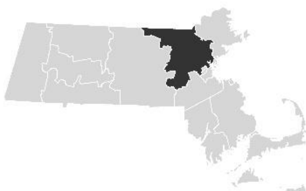
FLOOD INSURANCE STUDY

FEDERAL EMERGENCY MANAGEMENT AGENCY

VOLUME 4 OF 15



MIDDLESEX COUNTY, MASSACHUSETTS

(ALL JURISDICTIONS)

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

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FEMA

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Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Spencer Brook	Confluence with Assabet River	Angiers Pond	Discharge-frequency relationships	HEC-2 (USACE 1984)	2/1/1984	AE w/ Floodway	Discharge-frequency relationships and/or discharge-frequency-drainage area relationships were developed using hydrologic methods (SCS 1972a, SCS 1973a, Johnson and Tasker 1974). Cross-sections were from field surveys and topographic maps (Sewall 1976). Structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from known water-surface elevations on Assabet River.
Spot Pond Brook	Entrance to underground drainage above Cutter Street	Spot Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Spring Brook	Confluence with Shawsheen River	Approximately 2,300 feet above Alcott Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	4/1/1979	AE w/ Floodway	Due to large standard errors from the regional regression equations (Wandle 1977), discharges were also computed by rainfall-runoff techniques based on synthetic triangular unit hydrographs (SCS 1972a), and final discharges were a blend of the two methods resulting in a smooth curve. Cross-sections were from field surveys and topographic maps (Sewall 1976). Structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from the slope-area method.
Spring Brook pond	Entire shoreline	Entire shoreline	none	none	6/4/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (184.8 feet NAVD88).
Spruce Swamp Brook	Confluence with Catacoonamug Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Spruce Swamp Brook pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (378.2 feet NAVD88).
Squannacook River	Confluence with Nashua River	Headwaters at confluence of Pearl Hill Brook and Walker Brook 2	Rainfall-runoff routing (SCS 1972a)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	In the Town of Groton, discharge statistics were obtained by log-Pearson type III analyses on the streamgage on Squannacook River near West Groton (with data from 1949). Flood discharges were then routed through the river using runoff routing (SCS 1972a). Overbank portions of cross-sections were from topographic maps (Teledyne 1977, MADNR 1973). Underwater portions and all structures were from field surveys. Roughness factors were from field inspections and aerial imagery (Teledyne 1977). Starting water-surface elevations were from known water-surface elevations on Nashua River. The hydraulic model was calibrated to high-water marks from the historic flood of March 1936 (Bogart 1960, Townsend 1972).
Squannacook River Tributary B	Confluence with Squannacook River	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Squannacook River Tributary B1	Confluence with Squannacook River Tributary B	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Squannacook River Tributary C	Confluence with Squannacook River	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Squannacook River Tributary D	Confluence with Squannacook River	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Star House Lane swamp 1	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (247.7 feet NAVD88).
Star House Lane swamp 2	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (243.1 feet NAVD88).
Star House Lane swamp 3	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (257.9 feet NAVD88).
Stewart Brook	Confluence with Gulf Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Stewart Brook lower swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (412.1 feet NAVD88).

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Stewart Brook upper swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (422.3 feet NAVD88).

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Stony Brook	Confluence with Sudbury River	County boundary	Regression equations	HEC-RAS 4.1	10/1/2012	AE	Rural regression equations (Wandle 1983) were used to compute discharges for most locations. Locations with more than 10% impervious area used urban regression equations (Sauer et al. 1983) instead. Streamgage statistics, updated through 2010 using log-Pearson type III analysis (IACWD 1982) and either weighted skew coefficients or station skew (if gages were affected by urbanization or regulation), were compared to the statistics for the same gages from the 1983 reports to determine if the regression equations would predict well discharges from additional periods of record. The base-flood-frequency discharges increased 123% from 1983 to 2010, on average, which was applied as an adjustment factor to results from the regression equations. Computed discharges were reduced below flood-storage reservoirs based on average reduction of outflow compared to inflow as determined by flood-routing computations. Flood-routing computations were obtained from the NRCS or are original to this study. A HEC-HMS rainfall-runoff model was used to validate the results of the regression equations. The HEC-HMS model used the NRCS Curve Number method to compute runoff and the NRCS Unit Hydrograph method to transform it. The meteorological input was from a type III storm. Five-minute time-steps were used. The HEC-HMS model was calibrated to precipitation and streamflow data from the 2007 storm event. The regression equation discharges were adjusted at several locations based on comparison to the HEC-HMS model. Cross-sections were from a blend of field survey data and lidar data. Structures were from field surveys or, where available for large structures, construction plans. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from normal depth.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Stony Brook 1	Confluence with Charles River	Lincoln/ Weston corporate limits	Unit hydrograph theory	HEC-2 (USACE 1984)	5/1/1978	AE w/ Floodway	Unit hydrograph theory was selected because the basin is ungaged, has natural storage flow regulation, and has high urbanization. Synthetic triangular unit hydrographs were developed using data from Wandle 1977, adjusted for slopes and local inflows, and compared to results of regression equations (Wandle 1977). Cross-section data were from the City of Waltham (Waltham 1976), photogrammetric maps in Weston (NE Air 1977), or field surveys otherwise. Structures were from field surveys. Roughness factors were informed by field inspection and chosen from Chow 1959. Starting water-surface elevations were from the slope-area method in Waltham and rating curves in Weston.
Stony Brook 1	Approximately 0.1 mile south of Tower Road	Sandy Pond	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	3/1/1983	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Cross-sections were from field surveys and topographic maps (USGS various). Structures were from field surveys. Roughness factors were from field observations and engineering judgment.
Stony Brook 1	Lincoln/ Weston corporate limits	Approximately 0.1 mile south of Tower Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Stony Brook 1 Tributary A	Confluence with Stony Brook 1	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Stony Brook 1 Tributary B	Confluence with Stony Brook 1	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Stony Brook 1 Tributary B1	Confluence with Stony Brook 1 Tributary B	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Stony Brook 1 Tributary C	Confluence with Stony Brook 1	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Stony Brook 1 Tributary D	Confluence with Stony Brook 1	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Stony Brook 1 Tributary E	Confluence with Stony Brook 1	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Stony Brook 1 Tributary F	Confluence with Stony Brook 1	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Stony Brook 1 Tributary F1	Confluence with Stony Brook 1 Tributary F	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Stony Brook 2	Confluence with Merrimack River	Forge Pond	Regional frequency method (SCS 1972a)	HEC-2 (USACE 1984)	1/1/1981	AE w/ Floodway	Water-surface elevations for Forge Pond are determined by the elevations for this model at the upstream end. In the Town of Chelmsford, discharges were computed by applying streamgage statistics from USGS streamgage 01101000 (Parker River at Byfield, period of record through 1974) to Stony Brook based on flow-frequency relationships derived from similarities of basin characteristics. In the Town of Westford, the downstream discharges from Chelmsford were transferred upstream using discharge-drainage area curves developed with consideration of the Forge Pond/Beaver Brook wetland storage areas (Benson 1962). Overbank portions of cross-sections were from topographic maps (Teledyne 1979). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and engineering judgment. Starting water-surface elevations for the Westford reach were from a stage-discharge rating curve at a dam approximately 1,000 feet below the corporate limits. Starting water-surface elevations for the Chelmsford reach were from normal depth (for 10- and 2-percent-annual-chance profiles) and from known water-surface elevations on Merrimack River (1- and 0.2-percent-annual-chance profiles).
Stony Brook 2	Meadowbrook Road	Chelmsford/ Westford corporate limits	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Stony Brook 2 Tributary A	Confluence with Stony Brook 2	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Stony Brook 2 Tributary B	Confluence with Stony Brook 2	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Stony Brook Pond	Entire shoreline	Entire shoreline	none	none	4/30/2018	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (161.8 feet NAVD88).
Strawberry Meadow	Confluence with Ipswich River	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Strawberry Meadow Tributary A	Confluence with Strawberry Meadow	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Strawberry Meadow Tributary B	Confluence with Strawberry Meadow	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Strawberry Meadow Tributary B1	Confluence with Strawberry Meadow Tributary B	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Strong Water Brook	Confluence with Shawsheen River	Confluence of Meadow Brook and Marshall Brook	Drainage-area ratio	HEC-2 (USACE 1984)	12/1/1978	AE w/ Floodway	Discharges were calculated using a drainage-area ratio (Johnstone and Cross 1949), with an exponent between 0.5 and 0.8, proportional to discharges computed on Shawsheen River. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Shawsheen River.
Sucker Brook 1 Tributary A	County boundary	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Sucker Brook 2	Confluence with Nissitissit River	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Sucker Brook 2 Tributary A	Confluence with Sucker Brook 2	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Sudbury River	Approximately 600 feet above Lowell Road	Approximately 220 feet above Interstate 495	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ Floodway	Rural regression equations (Wandle 1983) were used to compute discharges for most locations. Locations with more than 10% impervious area used urban regression equations (Sauer et al. 1983) instead. Streamgage statistics, updated through 2010 using log-Pearson type III analysis (IACWD 1982) and either weighted skew coefficients or station skew (if gages were affected by urbanization or regulation), were compared to the statistics for the same gages from the 1983 reports to determine if the regression equations would predict well discharges from additional periods of record. The base-flood-frequency discharges increased 123% from 1983 to 2010, on average, which was applied as an adjustment factor to results from the regression equations. Computed discharges were reduced below flood-storage reservoirs based on average reduction of outflow compared to inflow as determined by flood-routing computations. Flood-routing computations were obtained from the NRCS or are original to this study. A HEC-HMS rainfall-runoff model was used to validate the results of the regression equations. The HEC-HMS model used the NRCS Curve Number method to compute runoff and the NRCS Unit Hydrograph method to transform it. The meteorological input was from a type III storm. Five-minute time-steps were used. The HEC-HMS model was calibrated to precipitation and streamflow data from the 2007 storm event. The regression equation discharges were adjusted at several locations based on comparison to the HEC-HMS model. Cross-sections were from a blend of field survey data and lidar data. Structures were from field surveys or, where available for large structures, construction plans. Geometry for the Saxonville Levee was imported from the HEC-RAS model created for accreditation. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from normal depth.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Sutton Brook	Confluence with Shawsheen River	Tewksbury/ Wilmington corporate limites	Regional relationships (USACE 1972)	HEC-2 (USACE 1984)	12/1/1978	AE w/ Floodway	Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Shawsheen River.
Sutton Brook	Tewksbury/ Wilmington corporate limites	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Sutton Brook Tributary A	Confluence with Sutton Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Swains Pond	Entire shoreline	Entire shoreline	none	none	4/30/2018	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (87.9 feet NAVD88).

