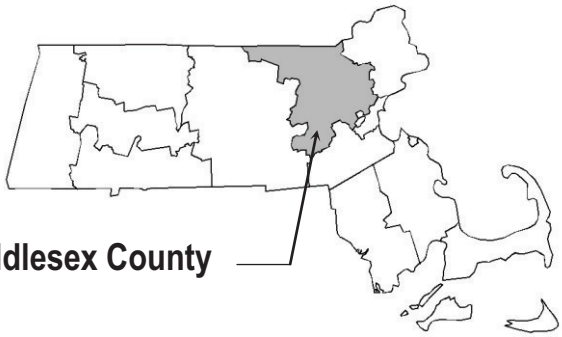
FLOOD INSURANCE STUDY

VOLUME 1 OF 8

MIDDLESEX COUNTY, MASSACHUSETTS
(ALL JURISDICTIONS)



COMMUNITY NAME	COMMUNITY NUMBER
ACTON, TOWN OF	250176
ARLINGTON, TOWN OF	250177
ASHBY, TOWN OF	250178
ASHLAND, TOWN OF	250179
AYER, TOWN OF	250180
BEDFORD, TOWN OF	255209
BELMONT, TOWN OF	250182
BILLERICA, TOWN OF	250183
BOXBOROUGH, TOWN OF	250184
BURLINGTON, TOWN OF	250185
CAMBRIDGE, CITY OF	250186
CARLISLE, TOWN OF	250187
CHELMSFORD, TOWN OF	250188
CONCORD, TOWN OF	250189
DRACUT, TOWN OF	250190
DUNSTABLE, TOWN OF	250191
EVERETT, CITY OF	250192
FRAMINGHAM, TOWN OF	250193
GROTON, TOWN OF	250194
HOLLISTON, TOWN OF	250195
HOPKINTON, TOWN OF	250196
HUDSON, TOWN OF	250197
LEXINGTON, TOWN OF	250198
LINCOLN, TOWN OF	250199
LITTLETON, TOWN OF	250200
LOWELL, CITY OF	250201
MALDEN, CITY OF	250202
MARLBOROUGH, CITY OF	250203
MAYNARD, TOWN OF	250204
MEDFORD, CITY OF	250205
MELROSE, CITY OF	250206
NATICK, TOWN OF	250207
NEWTON, CITY OF	250208
NORTH READING, TOWN OF	250209
PEPPERELL, TOWN OF	250210
READING, TOWN OF	250211
SHERBORN, TOWN OF	250212
SHIRLEY, TOWN OF	250213
SOMERVILLE, CITY OF	250214
STONEHAM, TOWN OF	250215
STOW, TOWN OF	250216
SUDBURY, TOWN OF	250217
TEWKSBURY, TOWN OF	250218
TOWNSEND, TOWN OF	250219
TYNGSBOROUGH, TOWN OF	250220
WAKEFIELD, TOWN OF	250221
WALTHAM, CITY OF	250222
WATERTOWN, TOWN OF	250223
WAYLAND, TOWN OF	250224
WESTFORD, TOWN OF	250225
WESTON, TOWN OF	250226
WILMINGTON, TOWN OF	250227
WINCHESTER, TOWN OF	250228
WOBURN, CITY OF	250229

EFFECTIVE:
JUNE 4, 2010

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
25017CV001A

NOTICE TO
FLOOD INSURANCE STUDY USERS

Communities participating in the National Flood Insurance Program have established repositories of flood hazard data for floodplain management and flood insurance purposes. This Flood Insurance Study (FIS) may not contain all data available within the repository. It is advisable to contact the community repository for any additional data.

Part or all of this FIS may be revised and republished at any time. In addition, part of this FIS may be revised by the Letter of Map Revision process, which does not involve republication or redistribution of the FIS. It is, therefore, the responsibility of the user to consult with community officials and to check the community repository to obtain the most current FIS components.

Initial Countywide FIS Effective Date: June 4, 2010

Revised Countywide FIS Date:

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FLOOD INSURANCE STUDY
MIDDLESEX COUNTY, MASSACHUSETTS (ALL JURISDICTIONS)

1.0 INTRODUCTION

1.1 Purpose of Study

This countywide Flood Insurance Study (FIS) investigates the existence and severity of flood hazards in, or revises and updates previous FISs/Flood Insurance Rate Maps (FIRMs) for the geographic area of Middlesex County, including: the Cities of Cambridge, Everett, Lowell, Malden, Marlborough, Medford, Melrose, Newton, Somerville, Waltham, and Woburn; the Towns of Acton, Arlington, Ashby, Ashland, Ayer, Bedford, Belmont, Billerica, Boxborough, Burlington, Carlisle, Chelmsford, Concord, Dracut, Dunstable, Framingham, Groton, Holliston, Hopkinton, Hudson, Lexington, Lincoln, Littleton, Maynard, Natick, North Reading, Pepperell, Reading, Sherborn, Shirley, Stoneham, Stow, Sudbury, Tewksbury, Townsend, Tyngsborough, Wakefield, Watertown, Wayland, Westford, Weston, Wilmington, and Winchester (hereinafter referred to collectively as Middlesex County).

This FIS aids in the administration of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. This FIS has developed flood risk data for various areas of the county that will be used to establish actuarial flood insurance rates. This information will also be used by Middlesex County to update existing floodplain regulations as part of the Regular Phase of the National Flood Insurance Program (NFIP), and will also be used by local and regional planners to further promote sound land use and floodplain development. Minimum floodplain management requirements for participation in the NFIP are set forth in the Code of Federal Regulations at 44 CFR, 60.3.

In some Commonwealths or communities, floodplain management criteria or regulations may exist that are more restrictive or comprehensive than the minimum Federal requirements. In such cases, the more restrictive criteria take precedence and the Commonwealth (or other jurisdictional agency) will be able to explain them.

1.2 Authority and Acknowledgments

The sources of authority for the Middlesex County countywide FIS are the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973.

Information on the authority and acknowledgments for each jurisdiction included in this countywide FIS, as compiled from their previously printed FIS reports, is shown below.

Acton, Town of: the hydrologic and hydraulic analyses in the FIS report dated January 6, 1988, were prepared by Camp Dresser & McKee, Inc., for the Federal Emergency Management Agency (FEMA), under

Contract No. EMW-84-C-1601. The work was completed in December 1985. The hydrologic and hydraulic analyses for the original FIS report were also prepared by Camp, Dresser & McKee, Inc., for FEMA. That work was completed in December 1976.

Arlington, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 5, 1982, were performed by Camp, Dresser, and McKee, Inc., for FEMA, under Contract No. H-3861. That work, which was completed in May 1978, covered all significant flooding sources affecting the Town of Arlington.

Ashland, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 16, 1981, were prepared by Howard, Needles, Tammen, and Bergendoff for the Federal Insurance Administration (FIA), under Contract No. H-4004. That work was completed in November 1979.

Ayer, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 19, 1982, were performed by Howard, Needles, Tammen, and Bergendoff for FEMA, under Contract No. H-4004. That work was completed in January 1978.

Bedford, Town of: the hydrologic and hydraulic analyses for the FIS report dated July 4, 1988, were prepared by Schoenfeld Associates, Inc., for FEMA under Contract No. EMW-C-0280. That work was completed in January 1984. The hydrologic and hydraulic analyses in the FIS report represent a revision of the original analyses prepared by the U.S. Army Corps of Engineers (USACE) for FEMA under Inter-Agency Agreement No. IAA-H-8-71. An updated version was prepared by C. E. Maguire, Inc. for FEMA under Contract No. H-4523. That work was completed in April 1979.

Belmont, Town of: the hydrologic and hydraulic analyses for the FIS report dated December 15, 1981, were prepared by C. E. Maguire, Inc., for FEMA, under Contract No. H-4523. That work, which was completed in February 1978, covered all significant flooding sources in the Town of Belmont.

Billerica, Town of: the hydrologic and hydraulic analyses for the FIS report dated February 5, 1985, were performed by

the Schoenfeld Associates, Inc. for FEMA, under Contract No. EMW-C-0280. That work was completed in July 1983. The hydrologic and hydraulic analyses for the FIS report dated May 1980 were performed by Camp, Dresser & McKee, Inc. for FEMA under Contract No. H-3861. That work was completed in January 1978.

Boxborough, Town of: the hydrologic and hydraulic analyses for the FIS report dated September 8, 1999, were prepared by the Green International Affiliates, Inc., for FEMA, under Contract No. EMB-96-CO-0403 (Task #4). That work was completed in August 1997. The hydrologic and hydraulic analyses for the FIS report dated March 1978 was prepared by Harris-Toups Associates for the FIA under Contract No. H-4024, That work was completed in April 1977.

Burlington, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 5, 1984, were prepared by Schoenfeld Associates, Inc., for FEMA, under Contract No. H-4794. That work was completed in March 1981.

Cambridge, City of: the hydrologic and hydraulic analyses for the FIS report dated January 5, 1982, were performed by C. E. Maguire, Inc., for FEMA, under Contract No. H-4523. That work, which was completed in April 1978, covered all significant flooding sources affecting the City of Cambridge. Preliminary findings for Alewife Brook (Little River) were revised using a later study done by Research Analysis, Inc.

Carlisle, Town of: the hydrologic and hydraulic analyses for the FIS report dated May 17, 1988, were prepared by the Schoenfeld Associates, Inc., for FEMA, under Contract No. EMW-C-0280. That work was completed in 1984. The hydrologic and hydraulic analyses for the FIS report dated October 15, 1980, were prepared by Harris-Toups Associates, Inc., for FEMA, under Contract No. H-4024. That work was completed in April 1978.

Chelmsford, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 16, 2004, for River Meadow Brook, were prepared by Roald Haestad, Inc., for FEMA, under Contract No. EMB-1999-CO-0564. That work was completed in October 2001. The

hydrologic and hydraulic analyses for the original December 1979 FIS and June 4, 1980, FIRM (hereinafter referred to as the 1980 FIS), were prepared by Camp, Dresser and McKee, Inc., for FEMA, under Contract No. H-3861. That work was completed in March 1978.

Concord, Town of: the hydrologic and hydraulic analyses for the FIS report dated June 3, 1988, were prepared by Schoenfeld Associates, Inc. for FEMA, under Contract No. EMW-C-0280. That work was completed in February 1984. The hydrologic and hydraulic analyses in the June 15, 1979, FIS report were prepared by Camp Dresser & McKee, Inc., for FEMA under Contract No. H-3861. That work was completed in April 1977.

Dracut, Town of: the hydrologic and hydraulic analyses for the FIS report dated June 5, 1989, were prepared by the New England Division of the USACE for FEMA, under Inter-Agency Agreement No. EMW-E-0941. This work was completed in July 1986. The hydrologic and hydraulic analyses for the FIS report dated June 2, 1980, were prepared by the New England Division of the USACE for FEMA, under Inter-Agency Agreement No. IAA-H-7-76, Project Order No. 24, and Inter-Agency Agreement No. IAA-H-10-77, Project Order No. 2. That work was completed in May 1978.

Dunstable, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 5, 1982, were prepared by Schoenfeld Associates, Inc., for FEMA, under Contract No. H-4794. That work was completed in February 1980.

Everett, City of: the hydrologic and hydraulic analyses for the FIS report dated June 3, 1986, were prepared by Camp, Dresser and McKee, Inc., for FEMA, under Contract No. EMW-84-C-1601. That work was completed in January 1985.

Framingham, Town of: For the FIS report dated March 16, 1992, Dewberry & Davis prepared updated hydraulic and hydrologic analyses. The data used in these analyses were provided by the Natural Resources Conservation Service (NRCS). The work was completed in July 1989. The hydrologic and hydraulic analyses for the original FIS report, were prepared by Howard,

Needles, Tammen and Bergendoff, for the FEMA, under Contract No. H-4004. That work was completed in November 1979. In the first revision, updated topographic data were provided by Dewberry & Davis, for FEMA, using contour maps provided by MacCarthy & Sullivan Engineering, Inc. In the second revision, updated zone designations were prepared by Dewberry & Davis for FEMA.

Groton, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 5, 1982, were prepared by Howard, Needle, Tammen and Bergendoff for FEMA, under Contract No. H-4004. That work was completed in January 1978.

Holliston, Town of: the hydrologic and hydraulic analyses for the FIS report dated March 1980 were prepared by the C-E Maguire, Inc., for FIA, under Contract No. H-4523. That work was completed in December 1978.

Hopkinton, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 5, 1982, were prepared by Howard, Needle, Tammen and Bergendoff for FEMA, under Contract No. H-4004. That work was completed in November 1979.

Hudson, Town of: the hydrologic and hydraulic analyses for the FIS report dated June 1979 were prepared by Harris-Toupes Association, for the FIA, under Contract No. H-4024. That work was completed in December 1977.

Lexington, Town of: the hydrologic and hydraulic analyses for the FIS report dated December 1977 were performed by Harris-Toups Associates for the FIA, under Contract No. H-4024. That work, which was completed in April 1977, covered all significant flooding sources affecting the Town of Lexington.

Lincoln, Town of: the hydraulic analyses for the FIS report dated June 17, 1986, were performed by Schoenfeld Associates, Inc., for FEMA, under Contract No. EMW-C-0280. They were limited to recalculating the flood profiles of Stony Brook and recalculating all floodways based upon the availability of more recent information. In addition, the floodplains and floodways of all streams studied in detail in Lincoln were delineated on topographic

maps obtained from the Town of Lincoln (American Air Surveys, Inc., 1968). This work was completed in March 1983. The hydrologic and hydraulic analyses for the FIS report dated December 1977 were prepared by Harris-Toups Associates for FEMA, under Contract No. H-4024. That work was completed in May 1977.

Littleton, Town of: the hydrologic and hydraulic analyses for the FIS report dated December 15, 1982, were prepared by Schoenfeld Associates Inc., for FEMA, under Contract No. H-4794. That work was completed in October 1980.

Lowell, City of: The hydraulic analyses for the FIS report dated September 30, 1992, were prepared by Roald Haestad, Inc., for FEMA, under Contract No. EMW-90-C-3126. That work was completed in March 1991. The original analyses were prepared by the USACE, New England Division, for FEMA, under Inter-Agency Agreement No. H-2-73, Project Order No. 1. In the first revision, the hydrologic and hydraulic analyses were prepared by the USACE for FEMA, under Inter-Agency Agreement No. EMW-E-0941. That work was completed in August 1986.

Malden, City of: The topographic information for Town Line Brook and Linden Brook for the FIS report dated August 20, 2002, was prepared by Roald Haestad, Inc., for FEMA, under Contract No. EMB-1999-CO-0564, Modification No. 5. That work was completed in November 2000. The hydrologic and hydraulic analyses for the original FIS report dated May 19, 1987, were prepared by Camp, Dresser and McKee, Inc., for FEMA, under Contract No. EMW-84-C-1601. That work was completed in October 1985.

Marlborough, City of: the hydrologic and hydraulic analyses for the FIS report dated July 6, 1981, were prepared by Howard, Needles, Tammen and Bergendoff for FEMA, under Contract No. H-4004. That work was completed in November 1979.

Maynard, Town of: the hydrologic and hydraulic analyses for the FIS report dated December 1978 were prepared by Harris-Toups Associates for FEMA, under Contract

No. H-4024. That work was completed in July 1977.

Medford, City of: the hydrologic and hydraulic analyses for the FIS report dated June 3, 1986, were prepared by Camp, Dresser & McKee, Inc., for FEMA, under Contract No. EMW-84-C-1601. That work was completed in January 1985.

Melrose, City of: the hydrologic and hydraulic analyses for the FIS report dated August 5, 1986, were prepared by Camp Dresser & McKee Inc., for FEMA, under Contract No. EMW-84-C-1601. That work was completed in March 1985.

Natick, Town of: the hydrologic and hydraulic analyses for the FIS report dated August 1979 were prepared by Harris-Toups Associates, for the FIA, under Contract No. H-4024. That work was completed in May 1978.

Newton, City of: the hydrologic and hydraulic analyses for the FIS report dated June 4, 1990, represent a revision of the original analyses by the USACE for FEMA, under Inter-Agency Agreement No. IAA-H-2-72, Project Order No. 4. The updated version was prepared by Schoenfeld Associates, Inc., for FEMA under Contract No. H-4794. This work was completed in November 1981. The FIS report was revised on July 17, 1986, for FEMA, to adjust the profiles for the Charles River. A further revision was completed in July 1988 by Dewberry & Davis, for FEMA, to reflect more accurate culvert data on South Meadow Brook.

North Reading, Town of: For the June 16, 2004, revision, the hydrologic and hydraulic analyses for Martins Brook, Martins Pond, and Skug River were prepared by Green International Affiliates, Inc., for the Town of North Reading under FEMA's Cooperating Technical Communities Program, Agreement No. EMB-2000-CA-0594. That work was completed in November 2001. The hydrologic and hydraulic analyses for Bear Meadow Brook were taken from the FIS for the Town of Reading. For the April 3, 1989, FIS, the hydrologic and hydraulic analyses for an updated study of a portion of Martins Brook were prepared by Dewberry & Davis LLC, under agreement with FEMA. That work was completed in January 1987. For the July 6, 1982, FIS, and the

January 6, 1983, FIRM (hereinafter referred to as the 1983 FIS), the hydrologic and hydraulic analyses were prepared by the New England Division of the USACE for FEMA under Inter-Agency Agreement No. IAA-H-10-77, Project Order No. 29. That work was completed in May 1979.

Pepperell, Town of: For the FIS report dated June 2, 1993, the hydrologic and hydraulic analyses were prepared by Green International Affiliates, Inc., under Contract No. EMW-89-C-2820, for FEMA. This work was completed in December 1989. The hydrologic and hydraulic analyses in the FIS report dated July 2, 1981, were prepared by Howard, Needles, Tammen and Bergendoff for FEMA, under Contract No. H-4004. Raytheon Autometrics, under subcontract to the study contractor, provided supplemental topographic mapping for areas along the Nashua River, the Nissitissit River, and Reedy Meadow Brook. Schofield Brothers, Incorporated, also under subcontract to the study contractor, provided field survey data and aerial photogrammetric mapping along portions of the Nissitissit River and Reedy Meadow Brook. The aerial photogrammetric mapping was provided by Teledyne Geotronics. That work was completed in January 1978.

Reading, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 2, 1981, were performed by Anderson-Nichols & Company, Inc., for the FIA, under Contract No. H-4524. That study was completed in August 1978.

Sherborn, Town of: the hydrologic and hydraulic analyses for the FIS report dated December 1979 were prepared by C. E. Maguire, Inc., for the FIA, under Contract No. H-4523. That work was completed in July 1978.

Shirley, Town of: the hydrologic and hydraulic analyses for the FIS report dated January 5, 1983, were prepared by Howard, Needles, Tammen and Bergendoff for FEMA, under Contract No. H-4004. That work was completed in January 1978.

Somerville, City of: the hydrologic and hydraulic analyses for the FIS report dated July 17, 1986, were prepared by Camp, Dresser & McKee, Inc., for FEMA, under Contract

	No. EMW-84-C-1601. That work was completed in January 1985.
Stoneham, Town of:	the hydrologic and hydraulic analyses for the FIS report dated July 3, 1986, were prepared by Camp Dresser & McKee, Inc., for FEMA, under Contract No. EMW-84-C-1601. That work was completed in March 1985.
Stow, Town of:	the hydrologic and hydraulic analyses for the FIS report dated February 1979 were prepared by Harris-Toups Associates for the FIA, under Contract No. H-4024. That work was completed in December 1977.
Sudbury, Town of:	the hydrologic and hydraulic analyses for the FIS report dated November 20, 1998, were prepared by Green International Affiliates, Inc., for FEMA, under Contract No. EMW-94-C-4406. That work was completed in February 1996. The hydrologic and hydraulic analyses for the FIS report dated December 1, 1981, were prepared by Howard, Needles, Tammen and Bergendoff for FEMA, under Contract No. H-4004. That work was completed in November 1979.
Tewksbury, Town of:	the hydrologic and hydraulic analyses for the FIS report dated January 2, 1981, represent a revision of the original analyses by Anderson-Nichols and Company, for the FIA under Contract No. H-3707. That work was completed in December 1978.
Townsend, Town of:	the hydrologic and hydraulic analyses for the FIS report dated February 2, 1982, were prepared by Howard, Needles, Tammen and Bergendoff for FEMA, under Contract No. H-4004. That work was completed in January 1978.
Tyngsborough, Town of:	the hydrologic and hydraulic analyses for the FIS report dated March 2, 1982, were prepared by Cullinan Engineering Co., Inc., for FEMA, under Contract No. H-4797. That work was completed in January 1981.
Wakefield, Town of:	the hydrologic and hydraulic analyses for the FIS report dated April 1978 were prepared by Camp Dresser & McKee, Inc., Environmental Engineers for the FIA, under Contract No. H-3861. That work was completed in December 1976.

Waltham, City of:	the hydrologic and hydraulic analyses for the FIS report dated July 5, 1984, represent a revision of the original analyses prepared by C. E. Maguire, Inc., for FEMA, under Contract No. H-4523. The original work was completed in April 1978. The updated version was completed in August 1983 from information supplied by C. E. Maguire, Inc., reflecting changes as of December 1981.
Watertown, Town of:	the hydrologic and hydraulic analyses for the FIS report dated August 1980 were prepared by C. E. Maguire, Inc., for the FIA, under Contract No. H-4523. That work was completed in August 1978.
Wayland, Town of:	the hydrologic and hydraulic analyses for the FIS report dated February 19, 1986, were prepared by Howard, Needles, Tammen and Bergendoff for FEMA, under Contract No. H-4004. That work was completed in November 1979. The revised hydrologic and hydraulic analyses for Hayward Brook were performed by Dewberry & Davis, under agreement with FEMA. That work was completed in April 1985.
Westford, Town of:	the hydrologic and hydraulic analyses for the FIS report dated December 15, 1982, were prepared by Cullinan Engineering Co., Inc., for FEMA, under Contract No. H-4797. That work was completed in January 1981.
Weston, Town of:	the hydrologic and hydraulic analyses for the FIS report dated January 1980 were prepared by C. E. Maguire, Inc., for the FIA, under Contract No. H-4523. That work was completed in May 1978.
Wilmington, Town of:	For the June 2, 1999, revision, the hydrologic and hydraulic analyses for Lubbers Brook, from Glen Road to the upstream corporate limits, were prepared by Green International Affiliates, Inc., for FEMA, under Contract No. EMB-96-CO-0403 (Task No. 3). That work was completed in April 1997. For the January 18, 1989, revision, the hydrologic and hydraulic analyses were prepared for Martins Brook and Tributary to Martins Brook by Dewberry & Davis, under contract to FEMA. The work for the updated study was completed in November 1986. For the original June 15, 1982, FIS, the hydrologic and hydraulic analyses were

prepared by Anderson-Nichols & Company, Inc., for FEMA, under Contract No. H-4524; approximate flood boundaries were prepared by Michael Baker, Jr., Inc. That work was completed in August 1978.

Winchester, Town of: the hydrologic and hydraulic analyses for the FIS report dated December 1979 were performed by Anderson-Nichols & Company, Inc., for the FIA, under Contract No. H-4524. That study, which was completed in July 1978, covered all significant flooding sources in the Town of Winchester.

Woburn, City of: the hydrologic and hydraulic analyses for the FIS report dated January 1980 were performed by Anderson-Nichols & Company, Inc., for the FIA, under Contract No. H-4524. That study was completed in July 1978.

The authority and acknowledgments for the Town of Ashby is not available because no FIS report was ever published for this community.

For this countywide FIS, hydrologic and hydraulic analyses were prepared by ENSR, under subcontract to CR Environmental, Green International Affiliates, under Contract No. EMB-2001-CO-0670. Revised hydrologic and hydraulic analyses were prepared and completed in June 2005. The following streams were restudied: Aberjona River, Aberjona River North Spur, Alewife Brook (Little River), Cummings Brook, Halls Brook, Horn Pond Brook/Fowle Brook, Little Brook, Mill Brook 3, Mystic River, Schneider Brook, Shakers Glen Brook, Sweetwater Brook, and Wellington Brook.

Floodplain boundaries were delineated using the Office of Geographic and Environmental Information (MassGIS), Commonwealth of Massachusetts, Executive Office of Energy and Environmental Affairs, 2002 LiDAR topography for the study area, at a scale of 1:5,000, suitable for 2-foot contour generation.

The digital base map information was provided by MassGIS. This information was derived from digital orthophotos produced at a scale of 1:5,000 from aerial photography dated April 2005.

1.3 Coordination

Consultation Coordination Officer's (CCO) meetings may be held for each jurisdiction in this countywide FIS. An initial CCO meeting is held typically with representatives of FEMA, the community, and the study contractor to explain the nature and purpose of a FIS, and to identify the streams to be studied by detailed methods. A final CCO meeting is held typically with representatives of FEMA, the community, and the study contractor to review the results of the study.

Prior to this countywide FIS, the dates of the initial and final CCO meetings held for all jurisdictions within Middlesex County from the historic FIS reports are shown in Table 1, "Initial and Final CCO Meetings."

