Draft Appendix C

Hydraulics

Rahway River Basin, New Jersey
Coastal Storm Risk Management Study

May 2017

New Jersey Department of Environmental Protection

**U.S. Army Corps of Engineers
New York District**

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1.0 INTRODUCTION

1.1 Area of Study

The Rahway River Basin is located in northeastern New Jersey. It lies within the metropolitan area of New York City. The basin is approximately 83.3 square miles (53,300 acres) in area. A feasibility study was recently conducted in September 2016 for the "fluvial," or inland, portion of the basin. This feasibility study focuses on the "tidal," or coastal, portion of the basin and includes the New Jersey municipalities of Rahway, Carteret, and Linden. A map of the Rahway River Basin, its municipalities, and the fluvial and tidal study areas is shown in Figure 1. The area of study specific to this report, "Rahway Tidal," is shown in Figure 2.

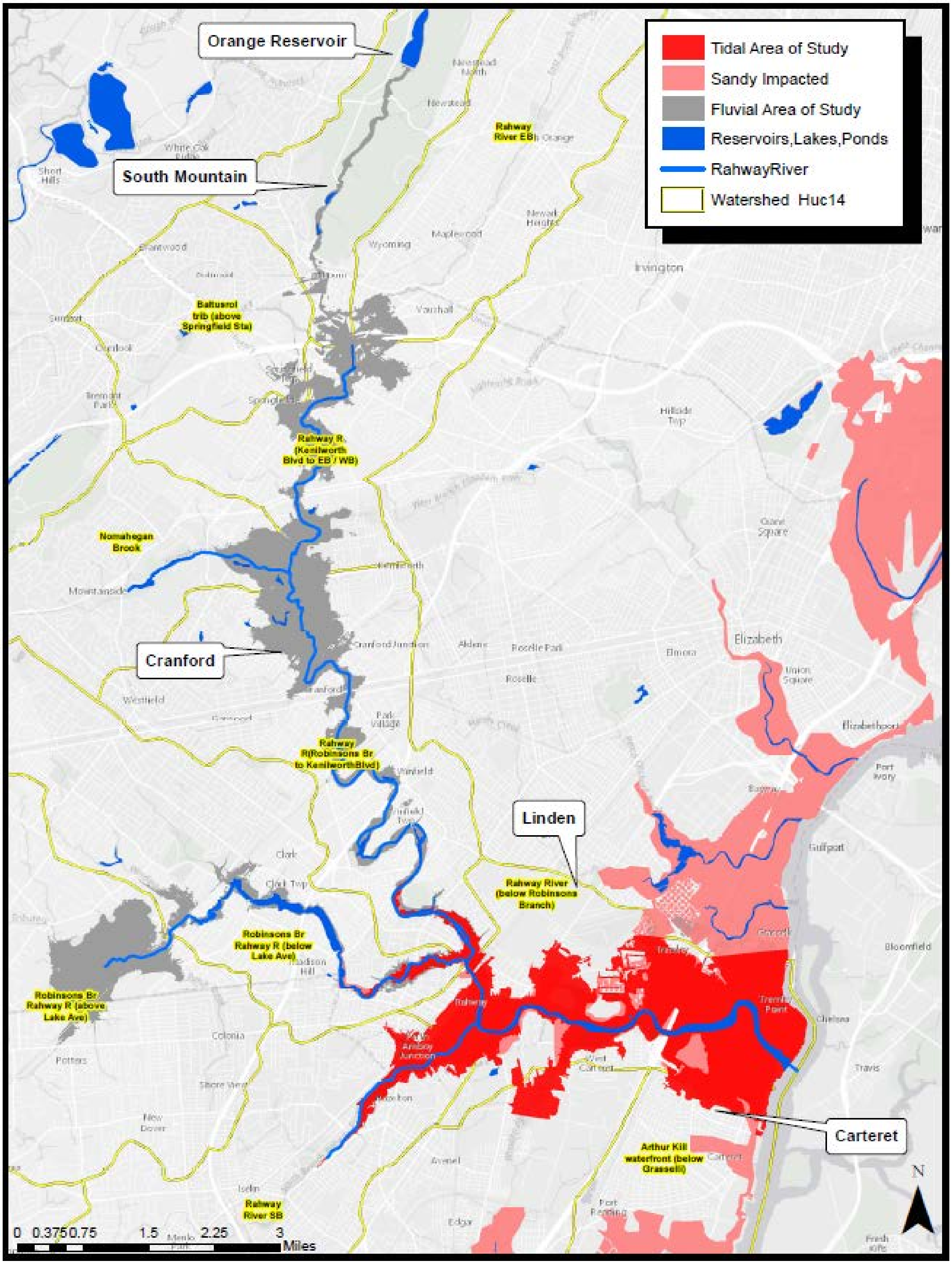


Figure 1: Rahway River Watershed.

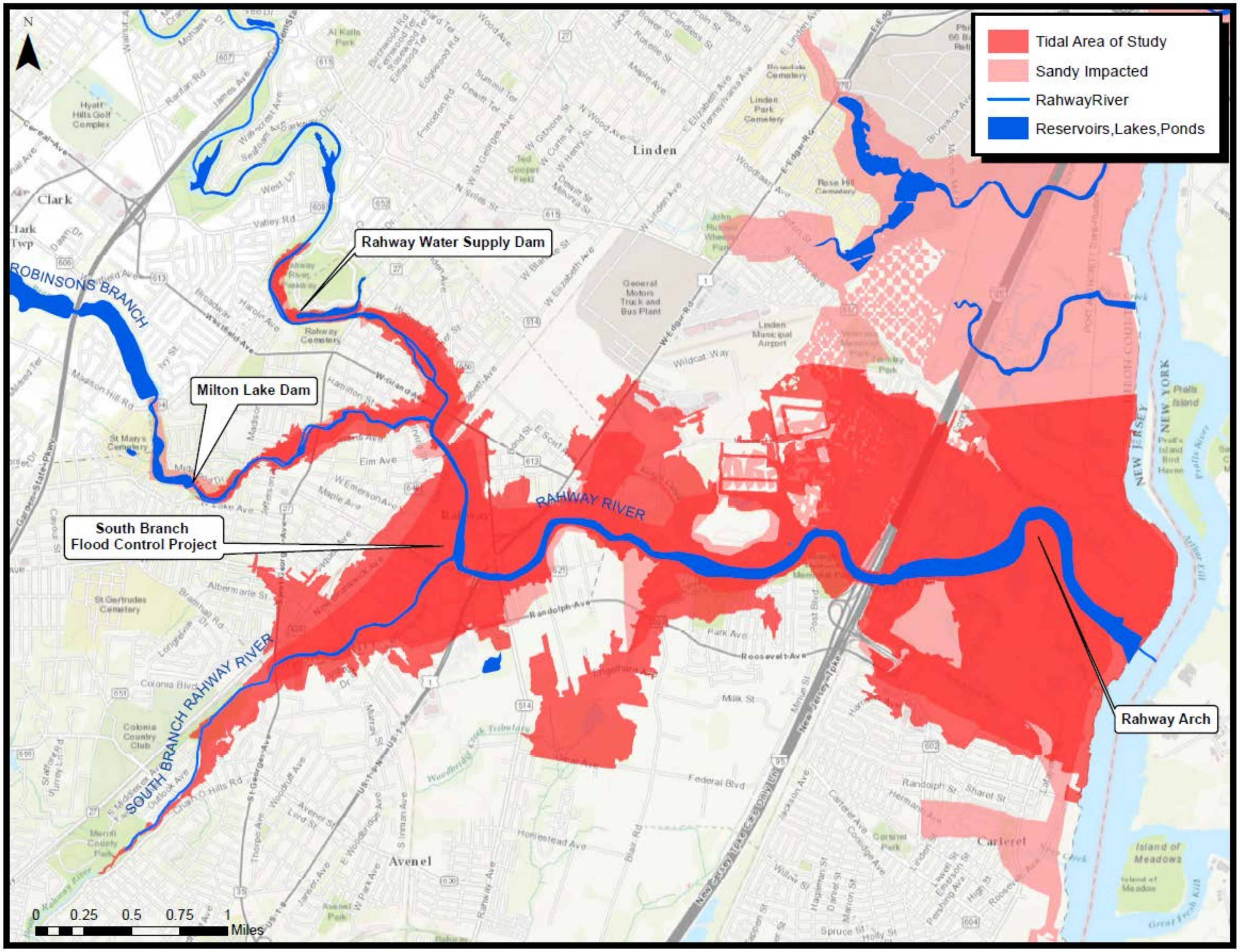


Figure 2: Rahway River Tidal Area of Study.

1.2 Present Flooding Problems

Periodic storms have caused severe tidal flooding along the Rahway River. There are three main areas with high flood risk: the mouth of the Rahway River at the confluence with the Arthur Kill, South Branch Rahway River, and the confluence of South Branch and Rahway Rivers. Flooding at the confluence of South Branch Rahway River and Mainstem Rahway River spans from the New Jersey Transit railroad bridge in Rahway, south towards the Rahway Yacht Club. This flooding is caused by a “U” shaped turn, six bridge constrictions within a mile of each other, and low channel capacity. The bridge constrictions and coastal surges at the confluence cause backwater along the South Branch of the Rahway River up to the St. Georges Avenue Bridge. Flooding at the confluence of the Arthur Kill and the Rahway River in Linden and Carteret is caused by low ground elevations and low roadway elevations at the New Jersey Turnpike. Most of the flooding at the mouth of the river is caused by low wetland elevations and severely affects the tank farms at the Tremley Point industrial area in Linden.

1.3 Objective

The objective of this study is to identify a feasible means of managing the risk of flooding in the most affected areas of the Rahway River in the most cost effective manner in an environmentally and culturally acceptable way. The flood risk management concepts considered in this study are: channel modification, bridge replacement, dams, levees, tide gates, pump stations, and non-structural plans.

2.0 RAHWAY RIVER DESCRIPTION

2.1 General

The head waters of the Rahway River start at the East and West Branch of the Rahway River. The Branches merge into the main stem Rahway River at Springfield and Union Township and flows south for approximately 2.5 miles from I-78 to Route 22. From this point it flows directly into Cranford, Winfield, and Clark Township, meeting with the Robinson’s Branch in Rahway. Robinson’s Branch runs through Clark and Rahway, and is impounded at Middlesex Reservoir and Milton Lake. Approximately half a mile downstream of the confluence of Robinson Branch with the Rahway River is the confluence with South Branch. South Branch has head waters in Edison at Roosevelt Park and runs through Iselin and Colonia to meet the main stem. Approximately 4.5 miles from the confluence of South Branch and the Rahway River is the confluence with the Arthur Kill in Carteret and Linden. The extents of this tidal study are from Rahway River Park south towards the Arthur Kill, Milton Lake Dam in the Robinson's Branch to the confluence at the Rahway River, and the entirety of the South Branch.

The channel banks in the tidal area are relatively low and vary from 2 to 6 feet in height in the area of study. Closer to the Arthur Kill with much of the overbanks being wetlands, channel banks are very low-lying. The channel bottom slope in that vicinity is also very mild at 2.0 ft/mile. South Branch has channel banks about 6 ft in height with a channel bottom slope of 3 ft/mile. Robinsons Branch has banks about 4 ft in height with a channel bottom slope of 10 ft/mile. The width of the channel by Arthur Kill is approximately 450 ft, tapering to 200 ft at the confluence with South Branch. South Branch has a channel width approximately 100 ft at the confluence tapering to 40 ft at the upstream end of the affected area. Robinson's Branch has an average channel width of approximately 40 ft in the flood-prone area.

The tidally influenced area of the Rahway River Basin is highly populated with dense suburban communities at South Branch and Robinson's Branch. Further downstream by the NJ Turnpike and Arthur Kill is industrial with many warehouses and tank farms. Much of the downstream area is believed to have HTRW due to its deep history in the chemical and oil refinery industries. Areas adjacent to the river are mostly protected by the non-federal sponsor (NJDEP) and the Green and Blue Acres Program.

There is an existing Corps of Engineers Flood Risk Management Project with levees along the left bank of the South Branch and along the right bank of the Rahway River at the confluence of these two rivers. The top of levee (TOL) elevation of this Corps of Engineers system is about 12.6 ft. NAVD '88 which is slightly above the present 0.01 annual exceedance probability (100-year) tidal event. This system is further described in the sections that follow.

2.2 Flood Prone Areas

The downstream reach of the Rahway River, by the Arthur Kill, starts producing minimal damages to the tank farms at the 0.99 annual exceedance probability (AEP), or 1-year, flood at 5.3 ft NAVD'88. Street flooding in this downstream reach begins at the 0.2 AEP (5-year) event and significant damages to structures begin at the 0.04 AEP (25-year) event at the Tower Trailer Park, Mileed Way Industrial Park, and Beverly Street residences in Carteret.

The confluence of the Rahway and South Branch Rivers at Edgar Road Bridge begins street flooding at the 0.5 AEP (2-year) event by Essex Street in Rahway. Significant damages begin at the 0.1 AEP event, including the automotive businesses and residences, without raised foundations, between Route 1 and Milton Avenue.

South Branch starts producing minimal damages to industrial areas at the 0.1 AEP flood at St. Georges Avenue and Elliot Street. Street flooding and residential damage in South Branch begins at the 0.02 AEP (50-year) event at Leesville Avenue.

Levee overtopping at South Branch and Rahway River currently begins slightly above the 0.01 AEP (100-year) event. For future conditions that include some increase in flow and sea level, the levees will be overtopped before the 0.01 AEP event.

Robinson's Branch has street flooding beginning at the 0.02 AEP (50-year) event at the intersection of Central Avenue and St. Georges Avenue and at Hamilton Avenue. Significant damages beginning at the 0.2 AEP (5-year) event occurs at the confluence with the Rahway River near the Rahway Arts District.

Flooding upstream is not heavily tidally influenced. Although tidal storm events alone would not cause significant damages upstream of the confluence, the joint-probability of a fluvial and tidal event occurring at Robinson's Branch and south of the Rahway Water Supply Dam suggests that a 0.04 AEP (25-year) event would cause damages.

2.3 Existing Hydraulic Features

Some areas along the Rahway River have seen a decrease in flood risk due to improvements implemented through the years. The USACE South Branch Flood Control Project of 1968 is the only project that falls within the tidal boundaries of this study. The flood control project was a combination of levees, floodwalls, and channel modification. The right bank of the Rahway River between Monroe Street and East Hazelwood Avenue has levees. The left bank of the South Branch River from Regina Avenue to Sterling Place is levee and from Sterling Place to Hazelwood Avenue is floodwall. This project also consists of a stop-log road closure structure at the Hazelwood Avenue Bridge. This system was constructed in the 1970's and is periodically inspected by the USACE Dam and Levee Safety Program.

The levee system was regraded in 2015 to the original design height of 12.6 ft NAVD'88 after the system was overtopped twice, slightly during Tropical Storm Irene in 2010 and by a few inches during Hurricane Sandy in 2011. Inspections had reported a settlement of about 1 ft. across the entire levee system.

3.0 HYDRAULIC BASIS OF DESIGN

3.1 Model Development

The hydraulic analysis of the Rahway River is based on an unsteady state numerical model using the Hydraulic Engineering Center River Analysis System (HEC-RAS) software. The hydraulic model used for this tidal flood risk management study encompasses the original fluvial study as well as new components of the coastal environment.

The fluvial analysis of the Rahway River is based on an unsteady state numerical model using HEC-RAS version 5.0. The boundaries of the model were to the north in West Orange by the Orange reservoir and to the south along the main stem to the mouth at Arthur Kill, including the Robinson's Branch and South Branch tributaries. This model was used to develop the without project and with project conditions for fluvial and tidal area.

The geometry was created using a combination of survey data, LiDAR, and previous model geometry. The 2009 topographic mapping of Cranford was developed by Roger Surveying PLLC and included surveys of utilities, bridges, and weirs. The channel cross sections were placed no more than 300 ft. apart, supplemented with 2 ft. contour topographic maps from June 2009 to create overbank cross sections. The 2012 topographic mapping of Robinson's Branch was developed by McKim & Creed and included channel cross sections (which were placed no more than 300 ft. apart), utilities, bridges, and weirs. 2006 FEMA Flood Insurance Study (No. 34039CV002A) channel profiles and 2007 LiDAR data of New Jersey were used to create the geometry of upstream fluvial reaches, South Branch, Upper Robinson's Branch, and tidal portions of the Rahway River by the Arthur Kill.

3.2 Model Calibration and Validation

The HEC-RAS model was calibrated for two events: Tropical Storm Irene and Hurricane Sandy. The 2012 Hurricane Sandy event was used to model a storm surge event in the tidal area of study. Hurricane Sandy is slightly less than a 0.01 AEP tidal event (100-year event) having a fluvial component that is negligible. The August 2011 Tropical Storm Irene was used to calibrate a storm with both fluvial and tidal influence. TS Irene is slightly greater than a 0.01 AEP fluvial event with a tidal component slightly less than the 0.1 AEP (10-year) event. Stage hydrographs of recorded tide elevations at Bergen Point were used as the downstream HEC-RAS boundary condition for Sandy and Irene. The storm surge of both Sandy and Irene can be determined by subtracting the predicted astronomical tide from the actual recorded "tide" of each event. The surge of each event can be seen in Figure 3 and Figure 4. Additionally, a hydrologic analysis using the Hydraulic Engineering Center Hydrologic Modeling System (HEC-HMS) software of the Rahway River Basin provided flow hydrographs for the fluvial Irene storm event. Evaluating the hydrology nodal diagram and the characteristics of the Rahway River Basin, the flows obtained from HEC-HMS were referenced to cross sections or locations in the HEC-RAS geometry. Refer to the Hydrology Appendix for details on hydrologic methodology and modeling.

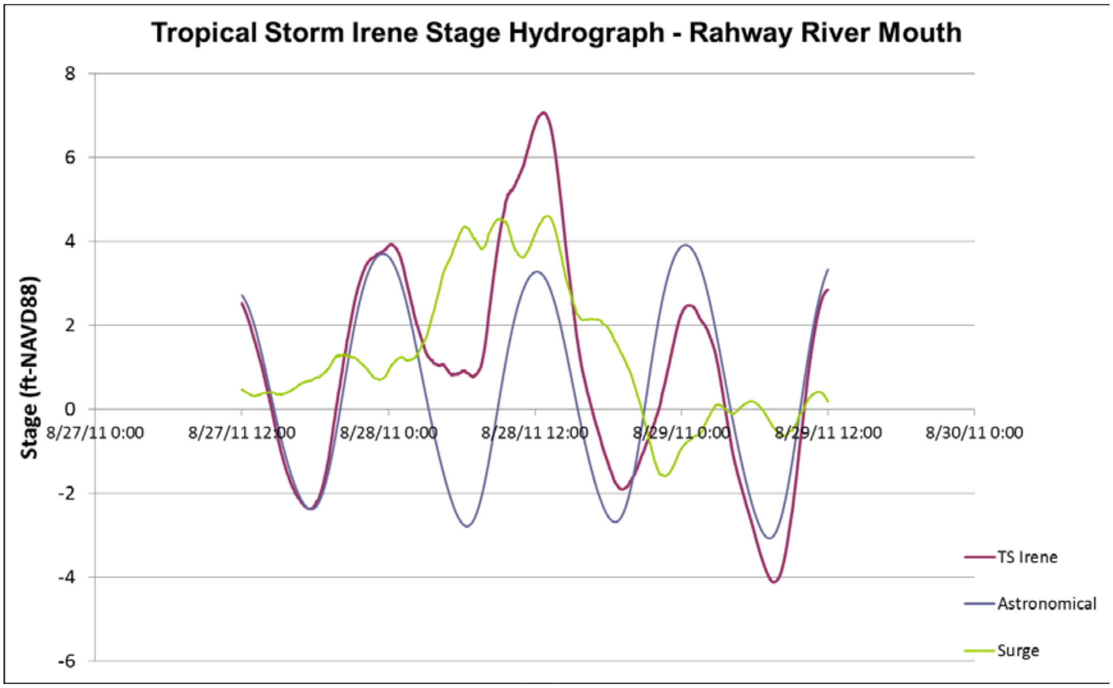


Figure 3: Stage Hydrograph for Tropical Storm Irene.

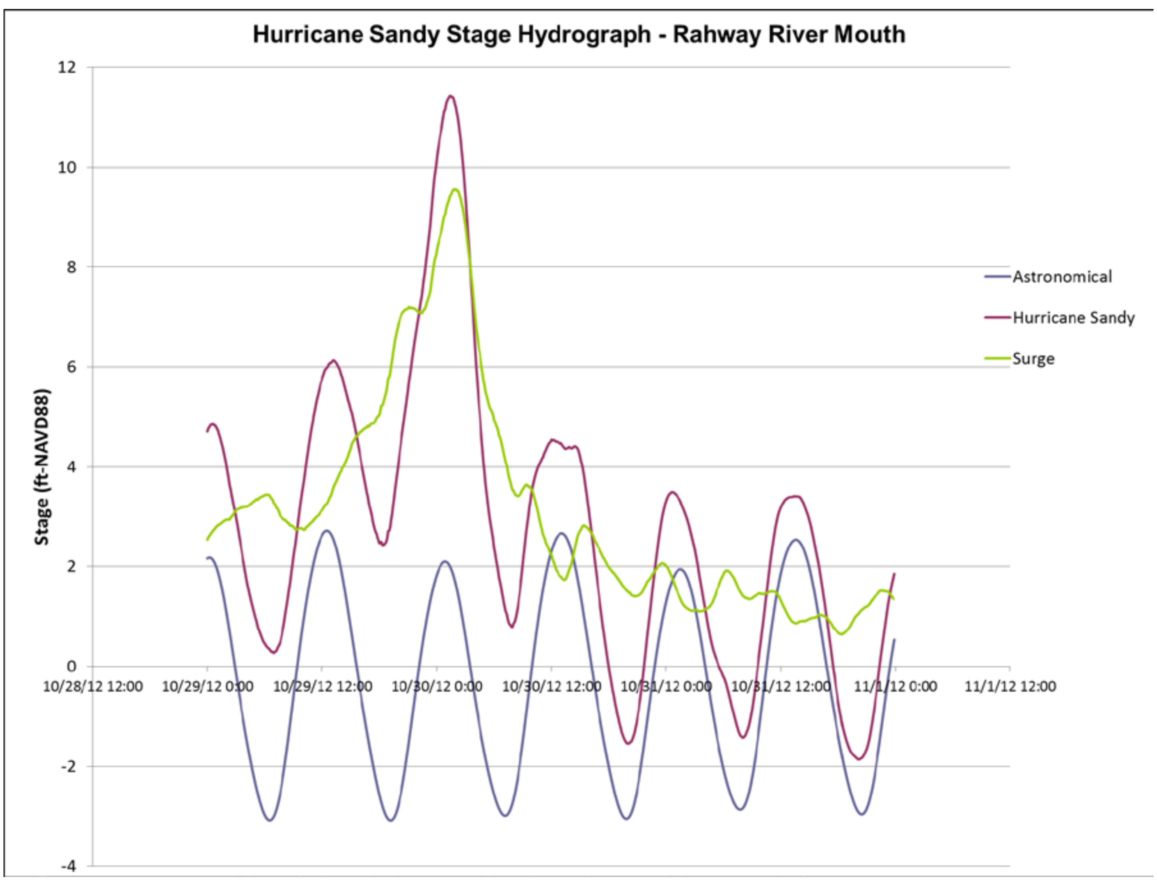


Figure 4: Stage Hydrograph for Hurricane Sandy.

In the first step of calibration, visual observations, Arc-GIS land cover, and aerial photographs were used to characterize the initial Manning's n-value. The overbanks varied from open spaces and parking lots to areas with high density vegetation or structures. Initial n-values were set between 0.025 and 0.045 for the channel, and overbank n-values were estimated to range between 0.025 and 1.5. Manning's n-values of 1.5 in the overbanks are for areas with no flow and large obstructions. Ineffective flow areas were identified in the overbanks at bridges and bends to better represent the effects of structures and topography on flow conveyance. Contraction and expansion coefficients were initially set at 0.1 and 0.3 for the open channel sections and at 0.3 and 0.5 for bridge sections.

In the second step of calibration, high water marks (HWM) were documented from multiple sources for both Hurricane Sandy and TS Irene. For tropical storm Irene, ten HWMs were obtained along the Robinson's Branch, two along the Rahway River in Rahway, and two along the Rahway River in Clark. High water marks were obtained from field surveys, eye-witness accounts, and gage data. Hurricane Sandy's five HWMs along the Rahway River, south of the Rahway River Park, came from the USGS Hurricane Sandy Data Viewer (http://stn.wim.usgs.gov/sandy/), eye witness accounts, and gage data. A USGS flow gage (013956000 Robinson's Branch at Rahway) HWM was added to the Robinson's Branch to verify the assumption of low flow contributions from the fluvial component of the storm event. The high water mark at the USGS gage 01395000 Rahway River at Rahway was not reliable for either event since it was submerged by the coastal surge for both events. Further adjustments to Manning's n-values, contraction and expansion coefficients, weir coefficients, ineffective flow areas, and other parameters were made in order to reproduce the WSEs to within ±0.5 ft. of the observed HWMs. The results show replicated results comparable to the historical events, especially the overtopping of the levees at the Rahway River and South Branch during Hurricane Sandy. Table 1 and Table 2 show the HWM elevations and locations for TS Irene and Hurricane Sandy, as well as the computed WSEs in that location from the RAS model. Figure 5, Figure 6, Figure 7, and Figure 8 are the HEC-RAS WSEs calibration profiles for the Irene and Sandy storm events.

Table 1: Tropical Storm Irene HWMs and HEC-RAS Calibration.

River Reach	HEC-STA	Computed WSE (ft, NAVD88)	HWM Elevation (ft, NAVD88)	Difference (ft)	Location
Robinson's Branch	8847.78	25.41	25.50	-0.09	***01396000 Robinsons Branch***
Robinson's Branch	6724.74	19.96	19.82	0.15	644 Maple
Robinson's Branch	5922.51	19.85	19.72	0.13	941 Jefferson
Robinson's Branch	5902.69	19.65	19.76	-0.11	Jeff-Elm-Bouman
Robinson's Branch	5282.55	19.28	19.58	-0.30	633 Bouman
Robinson's Branch	4008.99	18.78	18.99	-0.21	1229 St. Georges
Robinson's Branch	2583.05	18.29	18.30	-0.01	1452 Church
Robinson's Branch	1950.95	17.10	17.00	0.10	360 Hamilton
Robinson's Branch	962.53	16.80	16.80	0.00	277 Hamilton
Robinson's Branch	777.87	16.10	15.91	0.19	Irving 1653
Millburn-Clark	33116.94	19.59	19.81	-0.22	***01395000 Rahway***
Millburn-Clark	28743.80	15.03	14.98	0.05	182 Grand
Rahway	27995.02	14.49	14.43	0.06	***Confluence***
Rahway	26897.93	11.52	11.60	-0.08	Monroe Ave

Table 2: Hurricane Sandy HWMs and HEC-RAS calibration.

