Rahway River Basin, New Jersey
Coastal Storm Risk Management Feasibility Study

Appendix CII
Hydraulics

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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 conducted in September 2016 for the "fluvial," or inland portion of the basin. This feasibility study focuses on the coastal portion of the basin and includes the New Jersey municipalities of Rahway, Carteret, and Linden. These studies have been separated based on congressional appropriations, such that the Rahway Coastal study is funded by Hurricane Sandy project funds and is limited to a 3-year study period (See the Planning Appendix for further explanation). This study will only focus on alternatives that manage flood risk during coastal storms. A map of the Rahway River Basin, its municipalities, and the fluvial and coastal study areas is shown in Figure 1. The area of study specific to this report, "Rahway Coastal," is shown in Figure 2.

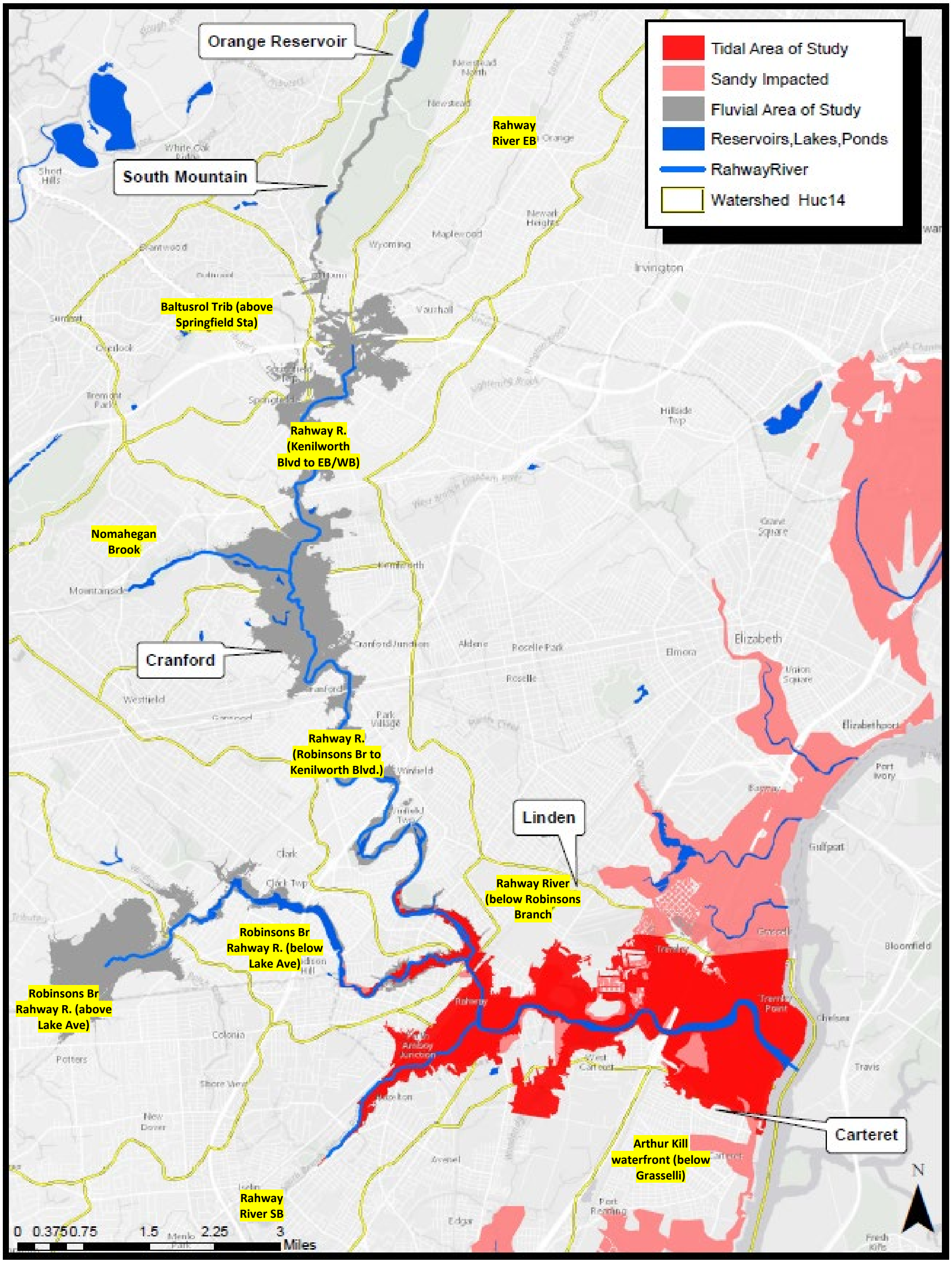


Figure 1: Rahway River Watershed.

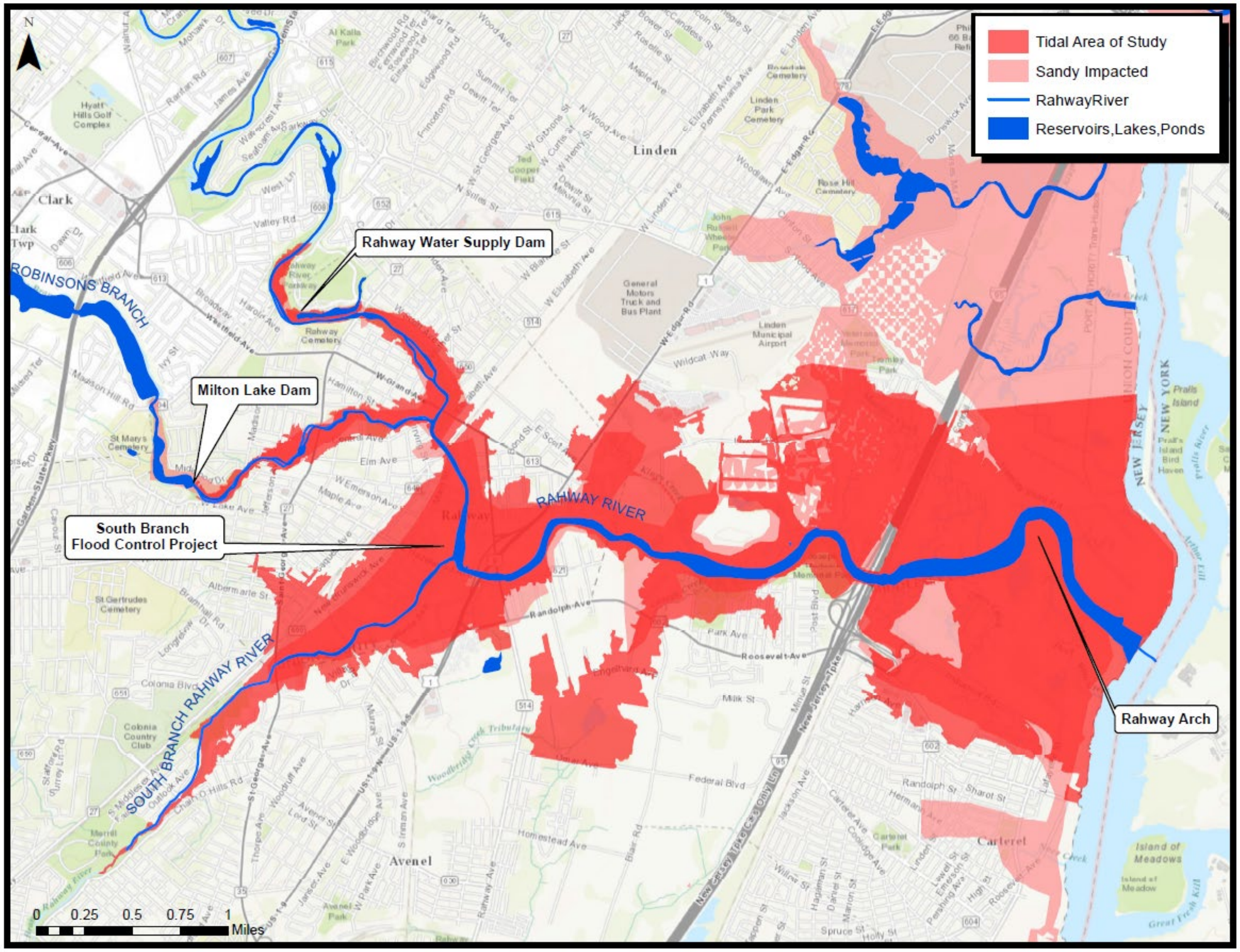


Figure 2: Rahway River Coastal Area of Study.

1.2 Present Flooding Problems

Periodic storms have caused severe coastal 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 Main stem 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 coastal 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 coastal 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 coastal 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 Hazardous, Toxic and Radioactive Waste (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 (FRMP) with levees and floodwalls 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) coastal 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 (10-year) 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 (10-year) 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 approximately at the 0.01 AEP (100-year) coastal storm event. For future conditions that include some increase in flow and sea level, the levees will be overtopped well before the 0.01 (100-year) AEP event.

There is street flooding beginning at the 0.2 AEP (5-year) at the confluence between Robinson's Branch and Rahway River.

2.3 Existing Hydraulic Features – City Of Rahway Levee and Floodwall

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 coastal 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 system is approximalty 5,300 ft long and was re-graded 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 coastal 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 the fluvial and coastal 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 coastal 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 (TS) Irene and Hurricane Sandy. The 2012 Hurricane Sandy event was used to model a storm surge event in the coastal area of study. Hurricane Sandy is slightly less than a 0.01 AEP coastal event (100-year storm event) having a fluvial component that is negligible. The August 2011 Tropical Storm Irene was used to calibrate a storm with both fluvial and coastal influence. TS Irene is slightly greater than a 0.01 AEP fluvial event with a coastal 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.

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, respectively, for the open channel sections and at 0.3 and 0.5, respectively, 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 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 (Water Surface Elevation) 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.

During the improved conditions lower n-values were use to characterize channel modification. This increased flow conveyance capacity of the channel, reducing flood during fluvial event, but not the same during coastal events. Coastal events 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 through Figure 8 are the HEC-RAS WSEs calibration profiles for the Irene and Sandy storm events

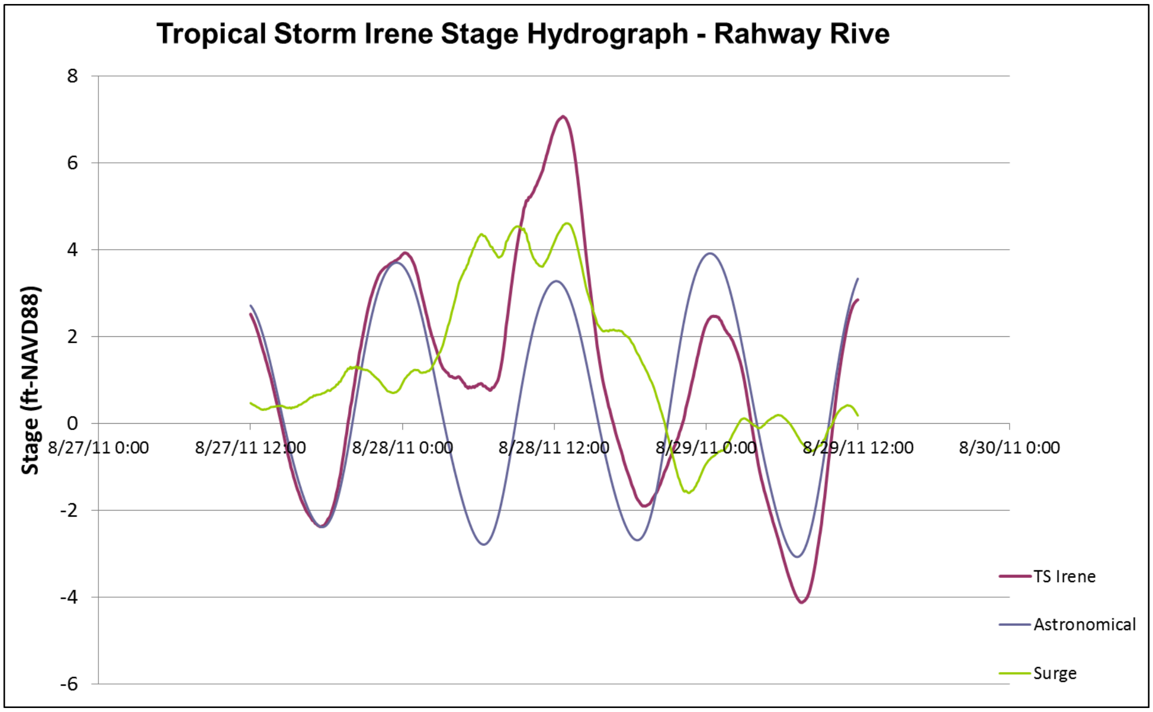


Figure 3: Stage Hydrograph for Tropical Storm Irene.

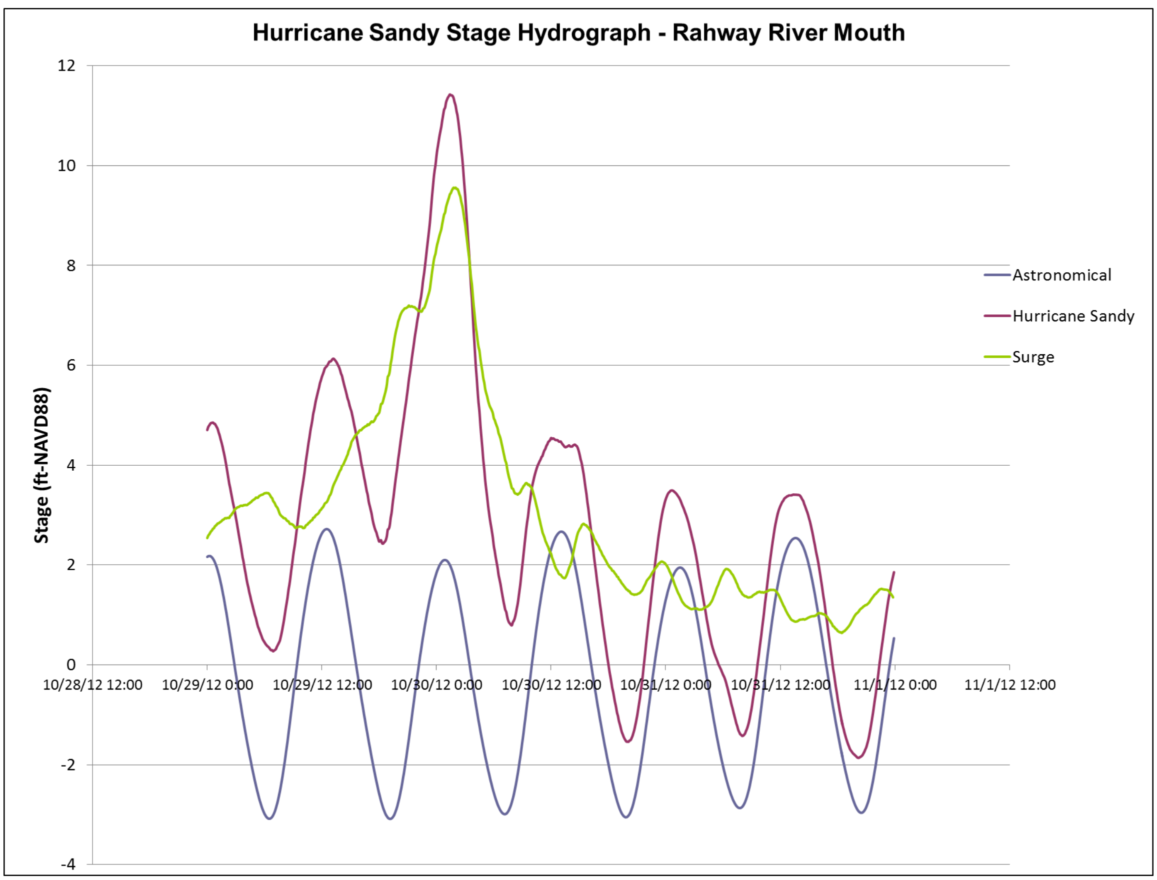


Figure 4: Stage Hydrograph for Hurricane Sandy.

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

*Stage are estimates, gage failed during storm event.

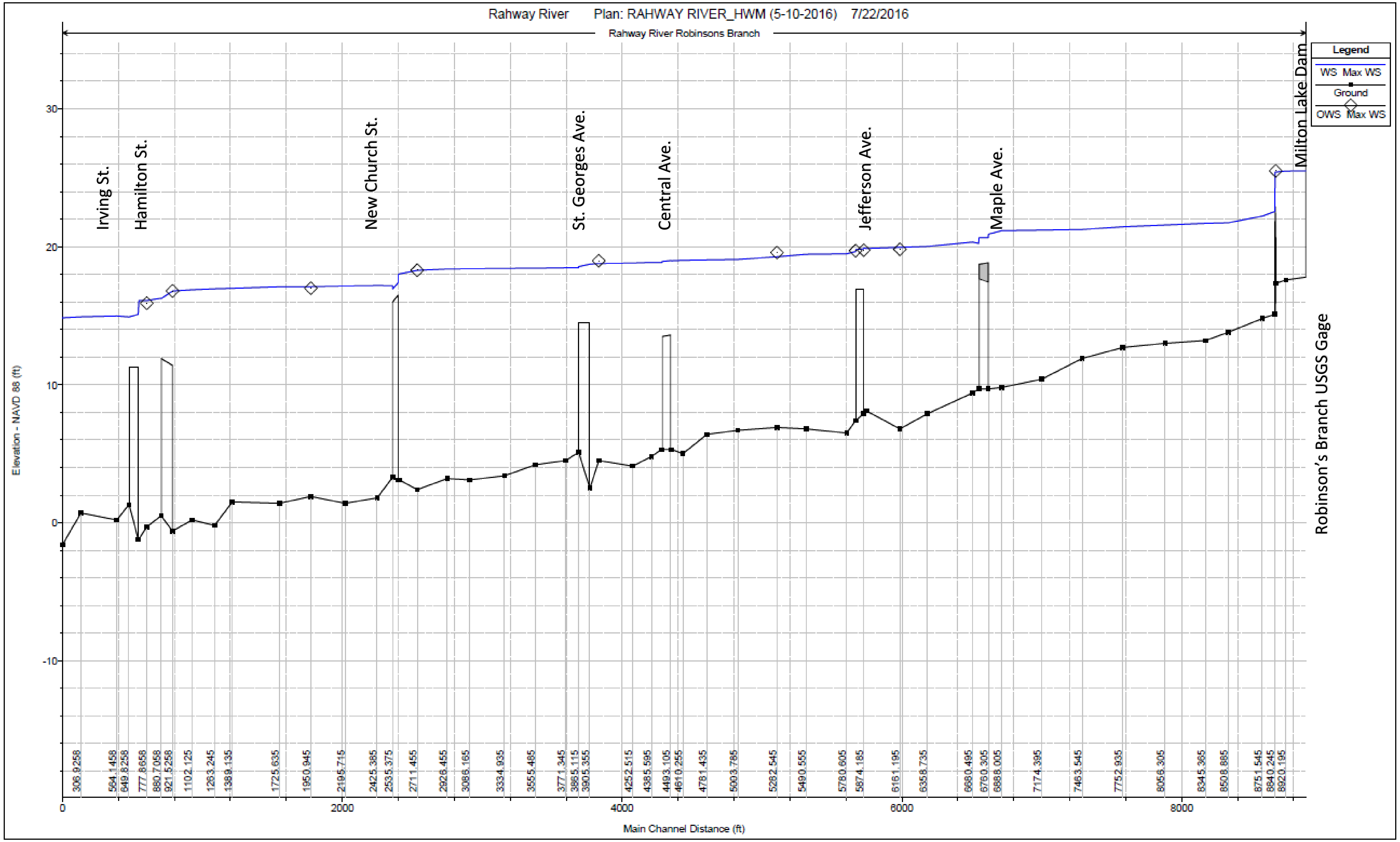


Figure 5: Computed water surface profile and observed HWMs for Tropical Storm Irene in Robinson's Branch.

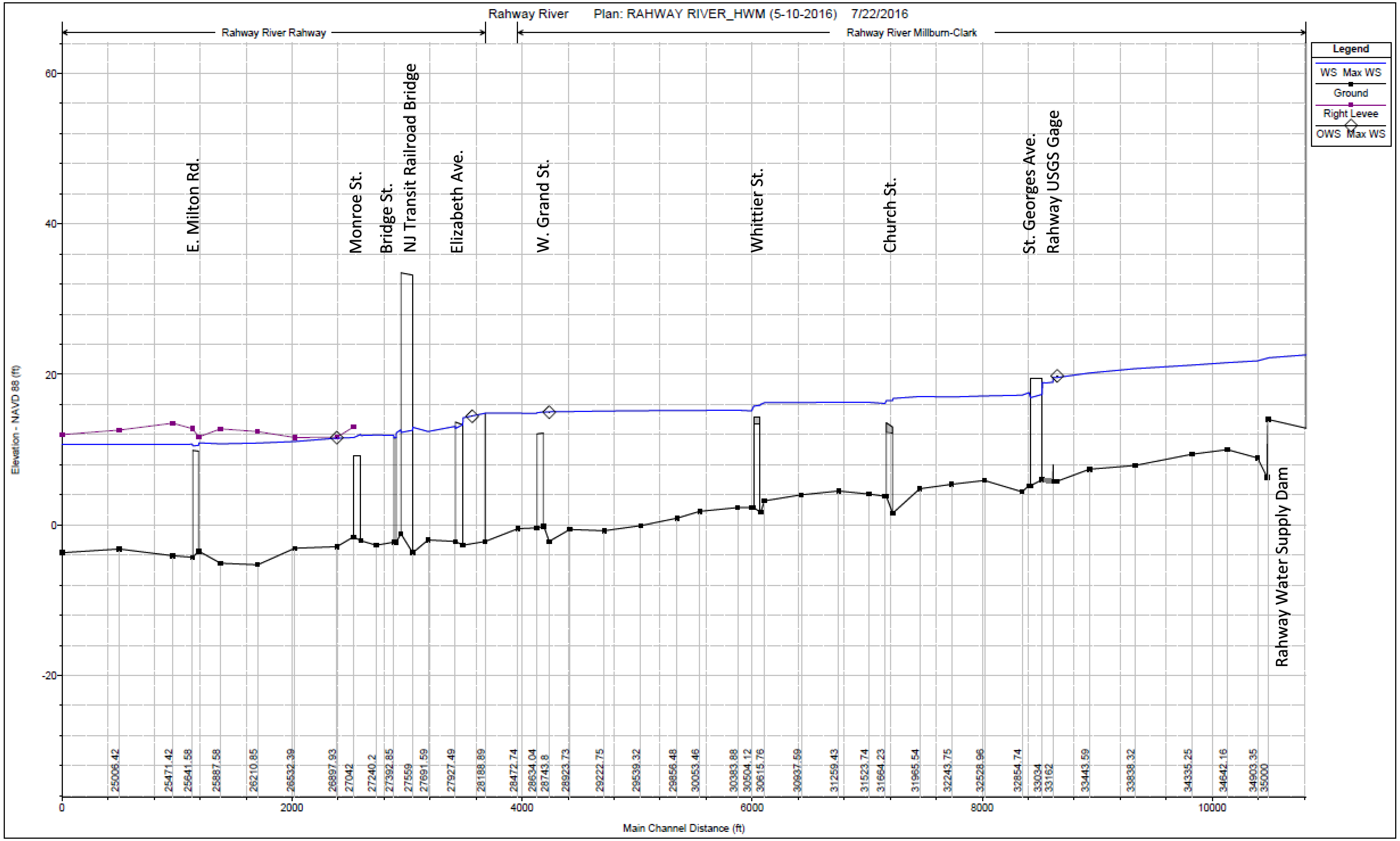


Figure 6: Computed water surface profile and observed HWMs for Tropical Storm Irene in the Rahway River between Rahway Water Supply and South Branch confluence.

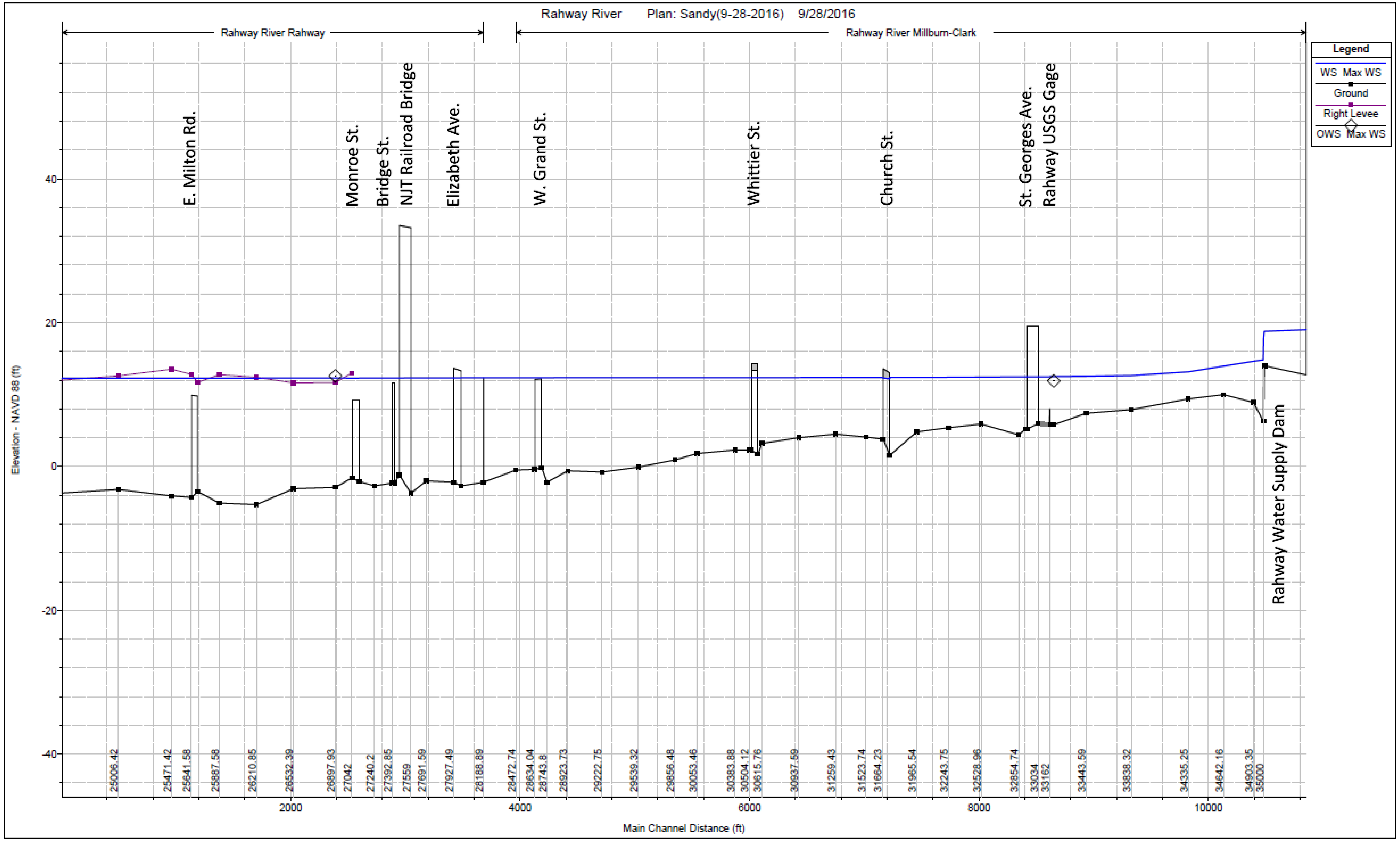


Figure 7: Computed water surface profile and observed HWMs for Hurricane Sandy in the Rahway River between Rahway Water Supply and South Branch confluence

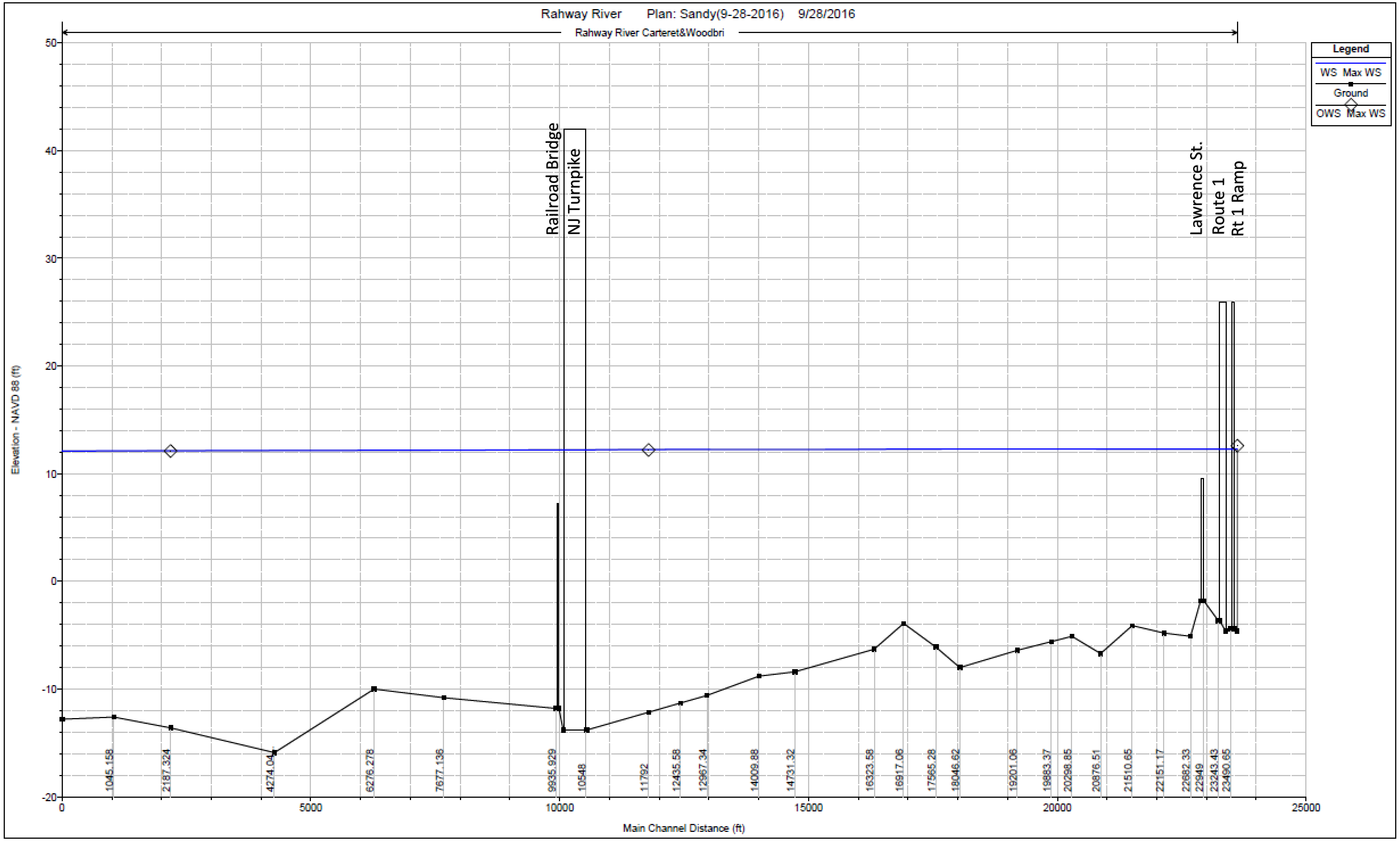


Figure 8: Computed water surface profile and observed HWMs for Hurricane Sandy downstream of South Branch to the Arthur Kill.

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 coastal 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 Coastal-Fluvial Joint Probability

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

For this assessment, both the NOAA tide 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 coastal 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 coastal 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 coastal stage.

Based on this coastal-fluvial assessment, it was determined that 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. For the remainder of this report, all frequency events referenced will be coastal dominant unless indicated otherwise. It should be noted that the stage of the tide cycle that the storm is coincident with is accounted for in the coastal storm modeling by running 96 random tides in the model and using statistics to reduce to a single height of water to be superimposed on the storm water surface elevation, such that the resulting water surface elevation accounts for the fact that the storm may arrive at any of the 96 random times in the tide cycle.