TABLE 1 - INITIAL AND FINAL CCO MEETINGS

Community	Initial CCO Date	Final CCO Date
Acton, Town of	April, 1984	August 19, 1986
Arlington, Town of	August 21, 1975	June 11, 1981
Ashby, Town of	Not available	Not available
Ashland, Town of	April 14, 1976	August 21, 1980
Ayer, Town of	April 21, 1976	April 16, 1981
Bedford, Town of	August 30, 1979	December 16, 1986
Belmont, Town of	May 3, 1977	July 27, 1981
Billerica, Town of	August 27, 1979	July 26, 1984
Boxborough, Town of	September 12, 1996	September 8, 1999
Burlington, Town of	May, 1978	April 5, 1982
Cambridge, City of	May 3, 1977	February 25, 1981
Carlisle, Town of	August 23, 1979	December 16, 1986
Chelmsford, Town of	November 29, 2000	August 19, 2002
Concord, Town of	August 30, 1979	December 17, 1986
Dracut, Town of	August 3, 1983	May 4, 1988
Dunstable, Town of	May, 1978	March 30, 1981
Everett, City, of	April, 1984	July 11, 1985
Framingham, Town of	April 9, 1976	March 5, 1981
Groton, Town of	April 12, 1976	May 26, 1981
Holliston, Town of	May 24, 1977	July 24, 1979
Hopkinton, Town of	April 15, 1976	November 4, 1981
Hudson, Town of	April 29, 1976	September 12, 1978
Lexington, Town of	April 13, 1976	June 23, 1977
Lincoln, Town of	August 30, 1979	April 24, 1984
Littleton, Town of	April, 1978	January 26, 1982
Lowell, City of	November 13, 1990	Not available
Malden, City of	February 23, 2000	Not available
Marlborough, City of	April 15, 1976	September 15, 1980
Maynard, Town of	April 29, 1976	March 22, 1978
Medford, City of	April, 1984	July 11, 1985
Melrose, City of	April, 1984	September 12, 1985
Natick, Town of	April 9, 1976	November 8, 1978
Newton, City of	May, 1978	December 9, 1982
North Reading, Town of	Not available	March 31, 2003
Pepperell, Town of	April 30, 1976	November 7, 1991
Reading, Town of	May, 1977	May 30, 1979
Sherborn, Town of	May 24, 1977	March 13, 1979
Shirley, Town of	October 18, 1976	November 4, 1981
Somerville, City of	April, 1984	August 29, 1985
Stoneham, Town of	April 4, 1984	August 19, 1985

TABLE 1 - INITIAL AND FINAL CCO MEETINGS - continued

Community	Initial CCO Date	Final CCO Date
Stow, Town of	May 12, 1976	August 8, 1978
Sudbury, Town of	August 4, 1993	December 10, 1997
Tewksbury, Town of	January 6, 1975	August 28, 1979
Townsend, Town of	April 23, 1976	September 22, 1981
Tyngsborough, Town of	May 19, 1978	August 10, 1981
Wakefield, Town of	August 28, 1975	June 7, 1977
Waltham, City of	May 5, 1977	October 25, 1978
Watertown, Town of	May 4, 1977	March 19, 1979
Wayland, Town of	April 14, 1976	January 26, 1981
Westford, Town of	May 19, 1978	February 8, 1982
Weston, Town of	May 17, 1977	March 20, 1979
Wilmington, Town of	September 16, 1996	April 15, 1998
Winchester, Town of	May 19, 1977	April 25, 1979
Woburn, City of	May 10, 1977	May 16, 1979

For this countywide FIS, which includes a restudy of the Mystic River basin, an initial CCO meeting was held on November 1, 2001, and was attended by representatives of FEMA Region I, ENSR International, Dewberry, Green International Affiliates, and the Massachusetts NFIP Coordinator.

Further, all communities in Middlesex County were notified by FEMA in a letter dated February 5, 2003, that FEMA will be preparing a FIS and FIRM for Middlesex County, Massachusetts (All Jurisdictions). The letter stated that the effective FIRMs and Flood Hazard Boundary Maps (FHBMs) of these communities will be digitally converted to a format that conforms to FEMA's Digital FIRM specifications.

For this countywide FIS, final CCO meetings were held on November 5, 7, and 8, 2007, and were attended by representatives of FEMA, Dewberry, ENSR, the Commonwealth, and various communities.

2.0 AREA STUDIED

2.1 Scope of Study

This FIS covers the geographic area of Middlesex County, Massachusetts.

As part of this countywide FIS, which includes a restudy of the Mystic River basin, updated analyses were included for the flooding sources shown in Table 2, "Scope of Revision."

TABLE 2– SCOPE OF REVISION

Stream	Limits of Revised or New Detailed Study
Aberjona River	From its confluence with Mystic River to the confluence of the Aberjona River North Spur
Aberjona River North Spur	From its confluence with Aberjona River to a point approximately 275 feet upstream of Willow Street
Alewife Brook (Little River)	From its confluence with Mystic River to the confluence of Wellington Brook
Cummings Brook	From its confluence with Shakers Glen Brook to a point approximately 130 feet upstream of Winn Street
Halls Brook	From its confluence with Aberjona River to a point approximately 220 feet upstream of Merrimac Street
Horn Pond Brook/Fowle Brook	From its confluence with Aberjona River to the confluence of Shakers Glen Brook
Little Brook	From its confluence with Cummings Brook to a point approximately 400 feet upstream of Bedford Road
Mill Brook 3	From its confluence with Lower Mystic Lake to a point approximately 40 feet upstream of Boston and Maine Railroad
Mystic River	From Amelia Earhart Dam to the outlet of Lower Mystic Lake
Schneider Brook	From its confluence with Aberjona River to a point approximately 800 feet upstream of Forbes Street
Shakers Glen Brook	From its confluence with Fowle Brook to a point approximately 190 feet upstream of Russell Street
Sweetwater Brook	From its confluence with Aberjona River to a point approximately 120 feet upstream of Lindenwood Road
Wellington Brook	From its confluence with Alewife Brook (Little River) to a point approximately 710 feet upstream of Library Private Drive

This FIS also incorporates the determinations of letters issued by FEMA resulting in map changes (Letter of Map Revision [LOMR], Letter of Map Revision - based on Fill [LOMR-F], and Letter of Map Amendment [LOMA], as shown in Table 3, "Letters of Map Change."

TABLE 3 - LETTERS OF MAP CHANGE

Community	Flooding Source(s)/Project Identifier	Effective Date	Type
Bedford, Town of	Concord River – Bedford Meadows Subdivision	May 20, 1996	LOMR
Bedford, Town of	Elm Brook – 135 South Road	March 31, 2009	LOMR
Billerica, Town of	Content Brook – Whipple Road Culvert	August 15, 1999	LOMR
Concord, Town of	Mill Brook 3 – Keyes Road Weir and Footbridge (2nd Submittal)	November 22, 2002	LOMR
Dunstable, Town of	Nashua River- N.R.L.C & Patenaude Gravel Pit	November 8, 1999	LOMR
Framingham, Town of	Baiting Brook – Belport Farms Subdivision	November 11, 1994	LOMR
Lowell, City of	Trull Brook Tributary	October 25, 1998	LOMR
Lowell, City of	River Meadow Brook, Former Wang Towers	January 8, 1996	LOMR
Pepperell, Town of	Nashua River (Data from Nashua, NH LMMP)	July 12, 2001	LOMR
Stow, Town of	Analysis of Zone A area for Elizabeth Brook – Land Realty Trust	November 17, 1989	LOMR
Tewksbury, Town of	Trull Brook Tributary	October 25, 1998	LOMR
Westford, Town of	Wyman's Beach	November 14, 2005	LOMR
Wilmington, Town of	Martins Brook	March 16, 2004	LOMR

The areas studied by detailed methods were selected with priority given to all known flood hazard areas and areas of projected development and proposed construction. All or portions of the flooding sources listed in Table 4, "Flooding Sources Studied by Detailed Methods," were studied by detailed methods. Limits of

detailed study are indicated on the Flood Profiles (Exhibit 1) and on the FIRM (Exhibit 2).

TABLE 4 - FLOODING SOURCES STUDIED BY DETAILED METHODS

Aberjona River
Aberjona River North Spur
Alewife Brook (Little River)
Angelica Brook
Assabet Branch No. 3
Assabet Branch No. 4
Assabet River
Atlantic Ocean
Baddacook Brook
Baiting Brook
Bear Meadow Brook
Beaver Brook 1
Beaver Brook 2
Beaver Brook 3
Beaver Brook 4
Beaver Brook 5
Beaverdam Brook
Bennetts Brook
Birch Meadow Brook
Black Brook
Bogastow Brook / Jar Brook
Bogle Brook 1
Bogle Brook 2
Boons Pond and Branch
Boutwell Brook
Bow Brook
Branch of Assabet River
Branch of Elizabeth Brook 1
Broad Meadow Brook
Brook A of Shawsheen River
Brook from Waushakum Pond
Butter Brook
Catacoonamug Brook
Charles River
Cheese Cake Brook
Cherry Brook
Chester Brook
Chicken Brook
Cochituate Brook
Cold Brook
Cold Spring Brook
Cole's Brook
Collins Brook
Conant Brook
Concord River
Content Brook - Middlesex Canal
Course Brook
Cow Pond Brook
Cummings Brook
Dakins Brook
Danforth Brook
Darby Brook
Davis Brook
Dirty Meadow Brook
Dopping Brook
Dudley Brook/Tributary A to Dudley Brook
East Outlet
Elizabeth Brook 1
Elizabeth Brook 2
Ell Pond
Elm Brook
Farrar Pond / Pole Brook
Farrar Pond Brook
Fort Meadow Brook
Fort Pond Brook
Fort Pond Brook Branch 1
Fort Pond Brook Branch 2
Grassy Pond Brook
Graves Pond Brook
Great Road Tributary
Greens Brook
Guggins Brook
Gumpas Pond Brook
Hales Brook
Halls Brook
Hayward Brook
Heath Brook
Hobbs Brook 1
Hobbs Brook 2
Hog Brook
Hop Brook
Horn Pond Brook / Fowle Brook
Indian Brook
Ipswich River
James Brook
Jones Brook
Kiln Brook
King Street Tributary
Landham - Allowance Brook
Lake Quannapowitt
Lawrence Brook
Linden Brook
Little Brook
Locke Brook
Lower Spot Pond Brook
Lower Mystic Lake
Lubbers Brook
Malden River
Maple Meadow Brook
Marginal Brook
Marshall Brook
Martins Brook
Martins Pond Brook
Mascuppic Brook
Massapoag Pond
Mason Brook
Meadow Brook
Meadow River Branch
Merrimack River
Mill Brook 1
Mill Brook 2
Mill Brook 3
Mill Pond Tributary
Mill River
Mineway Brook
Mongo Brook
Morse Brook
Mowry Brook
Mud Pond Brook

TABLE 4 - FLOODING SOURCES STUDIED BY DETAILED METHODS - continued

Mulpus Brook
Munroe Brook
Mystic River
Nagog Brook
Nagog Pond
Nashoba Brook
Nashua River
Nissitissit River
Nonacoicus Brook 1
Nonacoicus Brook 2
North Lexington Brook
Pages Brook
Pages Brook Branch
Pantry Brook
Pearl Hill Brook
Peppermint Brook
Pine Brook
Pratts Brook
Putnam Brook
Reedy Meadow Brook
Reservoir No. 1 – North Branch and Reservoir No. 3
Richardson Brook
River Meadow Brook
Run Brook
Salmon Brook
Sandy Brook
Saugus River
Saunders Brook
Sawmill Brook 1
Sawmill Brook 2
Schneider Brook
Shakers Glen Brook
Shawsheen River
Skug River
Snake Brook
South Meadow Brook/ Paul Brook
Spencer Brook 1
Spencer Brook 2
Spring Brook
Squannacook River
Stony Brook 1
Stony Brook 2
Strong Water Brook
Sudbury River
Sutton Brook
Sweetwater Brook
Tadmuck Brook
Tadmuck Swamp Brook
Taylor Brook
Town Line Brook
Tributary 1 to Cole's Brook
Tributary 1 to Sudbury River
Tributary 2 to Assabet River
Tributary 2 to Tributary 1 to Cole's Brook
Tributary 3 to Bogle Brook 2
Tributary 4 to Bogle Brook 2
Tributary A to Cold Brook
Tributary A to Course Brook
Tributary A to Hop Brook
Tributary A to Pantry Brook
Tributary A to Squannacook River
Tributary B to Hop Brook
Tributary B to Squannacook River
Tributary B to Vine Brook
Tributary C to Hop Brook
Tributary C to Vine Brook
Tributary D to Hop Brook
Tributary to Beaver Brook 3
Tributary to Cold Spring Brook
Tributary to Indian Brook
Tributary to Martins Brook
Tributary to Mill Brook
Tributary to Nonacoicus Brook 1/Long Pond Brook
Tributary to Waushakum Pond
Trout Brook 1
Trout Brook 2
Trull Brook
Trull Brook Tributary
Unkety Brook
Upper Mystic Lake
Valley Brook
Varnum Brook
Vine Brook
Walker Brook 1
Walker Brook 2
Walker Brook 3
Walkers Brook
Waushakum Pond
Wellington Brook
West Chester Brook
Whitehall Brook
Willard Brook
Winthrop Canal
Witch Brook

All or portions of numerous flooding sources in the county were studied by approximate methods. Approximate analyses were used to study those areas having a low development potential or minimal flood hazards. The scope and methods of the study were proposed to, and agreed upon, by FEMA and Middlesex County.

2.2 Community Description

Middlesex County is located in eastern Massachusetts. In Middlesex County, there are 54 communities. The Towns of Ashby, Ayer, Groton, Pepperell, Shirley, and Townsend are located in the northwestern section of the county. The Towns of Carlisle, Chelmsford, Dunstable, Tyngsborough, and Westford are located in the northern section of the county. In the northeastern part of the county, lie the City of Lowell and the Towns of Billerica, Burlington, Dracut, North Reading, Tewksbury, and Wilmington. In the eastern part of the county are the City of Boston suburbs including the Cities of Cambridge, Everett, Malden, Medford, Melrose, Somerville, and Woburn, and the Towns of Arlington, Belmont, Reading, Stoneham, Wakefield, and Winchester. The City of Boston suburbs also spill into the southeastern part of the county to include the Cities of Newton and Waltham and the Towns of Watertown and Weston. In the central part of Middlesex County lie the Towns of Bedford, Concord, Lexington, and Lincoln. In the far southern portion of the county are the Towns of Ashland, Framingham, Holliston, Hopkinton, Natick, and Sherborn. Southwestern Middlesex County contains the City of Marlborough and the Towns of Hudson, Maynard, Sudbury, and Wayland. The Towns of Acton, Boxborough, Littleton, and Stow are located in the western part of Middlesex County.

Middlesex County is bordered to the north by communities of Hillsboro County, New Hampshire: the Cities of Nashua and Manchester and the Towns of Antrim, East Merrimack, Hillsborough, Milford, and Peterborough. To the east, the county is bordered by communities of Essex County: the Cities of Lawrence and Peabody and the Towns of Andover, Lynnfield, Methuen, Middleton, and Saugus. It is bordered to the southeast by the City of Boston located in Suffolk County. Middlesex County is bordered to the south by communities of Norfolk County: the Towns of Dover, Medfield, Medway, Millis, Needham, and Wellesley. To the west, the county is bordered by the communities of Worcester County: the Cities of Fitchburg and Leominster and the Towns of Ashburnham, Berlin, Bolton, Harvard, Lancaster, Lunenburg, Milford, Northborough, Southborough, Upton and Westborough.

According to the U.S. Census Bureau, the estimated population of Middlesex County was 1,465,396 in 2000 (U.S. Census Bureau, 2006).

The topography of the county is flat coastal plains to the east with elevations near 10 feet in parts of Cambridge, gently rolling hills to the south and center of the county with elevations ranging from 200 feet to 800 feet, and more hilly terrains to the west and northwest with elevations from 800 feet to over 1,400 feet in Ashby and Townsend. Soils are generally made up of sediment in lower elevations and are quite rocky in the western and northwestern part of the county. The development in Middlesex County is primarily residential and commercial.

The climate of the county can be classified as modified continental courtesy of the Atlantic Ocean. The average high temperature in January is near 36 degrees Fahrenheit (°F) with average January lows near 21°F. Average July high temperatures are near 83°F with average low temperatures in July near 65°F. Low temperatures during winter infrequently drop below 0°F and high temperatures rarely rise above 100°F in summer (National Weather Service, Boston, 2006).

Average annual precipitation for Middlesex County ranges from 42 inches in the east to near 50 inches in the higher hills of the northwest (Oregon State University, 2006).

2.3 Principal Flood Problems

Historically, excessive rainfall along, or in combination with, snowmelt runoff have produced flooding in low-lying areas of Middlesex County. Severe flooding occurred during August 1955. The flood of August 1955 resulted from two hurricanes that arrived almost concurrently—Hurricane Connie, occurring between August 11 and 15; and Hurricane Diane occurring between August 17 and 20. As a result of these two storms, roads and bridges were overtopped, and residences and businesses were flooded. Further, significant recorded floods were those occurring in May 1850, December 1878, July 1891, July 1897, February and March 1900, November 1927, March 1936, July and September 1938, October 1942, October 1955, April 1960, March 1968, and January 1979.

Flooding in Middlesex County may be caused by a number of factors: inadequate and deteriorated river channels, constricting culverts and bridges, heavy precipitation in combination with frozen ground conditions, summer and fall hurricanes, winter northeasters, inadequate storm drain discharge, increased development, topographic conditions, and undersized culverts.

Chapter 131, Section 40 (310 CMR 10.00) of the General Laws of the Commonwealth of Massachusetts (revised, April 1, 1983) is commonly referred to as the Wetlands Protection Act. The law gives the responsibility for issuing permits to remove, fill, dredge, or alter wetlands to the local conservation commission. The commission has to determine if an area on which a permit is requested "is significant to public or private water supply, to *flood control,* to storm damage prevention, to prevention of pollution, to protection of land containing shellfish, or to the protection of fisheries." *After* a public *hearing,* the commission can impose such conditions as will contribute to the protection of these interests. The Department of Environmental Quality Engineering (DEQE) may also make a determination after a review of the commission's order. Conditions imposed by the DEQE supersede conditions imposed by the commission. Detailed rules and regulations concerning the administration of this act have been promulgated by the DEQE.

Section 40 now requires a conservation commission, if requested, to make a determination of whether a particular parcel of land is a wetland and governed by the Wetlands Protection Act. It also contains definitions of terms to aid this determination.

Chapter 131, Section 40A of the Acts of 1968, as amended by Chapter 782 of the Acts of 1972, gives the commissioner of the Department of Environmental Management the authority to protect inland wetlands and floodplains by establishing encroachment lines "for the purpose of preserving and promoting the public safety, private property, wildlife, fisheries, water resources, floodplain areas, and agriculture." The commissioner may adopt orders regulating, restricting, or prohibiting the altering or polluting of inland wetlands by

designating lines with which no obstruction or encroachment would be permitted without prior approval. These restrictions require notifications to each land owner affected, public hearings, and approval by the town. Section 40A was further amended by Chapter 818 by defining "inland wetlands" to include the definition of "freshwater wetlands" as set forth in Section 40 as "that portion of any bank that touches any inland waters or any freshwater wetland, and any freshwater wetland subject to flooding."

The Damondale Dam in West Concord and the old High Street Dam (Powder Mill Dam) in Acton affect flood elevations on the Assabet River. Warners Pond Dam in West Concord affects flooding in Warners Pond and the lower portions of Fort Pond Brook and Nashoba Brook. The Merriam, Cement, and Erikson Dams affect flooding on Fort Pond Brook. The Concord Road and Wheeler Lane Dams affect flooding on Nashoba Brook. The Amelia Earhart Dam in Arlington affects flood elevations on the Mystic River, Lower Mystic Lake, and Alewife Brook (Little River). The Cooke's Hollow Dam affects flooding on Mill Brook 3. The Charles River Dam at Warren Avenue in Boston controls the level of the Charles River within the City of Cambridge. This dam was designed to maintain the basin at a level of 4.35 feet during the 1-percent annual chance flood. It is estimated that damage to properties along the basin will not occur until the basin level reaches an elevation of 4.6 feet. With water being pumped at a rate of 8,400 cubic feet per second at the dam, a basin level of 4.6 feet can be expected to be exceeded approximately once in 175 years. The Talbot Mills Dam in Billerica affects flooding on the Concord and Sudbury Rivers. The Newton Lower Falls Dam, the Cordingly Dam, and the Metropolitan Dam affect flood elevations on the lower Charles River. The Cochrane Dam, which is located in Needham and Dover, affects flood elevations on the upper Charles River. The significance of the combination of the upstream control structures and natural valley storage can be explained when analyzing the March 1968 flood. Runoff within the lower basin crested at the old Charles River Dam within hours. The upper basin peak flow took four days to reach the dam. All storm runoff was drained from the watershed in about a month's time.

The following tabulation, taken from a USACE Flood Plain Information report, presents the relative flood heights at the Carlisle Road bridge (State Route 225) from Bedford to Carlisle for the 10 major floods in the Concord River basin in order of magnitude.

Date of Crest*	Estimated Elevation	Peak Discharge at Lowell** (cfs)
August 23, 1955	119.4	4,540
January 26, 1979	119.3	5,400
March 20, 1936	119.2	6,000
March 27, 1968	118.7	4,900
July 29, 1938	118.1	3,790
September 15, 1954	117.5	3,340
September 24, 1938	117.3	3,210
March 24, 1948	117.3	3,200

Date of Crest*	Estimated Elevation	Peak Discharge at Lowell** (cfs)
January 30, 1958	117.2	3,120
April 18, 1956	117.0	2,970

* The Carlisle Road bridge is located one mile upstream of the Bedford/ Billerica/ Carlisle corporate limits.
**The Lowell gage is located 9.15 miles downstream of the Bedford/Billerica/ Carlisle corporate limits.

Velocities of water during a 1-percent annual chance flood on the Concord River would be approximately 2.0 feet per second (fps) in the main channel and approximately 0.4 fps over the floodplain. For the Shawsheen River, velocities would be somewhat greater than 4 fps in the main channel and approximately 0.5 fps over the bank. During the 1936 and 1955 floods, it was estimated that velocities in the channel of the Concord River ranged up to 1.9 fps. Overbank velocities ranged up to 0.3 fps. These flood velocities are not considered hazardous.

The duration of flooding for most of the Concord River is generally sustained due to the large drainage area, shallow channel slopes, and wide meadow flood-storage areas. Records indicate that the 1936 flood remained higher than an elevation of 117.2 feet North American Vertical Datum (NAVD) of 1988 at the Carlisle Road bridge for more than 11 days. Hurricane Diane occurred on August 19 and 20, 1955, but the Concord River did not crest until late on August 22 with water levels remaining above an elevation of 117.2 feet NAVD for over 3 days. The Shawsheen River, on the other hand, rises fairly rapidly and crests within 36 to 48 hours after the time of maximum precipitation over the watershed.

Flooding along the coastline of the Town of Everett (downstream of Amelia Earhart Dam) is greatly influenced by storm surge elevations from Boston Harbor. The flood of record occurred in February 1978. A flood elevation of 10.25 feet was recorded at the U.S.S. Constitution in the nearby Charlestown section of Boston.

The flooding history of the Merrimack River includes information of floods dating back to 1785, although little factual information on these early floods exists. The dates and peak discharges of the five largest floods recorded at the USGS gage on the Merrimack River, below the mouth of the Concord River (Gage No. 01100000), are shown in the following tabulation:

Date	Peak Discharge (cfs)
March 20, 1936	173,000
September 23, 1938	121,000
April 23, 1852	108,000*
April 7, 1987	84,700
April 6, 1960	79,000**
November 5, 1927	76,800*

* Based on data furnished by Proprietors of Locks and Canals
**Modified by Franklin Falls, and the Blackwater and MacDowell Reservoirs.

The USGS has maintained continuous discharge records on the Concord River in Lowell since 1937. Based on the flow records of this gage, the greatest flood was recorded in March 1968 with a peak flow of 4,800 cfs. Flood records indicate that prior to the establishment of the gage, a water-surface elevation of approximately 77.2 feet NAVD was experienced in March 1936 in the vicinity of the gage site.

In March 1968, 5 inches of rain coupled with melting snow produced flood conditions in Natick. This storm was comparable to a 50-year event, according to the USGS gage record on the Charles River at Charles River Village (No. 01103500).

High-water marks gathered after the 1936 and 1968 floods on the Ipswich River are as follows:

Location	1936 Flood Height Elevation (feet NAVD)	1968 Flood Height Elevation (feet NAVD)
Main Street (upstream side)	69.5	70.1
200 feet downstream of Mill Street	N/A	70.6
Mill Street (upstream side)	N/A	70.7
State Route 93 (downstream side in Wilmington)	N/A	74.1

On the Aberjona River, high water marks gathered after the March 1936 flood are presented below:

Location	Elevation (feet NAVD)
Willow Street and Summer Avenue culvert	81.0
Boston & Maine Railroad	85.8
Lowell and Intervale Streets	88.4

The Massachusetts Geodetic Survey gathered high water mark data for the 1936 flood on the Merrimack River. Representative crest elevations for the Merrimack River are shown in the following tabulation:

Location	Elevation (feet NAVD)
At bridge to Tyngs Island	111.0
At country club at north end of Tyngs Island	111.3
State Route 113 at Butterfield Road	112.8
At Tyngsborough Bridge east abutment	114.0
At Tyngsborough Bridge west abutment	114.7
At Tyngsborough- Nashua, New Hampshire corporate limits	118.1

Two features, one man-made and one natural, affect flooding in Sudbury along the Sudbury River. The man-made feature is the Talbot Mill Dam on the Concord River in Billerica which creates a backwater effect from Billerica to Framingham. The crest of the Talbot Mill Dam is approximately 108.2 feet NAVD, while the bottom of the Sudbury River at the downstream corporate limits is approximately 102.2 feet NAVD. The natural feature is the low undefined banks of the Sudbury River which are adjacent to bordering vegetated wetlands. These two features make parts of Sudbury (as well as parts of Wayland, Concord, and Lincoln) a natural detention area for floodwaters.

2.4 Flood Protection Measures

Various measures have been taken in Middlesex County for flood protection. Among them: the adoption of local floodplain zoning ordinances (which are intended to regulate construction, excavation, filling, and grading of any land situated below specified elevations); construction of dams to control flooding (for example, along the Assabet River and its major tributaries); zoning by-laws (which may, for example, allow development within the floodplain only by special permit); stormwater drainage programs; dredging of channels; replacement of inadequate culverts; preserving natural runoff and flow patterns of streams and floodwater storage areas; wetland identification; flood retention structures; formation of Floodplain Conservancy Districts; The Flood Control Acts of 1936 and 1938; flood protection dikes and walls; holding pond storage; natural storage that exists in the many swamps and ponds; and establishing wetlands protection districts.

Ten dams have been constructed within the Upper Assabet River basin to control flooding and provide recreation. These dams are in Berlin, Bolton, Stow, Marlborough, Westborough, Northborough, and Shrewsbury, and they were designed to reduce the peak water-surface elevations of the 1-percent annual chance flood by 2.3 feet at the Maynard USGS gaging station.

