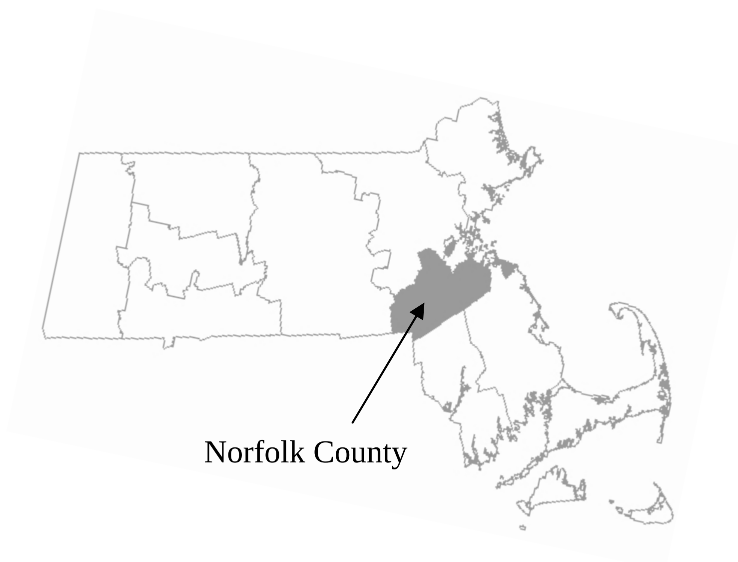
FLOOD INSURANCE STUDY

NORFOLK COUNTY, MASSACHUSETTS
(ALL JURISDICTIONS)

Volume 1 of 4



COMMUNITY NAME	COMMUNITY NUMBER
AVON, TOWN OF	250231
BELLINGHAM, TOWN OF	250232
BRAINTREE, TOWN OF	250233
BROOKLINE, TOWN OF	250234
CANTON, TOWN OF*	250235
COHASSET, TOWN OF	250236
DEDHAM, TOWN OF	250237
DOVER, TOWN OF	250238
FOXBOROUGH, TOWN OF	250239
FRANKLIN, TOWN OF	250240
HOLBROOK, TOWN OF	255212
MEDFIELD, TOWN OF	250242
MEDWAY, TOWN OF	250243
MILLIS, TOWN OF	250244
MILTON, TOWN OF	250245
NEEDHAM, TOWN OF	255215
NORFOLK, TOWN OF	255217
NORWOOD, TOWN OF	250248
PLAINVILLE, TOWN OF	250249
QUINCY, CITY OF	255219
RANDOLPH, TOWN OF	250251
SHARON, TOWN OF	250252
STOUGHTON, TOWN OF	250253
WALPOLE, TOWN OF	250254
WELLESLEY, TOWN OF	250255
WESTWOOD, TOWN OF	255225
WEYMOUTH, TOWN OF	250257
WRENTHAM, TOWN OF	250258

*The Town of Canton is not included in the partial countywide Flood Insurance Study; please refer to the separately published FIS, FIRM and FBFM

PRELIMINARY
OCTOBER 15, 2012

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
25021CV001B

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.

The Federal Emergency Management Agency (FEMA) may revise and republish part or all of this Preliminary FIS report at any time. In addition, FEMA may revise part of this FIS report by the Letter of Map Revision (LOMR) process, which does not involve republication or redistribution of the FIS report. Therefore, users should consult community officials and check the Community Map Repository to obtain the most current FIS components.

Initial Countywide FIS Effective Date: July 17, 2012

Revised Countywide FIS Date:

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Exhibit 1 – Flood Profiles – continued

FLOOD INSURANCE STUDY
NORFOLK COUNTY, MASSACHUSETTS (ALL JURISDICTIONS)

1.0 INTRODUCTION

1.1 Purpose of Study

This Partial Countywide Flood Insurance Study (FIS) revises and updates information on the existence and severity of flood hazards in the geographic area of Norfolk County, Massachusetts including the City of Quincy and the Towns of Avon, Bellingham, Braintree, Brookline, Cohasset, Dedham, Dover, Foxborough, Franklin, Holbrook, Medfield, Medway, Millis, Milton, Needham, Norfolk, Norwood, Plainville, Randolph, Sharon, Stoughton, Walpole, Wellesley, Westwood, Weymouth and Wrentham (referred to collectively herein as Norfolk 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.

Due to levee de-accreditation status at the time this FIS was finalized, the Town of Canton was not included within this Partial Countywide Study. Data related to the Town of Canton remains in this FIS report for informational purposes only, and users should refer to the separately published FIS report and FIRM for effective data.

In some States or communities, floodplain management criteria or regulations may exist that are more restrictive or comprehensive than the minimum Federal requirements. In such cases, the more restrictive criteria take precedence, and the 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.

The July 17, 2012 FIS (Reference 1) was prepared to incorporate all the communities listed above within Norfolk County in a partial countywide format. Information on the authority and acknowledgements for each jurisdiction included in the July 17, 2012 partial countywide FIS, as compiled from their previously printed FIS reports, is shown below:

Avon, Town of	In the original November 1979 study, the hydrologic and hydraulic analyses were prepared by Sverdrup & Parcel and Associates, Inc., for the Federal Insurance Administration (FIA), under Contract No. H-4037. That work was completed in March 1978. In the 1995 revision, the delineation of flood hazards for the swamp area along Mary Lee Brook was prepared by Dewberry & Davis, Inc.

Avon, Town of - continued	under agreement with the Federal Emergency Management Agency (FEMA). That work was completed in January 1994 (Reference 2).
Bellingham, Town of	The hydrologic and hydraulic analyses for the June 15, 1982 study were prepared by Vollmer Associates, Inc., for FEMA, under Contract No. H-4792. That work was completed in July 1980 (Reference 3).
Braintree, Town of	For the original June 1, 1978, FIRM and December 1977 FIS (hereinafter referred to as the 1978 FIS), the hydrologic and hydraulic analyses were performed by the U.S. Army Corps of Engineers (USACE) for the FIA under Inter-Agency Agreement No. IAA-H-19-74, Project Order No. 15. That work was completed in July 1977. For the revised November 19, 1986, FIS report and November 19, 1986, FIRM (hereinafter referred to as the 1986 FIS), the hydrologic and hydraulic analyses represented a revision of the original analyses prepared by the USACE for FEMA, under Inter-Agency Agreement No. IAA-H-19-74, Project Order No. 15. The updated November 1986 version was prepared by PRC Harris, Inc., for FEMA, under Contract Modification No. M010. That work was completed in January 1984. For the December 20, 2000 revision, the hydraulic analysis for the Cochato River was prepared by Green International Affiliates, Inc., for FEMA under Contract No. EMB-96-CO-0403 (Task No.8). That work was completed in July 1998 (Reference 4).
Canton, Town of	The hydrologic and hydraulic analyses in this study represent a revision of the original analyses prepared by Camp, Dresser and McKee, Inc., (CDM) for FEMA, under Contract No. H-3861. That work for the original study was completed in December 1976. The updated hydrologic and hydraulic analyses were prepared by CDM for FEMA, under Contract No. EMW-84-C-1601. That work was completed in February 1986 (Reference 5).
Cohasset, Town of	The hydrologic and hydraulic analyses for the June 4, 1987, study were prepared by PRC Harris for FEMA, under Contract No. H-4776. That work was completed in August 1983 (Reference 6).

Dedham, Town of	The hydrologic and hydraulic analyses for the September 29, 1988, study were performed by CDM, Environmental Engineers, for the FIA under Contract No. H-3861. That work, completed in June 1977, covered all significant flooding sources in the Town of Dedham. All field survey data for this study were collected and compiled by Harry R. Feldman, lnc., Civil Engineers and Land Surveyors, Boston, Massachusetts, under subcontract to CDM (Reference 7).
Dover, Town of	The June 1978 hydrologic and hydraulic analyses were performed by C. E. Maguire, Inc., for FEMA, under Contract No. H-2543. That work was completed June 1978. The updated version was prepared by CDM, for FEMA, under Contract No. EMW-84-C-1601. That work was completed in June 1985 (Reference 8).
Foxborough, Town of	The hydrologic and hydraulic analyses for the June 1979 study were performed by Sverdrup & Parcel and Associates, Inc., for the FIA, under Contract No. H-4037. That work completed in March 1978, covered all significant flooding sources affecting the Town of Foxborough (Reference 9).
Franklin, Town of	The hydrologic and hydraulic analyses for the August 17, 1981 study were prepared by Vollmer Associates, Inc., for FEMA, under Contract No. H-4792. That work was completed in February 1980 (Reference 10).
Holbrook, Town of	For the original July 15, 1988, FIS report, the hydrologic and hydraulic analyses represented a revision of the original analyses prepared by the U.S. Soil Conservation Service (USDA NRCS, formerly the SCS) for FEMA, under Inter-Agency Agreement No. IAA-H-9-71. The hydraulic analyses for the 1988 FIS were prepared by Schoenfeld Associates, Inc., for FEMA, under Contract No. EMW-C-0280. These analyses were limited to determining the floodways for the following previously studied streams: Cochato River/Lake Holbrook/Trout Brook, Tributary C2B, Tributary R2, Tributary R3, Tributary R4, and Beaver Brook. Floodways were not prepared for the low marsh areas of Tumbling Brook/Tumbling Brook Tributary and

Holbrook, Town of - continued	Great Pond Tributary since they were deemed unnecessary. That work was completed in May 1985. In the July 5, 2001 revision, the hydraulic analysis for the Cochato River was prepared by Green International Affiliates, Inc., for FEMA, under Contract No. EMB-96-CO-0403 (Task #9). That work was completed in July 1998 (Reference 11).
Medfield, Town of	The hydrologic and hydraulic analyses for the January 1979 study were performed by CDM for the FIA, under Contract No. H-3861. That work completed in January 1978, covered all significant flooding sources affecting the Town of Medfield (Reference 12).
Medway, Town of	The hydrologic and hydraulic analyses for the December 1979 study were prepared by C-E Maguire, Inc., for the FIA, under Contract No. H-4523. That work completed in November 1978, covered all significant flooding sources affecting the Town of Medway (Reference 13).
Millis, Town of	The hydrologic and hydraulic analyses for the February 5, 1985, study were performed by C-E Maguire, Inc. for the Department of Housing and Urban Development under Contract No. H-4523. That work was completed in August 1978 and resulted in the publication of the Millis FIS (Reference 13). The hydrologic and hydraulic analyses for the determination and delineation of the Bogastow Brook floodplain were performed by Schoenfeld Associates, Inc. for FEMA, under Contract No. EMW-C-0280. That work was completed in January 1983, and covered all flooding sources affecting the Town of Millis (Reference 14).
Milton, Town of	The hydrologic and hydraulic analyses for the March 1977 study were performed by CDM for the FIA, under Contract No. H-3861. That work, which was completed in October 1976, covered all significant flooding sources affecting the Town of Milton (Reference 15).
Needham, Town of	The hydrologic and hydraulic analyses in the June 5, 1989, study represent a revision of the original analyses prepared by the USACE for FEMA. That work was completed in November 1972 (Reference 15). The hydrologic and hydraulic analyses for the updated study were

Needham, Town of - continued	prepared by CDM for FEMA, under Contract No. EMW-86-C-2250. That work was completed in September 1987 (Reference 16).
Norfolk, Town of	The hydrologic and hydraulic analyses for the February 19, 1985, study were originally performed by the USDA NRCS for the Department of Housing and Urban Development, under Interagency Agreement IAA-H-9-71. That work resulted in the publication of the Norfolk FIS (Reference 17). The hydrologic and hydraulic analyses for Cress Brook, Myrtle Street Lateral and Harlow Pond Lateral were performed by Schoenfeld Associates, Inc. for FEMA, under Contract No. EMW-C-0280. That work completed in November 1982, covered all flooding sources affecting the Town of Norfolk (Reference 17).
Norwood, Town of	The hydrologic and hydraulic analyses for the June 1979 study were performed by Harris-Toups Associates for the FIA under Contract No. H-4024. That work completed in July 1977, covered all significant flooding sources affecting the Town of Norwood (Reference 18).
Plainville, Town of	The hydrologic and hydraulic analyses for the January 2, 1981 study were prepared by the United States Geological Survey (USGS) for the FIA, under Inter-Agency Agreement No. IAA-H-9-77, Project Order No.8, Amendment Nos. 2 and 3. That work was completed in January 1979 (Reference 19).
Quincy, City of	For the original December 4, 1985, FIS report, the hydrologic and hydraulic analyses were prepared by PRC Harris, Inc., for FEMA, under Contract No. H-4776 and under Contract Modification No. M010. That work was completed in July 1983. The hydrologic and hydraulic data for Furnace Brook, Town Brook, and Cunningham Brook were furnished by the USACE. For the May 16, 2006 revision, the floodplain boundaries were re-delineated by Applied Geographics, Inc., under contract to the City of Quincy. This work was completed in May 2000 (Reference 20).

Randolph, Town of	For the November 1977 FIS report and May 1, 1978, FIRM (hereinafter referred to as the 1978 FIS), the hydrologic and hydraulic analyses were done by Anderson-Nichols & Co., Inc., for the FIA, under Contract No. H-3707. That work was completed in June 1974. For the June 4, 1987, FIS, the hydrologic and hydraulic analyses were prepared by CDM for FEMA, under Contract No. EMW-84-R-1601. That work was completed in November 1985. For the August 2, 2000 revision, the hydraulic analysis for the Cochato River was prepared by Green International Affiliates, Inc., for FEMA under Contract No. EMB-96-CO-0403 (Task #8). That work was completed in July 1998 (Reference 21).
Sharon, Town of	The hydrologic and hydraulic analyses for the March 1978 study were performed by Harris-Toups Associates for the FIA, under Contract No. H-4024. That work completed in June 1977, covered all significant flooding sources affecting the Town of Sharon (Reference 22).
Stoughton, Town of	The hydrologic and hydraulic analyses for the December 1, 1981 study were prepared by Sverdrup & Parcel and Associates for FEMA, under Contract No. H-4037. That work completed in October 1978, covered all significant flooding sources in the Town of Stoughton (Reference 23).
Walpole, Town of	The hydrologic and hydraulic analyses for the November 18, 1988 study represent a revision of the original analyses prepared by the USDA NRCS of the U.S. Department of Agriculture for FEMA, under Inter-Agency Agreement No. IAA-H-18-75, Project Order No.3. That work was completed in December 1975. The hydrologic and hydraulic analyses for the updated study were prepared by Schoenfeld Associates, Inc., for FEMA, under Contract No. EMW-C-0280. That work was completed in May 1985 (Reference 24).
Wellesley, Town of	The hydrologic and hydraulic analyses for the March 1979 study were performed by CDM for the FIA, under Contract No. H-3861. That work completed in November 1977, covered all significant flooding sources affecting the Town of Wellesley (Reference 25).

Westwood, Town of	For the revised June 17, 2002 FIS, the hydrologic and hydraulic analyses for Bubbling Brook, Mill Brook, and Purgatory Brook were prepared by Hydraulic & Water Resources Engineers, Inc., (HWRE) for FEMA, under Contract No. EMB-96-CO-0406. That work was completed in December 1999. The hydrologic and hydraulic analyses for South Brook downstream of East Street were prepared by Green International Affiliates, Inc., for the Town of Westwood, Department of Public Works. That work was completed in January 2001 (Reference 26).
Weymouth, Town of	In the September 30, 1980 study, the hydrologic and hydraulic analyses were prepared by Sverdrup & Parcel and Associates for FEMA under Contract No. H-4037. In the June 5, 1989, revision, which included the effects of wave action, the hydrologic and hydraulic analyses were prepared by Dewberry & Davis for FEMA, under Contract No. EMW-85-C-2044. That work was completed in September 1987. In the August 19, 1991 revision, updated hydraulic analyses and updated topographic information for the Old Swamp River were prepared by Lamont R. Healy, Land Surveyors, and Reis Engineering, Inc., for FEMA. That work was completed in May 1990 (Reference 27).
Wrentham, Town of	The hydrologic and hydraulic analyses for the January 5, 1982 study were prepared by Vollmer Associates, Inc., for FEMA, under Contract No. H-4792. That work was completed in February 1980 (Reference 28).

For the July 17, 2012 partial countywide FIS, revised coastal analyses for the open water flooding sources in the communities of Braintree, Cohasset and Weymouth were prepared by Camp, Dresser &McKee, Inc. (CDM) for FEMA, under Contract No. EME-2003-CO-0340, and by Ocean & Coastal Consultants, Inc. (OCC) for CDM, under Contract No. 2809-999-003-CS. This study was completed in May 2009.

FIRM panels, base map information shown was provided in digital format by Massachusetts Geographic Information System (MassGIS). Ortho imagery was produced at a scale of 1:5,000. Aerial photography is dated April 2005 (Reference 29).

The coordinate system used for the production of the FIRM panels for the July 17, 2012 study was Massachusetts State Plane mainland zone (FIPSZONE2001). The horizontal datum was NAD83, GRS1980 spheroid (Reference 29).

Differences in datum, spheroid, projection or State Plane zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

The coastal wave height analysis for the partial countywide coastal revision in the City of Quincy was prepared by the Strategic Alliance for Risk Reduction (STARR) for FEMA under Contract No. HSFEHQ-09-D-0370 and completed in August 2012. This new analysis resulted in revisions to the Special Flood Hazards Areas (SFHA) within the City of Quincy and backwater effects to the Neponset River within the Town of Milton.

Base map information shown on the FIRM panels produced for this 2012 coastal revision was derived from USGS High Resolution orthophotography dated spring of 2008 and at 15 and 30 centimeter pixel resolution. The horizontal datum used was North American Datum of 1983 (NAD 83) (Reference 30).

1.3 Coordination

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

Prior to this partial countywide FIS, the dates of the initial, intermediate, and final CCO meetings held for the incorporated communities of Norfolk County are shown in Table 1, "Initial, Intermediate, and Final CCO Meetings."

TABLE 1 – INITIAL, INTERMEDIATE, AND FINAL CCO MEETINGS

Community Name	Initial CCO Date	Intermediate CCO Date	Final CCO Date
Town of Avon	May 1976	November 10, 1977	September 21, 1978
Town of Bellingham	May 25, 1978	*	October 8, 1981
Town of Braintree	May 3, 1978	September 13, 1983	June 11, 1985
Town of Brookline	*	*	*
Town of Canton	April 4, 1984	*	July 24, 1986
Town of Cohasset	March 29, 1978	October 12, 1983	January 28, 1985
Town of Dedham	September 4, 1975	*	January 11, 1978
Town of Dover	April 1984	*	April 28, 1986
Town of Foxborough	May 1976	November 7, 1977	September 28, 1978
Town of Franklin	May 24, 1978	*	December 29, 1980
Town of Holbrook	September 3, 1997	*	June 25, 1999
Town of Medfield	September 24, 1975	January 13, 1977	July 13, 1978
Town of Medway	May 12, 1977	*	May 8, 1979

*Information not available

TABLE 1 – INITIAL, INTERMEDIATE, AND FINAL CCO MEETINGS - CONTINUED

Town of Millis	August 27, 1979	*	April 30, 1984
Town of Milton	September 10, 1975	September 14, 1976	January 5, 1977
Town of Needham	January 1986	*	May 6, 1988
Town of Norfolk	August 24, 1979	*	May 7, 1984
Town of Norwood	April 13, 1976	October 1976	July 20, 1978
Town of Plainville	February 1976	*	January 21, 1980
City of Quincy	May 3, 1978	October 20, 1983	January 22, 1985
Town of Randolph	September 3, 1997	*	July 12, 1999
Town of Sharon	April 16, 1976	*	August 23, 1977
Town of Stoughton	May 1976	January 19, 1978	July 25, 1979
Town of Walpole	August 24, 1979	*	December 17, 1986
Town of Wellesley	September 4, 1975	May 18, 1977	June 20, 1978
Town of Westwood	September 29, 1995	November 13, 2000	June 25, 2001
Town of Weymouth	April 1, 1987	*	April 12, 1988
Town of Wrentham	May 23, 1978	*	December 4, 1980

*Information not available

For the July 17, 2012 Partial Countywide FIS, the initial CCO meeting was held on October 18, 2006, and attended by representatives of FEMA, Massachusetts Department of Conservation and Recreation (MADCR), OCC, CDM, and community members. The final CCO meeting held on June 24, 2009, and attended by representatives of FEMA, Massachusetts Department of Conservation and Recreation (MADCR), CDM, FEMA's Regional Service Center (RSC) and community members.

For this coastal study revision in City of Quincy, outreach meeting was held on March 30, 2011. A letter was sent to inform the community of the scope of the FIS, and to solicit pertinent local information. Work map discussion meeting was held with the City of Quincy on June 14, 2012, to discuss the initial results of the new coastal flood hazard analysis. The results of this coastal study were reviewed at the final CCO meetings held on __________________, and attended by representatives of the communities, the_______________. All problems raised at that meeting were addressed in this study.

2.0 AREA STUDIED

2.1 Scope of Study

This FIS report covers the geographic area of Norfolk County, Massachusetts, including the incorporated communities listed in Section 1.1. The Town of Canton was not included within this Partial Countywide Study. Data related to the Town of Canton remains in this FIS report for informational purposes only, and users should refer to the separately published FIS report and FIRM for effective data. The areas studied by detailed methods were selected with priority given to all known flood hazards and areas of projected development or proposed construction.

July 17, 2012 Partial Countywide FIS:

All or portions of the flooding sources listed in Table 2 were studied by detailed methods in the pre-partial countywide FISs. Limits of detailed study are indicated on the Flood Profiles (Exhibit 1) and on the DFIRM.

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS

Flooding Source Name	Description of Study Reaches
Arnolds Brook	From its confluence with the Peters River to a point approximately 480 feet upstream of Lizotte Drive in Bellingham
Beaver Brook (Town of Avon)	From Brockton Reservoir to the Avon/Stoughton corporate limits
Beaver Brook (Town of Bellingham)	From its confluence with the Charles River to Beaver Pond
Beaver Brook (Town of Holbrook)	From the Holbrook/Weymouth corporate limits to a point approximately 1,000 feet upstream of Weymouth Street
Beaver Brook (Town of Sharon)	From just downstream of Upland Road to approximately 3,400 feet upstream
Beaver Meadow Brook	From Bolivar Pond upstream to Pleasant Street in Canton
Billings Brook	From just downstream of Old Post Road to approximately 200 feet upstream of Dirt Road
Billings Brook Branch	From Dirt Road to just upstream of Wolomolopoag Street
Bogastow Brook	From confluence with Charles River to Town of Millis found next town over corporate limits
Bolivar Pond	For its entire shoreline in Canton
Brook A (Stetson Brook)	From its confluence with Glovers Brook to approximately 285 feet upstream of Allen Street

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – CONTINUED

Flooding Source Name	Description of Study Reaches
Brook B	From its confluence with Upper Reservoir to approximately 1,100 feet upstream of Vesey Road in Randolph
Brook No. 1	From Wrentham/Plainville corporate limits to its confluence with Rabbit Hill Pond
Bubbling Brook	From Willett Pond Dam to Walpole/Westwood corporate limits
Buckmaster Brook	From its confluence with Germany Brook to approximately 250 feet upstream of Arcadia Road
Bungay Brook	From its confluence with the Peters River to a point approximately 1,310 feet upstream of Wrentham Road in Bellingham
Burnt Swamp Brook	From the Massachusetts/Rhode Island State line, to a point 1,700 feet north of West Street in Wrentham
Canoe River (Town of Foxborough)	From Beaumont Road to Maple Street
Canoe River (Town of Sharon)	From 10,000 feet above East Street to 13,000 feet above East Street
Canton River	From its confluence with the Neponset River to approximately 355 feet upstream of Revere Court
Caroline Brook	From the confluence with Fuller Brook to just downstream of Forest Street
Charles River (Lower Reach)	From Concord Street in the City of Newton to Dover/Natick corporate limits
Charles River (Upper Reach)	From the Natick/Dover corporate limits to Mellen Street at the Bellingham/Milford corporate limits
Chicken Brook	From its influence with the Charles River to the Medway/Holliston corporate limits

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – CONTINUED

Flooding Source Name	Description of Study Reaches
Cobb's Brook	From its confluence with the Neponset River to a point approximately 50 feet upstream of North Street in Walpole
Cochato River/Trout River	From the confluence with the Monatiquot River to North Shore Road Dam
Cress Brook	From its confluence with the Mill River to Lake Street
Crocker Brook	From 1,700 feet upstream of Crocker Pond to a point 1,100 feet upstream of East Street
Cunningham Brook	From its confluence with Furnace Brook to a point approximately 400 feet upstream of Robertson Street in Quincy
Diamond Brook	From its confluence with the Neponset River to the upstream side of Washington Street
Dorchester Bay	Tidal flooding and wave action in Quincy
Dorchester Brook	From Atkinson Avenue to the Stoughton/Easton town line
Farm River	From the confluence with the Monatiquot River to approximately 1,700 feet upstream of West Street
Forge Pond	For its entire shoreline in the Town of Canton
Fuller Brook	From approximately 200 feet downstream of Wellesley High School fields to approximately 1,800 upstream feet of Smith Street
Furnace Brook	From its tidal limit to a point approximately 850 feet upstream of Hayden Street
Germany Brook	From its confluence with Hawes Brook to Westwood/Norwood corporate limits at Winter Street

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – CONTINUED

Flooding Source Name	Description of Study Reaches
Glovers Brook	From its confluence with Cochato River to approximately 1,000 feet upstream of Warren Street
Harlow Pond Lateral	From its confluence with the Charles River to a point 2,000 feet upstream of Phillips Pond
Hawes Brook	From its confluence with the Neponset River to Willet Pond Dam
Hawthorne Brook	From its confluence at Turnpike Lake to Cowell Street in Plainville
Herring Brook	From its confluence with Weymouth Back River to approximately 300 feet upstream of Iron Hill Street
Hingham Bay	Tidal flooding and wave action in Weymouth
Hopping Brook	From its confluence with the Charles River to a point approximately 1,400 feet upstream of Milford Street (State Route 109)
James Brook	From its confluence with Cohasset Cove to Sohier Street
Lake Holbrook	From the Holbrook/Randolph corporate limits to Spring Street
Lake Waban	For its entire shoreline with the Town of Wellesley
Lily Pond Stream	From its confluence with Lily Pond, to a point approximately 2,798 feet upstream
Lower Pequid Brook	From its confluence with Forge Pond to Reservoir Pond
Mann Pond Lateral	From its confluence with the Stop River to Boardman Street
Martin Brook	From its confluence with Cochato River to approximately 1,000 feet upstream of Oak Street in Randolph

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – CONTINUED

Flooding Source Name	Description of Study Reaches
Mary Lee Brook	From its confluence with Cochato River to South Main Street in Randolph
Massachusetts Bay	Tidal flooding and wave action in Cohasset
Massapoag Brook (Town of Canton)	From its outlet at Forge Pond to the Canton/Sharon corporate limits
Massapoag Brook (Town of Sharon)	From approximately 100 feet downstream of Wooden Foot Bridge to its confluence with Massapoag Lake
Massapoag Lake	For the entire area in Sharon
Meadow Brook	From Pleasant Street to its mouth at the Neponset River
Mill Brook	For its entire length within Westwood
Mill River (Town of Norfolk)	From its confluence with Charles River to Norfolk/Wrentham corporate limits
Mill River (Town of Weymouth)	From approximately 750 upstream of Mill Street to Hollis Street
Mill River Tributary A	From its confluence with the Mill River to a driveway approximately 550 feet upstream of Main Street
Mill River Tributary B	From its confluence with Mill River Tributary A to the Railroad tracks
Miller Brook	From its confluence with the Mill River to the Franklin/Norfolk corporate limits
Mine Brook (Town of Franklin)	From its confluence with the Charles River to a point approximately 200 feet upstream of Washington street
Mine Brook (Town of Walpole)	From its confluence with the Neponset River to the Walpole/Medfield corporate limits
Monatiquot River	From Quincy Avenue to the confluence with the Farm and Cochato Rivers

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – CONTINUED

Flooding Source Name	Description of Study Reaches
Morses Pond	For its entire shoreline with the Town of Wellesley
Mother Brook	From the Dedham-Boston corporate limits upstream to the diversion point with the Charles River
Myrtle Street Lateral	From its confluence with the Charles River to a point 3,000 feet upstream of Myrtle Street
Neponset River	From Boston/Milton corporate limits to Walpole/Foxborough corporate limits
Norraway Brook	From confluence with Upper Reservoir to approximately 285 feet upstream of Warren Street
Old Swamp River	From approximately 80 feet downstream of Libbey Industrial Parkway to 2,750 feet upstream of Ralph Talbot Street
Paintshop Pond	For its entire shoreline with the Town of Wellesley
Pequid Brook (Lower Reach)	From its confluence with Forge Pond to Pleasant Street
Pequid Brook (Upper Reach)	From its confluence with Reservoir Pond to Unnamed Bridge
Peters River	From Bellingham/Woonsocket corporate limits to Silver Lake
Pickerel Brook	From its confluence with Traphole Brook to a point approximately 1,800 feet upstream of Wolcott Avenue
Pine Tree Brook	From its confluence with Neponset River to approximately 2,000 feet upstream of I-95 Pope's point Dam
Plantingfield Brook	From Interstate Highway 95 (I-95) to the Westwood corporate limit

