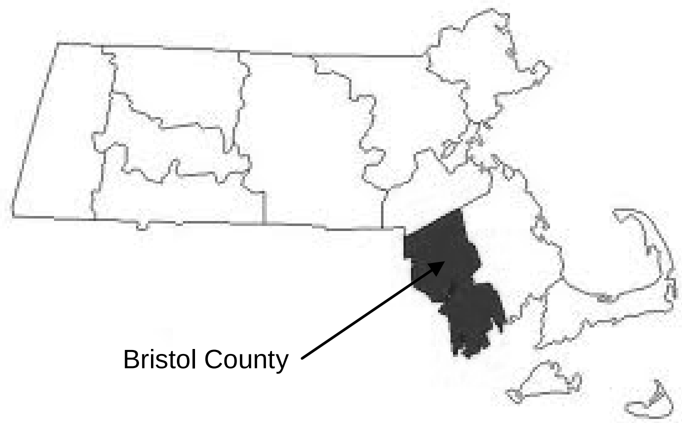
FLOOD INSURANCE STUDY

VOLUME 1 OF 4

BRISTOL COUNTY, MASSACHUSETTS
(ALL JURISDICTIONS)



COMMUNITY NAME	COMMUNITY NUMBER
ACUSHNET, TOWN OF	250048
ATTLEBORO, CITY OF	250049
BERKLEY, TOWN OF	250050
DARTMOUTH, TOWN OF	250051
DIGHTON, TOWN OF	250052
EASTON, TOWN OF	250053
FAIRHAVEN, TOWN OF	250054
FALL RIVER, CITY OF	250055
FREETOWN, TOWN OF	250056
MANSFIELD, TOWN OF	250057
NEW BEDFORD, CITY OF	255216
NORTH ATTLEBOROUGH, TOWN OF	250059
NORTON, TOWN OF	250060
RAYNHAM, TOWN OF	250061
REHOBOTH, TOWN OF	250062
SEEKONK, TOWN OF	250063
SOMERSET, TOWN OF	255220
SWANSEA, TOWN OF	255221
TAUTON, CITY OF	250066
WESTPORT, TOWN OF	255224

REVISED
JULY 16, 2014

Federal Emergency Management Agency

FLOOD INSURANCE STUDY NUMBER
25005CV001B

NOTICE TO
FLOOD INSURANCE STUDY USERS

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

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

Old Zone	New Zone
A1 through A30	AE
V1 through V30	VE (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

TABLE OF CONTENTS – Volume 1 – July 16, 2014

			Page
1.0	INTRODUCTION		1
	1.1	Purpose of Study	1
	1.2	Authority and Acknowledgments	1
	1.3	Coordination	6
2.0	AREA STUDIED		7
	2.1	Scope of Study	7
	2.2	Community Description	18
	2.3	Principal Flood Problems	18
	2.4	Flood Protection Measures	25
3.0	ENGINEERING METHODS		27
	3.1	Hydrologic Analyses	28
	3.2	Hydraulic Analyses	70
	3.3	Coastal Analyses	76
	3.4	Vertical Datum	115

FIGURES

Figure 1 – Frequency-Discharge, Drainage Area Curves for Anawan Brook-Bliss Brook	59
Figure 2 – Frequency-Discharge, Drainage Area Curves for Bad Luck Brook	60
Figure 3 – Frequency-Discharge, Drainage Area Curves for Buttonwood Brook	61
Figure 4 – Frequency-Discharge, Drainage Area Curves for Canoe River-Wading River	62
Figure 5 – Frequency-Discharge, Drainage Area Curves for East Branch Palmer River	63
Figure 6 – Frequency-Discharge, Drainage Area Curves for Hodges Brook-Rumford River	64
Figure 7 – Frequency-Discharge, Drainage Area Curves for Palmer River	65
Figure 8 – Frequency-Discharge, Drainage Area Curves for Paskamanset River	66
Figure 9 – Frequency-Discharge, Drainage Area Curves for Rocky Run	67

<u>TABLE OF CONTENTS</u> – Volume 1 - continued

<u>FIGURES</u> - continued

	Page
Figure 10 – Frequency-Discharge, Drainage Area Curves for Sabin Pond Brook-Oak Swamp Brook	68
Figure 11 - Frequency-Discharge, Drainage Area Curves for West Branch Palmer River	69
Figure 12 – July 7, 2009 Countywide Analysis Transect Location Map	97
Figure 13– 2012 Coastal Study Update Transect Location Map	110
Figure 14 – Transect Schematic	115

<u>TABLES</u>

Table 1 – CCO Meetings Dates for Precountywide FIS	6
Table 2 – Flooding Sources Studied by Detailed Methods	7
Table 3 – Scope of Revision	14
Table 4 - Flooding Sources Studied by Approximate Methods	15
Table 5 - Letters of Map Change	17
Table 6 - Summary of Discharges	33
Table 7 – Manning's "n" Values	75
Table 8 – Stage-Frequency Data	79
Table 9 - Precountywide Summary of Stillwater Elevations	81
Table 10 - Summary of July 7, 2009 Countywide Analysis Stillwater Elevations	85
Table 11 – July 7, 2009 Countywide Analysis Transect Descriptions	87
Table 12 – July 7, 2009 Countywide Analysis Transect Data	99
Table 13 – Summary of 2012 Coastal Study Update Stillwater Elevations	105
Table 14 – 2012 Coastal Study Update Transect Descriptions	106
Table 15 – 2012 Coastal Study Update Transect Data	113

TABLE OF CONTENTS – Volume 2 – July 16, 2014

		Page
4.0	FLOODPLAIN MANAGEMENT APPLICATIONS	117
	4.1 Floodplain Boundaries	117
	4.2 Floodways	118
5.0	INSURANCE APPLICATION	195
6.0	FLOOD INSURANCE RATE MAP	196
7.0	OTHER STUDIES	199
8.0	LOCATION OF DATA	199
9.0	BIBLIOGRAPHY AND REFERENCES	199

FIGURES

Figure 15– Floodway Schematic 121

TABLES

Table 16 – Floodway Data Table 122

Table 17 – Community Map History 197

TABLE OF CONTENTS – Volume 3 – July 16, 2014

EXHIBITS

Exhibit 1 -	Flood Profiles	
	Abbott Run	Panel 01P-02P
	Acushnet River	Panel 03P-07P
	Anawan Brook	Panel 08P-08P
	Armstrong Brook	Panel 09P-09P
	Assonet River	Panel 10P-12P
	Attleboro Industrial Stream	Panel 13P-13P
	Bad Luck Brook	Panel 14P-14P

EXHIBITS - continued

Exhibit 1 -	Flood Profiles	
	Black Brook	Panel 15P-18P
	Bliss Brook	Panel 19P-19P
	Bungay River	Panel 20P-22P
	Buttonwood Brook	Panel 23P-28P
	Buttonwood Brook East	Panel 29P-30P
	Buttonwood Brook West	Panel 31P-33P
	Canoe River (Lower Reach)	Panel 34P-35P
	Canoe River (Upper Reach)	Panel 36P-38P
	Chartley Brook	Panel 39P-40P
	Cobb Brook	Panel 41P-43P
	Coles Brook	Panel 44P-44P
	Dam Lot Brook	Panel 45P-45P
	Deep Brook	Panel 46P-47P
	East Branch Palmer River	Panel 48P-49P
	East Junction Stream	Panel 50P-50P
	Elmwood Street Brook	Panel 51P-51P
	Fall Brook	Panel 52P-54P
	Forge River	Panel 55P-58P
	Goose Branch Brook	Panel 59P-59P
	Gowards Brook	Panel 60P-60P
	Hodges Brook	Panel 61P-63P
	Lake Como Stream	Panel 64P-64P
	Landry Avenue Brook	Panel 65P-65P
	Mary Kennedy Brook	Panel 66P-66P
	Mason Park Brook	Panel 67P-67P
	Mill River	Panel 68P-70P
	Mulberry Brook and Poquanticut Brook	Panel 71P-71P
	Oak Hill Stream	Panel 72P-72P
	Oak Swamp Brook	Panel 73P-73P
	Palmer River	Panel 74P-77P
	Paskamanset River	Panel 78P-78P
	Poquanticut Brook	Panel 79P-81P
	Queset Brook	Panel 82P-86P
	Rattlesnake Brook (Freetown)	Panel 87P-87P
	Rattlesnake Brook (North Attleborough)	Panel 88P-88P
	Rocklawn Avenue Stream	Panel 89P-89P
	Rocky Run	Panel 90P-92P
	Rumford River (Lower Reach)	Panel 93P-94P
	Rumford River (Upper Reach)	Panel 95P-98P

TABLE OF CONTENTS – Volume 4 – July 16, 2014

EXHIBITS - continued

Exhibit 1 -	Flood Profiles	
	Runnins River	Panel 99P-103P
	Sabin Pond Brook	Panel 104P-104P
	Scotts Brook	Panel 105P-107P
	Segreganset River (Lower Reach)	Panel 108P-110P
	Segreganset River (Upper Reach)	Panel 111P-112P
	Seven Mile River	Panel 113P-118P
	Speedway Brook	Panel 119P-119P
	Sunken Brook	Panel 120P-120P
	Taunton River	Panel 121P-124P
	Ten Mile River	Panel 125P-139P
	Three Mile River	Panel 140P-144P
	Three Mile River West Channel	Panel 145P-145P
	Tributary to Dam Lot Brook	Panel 146P-146P
	Tributary to Forge River	Panel 147P-148P
	Wading River	Panel 149P-153P
	West Branch Palmer River	Panel 154P-157P
	Whiting Pond Bypass	Panel 158P-158P
	Whitman Brook	Panel 159P-159P
Exhibit 2 -	Flood Insurance Rate Map Index	
	Flood Insurance Rate Map	

FLOOD INSURANCE STUDY
BRISTOL COUNTY, MASSACHUSETTS (ALL JURISDICTIONS)

1.0 INTRODUCTION

1.1 Purpose of Study

This Flood Insurance Study (FIS) revises and updates information on the existence and severity of flood hazards in the geographic area of Bristol County, including the Cities of Attleboro, Fall River, New Bedford and Taunton and the Towns of Acushnet, Berkley, Dartmouth, Dighton, Easton, Fairhaven, Freetown, Mansfield, North Attleborough, Norton, Raynham, Rehoboth, Seekonk, Somerset, Swansea, and Westport (referred to collectively herein as Bristol County), and aids in the administration of the National Flood Insurance Act of 1968 and the Flood Disaster Protection Act of 1973. This study has developed flood-risk data for various areas of the community that will be used to establish actuarial flood insurance rates and to assist the community in its efforts to promote sound floodplain management. Minimum floodplain management requirements for participation in the National Flood Insurance Program (NFIP) are set forth in the Code of Federal Regulations at 44 CFR, 60.3.

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

1.2 Authority and Acknowledgments

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

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

Acushnet, Town of:	The hydrologic and hydraulic analyses for the January 19, 1982 study were prepared by PJR Consulting Engineers for the Federal Emergency Management Agency (FEMA), under Contract No. H-4795. This work was completed in April 1980.
Attleboro, City of:	The hydrologic and hydraulic analyses for the March 1978 study were performed by the U.S. Department of Agriculture, Soil Conservation Service (SCS) for the Federal Insurance Administration (FIA), under Interagency Agreement No. IAA-H-18-75, Project Order No. 8. This work, which was completed in June 1977, covered all significant flooding sources affecting the City of Attleboro.

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

Easton, Town of – continued:	Affiliates, Inc. for FEMA, under Contract No. EMB-96-CO-0403 (Task #l0). This work was completed in March 1998.
Fairhaven, 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-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 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

Somerset, Town of - continued:	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 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 Dartmouth, Fairhaven, New Bedford, 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 this 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 (FIPSZONE2001). 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

Community Name	Initial CCO Date	Intermediate CCO Date	Final CCO Date
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
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 this countywide FIS, which includes an updated coastal and backwater analysis along applicable areas of Berkley, Dighton, Fall River, 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.

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

Flooding Source Name	Description of Study Reaches
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

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS - continued

Flooding Source Name	Description of Study Reaches
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 Town 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
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

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS - continued

Flooding Source Name	Description of Study Reaches
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
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

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS - continued

Flooding Source Name	Description of Study Reaches
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 approximately 1 mile upstream
Lake Sabbatia	For the entire shoreline within the City of 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

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS - continued

Flooding Source Name	Description of Study Reaches
Mill Pond	For the entire shoreline within the City of Taunton
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

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS - continued

Flooding Source Name	Description of Study Reaches
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
Rumford River (Lower Reach)	From confluence with the Three Mile River to approximately 6,000 feet upstream of Cross Street in the Town of 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 the Town of 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

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS - continued

Flooding Source Name	Description of Study Reaches
Seven Mile River	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	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
Ten Mile River	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)	From its confluence with the Three Mile River to the upstream Town of Norton corporate limits

TABLE 2 –FLOODING SOURCES STUDIED BY DETAILED METHODS - continued

Flooding Source Name	Description of Study Reaches
Wading River (Upper Reach)	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	From its confluence with Ten Mile River to 1,700 ft 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

For flooding sources studied by detailed methods for the July 7, 2009 study, see Table 3, “Scope of Revision.”

