report was being prepared (1983), the Massachusetts Department of Public Works had evaluated its needs and announced it was withdrawing its share of support of the stream-gaging program. Because replacement funds were not certain, it was necessary to anticipate the discontinuance of nine stations in 1984. The program was budgeted at $371,000 for 76 stations in 1983 and projected to $353,000 for 67 stations in 1984.

Continuous-record streamflow data have been collected at 33 stations in Rhode Island. The stream-gaging program began in 1914 with one station. No stations were operated in 1925-28. From 1938 through 1941, the network increased from one station to ten stations, and remained at ten stations through 1960. A maximum of 22 stations were operated in 1965 and again in 1973. Since 1973, the number of stations has declined to its present level of 15 stations. Five stations were discontinued in 1982 when one cooperating State agency discontinued its funding support, but another State agency found that it needed continuing data at four of those stations and they were re-activated for 1983. The budget for 15 stations in 1984 is $60,500.

The history of the stream-gaging program in Massachusetts and Rhode Island can be reviewed in numbers of stations operated each year (fig. 1). Selected hydrologic data, including drainage area, period of record, and mean annual flow, for 91 stations in Massachusetts and Rhode Island are given in table 1. Station identification numbers used throughout this report (except in table 1) are the middle four digits of the Survey's eight-digit downstream-order station number; the first two digits of the standard Survey station number for all stations used in this report are 01. The last two digits for most stations are 00; if not, the middle four digits are followed by a decimal; for example, 1685 (01168500) or 1681.51 (01168151). Table 1 also provides the official name of each stream gage. In certain parts of the report abbreviated names will be used, either the name of the river or the name of the town the station is near, whichever is most clear in context. The locations of gaging stations in Massachusetts and Rhode Island are shown in figure 2.

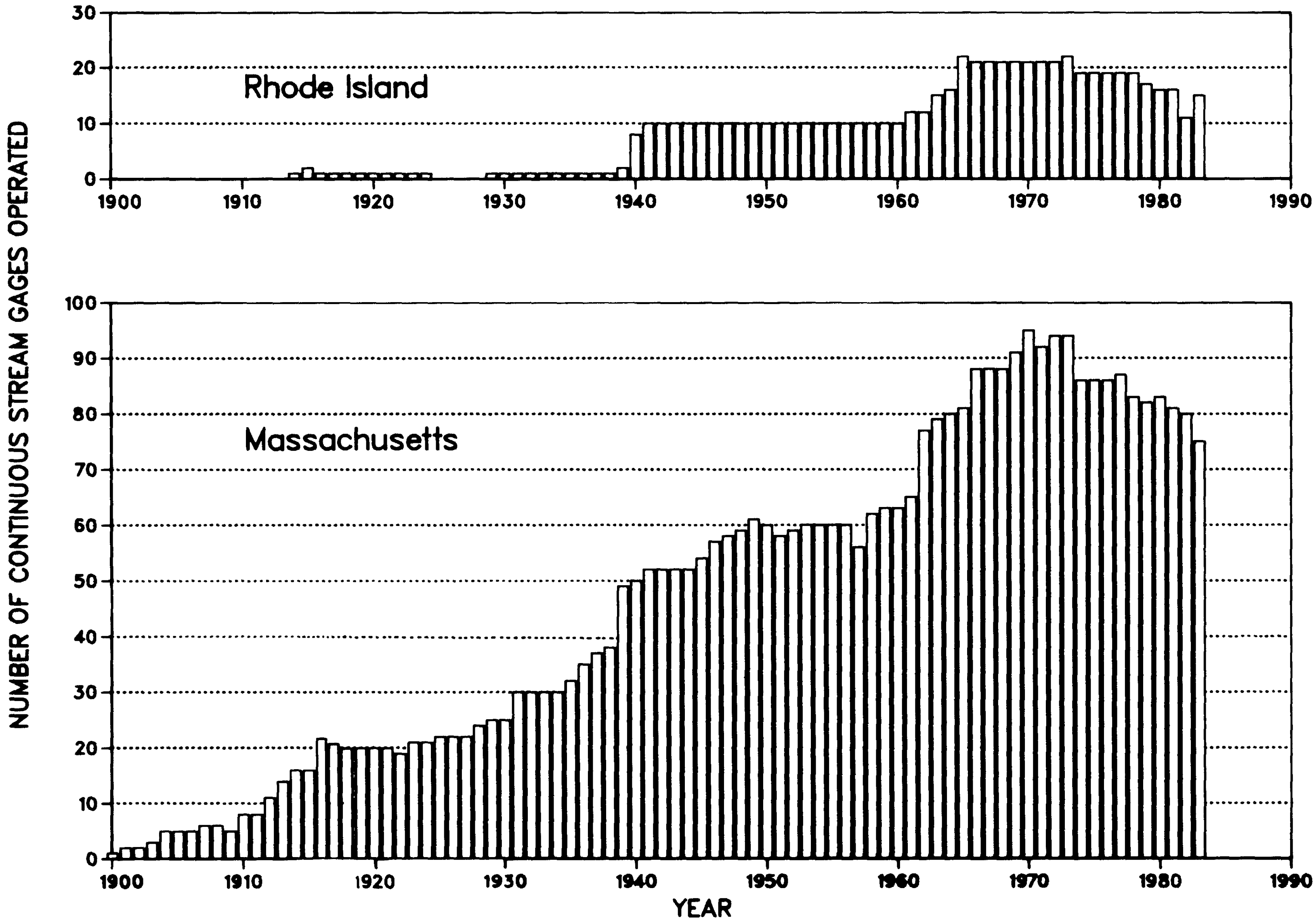

Figure 1.--History of continuous stream gaging in Massachusetts and Rhode Island.

Table 1.—Selected hydrologic data for stations in the Massachusetts and Rhode Island surface-water programs

Station number	Station name	Drainage area (square miles)	Period of record	Mean annual flow (cubic feet per second)
	MASSACHUSETTS			
	Merrimack River Basin			
01094400	North Nashua River at Fitchburg	63.6	October 1972-	119
01094500	North Nashua River near Leominster	110	September 1935-	191
01096000	Squannacook River near West Groton	62.8	October 1949-	109
01096500	Nashua River at East Pepperell	316	October 1935-	558
01096910	Boulder Brook at East Bolton	1.54	June 1971-	3.16
01097000	Assabet River at Maynard	116	July 1941-	182
01097300	Nashoba Brook near Acton	12.7	July 1963-[1]	20.6
01098530	Sudbury River at Saxonville	106	November 1979-	(2)
01099500	Concord River below River Meadow Brook, at Lowell	312	October 1936-	620
01100000	Merrimack River below Concord River, at Lowell	4425	June 1923-[3]	7452
01100600	Shawsheen River near Wilmington	36.5	November 1963-	56.8
	Parker River Basin			
01101000	Parker River at Byfield	21.6	October 1945-	35.6
	Ipswich River Basin			
01101500	Ipswich River at South Middleton	43.4	June 1938-	60.6
01102000	Ipswich River near Ipswich	124	June 1930-	184
	Mystic River Basin			
01102500	Aberjona River at Winchester	24.2	April 1939-	27.3
	Charles River Basin			
01103500	Charles River at Dover	184	October 1937-	298
01104000	Mother Brook at Dedham	(4)	October 1931-	78.0
01104200	Charles River at Wellesley	211	August 1959-	269
01104500	Charles River at Waltham	227	October 1903-October 1909, August 1931	(5) [6]298
	Neponset River Basin			
01105000	Neponset River at Norwood	35.2	October 1939-	52.9
01105500	East Branch Neponset River at Canton	27.2	October 1952-	50.9
	Weymouth Fore River Basin			
01105585	Town Brook at Quincy	4.25	September 1972-	8.11

Table 1.—Selected hydrologic data for stations in the Massachusetts and Rhode Island surface-water programs (continued)

Station number	Station name	Drainage area (square miles)	Period of record	Mean annual flow (cubic feet per second)
	MASSACHUSETTS (Continued)			
	Weymouth Back River Basin			
01105600	Old Swamp River near South Weymouth	4.29	May 1966-	8.92
	North River Basin			
01105730	Indian Head River at Hanover	30.3	July 1966-	60.2
	Jones River Basin			
01105870	Jones River at Kingston	15.8	August 1966-	30.3
	Herring River Basin			
01105880	Herring River at North Harwich	9.4	June 1966-	9.79
	Buzzards Bay			
01105884	Red Brook above Route 25 near Wareham	—	June 1981-	(2)
01105885	Red Brook below Route 25 near Wareham	—	September 1981-	(2)
	Taunton River Basin			
01108500	Wading River at West Mansfield	19.2	October 1953-	32.0
01109000	Wading River near Norton	42.4	June 1925-	72.5
01109060	Threemile River at North Dighton	83.8	July 1966-	169
01109070	Segreganset River near Dighton	10.6	July 1966-	22.0
	Blackstone River Basin			
01110000	Quinsigamond River at North Grafton	25.2	October 1939-	40.8
01111200	West River below West Hill Dam, near Uxbridge	27.9	March 1962-	45.1
	Quinebaug River Basin			
01123360	Quinebaug River below East Brimfield Dam, at Fiskdale	67.5	October 1972-	130
01123600	Quinebaug River below Westville Dam, near Southbridge	99.1	October 1962-	166
01124350	French River below Hodges Village Dam, at Hodges Village	31.0	March 1962-	53.8
01124500	Little River near Oxford	27.7	July 1939-	47.7

Table 1.—Selected hydrologic data for stations in the Massachusetts and Rhode Island surface-water programs (continued)

Station number	Station name	Drainage area (square miles)	Period of record	Mean annual flow (cubic feet per second)
			MASSACHUSETTS (Continued)	
			Connecticut River Basin	
01162000	Millers River near Winchendon	83.0	June 1916-	142
01162500	Priest Brook near Winchendon	19.4	May 1916-[7]	32.5
01163200	Otter River at Otter River	34.2	December 1964-	60.0
01164000	Millers River at South Royalston	187	July 1939-	320
01165000	East Branch Tully River near Athol	50.4	October 1915-[8]	80.8
01165300	Lake Rohunta Outlet near Athol	20.3	December 1964-	35.2
01166500	Millers River at Erving	375	August 1914 to June 1915[9], July 1915-	625
01167000	Connecticut River at Turners Falls	7163	January 1915-	11,830
01168151	Deerfield River near Rowe	254	May 1974-	756
01168500	Deerfield River at Charlemont	362	June 1913-	897
01169000	North River at Shattuckville	88.4	October 1939-	182
01169900	South River near Conway	24.0	June 1966-	52.7
01170000	Deerfield River near West Deerfield	558	1904-06[10] October 1940-	[11]1283
01170100	Green River near Colrain	41.4	October 1967-	93.5
01170500	Connecticut River at Montague City	7865	March 1904-	13,720
01171300	Fort River near Amherst	36.4	June 1966-	62.3
01171500	Mill River at Northampton	54.0	October 1938-	96.1
01172500	Ware River near Barre	55.0	July 1946-	91.4
01173000	Ware River at Intake Works, near Barre	96.8	January 1928-	165
01173500	Ware River at Gibbs Crossing	199	August 1912-	([12])
01174500	East Branch Swift River near Hardwick	43.7	January 1937-	68.6
01174600	Cadwell Creek near Pelham	.63	July 1961-	1.12
01174900	Cadwell Creek near Belchertown	2.81	July 1961-	4.73
01175500	Swift River at West Ware	188	July 1910- September 1912[9], October 1912-	([13])
01175670	Sevenmile River near Spencer	8.58	1960[14], October 1960-	14.0
01176000	Quaboag River at West Brimfield	151	August 1909 to July 1912, August 1912-	([9]) [15]242
01177000	Chicopee River at Indian Orchard	688	August 1928-	897
01179500	Westfield River at Knightville	162	August 1909-	326
01180500	Middle Branch Westfield River at Goss Heights	52.6	July 1910-	105
01181000	West Branch Westfield River at Huntington	93.7	September 1935-	189
01183500	Westfield River near Westfield	497	June 1914-	917
01185500	West Branch Farmington River near New Boston	92.0	May 1913-	182

Table 1.—Selected hydrologic data for stations in the Massachusetts and Rhode Island surface-water programs (continued)

Station number	Station name	Drainage area (square miles)	Period of record	Mean annual flow (cubic feet per second)
	MASSACHUSETTS (Continued)			
	Housatonic River Basin			
01197000	East Branch Housatonic River at Coltsville	57.1	March 1936-	114
01197500	Housatonic River near Great Barrington	280	May 1913-	526
	Hudson River Basin			
01331500	Hoosic River at Adams	46.3	October 1931-	89.6
01332000	North Branch Hoosic River at North Adams	39.0	June 1931-	96.5
01332500	Hoosic River near Williamstown	126	July 1940-	273
01333000	Green River at Williamstown	42.6	September 1949-	83.1
	RHODE ISLAND			
	Blackstone River Basin			
01111300	Nipmuc River near Harrisville	16.0	March 1964-	29.5
01111500	Branch River at Forestdale	91.2	January 1940 to September 1981, October 1982-	169
01112500	Blackstone River at Woonsocket	416	February 1929-	753
	Moshasuck River Basin			
01114000	Moshasuck River at Providence	23.1	June 1963-	40.0
	Woonasquatucket River Basin			
01114500	Woonasquatucket River at Centerdale	38.3	July 1941 to September 1981, October 1982-	71.4
	Pawtuxet River Basin			
01116000	South Branch Pawtuxet River at Washington	63.8	October 1940-	128
01116500	Pawtuxet River at Cranston	200	December 1939 to September 1981, October 1982-	339
	Potowomut River Basin			
01117000	Hunt River near East Greenwich	23.0	August 1940-	44.4

Table 1.—Selected hydrologic data for stations in the Massachusetts and Rhode Island surface-water programs (continued)

Station number	Station name	Drainage area (square miles)	Period of record	Mean annual flow (cubic feet per second)
	RHODE ISLAND (Continued)			
	Pawcatuck River Basin			
01117350	Chipuxet River at West Kingston	9.99	February 1958 to July 1960, September 1973-	19.8
01117420	Usquepaug River near Usquepaug	36.1	February 1958 to July 1960, December 1974-	73.3
01117468	Beaver River near Usquepaug	8.87	December 1974-	20.5
01117500	Pawcatuck River at Wood River Junction	100	October 1940-	191
01117800	Wood River near Arcadia	35.2	January 1964 to September 1981, October 1982-	76.0
01118000	Wood River at Hope Valley	72.4	March 1941-	153
01118500	Pawcatuck River at Westerly	295	November 1940-	566

[1] Occasional low-flow measurements in water years 1962-63.

[2] Less than 5 years of record.

[3] Prior to March 7, 1934, at Boott Mills 1800 feet upstream and 700 feet above mouth of Concord River. Flow included Concord River.

[4] Mother Brook is a diversion from Charles River basin to Neponset River basin.

[5] Figures of average weekly discharge.

[6] Water years 1932-82.

[7] Monthly discharge only October 1917 to July 1918, September 1935 to September 1936.

[8] Monthly discharge only October 1915 to May 1916.

[9] Twice daily gage heights and corresponding discharges.

[10] Months of March to November 1904, January, March to December 1905.

[11] Water years 1941-81.

[12] Affected by diversions from basin since 1931.

[13] Completely regulated by Quabbin Reservoir since 1939.

[14] Occasional low-flow measurements.

[15] Water years 1913-81.

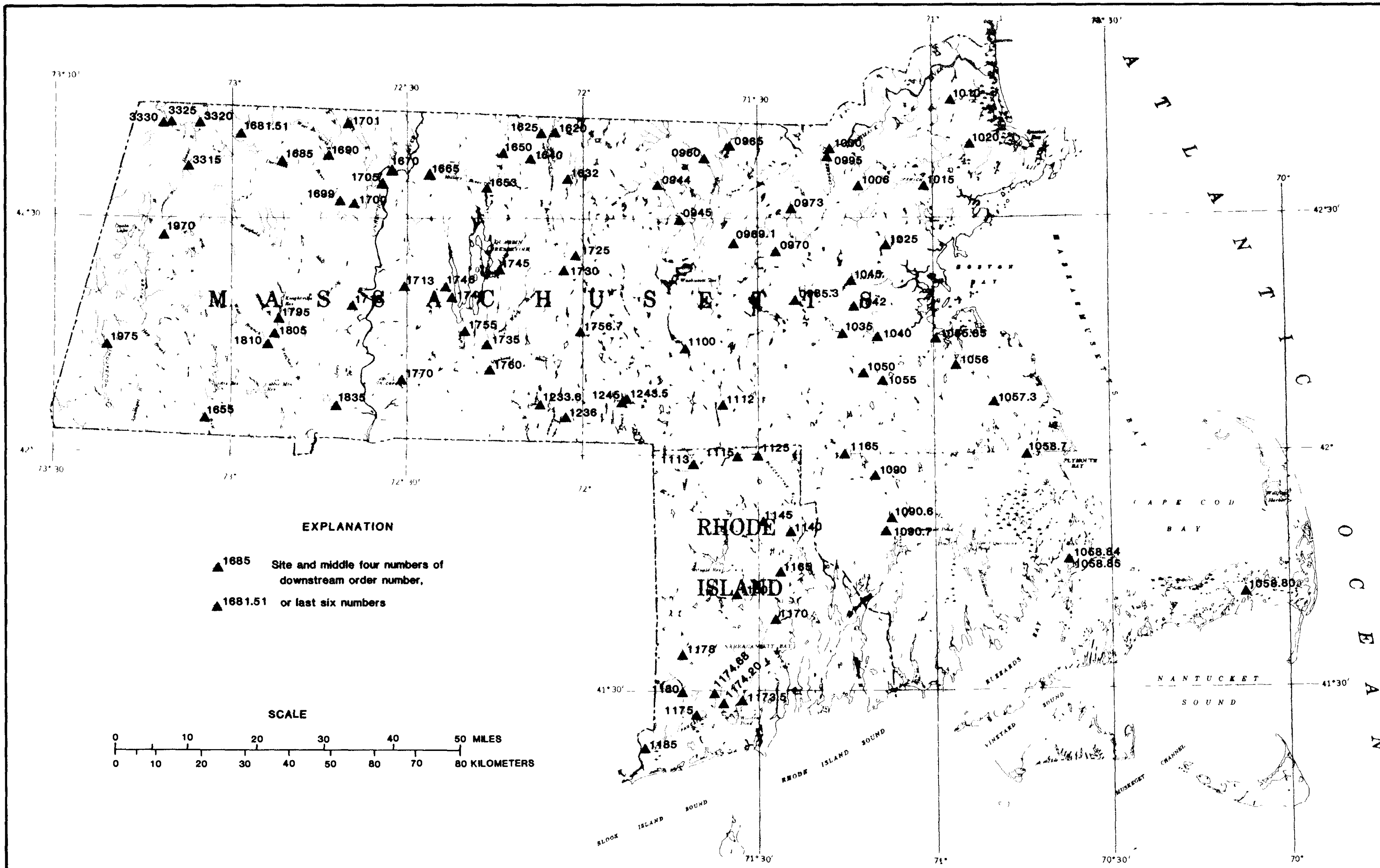

Figure 2.--Location of continuous-record gaging stations in Massachusetts and Rhode Island

USES, FUNDING, AND AVAILABILITY OF CONTINUOUS STREAMFLOW DATA

The value of a stream gage is determined by the uses that are made of the data that are produced from the gage. The uses of the data from each gage in the Massachusetts and Rhode Island programs were identified by a survey of known data users. The survey documented the importance of each gage and identified gaging stations that might be considered for discontinuance.

Data uses identified by the survey were divided into nine categories, defined below. The sources of funding for each gage and the frequency at which data are provided to the users were also compiled.

Data-Use Categories

The following definitions were used to categorize each known use of streamflow data for each continuous-record stream gage.

Regional Hydrology

For data to be useful in defining regional hydrology, a gaging station must be largely unaffected by manmade storage or diversion. In this class of uses, the effects of man on streamflow are not necessarily small, but the effects are limited to those caused primarily by land-use and climatic changes. Large amounts of manmade storage may exist in the basin, providing the outflow is uncontrolled. These stations are useful in developing regionally transferable information about the relationship between basin characteristics and streamflow.

Fifteen stations in Massachusetts and six in Rhode Island are classified in the regional hydrology data-use category. One station in Rhode Island is used to indicate current hydrologic conditions.

Hydrologic Systems

Stations that are used for accounting--that is, to define current hydrologic conditions and the sources, sinks, and fluxes of water through hydrologic systems including regulated systems, are designated as hydrologic systems stations. They include diversions and return flows and stations that are useful for defining the interaction of water systems.

Fifty-eight stations in Massachusetts and five in Rhode Island are included in the hydrologic systems category. Two stations are used by FERC (Federal Energy Regulatory Commission) to monitor the compliance of control structures to downstream flow requirements. One station is operated to ensure the compliance of wastewater-treatment plants to State-issued permits. One station in Massachusetts is used in the District's monthly report of current conditions of streamflow, ground-water levels, and reservoir storage.

Legal Obligations

Some stations provide records of flows for the verification or enforcement of existing treaties, compacts, and decrees. The legal obligation category contains only those stations that the Survey is required to operate to satisfy a legal responsibility.

There are no stations in the Massachusetts or Rhode Island programs used to fulfill a legal responsibility of the Survey.

Planning and Design

Gaging stations in this category of data use are used for the planning and design of a specific project—for example, a dam, levee, floodwall, navigation system, water-supply diversion, hydropower plant, or waste-treatment facility—or group of structures. The planning and design category is limited to those stations that were instituted for such purposes and where these purposes are still valid.

Currently, five stations in Massachusetts and seven in Rhode Island are being operated for planning or design purposes.

Project Operation

Gaging stations in this category are used, on an ongoing basis, to assist water managers in making operational decisions such as reservoir releases, hydropower operations, or diversions. The project operation use generally implies that the data are routinely available to the operators on a rapid-reporting basis. For projects on large streams, data may be needed only every few days.

There are 23 stations in Massachusetts and one in Rhode Island that are used in this manner. All of them are used in flood-control operations.

Hydrologic Forecasts

Gaging stations in this category are regularly used to provide information for hydrologic forecasting. These might be flood forecasts for a specific river reach, or periodic (daily, weekly, monthly, or seasonal) flow-volume forecasts for a specific site or region. The hydrologic forecast category generally implies that the data are routinely available to the forecasters on a rapid-reporting basis. On large streams, data may be needed only every few days.

Nine stations in Massachusetts and two in Rhode Island are included in the hydrologic forecast category and are used for flood forecasting and for forecasting inflows to flood control reservoirs. These data are used principally by the National Weather Service and the U.S. Army Corps of Engineers.

Water-Quality Sites

At some stations water-quality or sediment data are collected by the U.S. Geological Survey or some other agency. At other stations, the availability of streamflow data contributes to the utilization or is essential to the interpretation of the water-quality or sediment data. These stations are designated as water-quality sites.

Twenty-four stations in Massachusetts and ten in Rhode Island are included in the water-quality category. Two NASQAN (National Stream Quality Accounting Network) stations are part of a countrywide network designed to assess water-quality trends of significant streams. The remainder are used for interpreting water-quality samples collected near the station or, in some cases, for assessing changes to long-term trends subsequent to installation of wastewater treatment facilities.

Research

Gaging stations in this category are operated for a particular research or water-resources study. Typically, these are only operated for a few years.

Twelve stations in Massachusetts and five in Rhode Island are currently being used in support of water-resources studies.

Other

In addition to the eight data-use categories described above, one station in Massachusetts is used to provide streamflow information for recreational planning.

Funding

The four sources of funding for the streamflow-data program are:

1. Federal program.—Funds that have been directly allocated to the Survey.
2. OFA (Other Federal Agency) program.—Funds that have been transferred to the Survey by OFA's.
3. Coop program.—Funds that come jointly from Survey cooperative-designated funding and from a non-Federal cooperating State or local government agency. Cooperating agency funds may be in the form of direct services or cash.
4. Other non-Federal.—Funds that are provided entirely by a non-Federal agency or a private concern under the auspices of a Federal agency. In this study, funding from private concerns was limited to licensing and permitting requirements for hydropower development by the Federal Energy Regulatory Commission. Funds in this category are not matched by Survey cooperative funds.