Between 1967 and 1976, the Metropolitan District Commission (MDC) and the USACE constructed the Amelia Earhart Dam on the Mystic River between Somerville and Everett. The Earhart Dam can pump 4,200 cfs of flow from the Mystic River against high tide into Boston Harbor.

Originally a MDC water-supply reservoir and presently a recreation area, the Ashland Reservoir serves to moderate the flood flows of the lower portion of Cold Spring Brook.

An extensive portion of the Concord and Sudbury Rivers floodplain is further protected from development by being designated as part of the Great Meadows National Wildlife Refuge.

A dam is located on Sawmill Brook at the end of Sawmill Road approximately 10 feet from the Burlington-Wilmington corporate limits. The dam, locally known as the Noah Clapp Dam, controls a drainage area of approximately 960 acres, is approximately 80 feet long and ranges in height from 7 to 14 feet.

There are five flood control dams located upstream in New Hampshire that are operated in conjunction with each other to reduce flooding on the Merrimack River and its upstream tributaries. Flood discharges along the Merrimack River have been significantly reduced as a result of these projects. These structures are the Franklin Falls Dam on the Pemigewasset River, the Edward McDowell Dam on Nubanusit Brook, the Blackwater Dam on the Blackwater River (flood control); and the Everett Dam on the Piscataquog River and the Hopkinton Dam on the Contoocook River that control Hopkinton Lake. In addition to the upstream reservoirs, the USACE has also completed five local protection projects.

There are 14 non-Federal reservoir or lake systems existing in the Merrimack River basin with usable storage in excess of 4,000 acre-feet. These reservoirs have no storage specifically allocated for flood control; however, they are drawn down during the winter months and are capable of storing significant amounts of runoff during the spring snowmelt period.

Massapoag Pond Dam, located on Salmon Brook, provides storage in Massapoag Pond during periods of heavy runoff, provided storage capacity is available.

Chapter 131, Section 40 of the General Laws of the Commonwealth of Massachusetts (amended by Chapter 65 of the Acts of 1978) is commonly referred to as the Wetlands Protection Act. The law gives the responsibility for issuing permits to remove, fill, dredge, or alter wetlands to the town conservation commissions.

The Amelia Earhart Dam was constructed by the MDC and the USACE between 1967 and 1976. It is a multi-purpose structure under the jurisdiction of the MDC. The dam, which eliminated the tidal influence upstream, can pump 4,000 cfs of flow from the Mystic and Malden Rivers against high tide into Boston Harbor.

The USACE has constructed flood protection dikes and walls along a portion of the Sudbury River and the project eliminates flooding in much of the area of Saxonville. Two levees along the Sudbury River in Framingham currently meet accreditation criteria by the USACE and are designated as provisionally accredited levees following FEMA's Revised Procedure Memorandum No. 43. As such, the 0.2-percent annual chance floodplain has been extended to the landward topographic extent of the base flood elevation for the Sudbury River. The FIRM panel includes the following note: "*WARNING: This area protected by Provisionally Accredited Levee; accreditation expires on August 12, 2009*."

The NRCS has constructed a flood control project for the Baiting Brook watershed that reduces the severity of flooding along major portions of Baiting Brook and Birch Meadow Brook. The project includes a dry dam on Baiting Brook and culvert and channel modifications to the east outlet diversion channel.

Of the five dams in Holliston, three are used to create impoundments for industrial purposes. These dams are the Linden Pond Dam, located on the Winthrop Canal; the Houghton Pond Dam, located on Jar Brook; and the Factory Pond Dam, located on Bogastow Brook. The Waseeka Dam on Chicken Brook is used to create a wildlife habitat. The Winthrop Lake Dam is used to keep a

minimum water-surface elevation in Winthrop Lake, because of the recreation area located along this lake.

In the Town of Hopkinton, the Sudbury River watershed contains large amounts of natural storage upstream of the study area. The storage available in Cedar Swamp in Westborough and Hopkinton significantly reduces flooding on the Sudbury River. Whitehall Reservoir also provides storage capacity which reduces flooding along the Sudbury River and Whitehall Brook.

In the Town of Hudson, since the occurrence of the 1955 and the 1968 floods, flood retarding structures have been installed by the NRCS and are in operation. These structures are located on the Upper Assabet River tributaries, outside of the Hudson corporate limits. The overall effect of the structures would be to lower flood peaks, making a recurrence of floods of the magnitude of the 1955 or 1968 floods less likely.

Arlington Reservoir on Munroe Brook is for the control of floods downstream on Mill Brook 3 in Arlington.

An overflow spillway is located on Mill Pond approximately 35 feet upstream of Interstate Route 495. This structure causes flood flows to backup, increasing the level of Mill Pond.

Additional existing flood control measures within Lowell are a series of dikes and walls along the north bank of Beaver Brook 3 and the Merrimack River to Bridge Street. These structures, however, would not control a 1-percent annual chance flood and currently lack accreditation by the USACE. As such, the 1-percent annual chance floodplain has been extended to the landward topographic extent of the base flood elevation from Beaver Brook 3 and the Merrimack River as if there were no dikes or walls.

In the City of Malden, channel improvements were made to Lower Spot Pond Brook from the outlet of Spot Pond Brook Branch and Ell Pond Brook Branch near Wyoming Station in Melrose to the Melrose-Malden border. The existing streambed was deepened, widened, and lined with concrete in 1960. A 12.5-foot diameter tunnel built at the downstream end of Lower Spot Pond Brook in Malden provided the capacity to contain a 25-year flood discharge safely within the channel's banks. This tunnel was built to circumvent the construction of a surface drainage structure through the center of Malden, where the old stream channel flowed to its confluence with the Malden River. The tunnel outlet structure directs discharge into the Malden River near Charles Street and is the source of the Malden River flows. At the time the tunnel was constructed, the Malden River water-surface elevations were affected by tidal surge.

The Amelia Earhart Dam, under the jurisdiction of the MDC, is a multipurpose structure used for flood control and recreation. The dam was built across the confluence of the Malden and Mystic Rivers in the early 1970s to maintain the upstream water surface at constant elevations. The dam, however, has minor influences on elevations of Lower Spot Pond Brook during normal conditions. During high intensity, long duration storms, the elevation of the Malden River

may affect water-surface elevations at the tunnel intake and points further upstream along Lower Spot Pond Brook.

In the City of Marlborough, the NRCS has constructed a system of flood control reservoirs to reduce the severity of flooding along the Assabet River. There are no such man-made flood protection systems for the brooks in Marlborough. A limited amount of natural storage area, which aids in reducing peak flows, is available in the city's wetlands. Marlborough has adopted measures to protect its floodplain and wetland areas from development.

In the Town of Maynard, at the present time, nine flood-retarding structures under the supervision of the NRCS are in operation and one is under construction. These structures are located on the upper Assabet River tributaries, outside of the Maynard corporate limits. The overall effect of the structures would be to significantly lower flood peaks, reducing the chance of recurrence of floods of the magnitude of the 1955 or 1968 floods.

In the City of Medford, Amelia Earhart Dam is located at the confluence of the Mystic and Malden Rivers. Constructed by the MDC and the USACE between 1967 and 1976, it is a multi-purpose structure under the jurisdiction of the MDC. The dam, which eliminated the tidal influence upstream, can pump 4,000 cfs of flow from the Mystic and Malden Rivers against high tide into Boston Harbor.

Along portions of the Mystic River, open parklands operated by the MDC prevent floodplain encroachment, provide floodplain storage, and provide a buffer between the streams and the developed areas.

A number of flood control projects have been constructed in the City of Melrose. Spot Pond Brook Branch and Ell Pond Brook Branch were enclosed in a piecemeal manner that improved floodwater drainage. Further channel improvements were made to Lower Spot Pond Brook from the outlet of Spot Pond Brook Branch and Ell Pond Brook Branch near Wyoming Station to the Melrose-Malden border. The existing streambed was deepened, widened, and lined with concrete in 1960. A 12.5-foot diameter tunnel built at the downstream end of Lower Spot Pond Brook in Malden provided the capacity to contain a 25-year flood discharge safely within the new channel's banks. This tunnel was built to circumvent the construction of a surface drainage structure through the center of Malden, where the old stream channel ran down to its confluence with the Malden River. The tunnel outlet structure directs discharge into the Malden River near Charles Street in Malden and is the source of the Malden River flows. At the time the tunnel was constructed, the Malden River water-surface elevations were affected by tidal surge. Amelia Earhart Dam was built across the confluence of the Malden and Mystic Rivers in the early 1970s to maintain the upstream water surface at constant elevations. The dam, however, has minor influence on Lower Spot Pond Brook elevations during normal conditions. During high intensity storms of long duration, the elevation of the Malden River may affect water-surface elevations at the tunnel intake and points further upstream along Lower Spot Pond Brook.

In the City of Newton, a multi-purpose regulating dam and pumping station completed by the USACE in 1978 was designed to significantly reduce future flood stages in the Charles River basin. The facility replaced a structure built in 1910 which had become obsolete. The dam and pumping station were designed to reduce the elevation of a flood of the same magnitude as the August 1955 flood, from 6.9 feet to 4.0 feet. The facility has no effect on flood stages upstream of the Watertown Dam.

Another USACE flood control measure, in accordance with Public Law 93-351 (Water Resources Development Act of 1974), is the acquisition of flowage rights in 8,442 acres of 17 natural valley storage areas in the upper and middle Charles River basins. In order to justify the federal expenditure for this program, the Congressional authorization in Public Law 93-351 required a commitment from the Commonwealth of Massachusetts that floodplain and wetlands protection zoning will be adopted and enforced in the Charles River watershed.

Following the major flood of August 1955, the MDC initiated channel improvements on the middle Charles River in Newton Upper Falls, from the Silk Mill Dam to the Nahanton Street Bridge. The improvements included the installation of a hydraulically operated bascule gate at the dam and extensive channel excavation from the dam to Nahanton Street, a distance of approximately 2 miles. The gate has a 4.5-foot range in elevation from a lowered position of 80 feet to a raised position of 84.5 feet.

Flood protection is also generated by following the USACE recommendation concerning the operation of the Silk Mill Dam and Mother Brook Control Dam in Dedham in advance of and during storms.

The City of Newton has established regulations governing the use of its floodplains and watershed areas. The provisions of Section 30-20 of the zoning ordinances give the Board of Aldermen control over these areas in all matters pertaining to construction, maintenance of bridges, recreational areas, and agriculture. These regulations are considered to be more restrictive than the minimum regulations as required for a community's eligibility in the National Flood Insurance Program.

There are no flood protection measures for the rivers and streams in the Town of Pepperell. The two dams which do exist, the Pepperell Pond Dam on the Nashua River and a dam on the Nissitissit River, do not offer flood protection. However, the town has sought to limit floodplain development by designating parts of the undeveloped floodplain as conservation land.

In the City of Somerville, Amelia Earhart Dam is located at the confluence of the Mystic and Malden Rivers. Constructed by the MDC and the USACE between 1967 and 1976, it is a multi-purpose structure under the jurisdiction of the MDC. The dam, which eliminated the tidal influence upstream, can pump 4,000 cfs of flow from the Mystic and Malden Rivers against high tide into Boston Harbor.

In the Town of Stoneham, open parklands operated by the MDC surround Spot Pond Reservoir, Doleful Pond, Fells Reservoir, North Reservoir, and South

Reservoir in Stoneham. This prevents floodplain encroachment, provides floodplain storage, and provides a buffer between the ponds and reservoirs and the developed areas.

In the Town of Stow, on the upper Assabet River basin, nine flood control structures have been completed and one is under construction by the Natural Resources Conservation Service (NRCS). One such structure, the Delaney Dam, is located on Elizabeth Brook in Stow. These structures are designed to lower flood peaks on the Assabet River and on their respective tributaries such as Elizabeth Brook. Several floodplain management measures have been undertaken by town officials, including zoning controls to protect fringe areas along the Assabet River from encroachment, ongoing land management, and maintenance programs by the town conservation commission.

In the Town of Sudbury, Cedar Swamp provides a natural storage area for the Sudbury River. The storage area helps to mitigate peak flows and the severity of flooding along the Sudbury River as it passes through the town. The Sudbury Reservoir and the Framingham Reservoir system, which are an integral part of the MDC water-supply system, also provide significant storage volume which reduces peak flood flows on the Sudbury River.

In the Town of Tewksbury, above the study area there are five dams designed for flood control on the Merrimack River. They were constructed and are operated by the New England Division of the USACE. These structures are the Franklin Falls Dam on the Pemigewasset River, the Edward McDowell Dam on Nubanusit Brook, the Blackwater Dam on the Blackwater River (flood control), and two dams, the Everett Dam on the Piscataguog River and the Hopkinton Dam on the Contoocook River that control Hopkinton Lake. These reservoirs have the capacity to reduce flood stages on the Merrimack River in the Tewksbury area about 8 feet for a recurrence interval of the March 1936 event.

In the Town of Watertown, natural storage that exists in the many swamps and ponds in the upper Charles River is an important factor in dampening the potentially hazardous effects of the floodwaters. A program is presently underway to acquire extensive natural valley storage areas in the upper watershed in order to ensure preservation of these areas and allow for natural storage of floodwaters. This program is being administered by the USACE.

The reach of the Charles River downstream of the Watertown Dam is called the Charles River basin. The flood elevations in this area are controlled by the Charles River Dam located approximately eight miles downstream from Watertown.

In the Town of Wayland, Cedar Swamp, in the Towns of Westborough and Hopkinton, provides a natural upstream storage area for the Sudbury River which substantially decreases peak flows and thus the severity of flooding along the Sudbury River, as it passes through Wayland. The Sudbury Reservoir and the Framingham Reservoir systems provide significant storage volume which reduces peak flows on the Sudbury River.

Natural storage that exists in the many swamps and ponds throughout the Town of Weston is an important factor in dampening the potentially hazardous effects of the floodwaters. A program is presently underway to acquire extensive natural valley storage areas in the upper Charles River watershed in order to ensure preservation of these areas and allow for the natural storage of floodwaters. This program is being administered by the USACE.

In the Town of Winchester, two 30-inch pipes with gates at the base of the Main Street Falls are opened when a major storm is anticipated, dropping the water level in the Mill Pond approximately 12 inches. For normal and low flow conditions, this structure controls the water level of the Aberjona River upstream. During flood flow conditions, however, the falls do not significantly influence the water level of the Aberjona River.

A floodgate structure also exists at the mouth of Wedge Pond. The removal of flashboards when a major storm is anticipated drops the water level in Wedge Pond approximately 12 inches also.

Flooding Source and Location	Gage Number	Period of Record	Drainage Area (sq. mi.)	Datum (NGVD29)
Aberjona River at Winchester	01102500	April 1939 to 1980	24.8, excludes 0.6 drained by Winchester North Reservoir	"sea level", assumed 0

3.0 ENGINEERING METHODS

For the flooding sources studied in detail in the county, standard hydrologic and hydraulic study methods were used to determine the flood hazard data required for this FIS. Flood events of a magnitude which are expected to be equaled or exceeded once on the average during any 10-, 50-, 100-, or 500-year period (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 50-, 100-, and 500-year floods, have a 10-, 2-, 1-, and 0.2-percent chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long term average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood which equals or exceeds the 100-year flood (1-percent chance of annual exceedence) in any 50-year period is approximately 40 percent (4 in 10), and, for any 90-year period, the risk increases to approximately 60 percent (6 in 10). The analyses reported herein reflect flooding potentials based on conditions existing in the county at the time of completion of this FIS. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish the peak discharge-frequency relationships for the flooding sources studied in detail affecting the county.

Precountywide Analyses

For each community within Middlesex County that has a previously printed FIS report, the hydrologic analyses described in those reports have been compiled and are summarized below.

Discharge-frequency data for the flooding sources studied by detailed methods were determined from equations based on multiple-regression analyses of data from USGS gaged sites in Massachusetts and adjacent areas of bordering states (U.S. Department of the Interior, 1978). The equations contain the independent variables basin drainage area, main-channel slope, and a precipitation intensity index.

Flooding on the Assabet River is presently controlled by flood storage reservoirs constructed by the NRCS in the upper Assabet River basin (U.S. Department of Agriculture, 1975). Hydrographs of peak flows on the Assabet River for 10- and 1-percent annual chance recurrence interval floods were prepared by the NRCS at Maynard, Stow and Hudson, without consideration of the flood control structures (U.S. Department of Agriculture, 1970). The USGS has maintained a gage at Maynard for approximately 65 years (USGS, prior to 1950-1975). A log-Pearson Type III statistical analysis on uncontrolled records (U.S. Water Resources, 1976) yields results comparable to the 10- and 1-percent annual chance peak flows for the hydrographs developed without structural control by the NRCS. Therefore, the hydrograph analysis can be assumed to be valid. However, the unmodified flow record is no longer valid; therefore, the modified hydrograph at Maynard was employed to establish peak discharge-frequency estimates at that point. In the Town of Hudson, the NRCS then modified its hydrographs to reflect the effects of the flood control structures located upstream. At Acton and Concord, these flows were modified by the use of regression equations to reflect the change in intervening drainage area and the total flood control reservoir storage area at each point. Frequency curves on the 10- and 1-percent annual chance floods at each point were then extended on log-probability paper to establish the 2- and 0.2-percent annual chance flows.

Discharge-frequency estimates for the 10-, 2-, and 1-percent annual chance floods for Branch of Assabet River were computed by the USGS regional formula (U.S. Geological Survey, 1974). Peak discharge estimates for the 0.2-percent annual chance flood were determined by the extension of the frequency curve for the 10-, 2-, and 1-percent annual chance floods on log-probability paper.

Discharge-frequency-drainage area relationships and discharge-frequency relationships for Fort Pond Brook, Grassy Pond Brook, Nashoba Brook, Pratt's Brook, Conant Brook, Cole's Brook, Tributary 1 to Cole's Brook, Tributary 2 to Tributary 1 to Cole's Brook, Tributary 1 to Sudbury River, Dakins Brook, Tributary 2 to Assabet River, Mill Brook 2, and the portion of Spencer Brook in Concord were developed using the hydrologic methods developed by the NRCS and the USGS (U.S. Department of Agriculture, 1972, U.S. Department of Agriculture, 1973, U.S. Department of the Interior, 1974). These methodologies base flood flows on basin characteristics such as drainage area, basin slope, soil type, land use, and precipitation duration and intensity. The methodology used to establish the 10-, 2-, and 1-percent annual chance discharge-frequency

relationships for the restudied portions of Fort Pond Brook, Grassy Pond Brook, Inch Brook, and Nagog Brook is outlined in USGS Water-Supply paper No. 2214, Estimating Peak Discharges of Small, Rural Streams in Massachusetts (U.S. Department of the Interior, 1983). The 0.2-percent annual chance discharge was calculated from regression analyses of the 10-, 2-, and 1-percent annual chance discharges. In Boxborough, discharge-frequency relationships for Fort Pond Brook and Fort Pond Brook Branch were obtained by computations from a USGS regional study (U.S. Department of the Interior, 1974). The 0.2-percent annual chance discharge was computed based on a log-Pearson Type III distribution for the three lower floods, using regional skew coefficients (Water Resources Council, 1976). Discharge-frequency estimates were compared with downstream discharges computed for the Acton FIS (FEMA, 1988). These results were plotted with the results of the log-Pearson Type III analysis on Heath Hen Meadow gage, on frequency-discharge-drainage area curves. Discharges were found to vary with drainage area to the 0.90 exponential power. In Maynard, peak discharge-frequency estimates for Fort Pond Brook Branch and Taylor Brook for the 10-, 2-, and 1-percent annual chance floods were determined by a USGS regional formula (USGS, 1974). This formula accounts for the parameters of drainage area, slope, and mean annual precipitation. The precipitation data were obtained from the U.S. Weather Bureau (U.S. Weather Bureau, 1961). The 0.2-percent annual chance discharge was determined by extrapolation of a log-probability graph of flood discharges computed for frequencies of up to 100 years. For Nashoba Brook, the 0.2-percent annual chance discharge was calculated from regression analyses of the 10-, 2-, and 1-percent annual chance discharges.

Discharge-frequency data for the outflow of Nagog Brook at its outlet with Nagog Pond were determined using the USACE HEC-1 computer program (USACE, 1981). The outlet flow was used as starting flow at the upstream end of Nagog Brook.

In Acton, the methodology used to establish the 10-, 2-, and 1-percent annual chance discharge-frequency relationships for the restudied portions of Guggins Brook is outlined in USGS Water-Supply paper No. 2214, Estimating Peak Discharges of Small, Rural Streams in Massachusetts. (U.S. Department of the Interior, 1983) The 0.2-percent annual chance discharge was calculated from regression analyses of the 10-, 2-, and 1-percent annual chance discharges. In Boxborough, discharge-frequency relationships were obtained by computations from a USGS regional study (U.S. Department of the Interior, 1974.) The 0.2-percent annual chance discharge was based on a log-Pearson Type III distribution for the three lower floods, using regional skew coefficients (Water Resources Council, 1976.) The discharge-frequency estimates were plotted on frequency-discharge-drainage area curves and checked wit the log-Pearson Type III analysis on the nearby Heath Hen Meadow Brook gage. Flows at the gage were slightly lower than Guggins Brook flows for comparable drainage area, due to a large amount of storage area within the Heath Hen Meadow Brook drainage basin. Guggins Brook discharges were found to vary with drainage area to the 0.88 exponential power.

In Acton, the methodology used to establish the 10-, 2-, and 1-percent annual chance discharge-frequency relationships for the restudied portions of Butter

Brook is outlined in USGS Water-Supply paper No. 2214, Estimating Peak Discharges of Small, Rural Streams in Massachusetts (U.S. Department of the Interior, 1983). The 0.2-percent annual chance discharge was calculated from regression analyses of the 10-, 2-, and 1-percent annual chance discharges. In Westford, peak discharge-frequency relationships for Butter Brook, Boutwell Brook, Tadmuck Brook, and Tadmuck Swamp Brook were developed using procedures described by the USGS in Estimating the Magnitude and Frequency of Floods for Natural-Flow Streams in Massachusetts (U.S. Department of the Interior, 1977). The results were reconciled and verified with statistically analyzed data from nearby stream gages with similar watershed characteristics and the drainage area relationships described above.

In Ashland, Hopkinton, and Wayland, the result of a mathematical model developed by the Soil Conservation Service was the source of the 10-, 1-, and 0.2-percent annual chance peak discharges for the Sudbury River (U.S. Department of Agriculture, 1973). The peak discharges for the 2-percent annual chance flood was obtained graphically using Soil Conservation Service data. In Concord and Lincoln, a discharge-frequency relationship was established by modifying the log-Pearson Type III statistical analysis of the annual maximum daily discharge data for the 102 years of record on the Sudbury River gaging stations in Framingham (Water Resources Council, 1976; U.S. Department of Agriculture, 1972; U.S. Department of Agriculture, 1973). Natural river flows were modified to reflect the change in peak flow because of the intervening drainage area (U.S. Department of the Interior, 1974). In Framingham, discharge-frequency data were obtained using computer modeling techniques developed by the NRCS. In this method, a precipitation event is simulated over a basin, runoff is calculated, and the quantity of flow is routed through stream reaches and control structures. The 10-, 2-, 1- and 0.2-percent annual chance peak discharges were determined by applying the appropriate total rainfall depth associated with a particular event. Most of the watershed was analyzed in this way except for certain diversion areas where manual hand-routing procedures were used. In Sudbury, a computer model was developed for the entire watershed to Fairhaven Bay in Concord. The computer model was developed using the USACE HEC-1 computer program. (USACE, 1990). The base model for the watershed area above Sudbury was taken from a HEC-1 model developed by the USACE for the Saxonville Flood Control Project in Framingham (USACE, 1990). The model was then extended downstream by adding in the watershed areas for Sudbury and the surrounding communities downstream of Saxonville. The HEC-1 model developed for the Sudbury River in this FIS is based on the 1955 hurricane, which produced the most severe flooding on record upstream of the Town of Sudbury. The HEC-1 model was run and calibrated to match the historic flood elevation for the 1955 hurricane within 0.5 foot. The elevations in the HEC-1 model for the 1955 hurricane compared very closely with the base flood elevations developed for the 1982 FIS. Therefore, it was determined that a revision was not warranted for this stream.

Cold Spring Brook, Tributary to Cold Spring Brook, Tributary to Waushakum Pond, Indian Brook, Indian Brook Tributary, Beaverdam Brook, Angelica Brook, Davis Brook, James Brook, Martins Pond Brook, the Nissitissit River, Reedy Meadow Brook, Unkety Brook, Bennetts Brook, Mill Brook 3, Snake Brook, Pine

Brook, Hayward Brook, Trout Brook 2, Hop Brook, Mowry Brook, Broad Meadow Brook, Dudley Brook, Run Brook, Pantry Brook, Landham-Allowance Brook, Walker Brook 1, Walker Brook 2 Walker Brook 3, Assabet Branch 3, Assabet Branch 4, Morse Brook, Bow Brook, Catacoonamug Brook, Pearl Hill Brook, Mason Brook, Locke Brook, Willard Brook, Witch Brook, Tributaries A and B to Squannacook River, and Brook from Waushakum Pond discharge-frequency data was defined using regional equations (U.S. Department of the Interior, 1974). These equations, which relate basin characteristics to stream flow, provided the method for which the 10-, 2-, and 1-percent annual chance peak discharges were obtained. The 0.2-percent annual chance peak discharge was obtained graphically using the data obtained by the regional discharge-frequency equations. This data was compared with previously published data where such data was available and applicable. The result of a mathematical model developed by the Soil Conservation Service was the source of the 10-, 1-, and 0.2-percent annual chance peak discharges for the Cold Spring Brook between the Ashland Reservoir to Reservoir No.2 (U.S. Department of Agriculture, 1973). The peak discharges for the 0.2-percent annual chance flood was obtained graphically using Soil Conservation Service data. In Natick, the 0.2-percent annual chance peak discharge estimates for Beaverdam Brook and Davis Brook were determined by linear extrapolation of a log-Pearson Type III probability distribution on the 10-, 2-, and 1-percent annual chance floods (U.S. Water Resources Council, 1976). The revised hydrologic analysis for Hayward Brook was performed using USGS regional equations for small rural streams in Massachusetts (U.S. Department of the Interior, 1982). Gage data taken at the U.S. Route 20 culvert at Hayward Brook was used in the regression analysis to develop the regional equations for the 10-, 2-, and 1-percent annual chance discharges.

