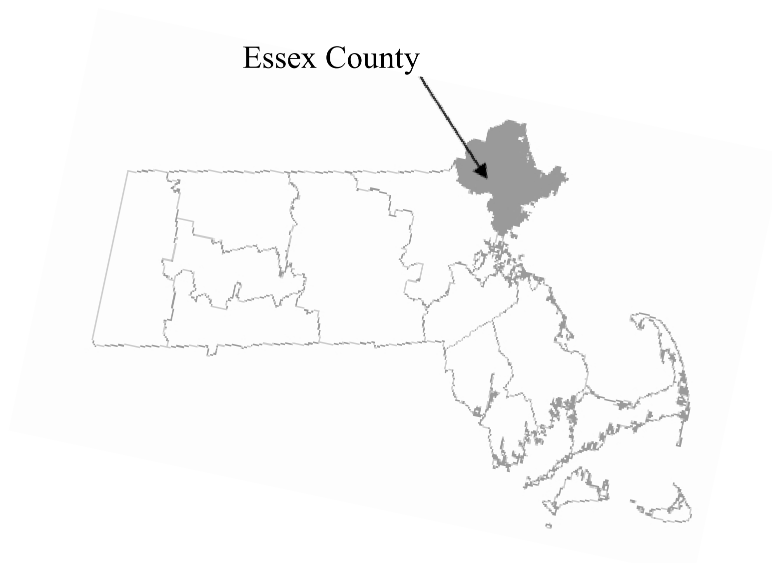
FLOOD INSURANCE STUDY

ESSEX COUNTY, MASSACHUSETTS

(ALL JURISDICTIONS)

Volume 1 of 4



COMMUNITY NAME	COMMUNITY NUMBER
AMESBURY, CITY OF	250075
ANDOVER, TOWN OF	250076
BEVERLY, CITY OF	250077
BOXFORD, TOWN OF	250078
DANVERS, TOWN OF	250079
ESSEX. TOWN OF	250080
GEORGETOWN, TOWN OF	250081
GLOUCESTER, CITY OF	250082
GROVELAND, TOWN OF	250083
HAMILTON, TOWN OF	250084
HAVERHILL, CITY OF	250085
IPSWICH, TOWN OF	250086
LAWRENCE, CITY OF	250087
LYNN, CITY OF	250088
LYNNFILED, TOWN OF	250089
MANCHESTER BY THE SEA, TOWN OF	250090
MARBLEHEAD, TOWN OF	250091
MERRIMAC, TOWN OF	250092
METHUEN, CITY OF	250093
MIDDLETON, TOWN OF	250094
NAHANT, TOWN OF	250095
NEWBURY, TOWN OF	250096
NEWBURYPORT, CITY OF	250097
NORTH ANDOVER, TOWN OF	250098
PEABODY, CITY OF	250099
ROCKPORT, TOWN OF	250100
ROWLEY, TOWN OF	250101
SALEM, CITY OF	250102
SALISBURY, TOWN OF	250103
SAUGUS, TOWN OF	250104
SWAMPSCOTT, TOWN OF	250105
TOPSFIELD, TOWN OF	250106
WENHAM, TOWN OF	250107
WEST NEWBURY, TOWN OF	250108

REVISION DATE:

JULY 16, 2014

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
25009CV001B

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.

Selected Flood Insurance Rate Map panels for the community contain information that was previously shown separately on the corresponding Flood Boundary and Floodway Map panels (e.g., floodways, cross sections). In addition, former flood hazard zone designations have been changed as follows:

Old Zone	New Zone
A1 through A30	AE
V1 through V30	VE
B	X
C	X

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

Initial Countywide FIS Effective Date: July 3, 2012

Revised Coastal FIS Effective Date: July 16, 2014

TABLE OF CONTENTS

Table of Contents - Volume 1 - July 16, 2014

FIGURES

Page

TABLES

TABLE OF CONTENTS – continued

Table of Contents - Volume 2 - July 16, 2014

TABLES

TABLE OF CONTENTS – VOLUME 3 - July 16, 2014

EXHIBITS

TABLE OF CONTENTS – continued

Table of Contents - Volume 3 - July 16, 2014

EXHIBITS - continued

Exhibit 1 - Flood Profiles – continued

TABLE OF CONTENTS – VOLUME 4 - July 16, 2014

EXHIBITS – continued

TABLE OF CONTENTS - continued

Table of Contents - Volume 4 - July 16,

EXHIBITS – continued

Exhibit 1 - Flood Profiles – continued

FLOOD INSURANCE STUDY
ESSEX COUNTY, MASSACHUSETTS (ALL JURISDICTIONS)

1.0 INTRODUCTION

1.1 Purpose of Study

This Flood Insurance Study (FIS) revises and updates information on the existence and severity of flood hazards in the geographic area of Essex County, including the Towns of Andover, Boxford, Danvers, Essex, Georgetown, Groveland, Hamilton, Ipswich, Lynnfield, Manchester by the Sea, Marblehead, Merrimac, Middleton, Nahant, Newbury, North Andover, Rockport, Rowley, Salisbury, Saugus, Swampscott, Topsfield, Wenham, and West Newbury; the Cities of Amesbury, Beverly, Gloucester, Haverhill, Lawrence, Lynn, Methuen, Newburyport, Peabody, and Salem (referred to collectively herein as Essex County), and aids in the administration of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. This study has developed flood-risk data for various areas of the community that will be used to establish actuarial flood insurance rates and to assist the community in its efforts to promote sound floodplain management. Minimum floodplain management requirements for participation in the National Flood Insurance Program (NFIP) are set forth in the Code of Federal Regulations at 44 CFR, 60.3.

In some states or communities, floodplain management criteria or regulations may exist that are more restrictive or comprehensive than the minimum Federal requirements. These criteria take precedence over the minimum federal criteria for purposes of regulating development in the floodplain, as set forth in the Code of Federal Regulations at 44 CFR, 60.3. In such cases, the more restrictive criteria take precedence and the state (or other jurisdictional agency) will be able to explain them.

1.2 Authority and Acknowledgments

The sources of authority for this FIS report are the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973.

This FIS was prepared to incorporate all the communities within Essex County into a countywide format. Information on the authority and acknowledgements for each jurisdiction included in this countywide FIS, as compiled from their previously printed FIS reports, is shown below:

Amesbury, City of	For the original December 1979 study, the hydrologic and hydraulic analyses were prepared by the U.S. Army Corps of Engineers (USACE) for the Federal Emergency Management Agency (FEMA), under Inter-Agency Agreement No. IAA-H-7-76,

Amesbury, City of (continued)	Project Order No. 25, and Inter-Agency Agreement No. IAA-H-10-77, Project Order No. 1. This work was completed in August 1978. In the October 29, 1982, revision, the hydrologic and hydraulic analyses for Lake Attitash were taken from the FIS for the Town of Merrimac, Essex County, Massachusetts, and were prepared by Cullinan Engineering Co, Inc., for FEMA under Contract No. H-4797. This work was completed in July 1980. The hydrologic and hydraulic analyses for this revision were taken from the FIS for the Town of South Hampton, Rockingham County, New Hampshire, and were prepared by the U.S Geological Survey (USGS) for FEMA, under Inter-Agency Agreement No. EM-89-E-2997, Project Order No. 5. This work was completed in September 1990 (Reference 1).
Andover, Town of	The hydrologic and hydraulic analyses for the June 5, 1989 study represent a revision of the original analyses prepared by the USACE for FEMA, under Inter-Agency Agreement No. IAA-H-2-73, Project Order No. 4. This work was completed in February 1975. The hydraulic analyses for the Merrimack River and the Shawsheen River; and the addition of detailed study analyses for Fish Brook, Hussey Brook, and Hussey Brook Tributary represent a revision to the original FIS for the Town of Andover. This updated study was performed by the USACE, New England Division, for FEMA, under Inter-Agency Agreement No. EMW-E-0941. This work was completed in July 1986 (Reference 2).
Beverly, City of	The hydrologic and hydraulic analyses for the March 18, 1986 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in July 1983 (Reference 3).
Boxford, Town of	The hydrologic and hydraulic analyses for the June 3, 1991 study were prepared by Camp, Dresser, and McKee Inc. (CDM) for FEMA, under Contract No. EM-86-C-2250. This work was completed in May 1989 (Reference 4).

Danvers, Town of	The hydrologic and hydraulic analyses for the January 1990 study were prepared by Anderson-Nichols and Company, Inc., for the Federal Insurance Administration (FIA), under Contract No. H-4524. This work, which was completed in November 1978, covered all significant flooding sources in the Town of Danvers (Reference 5).
Essex, Town of	The hydrologic and hydraulic analyses for the July 17, 1986 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in April 1984 (Reference 6).
Georgetown, Town of	The hydrologic and hydraulic analyses for the December 1979 study were performed by Sverdrup & Parcel and Associates, Inc., for the FIA under Contract No. H-4037. This work was completed in March 1978, and covered all significant flooding sources affecting the Town of Georgetown (Reference 7).
Gloucester, City of	The hydrologic and hydraulic analyses for the January 17, 1986 study were prepared by Stone & Webster Engineering Corporation for FEMA under Contract No. H-4772. This work was completed in September 1983 (Reference 8).
Groveland, Town of	The hydrologic and hydraulic analyses for the April 1980 study were performed by Sverdrup & Parcel and Associates, for the FIA under Contract No. H-4037. This work was completed in April 1978, and covered all significant flooding sources affecting the Town of Groveland (Reference 9).
Hamilton, Town of	The hydrologic and hydraulic analyses for the June 4, 1990 study were prepared by CDM for FEMA, under Contract No. EMW-86-R-2250. This work was completed in November 1987 (Reference 10).
Haverhill, City of	The hydrologic and hydraulic analyses for the August 16, 1982 study were prepared by Cullinan Engineering Co., Inc., for FEMA under Contract No. H-4797. The hydrologic and hydraulic analyses for a portion of the Merrimack River were performed by

Haverhill, City of (continued)	CDM. This work was completed in January 1981 (Reference 11).
Ipswich, Town of	The hydrologic and hydraulic analyses for the February 5, 1986 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in October 1983 (Reference 12).
Lawrence, City of	The hydrologic and hydraulic analyses for the February 2, 1982 study were prepared by Cullinan Engineering Co., Inc., for FEMA, under Contract No. H-4797. This work was completed in January 1981 (Reference 13).
Lynn, City of	The hydrologic and hydraulic analyses for the August 1, 1984 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No.H-4 772. This work was completed in May 1983 (Reference 14).
Lynnfield, Town of	The hydrologic and hydraulic analyses for the February 1, 1980 study were performed by CDM, for the FIA, under Contract No. H-3861. This work was completed in January 1978, and covered all significant flooding sources affecting the Town of Lynnfield (Reference 15).
Manchester by the Sea, Town of	The hydrologic and hydraulic analyses for the September 4, 1986 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in August 1983 (Reference 16).
Marblehead, Town of	The hydrologic and hydraulic analyses for the July 3, 1985 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in June 1983 (Reference 17).
Merrimac, Town of	The hydrologic and hydraulic analyses for the January 5, 1982 study were prepared by Cullinan Engineering Co. Inc. for FEMA, under Contract No. H-4797. This work was completed in July 1980 (Reference 18).

Methuen, City of	The hydrologic and hydraulic analyses for the June 18, 1987 study for the determination and delineation of floodplains in Methuen were originally performed by the USACE, for FEMA, under Inter- Agency Agreement No. IAA-H-10-77 Project Order No. 5. This work was completed in June 1978. The hydrologic and hydraulic analyses for Bartlett Brook, Peat Meadow Brook, Bare Meadow Brook (from Hawkes Brook to Hills Pond) and Hawkes Brook (from a point 3,750 feet upstream of Bare Meadow Brook to North Street) and the hydraulic analysis of the Merrimack River along the Methuen-Andover corporate limits were performed by Schoenfeld Associates, Inc. for the FEMA, under Contract No. EMW-C-0280. This work was completed in October 1983 (Reference 19).
Middleton, Town of	The hydrologic and hydraulic analyses for the May 1980 study were prepared by Anderson-Nichols and Company, Inc. for the FIA, under Contract No. H-4524. This work was completed in October 1978 (Reference 20).
Nahant, Town of	The hydrologic and hydraulic analyses in the March 28, 1984 study represent a revision of the original analyses performed by New England Division of the USACE for FEMA. The updated version was prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in April 1983 (Reference 21).
Newbury, Town of	The hydrologic and hydraulic analyses in the July 17, 1986 study represent a revision of the original analyses performed by New England Division of the USACE for FEMA. The updated 1986 version was prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in April 1983 (Reference 22).
Newburyport, City of	The hydrologic and hydraulic analyses in the November 1, 1985 study represent a revision of the original analyses performed by the USACE for FEMA. The updated version was prepared by Stone & Webster Engineering Corporation for FEMA,

Newburyport, City of (continued)	under Contract No. H-4772. This work was completed in December 1983 (Reference 23).
North Andover, Town of	For the original December 15, 1982 study, the hydrologic and hydraulic analyses were prepared by Cullinan Engineering Co., Inc. for FEMA, under Contract No. H-4797. This work was completed in November 1980. For the June 2, 1993 revision, the hydrologic and hydraulic analyses were prepared by CDM. For FEMA, under Contract No. EMW-88-R-2627. This work was completed in October 1990 (Reference 24).
Peabody, City of	The hydrologic and hydraulic analyses for the November 1979 study were prepared by Anderson-Nichols and Company, Inc., for the FIA, under Contract No. H-4524. This work, which was completed in November 1978, covered all significant flooding sources in the City of Peabody (Reference 25).
Rockport, Town of	The hydrologic and hydraulic analyses for the December 19, 1984 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract NO. H-4772. This work was completed in September 1983 (Reference 26).
Rowley, Town of	The hydrologic and hydraulic analyses for the August 5, 1986 study were prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in September 1980. The wave height and wave runup analyses were added by Dewberry & Davis using information supplied by Stone & Webster in the FIS's for the Towns of Ipswich and Newbury. The coastal analyses were completed in December 1983 (Reference 27).
Salem, City of	The hydrologic and hydraulic analyses in the February 5, 1985 study represents a revision of the original analyses by Anderson & Nichols, Inc., for FEMA, under Contract No. H-3715. The updated version was prepared by Stone& Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in July 1983. The hydrologic and hydraulic analyses in the

Salem, City of (continued)	updated study were completed by Stone & Webster (Reference 28).
Salisbury Town of	The hydrologic and hydraulic analyses in the September 4, 1986 study represents a revision of the original analyses by the USACE, New England Division, for FEMA, under Inter-Agency Agreement No. IAA-H-2-73, Project Order No. 13 and 14 and Amendment No. 1. The updated version was prepared by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This study was completed in January 1984. The hydrologic and hydraulic analyses in the updated study were computed by Stone & Webster Engineering Corporation (Reference 29).
Saugus, Town of	The hydrologic and hydraulic analyses for the July 19, 1982 study were prepared by Stone and Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in November 1980 (Reference 30).
Swampscott, Town of	The hydrologic and hydraulic analyses for the January 3, 1985 study were performed by Stone & Webster Engineering Corporation for FEMA, under Contract No. H-4772. This work was completed in May 1983 (Reference 31).
Topsfield, Town of	The hydrologic and hydraulic analyses in the original December 1979 study were prepared by Harris-Taupes Associates for FEMA under Contract No. H-4024. The work for the original study was completed in April 1978. The hydrologic and hydraulic analyses in the revision dated June 17, 1991 were prepared by CDM, for FEMA, under Contract No. EMW-86-C-2250. That work was completed in March 1989. In the June 2, 1994 revision, the hydrologic and hydraulic analyses were prepared by Roald Haestad, Inc., for FEMA, under Contract No. EMW-90-C-3126. This work was completed in January 1992 (Reference 32).
Wenham, Town of	The hydrologic and hydraulic analyses in the August 19, 1991 study represent a revision of the original 1989 analyses prepared by CDM, for FEMA, under Contract NO. EMW-86-C-2250. The work for the

Wenham, Town of (continued)	original study was completed in December 1987. The hydrologic and hydraulic analyses for the Ipswich River in the 1991 revision were taken from the June 17, 1991 FIS for the Town of Topsfield (Reference 33).
West Newbury, Town of	The hydrologic and hydraulic analyses for the December 1978 study were performed by CDM for the FIA, under Contract No. H-3861. This work, which was completed in January 1977, covered all significant flooding sources affecting the Town of West Newbury (Reference 34).

Base map information shown on the Flood Insurance Map (FIRM) was derived from digital orthophotography. Base map files were provided in digital form by Massachusetts Geographic Information System (MassGIS). Ortho imagery was produced at a scale of 1:5,000. Aerial photography is dated April 2005. The projection used in the preparation of this map was Massachusetts State Plane mainland zone (FIPSZONE2001). The horizontal datum was NAD83, GRS1980 spheroid.

Additionally for the countywide revision, the Shawsheen River was restudied from its confluence with Merrimack River to the county boundary with Middlesex County. Revised hydrologic and hydraulic analyses were prepared by URS for FEMA. The funding for the study was provided by the under the Hazard Mitigation and Technical Assistance Contract Number HSFEHQ-06-D-0162, Task Order 042. That study was completed in May 2008.

For Shawsheen River, LiDAR data were collected in 2006/2007 by URS Group, Inc., and its subconsultant, EarthData. The vertical and horizontal accuracy of the LiDAR data are summarized in a May 29, 2007, report entitled Final LiDAR Report, Shawsheen River, Middlesex/ Essex Counties (URS Group, Inc., 2007).

The digital base map information for the Shawsheen River was provided by MassGIS. This information was derived from 15 centimeter (cm) and 30 cm digital orthophotos from aerial photography dated April 2008.

The coastal hydrologic and hydraulic analyses for this revised coastal study, was performed by Strategic Alliance for Risk Reduction (STARR) for FEMA under Contract No. HSFEHQ-09-D-0370. This study was completed May 7, 2013. This new study resulted in revisions to the Special Flood hazard Areas (SFHAs) within the coastal communities of the Towns of Danvers, Essex, Ipswich, Manchester by the Sea, Marblehead, Nahant, Newbury, Rockport, Rowley, Salisbury, Saugus, Swampscott, and Wenham; the Cities of Beverly, Gloucester, Lynn, Newburyport, Peabody, and Salem.

In 2011, STARR collected Light Detection and Ranging (LiDAR) covering 20.3 square miles of the Essex County coastline. The LiDAR was captured to the ‘highest’ vertical accuracy requirement which is the equivalent of a 2-foot contour accuracy. A 2 meter Digital Elevation Model (DEM) was derived from the LiDAR data. The DEM was projected in State Plane Massachusetts Mainland FIPS 2001 NAD 1983 US foot and used as the basis for coastal analysis and floodplain boundary delineation.

The LiDAR data does not cover elevations below the water surface; therefore, bathymetry data was downloaded from the National Oceanic and Atmospheric Administration (NOAA) Coastal Relief Model (CRM). The source data for the bathymetric products were soundings collected by The National Ocean Service. The Bathymetry data gathered was referenced to the MLLW Datum. A datum conversion from MLLW datum to NAVD88 was not applied because the CRM metadata stated the precision of the Bathymetry data was equal to or greater than the conversion factor. Where the CRM failed to provide data, elevations were taken from current NOAA nautical charts.

Base Map information shown on this FIRM was derived from the Massachusetts Geographic Information System (MassGIS) and the U.S.D.A. Farm Service Agency National Agriculture Imagery Program (NAIP). Aerial photography is dated 2005, April 2008, and 2010.

1.3 Coordination

An initial Consultation Coordination Officer (CCO) meeting is held typically with representatives of FEMA and the communities 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 and the communities to review the results of the FIS.

The dates of the initial, intermediate and final CCO meetings held for the incorporated communities within Essex County are shown in Table 1, "CCO Meeting Dates for Pre-countywide FIS."

TABLE 1 - CCO MEETING DATES FOR PRE-COUNTYWIDE FIS

Community Name	Initial CCO Date	Intermediate CCO Date	Final CCO Date
Amesbury, City of	January 1976	*	March 19, 1979
Andover, Town of	August 8, 1983	*	May 10, 1988
Beverly, City of	April 5, 1978	September 7, 1983	November 7, 1984
Boxford, Town of	January 1986	*	February 23, 1990
Danvers, Town of	November 29, 1978	*	June 21, 1979
Essex, Town of	April 4, 1978	September 22, 1980	January 28, 1985
Georgetown, Town of	June 1976	November 21, 1977	November 20, 1978
Gloucester, City of	April 5, 1978	February 1980/ October 12, 1983	August 16, 1984

* Information not Available

TABLE 1 - CCO MEETING DATES FOR PRE-COUNTYWIDE FIS-continued

Community Name	Initial CCO Date	Intermediate CCO Date	Final CCO Date
Groveland, Town of	May 1976	September 12, 1977/ January 16, 1978	September 29, 1978
Hamilton, Town of	January 1986	*	April 8, 1989
Haverhill, Town of	May 1978	October 16, 1980	September 24, 1981
Ipswich, Town of	April 4, 1978	December 19, 1983	September, 18, 1984
Lawrence, City of	May 4, 1978	November 11, 1980	August 26, 1981
Lynn, City of	March 28, 1978	*	February 28, 1984
Lynnfield, Town of	August 28, 1975	*	September 27, 1978
Manchester by the Sea, Town of	April 5, 1979	January 1980/ September 7, 1984	April 9, 1984
Marblehead, Town of	April 25, 1978	December 12, 1979/ July 5, 1983	February 2, 1984
Merrimac, Town of	May 1978	May 1980	April, 15, 1981
Methuen, City of	August 23, 1979	*	May 2, 1985
Middleton, Town of	November 1, 1977	November 30, 1978	June 27, 1979
Nahant, Town of	*	*	November 17, 1983
Newbury, Town of	April 12, 1978	February 1980	October 9, 1984
Newburyport, City of	April 12, 1978	February 1980	November 27, 1984
North Andover, Town of	March 1987	*	September 26, 1991

* Information not Available

TABLE 1 - CCO MEETING DATES FOR PRECOUNTYWIDE FIS-continued

Community Name	Initial CCO Date	Intermediate CCO Date	Final CCO Date
Peabody, City of	May 13, 1977	November 30, 1978	June 21, 1979
Rockport, Town of	April 5, 1978	February 1980/ October 12, 1983	June 5, 1984
Rowley, Town of	April 4, 1978	September 29, 1980	April 8, 1985
Salem, City of	April 20, 1978	September 7, 1983	September 11, 1984
Salisbury, Town of	April 12, 1978	January 1980/ January 30, 1984	November 19, 1984
Saugus, Town of	March 28, 1978	January 26, 1981	February 3, 1982
Swampscott, Town of	April 20, 1978	December 12, 1979	March 5, 1984
Topsfield, Town of	January 23, 1990	*	February 23, 1990
Wenham, Town of	January 1986	January 13, 1988	August 17, 1988
West Newbury, Town of	August 20, 1975	*	August 20, 1975

*Information not available

For the countywide revision, the initial Consultation Coordination Officer (CCO) meetings were held on September 27 and 28, 2005, and attended by representatives of Essex County communities, Massachusetts Office of Coastal Zone Management, FEMA's Regional Management Center for Region I, Ocean Coastal Consultants, Inc. (OCC), and CDM.

The results of the countywide study were reviewed at the final CCO meeting held on June 16 and 17, 2009, and attended by representatives of FEMA, Essex County communities, Massachusetts Office of Coastal Zone Management, CDM, and Accenture (FEMA's Program Management contractor). All problems raised at that meeting have been addressed in this study.

For this 2013 coastal revision, initial CCO meeting was held on April 5, 2011. The meetings were attended by representatives of Essex County, FEMA, Massachusetts Department of Conservation and Recreation (MA DCR), STARR, and the communities.

A final CCO meeting for this coastal revision was held on July 29, 2013

2.0 **AREA STUDIED**

2.1 Scope of Study

This FIS report covers the geographic area of Essex County, Massachusetts, including the incorporated communities listed in Section 1.1. The areas studied by detailed methods were selected with priority given to all known flood hazards and areas of projected development or proposed construction.