In all four funding categories, the identified sources of funding pertain only to the collection of streamflow data; sources of funding for other activities, particularly collection of water-quality samples, that might be carried out at the site may not necessarily be the same as those identified for stream-gaging stations.

Nine Federal, State and local agencies currently are contributing funds to the Massachusetts and Rhode Island stream-gaging programs.

Frequency of Data Availability

Frequency of data availability refers to the times at which the streamflow data may be furnished to the users. Three distinct possibilities exist. Data can be furnished by direct-access telemetry equipment for immediate use, by periodic release of provisional data, or in publication format through the annual data reports published by the Survey. These three possibilities are designated T, P, and A, respectively, in table 2. Published daily records for two gaging stations in Massachusetts, designated by the letter F, are furnished by other agencies and reviewed by the Survey. In the current Massachusetts and Rhode Island programs, data for 89 of the 91 stations are made available through the annual report, data from 20 stations are available on a real-time basis, and data are released on a provisional basis at four stations.

Data-Use Presentation

Data-use and ancillary information are presented for each continuous-record gaging station in table 2, which includes footnotes to expand the information conveyed.

Data-Use Conclusions

The survey of data uses and funding sources indicated that the 15 stations in the Rhode Island network should be continued in the foreseeable future. In Massachusetts, the survey showed that data are being collected at the Red Brook stations (1058.84 and 1058.85) solely for the highway salting study; these stations will be discontinued at the conclusion of the data-collection period. The Herring River station (1058.80) on Cape Cod was established for the purpose of determining streamflow and ground-water discharge from a glacial outwash plain, which has been designated as a sole-source aquifer, but flow is affected by regulation and influenced by evaporation from several ponds, and has not been very satisfactory for this purpose. This gage should be discontinued if a replacement stream less influenced by regulation or evaporation can be found. There seemed to be little need for Wading River at West Mansfield, Massachusetts (1085), other than as background data for a current study. This station should be discontinued once the present data needs are fulfilled.

Table 2.—Uses, sources of funding, and availability of data at continuous-record gaging stations

	Data-use category									Funding category				
Station number	Regional hydrology	Hydro-logic systems	Legal obliga-tions	Planning and design	Project oper-ation	Hydro-logic forecasts	Water quality	Research	Other	Federal pro-gram	OFA pro-gram	Coop pro-gram	Other non-federal	Avail-ability of data
							MASSACHUSETTS							
0944		8					6					1		A,P
0945							6					1		A,P
0960		11										1		A
0965		11				10	6					1		A,T
0969.10	X	17										2		A
0970		12					6					1		A,P
0973							13					1		A
0985.30		14,17,20										3		A
0995		16,17				10	6,21					1		A,T
1000		22			28	10	6,21			Y				A,T
1006		20,25										1		A
1010	X	11,17										1		A
1015		17,19										1		A
1020		17,19										1		A
1025		15					23					3		A
1035		11,15,17				10	29	31				3		A,T
1040		15,17						31				3		A
1042		15,17,20						31				3		A,T
1045		15,17,20						31				3		A,T
1050		11,17				10						1		A,T
1055		16										1		A
1055.85				9							4			A
1056	X						6					1		A
1057.30		11					38					1		A
1058.70		24										1		A
1058.80		17										1		A
1058.84				7			7	7				2		
1058.85				7			7	7				2		
1085								32				1		A
1090		25						32				1		A
1090.60		17,25						32				1		A
1090.70		17,41						32				1		A
1100									18			1		A
1112					37						4			A
1233.60					37						4			A
1236		25			37						4			A
1243.50					37						4			A
1245					37						4			A
1620		17										1		A
1625	X	17										1		A
1632	X	17										3		A
1640		17,26			37						4			A
1650		17,26			37						4			A
1653		17,26										3		A
1665		17,20,26										1		A
1670		15										3		A,F
1681.51		30									5			A
1685		30				10					5			A,T
1690					33		6					1		A,T
1699	X	42					6					1		A
1700		20			28		6					1		A,T
1701	X											1		A
1705		22,34			28,33			35		Y				A,T
1713		17,25										1		A

Table 2.—Uses, sources of funding, and availability of data at continuous-record gaging stations (continued)

Station number	Data-use category: Regional hydrology	Hydrologic systems	Legal obligations	Planning and design	Project operation	Hydrologic forecasts	Water quality	Research	Other	Funding category: Federal program	OFA program	Coop program	Other non-federal	Availability of data
							MASSACHUSETTS (continued)							
1715	X						6					1		A
1725		17			37		6				4			A,T
1730		15,57			15		6					3		A,F,P
1735					28		6					1		A,T
1745	X	15										3		A
1746	X			27	27							3		A
1749	X			27	27							3		A
1755		17			40							1		A
1756.70		17,20					6					1		A
1760	X	12,17					6					1		A
1770		12,17,25			28,33	10	6					1		A,T
1795		17			37						4			A
1805		12,17			37						4			A
1810	X	17			28,33						4	1		A,T
1835		17			28,33							1		A,T
1855		17,20			28,36	10					4	1		A,T
1970		17,20										1		A
1975		17,20				10	6	39				1		A,T
3315		17										1		A
3320	X	17										1		A
3325		17,20,28									4			A
3330	X	17										1		A
							RHODE ISLAND							
1113	X			43								44		A
1115	X						47					45		A
1125		48			28	10	47			Y				A,T
1140	49			50			47					46		A
1145							47					45		A
1160		52		51								44		A
1165						10	47			Y	4			A
1170		52										44		A
1173.50		52					53	54				44		A
1174.20				55			53					44		A
1174.68	X			55			53	56				44		A
1175	X	57		55				54,56				44		A,T
1178	X			58				59				44		A
1180							53	59				44		A
1185							29			Y				A

1. WRC (Massachusetts Water Resources Commission), WPC (Massachusetts Division of Water Pollution Control), and DPW (Massachusetts Department of Public Works).
2. Massachusetts Department of Public Works.
3. MDC (Metropolitan District Commission).
4. U.S. Army Corps of Engineers.
5. Federal Energy Regulatory Commission.
6. WPC to monitor variously: specific sources of contamination, effectiveness of wastewater-treatment plants, hazardous waste and other permits, or changes to the flow regime since construction of treatment facilities.
7. Highway salting study.
8. WRC for monitoring wastewater-treatment plant.
9. Corps of Engineers local flood-protection project.

Table 2.—Uses, sources of funding, and availability of data at continuous-record gaging stations (continued)

10. Monitored by National Weather Service for flood forecasting.
11. WRC for index of water supplies.
12. WRC for effectiveness of flood retarding reservoirs under State Wetlands Protection or Restrictions Acts and National Flood Insurance program.
13. WRC for site specific industrial ground-water contamination.
14. To account for water released from MDC reservoir.
15. MDC for water quality or streamflow management.
16. WRC to assess the severity of flooding in many surrounding communities.
17. WPC for basin planning or modeling.
18. WRC for recreational management of Lake Quinsigamond. Also, possible lake study by WPC.
19. WRC monitors impacts of ground-water pumpage under requirements of site specified in State laws.
20. WRC water-management program to fulfill requirements of numerous State laws.
21. For computation of discharge of Merrimack River above Lowell (National Stream Quality Acounting Network station).
22. WRC and Federal agencies for accounting of flow entering Massachusetts.
23. Environmental Protection Agency, MDC, and WPC uses for superfund hazardous waste site.
24. WRC for effects of pumpage on low flows and for reservoir acquisition in limited water-supply area with many environmental concerns.
25. WRC monitors impacts of ground-water pumpage.
26. MDC in studies of methods for increasing available water supplies.
27. MDC and University of Massachusetts for an ongoing study of the effects of deforestation on enhancement of runoff to Quabbin Reservoir.
28. Corps of Engineers for forecast of storm runoff.
29. National Stream-Quality Accounting Network station.
30. Required under Federal Energy Regulatory Commission project permit number 2669 or 2323.
31. Charles River basin model study (05400).
32. Taunton River basin ground-water resources investigation (05800).
33. Used by power companies to monitor availability of streamflow for hydroelectric generation.
34. Used by several State agencies to analyze Northfield Mountain pumped storage operation upstream for purposes of possible diversions to Quabbin Reservoir.
35. U.S. Fish and Wildlife Service for management of migratory fish.
36. Hartford, Connecticut, Water Bureau for water availability.
37. Corps of Engineers for operation of flood-control reservoir.
38. Used by Ecosystems Center, Marine Biological Laboratory, Woods Hole, in study of nitrogen cycling in a tidal river.
39. Used by consultants and government agencies in studies of polychlorinated biphenyl contamination.
40. MDC for public water-supply operation.
41. WRC to determine runoff characteristics of the basin.
42. Town of Conway environmental impact statement for hydro powerplant.
43. For design of proposed Nipmuc River water-supply reservoir.
44. Rhode Island Water Resources Board.
45. DEM (Rhode Island Department of Environmental Management), Water Resources Division.
46. Narragansett Bay Water Quality Commission.
47. For interpreting water-quality data collected at or near station by the Survey, DEM, or University of Rhode Island.
48. Accounting for flow entering Rhode Island.
49. Runoff from heavily urbanized area.
50. For planning and design of storm drain/sewer system by Narragansett Bay Water Quality Commission.
51. For planning and design of Big River water-supply reservoir.
52. For monitoring streamflow trend caused by diversion for public water supplies.
53. For interpreting water-quality monitor data.
54. Study of ground-water development alternatives in Chipuxet River basin ground-water reservoir.
55. Water Resources Board for design of ground-water withdrawal schemes in the Pawcatuck River basin.
56. Study of ground-water development alternatives in Beaver River-Pasquiset Brook basin ground-water reservoir.
57. Monthly water resources bulletin.
58. For planning and design of Wood River water-supply reservoir.
59. Study of ground-water development alternatives in the Upper and Lower Wood River basin ground-water reservoirs.

A. Data published in annual report.
F. Furnished daily record by Western Massachusetts Electric Co. (station 01167000) or MDC (station 01173000).
P. Provided at specified intervals.
T. Auxiliary data transmitted by telemetry such as satellite, radio, or phone line.
X. Natural streamflow.
Y. Funded by U.S. Geological Survey.

Table 2 shows all stations that were in operation as of the end of the 1983 water year. One station (not listed) was discontinued at the end of April 1983, when its data needs were fulfilled. Daily discharges for Connecticut River at Turners Falls (1670) and Ware River at intake works near Barre (1730) are furnished by other agencies and published by the Survey. They are not included in the K-CERA (Kalman Filtering for Cost-Effective Resource-Allocation) analysis discussed in the next section, but are shown here because of their value as daily record stations.

For the K-CERA analysis to be effective, it must represent actual funding. With Massachusetts Department of Public Works withdrawing its support in 1984 and with no alternative funding being assured at the time of the analysis, it was necessary to exclude, in addition to the six stations mentioned above, seven more stations from the analysis. With the survey results as a guide, the following seven stations were excluded from K-CERA analysis: 0945, 0969.10, 1090.70, 1100, 1701, 1756.70, and 3330.

ALTERNATIVE METHODS OF DEVELOPING STREAMFLOW INFORMATION

The second step of the analysis of the stream-gaging program is to investigate alternative methods of providing daily streamflow information in lieu of operating continuous-flow gaging stations. The objective of the analysis is to identify gaging stations where alternative technology, such as flow-routing or statistical methods, will provide information about daily mean streamflow in a more cost-effective manner than operating a continuous-record stream gage. There are no guidelines concerning suitable accuracies for particular uses of the data; therefore, judgment is required in deciding if the accuracy of the estimated daily flows is suitable for the intended purpose. The data uses at a station will influence its potential for alternative methods. For example, those stations for which flood hydrographs are required in a real-time sense, such as hydrologic forecasts and project operation, are not candidates for the alternative methods. Likewise, there might be a legal obligation to operate an actual gaging station that would preclude using alternative methods. The primary candidates for alternative methods are stations that are operated upstream or downstream of other stations on the same stream. The accuracy of the estimated streamflow at these sites may be suitable because of the high redundancy of flow information between sites. Similar watersheds, located in the same physiographic and climatic area, also may have potential for alternative methods.

All stations in the Massachusetts and Rhode Island stream-gaging programs were categorized as to their potential use of alternative methods, and selected methods were applied at seven stations. The categorization of gaging stations and the application of the specific methods are described in subsequent sections of this report. This section briefly describes the two alternative methods that were used in the Massachusetts and Rhode Island analyses and documents why these specific methods were chosen.

Because of the short time frame of this analysis, only two methods were considered. Desirable attributes of a proposed alternative method are that the proposed method should (1) be computer oriented and easy to apply, (2) have an available interface with the Survey's WATSTORE (National Water Data Storage and Retrieval System) Daily Values File (Hutchinson, 1975), (3) be technically sound and generally acceptable to the hydrologic community, and (4) permit easy evaluation of the accuracy of the simulated streamflow records. The desirability of the first attribute above is rather obvious. Second, the interface with the WATSTORE Daily Values File is needed to easily calibrate the proposed alternative method. Third, the alternative method selected for analysis must be technically sound or it will not be able to provide data of suitable accuracy. Fourth, the alternative method should provide an estimate of the accuracy of the streamflow to judge the adequacy of the simulated data. The above selection criteria were used to select two methods—a flow-routing model and multiple-regression analysis.

Flow-Routing Model

Hydrologic flow-routing methods use the law of conservation of mass and the relationship between the storage in a reach and the outflow from the reach. The hydraulics of the system are not considered. The method usually requires only a few parameters and treats the reach in a lumped sense without subdivision. The input is usually a discharge hydrograph at the upstream end of the reach and the output, a discharge hydrograph at the downstream end. Several different types of hydrologic routing are available such as Muskingum, Modified Puls, Kinematic Wave, and the unit-response flow-routing method. The CONROUT unit-response flow-routing model was selected for this analysis (Doyle and others, 1983). This method uses two techniques—storage continuity (Sauer, 1973) and diffusion analogy (Keefer, 1974; Keefer and McQuivey, 1974). These concepts are discussed below.

The unit-response method was selected because it fulfilled the criteria noted above. Computer programs for the unit-response method can be used to route streamflow from one or more upstream locations to a downstream location. Downstream hydrographs are produced by the convolution of upstream hydrographs with their appropriate unit-response functions. This method can only be applied at a downstream station where an upstream station exists on the same stream. An advantage of this model is that it can be used for regulated stream systems. Reservoir routing techniques are included in the model so flows can be routed through reservoirs if the operating rules are known. Calibration and verification of the flow-routing model is achieved with observed upstream and downstream hydrographs and estimates of tributary inflows. The convolution model treats a stream reach as a linear one-dimensional system in which the system output (downstream hydrograph) is computed by multiplying (convoluting) the ordinates of the upstream hydrograph by the unit-response function and lagging them appropriately. The model has the capability of combining hydrographs, multiplying a hydrograph by a ratio, and changing the timing of a hydrograph. Routing can be accomplished using hourly data, but only daily data were used in this analysis.

Three options are available for determining the unit-response (system) function. Selection of the appropriate option depends primarily upon the variability of wave celerity (traveltime) and dispersion (channel storage) throughout the range of discharges to be routed. Adequate routing of daily flows can usually be accomplished using a single unit-response function (linearization about a single discharge) to represent the system response. However, if the routing coefficients vary drastically with discharge, linearization about a low-range discharge results in overestimated high flows that arrive late at the downstream site, whereas linearization about a high-range discharge results in low-range flows that are underestimated and arrive too soon. A single unit-response function may not provide acceptable results in such cases. Therefore, the option of multiple linearization (Keefer and McQuivey, 1974), which uses a family of unit-response functions to represent the system response, is available.

Determination of the system's response to the input at the upstream end of the reach is not the total solution for most flow-routing problems. The convolution process makes no accounting of flow from the intervening area between the upstream and downstream locations. Such flows may be totally unknown or estimated by some combination of gaged and ungaged flows. An estimating technique that should prove satisfactory in many instances is the multiplication of known flows at an index gaging station by a factor—for example, a drainage area ratio.

The objective in either the storage-continuity or diffusion analogy flow-routing method is to calibrate two parameters that describe the storage discharge relationship in a given reach and the traveltime of flow passing through the reach. In the storage-continuity method, a response function is derived by modifying a translation hydrograph technique developed by Mitchell (1962) to apply to open channels. A triangular pulse (Sauer, 1973) is routed through reservoir-type storage and then transformed by a summation curve technique to a unit response of desired duration. The two parameters that describe the routing reach are *Ks*, a storage coefficient, which is the slope of the storage-discharge relation, and *Ws*, which is the translation hydrograph time base. These two parameters determine the shape of the resulting unit-response function.

In the diffusion analogy theory, the two parameters requiring calibration in this method are K_0, a wave dispersion or damping coefficient, and C_0, the floodwave celerity. K_0 controls the spreading of the wave (analogous to K_s in the storage-continuity method) and C_0 controls the traveltime (analogous to *Ws* in the storage-continuity method). In the single linearization method, one K_0 and C_0 value is used. In the multiple linearization method, C_0 and K_0 are varied with discharge so a table of wave celerity (C_0) versus discharge (Q) and a table of dispersion coefficient (K_0) versus discharge (Q) are used.

In both the storage-continuity and diffusion-analogy methods, the two parameters are calibrated by trial and error. The analyst must decide if suitable parameters have been derived by comparing the simulated discharge to the observed discharge.

Regression Analysis

Simple- and multiple-regression techniques also can be used to estimate daily flow records. Regression equations can be computed that relate daily flows (or their logarithms) at a single station to daily flows at a combination of upstream, downstream, and (or) tributary stations. This statistical method is not limited, as is the flow-routing method, to stations where an upstream station exists on the same stream. The explanatory variables in the regression analysis can be stations from different watersheds, or downstream and tributary watersheds. The regression method has many of the same attributes as the flow-routing method in that it is easy to apply, provides indices of accuracy, and is generally accepted as a good tool for estimation. The theory and assumptions of regression analysis are described in several textbooks such as Draper and Smith (1966) and Kleinbaum and Kupper (1978). The application of regression analysis to hydrologic problems is described and illustrated by Riggs (1973) and Thomas and Benson (1970). Only a brief description of regression analysis is provided in this report.

A linear regression model of the following form was developed for estimating daily mean discharges in Massachusetts and Rhode Island:

$$Y_i = B_o + \sum_{j=1}^{p} B_j \, x_j + e_i$$

where

Y_i = daily mean discharge at station i (dependent variable),

x_j = daily mean discharges at nearby stations (explanatory variables),

B_0 and B_j = regression constant and coefficients, and

e_i = the random error term.

The above equation is calibrated (B_0 and B_j are estimated) with observed values of Y_i and x_j. These observed daily mean discharges can be retrieved from the WATSTORE Daily Values File. The values of x_j may be discharges observed on the same day as discharges at station i or may be for previous or future days, depending on whether station j is upstream or downstream of station i. Once the equation is calibrated and verified, future values of Y_i are estimated using observed values of x_j. The regression constant and coefficients (B_0 and B_j) are tested to determine if they are significantly different from zero. A given station j should only be retained in the regression equation if its regression coefficient (B_j) is significantly different from zero. The regression equation should be calibrated using one period of time and then verified or tested on a different period of time to obtain a measure of the true predictive accuracy. Both the calibration and verification period should be representative of the range of flows that could occur at station i. The equation should be verified by plotting (1) the residuals e_i (difference between simulated and observed discharges) against the dependent and all explanatory variables in the equation, and (2) simulated and observed discharges versus time. These tests are intended to identify (1) if the linear model is appropriate or whether some transformation of the variables is needed, and (2) if there is any bias in the equation such as overestimating low flows. These tests might indicate, for example, that a logarithmic transformation is desirable, that a nonlinear regression equation is appropriate, or that the regression equation is biased in some way. In this report, these tests indicated that a linear model with y_i and x_j, in cubic feet per second, was appropriate. The application of linear-regression techniques to seven watersheds in Massachusetts and Rhode Island is described in a subsequent section of this report.

It should be noted that the use of a regression relation to synthesize data at a discontinued gaging station entails a reduction in the variance of the streamflow record relative to that which would be computed from an actual record of streamflow at the site. The reduction in variance expressed as a fraction is approximately equal to one minus the square of the correlation coefficient that results from the regression analysis.

Stream Gages Used to Evaluate Alternative Methods

An analysis of the data uses presented in table 2 identified seven stations at which alternative methods for providing the needed streamflow information could be applied. These seven stations are North Nashua River near Leominster (0945), Squannacook River near West Groton (0960), North River at Shattuckville (1690), Cadwell Creek near Belchertown (1749), and Hoosic River near Williamstown (3325) in Massachusetts, and Branch River at Forestdale (1115) and Pawcatuck River at Westerly (1185) in Rhode Island. Based on the capabilities and limitations of the methods and data availability, flow-routing techniques were used only at the Leominster, Belchertown, Williamstown, and Westerly gaging stations. Regression methods were applied to all seven sites.

Leominster Flow-Routing Analysis

The purpose of this flow-routing analysis is to investigate the potential for use of the CONROUT model for streamflow routing to simulate daily mean discharges at Leominster (0945). A schematic diagram of the North Nashua River study area is presented in figure 3. (Squannacook River (0960), which is an easterly tributary to the Nashua River downstream of Leominster, is not shown in fig. 3). In this application, as with the other systems that were modeled, a best-fit model for the entire flow range is the desired product. Streamflow data available for this analysis are summarized in table 3.

The Leominster gage is located 6.8 miles downstream from the Fitchburg gage on the North Nashua River. In this reach, flow includes diversions for municipal supplies for Fitchburg from Mare Meadow Reservoir since 1955, for Leominster from Wachusett Reservoir since 1966, and for the Southeast well field since 1958. The intervening drainage area between Fitchburg and Leominster is 46.4 mi^2 or 42 percent of the total drainage area contributing to the Leominster site. There are no stream gages located within this 46.4 mi^2 intervening area.

