

FLOOD INSURANCE STUDY



VOLUME 1 OF 4

BRISTOL COUNTY, MASSACHUSETTS (ALL JURISDICTIONS)

COMMUNITY NAME

COMMUNITY NUMBER

ACUSHNET, TOWN OF
 ATTLEBORO, CITY OF
 BERKLEY, TOWN OF
 DARTMOUTH, TOWN OF
 DIGHTON, TOWN OF
 EASTON, TOWN OF
 FAIRHAVEN, TOWN OF
 FALL RIVER, CITY OF
 FREETOWN, TOWN OF
 MANSFIELD, TOWN OF
 NEW BEDFORD, CITY OF
 NORTH ATTLEBOROUGH, TOWN OF
 NORTON, TOWN OF
 RAYNHAM, TOWN OF
 REHOBOTH, TOWN OF
 SEEKONK, TOWN OF
 SOMERSET, TOWN OF
 SWANSEA, TOWN OF
 TAUNTON, CITY OF
 WESTPORT, TOWN OF

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 255224



REVISED
 JULY 16, 2015



Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
 25005CV001C

NOTICE TO
FLOOD INSURANCE STUDY USERS

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

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

<u>Old Zone</u>	<u>New Zone</u>
A1 through A30	AE
V1 through V30	VE (shaded)
B	X
C	X

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

Initial Countywide FIS Effective Date: July 7, 2009

Revised Countywide FIS Effective Date: July 16, 2014

Second Revised Countywide FIS Effective Date: July 16, 2015

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Berkley, Town of: The hydrologic and hydraulic analyses for the January 1978 study were performed by Camp, Dresser & McKee, Inc. (CDM) for the FIA, under Contract No. H-3861. This work, which was completed in August 1976, covered all significant flooding sources affecting the Town of Berkley.

Dartmouth, Town of: The original hydrologic and hydraulic analyses in the January 3, 1985 study were completed in December 1974 by the New England Division of the U.S. Army Corps of Engineers (USACE) for FEMA, under Inter-Agency Agreement No. IAA-H-2-73, Project Order No. 4. An updated version of the study for stillwater elevation re-analysis was completed in March 1979 by Stone and Webster Engineering Corporation for FEMA.

A second updated version to include wave runup and wave height analyses was also prepared by Stone and Webster Engineering Corporation for FEMA, under Contract No. H-4604. This work was completed in September 1983.

Dighton, Town of: The December 1979 study was conducted by Sverdrup & Parcel and Associates, Inc. for the FIA, under Contract No. H-4037. This work, which was completed in May 1978, covered all significant flooding sources affecting the Town of Dighton.

Easton, Town of: For the original August 3, 1981 FIS and February 3, 1982 FIRM, the hydrologic and hydraulic analyses were prepared by the U.S. Geological Survey (USGS) for FEMA, under Inter-Agency Agreement No. IAA-H-9-77, Project Order No. 8. That work was completed in June 1979.

For the May 16, 1995 revision, the hydrologic and hydraulic analyses were prepared by Green International Affiliates, Inc. for FEMA, under Contract No. EMW-89-C-2820. That work was completed in December 1991.

In the August 9, 2000 revision, the hydraulic analyses were prepared by Green International Affiliates, Inc. for FEMA, under Contract No. EMB-96-CO-0403 (Task #10). This work was completed in March 1998.

Fairhaven, Town of: The hydrologic and hydraulic analyses in the December 4, 1984 study represent an update of the original analyses performed by the New England Division of USACE for FEMA, under Inter-Agency Agreement No. IAA-H-2-73. The updated version was prepared by Stone and Webster Engineering Corporation for FEMA, under Contract No. H-4604. The updated study was completed in September 1983.

Fall River, City of: The hydrologic and hydraulic analyses in the July 5, 1983 study represent revisions of the original analyses by the New England Division of USACE. The updated version was prepared by Stone and Webster Engineering Corporation for FEMA, under Contract No. H-4604. The stillwater flooding portion of this study was completed in June 1979. The wave runup and wave height analyses were completed in April 1982.

Freetown, Town of: The December 1979 study was conducted by Sverdrup & Parcel and Associates, Inc. for the FIA, under Contract No. H-4037. This work, which was completed in February 1978, covered all significant flooding sources affecting the Town of Freetown.

Mansfield, Town of: The October 1976 study was conducted by Anderson-Nichols & Co., Inc. at the request of the FIA, U.S. Department of Housing and Urban Development. Authority and financing between the contractor and the FIA are contained in Contract No. H-3707.

New Bedford, City of: The hydrologic and hydraulic analyses in the July 5, 1983 study represent revisions of the original analyses by the New England Division of USACE, under Inter-Agency Agreement No. IAA-H-1572. The updated version was prepared by Stone and Webster Engineering Corporation under agreement with FEMA. The stillwater flooding portion of this study was completed in February 1979. The wave runup and wave height analyses were completed in April 1982.

North Attleborough, Town of: The hydrologic and hydraulic analyses for the March 1979 study were performed by the SCS for the FIA, under Inter-Agency Agreement No. IAA-H-9-76, Project Order No. 9. This work, which was completed in August 1977, covered

North Attleborough, Town of – continued:	all significant flooding sources affecting the Town of North Attleborough.
Norton, Town of:	The hydrologic and hydraulic analyses in the June 18, 1987 study represent a revision of the original analyses prepared by the SCS for FEMA. The work for the original study was completed in June 1977. The hydrologic and hydraulic analyses in this updated study were prepared by CDM for FEMA, under Contract No. EMW-84-C-1601. The work for this study was completed in August 1985.
Raynham, Town of:	The hydrologic and hydraulic analyses for the January 1980 study were prepared by the USGS for the FIA, under Inter-Agency Agreement No. IAA-H-9-77, Project Order No. 9. This work, which was completed in June 1978, covered all significant flooding sources in the Town of Raynham.
Rehoboth, Town of:	The hydrologic and hydraulic analyses for the September 1977 study were performed by Anderson-Nichols & Co., Inc. for the FIA, under Contract No. H-3715. This work, which was completed in April 1976, covered all flooding sources affecting the Town of Rehoboth.
Seekonk, Town of:	The hydrologic and hydraulic analyses for the March 1979 study were performed by the SCS for the FIA, under Inter-Agency Agreement No. IAA-H-9-76, Project Order No. 9. This work, which was completed in November 1977, covered all significant flooding sources affecting the Town of Seekonk.
Somerset, Town of:	The hydrologic and hydraulic analyses in the December 5, 1984 study represent an update of the original analyses performed by the New England Division of USACE for FEMA, under Inter-Agency Agreement No. IAA-H-8-71. The updated version was prepared by Stone and Webster Engineering Corporation for FEMA, under Contract No. H-4604. The stillwater flooding portion of this study was completed in April 1979. The wave runup and wave height analyses were completed in October 1983.
Swansea, Town of:	The hydrologic and hydraulic analyses in the July 17, 1986 study represent a revision of the original analyses by the USGS for FEMA. The updated

Swansea, Town of – continued:

version was prepared by Stone and Webster Engineering Corporation for FEMA, under Contract No. H-4604. The stillwater analysis in this study was completed in April 1979. The wave runup and wave height analyses were completed in October 1983. The hydrologic and hydraulic analyses for Rocky Run were performed by Anderson-Nichols & Co., Inc. for FEMA, during the preparation of the FIS for the Town of Rehoboth.

Taunton, City of:

The hydrologic and hydraulic analyses in the June 18, 1987 study represent a revision of the original study performed by the USGS for FEMA, under Inter-Agency Agreement No. IAA-H-9-77, Project Order No. 21. The original work was completed in March 1978. The hydrologic and hydraulic analyses in this updated study were prepared by CDM for FEMA, under Contract No. EMW-84-C-1601. This work was completed in October 1985.

Westport, Town of:

The hydrologic and hydraulic analyses in the September 18, 1984 study represent a revision of the original analyses by the USGS for FEMA, under Inter-Agency Agreement No. IAA-H-19-71. The updated version was prepared by Stone and Webster Engineering Corporation, under agreement with FEMA. The revised study was completed in July 1979. The addition of wave runup and wave height analyses was also performed by Stone and Webster Engineering Corporation for FEMA, under Contract No. H-4772. The wave runup and wave height analyses were completed in May 1983.

For the July 7, 2009 countywide FIS, coastal hydrologic and hydraulic analyses for the City of New Bedford and the Towns of Dartmouth, Fairhaven, and Westport were prepared by CDM for FEMA, under Contract No. EME-2003-CO-0340, and by Ocean and Coastal Consultants, Inc. for CDM, under Contract No. 2809-999-003-CS. This study was completed March 28, 2008.

The coastal analysis for the June 16, 2014 countywide revision was prepared by the Strategic Alliance for Risk Reduction (STARR) for FEMA, under Contract No. HSFEHQ-09-D-0370 and completed in November 2012. This new analysis resulted in revisions to the Special Flood Hazard Areas (SFHAs) within the City of Fall River and the Towns of Berkley, Dighton, Freetown, Rehoboth, Seekonk, Somerset, and Swansea.

Base map information shown on this FIRM was derived from digital orthophotography. Base map files were provided in digital form by Massachusetts Geographic Information

System (MassGIS). Bristol County orthophotography was collected at 15-cm or 30-cm pixel resolution. Aerial photography is dated April 2008 and March and April 2009 (Reference 1). The projection used in the preparation of this map was Massachusetts State Plane mainland zone (FIPZONE2001). The horizontal datum was NAD83, GRS1980 spheroid.

For the July 16, 2015 revision of the countywide FIS, hydrologic and hydraulic analyses were prepared by the U.S. Geological Survey (USGS) for FEMA under Contract No. HSFE01-11-X-0083, affecting SFHAs in the Cities of Attleboro and Taunton and the Towns of Acushnet, Berkley, Dighton, Freetown, Mansfield, North Attleborough, Norton, Raynham, and Seekonk. This study was completed March 24, 2014.

Base map information shown on this FIRM was derived from digital orthophotography. Base map files were provided in digital form by the USGS. Ortho imagery was produced at a scale of 1:2,400. Aerial photography is dated April 2013 or, in areas not covered by the 2013 project, March and April 2009. The projection used in the preparation of this map was Massachusetts State Plane mainland zone (FIPZONE2001). The horizontal datum was NAD83, GRS1980 spheroid.

1.3 Coordination

The purpose of an initial Consultation Coordination Officer’s (CCO) meeting is to discuss the scope of the FIS. A final meeting is held to review the results of the study.

The dates of the initial, intermediate and final CCO meetings held for the incorporated communities within Bristol County are shown in Table 1, “CCO Meeting Dates for Precountywide FIS.”

TABLE 1 – CCO MEETING DATES FOR PRECOUNTYWIDE FIS

<u>Community Name</u>	<u>Initial CCO Date</u>	<u>Intermediate CCO Date</u>	<u>Final CCO Date</u>
Town of Acushnet	March 1978	*	January 26, 1981
City of Attleboro	May 12, 1975	May 11, 1977	September 14, 1977
Town of Berkley	*	*	October 28, 1978
Town of Dartmouth	August 1, 1977	August 23, 1983	May 14, 1984
Town of Dighton	May 1976	October 1976	February 28, 1979
Town of Easton	September 11, 1997	*	March 22, 1999
Town of Fairhaven	August 1, 1977	September 15, 1983	July 17, 1984
City of Fall River	August 3, 1977	*	January 11, 1983
Town of Freetown	May 1976	June 26, 1978	April 2, 1979
Town of Mansfield	December 6, 1974	*	August 27, 1975
Town of North Attleborough	January 21, 1976	July 26, 1977	August 16, 1978
City of New Bedford	August 1, 1977	*	January 11, 1983
Town of Norton	April 1984	*	July 24, 1986
Town of Raynham	*	June 22, 1978	June 28, 1979
Town of Rehoboth	January 14, 1975	December 26, 1975	June 15, 1976
Town of Seekonk	January 20, 1976	July 26, 1977	March 3, 1978

*Data not available

TABLE 1 – CCO MEETING DATES FOR PRECOUNTYWIDE FIS - continued

<u>Community Name</u>	<u>Initial CCO Date</u>	<u>Intermediate CCO Date</u>	<u>Final CCO Date</u>
Town of Somerset	August 3, 1977	October 31, 1983	June 26, 1984
Town of Swansea	August 3, 1977	October 11, 1983	September 19, 1984
City of Taunton	April 11, 1984	*	September 5, 1986
Town of Westport	August 3, 1977	May 17, 1983	December 1, 1983

*Data not available

For the July 7, 2009 countywide study, the initial CCO meeting was held on March 8, 2005, and was attended by representatives of FEMA, Southeastern Regional Planning and Economic Development District Office (SRPEDD), the communities, and ENSR.

The results of the study were reviewed at the final CCO meeting held on June 24, 25, and 26 of 2008, and attended by representatives of FEMA, the communities, Massachusetts Department of Conservation and Recreation (MADCR), Regional Management Center for Region I (RMC I), and CDM. All problems raised at that meeting have been addressed in the July 7, 2009 countywide study.

For the June 16, 2014 countywide FIS, which includes an updated coastal and backwater analysis along the shorelines and rivers of the City of Fall River and the Towns of Berkley, Dighton, Freetown, Rehoboth, Seekonk, Somerset, and Swansea, two separate initial CCO meetings were held on February 16, 2011 in the City of Taunton. The results of this countywide study were reviewed at the final CCO meeting held on January 24, 2013 in the Town of Swansea. The meetings were attended by representatives of FEMA Region I, STARR, SRPEDD, and state and community officials. All issues raised at the meetings have been addressed in this study.

For the July 16, 2015 revision, the Discovery meeting for Bristol County was held on December 6, 2011 at the City Hall in Taunton. Two workmap meetings were held on September 16, 2013 that communities in Bristol County could attend: one at the public library in Attleboro, and the second at the public library in Lakeville. The initial CCO meeting was held on September 16, 2013, and was attended by representatives of FEMA Region I, STARR, USGS, the Massachusetts Department of Conservation and Recreation (MADCR), and the communities. The results of the study were reviewed at the final CCO meeting held on June 24, 2014, which was attended by representatives of the same organizations. All problems raised at that meeting have been addressed in this study.

2.0 AREA STUDIED

2.1 Scope of Study

July 7, 2009 Countywide Analysis

The July 7, 2009 FIS report covers the geographic area of Bristol County, Massachusetts, including the incorporated communities listed in Section 1.1. The areas studied by detailed methods were selected with priority given to all known flood hazards and areas of projected development or proposed construction.

All or portions of the flooding sources listed in Table 2, “Flooding Sources Studied by Detailed Methods,” were studied by detailed methods in the precountywide FISs. Limits of detailed study are indicated on the Flood Profiles (Exhibit 1) and on the FIRM. The areas studied by detailed methods were selected with priority given to all known flood hazards and areas of projected development or proposed construction.

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS

<u>Flooding Source Name</u>	<u>Description of Study Reaches</u>
Abbott Run	From approximately 200 ft. downstream of Meadow Road to the North Attleborough corporate limits
Acushnet River	From the downstream Acushnet corporate limits to the New Bedford Reservoir Flooding behind the hurricane barrier in Fairhaven
Anawan Brook	From its confluence with East Branch Palmer River to Kelton Street Extension Bridge in Rehoboth
Armstrong Brook	From its confluence with Bungay River to approximately 200 ft upstream of Lindsey Street in North Attleborough
Assonet River	For its entire length
Atlantic Ocean	Tidal flooding including its wave action from Buzzards Bay, Mount Hope Bay, Taunton River, Lee River, Cole River below Milford Pond Dam, the Palmer River, Tributary to the Barrington River, Three Mile River, the Mill River, and Cobb Brook Portions of the Acushnet River behind the hurricane barrier, and all estuaries within the City of New Bedford Coastal flooding, including its wave action, from Rhode Island Sound affecting Westport Harbor, the East Branch Westport River, and the West Branch Westport River
Attleboro Industrial Stream	From its confluence with Ten Mile River to Tiffany Street in Attleboro
Bad Luck Brook	From its confluence with East Branch Palmer River to a point approximately 0.76 miles upstream of the confluence

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS -
continued

<u>Flooding Source Name</u>	<u>Description of Study Reaches</u>
Black Brook	From Foundry Street in Easton to a point approximately 1,310 feet upstream of Randall Street
Bliss Brook	From its confluence with West Branch Palmer River to the Agricultural Avenue Bridge
Bungay River	From its confluence with Ten Mile River to just downstream of Bungay Road in North Attleboro
Buttonwood Brook	For the entire length within the Town of Dartmouth
Buttonwood Brook East	For the entire length within the Town of Dartmouth
Buttonwood Brook West	For the entire length within the Town of Dartmouth
Canoe River (Lower Reach)	From confluence with Winnecunnet Pond in the Town of Norton to approximately 5,000 feet upstream of Interstate Route 495
Canoe River (Upper Reach)	From a point approximately 31,850 feet upstream of confluence with Winnecunnet Pond in the Town of Norton to East Street in the Town of Mansfield
Chartley Brook	From the downstream Attleboro corporate limits to a point approximately 100 ft upstream of Wilmarth Street
Cobb Brook	From its confluence with the Taunton River upstream to Tremont Street in the City of Taunton
Coles Brook	From confluence with Central Pond in the Town of Seekonk to 1,500 feet upstream of Talbot Way
Dam Lot Brook	From its confluence with the Taunton River to its confluence with Tributary to Dam Lot Brook
Deep Brook	From its confluence with the Acushnet River to a point approximately 1 mile upstream

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS -
continued

<u>Flooding Source Name</u>	<u>Description of Study Reaches</u>
East Branch Palmer River	From its confluence with Palmer River to the Fairfield Street Bridge in the Town of Rehoboth
East Junction Stream	From its confluence with Ten Mile River to railroad crossing in the City of Attleboro
Elmwood Street Brook	From its confluence with Ten Mile River to 0.02 mile upstream of Parmenter Lane in the Town of North Attleborough
Fall Brook	From confluence with Long Pond to the dam 100 feet upstream of Chace Road in the Town of Freetown
Forge River	From confluence with Taunton River to the old railroad grade west of State Route 138 in the Town of Raynham
Goose Branch Brook	From its confluence with Wading River to approximately 50 feet upstream of West Hodges Street in the Town of Norton
Gowards Brook	From its confluence with the Canoe River to a point approximately 100 feet upstream of State Route 106 in the Town of Easton
Hodges Brook	From confluence with wading River to just downstream of the Penn Central Railroad in the Town of Mansfield
Lake Como Stream	From its confluence with Seven Mile River to a point 1 mile upstream
Lake Sabbatia	For the entire shoreline within Taunton
Landry Avenue Brook	From its confluence with Bungay River to 0.02 mile upstream of Hall Drive in the Town of North Attleborough
Mary Kennedy Brook	From its confluence with Bungay River to Kelly Boulevard in the Town of North Attleborough
Mason Park Brook	From its confluence with Ten Mile River to Landry Lane in the Town of North Attleborough
Mill Pond	For the entire shoreline within the City of Taunton

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS -
continued

<u>Flooding Source Name</u>	<u>Description of Study Reaches</u>
Mill River	From its confluence with the Taunton River to a point approximately 250 feet upstream of Whittenton Street in the City of Taunton
Mulberry Brook	From approximately 17,200 feet above Plain Street in the Town of Easton to its confluence with Beaver Brook
Oak Hill Stream	From the Seekonk corporate limits to the railroad crossing in the Town of Seekonk
Oak Swamp Brook	From its confluence with Rocky Run to a point approximately 4,600 ft upstream of the Providence Street Bridge
Palmer River	From the downstream Rehoboth corporate limits to its confluence with East and West Branch Palmer River
Paskamanset River	From a point approximately 28,000 feet above confluence with Slocums River to a point approximately 700 feet upstream from Mill Dam in the Town of Dartmouth
Poquanticut Brook	From its confluence with Beaver Brook to a point approximately 1,030 feet upstream of Rockland Street in the Town of Easton
Queset Brook	From 1,600 feet above Walnut Street in the Town of Easton to a point approximately 1,480 feet upstream of Canton Street in the Town of Easton
Rattlesnake Brook (Freetown)	From confluence with Assonet Bay to 350 feet upstream of State Route 24 in the Town of Freetown
Rattlesnake Brook (North Attleborough)	From its confluence with Ten Mile River to 0.03 mile upstream of Towne Street in the Town of North Attleborough
Rocklawn Avenue Stream	From its confluence with Seven Mile River to Rocklawn Avenue in the City of Attleboro
Rocky Run	From confluence with Palmer River to a point approximately 3,400 feet upstream of Private Road Dam in the Town of Rehoboth

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS -
continued

<u>Flooding Source Name</u>	<u>Description of Study Reaches</u>
Rumford River (Lower Reach)	From confluence with the Three Mile River to approximately 6,000 feet upstream of Cross Street in Norton
Rumford River (Upper Reach)	From Norton Reservoir to approximately 700 feet upstream of County Street in the Town of Mansfield
Runnins River	From Mobile Company Dam to Greenwood Avenue in Norton
Sabin Pond Brook	From confluence with Palmer River to a point approximately 0.83 miles upstream of the confluence
Scotts Brook	From its confluence with Ten Mile River to 0.17 mile upstream of High Street in the Town of North Attleborough
Segreganset River (Lower Reach)	From confluence with Taunton River to 700 feet upstream of confluence with Unnamed Tributary
Segreganset River (Upper Reach)	From 300 feet downstream of U.S. Route 44 / Winthrop Street at the Taunton corporate limits upstream to Glebe Street
Sevenmile River ¹	From Attleboro corporate limits to Hoppin Hill Road in the Town of North Attleborough
Speedway Brook	From confluence with Ten Mile River to Maple Street in the City of Attleboro
Sunken Brook	From confluence with Segreganset River to a point 3,500 feet upstream of Center Street in the Town of Dighton
Sweedens Swamp	For its entire length within the City of Attleboro
Taunton River ¹	From 1,200 feet downstream of its confluence with Forge River at the Town of Raynham corporate limits to 2,800 feet upstream of State Route 25, at Bristol County limits

¹Flooding source re-studied during July 16, 2015 revision (see Table 3)

TABLE 2 – FLOODING SOURCES STUDIED BY DETAILED METHODS -
continued

<u>Flooding Source Name</u>	<u>Description of Study Reaches</u>
Ten Mile River ¹	For its entire length within the City of Attleboro and Town of North Attleborough and from the Town of Seekonk's northern corporate limits to Old Mill Road
Three Mile River	From confluence with the Taunton River upstream to Tremont Street in the City of Taunton
Three Mile River – West Channel	From its confluence with the Three Mile River to its divergence from the Three Mile River
Tributary to Dam Lot Brook	From its confluence with Dam Lot Brook to a point approximately 3,000 feet upstream
Tributary to Forge River	From its confluence with Forge River to a point approximately 3,925 feet upstream of White Street
Wading River (Lower Reach) ¹	From its confluence with the Three Mile River to the upstream Town of Norton corporate limits
Wading River (Upper Reach) ¹	From the Town of Mansfield corporate limits to West Street Bridge
Warren Reservoir	For the entire shoreline within the Town of Somerset
Watson Pond	For the entire shoreline within the City of Taunton
West Branch Palmer River	From its confluence with Palmer River to the Fairfield Street in the Town of Rehoboth
Whiting Pond Bypass ¹	From its confluence with Ten Mile River to 1,700 feet upstream, at the North Attleborough corporate limits
Whitman Brook	From its confluence with Queset Brook to 2,000 feet upstream of the railroad crossing in the Town of Easton
Winnecunnet Pond	For the entire shoreline within the Town of Norton

¹Flooding source re-studied during July 16, 2015 revision (see Table 3)

Detail-studied streams that were not re-studied as part of the July 7, 2009 or July 16, 2015 studies 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.

June 16, 2014 Countywide Analysis

The coastal wave height analysis for the June 16, 2014 countywide coastal study was prepared by STARR. This new analysis resulted in revisions to the FIRM for the City of Fall River and the Towns of Berkley, Dighton, Freetown, Somerset, and Swansea. Additionally, new coastal analyses performed in adjacent counties resulted in revisions to the FIRM for the Towns of Rehoboth and Seekonk. One LOMC was incorporated into this revision as described in Table 5. The New Bedford-Fairhaven hurricane barrier is also now shown on the effective FIRM as accredited and providing protection from the 1-percent-annual- chance-flood.

July 16, 2015 Countywide Analysis

The riverine flooding analysis for the July 16, 2015 countywide study was prepared by USGS. This new analysis updated the hydrologic and hydraulic engineering data for the Sevenmile, Taunton, Ten Mile, and Wading Rivers, Whiting Pond Bypass, and Long Pond, as described in Table 3, in Bristol County and its neighboring counties. The analysis resulted in revisions to the FIRM for the Cities of Attleboro and Taunton and the Towns of Acushnet, Berkley, Dighton, Freetown, Mansfield, North Attleborough, Norton, Raynham, and Seekonk. No LOMCs were incorporated into this revision.

For flooding sources studied by detailed methods for the July 7, 2009 countywide study and June 16, 2014 and July 16, 2015 revisions, see Table 3, “Scope of Revision.”

TABLE 3 – SCOPE OF REVISION

<u>Flooding Source</u>	<u>Limits of Revised or New Detailed Study</u>
BUZZARDS BAY ¹	For the entire shoreline within the City of New Bedford and the Towns of Dartmouth and Fairhaven
RHODE ISLAND SOUND ¹	For the entire shoreline within the Town of Westport
MOUNT HOPE BAY ²	Coastal analysis within the City of Fall River and the Towns of Somerset and Swansea
TAUNTON RIVER ²	Coastal analysis within the City of Fall River and the Towns of Berkley, Dighton, Freetown, and Somerset

¹July 7, 2009 study

²June 16, 2014 study

TABLE 3 – SCOPE OF REVISION - continued

<u>Flooding Source</u>	<u>Limits of Revised or New Detailed Study</u>
LONG POND ³	For the entire shoreline within the Town of Freetown
TAUNTON RIVER ³	From the Cherry Street bridge between Halifax and Bridgewater to the Plain Street bridge just upstream of the confluence with the Three Mile River in Taunton
TEN MILE RIVER ³	From the High Street bridge in Plainville to the Railroad Bridge and Omega Pond Dam just upstream of the confluence with the Seekonk River in East Providence, RI
SEVENMILE RIVER ³	From the outflow of an unnamed pond about 100 feet east of Peck Road in Plainville to the Amtrak Railroad Bridge about 0.52 mile upstream of the confluence with the Ten Mile River in East Providence, RI
WADING RIVER ³	From the Cedar Street Bridge in Foxborough to the confluence with Three Mile River in Norton
WHITING POND BYPASS ³	For the entire reach within the Towns of North Attleborough and Plainville

³July 16, 2015 study

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 Bristol County. For the countywide revisions, no new approximate studies were executed. All or portions of the flooding sources listed in Table 4, “Flooding Sources Studied by Approximate Methods,” were studied by approximate methods in the precountywide FISs.

