US Army Corps of Engineers®
New York District

WESTCHESTER COUNTY STREAMS, BYRAM RIVER BASIN

FLOOD RISK MANAGEMENT FEASIBILITY STUDY

FAIRFIELD COUNTY, CONNECTICUT AND WESTCHESTER COUNTY, NEW YORK

FINAL INTEGRATED FEASIBILITY REPORT & ENVIRONMENTAL IMPACT STATEMENT

APPENDIX B.2:

Hydraulics

The purpose of the existing conditions and future without project conditions hydraulics analysis is to develop the inundation extents and peak water surface profiles associated with a set of design flood conditions: 100-, 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2-percent events. The flows developed in **Appendix B1** are the basis of the flow regimes used in the hydraulic simulations. This Appendix also analyzes the impacts of climate change and sea level rise as well as a sediment transport for conditions with and without the U.S. Route 1 Bridge Improvements.

1. PROJECT AREA OF INTEREST AND APPROACH

The Byram River basin is almost entirely within the extents of the City of Greenwich, Connecticut with headwaters north across the border in New York state. The total contributing area at the river mouth is 30 square miles. The riparian zone of the lower three miles of the Byram River is populated with suburban housing and commercial buildings. In the upper reach, upstream of the bridge at Bailiwick Road, the area is less densely developed.

The project area of interest is the main branch of the Byram River from the U.S. Route 1 bridge crossings upstream to the Comly Ave bridge and the area directly upstream of the Bailiwick Ave bridge. There has been historic flooding during extreme rainfall events in the Caroline Pond area just upstream of the U.S. Route 1 bridges. **Figure 1** shows the area of interest.

A flood control levee completed by the U.S. Army Corps of Engineers (USACE) in August 1961 on the east bank of the Byram River downstream of the Comly Ave bridge and adjacent to Halock Drive is also shown on **Figure 1**. The flood control project includes an earthen levee and approximately 3,000 feet of channel widening and deepening modifications. It was constructed under Public Law 685, 84th Congress.

A HEC-RAS model of the entire Byram River extending beyond the project area extents was built from prior studies and recent survey. This model was used to represent the hydraulic routing of historic floods and design discharge regimes described in **Appendix B1** of this report.

2. PRIOR HYDRAULIC ANALYSES

The analysis documented in this Appendix represents a refinement of prior hydraulic analyses conducted on the Byram River. The following reports served as a source of model input data.

2.1. USACE Flood Plain Information Summary Report (1964)

In 1964, USACE prepared a Flood Plain Information Report for the Byram River from the NYNH&H railroad bridge to about a mile upstream of the Merritt Parkway (USACE, 1964). The analysis included hydraulic modeling of the main reach to generate water surface profiles for design discharges, and estimates of the peak discharge associated with the October 1955 Flood.

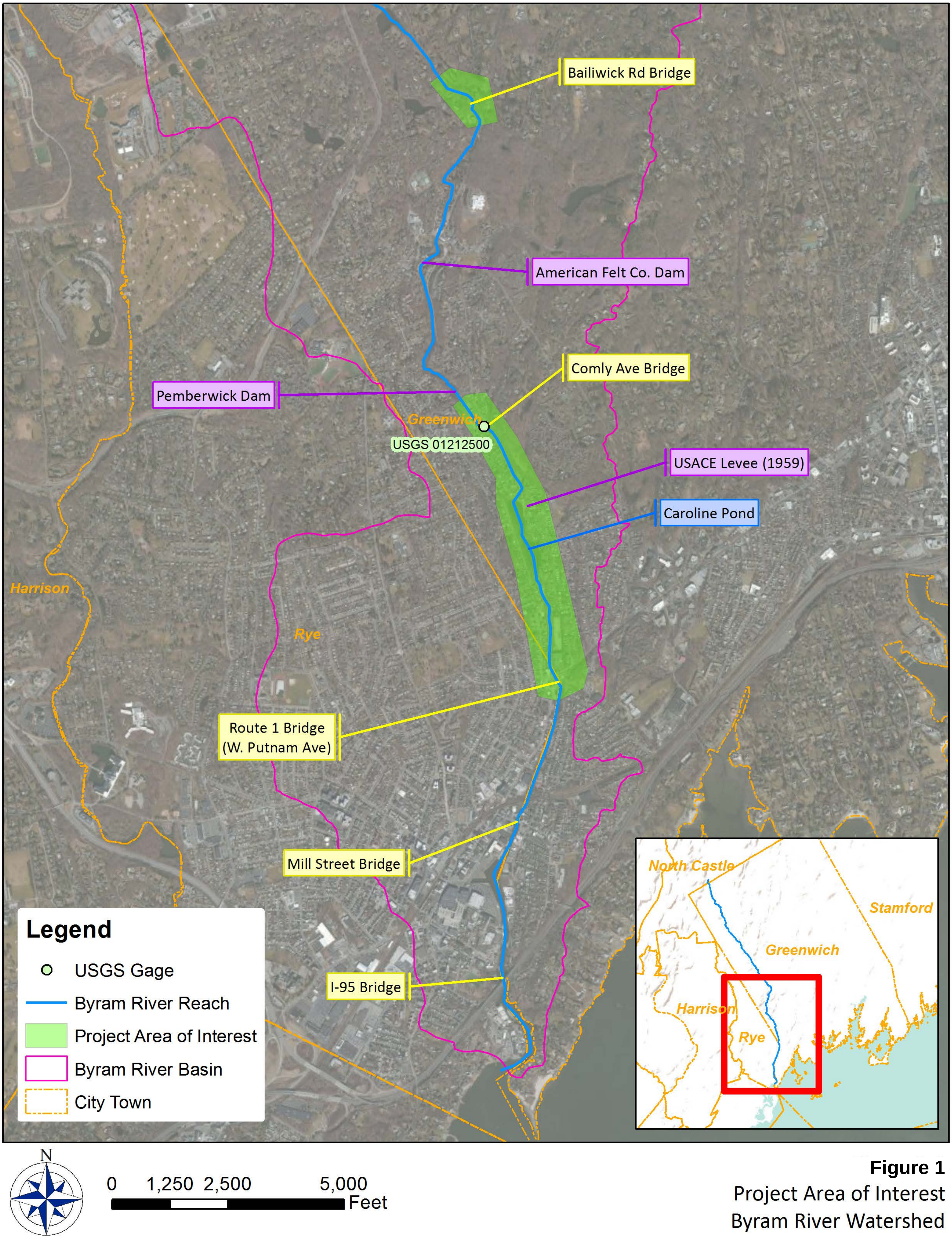


Figure 1
Project Area of Interest
Byram River Watershed

2.2. USACE Detailed Study (1975) and Feasibility Report (1977)

In 1975, USACE built a HEC-2 model of the Byram River from the Mill Street Bridge to the Toll Gate Pond Dam under Inter-Agency Agreement No. IAA-H-17-74, Project No. 15 (FEMA, 2010). The model was used to create flood profiles for the 1986 Flood Insurance Study (FIS) report published by the Federal Emergency Management Agency (FEMA).

The results of the detailed study are also reported in Appendix B of the 1977 Feasibility Report for Flood Control also published by the USACE.

2.3. CDM Drainage Study (2008)

In 2008 CDM completed a detailed study of the main branch of the Byram River. The hydraulic analysis was performed with a HEC-RAS model built in the HEC-GeoRAS environment (CDM, 2008). The 2008 model was an update of the 1975 HEC-2 detailed study in the effective FEMA FIS and used many of the same cross sections. Field survey was used to improve cross section accuracy and add detail to the original HEC-2 geometry. The model extents were also lengthened upstream by more than 4 miles ending at the Interstate 684 culvert. Record drawings from the Town of Greenwich and ConnDOT were used for verification of bridge and culvert geometry.

3. HYDRAULIC MODEL

The HEC-RAS model built by CDM in 2008 was further updated with additional detail based on field survey collected in 2012 and new cross sections were added with HEC-GeoRAS using recent LiDAR information collected by USACE in 2012 (Post-Sandy Coastal LiDAR). The U.S. Route 1 bridge crossings are a key feature in the project area of interest. Due to inconsistencies in data on the openings for these structures they were surveyed in early 2014. To review the effects of the bridge replacement on existing hydraulic conditions, the conceptual design of the U.S. Route 1 bridge replacement plan was incorporated into the HEC-RAS model.

3.1. Cross Sections

The 250 cross sections in the updated HEC-RAS model are constructed from a variety of data sources summarized in **Table 1**.

Table 1 HEC-RAS Cross Section Data Source

Number of Cross Sections	Bathymetry in Channel	Overbank Elevations
37	Original geometry from 1975 USACE HEC-2 model.	2012 LiDAR (ft NAVD88)
33	Survey conducted in 2007 for CDM Drainage Study	2012 LiDAR (ft NAVD88)
113	Trapezoidal channels added in the 2008 CDM Drainage	2012 LiDAR (ft NAVD88)

Number of Cross Sections	Bathymetry in Channel	Overbank Elevations
	Study based on thalwag interpolation between surveyed cross sections.	
38	Survey conducted in 2012 for this CDM Smith updated analysis. The survey focused on reach between Comly Ave Bridge and Mill Street Bridge	2012 LiDAR (ft NAVD88)
29	Interpolated bathymetry shape and thalwag between surveyed cross sections	2012 LiDAR (ft NAVD88)

The overbank elevations for all cross sections were developed with HEC-GeoRAS using the Town of Greenwich 2 ft contours developed from LiDAR collected by USACE in 2012. This updated the overbank elevation data even in the cross sections used from 1975 USACE model.

Since the available LiDAR does not have ground elevations below the water surface, the bathymetry of cross sections was taken from the best available source listed in **Table 1** and stitched into the overbank elevations. The basis of 71 cross sections is either (1) channel survey transects taken either in 2007 for the CDM Drainage Study or (2) channel survey transects taken in 2012 specifically for the analysis presented in this Appendix B2. The original HEC-2 bathymetry was available at 37 cross sections.

For cross sections where survey was not taken, the bathymetry was interpolated from upstream and downstream survey or HEC-2 data. Those cross sections created without survey or HEC-2 data during the 2008 CDM Drainage Evaluation are trapezoidal cross sections with a linearly interpolated thalwag. Cross sections interpolated from the 2012 survey between Comly Ave and Mill Street are based on both the thalwag and cross section shape of the upstream and downstream survey transects.

Figure 2 shows a map of all cross sections in the updated HEC-RAS model.

3.2. Bridges and Culverts

The updated HEC-RAS model includes 22 bridges and culverts which were either added or refined during the 2008 CDM Drainage Evaluation, during which structure geometry was obtained from field survey and record drawings obtained from the Town of Greenwich and ConnDOT and converted to the NAVD88 datum. **Table 2** lists the bridges and culverts in the updated HEC-RAS model.

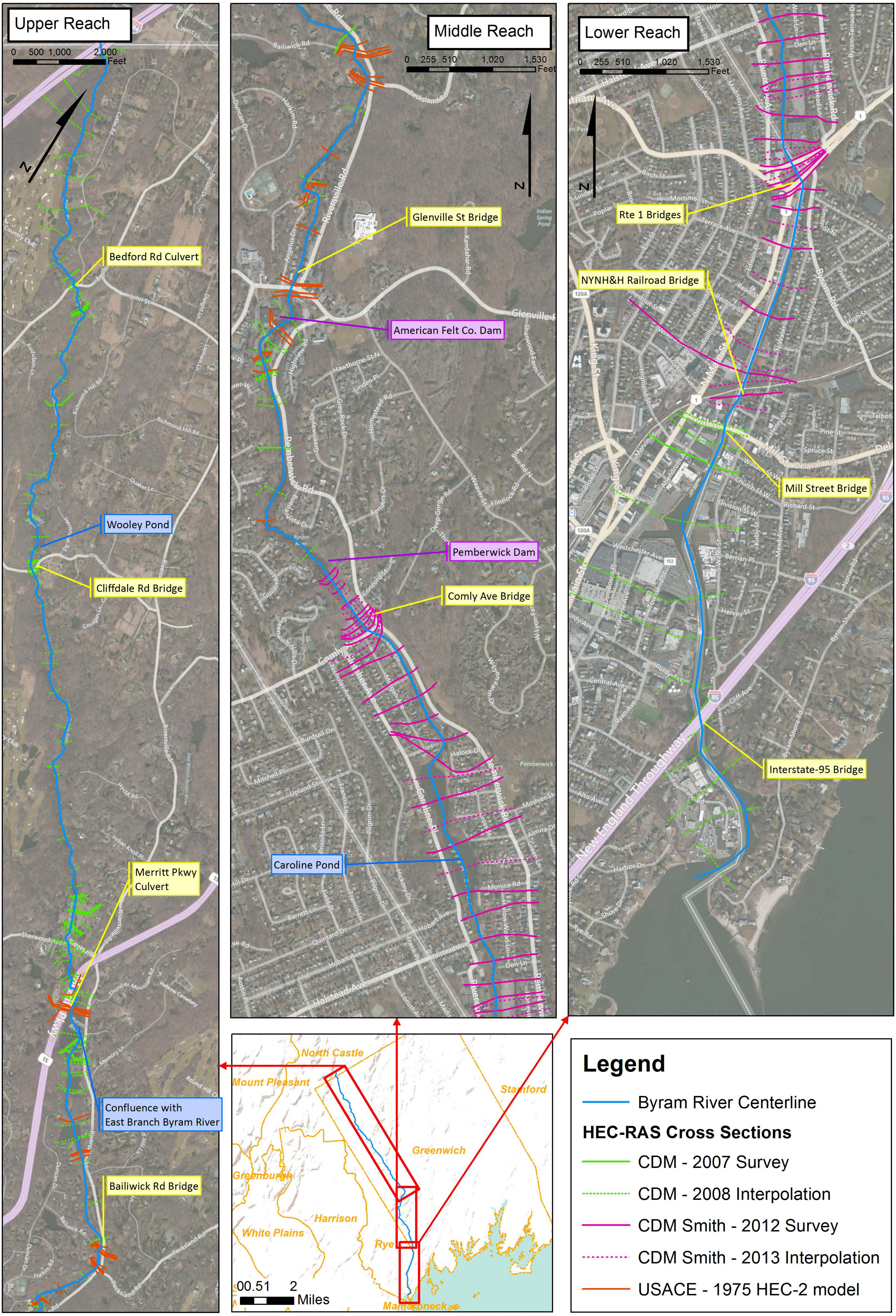


Figure 2
HEC-RAS Cross Sections
Byram River Main Branch

Table 2 HEC-RAS Bridges and Culverts

River Station (ft)	HEC-RAS Node Name	Structure Name and Location	Channel Bottom and Low Chord Elevation (NAVD88 ft)	Flow Area (ft^2)	Opening
46,846.4	BR_30	Bedford Rd Culvert	Crown = 362.30 ft Invert = 354.50 ft	~ 90 ft^2	One arched opening
39,666.6	BR_27	Cliffdale Rd Bridge	Low Chord = 286.00 ft Channel = 268.00 ft	~ 900 ft^2	One large opening
30,923.6	BR_25	Sherwood Ave Culvert	Crown = 160.30 ft Invert = 152.80 ft	~ 170 ft^2	Box culvert with angled edges
29,836.2	BR_24	Merritt Parkway - Rte 15	Crown = 152.00 ft Invert = 141.00 ft	~ 500 ft^2	Three box culverts 15' W x 11' H
29,270.2	BR_23	Abandoned Bridge	Low Chord = 147.70 ft Channel = 139.65 ft	~ 270 ft^2	One arched opening
28,537.0	BR_22	Footbridge	Low Chord = 143.00 ft Channel =138.00 ft	~ 200 ft^2	Small footbridge
28,169.9	BR_21	Footbridges	Low Chord = 141.00 ft Channel = 135.67 ft	~ 240 ft^2	Three small footbridges
23,650.8	BR_19	Bailiwick Rd Bridge	Low Chord = 130.85 Channel = 122.15 ft	~ 250 ft^2	One arched opening
23,232.1	BR_18	Pecksland Rd Bridge	Low Chord = 130.55 ft Channel = 118.45 ft	~ 310 ft^2	One opening
20,342.8	BR_16	Glenville St Bridge	Low Chord = 118.55 ft Channel = 103.75 ft	~ 575 ft^2	One arched opening
19,590.1	BR_14	Footbridge	Low Chord = 90.30 ft Channel = 79.14 ft	~ 400 ft^2	Small footbridge
19,252.7	BR_12	Footbridge	Low Chord = 87.80 ft Channel = 71.13 ft	~ 610 ft^2	Small opening
19,099.0	BR_11	Utility Line Crossing	Low Chord = 82.20 ft Channel = 70.69 ft	~ 500 ft^2	Small footbridge
15,813.0	BR_09	Footbridge	Low Chord = 38.00 ft Channel =28.85 ft	~ 310 ft^2	Small footbridge
15,587.8	BR_08	Footbridge	Low Chord = 37.50 ft Channel = 28.22 ft	~ 450 ft^2	Small footbridge
15,401.4	BR_07	Comly Ave Bridge	Low Chord = 37.25 ft Channel = 26.30 ft	~ 525 ft^2	One opening
10,474.1	BR_06	Footbridge	Low Chord = 7.00 ft Channel = 1.66 ft	~ 140 ft^2	Small footbridge
9,444.3	BR_05	W. Putnam Ave SB	Low Chord = 12.50 ft Channel = 1.78 ft	~ 500 ft^2	Double arched opening
9,230.4	BR_04	W. Putnam Ave NB	Low Chord = 10.75 ft Channel = -1.93 ft	~ 560 ft^2	Double arched opening

6,609.8	BR_03	Amtrak RR Bridge	Low Chord = 571.01 ft Channel = -3.19 ft	~ 1,400 ft²	Double arched opening
6,082.7	BR_02	Mill St Bridge	Low Chord = 7.00 ft Channel = -6.42 ft	~ 1,800 ft²	Two 4 ft wide piers
2,447.5	BR_01	I-95 Overpass	Low Chord = 64.2 ft Channel = -13.0 ft	~ 25,000 ft²	Four 7 ft wide piers

The energy equation was selected for bridge modeling on all low flow computations. For bridges with highly constrained channels where WSEL reaches decking during design storms, high flow computations were performed using "Pressure and/or Weir" methods. For bridges with larger conveyance openings and higher decking, the energy equation was used.

