

Final Appendix CII

Hydraulics

Rahway River Basin, New Jersey Flood Risk Management Findings Report

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Final Draft



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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. Another feasibility study was conducted in January 2020 for the coastal portion of the basin and includes the New Jersey municipalities of Rahway, Carteret, and Linden. This findings report focuses on the entirety of the Rahway River Basin. A map of the Rahway River Basin, its municipalities, and the fluvial and coastal study areas is shown in Figure 1.



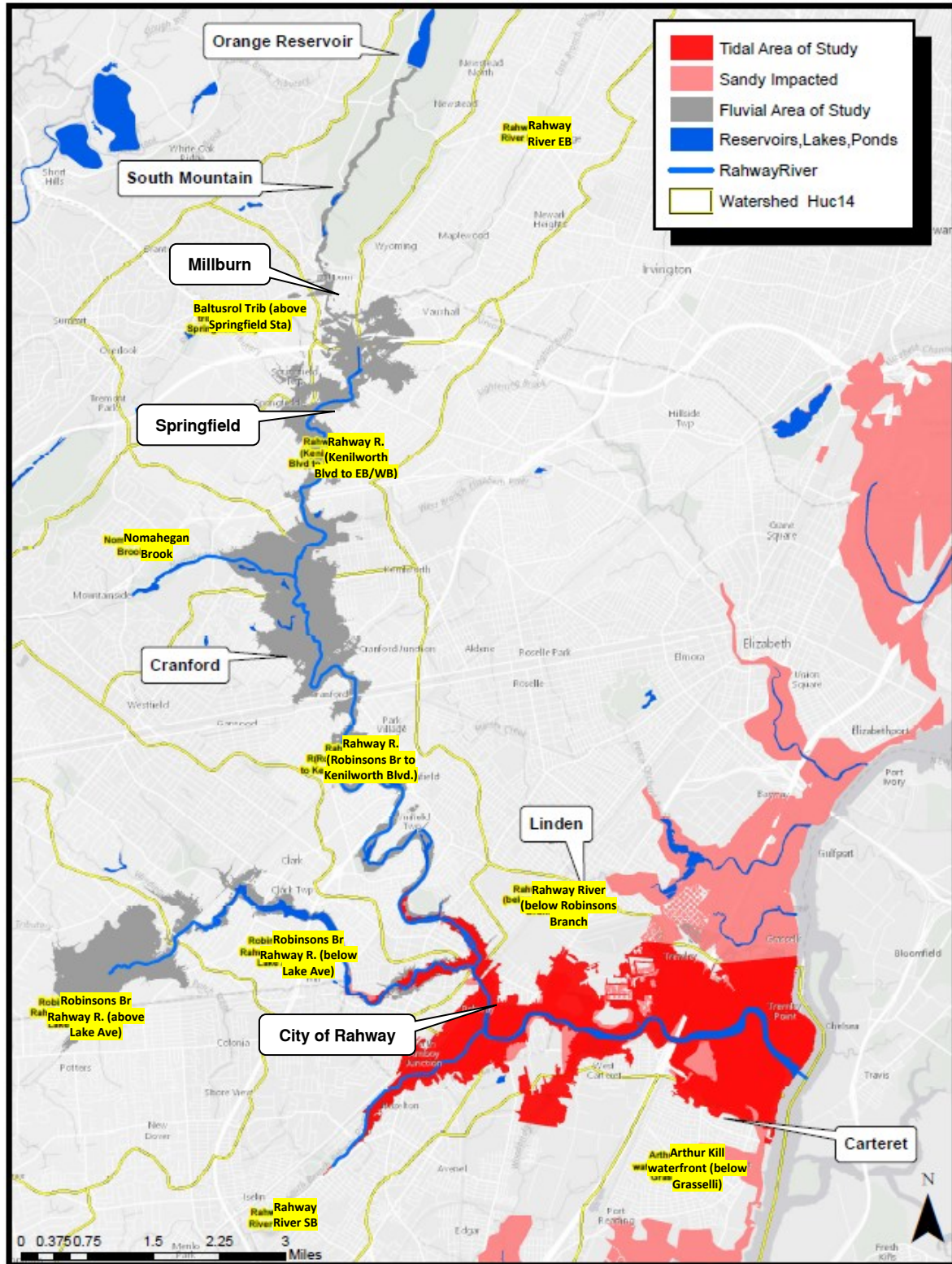


Figure 1. Rahway River Watershed



1.2 Present Flooding Problems

Periodic storms have caused severe fluvial flooding along the Rahway River. There are several main areas with high flood risk, the Townships of Millburn and Cranford and the Robinsons Branch in Rahway. Flooding in Millburn is caused by low channel capacity and the presence of several restricted bridges on the Rahway River. Flooding along the Rahway River in Cranford is caused by low channel capacity, constrictions of several bridges and dams along the river and two 90-degree bends forming a “U” turn at the Springfield Ave. just upstream of the center of the Township. The flood waters backup from the main Cranford area into the area of Lenape Park Detention Basin and Kenilworth Township. In City of Rahway at Robinson’s Branch the high risk of flooding is due to low channel capacity, the constrictions of several bridges, and the compound tidal/fluvial backwater from the main stem of the Rahway River, which is independent of the hydraulic conditions in the Robinson’s Branch.

Flooding problems are also identified in minor tributaries of Rahway River. Gallows Hill Brook experiences significant flooding due to local issues such as sediment buildup, which reduces channel capacity, and minimal maintenance since its 1981 flood control project. The rectangular channel, with 10-foot-wide and 5.5-foot-high walls, faces overtopping risks during short, high-intensity storms in its 1.2-square-mile drainage area. Flooding is primarily driven by local conditions and limited culvert capacity, rather than backwater from the Rahway River.

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, pump stations, and nonstructural measure. Authorized improvements to tidal gates are currently under design and not considered in this study.

1.4 Scope of Work

This findings report encompasses a range of activities, including updating models, analyzing stage frequency curves, conducting detailed analyses, and developing conceptual plans.



Existing hydrologic (HEC-HMS) and hydraulic (HEC-RAS) models were provided by the USACE. An additional reach was incorporated into the model to represent the East Branch of the river, and publicly available data (e.g., LIDAR and FEMA data) was utilized to estimate stream bed cross-sections and adjacent land data. These cross-sections were georeferenced to be used in the model.

The models were updated with the latest hydrology and hydraulic data available and calibrated to Hurricane Irene and Tropical Storm Ida. The boundary conditions for downstream end of the model determined in the Tidal Feasibility Study were used in this study. The models were calibrated to multiple points within the specified peak events (Irene and Ida).

The existing conditions of the floodplain were analyzed to confirm that they are reflective of observed conditions. The model was calibrated to publicly available floodmarks and stream gage data. A HEC-RAS model was developed and calibrated to produce stage-frequency data and profiles for existing conditions. The updated hydrology model (HEC-HMS) was calibrated to the full Rahway River Basin, including the tidally influenced areas. The analysis of the boundary between tidal/storm surge dominated areas and the fluvial dominated flood areas was evaluated using a compound flooding assessment. Statistical analysis was conducted to cover the full-range probabilities of all possible coincidental combinations.



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 head water for the East Branch is located in the vicinity of City of Orange, flowing downstream through South Orange and Maplewood Townships. The head water for the West Branch is located in the vicinity of West Orange, flowing downstream through the South Mountain Reservation into the Township of Millburn. 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 is impounded at Middlesex Reservoir and Milton Lake and runs through Clark and Rahway. 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 channel side slopes in the fluvial area are moderate and vary from 5 to 15 ft. in height. The channel banks in the coastal area are relatively low and vary from 2 to 6 feet in height. Robinsons Branch has channel banks about 4 ft in height. South Branch has channel banks about 6 ft in height.

The channel bottom in the Rahway River has a variable slope. Closer to the Arthur Kill with much of the overbanks being wetlands, channel banks are very low-lying. The channel bottom slope at the tidal influenced area is also very mild at 2.0 ft/mile. South Branch has a channel bottom slope of 10 ft/mile. Robinsons Branch a channel bottom slope of 3.0 ft/mile. The channel bottom slope is approximately 8.0 ft/mile from the confluence with Robinsons Branch to Cranford and 3.0 ft/mile from Cranford to the confluence between the East and West Branches. The West Branch of the Rahway River by the Township of Millburn and the South Mountain Reservation the slope becomes steep, approximately 55 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 tidal influenced area. Robinsons Branch has an average



channel width of approximately 40 ft in the tidal influenced area. In the fluvial area, the width of the channel at the banks varies in width from 30 to 40 ft. in the East and West Branches to 50 to 60 ft. just downstream of Route 22 to approximately 30 to 40 ft. through the Lenape and Nomahegan Parks (by Cranford Township), widening to 50 to 70 ft. near the confluence with Robinson's Branch.

Overall, although is a highly develop sub-burb of New Jersey, the banks of the river are densely cover by trees and shrubs. Areas adjacent to the river are mostly protected by the non-Federal sponsor (NJDEP) Green and Blue Acres Program. The debris produced by the high vegetation in combination with the quick rising flows results in floods in many areas of the Rahway River Basin. 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. Many areas adjacent to the river are mostly protected by the non-Federal sponsor (NJDEP) 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. NAVD88 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 Rahway River in the Townships of Millburn and Cranford and Robinson's Branch at Rahway begin to experience fluvial flooding at and above the 10% chance of annual exceedance (10-yr) event.

At this stage, the areas upstream of Essex St and Oakland Rd experience flooding due to low channel capacity and the presence of restricted bridges on the Rahway River. The low-lying area between Park Dr. and Springfield Ave. near the Nomahegan Park Back experiences flooding due to back water from a tributary of the Rahway River and some street flood upstream of Hansel Dam. For peak flows between the 10% chance of annual exceedance (10-yr) and the 4% chance of annual exceedance (25-yr) events, water surface elevations (WSEs) in the Rahway River overtop the



Nomahegan Park levees. Although there are some inconsistencies in the top elevation of the levees, both sides of the levee system can contain approximately the same event. For storm events above the 4% chance of annual exceedance (25-yr), the stage of the Rahway River waters starts producing floods in the following areas:

1. Kenilworth residential area due to backwater caused by the constrictions of the Kenilworth Blvd. Bridge.
2. At the right overbank between Willow St. and Brookside Place, near Cranford High School.
3. At the left and right sides overbanks and behind the existing levee system, the residential area at the residential area surrounding Riverside Dr., Brookdale Rd., Edgewood Rd., Glenwood Rd., Summit Rd., Edgar Ave., Franklin Ave., Balmiere Pkwy. and Doering Way.
4. And the commercial area surrounding Chestnut St.

2.3 Existing Hydraulic Features

Some areas along the Rahway River have seen a decrease in flood risk due to improvements implemented through the years. These are several of the existing Federal and non-Federal projects in place:

1. Springfield Levees: The levee system is located in the right bank of the Rahway River in Springfield Township. The system is divided into three (3) segment with varying top elevations. The north segment is approximately 1,560 long with a variable top elevation between 88.5 and 90 ft. NAVD88. The middle segment is approximately 1,500 ft. long with a top elevation of approximately 86 ft. NAVD88. This segment has the lowest top elevation of the three, with the smallest top width and is lacking in maintenance. The most downstream segment is approximately 1,900 ft. long and has with a top elevation of approximately 88 ft. NAVD88. The upstream end of the system is located at the Springfield Ave. Bridge (just downstream of I-78) and ends just upstream of the confluence between the Rahway River and Van Winkles Brook.