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – CONTINUED

Flooding Source Name	Description of Study Reaches
Ponkapoag Brook	From its confluence with the Neponset River upstream to Turnpike Street
Prison Farm Lateral	From its confluence with the Stop River to Spring Street
Purgatory Brook	From just downstream of U.S. Route 1 to approximately 6,500 feet upstream of Gay Street
Quincy Bay	Tidal flooding and wave action in Quincy
Rabbit Hill Brook	From the Wrentham/Plainville town boundary to Crocker Pond
Rattlesnake Run	From its confluence with Straits Pond to a point approximately 528 feet upstream
Redwing Brook	From just north of Pine Street to approximately 1,000 feet upstream of Pine Street
Reservoir Pond	For its entire shoreline within the Town of Canton
Richardsons Brook	From its confluence with Little Harbor to a point approximately 1,160 feet upstream
Robinson Brook	From Easton/Foxborough corporate limits to Central Street
	From approximately 200 feet downstream of Cocasset Street to 1,800 feet downstream of Cocasset Street, in Foxborough
Rock Meadow Brook	From County Club Road to approximately 1,600 feet upstream of Hartford Street
Rocky Brook	From its confluence with Trout Brook to just upstream of an abandoned railroad
Rumford River	From Vandys Pond south to the Foxborough corporate limits

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – CONTINUED

Flooding Source Name	Description of Study Reaches
School Meadow Brook	From its confluence with the Neponset River to a point approximately 350 feet upstream of U.S. Route 1
Shepards Brook	From its confluence with the Charles River to a point approximately 1,400 feet south of Partridge street
South Brook	From the confluence with Purgatory Brook to the downstream side of East Street
Steep Hill Brook	From just upstream of Brittons Pond to the Stoughton/Canton town line
Stony Brook	From its confluence with the Stop River to the Norfolk/Wrentham corporate limits
Stop River	From Walpole/Norfolk corporate limits to Norfolk/Wrentham corporate limits
Straits Pond	Tidal flooding and wave action in Cohasset
Sucker Brook	From the confluence of Massapoag Lake to approximately 2,100 feet upstream
The Gulf	Tidal flooding and wave action in Cohasset
Ten Mile River	From the North Attleborough corporate limits to Fuller Dam
Town Brook	From Elm Street to approximately 400 feet upstream of Wood Road
Town River Bay	Tidal flooding and wave action in Quincy
Traphole Brook	From Summer Street in Norwood to a point approximately 75 feet upstream of the U.S. Route 1 culvert
Tributary C2	From its confluence with the Cochato River to a point approximately 400 feet upstream of Kleen Way

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – CONTINUED

Flooding Source Name	Description of Study Reaches
Tributary C2B	From its confluence with Tributary C2 to a point approximately 250 feet upstream of Woodlawn Road
Tributary R1	From its confluence with Trout Brook to State Route 37 (South Franklin Street)
Tributary R2	From its confluence with Trout Brook to a point approximately 520 feet upstream of Reeds Lane
Tributary R3	From its confluence with Trout Brook to approximately 100 feet upstream of State Route 37 (South Franklin Street)
Tributary R4	From its confluence with Trout Brook to approximately 150 feet upstream of State Route 37 (South Franklin Street)
Tributary to Great Black Swamp	From the Millis/Medway corporate limits at the Great Black Swamp to a point 2,000 feet west of Saint Joseph's Cemetery on Oakland Street
Tributary to Steep Hill Brook	From Town Pond to its confluence with Steep Hill Brook
Trout Brook (Town of Avon)	From the Avon/Brockton corporate limits to Ladge Drive
Trout Brook (Town of Dover)	From its confluence with the Charles River to approximately 1,500 feet upstream of Access Road
Tumbling Brook	From the Holbrook/Randolph corporate limits to a point opposite Roberts Avenue
Tumbling Brook Tributary	From the Holbrook/Randolph corporate limits to a point opposite Roberts Avenue
Turkey Hill Run	Entire length within the Cohasset corporate limits
Turtle Brook	From Mirimichi Street Dam to its confluence with Hawthorne Brook

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS – CONTINUED

Flooding Source Name	Description of Study Reaches
Unnamed Tributary to Mary Lee Brook	From its confluence with Mary Lee Brook to just upstream of Union Street
Unnamed Tributary to Robinson Brook	From its confluence with Robinson Brook to approximately 1,720 feet upstream
Upper Pequid Brook	From Reservoir Pond to the unnamed bridge 1,050 feet upstream of Turnpike Street
Vine Brook	From its confluence with Charles River to just upstream of Industrial Culvert/Private Drive
Waban Brook	From its confluence with Charles River to Morses Pond Dam
Walnut Hill Stream	From its confluence with The Gulf to the manmade pond upstream of Beechwood Street
West Mill Brook	Charles River to Medfield Junction
Weymouth Back River	Tidal flooding and wave action in Quincy
Weymouth Fore River	Tidal flooding and wave action in Quincy
Whiting Pond Bypass	From North Attleborough/Plainville corporate limits to confluence with Ten Mile River

As part of the July 17, 2012 partial countywide update, revised coastal analyses were performed for the open water flooding sources in the communities of Braintree, Cohasset and Weymouth.

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 Norfolk County.

All or portions of the flooding sources listed in Table 3 were studied by approximate methods in the pre-partial countywide FISs.

TABLE 3 – FLOODING SOURCES STUDIED BY APPROXIMATE METHODS

Flooding Source Name	Community(ies)
Ames Long Pond	Stoughton
Areas of Shallow Flooding (James Street area and Beth Road near Acorn Terrace)	Franklin
Arnolds Brook	Bellingham
Beach Run	Quincy
Bear Swamp	Randolph
Beaver Brook	Sharon, Stoughton, Weymouth
Beaver Meadow Brook	Canton
Beaver Pond	Bellingham
Billings Brook	Sharon
Blue Hill River	Quincy, Randolph
Bound Brook	Cohasset
Brass Kettle Brook	Cohasset
Brockton Reservoir	Avon
Brook E	Medfield
Brook F	Medfield
Brook G	Medfield
Brook I	Medfield
Brook J	Medfield
Brook No. 2	Plainville
Brook No. 3	Plainville
Brook No. 4	Plainville
Bungay Brook	Bellingham
Bungay Swamp	Wrentham
Canoe River	Foxborough, Sharon
Cedar Swamp Brook	Walpole
Charles River	Norfolk
Charles River Tributaries A through G	Dover
Cranberry Brook	Braintree, Holbrook
Cranberry Pond	Weymouth
Cress Brook	Norfolk
Crocker Pond	Wrentham
Crystal Lake	Bellingham
Cunningham Brook	Quincy
Curtis Pond	Bellingham
Danielson Pond	Medfield
Dix Brook	Franklin
Eagle Brook	Wrentham
Elias Pond	Weymouth
Flynns Pond	Medfield
Franklin Reservoirs	Franklin

TABLE 3 – FLOODING SOURCES STUDIED BY APPROXIMATE METHODS – CONTINUED

Flooding Source Name	Community(ies)
Fuller Brook	Wellesley
Furnace Brook	Quincy
Great Cedar Swamp	Quincy
Great Pond	Braintree
Great Pond Brook	Medfield
Hales Pond	Wrentham
Harlow Pond Lateral	Norfolk
Hawthorne Brook	Wrentham
Hayward Creek	Braintree, Quincy
Henkes Brook	Foxborough
Herring Brook	Cohasset
Hopping Brook	Medway
James Brook	Cohasset
Jenks Reservoir	Bellingham
Jewells Pond	Medfield
Kingsbury Pond	Medfield
Lake Archer	Wrentham
Lake Hiawatha	Bellingham
Lake pearl	Wrentham
Lakeview Pond	Bellingham
Lily Hole Pond	Bellingham
Lily Pond	Cohasset
Lily Pond Stream	Cohasset
Long Pond	Bellingham
Lovett Brook	Stoughton
Lowder Brook	Dedham
Mann Pond Lateral	Norfolk
Mary Lee Brook	Avon, Randolph
Mary Lee Brook (swamp area along)	Avon
Massapoag Brook	Sharon
Massapoag Lake Branches	Sharon
Mill Brook	Dover, Medfield
Mill Brook Tributary A	Dover
Mill Pond	Wrentham
Mill River	Norfolk, Weymouth
Miller Brook	Franklin
Miller Brook	Norfolk
Mine Brook	Medfield
Miscoe Brook	Franklin
Miscoe Lake	Wrentham
Miscoe Swamp Brook	Wrentham
Myrtle Street Lateral	Norfolk
Nantasket Brook	Medfield
Neponset River	Foxborough
Noanet Brook	Dover
Noanet Brook Tributary A	Dover
Noanett Pond	Westwood

TABLE 3 – FLOODING SOURCES STUDIED BY APPROXIMATE METHODS – CONTINUED

Flooding Source Name	Community(ies)
North Brook	Medfield
Numerous Unnamed Lakes and Ponds	Foxborough
Numerous Unnamed Tributaries and Swamp Areas	Franklin, Medway, Sharon
Numerous Unnamed Ponds, Swamps and Streams	Wrentham
Numerous Unnamed streams	Countywide
Old Mill Brook	Plainville
Old Swamp River	Weymouth
Pecunit Brook	Canton
Pine Tree Brook Reservoir	Milton
Pinewood Pond	Stoughton
Plantingfield Brook	Norwood
Plymouth River	Weymouth
Ponkapoag Pond	Randolph
Powissett Brook	Dover, Westwood
Prison Farm Lateral	Norfolk
Purgatory Brook	Norwood
Rattlesnake Run	Cohasset
Rays Pond	Franklin
Richardsons Brook	Cohasset
Robinson Brook	Foxborough
Rocky Brook	Dover
Rosemary Brook	Needham
Rumford River	Foxborough
Sabina Lake	Wellesley
Sanctuary Pond	Cohasset
Sawmill Brook	Plainville
School Meadow Brook	Walpole
Sewell Brook	Medfield
Shepards Brook	Franklin
Spring Brook	Walpole
Spruce Pond	Franklin
Stall Brook	Bellingham, Medway
Stony Brook	Norfolk, Wrentham
Stop River	Norfolk
Storrow Pond	Westwood
Sucker Brook	Sharon
Sunset Lake	Braintree
Three Swamp Brook	Avon
Town Brook	Braintree
Traphole Brook	Norwood, Walpole
Tributary A to Beaver Brook	Avon
Tributary C2	Holbrook
Tributary C2B	Holbrook
Tributary R1	Holbrook
Tributary R2	Holbrook
Tributary R4	Holbrook
Tributary to Great Black Swamp	Medway

TABLE 3 – FLOODING SOURCES STUDIED BY APPROXIMATE METHODS – CONTINUED

Flooding Source Name	Community(ies)
Tributary to Norroway Brook	Randolph
Trout Brook	Avon, Dover
Trout Brook Tributaries A and B	Dover
Tuburek Brook	Medfield
Tubwreck Brook	Dover
Tumbling Brook	Randolph
Turtle Brook	Medfield
Uncas Brook	Franklin, Wrentham
Uncas Pond	Franklin
Unnamed brook located approximately 0.25 mile northwest of Main Street	Walpole
Unnamed Tributary	Westwood
Upper Pequid Brook	Canton
Upper Reservoir	Braintree, Randolph
Vine Brook	Medfield
Wading River	Foxborough
Waldo Lake	Avon
Walnut Hill Stream	Cohasset
West Mill Brook	Medfield
Weymouth Great Pond	Weymouth
Whitman Brook	Stoughton
Whitmans Pond	Weymouth
Whortleberry Pond	Weymouth

Detailed study streams that were not re-studied as part of the July 17, 2012 revision may include a profile baseline on the FIRM. The profile baselines 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 the transferred profile baseline may deviate significantly from the channel or may be outside of the floodplain.

The July 17, 2012 FIS also incorporated the determinations of letters issued by FEMA resulting in map changes Letter of Map Revision (LOMR), as shown in Table 4.

TABLE 4 – LETTERS OF MAP CHANGE

Community	Case Number	Flooding Source	Letter Date
Foxborough, Town of	04-01-041P	Robinson Brook Unnamed Tributary to Robinson Brook	1/12/2005
Needham, Town of	96-01-043P		
Stoughton, Town of	10-01-1148P	Steep Hill Brook	10/1/2010
Rose Mary Brook			
Stoughton, Town of	1-91-42	Unnamed Stream	11/20/1991
Wellesley, Town of	08-01-0508X	Fuller Brook/ Caroline Brook	7/11/2008
Westwood, Town of	07-01-0169P	Mill Brook	3/30/2007

For the July 17, 2012 partial countywide FIS, revised coastal analyses for the open water flooding sources in the communities of Braintree, Cohasset and Weymouth were prepared by CDM for FEMA.

2012 Coastal Study Update for City of Quincy

The coastal wave height analysis for this countywide coastal study was prepared by STARR. This new analysis resulted in revisions to the FIRM for the City of Quincy. Additionally, portions of Neponset River within the Town of Milton were revised based on the coastal backwater effect.

2.2 Community Description

Norfolk County is located to the South and West of Boston, Massachusetts. In Norfolk County, there are twenty-seven (27) towns and one (1) city. The Towns of Brookline, Dedham, Dover, Medfield, Millis, Milton, Needham and Wellesley are located in northern Norfolk County. The Towns of Canton, Norfolk, Norwood, Randolph, Walpole and Westwood are in the central portion of the county. The Towns of Avon, Foxborough, Holbrook, Plainville, Sharon and Stoughton are located in the southern portion of the county. The Towns of Bellingham, Franklin, Medway and Wrentham are located in the western portion of the county. The Towns of Braintree, Cohasset and Weymouth; and the City of Quincy are located in the eastern portion of the county.

Norfolk County is bordered on the north by Middlesex and Suffolk Counties in Massachusetts, and on the east by Plymouth County, Massachusetts and by the Atlantic Ocean. It is bordered on the west by Worcester County, Massachusetts and by Providence County, in Rhode Island. Norfolk County is bordered on the south by Bristol and Plymouth Counties in Massachusetts.

According to census records, the population of Norfolk County was 670,850 in 2010, 650,308 in 2000, and 616,087 in 1990 (Reference 31). The total land area in Norfolk County is 396 mi^2.

2.3 Principal Flood Problems

Major floods occur on streams in Norfolk County during the spring, fall, and winter seasons, although flooding incidences can occur at any time of the year. Some of the most severe flooding occurs in early spring as a result of snowmelt and heavy rains. Autumn is another critical season for floods due to heavy rainfall associated with hurricanes. Heavy thunderstorms can result in rapid runoff and flooding in the downstream portions of the smaller streams.

Flooding on the smaller streams of southern New England often results from either a combination of heavy rainfall and snowmelt or from high intensity rainfall alone. Seldom does flooding result from snowmelt alone.

Coastal towns are highly susceptible to northeasters and other coastal storms. A northeaster travels southwest to northeast along the Atlantic coast, collecting moisture over the ocean and sending it inland via northeast winds. Northeasters differ from hurricanes in that they cover a larger area, have less intense winds, and move slower causing a longer duration. Where a hurricane may last for several hours, a northeaster may last for several days. For this reason, northeasters often last long enough to be accompanied by at least one high tide, which results in the most severe coastal flooding conditions. In addition, northeasters can occur at different times of the year when there is existing snowmelt and frozen ground conditions, aggravating flooding conditions.

The flood problems for the communities within Norfolk County have been compiled from previous FISs and are described below:

Some major flood-producing storms were experienced along the coastal area of southern Massachusetts affecting the Towns of Braintree, Cohasset and Weymouth and the City of Quincy. The storm that accompanied Hurricane Carol on September 11-12, 1954, produced a total storm rainfall of 5.34 inches at Blue Hills in Milton and 5.69 inches at the National Weather Service station in Boston. The record rainfall for many areas of New England resulted from the rain accompanying Hurricane Diane in 1955. This storm produced record volumes as well as very high intensity rainfall. Storm rainfall amounts were 13.76 inches and 12.47 inches at Blue Hills and Boston, respectively. On March 17-18, 1968, a slow moving coastal storm produced high amounts of precipitation in southeastern New England. The highest amounts occurred in a triangular area formed by Boston, central Rhode Island, and the Cape Cod Canal, in which storm totals ranged from approximately 5 to 7 inches. Storm totals at Boston and Blue Hills were 5.07 and 7.53 inches, respectively.

Table 5 shows a list of maximum discharges from gages in the Braintree area and is indicative of regional streamflow.

TABLE 5 – GAGE DATA FOR THE BOSTON/BRAINTREE AREA

LOCATIONS OF GAGING STATION	DRAINAGE AREA (SQUARE MILES)	PERIOD OF RECORD	MAXIMUM DISCHARGE (cfs)	DATE
NEPONSET RIVER				
At Norwood, Massachusetts	35.20	1919-1940	1,490	August 19, 1955
EAST BRANCH NEPONSET RIVER				
At Canton, Massachusetts	27.20	1952-1974	1,790	August 19, 1955
DORCHESTER BROOK				
Near Brockton, Massachusetts	4.67	1962-1974	359	May 18, 1968

Tidal Data for the Boston/Braintree area is shown in Table 6:

TABLE 6 – TIDAL DATA FOR THE BOSTON/BRAINTREE AREA

TIDAL DATA	ELEVATION (feet NAVD[1])
NORMAL RANGE	
Mean Tide Range	8.7
Mean High Water	4.1
Mean Low Water	-5.4
High Predicted Spring Table	6.3
TIDES OF RECORD	
February 24, 1723	9.7
April 16, 1851	9.2
December 26, 1909	9.2
November 27, 1898	8.7
December 29, 1959	8.6

[1] North American Vertical Datum of 1988

In Cohasset, in addition to flooding, damaging waves may result in areas with sufficient fetch length, water depth, and exposure to winds. The outer coastline from the Cohasset-Hull town boundary to the White Head section of Cohasset Harbor is susceptible to damaging waves. Also in Cohasset, the seaward of the western side of Pleasant Beach is an ancient river channel which extends from the beach to approximately 1,500 feet offshore. The depth of water at this channel is significantly deeper than at other parts of the beach. These greater depths are a pathway for higher wave energy to reach the shore. The result is more overtopping and seepage at this part of the dune than at other sites in the system.

The City of Quincy, because of its coastal New England location, is highly susceptible to northeasters. In addition to flooding, damaging waves may result in areas with sufficient fetch length and water depth. Quincy's shoreline is subject to wave action from the northern end of the Boston Harbor Marina in Squantum to the confluence of the Weymouth Fore River. Houghs Neck is highly susceptible to flooding from northeasters, such as the storm of February 1978. The Boston Globe reported:

> Stormwaters which rose to three feet deep in some seaside living rooms and kitchens on Houghs Neck had receded yesterday, leaving two feet of gravel along Edgewater Drive and one home with a caved-in cellar. In yards on Sea Street, a dozen automobiles were strewn about having been flung as much as 60 feet from roadside by the tidal fury. Chairs and other pieces of furniture had washed up

onto Bayswater Road. Quincy Yacht Club and Harvey's Boat Landing were ripped partly from the shore (Reference 32).

Although other coastal locations in Quincy may be protected from wave attack, they are still vulnerable to inundation by storm surge. The Johns Street area of Montclair and parts of Snug Harbor and Germantown are some areas where this type of flooding has occurred in the past.

The Town of Weymouth has experienced flood damage in the past. Three areas in Weymouth have been severely flooded in the past and remain critical areas today. They are Weymouth Landing, East Weymouth, and the Derby Street area. Weymouth Landing, located at the Weymouth Fore River, was inundated by 5 feet of water in 1955. The 1955 flood was a result of hurricane Diane, which was estimated to have been between the 2- and 1.3-percent-annual-chance flood (50 to 75 years). A 96-inch culvert has since been placed on Smelt Brook to reduce backwater; however, the area still remains a critical area. In East Weymouth, serious flooding took place in 1955 due to the overflowing of Whitmans Pond. Heavy damage occurred along Picasant Street and Water Street. The Derby Street area is along the headwater of the Mill River. This is a low-lying area. An apartment complex floods during heavy rain storms as does a private company in that same area. This is a chronic flood problem. Minor flooding, mostly in the form of basement flooding, has occurred all along the low-lying areas alongside the Mill River and the Old Swamp River. At the Weymouth Back River, the sewer pumping station was also flooded in 1955. Weymouth Great Pond and Whitmans Pond, two very large bodies of water in Weymouth, have only caused flooding problems once. That was during the storm of 1955 when Whitmans Pond Dam overflowed. Tidal flooding in Weymouth including its wave action from Hingham Bay, the Weymouth Back River, and the Weymouth Fore river, is caused by the passage of hurricanes and northeasters.

Inland riverine flooding is a major concern in the Towns of Braintree, Cohasset and Quincy. In Braintree, during Hurricane Diane in 1955, the Monatiquot River flooded over Pearl Street, Hancock Street, Union Street, Adams Street, and River Street. Since then, some culverts have been enlarged and old dams removed along the Monatiquot River. Town Brook, from downstream of Old Quincy Reservoir to the Quincy corporate limits, has a history of flooding. Some areas which have experienced flooding in the past or are located in or near a flood plain include the neighborhood along Town Brook from State Route 3 to the dam for Old Quincy Reservoir, the Monatiquot River over Pearl Street, Hancock Street, Plain Street, Washington Street, River Street, and the Farm River in the vicinity of the shopping center on Granite Street. The Weymouth Fore River is subject to flooding during northeasters.

In Cohasset, during peak runoff seasons and high intensity storms, inland flooding occurs along Turkey Hill Run and in the downtown Pleasant Street area Where James Brook passes through a long culvert. In 1955, Ripley Road, Cushing Road, and Smith Place were rendered impassable due to flooding from James Brook caused by Hurricane Diane. Walnut Hill Stream from the manmade pond at Beechwood Street to the outlet at The Gulf has experienced flood problems, which are somewhat aggravated by undersized culverts. Another location of frequent flooding is the lowlands behind the gasoline station on State Route 3A near the Cohasset-Scituate town boundary. This flooding is caused by beaver dams, which are continually being built on Bound Brook.

In Quincy, during the storm of August 1955, precipitation exceeded the precipitation expected during a 1-percent-annual-chance storm for a period of more than three hours.

During this record flood, flooding extended essentially the entire length of Furnace Brook from Hancock Street to Willard Street. Town Brook caused flooding in the Bigelow Street area, in the parking area of the shopping center, along much of the length of Brook Road, and in the Center Street area near the Raytheon plant. On Cunningham Brook, flooding extended from its confluence with Furnace Brook to the vicinity of Unity Street. The flooding was also extensive on Ballou Street, and both Stedman and Robertson Streets were overtopped. Ponding occurred on Sheldon Street up to 4 feet deep (Reference 33). During this flood, floodwaters poured across Newport Avenue and down Furnace Brook Parkway, ponding to a depth of 4 feet in the underpass below the present Massachusetts Bay Transit Authority (MBTA) tracks. Upstream of this point at the Furnace Brook Parkway crossing, the waters swept across the road 200 feet wide and 12 inches deep. This water divided a portion following the brook channel and a portion flowing down Oakland Street, flooding a low area of several blocks through which the brook channel had once passed. High watermarks indicated that the roadway at Adams Street was topped for approximately 300 feet to a maximum depth of 8 inches. The floodwaters also overtopped Quarry Street, 6 inches deep and 140 feet wide. Ponded depths as much as 4.5 to 5 feet occurred between Quarry and Adams Streets and in the Cross Street, Furnace Avenue, and Copeland Street area.

The Town of Avon has experienced some extensive flood damage in the past. In August 1955, a storm, which resulted in 13 inches of rainfall in the town, caused considerable damage to cellars of houses and buildings and flooded many streets, rendering them impassable. Flooding along Trout Brook has been the main cause of concern regarding past flooding. East Main Street, West Main Street, Gill Street, and Spring Street were overtopped by flood waters from Trout Brook in 1955. During the 1955 flood, a trench had to be cut across Spring Street to reduce the high waters.

In Bellingham, numerous floods have occurred in the Crooks Corner area of Bellingham. This area is also one of the most urbanized areas of the town. Flood damage to structures in the floodplains occurred in 1936, 1938, 1955, 1968, and 1979. The 1955 flood caused by Hurricane Diane was the flood of record for the Charles River, Canton River and Neponset River. This storm was slightly less than a 1-percent-annual-chance storm (Reference 34).

In Braintree, the Monatiquot and lower Farm Rivers flow through Braintree in relatively well established channels, while upstream portions of the Farm River flow through wetlands in poorly defined channels. The Cochato River also flows through wetlands in a meandering, poorly defined channel. These flat, swampy areas provide considerable natural storage of floodwaters. Other streams in the study area tend to be well defined, and local drainage problems are found throughout the basin.

The Town of Canton has been besieged numerous times in the past with flooding problems. Major storms occurred in 1936, 1938, 1955, and 1968. Maximum flow of the Neponset River recorded at the USGS Gage 1-1050 (2.7 miles upstream of Neponset Street) was 1,490 cubic feet per second (cfs), recorded on August 19, 1955. Flow the same day at the gaging station located on the Canton River downstream from Washington Street (USGS Gage 1-1055) was 1,790 cfs. A severe rainstorm on March 18, 1968, caused extensive flooding from rivers in the Town of Canton. The Norwood gate recorded a peak flood of 1,140 cfs while the Canton River gage recorded 1,420 cfs (Reference 5). Though the rainfall total was only approximately 50 percent of the August 19, 1955 storm, this storm occurred with a high antecedent condition resulting in only a 30 percent reduction of the runoff of August 1955. Flooding problems occurred in numerous locations throughout the town, including the Washington Street Bridge over the Canton River; Bolivar, Mechanic, Rockland, and Neponset Streets; and an area near the viaduct opposite the Emerson and Cuming Plant. Homes were flooded in the Ponkapoag section, Pleasant Garden Road, and York and Revere Streets. Forge Pond, Bolivar Pond, and Shephard Pond Dams have all been overtopped in the past. In many instances, the flooding was a direct result of backwater from dams, bridges, and culverts not being capable of passing the flow.

In Cohasset, the February 1978 storm was a severe northeaster comparable to a 1-percent-annual-chance event. This storm caused inundation and damage in numerous areas, such as: the lowland area bordered by Cohasset Harbor, including parts of Lothrop Lane, Atlantic Avenue, and White Head Road; Cohasset Cove in the lowlands around Stockridge Road, including parts of Atlantic Avenue, Howard Gleason Road, Ennis Avenue, Margin Street, James Brook south of Elm Street, and the first floor of Hugo's Restaurant Wharf; south of the Border Street Bridge on The Gulf; along low sections of Atlantic Avenue bordering Pleasant Beach and Sandy Beach; Crescent Beach at the northwest tip of Straits Pond; on the Weir River and Straits Pond near Hull Street and the outlet of Turkey Hill Run; the Forest Avenue Extension Causeway at Black Rock Beach; and at the outlet to Rattlesnake Run at Jerusalem Road (Cohasset References 35 and 36).

The Town of Dedham has been besieged numerous times in the past with flooding problems. Historical records indicate that flooding may occur at any time of the year and can be the result of various complex hydrological characteristics in the watershed. Prior to 1900, there is very little data to base flood flow magnitudes on. In 1886, a flood occurred which was in all probability the largest flood up to that date. In 1818 and 1897, flooding caused problems but meager records preclude any realistic comparison. Since the turn of the century, five major floods have occurred in the Town of Dedham. In March 1936, melting snow and heavy rainfall from two major storms combined to cause extensive flooding throughout the Charles and Neponset watersheds. This storm caused the third highest elevations ever experienced on the Charles River and the second highest elevations on the Neponset River. In July 1938, torrential rains fell throughout both watersheds, resulting in the second highest elevations on the Charles River and near record setting elevations on the Neponset River. In August 1955, after two successive hurricanes within a one week span, all known flood elevations were exceeded in both the Neponset and Charles River Basins. Extensive flooding occurred throughout Dedham, particularly in the residential area of Readville Manor. The Riverdale area was a virtual island, surrounded by the slowly mounting waters of the Charles River. Many of the bridges into Riverdale were either under water or structurally unsafe because of excessive water pressure. Mother Brook suffered some flooding, but for the most part did not cause significant damage. Wigwam Brook backed up from rising waters in the Charles River, causing extensive flooding in the vicinity of High and Williams Streets. Several homes in the Readville Manor area were completely surrounded by waters from the Neponset River. Flood elevations experienced during this event were the highest to date on both the Neponset and Charles Rivers. In March 1968, record setting rains fell on the eastern portion of Massachusetts. The USGS Gaging Station No. 1-10350 (184 square mile drainage area) at Charles River Village, Needham, Massachusetts (7.8 miles upstream of the Mother Brook diversion) recorded a flow of 3,220 cubic feet per second (cfs), equaling the previous high established in August 1955. However, because of extensive channel improvements on both the Charles and Neponset Rivers, flood elevations were substantially lower than in 1955. Flooding did occur in Dedham, causing damages throughout many sections of the town. Dedham Housing for the Elderly on Bridge Street was evacuated as waters from the Charles River began to rise. The Maynard Road area caused considerable concern because of rising waters. Flooding also occurred again in the Readville Manor area, causing several families to be evacuated. Mother Brook overtopped its bank near the Mill Road area, and Lowder Brook caused considerable damage, especially in two locations: the rear of Dedham Plaza between Route 1 and Route 1A and an area near Robert and Booth Roads. These two locations were inundated mainly because of inadequate drainage facilities. Surface runoff from Route 128 caused flooding along Robert and Booth Roads. In March 1969, warm temperatures, heavy rains, and high snowmelt caused limited flooding in the town. Peak flood flows and maximum water-surface elevations for the Charles River are presented below. Peak flows are for the USGS Gaging Station No. 1-1035.0 on the Charles River at Charles River Village and for the USGS Gaging Station No. 1-1040.0 at Mother Brook. Water-surface elevations are from historical records at a site on the Charles River at the upstream side of Bridge Street, Dedham.