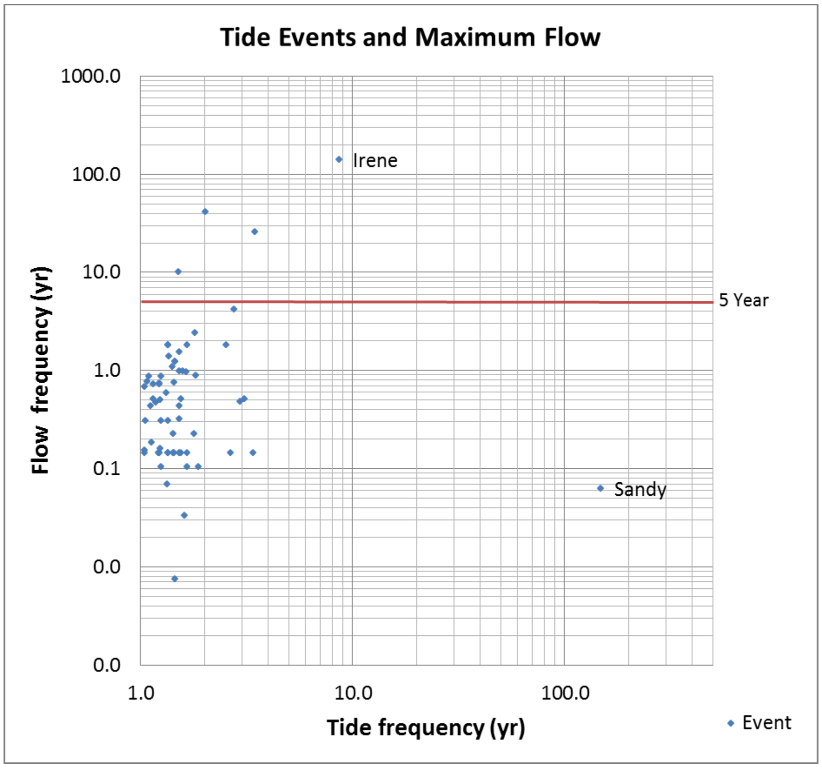


Figure 9: Coastal event and the maximum flow during the event.

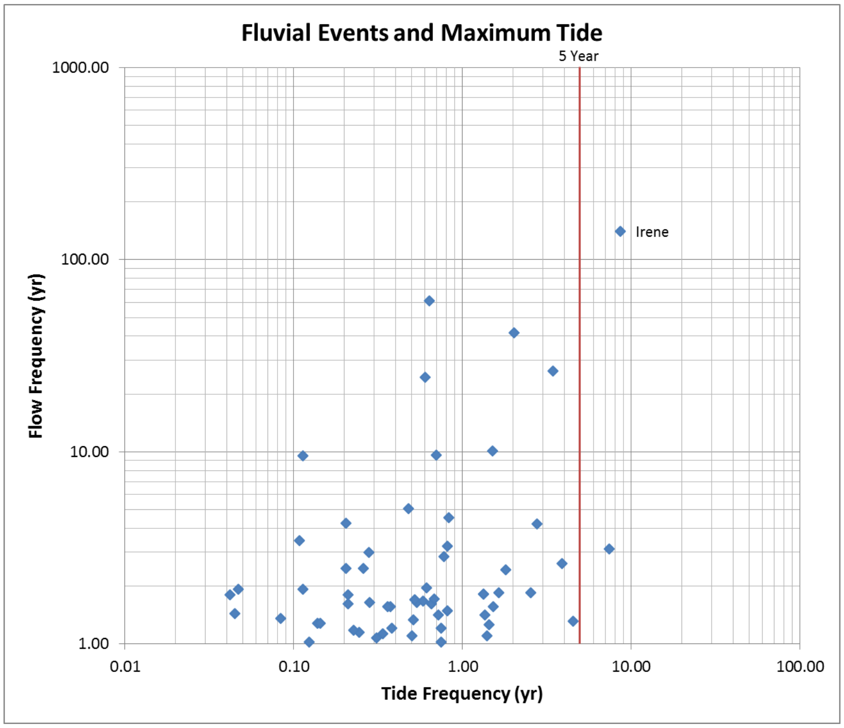


Figure 10: Fluvial event and the maximum coastal stage during the event

3.3.2 Coastal Stage Hydrographs and Downstream Boundary Condition

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 coastal boundary condition hydrographs. The stage frequency data for present conditions is shown in 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 stage-frequency curve (see Table 3) selected from the NACCS study was the base condition with 96 random tides superimposed. Sea level change was manually superimposed. Superposition requires assumption of negligible nonlinearity. Based on Figure D11 in Nadal-Caraballo et. Al. (2015), "Coastal Storm Hazards from Virginia to Maine", ERDC/CHL TR-15-5, Vicksburg, MS, Coastal and Hydraulics Laboratory, U.S. Army Corps of Engineer Research and Development Center, nonlinearity in the Rahway Coastal region is small. This assumption shall be tested in the PED phase. If significant enough nonlinearity is determined, hydrodynamic modeling shall be performed using tides and sea level change as starting conditions. The average of the tidal addition to the coastal storm surge is approximately 1.4 feet. This is equivalent to approximately 60% of the height of the average of NOAA's Mean High Water Datum representing the 1983 to 2001 epoch.

The NOAA Bergen Point gage (ID: 8519483) tide cycle characteristics were used to develop a basic shape for all the coastal 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 4. The local tide has no effect on the final maximum stages at the mouth on the river, as the astronomivcal stages are lower in elevation than all NACCS AEP evets. the 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 duration of each hypothetical storm had previously been obtained for the Port Monmouth CSRM study and it was reused for this study. In relation to the CSRM study, Port Monmouth is located approximately 15 miles southeast of the project area. 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 coastal 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 3: NACCS Stage-Frequency for PointID 11659 for year 1992 epoch 1983-2001mid point.

Frequency (YR)	Probability	Stage-Frequency MSL (M)	Stage-Frequency MSL (ft)	Stage-Frequency (ft-NAVD88)
1	0.99	1.59	5.22	5.10
2	0.5	1.88	6.17	6.05
5	0.2	2.27	7.45	7.33
10	0.1	2.58	8.46	8.35
20	0.05	2.9	9.51	9.40
50	0.02	3.37	11.06	10.94
100	0.01	3.78	12.40	12.28
200	0.005	4.22	13.85	13.73
500	0.002	4.78	15.68	15.56

*Note: Equation to convert MSL to NAVD88 is (0 m MSL = - 0.036 m NAVD88). Source of equation is from NACCS Study

Table 4: Bergen Point Gage Tide Datum.

Tide Characteristics for Bergen Point Gage ID: 8519483	
Coastal Datum	Elevation in ft. above NAVD'88
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

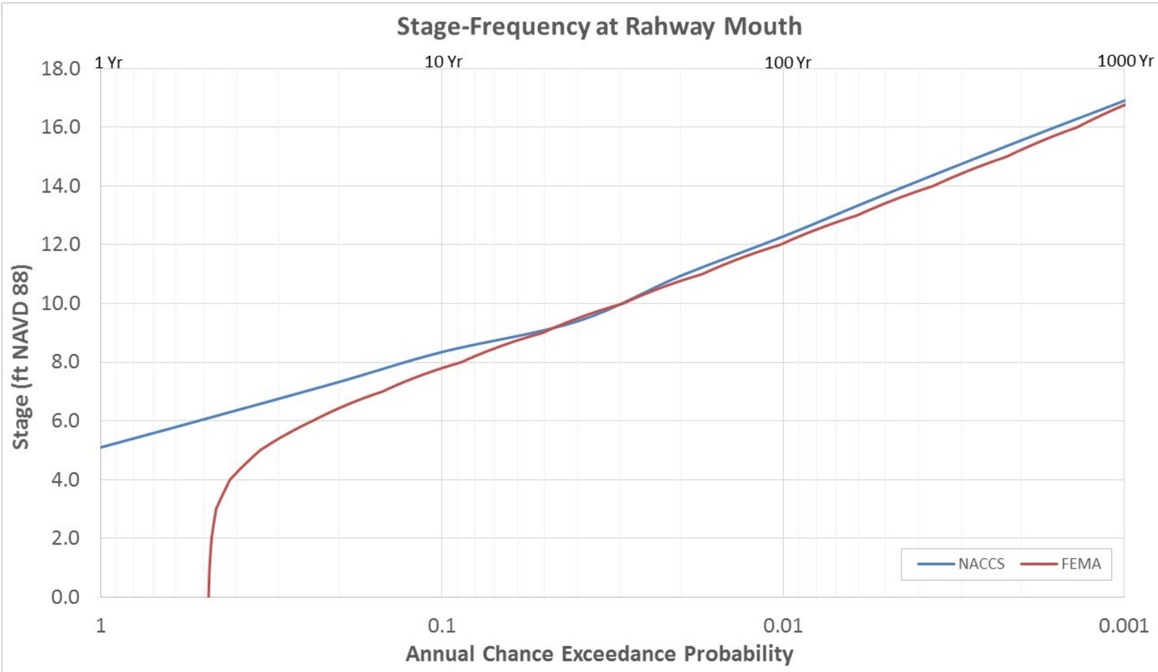


Figure 11: Stage-Frequency Curve at Rahway Mouth from NACCS and FEMA. *FEMA curve at Carteret (ID: 543829) approx. 0.5 mi. downstream of NACCS point on the Arthur Kill.

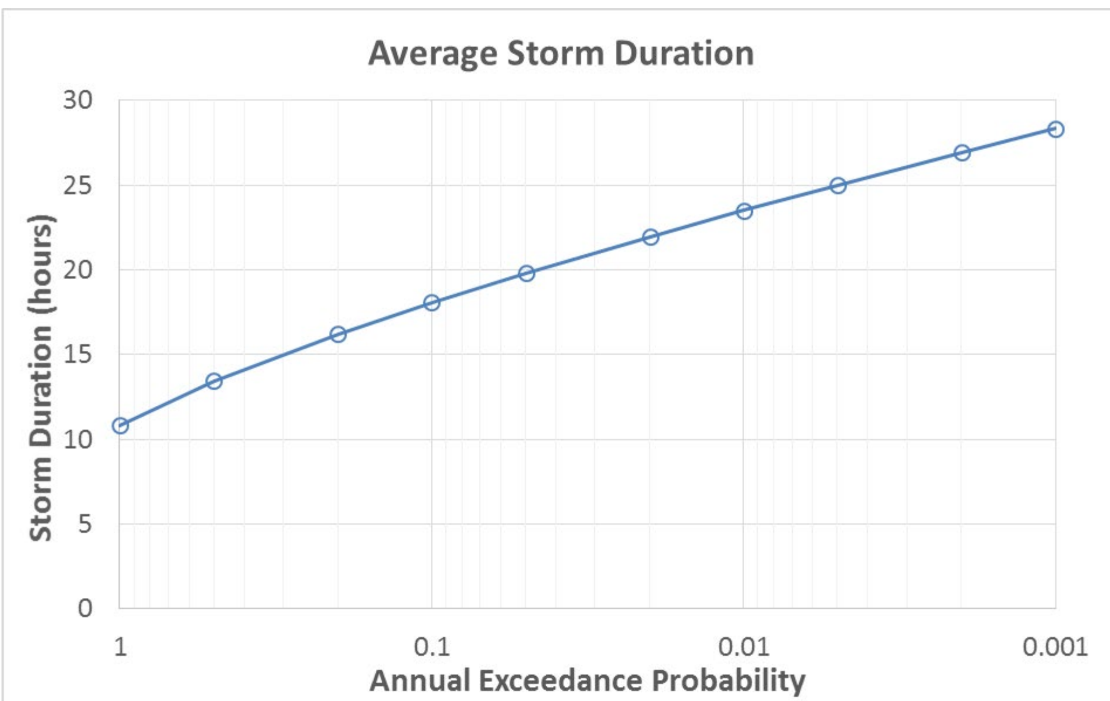


Figure 12: Storm duration curve from the NACCS study for Port Monmouth, NJ.

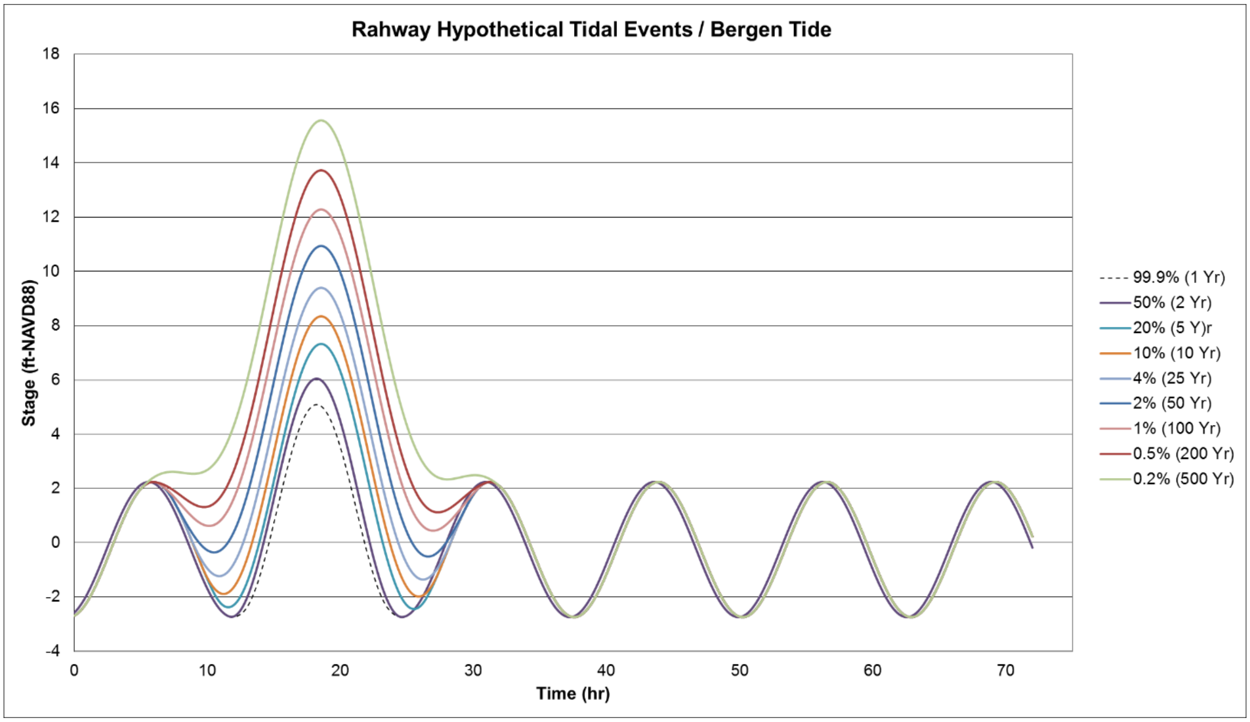


Figure 13: Stage hydrograph for hypothetical coastal events at the mouth of the Rahway River.

3.3.3 Frequency of Coincident Flows

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. Nonetheless, the resultant condition might be in function of the two independent events. In a hydrologic context, for this case according to EM 1110-2-1415 Chapter 11 - Frequency of Coincident Flows, it is necessary to consider those events which occur coincidentally with other events (i.e. all fluvial events that might occur coincidentally with the coastal events).

Hydrologic Engineering Center's (HEC) Statistical Software Package (HEC-SSP) was used to perform this analysis. The analysis was performed by running hypothetical coastal storm surge events up the Rahway River against hypothetical fluvial storm events down the Rahway River. Refer to Table 5 to see the different plans that were run through the hydraulic model. Both the present (Year 2023) and future conditions (Year 2073) were analyzed. All three coastal scenarios were analyzed. Refer to section 3.3.4 for more information about the coastal scenarios. The

NACSS and HEC-RAS model's output were input into HEC-SSP and a coincident frequency analysis was performed. Refer to Figure 14 through Figure 16 for joint probability analysis results at different locations of the Rahway River. The graphs compare the Rahway coastal stages obtained from NACCS versus the Rahway coastal stages with Rahway fluvial influence.

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 sensitive to fluvial flows, 2) the City of Rahway and the lower portion of Robinson's Branch have risk from both coastal 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.

Table 5: Joint Probability Model Runs – Rahway Fluvial vs. Rahway Coastal events.

Model Plan #	**Rahway Fluvial Storm Event**	**Rahway Coastal Storm Event**
1	99.99% AEP (1 Year Event)	99.99% AEP (1 Year Event)
2	99.99% AEP (1 Year Event)	50% AEP (2 Year Event)
3	99.99% AEP (1 Year Event)	20% AEP (5 Year Event)
4	99.99% AEP (1 Year Event)	10% AEP (10 Year Event)
5	99.99% AEP (1 Year Event)	4% AEP (25 Year Event)
6	99.99% AEP (1 Year Event)	2% AEP (50 Year Event)
7	99.99% AEP (1 Year Event)	1% AEP (100 Year Event)
8	99.99% AEP (1 Year Event)	0.4% AEP (250 Year Event)
9	99.99% AEP (1 Year Event)	0.2% AEP (500 Year Event)
10	50% AEP (2 Year Event)	99.99% AEP (1 Year Event)
11	50% AEP (2 Year Event)	50% AEP (2 Year Event)
12	50% AEP (2 Year Event)	20% AEP (5 Year Event)
13	50% AEP (2 Year Event)	10% AEP (10 Year Event)
14	50% AEP (2 Year Event)	4% AEP (25 Year Event)
15	50% AEP (2 Year Event)	2% AEP (50 Year Event)
16	50% AEP (2 Year Event)	1% AEP (100 Year Event)
17	50% AEP (2 Year Event)	0.4% AEP (250 Year Event)
18	50% AEP (2 Year Event)	0.2% AEP (500 Year Event)
19	20% AEP (5 Year Event)	99.99% AEP (1 Year Event)
20	20% AEP (5 Year Event)	50% AEP (2 Year Event)
21	20% AEP (5 Year Event)	20% AEP (5 Year Event)
22	20% AEP (5 Year Event)	10% AEP (10 Year Event)
23	20% AEP (5 Year Event)	4% AEP (25 Year Event)
24	20% AEP (5 Year Event)	2% AEP (50 Year Event)

Model Plan #	Rahway Fluvial Storm Event	Rahway Coastal Storm Event
25	20% AEP (5 Year Event)	1% AEP (100 Year Event)
26	20% AEP (5 Year Event)	0.4% AEP (250 Year Event)
27	20% AEP (5 Year Event)	0.2% AEP (500 Year Event)
28	10% AEP (10 Year Event)	99.99% AEP (1 Year Event)
29	10% AEP (10 Year Event)	50% AEP (2 Year Event)
30	10% AEP (10 Year Event)	20% AEP (5 Year Event)
31	10% AEP (10 Year Event)	10% AEP (10 Year Event)
32	10% AEP (10 Year Event)	4% AEP (25 Year Event)
33	10% AEP (10 Year Event)	2% AEP (50 Year Event)
34	10% AEP (10 Year Event)	1% AEP (100 Year Event)
35	10% AEP (10 Year Event)	0.4% AEP (250 Year Event)
36	10% AEP (10 Year Event)	0.2% AEP (500 Year Event)
37	4% AEP (25 Year Event)	99.99% AEP (1 Year Event)
38	4% AEP (25 Year Event)	50% AEP (2 Year Event)
39	4% AEP (25 Year Event)	20% AEP (5 Year Event)
40	4% AEP (25 Year Event)	10% AEP (10 Year Event)
41	4% AEP (25 Year Event)	4% AEP (25 Year Event)
42	4% AEP (25 Year Event)	2% AEP (50 Year Event)
43	4% AEP (25 Year Event)	1% AEP (100 Year Event)
44	4% AEP (25 Year Event)	0.4% AEP (250 Year Event)
45	4% AEP (25 Year Event)	0.2% AEP (500 Year Event)
46	2% AEP (50 Year Event)	99.99% AEP (1 Year Event)
47	2% AEP (50 Year Event)	50% AEP (2 Year Event)
48	2% AEP (50 Year Event)	20% AEP (5 Year Event)
49	2% AEP (50 Year Event)	10% AEP (10 Year Event)
50	2% AEP (50 Year Event)	4% AEP (25 Year Event)
51	2% AEP (50 Year Event)	2% AEP (50 Year Event)
52	2% AEP (50 Year Event)	1% AEP (100 Year Event)
53	2% AEP (50 Year Event)	0.4% AEP (250 Year Event)
54	2% AEP (50 Year Event)	0.2% AEP (500 Year Event)
55	1% AEP (100 Year Event)	99.99% AEP (1 Year Event)
56	1% AEP (100 Year Event)	50% AEP (2 Year Event)
57	1% AEP (100 Year Event)	20% AEP (5 Year Event)
58	1% AEP (100 Year Event)	10% AEP (10 Year Event)
59	1% AEP (100 Year Event)	4% AEP (25 Year Event)
60	1% AEP (100 Year Event)	2% AEP (50 Year Event)
61	1% AEP (100 Year Event)	1% AEP (100 Year Event)
62	1% AEP (100 Year Event)	0.4% AEP (250 Year Event)
63	1% AEP (100 Year Event)	0.2% AEP (500 Year Event)
64	0.4% AEP (250 Year Event)	99.99% AEP (1 Year Event)
65	0.4% AEP (250 Year Event)	50% AEP (2 Year Event)
66	0.4% AEP (250 Year Event)	20% AEP (5 Year Event)
67	0.4% AEP (250 Year Event)	10% AEP (10 Year Event)
68	0.4% AEP (250 Year Event)	4% AEP (25 Year Event)
69	0.4% AEP (250 Year Event)	2% AEP (50 Year Event)

Model Plan #	Rahway Fluvial Storm Event	Rahway Coastal Storm Event
70	0.4% AEP (250 Year Event)	1% AEP (100 Year Event)
71	0.4% AEP (250 Year Event)	0.4% AEP (250 Year Event)
72	0.4% AEP (250 Year Event)	0.2% AEP (500 Year Event)
73	0.2% AEP (500 Year Event)	99.99% AEP (1 Year Event)
74	0.2% AEP (500 Year Event)	50% AEP (2 Year Event)
75	0.2% AEP (500 Year Event)	20% AEP (5 Year Event)
76	0.2% AEP (500 Year Event)	10% AEP (10 Year Event)
77	0.2% AEP (500 Year Event)	4% AEP (25 Year Event)
78	0.2% AEP (500 Year Event)	2% AEP (50 Year Event)
79	0.2% AEP (500 Year Event)	1% AEP (100 Year Event)
80	0.2% AEP (500 Year Event)	0.4% AEP (250 Year Event)
81	0.2% AEP (500 Year Event)	0.2% AEP (500 Year Event)

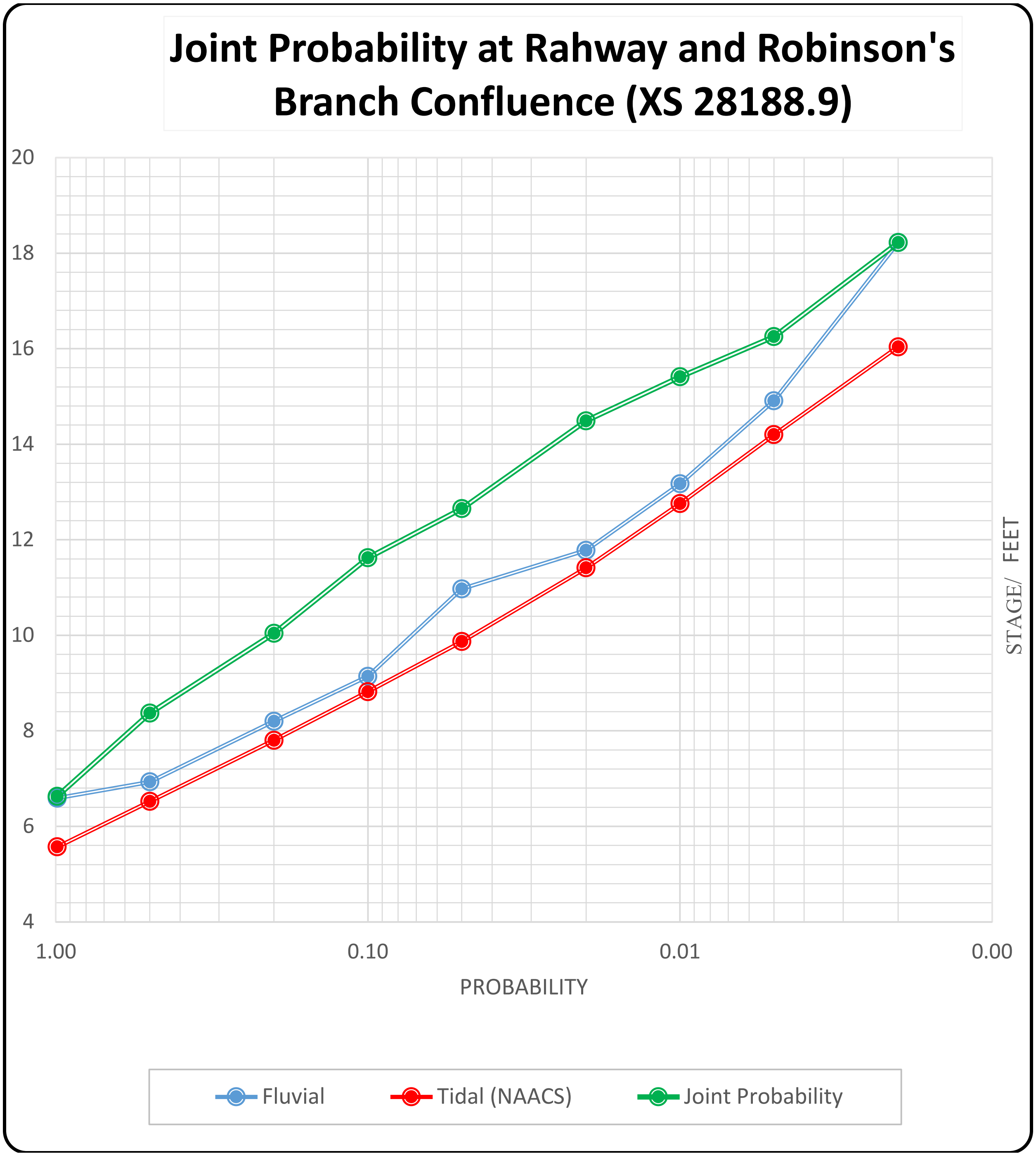


Figure 14: Joint Probability at cross section 28188.9 - Rahway and Robinson's Branch Confluence.

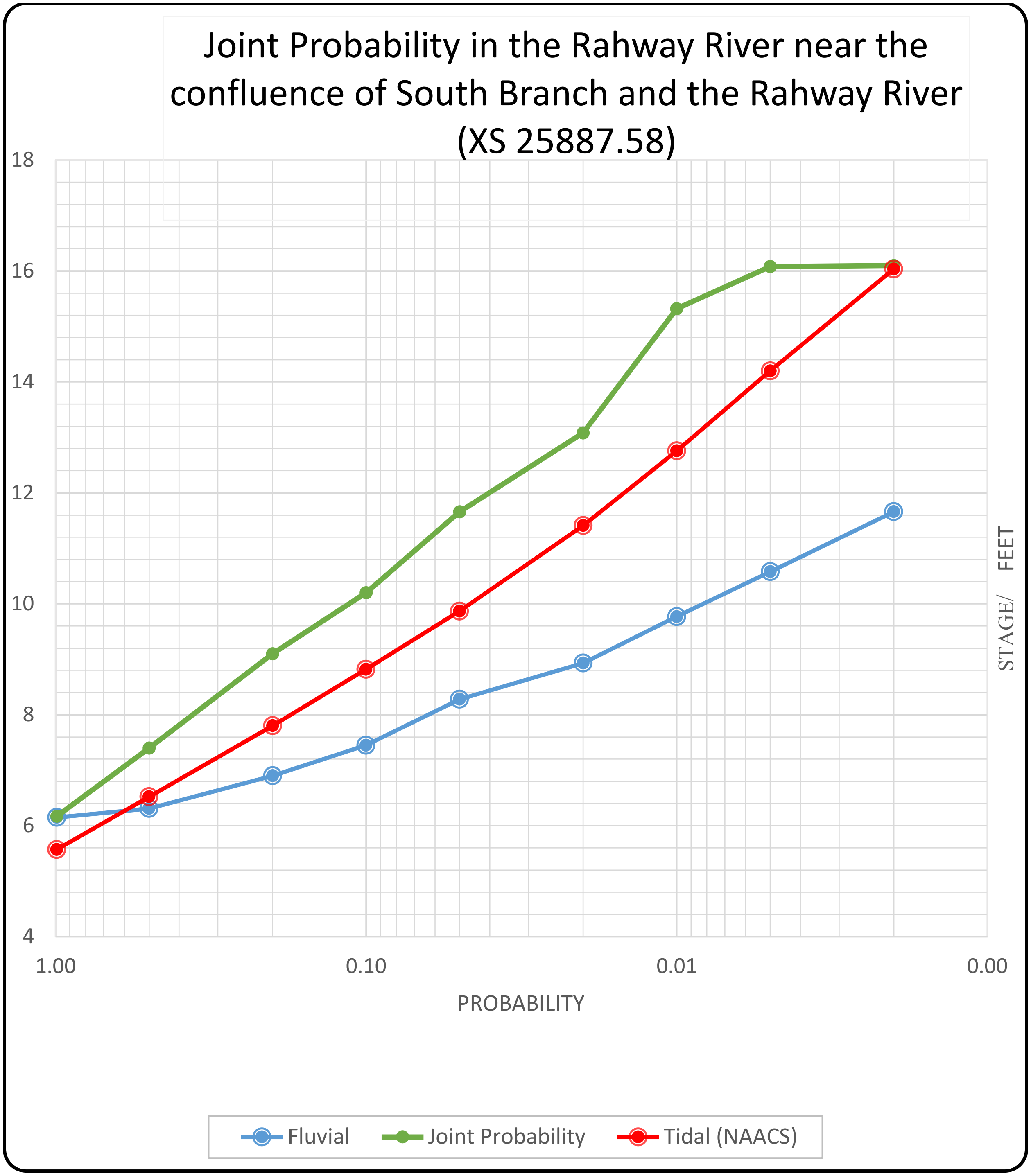


Figure 15: Joint Probability in the Rahway River 1600 feet upstream of the confluence of South Branch and the Rahway River- cross section 25887.58.

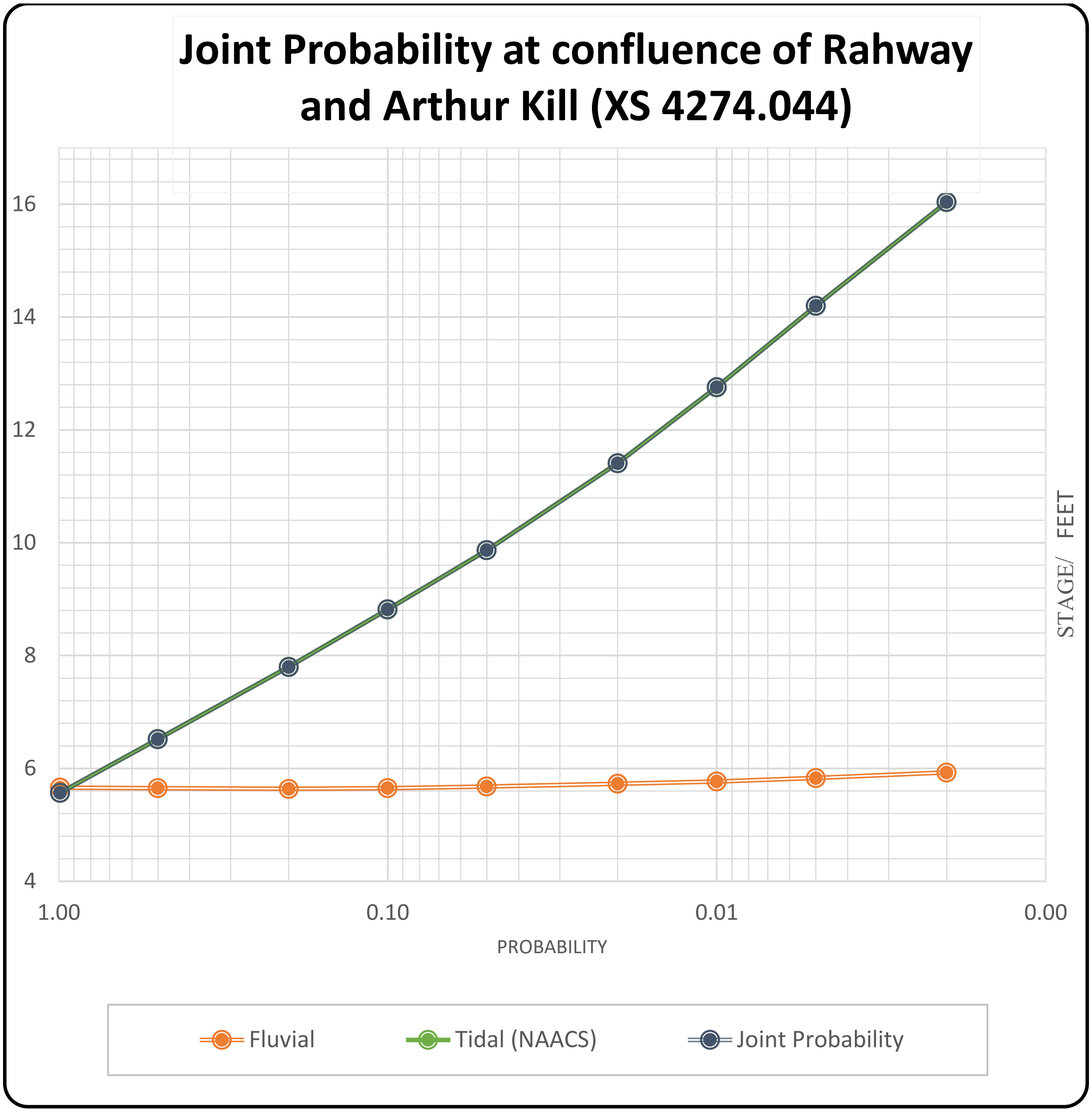


Figure 16: Joint Probability at confluence of Rahway River and Arthur Kill -cross section 4274.044.