Only the Merritt Parkway crossing was modeled with a HEC-RAS culvert. The three rectangular culverts are approximately 90 feet long. All other bridges were modeled using deck/roadway geometry.

3.3. Inline Structures

The updated HEC-RAS model includes 9 inline structures which were either added or refined during the 2008 CDM Drainage Evaluation, during which structure geometry was obtained from field survey, and record drawings obtained from the Town of Greenwich and ConnDOT and converted to the NAVD88 datum. **Table 3** lists the inline structures in the updated HEC-RAS model.

Table 3 HEC-RAS Inline Structures

River Station (ft)	HEC-RAS Node Name	Structure Name	CT Dam #	Spillway Length (ft)	Spillway Elev (ft NAVD88)	Height (ft)	Hazard Classification
46,220.4	BR_29	Dam near Bedford Rd	N/A	8	351.00	5	N/A
39,867.1	BR_28	Wooley Pond Dam	5710	55	280.00	6	BB
31,994.6	BR_26	Wilcox Pond Dam	5705	33	179.80	20	BB
31,369.5	BR_26B	Private Dam (Applecrest)	N/A	18	161.00	2	N/A
25,919.5	BR_20	Toll Gate Pond Dam	5737	70	136.25	10	A
21,626.0	BR_17	Dam near Angelus Drive	5708	74	121.55	10	BB
19,750.6	BR_15	American Felt Co. Dam	5704	49	109.25	30	C

19,330.5	BR_13	System Pond Dam	5729	8	79.00	8	A
16,211.1	BR_10	Pemberwick Dam	5703	62	70.92	36	C
A = Low Hazard BB = Moderate Hazard C = High Hazard							

In the 2012 survey completed in support of this Appendix, the spillway elevation and length was confirmed for two dams: (1) American Felt Co. Dam 500 feet downstream of the Glenville Street Dam, and (2) Pemberwick Dam, 800 feet upstream of Comly Ave Bridge.

3.4. Ineffective Flow Areas

There are three applications of ineffective flow area in the model: (1) the upstream and downstream face of bridge structures, (2) the upstream face of an inline structure, and (3) low-lying portions of a cross section that may create backwater inundation and some storage, but does not convey flow.

3.5. Energy Loss Coefficients

Manning's roughness was used for energy loss calculations. In each cross section, roughness coefficients were assigned to the main channel defined by the bank stations selected from cross section geometry and the left and right banks. Manning's roughness coefficients for the main channel of the Byram River ranged from n = 0.03 to n = 0.06 based on field observations of the size of rocks in channel and vegetation, and the associated coefficients described by Chow (1959) and Arcement and Schneider (1989). The overbank Manning's coefficients ranged from n=0.035 to 0.15. In some reaches, the Manning's coefficients were calibrated slightly to match the observed high-water marks as described in **Sections 4.1 and 4.2**.

The 1975 USACE used the same range of roughness coefficients in the effective FEMA FIS study for the main channel, and n=0.035 to n=0.125 for the overbanks (FEMA, 2010).

For the majority of channel cross sections, expansion and contraction loss coefficients of $K_{expansion} = 0.1$ and $K_{contraction} = 0.3$ were used. Where channel geometry created a rapid expansion or contraction between two cross sections, loss coefficients as high as $K_{expansion} = 0.5$ and $K_{contraction} = 0.7$ were used. Upstream of inline structures, loss coefficients as high as $K_{expansion} = 0.5$ and $K_{contraction} = 0.7$ were used. For the two cross sections upstream of a bridge and the cross section immediately downstream of a bridge, loss coefficients as high as $K_{expansion} = 0.3$ and $K_{contraction} = 0.5$ were used. For some cross sections, the Manning's coefficients were calibrated slightly to match the observed high-water marks as described in **Sections 4.1 and 4.2**.

3.6. Boundary Conditions

The downstream boundary condition for the Byram River hydraulic model is the stillwater elevation of Long Island Sound coinciding with the peak discharge routed through the main reach. The stillwater is defined as the coastal floodwater elevation in the absence of waves resulting from wind and includes storm surge associated with hurricanes and nor'easters.

There is considerable uncertainty regarding what stillwater elevation is expected to coincide with peak discharge from the Byram River. The analysis presented in this section determined that a single stillwater elevation of el. 6.9 ft. NAVD88 best represents the expected value for all discharge recurrence intervals. This stillwater elevation corresponds roughly with a 50-percent peak annual (2-year) recurrence interval. It is understood that there is a considerable uncertainty in this value, which is why it is included as one of the model parameter values in the uncertainty analysis described in **Section 4.3**.

Section 3.6.1 describes the effective FEMA coastal stillwater analysis that is available at the mouth of the Byram River. Section 3.6.2 describes the process of building a coincident record of peak discharge and tidal peaks from an available stillwater record at the Stamford Hurricane Barrier. **Section 3.6.3** presents the incorporation of sea level rise (SLR) over the life of the project. **Section 3.6.4** presents a summary of the statistics of the coincident record and the expected value of the stillwater boundary condition (el. 6.9 ft. NAVD88). The final **section (3.6.5)** summarizes the tidal boundary conditions for several historic storms.

3.6.1. Effective FEMA Coastal Stillwater Analysis

The closest coastal stillwater observations are 6 miles up the coast at the Stamford Hurricane Barrier where USACE maintains a 30-minute record that extends back to November 2001. The FEMA Flood Insurance Study (FIS) for Fairfield County CT (FEMA, 2010) provides extreme stillwater elevations at both the Stamford Hurricane Barrier (Transect 12) and at the mouth of the Byram River (Transect 1). **Figure 3** shows both transects on an excerpt from the Transect Location Map in the FIS (FEMA, 2010).

Figure 3 Transect Location Map in the FIS (FEMA, 2010)

At each location, the stillwater elevations were fitted to a logarithmic curve for percent annual recurrence in order to estimate the 1-, 2-, 5-, and 25-year peak annual stillwater elevations which are shown in Table 4.

Table 4 – Effective FEMA Peak Annual Coastal Stillwater Elevations

Scenario	Stillwater Elev. at Stamford Hurricane Barrier	Stillwater Elevation at Mouth of Byram River	Source of Data
100% Peak Annual (1-yr)	el. 6.2 ft NAVD88	el. 6.0 ft NAVD88	Extrapolated (log) from FIS
50% Peak Annual (2-yr)	el. 7.1 ft NAVD88	el. 6.9 ft NAVD88	Extrapolated (log) from FIS
20% Peak Annual (5-yr)	el. 7.9 ft NAVD88	el. 7.9 ft NAVD88	Extrapolated (log) from FIS
10% Peak Annual (10-yr)	el. 8.5 ft NAVD88	el. 8.7 ft NAVD88	FIS (FEMA, 2010)
4% Peak Annual (25-yr)	el. 9.3 ft NAVD88	el. 9.7 ft NAVD88	Interpolated (log) from FIS
2% Peak Annual (50-yr)	el. 10.0 ft NAVD88	el. 10.4 ft NAVD88	FIS (FEMA, 2010)
1% Peak Annual (100-yr)	el. 10.5 ft NAVD88	el. 11.2 ft NAVD88	FIS (FEMA, 2010)
0.2% Peak Annual (500-yr)	el. 11.9 ft NAVD88	el. 13.0 ft NAVD88	FIS (FEMA, 2010)

3.6.2. Coincident Tidal-Riverine Event Record

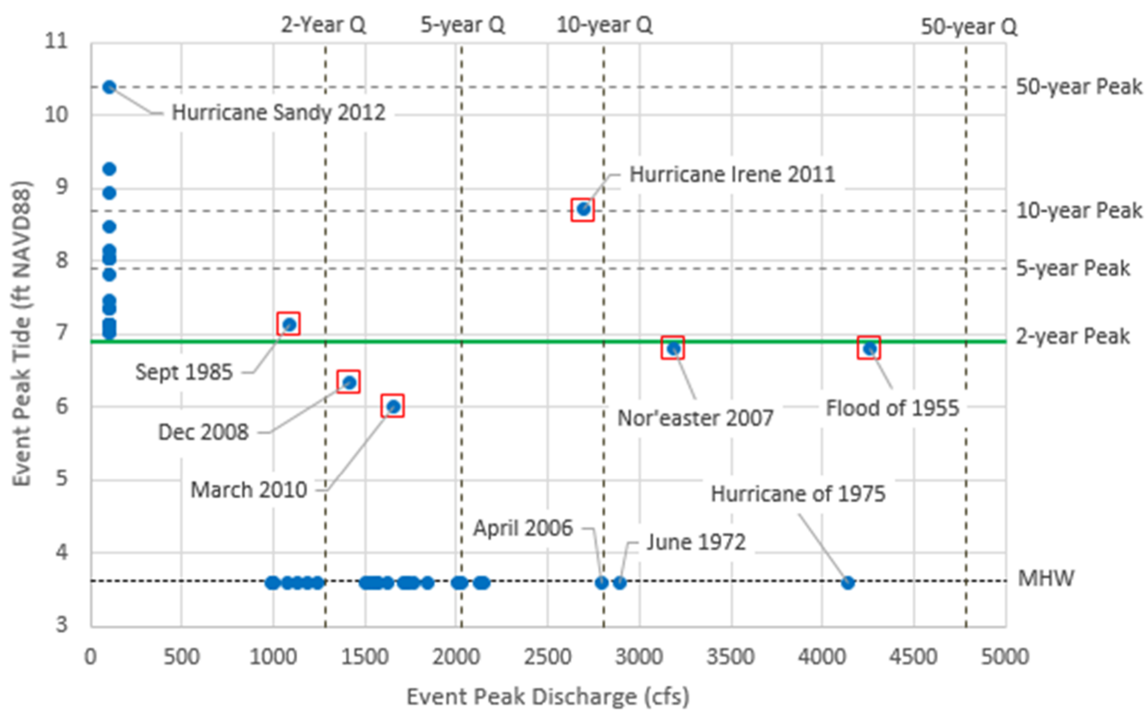
To determine the appropriate downstream boundary condition for riverine flooding on the Byram River, it is necessary to understand the coincidence between extreme riverine flood events and tidal-driven storm surge events. The Westchester County Streams - Byram River Basin Study is a fluvial flood risk management study. It is understood that the coastal region of Greenwich and Port Chester faces a combined hazard from both coastal flooding and riverine flooding. While previous studies published by USACE (1964, 1977) have indicated that the coastal flooding mechanisms are largely separate from the riverine flooding mechanisms, the riverine events are influenced by coastal storm surge. To better characterize the coastal influence on riverine events in the study area, coincident fluvial and coastal events were analyzed as described in this section. This study considers the coastal-fluvial relationship while formulating to reduce the risk of fluvial flooding events. Selection of a downstream boundary condition therefore should be based on the tidal condition which is coincident to the recurrent event (example: 100-year) that is associated with the same recurrence in extreme discharge (example: 100-year discharge).

USACE maintains a long record of maximum stillwater elevations observed at the Stamford Hurricane Barrier for all tides above 7.0 ft. NGVD29 (el. 5.9 ft. NAVD88) corresponding to roughly 6-month recurrence. There are 101 events in the record extending back to 1938. Each event the maximum stillwater record was fitted to a recurrence interval using the FEMA analysis shown in Table 4 to get the corresponding recurrence intervals.

For each extreme (tidal) stillwater event, a coincident record of peak discharge was identified from the synthetic record for the Byram River (described in **Section 3.2** of Appendix B1 – Hydrology). This peak annual discharge record is available from 1962 to 2013 (N=51). The Flood of 1955 was also included because it is still the flood of record within this watershed. For peak discharge events without a coincident maximum stillwater record it was conservatively assumed the stillwater was no more than Mean High Water (MHW) (el. 3.63 ft NAVD88).

In this period of coincident record there are 31 events in which the discharge exceeds the 1-year recurrence (830 cfs) and 22 events in which the tidal peak exceeds the 1-year recurrence (el. 6.0. ft NAVD88). Figure 4 shows a scatter plot of this subset of the coincident events (N=51) in which a further subset (in which both exceedance criteria are met, N=6) and are shown with red boxes. Each point represents the peak stillwater elevation and the peak discharge recurrence that coincided on the same day.

Figure 4 – Peak Discharge and Coincident Peak Coastal Stillwater from Coincident Record



While there is demonstrably a large degree of independence between extreme tidal and riverine events, the degree of dependence expressed by the six coincident extreme events (11% of all coincident events) should be considered for the selection of a boundary condition.

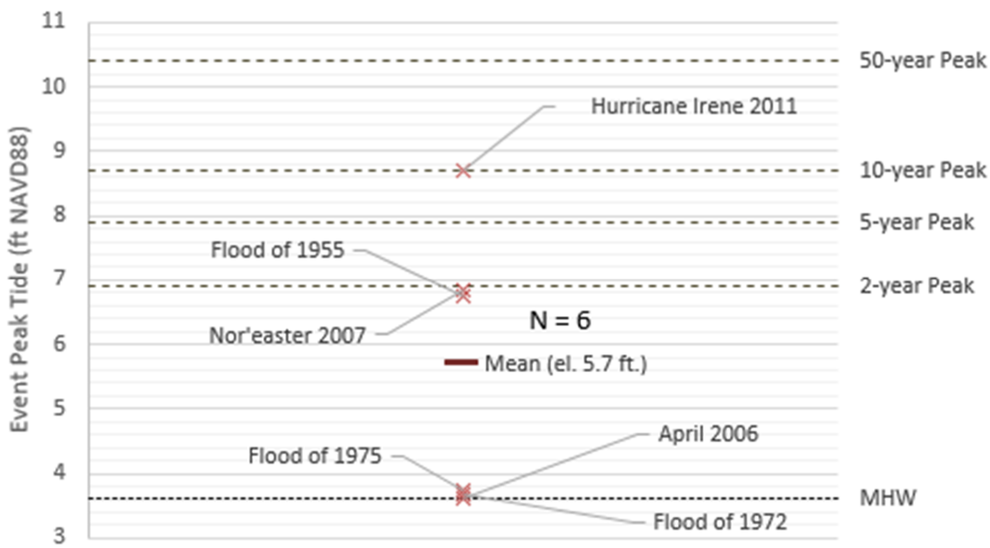
Table 5 – Summary of Coincident Peak Coastal Stillwater for Largest Peak Discharge Events in Record

Event	**Peak Discharge Recurrence**	**Peak Stillwater Elevation**	**Peak Stillwater Recurrence**
Flood of 1955	45.1 years	6.8 ft NAVD88	1.8 years
Nor'easter of 2007	21.2 years	6.8 ft NAVD88	1.8 years
Hurricane Irene 2011	13.5 years	8.7 ft NAVD88	10.3 years
Three additional events	*> 10-year*	*3.7 ft NAVD88*	*MHW*
Five additional events	*> 5-year*	*3.7 ft NAVD88*	*MHW*

March 2010	3.8 years	6.0 ft NAVD88	1.0 years
December 2008	2.5 years	6.3 ft NAVD88	1.2 years
September 1985	1.3 years	7.1 ft NAVD88	2.5 years

Table 5 summarizes the six events in which both stillwater exceeds 1-year recurrence and discharge exceeds 1-year recurrence. The table includes the eight additional events which exceed the 5-year peak recurrence for discharge but did not coincide with an extreme stillwater. Of the six discharge events greater than 10-year recurrence interval, only one (Hurricane Irene) coincided with a tidal stillwater of 2-years or above. Figure 5 summarizes the peak tidal stillwater associated with these six (N=6) events in which the peak discharge exceeds the 10-year event. The mean stillwater peak of this subset is el. 5.7 ft. NAVD88.

Figure 5 – Peak Tidal Stillwater Elevations Associated with Largest Discharge Events on Record



From analysis of the coincident record it is apparent that there is some degree of dependence between the extreme tidal and riverine events. Although there is a large degree of uncertainty for any given peak discharge event, the mean tidal stillwater for the record of events with large peak discharge is the best representation of the expected value of the tidal boundary condition (el. 5.7 ft. NAVD88).

3.6.3. Incorporating Sea Level Rise

In accordance with USACE ER 1100-2-8162, projected future Sea Level Rise (SLR) was incorporated into analysis of alternatives which includes the downstream boundary condition. As described in **Section 6.2**, the USACE Sea Level Change Calculator provides three SLR scenarios (“low”, “intermediate” and “high”). Following guidance in USACE ER 1100-2-8162 paragraph 6.d.(1), the approach selected for incorporating SLR into alternative selection is to perform the alternative analysis based on a single scenario followed by an evaluation under the other two scenarios to determine overall performance.

The single SLR scenario selected to evaluate alternative performance was the “intermediate” future over the anticipated project life until 2072. Of the three possible scenarios, "intermediate" was selected since it is the middle of the full range of anticipated scenarios provided by the USACE Sea Level Change Calculator. As described in **Section 6.2**, the anticipated SLR by 2072 is + 1.2 feet for the "intermediate" scenario. The evaluation of the Recommended Plan under the other two SLR scenarios ("low" and "high") required by USACE ER 1100-2-8162 is presented in **Section 6.2**. Note that these scenarios are also included in the upper and lower bound of the stage uncertainty analysis presented in **Section 4.5**.

3.6.4. Tidal Boundary Condition for Design Storms

The question of what downstream boundary condition to use for each steady-state discharge recurrence interval is complicated by several degrees of uncertainty and a limited coincident record of tidal and riverine discharge. In the 1977 Feasibility Report for Flood Control (USACE, 1977), the U.S. Army Corps of Engineers assumed a downstream boundary condition of 7.0 ft MSL (5.9 ft. NAVD88) for all riverine discharge events. This corresponds to approximately a 1-year recurrent tidal event.

For any given peak discharge event on the Byram River, there is significant uncertainty whether the event will coincide with a large tidal peak event on the same day as demonstrated by the available record and described in **Section 3.6.2**. The mean tidal peak from the record (N=6) of coincident events with riverine discharge in exceedance of the 10-year recurrence is el. 5.7 ft. NAVD88.