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Sweetwater Brook	Confluence with Aberjona River	Approximately 120 feet above Lindenwood Road	HEC-HMS 2.2.2	HEC-RAS 3.1.3 (USACE 2002)	6/1/2005	AE w/ Floodway	Sub-basin areas and characteristics for the hydrologic model were determined using automated GIS methods. The NRCS Runoff Curve Number (RCN) method was used for computing loss rate. To compute the RCN, surficial soils group information was read from paper maps, and landcover data was taken from IKONOS satellite imagery from 2001 and 2002. The RCN for each sub-basin was reduced as appropriate to account for impervious area. The Clark unit hydrograph was used as the transform method. The storage coefficient for each sub-basin was computed from the drainage area. Precipitation statistics were from the Northeast Regional Climate Center. The hydraulic model used unsteady flow. Cross-sections were from a seamless topo-bathy digital terrain model constructed from above-water lidar and underwater field-transect bathymetry. Structures were from field surveys or, where available, construction plans. Roughness factors were from GIS analysis of landcover data.
Sweetwater Brook	Approximately 120 feet above Lindenwood Road	Lindenwood Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Tadmuck Brook	Confluence with Stony Brook	Providence Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	1/1/1981	AE w/ Floodway	Results of the regression equations were verified against streamgage statistics from nearby streamgages with similar basin characteristics. Overbank portions of cross-sections were from topographic maps (Teledyne 1979). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and engineering judgment. Starting water-surface elevations were from known water-surface elevations on Stony Brook.
Tadmuck Brook	Providence Road	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Tadmuck Swamp Brook	Littleton Road	Approximately 3,325 feet above Interstate 495	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	1/1/1981	AE w/ Floodway	Results of the regression equations were verified against streamgage statistics from nearby streamgages with similar basin characteristics. Overbank portions of cross-sections were from topographic maps (Teledyne 1979). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and engineering judgment. Starting water-surface elevations were from normal depth.
Taylor Brook	Confluence with Assabet River	U.S. Military Reservation	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	7/1/1977	AE w/ Floodway	Precipitation data used in the regression equations was from Herschfield 1961. Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Overbank portions of cross-sections were from topographic maps (Teledyne 1976). Underwater portions and structures were from field surveys. Roughness factors were from aerial photographs (Teledyne 1976) and field reconnaisance. Starting water-surface elevations were from known water-surface elevations on Assabet River.
Trap Swamp Brook pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (289.3 feet NAVD88).
Trapfall Brook	Confluence with Willard Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Trapfall Brook Tributary A	Confluence with Trapfall Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Trapfall Brook Tributary B	Confluence with Trapfall Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Trapfall Brook Tributary C	Confluence with Trapfall Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Trapfall Brook Tributary D	Confluence with Trapfall Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Trapfall Brook Tributary E	Confluence with Trapfall Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Trapfall Brook Tributary F	Confluence with Trapfall Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Tributary 1 to Coles Brook	Confluence with Cole's Brook	Approximately 1,500 feet above Arborwood Road	Discharge-frequency relationships	HEC-2 (USACE 1984)	12/1/1976	AE w/ Floodway	Discharge-frequency relationships and/or discharge-frequency-drainage area relationships were developed using hydrologic methods (SCS 1972a, SCS 1973a, Johnson and Tasker 1974). Discharges were compared with weighted-average discharges computed from log-Pearson type III analyses (WRC 1976) of streamgage records from Nashoba Brook and Heath Hen Meadow Brook, which have similar basin characteristics. Overbank portions of cross-sections were from topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were based on field inspections and informed by Barnes 1967. Starting water-surface elevations were from known water-surface elevations on Coles Brook.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Tributary 1 to Sudbury River	Confluence with Sudbury River	Approximately 0.3 mile above confluence with Sudbury River	Discharge-frequency relationships	HEC-2 (USACE 1984)	2/1/1984	AE w/ Floodway	Discharge-frequency relationships and/or discharge-frequency-drainage area relationships were developed using hydrologic methods (SCS 1972a, SCS 1973a, Johnson and Tasker 1974). Cross-sections were from field surveys and topographic maps (Sewall 1976). Structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from known water-surface elevations on Sudbury River.
Tributary 2 to Assabet River	Confluence with Assabet River	Concord Turnpike	Discharge-frequency relationships	HEC-2 (USACE 1984)	2/1/1984	AE w/ Floodway	Discharge-frequency relationships and/or discharge-frequency-drainage area relationships were developed using hydrologic methods (SCS 1972a, SCS 1973a, Johnson and Tasker 1974). Cross-sections were from field surveys and topographic maps (Sewall 1976). Structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from known water-surface elevations on Assabet River.
Tributary 2 to Tributary 1 to Coles Brook	Confluence with Tributary 1 to Cole's Brook	Piper Road	Discharge-frequency relationships	HEC-2 (USACE 1984)	12/1/1976	AE w/ Floodway	Discharge-frequency relationships and/or discharge-frequency-drainage area relationships were developed using hydrologic methods (SCS 1972a, SCS 1973a, Johnson and Tasker 1974). Discharges were compared with weighted-average discharges computed from log-Pearson type III analyses (WRC 1976) of streamgage records from Nashoba Brook and Heath Hen Meadow Brook, which have similar basin characteristics. Overbank portions of cross-sections were from topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were based on field inspections and informed by Barnes 1967. Starting water-surface elevations were from known water-surface elevations on Tributary 1 to Coles Brook.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Tributary 3 to Bogle Brook 2	Confluence with Bogle Brook 2	Winter Street	Unit hydrograph theory	HEC-2 (USACE 1984)	5/1/1978	AE w/ Floodway	Unit hydrograph theory was selected because the basin is ungaged, has natural storage flow regulation, and has high urbanization. Synthetic triangular unit hydrographs were developed using data from Wandle 1977, adjusted for slopes and local inflows, and compared to results of regression equations (Wandle 1977). Cross-section data were from photogrammetric maps (NE Air 1977) or field surveys otherwise. Structures were from field surveys. Roughness factors were informed by field inspection and chosen from Chow 1959. Starting water-surface elevations were from the slope-area method.
Tributary 3 to Bogle Brook 2	Winter Street	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Tributary 4 to Bogle Brook 2	Confluence with Bogle Brook 2	Radcliffe Road	Unit hydrograph theory	HEC-2 (USACE 1984)	5/1/1978	AE w/ Floodway	Unit hydrograph theory was selected because the basin is ungaged, has natural storage flow regulation, and has high urbanization. Synthetic triangular unit hydrographs were developed using data from Wandle 1977, adjusted for slopes and local inflows, and compared to results of regression equations (Wandle 1977). Cross-section data were from photogrammetric maps (NE Air 1977) or field surveys otherwise. Structures were from field surveys. Roughness factors were informed by field inspection and chosen from Chow 1959. Starting water-surface elevations were from the slope-area method.
Tributary A to Cold Brook	Confluence with Cold Brook	Just below Dakin Road	Regression equations (Jennings et al. 1994)	HEC-2 (USACE 1984)	2/1/1996	AE	Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977).

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Tributary A to Course Brook	Confluence with Course Brook	Railroad approximately 2,100 feet above confluence with Course Brook	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ Floodway	Rural regression equations (Wandle 1983) were used to compute discharges for most locations. Locations with more than 10% impervious area used urban regression equations (Sauer et al. 1983) instead. Streamgage statistics, updated through 2010 using log-Pearson type III analysis (IACWD 1982) and either weighted skew coefficients or station skew (if gages were affected by urbanization or regulation), were compared to the statistics for the same gages from the 1983 reports to determine if the regression equations would predict well discharges from additional periods of record. The base-flood-frequency discharges increased 123% from 1983 to 2010, on average, which was applied as an adjustment factor to results from the regression equations. Computed discharges were reduced below flood-storage reservoirs based on average reduction of outflow compared to inflow as determined by flood-routing computations. Flood-routing computations were obtained from the NRCS or are original to this study. A HEC-HMS rainfall-runoff model was used to validate the results of the regression equations. The HEC-HMS model used the NRCS Curve Number method to compute runoff and the NRCS Unit Hydrograph method to transform it. The meteorological input was from a type III storm. Five-minute time-steps were used. The HEC-HMS model was calibrated to precipitation and streamflow data from the 2007 storm event. The regression equation discharges were adjusted at several locations based on comparison to the HEC-HMS model. Cross-sections were from a blend of field survey data and lidar data. Structures were from field surveys or, where available for large structures, construction plans. Roughness factors were from engineering judgment and field observations.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Tributary A to Dudley Brook	Confluence with Dudley Brook	Approximately 70 feet above U.S. Route 20	Regression equations (Jennings et al. 1994)	HEC-2 (USACE 1984)	2/1/1996	AE	Essentially an upstream continuation of Dudley Brook. Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977).
Tributary A to Hop Brook	Confluence with Hop Brook	Approximately 60 feet above Firecut Lane	Regression equations (Jennings et al. 1994)	HEC-2 (USACE 1984)	2/1/1996	AE	Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977).
Tributary A to Ipswich River	County boundary	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Tributary A to Pantry Brook	Confluence with Pantry Brook	Approximately 0.89 mile above confluence with Pantry Brook	Regression equations (Jennings et al. 1994)	HEC-2 (USACE 1984)	2/1/1996	AE	Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977).
Tributary A to Squannacook River	Confluence with Squannacook River	Just above Meadow Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Overbank portions of cross-sections were from topographic maps (Teledyne 1977, MADPW 1965). Underwater portions were from field surveys and existing data. Structures were from field surveys. Roughness factors were from field inspections and aerial photography. Starting water-surface elevations were from normal depth.
Tributary A to Squannacook River	Just above Meadow Road	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Tributary A1 to Squannacook River	Confluence with Tributary A to Squannacook River	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Tributary A1A to Squannacook River	Confluence with Tributary A1 to Squannacook River	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Tributary A2 to Squannacook River	Confluence with Tributary A to Squannacook River	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Tributary B to Hop Brook	Confluence with Hop Brook	Upstream side of Moore Road	Regression equations (Jennings et al. 1994)	HEC-2 (USACE 1984)	2/1/1996	AE	Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977).
Tributary B to Squannacook River	Confluence with Squannacook River	Approximately 1,000 feet above Dudley Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Overbank portions of cross-sections were from topographic maps (Teledyne 1977, MADPW 1965). Underwater portions were from field surveys and existing data. Structures were from field surveys. Roughness factors were from field inspections and aerial photography. Starting water-surface elevations were from normal depth.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Tributary B to Vine Brook	Confluence with Vine Brook	Burlington/ Lexington corporate limits	SWMM (EPA 1971)	HEC-2 (USACE 1984)	3/1/1981	AE w/ Floodway	The hydrologic analysis was taken from Metcalf and Eddy (1978). Overbank portions of cross-sections were from aerial photography (Col-East 1976). Underwater portions and structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from the slope-area method.
Tributary C to Vine Brook	Confluence with Vine Brook	Approximately 1,500 feet above Muller Road	SWMM (EPA 1971)	HEC-2 (USACE 1984)	3/1/1981	AE w/ Floodway	The hydrologic analysis was taken from Metcalf and Eddy (1978). Overbank portions of cross-sections were from aerial photography (Col-East 1976). Underwater portions and structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from the slope-area method.
Tributary to Beaver Brook 3	Confluence with Beaver Brook 3	Approximately 1,000 feet above Lakeview Avenue	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	7/1/1986	AE w/ Floodway	Results from regression equations were compared to statistically analyzed streamgage records in the region and found to be in general agreement. Overbank portions of cross-sections were from topographic maps (USACE undated). Underwater portions were from field survey, topographic maps, or engineering judgment. Structures were from field survey. Roughness factors were from field inspection, photographs, and engineering judgment. Starting water-surface elevations were from known water-surface elevations on Beaver Brook 3.
Tributary to Beaver Brook 3	Approximately 1,000 feet above Lakeview Avenue	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Tributary to Cold Spring Brook	Confluence with Cold Spring Brook	Approximately 3,600 feet above Voyagers Lane	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	11/1/1979	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Cross-sections were from photogrammetric maps (Teledyne 1977). Structures were from field surveys. Roughness factors were from field inspection and aerial photographs (Teledyne 1977). Starting water-surface elevations were from known water-surface elevations on Cold Spring Brook.
Tributary to Indian Brook	Limit of former detailed study	Limit of former detailed study	unknown	unknown	10/1/2012	A	Water-surface elevations for the entire detailed-study profile (from the original Town of Hopkinton study) were below the ground surface as determined from a digital elevation model. The entire study was replaced with an approximate study that inundates the whole wetland.
Tributary to Martins Brook	Confluence with Martins Brook	Interstate 93	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	11/1/1986	AE w/ Floodway	Overbank portions of cross-sections were from photogrammetric maps. Underwater portions and structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from the slope-area method.
Tributary to Mill Brook	Confluence with Mill Brook	Approximately 1,000 feet above Sweetwater Avenue	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	4/1/1979	AE w/ Floodway	Due to large standard errors from the regional regression equations (Wandle 1977), discharges were also computed by rainfall-runoff techniques based on synthetic triangular unit hydrographs (SCS 1972a), and final discharges were a blend of the two methods resulting in a smooth curve. Cross-sections were from field surveys and topographic maps (Sewall 1976). Structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from the slope-area method.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Tributary to Nonacoicus Brook 1	Confluence with Nonacoicus Brook 2	Headwaters at Balch Pond	Rainfall-runoff routing (SCS 1972a)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Essentially a downstream continuation of Long Pond Brook. 10-, 2-, 1-, and 0.2-percent-annual-chance rainfall depths were applied to each sub-basin, from which runoff was calculated and discharge routed through reaches and control structures. Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977, MADNR 1976). Underwater portions were from field surveys or existing data. Structures were from field surveys. Roughness factors were from field inspections and aerial imagery. Starting water-surface elevations were from normal depth.
Tributary to Waushakum Pond	Ashland/ Framingham corporate limits	Approximately 2,000 feet above Shore Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	11/1/1979	AE w/ Floodway	Essentially an upstream continuation of Brook from Waushakum Pond. Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Cross-sections were from photogrammetric maps (Teledyne 1977). Structures were from field surveys. Roughness factors were from field inspection and aerial photographs (Teledyne 1977). Starting water-surface elevations were from known water-surface elevations on Brook from Waushakum Pond.
Trout Brook	Confluence with Hop Brook	Marlborough/ Sudbury corporate limits	Regression equations (Jennings et al. 1994)	HEC-2 (USACE 1984)	2/1/1996	AE	Formerly known as Tributary D to Hop Brook. Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977).