Gaging stations on the Nashua River in East Pepperell and on the North Nashua River in Leominster were the principal sources of data utilized for defining discharge-frequency relationships for the Nashua River (U.S. Department of the Interior, 1964; U.S. Department of the Interior, 1976). These gages have been in operation since 1936. Values for the 10-, 2-, 1-, and 0.2-percent annual chance discharges at each gage were obtained from a log-Pearson Type III statistical analysis of annual peak flow data. In order to define discharge-frequency relationships for the Nashua River, a method of analysis in the Soil Conservation Service National Engineering Handbook was utilized (U.S. Department of Agriculture, 1972). This method provides for the hydrologic routing of flows. This method, which relates the discharge, in cfs per square mile, between any two points in the drainage basin by a ratio of the respective drainage areas, was used to account for the large floodplain storage available along the Nashua River.

To determine flood discharges on the Concord River, which is a gaged watercourse, it was necessary to determine and route flows on the Sudbury and Assabet Rivers, the rivers which form the Concord River. Because the Sudbury and Assabet Rivers are gaged, a log-Pearson Type III analysis (U.S. Water Resources Council, 1976) was performed on these rivers, and the analysis for the Assabet River was adjusted to reflect the effects of construction of a series of floodwater retention reservoirs on the Upper Assabet River during the past decade. Hydrographs produced at the headwaters of the Concord River were then routed downstream, adding in additional inflow from runoff from tributary

drainage areas and reducing flow because of storage in the Concord River floodplain. The routed flows were further modified based on data from the USGS gage on the Concord River in Lowell. Resultant flows were used as design discharges for the 10-, 2-, 1-, and 0.2-percent annual chance recurrence interval floods.

In Bedford, Billerica, and Wilmington, flood discharges for the Shawsheen River were computed using a log-Pearson Type III analyses of the USGS gage (No. 01100600) at Wilmington, and high-water marks recorded during the January 1979 flood. Using these developed flows and the frequency curve developed for the Shawsheen River at the Bedford/Billerica corporate limits, flows were calculated at this point. Using the NRCS area discharge relationship, these flows were transferred upstream (FEMA, 1983.) In Tewksbury peak discharges were developed based on data obtained from the USACE and the FIA (USACE, 1972; FEMA, unpublished).

In Bedford, the hydrologic data for Vine Brook, Elm Brook, Spring Brook, Tributary to Mill Brook**,** Mongo Brook, Dopping Brook, Chicken Brook, Bogastow Brook, Jar Brook, Dirty Meadow Brook, Course Brook, Tributary A and the Winthrop Canal were developed using the regional frequency method (U.S. Department on the Interior, 1977). Due to the inherent possibility of a large standard error in the regional frequency method, comparative computations of discharges by the rainfall-runoff technique based on a synthetic triangular unit hydrograph and NRCS methodology were utilized for assisting in the adoption of discharges for various frequencies into a smooth curve (U.S. Department of Agriculture, 1972). In Burlington, the hydrologic analysis for Vine Brook was obtained from the Burlington storm water management report (Metcalf and Eddy, Inc., 1978). Methods for developing hydrographs used in the preparation of this report were the NRCS TR-20 and the Environmental Protection Agency Storm Water Management Model (SWMM) reports (U.S. Department of Agriculture, 1973; U.S. Environmental Protection Agency, 1971). The TR-20 method was used for the upstream portions of Vine Brook. In Lexington, discharge-frequency estimates for Vine Brook for the 10-, 2-, 1-, and 0.2-percent annual chance storms were based on the USGS gage record on Beaver Brook. The gage is located in Belmont, downstream of Lexington, near the confluence of Beaver Brook and the Charles River. To insure a reasonable estimate of peak discharges, the 10-year gaging record on Beaver Brook was extended by correlation with the 44 year record of the USGS gaging station on the Charles River at Waltham. The correlation was carried out in accordance with statistical methods established in Water Resources Bulletin #17 (USGS, 1974). A log-Pearson type III analyses was run on both the 10-year record and on the extended record using a regionalized skew coefficient in both cases. The drainage areas contributing to peak discharges on Vine Brook have characteristics of runoff similar to those on Beaver Brook. Due to the similarity of watersheds, the 10-, 2-, and 1-percent annual chance floods for Vine Brook, Kiln Brook, Mill Brook 1, Munroe Brook, and North Lexington Brook were determine by applying the Beaver Brook coefficients to discharges computed by the USGS regional formula. The 0.2-percent annual chance discharge was determined by straight line extrapolations of a log-probability graph of discharges computed above. Discharges at other points were

determined by applying the discharge computed from the regional formula to a discharge-drainage area formula.

The hydrology for Wellington Brook was calculated using the unit hydrograph theory. This technique was selected because the watershed is ungaged, has natural storage flow regulation and high urbanization. Synthetic triangular unit hydrographs were developed to represent each watershed, utilizing available data and making adjustments for slopes and local inflows (USACE, 1976).

A method developed by the NRCS for ungaged watersheds was used to obtain the hydrologic analyses for Content Brook in Billerica (U.S. Department of Agriculture, 1975; U.S. Department of Agriculture, 1972). This method is based on soil types, type of land cover, and surface roughness. The soil cover-land use complex is given a curve number from which storm runoff and peak flow can be determined. In Tewksbury, peak discharges for Content Brook, Sutton Brook, Collins Brook, and Heath Brook were developed from regional relationships published for southeastern New England (USACE, 1972). The peak discharges obtained for Content Brook were used to calculate the frequency discharges for Mud Pond Brook, Saunders Brook, and Brook A, using the discharge-drainage area ratio formula.

Discharge-frequency relationship data for Beaver Brook No. 1 in Littleton and Westford was developed using the procedures described by the USGS in Estimating the Magnitude and Frequency of Floods for Natural-flow Streams in Massachusetts (U.S. Department of the Interior, 1977). The technique was developed using multiple regression analyses to estimate flood peaks in ungaged, natural-flow streams in Massachusetts by relating peak discharges to basin and climatic parameters. The resulting peak discharges were used to develop corresponding peak discharges at the inlet of Forge Pond using a multiplications factor equal to the ratio of the drainage areas to the 0.75 power. In Boxborough, discharge-frequency relationships were obtained by computations from a USGS regional study (U.S. Department of the Interior, 1974). The 0.2-percent annual chance discharge was computed based on a log-Pearson Type III distribution for the three lower floods, using regional skew coefficients (Water Resources Council, 1976). The discharges computed were compared with a downstream study by the USACE (USACE, 1975). These two results were plotted on a frequency-discharge-drainage area curve. The Shawsheen River was assumed, in the Littleton report, to have similar basin characteristics to Beaver Brook. The gaging record on the Shawsheen River (gage no. 01100600, 11 years of record) at Wilmington was extended by correlation with the Ipswich River gage (no. 01102000, 45 years of record) at Ipswich and a log-Pearson Type III analysis was run on the extended record (Water Resources Council, 1976). The gage discharges were also plotted on the curves and found to check well with the derived relationship that discharge varies with drainage area to the 0.73 exponential power.

Because most of the watercourses in Chelmsford are ungaged, it was necessary to investigate several hydrologically similar gaged streams. Requisite parameters needed for this investigation included the following: drainage area, main channel length, main channel slope, hydrologically similar gaged streams were located,

flow-frequency statistical analysis based on yearly maximum flows at the gages for their period of record through 1974. Discharge-frequency relationships for Stony Brook were developed using this method and used information from the USGS gage on the Parker River at Byfield (Gage No. 01101000). Hydrologically similar watersheds could not be found for Hales Brook, Putnam Brook, and Beaver Brook 2. Flow-frequency relationships for Hales Brook and Putnam Brooks were developed using regression analysis methods of the USGS (USGS, 1974). Flood-frequency discharges were adjusted to reflect differences in the sizes of the drainage areas contributing to the stream flows. Flow-frequency relationships for Beaver Brook No. 2 were developed using the NRCS method. This method is based on soil types found in the watershed, type of land cover, how the land is used, watershed slope, stream flow length, and surface roughness. The soil cover-land-use complex is given a curve number from which storm runoff and peak flow can be determined (U.S. Department of Agriculture, NRCS, 1975; NRCS, 1972).

Beaver Brook 3 discharges were developed by statistical analysis of available flow data in the region and by the use of empirical regression equations developed for Massachusetts by the USGS (U.S. Department of the Interior, 1977). There are no stream flow gaging stations on Beaver Brook. Two representative gaging stations within the region were used. Statistical analyses were performed using a log-Pearson Type III distribution on the Assabet River at Maynard and on the Ipswich River at Ipswich. Discharge frequencies were then transferred from each gage to Beaver Brook by ratio of respective drainage areas to the 0.7 exponential power. Also, discharge frequencies were developed by use of the referenced USGS regression equations. These equations were applied using physical characteristics of the Beaver Brook 3 watershed. It was determined that the discharge frequencies developed by the two methods were comparable and agreed with the discharges used in previous FISs in Dracut; therefore, the discharge frequencies in the original Dracut study were adopted in this study (FEMA, 1980).

For Beaver Brook 4 in Lexington, discharge-frequency estimates for the 10-, 2-, 1-, and 0.2-percent annual chance storms were based on the USGS gage record on Beaver Brook. The gage is located in Belmont, downstream of Lexington, near the confluence of Beaver Brook and the Charles River. To ensure a reasonable estimate of peak discharges, the 10-year gaging record on Beaver Brook was extended by correlation with the 44 year record of the USGS gaging station on the Charles River at Waltham. The correlation was carried out in accordance with statistical methods established in Water Resources Bulletin #17 (USGS, 1974). A log Pearson type III analyses was run on both the 10-year record and on the extended record using a regionalized skew coefficient in both cases. Peak discharges at the Beaver Brook gages for the 10-, 2-, and 1-percent annual chance floods were then computed based on the USGS regional formula (USGS, 1974). At the gage, these discharges were found to be low in comparison to the discharges computed by log-Pearson type III analysis on the gage record by a factor of 2.2 for the 10-percent annual chance flood, 2.9 for the 50-year flood and 3.2 for the 1-percent annual chance flood. Peak discharge estimates on Beaver Brook No. 4 in Lexington for the 10-, 2-, and 1-percent annual chance floods were computed using the USGS regional formula. The resultant discharges were modified by applying the above-mentioned coefficients established at the Belmont

gage. The 0.2-percent annual chance peak discharge estimate was determined by straight line extrapolation of a log-probability graph of discharges computed for the 10-, 2-, and 1-percent annual chance floods. Discharges on Beaver Brook No. 4 at two other locations in Lexington were determined using a peak discharge-drainage area formula. Hydrologic data for Beaver Brook No. 4 in Waltham were calculated using unit hydrograph theory. This technique was selected because the watersheds are ungaged, have natural storage flow regulation and have high urbanization. Synthetic triangular unit hydrographs were developed by representing each watershed, utilizing data available in USGS Water Resources Investigation 77-39, and making appropriate adjustments for slopes and local inflows (U.S. Department of the Interior, 1977). The results of these studies were compared to results obtained by using the peak discharge equations found in the USGS publication (U.S. Department of the Interior, 1977).

Discharge-frequency estimates for Elizabeth Brook and Branch of Elizabeth Brook 1 were developed by the NRCS in their study on Elizabeth Brook (U.S. Department of Agriculture, 1975). These discharges were developed using the NRCS computer program for project formulation, Hydrology, TR-20 (U.S. Department of Agriculture, 1975). The program computes surface runoff, taking into account conditions having an effect on runoff and routes the flow through stream channels and reservoirs. It combined the routed hydrograph with those from other tributaries and computes the peak discharges, the time to peak, and the water-surface elevation at selected cross sections. It takes into account the retarding effect of storage areas, such as the Delaney Reservoir, in decreasing discharges. Discharges were developed by the NRCS for floods of 10-, 2-, and 1-percent annual chance frequency. The frequency curve on these three floods was extended on log-probability paper to determine the 0.2-percent annual chance peak discharge. In Boxborough, peak discharges for Elizabeth Brook were calculated using the USGS Regional Regression Equations for Massachusetts (U.S. Department of the Interior, 1993).

In Concord, discharge-frequency-drainage area relationships and discharge-frequency relationships for Sawmill Brook were developed using the hydrologic methods developed by the NRCS and the USGS (U.S. Department of Agriculture, 1972, U.S. Department of Agriculture, 1973, U.S. Department of the Interior, 1974). These methodologies base flood flows on basin characteristics such as drainage area, basin slope, soil type, land use, and precipitation duration and intensity. In Burlington, the hydrologic analysis was obtained from the Burlington stormwater management report (Metcalf and Eddy, Inc., 1978). Methods for developing hydrographs used in the preparation of this report were the NRCS TR-20 and the Environmental Protection Agency Storm Water Management Model (SWMM) reports (U.S. Department of Agriculture, 1973; U.S. Environmental Protection Agency, 1971). Flows along Sawmill Brook were developed using the NRCS TR-55 model (U.S. Department of Agriculture, 1975).

Discharge-frequency-drainage area relationships were calculated for Pages Brook, Pages Brook Branch, Marshall Brook, Darby Brook, Trull Brook, Meadow Brook, and Meadow River Branch as well as the portion of Spencer Brook in Carlisle using the USGS regional method (U.S. Department of the Interior, 1977).

Peak flood discharges for River Meadow Brook in Chelmsford were calculated for the 10-, 2-, and 1-percent annual chance periods (recurrence interval) using regression equations. The regression equations used in the analysis are published in the USGS Water-Supply Paper 2214 (USGS, 1983). A regression equation was not available for the 0.2-percent annual chance flood; therefore, the flood peaks were extrapolated from the 50- to 1-percent annual chance data. The rural peak discharges calculated with the regression equations were not adjusted for urbanization because there is storage potential within the watershed. No urbanization adjustment would also be consistent with the methodology used in the downstream contiguous community of Lowell. In Lowell, the discharges were developed by statistical analysis of available flow data in the region and by use of empirical regression equations developed by the USGS for Massachusetts (U.S. Department of the Interior, 1977). Since stream flow gaging information was not available for River Meadow Brook, two representative gaging stations in the region were used; Aberjona River at Winchester and Nashoba Brook near Acton. The drainage areas of the two streams at the gage sites are 24.2 square miles and 12.7 square miles, respectively. Statistical analyses were performed using a log-Pearson Type III distribution. Discharge frequencies were then transferred from each gage to River Meadow Brook by ratio of respective drainage areas to the 0.7 exponential power. Also, discharge frequencies were developed by used of the reference USGS regression equations. The equations were applied to River Meadow Brook at its mouth using the physical parameters of drainage area (25.7 square miles) and main channel slope (5.5 feet/mile). The equations were also applied at the upstream end of the study limit where the drainage area was 22 square miles and the main channel slope was 5 feet/mile. It was determined that the discharge frequencies developed by the two methods were comparable and the results using the regression equations were adopted.

The 10-, 2-, 1-, and 0.2-percent annual chance discharges for the Merrimack River in Dracut, Lowell, and Tewksbury were developed by statistical analysis of recorded flow data in the region. The USGS gaging station located on the Merrimack River below the Concord River (gage no. 01100000) within the City of Lowell was used in the analysis. Statistical analysis was performed at the gaging station by using annual peak flows in a log-Pearson Type III distribution (U.S. Department of the Interior, 1981). The computed discharge frequency curve was adjusted for the modifying effects due to upstream USACE flood control reservoirs. In Tyngsborough and Chelmsford, flows developed for the Merrimack River by the USACE were reviewed and adopted (USACE, 1972). A discharge-frequency relationship was developed using a log-Pearson Type III statistical analysis of the discharges for the USGS Goffs Falls gage in Manchester, New Hampshire. The period of record for the gage extends from October 1936 to September 1977 (Water Resources Council, 1976). The discharges were then adjusted to reflect reductions caused by the flood control reservoirs between Manchester and Tyngsborough, as well as the additional runoff from the intervening drainage area.

Discharge frequencies for Richardson Brook were determined using a 13-year record of peak flows on the stream as measured by the USGS. A discharge frequency curve was developed using the log-Pearson Type III procedure with a computed standard deviation of 0.202, a mean of 2.06, and an adopted regional

skew of 0.5. The discharges for Trout Brook No. 1 were developed by using available flow data from Richardson Brook. Richardson Brook has similar basin characteristics to that of Trout Brook No. 1. Discharges were developed by statistical analysis of the Richardson Brook gage data using empirical regression equations developed for Massachusetts by the USGS (U.S. Department of the Interior, 1977).

The Mystic River Comprehensive Hydrology Study was completed in September 1981 to determine the flood potential upstream of the Amelia Earhart Dam (Camp, Dresser and McKee, Inc., 1981). This study uses the techniques and results obtained from the MDC Comprehensive Hydrology Study for the analysis of the Malden River. Hydrologic analyses were carried out to establish inflow storm hydrographs. The hydrologic analyses technique used for the Malden River was computer simulation of the rainfall-runoff processes as they affect the lands within the entire basin. Rainfall events were simulated using the M.I.T. Catchment (MITCAT) rainfall-runoff model (Camp, Dresser and McKee, Inc., 1980). Storm hydrographs from the rainfall events were generated and later used as input to the hydraulic model of the Mystic-Malden River Systems.

Because of both the Amelia Earhart Dam and the large amounts of storage found along the stream lengths, the Mystic-Malden River system behaves more like a series of reservoirs than as free flowing streams. Peak stages are not the result of peak discharges and generally do not occur at the time of peak discharge. They are not caused by the peak discharge but by the runoff volume and basin storage relationships. The relationship between inflows, outflows, and the resultant change in storage dominate the system. The discharges for Amelia Earhart Dam represent the pumping capacity at the damn during periods of high tide. The discharge through the dam can be greater when tidal conditions are lower than water levels above the dam, in which case water from upstream of the dam is allowed to pass through the lock gates. Elevations for Lower Mystic Lake and Upper Mystic Lake were determined using the MITCAT model described above.

Stillwater elevations for the 10-, 2-, and 1-percent annual chance floods for Ell Pond were obtained from the Mystic River Comprehensive Hydrology Study (Camp, Dresser and McKee, Inc., 1981). The 0.2-percent annual chance flood elevation for Ell Pond was not calculated in that study. The 0.2-percent annual chance discharge splits between the Ell Pond Conduit and a broad-crested weir formed along the northeastern corner of Emerson Street and Main Street. The 0.2-percent annual chance elevation was determined considering flow through the conduit and over the roadway.

A gaging station on the Squannacook River located 2.7 miles northwest of West Groton was the principal source of data utilized for defining discharge-frequency relationships for the river (U.S. Department of the Interior, 1964; U.S. Department of the Interior, 1976). The gage has been in operation since 1949. Values for the 10-, 2-, 1-, and 0.2-percent annual chance discharges at the West Groton gage were obtained from a log-Pearson Type III analysis of annual peak flow data. In order to define discharge-frequency relationships for the Squannacook River at sections upstream and downstream of the gaging station, the NRCS method for the hydrologic routing of flows was used (U.S. Department of Agriculture, 1972).

It relates the discharge in cubic feet per second per square mile between any two points in the drainage basin by a ratio of the respective drainage areas.

In Weston and Waltham, the hydrology for Hobbs Brook, Chester Brook, West Chester Brook, and Stony Brook was calculated using unit hydrograph theory. This technique was selected because the watershed is ungaged and has natural storage flow regulation. Synthetic triangular unit hydrographs were developed representing the watershed, utilizing the data available in USGS Water Resources Investigation 77-39 (U.S. Department of the Interior, 1977), and making appropriate adjustments for slopes and local inflows. The results of this study was compared to results obtained by using the peak discharge equations found in Estimating the Magnitude and Frequency of Floods on Natural Flow Stream in Massachusetts (USGS, 1977). In Lincoln, peak discharge frequency estimates for the 10-, 2-, and 1-percent annual chance floods on Hobbs Brook, Stony Brook, Farrar Pond/Pole Brook, Farrar Pond Brook, and Valley Pond were determined using the USGS regional method (U.S. Department of the Interior, 1974). The 0.2-percent annual chance peak discharge estimates for these streams were determined by using a straight line extrapolation of plotted flood discharges for frequencies up to 100 years on log-probability paper. In Westford, peak discharge-frequency relationships for the upstream portions of Stony Brook required the use of another USGS publication (U.S. Department of the Interior, 1962). The technique is similar to that used on the other portions of the brook, but permits an adjustment in consideration of the Forge Pond/Beaver Brook No. 1 wetland storage areas. Increases in the peak discharges at various locations along Stony Brook were calculated using a series of discharge-drainage area curves developed from the previously mentioned data and the peak discharges for Stony Brook used in the FIS for the Town of Chelmsford (FEMA, 1979).

Peak discharges for Lower Spot Pond Brook, Town Line Brook, Varnum Brook, and Greens Brook were developed using the standardized techniques presented in the USGS Water-Supply Papers Estimating Peak Discharges of Small, Rural Streams in Massachusetts and in Flood Characteristics of Urban Watersheds in the United States (Wandle, Jr., 1983; Sauer, et al., 1983). The two publications are complimentary, where a rural watershed discharge was computed and used as input for computing an urban watershed discharge. The urban discharge also depended on drainage areas but, in addition, the resulting discharges. The first publication provides a method for calculating discharges based on regional storm data. It is designed for ungaged streams and uses drainage basin area as the variable for determining discharge. Equations found in the latter publication adjust the regional rural discharge values to an urban condition by incorporating a basin development factor. The only portion of the Lower Spot Pond Brook watershed contributing flow through the Melrose-Malden corporate limits that is not fully developed is the area in the Fellsway Reservation. The rest of the basin has little attenuation of runoff.

Computer modeling techniques developed by the NRCS determined the discharge-frequency data for Mulpus Brook and Graves Pond Brook (U.S. Department of Agriculture, 1972). In this method, runoff was calculated and the resulting quantity of flow was routed through stream reaches and control structures. The 10-, 2-, 1-, and 0.2-percent annual chance peak discharges were

determined by applying the appropriate total rainfall depth associated with a particular frequency. This methodology takes into account excessive storage areas between Townsend Road Bridge and the confluence of Mulpus Brook with the Nashua River. The storage areas cause discharges on Mulpus Brook to increase going upstream.

In Weston, the hydrology for Bogle Brook, Tributary 3 to Bogle Brook 2, Tributary 4 to Bogle Brook 2, Cherry Brook and Stony Brook was calculated using unit hydrograph theory. This technique was selected because the watershed is ungaged and has natural storage flow regulation. Synthetic triangular unit hydrographs were developed representing the watershed, utilizing the data available in USGS Water Resources Investigation 77-39 (U.S. Department of the Interior, 1977), and making appropriate adjustments for slopes and local inflows. The results of this study was compared to results obtained by using the peak discharge equations found in Estimating the Magnitude and Frequency of Floods on Natural Flow Stream in Massachusetts (USGS, 1977). In Natick, discharge-frequency data for Bogle Brook and Stony Brook were defined using regional equations (U.S. Department of the Interior, 1974). These equations, which relate basin characteristics to stream flow, provided the method for which the 10-, 2-, and 1-percent annual chance peak discharges were obtained. The 0.2-percent annual chance peak discharge estimates were determined by linear extrapolation of a log-Pearson Type III probability distribution on the 10-, 2-, and 1-percent annual chance floods (U.S. Water Resources Council, 1976).

Peak discharge-frequency estimates for the Charles River were developed by the USACE using the log-Pearson Type III method (U.S. Water Resources Council, 1976), from records of the USGS gaging stations on the Charles River at the Village of Charles River and at Waltham (U.S. Department of Agriculture, 1972). The gage on the Charles River (no. 01103500) has been in effect since October 1937, and the gage at Waltham (no. 01104500) has been in effect since October 1909. A discharge-frequency-drainage area relationship was developed from this data. Watertown is located downstream of the gages. Approximately one-third of the Charles River flow is diverted from the basin via Mother Brook in Dedham, which is located midway between the two gaging stations.

For the Ipswich River in Reading and North Reading, flood-flow frequency data were based on statistical analyses of stage-discharge records covering a 35-year period at the South Middleton gage operated by the USGS. This analysis followed the standard log-Pearson Type III method as outlined by the Water Resources Council (Water Resources Council, 1967). Based on a previous study of the Ipswich River basin, a skew coefficient of 0.5 was adopted (USGS, 1971). Discharges at various locations on the Ipswich River were derived by multiplying the given discharge by a factor equal to the ratio of the drainage areas to the 0.7 exponential power.

Peak discharges for the 10-, 2-, 1-, and 0.2-percent annual chance floods on Lubbers Brook and Ipswich River in Wilmington were initially determined using the regional frequency-discharge formulas in Water Resources Investigation 77-39 (U.S. Department of the Interior, 1977). The formulas used drainage area, main channel slope, and storage to develop the calculated discharges. The percentage

of impervious land in each drainage basin was considered in determining the regional discharge values.

Hydrologic routings using the Muskingum Method were carried out to take into account the moderating influence of storage in the extensive swamplands along the Ipswich River, Maple Meadow Brook, and Lubbers Brook (R.K. Linsley and J.B. Franzini, 1972). Calculations were made to determine inflow and outflow hydrographs and storage curves. From these working curves, discharges for the 10-, 2-, 1-, and 0.2-percent annual chance floods at the downstream limit of the routed reaches were determined.

Two reaches of the Ipswich River were routed. The section from the Kelly Road area to a point just upstream of the confluence of Maple Meadow Brook was first routed and then, using the altered outflows, the section from a point downstream of the confluence of Maple Meadow Brook was routed to a point midway between Wildwood and Federal Streets. The local inflows along each reach were added to the routed outflows to calculate the peak flows just downstream of the routed sections. The discharges at the corporate limits were matched exactly to the North Reading FIS (FEMA, 1996).

One reach was routed on Maple Meadow Brook and one on Lubbers Brook. For Maple Meadow Brook, the regional peak flows were routed from the upstream limit of detailed study to the confluence of an unnamed tributary. USGS stream gaging station no. 01101300 on Maple Meadow Brook is a partial record station with only 11 years of record (U.S. Department of the Interior, 1974). The discharges calculated by the regional equation and routing analysis were coordinated with the gage record. The peak flows determined from the regional equation for Lubbers Brook were routed from the Boston & Maine Railroad bridge to Middlesex Avenue. The local inflows along the reach were added to the routed outflows to calculate peak flows at the downstream limit of the reach.