River Reach	HEC-STA	Computed WSE (ft, NAVD88)	HWM Elevation (ft, NAVD88)	Difference (ft)	Location
Millburn-Clark	33162.10	12.51	11.90	0.61	01395000 Rahway River
Rahway	26897.93	12.30	12.60	-0.30	Dock St
Carteret&Woodbridge	23622.28	12.29	12.60	-0.31	Confluence
Carteret&Woodbridge	11792.00	12.25	12.20	0.05	Medwick Park Trail
Carteret&Woodbridge	2187.32	12.13	12.10	0.03	Tremley Point Rd

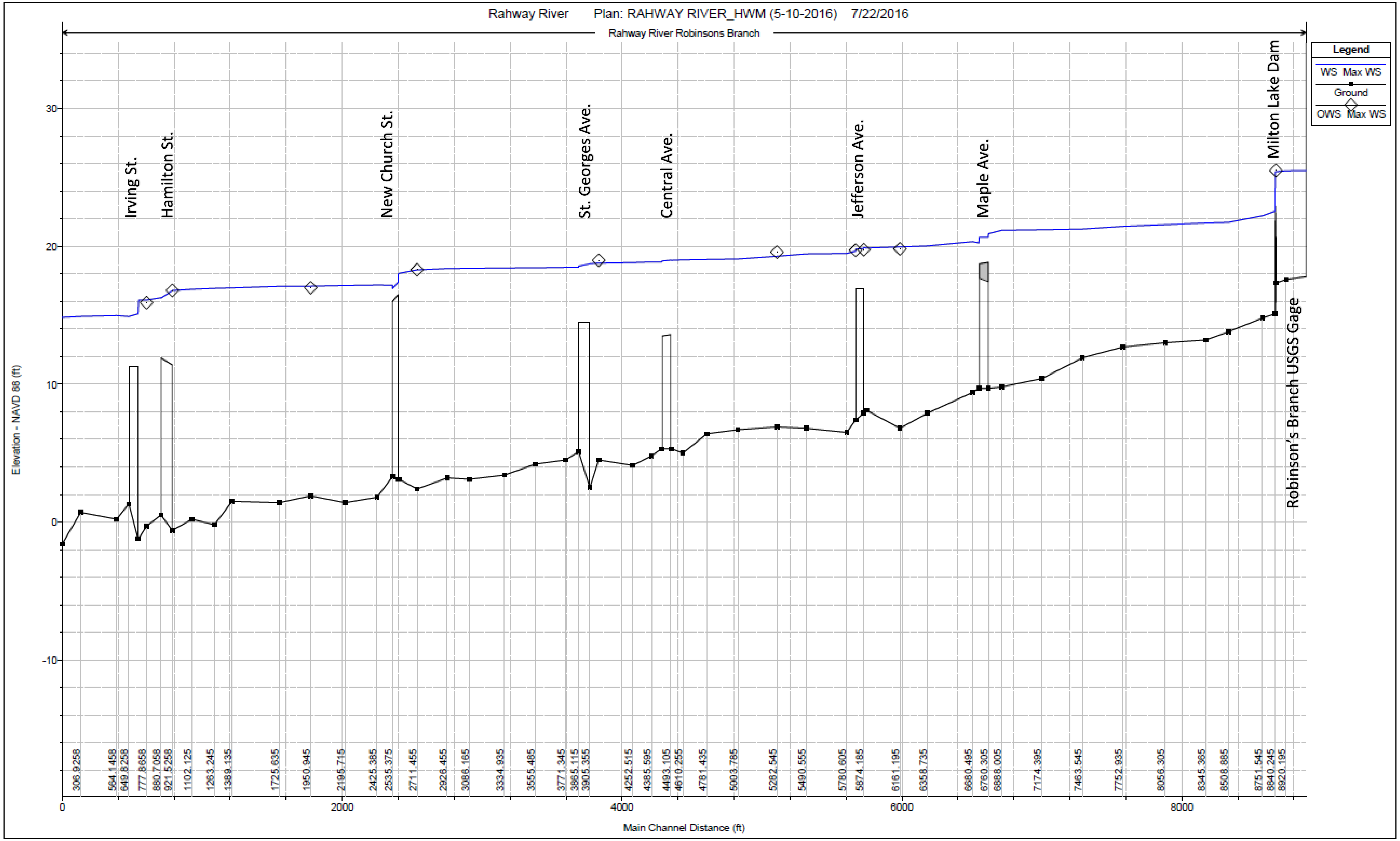


Figure 5: Computed water surface profile and observed HWMs for Tropical Storm Irene in Robinson's Branch.

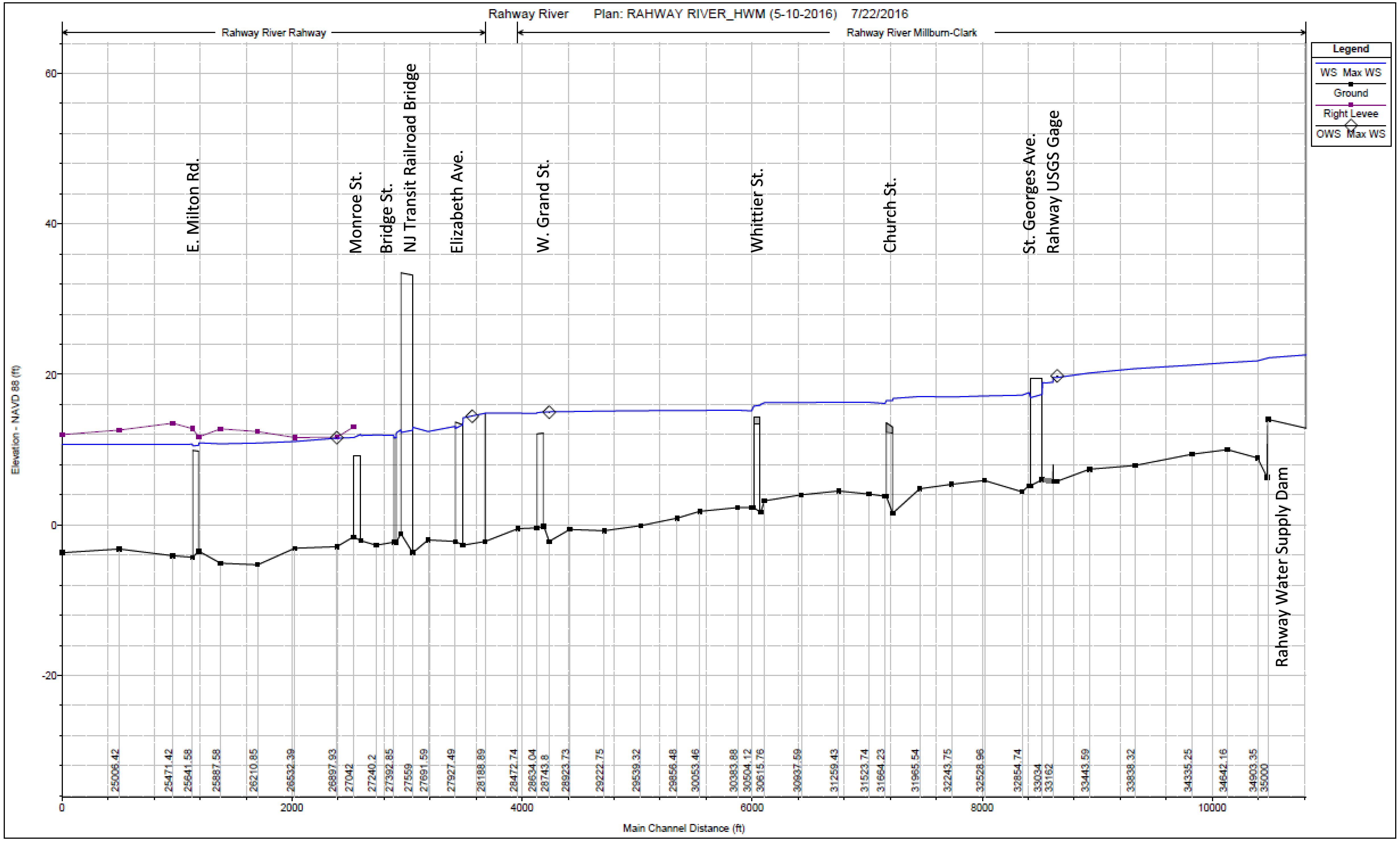


Figure 6: Computed water surface profile and observed HWMs for Tropical Storm Irene in the Rahway River between Rahway Water Supply and South Branch confluence.

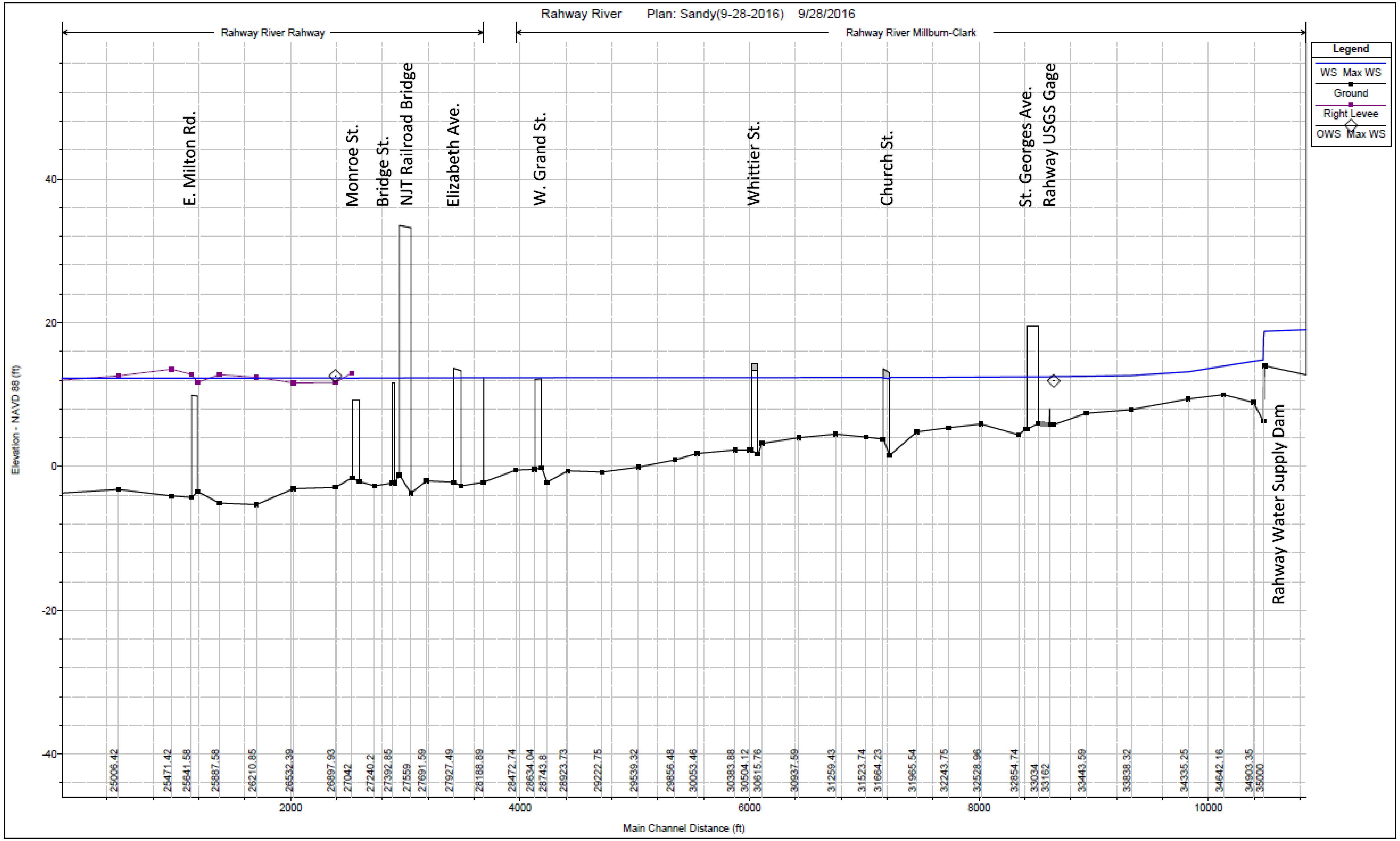


Figure 7: Computed water surface profile and observed HWMs for Hurricane Sandy in the Rahway River between Rahway Water Supply and South Branch confluence

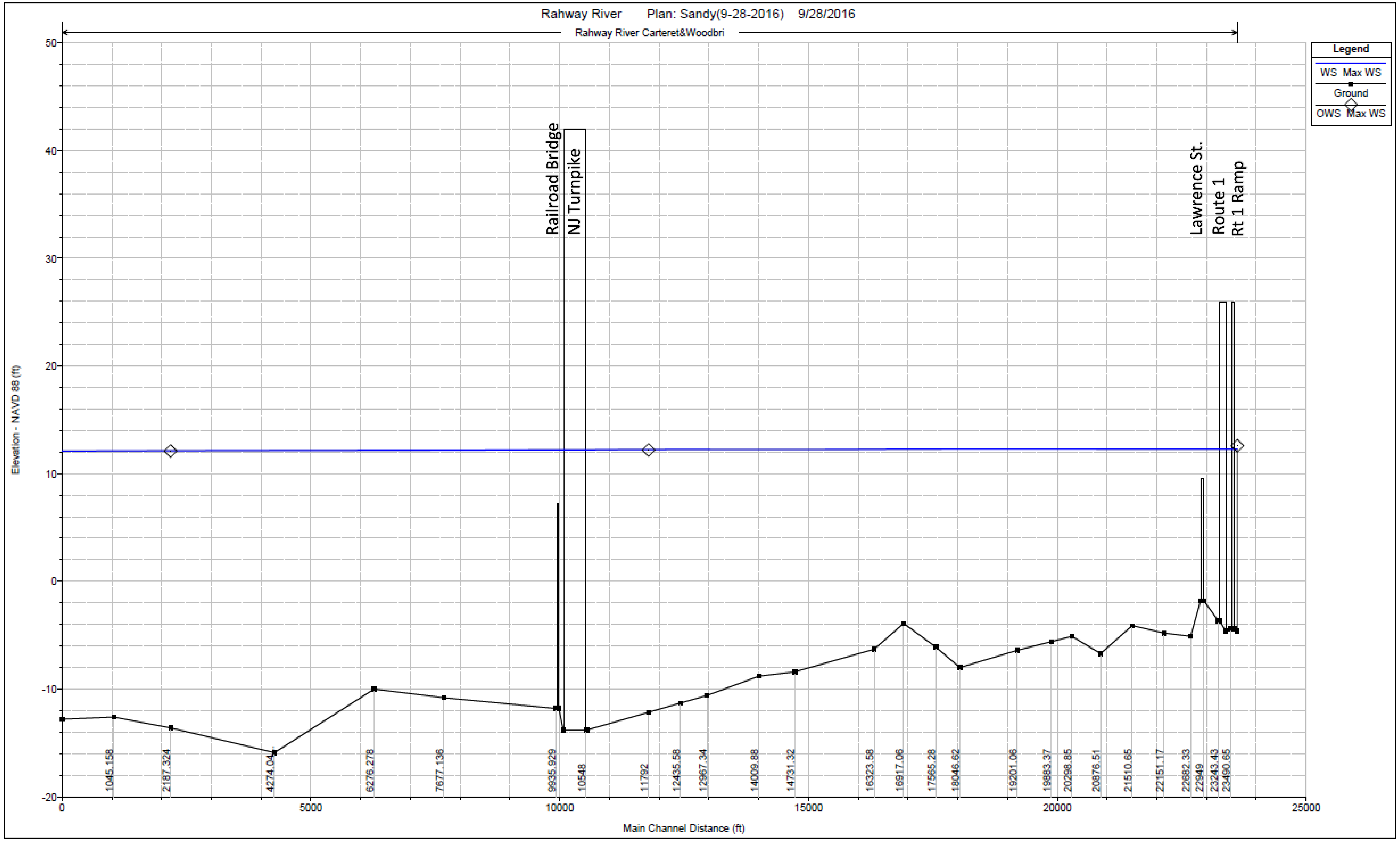


Figure 8: Computed water surface profile and observed HWMs for Hurricane Sandy downstream of South Branch to the Arthur Kill.

The next step of calibration includes replicating USGS rating curves and observed annual peak stages at the gages for TS Irene. However, this effort was previously completed in the Rahway River Fluvial Feasibility Study analysis. The calibration and comparisons between computed rating curves, USGS rating curves, and observed data can be seen in the Hydraulics Appendix of the 2016 Flood Risk Management Study of the Rahway River (Fluvial) feasibility report.

Due to the tidal nature of the model, much attention was put towards reproducing the stage hydrographs at the Arthur Kill boundary condition. Stage hydrographs for nine hypothetical events were developed, and their behaviors were compared to those of the observed Tropical Storm Irene and Hurricane Sandy events. This process will be described further in Section 3.3.2 Downstream Boundary Condition – Stage Hydrographs.

3.3 Boundary Conditions and Tidal-Fluvial Joint Probability

3.3.1 Tidal-Fluvial Assessment

In order to run the unsteady hydraulic model of the Rahway River with a set of hypothetical events, boundary conditions had to be established for the upstream reaches and the mouth of the Rahway River. Since the Rahway River flows into the Arthur Kill (an estuary), it was necessary to perform a Tidal-Fluvial assessment to establish the coincidental upstream flows that might be expected to occur during a storm surge, or coastal storm. There are three scenarios for storm events in the Rahway River basin:

(1) Local rainfall storms (large rain, no wind) producing fluvial floods without coastal impact,

(2) Offshore coastal events (large wind, no rain) producing coastal surges without high river flows, and

(3) Large storm events with both rain and coastal winds, with the possibility of producing floods associated with both coastal storm surges and high flows in the river.

This tidal-fluvial assessment focuses on scenarios 2 and 3, which will help determine if there are coincidental fluvial events associated with the tidal events. Scenarios 1 and 3 were used for the fluvial-tidal assessment during the Rahway Fluvial Study to determine the boundary conditions during a fluvial event and coincidental tidal stage. The results of the fluvial-tidal assessment will be used in the analysis of mixed-populations described further in section 3.3.3.

For this assessment, both the NOAA tidal gage at Bergen Point (ID: 8519483) and the fluvial gage at Rahway (USGS 10395000) were used to compare historical tide events with the coincidental fluvial data. Only tidal events greater than a 0.99 AEP (1-yr) and their corresponding maximum fluvial discharge were evaluated. The common data available for both gages is approximately 31 years. The results show that of 66 historic coastal events, only four events had a flow frequency greater than the 0.2 AEP (5-year) event. The results also show that the majority of tidal events are coupled with fluvial events having a 0.99 or less AEP (1-year) event.

Previous estuary studies at the NY District have determined that there was no correlation between coastal and fluvial events and it was common to use an average daily flow or in some other cases a 0.50 AEP (2-yr) fluvial flow with any significant coastal event. Since Tropical Storm Irene has now been added to this assessment, it appears to be more appropriate to use a 0.2 AEP (5-year) event with a significant coastal event. Figure 9 shows the frequency of tide events plotted with the frequency of the associated maximum flow for those events all at the Rahway gage. As mentioned previously, a similar assessment was performed for fluvial dominant storms (i.e. scenarios 1 and 3). Figure 10 shows the frequency of significant fluvial events plotted with the frequency of the associated maximum tidal stage.

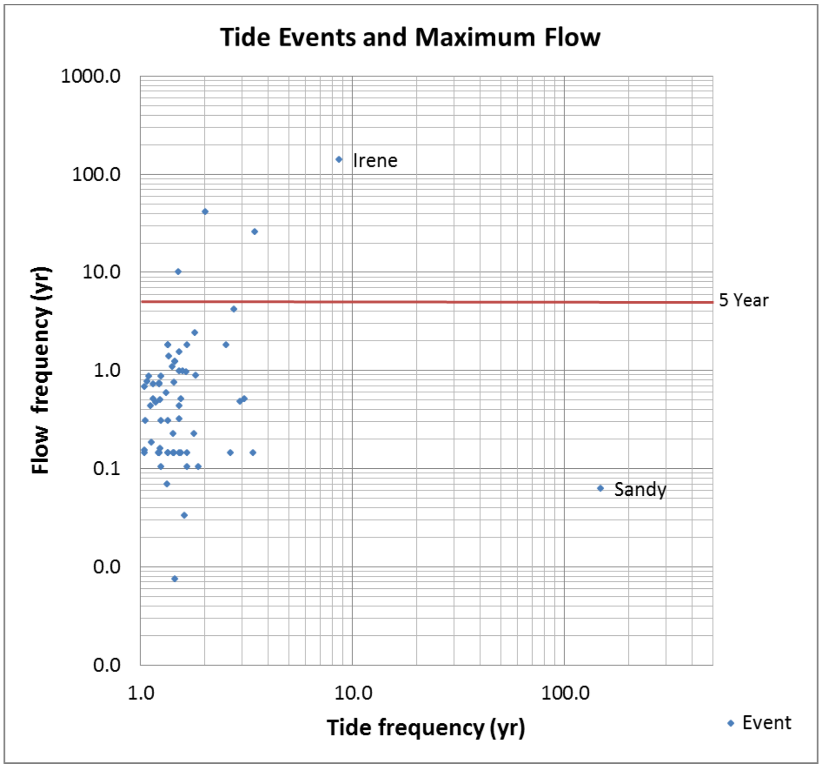


Figure 9: Tidal event and the maximum flow during the event.

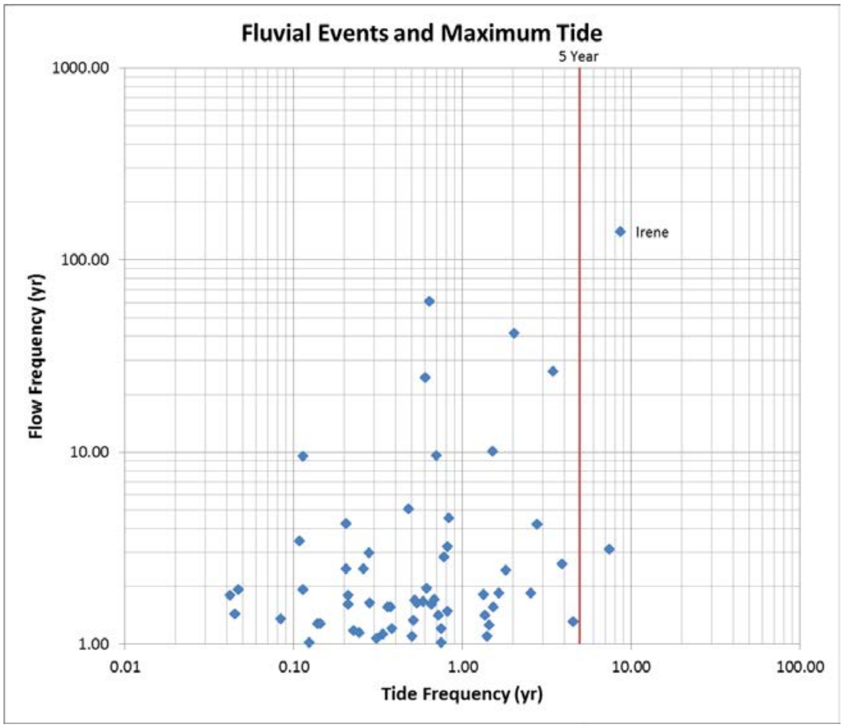


Figure 10: Fluvial event and the maximum tidal stage during the event

Based on this tidal-fluvial assessment, it was determined that tidal dominant coastal storms (scenarios 2 and 3) are historically associated with high frequency fluvial events (low flows). Coastal surges associated for each coastal frequency event were assigned a coincidental flow, which became the downstream and upstream boundary conditions, respectively. The boundary conditions are as follows: the 0.99 AEP tide was assigned a coincidental 0.99 AEP flow, the 0.5 AEP tide was assigned a coincidental 0.5 AEP flow, and all other tides (0.2 AEP and lower) were assigned a coincidental flow of 0.2 AEP. Likewise, the boundary conditions developed from the fluvial-tidal assessment (scenarios 1 and 3) have fluvial influenced boundary conditions. All boundary conditions used in this study both tidally influenced and fluvial influenced can be seen in Table 3.

Table 3: Upstream and downstream boundary conditions for coincidental storms.

Tidal Influenced Boundary Conditions		**Fluvial Influenced Boundary Conditions**	
Tidal AEP Event	Coincidental Fluvial AEP Event	Fluvial AEP Event	Coincidental Tidal AEP Event
0.99 (1 Yr)	0.99 (1 Yr)	0.99 (1 Yr)	0.99 (1 Yr)
0.5 (2 Yr)	0.5 (2 Yr)	0.5 (2 Yr)	0.5 (2 Yr)
0.2 (5 Yr)	0.2 (5 Yr)	0.2 (5 Yr)	0.2 (5 Yr)
0.1 (10 Yr)	0.2 (5 Yr)	0.1 (10 Yr)	0.2 (5 Yr)
0.04 (20 Yr)	0.2 (5 Yr)	0.04 (20 Yr)	0.2 (5 Yr)
0.02 (50 Yr)	0.2 (5 Yr)	0.02 (50 Yr)	0.2 (5 Yr)
0.01 (100 Yr)	0.2 (5 Yr)	0.01 (100 Yr)	0.2 (5 Yr)
0.005 (200 Yr)	0.2 (5 Yr)	0.005 (200 Yr)	0.2 (5 Yr)
0.002 (500 Yr)	0.2 (5 Yr)	0.002 (500 Yr)	0.2 (5 Yr)

For the remainder of this report, all frequency events referenced will be tidal dominant unless indicated otherwise.

3.3.2 *Downstream Boundary Condition – Stage Hydrographs*

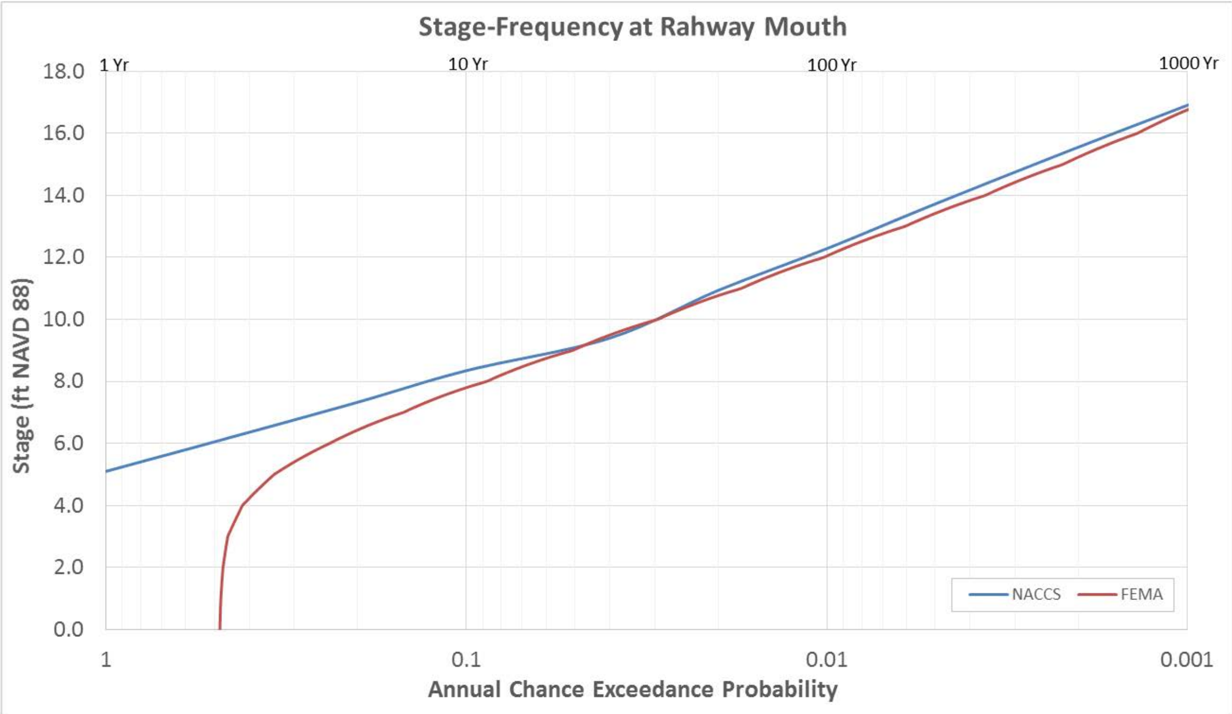
The 2015 USACE North Atlantic Coast Comprehensive Study (NACCS) coastal stage-frequency curve at Arthur Kill/Rahway Mouth (node ID: 11659) was used to obtain all annual exceedance probability peak stages for the tidal boundary condition hydrographs. The stage frequency data for present conditions is shown in Table 4 and Figure 11. The coastal stage-frequency data from the 2013 FEMA Region II Storm Surge Project was included for reference purposes and, as depicted in Figure 11, there is very good agreement between the FEMA study and the Corps NACCS study for all points greater than the 0.1 AEP event.