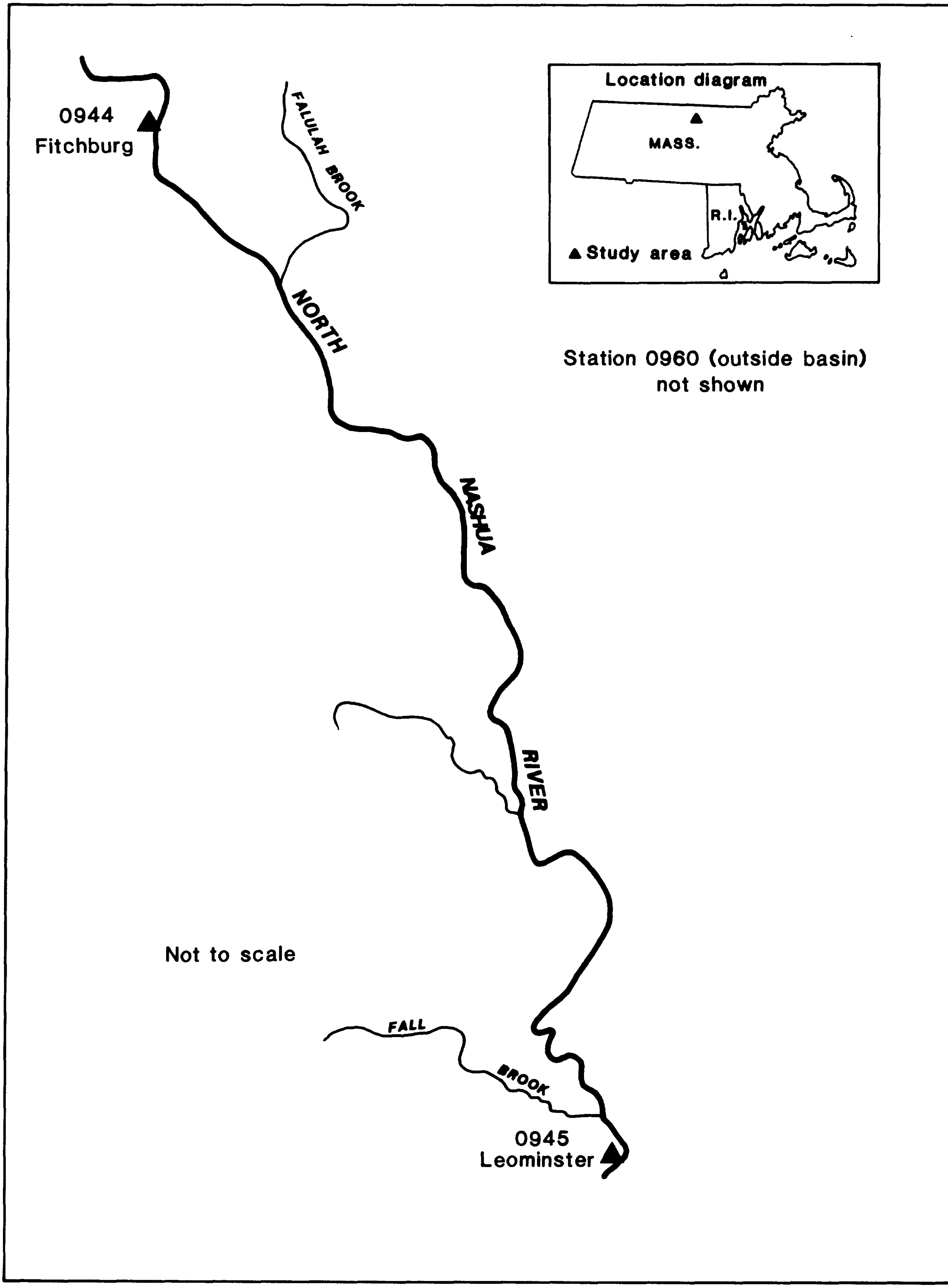

Figure 3.--Location of the North Nashua study area.

Table 3.—Gaging stations used in the Leominster flow-routing study

Station number	Station name	Drainage area (square miles)	Period of record
0944	Fitchburg	63.6	October 1972-present
0945	Leominster	110	September 1935-present
0960	West Groton	62.8	October 1949-present

Daily mean discharges were simulated by routing the flow along the North Nashua River from Fitchburg to Leominster with the CONROUT unit-response model and the single linearization option of the diffusion analogy method. The intervening drainage area would be accounted for by using data from station 0944 and (or) station 0960, adjusted (weighted) by drainage area ratios.

The model parameters C_0 (floodwave celerity) and K_0 (wave dispersion coefficient) were needed to route flow from Fitchburg to Leominster. The model parameters C_0 and K_0 are functions of channel width (W_0), in feet; channel slope (S_0), in feet per foot; slope of the stage-discharge relation (dQ_0/dY_0), in square feet per second; and discharge (Q_0), in cubic feet per second, representative of the reach in question, and are determined as follows:

$$C_0 = \frac{1}{W_0} \frac{dQ_0}{dY_0} \tag{1}$$

$$K_0 = \frac{Q_0}{2\ S_0\ W_0} \tag{2}$$

The discharge, Q_0, for which initial values of C_0 and K_0 were lineararized was the long-term mean daily discharge for the Fitchburg and Leominster gages. Channel width, W_0, was calculated as the average for the 6.8-mile reach between the site and was measured from topographic maps. Channel slope, S_0, was determined by converting the corresponding gage heights of the initial discharges, Q_0, taken from the stage-discharge relationships of each gage, to a common datum. The difference between these values was then divided by the channel length to obtain a slope. The slope of the stage-discharge relations, dQ_0/dY_0, was determined from the rating curves at each gage by using a 1-foot increment that bracketed the mean discharge, Q_0. The difference in the discharge through the 1-foot increment then represents the slope of the function at that point. The model parameters as determined above are listed in table 4.

Table 4.—Selected reach characteristics used in the Leominster flow-routing study

Site	Q_0[1] (cubic feet per second)	W_0 (feet)	S_0 (feet per foot)	dQ_0/dY_0 (square feet per second)	C_0 (feet per second)	K_0 (square feet per second)
Fitchburg	119	77	3.22×10^{-3}	251	3.27	241
Leominster	191	77		266	3.47	387

[1]Mean discharge for the period of record.

For the first trial, average values for the model parameters $C_0 = 3.37$ and $K_0 = 309$ were used. The intervening drainage was simulated by multiplying the flow at Fitchburg by the ratio of the intervening drainage area to the drainage area at Fitchburg (46.4/63.6 = 0.73).

Streamflow for water years 1973 through 1978 was used as a calibration data set. During calibration, C_0, K_0, and the drainage area ratio for simulating intervening drainage area were varied. Attempts were also made using Squannacook River (0960) to simulate all or some fraction of the intervening drainage area. The best-fit model from this analysis proved to be the one that used the initial values for C_0 and K_0 and simulated the intervening drainage by using the slightly revised value of 0.70 multiplied by the flow at Fitchburg.

The results of this calibrated model were verified by applying the model to an independent set of flow data for water years 1979 through 1981. Table 5 presents a summary of the verification results for the routing models to simulate flows at Leominster. Results of the verification were consistent with those obtained during calibration.

Table 5.—Verification results of routing model for Leominster

Mean absolute error for 1,096 days	=	9.87 percent
Mean negative error (594 days)	=	-10.06 percent
Mean positive error (502 days)	=	9.65 percent
Total volume error	=	1.82 percent

36 percent of total observations had errors less than or equal to 5 percent
63 percent of total observations had errors less than or equal to 10 percent
79 percent of total observations had errors less than or equal to 15 percent
88 percent of total observations had errors less than or equal to 20 percent
92 percent of total observations had errors less than or equal to 25 percent
8 percent of total observations had errors greater than 25 percent

Although the volume error was small (1.82 percent), mean absolute error for the water years 1979 through 1981 was 9.87 percent, and only 63 percent of the simulated data were within 10 percent of observed values at Leominster. Further study failed to determine any type of systematic pattern in the errors, and all attempts to improve the fit of this model failed. A typical example of the modeling results are shown in figure 4. Inability to simulate more accurately the mean daily streamflow at Leominster can be linked to the large percentage (42 percent) of ungaged intervening drainage and the regulating effects upstream from the site.

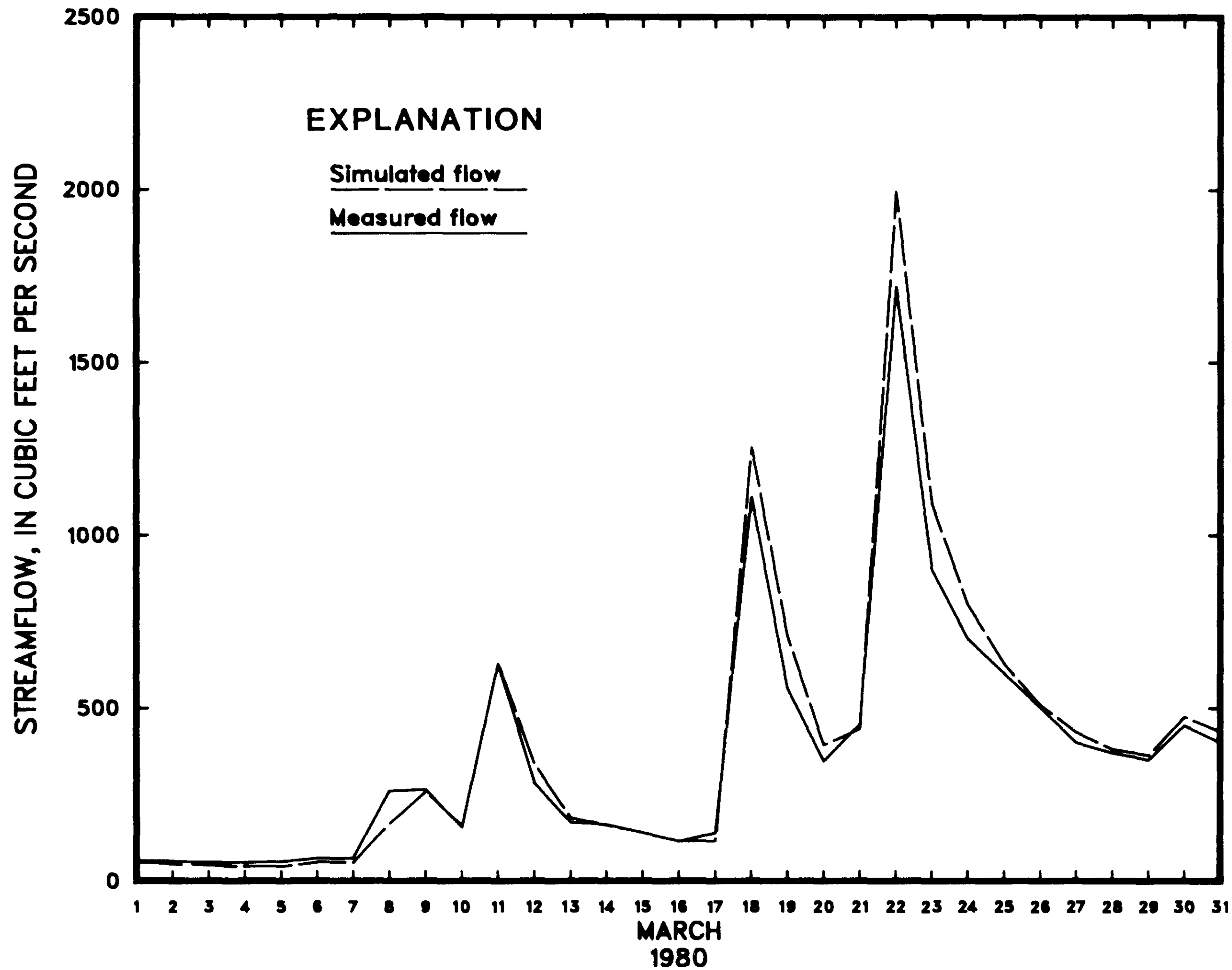

Figure 4.--Daily hydrograph at Leominster, spring 1980.

Belchertown Flow-Routing Analysis

The purpose of this flow-routing analysis is to investigate the potential for use of the CONROUT model for streamflow routing to simulate daily mean discharge at Cadwell Creek near Belchertown (1749). A schematic diagram of the Cadwell Creek study area is presented in figure 5. Streamflow data available for this analysis are summarized in table 6.

Table 6.—Gaging stations used in the Belchertown flow-routing study

Station number	Station name	Drainage area (square miles)	Period of record
1746	Pelham	0.63	July 1961–present
1749	Belchertown	2.81	July 1961–present

The Belchertown gage is located 200 feet upstream from the mouth of Cadwell Creek at Quabbin Reservoir and 2.0 miles downstream from the Pelham gage. The intervening drainage area between Pelham and Belchertown is 2.18 mi^2 or 78 percent of the total drainage area contributing to the Belchertown site. There are no stream gages located within this 2.18-mi^2 intervening area.

Daily mean discharges were simulated by routing the flow along Cadwell Creek from Pelham to Belchertown with the CONROUT unit-response model and the single linearization option of the diffusion analogy method. The intervening drainage area would be accounted for by using data from station 1746, adjusted by a drainage area ratio.

The routing parameters C_0 and K_0 were determined by using the techniques applied in the Leominster analysis and are summarized in table 7. One exception was the reach characteristic stream width W_0, which was calculated from discharge measurement data at each of the stream gages.

Table 7.—Selected reach characteristics used in the Belchertown flow-routing study

Site	Q_0[1] (cubic feet per second)	W_0 (feet)	S_0 (feet per foot)	dQ_0/dY_0 (square feet per second)	C_0 (feet per second)	K_0 (square feet per second)
Pelham	1.12	4	3.03×10^{-2}	4.19	1.05	4.62
Belchertown	4.73	14		10.1	.72	5.58

[1]Mean discharge for the period of record.

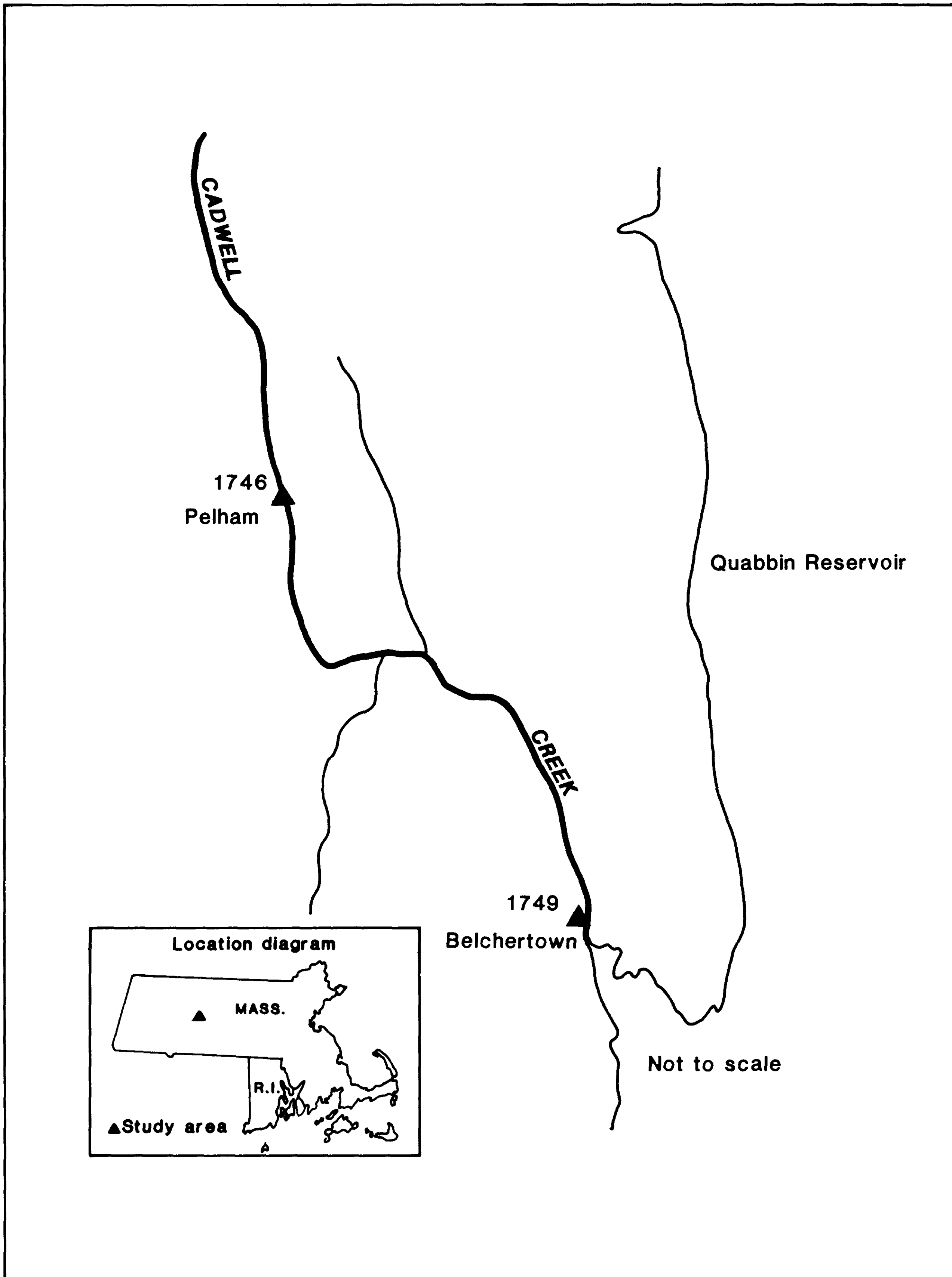

Figure 5.--Location of the Cadwell Creek study area.

For the first routing trial, average values for the model parameters C_0 = 0.90 and K_0 = 5.1 were used. The intervening drainage was simulated by multiplying the flow at Pelham by the ratio of the intervening drainage area to the drainage area at Pelham (2.18/0.63 = 3.46).

Streamflow data for water years 1973 through 1977 were used as a calibration data set. During calibration, C_0, K_0, and the computed ratio for simulating intervening drainage area were varied. The best-fit model from this analysis proved to be the one that used the initial values for C_0 and K_0 and simulated the intervening drainage by multiplying the streamflow at Pelham by 3.46.

The results of this calibrated model were verified by applying the model to an independent set of streamflow data from water years 1978 through 1980. Table 8 presents a summary of the verification results for the routing model to simulate flows at Belchertown. Results of the verification were consistent with those obtained during calibration.

Table 8.—Verification results of routing model for Belchertown

Mean absolute error for 1,096 days	=	17.06 percent
Mean negative error (680 days)	=	-16.80 percent
Mean positive error (416 days)	=	17.47 percent
Total volume error	=	0.87 percent

28 percent of total observations had errors less than or equal to 5 percent

50 percent of total observations had errors less than or equal to 10 percent

62 percent of total observations had errors less than or equal to 15 percent

70 percent of total observations had errors less than or equal to 20 percent

77 percent of total observations had errors less than or equal to 25 percent

23 percent of total observations had errors greater than 25 percent

Although the volume error was small (-0.87 percent), mean absolute error for the water years 1978 through 1980 was 17.06 percent, and only 50 percent of the simulated data were within 10 percent of observed figures at Belchertown. Further study failed to determine any type of systematic pattern in the errors, and all attempts to improve the fit of this model failed. A typical example of the modeling result are shown in figure 6. Inability to simulate more accurately the mean daily streamflow at Belchertown can be linked primarily to the high percentage (78 percent) of ungaged intervening drainage upstream from the site.

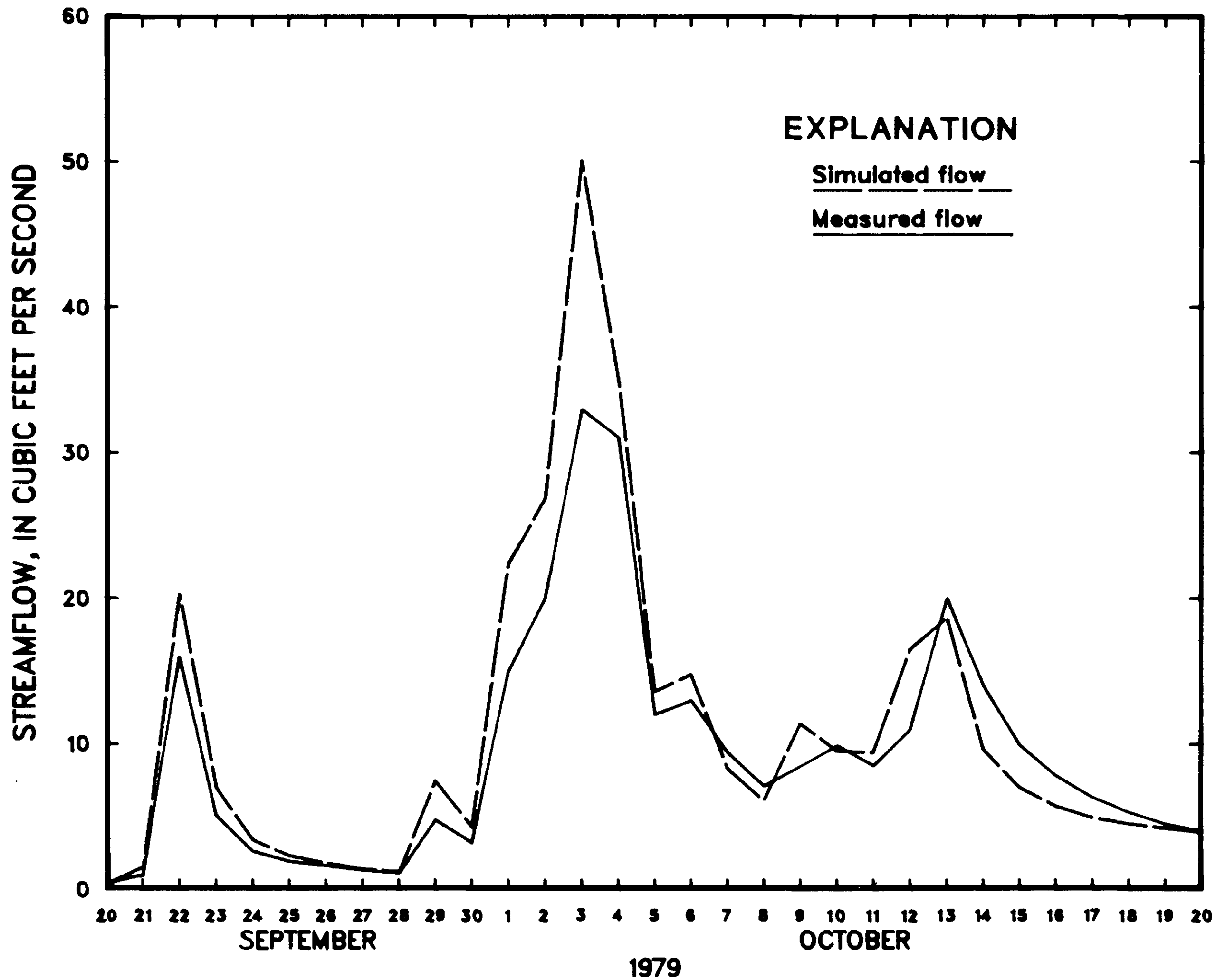

Figure 6.--Daily hydrograph at Belchertown, early fall 1979.

Williamstown Flow-Routing Analysis

The purpose of this flow-routing analysis is to investigate the potential for use of the CONROUT model for streamflow routing to simulate daily mean discharges at Hoosic River near Williamstown (3325). A schematic diagram of the Hoosic River study area is presented in figure 7. Streamflow data available for this analysis are summarized in table 9.

Table 9.—Gaging stations used in the Williamstown flow-routing study

Station number	Station name	Drainage area (square miles)	Period of record
3315	Adams	46.3	October 1931-present
3320	North Adams	39.0	June 1931-present
3325	Williamstown	[1]126	July 1940-present[1]
3330	Green River	42.6	September 1949-present

[1]Prior to June 6, 1979, at site located 1.2 miles downstream, with a drainage area of 132 mi^2.

The Williamstown gage is located 11.9 miles downstream from the Adams gage on the Hoosic River. The major tributary in this reach is the North Branch Hoosic River. The mouth of the North Branch is located approximately 5.2 miles upstream from the Williamstown gage. The ungaged intervening drainage area between Adams and Williamstown is 40.7 mi^2 (46.7 mi^2 prior to June 6, 1979) or 32.3 percent of the total drainage area contributing to the Williamstown site.

Daily mean discharges were simulated by routing the flow along the Hoosic River from Adams to Williamstown with the CONROUT unit-response model and the single linearization option of the diffusion analogy method. Data from the gage on the North Branch Hoosic River at North Adams was added to this routed flow. The ungaged intervening drainage area was accounted for by using data from a combination of the gages shown in table 9; adjusted by drainage area ratios.

The routing parameters C_0 and K_0 were determined by using the techniques applied in the Leominster analysis and are summarized in table 10.

Table 10.—Selected reach characteristics used in the Williamstown flow-routing study

Site	Q_0[1] (cubic feet per second)	W_0 (feet)	S_0 (feet per foot)	dQ_0/dY_0 (square feet per second)	C_0 (feet per second)	K_0 (square feet per second)
Adams	96.5	73	7.06×10^{-3}	253	3.46	93
Williamstown	273	73		520	7.10	264

[1]Mean discharge for the period of record.

