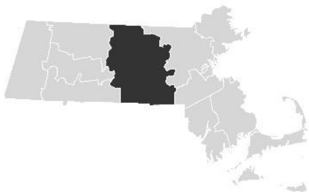
FLOOD INSURANCE STUDY

FEDERAL EMERGENCY MANAGEMENT AGENCY

VOLUME 2 OF 12



WORCESTER COUNTY, MASSACHUSETTS

(ALL JURISDICTIONS)

COMMUNITY NAME	NUMBER	COMMUNITY NAME	NUMBER
ASHBURNHAM, TOWN OF	250290	NEW BRAINTREE, TOWN OF	250320
ATHOL, TOWN OF	250291	NORTH BROOKFIELD, TOWN OF	250323
AUBURN, TOWN OF	250292	NORTHBOROUGH, TOWN OF	250321
BARRE, TOWN OF	250293	NORTHBRIDGE, TOWN OF	250322
BERLIN, TOWN OF	250294	OAKHAM, TOWN OF	250324
BLACKSTONE, TOWN OF	250295	OXFORD, TOWN OF	250325
BOLTON, TOWN OF	250296	PAXTON, TOWN OF	250326
BOYLSTON, TOWN OF	250297	PETERSHAM, TOWN OF	250327
BROOKFIELD, TOWN OF	250298	PHILLIPSTON, TOWN OF	250328
CHARLTON, TOWN OF	250299	PRINCETON, TOWN OF	250329
CLINTON, TOWN OF	250300	ROYALSTON, TOWN OF	250330
DOUGLAS, TOWN OF	250301	RUTLAND, TOWN OF	250331
DUDLEY, TOWN OF	250302	SHREWSBURY, TOWN OF	250332
EAST BROOKFIELD, TOWN OF	250303	SOUTHBOROUGH, TOWN OF	250333
FITCHBURG, CITY OF	250304	SOUTHBRIDGE, TOWN OF	250334
GARDNER, CITY OF	250305	SPENCER, TOWN OF	250335
GRAFTON, TOWN OF	250306	STERLING, TOWN OF	250336
HARDWICK, TOWN OF	250307	STURBRIDGE, TOWN OF	250337
HARVARD, TOWN OF	250308	SUTTON, TOWN OF	250338
HOLDEN, TOWN OF	250309	TEMPLETON, TOWN OF	250339
HOPEDALE, TOWN OF	250310	UPTON, TOWN OF	250340
HUBBARDSTON, TOWN OF	250311	UXBRIDGE, TOWN OF	250341
LANCASTER, TOWN OF	250312	WARREN, TOWN OF	250342
LEICESTER, TOWN OF	250313	WEBSTER, TOWN OF	250343
LEOMINSTER, CITY OF	250314	WEST BOYLSTON, TOWN OF	250345
LUNENBURG, TOWN OF	250315	WEST BROOKFIELD, TOWN OF	250346
MENDON, TOWN OF	250316	WESTBOROUGH, TOWN OF	250344
MILFORD, TOWN OF	250317	WESTMINSTER, TOWN OF	250347
MILLBURY, TOWN OF	250318	WINCHENDON, TOWN OF	250348
MILLVILLE, TOWN OF	250319	WORCESTER, CITY OF	250349

REVISED:

JULY 8, 2025

FLOOD INSURANCE STUDY NUMBER
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Table 9: Summary of Discharges

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Mahoney Brook	Above confluence with Foster Brook	5.2	175	*	295	355	555
Mahoney Brook	Below confluence with Greenwood Brook	5.0	165	*	270	325	510
Mahoney Brook	Above confluence with Greenwood Brook	2.0	70	*	115	140	215
McKinstry Brook	At confluence with Quinebaug River	8.0	405	*	680	835	1,285
Meadow Brook	At Oak Street	1.43	121	*	210	258	408
Middle River	At Worcester corporate limits	61.5	2,390	*	3,560	4,140	5,360
Mill Brook	At Hudson/ Bolton town line	4.9	190	*	305	362	536
Mill Brook	At Mill Road	3.5	145	*	226	262	360
Mill Brook No. 1	At confluence with French River	10.7	163	*	210	233	317
Mill Brook Conduit	At Salisbury Pond	6.39	903	*	1,144	1,303	1,587
Mill Brook Conduit	At Grove Street	6.01	792	*	986	1,113	1,344
Mill Brook Conduit	At Mill Brook Street	5.62	688	*	836	946	1,131
Mill Brook Conduit	At West Boylston Terrace	5.08	545	*	664	737	872
Mill Brook Conduit	At Neponset Street	4.5	453	*	542	602	705
Mill Brook Conduit	At confluence with Weasel Brook	4.31	414	*	498	554	656

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Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Mill Brook Conduit	At outflow from Indian Lake	2.81	78	*	106	143	195
Mill Brook Conduit	At inflow into Indian Lake	2.81	603	*	783	860	1,152
Mill River	At Harris Pond	*	1,470	*	2,590	3,100	4,600
Mill River	Above Hop Brook	*	1,300	*	2,250	2,750	3,850
Mill River	At Blackstone/ Mendon corporate limits	24.8	1,340	*	2,310	2,800	4,400
Mill River	At confluence with Round Meadow Brook	22.3	1,100	*	1,870	2,270	3,540
Mill River	At confluence with Muddy Brook	13.7	510	*	830	1,000	1,520
Mill River	1,200 feet above Neck Hill Road	13.1	470	*	760	910	1,390
Mill River	1,000 feet below Mill Street	12.3	430	*	690	820	1,240
Mill River	400 feet above Mendon Street	11.2	360	*	570	680	1,010
Mill River	At Hopedale Pond Dam	10.8	340	*	530	630	940
Mill River	At head of Hopedale Pond	5.8	780	*	1,320	1,590	2,480
Mill River	Below Milford Street	4.4	780	*	1,320	1,590	2,480

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Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Mill River	Above Fisk Pond	2.2	540	*	880	1,050	1,620
Mill River	800 feet above Camp Road	1	300	*	450	520	760
Mill River	400 feet below Milford Street	4.4	670	*	1,110	1,330	2,070
Mill River	2,000 feet above Milford Street	3	540	*	880	1,050	1,620
Mill River	At Fiske Pond	2.2	460	*	730	860	1,320
Mill River	800 feet above Camp Road	0.9	300	*	450	520	760
Miscoe Brook	At confluence with Silver Lake (West River)	5.6	203	*	310	365	652
Miscoe Brook	Below Cider Mill Pond Dam	5.4	195	*	296	348	616
Miscoe Brook	Below Merriam Road	2.9	122	*	183	214	475
Miscoe Brook	Below Adams Road	1.4	71	*	105	121	228
Monoosnoc Brook	At confluence with North Nashua River	11.2	679	920	1,120	1,340	1,910
Monoosnoc Brook	Above confluence with tributary from Distribution Reservoir	7.36	468	636	777	928	1,330
Monoosnoc Brook	Above Pierce Pond	5.91	384	524	641	767	1,100

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			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Monoosnoc Brook	Above confluence with tributary from Mirror Lake	5.17	336	459	562	673	970
Moulton Pond Brook	At inlet to Thayer Pond	2.0	57	*	92	110	166
Moulton Pond Brook	At outlet from Moulton Pond	1.7	47	*	75	90	136
Muddy Brook	At confluence with Mill River	6.3	350	*	600	740	1,140
Muddy Brook	At confluence with Spring Brook	3.8	250	*	420	520	800
Muddy Brook	At George Street	2.0	160	*	270	330	510
Mulpus Brook	At county boundary	13.00	810	*	1,920	2,140	3,820
Mumford River	At confluence with Blackstone River	54.9	1,510	*	2,880	3,740	6,750
Mumford River	At confluence with Cold Spring Brook	51.0	1,430	*	2,730	3,550	6,410
Mumford River	At State Route 122	50.4	1,430	*	2,730	3,540	6,390
Mumford River	At Uxbridge/ Sutton corporate limits	33.3	1,060	*	2,030	2,640	4,750
Mumford River	At State Route 146	31.6	1,030	*	1,970	2,560	4,620
Mumford River	At Linwood Pond Dam	49.8	1,420	*	2,700	3,510	6,340
Mumford River	At head of Linwood Pond	48.5	1,390	*	2,650	3,440	6,200

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			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Mumford River	At Meadow Pond Dam	47.4	1,370	*	2,620	3,400	6,130
Mumford River	Below confluence with Whitins Pond	34.3	1,090	*	2,090	2,710	4,890
Mumford River	Above Gilboa Pond Dam	29.8	990	*	1,880	2,440	4,410
Mumford River	Above Charles Street	28.4	950	*	1,820	2,370	4,270
Mumford River	Above confluence with Centerville Brook	23.0	830	*	1,590	2,060	3,720
Mumford River	Above confluence with Caswell Brook	21.9	800	*	1,530	1,990	3,580
Mumford River	Above Douglas/ Sutton corporate limits	12.0	520	*	1,000	1,300	2,340
Nashua River	Above confluence with Catacoonamug Brook	281	6,240	8,550	10,600	12,800	19,200
Nashua River	Above confluence with tributary from Slate Rock Pond	276	6,190	8,490	10,500	12,700	19,100
Nashua River	Above confluence with unnamed tributary approximately 650 feet above Harvard Road	272	6,150	8,450	10,500	12,700	19,000
Nashua River	Above confluence with North Nashua River	131	1,430	2,350	3,360	4,740	10,200

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			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Nashua River	Above confluence with Goodridge Brook	126	1,380	2,260	3,230	4,560	9,830
Nashua River	At USGS streamgage 01095503	124	1,350	2,220	3,170	4,470	9,640
Nashua River	At outlet of Wachusett Reservoir	123	1,350	2,220	3,170	4,470	9,640
North Brook	At mouth	16.94	1,217	*	2,036	2,510	3,712
North Brook	At Whitney Street	15.29	1,192	*	1,999	2,460	3,636
North Brook	At Crosby Street	14.38	1,031	*	1,681	2,020	2,904
North Brook	At Jones Road	11.29	666	*	1,042	1,239	1,750
North Brook	10.0 feet above Linden Street	10.31	636	*	970	1,131	1,546
North Brook	At West Street	7.35	1,409	*	2,300	2,736	3,825
North Brook	200 feet below Randall Road	7.33	1,403	*	2,291	2,725	3,810
North Brook	At Randall Road	3.45	672	*	115	1,336	1,889
North Brook	At Conrail bridge	2.01	494	*	809	963	1,253
North Brook	1,000 feet below Asphalt Road	1.47	385	*	622	735	944
North Nashua River	At confluence with Nashua River	132	6,070	8,360	10,400	12,600	18,800
North Nashua River	Above confluence with Pumpkin Brook	126	6,000	8,290	10,300	12,500	18,700

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Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
North Nashua River	At USGS streamgage 01094500	109	5,810	8,070	10,000	12,200	18,300
North Nashua River	Above confluence with Fall Brook	101	5,350	7,400	9,180	11,200	16,800
North Nashua River	Above confluence with Monoosnoc Brook	87.0	4,570	6,260	7,740	9,420	14,200
North Nashua River	Above confluence with Baker Brook 2	66.6	3,450	4,640	5,700	6,910	10,600
North Nashua River	At USGS streamgage 01094400	64.4	3,330	4,470	5,480	6,650	10,200
North Nashua River	Above confluence with tributary from Nichols Pond	62.7	3,240	4,340	5,320	6,450	9,910
North Nashua River	Above confluence with Sand Brook	60.1	3,100	4,140	5,060	6,140	9,450
North Nashua River	Above confluence with Phillips Brook	42.25	3,500	*	8,000	11,000	20,000
North Nashua River	Above confluence with Whitman River (Lower Reach)	13.71	1,280	*	2,490	2,700	4,140
O'Brien Brook	At confluence with Godfrey Brook	0.38	90	*	130	160	240
Otter River	At downstream Gardner/ Templeton corporate limits	37.3	740	*	1,165	1,410	2,085