TABLE 3 - SCOPE OF REVISION

Flooding Source	Limits of Revised or New Detailed Study
BUZZARDS BAY	For the entire shoreline within the Towns of Dartmouth, Fairhaven, and New Bedford
RHODE ISLAND SOUND	For the entire shoreline within the Town of Westport

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

Flooding Source Name	Community (s)
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
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

TABLE 4 - FLOODING SOURCES STUDIED BY APPROXIMATE METHODS - continued

Flooding Source Name	Community (s)
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
Noquochoke Lake	Dartmouth
North Watuppa Pond	Fall River, Westport
Norton Reservoir	Norton
Oak Hill Stream	Seekonk
Old Pond	Easton
Paskamanset 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

TABLE 4 - FLOODING SOURCES STUDIED BY APPROXIMATE METHODS - continued

Flooding Source Name	Community (s)
Unnamed Ponds	Countywide
Unnamed Streams	Countywide
Unnamed Swamps	Rehoboth, Swansea, Taunton
Unnamed Tributary to Black Brook	Easton
Unnamed Tributary to Poquanticut Brook	Easton
Wading River	Mansfield
Ward Pond	Easton
Whitesville Pond	Mansfield
Whitman Brook	Easton
Witch Pond and Swamp	Mansfield

Detail-studied streams that were not re-studied as part of the July 7, 2009 analysis may include a profile baseline on the FIRM. The profile baselines for these streams were based on the best available data at the time of their study and are depicted as they were on the previous FIRMs. In some cases the transferred profile baseline may deviate significantly from the channel or may be outside of the floodplain.

The July 7, 2009 FIS also incorporates 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

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

[1]Incorporated during 2012 Coastal Study Update

2012 Coastal Study Update

The coastal wave height analysis for this countywide coastal study was prepared by STARR. This new analysis resulted in revisions to the FIRM for the City of 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 LOMR 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.

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 the 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 North Attleborough. Attleboro received nearly eight inches of rain in less than 48 hours. Low-lying areas were flooded and downtown Attleboro in the vicinity of County Street and Riverbank Road resembled a small lake. 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 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 Attleboro center and in the vicinity of Route 1 in 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.

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 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 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 does not maintain gaging stations on the streams in Dartmouth; therefore, records of past riverine floods are limited. Based on the accounts of local residents and records at nearby gaging stations, the flood of March 1968 was considered a significant event.

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 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).

In 1968, major flood problems in 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.

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 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. Mansfield has suffered considerable damages from the floods that occurred in 1938, 1954, and 1968. The flood of March 1968, the most recent severe flood to occur in Mansfield, 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 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 on both Mansfield and neighboring Foxboro. Throughout the town some 370 homes reported water damage from this flood and several roads and culverts also suffered damage.

Also in 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 principal flood problems in 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 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 No. 01109000) since 1925 and at West Mansfield (gage No. 01108500) since 1953. The USGS has also collected gage information from gage No. 01109200 on the West Branch Palmer River near Rehoboth.

Most flood problems in 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 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 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 so 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 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 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, and 1968 caused the most damage. The 1938, 1954, and 1955 floods were caused by hurricanes; and the 1968 flood was caused by rainfall-induced spring thaw. 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. 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 1.0 foot of water lay over Brittania 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 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 New Bedford along Buzzards Bay is primarily residential and recreational. Floodplain development along the Acushnet River is primarily industrial.

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

The floodplains within 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 Dartmouth, Fairhaven, Fall River, New Bedford, 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 Fall River, 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 Dartmouth, Fairhaven, New Bedford, 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 Fall River, Somerset, and Swansea (Reference 6); 12.5 feet and 11.9 feet, respectively, in Dartmouth; 12.8 feet and 12.1 feet, respectively, in New Bedford and Fairhaven; and 12.2 feet and 11.2 feet, respectively, in Westport. These storms claimed lives and damaged residential, recreational, and small commercial developments, including harbors and marinas.

Riverine flooding along the New Bedford and Fairhaven waterfront occurs when the Acushnet River is ponded behind the New Bedford-Fairhaven hurricane barrier during concurrent periods of high runoff and surge activity. For the most part, this flooding is limited to parking lots and rail yards. Isolated flooding in 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 Norfolk County. Hurricane Gloria in September 1985 arrived at low tide and resulted in storm surges less than 5 feet above normal, minimizing damage to the coastline. Hurricane Bob in August 1991 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 and May 2006. The June 1998 storm was slow moving and produced rainfall of 6 to 12 inches over much of eastern Massachusetts. On May 24, 2009 Bristol, Plymouth, Norfolk, and Worcester Counties experienced an intense thunderstorm causing minor flooding, winds exceeding 70MPH, and quarter sized to golf-ball sized hail (Reference 8).

In March 2010, heavy rainfall of 6 to 10 inches fell over much of Southern New England resulting in major flooding across eastern Massachusetts and Rhode Island. The storm produced 6 to 10 inches of rain over Bristol County, causing several small streams to rise above flood stage, including 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 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 Fall River and Taunton, MA. 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 and are summarized below:

The most effective flood protection measures for many of the communities in Bristol County, including 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 Cities of New Bedford and 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 Dartmouth, Fairhaven, Fall River, New Bedford, 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 Fall River, 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 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 Foxboro. The remaining drainage area is controlled in Mansfield by Robinson's Pond, located above Williams Street, Blakes Pond, upstream of Balcolm 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 Foxboro 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 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, City of Taunton FIS, there were no flood protection structures in the city. Natural storage in swamps and ponds diminishes peak flows in 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 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 Attleboro along the Ten Mile River, there are several dams in North Attleborough at Falls Pond and Whiting Pond. 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 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 Atttleboro 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.

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 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, <u>average</u> 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 Acushnet, Mulberry, Poquanticut, Black and Queset Brooks in Easton, the Forge River, the Tributary to Forge River, Dam Lot Brook and the Tributary to Dam Lot Brook in Raynham, the Mill River in Taunton, and for certain waterways in 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 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 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 Raynham, an n value of 0.72 was used to estimate discharges at downstream sites on the Taunton River. In 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 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 Attleboro, 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 Berkley.

Peak discharges the 10-, 2-, 1-, and 0.2-percent-annual-chance floods in 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 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 Freetown and for the Segreganset River and Sunken Brook in Dighton (Reference 28). Standard USGS 7.5 minute quadrangle maps, with a scale of 1:24000 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 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, City of 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 Balcolm 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 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 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 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 and 38):

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

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 precountywide FIS (Reference 39). 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:

$$Q = CL\ g\ d^{1.5}$$

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 precountywide FIS for the Town of Warren, Rhode Island (Reference 40). 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 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 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 41). 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 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.

<u>Countywide Analyses</u>

For the July 7, 2009 and the 2012 Coastal Study Update countywide revisions, no new hydrologic analyses were conducted.

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

TABLE 6 – SUMMARY OF DISCHARGES

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
ABBOTT RUN	24.08	690	1120	1370	2120
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

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
ANAWAN BROOK - continued					
Location 3* in Rehoboth	0.50	60	100	130	220
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

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
ASSONET RIVER - continued					
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
ATTLEBORO INDUSTRIAL STREAM					
County Street in Attleboro	0.30	13	23	28	43
Tiffany Street in Attleboro	0.10	4	7	8	13

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
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

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
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
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

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
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
CHARTLEY BROOK					
Town Boundary with Norton	6.60	180	270	320	430
Wilmarth Street	1.50	60	90	100	150

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
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

* Values estimated from the Frequency-Discharge, Drainage Area Curves following this table
** Data not available

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
DAM LOT BROOK					
At mouth	3.00	150	260	320	490
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

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
ELMWOOD STREET BROOK - continued					
Washington Street in North Attleboro	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
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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
FALL BROOK - continued					
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
GOOSE BRANCH BROOK					
At confluence with Winnecunnet Pond	3.30	230	335	390	510

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
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

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
LANDRY AVENUE BROOK - continued					
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
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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
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
Janice Lane in North Attleborough	0.18	13	24	30	48
Landry Avenue in North Attleborough	0.07	5	9	11	19
At confluence with Taunton River	43.80	1100	1700	2100	3100
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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
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
Location 6* in Rehoboth	21.30	800	1250	1525	2275
PASKAMANSET RIVER					
Location 1* in Dartmouth	25.00	450	700	850	1200

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
PASKAMANSET RIVER - continued					
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 Long Pond	2.80	140	230	275	380
RATTLESNAKE BROOK					
Narrows Road	6.86	344	588	664	1115

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
RATTLESNAKE BROOK - continued					
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
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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
ROCKY RUN - continued					
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
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

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
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

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
SEGREGANSET RIVER - continued					
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
SEVEN MILE RIVER					
County Street	12.00	510	860	1040	1560
I-95 in Attleboro	11.00	500	850	1020	1530
Roy Avenue in Attleboro	9.10	430	730	870	1300
Attleboro Town Boundary	4.50	240	400	470	690
Attleboro corporate limit	4.20	240	390	470	690
Old Mill Dam	4.12	220	360	430	640
Footbridge in North Attleboro	3.71	200	320	390	570
Interstate Highway 295 in North Attleboro	3.56	190	310	370	540
Adams Avenue in North Attleboro	3.48	180	300	360	530

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
SEVEN MILE RIVER - continued					
Washington Street in North Attleboro	3.06	160	260	310	460
Hoppin Hill Road in North Attleboro	1.98	92	140	170	250
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					
At Raynham corporate boundary	312.00	4000	5500	6200	7900
Above confluence with Forge River	302.00	3800	5200	5900	7600
Above confluence with Poquoy Brook	273.00	3700	5000	5600	7200

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
TAUNTON RIVER - continued					
Above confluence of Three Mile River	369.00	4500	6200	6900	8900
Above confluence of Mill River	314.00	4000	5500	6200	7900
Above confluence of Forge River at Raynham	298.00	3800	5200	5900	7600
TEN MILE RIVER					
Mill Bridge	28.00	660	1040	1310	2130
Lamb Street in Attleboro	24.00	580	930	1170	1860
County Street in Attleboro	20.00	450	760	930	1500
I-95 in Attleboro	11.00	370	630	780	1260
Cedar Road in Attleboro	10.50	350	600	750	1210
Attleboro corporate limit	10.98	350	610	750	1210
Confluence with Rattlesnake Brook	10.77	350	600	730	1190
Freeman Street in North Attleborough	9.65	310	520	640	1050

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
TEN MILE RIVER - continued					
Sturdy Lane in North Attleborough	9.43	300	510	630	1040
Towne Street in North Attleborough	9.28	290	500	620	1020
Falls Pond Dam	8.56	270	470	580	960
Washington Street in North Attleborough	7.44	270	420	510	860
Confluence with Scotts Brook	6.99	250	390	480	810
Chestnut Street in North Attleborough	5.71	180	300	380	660
Elm Street in North Attleborough	5.56	170	300	370	660
Orne Street in North Attleborough	5.45	170	290	370	650
Fisher Street in North Attleborough	5.27	160	280	350	630
Footbridge	5.07	150	270	330	600
Confluence with Elmwood Street Brook	4.90	140	250	320	580
Whiting Pond Dam	4.23	86	150	200	390
Cross Section A	42.10	1100	1690	2100	3500

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
TEN MILE RIVER - continued					
Cross Section B	29.50	690	1080	1370	2220
Seekonk Corporate limits	29.20	680	1070	1360	2200
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

** Data not available

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
WADING RIVER					
At confluence with Three Mile River	42.40	800	1230	1450	2070
At confluence of Goose Branch Brook	37.80	705	1090	1290	1860
At West Main Street	28.70	515	820	980	1440
Location 1* in Mansfield	21.00	300	480	620	1000
Location 2* in Mansfield (USGS gaging station)	19.20	290	470	590	950
Location 3* in Mansfield	18.20	280	460	580	920
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(USGS Gaging Station)	5.00	300	490	510	1060
Location 4* in Rehoboth	4.30	280	420	630	950

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

TABLE 6 – SUMMARY OF DISCHARGES - continued

FLOODING SOURCE AND LOCATION	DRAINAGE AREA (SQUARE MILES)	PEAK DISCHARGES (CUBIC FEET PER SECOND) 10-PERCENT ANNUAL CHANCE	2-PERCENT ANNUAL CHANCE	1-PERCENT ANNUAL CHANCE	0.2-PERCENT ANNUAL CHANCE
WEST BRANCH PALMER RIVER - continued					
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					
Confluence with Ten Mile River	0.11	33	59	73	120
Plainville corporate limit	0.01	27	52	65	110
WHITMAN BROOK					
At Longwater Pond	2.97	150	240	290	420
At Contrail	1.94	110	180	220	335
At Stoughton-Easton corporate limits	1.55	95	155	190	295

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

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.