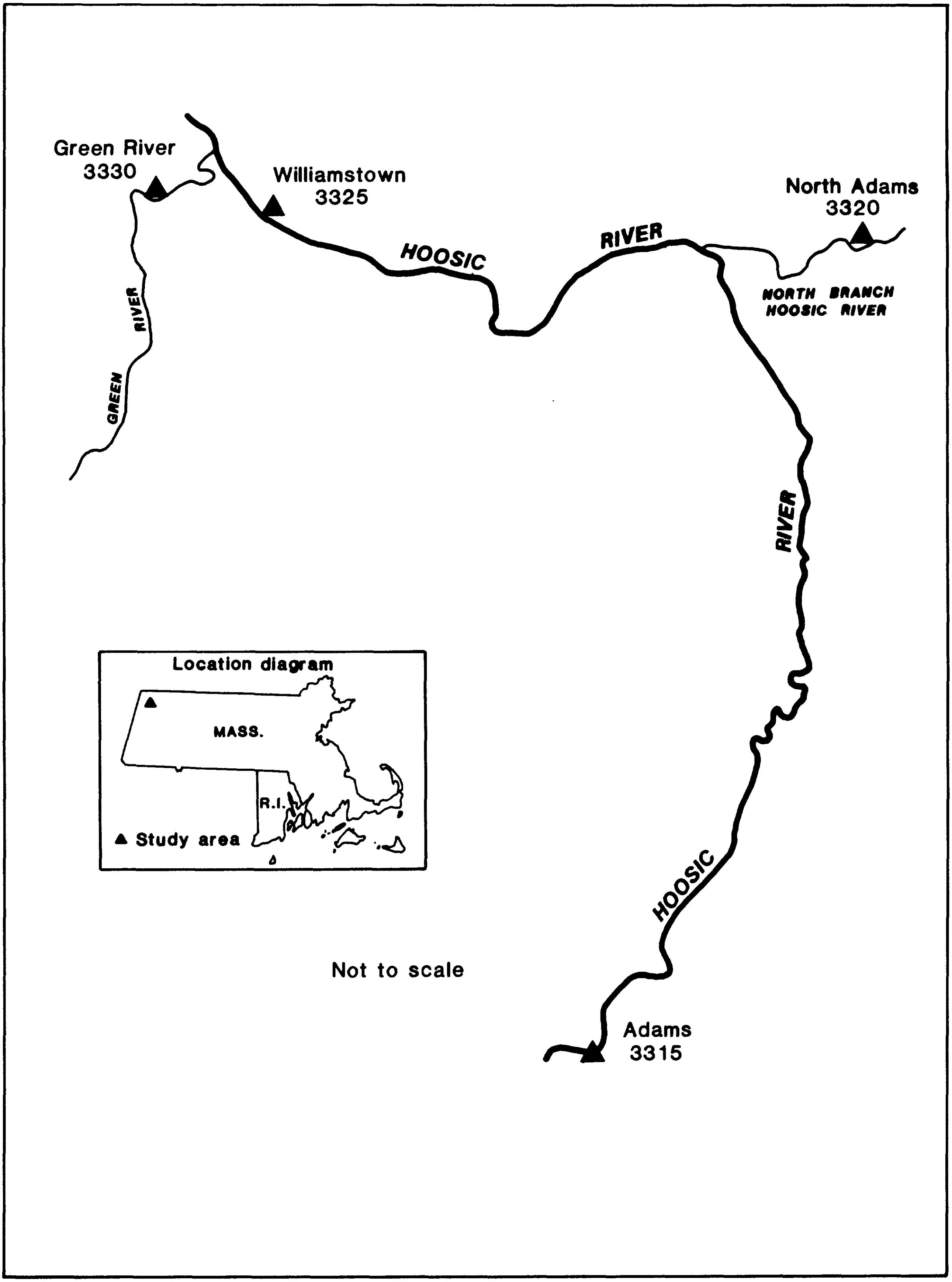

Figure 7.--Location of the Hoosic River study area.

For the first routing trial, average values for the model parameters C_0 = 5.28 and K_0 = 178 were used. Flow from the station on the North Branch (3320) was added directly to the routed discharge from Adams. The ungaged intervening drainage was simulated by multiplying streamflow at Adams by the ratio of the pre-1979 ungaged drainage area to the drainage area at Adams (46.7/46.3 = 1.01).

Streamflow data from water years 1973 through 1977 were used as a calibration data set. For the entire calibration period, the gage at Williamstown was located 1.2 miles downstream from the present site. This factor was taken into account when determining reach length for flow routing and drainage area ratios for simulating intervening drainage. During calibration, C_0, K_0, and the drainage area ratios and stations, given in table 9, used for simulating intervening drainage area were varied. The best-fit model from this analysis proved to be one with slightly revised figures of 5.50 for C_0 and 200 for K_0. Intervening drainage area was best simulated by multiplying the flow at Adams by 0.95 and the flow at North Adams by 1.0.

The results of this calibrated model were verified by applying the model to an independent set of flow data for the period from October 1, 1977, to June 6, 1979. Results from this verification analysis were consistent with those obtained during calibration. The applicability of this calibrated model to the current gage location was ensured by adjusting the reach length and ungaged drainage area ratios to account for changes when the gage was relocated in June 1979. The adjusted calibrated flow model was then verified by applying it to a data set for the period June 6, 1979, to September 30, 1981. Results of this verification were consistent with the previous verification and calibration results. Table 11 presents a summary of the final verification results.

Table 11.—Verification results of routing model for Williamstown

Mean absolute error for 848 days	=	8.91 percent
Mean negative error for (489 days)	=	-9.35 percent
Mean positive error for (359 days)	=	8.30 percent
Total volume error	=	-2.78 percent

34 percent of total observations had errors less than or equal to 5 percent

64 percent of total observations had errors less than or equal to 10 percent

82 percent of total observations had errors less than or equal to 15 percent

90 percent of total observations had errors less than or equal to 20 percent

97 percent of total observations had errors less than or equal to 25 percent

3 percent of total observations had errors greater than 25 percent

Mean absolute error for the period June 6, 1979, to September 30, 1981, was 8.91 percent, and 64 percent of the simulated data were within 10 percent of observed figures at Williamstown. Further study failed to determine any type of systematic pattern in the errors, and all attempts to improve the fit of this model failed. A typical example of the modeling results is shown in figure 8.

Inability to simulate more accurately mean daily streamflow at Williamstown can be linked primarily to the high percentage (32.3 percent) of ungaged intervening drainage upstream from the site.

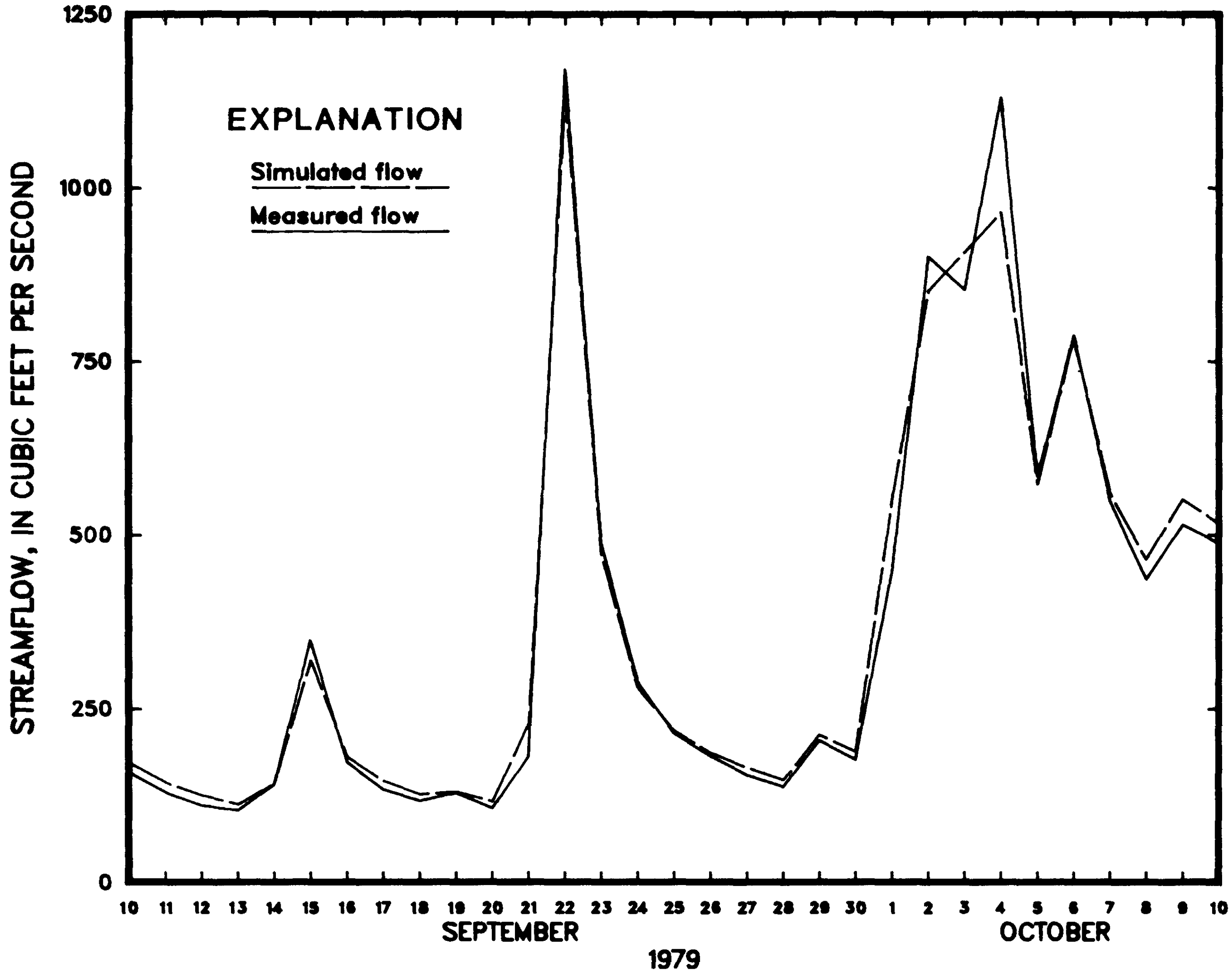

Figure 8.--Daily hydrograph at Williamstown, late summer 1979.

Westerly Flow-Routing Analysis

The purpose of this flow-routing analysis is to investigate the potential for use of the CONROUT model for streamflow routing to simulate daily mean discharges at Pawcatuck River at Westerly (1185). A schematic diagram of the Pawcatuck River study area is presented in figure 9. Streamflow data available for this analysis are summarized in table 12.

Table 12.—Gaging stations used in the Westerly flow-routing study

Station number	Station name	Drainage area (square miles)	Period of record
1175	Wood River Junction	100	October 1940–present
1180	Hope Valley	72.4	March 1941–present
1183	Pendleton Hill	4.02	July 1958–present
1185	Westerly	295	November 1940–present

The distance between the Westerly and Wood River Junction gages on the Pawcatuck River is 20.2 miles. The mouth of the Wood River is located 2.9 miles downstream from the Wood River Junction gage. The Hope Valley gage is located 5.7 miles upstream from the mouth of the Wood River. The ungaged intervening drainage area between Wood River Junction and Westerly is 119 mi^2 or 40.3 percent of the total drainage area contributing to the Westerly site. Streamflow data from the Westerly gage are influenced by diversion for the municipal supply of Westerly.

Daily mean discharges were simulated by routing the flow from Wood River Junction and Hope Valley gages to the confluence of the Wood and Pawcatuck Rivers, hereafter referred to as the confluence. The routed hydrographs were then combined and routed downstream along the Pawcatuck River to Westerly. All flow-routing used the CONROUT unit-response model and the single linearization option of the diffusion analogy method. Ungaged intervening drainage area was accounted for by using data from a combination of the gages in table 12, adjusted by drainage area ratios.

The routing parameters C_0 and K_0 were determined by using the techniques applied in the Leominster analysis and are summarized in table 13.

Table 13.—Selected reach characteristics used in the Westerly flow-routing study.

Site	Q_0[1] (cubic feet per second)	W_0 (feet)	S_0 (feet per foot)	dQ_0/dY_0 (square feet per second)	C_0 (feet per second)	K_0 (square feet per second)
Wood River Junction	191	110	4.1×10^{-4}	206	1.87	1,736
Hope Valley	153	110	5.0×10^{-4}	206	1.87	2,284
Westerly	566	110	4.1×10^{-4}	566	5.15	6,275

[1]Mean discharge calculated for the period of record.

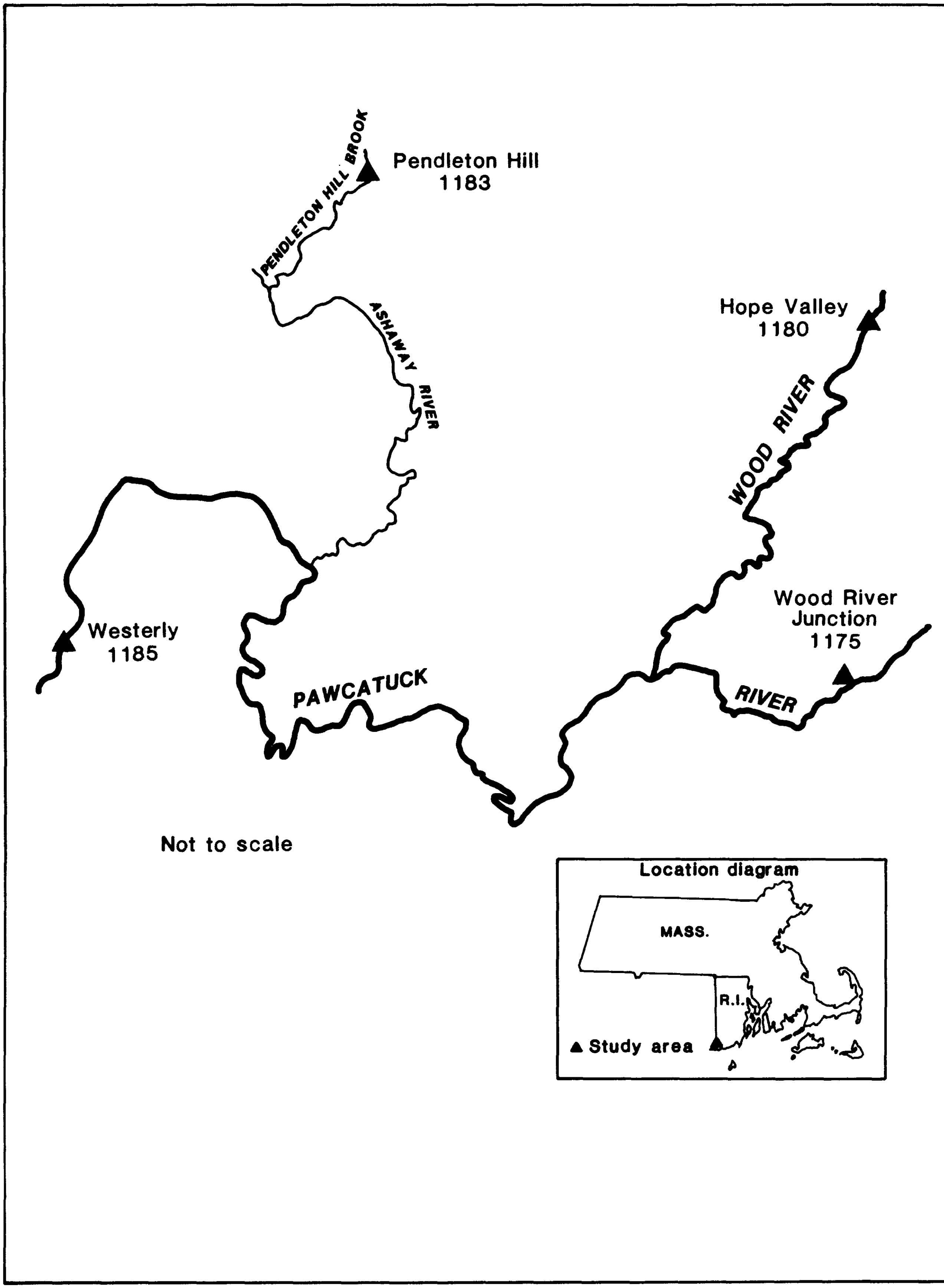

Figure 9.--Location of the Pawcatuck River study area.

For the first routing trial, average values for the model parameters $C_0 = 1.87$ and $K_0 = 2{,}010$ were used for the reaches from Wood River Junction and Hope Valley to the confluence. The routing parameters documented for Westerly, $C_0 = 5.15$ and $K_0 = 6{,}275$, were used as initial estimates for the reach of the Pawcatuck from the confluence to Westerly.

Intervening drainage was simulated by multiplying the flow at Wood River Junction by a drainage area ratio of 1.0, and the flow at Pendleton Hill Brook by a drainage area ratio of 1.89. These ratio-adjusted hydrographs were added to the routed streamflow to obtain simulated discharge at Westerly.

Streamflow data from water years 1973 through 1978 were used as a calibration data set. During calibration, C_0, K_0, and the drainage area ratios and stations, given in table 12, used for simulating intervening drainage area were varied. The best-fit model from this analysis proved to be the one with the initial values for C_0 and slightly revised figures of 2,280 for K_0 in the reach from Wood River Junction to the confluence, 1,740 for K_0 in the reach from Hope Valley to the confluence and the initial value of 6,275 for K_0 in the reach from the confluence to Westerly. Intervening drainage area was best simulated by multiplying using a ratio of 1.0 times the flow from the Wood River Junction gage.

The results of this calibrated model were verified by applying the model to an independent set of flow data for water years 1979 through 1981. Results of this verification were consistent with those obtained during calibration. Table 14 presents a summary of the final verification results.

Table 14.—Verification results of routing model for Westerly

Mean absolute error for 1,096 days	=	10.81 percent
Mean negative error for (437 days)	=	-6.93 percent
Mean positive error for (659 days)	=	13.38 percent
Total volume error	=	-0.85 percent

33 percent of total observations has errors less than or equal to 5 percent

57 percent of total observations has errors less than or equal to 10 percent

76 percent of total observations has errors less than or equal to 15 percent

87 percent of total observations has errors less than or equal to 20 percent

91 percent of total observations has errors less than or equal to 25 percent

9 percent of total observations has errors greater than 25 percent

Results for the Westerly routing study follow the pattern established in the previous studies. Volume errors are relatively small, but mean absolute errors are large. A typical example of the modeling results are presented in figure 10. Attempts to improve the fit of this model were restricted by the large percentage of ungaged intervening drainage area (40.3 percent) above Westerly as well as by the upstream regulation.

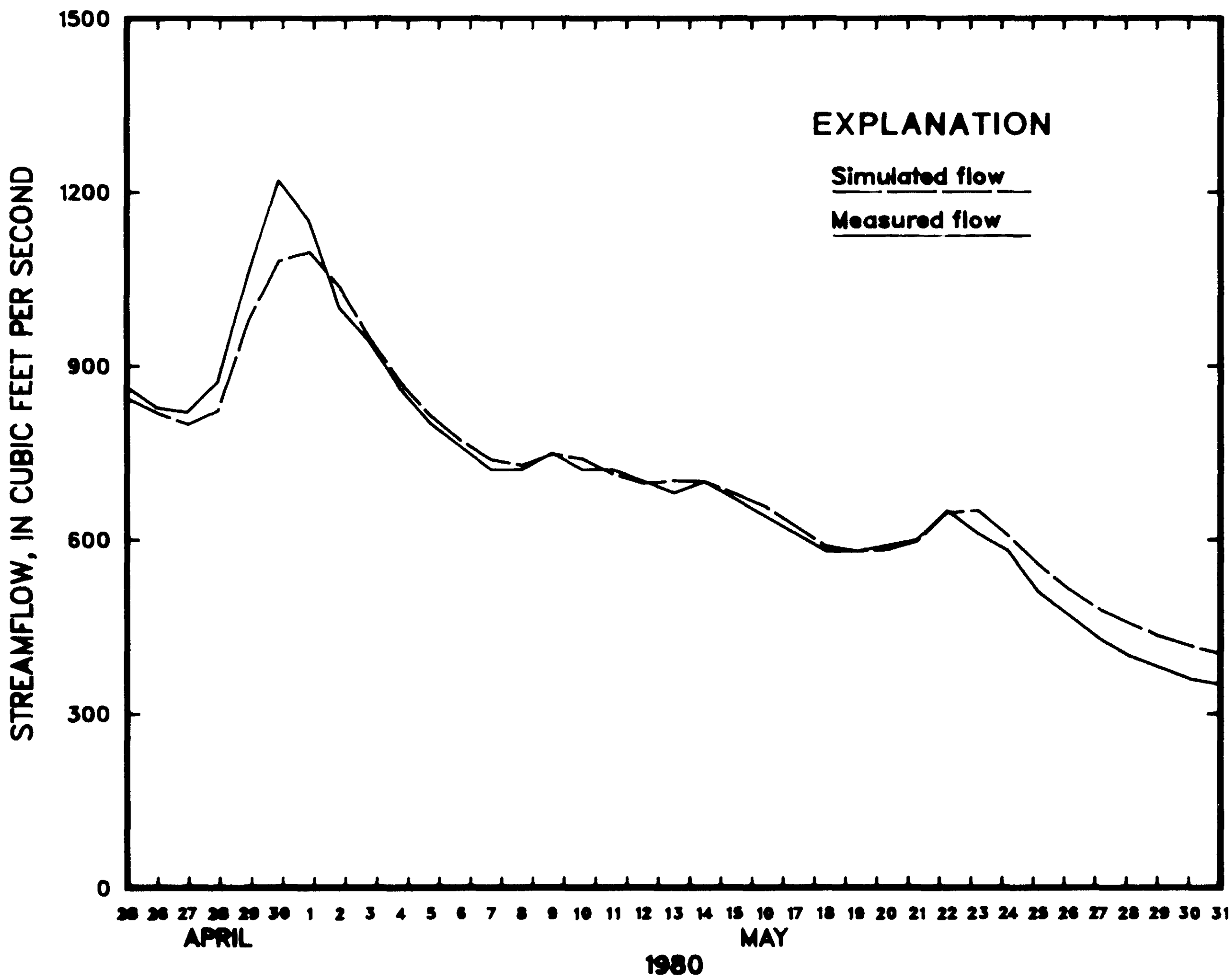

Figure 10.--Daily hydrograph at Westerly, spring 1980.

Regression Analysis Results

Linear regression techniques were applied to the seven selected sites. The streamflow record for each station considered for simulation (the dependent variable) was regressed against streamflow records at other stations (explanatory variables) during a given period of record (the calibration period). "Best-fit" linear regression models were developed and used to provide a daily streamflow record that was compared to the observed streamflow record. The percentage of difference between the simulated and actual record for each day was calculated. The results of the regression analysis for each site are summarized in table 15.

Table 15.—Summary of calibration for regression modeling of mean daily streamflow at selected gage sites in Massachusetts and Rhode Island

Station number	Model	Percentage of simulated flow within 5 percent of actual	Percentage of simulated flow within 10 percent of actual	Calibration period (water years)
0945	$Q_{0945} = 19.1 + 1.35\ (Q_{0944}) + 0.134\ (Q_{0960})$	33.7	64.1	1979-81
0960	$Q_{0960} = -0.289 + 0.428\ (Q_{0945}) + 0.259\ (Q_{0944})$	10.8	23.7	1979-81
1115	$Q_{1115} = 20.4 + 3.41\ (Q_{111300}) + 1.48\ (\text{LAG 1 } Q_{111300})$	18.1	37.3	1979-81
1185	$Q_{1185} = -39.4 + 1.37\ (Q_{1175}) + 1.03\ (Q_{1180}) + 1.89\ (Q_{1183}) + 0.770\ (\text{LAG 1 } Q_{1175})$	29.5	51.0	1979-81
1690	$Q_{1690} = -32.2 + 2.72\ (Q_{1701})$	12.7	24.1	1979-81
1749	$Q_{1749} = 0.242 + 4.07\ (Q_{1746}) + 0.243\ (\text{LAG 1 } Q_{1746})$	22.6	46.8	1979-81
3325	$Q_{3325} = 12.3 + 1.72\ (Q_{3315}) + 0.948\ (Q_{3320}) + 0.184\ (\text{LAG 1 } Q_{3315})$	34.4	61.3	June 6, 1979, to Sept. 30, 1981

Special explanatory variables, specified as LAG 1 Q, were created by lagging the discharge by 1 day. The insertion in a regression of the lagged and unlagged values for a given streamflow record acts to statistically route the flow from an upstream to a downstream site. The lagged discharge values account for the traveltime between the two sites.