2. Nomahegan levee system: The Nomahegan Park levee system is located on both sides of the banks in the Rahway River; protecting a commonly flooded residential area in Cranford. The left and right bank levees are approximately 1,800 ft. and 4,000 ft. long respectively. The levees are approximately 4 to 6 ft. high and have approximately a 6 ft. top width. There is also a flood reduction plan developed by the Township of Cranford in regards to the levees. It includes the construction of interior drainage stormwater pipes, pump stations for the east and west side of the existing levees, improvements to the stormwater sewer system and improvements of the existing levees. The Township plans are divided into the following phases:

Phase 1: Drainage swale approximately 500ft north of Belmont Ave. and express stormwater sewer pipeline, constructed in 2006.

Phase 2: Riverside drive stormwater pump station, and north and south gravity storm sewer interconnection, constructed in 2008.

Phase 3 & 4: Improvements to the Nomahegan Park and residential area existing levee system, currently on hold.

Phase 5: Park Ave. pumping station, Penn Rd. stormwater sewer pipeline and local collector system, currently on hold.

3. Lenape Park Dam: The dam creates dry detention area with a capacity of approximately 2100 acre-ft. at the top of the embankments, enough to hold a 1% chance of annual exceedance events (100-yr) without flood without overtopping. The secondary, or emergency, spillway is designed to overflow for the 4% chance of annual exceedance event (25-yr). The dam consist of a concrete spillway 100 ft. long and approximately 25 ft. high and earthen embankments approximately 10,000 ft. long with an approximately 10 ft. top width and one vertical to thee horizontal (1V:3H) side slopes. The right dam embankments located in the township of Cranford and Westfield are fairly well maintained. By contrast, the left embankment in Kenilworth, has a considerable amount of vegetation and trees growing on top.



4. Droescher's Dam: The dam is located in Cranford Township, Union County, New Jersey, on the Rahway River near Droescher's Mill at 347 Lincoln Avenue East. The dam features a floodgate that can be manually operated to regulate water levels in anticipation of significant rainfall. The Cranford Police Department and the Department of Public Works collaborate to monitor the Rahway River continuously, ensuring timely adjustments to the floodgate as necessary.
5. Sperry Dam: The dam is located in Cranford Township, Union County, New Jersey, and is equipped with a manual floodgate that allows for the control of water flow within the Rahway River. This operational capability is crucial for managing river levels during periods of excessive rainfall. The Office of Emergency Management (OEM) in Cranford routinely monitors weather conditions and forecasts. In the event of anticipated heavy rainfall or potential flooding, the OEM assesses the necessity of opening the river gates, including those at Sperry Dam, as a preventive measure to mitigate flood risks.
6. Jacksons Pond Dam: The dam is located in Clark Township, Union County, New Jersey. The dam is an earth and gravity dam with a height of approximately 15 feet and a length of 276 feet. The dam creates Jacksons Pond, which covers a surface area of about 15 acres and provides a normal storage capacity of 90 acre-feet. The drainage area feeding into the pond encompasses 40 square miles. The County of Union owns the dam, and it is cataloged under the National Inventory of Dams with the ID NJ00771.
7. USACE South Branch Flood Control Project of 1968. This is 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 approximately 5,300 ft long and was re-graded in 2015 to the original design height of 12.6 ft. NAVD88 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.

In-Progress Review Draft, Subject to Change



3.0 HYDRAULIC BASIS OF DESIGN

3.1 Model Development

3.1.1 *Original basin model*

In the 2020 Feasibility Study, the fluvial analysis of the Rahway River was 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. In this study, this model was extended to include the East Branch of the Rahway River. 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.

3.1.2 *Updated basin model*

In this study, 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 (Version 6.5). The hydraulic model used for this flood risk management study encompasses the original components in the fluvial and coastal area in the 2020 Feasibility Study as well as new components of the East Branch.

The modelling of bridges and weirs is generally as provided by NAN and was only changed as needed to improve model stability. Two bridges have been tested for impact and were found to be minor to minimal (i.e., no significant impacts) as shown in Figure 65 and Figure 66. Three lateral



weirs have been investigated for impact. The weirs are never or rarely overtopped, and the weir coefficients have minimal impact as shown in Figure 67, Figure 68, and **Figure 69**.

The East Branch reach HEC-RAS unsteady model was created using a prior steady-state HEC-2 model for the channel data supplemented with 1-meter LiDAR terrain for the overbanks. The model was then run as an unsteady model and updated with ineffective flow areas, obstructions, and Manning's n-values. Blocked obstructions were added into the model at locations where large buildings are located in the immediate overbanks. This was necessary to correctly reflect the floodplain storage in the unsteady model. A preliminary calibration was performed on the reach before adding to the overall Rahway unsteady model. After addition to the overall Rahway River hydraulic model, the calibration for East Branch was again adjusted. The East Branch and West Branch gage locations are shown in Figure 2. East and West Branch Rahway with gage locations



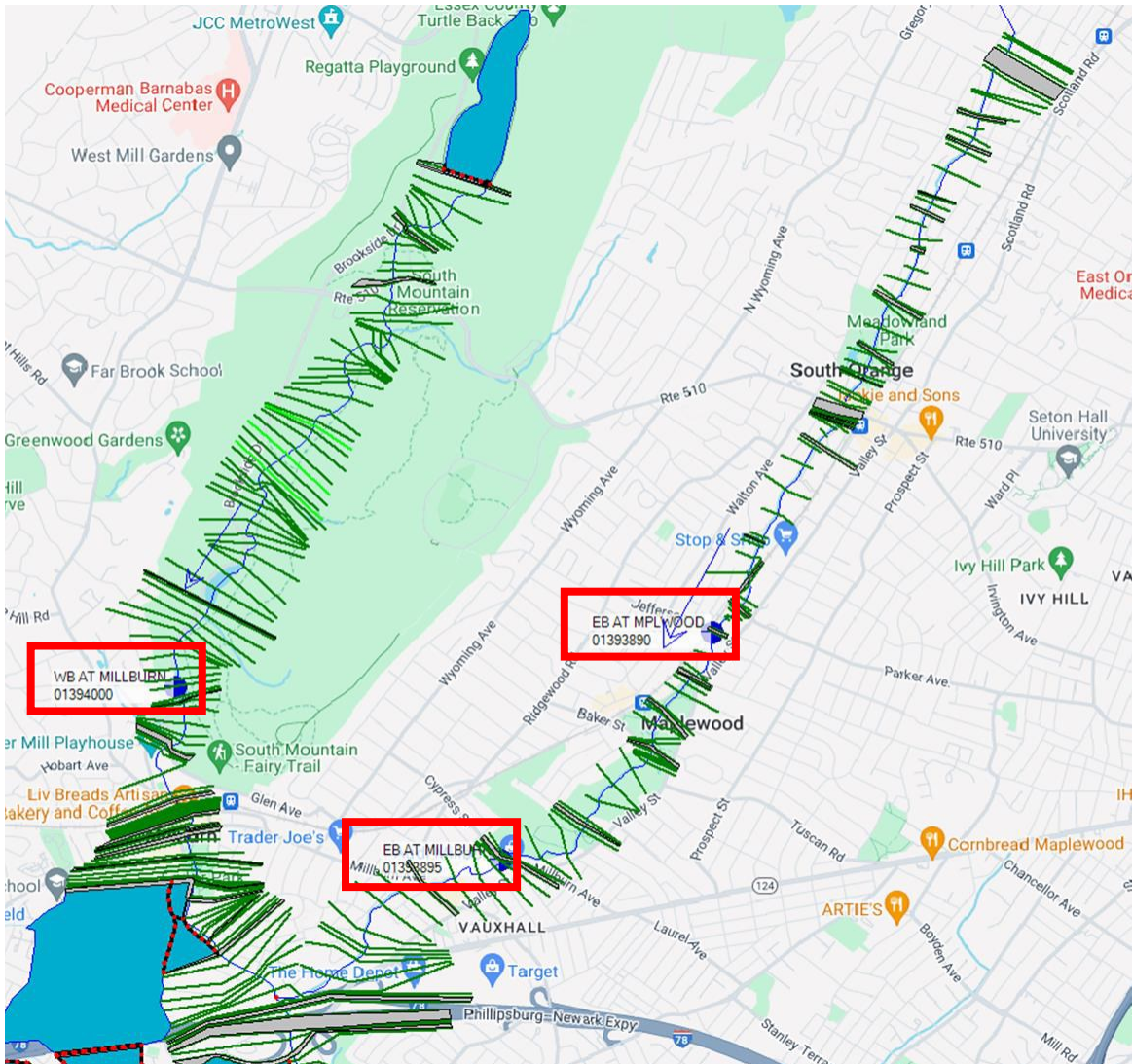


Figure 2. East and West Branch Rahway with gage locations



3.2 Model Calibration and Validation

3.2.1 Original basin model

This section presents the calibration efforts made in the 2020 Feasibility Study. In the 2020 Feasibility Study, 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 used to characterize channel modification. This increased flow conveyance capacity of the channel, reducing flood during fluvial event, but not the same during 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. The figures depict the good match between the observed HWMs and calculated WSEs.