Peak Discharges and Maximum Elevations for Dedham, Massachusetts are shown below in Table 7.

TABLE 7 – PEAK DISCHARGES AND MAXIMUM ELEVATIONS
DEDHAM, MA

DATE	PEAK FLOWS (cfs)	PEAK FLOWS (cfs)	MAXIMUM ELEVATION (feet NAVD[1])
	CHARLES RIVER VILLAGE	MOTHER BROOK	
March 1936	3,170	900	91.2
July 1938	3,110	909	92.2
August 1955	3,220	970	92.9
March 1968	3,220	1,040	89.8
March 1969	1,930	580	*

*Data not available
[1] North American Vertical Datum of 1988

Presented below in Table 8 are historical flood elevations for the Neponset River at a point opposite the Readville Manor area of Dedham.

TABLE 8 – MAXIMUM ELEVATIONS FOR THE NEPONSET RIVER
DEDHAM, MA

DATE	MAXIMUM ELEVATION (feet NAVD[1])
March 1936	43.8
August 1955	47.0
March 1968	42.6

[1] North American Vertical Datum of 1988

The Town of Foxborough has never experienced any substantial damage due to storm flooding. The town is essentially situated on top of a hill; thus, water drains out of the town. However, Foxborough does have three chronic flooding areas. One of these is the area of the Mansfield Bleachery on Morse Street at Glue Factory Pond. As much as 15 inches of water was reported on Moore Street between the main pond and the secondary pond during the 1955 storm. This condition exists both during major storms and storms of much lesser magnitude. A second chronic flooding area is on Oak Street, approximately 0.5 mile north of its intersection with Cocasset Street. At this point, two tributaries to Vandys Pond flow approximately 150 feet apart. In the 1955 storm, both of these sections of road were washed out. They have been replaced with larger culverts. Third, Wading River overflows every spring as a result of snowmelt. Fortunately, no property damage has occurred during any major storm.

In Holbrook, major floods have occurred in the area in 1936, 1938, 1955, 1968, and 1996. Flood damage in Holbrook has not been severe due to natural flood storage available in swampy areas along the streams. Because all the streams studied originate in Holbrook, there are relatively small drainage areas contributing to flood flows. Damage has occurred mainly in the area surrounding Lake Holbrook (Reference 37). The floods in August 1955 resulted from rainfall associated with the two hurricanes that swept through western New York and New England within a one-week period. Although rainfall in eastern Massachusetts during the period of August 11-15 from Hurricane Connie was not substantial, it left the ground saturated. Over two inches of rain were recorded in Brockton. Streams and reservoirs were already higher than normal due to Hurricane Connie when Hurricane Diane passed inland on August 17 over North Carolina and Virginia, then turned eastward along the coast and deposited record-breaking precipitation over southern New England. Precipitation began early on August 18 and heavy rain fell for more than thirty hours. During this period, 11.7 inches of rain were recorded in Brockton (Reference 11). In addition to Lake Holbrook, local officials have indicated several other areas, particularly roads that are subject to frequent minor inundation. Among these areas are: Center Street, State Route 139 (Union Street), Water Street, and Mear Road on the Cochato River; Morgan Road, State Route 37 (South Franklin Street), and Reeds Lane on the Trout Brook tributaries; and State Route 139 (Abington Avenue) and Weymouth Street on Beaver Brook. For the most part, damage has been limited to basement flooding and some minor first floor inundation. The combination of small drainage areas and relatively flat floodplains resulted in shallow depths of flooding (Reference 38).

In Medfield, during large flooding events, flood stages are increased by backwater caused by bridges and dams. The Cemetery Pond Dam affects flooding on Vine Brook. Large magnitude floods occurred in the Town of Medfield in 1936, 1938, 1955, 1968, and 1969. Reliable records of flood flows have been kept since 1938 at USGS gaging station No. 0103500, at Charles River Village, Needham, on the Charles River. Floods of record on the Charles River occurred in 1955 and 1968. Both of these floods were smaller than the 2-percent-annual-chance event. The 1955 flood was caused by a series of hurricanes; the 1968 flood resulted from a combination of rain and snowmelt.

Flooding in Milton is caused by intense rainstorms in the upland areas and by hurricanes and northeasters in the tidal areas. Floods can occur at any time of the year. The Neponset River has overflowed its banks on several occasions. The flood of August 1955, which was a 1.3-percent-annual-chance (75 years) flood, is believed to be second in magnitude, exceeded only by the flood of 1886. Several minor flooding problems occurred in 1806, 1898, 1936, and 1938. In March 1968, after an intense 3-day storm, the Neponset River once again released its waters to the surrounding areas. This storm, a 2.5-percent-annual-chance (40) storm, caused extensive flooding in many business establishments in the Lower Mills area of Milton. Trolley service along the Massachusetts Bay Transportation Authority's Mattapan to Ashmont line was suspended when the tracks became flooded at Lower Mills (Reference 39). Quick action by the Metropolitan District Commission officials and employees of the Perini Corporation, a private contractor, prevented further damages by releasing three floodgates on the Lower Mills Dam. Although this raised the water level downstream of the dam by a few inches, the level upstream dropped considerably, thus averting greater damages to the already flooded area. Pine Tree Brook experienced flooding problems in 1936, 1938, 1955, 1962, and, to a lesser extent, 1968 (Reference 40). Hurricane Diane in August 1955 caused the most severe flooding problems in recent history. Over 700 homes were affected and a few streets and bridges were under water. Restrictions caused by undersized culverts created a backwater effect along the brook, causing it to overtop its banks in several locations.

Floods in Norfolk, caused by excessive rainfall, snowmelt and hurricane storms, have occurred in 1936, 1938, 1955, and 1968. The flood caused by Hurricane Diane in August 1955 was approximated to be a 1-percent-annual-chance flood. The major flood problem area within Norfolk is in the vicinity of Populatic Pond on the Charles River. The March 1968 flood on the Charles River affected several summer cottages and a few permanent homes. This flooding primarily caused damage to basements and to access roads serving the area (Reference 16). In Norfolk Center, flooding has occurred which affected access to a commercial establishment and the water levels in two existing ponds. Flooding of roads in this area was caused primarily by inadequate culverts which have since been replaced. Also, minor flooding has occurred around two or three homes just above Needham Street near the state prison. These homes are relatively new and have not sustained actual flood damage. However, water does accumulate around the lower properties during periods of high runoff or snow melt. The stream has very little gradient in this vicinity (Reference 16). Elsewhere throughout the town, flooding has been limited to roads and bridges resulting in minor amounts of damage. Most of the more frequent water problem areas are caused by inadequate drainage and spring seepage from upland areas (Reference 16).

Norwood has experienced significant flooding in the past, especially during the August 1955 hurricane and the March 1968 storm. The 1955 hurricane was approximately a 0.5 percent-annual-chance storm. The March 1968 flood was a 1.3 percent-annual-chance (75 years) storm. The August 1955 storm caused millions of dollars' worth of damage to homes, streets, and local industry in Norwood. Examples of such damage are the homes on St. John's Avenue, which were surrounded due to overflow of Hawes Brook and the

Neponset River. Heavy damage was experienced in the Ellis Garden sector when Plantingfield Brook backed up at the Upland Road culvert, flowed under the bridge at Washington Street tearing up sections of Hill Street, and flowed into the Ellis Garden development. Homes in the area of Union and Summer Streets experienced inundation when the U.S. Route 1 Bridge over Traphole Brook collapsed and blocked the stream's flow. The Dean Street culvert over Meadow Brook backed up and caused ponding to six-foot depths upstream. The Norwood Airport suffered inundation of at least half of both of its runways. The Balch School, the Winslow School, and the Junior High School were flooded to varying degrees. Damage was heavy to local industries; the Bird and Son Plants suffered shutdowns and heavy losses, the George Morrill Division of Sun Chemical was flooded by backup at the Pleasant Street Bridge, and Factory Mutual was flooded by the overflowing of the Neponset River.

Flooding in Norwood during the March 1968 storm, damage was less severe. This was due partly to flood control measures taken between the 2 storms and partly to the storm being of a lesser magnitude than the 1955 storm. Damage was nevertheless severe. Over 200 homes suffered flooded cellars or yards, such as those along the Garden Parkway near Walpole. A portion of Pleasant Street Bridge over the Neponset River collapsed. New London mills, the old tannery property on Endicott Street, basements of the stores along Washington Street, and the Norwood airport all suffered degrees of inundation by floodwaters.

In Randolph, information from residents, newspaper articles, and other sources indicates that flooding in Randolph is a common occurrence due to high urbanization, high-water table, and local drainage problems. Prior to this revision, the five largest flooding events in Randolph were in 1807, March 1936, August 1955, spring 1968, and March 1969. Estimates of the frequencies of these floods were not available. A large flood event occurred in October of 1996. Increasing flow, caused by urbanization combined with debris, has clogged many already overtaxed drainage culverts and compounded the flooding problems, especially during the recent floods. Areas flooded during the storm of 1969 were inspected by the USACE and town officials (Reference 41). The following flooding was described in their report:

- The Cochato River - the entire lower basin was inundated.

- The Glovers Brook drainage area - flooding on Regina Road was limited to backyard areas; approximately 400 feet of Pleasant Street. was impassable with water 12 inches deep; from North Main Street to Warren Street, the entire area of Curhan Chevrolet parking lot and the southern area of Fernandes parking lot flooded to a depth of 16 inches; Doherty Lumber Yard was flooded to a depth of 24 inches with lumber stacks dislodged and deposited along the flooded railroad bed; a culvert at the intersection of Warren Street and Highland Avenue overflowed because of a severely obstructed downstream conduit; Bear Swamp culvert overflowed; minor flooding occurred at Highland Glen Estates west of Warren Street; flooding on Webster Street was 16 to 18 inches deep making the street impassable for several hundred feet; the flood discharged across private property between houses #67 and #71 in a stream 10 feet wide and 8 inches deep. Mary Lee Brook drainage areas - basements flooded badly in the vicinity of Summit Road.

- Unnamed Tributary to Mary Lee Brook - flooding near Barbara Road during and after the storm to a depth of 24 inches.

- Upper Reservoir - flooded the Oak Street area because of the inadequate culverts connecting the roadway to the reservoir.

- Maple Glen Court/Skyview Road area - flooded cellars were caused by a totally obstructed drain.

- Norroway Pond area - dam located at the outlet from the pond was breached with significant discharges passing through breach.

- USGS gage (01 1050 00) on the Neponset River at Norwood and USGS gage (01 1049 00) on Mill Brook at Westwood were used for the hydrologic analysis in this study.

Also in Randolph, Glovers Brook has overflowed its banks and caused significant flooding in the business district

In Sharon, the flood of August 1955 caused quite severe damage to personal and public property. In Sharon, the 1955 storm was approximately comparable to a 1-percent-annual-chance storm in discharge with 18 inches of rainfall in the three days the storm lasted (Reference 42). Numerous streets were completely washed-out and many instances of residential flooding were reported. Damage was estimated to be upwards of $125,000. Flooded areas included parts of East, Billings, Ames, and Quincy Streets, caused by overflow of Massapoag Brook. Beach Street was overflowed by the rising waters of Lake Massapoag. Damage was done to Moose Hill Parkway by Beaver Brook and South Walpole Street was flooded over by Billings Brook. Other flooded areas were U.S. Route 1, Richards Avenue, North Main Street, portions of New York, New Haven and Hartford railroad tracks and numerous incidences of minor damage to areas throughout the town (References 42 and Reference 43). In the most recent flood, March 1968, damage was limited to flooded cellars and other similar items.

The Town of Stoughton has experienced extensive flood damage in the past. The floods caused by the hurricane of September 1954 and a very severe rainstorm in March 1955 are the most damaging on record. During the 1955 flood which was the most severe of the two, two dams failed and many roads were washed out or overtopped.

The Town of Walpole has experienced damaging floods in 1936, 1938, 1955, and 1968. Flooding caused by Hurricane Diane in August 1955 was the most severe (Reference 44). Rainfall during the August 1955 storm was a maximum of approximately 11.5 inches for a 24-hour period and a total of 15 inches for the two-day storm period (Reference 44). Significant damage occurred along the Neponset River from the 1955 storm. Floodwaters from Diamond Brook inundated Walpole Center, flooding roads, bridges, 14 houses, a school, and 42 commercial establishments (Reference 44). The 1955 flood was approximately 1-percent-annual-chance flood at the USGS gaging station No. 01105000, located on the Neponset River near the Pleasant Street Bridge in Norwood.

Flood elevations in the Town of Wellesley are increased during large flood events by backwater caused by bridge crossings and by dams. The Newton Lower Falls, Cordingly and Metropolitan Circular Dams affect flood elevations on the Lower Charles River. The Cochrane Dam in Needham and Dover affects flooding elevations on the Upper Charles River. The maximum flood of record occurred in 1968. Other large magnitude floods have occurred in Wellesley in 1936, 1938, 1955, and 1969. Reliable records of flood flows on the Charles River have been kept on the USGS Gaging Station No. 01103500 at

Charles River Village, Needham, since 1938. USGS Gaging Station No. 01104500 at Waltham has recorded flows since 1932. USGS Gaging Station No. 01104200 at Wellesley has recorded flows since 1960. The floods of record at Charles River Village occurred in 1955 and 1968.

Flood damages in Westwood have not been as severe as the floodplains of the rivers, and streams have not been significantly encroached upon by development. Many of the streams have small drainage area and relatively low peak flood flows.

Excessive rainfall, alone or combined with snowmelt, has produced flooding in the past in Dover, Franklin, Medway, Millis, Needham, Plainville and Wrentham. In Dover, floods occurred in the town in 1936, 1938, 1955, 1968, and 1969. The floods of record at the USGS gage (No. 01103500) at Charles River Village occurred in 1955 and 1968. Both floods were approximately 1-percent-annual-chance floods. Areas near the Charles River Street Bridge, Turtle Lane, Mill Street, Claybrook Road, the Centre Street Bridge, and the Chestnut Street Bridge have experienced periodic flooding.

In Medway, floods on the Charles River have occurred in 1936, 1938, 1955, and 1968. One of the most severe storms in recent history was Hurricane Diane which occurred in August 1955. The August 1955 flood was considered to be close to a 1-percent-annual-chance flood in this area. The most extensive damage from this flood occurred in the populated area of town, along Village Street. Less severe damage occurred along most of the Charles River and the lower sections of Chicken and Hopping Brooks.

Floods in Millis have occurred in 1936, 1938, 1955, and 1968. The flood, caused by Hurricane Diane, in August 1955 was approximated to be a 1-percent-annual-chance flood. Several homes and businesses near the State Route 109 Bridge over the Charles River sustained extensive damage during the flood of August 1955 and again in the flood of March 1968.

Needham has experienced flooding problems of varying degrees on numerous occasions. Recent major floods on the Charles River occurred in March 1968 and August 1955. The March 1968 flood was caused by both a major storm that dropped approximately 6 to 7 inches of rain and a significant amount of snowmelt. Similar floods occurred in March 1936 and 1969, January 1979, and in April 1987. Torrential summer storms caused the floods of August 1955 and July 1938. The August 1955 flood in particular was caused by precipitation from the second of two hurricanes falling on ground already saturated by precipitation from the first. A flood in 1886 may have exceeded the 1955 and 1968 floods in magnitude, but historical records are not precise. The floods of March 1968 and August 1955 both recorded peaks of 3,220 cubic feet per second (cfs) at the USGS Charles River gage at Dover (No. 01103500). This corresponds to 4- to 2.5-percent-annual-chance (25 to 40 year) flood at the gage. During large floods, water elevations on the Charles River are affected by backwater at bridge crossings and by dams. The Newton Lower Falls, the Cardingly, and the Metropolitan Dams on the Lower Charles River and the Cochrane Dam on the Upper Charles River serve to regulate flood flow. Tables 9 and 10 show the historical flood level for various points along the Charles River and Fuller Brook provided by the Town of Needham Public Works Department and the USACE Water Resource Development Plan (Reference 45):

TABLE 9 – HISTORICAL FLOOD LEVELS ON CHARLES RIVER
NEEDHAM, MA

	ELEVATION (feet NAVD[1])				
CHARLES RIVER	MARCH 1936	AUGUST 1955	MARCH 1968	JANUARY 1979	APRIL 1987
CHARLES RIVER STREET	*	*	106.89	*	*
CENTRAL STREET (DOVER)	*	*	105.48	*	*
SOUTH STREET	*	*	103.45	*	*
USGS GAGE AT DOVER	98.0	98.2	97.7	97.3	*
CHESTNUT STREET	94.0	95.6	95.2	*	*
DEDHAM AVENUE	92.5	94.2	93.53	*	92.84
GREENDALE AVENUE	91.6	93.6	91.5	*	91.27
KENDRICK STREET	91.0	92.4	97.61	*	88.11
HIGHLAND AVENUE	90.6	92.4	88.2	*	87.78
CENTRAL AVENUE	88.4	88.6	87.5	*	87.17

*Data not available
[1] North American Vertical Datum of 1988

TABLE 10 – HISTORICAL FLOOD LEVELS ON FULLER BROOK
NEEDHAM, MA

	ELEVATION (feet NAVD[1])		
FULLER BROOK	FEBRUARY 1970	MARCH 1971	MARCH 1972
BROOK STREET (UPSTREAM WELLESLEY INCINERATOR)	130.8	130.4	130.8
PILGRIM ROAD (DOWNSTREAM)	131.8	131.3	131.7

[1] North American Vertical Datum of 1988

There are several areas of Needham subject to flooding during large storms. Some of these include the areas near Edgewater Drive and Edgewater Lane, Charles River Village, and to the south of Alden Road near Pine Swamp. During the 1968 flood, some homes on Edgewater Lane were reported to be completely surrounded by water. Flooding has also been a problem along Grosvenor Road and is believed to be caused by a poor drainage system. In addition, many homeowners report flooded basements throughout the town during most major storms. Establishment of the 10-, 2-, 1- and 0.2-percent-annual-chance flood elevations in Pine Swamp will aid in the formulation of a rational development policy for the Fuller Brook area.

Flooding in the Town of Plainville can occur anytime; however, major flooding usually occurs during the spring as a result of heavy rain combined with snowmelt or late summer-early fall due to tropical storms. The greatest flood in the memory of town officials occurred in March 1968. During that flood, overflow from Turnpike Lake flooded sections of U. S. Route 1 and Shepard Street, and the Ten Mile River flooded part of West Bacon Street.

In Wrentham, in January 1979, a heavy rainfall caused Hale's Pond to overflow and damaged a culvert and a portion of roadway on Jenks Street.

More than ten major flooding events have occurred in Massachusetts over the last 50 years. Many of these have caused minimal-to-moderate damage to Norfolk County. Hurricane Gloria in September 1985 arrived at low tide and resulted in storm surges less than 5 feet above normal, minimizing damage to the coastline. Hurricane Bob in August 1991 passed south of Norfolk County primarily affecting Southeastern Massachusetts, Cape Cod and the Islands. An unnamed coastal storm in October 1991 joined up with the remains of Hurricane Grace and produced the third highest tide recording at the Boston gage. This storm was labeled as the Perfect Storm by the National Weather Service. Winds measured over 80 MPH and waves were over 30 feet in some parts of the Massachusetts coastline, causing flooding and wind damage to several counties, including Norfolk (References 46 and 47).

Norfolk County also saw flooding from severe storms in October 1996, June 1998, March 2001, April 2004 and May 2006. The June 1998 storm was slow moving and produced rainfall of 6 to 12 inches over much of eastern Massachusetts. On May 24, 2009 Bristol, Plymouth, Norfolk, and Worcester Counties experienced an intense thunderstorm causing minor flooding, winds exceeding 70 MPH, and quarter sized to golf-ball sized hail (Reference 47).

In March 2010, heavy rainfall of 6 to 10 inches fell over much of Southern New England resulting in major flooding across eastern Massachusetts and Rhode Island. The Charles River at Dover and the Neponset River at Norwood both went into flood stage. The Neponset River rose to major flood stage, inundating the Norwood Memorial Airport with three and a half feet of water. Many roads throughout Norwood County were flooded including Furnace Brook Parkway in Quincy and two lanes of interstate 93 at Furnace Brook Parkway (Reference 48).

From December 2010 through February 2011, Southern New England, including Norfolk County, saw a series of winter storms that led to record snowfall for the season. Boston snowfall total was over 70 inches, more than 45 inches above average for the time of year. Heavy snow, combined with rain led to numerous flooding problems across the county, roof collapses, and downed trees and utility lines (References 49 and 50).

In August 2011, Hurricane Irene, weakened to a tropical storm, flooded numerous roads in the Greater Boston area, including Storrow Drive and Memorial Drive. Fallen trees and power outages were widespread (Reference 51).

2.4 Flood Protection Measures

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

As a result of past flooding in the Charles River basin, the USACE studied the flooding problems and recommended, and eventually implemented, a plan to acquire large floodplain, or Natural Valley Storage, areas as a method of controlling rates and quantities of runoff in the basin. It has long been known that the tremendous low-lying land areas throughout the Neponset and Charles River basins have contributed greatly to modifying peak flows during times of major flooding. These storage areas act as a sponge, absorbing high flows coming into the area and slowly releasing water at a much lower rate. Up until now, development has encroached more and more into these areas, resulting in higher rates of flow. The USACE, in accordance with the Water Resources Development Act of 1974 (Public Law 93-351), implemented its plan and actively sought to purchase undeveloped available areas, to insure that they would remain as such. The program involved the extensive acquisition of land in several communities within the Charles River basin including the Towns of Dedham, Dover, Medfield, Medway, Millis, Needham, Norfolk and Wellesley. (Reference 52, Reference 53). Most of this program was completed by 1983.

Several communities in Norfolk County, many of them in response to 1955 flood, have made improvements to drainage systems and other works to protect against future flood damage.

The coastal Towns of Cohasset and Weymouth and the City of Quincy have constructed tide gates, seawalls and dams in an effort to control flooding. In Cohasset, a new dam and reservoir have been constructed on Bound Brook at the southern tip of Cohasset. The dam is located at the end of Beechwood Street adjacent to the Wompatuck State Park boundary. Flooding in southern Cohasset has been controlled since the dam was put into operation. Flooding has specifically been controlled by the dam at the Doane Street Bridge, which at times in the past has been overtopped by 1.5 feet of floodwaters. Additional flood control of Bound Brook is provided by the control gate at the Beechwood Street Bridge just north of Mill Lane. This gate is used to control the elevation on Lily Pond. The tide gates at the mouths of James Brook and Richardson's Brook are both flap gate devices at the exits of culverts under Border Street and Jerusalem Road, respectively. These gates allow the streams to empty into tidal areas but do not allow the tidal surges to move upstream through the culverts. The elevations for the roads above these culverts are 7.9 feet and 8.3 feet for Border Street and Jerusalem Road, respectively. Both of these roads are inundated during the 1-percent-annual-chance storm.

In Weymouth, several flood protection works have been constructed. They are in the form of seawalls that are located along the banks of both the Weymouth Fore River and the Weymouth Rack River. Along the Weymouth Fore River, a seawall extends from Kings Cove to the other side of the neck and protects the power plant located there. Along the Weymouth Back River, a seawall is located on the south side of the neck below Upper Neck Cove. The above-mentioned seawalls provide flood protection from high tides and also act as retaining walls to eliminate shoreline erosion. Low elevation barriers have also been placed along the Hingham Bay shoreline to provide some protection from minor wave attack. Whitmans Pond Dam was used in industry for timber and wool scouring and, therefore, does not provide flood protection.

In Quincy, many projects have been undertaken to lessen the flooding problems over the years. Seawalls have been built along parts of Houghs Neck, Squantum, and Wollaston Beach. Dikes have been built around the lower area of Montclair. Drain pipes from many of the low flood prone coastal areas have tide gates at their outlets. In 1976, a major flood prevention project was undertaken on Hayward Creek (Reference 53). Flood prevention projects are under study for Furnace Brook and Town Brook (References 33 and Reference 54). A study is also being prepared to determine means of relieving the flooding in the low area of Montclair.

In Dedham and Needham, efforts to limits the disastrous effects of high floodwaters have been very successful throughout the reaches of the Charles River and for the Mother Brook in Dedham. In addition, Dedham is affected by the reaches of the Neponset River. The methods employed were a combination of construction of various public works projects and of utilizing existing natural storage areas to cut down on the damaging effects of high flood levels. For all practical purposes, the 1955 flood in the Neponset and Charles Rivers was probably the most damaging to the Town of Dedham. After the 1955 flood, extensive improvements were performed on both rivers. Work on the Charles River consisted of construction of two new adjustable weir dams, one at Silk Mill in Newton Upper Falls approximately 3.7 miles downstream from Dedham, and the other at the diversion structure for Mother Brook, in addition to extensive channel improvements. The new bascule dam at Silk Mill replaced an older fixed weir type of dam. Extensive channel excavation from the Silk Mill Dam upstream approximately 2 miles to Kendrick Street, Needham was also implemented at this time. While this work was being carried out, a bascule dam was also built at the downstream structure for Mother Brook. Though not originally designed as a flood control structure, the dam was designed so that, with proper control, approximately one third of the Charles River flow would be diverted to Mother Brook, further reducing flooding on the Lower Charles River. The USACE concluded the safe diversion capacity of Mother Brook to be approximately 1,000 cfs (Reference 45). The 1968 flood provided an excellent opportunity to evaluate these flood improvements in both Dedham and Needham. Peak flows at Charles River Village were exactly the same in 1955 and in 1968, but the maximum elevation at the Bridge Street Bridge was lower in 1968 by approximately 3.1 feet. This magnitude of difference held true for the entire reach of the Charles River in the Town of Dedham. The bascule dam at Mother Brook was an important factor in diminishing flood elevations along the brook. With floodwaters rising steadily, officials decided to raise the weir approximately 4 inches. By doing so, flow over the dam peaked at 1,040 cfs, just before the weir was raised. As soon as the elevation was increased, flow slowly decreased. At the same time the Mother Brook Dam was raised, the Silk Mill Dam was lowered approximately 8 inches, resulting in a dramatic increase in flow over the dam. This contributed greatly to decreasing flood elevations upstream throughout Dedham. The 1968 flood graphically illustrated that, by proper manipulation, flood elevations on the Charles River and Mother Brook can be controlled significantly, depending on timely action by responsible individuals. Unfortunately, lowering flood elevations throughout any particular reach of river invariably results in higher elevations elsewhere. Sound judgment must be practiced by those involved to best utilize these flood protection measures.

Also in Dedham, during the development of the Dedham 1978 FIS report, work was started to replace the Bussey Street Bridge over Mother Brook. It is assumed this work has been completed. The old bridge was a frequent source of trouble because of a very small bridge opening. The new bridge, with a substantially larger opening, should help alleviate some of the problems in this area. In addition, the small dam a few hundred feet downstream of Bussey Street was breeched to lower the brook level in order to facilitate construction required for the new bridge. It is now intended that the breech in the dam will remain, but stop planks will be added to allow some water level control upstream of the dam. Under normal flow conditions, the planks would be at existing crest elevations. During times of peak flow, these planks might be removed if the need arises. Additional work downstream of Mother Brook in Hyde Park, Massachusetts (Suffolk County) was planned or underway at the time of the Dedham 1978 FIS. These projects should prove beneficial to the Town of Dedham.

The Neponset River has also undergone extensive hydraulic changes since 1955 and prior to 1968, benefiting the Towns of Dedham and Milton. As shown previously, the 1955 flood elevation on the Neponset River in the vicinity of Dedham was approximately 4.4 feet higher than 1968, although flow was only approximately 25 percent higher in 1955. Following the 1955 flood, the Metropolitan District Commission (MDC) instituted a vast program of hydraulic improvements on the Neponset River from Lower Mills Dam in Milton, upstream to Paul's Bridge, approximately 0.7 mile downstream of the Dedham/Boston/Milton boundary. These improvements included channel excavation and realignments and dam replacement. The Lower Mills Dam renovation consisted of lowering the weir elevation of the dam and installing more stop planks, with the ultimate objective being greater flexibility of controlling peak flows upstream of the dam. A new bascule dam at the Tileston and Hollingsworth Company in Hyde Park, Massachusetts (Suffolk County) was installed 3.2 miles downstream of the Dedham/Boston/Milton line or approximately 2.7 miles upstream of Lower Mills in Milton. This dam was constructed by utilizing movable bascule gates which allow the crest elevation to vary. During times of peak flood discharge the crest is lowered allowing more flow to pass the dam for a given stage; this can be an important factor in flood control.

Also in Milton, an extensive program of channel improvements on Pine Tree Brook was completed in the mid- to late-fifties. Those improvements lessened, to a degree, the flooding potential throughout the watershed. About 1970, an earthen dam was constructed near the headwaters of Pine Tree Brook. This multi-purpose structure not only retards floodwaters but also creates a habitat for native wildlife. Normal flows are allowed to pass over the spillway, but any flows caused by excessive rainfall are impounded and released over a longer time period. This facility controls approximately 53 percent of the total watershed area and provides almost complete protection against major floodwater damage.

In Avon, a drainage project consisting of a series of canals, trap basins, and underground pipes has reduced the flooding potential within the community. Flooding is diverted to undeveloped swampy areas along Three Swamp Brook where property damage would be minimal.