The NOAA Bergen Point gage (ID: 8519483) tide cycle characteristics were used to develop a basic shape for all the tidal stage hydrographs. The project area experiences semidiurnal tide cycles, i.e. there are two high tides and two low tides every lunar day. The tide cycle characteristics can be seen in Table 5. The USACE Survey Section at Caven Point, New Jersey provided the standard conversion at this gage which is MLLW at -2.95 ft NAVD'88. The astronomic or predicted tide cycle during Hurricane Sandy was used as the base in the development of stage hydrograph boundary conditions for this study.

The peak of each of the nine hypothetical coastal stage frequency hydrograph was fixed to match the elevation from the Corps NACCS study. The duration for each of the nine coastal storms was also obtained from the NACCS. The duration of each hypothetical storm had previously been obtained for the Port Monmouth CSRM study and it was reused for this study. The duration of each storm increased as the size of the hypothetical storm got larger. Storm durations ranged from 11 hours (0.99 AEP event) to 28 hours (0.001 AEP event). Figure 12 shows the storm duration curve from the Port Monmouth study. The duration was used to determine the points where stage elevations would depart from and return to normal tide cycle. The maximum surge was uniformly reduced from the peak back to a normal tide on both sides of the peak. Figure 13 shows the stage hydrograph boundary condition for each event. Finally, the peak tidal stage was made to be coincidental to peak flow at the mouth of the Rahway River. The assumption that the peak coastal surge occurs at high tide was combined with the assumption that the peak surge also occurs at the same time as the peak fluvial flow to create a conservatively high maximum water surface elevation.

Table 4: NACCS Stage-Frequency data at Rahway Mouth.

Annual Exceedance Probability	Frequency Event (years)	Stage (ft-NAVD88)
0.99	1	5.10
0.5	2	6.05
0.2	5	7.33
0.1	10	8.35
0.04	25	9.40
0.02	50	10.94
0.01	100	12.28
0.005	200	13.73
0.002	500	15.56



*FEMA curve at Carteret (ID: 543829) approx. 0.5 mi. downstream of NACCS SavePoint on the Arthur Kill.
Figure 11: Stage-Frequency Curve at Rahway Mouth from NACCS and FEMA.

Table 5: Bergen Point Gage Tide Datum

Tide Characteristics for Bergen Point Gage ID: 8519483	
Tidal Datum	Elevation in ft above NAVD88
Mean Higher High Water	2.56
Mean High Water	2.24
Mean Sea Level	-0.18
Mean Tide Level	-0.25
Mean Low Water	-2.74
Mean Lower Low Water	-2.95

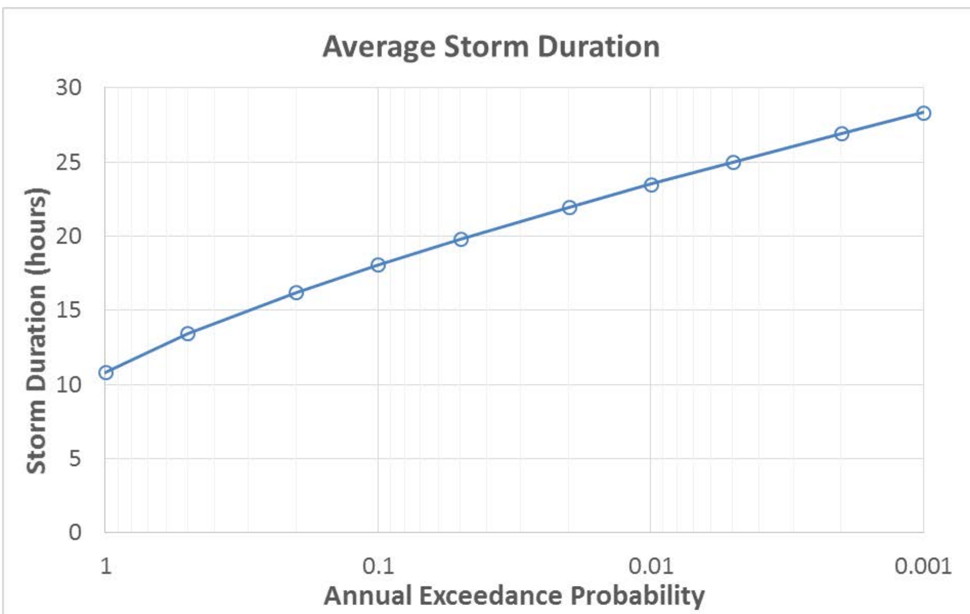


Figure 12: Storm duration curve from the NACCS study for Port Monmouth, NJ.

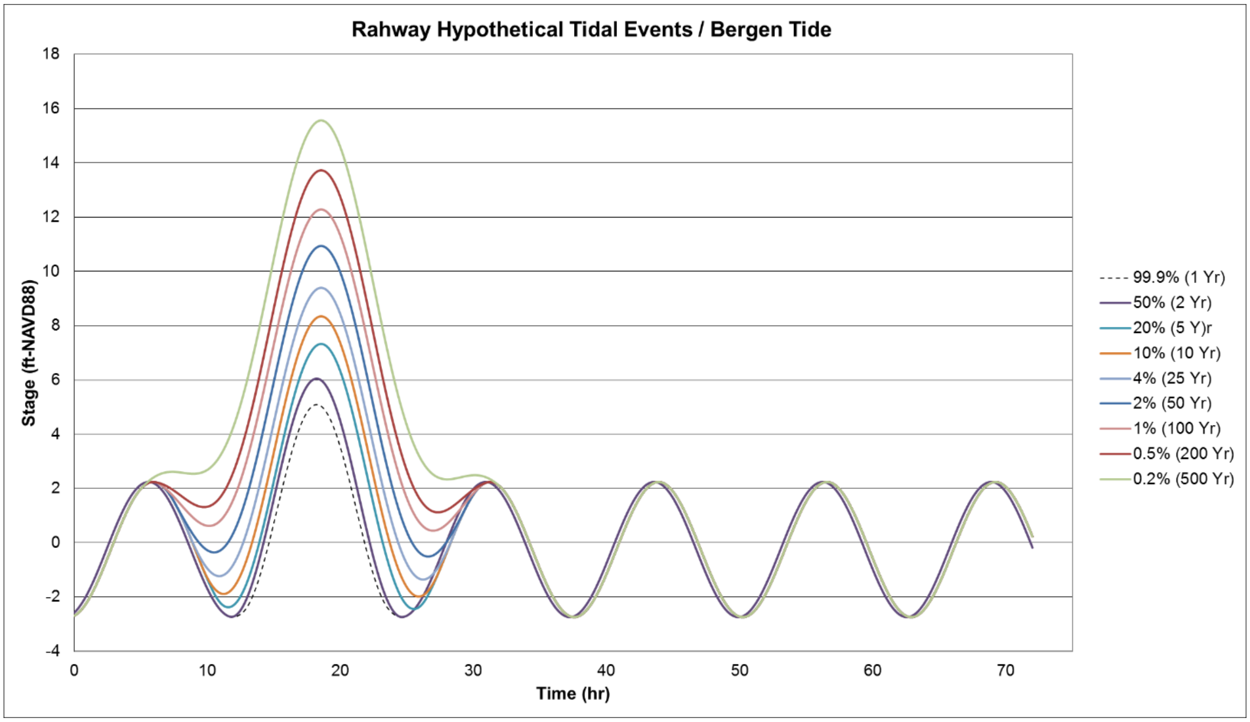


Figure 13: Stage hydrograph for hypothetical coastal events at the mouth of the Rahway River.

3.3.3 Analysis of Mixed Populations

Although coastal and fluvial flood events are sometimes related to the same storm event, the flooding is largely independent: one is based on wind, waves, and tide stages and the other is based on rainfall, runoff, and flow. In a hydrologic context, this condition is known as mixed population. According to EM 1110-2-1415 Chapter 10, mixed population is "applied to data that results from two or more different, but independent, causative conditions".

In this condition, a frequency curve derived by combining the frequency curves of each population can result in a computed frequency more representative of the observed data. The hydraulic runs for both the fluvial and the tidal conditions as explained in Section 3.3.1 and Table 3 above, were combined using the analysis of mixed populations to create a more accurate representation of flooding risk in the area of study. The resulting combined maximum water surface elevation curves do include some degree of coincidence as described and determined in the tidal-fluvial assessment from Section 3.31 above.

The mixed population joint probability is described by the following equation:

$$P_c = P_1 + P_2 - P_1 P_2$$

where:

P_c = Annual exceedance probability of combined populations for a selected magnitude

P_1 = Annual exceedance probability of same selected magnitude for population series 1

P_2 = Annual exceedance probability of same magnitude selected above for population series 2

The joint probability was computed for each cross section in all reaches to account for spatial sensitivities to flooding. The coastal probability of a particular water surface elevation was added to the fluvial probability of that same elevation (and the product of those probabilities was subtracted) to obtain the joint probability of being flooded at that elevation at that particular cross-section. The joint probabilities account for the fact that: 1) the lower portion of the Robinson's Branch and the upstream portion of the Rahway River by Clark are very sensitive to fluvial flows, 2) the City of Rahway and the lower portion of Robinson's Branch have a large risk from both tidal and fluvial flooding, and 3) Carteret and Linden are mainly flooded by coastal events.

The joint probability curves were computed for with and without project conditions. By using joint probability curves, the benefits of reducing the risk of flooding from both fluvial and coastal events was accounted for. Figure 14, Figure 15, and Figure 16 demonstrate the effects of joint-probability at three locations in the tidal area of study, i.e. Rahway and Robinson's Branch confluence, Rahway and South Branch confluence, and Rahway at Arthur Kill.

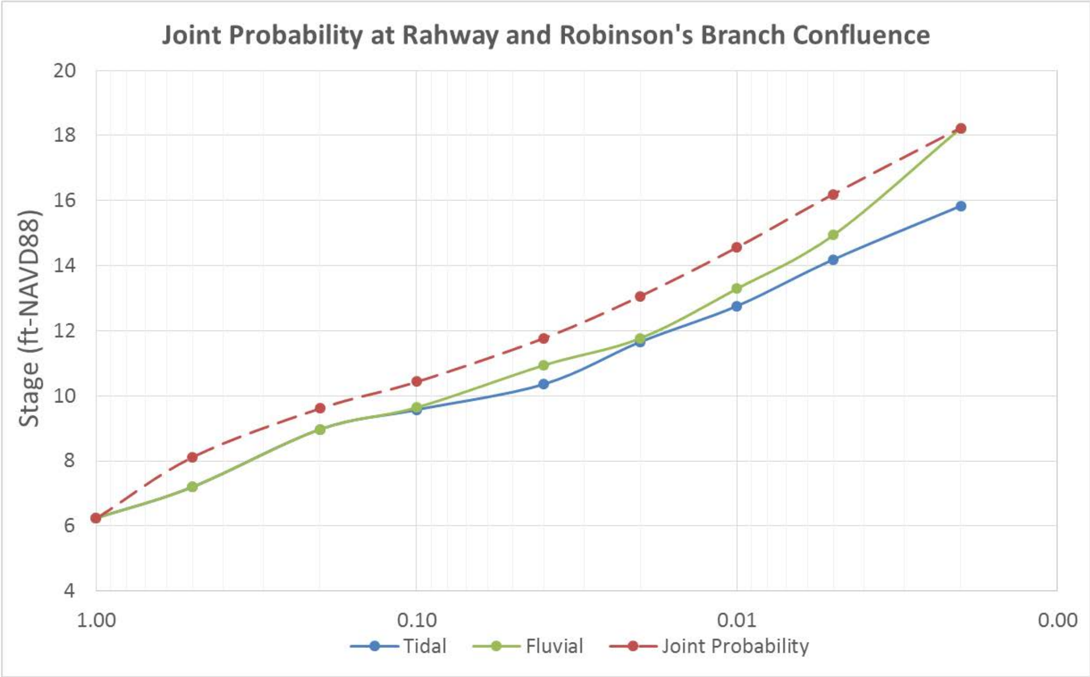


Figure 14: Joint Probability Curve at Robinson's Branch Confluence.

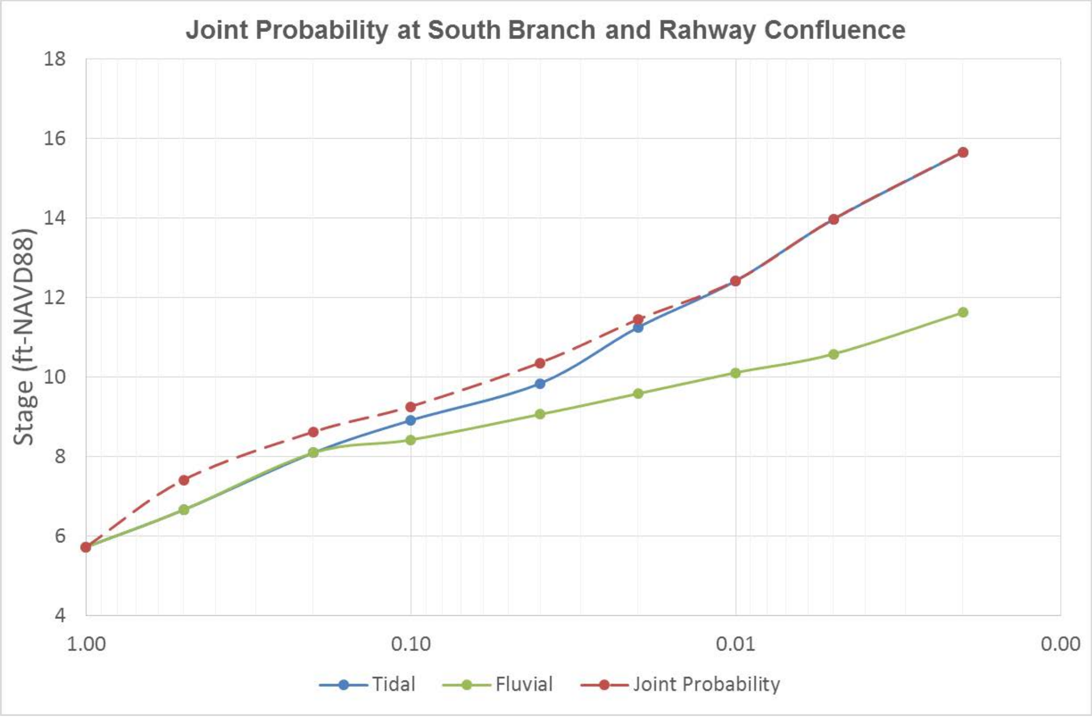


Figure 15: Joint Probability at South Branch Confluence.

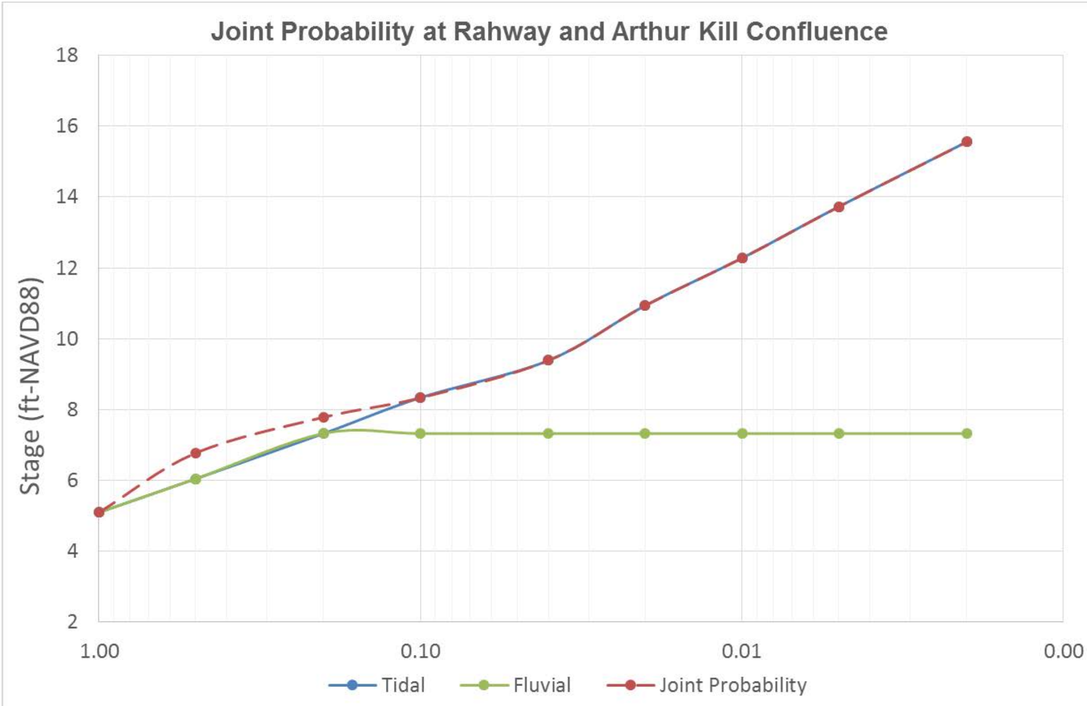


Figure 16: Joint Probability Curve at Arthur Kill

3.3.3.1 Mixed Population Confirmation

The stage frequency curve for the USGS gage at Rahway was plotted with the computed joint probability curve for cross section number 33162.10 at Millburn-Clark which is the nearest cross section upstream of the gage. Since the gage has a weir with a crest elevation of 7.68 feet NAVD88, any data near that elevation is primarily a fluvial flood; however elevations significantly above 7.68 ft NAVD88 represent the true observed tidal-fluvial conditions in this area of study. The comparison can be seen in Figure 17 and it appears that our assumptions were too conservative and this methodology will be re-evaluated during optimization.

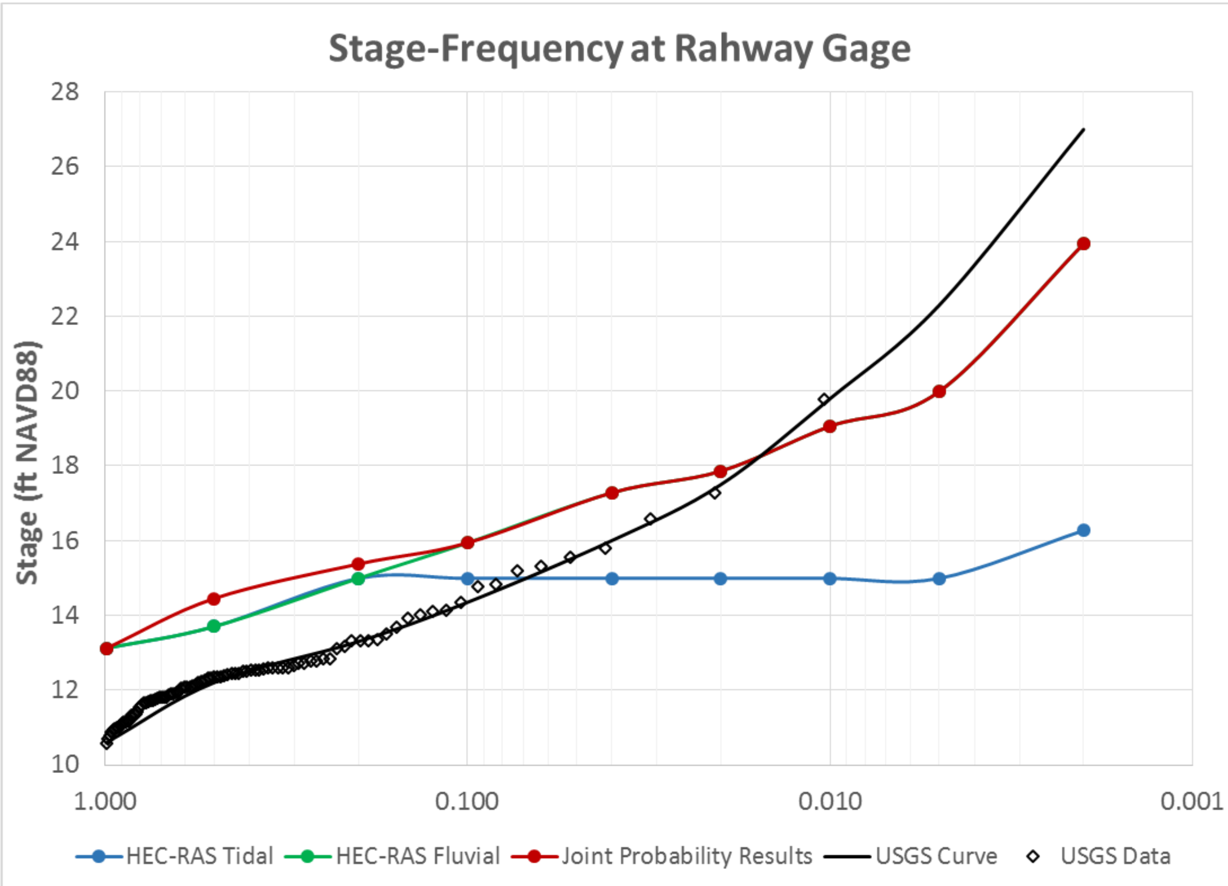


Figure 17: Stage-Frequency curves comparing gage data and computed data.

3.3.4 Sea Level Change (SLC)

Department of the Army, Engineering Regulation ER 1100-2-8162 provides guidance on incorporating the effect of projected SLC across the project life of USACE projects. Technical Letter ETL 1100-2-1 requires the use of at least three scenarios to estimate future sea levels. The USACE low rate of future SLC is based in the historic rate in the vicinity of the project area. Figure 18 shows the sea level rise trends and 33 years of data from the NOAA tide gage #8519483 at Bergen Point, New York. The plot shows the monthly mean sea level without the regular seasonal fluctuations due to coastal ocean temperatures, salinities, winds, atmospheric pressures, and ocean currents. The long-term linear trend is also shown, including its 95% confidence interval. The plotted values are relative to the most recent Mean Sea Level datum established by CO-OPS. The mean sea level trend is 4.65 millimeters/year with a 95% confidence interval of +/- 0.92 mm/yr based on monthly mean sea level data from 1981 to 2014 which is equivalent to a change of 1.53 feet in 100 years. This value was used to compute the expected low rate of SLC. The intermediate and high rates of future SLC are determined from the modified National Research Council (NRC -1987) eustatic sea-level change scenarios and the IPCC (2007) Types I and III respectively. The effects of vertical land movement (VLM) was also considered as a component of sea-level rise. The projected low, intermediate and high SLC scenarios are shown in Table 6 and Figure 19.

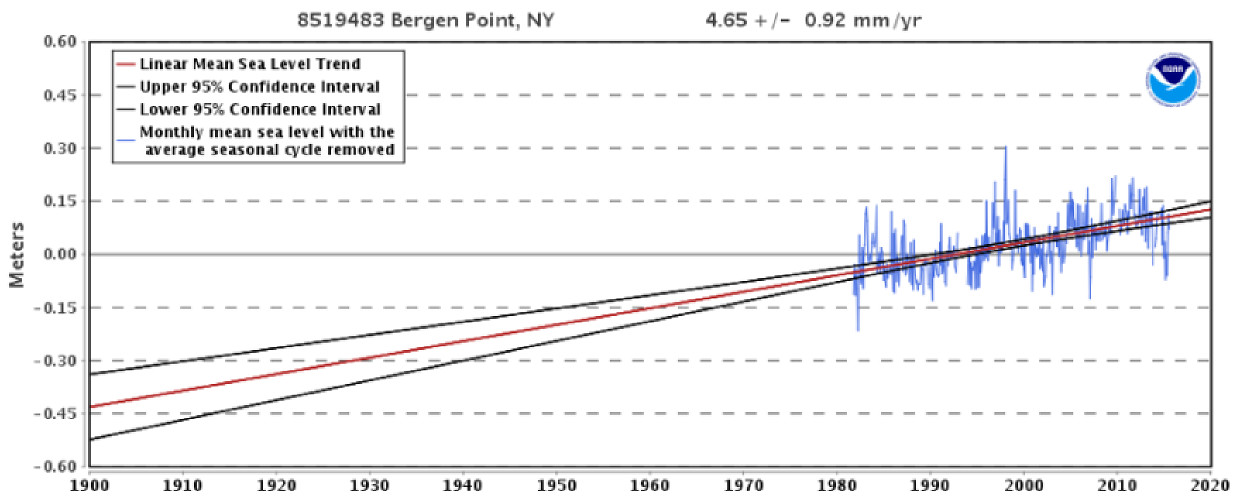


Figure 18: Sea level rise trends and monthly mean seal level at NOAA tide gage No. 8519483 at Bergen Point.

Table 6: Projected SLC for the period of analysis of 50 years at Bergen Point #8519483, and NRC/IPCC SLC scenarios.