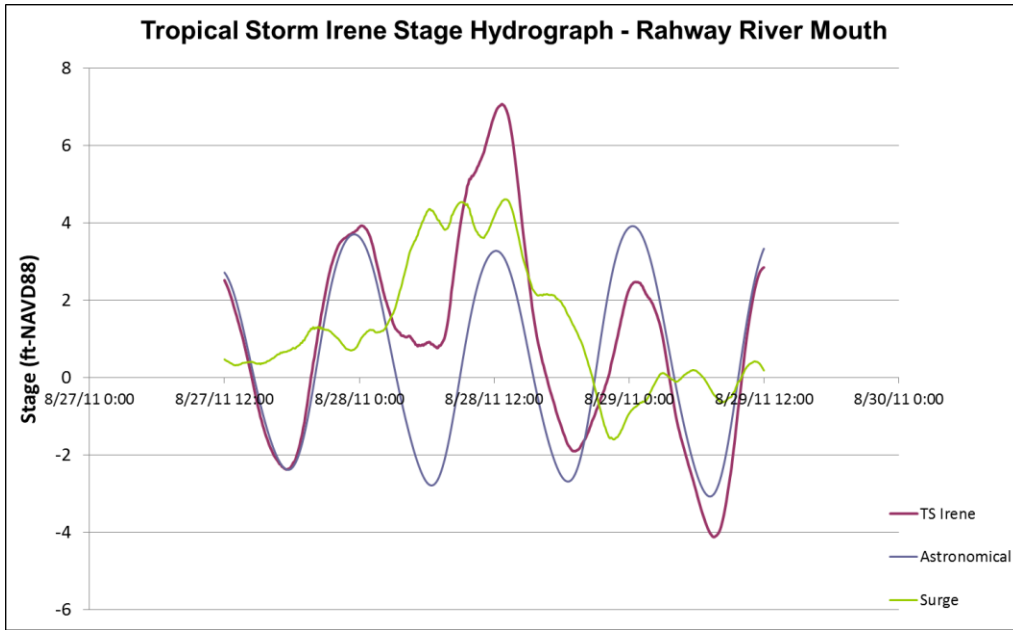


Figure 3. Stage hydrograph for Tropical Storm Irene

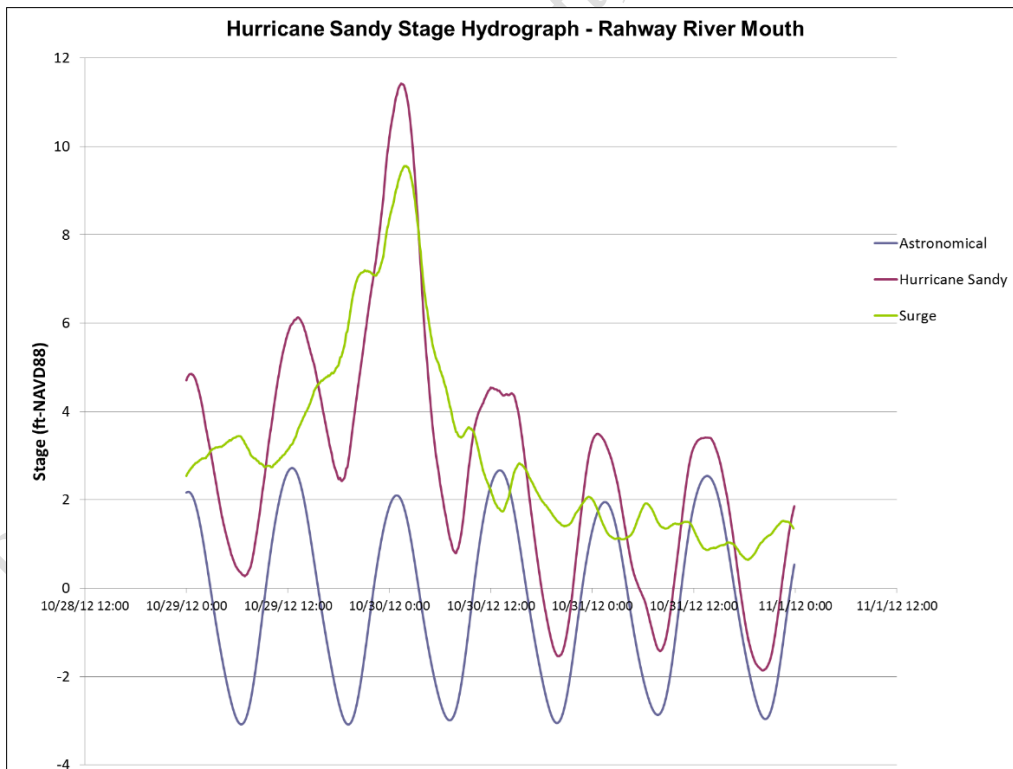


Figure 4. Stage hydrograph for Hurricane Sandy



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	<i>01396000 Robinsons Branch</i>
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	<i>01395000 Rahway</i>
Millburn-Clark	28743.80	15.03	14.98	0.05	182 Grand
Rahway	27995.02	14.49	14.43	0.06	<i>Confluence</i>
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.



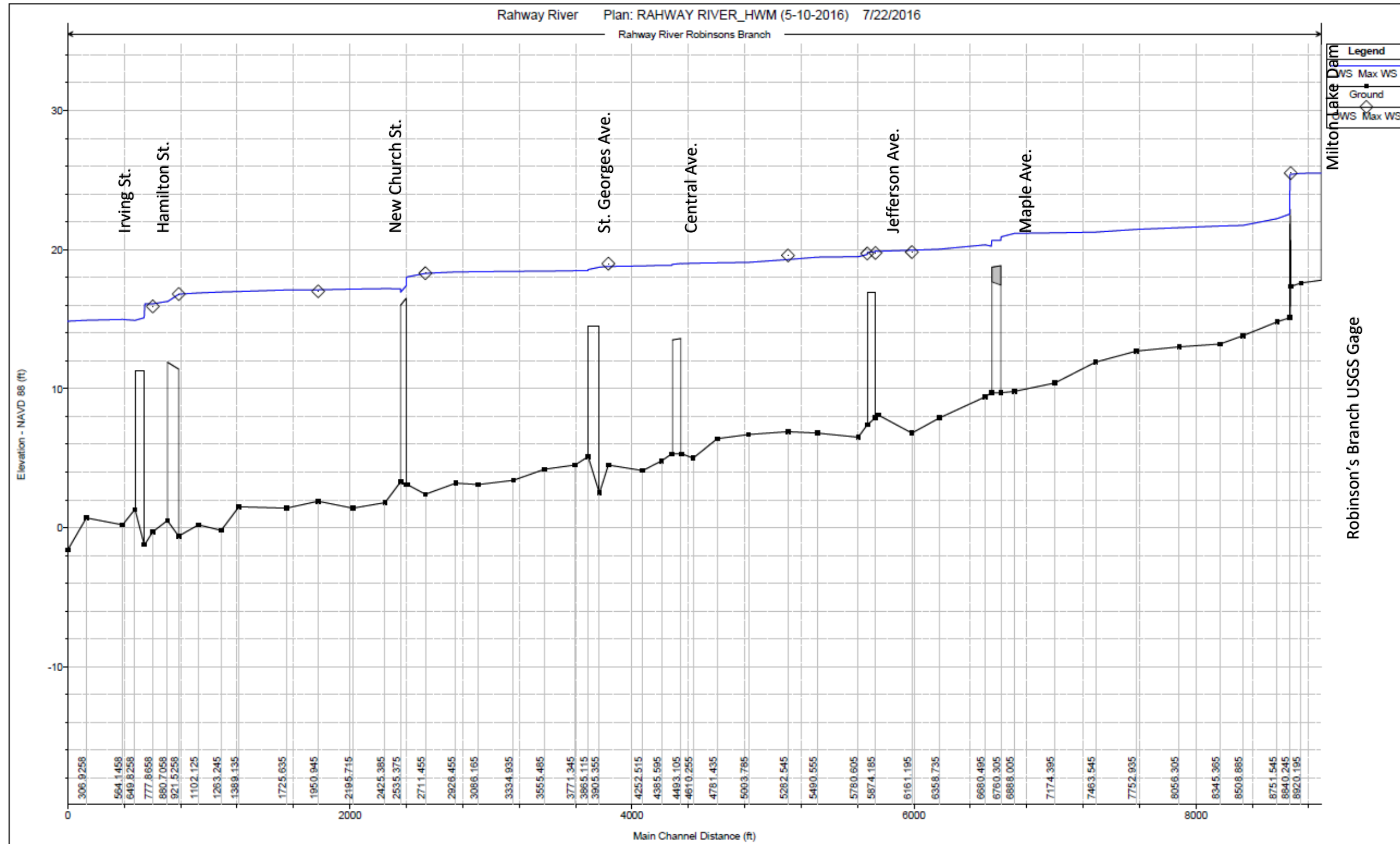


Figure 5. Computed water surface profile and observed HWMs for Tropical Storm Irene in Robinson's Branch

how



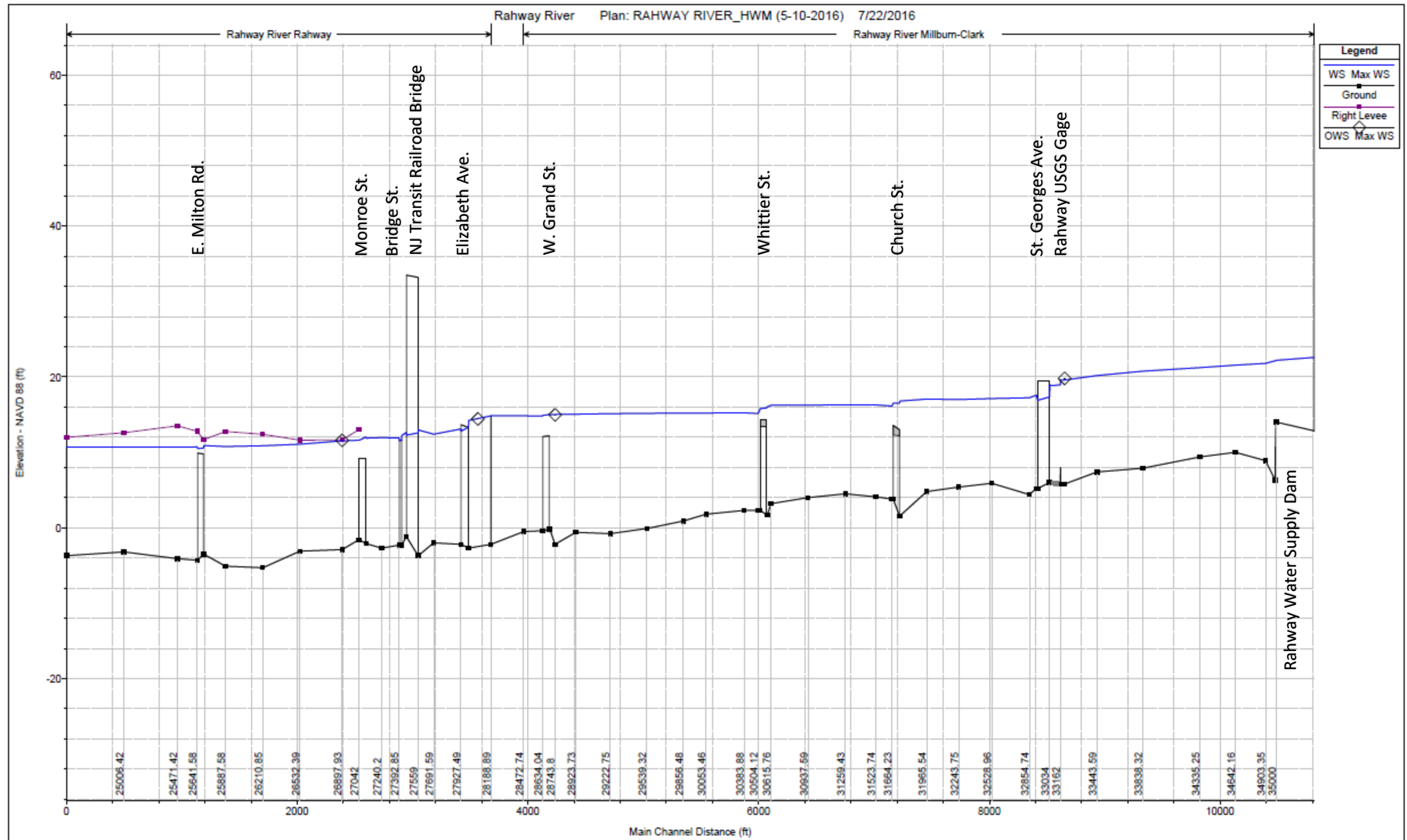


Figure 6. Computed water surface profile and observed HWMs for Tropical Storm Irene in the Rahway River between Rahway Water Supply and South Branch confluence