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Trout Brook 1	Confluence with Richardson Brook	Parker Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	7/1/1986	AE w/ Floodway	Streamgage statistics from Richardson Brook were considered applicable because of similar basin characteristics. A log-Pearson type III analysis of Richardson Brook's streamgage (13 data points, standard deviation 0.202, mean 2.06, regional skew 0.5) was leveraged in regression equations (Wandle 1977). Overbank portions of cross-sections were from topographic maps (USACE undated). Underwater portions were from field survey, topographic maps, or engineering judgment. Structures were from field survey. Roughness factors were from field inspection, photographs, and engineering judgment. Starting water-surface elevations were from known water-surface elevations on Richardson Brook.
Trout Brook 1 Tributary A	Confluence with Trout Brook 1	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Trout Brook 2	Confluence with Nashua River	Clark Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Overbank portions of cross-sections were from topographic maps (Teledyne 1977). Underwater portions were from field surveys or existing data. Structures were from field surveys. Roughness factors were from field inspection and photographs. Starting water-surface elevations were from normal depth.
Trull Brook	Confluence with Merrimack River	Interstate 495	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	12/1/1978	AE w/ Floodway	Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Merrimack River.
Trull Brook	Interstate 495	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Trull Brook Tributary	Confluence with Trull Brook	Approximately 350 feet above railroad	Drainage-area ratio	HEC-2 (USACE 1984)	2/15/1984	AE w/ Floodway	Results from regression equations (Wandle 1977) were compared to effective discharges on Trull Brook in the Town of Tewksbury. The results compared favorably. Therefore, Trull Brook discharges were transferred to Trull Brook Tributary locations by drainage-area ratio (with an exponent of 0.7). Overbank portions of cross-sections were from topographic maps (USACE undated). Underwater portions were from field survey, topographic maps, and engineering judgment. Topographic mapping was field-verified. Structures were from field surveys. Roughness factors were from field inspection, photographs, and engineering judgment. Starting water-surface elevations were from the slope-area method.
Trull Brook Tributary B	Confluence with Trull Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Tyler Road swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (451.0 feet NAVD88).
Unkety Brook	Confluence with Nashua River	Approximately 580 feet below Dunstable/ Groton corporate limits	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Unkety Brook	Approximately 580 feet below Dunstable/ Groton corporate limits	Approximately 6,890 feet above Dunstable/ Groton corporate limits	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Overbank portions of cross-sections were from topographic maps (Teledyne 1977). Underwater portions and all structures were from field surveys. Roughness factors were from field inspections and aerial imagery (Teledyne 1977). Starting water-surface elevations were from normal depth.
Unkety Brook	Approximately 6,890 feet above Dunstable/ Groton corporate limits	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Unkety Brook Tributary A	Confluence with Unkety Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Unkety Brook Tributary B	Confluence with Unkety Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Unkety Brook Tributary C	Confluence with Unkety Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Unkety Brook Tributary C1	Confluence with Unkety Brook Tributary C	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Unkety Brook Tributary D	Confluence with Unkety Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Unkety Brook Tributary E	Confluence with Unkety Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Unkety ponding	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (210.7 feet NAVD88).
Unnamed Tributary to Mill Brook 2	Confluence with Mill Brook 2	Approximately 0.67 mile above Lexington Road	unknown	HEC-RAS	8/14/2015	AE	Engineering was performed under LOMR 15-01-0902P. Engineering methods were not recorded.
Valley Pond	Entire shoreline	Entire shoreline	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	5/1/1977	AE	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Water-surface elevations were calculated from the ability of the spillway to pass the flood discharges.
Varnum Brook	Confluence with Nashua River	Approximately 75 feet above Hollis Street	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	12/1/1989	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Cross-sections and structures were from field surveys. Roughness factors were from field inspections and aerial photography. Starting water-surface elevations were from the slope-area method.
Vine Brook	Confluence with Shawsheen River	Hayes Lane	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	AE w/ Floodway	Roughness factors were estimated using field notes, photographs, and orthoimagery. Overbank portions of cross sections were taken from lidar topography (USGS 2011, 2014). Structures and underwater portions of cross sections were from field surveys. Starting water-surface elevations were from normal depth.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Vine Brook	Hayes Lane	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Vine Brook Tributary D	Confluence with Vine Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Vine Brook Tributary E	Confluence with Vine Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Vine Brook Tributary F	Confluence with Vine Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Vine Brook Tributary G	Confluence with Vine Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Vinton Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (508.3 feet NAVD88).
Walker Brook 1	Confluence with Nashua River	Approximately 6,740 feet above confluence with Nashua River	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Overbank portions of cross-sections were from topographic maps (Teledyne 1977). Underwater portions were from field surveys or existing data. Structures were from field surveys. Roughness factors were from field inspection and photographs. Starting water-surface elevations were from normal depth.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Walker Brook 2	Confluence with Pearl Hill Brook	County boundary	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Overbank portions of cross-sections were from topographic maps (Teledyne 1977, MADPW 1965). Underwater portions were from field surveys and existing data. Structures were from field surveys. Roughness factors were from field inspections and aerial photography. Starting water-surface elevations were from known water-surface elevations on Squannacook River.
Walker Brook 3	County boundary	Approximately 650 feet above State Route 85	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	11/1/1979	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Discharges were compared against existing data where possible. Overbank portions of cross-sections were from photogrammetric maps (Marlborough undated). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and aerial photography. Starting water-surface elevations were from known water-surface elevations on Sudbury Reservoir.
Walkers Brook	Reading/ Wakefield corporate limits	Ash Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	8/1/1978	AE w/ Floodway	Results from regression equations were adjusted where necessary to account for impervious land surface area resulting from urbanization. Overbank portions of cross-sections were from topographic maps (Sewall 1974, USGS various). Underwater portions were from field surveys. Structures were from construction plans where available or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from the slope-area method.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Warner Road swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (575.2 feet NAVD88).
Warren Road pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (374.4 feet NAVD88).
Washington Street ponding	Entire shoreline	Entire shoreline	none	none	4/30/2018	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (136.9 feet NAVD88).

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Wellington Brook	Confluence with Alewife Brook (Little River)	Approximately 710 feet above Library Private Drive	HEC-HMS 2.2.2	HEC-RAS 3.1.3 (USACE 2002)	6/1/2005	AE w/ Floodway	Sub-basin areas and characteristics for the hydrologic model were determined using automated GIS methods. The NRCS Runoff Curve Number (RCN) method was used for computing loss rate. To compute the RCN, surficial soils group information was read from paper maps, and landcover data was taken from IKONOS satellite imagery from 2001 and 2002. The RCN for each sub-basin was reduced as appropriate to account for impervious area. The Clark unit hydrograph was used as the transform method. The storage coefficient for each sub-basin was computed from the drainage area. Precipitation statistics were from the Northeast Regional Climate Center. The hydraulic model used unsteady flow. Cross-sections were from a seamless topo-bathy digital terrain model constructed from above-water lidar and underwater field-transect bathymetry. Structures were from field surveys or, where available, construction plans. Roughness factors were from GIS analysis of landcover data.
West Chester Brook	Confluence with Chester Brook	MDC Skating Rink on Totten Pond Road	Unit hydrograph theory	HEC-2 (USACE 1984)	8/1/1983	AE w/ Floodway	Unit hydrograph theory was selected because the basin is ungaged, has natural storage flow regulation, and has high urbanization. Synthetic triangular unit hydrographs were developed using data from Wandle 1977, adjusted for slopes and local inflows, and compared to results of regression equations (Wandle 1977). Cross-section data were from the City of Waltham (Waltham 1976) or field surveys otherwise. Structures were from field surveys. Roughness factors were informed by field inspection and chosen from Chow 1959. Starting water-surface elevations were from the slope-area method.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Western Canal	Confluence with Pawtucket Canal	Diversion from Northern Canal	Maximum flow capacity	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	Hamilton and Lawrence Canals were assumed to convey no flood flows. Maximum flow capacity of Northern and Pawtucket Canals and their inlet structures were from hydro-electric licensing documents (LIHI 2017). Some Northern Canal flow was diverted to Merrimack Canal through Moody Street Feeder (underground); the remainder was routed through Western Canal. HEC-RAS optimization routines determined flows at diversions of Eastern and Merrimack Canals from Pawtucket Canal. Overbank geometries were taken from lidar topography; channel geometries were estimated or interpolated from historical narratives (Malone 1975). Roughness was estimated based on land-use observations and historical narratives documenting channel material. Starting water-surface elevations were from normal depth using slope approximated from Merrimack and Pawtucket Canal outlet reaches. No structures were modeled hydraulically except Swamp Locks, Merrimack Canal Guard Gates, and Merrimack Dam, for all of which dimensions were estimated from imagery and topography.
Whitehall Brook	County boundary	Approximately 2,270 feet below Wood Street	Existing study (SCS 1973b)	WSP-2 (SCS 1976)	11/1/1979	AE w/ Floodway	The existing study (SCS 1973b) was the source of the 10-, 1-, and 0.2-percent-annual-chance discharges. The 2-percent-annual-chance discharges were obtained graphically from the other discharges. The same study was the source of the existing WSP-2 model used for the hydraulic model. Overbank portions of cross-sections were from topographic maps (MADPW 1965). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from aerial photography (Teledyne 1977) and field inspection. Starting water-surface elevations were from known water-surface elevations on Sudbury River.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Whitney Road swamp 1	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (343.5 feet NAVD88).
Whitney Road swamp 2	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (368.1 feet NAVD88).
Whittemore Pond	Entire shoreline	Entire shoreline	none	none	4/30/2018	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (39.6 feet NAVD88).
Willard Brook	Confluence with Walker Brook	Approximately 2,100 feet above West Meadow Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Overbank portions of cross-sections were from topographic maps (Teledyne 1977, MADPW 1965). Underwater portions were from field surveys and existing data. Structures were from field surveys. Roughness factors were from field inspections and aerial photography. Starting water-surface elevations were from normal depth.
Willard Brook	Approximately 2,100 feet above West Meadow Road	Confluence with Willard Brook Tributary E	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Willard Brook Tributary A	Confluence with Willard Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Willard Brook Tributary B	Confluence with Willard Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Willard Brook Tributary B1	Confluence with Willard Brook Tributary B	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Willard Brook Tributary C	Confluence with Willard Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Willard Brook Tributary C1	Confluence with Willard Brook Tributary C	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Willard Brook Tributary D	Confluence with Willard Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Willard Brook Tributary E	Confluence with Willard Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Winter Pond	Entire shoreline	Entire shoreline	none	none	4/30/2018	A	Analysis of lidar DEM (FEMA 2011, USGS 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (44.0 and 44.2 feet NAVD88).

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Winthrop Canal	Linden Pond	Lake Winthrop	HEC-HMS 3.5	HEC-RAS 4.1.0	9/17/2014	AE w/ Floodway	Engineering was performed under LOMR 13-01-2122P. The 24-hour peak precipitation storm was used for all profiles because the time of concentration for the basin does not exceed 24 hours. Total rainfall depth for the 24-hour storm was compared between the National Weather Service (Herschfield 1961) and the Northeast Regional Climate Center (NRCC undated); NRCC depths were chosen in consideration of the longer record of data. Temporal distribution of rainfall for the 24-hour storm was based on a type III synthetic rainfall distribution for eastern Massachusetts (SCS 1986). Rainfall losses were calculated using the runoff curve number approach (NRCS 2004). Spatially averaged curve numbers were computed for each sub-basin (SCS 1986) using soils data from the Soil Survey Geographic (SSURGO) database and landcover data from the USGS. Runoff response was calculated using the Unit Hydrograph method (SCS 1986). Time of concentration was calculated using the velocity method (NRCS 2010). Lag time was assumed to be 60% of time of concentration (NRCS 2010). The HEC-HMS model used a 1-minute time step for 48 hours to generate the input hydrographs for the unsteady-flow HEC-RAS hydraulic model. Overbank portions of cross-sections were from 2-foot photogrammetric contours provided by the Town of Holliston at a scale of 1:480. Underwater portions and structures were from field surveys. Roughness factors were based on field inspections and informed by Chow 1959 and Arcement and Schneider 1989. Starting water-surface elevations were from free outfall at Linden Pond Dam at the downstream end and known water-surface elevations on Winthrop Lake at the upstream end, determined by its stage-storage relationship.

Table 12: Summary of Hydrologic and Hydraulic Analyses

Flooding Source	Study Limits Downstream Limit	Study Limits Upstream Limit	Hydrologic Model or Method Used	Hydraulic Model or Method Used	Date Analyses Completed	Flood Zone on FIRM	Special Considerations
Witch Brook	Confluence with Squannacook River	Just above Pierce Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were obtained graphically from the other discharges. Overbank portions of cross-sections were from topographic maps (Teledyne 1977, MADPW 1965). Underwater portions were from field surveys and existing data. Structures were from field surveys. Roughness factors were from field inspections and aerial photography. Starting water-surface elevations were from normal depth.
Witch Brook	Just above Pierce Road	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Witch Brook Tributary A	Confluence with Witch Brook	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Wolf Brook	County boundary	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Wrangling Brook	Confluence with Nashua River	Diversion from Squannacook River	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Wrights Pond	Entrance to underground drainage above Parkway Road	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Aberjona River (approximate)	0.057	0.105
Aberjona River (detailed)	0.035-0.150	0.014-0.300
Aberjona River North Spur	0.035-0.150	0.014-0.300
Aberjona River Tributary A	0.057	0.105-0.106
Alewife Brook (approximate)	0.057	0.104
Alewife Brook (detailed)	0.035-0.150	0.014-0.300
Althea Lake	*	*
Angelica Brook	0.030-0.035	0.050-0.060
Assabet Branch No. 3	0.030-0.060	0.030-0.130
Assabet Branch No. 4	0.030-0.060	0.030-0.130
Assabet River	0.030-0.055	0.035-0.150
Baddacook Brook (detailed)	0.035-0.040	0.050-0.070
Baddacook Brook (approximate)	*	*
Baddacook Brook Tributary A	*	*
Baiting Brook	0.025-0.005	0.045-0.120
Bancroft Brook	*	*
Bartlett Brook	*	*
Bartlett Brook Tributary B	*	*
Bartlett Brook Tributary B1	*	*
Bartlett Brook Tributary C	*	*
Bayberry Hill Brook	*	*
Bear Meadow Brook	0.020-0.065	0.040-0.150
Bear Meadow Brook Tributary A	*	*
Bear Meadow Brook Tributary A1	*	*
Bear Meadow Brook Tributary B	*	*
Bear Meadow Brook Tributary B1	*	*
Beaver Brook 1 (approximate)	0.056-0.057	0.101-0.104
Beaver Brook 1 (detailed)	0.015-0.040	0.030-0.150
Beaver Brook 2	0.06	0.035-0.100
Beaver Brook 2 Split 1	0.030-0.090	0.030-0.070
Beaver Brook 2 Split 2	0.03	0.030-0.050
Beaver Brook 2 Split 3	0.06	0.050-0.090

Table 13: Roughness Coefficients

Flooding Source	Channel "n"	Overbank "n"
Beaver Brook 3	0.030-0.045	0.045-0.075
Beaver Brook 4 (detailed)	0.035-0.070	0.050-0.140
Beaver Brook 4 (approximate)	*	*
Beaver Brook 4 Tributary A	*	*
Beaver Brook 4 Tributary A1	*	*
Beaver Brook 4 Tributary B	*	*
Beaver Brook 4 Tributary C	*	*
Beaver Brook 4 Tributary D	*	*
Beaver Brook 4 Tributary E	*	*
Beaver Brook 4 Tributary E1	*	*
Beaver Brook 5 (approximate)	0.057	0.105-0.106
Beaver Brook 5 (detailed)	0.050-0.080	0.060-0.100
Beaver Brook 6	*	*
Beaver Brook 6 Tributary A	*	*
Beaver Brook 6 Tributary B	*	*
Beaver Brook 6 Tributary C	*	*
Beaver Dam Brook	0.015-0.050	0.050-0.110
Beaver Pond Brook	*	*
Bennetts Brook (detailed)	0.035	0.05
Bennetts Brook (approximate)	*	*
Bennetts Brook Tributary A	*	*
Bennetts Brook Tributary B	*	*
Bennetts Brook Tributary C	*	*
Bennetts Brook Tributary D	*	*
Bennetts Brook Tributary E	*	*
Bennetts Brook Tributary F	*	*
Bennetts Brook Tributary F1	*	*
Bennetts Brook Tributary G	*	*
Bennetts Brook Tributary H	*	*
Bennetts Pond Brook	0.057	0.103-0.104
Birch Meadow Brook	0.025-0.045	0.045-0.085
Bixby Brook	*	*