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 17 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 18.

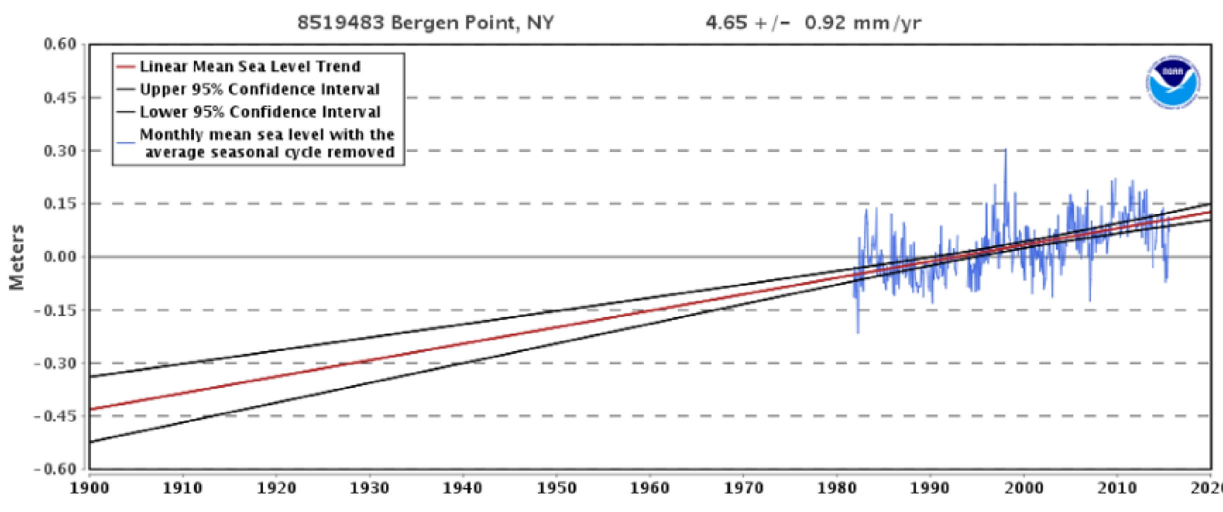


Figure 17: Sea level rise trends and monthly mean seal level at NOAA tide gage No. 8519483 at Bergen Point.

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

Year	VLM (ft.)	NET SLR (ft.)		
		Low	Intermediate	High
1992	0.00	0.00	0.00	0.00
2018	0.25	0.40	0.46	0.65
2023	0.30	0.47	0.56	0.83
2028	0.35	0.55	0.66	1.03
2033	0.40	0.63	0.77	1.25
2038	0.45	0.70	0.89	1.49
2043	0.49	0.78	1.01	1.74
2048	0.54	0.85	1.13	2.02
2053	0.59	0.93	1.26	2.31
2058	0.64	1.01	1.39	2.62
2063	0.69	1.08	1.53	2.95
2068	0.74	1.16	1.67	3.30
2073	0.78	1.24	1.82	3.67
2118	1.22	1.92	3.33	7.81

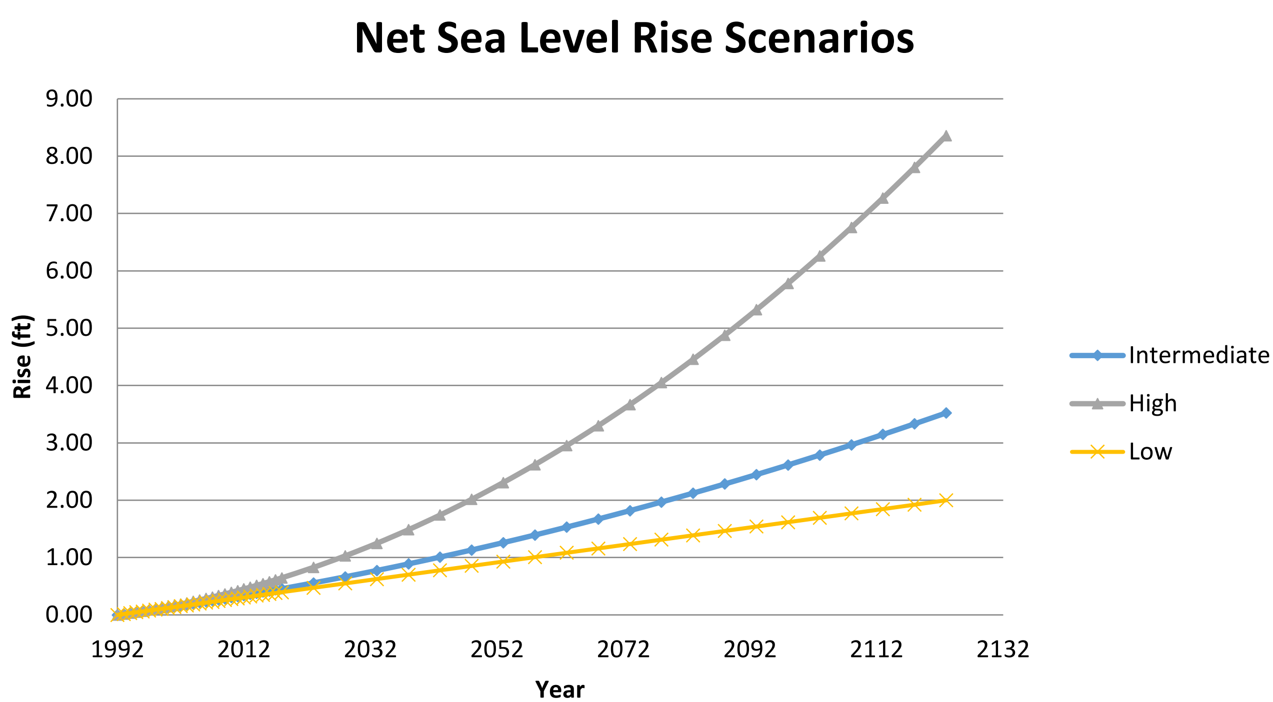


Figure 18: Projected SLC at Rahway for the local (low), NRC Type I (Intermediate), and NRC Type III (high) scenarios.

Sea level rise is expected to have impacts on direct coastal flooding along the Rahway River coastal 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. From the base feasibility study date of 2015, projected 50 years from the end of construction date of 2023, the sea level will rise 1.24 feet by 2073. 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.

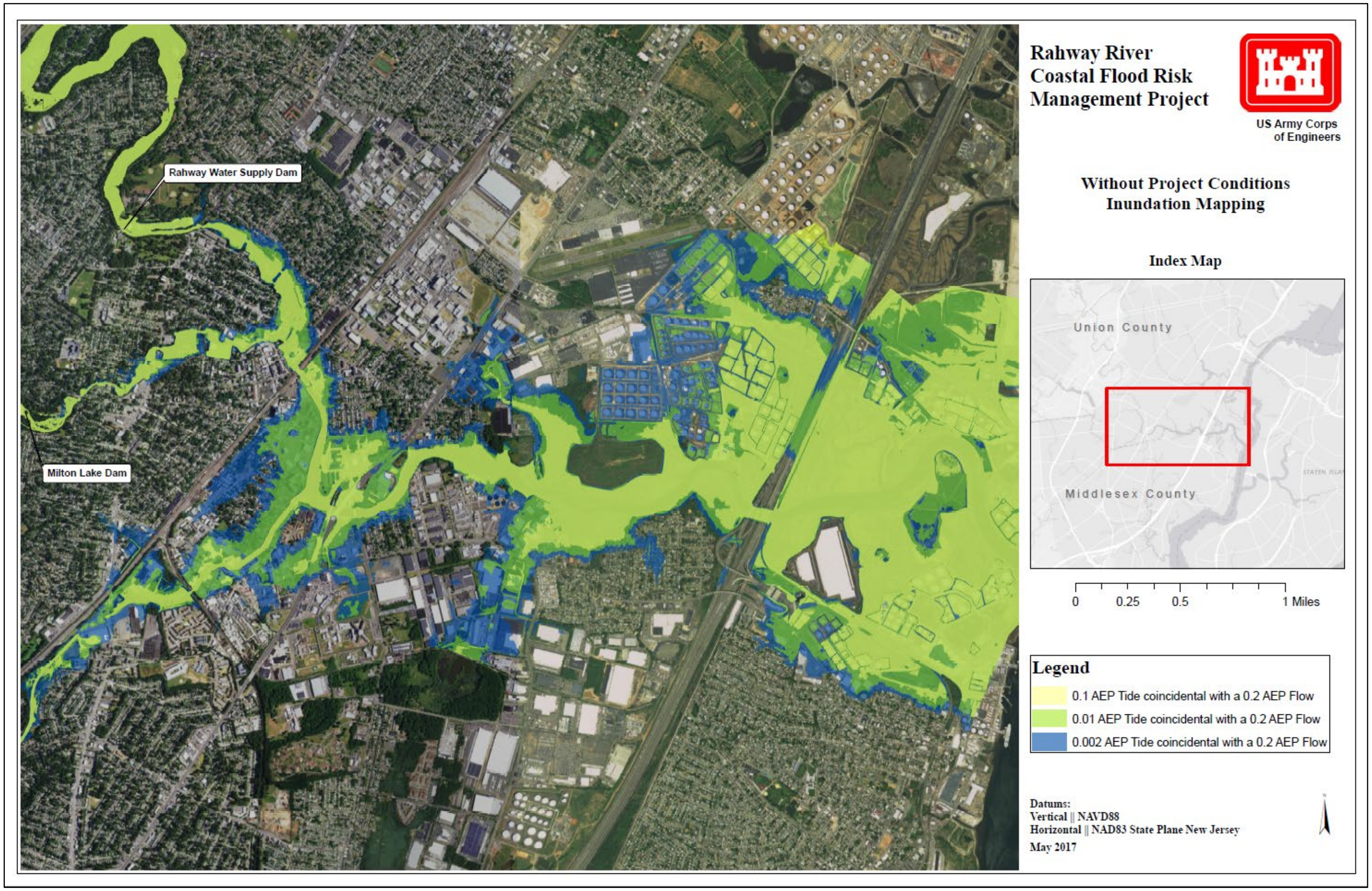
3.4 Present and Future Conditions – Hydraulic Profiles

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 19.

Future conditions hydraulics and hydrology includes sea level change, vertical land movement, and urbanization. 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 coastal area of study at the downstream boundary condition. The future conditions model was created using future hypothetical peak discharges, changes in MSL due to VLM and SLC, and the calibrated existing conditions HEC-RAS model. Both the increase in flow and tide elevations cause an increase in flooding for future without project conditions in the coastal area. Increased flows due to urbanization only have an impact in the coastal area up to the 0.2 (5-year) AEP event, with negligible impact near the mouth of the Rahway River. Coastal 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.

Figure 20 through 28 show the present "without project" WSE profiles for the Rahway River in the coastal 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) coastal events. Also shown is the Sandy WSE profile for reference.



*Note: This is USACE-NACCS inundation of coastal events coincidental with 0.2 AEP fluvial flow. This does not represent joint-probability inundations.

Figure 19: "Without project" present condition inundation map for the 0.1, 0.01, and 0.002 AEP events.

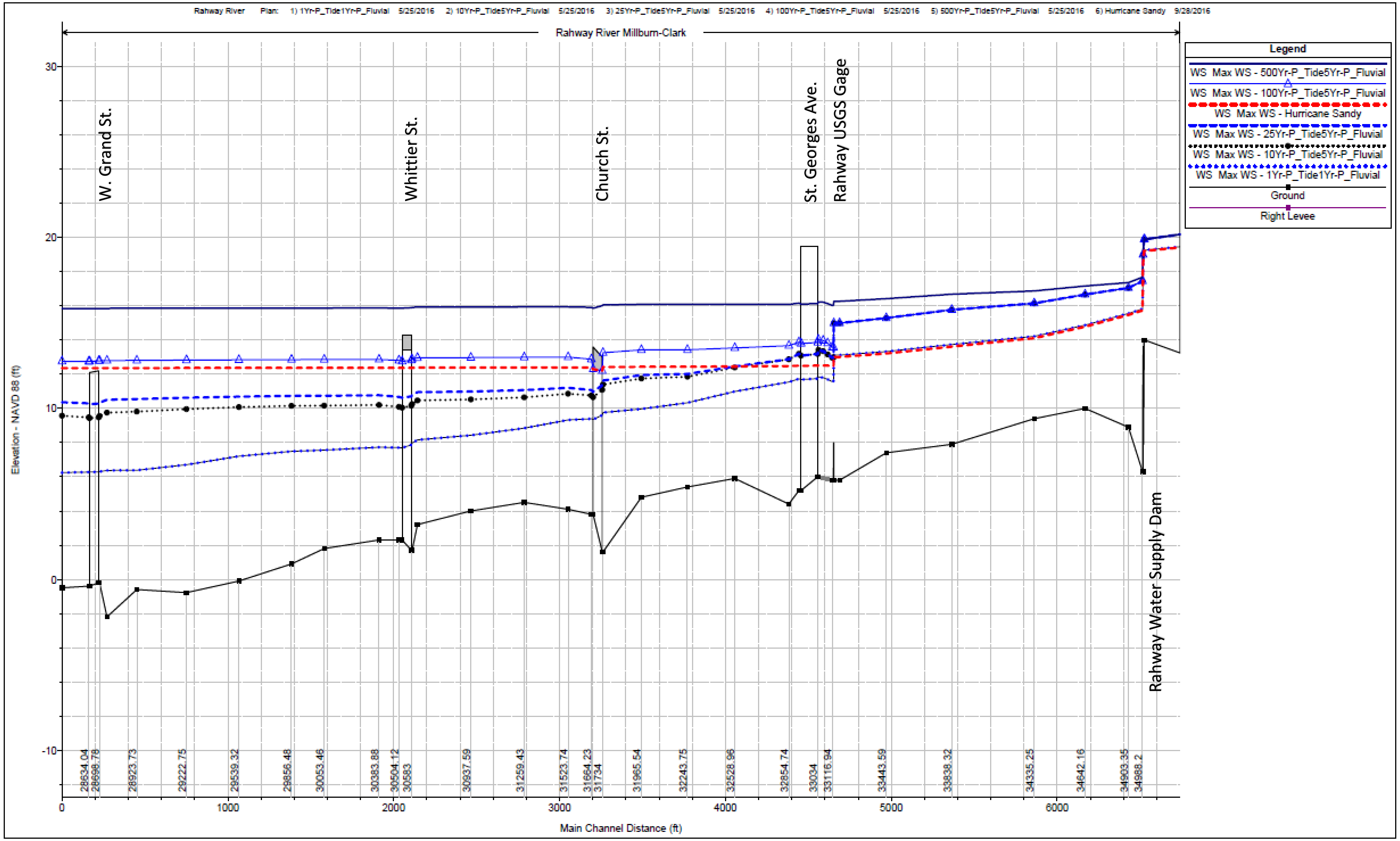


Figure 20: "Without project" condition computed water surface profile from Rahway Water Supply to Robinson's Branch Confluence.

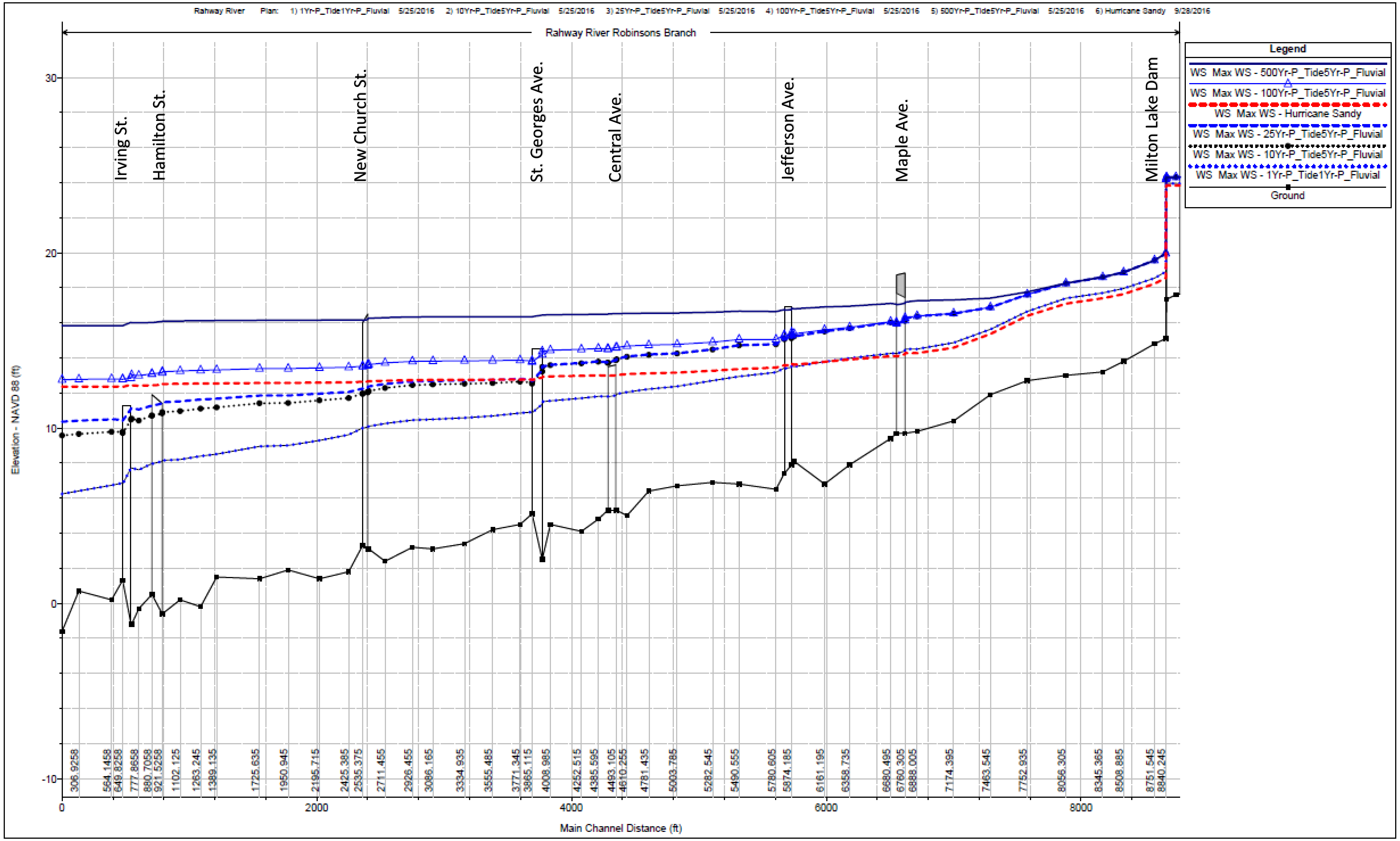


Figure 21: "Without project" condition computed water surface profile for Robinson's Branch from Milton Lake Dam to the confluence.

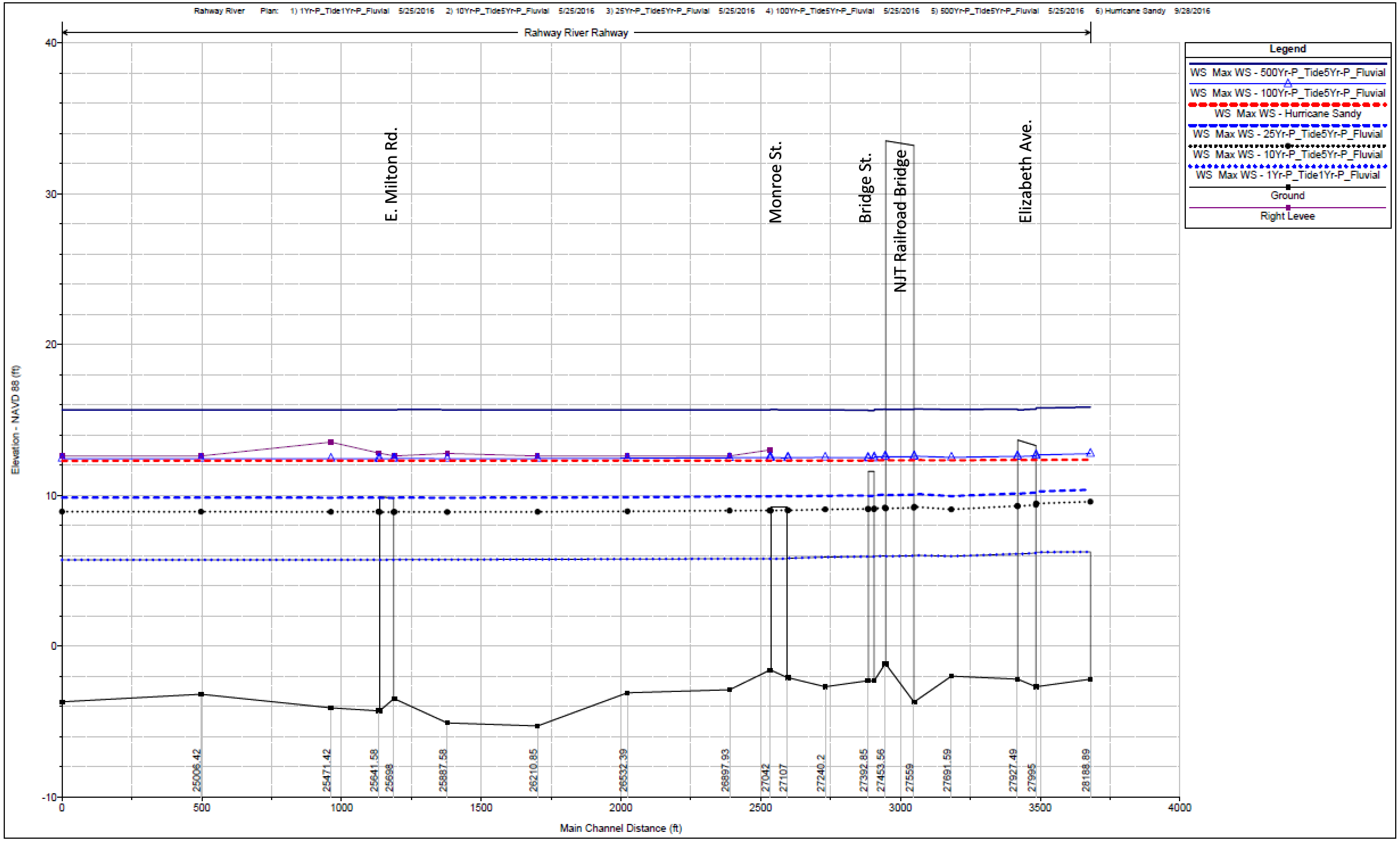


Figure 22: “Without project” condition computed water surface profile for Rahway River from Robinson’s Branch Confluence to South Branch Confluence.

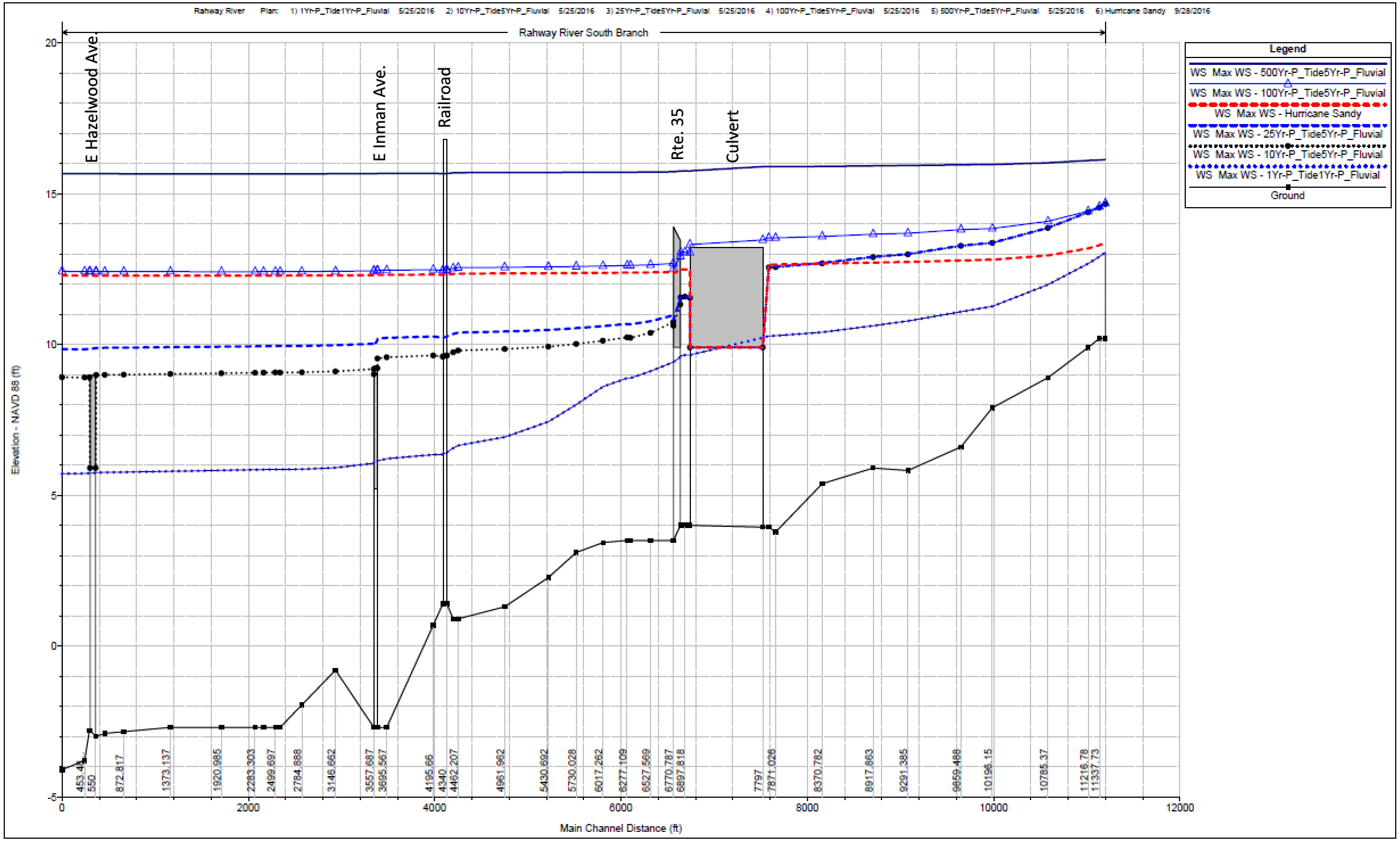


Figure 23: "Without project" condition computed water surface profile for South Branch.

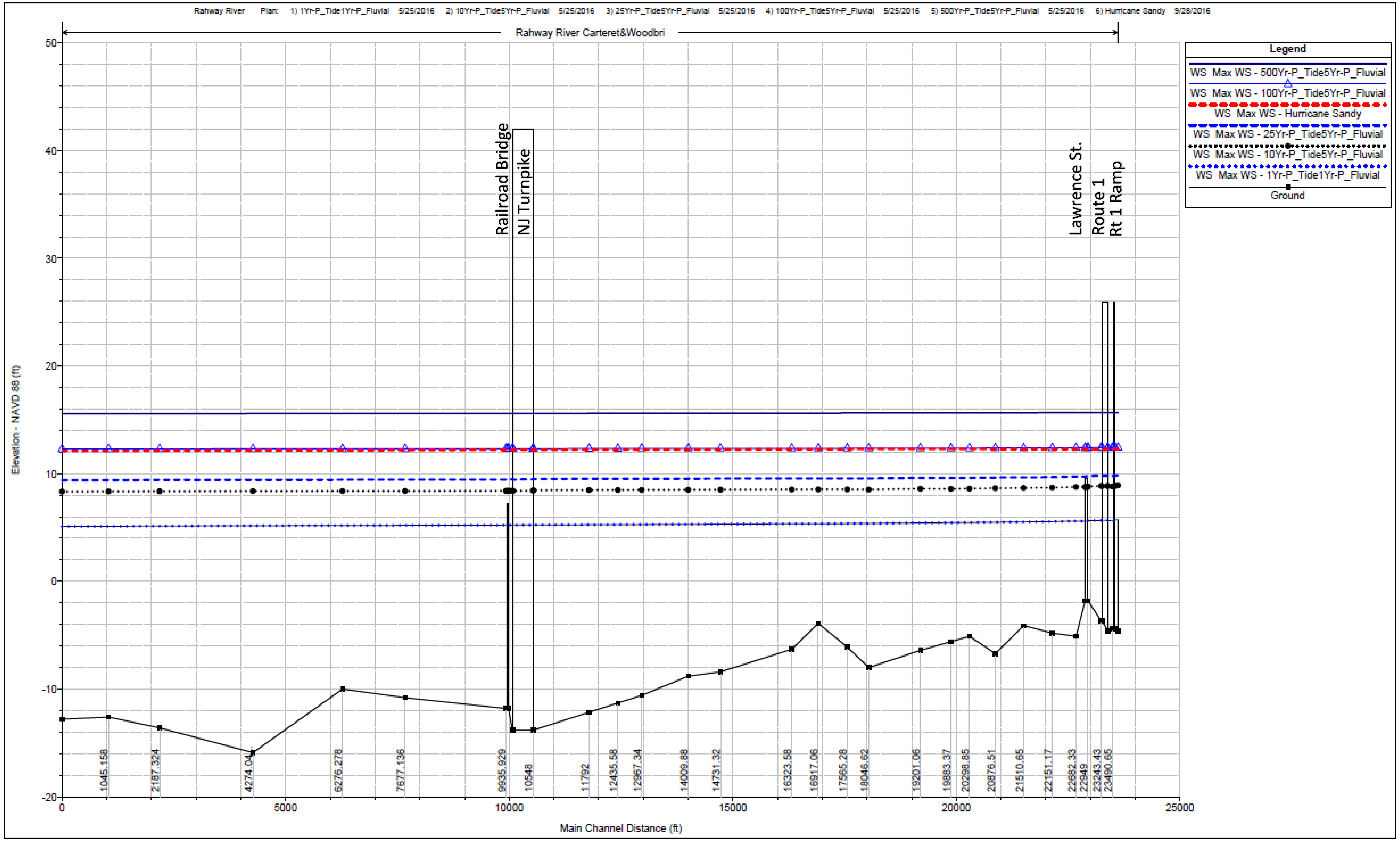


Figure 24: “Without project” condition computed water surface profile for Rahway River from South Branch Confluence to the Arthur Kill.

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 25 for the overview of the alternative and Figure 26 and Figure 27 for the plan layout of each component.

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 on 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 8.6 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.

