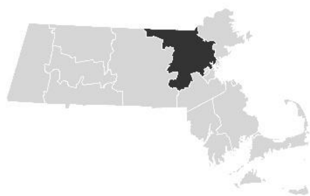
FLOOD INSURANCE STUDY

FEDERAL EMERGENCY MANAGEMENT AGENCY

VOLUME 3 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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TABLE OF CONTENTS
Volume 1

Figures

Tables

Volume 2

Figures

Tables

Volume 3

Tables

Volume 4

Figures

Tables

Volume 5

Tables

Volume 6

Tables

Volume 7

Tables

Page

Volume 8

Page

Tables

Page

Volume 9

Exhibits

Volume 10

Exhibits

Volume 11
Exhibits

Volume 12
Exhibits

Volume 13

Exhibits

Volume 14

Exhibits

Volume 15
Exhibits

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
Content Brook	Confluence with Shawsheen River	Billerica/ Tewksbury corporate limits	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.
Content Brook	Billerica/ Tewksbury corporate limits	Confluence with Middlesex Canal	TR-55 (SCS 1986)	HEC-2 (USACE 1984)	7/1/1983	AE w/ Floodway	Essentially a downstream continuation of Middlesex Canal. TR-55 (SCS 1986) was used in conjunction with the National Engineering Handbook (SCS 1972a). Cross-sections were from field surveys and topographic maps (Green 1971). Structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from known water-surface elevations from the downstream study in the Town of Tewksbury.
Content Brook	Confluence with Middlesex Canal	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.
Content Brook Tributary A	Confluence with Content 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.

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
Course Brook	Approximately 500 feet below Pond Street	Approximately 250 feet above Merchant Road	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. 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
Cow Pond Brook	Lower Massapoag Pond	Upper Massapoag Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Cow Pond Brook	Upper Massapoag Pond	Lost Lake	Rainfall-runoff routing (SCS 1972a)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	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 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.
Cow Pond Brook Tributary A	Confluence with Cow Pond 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.
Cow Pond Brook Tributary B	Confluence with Cow Pond 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.
Cranberry Brook	Confluence with Hop Brook	Approximately 0.8 mile above confluence with Hop Brook	Regression equations (Jennings et al. 1994)	HEC-2 (USACE 1984)	2/1/1996	AE	Formerly known as Tributary C 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).
Crooked Springs Brook	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
Crystal Lake 1	Entire shoreline	Entire shoreline	Regression equations (Zarriello 2017)	Weir rating	4/30/2018	A	Discharges were from regression equations (Zarriello 2017). Water-surface elevations were computed using the Francis formula for flow through a rectangular weir (slightly modified from Horton 1906) by modeling the outlet structure (dimensions and elevation provided by the City of Newton) as a vertical box outlet with weir flow over one of the long sides, the top of the structure acting as the control during high flow.
Crystal Lake 2	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 (88.3 feet NAVD88).
Cummings Brook	Confluence with Shakers Glen Brook	Approximately 130 feet above Winn Street	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.

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
Cummings Brook	Approximately 130 feet above Winn 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.
Dakins Brook	Confluence with Assabet River	Approximately 1,060 feet above Barnes Hill Road	Discharge-frequency relationships	HEC-2 (USACE 1984)	4/1/1977	AE	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.
Danforth Brook	Confluence with Assabet River	County boundary	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	12/1/1977	AE w/ Floodway	All geometry data were from field surveys. Roughness factors were from field investigations and aerial photographs. Starting water-surface elevations were from known water-surface elevations on Assabet River.
Darby Brook	Confluence with Marshall Brook	Approximately 1,000 feet above State Route 38	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 Marshall Brook.
Darby Brook	Approximately 1,000 feet above State Route 38	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.
Dark Hollow 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 (153.6 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
Davis Brook	Confluence with Charles River	Rockland Street	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	5/1/1978	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 using linear extrapolation of a log-Pearson type III distribution (WRC 1976). Overbank portions of cross-sections were from topographic maps (Avis 1977). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial imagery (Avis 1977). Starting water-surface elevations were from known water-surface elevations on Charles River. The hydraulic model was calibrated to high-water marks from the August 1955 historic flood.
Dead 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 (300.8 feet NAVD88).
Deep Brook	Confluence with Merrimack River	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.
Deep Brook Tributary A	Confluence with Deep 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.
Deer Brook	Confluence with Beaver Brook 6	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
Dirty Meadow Brook	Confluence with Bogastow Brook	Holliston/ Sherborn corporate limits	Regional frequency method (SCS 1972a)	HEC-2 (USACE 1984)	12/1/1978	AE w/ Floodway	Due to large standard errors from the regional frequency method, 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 aerial photogrammetry. Structures were from field surveys. Roughness factors were based on field inspections and informed by Chow 1959. Starting water-surface elevations were from the slope-area method.
Dirty Meadow Brook	Holliston/ Sherborn corporate limits	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.
Doleful 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 (146.9 feet NAVD88).
Dopping Brook	Confluence with Bogastow Brook	Approximately 3,000 feet above Washington Street	Regional frequency method (SCS 1972a)	HEC-2 (USACE 1984)	12/1/1978	AE w/ Floodway	Due to large standard errors from the regional frequency method, 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 aerial photogrammetry. Structures were from field surveys. Roughness factors were based on field inspections and informed by Chow 1959. Starting water-surface elevations were from the slope-area method.
Dopping Brook	Approximately 3,000 feet above Washington 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.

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
Dopping Brook Tributary A	Confluence with Dopping Brook	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.
Double Brook	Confluence with Beaver Brook 3	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Dudley Brook	Confluence with Hop Brook	Confluence with Tributary A to Dudley Brook	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	11/1/1979	AE w/ Floodway	Essentially a downstream continuation of Tributary A to Dudley Brook. 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 (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977). Starting water-surface elevations were from known water-surface elevations on Hop Brook.
Dug 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 (148.5 and 149.0 feet NAVD88).
East Outlet	Confluence with Sudbury River	Diversion from Baiting Brook	TR-20 (SCS 1965)	WSP-2 (SCS 1976)	7/1/1989	AE w/ Floodway	All geometry data were provided by the SCS and the Town of Framingham. Roughness factors were estimated based on field inspections and aerial imagery (Teledyne 1977). Starting water-surface elevations were determined by TR-20.

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
Eastern Canal	Confluence with Merrimack Canal	Diversion from Pawtucket 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.

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
Elizabeth Brook 1	Approximately 7,200 feet above confluence with Assabet River	Delany Street	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
Elizabeth Brook 2	Approximately 895 feet below Hoffman Dam	County boundary	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. Starting water-surface elevations were from known water-surface elevations at the downstream end.