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Bixby Brook Tributary A	*	*
Black Brook	0.030-0.035	0.055-0.085
Black Brook 4	*	*
Black Brook 4 Tributary A	*	*
Black Brook 4 Tributary B	*	*
Black Brook 4 Tributary B1	*	*
Black Brook 4 Tributary C	*	*
Blue Brook	*	*
Bogastow Brook	0.055	0.16
Bogastow Brook Tributary A	*	*
Bogastow Brook Tributary C	*	*
Bogle Brook 1 (approximate)	0.056-0.057	0.103-0.104
Bogle Brook 1 (detailed)	0.015-0.050	0.070-0.110
Bogle Brook 2	0.015-0.040	0.030-0.240
Boons Pond Branch	0.015-0.060	0.050-0.120
Boutwell Brook	0.03	0.05
Bow Brook (detailed)	0.035	0.050-0.070
Bow Brook (approximate)	*	*
Bow Brook Tributary A	*	*
Branch of Assabet River	0.015-0.060	0.050-0.120
Branch of Elizabeth Brook 1	0.015-0.060	0.050-0.120
Broad Meadow Brook	0.015-0.035	0.045-0.080
Brook A	*	*
Brook from Waushakum Pond	0.030-0.035	0.050-0.060
Butter Brook	0.035-0.045	0.050-0.085
Catacoonamug Brook (approximate)	*	*
Catacoonamug Brook (detailed)	0.035	0.050-0.070
Charles River (approximate)	0.057	0.104
Charles River (detailed) (lower)	0.034-0.065	0.04-0.08
Charles River (detailed) (upper)	0.015-0.060	0.040-0.150
Cheese Cake Brook (approximate)	0.03	0.06
Cheese Cake Brook (detailed)	0.030-0.035	0.01

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Cheese Cake Brook Tributary A	0.057	0.105-0.106
Cherry Brook (approximate)	0.057	0.105-0.106
Cherry Brook (detailed)	0.015-0.040	0.030-0.240
Cherry Brook Tributary A	0.057	0.105-0.106
Chester Brook	0.015-0.040	0.030-0.150
Chicken Brook	0.02-0.06	0.05-0.11
Claypit Brook	*	*
Cochituate Brook	0.030-0.035	0.050-0.060
Cold Brook	0.016-0.050	0.050-0.100
Cold Brook 1	*	*
Cold Spring Brook	0.035-0.050	0.050-0.100
Coles Brook	0.045	0.035-0.090
Collins Brook	*	*
Conant Brook	0.030-0.040	0.040-0.080
Concord River	0.032-0.050	0.032-0.100
Content Brook (detailed)	0.030-0.045	0.060-0.110
Content Brook (approximate)	*	*
Content Brook Tributary A	*	*
Course Brook	0.04	0.032-0.080
Cow Pond Brook	0.035-0.040	0.050-0.070
Cow Pond Brook Tributary A	*	*
Cow Pond Brook Tributary B	*	*
Cranberry Brook	0.04	0.1
Crooked Springs Brook	*	*
Cummings Brook (approximate)	0.057	0.104-0.105
Cummings Brook (detailed)	0.035-0.060	0.014-0.300
Dakins Brook	0.030-0.040	0.08
Danforth Brook	0.030-0.060	0.030-0.130
Darby Brook (detailed)	0.015-0.045	0.060-0.070
Darby Brook (approximate)	*	*
Davis Brook	0.015-0.050	0.070-0.110
Deep Brook	*	*

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Deep Brook Tributary A	*	*
Deer Brook	0.057	0.104-0.106
Dirty Meadow Brook (approximate)	0.056-0.057	0.103-0.104
Dirty Meadow Brook (detailed)	0.06	0.16
Dopping Brook (approximate)	0.057	0.104-0.106
Dopping Brook (detailed)	0.045-0.060	0.050-0.160
Dopping Brook Tributary A	0.057	0.106
Double Brook	*	*
Dudley Brook	0.016-0.045	0.050-0.090
East Outlet	0.030-0.055	0.050-0.095
Eastern Canal	*	*
Elizabeth Brook 1	0.015-0.060	0.050-0.120
Elizabeth Brook 2	0.040-0.055	0.040-0.120
Elm Brook (detailed)	0.015-0.050	0.020-0.180
Elm Brook (approximate)	*	*
Elm Brook Tributary A	*	*
Emerson Brook Tributary A	0.057	0.104-0.105
Emerson Brook Tributary A1	0.057	0.105-0.106
Emerson Brook Tributary A2	0.057	0.105-0.106
Farley Brook	0.05	0.035-0.090
Farley Brook Split 1	0.035-0.050	0.035-0.090
Farrar Pond Brook	0.045-0.075	0.05
Fiske Brook Tributary A	0.057	0.105-0.106
Fort Meadow Brook	0.030-0.060	0.030-0.130
Fort Pond Brook	0.012-0.070	0.015-0.140
Fort Pond Brook Branch 1	0.05	0.045-0.090
Fort Pond Brook Branch 2	0.035-0.070	0.050-0.140
Gilson Brook	*	*
Gilson Brook Tributary A	*	*
Grassy Pond Brook	0.015-0.050	0.015-0.085
Graves Pond Brook (detailed)	0.035	0.050-0.075
Graves Pond Brook (approximate)	*	*

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Great Road Tributary	0.035	0.08
Greens Brook	0.04	0.085-0.180
Guggins Brook	0.015-0.070	0.015-0.140
Gulf Brook	*	*
Gulf Brook Tributary A	*	*
Gulf Brook Tributary A1	*	*
Gulf Brook Tributary A2	*	*
Gulf Brook Tributary B	*	*
Gumpas Pond Brook	0.035	0.07
Hales Brook	0.035-0.055	0.080-0.100
Halls Brook	0.035-0.100	0.014-0.300
Hauk Brook	*	*
Hauk Brook Tributary A	*	*
Hayward Brook	0.035	0.050-0.075
Heath Brook (detailed)	*	*
Heath Brook (approximate)	*	*
Heath Hen Meadow Brook	0.033	0.050-0.090
Heath Hen Meadow Brook Split 1	0.033-0.050	0.050-0.090
Hobbs Brook 1 (approximate)	0.054-0.056	0.096-0.103
Hobbs Brook 1 (detailed)	0.015-0.040	0.030-0.240
Hobbs Brook 2 (approximate)	0.057	0.104-0.104
Hobbs Brook 2 (detailed)	0.045-0.075	0.045-0.075
Hobbs Brook Tributary A	*	*
Hobbs Brook Tributary B	*	*
Hobbs Brook Tributary C	*	*
Hobbs Brook Tributary D	*	*
Hobbs Brook Tributary E	*	*
Hog Brook	0.030-0.060	0.030-0.130
Hop Brook	0.015-0.035	0.045-0.080
Hopping Brook	*	*
Hopping Brook Tributary D	*	*
Hopping Brook Tributary D1	*	*

Table 13: Roughness Coefficients

Flooding Source	Channel "n"	Overbank "n"
Hopping Brook Tributary E	*	*
Horn Pond Brook / Fowle Brook	0.030-0.100	0.014-0.300
Inch Brook	0.025-0.045	0.025-0.085
Indian Brook	0.035-0.050	0.050-0.100
Indian Brook 2	0.056-0.057	0.102-0.106
Ipswich River (approximate)	0.057	0.105-0.106
Ipswich River (detailed)	0.030-0.065	0.04-0.12
Ipswich River Diversion	0.057	0.105-0.106
Ipswich River Tributary D	*	*
Ipswich River Tributary E	*	*
James Brook	0.035-0.040	0.050-0.070
James Brook Tributary A	*	*
James Brook Tributary B	*	*
James Brook Tributary B1	*	*
Jar Brook	0.055	0.12
Jenny Dugan Brook	0.033-0.050	0.050-0.090
Joint Grass Brook	*	*
Joint Grass Brook Tributary A	*	*
Joint Grass Brook Tributary B	*	*
Joint Grass Brook Tributary C	*	*
Joint Grass Brook Tributary D	*	*
Joint Grass Brook Tributary D1	*	*
Joint Grass Brook Tributary D2	*	*
Jones Brook (detailed)	0.030-0.040	0.11
Jones Brook (approximate)	*	*
Keyes Brook	*	*
Keyes Brook Tributary A	*	*
Kiln Brook (detailed)	0.050-0.080	0.060-0.100
Kiln Brook (approximate)	*	*
Kiln Brook Tributary A	*	*
Kiln Brook Tributary B	*	*
King Street Tributary	0.035-0.042	0.070-0.100

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Landham-Allowance Brook	0.016-0.035	0.050-0.070
Lawrence Brook	0.03	0.060-0.075
Little Brook	0.035-0.10	0.014-0.30
Locke Brook (detailed)	0.035	0.050-0.075
Locke Brook (approximate)	*	*
Locke Brook Tributary A	*	*
Long Pond Brook (detailed)	*	*
Long Pond Brook (approximate)	*	*
Long Pond Brook Tributary A	*	*
Lower Spot Pond Brook	0.020-0.024	0.044
Lubbers Brook	0.030-0.055	0.075-0.120
Malden River	0.020-0.050	0.020-0.050
Malden River Tributary A	0.057	0.105-0.106
Maple Meadow Brook (detailed)	0.035-0.055	0.065-0.088
Maple Meadow Brook (approximate)	0.057	0.104-0.105
Marginal Brook	0.035-0.045	0.045-0.085
Marshall Brook	0.014-0.045	0.060-0.070
Martins Brook	0.040-0.102	0.060-0.090
Martins Pond Brook (detailed)	0.035-0.040	0.050-0.070
Martins Pond Brook (approximate)	*	*
Martins Pond Brook Tributary A	*	*
Martins Pond Brook Tributary B	*	*
Mascuppic Brook	0.03	0.07
Mason Brook	0.035	0.050-0.075
Mason Brook Tributary A	*	*
Meadow Brook (detailed)	0.024-0.045	0.060-0.070
Meadow Brook (approximate)	*	*
Meadow Brook Tributary A	*	*
Meadow Brook Tributary A1	*	*
Meadow River Branch	0.015-0.050	0.1
Merrimack Canal	*	*
Merrimack River	0.020-0.055	0.040-0.200

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Merrimack River Tributary E	*	*
Merrimack River Tributary E1	*	*
Merrimack River Tributary E1A	*	*
Merrimack River Tributary E2	*	*
Merrimack River Tributary E2A	*	*
Merrimack River Tributary E2A1	*	*
Merrimack River Tributary E2B	*	*
Merrimack River Tributary E2C	*	*
Merrimack River Tributary E3	*	*
Merrimack River Tributary E4	*	*
Merrimack River Tributary E5	*	*
Merrimack River Tributary E5A	*	*
Merrimack River Tributary F	*	*
Merrimack River Tributary F1	*	*
Merrimack River Tributary F2	*	*
Middlesex Canal (detailed)	0.030-0.045	0.060-0.110
Middlesex Canal (approximate)	*	*
Mill Brook 1	0.035	0.050-0.075
Mill Brook 2	0.030-0.040	0.030-0.080
Mill Brook 3	0.035-0.150	0.014-0.300
Mill Pond Tributary	0.020-0.035	0.08
Mill River (approximate)	*	*
Mill River (detailed) (lower)	0.045-0.100	0.11
Mill River (detailed) (upper)	0.045-0.065	0.045-0.095
Mill River 2 (upper)	*	*
Mine Brook	*	*
Mineway Brook	0.030-0.045	0.070-0.100
Mongo Brook	0.040-0.050	0.120-0.160
Morse Brook	0.035	0.050-0.070
Mowry Brook	0.015-0.035	0.045-0.080
Mud Pond Brook (detailed)	*	*
Mud Pond Brook (approximate)	*	*

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Muddy Brook	0.05	0.050-0.090
Mulpus Brook	0.035	0.050-0.070
Munroe Brook	0.050-0.080	0.060-0.100
Mystic River	0.035	0.014-0.300
Nagog Brook	0.045	0.07
Nashoba Brook	0.015-0.045	0.040-0.120
Nashua River	0.03	0.060-0.070
Nashua River Tributary D	*	*
Nashua River Tributary E	*	*
Nashua River Tributary F	*	*
Nashua River Tributary H	*	*
New Meadow Brook Tributary A	*	*
New Meadow Brook Tributary E	*	*
Nissitissit River	0.04	0.060-0.090
Nissitissit River Tributary A	*	*
Nissitissit River Tributary A1	*	*
Nissitissit River Tributary B	*	*
Nissitissit River Tributary C	*	*
Nissitissit River Tributary D	*	*
Nissitissit River Tributary E	*	*
Nonacoicus Brook 1	0.035	0.05
Nonacoicus Brook 2	0.035	0.05
North Lexington Brook (detailed)	0.050-0.080	0.060-0.100
North Lexington Brook (approximate)	*	*
North Lexington Brook Tributary A	*	*
North Lexington Brook Tributary A1	*	*
North Lexington Brook Tributary B	*	*
Northern Canal	*	*
Pages Brook	0.013-0.050	0.l00
Pages Brook Branch	0.024-0.050	0.l00
Pantry Brook	0.016-0.040	0.050-0.100
Paul Brook	0.020-0.035	0.070-0.080