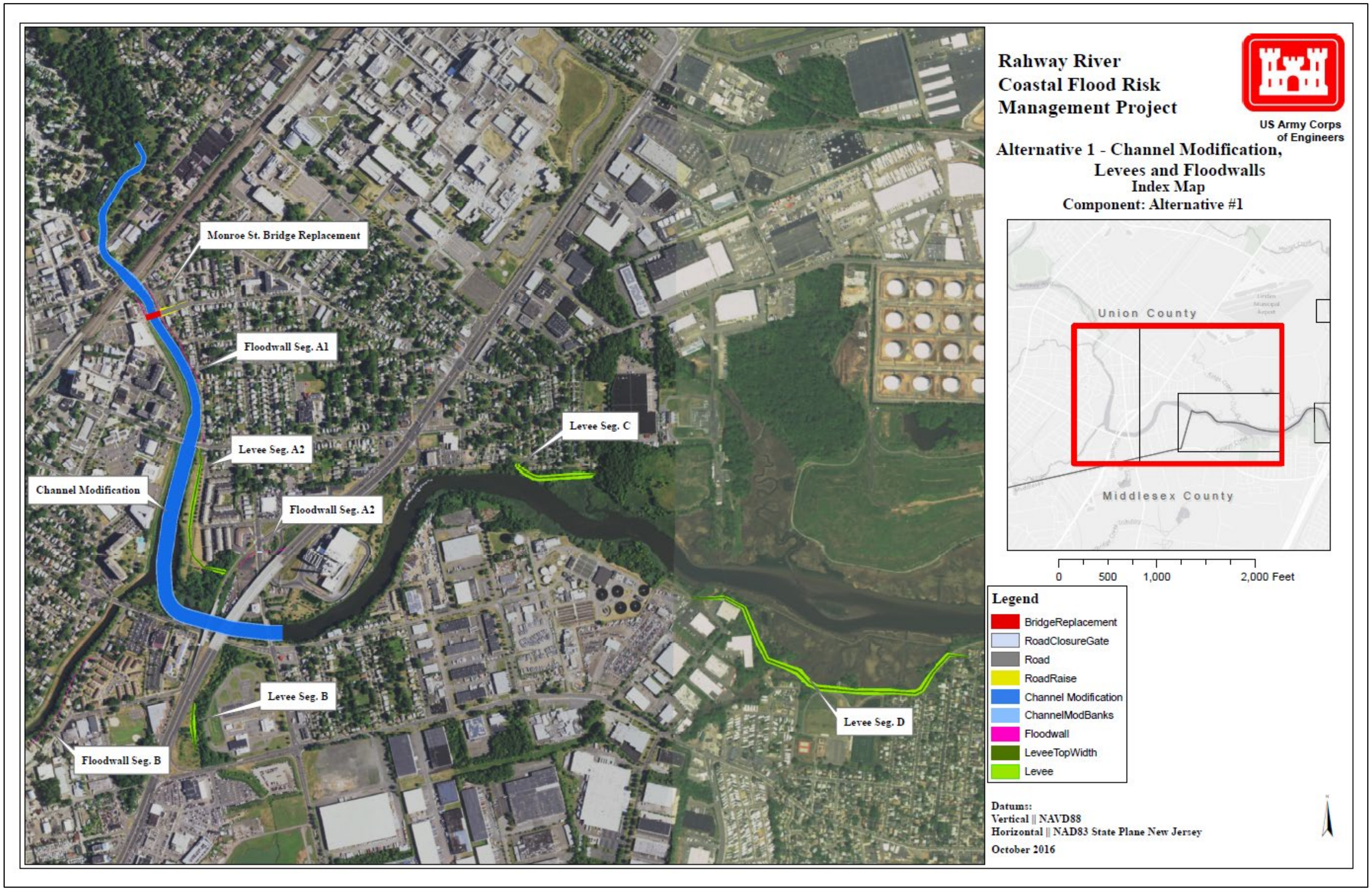


Figure 25: Alternative #1 Plan Overview

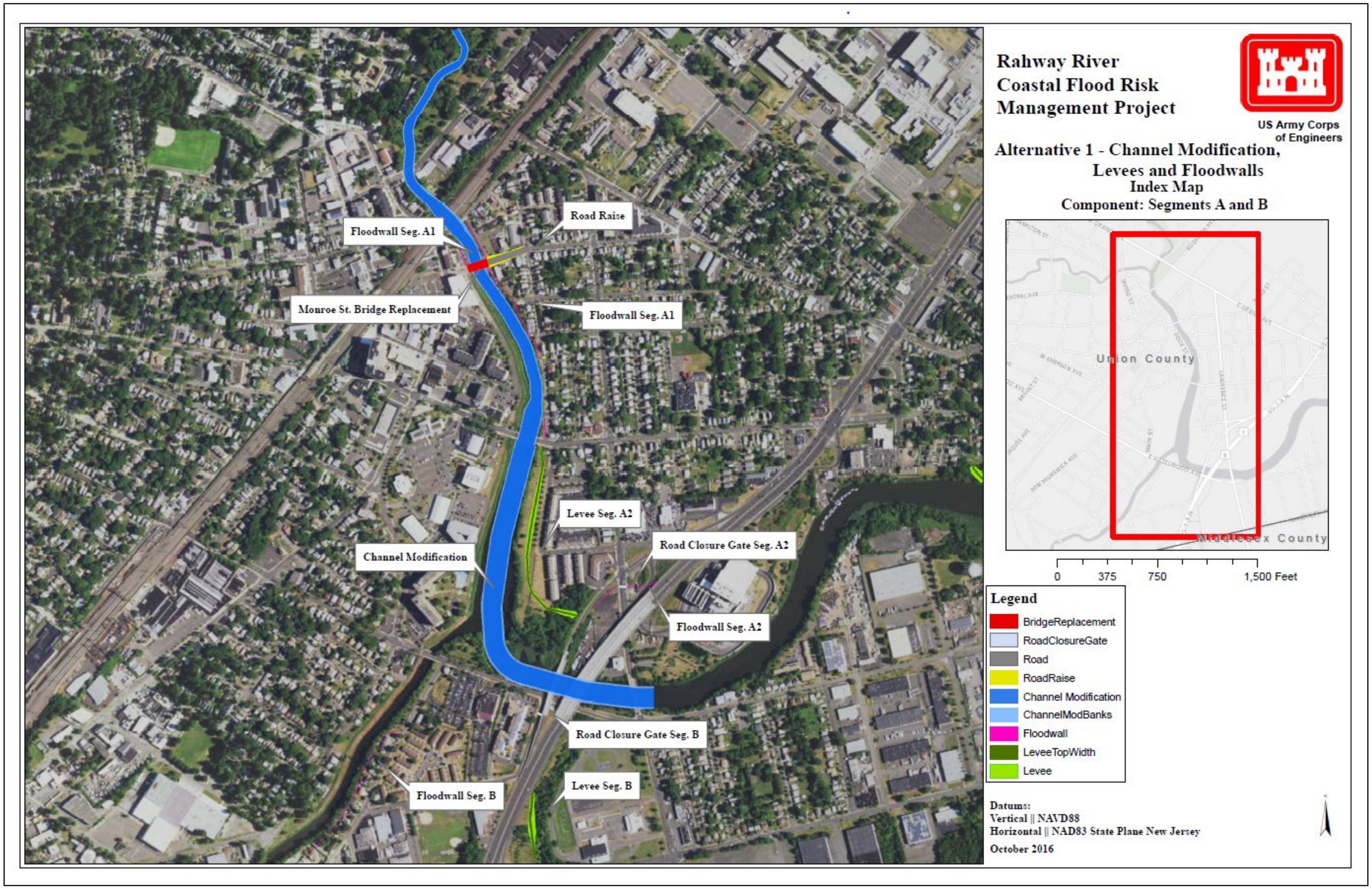


Figure 26: Alternative #1 Segments A and B layout.

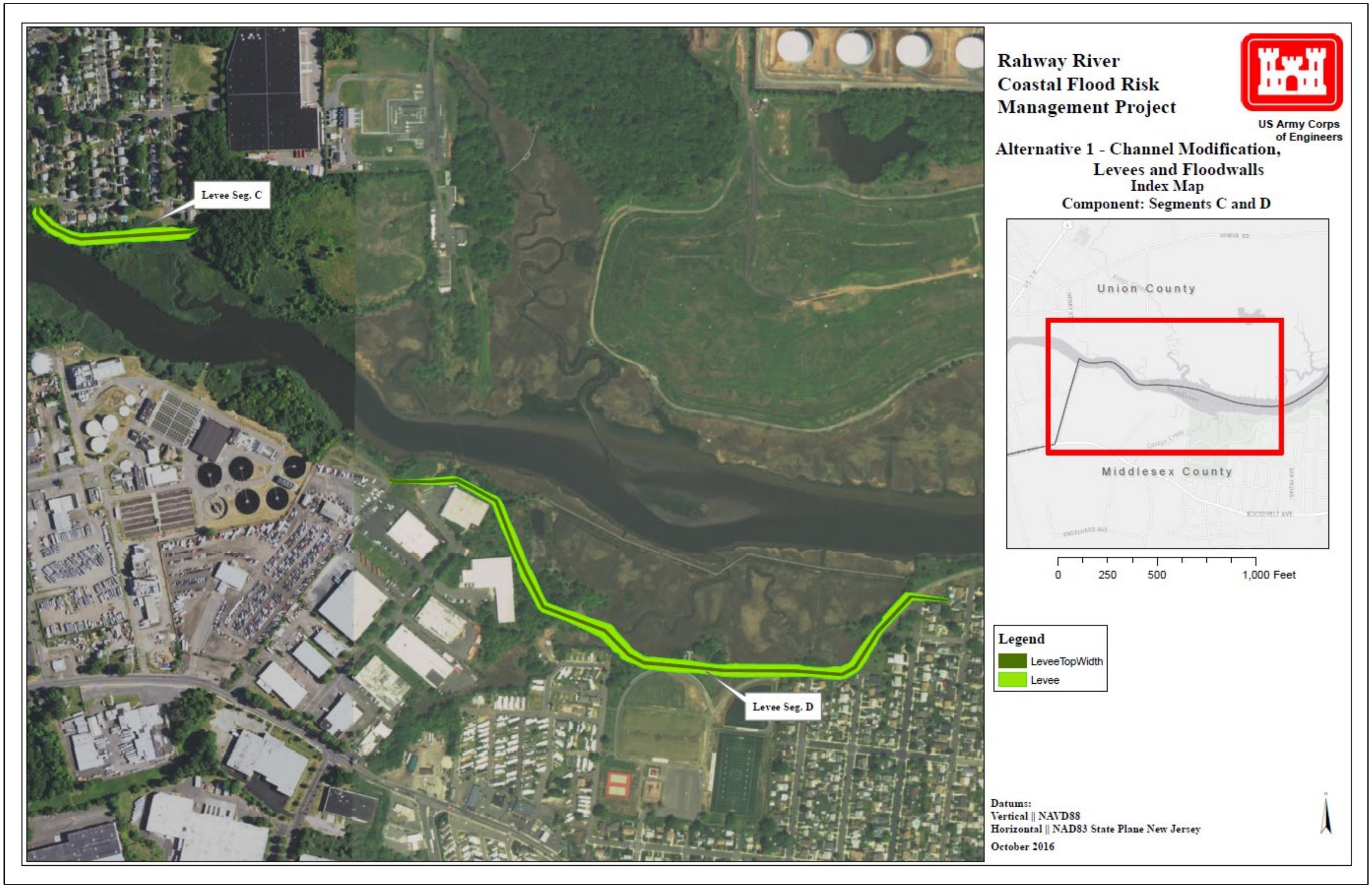


Figure 27: Alternative #1 Segments C and D layout.

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. Channel modification would involve channel widening, deepening and stabilization of channel banks. This is represented in the hydraulic model by lower mannings n-value (0.025) and cuts to the channel cross sections. The downside of channel modification is the lack flood reduction during coastal events, which control most of the damages at the mouth of the Rahway River, thus adding significant cost without providing necessary benefits. In addition, channel modification would also require environmental mitigation of contaminated areas along the Rahway River, adding to the total cost of the alternative. Figure 28 through Figure 32 show the present “with project” Alteranive #1 coastal WSE profiles.

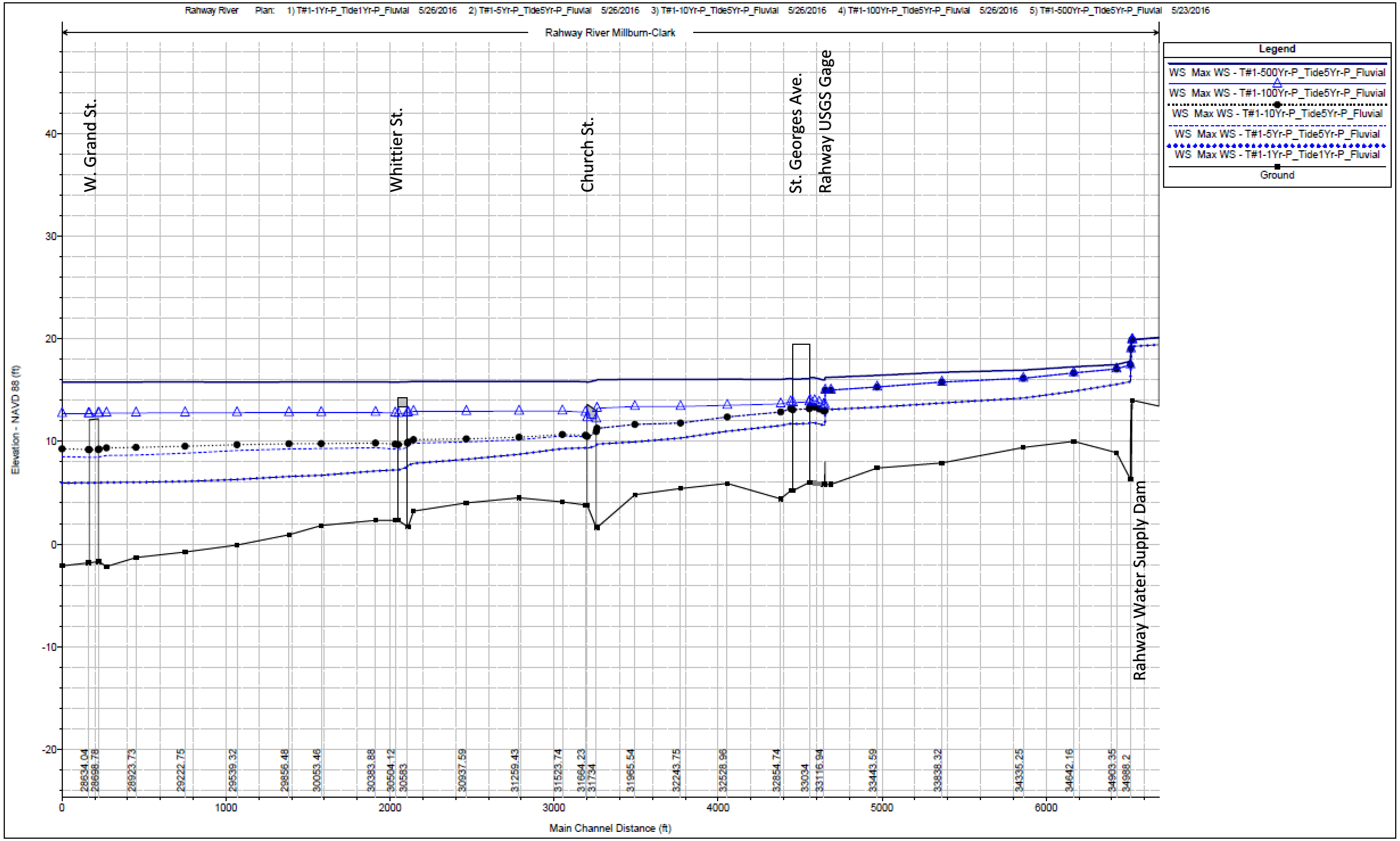


Figure 28: Alternative #1 computed water surface profile from Rahway Water Supply to Robinson's Branch Confluence.

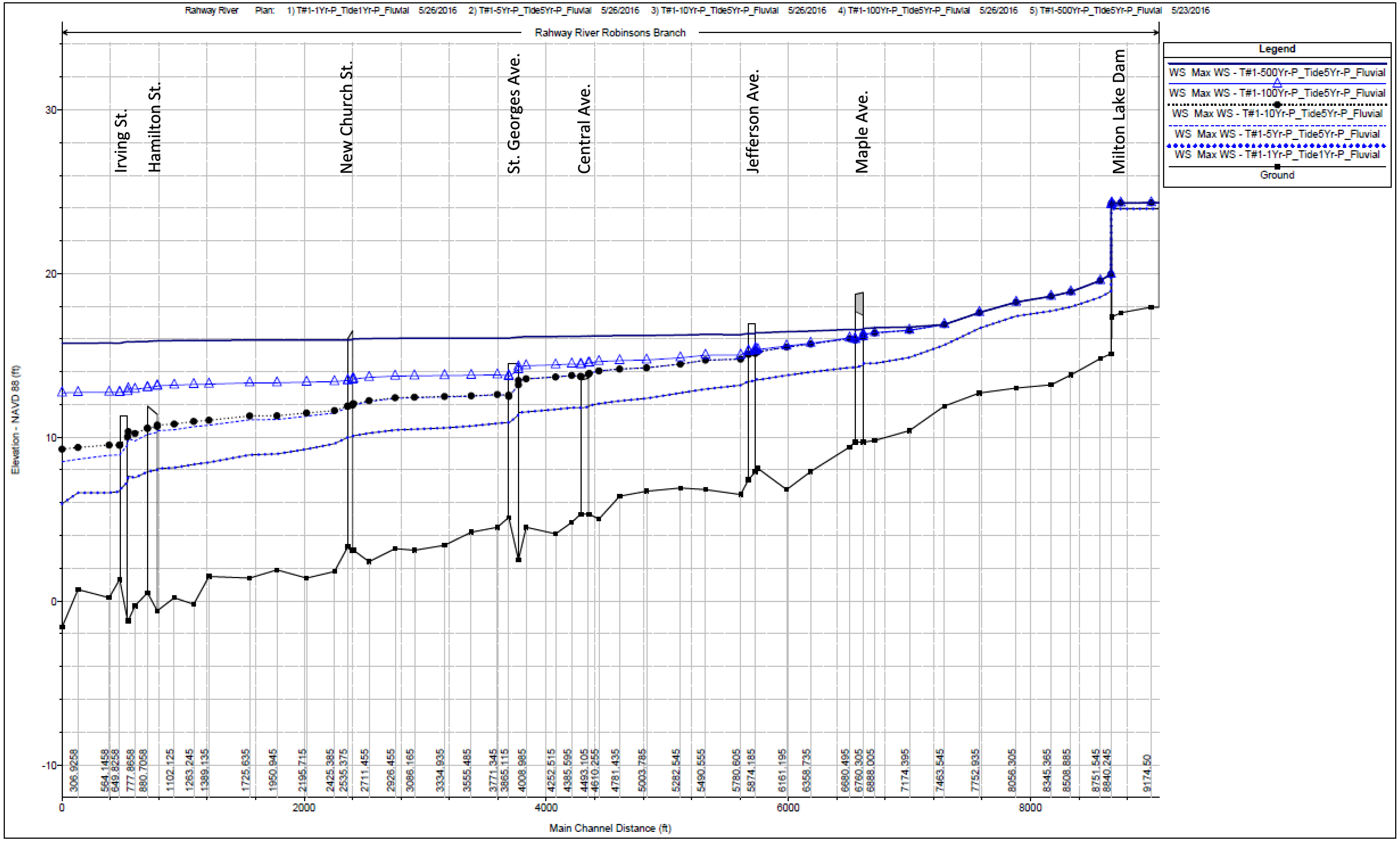


Figure 29: Alternative #1 computed water surface profile for Robinson's Branch from Milton Lake Dam to the confluence.

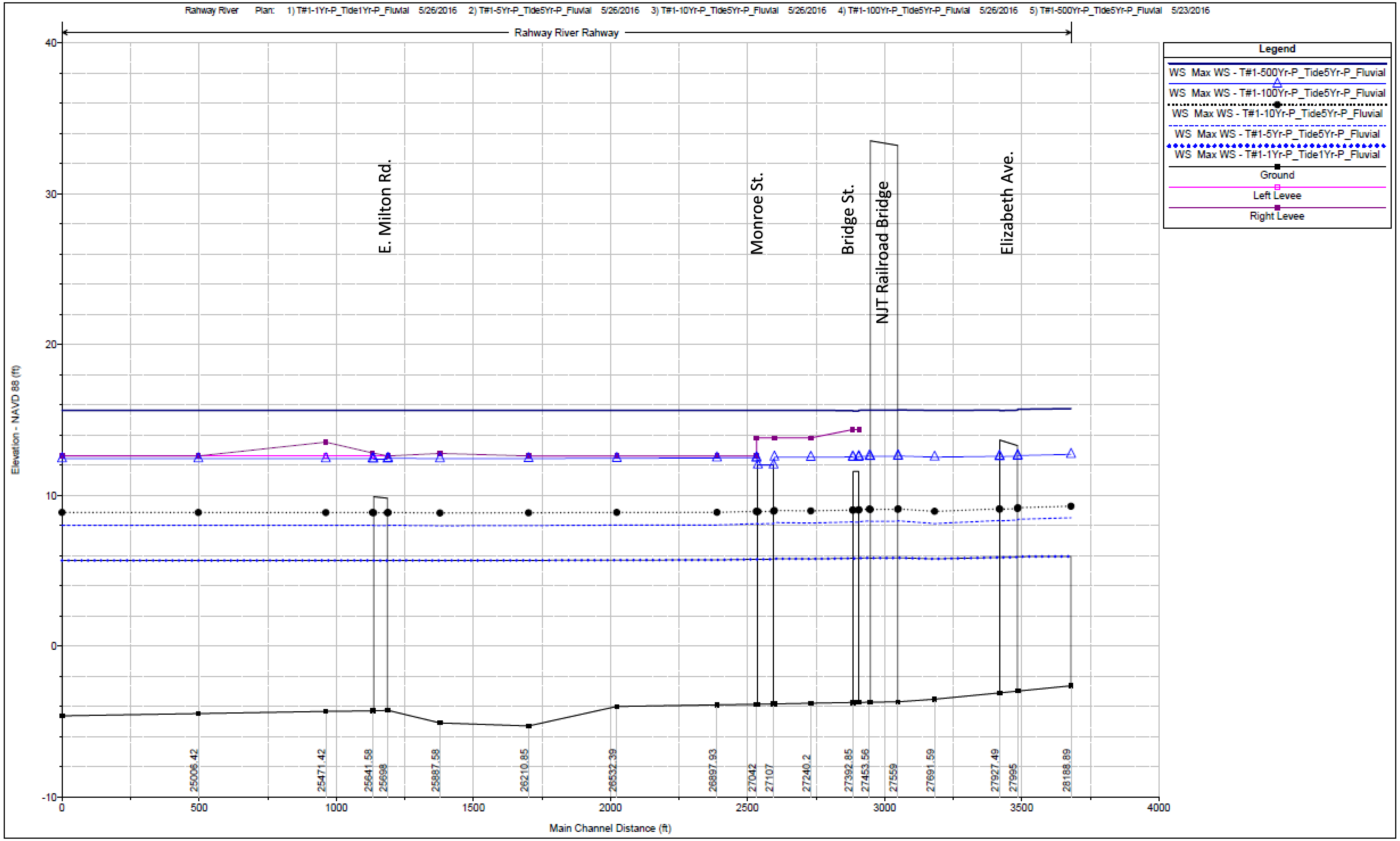


Figure 30: Alternative #1 computed water surface profile for Rahway River from Robinson's Branch Confluence to South Branch Confluence.

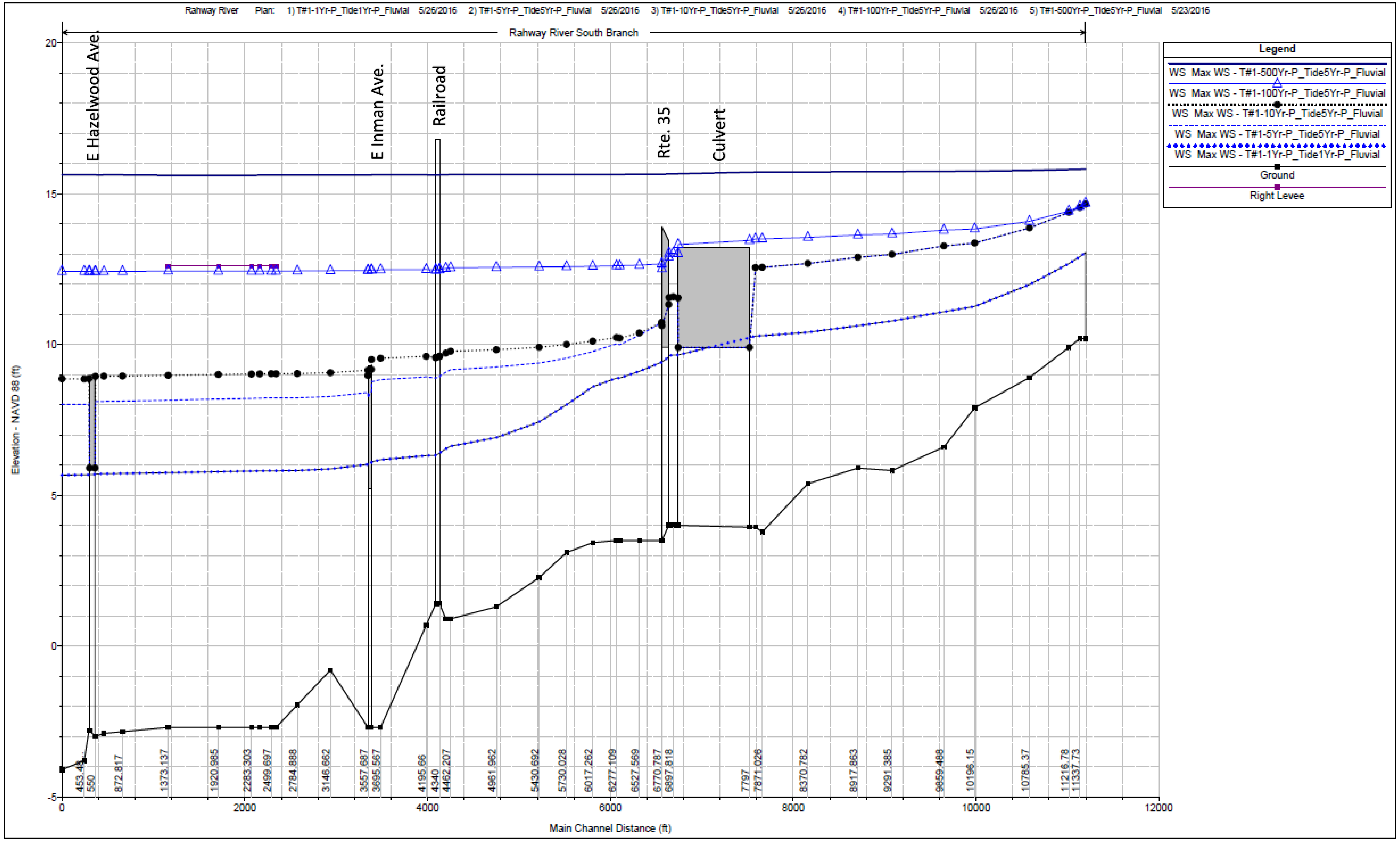


Figure 31: Alternative #1 computed water surface profile for South Branch.

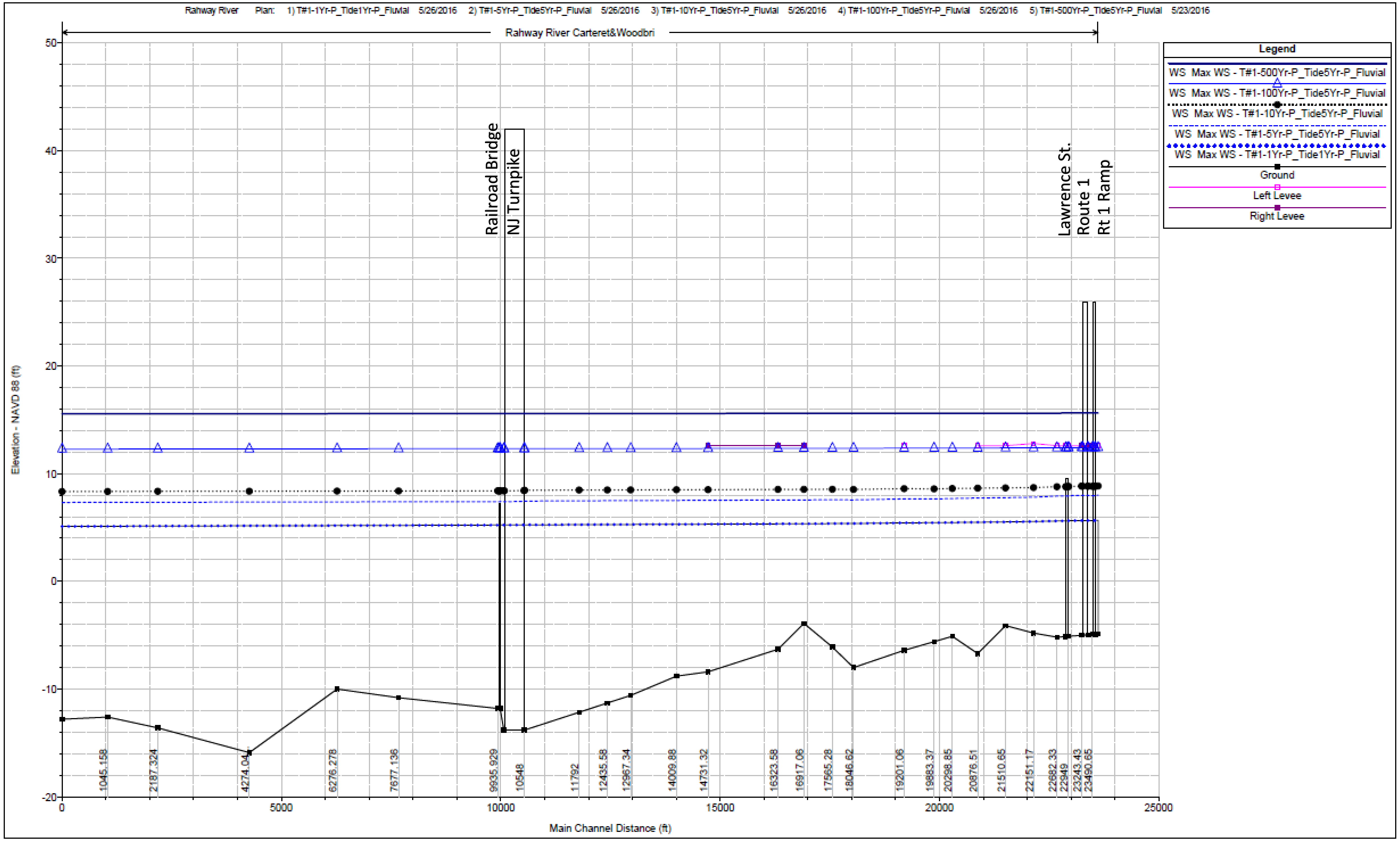


Figure 32: Alternative #1 computed water surface profile for Rahway River from South Branch Confluence to the Arthur Kill.

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 expected to have a 0.01 annual exceedance probability. See Figure 33 for the overview of the alternative and Figure 34, Figure 35, and Figure 36 for the plan layout of each component.

Figure 33: Alternative #2 Plan Overview.