The simulation of streamflow record at Leominster (0945) was one of the most successful of the seven regression models evaluated. The regression model for Leominster includes, as explanatory variables, the streamflow at stations 0944 and 0960. Station 0944 is the nearest upstream station in the basin and station 0960 is located in an adjacent basin. Estimates from the regression model for Leominster simulated the actual record within 10 percent for 64 percent of the calibration period and within 5 percent for 34 percent of the period. Attempts to improve this regression model were hindered by the paucity of streamflow data and the degree of regulation within the basin.

The streamflow record at West Groton (0960) was not reproduced with any acceptable degree of accuracy by regression techniques. The best-fit regression model for West Groton includes, as explanatory variables, the streamflow at stations 0944 and 0945. Stations 0944 and 0945 are located in a regulated adjacent basin. Estimates from the regression model for West Groton simulated the actual record within 10 percent for 24 percent of the calibration period and within 5 percent for 11 percent of the period. Attempts to improve this regression model were hindered by the paucity of streamflow data within the basin.

The streamflow record at Forestdale (1115) was not reproduced with any acceptable degree of accuracy by regression techniques. The best-fit regression model for Forestdale includes, as explanatory variables, the lagged and unlagged streamflow at station 1113. Station 1113 is the nearest upstream station within the basin. Estimates from the regression model for Forestdale simulated the actual record within 10 percent for 37 percent of the calibration period and within 5 percent for 18 percent of the period. Attempts to improve this regression model were hindered by the extent of ungaged drainage area within the basin and also by the regulation upstream from Forestdale.

The streamflow record at Westerly (1185) was simulated with a regression model that includes, as explanatory variables, the lagged and unlagged streamflow at station 1175, the streamflow at station 1180, and the streamflow at station 1183. Station 1175 is located upstream on the main stem of the river, and stations 1180 and 1183 are located on tributary streams within the basin.

Estimates from the regression model for Westerly simulated the actual record within 10 percent for 51 percent of the calibration period and within 5 percent for 30 percent of the period. Again, improvements in the regression model were hindered primarily by the upstream regulation and the amount of ungaged drainage area within the basin.

The streamflow record at Shattuckville (1690) was not reproduced with any acceptable degree of accuracy by regression techniques. The best-fit regression model for Shattuckville used the streamflow at Station 1701 as the explanatory variable. Station 1701 is located in an adjacent basin. The simulated data for Shattuckville were within 10 percent of the actual flows for 24 percent of the calibration period and within 5 percent for only 13 percent of the period. The inability to improve the regression model for Shattuckville is related to the paucity of streamflow data within the basin.

The streamflow record at Belchertown (1749) was simulated with a regression model that includes, as explanatory variables, the lagged and unlagged streamflow at station 1746. Station 1746 is the nearest upstream station within the basin. The simulated data for Belchertown were within 10 percent of the actual flows for 47 percent of the calibration period and within 5 percent for 23 percent of the period. Additional improvement in the regression model was restricted by the high percentage of ungaged drainage within the basin.

The simulation of streamflow at Williamstown (3325) was one of the most successful of the seven regression models evaluated. The regression model for Williamstown includes, as explanatory variables, the lagged and unlagged streamflow at station 3315 and the streamflow at station 3320. Station 3315 is the nearest upstream station in the basin, and station 3320 is located on a major tributary that enters the river between stations 3315 and 3325. The simulated data for Williamstown were within 10 percent of the actual flows for 61 percent of the calibration period and within 5 percent for 34 percent of the period. Additional improvement in the regression model was restricted by the high percentage of ungaged drainage within the basin.

Alternative methods conclusions

Simulation of streamflow with either flow-routing or regression methods at the seven stations evaluated was not sufficiently accurate to use these methods in lieu of operating a continuous-flow stream gage. All seven stations should remain in operation as part of the Massachusetts and Rhode Island stream-gaging programs.

COST-EFFECTIVE RESOURCE ALLOCATION

Introduction to K-CERA

In a study of the cost-effectiveness of a network of stream gages operated to determine water consumption in the Lower Colorado River Basin, a set of techniques called K-CERA (Kalman Filtering for Cost-Effective Resource Allocation) was developed (Moss and Gilroy, 1980). Because of the water-balance nature of that study, the measure of effectiveness of the network was chosen to be the minimization of the sum of variances of errors of estimation of annual mean discharges at each site in the network. This measure of effectiveness tends to concentrate stream-gaging resources on the larger, less stable streams where potential errors are greatest. Although such a tendency is appropriate for a water-balance network, in the broader context of the multitude of uses of the streamflow data collected in the Survey's Streamflow Information Program, this tendency causes undue concentration on larger streams. Therefore, the original version of K-CERA was extended to include as optional measures of effectiveness the sums of the variances of errors of estimation of the following streamflow variables: Annual mean discharge, in cubic feet per second; annual mean discharge, in percentage; average instantaneous discharge, in cubic feet per second; or average instantaneous discharge, in percentage. The use of percentage errors does not unduly weight activities at large streams to the detriment of records on small streams. In addition, the instantaneous discharge is the basic variable from which all other streamflow data are derived. For these reasons, this study used the K-CERA techniques with the sums of the variances of the percentage errors of the instantaneous discharges at all continuously gaged sites as the measure of the effectiveness of the data-collection activity.

The original version of K-CERA also did not account for error contributed by missing stage or other correlative data that are used to compute streamflow data. The probabilities of missing correlative data increase as the period between service visits to a stream gage increases. A procedure for dealing with the missing record has been developed and was incorporated into this study (Fontaine and others, 1984).

Brief descriptions of the mathematical program used to optimize cost-effectiveness of the data-collection activity and of the application of Kalman filtering (Gelb, 1974) to the determination of the accuracy of a stream-gaging record are presented below. For more detail on either the theory or the applications of K-CERA, see Moss and Gilroy (1980) and Gilroy and Moss (1981).

Description of Traveling Hydrographer

The mathematical program used to optimize the cost-effectiveness of the data-collection activity is called Traveling Hydrographer. Traveling Hydrographer attempts to allocate among stream gages a predefined budget for the collection of streamflow data in such a manner that the field operation is the most cost-effective possible. The measure of effectiveness is discussed above. The set of decisions available to the manager is the frequency of use (number of times per year) of each of a number of routes that may be used to service the stream gages and to make discharge measurements. The range of options within the program is from zero usage to daily usage for each route. A route is defined as a set of one or more stream gages and the least cost travel that takes the hydrographer from his base of operations to each of the gages and back to base. A route will have associated with it an average cost of travel and average cost of servicing each stream gage visited along the way. The first step in this part of the analysis is to define the set of practical routes. This set of routes commonly will include the path to an individual stream gage (with that gage as the lone stop) and a return to the home base, so that the individual needs of a stream gage can be considered in isolation from the other gages.

Another step in this part of the analysis is the determination of any special requirements for visits to each of the gages for such things as necessary periodic maintenance, rejuvenation of recording equipment, or required periodic sampling of water-quality data. Such special requirements are considered to be inviolable constraints in terms of the minimum number of visits to each gage.

The final step is to use all of the above to determine the number of times, N_i, that the i^{th} route for $i = 1, 2, ..., NR$, where NR is the number of practical routes, is used during a year such that (1) the budget for the network is not exceeded, (2) the minimum number of visits to each station is made, and (3) the total uncertainty in the network is minimized. Figure 11 represents this step in the form of a mathematical program. Figure 12 presents a tabular layout of the problem. Each of the NR routes is represented by a row in the table, and each of the stations is represented by a column. The zero-one matrix, (ω_{ij}), defines the routes in terms of the stations that comprise it. A value of one in row i and column j indicates that gaging station j will be visited on route i; a value of zero indicates that it will not. The unit travel costs, β_i, are the per-trip costs of the hydrographer's traveltime and any related per diem and operation, maintenance, and rental costs of vehicles. The sum of the products of β_i and N_i for $i = 1, 2, ..., NR$ is the total travel cost associated with the set of decisions $N = (N_1, N_2, ..., N_{NR})$.

The unit-visit cost, α_j, is comprised of the average service and maintenance costs incurred on a visit to the station plus the average cost of making a discharge measurement. The set of minimum visit constraints is denoted by the row λ_j, $j = 1, 2, ..., MG$, where MG is the number of stream gages. The row of integers M_j, $j = 1, 2, ..., MG$ specifies the number of visits to each station. M_j is the sum of the products of ω_{ij} and N_i for all i and must equal or exceed λ_j for all j if N is to be a feasible solution to the decision problem.

The total cost expended at the stations is equal to the sum of the products of α_j and M_j for all j. The cost of record computation, documentation, and publication is assumed to be influenced negligibly by the number of visits to the station and is included along with overhead in the fixed cost of operating the network. The total cost of operating the network equals the sum of the travel costs, the at-site costs, and the fixed cost, and must be less than or equal to the available budget.

The total uncertainty in the estimates of discharges at the MG stations is determined by summing the uncertainty functions, ϕ_j, evaluated at the value of M_j from the row above it, for $j = 1, 2, ..., MG$.

As pointed out in Moss and Gilroy (1980), the steepest descent search used to solve this mathematical program does not guarantee a true optimum solution. However, the locally optimum set of values for N obtained with this technique specifies an efficient strategy for operating the network, which may be the true optimum strategy. The true optimum cannot be guaranteed without testing all undominated, feasible strategies.

$$\underset{\underline{N}}{\text{Minimize}}\ V = \sum_{j=1}^{MG} \phi_j\ (M_j)$$

$V \equiv$ total uncertainty in the network

$\underline{N} \equiv$ vector of annual number times each route was used

$MG \equiv$ number of gages in the network

$M_j \equiv$ annual number of visits to station j

$\phi_j \equiv$ function relating number of visits to uncertainty at station j

Such that

Budget $\geq T_c \equiv$ total cost of operating the network

$$T_c = F_c + \sum_{j=1}^{MG} \alpha_j M_j + \sum_{i=1}^{NR} \beta_i N_i$$

$F_c \equiv$ fixed cost

$\alpha_j \equiv$ unit cost of visit to station j

$NR \equiv$ number of practical routes chosen

$\beta_i \equiv$ travel cost for route i

$N_i \equiv$ annual number times route i is used (an element of $\underline{N}$)

and such that

$$M_j \geq \lambda_j$$

$\lambda_j \equiv$ minimum number of annual visits to station j

Figure 11.--Mathematical programming form of the optimization of the routing of hydrographers

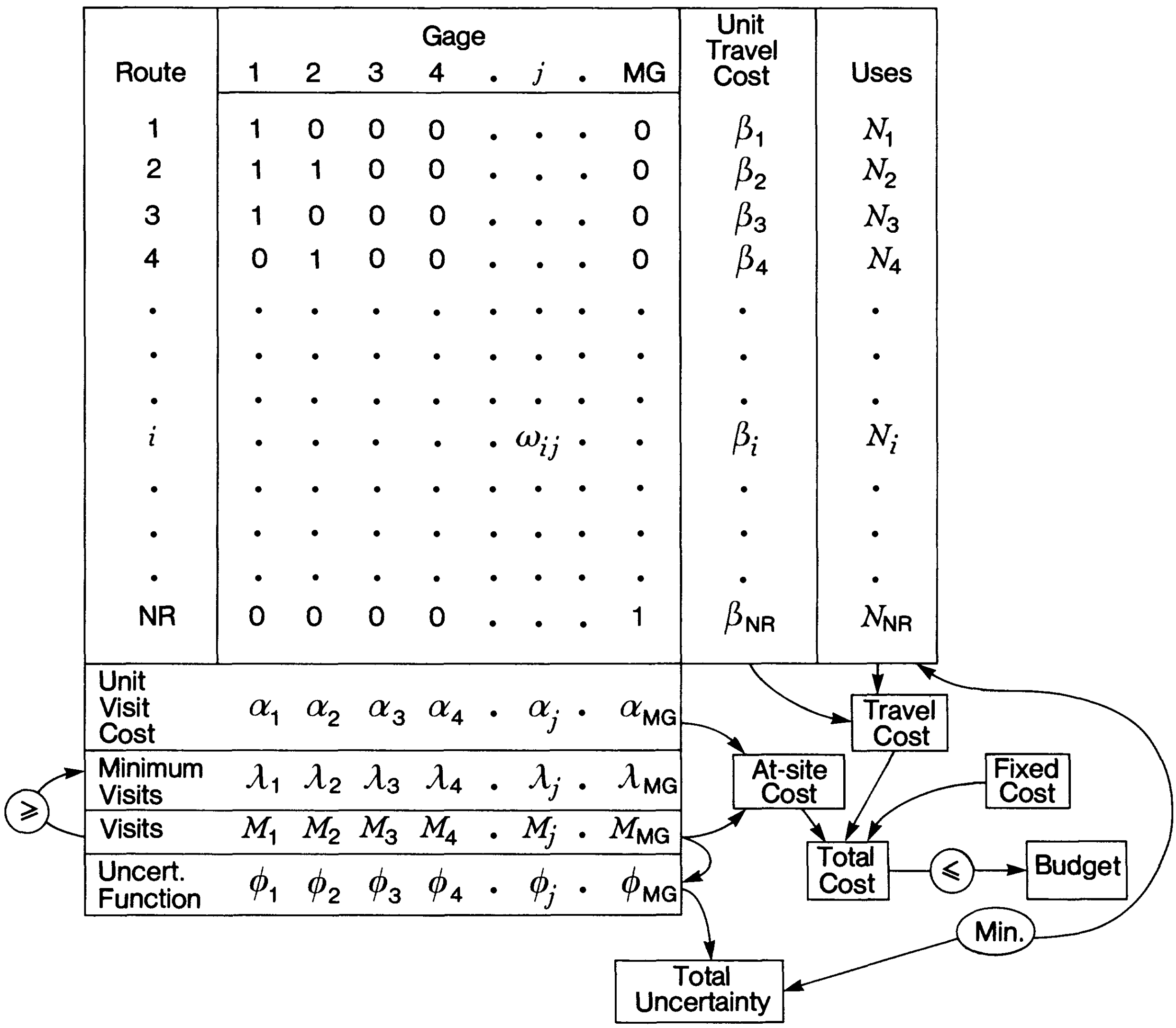

Figure 12.--Tabular form of the optimization of the routing of hydrographers

Description of Uncertainty Functions

As noted earlier, uncertainty in streamflow records is measured in this study as the average relative variance of estimation of instantaneous discharges. The accuracy of a streamflow estimate depends on how that estimate was obtained. Three situations are considered in this study: (1) Streamflow is estimated from measured discharge and correlative data using a stage-discharge relation (rating curve), (2) the streamflow record is reconstructed using secondary data at nearby stations because primary correlative data are missing, and (3) primary and secondary data are unavailable for estimating streamflow. The variances of the errors of the estimates of flow that would be employed in each situation were weighted by the fraction of time each situation is expected to occur. Thus, the average relative variance would be:

$$\bar{V} = \varepsilon_f V_f + \varepsilon_r V_r + \varepsilon_e V_e \tag{3}$$

with

$$1 = \varepsilon_f + \varepsilon_r + \varepsilon_e$$

where

$\bar{V}$ is the average relative variance of the errors of streamflow estimates,

ε_f is the fraction of time that the primary recorders are functioning,

V_f is the relative variance of the errors of flow estimates from primary recorders,

ε_r is the fraction of time that secondary data are available to reconstruct streamflow records given that the primary data are missing,

V_r is the relative variance of the errors of estimation of flows reconstructed from secondary data,

ε_e is the fraction of time that primary and secondary data are not available to compute streamflow records, and

V_e is the relative error variance of the third situation.

The fractions of time that each source of error is relevant are functions of the frequencies at which the recording equipment are serviced.

The time, τ, since the last service visit until failure of the recorder or recorders at the primary site is assumed to have a negative-exponential probability distribution truncated at the next service time; the distribution's probability density function is:

$$f(\tau) = ke^{-k\tau}/(1-e^{-ks}) \tag{4}$$

where

k is the failure rate in units of $(\text{day})^{-1}$,

e is the base of natural logarithms, and

s is the interval between visits to the site, in days.

It is assumed that, if a recorder fails, it continues to malfunction until the next service visit. As a result,

$$\varepsilon_f = (1-e^{-ks})/(ks) \tag{5}$$

(Fontaine and others, 1984, eq. 21).

The fraction of time, ε_e, that no records exist at either the primary or secondary sites also can be derived assuming that the times between failures at both sites are independent and have negative exponential distributions with the same rate constant. It then follows that:

$$\varepsilon_e = 1 - [2(1-e^{-ks}) + 0.5(1-e^{-2ks})]/(ks)$$

(Fontaine and others, 1984, eqs. 23 and 25).

Finally, the fraction of time, ε_r, that records are reconstructed, based on data from a secondary site, is determined by the equation:

$$\varepsilon_r = 1 - \varepsilon_f - \varepsilon_e. \qquad (6)$$

$$= [(1-e^{-ks}) + 0.5(1-e^{-2ks})] /(ks)$$

The relative variance, V_f, of the error derived from primary record computation is determined by analyzing a time series of residuals that are the differences between the logarithms of measured discharge and the rating curve discharge. The rating curve discharge is determined from a relationship between discharge and some correlative data, such as water-surface elevation at the gaging station. The measured discharge is the discharge determined by field observations of depths, widths, and velocities. If $q_T(t)$ is the true instantaneous discharge at time t and $q_R(t)$ is the value that would be estimated using the rating curve, then:

$$x(t) = ln\ q_T(t) - ln\ q_R(t) = ln\ [q_T(t)/q_R(t)] \qquad (7)$$

is the instantaneous difference between the logarithms of the true discharge and the rating curve discharge.

In computing estimates of streamflow, the rating curve may be continually adjusted on the basis of periodic measurements of discharge. This adjustment process results in an estimate, $q_c(t)$, that is a better estimate of the stream's discharge at time t. The difference between the variable $\hat{x}(t)$, which is defined as:

$$\hat{x}(t) = ln\ q_c(t) - ln\ q_R(t) \qquad (8)$$

and $x(t)$ is the error in the streamflow record at time t. The variance of this difference over time is the desired estimate of V_f.

Unfortunately, the true instantaneous discharge, $q_T(t)$, cannot be determined and thus $x(t)$ and the difference, $x(t) - \hat{x}(t)$, cannot be determined as well. However, the statistical properties of $x(t) - \hat{x}(t)$, particularly its variance, can be inferred from the available discharge measurements. Let the observed residuals of measured discharge from the rating curve be $z(t)$ so that

$$z(t) = x(t) + \nu(t) = ln\ q_m(t) - ln\ q_R(t) \qquad (9)$$

where

$\nu(t)$ is the measurement error, and

$ln\ q_m(t)$ is the logarithm of the measured discharge equal to $ln\ q_T(t)$ plus $\nu(t)$.

In the Kalman-filter analysis, the $z(t)$ time series was analyzed to determine three site-specific parameters. The Kalman filter used in this study assumes that the time residuals $x(t)$ arise from a continuous first-order Markovian process that has a Gaussian (normal) probability distribution with zero mean and variance (subsequently referred to as process variance) equal to p. A second important parameter is β, the reciprocal of the correlation time of the Markovian process giving rise to $x(t)$; the correlation between $x(t_1)$ and $x(t_2)$ is $exp[-\beta|t_1-t_2|]$. Fontaine and others (1984) also define q, the constant value of the spectral density function of the white noise that drives the Gauss-Markov x-process. The parameters, p, q, and β are related by:

$$Var[(x)t] = p = q/(2\beta) \tag{10}$$

The variance of the observed residuals $z(t)$ is:

$$Var[z(t)] = p + r \tag{11}$$

where r is the variance of the measurement error $v(t)$. The three parameters, p, β, and r, are computed by analyzing the statistical properties of the $z(t)$ time series. These three site-specific parameters are needed to define this component of the uncertainty relationship. The Kalman filter uses these three parameters to determine the average relative variance of the errors of estimation of discharges as a function of the number of discharge measurements per year (Moss and Gilroy, 1980).

If the recorder at the primary site fails and there are no concurrent data at other sites that can be used to reconstruct the missing record at the primary site, there are at least two ways of estimating discharges at the primary site. A recession curve could be applied from the time of recorder stoppage until the gage was once again functioning, or the expected value of discharge for the period of missing data could be used as an estimate. The expected-value approach is used in this study to estimate V_e, the relative error variance during periods of no concurrent data at nearby stations. If the expected value is used to estimate discharge, the value that is used should be the expected value of discharge at the time of year of the missing record because of the seasonal variation of streamflow. The variance of streamflow, which also is a seasonally variable parameter, is an estimate of the error variance that results from using the expected value as an estimate. Thus, the coefficient of variation squared $(C_v)^2$ is an estimate of the required relative error variance V_e. Because C_v varies seasonally and the times of failures cannot be anticipated, a seasonally averaged value of C_v is used:

$$\bar{C}_v = \left(\frac{1}{365} \sum_{i=1}^{365} \left(\frac{\sigma_i}{\mu_i} \right)^2 \right)^{1/2} \tag{12}$$

where

σ_i is the standard deviation of daily discharges for the i^{th} day of year,

μ_i is the expected value of discharge on the i^{th} day of the year, and

$(\bar{C}_v)^2$ is used as an estimate of V_e.

The variance V_r of the relative error during periods of reconstructed streamflow records is estimated on the basis of correlation between records at the primary site and records from other gaged nearby sites. The correlation coefficient ρ_c between the streamflows with seasonal trends removed at the site of interest and at the other sites is a measure of their linear relationship. The fraction of the variance of streamflow at the primary site that is explained by data from the other sites is equal to ρ_c^2. Thus, the relative error variance of flow estimates at the primary site obtained from secondary information will be:

$$V_r = (1-\rho_c^2)\, \bar{C}_v^2 \tag{13}$$

Because errors in streamflow estimates arise from three different sources with widely varying precisions, the resultant distribution of those errors may differ significantly from a normal or log-normal distribution. This lack of normality causes difficulty in interpretation of the resulting average estimation variance. When primary and secondary data are unavailable, the relative error variance V_e may be very large. This could yield correspondingly large values of $\bar{V}$ in equation (3); even if the probability that primary and secondary information are not available, ε_e, is quite small.

A new parameter, the EGS (equivalent Gaussian spread), is introduced here to assist in interpreting the results of the analyses. If it is assumed that the various errors arising from the three situations represented in equation (3) are log-normally distributed, the value of EGS is determined by the probability statement that:

$$\text{Probability } [e^{-\text{EGS}} \leq (q_c(t) \ / \ q_T(t)) \leq e^{+\text{EGS}}] = 0.683 \qquad (14)$$

Thus, if the residuals $\ln q_c(t) - \ln q_T(t)$ were normally distributed, $(\text{EGS})^2$ would be their variance. Here, EGS is reported in percent because EGS is defined so that nearly two-thirds of the errors in instantaneous streamflow data will be within plus or minus EGS percent of the reported values.

The Application of K-CERA in Massachusetts and Rhode Island

As a result of the analyses in the first two parts of this report, 63 stations in Massachusetts and 15 stations in Rhode Island were subjected to the K-CERA analysis with results that are described below.

Definition of Missing Record Probabilities

As was described earlier, the statistical characteristics of missing stage or other correlative data for computation of streamflow records can be defined by a single parameter, the value of k in the truncated negative exponential probability distribution of times to failure of the equipment. In the representation of f_τ as given in equation 4, the average time to failure is $1/k$. The value of $1/k$ will differ from site to site depending upon the type of equipment at the site and upon its exposure to natural elements and vandalism. The value of $1/k$ can be changed by advances in the technology of data collection and recording. A period of actual data collection of 6 years duration in which little change in technology occurred and in which stream gages were consistently visited nine times each year was used to estimate $1/k$ in Massachusetts and Rhode Island. During this 6-year period, a gage could be expected to malfunction an average of 5.2 percent of the time in Massachusetts and 5.6 percent of the time in Rhode Island. Use of these percentages and nine annual visits resulted in determinations of values of $1/k$ of 376 days for Massachusetts and 348 days for Rhode Island. These values of $1/k$ were used to determine ε_f, ε_r, and ε_e for the stream gages in the respective states. Tables 16 and 17 show how the missing record functions vary with visit frequency.