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Pawtucket Canal	*	*
Pearl Hill Brook (detailed)	0.035	0.050-0.075
Pearl Hill Brook (approximate)	*	*
Pearl Hill Brook Tributary A	*	*
Pearl Hill Brook Tributary B	*	*
Pearl Hill Brook Tributary C	*	*
Peppermint Brook	0.035	0.07
Peppermint Brook Tributary A	*	*
Pine Brook	0.035	0.050-0.075
Pole Brook	0.020-0.060	0.090-1.100
Pratts Brook	0.05	0.035-0.090
Pumpkin Brook	*	*
Putnam Brook	0.055	0.050-0.090
Reedy Meadow Brook	0.035-0.050	0.050-0.070
Reedy Meadow Brook Tributary A	*	*
Richardson Brook	0.035-0.045	0.045-0.080
River Meadow Brook	0.030-0.060	0.020-0.100
Robinson Brook	*	*
Run Brook	0.016-0.045	0.050-0.100
Salmon Brook	0.030-0.035	0.1
Salmon Brook Tributary A	*	*
Salmon Brook Tributary B	*	*
Salmon Brook Tributary C	*	*
Salmon Brook Tributary D	*	*
Salmon Brook Tributary E	*	*
Salmon Brook Tributary F	*	*
Salmon Brook Tributary G	*	*
Sandy Brook	0.035-0.045	0.050-0.100
Saugus River	0.035-0.075	0.04-0.10
Saunders Brook	*	*
Sawmill Brook 1 (approximate) (lower)	0.056-0.057	0.103-0.104
Sawmill Brook 1 (approximate) (upper)	0.057	0.104

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Sawmill Brook 1 (detailed)	0.038-0.040	0.080-0.090
Sawmill Brook 2	0.030-0.040	0.08
Sawmill Brook 3	0.055-0.057	0.100-0.103
Sawmill Brook 3 Tributary B	0.057	0.105-0.106
Scarlet Brook	*	*
Schneider Brook	0.035-0.15	0.014-0.300
Seaverns Brook	*	*
Seaverns Brook Tributary A	*	*
Sewall Brook	0.056-0.057	0.101-0.104
Shakers Glen Brook (approximate)	0.056-0.057	0.102-0.104
Shakers Glen Brook (detailed)	0.035	0.014-0.300
Shawsheen River	0.012-0.050	0.020-0.150
Shawsheen River Tributary E	*	*
Shawsheen River Tributary F	*	*
Shawsheen River Tributary F1	*	*
Shawsheen River Tributary G	*	*
Shute Brook	0.057	0.105-0.106
Silver Lake Outlet	0.057	0.105-0.106
Skug River	0.040-0.098	0.075
Skug River Tributary A	0.057	0.105-0.106
Skug River Tributary B	0.057	0.105-0.106
Skug River Tributary C	*	*
Skug River Tributary C1	*	*
Skug River Tributary C2	*	*
Skug River Tributary C3	*	*
Skug River Tributary D	0.057	0.105-0.106
Smelt Brook 1	0.057	0.104-0.105
Snake Brook	0.035	0.050-0.075
Snake Meadow Brook	*	*
Snake Meadow Brook Tributary A	*	*
South Branch Souhegan River	*	*
South Branch Souhegan River Tributary B	*	*

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
South Branch Souhegan River Tributary B1	*	*
South Branch Souhegan River Tributary B2	*	*
South Meadow Brook	0.020-0.035	0.070-0.080
Spaulding Brook	*	*
Spaulding Brook Tributary A	*	*
Spencer Brook	0.032-0.050	0.032-0.090
Spot Pond Brook	0.057	0.104-0.105
Spring Brook	0.024-0.045	0.030-0.160
Spruce Swamp Brook	*	*
Squannacook River	0.035-0.040	0.060-0.080
Squannacook River Tributary B	*	*
Squannacook River Tributary B1	*	*
Squannacook River Tributary C	*	*
Squannacook River Tributary D	*	*
Stewart Brook	*	*
Stony Brook	0.033-0.050	0.032-0.090
Stony Brook 1 (detailed)	*	*
Stony Brook 1 (approximate)	0.056	0.101-0.103
Stony Brook 1 Tributary A	*	*
Stony Brook 1 Tributary B	*	*
Stony Brook 1 Tributary B1	*	*
Stony Brook 1 Tributary C	*	*
Stony Brook 1 Tributary D	*	*
Stony Brook 1 Tributary E	*	*
Stony Brook 1 Tributary F	*	*
Stony Brook 1 Tributary F1	*	*
Stony Brook 2 (detailed)	*	*
Stony Brook 2 (approximate)	*	*
Stony Brook 2 Tributary A	*	*
Stony Brook 2 Tributary B	*	*
Strawberry Meadow	*	*
Strawberry Meadow Tributary A	*	*

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Strawberry Meadow Tributary B	*	*
Strawberry Meadow Tributary B1	*	*
Strong Water Brook	0.014-0.045	0.040-0.090
Sucker Brook 1 Tributary A	*	*
Sucker Brook 2	*	*
Sucker Brook 2 Tributary A	*	*
Sudbury River	0.032-0.090	0.032-0.100
Sudbury River Split 1	0.05	0.040-0.090
Sutton Brook (detailed)	*	*
Sutton Brook (approximate)	*	*
Sutton Brook Tributary A	*	*
Sweetwater Brook (approximate)	0.057	0.103
Sweetwater Brook (detailed)	0.035	0.014-0.300
Tadmuck Brook (detailed)	0.03	0.055-0.070
Tadmuck Brook (approximate)	*	*
Tadmuck Swamp Brook	0.03	0.07
Taylor Brook	0.035-0.050	0.015-0.120
Trapfall Brook	*	*
Trapfall Brook Tributary A	*	*
Trapfall Brook Tributary B	*	*
Trapfall Brook Tributary C	*	*
Trapfall Brook Tributary D	*	*
Trapfall Brook Tributary E	*	*
Trapfall Brook Tributary F	*	*
Tributary 1 to Coles Brook	0.015-0.040	0.040-0.120
Tributary 1 to Sudbury River	0.030-0.040	0.08
Tributary 2 to Assabet River	0.015-0.040	0.040-0.060
Tributary 2 to Tributary 1 to Coles Brook	0.015-0.040	0.040-0.120
Tributary 3 to Bogle Brook 2 (approximate)	0.057	0.104-0.105
Tributary 3 to Bogle Brook 2 (detailed)	0.015-0.040	0.030-0.240
Tributary 4 to Bogle Brook 2	0.015-0.040	0.030-0.240
Tributary A to Cold Brook	0.030-0.045	0.070-0.140

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Tributary A to Course Brook	0.04	0.040-0.090
Tributary A to Dudley Brook	0.016-0.045	0.050-0.090
Tributary A to Hop Brook	0.04	0.060-0.120
Tributary A to Ipswich River	0.057	0.104-0.105
Tributary A to Pantry Brook	0.030-0.040	0.090-0.110
Tributary A to Squannacook River (detailed)	0.035	0.050-0.070
Tributary A to Squannacook River (approximate)	*	*
Tributary A1 to Squannacook River	*	*
Tributary A1A to Squannacook River	*	*
Tributary A2 to Squannacook River	*	*
Tributary B to Hop Brook	0.035-0.040	0.12
Tributary B to Squannacook River	0.035	0.050-0.075
Tributary B to Vine Brook	0.040-0.045	0.040-0.090
Tributary C to Vine Brook	0.04	0.070-0.120
Tributary to Beaver Brook 3 (detailed)	0.040-0.050	0.075-0.095
Tributary to Beaver Brook 3 (approximate)	*	*
Tributary to Cold Spring Brook	0.035-0.045	0.050-0.085
Tributary to Indian Brook	0.035-0.050	0.050-0.100
Tributary to Martins Brook	0.045-0.065	0.060-0.090
Tributary to Mill Brook	0.030-0.065	0.030-0.150
Tributary to Nonacoicus Brook 1	0.035	0.05
Tributary to Waushakum Pond	0.035-0.045	0.050-0.085
Trout Brook	0.04	0.1
Trout Brook 1	0.035-0.050	0.070-0.090
Trout Brook 1 Tributary A	*	*
Trout Brook 2	0.035	0.050-0.070
Trull Brook (detailed)	0.030-0.060	0.050-0.080
Trull Brook (approximate)	*	*
Trull Brook Tributary	0.030-0.050	0.050-0.100
Trull Brook Tributary B	*	*
Unkety Brook (approximate) (lower)	*	*

Table 13: Roughness Coefficients

Flooding Source	Channel “n”	Overbank “n”
Unkety Brook (detailed)	0.035-0.040	0.050-0.070
Unkety Brook (approximate) (upper)	*	*
Unkety Brook Tributary A	*	*
Unkety Brook Tributary B	*	*
Unkety Brook Tributary C	*	*
Unkety Brook Tributary C1	*	*
Unkety Brook Tributary D	*	*
Unkety Brook Tributary E	*	*
Unnamed Tributary to Mill Brook 2	*	*
Varnum Brook	0.03	0.045-0.160
Vine Brook (detailed)	0.020-0.080	0.020-0.100
Vine Brook (approximate)	*	*
Vine Brook Tributary D	*	*
Vine Brook Tributary E	*	*
Vine Brook Tributary F	*	*
Vine Brook Tributary G	*	*
Walker Brook 1	0.035	0.050-0.070
Walker Brook 2	0.035	0.050-0.070
Walker Brook 3	0.015-0.035	0.045-0.080
Walkers Brook	0.020-0.065	0.040-0.150
Wellington Brook	0.035-0.150	0.014-0.300
West Chester Brook	0.015-0.040	0.030-0.150
Western Canal	*	*
Whitehall Brook	0.035-0.050	0.050-0.100
Willard Brook (detailed)	0.035	0.050-0.075
Willard Brook (approximate)	*	*
Willard Brook Tributary A	*	*
Willard Brook Tributary B	*	*
Willard Brook Tributary B1	*	*
Willard Brook Tributary C	*	*
Willard Brook Tributary C1	*	*
Willard Brook Tributary D	*	*

Table 13: Roughness Coefficients

Flooding Source	Channel "n"	Overbank "n"
Willard Brook Tributary E	*	*
Winthrop Canal	0.04	0.070-0.100
Witch Brook (detailed)	0.035	0.050-0.070
Witch Brook (approximate)	*	*
Witch Brook Tributary A	*	*
Wolf Brook	*	*
Wrangling Brook	*	*
Wrights Pond	0.057	0.105-0.106

5.3 Coastal Analyses

For the areas of Middlesex County that are impacted by coastal flooding processes, coastal flood hazard analyses were performed to provide estimates of coastal BFEs. Coastal BFEs reflect the increase in water levels during a flood event due to extreme tides and storm surge as well as overland wave effects.

The following subsections provide summaries of how each coastal process was considered for this FIS Report. Greater detail (including assumptions, analysis, and results) is available in the archived project documentation. Table 14 summarizes the methods and/or models used for the coastal analyses. Refer to Section 2.5.1 for descriptions of the terms used in this section.

Table 14: Summary of Coastal Analyses

Flooding Source	Study Limits From	Study Limits To	Hazard Evaluated	Model or Method Used	Date Analysis was Completed
Boston Harbor, Pines River	Coastal flooding along Mystic River, Charles River, Pines River, and Town Line Brook	Coastal flooding along Mystic River, Charles River, Pines River, and Town Line Brook	Stillwater elevation	Results from Suffolk County study	9/13/2013

5.3.1 Total Stillwater Elevations

The total stillwater elevations (stillwater including storm surge plus wave setup) for the 1% annual chance flood were determined for areas subject to coastal flooding. The models and methods that were used to determine storm surge and wave setup are listed in Table 14. The stillwater elevation that was used for each transect in coastal analyses is shown in Table 16, "Coastal Transect Parameters."

Figure 8: 1% Annual Chance Total Stillwater Elevations for Coastal Areas

[Not Applicable to this Flood Risk Project]

Astronomical Tide

Astronomical tidal statistics were generated directly from local tidal constituents by sampling the predicted tide at random times throughout the tidal epoch.

Storm Surge Statistics

Storm surge is modeled based on characteristics of actual storms responsible for significant coastal flooding. The characteristics of these storms are typically determined by statistical study of the regional historical record of storms or by statistical study of tidal gages.

When historic records are used to calculate storm surge, characteristics such as the strength, size, track, etc., of storms are identified by site. Storm data was used in conjunction with numerical hydrodynamic models to determine the corresponding storm surge levels. An extreme value analysis was performed on the storm surge modeling results to determine a stillwater elevation for the 1% annual chance event.

Tidal gages can be used instead of historic records of storms when the available tidal gage record for the area represents both the astronomical tide component and the storm surge component. Table 15 provides the gage name, managing agency, gage type, gage identifier, start date, end date, and statistical methodology applied to each gage used to determine the stillwater elevations. For areas between gages, peak stillwater elevations for selected recurrence intervals were estimated by combining interpolation between gages and observed high water marks during major storms. A regionalized statistical approach was applied to the gage data so that stillwater elevations in areas between gages could be identified.

Table 15: Tide Gage Analysis Specifics

[Not Applicable to this Flood Risk Project]

Combined Riverine and Tidal Effects

Riverine and surge rates for the lower reaches of the Inundation River were combined by developing curves for rate of occurrence vs. flood level for each flood source.

Wave Setup Analysis

Wave setup was computed during the storm surge modeling through the methods and models listed in Table 14 and included in the frequency analysis for the determination of the total stillwater elevations. The oscillating component of wave setup, *dynamic wave setup*, was calculated for areas subject to wave runup hazards.

5.3.2 Waves

A coastal wave model was used to calculate the nearshore wave fields required for the addition of wave setup effects. Three nested grids were used to obtain sufficient nearshore resolution to represent the radiation stress gradients required as ADCIRC inputs. Radiation stress fields output from the inner grids are used by ADCIRC to estimate the contribution of breaking waves (wave setup effects) to the total stillwater elevation.

5.3.3 Coastal Erosion

A single storm episode can cause extensive erosion in coastal areas. Storm-induced erosion was evaluated to determine the modification to existing topography that is expected to be associated with flooding events. Erosion was evaluated using the methods listed in Table 14. The post-event eroded profile was used for the subsequent transect-based onshore wave hazard analyses.