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
Ell Pond	Entire shoreline	Entire shoreline	HEC-HMS	SWMM 5.1	12/3/2018	AE	Engineering was performed under LOMR 18-01-0626P. The HEC-HMS rainfall-runoff model used the SCS Curve Number method (SCS 1986) to generate inflow hydrographs for each profile. Rainfall statistics were from recently updated hyetographs (NOAA 2015). The SWMM hydraulic model included representations of the stage and storage in the pond as well as the outlet hydraulics of Ell Pond Brook Culvert, Ell Pond Storm Drain, and the hydraulically actuated crest gate. The operations manual for the crest gate (maintained by City of Melrose) informed the logic used for activation.
Elliott Street 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 (338.2 feet NAVD88).
Elm Brook	Confluence with Shawsheen River	Bedford/ Concord corporate limits	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.
Elm Brook	Bedford/ Concord corporate limits	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
Elm Brook Tributary A	Confluence with Elm 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.
Elm Street 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 (265.7 feet NAVD88).
Emerson Brook Tributary A	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.
Emerson Brook Tributary A1	Confluence with Emerson Brook Tributary A	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Emerson Brook Tributary A2	Confluence with Emerson Brook Tributary A	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
Farley Brook	Confluence with River Meadow Brook	Approximately 1,800 feet above Sierra Drive	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
Farley Brook Split 1	Confluence with Farley Brook	Diversion from Farley Brook	none	HEC-RAS 4.1	10/1/2012	AE w/ Floodway	The mainstem HEC-RAS model was used to determine the amount of flow that can be conveyed by the mainstem. Overflow discharge was diverted into a side channel in a separate HEC-RAS model that covers the side channel from diversion to confluence downstream. The outputs of the two models were merged together at junctions.
Farm 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 (152.9 feet NAVD88).
Farrar Pond Brook	Farrar Pond	Small unnamed pond south of Mount Misery	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.
Fawn Lake	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 (179.8 feet NAVD88).
Fells Reservoir	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 (262.6 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
Fellsmere 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 (51.0 feet NAVD88).
First 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 (149.5 feet NAVD88).
Fiske Brook Tributary A	Entrance to underground drainage above Broadway	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.
Flat 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 (265.3 feet NAVD88).
Fort Hill 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 (775.4 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
Fort Meadow Brook	Approximately 0.2 mile north of Main Street	Fort Meadow Reservoir	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	12/1/1977	AE w/ Floodway	All geometry data were from field surveys. Roughness factors were from field investigations and aerial photographs. Starting water-surface elevations were from known water-surface elevations on Assabet River.
Fort Pond Brook	Confluence with Nashoba Brook	Acton/ Boxborough corporate limits	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	12/1/1985	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 computed from regression analyses of the other discharges. 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 aerial photographs (Sewall 1984a). 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 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
Fort Pond Brook Branch 1	Approximately 180 feet below Conant Street	Approximately 2,700 feet above Rockland Avenue	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ Floodway	Essentially an upstream continuation of Pratts Brook. 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 known water-surface elevations at Pratts 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
Fort Pond Brook Branch 2	Confluence with Fort Pond Brook	Approximately 0.25 mile above Sargent Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	4/1/1977	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 computed from a log-Pearson type III distribution of the three lower floods using regional skew coefficients (WRC 1976). Results were compared with a log-Pearson type III analysis of streamgage records on Heath Hen Meadow Brook and with downstream discharges from the Town of Acton. Comparisons supported a drainage-area ratio method on Fort Pond Brook Branch with an exponent of 0.89. Cross-sections and structures were from field surveys. Roughness factors were from field observation and aerial photographs (ASS 1970). Starting water-surface elevations were from known water-surface elevations on Fort Pond Brook. The hydraulic model was calibrated to the August 1955 historic flood.
Gilson Brook	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.
Gilson Brook Tributary A	Confluence with Gilson 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.
Grass 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 (195.8 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
Grassy Pond Brook	Confluence with Fort Pond Brook	Approximately 100 feet above State Route 2	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	12/1/1985	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 computed from regression analyses of the other discharges. 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 aerial photographs (Sewall 1984a). 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 Fort Pond Brook.
Graves Pond Brook	Confluence with Bixby Brook	Approximately 500 feet below Emery Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Graves Pond Brook	Approximately 500 feet below Emery Road	Approximately 700 feet above Emery Road	Rainfall-runoff routing (SCS 1972a)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	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 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.
Graves Pond Brook	Approximately 700 feet above Emery 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
Great Road Tributary	Confluence with Beaver Brook 4	Access road behind Littleton High School	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	10/1/1980	AE w/ Floodway	Discharges on the uppermost portion of the reach were computed by transposing the discharges from downstream using a drainage-area ratio (WRC 1977). Overbank portions of cross-sections were from aerial photographs (Quinn 1978a). Underwater portions and all structures were from field survey. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from the slope-area method.
Greens Brook	Confluence with Varnum Brook	Approximately 200 feet above Elm 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.
Greenwood 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 (68.4 and 69.1 feet NAVD88).
Groton School 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 (289.8 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
Guggins Brook	Confluence with Inch Brook	Approximately 800 feet north of Depot Road	Regression equations	HEC-2 (USACE 1984)	4/1/1977	AE w/ Floodway	In the Town of Boxborough, regression equations (Johnson and Tasker 1974) were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were computed from a log-Pearson type III distribution of the three lower floods using regional skew coefficients (WRC 1976). Results were compared with a log-Pearson type III analysis of streamgage records on Heath Hen Meadow Brook. Comparisons supported a drainage-area ratio method on Guggins Brook with an exponent of 0.88. In the Town of Acton, regression equations (Wandle 1983) were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were computed from regression analyses of the other discharges. 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. Cross-sections and structures were from field surveys. Roughness factors were from field observation and aerial photographs (ASS 1970). Starting water-surface elevations were from known water-surface elevations on Inch Brook. The hydraulic model was calibrated to the August 1955 historic flood.
Gulf 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.
Gulf Brook Tributary A	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.
Gulf Brook Tributary A1	Confluence with Gulf Brook Tributary A	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
Gulf Brook Tributary A2	Confluence with Gulf Brook Tributary A	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.
Gulf Brook Tributary B	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.
Gumpas Pond Brook	Confluence with Beaver Brook 3	County boundary	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	5/1/1978	AE	The reach in Middlesex County is very short and completely controlled by backwater from Beaver Brook 3. There are no mapped cross-sections or floodway data. Discharges were taken from the upstream study in the Town of Pelham, New Hampshire. 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 normal depth.
Hales Brook	Confluence with River Meadow Brook	Approximately 4,000 feet above Chelmsford/ Lowell corporate limits	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	3/1/1978	AE w/ Floodway	Cross-sections were from field surveys and topographic maps (EFB 1976). Roughness factors were based on field inspections and informed by Chow 1959 and Barnes 1967. 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
Halls Brook	Confluence with Aberjona River	Approximately 220 feet above Merrimac Street	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.
Hammond 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 (165.6 feet NAVD88).
Hanlon Road swamp	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 (283.0, 283.4, and 284.1 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
Hauk Brook	Confluence with Salmon 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.
Hauk Brook Tributary A	Confluence with Hauk 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.
Hawley Road 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 (81.0 feet NAVD88).
Hayward Brook	Confluence with Pine Brook	Approximately 250 feet above Boston and Maine Railroad	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	4/1/1985	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 photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial photography (Teledyne 1977). Starting water-surface elevations were from the slope-area method.
Heald Street 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 (428.2 feet NAVD88).
Heath Brook	Confluence with Shawsheen River	Foster Street	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.