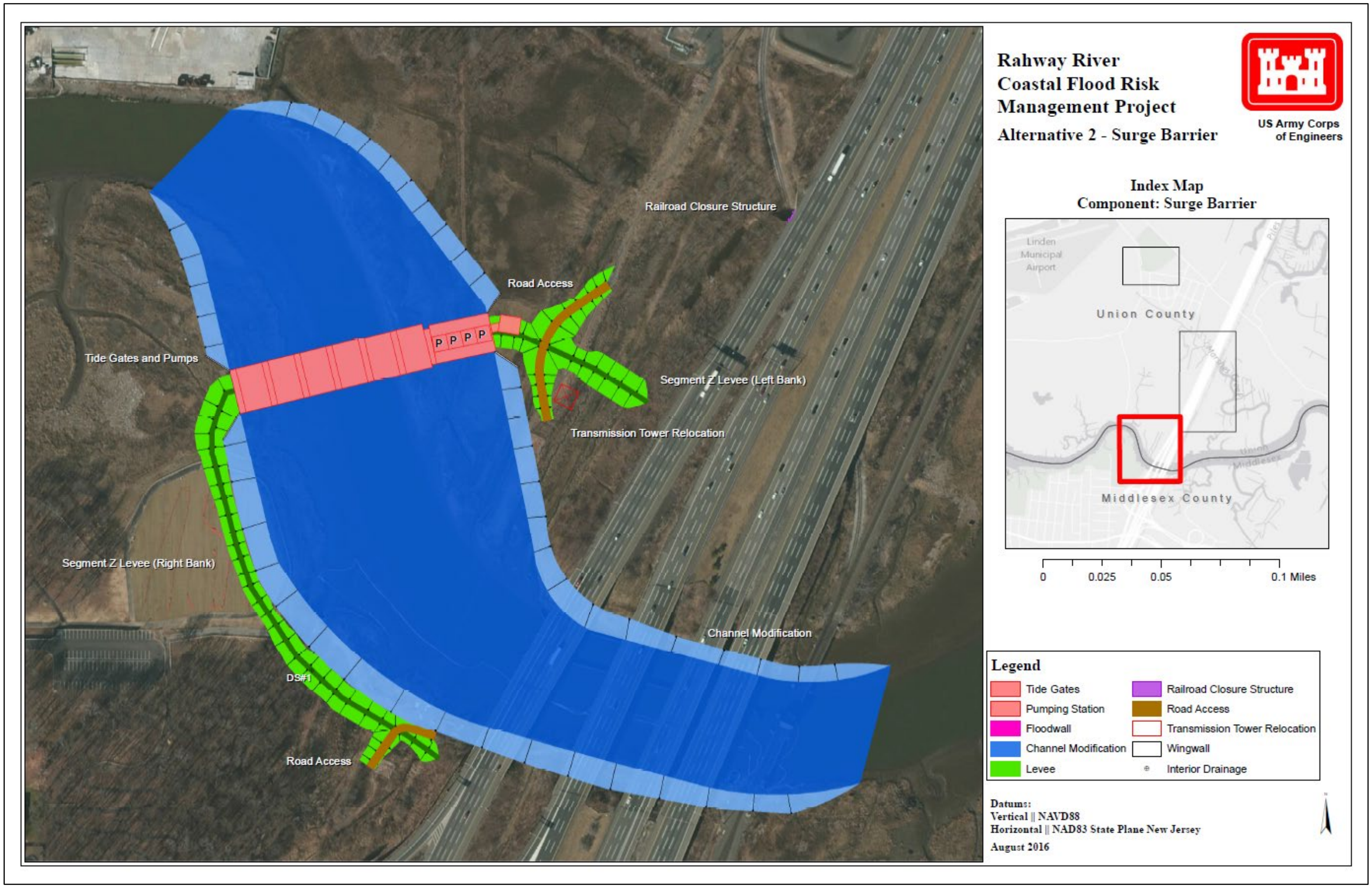


Figure 34: Alternative #2 Surge Barrier (Gates, Pumps, Levees, Channel Modification) layout.

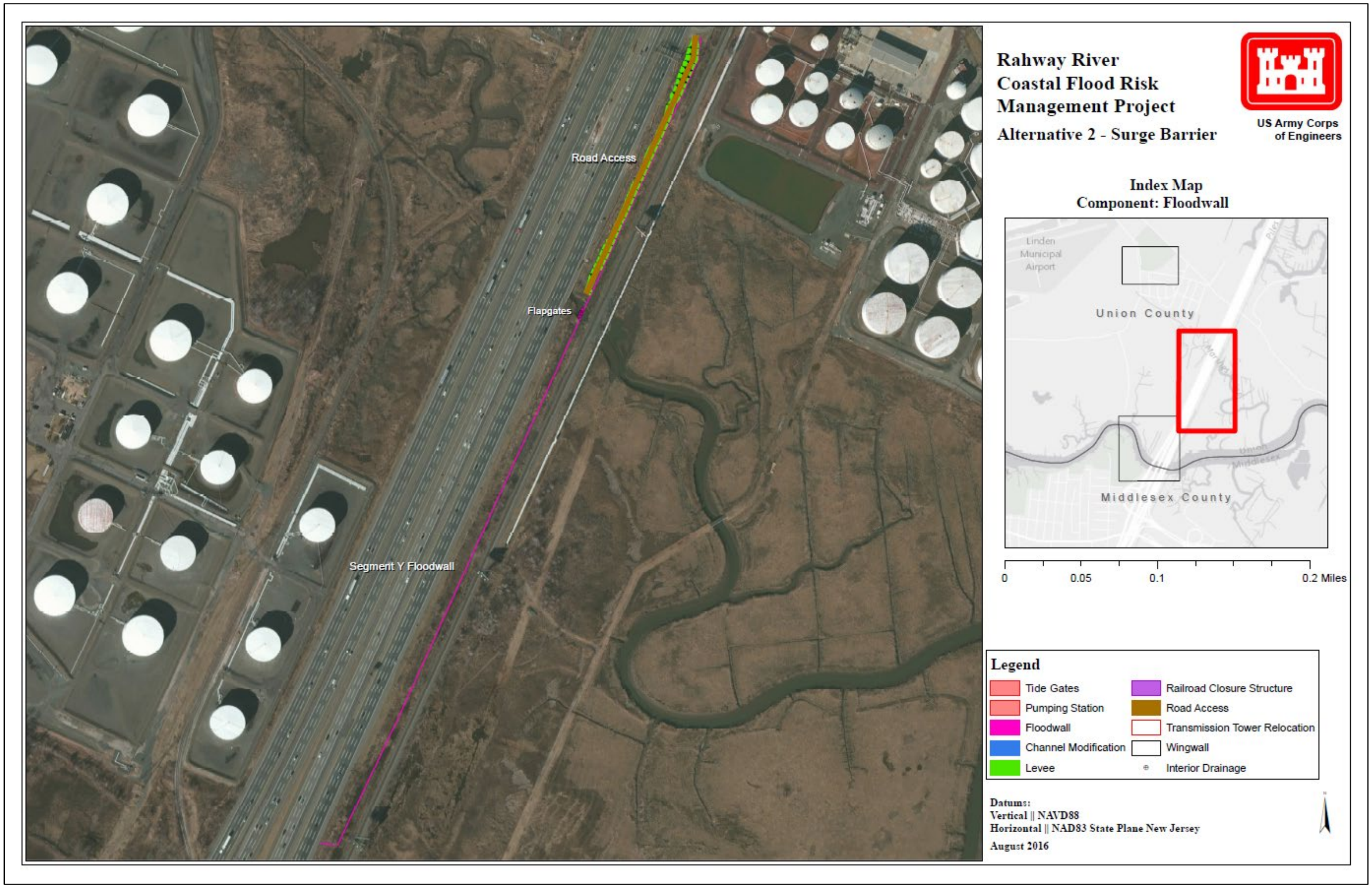


Figure 35: Alternative #2 Floodwall along Turnpike Northbound layout.

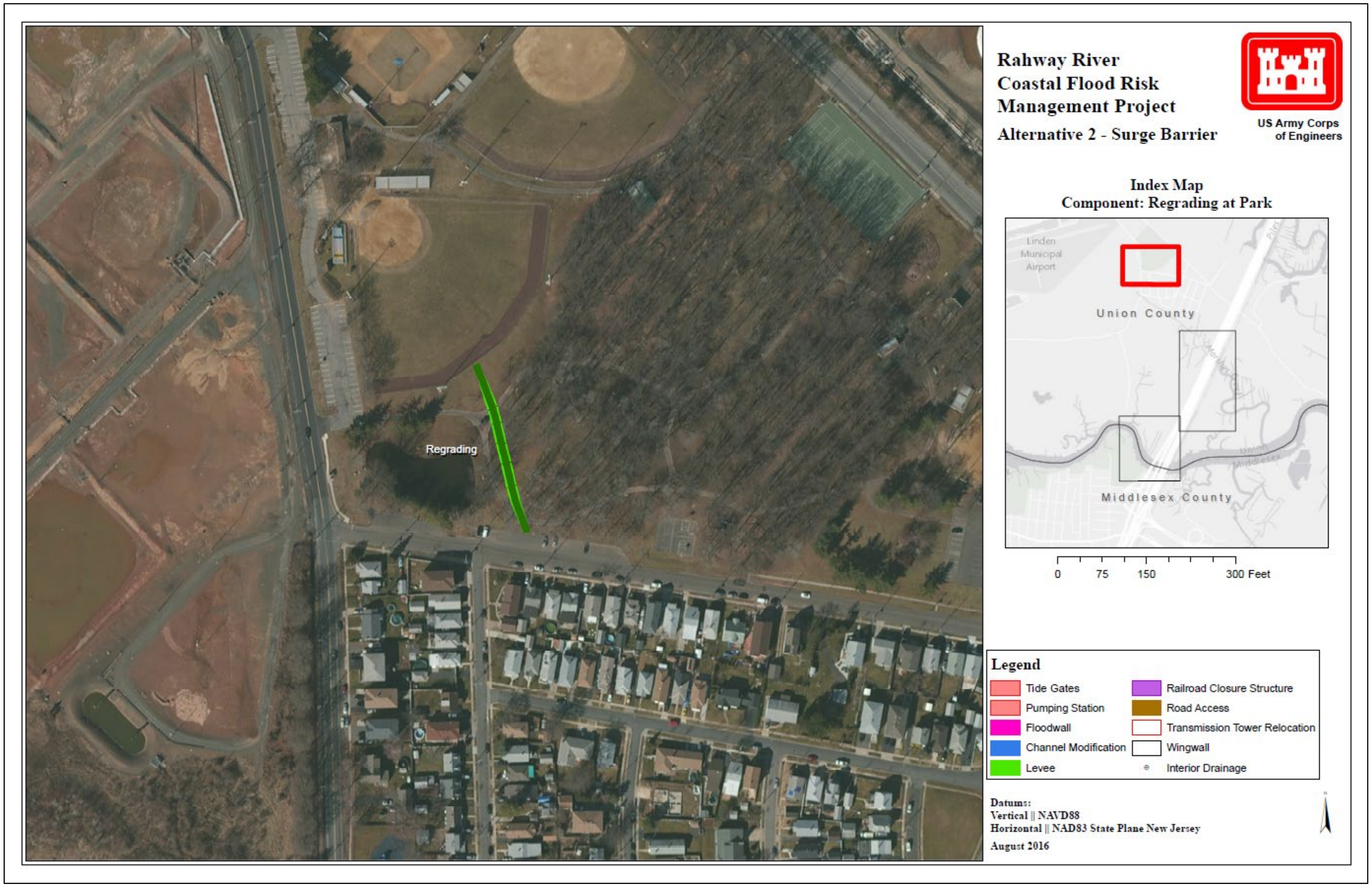


Figure 36: Alternative #2 re-grading at Memorial Field Park layout.

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 coastal 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) Re-grading approximately 300 linear ft of Memorial Field Park in Linden, NJ to an elevation of 13 ft NAVD '88,

(3) Three manual flap gates 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) Re-locating 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 flap gates 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 re-grading will prevent elevated water levels in the nearby basin from causing flooding in the Rahway coastal 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 37). 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 coastal 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) coastal event or less.

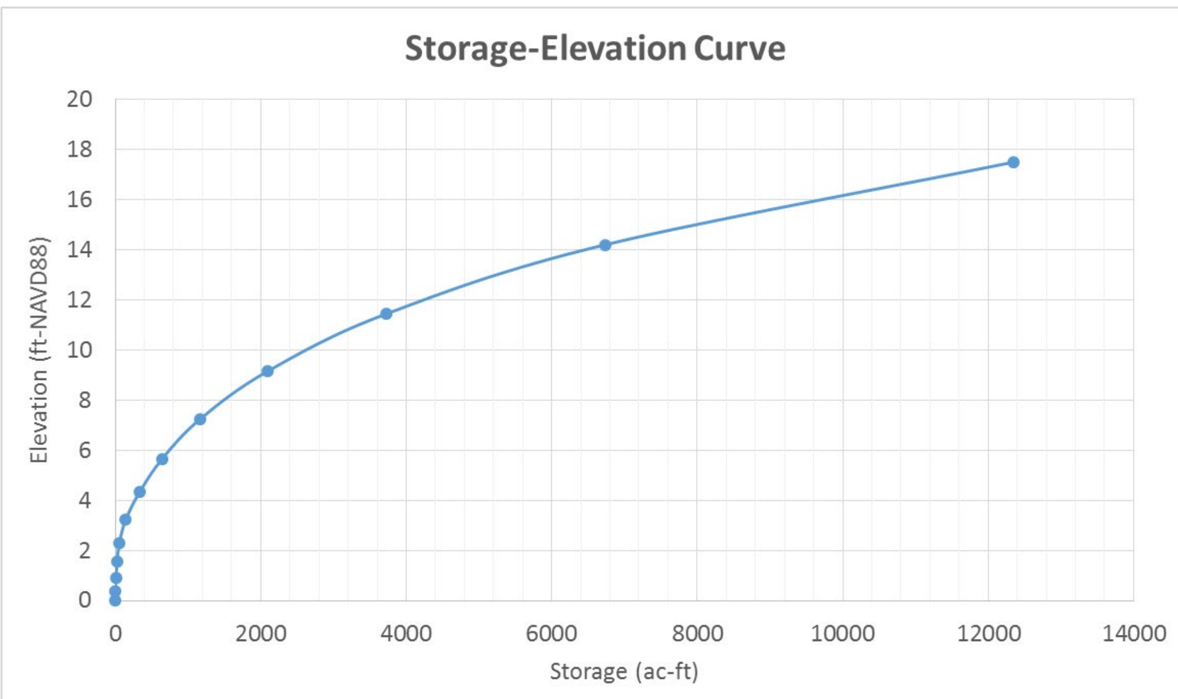


Figure 37: Storage-Elevation curve showing capacity of Coastal area of study.

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 38 through Figure 42 show the present “with project” coastal WSE profiles. Refer to Figure 20 through Figure 24 in Section 3.4 “Present and Future Conditions – Hydraulic Profiles” for the present “without project” coastal WSE profiles.

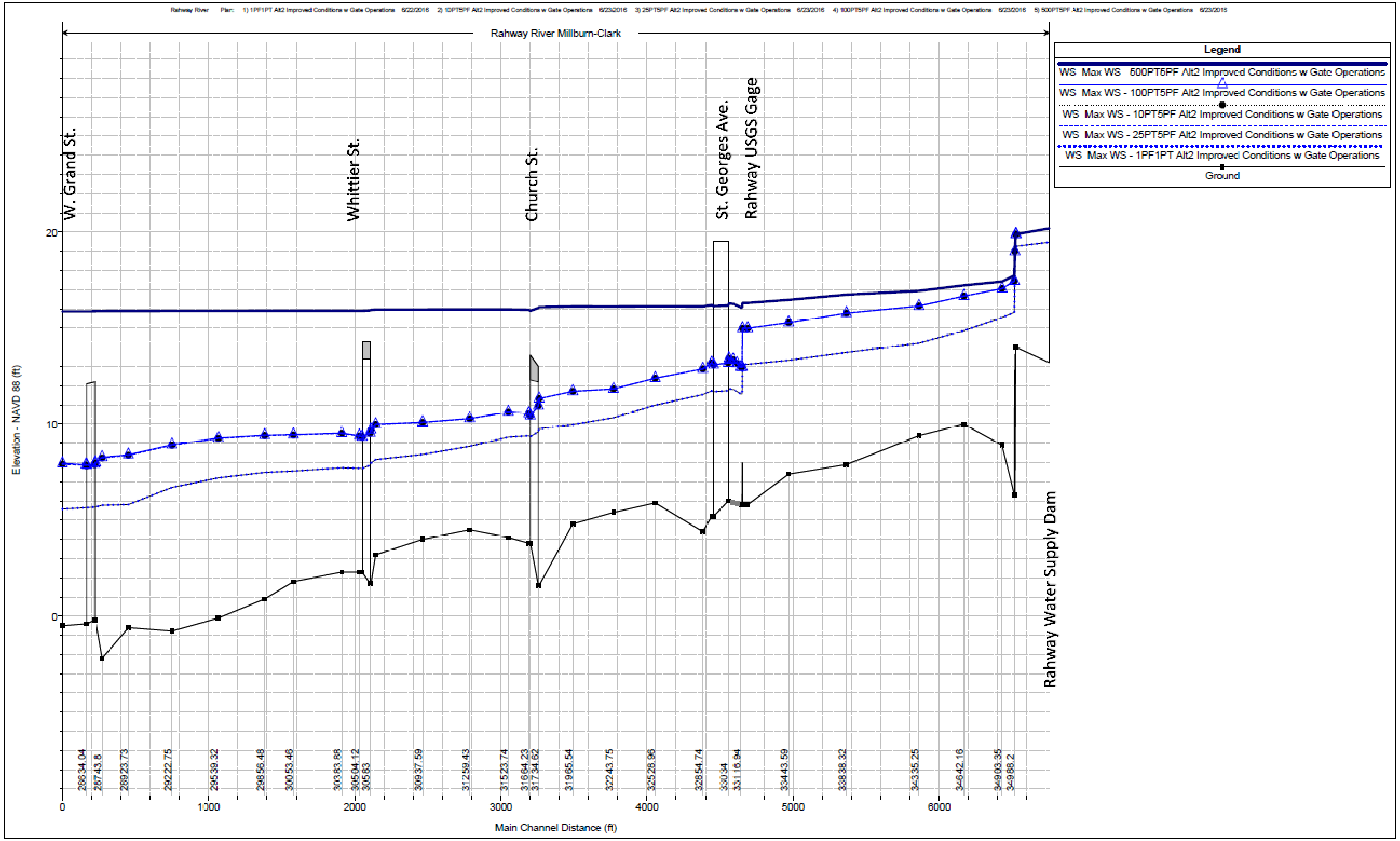


Figure 38: Alternative #2 computed water surface profile from Rahway Water Supply to Robinson's Branch Confluence.

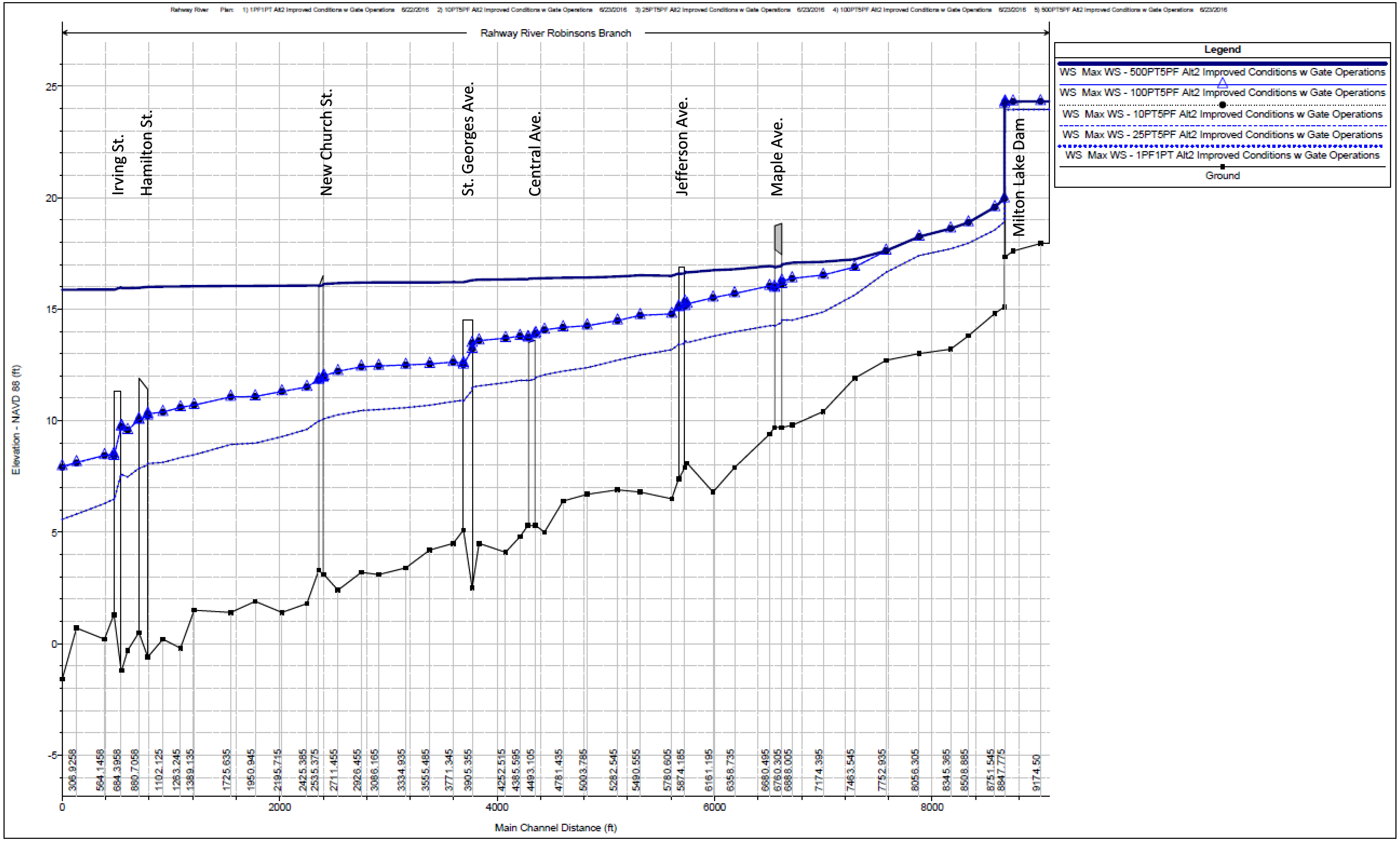


Figure 39: Alternative #2 computed water surface profile for Robinson's Branch from Milton Lake Dam to the confluence.

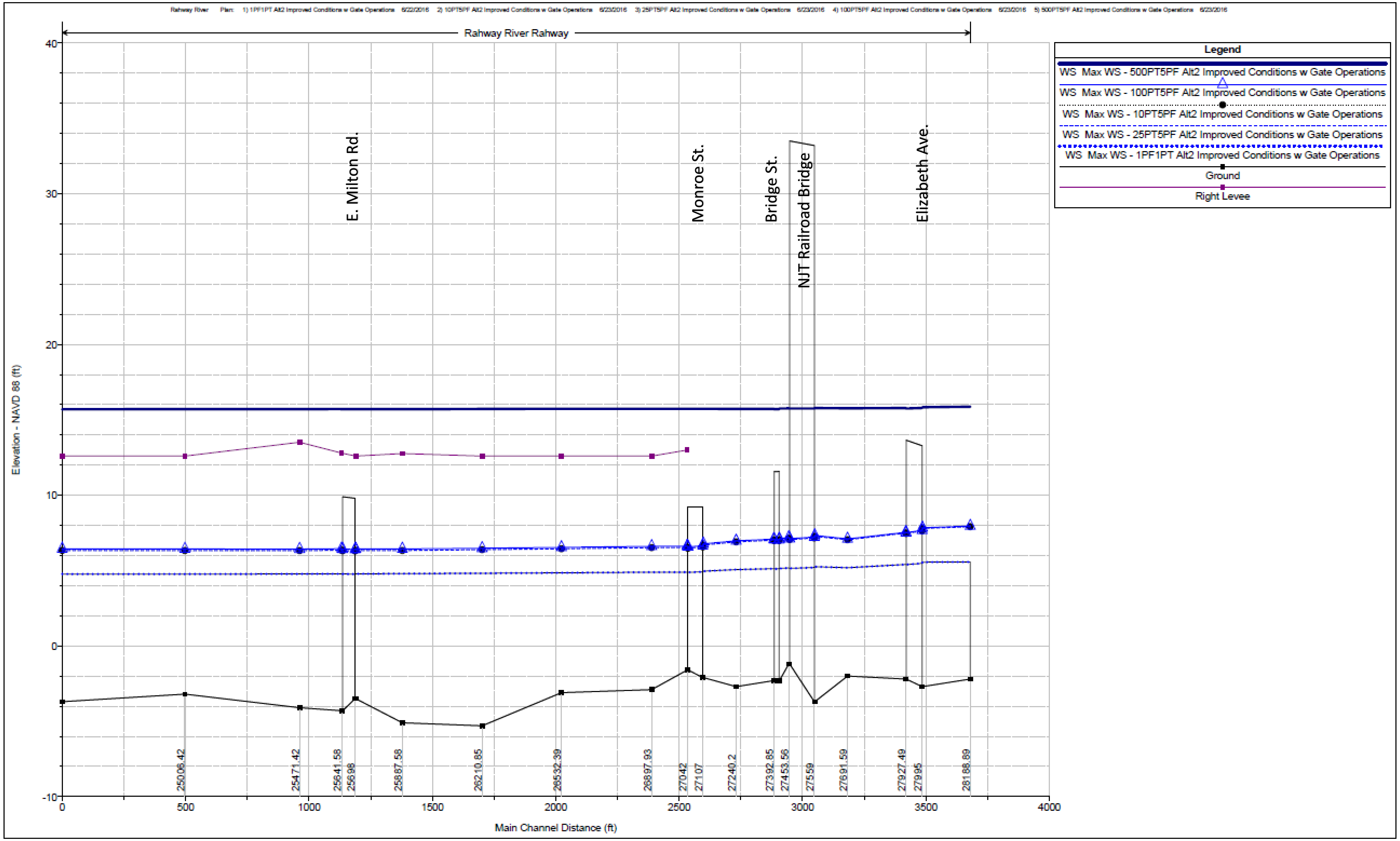


Figure 40: Alternative #2 computed water surface profile for Rahway River from Robinson's Branch Confluence to South Branch Confluence.

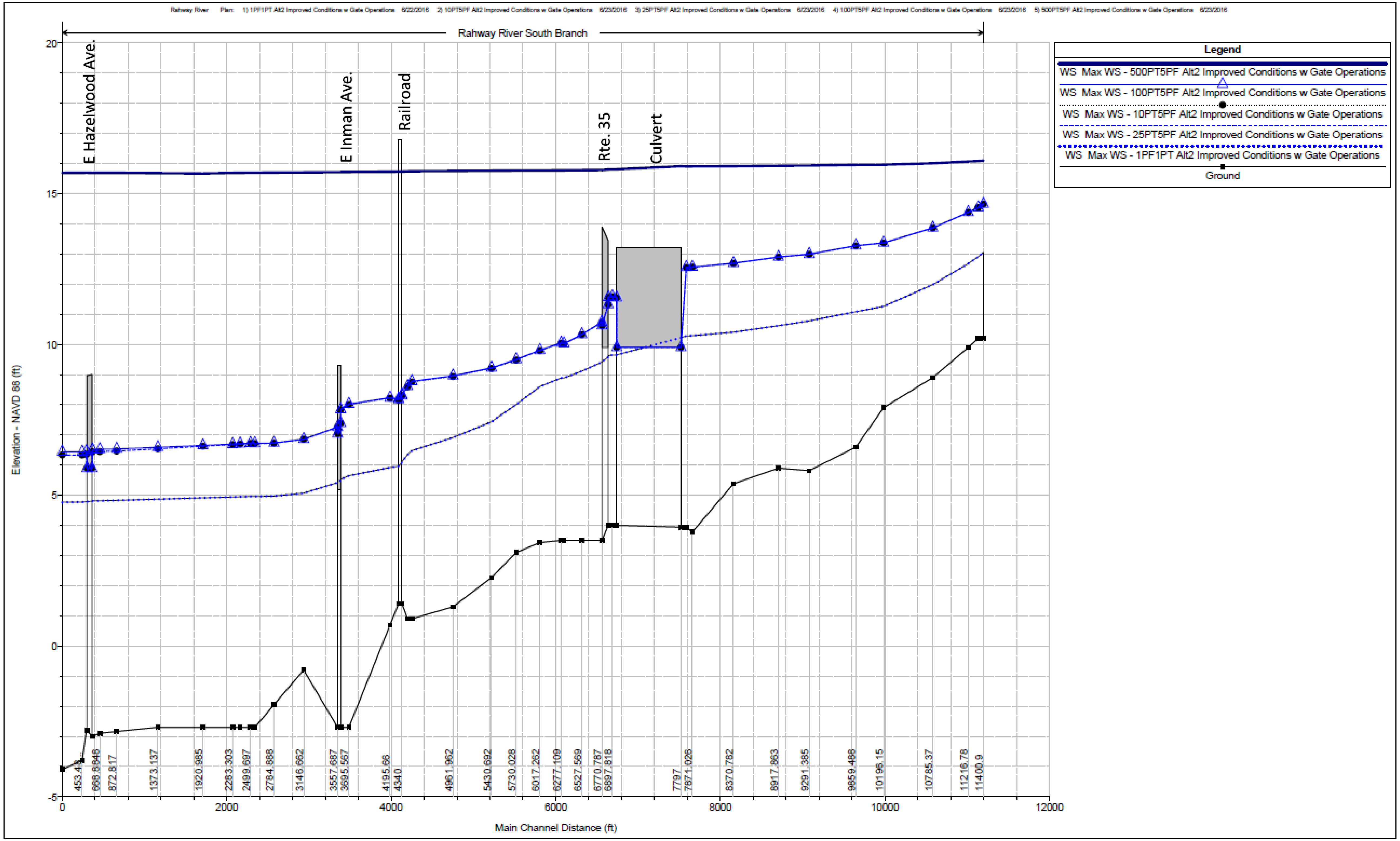


Figure 41: Alternative #2 computed water surface profile for South Branch

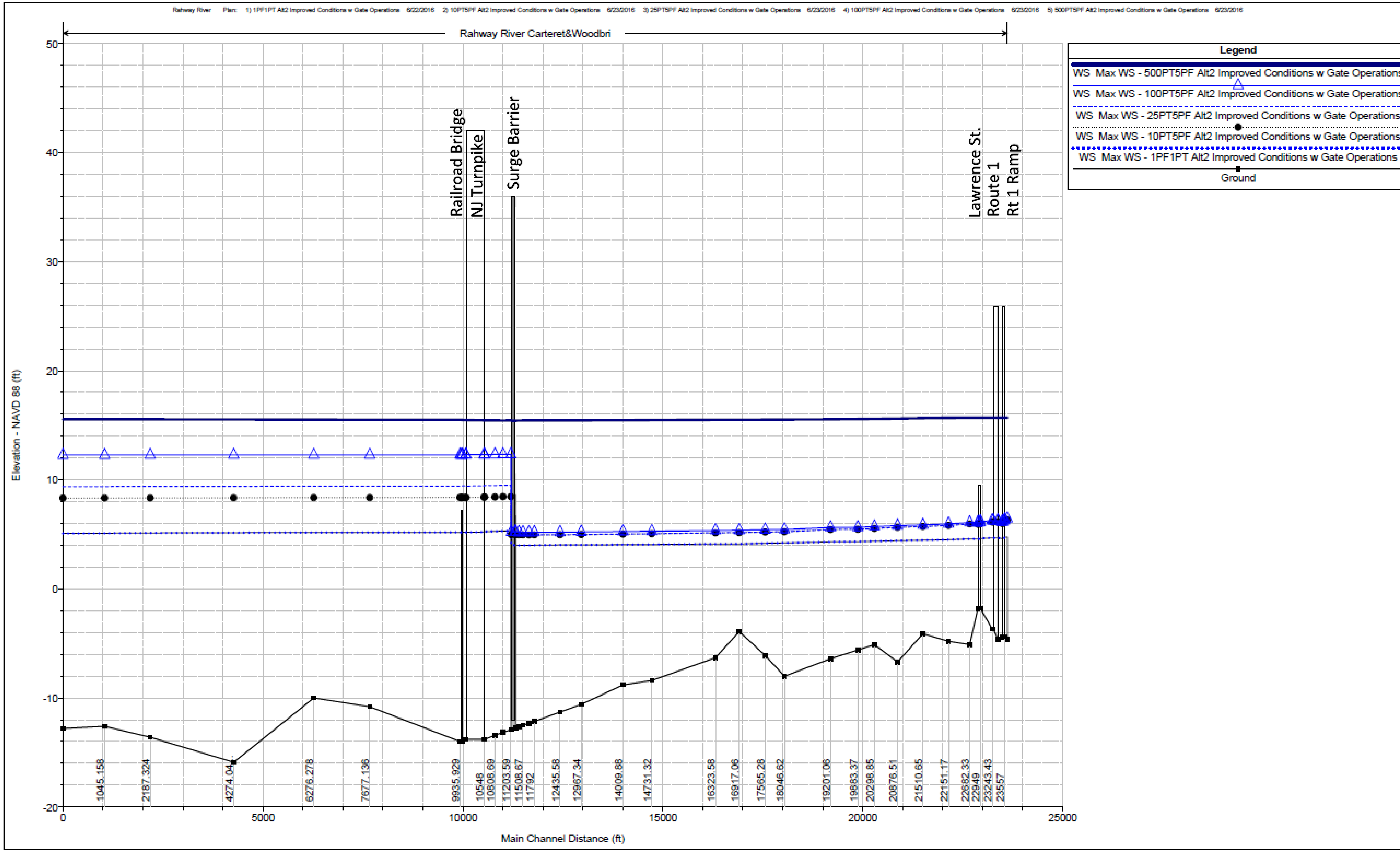


Figure 42: Alternative #2 computed water surface profile for Rahway River from South Branch Confluence to the Arthur Kill.