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Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Otter River	At gaging station just above Turner Street	33.7	675	*	1,055	1,270	1,865
Otter River	Below confluence with Perley Brook	27.1	580	*	920	1,110	1,655
Otter River	Above confluence with Perley Brook	20.3	480	*	750	895	1,330
Otter River	Below confluence with Pond Brook	18.8	460	*	715	855	1,265
Otter River	Above confluence with Mahoney Brook	16.3	400	*	620	735	1,090
Pearl Hill Brook	At confluence with Baker Brook 2	3.48	190	*	290	340	450
Perley Brook	At Parker Pond outlet	6.8	205	*	340	415	645
Perley Brook	Below confluence with Wilder Brook	6.1	195	*	325	395	615
Perley Brook	Above confluence with Wilder Brook	3.1	90	*	145	175	265
Perley Brook	At Perley Brook Reservoir	2.7	75	*	125	150	230
Perley Brook	At Green Street	1.6	60	*	95	115	175
Phillips Brook	At confluence with North Nashua River	14.76	1,610	*	2,680	3,220	4,550

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			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Phillips Brook	At Fitchburg/ Westminster corporate limits	10.40	1,260	*	2,100	2,520	3,550
Phillips Brook	At Ashburnham/ Westminster corporate limits	7.5	1,298	*	2,161	2,590	3,594
Phillips Brook	At Factory Village Pond Dam	5.0	1,176	*	1,956	2,339	3,210
Phillips Brook	At Winnekeag Pond outlet	2.1	1,021	*	1,692	2,017	2,712
Piccadilly Brook	At Westborough Reservoir	1.3	280	*	655	930	1,420
Piccadilly Brook	At Hopkinton Road	1.8	335	*	708	965	1,460
Pikes Pond Tributary	At Pikes Pond inlet	1.6	125	*	267	358	679
Pikes Pond Tributary	About 1,200 feet below Conrail right-of-way	1.2	98	*	210	282	535
Pond Brook	At confluence with Otter River	2.5	205	*	320	370	540
Pond Brook	At West Broadway Bridge	2.0	185	*	285	330	480
Quick Stream	In Town of Blackstone	*	130	*	360	440	630
Quinapoxet River	Approximately 1,200 feet below River Street	31.6	1,286	*	2,153	2,621	4,000

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Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Quinebaug River	At Fabyan Road	157.2	4,216	6,845	9,168	11,641	21,186
Quinebaug River	At confluence with Quinebaug River Tributary 2	130.2	3,275	5,353	7,296	9,382	17,255
Quinebaug River	At confluence with Cady Brook	115.8	2,549	4,184	5,741	7,394	13,666
Quinebaug River	Approximately 580 feet below River Street	102.3	2,108	2,585	3,493	4,488	8,414
Quinebaug River	At Westville Dam	93.4	2,109	2,109	2,569	3,325	6,359
Quinebaug River	At Stallion Hill Road	66.0	738	1,171	1,531	1,912	3,161
Quinebaug River	At East Brimfield Dam	61.8	43	44	45	46	48
Quinsigamond River	Above confluence with Blackstone River	37.3	650	*	1,105	1,330	2,290
Quinsigamond River	5,900 feet above confluence with Blackstone River	36.3	630	*	1,065	1,290	2,200
Quinsigamond River	Above Pleasant Creek	35.1	610	*	1,025	1,230	2,100
Quinsigamond River	Above Lake Ripple Dam	34.5	650	*	1,090	1,340	2,150
Quinsigamond River	Above head of Lake Ripple	31.2	570	*	950	1,160	1,840
Quinsigamond River	Above confluence with Big Bummet Brook	28.1	390	*	580	680	980

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Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Ramshorn Brook (Town of Auburn)	At Swanson Road	8.3	700	*	1,070	1,360	2,500
Ramshorn Brook (Town of Auburn)	At Pondville Pond	1.9	210	*	340	550	1,100
Ramshorn Brook (Town of Millbury)	At confluence with Pondville Pond	4.8	760	*	1,210	1,560	2,040
Ramshorn Brook (Town of Millbury)	At Carleton Street	4.2	465	*	720	950	1,280
Ramshorn Brook (Town of Millbury)	At West Main Street	3.3	230	*	325	465	675
Ramshorn Brook (Town of Millbury)	At outflow structure from Ramshorn Pond	2.5	103	*	200	261	453
Rawson Hill Brook	At Rawson Hill Dam	1.5	45	*	67	85	280
Riverdale Mills Sluice Gates and Tail Race	At Riverdale Street	*	445	*	1,075	1,284	2,060
Round Meadow Pond Brook	At Round Meadow Pond inlet	3.44	194	*	470	478	487
Rutters Brook	At confluence with Sullivan Brook	3.9	360	*	540	640	840
Rutters Brook	At confluence with Rutters Brook Tributary 1.1	1.2	160	*	240	290	380
Rutters Brook	Approximately 560 feet below East Main Street	0.7	120	*	180	210	280

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Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Rutters Brook Tributary 1	At confluence with Rutters Brook Tributary 1.1	1.3	170	*	260	300	400
Rutters Brook Tributary 1	Approximately 0.8 mile below Walkup Street	1.0	140	*	220	250	340
Rutters Brook Tributary 1	Approximately 1,150 feet below Walkup Street	0.3	60	*	100	110	150
Rutters Brook Tributary 1.1	At confluence with Rutters Brook Tributary 1	0.9	120	*	190	220	300
Sevenmile River	At Spencer/ East Brookfield corporate limits	39.1	1,550	*	2,900	3,800	6,150
Sevenmile River	Above confluence with Cranberry River	31.6	1,250	*	2,220	2,880	4,520
Sewall Brook	Below Sewall Pond	2.7	189	*	325	399	624
Sewall Brook	Above Sewall Pond	1.7	131	*	226	278	426
Sewall Brook	At dam 2,000 feet below State Route 140	1.2	101	*	175	216	340
Singletary Brook	At confluence with Blackstone River	6.3	520	*	895	1,040	1,380
Singletary Brook	At State Route 146	6.0	430	*	725	820	990

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Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Smith Brook	At confluence with Wyman Pond Brook	2.20	315	*	639	700	1,147
Smith Brook	At dam near State Route 140	2.08	384	*	775	850	1,392
Southwick Brook	Above confluence with Mumford River	0.9	80	*	140	170	280
Stall Brook	At Milford/ Medway town boundary	1.5	100	*	160	200	360
Still River	At Hudson/ Harvard town line	4.12	170	*	250	310	450
Stillwater River	At Muddy Pond Road	30.30	1,100	*	1,960	2,430	3,620
Stone Brook	At Brook Street	1.9	210	*	340	550	1,100
Stone Brook	Approximately 3,200 feet below South Street	0.6	120	*	190	310	610
Stony Brook	Approximately 1,400 feet below Stony Brook Reservoir Dam	22.5	1,240	*	1,820	2,150	2,830
Stony Brook	Approximately 1.0 mile above Stony Brook Reservoir Dam	12.7	760	*	1,130	1,340	1,770
Stony Brook	Approximately 1,900 feet below Boston Road	12.0	750	*	1,120	1,320	1,750

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Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Stony Brook	Approximately 1,600 feet above Boston Road	11.2	720	*	1,070	1,260	1,670
Stony Brook	Approximately 1,200 feet below White Bagley Road	10.5	680	*	1,020	1,210	1,600
Stony Brook	Approximately 470 feet below Cordaville Road	9.7	670	*	1,000	1,180	1,550
Stony Brook	Approximately 830 feet below Parkerville Road	7.9	580	*	870	1,030	1,360
Stony Brook Tributary 2	At Sudbury Reservoir	1.27	70	*	100	120	150
Sudbury River	Approximately 1,050 feet below Howe Street	23.8	1,290	*	1,890	2,240	2,940
Sudbury River	Approximately 190 feet below Cordaville Street	21.4	1,150	*	1,700	2,010	2,650
Sudbury River	Approximately 750 feet above Fay Court	19.7	1,130	*	1,670	1,970	2,590
Sudbury River	Approximately 140 feet above Fruit Street	18.4	1,080	*	1,590	1,880	2,470
Sudbury River Tributary 12	At confluence with Sudbury River	1.41	90	*	130	150	200

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			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Sullivan Brook	At Interstate 90/ Massachusetts Turnpike	17.6	1,090	*	1,610	1,900	2,490
Sullivan Brook	Approximately 0.8 mile above Interstate 495	9.4	680	*	1,010	1,190	1,570
Sullivan Brook	Approximately 1.2 mile above Interstate 495	7.0	530	*	800	940	1,250
Tatnuck Brook	At Coes Reservoir Outlet	10.8	1,430	*	2,330	2,770	3,450
Tatnuck Brook	At Patch Reservoir Outlet	9.0	1,670	*	2,610	3,120	4,250
Tatnuck Brook	At Cook Pond Outlet	7.0	1,100	*	1,830	2,250	3,300
Town Meadow Brook	Above Greenville Road	8.7	452	*	765	935	1,444
Town Meadow Brook	Above Pine Street	3.6	228	*	389	477	743
Town Meadow Brook	Above State Route 9 (Main Street)	2.9	184	*	314	385	600
Tributary 1	At outlet from New Pond	2.0	128	*	219	269	420
Tributary 1	At private road above New Pond	1.3	89	*	153	188	294
Tributary A to Fall Brook	At confluence with Fall Brook	1.0	64	*	95	108	145

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			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Tributary B to Fall Brook	At confluence with Fall Brook	1.84	101	*	152	174	233
Tributary C to Fall Brook	At confluence with Fall Brook	1.0	58	*	87	99	135
Tributary to Catacoonamug Brook	At confluence with Catacoonamug Brook	1.27	70	*	110	130	180
Tributary to Elizabeth Brook	At confluence with Elizabeth Brook	2.41	130	*	200	240	330
Tributary to Monoosnoc Brook	At confluence with Monoosnoc Brook	2.65	190	*	300	350	490
Tributary to Pearl Hill Brook	At confluence with Pearl Hill Brook	1.00	70	*	100	110	140
Tributary to Round Meadow Pond	At confluence with Whitman River (Lower Reach)	5.90	307	*	755	847	1,533
Tributary to Waushacum Brook	At confluence with Waushacum Brook	1.7	140	*	250	310	470
Tributary to Waushacum Brook	Above confluence with unnamed brook	0.2	17	*	30	40	60
Tributary to Wyman Pond	At confluence with Wyman Pond Brook	1.27	125	*	192	221	298
Unnamed Tributary	At Pondville Road	0.3	250	*	270	310	340
Unnamed Tributary to Mayo Pond	At confluence with Mayo Pond	0.6	310	*	540	615	745

Table 9: Summary of Discharges

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Walker Pond	Above Massachusetts Turnpike	3.8	213	*	362	443	688
Waushacum Brook	At confluence with Wachusett Reservoir	8.3	235	*	410	520	840
Waushacum Brook	Above confluence with Tributary to Waushacum Brook	6.4	95	*	170	210	340
Wekepeke Brook	At Pratt Junction Road	2.31	140	*	230	270	390
West Brook	At entrance to culvert in Shrewsbury	3.4	202	*	353	403	675
West Brook	At Main Street	2.46	174	*	298	366	574
West River	At Hartford Avenue	13.1	560	*	950	1,160	1,770
West River	At confluence with Baker Pond	7.3	370	*	630	770	1,170
West River	At Upton/ Grafton corporate limits	6.2	330	*	560	680	1,040
West River (Town of Uxbridge)	At confluence with Blackstone River	37.9	510	*	790	940	1,400
West River (Town of Uxbridge)	At confluence with Rock Meadow Brook	34.1	375	*	400	600	800
Whitman River (Lower Reach)	At confluence with North Nashua River	27.96	2,040	*	3,880	4,240	6,770