In Braintree, since the August 1955 flood, the bridges at Pearl Street, Unions Street, Plain Street, Adams Street, McCusker Street, and River Street have been rebuilt. The Ivory Street Bridge has been added, and Factory Pond and Ames Pond Dam have been removed along the Monatiquot River. Along the Farm River, the bridge for Lundquist

Drive, Campenevelle Drive, Granite Street, and Pond Street have been replaced. The USACE has built flood protection works on Smelt Brook and Hayward Creek.

In Canton, following the 1955 flood, the USACE determined that some degree of protection against future flood damage was necessary for the Canton River. In the early sixties, flood protection measures known as the Canton Local Protection Project were constructed along the Canton River in the vicinity of the Plymouth Rubber Company (Reference 55). The project consisted of a diversion spillway and channel that would safely pass peak flood flows around the portion of the Canton River that passed near the Plymouth Rubber Company. This project was successful in lowering potential flood damage from the 1968 storm. The periodic cleaning of numerous culverts and streams and the enlargement of various culverts would play an important role in minimizing flood damage in Canton. The USACE also constructed a diversion channel connecting Silk Mill Pond to Bolivar Pond. Flooding of Lower Massapoag Brook is partially alleviated by this flood protection measure; however, Bolivar Pond is unable to store the additional inflows, and consequently, Bolivar Pond Dam overtopped during the storms of March 1968 and February 1978. These events precipitated the development of a flood control communication system between the dam owners. Ponds on and off Massapoag Brook are now drawn down in preparation of large storms. The hydrologic and hydraulic analysis for this FIS assumes that all stop logs are removed from Bolivar, Forge, and Reservoir Pond Dams during flooding events.

Between the August 1955 flood and the March 1968 storm, both major and minor flood protection measures were taken in Norwood. One major undertaking was the relocation and dredging of the Neponset River during the construction of I-95 in the vicinity of the Neponset Street, I-95 interchange. The Neponset Street culvert was also enlarged at this time, with the result that the street, which was impassable during the 1955 hurricane, was not flooded in the 1968 storm. The Factory Mutual buildings were protected in 1968 by a dike around the plant which had been constructed after the 1955 storm. Culverts on Plantingfield, Meadow, Hawes, and Traphole Brooks were rebuilt or enlarged. Portions of Hawes and Purgatory Brooks were relocated and a protective berm was constructed in the David Terrace area of Purgatory Brook. Although Norwood does not have floodplain or wetlands zoning, the importance of the preservation of wetlands as flood storage areas was emphasized for the Neponset River in a report for the Massachusetts Water Resources Commission in April 1971 entitled “Neponset River Basin Flood Plain and Wetland Encroachment Study” (Reference 56). Norwood has set aside several parcels of Conservation Land along streams, including Traphole Brook, Purgatory Brook, and Hawes Brook, near Ellis Pond.

The town of Randolph has pursued a vigilant and successful program of channel and drainage system maintenance and upgrading. The policy of channel debris removal has been very helpful in mitigating drainage problems. Hydraulic modifications of bridges and culverts increased discharge capacities for the culverts for Glovers Brook at Pearl Street, Norroway Brook at Grove Street, Brook B at Oak Street, and Unnamed Tributary to Mary Lee Brook at Petipas Lane. Improvements have been made in the town through cooperation among the town, the Commonwealth of Massachusetts, and the USACE. Mary Lee Brook, which once presented a severe flooding threat, has had its inadequate flood-control structures replaced with adequate structures and other major channel improvements have occurred.

In Stoughton, since the March 1955 disaster, the town has replaced many of its undersized culverts with larger culverts designed to handle future major storms. The two dams which were washed out have been rebuilt, but are relatively small and not used for flood-control purposes. There is no other flood protection works existing or planned that would affect flooding in Stoughton.

In Walpole, following damage caused in Walpole Center by the 1955 and 1968 floods, the Norfolk Conservation District and the Town of Walpole joined in requesting assistance from the USDA NRCS in developing a plan to reduce flood damage. A watershed work plan for the Diamond and Spring Brooks watershed was developed under the authority of the Watershed Protection and Flood Prevention Act (Reference 44). The final plan was submitted to Congress and approved in early 1976. Measures included in the plan consist of land use conservation, a multiple-purpose reservoir site with storage for floodwater, approximately 888 feet of reinforced concrete conduit, and an enlargement of 350 feet of stream channel. The project provides an estimated 99 percent reduction in average annual floodwater damages on Diamond Brook (Reference 44). The dam was completed in 1979. Design of the culvert and the channelization began in early 1981 and was completed in 1985. Some flood flow reduction along the Neponset River is the result of operations of the Neponset Reservoir Company. The company's operating committee is attempting to impound the winter-spring runoff in both the Neponset Reservoir and Willet Pond for release during the dry summer period. The 270-acre Neponset Reservoir can provide significant flood peak reductions on its 1.6 square mile drainage area if flooding occurs during a period when the reservoir is drawn down. Even if Neponset Reservoir is at its maximum operating level, freeboard storage has a significant effect in reducing peak flow (Reference 44).

The Town of Plainville does not have flood protection structures; however, natural storage in swamps and ponds combined with the low topographic and stream gradients in the area diminish peak flows. The Plainville Highway Department monitors Turnpike Lake, releasing water when the water level rises too high. Currently, the town does not have zoning ordinances pertaining to flood-prone areas.

In Norfolk, there are numerous dams located on the streams. They are principally used for agricultural, industrial and recreational purposes.

In Sharon, since the 1955 storm, flood control measures have been implemented in the town. During a major storm, a patrol is dispatched by the Department of Public Works, which checks water levels on the numerous ponds and lakes in the town. By varying the height of planking at the outlet structures, this patrol controls, by common sense methods, the storage capacity of these lakes and ponds. The Director of Public Works in Sharon believes that these flood control measures were, in part, a reason for the considerably reduced damage caused by the 1968 flood (as compared to the 1955 flood). In recent years, the Department of Public Works has made a point of keeping the hydraulic structures in the town clear of debris and in good working order. Several flood plain management measures have been taken by town officials. The town amended the zoning bylaws to regulate construction in flood hazard areas as designated on the Flood Hazard Boundary Maps (Reference 57). Sharon has a conservation fund for acquiring land for conservation, including on-going programs for wetland protection. Coordination with state agencies and neighboring towns was maintained in areas where there is a common interest.

The Towns of Dedham, Holbrook, Millis, Norfolk and Walpole use non-structural measures of flood protection to aid in the prevention of future flood damage. Chapter 131, Section 40 (310 C.M.R. 10.00) of the General Laws of the Commonwealth of

Massachusetts (most recently revised on April 1, 1983) is commonly referred to as the Wetlands Protection Act. The law gives the responsibility for issuing permits to remove, fill, dredge, or alter wetlands to the local conservation commission. The commission has to determine if an area on which a permit requested is significant to public or private water supply, to the ground water supply, to flood control, to storm damage prevention, to prevention of pollution, to protection of land containing shellfish, or to the protection of fisheries. After a public hearing, the commission can impose such conditions as will contribute to the protection of these interests. The Department of Environmental Quality Engineering (DEQE) may also make a determination after a review of the commission's order. Conditions imposed by the DEQE supersede conditions imposed by the commission. Detailed rules and regulations concerning the administration of this act have been promulgated by the DEQE. Section 40 now requires a conservation commission, if requested, to make a determination of whether a particular parcel of land is a wetland and governed by the Wetlands Protection Act. It also contains definitions of terms to aid this determination. Chapter 131, Section 40A of the Acts of 1968 (amended by Chapter 782 of the Acts of 1972), gives the commissioner of the Department of Environmental Management the authority to protect inland wetlands and floodplains by establishing encroachment lines for the purpose of preserving and promoting the public safety, private property, wildlife, fisheries, water resources, floodplain areas, and agriculture. The commissioner may adopt orders regulating, restricting, or prohibiting the altering of polluting of inland wetlands by designating lines with which no obstruction or encroachment would be permitted without prior approval. These restrictions require notifications to each land owner affected, public hearings, and approval by the town. Section 40A was further amended by Chapter 818 by defining "inland wetlands" to include the definition of "freshwater wetlands" as set forth in Section 40 as "portions of any bank which touches any inland waters or any freshwater wetland, and any freshwater wetland subject to flooding".

In addition, the Town of Dedham is one of the first communities in the Commonwealth of Massachusetts to have a large percentage of its wetlands subjected to an Order of Restriction in accordance with Chapter 131, Section 40A, of the Massachusetts General Laws. These wetlands may not be developed and, as such, add a great deal of flood protection for the waterways throughout the town. In addition, local zoning laws support this action of prohibiting building within the floodplain.

The Towns of Braintree, Cohasset, Medfield and Wellesley have enacted floodplain and watershed protection plans to combat flooding. The Town of Braintree’s Wetland and Floodplain Protection District serves to regulate future development within the floodplain areas. This not only protects against flood damage to new structures, but also assures that the natural flood storage areas in the town will be protected. The Towns of Cohasset, Medfield and Wellesley maintain Floodplain and Watershed Protection District(s). In Cohasset, the district restricts land use to protect persons and property, to preserve and protect the water supplies of Cohasset and adjacent towns, and to provide adequate and safe water storage and runoff capacity (Reference 58). In Medfield and Wellesley, the districts created in 1969 and 1974 respectively, restrict construction, excavation, fills, and grades. In Medfield the districts are defined by elevation in some areas and distance from the stream centerline in other areas, while in Wellesley the restrictions are at or below specified elevations; these elevations vary with the location in the town.

The Town of Needham has adopted zoning laws that define floodplain "use districts." The zoning bylaws and the Needham zoning maps define the extent of the floodplain districts as well as the elevations to which these zoning regulations apply (Reference 59).

Within the floodplain districts, hazardous/toxic material manufacturing, handling, storage, and disposal are prohibited, as are any forms of solid waste disposal. Also encroachments including fill, replacement of soil with impervious material, new construction, substantial improvement or other development unless certification by a registered professional engineer is provided demonstrating that encroachments shall not result in any increase in flood levels in the town during the occurrence of a 1-percent-annual-chance flood, were prohibited. This zoning regulation encompasses all of the detailed studied streams in this study. The restriction on increases in the 1-percent-annual-chance flood levels rules out the establishment of floodways in these floodplain districts of Needham.

There are no existing or planned flood protection measures in the Towns of Bellingham, Franklin and Foxborough. Also in Bellingham, there are also no flood fighting or emergency evacuation plans. The Civil Defense Office of Bellingham is responsible for alerting residents of impending disasters and coordinating any emergency operations with town and state public service agencies.

In Needham and Randolph, the police and fire departments with support from the local Civil Defense, the Highway and Engineering Departments, and the town's administrative offices are responsible for local flood warnings. The National Weather Service at Logan Airport provides flood warning and forecasts on a regional scale.

There are presently no flood control structures which would have an effect on the base flood elevations (BFEs) computed for Holbrook, Medway, Millis, Norfolk, Westwood and Wrentham.

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-, 50-, 100-, or 500-year period (recurrence interval) have been selected as having special significance for floodplain management and for flood insurance rates. These events, commonly termed the 10-, 50-, 100-, and 500-year floods, have a 10-, 2-, 1-, and 0.2-percent chance, respectively, of being equaled or exceeded during any year. Although the recurrence interval represents the long-term, average period between floods of a specific magnitude, rare floods could occur at short intervals or even within the same year. The risk of experiencing a rare flood increases when periods greater than 1 year are considered. For example, the risk of having a flood 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 Riverine Hydrologic Analyses

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

For each community within Norfolk County that has a previously printed FIS report, the hydrologic analyses described in those reports have been compiled and are summarized below. Due to levee de-accreditation status at the time this FIS was finalized, the Town of Canton was not included within this Partial Countywide Study. Data related to the Town of Canton remains in this FIS report for informational purposes only, and users should refer to the separately published FIS report and FIRMs for effective data.

Pre-partial Countywide Analyses

Peak discharges and discharge-frequency relationships for the Charles River in Franklin, Medfield, Needham and Wellesley were obtained from previous FISs. In Franklin, peak discharges for the upstream portion of the Charles River were obtained from the FIS for the Town of Medway (Reference 13). Peak discharges for the downstream portion of the Charles River were computed by reducing the discharges at the Medway/Bellingham/Franklin town boundary using all appropriate reduction factor based on the ratio of drainage areas. In Medfield, a discharge-frequency relationship for the Charles River was established by utilizing the relationship developed for the Needham FIS (Reference 16) and information presented in the USACE hydrologic analyses (Reference 16). In Needham, peak discharge relationships for the Charles River were taken from the FISs for the Towns of Wellesley, Dover, Westwood and Dedham and the City of Newton (References 25, 8, 26, 7, and 60). The discharges were compared with peak flows estimated using, methodologies outlined in USGS Bulletin 17B and incorporating updated stream-flow records (Reference 61). Flood flows adopted for the Charles River at Needham were those previously published peak flows that did not differ significantly from the latest estimates, and that were consistent throughout the various earlier studies performed on the Charles River in the vicinity of Needham. The large natural storage capacity of the wetlands along the Charles River further reduces flood peak magnitudes. In Wellesley, a discharge-frequency-drainage area relationship for the Charles River in Wellesley was established by utilizing the relationships developed for the Newton and Needham FISs (References 60 and Reference 16).

Discharge-frequency relationships and peak flows for the Charles River in Bellingham, Dedham, Dover, Medway, Millis and Norfolk were established from the USGS Charles River Village gage (No. 01103500) in Dover having a period of record of 41 years, using the log-Pearson Type III distribution (Reference 62, Reference 63, Reference 64, Reference 65). Reliable records have been kept at the USGS gage on the Charles River at Charles River Village since 1938. In Bellingham, a log-Pearson Type III regression analysis of annual maximum discharges (Reference 34). The peak discharges computed were then transposed to other locations on the Charles River in Bellingham by multiplying by the ratio of drainage areas raised to the 0.7 power. In Medway, Millis and Norfolk, the computed discharges were extrapolated using USDA NRCS methods to obtain peak flow discharges (References 66 and 67). In Millis and Norfolk, stationing along the Charles River and the associated drainage areas were obtained from a water resources development plan published by the USACE (References 68 and Reference 69). In Dedham, a standard log-Pearson Type III analysis (Reference 70) was utilized to determine peak flows for selected recurrence intervals on the Charles River and on Mother Brook. Flows determined from the Charles River Village gage were adjusted by a method developed by the USDA NRCS (Reference 71), utilizing a discharge drainage area relationship. According to Massachusetts General Laws, one third of the peak flows on the Charles River would be diverted to Mother Brook. There is a possibility that in the future, maximum rate of diversion to Mother Brook would be limited by legislative act to 1,000 cfs, which has been determined to be the approximate non-damaging capacity of the brook. At this time, no action to implement this change seems likely in the near future. Therefore, peak flows to Mother Brook have been determined by assuming the diversion of one third the peak on the Charles River to the brook.

Flows and hydrologic analyses for the Neponset River in Sharon, Norwood, Westwood, Canton, Dedham and Milton were based on the 1971 completed by the Anderson-Nichols & Company for the Massachusetts Water Resources Commission (MWRC) (Reference 56). This study was quite extensive and thoroughly investigated the complex hydrologic phenomena associated with riverine flooding compounded by wetland storage. This study determined peak discharges for the 1-percent-annual-chance flood for points on the Neponset River below the USGS gage (#0110500) in Norwood. The gage, maintained since 1940, is located on the left bank, 200 feet upstream of the Pleasant Street Bridge. At this station, the 1955 flood was the greatest flood since 1886. To establish the peak discharge-frequencies at this point, a log-Pearson Type III analysis was run on the record, with regional skew adjustment and historic weighting, according to guidelines set forth in U.S. Water Resource Council Bulletin No. 17 (Reference 70).

The study, entitled "Neponset River Basin Flood Plain and Wetland Encroachment Study", compared flood discharges and stages to the degree of wetlands encroachment within the basin (Reference 56). For the MWRC study, the USDA NRCS TR-20 Project Formulation Computer Program-Hydrology was used to compute peak discharges for points downstream of the Neponset River gage in Norwood (Reference 72). The TR-20 program computes surface runoff, taking into account conditions having a bearing on runoff, and routes the flow through stream channels and natural and artificial reservoirs. It combines the routed hydrograph with those from other tributaries and computes peak discharge, time to peak, and the water-surface elevation at selected cross sections. Since the 1-percent-annual-chance peak discharge at the gage established by the MWRC study falls within the 95 percent confidence interval of the gage frequency curve, and because the study was able to duplicate numerous historical flood peaks, the study therefore may be assumed to be a valid analysis of flooding on the Neponset River. Utilizing the discharge-frequency curve established at the gage and the generated discharge-drainage area relationship by the MWRC 1-percent-annual-chance flood peaks, a discharge-drainage area frequency curve was developed for points downstream of the Neponset River gage. Peak flows for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods at selected points on the Neponset River were obtained from the curve. Downstream of the gage, natural reservoir storage is an important factor in modifying peak discharges. Upstream of the gage, however, this factor is of less importance. A regional discharge-drainage area relationship was employed to establish flows upstream of the gage through the following steps.

Peak flows for the 10-, 2- and 1-percent-annual-chance floods were computed at the gage according to the USGS regional formula (Reference 62), which is of the form:

$$Q_n = C_1 A^{C2} S^{C3} P^{C4}$$

Where Q_n equals the peak flow for return interval n, A equals the drainage area, S equals the stream slope and P equals the mean annual precipitation. C_1, C_2, C_3, C_4 equal coefficients specific to Q_n.

The flows computed according to the USGS formula were found to be approximately twice as large as the flows established by the gage record. Therefore, a coefficient of 0.50 for Q10, 0.49 for Q50, and 0.49 for Q100 was applied to the USGS formula for the Neponset River upstream of the gage in Norwood. Flows at points upstream of the gage were then computed according to the USGS formula modified by the above coefficients. The 0.2-percent-annual-chance flows were determined by extrapolation of a log-probability graph of flood discharges computed for frequencies of up to 100 years. Loss of discharge along the Neponset River downstream of I-95 is due to the storage effect on the overbank areas.

The hydrology for the Neponset River in Canton was taken from the previous Flood Insurance Study for the Town of Canton (Reference 5). For the Neponset River in Canton and Dedham, peak discharges for the selected recurrence intervals were, for the most part, based on the previous report for the Neponset River Basin by the MWRC referenced above (References 55 and 73). The Neponset River, as it passes through Canton and Dedham, is characterized by sizable swampy areas that are extremely important in modifying peak flood flows throughout the course of the river. The methods used to derive the flood flows recorded in this report were sound and the results can be used with confidence. The conclusions of the report have therefore been adopted for this study.

Discharge-frequency curves for the Neponset River in Milton were derived by use of the USDA NRCS discharge-drainage area relationship (Reference 74), based on flood flows previously published for the Neponset River (Reference 56). These published flows were computed for the Tileston and Hollingsworth Dam, 2.7 miles upstream of the Lower Mills Dam in Milton. These flows were then adjusted by the USDA NRCS method (Reference 74) to reflect actual flows through the Town of Milton.

Neponset River profiles in Westwood were also based on data used in the development of the 1971 MWRC report.

In Walpole, the drainage area of the Neponset River was divided into 68 subwatersheds for flood-routing purposes. Subwatershed boundaries were delineated and the drainage areas planimetered from USGS topographic maps (Reference 75). Hydrologic soil group data were obtained from county soil maps (Reference 76). Land use data were obtained from the 1971 Massachusetts Map Down Project and topographic maps (References 75 and 77). The soil and land use data were then used to compute composite runoff curve numbers. Travel times and times of concentration were developed for each subwatershed based on estimated water velocities for overland and channel flows. Storage capacity and stage discharge curves were computed for all significant reservoirs and natural valley storage areas. The 10-, 2-, 1- and 0.2-percent-annual-chance storms for the Neponset River were flood routed through the upstream areas of the Neponset watershed using the USDA NRCS Technical Release No. 20 computer program (Reference 78). Rainfall data for the various frequency storms were obtained from Technical Paper No. 40 (Reference 79). A standard USDA NRCS 24-hour Type II rainfall distribution was assumed for all frequency storms. The hydrologic analysis for the Neponset River upstream of South Street is complicated by Cedar Swamp, which is actually four interconnected natural storage areas. The flow in the swamp areas is restricted by culverts under railroads and streets. Water-surface elevations vary in different sections of the swamp. Since the Technical Release No. 20 program does not accept stage-discharge ratings that vary with tailwater elevation, a USDA NRCS computer program known as SWAMP was used to flood route through the Cedar Swamp area (Reference 80). Computer runs were made using the SWAMP program for the 10-, 2-, 1- and 0.2-percent-annual-chance frequency storms. The SWAMP program provided an outflow hydrograph for the entire Cedar Swamp area that was used as input data in the Technical Release No. 20 program for the downstream section of the Neponset watershed. An analysis of stream gage records on the Neponset River was made as a calibration check of the hydrologic model of the watershed using Technical Release No. 20 and the SWAMP programs (References 78 and 80). Flood flow frequency data were based on statistical analysis of discharge records covering a 34-year period at the USGS gaging station on the Neponset River at Norwood (Reference 81). This analysis followed the standard log-Pearson Type III method as outlined by the Water Resources Council in September 1989 (Reference 82).

A multiple regression analysis, developed by Johnson and Tasker, was employed to find runoff discharges in Foxborough; Stoughton; riverine flooding in Weymouth; and for Beaver and Trout Brooks in Avon (Reference 63). Standard USGS topographic maps were used to determine watershed areas and local topography (References 83, and 84). An annual precipitation value of 3.67 feet per year, representative of the southeastern Massachusetts region, was obtained from the U. S. Weather Bureau Technical Paper 40 (TP-40) and was used throughout southeastern Massachusetts (Reference 79). By determining values for slope and area and using them in conjunction with the precipitation value in the Johnson-Tasker formulas (Reference 63), values for runoff from 10-, 2- and 1-percent-annual-chance storms were predicted. Exponents for the 0.2-percent-annual-chance storm frequency equation, though not given in the Johnson-Tasker report, were arrived at by extrapolating the given values for the 10-, 2- and 1-percent-annual-chance storms. Wherever possible, stream gage records were compared to these figures. Contributing flows from neighboring towns were obtained from other studies when available in Avon and in Foxborough (References 85, 86, 22, 19, and 24); or, where no other study had been conducted, the associated watershed was isolated and the Johnson-Tasker method was applied. In Weymouth, there is one gage located along the Old Swamp River near Whitmans Pond (10 years of record). A log-Pearson Type III analysis of this gage found discharge values of the Johnson and Tasker method to be compatible (Reference 63). Contributing flows from the Flood Insurance Studies for the Towns of Braintree and Holbrook were also obtained (References 87 and 88). Where no other study has been conducted, isolating the associated watershed and applying the Johnson-Tasker regression analysis was used.

In Stoughton, peak discharges obtained using the Johnson-Tasker method were compared to discharges obtained from analysis of stream gage records on the Neponset River in the neighboring Town of Canton using the correlation formula

$$Q_s/Q_g = A_s/A_g^{\ x}$$

where Q_s and Q_g are the flows at the site and gage, respectively, A_s and A_g are the drainage areas at the site and gage, respectively, and x is the regional drainage-area ratio exponent. After comparison of predicted discharges with experienced 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 run-off. The adjusted discharge figures in Avon, Foxborough, Stoughton and Weymouth more closely reflect the true nature of the basins involved.

The analyses for the Monatiquot, Cochato, and Farm Rivers in Braintree and the Canton River in Canton were based on stream flow records at USGS gaging station No. 01105500 located approximately 100 feet downstream of Washington Street on the East Branch Neponset River in Canton, Massachusetts. In Braintree, the 10-, 2-, 1- and 0.2-percent-annual-chance discharges for the Monatiquot, Cochato, and Farm Rivers were determined from the previously effective FIS for the Town of Braintree (Reference 89). The analyses for these streams were based on stream flow records at USGS gaging station No. 01105500, for the period from 1952 to 1974. The Monatiquot River and Farm River discharges for the 10-, 2-, 1- and 0.2-percent-annual-chance year floods are based on a statistical analysis of the East Branch Neponset River flow records from 1952 to 1974, using a log-Pearson Type III distribution and a regional skew of 0.5 (Reference 90). The data were also adjusted to reflect partial duration and the size of the sample. Flood discharges from the East Branch Neponset River were compared to the discharges calculated by the USACE at the Armstrong Cork Company Dam, located on the Monatiquot River. The discharges were found to be in agreement; therefore, the discharge-frequency data developed at the East Branch Neponset River gage, with a drainage area of 27.2 square miles, was considered applicable for use at the Armstrong Cork Company Dam, with a drainage area of 25.6 square miles.

The discharges for other locations along the Monatiquot and Farm Rivers were determined by using the drainage area-discharge ratio formula that follows:

$$Q_1/Q_2 = (DA_1)^{0.7}/DA_2$$

where Q_1 and Q_2 are the discharges at the specific locations, and $DA_1$and DA_2 are the drainage areas at these locations. The exponent 0.7 reflects a value developed for the area. The discharge for the Cochato River was developed as part of the Draft Flood Insurance Study for the Town of Randolph (Reference 91).

Hydrology for the Canton River in Canton was taken from the previous Flood Insurance Study for the Town of Canton (Reference 5). A standard log-Pearson Type III analysis was utilized to determine peak flows for the selected recurrence intervals (Reference 92). Flow records (1953 through 1973) for the USGS gaging station No. 1-10550 were analyzed, and a discharge-frequency curve was developed. The HEC-1 analysis used these gage data as calibration targets (Reference 93). HEC-1 hydrologic parameters were adjusted until the computed discharges matched the discharges measured for the March 1968 storm and the discharges computed in the standard log-Pearson Type III analysis. The rainfall totals and distributions for the 10-, 2-, 1- and 0.2-percent-annual-chance storms were taken from a statistical analysis of local historical precipitation data.

Discharges for Lower Pequid Brook, Upper Pequid Brook, Massapoag Brook, and Beaver Meadow Brook in Canton, which form the drainage basin above Forge Pond Dam, were estimated using the HEC-1 computer program (Reference 93). HEC-1 is a rainfall-runoff and hydrologic routing model. The HEC-1 analysis described above also provided elevation frequency relationships for Bolivar, Forge, and Reservoir Ponds (Reference 93).

For Ponkapoag Brook in Canton, discharge-frequency relationships were determined using peak discharges assuming the brook was located in a rural watershed. These rural flows were then transformed to urban flows based on basin development characteristics (Reference 94 and Reference 95).

In Needham, storm depths and patterns as well as runoff volumes and hydrographs from the Fuller Brook basin resulting from 24-hour storms of 10-, 2- and 1-percent-annual-chance return periods were computed. The 0.2-percent-annual-chance storm depth was extrapolated using the values for storms of the 10-, 2- and 1-percent-annual-chance return periods. The methodology followed standard procedures developed by the USDA NRCS (Reference 96 and Reference 70). Catchment areas, curve numbers, concentration times, and other parameters were determined from USDA NRCS soil survey maps at a scale of 1:24,000 and from USGS topographic maps at a scale of 1:25,000 with a contour interval of 10 feet (References 97 and 98). For each storm event, the inflow to Pine Swamp storage along Fuller Brook was routed to the downstream control (outlet at corporate limits) by use of the USACE HEC-1 computer program (Reference 93).

In Randolph, hydrologic analyses were based on flow records of the USGS Gaging Stations (01 1050 00) on the Neponset River at Norwood and (01 1049 00) on Mill Brook at Westwood. Periods of record for the stream gages are from 1939 to 1972 and from 1964 to 1973, respectively. The gages are from 1939 to 1972 and from 1964 to 1973, respectively. The 10-, 2-, 1- and 0.2-percent-annual-chance discharges for the Cochato River were based on statistical analysis using a log-Pearson Type III distribution of the flow records with a regional skew equal to 1.0 from 1939 to 1972 for the similar Neponset River watershed (Reference 63). These values check well with design discharges computed by the Massachusetts Flood Magnitude formulas developed by the USGS (Reference 63). As a further check, the discharges were compared to the discharges calculated in conjunction with modifications to the Armstrong Cork Dam located downstream on the Monatiquot River in the Town of Braintree, Massachusetts. The discharges were found to be essentially in agreement. Minor differences were due to effects of urbanization on discharges in the 25 years since the construction at the Armstrong plant.

The discharges for the streams passing through urbanized areas were based on adjusted record for the similar Mill Brook drainage area. An adjustment to the discharges along Glovers and Norroway Brooks were based on an analysis of rainfall data, discharge carrying capacity of the culverts, and available storage to account for the storage effect of Bear Swamp.

The discharges for the other major sub-watersheds in Randolph were determined using the following drainage area-discharge ratio formula:

$$(Q_1/ Q_2) = (D\,A_1/DA_2)^{n}$$

Where Q_1, and Q_2 are the discharges at specific locations; and DA_1 and DA_2 are the drainage areas at these locations with the exponent n varying from 0.70 to 0.80. For the New England areas, an average value of 0.75 was used (Reference 99). The discharges for minor sub-watersheds were determined using the Rational Method or the drainage area-discharge ratio formula. The discharges for Brook A (Stetson Brook) were computed using the Rational Method.