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
Heath Brook	Foster Street	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
Heath Hen Meadow Brook	Confluence with Fort Pond Brook	Approximately 3,900 feet below Acton Road	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
Heath Hen Meadow Brook Split 1	Confluence with Heath Hen Meadow Brook	Diversion from Heath Hen Meadow Brook	none	HEC-RAS 4.1	10/1/2012	AE	The mainstem HEC-RAS model was used to determine the amount of flow that can be conveyed by the mainstem. Overflow discharge was diverted into a side channel in a separate HEC-RAS model that covers the side channel from diversion to confluence downstream. The outputs of the two models were merged together at junctions.
Heron 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 (74.1 feet NAVD88).
Hobbs Brook 1	Confluence with Stony Brook 1	Cambridge Reservoir	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.
Hobbs Brook 1	Cambridge Reservoir	Lexington/ Lincoln corporate limits	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
Hobbs Brook 2	Lexington/ Lincoln corporate limits	Approximately 0.5 mile above Lexington/ Lincoln corporate limits	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.
Hobbs Brook 2	Approximately 0.5 mile above Lexington/ Lincoln corporate limits	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.
Hobbs Brook Tributary A	Confluence with Hobbs 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.
Hobbs Brook Tributary B	Confluence with Hobbs 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.
Hobbs Brook Tributary C	Confluence with Hobbs Brook 2	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.
Hobbs Brook Tributary D	Confluence with Hobbs Brook 2	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.
Hobbs Brook Tributary E	Confluence with Hobbs Brook 2	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
Hog Brook	Confluence with Assabet River	Approximately 0.6 mile above River Street	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	12/1/1977	AE w/ Floodway	All geometry data were from field surveys. Roughness factors were from field investigations and aerial photographs. Starting water-surface elevations were from known water-surface elevations on Assabet River.
Hollis Street 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 (360.3 feet NAVD88).
Hollis Street 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 (343.2 feet NAVD88).
Hop Brook	Confluence with Landham-Allowance Brook	Hager Pond	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. Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977, Marlborough undated). Underwater portions and all structures were from field surveys. Roughness factors were estimated based on field inspections and aerial imagery (Teledyne 1977). Starting water-surface elevations were from known water-surface elevations on Landham-Allowance Brook.
Hopping Brook	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.

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
Hopping Brook Tributary D	Confluence with Hopping Brook	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.
Hopping Brook Tributary D1	Confluence with Hopping Brook Tributary D	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.
Hopping Brook Tributary E	Confluence with Hopping Brook	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.
Horn Pond Brook / Fowle Brook	Confluence with Aberjona River	Confluence with Shakers Glen Brook	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.

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
Inch Brook	Confluence with Fort Pond Brook	Confluence of Guggins Brook	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	12/1/1985	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 computed from regression analyses of the other discharges. 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 aerial photographs (Sewall 1984a). 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 Fort Pond Brook.
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.
Indian Brook 2	Confluence with Charles 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.

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
Ipswich River	County boundary	Burlington/ Wilmington corporate limits	Log-Pearson type III flood frequency analysis	HEC-RAS 5.0 (USACE 2016)	6/1/2017	AE w/ Floodway	Bulletin 17B flood-frequency analyses (IACWD 1982), modified with the expected moments algorithm (Cohn 1997, Cohn 2001, Griffis 2004), were performed on USGS streamgages 01101500 and 01102000 with data from water years 1938 to 2015 and 1931 to 2015, respectively. Estimated at-site discharges were weighted with regression estimates. Peak flows upstream of, downstream of, or between the gages were computed using an equation for ungaged sites on gaged streams (Guimaraes and Bohman 1992, Stamey and Hess 1993). For sites above Bear Meadow Brook, where the drainage area is less than 50% of the nearest streamgage's, regression estimates were applied instead of the equation. Just above Main Street in Wilmington, peak flows were decreased to account for a small flow diversion during large events. The diversion drains into Maple Meadow Brook, where the diverted flows were added back into the model. 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.
Ipswich River	Burlington/ Wilmington corporate limits	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.
Ipswich River Diversion	Confluence with Ipswich River	Diversion from Ipswich River	Conveyance computations	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	Discharges were determined by conveyance computations at the point of diversion from Ipswich River. Cross sections were placed at entrances and exits of structures and at significant changes in stream morphology. Cross-section geometries were taken from lidar topography (FEMA 2011, USGS 2011, USGS 2014b). Roughness was estimated from drainage area. Starting water-surface elevations were from normal depth using slope of lower end of reach. Ineffective flow was applied where applicable.

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
Ipswich River Tributary D	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.
Ipswich River Tributary E	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.
James Brook	Confluence with Nashua River	Just above Court Street	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 known water-surface elevations from the Town of Ayer.
James Brook Tributary A	Confluence with James 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.
James Brook Tributary B	Confluence with James 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.
James Brook Tributary B1	Confluence with James 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.

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
Jar Brook	Factory Pond	Meadowbrook Lane	Regional frequency method (SCS 1972a)	HEC-2 (USACE 1984)	12/1/1978	AE w/ Floodway	Essentially an upstream continuation of Bogastow Brook. Due to large standard errors from the regional frequency method, 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 aerial photogrammetry. Structures were from field surveys. Roughness factors were based on field inspections and informed by Chow 1959. Starting water-surface elevations were from rating curves developed for Factory Pond Dam and Houghton Pond Dam.

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
Jenny Dugan Brook	Confluence with Sudbury River	Approximately 3,200 feet above Williams Road	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
Jodi Drive ponding	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 (133.9 feet NAVD88).
Joint Grass Brook	Confluence with Salmon 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.
Joint Grass Brook Tributary A	Confluence with Joint Grass 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.
Joint Grass Brook Tributary B	Confluence with Joint Grass 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.
Joint Grass Brook Tributary C	Confluence with Joint Grass 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.
Joint Grass Brook Tributary D	Confluence with Joint Grass 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.
Joint Grass Brook Tributary D1	Confluence with Joint Grass Brook Tributary D	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.
Joint Grass Brook Tributary D2	Confluence with Joint Grass Brook Tributary D	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
Jones Brook	Confluence with Shawsheen River	Approximately 780 feet above Baldwin Road	TR-55 (SCS 1986)	HEC-2 (USACE 1984)	7/1/1983	AE w/ Floodway	TR-55 (SCS 1986) was used in conjunction with the National Engineering Handbook (SCS 1972a). Cross-sections were from field surveys and topographic maps (Green 1971). Structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from the slope-area method.
Jones Brook	Approximately 780 feet above Baldwin 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.
Keyes Brook	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.
Keyes Brook Tributary A	Confluence with Keyes 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.