Table 9: Summary of Discharges

Flooding Source	Location	Drainage Area (Square Miles)	Peak Discharge (cfs)				
			10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Whitman River (Upper Reach)	At Ashburnham/ Westminster corporate limits	8.2	816	*	1,389	1,701	2,642
Whitman River (Upper Reach)	At Whitney Pond Dam	3.9	430	*	738	906	1,416
Wilder Brook	At confluence with Perley Brook	3.0	100	*	160	200	355
Wilder Brook	At West Street	2.8	90	*	150	185	270
Wrack Meadow Brook	At mouth	7.33	1,403	*	2,291	2,725	3,810
Wrack Meadow Brook	At West Street	3.87	740	*	1,199	1,422	1,980
Wrack Meadow Brook	At Boylston Road	3.78	723	*	1,171	1,389	1,934
Wyman Pond Brook	At Princeton Road	9.06	350	*	814	906	1,594
Wyman Pond Brook	At Fitchburg/ Westminster corporate limits	8.55	261	*	649	727	1,326
Wyman Pond Brook	Below Narrows Road	7.81	197	*	461	510	953

*Not calculated for this Flood Risk Project

Figure 7: Frequency Discharge-Drainage Area Curves

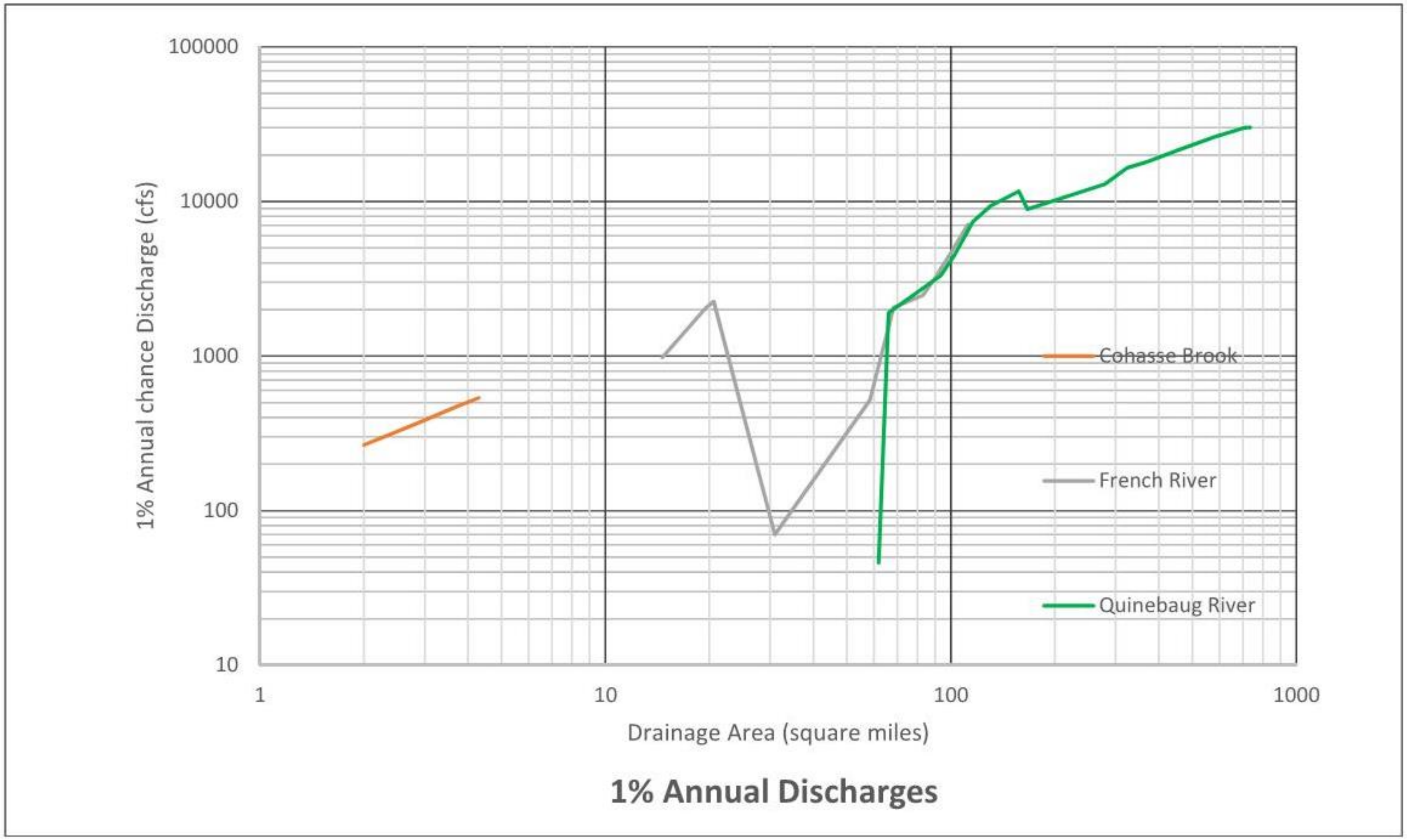

Table 10: Summary of Non-Coastal Stillwater Elevations

Flooding Source	Location	Elevations (feet NAVD88)				
		10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Auburn Pond	Auburn, Town of	498.7	*	499.7	500.5	502.1
Brierly Pond	Millbury, Town of	518.2	*	518.8	519.0	519.5
Curtis Pond	Worcester, City of	471.4	*	474.5	475.4	477.4
East Waushacum Pond	Sterling, Town of	438.8	*	439.3	439.4	440.0
Fall Brook Reservoir	Leominster, City of	653.2	653.7	654.1	654.4	655.2
Flagg Street Pond	Worcester, City of	566.0	*	566.3	566.4	566.6
Harris Pond	Blackstone, Town of (above railroad embankment)	168.9	*	172.7	175.1	180.5
Harris Pond	Blackstone, Town of (at dam)	168.9	*	169.7	170.2	171.1
Indian Lake	Worcester, City of	540.6	*	541.4	542.4	542.9
Lake Shirley	Lunenburg, Town of	299.9	*	300.7	301.4	301.8
Lake Webster/ Mill Brook	Webster, Town of	477.8	479.0	479.7	480.3	482.2
Lake Whalom	Leominster, City of; Lunenburg, Town of	512.6	*	512.8	512.9	513.1
Leesville Pond	Auburn, Town of; Worcester, City of (below Interstate 290)	485.7	*	485.8	485.9	488.5
Leesville Pond	Auburn, Town of (above Interstate 290)	486.6	*	486.8	487.1	492.7
Mill Brook Conduit	Worcester, City of	440.3	*	440.8	449.3	449.8
Paradise Pond	Princeton, Town of	807.1	*	807.5	807.9	808.0

Table 10: Summary of Non-Coastal Stillwater Elevations

Flooding Source	Location	Elevations (feet NAVD88)				
		10% Annual Chance	4% Annual Chance	2% Annual Chance	1% Annual Chance	0.2% Annual Chance
Partridge Pond	Westminster, Town of	1,105.2	*	1,105.9	1,106.3	1,106.6
Pondville Pond	Auburn, Town of; Millbury, Town of	514.4	*	515.7	516.7	525.1
Quacumquasit Pond	Sturbridge, Town of	602.5	*	604.1	605.6	608.2
Scott Reservoir	Fitchburg, City of	882.3	882.6	882.8	883.1	883.7
Singletary Pond	Millbury, Town of; Sutton, Town of	558.3	*	558.9	559.1	559.4
Stoneville Pond	Auburn, Town of	516.3	*	518.4	520.0	522.7
Tributary A Dam Pond	Leominster, City of	451.2	*	451.4	451.7	451.9
Upper Crocker Pond	Ashburnham, Town of; Westminster, Town of	825.6	*	827.5	827.8	830.3
Wachusett Reservoir	Boylston, Town of; Clinton, Town of; Sterling, Town of; West Boylston, Town of	384.9	386.1	387.2	388.5	390.5
Whitins Pond	Northbridge, Town of; Sutton, Town of	309.6	*	311.3	311.9	314.0
Winnekeag Lake	Ashburnham, Town of	1,129.6	*	1,130.7	1,131.3	1,132.2
Wyman Pond	Princeton, Town of; Westminster, Town of	891.4	*	891.9	892.2	892.5

*Not calculated for this Flood Risk Project

Table 11: Stream Gage Information used to Determine Discharges

Flooding Source	Gage Identifier	Agency that Maintains Gage	Site Name	Drainage Area (Square Miles)	Period of Record	
					From	To
Assabet River	01097000	USGS	Assabet River at Maynard, MA	116	1941	1977
Blackstone River	01110500	USGS	Blackstone River at Northbridge, MA	141	1936	2006
Blackstone River	01112500	USGS	Blackstone River at Woonsocket, RI	416	1929	2006
Charles River	01103500	USGS	Charles River at Dover, MA	183	1938	1979
East Branch Tully River	01165000	USGS	East Branch Tully River near Athol, MA	50.5	1917	1979
French River	01124350	USGS	French River below dam, at Hodges Village, MA	31.2	1962	1989
French River	01125000	USGS	French River at Webster, MA	86	1936	1981
Kettle Brook	01109500	USGS	Kettle Brook at Worcester, MA	31.6	1960	1980
Millers River	01166500	USGS	Millers River at Erving, MA	372	1915	1979
Millers River	01164000	USGS	Millers River at South Royalston, MA	189	1940	1979
Mumford River	01111000	USGS	Mumford River at East Douglas, MA	27.8	1966	1979
Nashua River	01095500	USGS	Nashua River at Clinton, MA	108	2003	2007
Nashua River	01095503	USGS	Nashua River, Water Street Bridge, at Clinton, MA	110	2012	2016
Nashua River	01095505	USGS	Nashua River, 0.4 mi upstream of Route 110, at Clinton, MA	125	2008	2011
Nashua River	01096500	USGS	Nashua River at East Pepperell, MA	435	1936	2016
North Nashua River	01094400	USGS	North Nashua River at Fitchburg, MA	64.2	1973	2016
North Nashua River	01094500	USGS	North Nashua River near Leominster, MA	110	1936	2016

Table 11: Stream Gage Information used to Determine Discharges

Flooding Source	Gage Identifier	Agency that Maintains Gage	Site Name	Drainage Area (Square Miles)	Period of Record	
					From	To
Otter River	01163200	USGS	Otter River at Otter River, MA	34.1	1964	1975
Quinebaug River	01123360	USGS	Quinebaug River below East Brimfield Dam at Fiskdale, MA	62.6	1936	2016
Quinebaug River	01123500	USGS	Quinebaug River at Westville, MA	93.6	1938	1962
Quinebaug River	01123600	USGS	Quinebaug River below Westville Dam near Southbridge, MA	94.4	1963	2016
Quinsigamond River	01110000	USGS	Quinsigamond River at North Grafton, MA	25.6	1940	1978
Ware River	01173500	USGS	Ware River at Gibbs Crossing, MA	197	1913	1978
West River	01111200	USGS	West River below West Hill Dam near Uxbridge, MA	27.9	1962	1979
Wilder Brook	01163100	USGS	Wilder Brook near Gardner, MA	2.35	1963	1973

5.2 Hydraulic Analyses

Analyses of the hydraulic characteristics of flooding from the sources studied were carried out to provide estimates of the elevations of floods of the selected recurrence intervals. Base flood elevations on the FIRM represent the elevations shown on the Flood Profiles and in the Floodway Data tables in the FIS Report. Rounded whole-foot elevations may be shown on the FIRM in coastal areas, areas of ponding, and other areas with static base flood elevations. These whole-foot elevations may not exactly reflect the elevations derived from the hydraulic analyses. Flood elevations shown on the FIRM are primarily intended for flood insurance rating purposes. For construction and/or floodplain management purposes, users are cautioned to use the flood elevation data presented in this FIS Report in conjunction with the data shown on the FIRM. The hydraulic analyses for this FIS were based on unobstructed flow. The flood elevations shown on the profiles are thus considered valid only if hydraulic structures remain unobstructed, operate properly, and do not fail.

For streams for which hydraulic analyses were based on cross sections, locations of selected cross sections are shown on the Flood Profiles (Exhibit 1). For stream segments for which a floodway was computed (Section 6.3), selected cross sections are also listed on Table 23, "Floodway Data."