Year	USACE Net SLC (ft.)		
	Low	Intermediate	High
2018	0.05	0.00	0.00
2023	0.12	0.10	0.18
2028	0.20	0.21	0.38
2033	0.27	0.32	0.60
2038	0.35	0.43	0.84
2043	0.43	0.55	1.09
2048	0.50	0.68	1.37
2053	0.58	0.80	1.66
2058	0.66	0.94	1.97
2063	0.73	1.07	2.30
2068	0.81	1.22	2.65

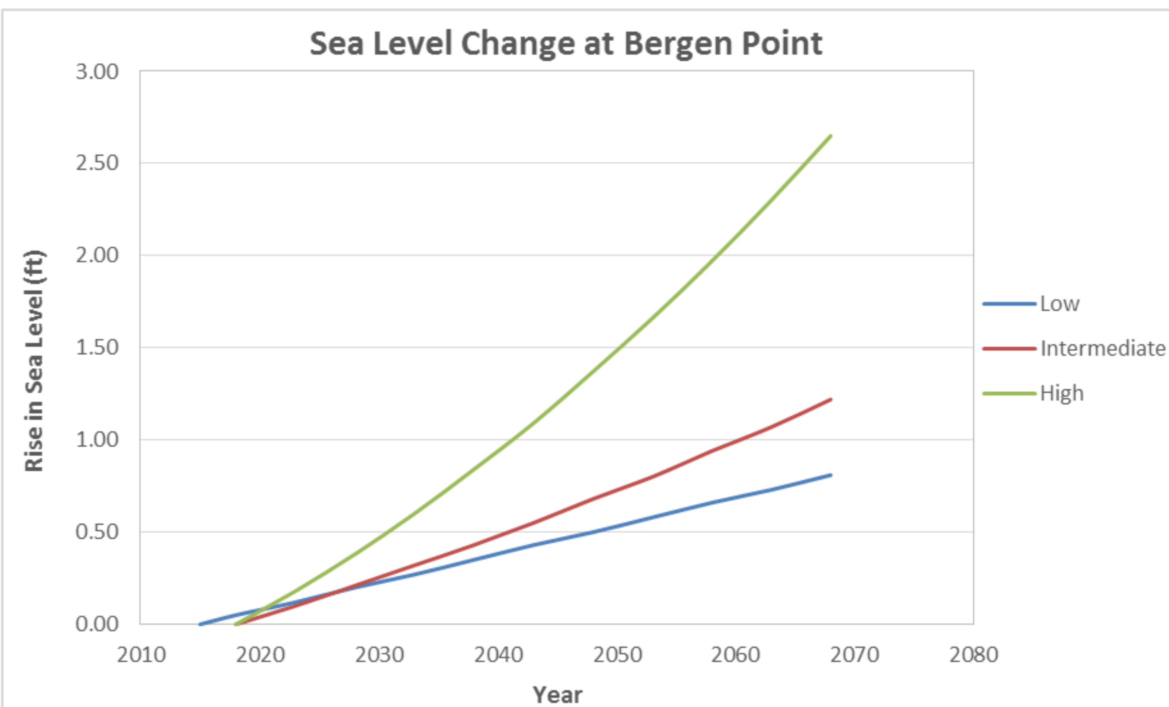


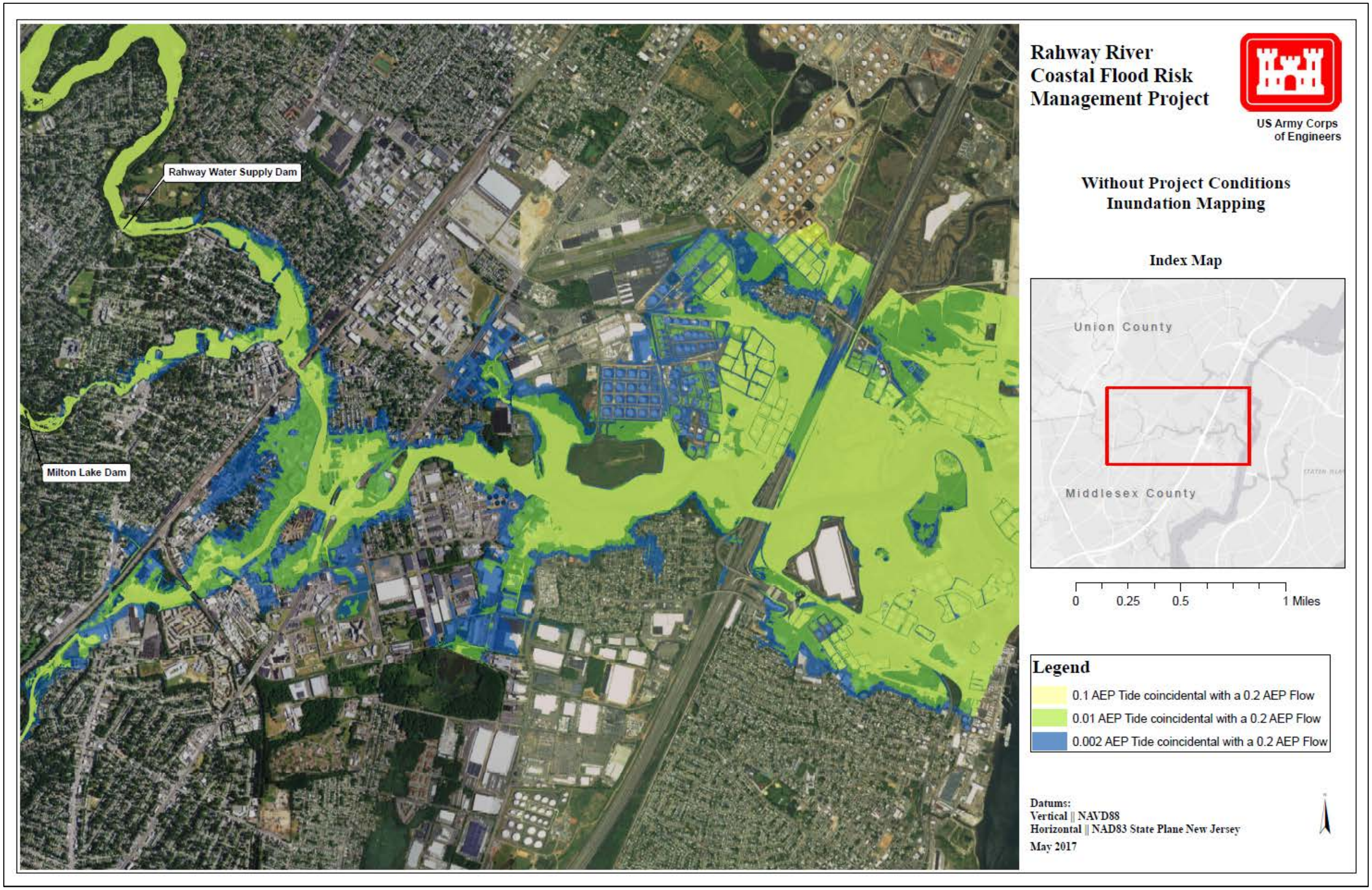
Figure 19: Projected SLC at Bergen Point for the local (low), NRC Type I (intermediate), and NRC Type III (high) scenarios.

Sea level rise is expected to have impacts on direct coastal flooding along the Rahway River tidal influenced area, including impacts to properties and critical infrastructure. Future conditions, with and without project includes the historic local rate of SLR, projected 50 years into the future (i.e. 0.76 ft rise in 50 years). From the base feasibility study date of 2015, projected 50 years from the construction date of 2018, the sea level will rise 0.81 feet by 2068. All future conditions runs used tidal stage hydrograph boundary conditions that included the historic rate of SLR. The impact of SLR projections are implicit to the hydraulic and economic computation due to the use of joint stage-probability curves that were modified for future conditions to included SLR.

3.4 Present and Future Conditions – Hydraulic Profiles

3.4.1 Flow Line Computation

The calibrated HEC-RAS model of the Rahway River was used to determine the present and future, "with-" and "without project" WSE for the 0.99, 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.005, and 0.002 AEP (1, 2, 5, 10, 25, 50, 100, 200, and 500-year) storm events. Inundation maps for "without project" present conditions for the 0.1, 0.01 and 0.002 AEP events are shown in Figure 20. The future "without project" model was created using the future hypothetical peak discharges, future sea level change, future vertical land movement, and the calibrated existing conditions HEC-RAS model. There is expected to be increases in WSEs due to urbanization in the fluvial area of study at the upstream boundary conditions and due to SLC in the tidal area of study at the downstream boundary condition. Both the increase in flow and tide elevations cause an increase in flooding for future without project conditions in the tidal area. Increased flows due to urbanization only have an impact in the tidal area up to the 0.2 (5-year) AEP event, with negligible impact near the mouth of the Rahway River. Tidally influenced flooding does not go beyond the Milton Lake dam or the Rahway Water Supply dam for future unimproved conditions due to the steep bed slope and topographic characteristics of the overbanks. Table 7 demonstrates the joint-probability increase in flood elevations due to urbanization and SLC in the next 50 years for the 0.2, 0.04, 0.01, and 0.002 AEP events.



*Note: This is the inundation of tidal events coincidental with 0.2 AEP flows. This does not represent joint-probability inundations.

Figure 20: "Without project" present condition inundation map for the 0.1, 0.01, and 0.002 AEP events.

Table 7: Difference in WSEs between future and present "without project" conditions.

Location	HEC-STA	W/O Project WSE (ft.)	W/O Project Future Increase in WSEs (ft.)			
			0.2 AEP (5-yr)	0.04 AEP (25-yr)	0.01 AEP (100-yr)	0.002 AEP (500-yr)
Rahway River at Rahway Water Supply Dam	34903.35	17.87	0.18	0.09	0.09	0.13
Robinson's Branch at Milton Lake Dam	8751.545	20.29	0.03	0.04	0.04	0.04
Robinson's Branch at Rahway Confluence	175.4458	10.43	0.47	0.42	0.46	0.14
South Branch Upstream	11216.78	9.21	0.04	0.04	0.01	0.02
South Branch and Rahway River Confluence	210.7962	15.40	0.62	0.62	0.94	0.77
Rahway at Arthur Kill	5.520991	9.26	0.81	0.81	0.81	0.81

Figure 21 through Figure 25 show the present “without project” WSE profiles for the Rahway River in the tidal area of study. The highlighted WSE profiles are the 0.99, 0.1, 0.04, 0.01, and 0.002 AEP (1, 10, 25, 100, and 500 year) tidal events. Also shown is the Sandy WSE profile for reference.

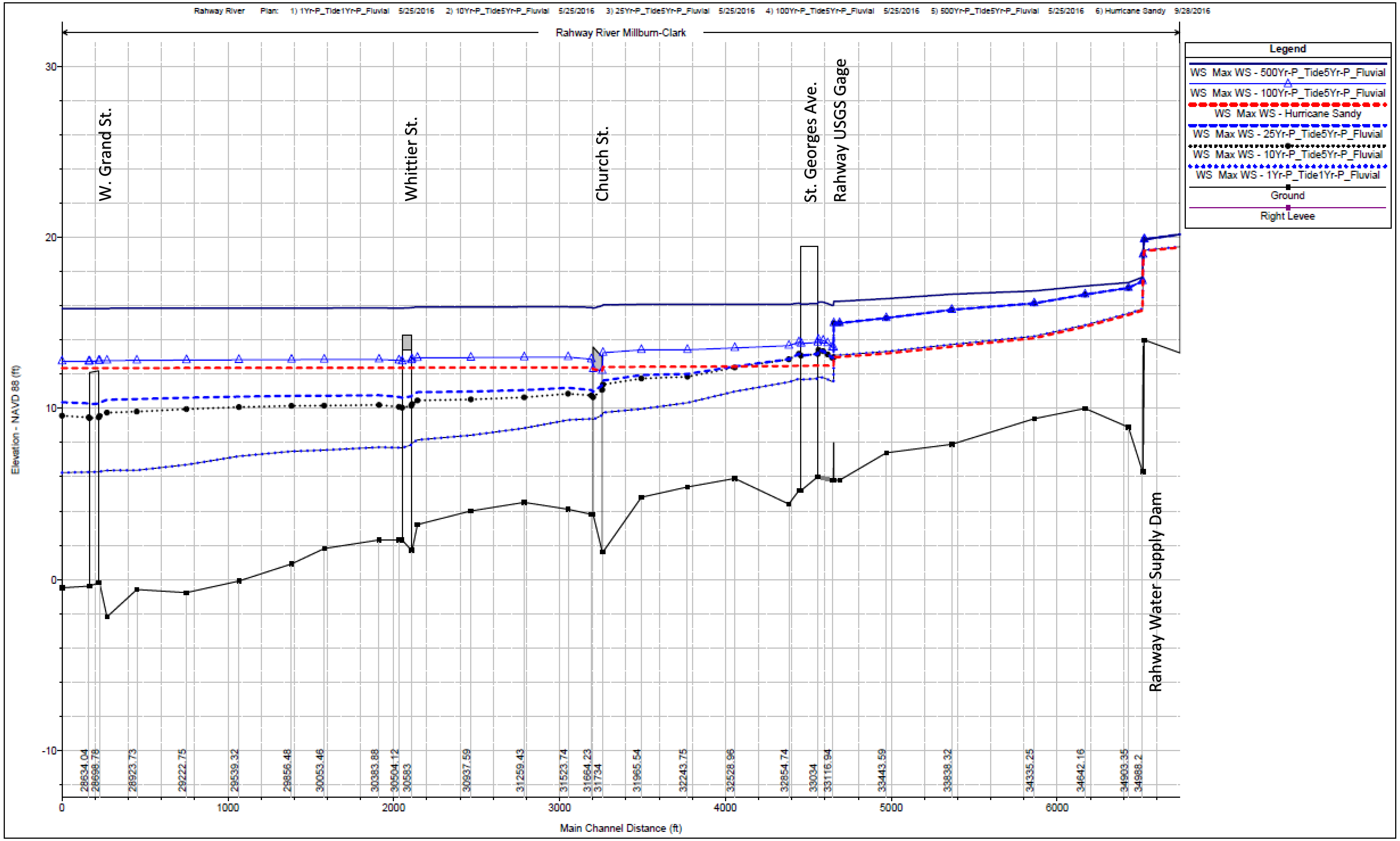


Figure 21: “Without project” condition computed water surface profile from Rahway Water Supply to Robinson’s Branch Confluence.

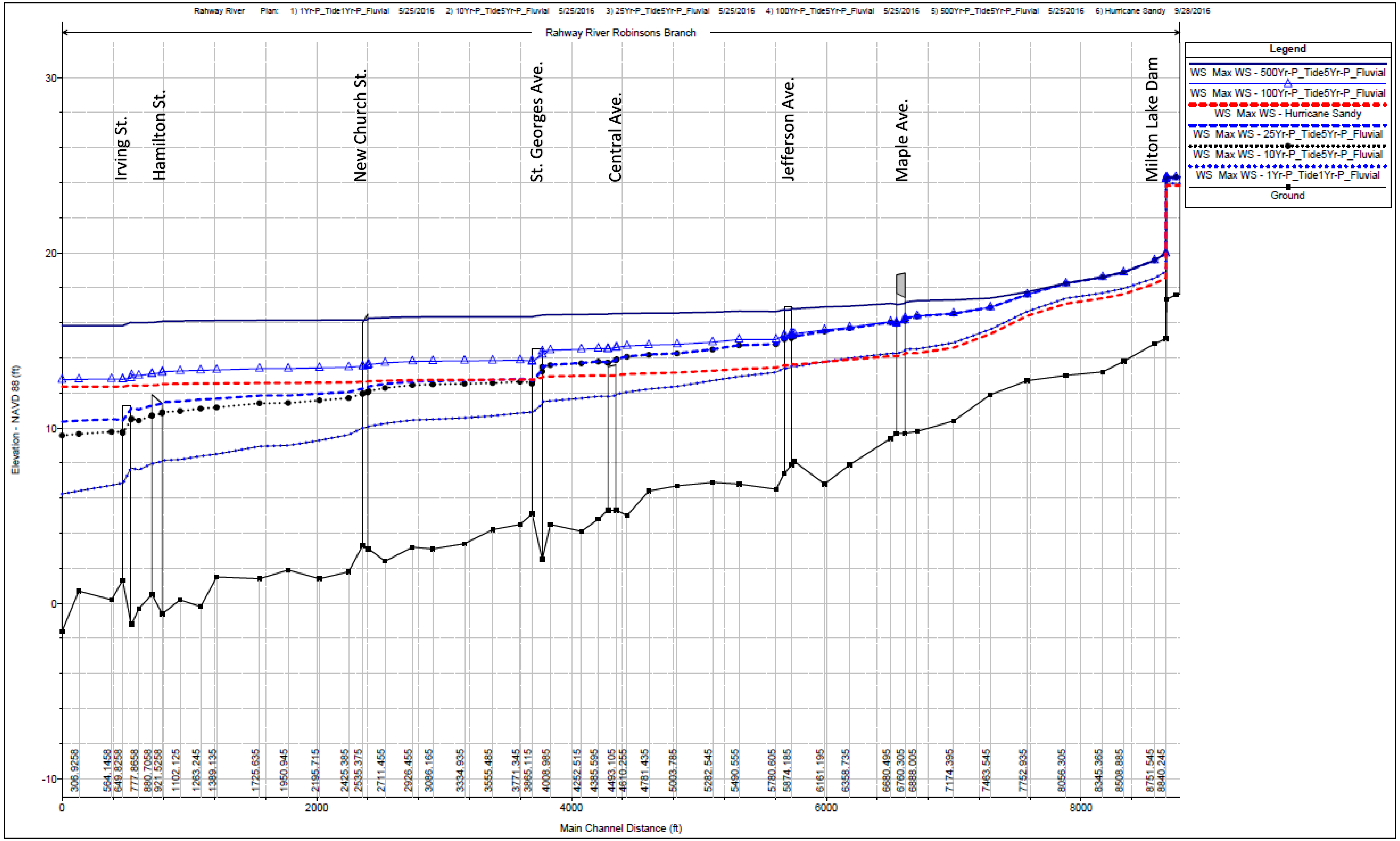


Figure 22: "Without project" condition computed water surface profile for Robinson's Branch from Milton Lake Dam to the confluence.

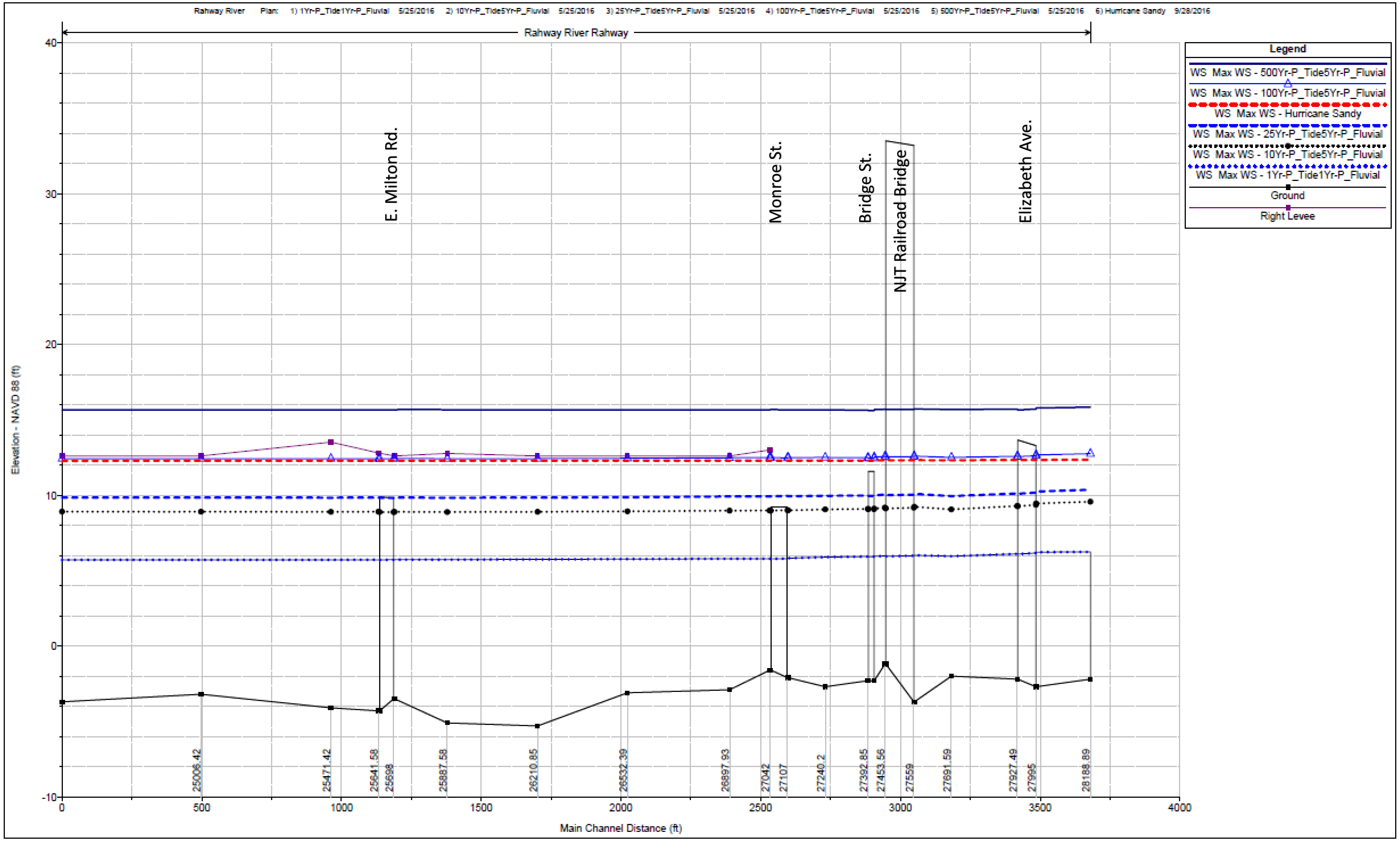


Figure 23: "Without project" condition computed water surface profile for Rahway River from Robinson's Branch Confluence to South Branch Confluence.

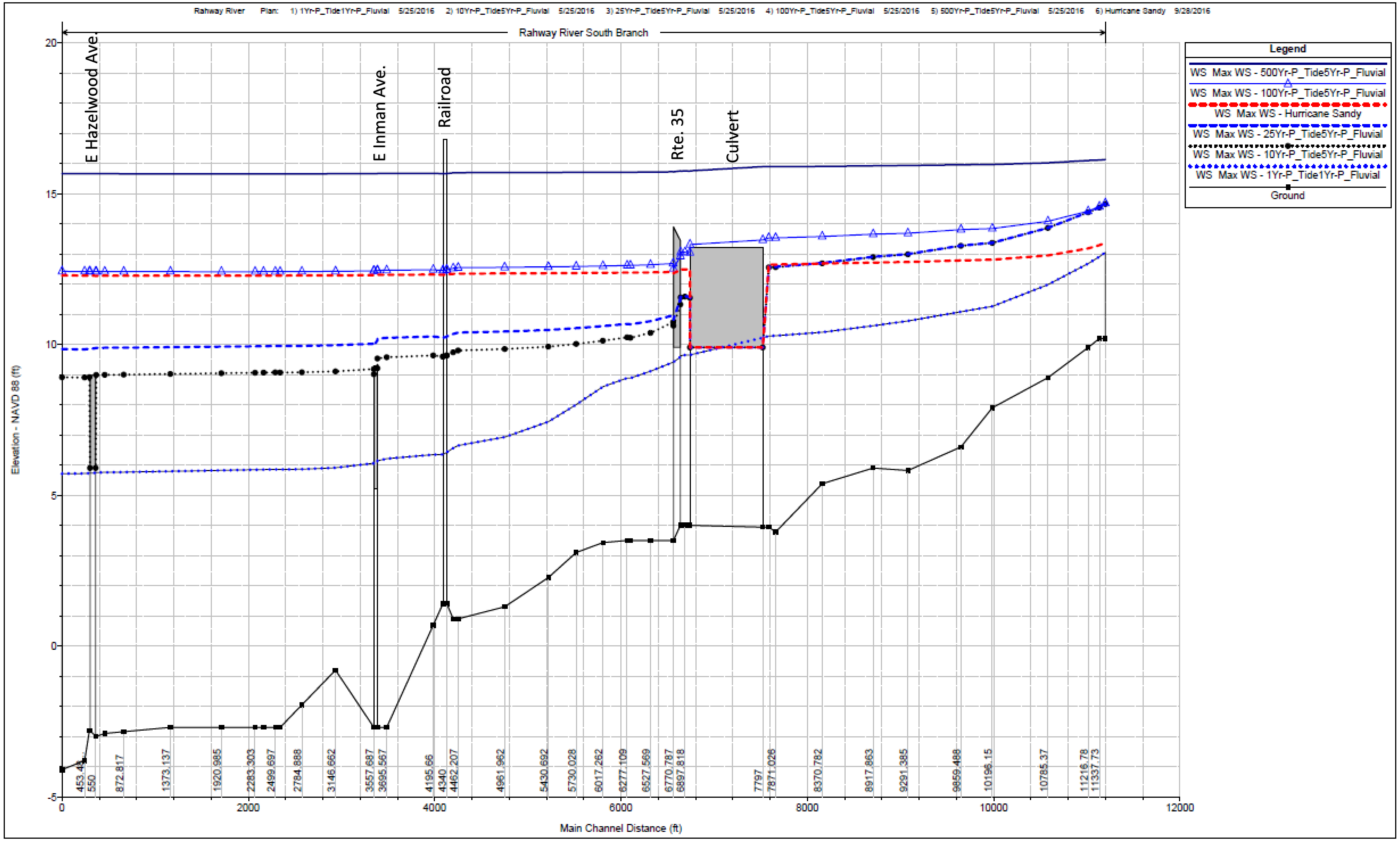


Figure 24: "Without project" condition computed water surface profile for South Branch.

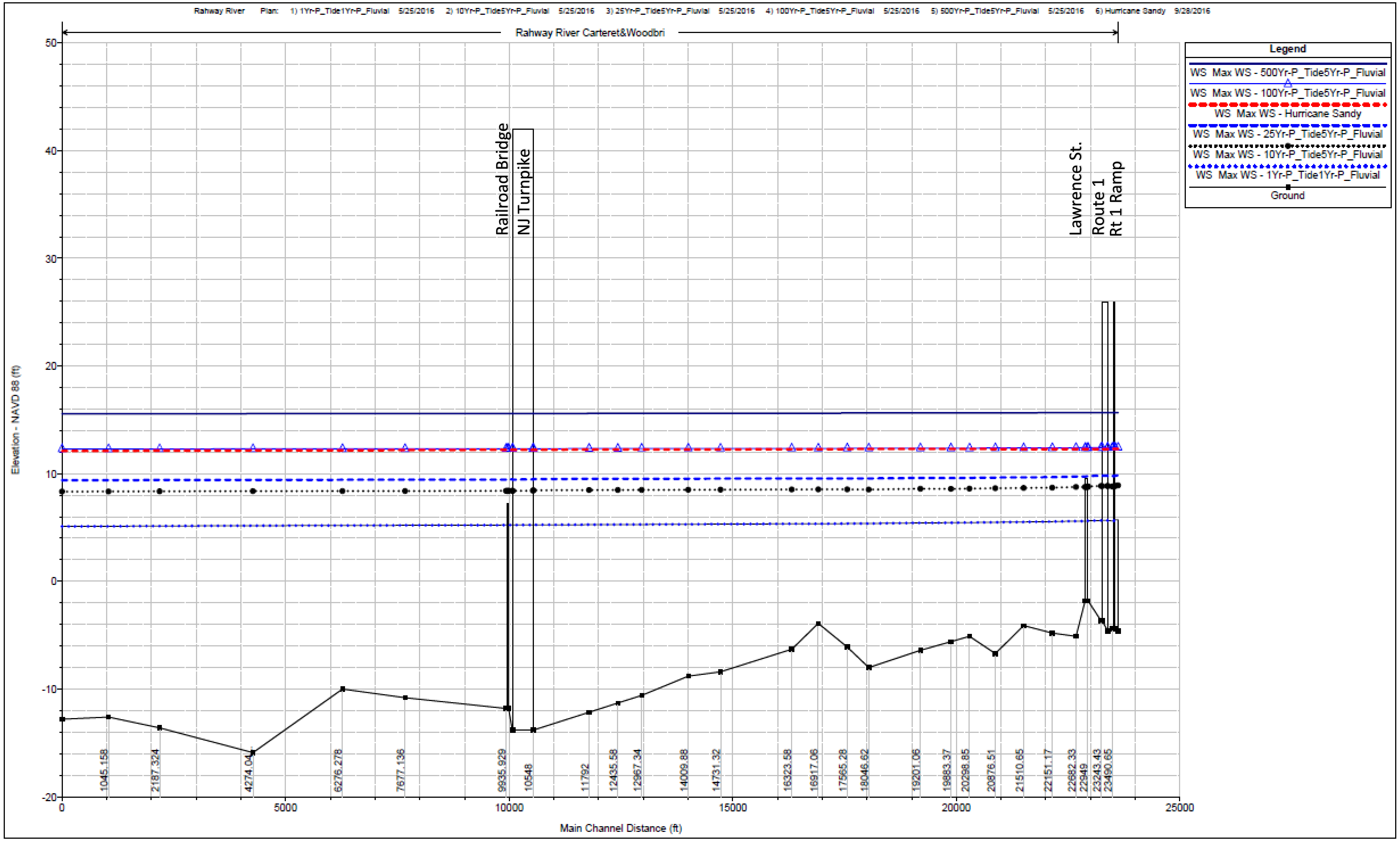


Figure 25: “Without project” condition computed water surface profile for Rahway River from South Branch Confluence to the Arthur Kill.