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
Kiln Brook	Confluence with North Lexington Brook	Interstate 95	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	4/1/1977	AE w/ Floodway	The 10-year record of the USGS streamgage on Beaver Brook in Belmont was extended by correlation with the 44-year record of the Charles River streamgage in Waltham (WRC 1976). A log-Pearson type III analysis was performed on the 10-year and the extended record using a regionalized skew coefficient. These streamgage statistics were divided by results of the regression equations (Johnson and Tasker 1974) at the streamgage for the 10-, 2-, and 1-percent-annual-chance discharges to obtain multiplication factors for regression equation results upstream of the gage. At Kiln Brook locations with similar basin characteristics to Beaver Brook at the gage, the 10-, 2-, and 1-percent-annual-chance discharges were computed from the regression equations, multiplied by the above streamgage factors, and used to extrapolate the 0.2-percent-annual-chance discharge. Discharges for locations elsewhere on Kiln Brook were determined from a drainage-area ratio formula. Overbank portions of cross-sections were from topographic maps (Sewall 1972). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial photographs (ASS 1970). Starting water-surface elevations were from known water-surface elevations on North Lexington Brook.
Kiln Brook	Interstate 95	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.
Kiln Brook Tributary A	Confluence with Kiln 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.
Kiln Brook Tributary B	Confluence with Kiln 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.

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
King Street Tributary	Confluence with Beaver Brook 4	Access road behind Littleton High School	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	10/1/1980	AE w/ Floodway	Overbank portions of cross-sections were from aerial photographs (Quinn 1978a). Underwater portions and all structures were from field survey. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from the slope-area method.
Lake Maspenock	Entire shoreline	Entire shoreline	Regression equations (Zariello 2017)	HEC-RAS 5.0 (USACE 2016)	12/1/2021	A	The lake was mapped with the water-surface elevations at the upper end of the Mill River model.
Lake Quannapowitt	Entire shoreline	Entire shoreline	TR-55 (SCS 1986)	Weir rating	12/1/1976	AE	Water-surface elevations were determined independently of the Saugus River model. Volume and rate of runoff tributary to Lake Quannapowitt were computed using TR-55 (SCS 1986) and TP-149 (SCS 1973a). Rainfall data input for the models were from TR-40 (USWB 1963). The computed runoff was routed through the lake (Fair and Geyer 1954) to compute an outflow hydrograph. A stage-discharge rating curve at the outlet weir was used to compute water-surface elevations.
Landham-Allowance Brook	Confluence with Sudbury River	Edmands Road	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 (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977). 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
Laurelwoods Drive 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 (600.4 feet NAVD88).
Lawrence Brook	Confluence with Merrimack River	Lakeview Avenue	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	1/1/1981	AE w/ Floodway	Discharges were compared to statistically analyzed data from nearby streamgages with similar basin characteristics using a drainage-area ratio with an exponent of 0.75. Overbank portions of cross-sections were from topographic maps (Teledyne 1979). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and engineering judgment. Starting water-surface elevations were from normal depth.
Little Brook	Confluence with Cummings Brook	Approximately 400 feet above Bedford 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.

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
Locke Brook	Confluence with Willard Brook	Approximately 2,000 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.
Locke Brook	Approximately 2,000 feet above West Meadow Road	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Locke Brook Tributary A	Confluence with Locke Brook	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Long Pond Brook	Balch Pond	Approximately 1.8 miles above confluence with Tributary to Nonacoicus Brook 1	Rainfall-runoff routing (SCS 1972a)	HEC-2 (USACE 1984)	1/1/1978	AE	Essentially an upstream continuation of Tributary to Nonacoicus Brook 1. 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 known water-surface elevations on Tributary to Nonacoicus Brook 1.

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
Long Pond Brook	Approximately 1.8 miles above confluence with Tributary to Nonacoicus Brook 1	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.
Long Pond Brook Tributary A	Confluence with Long Pond 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.
Lower Spot Pond Brook	Malden Tunnel inlet	Outlet of Spot Pond Brook Branch and Ell Pond Brook Branch	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	10/1/1985	AE	Essentially an upstream continuation (disconnected by tunnel) of Malden River. Discharges were calculated using regression equations (Wandle 1983) and then adjusted by a basin development factor to account for urbanization (Sauer et al. 1983). Overbank portions of cross-sections were from topographic maps (RMS 1978, Sewall 1984b). Underwater portions and structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from the intake of the Winter Street shaft.
Lubbers Brook	Confluence with Ipswich River	Billerica/ Burlington corporate limits	HEC-1 (USACE 1981)	HEC-2 (USACE 1984)	4/1/1997	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.

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
Malden River	Amelia Earhart Dam	Malden Tunnel outlet	MITCAT (CDM 1980)	FLOW2D (CDM 1983)	10/1/1985	AE	Essentially a downstream continuation (disconnected by tunnel) of Lower Spot Pond Brook. Rainfall-runoff analysis using MITCAT (CDM 1980) was performed under the Mystic River Comprehensive Hydrology Study (CDM 1981) to generate hydrographs to be used in the hydraulic model. Because of the Amelia Earheart Dam and the lake systems, the Mystic-Malden River system behaves more like a series of reservoirs than like free-flowing streams. Peak stages are not the result of peak discharges and generally do not occur at the time of peak discharge. They are not caused by peak discharge but rather by runoff volume and basin storage relationships. Cross-sections and structures were from the Mystic River Comprehensive Hydrology Study (CDM 1981). Roughness factors were from field observations and engineering judgment. The FLOW2D hydraulic model takes into account the hydraulic properties of the reach and structures, storm hydrographs, storage-elevation relationships, and the operating plan of the Amelia Earheart Dam. The hydraulic model was calibrated to historic flood profiles. The output time-series of elevations for each profile was analyzed statistically to determine recurrence intervals of peak stages, which were used to create the water-surface profile.
Malden River Tributary A	Entrance to underground drainage at Forest Dale Cemetery	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.
Mallard Way 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 (162.1 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
Maple Meadow Brook	Confluence with Ipswich River	Power company easement bridge	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	8/1/1978	AE w/ Floodway	Percentage of impervious land was considered in determining the discharge. Hydrologic routings using the Muskingum Method (Linsley and Franzini 1972) were also used to take into account the moderating influence of storage in the swamplands along the brook. The routed reach was from the upstream limit of detailed study to the confluence of an unnamed tributary. Discharges computed by regression equations and routing analysis were compared to streamflow records at USGS streamgage 01101300 (11 years of record). 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.
Maple Meadow Brook	Power company easement bridge	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.
Marginal Brook	Confluence with Concord River	Lowell/ Tewksbury corporate limits	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	2/15/1984	AE w/ Floodway	Results from regression equations were compared to log-Pearson type III streamgage statistics at nearby gages (Aberjona River at Winchester, Nashoba Brook near Acton, Stony Brook at Temple NH, and Richardson Brook near Lowell) transferred to Marginal Brook 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 known water-surface elevations on Concord 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
Marshall Brook	Confluence with Meadow Brook	Approximately 2,000 feet above State Route 38	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 Strong Water Brook.
Martins Brook	Confluence with Ipswich River	Headwaters at Martins Pond	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	11/1/1986	AE w/ Floodway	Water-surface elevations for Martins Pond are determined by the elevations for this model at the upstream end. 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.
Martins Pond Brook	Mouth at Lost Lake	Approximately 4,600 feet above mouth at Lost Lake	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.
Martins Pond Brook	Approximately 4,600 feet above mouth at Lost Lake	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.
Martins Pond Brook Tributary A	Confluence with Martins Pond 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.
Martins Pond Brook Tributary B	Confluence with Martins Pond 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.