In Wilmington, peak discharges on Lubbers Brook upstream of Glen Road were computed using the USACE HEC-1 Computer Program (USACE, 1990).

Peak discharges and hydrographs for the 10-, 2-, 1-, and 0.2-percent annual chance flood events for Martins Brook, Martins Pond, and Skug River were developed using the USACE HEC-1 computer program (USACE, 1990). Flood storage effects in Martins Brook upstream of Salem Street (Route 62) in the Town of Wilmington and in Martin Pond, upstream of Burrough Road, were considered. Stage-storage and stage-discharge rating curves for both location were developed during the study. Runoff curve numbers were computed for all drainage subareas in the watershed area for Martins Brook, Martins Pond, and Skug River and were used to develop NRCS unit hydrographs in HEC-1. Rainfall depths were taken from U.S. Weather Bureau's Technical Paper No. 40. Stage-frequency relationships used for Martins Pond were obtained from the elevations computed for Martins Brook at the outlet of the pond. In Wilmington, peak discharges for Martins Brook upstream of Salem Street and Tributary to Martins Brook were determined using regional equations developed by the USGS in Open-File Report 80-676 (U.S. Department of the Interior, 1982).

Elevations for Massapoag Pond were developed in the HEC-2 step-backwater analysis of Salmon Brook in the Flood Insurance Study for the Town of Dunstable (FEMA, unpublished).

Flooding of Alewife Brook (Little River) and the Lower Mystic Lake is caused by the elevated water-surface of the Mystic River. This backwater condition causes higher water-surface elevations than the natural drainage from the tributary area.

Discharge-frequency data for Nonacoicus Brook 1, Nonacoicus Brook 2, Tributary to Nonacoicus Brook, Long Pond Brook, Baddacook Brook, and Cow Pond Brook were obtained using computer modeling techniques developed by the NRCS (U.S. Department of Agriculture, 1972). In this method, a precipitation event is simulated over a basin, runoff is calculated, and the resulting quantity of flow routed through stream reaches and control structures. The 10-, 2-, 1-, and 0.2-percent annual chance peak discharges were determined by applying the appropriate total rainfall depth associated with a particular frequency.

Peak discharge frequencies for Tributary to Beaver Brook 3 and Peppermint Brook were derived using procedures presented in the USGS report developed for Massachusetts (U.S. Department of the Interior, 1977). The resulting flow values were also compared with statistically analyzed gaged stream records in the region and were found to be in general agreement.

Discharges for Gumpas Pond Brook were obtained from the FIS for the Town of Pelham, New Hampshire (FEMA, 1980). In that study, the discharges were determined by averaging the results of the regional equation by Johnson and Tasker and an area-weighted transposition with an adjusted log-Pearson Type III frequency analysis of the gages at Hop Brook (No. 01147000 with 27 years of record), Bungay Brook (No. 01112300 with 11 years of record), and East Meadow Brook (No. 01100700 with 11 years of record) (U.S. Department of the Interior, 1974; U.S. Department of the Interior 1981). The regional equations by Johnson and Tasker consist of parameters that include drainage area, ground slope, and average rainfall per year. Hop Brook, Bungay Brook, and East Meadow Brook have drainage areas with similar hydrologic characteristics as those of this study. Discharges for Gumpas Pond Brook were modified to include some storage effects. This modification utilizes a numerical reservoir routing technique known as the Unit Hydrograph Method, which developed an inflow hydrograph (Viessman, Harbough, and Knapp, 1972).

Discharges for Salmon Brook, Great Road Tributary, King Street Tributary, Mill Pond Tributary, Bear Meadow Brook, and Walkers Brook were determined using the Wandle method developed by the USGS specifically for Massachusetts. This methodology takes into consideration channel slope and drainage area in its evaluation of a stream (U.S. Department of the Interior, 1977). The USGS derived the equations used in the Wandle method by applying multiple regression techniques to flow data and physical characteristics of 113 stream gaging stations in or near Massachusetts. The regional equation flows were adjusted for Bear Meadow Brook, and Walkers Brook to account for impervious land surface area resulting from urbanization. Flows on the upstream portion of Great Pond were

determined using the transposition of drainage area methods established by the Water Resources Council (Water Resources Council, 1977).

Storm surge elevations for the Atlantic Ocean affecting the Mystic River and the Island End River were determined by FEMA. The storm surge elevations were obtained from the FIS for the City of Chelsea (FEMA, 1982).

The results of a mathematical model developed by the RCS were the source of the 10-, 1-, and 0.2-percent annual chance peak discharges for Reservoir No.2 (U.S. Department of Agriculture, 1973). For Cochituate Brook, Reservoir No 1, Reservoir No.1 North Branch, and Reservoir No. 3, discharge-frequency data were obtained using computer modeling techniques developed by the NRCS (U.S. Department of Agriculture, 1972). In this method, a precipitation event is simulated over a basin, runoff is calculated, and the quantity of flow is routed through stream reaches and control structures. The 10-, 2-, 1-, and 0.2-percent annual chance peak discharges were determined by applying the appropriate total rainfall depth associated with a particular event. Most of the watershed was analyzed in this way except for certain diversion areas where manual hand-routing procedures were used.

Originally, 10-, 2-, 1-, and 0.2-percent annual chance discharges for Baiting Brook were obtained from previous hydrologic analyses conducted by the NRCS (U.S. Department of Agriculture, 1957). Discharges for Baiting Brook, and for East Outlet and Birch Meadow Brook, were determined using the NRCS TR-20 hydrologic computer program (U.S. Department of Agriculture, 1965).

The result of a mathematical model developed by the Soil Conservation Service was the source of the 10-, 1-, and 0.2-percent annual chance peak discharges for Whitehall Brook (U.S. Department of Agriculture, 1973). The peak discharges for the 2-percent annual chance flood was obtained graphically using Soil Conservation Service data.

Discharges for Black Brook and Marginal Brook were developed by statistical analysis of available flow data in the region and by use of the regression equations developed by the USGS for Massachusetts (U.S. Department of the Interior, 1977). As there are no stream flow gaging stations on either Black Brook or Marginal Brook, the following four representative gaging stations within the region were used:

Station	Drainage Area (square miles)	Years of Record
Aberjona River At Winchester (Gage No. 01102500)	24.2	43
Nashoba River At Acton (Gage No. 01097300)	12.7	19
Stony Brook At Temple, New Hampshire (Gage No. 01093800)	3.6	19

Station	Drainage Area (square miles)	Years of Record
Richardson Brook Near Lowell (Gage No. 01100100)	4.2	21

Statistical analyses were performed using a log-Pearson Type III distribution (U.S. Department of the Interior, 1981). Discharge frequencies were then transferred from each gage to Black Brook and Marginal Brook by ratio of respective drainage areas to the 0.7 exponential power. Also, discharge frequencies were developed using the reference USGS regression equations. In applying the equations to Black Brook, the following physical parameters were used: drainage area = 3.0 square miles and main channel slope = 10 feet/mile. For Marginal Brook, the parameters were: drainage area = 1.2 square miles and main channel slope = 20 feet/mile. It was determined that the statistical analyses and regression analyses were comparable and the results using the regression equations were adopted for both Black Brook and Marginal Brook.

Discharge-frequencies for Trull Brook Tributary were obtained from the FIS for the Town of Tewksbury (FEMA, 1981). The published discharges agreed with results using the reference regression equations (U.S. Department of the Interior, 1977). Therefore, discharges along the tributary were considered proportional to those on Trull Brook Tributary by ratio of respective drainage area to the 0.7 exponential power.

Discharges for South Meadow Brook/Paul Brook, and Cheese Cake Brook were determined using a method developed by the NRCS (U.S. Department of Agriculture, 1974). The method takes into consideration the type of storm, antecedent moisture conditions, hydrologic soil groups, and topographic characteristics.

No hydrologic routings were performed on lakes in Reading. In Woburn, the Aberjona River enters the Aberjona Holding Pond in the Woburn Industrial Park. A hydrologic reservoir routing of this pond was performed to establish the water-surface elevations during the 10-, 2-, 1-, and 0.2-percent annual chance floods on the Aberjona River upstream.

Discharge-frequency estimates for the 10-, 2-, and 1-percent annual chance floods on Boons Pond and Branch were derived by computations using the NRCS formulation for runoff determination of small watershed (U.S. Department of Agriculture, 1975). This formula takes into account drainage area, watershed land use, 24-hour rainfall, hydraulic length of watershed, land slope, and amount of ponded areas. Peak discharge estimates for the 0.2-percent annual chance flood were determined by extending the frequency curve of the three other floods on log-probability paper.

For Tributary A to Dudley Brook, Mineway Brook, Tributary A to Cold Brook, Tributary A to Pantry Brook, and Tributaries A, B, C, and D to Hop Brook, peak discharges were developed using USGS regression equations for the region (U.S. Department of the Interior, 1993).

Frequency-discharge data for Strong Water Brook were developed by a discharge-drainage area ratio formula, where Q_1 and Q_2 are the discharges at Strong Water Brook and the Shawsheen River, A_1 and A_2 are the drainage areas of the brook and river, respectively and n is an exponent varying from 0.5 to 0.8 (Johnstone and Cross, 1949).

$$Q_{1/} Q_2 = [A_{1/}A_2]^n$$

Peak discharge-frequency relationships for Lawrence Brook and Mascuppic Brook were developed using procedures described by the USGS in Estimating the Magnitude and Frequency of Floods for Natural-Flow Streams in Massachusetts. (U.S. Department of the Interior, 1977). This technique was developed using multiple regression analyses to estimate flood peaks on ungaged, natural-flow streams in Massachusetts by relating peak discharges to basin and climatic parameters. The resulting peak discharges were verified and/or reconciled with statistically analyzed data from nearby stream gages with similar watershed characteristics by using a multiplication factor equal to the ratio of the drainage areas to the 0.75 exponential power. They were found to be in general agreement. A numerical integration reservoir routing of triangular inflow hydrographs was used in conjunction with the previously mentioned procedures for Mascuppic Brook (U.S. Department of the Interior, 1977; Viessman, Jr., 1972). The routing process was incorporated to take into account the effects of storage in Mascuppic Lake, upstream of Mascuppic Brook.

The Saugus River watershed is a complex hydrologic system, containing three major storage areas: Lake Quannapowitt, a large swampy area in Reading; the large swamp north of Route 128 on the Wakefield-Lynnfield line by the Wakefield Industrial Park; and two major tributary streams, the Reading Drainage Canal and Beaverdam Brook.

Because there are no hydrologically similar gaged streams in the area, runoff and flows tributary to Lake Quannapowitt were calculated by methods developed by the NRCS. The NRCS TP-149 (U.S. Department of Agriculture, 1973) and TR-55 (U.S. Department of Agriculture, 1975) are methods of estimating volume and rates of runoff on watersheds. Rainfall data were obtained from the U.S. Weather Bureau Technical Release No. 40 (U.S. Department of Commerce, 1963). The discharges determined by these NRCS methods can then be routed through the lake (Fair and Geyer, 1954). Because of the storage capacity of Lake Quannapowitt, flood flows could be significantly reduced. By calculating a stage-discharge curve for the outlet weir, a stage-discharge-frequency curve was developed for outflows from Lake Quannapowitt. The outflow hydrograph for Lake Quannapowitt, developed for the 10-, 2-, 1-, and 0.2-percent annual chance recurrence intervals, was hydrographically combined with flood flows developed for the Reading Drainage Canal. These flows were routed and again hydrographically combined with flows developed for Beaverdam Brook and the

Pilling Pond outflow. Flows through the swamp were then reduced to take into account the effect of storage provided by the swamp and to obtain outflows over the Saugus River Dam (Lynn Diversion works). Flows over the dam were then combined with flows developed from the incremental drainage area between Water Street and the dam.

Flows were developed for the Mill River, upstream of its confluence with the Saugus River for the 10-, 2-, 1-, and 0.2-percent annual chance recurrence intervals by methods used by the NRCS. In studying the Mill River watershed, it was found that overflows from Crystal Lake are tributary to the Mill River via storm drain pipes. Analysis showed that overflows from the lake reaching Mill River are very infrequent and generally occur long after peak flows on the river; therefore, they would not affect peak flow discharges.

Water-surface elevations of selected recurrence intervals for Forge Pond were computed using a set of empirical equations in conjunction with a set of stage-discharge curves for the multiple outlets of the pond (during periods of significant flooding) (U.S. Department of the Interior, 1962).

The 1-percent annual chance discharges for streams studied by approximate methods in Ashland, Ayer, Framingham, Groton, Hopkinton, Lexington, Marlborough, Townsend, were calculated using regional discharge frequency equations (U.S. Department of the Interior, 1974).

Although Todd Pond Brook was not studied in detail, the 1-percent annual chance peak discharge was calculated using the USGS regional method (U.S. Department of the Interior, 1974). The drainage area at the outlet of Todd Pond is approximately 0.8 square mile, and the 1-percent annual chance peak discharge is approximately 68 cfs.

A summary of the drainage area-peak discharge relationships for the streams studied by detailed methods is shown in Table 5, "Summary of Discharges."

TABLE 5 - SUMMARY OF DISCHARGES

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
ABERJONA RIVER					
Upstream of confluence with Mill Brook 3	28.0	700	1,370	1,820	3,910
At Mid Lake Dam	27.7	700	1,380	1,930	3,710
At USGS gaging station	24.8	730	1,380	1,830	3,510
Downstream of confluence of Horn Pond Brook	24.3	710	1,350	1,800	3,560
Upstream of confluence of Horn Pond Brook	14.5	600	930	1,190	2,410
At Washington Street	12.5	560	900	1,150	2,160
Downstream of confluence of Sweetwater Brook	11.5	520	870	1,080	1,970
Upstream of confluence of Sweetwater Brook	9.0	400	640	820	1,560
Downstream of confluence of Schneider Brook	8.4	380	610	790	1,260
Upstream of confluence of Schneider Brook	7.0	330	520	670	1,030
Downstream of confluence of Halls Brook	5.5	270	460	550	810
Upstream of confluence of Halls Brook	2.5	200	370	480	640
Downstream of confluence of Aberjona River North Spur	2.0	110	190	200	250
Upstream of confluence of Aberjona River North Spur	1.4	40	100	110	120
At West Street/Willow Street culvert	0.9	20	50	80	130
ABERJONA RIVER NORTH SPUR					
At Holding Pond at Woburn Industrial Park	1.91	20	30	38	140
ALEWIFE BROOK (LITTLE RIVER)					
At Cambridge/Somerville corporate limits	8.3	230	360	460	410
ANGELICA BROOK					
At confluence with Reservoir No. 3	1.62	90	140	160	220
ASSABET BRANCH NO. 3					
At confluence with Assabet River	1.1	60	72	99	134

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
ASSABET BRANCH NO. 4					
At confluence with Assabet River	1.0	72	104	118	159
ASSABET RIVER					
At confluence of Sudbury River	176.0	1,980	3,250	4,110	6,500
At confluence of Nashoba Brook	167.0	1,920	3,160	3,990	6,310
At Damondale Dam	118.0	1,600	2,630	3,320	5,260
At Acton/Concord corporate limits	117.0	1,600	2,630	3,320	5,260
Just downstream of Elizabeth Brook 1	112.41	1,580	2,650	3,250	5,080
Just upstream of Elizabeth Brook 1	92.42	1,486	2,610	3,250	5,080
Just downstream of confluence of Boons Pond and Branch	89.71	1,392	2,440	3,042	4,930
Just upstream of confluence of Boons Pond and Branch	87.35	1,386	2,430	3,030	4,920
Just downstream of confluence of Fort Meadow Brook	86.89	1,385	2,430	3,030	4,920
Just upstream of confluence of Fort Meadow Brook	80.01	1,223	2,140	2,670	4,330
Just downstream of confluence of Branch of Assabet River	79.05	1,221	2,140	2,664	4,320
At the Hudson/Stow corporate limits	76.3	1,153	2,020	2,520	4,090
Downstream of confluence of Danforth Brook	72.7	1,090	1,910	2,380	3,860
Downstream of confluence of Hog Brook	63.6	927	1,610	1,998	3,210
At Interstate Route 495	58.5	850	1,490	1,850	2,990
At Baker Avenue	0.1	45	85	100	120
BADDACOOK BROOK					
At confluence of Whitney Pond	2.05	190	520	590	1,080
BAITING BROOK					
At confluence with Sudbury River	3.58	288	425	488	625
At Constance M. Fiske Dam	1.92	68	77	80	87

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
BEAR MEADOW BROOK					
At confluence with Ipswich River	4.65	160	270	330	500
Approximately 1,200 feet downstream of Haverhill Street	3.99	140	230	280	430
Approximately 680 feet upstream of Haverhill Street	2.30	100	160	190	290
Approximately 2,700 feet upstream of Haverhill Street	1.35	57	96	120	180
BEAVER BROOK 1					
At confluence with Charles River	11.5	1,020	1,370	1,800	2,450
Upstream of confluence of Chester Brook	7.8	670	900	1,180	1,600
Upstream of Beaver Street Bridge	6.9	570	760	1,000	1,375
Upstream of Beaver Street Bridge	5.89	570	760	1,000	1,375
Approximately 15 feet downstream of Linden Street/ State Route 60	1.79	129	234	289	477
BEAVER BROOK 2					
At confluence with River Meadow Brook	5.8	170	225	260	330
BEAVER BROOK 3					
At Dracut/Lowell corporate limits	96.5	1,850	3,500	4,200	6,650
Upstream of confluence of Peppermint Brook	93.7	1,800	3,400	4,050	6,450
Upstream of Lakeview Avenue	87.4	1,710	3,220	3,830	6,090
Upstream of confluence of Gumpas Pond Brook	83.7	1,690	3,190	3,790	6,080
BEAVER BROOK 4					
At inlet to Forge Pond	13.6	420	690	845	1,280
Downstream of Westford/ Littleton corporate limits	11.8	380	620	760	1,150
Approximately 200 feet upstream of King Street	9.8	339	563	686	1,045
Upstream of Mill Pond	7.9	296	494	601	920
At State Route 2	5.8	241	403	493	756
At Boxborough/Littleton corporate limits	4.3	145	215	250	330

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
BEAVER BROOK 4 (continued)					
Approximately 7,280 feet upstream of Captain Isaac Davis Highway/State Route 2	3.0	92	140	160	220
Approximately 3,260 feet downstream of West Whitcomb Road	1.9	66	100	120	150
At Interstate Route 495	1.4	55	84	98	120
BEAVER BROOK 5					
At State Route 2	2.08	147	269	334	554
At Cross Section H	1.79	129	234	289	477
BEAVERDAM BROOK					
At River Mile 0.0	5.6	221	369	450	690
At River Mile 1.733	3.5	183	309	378	586
At the Framingham/Natick corporate limits	5.54	180	310	380	590
At the Ashland/Framingham corporate limits	1.0	180	310	380	590
BENNETTS BROOK					
At the Ayer/Littleton corporate limits	4.93	180	280	330	440
Approximately 1,500 feet downstream of Shaker Mille Pond	2.82	120	180	210	280
BIRCH MEADOW BROOK					
At confluence with East Outlet	1.0	50	80	99	149
BLACK BROOK					
At confluence with Merrimack River	3.0	120	200	250	380
BOGASTOW BROOK-JAR BROOK					
At county boundary	12.43	690	1,000	1,400	2,200
Upstream of confluence of Dirty Meadow Brook	9.77	540	770	1,050	1,600
Upstream of confluence of Dopping Brook	6.7	450	610	800	1,170
Upstream of Factory Pond	2.85	270	420	540	800
Upstream of Houghton Pond	2.45	250	400	500	750
At Meadowbrook Lane	0.41	50	80	100	150

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
BOGLE BROOK 1					
At the county boundary	1.9	50	82	99	151
At State Route 9	1.0	32	54	65	101
BOGLE BROOK 2					
At county boundary	3.58	325	490	630	1,000
Upstream of confluence of Tributary 4 to Bogle Brook 2	3.05	325	490	630	930
At Nonesuch Pond Outlet	2.82	300	460	600	890
At Nonesuch Pond Inlet	2.42	280	430	560	840
Upstream of confluence of Tributary 3 to Bogle Brook 2	1.35	200	260	350	510
At Pine Street	1.1	150	200	270	400
BOONS POND AND BRANCH					
At Barton Road	2.31	120	285	350	667
BOUTWELL BROOK					
At confluence with Stony Brook 2	1.3	90	150	180	280
BOW BROOK					
At confluence with Catacoonamug Brook	2.41	110	180	200	280
BRANCH OF ASSABET RIVER					
Approximately 1,380 feet downstream of Hudson Road/Walcott-Randall Road	4.23	186	294	347	505
At Hudson Road/Walcott-Randall Road	1.45	78	118	135	188
At Goshen Lane	1.09	60	91	104	145
At Athens Street	1.01	57	78	98	136
BRANCH OF ELIZABETH BROOK 1					
At confluence of Ministers Pond	1.08	84	113	125	175
BROAD MEADOW BROOK					
At confluence with Sudbury Reservoir	1.1	70	100	110	150
BROOK A OF SHAWSHEEN RIVER					
At confluence with Shawsheen River	0.7	65	120	150	255

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
BROOK FROM WAUSHAKUM POND					
At confluence with Beaverdam Brook	2.9	30	40	50	60
BUTTER BROOK					
At confluence with Nashoba Brook	3.1	105	180	225	440
At Acton/Westford corporate limits	1.4	65	110	140	275
At Concord Road	1.0	60	120	160	270
At Griffin Road	0.4	30	50	70	110
CATACOONAMUG BROOK					
At confluence with the Nashua River	20.3	670	1,150	1,400	2,080
CHARLES RIVER					
At the Waltham/Newton/ Watertown corporate limits	250.6	2,200	3,200	3,700	5,200
At Moody Street Dam	250.6	2,200	3,200	3,700	5,200
At Waltham Gaging Station	227.0	2,200	2,900	3,500	4,500
At diversion from Mother Brook	200.0	1,780	2,480	3,200	4,270
At confluence of Mother Brook	200.0	2,650	3,610	4,680	6,210
At the Charles River Village/ Dover Gage (No. 01103500)	184.0	2,500	3,500	4,500	6,000
At Natick/Sherborn corporate limits	176.0	2,500	3,500	4,500	6,000
At Medfield	156.0	2,450	3,430	4,410	5,925
CHEESE CAKE BROOK					
At Eddy Street	2.0	410	680	800	1,080
CHERRY BROOK					
At confluence with Stony Brook 1	3.24	400	500	700	1,080
At Concord Road	2.03	310	350	550	800
CHESTER BROOK					
At its confluence with Beaver Brook 1	3.67	300	460	600	850
At the confluence of West Chester Brook	1.74	200	300	400	450
At the lower end of the Lexington Street culvert	0.53	100	150	200	300

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
CHICKEN BROOK					
At county boundary	2.98	150	250	300	450
Upstream of Waseeka Wildlife Dam	0.22	40	50	60	80
COCHITUATE BROOK					
At confluence with Sudbury River	18.21	420	690	800	1,100
COLD BROOK					
At confluence of Pantry Brook	2.1	120	190	230	345
Above confluence of Tributary A to Cold Brook	0.41	40	65	80	125
COLD SPRING BROOK					
Downstream of Ashland Reservoir at Chestnut Street	8.54	550	1,030	1,250	1,610
Upstream of Ashland Reservoir	5.68	250	370	430	570
At the Ashland/Hopkinton corporate limits	5.48	250	370	430	570
COLE'S BROOK					
At School Street	1.83	275	455	530	655
At Brucewood Road	1.46	275	455	530	655
At confluence of Tributary 1 to Cole's Brook	1.13	225	370	430	530
COLLINS BROOK					
At confluence with Sutton Brook	0.5	55	100	130	220
At Pringle Street	0.2	40	65	85	145
CONANT BROOK					
At confluence with Nashoba Brook	2.22	290	490	550	630
At Nagog Hill Road	1.16	200	330	370	430
CONCORD RIVER					
At Billerica/Chelmsford corporate limits	373.0	3,105	4,924	5,995	8,870
At the Billerica/Tewksbury corporate limits	373.0	3,100	4,900	6,000	8,900
At Talbot Mill Dam	370.0	2,940	4,660	5,675	8,395
At U.S. Route 3 Bridge, In Billerica	363.0	2,885	4,577	5,575	8,245

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
CONCORD RIVER (continued)					
At the Billerica/Carlisle corporate limits	360.0	2,885	4,577	5,575	8,245
At Concord/Carlisle corporate limits	352.0	2,950	4,680	5,700	8,430
CONTENT BROOK – MIDDLESEX CANAL					
At confluence with Shawsheen River	3.3	145	260	330	560
At Billerica/Tewksbury corporate limits	5.8	205	370	455	585
At Gray Street	4.9	180	330	400	520
Just upstream of confluence of Content Brook and Middlesex Canal	2.2	95	175	210	275
COURSE BROOK					
At Sherborn/Natick corporate limits	2.77	200	350	430	700
Upstream of confluence of Tributary A to Course Brook	2.3	170	300	370	600
COW POND BROOK					
At the abandon railroad	9.3	210	510	570	980
At the outlet from Whitney Pond	7.3	50	100	110	195
At the outlet from Lost Lake	4.84	30	50	50	70
CUMMINGS BROOK					
Upstream of confluence with Shakers Glen Brook	3.4	120	230	330	690
Downstream of confluence of Little Brook	2.8	90	190	260	600
Upstream of confluence of Little Brook	1.5	40	110	140	320
At Winn Street	1.2	30	70	90	170
DAKINS BROOK					
At Lowell Road	0.5	120	215	250	275
DANFORTH BROOK					
At confluence with Assabet River	6.4	236	379	450	664
At county boundary	4.9	176	274	320	458