Table 16.—Summary of probabilities used in computing uncertainty functions in the Massachusetts study

Number of visits per year	ε_f	ε_r	ε_e
0	0.000	0.000	1.000
1	.640	.199	.161
2	.792	.152	.055
4	.888	.096	.016
6	.923	.069	.008
8	.942	.054	.004
9	.948	.048	.004
10	.953	.044	.003
12	.961	.037	.002
15	.968	.030	.001
20	.976	.023	.001
24	.980	.019	.001
36	.987	.013	.000

Table 17.—Summary of probabilities used in computing uncertainty functions in the Rhode Island study

Number of visits per year	ε_f	ε_r	ε_e
0	0.000	0.000	1.000
1	.620	.201	.179
2	.778	.159	.063
4	.880	.101	.019
6	.918	.073	.009
8	.937	.058	.005
9	.944	.052	.004
10	.949	.047	.003
12	.958	.040	.002
15	.966	.033	.002
20	.974	.025	.001
24	.978	.021	.001
36	.986	.014	.000

Definition of Cross-Correlation Coefficient and Coefficient of Variation

The values of V_e and V_r of the needed uncertainty functions were computed using daily streamflow records for each of the 78 stations for the last 30 years or the part of the last 30 years for which daily streamflow values are stored in WATSTORE (Hutchinson, 1975). For each of the stream gages that had three or more complete water years of data, the value of C_v was computed and various options, based on combinations of other stream gages, were explored to determine the maximum ρ_c. Values of C_v and ρ_c were estimated for one station, Sudbury River at Saxonville, that had less than 3 water years of data. In addition to other nearby stream gages, some of the stations have other means by which streamflow data can be reconstructed when the primary recorder is malfunctioning: Some stations are equipped with telemetry systems that operate independently from the primary recorder and are frequently queried; some stations have a local observer who reads and records once or twice daily; operators of hydropower plants and dams upstream may have rated their turbines and spillways to determine the discharge that passes through them and keep flow records that can be used for streamflow reconstruction; auxiliary recorders are operated at some station to provide backup stage record.

In the cases where once- or twice-daily readings of stage (observer or telemetry) are available, values of ρ_c were assumed to be 0.96 for daily readings and 0.99 for twice-daily readings. These values were taken from results obtained in Maine (Fontaine and others, 1984) and were assumed to be applicable in Massachusetts and Rhode Island as well.

At stations where dam records are available for record reconstruction, analyses were performed to determine cross correlations, ρ_c, between daily discharges at sites and the furnished record. Three station-dam pairs were studied--West Deerfield, Charlemont, and Rowe—and an average value of $\rho_c = 0.96$ was determined. This value was used at all locations where dam records are available to reconstruct station records.

For all stations where an auxiliary and independent recorder is available at the station, a value of ρ_c of 0.99 was assumed.

In the Maine study (Fontaine and others, 1984), the uncertainty V_e was assumed to be equal to C_v^2, the coefficient of variation. For Massachusetts and Rhode Island, this assumption was felt to be overly conservative. It was reasoned that some source of auxiliary data would always be available to reconstruct record at a station even if the primary source of data for reconstruction (maximum ρ_c) were not available. In this study, a new variable, R_2, the secondary cross-correlation coefficient, is used. The value of R_2 is assumed to be the second highest cross-correlation value obtained in the ρ_c analysis. The value of uncertainty, V_e, now is estimated by the product $(1 - R_2^2)C_v^2$.

The set of parameters for each station and the auxiliary records that gave the highest cross-correlation coefficient, ρ_c, and the second highest cross correlation, R_2, are listed in table 18.

Table 18.—Statistics of record reconstruction

Station number	C_v	P_c	R_2	Station or other source of reconstructed records
		MASSACHUSETTS		
0944	73.0	0.838	—	0945, 0960
	73.0	—	0.776	0960
0945	88.6	.916	—	0944, 0960
	88.6	—	.834	0944
0960	107	.895	—	0945
	107	—	.776	0944
0965	89.5	.99	—	Supplemental recorder at site
	89.5	—	.746	0995
0969.10	108	.684	—	0973
	108	—	.672	0960
0970	107	.892	—	0973, 0965
	107	—	.812	0973
0973	107	.852	—	0970, 1006
	107	—	.808	1006
0985.30	105	.85e	—	None—less than 3 years of data
	105	—	.80e	
0995	88.7	.99	—	Supplemental recorder at site
	88.7	—	.863	0970, 0965
1000	78.4	.952	—	0920
	78.4	—	.808	0995
1006	94.9	.830	—	0973, 1010
	94.9	—	.709	1010
1010	115	.914	—	1015, 1006
	115	—	.709	1006
1015	134	.828	—	1025
	134	—	.752	1006
1020	123	.950	—	1010, 0995
	123	—	.895	0995
1025	139	.99	—	Supplemental recorder at site
	123	—	.830	1010, 1015
1035	102	.99	—	Supplemental recorder at site
	102	—	.969	1040, 1042
1040	158	.896	—	1035, 1042
	158	—	.751	1042
1042	74.2	.950	—	1045, 1035, 1040
	74.2	—	.940	1035, 1040
1045	93.2	.968	—	1035, 1040
	93.2	—	.945	1042
1050	101	.926	—	1055
	101	—	.804	1057.30
1055	100	.99	—	Supplemental recorder at site
	100	—	.926	1050
1055.85	95.8	.638	—	1050, 1025
	95.8	—	.616	1050

e, estimated.

Table 18.—Statistics of record reconstruction (continued)

Station number	C_v	P_c	R_2	Station or other source of reconstructed records
		MASSACHUSETTS (Continued)		
1056	107	0.892	—	1057.30
	107	—	0.798	1090.70
1057.30	94.6	.929	—	1056, 1090
	94.6	—	.809	1090
1058.70	68.7	.804	—	1057.30, 1056
	68.7	—	.707	1056
1058.80	58.8	.443	—	1058.70
	58.8	—	0	—
1085	114	.939	—	1090
	114	—	.870	1090.60
1090	109	.939	—	1085
	109	—	.920	1090.60
1090.60	80.8	.920	—	1090
	80.8	—	.870	1085
1090.70	128	.888	—	1090.60, 1056
	128	—	.800	1090
1100	103	.776	—	1115
	103	—	.762	1112
1112	97.1	.762	—	1110
	97.1	—	.735	1115
1233.60	69.9	.96	—	Upstream reservoir record
	69.9	—	.876	1236
1236	84.9	.96	—	Upstream reservoir record
	84.9	—	.924	1233.60, 1240
1243.50	94.4	.96	—	Upstream reservoir record
	94.4	—	.840	1245
1245	120	.96	—	Upstream reservoir record
	120	—	.840	1243.50
1620	106	.896	—	1625
	106	—	.834	0960
1625	134	.910	—	1620, 1632
	134	—	.816	1632
1632	89.4	.869	—	1625, 1653
	89.4	—	.816	1625
1640	74.5	.958	—	1620, 1625, 1665
	74.5	—	.909	1620, 1625
1650	125	.96	—	Upstream reservoir record
	125	—	.96	Upstream reservoir record
1653	97.5	.826	—	1632
	97.5	—	.787	1625
1665	94.2	.976	—	1640, 1650, 1653
	94.2	—	.925	1650, 1653
1681.51	67.4	.96	—	Upstream hydropower plant
	67.4	—	.829	1685

Table 18.—Statistics of record reconstruction (continued)

Station number	C_v	P_c	R_2	Station or other source of reconstructed records
		MASSACHUSETTS (Continued)		
1685	82.7	0.99	—	Upstream hydropower plant
	82.7	—	0.980	1700, 1681.51
1690	135	.99	—	Telemetry; read twice daily
	135	—	.942	1701, 3320
1699	102	.882	—	1690, 1715
	102	—	.852	1690
1700	89.4	.97	—	Upstream hydropower plant
	89.4	—	.961	1685
1701	104	.916	—	1699, 3320
	104	—	.911	1690
1705	77.2	.993	—	1700, 1670
	77.2	—	.755	1700
1713	101	.829	—	1715
	101	—	.806	1745
1715	126	.923	—	1699, 1810
	126	—	.860	1699
1725	119	.96	—	Upstream reservoir record
	119	—	.96	Telemetry; read daily
1735	101	.900	—	1730
	101	—	.898	1745
1745	118	.914	—	1730
	118	—	.868	1632
1746	149	.99	—	Supplemental recorder at site
	149	—	.971	1749
1749	132	.99	—	Supplemental recorder at site
	132	—	.971	1746
1755	96.2	.96	—	Upstream reservoir record
	96.2	—	.96	Upstream reservoir record
1756.70	112	.902	—	1760, 1745
	112	—	.831	1745
1760	107	.919	—	1770, 1750
	107	—	.858	1756.70
1770	83.4	.958	—	1760, 1735
	83.4	—	.902	1760
1795	136	.99	—	Supplemental recorder at site
	136	—	.96	Upstream reservoir record
1805	151	.96	—	Upstream reservoir record
	151	—	.841	1810
1810	149	.916	—	1855
	149	—	.841	1805
1835	118	.957	—	1795, 1805, 1810
	118	—	.920	1805, 1810
1855	132	.99	—	Supplemental recorder at site
	132	—	.916	1810

Table 18.—Statistics of record reconstruction (continued)

Station number	C_v	P_c	R_2	Station or other source of reconstructed records
		MASSACHUSETTS (Continued)		
1970	111	0.912	—	1810
	111	—	0.904	1975
1975	89.5	.872	—	1970
	89.5	—	.809	1810
3315	92.8	.946	—	3325
	92.8	—	.872	3320
3320	131	.954	—	3325
	131	—	.902	3340
3325	93.9	.982	—	3315, 3320
	93.9	—	.946	3315
3330	113	.917	—	3315, 3320
	113	—	.874	3320
		RHODE ISLAND		
1113	112	0.888	—	1115
	112	—	0.862	1125
1115	101	.99	—	Supplemental recorder at site
	101	—	.964	1113, 1125
1125	94.6	.96	—	Telemetry; read daily
	94.6	—	.945	1115
1140	91.4	.777	—	1115, 1170
	91.4	—	.712	1115
1145	76.0	.836	—	1165, 1180
	76.0	—	.794	1180
1160	73.2	.853	—	1165
	73.2	—	.705	1115
1165	72.4	.99	—	Supplemental recorder at site
	72.4	—	.901	1145, 1160
1170	94.1	.934	—	1180, 1140
	94.1	—	.759	1140
1173.50	52.8	.867	—	1175, 1180
	52.8	—	.790	1180
1174.20	58.3	.939	—	1174.68, 1175
	58.3	—	.864	1175
1174.68	58.2	.909	—	1174.20, 1175
	58.2	—	.794	1175
1175	62.9	.966	—	1185, 1180
	62.9	—	.96	Telemetry; read daily
1178	69.7	.936	—	1180
	69.7	—	.844	1175
1180	76.4	.954	—	1178, 1175
	76.4	—	.884	1175
1185	72.4	.979	—	1180, 1175
	72.4	—	.930	1180

Kalman-Filter Definition of Variance

The determination of the variance V_f for each of the 78 stations required the execution of three distinct steps: (1) Long-term rating analysis and computation of residuals of measured discharges from the long-term rating, (2) time-series analysis of the residuals to determine the input parameters of the Kalman-filter streamflow records, and (3) computation of the error variance, V_f, as a function of the time-series parameters, the discharge-measurement-error variance, and the frequency of discharge measurements.

In the Massachusetts and Rhode Island analyses, all long-term rating functions for open-water seasons were determined by applying SAS (statistical analysis system), NLIN (nonlinear fitting routines) to discharge measurements and the correlative data. The rating functions determined were of the general form:

$$LQM = B1 + B3 * LOG(GHT - B2) \tag{15}$$

where

LQM is the logarithmic (base *e*) value of the measured discharge,
GHT is the recorded gage height corresponding to the measured discharge,
B1 is the logarithm of discharge for a flow depth of 1 foot,
B2 is the gage height of zero flow, and
B3 is the slope of the rating curve.

The rating functions determined above were then used to compute residuals of the discharge measurements. The time series of these residuals was used to compute sample estimates of q and β, two of the three parameters required to compute V_f, by determining a best fit autocovariance function to the time series of residuals. Autocovariance functions for sample stations in Massachusetts and Rhode Island are illustrated in figures 13 and 14. Measurement variance, the third parameter, is determined from an assumed constant percentage standard error. For the Massachusetts and Rhode Island programs, all open-water measurements were assumed to have a measurement error of 2 percent except for those at North Adams, where the measurement error was assumed to be 5 percent.

As discussed earlier, q and β can be expressed as the process variance of the shifts from the rating curve and the 1-day autocorrelation coefficient of these shifts. Table 19 presents a summary of the autocovariance analysis expressed in terms of process variance and 1-day autocorrelation.

The last column in table 19 is the length of period, in days, to which the computed parameters were applied. The parameters are not applicable for the entire year when the average winter ice-backwater period exceeds about 45 days. For 12 stations in Massachusetts, the average ice period exceeded 45 days and alternate analyses were required.

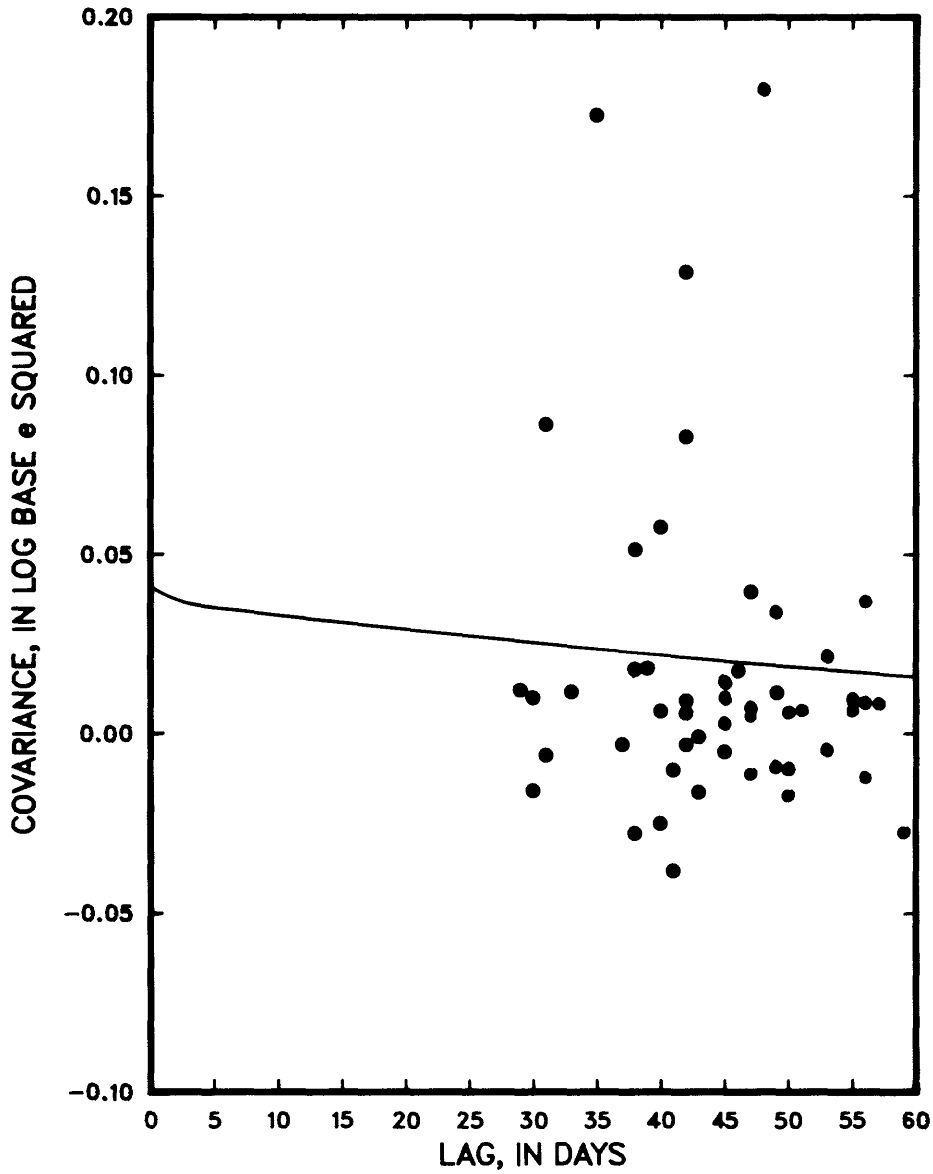

Figure 13.--Autocovariance function for Town Brook (1055.85)

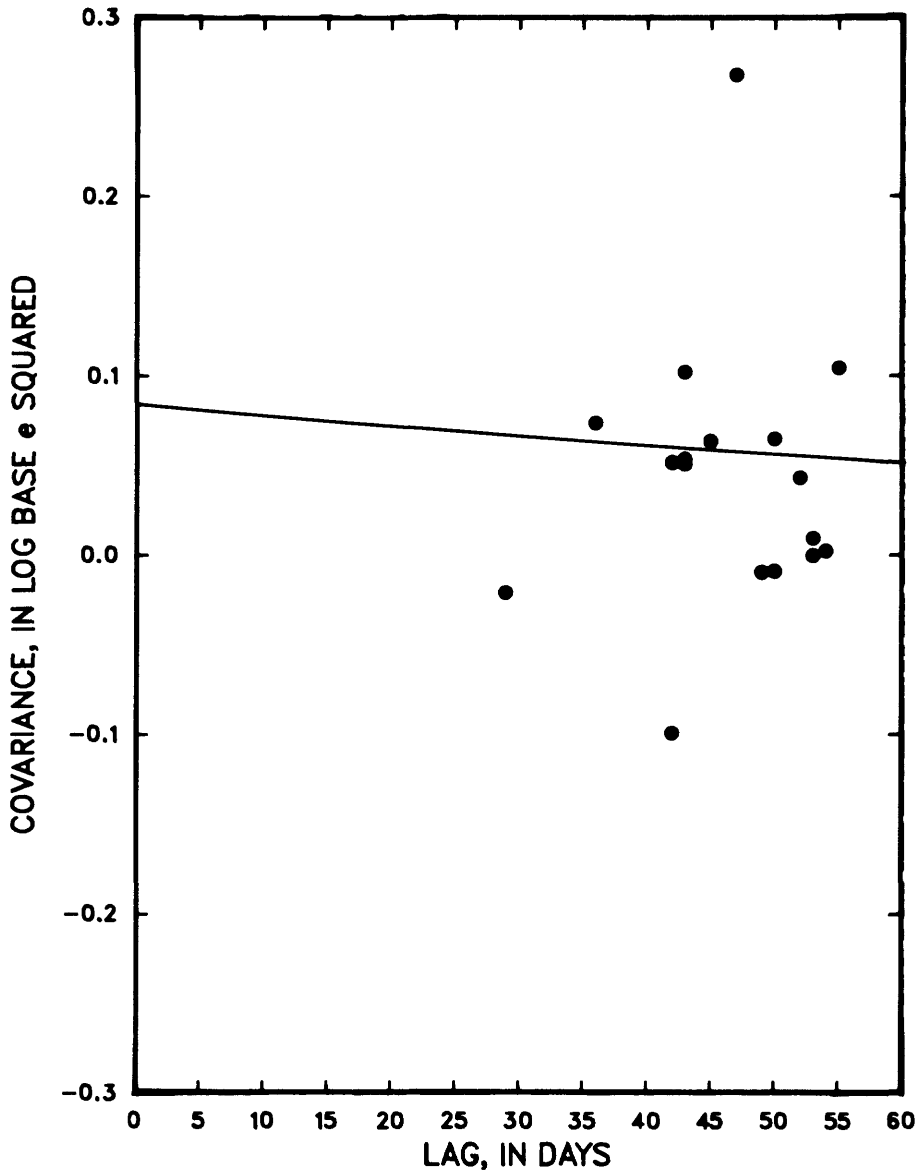

Figure 14.—–Autocovariance function for Hunt River (1170)

Table 19.—Summary of the autocovariance analysis

Station number	RHO (1-day autocorrelation coefficient)	Measurement variance (log base e)^2	Process variance (log base e)^2	Length of period (days)
		MASSACHUSETTS		
0944	0.986	0.0004	0.0008	365
0960	.673	.0004	.0009	365
0965	.903	.0004	.0005	365
0970	.987	.0004	.0648	365
0973	.725	.0004	.0023	365
0985.30	.959	.0004	.0072	365
0995	.945	.0004	.0011	365
1000	.938	.0004	.0017	365
1006	.996	.0004	.0135	365
1010	.955	.0004	.0008	365
1015	.685	.0004	.0010	365
1020	.982	.0004	.0012	365
1025	.984	.0004	.0005	365
1035	.981	.0004	.0016	365
1040	.973	.0004	.0042	365
1042	.958	.0004	.0049	365
1045	.959	.0004	.0093	365
1050	.656	.0004	.0009	365
1055	.996	.0004	.0567	365
1055.85	.986	.0004	.0372	365
1056	.971	.0004	.0534	320
1057.30	.696	.0004	.0011	365
1058.70	.990	.0004	.0729	365
1090	.964	.0004	.0011	365
1090.60	.963	.0004	.0028	365
1112	.961	.0004	.0020	365
1233.60	.932	.0004	.0008	365
1236	.980	.0004	.0027	365
1243.50	.627	.0004	.0002	365
1245	.915	.0004	.0130	365
1620	.920	.0004	.0058	315
1625	.992	.0004	.0095	365
1632	.735	.0004	.0004	365
1640	.990	.0004	.0005	305
1650	.631	.0004	.0015	365
1653	.981	.0004	.0004	365
1655	.976	.0004	.0013	317
1681.51	.677	.0004	.0043	365
1685	.958	.0004	.0013	365
1690	.975	.0004	.0025	311

Table 19.—Summary of the autocovariance analysis (continued)

Station number	RHO (1-day autocorrelation coefficient)	Measurement variance (log base e)^2	Process variance (log base e)^2	Length of period (days)
		MASSACHUSETTS (Continued)		
1699	0.983	0.0004	0.0126	285
1700	.619	.0004	.0001	365
1705	.970	.0004	.0004	365
1713	.989	.0004	.0077	318
1715	.991	.0004	.0030	315
1725	.985	.0004	.0016	365
1735	.976	.0004	.0028	310
1745	No measurements	0	.0025	365
1746	.989	.0004	.0039	365
1749	.755	.0004	.0012	365
1755	.699	.0004	.0020	365
1760	.912	.0004	.0010	310
1770	.767	.0004	.0018	365
1795	.707	.0004	.0011	365
1805	.527	.0004	.0044	365
1810	.996	.0004	.0461	365
1835	.979	.0004	.0035	365
1855	.965	.0004	.0020	313
1970	.907	.0004	.0012	365
1975	.984	.0004	.0055	320
3315	.980	.0004	.0075	365
3320	.625	.0025	.0003	365
3325	.984	.0004	.0719	365
		RHODE ISLAND		
1113	0.982	0.0004	0.0034	365
1115	.966	.0004	.0012	365
1125	.963	.0004	.0017	365
1140	.973	.0004	.0010	365
1145	.995	.0004	.0648	365
1160	.984	.0004	.0151	365
1165	.983	.0004	.0084	365
1170	.991	.0004	.0831	365
1173.50	.925	.0004	.0219	365
1174.20	.976	.0004	.0211	365
1174.68	.991	.0004	.0393	365
1175	.630	.0004	.0006	365
1178	.996	.0004	.0060	365
1180	.988	.0004	.0018	365
1185	.985	.0004	.0444	365

In the Maine study (Fontaine and others, 1984) this was accomplished by computing a rating function for the ice-backwater (winter) period and then proceeding with an analysis as was done for the open-water (summer) portion of the year. This type of analysis requires more discharge measurement data than was available for the Massachusetts stations. This difficulty was overcome by assuming that the variance for the winter period, V_{f_w}, could be approximated by the expression $(1 - \rho_c^2)C_v^2$. This assumption was made seasonally correct by recomputing C_v, which had been computed for the entire year, to reflect only that portion of the year to which it would be applied. This was accomplished by applying the following revised forms of equation 12.

$$C_{v_w} = 100 \left(\frac{1}{X} \sum_{i=1}^{X} \left(\frac{\sigma_i}{\mu_i} \right)^2 \right)^{1/2} \qquad (16)$$

$$C_{v_s} = 100 \left(\frac{1}{365-X} \sum_{i=1}^{365-X} \left(\frac{\sigma_i}{m_i} \right)^2 \right)^{1/2} \qquad (17)$$

where

C_{v_w} is the coefficient of variation for the ice-backwater (winter) season,
C_{v_s} is the coefficient of variation for the open-water (summer) season, and
X is the length of ice-backwater season, in days.