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
Mascuppic Brook	Confluence with Lawrence Brook	Mascuppic Lake	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	1/1/1981	AE w/ Floodway	Results of the regression equations were augmented by numerical integration reservoir routing using triangular inflow hydrographs for Mascuppic Lake (USBR 1977, Viessman 1972). Discharges were compared to statistically analyzed data from nearby streamgages with similar basin characteristics using a drainage-area ratio with an exponent of 0.75. Overbank portions of cross-sections were from topographic maps (Teledyne 1979). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and engineering judgment. Starting water-surface elevations were from normal depth.
Mason Brook	Confluence with Walker 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 normal depth.
Mason Brook Tributary A	Confluence with Mason Brook	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Mayo Road 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 (1,023.1 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
McCann's Hill 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 (356.5 feet NAVD88).
McKee 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 (140.9 feet NAVD88).
Meadow Brook	Confluence with Marshall Brook	Kendall Road	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 Strong Water Brook.
Meadow Brook	Kendall 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.
Meadow Brook Tributary A	Confluence with Meadow Brook	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Meadow Brook Tributary A1	Confluence with Meadow Brook Tributary A	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Meadow River Branch	Approximately 250 feet below Lowell Street	Approximately 800 feet below corporate limits	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	4/1/1978	AE w/ Floodway	Overbank portions of cross-sections were from topographic maps (Avis 1980, Teledyne 1977). 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.

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
Merrimack Canal	Confluence with Merrimack River	Diversion from Pawtucket 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.

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
Merrimack River	County boundary	County boundary	Log-Pearson type III flood frequency analysis	HEC-2 (USACE 1984)	3/1/1991	AE w/ Floodway	The 10-, 2-, 1-, and 0.2-percent-annual-chance discharges were determined from a log-Pearson type III flood-frequency analysis (IACWD 1982) on USGS streamgage 01100000 (Merrimack River below Concord River in Lowell). Computed discharges were modified to account for the effect of upstream flood control reservoirs. Overbank portions of cross-sections were from topographic maps (Lowell undated, CDM undated, USACE undated). Underwater portions were from field surveys and mapping (USACE 1937, L&C 1936, P&P 1952). Topographic mapping was field-verified. Structures were from field surveys. Roughness factors were from field inspection, photographs, and engineering judgment. Starting water-surface elevations at Pawtucket Dam were from a discharge rating curve for the dam. Downstream starting water-surface elevations were from known water-surface elevations on existing downstream studies. The hydraulic model was calibrated to high-water marks from the April 1987 historic flood.
Merrimack River Tributary E	Confluence with Merrimack River	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.
Merrimack River Tributary E1	Confluence with Merrimack River Tributary E	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.
Merrimack River Tributary E1A	Confluence with Merrimack River Tributary E1	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.
Merrimack River Tributary E2	Confluence with Merrimack River Tributary E	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
Merrimack River Tributary E2A	Confluence with Merrimack River Tributary E2	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.
Merrimack River Tributary E2A1	Confluence with Merrimack River Tributary E2A	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.
Merrimack River Tributary E2B	Confluence with Merrimack River Tributary E2	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.
Merrimack River Tributary E2C	Confluence with Merrimack River Tributary E2	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.
Merrimack River Tributary E3	Confluence with Merrimack River Tributary E	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.
Merrimack River Tributary E4	Confluence with Merrimack River Tributary E	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.
Merrimack River Tributary E5	Confluence with Merrimack River Tributary E	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.
Merrimack River Tributary E5A	Confluence with Merrimack River Tributary E5	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.
Merrimack River Tributary F	Confluence with Merrimack River	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
Merrimack River Tributary F1	Confluence with Merrimack River Tributary F	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.
Merrimack River Tributary F2	Confluence with Merrimack River Tributary F	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
Middlesex Canal	Confluence with Content Brook	Pond Street	TR-55 (SCS 1986)	HEC-2 (USACE 1984)	7/1/1983	AE	Essentially an upstream continuation of Content Brook. TR-55 (SCS 1986) was used in conjunction with the National Engineering Handbook (SCS 1972a). Cross-sections were from field surveys and topographic maps (Green 1971). 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 Content Brook.
Middlesex Canal	Pond Street	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.
Middlesex Turnpike ponding	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 (150.8 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
Mill Brook 1	Confluence with Pine Brook	Claypit Hill Road	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 to existing data, where available. Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial photography (Teledyne 1977). Starting water-surface elevations were from the slope-area method.
Mill Brook 2	Confluence with Concord River	Approximately 1,000 feet above Cambridge Turnpike Dam	unknown	HEC-RAS	8/14/2015	AE w/ Floodway	Engineering was performed under LOMR 15-01-0902P. Engineering methods were not recorded.
Mill Brook 3	Confluence with Lower Mystic Lake	Approximately 40 feet above Boston and Maine Railroad	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.

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
Mill Pond Tributary	Confluence with Beaver Brook 4	Intersection of Interstate 495 and State Route 2	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	10/1/1980	AE w/ Floodway	Overbank portions of cross-sections were from aerial photographs (Quinn 1978a). Underwater portions and all structures were from field survey. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from the slope-area method.
Mill River	Confluence with Saugus River	Lowell Street	unknown	HEC-2 (USACE 1984)	12/1/1976	AE w/ Floodway	Discharges were computed by methods used by the SCS. Overflows from Crystal Lake enter Mill River via storm-drain pipes but do not contribute to peakflows because they reach Mill River long after it peaks. Cross-sections were from field surveys. Roughness factors were based on field inspections and informed by Chow 1959 and Barnes 1967. Starting water-surface elevations were from known water-surface elevations on Saugus River.
Mill River	Lowell Street	Approximately 400 feet below Interstate 95	none	none	4/30/2018	A	This reach was not modeled. A straight-line water-surface profile was interpolated between the lower end of the upper reach of Mill River (the diversion from Saugus River) and the upper end of the lower reach of Mill River (the detailed study).
Mill River	Approximately 400 feet below Interstate 95	Diversion from Saugus River	Conveyance computations	HEC-RAS 5.0 (USACE 2016)	4/30/2018	AE	Discharges were determined by conveyance computations at the point of diversion from Saugus River. Cross sections were placed at entrances and exits of structures and at significant changes in stream morphology. Overbank geometries were taken from lidar topography; channel geometries and structures were from field surveys. Roughness was estimated using engineering judgment.