As described in **Section 3.6.3**, a single SLR future ("Intermediate") was incorporated into the downstream boundary condition for the selection of alternatives. As described in detailed in **Section 6.2**, the "Intermediate" climate scenario anticipates a stillwater rise of +1.2 feet by 2072 which is the end of the project life.

The overall expected tidal peak that coincides with peak discharges on the Byram River is a combination of the various expected terms described in **Section 3.6**. Table 6 shows the addition of the sea-level rise to the tidal elevation to be used in the design, which is 6.9 ft. The tidal peak happens to coincide with the 2-year peak annual stillwater el. 6.9 ft. NAVD88, which is also referred to as the 50%-peak annual recurrence.

Table 6 – Summary of Contributing Factors for Expected Coincident Peak Tidal Stillwater Elevation

Mean coincident tidal peak for peak discharge events	Section 3.6.2	el. 5.7 ft. NAVD88
"Intermediate" Sea Level Rise for 2072	Section 3.6.3	+1.2 ft.
Overall expected tidal peak associated with peak discharge events		el. 6.9 ft. NAVD88

It is understood that all of the discussed factors contribute to the uncertainty above and below what is an expected value for the tidal boundary. The stage uncertainty analysis presented in **Section 4.3** incorporates uncertainty of the downstream stillwater into the "likely combinations" of uncertain parameters.

3.6.5. Tidal Boundary Condition for Historic Validation Events

For the historic calibration events, estimates of the observed stillwater elevations at the mouth of the Byram River were developed from the available record at the Stamford Hurricane Barrier described in **Section 3.6.2**. For two of the events (Flood of June 1972 and Flood of September 1975) there is no record of an extreme stillwater tide. The peak stillwater for each of the four available events was translated into stillwater elevations at the mouth Byram River by the logarithmic curve relating recurrence interval and elevation at both sites and assuming the same recurrence interval for tidal storm surge along the 6-mile coast. Table 7 summarizes the estimated tailwater conditions for each event.

Table 7 – Summary of Historical Coincident Peak Tidal Stillwater Elevations

Historic Event	Peak Tidal Stillwater Elevation
Flood of October 1955	el. 6.8 ft NAVD88
Flood of June 1972	Not recorded
Flood of September 1975	Not recorded
Nor'easter of April 2007	el. 7.1 ft NAVD88
Hurricane Irene of August 2011	el. 8.7 ft NAVD88
Hurricane Sandy of October 2012	el. 10.4 ft NAVD88

3.7. U.S. Route 1 Bridge Replacement Hydraulic Model

To analyze the effect of the U.S. Route 1 bridge replacement on the current flood conditions, a new HEC-RAS model was developed to reflect a proposed condition geometry. U.S. Route 1, also known as Putnam Avenue, traverses Byram River as two separate bridges for northbound and southbound traffic. In the existing condition, both the north and south U.S. Route 1 bridges experience overtopping in the 2-percent flood event according to the HEC-RAS model.

The proposed improvements will replace both bridges in their existing locations and raise the roadway profile. The north and south bridge will have lengths of 82 feet and 93 feet, respectively. Each bridge will have a total depth of 4.5 feet which includes a 6-inch deck, 9-inch cross slope, and 3.25-foot beam depth. Barrier walls, matching the existing barrier wall height, will be incorporated on both sides of each bridge for pedestrian and vehicular safety. This will provide a larger opening of the bridge restricting less water.

The low chord of the proposed north bridge is approximately 12.71 feet and the low chord of the south bridge is 13.23 feet. The U.S. Route 1 bridge replacements will not experience overtopping until the 0.2-percent flood event, based on the six simulated events within the HEC-RAS model. **Figure 6** provides the flood profile for the existing condition flows at the proposed U.S. Route 1 bridge for all simulated events.

As reflected in **Table 8**, the U.S. Route 1 bridge replacements will significantly reduce peak stages in the vicinity of the bridges. There are negligible increases on the downstream side of the bridge as a result of the proposed bridges. All stage comparisons are with respect to vertical datum NAVD88 and are based on the existing condition flows. **Figure 7** displays the below information in a water surface profile.

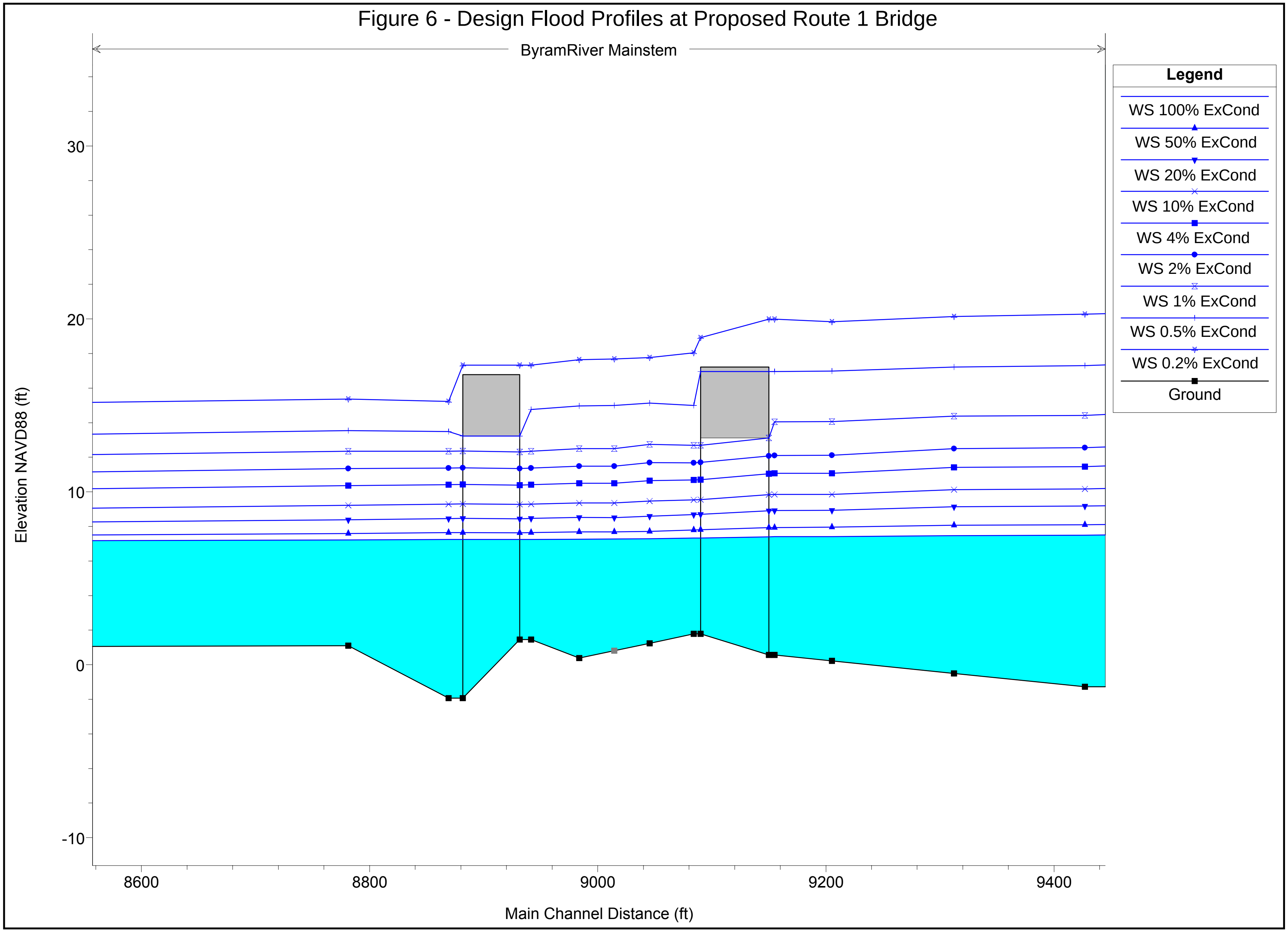
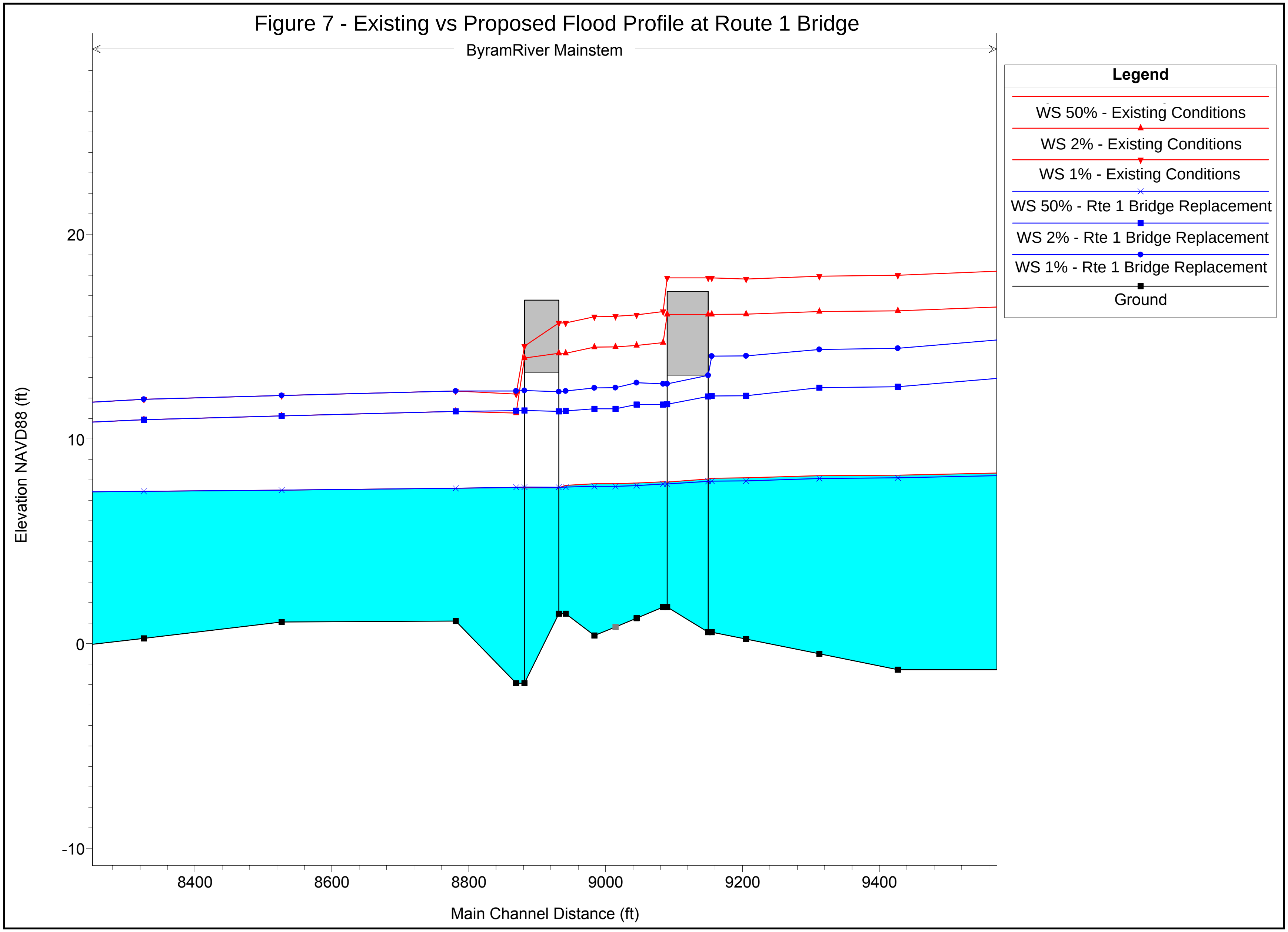


Figure 6 - Design Flood Profiles at Proposed Route 1 Bridge
ByramRiver Mainstem
Legend
WS 100% ExCond
WS 50% ExCond
WS 20% ExCond
WS 10% ExCond
WS 4% ExCond
WS 2% ExCond
WS 1% ExCond
WS 0.5% ExCond
WS 0.2% ExCond
Ground
Elevation NAVD88 (ft)
30
20
10
0
-10
8600
8800
9000
9200
9400
Main Channel Distance (ft)



(Note: Geometry reflects proposed bridge)

Table 8 Existing vs Proposed Stages at U.S. Route 1 Bridge

Location	HEC-RAS Cross section	Existing Condition Stage (ft)			Proposed Condition Stage (ft)		
		50% Flood	2% Flood	1% Flood	50% Flood	2% Flood	1% Flood
Upstream of North bridge	9633.9	8.20	16.22	17.95	8.06	12.50	14.37
Upstream of North bridge	9526.8	8.10	16.10	17.81	7.95	12.11	14.06
Immediately upstream of North Bridge	9476.7	8.08	16.08	17.87	7.94	12.10	14.04
North Bridge	9444.3	-	**	**	-	-	*
In between bridges	9405.8	7.9	14.71	16.23	7.8	11.68	12.69
In between bridges	9367.1	7.84	14.57	16.06	7.72	11.68	12.75
In between bridges	9336.19	7.81	14.5	15.99	7.68	11.48	12.5
In between bridges	9305.3	7.81	14.49	15.97	7.68	11.48	12.5
In between bridges	9263.3	7.73	14.18	15.67	7.64	11.37	12.34
South Bridge	9230.4	-	**	**	-	-	-
Immediately downstream of South Bridge	9190.9	7.62	11.26	12.19	7.64	11.38	12.35
Downstream of South bridge	9102.9	7.59	11.35	12.35	7.59	11.35	12.35
*Indicates stage at or above low chord but no bridge overtopping							
**Indicates bridge overtopping							

4. MODEL CALIBRATION AND UNCERTAINTY ANALYSIS

High water marks were available for three historic flood events, and two were used to calibrate some parameters in the HEC-RAS model. Several of the reported high-water marks are questionable and appear to be unreasonable in that they yield wildly inconsistent rating curves. These high-water marks may have been incorrectly recorded and/or result from an error in the reference elevation. Each of these questionable high-water marks are described in detail and were not used to calibrate the hydraulics of the model. In addition to calibration efforts, an uncertainty analysis of stages at the U.S. Route 1 proposed bridges was conducted in accordance with Section 5-5 of USACE EM 1110-2-1619.

4.1. April 2007 Flood (Nor'easter)

The April 2007 Nor'easter caused major flooding along the banks of the Byram River. Milone and MacBroom collected high water marks at 5 locations listed in **Table 9**.

The first 4 high water marks were collected in the neighborhood adjacent to Caroline Pond as shown in the map in **Figure 8**. There is one additional high-water mark recorded just upstream of the American Felt Co. Dam at Glenville Street as shown in **Figure 9**.

The HEC-RAS simulation of the 2007 Nor'easter flooding shows that only 2 of the 12 major crossings were overtopped during the peak discharge. The major crossings do not include privately owned footbridges, an abandoned stone archway, and a utility crossing, many of which were overtopped during the 2007 event. The two major bridges shown to have overtopped in the simulation of the 2007 flood are, the Sherwood Ave Culvert and the Bailiwick Road bridge. The backwater elevation from the observed storm surge (El. 7.9 ft NAVD88) extended upstream as far as the Mill Street Bridge.

Table 9 High Water Marks from April 2007 Flood on Byram River

Location No.	Greenwich, CT Address	Description of Flood Mark	Reference	Flood Elevation (ft NAVD88)	Computed Elevation (ft NAVD88)	Approximate HEC-RAS Station (ft)	Note
1	24 Homestead Lane	Flood mark is below siding on white wall, measuring down from siding distance of 1 ¼ siding boards for flood mark.	Light green house with white trim. Mark was on left side of house (adjacent to driveway)	El. 17.03	El. 13.95	10,290	Suspect
2	Den Lane Pump Station	Flood mark is middle bar on chain link fence at front left side	Small yellow building with dark trim;	El. 13.91	El. 14.31	10,840	Used

		of fence behind utility pole.	w/chain link fence enclosure				
3	10 Hollow Wood Lane	Flood Mark is top of bottom panel in white garage door located to left of concrete staircase.	2-Fam. Dwelling- Photo provided is at garage door on left side of building.	El. 14.30	El. 14.50	11,250	Used
4	14 Hollow Wood Lane	Flood Mark is on 3rd garage door from back of building- middle of 2nd panel (from bottom) of garage door, just above door handle.	2 story building (Yellow siding w/green shutters)- bldg. is on left side of driveway	El. 14.47	El. 14.62	11,130	Used
5	10 Glenville Street	Flood Mark is on rear wall of brick building near entrance door at ground level. Yellow keel mark is at mortar joint at bottom of 7th brick below ledge of window (with screen), between window and downspout.	Yellow keel mark on building between window and downspout.	El. 119.95	El. 115.97	19,850	Suspect

The simulated 2007 Flood profile in the vicinity of the recorded high-water marks at Locations 1 through 4 is shown in **Figure 10**. Observed water surface elevations at Locations 2, 3, and 4 are close to the simulated water surface with a difference of +0.40 ft, +0.20 ft, and +0.14 ft respectively. Although Location 1 is about 600 ft downstream of the other observed high-water marks with no significant hydraulic structures or changes in channel geometry, the recorded elevation there is more than 3 ft higher than those recorded upstream. The simulated water surface elevation is 3 ft lower. This high-water mark was not considered in model calibration.