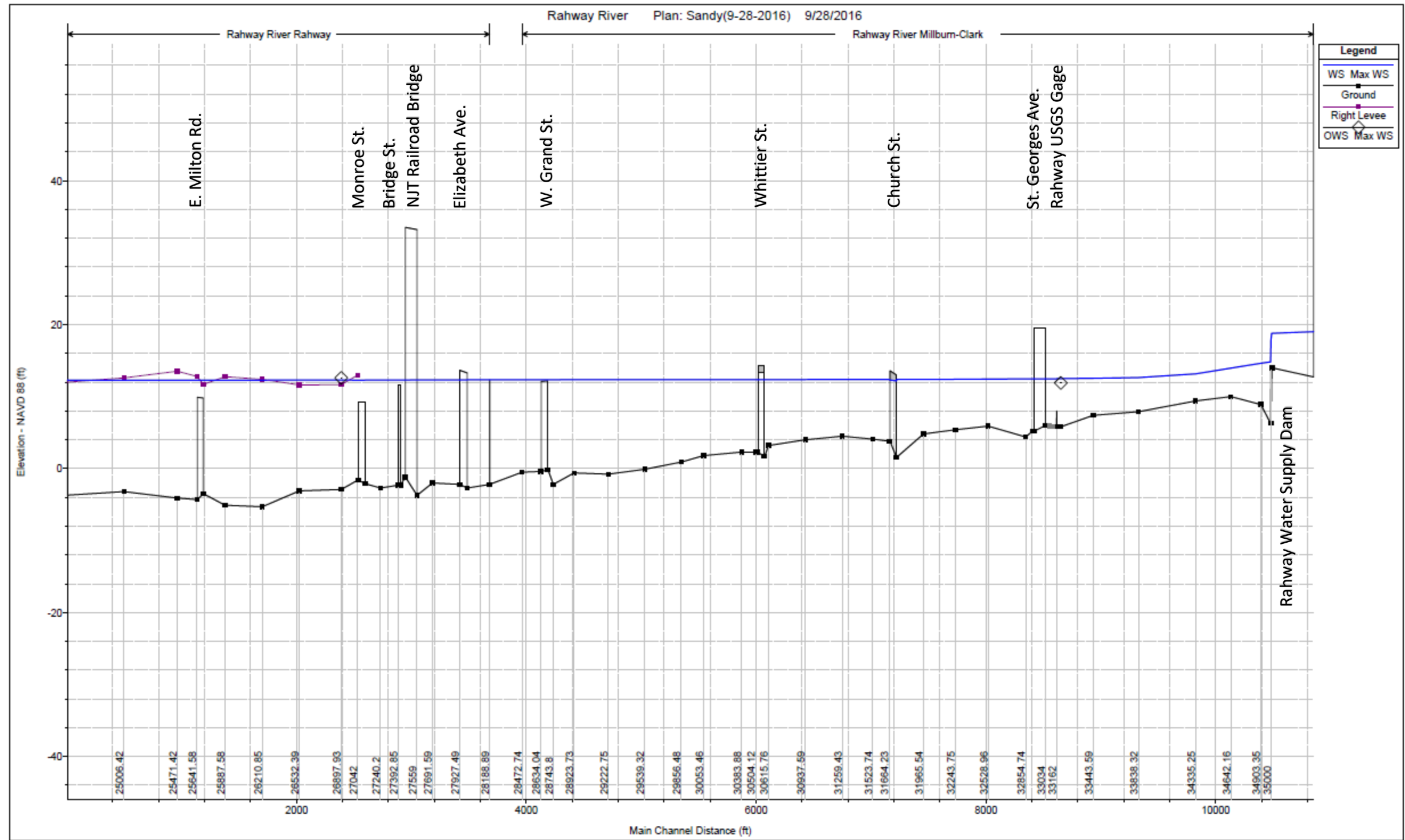


Figure 7. Computed water surface profile and observed HWMs for Hurricane Sandy in the Rahway River between Rahway Water Supply and South Branch confluence



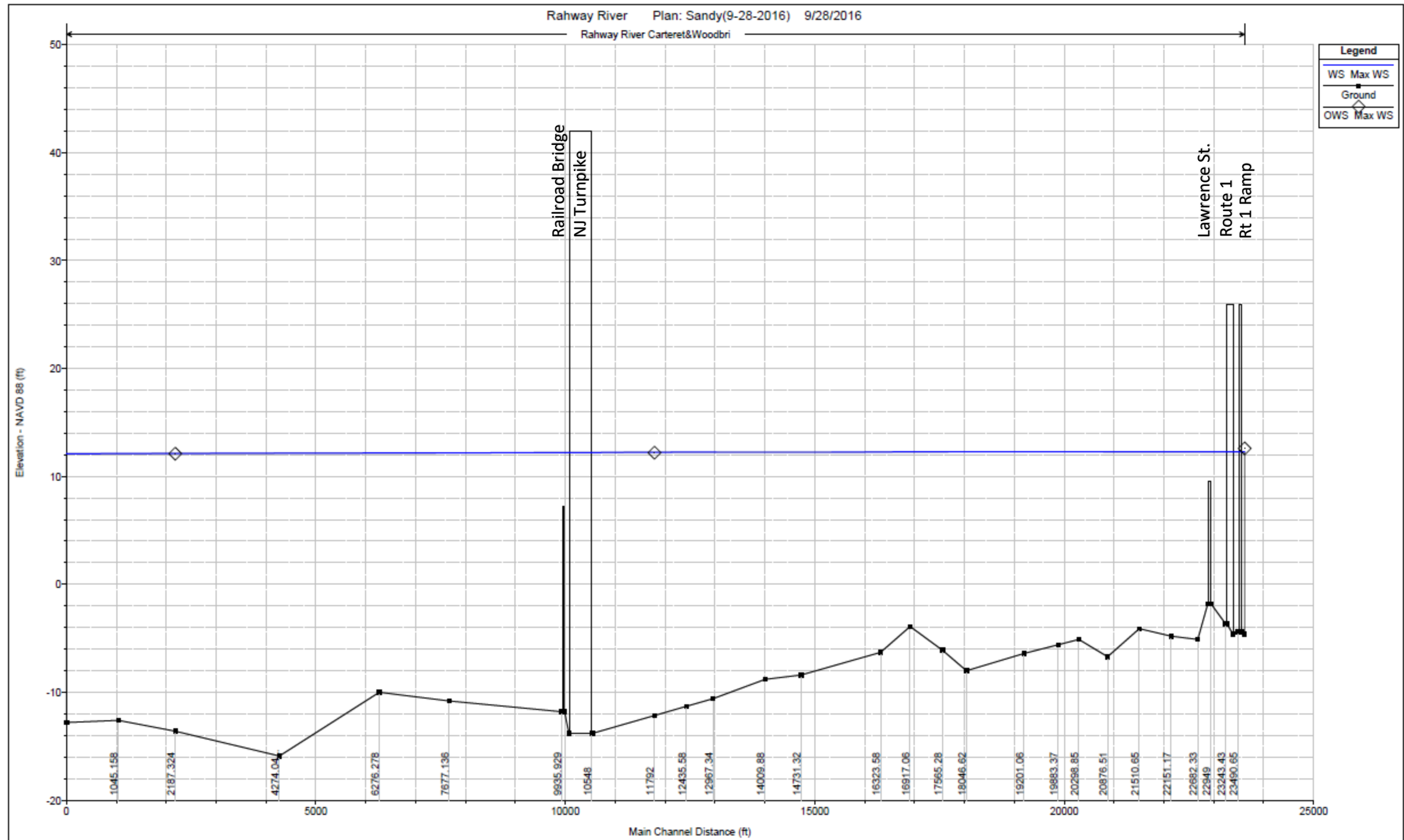


Figure 8. Computed water surface profile and observed HWMs for Hurricane Sandy downstream of South Branch to the Arthur Kill



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 2016 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 Fluvial Study.

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 Coastal Stage Hydrographs and Downstream Boundary Condition

3.2.2 Updated basin model

The goal of this section is to document the calibration of the updated USACE model (East Branch incorporated) for Hurricane Irene (2011) and Tropical Storm Ida (2021), prioritizing peak discharge, peak timing, and stage.

1) Gage Data Availability

Three (3) USGS gages in the watershed were utilized to calibrate modeling results with observed and recorded data:

1. USGS 01394500 Rahway River at Springfield: Located in Springfield Township by Route 22
 - Partially captures Hurricane Irene but the peak flow is estimated.
 - Captures Tropical Storm Ida.
2. USGS 01395000 Rahway River at Rahway: Located in the Town of Rahway in the Rahway River Park.
 - Partially captures Hurricane Irene, peak is not available.
 - Partially captures Tropical Storm Ida, peak and tail are estimated.
3. USGS 01396000 Robinson Branch at Rahway: Located in the Town of Rahway after the Milton Lake



- No available data for any of these two storms.

The locations of these gages and availability of flow and stage data are presented in Figure 9, Figure 10, and Table 20.



Figure 9. Location and data availability of the USGS stream gages in the Rahway basin



Table 3. Data availability of the USGS stream gages in the Rahway basin

Gages	Flow	Stage
EB at Maplewood		✓
EB at Millburn		✓
WB at Millburn		✓
Springfield	✓	✓
Kenilworth		✓
Rahway	✓	✓
Robinsons	✓	✓

In-Progress Review Draft, Subject to Change



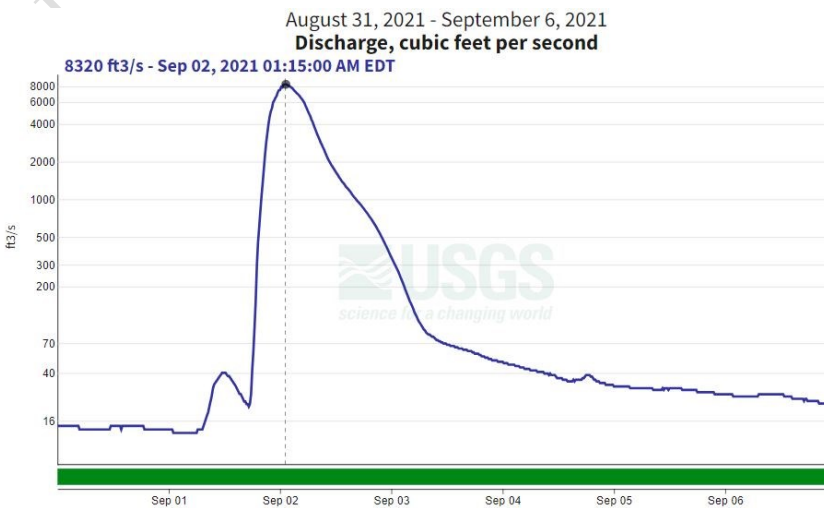
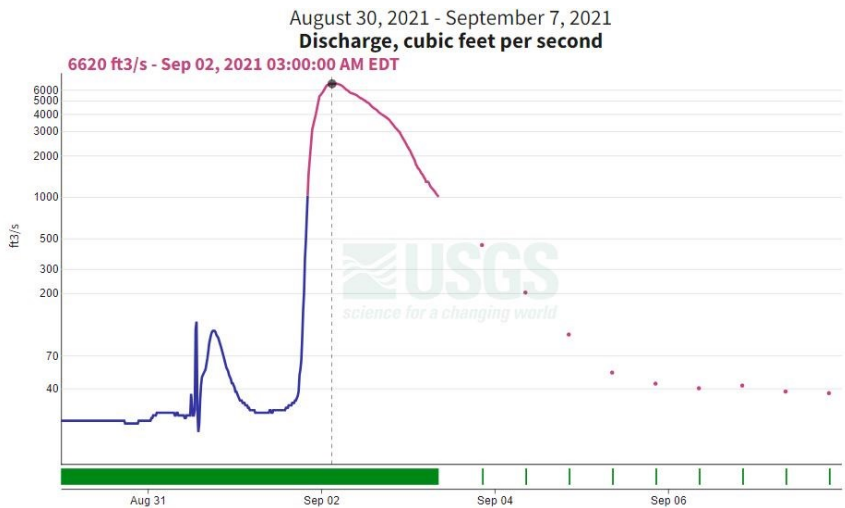
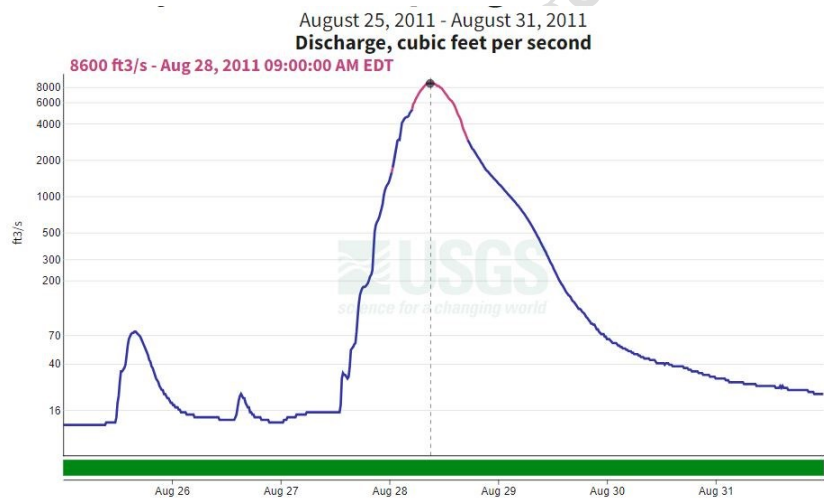


Figure 10. USGS gages flow data for Hurricane Irene (top) and Ida (bottom) at Springfield (right) and Rahway (left)



The Hydrology Appendix of the 2020 Feasibility Study provides the estimated hydrograph for the Rahway gage and Robinsons Branch gage during Hurricane Irene, which has been utilized in this report for model calibration. Figure 11 depicts the reconstituted hydrographs from the USACE perspective.