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
Mill River 2 (upper)	Lake Maspenock	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	12/1/2021	A	Flow-change locations were selected based on 50% change in drainage area. Sub-basin delineation used hydro-conditioned lidar topography (FEMA 2011, USGS 2011, USGS 2014b). Cross sections were placed at entrances and exits of structures, at flow-change locations, and at significant changes in stream morphology. Overbank geometries were taken from lidar topography; channel geometries were calculated from regional bankfull equations (Bent 2006). Roughness was estimated from drainage area. Starting water-surface elevations were from normal depth using slope of lower end of reach. Ineffective flow was applied where applicable.
Millstone Hill swamp	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 (264.7 feet NAVD88).
Mine Brook	Confluence with Nissitissit River	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Mineway Brook	Confluence with Pantry Brook	Approximately 250 feet above Concord 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
Mongo Brook	Confluence with Elm Brook	Approximately 500 feet above Fern Way	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.
Morse Brook	Confluence with Nashua River	Patterson 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.
Mount Lebanon Street 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 (213.1 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
Mowry Brook	Mouth at Sudbury Reservoir	Approximately 700 feet above Phelps Street	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.
Mud Pond Brook	Confluence with Shawsheen River	Wolcott Street	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 Content Brook. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Shawsheen River.
Mud Pond Brook	Wolcott Street	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
Muddy Brook	Confluence with Heath Hen Meadow Brook	Approximately 1,400 feet above Willow Street	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
Mulpus Brook	Confluence with Nashua River	County boundary	Rainfall-runoff routing (SCS 1972a)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	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, to account for significant storage areas along the reach, which cause discharges to increase going upstream. 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
Munroe Brook	Arlington Reservoir	Approximately 100 feet above Bryant Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	4/1/1977	AE w/ Floodway	The 10-year record of the USGS streamgage on Beaver Brook in Belmont was extended by correlation with the 44-year record of the Charles River streamgage in Waltham (WRC 1976). A log-Pearson type III analysis was performed on the 10-year and the extended record using a regionalized skew coefficient. These streamgage statistics were divided by results of the regression equations (Johnson and Tasker 1974) at the streamgage for the 10-, 2-, and 1-percent-annual-chance discharges to obtain multiplication factors for regression equation results upstream of the gage. At Munroe Brook locations with similar basin characteristics to Beaver Brook at the gage, the 10-, 2-, and 1-percent-annual-chance discharges were computed from the regression equations, multiplied by the above streamgage factors, and used to extrapolate the 0.2-percent-annual-chance discharge. Discharges for locations elsewhere on Munroe Brook were determined from a drainage-area ratio formula. Overbank portions of cross-sections were from topographic maps (Sewall 1972). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial photographs (ASS 1970). Starting water-surface elevations were from a stage/discharge rating curve on the Arlington Reservoir outlet.

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
Mystic River	Amelia Earhart Dam	Outlet of Lower Mystic Lake	HEC-HMS	HEC-RAS 3.1.3	6/1/2005	AE w/ Floodway	Essentially a downstream continuation of Aberjona River. Lower and Upper Mystic Lakes are included in this model. 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.

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
Nagog Brook	Confluence with Nashoba Brook	Nagog Pond	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	12/1/1985	AE w/ Floodway	Water-surface elevations for Nagog Pond are determined by the elevations for this model at the upstream end. For most of the reach, regression equations were used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharges were computed from regression analyses of the other discharges. At the upstream end (outlet of Nagog Pond), discharges were determined using HEC-1 (USACE 1981). 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 aerial photographs (Sewall 1984a). 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 Nashoba Brook.
Nashoba Brook	Confluence with Assabet River	Acton/ Westford corporate limits	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	12/1/1985	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 computed from regression analyses of the other discharges. Discharges were compared with weighted-average discharges computed from log-Pearson type III analyses (WRC 1976) of streamgage records from Heath Hen Meadow Brook, which has similar basin characteristics. Overbank portions of cross-sections were from aerial photographs (Sewall 1984a). 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 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
Nashua River	County boundary	County boundary	Log-Pearson type III flood-frequency analysis (IACWD 2018)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	AE w/ Floodway	Bulletin 17C flood-frequency analysis (IACWD 2018), modified with the expected moments algorithm (Cohn 1997, Cohn 2001, Griffis 2004), was performed on USGS streamgages 01094400 (water years 1973 to 2016, augmented with the 1936 peak [USGS 1937]), 01094500 (1936 to 2016), 01096500 (1936 to 2016), and 01095503 (2012 to 2016, augmented with the 1936 peak [USGS 1937], 2008 to 2011 peaks from streamgage 01095505, and 2003 to 2007 and 2017 peaks from streamgage 01095500, furnished by reservoir management at Massachusetts Water Resources Authority). Estimated at-site discharges were weighted by regression estimates (USGS 2014a, Zarriello 2017) for 01094500 only; other gages are too significantly affected by regulation. For the same reason, generalized skew (Zarriello 2017) was used for 01094500 and station skew for the others. Nashua River above the confluence with North Nashua River was assumed to be non-contributing drainage area, being dominated by Wachusett Reservoir. Instead, North Nashua River was considered a hydrologic continuation of Nashua River at the confluence. Therefore, peak flows at ungaged sites were logarithmically-linearly interpolated on this assumed reach (between 01094400 and 01094500, and between 01094500 and 01096500) or extrapolated (above 01094400 and below 01096500). Roughness factors were estimated using field notes, photographs, and orthoimagery. Overbank portions of cross sections were taken from lidar topography (FEMA 2011). Structures and underwater portions of cross sections were from field surveys. Starting water-surface elevations were from normal depth.
Nashua River Tributary D	Confluence with Nashua 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
Nashua River Tributary E	Confluence with Nashua 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.
Nashua River Tributary F	Confluence with Nashua 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.
Nashua River Tributary H	Confluence with Nashua 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.
Nashua Road 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 (200.7 feet NAVD88).
New Meadow	Entire shoreline	Entire shoreline	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	AE	Stillwater elevation was taken from the upper limit of Peppermint Brook. Elevations east of New Meadow slope downhill linearly to tie in with New Meadow Brook in Hillsborough County, New Hampshire.
New Meadow Brook Tributary A	County boundary	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.
New Meadow Brook Tributary E	County boundary	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
Nissitissit River	Confluence with Nashua River	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	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.
Nissitissit River Tributary A	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.
Nissitissit River Tributary A1	Confluence with Nissitissit River Tributary A	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.
Nissitissit River Tributary B	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.
Nissitissit River Tributary C	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.
Nissitissit River Tributary D	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.
Nissitissit River Tributary E	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.