All or portions of the flooding sources listed in Table 2, "Flooding Sources Studied by Detailed Methods," were studied by detailed methods in the pre-countywide FIS's. Limits of detailed study are indicated on the Flood Profiles (Exhibit 1) and on the FIRM.

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS

Flooding Source Name	Description of Study Reaches
Argilla Brook	From its confluence with Johnson Creek to the Center Street Bridge
Artichoke River - Reservoir	From its confluence with the Merrimack River to its confluence with the North Tributary Brook
Atlantic Ocean	Along the entire eastern coastline of Essex County
Bare Meadow Brook	In the City of Methuen, from its confluence with the Merrimack River to approximately 400 feet upstream of the Hills Pond Dam
Bartlett Brook	From its confluence with the Merrimack River to a point approximately 6,150 feet upstream of North Lowell Street

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
Bates Brook	From its confluence with Pillings Pond to approximately 575 feet upstream of Chatham Way
Beaver Brook (Town of Danvers)	From the Sylvan Street Dam to Nichols Street
Beaver Brook (Town of West Newbury)	From Middle Street to approximately 4,400 feet upstream from Tewksbury Street
Beaverdam Brook	From its confluence with the Saugus River to the Main Street culvert
Bennett's Pond Brook	From its confluence with the Saugus River to approximately 1,000 feet upstream of Lewis O. Gray Drive
Beverly Harbor	At the Danvers River Estuary, from State Highway 1A to the Atlantic Ocean
Boston Brook	From its confluence with the Ipswich River in the Town of Middleton to approximately 4,000 feet upstream from Hawkins Lane in the Town of North Andover
Branch of Ipswich & Cleveland Brook	From its confluence with the Ipswich River to approximately 300 feet upstream from Washington Street

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
Bulford Brook	From its confluence with Penn Brook to approximately 2,400 feet upstream from State Highway 133 (East Main Street)
Centerville Creek	From approximately 350 feet downstream of Hale Street to approximately 75 feet upstream from Common Lane
Chubb Creek	On the border of the City of Beverly and the Town of Manchester by the Sea, from its confluence with Manchester Bay to State Highway 127 (Hale Street)
Chubbs Brook	From its confluence with Chubb Creek to approximately 65 feet upstream from State Highway 127 (Hale Street) in the City of Beverly
Coastal Flooding	Affecting the entire eastern coastline of Essex County, resulting from the Atlantic Ocean
Cochichewick Brook	In the Town of North Andover, from its confluence with the Merrimack River to Stevens Pond
Crane River & Crane Brook	In the Town of Danvers, from its confluence with Porter River to approximately 650 feet past the Border-to-Boston Bike Trail
Creek Brook	From its confluence with the Merrimack River to Crystal Lake
Egg Rock	Off the coast of the Town of Nanhant, island located in Nahant Bay

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
Emerson Brook	From its confluence with the Ipswich River to approximately 20 feet upstream from Liberty Street in Middleton
Fish Brook	From its confluence with the Ipswich River to confluence with Mosquito Brook
Fish Brook (Town of Andover)	From confluence with Merrimack River to Greenwood Road
Fiske Brook	In the Town of Saugus, from its confluence with Shute Brook to approximately 820 feet upstream from its confluence with Shute Brook
Goldthwaite Brook	In the City of Peabody, from its confluence with Proctor Brook to approximately 100 feet upstream from First Avenue
Harris Brook	In the City of Methuen, from its confluence with the Spicket River to approximately 75 feet upstream from Hampshire Road
Riverside Airport Brook	In the City of Haverhill, from its confluence with the Merrimack River to upstream to approximately 100 feet upstream from Kenoza Street
Hawkes Brook	In the City of Methuen, from its confluence with Bare Meadow Brook to approximately 100 feet upstream from North Street

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
Howlett Brook & Pye Brook	In the Town of Topsfield, from its confluence with the Ipswich River to approximately 600 upstream from State Highway 97 (Haverhill Road)
Hussey Brook	In the Town of Andover, from its confluence with the Shawsheen River to approximately 3700 feet upstream from Beacon Street
Hussey Brook Tributary	In the Town of Andover, from its confluence with Hussey Brook to approximately 200 feet upstream from Beacon Street
Ipswich River	In the Town of Ipswich, from State Route 133 to the Essex County (Town of Lynnfield)/Middlesex County (Town of North Reading) County Limits
Jackman Brook	In the Town of Georgetown, from Parish Road to Jewett Street
Johnson Creek	In the Town of Groveland, from its confluence with the Merrimack River to Washington Street
Lake Attitash	On the City of Amesbury/Town of Merrimac Corporate Limits
Little River	In the City of Haverhill, from its confluence with the Merrimack River to approximately 5000 feet upstream from Rosemont Street
Massachusetts Bay	Along the eastern coastline of Essex County from its southern border to Cape Anne

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
Merrimack River	In the City of Amesbury, from the Newburyport Lighthouse to approximately 6700 feet upstream of Ravens Bluff
Mile Brook	In the Town of Topsfield, from its confluence with the Ipswich River to its divergence from Howlett Brook and Pye Brook
Miles River	From its confluence with the Ipswich River to Dodge Row in the Town of Wenham
Mill River (Town of Gloucester)	From Dr. Osman Babson Road to approximately 400 feet upstream of Access Road
Mill River (Town of Rowley)	From just downstream of U.S. Route 1 to approximately 3,700 feet upstream of Mill Dam
Millvale Reservoir Brook	From its confluence with the Merrimack River to Millvale Reservoir
Mosquito Brook	From Boxford/North Andover corporate boundary to approximately 50 feet upstream of Chestnut Street in the Town of North Andover
North Beverly Drainage Ditch	From its confluence with Bass River to approximately 1300 feet upstream of Russell Street in the City of Beverly
North River & Proctor Brook	From Grove Street in Salem to approximately 120 feet upstream from Peabody Road in the City of Peabody

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
North Tributary Brook	From its confluence with Artichoke River – Reservoir to approximately 6000 feet upstream of Garden Street in the Town of West Newbury
Parker River (Town of Boxford)	From approximately 14000 feet downstream from Byfield Road to approximately 50 feet upstream from State Route 133
Parker River (Town of Georgetown)	From approximately 1500 feet upstream from Thurlow Street to approximately 2400 feet upstream from Bailey Lane
Parker River (Town of Newbury)	In the Town of Newbury, from 500 feet downstream of the Central Street Dam to approximately 1800 feet upstream from River Street
Peat Meadow Brook	In the City of Methuen, from its confluence with the Spicket River to approximately 30 feet upstream from Forest Street
Penn Brook	In the Town of Georgetown, from its confluence with Parker River (Town of Georgetown) to approximately 450 upstream from Newburyport Branch Railroad
Pillings Pond	In the Town of Lynnfield, from its confluence with Bates Brook to approximately 5000 feet downstream from its confluence with Bates Brook

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
Porter River & Frost Fish Brook	In the Town of Danvers, from approximately 1,500 feet upstream from Kernwood Avenue to Coolidge Road
Powwow River	In the City of Amesbury, from its confluence with the Merrimack River to its confluence with Lake Gardner
Saugus River	In the Town of Saugus, from the Hamilton Street Bridge (Upstream Face) to approximately 460 feet upstream from Main Street in the Town of Lynnfield
School Brook	In the Town of Topsfield, from its confluence with Branch of Ipswich and Cleveland Brook to approximately 150 feet upstream of State Highway 97 (High Street)
Shawsheen River	In the Town of North Andover, from its confluence with the Merrimack River to approximately 5000 feet upstream of Interstate Highway 93 in the Town of Andover
Shute River	In the Town of Saugus, from approximately 2500 feet downstream from Central Street Culvert (Upstream Face) to approximately 250 feet upstream from Pennybrook Road

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
Skug River	In the Town of Andover, from approximately 1600 feet downstream from State Highway 28 to approximately 1000 feet upstream from State Highway 28
Spicket River	In the City of Lawrence, from its confluence with the Merrimack River to approximately 80 feet upstream of Hampshire Road in the City of Methuen
Strongwater Brook	In the City of Peabody, from its confluence with North River and Proctor Brook to Pierpont Street
Tidal Flooding	Flooding from the Atlantic Ocean affecting all Essex County coastline, bays, estuaries, tidal rivers, tidal flats and tidal streams and surrounding areas
Tapley Brook	In the City of Peabody, from its confluence with Goldthwaite Brook to approximately 1600 feet upstream from Sidneys Pond Dam
Tributary to the Ipswich River	In the Town of Middleton, from its confluence with the Ipswich River to approximately 1050 feet upstream from Pleasant Street
Tributary to Neal Pond	In the Town of Merrimac, from Birch Meadow Road to Birch Meadow Loop
Unnamed Tributary to Fish Brook	In the Town of Topsfield, from its confluence with Fish Brook to approximately 1500 feet upstream of Boxford Road

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS-continued

Flooding Source Name	Description of Study Reaches
Upper Artichoke Reservoir	On the Town of West Newbury/City of Newburyport Corporate Limits from its confluence with Merrimack River to its confluence with North Tributary Brook
Shallow Flooding	Sluice Pond, Flax Pond, and Cedar Pond in the Town of Lynn
Waters River	In the Town of Danvers, from its confluence with Danvers River to approximately 4600 feet upstream from State Highway 35 (Water Street)

For the countywide revision, revised coastal analyses were performed for the open water flooding sources in the communities of Salisbury and Newburyport. In addition, redelineation of coastal flood hazard data was performed for open water flooding sources in the communities of Beverly, Danvers, Essex, Gloucester, Ipswich, Lynn, Manchester by the Sea, Marblehead, Nahant, Newbury, Peabody, Rockport, Rowley, Salem, Saugus and Swampscott.

Detailed study flooding sources that were not restudied as part of this revision may include a profile baseline on the FIRM. The profile baselines for these flooding sources were based on the best available data at the time of their study and are depicted as they were on the previous FIRMs. In some cases the transferred profile baseline may deviate significantly from the channel or may be outside of the floodplain.

Approximate analyses were used to study those areas having a low development potential or minimal flood hazards. The scope and methods of study were proposed to, and agreed upon, by FEMA and the individual communities within Essex County. For this countywide revision, no new approximate studies were executed.

All or portions of the flooding sources listed in Table 3, "Flooding Sources Studied by Approximate Methods," were studied by approximate methods in the pre-countywide FISs.

TABLE 3 - FLOODING SOURCES STUDIED BY APPROXIMATE METHODS

Flooding Source Name	Community (ies)
Alewife Brook	Essex Argilla
Brook	Groveland
Bachelder Brook	Rowley
Back River	Amesbury, Merrimac
Bagness School	Groveland
Baldpate Pond	Boxford Bare
Meadow Brook	Methuen Bare
Meadow Brook Tributary	Methuen Bass
River	Beverly
Beaver Brook	Danvers, Groveland Beaver
Brook Tributary	West Newbury Beaver
Pond	Beverly
Beavor Brook	West Newbury
Bennetts Pond	Saugus
Birch Pond	Lynn, Saugus
Boston Brook	Middleton, North Andover
Breeds Pond	Lynn Browns
Pond	Peabody Bull
Brook Reservoir	Ipswich
Cat Brook	Manchester
Cedar Pond	Boxford, Wenham
Centerville Creek	Beverly
Chadwick Pond	Boxford, Haverhill
Chebacco Lake	Essex Chubb
Creek	Beverly Chubbs
Brook	Beverly Cobbler
Brook	Merrimac Coy
Pond	Wenham Crane
Brook	Danvers Crystal
Lake	Haverhill Dow
Brook Reservoir	Ipswich East
Meadow	Haverhill Egypt
Pond	Ipswich Emerson
Brook	Middleton
Fish Brook	Andover, Boxford
Fiske Brook	Saugus Flood
Prone Areas	Groveland Forest
River	Salem Fourmile
Pond	Boxford

TABLE 3 - FLOODING SOURCES STUDIED BY APPROXIMATE METHODS-continued

Flooding Source Name	Community (ies)
Fowler Brook	Danvers Frost Fish
Brook	Danvers
Goldthwaite Brook	Peabody
Gravelly Brook	Ipswich
Grindle Brook	Groveland, Haverhill
Hawkes Brook	Methuen Hawkes
Pond	Saugus Hood
Pond	Ipswich Hoveys
Pond	Boxford Howlett
Brook	Topsfield Hussey
Brook	Andover
Jackman Brook	Georgetown Johnson
PondBoxford, Grovel	
and Kimballs Pond	Boxford
Lake Cochichewick	North Andover
Lake Pentucket	Haverhill
Lake Saltonstall	Haverhill
Low development potential	Gloucester, Lynnfield
Lowe Pond	Boxford
Lower Millpond	Rowley
Low-Lying Area	Newburyport
Lufkin Creek	Essex, Methuen, Middleton, Topsfield
Mile Brook	Topsfield
Mill River	Rowley
Minimal Flood Hazards	Gloucester, Lynnfield
Mosquito Brook	North Andover Muddy
Pond	Wenham Muddy
RunIpswich	
Mystic Pond	Methuen
Neal Pond	Merrimac
Nichols Brook	Danvers, Middleton, Topsfield
Norris Brook	Danvers, Peabody
North Beverly Drainage Ditch	Beverly North
River	Salem
Norwood Pond	Beverly Ox
Pasture Brook	Rowley
Parker River	Georgetown, Groveland
Peat Meadow Brook	Methuen
Pennys Brook	Saugus

TABLE 3 - FLOODING SOURCES STUDIED BY APPROXIMATE METHODS-continued

Flooding Source Name	Community (ies)
Ponding Areas	Hamilton, Lawrence
Proctor Brook	Peabody
Putnamville Reservoir	Danvers, Topsfield
Pye Brook	Boxford, Topsfield
Rantoul Pond	Ipswich
Rogers Brook	Andover
Salen Pond	North Andover
Saw Mill Brook	Manchester
Sharpners Pond	North Andover
Shoe Pond	Beverly
Skug River	Andover
Smaller Watercourses	Georgetown
Snows Brook	Haverhill
Soginese Creek	Essex
Sperrys Pond	Boxford
Spofford Pond	Boxford
Stearns Pond	North Andover
Stevend Pond	Boxford
Stevens Pond	North Andover
Stiles Pond	Boxford
Strawberry Brook	Lynn
Stream Tributary	Groveland
Streams feeding water bodies	Haverhill
Strongwater Brook	Peabody
Sudden Pond	North Andover
Swamp Brook	Rowley
Tapley Brook	Peabody
Thompson's Meadow	Salem Towne
Pond	Boxford
Tributary A	Middleton
Unnamed Areas	Beverly, Saugus
Unnamed Bogs	Essex
Unnamed Drainage Areas	Merrimac
Unnamed Ponding Area	Wenham
Unnamed Ponds	Boxford, North Andover
Unnamed Streams	Amesbury, Danvers, Essex, North Andover, Peabody
Unnamed Swamps	Danvers, Ipswich, Peabody
Unnamed Tributaries	Andover, Groveland, Manchester

TABLE 3 - FLOODING SOURCES STUDIED BY APPROXIMATE METHODS-continued

Flooding Source Name	Community (ies)
Upper Artichoke Reservoir	West Newbury
Upper Millpond	Rowley
Walden Pond	Lynn, Saugus
Waters River	Danvers
Wenham Lake	Beverly, Wenham
West Meadow River	Haverhill
Wetlands	Groveland, Haverhill
Wheeler Brook	Georgetown
Wilson Pond	Rowley

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 4, "Letters of Map Change."

TABLE 4 – LETTERS OF MAP CHANGE

Community	Case Number	Flooding Source	Letter Date
Andover, Town of	04-01-007P	Skug River	03/16/2004
Beverly, City of	08-01-0002P	Massachusetts Bay	08/01/2008
Boxford, Town of	96-01-031P	Upper and Lower Kimball Pond	8/8/1996
Wenham, Town of	06-01-B791P	Pond 1 and Pond 2	02/15/2007

For this 2013 coastal revision, the coastal analysis establishes the flood elevations for selected recurrence intervals primarily in the coastal communities of the Towns of Essex, Danvers, Ipswich, Manchester by the Sea, Marblehead, Nanhunt, Newbury, Rockport, Rowley, Saugus, and Swampscott; the Cities of Beverly, Gloucester, Newburyport, Lynn, Peabody, and Salem, There were no new LOMR determinations that resulted in FIRM revisions. No new riverine or approximate studies were performed as part of this coastal revision.

2.2 Community Description

Essex County is located in northeastern Massachusetts. In Essex County, there are twenty-six (26) towns and eight (8) cities. The Towns of Georgetown, Groveland, Manchester, Merrimac, Middleton, Newbury, Rockport, Salisbury, West Newbury and the Cities of Amesbury, Gloucester, Haverhill, Methuen, Newburyport, and Salem are located in northern Essex County. The Towns of

Hamilton and Topsfield are in the central portion of the county. The Cities of Beverly, Lynn, Peabody, Salem and Towns of Danvers, Lynnfield, Marblehead, Swampscott are located in the southern portion of the county. The Town of Andover is located in the northeastern portion of the county. The Towns of Boxford, North Andover and the City of Lawrence are located in the western portion of the county. The Towns of Essex, Ipswich, Nahant, Rowley and Wenham are located in the eastern portion of the county.

Essex County is bordered on the north by the State of New Hampshire and on the east by Atlantic Ocean. It is bordered on the west by Middlesex and to the south by Suffolk County.

According to census records, the population of Essex County was 743,159 in 2010, 723,419 in 2000 and 670,080 in 1990 (Reference 35). The total area in Essex County consists of 1,444 mi^2, including 328 mi^2 of water area. All communities in Essex County, along with their population and total area, are listed in Table 5, "Population and Total Area by Community."

TABLE 5 – POPULATION AND TOTAL AREA BY COMMUNITY

Community	Total Area (sq. mi)[1]	Population[1]
Amesbury, City of	13.7	16,283
Andover, Town of	32.1	33,201
Beverly, City of	22.7	39,502
Boxford, Town of	24.6	7,965
Danvers, Town of	14.1	26,493
Essex, Town of	15.9	3,504
Georgetown, Town of	13.2	8,183
Gloucester, City of	41.5	28,789
Groveland, Town of	9.4	6,459
Hamilton, Town of	14.9	7,764
Haverhill, Town of	42.1	60,879
Ipswich, Town of	32	13,175
Lawrence, City of	7.4	76,377
Lynn, City of	13.5	90,329
Lynnfield, Town of	10.5	11,596
Manchester, Town of	18.3	5,136
Marblehead, Town of	19.6	19,808
Merrimac, Town of	8.8	6,338
Methuen, City of	23.1	47,255
Middleton, Town of	14.4	8,987
Nahant, Town of	15.5	3,410

[1]U.S Census Bureau (Reference 35)

TABLE 5 – POPULATION AND TOTAL AREA BY COMMUNITY-continued

Community	Total Area (sq. mi)[1]	Population[1]
Newbury, Town of	26.5	6,666
Newburyport, City of	10.6	17,416
N. Andover, Town of	27.8	28,352
Peabody, City of	16.9	51,251
Rockport, Town of	17.6	6,952
Rowley, Town of	20.6	5,856
Salem, City of	18.1	41,340
Salisbury, Town of	17.9	8,283
Saugus, Town of	11.8	26,628
Swampscott, Town of	6.7	13,787
Topsfield, Town of	12.8	6,085
Wenham, Town of	8.1	4,875
W. Newbury, Town of	14.6	4,235

[1]U.S Census Bureau (Reference 35)

2.3 Principal Flood Problems

Past flooding on the streams within Essex County indicates that flooding can occur during any season of the year. Most major floods have occurred during February, March, and April and are usually the result of spring rains and/or snowmelt. Floods occurring during the midsummer and late summer are often associated with tropical storms moving up the Atlantic coastline. Severe flooding in Essex County generally occurs as a result of hurricanes or melting snows and spring rains, with more localized flooding caused by summer thunder-storms.

Trees, brush, and other vegetation growing along stream banks impede flood flows during high waters, thus creating backwater and increasing flood heights. Furthermore, trees, ice, and other debris may be washed away and carried downstream to collect on bridges and other obstructions. As the flood flow and debris surges downstream until another obstruction is encountered. Debris may collect against a bridge or culvert until the load exceeds the structural capacity, causing its destruction. It is difficult to predict the degree to which, or the location where, debris may accumulate. Therefore, in the development of the flood profiles it has been necessary to assume no accumulation of debris or obstruction of flow.

The flood problems for the communities within Essex County have been compiled and are described below, this information may not include the latest flood events:

Historical records indicate that since December 1740, numerous storms and floods have occurred in the Merrimack River Basin. The two latest and the most significant occurred in March 1936 and September 1938. The most severe, the 1936 flood resulted from a combination of heavy rains and extensive snow melts. The other, in 1938, was from torrential rains associated with a hurricane passing over the Merrimack River basin at a time when the ground was saturated from an earlier storm. While studies determining the recurrence frequency are not available, it is estimated that floods of the 1936 magnitude and the 1938 magnitude recur approximately every 200 years and every 50 years, respectively. Flooding from the mouth of the Merrimack River in Salisbury and Newburyport, Massachusetts, to the Deer Island Bridge between Amesbury and Newburyport is primarily affected by high tides. Upstream of this bridge, flooding has been caused by either high tides or high-river flows, separately or in coincidence.

Outstanding floods caused by heavy rainfall alone or by a combination of heavy rain and melting snow, have occurred along both the Shawsheen and Merrimack Rivers in Andover. Records of river stages and discharges on the Merrimack River at Lawrence, Massachusetts, have been maintained by the Essex Company at the Essex Company Dam (2 miles downstream of the Andover town line) since 1848. No records of stream stages or discharges are available for the lower portion of the Shawsheen River in Andover. However, flood information has been obtained from records of the Massachusetts Geodetic Survey and the USACE, as well as from local authorities and residents. The Essex Company Dam is the hydraulic control for the entire Merrimack River in Andover. Discharges in cubic feet per second (cfs) and stages of the greatest known floods of record measured at the Essex Company Dam since 1852 are shown in the following tabulation:

Date	Stage[1] (Feet NAVD 88)	Estimated Peak Discharge (cfs)
March 20-21, 1936	52.3	174,000
September 23, 1938	49.2	121,000
April 23, 1852	48.48	108,000
March 3, 1896	48.10	105,000
April 16, 1895	47.21	89,900
April 6, 1960	46.53[2]	79,000[2]
November 6, 1927	46.51	78,900
April 20, 1933	46.36	76,200
March 4, 1902	46.14	73,100

[1] Does not reflect the peak discharge or stages that would have occurred had they been modified by the five upstream flood control reservoirs that were constructed subsequent to the tabulated events

[2]Modified by Franklin Falls, Blackwater, and MacDowell Dams

The Shawsheen River is characterized by a shallow, meandering channel with predominantly low banks. Floodplains are generally wide and flat and covered with grasses and trees. Investigations indicate that the flood of record on the Shawsheen River also occurred as often on the Shawsheen River as on the Merrimack River.

The most significant recorded floods were those occurring in March 1936 and in September 1938. The 1936 flood was the result of a combination of heavy rains and extensive snow melt, while the 1938 flood was caused by heavy precipitation over the Merrimack River basin at a time when the ground was saturated from an earlier storm. The USACE estimated the 1936 flood to have a recurrence interval in excess of the 1-percent-annual-chance, while the 1938 flood had a recurrence interval of approximately the 2-percent-annual-chance (References 2 and 13).

Low-lying areas of Beverly, Essex, Gloucester, Ipswich, Lynn, Manchester, Marblehead, Nahant, Newbury, Newburyport, Rockport, Rowley, Salem, Salisbury, Saugus, and Swampscott are subject to the periodic flooding and wave attack that accompany coastal storms, such as northeasters and hurricanes. The Town of Essex is protected from the heaviest wave attack by the barrier islands at Crane and Coffin Beaches. The majority of these storms cause damage only to low coastal roads, boats, beaches, and seawalls. Occasionally, a major storm accompanied by strong onshore winds and high tides results in surge and wave activity that causes extensive property damage and erosion. Some of the more significant storms in the area include those of December 1909 and 1959, November 1945, 1963, and 1968, and February 1972 and 1978. These storms damaged harbors and marinas and residential and commercial developments in the coastal areas.