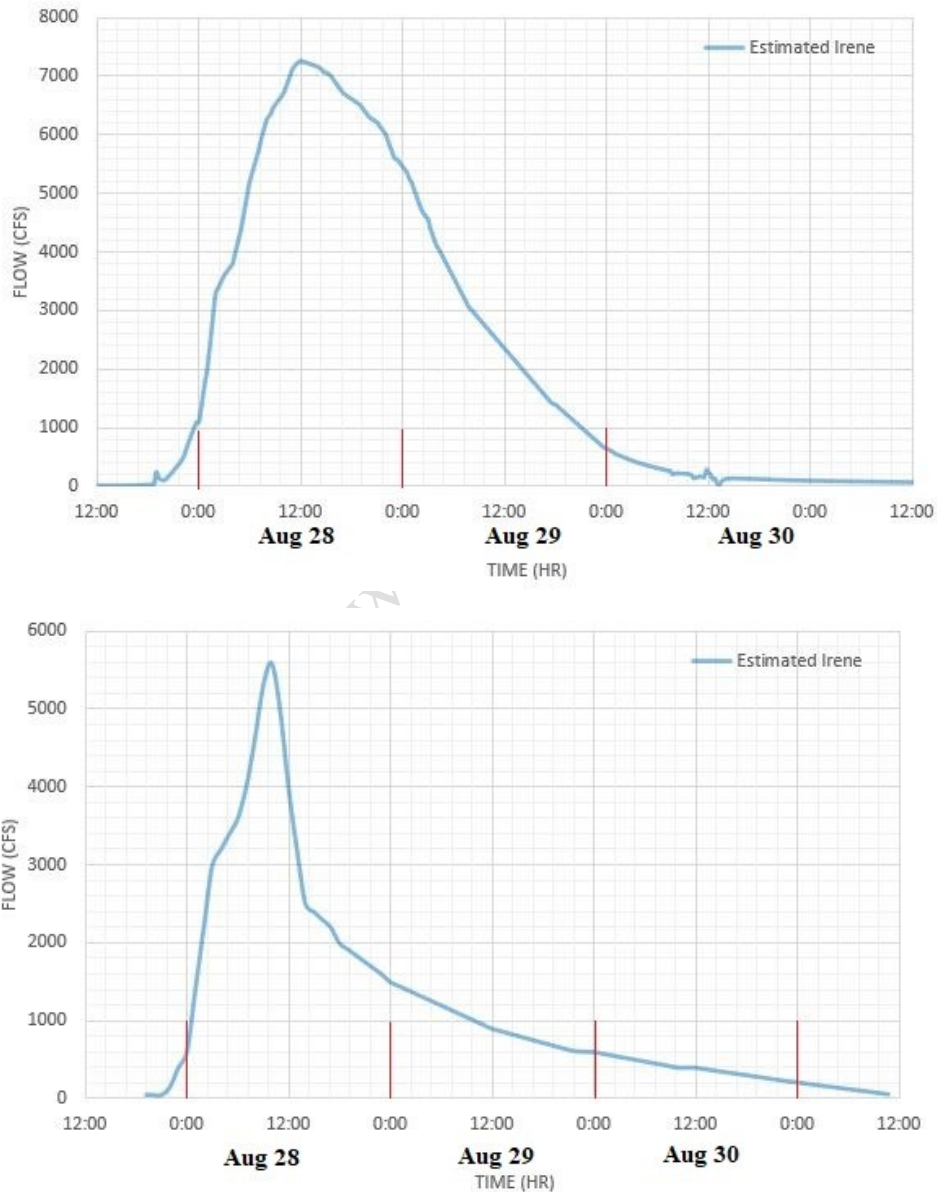


Figure 11. Estimated flow data from the USACE report for Hurricane Irene at Rahway gage (top) and Robinsons Branch (bottom)



2) Calibration of the updated overall model

As depicted in Figure 12, the following adjustments were made to the updated model to improve the stability of the model and consistence of results.

- Mannings 'n' values – affect stream velocity – routing/water surface
- Ineffective flow areas – block flow but not storage
- Blocked obstructions - large buildings – block flow and storage
- Storage Areas – many areas included buildings – problem with LiDAR showing adjacent grade at buildings and not the volume of the building structures.



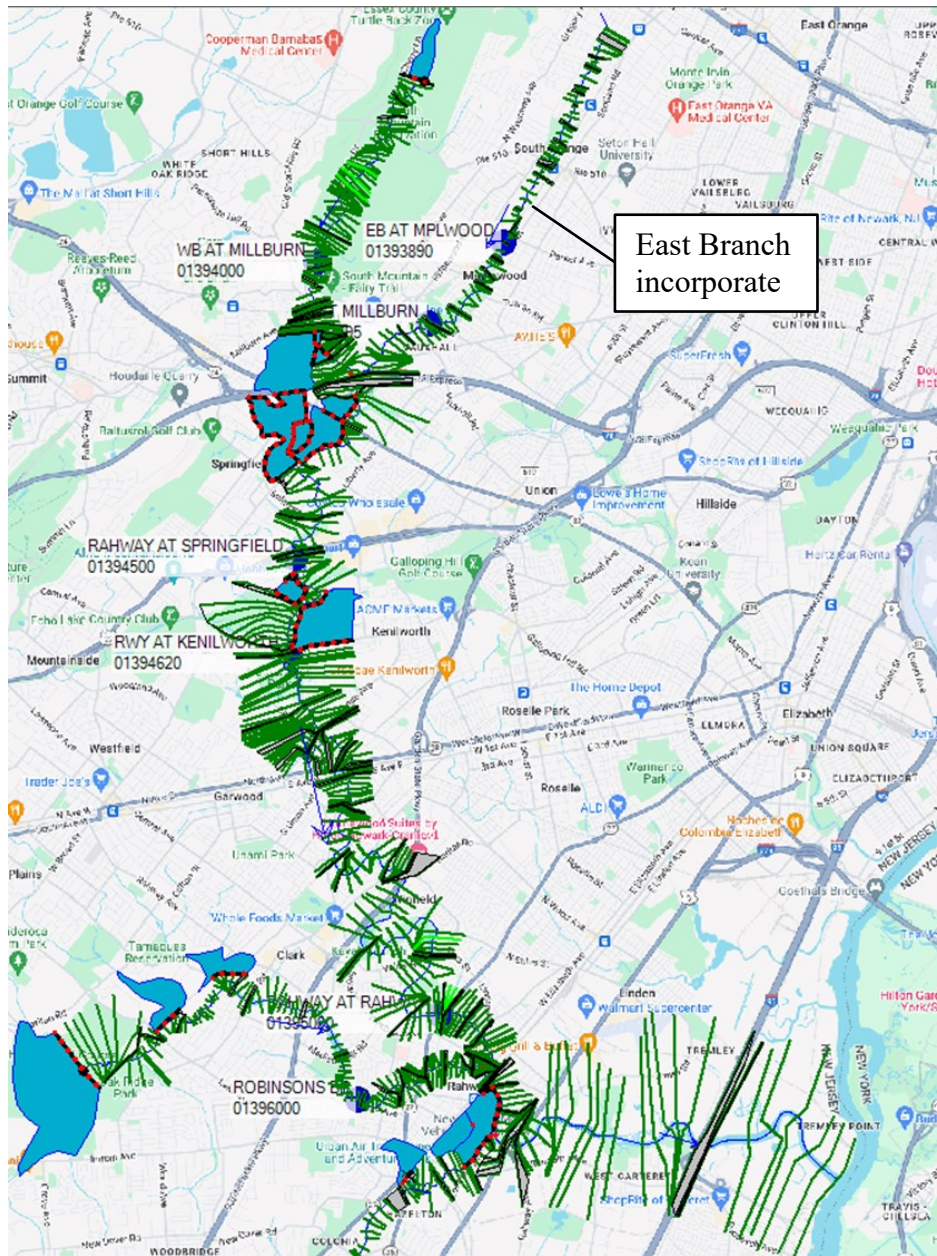


Figure 12. Updated Rahway basin model

3) Calibration for Hurricane Irene and Ida

The results presented below are comparisons between the observed data and calibrated HEC-RAS model output for Hurricane Irene and Ida at two representative gages. Summary tables of the stages and flows for Irene and Ida are provided in tables followed by the graphical figures.



West Branch at Millburn - Irene

As depicted in Table 4 and Figure 13, the calibrated model demonstrates a good approximation of peak stage and time of peak compared to the observed data. The peak stage differs by 0.08 ft at this location.

Table 4. Calibration of peak stage and flow at West Branch at Millburn during Irene

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	177.51	N/A
Model	177.43	2831

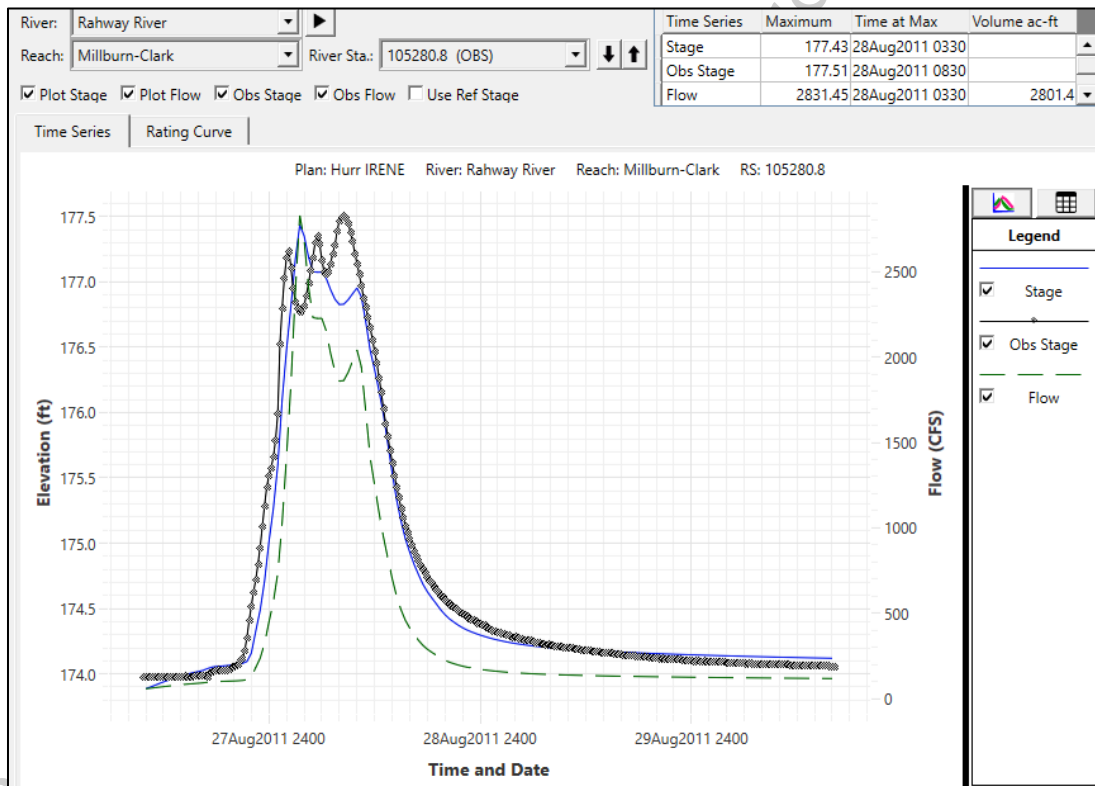


Figure 13. Hydrograph of West Branch at Millburn during Irene



West Branch at Millburn - Ida

As depicted in Table 5 and Figure 14, the calibrated model produces a good approximation of peak stage and timing compared to the observed data. The peak stage differs by 0.31 ft at this location.