TABLE 4 – FLOODING SOURCES STUDIED BY APPROXIMATE METHODS

<u>Flooding Source Name</u>	<u>Community (s)</u>
Acushnet Cedar Swamp	New Bedford
Ames Long Pond	Easton
Ames Pond	Easton
Ashley Brook	Freetown
Bassett Brook	Raynham
Beaver Brook	Easton
Bigney Pond	Easton
Birch Brook	Norton
Black Brook	Easton
Bleachery Pond	Fall River
Blossom Brook	Fall River
Bolton Cedar Swamp	Freetown

TABLE 4 – FLOODING SOURCES STUDIED BY APPROXIMATE METHODS -
continued

<u>Flooding Source Name</u>	<u>Community (s)</u>
Canoe River	Mansfield, Norton
Chartley Brook	Attleboro
Clear Run Brook	Seekonk
Cole River	Dighton, Swansea
Coles Brook	Seekonk
Cone River	Easton
Cook Pond	Fall River
Cooper Pond	Attleboro
Copicut River	Dartmouth, Fall River
Cotley River	Berkley, Taunton
Cranberry bogs	Acushnet
Daley Brook	Easton
Deerfield Swamp	Dartmouth
Destruction Brook	Dartmouth
Dorchester Brook	Easton
Fall Brook	Freetown, Taunton
French Pond	Easton
Fuller Hammond Reservoir	Easton
Furnace Brook	Taunton
Goose Branch Brook	Norton
Gowards Brook	Easton
Greenwood Lake	North Attleborough
Hathaway Swamp	Acushnet
Heath Brook	Swansea
Hemlock Swamp	Norton
Henkes Brook	Mansfield
Hockomock Swamp	Easton
Hodges Brook	Mansfield
Hoppin Hill Reservoir	North Attleborough
Keene River	Acushnet, Freetown
Kickamuit River	Swansea
King Phillip Brook	Fall River
Labor in Vain Brook	Dighton
Leach Pond	Easton
Lewin Brook	Swansea
Little Cedar Swamp	Easton
Long Pond	Freetown
Manchester Pond	Attleboro
Meadow Brook	Norton
Meadowbrook Pond	Norton
Mill Brook	Fall River
Monte Pond	Easton
Muddy Cove Brook	Dighton
Mulberry Brook	Easton
Mulberry Meadow Brook	Norton
New Bedford Reservoir	Acushnet

TABLE 4 – FLOODING SOURCES STUDIED BY APPROXIMATE METHODS -
continued

<u>Flooding Source Name</u>	<u>Community (s)</u>
Noquochoke Lake	Dartmouth
North Watuppa Pond	Fall River, Westport
Norton Reservoir	Norton
Oak Hill Stream	Seekonk
Old Pond	Easton
Paskamaset River	Dartmouth
Pine Swamp Brook	Raynham
Poppasquash Swamp	Dighton
Poquanticut Brook	Easton
Puds Pond	Easton
Quaker Brook	Berkley, Freetown
Queen Gutter Brook	Fall River
Queset Brook	Easton
Rattlesnake Brook	Freetown
Robin Hollow Pond	North Attleborough
Robinson Brook	Mansfield
Rumford River	Norton
Runnins River	Seekonk
Sawdy Pond	Fall River, Westport
Segreganset River	Dighton
Seven Mile Bypass	Attleboro
Shingle Island River	Dartmouth
Slab Brook	Freetown
Snake River	Taunton
South Watuppa Pond	Fall River, Westport
Squam Brook	Acushnet, Freetown
Sunken Brook	Dighton
Swampy areas	Acushnet
Terry Brook	Freetown
Three Mile River	Norton, Taunton
Tinkham Pond	Acushnet
Torrey Creek	Seekonk
Unnamed Areas	Countywide
Unnamed Ponds	Countywide
Unnamed Streams	Countywide
Unnamed Swamps	Rehoboth, Swansea, Taunton
Unnamed Tributary to Black Brook	Easton
Unnamed Tributary to Poquanticut Bk	Easton
Wading River	Mansfield
Ward Pond	Easton
Whitesville Pond	Mansfield
Whitman Brook	Easton
Witch Pond and Swamp	Mansfield

The initial and revised countywide FISs also incorporate the determinations of letters issued by FEMA resulting in map changes (Letters of Map Revision [LOMR], Letters of

Map Revision - based on Fill [LOMR-F], and Letters of Map Amendment [LOMA]), as shown in Table 5, “Letters of Map Change.”

TABLE 5 – LETTERS OF MAP CHANGE

<u>Community</u>	<u>Case Number</u>	<u>Flooding Source</u>	<u>Letter Date</u>
Easton, Town of ¹	00-01-021P	Gowards Brook	08/10/2000
Easton, Town of ¹	01-01-003P	Unnamed Tributary	02/01/2001
Mansfield, Town of ¹	95-01-035P	Wading River	05/02/1996
Swansea, Town of ²	10-01-1791P	Warren Reservoir	10/04/2010
Taunton, City of ¹	06-01-B096P	Taunton River	10/24/2006

¹Incorporated during July 7, 2009 study

²Incorporated during June 16, 2014 study

2.2 Community Description

Bristol County is located in southeast Massachusetts. There are four cities and sixteen towns in Bristol County. The Cities of Attleboro and Taunton and the Towns of North Attleborough, Mansfield, Easton, Norton, Raynham are located in northern Bristol County. The City of Fall River and the Towns of Seekonk, Rehoboth, Dighton, Berkley, Swansea, Somerset, and Freetown are located in the central portion of the county. The City of New Bedford and the Towns of Westport, Dartmouth, Acushnet, and Fairhaven are located in the southern portion of the county.

Bristol County is bordered on the north by Norfolk County, Massachusetts, and on the east by Plymouth County, Massachusetts. It is bordered on the west by Providence, Bristol, and Newport Counties in Rhode Island. Bristol County is bordered on the south and southeast by the Rhode Island Sound and Buzzards Bay and on portions of the west by Narragansett Bay and Mt. Hope Bay.

According to the U.S. Census Bureau, the population of Bristol County was 548,285 in 2010, and the total area was 691 square miles, including 138 square miles of water (Reference 2).

2.3 Principal Flood Problems

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

Trees, brush, and other vegetation growing along stream banks impede flood flows during high waters, thus creating backwater and increasing flood heights. Furthermore, trees, ice, and other debris may be washed away and carried downstream to collect on bridges and other obstructions. As the flood flow increases, significant amounts of this debris often break loose, and a wall of water and debris surges downstream until another

obstruction is encountered. Debris may collect against a bridge or culvert until the load exceeds the structural capacity, causing its destruction. It is difficult to predict the degree to which, or the location where, debris may accumulate. Therefore, in the development of the flood profiles it has been necessary to assume no accumulation of debris or obstruction of flow.

The flood problems for the communities within Bristol County have been compiled and are described below:

There has been no history of major flooding in the Town of Acushnet. There has been little flood damage in the town due to the lack of development in the floodplains.

Severe flooding in the City of Attleboro and the Town of North Attleborough generally occurs as a result of hurricanes or melting snows and spring rains, with more localized flooding caused by summer thunderstorms. The major floods in these communities have been the result of multiple-day rainfalls. The more recent floods occurred in August 1955 and March 1968. The flood of August 17-19, 1955 (Hurricane Diane) was a tropical storm accompanied by high winds. As much as 19 inches of rain fell in some parts of Massachusetts as the hurricane path crossed the state only 15 miles north of the Town of North Attleborough. The City of Attleboro received nearly eight inches of rain in less than 48 hours. Low-lying areas were flooded and downtown in the vicinity of County Street and Riverbank Road resembled a small lake. The Town of North Attleborough received nearly ten inches of rain during the same period. The flood of March 17-18, 1968 was caused by a low intensity, long duration rainfall. About three inches of rain the previous week plus water from melting snow ponded in woods and other protected areas on top of frozen ground, resulting in a very saturated watershed condition, and setting the stage for the worst flood in the City of Attleboro's history. Although only six inches of rain fell during the March 17-18 period, the resulting flood crests reported were higher than those of the 1955 storm. A flood of this magnitude or greater is expected to have a 2-percent chance of occurring in any given year. The expected 1-percent-annual-chance flood elevation along the Ten Mile River in the vicinity of the City of Attleboro center and in the vicinity of Route 1 in the Town of North Attleborough would be about one foot higher than was experienced during the 1968 flood and a 0.2-percent-annual-chance flood would be about four feet higher than the 1968 flood. The 2010 flood was caused by 17-23 inches of rain from three primary storms over about a five-week period. It resulted in several bridges being overtopped. The recurrence interval for the 2010 flood was generally from a 50- to 100-year event and greater than 100-year in some locations. The USGS stream gage on the Ten Mile River at East Providence, Rhode Island (01109403), downstream of the City of Attleboro, has operated since 1987 and experienced about a 100-year event from the February to March 2010 storm events.

Although the Town of Berkley is located on the Taunton River, approximately 12 miles from the ocean, the greatest flood to occur in recorded history resulted from an exceptionally high tide accompanying a hurricane. This occurred in September 1938. The water level of the Taunton River rose on this occasion to a height of 13.8 feet. In August of 1954, another hurricane produced an elevation of 13.4 feet, the second highest water level ever recorded.

In March of 1968, the record flood for the reaches of the Taunton River upstream of the Towns of Berkley and Somerset occurred with a 1-percent-annual-chance frequency. Although the total rainfall that fell during this storm was substantial, the system of

swamps and wetlands throughout the watershed kept damages to a minimum in the lower portions of the river, especially in the vicinities of the Towns of Berkley and Somerset. A water-surface elevation of 7.7 feet was recorded at a point opposite the Three Mile River, approximately 1.5 miles upstream of the Berkley Bridge.

Riverine flooding on streams in the Town of Dartmouth results either from high intensity rainfall over a small area or moderate to heavy rainfall for a longer period of time over a large area. Flooding in the Paskamanset River watershed is mitigated by the extensive swamp and wetland areas, including Acushnet Cedar Swamp and Apponagansett Swamp, which form its headwaters; however, flooding has occurred in certain developed areas due to flat river gradients or backwater caused by restrictive river crossings. The USGS did not maintain gaging stations on the streams in the Town of Dartmouth until 1995; therefore, records of past riverine floods are limited. Since 1995, the USGS has operated a stream gage on the Paskamanset River near South Dartmouth. Based on the accounts of local residents and records at nearby gaging stations, the flood of March 1968 was considered a significant event. The stream gage's peak flows show that the October 2005, March 2001, and March 2010 events were likely the largest floods since the 1968 flood.

The Town of Dighton has experienced major flooding from hurricanes along the Taunton River as well as less severe flooding along the Three Mile River, the Segreganset River, and Muddy Cove Brook. Flooding along the Taunton River in the Town of Dighton occurs along Pleasant Street from its intersection with Main Street south to the Dighton-Somerset town line. Included in this stretch is the Old Dighton Rock Park area of town located just south of Hart Street. This area experienced some very heavy storm damage in both 1938 and 1954-1955 and, in general, floods out during every major storm. Water rose to the window sills at 2185 and 2177 Pleasant Street and reached a depth of at least 24 inches inside the house for both storms. Boats were washed up onto the other side of Pleasant Street; in general, extensive property damage occurred here. Flooding also occurred in the lower Main Street-Water Street area further up the Taunton River. Here, property damage was minimal but much land was inundated, as was the case along most of Pleasant Street (Reference 3). High moon tides will flood this area. This area, then, is apt to be hit very hard by future storms. The Segreganset River overtopped the following roadways during the 1938, 1954, and 1955 storms: Wheeler Street, Maple Street, Center Street, and Brook Street (Reference 3). It is felt that these roads will indeed be overtopped during future storms as has been experienced in the past. Muddy Cove Brook overtopped Main Street in 1938, 1954, and 1955 and will no doubt continue to do so in future storms. The Three Mile River in North Dighton caused some rather harsh flooding also, as Spring Street was washed out where the Three Mile River passes beneath it. The road here has been rebuilt with large culverts. The road may be overtopped by future storms due to the bend in the river at this location. The 1938, 1954, and 1955 storms were severe, but are estimated to have a frequency occurrence of less than 1-percent. The 1968 storm produced the highest level of water ever recorded for the upper reaches of the Taunton River, but the storm had a relatively minor effect on lower reaches of the river in the vicinity of the Town of Dighton. Statistical analysis has indicated that this storm was equivalent to the 100-year flood for the upper reaches of the Taunton River (Reference 4). The 2010 flood was caused by 17-23 inches of rain from three primary storms over about a five-week period. It resulted in several bridges being overtopped. The recurrence interval for the 2010 flood was generally from a 50- to 100-year event and greater than 100-year in some locations. The USGS stream gage on the Threemile River at North

Dighton (01109060), operated since 1967, experienced only about a 10- to 25-year event from the February to March 2010 storm events.

In 1968, major flood problems in the Town of Easton occurred on Queset Brook at State Route 138 and in the area of Morse Pond. Poquanticut Brook overtopped New Pond and flooded State Route 106 during this flood. Based on gaging stations near the town, the recurrence interval of the 1968 flood was determined to be 60 years.

The Town of Freetown has experienced very little flood damage due to past hurricanes and storms. The greatest flood on record resulted from an exceptionally high tide accompanying a hurricane that occurred in September 1938. The estimated frequency of occurrence was approximately 75 years. In August 1954, another hurricane produced the second highest flood elevation and was estimated to have a frequency interval greater than 50 years. In the event of a 1-percent-annual chance storm, Mill Street and State Route 79 along the Assonet River would be flooded with 2.5 feet of water and Narrows Road along Rattlesnake Brook would be overtopped with over five feet of water.

Information from town officials and previous engineering studies indicate that flooding in the Town of Mansfield is caused by hurricanes or other major storms that occasionally visit the area. When substantial flooding occurs it is generally confined to the lower portion of the Rumford River, the upper reaches of the Canoe River and the Whiteville Pond area, and the lowlands along Hodges Brook between the Penn Central Railroad and West Street. The Town of Mansfield has suffered considerable damages from the floods that occurred in 1938, 1954, and 1968. The flood of March 1968 reflects damages to contemporary development and is discussed in detail in this report. The spring flood of 1968 resulted from the runoff of a record rainfall which occurred on March 18-19. The antecedent conditions in 1968 were the primary cause of flooding in the Town of Mansfield. There was heavy snow cover and a storm the previous week which saturated the ground. The streamflow on the Wading River in Mansfield was a record 541 cubic feet per second (cfs) on March 19, 1968. The Rumford River suffered the most serious flooding. The dams at both Fulton and Kingman Ponds were breached, and at Willow Street water flowed over the top of the road. School Street and Oak Street were overtopped and an estimated 20 acres of land in the lowlands were inundated by Hodges Brook. The Canoe River was adequately controlled by the Mill Street dams, with the only notable flood problem occurring when water draining from Whiteville Pond flowed over Franklin Street. Flooding on the Wading River was controlled by lake storage in both the Town of Mansfield and neighboring Foxborough. Throughout the town some 370 homes reported water damage from this flood and several roads and culverts also suffered damage. Also in the Town of Mansfield, information from past reports suggests that minor or localized flooding, which occurs along Back Bay Brook and some portions of the Rumford River, is the result of inadequate or undersized drainage systems. The 2010 flood was caused by 17-23 inches of rain from three primary storms over about a five-week period. It resulted in several bridges being overtopped. The recurrence interval for the 2010 flood was generally from a 50- to 100-year event and greater than 100-year in some locations.

The principal flood problems in the Town of Norton are caused by the overflow of Norton Reservoir, Chartley Pond, and the Rumford, Canoe, and Wading Rivers. Damage is caused by inundation because stream velocities are generally low. Natural storage in swamps and ponds generally diminishes peak flows in the Town of Norton. Much of this storage capacity is located in adjoining towns. A number of major floods have occurred

in the Taunton River basin during the 20th century. The worst floods occurred in August 1955 and March 1968. The estimated return period for the 1968 flood is 100 years. These floodwaters caused damage to the industries in the vicinity of West Main Street and South Worcester Street, as well as inundating bridges at Plain Street on the Canoe River and Walker Street and West Main Street on the Wading River. The USGS has collected data at gaging stations on the Wading River near Norton (gage 01109000) from 1925 to present and at West Mansfield (gage 01108500) since from 1953 to 1986. The USGS has also collected gage information from gage 01109200 on the West Branch Palmer River near Rehoboth from 1964 to 1974. The 2010 flood was caused by 17-23 inches of rain from three primary storms over about a five-week period. It resulted in several bridges being overtopped. The recurrence interval for the 2010 flood was generally from a 50- to 100-year event and greater than 100-year in some locations.

Most flood problems in the Town of Raynham are caused by the Forge and Taunton Rivers. In March 1968, Gardner Street was flooded by the Forge River, and the downstream side of the embankment which dams Kings Pond was seriously eroded. The Church Street bridge over the Taunton River was almost submerged by the 1968 flood, which had a recurrence interval of approximately 60 years. Tidal flooding from Assonet Bay, as well as riverine flooding, occurs along the Taunton River. However, for the estimated 1-percent-annual- chance flood, riverine flooding would exceed tidal flooding, except along the reach at the mouth of the Forge River. Tidal flooding is caused by hurricane tides, and riverine flooding associated with a hurricane tends not to occur until about two days after the hurricane.

In the Town of Rehoboth, flood problems resulting from hurricanes or northeasters have inundated basements, causing financial difficulties for the town and its residents. Local newspaper accounts mention little of the flooding situations within the town, and, because past flooding has existed mostly in undeveloped areas, there are only scant records of the extent and depths of flooding encountered. Some homes have been partially flooded, but records are not explicit as to their number or the cause of flooding. The geological structure throughout the town is such that ground water infiltration into basements is a seasonal occurrence, and residents have provided for this situation. Because of the predominantly rural nature of Rehoboth's development, flooding of private property has not been extensive. In the past, however, bridges and roads have been washed away by flood waters, such as during the storm of March 19, 1968. Bridges on Providence Street were lost, and those on Danforth, Carpenter, Pleasant, and Water Streets were damaged. The bridge on Water Street was later replaced.

The low-density development pattern of the Town of Rehoboth does much to decrease the possibility of flood-related damage. As the land is not extensively developed and the wetlands are essentially in a natural state, a great deal of natural storage is still available to reduce flood flows. Certain streams, however, will not be able to carry the volume of water anticipated to result from the 1-percent-annual-chance event because of insufficient channel cross sectional areas or structural limitations (in terms of small bridge openings). The 1- and 0.2- percent-annual-chance floods will overtax many of the town's bridges and induce sheet flooding in many areas. With the exception of washed-out bridges and many flooded basements, however, the actual flood will not cause much damage, as long as development does not proliferate and the townspeople are given sufficient warning. One exception to this is the lower portion of the East Branch Palmer River, in the vicinity upstream of County Street, which will experience extensive flooding. The greatest danger will be the possibility of loss of access by

emergency vehicles and personnel to a particular location. Alternate routes could be prepared well in advance of an actual flood emergency, according to the flooding patterns illustrated in this report. It should be noted that the 1968 storm is the flood of record. This storm corresponds to the 2-percent-annual-chance event, as shown on the profile at the stream gage located on the West Branch Palmer River. Other notable flooding events occurred in 1955, 1938, 1936, 1935, and 1933.

In the Town of Seekonk, the more serious flooding is usually a result of large volumes of runoff which exceed the natural storage of the extensive wetlands. Tidal flooding along the Runnins River south of I-195 is a threat. Although infrequent, the larger floods do have the potential to be devastating. In the past, over-road flooding has occurred on the small streams (drainage areas of one to three square miles) as a result of above-average rainfall and obstruction of the numerous culvert road crossings. Often the obstruction is caused by debris, but just as often, the inlets are frozen and clogged with ice. Historically, the Town of Seekonk has not experienced large devastating floods. This is principally due to three factors: (1) the path and pattern of historical storms; (2) the relatively low development along watercourses; and (3) the large natural storage areas in the headwaters of and along the principal streams, including the Ten Mile River. The potential for the most extensive flooding is from stillwater tide levels below Highland Avenue.

The principal flood problems in the City of Taunton are caused by the overflow of the Taunton River, Three Mile River, Mill River, and Cobb Brook. Stream velocities are generally low, and flood damage is caused by inundation. According to residents and local officials, the floods of 1886, 1938, 1954, 1955, 1968, 2005, and 2010 caused the most damage. The 1938, 1954, and 1955 floods were caused by hurricanes; the 1968 flood was caused by rainfall-induced spring thaw, and the 2010 flood was caused by 17-23 inches of rain from three primary storms over about a five-week period. Tidal flooding as well as riverine flooding occurs along the lower portion of the Taunton River. Hurricane tides in 1938 and 1954 and riverine flooding in 1968 inundated the area between First and Third Streets. Floodwaters rose to 12.6 feet in the municipal lighting plant on West Water Street during the flood of 1938. Areas along U.S. Route 44 were inundated to a depth of 1.5 feet during the flood of 1968. The recurrence intervals of the 1938 tidal flood and the 1968 riverine flood on the Taunton River have been determined to be approximately 70 and 60 years, respectively. The recurrence interval for the 2010 flood was generally from a 50- to 100-year event and greater than 100-year in some locations.

In 1886, the Three Mile River inundated U.S. Route 44 and destroyed many bridges. In 1968, several businesses and residences on Warner Boulevard and Spring Street were damaged by the Three Mile River floodwaters. The Mill River also has a long history of flooding. In 1889, the Mill River floodwaters rose almost to the City Common and destroyed many bridges. In 1968, its headwater, Lake Sabbatia, rose to 65.8 feet, overflowing its banks. This elevation was determined from historical watermarks. Approximately 1 foot of water lay over Britannia Street downstream of Lake Sabbatia. Large areas were evacuated when it was feared that Whittenton Dam or Moreys Bridge Dam might fail. Serious flooding occurred along the downtown area, especially in the vicinity of Spring and Winthrop Streets. Cobb Brook overflowed Somerset Avenue in 1938, 1954, and 1968. In 1968, an area between Oak Street and East Whitehill Street was inundated. This area is subject to periodic flooding. USGS gaging stations on the Taunton River near Bridgewater (station No. 01108000), the Three Mile River at North Dighton

(station No. 01109060), and on the Segreganset River at Dighton (station No. 01109070) were used for the hydrologic analyses in this study.

Most of the City of Fall River lies above coastal flood levels. Floodplain development is restricted to a narrow strip of land along the Taunton River and Mount Hope Bay. This area is zoned for industrial and unrestricted development and is the site of many industrial complexes, deep draft wharves and piers, tourist and recreational facilities, small residential developments, and several small craft facilities (Reference 5).

Floodplain development in the City of New Bedford along Buzzards Bay is primarily residential and recreational. Floodplain development along the Acushnet River is primarily industrial.

Floodplain development in the Town of Somerset is primarily residential, with the exception of power plants and several small industrial firms and commercial developments. Loading docks are located at both the New England Power Company and Montaup Electric Company power stations. A shipbuilding yard and several small craft facilities are located at both Somerset and South Somerset.

In the Town of Swansea, floodplain development is primarily residential and recreational, with the exception of several small commercial developments that support fishing and boating.

The floodplains within the Town of Westport contain residential, recreational, and commercial development. Seasonal dwellings, consisting primarily of mobile homes and trailers, are located along East Beach. Several homes and commercial establishments are located at the junction of East Beach with West Beach and at Westport Point. At State Beach, several buildings, camping facilities, and parking areas support recreational interests.

The low-lying coastal areas of the Cities of Fall River and New Bedford and the Towns of Dartmouth, Fairhaven, Somerset, Swansea, and Westport are subject to the periodic flooding and wave attack that accompany coastal storms and hurricanes. Most of these storms cause damage only to boats, low coastal roads, beaches, and seawalls. Occasionally, a major northeaster or hurricane accompanied by strong onshore winds and high tides results in surge and wave activity that causes extensive property damage and erosion.

In the City of Fall River and the Towns of Somerset and Swansea, the worst storm damage results when southern winds cause funneling through Narragansett Bay and Mount Hope Bay. Many times a storm of relatively minor proportions will linger over these areas for a substantial period of time and will cause excessive buildup of the tidal levels. In the City of New Bedford and the Towns of Dartmouth, Fairhaven, and Westport, the worst storm damage results when southerly winds cause funneling through Buzzards Bay. Some of the more significant coastal storms in these communities include the hurricanes of September 1938 and August 1954. The resultant flood levels were estimated by the USACE at 13.7 and 13.4 feet, respectively, in the City of Fall River and the Towns of Somerset and Swansea (Reference 6); 12.5 feet and 11.9 feet, respectively, in the Town of Dartmouth; 12.8 feet and 12.1 feet, respectively, in the City of New Bedford and the Town of Fairhaven; and 12.2 feet and 11.2 feet, respectively, in

the Town of Westport. These storms claimed lives and damaged residential, recreational, and small commercial developments, including harbors and marinas.

Riverine flooding along the City of New Bedford and the Town of Fairhaven waterfront occurs when the Acushnet River is ponded behind the New Bedford-Fairhaven hurricane barrier during periods of concurrent high runoff and surge activity. For the most part, this flooding is limited to parking lots and rail yards. Isolated flooding in the City of New Bedford can also occur southwest of the municipal airport along the upper tributaries of the Paskamanset River.

Minor flooding occurs in various locations throughout the Town of Somerset, primarily as a result of inadequate or blocked culverts. Storms of great intensity and short duration are usually the cause of this type of flooding.

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 Bristol 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 made landfall over Block Island, RI and crossed into Massachusetts 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 Bristol County (References 7 and 8).

Bristol County also saw flooding from severe storms in October 1996, June 1998, March 2001, April 2004, October 2005, May 2006, April 2007, and March and April 2010. 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 8).

In late February through March 2010, three separate rainfall events resulted in about 17 to 23 inches over much of southern New England, causing major flooding across eastern Massachusetts and Rhode Island. These rain storms caused several small streams in Bristol County to rise above flood stage, including the USGS stream gages on the Wading River at Norton, the Mill River at Taunton, and the Segreganset River at Dighton. Several communities had areas that were closed for several days due to small stream, urban, and poor drainage flooding (Reference 9).

From December 2010 through February 2011, southern New England, including Bristol County, saw a series of winter storms that led to record snowfall for the season. The City of Attleboro snowfall total was over 60 inches. Heavy snow, combined with rain led to numerous flooding problems across the county, roof collapses, and downed trees and utility lines (References 10 and 11).

In August 2011, Hurricane Irene, weakened to a tropical storm, flooded numerous roads throughout Bristol County. Maximum storm tides of 5 to 7 feet above Mean Lower Low Water (MLLW) were recorded at the Cities of Fall River and Taunton. Fallen trees

and power outages were widespread leaving residents and businesses without power for days (Reference 12 and 13).

2.4 Flood Protection Measures

Flood protection measures for Bristol County have been compiled from pre-countywide FISs and are summarized below. Flood protection measures that have been implemented since the date of each community's pre-countywide FIS may not be reflected here.

The most effective flood protection measures for many of the communities in Bristol County, including the Towns of Acushnet, Easton, Raynham, and Seekonk, are provided by the natural system of swamps which tend to attenuate the flood flows by creating storage areas, and by the generally flat terrain that tends to reduce flood velocities.

The major protective structures along the Town of Dartmouth coast are the Padanaram breakwater at the mouth of Apponagansett Bay and the Hunts Rock breakwater between Round Hill Point and Salters Point. Stone and concrete walls have been built along portions of the Paskamanset River and Buttonwood Brook.

The New Bedford-Fairhaven Hurricane Barrier is located in the City of New Bedford, the Town of Fairhaven, and the Town of Acushnet. The project was completed in May 1966 and is operated and maintained by the USACE, the City of New Bedford, and the Town of Fairhaven. The New Bedford-Fairhaven Hurricane Barrier consists of three separate barrier structures: the main barrier, the Clarks Cove Dike, and the Fairhaven Dike. The main barrier spans across the Acushnet River at the mouth of the New Bedford harbor and extends from near Cove Road to the street gate between Rodney and Frederick Streets. The street gate serves as a barrier in the event of potential flooding. The Clarks Cove Dike extends from the street gate on Cove Road (west) to the street gate on Rodney French Boulevard West (near Woodlawn Street). These two structures provide protection from coastal flooding to all but the properties in the southernmost areas of the City of New Bedford, such as along Padnaram Avenue and Rodney French Boulevard and south of Rodney and Woodlawn Streets to the Fort Rodman Military Reservation at Clark Point. Fairhaven Dike is located across the tidal marshes at the head of Priests Cove. This structure provides protection from coastal flooding to all but Sconticut Neck, West Island, and the northern shore of Nasketucket Bay.

It has been assumed that the New Bedford-Fairhaven Hurricane Barrier would fail in a 0.2-percent-annual-chance flood event (Reference 14). On July 14, 2011, the City of New Bedford and the Town of Fairhaven received notification of the New Bedford-Fairhaven Hurricane Barrier accreditation, which states that the barrier system complies with the minimum requirements outlined in Title 44 of the Code of Federal Regulations, Section 65.10 (44 CFR 65.10). The accredited barrier system is shown on the effective FIRM as providing protection from the 1-percent-annual-chance flood.