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
Nonacoicus Brook 1	Confluence with Nashua River	Headwaters at confluence of Tributary to Nonacoicus Brook 1 and Nonacoicus Brook 2	Rainfall-runoff routing (SCS 1972a)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Essentially a downstream continuation of Nonacoicus Brook 2. 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.
Nonacoicus Brook 2	Confluence with Tributary to Nonacoicus Brook 1	County boundary	Rainfall-runoff routing (SCS 1972a)	HEC-2 (USACE 1984)	1/1/1978	AE w/ Floodway	Essentially an upstream continuation of Nonacoicus Brook 1; known as Bowers Brook in Worcester County. 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.

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
North Lexington Brook	Confluence with Shawsheen River	Interstate 95 ramp	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	4/1/1977	AE w/ Floodway	The 10-year record of the USGS streamgage on Beaver Brook in Belmont was extended by correlation with the 44-year record of the Charles River streamgage in Waltham (WRC 1976). A log-Pearson type III analysis was performed on the 10-year and the extended record using a regionalized skew coefficient. These streamgage statistics were divided by results of the regression equations (Johnson and Tasker 1974) at the streamgage for the 10-, 2-, and 1-percent-annual-chance discharges to obtain multiplication factors for regression equation results upstream of the gage. At North Lexington Brook locations with similar basin characteristics to Beaver Brook at the gage, the 10-, 2-, and 1-percent-annual-chance discharges were computed from the regression equations, multiplied by the above streamgage factors, and used to extrapolate the 0.2-percent-annual-chance discharge. Discharges for locations elsewhere on North Lexington Brook were determined from a drainage-area ratio formula. Overbank portions of cross-sections were from topographic maps (Sewall 1972). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial photographs (ASS 1970). Starting water-surface elevations were from known water-surface elevations on Shawsheen River.
North Lexington Brook	Interstate 95 ramp	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.
North Lexington Brook Tributary A	Confluence with North Lexington 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.
North Lexington Brook Tributary A1	Confluence with North Lexington Brook Tributary A	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
North Lexington Brook Tributary B	Confluence with North Lexington 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.
North Reservoir	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 (140.1 feet NAVD88).
Northern Canal	Confluence with Western Canal	Diversion from Merrimack River	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.

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
Norumbega Reservoir	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 (238.6 and 266.0 feet NAVD88).
Pages Brook	Confluence with Concord River	Approximately 3,850 feet above Brook Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	4/1/1978	AE w/ Floodway	Overbank portions of cross-sections were from topographic maps (Avis 1980, Teledyne 1977). 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.
Pages Brook Branch	Confluence with Pages Brook	Approximately 1,430 above East Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	4/1/1978	AE w/ Floodway	Results of the regression equations were compared favorably to computations using TR-55 (SCS 1986). Overbank portions of cross-sections were from topographic maps (Avis 1980, Teledyne 1977). 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.
Pantry Brook	Confluence with Sudbury River	Approximately 2,800 feet above Marlboro Road	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 (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977). 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
Paul Brook	Confluence with South Meadow Brook	Boylston Street	Rainfall-runoff routing (SCS 1974)	HEC-2 (USACE 1984)	11/1/1981	AE	Essentially an upstream continuation of South Meadow Brook. Overbank portions of cross-sections were from topographic maps (Berger 1977). Underwater portions and 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 South Meadow Brook.
Pawtucket Canal	Confluence with Concord River	Diversion from Merrimack River	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.

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
Pearl Hill Brook	Confluence with Walker Brook	Approximately 8,300 feet above Main Street	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.
Pearl Hill Brook	Approximately 8,300 feet above Main Street	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	A	See special considerations for Aberjona River Tributary A.
Pearl Hill Brook Tributary A	Confluence with Pearl Hill 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.
Pearl Hill Brook Tributary B	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.
Pearl Hill Brook Tributary C	Confluence with Pearl Hill 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.
Peppermint Brook	Confluence with Beaver Brook 3	Approximately 500 feet above State Route 38	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.
Peppermint Brook Tributary A	Confluence with Peppermint 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.

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
Pine Brook	Confluence with Sudbury River	Confluence with Hayward Brook	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 to existing data, where available. Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial photography (Teledyne 1977). Starting water-surface elevations were from the slope-area method.
Pines River	Coastal flooding along Pines River and Town Line Brook	Coastal flooding along Pines River and Town Line Brook	none	none	5/30/2015	AE	Coastal stillwater elevations were taken from Suffolk County's FIS.
Pole Brook	Confluence with Sudbury River	Lincoln Road (State Route 126)	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1984)	5/1/1977	AE w/ Floodway	Farrar Pond is included in the downstream end of this model. 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.

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
Pratts Brook	Confluence with Fort Pond Brook	Approximately 180 feet below Conant Street	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ Floodway	Essentially a downstream continuation of Fort Pond Brook Branch 1. 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
Pumpkin Brook	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
Putnam Brook	Confluence with River Meadow Brook	Approximately 750 feet above Hall Road	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
Quarter Mile 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 (155.6 feet NAVD88).
Reedy Meadow Brook	Confluence with Nashua River	Approximately 2,440 feet above Groton/ Pepperell corporate limits	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	11/1/2019	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.
Reedy Meadow Brook Tributary A	Approximately 2,440 feet above Groton/ Pepperell 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.
Reedy Meadow 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 (260.4 feet NAVD88).
Reedy Meadow 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 (250.5 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
Reedy Meadow 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 (249.8 feet NAVD88).
Richardson Brook	Confluence with Merrimack River	Approximately 100 feet above Broadway Street	Rational method	HEC-RAS	9/24/2015	AE w/ Floodway	Engineering was performed under LOMR 15-01-0572P. Engineering methods were not recorded.
River Meadow Brook	Chelmsford/ Lowell corporate limits	Mill Road	Regression equations (Wandle 1983)	HEC-2 (USACE 1984)	10/1/2001	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. No adjustment for urbanization was applied because there is storage potential in the watershed. Cross-sections and 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 the downstream study in the City of Lowell (LOMR 95-01-057P).
River Meadow Brook	Confluence with Concord River	Chelmsford/ Lowell corporate limits	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	2/15/1984	AE w/ Floodway	Results from regression equations were compared to log-Pearson type III streamgage statistics at nearby gages (Aberjona River at Winchester and Nashoba Brook near Acton) transferred to River Meadow Brook 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 known water-surface elevations on Concord 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
River Street 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 (177.8 feet NAVD88).
Robinson Brook	Confluence with Bancroft 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.
Roosevelt Park 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 (9.0 feet NAVD88).
Run Brook	Confluence with Hop Brook	Approximately 150 feet above Fairbank Road	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 (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field observation and aerial photography (Teledyne 1977). Starting water-surface elevations were from known water-surface elevations on Hop 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
Salmon Brook	County boundary	Headwaters at Massapoag Pond	Regression equations (Wandle 1977)	HEC-2 (USACE 1984)	2/1/1980	AE w/ Floodway	Water-surface elevations for Massapoag Pond are determined by the elevations for this model at the upstream end. Overbank portions of cross-sections were from aerial photography (Quinn 1978b). Underwater portions and structures were from field surveys. Roughness factors were from field observations and engineering judgment. Starting water-surface elevations were from known water-surface elevations in the City of Nashua.
Salmon Brook Tributary A	County boundary	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.
Salmon Brook Tributary B	Confluence with Salmon 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.
Salmon Brook Tributary C	Confluence with Salmon 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.
Salmon Brook Tributary D	Confluence with Salmon 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.
Salmon Brook Tributary E	Confluence with Salmon 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.
Salmon Brook Tributary F	Confluence with Salmon 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.
Salmon Brook Tributary G	Confluence with Salmon 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.