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
DARBY BROOK					
At confluence with Marshall Brook	0.6	35	70	90	145
DAVIS BROOK					
At confluence with Charles River	1.9	109	185	227	353
DIRTY MEADOW BROOK					
At confluence with Bogastow Brook	2.45	140	230	340	570
DOPPING BROOK					
At confluence with Bogastow Brook	2.12	130	180	240	350
DUDLEY BROOK/TRIBUTARY TO DUDLEY BROOK					
At confluence with Hope Brook	2.26	110	160	190	250
Approximately 1,000 feet downstream of Bent Road	1.06	75	125	150	225
At U.S. Route 20	0.27	30	50	60	100
EAST OUTLET					
At confluence with Sudbury River	2.24	121	192	237	356
ELIZABETH BROOK 1					
Approximately 1,500 feet downstream of Box Mill Road	18.26	473	803	967	1,390
At Fletcher Road	17.99	462	787	949	1,367
At Gleasondale Road	17.8	446	760	918	1,324
At Wheeler Dam	16.88	367	632	768	1,113
At Great Road	15.89	289	487	588	844
At Hiley Brook Road	15.4	244	397	478	759
At Delaney Road	14.92	200	308	371	674
ELIZABETH BROOK 2					
At county boundary	1.6	100	160	190	290
Approximately 140 feet downstream of Rushwood Road	1.1	80	130	155	235
Approximately 470 feet downstream of Massachusetts Avenue/State Route 111	0.7	60	100	120	180

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
ELM BROOK					
At confluence with Shawsheen River	4.6	200	270	355	527
FARRAR POND/ POLE BROOK					
At Sudbury River	2.19	100	147	169	237
At confluence with Farrar Pond/ Pole Brook confluence	1.04	54	80	92	129
At Concord Road	0.54	38	57	65	91
FARRAR POND BROOK					
At confluence with Farrar Pond	1.06	51	75	85	110
FORT MEADOW BROOK					
At Chestnut Street	4.6	245	385	450	649
At Fort Meadow Reservoir	3.3	169	252	289	399
FORT POND BROOK					
At confluence with Nashoba Brook	24.6	570	850	975	1,250
At Laws Brook Road	24.7	570	850	980	1,250
At Merriam Dam	24.3	565	850	975	1,245
At Erikson Dam	20.5	555	840	965	1,235
At confluence of Heath Hen Meadow Brook	19.8	545	835	955	1,220
At Boston & Main Railroad near Elm Street	10.1	375	650	790	1,210
Upstream of confluence of Inch Brook	4.4	130	230	285	520
At Boxborough/Acton corporate limits	2.8	97	148	175	235
Approximately 990 feet upstream of Littlefield Road	2.6	90	138	165	220
FORT POND BROOK BRANCH 1					
At Rockland Street	1.1	50	73	82	111
FORT POND BROOK BRANCH 2					
At confluence with Fort Pond Brook	4.3	143	215	260	350
At Boston and Maine Railroad, Southern Crossing	4.2	140	213	255	340
At Sargent Road	3.0	103	165	190	255

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
GRASSY POND BROOK					
At confluence with Fort Pond Brook	1.6	70	120	150	290
GRAVES POND BROOK					
At the outlet of Graves Pond	1.49	90	150	170	240
GREAT ROAD TRIBUTARY					
At the confluence with Beaver Brook 4	0.4	46	81	100	159
At Great Road	0.1	17	30	37	58
Approximately 290 feet upstream of Interstate Route 495	0.05	9	16	20	32
GREENS BROOK					
At confluence with Varnum Brook	0.54	50	80	100	155
GUGGINS BROOK					
At confluence with Inch Brook	4.6	140	240	295	540
At Boxborough/Acton corporate limits	2.2	77	118	143	190
Approximately 3,340 feet downstream of Liberty Square Road	2.1	74	115	135	185
At Liberty Square Road	1.8	64	98	120	160
At eastern crossing of Massachusetts Avenue	1.5	59	84	100	135
Approximately 560 feet upstream of Massachusetts Avenue	1.0	39	60	73	98
GUMPAS POND BROOK					
At Dracut/Pelham corporate limits	3.7	200	345	425	715
HALES BROOK					
At confluence with River Meadow Brook	1.8	65	90	102	130
HALLS BROOK					
Upstream of confluence with Aberjona River	3.0	70	90	80	170
Downstream of Boston and Maine Railroad	2.6	100	180	240	370
Upstream of Boston and Maine Railroad	2.1	70	130	170	270
At Merrimac Street and School Street	0.3	20	40	50	90

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
HAYWARD BROOK					
At confluence with Pine Brook	3.39	161	260	312	442
At Boston Post Road	2.32	83	140	175	272
At private drive bridge	1.48	62	104	130	202
HEATH BROOK					
At confluence with Shawsheen River	2.0	110	200	260	445
Approximately 3,600 feet upstream of Shawsheen Street	1.1	80	150	190	325
At Foster Road	0.9	70	130	165	290
HOBBS BROOK 1					
At confluence with Stony Brook 1	24.7	300	400	525	775
At inlet to pond upstream of North Avenue	8.61	280	380	500	730
At Weston/Waltham corporate limits	7.2	150	200	260	390
HOBBS BROOK 2					
At Lexington Road	2.35	97	145	167	221
HOG BROOK					
At confluence with Assabet River	3.5	214	341	400	583
HOP BROOK					
At confluence with Landham-Allowance Brook	15.58	470	770	920	1,300
Above confluence of Dudley Brook	11.65	390	630	750	1,050
Above confluence of Run Brook	9.16	320	530	630	890
At Dutton Road	3.54	160	260	310	440
At the Sudbury/Framingham corporate limits	2.0	180	280	320	440
At the Marlborough/Sudbury corporate limits	1.32	160	260	310	435
HORN POND BROOK/ FOWLE BROOK					
At confluence with Aberjona River	9.8	200	430	610	1,240
At Horn Pond Dam	8.8	180	400	570	1,080
Downstream of confluence of Cummings Brook and Shakers Glen Brook	6.2	170	350	490	910

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
INCH BROOK					
At confluence with Fort Pond Brook	5.7	155	270	335	615
INDIAN BROOK					
Approximately 1,150 feet downstream of the confluence of Tributary to Indian Brook	2.77	170	270	320	430
IPSWICH RIVER					
At downstream North Reading/ Reading corporate limits	18.4	360	520	600	830
At Reading/North Reading/ Wilmington corporate limits	14.7	320	480	560	760
Upstream of confluence of Lubbers Brook	8.9	150	250	320	550
Downstream of confluence of Maple Meadow Brook	8.4	280	450	540	820
Upstream of confluence of Maple Meadow Brook	2.7	50	80	110	190
Approximately 1,050 feet downstream of Adams Street Culvert	2.6	130	220	260	390
Downstream of Clark Street	2.0	100	160	200	300
At the Burlington/Wilmington corporate limits	1.1	70	120	140	220
JAMES BROOK					
At confluence with Nashua River	4.9	180	290	340	460
At the Ayer/Groton corporate limits	2.9	130	210	240	340
At Old Ayer Road	2.6	110	170	240	340
At Indian Hill Road	1.72	80	120	140	200
At Ayer Road	1.1	50	80	100	140
JONES BROOK					
At confluence with Shawsheen River	1.7	215	380	450	540
At Golf Course Culvert	1.6	195	355	425	510
At Baldwin Road	1.3	160	290	345	415
KILN BROOK					
Approximately 2,700 feet downstream of Interstate Route 95	1.73	177	350	450	803
At State Route 128/ Interstate Route 95	1.07	125	243	312	550

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
KING STREET TRIBUTARY					
At confluence with Beaver Brook 4	0.5	49	85	104	166
Approximately 1,500 feet upstream of King Street	0.3	39	68	84	136
LANDHAM-ALLOWANCE BROOK					
At Landham Road	21.0	580	940	1,130	1,590
At the Sudbury/Framingham corporate limits	2.0	180	280	330	450
LAWRENCE BROOK					
At confluence with Merrimack River	3.42	110	170	200	305
Upstream of confluence of Mascuppic Brook	0.7	65	110	135	210
LITTLE BROOK					
Upstream of confluence with Cummings Brook	1.4	50	100	140	360
At Bedford Road	1.1	40	80	120	230
LOCKE BROOK					
At confluence with Willard Brook	4.66	340	540	640	890
LOWER SPOT POND BROOK					
At the intake to Winter Street in Malden	6.0	640	900	1,060	1,480
LUBBERS BROOK					
Upstream of confluence with Ipswich River	5.5	160	220	270	410
Upstream of Middlesex Avenue	4.5	140	200	240	360
Upstream of Boston & Maine Railroad bridge at Lawrence Street	3.4	180	270	310	450
At Glen Road	3.18	135	215	250	350
At State Route 38 (Main Street)	2.76	80	115	130	180
At State Route 129 (Shawsheen Avenue)	2.02	90	135	150	290
Approximately 2,200 feet upstream of Shawsheen Avenue/ State Route 29	1.46	90	150	180	275
At Billerica/Wilmington corporate limits	1.3	63	106	129	200
At Billerica/Burlington corporate limits	0.7	47	80	98	153

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
MALDEN RIVER					
At the upstream side of the Amelia Earhart Dam	61.9	4,000	4,000	4,000	4,000
At Medford/Everett corporate limits	9.2	850	1,200	1,410	1,890
MAPLE MEADOW BROOK					
Upstream of confluence with Ipswich River	5.7	230	360	430	620
Upstream of tributary, approximately 525 feet northeast of Paddock Street	5.6	190	290	350	520
Upstream of Main Street bridge	4.1	160	260	310	460
Approximately 1,300 feet upstream of Middlesex Canal	1.5	30	50	60	90
MARGINAL BROOK					
At confluence with Concord River	1.2	70	115	140	220
MARSHALL BROOK					
At confluence with Strong Water Brook	4.0	180	295	350	515
Upstream of confluence of Darby Brook	3.2	150	240	290	425
Upstream of the tributary at station 1.075	2.6	105	170	205	300
MARTINS BROOK					
At the Wilmington/North Reading corporate limits	10.9	460	700	830	1,190
Approximately 2,000 feet downstream of State Route 62 (Salem Street)	10.3	370	570	670	980
MARTINS POND BROOK					
At confluence with Lost Lake	2.11	90	130	150	200
MASCUPPIC BROOK					
At confluence with Lawrence Brook	2.35	50	70	80	115
MASON BROOK					
At the confluence with Walker Brook 2	7.42	320	540	660	940

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
MEADOW BROOK					
At confluence with Strong Water Brook	5.1	260	425	510	760
Upstream of tributary at Station 1.145	2.6	145	240	285	425
MEADOW RIVER BRANCH					
At Lowell Street	7.9	266	441	537	819
At Curve Street	4.4	177	296	361	554
MERRIMACK RIVER					
At mouth	4,180.0	54,000	85,000	102,000	145,000
At the Andover/Tewksbury corporate limits	4,635.0	58,000	90,000	111,000	156,000
At Dracut/Methuen corporate limits	4,644.0	58,000	90,000	111,000	156,000
At Nashua, New Hampshire (State Route 111)	3,982.0	53,000	85,000	102,000	148,000
MILL BROOK 1					
At confluence with Pine Brook	5.0	70	90	100	130
At Lexington/Wayland corporate limits	1.32	132	225	322	558
At Fottler Avenue	1.03	107	208	264	461
MILL BROOK 2					
At Lang Street	3.8	275	495	570	670
At Cambridge Turnpike (State Route 2A)	2.6	116	188	227	322
At confluence of Crosby Pond	0.8	62	101	123	180
MILL BROOK 3					
At Mystic Valley Parkway	5.5	210	370	480	900
At Mill Street	5.0	150	310	450	730
At Brattle Street	4.3	120	210	260	820
At Park Avenue	3.6	80	150	200	390
MILL POND TRIBUTARY					
At confluence with Beaver Brook 4	0.9	37	62	76	117
Upstream of Boston & Maine Railroad	0.5	18	29	36	55

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
MILL RIVER					
At confluence with Saugus River	3.6	150	260	300	350
At Water Street bridge above Crystal Lake Storm Drain	0.8	75	130	145	170
At Salem Street bridge	0.35	34	58	66	78
MINEWAY BROOK					
At confluence with Pantry Brook	1.54	100	160	190	285
Approximately 1,500 feet upstream of confluence with Pantry Brook	1.25	85	140	165	250
Approximately 1,500 feet upstream of Morse Road	0.95	70	115	140	210
Approximately 3,100 feet upstream of Morse Road	0.74	60	100	120	180
At Abandon Railroad line	0.41	40	65	80	125
At Concord Road and Candy Hill Road Intersection	0.25	30	50	60	95
MONGO BROOK					
At confluence with Elm Brook	0.59	26	41	50	63
MORSE BROOK					
At confluence with the Nashua River	1.0	50	70	80	100
MOWRY BROOK					
At confluence with the Sudbury Reservoir	1.59	50	70	80	110
MUD POND BROOK					
At confluence with Shawsheen River	0.3	45	80	100	175
MULPUS BROOK					
At confluence with the Nashua River	15.9	720	1,740	1,950	3,440
At Townsend Road Culvert	13.95	810	1,920	2,140	3,820
MUNROE BROOK					
At Lexington/Arlington corporate limits	2.16	179	345	434	754
At Lilian Road	2.01	165	313	399	665
At Trail	1.45	130	242	302	511
At Bryant Road	1.02	100	188	238	359

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
MYSTIC RIVER					
At confluence with Maiden River	62.9	1,150	2,130	2,530	3,700
Downstream of confluence of Alewife Brook (Little River)	43.7	990	1,840	2,110	3,520
Upstream of confluence of Alewife Brook (Little River)	34.8	800	1,560	2,040	4,250
NAGOG BROOK					
At confluence with Nashoba Brook	2.4	70	120	155	310
At Nagog Pond outlet	1.2	12	16	18	27
NASHOBA BROOK					
At confluence of Fort Pond Brook	20.3	450	710	845	1,140
At State Route 27	11.8	410	695	840	1,340
Upstream of confluence of Butter Brook	8.7	340	590	715	1,130
NASHUA RIVER					
At the Massachusetts/New Hampshire State Line	396.0	8,300	14,300	17,800	28,300
At the Dunstable/Groton corporate limits	390.0	8,400	15,400	19,800	33,900
At confluence of Nissitissit River	352.0	7,055	11,945	14,651	22,829
At Fitch Bridge Road	312.6	6,950	11,700	14,400	22,600
At confluence of Squannacook River	220.5	5,850	9,900	12,500	19,200
At confluence of Mulpus Brook	204.5	5,650	9,600	12,200	18,600
At confluences of Walker Brook 1 and Nonacoicus Brook 1	183.9	5,400	9,100	11,600	18,000
At confluence of Catacoonamug Brook	161.0	5,100	8,600	11,800	17,000
NISSITISSIT RIVER					
At confluence with the Nashua River	59.76	1,497	2,642	3,642	5,000
NONACOICUS BROOK 1					
At confluence with Nashua River	18.36	840	2,120	2,370	4,160
At Main Street	160.7	400	670	720	1,070

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
NONACOICUS BROOK 2					
At confluence with Nonacoicus Brook 1	10.95	370	980	1,120	2,230
NORTH LEXINGTON BROOK					
At Bedford/Lexington corporate limits	4.92	396	817	1,072	1,986
At Hartwell Avenue	3.15	273	548	708	1,217
Approximately 1,260 feet downstream of Interstate 95 Interchange	1.71	168	330	421	746
At Interstate 95 Interchange	1.0	100	183	235	395
PAGES BROOK					
At confluence with Concord River	4.0	171	286	349	538
At Maple Street	1.8	95	162	199	309
PAGES BROOK BRANCH					
At Brook Street	1.4	265	350	384	472
At East Street	0.8	230	300	334	410
PANTRY BROOK					
At confluence with the Sudbury River	6.04	240	380	450	670
Above confluence of Mineway Brook	1.06	75	125	150	225
Above confluence of Tributary A to Pantry Brook	0.3	35	55	70	105
PEARL HILL BROOK					
At confluence with Walker Brook 2	7.01	280	460	550	790
PEPPERMINT BROOK					
At confluence with Beaver Brook 3	2.4	140	240	300	460
At Pleasant Street	2.3	130	230	290	440
At Hildreth Street	2.0	120	200	250	380
PINE BROOK					
At confluence of Mill Brook 1	5.84	220	340	440	540
At confluence of Hayward Brook	3.97	160	251	294	400

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
PRATTS BROOK					
At High Street	1.45	180	275	315	375
PUTNAM BROOK					
At confluence with River Meadow Brook	0.8	45	60	70	90
REEDY MEADOW BROOK					
At confluence with the Nashua River	2.72	124	190	220	299
At the Groton/Pepperell corporate limits	2.0	120	190	220	300
RESERVOIR NO. 1 - NORTH BRANCH AND RESERVOIR NO. 3					
At Salem End Road	27.68	1,220	2,130	5,130	3,420
At the outlet of Reservoir No. 3	27.68	1,220	2,130	2,490	3,420
At the county boundary	22.28	1,170	2,020	2,360	3,200
RICHARDSON BROOK					
At confluence with Merrimack River	4.5	210	330	390	570
Upstream of confluence of Trout Brook 1	1.7	90	150	180	260
RIVER MEADOW BROOK					
At Chelmsford/Lowell corporate limits	22.0	555	870	1,030	1,450
At confluence of Beaver Brook 2	13.2	400	625	740	1,015
At confluence of Putnam Brook	12.4	380	600	715	980
At confluence of Farley Brook	11.1	355	560	665	910
RUN BROOK					
At the confluence of Hop Brook	0.56	50	80	100	150
Downstream of Hudson Road	0.43	40	70	85	130
Downstream of Fairbank Road	0.14	20	35	45	70

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
SALMON BROOK					
At the Dunstable/Nashua, New Hampshire, corporate limits	22.4	550	920	1,120	1,620
Above confluence of Joint Grass Brook	17.7	190	320	390	605
Above confluence of Hauk Brook	13.4	180	300	365	575
SANDY BROOK					
At the Sandy Brook Road	0.95	360	680	900	1,300
At Maude Graham Circle	0.74	215	420	545	790
At Bedford Street	0.46	140	275	365	575
SAUGUS RIVER					
At county boundary	15.7	340	570	655	840
At Water Street bridge	12.1	230	380	435	595
Above confluence of Montrose Avenue Tributary	11.3	115	185	215	340
At State Route 128 upstream crossing	5.4	190	310	330	395
Above confluence of Reading Drainage Canal	1.8	35	50	57	65
SAUNDERS BROOK					
At confluence with Shawsheen River	2.7	130	235	300	515
At Wilmington/Burlington corporate limits	1.5	95	175	220	380
SAWMILL BROOK 1					
At Wilmington/Tewksbury corporate limits	1.62	354	595	743	1,250
At Mill Street	1.47	350	585	732	1,220
SAWMILL BROOK 2					
At Monument Street	2.5	285	500	550	600
SCHNEIDER BROOK					
Upstream of confluence with Aberjona River	1.4	60	110	140	260
At Forbes Street	0.7	30	50	60	110
SHAKERS GLEN BROOK					
Upstream of confluence of Cummings Brook	2.7	60	130	180	340
Upstream of Russell Street culvert	2.5	50	110	160	290

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
SHAWSHEEN RIVER					
At the northern Andover/ Tewksbury corporate limits	60.1	1,350	2,015	2,340	3,300
At the southern Andover/ Tewksbury corporate limits	58.7	1,325	1,980	2,300	3,240
Downstream of confluence of Strong Water Brook	54.5	1,260	1,875	2,170	3,070
Upstream of confluence of Strong Water Brook	43.9	1,060	1,580	1,840	2,585
Downstream of confluence of Mud Pond Brook	41.1	1,010	1,515	1,765	2,480
Downstream of confluence of Content Brook	37.0	930	1,400	1,610	2,280
At State Road (SR 129)	36.5	1,115	1,825	2,200	3,285
At the Billerica/Tewksbury corporate limits	35.3	1,000	1,350	1,500	1,850
Above confluence of Jones Brook	33.0	1,040	1,710	2,060	3,070
At Boston Road (SR3A)	31.2	1,020	1,650	1,985	2,960
At Bedford/Billerica corporate limits	27.2	1,000	1,500	1,800	2,660
Upstream of confluence of Vine Brook	16.5	885	1,410	1,695	2,500
Upstream of confluence of Spring Brook	13.4	830	1,320	1,590	2,345
Upstream of confluence of Elm Brook	8.1	665	1,056	1,276	1,875
Upstream of confluence of Kiln Brook	2.3	280	440	530	650
SKUG RIVER					
At confluence with Martins Pond	6.9	557	991	1,171	1,642
At Central Street	6.2	535	909	1,082	1,525
SNAKE BROOK					
At confluence with Lake Cochituate	2.52	120	180	210	280
SOUTH MEADOW BROOK-PAUL BROOK					
At Tower Road	2.8	605	1,045	1,265	1,620
At Dedham Street	2.0	480	835	1,015	1,315
At Mildred Road	0.9	285	520	615	845
SPENCER BROOK 1					
At Barrett's Mill Road	6.2	225	350	410	460

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
SPENCER BROOK 2					
At Concord/Carlisle corporate limits	1.5	82	139	171	265
SPRING BROOK					
At confluence with Shawsheen River	2.7	110	125	130	150
Upstream of Alcott Street	0.6	34	45	60	70
SQUANNACOOK RIVER					
At confluence with the Nashua River	62.8	3,540	6,880	8,840	15,160
At Elm Street	51.48	2,950	5,740	7,380	12,650
At Mason Road	42.32	2,620	5,090	6,550	11,230
STONY BROOK 1					
At confluence with the Charles River	24.68	300	400	500	700
At the inlet to Stony Brook Reservoir	22.85	1,140	1,520	2,000	2,950
Upstream of confluence of Hobbs Brook 1	11.38	890	1,200	1,560	2,300
At confluence of Iron Mine Brook	2.3	95	144	167	230
At Tower Road	1.2	54	82	95	131
At Brooks School	1.0	45	68	79	96
STONY BROOK 2					
At confluence with the Merrimack River	43.2	915	1,320	1,490	1,835
STRONG WATER BROOK					
At confluence with the Shawsheen River	10.2	345	515	595	840
SUDBURY RIVER					
At Fairhaven Bay Outlet	158.4	1,770	2,880	3,340	4,930
At Lincoln/Concord/ Wayland corporate limits	153.78	2,500	3,860	4,490	5,590
Downstream of Sherman Bridge Road	146.21	2,570	3,970	4,610	6,090
Downstream of Old Sudbury Road	140.95	2,810	4,330	5,080	6,800
Downstream of Boston Post Road	140.07	2,830	4,440	5,235	7,030
Downstream of Pelham Island Bridge	116.89	2,850	4,500	5,310	7,130

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
SUDBURY RIVER (continued)					
Downstream of Stone Bridge Road	107.9	2,990	4,740	5,590	7,500
At Sudbury/Wayland corporate limits – downstream of Potter Road	107.6	2,990	4,740	5,590	7,500
At Saxonville Dam	83.38	2,580	4,060	4,810	6,410
At Reservoir No. 1	74.66	2,850	4,300	5,130	6,890
At Reservoir No. 2	44.19	2,650	4,130	4,930	6,590
At Myrtle Street	34.23	2,420	3,790	4,480	6,040
At Conrail tracks near Howe Street	23.45	1,680	2,630	3,070	4,130
At confluence of Concord River	166.0	2,030	3,220	3,840	5,670
At Fairhaven Bay Outlet	158.0	1,770	2,880	3,340	4,930
At State Route 85	21.8	1,100	1,740	2,020	2,660
At the Massachusetts Turnpike	17.52	450	740	820	1,090
SUTTON BROOK					
At confluence with Shawsheen River	2.7	130	235	300	515
At Wilmington/Tewksbury corporate limits	1.5	95	175	220	380
SWEETWATER BROOK					
Upstream of confluence with Aberjona River	2.4	210	400	530	970
At I-93	2.3	200	400	530	960
At Lindenwood Road	1.9	170	350	470	840
TADMUCK BROOK					
At confluence with Stony Brook 2	2.1	120	210	250	400
At Main Street	1.6	90	160	190	300
At Providence Road	1.0	50	90	110	170
TADMUCK SWAMP BROOK					
At Westford/Chelmsford corporate limits	1.8	110	190	230	360
At Interstate Route 495	1.2	90	150	180	290
TAYLOR BROOK					
At confluence with Assabet River	4.56	102	136	152	200
TRIBUTARY 1 TO COLE'S BROOK					
At Arborwood Road	0.17	50	95	115	155

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
TRIBUTARY 1 TO SUDBURY RIVER					
At Coolidge Road	0.3	65	126	145	175
TRIBUTARY 2 TO ASSABET RIVER					
At Baker Avenue	0.1	45	85	100	120
TRIBUTARY 2 TO TRIBUTARY 1 TO COLE'S BROOK					
At Fernwood Road	0.16	120	180	200	215
TRIBUTARY 3 TO BOGLE BROOK 2					
At confluence with Bogle Brook 2	1.0	120	175	240	380
TRIBUTARY 4 TO BOGLE BROOK 2					
At confluence with Bogle Brook 2	0.53	80	120	170	250
TRIBUTARY A TO COLD BROOK					
At confluence with Cold Brook	0.64	55	90	110	165
At Abandon Railroad line	0.34	35	60	75	110
TRIBUTARY A TO COURSE BROOK					
At confluence with Course Brook	0.68	90	140	180	270
TRIBUTARY A TO HOP BROOK					
At confluence with Hop Brook	0.61	50	90	105	160
At Firecut Lane	0.09	20	35	45	70
TRIBUTARY A TO SQUANNACOOK RIVER					
At confluence with the Squannacook River	2.54	130	200	230	310

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
TRIBUTARY B TO HOP BROOK					
At confluence with Hop Brook	0.36	40	60	75	115
TRIBUTARY B TO SQUANNACOOK RIVER					
At confluence with the Squannacook River	0.4	30	50	50	70
TRIBUTARY B TO VINE BROOK					
At Middlesex Street	0.71	425	590	685	960
At Third Avenue	0.56	100	130	150	195
At US Route 3	0.49	70	85	95	115
TRIBUTARY C TO HOP BROOK					
At confluence with Hop Brook	1.76	105	170	205	310
TRIBUTARY C TO VINE BROOK					
At Wheeler Road	0.70	195	300	360	560
At Muller Road	0.52	170	340	465	780
TRIBUTARY D TO HOP BROOK					
At confluence with Hop Brook	1.88	110	180	215	325
TRIBUTARY TO BEAVER BROOK 3					
At confluence with Beaver Brook 3	1.2	80	140	175	275
TRIBUTARY TO COLD SPRING BROOK					
At confluence with Cold Spring Brook	1.17	80	110	130	170
TRIBUTARY TO INDIAN BROOK					
At confluence with Indian Brook	1.0	70	110	130	180

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
TRIBUTARY TO MARTINS BROOK					
At confluence with Martins Brook	1.5	94	153	185	250
TRIBUTARY TO MILL BROOK					
At confluence with Mill Brook	1.1	60	80	105	155
TRIBUTARY TO NONACOICUS BROOK 1/ LONG POND BROOK					
At the confluence with Nonacoicus Brook 1	4.09	120	150	160	240
At Snake Hill Road	2.24	30	60	70	120
TRIBUTARY TO WAUSHAKUM POND					
At the south end of Waushakum Pond	1.49	100	140	150	200
TROUT BROOK 1					
At confluence with Richardson Brook	2.6	140	220	270	390
TROUT BROOK 2					
At confluence with the Nashua River	0.56	40	50	60	70
TRULL BROOK					
At confluence with Merrimack River	4.4	200	300	355	475
Upstream of River Road	4.1	175	250	285	335
Upstream of tributary at Station 1.145	2.3	125	170	200	235
TRULL BROOK TRIBUTARY					
At Nesmith Street	0.6	40	60	70	95
UNKETY BROOK					
At the Groton/Dunstable corporate limits	2.6	110	160	190	250
VALLEY POND					
At Valley Pond Outlet	1.78	77	116	133	185

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
VARNUM BROOK					
At confluence with the Nashua River	0.92	70	110	135	210
VINE BROOK					
At confluence with Shawsheen River	8.3	495	700	820	1,195
At Wilson Road	8.2	485	690	805	1,175
At Butterfield Pond	2.32	197	379	486	852
At Emerson Road	2.2	188	350	444	758
At Trail	1.81	159	304	383	660
At Brookwood Road	1.67	150	287	360	620
At downstream end of 2,600-foot culvert	1.49	132	249	309	522
WALKER BROOK 1					
At confluence with the Nashua River	1.12	60	90	100	130
WALKER BROOK 2					
At confluence with the Squannacook River	41.8	1,470	2,760	3,520	5,600
Above confluence of Willard Brook	15.05	750	1,410	1,800	2,830
Above confluence of Mason Brook	6.58	310	520	630	910
WALKER BROOK 3					
At confluence with Sudbury Reservoir	1.85	100	160	190	260
WALKERS BROOK					
Downstream Reading corporate limits	2.55	140	230	280	420
Approximately 2,900 feet downstream of John Street	1.66	120	200	240	350
Approximately 900 feet downstream of John Street	1.16	96	150	180	280
WELLINGTON BROOK					
At Boston and Maine Railroad	1.7	70	130	180	320
WEST CHESTER BROOK					
At its confluence with Chester Brook	1.13	120	160	200	290

TABLE 5 - SUMMARY OF DISCHARGES – continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (sq. miles)	PEAK DISCHARGES (cfs) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
WHITEHALL BROOK					
At confluence with Sudbury River	7.22	660	990	1,130	1,470
WILLARD BROOK					
At confluence with Walker Brook No. 2	26.9	1,330	2,360	2,920	4,440
WINTHROP CANAL					
Upstream of Linden Pond	2.48	175	235	310	460
Upstream of Arch Street	1.81	60	80	100	150
WITCH BROOK					
At confluence with the Squannacook River	3.51	150	240	280	380

The stillwater elevations have been determined for the 10-, 2-, 1-, and 0.2-percent annual chance floods for the flooding sources studied by detailed methods and are summarized in Table 6, "Summary of Stillwater Elevations."