Table 5. Calibration of peak stage and flow at West Branch at Millburn during Ida

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	177.85	N/A
Model	177.54	3024

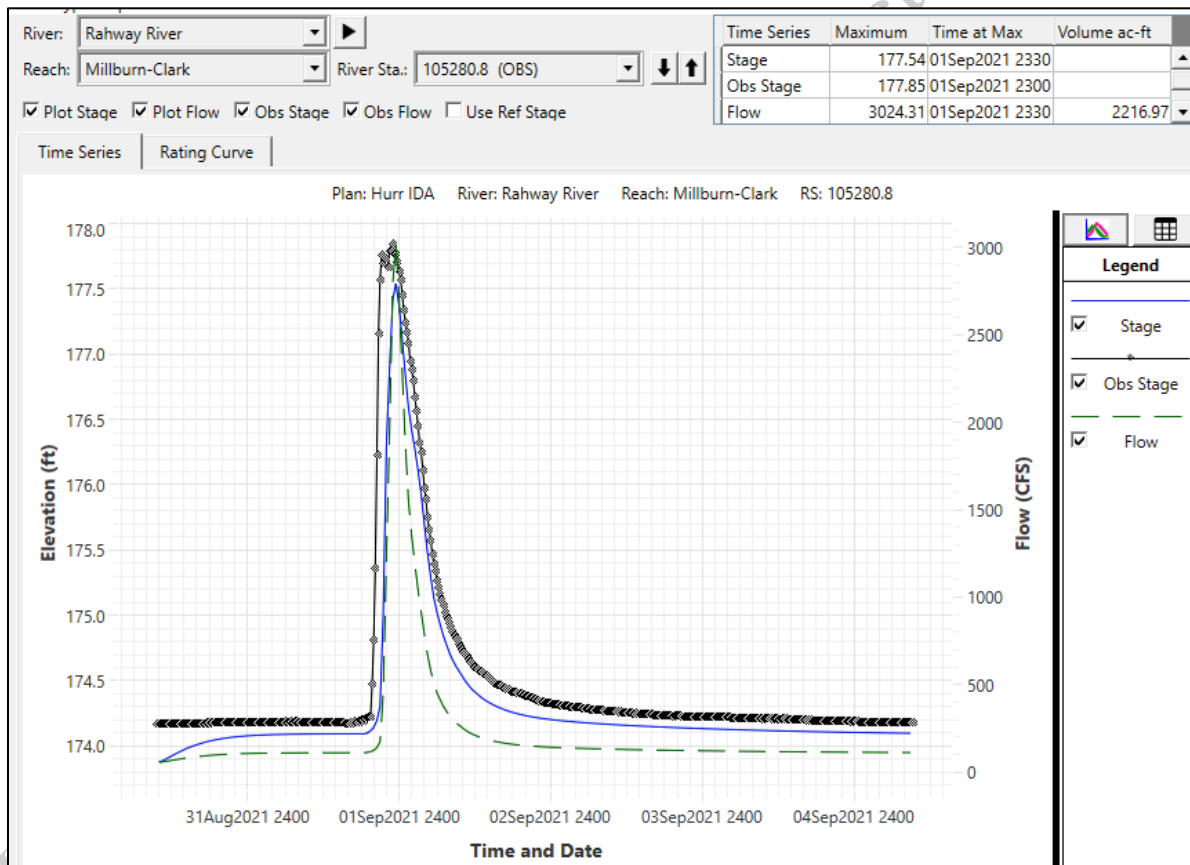


Figure 14. Hydrograph of West Branch at Millburn during Ida



East Branch at Maplewood - Irene

As depicted in Table 6 and Figure 15, the calibrated model produces a good approximation of peak stage and timing compared to the observed data. The peak stage differs by 0.37 ft at this location. The fluctuation in the peak stage is due to the fluctuation in the rainfall. The RAS model utilized a higher base flow to address the instability caused by low flow condition and rapid change in rainfall. This higher base flow flattens the fluctuation in the peak stage.

Table 6. Calibration of peak stage and flow at East Branch at Maplewood during Irene

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	122.55	N/A
Model	122.18	1661

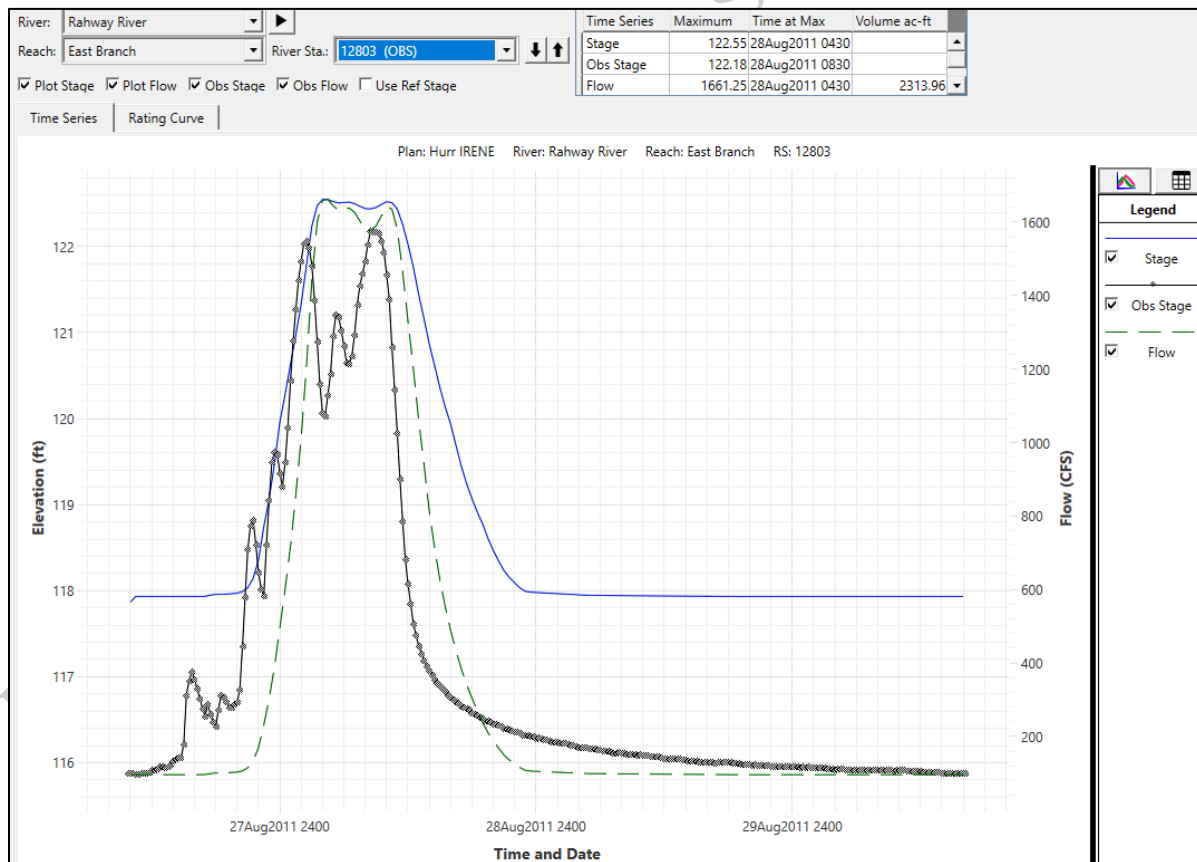


Figure 15. Hydrograph of East Branch at Maplewood during Irene



East Branch at Maplewood - Ida

As depicted in Table 7 and Figure 16, the calibrated peak stage is 123.74 ft compared to the observed stage of 123.4 ft. The calibrated time to peak is 23:30 compared to the observed time to peak of 22:00. The discrepancies of 0.34 ft (less than the agreed upon ± 0.5 ft) in peak stage and 1.5 hours in time to peak are acceptable.

Besides, the RAS model utilized a higher base flow to address the instability caused by low flow condition and rapid change in rainfall.

Table 7. Calibration of peak stage and flow at East Branch at Maplewood during Ida

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	123.74	N/A
Model	123.29	2180

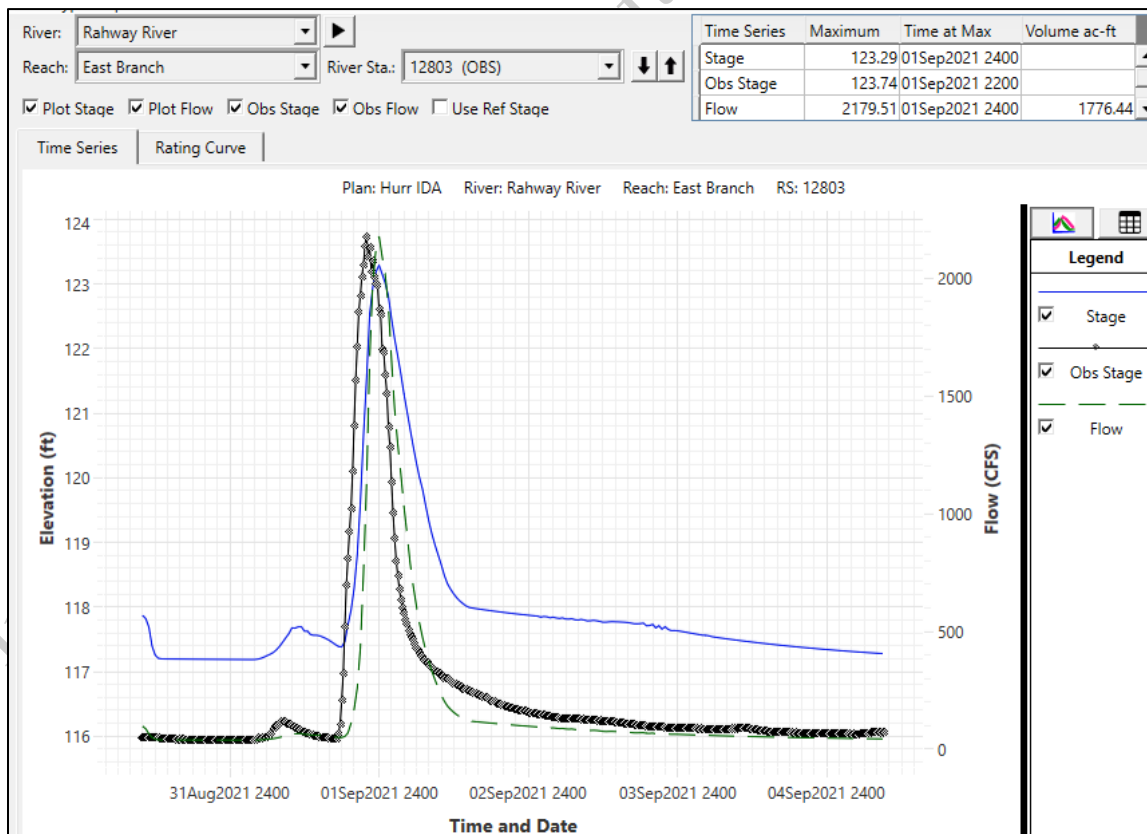


Figure 16. Hydrograph of East Branch at Maplewood during Ida



East Branch at Millburn - Irene

As depicted in Table 8 and Figure 17, the calibrated model produces a good approximation of peak stage and timing compared to the observed data. The peak stage differs by 0.51 ft at this location.