In the Cities of Fall River and New Bedford and the Towns of Dartmouth, Fairhaven, Somerset, Swansea, and Westport, protective structures have generally been built and are maintained by the municipality or private property owners to satisfy their individual requirements. Limited financial resources sometimes result in less than adequate protection.

A hurricane survey report for the Narragansett Bay area, including the City of Fall River and the Towns of Somerset and Swansea, was published by the USACE in 1966. It recommended a system of three massive ungated rock barriers across the three entrances to protect the bay areas from tidal flood damage. These barriers have not been constructed.

Normal runoff from a large portion of the drainage areas of the rivers flowing through the Town of Mansfield is controlled by either lake and pond storage or dams. These dams were not built as flood control projects and no reliance should be placed on them to reduce major flooding. Three-fourths of the drainage area of the Wading River above Otis Street is controlled by lake storage in the neighboring town of Foxborough. The remaining drainage area is controlled in the Town of Mansfield by Robinson's Pond, located above Williams Street, Blakes Pond, upstream of Balcom Street, and Sweet's Pond, located above Otis Street. On the Rumford River, nearly one-half of its total drainage area above the Norton Reservoir is controlled by pond storage in Foxborough and Sharon. A small degree of control is exercised at Cabot Pond, located above Willow Street, and Fulton Pond, where a recently completed structure has replaced the dam that was breached by the spring 1968 floods. Replacement of the structure at Kingman's Pond, which also failed during the 1968 floods, is being given serious consideration. The Canoe River is controlled by a dam on Mill Street. This has afforded reasonable flood control on the lower reaches of the Canoe River in the past, especially during the 1968 floods. There is some control of Hodges Brook, other than that naturally exerted by swamps and lowlands, which is provided by the retention ponds, modified channel, and drainage ditches that are in the Industrial Park near Interstate Highway 95. These retention ponds and drainage ditches have been calculated to be capable of carrying the flows of the 100-year flood. The Police and Fire Departments are responsible for local flood warnings.

In 1974, the Town of Norton established Wetland Protection Districts to "...protect persons and property against the hazards of flood inundation by providing for the unimpeded natural flow of watercourses and for adequate and safe flood storage capacity" (Reference 15).

In the Town of Raynham, most of the storage capacity for the Taunton River is located in upstream towns. The Church Street bridge, which created backwater problems during past floods, has been demolished. It will be replaced by a bridge designed to accommodate a 2- percent-annual-chance flood. The Raynham Highway Department keeps close watch on the dams in the town, releasing water when there is a flood threat; however, these dams were not designed as flood-control structures. Another dam was built on Wilbur Pond, in west-central Raynham, but this dam is not a flood-control structure, either.

The City of Taunton has established floodplain districts and has adopted zoning regulations corresponding to those districts. As of the June 18, 1987 Taunton FIS, there were no flood protection structures in the city. Natural storage in swamps and ponds diminishes peak flows in the City of Taunton. Much of this storage capacity is located in adjoining towns; however, the city sometimes makes use of available local storage. According to the City Engineer, much of the inflow into Lake Sabbatia during the flood of 1968 was diverted to and stored in Watson Pond. Inflow to Lake Sabbatia was estimated to be approximately 3,300 cubic feet per second (cfs), but peak flow at the outlet, the Mill River, was estimated by the USACE to be only 1,700 cfs. The various dams on the Three Mile River, the Three Mile River -West Channel, the Mill River, and

Cobb Brook are for industrial use and offer no protection during large floods. The dams on the Segreganset River are used to create in-stream ponds for the golf course for irrigation and water hazards and offer no protection during large floods.

In the City of Attleboro, Chartley Brook and Bungay River are central watercourses which drain enormous wetland areas. During periods of large floods, the waters are held back and spread out over the wetlands thus preventing this water from piling up downstream and causing damages. The Bungay River wetland is especially important because the large amount of floodwaters that are held back are not allowed to coincide with the earlier arriving peak flood flows of the Ten Mile River.

Upstream from the City of Attleboro along the Ten Mile River, there are several dams in the Town of North Attleborough at Falls Pond and Whiting Pond. The Town of North Attleborough releases waters in these dams in anticipation of storm events, in order to avoid localized flooding. Lack of communication and coordination has caused problems in the past. An upstream release will cause flooding problems in the City of Attleboro if the city does not react by correspondingly adjusting levels within holding areas of the Ten Mile River at Mechanics Pond and monitoring culverts under bridges and roadways. If the release is not managed, the City of Attleboro can experience flooding of the Willet School Field and the Riverbank Road/County Street area. In October of 2002, an agreement was developed between the two communities that outlines the coordination steps necessary to avoid flooding problems.

In addition to this agreement, the Attleboro Department of Water and Wastewater maintains a water level monitoring station at the treatment facility. This monitoring station has data on rainfall going back over 40 years and the records indicate by day the precipitation in inches. Street flooding is possible for storm events that are over 2 inches of precipitation.

The Town of Rehoboth has no formal flood protection measures. While there are a number of dams throughout the town, they were not designed with sufficient capacity for flood control.

The dams located below the Warren Reservoir and Milford Pond in the Town of Swansea were also not treated as flood control structures in the July 17, 1986 Swansea FIS.

There are no flood protection structures to prevent flooding affecting the Towns of Berkley, Dighton, Easton, Freetown, Norton, and Seekonk.

The Town of Easton does not have ordinances related to flooding.

The New England River Basins Commission recommends that flood-prone areas be protected by non-structural floodplain management measures and that wetlands be preserved as natural flood retention areas since they help minimize tidal flood damage (Reference 16).

Flood warning and forecasting services are performed by the National Weather Service on a regional scale. Adoption of federal, state, and local development regulations concerning floodplain management will help alleviate storm-related losses.

3.0 ENGINEERING METHODS

For the flooding sources studied by detailed methods in the county, 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 county at the time of completion of this study. Maps and flood elevations will be amended periodically to reflect future changes.

3.1 Hydrologic Analyses

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

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

Precountywide Analyses

S. William Wandle's regional discharge-frequency equations for ungaged streams were used to determine peak discharges for the following flooding sources: a portion of the Acushnet River watershed and Deep Brook in the Town of Acushnet, Mulberry, Poquanticut, Black and Queset Brooks in the Town of Easton, the Forge River, the Tributary to Forge River, Dam Lot Brook and the Tributary to Dam Lot Brook in the Town of Raynham, the Mill River in the City of Taunton, and for certain waterways in the Towns of Mansfield and Norton (Reference 17). In the formula below, the discharge is determined as a function of the drainage area and the main channel slope:

$$Q_1 = Q_2 (A_1/A_2)^n$$

where Q_1 and Q_2 are the flows at the site and gage, respectively, A_1 and A_2 are the drainage areas at the site and gage, respectively, and n is the regional drainage area ratio exponent (References 18 and 19). In the Town of Mansfield, discharges were estimated above and below the gage on the Wading River and above Norton Reservoir on the Rumford River using an n -value of 0.75. In the Town of Norton, discharges were estimated for the Rumford River and the Canoe River with an n -value of 0.72, and for Goose Branch Brook, using gage No. 01109200 on West Branch Palmer River near Rehoboth (Reference 20), with an n -value of 0.66. In the Town of Raynham, an n -value of 0.72 was used to estimate discharges at downstream sites on the Taunton River. In the City of Taunton, sites along the Taunton River and Three Mile River were estimated using an n -value of 0.72, and the discharges of the Segreganset River were estimated with an n -value of 0.66 (Reference 19).

The discharges on the Acushnet River in the Town of Acushnet were obtained from the routing of the New Bedford Reservoir in conjunction with use of regional equations. Discharges in the non-tidal portion of the Acushnet River were determined as follows: An inflow flood hydrograph was determined at the outlet of the New Bedford Reservoir (Reference 21). A reservoir routing of the flood hydrograph was performed to determine the outflow discharges of the reservoir. The final discharges on the Acushnet River are the sum of the discharges from the results of the routing of the New Bedford Reservoir and the results from the use of the regional frequency-discharge equations. The discharges from the New Bedford Reservoir routing are taken from the rising limb of the outflow hydrograph to account for the fact that the peak discharges from the reservoir occur much later in time than peak discharges from the remaining watershed. Although the drainage area for the Acushnet River is larger than that of Deep Brook, the discharges are not as high due to New Bedford Reservoir's storage capabilities and flow controls of the Acushnet River. Flood elevations for the tidal portions of the Acushnet River were taken from the City of New Bedford FIS (described below). The New Bedford-Fairhaven Barrier was assumed to have failed for the 0.2-percent-annual-chance flood (Reference 14).

The discharge-frequency relationships for all streams in the City of Attleboro and the Towns of North Attleborough and Seekonk watersheds were determined from a methodology developed by the SCS which analyzes anticipated rainfall and resulting runoff (Reference 22). The watershed was divided into areas of relatively uniform hydrologic characteristics. An analysis of the slope, soils, vegetative cover, land use, and stream channels for these areas was made to compute composite runoff curve numbers, times of concentration, and travel times. Storage capacity and stage-discharge curves were computed for all significant reservoirs and natural valley storage areas. Discharges were not determined for streams studied by approximate methods. The storm of March 1968 was flood-routed through the watershed by use of the SCS Computer Program for Project Formulation-Hydrology, TR20 (Reference 23) to verify the model. The results of this historical storm flood routing showed a good correlation between actual high-water marks and the computed flood elevations. There were no stream gage records available in the watershed for comparison. 10-, 2-, 1-, and 0.2-percent-annual-chance synthetic storms were then flood routed through the upstream areas of the watershed using the Computer Program for Project Formulation-Hydrology (Reference 23). This program computes surface runoff resulting from synthetic or natural rainstorms. It takes into account conditions having a bearing on runoff and routes the flow through stream channels and reservoirs. It also combines the routed hydrograph with those from other tributaries and computes peak discharge, time of occurrence, and the water-surface elevation at selected cross sections and reservoirs. Rainfall data for the various frequency storms were obtained from U.S. Weather Bureau publications (References 24 and 25). A 48-hour rainfall distribution was assumed for all frequency storms.

A method was developed by the Water Resources Division of USGS (Reference 26) to determine the peak discharge 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 that flood peaks for any stream, whether it be gaged or ungaged, may be estimated from knowledge of the drainage characteristics of the area, main channel slope, and the mean precipitation on the basin. This method was utilized for the approximate study areas in the Town of Berkley.

Peak discharges for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods in the Town of Dartmouth were determined by the Rational Method and by comparison of flow data from other streams with similar hydrologic characteristics (Reference 27). Frequency discharge curves for the Paskamanset River and Buttonwood Brook were prepared from these data. Tide stage-frequency relationships were determined for the waters at the Town of Dartmouth using flood profiles developed by statistically analyzing maximum high-water elevations in the study area.

A multiple regression analysis developed by Johnson and Tasker was employed to find runoff discharges in the Town of Freetown and for the Segreganset River and Sunken Brook in the Town of Dighton (Reference 28). Standard USGS 7.5-minute quadrangle maps, with a scale of 1:24,000 and a contour interval of 10 feet (Reference 29), were used to determine watershed areas and local topography. An annual precipitation value, representative for the region, of 3.67 feet per year was obtained from the U.S. Weather Bureau and used throughout southeastern Massachusetts (Reference 30). By determining values for slope and area and using them in conjunction with the precipitation value in the Johnson- Tasker formulae, values for runoff from 10-, 2-, 1-, and 0.2-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 compared from other available studies, including the City of Taunton FIS (described below), or by isolating the associated watershed and applying the Johnson-Tasker regression analysis where no other study has been conducted. In the Town of Freetown, none of the rivers studied have gaging stations; however, certain rivers in the region having similar topography are gaged. Gage records for these rivers were compared by log-Pearson Type III analysis and discharge values were found to be compatible (Reference 28). These rivers are the Three Mile River at North Dighton (10 years of record) and the Segreganset River at Dighton (10 years of record). After comparison of predicted discharges with experienced floods, it was found that the Johnson-Tasker methods break down in regions of flat slope or high storage. To correct these discrepancies, areas of swamp, bog, open water, and urban development were computed and assigned weighting values to account for storage and rapid urban runoff. The adjusted discharge figures more closely reflect the true nature of the basins involved.

The following stream gages and lengths of record were used in the Dighton study: the Taunton River at State Farm in Bridgewater, Massachusetts, with a record of 46 years; the Wading River at West Mansfield with a record of 23 years; the Wading River at Norton with a record of 51 years; the Three Mile River at North Dighton with a record of 10 years; and the Segreganset River at Dighton with a record of 10 years. The Three Mile River and the West Channel Three Mile River were studied in detail by the USGS for the June 18, 1987 Taunton FIS (described below).

Hydrologic analyses for the October, 1976 Mansfield FIS were based upon 22 years of record of the USGS Gaging Station (Number 01108500) on the Wading River at West Mansfield, Massachusetts, 200 feet downstream of the Balcom Street bridge, and also on regional discharge-drainage area relationships developed by the USACE (Reference 31). A rating curve for the gaging station on the Wading River at West Mansfield was used to calculate the 10-, 50-, 100-, and 500-year flood discharges on the Wading River.

These values were checked against the regional discharge-drainage area curves, which yielded comparable results. For Hodges Brook and the Canoe River, due to their close proximity and similar drainage area, discharges were calculated based on these gage values. Discharges for the Rumford River were developed from an Average Regional Relationship of discharge-drainage area as contained in a study of southeastern New England by the USACE. These 10-, 2-, 1- and 0.2-percent-annual-chance discharges were compared and found to favorably agree with values determined according to the Massachusetts Flood Magnitude Formulas developed by the USGS (Reference 28).

In the Town of Norton, data collected at gage No. 01109000 near Norton and gage No. 01108500 at West Mansfield on the Wading River were used to determine the 10-, 2-, 1- and 0.2- percent-annual-chance peak discharges on the Wading River. Peak discharges at these gages were determined from a log-Pearson Type III distribution using a weighted skew coefficient (Reference 32). Discharges at intervening sites along the Wading River were estimated by interpolating on the basis of drainage area. For Winnecunnet Pond, the water-surface elevations for floods of the selected recurrence intervals were calculated based on a reservoir analysis at Lake Sabbatia in the City of Taunton. The flood elevations for Winnecunnet Pond were calculated using the Lake Sabbatia elevations and the characteristics of the hydraulic connection between Lake Sabbatia and the Wading River.

Peak discharges for the Taunton River in Raynham were developed from a log-Pearson Type III analysis of annual peak discharge records following methods outlined in the Water Resources Bulletin No. 17 (Reference 32). The discharge data were obtained from a USGS gaging station on the Taunton River near Bridgewater that covered a 47-year period, from October 1929 to April 1976. The peak elevations for floods of selected recurrence intervals on the tidal reach of the Taunton River were based on a tidal frequency analysis in a hurricane study of Narragansett Bay and flood-profiles of the 1938 and 1954 tides on the Taunton River, furnished by the USACE (Reference 33). In conjunction with the USACE, the original tide profiles were modified and extended on the basis of additional historical tide data furnished by the City Engineer of Taunton.

Hydrologic analyses for Rocky Run in the Town of Rehoboth were based on flow records of the USGS gaging stations on the West Branch Palmer River in Rehoboth, Massachusetts (12 years of record), on the Ipswich River at South Middleton, Massachusetts (38 years of record), and on the Wading River near Norton, Massachusetts (51 years of record). The 10-, 2-, 1- and 0.2-percent-annual-chance discharges were determined by statistical analysis, using the log-Pearson Type III distribution (Reference 34), with a regional skew of 0.5 (Reference 31). Information obtained from those gages located outside the actual study area was transposed and then altered using a discharge-area ratio (Reference 35). Certain downstream reaches of the Palmer River and Rocky Run are subject to tidal influence. Elevations for tidal events, with recurrence intervals of 10-, 2-, 1- and 0.2- percent-annual-chance, were obtained from the October 1973, Town of Warren, Rhode Island FIS (Reference 36). For subdivisions within each reach, discharge relationships were analyzed to ensure that flows were representative. The July 17, 1986 Swansea FIS also used the hydrologic analyses of Rocky Run.

The 10- and 1-percent-annual-chance riverine flows were determined for the Cole River in the Town of Swansea. These flows were used to calculate water-surface elevations for Milford Pond. The 1-percent-annual-chance flow was calculated using the Kinnison and Colby method, which employs the following equation (References 37):

$$Q = (0.0344 \times S^{1.5} + 200) M^{0.05} / L^{0.5}$$

where Q is the peak discharge in cubic feet per second, M is the drainage area in square miles, S is the mean altitude of the drainage basin in feet above the outlet, and L is the average distance in miles which water from runoff uniformly distributed over the basin must travel to the outlet. The 10-percent-annual-chance flow was estimated by transposing data from a gaged watershed with similar hydrologic characteristics in the study area. The drainage areas of the gaged and ungaged (Cole River) streams are proportioned and the 10-year flow adjusted accordingly. The analysis to determine the water-surface elevation for the Cole River above Milford Pond Dam was obtained from the February 6, 1971 Swansea FIS (Reference 38). This study assumed that the sluice gates at Milford Pond Dam were not operated to reduce upstream flooding. Water-surface elevations were calculated using the broad crested weir formula:

$$Q = CLgd^{1.5}$$

where C is the coefficient of discharge, L is the length of the weir crest, and d is the depth of flow over the weir.

Elevations for the Warren Reservoir and Heath Brook were taken from the FIS for the Town of Warren, Rhode Island (Reference 39). For the Warren Reservoir and Heath Brook, storm surge hydrographs were developed for overflow from the Palmer River and for flow from the Kickmuit River at the Child Street dike. The storm surge was routed over the Child Street dike and combined with flow from the Palmer River and Belcher Cove to determine total volumes of overflow. These volumes were compared with curves of elevation versus storage for the reservoir to determine the flood elevation-frequency relationship for the Warren Reservoir. The stillwater elevations for the 10-, 2-, 1- and 0.2-percent-annual-chance floods were determined for Mount Hope Ray, the Cole River, the Lee River, the Palmer River, Tributary to Barrington River, and the Warren Reservoir.

In the City of Taunton, peak discharges for the Taunton River, the Three Mile River, Three Mile River - West Channel, and the Segreganset River for 10-, 2-, 1- and 0.2-percent-annual-chance floods were determined from a log-Pearson Type III distribution, using a weighted skew coefficient as recommended by the Water Resources Council (Reference 32). This analysis used data that the USGS collected at gaging stations on the Taunton River near Bridgewater (station No. 01108000) since 1929, the Three Mile River at North Dighton (station No. 01109060) since 1966, and on the Segreganset River at Dighton (station No. 01109070) since 1967. Historical information was included in the Three Mile River computation to supplement the gage data. To determine discharge-frequency relationships for Cobb Brook in the City of Taunton, it was assumed to be located in a rural watershed. The rural flows were then transformed into urban flows based on basin development characteristics. The analytical relationships that were used to compute the rural peak discharges are found in Estimating Peak Discharges of Small, Rural Streams in Massachusetts (Reference 20). The equations for eastern Massachusetts were used to determine peak discharges for Cobb Brook. Rural peak discharges were computed for the 10-, 2-, 1- and 0.2-percent-annual-chance flood frequencies. The three-parameter estimating equations were used to transform the rural peak discharges to urban peak discharges (Reference 40). The Basin Development Factor (BDF) used in the calculations varied from 2 to 6 for sub-drainage areas of Cobb Brook.

The Taunton River, which acts as the town boundary between the Towns of Freetown and Somerset, was studied by the USACE for the December 5, 1984 Somerset FIS (described below). Because the Taunton River is entirely tidal in the Freetown study area, there was no need to perform hydrologic calculations.

In the May 16, 1995 Easton FIS revision, no information is available regarding the hydrologic analyses that were utilized.

Countywide Analyses

For the July 7, 2009 and the June 16, 2014 countywide revisions, no new hydrologic analyses were conducted.

For the July 16, 2015 countywide analysis, hydrologic analyses were conducted for the Taunton, Ten Mile, Sevenmile, and Wading Rivers and Whiting Pond Bypass.

Taunton River

Discharges for given Annual Exceedance Probabilities (AEPs) at the Taunton River near Bridgewater, MA USGS streamgage number 01108000 are weighted values calculated with the USGS Weighted Independent Estimator (WIE) program developed by Cohn and others (Reference 41). The program combines at-site log-Pearson Type III flow frequency estimates with regional regression estimates. At-site estimates of the 10-, 2-, 1-, and 0.2-percent AEPs at the Taunton River gage were taken from Zarriello and others (Reference 42). These at-site estimates were based on 65 years of continuous record and 20 additional years of estimated peaks. The estimates calculated using regional regression equations are based on the basin characteristics drainage area (mi²), stream density (mi/mi²), and open water and wetland storage (percent) at the streamgage.

The Taunton River streamgage is just upstream of the Titicut Street bridge. Theoretical AEP flows were transferred 8 miles upstream and 16 miles downstream from the gage using a weighted hybrid method from Guimaraes and Bohman (Reference 43) that combines regression equation estimates at the new location with the weighted estimate determined at the gaged site.

A section of splitflow occurs around two railroad bridges starting upstream of County Street/Route 140 above the Mill River in the City of Taunton, and downstream of the Honorable Gordon Owen Parkway which is downstream of Route 24. A flow optimization routine in HEC-RAS resulted in from 3,100 to 3,500 cfs going through the main channel under the railroad bridges at all of the theoretical flows modeled. Since the total flows vary from 4,230 at the 10-percent AEP to 6,890 cfs at the 0.2-percent AEP in this section of the river, the remainder of the flow that flowed outside the banks was from 700 to 3,650 cfs (16% to 53%) of the total.

The hydraulic model for the Taunton River was calibrated for the March and April 2010 event using the flood flow calculated at the streamgage for that event and high-water marks (HWMs) documented throughout the reach.

Ten Mile River

Discharges for given AEPs at the Ten Mile River at Pawtucket Avenue at East Providence, RI (USGS streamgage 01109403) are weighted values calculated with the USGS WIE program (Reference 41). The program combines at-site log-Pearson Type III

flow frequency estimates with regional regression estimates. At-site estimates of the 10-, 2-, 1- and 0.2-percent AEPs at the Ten Mile River streamgage were determined (Reference 42). These at-site estimates were based on 24 years of continuous record and 61 additional years of estimated peaks. The estimates calculated using regional regression equations are based on the basin characteristics drainage area (mi²), stream density (mi/mi²), and open water and wetland storage (percent) at the streamgage.

The Ten Mile River streamgage is just upstream of the Pawtucket Avenue bridge at East Providence, RI. Theoretical AEP flows were transferred upstream and downstream from the streamgage using the weighted hybrid method (Reference 43).

Whiting Pond Bypass

Flows for Whiting Pond Bypass (a diversion of the Ten Mile River) were determined using the split-flow optimization routine in HEC-RAS. Flows on the Ten Mile River at Broad Street (downstream of Whiting Pond Bypass) were split between the main stem of the Ten Mile River and Whiting Pond Bypass using this routine.

Sevenmile River

Discharges for given AEPs on the Sevenmile River were calculated using regional regression equations (Reference 42) and a drainage-area ratio method. The regional regression equations are based on the basin characteristics drainage area (mi²), stream density (mi/mi²), and open water and wetland storage (percent) at the streamgage.

The regional regression equations were used to estimate discharges for the 10-, 2-, 1-, and 0.2-percent AEPs for those sites from Orrs Pond Dam in the City of Attleboro downstream to the Amtrak railroad bridge (about 0.5 miles upstream of the confluence with the Ten Mile River). These sites' drainage areas were within the range of those USGS streamgages used in the regional regression equations by Zarriello and other (2012). For those sites from the Russell Tennant Water Treatment Facility Footbridge, Dam #2 in the City of Attleboro (upstream of Orrs Pond Dam) upstream to the outflow of the unnamed pond 100 feet east of Peck Road in Plainville, a drainage area ratio method was used to estimate discharges for the 10-, 2-, 1-, and 0.2-percent AEPs at Orrs Pond Dam in the City of Attleboro.

Wading River

Discharges for given AEPs at the Wading River near Norton, MA (USGS streamgage number 01109000) and at the Wading River at West Mansfield, MA (USGS streamgage number 01108500) are weighted values calculated with the USGS WIE program (Cohn and others, 2012). The program combines at-site log-Pearson Type III flow frequency estimates with regional regression estimates. At-site estimates of the 10-, 2-, 1-, and 0.2-percent AEPs at the Wading River gages were done (Reference 43). These at-site estimates were based on 86 and 33 years of peak flow record, respectively. The estimates calculated using regional regression equations are based on the basin characteristics drainage area (mi²), stream density (mi/mi²), and open water and wetland storage (percent) at the streamgage.

The Wading River near Norton gage (01109000, period of record 1926-2011) is just downstream of the Taunton Avenue bridge in Norton, MA. The Wading River at West Mansfield gage (01108500, period of record 1954-1986) is just downstream of the Balcom Street bridge in Mansfield, MA. Theoretical AEP flows from USGS gage 01108500 were transferred downstream to Richardson Avenue and AEP flows from

USGS gage 01109000 were transferred upstream to Power Street using a weighted hybrid method (Reference 43) that combines regression equation estimates at the new location with the weighted estimate determined at the gaged site. Flows between Power Street and Richardson Avenue were adjusted based on drainage area to provide a smooth transition between gage AEP flows.

Peak discharge-drainage area relationships for Bristol County are shown in Table 6, Summary of Discharges.