Chebacco Lake occasionally floods in the spring but is not believed to threaten property in Essex.

Coastal flooding in Beverly has been particularly evident in the low-lying areas on the north end of Dane Street Beach. Riverine flooding has not generally been a serious problem in the city. The only significant problem reported is the frequent flooding of a portion of Cabot Street by North Beverly Drainage Ditch. This flooding is due primarily to an undersized culvert at Cabot Street and heavy siltation of the Boston and Maine rail road culverts just downstream of Cabot Street.

Few detailed records of flooding are available for the Town of Boxford or for the Town of Hamilton. Major storms in the region occurred in 1936, 1938, 1949, 1955, 1968, 1976, 1979 and 1987. Little information is available for any of these storms. In Boxford, the April 1987 storm caused overtopping of several roads located around local streams. During the April 1987 storm, the town suffered approximately $37,000 worth of flood damages, of which the town recovered approximately $14,000 of eligible damages from Federal Emergency Relief. Numerous residential and commercial basements were pumped by the Fire Department. At least 5 roads were overtopped by floodwaters. Most of these were caused by high backwater from downstream ponds, although in one case the culvert system serving as the Stiles Pond outlet was overtopped and required replacement. In Hamilton, the April 1987 storm caused overtopping of the approaches to the Winthrop Street Bridge by the Ipswich River. The bridge could not be crossed for several days until the floodwaters receded. The Winthrop Street crossing is located downstream of the Town of Hamilton, but was covered in detail in the Flood Insurance Study for the Town of Ipswich (Reference 12). During the April 1987 storm, flow from the Ipswich River was contained by Wenham Swamp and its large storage capacity. The area is not monitored for flood elevations, and therefore no high-water marks are available. The area contiguous to the Miles River has not experienced any major flooding problems. There were incidents of flooded basements and ponded yards during the April 1987 storm, but these occurrences can be attributed to the high local groundwater table and the inability of the soil to percolate rainfall.

Major storms in the North Andover region occurred in 1936, 1938, 1949, 1955, 1968, 1976, 1979, and 1987. The Shawsheen and Merrimack Rivers are significantly larger than Boston and Mosquito Brooks and have historical records of flooding predating the 1936 flood. Each of these rivers experienced their most severe flooding during the 1936 storm (Reference 36). However, no flooding information for these early period storms is available for either Boston or Mosquito Brook, probably because each brook flows through lightly populated or underdeveloped areas where flooding impacts imposed little economic damage. The history of the Merrimack River contains information on flood dating back to 1846. The most significant recorded floods occurred in March 1936 and September 1938. The 1936 flood resulted from a combination of heavy rains and extensive snowmelt. The 1938 flood was caused by torrential rains associated with a hurricane passing over the Merrimack River basin at a time when the ground was saturated from an earlier storm. The USACE estimated the 1936 flood had a recurrence interval in excess of the 1-percent-annual-chance flood and estimated the 1938 flood had a recurrence interval of about 2-percent-annual-chance (Reference 37). The flood of 1936 is also the flood of record on the Shawsheen River. Flooding along the lower portion of the Shawsheen River is influenced by the rise in water-surface elevation of the Merrimack River. During severe floods, backwater from the Merrimack River has extended up the Shawsheen River through North Andover and Lawrence as far south as Andover, a distance of 5 miles. Following the 1936 flood, the Massachusetts Geodetic Survey gathered high-water mark data for the Merrimack River and the Shawsheen River in North Andover (Reference 38). High-water marks for the Merrimack River included elevations of 39.1 feet at the North Andover-Haverhill corporate limits, 43.4 feet at the Boston and Maine Railroad signal located 475 feet northeast of North Main Street, 43.2 feet at the building at 45 Riverview Street, and 43.2 feet at the building at 16 Sutton Street. High-water marks for the Shawsheen River included elevations of 43.4 feet at Marblehead Street near Sutton Street Bridge, 43.8 feet at Massachusetts Avenue approximately 1,200 feet east of the bridge over the Shawsheen, 43.5 feet at Loring Street Bridge at the North Andover-Lawrence corporate limits, and 43.4 feet at Green Street Bridge at the North Andover-Lawrence corporate limits.

Both riverine and tidal waters cause flood problems in Danvers. The history of tidal flooding in the study area can be traced by referring to the records of storm tides measured at Boston since 1851. Riverine flooding on the Ipswich River has been documented by gage records and high-water marks. Records at the Middleton gage on the Ipswich River give an indication of the magnitude of flood discharges in Danvers, located just downstream. The Middleton gage began operation in 1938. The flood of record occurred on January 26, 1979, and had a measured peak discharge of 835 cfs.

Georgetown has not experienced any severe storm damage in the past. Minor flooding has occurred throughout the town. Two of these areas are Baldpate Road, which floods due to runoff from Littles Hill, and Andover Street, North Street, and Bailey Lane at Parker River. Rock pond and Pentucket Pond have not had flood problems in the past.

Groveland has experienced extensive flooding in the past. The most notable storm was in 1936, a storm classified as being greater than a 1-percent-annual-chance storm. Major flooding inundated the banks of the Merrimack River. The hardest hit area was along Main Street, near Gardener Street. Bates Bridge over the Merrimack River was nearly overtopped. Also hit hard were areas along Johnson Creek from its confluence with Merrimack River to a point upstream of Main Street. Minor flooding is caused by blocked and inadequate culverts throughout the town.

History of flooding on the Merrimack River in Hamilton, Lawrence, and Merrimac contains information dating back to 1846. The most significant recorded floods were those occurring in March 1936 and September 1938. The 1936 flood was the result of a combination of heavy rains and excessive snowmelt. The 1938 flood was caused by torrential rains associated with a tropical storm passing over the Merrimack River basin at a time when the ground was saturated from an earlier storm. The USACE estimated that the 1936 flood has a recurrence interval in excess of 1-percent-annual-chance and the 1938 flood has a recurrence interval of approximately 2-percent-annual-chance. Flooding along the Little River is influenced by the rise in water-surface elevations from the Merrimack River, During severe flooding, the backwater from the Merrimack River extends up the Little River above the entrance to the arch culvert running under the city. During these periods of high backwater, pumps have been utilized to convey floodwaters from the Little River to the Merrimack River. The Massachusetts Geodetic Survey gathered Merrimack, Shawsheen and Spicket River high-water mark data in the City of Lawrence resulting from the 1936 flood (Reference 38).

Riverine flooding in Ipswich caused by inland precipitation runoff has led to little damage from the Ipswich and Miles Rivers due to limited construction in vulnerable areas. In March 1936 and March 1968, floods with recurrence intervals of approximately 50 years were recorded on the Ipswich River at the gage located at the Willowdale Dam. There is no gage on the Miles River, but it can be assumed that this river had floods of similar severity in 1936 and 1968. Neither flood had significant effects on developed land. Kimball Brook, another tributary of the Ipswich River, has flooded Peabody Street, Safford Street, and Cherry

Street. Due to the nature of the flooding and limited scope of the stream, it was not studied in detail.

Principal flood problems in the Lawrence are localized in nature because of undersized culverts. In October 1962 and March 1968, major flooding was experienced on the Saugus River and the Ipswich River. Damage was confined to flooded basements because of standing water.

During the February 6-7, 1978 in Manchester, blizzard, the coastal areas particularly prone to flooding and damage were Ocean Street and Raymond Street. In the Ocean Street area, considerable debris washed over the revetment structure and onto residential property. Residences off Raymond Street, on the beach at Kettle Cove, experienced flooding and wave attacks. There was less severe flooding occurred around Lobster Cove and Boardman Avenue near the entrance to Manchester Harbor. Further, there was extensive damage to the riprap which protects the back side of Singing Beach.

Marblehead Harbor in Marblehead is exposed to winds from the northeast. During the February 6-7, 1978, a blizzard there was overtopping of the seawall at the south end of the harbor, and waters flooded the causeway that links Marblehead and Marblehead Neck. Although the seawall was not breached, it did sustain structural damage. In addition, there was some damage to structures on Front Street, where foundations rested on rocks at the water's edge. This damage, however, was not extensive.

Causes of significant flooding in Methuen are generally similar to those of the lower Merrimack River basin. Of the 76 largest floods experienced since 1846 at Lowell, upstream on the Merrimack River, 59 occurred in the months of March, April, or May and resulted from snowmelt augmented by rainfall, as happened during the record March 1936 flood. Floods resulting from heavy rainfall alone, however, may also be expected during the other seasons of the year, as in the cases of the floods of November 1927 and September 1938 (Reference 19) in Methuen. The flood history of the Merrimack River contains information of floods as far back as 1785. There is little documentation available for the early floods. However, dates and peak discharges of the five largest floods at the USGS gaging station in Lowell, Massachusetts (No. 01100000) 1,100 feet downstream of the confluence of the Merrimack and Concord Rivers and 4.2 miles upstream of the Andover-Dracut-Methuen corporate limits are listed below. The total drainage area is 4,635 square miles (Reference 19).

Although overbank flooding occurs nearly every year along some reaches of the Ipswich River in Middleton, floods of a catastrophic nature are rare due to the large storage capacity of the many marshes through which the Ipswich flows (Reference 39). The flood of record at the South Middleton gage (No. 01101500) on the Ipswich River occurred on January 26, 1979, with a recorded peak discharge of 835 cfs. Prior to 1979 the worst flooding occurred in March 1968, which was the most severe since February 1886, although it is uncertain exactly how the two floods compare since the latter is known only through reports of local residents (Reference 39). In March 1936, a flood similar in magnitude to the 1968 flood occurred on the Ipswich River.

During the February 1978 blizzard in Nahant, seawall or tidal reinforcement damage was sustained on Castle Road, Nahant Road, Wendell Road, Wilson Road, Willow Road to Wharf Street, on the coast near the intersection of Cliff Road and Nahant Road and near the intersection of Marginal Road and Winter Street, and at the beach between Little Nahant and Nahant. Major areas prone to flooding within Nahant include the causeway leading into the main part of the town, the interior region between Pond Beach and Black Back Beach, and the area between Wharf Street and Furbush Road. Housing in the flood-prone areas of the town consists principally of year-round residences. The majority of the homes throughout the flood prone areas are single-family, wood-frame structures with basements.

In addition to flooding in Newbury, serious shorefront erosion has occurred at Plum Island since the early 1880s, when the mouth of the Merrimack River was located approximately 0.5 mile south of its present position. Jetties, which were constructed at the turn of the century, had stabilized the entrance of the river at its present location and tended to create a buildup of the oceanfront shores on the northern end of the island. However, since 1938, continuous recession of the shoreline has occurred, resulting primarily from severe storm surges and coincident wave action.

One of the most severe storms in Newbury occurred on February 19, 1972, destroying a wide fronting beach and back lying dunes and damaging or destroying several cottages. This storm rendered the island susceptible to further damage. Riverine flooding has not generally been as serious of a problem as coastal flooding in the Newbury area. Extreme water levels on the Parker River upstream of Central Street and on the Little River upstream of Boston Street are primarily caused by runoff from heavy rainfall and snowmelt.

The problem of flooding and erosion on Plum Island in Newburyport dates back to the early 1800s when the Merrimack River was approximately 0.5 mile south of its present position. Jetties which were built at the turn of the century stabilized the entrance to the Merrimack River to its present location and created a buildup of oceanfront shores at the north end of the island. Since 1928, continuous recession of the shoreline has occurred, primarily as a result of severe storm surge and wave attack. In 1970, a revetment was placed along the south shore of the mouth of the river to protect the U.S. Coast Guard Station, which has since moved upstream to a more protected area. One of the most severe storms recorded in this area occurred on February 19, 1972, and destroyed a wide fronting beach and the back-lying sand dunes. This storm rendered the island even more susceptible to severe damage in the future. Flooding on the Merrimack River upstream of the

Newburyport Bridge has been caused by high tides or high river flows, separately or in coincidence. The high river flows have resulted from heavy rainfall or from a combination of rainfall and snowmelt. Since 1923, the USGS has continuously recorded Merrimack River flows at the gage 28 miles upstream of Newburyport in Lowell, Massachusetts. This station measures runoff from a 4,635 square mile drainage area, or 93 percent of the drainage area that contributes flow to the Merrimack River at Newburyport.

The major floods in Peabody have resulted from rainfall alone or in combination with snowmelt. Flooding on the Ipswich River has been documented by gage records, high-water marks, and personal accounts. Records at the Middleton and Ipswich gages on the Ipswich River give an indication of the magnitude of flood discharges in Peabody, which is located between the two gages. The Middleton gage began operation in 1938. The highest flood of record occurred in January 1979 and had a measured peak flow of 835 cfs. Other major floods, in order of decreasing peak discharge, are as follows: March 1968, October: 1962, March 1969, January 1958, March 1948, July 1938, December 1969, and March 1954. The gage at Ipswich began recording flows in 1931. Its highest flood of record also occurred in 1968, but a flood that occurred in March 1936, showed a peak discharge of nearly the same magnitude (References 40 and 41). Based upon accounts of Ipswich residents, a flood that occurred in February 1886 reached a higher elevation than that for the 1936 flood, however, no measurements of this 1886 flood are available (Reference 39). The 1936 flood had a flood height of 51.2 feet on the Ipswich River at Boston Street. The Ipswich River had 1968 flood heights of 52.8 and 51.3 feet at Boston Street and the USGS gaging station at South Middleton, respectively (References 38 and 42). Gage records and published high-water marks are not available to document floods on the Goldthwaite Brook, Proctor Brook, North River, Strongwater Brook, and Tapley Brook. The hydrologic conditions that cause major flooding on the Ipswich River can result in flooding on these smaller streams.

During the February 6-7, 1978 blizzard in Rockport, damage occurred to coastal areas around Penzance Road, Old Harbor, and Pigeon Cove.

The area of the City of Salem that has been consistently the most heavily damaged is Salem Willows. Also subject to damage are areas within Salem Harbor, such as Derby Wharf, Palmerus Cove, and Forest River Park.

Continuing erosion in Salisbury associated with severe storms also acts to reduce beach and dune width to below protective and recreational use requirements (Reference 43).

River flooding in Saugus has not been a serious problem in the tidal area. The non-tidal flooding problems are primarily due to flooding' along the Saugus River and its tributaries.

During the February 6-7, 1978, blizzard in Swampscott, damage occurred to the Ocean Avenue Extension and its seawall and in the area adjacent to the beach club at Phillips Beach. These sites have greater exposure to northeast winds than other locations on the Swampscott coastline.

Investigations have revealed instances of severe flooding in Topsfield during the floods of March 1936, March 1968, and April 1987. Existing records from gaging stations in the Ipswich River basin show that the 1987 flood had the greatest peak discharge of the three events and was rated larger than the 1-percent-annual-chance recurrence interval flood. The March 1936 flood resulted from inadequate culverts and debris blockage at culvert entrances. Flooded areas included Salem Road at River Road; State Route 97 at the Ipswich River; Topsfield Fairgrounds; Grove Street; Prospect Street near the branch of Ipswich River (locally known as Cleveland Brook); Haverhill Road; and Pond Street. Wenham Swamp caused residential flooding. Damage was minimal, however, due to marginal development of the areas. Based on the Ipswich River gage No. 01102000 at Ipswich, Massachusetts, the peak discharges of the 1936, 1968, and 1987 events were 2,610; 2,680; and 3,550 cfs, respectively. The 1936 and 1968 events were estimated as having an approximate 2-percent-annual-chance frequency and, as mentioned in the previous paragraph, the 1987 event was estimated to have an average frequency greater than 1-percent-annual-chance. Precipitation measurements for the 1987 event were indicative of a rainfall that occurs much more frequently than 1-percent-annual-chance; the resulting high flow rate was due to antecedent conditions.

Major storms in Wenham occurred in 1936, 1938, 1949, 1955, 1968, 1976, 1979, and 1987. Little information is available for any of these early period storms. The April 1987 storm caused overtopping of an access road located just downstream of the outlet to Longham Reservoir. Flooding along the Miles River is minimal. Some low-lying houses adjacent to the river experienced flooded basements and ponded yards during the 1987 storm, but these occurrences can be attributed to the high local groundwater table, and the inability of the soil to percolate rainfall.

The only major flooding the Town of West Newbury has experienced was along the Merrimack and the Artichoke Rivers in 1936 and 1938. As a result of these floods, any building which occurred subsequent to 1936 along the Merrimack has been carried out with foundations set above the 1938 flood levels. No factors have aggravated the flood problems. The frequencies of the 1936 and 1938 floods cannot be estimated.

2.4 Flood Protection Measures

Flood protection measures for Essex County have been compiled and are summarized below:

There are four completed reservoirs located upstream of the Merrimack River in New Hampshire. These projects are operated in conjunction with each other to reduce flooding on the upstream tributaries and main stem of the Merrimack River. As a result of these projects, flood discharges along the Merrimack River in Essex County have been significantly reduced. In addition to the upstream reservoirs, the USACE has also completed five local protection projects, but they are not designed to affect flooding in Essex County. 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; and two dams that control Hopkinton Lake, the Everett Dam on the Piscataquog River, and the Hopkinton Dam on the Contoocook River. The USACE studies indicate that the five completed projects would have reduced the peak discharge of the 1936 flood on the Merrimack River at Andover by approximately 35 percent. No flood control reservoirs have been built in the Massachusetts portion of the Merrimack River basin, and none are contemplated. In part, this is because of the relatively flat topography, which does not permit storage of large volumes of floodwaters behind a single large dam without flooding adjacent developed areas. There are also 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. Dams located at Tuxbury Pond, Lake Gardner, and immediately downstream of Pond Street on the Powwow River do not provide flood protection measures.

The State of Massachusetts provides concrete seawalls and stone revetments to protect coastal highways in Essex County. Other protective structures were generally constructed and are maintained by the communities and private property owners to satisfy their individual requirements and financial capabilities. These structures include such backshore protection as timber and steel sheet piles, bulkheads, stone revetments, concrete seawalls, and pre-cast concrete units (Reference 43).

The principal protection along the Essex County coastline consists of a system of concrete and stone seawalls, approximately half of which are maintained by the communities and the remaining half by private owners.

At Kettle Cove in Manchester, each house has its own seawall, some of which were rebuilt after the February 1978 storm.

There is a partially submerged breakwater approximately 2 miles offshore of Rockport which affords some protection to the general area of Sandy Bay. At Rockport Harbor, there is a shore attached breakwater.

Regulation of the outlet structures of the major ponds in the Town of Boxford provide a limited means of controlling flood levels both upstream and downstream of the ponds. No other structural control measures exist on the study streams in the town. The Town of Boxford has adopted zoning laws which define Conservancy Districts that coincide with the extent of wetlands along river channels in the town. The zoning bylaws and the zoning map for the Town of Boxford defines the extent of the Conservancy Districts (Reference 44). The zoning map indicates the locations and elevations of the Conservancy Districts, and the bylaws extend the coverage to adjacent wetland areas as defined by Massachusetts wetland statutes. Furthermore, the zoning bylaws specify that residential lots include at least one acre of land not subject to the 1-percent-annual-chance flood as defined by the FIS or as determined by engineering methods specified by Massachusetts wetland regulations.

There are no flood control structures in Danvers. The swampy nature of the Ipswich River Basin and of some of the smaller streams of Danvers is a natural form of flood protection, since swamps store water and reduce peak flood discharges. The Town of Danvers has floodplain zoning regulations as a protection against flood hazards. The regulations are intended to restrict construction within the floodplain districts thereby minimizing flood damage. In an effort to minimize flooding problems, several stream improvement programs have been carried out in Danvers. Channelization and culvert design projects have been completed on Beaver Brook, Crane River, and Crane Brook (References 45, 46, 47, 48, and 49).

No structural flood protection measures exist within the Town of Hamilton to alleviate flooding within the community. The town has zoned a Conservancy District that is intended, among other purposes, to "protect the public health and safety, persons and property against hazards of flood water inundation; for the protection of the community against the costs which may be incurred when unsuitable development occurs in swamps, marshes, along watercourses, or in areas subject to floods;" (Reference 50). Parts of both the Ipswich and Miles Rivers are located within the Conservancy District.

There are several federal and non-federal water resource developments which significantly affect flood flows in the Merrimack River. These projects consist of reservoirs, dams, lakes, channel improvements, and other flood-retarding structures and are described in greater detail below. As a result of these projects, flood discharges along the river have been significantly reduced.

The Soil Conservation Service (SCS) constructed a number of small flood-retarding structures on three tributaries to the Merrimack River. These are located in the Assabet, Baker, Concord, Souhegan, and Sudbury watersheds and designed primarily for flood protection to specific downstream damage areas. None of these projects affect flooding conditions in the Essex County communities.

Five USACE flood control dams are located in the Merrimack River Basin. The Blackwater Dam, completed in 1941 at Webster, New Hampshire, has a drainage area of 28 square miles and flood control storage of 46,000 acre-feet. The Edward MacDowell Dam, completed in 1950 at Peterborough, New Hampshire has a drainage area of 44 square miles and flood control storage of 12,800 acre-feet. The Franklin Falls Dam, completed in 1943 at Franklin, New Hampshire, has a drainage area of 1,000 square miles and flood control storage of 150,600 acre-feet. Hopkinton Lake Dam, completed in 1962 at Hopkinton, New Hampshire, has a drainage area of 382 square miles and a flood control storage of 70,100 acre-feet. Everett Lake Dam, completed in 1961 at Weare, New Hampshire, has a drainage area of 64 square miles and flood control storage of 85,500 acre-feet. Hopkinton Lake and Everett Lake Dam projects are considered to operate as a single unit since they are connected by a canal. By the operation of these five projects, peak flows at the Lowell gage would be reduced from 173,000 cfs to 112,000 cfs during a recurrence of the record 1936 flood. Tidal flood barriers have not been constructed and are not planned for this area; however, the USACE has constructed and maintains two jetties, 4,118 and 2,445 feet long respectively, at the confluence of the Merrimack River to maintain a good navigational opening and to reduce the erosion of Plum Island and Salisbury Beach.

In addition, several reservoirs exist in Haverhill that do not significantly contribute to flood control, including Crystal Lake Reservoir and Millvale Reservoir. Prior to the flood of September 1938, the Emergency Relief Administration completed construction of concrete floodwalls and a concrete pressure conduit along the north bank of the Merrimack River and the Little River in the vicinity of the center of the City of Haverhill. The concrete floodwalls are equipped with two foot flashboards with a crest elevation of 25.2 feet. There are no new or planned flood control measures which would significantly affect flood conditions in the City of Haverhill.

The Essex Company Dam, also known as Great Stone Dam, is located on the Merrimack River in Lawrence (Reference 51), It was built to utilize the water power of Bodwells Falls. There are canals on both the north and south sides of the river, Under normal conditions, the canals would never be allowed to exceed an elevation of 41.77 feet due to control gates operated by the Essex Company. The dam does not provide flood protection other than to control water levels in the canals. The Malden Mills Dam is located at the outlet of Stevens Pond on the Spicket River. This dam provides flood control during periods of large flows in the Spicket River.

No tidal or riverine flood control structures have been built that would significantly affect flood conditions in Ipswich. The Willowdale dam is located approximately 4.6 miles upstream of State Route 133 on the Ipswich River. Also, there are several other small dams on the Ipswich and Miles Rivers within the town. These dams are used for water power, and none affect flood flows.

The Town of Lynnfield has a zoning ordinance which establishes a Floodplain District. The purpose of this bylaw is to ensure that lands in the town subjected to seasonal or periodic flooding shall not be used for residences or other purposes which would endanger the health, safety, or welfare of its citizens. Currently, there are no flood protection structures in Lynnfield.

The City of Methuen has dredged Searles Pond and reconstructed a breached spillway at its lower end. The City also has replaced culverts at East Street in this area. All of this work is designed to reduce the effects of flooding along Bloody Brook, which flows from Searles Pond south to the Methuen-Lawrence corporate limits. The dams in Methuen are used for industrial purposes and have no effect on flood control.