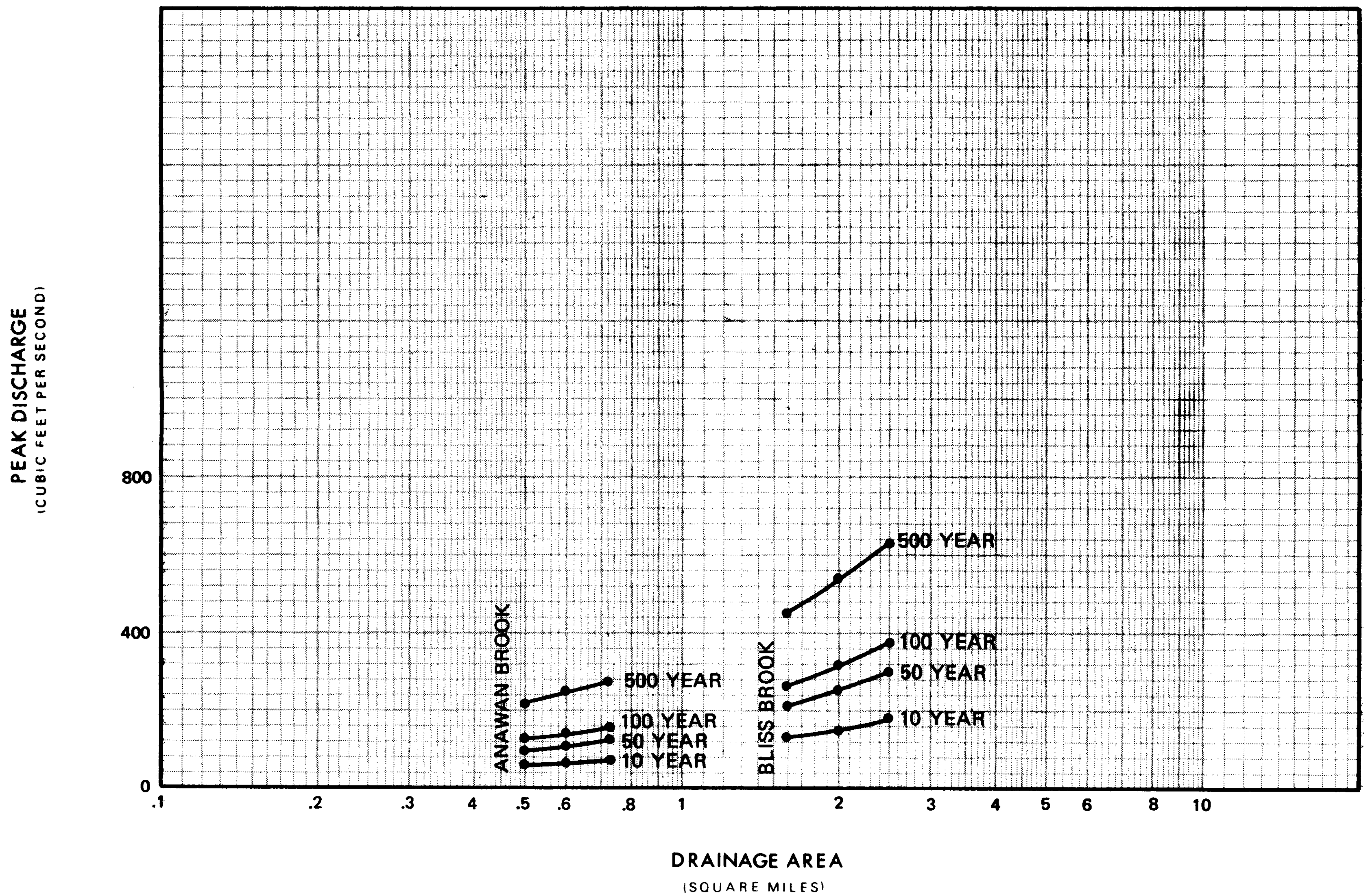


Figure 1

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

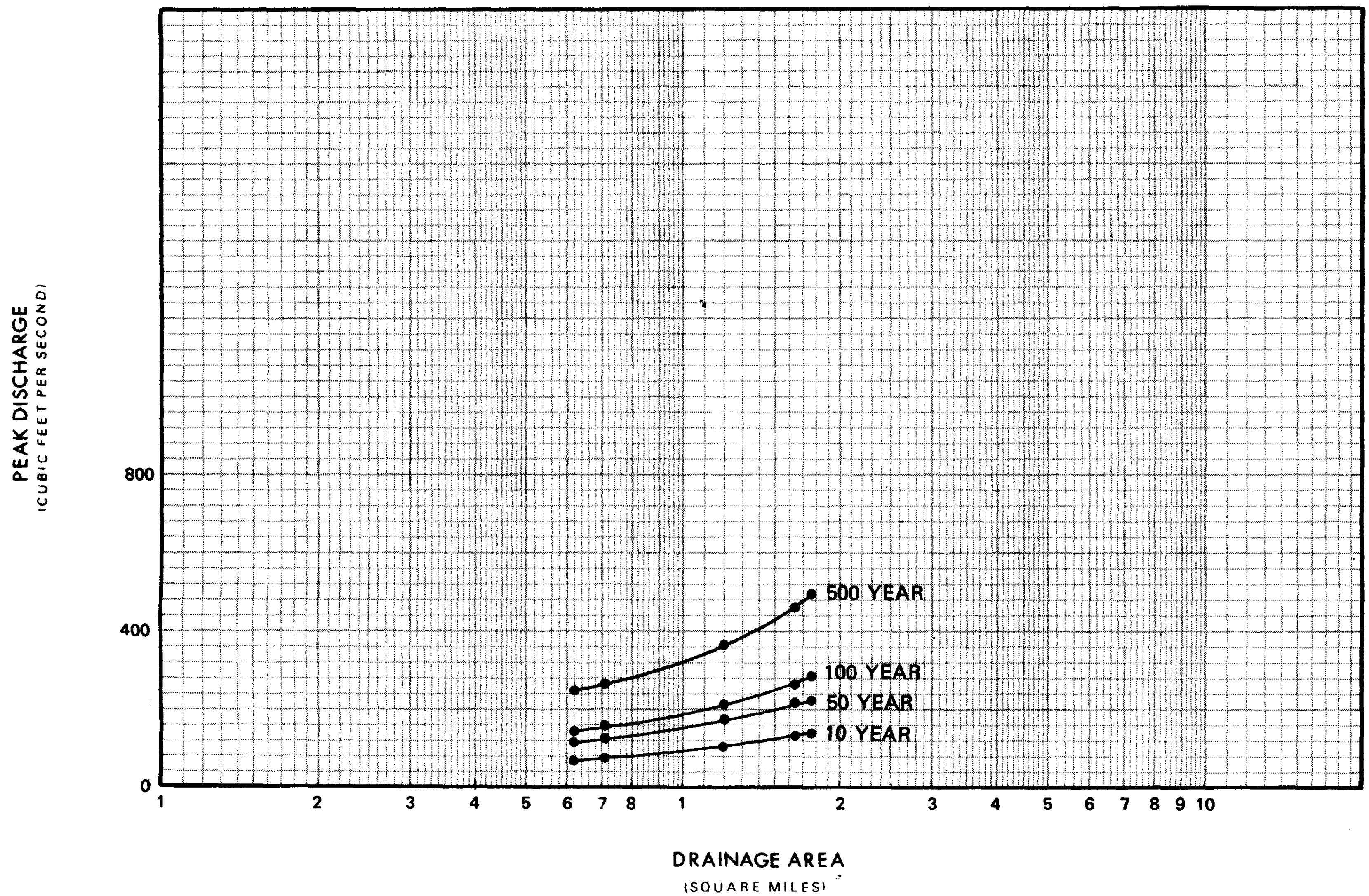


Figure 2

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

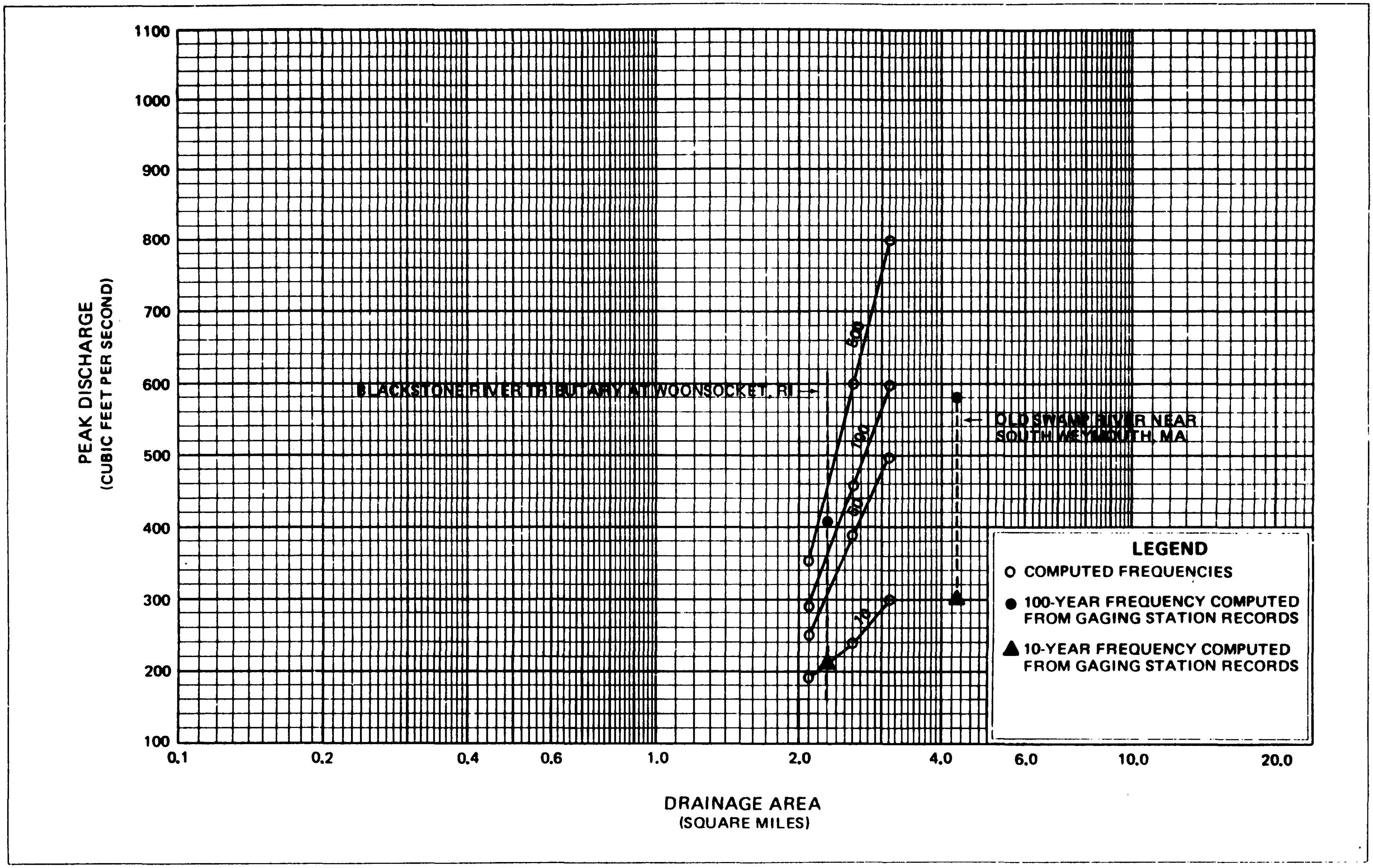


Figure 3

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF DARTMOUTH, MA

BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES

BUTTONWOOD BROOK

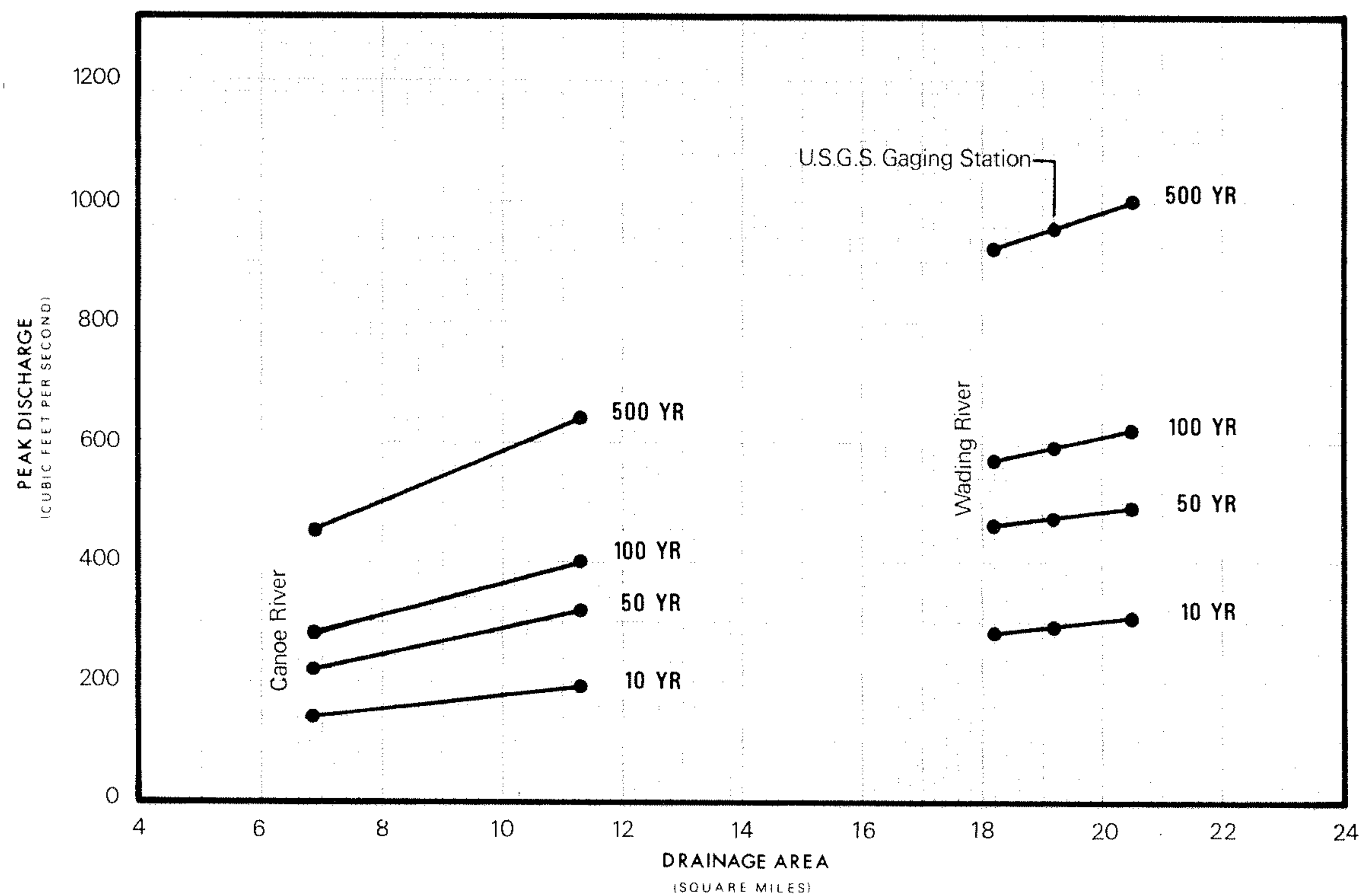


Figure 4

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