The simulated 2007 Flood profile in the vicinity of the American Felt Co. Dam is shown in **Figure 11** along with the 2007 high water mark at Location 5. High water mark observations from 2011 Hurricane Irene, which are described in **Section 4.2**, are also shown. The high-water mark recorded for the 2007 flood (El. 119.95 feet) is more than 10 feet higher than the 49-foot-long spillway crest (El. 109.25 feet), which was surveyed in 2012 as mentioned in **Section 3.3**. The high-water mark at Location 5 is also more than 3 feet above the simulated water surface

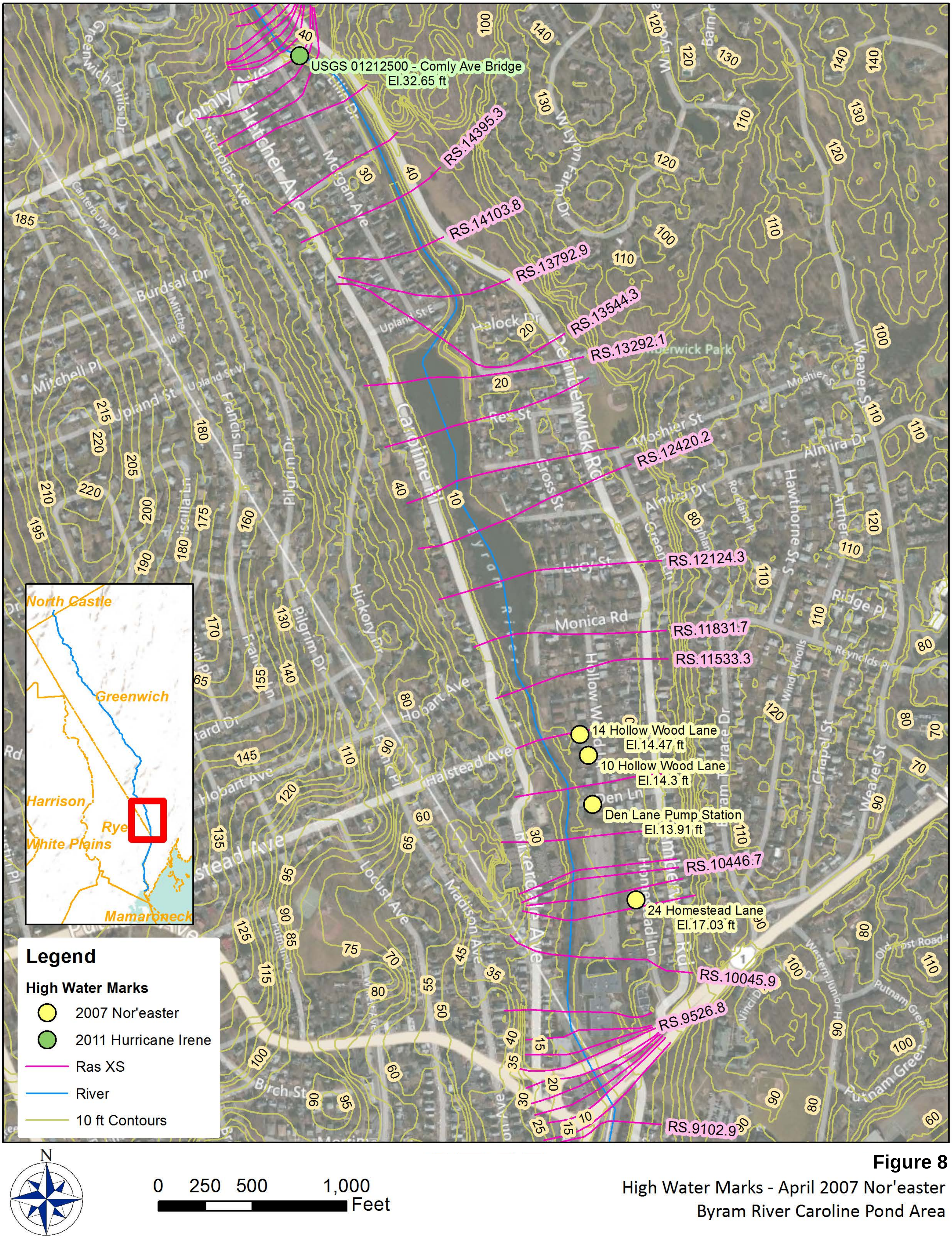


Figure 8

High Water Marks - April 2007 Nor'easter

Byram River Caroline Pond Area

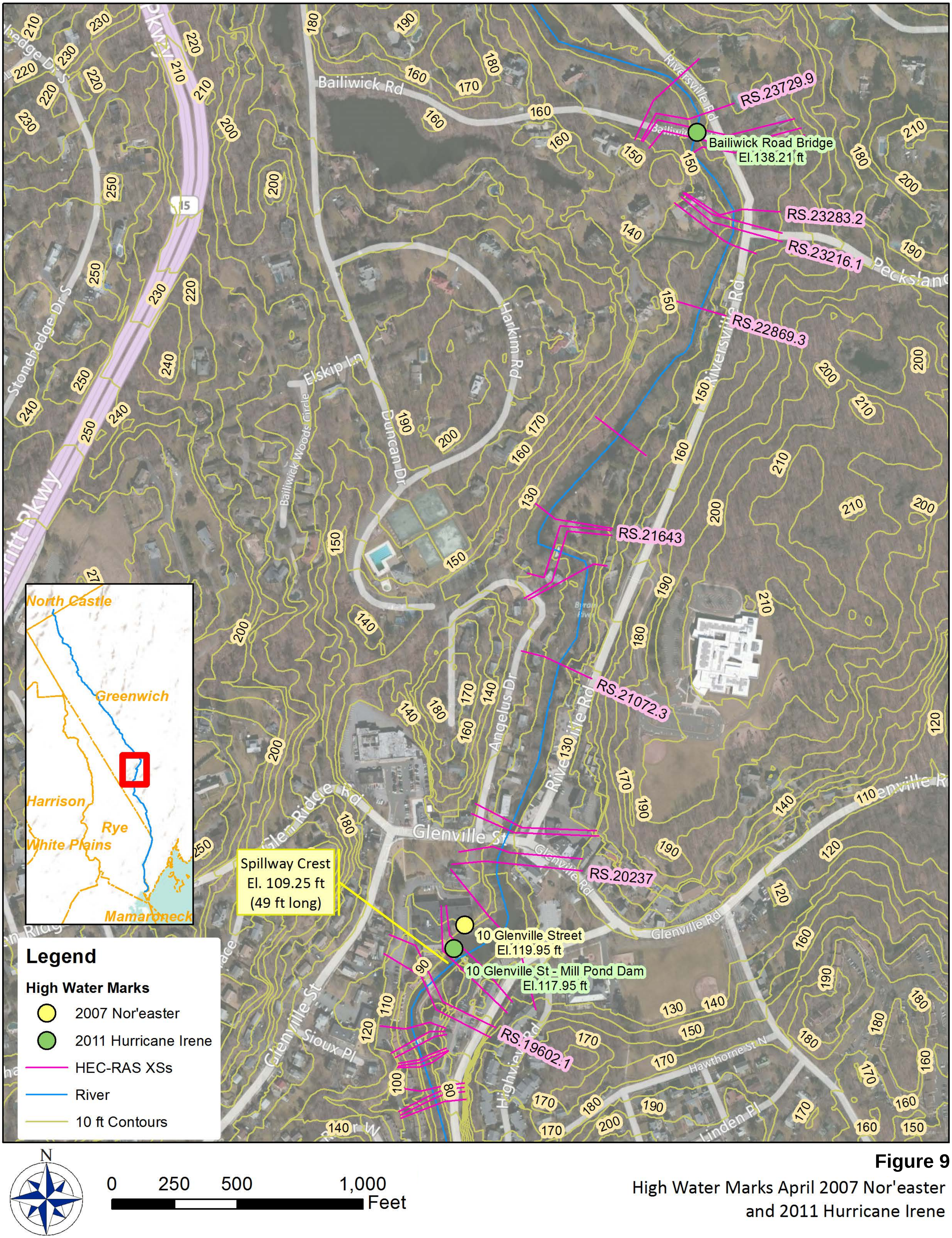


Figure 9

High Water Marks April 2007 Nor'easter
and 2011 Hurricane Irene

Figure 10 - Flood of 2011 Profile with High Water Marks

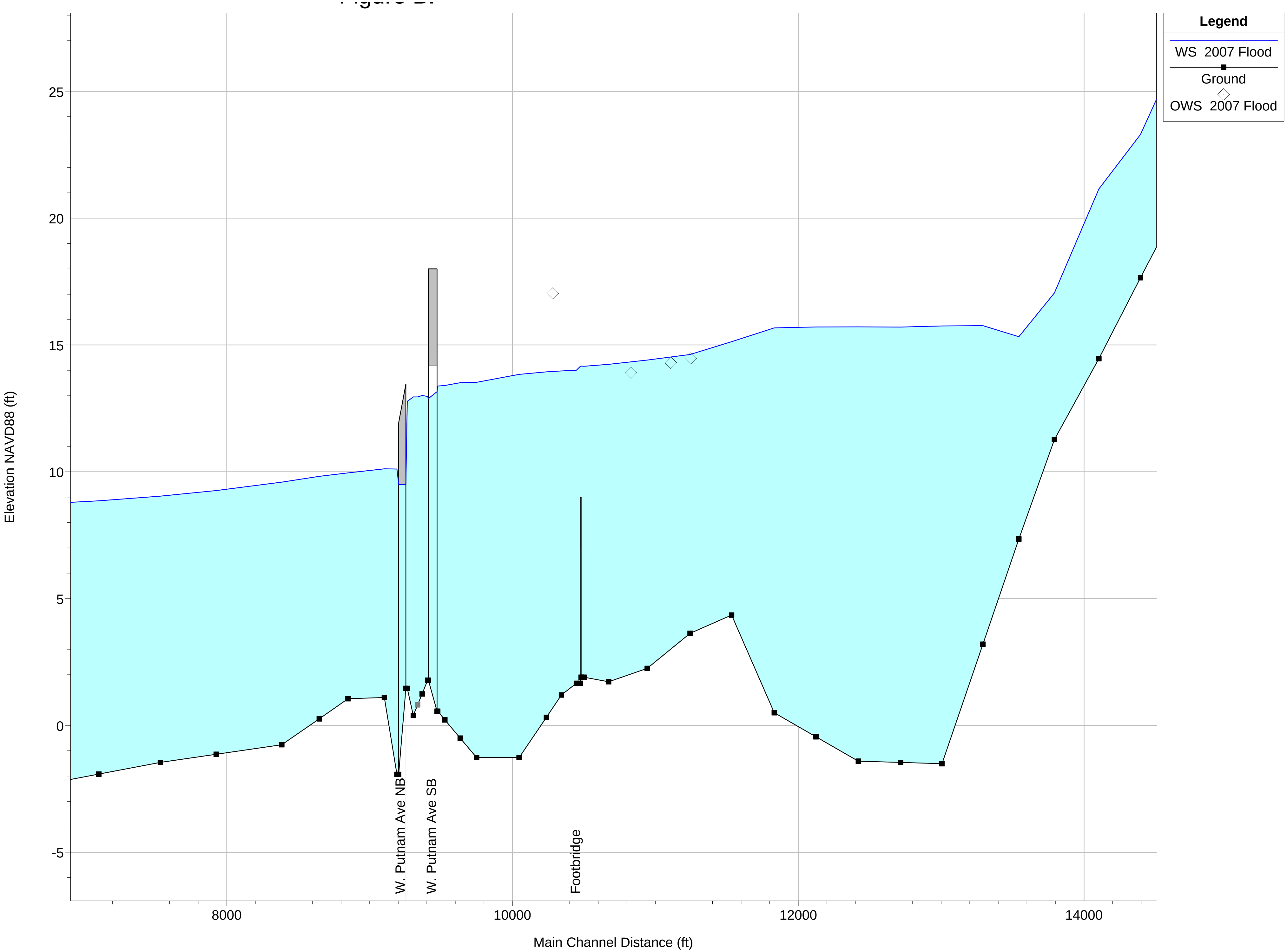
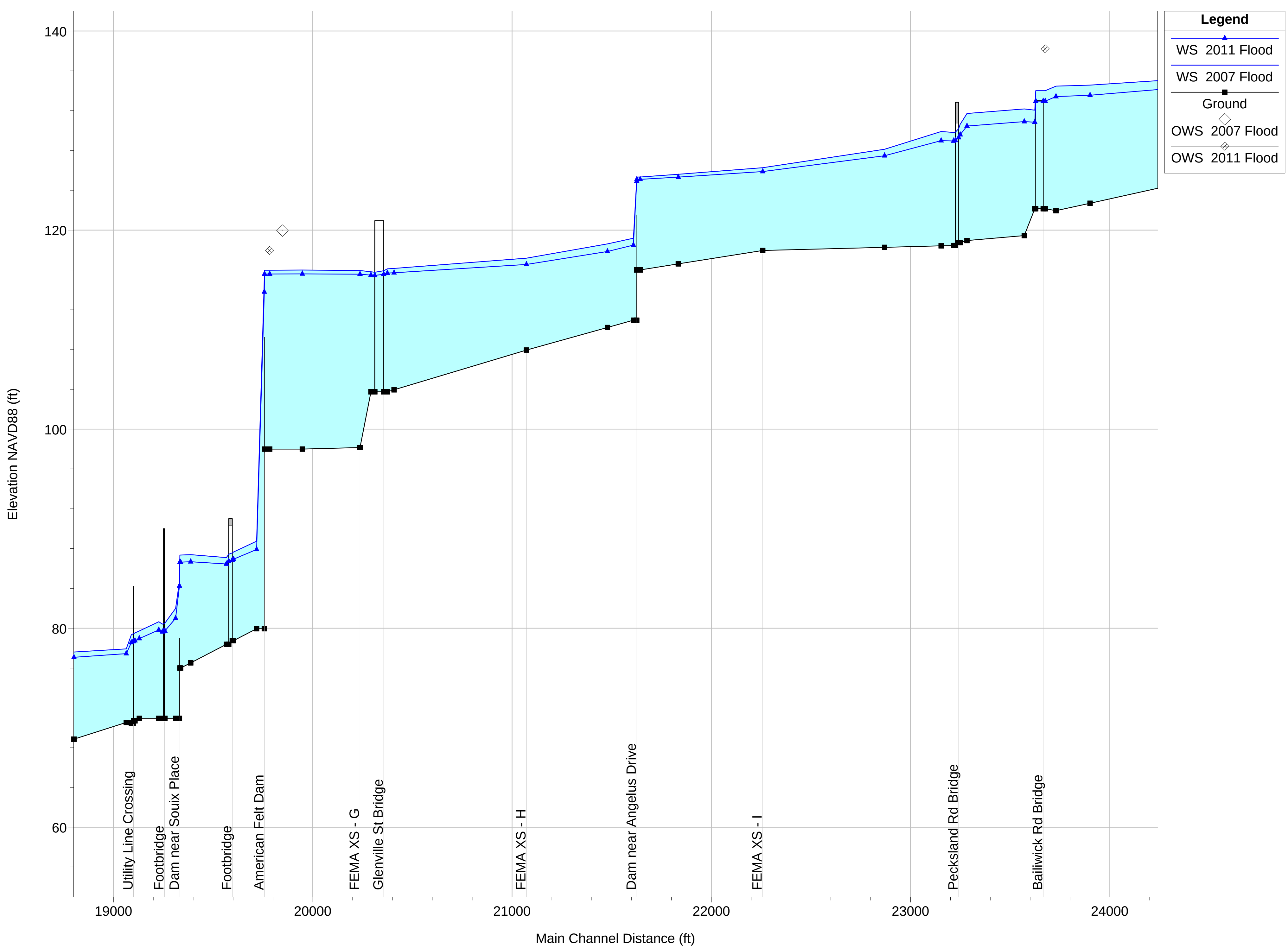


Figure 11- Floods of 2007 and 2011 Profiles with High Water Marks



elevation upstream of the dam (El. 115.98 feet) for the peak discharge simulated at the dam (3,180 cfs). Assuming the geometry from the 2012 survey is correct, including the embankments (El. 114.25 feet), it would take a discharge of 13,000 cfs equivalent to twice the estimated 0.2-percent flood to create such a water surface elevation directly upstream of the dam. The rating curve for the American Felt Co. Dam spillway used in the effective FIS is similar to that in in this study (FEMA, 2010). The effective 0.2-percent flood discharge is 8,780 cfs resulting in an upstream water surface of 118.9 at cross section-G, one foot lower than the observed high-water mark at more than twice the discharge rate. This study assumes that the elevation associated with the Location 5 high water mark does not represent the 2007 peak flood profile.

4.2. Hurricane Irene (August 2011)

In August 2011 Hurricane Irene caused major flooding along the banks of the Byram River. Milone and MacBroom collected high water marks at 2 locations listed in **Table 10**.

Table 10 High Water Marks from Hurricane Irene (2011) on Byram River

Location No.	Greenwich, CT Address	Description of Flood Mark	Reference	Flood Elevation (ft NAVD88)	Computed Elevation (ft NAVD88)	Approximate HEC-RAS Station (ft)	Note
6	10 Glenville Street American Felt Co. Dam	Flood Mark is on top of the dam structure behind brick office building.	See photos of top of dam structure behind 2-story brick office building.	117.95	115.59	19,750	Suspect
7	Bailiwick Road Bridge	Flood Mark is the bottom of the bridge railing on North side of bridge, located at the 3rd post from the stone wall on West side.	See bridge photo- flood mark is where debris is piled up on upstream side of bridge.	138.21	133.42	23,660	Suspect
USGS	Comly Ave Bridge	Depth records at USGS gage on Byram River at Pemberwick, CT (USGS 01212500)	Datum = 25.67 ft NAVD88 from 2012 survey and field measurements of depth gage	32.65	32.68	15,358	Used

Figure 11 shows the simulated 2011 Flood profile along with the two high water marks recorded during the same event.

The high-water mark at Location 6 recorded for the 2011 flood (El. 117.95 feet) at the American Felt Co. Dam is more than 8 feet higher than the 49-foot-long spillway crest (El. 109.25 feet), surveyed in 2012. Assuming the geometry from the survey is accurate, including the embankments (El. 114.25), this corresponds to a discharge of approximately 8,000 cfs, more than twice the estimated discharge from the hydrologic analysis for this event (2,940 cfs), and greater than the estimated 0.2-percent flood discharge at this location. The rating curve for the American Felt Co. Dam spillway used in the effective FIS is similar to that in in this study (FEMA, 2010). While the high-water mark is very close to the effective 1-percent flood elevation just upstream of the dam (El. 117.5 at cross section G), the associated discharge from the FEMA study (5,850 cfs) is more than twice the estimated peak discharge during Hurricane Irene.

Additionally, the exact location of the flood mark for Location 6 is unclear. The surveyor recorded the elevation "on top of the [embankment] structure" approximately 3.7 feet higher than the top of the embankment. If the high-water mark was reported from debris observed on the railing, it is possible that this represents wave action or debris pushed up higher than the actual water surface elevation. This study assumes that the elevation with Location 6 does not represent the 2011 peak flood profile.