A summary of the methods used in hydraulic analyses performed for this project is provided in Table 12. Roughness coefficients are provided in Table 13. Roughness coefficients are values representing the frictional resistance water experiences when passing overland or through a channel. They are used in the calculations to determine water surface elevations. Greater detail (including assumptions, analysis, and results) is available in the archived project documentation.

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
Asnebumskit Brook	Confluence with Quinapoxet River	Pine Hill Reservoir	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	Flow-change locations were selected based on 50% change in drainage area. Sub-basin delineation used hydro-conditioned lidar topography (FEMA 2011). 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. These special considerations apply to all Zone A flooding sources in this table dated 7/15/2019.
Asnebumskit Brook Tributary A	Confluence with Asnebumskit Brook	Stump Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Asnebumskit Brook Tributary B	Confluence with Asnebumskit Brook	Kendall Reservoir	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Assabet River	County boundary	Approximately 2,300 feet below Allen 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). Streamgage statistics, updated through 2010 using log-Pearson type III analysis (IACWD 1982) with 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. Average base-flood-frequency discharges increased 123%, 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 (from NRCS or this study). A HEC-HMS rainfall-runoff model (NRCS Curve Number and Unit Hydrograph methods, type III storm, 5-minute time steps), calibrated to precipitation and streamflow data from the 2007 storm event, was used to validate the results of the regression equations. 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 Concord River. These special considerations also apply to all other Zone AE flooding sources dated 10/1/2012.

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
Assabet River	Approximately 2,300 feet below Allen Street	Approximately 240 feet above Main Street	Hydrograph analysis	HEC-2 (USACE 1973a)	11/1/1977	AE w/ floodway	SCS prepared peak-flow hydrographs for the 10- and 1-percent-annual-chance events on Assabet River at Maynard and Hudson for flow conditions both with and without flood storage reservoirs. The Maynard hydrograph without reservoirs compared favorably to a log-Pearson type III statistical analysis (WRC 1976) of records at USGS streamgage 01097000 (Assabet River at Maynard). The hydrographs with reservoirs were used to develop peak discharges for the 10- and 1-percent-annual-chance floods at Maynard and Hudson. At other points, discharges were modified to reflect drainage area and total storage. A log-Pearson type III distribution was used to estimate the 2- and 0.2-percent-annual-chance discharges. Overbank portions of cross sections were from topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from USACE 1966. The hydraulic model was calibrated using high-water marks from the August 1955 flood, information from USACE 1966, and interviews with local residents.
Assabet River (Lower Reach)	Approximately 29.6 miles above confluence with Concord River	Assabet River Dam (George Nichols Dam)	Hydrograph analysis	HEC-2 (USACE 1973a)	3/1/1978	AE w/ floodway	SCS prepared peak-flow hydrographs for the 10- and 1-percent-annual-chance events on Assabet River at Maynard and Hudson for flow conditions both with and without flood storage reservoirs. The Maynard hydrograph without reservoirs compared favorably to a log-Pearson type III statistical analysis (WRC 1976) of records at USGS streamgage 01097000 (Assabet River at Maynard). The hydrographs with reservoirs were used to develop peak discharges for the 10- and 1-percent-annual-chance floods at Maynard and Hudson. At other points, discharges were modified to reflect drainage area and total storage. A log-Pearson type III distribution was used to estimate the 2- and 0.2-percent-annual-chance discharges. Overbank portions of cross sections were from topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations at Tyler Dam (USACE 1966).

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
Assabet River (Upper Reach)	Mouth at Assabet Reservoir	Approximately 1,800 feet above Nourse Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	3/1/1978	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Wandle 1977). The 0.2-percent-annual-chance discharge was extrapolated using a log-Pearson type III distribution. Overbank portions of cross sections were from topographic maps. Underwater portions and structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations at Nichols Dam (SCS 1975a). The hydraulic model was calibrated to high-water marks from the March 1968 flood obtained from newspaper articles (Worcester Telegram 1968), a USGS report (USGS 1970), and interviews with local residents.
Assabet River Branch No. 2	County boundary	Approximately 765 feet above Gates Pond Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1973a)	11/1/1977	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Johnson and Tasker 1974). The 0.2-percent-annual-chance discharge was extrapolated using a log-Pearson type III distribution. Overbank portions of cross sections were from topographic maps (Teledyne 1976). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial photography (Teledyne 1976). Starting water-surface elevations were from the slope-area method. The hydraulic model was calibrated to high-water marks from the August 1955 flood obtained from newspaper articles, USACE 1966, and interviews with local residents.
Auburn Pond	Entire shoreline	Entire shoreline	Stage-frequency relationship	Stage-frequency relationship	1/1/1977	AE w/ floodway	

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
Axtell Brook	Mouth at Lake Ripple	Massachusetts Turnpike	Regression equations (Wandle 1983)	HEC-2 (USACE 1973a)	11/1/1989	AE w/ floodway	Wandle 1983 was used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharge was based on Wandle 1977. Overbank portions of cross sections were from field surveys and topographic maps. Underwater portions were from field surveys or interpolated from topographic maps. Structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Lake Ripple.
Babcock Brook	Confluence with East Wachusett Brook	Bullard Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1979	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Babcock Brook	Bullard Road	Approximately 3,300 feet below Gregory Hill Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Babcock Brook Tributary A	Confluence with Babcock Brook	Approximately 1,300 feet above confluence	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Babcock Brook Tributary B	Confluence with Babcock Brook	Approximately 550 feet above confluence	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Baker Brook 1	Confluence with Mahoney Brook	Approximately 1,000 feet above Boston and Maine Railroad	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	5/1/1978	AE w/ floodway	Overbank portions of cross sections were from topographic 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
Baker Brook 2	Confluence with North Nashua River	Headwaters at Scott Reservoir	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	AE w/ floodway	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.
Baker Brook 2 Tributary A	Confluence with Baker Brook 2	Confluence with unnamed tributary above Ashby West Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Baldwin Hill pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (834.9 feet NAVD88).
Barrett Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (460.9 feet NAVD88).
Bartlett Pond Brook	Confluence with Steam Mill Brook	Bartlett Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Beaver Brook	Confluence with Middle River	Approximately 1,855 feet above May Street	HEC-1 (USACE 1973b)	HEC-2 (USACE 1973a)	9/6/2001	AE w/ floodway	Rainfall data were taken from the U.S. Weather Bureau's Technical Paper No. 40. The Clark method was used to estimate parameters for synthetic unit hydrographs. The SCS curve number method was selected to estimate infiltration losses. The Muskingum method was selected for flood routing. Overbank portions of cross sections were from topographic maps (Worcester 1996). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from known water-surface elevations on Middle River.
Beaver Brook (Bellingham)	County boundary	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	Flow-change locations were selected based on 50% change in drainage area. Sub-basin delineation used hydro-conditioned lidar topography (FEMA 2011). 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. These special considerations apply to all Zone A flooding sources in this table dated 4/30/2018.
Beaver Brook 4 Tributary B	County boundary	Headwaters at unnamed pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Beaver Brook 4 Tributary B pond	Entire shoreline	Entire shoreline	none	none	6/4/2019	A	Analysis of lidar DEM (FEMA 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 (381.6 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
Beaver Brook 4 Tributary C	County boundary	Approximately 5,900 feet above confluence	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Beaver Pond Brook	County boundary	Headwaters at swamp above Page Street	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Bennetts Brook	County boundary	Shaker Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1973a)	1/1/1978	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Johnson and Tasker 1974). The 0.2-percent-annual-chance discharge was extrapolated using a log-Pearson type III distribution. Overbank portions of cross sections were from topographic maps (Teledyne 1977). 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 downstream studies. Floodplain was redelineated (FEMA 2011) in 2022 Merrimack Watershed revision.
Bennetts Brook	Shaker Road	State Route 2	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Bennetts Brook Tributary H	County boundary	Approximately 3,500 feet above confluence	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Bennetts Brook Tributary I	Confluence with Bennetts Brook	Approximately 2,600 feet above Ann Lees Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Bennetts Brook Tributary J	Confluence with Bennetts Brook	Approximately 3,000 feet above confluence	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Big Bummet Brook	Confluence with Quinsigamond River	Approximately 2,140 feet above Gold Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	3/1/1978	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Wandle 1977). The 0.2-percent-annual-chance discharge was extrapolated using a log-Pearson type III distribution. Overbank portions of cross sections were from field surveys and topographic maps (Teledyne 1976). Underwater portions were from field surveys or interpolated from topographic maps. Structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Quinsigamond River.
Blackstone River	County boundary	Confluence with Middle River	Log-Pearson type III flood-frequency analysis (IACWD 1982)	HEC-RAS 3.1.3	3/30/2007	AE w/ floodway	Peak discharges were calculated using Bulletin 17B analysis (IACWD 1982) with generalized skew coefficients at USGS streamgages 01109500 (Kettle Brook at Worcester, period of record 1924 to 1959), 01110500 (Blackstone River at Northbridge, period of record 1936 to 2006), and 01112500 (Blackstone River at Woonsocket, period of record 1929 to 2006). Discharges were transferred to ungaged locations using a drainage-area ratio equation with an exponent of 0.75 (Krejmas and Wandle 1982). Cross sections were from lidar topography and field surveys. Structures were from effective studies, field surveys, lidar data, and as-built records from DOT or affected communities. Starting water-surface elevations were from known water-surface elevations in the City of Pawtucket. Saranac Canal was assumed to convey no flow. Tupperware Mill Canal was assumed to convey the same discharges computed in the 1977 Town of Blackstone study. Riverdale Street Dam in Northbridge was modeled to provide flood relief according to the Emergency Action Plan by Riverdale Mills Corporation. The tail races were modeled using a separate HEC-RAS model. The diversion model used starting water-surface elevations from normal depth and was calibrated to match water-surface elevations of the mainstem model at the upstream 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
Blood Hill ponding	Entire shoreline	Entire shoreline	none	none	6/4/2019	A	Analysis of lidar DEM (FEMA 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 (1,258.4 feet NAVD88).
Bow Brook	County boundary	Approximately 1,200 feet above Shirley Airport runway	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Bow Brook Tributary A	County boundary	Swamp approximately 2,700 feet above Lunenburg Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Bowers Brook	County boundary	Woodside Road	multiple	HEC-2 (USACE 1973a)	1/1/1978	AE w/ floodway	The upper portion of Bowers Brook (above Bare Hill Pond) was studied using regression equations (Johnson and Tasker 1974) for the 10-, 2-, and 1-percent-annual-chance events. The 0.2-percent-annual-chance discharge was extrapolated using a log-Pearson type III distribution. The lower portion of Bowers Brook (below Bare Hill Pond) was studied using rainfall-runoff routing (SCS 1972). 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. Discharge increased with decreasing drainage area due to effects of storage. Overbank portions of cross sections were from topographic maps (Teledyne 1977). 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 downstream studies. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.