The results of this ice-backwater variance analysis are summarized in table 20. In this table, the last digit of the station number was changed to a 9 to identify it as an ice-backwater station.

The autocovariance parameters summarized in tables 19 and 20 and data from the definition of missing record probabilities summarized in table 16 are used jointly to define uncertainty functions for each gaging station. The uncertainty functions give the relationship of total error variance to the number of visits and discharge measurements. The stations for which graphical fits of the autocovariance functions were previously given present typical examples of uncertainty functions and are given in figure 15. These functions are based on the assumption that a measurement was made during each visit to the station.

Table 20.—Summary of ice-backwater variance analysis

Station number	Summer (C_{v_s})	Winter (C_{v_w})	Variance (V_{f_w}) (log base e)^2	Length of period (days)
1056.09	1.113	1.025	0.1939	45
1620.09	1.101	.940	.1606	50
1640.09	.980	.849	.0575	60
1665.09	.979	.839	.0331	48
1690.09	1.448	1.145	.0258	54
1699.09	1.061	1.040	.2156	80
1713.09	1.056	1.001	.2731	47
1715.09	1.346	1.035	.1476	50
1735.09	1.075	.779	.1093	55
1760.09	1.167	.725	.0784	46
1855.09	1.448	.911	.0164	55
1975.09	.923	.836	.1550	52

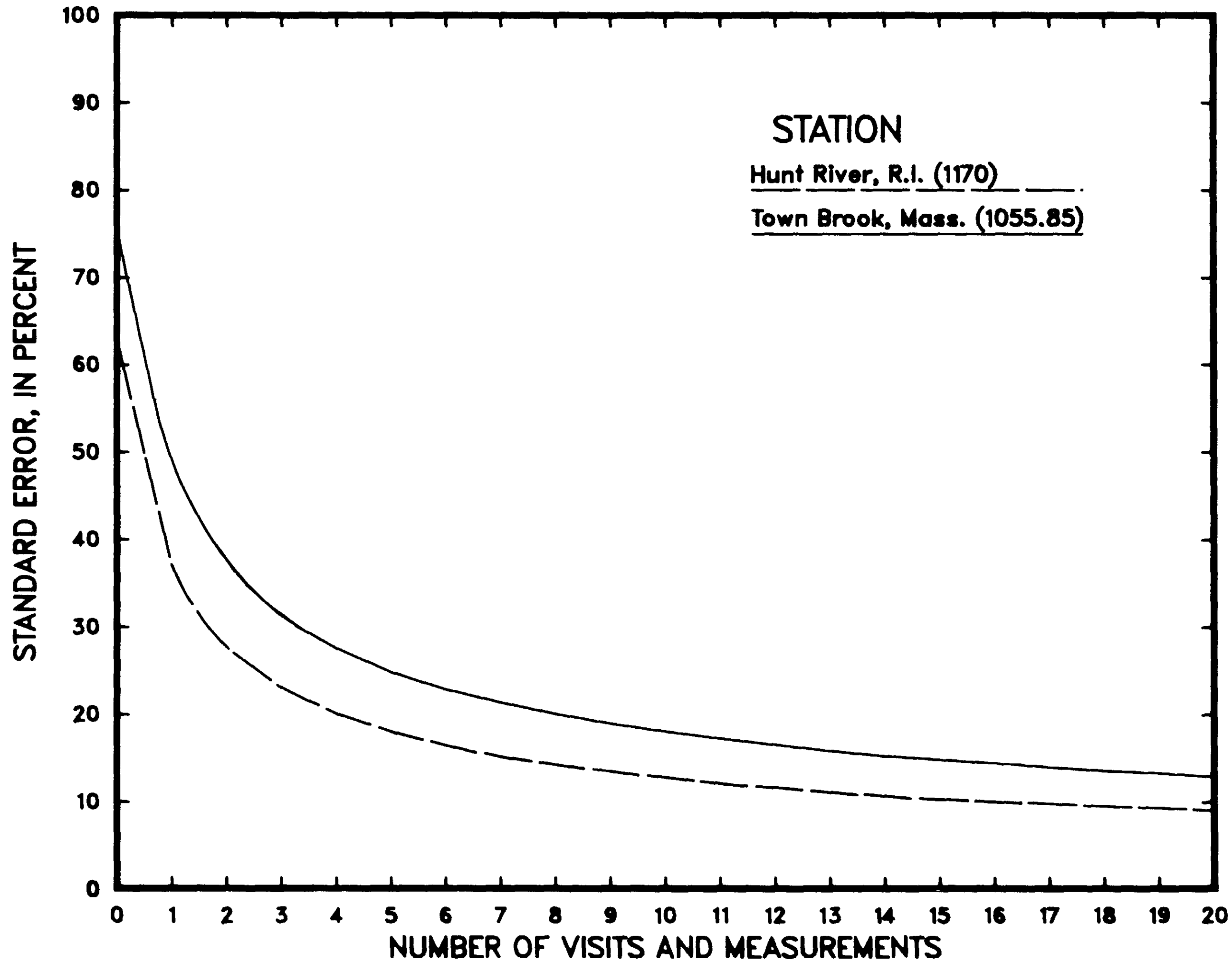

Figure 15.--Typical uncertainty function for instantaneous discharge.

Costs and Routes

Fixed costs to operate each station were estimated. Fixed costs include equipment rental, vehicle rental, batteries, miscellaneous supplies, data processing and storage, computer charges, maintenance, analysis, and supervision. Cost of analysis and supervision, especially analysis, forms a high percentage of the cost of each station and can vary widely. These costs were determined on a station by station basis from past experience. Supervision includes management and data review functions.

Visit costs are those associated with paying the hydrographer for time actually spent at a station servicing the equipment and making a discharge measurement. These costs differ among stations and are functions of the difficulty and time required to make a discharge measurement, amount and complexity of equipment to be serviced, time spent walking to and from the gage structure and (or) the measuring sections, and time to complete documentation of the visit. Average visit times were calculated for each station and ranged from about 30 minutes to about 6 hours.

Part of the visit cost is the time needed to make a discharge measurement. A modification of the Traveling Hydrographer program permits a measurement probability factor to be assigned, from 0 (no measurement) to 1.0 (always measure). A factor was assigned to each station.

Route costs include vehicle costs associated with driving the number of miles it takes to cover the route, cost of the hydrographer's time while in transit, and any per diem associated with the time it takes to complete the trip. Route costs ranged from about $3 for visiting a nearby station in Rhode Island to about $167 for visiting a single station in western Massachusetts (requiring overnight lodging).

In the Maine study (Fontaine and others, 1984), a separate set of costs were developed for the summer and winter seasons for some stations, and those costs were apportioned on the basis of the number of days in the two seasons. It was not considered necessary to do that in the Massachuetts or Rhode Island studies because the route and visit costs, which are the principal variables in the Traveling Hydrographer program, are about the same in both seasons. Stations with ice-backwater problems were accommodated by increasing the fixed costs appropriately.

Eighty-four feasible routes to service 63 stations in Massachusetts and 27 routes to service 15 stations in Rhode Island were developed. Separate Traveling Hydrographer programs were run for the two States because the stations are operated from separate offices, with separate budgets, and with different route limitations.

The routes, and stations visited on each, are summarized in table 21. The routes include combinations that describe the current operating practice, alternatives already under consideration, routes that visit individual stations (as might happen on a "high-water" trip), and combinations that grouped proximate gages where the level of uncertainty indicated more frequent visits might be useful.

Table 21.—Stations on the routes that may be used to visit gaging stations

Route number	Stations serviced on the route							
				ROUTES IN MASSACHUSETTS				
1	1681.51	1685	1690	1699	1700	1705	1970	3315
	3320	3325						
2	1713	1715	1735	1746	1749	1755	1760	1770
	1795	1805	1810	1835	1855	1975		
3	0944	0960	0965	0970	0973	1620	1625	1632
	1640	1650	1653	1665	1725	1745		
4	0985.30	1035	1112	1233.60	1236	1243.50	1245	
5	0995	1000	1006	1010	1015	1020	1025	1042
	1045							
6	1040	1050	1055	1055.85	1056	1057.30	1058.70	1090
	1090.60							
7	1650	1653	1665.09	1681.51	1685	1690.09	1699.09	1700
	1705	1970	3315	3320	3325			
8	1713.09	1715.09	1746	1749	1755	1770	1795	1805
	1810	1835	1855.09	1975.09				
9	0944	0960	0965	0970	0973	0995	1000	1006
	1620.09	1625	1632	1640.09	1725	1745		
10	0985.30	1035	1112	1233.60	1236	1243.50	1245	1735.09
	1760.09							
11	1010	1015	1020	1025	1042	1045		
12	1035	1040	1050	1055	1055.85	1056.09	1057.30	1058.70
	1090	1090.60						
13	1650	1653	1665	1681.51	1685	1690	1699	1700
	1705	1970	3315	3320	3325			
14	1713	1715	1746	1749	1755	1770	1795	1805
	1810	1835	1855	1975				
15	0944	0960	0965	0970	0973	0995	1000	1006
	1620	1625	1632	1640	1725	1745		
16	0985.30	1035	1112	1233.60	1236	1243.50	1245	1735
	1760							
17	1010	1015	1020	1025	1042	1045		
18	1040	1050	1055	1055.85	1056	1057.30	1058.70	1090
	1090.60							
19	1056	1058.70						
20	0985.30	1055.85						
21	1810							

Table 21.--Stations on the routes that may be used to visit gaging stations (continued)

Route number	Stations serviced on the route	
22	0970	0973
23	1090	
24	1699	
25	1713	
26	0944	
27	0960	
28	0965	
29	0970	
30	0973	
31	0985	
32	0995	
33	1000	
34	1006	
35	1010	
36	1015	
37	1020	
38	1025	
39	1035	
40	1040	
41	1042	
42	1045	
43	1050	
44	1055	
45	1055.85	
46	1056	
47	1057.30	
48	1058.70	
49	1090.60	
50	1112	
51	1233.60	
52	1236	
53	1243.50	

Table 21.--Stations on the routes that may be used to visit gaging stations (continued)

Route number	Stations serviced on the route
54	1245
55	1620
56	1625
57	1632
58	1640
59	1650
60	1653
61	1665
62	1681.51
63	1685
64	1690
65	1700
66	1705
67	1715
68	1725
69	1735
70	1745
71	1746
72	1749
73	1755
74	1760
75	1770
76	1795
77	1805
78	1835
79	1855
80	1970
81	1975
82	3315
83	3320
84	3325

Table 21.--Stations on the routes that may be used to visit gaging stations (continued)

Route number	Stations serviced on the route		
	ROUTES IN RHODE ISLAND		
1	1113	1115	1125
2	1140	1145	1165
3	1160	1170	1178
4	1175	1180	1185
5	1173.50	1174.20	1174.68
6	1113	1115	
7	1180	1185	
8	1175	1178	
9	1173.50	1174.20	1174.68
10	1125	1145	
11	1140	1165	
12	1160	1170	
13	1113		
14	1115		
15	1125		
16	1140		
17	1145		
18	1160		
19	1165		
20	1170		
21	1173.50		
22	1174.20		
23	1174.68		
24	1175		
25	1178		
26	1180		
27	1185		

K-CERA Results

The Traveling Hydrographer program uses the uncertainty functions, along with appropriate cost data and route definitions, to compute the most cost-effective way of operating the stream-gaging program. In this application, the first step is to simulate the current practice and determine the associated total uncertainty. To accomplish this, the number of visits made to each stream gage and the specific routes used to make these visits are fixed. The resulting average standard errors for current practices are plotted as points in figures 16 and 17, and are 12.3 percent in Massachusetts and 9.7 percent in Rhode Island.

The solid lines on figures 16 and 17 represent the minimum levels of average uncertainty that can be obtained for given budgets with the existing instrumentation and technology. The lines were defined by several runs of the Traveling Hydrographer program with different budgets. Constraints on the operations, other than budget, were defined as described below.

The minimum number of times each station must be visited was determined by giving consideration only to the physical limitations of the method used to record data. The effect of visitation frequency on the accuracy of the data and amount of lost record is taken into account in the uncertainty analysis. In Massachusetts and Rhode Island, a minimum requirement of five visits per year was calculated and applied to all stations. A minimum of two visits for the winter season and three visits for the summer season was applied at stations where the year was split into winter and summer seasons. These values were based on limitations of the batteries used to drive recording equipment, capacities of the uptake spools on the digital recorders, the need to protect gages from freezing winter conditions, and the need for general maintenance as an extra trip during the summer.

The results in figures 16 and 17 and table 22 summarize the K-CERA analysis. It should be emphasized that the results are based on various assumptions (stated previously) concerning both the time series of shifts to the stage-discharge relationship and the methods of record reconstruction. Where a choice of assumptions was available, the assumption that would not underestimate the magnitude of the error variances was chosen.

There was not a sufficient amount of winter-measurement data to perform a winter-rating analysis, and there is not much variation in the number of winter trips. Two winter visits were assumed for all summer-winter stations. Therefore, the standard errors for winter periods do not change, regardless of the budget used. In table 22, the standard error for an entire year for a summer-winter station can be calculated by weighting the variance by the percentage of year used. For example, the standard error at station 1056 for the current operating budget is 23.1 percent, and was computed from the equation: standard error = $(\text{variance})^{1/2} = [\,0.88\,(17.0)^2 + 0.12\,(48.2)^2\,]^{1/2}$. The fraction of year used for each season is indicated beside the station number.

In Massachusetts, the current operational policy results in an average standard error of 12.3 percent. This policy requires a budget of $353,000 to maintain the 63 continuous-record stream-gaging station program. Standard errors range from a low of 3.0 percent at station 1705 to a high of 25.8 percent (weighted summer-winter average) at station 1699. The highest standard error (17.2 percent) for a station with no winter season occurred at station 1055.85. A higher standard error (19.2 percent) is shown for station 3325, but it is based on earlier data when the rating errors and missing record were more significant. The standard error for that station should now be much like those for stations 3315 or 3320.

The same standard error (12.3 percent) of the present operational policy could be accomplished with a budget of $347,000 with a change of policy. The minimum budget that could sustain the present number of stations is $340,000, for which the standard error would be 12.8 percent. For a budget of $700,000, almost double the present budget, the standard error would be 8.1 percent. As can be seen in figure 16, little improvement in standard error would be accomplished by further increasing the budget.

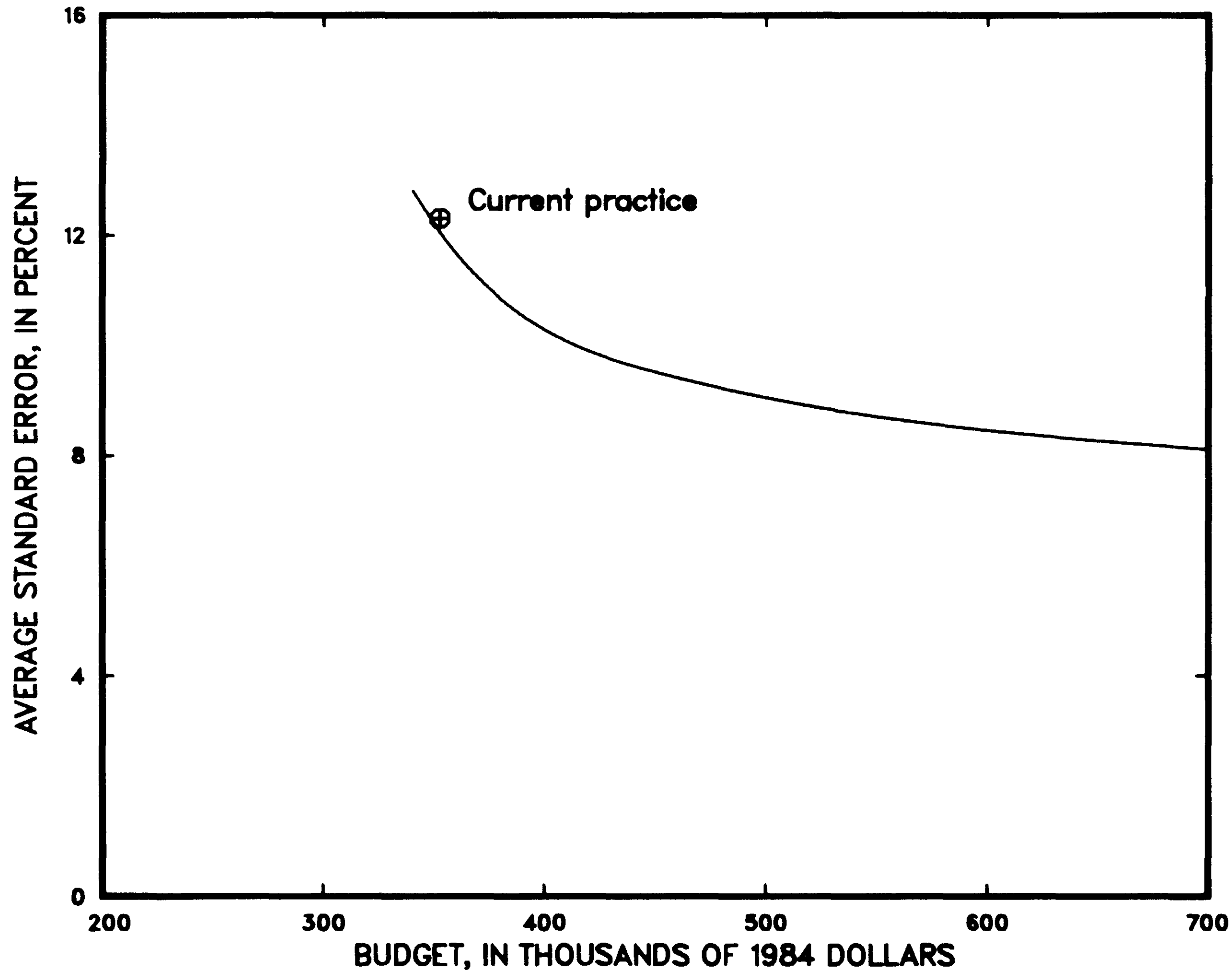

Figure 16.--Average standard error for Massachusetts stations for various budgets.

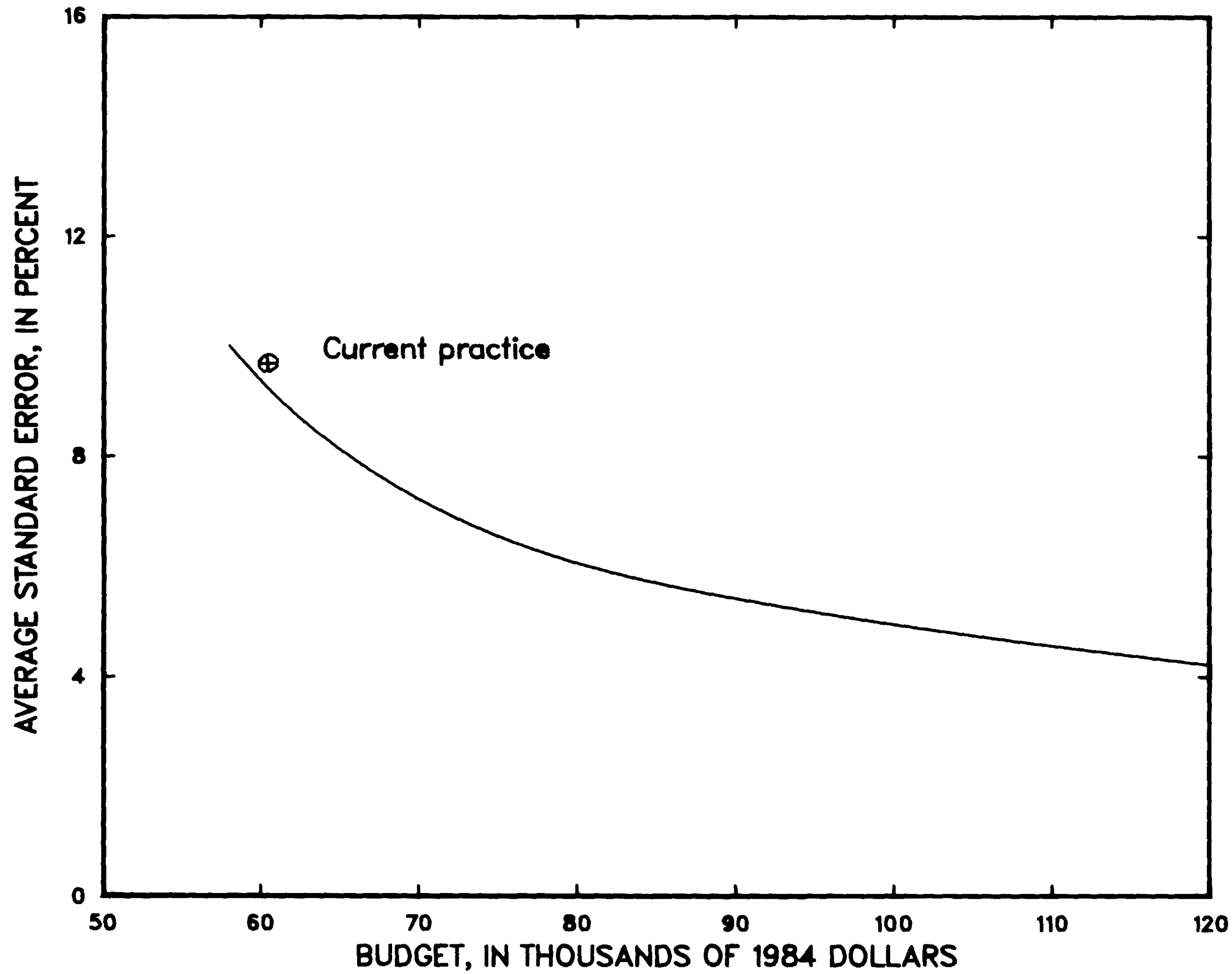

Figure 17.--Average standard error for Rhode Island stations for various budgets.

In Rhode Island, the current operational policy results in an average standard error of 9.7 percent for a budget of $60,500. Standard errors range from a low of 4.2 percent (station 1115) to a high of 13.8 percent (station 1140). With a change of policy, the same standard error (9.7 percent) could be achieved with a budget of $59,000. The minimum budget that could sustain the present number of stations is $58,000, for which the average standard error would be 10.0 percent. For a budget of $120,000, the standard error would be 4.2 percent. Figure 17 shows the change in average standard errors for various budgets.