Hydrologic data were computed based on equations found in “Estimating Peak Discharges of Small Rural Streams in Massachusetts” (References 56, 90, and 94) for Chicken Brook, Hopping Brook, and the tributary to Great Black Swamp in Medway. Analytical relationships found in the above text were also used to compute discharges for upper portion of Trout Brook and Rocky Brook in Dover and for Mary Lee Brook in Randolph. In Medway, discharges for Chicken and Hopping Brook and the tributary to Great Black Swamp were then compared to discharges developed from the unit hydrograph theory. In Dover, the drainage basins of Trout Brook and Rocky Brook were considered rural, having a usable manmade storage less than 4.5 million cubic feet per square mile and having less than 10 percent of the area affected by urbanization. Rural peak discharges were computed for the 10-, 2- and 1-percent-annual-chance floods. The peak discharge for the 0.2-percent-annual-chance flood was extrapolated from the 50-, 20-, 10-, 4-, 2-, and 1-percent-annual-chance computed rural peaks. Data required for the analysis of the discharge-frequency relationship included the measurement of the respective drainage basin areas and an evaluation of the significance of usable manmade storage and urbanization. The significance of basin storage and urbanization was determined from a review of topographic maps and field reconnaissance (Reference 100). The hydrology for the lower portion of Trout Brook was obtained using peak discharge equations (Reference 90). These discharges were then compared to discharges developed from the unit hydrograph theory. In Randolph, peak discharges were computed for the 10-, 2- and 1-percent-annual-chance flood frequencies. The 0.2-percent-annual-chance peak discharge was extrapolated from the computed discharges. The peak discharges computed for Mary Lee Brook compared the equations described in "Flood Characteristics of Urban Watersheds in the United States" with a log-Pearson analyses performed on Old Swamp River in South Weymouth, Massachusetts, and Town Brook in Quincy, Massachusetts (Reference 101). Old Swamp River and Town Brook were chosen for comparison because their drainage areas are similar in size and character to the drainage area of Mary Lee Brook. The values for peak discharges were assessed to be reasonable. The three-parameter estimating equations described in Flood Characteristics of Urban Watersheds in the United States were used to transform the rural peak discharge to urban peak discharges (Reference 101). The basin development factor used in the calculations was 4.

The USDA NRCS Technical Release No. 20 computer program was used to develop discharge-frequency relationships for the Mill River, Miller Brook, the Stop River, Mann Pond Lateral, Prison Farm Lateral and Stony Brook in Norfolk (Reference 95); and the Stop River in Walpole (Reference 78). In Walpole, this material was previously developed for a FIS for the Town of Norfolk (Reference 17).

In Holbrook, sub-watershed boundaries for each stream were located on USGS topographic maps and their areas were determined (Reference 16). Times of concentration for each sub-watershed were based on travel times calculated from the watershed hydraulic characteristics. Soil-cover complex numbers were derived from a study of the 1926 soils map of Norfolk County and the land use map from the 1966 Holbrook Planning Study (References 103 and Reference 104). Flood frequencies were related to rainfall records published by the U.S. Weather Bureau (Reference 79). Tributary discharges were determined by flood routing various frequency l-day rainfalls using the USDA NRCS Technical Release No. 20 computer program and by the USDA NRCS tabular method of flood routing for some of the smaller areas (References 66 and 78). Discharges for the Cochato River in Holbrook and Randolph were later revised.

For the April 17, 1984, Town of Westwood FIRM, subwatershed boundaries were located on USGS 7.5-Minute Series Topographic Maps for each stream reach and the drainage areas measured. Time of concentration for each subwatershed was based on travel times calculated using the watershed hydraulic characteristics. Soil-cover complex numbers were derived from a study of the general soils map of Norfolk County and aerial photographs, combined with field observations. Flood frequencies were related to rainfall records using the U.S. Weather Bureau Publication Technical Paper 40 (Reference 79). Tributary discharges were determined by flood routing various frequency l-day rainfalls using the Project Formulation Hydrology (USDA NRCS Technical Release 20) computer program. Flood profiles for the l0-, 1-, and 0.2-percent-annual-chance floods were developed based on the combination of the routed discharge-frequency and the elevation-discharge relationships at selected cross sections. Flood profiles were prepared to a horizontal scale of 1"=800'.

Profiles for the Charles River in Westwood were extrapolated from data presented in the "Charles River Study Report" and other information prepared by the USACE, New England Division.

For the Town of Westwood 2002 FIS revision, the analyses of Bubbling Brook, Mill Brook, and Purgatory Brook were performed using regression equations for estimation of peak discharges for the 10-, 2-, and 1-percent-annual-chance probabilities provided for ungaged sites in Eastern Massachusetts (Reference 105). Multiple-regression techniques were used to develop these regression equations. Peak discharges for the 0.2-percent-annual-chance exceedance probability were extrapolated from peak discharges available for 10-, 2-, and 1-percent-annual-chance probabilities. In September 1990, for South Brook, the USACE HEC-l Computer Program was used to develop a rainfall computer model of the watershed (Reference 93). The Natural Resources Conservation Service (USDA NRCS) unit hydrograph was used within the HEC-l program to develop hydrographs for each subarea, and the NRCS lag formula was used to calculate lag time for each subarea. Hydrographs were routed through large flood storage areas and natural wetland areas using the Modified Puls method within the HEC-l program. The peak flows computed by HEC-l were verified using the Nationwide Urban Equations found in USGS WRI Report 94-4002 (Reference 106). Computer models were developed for existing conditions and for several alternative mitigation measures. Computer simulations were run for the 50-, 10-, 4-, 2-, 1-, and 0.2-percent-annual-chance storm events using rainfall depths from Technical Paper No. 40 (Reference 79).

Peak discharge-frequency estimates for Traphole Brook in Norwood were developed by the USDA NRCS in their studies on Diamond and Traphole Brooks (References 107 and 108) and their FIS in Walpole (Reference 24). These discharges were developed using the USDA NRCS computer program for Project Formulation-Hydrology, TR-20.

For Diamond Brook in Walpole, the hydrologic frequency-discharge analyses were updated due to the 1979 completion of a dam and reservoir with floodwater storage potential. The regional equation for Massachusetts developed by the USGS was used to determine flows along the stream (Reference 90). The flows were modified to account for floodwater storage provided by Allens Pond. A reservoir routing, incorporating USDA NRCS methodologies of Allens Pond, yielded flow reductions of 51 to 65 percent at the pond (Reference 109).

Prior to 1984, the Massachusetts Department of Public Works (MDPW) planned various improvements along Traphole Brook in Norwood and Walpole. The MDPW contracted with Schoenfeld Associates, Inc., in 1984 to assess the effect of such improvements on the watershed and to propose remedial measures, if necessary, to offset any increases in flood hazard (Reference 110). The discharges developed for Traphole Brook in that study, using Technical Release No. 20, were used in the 1988 Walpole FIS study.

Also in Walpole, discharges for Cobb’s Brook, Mine Brook, Pickerel Brook, and School Meadow Brook were obtained from the original FIS for the Town of Walpole (Reference 24). Data for Bubbling Brook and Willett Pond were obtained from the original FIS for the Town of Walpole using the FIS for the Town of Westwood (References 24 and 26).

In September 1990, the hydrologic analysis for the Cochato River in Braintree, Holbrook and Randolph was revised using the USACE HEC-l Computer Program (Reference 93). This analysis was performed to resolve discrepancies in the peak discharges for the Cochato River that occurred between the Town of Holbrook FIS and the Town of

Randolph FIS. Peak flows in the Cochato River were calculated for the 10-, 2-, 1- and 0.2-percent-annual-chance discharges

In Cohasset, peak discharges for the 10-, 2-, 1- and 0.2-percent-annual-chance floods for the riverine streams studied by detailed methods were computed using the USGS regional formula for estimating flood magnitude and frequency (Reference 90). This formula is based on an analysis of all gaging stations in eastern Massachusetts and is in the following form:

$$Q_n = C_1 A^{C2} S^{C3}$$

where Q_n is the peak discharge for recurrence interval "n" in cubic feet per second (cfs), A is the drainage area, S is the stream slope, and $C_{1,}$ C_2 and C_3 are coefficients specific to recurrence interval "n". Runoff volumes for Bound Brook were computed by the USDA NRCS tabular method (Reference 73). The watershed was divided into sub-areas, and drainage area, time of concentration (Tc) and travel time (Tt) were computed. A runoff curve number (RCN) was assigned to each sub-area based on soil and land-use characteristics. The 24-hour rainfall for the flood events was determined using Technical Paper No. 40 (Reference 79). Based on the RCN and the 24-hour rainfall, the runoff in inches was determined from tables prepared by the USDA NRCS (Reference 111). Hydrographs of flow in cubic feet per second per square mile (CSM) for each point were taken from tables prepared by the USDA NRCS, based on Tc and Tt.

To establish peak discharge-frequencies at the Plantingfield Brook gage in Norwood, USGS gage (#01105550) maintained at the U.S. Route 1 culvert since 1964, a log-Pearson Type III analysis was run on the gage record using a regional skew coefficient, according to guidelines set forth in the U.S. Water Resources Bulletin No. 17 (Reference 70). To establish peak flows for the 10-, 2-, 1- and 0.2-percent-annual-chance floods at other points on Plantingfield Brook, the following steps were employed. Peak flows for the 10-, 2-, and 1-percent-annual-chance flood were computed at the gage according to the USGS regional formula (Reference 63). The flows computed according to the USGS formula were found to be approximately one-half of the volume of those established by the gage record. Therefore, a coefficient of 2.04 for Q10, 1.84 for Q50, and 1.79 for Q100 were applied to the USGS formula for Plantingfield Brook in Norwood. Flows at points other than at the gage were then computed according to the USGS formula modified by the above coefficients. The 0.2-percent-annual-chance flows were determined by extrapolation of a log-probability graph of flood discharges computed for frequencies of up to 100 years. Norwood, being more urbanized than much of the region used to establish the USGS regional formula discharges, would be expected to produce higher discharges than the regional formula would produce for streams with small drainage areas. The coefficients established account for runoff characteristics particular to Norwood.

The watershed area contributing to Germany, Hawes, Meadow, and Purgatory Brooks in Norwood have characteristics of runoff similar to those of Plantingfield Brook, including the degree of urbanization, area of wetlands, and soil types. Therefore, peak discharge estimates for the 10-, 2-, and 1-percent-annual-chance flood were computed by the USGS regional formula for these streams, and the coefficients established for Plantingfield Brook were applied. The 0.2-percent-annual-chance peak discharge was determined by extrapolation of a log-probability graph of flood discharges computed for frequencies of up to 100 years.

In Sharon, on Beaver Brook, Billings Brook, Billings Brook Branch, Canoe River, Massapoag Brook, Massapoag Lake, and Sucker Brook peak discharge frequency estimates for the 10-, 2-, and 1-percent-annual-chance floods were determined by the USGS regional formula (Reference 63). The 0.2-percent-annual-chance discharge was determined by extrapolation of a log-probability graph of flood discharges computed for frequencies of up to 1-percent-annual-chance.

Discharge-frequency relationships for all streams in Wrentham were developed using regional equations prepared by the USGS. These equations relate streamflow to the parameters of drainage area, main channel slope, and mean annual precipitation (Reference 90).

Peak discharges in Bellingham for Beaver Brook, Arnolds Brook, Bungay Brook, and Hopping Brook were developed through the use of regional equations for eastern Massachusetts (Reference 90).

Discharge-frequency relationships for Bogastow Brook in Millis were developed using a method developed by the USGS specifically for Massachusetts (Reference 105). The method takes into consideration both the slope of the main channel and the drainage area in its evaluation.

Peak discharges for Mine Brook and Shepards Brook in Franklin were defined using regional equations prepared by the USGS (Reference 83). These regional equations relate stream flow to parameters of drainage area, main channel slope, and mean annual precipitation.

Discharge-frequency relationships for Cress Brook, Myrtle Street Lateral, and Harlow Pond Lateral in Norfolk were developed using the Regional Frequency Method (Reference 112).

A discharge-frequency drainage area relationship was developed for Vine Brook in Medfield by using hydrologic methods developed by the USDA NRCS (References 113 and 111). This methodology bases flood flows on basin characteristics, such as drainage area, basin slope, soil type, land use and precipitation duration and intensity. The total drainage area for Vine Brook is 1.2 square miles at its confluence with the Charles River.

Peak discharges for Town Brook in Braintree were determined by the USACE in September 1989 (Reference 82).

The 10-, 2-, 1- and 0.2-percent-annual-chance discharges for Furnace Brook, Town Brook, and Cunningham Brook in Quincy were determined by the USACE (References 33 and 54).

In Bellingham, peak discharges for the Peters River at the Woonsocket boundary were taken from the Flood Insurance Study for the City of Woonsocket, Rhode Island (Reference 89). Peak discharges for the upstream portion of the Peters River in Bellingham were calculated by applying the ratio of drainage areas raised to the 0.7 power.

In Milton, discharge-frequency-drainage area relationships for Pine Tree Brook were developed from information received from the Amherst office of the USDA NRCS

(Reference 40). These flows were utilized by the USDA NRCS in 1970 and were checked and found to be in agreement with current USDA NRCS criteria (Reference 66).

In Plainville, with the exception of the Ten Mile River and the Whiting Pond Bypass, peak discharges for floods with 10-, 2-, 1- and 0.2-percent-annual-chance recurrence intervals were estimated by use of formula developed by S. William Wandle, Jr. (Reference 90). Discharges for the Ten Mile River and the Whiting Pond Bypass were estimated by the USDA NRCS during the preparation of an Federal Insurance Agency (currently FEMA) Type 15 study of the adjoining Town of North Attleborough (Reference 116). Near the corporate limits, peak discharge of the Ten Mile River does not relate to drainage area because of flow diversion into the Whiting Pond Bypass.

In Wellesley, by using the hydrologic methods developed by the USDA NRCS (References 8 and 117) and the 1974 USGS (Reference 73), discharge-frequency relationships were developed for the remaining detailed study areas. These methodologies base flood flows on basin characteristics, such as drainage area, basin slope, land use, and precipitation duration and intensity. The total drainage area for Morses and Paintshop Ponds and Lake Waban are 8.8, 8.9, and 10.9 square miles, respectively. Inflows were calculated for the various flood frequencies and were routed through the ponds, using a standard routing methodology (Reference 118). Flooding of the lower Waban reaches of Fuller and Waban Brooks and Lake Waban in its entirety is caused by the elevated water-surface of the Upper Charles River. This backwater condition causes higher water-surface elevations on the brooks than the natural drainage from the brooks' own tributary areas.

For the approximately studied area in Dedham, a method was developed by the Water Resources Division of the USGS (Reference 63), to determine peak discharges for a selected recurrence interval from an ungaged drainage basin. This method was developed after many years of monitoring an extensive number of gaged streams throughout the Commonwealth of Massachusetts. The results of this study indicate flood peaks for any stream may be estimated from knowledge of the drainage characteristics of the area, main channel slope, and the mean precipitation of the basin.

Partial Countywide Analyses

No new riverine hydrologic analyses were performed for the July 17, 2012 partial countywide FIS.

Peak discharge-drainage area relationships for Norfolk County are shown in Table 11.

TABLE 11 – SUMMARY OF DISCHARGES

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
ARNOLDS BROOK					
At the confluence with the Peters River	1.40	100	170	210	330
At confluence with Brockton Reservoir	2.20	205	330	395	660
At Old Pond Street	2.10	202	326	386	647
At State Highway 24 exit ramp	2.00	196	316	373	624
At New Pond Street	1.90	190	300	340	604
At State Highway 24 entrance ramp	1.60	181	283	329	578
At Stockwell Drive	1.10	105	162	188	299
At Old Railroad Grade near Avon/Stoughton corporate limits	0.90	92	140	162	257
BEAVER BROOK (Bellingham)					
At the confluence with the Charles River	2.60	70	130	160	220
At the Taunton Street	2.10	60	110	140	190
BEAVER BROOK (Holbrook)					
At Holbrook/Weymouth corporate limits	1.80	790	*	1350	1760
At Plymouth Street	0.90	300	*	500	650

*Data Not Available

TABLE 11 – 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 (Sharon)					
At Upland Road	1.95	104	156	179	249
BEAVER MEADOW BROOK					
At Bolivar Pond Outlet	9.70	805	525	1890	2680
At Bolivar Pond	3.10	270	600	750	1070
BILLINGS BROOK					
Cranberry Bogs (Mile 1.8)	1.10	51	72	80	105
BILLINGS BROOK BRANCH					
950 feet from Main Branch (Cross Section A)	1.40	63	87	95	122
BOGASTOW BROOK					
At confluence with Charles River	19.20	475	780	940	1420
At Bogastow Pond Outlet	17.60	505	835	1010	1530
At Orchard Street (Schoolhouse Hill)	14.80	460	760	925	1400
At Orchard Street (Golf Course)	13.10	450	750	910	1390
BOUND BROOK					
At Turtle Island in Cohasset	0.49	68	101	116	160

TABLE 11 – 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
BROOK A	*	*	*	*	*
BROOK B	*	*	*	*	*
BROOK NO. 1					
At Lake Mirimichi	12.00	400	670	810	1230
At inlet to Lake Mirimichi	5.83	220	360	440	670
At Interstate Route 495	4.55	170	280	340	520
BUBBLING BROOK					
At Brook Street	3.56	166.8	268.0	321.9	500
At Pettees Pond Lane	0.55	48.5	80.1	97.5	165.0
At North Street	0.29	31.6	52.8	64.5	106.5
BUNGAY BROOK					
At the confluence with the Peters River in Bellingham	4.10	210	360	440	680
4,000 feet upstream of the confluence with the Peters River	2.90	180	320	390	600
BURNT SWAMP BROOK					
At Wrentham/ Cumberland, Rhode Island, corporate limits	4.60	200	330	410	630
At Burnt Swamp Road	3.50	180	257	370	567

*Data Not Available

TABLE 11 – 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
BURNT SWAMP BROOK - Continued					
At West Street	2.60	140	233	290	442
At Terminus of Study 1,700 feet upstream of West Street in Wrentham	1.00	120	200	250	380
CANOE RIVER (Foxborough)					
At Beaumont Pond	2.50	136	205	234	363
At East Street	2.30	133	200	229	354
Approximately 1,900 feet upstream of East Street	2.10	125	190	215	336
At Willow Street	2.00	122	182	208	322
Approximately 1,450 feet upstream of Willow Street	1.80	113	167	191	295
CANOE RIVER (Sharon)					
Approximately 900 feet from Sharon/Foxborough corporate limit	1.48	71	99	111	146
CANTON RIVER					
At gaging station	27.20	930	1910	2570	4980
CAROLINE BROOK					
At Wellesley	0.26	*	*	480	*

*Data Not Available

TABLE 11 – 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
CHARLES RIVER (LOWER REACH)					
At Wellesley	216.00	2200	2900	3500	4500
At Wellesley/Needham corporate limits (USGS Wellesley gage)	211.00	1965	2660	2990	3825
At Dedham Corporate boundary	200.00	1534	2493	3019	4585
Mother Brook diversion	198.20	2301	3740	4528	6878
Downstream of Mother Brook Division Channel	198.00	1780	2480	3200	4270
Upstream of Mother Brook Division Channel	198.00	2650	3610	4680	6210
Route 128	192.70	2267	3685	4461	6776
At the Charles River Village gage	184.00	2500	3500	4500	6000
At Medfield	156.00	2450	3430	4410	5925
At Medfield/Dover/ Sherborn Corporate Limits	145.00	2450	3430	4410	5925
CHARLES RIVER (UPPER REACH)					
At Myrtle Street in Norfolk	85.90	1900	2500	3500	5100
At the Medway/Norfolk/ Franklin town boundary	66.00	1900	2500	3500	5100
Upstream of Medway Dam	65.00	1200	2300	3100	4700

TABLE 11 – 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
CHARLES RIVER (UPPER REACH) - Continued					
Upstream of West Medway Dam	52.60	1056	2024	2728	4136
At the Medway/Bellingham/ Franklin corporate limits	36.10	990	1860	2540	3830
Upstream of confluence with Hopping Brook	24.20	740	1400	1910	2870
At confluence with Stall Brook	20.20	650	1230	1670	2520
At Interstate 495	19.50	640	1210	1640	2470
At confluence with Beaver Brook	13.80	460	860	1180	1800
At the head of Box Pond	12.90	430	820	1130	1780
At Billingham/Milford upstream corporate limits	11.90	430	800	1130	1670
CHICKEN BROOK					
At confluence with Charles River in Medway	7.33	300	500	600	900
Upstream of Park Pond Dam	6.80	284	473	568	851
Upstream of Milk Pond Dam	6.35	270	450	540	810
Upstream of Milk Pond	5.73	250	415	500	750

TABLE 11 – 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
COBB'S BROOK					
At confluence with Neponset River in Walpole	1.90	108	128	163	704
COCHATO RIVER					
At the confluence with Monatiquot River	11.43	864	1405	1924	3000
Upstream of confluence with Glover's Brook	6.24	642	990	1375	2748
COCHATO RIVER - Continued					
At Randolph/Holbrook corporate limits	4.34	376	629	963	1965
At Lake Holbrook	2.52	155	274	488	1088
CRESS BROOK					
At confluence with Mill River	1.80	115	200	245	385
At Lake Street	0.20	25	45	60	90
CROCKER BROOK					
2,000 feet downstream of East Street	2.10	80	130	150	240
At Railroad	1.40	60	100	120	190
CUNNINGHAM BROOK					
At Wallace Road	0.75	128	190	210	260

TABLE 11 – 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
DIAMOND BROOK					
At confluence with Neponset River	2.10	160	300	380	500
At Diamond Pond Dam	1.60	130	250	330	420
At Washington Street	1.10	110	220	290	360
DORCHESTER BROOK					
At Stoughton/Easton Corporate limits	2.10	192	304	357	586
At Atkinson Avenue	1.50	154	233	266	417
FARM RIVER					
** in Braintree	12.00	570	1100	1300	2700
** in Braintree	10.00	500	920	1200	2400
FULLER BROOK					
At Wellesley	2.83	*	*	820	*
FURNACE BROOK					
At Hancock Street	3.18	630	880	970	1090
At Newport Avenue	3.01	570	830	910	970
At Adams Street	2.43	460	640	710	830
At Crescent Street	1.52	140	165	173	190

*Data Not Available
**Locations not available in 1977 FIS

TABLE 11 – 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
GERMANY BROOK					
At 100 feet downstream of Nichols Street	2.40	250	342	383	497
At 70 feet upstream of Westover Parkway	1.80	200	269	299	382
GLOVERS BROOK	*	*	*	*	*
GREAT POND TRIBUTARY					
At Holbrook/Weymouth corporate limits	0.50	140	*	250	325
HARLOW POND LATERAL					
At confluence with Charles River	1.40	90	150	185	285
At 4th Dam on Brook	0.10	22	26	30	36
HAWES BROOK					
At 140 feet upstream of Washington Street	8.80	778	1152	1342	1873
Hawthorne Brook at Inlet to Turnpike Lake	1.61	100	170	210	320
HERRING BROOK					
At confluence with Weymouth Back River	15.10	615	1006	1203	2032
At Bituminous Road	14.70	598	981	1176	1993
At Railroad	14.60	590	970	1160	1965

*Data Not Available

TABLE 11 – 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
HERRING BROOK - Continued					
Approximately 300 feet upstream of Railroad	14.50	583	956	1145	1934
At Broad Street	14.10	567	924	1104	1858
Weir just downstream of Commercial Street	14.00	566	922	1099	1846
At Commercial Street	14.00	565	920	1096	1835
At Pleasant Street	13.90	563	915	1088	1818
At Ironhill Street	13.50	539	875	1044	1751
Approximately 300 feet upstream of Ironhill Street	13.40	537	872	1040	1743
HOPPING BROOK					
At confluence with Charles River	11.40	400	670	1000	1800
At south of Main Street	10.71	350	600	900	1400
JAMES BROOK					
At confluence with Cohasset Cove	1.31	76	130	159	249
LAKE WABAN					
At Wellesley	10.90	100	145	170	235
LILY POND STREAM					
At confluence with Lily Pond	0.49	50	87	107	171

TABLE 11 – 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
MANN POND LATERAL					
At confluence with Stop River	1.40	100	*	330	570
At Railroad Culvert	1.10	86	*	137	202
At Boardman Street	0.70	120	*	280	405
MARTIN BROOK	*	*	*	*	*
MARY LEE BROOK					
At confluence with Cochato River	1.40	160	205	245	285
Just upstream of confluence of Unnamed Tributary to Mary Lee Brook	1.17	140	180	215	255
MASSAPOAG BROOK (Canton)					
Just downstream of the Silk Mill Pond Dam	10.35	140	230	270	530
At upstream Sharon/Canton corporate limits	9.90	400	710	880	1250
MASSAPOAG BROOK (Sharon)					
Just downstream of Footbridge	5.58	265	426	505	748
Confluence with Devil Brook	4.73	233	372	439	646

*Data Not Available

TABLE 11 – 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
MASSAPOAG BROOK (Sharon) - Continued					
Confluence with Sub-Branch of Massapoag Brook	3.94	200	315	371	541
Massapoag Lake	3.55	195	312	368	541
MEADOW BROOK					
Confluence with Neponset River	1.50	183	249	278	359
Downstream of U.S. Route 1	1.20	147	197	216	274
MILL BROOK					
At Brook Road	3.02	149.4	240.7	289.4	425.0
At Winslow Road	2.03	114.9	186.2	224.5	350.0
At Tamarack Road	1.82	107.2	173.9	209.8	331.8
At High Street	1.33	87.2	142.2	172.0	275.0
At Hartford Street	0.83	63.8	104.7	127.0	200.0
MILL RIVER (Norfolk)					
At confluence with Charles River	16.80	500	*	1153	1707
At Miller Street	13.40	249	*	580	877
At City Mills Pond Dam (Main Street)	10.70	180	*	325	447
At Railroad Culvert	10.40	290	*	685	1009
At Bush Pond Dam	9.40	175	*	264	590

*Data Not Available

TABLE 11 – 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
MILL RIVER (Norfolk) - Continued					
Opposite Maple Street	9.00	238	*	533	774
MILL RIVER (Weymouth)					
Approximately 1,750 feet downstream of West Street	5.80	239	365	423	668
At West Street	5.70	238	364	422	666
Approximately 1,150 feet upstream of West Street	5.60	237	363	421	664
MILL RIVER (Weymouth) – Continued					
At Railroad	4.40	190	300	340	532
Approximately 400 feet upstream of confluence with Mill River Tributary A	3.50	146	220	252	389
Approximately 550 feet downstream of Hollis Street / Randolph Street	2.80	139	206	241	369
At Hollis Street / Randolph Street	2.40	136	203	232	357

*Data Not Available

TABLE 11 – 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
MILL RIVER TRIBUTARY A					
At Randolph Street	0.80	62	88	98	143
At Gravel Road	0.70	60	84	92	138
At Pond Street	0.60	53	61	73	103
At Railroad	0.60	51	58	69	98
At Main Street	0.40	31	42	49	70
Approximately 630 feet downstream of State Highway 18	0.30	22	33	43	60
MILL RIVER TRIBUTARY B					
At Railroad	0.10	10	16	20	28
MILLER BROOK					
Confluence with Mill River	1.70	195	*	450	660
At Main Street	1.50	190	*	440	650
MINE BROOK (Franklin)					
Upstream of the confluence with Charles River	14.90	500	830	1010	1540
Upstream of Beech Street	13.50	460	770	940	1430
Upstream of Interstate 495	9.40	360	600	730	1110

*Data Not Available

TABLE 11 – 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
MINE BROOK (Franklin) - Continued					
Upstream of Beaver Street	7.70	310	520	630	960
Upstream of Spring Pond	2.60	150	250	300	460
MINE BROOK (Walpole)					
At confluence with Neponset River	7.20	175	325	450	1025
At downstream crossing of Mill Pond Road	7.10	175	350	525	1150
At Railroad bridge	6.00	175	350	500	1050
At Walpole/Medfield corporate limits	5.00	400	500	575	950
MONATIQUOT RIVER					
** in Braintree	29.70	1020	1900	2200	4800
** in Braintree	27.80	990	1800	2150	4700
** in Braintree	25.50	930	1700	2100	4400
**in Braintree	24.00	900	1600	2000	4200
MORSES POND					
At Wellesley	8.80	125	175	210	285
MOTHER BROOK					
Diversion Dam From Charles River	198.20	767	1247	1509	2293

**Locations not available in 1977 FIS

TABLE 11 – 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
MYRTLE STREET LATERAL					
At confluence with Charles River	0.90	60	100	120	190
NEPONSET RIVER					
Dam at Lower Mills	*	2450	3410	3730	4750
Dedham/Boston corporate limits	93.00	552	1401	1945	5500
At Interstate Route 95 bridge	*	*	*	1510	*
Canton/Dedham Street Bridge	*	*	*	1790	*
At Greenlodge Street Bridge	*	*	*	1960	*
Dedham Street	86.30	820	1440	1786	2850
Upstream of Purgatory Brook (Section C)	79.70	1010	1730	2050	3350
Downstream of 1-95 Interchange near Canton/Norwood corporate limits	78.20	1030	1800	2070	3450
Neponset Street	76.50	1060	1850	2254	3550
Upstream crossing of I-95	41.90	720	1150	1508	2300
Upstream of Traphole Brook	38.10	633	1098	1354	2189
USGS gaging station (Section Y)	35.20	609	1020	1260	1980

*Data Not Available

TABLE 11 – 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
NEPONSET RIVER - Continued					
Upstream of Hawes Brook confluence	26.20	463	786	958	1515
At Walpole/Norwood downstream corporate limits	25.80	700	1025	1225	2575
At Washington Street	25.70	700	1025	1225	2550
NEPONSET RIVER – Continued					
At Bird and Son Co. Dam	25.70	695	1032	1234	2565
At Plimpton Pond Dam	24.90	683	1024	1235	2527
At State Route 1A	22.90	575	900	1100	2350
At Stetson Pond Dam	22.20	574	906	1114	2336
At Elm Street	10.60	300	475	550	1025
At South Street	*	261	416	498	1050
At Summer Street	3.50	232	456	570	928
NORWAY BROOK	*	*	*	*	
OLD SWAMP RIVER					
At Libbey Industrial Parkway	4.90	241	360	422	657
At State Route 3 Northbound lane	4.70	222	336	389	608