TABLE 6 – SUMMARY OF DISCHARGES

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
ABBOTT RUN					
Mendon Road	24.08	690	1120	1370	2120
Cushman Road	23.79	680	1110	1360	2100
Old Railroad Grade	23.37	670	1100	1340	2070
Hunts Bridge Road	22.87	660	1080	1310	2030
Corporate limit of North Attleborough	21.35	620	1010	1230	1910
ACUSHNET RIVER					
Dam at Station 79-30	17.90	280	475	620	935
Upstream of Hamilton Street	15.60	220	380	505	760
Upstream of Deep Brook	10.00	90	180	285	430
Below New Bedford Reservoir	6.80	40	90	170	250
ANAWAN BROOK					
Location 1* in Rehoboth	0.72	80	130	160	280
Location 2* in Rehoboth	0.60	70	110	140	250
Location 3* in Rehoboth	0.50	60	100	130	220

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
ARMSTRONG BROOK					
Confluence with Bungay River	0.19	24	41	49	75
Gravel Road	0.17	21	37	44	67
Lindsey Street	0.10	13	22	26	39
Cross Section B	0.09	11	19	23	36
ASSONET RIVER					
State Route 24 in Freetown	22.50	650	1022	1206	1948
State Route 79 in Freetown	22.20	640	1007	1191	1914
Mill Street in Freetown	22.00	637	1001	1180	1904
Dam No. 1	21.20	616	966	1137	1830
Gravel Road in Freetown	21.00	600	931	1079	1744
Locust Street in Freetown	20.90	580	897	1051	1666
Dam No. 2	20.80	577	893	1046	1660
Forge Road in Freetown	20.60	573	885	1036	1641
Dam No. 3	20.50	570	880	1030	1630
1,500 feet downstream of Myricks Street	16.80	500	765	885	1405
Myricks Street in Freetown	16.40	481	733	852	1333
Dam No. 4	16.30	469	718	836	1311
Northern corporate limit of Freetown	15.80	460	704	820	1288

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
ATTLEBORO INDUSTRIAL STREAM					
County Street in Attleboro	0.30	13	23	28	43
Tiffany Street in Attleboro	0.10	4	7	8	13
BAD LUCK BROOK					
Location 1* in Rehoboth	1.75	140	220	280	500
Location 2* in Rehoboth	1.65	130	210	270	460
Location 3* in Rehoboth	1.20	110	170	210	360
Location 4* in Rehoboth	0.71	80	130	160	260
Location 5* in Rehoboth	0.62	70	120	140	250
BLACK BROOK					
Above unnamed tributary below Foundry Street in Easton	6.20	270	450	550	850
Above Little Cedar Swamp	4.10	200	330	410	630
At private road below Depot Street	1.80	110	185	230	350
At Depot Street in Easton	1.40	85	140	180	270
At Summer Street in Easton	0.90	70	120	140	230
BLISS BROOK					
Location 1* in Rehoboth	2.50	180	300	380	530
Location 2* in Rehoboth	2.00	150	250	320	540
Location 3* in Rehoboth	1.60	130	210	260	450

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
BUNGAY RIVER					
Route 152 in Attleboro	8.00	82	130	160	230
Holden Street in Attleboro	7.00	81	130	150	230
Attleboro corporate limit	5.16	81	130	154	228
Confluence with Mary Kennedy Brook	5.05	81	130	154	228
Confluence with Armstrong Brook	4.09	81	130	154	228
Confluence with Landry Avenue Brook	3.22	46	87	110	180
Bungay Road in North Attleborough	2.14	27	52	66	110
BUTTONWOOD BROOK					
Location 1* in Dartmouth	3.10	300	495	595	800
Location 2* in Dartmouth	2.60	240	385	435	600
Location 3* in Dartmouth	2.10	190	250	290	355
CANOE RIVER					
At confluence with Winnecunnet Pond	19.10	450	695	815	1170
Approximately 1,150 feet downstream of upstream crossing of Interstate Route 495	13.10	345	530	620	890
Location 1* in Mansfield	11.30	190	260	400	640
Location 2* in Mansfield	6.80	140	220	280	460

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
CHARTLEY BROOK					
Town Boundary with Norton	6.60	180	270	320	430
Wilmarth Street	1.50	60	90	100	150
COBB BROOK					
At confluence with Taunton River	2.50	210	325	390	570
Above confluence of tributary at Godfrey Street	1.80	180	275	325	470
At Winthrop Street in Taunton	1.30	130	200	235	345
At East Whitehill Street in Taunton	1.10	105	160	190	280
At Kilmer Street in Taunton	0.70	65	110	130	185
At Tremont Street in Taunton	0.30	25	50	55	95
COLE RIVER					
At Milford Pond Dam	12.00	350	**	1,055	**
COLES BROOK					
Newman Avenue in Seekonk	3.00	110	185	235	345
Talbot Way in Seekonk	2.70	100	165	200	300
Cross Section E in Seekonk	2.50	90	150	185	275
DAM LOT BROOK					
At mouth	3.00	150	260	320	490

**Data not available

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
DEEP BROOK					
At confluence with Acushnet River	2.80	150	250	305	475
Downstream of Morses Lane in Acushnet	1.60	80	135	165	260
EAST BRANCH PALMER RIVER					
Location 1* in Rehoboth	13.50	550	830	980	1450
Location 2* in Rehoboth	10.25	440	660	780	1210
Location 3* in Rehoboth	5.50	310	500	620	1100
Knight Avenue in Attleboro	1.10	18	34	41	64
Thurber Avenue in Attleboro	0.90	14	26	31	49
ELMWOOD STREET BROOK					
Confluence with Ten Mile River	0.20	14	22	26	38
Washington Street in North Attleborough	0.19	13	21	25	36
Parmenter Lane in North Attleborough	0.11	8	12	14	21
FALL BROOK					
1,800 feet downstream of Dam No. 1 in Freetown	13.40	457	714	836	1356
Dam No. 1	13.30	455	712	835	1350
County Road in Freetown	13.30	453	710	834	1345

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
FALL BROOK – continued					
State Route 140 in Freetown	10.00	369	572	668	1067
Dam No. 2	9.90	367	570	666	1065
Braley Road in Freetown	9.70	365	566	662	1060
1,500 feet upstream of Braley Road	9.00	344	534	628	1014
Cross section H	8.40	323	502	591	957
Cross section I	7.80	309	470	554	890
Cross section J	7.20	288	439	517	826
Cross section K	6.60	267	411	480	763
1,150 feet downstream of Conrail	6.00	248	380	443	701
Conrail	5.40	228	350	406	641
Chace Road in Freetown	5.30	226	347	403	637
Dam No. 3	5.20	225	345	400	635
FORGE RIVER					
At mouth	9.30	340	570	690	1060
Above Tributary to Forge River	5.70	230	390	480	730
Above Pine Swamp Outlet	2.90	130	220	260	410
Above Wilbur Pond	1.40	68	115	141	219
Above Tributary No. 2	0.93	36	60	73	113

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
GOOSE BRANCH BROOK					
At confluence with Winnecunnet Pond	3.30	230	335	390	510
GOWARDS BROOK					
At Norton Avenue in Easton	1.83	110	175	210	300
At Highland Street in Easton	1.42	90	150	180	255
At State Route 106 in Easton	1.05	75	125	150	215
HODGES BROOK					
Location 1* in Mansfield	3.70	85	145	175	285
Location 2* in Mansfield	2.50	60	100	135	210
LAKE COMO STREAM					
Newport Avenue in Attleboro	1.30	87	150	180	270
Route 1 in Attleboro	0.30	15	23	26	31
LANDRY AVENUE BROOK					
Confluence with Bungay River	1.06	20	39	50	87
Bungay Road in North Attleborough	1.02	19	38	48	84
Irrigation Pond	1.00	19	37	47	82
Kelley Boulevard in North Attleborough	0.94	18	35	44	77
Interstate Highway 95 in North Attleborough	0.91	17	33	43	75

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
LANDRY AVENUE BROOK					
– continued					
Landry Avenue in North Attleborough	0.86	16	32	41	70
Kostka Drive in North Attleborough	0.82	15	30	39	67
Hall Drive in North Attleborough	0.77	15	28	36	63
MARY KENNEDY BROOK					
Confluence with Bungay River	0.96	49	82	98	150
Gravel Road in North Attleborough	0.95	48	81	97	150
Mary Kennedy Drive Extension	0.93	47	79	95	140
Mary Kennedy Drive in North Attleborough	0.78	40	67	80	120
Kelley Boulevard in North Attleborough	0.77	39	66	79	120
MASON PARK BROOK					
Confluence with Ten Mile River	0.50	35	66	82	130
Commonwealth Avenue in North Attleborough	0.48	35	63	79	130
Elm Street in North Attleborough	0.43	30	57	71	110
Mount Hope Cemetery	0.35	25	46	57	93
Spring and Lyman Streets	0.24	17	32	39	64

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
MASON PARK BROOK – continued					
Janice Lane in North Attleborough	0.18	13	24	30	48
Landry Avenue in North Attleborough	0.07	5	9	11	19
MULBERRY BROOK					
Above Ward Pond	9.00	370	620	760	1200
OAK HILL STREAM					
Oak Hill Avenue	0.80	32	60	75	120
Bishop Avenue	0.10	29	55	68	109
Conrail Crossing	0.50	22	43	53	86
OAK SWAMP BROOK					
Location 1* in Rehoboth	2.40	180	300	360	620
Location 2* in Rehoboth	2.00	150	260	310	530
Location 3* in Rehoboth	0.98	90	140	180	310
PALMER RIVER					
Location 1* in Rehoboth	46.50	1480	2360	2930	4750
Location 2* in Rehoboth	43.50	1420	2250	2750	4330
Location 3* in Rehoboth	32.50	1125	1750	2125	3275
Location 4* in Rehoboth	29.50	1050	1650	1990	3025
Location 5* in Rehoboth	26.40	950	1500	1800	2750

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
PALMER RIVER – continued					
Location 6* in Rehoboth	21.30	800	1250	1525	2275
PASKAMANSET RIVER					
Location 1* in Dartmouth	25.00	450	700	850	1200
Location 2* in Dartmouth	16.00	305	460	555	795
POQUANTICUT BROOK					
At Beaver Brook	5.70	270	450	550	840
At Chestnut Street in Easton	4.50	240	400	490	760
At Rockland Street in Easton	3.20	170	290	350	540
QUESET BROOK					
Above Coweeset Brook	10.40	400	670	820	1250
At State Route 138 in Easton	9.50	320	505	600	815
At Longwater Pond	7.34	270	425	510	690
At Shovelshop Pond	4.38	190	305	365	500
At Ames Lond Pond	2.80	140	230	275	380
RATTLESNAKE BROOK					
Narrows Road	6.86	344	588	664	1115
South Main Street in Freetown	4.29	246	388	457	726
Conrail	4.26	233	308	432	706
Confluence with Ten Mile River	1.05	52	88	106	160
Commonwealth Avenue	0.98	49	82	99	150

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
RATTLESNAKE BROOK – continued					
Ivy Street in North Attleborough	0.92	47	77	93	140
Towne Street in North Attleborough	0.84	42	70	85	130
ROCKLAWN AVENUE STREAM					
Todd Drive in Attleboro	0.40	15	26	32	51
Rocklawn Avenue in Attleboro	0.30	13	23	28	45
ROCKY RUN					
At the upstream Swansea corporate limits	6.10	357	575	719	1242
Location 1* in Rehoboth	10.50	540	870	1100	1890
Location 2* in Rehoboth	9.50	500	810	1000	1740
Location 3* in Rehoboth	6.60	410	650	810	1410
Location 4* in Rehoboth	6.60	350	570	710	1250
Location 5* in Rehoboth	5.10	310	500	620	1090
Location 6* in Rehoboth	3.30	220	360	450	780
RUMFORD RIVER					
At confluence with Three Mile River	22.30	500	770	910	1300
Location 1* in Mansfield	13.10	360	620	790	1260

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
RUMFORD RIVER – continued					
Location 2* in Mansfield	10.80	310	540	680	1090
Location 3* in Mansfield	8.00	250	445	560	880
Location 4* in Mansfield	6.80	230	405	505	800
RUNNINS RIVER					
School Street in Seekonk	9.60	275	450	535	800
Mink Street in Seekonk	9.10	260	430	510	755
Cross Section C	8.90	250	410	490	725
Cross Section D	8.20	230	375	450	665
Highland Avenue in Seekonk	7.50	195	315	375	605
Leonard Street in Seekonk	6.00	165	265	335	545
Fall River Avenue in Seekonk	5.90	160	255	330	535
Pleasant Street in Seekonk	4.20	105	175	235	405
Cross Section R	3.90	100	155	225	390
Arcade Avenue in Seekonk	3.30	85	135	205	355
Ledge Road in Seekonk	3.10	80	130	195	350
Greenwood Avenue in Seekonk	2.40	60	105	155	305

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
SABIN POND BROOK					
Above confluence of tributary at U.S. Route 44 in Taunton	4.80	385	565	650	870
Above confluence of tributary at Dirt Path No. 1 in Taunton	2.70	265	385	440	590
At Glebe Street in Taunton	1.20	155	225	260	345
Location 1* in Rehoboth	0.50	60	100	130	230
SCOTTS BROOK					
Confluence with Ten Mile River	1.21	110	190	230	340
Washington Street in North Attleborough	1.18	110	180	220	330
Avery Street in North Attleborough	1.15	100	170	210	310
Arnold Road in North Attleborough	1.07	87	150	180	270
High Street in North Attleborough	0.98	79	130	160	250
SEGREGANSET RIVER					
Confluence of Sunken Brook	13.40	600	995	1255	1269
Center Street in Dighton	11.00	504	849	1027	1797
Near Briggs Road in Dighton	1.20	70	100	110	165
At Taunton corporate limits	5.40	415	610	700	935

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
SEVENMILE RIVER					
Amtrak railroad bridge in Attleboro	12.6	577	961	1,144	1,620
County Street in Attleboro	12	532	886	1,055	1,532
Pitas Avenue in Attleboro	10.2	468	779	928	1,311
Roy Avenue in Attleboro	9.31	417	694	827	1,166
Read Street in Attleboro	7.28	322	536	640	900
Orrs Pond Dam in Attleboro	7.14	306	509	607	852
West Street in Attleboro	5.01	230	386	462	651
Luther Reservoir Dam in Attleboro	4.44	209	351	421	594
Old mill dam upstream of Old Post Road in North Attleborough	3.6	179	302	363	513
Draper Avenue in North Attleborough	3.5	173	292	351	496
Riverview Drive in North Attleborough	3.05	155	262	315	446
Hoppin Hill Avenue in North Attleborough	1.94	108	184	223	317
Hickory Road in North Attleborough	0.53	38	67	82	118
Culvert in farm field 1,700 feet south of High Street in North Attleborough	0.41	31	55	67	97

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
SPEEDWAY BROOK					
South Main Street in Attleboro	3.10	170	280	340	510
Maple Street in Attleboro	0.75	80	140	170	260
SUNKEN BROOK					
Center Street in Dighton	2.20	123	187	216	341
3,850 feet upstream of Center Street in Dighton	1.40	75	106	125	198
TAUNTON RIVER					
Plain Street above Three Mile River in Taunton	363	4,890	7,260	8,420	11,100
County Street/Route 140 above Mill River in Taunton	317	4,230	5,940	6,770	8,690
Route 24 above Forge River	302	4,080	5,630	6,380	8,120
South Street	293	3,970	5,430	6,120	7,750
US Route 44	283	3,860	5,210	5,850	7,370
Green Street/Plymouth Street	271	3,740	4,990	5,570	6,970
Titicut Street (Taunton River near Bridgewater, MA streamgage number 01108000)	262	3,660	4,830	5,380	6,690
Auburn Street	183	2,820	4,140	4,780	6,260
Cherry Street	129	2,280	3,590	4,230	5,720

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
TEN MILE RIVER					
Railroad Bridge and Omega Pond Dam, East Providence, RI	55.4	1510	2440	2940	4200
Pawtucket Avenue (Routes 114, 1, and 1A), East Providence, RI (USGS streamgage 01109403)	53.7	1440	2330	2820	4030
Dam downstream of Pond Street near Maple Avenue in Seekonk	28.7	1290	2,140	2550	3640
Old unnamed road at Elks Lodge, 887 South Main Street, Attleboro	25.2	1140	1900	2260	3220
West Street in Attleboro	11.7	875	1500	1780	2580
Freeman Street in North Attleborough	9.54	637	1090	1290	1860
Mount Hope Street in North Attleborough	8.39	506	864	1020	1450
Trailer Park Arch at 300 East Washington Street, North Attleborough	5.56	388	679	807	1160
East Washington Street (Route 1) in North Attleborough	4.91	324	566	672	962
Broad Street in North Attleborough	3.65	179	309	365	507
Abandoned dirt road at crushed stone operation off Cross Street in Plainville	1.43	122	224	268	394

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
TEN MILE RIVER – continued					
High Street in Plainville	0.59	56	102	122	177
THREE MILE RIVER					
At confluence with Taunton River	84.60	1820	2710	3170	4440
THREE MILE RIVER – WEST CHANNEL					
At confluence with Three Mile River	**	900	1430	1690	2440
TRIBUTARY TO DAM LOT BROOK					
At mouth	0.54	41	71	87	140
TRIBUTARY TO FORGE RIVER					
At mouth	2.80	110	180	220	340
At White Street in Raynham	2.30	100	165	205	310
WADING RIVER					
At confluence with Three Mile River, Norton	44.1	910	1,410	1,680	2,330
At Taunton Avenue (Route 140) Norton, MA (USGS gage 01109000)	43.7	900	1,400	1,660	2,310
At Power Street in Norton	37	850	1,340	1,590	2,200
Above confluence with Chartley Brook in Norton	29.6	680	1,140	1,380	1,950

**Data not available

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
WADING RIVER - continued					
At Walker Street in Norton	26.5	610	1060	1290	1840
Above confluence with Hodges Brook, near Richardson Avenue in Norton	21.5	500	920	1,150	1,670
At Balcom Street in Mansfield (USGS gage 01108500)	19.9	430	820	1,040	1,520
At Cedar Street in Foxborough	18.2	430	810	1,010	1,465
WEST BRANCH PALMER RIVER					
Location 1* in Rehoboth	7.90	430	700	870	1500
Location 2* in Rehoboth	6.90	380	520	780	1340
Location 3* in Rehoboth	5.00	300	490	510	1060
Location 4* in Rehoboth	4.30	280	420	630	950
Location 5* in Rehoboth	3.65	240	380	500	850
Location 6* in Rehoboth	1.15	100	160	210	360
Location 7* in Rehoboth	0.90	80	130	160	300
WHITING POND BYPASS					
At confluence with Ten Mile River	**	89	154	182	253

*Values estimated from the Frequency-Discharge, Drainage Area Curves following this table

**Data not available

TABLE 6 – SUMMARY OF DISCHARGES - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>DRAINAGE AREA (SQUARE MILES)</u>	<u>PEAK DISCHARGES (CUBIC FEET PER SECOND)</u>			
		<u>10- PERCENT ANNUAL CHANCE</u>	<u>2- PERCENT ANNUAL CHANCE</u>	<u>1- PERCENT ANNUAL CHANCE</u>	<u>0.2- PERCENT ANNUAL CHANCE</u>
WHITMAN BROOK					
At Longwater Pond	2.97	150	240	290	420
At Conrail	1.94	110	180	220	335
At Stoughton-Easton corporate limits	1.55	95	155	190	295

Frequency-Discharge, Drainage Area relationships are shown in Figures 1-11 for Anawan Brook-Bliss Brook, Bad Luck Brook, Buttonwood Brook, Canoe River-Wading River, East Branch Palmer River, Hodges Brook-Rumford River, Palmer River, Paskamanset River, Rocky Run, Sabin Pond Brook-Oak Swamp Brook, and West Branch Palmer River, respectively.

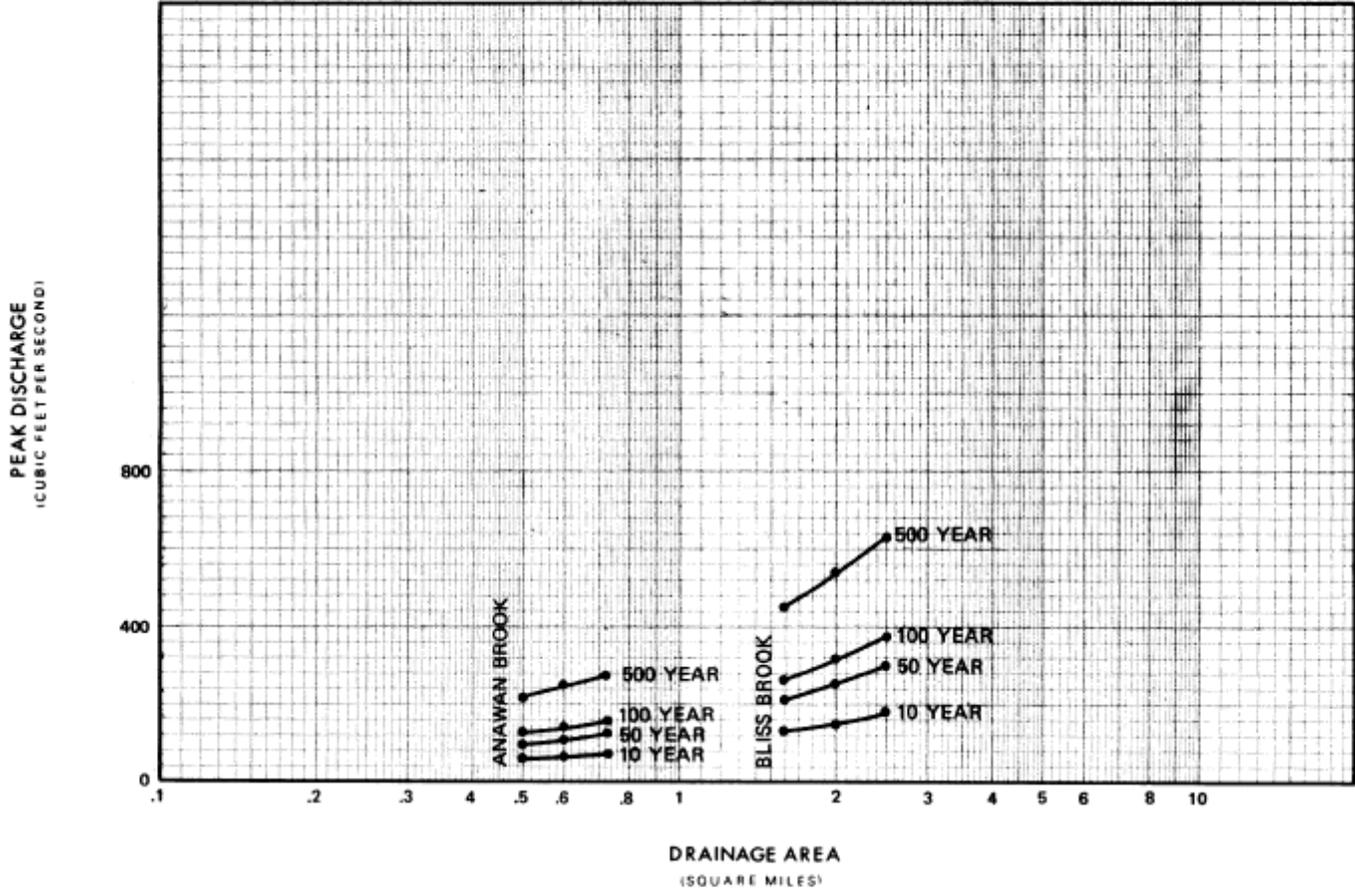


Figure 1

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
 Federal Insurance Administration
TOWN OF REHOBOTH, MA
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES
ANAWAN BROOK - BLISS BROOK

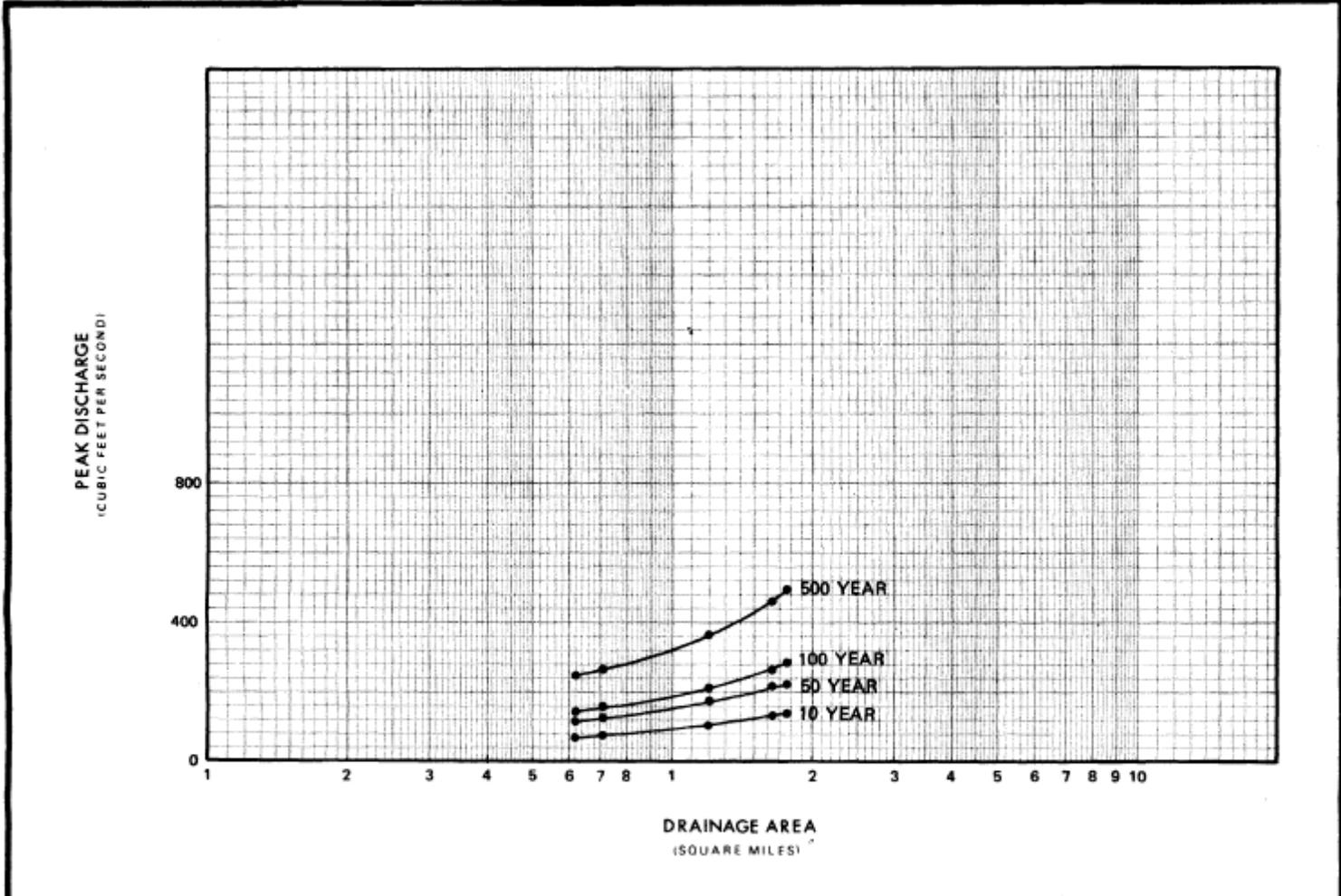


Figure 2

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
 Federal Insurance Administration
TOWN OF REHOBOTH, MA
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES
BAD LUCK BROOK

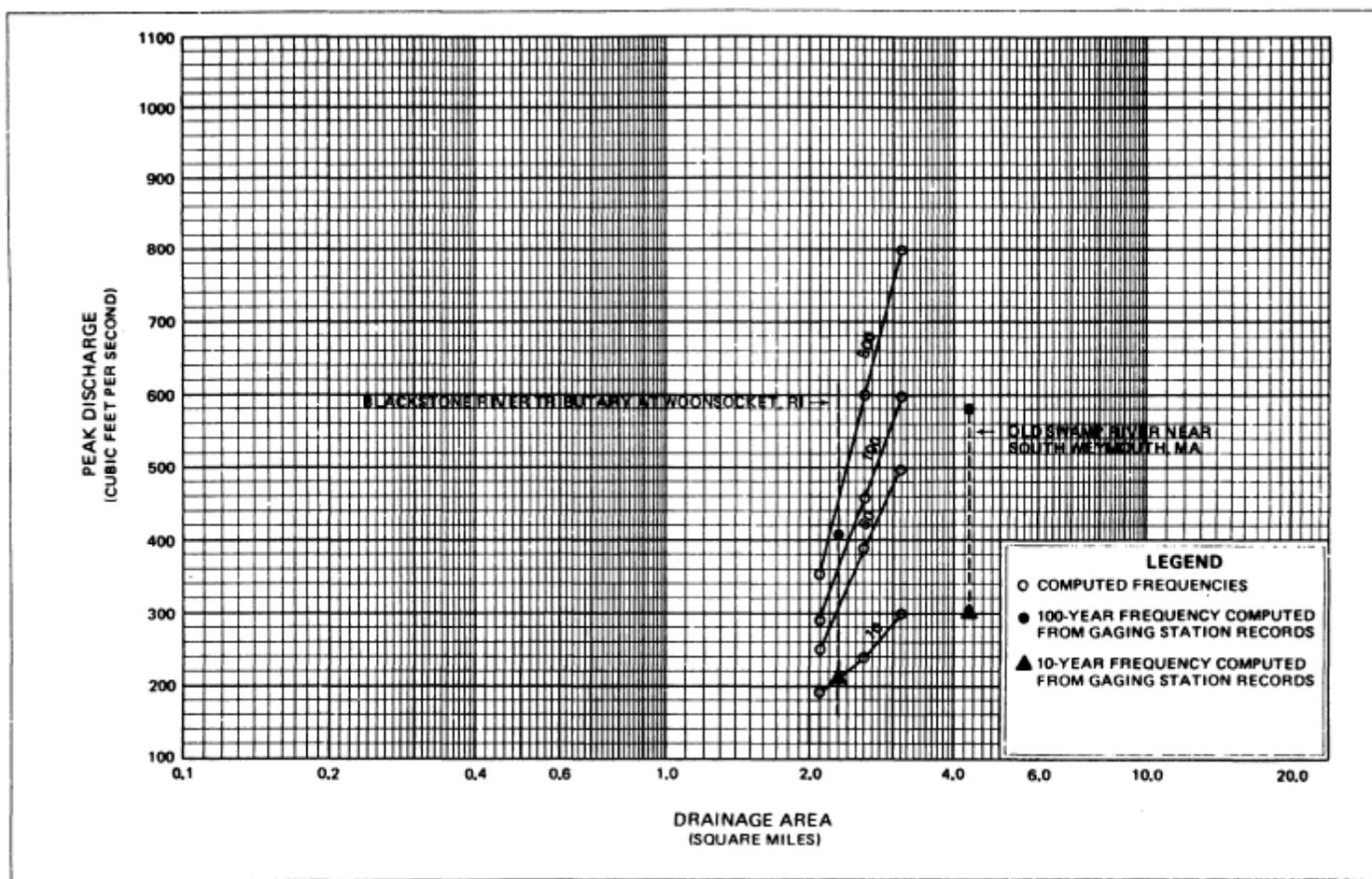


Figure 3

FEDERAL EMERGENCY MANAGEMENT AGENCY
TOWN OF DARTMOUTH, MA
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES
BUTTONWOOD BROOK