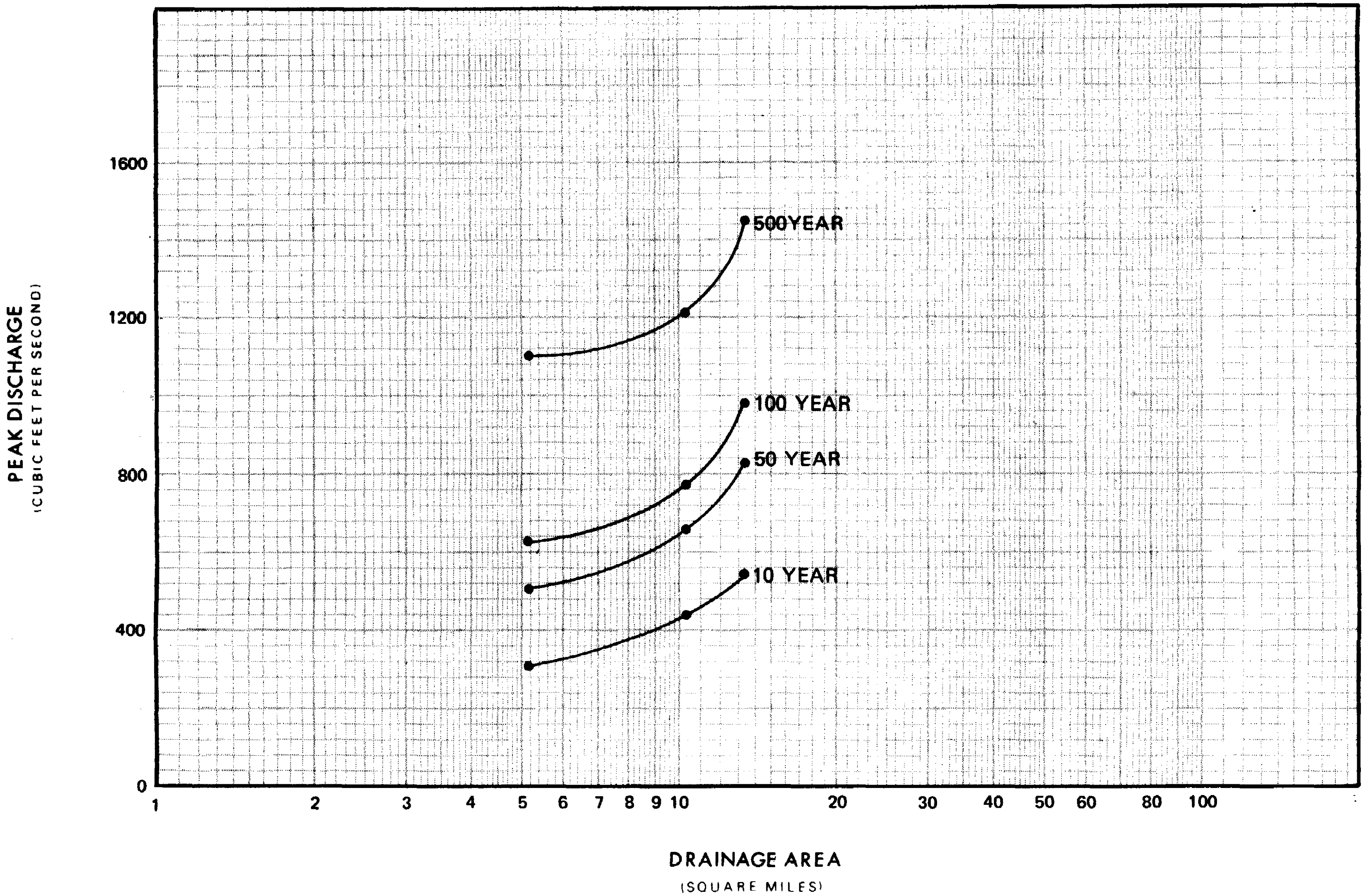


Figure 5

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

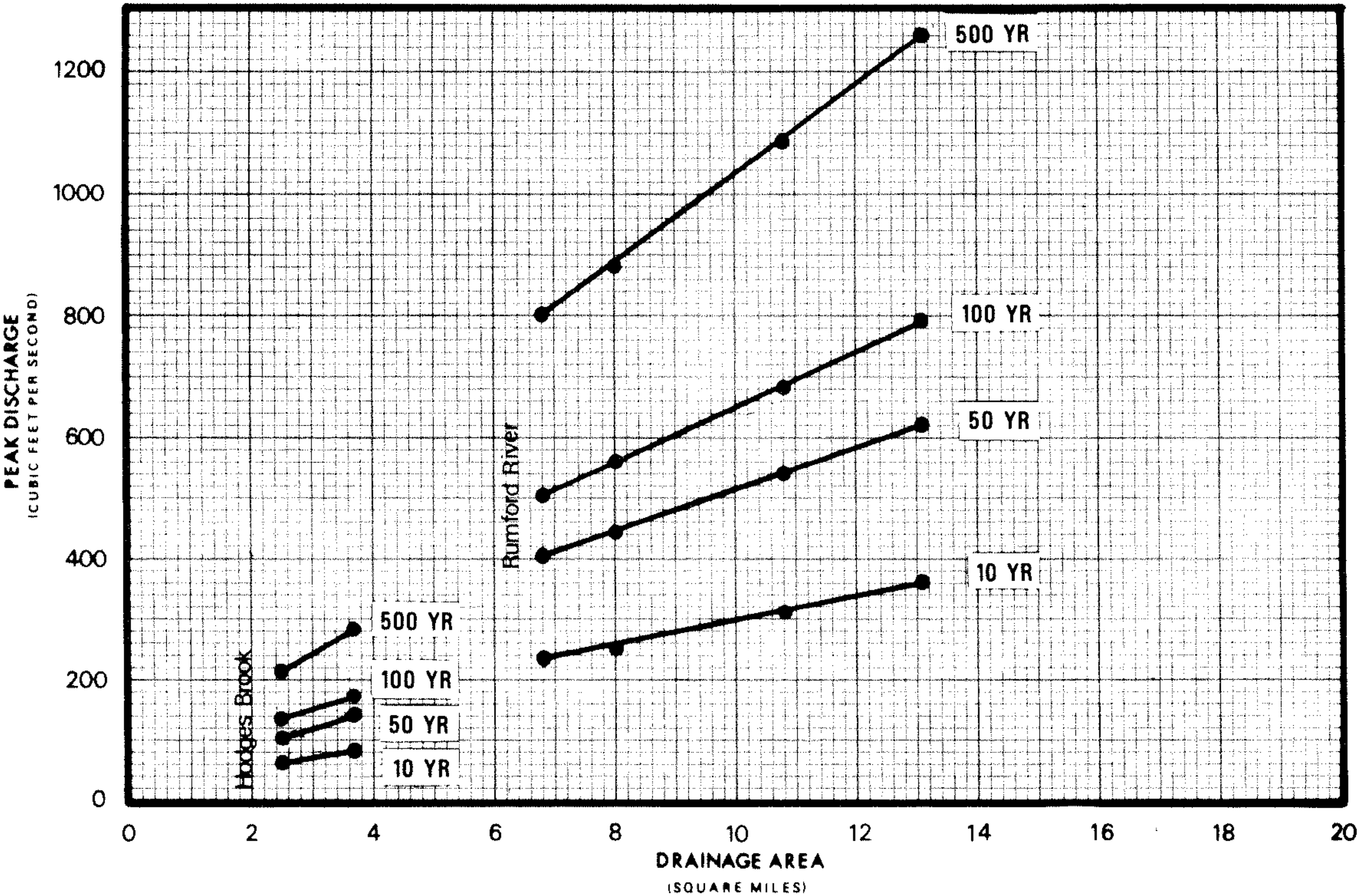


Figure 6

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

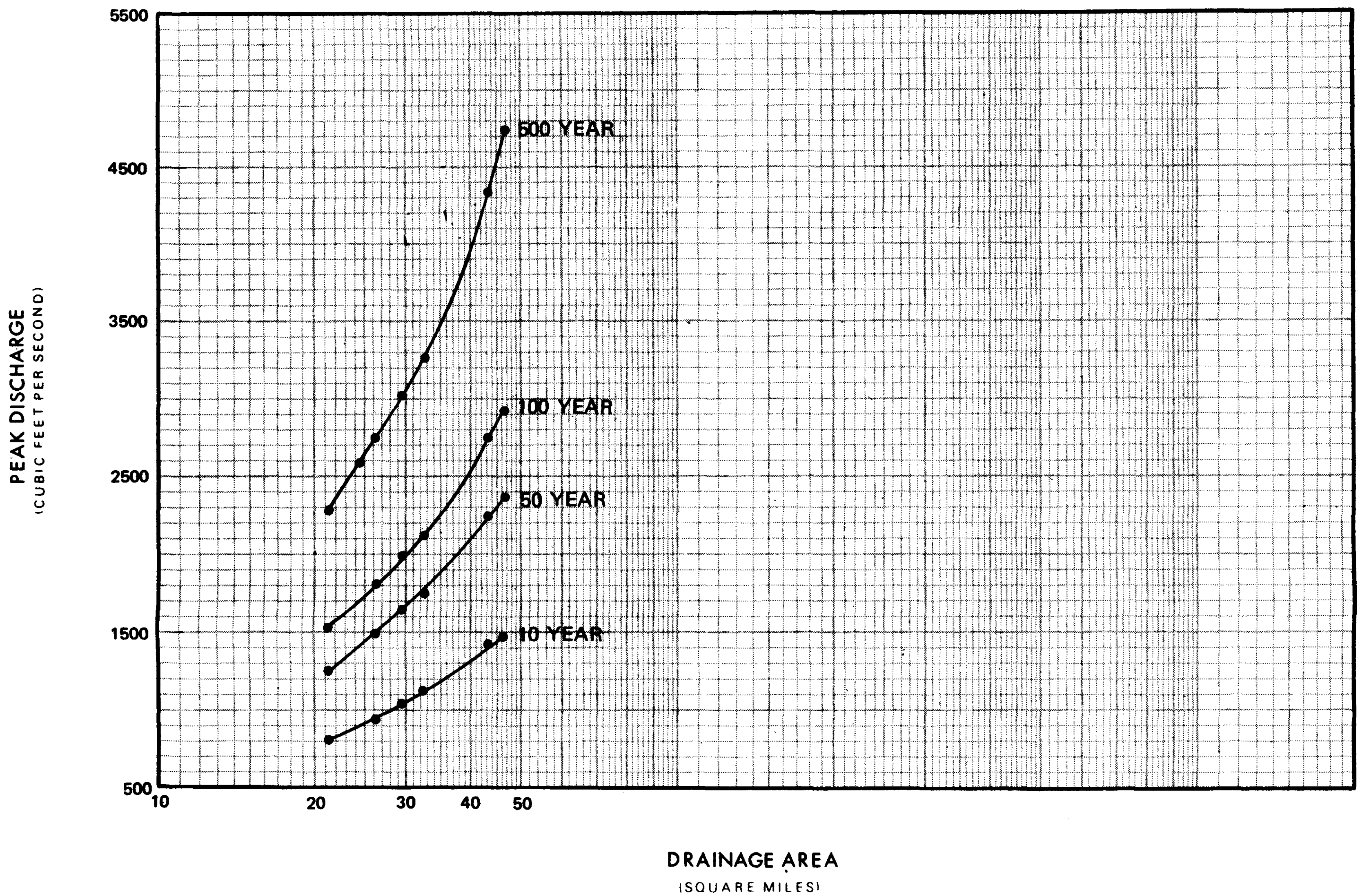


Figure 7

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

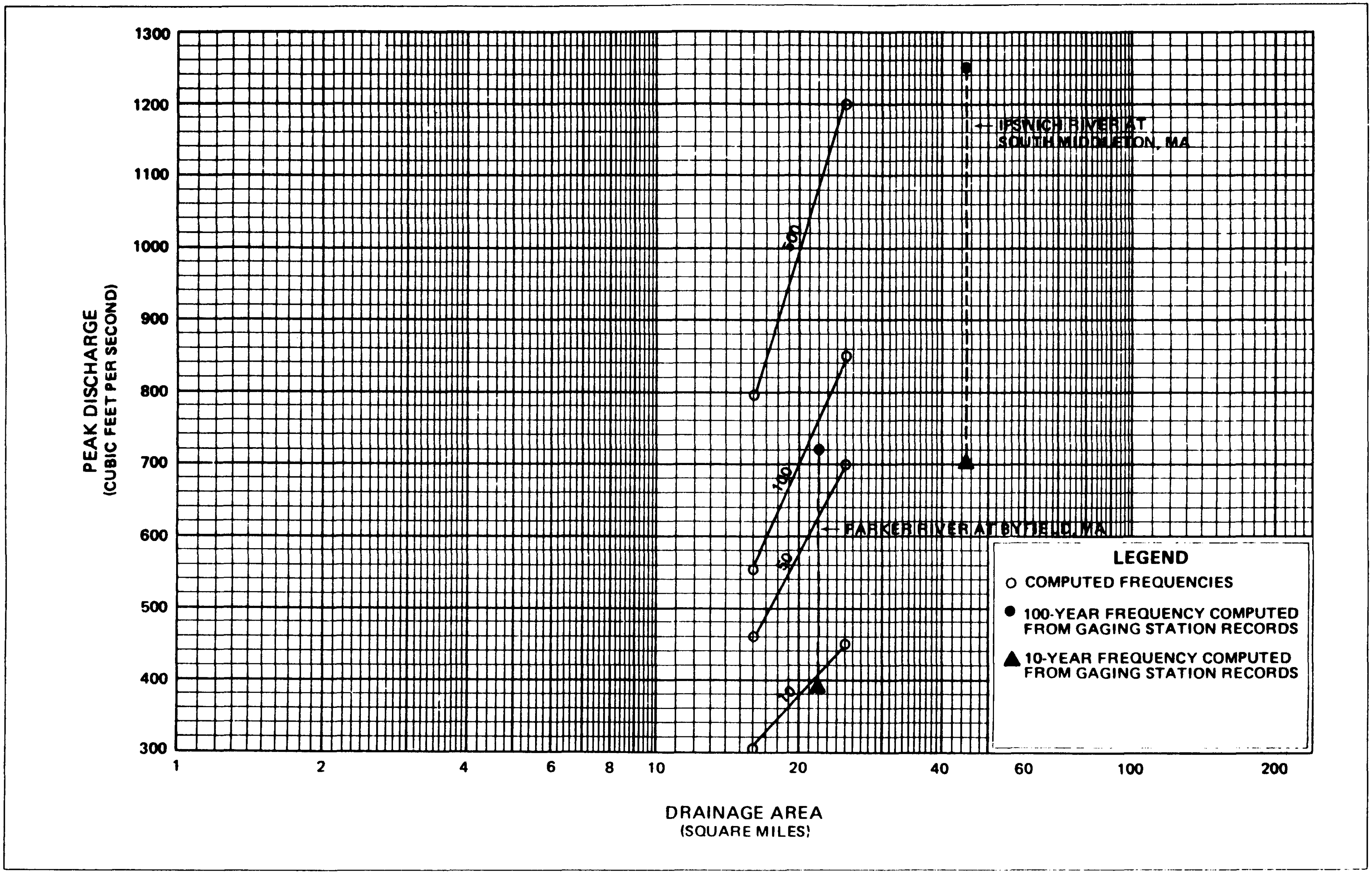


Figure 8

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOWN OF DARTMOUTH, MA

BRISTOL COUNTY, MA (ALL JURISDICTIONS)