Table 22.—Selected results of K-CERA analysis

Station	Number of visits (Standard error of instantaneous discharge, in percent) Equivalent Gaussian spread, in percent					
	Current operation (1983)	U.S. Geological Survey's stream-gaging budget in Massachusetts, in 1984 dollars				
		340,000	353,000	370,000	450,000	700,000
Average per station[1]	(12.3)	(12.8)	(12.0)	(11.2)	(9.5)	(8.1)
0944 North Nashua River	9 (9.2) 1.5	6 (11.1) 1.8	8 (9.7) 1.6	9 (9.2) 1.5	18 (6.6) 1.0	46 (4.1) .7
0965 Nashua River	9 (3.5) 2.1	5 (4.3) 2.4	5 (4.3) 2.4	6 (4.0) 2.3	13 (3.1) 2.0	40 (2.0) 1.5
0970 Assabet River	9 (15.5) 11.8	12 (13.5) 10.1	15 (12.1) 9.0	19 (10.8) 7.9	34 (8.1) 5.8	90 (5.0) 3.6
0973 Nashoba Brook	9 (13.5) 4.9	10 (12.9) 4.9	15 (10.9) 4.7	18 (10.1) 4.6	31 (8.1) 4.2	82 (5.4) 3.3
0985.30 Sudbury River	10 (13.3) 6.2	11 (12.7) 6.0	14 (11.4) 5.4	18 (10.1) 4.8	35 (7.3) 3.5	84 (4.8) 2.3
0995 Concord River	8 (4.2) 3.2	5 (4.8) 3.5	5 (4.8) 3.5	5 (4.8) 3.5	13 (3.7) 3.0	39 (2.6) 2.2
1000 Merrimack River	8 (6.7) 3.8	5 (8.1) 4.2	5 (8.1) 4.3	5 (8.1) 4.2	13 (5.6) 3.4	39 (3.4) 2.2
1006 Shawsheen River	8 (13.1) 3.3	9 (12.4) 3.1	12 (10.8) 2.7	15 (9.7) 2.4	28 (7.1) 1.8	65 (4.7) 1.2

[1]Square root of seasonally averaged station variance.

Table 22.—Selected results of K-CERA analysis (continued)

Station	Current operation (1983)	340,000	353,000	370,000	450,000	700,000
	Number of visits (Standard error of instantaneous discharge, in percent) Equivalent Gaussian spread, in percent	U.S. Geological Survey's stream-gaging budget in Massachusetts, in 1984 dollars				
1010 Parker River	11 (10.0) 2.6	8 (11.5) 2.8	11 (10.0) 2.6	15 (8.6) 2.4	27 (6.5) 2.0	71 (4.1) 1.4
1015 Ipswich River, South Middleton	11 (15.8) 3.3	14 (14.2) 3.3	19 (12.3) 3.2	25 (10.8) 3.2	48 (8.1) 3.0	123 (5.4) 2.8
1020 Ipswich River, Ipswich	11 (8.4) 2.8	7 (10.3) 3.2	10 (8.7) 2.9	13 (7.7) 2.6	26 (5.6) 2.0	64 (3.6) 1.3
1025 Aberjona River	11 (4.3) 1.5	8 (5.0) 1.7	8 (5.0) 1.7	8 (5.0) 1.7	14 (3.8) 1.3	43 (2.2) .8
1035 Charles River, Dover	10 (3.8) 2.4	6 (4.8) 3.0	6 (4.8) 3.0	8 (4.2) 2.7	16 (3.1) 1.9	39 (2.0) 1.3
1040 Mother Brook	11 (15.0) 4.3	14 (13.4) 3.8	18 (11.9) 3.4	23 (10.5) 3.0	44 (7.7) 2.2	108 (4.9) 1.5
1042 Charles River, Wellesley	11 (7.2) 5.8	5 (9.2) 6.9	5 (9.2) 6.9	7 (8.3) 6.5	19 (5.9) 4.8	53 (3.8) 3.1
1045 Charles River, Waltham	11 (8.6) 7.5	5 (10.8) 9.2	6 (10.3) 8.8	9 (9.1) 8.0	22 (6.6) 5.8	60 (4.1) 3.6
1050 Neponset River	11 (8.4) 3.1	7 (10.2) 3.2	8 (9.6) 3.2	10 (8.7) 3.1	21 (6.4) 2.9	54 (4.4) 2.6
1055 East Branch Neponset River	11 (5.9) 5.4	6 (7.8) 7.4	6 (7.8) 7.4	8 (6.9) 6.4	14 (5.3) 4.8	31 (3.6) 3.2
1055.85 Town Brook	11 (17.2) 8.4	15 (14.8) 7.1	20 (12.8) 6.1	25 (11.5) 5.4	48 (8.3) 3.9	120 (5.3) 2.5

Table 22.—Selected results of K-CERA analysis (continued)

Station	Number of visits (Standard error of instantaneous discharge, in percent) Equivalent Gaussian spread, in percent					
	Current operation (1983)	U.S. Geological Survey's stream-gaging budget in Massachusetts, in 1984 dollars				
		340,000	353,000	370,000	450,000	700,000
1056 (s 0.88)[2]	9	12	17	22	42	84
Old Swamp River	(17.0)	(15.0)	(12.7)	(11.3)	(8.2)	(5.8)
	14.5	12.6	10.6	9.3	6.7	4.7
1056.09 (w 0.12)[3]	2	2	2	2	2	2
Old Swamp River	(48.2)	(48.2)	(48.2)	(48.2)	(48.2)	(48.2)
	48.2	48.2	48.2	48.2	48.2	48.2
1057.30	11	6	8	10	20	53
Indian Head River	(7.9)	(10.2)	(9.0)	(8.2)	(6.2)	(4.2)
	3.4	3.6	3.5	3.4	3.2	3.8
1058.70	11	11	13	17	31	69
Jones River	(12.5)	(12.5)	(11.5)	(10.1)	(7.5)	(5.1)
	9.9	9.9	9.0	7.8	5.7	3.8
1090	11	7	9	11	20	51
Wading River	(8.0)	(9.9)	(8.8)	(8.0)	(6.0)	(3.8)
	2.3	2.7	2.5	2.3	1.8	1.1
1090.60	11	5	7	9	18	49
Threemile River	(7.7)	(10.6)	(9.3)	(8.4)	(6.3)	(4.0)
	4.5	5.4	5.0	4.7	3.8	2.5
1112	10	11	14	18	34	84
West River	(14.1)	(13.5)	(12.0)	(10.7)	(7.9)	(5.1)
	3.9	3.8	3.6	3.3	2.6	1.7
1233.60	10	6	6	8	16	39
Quinebaug River,	(4.8)	(6.0)	(6.0)	(5.3)	(4.0)	(2.7)
Fiskdale	2.5	2.8	2.8	2.7	2.2	1.6
1236	10	6	6	8	16	39
Quinebaug River,	(5.9)	(7.4)	(7.4)	(6.6)	(4.8)	(3.1)
Southbridge	3.2	4.0	4.0	3.5	2.5	1.7
1243.50	10	6	6	8	16	39
French River,	(5.9)	(7.4)	(7.4)	(6.5)	(4.7)	(3.2)
Hodges Village	1.5	1.6	1.6	1.5	1.4	1.3
1245	10	6	7	10	25	74
Little River	(11.9)	(13.6)	(13.1)	(11.9)	(8.7)	(5.3)
	10.0	11.1	10.8	10.0	7.6	4.6

[2](s 0.88) summer season, 88 percent of year.
[3](w 0.12) winter season, 12 percent of year.

Table 22.—Selected results of K-CERA analysis (continued)

Station	Current operation (1983)	340,000	353,000	370,000	450,000	700,000
	Number of visits (Standard error of instantaneous discharge, in percent) Equivalent Gaussian spread, in percent					
		U.S. Geological Survey's stream-gaging budget in Massachusetts, in 1984 dollars				
1620 (s 0.86)	7	6	8	10	22	56
Millers River,	(13.0)	(13.8)	(12.3)	(11.2)	(8.0)	(5.2)
Winchendon	7.0	7.3	6.8	6.4	4.9	3.2
1620.09 (w 0.14)	2	2	2	2	2	2
Millers River,	(47.0)	(47.0)	(47.0)	(47.0)	(47.0)	(47.0)
Winchendon	47.0	47.0	47.0	47.0	47.0	47.0
1625	9	9	11	14	27	69
Priest Brook	(13.2)	(13.2)	(12.0)	(10.7)	(7.7)	(4.9)
	4.1	4.1	3.7	3.2	2.3	1.5
1632	9	6	8	10	19	49
Otter River	(10.3)	(12.4)	(10.8)	(9.8)	(7.2)	(4.7)
	2.1	2.2	2.1	2.0	1.9	1.6
1640 (s 0.84)	7	3	4	5	11	38
Millers River,	(6.6)	(9.7)	(8.5)	(7.7)	(5.3)	(2.9)
South Royalston	1.7	2.3	2.1	1.9	1.4	.8
1640.09 (w 0.16)	2	2	2	2	2	2
Millers River,	(27.1)	(27.1)	(27.1)	(27.1)	(27.1)	(27.1)
South Royalston	27.1	27.1	27.1	27.1	27.1	27.1
1650	8	5	7	9	18	46
East Branch Tully River	(9.2)	(11.2)	(9.7)	(8.8)	(6.8)	(5.0)
	4.1	4.3	4.2	4.1	3.8	3.5
1653	8	7	10	13	23	58
Lake Rohunta Outlet	(13.3)	(14.2)	(12.0)	(10.5)	(8.0)	(5.0)
	1.4	1.5	1.2	1.1	.8	.5
1665 (s 0.87)	6	3	4	7	16	44
Millers River,	(6.1)	(8.0)	(7.1)	(5.8)	(4.2)	(2.6)
Erving	3.5	4.0	7.4	3.3	2.6	1.8
1665.09 (w 0.13)	2	2	2	2	2	2
Millers River,	(20.6)	(20.6)	(20.6)	(20.6)	(20.6)	(20.6)
Erving	20.6	20.6	20.6	20.6	20.6	20.6
1681.51	7	5	7	8	17	44
Deerfield River,	(7.9)	(8.4)	(7.9)	(7.8)	(7.0)	(6.3)
Rowe	6.9	7.1	6.9	6.8	6.5	6.1

Table 22.—Selected results of K-CERA analysis (continued)

Station	Number of visits (Standard error of instantaneous discharge, in percent) Equivalent Gaussian spread, in percent					
	Current operation (1983)	U.S. Geological Survey's stream-gaging budget in Massachusetts, in 1984 dollars				
		340,000	353,000	370,000	450,000	700,000
1685 Deerfield River, Charlemont	7 (4.5) 3.7	5 (4.8) 3.8	7 (4.5) 3.7	8 (4.4) 3.6	17 (3.7) 3.3	44 (2.9) 2.7
1690 (s 0.85) North River	5 (6.4) 4.1	3 (7.8) 4.9	5 (6.4) 4.1	6 (6.0) 3.8	15 (4.0) 2.6	42 (2.5) 1.6
1690.09 (w 0.15) North River	2 (19.0) 19.0	2 (19.0) 19.0	2 (19.0) 19.0	2 (19.0) 19.0	2 (19.0) 19.0	2 (19.0) 19.0
1699 (s 0.78) South River	5 (14.4) 7.2	5 (14.4) 7.2	5 (14.4) 7.2	6 (13.2) 6.5	15 (8.5) 4.1	42 (5.1) 2.4
1699.09 (w 0.22) South River	2 (47.9) 47.9	2 (47.9) 47.9	2 (47.9) 47.9	2 (47.9) 47.9	2 (47.9) 47.9	2 (47.9) 47.9
1700 Deerfield River, West Deerfield	7 (5.7) 1.1	5 (6.6) 1.4	7 (5.7) 1.1	8 (5.3) 1.1	17 (3.8) 1.0	44 (2.5) 1.0
1705 Connecticut River	7 (3.0) 2.0	5 (3.3) 2.1	7 (3.0) 2.0	8 (2.8) 2.0	17 (2.3) 1.8	44 (1.6) 1.3
1713 (s 0.87) Fort River	6 (15.2) 4.5	7 (14.1) 4.1	9 (12.5) 3.6	11 (11.3) 3.2	21 (8.3) 2.3	52 (5.3) 1.5
1713.09 (w 0.13) Fort River	2 (57.7) 57.7	2 (57.7) 57.7	2 (57.7) 57.7	2 (57.7) 57.7	2 (57.7) 57.7	2 (57.7) 57.7
1715 (s 0.86) Mill River	6 (12.7) 2.6	5 (13.9) 2.9	7 (11.8) 2.4	9 (10.5) 2.1	18 (7.5) 1.5	43 (4.9) 1.0
1715.09 (w 0.14) Mill River	2 (48.4) 48.4	2 (48.4) 48.4	2 (48.4) 48.4	2 (48.4) 48.4	2 (48.4) 48.4	2 (48.4) 48.4

Table 22.—Selected results of K-CERA analysis (continued)

Station	Current operation (1983)	340,000	353,000	370,000	450,000	700,000
	Number of visits (Standard error of instantaneous discharge, in percent) Equivalent Gaussian spread, in percent					
		U.S. Geological Survey's stream-gaging budget in Massachusetts, in 1984 dollars				
1725	9	5	8	8	16	42
Ware River,	(8.2)	(10.6)	(8.6)	(8.6)	(6.3)	(3.9)
Barre	3.3	3.9	3.4	3.4	2.6	1.7
1735 (s 0.85)	6	6	9	9	17	43
Ware River,	(11.8)	(11.8)	(9.8)	(9.8)	(7.2)	(4.6)
Gibbs Crossing	4.0	4.0	3.4	3.4	2.5	1.6
1735.09 (w 0.15)	2	2	2	2	2	2
Ware River,	(44.0)	(44.0)	(44.0)	(44.0)	(44.0)	(44.0)
Gibbs Crossing	44.0	44.0	44.0	44.0		
1745	9	8	13	13	24	59
East Branch Swift River	(12.0)	(12.5)	(10.4)	(10.4)	(8.4)	(6.6)
	5.4	5.4	5.2	5.2	5.1	5.1
1746	8	6	8	8	16	42
Cadwell Creek,	(5.7)	(6.6)	(5.7)	(5.7)	(4.1)	(2.6)
Pelham	3.0	3.4	3.0	3.0	2.1	1.3
1749	8	7	8	8	16	42
Cadwell Creek,	(5.6)	(5.8)	(5.6)	(5.6)	(4.5)	(3.5)
Belchertown	3.6	3.6	3.6	3.6	3.4	2.9
1755	8	6	8	8	16	42
Swift River	(7.8)	(8.6)	(7.8)	(7.8)	(6.3)	(5.0)
	4.7	4.9	4.7	4.7	4.5	4.2
1760 (s 0.85)	6	6	9	9	16	43
Quaboag River	(11.2)	(11.2)	(9.3)	(9.3)	(7.2)	(4.6)
	3.3	3.3	3.1	3.1	2.8	2.1
1760.09 (w 0.15)	2	2	2	2	2	2
Quaboag River	(42.2)	(42.2)	(42.2)	(42.2)	(42.2)	(42.2)
	42.2	42.2	42.2	42.2	42.2	42.2
1770	8	6	8	8	16	42
Chicopee River	(7.0)	(7.7)	(7.0)	(7.0)	(5.7)	(4.5)
	4.5	4.6	4.5	4.5	4.2	3.8
1795	8	6	8	8	16	42
Westfield River,	(5.6)	(6.2)	(5.6)	(5.6)	(4.6)	(3.8)
Knightville	3.5	3.6	3.5	3.5	3.4	3.2

Table 22.—Selected results of K-CERA analysis (continued)

Station	Number of visits (Standard error of instantaneous discharge, in percent) Equivalent Gaussian spread, in percent					
	Current operation (1983)	U.S. Geological Survey's stream-gaging budget in Massachusetts, in 1984 dollars				
		340,000	353,000	370,000	450,000	700,000
1805 Middle Branch Westfield River	8 (12.0) 7.0	6 (13.3) 7.2	8 (12.0) 7.0	8 (12.0) 7.0	16 (9.7) 6.7	42 (7.7) 6.3
1810 West Branch Westfield River	8 (15.4) 6.0	8 (15.4) 6.0	11 (13.2) 5.0	11 (13.2) 5.0	21 (9.7) 3.6	50 (6.3) 2.4
1835 Westfield River, Westfield	8 (9.3) 4.7	6 (10.5) 5.2	8 (9.3) 4.7	8 (9.3) 4.7	16 (6.8) 3.6	42 (4.3) 2.3
1855 (s 0.86) West Branch Farmington River	6 (6.1) 4.2	4 (7.0) 4.5	6 (6.1) 4.2	6 (6.1) 4.2	14 (4.4) 3.3	40 (2.8) 2.1
1855.09 (w 0.14) West Branch Farmington River	2 (18.6) 18.6	2 (18.6) 18.6	2 (18.6) 18.6	2 (18.6) 18.6	2 (18.6) 18.6	2 (18.6) 18.6
1970 East Branch Housatonic River	7 (12.1) 3.5	5 (14.1) 3.7	8 (11.4) 3.4	8 (11.4) 3.4	17 (8.1) 3.0	44 (5.2 2.2
1975 (s 0.88) Housatonic River	6 (12.2) 4.9	4 (14.7) 6.0	6 (12.2) 4.9	6 (12.2) 4.9	14 (8.2) 3.2	40 (4.9) 1.9
1975.09 (w 0.12) Housatonic River	2 (43.8) 43.8	2 (43.8) 43.8	2 (43.8) 43.8	2 (43.8) 43.8	2 (43.8) 43.8	2 (43.8) 43.8
3315 Hoosic River, Adams	7 (9.5) 6.1	5 (10.9) 7.0	8 (9.0) 5.8	8 (9.0) 5.8	17 (6.3) 4.0	44 (4.0) 2.5
3320 North Branch Hoosic River	7 (10.2) 1.9	5 (12.0) 2.0	8 (9.6) 1.9	8 (9.6) 1.9	17 (6.8) 1.8	44 (4.4) 1.7
3325 Hoosic River, Williamstown	7 (19.2) 19.1	6 (20.0) 20.0	12 (15.7) 15.7	12 (15.7) 15.7	24 (11.5) 11.4	60 (7.3) 7.2

Table 22.—Selected results of K-CERA analysis (continued)

Station	Current operation (1983)	58,000	60,500	65,000	90,000	120,000
	Number of visits (Standard error of instantaneous discharge, in percent) Equivalent Gaussian spread, in percent					
		U.S. Geological Survey's stream-gaging budget in Rhode Island, in 1984 dollars				
Average per station[1]	(9.7)	(10.0)	(9.2)	(8.1)	(5.4)	(4.2)
1113	9	9	11	13	28	53
Nipmuc River	(11.2)	(11.2)	(10.2)	(9.4)	(6.5)	(4.7)
	2.9	2.9	2.7	2.4	1.7	1.2
1115	9	5	5	6	17	32
Branch River	(4.2)	(5.3)	(5.3)	(4.9)	(3.2)	(2.4)
	2.8	3.3	3.3	3.2	2.2	1.7
1125	9	5	6	8	20	41
Blackstone River	(7.1)	(9.0)	(8.4)	(7.5)	(5.1)	(3.6)
	3.7	4.2	4.1	3.8	2.9	2.1
1140	9	10	14	17	38	72
Moshassuck River	(13.8)	(13.1)	(11.1)	(10.1)	(6.8)	(5.0)
	2.1	2.0	1.8	1.6	1.1	.8
1145	9	10	12	16	35	67
Woonasquatucket River	(11.9)	(11.4)	(10.4)	(9.0)	(6.2)	(4.5)
	7.4	7.0	6.3	5.4	3.6	2.7
1160	9	11	14	15	39	66
South Branch	(5.5)	(6.9)	(6.9)	(5.8)	(3.8)	(2.9)
Pawtuxet River	5.3	6.7	6.7	5.6	3.5	2.7
1165	9	5	5	8	21	37
Pawtuxet River	(5.5)	(6.9)	(6.9)	(5.8)	(3.8)	(2.9)
	5.3	6.7	6.7	5.6	3.5	2.7
1170	9	13	15	19	45	69
Hunt River	(13.0)	(10.9)	(10.1)	(9.0)	(5.9)	(4.8)
	11.1	9.1	8.4	7.4	4.8	3.9
1173.50	9	7	9	15	45	66
Chipuxet River	(13.6)	(14.3)	(13.6)	(11.8)	(7.6)	(6.3)
	12.9	13.6	12.9	11.2	7.2	5.9
1174.20	9	7	9	14	30	43
Usquepaug River	(9.5)	(10.5)	(9.5)	(7.8)	(5.4)	(4.6)
	8.8	9.8	8.8	7.2	4.9	4.1

[1]Square root of seasonally averaged station variance.

Table 22.—Selected results of K-CERA analysis (continued)

Station	Number of visits (Standard error of instantaneous discharge, in percent) Equivalent Gaussian spread, in percent					
	Current operation (1983)	U.S. Geological Survey's stream-gaging budget in Rhode Island, in 1984 dollars				
		58,000	60,500	65,000	90,000	120,000
1174.68 Beaver River	9 (9.1) 7.7	7 (10.2) 8.8	9 (9.1) 7.7	14 (7.4) 6.0	30 (5.1) 4.1	42 (4.3) 3.4
1175 Pawcatuck River, Wood River Junction	9 (4.5) 2.6	5 (5.6) 2.8	6 (5.2) 2.7	7 (4.9) 2.6	17 (3.7) 2.5	26 (3.3) 2.4
1178 Wood River, Arcadia	9 (6.2) 2.4	5 (8.2) 3.3	6 (7.5) 3.0	8 (6.5) 2.5	18 (4.4) 1.7	32 (3.4) 1.3
1180 Wood River, Hope Valley	9 (6.0) 2.7	5 (7.7) 3.5	6 (7.1) 3.2	8 (6.3) 2.9	21 (4.0) 1.8	35 (3.1) 1.4
1185 Pawcatuck River, Westerly	9 (10.3) 10.2	7 (11.4) 11.4	9 (10.3) 10.2	12 (9.0) 8.8	27 (6.1) 5.8	43 (4.8) 4.6

K-CERA Conclusions

The K-CERA analysis revealed no outstanding discrepancies in the operational policies of the stream-gaging programs in Massachusetts or Rhode Island. The differences between the currrent operations budgets and the budgets for improved operations at the same levels of error are 1.7 percent in Massachusetts and 2.5 percent in Rhode Island. These differences are well within the limitations of estimating fixed costs.

Any decision to change current operational policy should take into consideration the improvements that can be made to both standard error and EGS, by making more visits to particular stations. EGS is strongly influenced by the stability of the stage-discharge relation; a lower percentage indicates a more stable relation. The Concord River station (0995) has had the same rating since 1975; only two or three measurements are necessary each year to confirm the rating. It has two independent recorders, lost record is slight, and ice effect is usually readily apparent. Under the current budget, this station has a standard error of 4.2 percent and an EGS of 3.2 percent, for eight visits per year. Traveling Hydrographer shows that, at twice the present budget, standard error could be reduced to 2.6 percent and EGS to 2.2 percent for 39 visits per year. By contrast, Jones River (1058.70), for 11 visits per year, has a standard error of 12.5 percent and an EGS of 9.9 percent. At this station, the rating changes constantly, there is no backup record, and comparisons with other stations are poor. Traveling Hydrographer shows that, at twice the present budget, standard error could be reduced to 5.1 percent and EGS to 3.8 percent for 69 visits per year. If additional financial resources were available and a need for greater accuracy were identified, the greater proportion of those resources would be directed to improving the quality of record at stations such as Jones River.

SUMMARY

The data-use survey showed that the 15 stations in the Rhode Island network should be kept in operation in the foreseeable future. In Massachusetts, two special-purpose stations will be discontinued at the end of the data-collection phases, one long-term station could probably be discontinued at the conclusion of a study in the basin, and one station on Cape Cod would provide more useful data if it were relocated to another stream less influenced by regulation or evaporation from ponds.

Simulation of streamflow by either flow-routing or regression techniques was not sufficiently accurate to use these methods in lieu of operating continuous-record stream gages.

No major changes in operational policy in either State were indicated. Actual budgets are only 1.7 percent higher than the minimum possible budget in Massachusetts and only 2.5 percent higher than the minimum possible budget in Rhode Island. At minimum budgets, standard errors would increase by about half a percent in both States. If the present budget levels were doubled, a one-third reduction in the standard error could be achieved in Massachusetts, and slightly more than a 50-percent reduction could be achieved in Rhode Island. Further budget increases would not improve the standard errors significantly.

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