Table 8. Calibration of peak stage and flow at East Branch at Millburn during Irene

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	97.68	N/A
Model	98.17	2478

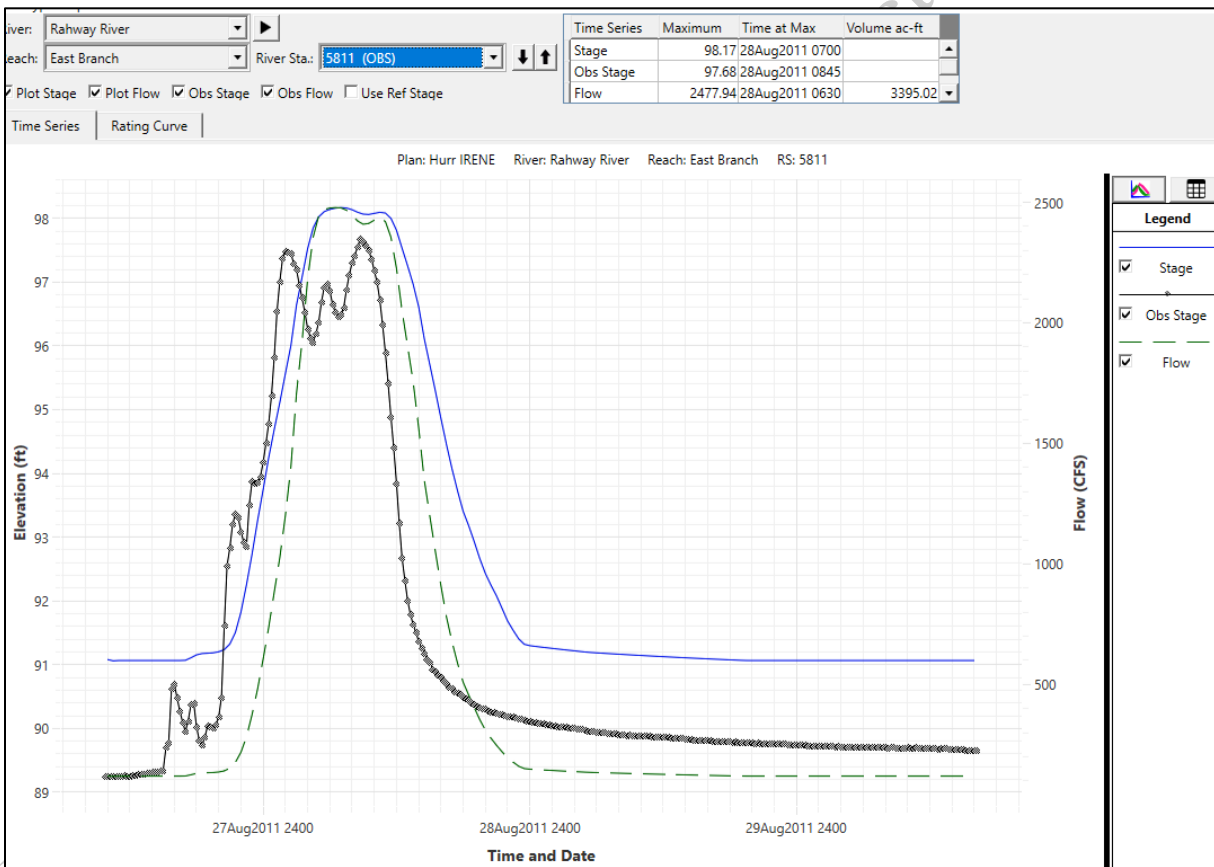


Figure 17. Hydrograph of East Branch at Millburn during Irene



East Branch at Millburn - Ida

As depicted in Table 9 and Figure 18, the calibrated model produces a good approximation of peak stage and timing compared to the observed data. The peak stage differs by 0.01 ft at this location.

Table 9. Calibration of peak stage and flow at East Branch at Millburn during Ida

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	99.23	N/A
Model	99.22	3112

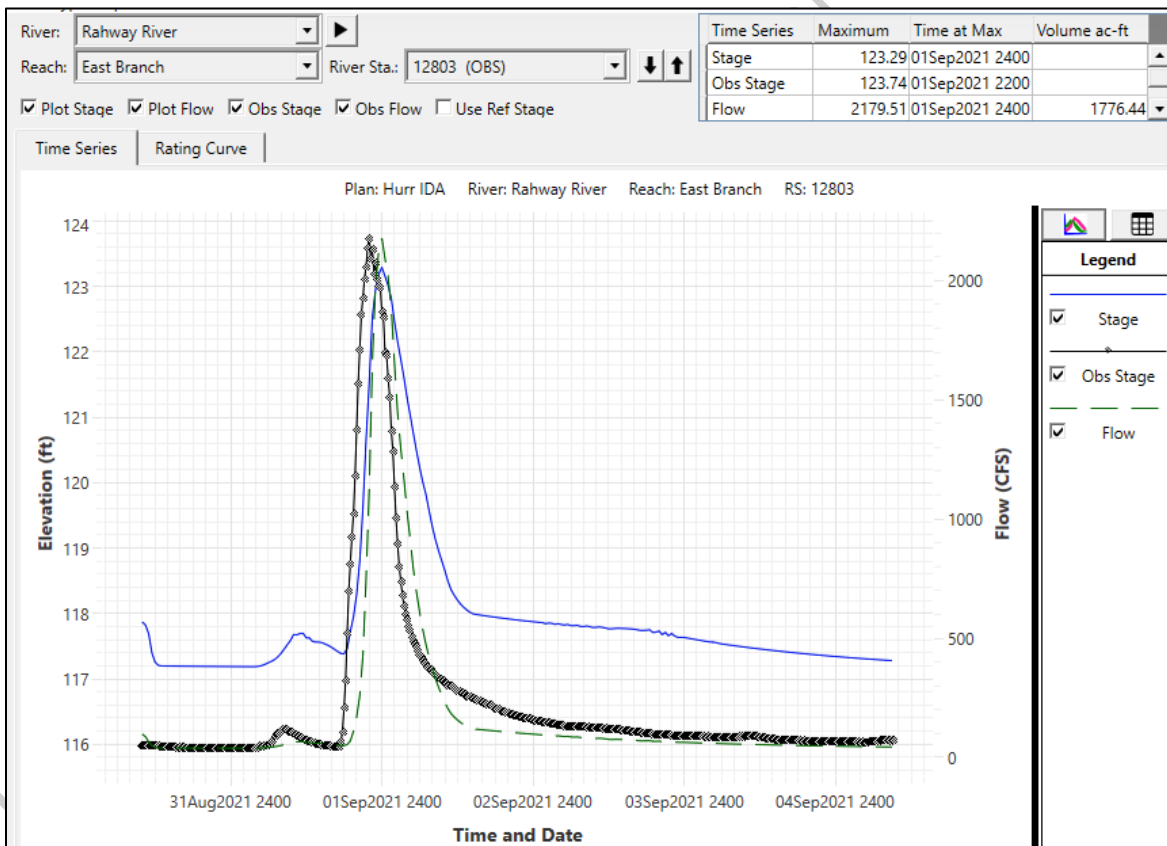


Figure 18. Hydrograph of East Branch at Millburn during Ida



Rahway at Springfield – Irene

As shown in Table 10 and Figure 19, the Springfield gage has a long record length of continuous data, and the calibration of the hydrographs are reasonably good at this location. The peak stage differs by 0.41 ft.

Table 10. Calibration of peak stage and flow at Rahway at Springfield during Irene

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	76.02	8620
Model	76.43	7812

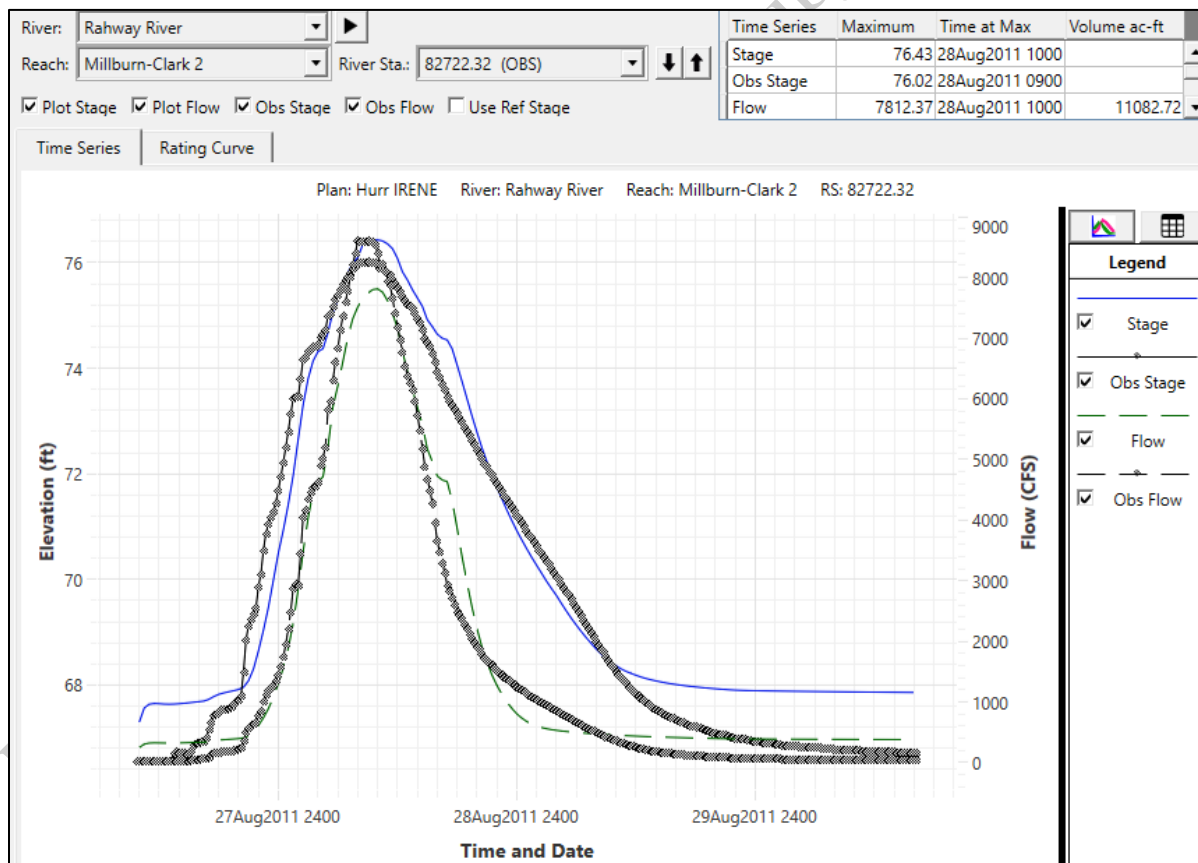


Figure 19. Hydrograph of Rahway at Springfield during Irene



Rahway at Springfield – Ida

As depicted in Table 11 and Figure 20, the calibrated model demonstrates a good approximation of peak stage, discharge, and timing compared to the gage data. The peak stage differs by 0.10 ft.