FREQUENCY-DISCHARGE, DRAINAGE AREA CURVES

PASKAMANSET RIVER

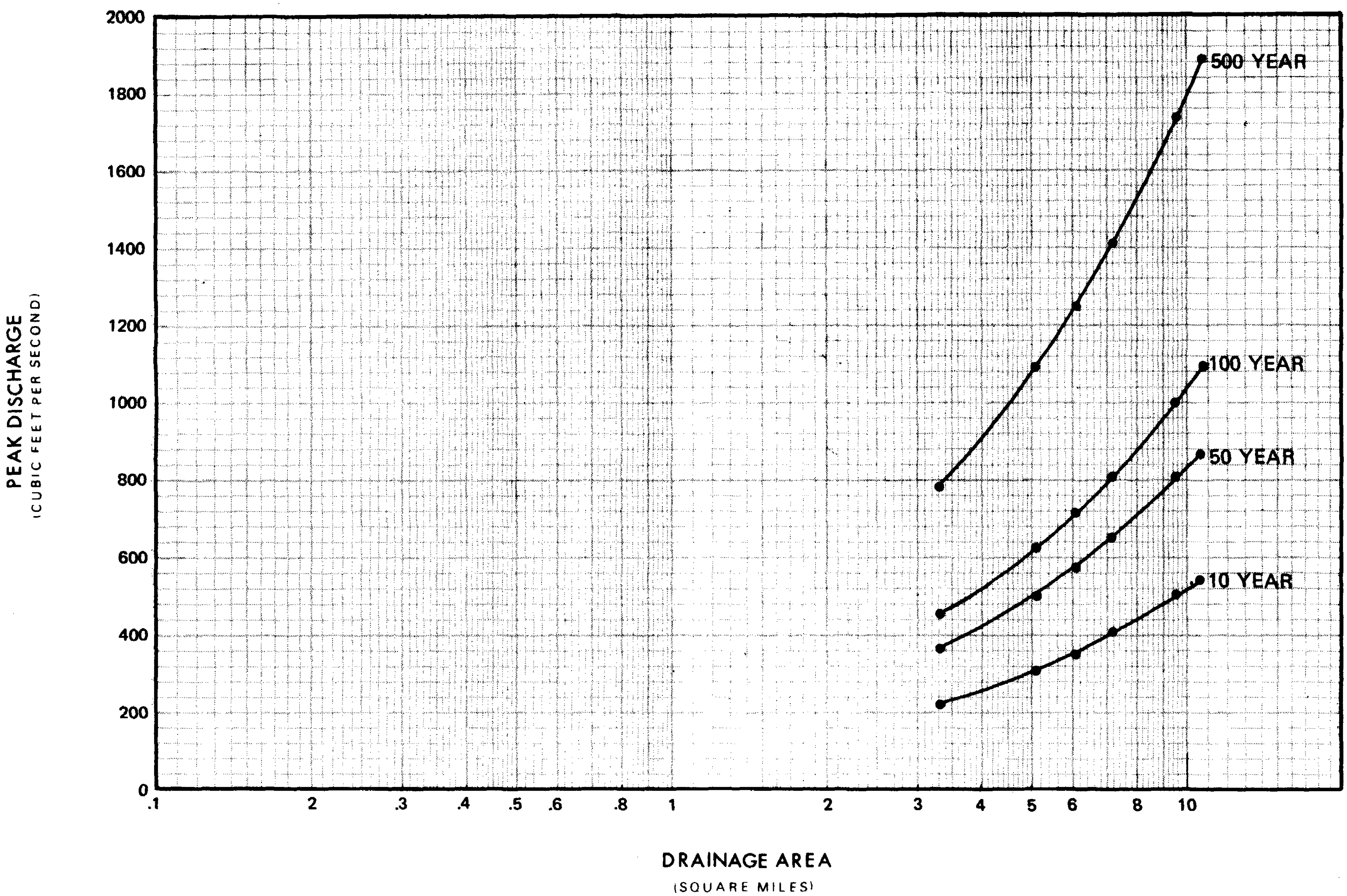


Figure 9

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

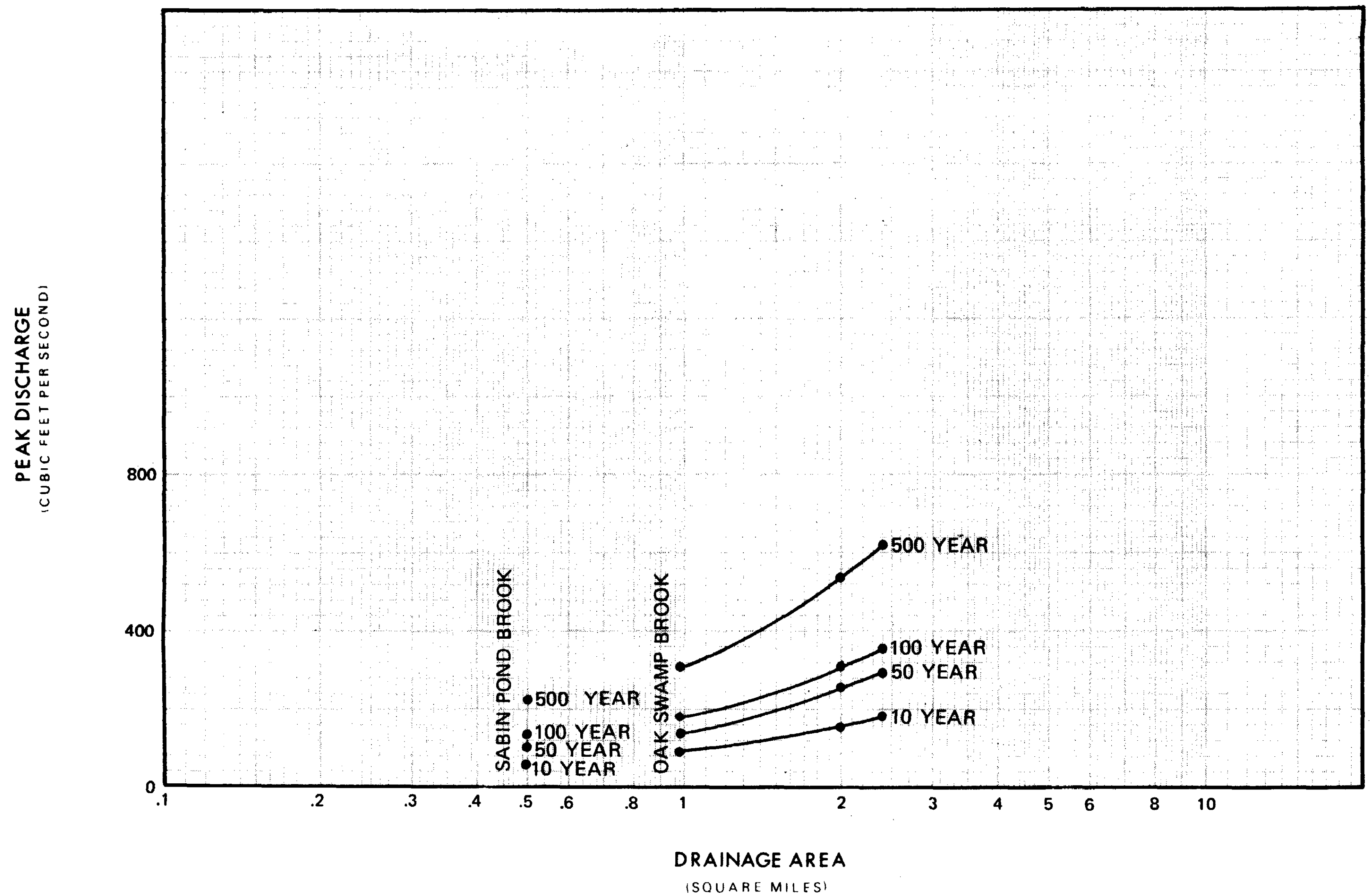


Figure 10

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

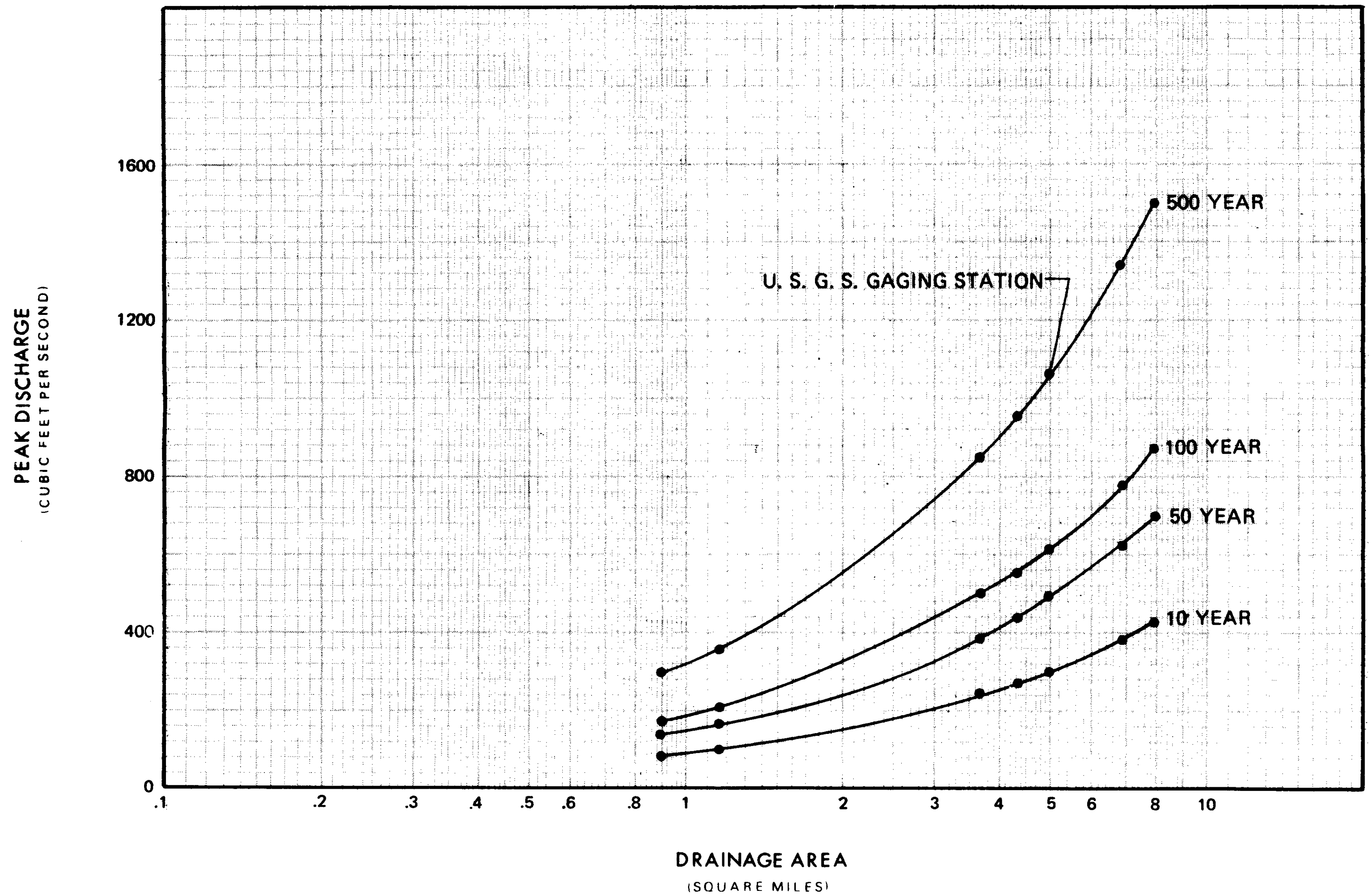


Figure 11

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.

Precountywide Analyses

Starting water-surface elevations for Ten Mile River in Attleboro and North Attleborough and Chartley Brook in Attleboro were obtained by rating curves. Lake Como, Rocklawn Avenue Stream starting water-surface elevations were obtained from Seven Mile River backwater in Attleboro. All other starting water-surface elevations in Attleboro and North Attleborough were obtained from Ten Mile River backwater. Stevens Swamp water-surface elevations are controlled by backwater from the Seven 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, TR-61 (Reference 42). 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 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 43 and 44) 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 approximately1-percent-annual-chance.

In Dartmouth, water-surface elevations of floods of the selected recurrence intervals were computed using the USACE HEC-2 step-backwater computer program (Reference 45). 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 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 46 and 47). 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 48). 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 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 49). The hydraulic analysis was performed by utilizing the USGS step-backwater program E432 (Reference 50). 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 51). 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 Fall River and Somerset, Massachusetts (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 43, 44, 52, and 53) in the area.

In both the August 3, 1981 and May 16, 1995 precountywide Easton FISs, water-surface elevations of floods of the selected recurrence intervals were computed using the USGS E431 step-backwater computer program (Reference 50). 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 precountywide Easton FIS, water-surface profiles for Gowards Brook were developed using the USACE HEC-2 step-backwater computer program (Reference 54). 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 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 49). 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 55) 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 Freetown lies within the 1-percent-annual-chance floodplain determined for the Assonet River.

Water-surface elevations of floods in Mansfield were computed through use of the USACE HEC-2 step-backwater computer program (Reference 56). For the Wading River at the gage location at Balcolm Street, the computed profiles agree with recorded flood elevations. Starting elevations for the study streams were developed by the slope-area method.

In 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 57). 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 50). 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 58). 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 Raynham were computed through the use of the USGS E431 step-backwater computer program (Reference 50). Starting water-surface elevations for the Taunton River were obtained from the June 18, 1987, precountywide 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 Rehoboth were computed through use of the HEC-2 step-backwater computer program (Reference 46). Flooding along the lower Palmer River in 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 88) 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 88), 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 Bridge 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 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. 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, precountywide Rehoboth FIS.

In Seekonk, starting water-surface elevations for the Ten Mile River, 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 42). The water-surface elevation for downstream of School Street on the Runnins River was obtained from the East Providence, Rhode Island, FIS (Reference 59). 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 60) and an analysis of the 1-percent-annual-chance flood stages developed by the watershed models along the streams studied in detail.

In Swansea, water-surface elevations of floods of the selected recurrence intervals were computed using the USACE HEC-2 step-backwater computer program (Reference 46). 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 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 61). Water-surface elevations of floods of the selected recurrence intervals for the Taunton River, the Three Mile River, the Three Mile River - West Channel, and the Mill River were computed using the USGS E431 computer program (Reference 50). Water-surface elevations for the Segreganset River and Cobb Brook were computed using the COE HEC-2 step-backwater computer program (Reference 58). 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 50). 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 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 62). 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. Table 7, "Manning's "n" values" shows the channel and overbank "n" values for the streams studied by detailed methods:

TABLE 7 – MANNING'S "n" VALUES

Flooding Source	Channel "n"	Overbanks
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
Buttonwood Brook West	0.035	0.100
Canoe River	0.023-0.080	0.023-0.100
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
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

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

Flooding Source	Channel "n"	Overbanks
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
Seven Mile River	0.03-0.04	0.06-0.09
Speedway Brook	0.03-0.04	0.06-0.09
Sunken Brook	0.03-0.06	0.05-0.1
Taunton River (Raynham)	0.035-0.065	0.080-0.150