TABLE 6 - SUMMARY OF STILLWATER ELEVATIONS

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD*) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
ASSABET RIVER				
At Acton-Concord corporate limits	129.2	130.5	131.1	132.9
At Powder Mill Dam	143.1	144.4	145.0	145.9
ATLANTIC OCEAN				
Mystic River downstream of Amelia Earhart Dam	8.3	9.1	9.5	10.3
ELL POND				
Entire shoreline within Melrose	48.2	51.6	53.4	53.9
FORT POND BROOK				
At Acton/Concord corporate limits	122.5	123.4	123.9	124.6
At Merrimack Dam	148.3	149.3	149.8	150.6
At Cement Dam	174.7	175.2	175.3	175.6
At Erikson Dam	192.5	193.3	193.6	194.2

*North American Vertical Datum of 1988

TABLE 6 - SUMMARY OF STILLWATER ELEVATIONS

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD*) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
LAKE QUANNAPOWITT				
Entire shoreline within Wakefield	82.5	82.8	83.0	83.1
LINDEN BROOK				
At confluence with Town Line Brook	**	**	8.2	**
At Beach Street	**	**	8.2	**
LOWER MYSTIC LAKE				
Entire shoreline within Arlington and Medford	**	**	7.2	**
MASSAPOAG POND				
Entire shoreline within Groton, Dunstable, and Tyngsborough	166.5	167.3	167.6	168.6
NAGOG POND				
At Dam	225.8	225.9	225.9	226.2
NASHOBA BROOK				
At Acton/Westford corporate limits	121.4	121.9	122.4	124.9
At Concord Road Dam	135.7	136.1	136.3	136.6
At Wheeler Lane Dam	167.5	167.8	168.0	168.4
TOWN LINE BROOK				
At county boundary	**	**	8.2	**
At Broadway Drive	**	**	8.2	**
UPPER MYSTIC LAKE				
Entire shoreline within Winchester	**	**	12.6	14.0
WAUSHAKUM POND				
Entire shoreline within Ashland and Framingham	158.6	159.2	159.3	160.0

*North American Vertical Datum of 1988
**data not available

Postcountywide Analyses

The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS), version 2.2.2, was used to develop runoff hydrographs for use in the HEC-RAS unsteady flow model. Geographic information systems (GIS) based automation was employed to efficiently develop sub-basin areas and characteristics. The steps that went into building and the hydrologic model were:

- Basin delineation
- SCS Curve Number determination
- Time of Concentration calculations

- Storage coefficient calculations
- Determination of precipitation extremes

Basin Delineation

GIS-based digital terrain modeling was employed to automate delineation of sub-watersheds within the study area. These techniques were used to identify the contributing watershed to approximately 200 reaches of study area flooding sources. To accomplish this task, the methodology established for processing the National Elevation Dataset (2003) for use with the National Hydrography Dataset (USGS 2003) was followed, which included the following steps:

- Establish stream centerline for flooding sources from NHD
- Segment stream centerline at structure locations to generate desired reaches.
- Modification of NED elevations to fill any sinks and enforce previously delineated stream channel locations (centerline) and basin boundaries (those done by the Massachusetts Department of Environmental Protection).
- Calculate flow direction grid based on the modified NED coverage.

Using the flow direction grid, delineate the contributing area to each of the stream reaches. The filling of sinks, calculation of flow direction, and watershed delineation are functions that are built in to the Spatial Analyst product.

SCS Curve Number Determination

The SCS Runoff Curve Number method of hydrologic abstractions as implemented in HEC-HMS (USACE, 2000) was selected as the loss rate method for the hydrologic modeling. This methodology depends on a Runoff Curve Number (RCN) that defines the rainfall-runoff relationship for the basin, and a time of concentration defined by the longest hydrologic flow path in the basin. The RCN was determined in accordance with SCS TR-55 methodology based on the surficial hydrologic soil group and landcover type. GIS analysis was used to automate calculation of RCNs for each of the approximately 200 basins in the study area.

Hydrologic Soils Data

A Digital NRCS Soil Survey is not yet available for Middlesex County, Massachusetts; therefore, for use in GIS analysis this data had to be captured from the existing paper maps.

Landcover Data

IKONOS satellite imagery from 2001 and 2002 at four meters per pixel resolution was acquired, and classified according to a few basic landcover types: Water, Wetland, Grass, Urban Recreation, Residential, Roads/Parking, Industrial/Commercial, Quarry/Landfill, Forest, Scrub/Shrub, Fallow/Bare Earth, and Agriculture.

Composite Curve Number Generation Method

GIS was used to automate the calculation of composite RCNs for each sub-watershed using separate grid coverages of hydrologic soil groups, landcover, and sub-watershed boundaries. This automated process also calculated the percent directly connected impervious area (DCIA) for each subwatershed. A percent DCIA was assigned to the landcover categories residential, roads/parking, and Industrial/Commercial, the remaining lancover categories were assumed to have no DCIA. The table below summarizes the DCIA used to calculate % Impervious for input into the HMS model. Based on the percent DCIA of each subwatershed, the RCN input into the hydrologic model was reduced by the following expression:

(RCN – 98 * (DCIA)) / (1 – DCIA)

Percent Directly Connected Impervious Area

Landuse	% DCIA
Residential	25
Roads/Parking	90
Industrial/Commercial	50

Time of Concentration Calculations

Time of concentration was calculated for each of the sub-basins.

Storage Coefficient Calculations

The Clark Unit hydrograph method was used as the transform method in HEC-HMS. The following relationship was developed to relate basin area to storage coefficient for each of the subbasins, with a minimum R of 0.1:

R = 20.84 * (Log(Area) + 1) + 0.1

Determination of Precipitation Extremes

The precipitation statistics from the Northeast Regional Climate Center (Wilks & Cember, 1993) were chosen as the basis for the design storm values for the hydrologic modeling in the Mystic River basin study.

NRCC and TP-40 Precipitation Extremes

	Rainfall in Inches	
Return Period	**24-hour**	**1-day**
10-percent annual chance	4.8	4.3
2-percent annual chance	7.1	6.3
1-percent annual chance	8.5	7.5
0.2-percent annual chance (extrapolated)	12.5	11.1

NRCC gives 1-day values, and a factor of 1.13 to convert to 24-hour

3.2 Hydraulic Analyses

Precountywide Analyses

Analyses of the hydraulic characteristics of flooding from the sources studied were carried out to provide estimates of the elevations of floods of the selected recurrence intervals. Information below up to and including the vertical datum is NAVD is common throughout all communities. Other information specific to each community is listed starting with the next page.

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles. For stream segments for which a floodway was computed, selected cross-section locations are also shown on the Flood Insurance Rate Map.

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

Cross sections for the hydraulic model were developed using GIS-based automated modeling techniques from a digital terrain model of the study area. The floodplain digital terrain model developed from aerial photogrammetric topographic survey of the above water areas and boat-based bathymetric transect survey of the under water areas.

Dimensions of the hydraulic structures were determined by field survey and/or from available plan information.

Manning's 'n' values were assigned using GIS-based automated modeling techniques based on a land cover data layer developed from project planimetric and orthophoto maps. Each land cover type was assigned a representative Manning's "n" value.

0.1, 0.02, 0.01, and 0.002 annual chance water-surface elevations were determined using an unsteady flow, step backwater hydraulic model, HEC-RAS version 3.1.3.

Flood profiles were drawn showing computed water—surface elevations to an accuracy of 0.5 foot for floods of the selected recurrence intervals. Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway is computed (Section 4.2), selected cross—section locations are also shown on the FIRM (Exhibit 3). All elevations in this study are referenced to NAVD.

For the streams studied by approximate methods, the flood boundaries were determined using normal depth calculations. Field investigations and historical observations in conjunction with manual calculations were used to determine elevations for areas prone to flooding with an estimated recurrence probability of less than one percent. In many instances, flooding was determined by backwater conditions from rivers and streams that were studied using detailed methods.

Cross sections for the streams studied by detailed methods within Middlesex County involved the following techniques: aerial photographs at various scales, below—water sections were obtained by field measurement, field survey, photogrammetric mapping, existing mapping supplied by the Massachusetts Department of Public Works, topographic mapping, and culvert analysis.

Water—surface elevations of floods of the selected recurrence intervals within Middlesex County involved the following computer programs: USACE HEC—2 step—backwater computer program, Soil Conservation Service WSP-2 water-surface profiles computer program, FLOW2D computer simulation model, storage-discharge relationships, and normal bridge option.

Starting water—surface elevations for the streams studied by detailed methods within Middlesex County involved the following techniques: use of the slope/area method, use of backwater computations, use of the HEC-2 program, use of normal depth analysis, use of a discharge rating curve, using the USACE report in Littleton, use of profiles of the main streams, use of various previously printed FISs for several communities within Middlesex County, developing a stage-discharge curve and then computing a backwater profile to this cross section, developing a stage-discharge relationship for the Wamesit Power Company Dam, use of critical depth, use of the NRCS TR-20 computer program, and analyzing the effects of the tide gates.

For the Middlesex County countywide FIS, revised hydraulic analyses were prepared for the following streams: Aberjona River, Aberjona River North Spur, Alewife Brook (Little River), Cummings Brook, Halls Brook, Horn Pond Brook/Fowle Brook, Little Brook, Mill Brook 3, Mystic River, Schneider Brook, Shakers Glen Brook, Sweetwater Brook, and Wellington Brook. Cross sections for the hydraulic model were developed using GIS-based automated modeling techniques from a digital terrain model of the study area. The floodplain digital terrain model developed from LiDAR survey of the water areas and boat-based bathymetric/field transect survey of the underwater areas. Dimensions of the hydraulic structures were determined by field survey and/or from available plan information.

Flood levels along the Charles River downstream of the Watertown Dam are controlled by the operation of the Charles River Dam located 6 miles downstream in the City of Boston. The MDC operates the 8,400 cfs capacity pumps at the dam so as to prevent the 1-percent annual chance flood from achieving 4.6 feet (beginning of damages). Stages for the 10-, 2-, 1-, and 0.2-percent annual chance flood events along this reach of the Charles River are presented on the Flood Profiles.

Roughness factors (Manning's "n") used in the hydraulic computations were chosen by engineering judgment and were based on field observations of the streams and floodplain areas. Roughness factors for all streams studied by detailed methods are shown in Table 7, "Manning's "n" Values."

TABLE 7 - MANNING'S "n" VALUES

Stream	Channel "n"	Overbank "n"
Aberjona River	0.035-0.150	0.014-0.300
Aberjona River North Spur	0.035-0.150	0.014-0.300
Alewife Brook (Little River)	0.035-0.150	0.014-0.300
Angelica Brook	0.030-0.035	0.050-0.060
Assabet Branch No. 3	0.030-0.060	0.030-0.130
Assabet Branch No. 4	0.030-0.060	0.030-0.130
Assabet River	0.015-0.060	0.015-0.130
Baddacook Brook	0.035-0.040	0.050-0.070
Baiting Brook	0.025-0.005	0.045-0.120
Bear Meadow Brook	0.020-0.065	0.040-0.150
Beaver Brook 1	0.015-0.040	0.030-0.150
Beaver Brook 2	0.035-0.055	0.080-0.100
Beaver Brook 3	0.030-0.045	0.045-0.075
Beaver Brook 4	0.035-0.070	0.050-0.140
Beaver Brook 5	0.050-0.080	0.060-0.100
Beaverdam Brook	0.015-0.050	0.050-0.110
Bennetts Brook	0.035	0.050
Birch Meadow Brook	0.025-0.045	0.045-0.085
Black Brook	0.030-0.035	0.055-0.085
Bogastow Brook – Jar Brook	0.055	0.160
Bogle Brook 1	0.015-0.050	0.070-0.110
Bogle Brook 2	0.015-0.040	0.030-0.240
Boons Pond and Branch	0.015-0.060	0.050-0.120
Boutwell Brook	0.030	0.050
Bow Brook	0.035	0.050-0.070
Branch of Assabet River	0.015-0.060	0.050-0.120
Branch of Elizabeth Brook 1	0.015-0.060	0.050-0.120
Broad Meadow Brook	0.015-0.035	0.045-0.080
Brook A of Shawsheen River	*	*
Brook from Waushakum Pond	0.030-0.035	0.050-0.060
Butter Brook	0.035-0.045	0.050-0.085
Catacoonamug Brook	0.035	0.050-0.070
Charles River	0.015-0.060	0.040-0.150
Cheese Cake Brook	0.030-0.035	0.010
Cherry Brook	0.015-0.040	0.030-0.240
Chester Brook	0.015-0.040	0.030-0.150
Chicken Brook	0.060	0.120
Cochituate Brook	0.030-0.035	0.050-0.060
Cold Brook	0.016-0.050	0.050-0.100
Cold Spring Brook	0.035-0.050	0.050-0.100
Cole's Brook	0.015-0.040	0.040-0.120
Collins Brook	*	*
Conant Brook	0.030-0.040	0.040-0.080
Concord River	0.015-0.077	0.040-0.125

*Data not available

TABLE 7 - MANNING'S "n" VALUES - continued

Stream	Channel "n"	Overbank "n"
Content Brook – Middlesex Canal	0.030-0.045	0.060-0.110
Course Brook	0.020-0.060	0.040-0.150
Cow Pond Brook	0.035-0.040	0.050-0.070
Cummings Brook	0.035-0.060	0.014-0.300
Dakins Brook	0.030-0.040	0.080
Danforth Brook	0.030-0.060	0.030-0.130
Darby Brook	0.015-0.045	0.060-0.070
Davis Brook	0.015-0.050	0.070-0.110
Dirty Meadow Brook	0.060	0.160
Dopping Brook	0.045-0.060	0.050-0.160
Dudley Brook – Tributary to Dudley Brook	0.016-0.045	0.050-0.090
East Outlet	0.030-0.055	0.050-0.095
Elizabeth Brook 1	0.015-0.060	0.050-0.120
Elizabeth Brook 2	0.035-0.070	0.050-0.140
Elm Brook	0.015-0.050	0.020-0.180
Farrar Pond Brook	0.045-0.075	0.050
Farrar Pond – Pole Brook	0.020-0.060	0.090-1.100
Fort Meadow Brook	0.030-0.060	0.030-0.130
Fort Pond Brook	0.012-0.070	0.015-0.140
Fort Pond Brook Branch 1	0.035-0.050	0.015-0.120
Fort Pond Brook Branch 2	0.035-0.070	0.050-0.140
Grassy Pond Brook	0.015-0.050	0.015-0.085
Graves Pond Brook	0.035	0.050-0.075
Great Road Tributary	0.035	0.080
Greens Brook	0.040	0.085-0.180
Guggins Brook	0.015-0.070	0.015-0.140
Gumpas Pond Brook	0.035	0.070
Hales Brook	0.035-0.055	0.080-0.100
Halls Brook	0.035-0.100	0.014-0.300
Hayward Brook	0.035	0.050-0.075
Heath Brook	*	*
Hobbs Brook 1	0.015-0.040	0.030-0.240
Hobbs Brook 2	0.045-0.075	0.045-0.075
Hog Brook	0.030-0.060	0.030-0.130
Hop Brook	0.015-0.035	0.045-0.080
Horn Pond Brook / Fowle Brook	0.030-0.100	0.014-0.300
Inch Brook	0.025-0.045	0.025-0.085
Indian Brook	0.035-0.050	0.050-0.100
Ipswich River	0.020-0.065	0.040-0.150
James Brook	0.035-0.040	0.050-0.070
Jar Brook	0.055	0.120
Jones Brook	0.030-0.040	0.110
Kiln Brook	0.050-0.080	0.060-0.100
King Street Tributary	0.035-0.042	0.070-0.100

*Data not available

TABLE 7 - MANNING'S "n" VALUES - continued

Stream	Channel "n"	Overbank "n"
Landham-Allowance Brook	0.016-0.035	0.050-0.070
Lawrence Brook	0.030	0.060-0.075
Little Brook	0.035-0.10	0.014-0.30
Locke Brook	0.035	0.050-0.075
Lower Spot Pond Brook	0.020-0.024	0.044
Lubbers Brook	0.030-0.055	0.075-0.120
Malden River	0.020-0.050	0.020-0.050
Maple Meadow Brook	0.035-0.055	0.065-0.088
Marginal Brook	0.035-0.045	0.045-0.085
Marshall Brook	0.014-0.045	0.060-0.070
Martins Brook	0.040-0.102	0.060-0.090
Martins Pond Brook	0.035-0.040	0.050-0.070
Mascuppic Brook	0.030	0.070
Mason Brook	0.035	0.050-0.075
Meadow Brook	0.024-0.045	0.060-0.070
Meadow River Branch	0.015-0.050	0.100
Merrimack River	0.020-0.055	0.040-0.200
Mill Brook 1	0.035	0.050-0.075
Mill Brook 2	0.030-0.040	0.030-0.080
Mill Brook 3	0.035-0.150	0.014-0.300
Mill Pond Tributary	0.020-0.035	0.080
Mill River	0.045-0.100	0.110
Mineway Brook	0.030-0.045	0.070-0.100
Mongo Brook	0.040-0.050	0.120-0.160
Morse Brook	0.035	0.050-0.070
Mowry Brook	0.015-0.035	0.045-0.080
Mud Pond Brook	*	*
Mulpus Brook	0.035	0.050-0.070
Munroe Brook	0.050-0.080	0.060-0.100
Mystic River	0.035	0.014-0.300
Nagog Brook	0.045	0.070
Nashoba Brook	0.015-0.045	0.040-0.120
Nashua River	0.030	0.060-0.070
Nissitissit River	0.040	0.060-0.090
Nonacoicus Brook 1	0.035	0.050
Nonacoicus Brook 2	0.035	0.050
North Lexington Brook	0.050-0.080	0.060-0.100
Pages Brook	0.013-0.050	0.100
Pages Brook Branch	0.024-0.050	0.100
Pantry Brook	0.016-0.040	0.050-0.100
Pearl Hill Brook	0.035	0.050-0.075
Peppermint Brook	0.035	0.070
Pine Brook	0.035	0.050-0.075
Pratts Brook	0.025-0.040	0.080

*Data not available

TABLE 7 - MANNING'S "n" VALUES - continued

Stream	Channel "n"	Overbank "n"
Putnam Brook	0.035-0.055	0.080-0.100
Reedy Meadow Brook	0.035-0.050	0.050-0.070
Reservoir No. 1-North Branch	0.030-0.035	0.050-0.060
Reservoir No. 3	0.030-0.035	0.050-0.060
Richardson Brook	0.035-0.045	0.045-0.080
River Meadow Brook	0.030-0.060	0.020-0.100
Run Brook	0.016-0.045	0.050-0.100
Salmon Brook	0.030-0.035	0.100
Sandy Brook	0.035-0.045	0.050-0.100
Saugus River	0.045-0.100	0.110
Saunders Brook	*	*
Sawmill Brook 1	0.038-0.040	0.080-0.090
Sawmill Brook 2	0.030-0.040	0.080
Schneider Brook	0.035-0.15	0.014-0.300
Shakers Glen Brook	0.035	0.014-0.300
Shawsheen River	0.012-0.050	0.020-0.150
Skug River	0.040-0.098	0.075
Snake Brook	0.035	0.050-0.075
South Meadow Brook/Paul Brook	0.020-0.035	0.070-0.080
Spencer Brook 1	0.020-0.040	0.080
Spencer Brook 2	0.015-0.050	0.100
Spring Brook	0.024-0.045	0.030-0.160
Squannacook River	0.035-0.040	0.060-0.080
Stony Brook 1	0.015-0.045	0.030-0.150
Stony Brook 2	0.030-0.055	0.070-0.110
Strong Water Brook	0.014-0.045	0.040-0.090
Sudbury River	0.025-0.050	0.030-0.120
Sutton Brook	*	*
Sweetwater Brook	0.035	0.014-0.300
Tadmuck Brook	0.030	0.055-0.070
Tadmuck Swamp Brook	0.030	0.070
Taylor Brook	0.035-0.050	0.015-0.120
Town Line Brook	0.020	0.035
Tributary 1 to Cole's Brook	0.015-0.040	0.040-0.120
Tributary 1 to Sudbury River	0.030-0.040	0.080
Tributary 2 to Assabet River	0.015-0.040	0.040-0.060
Tributary 2 to Tributary 1 to Cole's Brook	0.015-0.040	0.040-0.120
Tributary 3 to Bogle Brook 2	0.015-0.040	0.030-0.240
Tributary 4 to Bogle Brook 2	0.015-0.040	0.030-0.240
Tributary A to Cold Brook	0.030-0.045	0.070-0.140
Tributary A to Course Brook	0.020-0.060	0.040-0.150
Tributary A to Hop Brook	0.040	0.060-0.120
Tributary A to Pantry Brook	0.030-0.040	0.090-0.110
Tributary A to Squannacook River	0.035	0.050-0.070

*Data not available

TABLE 7 - MANNING'S "n" VALUES - continued

Stream	Channel "n"	Overbank "n"
Tributary B to Hop Brook	0.035-0.040	0.120
Tributary B to Squannacook River	0.035	0.050-0.075
Tributary B to Vine Brook	0.040-0.045	0.040-0.090
Tributary C to Hop Brook	0.040	0.100
Tributary C to Vine Brook	0.040	0.070-0.120
Tributary D to Hop Brook	0.040	0.100
Tributary to Beaver Brook 3	0.040-0.050	0.075-0.095
Tributary to Cold Spring Brook	0.035-0.045	0.050-0.085
Tributary to Indian Brook	0.035-0.050	0.050-0.100
Tributary to Martins Brook	0.045-0.065	0.060-0.090
Tributary to Mill Brook	0.030-0.065	0.030-0.150
Tributary to Nonacoicus Brook 1/Long Pond Brook	0.035	0.050
Tributary to Waushakum Pond	0.035-0.045	0.050-0.085
Trout Brook 1	0.035-0.050	0.070-0.090
Trout Brook 2	0.035	0.050-0.070
Trull Brook	0.030-0.060	0.050-0.080
Trull Brook Tributary	0.030-0.050	0.050-0.100
Unkety Brook	0.035-0.040	0.050-0.070
Varnum Brook	0.030	0.045-0.160
Vine Brook	0.020-0.080	0.020-0.100
Walker Brook 1	0.035	0.050-0.070
Walker Brook 2	0.035	0.050-0.070
Walker Brook 3	0.015-0.035	0.045-0.080
Walkers Brook	0.020-0.065	0.040-0.150
Wellington Brook	0.035-0.150	0.014-0.300
West Chester Brook	0.015-0.040	0.030-0.150
Whitehall Brook	0.035-0.050	0.050-0.100
Willard Brook	0.035	0.050-0.075
Winthrop Canal	0.040	0.070-0.100
Witch Brook	0.035	0.050-0.070

All qualifying bench marks within a given jurisdiction that are cataloged by the National Geodetic Survey (NGS) and entered into the National Spatial Reference System (NSRS) as First or Second Order Vertical and have a vertical stability classification of A, B, or C are shown and labeled on the FIRM with their 6-character NSRS Permanent Identifier.