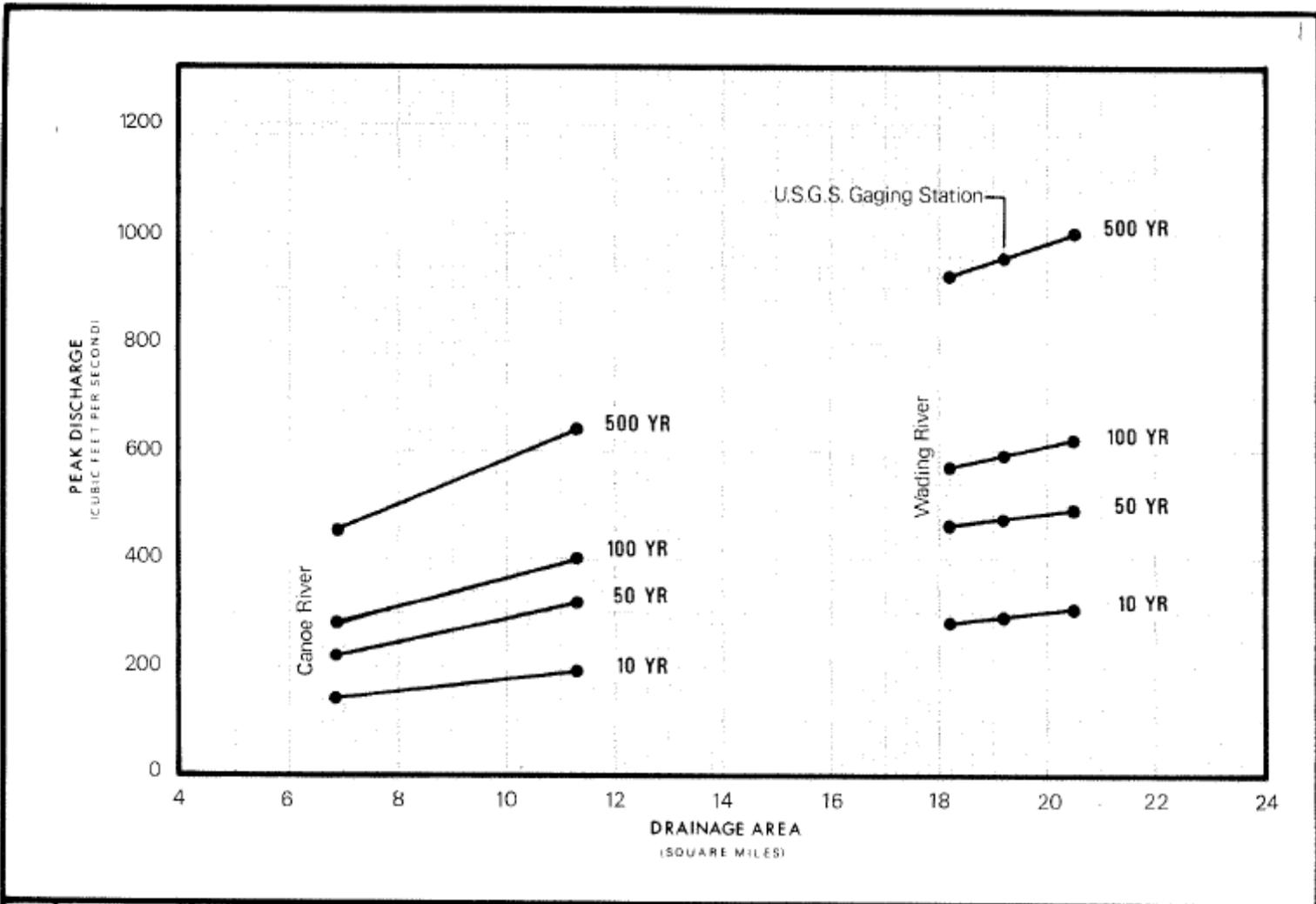


Figure 4

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
 Federal Insurance Administration
 Town of Mansfield
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES

CANOE RIVER - WADING RIVER

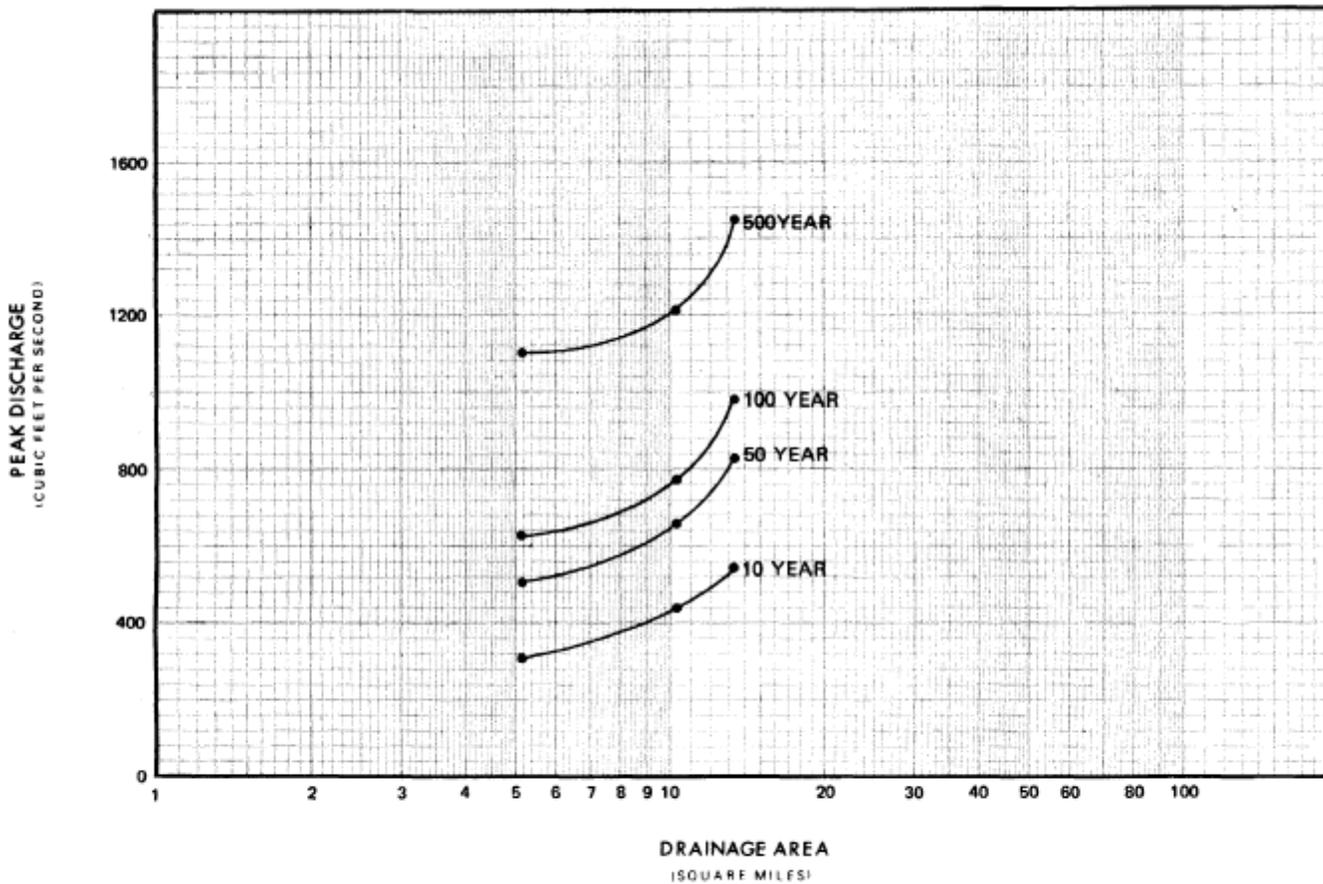


Figure 5

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
 Federal Insurance Administration
TOWN OF REHOBOTH, MA
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES

EAST BRANCH PALMER RIVER

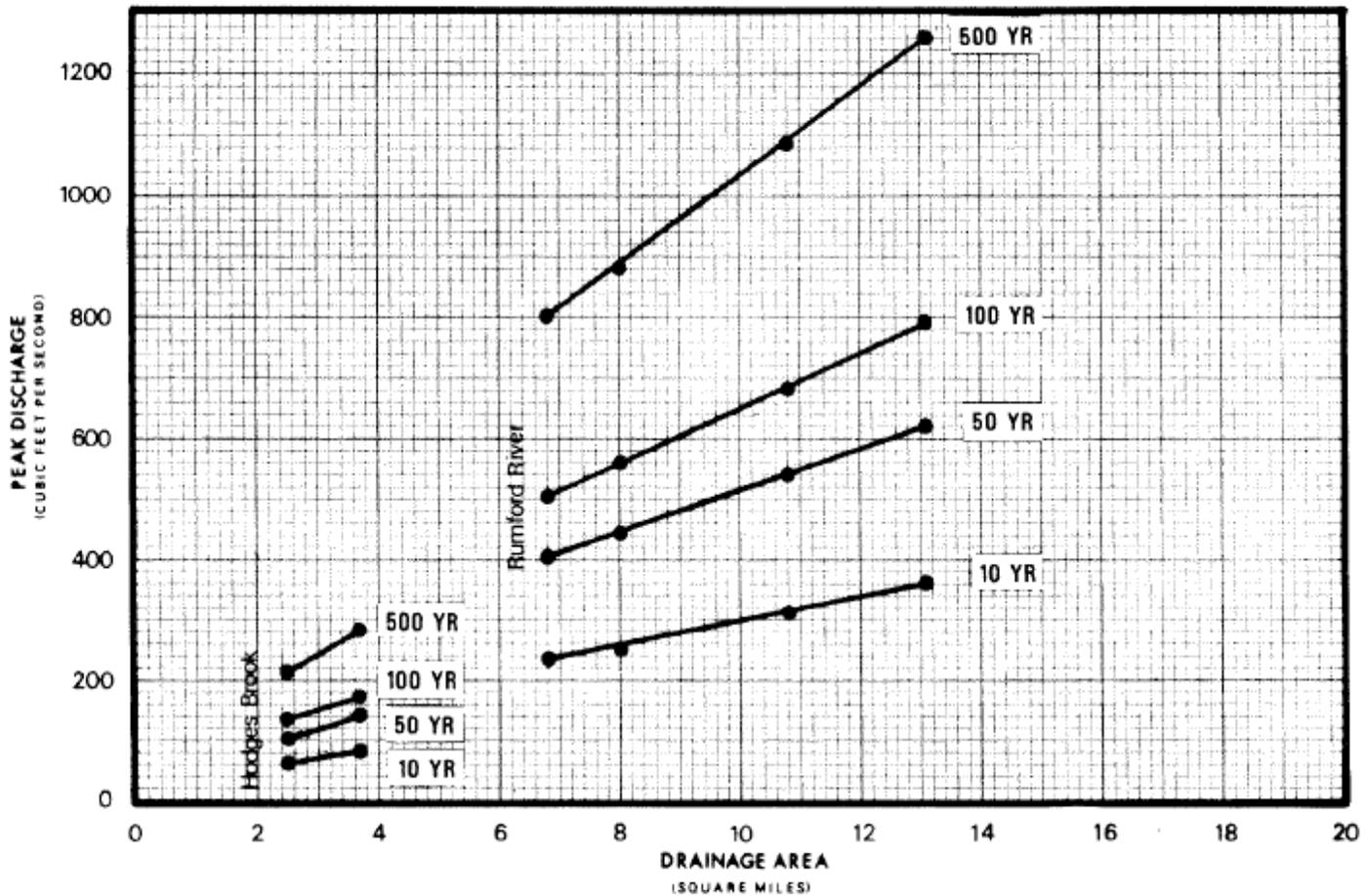


Figure 6

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
 Federal Insurance Administration
 Town of Mansfield
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES

HODGES BROOK-RUMFORD RIVER

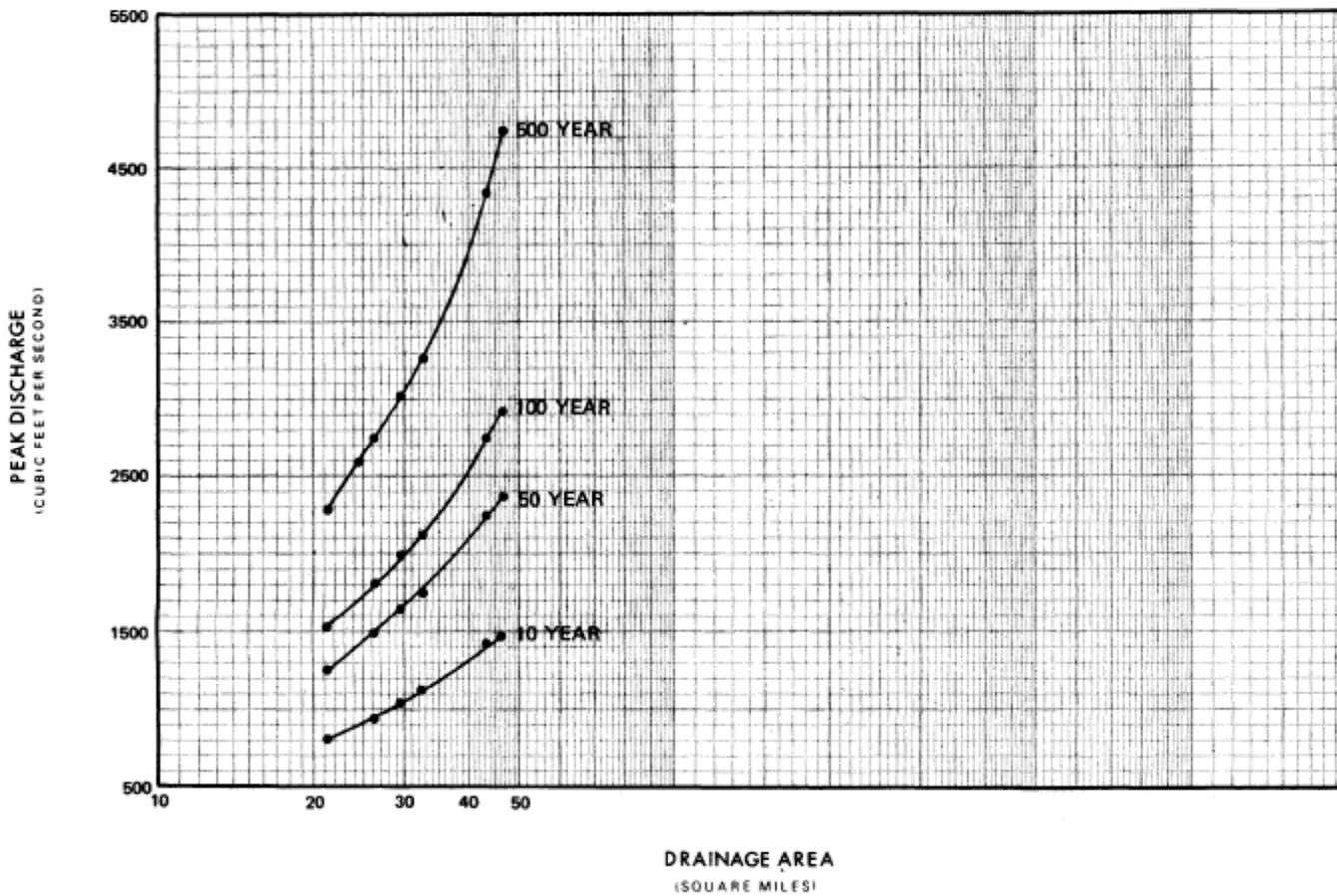


Figure 7

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
 Federal Insurance Administration
TOWN OF REHOBOTH, MA
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES

PALMER RIVER

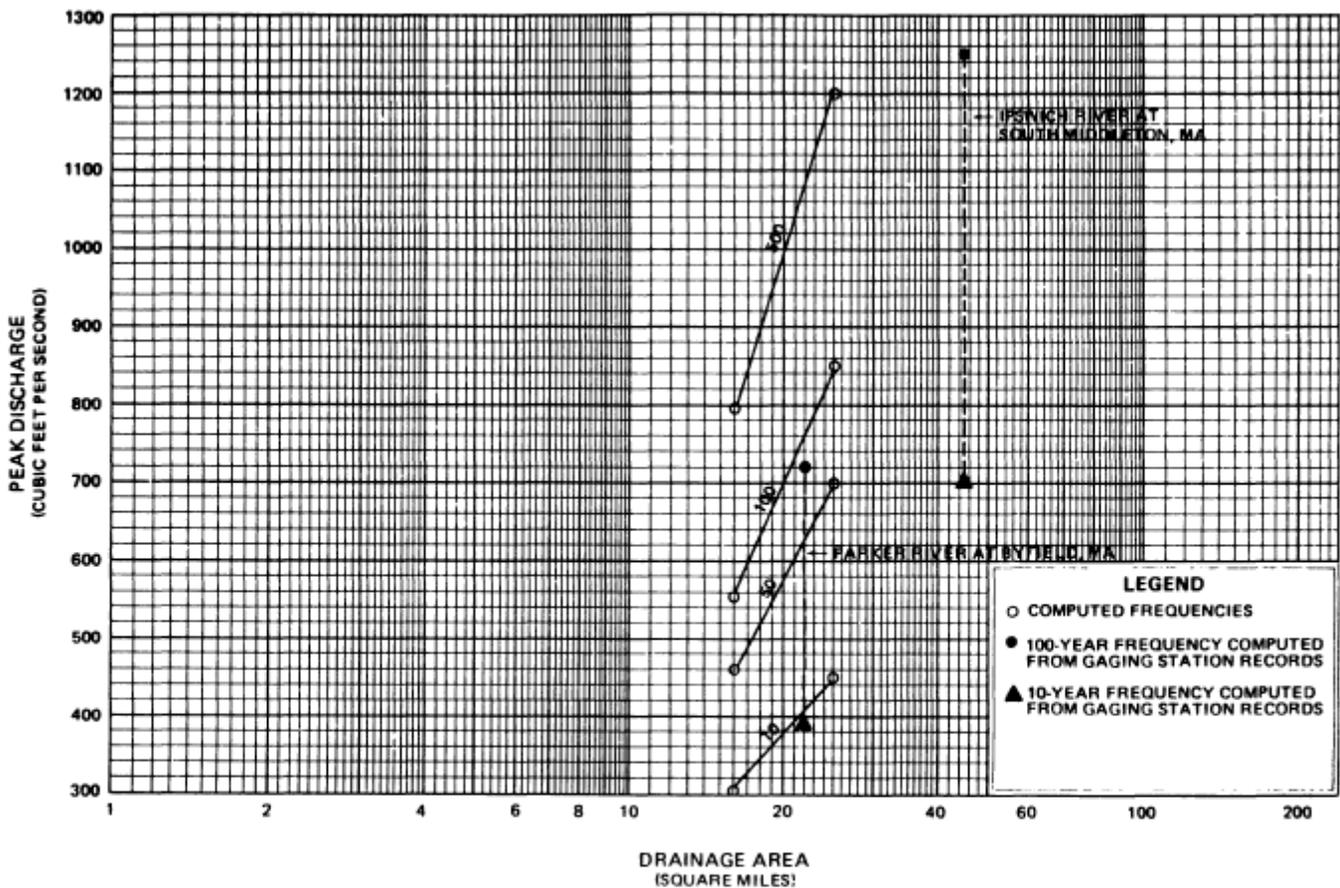


Figure 8

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF DARTMOUTH, MA
BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES

PASKAMANSET RIVER

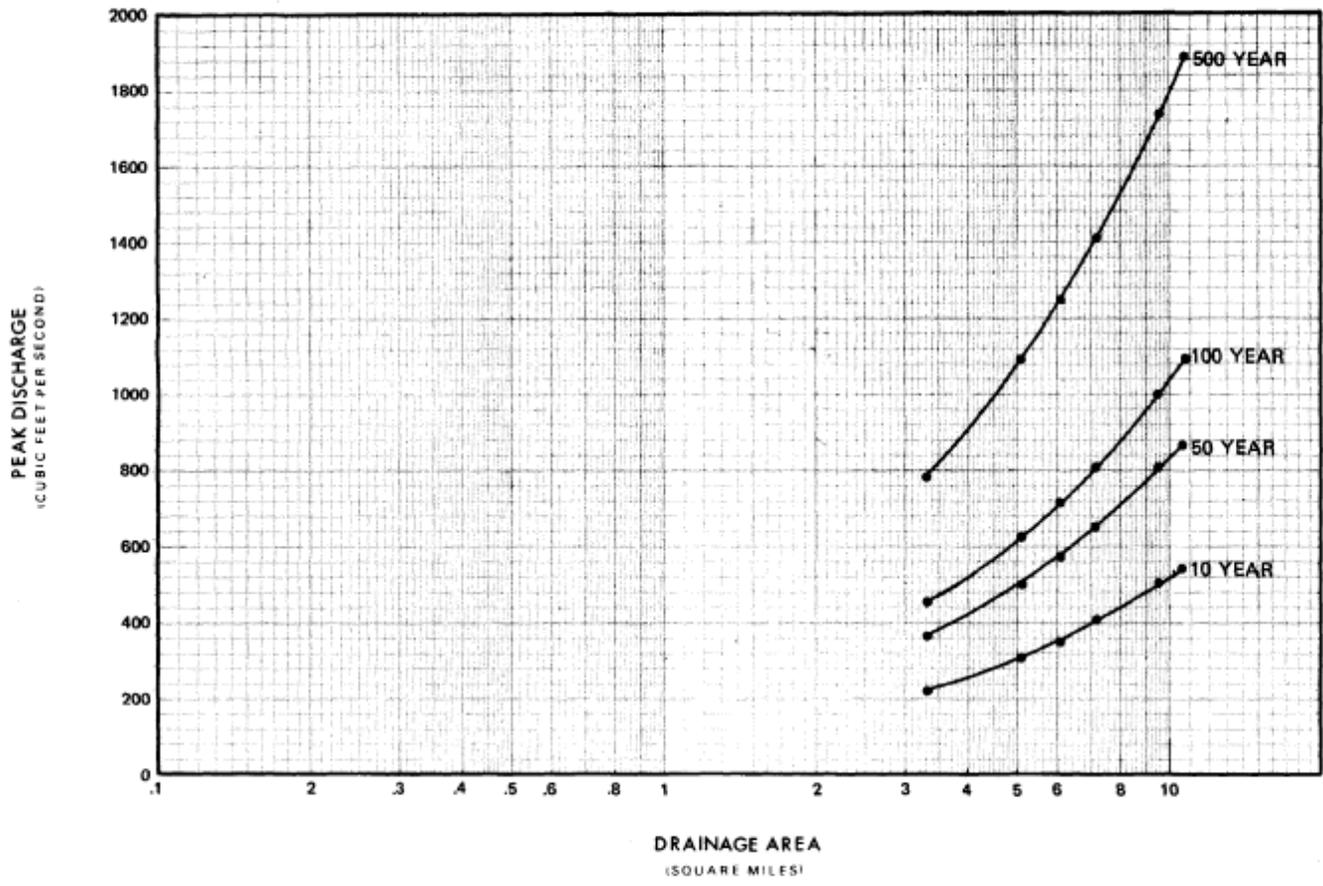


Figure 9

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
 Federal Insurance Administration
TOWN OF REHOBOTH, MA
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES

ROCKY RUN

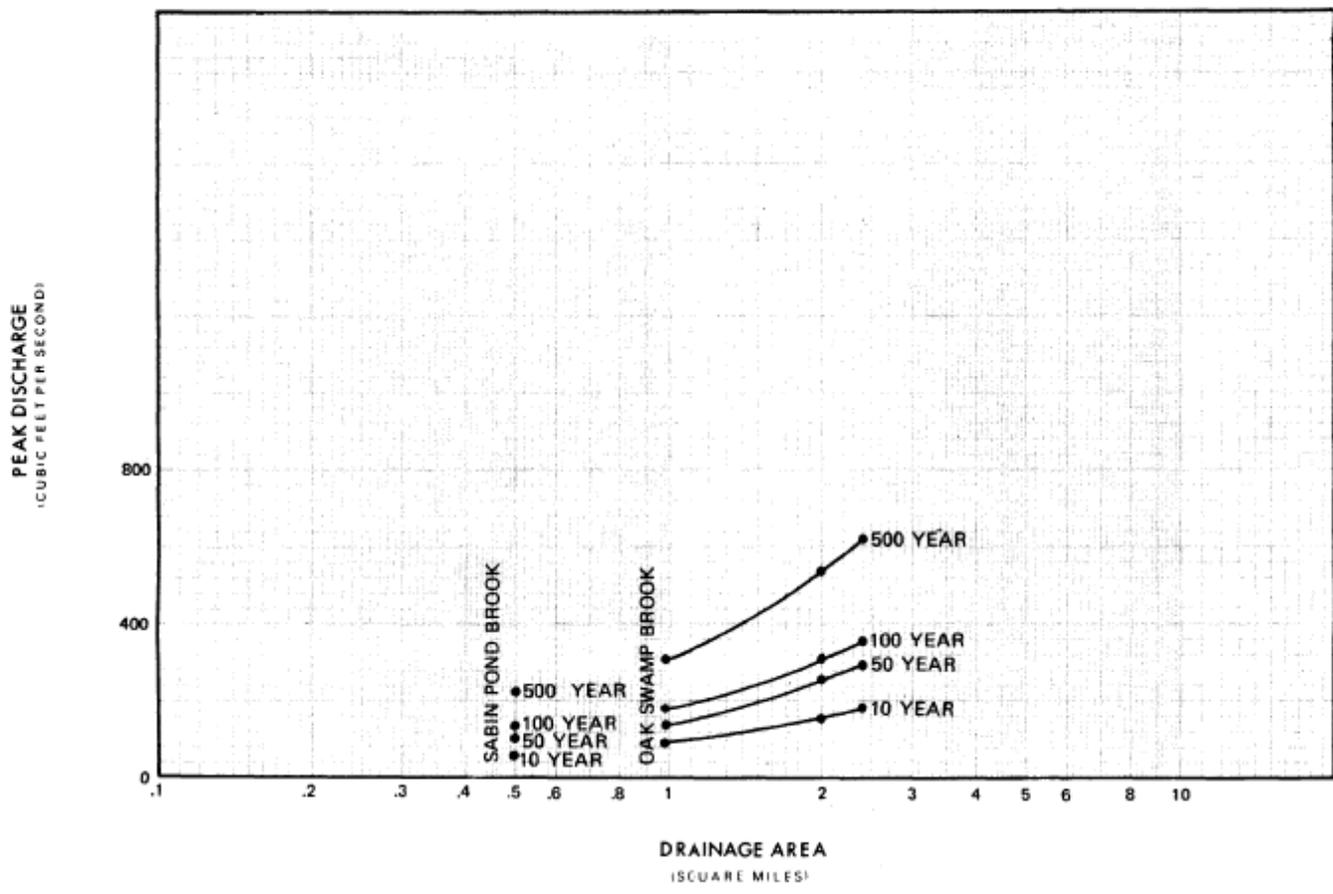


Figure 10

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
 Federal Insurance Administration
TOWN OF REHOBOTH, MA
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES
SABIN POND BROOK - OAK SWAMP BROOK

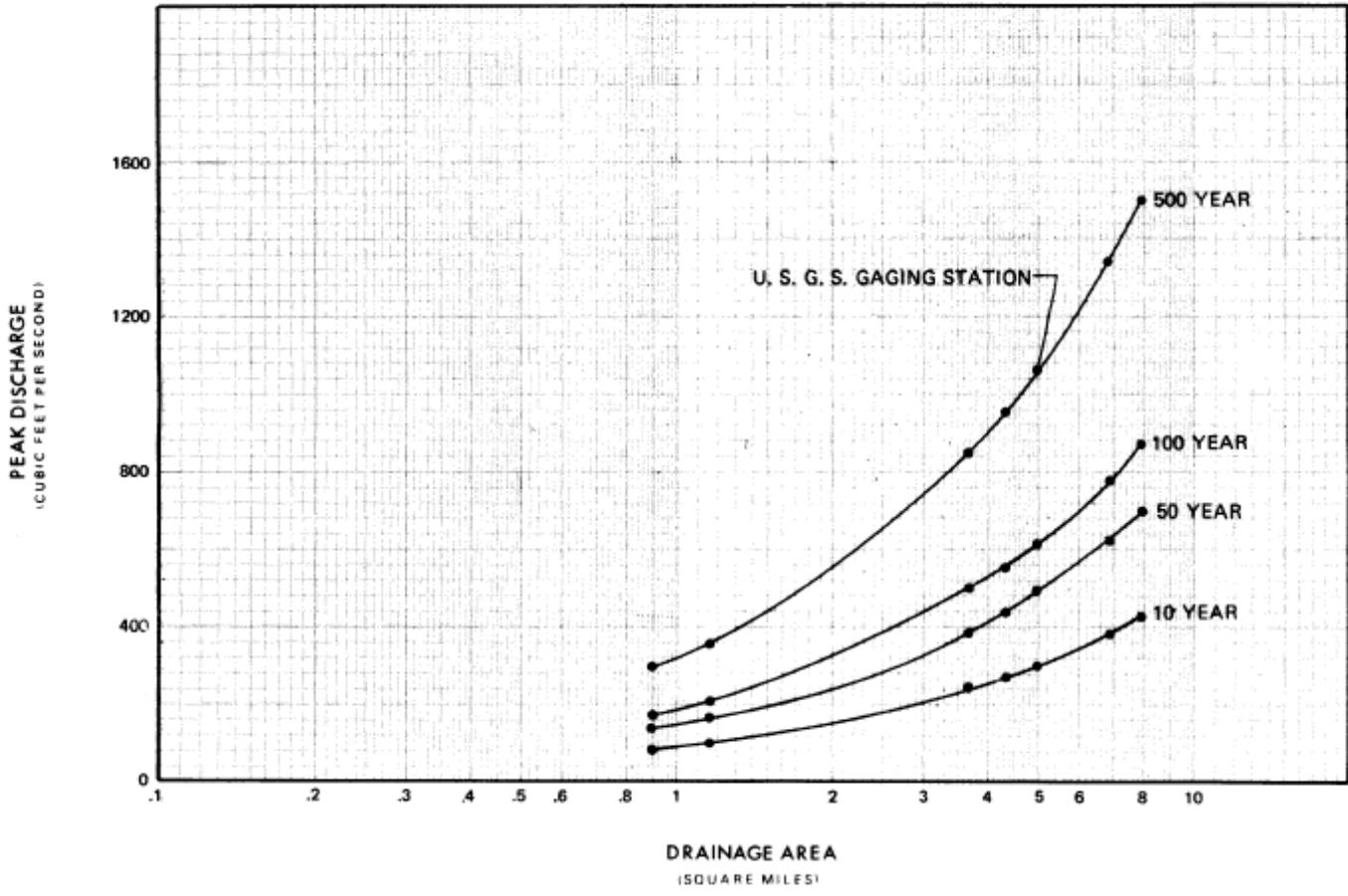


Figure 11

DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT
 Federal Insurance Administration
TOWN OF REHOBOTH, MA
 BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES
WEST BRANCH PALMER RIVER

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 [Flood Insurance Rate Map (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.