Taunton River (Taunton)	0.030-0.065	0.060-0.300
Ten Mile River	0.03-0.04	0.06-0.09
Ten Mile River (Seekonk)	0.035-0.05	0.05-0.11
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	0.035-0.080	0.090-0.100
Whiting Pond Bypass	0.03-0.04	0.06-0.09
Whitman Brook	0.012-0.045	0.050-0.140

Countywide Analyses

For the July 7, 2009 and the 2012 Coastal Study Update countywide revisions, no new hydraulic analyses were conducted.

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 63). 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.

<u>Precountywide Analysis</u>

Prior to the countywide updates, coastal hydrologic and hydraulic analyses were carried out to estimate the 1-percent-annual-chance storm characteristics in Fall River, Somerset, and Swansea and areas of Fairhaven and New Bedford behind the hurricane barrier. As part of the July 7, 2009 countywide study, new coastal analyses were performed for the communities of Dartmouth, Fairhaven, New Bedford 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 Fall River, Somerset, and Swansea have been superseded by the 2012 Coastal Study Update. A description of the revised analyses is presented in the subsequent 2012 Coastal Study Update 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 Fall River and Somerset, Massachusetts (Reference 43). 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 43, 44, 52 and 53). 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 Fairhaven and 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 the distance which runoff travels in unit time is equal to the longest length of travel divided by the time of concentration (Reference 64). 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 65 and 66). 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 67). Runoff volumes were calculated by dividing the 1.5-square mile contributory drainage basin into three subbasins: 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 Fairhaven and 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 Fall River, Somerset, and Swansea, using flood level profiles developed by statistically analyzing high-water elevations in the study area (References 14 and 68). Information evaluated at Fall River, Somerset, 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 43). 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 69). 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 Somerset, Swansea, and Fall River is shown in Table 8. Data is based on tide stage data at Newport, Rhode Island, as related to New Bedford, Massachusetts.

TABLE 8 – STAGE-FREQUENCY DATA

Hurricane or Storm	Date	Elevation (NAVD[1])
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

[1] North American Vertical Datum of 1988

TABLE 8 – STAGE-FREQUENCY DATA - continued

Hurricane or Storm	Date	Elevation (NAVD[1])
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
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

[1] North American Vertical Datum of 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

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD[1]) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
ACHUSHNET 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
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

[1] North American Vertical Datum of 1988
*Data Not Available

TABLE 9 – PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS – continued

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD[1])			
	10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
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
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

[1] North American Vertical Datum of 1988
*Data Not Available

TABLE 9 – PRECOUNTYWIDE SUMMARY OF STILLWATER ELEVATIONS – continued

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD[1]) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT
TAUNTON RIVER - continued				
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
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

[1] North American Vertical Datum of 1988
*Data Not Available

The methodology for analyzing wave heights and corresponding wave crest elevations was developed by the National Academy of Sciences (NAS; Reference 70). 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:

$$Hb = 0.78d$$

Where Hb 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:

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

Where Zw, 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. Hb 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 Ht 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 70).

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 71). 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 70 and 72).

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 communities of Dartmouth, Fairhaven, New Bedford, 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 73) of the Guidelines and Specifications as well as the "Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update" (Reference 74).

For the revised communities, published values in the Tidal Flood Survey (Reference 75) 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 the 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

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD[1])			
	10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT*
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

[1] North American Vertical Datum of 1988
* Extrapolated from USACE data

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

FLOODING SOURCE AND LOCATION	ELEVATION (feet NAVD[1]) 10-PERCENT	2-PERCENT	1-PERCENT	0.2-PERCENT*
BUZZARDS BAY - continued				
Acushnet River	4.4	4.7	5.5**	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/Apponag. 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
Upstream End of East Branch of 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

[1] North American Vertical Datum of 1988
* Extrapolated from USACE data
** Computed from the City of New Bedford and Town of Fairhaven, May 2011 Hurricane Dike and Barrier System Accreditation Package

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. Datum conversion factor of +0.15 was applied to the data in the Tidal Flood Survey.