4.4 Non-Structural Alternatives

4.4.1 Description of Non-Structural Treatment Method and Selection

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), with the floodplain defined using joint probability HEC RAS results which utilized the NACCS stage frequency curve that incorporated 96 random tides at the mouth of the river as a boundary condition. 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 Coastal 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 flotation 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 are 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 is assumed to be feasible for structures having footprint of less than 3,000 sf. The elevation of the treated structures shall be at a minimum, one foot above the HEC RAS joint probability 0.01 AEP (100-year) bound by the North Atlantic Comprehensive Coastal Study water surface elevation with 96 random tides and intermediate sea level change superimposed on it.

- *Barriers (Ringwalls or Ring Levees)*. Although barriers, such as in the case of ringwalls, levees, or berms, are not considered a non-structural meassure, its need is determined during the non-structural analysis. Structures where is not feasible to provide flood reduction with the previous non-structural treatments due to the flood elevation are treated with this structural method.

4.4.2 Areas for Non-Structural Analysis

Floodplains corresponding to a flood frequency of 0.1 and 0.02 annual exceedance probability (10 and 50 year events) were evaluated considering future conditions flows and boundary conditions. The analysis is based on fluvial-coastal 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 7.

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 7.

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 7: 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**

*Strcutural measure considered within the non-strcutural plan.

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 8 and Table 9.

Table 8: 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 9: 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 43.

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 1 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 and Alternative #4a can be found in Table 10. Ringwall characteristics can be found in Table 11.

Table 10: Nonstructural Treatments for structures within Alternative #4.

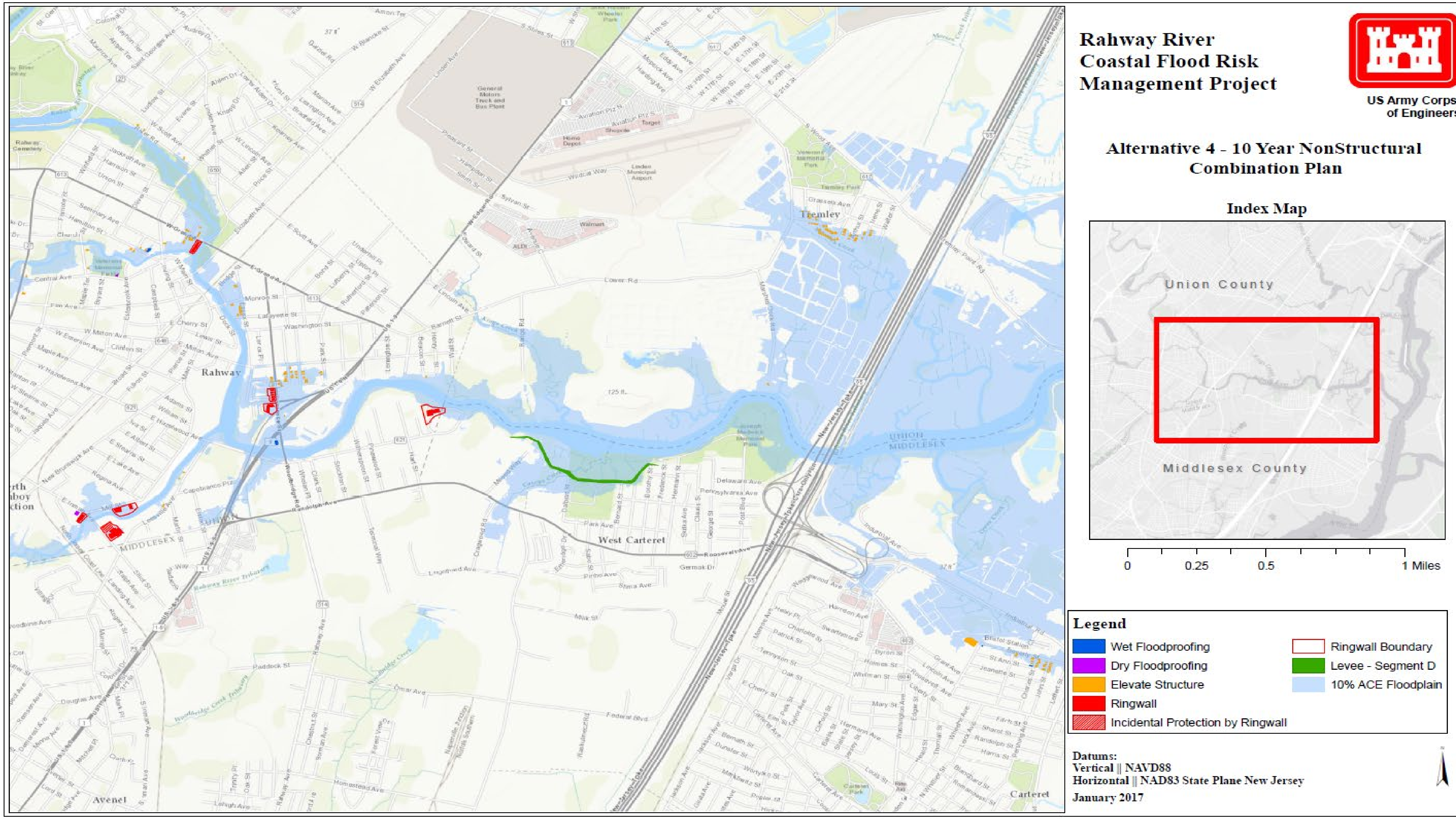
Nonstructural Flood Proofing Measure	**0.1 AEP ACE Combination Plan**		
	Residential	**Non-Residential**	**Total**
Dry Flood Proofing	0	2	**2**
Wet Flood Proofing	1	3	**4**
Elevation	123	4	**127**
Elevation - Demolish and Rebuild	1	2	**3**
Ringwall	6	7*	**13**
Total of Structures	**131**	**18**	**149**

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

Table 11: 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 coastal inundation only. Representation does not include joint-probability WSEs.

Figure 43: 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.

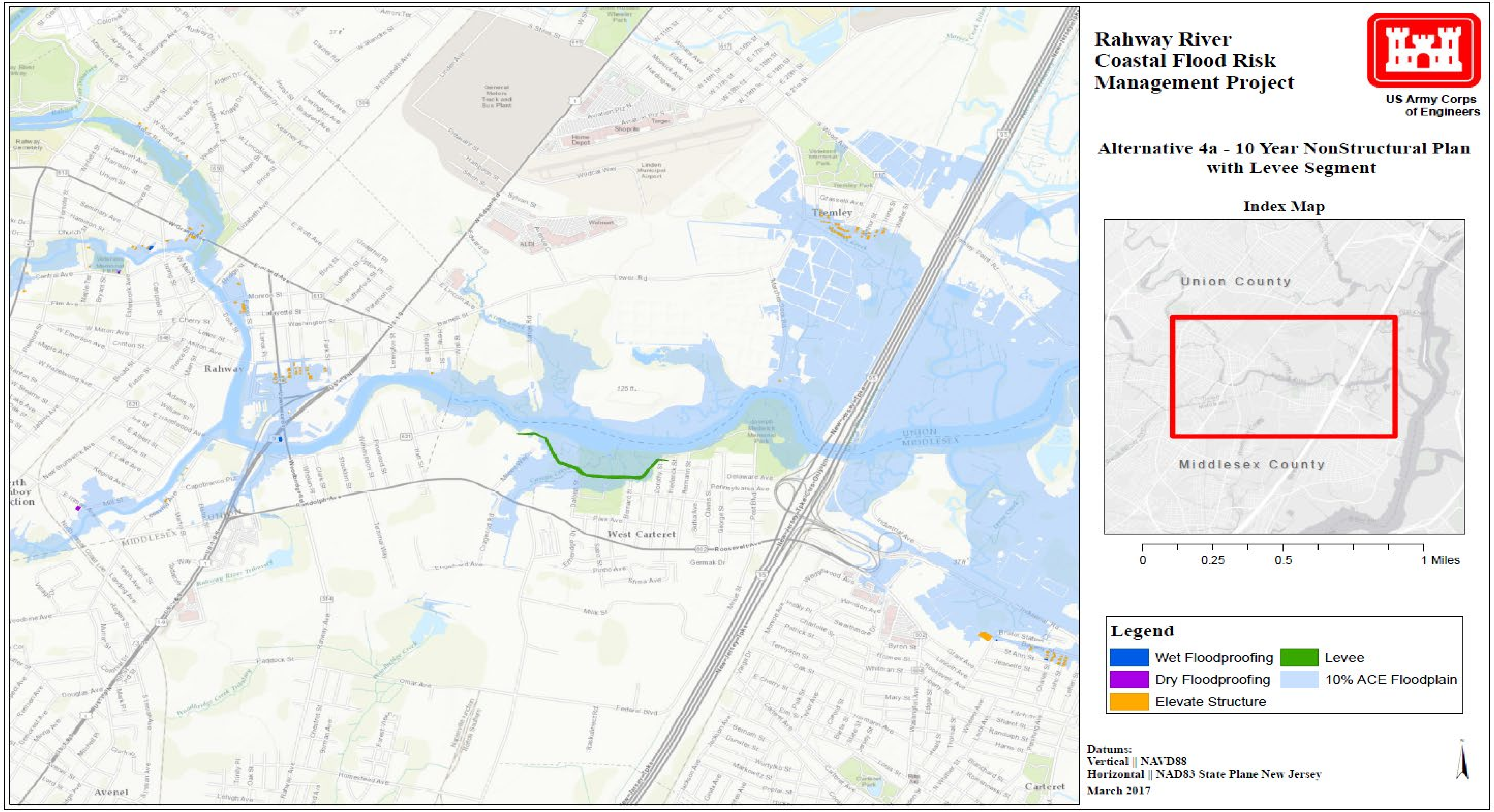
Alternative #4a thus determined nonstructural treatment for approximately 136 structures (125 residential, 11 non-residential) of the 577 structures (211 residential, 366 non-residential) contained in the 0.1 AEP (10-yr) floodplain. Nonstructural measures were designed to the future conditions 0.01 AEP (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 8.6 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 44. A summary of the treated structures in Alternative #4a can be found in Table 12.

Table 12: 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 a USACE NACCS coastal inundation only. Representation does not include joint-probability WSEs.

Figure 44: Alternative #4a Plan Overview.

5.0 OPTIMIZATION OF THE TSP

5.1.1 The Tentatively Selected Plan (TSP)

As a result of the plan formulation process, the conclusion of the analysis indicated that Alternative #4a was the plan with the highest net benefits. Therefore, alternative #4a was identified as the Tentatively Selected Plan (TSP).

5.1.2 Optimization of Alternative #4a

Alternative #4a featured 3 options for the sizing of the levees. There was a small plan, a medium plan, and a large plan. All these levee features consisted of varying lengths, varying elevations, and the medium and large plan also consisted of a road raise. Additionally, the large plan had sections of flood wall. Refer to Table 13 for a table of the different features and details and design height for each plan within this TSP.

The design elevations for the optimization plan small, medium and high, were selected based on; USACE existing project levee height at Rahway, intermediate and high SLR projection at 2073, respectively. For the small plan, the levee height were set at 12.6 feet NAVD'88, same elevation as the existing levees along the right bank of the Rahway River between Monroe Street and East Hazelwood Avenue. The medium plan was set to 14.2 ft NAVD'88 and the large plan at elevation 16.0 ft NAVD'88. See Figure 45 through Figure 47 for layout of the optimization the small, medium, and large levee/ floodwalls.

The plan will also contains of a non-structural aspect. Refer to Table 14 for details of the amount of structures and treatment per optimization plan.

The preliminary interior drainage analysis yielded in interior drainage structures approximately every 400 ft. The interior drainage structures will provide drainage to the protected levee areas and prevent negative flow. No pump station was deemed necessary due to the small size of drainage areas behind the levee segment and available ponding areas. For the selected plan, Casey Creek will be open during no storm conditions. The interior drainage structures are fairly typical for all cases and is an overall small feature of the selected plan. Each interior drainage structure have minimum variation in size of flap gates, pipes diameter (18" to 36"), length, and stormwater runoff catch basins. Detailed analysis will be performed during PED phase.

Table 13: Pre-optimized TSP Structural Options.

Details	Small Plan	Medium Plan	Large Plan
Levee Elevation	12.6 ft. NAVD'88	14.2 ft. NAVD'88	16 ft. NAVD'88
Levee Length	3520 ft.	4372 ft.	5471 ft.
Flood Wall Elevation	N/A	N/A	16 ft.
Flood Wall Length	N/A	N/A	420 ft.
Road Raise Elevation	N/A	14.2 ft.	17.2 ft.
Road Raise Length	N/A	1308 ft.	2619 ft.

Table 14: Non-structural treatments for optimization plans.

Total Structures	Residential	Non-Residential	Subtotal
	327	391	718
Small Plan			
Dry Floodproofing	0	0	0
Wet Floodproofing	10	3	13
Elevation	84	2	86
Buyout	6	0	6
Subtotal	100	5	105
Intermediate Plan			
Dry Floodproofing	0	0	0
Wet Floodproofing	7	2	9
Elevation	89	2	91
Buyout	10	0	10
Subtotal	106	4	110
Large Plan			
Dry Floodproofing	2	1	3
Wet Floodproofing	0	0	0
Elevation	69	4	73
Buyout	67	3	70
Subtotal	138	8	146

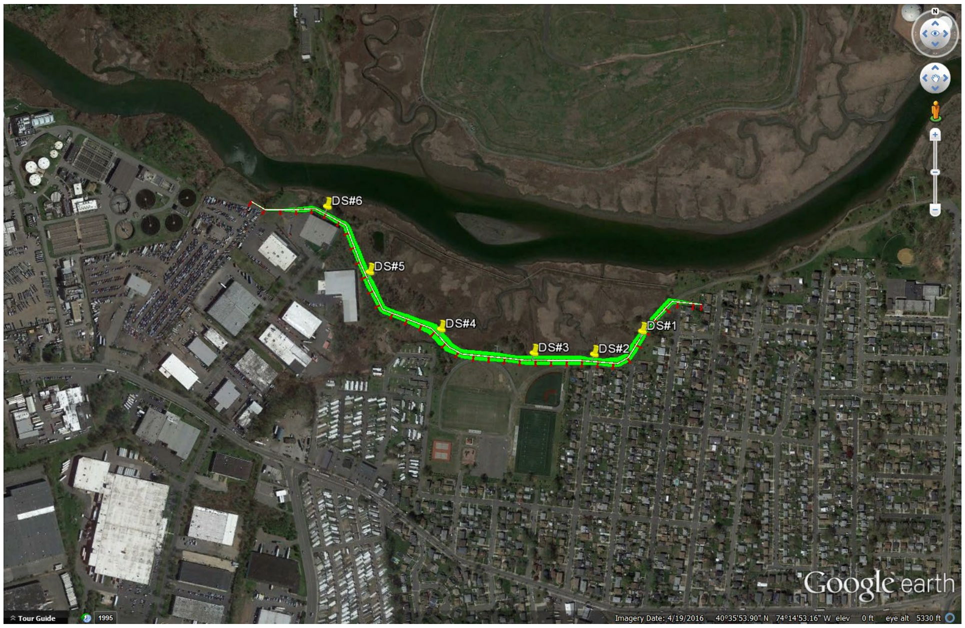


Figure 45: Small Plan Layout

Figure 46: Medium Plan Layout

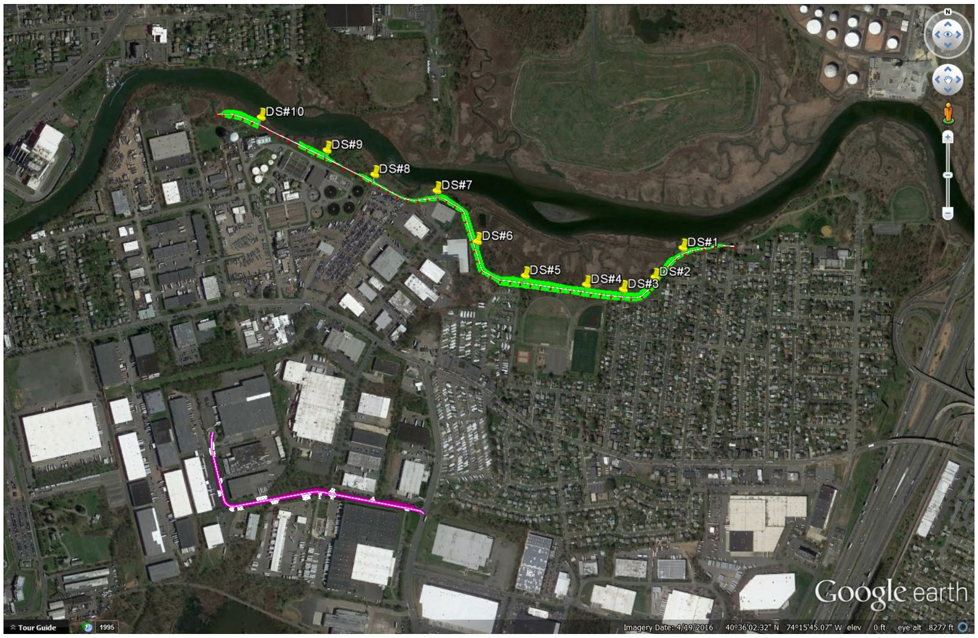


Figure 47: Large Plan Layout

6.0 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 48. (See Section 3.3.2 "Downstream Boundary Condition – Stage Hydrographs" regarding the rating curve chosen.). The uncertainty data from the NACCS was used in the economic analysis and is in compliance with the recommended procedure provided in the EM 1110-2-1619 (USACE 1996).

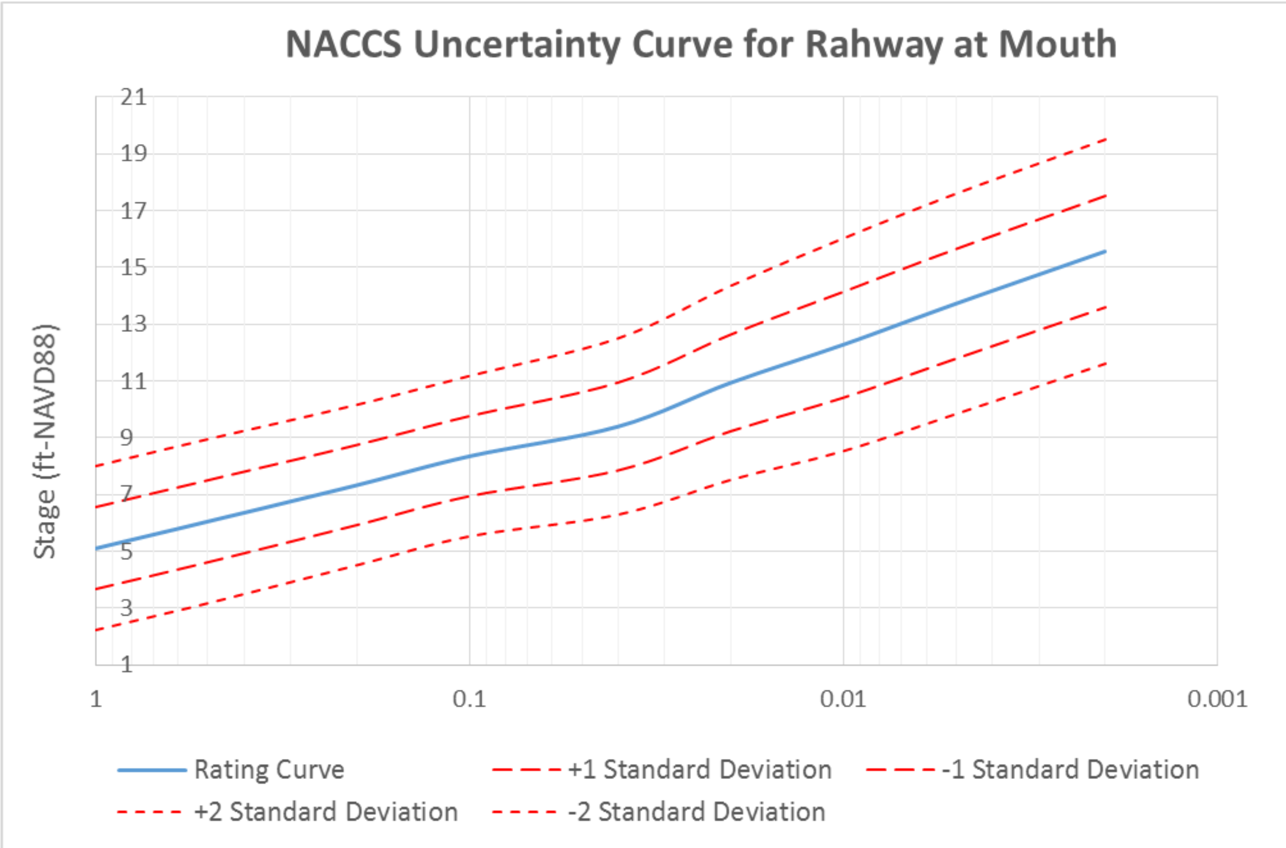


Figure 48: NACCS 1.0 and 2.0 times the standard deviation from expected SWL curve.

7.0 RECOMMENDED PLAN AND ADAPTABILITY

After careful analyses performed by the PDT, the optimized net benefit and benefit to cost ratio was achieved. The recommended plan or National Economic Development (NED) Plan is the medium size plan for alternative 4a. The medium plan for Alternative 4a features a levee, floodwalls and road raise to elevation 14.2 ft NAVD'88. It also includes the non-structural treatment 110 structures within the lower portion of the basin. The levee top width is 12 ft wide with side slopes of 1vertical to 3 horizontal (1V:3H), and a levee and floodwall length of 2520 ft and 1968 ft, respectively. The change to a section of floodwall, prevouisly a levee, is due to land acquisition limitation and to avoid utility relocation. The line of protection aligment goes thru areas with limited space adjacent to existing commercial buildings and major utilities. Refer to Table 15 and Table 16 to see general details of the NED plan. Figure 50 thru Figure 54 show the water surface profiles of the NED plan. Refer Figure 55 thru Figure 65 for NED plans view details.

For coastal SLR and riverine adaptability purposes, it is estimated that the recommended plan can be raised approximately 2.8 ft above current elevation. The conceptual design typical sections used for quantification and cost estimate do allow room for future modification and raising. This includes reducing the levee top width, from 12 ft to 10 ft, and side slopes from 1V:3H to 1V:2.5H, producing a higher levee. The floodwall footing and underpinning was designed in average for an approximately 3.0 ft higher floodwall. This additional height would result in levee and floodwall elevation above the intermediate SLR for the 100 yr. adaptation horizon. See Figure 49 for representation of SLR scenarios and adaptavility summary. See Table 17 for the existing main floor elevatiosn of the structures to be elevated, their proposed main floor elevation, and FEMA Base Flood Elevation, as a comparison.

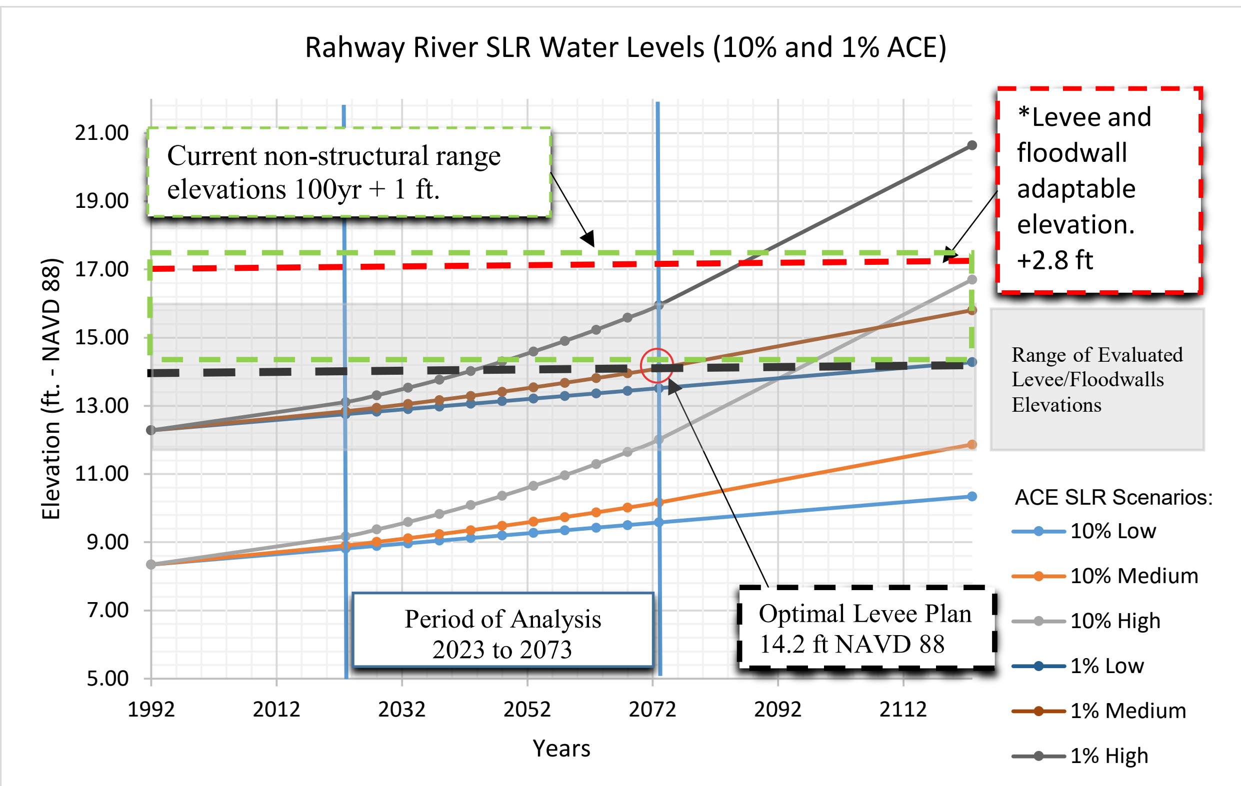
Table 15: Non-structural component of the NED plan.

Total Structures	Residential	Non-Residential	Subtotal
NED Plan			
Dry Floodproofing	0	0	0
Wet Floodproofing	7	2	9
Elevation (Range: 2.0 to 7.5 ft)	89	2	91
Buyout	10	0	10
Subtotal	106	4	110

Table 16: Structural components of the NED plan.

Details	Medium Plan
Levee Elevation	14.2 ft. NAVD'88
Average Levee/Floodwall Height	10.2 ft.
Levee Length	2520 ft.
Levee Top Width	12 ft.
Floodwall Elevation	14.2 ft. NAVD'88
Floodwall Length	1968 ft.
Englehard Ave. Road Raise to Elevation	14.2 ft. NAVD'88
Englehard Ave. Road Raise Length	1350 ft.
*Number of Interior Drainage Structures (and flap gates)	8

*Flap gae on Caseys Creek to remain open during non-storm conditions.



*Based on average height and typical sections.

Figure 49: Graphic presentation of SLR scenarios, optimization and coastal adaptability of recommended plan from 1992 to 2123.

Table 17: Comparative Main Floor Elevations for the Structures Proposed to be Elevated.