There are several dams located on the Parker River within Newbury. These dams were not built for flood control, and they have no affect on flows from the river. Present and future demands associated with the seasonal tourist industry will further intensify the pressure for development of flood-prone coastal lands. However, the adoption of local and state development regulations concerning floodplain management will help alleviate storm-related losses (Reference 52).

No new or planned flood control measures have been reported which would significantly affect flood conditions in the Town of North Andover. There are five dams along the Shawsheen River located outside of the town which have little effect on flood flows in North Andover (Reference 36). Sutton Pond Dam, Osgood Pond Dam, and two spillways are located on Cochichewick Brook within the Town of North Andover. These structures have no effect on flood control. There are no structural flood control measures for either Boston Brook or Mosquito Brook. North Andover has zoned a floodplain district which includes all special flood hazard areas designated as Zone A or Zone AE. The Watershed Protection District is intended to protect Lake Cochichewick, the town's sole drinking water supply, and has no impact on this flood study. Both the Floodplain District and the Watershed Protection District are overlay districts.

There are no flood-control structures in Peabody. The Spring Pond reservoirs on the upstream end of Tapley Brook are operated for water supply, and not flood-control purposes. The swampy nature of the Ipswich River Basin and some of the smaller streams of Peabody is a natural form of flood protection, since swamps store water and reduce peak flood discharges. The City of Peabody has floodplain zoning laws as a protection against flood hazards. Floodplain districts are designated in the watersheds of the following streams: the Ipswich River, the Norris Brook, the North River, the Proctor Brook, the Goldthwaite Brook, the Tapley Brook, and the Strongwater Brook. The boundaries of the districts are described by elevation or by horizontal distance from the waterway. The construction of new structures, the dumping of trash, the alteration of topography that may increase flood hazards, and the storage of toxic or floatable materials is prohibited in these floodplain districts (Reference 53).

No structural flood protection measures exist within the town of Topsfield. However, as a result of the severe flooding in 1936, most of the culverts were modified or enlarged to handle increased discharge. As a result, flooding decreased significantly during the 1968 and 1987 flood in comparison with the 1936 flood. The Town of Topsfield has zoning measures to control development in wetland areas. These measures regulate development that would cause excessive increases in storm runoff, and impose strict control on public buildings and proposed private development (Reference 54).

There are no structural flood control measures currently located within the town of Wenham. However, as discussed in Section 2.2, the area along the Miles River is zoned as a Floodplain District by the Town of Wenham. Zoning regulations in this Floodplain District, with the exception of the area upstream of Longham Reservoir Dam, contain provisions that are more restrictive concerning floodplain development than those required by the NFIP. Longham Reservoir Dam does not provide any flood control for the community; it is utilized as a water supply source only.

There are no flood protection works that have been constructed or that are planned which would significantly affect flood conditions in the Town of Georgetown, Middleton, Saugus, Rowley and Groveland.

3.0 ENGINEERING METHODS

For the flooding sources studied by detailed methods in the community, standard hydrologic and hydraulic study methods were used to determine the flood-hazard data required for this study. Flood events of a magnitude that is expected to be equaled or exceeded once on the average during any 10-, 2-, 1-, or 0.2-percent-annual-chance (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 2-, 1-, and 0.2-percent-annual-chance 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 that equals or exceeds the 1-percent-annual-chance flood in any 50-year period is approximately 40 percent (4 in 10); 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 community at the time of completion of this study. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

Hydrologic analyses were carried out to establish peak discharge-frequency relationships for each flooding source studied by detailed methods affecting the county.

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

Pre-countywide Analyses

The 10-, 2-, l-, and 0.2-percent-annual-chance peak discharges for the Porter River Frost Fish Brook, the Crane River, Crane Brook, and Beaver Brook were generated by applying regional equations developed by the USGS (Reference 55). These regional equations relate flows of various return periods to drainage area and main channel slope. The equations were derived by applying multiple regression techniques to the flow data and basin characteristics of 113 gaging stations located in Massachusetts; and in Vermont, New Hampshire, and Rhode Island near the Massachusetts border. The 10-, 2-, l-, and 0.2-percent-annual- chance peak flows at several stations on these 3 streams were calculated from the regional equations. The regional equation discharges were adjusted to account for impervious land surface area resulting from urbanization. This urbanization adjustment was based on runoff characteristics developed by the Soil Conservation Service (SCS) (Reference 56).

The natural peak discharge frequencies used for the Merrimack River in Amesbury, Andover, Newburyport, Salisbury and West Newbury are from a report entitled Water Resources investigation, Merrimack River Basin, completed by the USACE, New England Division in August 1972 (Reference 57). This report presented a discharge-frequency relationship for the Merrimack River at Lowell, Massachusetts, that was developed using the log-Pearson Type III method in a statistical analysis of the annual peak discharge data recorded at the Lowell USGS gage (Reference 58). The period of record for this gage extends from 1923 to the present. The probable river flows computed for Lowell was adjusted to conditions in the region by the addition of runoff from the intervening drainage area. These flows were then adjusted to reflect reductions caused by the operation of the upstream flood control reservoirs. These flows were adjusted by computing the storage of the reservoirs and routing the inflow hydrograph by the Straddle-Stagger method to the Lowell gage (Reference 59). The frequency of the routed discharge was developed by the log-Pearson Type III method (Reference 58).

There are no discharge records for the Powwow River. Originally, peak discharge frequencies for this river were derived using procedures presented in the report entitled, Estimating the Magnitude and Frequency of Floods on Natural Flow Streams in Massachusetts (Reference 55). The resulting flow values were also compared with the statistically analyzed gaged stream records, mentioned above, in the region and were found to be in general agreement.

The hydrologic analyses for Lake Attitash for the revised August 3, 1992 City of Amesbury FIS were taken from the Flood Insurance Study for the Town of Merrimac (Reference 18). The hydrologic analyses for the Powwow River for the revised August 3, 1992 City of Amesbury FIS were taken from the FIS for the Town of South Hampton, New Hampshire (Reference 60). The drainage areas for the Powwow River were taken from a USGS report on hydrologic characteristics of streams in the Merrimack River Basin (Reference 59). The 1-percent-annual-chance discharges for the Powwow River were computed using regression equations published in a 1983 report entitled Estimating Peak Discharges of Small, Rural Streams in Massachusetts (Reference 61). These equations supersede those previously published in the 1977 Amesbury report.

Several steps were taken to compute the discharges for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods for the Shawsheen River in Andover. First, statistical analysis, using a log-Pearson Type III distribution was performed at the USGS gaging station located on the Shawsheen River near Wilmington (gage No. 01100600). Since the Wilmington gage has a limited period of record of only 19 years, comparative analysis was also performed on longer term records in the region. The Ipswich River at South Middleton (gage No. 01101500), having a period of record of 45 years, was selected as being hydrologically similar to the Shawsheen River. A two station statistical comparison was then made between the Wilmington gage and the Ipswich River gage. The developed discharges compared quite closely with those used in the pre-countywide Andover FIS and, therefore, the existing analysis of the Ipswich River gage was used (Reference 2).

Peak discharges for the Shawsheen River were calculated by multiplying values adopted for the Town of Andover by the ratio of the drainage areas to the 0.75 exponential power (Reference 2).

The Fish Brook discharges were developed by statistical analysis and by empirical regression equations developed for Massachusetts by the USGS (Reference 55). As there are no stream flows gaging stations on Fish Brook, four representative gaging stations throughout the region were used: Boulder Brook at East Bolton; Nashoba Brook at Acton; the Aberjona River at Winchester; and Stony Brook at Temple, New Hampshire. Statistical analyses were performed using a log-Pearson Type III distribution. Discharge frequencies were then transferred from each gage to Fish Brook by ratio of respective drainage areas to the 0.7 exponential power. Additionally, discharge frequencies were derived from the reference USGS empirical regression equations. These equations were applied using physical characteristics of the Fish Brook watershed. It was determined that the discharge frequencies developed by the two methods were comparable and the regression equation results were adopted. Hussey Brook discharge frequency information was developed using the same methodology as

Fish Brook. It was determined that discharge frequencies developed by the two methods were comparable and the regression equation results were adopted. Discharge frequency for Hussey Brook Tributary was considered proportional to the adopted discharge frequencies for Hussey Brook by ratio of respective drainage areas.

Since the streams in Beverly are ungaged, the 10-, 2-, and 1-percent-annual-chance discharges were computed based on the Massachusetts flood magnitude and frequency formulas developed by the USGS for Centerville Creek, Chubbs Brook, and North Beverly Drainage Ditch (Reference 55). The study contractor performed a separate evaluation of these formulas and found them to be applicable to non-urban areas in the vicinity of Beverly. The USGS formulas predict discharges based on the parameters of watershed drainage area and main channel slope. The 0.2-percent-annual-chance discharge was determined by extrapolation. To account for the impact of urbanization on the North Beverly drainage area, adjustments were required in computing discharges. Existing methodology was modified to be compatible with Beverly's meteorological and hydrologic parameters (Reference 61).

The streams and rivers in Gloucester are ungaged; therefore, the l0-, 2-, l-, and 0.2-percent-annual-chance discharges for the Mill River were computed based on the Massachusetts flood magnitude and frequency formulas developed by the USGS (Reference 55). Stone & Webster performed a separate evaluation of these formulas and found them to be applicable to the Gloucester region. The USGS formulas predict discharges based on the parameters of watershed drainage area and main channel slope.

Since the Miles River in Ipswich is ungaged, the l0-, 2-, l-, and 0.2-percent-annual-chance discharges were computed based on the Massachusetts flood magnitude and frequency formulas developed by the USGS (Reference 55). The study contractor performed a separate evaluation of these formulas and found them to be applicable to the Ipswich region. The USGS formulas predict discharges based on the parameters of watershed drainage area and main channel slope.

Since the streams in Rowley are ungaged, the l0-, 2-, l-, and 0.2-percent-annual-chance discharges for the Mill River were computed based on the Massachusetts flood magnitude and frequency formulas developed by the USGS (Reference 55). The formulas predict discharges based on the parameters of watershed drainage area and main channel slope. The study contractor performed a separate evaluation of these formulas and found them to be applicable to the Rowley region.

A multiple regression analysis, developed by Johnson and Tasker (Reference 62), was employed to find runoff discharges in Georgetown. Standard USGS topographic quadrangle maps (Reference 63) were used to determine watershed areas and local topography. An annual precipitation value, representative for the region, of 3.67 feet per year was obtained from the U.S. Weather Bureau and used throughout southeastern Massachusetts (Reference 64) by determining values for slope and area and using them in conjunction with the precipitation value in the Johnson-Tasker formulas, values for runoff from 10-, 2-, l-, and 0.2 percent-annual-chance predicted. Exponents for the 0.2-percent-annual-chance storm frequency equation, though not given in the Johnson-Tasker Report, were determined by extrapolating the given values for the l0-, 2-, and 1-percent-annual-chance. Wherever possible, stream gage records were compared to these figures contributing flows from neighboring towns were obtained from other studies when available, or by isolating the associated watershed and applying the Johnson-Tasker regression analysis where no other study has been conducted. After comparison of predicted discharges with past floods, it was found that the Johnson-Tasker method breaks down in regions of flat slope or high storage. To correct these discrepancies, areas of swamp, bog, open water, and urban development were computed and assigned weighting values to account for storage and rapid urban runoff. The adjusted discharge figures more closely reflect the true nature of the basins involved. This method was used for the Town of Groveland as well.

For the Ipswich River in Gloucester, peak discharges for floods of the selected recurrence intervals were determined using a standard log-Pearson Type III analysis (Reference 65). Flow records (1931 through 1984) on the Ipswich River from the USGS gage, Ipswich River near Ipswich, Massachusetts (No. 01102000) were analyzed, and a discharge-frequency curve was developed. These discharges were then transposed using drainage area ratios and regional exponents for eastern Massachusetts. Peak discharges on the Miles River for floods of the selected recurrence intervals were estimated using Massachusetts regional equations developed by the USGS for small, rural watersheds (References 43 and 55).

Peak discharge-frequency relationships for the Ipswich River in Lynnfield and North Andover were taken from the pre-countywide North Reading FIS Middlesex County, Massachusetts (Reference 66). Frequency data were based on statistical analyses of stage-discharge records. Because no hydrologically similar gaged streams are in the area, flood flows for all other streams studied by detailed methods in Lynnfield, including Saugus River, Beaverdam and Bates Brooks, and Pillings Pond, were developed using the SCS method for estimating volume and rate of runoff in small watersheds (References 67, 68, and 69 in Lynnfield).

In Boxford, Middleton, Topsfield and Wenham, data from two gaging stations on the Ipswich River, USGS gages No. 01101500 at South Middleton, Massachusetts and No. 01102000 near Ipswich Massachusetts were used to define frequency-discharge relationships on the Ipswich River. The discharges were determined by drainage weighted correlations.

Flow frequencies for the Ipswich River in Ipswich were developed by the USGS from data at the gage located at the Willowdale Dam. These frequencies were based on a statistical analysis of the systematic discharge record of 46 years and a historic record of 92 years. The standard log-Pearson Type III method outlined by Water Resources Council Bulletin No. 17 was followed in the analysis (Reference 58). Once the 10-, 2-, l-, and 0.2-percent-annual-chance discharges were obtained at the gage, the flows downstream of the gage were adjusted by using a formula that relates the flows between two basins as a function of the drainage areas (Reference 70). Since the Miles River in Ipswich is ungaged, the l0-, 2-, l-, and 0.2-percent-annaul-chance discharges were computed based on the Massachusetts flood magnitude and frequency formulas developed by the USGS (Reference 55). The study contractor performed a separate evaluation of these formulas and found them to be applicable to the Ipswich region. The USGS formulas predict discharges based on the parameters of watershed drainage area and main channel slope.

Peak discharge-frequency relationships for the Little River and Riverside Airport Brook were derived using procedures described by the USGS in Estimating the Magnitude and Frequency of Floods for Natural-Flow Streams (Reference 55). The 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 using statistically analyzed data from nearby stream gages with similar watershed characteristics 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. The derivation of peak discharge-frequency relationships for Creek Brook and Millvale Reservoir Brook used the previously referenced USGS method in conjunction with a numerical integration reservoir routing of triangular inflow hydrographs (References 71 and 72). The routing process was incorporated to take into account the effects of storage in Crystal Lake and Millvale Reservoir upstream of Creek Brook and Millvale Reservoir Brook, respectively.

For the Spicket River, flows of selected recurrence intervals were developed utilizing a drainage area-peak discharge relationship in conjunction with corresponding peak discharges from the pre-countywide FIS for the Town of Methuen (Reference 73). The discharges were further refined by applying an adjustment for impervious land area in consideration of extensive urbanization over the lower reaches of the stream (Reference 74).

The Saugus River watershed is a complex hydrologic system. It contains three major storage areas: Lake Quannapowitt, a large swampy area in Reading; the swamp by the Wakefield Industrial Park; and two major tributary streams (the Reading Drainage Canal and Beaverdam Brook).

Because the culvert at Chestnut Street in Lynnfield acts as a control structure on Beaverdam Brook during periods of high flow, flood flows are reduced by routing through the swampy area upstream of Chestnut Street (Reference 75).

Runoff and flows tributary to Lake Quannapowitt were calculated by methods developed by the SCS and then routed through the lake (Reference 75). Because of the lake's storage capacity, flood flows could be significantly reduced. The outflow hydrograph for Lake Quannapowitt developed for the l0-, 2-, 1- and 0.2-percent-annual-chance recurrence intervals were hydrologically combined with flood flows developed for the Reading Drainage Canal. These flows were routed and again hydrologically combined with flows developed for Beaverdam Brook and the Pillings Pond outflow. Flows through the swamp by the Wakefield Industrial Park were then reduced (Reference 75) to take into account the effect of storage provided by the swamp and to obtain outflows over the Saugus River Dam (City of Lynn Diversion Works). Flows over the dam were then combined with flows developed from the incremental drainage areas below the dam to obtain flood flows on the Saugus River between the Town of Lynnfield's corporate limits and the Saugus River Dam.

The Pillings Pond basin complex is able to significantly reduce flood flows leaving the pond. Flood flows were calculated for Bates Brook from methods developed by the SCS (References 67, 68, and 69). The box culvert under the Boston and Maine Railroad embankment, which Bates Brook crosses, acts as a control structure during flood flows with the railroad embankment acting as a dike. This causes the swampy area on the upstream side of the railroad embankment to pond and store excess incoming flows not able to immediately pass through the box culvert. Flows were routed through this storage area with the reduced flows allowed to enter Pillings Pond. These flows were then routed through the pond in combination with runoff tributary to Pillings Pond, to arrive at expected flood elevations for the pond.

There are no discharge records available for Tributary to Neal Pond. Peak discharge-frequency relationships were derived using procedures described in the USGS publication, Estimating the Magnitude and Frequency of Floods on Natural Flow Streams in Massachusetts (Reference 55). The technique was developed using multiple-regression analyses to estimate flood peaks on ungaged, natural-flow streams in Massachusetts by relating the peak discharge to basin and climatic parameters. The resulting peak discharges were verified with statistically analyzed data from nearby stream gages with similar watershed characteristics.

There are no discharge records for Bare Meadow Brook, Bartlett Brook, Harris Brook, Hawkes Brook, or Peat Meadow Brook. Peak discharge-frequencies for Bare Meadow Brook, Bartlett Brook, Harris Brook, and Peat Meadow Brook were derived by using procedures developed by the USGS (Reference 55). Resulting flows were also compared with statistically-analyzed stream records from the USGS gage at Lowell on the Merrimack River with 55 years of record (Reference

75). They were found to be in general agreement. Discharge-frequencies for Hawkes Brook, a tributary of Bare Meadow Brook, were developed by multiplying the adopted discharges for Bare Meadow Brook by a factor equal to the ratio of the drainage areas raised exponentially to the 0.7 power.

For Emerson Brook and Boston Brook, peak discharges were obtained using the regional equation for Massachusetts developed by the USGS (Reference 55). The regional equation relates stream flow to the parameters of drainage area and main channel slope. Peak discharges for Tributary A to the Ipswich River were calculated by routing peak flows through Middleton Pond using the reservoir routing and hydrograph methods developed by the SCS (Reference 56). The storage effects of Middleton Pond are thus accounted for in the flow values of Tributary A.

Flow frequency for the Parker River in Newbury was based on a statistical analysis of USGS gage data. These data were analyzed in accordance with criteria outlined by the Water Resources Council (Reference 58). Frequency discharge data were based on a USGS computer model. The model was run on November 20, 1978, using a systematic record of 32 years and a generalized skew coefficient. The study contractor reviewed the input and assumption of the analysis and used it for this study. The discharges are based on Water Resources Council adjusted values. For the ungaged portion of the Parker River, short distances upstream and downstream of the gage were adjusted by means of proportional drainage basins as outlined by Chow (Reference 70).

Peak discharges for Parker River in Boxford were estimated using Massachusetts regional equations developed by the USGS for small rural watersheds (References 58 and 60).

Discharges for the Mill River were obtained from the pre-countywide FIS for the Town of Rowley (Reference 76).

Peak discharge-frequency relationships for Cochichewick Brook were determined using a method developed by the USGS (Reference 55). The method 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 with statistically analyzed data from nearby stream gages having similar watershed characteristics by using a multiplication factor. The reservoir routing of a triangular inflow hydrograph was used to evaluate the effects of storage in Lake Cochichewick (References 71 and 72). The results of the analysis indicated no outflow due to regulation and available storage. The peak discharges for Boston and Mosquito Brooks for the l0-, 2-, l-, and 0.2-percent-annual-chance floods were estimated using Massachusetts regional equations developed by the USGS for small rural watersheds (References 60 and 77).

The 10-, 2-, l-, and 0.2-percent-annual-chance peak flows for Proctor Brook, the Goldthwaite Brook, North River, and Strongwater Brook were generated by applying regional equations developed by the USGS (Reference 55). These regional equations relate flows of various turn periods to drainage area and main channel slope. The equations were derived by applying multiple regression techniques to the flow data and basin characteristics of 113 gaging stations located both in Massachusetts and in Vermont, New Hampshire and Rhode Island near the Massachusetts border. The 10-, 2-, l-, and 0.2-percent-annual-chance peak flows at several stations on these three streams were calculated from the regional equations. The regional equation flows were adjusted to account for impervious land surface area resulting from urbanization.

In determining peak discharges on Tapley Brook, hydrologic reservoir routings were performed on Browns Pond and on the upper and lower parts of Spring Pond (Reference 72). Browns Pond drains into Tapley Brook downstream of Spring Pond. The routing of Browns Pond revealed that peak flows were in significant to flooding on Tapley Brook. The l0-, 2-, l-, and 0.2-percent-annual-chance peak discharges used for Tapley Brook were determined by adding the peak discharges that resulted from the Spring Pond routing to peak discharges determined by applying the regional equation to the area draining directly in to Tapley Brook downstream of Spring Pond (Reference 55). It was determined that these two independent sets of discharges would reach the mouth of Tapley Brook at approximately the same time.

To account for the impact of urbanization on the drainage basins of the area, adjustments were required in computing discharges in Saugus for the Bennett's Pond Brook, Fiske Brook, Saugus River, and Shute Brook. Existing methodology in USGS Water Resources Investigation 23-74 was modified to be compatible with meteorological and hydrologic parameters for Saugus (Reference 61). This methodology was implemented for the flooding sources in Saugus. Above the most downstream point, of the area along which the Saugus River forms the border between Saugus and Wakefield, the flows used in this study were taken from the FIS for the Town of Wakefield (Reference 78). In the Wakefield study, determination of flows on the Saugus River was performed using a different method than that employed for this study. When an attempt was made to match flows determined at the Saugus town boundary using both methods, it was found that the methods produced flows which were not in agreement at this common point. Return period flows above the Hamilton Street Bridge in Saugus were determined using the Massachusetts flood magnitude and frequency formulas (Reference 55). The flows above the town boundary were obtained from the Wakefield study (Reference 78). Between the Hamilton Street Bridge and the Saugus town boundary, return period flows were determined at the U.S. Route 1 culvert. These flows were computed using a direct proportion, based on drainage basin size, of the flows between Hamilton Street and the Saugus town boundary. The drainage basin, area above U.S. Route 1 is slightly less than halfway between the area at the Saugus town boundary and Hamilton Street.

Thus, each return period flow at the U.S. Route 1 culvert is slightly less than halfway between the flow at the town boundary and the flow at Hamilton Street. This method of interpolation provided a reasonable means of tying together flows computed using the two different methods.

In the original 1979 study for Topsfield, the peak discharge-frequency relationships for Howlett Brook and Mile Brook depended on the hydraulics of the immediate and downstream areas. For this reason, the peak discharge frequency relationship was determined using the USACE HEC-2 step-backwater computer program (References 79 and 80). The computer program uses the geometries and relative elevations of significant features of the waterway to predict the water-surface elevation (as well as other properties). The total flow for the Howlett-Pye-Mile Basin, as determined by the regression equations at each flood frequency, was divided between Howlett Brook and Mill Brook at various ratios and used in the computer program to predict the water-surface elevations at the origin of the two streams. The ratio of the flow split at which the elevations matched was then considered to be the appropriate flow assignment for the two brooks at that flood frequency (Reference 70).

In the pre-countywide June 1994 Topsfield revision, the peak discharge-frequency relationships for Howlett, Pye, and Mile Brooks were calculated using the Massachusetts regional regression equations, assuming no diversion from Howlett Brook to Mile Brook (Reference 77). The quantity of flow diverted from Howlett Brook to Mile Brook is controlled by the North Street culvert at Mile Brook. The capacity of the North Street culvert is affected by tailwater created by a dam located approximately 1,000 feet east of North Street. The diversion of flow from Howlett Brook to Mile Brook was assumed to occur with the pond at peak stage. The pond's peak stages for the various frequency floods were computed using the procedures outlined by the USACE (Reference 81). The flow rates diverted to Mile Brook for the various frequency floods were determined by trial and error.