*Data Not Available

TABLE 11 – 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
OLD SWAMP RIVER - Continued					
Approximately 800 feet downstream of Pleasant Street	4.10	190	288	334	537
At State Route 3 southbound lane	4.00	183	273	313	480
At Pleasant Street	4.00	182	272	310	475
Approximately 750 feet upstream of Pleasant Street	3.90	180	270	308	472
At Elm Street	3.80	179	267	305	469
Approximately 1,150 feet downstream of Talbot Street	3.60	170	254	300	453
At Talbot Street	3.40	160	239	289	437
Approximately 950 feet downstream of Ralph Talbot Street	3.10	147	220	268	396
At Ralph Talbot Street	3.00	143	212	250	375
Approximately1,400 feet upstream of Ralph Talbot Street	2.90	140	206	235	356
PAINTSHOP POND					
At Wellesley	8.90	125	175	210	285
PEQUID BROOK (LOWER REACH)					
At Reservoir Pond	6.23	180	190	210	300

TABLE 11 – 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
PEQUID BROOK (UPPER REACH)					
At unnamed bridge approximately 1,050 feet upstream of Turnpike Street	4.02	220	410	480	720
PETERS RIVER					
At the downstream corporate limits of Providence County, RI/Norfolk County, MA	12.50	750	1150	1600	2600
At confluence of Arnolds Brook	10.50	670	1020	1420	2310
At the confluence of Bungay Brook	6.40	470	720	1010	1640
Upstream of Jenks Reservoir	5.60	430	660	910	1480
At the Pulaski Boulevard Bridge	4.20	350	540	750	1220
At the Abandoned Railroad Bridge	4.20	350	540	750	1220
At confluence of a tributary upstream of the New York/New Haven and Hartford Railroad	2.80	260	400	560	910
PICKEREL BROOK					
At confluence with Traphole Brook	*	80	330	430	740
At Walcott Avenue	0.80	86	386	471	816

*Data Not Available

TABLE 11 – 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
PINE TREE BROOK					
At Confluence with Neponset River	8.28	550	870	1030	1400
At School Street	8.15	460	750	900	1300
At Thatcher Street	7.33	370	640	760	1125
At Elm Street	7.15	305	530	640	950
At Blue Hill Parkway	6.98	250	410	480	690
PLANTINGFIELD BROOK					
USGS crest-stage gage station at U.S. Route 1 culvert	1.52	190	258	290	374
Downstream of 1,400-foot culvert, approximately 1,950 feet upstream of U.S. Route 1	1.20	149	196	219	279
Upstream of 1,400-foot culvert	1.02	130	171	188	235
PONKAPOAG BROOK					
At confluence with Neponset River	3.36	165	265	320	470
PRISON FARM LATERAL					
At Needham Street	0.60	110	*	260	370

*Data Not Available

TABLE 11 – 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
PURGATORY BROOK					
Downstream of U.S. Route 1	2.90	307	432	487	647
Downstream of Everett Street	2.50	265	366	370	454
At Everett Street	4.42	192.2	307.9	369.2	600.0
At confluence with South Brook	4.12	183.8	294.7	353.5	570.0
At Washington Street	1.69	102.0	165.8	200.2	300.0
At Gay Street	1.06	74.9	122.6	148.5	250.0
RABBIT HILL BROOK					
At Wrentham/Plainville corporate limits	3.70	160	260	310	480
At Myrtle Street	3.00	140	220	270	410
RATTLESNAKE RUN					
At confluence with Straits Pond	0.54	42	73	90	141
REDWING BROOK					
At Stoughton/Canton corporate limits	1.70	164	260	304	502
At York Street and Meadow Brook Lane	1.40	120	187	217	351
At Pine Street	1.10	89	136	157	248

TABLE 11 – 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
RICHARDSONS BROOK					
At confluence with Little Harbor	0.29	31	54	66	106
ROBINSON BROOK					
At Foxborough/ Mansfield Corporate Limits	2.40	162	249	289	456
At Commercial Street	2.30	159	242	281	440
ROBINSON BROOK – Continued					
Approximately 60 feet upstream of Foxborough Boulevard	2.30	156	236	272	428
Approximate 1,000 feet downstream of Interstate 95	2.20	152	228	265	415
At Interstate Route 95	2.10	148	224	259	405
At Walnut Street	1.90	141	216	248	384
Approximately 400 feet upstream of Hershey Pond	1.60	127	197	225	352
At Central Street	1.40	118	186	212	339
Just downstream of Cocasset Street	0.80	*	*	128	*
Approximately 1,800 feet upstream of Cocasset Street	0.33	*	*	56	*

*Data Not Available

TABLE 11 – 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
ROCKY BROOK					
At confluence with Trout Brook	0.65	52	89	109	197
RUMFORD RIVER					
At Foxborough/Mansfield Corporate Limits	3.80	234	375	443	736
At Spring Street	3.70	229	367	432	719
At 1st Private Road	3.50	222	354	417	691
At 2nd Private Road	3.40	219	350	409	684
Approximately 600 feet upstream of Private Road	3.40	218	348	405	680
Approximately 4,400 feet downstream of Cocasset Street	3.30	215	346	400	670
Approximately 1,920 feet downstream of Cocasset Street	3.30	214	344	399	665
Approximately 1,600 feet downstream of Cocasset Street	3.30	213	342	398	660
At Cocasset Street	3.20	210	337	395	655
At Sand Street	3.10	208	331	387	640
Approximately 500 feet upstream of Sand Street	2.60	206	326	382	625

TABLE 11 – 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 MEADOW BROOK					
At confluence with Neponset River	3.10	80	110	120	380
At Washington Street	2.90	110	220	290	580
At U.S. Route 1	1.70	100	190	230	400
SHEPARDS BROOK					
Upstream of the confluence with Charles River in Franklin	4.40	240	400	490	760
SOUTH BROOK					
At confluence with Purgatory Brook	1.15	195	325	380	530
Upstream of confluence with tributary from subarea 2	0.87	165	285	335	480
Downstream of Southwest Park	0.54	105	180	210	320
Downstream of Boston Providence Turnpike	0.44	100	165	195	295
STEEP HILL BROOK					
At Stoughton/Canton corporate limits	5.30	583	1004	1225	2193
At Erin Road	5.00	535	904	1110	1947
At Mill Street	4.30	497	846	1026	1814
At Pratt Court	2.40	365	603	722	1236

TABLE 11 – 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
STEEP HILL BROOK - Continued					
At Southworth Pond Dam	2.00	328	536	639	1084
At Sheehan Street	0.80	120	184	220	341
STONY BROOK					
At confluence with Stop River	3.60	210	*	460	590
At Stony Brook Pond Dam	3.20	190	*	470	660
At Diamond Street	2.00	170	*	370	540
At Union Street	1.10	130	*	280	420
At Mirror Lake Avenue	0.40	4	*	8	11
STOP RIVER					
At Walpole/Norfolk corporate limits	12.60	360	6.50	820	1200
At Highland Lake Dam	10.10	150	*	240	290
At Main Street	8.50	170	270	330	470
At Winter Street	7.30	160	250	290	380
At Prison Road	7.00	160	*	290	350
Above Confluence with Stony Brook	3.40	470	*	580	870
At Dedham Street (SR Rte 1A)	1.70	315	*	725	1060
At Pine Street (SR 115)	1.00	200	*	460	670

*Data Not Available

TABLE 11 – 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
SUCKER BROOK					
At confluence with Massapoag Lake	1.10	63	92	104	141
TEN MILE RIVER					
At Plainville downstream corporate limits	4.23	86	150	200	390
At confluence with Whiting Pond Bypass	3.48	94	180	230	420
TOWN BROOK					
At USGS gage between Bigelow Street and Miller Stile Road	4.46	480	650	730	970
At upstream end of Railroad	4.04	330	410	450	530
At Braintree/Quincy corporate limits	2.35	356	420	440	495
Downstream of Common Street	2.22	255	325	360	445
At Worthington Circle	1.92	125	200	240	355
At Acorn Street	1.67	15	75	160	280
At Walnut Street	1.56	15	70	140	245
At upstream inlet of Old Quincy Reservoir	1.22	350	468	525	670
Upstream of Granite Street	0.56	200	268	300	384
Downstream of Wood Road	0.25	115	152	170	216

TABLE 11 – 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
TRAPHOLE BROOK					
Sumner Street	3.40	400	1050	1390	2350
U.S. Route 1	2.90	381	1000	1330	2238
At Norwood/Walpole downstream corporate limits	2.90	486	1159	1404	2229
At Union Street	2.10	422	848	1034	1574
At Coney Street	2.10	434	857	1042	1575
At U.S. Route 1	1.90	518	972	1130	1642
TRIBUTARY C2					
At English Road	0.90	200	*	350	500
At Kleen Road	0.30	100	*	360	510
TRIBUTARY C2B					
At Railroad Culvert 32	0.50	295	*	605	850
At Woodlawn Road	0.30	180	*	360	510
TRIBUTARY OF GREAT BLACK SWAMP					
At upstream Millis/Medway corporate limits	1.16	130	200	300	400
Approximately 3,350 feet upstream of State Route 109	0.61	60	90	110	170

*Data Not Available

TABLE 11 – 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
TRIBUTARY R1					
At confluence with Trout Brook	0.60	200	*	400	550
At State Route 37 (South Franklin Street)	0.40	100	*	250	350
TRIBUTARY R2					
At confluence with Trout Brook	0.50	150	*	300	400
At Dean Street	0.40	100	*	200	300
TRIBUTARY R3					
At confluence with Trout Brook	0.40	100	*	250	300
TRIBUTARY R4					
At confluence with Trout Brook	0.20	150	*	250	350
TRIBUTARY TO STEEP HILL BROOK					
Town Pond Dam at Pratt Court	1.70	149	235	275	450
TROUT BROOK (Avon)					
At Avon/Brockton corporate limits	1.90	283	462	550	932
Connelly Road	1.4	226	365	444	740
Approximately 850 feet upstream of Connelly Road	1.00	173	274	321	529
At Ladge Drive	0.90	151	245	287	464

*Data Not Available

TABLE 11 – 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
TROUT BROOK (Dover)					
At confluence with the Charles River	4.29	200	325	480	800
At Haven Street	3.50	165	265	318	570
TROUT BROOK (HOLBROOK)					
At Spring Road	0.30	300	*	550	750
At Braintree/Holbrook corporate limits	1.10	300	*	600	800
TUMBLING BROOK TRIBUTARY					
At Braintree/Holbrook corporate limits	1.10	300	*	600	800
TURTLE HILL RUN					
At confluence with Straits Pond	1.40	89	152	187	292
TURTLE BROOK					
At Mirimichi Street	5.29	260	440	540	830
Above confluence with Sawmill Brook	3.50	145	215	285	495
At Shepard Street	1.88	110	190	235	365
UNNAMED TRIBUTARY TO MARY LEE BROOK	*	*	*	*	*

*Data Not Available

TABLE 11 – 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
UNNAMED TRIBUTARY TO ROBINSON BROOK					
Just upstream of confluence with Tributary to Robinson Brook	0.42	*	*	72	*
VINE BROOK					
At State Route 109	1.11	90	145	165	225
At Upham Road	1.00	45	70	80	110
WALNUT HILL STREAM					
At confluence with The Gulf	0.45	44	76	94	149
WHITING POND BYPASS					
At North Attleborough/Plainville downstream corporate limits	0.01	27	52	65	110

*Data Not Available

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.

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.

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 flooding sources studied by approximate methods, only 1-percent-annual-chance flood elevations were computed.

For each community within Norfolk County that has a previously printed FIS report, the hydraulic analyses described in those reports have been compiled and are summarized below. Due to levee de-accreditation status at the time this FIS was finalized, the Town of Canton was not included within this Partial Countywide Study; Data related to the Town of Canton remains in this FIS report for informational purposes only, and users should refer to the separately published FIS report and FIRMs for effective data.

<u>Pre-partial Countywide Analyses</u>

In Avon, Bellingham, Dedham, Foxborough, Franklin, Medfield, Medway, Milton, Plainville, Stoughton, Wrentham, flood profiles were drawn showing computed water-surface elevations to an accuracy of 0.5 foot for floods of the selected recurrence intervals.

In Holbrook, 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. For construction and/or floodplain management purposes, users are encouraged to use the flood elevation data presented in this FIS in conjunction with the data shown on the FIRM.

A profile base line is shown on the maps to represent channel distances as indicated on the flood profiles and floodway data tables along certain portions of and Diamond Brooks in Walpole, Buckmaster Brook and Mill Brook in Westwood, Town Brook in Braintree and the Cochato River in Braintree and Randolph.

No profile is published for Tumbling Brook/Tumbling Brook Tributary in Holbrook since it is all affected by backwater from the Cochato River. No profile is published for the restudied portion of Tributary C2 since it is all affected by backwater from Lake Holbrook and the Cochato River. Also, insufficient data were found to utilize profiles for Great Pond Tributary and Tributary Rl.

In Avon, Braintree, Foxborough, Plainville, Walpole, and for the Neponset River in Milton, Westwood, Weymouth, cross sections for the flooding sources studied by detailed methods were obtained from field surveys.

All bridges and culverts in Avon, Braintree, Cohasset, Dover, Foxborough, Franklin, Medway, Norfolk, Plainville, Westwood, Weymouth (Reference 112) and Wrentham were field surveyed to obtain elevation data and structural geometry. All bridges, dams, and culverts in Millis and Randolph were field checked to obtain elevation data and structural geometry. In Bellingham, bridge plans were utilized to obtain elevation data and structural geometry. All bridges for which plans were unavailable or out of date were surveyed.

In Avon, Foxborough and Weymouth, cross sections were taken at close intervals upstream, downstream and along the centerline of each obstruction to compute representative flood profiles. Cross sections between obstructions in Avon and Foxborough were taken at intervals of 0.25 mile or less.

In Braintree, surveyed cross sections by the USACE were augmented by previously defined cross sections furnished by the USDA NRCS and from the Draft FIS for the City of Randolph (References 120 and Reference 121). For Town Brook, cross sections for the culvert under State Route 3 were taken from construction drawings (Reference 122). Overbank extensions of field surveyed cross sections, and additional sections needed for hydraulic continuity were taken from topographic maps (Reference 123).

In Bellingham, the valley portions of the cross-section data for the detailed study streams were obtained from topographic maps (Reference 121). The below-water sections in Bellingham, Millis and Norfolk were obtained from field measurement.

In Canton, cross sections for the Neponset and Canton Rivers were obtained from the previous Flood Insurance Study for the Town of Canton (Reference 5).

In Canton and for the Neponset River in Milton, the cross sections were placed at specific intervals along the river channels to enable hydraulic properties to be accurately modeled by the computer. Sections were interpolated between certain surveyed sections as deemed necessary. These interpolated sections were prepared from survey data, along with the aid of topographic mapping at a scale of 1:4,800 with a contour interval of 4 feet in Canton (Reference 124) and topographic mapping in Milton (Reference 125). In addition, cross sections for Pine Tree Brook in Milton were taken by the USDA NRCS in 1966 (Reference 126).

In Walpole, cross sections were located at typical valley sections and restrictions such as roads, bridges, and dams. Stream profiles were developed from the survey data using valley lengths from new detailed topographic maps at a scale of 1:4,800 with a contour interval of 5 feet (Reference 127).

In Dover, cross-section data for the Charles River and the lower portion of Trout Brook were obtained by field measurement and topographic maps (scale 1:2,400, contour interval 5 feet) compiled from aerial photographs (Reference 128). Cross-section data for the upper portion of Trout Brook and Rocky Brook in Dover were obtained from topographic maps (scale 1:4,800, contour interval 4 feet) compiled from aerial photographs (Reference 129). The geometry of the active stream channel at selected sections, culverts, and road crossings was determined by a field survey in January 1985.

In Needham, cross section data for the Charles River were taken from the Flood Insurance Studies for the Towns of Wellesley, Dover, Westwood, and Dedham; and the Cities of Newton and Boston (References 25, 8, 26, 7, 60, 11 and 82).

Cross-section data were obtained from field measurement and aerial photographs in Franklin (Reference 130), Medway and Wrentham (Reference 131). Cross sections for Franklin, Medway and Wrentham were located using guidelines prepared by the USACE (Reference 132).

Cross sections for the backwater analysis of detailed study areas were field surveyed in Dedham, Medfield, Norwood and Wellesley; and for Ponkapoag Brook, Lower Pequid Brook, Upper Pequid Brook, Massapoag Brook, Beaver Brook, and Meadow Brook in Canton; for Randolph (except for Mary Lee Brook) and for Furnace Brook in Quincy.

In Cohasset, cross-section data for the backwater analysis were obtained from field surveys and from topographic maps compiled from aerial photographs (Reference 126). Also in Dedham, sections were interpolated between certain surveyed sections as deemed necessary. These interpolated sections were prepared from survey data along with the aid of topographic mapping (References 133, 134, and 135).

In Norwood and Sharon, structures and channel cross sections were field surveyed by R.E. Cameron and Associates, Inc., a subsidiary of Teledyne Geotronics, under subcontract to Harris-Toups Associates.

Overbank extensions of field surveyed cross sections and additional sections needed for hydraulic continuity were taken from topographic maps in Quincy (Reference 136) and from four-foot contour interval topographic maps prepared by Teledyne Geotronics (Reference 137) in Norwood and Sharon (Reference 138 and Reference 139).

In Quincy, structure sections for the backwater analysis for Cunningham Brook were taken from the USACE study, Furnace Brook Local Protection, Massachusetts Coastal Streams (Reference 33). Channel and overbank cross sections were taken from topographic maps (Reference 137).

In Bellingham, Cohasset, Dedham, Dover, Norwood, Quincy, Randolph, Sharon and Stoughton, cross sections for the backwater analyses were located at close intervals above and below bridges in order to compute the significant backwater effects of these structures in the developed areas. In Bellingham, Cohasset, Dedham, in long reaches between structures, appropriate valley cross sections were also used.

In Medfield and Wellesley, cross sections were taken at specific intervals along the stream channels, such that hydraulic properties would be accurately modeled by the computer. To increase the accuracy of backwater computations, cross sections were interpolated between surveyed cross sections when necessary. These interpolated cross sections were prepared from survey data with the aid of USGS maps at a scale of 1:24,000, with a contour interval of 10 feet (Reference 140) in Medfield and from survey data with the aid of town mapping (Reference 141).

Cross sections for the backwater analyses of the Charles River along the Millis/Norfolk corporate limits and from Forest Road along the Medfield/Millis corporate limits to the Medfield/Millis/Sherborn corporate limits were obtained from photogrammetric maps (References 142 and 143) and field measurements. Cross sections for the backwater analyses of the Charles River from the northern end of the Millis-Norfolk corporate limits to Forest Road were obtained from the Millis topographic map (Reference 144) and field measurement. New photogrammetric maps (Reference 114) were generated for the Millis 1985 FIS which supplant the topographic map used in this area. Cross sections for the backwater analyses of Bogastow Brook were obtained from photogrammetric maps (References 142 and 115) and field measurement.

In Norfolk cross sections for the backwater analyses of the Charles River, Mill River, Miller Brook, Stop River, Mann Pond Lateral, Prison Farm Lateral and Stony Brook were obtained from USGS topographic Maps (Reference 34) and from other engineering studies and construction plans, where available. Cross sections for the backwater analyses of Cress Brook and Myrtle Street and Harlow Pond Laterals were obtained from photogrammetric maps (Reference 145) and field measurement.

In Randolph, cross-section data for the updated backwater analysis of Mary Lee Brook were taken from topographic maps obtained from aerial photographs at a scale of 1:4,800 with a contour interval of 4 feet (Reference 124). Below water sections were obtained by field measurement.

In Holbrook, a general field reconnaissance and map study was made using USGS topographic maps (Reference 62). Major streams and tributaries were located and upper limits of study on the tributaries designated. Field surveys were selected to provide typical and restrictive cross sectional data for hydraulic studies. These included stream and valley cross sections, road-crossing restrictions, existing dams, and other key points controlling flood elevations. In addition, building elevations, low ground elevations, road profiles, high-water marks, and other pertinent information were obtained. Locations of field surveys were marked on a composite USGS topographic map and checked by field observation. Forty cross sections were taken on approximately 8 miles of streams. Information as to type, size, and condition was gathered on 30 stream crossings. In addition, available surveys, such as sewer plans and highway drawings, were used to supplement field surveys. Cross sections and stream crossings were plotted on graph paper. Stream profiles were developed from the survey data using valley lengths from new detailed topographic maps at a scale of 1:4,800 with a contour interval of 5 feet (Reference 146). Where available, additional location and elevation information from other engineering studies and construction plans were used. Tumbling Brook/Tumbling Brook Tributary and Great Pond Tributary are very swampy and covered with dense growth that makes field surveys expensive and time consuming. Both areas are undeveloped and were recommended by the town's planning consultants for acquisition by the town to protect water supplies and prevent unwise building in wet locations. Because of these conditions, cross sections were surveyed only at road crossings and accessible areas and normal depth computations were performed at these cross sections to obtain the water-surface elevations for these streams.

On Turnpike Lake in Plainville there are two small dams. The Plainville Highway Department removes the flashboards of these dams when the water level of the lake approaches flood stage. For the dam computations it has been assumed that all flashboards would be removed. Water can be diverted from Turtle Brook into a canal just below Turnpike Lake Dam No.1. The diverted water can be returned to Turtle Brook upstream from the site of an abandoned mill at Taunton Street. Furthermore, there is a leakage from the canal which is at a higher elevation than the brook. However, because there is no way of knowing how much, if any, water would be diverted into the canal during a flood, it has been assumed that canal flow would be negligible.

In Avon, Foxborough and Stoughton, elevations were taken from USGS topographic maps (References 114, 90 and 147).

In Bellingham, flood elevations were determined using a regional relationship developed between the drainage area and depth of flooding based on a regression analysis of gaged, small drainage area streams in Massachusetts.

Flood elevations for areas of Wigwam and Lowder Brooks and Wigwam and Little Wigwam Ponds in Dedham and for the Pine Tree Brook Reservoir in Milton were determined using historical information, field observations, and basic hydraulic calculations.

In Medfield and Wellesley, field investigations, historical observations, manual calculations and backwater effects from streams studied by detailed methods were used to determine approximate elevations. In some instances in Wellesley, flooding was determined by backwater conditions from streams which were studied using detailed methods.

In Foxborough and Stoughton, approximate methods were used to study flood boundaries along streams flowing through undeveloped areas. In Stoughton, values for roughness coefficients and selected channel data were obtained by field investigation. The 1-percent-annual-chance flood elevations were computed using topographic maps (Reference 63).

For streams studied by the approximate method in Norwood and Sharon, the best available information on flooding was utilized, including backwater elevations for detailed reaches, wetlands information, aerial photos, historic observation, field survey, and the emergency phase Flood Hazard Boundary Map (Reference 148) for Norwood and for Sharon (Reference 64).

In Randolph, for the streams studied by approximate methods, flood magnitudes were estimated based on the Town of Randolph Watershed and Wetlands Protection maps and the USGS Blue Hills topographic map (References 63 and 149).

In Walpole, the depth of flooding for the areas studied by approximate methods were determined using a drainage area-depth of flooding curve developed by the USDA NRCS in a previous study (Reference 150).

Approximate flood elevations in Weymouth were taken from the previous Flood Insurance Study for the Town of Weymouth (Reference 27). Field inspection was used to verify approximate flood areas.

Water-surface elevations of floods of the selected recurrence intervals were computed using the USACE HEC-2 step-backwater computer program for most of Norfolk County including; using the May 1991 for the Towns of Avon (Reference 82), Bellingham (Reference 82), Canton (References 151 and 152), Cohasset (Reference 82), in September 1989 for Dedham (References 82 and 153), Dover (Reference 82), Foxborough (Reference 82), Franklin (Reference 82), Medfield (Reference 82), Medway (Reference 82), Millis (Reference 82), Randolph (1978 & 1987 FIS) (Reference 82), Sharon (Reference 82), Stoughton (Reference 82), Wellesley (References 82, 63, and 153) and Wrentham (Reference 82); and for the Town Brook in Braintree (Reference 82;

riverine flooding in Milton (References 82 and 153); Cress Brook and Myrtle Street and Harlow Pond Laterals in Norfolk (Reference 82); Bubbling, Mill and Purgatory Brooks in Westwood (Reference 87); Herring Brook, the Mill River, Mill River Tributary A, and Mill River Tributary B in Weymouth (Reference 78); and for Furnace and Cunningham Brooks in the City of Quincy.

Water-surface elevations of floods of the selected recurrence intervals were computed using the USDA NRCS WSP-2 computer program in Holbrook, (Reference 154), Walpole (Reference 148); for Mill River, Miller Brook, Stop River, Mann Pond Lateral, Prison Farm Lateral and Stony Brook in Norfolk (Reference 154); and for the Ten Mile River and the Whiting Pond Bypass in Plainville (Reference 154).

Water-surface elevations for South Brook in Westwood were computed using the USACE HEC-RAS September 1998 step-backwater computer program (Reference 87). Starting water-surface elevations were taken from a plot of the discharge versus elevation for the cross section at Purgatory Brook 75 feet upstream of Everett Street.

Water-surface elevations for the 10-, 2-, 1- and 0.2-percent-annual-chance peak discharges in the Cochato River were re-computed using the USACE HEC-RAS computer program in Braintree (Reference 87), Holbrook (Reference 87) and Randolph (Reference 87). In Randolph, starting water-surface elevations for the Cochato River were based on the hydraulic analysis of the reach of the Cochato River in Braintree and the Monatiquot River from its junction with the Cochato River downstream to the Armstrong Cork Dam.

Starting water-surface elevations for the Town of Foxborough, Stoughton; Beaver Brook, Arnolds Brook, and Hopping Brook in Bellingham; Mine Brook and Shepards Brook in Franklin; Chicken Brook, Hopping Brook, and Tributary to Great Black Swamp in Medway; for Bogastow Brook in Millis; for Traphole Brook in Norwood; for Cress Brook and Myrtle Street and Harlow Pond Laterals in Norfolk; for Hawthorne Brook in Plainville; and for Beaver Brook, Billings Brook and Branch, Canoe River and Sucker Brook in Sharon, were developed using the slope/area method.

Starting water-surface elevations were developed for the Charles River in Bellingham, Dedham, Dover, Franklin, Medfield, Medway, Millis, Needham, Norfolk and Wellesley.

In Medfield, starting water-surface elevations were based on hydraulic calculations begun at the nearest elevation control. The Charles River studies were begun at the South Natick Dam. The Vine Brook studies were based on normal depth elevation solution for the downstream section of the reach. Starting elevations in Medway and water-surface elevations of the floods of the selected recurrence intervals for the Charles River in Norfolk were adopted from the Millis Flood Insurance Study (References 14). Starting water surface elevations for the Charles River in Millis and Norfolk were adopted from flood profiles in the Medfield Flood Insurance Study (Reference 11). Starting water-surface elevations for the Charles River in Bellingham and Franklin were obtained from the Flood Insurance Study for the Town of Medway (Reference 13). In Wellesley and Needham, starting water-surface elevations for the Charles River were based on studies prepared by the USACE (References 5, 61, 156, and 157).

The starting water-surface elevations for the Charles River in Dedham were determined from published information (References 73 and 5). Starting water-surface elevations for

Mother Brook in Dedham were determined from basic hydraulic calculations using Manning's equation.

To ensure the reasonableness of the expected flood levels in Dover, the resulting profiles for the Charles River and the lower portion of Trout Brook were verified by checking against high-water marks resulting from the floods of August 1955 and March 1968. Starting water-surface elevations for the Charles River were obtained from the Flood Insurance Study for the Town of Needham (Reference 16). Starting water-surface elevations for Trout Brook and Rocky Brook were determined by the slope/area method. For the concrete culverts located at road crossings, a roughness factor of 0.024 was used.

Flooding of the lower segment of West Mill Brook and the entire Stop River in Medfield is caused by the elevated water surface of the Charles River. This backwater condition causes higher water-surface elevations on the streams than the natural drainage from the streams own tributary watershed, thus no profiles were developed for these streams.

Starting water-surface elevations were developed for the Neponset River in Canton, Dedham, Milton, Norwood, Sharon and Walpole. In Canton and Dedham, the starting water-surface elevation for the Neponset River was determined from published information (References 5, 55, and 73). Starting water-surface elevations for the Neponset River in Norwood and Sharon were taken from the FIS for Canton (Reference 5), and the elevations in Sharon were coordinated with the Flood Insurance Study presently being conducted on the Neponset River in Norwood, by Harris-Toups Associates (References 5 and 18). In Walpole, starting water-surface elevations for the Neponset River were taken from the Flood Insurance Study for the Town of Norwood (Reference 18).

The starting water-surface elevations for the Neponset River upstream of the Lower Mills Dam in Milton were assumed to occur with the stop planks removed. Starting elevations for Pine Tree Brook were taken from the Neponset River.

In Norwood, the computer model for the Neponset River was calibrated with flood marks from the March 1968 flood and was checked for agreement with the MWRC report (Reference 56) and with profiles developed for the FIS for Canton, Massachusetts.