Wave setup along the open coast areas of Dartmouth, Fairhaven, New Bedford, and Westport was calculated using the procedures detailed in the “Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update”, (Reference 74). 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 Dartmouth, Fairhaven, New Bedford, 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
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

[1] Because 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
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
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

[1] Because 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
9	The transect is located along the eastern shoreline of Sconticut Neck at a point approximately 250 feet south of Island View Road extending to the northwest towards Sconticut Neck Road.	12	17.91
10	The transect is located along the southern end of Sconticut 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 Sconticut Neck Road.	12	23.23
12	The transect is located along Chambers Street extending from the Buzzards Bay shoreline east to Sconticut 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
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

[1] Because 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
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

[1] Because 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
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

[1] Because 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
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
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

[1] Because 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
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
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

[1] Because 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
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
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

[1] Because 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
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
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

[1] Because 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

TRANSECT	LOCATION	1-PERCENT-ANNUAL-CHANCE STILLWATER	MAXIMUM 1-PERCENT-ANNUAL-CHANCE WAVE CREST[1]
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
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

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

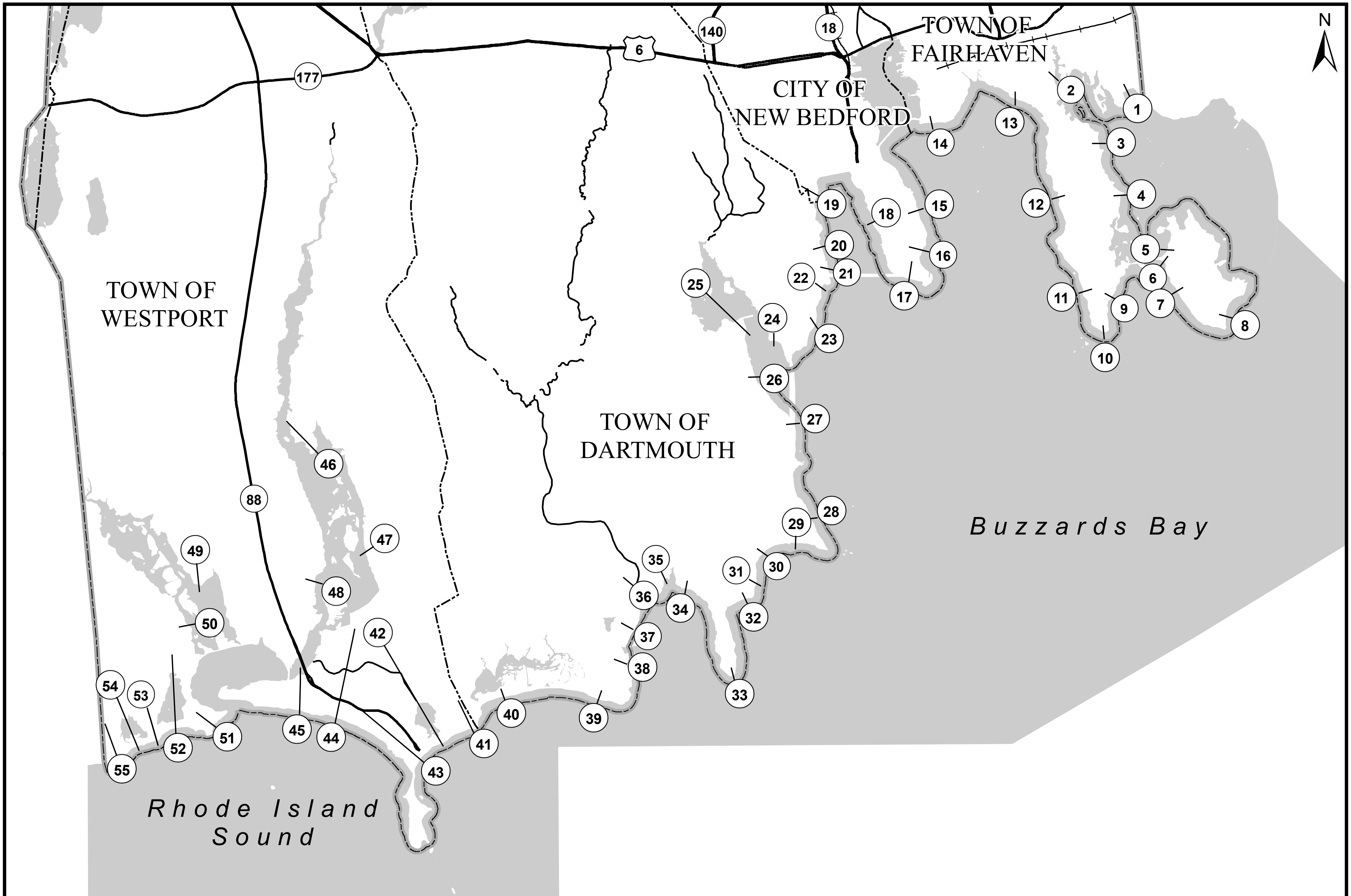


FIGURE 12

FEDERAL EMERGENCY MANAGEMENT AGENCY

BRISTOL COUNTY, MA

(ALL JURISDICTIONS)

APPROXIMATE SCALE

0 9,200 18,400 27,600 36,800 Feet

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 ft 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 74). WHAFIS 3.0 was used to predict wave heights.

The FEMA Guidelines (Reference 73) allow for the following methods to be used to determine wave runup: RUNUP 2.0; “Technical Advisory Committee for Water Retaining Structures” (Reference 76); Automated Coastal Engineering System (ACES); and the Shore Protection Manual (Reference 77). 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 Dartmouth, Fairhaven, New Bedford, 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 78). Depths below mean low water were determined from National Ocean Survey Coastal Charts (Reference 79). The land-use and land cover data were obtained by field surveys and aerial photographs (Reference 80).

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

FLOODING SOURCE	STILLWATER ELEVATIONS (feet NAVD[1]) 10-PERCENT-ANNUAL CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD)[1]
BUZZARDS BAY						
1	6.8	10.4	12.2	15.8	VE	15-18
					AE	13-15
2	6.8	10.4	12.2	15.8	VE	15-19
					AE	13-15
3	6.8	10.4	12.2	15.8	VE	15-19
					AE	13-15
4	6.8	10.4	12.2	15.8	VE	15-19
					AE	13-15
5	6.8	10.4	12.2	15.8	VE	15-17
					AE	13-15
6	6.7	10.2	12.0	15.7	VE	15-19
					AE	12-15
7	6.7	10.2	12.0	15.7	VE	18-23
					AE	18
8	6.7	10.2	12.0	15.7	VE	18-23
					AE	15-17
9	6.7	10.2	12.0	15.7	VE	15-18
10	6.7	10.2	12.0	15.7	VE	18-23
					AE	16-17
11	6.7	10.2	12.0	15.7	VE	19-23

[1]North American Vertical Datum of 1988

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

FLOODING SOURCE	STILLWATER ELEVATIONS (feet NAVD[1]) 10-PERCENT-ANNUAL CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD)[1]
BUZZARDS BAY – cont.						
12	6.6	10.1	11.9	15.5	VE	17-23
					AE	15-17
13	6.6	10.1	11.9	15.5	VE	15-19
					AE	13-15
14	6.6	10.1	11.9	15.5	VE	19-15
					AE	13-15
15	6.3	9.8	11.7	15.3	VE	15-18
					AE	12-14
16	6.3	9.8	11.7	15.3	VE	18-24
					AE	17
17	6.3	9.8	11.7	15.3	VE	20-26
					AE	12
18	6.3	9.8	11.7	15.3	VE	15-18
					AE	12-15
19	6.3	9.8	11.7	15.3	VE	15-19
					AE	13-15
20	6.2	9.7	11.7	15.2	VE	23
21	6.2	9.7	11.7	15.2	VE	18 - 24
					AE	16-18

[1]North American Vertical Datum of 1988

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

FLOODING SOURCE	STILLWATER ELEVATIONS (feet NAVD[1]) 10-PERCENT-ANNUAL CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD)[1]
BUZZARDS BAY – cont.						
22	6.2	9.7	11.7	15.2	VE	18-24
					AE	15-17
23	6.2	9.7	11.7	15.2	VE	18-24
					AE	18
24	6.0	9.6	11.7	15.2	VE	15-19
					AE	13-15
25	6.0	9.6	11.7	15.2	VE	17-23
					AE	12-13
26	6.0	9.6	11.7	15.2	VE	17-23
					AE	15-17
27	6.0	9.6	11.7	15.2	VE	26
					AE	16-18
28	6.0	9.6	11.7	15.2	VE	24
					AE	16-18
29	6.0	9.6	11.7	15.2	VE	18-24
					AE	15-17
30	5.9	9.5	11.7	15.3	VE	17-23
					AE	15-17
31	5.9	9.5	11.7	15.3	VE	18-24
					AE	16-18

[1]North American Vertical Datum of 1988

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

FLOODING SOURCE	STILLWATER ELEVATIONS (feet NAVD[1]) 10-PERCENT-ANNUAL CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD)[1]
BUZZARDS BAY – cont.						
32	5.9	9.5	11.7	15.3	VE	17-22
					AE	15-17
33	5.9	9.5	11.7	15.3	VE	25
34	5.9	9.5	11.7	15.3	VE	14-23
					AE	12-14
35	5.9	9.5	11.7	15.3	VE	17-22
					AE	12-17
36	5.9	9.5	11.7	15.3	VE	16-22
					AE	12-16
37	5.9	9.5	11.7	15.3	VE	15-22
					AE	12-14
38	5.8	9.5	11.7	15.3	VE	17-23
					AE	15-16
39	5.8	9.5	11.7	15.3	VE	18-23
					AE	15-17
40	5.8	9.6	11.7	15.5	VE	17-22
					AE	15-17

[1]North American Vertical Datum of 1988

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

FLOODING SOURCE	STILLWATER ELEVATIONS (feet NAVD[1]) 10-PERCENT-ANNUAL CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD)[1]
RHODE ISLAND SOUND						
41	5.7	9.6	11.7	15.5	VE	16-24
					AE	15-16
42	5.7	9.6	11.7	15.5	VE	18-24
					AE	15-17
43	5.7	9.6	11.7	15.5	VE	14-23
					AE	12-14
44	5.7	9.6	11.7	15.5	VE	14-24
					AE	12-14
45	5.7	9.6	11.7	15.5	VE	17-23
					AE	12-17
46	5.7	9.6	11.7	15.5	VE	16
47	5.7	9.6	11.7	15.5	VE	14-15
					AE	12-14
48	5.7	9.6	11.7	15.5	VE	15-16
					AE	12-14
49	5.7	9.6	11.7	15.5	VE	14-17
					AE	12-14
50	5.7	9.6	11.7	15.5	VE	14-16
					AE	12-14

[1]North American Vertical Datum of 1988

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

FLOODING SOURCE	STILLWATER ELEVATIONS (feet NAVD[1]) 10-PERCENT-ANNUAL CHANCE	2-PERCENT-ANNUAL-CHANCE	1-PERCENT-ANNUAL-CHANCE	0.2-PERCENT-ANNUAL-CHANCE	ZONE	BASE FLOOD ELEVATION (feet NAVD)[1]
RHODE ISLAND SOUND – cont.						
51	5.7	9.6	11.7	15.5	VE	18-24
					AE	15-17
52	5.7	9.6	11.7	15.5	VE	18-24
					AE	16-18
53	5.7	9.6	11.7	15.5	VE	19-26
					AE	17-19
54	5.7	9.6	11.7	15.5	VE	18-24
					AE	15-17
55	5.7	9.6	11.7	15.5	VE	21-23

[1]North American Vertical Datum of 1988

2012 Coastal Study Update

As part of this countywide update, revised coastal analyses were performed for the open water flooding sources along Mount Hope Bay in the communities of Fall River, Somerset, and Swansea, as well as along the Taunton River in the communities of Berkley, Dighton, Fall River, Freetown, and Somerset. Portions of Heath Brook, the Palmer River, and the Runnins River in the communities 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 73) of the Guidelines and Specifications as well as the "Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update" (Reference 74).

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 81). Note that stillwater elevations upstream of Interstate 195 on the Palmer River resulted back to 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 2012 COASTAL UPDATE STILLWATER ELEVATIONS

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

[1]From Bristol County, RI Analysis
[2]From Precountywide Analysis
[3]North American Vertical Datum of 1988

Offshore (deepwater) wave heights, wave setup, and wave runup were calculated for each transect using Mathcad (Reference 82) sheets developed by STARR to apply methodologies from the USACE's Coastal Engineering Manual (Reference 83). 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 72). 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 2012 Coastal Study Update are shown in Table 14. The locations of these transects are depicted in Figure 13.