In the June 17, 1991 Topsfield revision, peak discharge-frequency relationships for the 10-, 2-, and 1-percent-annual-chance floods for Pye Brook, Cleveland Brook, School Brook, and Fish Brook were developed using Massachusetts regional regression equations developed by the USGS for small rural streams. In this revision, for Cleveland Brook, School Brook, Fish Brook, and Unnamed Tributary to Fish Brook, peak discharge-frequency relationships of the selected recurrence intervals, excluding the 0.2-percent-annual-chance flood, were developed using Massachusetts regional regression equations developed by the USGS for small rural streams (References 58 and 77). These relationships were then modified, where appropriate, to reflect urbanization within the basin. This adjustment was performed using regional regression equations developed by the Soil Conservation Commission (Reference 60). The 0.2-percent-annual-chance discharges were determined by a straight-line extrapolation of the 10-, 2-, and 1-percent-annual-chance discharges.

Peak discharges of the selected recurrence intervals for the Miles River including Longham Reservoir were estimated using Massachusetts regional equations developed by the USGS for small rural watersheds (References 59 and 77).

Because no hydrologically similar gaged streams are in the area, flood flows for North Tributary Brook and Beaver Brook were developed using the SCS method for estimating volume and rate of runoff in small watersheds. This method was developed using both land use and ground slope. Peak discharges for the approximate study streams were established using the above-mentioned SCS method (References 59, 82, and 83). Flood flows were routed (graphical method) through the upper Artichoke Reservoir to determine the peak discharges and maximum water-surface elevation to be expected (Reference 84).

Countywide Analyses

For the Shawsheen River revision, peak flow discharges were computed using the 1983 USGS Rural Regression equations for Massachusetts developed by Wandel and nationwide urban equations described in the USGS Water Supply Paper 2207. USGS operates a stream gage near Wilmington, MA (1100600) since 1964 and has experienced extensive urbanization since 1964 making the USGS urban equations the preferred method for the Shawsheen River area.

Peak discharges for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods of each flooding source studied in detail in Essex County are shown in Table 6.

TABLE 6 – SUMMARY OF DISCHARGES

		PEAK DISCHARGES (CUBIC FEET PER SECOND)			
FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
ARGILLA BROOK					
At Main Street	1.70	212	290	342	616
Approximately 2,100 feet upstream of Main Street	1.40	193	262	300	565
At Center Street	0.90	174	233	277	516
ARTICHOKE RIVER - RESERVOIR					
At upper Artichoke Reservoir Dam	5.60	80	180	240	290

<u>TABLE 6 – SUMMARY OF DISCHARGES</u>-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
BARE MEADOW BROOK					
At confluence with Merrimack River	7.70	350	580	710	1,090
At confluence with Hawkes Brook	2.70	180	320	400	620
450 feet downstream of Oak Street	1.10	110	190	230	370
Hills Pond	0.20	34	61	80	123
BARTLETT BROOK					
Approximately 3800 feet upstream of North Lowell Street	6.30	310	520	630	970
BATES BROOK					
Upstream of Confluence with Pillings Pond	1.10	50*	112*	120*	132*
Upstream of Private Driveway	0.70	125	230	275	345
BEAVER BROOK (TOWN OF DANVERS)					
At mouth in Danvers	2.20	170	270	320	470
At Maple Street	1.70	150	240	290	430
Approximately 790 feet downstream of Spring Street	1.30	140	220	260	390

*Decrease in Discharges Over Lager Drainage Area Due to Attenuation of Flow by Swamps

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
BEAVER BROOK (TOWN OF WEST NEWBURY)					
Middle Street in	1.58	65	125	150	170
Confluence with Beaver Brook Tributary	0.72	25	55	70	80
BEAVERDAM BROOK					
At Main Street	1.50	80	100	105	112
At Chesterbrook Street	1.20	80	100	105	112
BENNETT'S POND BROOK					
Confluence with the Saugus River	3.32	374	539	618	828
BOSTON BROOK					
At confluence with Ipswich River in Middleton	10.40	450	740	910	1,390
At Liberty River in Middleton	8.50	360	600	730	1,120
Downstream of Creighton Pond Tributary in Middleton	7.30	330	560	680	1,040
At downstream North Andover Corporate Limits	5.70	230	365	435	580
At confluence of unnamed Tributary downstream of Footpath in North Andover	4.90	205	330	395	530
At confluence of unnamed Tributary	4.20	185	300	355	490

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
BRANCH OF IPSWICH AND CLEVELAND BROOK					
At its confluence with School Brook	0.40	70	110	130	170
BULFORD BROOK					
East Main Street	0.49	5	7	9	14
Approximately 1,300 feet upstream of East main Street	0.35	4	6	8	13
Approximately 2400 feet of East Main Street	0.23	4	6	8	12
CENTERVILLE CREEK					
At its confluence with Massachusetts Bay	1.74	96	163	199	310
CHUBBS BROOK					
At its confluence with Chubb Creek	1.36	80	135	166	259
COCHICHEWICK BROOK					
At the confluence with Merrimack River	2.20	150	250	310	480

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
CRANE BROOK					
Approximately 80 feet Street Dam	2.90	190	300	350	500
Boston and Maine Railroad near Pine Street	2.60	170	260	310	450
At Collins Street	2.10	140	220	260	390
Approximately 1,320 feet downstream of Andover Street	1.60	110	180	210	310
At Andover Street	1.30	90	140	170	250
Boston and Maine Railroad near Andover Street	1.10	80	130	150	230
CRANE RIVER AND CRANE BROOK					
At mouth in Danvers	5.70	360	530	620	880
CREEK BROOK					
At confluence with Merrimack River	4.00	250	430	530	820
At Broadway Street	1.40	120	220	260	410
EMERSON BROOK					
At confluence with Ipswich River	5.80	230	390	470	720
FISH BROOK (TOWN OF ANDOVER)					
At confluence with Merrimack River	5.90	265	450	545	840

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
FISH BROOK					
At confluence with the Ipswich River	17.80	480	760	900	1,190
Approximately 160 feet downstream of I-95 crossing in Boxford	15.80	450	700	830	1,065
At Towne Road crossing in Boxford	9.60	300	510	600	790
FISKE BROOK					
At the confluence with Shute Brook	1.12	157	237	278	391
GOLDTHWAITE BROOK					
At confluence with Proctor Brook	4.93	350	530	630	910
Downstream of Allens Lane	4.53	310	490	580	840
Downstream of Boston and Maine Railroad crossing	3.90	260	410	480	710
1,750 feet downstream of Summit Street	2.56	190	300	350	520
180 feet upstream of Summit Street	2.17	150	230	270	400
Upstream of granite slab Bridge	1.93	130	200	240	350
Upstream of pond above Corvin Street	1.69	110	170	210	310
Downstream of First Avenue	1.34	73	110	140	200
HARRIS BROOK					
At its confluence with Spicket River	4.80	200	330	400	600

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
At Pelham Street	2.90	140	230	280	420
HAWKER BROOK					
At confluence with Bare Meadow Brook	4.20	210	360	440	690
3,750 feet upstream of Confluence with Bare Meadow Brook	3.90	160	280	340	520
HAWKER BROOK – cont'd					
At Washington Street	3.30	150	250	300	470
400 feet upstream of Maple Street	1.30	90	150	180	280
HOWETT BROOK AND PYE BROOK					
At the confluence Of the Ipswich River	8.70	275	450	535	730
HOWETT BROOK AND PYE BROOK – cont'd					
At the confluence with Unnamed Tributary Upstream of East Street	7.04	235	380	465	630
At divergence into Howlett and Mile Brooks	6.14	240	380	455	615
HUSSEY BROOK					
At confluence with Shawsheen River	2.10	130	225	280	435
HUSSEY BROOK TRIBUTARY					
At confluence with	0.80	50	90	110	170

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
Hussey Brook					
IPSWICH RIVER					
At Central Street in Ipswich	148.00	2,023	3,016	3,251	4,196
At corporate limits in Topsfield	120.90	1,880	2,700	3,070	3,980
At confluence with Mile Brook	109.30	1,755	2,520	2,860	3,725
IPSWICH RIVER – cont'd					
At confluence of Branch of Ipswich	92.60	1,360	2,080	2,440	3,430
At Middleton/Topsfield / Boxford Corporate Limits in Middleton	76.10	1,077	1,584	1,829	2,478
Downstream of Boston Brook in Middleton	71.90	1,023	1,506	1,741	2,366
Downstream of Tributary A to Ipswich River	53.80	790	1,173	1,362	1,872
Middleton/Danvers Corporate Limits south of State Route 114	50.90	750	1,120	1,300	1,790
Downstream of Norris Brook in Danvers	48.20	720	1,070	1,240	1,710
At Peabody/Danvers/ Middleton Corporate Limits	44.60	630	930	1,130	1,620
At Middleton/North Reading Corporate Limits	42.50	630	930	1,130	1,620
JACKMAN BROOK					
Georgetown/Newbury	1.38	24	39	47	72

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
Corporate Limits (Parish Road)					
Jackman Street	0.66	12	19	22	34
Approximately 1,200 feet upstream of Jackman Street	0.45	8	13	15	24
Jewett Street	0.24	4	7	8	13
JOHNSON CREEK					
Approximately 430 feet downstream of Haverhill/Groveland Corporate Limits	6.00	511	731	877	1,623
At Main Street	4.30	225	350	410	720
At Gravel Road Over Dam	3.00	200	320	385	650
At Center Street	2.90	190	308	362	603
Approximately 620 feet upstream of Center Street	2.20	164	270	315	525
At Salem Street	2.10	148	233	270	442
At Uptrack Road	1.70	110	170	200	310
At Washington Street	1.50	98	145	164	252
LITTLE RIVER					
At Winter Street	37.00	1,160	1,920	2,330	3,520
Upstream of I-95	27.70	980	1,640	1,990	3,030
Downstream of Haverhill/ Plaistow corporate limits	20.80	660	1,065	1,275	1,865
MERRIMACK RIVER					
At Salisbury/Amesbury /Newburyport	5010.00	61,000	92,000	115,000	172,100

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
Corporate Limits					
At Haverhill/Methuen /North Andover Corporate Limits	4980.00	61,100	92,100	115,100	172,100
At USGS gage No. 1005 in Lawrence	4672.00	58,000	90,000	111,000	156,000
At Andover/ Tewksbury Corporate Limits	4644.00	58,000	90,000	111,000	156,000
MILE BROOK					
At U.S Route 1	0.24	52	70	81	108
At dam approximately 1,000 feet downstream of North Street	0.16	42	60	71	88
MILES RIVER					
At its confluence with the Ipswich River	16.00	359	584	706	1,061
At downstream Corporate Limits in Hamilton	12.70	385	610	725	1,150
At downstream Corporate Limits in Wenham	8.60	298	472	563	900
At confluence of Unnamed Tributary above Wenham Lake	7.50	272	434	518	820
At Longham Reservoir Dam in Wenham	6.80	255	407	486	769
MILL RIVER(CITY OF GLOUCESTER)					
At tide gate under Washington Street	2.27	139	238	292	455
Adjacent to inter-Section of Poplar Street and York	1.59	107	182	224	351

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
Road					
MILL RIVER(TOWN OF ROWLEY)					
At U.S Route 1 In Rowley	13.60	415	685	831	1,261
MILLVALE RESERVOIR BROOK					
At confluence with Merrimack River	8.60	240	400	490	790
At Millvale Road	6.70	160	270	350	620
MOSQUITO BROOK					
At downstream limits of North Andover Corporate Limits	9.40	295	500	590	780
At confluence of Unnamed Tributary Downstream of Boxford Stream	7.50	275	435	520	700
At confluence of unnamed Tributary upstream of Boxford Street	5.10	210	340	405	540
At confluence of tributary from Stiles Pond	3.10	155	245	295	400
Approximately 1240 feet upstream of Foster Street	2.60	135	220	265	350
At confluence of unnamed Tributary downstream of Salem Street	1.40	95	150	180	245
At confluence of	0.80	60	100	120	165

TABLE 6 – SUMMARY OF DISCHARGES-continued

		PEAK DISCHARGES (CUBIC FEET PER SECOND)			
FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
unnamed Tributary downstream of Abandoned dam					
NORTH BEVERLY DRAINAGE DITCH					
Upstream of Boston and Maine Railroad embankment in	1.16	174	253	290	393
NORTH RIVER AND PROCTOR BROOK					
At Salem corporate Limits	9.96	640	990	1,140	1,620
Upstream of Strongwater Brook	8.86	580	880	1,030	1,470
Upstream of Goldthwaite Brook	3.52	240	360	420	610
Upstream of State Route 128	2.52	170	260	310	450
Upstream of Downing Road	2.10	150	240	280	420
150 feet upstream of Downing Road	1.48	140	220	260	380
Downstream of Albert Road	1.22	130	200	230	340
NORTH TRIBUTARY BROOK					
At Pikes Bridge Road	1.35	70	125	150	170
PARKERS RIVER(TOWN OF BOXFORD)					
At downstream Corporate Limits	3.60	170	270	325	460

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
Upstream of Willow Road crossing	2.70	140	225	271	460
PARKERS RIVER(TOWN OF BOXFORD) – cont'd					
Approximately 640 feet Downstream of Main Street Crossing	2.10	125	190	230	305
PARKER RIVER (TOWN OF GEORGETOWN)					
Georgetown/Groveland Corporate Limits	11.24	198	317	382	585
Thurlow Street	10.80	190	305	368	562
Railroad Track Bed	9.94	175	280	338	517
Mill Street	9.83	155	245	285	450
Railroad Track Bed	6.73	120	200	255	395
Pond Street	6.59	116	186	224	343
Railroad Track Bed	5.90	105	170	208	310
West Main Street	5.81	102	164	198	302
Bailey Lane	5.12	95	150	180	285
Approximately 2420 feet downstream of Bailey Lane	4.71	83	133	160	245
PARKER RIVER (TOWN OF NEWBURY)					
At central Street	24.20	393	605	714	1,019
Approximately 1,150 feet downstream of Larkin Street	21.60	359	552	652	930

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
PEAT MEADOW BROOK					
At confluence with Spicket River	2.00	100	160	200	310
At Interstate Highway 93	1.50	80	140	170	270
At Forest Street	0.20	20	30	40	70
PENN BROOK					
North Street	3.10	55	87	105	161
Summer Street	2.96	52	83	101	154
Road to High School	2.84	50	80	97	148
Penn Brook Avenue	2.74	47	76	92	141
East Main Street	2.20	43	70	85	130
Approximately 1650 feet upstream of East Main Street	2.04	41	66	80	122
Approximately 1680 feet downstream of East Street	1.87	37	57	71	109
East Street	1.64	33	50	62	95
State Highway 97	1.31	28	43	53	81
Railroad Track Bed	1.28	23	36	44	67
PORTER RIVER AND FROST FISH BROOK					
At mouth in Danvers	12.50	720	1,070	1,240	1,750
Upstream of Waters River	10.30	600	900	1,050	1,490
Upstream of Crane River	4.40	260	410	490	720
At U.S Route 128	3.50	220	350	420	620
At Conant Street	3.00	200	310	380	560

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
At Coolidge Road	2.50	170	270	330	490
POWWOW RIVER					
At Lake Gardner Dam	49.10	*	*	1,720	*
At Tuxbury Pond Dam	45.90	*	*	1,640	*
RIVERSIDE AIRPORT BROOK					
At confluence with Merrimack River	0.70	50	90	120	180
SAUGUS RIVER					
At Hamilton Street Bridge in Saugus	22.60	564	923	1,118	1,683
At the U.S Route 1 Culvert in Saugus	18.60	432	715	846	1,187
At a point approximately 1,250 feet downstream of the Water Street culvert	15.70	340	570	655	840
At the Water Street culvert in Saugus	12.10	230	380	435	595
Above Confluence with Unnamed Stream from Montrose Avenue (Wakefield)	11.30	115	185	215	340
At State Route 128 Upstream Crossing (Main Street, Lynnfield)	5.40	190	310	330	395
Above Confluence With Reading Drainage Canal in Lynnfield	1.80	35	50	57	65

* Data not computed

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
SCHOOL BROOK					
At its confluence with Cleveland Brook	0.40	70	110	130	170
SHAWSHEEN RIVER					
At confluence with Merrimack River in Lawrence	78.07	2,231	3,137	3,707	4,667
At Andover/Lawrance Corporate Limits	75.34	2,149	3,026	3,577	4,506
At U.S. Route 128	71.09	2,008	2,834	3,350	4,212
At Interstate Highway 93	60.84	1,854	2,618	3,093	3,898
SHUTE BROOK					
At the confluence with the Saugus River	3.22	470	664	757	1,000
SPICKET RIVER					
At confluence with Merrimack River in Lawrence	74.50	1,200	1,950	2,400	3,550
At Spruce Street in Lawrence	72.00	1,100	1,800	2,200	3,300
At Methuen Dam in Methuen	73.80	1,100	1,800	2,200	3,300
Below Harris Brook in Methuen	67.60	1,000	1,700	2,000	3,100
At the Massachusetts New Hampshire state line in Methuen	61.60	900	1,600	1,900	2,900

TABLE 6 – SUMMARY OF DISCHARGES-continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
STRONGWATER BROOK					
At confluence with Proctor Brook	1.08	77	120	140	210
TAPLEY BROOK					
At confluence with Goldthwaite Brook	1.34	81	135	165	250
TRIBUTARY A TO IPSWICH RIVER					
At confluence with Ipswich River	2.00	76	143	175	236
At downstream end of Middleton Pond	1.60	41	85	111	170
TRIBUTARY TO NEAL POND					
At Birch Meadow Road No.2	0.80	80	140	170	250
UNNAMED TRIBUTARY TO FISH BROOK					
At confluence with Fish Brook	0.33	*	*	70	*
At Boxfield Road in Topsfield	0.25	*	*	29	*
At corporate limits in Topsfield	0.16	*	*	59	*

* Data not computed

3.2 Hydraulic 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. Users should be aware that flood elevations shown on the FIRM represent rounded whole foot elevations and may not exactly reflect the elevations shown on the Flood Profiles or in the Floodway Data tables in the FIS report. Flood elevations shown on the FIRM are primarily intended for flood insurance rating purposes. For construction and/or floodplain management purposes, users are cautioned to use the flood elevation data presented in this FIS in conjunction with the data shown on the FIRM.

All bridges, dams and culverts were field surveyed to obtain elevation data and structural geometry. Cross section data for the below-water sections were obtained from field surveys and topographic maps compiled by photogrammetric methods. Cross sections were located at close intervals above and below bridges, culverts, and dams in order to compute the significant backwater effects of these structures. In addition, cross sections were taken between hydraulic controls whenever warranted by topographic changes.

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 was computed (Section 4.2), selected cross-section locations are also shown on the FIRM.

The hydraulic analyses for this study were based on unobstructed flow. The flood elevations shown on the Flood Profiles (Exhibit 1) are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

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

<u>Pre-countywide Analyses</u>

In Merrimac, USACE bridge plans and intermediate valley cross sections were used to supplement field survey data. Base mapping, at a scale of 1:4,800 (1"= 400') with a contour interval of 5 feet was used to develop overbank cross section data and additional valley cross sections as necessary (Reference 84). Below-water sections were also field surveyed at representative locations along Tributary to Neal Pond. Topographic maps at a scale of 1:4,800 with a contour interval of 5 feet were used to develop overbank cross section data and additional valley sections as necessary to satisfy hydraulic computation requirements (Reference 85).

In those areas where the analysis indicated super critical flow conditions, critical depth was assumed for the flood elevations because of the inherent instability of supercritical flow.

Water-surface elevations of floods of the selected recurrence intervals for the communities in Essex County were computed using the USACE HEC-2 step-backwater computer program (Reference 79). Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals.

The Powwow River starting water-surface elevation for the 10-percent-annual-chance flood elevation was taken from the annual tidal stage-elevation; the 2-percent-annual-chance was taken from the 5-year tidal stage-elevation; the 1-percent-annual-chance flood elevation was taken from the 10- tidal stage-elevation; and the 0.2-percent-annual-chance flood elevation was taken from the 25-year tidal stage elevation (Reference 59). For the pre-countywide revised portion of the Powwow River, the starting water-surface elevation for the 1-percent-annual-chance flood was taken from the FIS for the Town of South Hampton, New Hampshire (Reference 60).

Starting water-surface elevations for Unnamed Tributary to Fish Brook was determined using the discharge characteristics of the dam downstream of Lockwood Lane.

Starting water-surface elevations for Ipswich River, Bartlett Brook, Boston Brook, Emerson Brook, Howlett Brook, Pye Brook, Mile Brook, Branch of Ipswich, Bennett's Pond Brook, Cleveland Brook, School Brook, Shute Brook, Fish Brook, Parker River, North Beverly Drainage Ditch, Miles River, Tributary A to the Ipswich River, Fiske Brook, and Fish Brook were computed using the slope/area method.

Starting water-surface elevations for the Ipswich River in Ipswich and for Parker River in Newbury were determined using critical depth.

Starting water-surface elevations for the Little River, Creek Brook, Millvale Reservoir Brook, Shawsheen River, Spicket River, Cochichewick Brook, Bare Meadow Brook, and Riverside Airport Brook were determined to be the normal water-surface elevation of the Merrimack River at their respective confluences.

Starting water-surface elevations for Fish Brook were based on a stage discharge rating curve developed at the dam located at its mouth. Starting water-surface elevations on Hussey Brook were found to be coincident with developed flood profiles on the Shawsheen River. Starting water-surface elevations for Hussey Brook Tributary were found to be coincident with developed flood profiles on Hussey Brook.

Starting water-surface elevations for Chubbs Brook, Mill River, Saugus River, and Centerville Creek were based on the average spring high tide level.

Mean high tide in Beverly Harbor was used as the starting water-surface elevation for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods on the Porter River and Frost Fish Brook, and on the Crane River and Crane Brook. Hydraulic analysis of the Ipswich River was performed in conjunction with the Middleton, Massachusetts FIS, which involved a continuous backwater run from North Reading to Topsfield (Reference 85). Starting water-surface elevations on Beaver Brook were set equal to the flood elevations at Mill Pond, as determined in the Crane River and Crane Brook hydraulic analysis. The Waters River was analyzed using the tidal effects from Beverly Harbor (Reference 86).

Starting water-surface elevations for Parker River in Georgetown were taken from its confluence with the Merrimack River. The starting water-surface elevations for Penn Brook were taken at its confluence with Parker River. Bulford Brook starting elevations were taken at its confluence with Penn Brook. Starting elevations for Jackman Brook were obtained from field notes taken at the Georgetown corporate limits.

For Hawkes Brook, the starting water surface elevations were derived from its confluence elevations with Bare Meadow Brook (Reference 87). Starting water surface elevations for Harris Brook were derived from its confluence elevations with the Spicket River. For Peat Meadow Brook the starting water-surface elevations were determined by the slope-area method.

Starting water-surface elevations for Unnamed Tributary to Fish Brook was determined using the discharge characteristics of the dam downstream of Lockwood Lane.

Maximum elevations of the Upper Artichoke Reservoir were used in starting water-surface elevations for North Tributary Brook. Starting water-surface elevations for all flood flows on Beaver Brook were started at the normal water elevations.

Starting water-surface elevations for the Merrimack River in Merrimac were taken from the 10-year tide elevation in Newburyport.

In Gloucester, for Mill River, water-surface elevations of floods of the selected recurrence intervals were computed using the One Dimensional Storm Surge Model for Coastal Rivers for the tidal reaches and the USACE HEC-2 step-backwater computer program for the inland reaches (References 80 and 87).

In some locations, water levels shown on the maps were computed by correlating synthetically produced water levels with elevations obtained during historic floods (Reference 88).

The areas analyzed by approximate methods were delineated after consideration of the 1-percent-annual-chance flood elevations from the backwater analysis on the detailed study areas and the communities respective pre-countywide Flood Hazard Boundary Maps.