5.3.4 Wave Hazard Analyses

Overland wave hazards were evaluated to determine the combined effects of ground elevation, vegetation, and physical features on overland wave propagation and wave runup. These analyses were performed at representative transects along all shorelines for which waves were expected to be present during the floods of the selected recurrence intervals. The results of these analyses were used to determine elevations for the 1% annual chance flood.

Transect locations were chosen with consideration given to the physical land characteristics as well as development type and density so that they would closely represent conditions in their locality. Additional consideration was given to changes in the total stillwater elevation. Transects were spaced close together in areas of complex topography and dense development or where total stillwater elevations varied. In areas having more uniform characteristics, transects were spaced at larger intervals. Transects shown in Figure 9, "Transect Location Map," are also depicted on the FIRM. Table 16 provides the location, stillwater elevations, and starting wave conditions for each transect evaluated for overland wave hazards. In this table, "starting" indicates the parameter value at the beginning of the transect.

Wave Height Analysis

Wave height analyses were performed to determine wave heights and corresponding wave crest elevations for the areas inundated by coastal flooding and subject to overland wave propagation hazards. Refer to Figure 6 for a schematic of a coastal transect evaluated for overland wave propagation hazards.

Wave heights and wave crest elevations were modeled using the methods and models listed in Table 14, "Summary of Coastal Analyses".

Wave Runup Analysis

Wave runup analyses were performed to determine the height and extent of runup beyond the limit of stillwater inundation for the 1% annual chance flood. Wave runup elevations were modeled using the methods and models listed in Table 14.

Table 16: Coastal Transect Parameters

[Not Applicable to this Flood Risk Project]

Figure 9: Transect Location Map

[Not Applicable to this Flood Risk Project]

5.4 Alluvial Fan Analyses

This section is not applicable to this Flood Risk Project.

Table 17: Summary of Alluvial Fan Analyses

[Not Applicable to this Flood Risk Project]

Table 18: Results of Alluvial Fan Analyses

[Not Applicable to this Flood Risk Project]

SECTION 6.0 – MAPPING METHODS

6.1 Vertical and Horizontal Control

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 (NGVD29). With the completion of the North American Vertical Datum of 1988 (NAVD88), many FIS Reports and FIRMs are now prepared using NAVD88 as the referenced vertical datum.

Flood elevations shown in this FIS Report and on the FIRMs are referenced to NAVD88. These flood elevations must be compared to structure and ground elevations referenced to the same vertical datum. For information regarding conversion between NGVD29 and NAVD88 or other datum conversion, visit the National Geodetic Survey website at www.ngs.noaa.gov.

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 archived project documentation associated with the FIS Report and the FIRMs for this community. Interested individuals may contact FEMA to access these data.

To obtain current elevation, description, and/or location information for benchmarks in the area, please visit the NGS website at www.ngs.noaa.gov.

The datum conversion locations and values that were calculated for Middlesex County are provided in Table 19.

Table 19: Countywide Vertical Datum Conversion

Quadrangle Name	Quadrangle Corner	Latitude	Longitude	Conversion from NGVD29 to NAVD88 (feet)
All in Middlesex County				-0.8
Average Conversion from NGVD29 to NAVD88 = -0.8 feet				

Table 20: Stream-Based Vertical Datum Conversion

[Not Applicable to this Flood Risk Project]

6.2 Base Map

The FIRMs and FIS Report for this project have been produced in a digital format. The flood hazard information was converted to a Geographic Information System (GIS) format that meets FEMA's FIRM database specifications and geographic information standards. This information is provided in a digital format so that it can be incorporated into a local GIS and be accessed more easily by the community. The FIRM Database includes most of the tabular information contained in the FIS Report in such a way that the data can be associated with pertinent spatial features. For example, the information contained in the Floodway Data table and Flood Profiles can be linked to the cross sections that are shown on the FIRMs. Additional information about the FIRM Database and its contents can be found in FEMA's *Guidelines and Standards for Flood Risk Analysis and Mapping*, www.fema.gov/guidelines-and-standards-flood-risk-analysis-and-mapping.

Base map information shown on the FIRM was derived from the sources described in Table 21.

Table 21: Base Map Sources

Data Type	Data Provider	Data Date	Data Scale	Data Description
Digital orthophoto	MassGIS	2005	1:5,000	Orthoimagery for all FIRMs dated June 4, 2010, or July 7, 2014 (MassGIS 2005)
Digital orthophoto	USGS	2019	0.15-m resolution	Orthoimagery for all FIRMs dated December 31, 9999 (USGS 2019)
Political boundaries	MassGIS	2017	1:5,000	Municipal and county boundaries (MassGIS 2017)
Transportation features	MassGIS	2005	1:5,0000	Roads and railroads derived from orthophotography for all FIRMs dated June 4, 2010, or July 7, 2014 (MassGIS 2005)
Transportation features	MassGIS	2015	-	Railroads for all FIRMs dated December 31, 9999 (MassGIS 2015)
Transportation features	USCB	2016	-	Roads for all FIRMs in Charles Watershed dated December 31, 9999 (USCB 2016)
Transportation features	USCB	2019	-	Roads for all FIRMs in Merrimack and Nashua Watersheds dated December 31, 9999 (USCB 2019)

6.3 Floodplain and Floodway Delineation

The FIRM shows tints, screens, and symbols to indicate floodplains and floodways as well as the locations of selected cross sections used in the hydraulic analyses and floodway computations.

For riverine flooding sources, the mapped floodplain boundaries shown on the FIRM have been delineated using the flood elevations determined at each cross section; between cross sections, the boundaries were interpolated using the topographic elevation data described in Table 22. For each coastal flooding source studied as part of this FIS Report, the mapped floodplain boundaries on the FIRM have been delineated using the flood and wave elevations determined at each transect; between transects, boundaries were delineated using land use and land cover data, the topographic elevation data described in Table 22, and knowledge of coastal flood processes. In ponding areas, flood elevations were determined at each junction of the model; between junctions, boundaries were interpolated using the topographic elevation data described in Table 22.

In cases where the 1% and 0.2% annual chance floodplain boundaries are close together, only the 1% annual chance floodplain boundary has been shown. Small areas within the floodplain boundaries may lie above the flood elevations but cannot be shown due to limitations of the map scale and/or lack of detailed topographic data.

The floodway widths presented in this FIS Report and on the FIRM were computed for certain stream segments on the basis of equal conveyance reduction from each side of the floodplain. Floodway widths were computed at cross sections. Between cross sections, the floodway boundaries were interpolated. Table 2 indicates the flooding sources for which floodways have been determined. The results of the floodway computations for those flooding sources have been tabulated for selected cross sections and are shown in Table 23, “Floodway Data.”

Table 22: Summary of Topographic Elevation Data used in Mapping

Community	Flooding Source	Source for Topographic Elevation Data			
		Description	Vertical Accuracy	Horizontal Accuracy	Citation
Arlington, Town of; Belmont, Town of; Billerica, Town of; Burlington, Town of; Cambridge, City of; Everett, City of; Lexington, Town of; Lincoln, Town of; Malden, City of; Medford, City of; Melrose, City of; Newton, City of; Reading, Town of; Somerville, City of; Stoneham, Town of; Wakefield, Town of; Waltham, City of; Watertown, Town of; Weston, Town of; Wilmington, Town of; Winchester, Town of; Woburn, City of	All sources in lidar project area studied or redelineated in 2020 Charles Watershed revision (mostly in the greater Boston area)	Lidar	5.0 cm	9.25 cm	USGS 2014B

Table 22: Summary of Topographic Elevation Data used in Mapping

Community	Flooding Source	Source for Topographic Elevation Data			
		Description	Vertical Accuracy	Horizontal Accuracy	Citation
Billerica, Town of; North Reading, Town of; Wakefield, Town of; Wilmington, Town of	All sources in lidar project area studied or redelineated in 2020 Charles Watershed revision (mostly in the northeast of the county)	Lidar	8.5 cm	16.6 cm	USGS 2011
Holliston, Town of; Hopkinton, Town of; Lincoln, Town of; Natick, Town of; Sherborn, Town of; Weston, Town of	All sources in lidar project area studied or redelineated in 2020 Charles Watershed revision (mostly in the metrowest area)	Lidar	8.5 cm	16.6 cm	FEMA 2011
Acton, Town of; Ashland, Town of; Bedford, Town of; Billerica, Town of; Boxborough, Town of; Carlisle, Town of; Chelmsford, Town of; Concord, Town of; Framingham, Town of; Hopkinton, Town of; Hudson, Town of; Lincoln, Town of; Littleton, Town of; Lowell, City of; Marlborough, City of; Maynard, Town of; Natick, Town of; Sherborn, Town of; Stow, Town of; Sudbury, Town of; Tewksbury, Town of; Wayland, Town of; Westford, Town of	All sources studied or redelineated in 2014 Concord Watershed revision	Lidar	0.03 ft	N/A	PS 2010

BFEs shown at cross sections on the FIRM represent the 1% annual chance water surface elevations shown on the Flood Profiles and in the Floodway Data tables in the FIS Report. Rounded whole-foot elevations may be shown on the FIRM in coastal areas, areas of ponding, and other areas with static base flood elevations.

Table 23: Floodway Data

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	30	1,694	73,950	0.0	7.2	7.2	7.6	0.4
B	2,770	329[2]	10,014	0.3	7.2	7.2	7.6	0.4
C	2,950	533[2]	6,061	0.5	12.6	12.6	12.7	0.1
D	9,560	537	537	3.7	13.6	13.6	13.9	0.3
E	10,260	51[2]	432	4.9	14.2	14.2	14.5	0.3
F	10,340	43[2]	405	5.0	14.9	14.9	15.2	0.3
G	10,860	120	829	2.4	16.4	16.4	16.8	0.4
H	11,880	214	969	2.0	17.3	17.3	17.8	0.5
I	12,100	112	825	2.4	17.5	17.5	18.4	0.9
J	13,610	88	578	3.4	18.5	18.5	19.0	0.5
K	13,850	79	752	2.6	19.2	19.2	19.7	0.5
L	13,980	103[2]	990	2.1	19.8	19.8	20.4	0.6
M	14,240	203	1,537	1.3	19.9	19.9	20.5	0.6
N	14,460	75[2]	587	3.5	19.9	19.9	20.4	0.5
O	15,540	383[2]	2,373	0.8	22.5	22.5	23.4	0.9
P	15,620	144[2]	1,028	2.0	22.5	22.5	23.5	1.0
Q	15,800	621	2,603	0.5	22.7	22.7	23.6	0.9
R	15,900	655	2,642	0.5	22.7	22.7	23.6	0.9
S	17,590	49[2]	326	4.1	23.5	23.5	23.7	0.2
T	19,460	123[2]	922	1.1	24.9	24.9	25.6	0.7
U	21,070	284	786	1.8	28.5	28.5	28.5	0.0

[1]Feet above outlet of Lower Mystic Lake

[2]The measured top width on the FIRM may differ due to the effects of ineffective flow, the exclusion of small pocket areas due to map scale limitations, or is estimated due to HEC-RAS modeling limitations

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ABERJONA RIVER

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
V	21,250	458	2,676	0.5	28.6	28.6	29.4	0.8
W	21,270	496	2,887	0.5	28.6	28.6	29.4	0.8
X	22,910	557	2,960	0.7	28.6	28.6	29.4	0.8
Y	23,770	552	1,323	0.9	29.1	29.1	29.4	0.3
Z	25,100	578	1,529	0.8	29.5	29.5	29.7	0.2
AA	27,310	95[2]	382	2.4	36.4	36.4	37.2	0.8
AB	27,760	37	227	4.0	37.1	37.1	37.8	0.7
AC	30,130	46	209	4.2	39.1	39.1	39.5	0.4
AD	30,360	76[2]	462	3.0	40.6	40.6	41.2	0.6
AE	31,380	347	1,606	0.9	43.9	43.9	44.4	0.5
AF	33,290	40[2]	160	4.7	45.3	45.3	45.5	0.2
AG	37,080	39[2]	173	3.9	48.3	48.3	48.5	0.2
AH	37,810	577[2]	1,722	0.4	50.3	50.3	50.5	0.2
AI	38,060	374[2]	631	2.1	50.6	50.6	50.7	0.1
AJ	38,360	83[2]	380	1.7	51.3	51.3	51.4	0.1
AK	38,860	49[2]	276	2.1	52.9	52.9	53.2	0.3
AL	42,150	18	43	3.4	54.8	54.8	54.8	0.0
AM	43,790	20[2]	80	3.2	58.8	58.8	58.9	0.1
AN	43,990	34	186	1.4	61.2	61.2	61.2	0.0
AO	44,890	25[2]	122	1.7	62.8	62.8	62.8	0.0
AP	45,140	43[2]	196	0.5	64.3	64.3	64.3	0.0

[1]Feet above outlet of Lower Mystic Lake

[2]The measured top width on the FIRM may differ due to the effects of ineffective flow, the exclusion of small pocket areas due to map scale limitations, or is estimated due to HEC-RAS modeling limitations

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ABERJONA RIVER

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
AQ	45,860	41	220	0.6	65.1	65.1	65.1	0.0
AR	46,580	39[2]	192	0.6	65.2	65.2	65.2	0.0
AS	46,940	20[2]	74	1.6	65.3	65.3	65.4	0.1
AT	50,340	15	28	3.8	78.8	78.8	78.8	0.0

[1]Feet above outlet of Lower Mystic Lake

[2]The measured top width on the FIRM may differ due to the effects of ineffective flow, the exclusion of small pocket areas due to map scale limitations, or is estimated due to HEC-RAS modeling limitations