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
Sandy Brook	Confluence with Vine Brook	State Route 3A	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.

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
Saugus River	County boundary	Lake Quannapowitt	Log-Pearson type III flood frequency analysis	HEC-RAS 5.0 (USACE 2016)	6/1/2017	AE w/ Floodway	Bulletin 17B flood-frequency analysis (IACWD 1982), modified with the expected moments algorithm (Cohn 1997, Cohn 2001, Griffis 2004), was performed on USGS streamgage 01102345 with data from water years 1994 to 2015. Estimated at-site discharges were weighted with regression estimates despite the presence of Lynnfield municipal water-supply reservoirs (Hawkes Pond and Walden Pond) in the basin. The reservoirs were assumed to contribute to peak flows. Peak flows upstream of the gage as far as the confluence with Mill River were computed using an equation for ungaged sites on gaged streams (Guimaraes and Bohman 1992, Stamey and Hess 1993). For sites above Mill River, regression estimates were applied instead of the equation. Peak flows for sites between Mill River and Reedy Meadow were decreased to account for a small amount of flow that leaves Saugus River under Interstate 95 at an abandoned railroad bridge and diverts into Mill River, which drains into Saugus River further downstream. 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. Since no dam operations plans were available for Lake Quannapowitt, the dam outlet structures were modeled to reproduce the effective Lake Quannapowitt elevations at the upstream end of the model.
Saunders Brook	Confluence with Shawsheen River	Shawsheen Street	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 Content Brook. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Shawsheen 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
Sawmill Brook 1	Burlington/ Wilmington corporate limits	Lucaya Circle	TR-55 (SCS 1986)	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 a rating curve on Noah Clapp Dam.
Sawmill Brook 1	Confluence with Maple Meadow Brook	Burlington/ Wilmington corporate limits	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Sawmill Brook 1	Lucaya Circle	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.
Sawmill Brook 2	Confluence with Concord River	Hutchins 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 Concord River.
Sawmill Brook 3	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.
Sawmill Brook 3 Tributary B	Confluence with Sawmill Brook 3	County boundary	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
Scarlet Brook	Confluence with Merrimack River	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.
Schneider Brook	Confluence with Aberjona River	Approximately 800 feet above Forbes Street	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.
Seaverns Brook	Confluence with Charles 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.
Seaverns Brook Tributary A	Confluence with Seaverns Brook	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.
Sewall Brook	Confluence with Charles 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.

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
Shakers Glen Brook	Confluence with Fowle Brook	Approximately 190 feet above Russell Street	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.
Shakers Glen Brook	Approximately 190 feet above Russell 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.
Shawsheen River	County boundary	Approximately 1,100 feet above access road to Hanscom Air Force Base	Regression equations (Sauer et al. 1983)	HEC-RAS 3.1.3 (USACE 2002)	4/1/2015	AE w/ Floodway	Urban regression equations were considered more applicable than rural regression equations (Wandle 1983) considering extensive watershed development since 1964. The hydraulic model was calibrated to high-water marks from historic floods in 2006 and 2007. Some structure dimensions from the previously effective HEC-2 model. Other structure dimensions and underwater portions of cross-sections were from field surveys. Overbank portions of cross-sections were from lidar data (URS 2007). The hydraulic model includes the 2009 modification of the Page Road bridge. 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
Shawsheen River Tributary E	Confluence with Shawsheen River	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.
Shawsheen River Tributary F	Confluence with Shawsheen River	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.
Shawsheen River Tributary F1	Confluence with Shawsheen River Tributary F	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.
Shawsheen River Tributary G	Confluence with Shawsheen River	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.
Sherman Lane 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 (184.5 feet NAVD88).
Shipley Hills 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 (496.1 feet NAVD88).
Shirley Street 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 (236.9 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
Shute Brook	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.
Silver Lake Outlet	Confluence with Lubbers Brook	Silver Lake	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Aberjona River Tributary A.
Skug River	Martins Pond	County boundary	Drainage-area ratio	HEC-2 (USACE 1984)	5/1/1979	AE w/ Floodway	Discharges were determined by applying a drainage-area ratio (with an exponent of 0.7) to discharges from Martins Brook downstream. Martins Brook discharges were first computed using regression equations (Wandle 1983). Roughness factors were from engineering judgment. Starting water-surface elevations were from known water-surface elevations on Martins Brook.
Skug River Tributary A	Confluence with Skug 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.
Skug River Tributary B	Confluence with Skug 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.
Skug River Tributary C	Confluence with Skug 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.
Skug River Tributary C1	Confluence with Skug River Tributary C	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.
Skug River Tributary C2	Confluence with Skug River Tributary C	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
Skug River Tributary C3	Confluence with Skug River Tributary C	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.
Skug River Tributary D	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.
Smelt Brook 1	Entrance to underground drainage above Lawrence 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.
Snake Brook	Commonwealth Avenue	Thompson Street	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 to existing data, where available. Overbank portions of cross-sections were from photogrammetric maps (Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial photography (Teledyne 1977). Starting water-surface elevations were from the slope-area method. Backwater elevations from Lake Cochituate were from a rating curve for the culvert on Commonwealth Avenue.
Snake Meadow Brook	Confluence with Keyes 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.
Snake Meadow Brook Tributary A	Confluence with Snake Meadow 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.

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
South Branch Souhegan River	County boundary	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/4/2019	A	See special considerations for Aberjona River Tributary A.
South Branch Souhegan River Tributary B	Confluence with South Branch Souhegan River	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.
South Branch Souhegan River Tributary B1	Confluence with South Branch Souhegan River Tributary B	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.
South Branch Souhegan River Tributary B2	Confluence with South Branch Souhegan River Tributary B	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.
South Meadow Brook	Approximately 250 feet above Tower Road	Confluence with Paul Brook	Rainfall-runoff routing (SCS 1974)	HEC-2 (USACE 1984)	7/1/1988	AE w/ Floodway	Essentially a downstream continuation of Paul Brook. Overbank portions of cross-sections were from topographic maps (Berger 1977). 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.
South Row 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 (376.1 feet NAVD88).
Spaulding Brook	Confluence with Keyes 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.

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
Spaulding Brook Tributary A	Confluence with Spaulding 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.

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	Angiers Pond	Approximately 1,600 feet above Russell Street	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. Starting water-surface elevations were from known water-surface elevations at the lower portion of Spencer Brook.