<u>Countywide Analyses</u>

For the Shawsheen River, the USACE HEC-RAS Version 3.1.3 was used to perform the hydraulic analysis. HEC GeoRAS Version 4.1 for ArcGIS 9.2 was used as a pre-processor for inputs to the hydraulic model and a post-processor for delineation of the floodplains. The Shawsheen River HEC-RAS model was calibrated to flood elevations recorded for the 2006 and 2007 flood events and used to compute the 10-percent, 2-percent, 1-percent and 0.2-percent-annual-chance water-surface elevations for the study reach as well as the floodway. The topographic information reflected in the hydraulic model is based on field survey and Light Detection and Ranging (LiDAR) data; in some cases, effective HEC-2 model input data was also used to obtain structure dimensions.

Roughness factors (Manning's "n" values) used in the hydraulic computations were determined from field observations, guided by U.S. Geological Water Supply Publications. Table 7, "Manning's "n" values" shows the channel and overbank "n" values for the streams studied by detailed methods:

TABLE 7 – MANNING'S "n" VALUES

Flooding Source	Channel "n"	Overbanks
Bare Meadow Brook	0.030-0.040	0.060-0.080
Bartlett Brook	0.030-0.040	0.030-0.080
Bates Brook (Lynnfield)	0.020-0.110	0.11
Beaver Brook (Danvers)	0.020-0.040	0.035-0.090
Beaverdam Brook (Lynnfield)	0.020-0.110	0.11
Bennett's Pond Brook	0.013-0.050	0.07
Boston Brook (Middleton)	0.013-0.045	0.05-0.10
Boston Brook (North Andover)	0.020-0.060	0.020-0.080
Branch of Ipswich (Topsfield)	0.010-0.060	0.11
Centerville Creek	0.020-0.050	0.040-0.070
Channel Bottoms/Overbanks (West Newbury)	0.013-0.060	0.01
Chubbs Brook	0.030-0.050	0.030-0.090
Cleveland Brook (Topsfield)	0.010-0.060	0.11
Cochichewick Brook	0.030-0.042	0.035-0.100
Crane Brook (Danvers)	0.020-0.040	0.035-0.090
Crane River (Danvers)	0.020-0.040	0.035-0.090
Creek Brook	0.015-0.065	0.017-0.125
Drainage Ditch (North Beverly)	0.014-0.050	0.05
Emerson Brook	0.015-0.06	0.07-0.08
Fish Brook	0.030-0.050	0.050-0.085
Fish Brook (Topsfield)	0.010-0.060	0.11
Fiske Brook	0.05	0.06
Frost Fish Brook (Danvers)	0.020-0.040	0.035-0.090
Georgetown Floodplains	0.030-0.060	0.050-0.100
Goldthwaite Brook (Peabody)	0.023-0.045	0.030-0.15
Groveland Floodplains	0.03-0.06	0.05-0.10
Hamilton Streams	0.020-0.050	0.020-0.070
Harris Brook	0.035	0.06
Riverside Airport Brook	0.035-0.040	0.100-0.060
Hawkes Brook	0.035-0.040	0.030-0.100
Howlett Brook (Topsfield)	0.010-0.100	0.050-0.100
Hussey Brook	0.030-0.055	0.080-0.100
Hussey Brook Tributary	0.030-0.055	0.080-0.100
Ipswich River (Danvers)	0.020-0.040	0.035-0.090
Ipswich River (Ipswich)	0.020-0.040	0.030-0.100
Ipswich River (Lynnfield)	0.020-0.110	0.11

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

Flooding Source	Channel "n"	Overbanks
Ipswich River (Middleton)	0.03-0.045	0.033-0.15
Ipswich River (Topsfield)	0.010-0.060	0.11
Ipswich River (Wenham)	0.010-0.060	0.11
Little River	0.029-0.065	0.030-0.125
Merrimack River (Andover)	0.035-0.020	0.065-0.075
Merrimack River (Haverhill)	0.025-0.035	0.065
Merrimack River (Lawrence)	0.028	0.07
Merrimack River (Merrimac)	0.030-0.040	0.060-0.100
Merrimack River (Methuen)	0.023	0.08
Merrimack River (Newburyport)	0.04	0.075
Merrimack River (North Andover)	0.022-0.032	0.060-0.080
Merrimack Tributary to Neal Pond	0.030-0.040	0.060-0.100
Mile Brook (Topsfield)	0.010-0.100	0.050-0.100
Miles River (Ipswich)	0.020-0.050	0.035-0.100
Miles River (Wenham)	0.020-0.050	0.020-0.070
Mill River (Gloucester)	0.013-0.040	0.1
Mill River (Newbury)	0.015-0.050	0.015-0.050
Mill River (Rowley)	0.015-0.060	0.015-0.090
Millvale Reservoir Brook	0.035-0.042	0.112-0.125
Mosquito Brook	0.020-0.070	0.020-0.070
North River (Peabody)	0.023-0.045	0.030-0.15
Parker River	0.020-0.040	0.1
Peat Meadow Brook	0.025-0.050	0.090-0.100
Porter River (Danvers)	0.020-0.040	0.035-0.090
Powwow River	0.035	0.06
Proctor Brook (Peabody)	0.023-0.045	0.030-0.15
Pye Brook (Topsfield)	0.010-0.100	0.050-0.100
Riverside Airport Brook	0.035-0.040	0.100-0.060
Saugus River	0.020-0.040	0.050-0.100
Saugus River (Lynnfield)	0.020-0.110	0.11
School Brook (Topsfield)	0.010-0.060	0.11
Shawsheen River	0.035-0.065	0.030-0.170
Shute River	0.012-0.050	0.035-0.100
Spicket River (Methuen)	0.035-0.050	0.060-0.080
Spicket River (Lawrence)	0.03	0.07

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

Flooding Source	Channel "n"	Overbanks
Strongwater Brook (Peabody)	0.023-0.045	0.030-0.15
Tapley Brook (Peabody)	0.023-0.045	0.030-0.15
Topsfield Streams	0.010-0.100	0.050-0.100
Tributary A to the Ipswich River	0.015-0.04	0.04-0.09
Unnamed Tributary to Fish Brook	0.010-0.100	0.050-0.100
Waters River (Peabody)	0.023-0.045	0.030-0.15
West Newbury Channels and overbanks	0.030-0.040	0.060-0.085

3.3 Coastal Analyses

In New England, the flooding of low-lying areas is caused primarily by storm surges generated by extra-tropical coastal storms called northeasters. Hurricanes also occasionally produce significant storm surges in New England, but they do not occur nearly as frequently as northeasters. Due to its geographic location, Essex County is susceptible to flooding from both hurricanes and northeasters.

A northeaster is typically a large counterclockwise wind circulation around a low pressure. The storm is often as much as 1,000 miles wide, and the storm speed is approximately 25 mph as it travels up the eastern coast of the United States. Sustained wind speeds of 10-40 mph are common, with short-term wind speeds of up to 70 mph. Such information is available on synoptic weather charts published by the National Weather Service.

Areas of coastline subject to significant wave attack are referred to as coastal high hazard zones. The USACE has established the 3-foot breaking wave as the criterion for identifying the limit of coastal high hazard zones. The 3-foot wave has been determined as the minimum size wave capable of causing major damage to conventional wood frame or brick veneer structures. Wave height analyses were performed in the coastal communities of Essex County to determine wave heights and corresponding wave crest elevations for the areas inundated by the tidal flooding, wave runup analyses were performed to determine the height and extent of runup beyond the limit of tidal inundation. The results of these analyses were combined into wave envelopes, which were constructed by extending the maximum wave runup elevation seaward to its intersection with the wave crest profile.

The methodology for analyzing wave heights and corresponding wave crest elevations was developed by NAS (References 89, 90, 91, and 92). The wave runup was determined using the methodology developed by Stone and Webster

Engineering Corporation for FEMA (Reference 91). The NAS methodology is based on three major concepts.

First, a storm surge on the open coast is accompanied by waves. The maximum height of these waves is related to the depth of water by the following equation:

$$H_b = 0.78d$$

where H_b is the crest to trough height of the maximum or breaking wave and d is the stillwater depth. The elevation of the crest of an unimpeded wave is determined using the equation:

$$Z_w = S_* + 0.7H_* = S_* + 0.55d$$

where Z_w is the wave crest elevation, S_* is the stillwater elevation at the site, and H_* is the wave height at the site. The 0.7 coefficient is the portion of the wave height which reaches above the stillwater elevation. H_b is the upper limit for H_*.

The second major concept is that the breaking wave height may be diminished by dissipation of energy by natural or man-made obstructions. The wave height transmitted past a given obstruction is determined by the following equation:

$$H_t = BH_i$$

where H_t is the transmitted wave height, H_i is the incident wave height, and B is a transmission coefficient ranging from 0.0 to 1.0. The coefficient is a function of the physical characteristics of the obstruction. Equations have been developed by NAS to determine B for vegetation, buildings, natural barriers such as dunes, and man-made barriers such as breakwaters and seawalls (Reference 89).

The third concept deals with unimpeded reaches between obstructions. New wave generation can result from wind action. This added energy is related to distance and mean depth over the unimpeded reach.

As part of this countywide update, revised coastal analyses were performed for the open water flooding sources in the communities of Salisbury and Newburyport. In addition, redelineation of coastal flood hazard data was performed for open water flooding sources in the communities of Beverly, Danvers, Essex, Gloucester, Ipswich, Lynn, Manchester, Marblehead, Nahant, Newbury, Peabody, Rockport, Rowley, Salem, Saugus and Swampscott. A description of the revised analyses is presented in the Countywide section below.

For each coastal community within Essex County that has been studied prior to this countywide update, the coastal analyses described in the previous FIS reports have been compiled and are summarized below.

Pre-countywide Analyses

The extent and frequency of recurrence of coastal flooding were determined by conducting a frequency analysis of annual maximum tidal heights along the coastlines of Essex County. Some historic storm-tide heights, consisting of an astronomical tide and a storm surge contribution, were determined by the mathematical simulation of historic northeasters and hurricanes as described above; others, for which associated storm data were not available, were obtained by a correlation analysis using tide data from Boston or Portsmouth, The data base at the Boston gage extended from 1978 discontinuously back to 1848; the shorter record at Portsmouth was lengthened by a statistical correlation with data at Boston and Portland. The annual maxima of these reproduced historic water elevations were fitted with a Pearson Type III distribution. The goodness of fit was tested with the chi-square test and accepted at the 95 percent confidence level. The variations in location and bathymetry require the reporting of separate storm-tide elevations for Lynn Harbor and Nahant Bay. A detailed description of the methodology employed in this analysis can be found in the report entitled Determination of Coastal Storm Tide Levels (Reference 93). Tidal flood elevations for the Saugus River were obtained from an unpublished USACE study. The USACE study is a detailed analysis of flooding on the Saugus and Pines Rivers and is appropriate for use in this study. For Cedar Pond, Sluice Pond, and Flax Pond, a volume-elevation analysis was performed for the 10- and 1-percent-annual-chance rainfalls. The water-surface elevation for each pond was developed by determining the respective drainage areas, time of concentration, and rainfall duration and amount.

Stillwater elevations for the Parker River were determined using a one dimensional storm surge model for coastal rivers (Reference 87). The one- dimensional model is based on the hydrodynamic equations of motion and conservation of mass. The model was used where applicable for estuaries within Rowley.

Areas of shallow flooding have been determined for the lee side of the dunes along Massachusetts Bay. In these areas, the wave runup elevation exceeded the dune crest elevation. The difference between the runup elevation and the dune crest was used to determine the depth of shallow flooding behind the dune (Reference 94). Areas of ponding have been determined along Massachusetts Bay and Beverly Harbor. In these areas, the wave runup elevation exceeded the bluff elevation. The amount of overtopping and flooding behind the bluff were determined based on the bluff elevation and surrounding topography (References 94 and 95).

In Marblehead, in some locations, water levels shown on the maps were computed by correlating synthetically produced water levels with elevations obtained during historic floods in Marblehead (Reference 17). Historic flood damage information was also used to ensure reasonable delineation of flood-prone areas along the Marblehead shoreline (Reference 88).

The pre-countywide stillwater elevations for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods have been determined and are shown in Table 8. The analyses reported in this study reflect the stillwater elevations, shown in Table 8, due to tidal and wind setup effects and include the contributions from wave action effects.

TABLE 8 - PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS

	ELEVATION (feet NAVD 88)[1]			
FLOODING SOURCE AND LOCATION	10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
ATLANTIC OCEAN				
Affecting Essex Bay for Its entire length within The corporate limits of Essex	7.8	8.5	8.8	9.5
North Coast In Gloucester	7.8	8.6	8.9	9.6
South Coast In Gloucester	7.1	7.9	8.2	9.0
Entire shoreline within Ipswich	7.7	8.4	8.7	9.4
At Nahant Bay In Lynn	7.6	8.4	8.8	9.6
Saugus River General Edwards Bridge to Salem Turnpike in Lynn	7.2	7.8	8.2	9.0
Salem Turnpike to Boston Street in Lynn	6.7	7.3	7.6	8.4
At Lynn Harbor In Lynn	8.0	8.8	9.2	9.9
Entire coastline of Manchester	7.3	8.1	8.4	9.2
Entire coastline of Marblehead	7.6	8.4	8.8	9.5
Entire coastline within Nanhant	7.7	8.5	8.9	9.7

[1]North American Vertical Datum of 1988

TABLE 8 - PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS-continued

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD 88)[1] 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
ATLANTIC OCEAN – cont'd				
Entire coastline within Newbury	7.4	8.1	8.4	9.0
Entire coastline of Rockport	7.5	8.3	8.6	9.2
Shoreline of Massachusetts Bay, Salem Harbor, and Beverly Harbor In Salem	7.7	8.5	8.8	9.6
Entire shoreline within Community of Salisbury	7.4	8.1	8.3	9
Saugus tidal area	8.0	8.8	9.2	10
Entire coastline of Swampscott	7.6	8.4	8.7	9.5
At the Rowley/Newbury Corporate limits	7.5	8.2	8.5	9.2
At the Ipswich/Rowley Corporate limits	7.7	8.4	8.7	9.4
BEVERLY HARBOR				
Waters River, Porter River, and Crane River In Danvers	8.4	9.5	10.0	11.6
Waters River Within Peabody Limits	8.4	9.5	10.0	11.6
At the Danvers River In Beverly	7.5	8.3	8.7	9.4

[1]North American Vertical Datum of 1988

TABLE 8 - PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS-continued

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD 88)[1] 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
CEDAR POND				
In Lynn	98.2	*	109.7	*
FLAX POND				
In Lynn	54.2	*	54.8	*
* Data not available				
LAKE ATTITASH				
Entire shoreline within Amesbury/Merrimac	96.9	97.2	97.3	98
MASSACHUSETTS BAY				
At West Bay In Beverly	7.5	8.3	8.6	9.4
MERRIMACK RIVER				
In Amesbury	7.9	8.8	9.4	10.7
PARKERS RIVER				
Upstream of mouth to Boston and Main Railroad bridge	7.4	8.1	8.4	9.0
Upstream of Boston & Maine Railroad of Mill River downstream of U.S Route 1 bridge In Rowley	6.6	7.2	7.4	8
POND 1				
Approximately 1,300 feet north of intersection of Pleasant and Cherry Streets	*	*	47.1	*

* Data not available

[1]North American Vertical Datum of 1988

TABLE 8 - PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS-continued

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD 88)[1] 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
POND 2				
Approximately 700 feet north of intersection of Pleasant and Cherry Streets	*	*	53.6	*
PILLINGS POND				
At Lynnfield	97.4	98.1	98.3	98.8
SLUICE POND				
In Lynn	63.8	*	64.2	*
UPPER ARTICHOKE RESERVOIR				
Upstream of the Dam in Newburyport	12.6	12.9	13	13.1

* Data not available

[1]North American Vertical Datum of 1988

Countywide Analyses

As part of the countywide update, revised coastal analyses were performed for the open water flooding sources in the communities of Salisbury and Newburyport. In addition, redelineation of coastal flood hazard data was performed for open water flooding sources in the communities of Beverly, Danvers, Essex, Gloucester, Ipswich, Lynn, Manchester, Marblehead, Nahant, Newbury, Peabody, Rockport, Rowley, Salem, Saugus and Swampscott. Redelineation of coastal flood hazards is defined as applying the results of previous coastal analyses to new or more detailed topographic data. Provided below is a summary of the analyses performed. All revised coastal analyses and redelineation of coastal flood hazards were performed in accordance with Appendix D "Guidance for Coastal Flooding Analyses and Mapping," (Reference 96) of the Guidelines and Specifications (G&S), as well as, the "Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update" (Reference 97).

For the communities with revised coastal analyses, published values in the Tidal Flood Survey (Reference 98) were used to estimate the stillwater elevations for the 10-, 2-, and 1-percent-annual-chance floods for open water flooding sources.

The 0.2-percent-annual-chance stillwater elevations were extrapolated from the more the frequent stillwater elevations in the Tidal Flood Survey. For communities with redelineation of coastal flood hazard data, the 10-, 2-, 1- and 0.2-percent-annual-chance stillwater elevations are the same as published in the Previous FIS. Stillwater elevations for the revised and redelineated flooding sources are presented in Table 9.

TABLE 9 – SUMMARY OF REVISED STILLWATER ELEVATIONS

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD 88)[1]			
	10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
Newburyport				
Atlantic Ocean	7.8	8.6	8.9	9.7
Salisbury				
Atlantic Ocean	7.4	8.1	8.3	9

[1]North American Vertical Datum of 1988

For the communities with revised coastal analyses, the elevations presented in the Tidal Flood Survey (Reference 98) are referenced to the National Tidal Datum Epoch (NTDE) of 1960-1978. The current tidal datum is based on the NTDE of 1983-2001. The NTDE is a specific 19 year period that includes the longest periodic tidal variations caused by the astronomic tide-producing forces. The value averages out long term seasonal meteorological, hydrologic, and oceanographic fluctuations and provides a nationally consistent tidal datum network (bench marks) by accounting for seasonal and apparent environmental trends in sea level rise that affect the accuracy of tidal datums. For use in this coastal analysis revision, the stillwater elevations presented in the Tidal Flood Survey (Reference 98) were converted to the current tidal datum. A datum conversion factor of +0.05 feet was applied to the data in the Tidal Flood Survey for the communities of Salisbury and Newburyport, thus the data were converted to National Geodetic Vertical Datum of 1929 (NGVD 29), and the datum was then shifted to North American Vertical Datum of 1988 (NAVD 88).

For the communities with redelineation of coastal flood hazard data, the elevations presented in the previous FIS's are referenced to the National Geodetic Vertical Datum of 1929 (NGVD 29). These elevations were converted to the North American Vertical Datum of 1988 (NAVD 88). The vertical datum shift between NGVD 29 and NAVD 88 was determined in accordance with Appendix B "Guidance for Converting to the North American Vertical Datum of 1988," of the Guidelines and Specifications, as well as the Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update, (Reference 96).

For the communities with revised coastal analyses, wave setup along the open coast was calculated using the procedures detailed in the Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update, (Reference 96). Specifically, the Direct Integration Method (DIM) was applied. Because much of the Essex County coastline has experienced historical flooding and damage above predicted surge and runup elevations, setup was assumed to be an important component of the analyses and was applied to the entire open coast shoreline in the revised community, except for areas inundated by wave runup.

For the communities with revised coastal analyses, offshore wave characteristics representing a 1-percent-annual-chance storm were determined using hindcast wave data from the USACE Wave Information Studies (WIS) stations. A Peaks-Over-Threshold statistical analysis (Reference 99) was applied on 20 years (1980-1999) of wave characteristic data from WIS Station 45, located offshore of the Town of Salisbury. For areas sheltered from direct ocean waves, such as the Merrimac River and west facing shorelines, wave characteristics representing a 1-percent-annual-chance storm were determined using a restricted fetch analysis and the USACE Automated Coastal Engineering System (ACES) software package. Mean wave characteristics were determined as specified in the FEMA guidance for V Zone mapping.

Wave heights and wave runup in the communities with revised coastal analyses were computed along transects that were located perpendicular to the average shoreline. The transects were located with consideration given to the physical and cultural characteristics of the land so that they would closely represent local conditions. Transects were spaced close together in areas of complex topography and dense development. In areas having more uniform characteristics, the transects were spaced at larger intervals. It was also necessary to locate transects in areas where unique flooding existed and in areas where computed wave heights varied significantly between adjacent transects.

Transect data for the communities with redelineation of coastal hazard data are referenced to each community's previous FIS.

2013 Coastal Update

Coastal Hydrologic Analyses

The stillwater elevation is the elevation of the water surface due to storm surge and the astronomical tides coincident with a storm. In 1988, the USACE developed coastal flood frequency curves for the New England coastline, covering the Long Island Sound to the U.S.-Canada border in Maine. The data for this work was derived from high water marks collected after historical storm events and tide gauge records maintained by the USACE and NOAA. A Pearson Type III distribution was fitted to the data, from which inferences about flood recurrence intervals were made. The statistics at the gauge locations were then extrapolated along the coastline based on considerations of tidal hydrodynamics and high water marks from historic storms. This document has historically been the primary source of storm surge elevations (SWELs) for FEMA coastal studies.

In 2012, STARR, under contract to FEMA, published a revision to the flood frequency profiles (Reference 100). The revision incorporates approximately 20 additional years of tide gauge data collected since the 1988 USACE report and it uses the more statistically robust regionalized L-moments distribution fitting approach (Reference 101). The 10-percent, 2-percent, 1-percent, and 0.2-percent-annual-chance SWELs for this Study were obtained from the updated STARR flood frequency profiles. Stillwater elevations were linearly interpolated to all coastal transects throughout Essex County and are included in the digital data files compiled for the coastal submittal.

Coastal Hydraulic Analyses

At each station, the USACE has conducted return period analysis from which wave heights for different return periods were obtained. The corresponding wave periods were determined by considering wave steepness values typical of North Atlantic hurricanes as described in the G&S. The wave conditions were deshoaled to deep water to obtain the equivalent deep water wave conditions. The wave heights were averaged to provide a single wave height and wave period for the open coast transects.

The southern portions of the Essex County coastline has some areas where islands and penisulas cause parts of the shoreline to be largely sheltered from wind waves generated in the Atlantic Ocean. Wave conditions for transects along these coastlines were derived using the methodology prescribed by the USACE for computing wave growth in fetch-restricted water bodies. The approach is implemented in the ACES software package, which was used for this work.

For each transect, the geometry of the basin was defined by wind fetches parallel to the transect direction. As recommended in the Coastal Engineering Manual (CEM) and FEMA guidance, the deep water wave growth option was used in all cases irrespective of the average depth of the wind basin. The ACES technical report notes that the shallow-water forms of the wave growth equations attempt to incorporate the effects of bottom friction and percolation but that the formulations are still largely experimental and unverified. The CEM instead recommends that the computed wave height be capped by depth-limited wave breaking considerations and the wave period (T_P) be capped by the limiting wave period:

$$T_P \approx 9.78\left(\frac{d}{g}\right)^{1/2}$$

where d is the average water depth and g is standard gravity.

For a given effective wind fetch, the wind duration for which waves attain a fully-developed sea state is found by marching through a range (zero to six hours) of wind speed averaging intervals.

The SWELs for different recurrence intervals were derived by statistical analysis of tide gauge records in New England. The results of the analysis at the tide gauge stations were used to develop flood profiles along the New England coastline.

Wave setup was computed at each transect using the DIM as described in the G&S. On low-lying transects inundated by storm surge, the propagation of waves overland was modeled using the Wave Height Analysis for Flood Insurance Studies (WHAFIS 4.0) tool (Reference 102).

Transects (profiles) were located for coastal hydrologic and hydraulic analyses perpendicular to the average shoreline along areas subject to coastal flooding extending off-shore to areas representative of deep water conditions and extending inland to a point where wave action ceased, in accordance with the User's Manual for Wave Height Analysis (Reference 103). Transects were placed with consideration of topographic and structural changes of the land surface, as well as the cultural characteristics of the land, so that they would closely represent local conditions.