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ABERJONA RIVER

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	130	33	148	0.9	64.3	64.3	64.3	0.0
B	2,260	68[2]	324	0.6	68.1	68.1	68.1	0.0
C	2,860	152	203	0.9	68.2	68.2	68.2	0.0
D	4,400	124[2]	713	0.5	75.8	75.8	75.8	0.0
E	6,500	18	15	2.1	78.3	78.3	78.3	0.0
F	7,880	47[2]	68	1.1	81.5	81.5	81.5	0.0
G	9,410	18[2]	27	0.5	83.0	83.0	83.0	0.0

[1]Feet above confluence with Aberjona River

[2]The measured top width on the FIRM may differ due to the effects of ineffective flow, the exclusion of small pocket areas due to map scale limitations, or is estimated due to HEC-RAS modeling limitations

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ABERJONA RIVER NORTH SPUR

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	100	77[2]	427	1.1	6.7	3.9[3]	4.1	0.2
B	250	101[2]	399	1.2	6.7	3.9[3]	4.1	0.2
C	2,960	74	381	1.2	6.7	4.1[3]	4.3	0.2
D	3,970	56[2]	372	1.5	6.7	4.5[3]	4.8	0.3
E	5,220	84	327	1.2	6.8	4.7[3]	5.0	0.3
F	7,330	500[2]	1,135	0.3	6.8	5.0[3]	5.4	0.4
G	7,770	1,556[2]	2,294	0.2	6.8	5.2[3]	5.5	0.3
H	8,010	1,675[2]	3,477	0.1	6.8	5.2[3]	5.5	0.3
I	11,625	70	569	0.8	6.9	6.4[3]	7.2	0.8

[1]Feet above confluence with Mystic River

[2]The measured top width on the FIRM may differ due to the effects of ineffective flow, the exclusion of small pocket areas due to map scale limitations, or is estimated due to HEC-RAS modeling limitations

[3]Elevation computed without consideration of backwater effects from Mystic River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ALEWIFE BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	500	16	23	6.9	190.1	190.1	190.1	0.0
B	1,360	8	25	6.4	207.1	207.1	207.9	0.8
C	2,770	100	525	0.3	223.4	223.4	223.4	0.0

[1]Feet above confluence with Reservoir No. 3

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ANGELICA BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	0	61	285	0.4	198.5	197.2[2]	198.2	1.0
B	26	53	205	0.5	198.5	197.2[2]	198.2	1.0
C	243	53	136	0.7	198.5	197.2[2]	198.2	1.0
D	502	8	26	3.9	198.5	197.2[2]	198.2	1.0
E	1,225	15	44	2.3	200.4	200.4	200.7	0.3
F	1,368	13	21	4.7	200.6	200.6	200.9	0.3
G	1,526	35	80	1.2	201.5	201.5	201.5	0.0
H	2,165	34	80	1.1	201.6	201.6	201.9	0.3
I	2,313	10	19	4.4	201.8	201.8	201.8	0.0
J	2,476	15	40	2.1	202.7	202.7	202.7	0.0
K	2,524	15	41	2.0	202.7	202.7	202.8	0.1
L	2,904	11	15	5.6	203.3	203.3	203.3	0.0
M	3,036	26	44	1.9	203.9	203.9	204.2	0.3

[1]Feet above confluence with Assabet River

[2]Elevation computed without consideration of backwater effects from Assabet River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ASSABET BRANCH NO. 3

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	2,376	12	26	4.0	192.3	192.3	193.3	1.0
B	3,796	20	54	1.9	197.1	197.1	197.6	0.5
C	4,108	8	14	7.5	202.7	202.7	203.2	0.5
D	4,224	20	35	2.4	205.8	205.8	206.5	0.7
E	4,345	14	27	3.8	206.6	206.6	207.6	1.0

[1]Feet above confluence with Assabet River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ASSABET BRANCH NO. 4

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	102	770	7,236	0.9	120.3	120.3	121.3	1.0
B	5,724	376	3,262	1.6	121.0	121.0	121.9	0.9
C	10,328	367	3,927	1.3	121.9	121.9	122.9	1.0
D	14,714	264	3,029	1.7	123.8	123.8	124.5	0.7
E	16,510	122	1,303	3.1	125.0	125.0	125.7	0.7
F	17,296	88	1,128	3.6	126.2	126.2	126.8	0.6
G	18,091	322	3,884	1.1	126.5	126.5	127.1	0.6
H	23,378	240	2,042	2.0	127.3	127.3	127.9	0.6
I	24,177	151	1,337	3.0	127.9	127.9	128.8	0.9
J	26,031	159	1,744	2.3	129.4	129.4	129.7	0.3
K	30,214	320	1,932	2.1	131.1	131.1	131.5	0.4
L	31,995	162	1,475	2.7	133.3	133.3	133.6	0.3
M	32,999	89	632	6.3	135.2	135.2	135.4	0.2
N	35,035	305	2,925	1.4	143.0	143.0	143.0	0.0
O	37,412	124	857	4.6	143.7	143.7	144.0	0.3
P	39,483	104	753	5.3	147.0	147.0	147.4	0.4
Q	40,509	105	786	5.0	151.5	151.5	151.9	0.4
R	41,398	48	424	9.3	155.3	155.3	155.4	0.1
S	41,802	52	566	7.0	159.6	159.6	159.7	0.1
T	42,065	68	691	5.7	161.4	161.4	161.4	0.0
U	43,612	93	632	6.3	163.1	163.1	163.6	0.5

[1]Feet above confluence with Concord River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ASSABET RIVER

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
V	45,644	193	1,355	2.9	172.0	172.0	172.0	0.0
W	46,318	139	1,247	3.2	175.3	175.3	175.3	0.0
X	46,897	543	4,534	0.9	179.4	179.4	179.4	0.0
Y	53,433	215	2,218	1.5	180.2	180.2	180.5	0.3
Z	56,986	328	2,990	1.1	180.4	180.4	180.6	0.2
AA	64,088	239	2,480	1.3	181.4	181.4	182.1	0.7
AB	67,647	1,082	6,645	0.5	181.5	181.5	182.3	0.8
AC	73,564	61	532	5.8	182.5	182.5	183.5	1.0
AD	74,014	95	708	4.3	186.6	186.6	186.6	0.0
AE	79,685	390	2,403	1.2	194.9	194.9	195.3	0.4
AF	84,551	290	2,267	1.3	198.5	198.5	199.5	1.0
AG	87,918	239	1,783	1.7	200.0	200.0	200.9	0.9
AH	91,272	207	1,409	2.1	201.5	201.5	202.3	0.8
AI	92,114	96	915	3.2	203.0	203.0	203.9	0.9
AJ	92,715	219	1,570	1.9	203.9	203.9	204.7	0.8
AK	93,292	81	608	4.5	204.4	204.4	205.2	0.8
AL	95,000	579	8,200	0.3	212.4	212.4	212.4	0.0
AM	97,763	324	1,789	1.5	212.6	212.6	212.7	0.1
AN	105,507	593	3,711	0.7	213.4	213.4	213.8	0.4
AO	113,023	233	1,305	1.8	214.2	214.2	214.8	0.6
AP	117,076	342	1,798	1.3	215.8	215.8	216.7	0.9

[1]Feet above confluence with Assabet River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ASSABET RIVER

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
AQ	120,493	68	501	1.3	216.2	216.2	217.0	0.8
AR	122,445	119	583	1.0	216.9	216.9	217.7	0.8
AS	123,577	445	3,903	0.2	217.2	217.2	218.0	0.8
AT	124,647	946	15,468	0.2	232.6	232.6	232.7	0.1

[1]Feet above confluence with Concord River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: ASSABET RIVER

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	1,010	130	362	1.6	198.2	198.2	198.2	0.0
B	1,950	40	80	7.4	207.3	207.3	207.5	0.2

[1]Feet above confluence with Cow Pond Brook

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BADDACOOK BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	1,700	130	850	0.6	165.1	165.1	166.1	1.0
B	2,920	50	107	4.6	172.3	172.3	173.3	1.0
C	3,800	100	615	0.5	178.1	178.1	179.1	1.0
D	5,700	150	953	0.4	182.2	182.2	183.2	1.0
E	6,730	220	1,405	0.3	182.6	182.6	183.6	1.0
F	7,430	490	2,607	0.1	182.7	182.7	183.7	1.0
G	8,575	90	475	0.4	185.9	185.9	186.9	1.0
H	9,710	70	155	1.3	187.2	187.2	188.2	1.0
I	11,125	102	192	0.7	190.1	190.1	191.1	1.0
J	11,900	25	51	3.8	193.0	193.0	194.0	1.0
K	12,400	35	110	1.8	196.4	196.4	197.4	1.0

[1]Feet above confluence with Sudbury River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BAITING BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	3,390	140	266	1.1	70.1	69.9[2]	70.9	1.0
B	4,551	180	399	0.7	70.5	70.5	71.5	1.0
C	7,313	60	117	1.6	73.9	73.9	74.9	1.0

[1]Feet above confluence with Ipswich River
[2]Elevation computed without consideration of backwater effects from Ipswich River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAR MEADOW BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A-L*	*	*	*	*	*	*	*	*
M	4,504	46	398	3.0	39.7	39.7	39.7	0.0
N	4,636	25	234	5.0	39.7	39.7	39.7	0.0
O	4,774	141	597	2.0	39.7	39.7	40.0	0.3
P	5,523	24	188	6.3	40.3	40.3	40.9	0.6
Q	6,273	77	553	2.1	40.8	40.8	41.8	1.0
R	6,490	80	490	2.4	42.8	42.8	42.8	0.0
S	6,806	79	513	2.3	42.9	42.9	43.1	0.2
T	7,155	200	330	3.6	43.7	43.7	44.0	0.3
U	7,466	80	346	3.4	45.2	45.2	45.7	0.5
V	8,169	50	221	5.3	48.4	48.4	48.7	0.3
W	8,285	50	280	4.2	50.8	50.8	51.1	0.3
X	8,427	60	325	3.6	51.2	51.2	51.2	0.0
Y	8,929	20	153	6.5	51.5	51.5	52.1	0.6
Z	9,198	120	578	1.7	51.8	51.8	52.8	1.0
AA	9,568	318	2,279	0.4	51.9	51.9	52.9	1.0
AB	9,996	217	1,821	0.5	51.9	51.9	52.9	1.0
AC	10,677	134	934	1.1	51.9	51.9	52.9	1.0
AD	12,086	86	385	2.6	52.8	52.8	53.7	0.9
AE	13,259	109	177	5.7	60.7	60.7	61.3	0.6
AF	13,861	51	125	8.0	72.4	72.4	72.8	0.4

[1]Feet above confluence with Charles River
*No data available

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 1

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	863	58	155	6.6	110.8	110.8	111.2	0.4
B	1,663	81	248	4.2	118.6	118.6	118.8	0.2
C	2,264	26	178	4.6	124.0	124.0	124.1	0.1
D	3,370	174	466	2.2	124.3	124.3	124.6	0.3
E	3,842	11	69	7.7	129.4	125.8[2]	126.2	0.4
F	4,020	54	182	3.6	131.4	130.6[2]	130.9	0.3
G	4,209	25	83	11.2	132.5	132.5	132.6	0.1
H	4,384	64	309	3.7	141.5	141.5	141.5	0.0
I	5,012	68	332	2.7	142.4	142.4	142.5	0.1
J	5,970	100	338	3.7	156.9	156.9	156.9	0.0
K	7,159	60	243	4.9	162.4	162.4	162.8	0.4
L	8,050	55	318	3.0	166.1	166.1	166.9	0.8
M	9,153	95	574	1.8	168.4	168.4	169.4	1.0
N	10,296	170	790	1.5	168.6	168.6	169.6	1.0
O	11,615	115	321	4.0	169.2	169.2	170.2	1.0
P	12,615	70	236	3.2	170.5	170.5	171.5	1.0
Q	13,250	55	253	3.3	173.6	173.6	174.5	0.9
R	14,046	55	332	2.5	175.4	175.4	175.9	0.5
S	14,932	27	135	1.4	179.8	179.8	180.2	0.4
T	15,351	40	339	0.6	186.8	186.8	187.4	0.6
U	16,281	75	364	2.4	186.8	186.8	187.5	0.7

[1]Feet above confluence with River Meadow Brook
[2]Elevation computed without consideration of flooding controlled by Beaver Brook 2 Split 2

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 2

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
V	16,657	105	585	1.7	187.7	187.7	188.6	0.9
W	17,026	53	396	2.0	188.8	188.8	189.7	0.9
X	18,112	90	468	1.7	190.4	190.4	191.3	0.9
Y	19,268	46	128	4.8	193.2	193.2	193.4	0.2
Z	20,291	45	111	5.3	195.3	195.3	195.6	0.3
AA	21,221	100	486	1.8	198.9	198.9	199.1	0.2
AB	22,193	135	573	1.3	199.0	199.0	199.3	0.3
AC	23,455	33	107	4.4	199.1	199.1	199.4	0.3

[1]Feet above confluence with River Meadow Brook

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 2

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	118	33	50	3.1	112.7	112.7	112.8	0.1
B	281	33	28	5.3	113.7	113.7	113.7	0.0
C	1,112	33	34	4.4	123.6	123.6	123.6	0.0

[1]Feet above confluence with Beaver Brook 2

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 2 SPLIT 1

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	91	19	38	8.6	128.6	128.6	128.6	0.0
B	241	34	106	3.0	131.3	131.3	131.7	0.4

[1]Feet above confluence with Beaver Brook 2

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 2 SPLIT 2

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	257	36	78	6.4	177.0	177.0	177.3	0.3
B	582	36	114	4.3	182.2	182.2	182.4	0.2
C	1,155	70	105	5.3	186.7	186.7	187.1	0.4