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
Bowers Brook	Woodside Road	Approximately 1,500 feet above Woodside Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Brierly Pond	Entire shoreline	Entire shoreline	unknown	unknown	1/1/1978	AE	
Broad Meadow Brook	Millbury/ Worcester corporate limits	Approximately 8,630 feet above U.S. Highway 20	HEC-1 (USACE 1973b)	HEC-2 (USACE 1973a)	2/1/2000	AE w/ floodway	Rainfall data were taken from the U.S. Weather Bureau's Technical Paper No. 40. The Clark method was used to estimate parameters for synthetic unit hydrographs. The SCS curve number method was selected to estimate infiltration losses. The Muskingum method was selected for flood routing. Overbank portions of cross sections were from topographic maps (Worcester 1996). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Brook to Saima Pond	Confluence with Falulah Brook	Approximately 1,000 feet above Scripture Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1973a)	6/1/1980	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Johnson and Tasker 1974). The 0.2-percent-annual-chance discharge was extrapolated using a log-Pearson type III distribution. Overbank portions of cross sections were from topographic maps (MADPW 1968). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and photography. Starting water-surface elevations were from normal depth. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Brown Brook	Confluence with Phillips Brook	Approximately 2,200 feet above Russell Hill Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Brown Brook Tributary A	Confluence with Brown Brook	Approximately 1,100 feet above Crosby Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Bryant Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (746.1 feet NAVD88).
Cady Brook	Charlton/ Southbridge corporate limits	Dam approximately 1,200 feet above U.S. Route 20	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Cady Brook	Confluence with Quinebaug River	Charlton/ Southbridge corporate limits	Effective study	HEC-2 (USACE 1973a)	8/1/1980	AE w/ floodway	Discharges for the 2- and 1-percent-annual-chance floods were taken from the Southbridge Flood Plain Information report (USACE 1972). The 10- and 0.2-percent-annual-chance discharges were computed from the others using exponential extrapolation. Cross sections and structures were from the Southbridge Flood Plain Information report (USACE 1972). Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Calamint Hill swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (998.6 feet NAVD88).
Canada Mills pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (574.4 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
Canesto Brook	Barre/ Hubbardston corporate limits	Approximately 2,000 feet above Williamsville Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	6/1/1980	AE w/ floodway	Drainage area and storage percent were measured from topographic maps (USGS various), and precipitation was interpolated from an isohyetal map at the basin centroid (USGS 1955). Overbank portions of cross sections were from aerial photography (Moore 1980). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and reference texts (Barnes 1967). Starting water-surface elevations were from normal depth.
Catacoonamug Brook	County boundary	Confluence with Tributary to Catacoonamug Brook	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	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 structures were from field surveys. Roughness factors were from field inspection and photography. Starting water-surface elevations were from known water-surface elevations on downstream studies. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Catacoonamug Brook	Confluence with Tributary to Catacoonamug Brook	Swamp above West Street	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Catacoonamug Brook Tributary A	Confluence with Catacoonamug Brook	Outlet of Lake Whalom	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Cedar Meadow Brook	Confluence with Quinebaug River	Cooper Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	11/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. 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
Cedar Pond	Cedar Pond Dam	Approximately 1.6 miles above Cedar Pond Dam	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	11/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from elevation-discharge curves.
Center Brook	Station Street	Pratt Pond Dam	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	8/1/1980	AE w/ floodway	Overbank portions of cross sections were from topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were estimated. Starting water-surface elevations were from the slope-area method.
Chace Hill Road swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (376.8 feet NAVD88).
Chaffin Pond Tributary A	Mouth at Chaffin Pond	Approximately 500 feet above Bailey Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Chaffin Pond Tributary B	Mouth at Chaffin Pond	Approximately 300 feet below Salisbury Street	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Charles River	County boundary	Milford Pond	Drainage-area ratio equation	HEC-2 (USACE 1973a)	7/1/1980	AE	USACE performed a log-Pearson type III analysis of the USGS Charles River Village streamgage data using 41 years of record (USACE 1976c). Discharges were transposed to ungaged locations using a drainage-area ratio equation with an exponent of 0.7. Overbank portions of cross sections were from topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on downstream studies. Floodplain was redelineated (FEMA 2011) in 2022 Charles Watershed revision.
Charles River	Milford Pond	County boundary	Drainage-area ratio equation	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	USACE performed a log-Pearson type III analysis of the USGS Charles River Village streamgage data using 41 years of record (USACE 1976c). Discharges were transposed to ungaged locations using a drainage-area ratio equation with an exponent of 0.7. Overbank portions of cross sections were from topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on downstream studies. Floodplain was redelineated (FEMA 2011) in 2022 Charles Watershed revision.
Cobb Brook	Confluence with South Wachusett Brook	Approximately 1,800 feet above Brooks Station Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Cohasse Brook	Confluence with Quinebaug River	Approximately 500 feet below Cohasse Brook Reservoir	HEC-HMS 3.0 and up (Dec 2005)	HEC-RAS 5.0 and up	10/10/2019	AE w/ floodway	

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
Cold Brook	Confluence with Governor Brook	Approximately 5,700 feet above Cournoyer Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Cold Harbor Brook (Lower Reach)	Confluence with Assabet River	Hudson Street	TR-20 (SCS 1965)	WSP-2 (SCS 1976)	11/1/1977	AE w/ floodway	Hydrologic computations and hydraulic profiles were taken from an SCS study of the upper Assabet River and its tributaries (SCS 1975a). At the confluence with Assabet River, the profiles were modified to incorporate backwater.
Cold Harbor Brook (Lower Reach)	Hudson Street	Approximately 110 feet below Church Street	TR-20 (SCS 1965)	HEC-RAS	10/4/2013	AE w/ floodway	Updated by LOMR 13-01-0608P. Hydrology was unchanged from the original study. Hydraulics was updated to reflect structure and topography changes due to Laurence Falls Condominiums.
Cold Harbor Brook (Lower Reach)	Approximately 110 feet below Church Street	Approximately 1,800 feet above Lincoln Street	TR-20 (SCS 1965)	WSP-2 (SCS 1976)	11/1/1977	AE w/ floodway	Hydrologic computations and hydraulic profiles were taken from an SCS study of the upper Assabet River and its tributaries (SCS 1975a).
Cold Harbor Brook (Town of Boylston)	Boylston/ Northborough corporate limits	Approximately 1,700 feet above Reservoir Road	TR-20 (SCS 1965)	WSP-2 (SCS 1976)	6/1/1979	AE w/ floodway	Hydrologic computations and hydraulic profiles were taken from an SCS study of the upper Assabet River and its tributaries (SCS 1975a).
Cold Harbor Brook (Upper Reach)	Cherry Street	Approximately 700 feet above Fisher Street	TR-20 (SCS 1965)	WSP-2 (SCS 1976)	11/1/1977	AE w/ floodway	Hydrologic computations and hydraulic profiles were taken from an SCS study of the upper Assabet River and its tributaries (SCS 1975a).
Cold Spring Brook (Town of Harvard)	Confluence with Bowers Brook	Approximately 1,900 feet above Boston and Maine Railroad	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	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 structures were from field surveys. Roughness factors were from field inspection and aerial photography. Starting water-surface elevations were from normal depth. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.

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
Cold Spring Brook (Town of Sutton)	Confluence with Blackstone River	Approximately 5,400 feet above confluence with Blackstone River	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	11/1/1980	AE w/ floodway	Overbank portions of cross sections were from topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from the slope-area method.
Connelly Brook	Mouth at West Waushacum Pond	Approximately 280 feet below State Route 12	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Connelly Brook	Approximately 280 feet below State Route 12	Approximately 80 feet above State Route 62	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1973a)	1/1/1978	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Johnson and Tasker 1974). The 0.2-percent-annual-chance discharge was computed from the others using a log-Pearson type III distribution. Overbank portions of cross sections were from topographic maps (MADPW 1968). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and photography. Starting water-surface elevations were from normal depth. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Connelly Brook	Approximately 80 feet above State Route 62	Approximately 900 feet above Interstate 190	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Counterpane Brook	Confluence with Nashua River	Coachlace Pond	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	5/1/1979	AE w/ floodway	Results of regression equations were compared to a state report (MADPW 1972) and found to be in general agreement. Cross sections and structures were from field surveys. Roughness factors were from engineering judgment. Starting water-surface elevations were from the slope-area method. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.

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
Counterpane Brook	Coachlace Pond	Fitch Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Cowees Hill swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (925.3 feet NAVD88).
Cranberry Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (240.5 feet NAVD88).
Cronin Brook	Confluence with Blackstone River	Millbury Street	Regression equations (Wandle 1983)	HEC-2 (USACE 1973a)	11/1/1989	AE w/ floodway	Wandle 1983 was used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharge was based on Wandle 1977. Overbank portions of cross sections were from field surveys and topographic maps. Underwater portions were from field surveys or interpolated from topographic maps. Structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from the slope-area method.
Curtis Pond	Entire shoreline	Entire shoreline	Rainfall-runoff routing (SCS 1972)	unknown	1/1/1978	AE	Runoff curve numbers used for the hydrologic analysis were from soils mapping (SCS 1971).
Dark Brook	Confluence with Mumford River	Approximately 1,950 feet above Tucker's Pond Dam	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	11/1/1980	AE w/ floodway	Overbank portions of cross sections were from topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Mumford 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
Dark Brook No. 1	Mouth at Auburn Pond	Central Street	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	6/1/1989	AE w/ floodway	The SCS runoff curve number method (SCS 1972) was used to compute inflows to Eddy Pond, route the flows through the pond to account for attenuation, and increase discharges downstream due to tributaries. Development did not have significant effect on discharges. Overbank portions of cross sections were from topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and reference texts (Barnes 1967). Starting water-surface elevations were from known water-surface elevations on Auburn Pond.
Dark Brook No. 2	Mouth at Stoneville Pond	Leicester Street	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	1/1/1977	AE	Overbank portions of cross sections were from topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and reference texts (Barnes 1967). Starting water-surface elevations were from known water-surface elevations on Stoneville Pond.
Deans Brook	Charlton/ Southbridge corporate limits	Approximately 0.35 mile above McIntyre Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	Discharge was found to increase with decreasing drainage area due to the effects of storage. Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Deer Brook	County boundary	County boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Beaver Brook (Bellingham).
Denny Brook	Confluence with Jackstraw Brook	South Street	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ floodway	See special considerations for Assabet River dated 10/1/2012.