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

Locations of selected cross sections used in the hydraulic analyses are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway was computed (Section 4.2), selected cross-section locations are also shown on the FIRM.

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

For each community within Bristol County that has a previously printed FIS report, the hydraulic analyses described in those reports have been compiled and are summarized below. Note that for Sevenmile River, Ten Mile River, Taunton River, Wading River, and Whiting Pond Bypass, all precountywide analyses have been superseded by updated analyses in countywide studies described after the precountywide analyses below.

Precountywide Analyses

Starting water-surface elevations for Ten Mile River in the City of Attleboro and the Town of North Attleborough and Chartley Brook in the City of Attleboro were obtained by rating curves. Lake Como and Rocklawn Avenue Stream starting water-surface elevations were obtained from Sevenmile River backwater in the City of Attleboro. All other starting water-surface elevations in the City of Attleboro and the Town of North Attleborough were obtained from Ten Mile River backwater. Stevens Swamp water-surface elevations are controlled by backwater from the Sevenmile River. Water-surface elevations of floods of the selected recurrence intervals were computed through the use of the SCS WSP 2 (water-surface profile) Computer Program, TR-61 (Reference 44). Water-surface elevations for the reservoirs and other large water storage areas in the community were determined by routing floods of the selected recurrence intervals using the Computer Program for Project Formulation-Hydrology (Reference 23). Flood elevations along streams studied by approximate methods were determined from the best topographic maps available for each area (Reference 29), and were field checked for reasonableness where practicable and engineering judgment was applied.

In the Town of Berkley, historical data from previous hurricanes and storms were investigated, and the results plotted on log-probability paper. Most of the information was

obtained from the USACE reports (References 45 and 46) on the area. The 10-percent-annual-chance frequency tide level will produce a water-surface elevation adjacent to the Town of Berkley much greater than that observed during the 1968 storm, when the water-surface elevation was 7.7 feet at a point 1.5 miles above the Berkley Bridge. This storm produced the highest level of water ever recorded for the upper reaches of the Taunton River, but had a relatively minor effect on the lower reaches of the river in the vicinity of the Town of Berkley. Statistical analysis has indicated that this storm was equivalent to the 1-percent-annual-chance flood for the upper reaches of the Taunton River (Reference 4). Subsequent analysis indicated that riverine flooding along the Taunton and Assonet Rivers would be negligible compared to flooding caused by excessive high tide and, therefore, no backwater program was performed. Utilizing historical information, field observation, and basic hydraulic calculations, areas prone to flooding were delineated for the recurrence interval of approximately 1-percent-annual-chance.

In the Town of Dartmouth, water-surface elevations of floods of the selected recurrence intervals were computed using the USACE HEC-2 step-backwater computer program (Reference 47). Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals. Mean high water was used as the starting water-surface elevation in the riverine areas affected by tidal backwater. In areas not affected by tidal backwater, starting water-surface elevations were based on discharge-rating curves. Hydraulic analyses, considering storm characteristics and the shoreline and bathymetric characteristics of the flooding sources studied, were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along each of the shorelines.

In the Town of Acushnet, water-surface elevations of the floods of selected recurrence intervals were computed also through the use of the USACE HEC-2 step-backwater computer program (References 48 and 49). Starting water-surface elevations for the Acushnet River were obtained using the mean high tide for the 10- and 2-percent-annual-chance floods, and by slope/area determinations for the 1- and 0.2-percent-annual-chance frequency floods. Flood elevations for the tidal portion of the Acushnet River were taken from the FIS for the City of New Bedford (Reference 50). Known water-surface elevations for the Acushnet River were used as starting elevations for Deep Brook.

The starting water-surface elevations for the Segreganset River in the Town of Dighton were determined from elevations of the Taunton River. The starting water-surface elevations for Sunken Brook were determined from elevations of the Segreganset River. Starting water-surface elevations for the Three Mile River and the West Channel Three Mile River were determined by the slope conveyance method.

Water-surface profiles for the Segreganset River and Sunken Brook were developed using a modified HEC-2 computer program and step-backwater model (Reference 51). The hydraulic analysis was performed by utilizing the USGS step-backwater program E432 (Reference 52). Profiles were determined for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods. Because riverine flooding is negligible compared to the dominance of tidal flooding, no free-flow hydraulic analysis was performed on the Taunton River. The lower reaches of the Three Mile River, the West Channel Three Mile River, and the Segreganset River are under the influence of tidal flooding from the Taunton River. The Taunton River was studied in detail by CDM (Reference 53). In 1964, the New England Division USACE conducted a study to determine the flood frequency potential for waters

adjacent to the Taunton River in the vicinity of the City of Fall River and the Town of Somerset (Reference 43). Historical data from previous hurricanes and storms were investigated, and the results were plotted on log-probability paper. Most of the information was obtained from the USACE reports (References 45, 46, 54, and 55) in the area.

In both the August 3, 1981 and May 16, 1995 Easton FISs, water-surface elevations of floods of the selected recurrence intervals were computed using the USGS E431 step-backwater computer program (Reference 52). Starting water-surface elevations were determined by step-backwater methods applied at reaches below detailed study areas; starting water-surface elevations for Poquanticut Brook at New Pond and Queset Brook at Dean Pond were determined from water-surface elevations versus discharge relationships developed for dams. Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals.

For the August 9, 2000 Easton FIS, water-surface profiles for Gowards Brook were developed using the USACE HEC-2 step-backwater computer program (Reference 56). Starting water-surface elevations were calculated using step-backwater methods applied at reaches below the detailed studied areas. Hydraulic structure and cross section data for Gowards Brook was obtained from field surveys. For areas inaccessible by survey field crews, land cover and elevations were estimated from available mapping and field observations.

In the Town of Freetown, starting water-surface elevations for the Assonet River were taken from the Taunton River mean annual tide. Fall Brook and Rattlesnake Brook starting elevations were taken from normal flow elevations determined by field inspections and field surveys. Water-surface profiles for the Assonet River, Fall Brook, and Rattlesnake Brook were developed using a HEC-2 backwater computer (Reference 51). No backwater program on the Taunton River was performed because analysis by the USACE indicated that riverine flooding along the Taunton River would be negligible compared to flooding caused by excessive high tide. Flood boundaries along streams flowing through undeveloped areas were determined by approximate methods. Approximate flood elevations were determined by overlaying USGS topographic maps (Reference 29) on the Flood Hazard Boundary Map (FHBM) (Reference 57) and determining elevations from the topographic contour intervals. Normal depth calculations were used to check elevations from the topographic maps. The portion of Quaker Brook within the corporate limits of the Town of Freetown lies within the 1-percent-annual-chance floodplain determined for the Assonet River.

Water-surface elevations of floods in the Town of Mansfield were computed through use of the USACE HEC-2 step-backwater computer program (Reference 58). For the Wading River at the gage location at Balcom Street, the computed profiles agree with recorded flood elevations. Starting elevations for the study streams were developed by the slope-area method.

In the Town of Norton, cross sections for the backwater analyses of the streams studied by detailed methods were obtained from aerial photographs at a scale of 1:12,000 (Reference 59). Water-surface elevations of floods of the selected recurrence intervals for the Rumford River were computed using the USGS E431 step-backwater computer program (Reference 53). Water-surface elevations of floods of the selected recurrence intervals for the Canoe River, the Wading River, and Goose Branch Brook were

computed using the USACE HEC-2 step-backwater computer program (Reference 60). Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals. Starting water-surface elevations for the Rumford River, the Wading River, and Goose Branch Brook were calculated using the slope/area method. Starting water-surface elevations for the Canoe River were taken from the flood elevations at Winnecunnet Pond.

Water-surface elevations of floods of the selected recurrence intervals in the Town of Raynham were computed through the use of the USGS E431 step-backwater computer program (Reference 52). Starting water-surface elevations for the Taunton River were obtained from the June 18, 1987 Taunton FIS (described below). For the Forge River and Dam Lot Brook, starting elevations were computed using a slope/area method; the tributaries studied were based on the main stream flood elevation. Elevations above dams were based on computations of head over dam for selected discharges. At Kings Pond, elevations were also based on the regulation of flow through orifices. Computations on the Taunton River at the site of the Church Street bridge were based on construction plans for the new bridge to be built. Also, computations in this area were based on construction plans for the new dam under construction at Wilbur Pond.

Water-surface elevations of floods in the Town of Rehoboth were computed through use of the HEC-2 step-backwater computer program (Reference 48). Flooding along the lower Palmer River in the Town of Rehoboth may result from tidal action. This necessitated the determination of whether the governing influence for inundation would result from riverine flow, tidal flood, or a combination of both. To find the location of this interface, riverine and tidal flood heights were graphically compared for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods. For the riverine portion of this study phase, the mean spring high-water elevation of 3.8 feet NGVD (3.0 feet NAVD) was used as a starting water-surface elevation. The tidal portion of this study phase, on the other hand, was based on the 1-percent-annual-chance tidal flood height of 10 feet NGVD (9.2 feet NAVD), as determined by the FIS for the Town of Warren, Rhode Island (Reference 36). It is recognized that tidal conditions are extremely transitory, and peak tide levels are maintained for relatively short time periods. However, it was calculated that, during the course of the 1-percent-annual-chance event tidal cycle, the gradual level would create a weir flow condition at the Route 6 bridge, though low flow would still prevail at the Interstate 195 bridge further upstream. The volume of water passed by these bridges over the course of the tidal cycle was calculated as sufficient to fill the area adjacent to the stream and below the 10.0-foot NGVD (9.2-foot NAVD) contour level. This 10.0-foot NGVD (9.2-foot NAVD) elevation was computed to flood depths calculated by a riverine backwater analysis, starting at the corporate limits and using mean spring high tide as a starting water-surface elevation. Tidal influences extend upstream to the area between Providence Street ridge and the dam at Shad Factory Pond and, for the base (1-percent-annual-chance) flood, extend approximately 3.5 river miles above the corporate limits.

Because the Town of Rehoboth has several expansive swamps that do not lend themselves to riverine-type analyses, it was necessary to devise a general methodology to accurately reflect their potential for flooding. Proper identification of such areas is often difficult because of scanty flood data, low velocities, and the variability of flows. Flooding of these swamp and wetland areas is dependent on the immediate drainage area, soil characteristics, and the amount, type, and duration of precipitation. An amount of precipitation equivalent to the 1-percent-annual-chance event, taken over a duration of 6 hours, was selected as the base condition, and inflow accumulation was calculated

according to drainage area and soil types for each area. For these swamps studied in detail, which feed into a detailed study stream, the outflows were known and accumulations were determined by the relationship of inflow to outflow. In those areas where the results of the methodology indicated water depths of 1 foot or less, flooding was considered minimal; those areas were designated as Zone X. Water-surface elevations of areas studied by approximate methods were based on hydrologic considerations, onsite examination, and detailed results of inundation of similar locations in the immediate area, all weighted according to past flood history and engineering judgment. The Town of Rehoboth also contains many swamp-like areas of various sizes that have no definite inlet or outlet. In these areas, the accumulation of surface water is primarily dependent on the elevation of the ground water table. Areas displaying these characteristics are classified as perched swamps and, as such, were not considered as part of the September 1977 Rehoboth FIS.

In the Town of Seekonk, starting water-surface elevations for the Runnins River and Oak Hill Stream were obtained by rating curves. The Coles River starting water-surface elevations were obtained from the backwater of the Ten Mile River. Water-surface elevations of floods of the selected recurrence intervals were computed through the use of the SCS WSP-2 water-surface profile computer program (Reference 44). The water-surface elevation for downstream of School Street on the Runnins River was obtained from the East Providence, Rhode Island, FIS (Reference 61). The stillwater tide elevations along the lower reaches of the Runnins River for the frequency floods studied are the same as those for the upper reach of the Barrington River. The hurricane tidal effect of Narragansett Bay on the Runnins River was evaluated by analysis of historical high water measurements dating from 1935 and recent gage readings by the U.S. Coast and Geodetic Survey. The percent chance of occurrence was calculated for each data point and the information was plotted as a tidal-flood elevation frequency curve. The backwater elevation of Narragansett Bay up the Runnins River was derived from this curve. Flood stages for the 1-percent-annual-chance flood along streams studied by approximate methods were determined from Regional Stage versus Drainage Area Curves for Massachusetts (Reference 62) and an analysis of the 1-percent-annual-chance flood stages developed by the watershed models along the streams studied in detail.

In the Town of Swansea, water-surface elevations of floods of the selected recurrence intervals were computed using the USACE HEC-2 step-backwater computer program (Reference 48). Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals. Starting water-surface elevations for Rocky Run were determined using the mean spring high-tide elevation.

Cross-section data for the backwater analyses for the Segreganset River in the City of Taunton were obtained from topographic maps compiled from aerial photographs at a scale of 1:4,800 with a contour interval of 4 feet (Reference 63). Water-surface elevations of floods of the selected recurrence intervals for the Three Mile River, the Three Mile River - West Channel, and the Mill River were computed using the USGS E431 computer program (Reference 52). Water-surface elevations for the Segreganset River and Cobb Brook were computed using the COE HEC-2 step-backwater computer program (Reference 61). Flood profiles were drawn showing computed water-surface elevations for floods of the selected recurrence intervals. Starting water-surface elevations for the Taunton River were determined using the step-backwater method utilizing the USGS E431 computer program (Reference 52). Starting water-surface elevations for the Three Mile River, the Mill River, and Cobb Brook were determined

from elevations of the Taunton River. Starting water-surface elevations for the Three Mile River - West Channel were based on the principles of divided flow. Starting water-surface elevations for the Segreganset River were determined using the slope/area method.

The computed profile for the 1-percent-annual-chance flood in the vicinity of the Three Mile River gage in the City of Taunton was used to check the high-water elevations of the flooding of 1968. The flood profiles for the Taunton River have been compared to the profile of the March 1968 flood. Computations on the Taunton River at the site of the Church Street bridge were based on construction plans for the new bridge to be built. Because riverine flooding is negligible compared to tidal flooding, no hydraulic analysis was performed on the Taunton River from the downstream corporate limits to approximately 1,000 feet upstream of the confluence with Dam Lot Brook. Reservoir routing with the HEC-1 rainfall-runoff computer model was used to establish the 10-, 2-, 1-, and 0.2-percent- annual-chance flood elevations for Lake Sabbatia, Watson Pond, and Mill Pond (Reference 64). The model was calibrated to high-water marks for the 1968 flood, which is the flood of record in this basin. The analysis of flood elevations at Watson Pond takes into consideration the constricted culvert at Bay Street, which is the exit to Lake Sabbatia.

Roughness factors (Manning’s “n” values) used in the hydraulic computations were determined from field observations, guided by U.S. Geological Water Supply Publications (References 71 and 72). Table 7, “Manning’s “n” values” shows the channel and overbank “n” values for the streams studied by detailed methods.

TABLE 7 – MANNING’S “n” VALUES

<u>Flooding Source</u>	<u>Channel “n”</u>	<u>Overbanks</u>
Abbott Run	0.03-0.04	0.06-0.09
Acushnet River	0.035-0.050	0.060-0.100
Armstrong Brook	0.03-0.04	0.06-0.09
Assonet River	0.03-0.06	0.05-0.10
Attleboro Industrial Stream	0.03-0.04	0.06-0.09
Black Brook	0.035-0.080	0.035-0.100
Bungay River	0.03-0.04	0.06-0.09
Buttonwood Brook	0.035	0.100
Buttonwood Brook East	0.035	0.100
Chartley Brook	0.03-0.04	0.06-0.09
Cobb Brook	0.025-0.070	0.030-0.100
Coles Brook	0.035-0.05	0.05-0.11
Dam Lot Brook	0.035-0.040	0.035-0.060
Deep Brook	0.030-0.035	0.060-0.080
East Junction Stream	0.03-0.04	0.06-0.09
Elmwood Street Brook	0.03-0.04	0.06-0.09
Fall Brook	0.03-0.06	0.05-0.10
Forge River	0.025-0.040	0.030-0.070
Goose Branch Brook	0.040-0.100	0.050-0.100
Gowards Brook	0.035-0.065	0.080-0.130
Lake Como Stream	0.03-0.04	0.06-0.09

TABLE 7 – MANNING’S “n” VALUES - continued

<u>Flooding Source</u>	<u>Channel “n”</u>	<u>Overbanks</u>
Landry Avenue Brook	0.03-0.04	0.06-0.09
Mary Kennedy Brook	0.03-0.04	0.06-0.09
Mason Park Brook	0.03-0.04	0.06-0.09
Mill River	0.030-0.065	0.060-0.300
Mulberry Brook	0.040-0.060	0.040-0.060
Oak Hill Stream	0.035-0.05	0.05-0.11
Paskamanset River	0.035	0.100
Poquanticut Brook	0.040-0.060	0.040-0.060
Queset Brook	0.030-0.060	0.035-0.140
Rattlesnake Brook (Freetown)	0.03-0.06	0.05-0.10
Rattlesnake Brook (North Attleborough)	0.03-0.04	0.06-0.09
Rocklawn Avenue Stream	0.03-0.04	0.06-0.09
Rocky Run	0.035-0.045	0.050-0.120
Rumford River	0.018-0.150	0.020-0.120
Runnins River	0.035-0.05	0.05-0.11
Scotts Brook	0.03-0.04	0.06-0.09
Segreganset River (Dighton)	0.03-0.06	0.05-0.1
Segreganset River (Taunton)	0.035-0.050	0.040-0.100
Sevenmile River ¹	0.03-0.04	0.04-0.1
Speedway Brook	0.03-0.04	0.06-0.09
Sunken Brook	0.03-0.06	0.05-0.1
Taunton River ¹	0.035-0.055	0.08-0.10
Ten Mile River ¹	0.025-0.055	0.03-0.085
Three Mile River (Dighton)	0.03-0.06	0.05-0.1
Three Mile River (Taunton)	0.030-0.065	0.060-0.300
Three Mile River – West Channel	0.030-0.065	0.060-0.300
Tributary to Dam Lot Brook	0.035-0.045	0.035-0.060
Tributary to Forge River	0.035-0.060	0.040-0.060
Wading River ¹	0.02-0.05	0.06-0.12
Whiting Pond Bypass ¹	0.036-0.060	0.09-0.11
Whitman Brook	0.012-0.045	0.050-0.140

¹July 16, 2015 study

Countywide Analyses

For the July 7, 2009 and the June 16, 2014 countywide revisions, no new hydraulic analyses were conducted.

For the July 16, 2015 countywide analysis, hydrologic analyses were conducted for the Taunton, Ten Mile, Sevenmile, and Wading Rivers and Whiting Pond Bypass. The U.S. Army Corps of Engineers computer programs HEC-RAS 4.1.0 and HEC-GeoRAS 10.0

for ArcGIS 10.1 (References 65 and 66) were used to model stream profiles with 10-, 2-, 1-, and 0.2-percent annual exceedance probabilities for the four rivers. Cross-section data and structure elevations for these rivers were obtained from field surveys in March and April of 2012 and from cross sections obtained from prior FIS models (References 67 and 68).

Field data collected by USGS staff for these four models include elevation data and underwater depths (referenced to the elevation of the water surface at the time of the survey) collected with a total station theodolite. Underwater and channel bank field survey data were merged with LiDAR data describing the elevations of the overbanks. LiDAR data were collected and processed by Photo Science Inc., under contract with the USGS. LiDAR was collected in the winter and spring of 2011 and processed and published in 2012. It was collected to a vertical accuracy of 30 cm with a 95% confidence interval.

Sevenmile River

Underwater points for selected cross sections were obtained from SCS WSP2 (TR-61) input files used in the countywide FIS for Bristol County, MA (Reference 67) and adapted for use here.

The starting water-surface elevations for the 10-, 2-, 1-, and 0.2-percent annual exceedance probability flow profiles near the confluence with the Ten Mile River were estimated from normal depth slope calculations. Normal depth slope was set at 0.0009 based on the slope at the lower end of the surveyed reach of the Sevenmile River.

The hydraulic model for the Sevenmile River was calibrated for the March and April 2010 event using high-water marks (HWMs) documented at Read Street in the City of Attleboro by Zarriello and Bent (Reference 69).

Taunton River

Underwater points for cross sections in the towns of Bridgewater and Middleborough were obtained from survey data collected by Sverdrup & Parcel and Associates, Inc. in 1977-78. These models were adapted for use in the subsequent countywide FISs for Plymouth and Bristol counties (References 67 and 68) and then adapted for use here.

The starting water-surface elevations for the 10-, 2-, 1-, and 0.2-percent annual exceedance probability flow profiles downstream from the Plain Street bridge in the City of Taunton were estimated from normal depth slope calculations. Normal depth slope was set at 0.00033 based on the slope at the lower end of the surveyed reach of the Taunton River, resulting in starting water surfaces greater than 4 feet NAVD for all modeled profiles in the vicinity of the Plain Street bridge. Although the Taunton River becomes tidal at the downstream end, the Mean High Water from the Tidal Flood Profile is between 3-4 feet NAVD in this location (Reference 70) and thus the riverine flooding was expected to control the downstream water surface elevations as opposed to tidal flooding.

Although mean high tides are not expected to control riverine floods with from 10- to 0.2-percent AEPs, tidal floods with from 10- to 0.2-percent AEPs will exceed riverine floods with from 10- to 0.2-percent AEPs at the downstream end of the Taunton River (up through the City of Taunton). Flood profiles and flood mapping are drawn to selected AEPs without regard to whether the flood is tidal or riverine. For the 10% AEP, the tidal

flood exceeds the riverine flood up to the County Street (Route 140) bridge in the City of Taunton. For the 1% and 0.2% AEP the tidal flood exceeds the riverine flood up to the Taunton/Raynham town line.

Ten Mile River

The starting water-surface elevations for the 10-, 2-, 1-, and 0.2-percent annual exceedance probability flow profiles at the confluence with the Seekonk River were estimated from normal depth slope calculations. Normal depth slope was set at 0.0005 based on the slope at the lower end of the reach of the Ten Mile River. The downstream reach was set to a water surface elevation of 15.0 feet due to backwater from the Seekonk River for mapping purposes. The model was calibrated with high water marks for the Ten Mile River collected during the April 2010 flood event (Reference 69).

Whiting Pond Bypass

The starting water-surface elevations for the 10-, 2-, 1-, and 0.2-percent annual exceedance probability flow profiles at the confluence with the Ten Mile River were calculated by HEC-RAS as a part of its split-flow routine.

Wading River

The starting water-surface elevations for the 10-, 2-, 1-, and 0.2-percent annual exceedance probability flow profiles downstream of Taunton Avenue (Route 140) in the Town of Norton were estimated from normal depth slope calculations. Normal depth slope was set at 0.0005 based on the slope at the lower end of the surveyed reach of the Wading River.

The hydraulic model for the Wading River was calibrated for the March and April 2010 event using the flood flow calculated at the streamgage for that event and high-water marks (HWMs) documented throughout the reach (Reference 69).

3.3 Coastal Analysis

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, Bristol County is susceptible to flooding from both hurricanes and northeasters.

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

Areas of coastline subject to significant wave attack are referred to as coastal high hazard zones. The USACE has established the 3-foot breaking wave as the criterion for identifying the limit of coastal high hazard zones (Reference 73). The 3-foot wave has been determined as the minimum size wave capable of causing major damage to conventional wood frame or brick veneer structures. Wave height analyses were performed in the coastal communities of Bristol County to determine wave heights and

corresponding wave crest elevations for the areas inundated by the tidal flooding, and wave runup analyses were performed to determine the height and extent of runup beyond the limit of tidal inundation. The results of these analyses were combined into wave envelopes, which were constructed by extending the maximum wave runup elevation seaward to its intersection with the wave crest profile.