Structure ID	Residential or Non-Residential	Ground Elevation (ft NAVD88)	Main Floor Elevation (ft NAVD88)	Water Surface Elevation 0.01 AEP (100-yr) Future in ft. NAVD88	Protection Level (WSEL +1) AND Proposed Main Floor Elevation in ft. NAVD88	Category of Treatment	FEMA Base Flood Elevation in ft. NAVD88
1330	Residential	8.4	11.9	15.6	16.6	Elevate	9.9
1332	Residential	8.1	11.1	15.6	16.6	Elevate	9.9
1333	Residential	8.6	11.1	15.6	16.6	Elevate	9.9
1334	Residential	8.7	11.7	15.6	16.6	Elevate	9.9
1335	Residential	9.0	10.5	15.6	16.6	Elevate	9.9
1369	Residential	9.8	13.3	15.6	16.6	Elevate	9.1
1374	Residential	7.9	10.9	15.6	16.6	Elevate	9.1
1375	Residential	8.1	11.1	15.6	16.6	Elevate	9.1
1377	Residential	9.6	12.1	15.6	16.6	Elevate	9.1
1501	Residential	7.8	8.3	15.6	16.6	Elevate	9.1
1502	Residential	8.6	12.6	15.6	16.6	Elevate	9.1
1522	Residential	10.0	12.5	15.2	16.2	Elevate	9.1
1524	Residential	9.0	13.0	15.2	16.2	Elevate	9.1
1525	Residential	9.0	13.0	15.2	16.2	Elevate	9.1
1529	Residential	8.8	11.3	15.6	16.6	Elevate	9.1
1538	Residential	8.8	11.8	15.6	16.6	Elevate	9.1
1539	Residential	9.1	13.1	15.6	16.6	Elevate	9.1
1542	Residential	9.6	12.6	15.6	16.6	Elevate	9.1
1543	Residential	9.4	12.4	15.6	16.6	Elevate	9.1
1544	Residential	9.1	11.6	15.6	16.6	Elevate	9.1
1545	Residential	8.9	10.9	15.6	16.6	Elevate	9.1
1548	Residential	8.7	10.7	15.6	16.6	Elevate	9.1
1550	Residential	9.5	12.5	15.6	16.6	Elevate	9.1
1551	Residential	9.8	12.3	15.6	16.6	Elevate	9.1
1552	Residential	9.4	11.4	15.6	16.6	Elevate	9.1
1554	Residential	9.0	12.0	15.6	16.6	Elevate	9.1
1566	Residential	9.5	11.5	15.6	16.6	Elevate	9.1
1567	Residential	9.6	11.6	15.6	16.6	Elevate	9.1
1570	Residential	9.1	10.1	15.6	16.6	Elevate	9.1
3017	Residential	8.2	10.9	15.7	16.7	Elevate	9.1
3018	Residential	8.5	11.8	15.7	16.7	Elevate	9.9
3206	Residential	8.7	11.2	14.1	15.1	Elevate	9.4
3218	Residential	9.1	11.6	14.1	15.1	Elevate	9.4
3220	Residential	8.2	11.2	14.1	15.1	Elevate	9
3224	Residential	7.3	13.8	14.1	15.1	Elevate	9
3225	Residential	8.4	12.4	14.1	15.1	Elevate	16
3240	Residential	7.4	11.4	14.1	15.1	Elevate	9
3241	Residential	7.6	10.1	14.1	15.1	Elevate	9
3243	Residential	7.2	10.2	14.1	15.1	Elevate	9
3244	Residential	7.4	7.9	14.1	15.1	Elevate	9
3245	Residential	7.7	8.2	14.1	15.1	Elevate	9
3246	Residential	7.7	8.2	14.1	15.1	Elevate	9
3248	Residential	7.0	7.0	14.1	15.1	Elevate	9
3249	Residential	7.0	12.0	14.1	15.1	Elevate	9
3254	Residential	8.2	8.2	14.1	15.1	Elevate	9
3255	Residential	8.7	8.7	14.1	15.1	Elevate	9
3258	Residential	7.9	10.9	14.1	15.1	Elevate	9
3259	Residential	7.2	7.2	14.1	15.1	Elevate	9

Structure ID	Residential or Non-Residential	Ground Elevation (ft NAVD88)	Main Floor Elevation (ft NAVD88)	Water Surface Elevation 0.01 AEP (100-yr) Future in ft. NAVD88	Protection Level (WSEL +1) AND Proposed Main Floor Elevation in ft. NAVD88	Category of Treatment	FEMA Base Flood Elevation in ft. NAVD88
3258	Residential	7.9	10.9	14.1	15.1	Elevate	9
3259	Residential	7.2	7.2	14.1	15.1	Elevate	9
3265	Residential	7.7	11.7	14.1	15.1	Elevate	9
3266	Residential	8.7	12.7	14.1	15.1	Elevate	9
3276	Residential	8.0	12.5	14.1	15.1	Elevate	9
3277	Residential	6.1	10.6	14.1	15.1	Elevate	9
3279	Residential	7.5	12.0	14.1	15.1	Elevate	9
3280	Residential	7.9	12.4	14.1	15.1	Elevate	9
3291	Residential	8.5	12.5	14.1	15.1	Elevate	9
3293	Residential	6.5	10.0	14.1	15.1	Elevate	9
3300	Residential	7.6	11.6	14.1	15.1	Elevate	9
3301	Residential	8.8	13.3	14.1	15.1	Elevate	9
3829	Residential	8.3	10.3	14.1	15.1	Elevate	7
3830	Residential	7.8	9.3	14.1	15.1	Elevate	7
3831	Residential	8.4	10.9	14.1	15.1	Elevate	7
3832	Residential	6.9	10.9	14.1	15.1	Elevate	7
3833	Residential	7.0	11.5	14.1	15.1	Elevate	7
3834	Residential	6.8	11.3	14.1	15.1	Elevate	7
3835	Residential	6.7	11.2	14.1	15.1	Elevate	7
3836	Residential	7.3	10.3	14.1	15.1	Elevate	7
3837	Residential	7.9	11.9	14.1	15.1	Elevate	7
3838	Residential	7.8	8.8	14.1	15.1	Elevate	7
3839	Residential	8.0	12.5	14.1	15.1	Elevate	7
3851	Residential	8.8	12.8	14.1	15.1	Elevate	7
3852	Residential	8.2	13.7	14.1	15.1	Elevate	7
3853	Residential	7.6	12.1	14.1	15.1	Elevate	7
3854	Residential	7.9	13.9	14.1	15.1	Elevate	7
3855	Residential	6.9	12.9	14.1	15.1	Elevate	7
3877	Residential	7.7	11.7	14.1	15.1	Elevate	7
3878	Residential	8.3	10.8	14.1	15.1	Elevate	7
4583	Residential	10.8	12.8	16.8	17.8	Elevate	18
4616	Residential	13.0	15.0	17.4	18.4	Elevate	18
4740	Residential	8.1	11.6	15.6	16.6	Elevate	10
4921	Residential	14.0	14.0	18.1	19.1	Elevate	18.3
5028	Residential	11.4	12.7	16.1	17.1	Elevate	14.7
5029	Residential	10.7	14.0	17.4	18.4	Elevate	15.5
5051	Non-Residential	10.8	12.1	16.1	17.1	Elevate	14.3
5086	Residential	8.4	11.7	16.1	17.1	Elevate	14.8
5095	Residential	8.8	11.5	16.1	17.1	Elevate	14.4
5097	Residential	8.5	11.8	16.1	17.1	Elevate	14.4
5099	Residential	8.4	15.1	16.0	17.0	Elevate	14.4
5434	Non-Residential	9.8	12.3	15.6	16.6	Elevate	9.3
5453	Residential	9.9	11.9	15.6	16.6	Elevate	9.3
5454	Residential	10.4	11.9	15.6	16.6	Elevate	9.3
5455	Residential	9.2	11.2	15.6	16.6	Elevate	9.3

Structure ID	Residential or Non-Residential	Ground Elevation (ft NAVD88)	Main Floor Elevation (ft NAVD88)	Water Surface Elevation 0.01 AEP (100-yr) Future in ft. NAVD88	Protection Level (WSEL +1) AND Proposed Main Floor Elevation in ft. NAVD88	Category of Treatment	FEMA Base Flood Elevation in ft. NAVD88
3265	Residential	7.7	11.7	14.1	15.1	Elevate	9
3266	Residential	8.7	12.7	14.1	15.1	Elevate	9
3276	Residential	8.0	12.5	14.1	15.1	Elevate	9
3277	Residential	6.1	10.6	14.1	15.1	Elevate	9
3279	Residential	7.5	12.0	14.1	15.1	Elevate	9
3280	Residential	7.9	12.4	14.1	15.1	Elevate	9
3291	Residential	8.5	12.5	14.1	15.1	Elevate	9
3293	Residential	6.5	10.0	14.1	15.1	Elevate	9
3300	Residential	7.6	11.6	14.1	15.1	Elevate	9
3301	Residential	8.8	13.3	14.1	15.1	Elevate	9
3829	Residential	8.3	10.3	14.1	15.1	Elevate	7
3830	Residential	7.8	9.3	14.1	15.1	Elevate	7
3831	Residential	8.4	10.9	14.1	15.1	Elevate	7
3832	Residential	6.9	10.9	14.1	15.1	Elevate	7
3833	Residential	7.0	11.5	14.1	15.1	Elevate	7
3834	Residential	6.8	11.3	14.1	15.1	Elevate	7
3835	Residential	6.7	11.2	14.1	15.1	Elevate	7
3836	Residential	7.3	10.3	14.1	15.1	Elevate	7
3837	Residential	7.9	11.9	14.1	15.1	Elevate	7
3838	Residential	7.8	8.8	14.1	15.1	Elevate	7
3839	Residential	8.0	12.5	14.1	15.1	Elevate	7
3851	Residential	8.8	12.8	14.1	15.1	Elevate	7
3852	Residential	8.2	13.7	14.1	15.1	Elevate	7
3853	Residential	7.6	12.1	14.1	15.1	Elevate	7
3854	Residential	7.9	13.9	14.1	15.1	Elevate	7
3855	Residential	6.9	12.9	14.1	15.1	Elevate	7
3877	Residential	7.7	11.7	14.1	15.1	Elevate	7
3878	Residential	8.3	10.8	14.1	15.1	Elevate	7
4583	Residential	10.8	12.8	16.8	17.8	Elevate	18
4616	Residential	13.0	15.0	17.4	18.4	Elevate	18
4740	Residential	8.1	11.6	15.6	16.6	Elevate	10
4921	Residential	14.0	14.0	18.1	19.1	Elevate	18.3
5028	Residential	11.4	12.7	16.1	17.1	Elevate	14.7
5029	Residential	10.7	14.0	17.4	18.4	Elevate	15.5
5051	Non-Residential	10.8	12.1	16.1	17.1	Elevate	14.3
5086	Residential	8.4	11.7	16.1	17.1	Elevate	14.8
5095	Residential	8.8	11.5	16.1	17.1	Elevate	14.4
5097	Residential	8.5	11.8	16.1	17.1	Elevate	14.4
5099	Residential	8.4	15.1	16.0	17.0	Elevate	14.4
5434	Non-Residential	9.8	12.3	15.6	16.6	Elevate	9.3
5453	Residential	9.9	11.9	15.6	16.6	Elevate	9.3
5454	Residential	10.4	11.9	15.6	16.6	Elevate	9.3
5455	Residential	9.2	11.2	15.6	16.6	Elevate	9.3

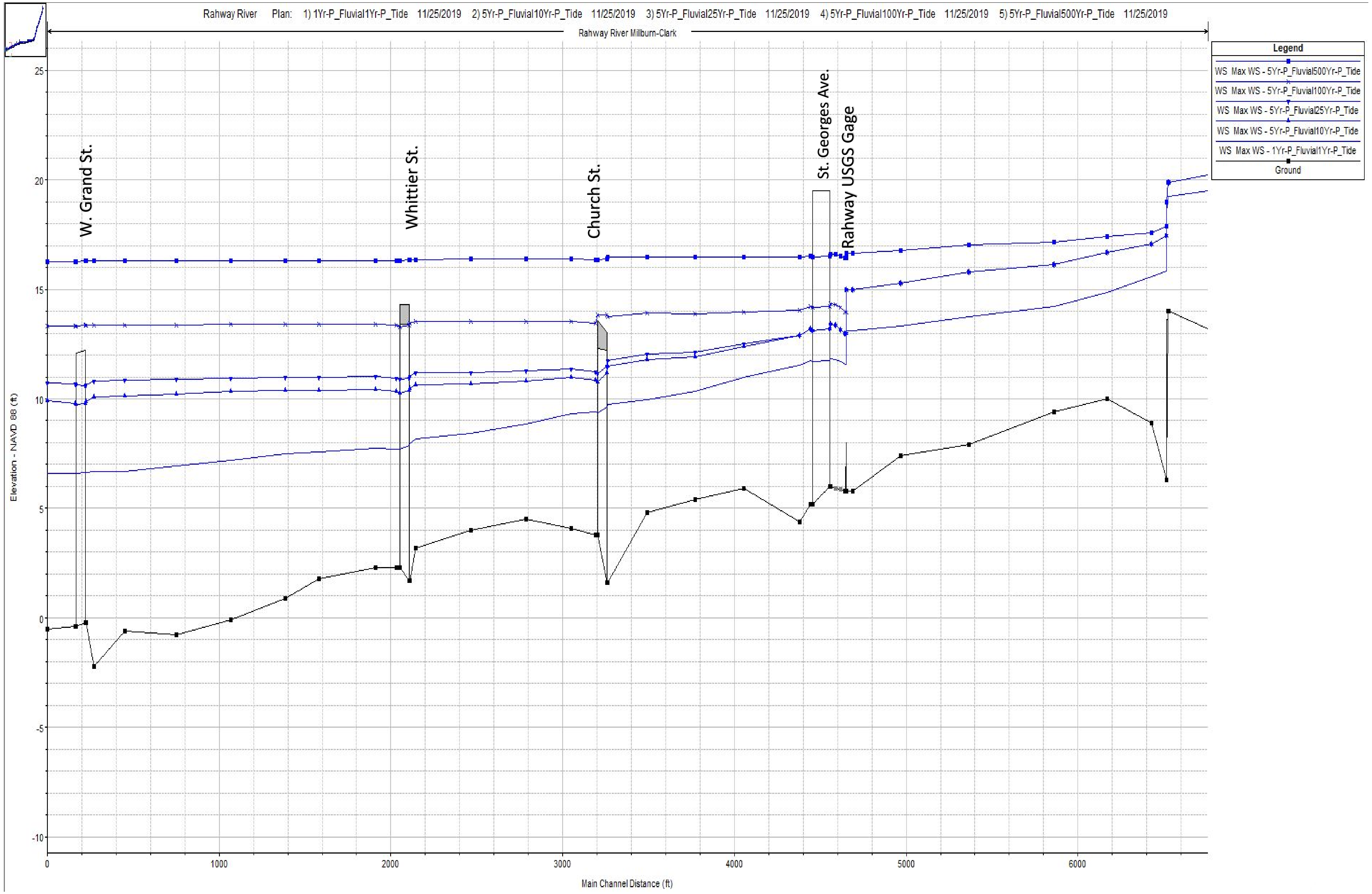


Figure 50: NED plan computed water surface profile from Rahway Water Supply to Robinson's Branch Confluence.

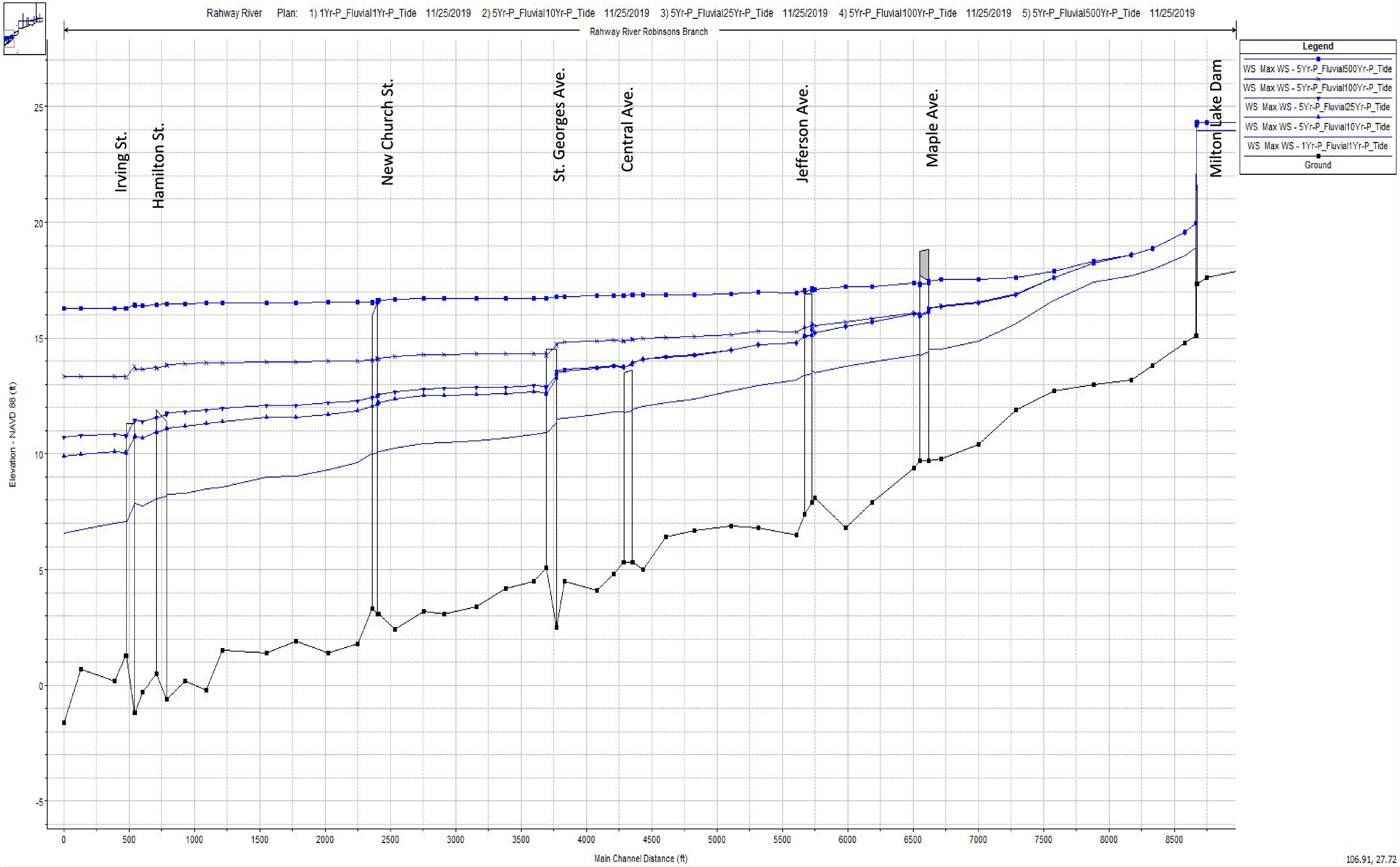


Figure 51: NED plan computed water surface profile for Robinson's Branch from Milton Lake Dam to the confluence.

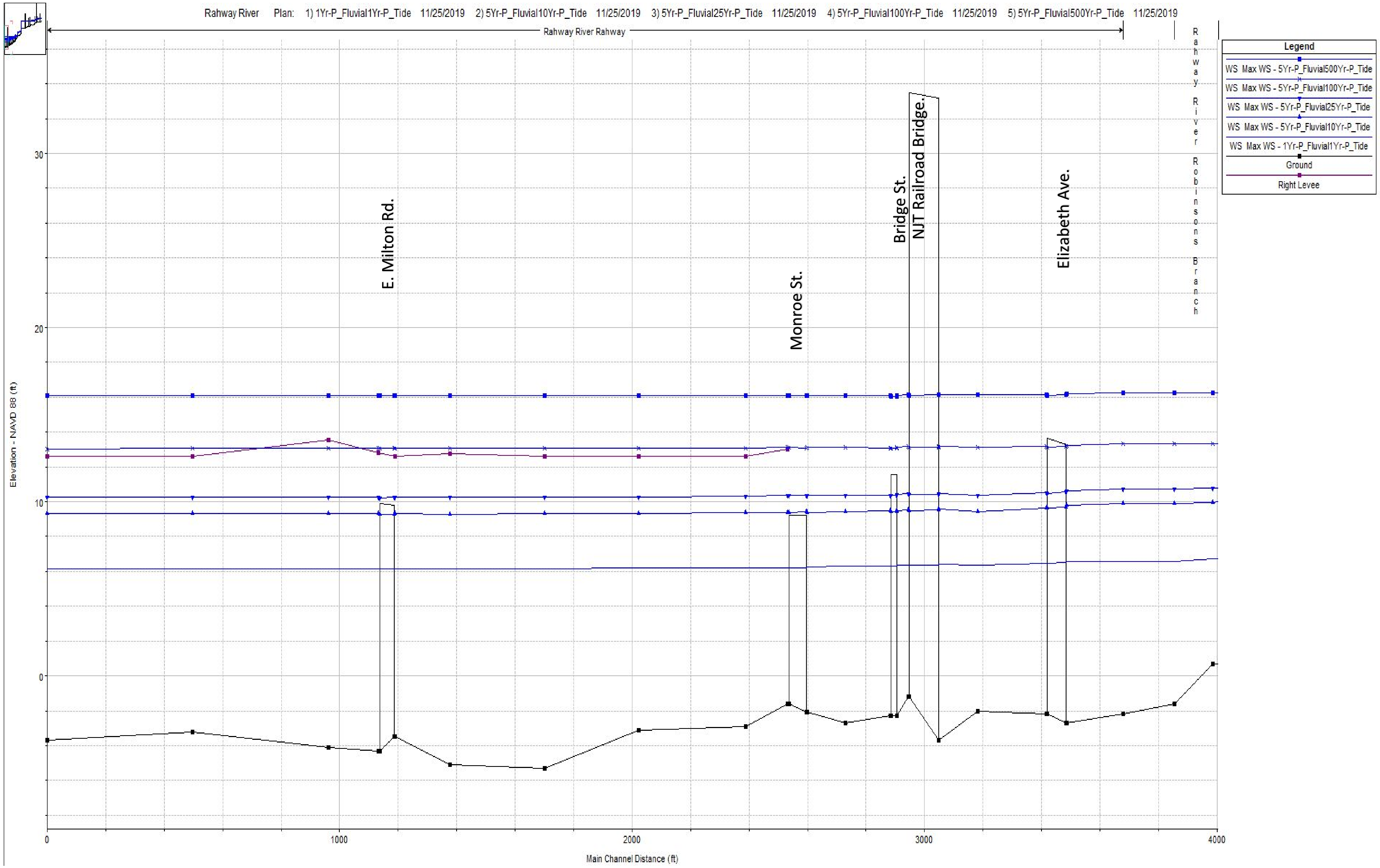


Figure 52: NED plan computed water surface profile for Rahway River from Robinson's Branch Confluence to South Branch Confluence.

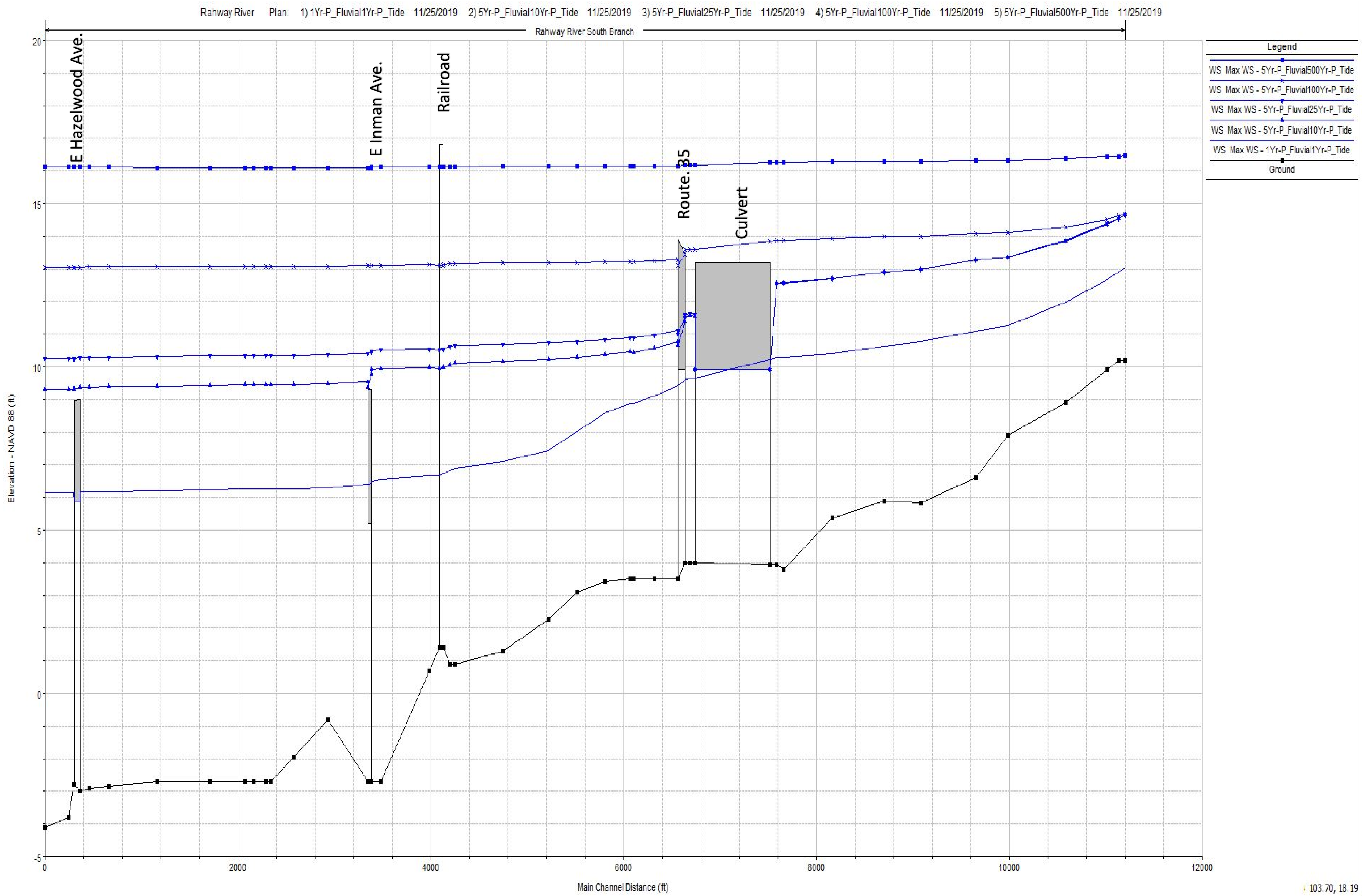


Figure 53: NED plan computed water surface profile for South Branch

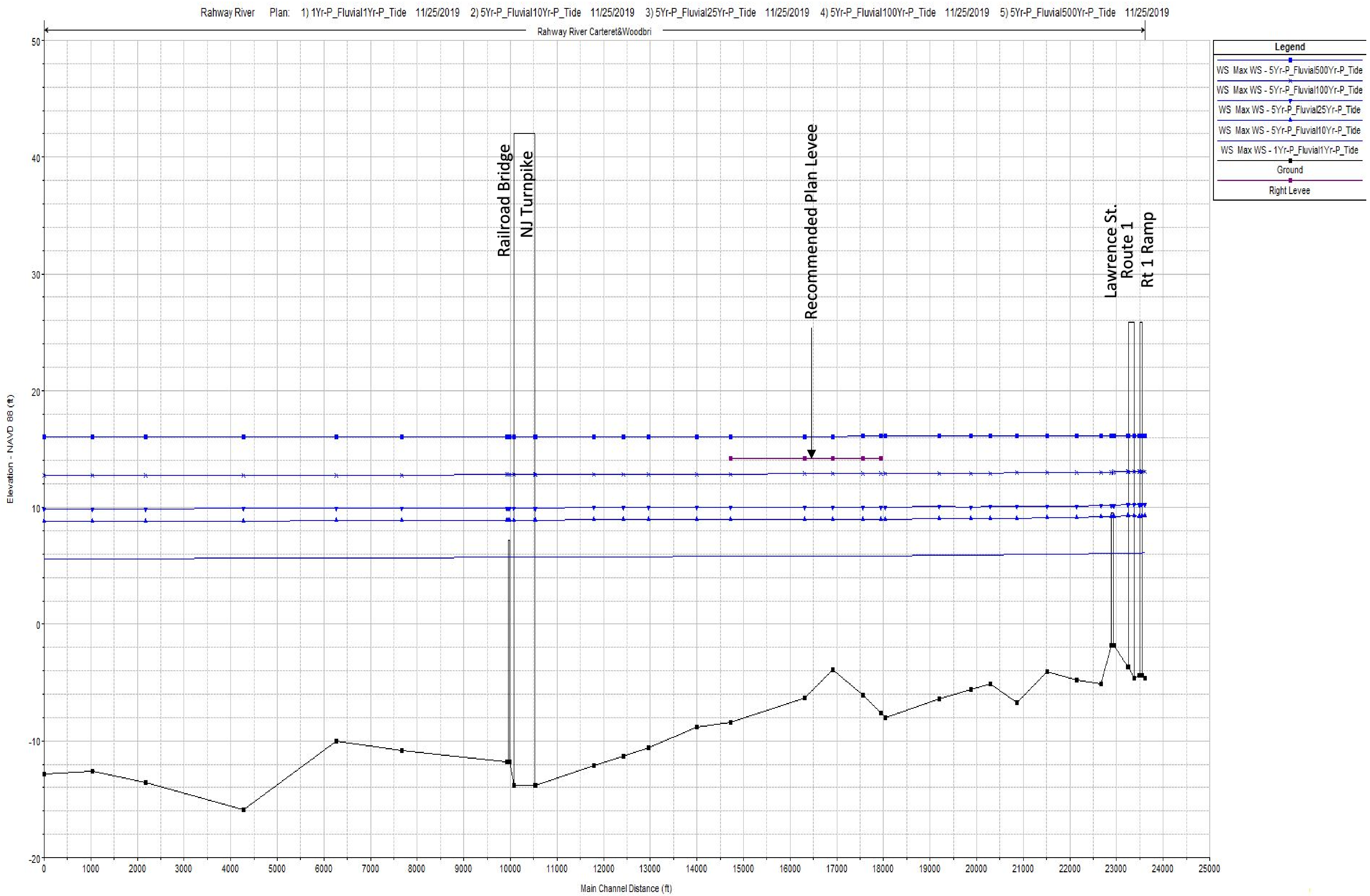


Figure 54: NED plan computed water surface profile for Rahway River from South Branch Confluence to the Arthur Kill.