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
Denny Brook	South Street	Approximately 700 feet above High School Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	3/1/1978	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Wandle 1977). The 0.2-percent-annual-chance discharge was extrapolated using a log-Pearson type III distribution. Overbank portions of cross sections were from topographic 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. The hydraulic model was calibrated to high-water marks from the March 1968 flood obtained from newspaper articles (Worcester Telegram 1968), a USGS report (USGS 1970), and interviews with local residents.
Denny Brook	Approximately 700 feet above High School Road	Approximately 2,050 feet above Harvey Lane	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ floodway	See special considerations for Assabet River dated 10/1/2012.
Denny Brook Tributary 1	Confluence with Denny Brook	Approximately 550 feet above Chestnut Street	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ floodway	See special considerations for Assabet River dated 10/1/2012.
Dorothy Brook	Confluence with Blackstone River	Approximately 1,030 feet above confluence with Blackstone River	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	1/1/1978	AE w/ floodway	Runoff curve numbers used for the hydrologic analysis were from soils mapping (SCS 1971). Overbank portions of cross sections were from field surveys and topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from normal depth.
Dorothy Pond	Riverlin Street	Approximately 700 feet above Wheelock Avenue	HEC-1 (USACE 1973b)	HEC-2 (USACE 1973a)	6/1/1996	AE	The HEC-1 hydrologic simulation used the NRCS runoff curve number method. Cross sections 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
Dunns Brook	Confluence with Kettle Brook	Auburn Pond	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	1/1/1977	AE w/ floodway	Overbank portions of cross sections were from topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and reference texts (Barnes 1967). Starting water-surface elevations were from known water-surface elevations on Kettle Brook. The upstream end of the model determined water-surface elevations for Auburn Pond.
East Branch Ware River (Hubbardston)	Hubbardston/ Princeton corporate limits	Bickford Pond	Reservoir routing	HEC-2 (USACE 1973a)	6/1/1980	AE w/ floodway	Flow on East Branch Ware River is controlled by outflows from Mare Meadow Reservoir and Bickford Pond Reservoir. Inflow hydrographs were constructed assuming trapezoidal shape (linear rising and receding limbs). Inflows were computed using regression equations (Wandle 1977) excluding the storage for the reservoir in question. Storm duration was from statistical analysis of hourly rainfall records at a NOAA station at Worcester Airport. Outflow hydrographs were constructed using storage-elevation relations of the impoundments. The differential equation resulting from the two relations being related by conservation of mass was solved using numerical techniques. Peak discharges were obtained from the intersection of the inflow and outflow hydrographs. Overbank portions of cross sections were from aerial photography (Moore 1980). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and reference texts (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
East Branch Ware River (Rutland)	Barre/ Rutland corporate limits	Princeton/ Rutland corporate limits	Reservoir routing	HEC-2 (USACE 1973a)	6/1/1980	AE w/ floodway	Flow on East Branch Ware River is controlled by outflows from Mare Meadow Reservoir and Bickford Pond Reservoir. Inflow hydrographs were constructed assuming trapezoidal shape (linear rising and receding limbs). Inflows were computed using regression equations (Wandle 1977) excluding the storage for the reservoir in question. Storm duration was from statistical analysis of hourly rainfall records at a NOAA station at Worcester Airport. Outflow hydrographs were constructed using storage-elevation relations of the impoundments. The differential equation resulting from the two relations being related by conservation of mass was solved using numerical techniques. Peak discharges were obtained from the intersection of the inflow and outflow hydrographs. Discharges were then transferred to Rutland using regional equations (Wandle 1977). Overbank portions of cross sections were from topographic maps (Moore 1980). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and reference texts (Barnes 1967). Starting water-surface elevations were from known water-surface elevations on downstream studies of Barre Falls Dam (USACE 1964).
East Wachusett Brook	Town Farm Road	Approximately 900 feet above Bullard Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1979	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
East Wachusett Brook	Approximately 900 feet above Bullard Road	Confluence with unnamed tributary above Mirick Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
East Wachusett Brook Tributary A	Confluence with East Wachusett Brook	Approximately 4,500 feet above confluence	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
East Waushacum Pond	Entire shoreline	Entire shoreline	Rainfall-runoff routing (SCS 1972)	unknown	1/1/1978	AE	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. Because of the large storage area relative to the drainage area, essentially all storm runoff is stored in the pond, and peak discharges do not exceed 5 cfs. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Easter Brook	Mouth at Lake Shirley	Confluence with unnamed tributary above Lancaster Avenue	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Easter Brook Tributary A	Confluence with Easter Brook	Swamp above last railroad crossing	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Echo Lake	Entire shoreline	Entire shoreline	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	4/30/2018	A	See special considerations for Beaver Brook (Bellingham).
Elizabeth Brook	Approximately 4,500 feet above Harvard Road	Approximately 1,000 feet above Sherry Road	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ floodway	See special considerations for Assabet River dated 10/1/2012.
Fairbanks Street swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (437.7 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
Fall Brook	Confluence with North Nashua River	Headwaters at Fall Brook Reservoir	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	AE w/ floodway	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.
Fall Brook Reservoir	Entire shoreline	Entire shoreline	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	AE	Stillwater water-surface elevations were taken from the upper limit of study of Fall Brook.
Fitch Basin	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (633.8 feet NAVD88).
Flagg Brook	Confluence with Whitman River (Lower Reach)	Confluence with Wyman Pond Brook	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	6/1/1980	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 (MADPW 1968). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and photography. Starting water-surface elevations were from normal depth. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Flagg Brook	Confluence with Wyman Pond Brook	Crows Hill Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Flagg Brook Tributary A	Confluence with Flagg Brook	Notown Reservoir	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Flagg Street Pond	Entire shoreline	Entire shoreline	Rainfall-runoff routing (SCS 1972)	unknown	1/1/1978	AE	Runoff curve numbers used for the hydrologic analysis were from soils mapping (SCS 1971).
Foster Brook	Confluence with Mahoney Brook	Dunn Pond	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	5/1/1978	AE w/ floodway	Overbank portions of cross sections were from topographic 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.
French Brook	Mouth at Wachusett Reservoir	Approximately 2,300 feet above Cross Street	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
French Brook pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (494.2 feet NAVD88).
French River	Confluence with Quinebaug River	Approximately 400 feet above Augetteback Pond	HEC-HMS 3.0 and up (Dec 2005)	HEC-RAS 5.0 and up	10/10/2019	AE w/ floodway	
French River	Approximately 2,550 feet below Clara Barton Road	Approximately 500 feet above Pleasant Street	HEC-HMS 3.0 and up (Dec 2005)	HEC-RAS 5.0 and up	10/10/2019	AE w/ floodway	
Gates Brook	Mouth at Wachusett Reservoir	Approximately 120 feet below Boston and Maine Railroad	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Gates Brook	Approximately 120 feet below Boston and Maine Railroad	Approximately 80 feet above Pierce Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	5/1/1988	AE w/ floodway	Results of regression equations (Wandle 1977) were modified to account for the effects of urbanization (Sauer 1974). Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Gates Brook	Approximately 80 feet above Pierce Street	Lombard Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Godfrey Brook	Confluence with Charles River	Headwaters at pond at Milford High School	2017 state regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	6/1/2017	AE w/ floodway	For locations within the bounds of applicable basin characteristics for the regression equations, dicharges were computed using regression equations (Zarriello 2017) adjusted for urbanization (Sauer et al. 1983). For other locations, discharges were transferred using a drainage-area ratio equation with exponent ranging from 0.76 to 0.78. Discharges between Water Street and Vine Street were decreased to account for a flood mitigation project that diverts water through an underground storm drain system to a point below Vine Street. The diversion system was modeled in HEC-RAS for hydrologic purposes only and was not mapped. 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.

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
Goodridge Brook	Just below Clinton/ Lancaster corporate limits	Approximately 1,000 feet above Parker Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1973a)	1/1/1978	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Johnson and Tasker 1974). The 0.2-percent-annual-chance discharge was computed from the others using a log-Pearson type III distribution. Overbank portions of cross sections were from photogrammetric maps (Teledyne 1977). 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 Nashua River. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Goodridge Brook	Approximately 1,000 feet above Parker Road	Approximately 1,400 feet above State Route 62	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Governor Brook	Confluence with Cold Brook	Holden/ Princeton corporate limits	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Governor Brook	Holden/ Princeton corporate limits	Approximately 2,100 feet below Coal Kiln Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1979	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Governor Brook Tributary A	Confluence with Governor Brook	Approximately 2,600 feet above corporate limits	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Great Brook	County boundary	Nourse Road	multiple	HEC-2 (USACE 1973a)	4/1/1978	AE w/ floodway	Below East Bolton Dam, discharges were taken from the effective study on Elizabeth Brook in the Town of Stow. Those discharges were developed using TR-20 and account for the attenuating effect of East Bolton Dam and for the diversion channel to the Delaney Dam complex. Above East Bolton Dam, discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Wandle 1977). The 0.2-percent-annual-chance discharge was computed from the others using a log-Pearson type III distribution. Overbank portions of cross sections were from topographic maps (Teledyne 1976). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and aerial photography (Teledyne 1976). Starting water-surface elevations for the reach below East Bolton Dam were from known water-surface elevations on downstream studies. Starting water-surface elevations for the reach above East Bolton Dam were from computations of flood storage elevations at the dam.
Greenwood Brook	Confluence with Mahoney Brook	East Broadway	Modified Puls method (Viessman et al. 1972)	HEC-2 (USACE 1973a)	5/1/1978	AE w/ floodway	Discharges were calculated by routing flows through Wrights Reservoir using the modified Puls method (Viessman et al. 1972). The inflow to Wrights Reservoir was calculated using the unit hydrograph method (SCS 1972). Overbank portions of cross sections were from topographic maps. Underwater portions and structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Mahoney Brook.
Hamant Brook	Access Road	Approximately 0.6 mile above Interstate 84	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	11/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. 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
Haynes Reservoir	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (840.0 feet NAVD88).
Hop Brook	Confluence with Hop Brook Tributary 4	Approximately 650 feet above Spring Street	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ floodway	See special considerations for Assabet River dated 10/1/2012.
Hop Brook Tributary 4	Confluence with Hop Brook	Approximately 1,000 feet above Flanagan Drive	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ floodway	See special considerations for Assabet River dated 10/1/2012.
Hop Brook Tributary 4.1	Confluence with Hop Brook Tributary 4	Approximately 200 feet below Main Street	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ floodway	See special considerations for Assabet River dated 10/1/2012.
Howard Brook	Whitney Street	Brewer Street	TR-20 (SCS 1965)	WSP-2 (SCS 1976)	11/1/1977	AE w/ floodway	Hydrologic computations and hydraulic profiles were taken from an SCS study of the upper Assabet River and its tributaries (SCS 1975a). At the confluence with Cold Harbor Brook, the profiles were modified to incorporate backwater.
Howard Brook	Confluence with Cold Harbor Brook (Lower Reach)	Whitney Street	TR-20 (SCS 1965)	HEC-RAS	10/4/2013	AE w/ floodway	Updated by LOMR 13-01-0608P. Hydrology was unchanged from the original study. Hydraulics was updated to reflect structure and topography changes due to Laurence Falls Condominiums.
Howard Brook Split Flow	Confluence with Cold Harbor Brook (Lower Reach)	Diversion from Howard Brook	TR-20 (SCS 1965)	HEC-RAS	10/4/2013	AE w/ floodway	Updated by LOMR 13-01-0608P. Hydrology was unchanged from the original study. Hydraulics was updated to reflect structure and topography changes due to Laurence Falls Condominiums.

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
Huckleberry Brook	Mouth at Cedar Swamp Pond	Erin Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	Overbank portions of cross sections were from topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Charles River. Floodplain was redelineated (FEMA 2011) in 2022 Charles Watershed revision.
Indian Lake	Entire shoreline	Entire shoreline	Rainfall-runoff routing (SCS 1972)	unknown	1/1/1978	AE	Runoff curve numbers used for the hydrologic analysis were from soils mapping (SCS 1971).
Ivy Brook	Confluence with Huckleberry Brook	Approximately 1,300 feet above Silver Hill Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	Overbank portions of cross sections were from topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Huckleberry Brook. Floodplain was redelineated (FEMA 2011) in 2022 Charles Watershed revision.
Jackstraw Brook	Confluence with Sullivan Brook	Approximately 650 feet above Garfield Drive	Regression equations	HEC-RAS 4.1	10/1/2012	AE w/ floodway	See special considerations for Assabet River dated 10/1/2012.
Justice Brook	Confluence with Keyes Brook	Headwaters at confluence of Bartlett Pond Brook and Steam Mill Brook	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Kettle Brook (East)	Mouth at Curtis Pond	Leesville Pond	Log-Pearson type III flood-frequency analysis (WRC 1976)	HEC-2 (USACE 1973a)	1/1/1978	AE	A log-Pearson type III analysis (WRC 1976) was performed on data from USGS streamgage 01109500 using discharge records since 1960 (when flows began to be diverted at the Leesville Diversion weir). Overbank portions of cross sections were from topographic maps (Moore 1975). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from known water-surface elevations on Curtis Pond. The upstream end of the model determined water-surface elevations for Leesville Pond.
Kettle Brook (Town of Auburn)	Mouth at Leesville Pond	Stoneville Pond	Log-Pearson type III flood-frequency analysis (WRC 1976)	HEC-2 (USACE 1973a)	1/1/1977	AE	A log-Pearson type III analysis (WRC 1976) was performed on data from USGS streamgage 01109500 using discharge records from before 1960 (when flows began to be diverted at the Leesville Diversion weir). Resulting discharges were transposed to ungaged locations using drainage-area ratio equation (SCS 1972). Overbank portions of cross sections were from topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and reference texts (Barnes 1967). Starting water-surface elevations were from known water-surface elevations on Leesville Pond. The upstream end of the model determined water-surface elevations for Stoneville Pond.
Kettle Brook (West)	Auburn/ Worcester corporate limits	Leicester/ Worcester corporate limits	Log-Pearson type III flood-frequency analysis (WRC 1976)	HEC-2 (USACE 1973a)	1/1/1978	AE w/ floodway	A log-Pearson type III analysis (WRC 1976) was performed on data from USGS streamgage 01109500 using discharge records from before 1960 (when flows began to be diverted at the Leesville Diversion weir). Resulting discharges were transposed to ungaged locations using drainage-area ratio equation (SCS 1972). Overbank portions of cross sections were from topographic maps (Moore 1975). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from known water-surface elevations on Stoneville Pond.