Bench marks cataloged by the NGS and entered into the NSRS vary widely in vertical stability classification. NSRS vertical stability classifications are as follows:

- Stability A: Monuments of the most reliable nature, expected to hold position/elevation well (e.g., mounted in bedrock)

- Stability B: Monuments which generally hold their position/elevation well (e.g., concrete bridge abutment)

- Stability C: Monuments which may be affected by surface ground movements (e.g., concrete monument below frost line)

- Stability D: Mark of questionable or unknown vertical stability (e.g., concrete monument above frost line, or steel witness post)

In addition to NSRS bench marks, the FIRM may also show vertical control monuments established by a local jurisdiction; these monuments will be shown on the FIRM with the appropriate designations. Local monuments will only be placed on the FIRM if the community has requested that they be included, and if the monuments meet the aforementioned NSRS inclusion criteria.

To obtain current elevation, description, and/or location information for bench marks shown on the FIRM for this jurisdiction, please contact the Information Services Branch of the NGS at (301) 713-3242, or visit their Web site at www.ngs.noaa.gov.

It is important to note that temporary vertical monuments are often established during the preparation of a flood hazard analysis for the purpose of establishing local vertical control. Although these monuments are not shown on the FIRM, they may be found in the Technical Support Data Notebook associated with this FIS and FIRM. Interested individuals may contact FEMA to access this data.

Postcountywide Analyses

Cross-sections for the hydraulic model were developed using GIS-based automated modeling techniques from a digital terrain model of the study area. The floodplain digital terrain model developed from LiDAR survey of the above water areas and boat-based bathymetric/field transect survey of the under water areas.

Dimensions of the hydraulic structures were determined by field survey and/or from available plan information.

Manning's 'n' values were assigned using GIS-based automated modeling techniques based on a landcover datalayer developed for the project. Each land cover type was assigned a representative Manning's "n" value. Below is the range for each flooding source.

Aberjona River- Channel Manning's "n" values ranged from 0.035 to 0.15, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Alewife Brook (Little River) - Channel Manning's "n" values ranged from 0.035 to 0.15, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Cummings Brook - Channel Manning's "n" values ranged from 0.035 to 0.06, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Halls Brook - Channel Manning's "n" values ranged from 0.035 to 0.1, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Horn Pond Brook / Fowle Brook - Channel Manning's "n" values ranged from 0.035 to 0.1, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Little Brook - Channel Manning's "n" values ranged from 0.035 to 0.1, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Mill Brook - Channel Manning's "n" values ranged from 0.035 to 0.15, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Mystic River - Channel Manning's "n" used was 0.035, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Aberjona River – North Spur - Channel Manning's "n" values ranged from 0.035 to 0.15, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Schneider Brook - Channel Manning's "n" values ranged from 0.035 to 0.15, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Shakers Glen Brook - Channel Manning's "n" used was 0.035, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Sweetwater Brook - Channel Manning's "n" used was 0.035, and overbank Manning's "n" values ranged from 0.014 – 0.3.

Wellington Brook - Channel Manning's "n" values ranged from 0.035 to 0.15, and overbank Manning's "n" values ranged from 0.014 – 0.3.

10-, 2-, 1-, and 0.2-percent annual chance water-surface elevations were determined using an unsteady flow, step backwater hydraulic model, HEC-RAS version 3.1.3.

3.3 Vertical Datum

All FISs and FIRMs are referenced to a specific vertical datum. The vertical datum provides a starting point against which flood, ground, and structure elevations can be referenced and compared. Until recently, the standard vertical datum in use for newly created or revised FISs and FIRMs was National Geodetic Vertical Datum (NGVD) of 1929. With the finalization of the North American Vertical Datum (NAVD) of 1988, many FIS reports and FIRMs are being prepared using NAVD as the referenced vertical datum.

All flood elevations shown in this FIS report and on the FIRM are referenced to NAVD. Structure and ground elevations in the community must, therefore, be referenced to NAVD. It is important to note that adjacent communities may be referenced to NGVD. This may result in differences in base flood elevations across the corporate limits between the communities.

Prior versions of the FIS report and FIRM were referenced to NGVD. When a datum conversion is effected for an FIS report and FIRM, the Flood Profiles, base flood elevations (BFEs) and ERMs reflect the new datum values. To compare structure and ground elevations to 1-percent annual chance (100-year) flood elevations shown in the FIS and on the FIRM, the subject structure and ground elevations must be referenced to the new datum values.

As noted above, the elevations shown in the FIS report and on the FIRM for Middlesex County are referenced to NAVD. Ground, structure, and flood elevations may be compared and/or referenced to NGVD by applying a standard conversion factor. The conversion factor to NGVD is +0.8 foot. The BFEs shown on the FIRM represent whole-foot rounded values. For example, a BFE of 102.4 will appear as 102 on the FIRM and 102.6 will appear as 103. Therefore, users that wish to convert the elevations in this FIS to NGVD should apply the stated conversion factor(s) to elevations shown on the Flood Profiles and supporting data tables in the FIS report, which are shown at a minimum to the nearest 0.1 foot.

For more information on NAVD, see Converting the National Flood Insurance Program to the North American Vertical Datum of 1988, FEMA Publication FIA-20/June 1992, or contact the Vertical Network Branch, National Geodetic Survey, Coast and Geodetic Survey, National Oceanic and Atmospheric Administration, Rockville, Maryland 20910 (Internet address http://www.ngs.noaa.gov).

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The National Flood Insurance Program encourages commonwealth and local governments to adopt sound floodplain management programs. Therefore, each FIS includes a flood boundary map designed to assist communities in developing sound floodplain management measures.

4.1 Floodplain Boundaries

In order to provide a national standard without regional discrimination, the 1-percent annual chance flood has been adopted by the FIA as the base flood for purposes of floodplain management measures. The 0.2-percent annual chance flood is employed to indicate additional areas of flood risk in the community. For each stream studied in detail, the boundaries of the 1- and 0.2-percent annual chance floods have been delineated using the flood elevations determined at each cross section; between cross sections, the boundaries were interpolated using topographic maps at a scale of 1:2,400 and a scale of 1:24,000 with contour intervals of 4 and 10 feet, respectively. In cases where the 1- and 0.2-percent annual chance flood boundaries are close together, only the 1-percent annual chance boundary has been shown.

For the streams studied by approximate methods, the boundary of the 1-percent annual chance flood was delineated using the Flood Hazard Boundary Map for the municipalities of Middlesex County.

Small areas within the flood boundaries may lie above the flood elevations and, therefore, may not be subject to flooding. Owing to limitations of the map scale and lack of detailed topographic data, such areas are not shown.

For these streams, the following data bulleted below was used: Aberjona River, Aberjona River North Spur, Alewife Brook (Little River), Cummings Brook, Halls Brook, Horn Pond Brook/Fowle Brook, Little Brook, Mill Brook 3, Mystic River, Schneider Brook, Shakers Glen Brook, Sweetwater Brook, and Wellington Brook.

- 7.5-Minute USGS Digital Quadrangles for the study area.
  1:24,000 scale
  10-foot topographic contour interval

- MassGIS 2005 (April 2005) Color Digital Orthophotos for the study area.
  1:5,000 scale
  0.5 meter per pixel

- MassGIS 2002 LIDAR topography for the study area.
  1:5,000 scale
  Suitable for 2-foot contour generation

- Town of Winchester basemap for Winchester only.
  1:5,000 scale; 2-foot topographic contour interval

Floodplain boundaries were delineated on the project digital terrain (DTM) model using GIS-based automated techniques.

4.2 Floodways

The project HEC-RAS unsteady flow model was used to compute a regulatory floodway for the one percent annual chance event.

The initial encroachment analysis was performed using the steady flow option in HEC-RAS. A steady flow file model was developed using the peak flows predicted in the one percent annual chance unsteady flow model. The steady flow encroachment analysis used the equal conveyance reduction and Method 5, where the model optimizes the encroachments to match a target water surface rise, in this case 1.0 foot. The results produced a reasonable approximation of potential floodway encroachments. The steady flow encroachment stations were applied to the unsteady flow model. The model results from the unsteady flow model run with the steady flow encroachments predicted that the observed water surface at many cross sections would be greater than 1.0 foot.

A method was developed to determine encroachment stations that could be applied fairly to all rivers and reaches within the unsteady flow model and result in a maximum water surface rise at any cross section of 1.0 foot. All reaches within the model were divided into subreaches defined both upstream and downstream by hydraulic structures. The average top width of the right and left overbank flow areas, determined from the results of the 1-percent annual chance model, were assigned to all cross sections within each subreach.

The model encroachment stations were calculated by encroaching on the left and right overbank by a percentage of the average overbank top width for each

subreach. The same encroachment percentage was applied to all sub-reaches. An encroachment of one percent resulted in a maximum water-surface rise of 1.0 foot at several locations within the study area. Many reaches are far below the target 1.0-foot increase, but if the encroachments are increased uniformly across the watershed, the predicted water surface will increase to greater than 1.0 foot in the downstream sections of the model domain.

The following detailed study streams in Middlesex County did not have a floodway calculated: the Charles River, Dakins Brook, Gumpas Pond Brook, Lower Spot Pond Brook, the Malden River, Mineway Brook, Pratts Brook, Putnam Brook, Reservoir No. 1 – North Branch and Reservoir No. 3, Tributary A to Cold Brook, Tributary A to Dudley Brook, Tributary A to Hop Brook, Tributary A to Pantry Brook, Tributary B to Hop Brook, Tributary C to Hop Brook, Tributary D to Hop Brook, and Valley Pond. Portions of the following detailed study streams in Middlesex County did not have a floodway calculated: Beaver Brook 1, Chester Brook, Hales Brook, Martins Brook, Pantry Brook, and Stony Brook 2. In addition, portions of the Sudbury River in Middlesex County did not have a floodway shown due to the pooling effects from Reservoir No. 2 Dam and Weir Dam.

A floodway has been calculated for all of the streams included in the hydrodynamic model of the Mystic Watershed.

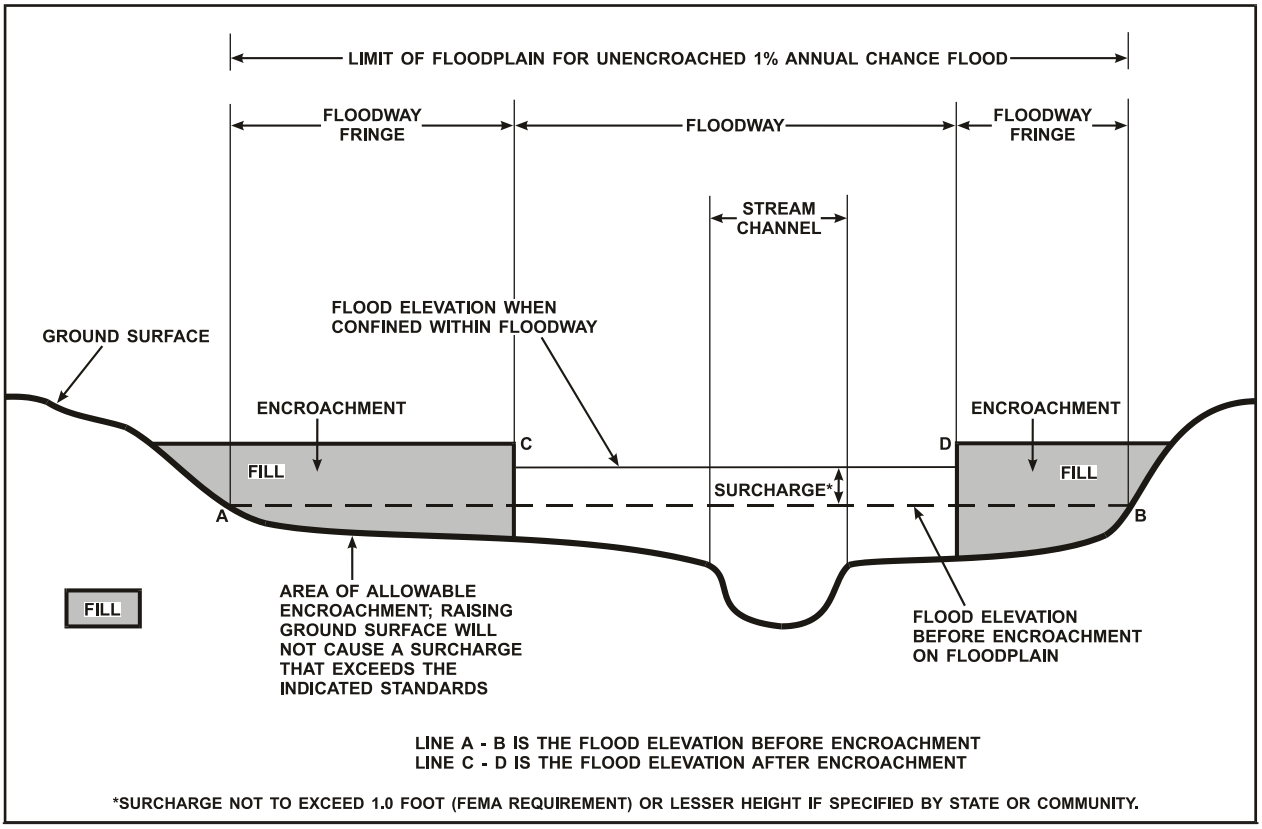
The Commonwealth of Massachusetts requires no rise in flood elevations for projects in the floodplain.

The floodways presented in this FIS were computed for certain stream segments on the basis of equal conveyance reduction from each side of the floodplain.

Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. The results of the floodway computations are tabulated for selected cross sections (Table 8, located in Volume 2). The computed floodways are shown on the FIRM (Exhibit 2). In cases where the floodway and 1-percent annual chance floodplain boundaries are either close together or collinear, only the floodway boundary is shown.

Encroachment into areas subject to inundation by floodwaters having hazardous velocities aggravates the risk of flood damage, and heightens potential flood hazards by further increasing velocities. A listing of stream velocities at selected cross sections is provided in Table 8, "Floodway Data" (located in Volume 2). In order to reduce the risk of property damage in areas where the stream velocities are high, the community may wish to restrict development in areas outside the floodway.

The area between the floodway and 1-percent annual chance floodplain boundaries is termed the floodway fringe. The floodway fringe encompasses the portion of the floodplain that could be completely obstructed without increasing the water-surface elevation of the 1-percent annual chance flood by more than 1.0 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 1.



FLOODWAY SCHEMATIC Figure 1

5.0 INSURANCE APPLICATIONS

For flood insurance rating purposes, flood insurance zone designations are assigned to a community based on the results of the engineering analyses. The zones are as follows:

Zone A

Zone A is the flood insurance rate zone that corresponds to the 1-percent annual chance floodplains that are determined in the FIS by approximate methods. Because detailed hydraulic analyses are not performed for such areas, no base flood elevations or depths are shown within this zone.

Zone AE

Zone AE is the flood insurance rate zone that corresponds to the 1-percent annual chance floodplains that are determined in the FIS by detailed methods. In most instances, whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone AH

Zone AH is the flood insurance rate zone that corresponds to the areas of 1-percent annual chance shallow flooding (usually areas of ponding) where average depths are between 1 and 3 feet. Whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone AO

Zone AO is the flood insurance rate zone that corresponds to the areas of 1-percent annual chance shallow flooding (usually sheet flow on sloping terrain) where average depths are between 1 and 3 feet. Average whole-foot depths derived from the detailed hydraulic analyses are shown within this zone.

Zone AR

Area of special flood hazard formerly protected from the 1-percent annual chance flood event by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1-percent annual chance or greater flood event.

Zone A99

Zone A99 is the flood insurance rate zone that corresponds to areas of the 1-percent annual chance floodplain that will be protected by a Federal flood protection system where construction has reached specified statutory milestones. No base flood elevations or depths are shown within this zone.

Zone V

Zone V is the flood insurance rate zone that corresponds to the 1-percent annual chance coastal floodplains that have additional hazards associated with storm waves. Because approximate hydraulic analyses are performed for such areas, no base flood elevations are shown within this zone.

Zone VE

Zone VE is the flood insurance rate zone that corresponds to the 1-percent annual chance coastal floodplains that have additional hazards associated with storm waves. Whole-foot base flood elevations derived from the detailed hydraulic analyses are shown at selected intervals within this zone.

Zone X

Zone X is the flood insurance rate zone that corresponds to areas outside the 0.2-percent annual chance floodplain, areas within the 0.2-percent annual chance floodplain, and to areas of 1-percent annual chance flooding where average depths are less than 1 foot, areas of 1-percent annual chance flooding where the contributing drainage area is less than 1 square mile, and areas protected from the 1-

percent annual chance flood by levees. No base flood elevations or depths are shown within this zone.

Zone D

Zone D is the flood insurance rate zone that corresponds to unstudied areas where flood hazards are undetermined, but possible.

6.0 FLOOD INSURANCE RATE MAP

The FIRM is designed for flood insurance and floodplain management applications.

For flood insurance applications, the map designates flood insurance rate zones as described in Section 5.0 and, in the 1-percent annual chance floodplains that were studied by detailed methods, shows selected whole-foot base flood elevations or average depths. Insurance agents use the zones and base flood elevations in conjunction with information on structures and their contents to assign premium rates for flood insurance policies.

For floodplain management applications, the map shows by tints, screens, and symbols, the 1- and 0.2-percent annual chance floodplains. Floodways and the locations of selected cross sections used in the hydraulic analyses and floodway computations are shown where applicable.

The current FIRM presents flooding information for the entire geographic area of Middlesex County. Previously, separate FIRMs were prepared for each identified floodprone incorporated community and the unincorporated areas of the county. This countywide FIRM also includes flood hazard information that was presented separately on FBFMs, where applicable. Historical data relating to the maps prepared for each community, are presented in Table 9, "Community Map History."

7.0 OTHER STUDIES

FISs and FIRMs have been prepared for the following towns in Hillsborough County, New Hampshire: New Ipswich (FEMA, 1991), Hollis (FEMA, 1979), Hudson (FEMA, 1979) and Pelham (FEMA, 1980). FISs have been prepared for the City of Nashua (FEMA, 1978) in Hillsborough County, New Hampshire. FIRMs have been prepared for the following towns in Hillsborough County, New Hampshire: Mason (FEMA, 1981) and Brookline (FEMA, 1987).

FISs and FIRMs have been prepared for the following towns in Worcester County, Massachusetts: Ashburnham (FEMA, 1984), Westminster (FEMA, 1982), Lunenburg (FEMA, 1983), Lancaster (FEMA, 1982), Harvard (FEMA, 1983), Bolton (FEMA, 1980), Berlin (FEMA, 1980), Northborough (FEMA, 1979), Southborough (FEMA, 1981), Milford (FEMA, 1984), Upton (FEMA, 1982) and Westborough (FEMA, 1979). FISs and FIRMs have been prepared for the City of Fitchburg (FEMA, 1991) in Worcester County, Massachusetts.

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE
Acton, Town of	July 26, 1974	None	June 15, 1978	January 6, 1988
Arlington, Town of	June 28, 1974	January 14, 1977	July 5, 1982	
Ashby, Town of	April 29, 1977	None	August 1, 1996	
Ashland, Town of	February 8, 1974	August 6, 1976	July 16, 1981	
Ayer, Town of	March 22, 1974	September 3, 1976	July 19, 1982	March 18, 1991
Bedford, Town of	September 7, 1973	None	September 7, 1973	July 1, 1974 February 26, 1976 June 15, 1983 July 4, 1988
Belmont, Town of	July 26, 1974	December 10, 1976	June 15, 1982	
Billerica, Town of	September 20, 1974	September 17, 1976	November 5, 1980	August 5, 1985
Boxborough, Town of	September 20, 1974	December 17, 1976	September 15, 1978	September 8, 1999
Burlington, Town of	August 9, 1977	None	July 5, 1984	
Cambridge, City of	June 21, 1974	November 19, 1976	July 5, 1982	

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA (ALL JURISDICTIONS)

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE
Carlisle, Town of	August 16, 1974	December 10, 1976	October 15, 1980	May 17, 1988
Chelmsford, Town of	October 25, 1974	October 8, 1976	June 4, 1980	January 16, 2004
Concord, Town of	September 6, 1974	None	June 15, 1979	June 3, 1988
Dracut, Town of	August 9, 1974	June 25, 1976	July 2, 1980	June 5, 1989
Dunstable, Town of	November 29, 1974	July 16, 1976	July 5, 1982	
Everett, City of	June 7, 1974	July 30, 1976	June 3, 1986	
Framingham, Town of	August 2, 1974	December 13, 1977	February 3, 1982	March 15, 1984 November 19, 1986 March 16, 1992
Groton, Town of	September 6, 1974	November 12, 1977	July 5, 1982	
Holliston, Town of	August 2, 1974	November 5, 1976	September 30, 1980	September 10, 1982
Hopkinton, Town of	July 19, 1974	October 8, 1976	July 5, 1982	
Hudson, Town of	July 26, 1974	November 12, 1976	December 15, 1979	

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA (ALL JURISDICTIONS)

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE
Lexington, Town of	June 28, 1974	December 10, 1976	June 1, 1978	September 30, 1983
Lincoln, Town of	December 13, 1974	October 15, 1976	June 1, 1978	June 17, 1986
Littleton, Town of	July 19, 1974	August 20, 1976	June 15, 1983	
Lowell, City of	May 31, 1974	None	April 16, 1979	February 15, 1984 May 15, 1991 September 30, 1992
Malden, City of	July 26, 1974	May 24, 1977	May 19, 1987	August 20, 2002
Marlborough, City of	July 26, 1974	November 15, 1977	January 6, 1982	
Maynard, Town of	July 26, 1974	December 10, 1976	June 15, 1979	
Medford, City of	July 26, 1974	September 24, 1976	June 3, 1986	
Melrose, City of	June 28, 1974	June 18, 1976	August 5, 1986	
Natick, Town of	July 26, 1974	April 9, 1976	February 1, 1980	
Newton, City of	June 28, 1974	None	June 1, 1978	November 2, 1983 July 17, 1986 June 4, 1990

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA (ALL JURISDICTIONS)

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE
North Reading, Town of	August 30, 1974	None	April 3, 1978	January 6, 1983 April 3, 1989 March 5, 1996 June 16, 2004
Pepperell, Town of	August 2, 1974	August 13, 1976	July 2, 1981	June 2, 1993
Reading, Town of	June 21, 1977	None	July 2, 1981	
Sherborn, Town of	May 27, 1977	None	June 18, 1980	
Shirley, Town of	June 28, 1974	November 19, 1976	July 5, 1983	
Somerville, City of	July 26, 1974	November 27, 1976	July 17, 1986	
Stoneham, Town of	August 2, 1974	December 13, 1977	July 3, 1986	
Stow, Town of	October 18, 1974	December 6, 1977	August 1, 1979	
Sudbury, Town of	August 23, 1974	December 10, 1976	June 1, 1982	November 20, 1998
Tewksbury, Town of	December 10, 1971	August 2, 1974	July 18, 1977	July 2, 1981
Townsend, Town of	September 20, 1974	August 13, 1976	August 2, 1982	

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA (ALL JURISDICTIONS)

COMMUNITY MAP HISTORY

COMMUNITY NAME	INITIAL IDENTIFICATION	FLOOD HAZARD BOUNDARY MAP REVISIONS DATE	FIRM EFFECTIVE DATE	FIRM REVISIONS DATE
Tyngsborough, Town of	August 2, 1974	November 26, 1976	September 2, 1982	
Wakefield, Town of	August 2, 1974	None	October 17, 1978	September 2, 1988
Waltham, City of	June 28, 1974	April 15, 1977	December 18, 1979	December 19, 1979 July 5, 1984
Watertown, Town of	June 28, 1974	December 3, 1976	September 30, 1980	
Wayland, Town of	July 26, 1974	December 24, 1976	June 1, 1982	February 19, 1986
Westford, Town of	October 18, 1974	August 6, 1976	June 15, 1983	
Weston, Town of	July 26, 1974	October 1, 1976	July 2, 1980	
Wilmington, Town of	March 1, 1974	July 2, 1976	June 15, 1982	January 18, 1989 June 2, 1999
Winchester, Town of	July 19, 1974	November 19, 1976	June 18, 1980	
Woburn, City of	August 2, 1974	June 28, 1977	July 2, 1980	

TABLE 9

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA
(ALL JURISDICTIONS)

COMMUNITY MAP HISTORY

FISs and FIRMs have been prepared for the following towns in Norfolk County, Massachusetts: Medway (FEMA, 1980), Millis (FEMA, 1985), Medfield (FEMA, 1979), Dover (FEMA, 1987), Wellesley (FEMA, 1987), and Needham (FEMA, 1989). FIRMs have been prepared for the Town of Brookline in Norfolk County, New Hampshire.

FISs and FIRMs have been prepared for the following towns in Essex County, Massachusetts: Saugus (FEMA, 1983), Lynnfield (FEMA, 1990), Middleton (FEMA, 1980), North Andover (FEMA, 1993), Andover (FEMA, 1989) and Methuen (FEMA, 1987).

FISs and FIRMs have been prepared for the following cities in Suffolk County, Massachusetts: Chelsea (FEMA, 1982) and Boston (FEMA, 1982). FISs have been prepared for the City of Revere (FEMA, 1984) in Suffolk County, Massachusetts.

Information pertaining to revised and unrevised flood hazards for each jurisdiction within Middlesex County has been compiled into this FIS. Therefore, this FIS supersedes all previously printed FIS Reports, FHBMs, FBFMs, and FIRMs for all jurisdictions within Middlesex County.

8.0 LOCATION OF DATA

Information concerning the pertinent data used in the preparation of this FIS can be obtained by contacting FEMA, Federal Insurance and Mitigation Division, 99 High Street, 6th Floor, Boston, Massachusetts 02110.

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