4.0 DEVELOPMENT AND EVALUATION OF ALTERNATIVES

4.1 General

The objective for the development of alternatives is to better manage the risk of flooding in the project area. The alternatives were focused on reducing flood risk in the areas of Linden, Carteret, and City of Rahway in South Branch and Robinson's Branch. The alternatives evaluated can be classified as No Action (same as Future without Project Conditions), Structural, and Non-structural alternatives. The Structural alternatives involve channel work, levees, floodwalls, tide gates, and/or a combination of the above. Non-structural measures are permanent or temporary procedures applied to a structure and/or its parts preventing or resisting damage from a flood event. Examples of such measures are dry flood proofing, wet flood proofing, elevating/raising structures, and buyouts. While ringwalls and ring levees are structural measures, they are included in the non-structural plans. Other alternatives were preliminarily evaluated and omitted due to low levels of performance, high cost, and/or potentially high environmental impacts.

4.2 No Action Alternative

This plan involves no Federal action to manage the flood risk in the Rahway River Basin. The no action alternative provides some indication as to what future conditions would be in the absence of the project. The No Action alternative avoids environmental and other impacts associated with implementation of other plans for flood risk management. The population, industries, and businesses are either stable or growing, indicating land-use and rainfall runoff increase. Sea level change analysis indicates an increase of 0.81 ft by the year 2068. Since future trends indicate higher flows and sea level rise, this plan fails to meet any of the study objectives. The result would be the continuation and future increase of flooding problems in the study area. This alternative represents the default condition if no other plan is recommended for further action and is a basis of comparison for all other plans.

4.3 Structural Alternatives

4.3.1 Alternative #1: Levees and Floodwalls

4.3.1.1 Alternative #1 – Summary and Features

This structural alternative consists of a combination of four (4) levee/floodwall segments, two (2) road closure gates, interior drainage structures, and channel modification. The improvements are located in Clark, Carteret, and Linden Townships. This alternative, at present conditions, is likely to have a 0.01 annual exceedance probability in the protected areas. See Figure 26 for the overview of the alternative and Figure 27 and Figure 28 for the plan layout of each component.

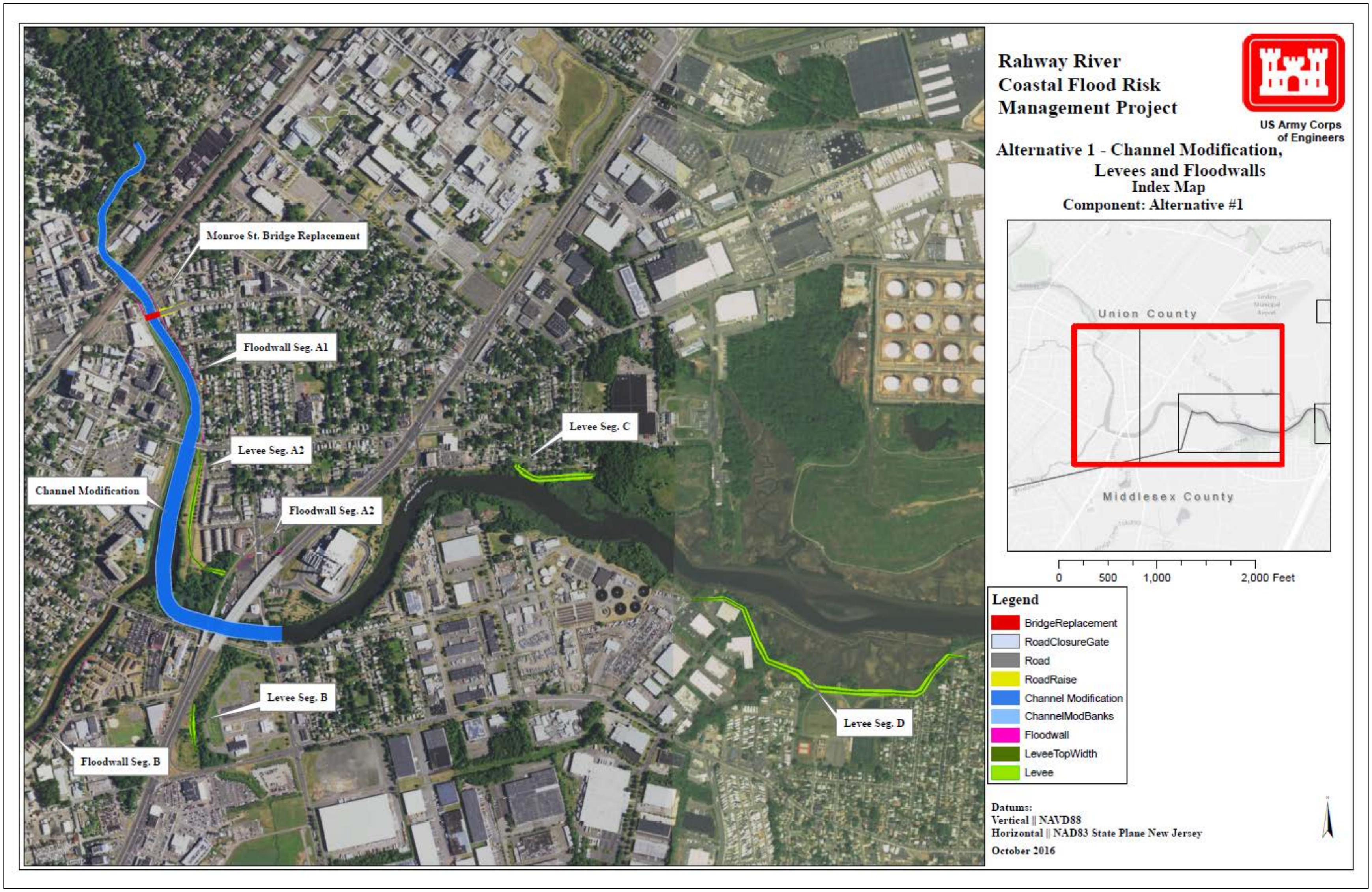


Figure 26: Alternative #1 Plan Overview

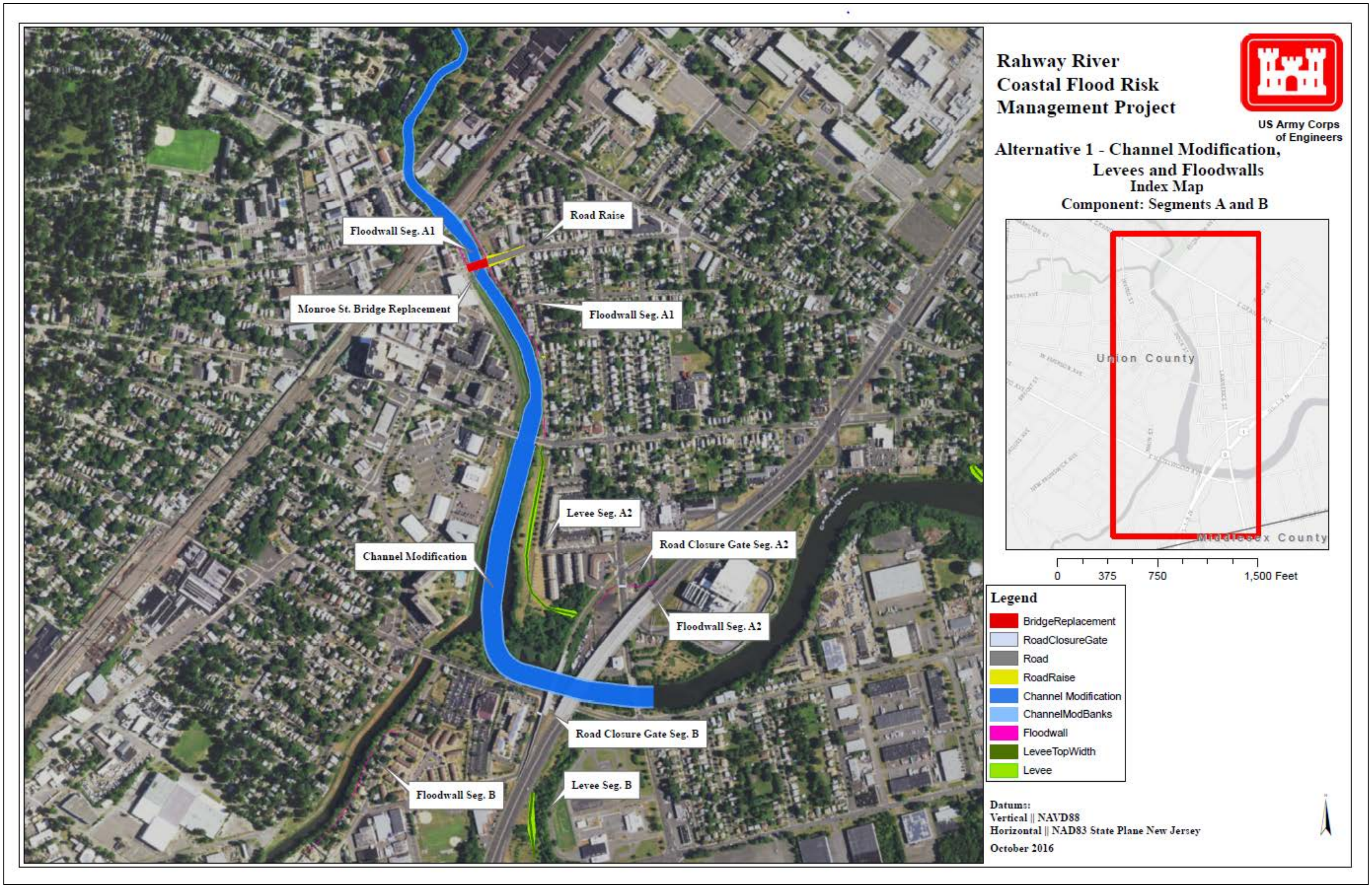


Figure 27: Alternative #1 Segments A and B layout.

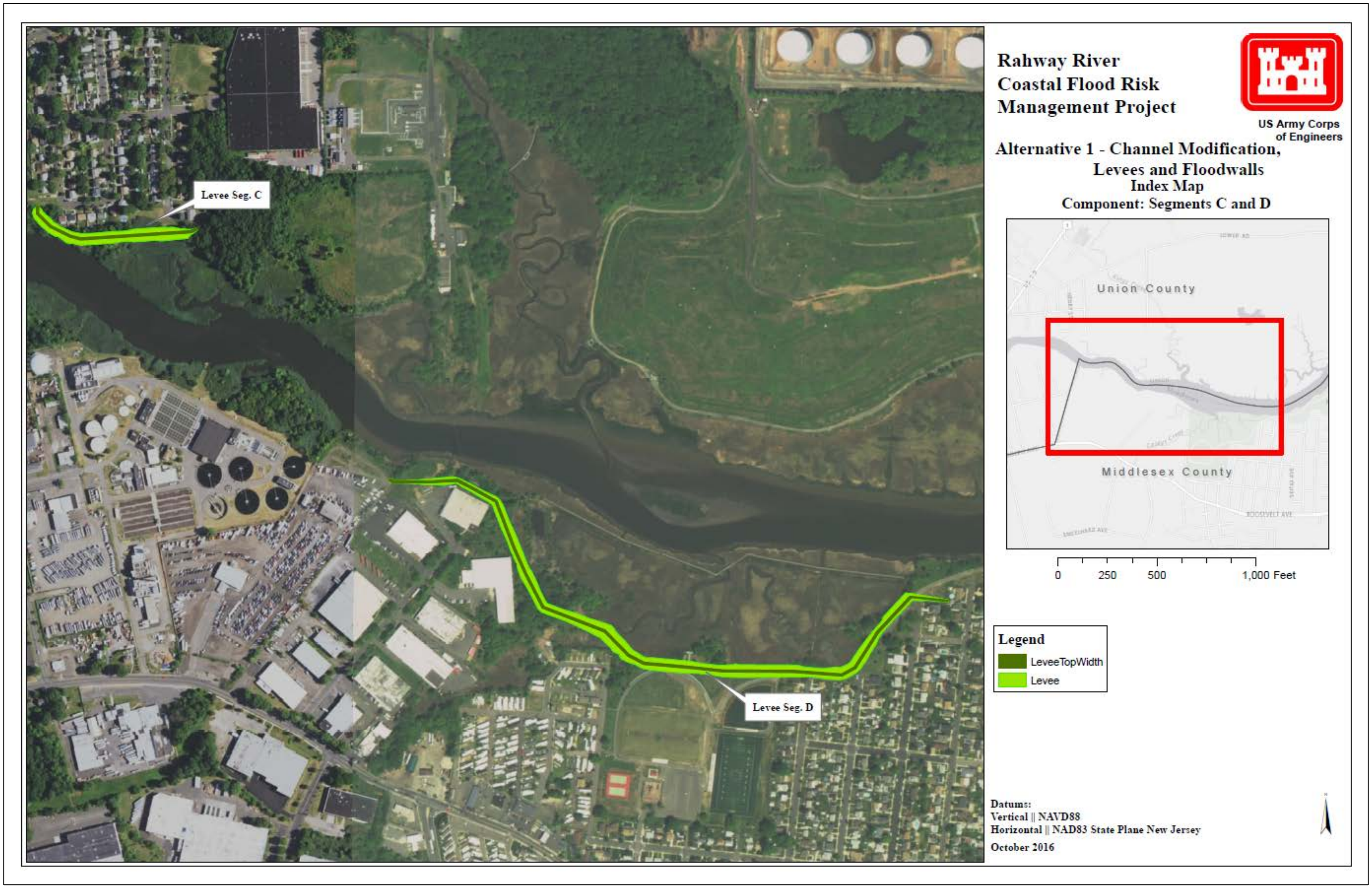


Figure 28: Alternative #1 Segments C and D layout.

The segments are the followings:

(1) Segment A: Levees and floodwalls, channel modification, bridge replacement, and road closure gate.

The upstream section, Segment A1, starts with “T-wall” floodwalls in both banks of the Rahway River near Bridge St. The left bank floodwall is approximately 325 ft. long while the right bank floodwall is approximately 210 ft. long, each at elevation 13.8 ft. NAVD’88. This section of floodwalls in both banks of the river ends at Monroe Street Bridge. The bridge shall be raised by 2.8 ft., and the left abutment shall be moved inland by 15 ft. As result of bridge modification, approximately 300 ft. of Monroe St. shall be raised by a maximum of 2.8 ft. The raised section of road ties in into the existing roadway surface at the intersection of Monroe St. and Essex St.

The left bank floodwall continues downstream towards Essex St. with a top elevation of 12.6 ft. NAVD ’88. The floodwall tie-in to Essex St. requires the road to be raised by approximately 1.5 ft. The raised section is approximately 150 ft. long and starts 50 ft. south the intersection of Essex St. and Washington St.

Segment A2 starts on the left bank of the Rahway River, approximately 150 ft. north of E. Milton Avenue Bridge. This section is a sheet pile wall with a maximum height of approximately 2 ft. Sheet pile ties into high ground at the recently modified bridge. A levee section starts downstream of E. Milton Avenue Bridge and ties into high ground on the abutments of the Edgar Rd. exit (Route 1). The levee is approximately 1,510 ft. long, with an average height of 4 ft., having a 12 ft. top width and one vertical to three horizontal (1:3) side slopes.

The final section of Segment A2 is a floodwall approximately 580 ft. long with an average height of 5.5 ft., located between the Route 1 exit and Route 1 itself. This section will also include a flood hydrostatic gate (road closure structure) approximately 65 ft. wide by 6 ft. high. The gate is located on Lawrence St. approximately 300 ft. south of the Hancock St. and Lawrence St. intersection.

Channel modification is necessary in order to mitigate for the impact (induced flooding) of bank encroachments caused by existing levees in the Rahway River and the additional features of Segment A. The upstream and downstream ends of channel modification are: 500 ft. upstream of W. Grand Avenue Bridge upstream of the confluence with Robinson's Branch and approximately 100 ft. downstream of Lawrence Street Bridge downstream of the confluence with the South Branch, respectively. The channel modification consists of a natural trapezoidal channel with one vertical to two and a half horizontal (1:2.5) side slopes. It is approximately 6,540 ft. long, totaling 60,000 cyd. of dredged material. The channel modification slope and bottom width are variable. The slope upstream of the NJ Transit Railroad Bridge is approximately 9.5 ft./mile and downstream is approximately 1.6 ft./mile, having bottom widths ranging from 35 ft. to 140 ft. This channel modification mostly removes high ground sections along the channel caused by high deposits of sediment. The channel modification will not only reduce upstream impacts but will also reduce flood risk during frequent fluvial events.

(2) Segment B: Levees, floodwalls and road closure gate.

This segment is a combination of levee and floodwall. The levee has a 12 ft. top width and one vertical to three horizontal (1:3) side slopes. It is approximately 640 ft. long with an average height of approximately 8 ft. This levee is located on the right side of Edgar Rd. just north of Randolph Ave.

The floodwall is a sheet pile approximately 5,700 ft. long with an average height of approximately 3.8 ft. The floodwall is located on the right bank of the South Branch, between the riverine and Leesville Ave. The upstream end of the floodwall is approximately 1,300 ft. downstream of E Inman Ave. and the downstream ends is approximately 600 ft. upstream of E Hazelwood Ave. Segment B also includes a flood hydrostatic gate (road closure structure). The dimension of the road closure structure is 40 ft. wide by 5 ft. high and it is located in the north end of Capobianco Plaza Rd.

(3) Segment C: Levee.

This levee segment is 890 ft. long with a 12 ft. top width and one vertical to three horizontal (1:3) side slopes. The average height of approximately 7.5 ft. The levee is located on the left bank of the Rahway River, approximately one mile downstream of the confluence with the South Branch. The upstream end is located by Beacon St., continues downstream, and ties in into high ground approximately 150 ft. downstream of Wall St.

(4) Segment D: Levee.

This levee segment is 3,360 ft. long with a 12 ft. top width and one vertical to three horizontal (1:3) side slopes. The average height is approximately 7.5 ft. The levee is located next to the right bank of the Rahway River, approximately 1.2 mile downstream of the confluence with the South Branch. The upstream end is located at the industrial/commercial area by Ardemore Ave., continuing downstream to Dorothy St.

4.3.1.2 Alternative #1 – Hydraulic Analysis

The design height of hydraulic features will be at elevation 12.6 ft. NAVD '88, consistent with the existing levees in the City of Rahway. Levees, floodwalls, and road closure structures were designed to this height and evaluated based on their performance during the 0.01 AEP hypothetical event in HEC-RAS. The bank encroachment caused by existing levees in the Rahway and the proposed levees in Segment A induced flooding upstream during model simulation, especially during significant fluvial events. Channel modification was necessary to reduce WSEs to "without project" condition levels. This channel modification will not only reduce upstream impacts but will also reduce flood risk during frequent fluvial events, providing additional benefits to City of Rahway and Clark Township. Figure 29 through Figure 33 show the present "with project" tidal WSE profiles.

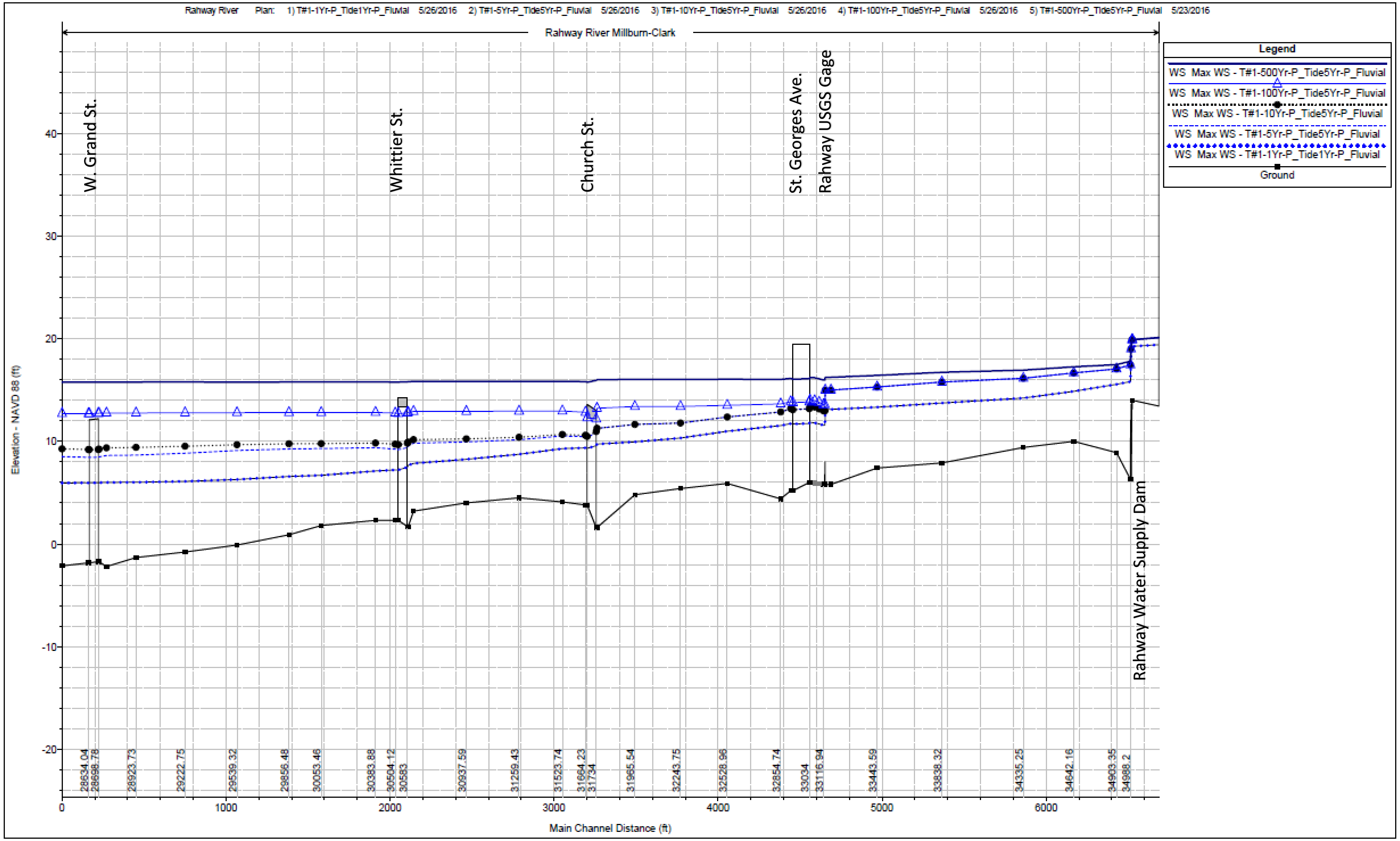


Figure 29: Alternative #1 computed water surface profile from Rahway Water Supply to Robinson's Branch Confluence.

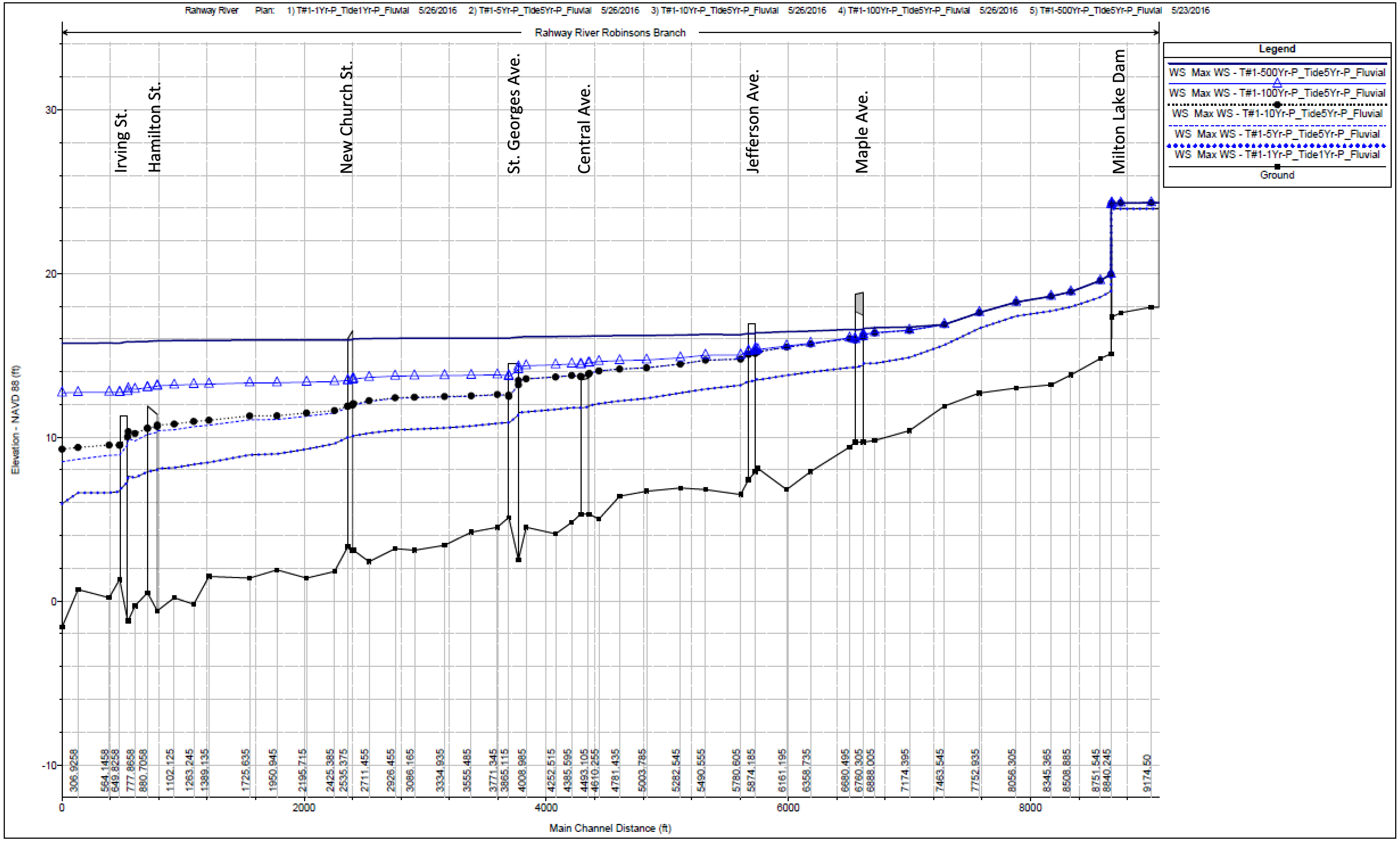


Figure 30: Alternative #1 computed water surface profile for Robinson’s Branch from Milton Lake Dam to the confluence.