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
Keyes Brook	Confluence with Justice Brook	Approximately 2,200 feet below Hobbs Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Keyes Brook	Approximately 2,200 feet below Hobbs Road	Paradise Pond	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1979	AE w/ floodway	Overbank portions of cross sections were from aerials photography (Chicago Aerial 1967, Sewall 1969). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Keyes Brook	Paradise Road	Headwaters swamp above corporate limits	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Keyes Brook Tributary A	Confluence with Keyes Brook	Swamp above Wolf Rock Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Keyes Brook Tributary C	Confluence with Keyes Brook	Approximately 1,600 feet above confluence	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Kilburn Street swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (362.5 feet NAVD88).
Lake Shirley	Entire shoreline	Entire shoreline	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	1/1/1978	AE	Stillwater water-surface elevations were derived from model developed for Catacoonamug 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
Lake Webster	Entire shoreline	Entire shoreline	HEC-HMS 3.0 and up (Dec 2005)	HEC-RAS 5.0 and up	10/10/2019	AE	
Lake Whalom	Entire shoreline	Entire shoreline	unknown	unknown	1/1/1978	AE	Elevation-frequency data were computed using computer modeling techniques (SCS 1972). Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Lane Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (373.1 feet NAVD88).
Leadmine Brook	County boundary	Approximately 3,000 feet above Leadmine Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	11/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Lebanon Brook	Confluence with Quinebaug River	Approximately 6,500 feet above State Route 169 (North Woodstock Road)	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	8/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Leesville Pond	Entire shoreline	Entire shoreline	Log-Pearson type III flood-frequency analysis (WRC 1976)	HEC-2 (USACE 1973a)	1/1/1977	AE	Stillwater water-surface elevations were derived from model developed for Kettle 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
Lily Ponds	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (393.6, 394.3, and 394.5 feet NAVD88).
Linden Street swamp 1	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (519.4 feet NAVD88).
Linden Street swamp 2	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (514.9 feet NAVD88).
Little Mirror Lake	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 2011, USGS 2014b), guided by shape of existing waterbody feature (e.g., effective FIRM, National Wetland Inventory, or National Hydrography Dataset), if extant, was used to determine a stillwater elevation corresponding to the expected 1-percent-annual-chance floodplain (243.9 feet NAVD88).
Little Nugget Brook	Pikes Pond	Little Nugget Lake	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Little River	Just below Turner Road	Approximately 0.5 mile above Oxbow Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. 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
Lowes Brook	Confluence with French River	Approximately 0.3 mile above Sutton Avenue	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	3/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Lynde Brook	State Route 9 (Main Street)	Lynde Brook Reservoir	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	3/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Mahoney Brook	Confluence with Otter River	Just above Partridge Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	5/1/1978	AE w/ floodway	Overbank portions of cross sections were from topographic 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.
Malagasco Brook	Mouth at Wachusett Reservoir	Swamp above School Street	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Malden Brook	Mouth at Wachusett Reservoir	Swamp above Lee Street	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
McGovern Brook	Confluence with North Nashua River	Headwaters at swamp above trail from State Route 70	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
McKinstry Brook	Confluence with Quinebaug River	Approximately 0.7 mile above confluence with Quinebaug River	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	8/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Meadow Brook	Approximately 120 feet below Oak Street	Approximately 4,480 feet above Oak Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	3/1/1978	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Wandle 1977). The 0.2-percent-annual-chance discharge was computed from the others using a log-Pearson type III distribution. Overbank portions of cross sections were from topographic maps (Teledyne 1976). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and aerial photography (Teledyne 1976). Starting water-surface elevations were from the slope-area method.
Middle River	Confluence with Blackstone River	Curtis Pond	Discharge summation	HEC-2 (USACE 1973a)	1/1/1978	AE w/ floodway	Discharges for Middle River were computed as the sums of discharges from Kettle Brook (East), Tatnuck Brook, and Beaver Brook. Discharges were verified against a log-Pearson type III analysis (WRC 1976) of records from USGS streamgage 01110500 (Blackstone River at Northbridge), modified by subtracting the runoff for all drainage area between Worcester and Northbridge. Overbank portions of cross sections were from topographic maps (Moore 1975). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from known water-surface elevations on downstream studies. The upstream end of the model determined water-surface elevations for Curtis Pond.

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 (Town of Bolton)	County boundary	Spectacle Hill Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	4/1/1978	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Wandle 1977). The 0.2-percent-annual-chance discharge was computed from the others using a log-Pearson type III distribution. Discharges were coordinated with those downstream on Danforth Brook in the Town of Hudson. Overbank portions of cross sections were from topographic maps (Teledyne 1976). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and aerial photography (Teledyne 1976). Starting water-surface elevations were from known water-surface elevations on downstream studies.
Mill Brook (Town of Webster)	Confluence with French River	Approximately 0.185 mile above Arkwright Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	2/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from known water-surface elevations on French River.
Mill Brook Conduit	Salisbury Pond	Indian Lake	HEC-1 (USACE 1973b)	HEC-2 (USACE 1973a)	2/1/2000	AE	Rainfall data were taken from the U.S. Weather Bureau's Technical Paper No. 40. The Clark method was used to estimate parameters for synthetic unit hydrographs. The SCS curve number method was selected to estimate infiltration losses. The Muskingum method was selected for flood routing. Overbank portions of cross sections were from topographic maps (Worcester 1996). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the crest elevation of the embankment of Salisbury Pond.

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	Harris Pond	County boundary	HEC-1 (USACE 1973b)	HEC-2 (USACE 1973a)	7/1/1976	AE w/ floodway	A HEC-1 model (USACE 1973b) was developed for the entire Mill River watershed. Discharges between computation points were calculated using a drainage-area ratio equation with an exponent of 0.7. Discharges for the portion of the reach above Hop Brook were adjusted by subtracting the Hop Brook runoff from the downstream discharges. Discharges from the Mill River model were used along with survey data surrounding Harris Pond to develop a stage-discharge-frequency relationship for Harris Pond. Cross sections were from field surveys or topographic maps (Quinn 1979). Roughness factors were from field inspection, references (Barnes 1967), and calibration using high-water marks. Starting water-surface elevations were from known water-surface elevations on Harris Pond.
Mirror Lake	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (640.3 feet NAVD88).
Mirror Lake 2	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (316.9 feet NAVD88).
Miscoe Brook	Silver Lake	Adams Road	Regression equations (Wandle 1983)	HEC-2 (USACE 1973a)	11/1/1989	AE w/ floodway	Wandle 1983 was used to compute 10-, 2-, and 1-percent-annual-chance discharges. The 0.2-percent-annual-chance discharge was based on Wandle 1977. Overbank portions of cross sections were from field surveys and topographic maps. Underwater portions were from field surveys or interpolated from topographic maps. Structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on downstream studies.

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
Monoosnoc Brook	Confluence with North Nashua River	Downstream crossing of State Route 2	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	AE w/ floodway	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.
Monoosnoc Brook	Downstream crossing of State Route 2	Notown Reservoir	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Morse Reservoir	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (674.2 feet NAVD88).
Moulton Pond Brook	Thayer Pond	Moulton Pond	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	6/1/1980	AE w/ floodway	For the regression equations, drainage area and storage percent were measured from topographic maps (USGS various), and precipitation was interpolated from an isohyetal map at the centroid of the basin (USGS 1955). Overbank portions of cross sections were from topographic maps (Moore 1980). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and reference texts (Barnes 1967). Starting water-surface elevations were from the slope-area method.
Mountain Laurel Lane swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (345.8 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
Muddy Brook	Confluence with Mill River	Approximately 1,000 feet above Milford Street	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	Overbank portions of cross sections were from topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from the slope-area method.
Muddy Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (1,046.4 feet NAVD88).
Muddy Pond Road swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (415.4 feet NAVD88).
Mulpus Brook	County boundary	Approximately 450 feet above West Groton Road	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	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 structures were from field surveys. Roughness factors were from field inspection and photography. Starting water-surface elevations were from known water-surface elevations on downstream studies. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Mulpus Brook	Approximately 450 feet above West Groton Road	Approximately 5,000 feet above Howard Street	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Mulpus Brook Tributary A	Confluence with Mulpus Brook	Confluence with unnamed tributary below Northfield Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Mulpus Brook Tributary B	Confluence with Mulpus Brook	Approximately 250 feet below State Route 13	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Mumford River	Confluence with Blackstone River	Manchaug Road	Log-Pearson type III flood-frequency analysis (IACWD 1982)	HEC-2 (USACE 1973a)	11/1/1980	AE w/ floodway	Peak discharges were calculated in USGS 1979 using Bulletin 17B log-Pearson type III flood-frequency analysis (IACWD 1982) on USGS streamgage 01111000 (Mumford River at East Douglas, period of record 13 years). Discharges were transferred downstream to ungaged locations using a drainage-area ratio equation with an exponent of 0.7 (Chow 1967). Since the gage record was short, discharges at the mouth were computed as the average of these discharges and the discharges computed in the hydrologic analysis for Blackstone River. These averaged discharges were then transferred back upstream using the drainage-area ratio equation. Discharges were verified against discharges computed using regional equations. Overbank portions of cross sections were from field surveys or topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from the slope-area method.
Muschopauge Brook	Mouth at Quinapoxet Reservoir	Approximately 4,800 feet above Glenwood Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Nashua River	County boundary	Headwaters at Wachusett Reservoir	Log-Pearson type III flood-frequency analysis (IACWD 2018)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	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 G	Confluence with Nashua River	Headwaters at unnamed ponds	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Nashua River Tributary I	Confluence with Nashua River	Approximately 1,200 feet above Old Shirley Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Newell Road pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (735.4 feet NAVD88).
North Brook	Confluence with Assabet River	Berlin/ Bolton corporate limits	TR-20 (SCS 1965)	HEC-2 (USACE 1973a)	11/1/1977	AE w/ floodway	Hydrologic computations were taken from an SCS study of the upper Assabet River and its tributaries (SCS 1975a). The study accounted for decreasing discharge caused by the storage behind Ross Dam. Hydraulic profiles for the lower portion of the reach were from the SCS study. At the confluence with Assabet River, the profiles were modified to incorporate backwater. For the upper portion of the reach, a HEC-2 model was developed. Overbank portions of cross sections were from topographic maps (Teledyne 1976). Underwater portions and structures were from field surveys. Roughness factors were from field inspections and aerial photography (Teledyne 1976). Starting water-surface elevations were from known water-surface elevations on the lower portion of the reach. The hydraulic model was calibrated to high-water marks from the August 1955 flood obtained from newspaper articles, USACE 1966, and interviews with local residents.
North Lancaster pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (357.1 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
North Nashua River	Confluence with Nashua River	Confluence with Phillips Brook	Log-Pearson type III flood-frequency analysis (IACWD 2018)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	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), and 01096500 (1936 to 2016). 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. 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). 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 the known water surface at New Charles River Dam.
North Nashua River	Confluence with Phillips Brook	Headwaters at confluence of Whitman River (Lower Reach) and Flagg Brook	Log-Pearson type III flood-frequency analysis (WRC 1976)	HEC-2 (USACE 1973a)	6/1/1980	AE w/ floodway	A log-Pearson type III statistical analysis (WRC 1976) of records at the USGS streamgage in Leominster was developed using data since 1936. Discharges for gaged and ungaged locations were taken from a USACE analysis (USACE 1976a). The hydraulic model and its results, including cross sections and roughness factors, were also taken from the USACE analysis (USACE 1976a). Starting water-surface elevations were from known water-surface elevations on downstream studies. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
North Nashua River Tributary A	Confluence with North Nashua River	Goss Lane	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
North Nashua River Tributary A1	Confluence with North Nashua River Tributary A	Headwaters at unnamed ponds	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
North Nashua River Tributary A2	Confluence with North Nashua River Tributary A	Approximately 1,400 feet above drive off Bull Hill Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
North Nashua River Tributary B	Confluence with North Nashua River	Approximately 800 feet above Old County Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
North Nashua River Tributary C	Confluence with North Nashua River	Approximately 500 feet above White Pond Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
North Nashua River Tributary D	Confluence with North Nashua River	Williams Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
O'Brien Brook	Confluence with Godfrey Brook	Vincenzo Road	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	Overbank portions of cross sections were from topographic maps (Quinn 1979). Underwater portions were from field surveys. Structures were from construction plans, where available, or field surveys otherwise. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations on Godfrey Brook. Floodplain was redelineated (FEMA 2011) in 2022 Charles Watershed revision.