Precountywide Analysis

Prior to the countywide updates, coastal hydrologic and hydraulic analyses were carried out to estimate the 1-percent-annual-chance storm characteristics in the City of Fall River and the Towns of Somerset and Swansea and areas of the Town of Fairhaven and the City of New Bedford behind the hurricane barrier. As part of the July 7, 2009 countywide study, new coastal analyses were performed for the City of New Bedford and the Towns of Dartmouth, Fairhaven, and Westport. A description of the revised analyses is presented in the subsequent July 7, 2009 Countywide Analysis section. Portions of the coastal analyses described in this section performed for the City of Fall River and the Towns of Somerset and Swansea have been superseded by the June 16, 2014 revision. A description of the revised analyses is presented in the subsequent June 16, 2014 Countywide Analysis section.

In 1964, the New England Division USACE conducted a study to determine the flood frequency potential for waters adjacent to the Taunton River in the vicinity of the City of Fall River and the Town of Somerset (Reference 45). This report, although initially conducted to determine the cost-benefit ratio of constructing hurricane barriers in Narragansett Bay, indicated the frequency of tidal flooding caused by hurricanes and high intensity storms. Many times a storm of relatively minor proportions will linger over the area for a substantial period of time and will cause excessive buildup of tidal levels throughout the area. Historical data from previous hurricanes and storms were investigated, and the results were plotted on log-probability paper. Most of the information was obtained from the USACE reports in the area (References 45, 46, 54 and 55). The lower portions of the Three Mile River, the Mill River, and Cobb Brook are under the influence of tidal flooding from the Taunton River. Rainfall data used for the rainfall-runoff model simulations of Mill Pond, Lake Sabbatia, and Watson Pond were taken from the U.S. Weather Bureau's Technical Paper No. 40 (Reference 30). The stillwater elevations for the 10-, 2-, 1- and 0.2-percent-annual-chance floods have been determined for the Taunton River, the Three Mile River, the Mill River, Cobb Brook, Mill Pond, Lake Sabbatia, and Watson Pond.

In the Town of Fairhaven and the City of New Bedford, studies were performed to determine ponding levels behind the hurricane barrier when closed during periods of abnormally high tides. According to an operations summary provided by the USACE, the longest period of closure since operation began in 1966 has been four hours. Storms of equal duration were selected to compute runoff volumes. Four-hour rainfalls were obtained for the 10-, 2-, 1- and 0.2-percent-annual-chance storms from the Rainfall Frequency Atlas of the United States (Reference 22). It was assumed that the watershed areas that would contribute runoff to ponding behind the hurricane barrier during the four-hour closure period would include the entire 11-square mile Acushnet River watershed downstream of Saw Mill Dam and an additional area upstream of the dam (Reference 14).

The upstream drainage area was estimated by assuming that the distance which runoff travels in unit time is equal to the longest length of travel divided by the time of concentration (Reference 74). The time of concentration can be defined as "the travel time of water from the hydraulically-most distant point of a drainage basin to the point of interest in hours" (References 75 and 76). The Acushnet River basin characteristics, upstream of Saw Mill Dam, were obtained from the USACE (Reference 14). The results indicated that runoff from watersheds adjacent to an approximately 4,000-foot reach of the Acushnet River, upstream of Saw Mill Dam, would contribute to ponding behind the barrier during the four-hour period. The drainage area for the contributory reach of the main channel and its tributaries was delineated on topographic maps and calculated to be approximately 1.5 square miles (Reference 77). Runoff volumes were calculated by dividing the 1.5-square mile contributory drainage basin into three sub-basins: urban, suburban, and water surface. Approximate runoff coefficients, C, were assigned to each sub-basin, and the volume, V, in acre-feet (volume of water in a 1-acre area at a depth of 1 foot), was calculated using the following formula:

$$V = R (C_1A_1 + C_2A_2 + C_3A_3)/12$$

where R equals rainfall in inches for the selected recurrence intervals and A the sub-basin in acres.

Resultant flood levels behind the hurricane barrier in the Town of Fairhaven and the City of New Bedford were calculated from a USACE stage-capacity curve, assuming an average closure elevation of 4 feet, based on 12 years (January 23, 1966 to February 7, 1978) of historic records for barrier operations (Reference 14). If heavy runoff occurred or was anticipated from heavy rainfall that had previously occurred, the gates at the barrier would be closed when the ocean tide reached 2 feet (Reference 14). At an initial pond elevation of 4 feet, approximately 4,900 acre-feet of water are stored behind the closed barrier. The analyses of the storm surge elevations for the 10-, 2-, 1- and 0.2-percent-annual-chance floods for coastal waters reflect the stillwater elevations due to tidal and wind set-up effects.

Tidal stage-frequency relationships were determined for the coastal communities of the City of Fall River and the Towns of Somerset and Swansea using flood level profiles developed by statistically analyzing high-water elevations in the study area (References 14 and 78). Information evaluated at the City of Fall River and the Towns of Somerset and Swansea consisted of a 33-year (1931-1963) systematic record and several extreme historic events representing both a 149-year (1815-1863) and a 329-year (1635-1963) period of record. In the Town of Swansea, the Cole River below Milford Pond Dam, the Lee River, and Mount Hope Bay were evaluated. The two greatest storms in 30 years of record were the hurricanes of 1938 and 1954. The incorporation of the historic events improves the frequency distribution by extending the record of greatest events and includes actual community experience. The coastal surge evaluation was based primarily on tide stage-frequency curves and flood level profiles developed for the study area. The data were plotted on probability paper using the following formula:

$$P = 100(M-0.5)/y$$

where P equals the percent chance of occurrence in any one year, M the number of the event ranked in order of decreasing magnitude, and y the number of years of record (Reference 45). The "Design Basis Hurricane," a hypothetical worst-case storm with an

assigned recurrence interval of 0.2-percent-annual-chance was also used as a plotting point. The tide stage for the floods of the selected recurrence intervals were read from the curve drawn to best fit the data. A similar study was also performed by the USACE for the Palmer and Barrington Rivers, which are tributaries to the Warren River (Reference 79). The analyses reported in this study reflect the stillwater elevations due to tidal and wind setup effects.

A summary of significant data for hurricanes and severe storms in the Towns of Somerset and Swansea and the City of Fall River is shown in Table 8. Data is based on tide stage data at Newport, Rhode Island, as related to the City of New Bedford.

TABLE 8 – STAGE-FREQUENCY DATA

<u>Hurricane or Storm</u>	<u>Date</u>	<u>Elevation (NAVD¹)</u>
Hurricane	August 3, 1638	15.8
Hurricane	August 15, 1635	14.9
Hurricane	September 21, 1938	12.9
Hurricane	September 23, 1815	12.2
Hurricane	September 14, 1944	8.6
Hurricane	September 21, 1961	5.3
Hurricane Carol	August 31, 1954	12.6
Hurricane Donna	September 12, 1960	6.5
Storm	November 30, 1963	7.2
Storm	November 30, 1944	6.7
Storm	November 7, 1962	6.2
Storm	March 7, 1962	6.1
Storm	March 3, 1947	6.0
Storm	February 19, 1960	5.9
Storm	March 3, 1942	5.7
Storm	November 12, 1947	5.7
Storm	February 14, 1960	5.7
Storm	February 7, 1951	5.6
Storm	April 3, 1958	5.6
Storm	December 29, 1959	5.6
Storm	January 3, 1960	5.6
Storm	January 27, 1933	5.5
Storm	November 3, 1951	5.5
Storm	January 16, 1961	5.5
Storm	February 15, 1953	5.3
Storm	November 10, 1958	5.3
Storm	November 23, 1961	5.3
Storm	December 2, 1942	5.2
Storm	October 31, 1947	5.2
Storm	October 22, 1949	5.2
Storm	October 23, 1953	5.2

¹North American Vertical Datum 1988

TABLE 8 – STAGE-FREQUENCY DATA - continued

<u>Hurricane or Storm</u>	<u>Date</u>	<u>Elevation (NAVD¹)</u>
Storm	October 16, 1955	5.2
Storm	December 6, 1962	5.2
Storm	October 1, 1936	5.0
Storm	November 25, 1950	5.0
Storm	April 13, 1953	5.0
Storm	March 20, 1958	5.0
Storm	January 27, 1963	5.0
Storm	November 2, 1963	5.0

¹North American Vertical Datum 1988

The precountywide stillwater elevations have been determined for the 10-, 2-, 1-, and 0.2-percent-annual-chance floods for the flooding sources studied by detailed methods and are summarized in Table 9, “Precountywide Summary of Stillwater Elevations.”

TABLE 9 – PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS

<u>FLOODING SOURCE AND LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>			
	<u>10-PERCENT</u>	<u>2-PERCENT</u>	<u>1-PERCENT</u>	<u>0.2-PERCENT</u>
ACUSHNET RIVER				
At confluence with New Bedford Harbor	4.4	4.7	4.9	15.2
ASSONET RIVER				
In Berkley	8.4	12.2	13.8	17.6
BUZZARDS BAY				
Entire shoreline within Dartmouth	7.4	10.4	11.7	14.7
Entire shoreline within Fairhaven and New Bedford	7.6	10.6	12.0	15.2
COBB BROOK				
At confluence with Taunton River	7.2	11.0	12.6	16.3

¹North American Vertical Datum of 1988

TABLE 9 – PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>			
	<u>10-PERCENT</u>	<u>2-PERCENT</u>	<u>1-PERCENT</u>	<u>0.2-PERCENT</u>
COLE RIVER				
Below Milford Pond Dam	8.4	12.3	13.9	17.6
Above Milford Pond Dam	23.7	*	24.7	*
LAKE SABBATIA				
Entire shoreline	63.2	64.5	65.0	66.3
LEE RIVER				
Entire length within Somerset and Swansea	8.4	12.3	13.9	17.6
MILL POND				
Entire shoreline	60.1	60.6	60.8	61.2
MILL RIVER				
At confluence with Taunton River	6.9	10.8	12.3	16.0
MOUNT HOPE BAY				
Entire length	8.4	12.3	13.9	17.6
RHODE ISLAND SOUND				
Entire shoreline within Westport	7.1	10.1	11.5	14.6
West Branch Westport River	7.1	10.1	11.5	14.6
At the downstream end of the East Branch Westport River	7.1	10.1	11.5	14.6
At the upstream end of the East Branch Westport River	7.5	10.5	11.8	14.8

¹North American Vertical Datum of 1988

*Data not available

TABLE 9 – PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>			
	<u>10-PERCENT</u>	<u>2-PERCENT</u>	<u>1-PERCENT</u>	<u>0.2-PERCENT</u>
RUNNINS RIVER				
In Seekonk	5.8	7.9	9.2	12.2
SWEEDENS SWAMP				
At Attleboro	74.3	75.0	75.4	76.7
TAUNTON RIVER				
In Fall River	8.4	12.3	13.9	17.6
South of Poplar Road	8.4	12.3	13.9	17.6
North of Poplar Road	8.4	12.2	13.8	17.6
At Assonet River	8.4	12.2	13.8	17.6
At Peters Point	8.4	12.2	13.8	17.6
At Berkley Bridge	8.0	11.9	13.5	17.2
At confluence of Three Mile River	7.7	11.5	13.1	16.9
At Berkley-Taunton Line	7.3	11.1	12.7	16.5
At confluence of Mill River	6.9	10.8	12.3	16.0
At the Raynham corporate boundary	8.5	10.6	11.9	15.6
At confluence of Forge River	*	10.2	11.8	15.5
THREE MILE RIVER				
At Dam Number 3	7.5	11.2	12.8	16.6
At Old Somerset Avenue	7.7	11.5	13.0	16.6

¹North American Vertical Datum of 1988

*Data not available

TABLE 9 – PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>			
	<u>10-PERCENT</u>	<u>2-PERCENT</u>	<u>1-PERCENT</u>	<u>0.2-PERCENT</u>
WARREN RESERVOIR				
Entire shoreline within Town of Swansea	4.4	6.3	9.5	12.4
WATSON POND				
Entire shoreline	62.6	63.7	64.0	64.8
WEST CHANNEL THREE MILE RIVER				
Above downstream confluence with Three Mile River	7.5	11.2	12.8	16.6
WINNECUNNET POND				
Entire shoreline	71.0	72.8	73.4	75.0

¹North American Vertical Datum of 1988

The methodology for analyzing wave heights and corresponding wave crest elevations was developed by the National Academy of Sciences (Reference 80). The NAS methodology is based on three major concepts.

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

$$H_b = 0.78d$$

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

$$Z_w = S^* + 0.7H^* = S^* + 0.55d$$

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

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

$$H_t = BH_i$$

where H_t is the transmitted wave height, H_i is the incident wave height, and B is a transmission coefficient ranging from 0.0 to 1.0. The coefficient is a function of the physical characteristics of the obstruction. Equations have been developed by NAS to determine B for vegetation, buildings, natural barriers such as dunes, and man-made barriers such as breakwaters and seawalls (Reference 80).

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

Hydraulic analyses of the shoreline characteristics of the flooding sources studied in detail were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along the shoreline.

The methodology for analyzing wave runup was developed by Stone and Webster Engineering Corporation (Reference 81). The wave runup computer program (based on earlier work done by the USACE) operates using an ensemble of deepwater wave heights, H_i , the surge stillwater elevation, a wave period, T_s , and beach slope, m .

Wave heights were computed along transects which were located perpendicular to the average mean 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.

Along each transect, wave heights, wave crest elevations, and wave runup were computed considering the combined effects of changes in ground elevation, vegetation, and physical features. The calculations were carried inland along the transect until the wave crest elevation was permanently less than 0.5 foot above the stillwater surge elevation or until the coastal flooding met another flooding source (i.e. riverine) with an equal water-surface elevation. The results of the calculations are accurate until local topography, vegetation, or cultural development within the community undergoes any major changes.

For each transect, the program produced a maximum wave runup elevation which defines the inland extent of flooding. Between transects, runup elevations were interpolated to give the area extent of flooding. Wave crest profiles are constructed for each transect by extending the maximum wave runup elevation seaward to its intersection with the wave profile determined by the NAS wave height analyses (References 80 and 82).

July 7, 2009 Countywide Analysis

As part of the July 7, 2009 countywide update, revised coastal analyses were performed for the open water flooding sources in the City of New Bedford and the Towns of Dartmouth, Fairhaven, and Westport. Provided below is a summary of the analyses performed. All revised coastal analyses were performed in accordance with Appendix D "Guidelines for Coastal Flooding Analyses and Mapping," (Reference 83) of the

Guidelines and Specifications as well as the “Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update” (Reference 84).

For the revised communities, published values in the Tidal Flood Survey (Reference 70) were used to estimate the stillwater elevations for the 10-, 2-, and 1-percent-annual-chance floods for Buzzards Bay and Rhode Island Sound. The 0.2-percent-annual-chance stillwater elevations for the revised flooding sources were extrapolated based on the more frequent stillwater elevations in the Tidal Flood Survey. Stillwater elevations for the revised flooding sources are presented in Table 10.

TABLE 10 – SUMMARY OF JULY 7, 2009 COUNTYWIDE ANALYSIS STILLWATER ELEVATIONS

<u>FLOODING SOURCE AND LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>			
	<u>10-PERCENT</u>	<u>2-PERCENT</u>	<u>1-PERCENT</u>	<u>0.2-PERCENT</u>
BUZZARDS BAY				
Nasketucket Bay	6.8	10.4	12.2	15.8
West Island	6.7	10.2	12.0	15.7
Harbor View/Pope Beach	6.6	10.1	11.9	15.5
Acushnet River	4.4	4.7	5.5 ²	15.2
New Bedford Harbor	6.4	10.0	11.7	15.4
Fort Rochman/Clark Point	6.3	9.8	11.7	15.3
Clark Cove	6.2	9.7	11.7	15.2
Round Hill Point/Apponaug Bay	6.0	9.6	11.7	15.2
Little River/Mishaum Point	5.9	9.5	11.7	15.3
Barney’s Point	5.8	9.5	11.7	15.3
Little Beach	5.8	9.6	11.7	15.5
RHODE ISLAND SOUND				
East Horseneck Beach	5.7	9.6	11.7	15.5
Horseneck Beach	5.7	9.6	11.7	15.5

¹North American Vertical Datum of 1988

²Computed from the City of New Bedford and Town of Fairhaven, May 2011 Hurricane Dike and Barrier System Accreditation Package

TABLE 10 – SUMMARY OF JULY 7, 2009 COUNTYWIDE ANALYSIS STILLWATER
ELEVATIONS - continued

<u>FLOODING SOURCE AND LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>			
	<u>10-PERCENT</u>	<u>2-PERCENT</u>	<u>1-PERCENT</u>	<u>0.2-PERCENT</u>
RHODE ISLAND SOUND - continued				
Upstream end of East Branch Westport River	5.7	9.6	11.7	15.5
Westport Harbor	5.7	9.6	11.7	15.5
Richmond Pond	5.7	9.6	11.7	15.5

¹North American Vertical Datum of 1988

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 +0.15 foot was applied to the data in the Tidal Flood Survey.

Wave setup along the open coast areas of the City of New Bedford and the Towns of Dartmouth, Fairhaven, and Westport was calculated using the procedures detailed in the “Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update”, (Reference 84). Specifically, the Direct Integration Method (DIM) was applied. Because much of the New England 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 coastal portions of Bristol County, offshore wave characteristics representing a 1-percent-annual-chance storm were determined using data from the Wave Information Study (WIS). A Peaks-Over-Threshold statistical analysis was applied on 20 years (1980-1999) of wave characteristic data from WIS Station No. 53. Mean wave characteristics were determined as specified in the FEMA guidance for V Zone mapping.

Wave heights and wave runup in the City of New Bedford and the Towns of Dartmouth, Fairhaven, and Westport 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 computer wave heights varied significantly between adjacent transects.

Transect descriptions for the July 7, 2009 countywide analysis are shown in Table 11. The locations of these transects are depicted in Figure 12.

TABLE 11 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DESCRIPTIONS

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>	
		<u>1-PERCENT- ANNUAL-CHANCE STILLWATER</u>	<u>MAXIMUM 1- PERCENT- ANNUAL-CHANCE WAVE CREST²</u>
1	The transect is located at a point approximately 300 feet southeast of the western end of Shaws Cove Road, extending to the northwest towards Shaw Road.	12.2	18.44
2	The transect is located at the mouth of the Nasketucket River, extending to the northwest from Camp Echo towards U.S. Route 6 (Huttelston Avenue).	12.2	18.65
3	The transect is located along the Sconticut Neck shoreline at the eastern extent of Ocean Avenue, extending to the west towards Sconticut Neck Road.	12.2	19.22
4	The transect is located along the eastern shoreline of Sconticut Neck at a point approximately 1,500 feet south of Wapatma Lane, extending to the west towards Sconticut Neck Road.	12.2	19.3
5	The transect is located at the Nasketucket Bay shoreline extending east along Bluepoint Road to the intersection with Fir Street.	12.2	16.83
6	The transect is located along the Buzzards Bay shoreline at a point approximately 380 feet southwest of the intersection of Causeway Road and Alder Street, extending to the northeast towards Almond Street.	12	18.54

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 11 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DESCRIPTIONS - continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>	
		<u>1-PERCENT-ANNUAL-CHANCE STILLWATER</u>	<u>MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST²</u>
7	The transect is located along the West Island shoreline, extending to the northeast along Gull Island Road towards Fir Street.	12	23.08
8	The transect is located along the southeast shoreline of West Island at a point approximately 975 feet north of Rocky Point, extending to the northwest towards Fir Street.	12	23.38
9	The transect is located along the eastern shoreline of Scoticut Neck at a point approximately 250 feet south of Island View Road extending to the northwest towards Scoticut Neck Road.	12	17.91
10	The transect is located along the southern end of Scoticut Neck Road extending north to the intersection with Manomet Street.	12	23.08
11	The transect is located at the western end of Potter Street along the Buzzards Bay shoreline, extending to the northeast towards Scoticut Neck Road.	12	23.23
12	The transect is located along Chambers Street extending from the Buzzards Bay shoreline east to Scoticut Neck Road.	11.9	23.38
13	The transect is located along the Buzzards Bay shoreline extending north along Manhattan Avenue from the shoreline of Buzzards Bay north to the intersection with Grove Street.	11.9	18.67

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 11 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DESCRIPTIONS - continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>	
		<u>1-PERCENT- ANNUAL-CHANCE STILLWATER</u>	<u>MAXIMUM 1- PERCENT- ANNUAL-CHANCE WAVE CREST²</u>
14	The transect is located at Fort Phoenix Beach State Reservation extending from the shoreline of Buzzards Bay north to the intersection of Phoenix Street and Laurel Street.	11.9	19.08
15	The transect is located along the Buzzards Bay shoreline and extends west along Apponagansett Street towards Brock Avenue.	11.7	17.65
16	The transect is located along the Buzzards Bay shoreline approximately 200 feet southeast of Hudson Street extending northwest towards Brock Avenue.	11.7	23.54
17	The transect is located along the Buzzards Bay shoreline approximately 1,200 feet south of the intersection of South Rodney French Boulevard and Brock Avenue, extending north towards South Rodney French Boulevard.	11.7	26.1
18	The transect is located along the Clarks Cove shoreline at the west end of Lucas Street, extending northeast towards Brock Cove.	11.7	17.48
19	The transect is located along the Clarks Cove shoreline at a point approximately 225 feet east of the intersection of Osborn Street and Padanaram Avenue.	11.7	19.19
20	The transect is located along the Clarks Cove shoreline at a point approximately 630 feet east of the intersection of Flagship Drive and Spinnaker Lane, extending west towards Dartmouth Street.	11.7	19.96

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 11 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DESCRIPTIONS - continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>	
		<u>1-PERCENT- ANNUAL-CHANCE STILLWATER</u>	<u>MAXIMUM 1- PERCENT- ANNUAL-CHANCE WAVE CREST²</u>
21	The transect is located along the Clarks Cove shoreline at a point approximately 175 feet east of the intersection of Mosher Street and Clarks Cove Drive, extending west towards Prospect Street.	11.7	23.84
22	The transect is located along the Buzzards Bay shoreline at a point approximately 525 feet southeast of the intersection of Stone Ledge Road and William Street.	11.7	23.54
23	The transect is located along the Buzzards Bay shoreline at the southern end of Rockland Farm Road, extending northwest towards Dartmouth Street.	11.7	24.14
24	The transect is located at the Buzzards Bay shoreline at a point approximately 1,800 feet north of Ricketsons Point extending northeast towards Prospect Street.	11.7	18.92
25	The transect is located along the Buzzards Bay shoreline at a point approximately 885 feet south of the Padanaram Bridge extending northwest across Apponagansett Bay to the southern end of Star of the Sea Drive.	11.7	23.08
26	The transect is located at the eastern end of Bayview Avenue extending west towards Smith Neck Road.	11.7	23.38
27	The transect is located at the east end of Pokanoket Lane extending west towards Smith Neck Road.	11.7	24.14

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 11 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DESCRIPTIONS – continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>	
		<u>1-PERCENT-ANNUAL-CHANCE STILLWATER</u>	<u>MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST²</u>
28	The transect is located at the southeastern end of Mattarest Lane extending west towards Smith Neck Road.	11.7	23.69
29	The transect is located along the Buzzards Bay shoreline at a point approximately 2,400 feet east of Round Hill Road, extending north towards Hetty Green Drive.	11.7	23.54
30	The transect is located along the Buzzards Bay shoreline at a point approximately 270 feet southwest of Ray Peck Drive, extending northwest towards Smith Neck Road.	11.7	22.93
31	The transect is located along the Buzzards Bay shoreline at a point approximately 390 feet northeast from the intersection of Naushon Avenue and Gosnold Avenue, extending northwest towards Naushon Avenue.	11.7	23.69
32	The transect is located along the Buzzards Bay shoreline at a point approximately 500 feet west from Naushon Avenue extending north towards Mishaum Point Road.	11.7	22.33
33	The transect is located at the southern end of Mishaum Point extending north along Mishaum Point Road.	11.7	25.35
34	The transect is located along the Buzzards Bay shoreline at a point approximately 1,600 feet south of Little River Road, extending to the northeast towards Little River Road.	11.7	23.38

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 11 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DESCRIPTIONS - continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>	
		<u>1-PERCENT-ANNUAL-CHANCE STILLWATER</u>	<u>MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST²</u>
35	The transect is located along the Buzzards Bay shoreline approximately 850 feet southwest of Little River Road extending north towards Potomska Road.	11.7	22.33
36	The transect is located along the Buzzards Bay shoreline at a point approximately 1,700 feet north of Demarest Lloyd Memorial State Park, extending northwest across Giles Creek and towards Great Neck.	11.7	21.87
37	The transect is located along the Buzzards Bay shoreline at a point approximately 750 feet south of Demarest Lloyd State Park Road and extending to the northwest towards Barney's Joy Road.	11.7	22.17
38	The transect is located along the Buzzards Bay shoreline at a point approximately 2,400 feet south of Barney's Joy Road extending to the northwest.	11.7	22.78
39	The transect is located along the Buzzards Bay shoreline at a point approximately 1,800 feet west of Barney's Joy Point extending to the north towards Jordan Road.	11.7	23.08
40	The transect is located along the Buzzards Bay shoreline at a point approximately 2,900 feet east of Horseneck Road extending to the north towards Division Road.	11.7	22.33

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 11 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DESCRIPTIONS - continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>	
		<u>1-PERCENT-ANNUAL-CHANCE STILLWATER</u>	<u>MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST²</u>
41	The transect is located along the Buzzards Bay shoreline at a point approximately 300 feet west of the Town of Dartmouth corporate limits, extending north towards Third Street.	11.7	23.54
42	The transect is located along the Buzzards Bay shoreline at a point approximately 2,759 feet west of the Town of Dartmouth corporate limits, extending north across the Let and towards Taber Point.	11.7	23.54
43	The transect is located along the Buzzards Bay shoreline at a point approximately 500 feet south of the intersection of East Beach Road and Grove Lane.	11.7	23.38
44	The transect is located along the Buzzards Bay shoreline at a point approximately 1,000 feet south of the Horseneck Beach State Park access road; extending northeast across the Westport River East Branch towards the south end of Lower Way.	11.7	23.54
45	The transect is located along the Buzzards Bay shoreline, extending north along Bridge Street towards Cherry & Webb Lane.	11.7	23.38
46	The transect is located along the East Branch shoreline extending northwest along Cadman's Neck Road.	11.7	16.25

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 11 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DESCRIPTIONS – continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>	
		<u>1-PERCENT-ANNUAL-CHANCE STILLWATER</u>	<u>MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST²</u>
47	The transect is located along the East Branch shoreline at a point approximately 3,000 feet west of Horseneck Road extending east along Pettey Lane.	11.7	14.76
48	The transect is located along the west shoreline of East Branch at a point approximately 1,100 feet east of Olin Howard Way, extending northwest towards Drift Road.	11.7	15.98
49	The transect is located at Toms Point extending north along Judge's Way towards Cornell Road.	11.7	17.13
50	The transect is located along the western shoreline of the West Branch, extending west along Palmer Lane towards River Road.	11.7	16.12
51	The transect is located along the Buzzards Bay shoreline at a point approximately 600 feet east of Acoaxet Street, extending northwest towards River Road.	11.7	23.54
52	The transect is located along the Buzzards Bay shoreline at a point approximately 1,400 feet east of Lakeside Avenue, extending north towards Cross Road.	11.7	23.99
53	The transect is located along the Buzzards Bay shoreline at a point approximately 500 feet south of Atlantic Avenue, extending north along Hillside Road towards Cross Road.	11.7	25.8

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 11 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DESCRIPTIONS - continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>	
		<u>1-PERCENT- ANNUAL-CHANCE STILLWATER</u>	<u>MAXIMUM 1- PERCENT- ANNUAL-CHANCE WAVE CREST²</u>
54	The transect is located along the Buzzard's Bay shoreline at a point approximately 1,250 feet west of Howland Avenue, extending north towards Brayton Point Road.	11.7	22.33
55	The transect is located along the Buzzards Bay shoreline extending north along Brayton Point Road toward Ellsworth Drive.	11.7	23.38

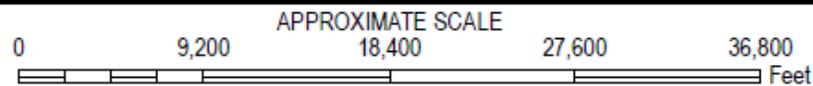
¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.