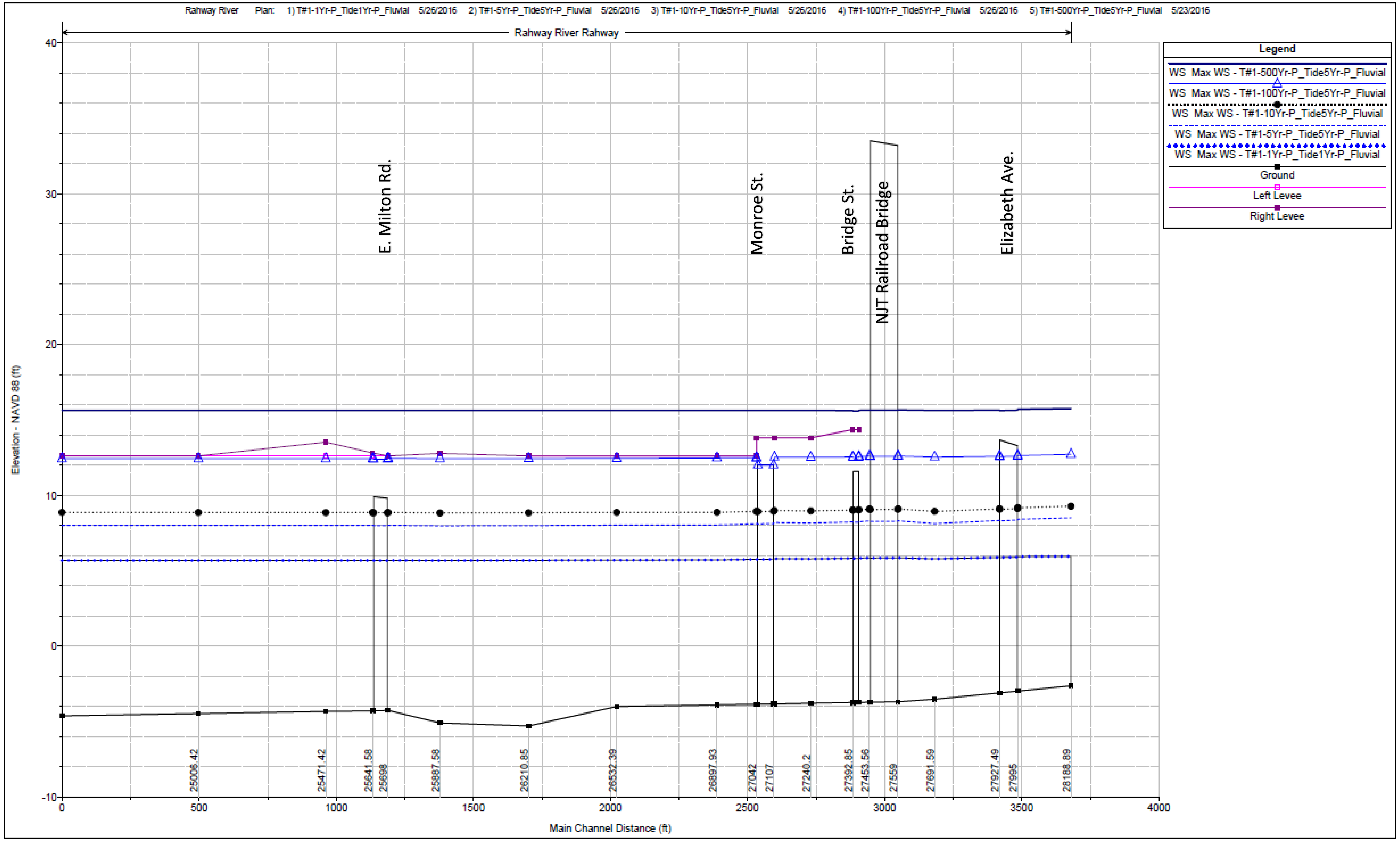


Figure 31: Alternative #1 computed water surface profile for Rahway River from Robinson's Branch Confluence to South Branch Confluence.

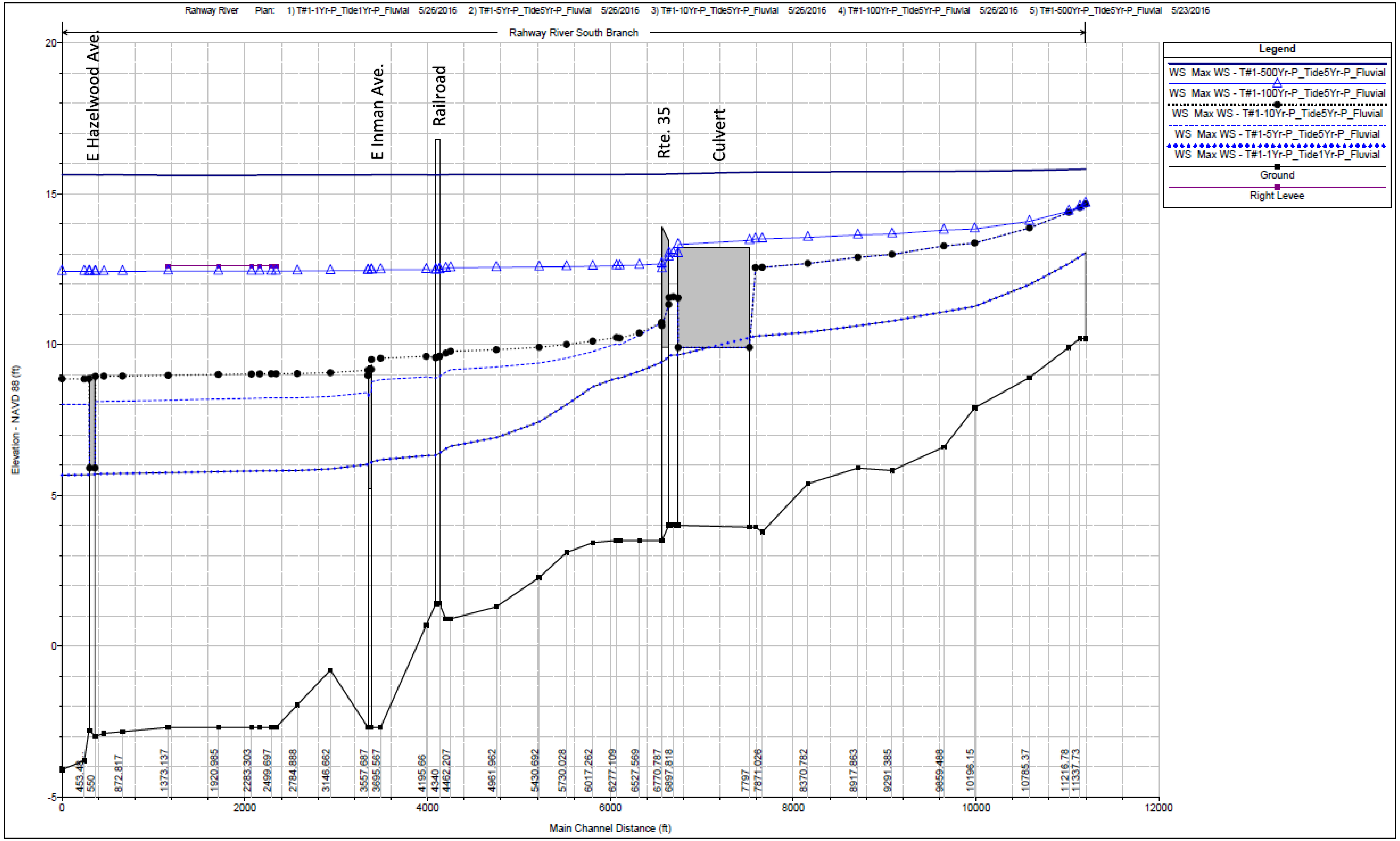


Figure 32: Alternative #1 computed water surface profile for South Branch.

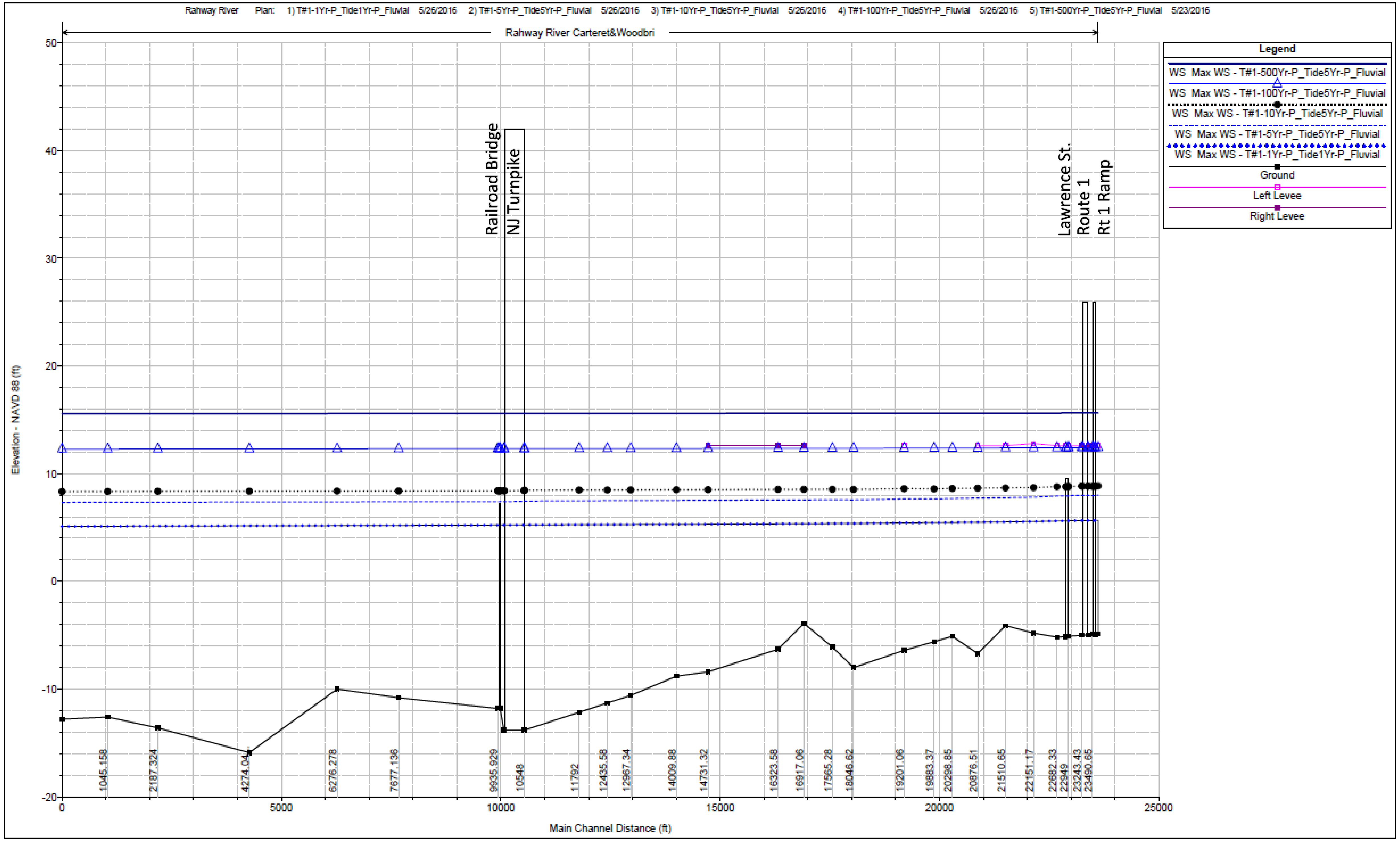


Figure 33: Alternative #1 computed water surface profile for Rahway River from South Branch Confluence to the Arthur Kill.

4.3.2 *Alternative #2: Surge Barrier*

4.3.2.1 Alternative #2 - Summary and Features

This structural alternative's main feature is a surge barrier consisting of tide gates and a pumping station at the New Jersey Turnpike Bridge. A surge barrier is a specific type of floodgate designed to prevent a storm surge from flooding the area behind the barrier up to a specified design height. The barrier would be upstream of the bridge, i.e. to the west of the Turnpike, spanning across the width of the river from Carteret to Linden. Additional channel modification, levees and floodwalls in both Carteret and Linden, and closure structures complete the plan. This alternative is likely to have a 0.01 annual exceedance probability. See Figure 34 for the overview of the alternative and Figure 35, Figure 36, and Figure 37 for the plan layout of each component.

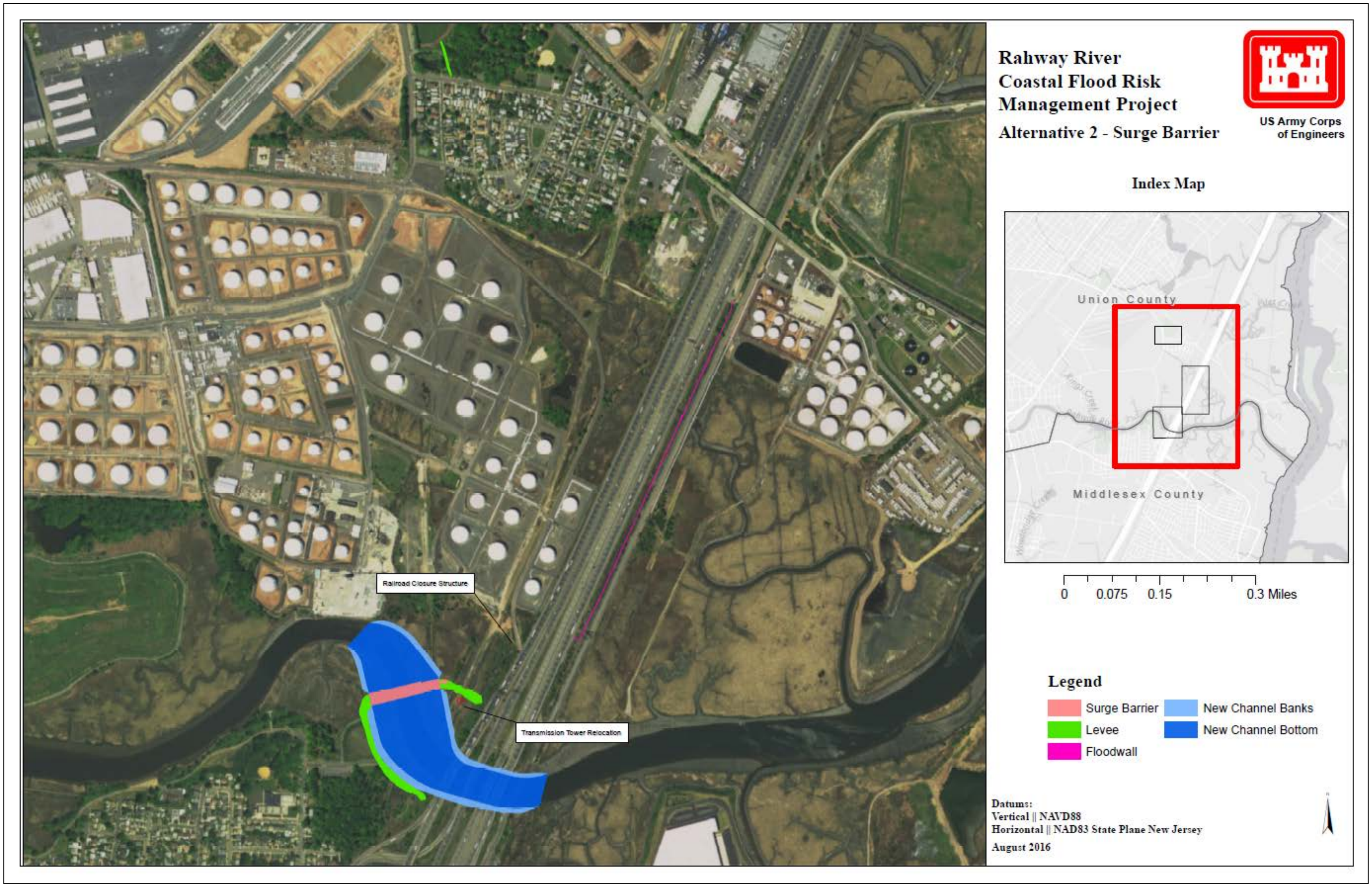


Figure 34: Alternative #2 Plan Overview.

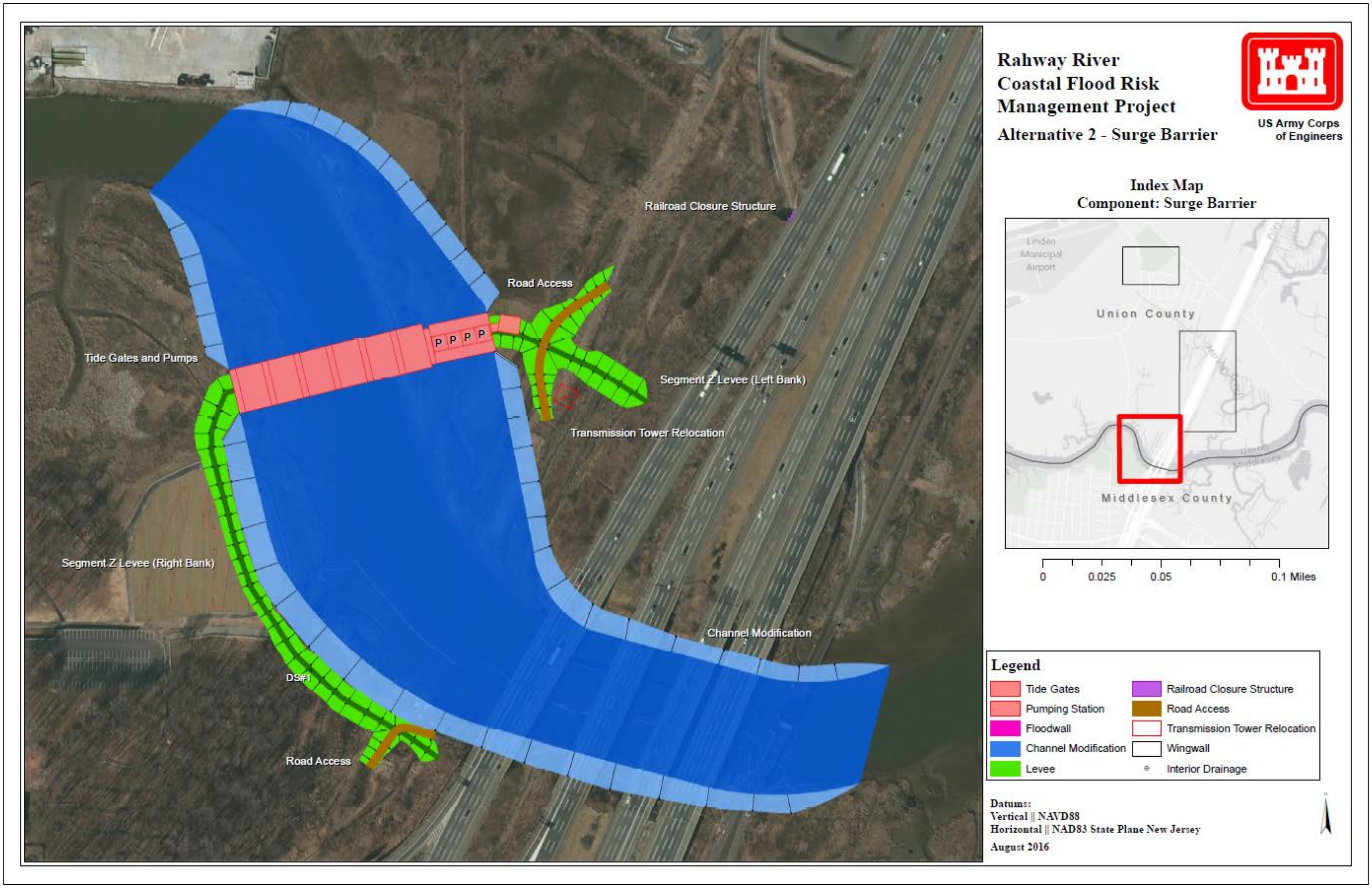


Figure 35: Alternative #2 Surge Barrier (Gates, Pumps, Levees, Channel Modification) layout.

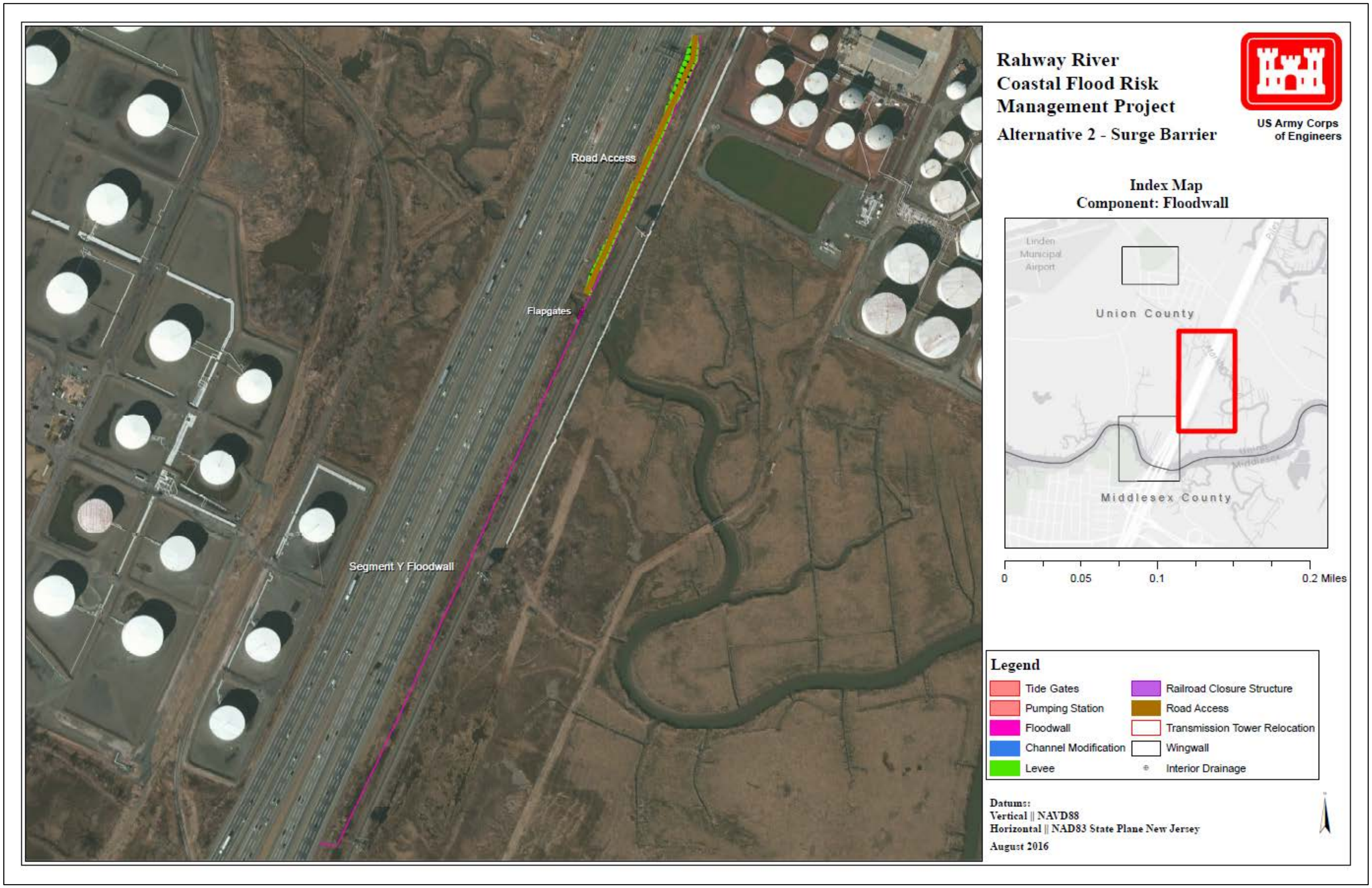


Figure 36: Alternative #2 Floodwall along Turnpike Northbound layout.

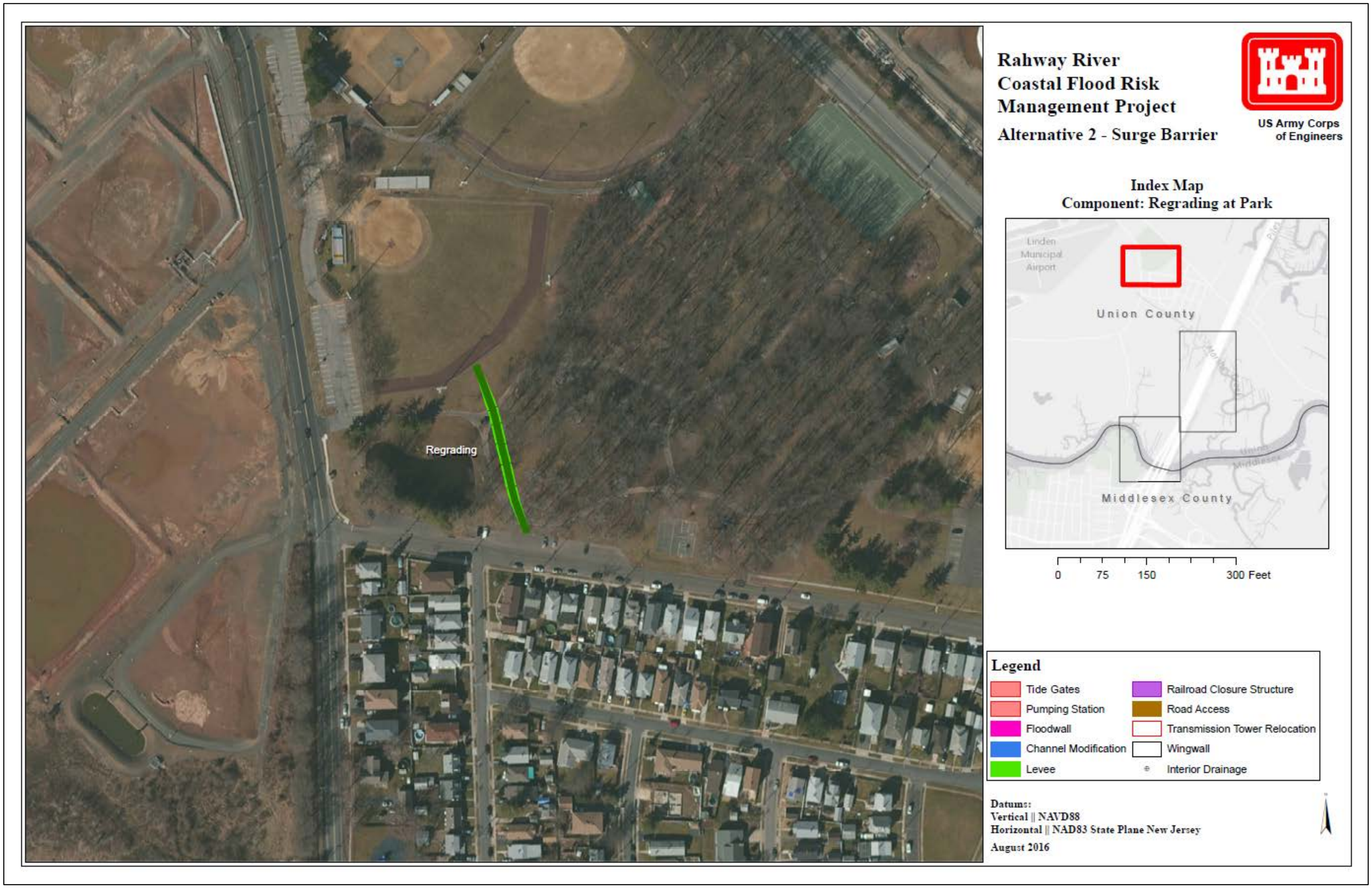


Figure 37: Alternative #2 regrading at Memorial Field Park layout.

The surge barrier is located approximately 775 ft upstream of the New Jersey Turnpike with a design elevation of 13 feet NAVD ‘88. It includes:

(1) Six tainter gates allowing navigable passage,

(2) A pumping station with four pumps at a total capacity of 2.7 million gpm,

(3) Levee tie-ins to high ground (the turnpike) on the left and right banks, and

(4) Channel modification at the surge barrier for a length of approximately 2,000 ft.