[1]Feet above confluence with Beaver Brook 2

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 2 SPLIT 3

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	418	74	585	8.1	68.9	54.0[2]	54.4	0.4
B	803	136	1,106	4.3	68.9	62.2[2]	62.2	0.0
C	1,060	132	1,261	3.8	68.9	62.7[2]	62.7	0.0
D	1,575	105	1,179	4.0	68.9	63.0[2]	63.0	0.0
E	2,990	114	826	5.6	68.9	64.2[2]	64.2	0.0
F	3,230	85	986	4.7	68.9	67.7[2]	67.7	0.0
G	3,237	85	984	4.7	68.9	67.7[2]	67.7	0.0
H	3,390	136	1,688	2.7	79.3	79.3	79.3	0.0
I	3,680	318	3,004	1.5	79.4	79.4	79.4	0.0
J	4,398	344	2,696	1.7	79.5	79.5	79.5	0.0
K	4,937	377	3,028	1.5	79.5	79.5	79.6	0.1
L	5,370	275	2,193	2.1	80.2	80.2	80.2	0.0
M	5,741	221	1,995	2.3	80.3	80.3	80.3	0.0
N	5,857	191	1,252	3.7	80.3	80.3	80.3	0.0
O	6,034	357	1,787	2.6	85.3	85.3	86.2	0.9
P	6,877	186	1,353	3.4	86.1	86.1	86.7	0.6
Q	7,134	96	640	7.2	88.0	88.0	88.0	0.0
R	9,027	330	2,319	2.0	90.2	90.2	90.6	0.4
S	10,250	470	2,967	1.6	90.5	90.5	91.1	0.6
T	11,198	571	3,425	1.4	90.7	90.7	91.3	0.6
U	11,307	534	2,868	1.6	90.7	90.7	91.3	0.6

[1]Feet above confluence with Merrimack River

[2]Elevation computed without consideration of backwater effects from Merrimack River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 3

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
V	11,838	505	3,244	1.4	90.8	90.8	91.4	0.6
W	12,000	432	1,747	2.6	90.8	90.8	91.4	0.6
X	12,096	445	2,423	1.9	90.8	90.8	91.5	0.7
Y	12,596	240	1,643	2.8	91.0	91.0	91.7	0.7
Z	13,000	204	1,378	3.4	91.3	91.3	92.0	0.7
AA	13,342	180	1,536	3.0	92.4	92.4	93.4	1.0
AB	14,327	330	2,522	1.8	93.1	93.1	94.0	0.9
AC	14,828	292	1,543	3.0	93.3	93.3	94.2	0.9
AD	15,115	249	1,321	3.5	93.6	93.6	94.4	0.8
AE	15,828	200	1,125	3.9	94.8	94.8	95.3	0.5
AF	15,942	72	553	6.2	94.8	94.8	95.4	0.6
AG	17,216	62	374	9.1	100.3	100.3	100.3	0.0
AH	17,292	114	560	7.9	101.1	101.1	101.2	0.1
AI	17,346	72	412	10.7	101.3	101.3	101.3	0.0
AJ	17,678	68	476	9.2	105.2	105.2	105.2	0.0
AK	17,829	76	899	4.9	110.7	110.7	110.7	0.0
AL	17,868	110	1,449	3.0	111.0	111.0	111.0	0.0
AM	18,060	259	2,446	1.8	120.7	120.7	120.8	0.1
AN	20,123	82	785	5.6	121.0	121.0	121.1	0.1
AO	21,607	263	1,983	2.2	122.2	122.2	122.4	0.2
AP	22,881	325	2,577	1.7	122.6	122.6	122.9	0.3

[1]Feet above confluence with Merrimack River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 3

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
AQ	23,533	300	3,409	1.3	122.7	122.7	123.0	0.3
AR	23,722	289	2,238	2.0	123.1	123.1	123.5	0.4
AS	24,934	657	4,748	0.9	123.3	123.3	123.9	0.6
AT	25,385	705	5,092	0.8	123.4	123.4	123.9	0.5
AU	25,838	612	4,533	0.9	123.4	123.4	123.9	0.5

[1]Feet above confluence with Merrimack River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 3

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	650	27	101	8.3	207.4	204.2[2]	204.2	0.0
B	700	27	93	9.1	207.4	204.5[2]	204.7	0.2
C	740	303	1,368	0.6	207.4	206.5[2]	206.5	0.0
D	3,640	243	759	1.1	207.4	206.6[2]	206.6	0.0
E	5,580	30	105	8.1	207.4	206.9[2]	206.9	0.0
F	5,600	30	142	5.9	207.4	207.1[2]	207.7	0.6
G	5,650	96	598	1.4	208.0	208.0	208.4	0.4
H	8,170	350	1,230	0.6	208.2	208.2	208.6	0.4
I	14,219	120	462	1.6	209.0	209.0	209.5	0.5
J	16,521	200	901	0.8	210.6	210.6	211.0	0.4
K	17,815	150	660	1.1	210.7	210.7	211.2	0.5
L	18,686	100	528	1.4	211.0	211.0	211.6	0.6
M	19,679	200	1,090	0.7	211.1	211.1	211.8	0.7
N	21,131	60	363	2.1	211.1	211.1	211.8	0.7
O	22,440	100	445	1.7	211.6	211.6	212.4	0.8
P	23,575	80	509	1.3	214.0	214.0	214.5	0.5
Q	25,239	60	313	2.2	214.2	214.2	215.2	1.0
R	26,981	328	2,273	0.3	216.2	216.2	216.2	0.0
S	29,331	36	80	8.6	217.1	217.1	217.1	0.0
T	29,991	65	327	1.8	223.0	223.0	223.3	0.3
U	30,487	40	385	1.6	225.1	225.1	225.9	0.8

[1]Feet above confluence with Forge Pond
[2]Elevation computed without consideration of backwater effects from Forge Pond

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 4

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
V	33,497	200	1,792	0.3	225.2	225.2	226.0	0.8
W	34,991	80	625	1.0	225.7	225.7	226.4	0.7
X	36,316	60	445	1.4	225.7	225.7	226.5	0.8
Y	37,858	75	564	0.9	226.2	226.2	226.9	0.7
Z	39,125	125	854	0.6	226.2	226.2	227.0	0.8
AA	40,503	125	633	0.8	226.3	226.3	227.2	0.9
AB	40,714	41	149	1.7	226.3	226.3	227.3	1.0
AC	40,820	179	727	0.3	226.9	226.9	227.5	0.6
AD	41,453	110	470	0.5	226.9	226.9	227.5	0.6
AE	41,559	60	353	0.7	228.8	228.8	229.4	0.6
AF	41,717	33	249	1.0	228.8	228.8	229.4	0.6
AG	42,668	158	824	0.3	228.8	228.8	229.5	0.7
AH	43,777	121	613	0.4	228.9	228.9	229.5	0.6
AI	44,674	333	1,573	0.1	228.9	228.9	229.5	0.6
AJ	45,519	344	1,488	0.2	228.9	228.9	229.5	0.6
AK	46,406	368	1,635	0.1	228.9	228.9	229.5	0.6
AL	47,568	212	844	0.1	228.9	228.9	229.5	0.6
AM	48,571	202	767	0.2	228.9	228.9	229.5	0.6
AN	49,152	98	722	0.1	228.9	228.9	229.5	0.6
AO	50,102	212	1,031	0.1	228.9	228.9	229.5	0.6
AP	51,000	90	34	3.6	236.2	236.2	236.2	0.0

[1]Feet above confluence with Forge Pond

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 4

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
AQ	51,105	120	174	0.7	244.6	244.6	244.6	0.0
AR	51,739	50	293	0.4	244.7	244.7	244.7	0.0
AS	51,897	211	157	0.8	247.9	247.9	247.9	0.0
AT	52,425	7	23	5.3	248.0	248.0	248.2	0.2
AU	52,584	8	30	4.0	255.2	255.2	255.2	0.0
AV	52,689	8	37	3.2	256.2	256.2	256.2	0.0
AW	52,848	71	305	0.4	256.8	256.8	257.7	0.9
AX	53,587	5	14	7.1	256.9	256.9	257.7	0.8
AY	54,527	34	71	1.4	260.0	260.0	261.0	1.0
AZ	55,319	10	38	2.6	269.5	269.5	270.5	1.0
BA	55,424	10	14	6.8	274.1	274.1	274.2	0.1
BB	56,375	19	41	2.3	284.1	284.1	284.5	0.4
BC	56,480	153	478	0.2	287.3	287.3	287.3	0.0
BD	57,483	100	207	0.5	287.5	287.5	287.5	0.0

[1]Feet above confluence with Forge Pond

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 4

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	21,595	32	150	2.2	194.6	194.6	195.6	1.0
B	22,044	24	79	4.3	195.8	195.8	196.2	0.4
C	22,387	20	68	4.9	199.2	199.2	199.5	0.3
D	22,704	34	166	2.0	200.1	200.1	200.4	0.3
E	22,968	15	71	4.8	200.6	200.6	201.0	0.4
F	23,206	19	90	3.7	202.1	202.1	202.2	0.1
G	24,314	167	545	0.6	202.4	202.4	202.6	0.2
H	24,473	241	433	0.7	202.4	202.4	202.6	0.2

[1]Feet above confluence with Charles River

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER BROOK 5

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	132	155	761	0.6	137.9	137.9	138.9	1.0
B	232	158	1,243	0.4	138.7	138.7	139.7	1.0
C	576	137	927	0.5	138.7	138.7	139.7	1.0
D	681	131	698	0.6	138.7	138.7	139.7	1.0
E	781	200	1,140	0.4	138.7	138.7	139.7	1.0
F	1,542	35	60	7.5	139.7	139.7	139.7	0.0
G	2,561	30	176	2.6	142.9	142.9	143.4	0.5
H	2,699	30	158	2.9	142.9	142.9	143.9	1.0
I	2,798	30	195	2.3	143.1	143.1	144.0	0.9
J	3,701	27	194	2.3	143.7	143.7	144.6	0.9
K	3,839	27	254	1.8	146.2	146.2	147.1	0.9
L	3,981	37	309	1.5	146.2	146.2	147.1	0.9
M	5,116	80	512	0.9	146.4	146.4	147.3	0.9
N	5,238	25	248	1.8	147.4	147.4	148.4	1.0
O	5,359	72	584	0.8	147.4	147.4	148.4	1.0
P	5,977	30	322	1.4	147.5	147.5	148.5	1.0
Q	6,098	30	310	1.5	148.2	148.2	149.2	1.0
R	6,220	62	424	1.1	148.2	148.2	149.2	1.0
S	6,801	50	356	1.3	148.3	148.3	149.3	1.0
T	7,498	37	297	1.5	148.5	148.5	149.5	1.0
U	7,619	35	259	1.7	148.5	148.5	149.5	1.0

[1]Feet above Fisk Pond

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER DAM BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
V	7,740	35	248	1.8	148.5	148.5	149.5	1.0
W	7,941	31	250	1.8	148.6	148.6	149.5	0.9
X	8,073	25	238	1.9	149.0	149.0	150.0	1.0
Y	8,321	52	467	1.0	149.0	149.0	150.0	1.0
Z	8,469	44	470	1.0	150.2	150.2	151.2	1.0
AA	8,617	41	362	1.2	150.2	150.2	151.2	1.0
AB	8,939	173	1,044	0.4	150.2	150.2	151.2	1.0
AC	9,619	120	568	0.7	150.6	150.6	151.2	0.6
AD	11,124	80	320	1.2	150.7	150.7	151.4	0.7
AE	12,019	300	1,697	0.2	151.3	151.3	151.9	0.6
AF	14,319	100	377	1.0	152.8	152.8	153.8	1.0
AG	16,309	200	722	0.5	153.5	153.5	154.0	0.5
AH	17,139	400	1,728	0.2	153.8	153.8	154.2	0.4
AI	18,439	80	314	1.2	153.8	153.8	154.2	0.4
AJ	20,479	280	611	0.6	156.8	156.8	156.8	0.0
AK	20,989	450	1,437	0.3	156.8	156.8	156.9	0.1
AL	22,484	585	1,969	0.2	156.8	156.8	159.9	0.1

[1]Feet above Fisk Pond

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BEAVER DAM BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	1,140	195	85	3.8	212.2	212.2	212.7	0.5
B	1,880	200	446	0.7	212.7	212.7	213.3	0.6
C	2,630	210	489	0.7	212.8	212.8	213.5	0.7
D	3,090	250	1,138	0.3	218.9	218.9	218.9	0.0
E	3,375	205	643	0.5	218.9	218.9	218.9	0.0
F	3,810	175	1,307	0.2	225.7	225.7	225.7	0.0
G	4,300	70	207	1.6	225.8	225.8	225.8	0.0
H	5,240	120	597	0.5	243.4	243.4	243.4	0.0
I	5,930	75	109	3.0	244.2	244.2	244.5	0.3
J	7,520	40	100	3.2	250.5	250.5	250.8	0.3
K	8,750	60	127	1.6	254.9	254.9	255.5	0.6
L	10,105	50	125	1.7	256.1	256.1	256.6	0.5
M	10,540	157	1,013	0.2	261.8	261.8	261.8	0.0
N	11,235	166	747	0.3	261.9	261.9	261.9	0.0
O	11,680	68	165	1.3	261.9	261.9	261.9	0.0
P	12,080	130	474	0.4	265.5	265.5	265.5	0.0

[1]Feet above confluence with Spectacle Pond

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BENNETTS BROOK

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE[1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/ SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	420	100	45	2.2	185.6	185.6	186.6	1.0
B	4,580	90	23	7.6	192.2	192.2	193.2	1.0
C	2,460	60	115	1.6	193.7	193.7	194.7	1.0

[1]Feet above confluence with East Outlet

TABLE 23

FEDERAL EMERGENCY MANAGEMENT AGENCY

MIDDLESEX COUNTY, MA

(ALL JURISDICTIONS)

FLOODWAY DATA

FLOODING SOURCE: BIRCH MEADOW BROOK