Transects were closely spaced in areas of complex topography and dense development, and spaced at larger intervals in areas having more uniform characteristics. It was also necessary to locate transects in areas where unique flooding existed and in areas where computed wave heights varied significantly between adjacent transects. Figure 1 shows the transect layout for Essex County. A total of 67 transects were used to capture the coastal characteristics in Essex County.

On steep transects where wave runup, rather than storm surge inundation is the dominant source of flooding, wave runup was computed using the RUNUP 2.0 tool or the Technical Advisory Committee for Water Retaining Structures (TAW) method, as described in the G&S. The choice of runup methodology was dictated by the steepness of the near shore profile. Both WHAFIS 4.0 and RUNUP 2.0 are implemented in the Coastal Hazard Analysis Modeling Program (CHAMP) (Reference 103). In some cases the above mentioned run-up methods were not applicable. In that case a more applicable runup method was chosen in accordance with the G&S.

During significant coastal storms shoreline profiles are altered due to episodic erosion and can allow for greater landward propagation of waves. Erosion analysis, using CHAMP, is performed on profiles with erodible dunes with no coastal structures. Dunes subject to erosion must be sandy features with potentially light vegetation. Any thickly vegetated, rocky, silty, or clayey dune features or bluffs were not eroded.

On transects with significant inland excursion of the 1-percent-annual-chance SWEL, WHAFIS 4.0 was used to compute the propagation of waves inland. Along each transect, WHAFIS takes as input, the 1-percent SWEL and corresponding wave conditions (i.e. the significant wave height and peak wave period), a bathymetric and topographic profile (entered as station-elevation pairs), and input "cards" at each station describing vegetation and land-use characteristics. WHAFIS uses this information to compute wave heights, wave crest elevations, flood insurance risk zone designations, and flood zone boundaries along each transect.

The wave action conservation equation used within the model governs both wave regeneration caused by wind and wave dissipation resulting from marsh plants. This equation is supplemented by the conservation of wave equation, which expresses the spatial variation of the wave period at the peak of the wave spectrum. The wave heights and period respond to changes in wind conditions, water depths, and obstructions as a wave propagates. These equations are solved as a function of distance along the wave analysis transects.

Table 10, "Transect Descriptions," provides a listing of the transect locations, stillwater elevations, and maximum wave crest (or wave runup) elevations along the shoreline. Transects have been re-numbered to conform to countywide standard.

Along each transect, WHAFIS computes wave heights and wave crest elevations taking into account the combined effects of changes in ground elevation, vegetation, and other obstructions. Wave heights are calculated to the nearest 0.1 foot, and wave crest elevations are computed at whole-foot intervals. The calculations are carried inland along the transect until the wave crest elevation is permanently less than 0.5 foot above the SWEL or until the coastal flooding meets another flood source (e.g., a riverine flood source). The results of this analysis are summarized in Table 11, "Transect Data."

TABLE 10 - TRANSECTS DESCRIPTIONS

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
1	The transect is located approximately 600 feet Southwest of the intersection of Commercial Street and Bennett Street, extending Southeast towards Broad Sound.	11.0	*
2	The transect is located approximately 600 feet Southeast of the intersection of Broad Street and Newhall Street, extending Southwest towards Broad Sound.	11.1	*
3	The transect is located approximately 300 feet Northwest of the intersection of Nahant Road and Castle Road, extending northeast towards Nahant Bay.	13.9	21.3
4	The transect is located approximately 100 feet northwest of the intersection of Castle Road and Pearl Road extending west towards Broad Sound.	10.5	14.7
5	The transect is located at a point approximately 300 feet south of the intersection of Gardner Road and Breesy Hill Ter. extending south towards the Atlantic Ocean.	14.5	*
6	The transect is located at a point approximately 100 feet northeast of the intersection of Ocean Street and Intervale Road, extending south towards the Atlantic Ocean.	14.5	22.2
7	The transect is located at a point approximately 300 feet southeast of the intersection of Nahant Road and Swallow Cave Road, extending south towards the Atlantic Ocean.	14.6	22.4

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 10 - TRANSECTS DESCRIPTIONS-continued

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
8	The transect is located at a point approximately 300 feet southwest of the intersection of Nahant Street and Forty Steps Lane.	14.6	*
9	The transect is located in Nahant Bay at a point approximately 100 feet east of the intersection of Ocean Street and Highland Road, extending northeast towards the Atlantic Ocean.	14.2	*
10	The transect is located in Nahant Bay at a point approximately 500 feet northeast of the intersection of Little Nahant Road and Lenox Road, extending east towards the Atlantic Ocean.	13.8	*
11	The transect is located in Nahant Bay at a point approximately 500 feet east of the intersection of Sagamore Street and Sachem Street, extending southeast towards the Atlantic Ocean..	13.8	*
12	The transect is located in Nahant Bay at a point approximately 300 feet north of the intersection of Red Rock Street and Ocean Street, extending southeast towards the Atlantic Ocean..	13.8	21.2
13	The transect is located in Nahant Bay at a point approximately 200 feet northeast of the intersection of Humphry Street and Millett Road, extending southeast towards the Atlantic Ocean..	14.2	21.8

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 10 - TRANSECTS DESCRIPTIONS-continued

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
14	The transect is located at a point approximately 700 feet east of the intersection of Puritan Road and Winshaw Road, extending southeast towards the Atlantic Ocean.	14.1	*
15	The transect is located at a point approximately 300 feet southeast of the intersection of Humphry Street and Palmer Avenue, extending southeast towards the Atlantic Ocean.	14.0	21.4
16	The transect is located at a point approximately 100 feet northwest of the intersection of Phillips Avenue and Stanwood Road, extending southeast towards the Atlantic Ocean.	13.7	*
17	The transect is located at a point approximately 100 feet northwest of the intersection of Atlantic Avenue and Garden Road, extending southeast towards the Atlantic Ocean.	13.7	*
18	The transect is located at a point approximately 250 feet northeast of the intersection of Mohawk Road and Sagamore Road, extending southeast towards the Atlantic Ocean.	13.7	21.0
19	The transect is located at a point approximately 500 feet north of the intersection of Flint Street and Bonad Road, extending southwest towards the Atlantic Ocean.	13.9	21.3
20	The transect is located at a point approximately 200 feet northwest of the intersection of Dennett Road and Risley Road, extending southeast towards the Atlantic Ocean.	14.0	*

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 10 - TRANSECTS DESCRIPTIONS-continued

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
21	The transect is located at a point approximately 200 feet northwest of the intersection of Ocean Avenue and Brown Street, extending east towards the Atlantic Ocean.	14.3	*
22	The transect is located in Marblehead Harbor at a point approximately 500 feet southeast of the intersection of Ocean Avenue and Beach Street, extending northeast towards the Atlantic Ocean..	10.8	15.9
23	The transect is located at a point approximately 200 feet south of the intersection of Pond Street and Cowell Street, extending east towards the Atlantic Ocean.	13.8	*
24	The transect is located at a point approximately 400 feet northeast of the intersection of Davis Road and Crownisshield Road, extending east towards the Atlantic Ocean.	13.7	*
25	The transect is located at a point approximately 500 feet northwest of the intersection of Bradlee Road and Mooring Road, extending northeast towards the Atlantic Ocean.	13.8	*
26	The transect is located in Salem Harbor at a point approximately 300 feet east of the intersection of Beverly Avenue and Knight Avenue, extending northwest towards Salem Harbor	10.5	*

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 10 - TRANSECTS DESCRIPTIONS-continued

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
27	The transect is located along Forest River at a point approximately 100 feet northeast of the intersection of Loring Avenue and Leggs Hill Road, extending northeast towards Salem Harbor.	10.5	*
28	The transect is located at a point approximately 250 feet west of the intersection of Essex Street and Forrester Street, extending southeast towards Salem Harbor.	10.3	13.2
29	The transect is located at a point approximately 1,200 feet southeast of the intersection of Fort Avenue and Winter Island Road, extending east towards the Atlantic Ocean.	13.9	21.3
30	The transect is located at a point approximately 700 feet northwest of the intersection of Fort Avenue and Memorial Drive, extending northeast towards the Atlantic Ocean.	13.7	21.0
31	The transect is located at a point approximately 300 feet south of the intersection of Hawthorne Street and Endicott Street, extending northeast towards Beverly Harbor.	10.3	14.0
32	The transect is located in Beverly Harbor at a point approximately 100 feet southeast of the intersection of Lovett Street and Abbott Street, extending southeast towards the Atlantic Ocean..	13.7	*
33	The transect is located at a point approximately 800 feet southeast of Neptune Street and Bay View Road, extending south towards the Atlantic Ocean.	13.6	*

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 10 - TRANSECTS DESCRIPTIONS-continued

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
34	The transect is located at a point approximately 2,200 feet east of the intersection of Hale Street and Prince Street, extending southeast towards the Atlantic Ocean.	13.6	20.9
35	The transect is located at a point approximately 1,700 feet southeast of the intersection of Hale Street and Paine Avenue, extending southeast towards the Atlantic Ocean.	13.9	21.3
36	The transect is located at the center of Baker Island, extending northeast towards the Atlantic Ocean.	14.1	*
37	The transect is located at a point approximately 1,800 feet southeast of the intersection of Bridge Street and Boardman Avenue, extending southeast towards the Atlantic Ocean.	14.5	*
38	The transect is located at a point approximately 400 feet northwest of the intersection of Gales Point Road and Masconomo Street, extending southwest towards the Atlantic Ocean.	10.6	15.6
39	The transect is located at a point approximately 1,800 feet southwest of the intersection of Masconomo Street and Proctor Street, extending southeast towards the Atlantic Ocean.	14.6	*
40	The transect is located at a point approximately 200 feet southwest of the intersection of Masconomo Street and Proctor Street, extending southeast towards the Atlantic Ocean.	14.3	*

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 10 - TRANSECTS DESCRIPTIONS-continued

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
41	The transect is located in Kettle Cove at a point approximately 300 feet northeast of the intersection of Summer Street and Ocean Street, extending southwest towards the Atlantic Ocean.	14.1	*
42	The transect is located in Magnolia Harbor at a point approximately 800 feet southeast of the intersection of Summer Street and Raymond Street, extending southeast towards the Atlantic Ocean.	14.4	22.1
43	The transect is located at a point approximately at the intersection of Lexington Avenue and Cliff Avenue, extending south towards the Atlantic Ocean.	14.5	*
44	The transect is located at a point approximately 200 feet southeast of the intersection of Hesperus Avenue and Castle Hill Road, extending southeast towards the Atlantic Ocean.	14.2	*
45	The transect is located in Gloucester Harbor at a point approximately 100 feet northwest of the intersection of Church Street and Pine Street, extending southwest towards the Atlantic Ocean.	14.1	22.1
46	The transect is located at a point approximately 500 feet south of the intersection of Fort Hill Avenue and Ramparts Field Road, extending southeast towards the Atlantic Ocean.	14.2	*

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 10 - TRANSECTS DESCRIPTIONS-continued

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
47	The transect is located at a point approximately 300 feet south of the intersection of Souther Road and Tagabigazanda Road, extending east towards the Atlantic Ocean.	14.4	*
48	The transect is located at a point approximately 200 feet southwest of the intersection of Eastern Avenue and Old Country Road, extending southeast towards the Atlantic Ocean.	14.2	21.8
49	The transect is located at a point approximately at the intersection of Long Beach Road and Twilight Avenue, extending south towards the Atlantic Ocean.	14.0	*
50	The transect is located approximately 1700 feet northwest of the intersection of Thatcher Road and Highview Road, extending southeast towards the Atlantic Ocean.	11.6	17.8
51	The transect is located approximately 300 feet north of the intersection of Thatcher Road and Ridgewood Road, extending southeast towards the Atlantic Ocean.	11.6	17.8
52	The transect is located in Lobiolly Cove at a point approximately 700 feet northwest of the intersection of Eden Road and Penzance Road, extending east towards the Atlantic Ocean.	14.2	21.8
53	The transect is located approximately 200 feet southeast of the intersection of Marmion Way and Old Garden Road, extending north towards the Atlantic Ocean.	14.3	*

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 10 - TRANSECTS DESCRIPTIONS-continued

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
54	The transect is located in Back Harbor at a point approximately 400 feet southwest of the intersection of King Street and Forest Street, extending northeast towards the Atlantic Ocean.	14.7	*
55	The transect is located approximately 400 feet northwest of the intersection of Granite Street and Landmark Lane, extending northeast towards the Atlantic Ocean.	14.7	*
56	The transect is located approximately 100 feet southeast of the intersection of Phillips Avenue and Haven Avenue, extending northeast towards the Atlantic Ocean.	14.7	*
57	The transect is located in Folly Cove at a point approximately 700 feet southeast of the intersection of Washington Street and Woodbury Street, extending north towards the Atlantic Ocean.	14.2	21.8
58	The transect is located approximately 100 feet north of the intersection of Langsford Street and Langsford Way, extending northwest towards the Atlantic Ocean.	13.3	*
59	The transect is located approximately 500 feet southwest of the intersection of Ocean Avenue and Norrock Road, extending northwest towards the Atlantic Ocean.	12.9	19.8
60	The transect is located approximately 100 feet south of the intersection of Wyona Road and Point Road, extending north towards the Atlantic Ocean.	12.5	*

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 10 - TRANSECTS DESCRIPTIONS-continued

Transect	Location	ELEVATION (feet NAVD88[1]) 1-PERCENT-ANNUAL-CHANCE STILLWATER[2]	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[3]
61	The transect is located approximately 2,200 feet northeast of the intersection of Concord Street and Coles island Road, extending northeast towards the Atlantic Ocean.	12.5	19.2
62	The transect is located in Essex Bay at a point approximately 350 feet northwest of the intersection of Conomo Point Road and Robbins Island Road, extending northeast towards the Atlantic Ocean.	12.5	19.3
63	The transect is located in Essex Bay at a point approximately at the confluence of Hog Island Channel and Essex Bay, extending northeast towards the Atlantic Ocean.	12.5	19.2
64	The transect is located at a point approximately 4,200 feet northeast of the intersection of Argilla Road and Fox Creek Road, extending northeast towards the Atlantic Ocean.	12.4	*
65	The transect is located at a point approximately 100 feet northeast of the intersection of Hilltop Road and Third Street, extending east towards the Atlantic Ocean.	12.5	*
66	The transect is located in Plum Island Sound at a point approximately 1500 feet northeast of the intersection of Main Street and Cross Street, extending northeast towards the Atlantic Ocean.	12.6	19.3
67	The transect is located in Essex Bay at a point approximately 100 feet southwest of the intersection of Hancock Street and Marlboro Street.	12.9	15.9

[1]North American Vertical Datum of 1988
[2]Including stillwater elevation and effects of wave setup.
[3]Because of map scale limitations, maximum wave elevations may not be shown on the FIRM.
*1% annual chance water level governed by wave runup

TABLE 11 - TRANSECT DATA

STILLWATER ELEVATION

TRANSECT	FLOODING SOURCE	10-PERCENT-ANNUAL-CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD88*)[1]
1	Broad Sound	8.4	9.5	10.0	11.4	VE	14
						AE	10-14
2	Broad Sound	8.4	9.5	10.0	11.4	VE	18
3	Nahant Bay	8.4	9.5	10.0	11.4	VE	16-21
						AE	15
						VE	15
4	Broad Sound	8.4	9.5	10.0	11.4	VE	15
5	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	24
						AO	2
6	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	22
						AE	15
7	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	22
						AE	15
8	Nahant Bay	8.4	9.5	10.0	11.4	VE	48
9	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	38

[1]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*North American Vertical Datum of 1988

TABLE 11 - TRANSECT DATA-continued

STILLWATER ELEVATION

TRANSECT	FLOODING SOURCE	10-PERCENT-ANNUAL-CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD88*)[1]
10	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	40
11	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	21
						AO	3
12	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	20
13	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	22
						AE	15
14	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	41
15	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	16-21
						AE	15-16
16	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	23
						AO	3
17	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	40

[1]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*North American Vertical Datum of 1988

TABLE 11 - TRANSECT DATA-continued

STILLWATER ELEVATION

TRANSECT	FLOODING SOURCE	10-PERCENT-ANNUAL-CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD88*)[1]
18	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	16-21
						AE	16
19	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	21
						AE	14
20	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	47
21	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	21
22	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	16
						AE	16
23	Atlantic Ocean	8.4	9.5	10.0	11.4	VE	43
						AO	3
						AE	14
24	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	24
25	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	20

[1]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*North American Vertical Datum of 1988

TABLE 11 - TRANSECT DATA-continued

STILLWATER ELEVATION

TRANSECT	FLOODING SOURCE	10-PERCENT-ANNUAL-CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD88*)[1]
26	Salem Harbor	8.4	9.4	10.0	11.4	VE	17
27	Salem Harbor	8.4	9.4	10.0	11.4	VE	16
						AE	16
28	Salem Harbor	8.4	9.4	10.0	11.4	VE	13
						AE	10
29	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	19-21
						AE	19
30	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	21
						AE	14
31	Beverly Harbor	8.4	9.4	10.0	11.4	VE	14
						AE	10
32	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	30
33	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	25
34	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	21
35	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	21
						AE	14

[1]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*North American Vertical Datum of 1988

TABLE 11 - TRANSECT DATA-continued

		STILLWATER ELEVATION					
TRANSECT	**FLOODING SOURCE**	**10-PERCENT-ANNUAL-CHANCE**	**2-PERCENT-ANNUAL-CHANCE**	**1-PERCENT-ANNUAL-CHANCE**	**0.2-PERCENT-ANNUAL-CHANCE**	**ZONE**	**BASE FLOOD ELEVATION (feet NAVD88*)[1]**
36	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	35
37	Atlantic Ocean	8.4	9.4	10.0	11.4	VE	35
38	Atlantic Ocean	8.4	9.4	9.9	11.4	VE	16
						AE	11
39	Atlantic Ocean	8.4	9.4	9.9	11.3	VE	38
						AO	3
40	Atlantic Ocean	8.4	9.4	9.9	11.3	VE	23
41	Atlantic Ocean	8.4	9.4	9.9	11.3	VE	26
						AO	2
42	Atlantic Ocean	8.4	9.4	9.9	11.3	VE	17-22
						VE	20
43	Atlantic Ocean	8.4	9.4	9.9	11.3	VE	33
						AE	33
44	Atlantic Ocean	8.3	9.4	9.9	11.3	VE	42

[1]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*North American Vertical Datum of 1988

TABLE 11 - TRANSECT DATA-continued

STILLWATER ELEVATION

TRANSECT	FLOODING SOURCE	10-PERCENT-ANNUAL-CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD88*)[1]
45	Atlantic Ocean	8.3	9.4	9.9	11.3	VE	22
						AE	15-16
46	Atlantic Ocean	8.3	9.4	9.9	11.3	VE	29
47	Atlantic Ocean	8.3	9.3	9.9	11.3	VE	34
48	Atlantic Ocean	8.3	9.3	9.9	11.2	VE	17-22
						AE	15-16
49	Atlantic Ocean	8.3	9.3	9.8	11.2	VE	34
50	Atlantic Ocean	8.3	9.3	9.8	11.2	VE	16-18
						AE	14
51	Atlantic Ocean	8.3	9.3	9.8	11.2	VE	16-18
						AE	15
52	Atlantic Ocean	8.3	9.3	9.8	11.2	VE	22

[1]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*North American Vertical Datum of 1988

TABLE 11 - TRANSECT DATA-continued

		STILLWATER ELEVATION					
TRANSECT	**FLOODING SOURCE**	**10-PERCENT-ANNUAL-CHANCE**	**2-PERCENT-ANNUAL-CHANCE**	**1-PERCENT-ANNUAL-CHANCE**	**0.2-PERCENT-ANNUAL-CHANCE**	**ZONE**	**BASE FLOOD ELEVATION (feet NAVD88*)[1]**
53	Atlantic Ocean	8.2	9.3	9.8	11.2	VE	25
54	Atlantic Ocean	8.2	9.3	9.8	11.2	VE	34
55	Atlantic Ocean	8.2	9.2	9.8	11.1	VE	36
56	Atlantic Ocean	8.2	9.2	9.7	11.1	VE	34
57	Atlantic Ocean	8.2	9.2	9.7	11.1	VE	22
						VE	20
						AE	20
58	Atlantic Ocean	8.2	9.2	9.7	11.1	VE	27
59	Atlantic Ocean	8.2	9.2	9.7	11.1	VE	20
60	Atlantic Ocean	8.2	9.2	9.7	11.1	VE	31

[1]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*North American Vertical Datum of 1988

TABLE 11 - TRANSECT DATA-continued

TRANSECT	FLOODING SOURCE	STILLWATER ELEVATION 10-PERCENT-ANNUAL-CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD88*)[1]
61	Atlantic Ocean	8.2	9.2	9.7	11.1	VE	19
						AE	10-12
62	Atlantic Ocean	8.2	9.2	9.7	11.1	VE	19
						AE	10
						VE	16
63	Atlantic Ocean	8.2	9.2	9.7	11.0	VE	19
						VE	15
						AE	10
64	Atlantic Ocean	8.1	9.2	9.7	11.0	VE	28
65	Atlantic Ocean	8.1	9.2	9.7	11.0	VE	27

[1]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*North American Vertical Datum of 1988

TABLE 11 - TRANSECT DATA-continued

TRANSECT	FLOODING SOURCE	STILLWATER ELEVATION 10-PERCENT-ANNUAL-CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD88*)[1]
66	Atlantic Ocean	8.1	9.2	9.6	11.0	VE	19
						AE	10-12
67	Atlantic Ocean	8.1	9.1	9.6	11.0	VE	16
						AE	10-14

[1]Because of map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted.
*North American Vertical Datum of 1988

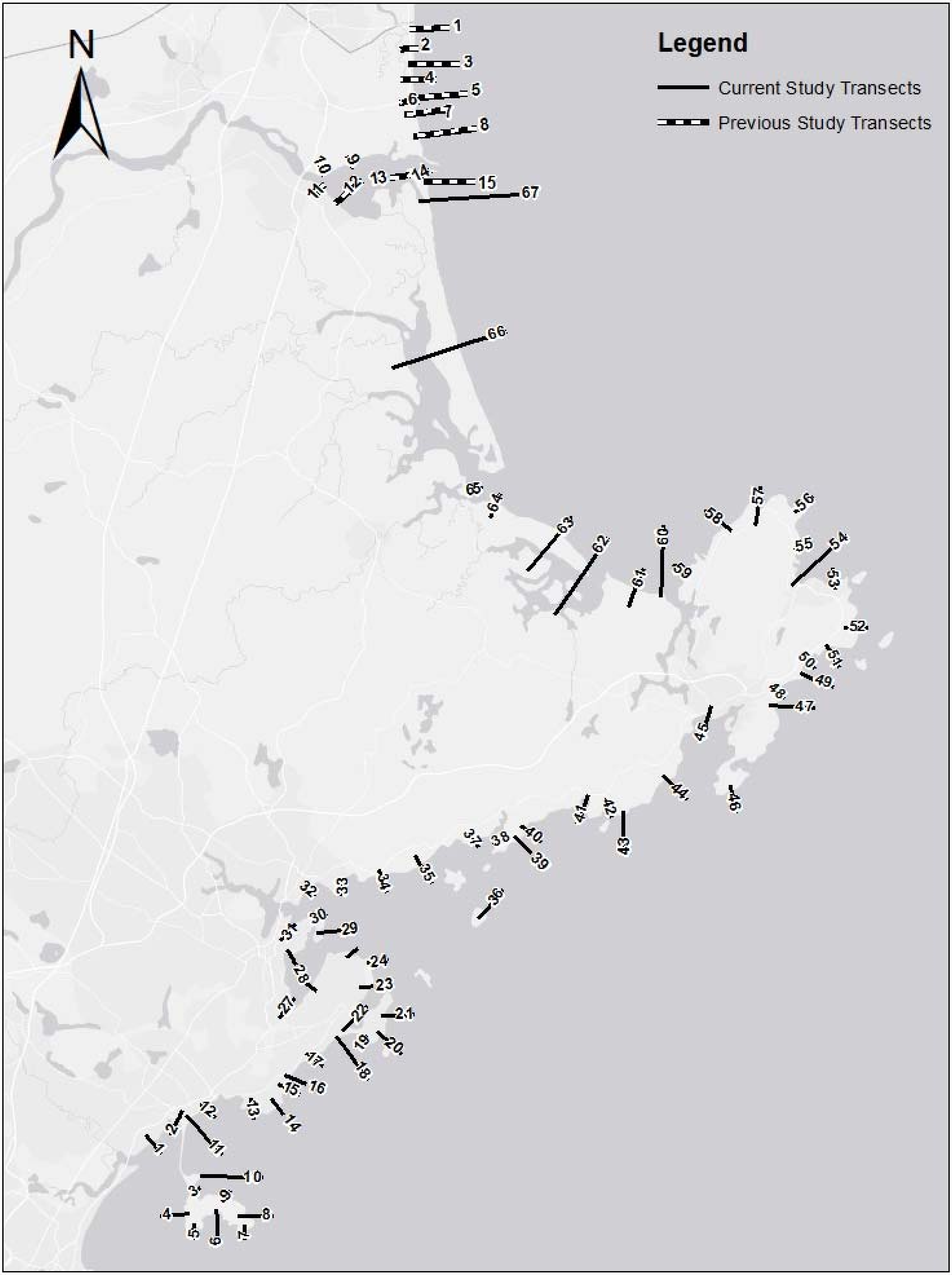


Figure 1 - Transect Location Map

Figure 2, “Transect Schematic” represents s sample transect that illustrates the relationship between stillwater elevation, the wave crest elevation, the ground elevation profile and the location of the V/A zone boundary.