The surge barrier contains six tainter gates, each 60 ft wide and 30 ft tall from invert to top of gate. Gates will be open during normal tide conditions and fluvial events. During tidal events, the gates will close during a rising tide as long as the headwater (landside) has a lower WSE than the tailwater (ocean-side). The pump station is located on the left bank and will tie into the line of protection of the gate components. It contains four 1,500 cfs pumps with a total capacity of 6,000 cfs, or 2.7 million gpm. Pump operation is necessary when the gates are operating so that damage is not incurred to structures upstream of the barrier. A more detailed explanation of pump design is in Section 4.3.2.2 Alternative #2 - Hydraulic Analysis

Levees on the left and right banks of the surge barrier will tie into the NJ Turnpike. Levees will have a top width of 12 ft and a 1 vertical to 3 horizontal (1:3) side slope. Levee length on the left bank is approximately 380 ft with a design height of 13 ft NAVD ’88, having a maximum exposed levee height of 11 ft. Levee length on the right bank is approximately 1,040 ft with a design height of 13 ft NAVD ’88, having a maximum exposed levee height of 11 ft. The right bank levee includes an 18 inch diameter interior drainage structure.

The surge barrier involves approximately 2,000 ft of channel modifications, totaling 322,000 cubic yards of dredged material. Modification begins approximately 500 ft upstream of the barrier to just downstream of the railroad bridge. Channel modification includes a new alignment of the left bank at the pump station, rectangular cuts immediately upstream and downstream of the barrier, trapezoidal cuts along the length of the channel with a 1:3 side slope, and 1:5 side slopes under the Turnpike and railroad bridges. The channel bed slope will be constant at a natural slope of 0.0013 ft/ft.

The remainder of the project will include:

(1) A floodwall along New Jersey Turnpike Northbound,

(2) Regrading approximately 300 linear ft of Memorial Field Park in Linden, NJ to an elevation of 13 ft NAVD ‘88,

(3) Three manual flapgates in the floodwall on the Northbound side of the Turnpike at Marshes Creek,

(4) A 6 ft high swing gate railroad closure structure on the Southbound side of the Turnpike by the Citgo oil tank farm, and

(5) Relocating the transmission tower on the left bank approximately 130 ft toward the left bank levee, away from the river.

The floodwall component of the alternative is located along the northbound side of the Turnpike between the highway and the railroad running parallel. Length of the floodwall is approximately 3,090 ft with design height 13 ft NAVD ’88 and having a maximum exposed height of 13 ft. The floodwall includes three 8 ft diameter manually operated flapgates at the Marshes Creek outlet. The flapgates will be open during normal conditions as to not affect the tidal environment.

Regrading at Memorial Field Park is minor but necessary to distinguish the Rahway River basin from the Arthur Kill-Upper Bay basin, including Elizabeth River and Morses Creek. The one foot regrading will prevent elevated water levels in the nearby basin from causing flooding in the Rahway tidal area of study.

4.3.2.2 Alternative #2 - Hydraulic Analysis

This alternative was developed based on a design height of 13.0 ft NAVD ’88, which is approximately the future conditions 0.01 AEP event. All levees, floodwalls, and tide gates were designed to this height and evaluated based on their performance during the 0.01 AEP hypothetical event in HEC-RAS.

The pump station was designed based on guidance from EM 1110-2-1413 Hydrologic Analysis of Interior Areas (chapter 3), which describes the “minimum facility” of flood relief for storm drainage. Pump necessity was first determined based on the storage-elevation curve of the area of study (Figure 38). Given the lack of natural detention storage and the parallel functionality of a levee to a surge barrier, the minimum facility design concept was applied to pump capacity design. The language of the EM suggests that flooding “with project” cannot be any worse than “without project” conditions. In the tidal area of study, the “without project” WSEs cause damages beginning at approximately 5.25 ft NAVD ’88, which occurs below the 0.5 AEP (2-yr) event. Damages can be defined as street flooding and structures completely surrounded by inundation at this WSE. The goal of pump design is to have enough capacity and efficiency to lower “with project” WSEs to “without project” WSEs. The pump was designed to decrease WSEs to 5.25 ft NAVD 88’ at approximately the 0.02 AEP (50-yr) tidal event or less.

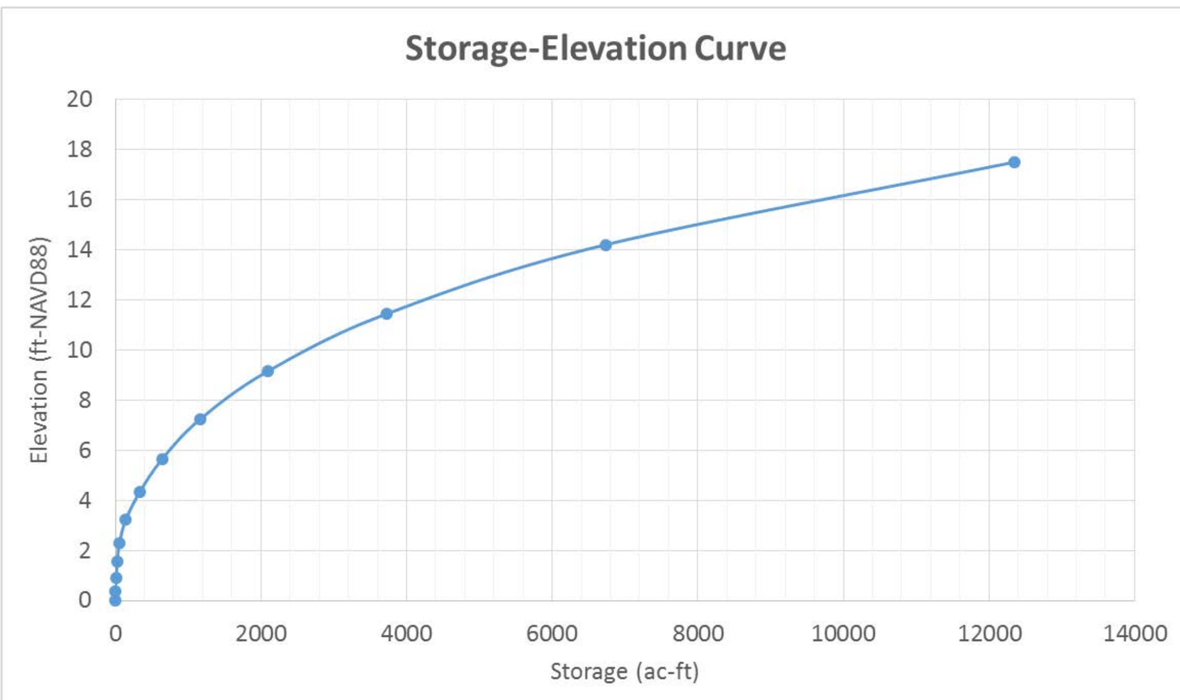


Figure 38: Storage-Elevation curve showing capacity of Tidal area of study.

HEC-RAS hydraulic runs were used to create stage-frequency curves in order to determine the capacity and ramp-up/down elevations for pump operation. The feasibility stage pump capacity design was determined to be four 1,500 cfs pumps, having a total capacity of 6,000 cfs. Figure 39 through Figure 43 show the present “with project” tidal WSE profiles. Refer to Figure 21 through Figure 25 in Section 3.4 “Present and Future Conditions – Hydraulic Profiles” for the present “without project” tidal WSE profiles.

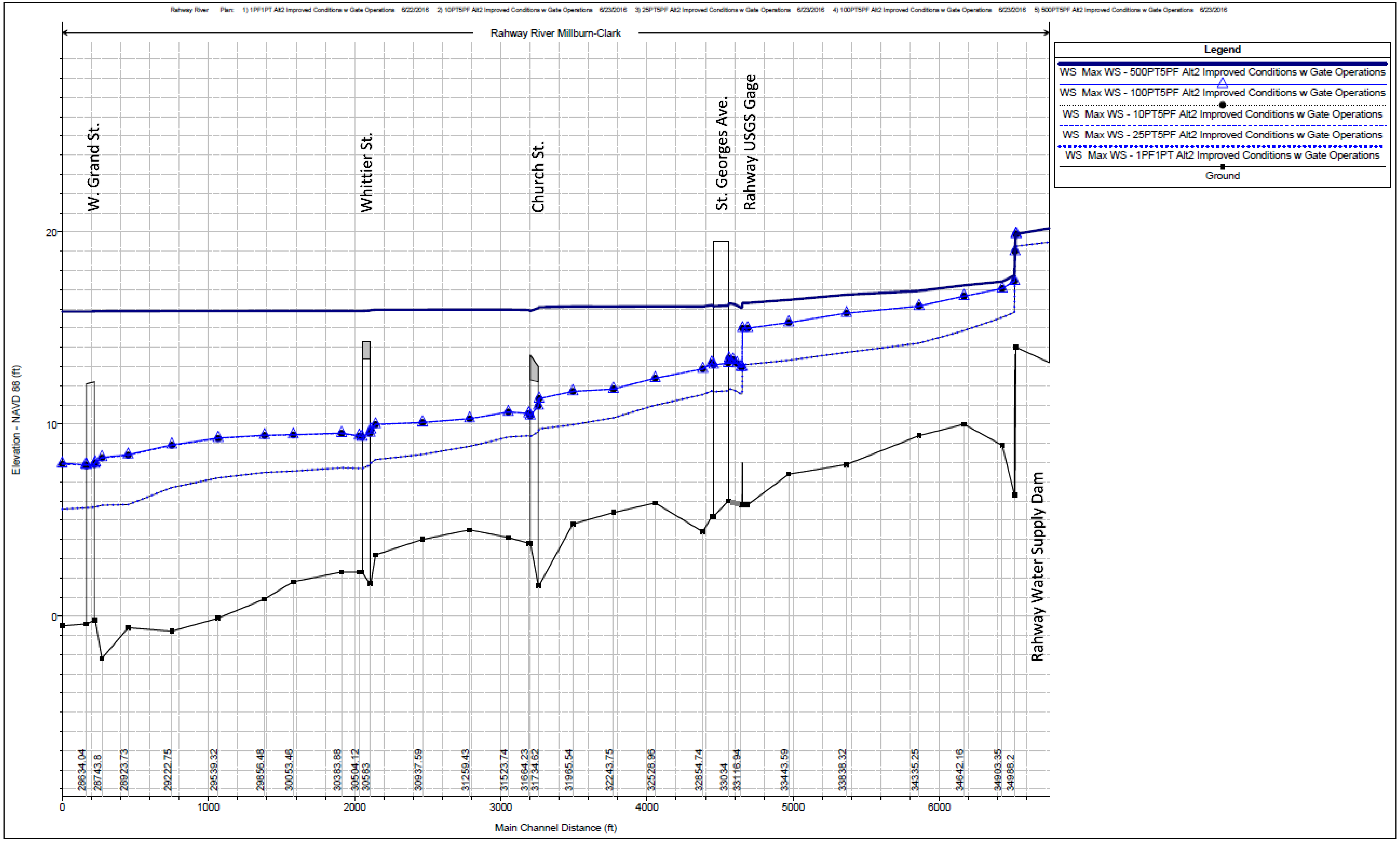


Figure 39: Alternative #2 computed water surface profile from Rahway Water Supply to Robinson's Branch Confluence.

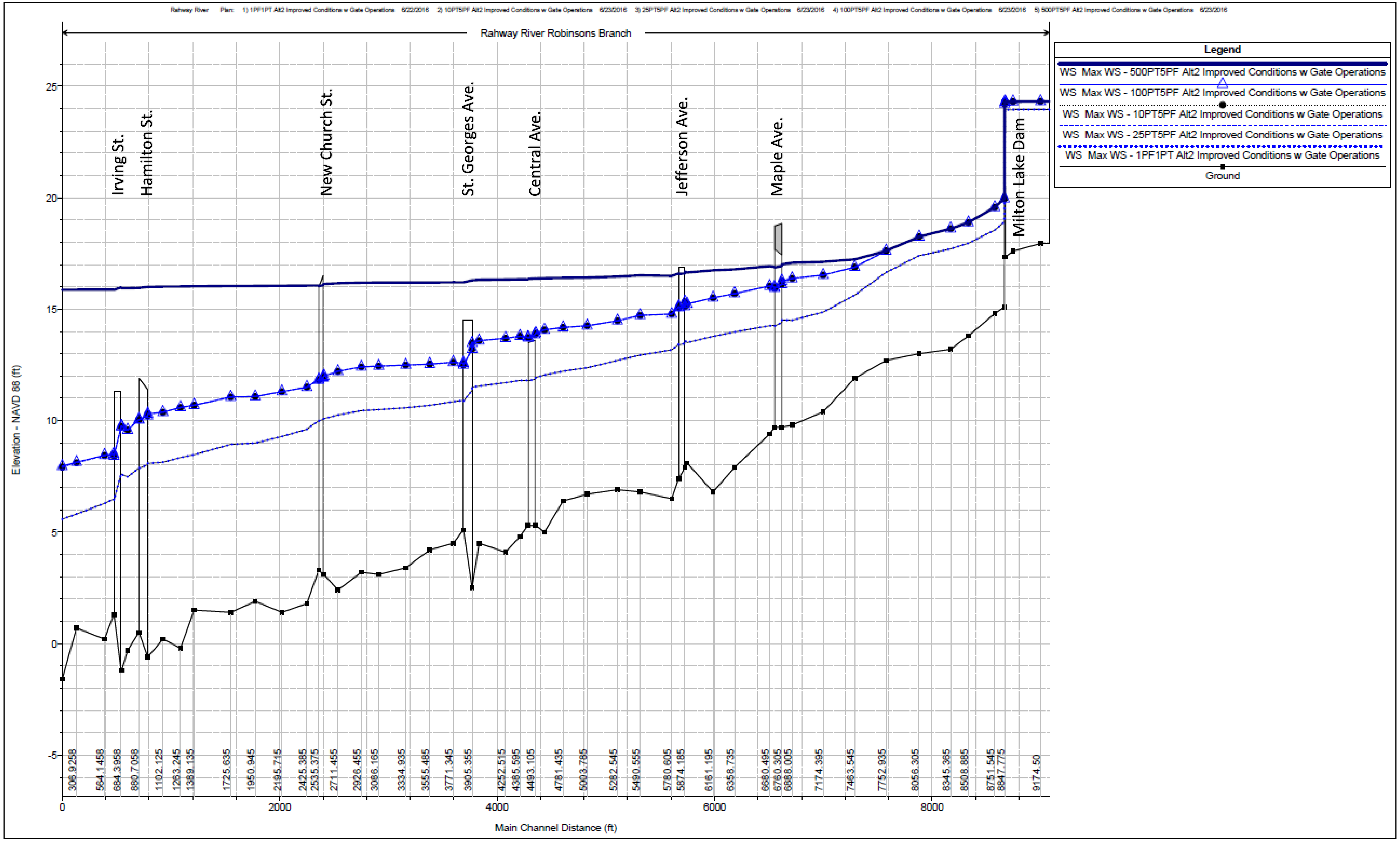


Figure 40: Alternative #2 computed water surface profile for Robinson's Branch from Milton Lake Dam to the confluence.

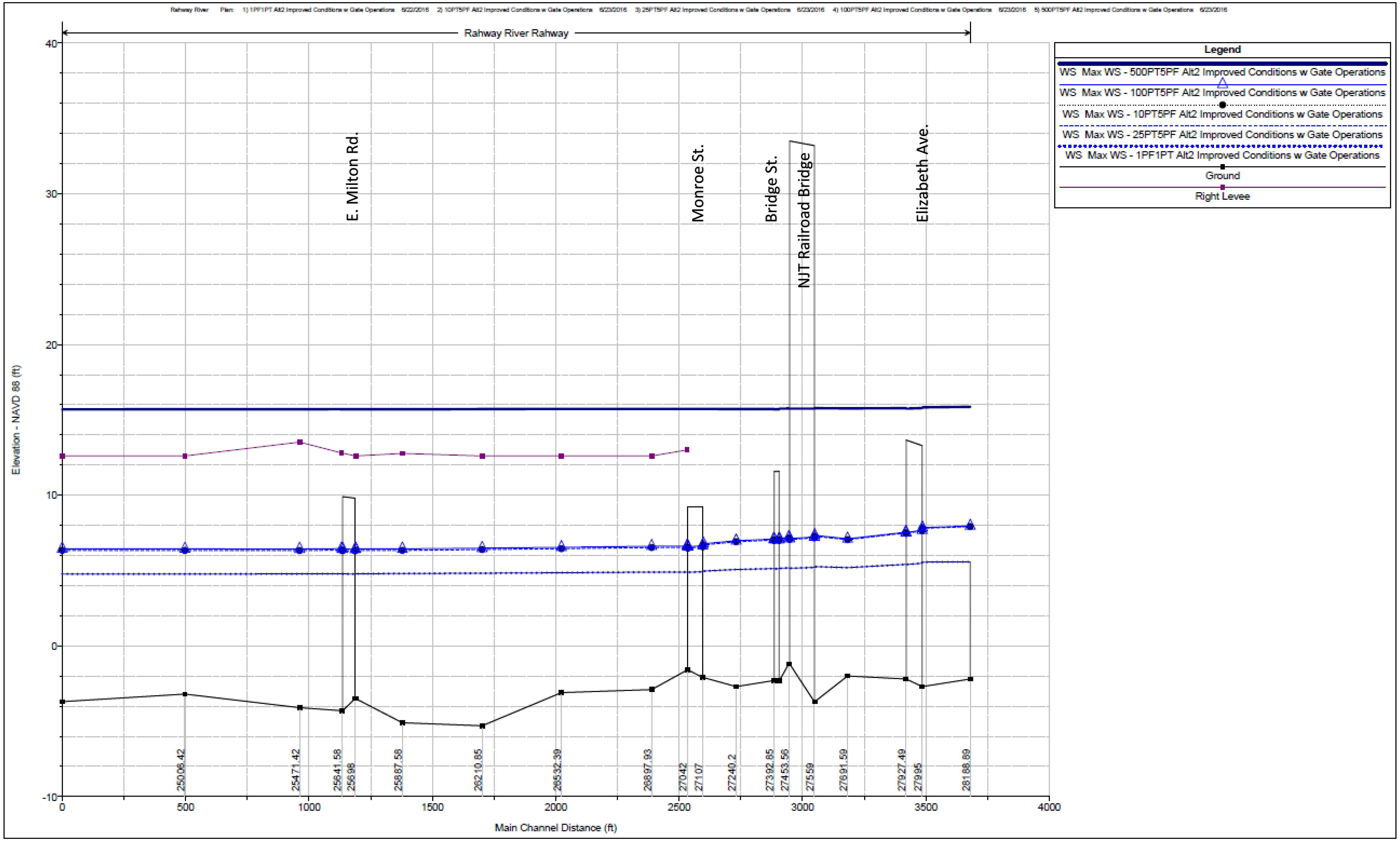


Figure 41: Alternative #2 computed water surface profile for Rahway River from Robinson's Branch Confluence to South Branch Confluence.

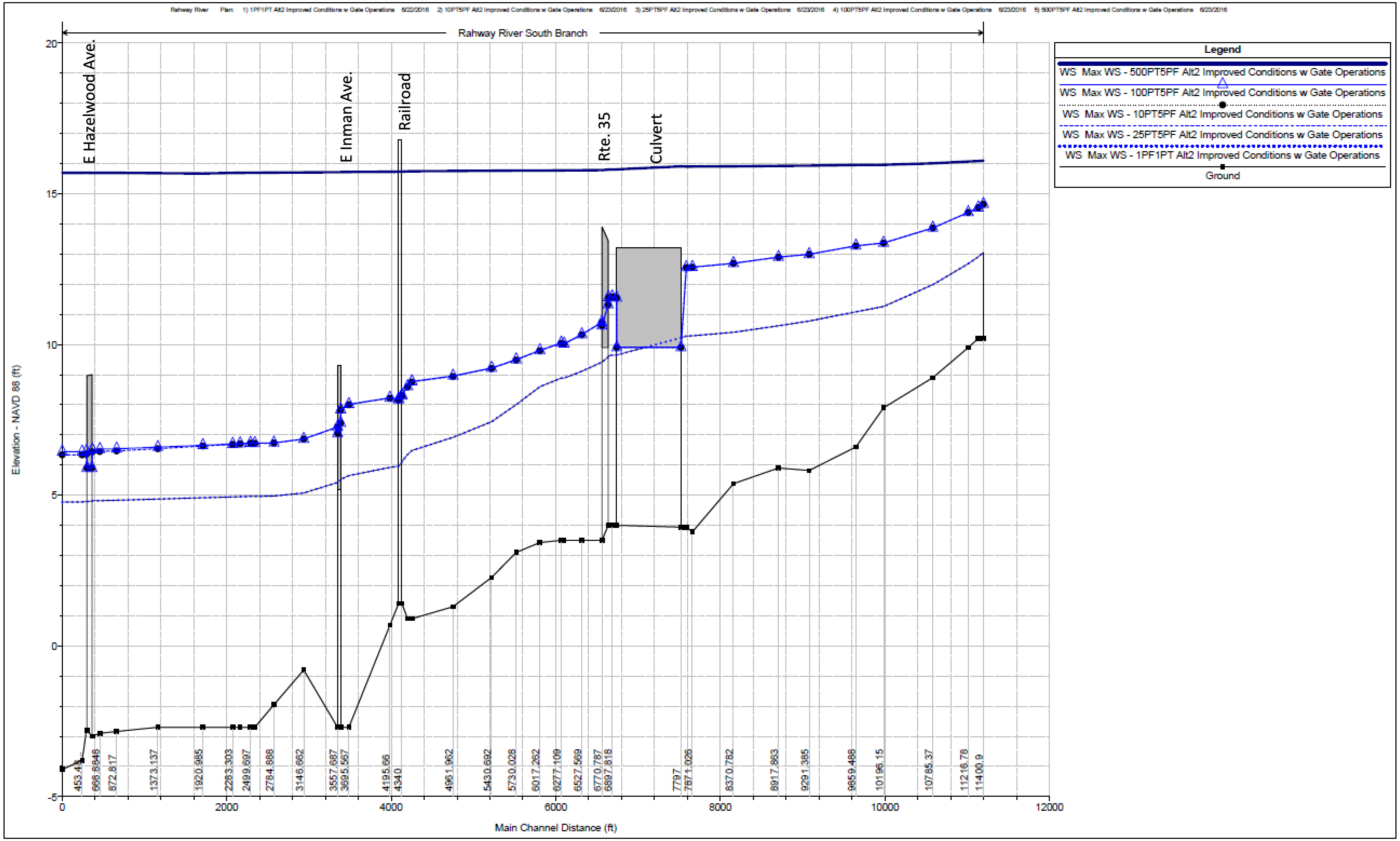


Figure 42: Alternative #2 computed water surface profile for South Branch

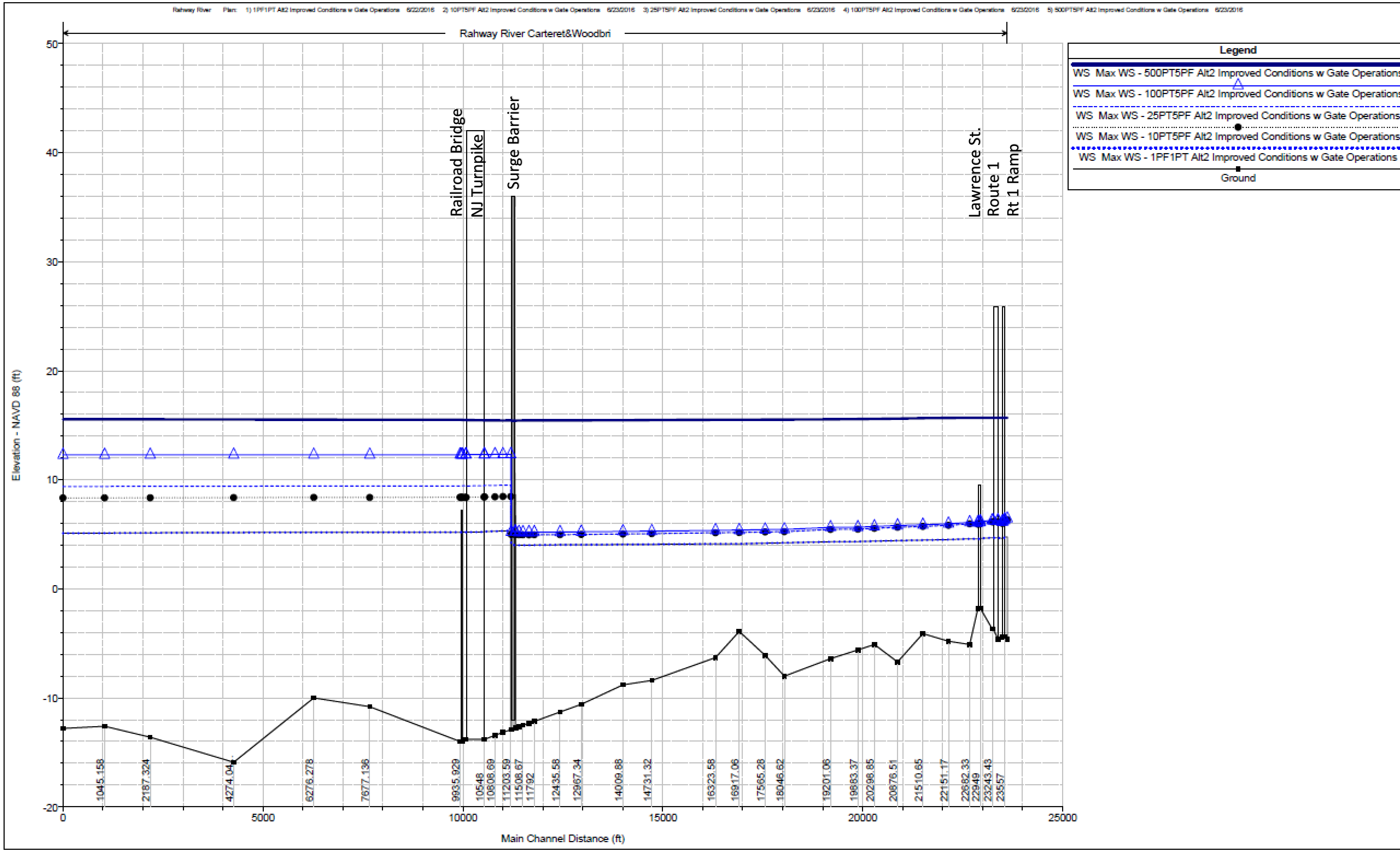


Figure 43: Alternative #2 computed water surface profile for Rahway River from South Branch Confluence to the Arthur Kill.

4.4 Non-Structural Alternatives

4.4.1 Description of Non-Structural Treatment Methods

Non-structural flood risk management measures are authentic techniques for reducing accountable flood damages within floodplains. These techniques mainly consist of measures such as relocation, acquisition, flood proofing (wet/dry), raising/elevation, flood warning system, flood emergency preparedness plans, and public education. Some of the measures (i.e., flood proofing and raising) maintain residential, commercial, and industrial areas, reducing flood damages through modifications of the existing structures. Other treatments are more invasive non-structural measures like buying and removing low-lying high risk properties from the floodplain. These non-structural measures are generally used for the reduction of damages for frequently flooded properties (i.e., 0.04 AEP (25 year event) or less). For areas or structures where non-structural measures are not appropriate, structural measures such as ring levees and ringwalls are considered. These structural treatments however have the potential to affect the floodplain and require further hydraulic analysis.

The non-structural measures to be considered in the feasibility study of the Rahway River Tidal project include dry flood proofing (e.g., sealing basement windows on residential properties), wet flood proofing, elevation (raising buildings), barriers (ring floodwalls/ring berms), and pump replacements. Relocations and acquisitions (buyouts) were not considered in this analysis. Buyouts are considered where the cost of the treatment exceeds the cost of the buyout. This evaluation occurs in the later design stages.