In Sharon, the computer model, in general, was checked with information (obtained from interviews with local officials and residents) of flooding which occurred during the August 1955 flood. In the case of the Neponset River, the computer model was calibrated by the March 1968 flood water-surface elevations, obtained through the Water Resources Commission (Reference 158). Profiles for the 1- and 0.2-percent-annual-chance floods were obtained from a previous report by the firm of Anderson-Nichols & Company (Reference 56).

In Norfolk, starting water surface elevations for the Stop River were based upon a combination of the routed discharge frequency and the elevation-discharge relationship at the furthermost downstream cross-section. Starting water surface elevations for the Mill River, Miller Brook, Mann Pond Lateral, Prison Farm Lateral and Stony Brook were based upon the combination of the routed discharge-frequency and the elevation-discharge relationship at selected cross sections. If the backwater elevation of the stream into which any of this last group discharges were found to be higher for the same recurrence interval, the higher elevation was used.

For the smaller streams in Norwood, the computer models were checked for agreement with flood data from the March 1968 flood and the August 1955 flood. Information on these floods was obtained through interviews with local residents. Present culvert conditions were taken into consideration in the use of flood marks. Starting water-surface elevations for the computer model for Plantingfield and Purgatory Brooks were determined by the slope-area method. These elevations were within 0.5 foot of backwater elevations from the Neponset River. The starting water-surface elevations for Hawes Brook and Meadow Brook were taken from backwater elevations for the Neponset River. Backwater from Hawes Brook determined the starting water-surface elevations for Germany Brook. Water-surface elevations of floods of the selected recurrence intervals were computed through use of the USACE September 1990 HEC-2 backwater computer program (Reference 93).

In Norwood, Willett Pond was studied in the detailed analysis of Bubbling Brook in the Flood Insurance Study for the Town of Walpole, Massachusetts (Reference 24). Since Willett Pond lies partially in Norwood, it was designated Zone AE, as it was in the Walpole FIS.

Starting water-surface elevations for Bubbling Brook and Mill Brook in Westwood were determined by using the known water-surface elevation of Willett Pond, taken from the FIS for the Town of Norwood, Massachusetts (Reference 18). The slope/area method was used to provide a computed water-surface elevation corresponding to a specified energy grade line slope at the first downstream cross section for Purgatory Brook.

In Walpole, flood profiles for the Stop River were obtained from the Flood Insurance Study prepared for the Town of Norfolk (Reference 17). Profiles of Bubbling Brook were obtained from the original Flood Insurance Study for the Town of Walpole using the FIS for the Town of Westwood (References 24 and 26). Water-surface elevations of the numerous reservoirs in the town were determined by routing floods of the selected recurrence interval using the Technical Release No. 20 computer program (Reference 78). Many of the ponds in the Neponset watershed are managed for industrial use; therefore, during an actual flood, the starting reservoir stages may be higher or lower than those assumed in this study. Field surveys used to locate and delineate cross sections were made in the spring when most reservoirs were nearly full.

Also in Walpole, starting water-surface elevations for the Stop River were computed using normal depth calculations. Backwater elevations from the Charles River are superimposed over the Stop River values to obtain the final profile elevations (Reference 12). The downstream portion of the Stop River within Walpole is affected by Charles River backwater elevations; hence, no profile is published for that portion. Starting water-surface elevations for Traphole Brook were taken directly from the 1984 elevations of proposed culvert modifications to Traphole Brook in Norwood and Walpole (Reference 121). Starting water-surface elevations for Cobb's, Diamond, Mine, Pickerel, School Meadow, and Bubbling Brooks were assumed to be at normal depth for the discharge related to the specific recurrence interval on these streams.

In Avon, starting water-surface elevations for Beaver and Trout Brooks at the downstream most cross sections were determined by the drainage area contributing to the section being analyzed and geometry of the stream channel, using the USACE HEC-2 May 1991 step-backwater computer model (Reference 82).

Starting water-surface elevations for the Peters River in Bellingham were obtained from the FIS for the City of Woonsocket (Reference 89). Starting water-surface elevations for Bungay Brook were taken from the Peters River profiles.

The HEC-2 model for Town Brook in Braintree was calibrated to the 1-percent-annual-chance profile developed by the USACE in their study of Town Brook (Reference 54). The model was then used to calculate elevations for the 10-, 2-, 1- and 0.2-percent-annual-chance flood elevations. Present culvert conditions were used, and recent modifications were taken into consideration in the use of these flood data. The culvert under State Route 3 was used in computing flood profiles for Town Brook. The starting water-surface elevations for Town Brook were taken from the elevations developed by the USACE in their study for Town Brook (Reference 54).

Also in Braintree, the water-surface elevations for the Monatiquot River were taken from the previously effective FIS for Braintree except for the reach of the river from just downstream of the River Street bridge to upstream of the Conrail bridge (downstream of the Lower Armstrong Dam) (Reference 4). The River Street bridge has been replaced, the Ivory Street Bridge has been added, and Ames Pond Dam has been removed from the since the previous study had been completed. These changes required that the hydraulic analysis for this reach be redone to take those changes into account. The USACE HEC-2 input data for this reach were modified to reflect changes, and the HEC-2 program was rerun to calculate the elevations for the 10-, 2-, 1- and 0.2-percent-annual-chance floods. The starting water-surface elevations were taken from the June 1, 1978, FIS for Braintree at the downstream end of this reach (Reference 4). The water-surface profiles are affected by the reconstruction only through that reach; at the upstream end of the reach, the new profiles have been tied into the previously computed USACE profiles. The water-surface elevations and starting water-surface elevations for the Farm River were determined using the previously effective study for the Town of Braintree (Reference 4).

The starting water-surface elevation for the Canton River and Ponkapoag Brook in Canton were taken from the calculated elevation of the Neponset River at their respective confluences. The starting water-surface elevation for the remaining streams studied by detailed methods was determined from the HEC-1 January 1973 analysis of the watershed upstream of the Forge Pond Dam (Reference 93).

In Cohasset, the computer model for each stream was calibrated to historic records obtained through interviews with local residents and using the Cohasset flood plain map (Reference 58). Present culvert conditions were used, and recent modifications were taken into consideration in the use of historic floodmarks. Starting water-surface elevations for Walnut Hill Stream, Rattlesnake Run, James Brook, and Richardsons Brook were determined using normal depth calculations. Starting water-surface elevations for Turkey Hill Run were taken as the mean high tide level at the outlet to Straits Pond. Starting water-surface elevations for Lily Pond Stream were taken as the stillwater elevation on Lily Pond.

In Holbrook, stage-storage-discharge data were developed for the outlet structures of Lake Holbrook. The starting water-surface elevations for the streams studied by detailed methods in Holbrook were determined by normal depth computations.

Water-surface elevations for Needham were computed using the USACE HEC-2 step-backwater computer program taken from the Flood Insurance Studies for the Towns of Wellesley, Dover, Westwood and Dedham; and the Cities of Newton and Boston

(References 11, 25, 8, 26, 7, 60, and 159). The flood elevations for Fuller Brook in Needham were estimated through the use of the January 1985 USACE HEC-l (Reference 93).

With the exception of the Ten Mile River and the Whiting Pond Bypass, water-surface elevations of floods of the selected recurrence intervals in Plainville were computed using USGS step-backwater computer program E431 (Reference 160). The elevations for the Ten Mile River and the Whiting Pond Bypass were computed at the time of the USDA NRCS Flood Insurance Study of the Town of North Attleboro (Reference 116). Elevations obtained for the Ten Mile River using USDA NRCS field data in the USGS computer program verify those obtained by the USDA NRCS. The flood elevations of Lake Mirimichi were used as starting elevations for Turtle Brook. The starting elevations on Brook No. 1 were determined by dam computations.

In Quincy, the computer model for Furnace and Cunningham Brooks was calibrated to the 1-percent-annual-chance profile developed by the USACE in their study of Furnace Brook (Reference 33). The model was then used to calculate elevations for the 10-, 2-, 1- and 0.2-percent-annual-chance profiles. Present culvert conditions were used and recent modifications were taken into consideration in the use of these flood data. The starting water-surface elevations for Furnace Brook were computed by calculating normal depth in its tidal reach at Blacks Creek. Starting water-surface elevations for Cunningham Brook were taken from the profiles of Furnace Brook at the confluence of Cunningham Brook. Elevations for Town Brook were taken from the USACE report, Town Brook Local Protection, Massachusetts Coastal Streams (Reference 54). Certain elevations were interpolated from the published data.

In Randolph, starting water-surface elevations for Martin Brook, Glovers Brook, and Mary Lee Brook were taken from the Cochato River profile for floods of the selected recurrence intervals at the tributary junction locations. Starting water-surface elevations for Brook A (Stetson Brook) were based on the peak flood stage for each of the 10-, 2-, 1- and 0.2-percent-annual-chance flood profiles along Glovers Brook. The starting water-surface elevations for Norroway Brook and Brook B were based on an analysis of the Upper Reservoir; that analysis consisted of an evaluation of the reservoir's storage capacity, the discharge versus data for the embankment and spillway between the Upper Reservoir and Great Pond, and local rainfall and runoff data. Starting water-surface elevations for the Unnamed Tributary to Mary Lee Brook were taken from the peak flood elevations of Mary Lee Brook at their point of confluence.

In Sharon, the water-surface elevation of Massapoag Lake is controlled by the flume house at the most northerly end of the lake which serves as the outlet structure to Massapoag Brook. According to the Town Engineer in Sharon, approximately 3.5 to 4.0 feet of planking controls the flow at the flume house. Through hydraulic computations it was determined that the spillway in the flume house has a limited capacity and will not adequately handle the floodwaters of the 2-, 1- and 0.2-percent-annual-chance year floods. It was for this reason, along with the fact that the town does implement flood control measures, that the spillway was assumed to be fully opened in time of a major storm. The final water-surface elevations on the lake were determined through a stage-discharge curve based on the combined effects of the outlet structure at the flume house and flow over the road on low-lying areas of Beach Street, which drains into the swampy area west of Lake Massapoag.

For the Waban Brook, Lake Waban, Morses Pond, and Paintshop Pond in Wellesley, studies were begun at the dams which control the water-surface elevations.

In the 1991 Weymouth FIS revision, an updated version of the April 1984 USACE HEC-2 step-backwater computer program was used to develop water-surface profiles for Old Swamp River (Reference 82). Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals. Starting water-surface elevations for Herring Brook, the Mill River, Mill River Tributary A, Mill River Tributary B, and the Old Swamp River were taken from normal flow elevations determined by field inspections and field surveys.

In Wrentham, the resulting profiles were verified by checking against recent high-water marks to ensure the reasonableness of the expected flood levels. The starting water-surface elevations were obtained using the slope/area option of the HEC-2 computer program. It was determined that certain hydraulic structures on the streams studied by detailed methods would not be included in the analyses due to the minimal effects they would have on flood elevations. In these cases the hydraulic structures are shown on the map but not on the profiles

Roughness factors (Manning's "n") used in the hydraulic computations were estimated based on field inspection of flood plain areas. The channel "n" and overbank "n" values for the streams studied by detailed methods are shown in Table 12.

TABLE 12 –MANNING'S "N" VALUES

Flooding Source	Channel "n"	Overbanks
Arnolds Brook	0.030-.0.55	0.050-0.100
Beaver Brook (Avon)	0.030-0.060	0.050-0.100
Beaver Brook (Bellingham)	0.030-.0.55	0.050-0.100
Beaver Brook (Holbrook)	0.040-0.045	0.030-0.11
Beaver Brook (Sharon)	0.02-0.05	0.04-0.075
Beaver Meadow Brook	0.015-0.040	0.045-0.080
Billings Brook	0.02-0.05	0.04-0.075
Billings Brook Branch	0.02-0.05	0.04-0.075
Bogastow Brook	0.035-0.043	0.060-0.100
Brook A (Stetson Brook)	0.050	0.060-0.1 00
Brook B	0.050	0.100
Brook No.1	0.015-0.045	0.040-0.080
Bubbling Brook (Walpole)	(data not available)	(data not available)
Bubbling Brook (Westwood)	0.013-0.045	0.035-0.110
Bubbling Brook (Willett Pond) (Norwood)	0.02-0.05	0.04-0.10
Bungay Brook	0.030-.0.55	0.050-0.100
Burnt Swamp Brook	0.040-0.050	0.070-0.100
Canoe River (Foxborough)	0.030-0.060	0.050-0.100
Canoe River (Sharon)	0.02-0.05	0.04-0.075

TABLE 12 –MANNING'S "N" VALUES – (CONTINUED)

Flooding Source	Channel "n"	Overbanks
Canton River	0.025-0.040	0.060-0.080
Charles River (Bellingham)	0.030-.0.55	0.050-0.100
Charles River (Dedham)	0.025-0.035	0.065-0.09
Charles River (Dover, Needham)	0.014-0.040	0.030-0.100
Charles River (Franklin)	0.030-0.044	0.040-0.120
Charles River (Medfield)	0.015-0.040	0.040-0.080
Charles River (Medway)	0.030-0.044	0.020-0.120
Charles River (Millis, Norfolk)	0.014-0.040	0.030-0.200
Charles River (Wellesley)	0.015-0.050	0.040-0.080
Chicken Brook	0.025-0.050	0.020-0.150
Cobb's Brook	0.035-0.045	0.045-0.090
Cochato River (Braintree)	0.015-0.090	0.016-0.120
Cochato River (Holbrook)	0.035-0.040	0.020-0.110
Cochato River (Randolph)	0.04-0.05	0.07-0.12
Cress Brook	0.040	0.080
Crocker Brook	0.040	0.080
Cunningham Brook	0.040	0.060
Diamond Brook	0.040-0.060	0.050-0.080
Dorchester Brook	0.013-0.06	0.029-0.08
Farm River	0.015-0.090	0.016-0.120
Furnace Brook	0.015-0.060	0.070-0.110
Germany Brook	0.02-0.05	0.04-0.10
Glovers Brook	0.050	0.060-0.1 00
Harlow Pond Lateral	0.030	0.080
Hawes Brook	0.02-0.05	0.04-0.10
Hawthorne Brook	0.025-0.035	0.045-0.070
Herring Brook	0.030-0.060	0.050-0.100
Hopping Brook (Bellingham)	0.030-.0.55	0.050-0.100
Hopping Brook (Medway)	0.018-0.060	0.020-0.160
James Brook	0.015-0.040	0.060-0. 120
Lake Holbrook	0.035-0.040	0.020-0.110
Lake Waban	0.015-0.050	0.040-0.080
Lily Pond Stream	0.013-0.040	0.090-0.100
Lower Pequid Brook	0.015-0.040	0.045-0.080
Mann Pond Lateral	0.010-0.065	0.050-0.100
Martin Brook	0.050	0.080
Mary Lee Brook	0.033-0.064	0.064-0.085
Massapoag Brook (Canton)	0.015-0.040	0.045-0.080
Massapoag Brook (Sharon)	0.02-0.05	0.04-0.075
Meadow Brook	0.02-0.05	0.04-0.10
Mill Brook	0.013-0.045	0.035-0.110
Mill River (Norfolk)	0.025-0.060	0.050-0.100
Mill River (Weymouth)	0.030-0.060	0.050-0.100
Mill River Tributary A	0.030-0.060	0.050-0.100
Mill River Tributary B	0.030-0.060	0.050-0.100

TABLE 12 –MANNING'S "N" VALUES – (CONTINUED)

Flooding Source	Channel "n"	Overbanks
Miller Brook	0.015-0.070	0.050-0.200
Mine Brook (Franklin)	0.040-0.070	0.060-0.090
Mine Brook (Walpole)	0.030-0.100	0.010-0.110
Monatiquot River	0.015-0.090	0.016-0.120
Morses Pond	0.015-0.050	0.040-0.080
Mother Brook	0.025-0.035	0.065-0.09
Myrtle Street Lateral	0.025-0.050	0.070
Neponset River (Canton)	0.025-0.040	0.060-0.080
Neponset River (Dedham)	0.025-0.035	0.065-0.09
Neponset River (Milton)	0.035	0.06
Neponset River (Norwood)	0.02-0.05	0.04-0.10
Neponset River (Sharon)	0.02-0.05	0.04-0.075
Neponset River (Walpole)	0.020-0.075	0.070-0.110
Norroway Brook	0.050	0.060-0.1 00
Old Swamp River	0.030-0.060	0.050-0.100
Paintshop Pond	0.015-0.050	0.040-0.080
Peters River	0.030-.0.55	0.050-0.100
Pickerel Brook	0.025-0.070	0.030-0.095
Pine Tree Brook	0.04-0.05	0.08
Plantingfield Brook	0.02-0.05	0.04-0.10
Ponkapoag Brook	0.015-0.040	0.045-0.080
Prison Farm Lateral	0.040	0.090
Purgatory Brook (Norwood)	0.02-0.05	0.04-0.10
Purgatory Brook (Westwood)	0.013-0.045	0.035-0.110
Rabbit Hill Brook	0.040	0.080
Rattlesnake Run	0.020-0.040	0.080-0.090
Redwing Brook	0.013-0.06	0.016-0.08
Richardsons Brook	0.020-0.040	0.070
Robinson Brook	0.030-0.060	0.050-0.100
Rocky Brook	0.030	0.064
Rumford River	0.030-0.060	0.050-0.100
School Meadow Brook	0.050	0.060-0.090
Shepards Brook	0.050-0.060	0.070-0.080
South Brook	0.050-0.093	0.050-0.090
Steep Hill Brook	0.013-0.06	0.016-0.08
Stony Brook	0.040-0.065	0.090-0.100
Stop River (Medfield)	0.015-0.040	0.040-0.080
Stop River (Norfolk)	0.020-0.065	0.050-0.100
Stop River (Walpole)	0.010-0.060	0.060-0.095
Sucker Brook	0.02-0.05	0.04-0.075
Ten Mile River	0.010-0.040	0.020-0.090
Town Brook	0.015-0.090	0.016-0.120
Traphole Brook (Norwood)	0.02-0.05	0.04-0.10
Traphole Brook (Walpole)	0.040-0.080	0.060-0.110
Tributary C2	(data not available)	(data not available)
Tributary C2B	0.040	0.030-0.090

TABLE 12 –MANNING’S “N” VALUES – (CONTINUED)

Flooding Source	Channel "n"	Overbanks
Tributary R1	(data not available)	(data not available)
Tributary R2	0.040	0.040-0.090
Tributary R3	0.040	0.030-0.110
Tributary R4	0.040-0.005	0.040-0.100
Tributary to Great Black Swamp	0.015-0.055	0.020-0.120
Tributary to Steep Hill Brook	0.013-0.06	0.016-0.08
Trout Brook (Avon)	0.030-0.060	0.050-0.100
Trout Brook (Holbrook)	0.035-0.040	0.020-0.110
Trout Brook (lower portion) (Dover)	0.014-0.040	0.030-0.100
Trout Brook (upper portion) (Dover)	0.030	0.064
Turkey Hill Run	0.015-0.070	0.090-0.110
Turtle Brook	0.025-0.080	0.035-0.080
Unnamed tributary to Mary Lee Brook	0.050	0.100
Upper Pequid Brook	0.015-0.040	0.045-0.080
Vine Brook	0.015-0.040	0.040-0.080
Waban Brook	0.015-0.050	0.040-0.080
Walnut Hill Stream	0.015-0.040	0.090-0.120
West Mill Brook	0.015-0.040	0.040-0.080
Weymouth Back River	0.030-0.060	0.050-0.100
Weymouth Fore River (Braintree)	0.015-0.090	0.016-0.120
Weymouth Fore River (Weymouth)	0.030-0.060	0.050-0.100
Whiting Pond Bypass	0.035-0.040	0.050-0.080

Stillwater elevations for the flooding sources that were not revised by the countywide analyses and City of Quincy Coastal update are presented in Table 13.

TABLE 13 – SUMMARY OF STILLWATER ELEVATIONS

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD)[1]			
	10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
BOLIVAR POND				
Entire Shoreline within Canton	105.4	105.9	106.1	106.5
BUCKMASTER POND				
Entire Shoreline within Westwood	181.2	*	182.4	183.3
FORGE POND				
Entire Shoreline within Canton	91.2	93.5	94.2	94.9
FULLER BROOK				
Upstream of Wellesley/ Needham Corporate Limits in Needham	132.1	132.8	133.2	134.0
MASSAPAG LAKE				
At Sharon	252.8	254.0	254.2	254.6
MORSES POND				
At Wellesley	122.9	123.7	124.2	125.4
MASSAPAG LAKE				
At Sharon	252.8	254.0	254.2	254.6
PETTEE ROAD				
Entire Shoreline within Westwood	143.3	*	144.4	145.2
RESERVOIR POND				
Entire Shoreline within Canton	145.4	146.2	147.0	148.4

*data not available
[1]North American Vertical Datum of 1988

<u>July 17, 2012 Partial Countywide Analyses</u>

No new riverine hydraulic analyses were performed for the July 17, 2012 partial countywide FIS.

<u>City of Quincy Coastal Update</u>

No new riverine hydraulic analyses were performed for the City of Quincy Coastal Update.

3.3 Coastal Hydrologic Analyses

In New England, the flooding of low-lying areas is caused primarily by storm surges generated by extratropical coastal storms called northeasters. Hurricanes also occasionally produce significant storm surges in New England, but they do not occur nearly as frequently as northeasters. Hurricanes in New England typically have a more severe impact on the south facing coastlines. Due to its geographic location, Norfolk 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 (Reference 152).

As part of the July 17, 2012 countywide update, new coastal analysis was performed for the communities of Braintree, Cohasset, and Weymouth. A description of the revised analyses is presented below. For the City of Quincy coastal study update, a description of the methods is described in the City of Quincy Coastal Study Update section.

<u>July 17, 2012 Partial Countywide Analyses</u>

As part of the July 17, 2012 partial countywide update, revised coastal analyses were performed for the open water flooding sources in the communities of Braintree, Cohasset and Weymouth. Provided below is a summary of the analyses performed. All revised coastal analyses were performed in accordance with Appendix D “Guidance for Coastal Flooding Analyses and Mapping,” (Reference 161) of the Guidelines and Specifications, as well as, the “Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update”, (Reference 162).

For the revised communities, published values in the Tidal Flood Survey (Reference 163) were used to estimate the stillwater elevations for the 10-, 2-, and 1-percent-annual-chance floods for Hingham Bay, Straits Pond, The Gulf, Massachusetts Bay, and Weymouth Fore River and Weymouth Back River. The 0.2-percent-annual-chance stillwater elevations for the revised flooding sources were extrapolated based on the more the frequent stillwater elevations in the Tidal Flood Survey. Stillwater elevations for the revised flooding sources are presented in Table 14.

TABLE 14 – SUMMARY OF COASTAL STILLWATER ELEVATIONS

FLOODING SOURCE AND LOCATION	ELEVATION (feet, North American Vertical Datum of 1988) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
Town of Cohasset				
MASSACHUSETTS BAY				
From Cohasset-Hull corporate limits to Government Island, including Little Harbor and Cohasset Cove	8.4	9.3	9.6	10.4
TRANSECTS 40-43 and 47-79				
THE GULF				
From Border Street to 300 feet north of Supper Island	8.4	9.3	9.6	10.4
TRANSECTS 50-51				
Near Supper Island	8.4	9.3	9.6	10.4
200 feet south of Supper Island to Stanton Road	8.4	9.3	9.6	10.4
Stanton Road to Scituate-Cohasset corporate limits	8.4	9.3	9.6	10.4
STRAITS POND				
Entire Shoreline within the corporate limits	8.4	9.3	9.6	10.4
JAMES BROOK				
From Border Street tide gate to Elm Street	8.4	9.3	9.6	10.4
TRANSECTS 45-46				
RICHARDSONS BROOK				
Upstream of Jerusalem Road	8.4	9.3	9.6	10.4
TRANSECT 44				

TABLE 14 – SUMMARY OF COASTAL STILLWATER ELEVATIONS – (CONTINUED)

FLOODING SOURCE AND LOCATION	ELEVATION (feet, North American Vertical Datum of 1988) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
Town of Weymouth				
HINGHAM BAY				
At Grape Bay	8.3	9.2	9.5	10.4
TRANSECTS 36-38				
WEYMOUTH BACK RIVER				
Entire length within community	8.3	9.2	9.5	10.4
TRANSECT 39				
WEYMOUTH FORE RIVER				
At Rose Cliff	8.3	9.2	9.5	10.4
TRANSECTS 31-32				
At Wassagusset Beach	8.3	9.2	9.5	10.4
TRANSECTS 34-35				
Kings Cove	8.3	9.2	9.5	10.4
TRANSECT 33				
Town of Braintree				
WEYMOUTH FORE RIVER				
Quincy corporate limits to Venus Road	8.4	9.3	9.5	10.4
TRANSECTS 27B-29				
Venus Road to Shaw Street	8.4	9.3	9.5	10.4
TRANSECT 30				

The elevations presented in the Tidal Flood Survey 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 were converted to the current tidal datum. A datum conversion factor of +0.11 feet was applied to the data in the Tidal Flood Survey for each community.

City of Quincy Coastal Study Update

For the City of Quincy, the 10-, 2-, 1-, and 0.2-percent-annual-chance stillwater elevations were obtained from the "Regional Frequency Analyses using L-Moments" memorandum developed by STARR (Reference 163), a statistical analysis of available tide gage records for areas subject to coastal flooding. There is one gage located near Norfolk County within Suffolk County. The stillwater elevations for each transect were linearly scaled based on a ratio between the effective FIS stillwater elevation and the Boston Harbor tide gage. Stillwater elevations for the City of Quincy are presented in Table 15.

TABLE 15 – SUMMARY OF COASTAL STILLWATER ELEVATIONS - QUINCY

FLOODING SOURCE AND LOCATION	ELEVATION (feet, North American Vertical Datum of 1988) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
QUINCY BAY				
Long Island Causeway to Fenno Street	9.0	9.9	10.7	11.9
Fenno Street to Adams Shore	9.2	10.3	10.9	12.3
Adams Shore to Nut Island	9.0	10.0	10.8	12.1
Nut Island to Rock Island Head	9.4	10.8	11.8	12.8
DORCHESTER BAY				
Neponset River to Long Island Causeway	9.3	10.8	11.3	12.8
WEYMOUTH FORE RIVER				
Rock Island Head to Germantown Point	9.5	10.9	12.0	13.0
Gernantown Point to Braintree Corporate Limits	9.8	11.2	12.1	13.5

TABLE 15 – SUMMARY OF COASTAL STILLWATER ELEVATIONS - QUINCY - CONTINUED

FLOODING SOURCE AND LOCATION	ELEVATION (feet, North American Vertical Datum of 1988) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
TOWN RIVER BAY				
Gernantown Point to Mound Street	9.6	11.0	12.1	13.2
Mound Street to Southern Artery	9.7	11.0	12.1	13.3
Southern Artery to Elm Street	9.1	10.9	11.8	13.3

3.4 Coastal Hydraulic Analyses

July 17, 2012 Partial Countywide Analyses

Wave setup along the open coast areas of Braintree, Cohasset and Weymouth was calculated using the procedures detailed in the “Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update”, (Reference 162). Specifically, the Direct Integration Method (DIM) was applied. Because much of the Norfolk 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 communities, except for areas inundated by wave runup.

For the revised open coast portions of Norfolk County, offshore wave characteristics representing a 1-percent-annual-chance storm were determined using recorded buoy data from the National Oceanic and Atmospheric Administration's National Data Buoy Center (NDBC). A Peaks-Over-Threshold statistical analysis (Reference 165) was applied on 21 years (1987-2007) of wave characteristic data from NDBC Station 44013, located offshore of the Town of Cohasset. For fetch limited cases such as Hingham Bay, Weymouth Fore River and Weymouth Back River, 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 FEMA guidance for V Zone mapping.

Wave heights and wave runup in Braintree, Cohasset and Weymouth 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 conditions in their locality. 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 Descriptions for the restudied coastal analyses are shown in Table 16 below and have been re-numbered to conform to countywide standards.