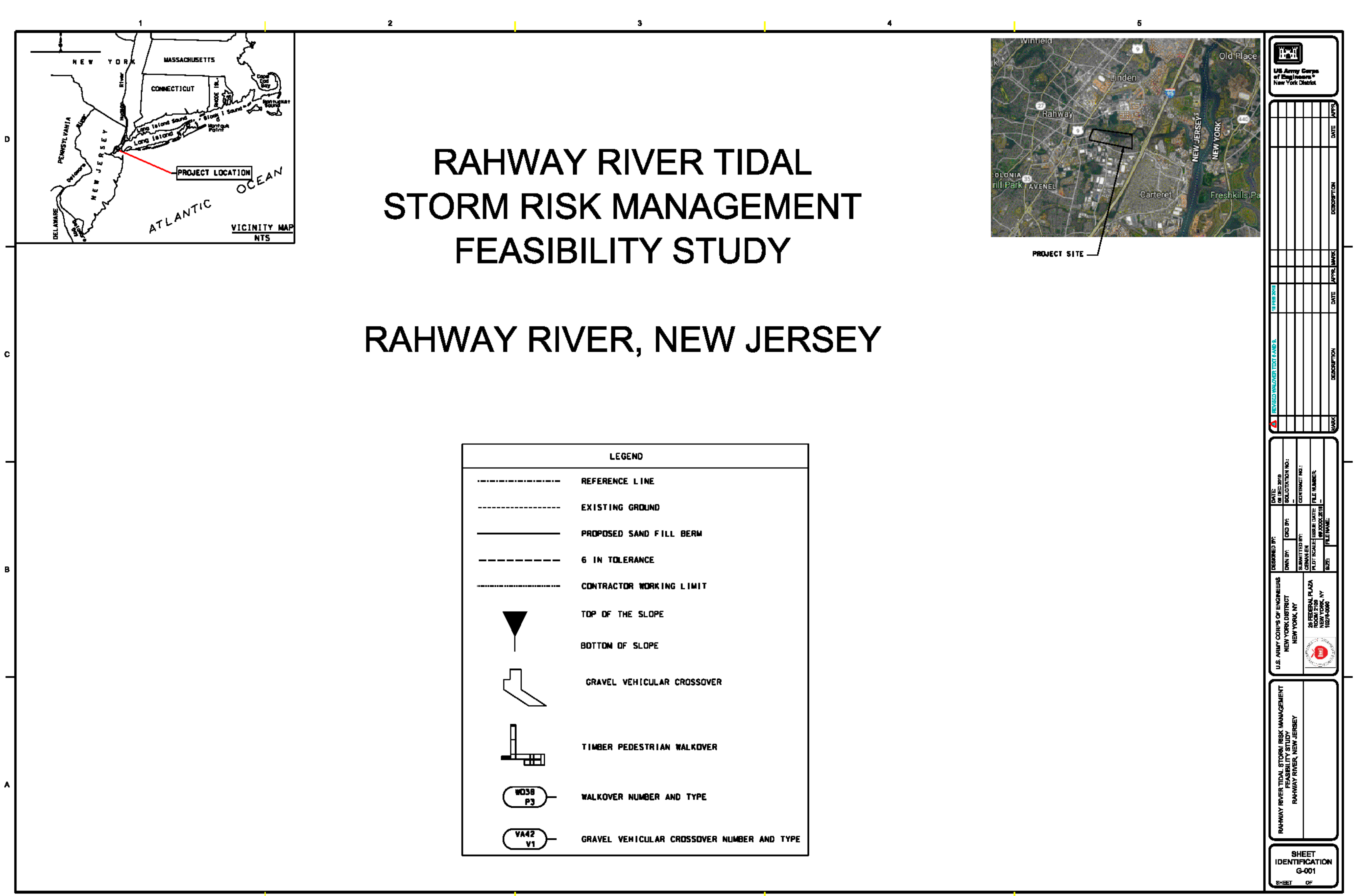


Figure 55: NED Plan Layout

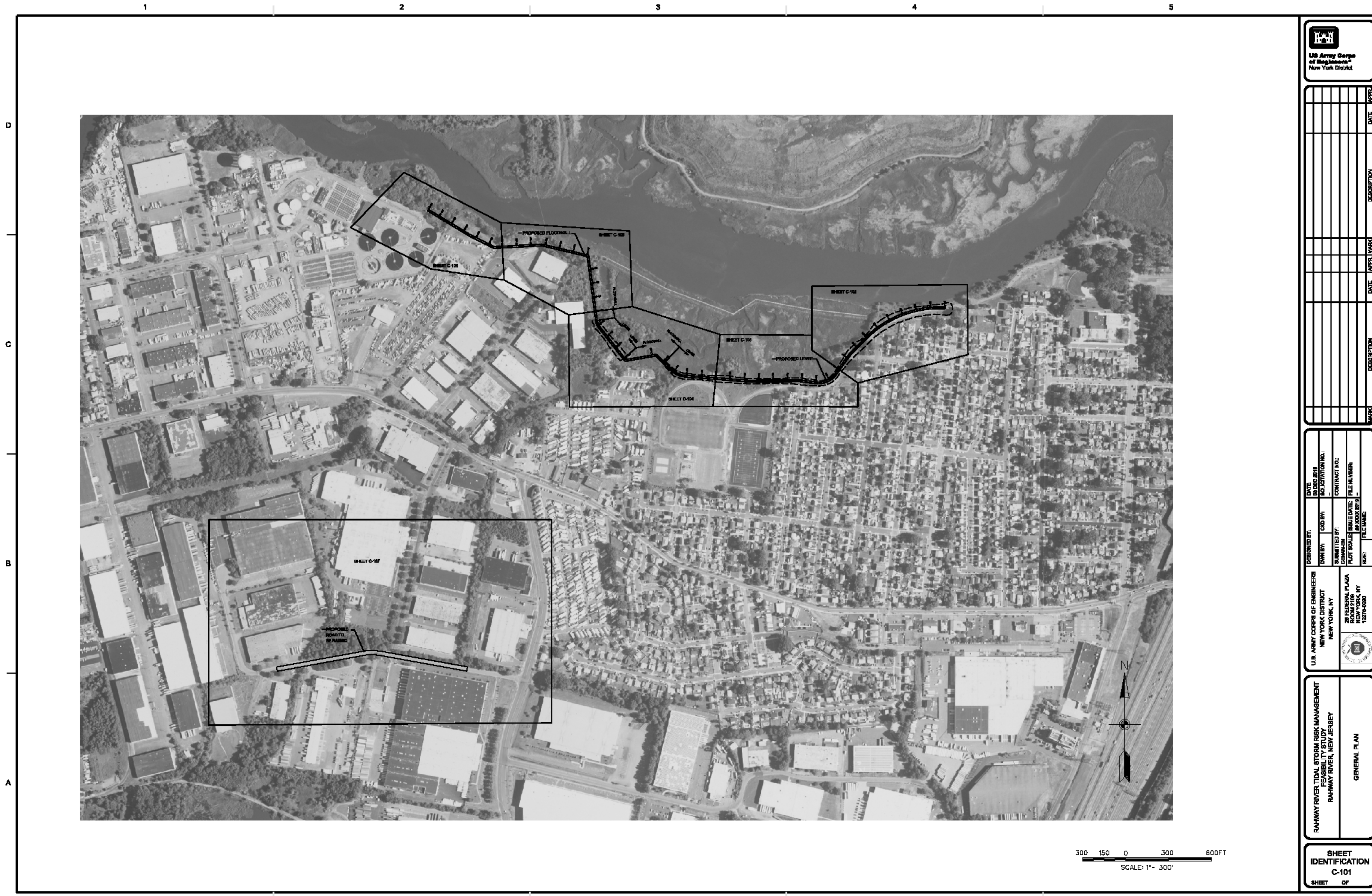


Figure 56: NED Plan Layout

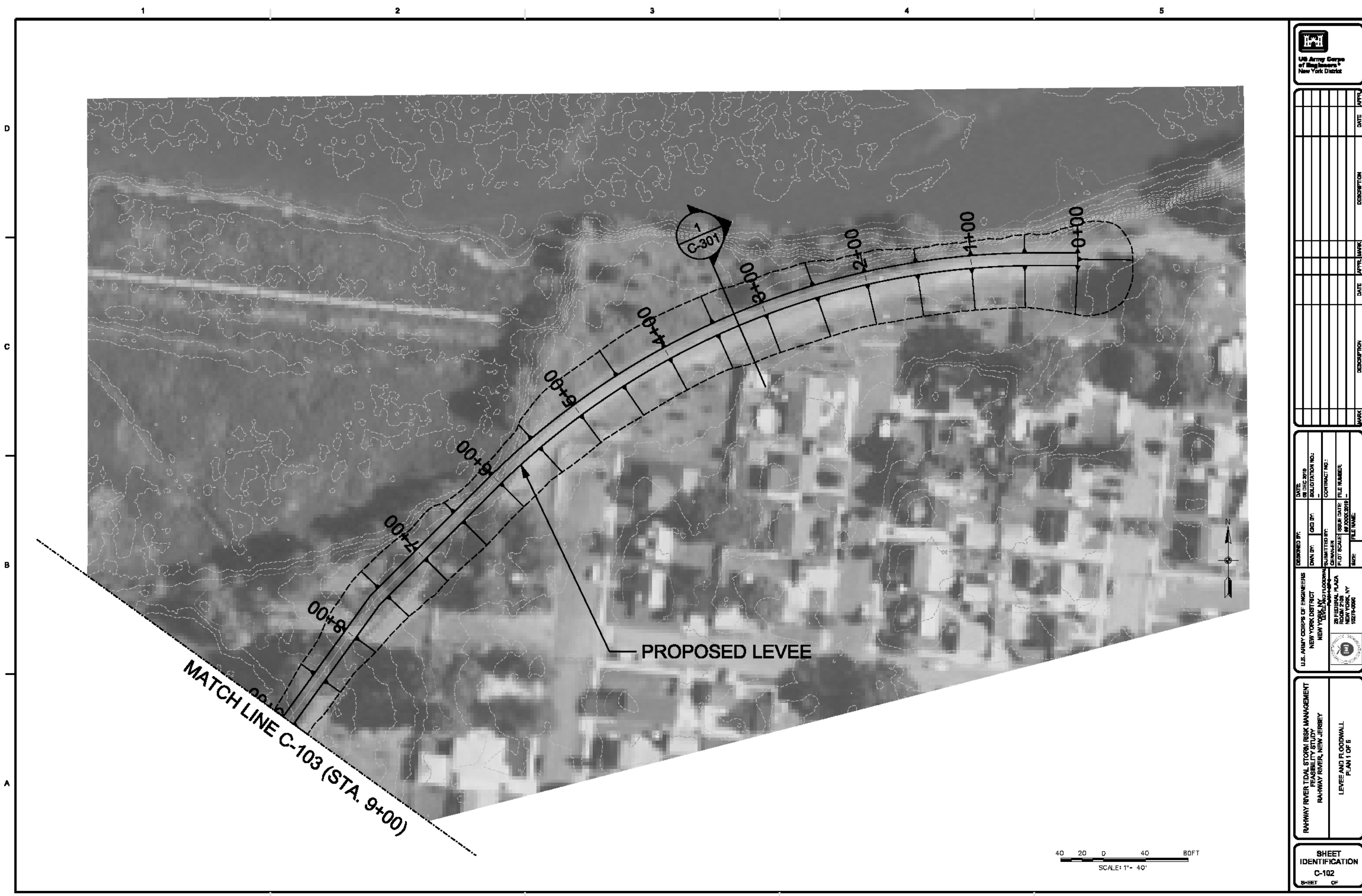


Figure 57: NED Plan Layout

Figure 58: NED Plan Layout

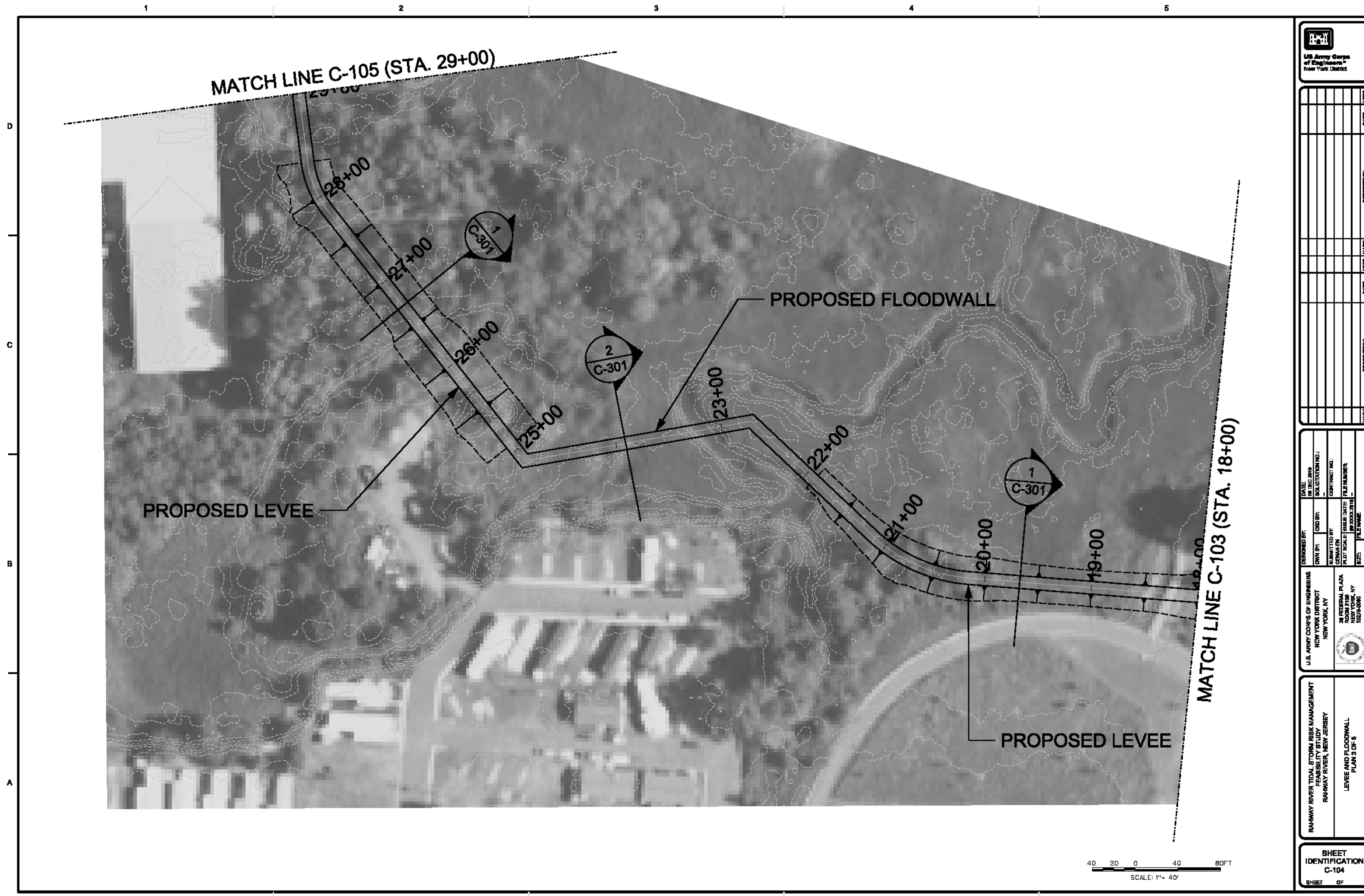


Figure 59: NED Plan Layout

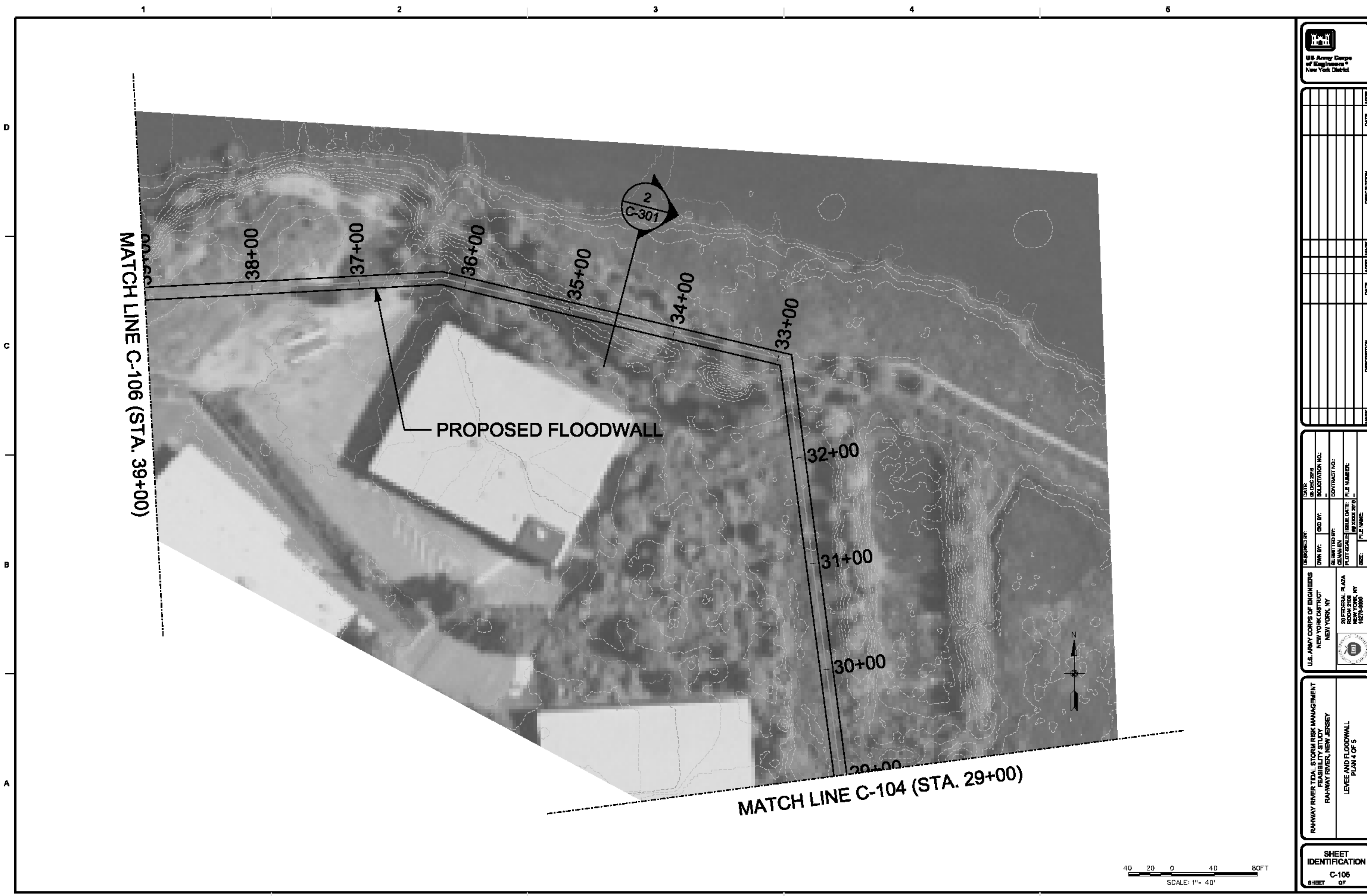


Figure 60: NED Plan Layout

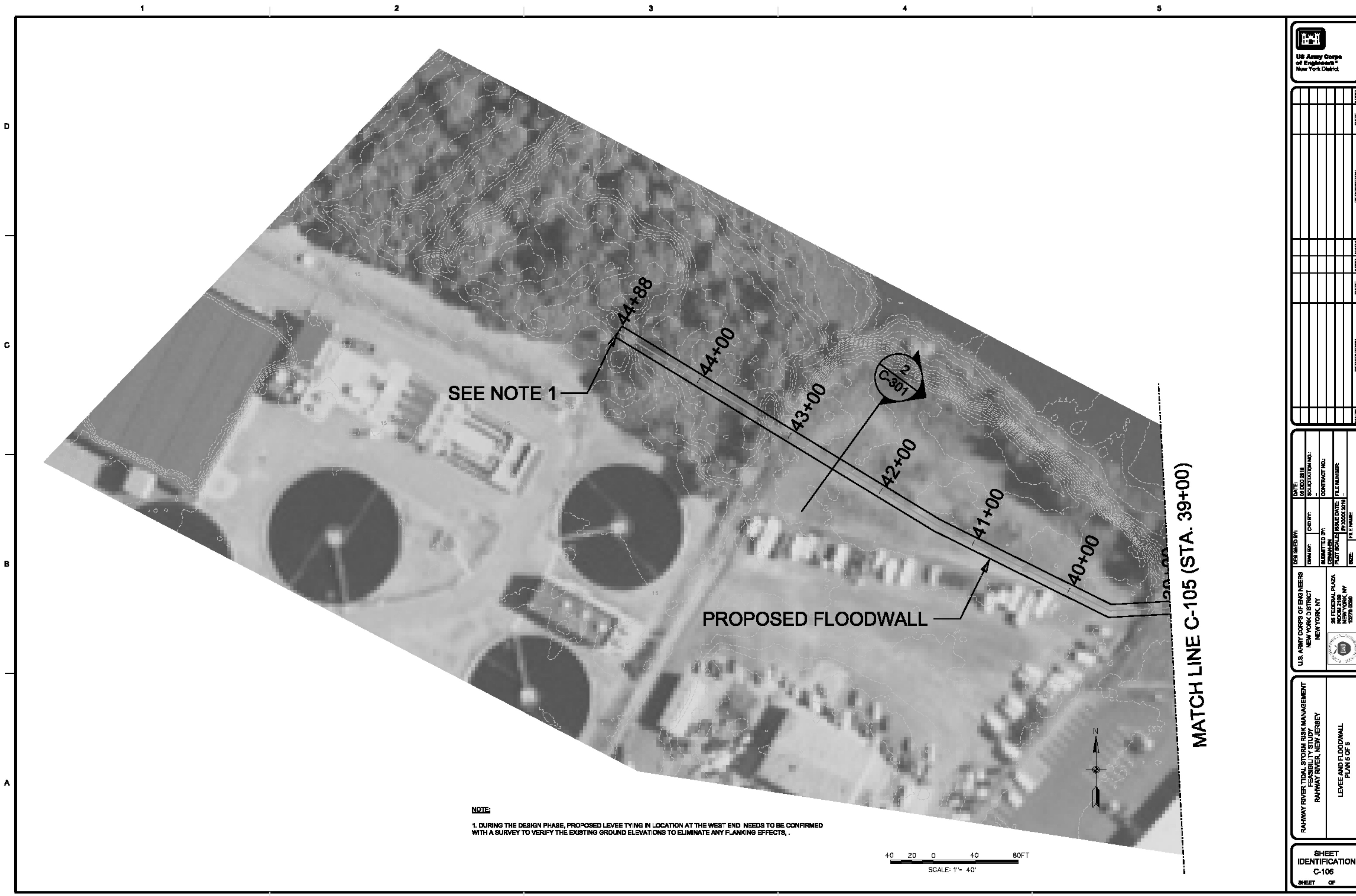


Figure 61: NED Plan Layout

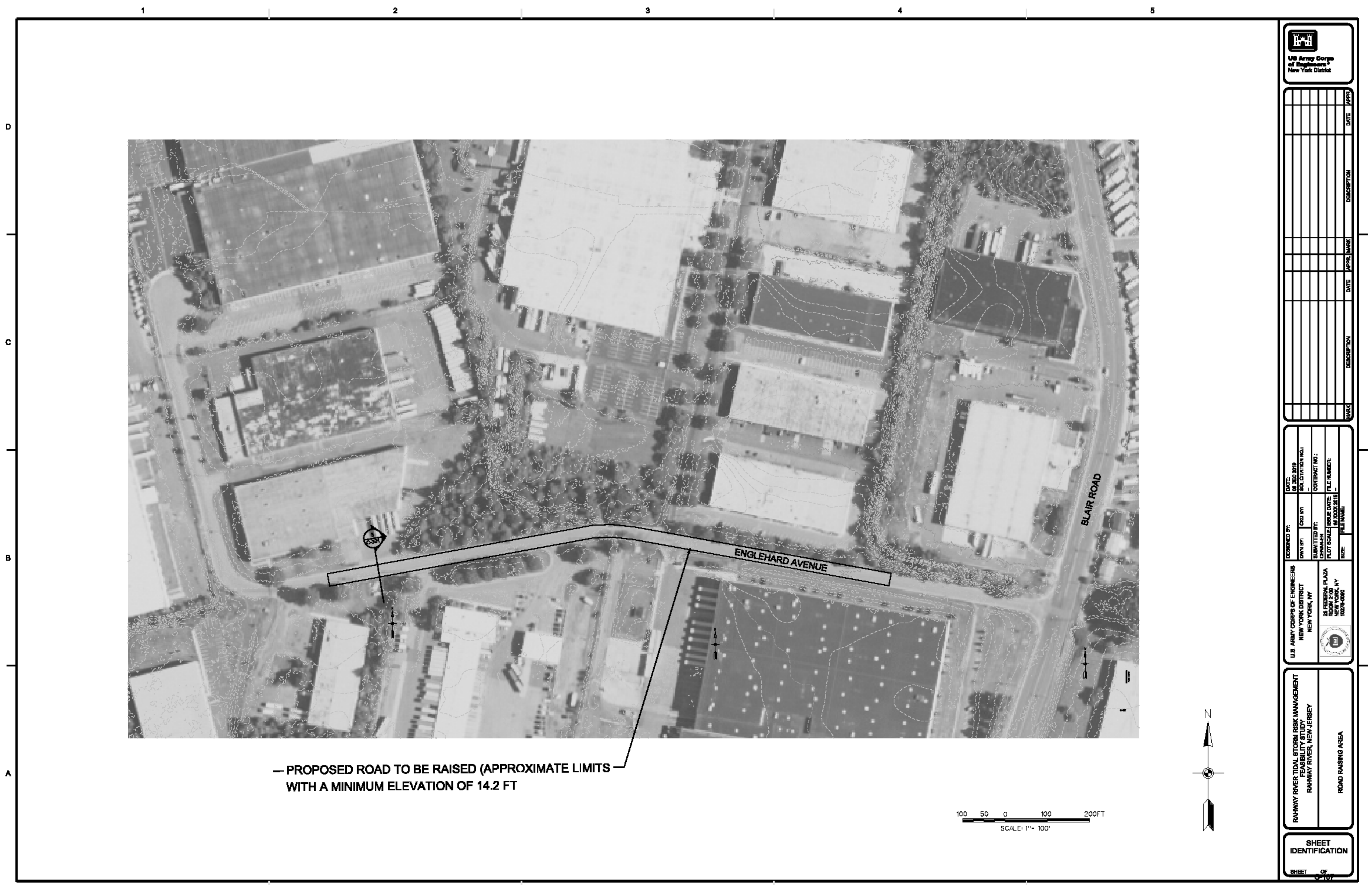


Figure 62: NED Plan Layout

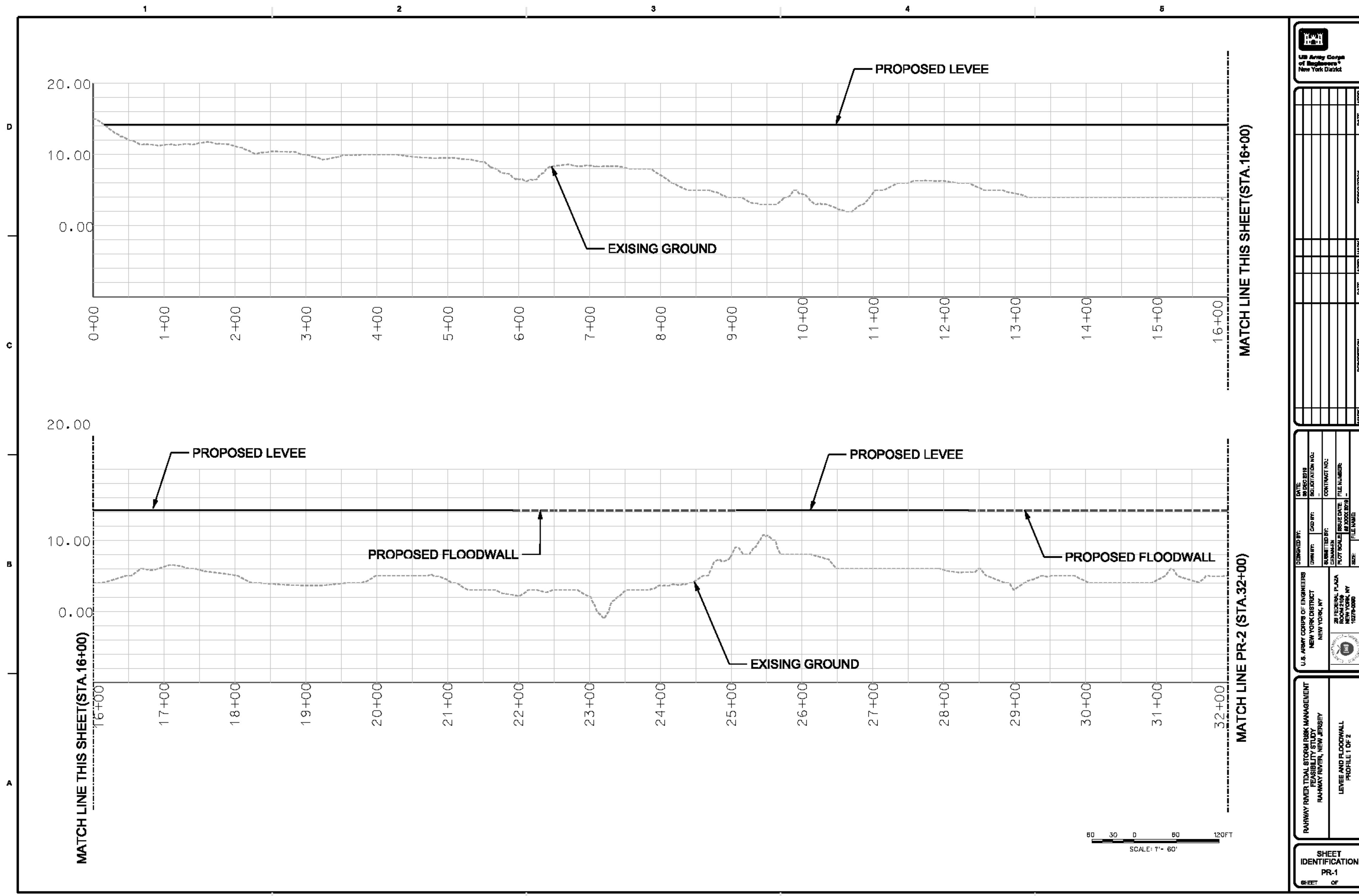


Figure 63: NED Plan Layout

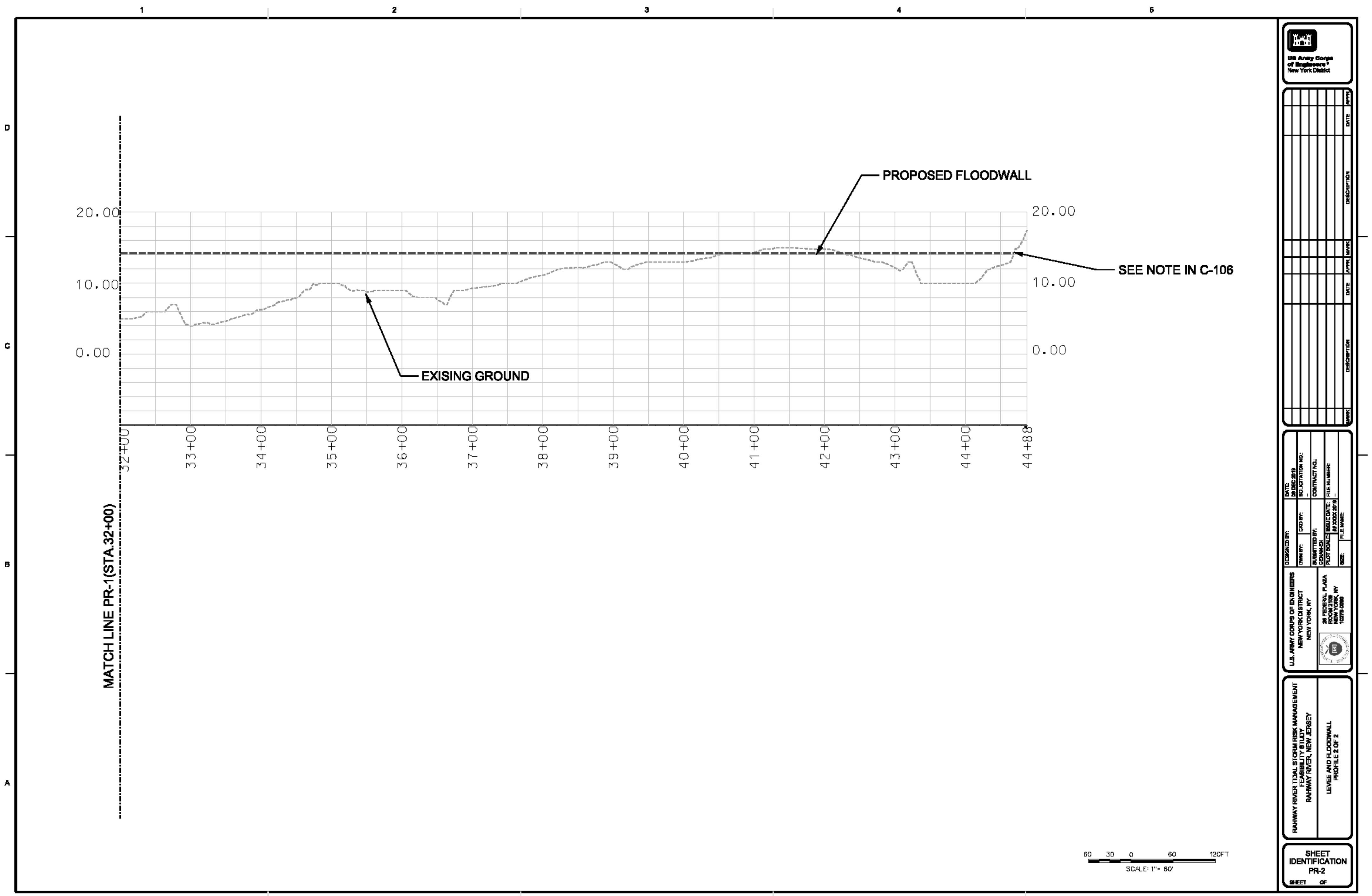


Figure 64: NED Plan Layout

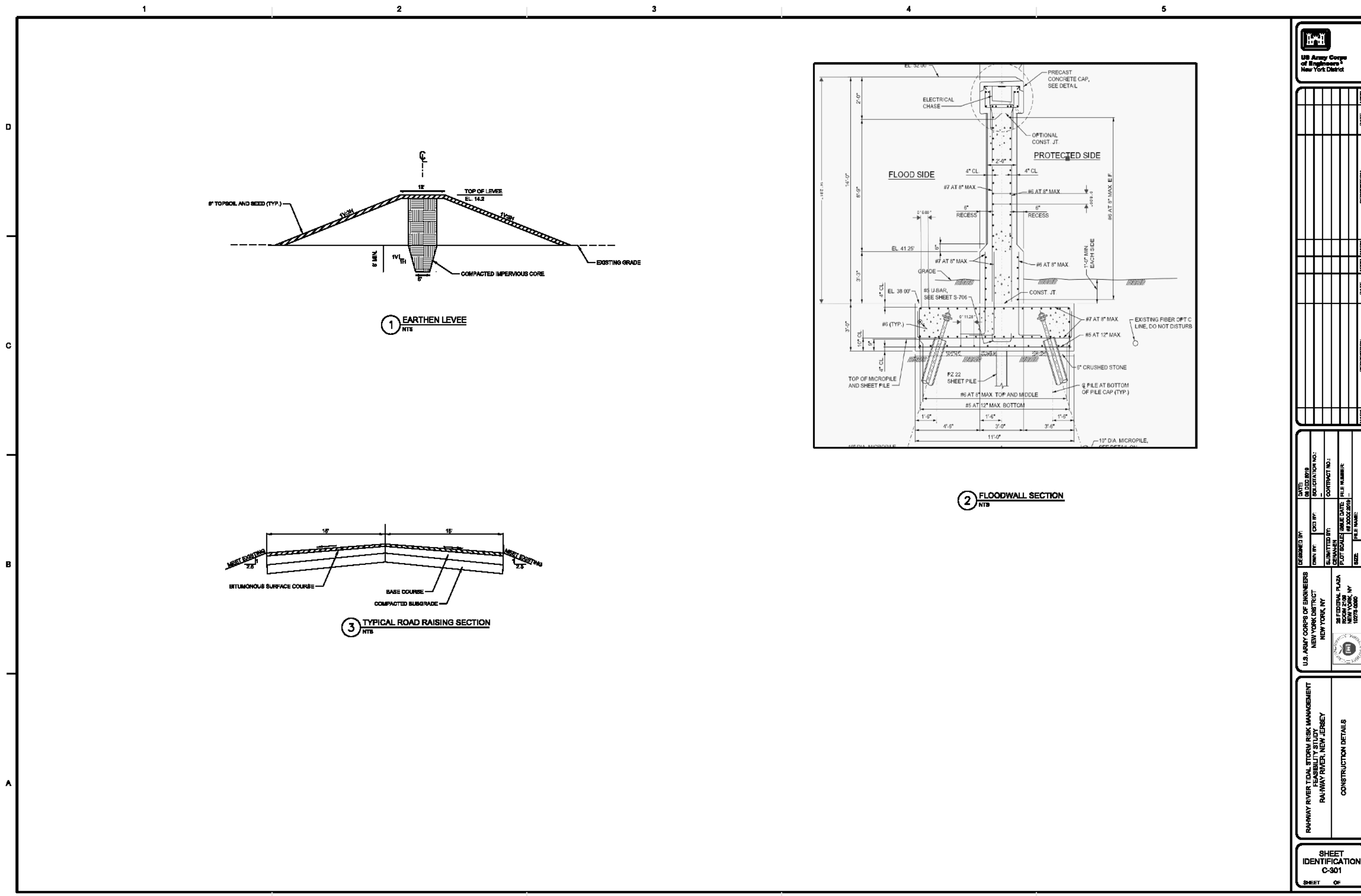


Figure 65: NED Plan Layout

n8