The high-water mark at Location 7 recorded for the 2011 flood (El. 138.21) upstream of the Bailiwick Road Bridge is more than 5 feet above the top deck of the Bailiwick Road Bridge where it was taken where "debris [was] piled up on [the] upstream side of bridge". The bridge deck and elevations of the approaching roadway are based on 2-foot contours from the Town of Greenwich and checked against bridge inspection reports and field measurements. This study assumes that the elevation with Location 7 does not represent the 2011 peak flood profile.

The calibration also made use of the depth observations recorded at the USGS gage on the Byram River at Pemberwick (USGS 01212500) at the Comly Ave bridge. The gage was operational at the time of Hurricane Irene. The peak water surface elevation (El. 32.65 ft NAVD88) is shown in **Table 10**.

The datum listed for USGS gage 01212500 in the official USGS record (El. 40 NAVD88) is a very rough estimate based on the general area topography. For the purposes of estimating a high-water mark elevation a more accurate datum was required. The survey collected in fall 2012 for the model cross sections included survey of the Comly Ave bridge. The Town of Greenwich took field measurements of the USGS depth gage relative to features of the bridge on August 15, 2013. From the survey of the top of the west bank wing-wall (El. 41.77 ft NAVD88), the bottom of the USGS depth gage was reported to be El. 25.67 ft NAVD88. Using this refined datum, the peak water surface elevation from Hurricane Irene was calculated.

4.3. Stage Uncertainty Analysis

To account for stage uncertainty in the hydraulic analysis of the project area, an analysis in accordance with an approach described in Chapter 5 of EM 1110-2-1619, to estimate an upper and lower bound for each simulated steady state discharge, was completed. As recommended in Section 5-7 of EM 1110-2-1619, likely combinations of model parameters were selected to represent the associated upper and lower bounds of uncertainty. These likely combinations were used to generate hydraulic profiles for the upper and lower bounds above and below the "expected" hydraulic profile described in **Sections 3 and 5**.

4.3.1. Selected Parameters for Upper and Lower Bounds

Three parameters were selected as having the most uncertainty and impact on the hydraulic profile: (1) Manning's roughness coefficients, (2) contraction and expansion coefficients, and (3) the downstream boundary condition representing the tidal stillwater coinciding with peak discharge. The roughness, contraction, and expansion coefficients were adjusted along the entire model reach to generate an upper and lower bound.

Each roughness coefficient selected to represent the energy losses for a particular reach, as described in **Section 3.5**, an upper and lower bound was selected based on the uncertainty range described in Chow (1959). Table 11 shows the expected Manning's n value for channel and overbanks along with the upper and lower value from Chow (1959).

Table 11 – Expected Value of Manning's n and associated Upper and Lower Bound

Lower	Expected	Upper
0.03	0.035	0.04
0.033	0.04	0.05
0.045	0.05	0.06
0.05	0.07	0.08
0.07	0.1	0.16
0.11	0.15	0.2

Contraction and expansion coefficients at each cross section were provided with a lower and upper bound depending on their proximity to bridge structures as summarized in Table 12. These values are based on Chapter 3 of the HEC-RAS Hydraulic Reference Manual (USACE, 2010).

Table 12 Expected Value of Contraction and Expansion Coefficients and Associated Upper and Lower Bounds

	Lower Bound	Expected	Upper bound
At bridge transitions	K_{exp} = 0.3, K_{con} = 0.1	K_{exp} = 0.5, K_{con} = 0.3	K_{exp} = 0.8, K_{con} = 0.6
Outside of bridge transitions	K_{exp} = 0, K_{con} = 0	K_{exp} = 0.3, K_{con} = 0.1	K_{exp} = 0.3, K_{con} = 0.1

As described in **Section 3.6**, the downstream boundary condition representing the tidal stillwater that coincides with the peak riverine discharge is highly uncertain. There are two factors contributing to this uncertainty: (1) the correlation between peak tidal events and riverine events as described in **Section 3.6.2**, (2) and the impact of sea level rise (SLR) over the project life as described in **Section 6.2**. Table 6 in **Section 3.6**.4 summarizes how these factors were integrated into the median, or "expected", value to represent the coincident downstream boundary condition (el. 6.9 ft. NAVD88). Table 13 describes how likely combinations of these factors were used to generate downstream boundaries representing the upper bound (el. 9.5 ft. NAVD88) and lower bound (el. 4.2 ft. NAVD88)

Table 13 Summary of Components for Upper and Lower Bound Tidal Downstream Boundary Condition

	Expected	**Lower Bound**	**Upper bound**
Event Coincidence (Section 3.6.2)	el. 5.7 ft. NAVD88	el. 3.6 ft. NAV88	el. 8.7 ft. NAV88
	Expected value determined by coincident record.	The lower bound for event coincidence is the Mean High Water (MHW) and corresponds to a complete non-coincidence between a peak riverine discharge event and a peak tidal event, which is a regular occurrence in the coincident record described in Section 3.6.2.	The upper bound for event coincidence is based on the highest observed tidal stillwater coincident with a peak riverine discharge on the Byram River which occurred during Hurricane Irene in 2011.
Sea Level Rise (Section 3.6.3) (Section 6.2)	+1.2 ft.	+0.6 ft.	+3.0 ft
	"Intermediate" scenario for 2072 based on USACE ER 1100-2-8162	"Low" scenario for 2072 based on observed SLR from the recent historic record.	"High" scenario for 2072 represents a high emissions future.
Overall downstream boundary condition	el. 6.9 ft. NAVD88	el. 4.2 ft. NAVD88	el. 9.5 ft. NAVD88
	Overall adjusted expected value described in Section 3.6.4.	The lower bound is the MHW (el. 3.6 ft. NAVD88) adjusted up by the "Low" SLR scenario	The "high" SLR scenario adjustment (+3.0 ft) was applied to the the upper bound in the uncertainty of mean coincidence (el. 8.7 ft. NAVD88) with an additional adjustment (+0.5 ft) to account for compounding factors between SLR and coincidence.

4.3.2. Upper and Lower Bound Flood Profiles

Both the upper and lower bound parameters were input for the existing conditions model described in **Section 3** and the Recommended Plan model described in **Section 3.7**. The 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year peak discharge events were simulated.

Figure 12 shows the upper and lower bound for the 100-year event for the existing condition around the U.S. Route 1 Bridges that are associated with the Recommended Plan. The difference between the upper and lower bound ranges from around 5.4 ft at the Byram River outlet where the profile is dominated by the tidal boundary condition to around 1.5 ft upstream of the U.S. Route 1 bridges. Where the river transitions from total tidal control (RS. 6860) the band between the upper and lower bound is approximately 3.3 ft.

Figure 12 Hydraulic Profile of Existing Condition 100-year with Upper and Lower Bound

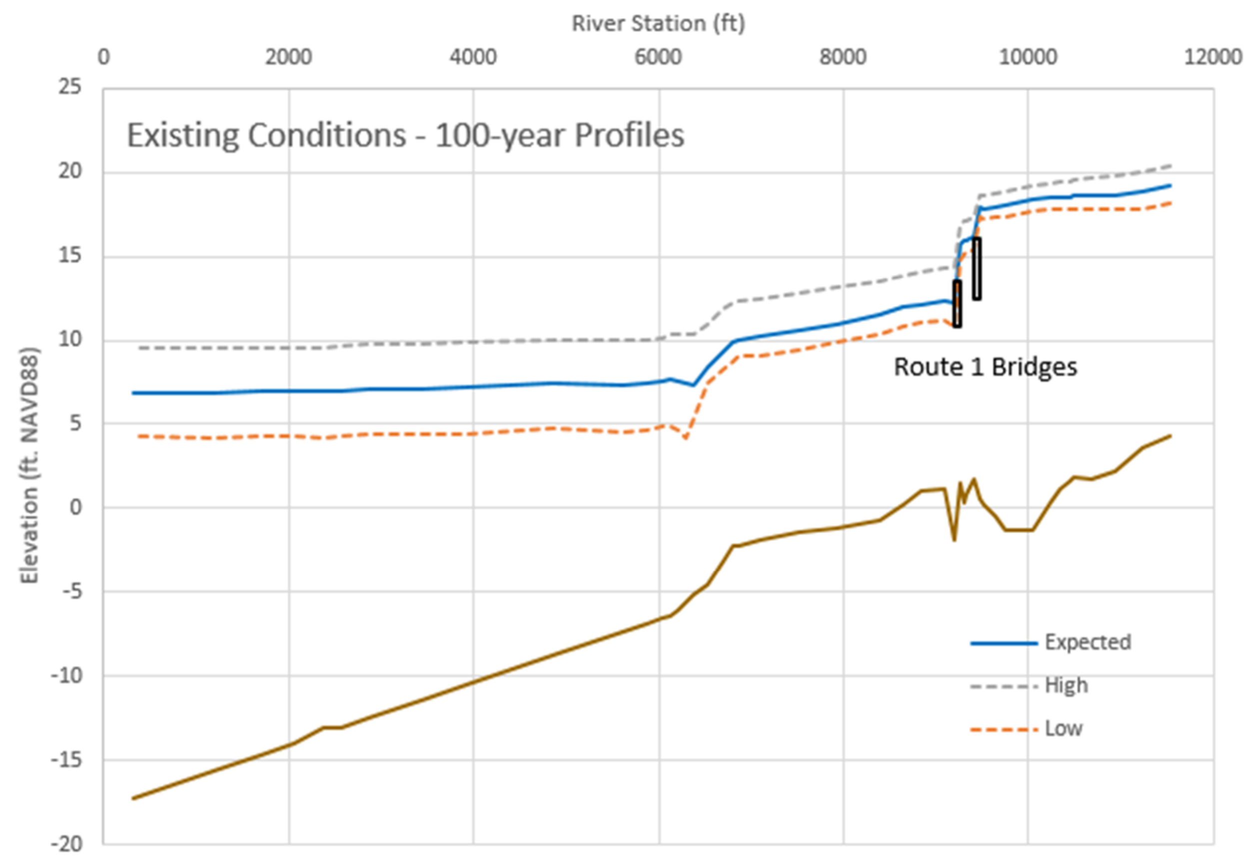
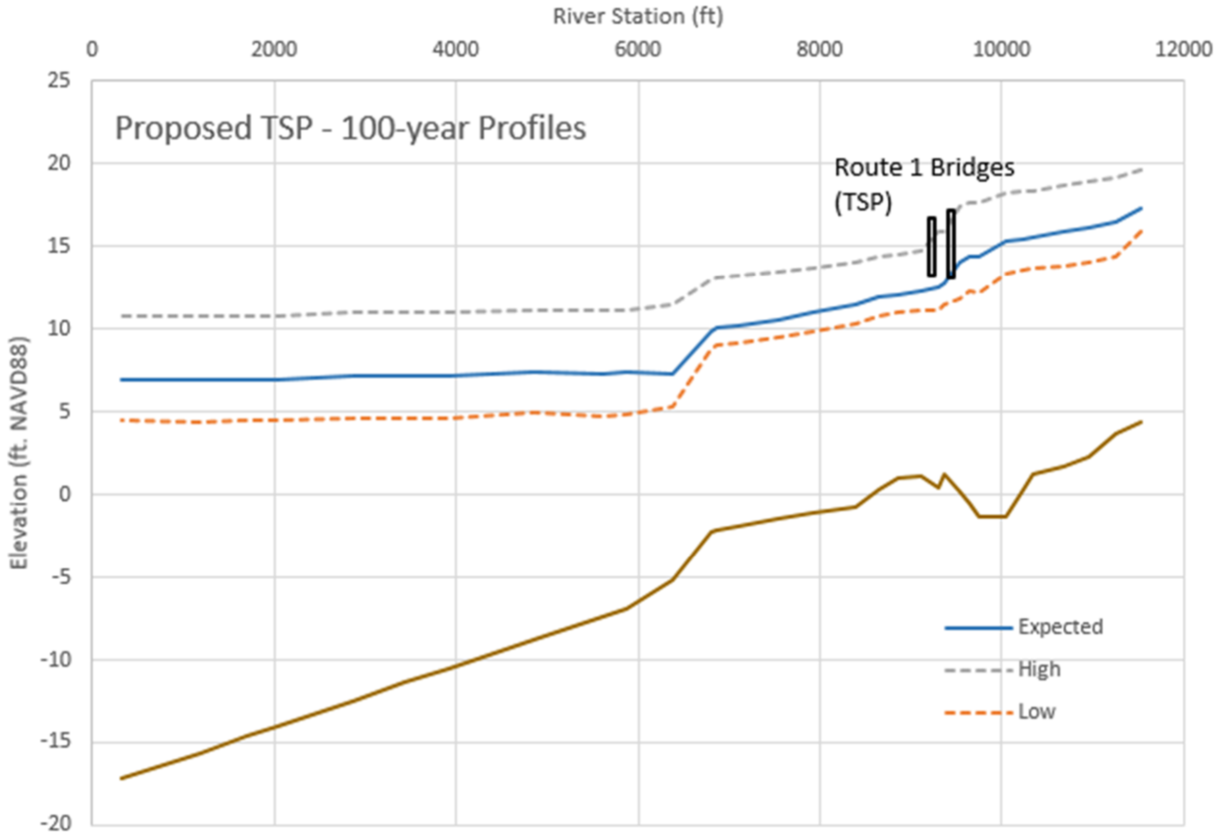


Figure 13 shows the upper and lower bound for the 100-year event for the Recommended Plan. The difference between the upper and lower bound at the Byram River outlet to the U.S. Route 1 bridges is identical to the Existing condition shown in Figure 14. Upstream of the U.S. Route 1 Bridges under the Recommended Plan, the range between the upper and lower band ranges from 3.0 to 5.0 feet. It should be noted that the flood profiles associated with the Recommended Plan are consistently lower for the Recommended Plan, even if the uncertainty range is larger for the 100-year event.

Figure 13 Hydraulic Profile of Recommended Plan Condition 100-year with Upper and Lower Bound



4.3.3. Standard Deviation of Stage Uncertainty

The length-weighted, aggregated range of uncertainty between the upper and lower bounds for the 100-year profile was determined for fifteen economic reaches. Table 14 shows the aggregated range for each reach.

Table 14 Stage Sensitivity Analysis Standard Deviation by Economic Reach

Reach Name	Beginning Station (ft)	Ending Station (ft)	Existing		Recommended Plan	
			Aggregated Range (ft)	Standard Deviation (ft)	Aggregated Range (ft)	Standard Deviation (ft)
1	0	6,609.8	5.36	1.34	5.36	1.34
2	6,609.8	9,230.4	3.16	0.79	3.23	0.81
3	9,230.4	9,476.7	1.81	0.45 [1]	4.34	1.09
31	9,476.7	10,474.1	1.50	0.38 [1]	4.63	1.16
32	10,474.1	13,006.3	1.96	0.49 [1]	4.22	1.06

33	13,006.3	13,544.3	2.85	0.71	4.53	1.13
4	13,544.3	15,401.2	1.59	0.40 [1]	2.04	0.51 [1]
5	15,401.2	15,562.2	1.47	0.37 [1]	1.47	0.37 [1]
6	15,562.2	15,813.1	0.99	0.25 [1]	0.99	0.25 [1]
7	15,813.1	16,211.1	2.34	0.59 [1]	2.34	0.59 [1]
8	16,211.1	19,098.9	0.56	0.14 [1]	0.56	0.14 [1]
9	19,098.9	19,330.47	3.00	0.75	3.00	0.75
10	19,330.47	19,750.64	2.27	0.57 [1]	2.27	0.57 [1]
11	19,750.64	23,650.82	1.13	0.28 [1]	1.12	0.28 [1]
12	23,650.82	28,169.85	1.59	0.40 [1]	1.59	0.40 [1]

[1] The recommended minimum standard deviation of error developed from a model with "fair" reliability in Manning's coefficients is 0.7 feet based on Table 5-2 in USACE EM 1110-2-1619. While the calculated standard deviation of effort is listed in the table, in the economic analysis these reaches used the minimum recommended value (0.7 feet).

Using guidance in USACE EM 1110-2-1619, the standard deviation of this uncertainty can be estimated using SD = E_{mean} /4 where E_{mean} is the mean stage difference between the upper and lower water surface profiles. The standard deviation for each economic reach is shown in Table 14 as well. As defined in Table 5-2 of USACE EM 1110-2-1619 for a model developed from field survey and "fair" reliability in the selection of Manning's coefficients based on limited high water mark data the minimum recommend standard deviation of error for an economic analysis is 0.7 feet. Since the standard deviation of error for several reaches does not exceed this minimum under the existing condition or Recommended Plan, the value was raised to the minimum (0.7 feet) for the economic analysis.

4.4. Summary

The HEC-RAS model is based on the best available LiDAR and field survey. The focus of model calibration was the recent 2007 and 2011 storm events. Where there appear to be differences between the modeled profile and observed high water marks, sensitivity analysis of the profile and inconsistencies in the observed data may shed doubt on the quality of the high-water marks. As referenced in **Section 3.5**, energy loss parameters were adjusted to match the high-water marks determined reliable in the previous sections. The model is well-calibrated (+/- 0.5 feet) to the high-water marks that are consistent with the 2012 structure and cross section survey, as well as with each other.

The uncertainty analysis was performed to determine effects of adjusting input parameters within the HEC-RAS model to be at the low and high end of an acceptable range. The model showed most sensitivity to roughness coefficients and expansion and contraction coefficients. All stages within the vicinity of the U.S. Route 1 proposed bridge for the high were below the low edge of pavement for the 1-percent flood event and thus the profile elevation of the conceptual design is adequate.

The standard deviation of error for each economic reach described in Section 4.3.3 was .

5. DESIGN STORM SIMULATIONS

The refined and calibrated HEC-RAS model was used to simulate design floods. The steady flow regime representing the peak discharge for each design condition is described in **Section 7.0** of Appendix B1 – Hydrology. The downstream boundary conditions are discussed in **Section 3.6** of this appendix.