TABLE 16 – REVISED TRANSECT DESCRIPTIONS

TRANSECT	LOCATION	1-PERCENT ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT ANNUAL-CHANCE WAVE CREST[1]
27B	This transect represents the shoreline of Braintree from the Corporate Limits of the Town of Quincy southeast to approximately 500 feet west of the CITGO Petroleum property.	9.5	13
28	This transect represents the shoreline of Braintree along the easternmost 500 feet of the CITGO Petroleum property.	9.5	13
29	This transect represents the shoreline of Braintree from the eastern property limit of the CITGO Petroleum property to the northeast end of Audubon Avenue.	9.5	12
30	This transect represents the shoreline of Braintree from the northeast end of Audubon Avenue to the Corporate Limits of the Town of Weymouth.	9.5	13
31	This transect represents the shoreline of Weymouth from the marina just east of the intersection of Gilmore Street and Brewster Road to the Corporate Limits of the Town of Braintree.	9.5	14
32	This transect represents the shoreline of Weymouth from the Fore River Bridge to the marina just east of the intersection of Gilmore Street and Brewster Road.	9.5	14
33	This transect represents the shoreline of Weymouth from the intersection of Babcock Avenue and Kings Cove Road to the Fore River Bridge.	9.5	22

[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 16 – REVISED TRANSECT DESCRIPTIONS – (CONTINUED)

TRANSECT	LOCATION	1-PERCENT ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT ANNUAL-CHANCE WAVE CREST[1]
34	This transect represents two non-adjacent sections of shoreline of Weymouth. The first is between the intersection of Regatta Road and Neck Street and the intersection of Wessagussett Road and North Street and the second is along Wessagussett Road between Pilgrim Road and Paomet Road.	9.5	15
35	This transect represents two non-adjacent sections of shoreline of Weymouth. The first is along Wessagussett Road between North Street and Pilgrim Road and the second is from the intersection of Paomet Road and Wessagussett Road to the intersection of Babcock Avenue and Kings Cove Road.	9.5	20
36	This transect represents the shoreline of Weymouth from the northeastern extent of Fort Point Road to the intersection of Regatta Road and Neck Street.	9.5	16
37	This transect represents the shoreline of Weymouth from eastern extent of River Street and the entrance to William Webb Park to the northeastern extent of Fort Point Road.	9.5	24
38	This transect represents the shoreline of Weymouth along the perimeter of William Webb Park at the eastern extent of River Street.	9.5	18
39	This transect represents the shoreline of Weymouth from the southern extent of William Webb Park near the intersectioı of River Street and Broad Reach to the Bridge Street (Route 3A) Bridge.	9.5	13

[1]Because of map scale limitations, the maximum wave elevation may not be shown on the

TABLE 16 – REVISED TRANSECT DESCRIPTIONS – (CONTINUED)

TRANSECT	LOCATION	1-PERCENT ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT ANNUAL-CHANCE WAVE CREST[1]
FIRM.			
40	This transect represents the shoreline of Cohasset from intersection of Jerusalem Road and Linden Drive to the Hull Corporate Boundary.	9.6	21
41	This transect represents the shoreline of Cohasset from the western extent of Pleasant Beach near the intersection of Jerusalem Road and Jerusalem Lane to the intersection of Linden Drive and Jerusalem Road.	9.6	26
42	This transect represents the shoreline of Cohasset from the eastern extent of Pleasant Beach along Atlantic Avenue to the western extent of Pleasant Beach near the intersection of Jerusalem Road and Jerusalem Lane.	9.6	20
43	This transect represents the shoreline of Cohasset from the western extent of Sandy Beach to the eastern extent of Pleasant Beach along Atlantic Avenue.	9.6	24
44	This transect represents the shoreline of Cohasset in Little Harbor from the intersection of Jerusalem Road and Whites Way to the inlet of Little Harbor near the intersection of Hobart Lane and Atlantic Avenue.	9.6	14
45	This transect represents the shoreline of Cohasset from the fork in Joy Place near Gammons Road to intersection of Jerusalem Road and Bow Street.	9.6	21

[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 16 – REVISED TRANSECT DESCRIPTIONS – (CONTINUED)

TRANSECT	LOCATION	1-PERCENT ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT ANNUAL-CHANCE WAVE CREST[1]
46	This transect represents the shoreline of Cohasset along Sandy Beach and any overtopping resulting in wave transmission to the shoreline along Little Harbor to the fork in Joy Place near Gammons Road.	9.6	21
47	This transect represents the shoreline of Cohasset from the inlet of Little Harbor near the intersection of Hobart Lane and Atlantic Avenue to eastern extent of Sandy Beach.	9.6	20
48	This transect represents the shoreline of Cohasset from Quarry Point, just north of Lothrop Lane to the inlet of Little Harbor near the intersection of Hobart Lane and Atlantic Avenue.	9.6	20
49	This transect represents the shoreline of Cohasset from the eastern extent of Whitehead Road to Quarry Point, just north of Lothrop Lane.	9.6	21
50	This transect represents the shoreline of Cohasset from Cohasset Harbor near the intersection of Howard Gleason Road and Atlantic Avenue to the eastern extent of Whitehead Road.	9.6	23
51	This transect represents the shoreline of Cohasset from the North Scituate Corporate Boundary, near the intersection of Gannett Road and Hollett Street, to Cohasset Harbor near the intersection of Howard Gleason Road and Atlantic Avenue.	9.6	21

[1]Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

For the revised open water flooding sources in Cohasset, coastal transect data was extracted from topographic data acquired from 3-meter contour data obtained from the Massachusetts Office of Geographic and Environmental Information (MassGIS). For the revised open water flooding sources in Braintree and Weymouth, coastal transect data was extracted from 2-foot contour data provided by the community. Additionally, portions of twenty (20) coastal transects were field surveyed to supplement the contour data for the restudy area. As appropriate, coastal protection structure details and 0.0 ft NAVD elevation were included and noted in the transect field surveys. Bathymetric data from NOAA Nautical Charts were used to extend the transects offshore for wave runup calculations. Coastal processes that may affect the transect profile, such as dune erosion and seawall scour and failure, were estimated in accordance with Appendix D "Guidance for Coastal Flooding Analyses and Mapping," (Reference 161) of the Guidelines and Specifications, as well as, the "Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update" (Reference 162).

Along each transect in the revised areas, wave envelopes were computed considering the combined effects of changes in ground elevation, vegetation, and physical features. Between transects, elevations were interpolated using topographic maps, land-use and land-cover data, and engineering judgment to determine the extent of flooding. The results of the calculations are accurate until local topography, vegetation, or land development within the community, undergo major changes.

Wave height and runup calculations used in the revised coastal analysis follow the methodologies described in FEMA guidance for V Zone mapping (Reference 161). WHAFIS 3.0 was used to predict wave heights.

FEMA Guidelines (Reference 162) allow for the following methods to be used to determine wave runup: RUNUP 2.0; "Technical Advisory Committee for Water Retaining Structures" (TAW); Automated Coastal Engineering System (ACES); and the Shore Protection Manual (Reference 166). Each of the aforementioned methods has an appropriate set of nearshore conditions for which it should be applied. For example the methods described in the Shore Protection Manual are to be used to determine runup on vertical structures. These methods were applied for each of the restudied coastal transects, as appropriate.

The runup methodologies were used to compute wave envelope elevations associated with the 1-percent-annual-chance storm in Braintree, Cohasset and Weymouth. Accurate topographic, land-use, and land cover data are required for the coastal analyses. Best available contour data were used for revised analyses. Depths below mean low water were determined from National Ocean Survey Coastal Charts (Reference 167). The land-use and land cover data were obtained by field surveys and aerial photographs (Reference 168).

Areas of shallow flooding, designated AO zones, are shown along portions of the shoreline. These areas are the result of wave runup overtopping and ponding behind seawalls and berms with average depths of 1 to 3 feet.

In accordance with FEMA Guidelines (Reference 162) the effect of the Primary Frontal Dune (PFD) on coastal flood hazard mapping was evaluated for all communities. In areas that had appropriate topographic data, the extent of the PFD was calculated in accordance with the Massachusetts Office of Coastal Zone Management methodology (Reference 169), then field verified. For other areas, the extent of the PFD was determined from field survey.

Table 17 "Revised Transect Data," lists the flood hazard zone and base flood elevations for each revised transect, along with the 1-percent-annual-chance stillwater elevation for the respective flooding source.

TABLE 17 – REVISED TRANSECT DATA STILLWATER ELEVATION (FEET NAVD)[1]

FLOODING SOURCE	10-PERCENT ANNUAL - CHANCE	2-PERCENT ANNUAL – CHANCE	1-PERCENT ANNUAL – CHANCE	0.2-PERCENT ANNUAL – CHANCE	ZONE	BASE FLOOD ELEVATION[2]
Town of Cohasset						
MASSACHUSETTS BAY From Cohasset-Hull corporate limits to Government Island, including Little Harbor and Cohasset Cove						
TRANSECT 40	8.4	9.3	9.6	10.4	VE	21
					AE	10
TRANSECT 41	8.4	9.3	9.6	10.4	VE	26
TRANSECT 41	8.4	9.3	9.6	10.4	VE	26
TRANSECT 42	8.4	9.3	9.6	10.4	VE	16-20
TRANSECT 43	8.4	9.3	9.6	10.4	VE	24
					AE	10-13
TRANSECT 47	8.4	9.3	9.6	10.4	VE	16-20
					AE	14
TRANSECT 48	8.4	9.3	9.6	10.4	VE	17-20
					AE	14

[1]North American Vertical Datum of 1988

[2]Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted

TABLE 17 – REVISED TRANSECT DATA STILLWATER ELEVATION (FEET NAVD)[1] – (CONTINUED)

FLOODING SOURCE	10-PERCENT ANNUAL - CHANCE	2-PERCENT ANNUAL – CHANCE	1-PERCENT ANNUAL – CHANCE	0.2-PERCENT ANNUAL – CHANCE	ZONE	BASE FLOOD ELEVATION[2]
TRANSECT 49	8.4	9.3	9.6	10.4	VE	16-21
THE GULF						
From Border Street to 300 feet north of Supper Island						
TRANSECT 50	8.4	9.3	9.6	10.4	VE	18-23
					AE	10-14
TRANSECT 51	8.4	9.3	9.6	10.4	VE	18-21
					AE	10-14
JAMES BROOK						
From Border Street tide gate to Elm Street						
TRANSECT 45	8.4	9.3	9.6	10.4	VE	14
					AE	10-12
TRANSECT 46	8.4	9.3	9.6	10.4	VE	12;18-21
					AE	10-15
RICHARDSONS BROOK						
Upstream of Jerusalem Road						
TRANSECT 44	8.4	9.3	9.6	10.4	AE	10
Town of Weymouth						
HINGHAM BAY						
At Grape Bay						
TRANSECT 36	8.3	9.2	9.5	10.4	VE	13-16

[1]North American Vertical Datum of 1988

[2]Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted

TABLE 17 – REVISED TRANSECT DATA STILLWATER ELEVATION (FEET NAVD)[1] – (CONTINUED)

FLOODING SOURCE	10-PERCENT ANNUAL - CHANCE	2-PERCENT ANNUAL – CHANCE	1-PERCENT ANNUAL – CHANCE	0.2-PERCENT ANNUAL – CHANCE	ZONE	BASE FLOOD ELEVATION[2]
TRANSECT 37	8.3	9.2	9.5	10.4	VE	24; 13-15
					AE	12-14
TRANSECT 38	8.3	9.2	9.5	10.4	VE	18; 13-15
					AE	11-13
WEYMOUTH BACK RIVER						
Entire length within community						
TRANSECT 39	8.3	9.2	9.5	10.4	VE	12-13
					AE	10-12
WEYMOUTH FORE RIVER						
At Rose Cliff						
TRANSECT 31	8.3	9.2	9.5	10.4	VE	14; 12-13
					AE	10-12
TRANSECT 32	8.3	9.2	9.5	10.4	VE	13-14
					AE	11-13
At Wassagusset Beach						
TRANSECT 34	8.3	9.2	9.5	10.4	VE	12-15
					AE	10-12
TRANSECT 35	8.3	9.2	9.5	10.4	VE	20; 12-15
					AE	10-12
Kings Cove						
TRANSECT 33	8.3	9.2	9.5	10.4	VE	22; 15-18
					AE	10-15

[1]North American Vertical Datum of 1988

[2]Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted

TABLE 17 – REVISED TRANSECT DATA STILLWATER ELEVATION (FEET NAVD)[1] – (CONTINUED)

FLOODING SOURCE	10-PERCENT ANNUAL - CHANCE	2-PERCENT ANNUAL – CHANCE	1-PERCENT ANNUAL – CHANCE	0.2-PERCENT ANNUAL – CHANCE	ZONE	BASE FLOOD ELEVATION[2]
Town of Braintree						
WEYMOUTH FORE RIVER						
Quincy corporate limits to Venus Road						
TRANSECT 27	8.4	9.3	9.5	10.4	VE	12-13
					AE	10-12
TRANSECT 28	8.4	9.3	9.5	10.4	VE	12-13
					AE	10-12
TRANSECT 29	8.4	9.3	9.5	10.4	VE	12
					AE	10-12
Venus Road to Shaw Street						
TRANSECT 30	8.4	9.3	9.5	10.4	VE	12-13
					AE	10-12

[1]North American Vertical Datum of 1988

[2]Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted

City of Quincy Coastal Study Update

The energy-based significant wave height (Hmo) and peak wave period (Tp) are used as inputs to wave setup and wave runup calculations and were calculated using the Steady-State Spectral Wave Model (STWAVE) . STWAVE is a phased-averaged spectral wave model that simulates depth-induced wave refraction and shoaling, depth- and steepness-induced wave breaking, diffraction, wind-wave growth, and wave-wave interaction and white capping that redistribute and dissipate energy in a growing wave field. The model accepts a spectral form of the wave as an input condition and provides Hmo and Tp results over the gridded model domain.

Offshore (deepwater) wave heights, wave setup, and wave runup for each transect were calculated using Mathcad sheets developed by STARR to apply methodologies from the USACE's Coastal Engineering Manual (Reference 170) and FEMA Guidelines and Specifications (Reference 162). Methodologies for each type of calculation are discussed in more detail below. Results from the Mathcad calculations performed for each transect were compiled in a summary spreadsheet.

Overland wave heights were calculated for restricted and unrestricted fetch settings using the Wave Height Analysis for Flood Insurance Studies (WHAFIS), Version 4.0 (Reference 171), within the Coastal Hazard Analysis for Mapping Program (CHAMP) (Reference 172), following the methodology described in the FEMA Guidelines and Specifications for each coastal transect.

The general working procedure included eight steps: 1) laying out transects; 2) determining off-shore significant wave heights and corresponding wave periods from STWAVE outputs; 3) performing the off-shore engineering analysis; 4) preparing WHAFIS input data and populating the CHAMP database; 5) performing erosion analysis for erodible transects without a coastal structure; 6) performing WHAFIS modeling runs on eroded transects and transects with both intact and failed structures, as applicable; 7) performing wave runup analysis on intact and failed structures; and 8) identifying primary frontal dunes.

Coastal engineering analysis was performed for each coastal transect using wave condition extracted from the STWAVE model and SWEL data to generate wave setup and wave runup values for open coast transects and transects with vertical structures or revetments, and to generate input used in developing CHAMP and WHAFIS input data. Mathcad sheets were developed and applied by STARR for the calculations to help ensure consistency and accuracy. The input data and results of the analysis were compiled for each transect in a summary spreadsheet. The Mathcad sheets and summary spreadsheet are included in the digital data files compiled for the coastal submittal.

CHAMP is a Microsoft (MS) Windows-interfaced Visual Basic language program that allows the user to enter data, perform coastal engineering analyses, view and tabulate results, and chart summary information for each representative transect along a coastline within a user-friendly graphical interface. With CHAMP, the user can import digital elevation data, perform storm-induced erosion treatments, wave height and wave runup analyses, plot summary graphics of the results, and create summary tables and reports in a single environment. CHAMP version 2.0 (Reference 172) was used to perform erosion analysis, run WHAFIS, and apply RUNUP 2.0 to transects without coastal structures. Application of CHAMP followed the instructions in the FEMA Guidelines and Specifications (Reference 162) and the Coastal Hazard Analysis Modeling Program user's guide found in the software documentation (FEMA, 2007).

Wave setup can be a significant contributor to the total water level at the shoreline and was included in the determination of coastal base flood elevations. Wave setup is defined as the increase in total stillwater elevation against a barrier caused by the attenuation of waves in shallow water. Wave setup is based upon wave breaking characteristics and profile slope. Wave setup values were calculated for each coastal transect using the Direct Integration Method (DIM), developed by Goda (Reference 165), as described in the FEMA Guidelines and Specifications, Equation D.2.6-1. For those coastal transects where a structure was located, documentation was gathered on the structure, and the wave setup against the coastal structure was also calculated.

The fundamental analysis of overland wave effects for an FIS is provided by FEMA's Wave Height Analysis For Flood Insurance Studies computer program, WHAFIS 4.0, a computer program that uses representative transects to compute wave crest elevations in a given study area. Topographic, vegetative, and cultural features are identified along each specified transect landward of the shoreline. WHAFIS uses this and other input information to calculate wave heights, wave crest elevations, flood insurance risk zone designations, and flood zone boundaries along the transects.

The original basis for the WHAFIS model was the 1977 National Academy of Sciences (NAS) report "Methodology for Calculating Wave Action Effects Associated with Storm Surges" (Reference 119). The NAS methodology accounted for varying fetch lengths, barriers to wave transmission, and the regeneration of waves over flooded land areas. Since the incorporation of the NAS methodology into the initial version of WHAFIS, periodic upgrades have been made to WHAFIS to incorporate improved or additional wave considerations.

WHAFIS 4.0 was applied using CHAMP to calculate overland wave height propagation and establish base flood elevations. For profiles with vertical structures or revetments, a failed structure analysis was performed and a new profile of the failed structure was generated and analyzed.

Wave runup is the uprush of water caused by the interaction of waves with the area of shoreline where the stillwater hits the land or other barrier intercepting the stillwater level. The wave runup elevation is the vertical height above the stillwater level ultimately attained by the extremity of the uprushing water. Wave runup at a shore barrier can provide flood hazards above and beyond those from stillwater inundation. Guidance in the FEMA Guidelines and Specifications (Reference 162) suggests using the 2-percent wave runup value, the value exceeded by 2 percent of the runup events. The 2-percent wave runup value is particularly important for steep slopes and vertical structures.

Wave runup was calculated for each coastal transect using methods described in the FEMA Guidelines and Specifications (Reference 162). Runup estimates were developed for vertical walls using the guidance in Figure D.2.8-3 of the FEMA Guidelines and Specifications (Reference 162), taken from the Shore Protection Manual (Reference 166). Technical Advisory Committee for Water Retaining Structures (TAW) method was applied for sloped structures with a slope steeper than 1:8. For slopes milder than 1:8, the FEMA Wave Runup Model RUNUP 2.0 was used. Both the SPM and RUNUP 2.0 provide mean wave runup. The mean wave runup was multiplied by 2.2 to obtain the 2-percent runup height. Wave runup elevation was added to the stillwater elevation and does not include wave setup.

The LiMWA is determined and defined as the location of the 1.5-foot wave. Typical constructions in areas of wave heights less than 3-feet high have experienced damage, suggesting that construction requirements within some areas of the AE zone should be more like those requirements for the VE zone. Testing and investigations have confirmed that a wave height greater than 1.5 feet can cause structure failure. The LiMWA was determined for all areas subject to significant wave attack in accordance with "Procedure Memorandum No. 50 – Policy and Procedures for Identifying and Mapping Areas Subject to Wave Heights Greater than 1.5 feet as an Informational Layer on Flood Insurance Rate Maps (FIRMs)" (Reference 173). The effects of wave hazards in the Zone AE areas (or shoreline in areas where VE Zones are not identified) and the limit of the LiMWA boundary are similar to, but less severe than, those in Zone VE where 3-foot breaking waves are projected during a 1-percent-annual-chance flooding event.

The effects of wave hazards in the Zone AE areas (or shoreline in areas where VE Zones are not identified) and the limit of the LiMWA boundary are similar to, but less severe than, those in Zone VE where 3-foot breaking waves are projected during a 1-percent-annual-chance flooding event.

No significant Primary Frontal Dunes (PFDs) were identified in the City of Quincy, therefore no further PFD analysis was performed in Norfolk County.

The transect schematic Figure 1 represents a sample transect that illustrates the relationship between the stillwater elevation , the wave crest elevation, the ground elevation profile, and the location of the A/V zone boundary.

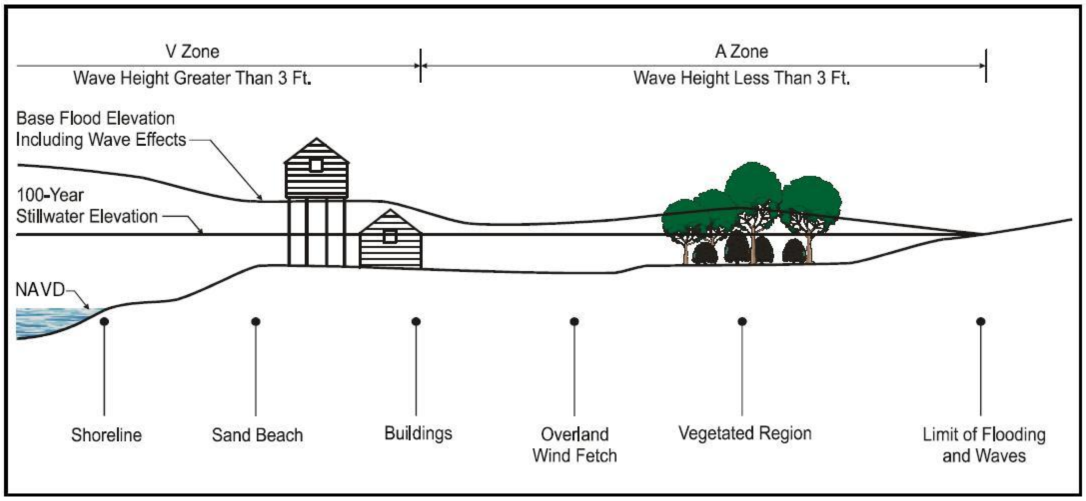

FIGURE 1 – TRANSECT SCHEMATIC

Transects (profiles) were located for coastal hydrologic and hydraulic analyses perpendicular to the average shoreline along areas subject to coastal flooding; transects extend off-shore to areas representative of deep water conditions and extend inland to a point where wave action ceases, in accordance with the User’s Manual for Wave Height Analysis (Reference 174). 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 spaced close together in areas of complex topography and dense development. In areas having more uniform characteristics, 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.

Table 18 provides a description of the transect locations, the 1-percent-annual-chance stillwater elevations, and the maximum 1-percent-annual-chance wave crest elevations. Figure 2, "Transect Location Map," illustrates the location of the transects for the county.

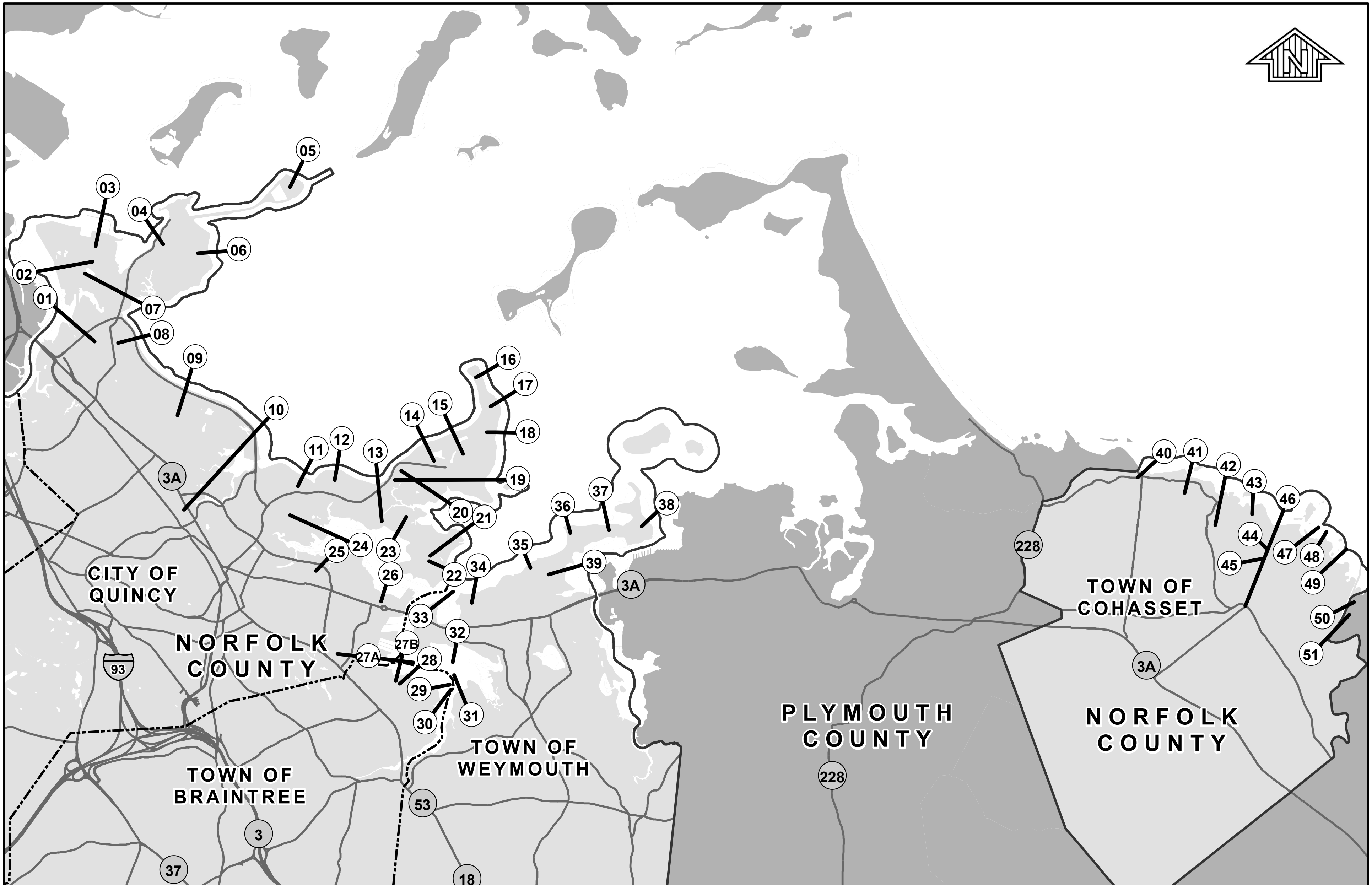

FIGURE 2

FEDERAL EMERGENCY MANAGEMENT AGENCY

NORFOLK COUNTY, MA

(ALL JURISDICTIONS)

APPROXIMATE SCALE

0 3,750 7,500 15,000 22,500 30,000 Feet

TRANSECT LOCATION MAP

TABLE 18 – REVISED TRANSECT DATA STILLWATER ELEVATION - QUINCY (FEET NAVD)[1]

FLOODING SOURCE	10-PERCENT ANNUAL - CHANCE	2-PERCENT ANNUAL – CHANCE	1-PERCENT ANNUAL – CHANCE	0.2-PERCENT ANNUAL – CHANCE	ZONE	BASE FLOOD ELEVATION[2]
City of Quincy						
Entire shoreline						
TRANSECT 1	9.3	10.8	11.3	12.8	VE	14
					AE	12-14
TRANSECT 2	9.3	10.8	11.3	12.8	VE	15-16
					AE	13-15
TRANSECT 3	9.3	10.8	11.3	12.8	VE	16
					AE	14-16
TRANSECT 4	9.3	10.8	11.3	12.8	VE	15
					AE	13-15
TRANSECT 5	9.0	9.9	10.7	11.9	VE	15-16
					AE	12-15
TRANSECT 6	9.0	9.9	10.7	11.9	VE	14-17
					AE	14
TRANSECT 7	9.0	9.9	10.7	11.9	VE	13-16
					AE	13
TRANSECT 8	9.0	9.9	10.7	11.9	VE	14-16
					AE	12-14
TRANSECT 9	9.0	9.9	10.7	11.9	VE	14-16
					AE	12-14
TRANSECT 10	9.2	10.3	10.9	12.2	VE	15-17
					AE	13-15

[1]North American Vertical Datum of 1988

[2]Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted

TABLE 18 – REVISED TRANSECT DATA STILLWATER ELEVATION - QUINCY (FEET NAVD)[1] - CONTINUED

FLOODING SOURCE	10-PERCENT ANNUAL - CHANCE	2-PERCENT ANNUAL – CHANCE	1-PERCENT ANNUAL – CHANCE	0.2-PERCENT ANNUAL – CHANCE	ZONE	BASE FLOOD ELEVATION[2]
TRANSECT 11	9.2	10.3	10.9	12.2	VE	14-17
					AE	12-14
TRANSECT 12	9.2	10.3	10.9	12.2	VE	15-18
					AE	12-15
TRANSECT 13	9.0	10.0	10.8	12.1	VE	14-15
					AE	12-14
TRANSECT 14	9.0	10.0	10.8	12.1	VE	15-18
					AE	13-15
TRANSECT 15	9.0	10.0	10.8	12.1	VE	14-16
					AE	12-14
TRANSECT 16	9.4	10.8	11.7	12.8	VE	16
					AE	-
TRANSECT 17	9.4	10.8	11.7	12.8	VE	16
					AE	-
TRANSECT 18	9.4	10.8	11.7	12.8	VE	16
					AE	14-16
TRANSECT 19	9.4	10.8	11.7	12.8	VE	15
					AE	13-15
TRANSECT 20	9.5	10.9	12.0	13.0	VE	16
					AE	14-16
TRANSECT 21	9.5	10.9	12.0	13.0	VE	15
					AE	13-15
TRANSECT 22	9.5	10.9	12.0	13.0	VE	15
					AE	13-15

[1]North American Vertical Datum of 1988

[2]Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted

TABLE 18 – REVISED TRANSECT DATA STILLWATER ELEVATION - QUINCY (FEET NAVD)[1] - CONTINUED

FLOODING SOURCE	10-PERCENT ANNUAL - CHANCE	2-PERCENT ANNUAL – CHANCE	1-PERCENT ANNUAL – CHANCE	0.2-PERCENT ANNUAL – CHANCE	ZONE	BASE FLOOD ELEVATION[2]
TRANSECT 23	9.6	11.0	12.1	13.1	VE	16
					AE	14-16
TRANSECT 24	9.6	11.0	12.1	13.1	VE	15
					AE	13-15
TRANSECT 25	9.1	10.9	11.7	13.3	VE	15-16
					AE	14-15
TRANSECT 26	9.1	10.9	11.7	13.3	VE	16
					AE	-
TRANSECT 27A	9.1	10.9	11.7	13.3	VE	16
					AE	14-16

[1]North American Vertical Datum of 1988

[2]Due to map scale limitations, base flood elevations shown on the FIRM represent average elevations for the zones depicted

3.5 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 partial countywide 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 feet, and from NAVD 88 to NGVD 29 is +0.8 feet.**

For information regarding conversion between the NGVD and NAVD, 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.