- *Dry Flood Proofing*. Dry flood proofing measures allow flood waters to reach the structure but diminish the flood threat by preventing the water from getting inside the structure. Dry flood proofing measures considered in this screening make the portion of a building that is below the flood level watertight through attaching watertight membranes and installing closure structures in doorway and window openings, referred to as sealants and closures.
- *Dry Flood Proofing with Liquid Storage Tank Modifications.* Liquid storage tanks are subject to floatation during flooding. The International Building Code Appendix G: Flood Resistant Construction specifies that tanks, if not located above the design flood elevation, are to be designed and anchored to prevent flotation, collapse, or lateral movement from

hydrostatic loads (including the effect of buoyancy). All tank inlets and vents not above the design flood elevation are to be fitted with covers designed to prevent the inflow of floodwater and the outflow of tank contents, and that these inlets and vents be properly anchored. Anchoring involves installing anti-flotation measures, elevating sensitive equipment, and adding back-up power sources such as generators. Common operational measures include pre-filling the tanks prior to the high water storm event. If an above-ground tank is no longer in use, holes may be cut in the tank to allow the flow of water in and out preventing floatation. In this study, liquid storage tanks were found in conjunction with masonry buildings with slab foundations for which dry flood proofing was appropriate.

- *Wet Flood Proofing*. Wet flood proofing measures allow flood water to get inside lower, non-living space areas of the structure via vents and openings in order to reduce the effects of hydrostatic pressure and, in turn, reduce flood-related damages to the structure's foundation. When a basement is involved, it is filled with compacted earth for foundational stability. Wet flood proofing also involves elevating and/or protecting utilities.
- *Wet Flood Proofing by Pump Modification*. For storm water pump stations, continued operation during floods is desirable. Nonstructural measures involve replacing non-submersible pumps with submersible pumps, elevating sensitive equipment, and adding back up power sources such as generators. Pump controls and motors may be modified by replacing the pump shaft with a longer shaft and mounting the controls and motors at elevation above the design water surface elevation.
- *Elevation (Raise)*. Elevation involves raising the lowest finished floor of a building to a height that is above the flood level. In most cases, the structure is lifted in place and the foundation walls are extended up to the new level of the lowest floor. When a building is in poor condition, elevation is not feasible; in these cases demolition and rebuilding is recommended with the lowest finished floor above the flood levels. The elevation process differs for different foundation types: slab-on-grade, sub grade basement, walkout basement, raised (crawlspace) foundation, bi-levels/raised ranches, or split levels. In this

study, no structures were assumed to be elevated on piers, posts, or piles. Elevation was assumed to be feasible for structures having footprint of less than 3,000 sf.

- *Barriers (Ringwalls or Ring Levees)*. Barriers usually surround the building but are not attached, such as in the case of ringwalls, levees, or berms. It is used where the elevation is not feasible.

4.4.2 Non-Structural Analysis

Floodplains corresponding to a flood frequency of 0.01 and 0.2 annual exceedance probability (10 and 50 year events) were evaluated considering future conditions flows and boundary conditions. The analysis is based on fluvial-tidal joint-probability WSEs for these two events. Structures within the corresponding joint-probability floodplains were analyzed for treatment type based on structure type, condition, and build characteristics. Treatments for buildings were selected based on the USACE National Nonstructural/Flood Proofing Committee (NFPC) Flood Damage Reduction Matrix (March 2016).

4.4.2.1 Alternative #3a: 0.1 AEP Floodplain

Nonstructural measures were determined for approximately 577 structures (211 residential, 366 non-residential) contained in the 0.1 AEP (10-yr) floodplain. Results for the 0.1 AEP floodplain show that 257 structures will be treated and no treatment is recommended for the remaining 320 structures. This alternative requires approximately 33 ringwalls, each surrounding from one to 30 structures, varying in length from 300 to 3,500 linear feet, and varying in height above grade from 5 to 15 feet. All structures will be treated to an elevation of one foot above the 0.01 AEP event, including sea level change. Non-structural treatments for the 0.1 AEP floodplain plan are summarized in Table 8.

Additional flood risk management measures would be required to mitigate backwater during fluvially influenced events. The WSEs at the confluence of Robinson’s Branch and Rahway River down to Monroe Street were increased due to the constriction of flow by structural ringwalls. Proximity of ringwalls to the river, expansiveness of ringwalls, and minimal storage capacity contribute to the localized increases in flooding upstream. In this situation, mitigation for flooding was accounted for by including channel modification and bridge replacement at Monroe Street.

Channel modification comprised of deepening approximately 3,300 linear feet along mainstem Rahway River and widening the river near Monroe Street Bridge, for a total dredged capacity of approximately 17,000 cy.

4.4.2.2 Alternative #3b: 0.02 AEP Floodplain

Nonstructural measures were determined for approximately 983 structures (561 residential, 422 non-residential) contained in the 0.02 AEP (50-yr) floodplain. Results for the 0.02 AEP floodplain show that 597 structures will be treated and no treatment is recommended for the remaining 386 structures. This alternative requires approximately 40 ringwalls, each surrounding from one to 62 structures, varying in length from 300 to 10,000 linear feet, and varying in height above grade from 5 to 15 feet. All structures will be treated to an elevation of one foot above the 0.01 AEP event, including sea level change. Non-structural treatments for the 0.02 AEP floodplain plan are summarized in Table 8.

Additional flood risk management measures would be required to mitigate backwater during fluvially influenced events. Mitigation efforts would increase for Alternative #3b from Alternative #3a due to greater constrictions for longer reaches. Channel modification comprised of deepening approximately 4,500 linear feet along mainstem Rahway River, widening the river near Monroe Street Bridge, and deepening approximately 2,000 linear feet along South Branch from the existing levee upstream towards the railroad bridge. Bridge replacements and road raising would be required as well.

Table 8: Non-Structural Treatments for the 0.1 (10-yr) and 0.02 (50-yr) AEP Floodplains.

Nonstructural Flood Proofing Measure	Alt #3a: 0.1 AEP Floodplain			Alt #3b: 0.02 AEP Floodplain		
	Residential	Non-Residential	Total	Residential	Non-Residential	Total
Dry Flood proofing	0	2	**2**	12	34	**46**
Dry Flood Proofing with Tank Anchoring	0	0	**0**	0	3	**3**
Wet Flood Proofing	10	1	**11**	66	1	**67**
Pump Replacement	0	3	**3**	0	3	**3**
Elevation	138	3	**141**	292	4	**296**
Ringwalls	47	53	**100**	92	90	**182**
Total of Structures	**195**	**62**	**257**	**462**	**135**	**597**

4.5 Alternatives Results

The improved hydraulic condition analysis shows that the alternative with the greatest flood risk reduction is Alternative #2. Reduction in WSE is up to 3.4 ft in the location of the Turnpike Bridge for Alternative #2. However, this alternative is the most costly of all the alternatives. Alternative #1 reduces WSE by about half a foot at the confluence with Robinson's Branch and South Branch, but only at smaller flood events. The reduction in WSE from "without project" WSEs to those of Alternatives #1 and #2 are seen in Table 9 through Table 11.

Table 9: Decrease in WSE from "without project" condition for the 0.1 AEP (10-yr) event.

Location	HEC-STA	W/O Project WSE (ft.)	Reduction in the 0.1 AEP WSE (ft.)	
			Alt #1	Alt #2
Rahway River at Rahway Water Supply Dam	34903.35	17.87	0.01	0.01
Robinson's Branch at Milton Lake Dam	8751.545	20.29	0.10	0.10
Robinson's Branch at Rahway Confluence	175.4458	10.43	0.64	1.05
Rahway River Levee at Milton Ave Bridge	25887.58	9.21	0.14	1.26
South Branch Upstream	11216.78	15.40	0.00	0.00
South Branch and Rahway River Confluence	210.7962	9.26	0.12	1.27
Rahway River at Turnpike Bridge	11792	8.50	0.00	1.41

Table 10: Decrease in WSE from "without project" condition for the 0.02 AEP (50-yr) event.

Location	HEC-STA	W/O Project WSE (ft.)	Reduction in the 0.02 AEP WSE (ft.)	
			Alt #1	Alt #2
Rahway River at Rahway Water Supply Dam	34903.35	20.08	0.00	0.00
Robinson's Branch at Milton Lake Dam	8751.545	21.30	0.00	0.00
Robinson's Branch at Rahway Confluence	175.4458	13.07	0.61	0.87
Rahway River Levee at Milton Ave Bridge	25887.58	11.46	0.09	1.48
South Branch Upstream	11216.78	17.43	0.00	0.00
South Branch and Rahway River Confluence	210.7962	11.44	0.03	1.46
Rahway River at Turnpike Bridge	11792	11.04	0.00	2.60

Table 11: Decrease in WSE from "without project" condition for the 0.01 AEP (100-yr) event

Location	HEC-STA	W/O Project WSE (ft.)	Reduction in the 0.01 AEP WSE (ft.)	
			Alt #1	Alt #2
Rahway River at Rahway Water Supply Dam	34903.35	21.30	0.00	0.00
Robinson's Branch at Milton Lake Dam	8751.545	21.75	0.00	0.00
Robinson's Branch at Rahway Confluence	175.4458	14.56	0.43	0.43
Rahway River Levee at Milton Ave Bridge	25887.58	12.42	0.00	2.06
South Branch Upstream	11216.78	17.84	0.00	0.00
South Branch and Rahway River Confluence	210.7962	12.42	0.00	2.07
Rahway River at Turnpike Bridge	11792	12.33	0.00	3.41

An initial economic analysis and cost estimate collectively determined that a combination plan of nonstructural treatments and a levee segment would provide the greatest benefit to cost ratio. It was determined from the analysis that Alternative #2 did not produce a positive benefit-to-cost ratio within the entirety of the hydraulically dependent alternative. Nonetheless, Alternative #1 produced one levee segment with a positive BC ratio as determined by economic reach due to hydraulic independence. The pre-TSP economic analysis therefore determined that a nonstructural plan in conjunction with levee Segment D from Alternative #1 would be used for TSP determination. This combination plan and its modifications will be described in the following sections.

4.5.1 The Combination Plan

In order to reach an acceptable alternative for the TSP milestone, a re-evaluation of non-structural measures (i.e. ringwalls) based on new engineering guidelines was necessary. Although ringwalls were previously determined as a nonstructural measure, they are in fact "structural" measures analyzed and treated as structural features, i.e. floodwalls. Appropriate ringwall buffers for construction and inspection were included in the combination plan reassessment of the 0.1 AEP floodplain.

4.5.1.1 Alternative #4: 0.1 AEP Non-Structural Plan + Levee

This plan consists of a subset of structures within the 0.1 AEP floodplain nonstructural plan (Alternative #3a) and levee segment D from Alternative #1. Nonstructural measures were designed to the future conditions 0.01 AEP (100-yr) WSE plus one foot to account for water surface perturbations. The design height of the levee was evaluated at elevation 12.6 ft. NAVD '88, consistent with the existing levees in the City of Rahway. Nonstructural recommendations on the protected side of this levee were omitted. This plan included a preliminary investigation of ringwall suitability, including the engineering feasibility given new guidelines and the economic practicability. A map of the combination plan can be found in Figure 44.

Alternative #4 determined nonstructural treatment for approximately 149 structures (131 residential, 18 non-residential) of the 577 structures (211 residential, 366 non-residential) contained in the 0.1 AEP (10-yr) floodplain. This alternative required 7 ringwalls, each surrounding from one to 5 structures, varying in length from 600 to 1,500 linear feet, and varying in height above grade from 5 to 10 feet. This is a reduction of 26 ringwalls from Alternative #3a, which in turn also reduced the need for channel modification and bridge replacement. No treatment was recommended at the time for the remaining 428 structures within the floodplain. A summary of the treated structures in Alternative #4 can be found in Table 12. Ringwall characteristics can be found in Table 13.

Table 12: Nonstructural Treatments for structures within Alternative #4.

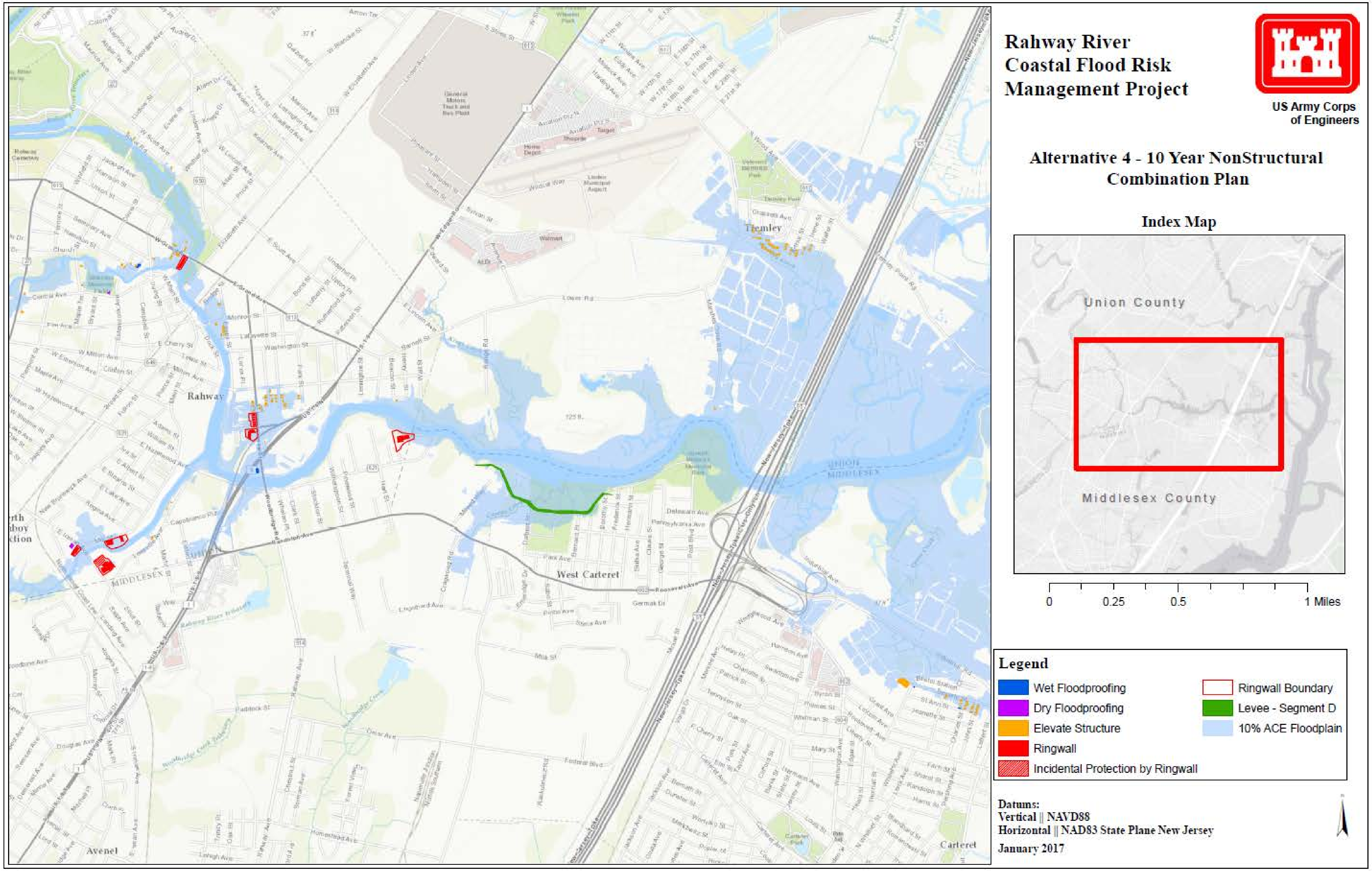
Nonstructural Flood Proofing Measure	**0.1 AEP ACE Combination Plan**		
	Residential	**Non-Residential**	**Total**
Dry Flood Proofing	0	2	**2**
Wet Flood Proofing	1	3	**4**
Elevation	123	4	**127**
Elevation - Demolish and Rebuild	1	2	**3**
Ringwall	6	7*	**13**
Total of Structures	**131**	**18**	**149**

* Structure is incidentally protected by ringwall. There is no associated cost with the additional structure but there are additional benefits.

Table 13: Characteristics of Ringwalls in the 0.1 AEP Combination Plan.

Ringwall	**Structures within Ringwall**	**Avg Height of Ringwall (in feet)**	**Top of Ringwall (EL ft. NAVD)**	**Perimeter (ft)**
R001	2*	10	14.4	1226.362
R002	1	5	14.4	608.715
R003	2	10	14.4	1192.455
R004	1	10	14.3	1436.819
R005	1	10	14.4	858.846
R006	5	10	14.4	812.531
R007	1	10	16	789.54

* Structure is incidentally protected by ringwall. There is no associated cost with the additional structure but there are additional benefits.



*Note: This is the tidal inundation only. Representation does not include joint-probability WSEs.

Figure 44: Alternative #4 Plan Overview

4.5.1.2 Alternative #4a: 0.1 AEP Non-Structural Plan + Levee, No Ringwalls

Alternative #4a consists of the 0.1 AEP floodplain nonstructural plan (Alternative #4) and a levee (Alternative #1 Segment D Levee) with the removal of all ringwalls from the nonstructural plan. The incremental justification of Alternative #4 resulted in all ringwalls being economically infeasible. As it was determined during the preliminary ringwall suitability evaluation in Alternative #4, structures given ringwall treatment had no other feasible nonstructural treatment method. The removal of all ringwalls would consequently remove all the structures enclosed by ringwalls from the plan entirely.

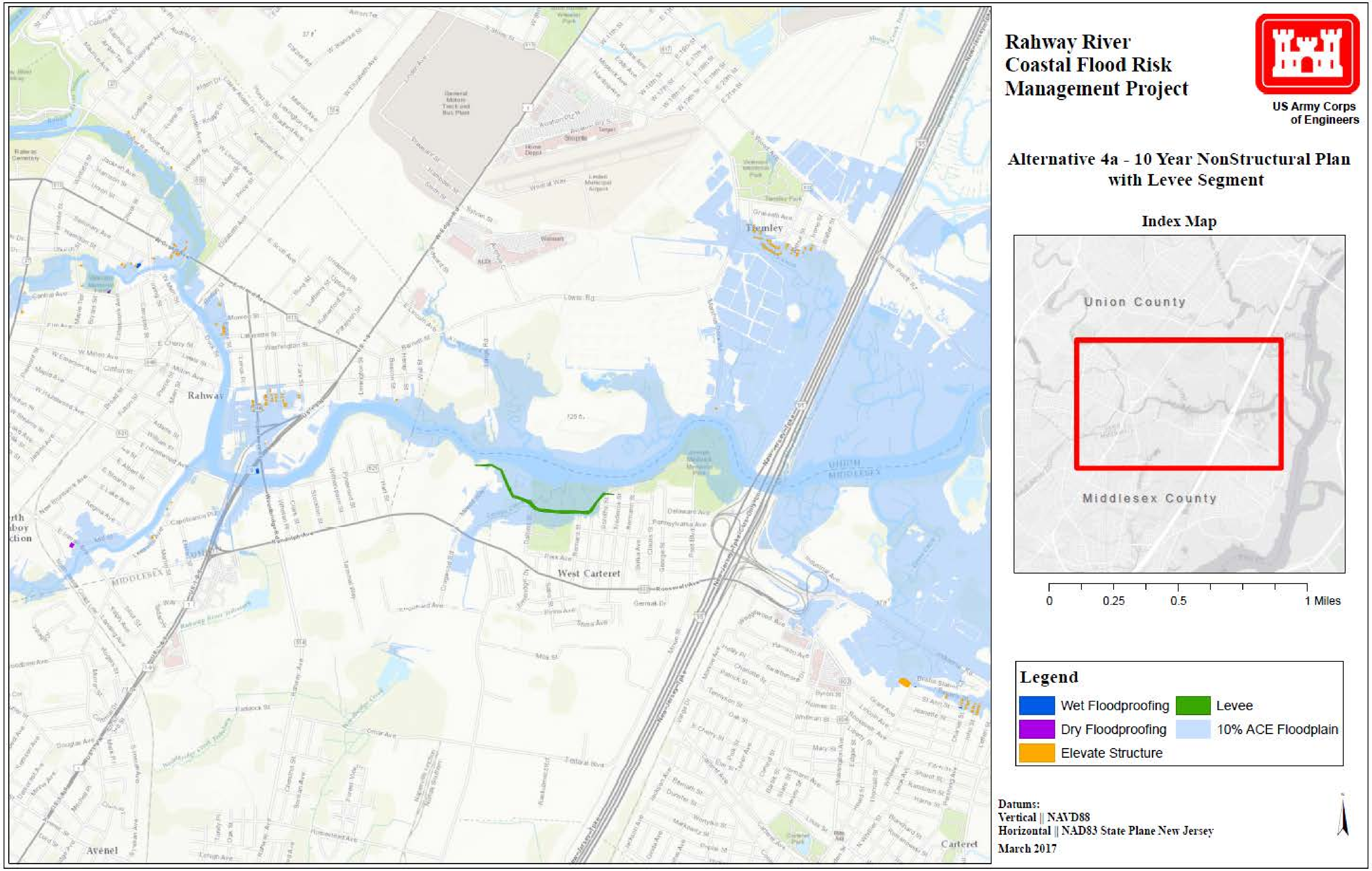
Alternative #4a thus determined nonstructural treatment for approximately 136 structures (125 residential, 11 non-residential) of the 577 structures (211 residential, 366 non-residential) contained in the 0.1 AEP (10-yr) floodplain. Nonstructural measures were designed to the future conditions 0.01 AEP (100-yr) WSE plus one foot to account for water surface perturbations. No treatment is recommended at this time for the remaining 441 structures within the floodplain.

The levee segment is 3,360 ft. long with a 12 ft. top width and one vertical to three horizontal (1:3) side slopes. The average height is approximately 7.5 ft. The design height of the levee was evaluated at elevation 12.6 ft. NAVD '88, consistent with the existing levees in the City of Rahway. The levee is located next to the right bank of the Rahway River, approximately 1.2 miles downstream of the confluence with the South Branch. The upstream end is located at the industrial/commercial area by Ardemore Ave., continuing downstream to Dorothy St. Nonstructural recommendations on the protected side of this levee were omitted.

Optimization of Alternative #4a is the next step of the hydraulic analysis, during which nonstructural treatments and the levee segment will be revisited for analysis at various flood frequency design heights. A map of this Tentatively Selected Plan can be found in Figure 45. A summary of the treated structures in Alternative #4a can be found in Table 14.

Table 14: Nonstructural Treatments for structures within Alternative #4a.

Nonstructural Flood Proofing Measure	**10% ACE Combination Plan**		
	Residential	**Non-Residential**	**Total**
Dry Flood Proofing	0	2	**2**
Wet Flood Proofing	1	3	**4**
Elevation	123	4	**127**
Elevation - Demolish and Rebuild	1	2	**3**
Total of Structures	**125**	**11**	**136**



*Note: This is the tidal inundation only. Representation does not include joint-probability WSEs.

Figure 45: Alternative #4a Plan Overview.

5.0 UNCERTAINTY ANALYSIS ON EXISTING AND FUTURE WITH AND WITHOUT PROJECT CONDITIONS

To determine the uncertainty of the Coastal WSELs, the standard-of-practice is to develop a probabilistic model of the storm forcing parameters. The primary parameters include at coastal reference location are central pressure deficit, radius of maximum winds, translation speed and heading direction. Statistical approaches for estimating the joint probability of coastal storm response, such as surge and waves, have been greatly improved. Within the North Atlantic Coast Comprehensive Study (NACCS), still water level (SWL) curves were computed from statistical analysis at nearly 19,000 “save points” along the east coast. The upper limits of three confidence intervals (68%, 90%, and 95%) were provided, corresponding to 1.0, 1.6, and 2.0 times the standard deviation above the expected SWL curve. For Rahway River at Arthur Kill, the save point (No. 11659) had a rating curve with 68% and 95% confidence intervals as seen in Figure 46. (See Section 3.3.2 “Downstream Boundary Condition – Stage Hydrographs” regarding the rating curve chosen.) These data were used in the economic analysis are in compliance with the recommended procedure provided in the EM 1110-2-1619 (USACE 1996).

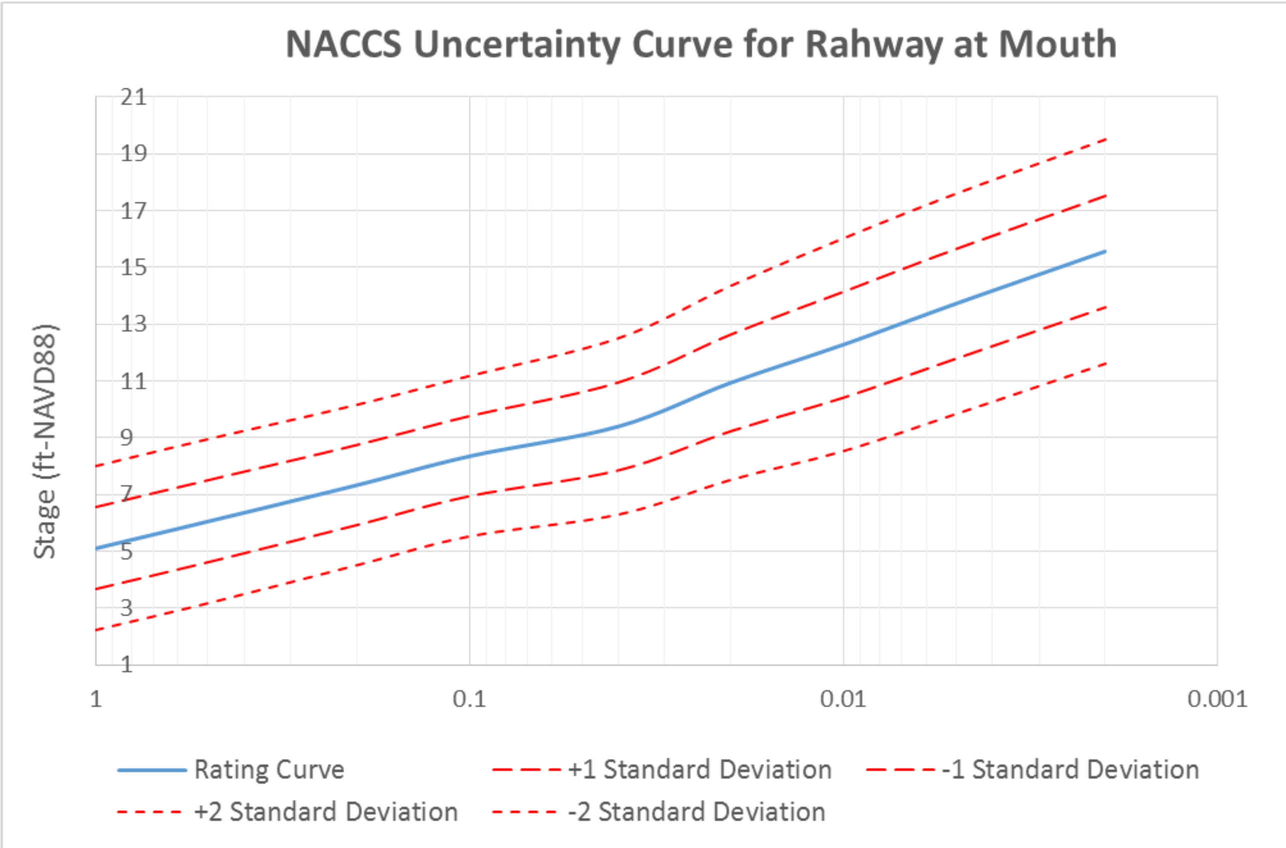


Figure 46: NACCS 1.0 and 2.0 times the standard deviation from expected SWL curve.