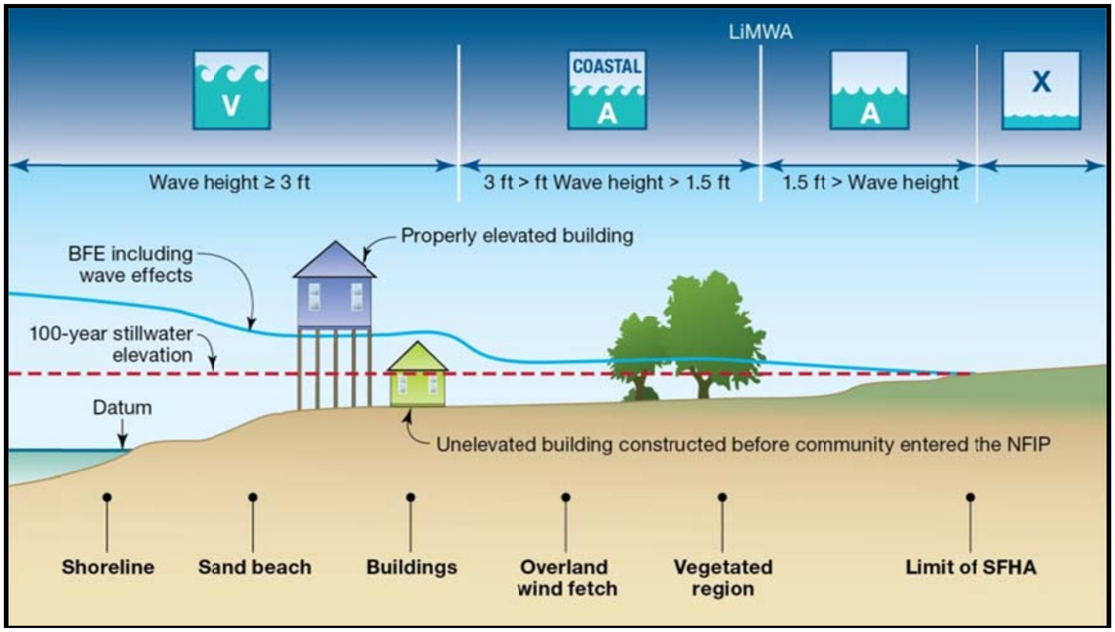


Figure 2. Transect Schematic

3.4 Vertical Datum

All FIS reports 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 used for newly created or revised FIS reports and FIRMs was the National Geodetic Vertical Datum of 1929 (NGVD 29). With the completion of the North American Vertical Datum of 1988 (NAVD 88), many FIS reports and FIRMs are now prepared using NAVD 88 as the referenced vertical datum.

All flood elevations shown in this FIS report and on the FIRM are referenced to the NAVD 88. These flood elevations must be compared to structure and ground elevations referenced to the same vertical datum. Ground, structure, and flood elevations may be compared and/or referenced to NGVD 29 by applying a standard conversion factor. **The conversion factor from NGVD 29 to NAVD 88 is -0.8, and from NAVD 88 to NGVD 29 is +0.8.**

For information regarding conversion between the NGVD 29 and NAVD 88, visit the National Geodetic Survey website at www.ngs.noaa.gov, or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3282
(301) 713-3242

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 the FIS report and FIRM for this county. Interested individuals may contact FEMA to access these data.

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 29 should apply the stated conversion factor 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.

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

4.0 FLOODPLAIN MANAGEMENT APPLICATIONS

The NFIP encourages State and local governments to adopt sound floodplain management programs. To assist in this endeavor, each FIS report provides 1-percent-annual-chance floodplain data, which may include a combination of the following: 10-, 2-, 1-, and 0.2-percent-annual-chance flood elevations; delineations of the 1- and 0.2-percent-annual-chance floodplains; and a 1-percent-annual-chance floodway. This information is presented on the FIRM and in many components of the FIS report, including Flood Profiles, Floodway Data tables, and Summary of Stillwater Elevation tables. Users should reference the data presented in the FIS report as well as additional information that may be available at the local community map repository before making flood elevation and/or floodplain boundary determinations.

4.1 Floodplain Boundaries

In order to provide a national standard without regional discrimination, the 1-percent-annual-chance flood has been adopted by FEMA as the base flood for floodplain management purposes. The 0.2-percent-annual-chance flood is employed to indicate additional areas of flood risk in the community.

The 1- and 0.2-percent-annual-chance floodplain boundaries are shown on the FIRM (Exhibit 2). On this map, the 1-percent-annual-chance floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones A, AE, AO, and VE), and the 0.2-percent-annual-chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1- and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations, but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

For the streams studied by approximate methods, only the 1-percent-annual-chance floodplain boundary is shown on the FIRM.

Redelineation of coastal flood hazard data was performed for open water flooding sources in the communities of Beverly, Danvers, Essex, Gloucester, Ipswich, Lynn, Manchester, Marblehead, Nahant, Newbury, Peabody, Rockport, Rowley, Salem, Saugus and Swampscott by applying the results of previous coastal analyses to new or updated topographic data.

For the countywide revision, the following communities provided high resolution topographic data. For these communities, this data was used for redelineation of the detailed and approximate study reaches.

- City of Beverly – Town Provided, Contour Interval 2 feet
- City of Methuen – Town Provided, Contour Interval 2 feet
- Town of Topsfield – James W. Sewall Company, Contour Interval 2 feet

For unrevised flooding sources in Essex County, data was taken from previously printed FISs for each individual community and are compiled below.

For the streams studied in detail, the 1- and 0.2-percent-annual-chance floodplain boundaries have been delineated using the flood elevations determined at each cross section. The boundaries were interpolated between cross sections using topographic maps at a scale of 1:4,800 with a contour interval of 5 feet in Andover (Reference 108); using photogrammetric maps in Beverly (Reference 109); at a scale of 1:4,800 with a contour interval of 4 feet in Boxford (Reference 110); at a scale of 1:2,400 and 1: 4,800 with contours intervals of 5 feet in Danvers, Lawrence, and Haverhill, Manchester, Merrimac, Methuen, Middleton, Salem, and Saugus (Reference 111 and 112); at a scale of 1:24,000 with a contour interval of 10 feet in Amesbury, Groveland, Hamilton, Topsfield, Wenham, West Newbury, and Georgetown (Reference 112); using topographic maps in Ipswich, Lynn, Gloucester, and Essex, Newbury, Newburyport, Rowley, Salisbury and Swampscott (Reference 113); at scales of 1:24,000 and 1:2,400 with contour intervals of 10 and 5 feet in Lynnfield (References 114 and 115); at a scale of

1:2,400 with a contour interval of 5 feet in Nahant (Reference 115); at a scale of 1:4,800 feet with a contour interval of 10 feet in North Andover (Reference 116)

For the 2011 revision, floodplain boundaries for the Shawsheen River have been delineated using LiDAR collected in 2006/2007. The vertical and horizontal accuracy of the LiDAR data are summarized in a May 29, 2007, report entitled Final LiDAR Report, Shawsheen River, Middlesex/ Essex Counties (URS Group Inc., 2007).

The bare earth LiDAR data were used to develop a digital terrain model in the form of a TIN (Triangulated Irregular Network). The TIN surface was generated as the source of ground elevations for the hydraulic model preparation and mapping work.

Detailed study reaches that were redelineated based on the improved topographic information may include a profile base line which provides a link to the flood profiles included in the FIS report. The profile base lines for these streams were based on the best available data at the time of their study and are depicted as they were on the previous FIRMs. In some cases where improved topographic data was used to redelineate floodplain boundaries, the profile base line may deviate from the channel centerline or may be outside the floodplain boundaries.

For the areas studied by approximate methods, the boundary of the 1-percent-annual-chance year flood was delineated using the Flood Hazard Boundary Map (FHBM) and topographic maps for the respective communities.

For this 2013 coastal revision STARR performed coastal flood hazard analysis for the study area that included the collection of storm surge (coastal hydrology) data and conducting overland wave height analysis (coastal hydraulics). For storm surge or stillwater elevations, the STARR team used the "Tidal Flood Profiles New England Coastline," prepared by New England Division, USACE, dated September 1988. STARR has reviewed the FEMA HQ report titled, "Updating Tidal Profiles for New England Coastline," dated December 3, 2008, for the 10-, 2-, 1-, and 0.2- percent-annual-chance flood events. The 1988 profiles also reflect highwater information for multiple areas resulting from the Hurricanes of 1938 and February 1978 extratropical events.

The overland wave height analysis was performed using CHAMP. Results of the overland wave height analysis were transferred to topographic work maps.

After the wave models were reviewed, the model outputs were imported into ArcMap and zone point shapefiles were generated. The zone point shapefiles delineate the change in BFEs along the transect and can be used to map the BFE changes. The BFEs were separated by drawing gutter lines which connect the zone point breaks between transects.

STARR delineated the 1- and 0.2-percent-annual-chance floodplain boundaries for

Essex County using standard GIS utilities. The STARR team manually drew the floodplain boundaries on the on 2-foot topographic contours derived from the terrain model using LiDAR collected in 2011. Aerial imagery and land use data assisted in the development of these features.

Zone VE (high wave velocity action area) was assigned to areas where the wave height is at least 3 feet. Since the wave crest is 70 percent of the controlling wave height above the stillwater plus setup surface, the wave crest in Zone VE is at least 2.1 feet higher than the stillwater plus wave setup elevation. Zone AE was assigned to areas where the total wave height is less than 3 feet and the wave crest is less than 2 feet above the stillwater plus wave setup elevation. Any zone width that is less than 0.2 times the FIRM scale was merged into the adjacent higher elevation zone. In the case of Essex County, the FIRM scales are 1 inch equals 500 feet, so zone widths of less than 100 feet were usually merged to the adjacent higher zone.

In March 2007, FEMA developed the guidance on the identification and mapping of the LiMWA. For Essex County, MA this mapping was done by identifying the LiMWA location(s) along each transect using the WHAFIS output and connecting those points between transects using gutter lines. In areas where runup elevations dominate over WHAFIS wave height, such as areas with steeply sloping beaches or high bluffs, there is no need to delineate the LiMWA. To retain continuous LiMWA lines in runup areas, the LiMWA was placed immediately landward of the mapped VE/AE Zone boundary and coincident with the 1-percent-annual-chance floodplain boundary in areas without an AE zone.

The 1- and 0.2-percent-annual-chance floodplain boundaries are shown on the FIRM (Exhibit 1). The 1-percent-annual-chance floodplain boundary corresponds to the boundary of the areas of special flood hazards (Zones AE and VE), and the 0.2-percent-annual-chance floodplain boundary corresponds to the boundary of areas of moderate flood hazards. In cases where the 1- and 0.2-percent-annual-chance floodplain boundaries are close together, only the 1-percent-annual-chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

4.2 Floodways

Encroachment on floodplains, such as structures and fill, reduces flood-carrying capacity, increases flood heights and velocities, and increases flood hazards in areas beyond the encroachment itself. One aspect of floodplain management involves balancing the economic gain from floodplain development against the resulting increase in flood hazard. For purposes of the NFIP, a floodway is used as a tool to assist local communities in this aspect of floodplain management. Under this concept, the area of the 1-percent-annual-chance floodplain is divided into a floodway and a floodway fringe. The floodway is the channel of a stream, plus any adjacent floodplain areas, that must be kept free of encroachment so that the base flood can be carried without substantial increases in flood heights. Minimum Federal standards limit such increases to 1 foot, provided that hazardous velocities are not produced. The floodways in this study are presented to local agencies as minimum standards that can be adopted directly or that can be used as a basis for additional floodway studies.

Floodway widths were computed at cross sections. 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 (see Table 12, “Floodway Data”). In cases where the floodway and 1-percent-annual-chance floodplain boundaries are either close together or collinear, only the floodway boundary is shown.

Since the communities of Newburyport and West Newbury are located directly across the river from Amesbury, the floodway for the Merrimack River in the City of Amesbury was taken from the completed FIS’s for these two communities (References 118 and 119). It is noted that the floodway limits on the Merrimack River are generally at the edge of the river bank with little resulting rise in water-surface. The floodway boundaries for Hussey Brook Tributary are coincident with the channel banks.

Wherever applicable, the Powwow River floodway was computed by Method 6 of the HEC-2 computer program. Method 6 reduces conveyance equally on both sides until a one-foot rise in energy gradient is obtained. On portions of the Powwow River, the floodway concept is generally not applicable for bodies of water with significant impoundment effects. Because Lake Gardner and the large swampy area between West Whitehall Road and Tuxbury Pond in northwest Amesbury do provide storage, no floodway was computed along those portions of the Powwow River.

A floodway was not computed for North Beverly Drainage Ditch since the concept of a floodway does not apply to this drainage area. The flooding in this area is the result of ponding behind an insufficiently-sized culvert under the Boston and Maine Railroad tracks. The ponding upstream of the culvert reaches a 1-percent-annual-chance recurrence elevation of 19 feet regardless of any floodway that may or may not be present. Since water exits only through the culvert and not over the railroad tracks, it is inappropriate to incorporate a floodway there.

Portions of the floodway widths along the Merrimack River, Little River, Creek Brook, Millvale Reservoir Brook and Riverside Airport Brook, Shawsheen River, Spicket Brook are contained within the channel banks.

No floodway has been computed for Pillings Pond, because it is not appropriate to delineate a floodway for an impoundment area.

Because of the general hydrological makeup of Beaverdam Brook in the vicinity of the Chestnut Street crossing and Bates Brook in the vicinity of the Boston and Maine Railroad embankment crossing, encroachments could theoretically be allowed up to the existing channel banks without increasing the 1-percent-annual-chance elevation more than 1.0 foot. However, caution should be used in adopting this concept, because the loss of extensive overbank storage could possibly result in hazardous velocity conditions along portions of these brooks.

Floodways were computed on the Saugus River on Beaverdam Brook from its confluence with the Saugus River to the limit of detailed study, and on Bates Brook from Bourque Road to the limit of detailed study. Extreme caution should be exercised in allowing encroachments on the Saugus River above the Saugus River Dam, and on Beaverdam Brook for the following reasons. Loss of natural valley storage will mean a loss of attenuation.

Depending on the amount of encroachment permitted, there exists a possibility of a significant increase in flood discharges, due to this loss of attenuation, over those values used in computations in this report. Also, it should be noted that the City of Lynn uses the portion of the Saugus River above the dam as a part of their water supply system. Taking water from the Saugus River for this purpose was authorized by Chapter 256, Acts of 1883, and by Chapter 400, Acts of 1893, enacted by the legislature of the Commonwealth of Massachusetts. Because no survey information was available, that portion of the floodway on Beaverdam Brook between its confluence with the Saugus River and Main Street was based on the floodway computed for this brook above Chestnut Street.

Establishment of floodways on the Spicket River, Harris Brook, Bare Meadow Brook to its confluence with Hawkes Brook and to a point 3,750 feet above its confluence with Bare Meadow Brook was accomplished using Types 2, 5, and 6 of the HEC-2 program (Reference 80). Type 5 reduces conveyance equally on both sides of a stream until a 1.0-foot rise in water-surface is indicated, and Type 6 encroaches by equal conveyance until a 1.0-foot rise in the energy grade line is indicated. Although Types 5 and 6 were used for guidance, because of the small size and tortuous flow paths of many of the streams, Type 2 was utilized in an attempt to arrive at practical floodways with reasonably uniform widths. For Type 2, the left and right encroachment stations are made equidistant from one center line of the channel.

Establishment of floodways on the Merrimack River, Bare Meadow Brook from its confluence with Hawkes Brook to Hills Pond, Hawkes Brook from a point 3,750 feet above the confluence with Bare Meadow Brook to North Street, Bartlett Brook and Peat Meadow Brook was accomplished using Types 1 and 6 of the HEC-2 program. Type 6 was first used for guidance. Type 1 provides for the setting of floodway widths at each cross section and thus is made to delineate a smooth floodway.

In the downtown area of Peabody, flooding problems on North River, Proctor Brook and Goldthwaite Brook are compounded by culverts with insufficient capacity. Floodways for this area were not computed due to heavy urban development in the floodplain. It would he technically inaccurate to model a floodway in this area using conventional backwater analyses, where the natural stream channel frequently is nonexistent. However, in the event of redevelopment, detailed analysis of the area should be made to prevent an increase in the base flood elevation greater than 1.0 foot.

No floodway has been computed for the Merrimack River in the Town of Salisbury. No floodway is shown for Unnamed Tributary to Fish Brook.

No floodway was run on the Artichoke River, from its confluence with the Merrimack River to Curzon Mill Dam, because this reach is a tidal estuary. Between the Curzon Mill Dam and the Lower Artichoke Reservoir Dam on the Artichoke River, no floodway was run because this reach of the river is a water impoundment area. From the Lower Artichoke Reservoir Dam to Pikes Bridge Road, which includes the lower reach of North Tributary Brook, no floodway was run because this reach of the Artichoke River is a water supply reservoir. The wide floodway between cross sections C and D of North Tributary Brook is due to the influence of the meandering stream.

Protection against the filling of flood storage areas is possible under the Massachusetts Wetlands Protection Act, (General Laws Chapter 131, Section 40). This act controls, but does not ban development on wetlands. Wetlands are defined here, for the purpose of brevity, as inland wetlands, marshes, swamps bordering on rivers, streams, and ponds--most any land which is periodically wet. The law requires that any person or governmental agency intending to remove, fill, dredge, or alter a wetland must ensure, by following various procedural and technical steps, that the activity will have no adverse effect on water supplies, storm and flood prevention, pollution prevention, or fisheries. In effect, the owner must develop his wetlands in accordance with the public's interest and safety.

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 base flood more than 1 foot at any point. Typical relationships between the floodway and the floodway fringe and their significance to floodplain development are shown in Figure 3, “Floodway Schematic”.

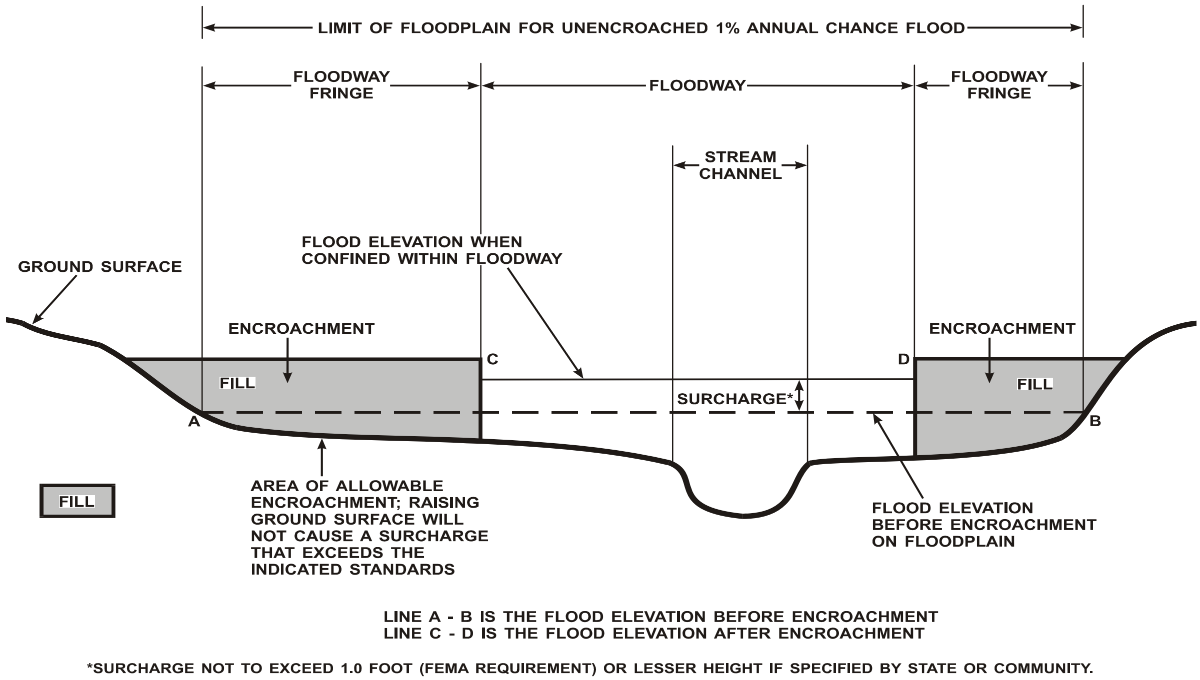


Figure 3. Floodway Schematic

One aspect of floodway and floodplain encroachment is sometimes overlooked and more often neglected: the cumulative effect of encroachment on flood discharge magnitude. Generally, as encroachment occurs, temporary storage areas are lost, velocities increase, and the magnitude of the discharge increases. As floodwaters move downstream, that increase can become more significant. The combined effect of a narrower floodplain and greater discharge can, due to hydraulic effects alone, produce a flood stage that exceeds the anticipated 1-percent-annual-chance flood.

FEMA does not encourage the filling in of the floodway fringe area. Local officials should be aware that even a 1-foot rise in the water surface elevation can cause flooding in areas which would have received little or no flooding if such filling had not taken place. Careful consideration of the economic and human dislocation which will be caused by a rise in flood heights should be made before filling is allowed. Large quantities of fill in the fringe area could also disrupt the floodplain ecosystem, causing a major impact on local environmental resources.

Communities are encouraged by FEMA to adopt wider, more restrictive floodways and to minimize the amount of fill allowed in the fringe areas. Such actions also meet the intent of the Massachusetts Wetlands Protection Act (Massachusetts General Law, Chapter 131, Section 40). Under the provisions of the act, the local conservation commission and the Massachusetts Department of Environmental Quality Engineering have the authority to impose "orders of condition" regulating floodplain areas subject to flooding and wetland alterations. The orders normally require compensatory storage to replace any loss resulting from proposed floodplain alterations. "Compensatory storage" is the volume of floodplain storage which must be created for floodwater retention equaling the storage removed by alteration. Such requirements in floodplain areas are designed to minimize adverse effects on floodplain hydrology.

In order to achieve a unified floodplain and wetlands management program, numerous Massachusetts communities have adopted local zoning by-laws, ordinances, subdivision regulations, and local Board of Health regulations augmenting the minimum requirements of the NFIP and the Wetlands Protection Act. FEMA encourages the use of this FIS as the technical basis for adoption of a broader, more encompassing local floodplain management program than is required to meet the minimum standards of the NFIP.