5.1. Peak Flow Profiles

Figures 14 through **16** show the water surface profiles for the existing conditions 10-, 2-, 1-, and 0.2-percent design floods along the entire extents of the HEC-RAS model of the Byram River from Long Island Sound to the I-684 crossing near the corporate limits of the Town of Greenwich. **Table 15** shows water surface

Table 15 Flood Elevations (Existing Conditions) – Selected Area of Interest Cross Sections

Location	HEC-RAS Station (ft)	Peak Water Surface Elevations (ft NAVD88)					
		50% Flood	10% Flood	4% Flood	2% Flood	1% Flood	0.2% Flood
Long Island Sound (50-percent flood stillwater)	321.6	6.9	6.9	6.9	6.9	6.9	6.9
U/S of Amtrak RR Bridge	6,805.3	7.1	7.7	8.3	9.0	9.9	12.8
D/S of Northbound Rte. 1 Bridge	9,102.9	7.6	9.2	10.4	11.4	12.4	15.4
U/S of Southbound Rte. 1 Bridge	9,526.8	8.1	10.6	14.4	16.1	17.8	20.6
Caroline Pond	11,831.7	11.9	14.5	16.5	18.0	19.5	22.6
U/S of Comely Ave Bridge	15,435.3	31.3	33.6	34.9	35.9	37.0	42.0

5.2. Flood Inundation Maps

Figures 17 through **20** show the peak flood inundation on the Byram River for the existing conditions 10-, 2-, 1-, and 0.2-percent design floods from the Mill Street Bridge to the Bailiwick Road Bridge.

Figure 14. Design Flood Profiles

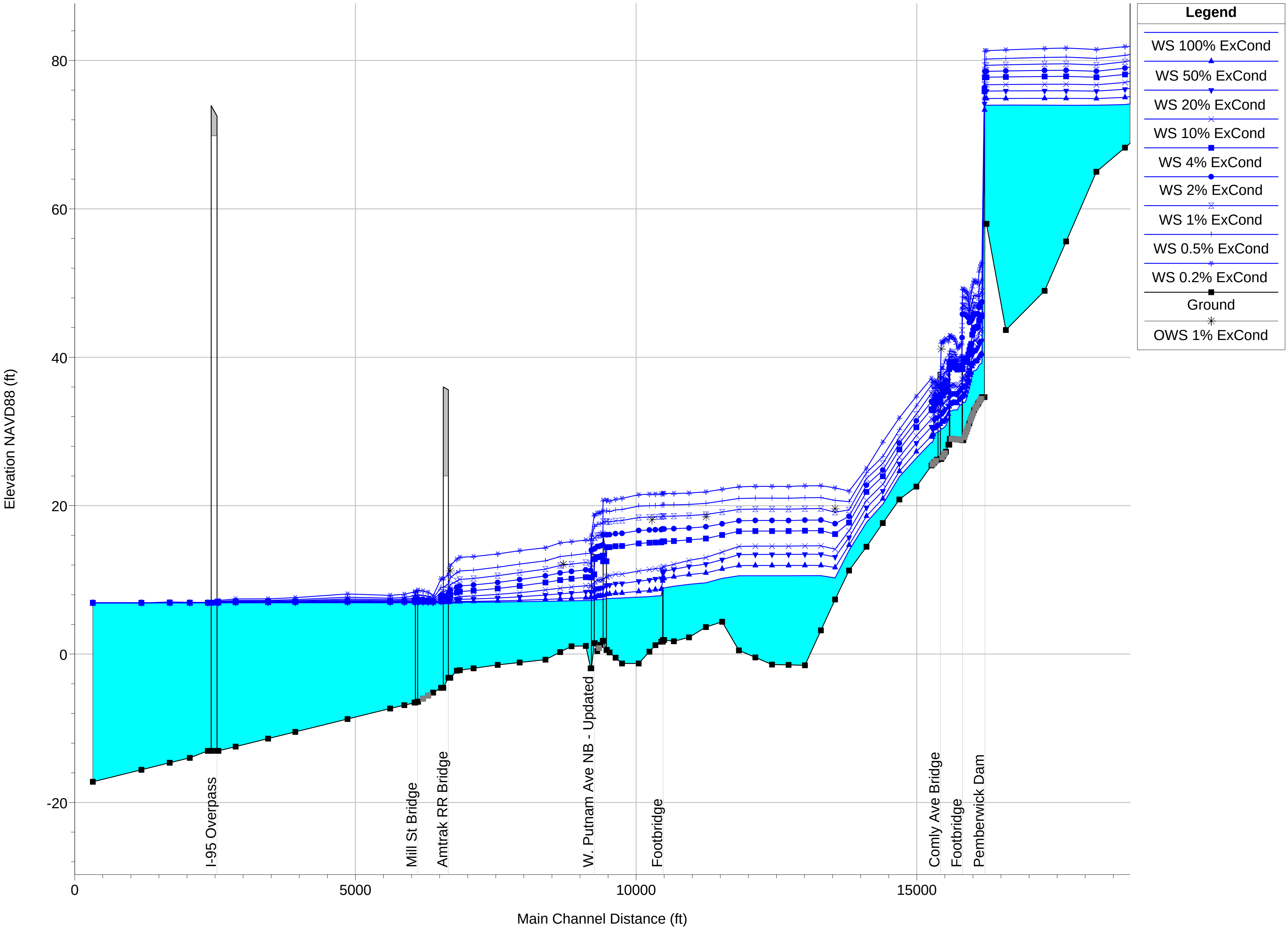
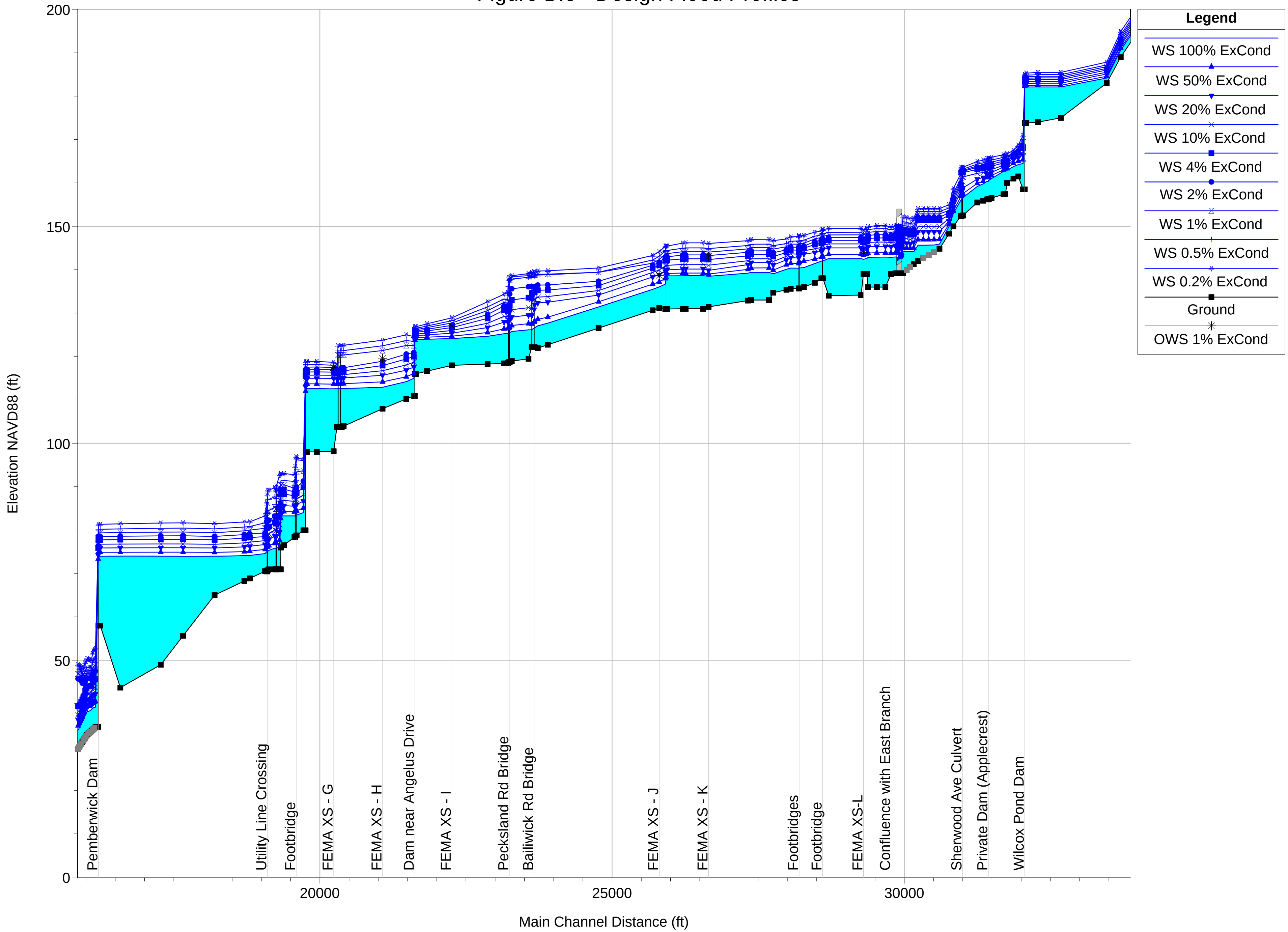


Figure 15

Design Flood Profiles



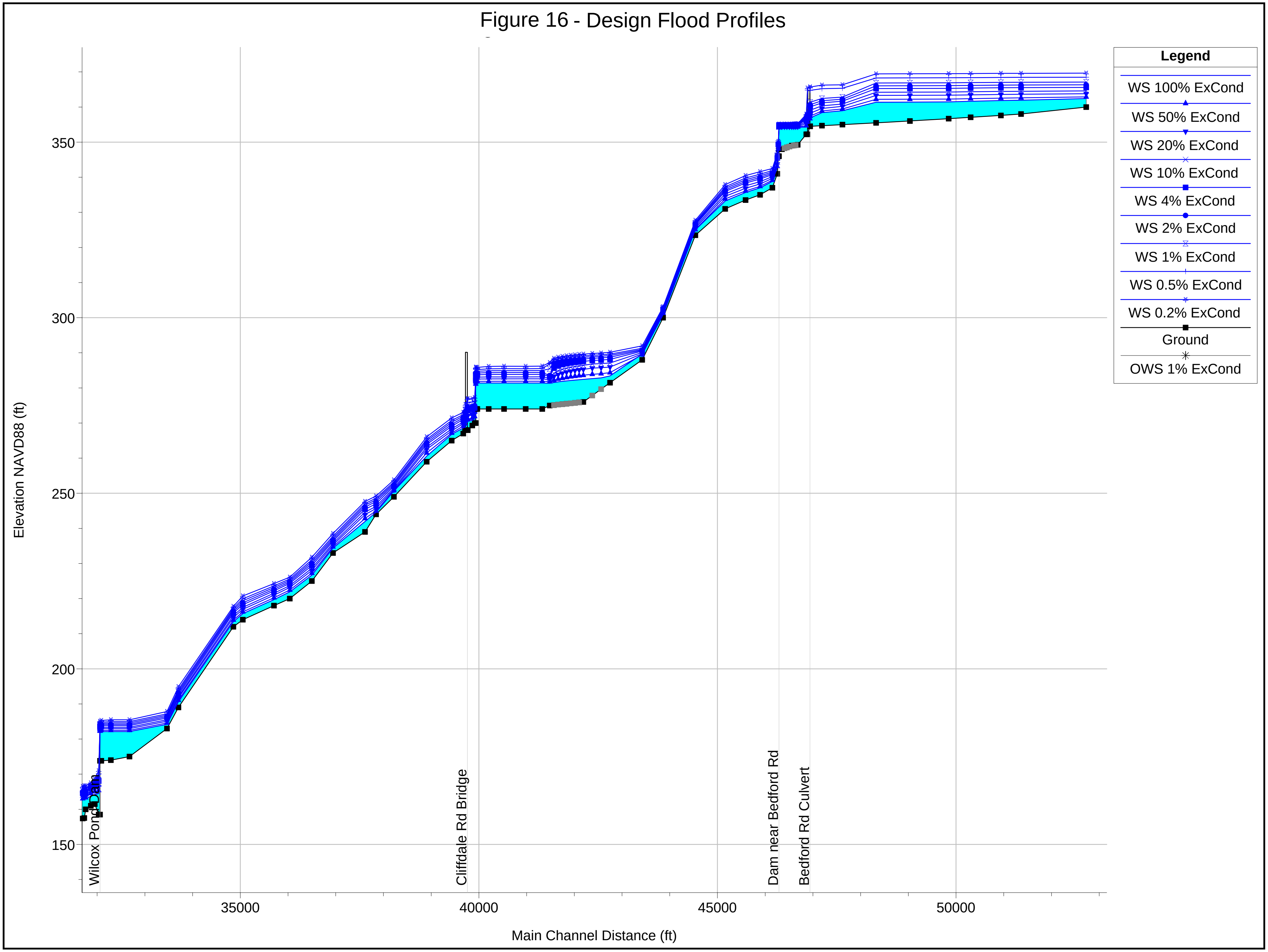


Figure 16 - Design Flood Profiles
Legend
WS 100% ExCond
WS 50% ExCond
WS 20% ExCond
WS 10% ExCond
WS 4% ExCond
WS 2% ExCond
WS 1% ExCond
WS 0.5% ExCond
WS 0.2% ExCond
Ground
OWS 1% ExCond
Elevation NAVD88 (ft)
Main Channel Distance (ft)
350
300
250
200
150
35000
40000
45000
50000
Wilcox Pond Dam
Cliffdale Rd Bridge
Dam near Bedford Rd
Bedford Rd Culvert