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
Oak Hill Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (360.5 feet NAVD88).
Otter River	Templeton/ Winchendon corporate limits	Gardner/ Templeton corporate limits	Streamgage analysis and regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	5/1/1978	AE w/ floodway	A log-Pearson type III statistical analysis (WRC 1976) of records at USGS streamgage 01163200 (Otter River at Turner Street, period of record 11 years) was developed. Final discharges at the gage were calculated as a weighted average between results of the streamgage analysis and regression equations (Wandle 1977). Discharges were transposed to ungaged locations using a drainage-area ratio equation with an exponent of 0.75 (USACE 1970). Overbank portions of cross sections were from topographic maps (Lockwood 1979). Underwater portions and structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from the slope-area method.
Overlook Reservoir	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (829.6 feet NAVD88).
Overlook Road swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (1,013.2 feet NAVD88).
Paradise Pond	Entire shoreline	Entire shoreline	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1979	AE	Stillwater water-surface elevations were taken from the upper limit of study of Keyes 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
Partridge Pond	Entire shoreline	Entire shoreline	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	1/1/1978	AE	Stillwater water-surface elevations were taken from the upper limit of study of Tributary to Round Meadow Pond.
Patton Road swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (245.7 feet NAVD88).
Pearl Hill Brook	Confluence with Baker Brook 2	Second crossing of Fitchburg/ Lunenburg corporate limits near New West Townsend Road	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1973a)	6/1/1980	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Johnson and Tasker 1974). The 0.2-percent-annual-chance discharge was computed from the others using a log-Pearson type III distribution. Overbank portions of cross sections were from topographic maps (MADPW 1968, Teledyne 1977). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and photography. Starting water-surface elevations were from normal depth. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Pearl Hill Brook 2	County Boundary	Point of one square mile of drainage area	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Pearl Hill Brook 2 Tributary B	Confluence with Pearl Hill Brook 2	County Boundary	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Perley Brook	Confluence with Otter River	Just below Cowee Pond	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	5/1/1978	AE w/ floodway	Overbank portions of cross sections were from topographic 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
Phillips Brook	Confluence with North Nashua River	Winnekeag Lake	Regression equations (Johnson and Tasker 1974)	HEC-2 (USACE 1973a)	6/1/1980	AE w/ floodway	Discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Johnson and Tasker 1974). The 0.2-percent-annual-chance discharge was computed from the others using a log-Pearson type III distribution. Results were adjusted to account for a high-water mark on a control structure from the 1936 flood (MAGS 1939). Overbank portions of cross sections were from topographic maps (MADPW 1968). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and photography. Starting water-surface elevations were from normal depth. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Phillips Brook	Winnekeag Lake	Approximately 1,200 feet above Stowell Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Phillips Brook Tributary A	Confluence with Phillips Brook	Confluence with unnamed tributary below corporate limits	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Phillips Brook Tributary B	Confluence with Phillips Brook	Approximately 1,200 feet above River Styx Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Phillips Brook Tributary C	Confluence with Phillips Brook	Approximately 500 feet above State Route 12	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Phillips Brook Tributary D	Confluence with Phillips Brook	Lincoln Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Piccadilly Brook	County boundary	Just above Upton Road	TR-20 (SCS 1965)	HEC-2 (USACE 1973a)	3/1/1978	AE w/ floodway	Hydrologic computations were taken from an SCS study of the upper Sudbury River (SCS 1973). Overbank portions of cross sections were from topographic maps. Underwater portions and structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from known water-surface elevations (SCS 1973). The hydraulic model was calibrated to high-water marks from the March 1968 flood obtained from newspaper articles (Worcester Telegram 1968), a USGS report (USGS 1970), and interviews with local residents.
Pikes Pond Tributary	Pikes Pond	Approximately 900 feet above railroad	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	7/1/1980	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Quinn 1978a). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method.
Pine Hill Reservoir Tributary A	Pine Hill Reservoir	Asnebumskit Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Pine Hill Reservoir Tributary B	Pine Hill Reservoir	Inwood Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Pond Brook	Confluence with Otter River	Approximately 100 feet below Timpany Boulevard	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	5/1/1978	AE w/ floodway	Results of regression equations (Wandle 1977) were adjusted to account for extensive amounts of impervious cover in the upper portions of the watershed. Crystal Lake's drainage area was assumed to add no significant discharge to Pond Brook. Overbank portions of cross sections were from topographic 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.
Pondville Pond	Entire shoreline	Entire shoreline	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	1/1/1977	AE	Stillwater water-surface elevations were derived from model developed for Ramshorn Brook.
Poor Farm Brook 1	Mouth at Chaffin Pond	Swamp above Newell Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Quacumquasit Pond	Entire shoreline	Entire shoreline	SWAMP (SCS 1975b)	SWAMP (SCS 1975b)	11/1/1980	AE	Quaboag River, Quaboag Pond, East Brookfield River, and adjacent wetland areas were modeled as a series of interconnected storage areas rather than stream reaches to more adequately represent available flood storage and known flow-reversal situations.
Quick Stream	Harris Pond	Approximately 3,200 feet above Harris Pond	HEC-1 (USACE 1973b)	HEC-2 (USACE 1973a)	7/1/1976	AE w/ floodway	A HEC-1 model (USACE 1973b) was developed for the entire Mill River watershed, which includes Quick Stream. Discharges from the Mill River model were used along with survey data surrounding Harris Pond to develop a stage-discharge-frequency relationship for Harris Pond. Cross sections were from field surveys. Roughness factors were from field inspection, references (Barnes 1967), and calibration using high-water marks. Starting water-surface elevations were from known water-surface elevations on Harris Pond.

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
Quinapoxet River	Mouth at Wachusett Reservoir	Approximately 2,000 feet below River Street	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Quinapoxet River	Approximately 2,000 feet below River Street	Approximately 1,200 feet above State Route 31	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	6/1/1979	AE w/ floodway	Overbank portions of cross sections were from aerial photography (Berger 1973). Underwater portions and structures were from field surveys. Roughness factors were from engineering judgment and field observations. Starting water-surface elevations were from the slope-area method. Floodplain was redelineated (FEMA 2011) in 2022 Nashua Watershed revision.
Quinapoxet River	Approximately 1,200 feet above State Route 31	Muschopauge Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Quinapoxet River Tributary A	Confluence with Quinapoxet River	Raymond Huntington Highway	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Quinapoxet River Tributary B	Confluence with Quinapoxet River	Swamp above Malden Street	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Quinapoxet River Tributary C	Confluence with Quinapoxet River	Chaffin Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Quinapoxet River Tributary D	Confluence with Quinapoxet River	Confluence with unnamed tributary below Cutler Road	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit 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
Quinebaug River	Approximately 145 feet above Fabyan Woodstock Road	Approximately 4,200 feet above Main Street	HEC-HMS 3.0 and up (Dec 2005)	HEC-RAS 5.0 and up	10/10/2019	AE w/ floodway	
Quinebaug River	Stallion Hill Road	Approximately 300 feet below Lond Pond	HEC-HMS 3.0 and up (Dec 2005)	HEC-RAS 5.0 and up	10/10/2019	AE w/ floodway	

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
Quinsigamond River	Confluence with Blackstone River	Hovey Pond Dam	Log-Pearson type III flood-frequency analysis (IACWD 1982)	HEC-2 (USACE 1973a)	11/1/1989	AE w/ floodway	Peak discharges were calculated in USGS 1979 using Bulletin 17B log-Pearson type III flood-frequency analysis (IACWD 1982) on USGS streamgage 01110000 (Quinsigamond River at North Grafton, period of record 1940 to 1978). Discharges were transferred downstream to ungaged locations using a drainage-area ratio equation with an exponent of 0.7 (Chow 1967). Discharges calculated in this manner at the outlet of Lake Ripple were routed through the lake to account for the attenuation effect of lake storage (MADEQ 1980). Routed discharges were transferred downstream to the mouth using the drainage-area ratio equation. Discharges at the mouth were computed as the average of these discharges and the discharges computed in the hydrologic analysis for Blackstone River. These averaged discharges were then transferred back upstream as far as Lake Ripple using the drainage-area ratio equation. Overbank portions of cross sections were from field surveys and topographic maps. Underwater portions were from field surveys or interpolated from topographic maps. Structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations at the downstream end were from the slope-area method. Starting water-surface elevations at Lake Ripple Dam were based on a rating curve for the dam. Dam analysis assumed conservatively that manually operated sluice gates between the two spillways and a flume on the west bank adjacent to the dam would not pass flow. However, the Town of Grafton's Emergency Action Plan contains provisions formally addressing dam operation and maintenance that could be used in future analysis to reduce the computed water-surface elevations for Lake Ripple.

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
Ramshorn Brook (Town of Auburn)	Auburn Pond	Pondville Pond	Rainfall-runoff routing (SCS 1972)	HEC-2 (USACE 1973a)	1/1/1977	AE w/ floodway	Overbank portions of cross sections were from topographic maps (USGS various). Underwater portions and structures were from field surveys. Roughness factors were from field inspection and reference texts (Barnes 1967). Starting water-surface elevations were from known water-surface elevations on Auburn Pond. The upstream end of the model determined water-surface elevations for Pondville Pond.
Ramshorn Brook (Town of Millbury)	Pondville Pond	Griggs Road	HEC-1 (USACE 1973b)	HEC-2 (USACE 1973a)	6/1/1996	AE w/ floodway	The HEC-1 hydrologic simulation used the NRCS runoff curve number method. Cross sections and structures were from field surveys. Roughness factors were from field inspection. Starting water-surface elevations were from the slope-area method.
Rawson Hill Brook	Approximately 0.025 mile below Prospect Street	Boylston/ Shrewsbury corporate limits	Regression equations (Wandle 1977)	HEC-2 (USACE 1973a)	3/1/1978	AE w/ floodway	Above Rawson Hill Dam, discharges for the 10-, 2-, and 1-percent-annual-chance events were determined from regression equations (Wandle 1977). The 0.2-percent-annual-chance discharge was computed from the others using a log-Pearson type III distribution. At the dam, discharges were computed by SCS (SCS 1975a). Discharges were found to decrease with increasing drainage area due to storage at Rawson Hill Dam. Overbank portions of cross sections were from topographic maps (Teledyne 1976). Underwater portions and structures were from field surveys above Rawson Hill Dam and from SCS 1975a below the dam. Roughness factors were from field inspection and aerial photography (Teledyne 1976). Starting water-surface elevations were from known water-surface elevations on downstream studies (SCS 1975a).
Rice Meadow Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (832.0 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
Riverdale Mills Sluice Gates & Tail Race	Confluence with Blackstone River	Diversion from Blackstone River	Conveyance computations	HEC-RAS	9/1/1999	AE	Peak discharges were calculated on Blackstone River. The Riverdale Street Dam in Northbridge was modeled to provide flood relief according to the Emergency Action Plan by Riverdale Mills Corporation. According to the plan, the corporation opens a flood modulator gate, removes stoplogs, and opens two sluice gates allowing flow to pass through a tail race under the mill, preventing overtopping of Riverdale Street and damage to the mill complex and upstream properties. The tail races were modeled using a separate HEC-RAS model. The diversion model used starting water-surface elevations from normal depth and was calibrated to match the water-surface elevations of the mainstem model at the upstream end.
Robbins Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (245.5 feet NAVD88).
Rocky Brook 2	Confluence with Stillwater River	Hy-Crest Pond	Regression equations (Zarriello 2017)	HEC-RAS 5.0 (USACE 2016)	7/15/2019	A	See special considerations for Asnebumskit Brook.
Rocky Hill swamp	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (754.1 feet NAVD88).
Rocky Pond	Entire shoreline	Entire shoreline	none	none	11/1/2019	A	Analysis of lidar DEM (FEMA 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 (891.7 feet NAVD88).