FIGURE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY
BRISTOL COUNTY, MA
 (ALL JURISDICTIONS)



TRANSECT LOCATION MAP

For the July 7, 2009 revised study, coastal transect data was extracted from topographic data collected by Sanborn Map Company, Inc. This data was collected within the restudy area by Light Detection and Ranging (LiDAR) technology. Additionally, portions of nineteen (19) coastal transects were land surveyed by Green International Affiliates, Inc. (GIA) to supplement the LiDAR data. As appropriate, coastal protection structure details and 0.0 foot NAVD elevation were included and noted in the transect land surveys performed by GIA. Bathymetric data from NOAA Nautical Charts were used to extend the transects offshore. Coastal processes that may affect the transect profile, such as dune erosion and seawall scour and failure, were estimated following the FEMA Guidelines.

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 aerial extent of flooding. The results of the calculations are accurate until local topography, vegetation, or cultural development within the community undergoes major changes.

Wave height and runup calculations used in the revised coastal analysis follow the methodologies described in the FEMA guidance for V Zone mapping (Reference 85). WHAFIS 3.0 was used to predict wave heights.

The FEMA Guidelines (Reference 83) allow for the following methods to be used to determine wave runup: RUNUP 2.0; “Technical Advisory Committee for Water Retaining Structures” (Reference 85); Automated Coastal Engineering System (ACES); and the Shore Protection Manual (Reference 86). 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.

These methodologies were used to compute wave envelope elevations associated with the 1-percent-annual-chance storm surge in the City of New Bedford and the Towns of Dartmouth, Fairhaven, and Westport. Accurate topographic, land-use, and land cover data are required for the coastal analyses. LiDAR data which meets the accuracy standards for flood hazard mapping were used for the topographic data (Reference 87). Depths below mean low water were determined from National Ocean Survey Coastal Charts (Reference 88). The land-use and land cover data were obtained by field surveys and aerial photographs (Reference 89).

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 2 feet.

Table 12 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 12 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DATA

<u>FLOODING SOURCE</u>	<u>STILLWATER ELEVATIONS (feet NAVD¹)</u>				<u>ZONE</u>	<u>BASE FLOOD ELEVATION (feet NAVD¹)</u>
	<u>10- PERCENT- ANNUAL- CHANCE</u>	<u>2- PERCENT- ANNUAL- CHANCE</u>	<u>1- PERCENT- ANNUAL- CHANCE</u>	<u>0.2- PERCENT- ANNUAL- CHANCE</u>		
BUZZARDS BAY						
1	6.8	10.4	12.2	15.8	VE AE	15-18 13-15
2	6.8	10.4	12.2	15.8	VE AE	15-19 13-15
3	6.8	10.4	12.2	15.8	VE AE	15-19 13-15
4	6.8	10.4	12.2	15.8	VE AE	15-19 13-15
5	6.8	10.4	12.2	15.8	VE AE	15-17 13-15
6	6.7	10.2	12.0	15.7	VE AE	15-19 12-15
7	6.7	10.2	12.0	15.7	VE AE	18-23 18
8	6.7	10.2	12.0	15.7	VE AE	18-23 15-17
9	6.7	10.2	12.0	15.7	VE	15-18
10	6.7	10.2	12.0	15.7	VE AE	18-23 16-17
11	6.7	10.2	12.0	15.7	VE	19-23
12	6.6	10.1	11.9	15.5	VE AE	17-23 15-17
13	6.6	10.1	11.9	15.5	VE AE	15-19 13-15
14	6.6	10.1	11.9	15.5	VE AE	19-15 13-15

¹North American Vertical Datum of 1988

TABLE 12 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DATA - continued

<u>FLOODING SOURCE</u>	<u>STILLWATER ELEVATIONS (feet NAVD¹)</u>				<u>ZONE</u>	<u>BASE FLOOD ELEVATION (feet NAVD¹)</u>
	<u>10- PERCENT- ANNUAL- CHANCE</u>	<u>2- PERCENT- ANNUAL- CHANCE</u>	<u>1- PERCENT- ANNUAL- CHANCE</u>	<u>0.2- PERCENT- ANNUAL- CHANCE</u>		
BUZZARDS BAY - continued						
15	6.3	9.8	11.7	15.3	VE AE	15-18 12-14
16	6.3	9.8	11.7	15.3	VE AE	18-24 17
17	6.3	9.8	11.7	15.3	VE AE	20-26 12
18	6.3	9.8	11.7	15.3	VE AE	15-18 12-15
19	6.3	9.8	11.7	15.3	VE AE	15-19 13-15
20	6.2	9.7	11.7	15.2	VE	23
21	6.2	9.7	11.7	15.2	VE AE	18-24 16-18
22	6.2	9.7	11.7	15.2	VE AE	18-24 15-17
23	6.2	9.7	11.7	15.2	VE AE	18-24 18
24	6.0	9.6	11.7	15.2	VE AE	15-19 13-15
25	6.0	9.6	11.7	15.2	VE AE	17-23 12-13
26	6.0	9.6	11.7	15.2	VE AE	17-23 15-17
27	6.0	9.6	11.7	15.2	VE AE	26 16-18

¹North American Vertical Datum of 1988

TABLE 12 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DATA - continued

<u>FLOODING SOURCE</u>	<u>STILLWATER ELEVATIONS (feet NAVD¹)</u>				<u>ZONE</u>	<u>BASE FLOOD ELEVATION (feet NAVD¹)</u>
	<u>10- PERCENT- ANNUAL- CHANCE</u>	<u>2- PERCENT- ANNUAL- CHANCE</u>	<u>1- PERCENT- ANNUAL- CHANCE</u>	<u>0.2- PERCENT- ANNUAL- CHANCE</u>		
BUZZARDS BAY - continued						
28	6.0	9.6	11.7	15.2	VE AE	24 16-18
29	6.0	9.6	11.7	15.2	VE AE	18-24 15-17
30	5.9	9.5	11.7	15.3	VE AE	17-23 15-17
31	5.9	9.5	11.7	15.3	VE AE	18-24 16-18
32	5.9	9.5	11.7	15.3	VE AE	17-22 15-17
33	5.9	9.5	11.7	15.3	VE	25
34	5.9	9.5	11.7	15.3	VE AE	14-23 12-14
35	5.9	9.5	11.7	15.3	VE AE	17-22 12-17
36	5.9	9.5	11.7	15.3	VE AE	16-22 12-16
37	5.9	9.5	11.7	15.3	VE AE	15-22 12-14
38	5.8	9.5	11.7	15.3	VE AE	17-23 15-16
39	5.8	9.5	11.7	15.3	VE AE	18-23 15-17
40	5.8	9.6	11.7	15.5	VE AE	17-22 15-17

¹North American Vertical Datum of 1988

TABLE 12 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DATA - continued

<u>FLOODING SOURCE</u>	<u>STILLWATER ELEVATIONS (feet NAVD¹)</u>				<u>ZONE</u>	<u>BASE FLOOD ELEVATION (feet NAVD¹)</u>
	<u>10- PERCENT- ANNUAL- CHANCE</u>	<u>2- PERCENT- ANNUAL- CHANCE</u>	<u>1- PERCENT- ANNUAL- CHANCE</u>	<u>0.2- PERCENT- ANNUAL- CHANCE</u>		
RHODE ISLAND SOUND						
41	5.7	9.6	11.7	15.5	VE AE	16-24 15-16
42	5.7	9.6	11.7	15.5	VE AE	18-24 15-17
43	5.7	9.6	11.7	15.5	VE AE	14-23 12-14
44	5.7	9.6	11.7	15.5	VE AE	14-24 12-14
45	5.7	9.6	11.7	15.5	VE AE	17-23 12-17
46	5.7	9.6	11.7	15.5	VE	16
47	5.7	9.6	11.7	15.5	VE AE	14-15 12-14
48	5.7	9.6	11.7	15.5	VE AE	15-16 12-14
49	5.7	9.6	11.7	15.5	VE AE	14-17 12-14
50	5.7	9.6	11.7	15.5	VE AE	14-16 12-14
51	5.7	9.6	11.7	15.5	VE AE	18-24 15-17
52	5.7	9.6	11.7	15.5	VE AE	18-24 16-18
53	5.7	9.6	11.7	15.5	VE AE	19-26 17-19

¹North American Vertical Datum of 1988

TABLE 12 – JULY 7, 2009 COUNTYWIDE ANALYSIS TRANSECT DATA - continued

<u>FLOODING SOURCE</u>	<u>STILLWATER ELEVATIONS (feet NAVD¹)</u>				<u>ZONE</u>	<u>BASE FLOOD ELEVATION (feet NAVD¹)</u>
	<u>10- PERCENT- ANNUAL- CHANCE</u>	<u>2- PERCENT- ANNUAL- CHANCE</u>	<u>1- PERCENT- ANNUAL- CHANCE</u>	<u>0.2- PERCENT- ANNUAL- CHANCE</u>		
RHODE ISLAND SOUND - continued						
54	5.7	9.6	11.7	15.5	VE AE	18-24 15-17
55	5.7	9.6	11.7	15.5	VE	21-23

¹North American Vertical Datum of 1988

June 16, 2014 Countywide Analysis

As part of the June 16, 2014 countywide update, revised coastal analyses were performed for the open water flooding sources along Mount Hope Bay in the City of Fall River and the Towns of Somerset and Swansea, as well as along the Taunton River in the City of Fall River and the Towns of Berkley, Dighton, Freetown, and Somerset. Portions of Heath Brook, the Palmer River, and the Runnins River in the Towns of Rehoboth, Seekonk, and Swansea were reviewed to ensure that a tie-in with updated coastal analyses conducted in adjacent counties was made. Additionally, tributaries to the Palmer and Taunton Rivers were reviewed to ensure that the appropriate backwater elevation was depicted. A summary of the analyses performed is provided below. All revised coastal analyses were performed in accordance with Appendix D “Guidelines for Coastal Flooding Analyses and Mapping,” (Reference 83) of the Guidelines and Specifications as well as the “Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update” (Reference 84).

The stillwater elevations for the revised coastal analysis study area, namely the open waters of Mount Hope Bay and the portions of the Cole, Lee, and Taunton Rivers and their tributaries under tidal influences, remain unchanged from the precountywide analysis. Heath Brook and the Palmer River used new stillwater elevations calculated during the revised Bristol County, Rhode Island countywide study (Reference 90). Note that stillwater elevations upstream of Interstate 195 on the Palmer River resulted in the values calculated during the precountywide analysis. The stillwater elevations on the Segreganset were also revised based on values from the precountywide analysis at the Taunton River’s confluence with the Assonet River. These revised stillwater elevations are show in Table 13.

TABLE 13 – SUMMARY OF JUNE 16, 2014 COUNTYWIDE REVISION STILLWATER ELEVATIONS

<u>FLOODING SOURCE AND LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>			
	<u>10-PERCENT</u>	<u>2-PERCENT</u>	<u>1-PERCENT</u>	<u>0.2-PERCENT</u>
HEATH BROOK				
At the downstream Swansea Corporate Limits ²	6.8	9.6	11.6	18.1
PALMER RIVER AND TRIBUTARY TO BARRINGTON RIVER				
From the downstream Swansea Corporate Limits to Interstate 195 ²	6.6	9.5	11.3	17.9
Upstream of Interstate 195 ³	5.9	8.2	9.2	12.2
SEGREGANSET RIVER				
At Taunton River ³	8.4	12.2	13.8	17.6

¹North American Vertical Datum of 1988

²From Bristol County, RI Analysis

³From Precountywide Analysis

Offshore (deepwater) wave heights, wave setup, and wave runup were calculated for each transect using Mathcad (Reference 91) sheets developed by STARR to apply methodologies from the USACE’s Coastal Engineering Manual (Reference 92). Methodologies for each type of calculation are discussed in more detail below. Results from the Mathcad calculations have been summarized in a spreadsheet and both the Mathcad sheets and summary spreadsheet are included in the digital data files compiled for the coastal submittal.

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 92). 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.

Transect descriptions for the June 16, 2014 countywide revision are shown in Table 14. The locations of these transects are depicted in Figure 13.

TABLE 14 – JUNE 16, 2014 COUNTYWIDE REVISION TRANSECT DESCRIPTIONS

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>		<u>V ZONE MAPPING METHOD</u>
		<u>STILLWATER 1-PERCENT-ANNUAL-CHANCE</u>	<u>MAX WAVE CREST 1-PERCENT-ANNUAL-CHANCE²</u>	
56	The transect is located at the shoreline of Mt. Hope Bay, in the Town of Swansea, from the Rhode Island / Massachusetts State Boundary to Barton Avenue	13.9	21.3	Runup
57	The transect is located at the shoreline of Mt. Hope Bay, in the Town of Swansea, from Barton Avenue to Cole Street	13.9	22.0	Breaking wave height
58	The transect is located at the shoreline of Mt. Hope Bay, in the Town of Swansea, from Cole Street to Calef Avenue	13.9	20.5	Runup
59	The transect is located at the shoreline of Mt. Hope Bay, in the Town of Swansea, from Calef Avenue to the intersection of Bay Point Street and Susan Place	13.9	21.5	Runup
60	The transect is located at the shoreline of Mt. Hope Bay, in the Town of Swansea, from the intersection of Bay Point Street and Susan Place to Gardners Neck Road	13.9	22.9	Runup
61	The transect is located at the shoreline of Mt. Hope Bay, at the mouth of the Lee River, in the Town of Swansea, from Gardners Neck Road to Mattapoissett Road	13.9	22.9	Wave overtopping splash zone

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 14 – JUNE 16, 2014 COUNTYWIDE REVISION TRANSECT DESCRIPTIONS - continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>		<u>V ZONE MAPPING METHOD</u>
		<u>STILLWATER 1-PERCENT-ANNUAL-CHANCE</u>	<u>MAX WAVE CREST 1-PERCENT-ANNUAL-CHANCE²</u>	
62	The transect is located at the shoreline of Mt. Hope Bay, at the mouth of the Lee River, in the Town of Swansea, from Mattapoissett Road to the power generation facility approximately 2,000 feet south of Kenneth Avenue	13.9	22.5	Runup
63	The transect is located at the shoreline of Mt. Hope Bay, in the Town of Somerset, from the power generation facility approximately 2,000 feet south of Kenneth Avenue to Farren Street (extended)	13.9	22.3	Wave overtopping splash zone
64	The transect is located at the shoreline of the Taunton River, in the Town of Somerset, from Farren Street (extended) to the intersection of Riverside Avenue and Alden Place	13.9	22.1	Wave overtopping splash zone
65	The transect is located at the shoreline of the Taunton River, in the Town of Somerset, from the intersection of Riverside Avenue and Alden Place to Slades Ferry Avenue	13.9	18.4	Wave overtopping splash zone
66	The transect is located at the shoreline of the Taunton River, in the Town of Somerset, from Slades Ferry Avenue to approximately 700 feet northeast of the Riverside Avenue / Stevens Street intersection	13.9	17.6	Runup
67	The transect is located at the shoreline of the Taunton River, in the Town of Somerset, from approximately 700 feet northeast of the Riverside Avenue / Stevens Street intersection to Cusick Lane	13.9	17.6	Runup

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

TABLE 14 – JUNE 16, 2014 COUNTYWIDE REVISION TRANSECT DESCRIPTIONS - continued

<u>TRANSECT</u>	<u>LOCATION</u>	<u>ELEVATION (feet NAVD¹)</u>		<u>V ZONE MAPPING METHOD</u>
		<u>STILLWATER 1-PERCENT-ANNUAL-CHANCE</u>	<u>MAX WAVE CREST 1-PERCENT-ANNUAL-CHANCE²</u>	
68	The transect is located at the shoreline of the Taunton River, in the Town of Somerset, from Cusick Lane to Euclid Avenue	13.9	18.0	Runup
69	The transect is located at the shoreline of the Taunton River, in the Town of Somerset, from Euclid Avenue to Broad Cove Street (extended)	13.9	17.9	Runup
70	The transect is located at the shoreline of the Taunton River, in the City of Fall River, from Broad Cove Street (extended) to Haskell Street (extended)	13.9	17.6	Runup
71	The transect is located at the shoreline of the Taunton River, in the City of Fall River, from Haskell Street (extended) to Ferry Street	13.9	20.7	Runup
72	The transect is located at the shoreline of the Taunton River, in the City of Fall River, from Ferry Street to Bradford Avenue (extended)	13.9	21.7	Runup
73	The transect is located at the shoreline of Mt. Hope Bay, in the City of Fall River, from Bradford Avenue (extended) to Riverview Street (extended)	13.9	22.3	Runup
74	The transect is located at the shoreline of Mt. Hope Bay, in the City of Fall River, from Riverview Street (extended) to the Massachusetts / Rhode Island State Boundary	13.9	21.1	Wave overtopping splash zone

¹North American Vertical Datum of 1988

²Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

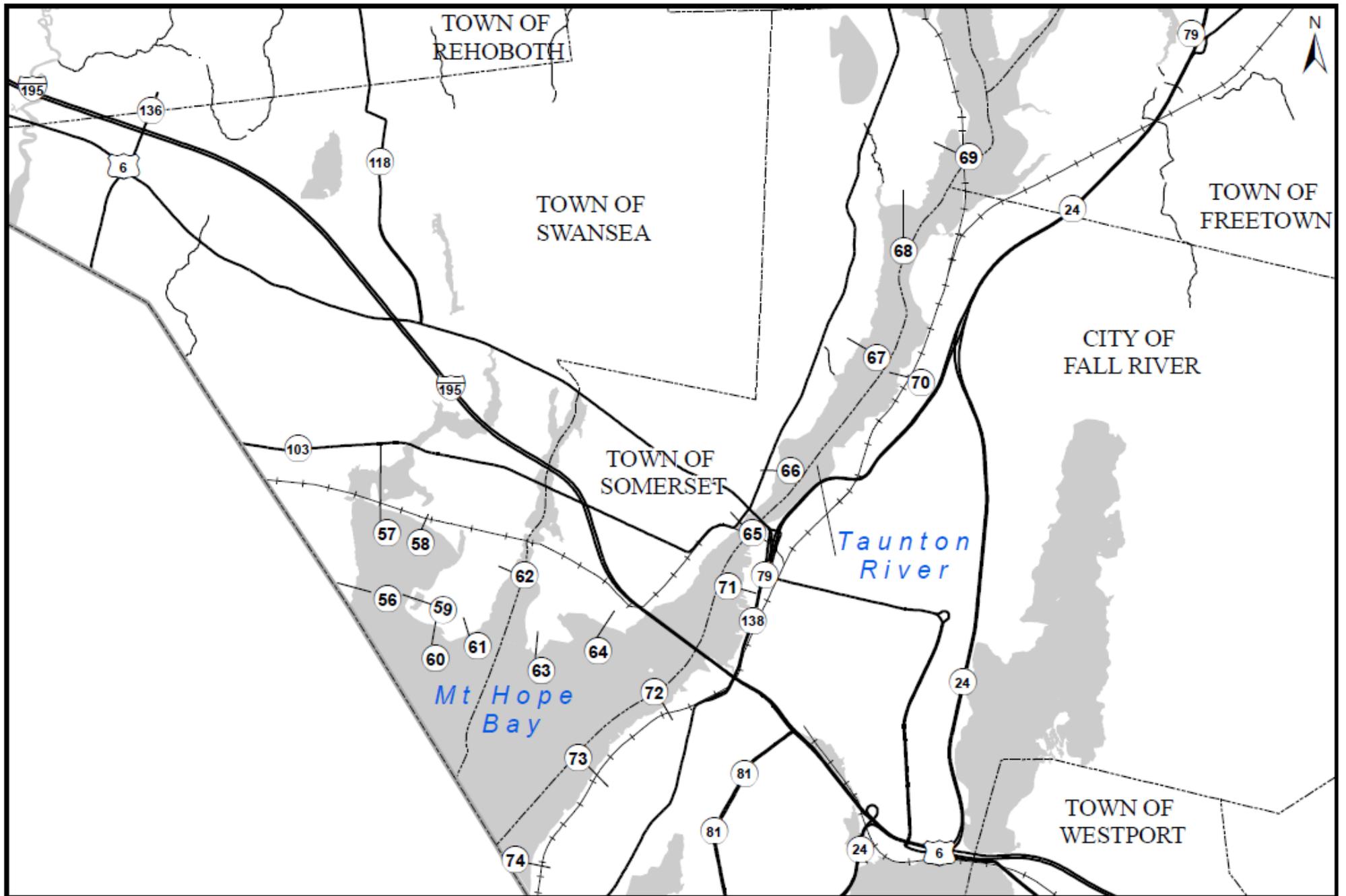


FIGURE 13

FEDERAL EMERGENCY MANAGEMENT AGENCY
 BRISTOL COUNTY, MA
 (ALL JURISDICTIONS)

TRANSECT LOCATION MAP

For the June 16, 2014 countywide revision, coastal transect data was extracted from topographic data collected by Photo Science using LiDAR technology. Additionally, portions of nine coastal transects were surveyed by Green International Affiliates (GIA) to supplement the LiDAR data. As appropriate, coastal protection structure details and 0.0 foot NAVD elevations were included and noted in the transect land surveys performed by GIA. Bathymetric data from NOAA Nautical Charts were used to extend the transects offshore. Coastal processes that may affect the transect profile, such as dune erosion and seawall scour and failure, were estimated using FEMA Guidelines and Specifications.

The energy-based significant wave height (H_{mo}) and peak wave period (T_p) are used as inputs to wave setup and wave runup calculations and were calculated at each transect 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 (Reference 93). The model accepts a spectral form of the wave as an input condition and provides H_{mo} and T_p results over the gridded model domain.

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 94), 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.

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 95), within the Coastal Hazard Analysis Modeling Program (CHAMP), Version 2.0 (Reference 96), following the methodology described in the FEMA Guidelines and Specifications for each coastal transect.

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. Application of CHAMP followed the instruction in the FEMA Guidelines and Specifications and the CHAMP user's guide found in the software documentation (Reference 97).

Topographic, vegetative, and cultural features were identified along each specified transect landward of the shoreline. WHAFIS uses this and other information to calculate the 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 80). 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 the 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 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. Runup estimates were developed for vertical walls using the guidance in Figure D.2.8-3 of the FEMA Guidelines and Specifications, taken from the Shore Protection Manual (Reference 86). The 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 within CHAMP (Reference 97). 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.

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 aerial extent of flooding. The results of the calculations are accurate until local topography, vegetation, or cultural development within the community undergoes major changes.

The LiMWA is determined and defined as the location of the 1.5-foot wave. Typical construction 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 Information Layer on Flood Insurance Rate Maps (FIRMs)” (Reference 98). 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 during the June 16, 2014 countywide revision; therefore no further PFD analysis was performed in Bristol County.

Table 15 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 15 – JUNE 16, 2014 COUNTYWIDE REVISION TRANSECT DATA

<u>Flooding Source and Transect Number</u>	Stillwater Elevations (feet NAVD ¹)				Total Water Level ² 1- percent- annual- chance	<u>Zone</u>	Base Flood Elevation ³ (feet NAVD ¹)
	10- percent- annual- chance	2- percent- annual- chance	1- percent- annual- chance	0.2- percent- annual- chance			
MOUNT HOPE BAY							
Transect 56	8.4	12.3	13.9	17.6	14.8	VE AE	17 15-16
Transect 57	8.4	12.3	13.9	17.6	16.5	VE AE	19 17
Transect 58	8.4	12.3	13.9	17.6	15.0	VE AE	17 *
Transect 59	8.4	12.3	13.9	17.6	15.0	VE AE	18 17
Transect 60	8.4	12.3	13.9	17.6	15.8	VE AE	21 *
Transect 61	8.4	12.3	13.9	17.6	15.8	VE AE	21 *
Transect 62	8.4	12.3	13.9	17.6	15.5	VE AE	18 *
Transect 63	8.4	12.3	13.9	17.6	15.4	VE AE	18 *

¹North American Vertical Datum of 1988

²Including stillwater elevation and the effects of wave setup

³Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

*Data not available

TABLE 15 – JUNE 16, 2014 COUNTYWIDE REVISION TRANSECT DATA - continued

<u>Flooding Source and Transect Number</u>	Stillwater Elevations (feet NAVD ¹)				Total Water Level ² 1- percent- annual- chance	<u>Zone</u>	Base Flood Elevation ³ (feet NAVD ¹)
	10- percent- annual- chance	2- percent- annual- chance	1- percent- annual- chance	0.2- percent- annual- chance			
MOUNT HOPE BAY - continued							
Transect 73	8.4	12.3	13.9	17.6	16.1	VE AE	24 *
Transect 74	8.4	12.3	13.9	17.6	15.1	VE AE	20 *
TAUNTON RIVER							
Transect 64	8.4	12.3	13.9	17.6	16.6	VE AE	19 17
Transect 65	8.4	12.3	13.9	17.6	15.5	VE AE	18 15-16
Transect 66	8.4	12.3	13.9	17.6	14.4	VE AE	16 15
Transect 67	8.4	12.3	13.9	17.6	14.6	VE AE	17 *
Transect 68	8.4	12.3	13.9	17.6	15.1	VE AE	19 *
Transect 69	8.4	12.3	13.9	17.6	14.9	VE AE	18 *
Transect 70	8.4	12.3	13.9	17.6	14.7	VE AE	17 *
Transect 71	8.4	12.3	13.9	17.6	15.2	VE AE	17 15
Transect 72	8.4	12.3	13.9	17.6	15.6	VE AE	24 *

¹North American Vertical Datum of 1988

²Including stillwater elevation and the effects of wave setup

³Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.

*Data not available

The transect schematic (Figure 14) 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. Actual wave conditions in the community may not include all situations illustrated in Figure 14.

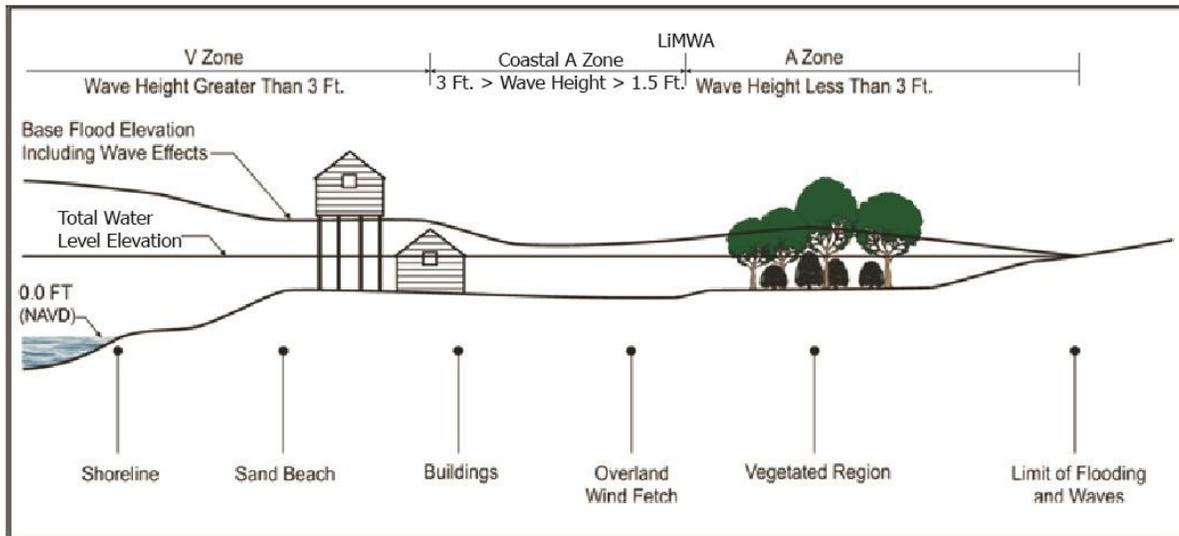


Figure 14 – TRANSECT SCHEMATIC

3.4 Vertical Datum

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

All flood elevations shown in this FIS report and on the FIRM are referenced to the NAVD. 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 by applying a standard conversion factor. **The conversion factor from NGVD to NAVD is -0.8, and from NAVD to NGVD is +0.8.**

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 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.