Figure 17
Design Flood Inundation
Existing Conditions
Byram River 1 of 4

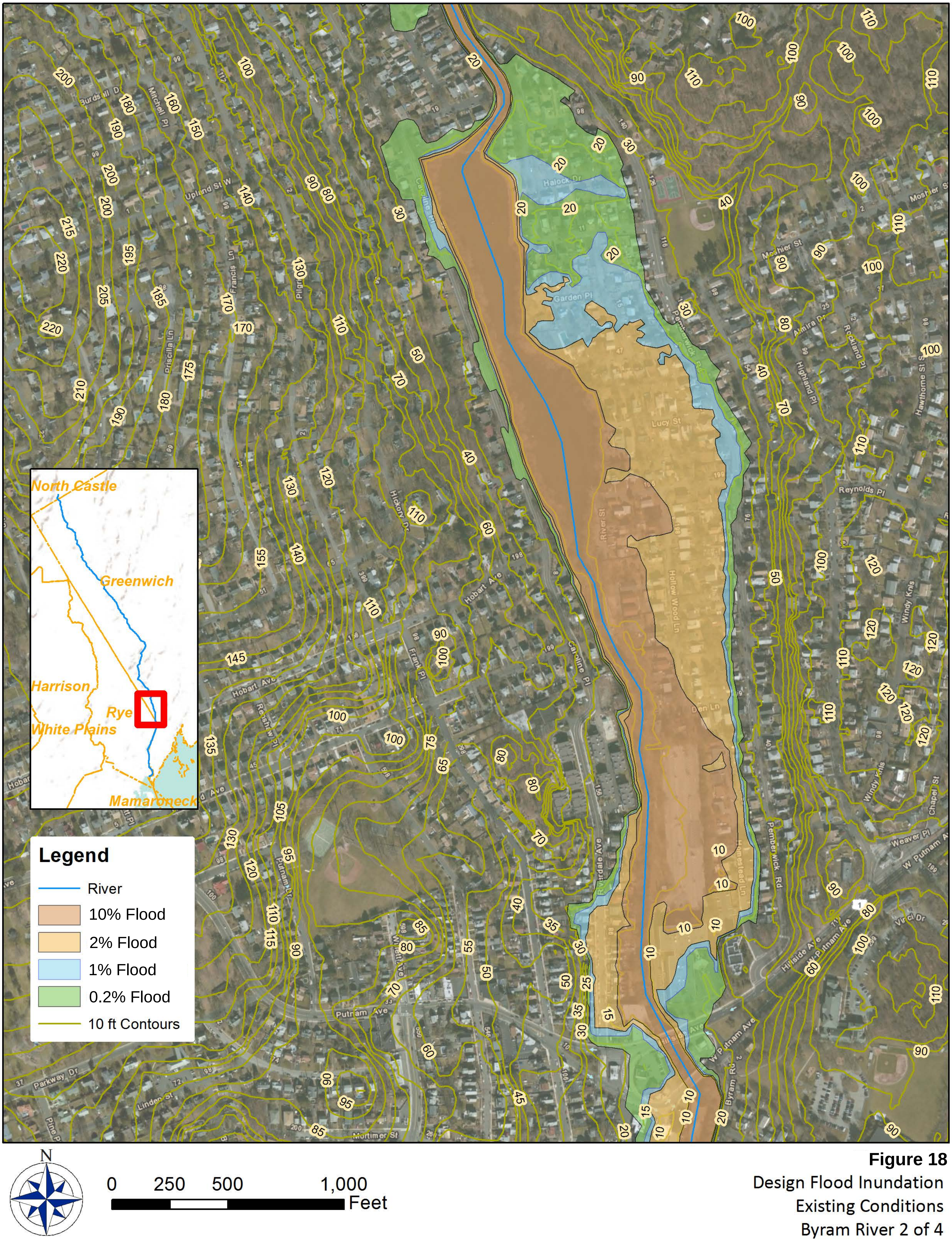


Figure 18
Design Flood Inundation
Existing Conditions
Byram River 2 of 4

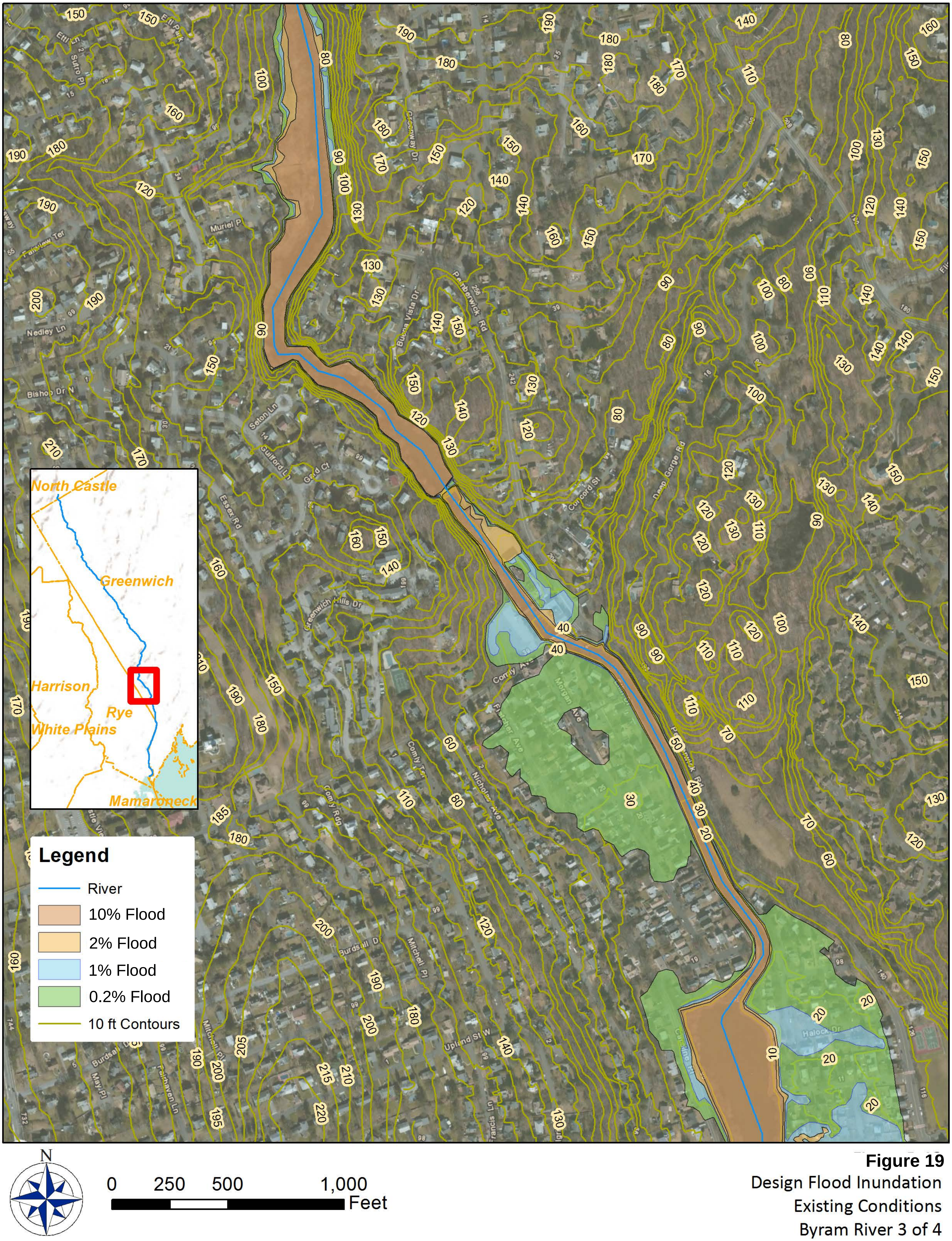


Figure 19
Design Flood Inundation
Existing Conditions
Byram River 3 of 4

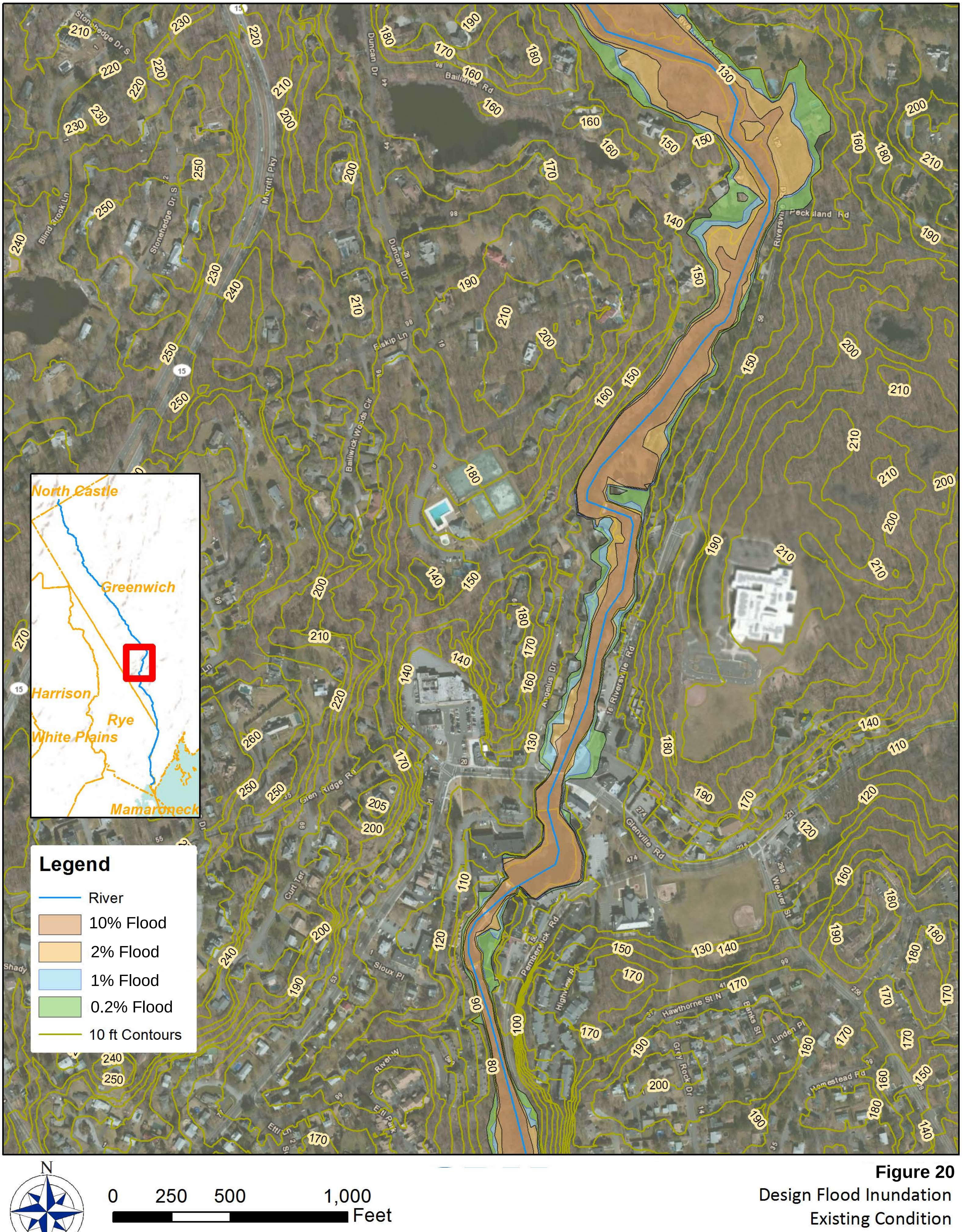


Figure 20
Design Flood Inundation
Existing Condition
Byram River 4 of 4

6. CLIMATE AND SEA LEVEL RISE ANALYSES

A qualitative assessment with respect to climate trends and quantitatively has been performed to establish the effects of sea level rise under a low, medium, and high rate of rise. The effects of these two phenomena will be discussed with respect to the project area and more specifically at the U.S. Route 1 bridge.

6.1. Climate Trend Analysis

The limits of the Byram River fall just within USACE Region 02, New York District. Byram River is located at the border between the states of New York and Connecticut closest to the cities of Greenwich, CT and Port Chester, NY. This area is characterized by relatively cold winters with average lows in the low 20's and warm summers with average highs in the low 80's. Precipitation is generally continuous throughout the year with the area annually receiving about 49 inches. While rainfall is continuous, winter and spring tends to have higher rates, while fall has the lowest. Humidity peaks during summer and into early fall.

In accordance with USACE ECB 2016-25 "Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs and Projects", CDM Smith has performed a qualitative analysis to determine whether any climate trends may exist within the Byram River study area. Trend climates with reviewed with respect to temperature, precipitation, and streamflow. Increases in temperature may stimulate processes such as snowmelt and precipitation trends may affect both the frequency and intensity of rainfall events. A review of streamflow trends is pertinent to determine whether the latter two trends could have a direct correlation on flows and stages experienced within the Byram River.

A review of the "Recent Climate Change and Hydrology Literature Applicable to USACE, Mid-Atlantic Region 02" (USACE, 2015) report indicates generally increasing trends in annual temperature and precipitation but a lack of trends regarding streamflow.

USACE, 2015 discusses 13 studies with respect to temperature trends. Although some researchers agree that the southern portion of the Mid-Atlantic region is within the "warming hole" where cooling trends are found during cooler months rather than warming trends, most data points towards an increase in extreme heat days and annual temperatures. This is also characterized by earlier onset of Spring and fewer extreme cold days with statistically significant ($p < 0.05$) recorded rates of annual temperature increases of around 0.01-0.05 degrees Celsius per year.

18 studies were included within USACE, 2015 with respect to precipitation trending. There is generally good consensus that precipitation and the occurrence of extreme storms has increased in the study region over the past century. This is also joined by a decrease in the occurrence of droughts and increase in soil moisture. Cook et al., 2010 found a statistically significant ($p < 0.05$) trend of increasing precipitation for September through November, with an overall precipitation increase of nearly 1 millimeter per year.

According to three studies discussed within USACE, 2015, no statistically significantly data points towards a trend in streamflow in either direction. Results discussed suggest that the balance between increasing temperatures and increasing precipitation simultaneously may contribute to the lack of streamflow sensitivity to changes in climate. NOAA Technical Report NESDIS 142-1, "Regional Climate Trends and Scenarios for the U.S. National Climate Assessment, Northeast U.S." (NOAA, 2013) analyzed spring center of volume date which represents the date at which half of the total river flow volume over the period of January through May passes a point. Trending of an earlier spring center of volume date could indicate an overall higher baseflow of the stream and/or an increase in the frequency of high flow events. A study done by Hodgkins et al. 2003 analyzed 27 unregulated streams in the New England area, all of which have been experiencing earlier spring center of volume dates. Though, trends on the three rivers analyzed within Connecticut are less substantial than rivers further north in the New England region. This could be due to the more significant presence of snowmelt in the northern rivers.

As outlined in USACE ECB 2016-25, the USACE Climate Assessment Tool was used to review historic trends of streamflow in the Byram River project area. This tool requires use of active gages with at least 30 years of data only. The only gage on Byram River, USGS Gage 01212500 Byram River at Pemberwick, CT, does not meet this criteria as it was installed in 2009. The nearest active stream gage with at least 30 years of data is USGS Gage 01209700 Norwalk River at South Wilton, CT, approximately 16 miles northeast of U.S. Route 1 over Byram River. USGS Gage 01209710 has a period of record of 1962 to present. **Figure 21** shows the time series graph of annual peak streamflow for this gage. As exhibited on the graph, there is a slight declining trend in annual peak streamflow though the p-value indicates this is not statistically significant. Typically, a p-value less than 0.05 demonstrates a statistically significant trend which is far below the p-value of 0.132 exhibited at USGS Gage 01209710.

The U.S. Route 1 proposed bridges will substantially decrease stages within the vicinity of U.S. Route 1. Regardless of potential future climate changes, there will still be a net improvement in implementation of the proposed design. Though, based on the lack of correlation in climate trends climate change is expected to have none or minimal impacts on inland hydrology.

A table of the studies reviewed for the qualitative analysis and their outcomes is provided in **Table 16**.

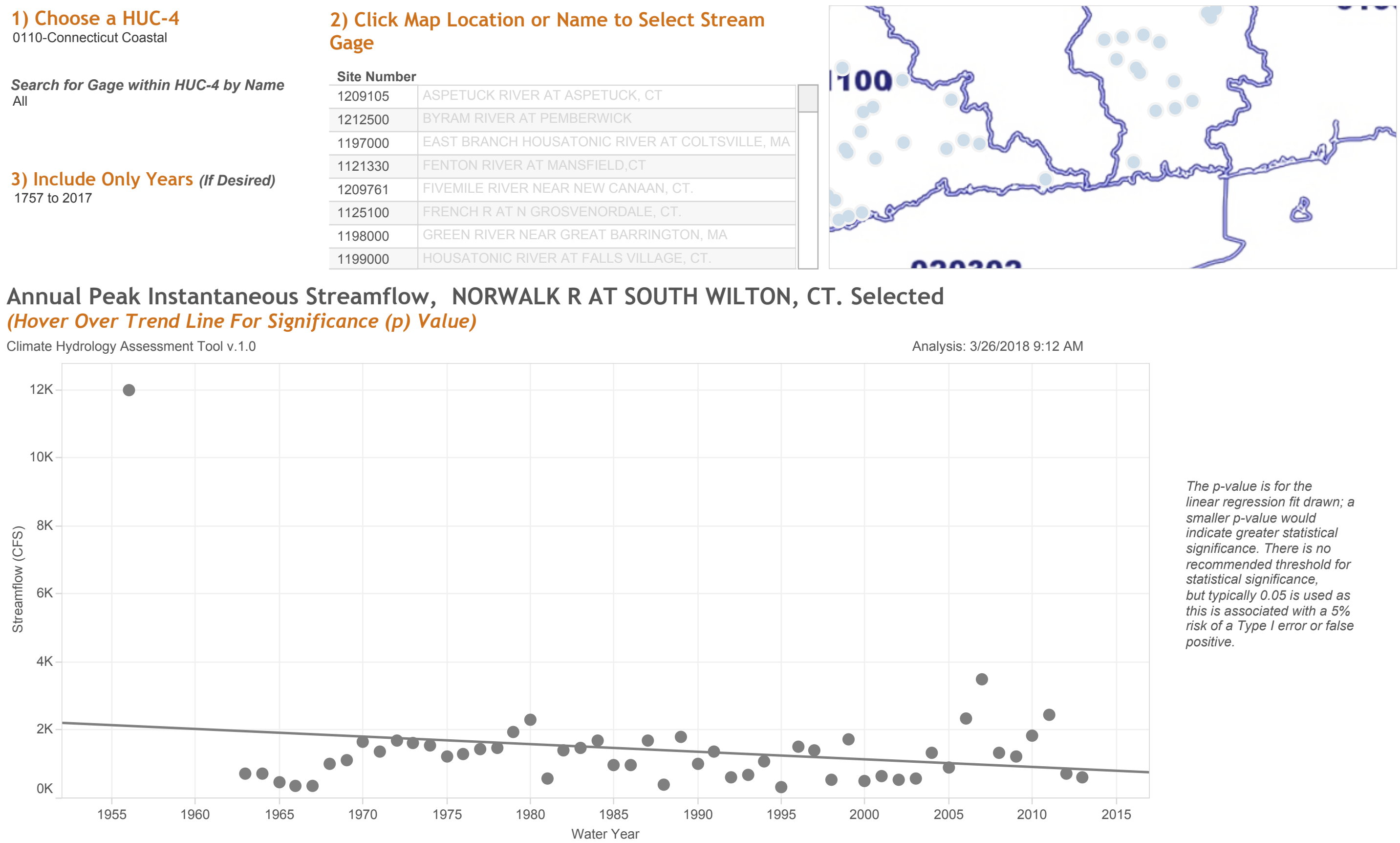


Figure 21 - USACE Climate Assessment Tool: USGS Gage 01209700

Trendline Equation: Q = -22.33*(Water Year) + 45814.1

p = .132, R-squared = 0.045

Table 16 **Summary of Studies Reviewed for Climate Trend Analysis**

Study	Period of Record	Location	Temperature		Precipitation		Streamflow	
			Study Results	Statistical Significance	Study Results	Statistical Significance	Study Results	Statistical Significance
Anandhi et al., 2013	1960-2008	Catskills, southern NY	Increasing trends in annual average daily min temps (up to 0.5C per decade) Corresponding decreasing trends in number of frost days (7 days per decade)	Statistically Significant				
Bonnin et al, 2011	1908-2007	Ohio river and most of Mid-Atlantic Region			Increasing trends in occurrence of large storm events	Statistically Significant $p<0.05$		
Brown et al., 2010	1893-2005	New York and Pennsylvania	Mixed trend for NY vs Penn., Early record = increasing stat significant number summer high heat days Latter record = exhibits primarily decreasing trends/no trend Decrease in number of cold spells in recent records	Statistically Significant $p<0.05$ Not Significant Not Discussed	Increasing trends in number of annual extreme wet days since 1950 Prior to 1950 there are a few decreasing trends	Statistically Significant $p<0.05$		
Burns et al., 2007	1952-2005	Catskills, Southern NY	Statistically Significant increasing trends in mean air temperature for majority of climate stations in Catskills Average rate of increase 0.1° C per decade	Statistically Significant $p<0.05$ Statistically Significant $p<0.05$ Statistically Significant	Seasonal trends for decreasing winter and summer monthly precipitation	Statistically Significant $p<0.05$		
Carter et al., 2014	Early part of 20th century until recent (at time of publishing,2015)	Southeast region (includes southern portions of Mid Atlantic region)	Mild warmings of average annual temperatures in early part 20th century Then a few decades of cooling trends Most recently there have been warming trends	Not Provided				
Chen et a, 2012	1895 - 2007	Virginia			Slightly increasing Significant trend in 12 and 6month SPI averaged over the entire study region = higher precipitation rates and decreased drought risk	Not Provided		
Cook et al, 2014	1000-2005	Virginia			Decrease in drought frequency (droughts per century over last 1000 years) General increase in soil moisture as defined by PDSI over same period	Statistically Significant $p<0.05$		
Cook et al., 2010	1896-2006	Southern NY	Increasing trends in minimum, maximum, and mean annual temperature of 0.01° C to 0.02° C per year Increasing trends in occurrence of extreme heat days Decreasing trends occurrence of extreme cold trends No evidence of the warming hole	Statistically Significant $p<0.05$ Statistically Significant $p<0.05$ Not Significant	Increasing trends for three month autumn (Sept - Nov) precipitation Delta = 1mm/ yr	Statistically Significant $p<0.05$		
Grundstein 2009	1895-2006	Northern portion of Mid Atlantic			Increasing trends in soil moisture Increasing trends in total annual precipitation	Statistically Significant $p<0.05$		
Grundstein and Dowd, 2011	1949-2010	Mid-Atlantic Region	Statistically Significant increasing trends in number of one-day extreme minimum temperatures throughout region, No significant trends for extreme max	Statistically Significant Not Significant				
Hodgins et al. 2003	early 1900s to 2000	Connecticut, New Hampshire, Vermont, Maine					On average over past 30 years, spring center of volume date occurs two weeks earlier	Not Provided
Horton et al., 2014	1895-2011	Northeast Region	2° F increase in average annual temperature	Not Provided	10% increase in average annual precipitation between 1895-2011 Increase in amount of precipitation from extreme heavy events	Not Provided		
Huntington et al., 2009	Unknown	New York State	1° F-3° F increase in average annual temperature	Not Provided				
Kalra et al, 2008	1952-2001	Mid-Atlantic Region					No Statistically Significant trends in either annual or seasonal streamflow	Not Significant
Kunkel et al, 2009	1950-?	Northeast Region			Extreme snowfall not increasing Increasing trends in occurrence of extreme low snowfall years	Statistically Significant		
Maxwell et al, 2012	1200-2000 (May only)	Mid-Atlantic Region			Increased variability with disproportionate number of extreme wet and extreme dry periods for past 100 years compared to previous centuries Statistically Significant increasing trend in May precipitation for 1895-1997	Statistically Significant $p<0.05$		
McRoberts and Nielsen-Gammon, 2011	1895-2009	Mid-Atlantic Region			Linear positive trends in annual precipitation for most of US Mildly increasing trends 2-10% per century	Not Provided		