TABLE 14 – 2012 COASTAL STUDY UPDATE TRANSECT DESCRIPTIONS

		Elevation (Feet NAVD[2])		
Transect	Location	Stillwater, 1-Percent-Annual-Chance	Max Wave Crest, 1-Percent-Annual-Chance[1]	V Zone Mapping Method
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 the confluence with the Cole River	13.9	21.3	Runup
57	The transect is located at the shoreline of Mt. Hope Bay, in the Town of Swansea, from the confluence with the Cole River to Cole Street	13.9	22.0	Breaking Wave Ht
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	Wave Overtopping Splash Zone
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

[1] Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[2] North American Vertical Datum of 1988

TABLE 14 – 2012 COASTAL STUDY UPDATE TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (Feet NAVD[2]) Stillwater, 1-Percent-Annual-Chance	Max Wave Crest, 1-Percent-Annual-Chance[1]	V Zone Mapping Method
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 Mattapoisett Road	13.9	22.9	Wave Overtopping Splash Zone
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 Mattapoisett 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

[1] Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[2] North American Vertical Datum of 1988

TABLE 14 – 2012 COASTAL STUDY UPDATE TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (Feet NAVD[2]) Stillwater, 1-Percent-Annual-Chance	Max Wave Crest, 1-Percent-Annual-Chance[1]	V Zone Mapping Method
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
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 Essex 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 Essex 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 Sprague Street (extended)	13.9	21.7	Wave Overtopping Splash Zone
73	The transect is located at the shoreline of Mt. Hope Bay, in the City of Fall River, from Sprague Street (extended) to Riverview Street (extended)	13.9	22.3	Runup

[1] Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[2] North American Vertical Datum of 1988

TABLE 14 – 2012 COASTAL STUDY UPDATE TRANSECT DESCRIPTIONS - continued

Transect	Location	Elevation (Feet NAVD[2]) Stillwater, 1-Percent-Annual-Chance	Elevation (Feet NAVD[2]) Max Wave Crest, 1-Percent-Annual-Chance[1]	V Zone Mapping Method
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

[1] Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[2] North American Vertical Datum of 1988

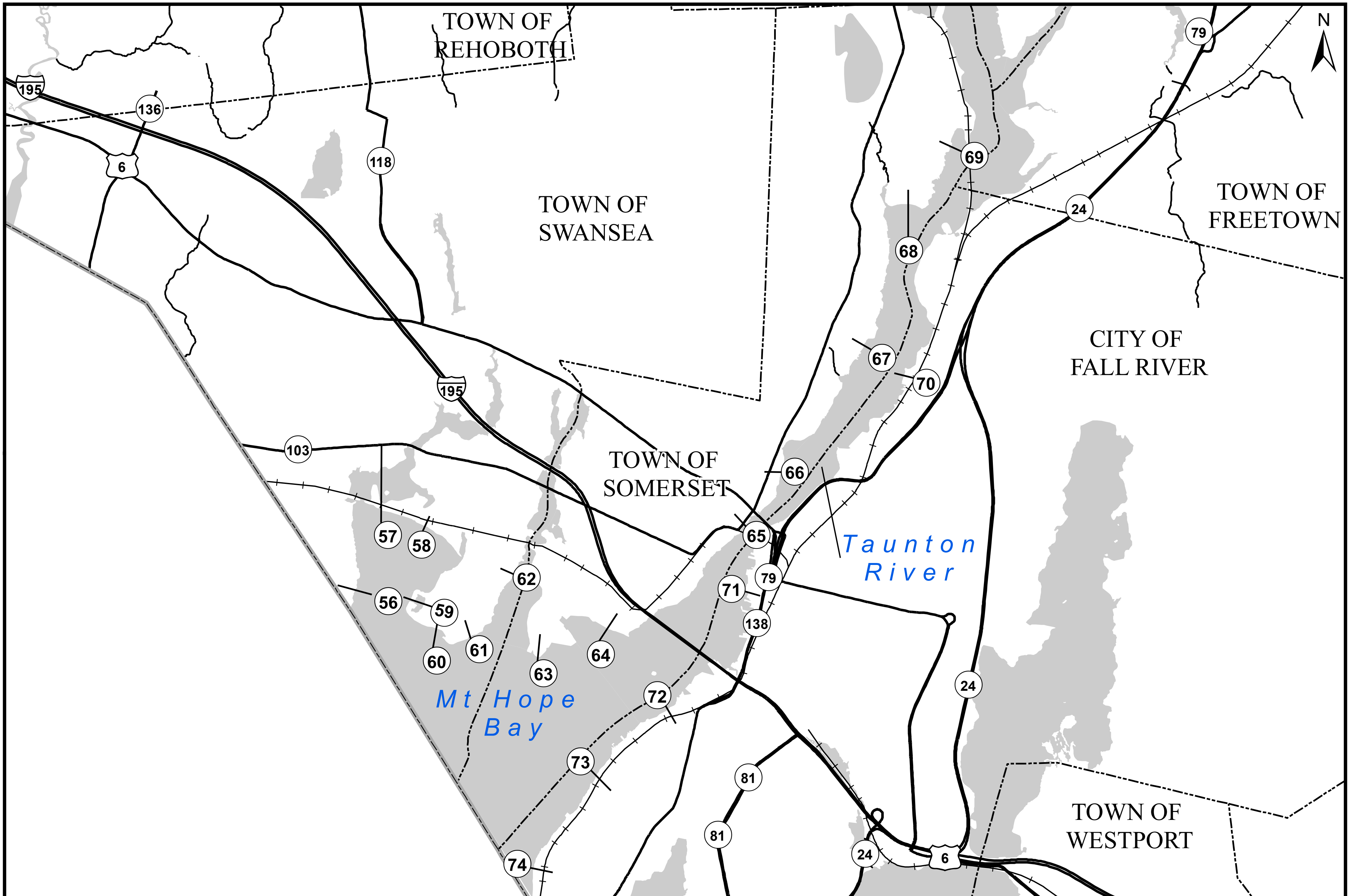


FIGURE 13

FEDERAL EMERGENCY MANAGEMENT AGENCY

BRISTOL COUNTY, MA

(ALL JURISDICTIONS)

APPROXIMATE SCALE

0 5,900 11,800 17,700 23,600 Feet

TRANSECT LOCATION MAP

For the 2012 Coastal Study Update, 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 ft 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 (Hmo) and peak wave period (Tp) 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 84). The model accepts a spectral form of the wave as an input condition and provides Hmo and Tp 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 85), 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 86), within the Coastal Hazard Analysis Modeling Program (CHAMP), Version 2.0 (Reference 87), 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 88).

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 70). 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 77). 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. 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 89). 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 2012 Coastal Study Update; 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 – 2012 COASTAL STUDY UPDATE TRANSECT DATA

Flooding Source and Transect Number	Stillwater elevations (feet NAVD[3]) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance	Total Water Level[1] 1- percent annual-chance	Zone	Base Flood Elevation[2] (Feet NAVD[3])
MOUNT HOPE BAY							
Transect 56	8.4	12.3	13.9	17.6	14.8	VE	17
						AE	15-16
Transect 57	8.4	12.3	13.9	17.6	16.5	VE	19
						AE	17
Transect 58	8.4	12.3	13.9	17.6	15.0	VE	17
						AE	*
Transect 59	8.4	12.3	13.9	17.6	15.0	VE	18
						AE	17
Transect 60	8.4	12.3	13.9	17.6	15.8	VE	21
						AE	*
Transect 61	8.4	12.3	13.9	17.6	15.8	VE	21
						AE	*
Transect 62	8.4	12.3	13.9	17.6	15.5	VE	18
						AE	*
Transect 63	8.4	12.3	13.9	17.6	15.4	VE	18
						AE	*
Transect 73	8.4	12.3	13.9	17.6	16.1	VE	24
						AE	*
Transect 74	8.4	12.3	13.9	17.6	15.1	VE	20
						AE	*
TAUNTON RIVER							
Transect 64	8.4	12.3	13.9	17.6	16.6	VE	19
						AE	17
Transect 65	8.4	12.3	13.9	17.6	15.5	VE	18
						AE	15-16
Transect 66	8.4	12.3	13.9	17.6	14.4	VE	16
						AE	15

[1] Including stillwater elevation and effects of wave setup.
[2] Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[3] North American Vertical Datum of 1988
* Data not available

TABLE 15 – 2012 COASTAL STUDY UPDATE TRANSECT DATA - continued

Flooding Source and Transect Number	Stillwater elevations (feet NAVD[3]) 10-percent-annual-chance	2-percent-annual-chance	1-percent-annual-chance	0.2-percent-annual-chance	Total Water Level[1] 1- percent annual-chance	Zone	Base Flood Elevation[2] (Feet NAVD[3])
TAUNTON RIVER - continued							
Transect 67	8.4	12.3	13.9	17.6	14.6	VE	17
						AE	*
Transect 68	8.4	12.3	13.9	17.6	15.1	VE	19
						AE	*
Transect 69	8.4	12.3	13.9	17.6	14.9	VE	18
						AE	*
Transect 70	8.4	12.3	13.9	17.6	14.7	VE	17
						AE	*
Transect 71	8.4	12.3	13.9	17.6	15.2	VE	17
						AE	15
Transect 72	8.4	12.3	13.9	17.6	15.6	VE	24
						AE	*

[1] Including stillwater elevation and effects of wave setup.
[2] Because of map scale limitations, the maximum wave elevation may not be shown on the FIRM.
[3] North American Vertical Datum of 1988
* 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.

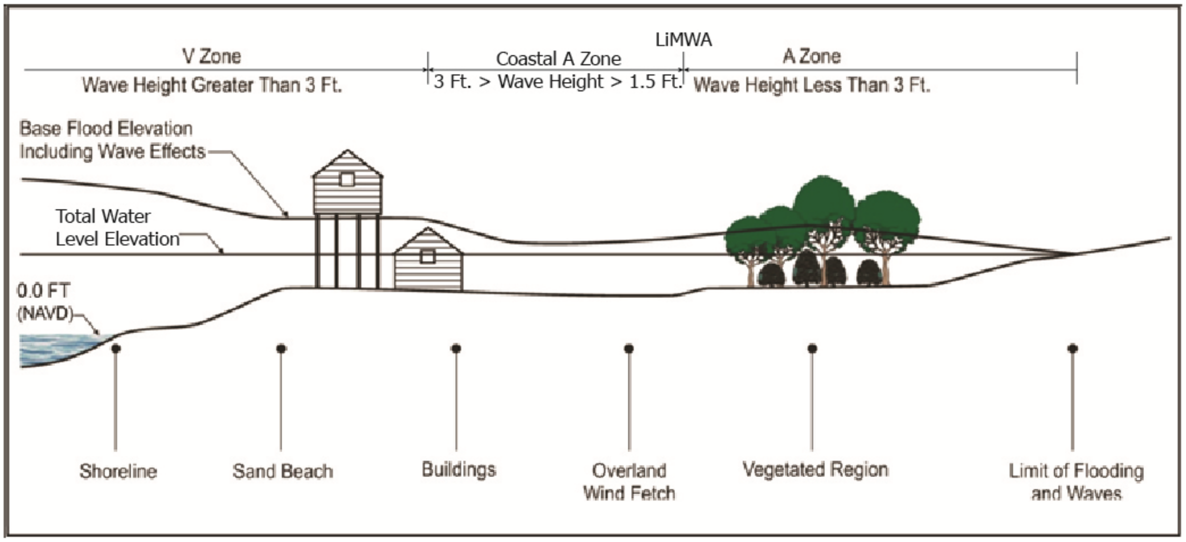


Figure 14 - TRANSECT SCHEMATIC

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 29). With the completion of the North American Vertical Datum of 1988 (NAVD 88), many FIS reports and FIRMs are now prepared using NAVD 88 as the referenced vertical datum.

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

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

NGS Information Services
NOAA, N/NGS12
National Geodetic Survey
SSMC-3, #9202

1315 East-West Highway
Silver Spring, Maryland 20910-3282
(301) 713-3242

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

The BFEs shown on the FIRM represent whole-foot rounded values. For example, a BFE of 102.4 will appear as 102 on the FIRM and 102.6 will appear as 103. Therefore, users that wish to convert the elevations in this FIS to NGVD 29 should apply the stated conversion factor to elevations shown on the Flood Profiles and supporting data tables in the FIS report, which are shown at a minimum to the nearest 0.1 foot.

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