Table 10 Summary of Studies Reviewed for Climate Trend Analysis

Study	Period of Record	Location	Temperature		Precipitation		Streamflow	
			Study Results	Statistical Significance	Study Results	Statistical Significance	Study Results	Statistical Significance
Meehl et al., 2011	1950-1999	Eastern US	Warming hole Last 50 years there has been a decreasing trend (-1° C) in the warming hole In Dec-Feb WH covers entire mid Atlantic In summer WH covers southern half of mid Atlantic In summer the northern half mid-Atlantic has an increasing trend (+1°C) the last 50 years	Not Provided				
Nguyen and DeGaetano, 2012	1948-2007	Northeast Region			Increasing trends in frequency and magnitude of high precipitation events characterized by closed low precipitation	Statistically Significant $p<0.05$		
Palecki et al., 2005	1972-2002	Mid-Atlantic Region			No trends found for storm magnitude, duration, or intensity for any season	Not Significant		
Patterson et al, 2012	1934-2005	Virginia/Maryland			Only 1 of 15 stations exhibited statistically significant increasing trends Most others exhibited increasing trends but not Statistically Significant	1 station Statistically Significant other stations overall increasing trend	1934-1969 Significant decreasing trends in streamflow for Virginia 1970-2000 small number of stations had Significant decreasing trends Entire POR No Significant trends detected	Statistically Significant $p<0.05$ Not significant
Pryor et al 2009		Mid-Atlantic Region			General increasing total annual precipitation and days of precipitation Mixed results for trends in extreme high precipitation events (90th percentile) More stations exhibit significant decreasing trend for intensity	Statistically Significant $p<0.1$		
Schwartz et al., 2013	POR through 2010	Mid-Atlantic Region	Spring onsets a few days earlier in 2001-2010 compared to 1951-1960 (baseline reference decade) Warmer winter and spring	Not Provided				
Small et all., 2006	1948-1997	Mid-Atlantic Region			Increasing trends in fall precipitation for some regional locations (none for others) No trends for total annual precipitation	Statistically Significant $p<0.05$ for some regions Not Significant for others	No Statistically Significant trends in annual flow for any of multiple stations in mid Atlantic , Small number had Statistically Significant for increasing trends in fall low flow	Not Significant
Wang and Zhang 2008		Mid-Atlantic Region			Significant changes for period 1977-1999 compared to 1949 to 1976 25-100% frequency increase	Not Provided		
Wang et al., 2009	1950-2000	Mid-Atlantic Region	Winter, Spring, Summer: Warming (up to 1°C), Autumn: Cooling (<1°C)	Not Provided	Positive trends precipitation for summer, fall, and spring Negative trends for winter precipitation	Not Provided		
Warrach et al., 2006	20th century	Single Station in Southern NY	Differences in trend rates between first and second half of 20th century 0.01° C increase over entire 20th cent per year; Rate of increase in 1st half much higher than 2nd half	Statistically Significant $p<0.05$				
Westby et al., 2013	1949-2011	Mid-Atlantic Region	General winter cooling trend	Not Significant				
Xu et al 2013	1950-2000	Mid-Atlantic Region					No Statistically Significant trends in either annual streamflow or baseflow for any Mid-Atlantic stations	Not Significant

Legend

General consensus of increasing trend
Decreasing Trends
No observed trends
Split trends

6.2. Sea Level Rise Analysis

In accordance with USACE ER 1100-2-8162, "Incorporating Sea Level Change in Civil Works Programs", CDM Smith has appended the HEC-RAS model to include adjustments to the downstream tailwater condition of the Byram River outlet under three rates of sea level change (SLC). The three rates of sea level change are referred to as low, intermediate, and high scenarios.

The historic rate of SLR represents the low rate and does not account for future acceleration of SLC. The intermediate rate of SLC is estimated using the modified NRC Curve I and is corrected for the local rate of vertical land movement. The high rate of SLR is estimated using the modified NRC Curve III and is also corrected for the local rate of vertical land movement. Locally experienced SLC is higher in areas where vertical land movement is downward, or sinking. These scenarios were analyzed within the USACE Sea Level Change Calculator, an online tool which provides a net change in sea level based on input of a project time, SLC rate, and NOAA gage.

The economic period of analysis for the evaluation of alternatives is 50 years, from the completion of construction in 2023 to 2072. The nearest gage to the Byram River outlet is at Kings Point, New York and was selected for input in the sea level change calculator. The SLC Rate used was the regionally corrected one, with an estimated rise of 0.00778ft/yr.

Results of the SLC indicated an expected rise of 0.62 feet, 1.19 feet, and 3.00 feet for the low, intermediate, and high scenario, respectively, by 2072. As discussed in **Section 3.6**, the intermediate scenario was incorporated into the expected tidal boundary condition. As described in **Section 4.3**, the low and high scenarios were incorporated into the lower and upper bound respectively for the stage uncertainty analysis. The expected peak coincident downstream tidal boundary condition is el. 5.7 ft. NAVD88 as described in **Section 3.6.2**. The boundary condition associated with low SLR for the project life is el. 6.3 ft. NAVD88. The intermediate is el. 6.9 ft. NAVD88. The high is el. 8.7 ft. NAVD88.

To understand the impact of the range of possible SLR futures, the boundary condition of the Byram River outlet was increased by these values within HEC-RAS to simulate potential future conditions of the Byram River. The output was specifically reviewed within the vicinity of the U.S. Route 1 bridge replacements to determine whether sea level rise will impact existing flood behavior at U.S. Route 1 over the lifetime of the proposed north and south bridge.

The analysis indicates the U.S. Route 1 bridges are close enough to the Byram River outlet that tailwater fluctuations will influence water elevations. Results have been tabulated in **Table 17** with respect to the 1-percent flood stages. **Figure 22** displays the below information in a flood profile. All stage comparisons are with respect to vertical datum NAVD88 and are based on the existing condition flows.

Table 17 Stages at U.S. Route 1 Bridge due to Sea Level Rise

Location	HEC-RAS Cross section	1-Percent Flood Stage (ft), U.S. Route 1 Bridge Replacement		
		Low Scenario	Intermediate Scenario	High Scenario
Upstream of North bridge	9633.9	14.37	14.37	15.75
Upstream of North bridge	9526.8	14.06	14.06	15.54
Immediately upstream of North Bridge	9476.7	14.04	14.04	15.53
North Bridge	9444.3	*	*	*
In between bridges	9405.8	12.63	12.69	14.29
In between bridges	9367.1	12.69	12.75	14.37
In between bridges	9336.19	12.44	12.5	14.26
In between bridges	9305.3	12.43	12.5	14.24
In between bridges	9263.3	12.28	12.34	14.1
South Bridge	9230.4	*	*	*
Immediately downstream of South Bridge	9190.9	12.28	12.35	12.73
Downstream of South bridge	9102.9	12.28	12.35	12.75
*Indicates stage at or above low chord but no bridge overtopping				

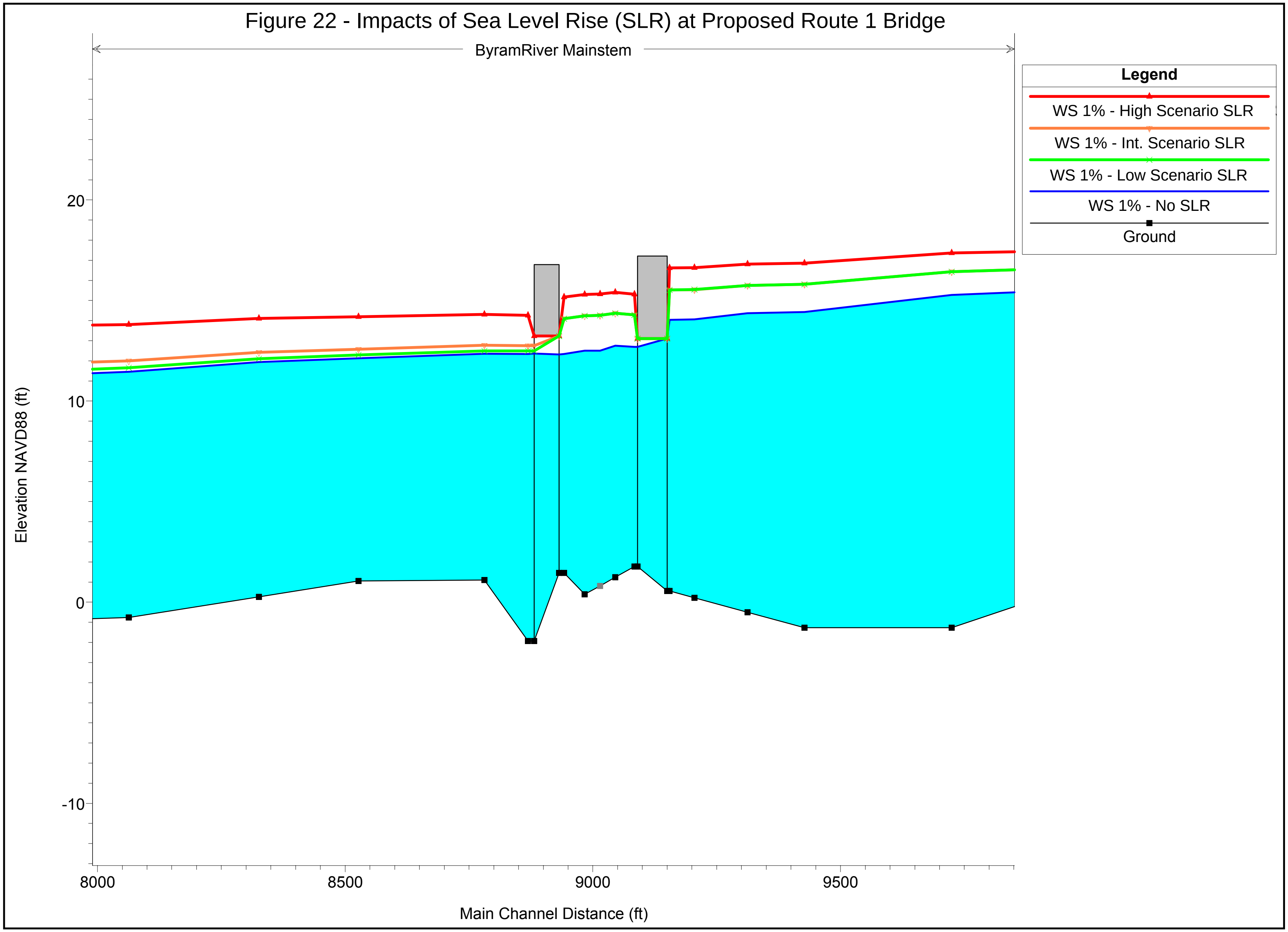


Figure 22 - Impacts of Sea Level Rise (SLR) at Proposed Route 1 Bridge
ByramRiver Mainstem
Legend
WS 1% - High Scenario SLR
WS 1% - Int. Scenario SLR
WS 1% - Low Scenario SLR
WS 1% - No SLR
Ground
Elevation NAVD88 (ft)
20
10
0
-10
8000
8500
9000
9500
Main Channel Distance (ft)

7. SEDIMENT TRANSPORT ANALYSIS

The Byram River sediment transport analysis focused on the review of available watershed information plus review of HEC-RAS simulation results. Watershed information included a characterization of the watershed conveyance network and limited sampling data. The HEC-RAS analysis included review of velocities in the vicinity of the bridge improvements, with and without the improvements, to assess the potential for impacts.

7.1. Byram River Watershed Conveyance Network

According to the *Byram River Watershed Management Plan* (Steven Danzer Ph.D & Associates LLC, September 2011), there are over 40 dams in the Byram River watershed conveyance network, creating impoundments of various sizes. As a result, there are a number of locations at which stream velocity is slow and sediment accumulation will occur. As a result, the sediment loads to the location of the proposed bridge improvements and the downstream portion of the river are limited under current conditions.

Limited sediment monitoring data are presented in the *Final Report on Byram River Watershed Model Development* (Earth and Environmental Engineering Department, Columbia University, NY, NY, January 2012) for the Byram River. Sampling station BR8 in this report is located near the Putnam Avenue/U.S. Route 1 bridges that are proposed for modification. During 6 sampling events, which include 3 dry weather sampling events and three events taken with prior 24-hour rainfall of 0.5 to 2.9 inches, the measured settleable solids ranged from <0.1 to 0.4 ml/l.

It is not obvious how these measurements in units of ml/l would relate to a suspended solids concentration in mg/l, *Wastewater Engineering: Treatment/Disposal/Reuse* (Metcalf and Eddy, 1979) tabulates typical values of settelable solids in ml/l and total suspended solids in mg/l for wastewater, and the ratio is approximately 20. Therefore, a settleable solids range of <0.1 to 0.4 ml/l is expected to correspond to a TSS range of < 2 to 8 mg/l. These values confirm that sediment transport in the watershed at the location of the bridge improvements is limited.

Consequently, the evaluation of sediment transport will focus on the expected change in velocity in the vicinity of the proposed bridge improvements.

7.2. HEC-RAS Model Results for Proposed Bridge Improvements

To evaluate the potential impacts of the proposed bridge improvements on sediment transport, the HEC-RAS model results for current conditions with and without the bridge improvements were evaluated. The model impacts include the following considerations:

- Model was evaluated for the 100-percent and 50-percent design storms. Typically, potential erosion impacts include the evaluations of less extreme design storms (e.g., the 100-percent and 50-percent storms).

- Model was evaluated for the mean high water (MHW) and mean low water (MLW) tailwater conditions. This represents a more typical range of tailwater conditions, in contrast to the high tailwater condition that is the basis for evaluating potential flooding impacts.

The impacts on velocities were evaluated upstream of the bridges, between the bridges, and downstream of the bridges. The upstream evaluation considered model results from the upstream bridge to the next upstream crossing (footbridge at station 10474.1), and the downstream evaluation considered model results from the downstream bridge to the next downstream crossing (Amtrak Railroad Bridge at station 6609.8). Beyond that downstream point, any impacts of the bridge improvements will be minimal.

The model results are presented in **Table 18**. In general, the results show that the proposed improvements will results in a slight increase in velocity upstream of the improved bridges and between the improved bridges, with no impact downstream of the improved bridges. The small velocity increases are unlikely to have an adverse impact on sediment delivery to the tidal area downstream of the improved bridges.

Table 18 HEC-RAS Velocity Results with and without Bridge Improvements.

Location	Design Storm	Current Bridge - Velocity (ft/s)			Proposed Bridge - Velocity (ft/s)		
		Base	MLW	MHW	Base	MLW	MHW
Upstream of bridges	1-year	2.57	3.22	3.20	2.61	3.27	3.24
Between bridges		2.54	4.00	3.88	2.51	4.03	3.90
Downstream of bridges		1.28	3.18	2.32	1.28	3.18	2.32
Upstream of bridges	2-year	3.50	4.01	3.98	3.58	4.09	4.06
Between bridges		3.57	4.76	4.66	3.56	4.85	4.72
Downstream of bridges		1.90	3.55	2.99	1.90	3.55	2.99

Base Tailwater = 6.90 feet NAVD; MLW Tailwater = -3.89 feet NAVD; MHW Tailwater = 3.40 feet NAVD

7.3. Summary of Results

Review of existing watershed information indicates that there is limited sediment delivery to the proposed bridge improvement location, because of multiple dams in the watershed conveyance network. Comparison of modeled HEC-RAS velocities with and without the bridge improvements indicate little or no velocity increase would be expected. Consequently, the bridge improvements are expected to have no potential impacts to sediment delivery to the Byram River tidal area.

8. SUMMARY

The hydraulic simulation presented in this Appendix represents the best available information to date on the Byram River, with detailed cross sections of the bathymetry and structures in the area of interest, and a high-water mark calibration to a gaged event (Hurricane Irene, 2011) and the estimated discharge associated with a recent 20-year event (April 2007).

In the 1977 Feasibility Report for Flood Control (USACE, 1977), USACE estimated that the 1-percent storm peak design flow in the vicinity of the Caroline Pond was approximately 6,920 cfs resulting in a flood elevation of 21.7 ft NAVD88. By comparison, in the updated analysis for the same reach the 1-percent storm design flow of 6,690 cfs (presented in **Appendix B1**) resulting in a flood elevation of 19.5 ft NAVD88. The updated 2013 analysis demonstrates design conditions which are slightly lower than those presented in the 1977 Feasibility Report. In this way the updated analysis shows that the improvements proposed in the 1977 Feasibility Report may be slightly conservative for the return interval of interest.

Improvements to replace the U.S. Route 1 bridges for a design with a higher profile show significant benefits at U.S. Route 1. The overtopping frequency of U.S. Route 1 is anticipated to decrease from a 2-percent event to almost a 0.2-percent event with implementation of the conceptual design. With respect to climate change, a review of temperature, precipitation, and streamflow data indicate climate change will likely have none or minimal impacts on inland hydrology for this project.

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