Table 11. Calibration of peak stage and flow at Rahway at Springfield during Ida

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	75.93	8320
Model	75.83	6970

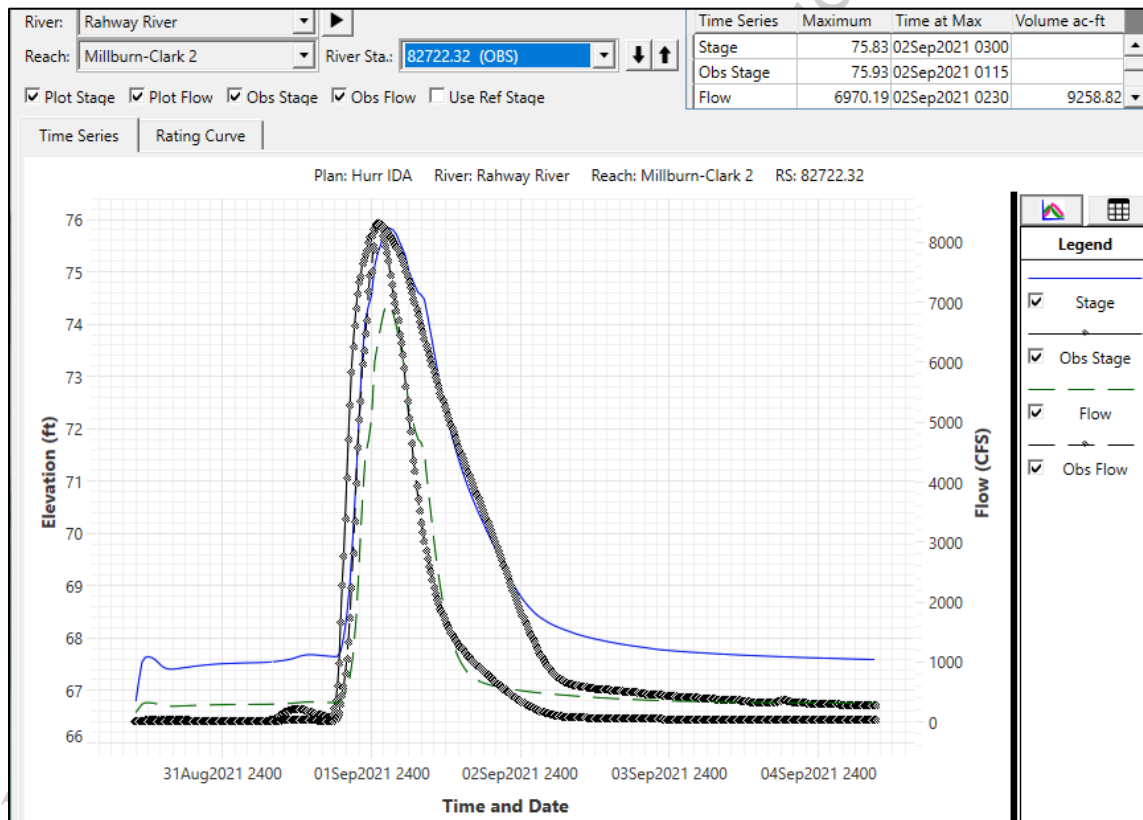


Figure 20. Hydrograph of Rahway at Springfield during Ida



Rahway at Kenilworth – Irene

As shown in Table 12 and Figure 21, the Springfield gage has a long record length of continuous data, and the calibration of the hydrographs are reasonably good at this location. The peak stage differs by 0.37 ft.

Table 12. Calibration of peak stage and flow at Rahway at Kenilworth during Irene

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	70.55	N/A
Model	70.92	9132

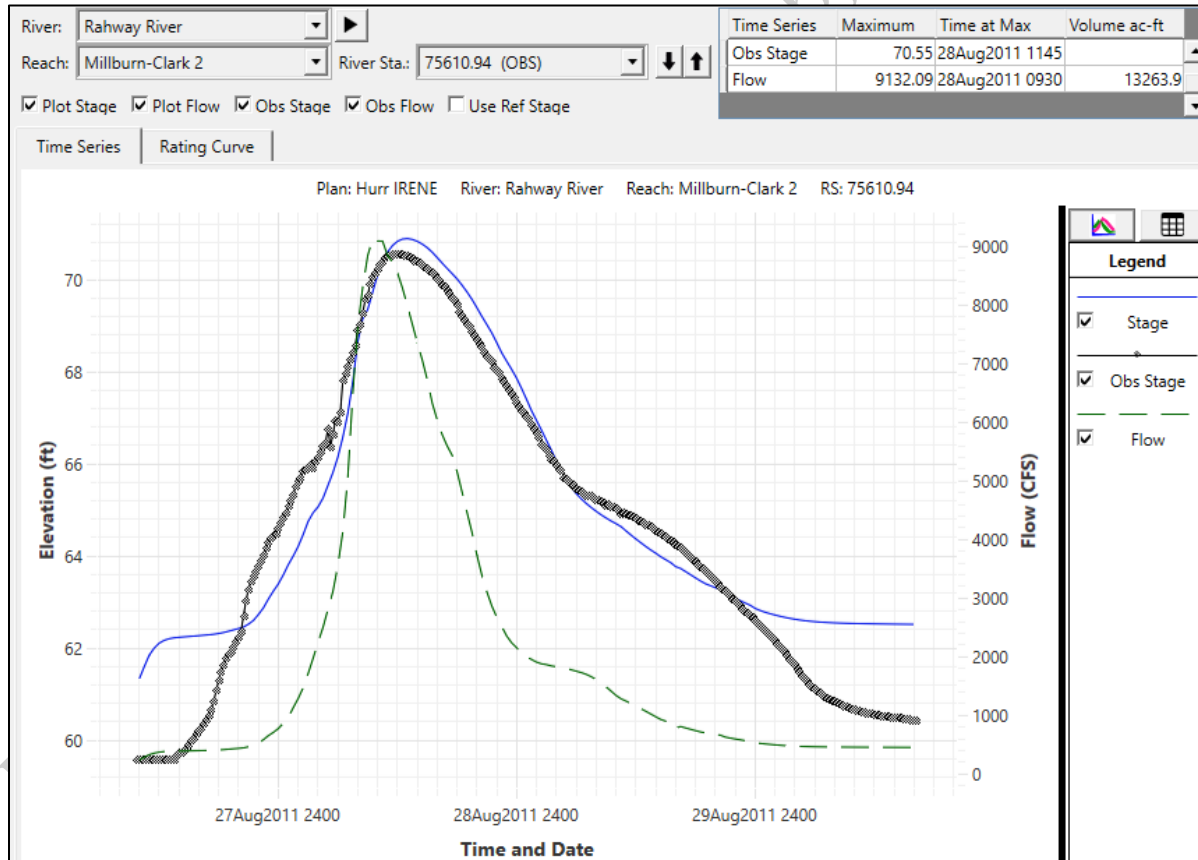


Figure 21. Hydrograph of Rahway at Kenilworth during Irene



Rahway at Kenilworth – Ida

As depicted in Table 13 and Figure 22, the calibrated model demonstrates a good approximation of peak stage, discharge, and timing compared to the gage data. The peak stage differs by 0.18 ft.

Table 13. Calibration of peak stage and flow at Rahway at Kenilworth during Ida

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	69.76	N/A
Model	69.94	7836

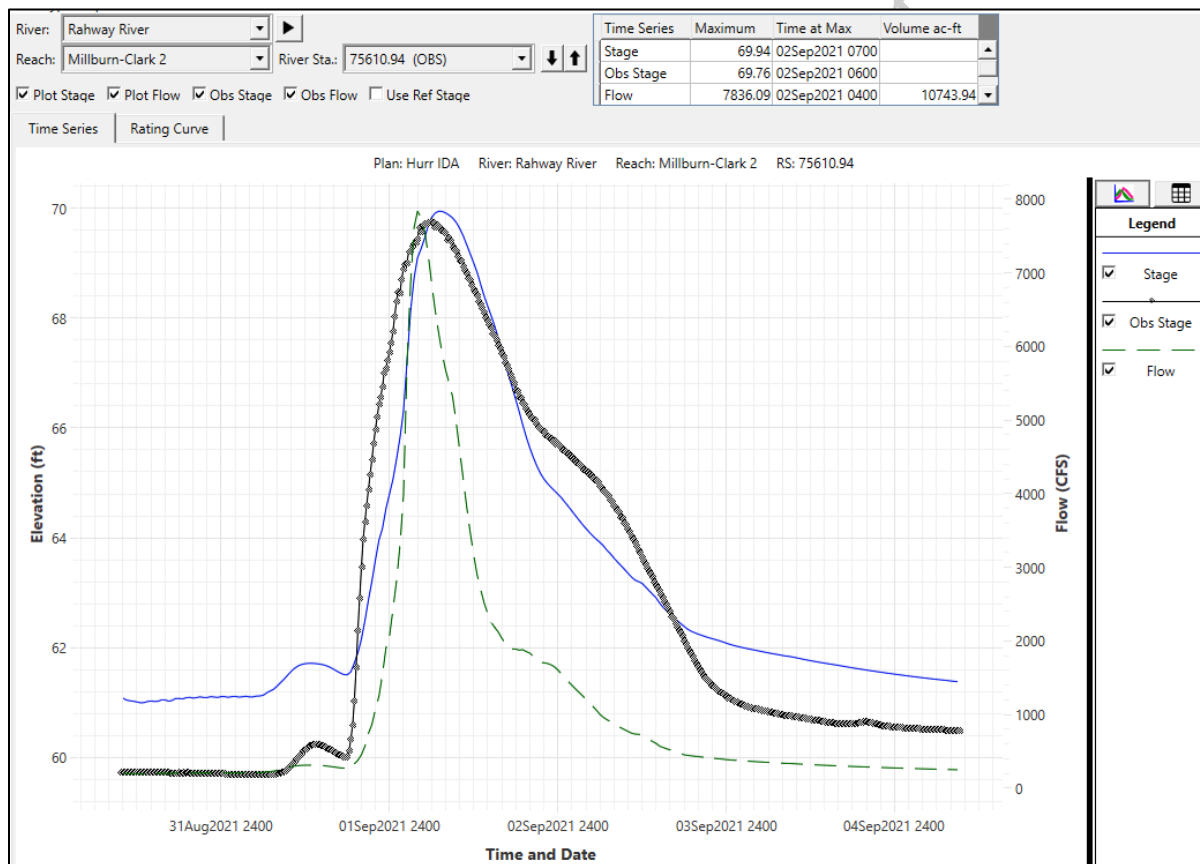


Figure 22. Hydrograph of Rahway at Kenilworth during Ida



Rahway at Rahway – Irene

As shown in Table 14 and Figure 23, the discrepancy in peak stage, time to peak, and peak flow is due to gage failure during Hurricane Irene. The observed peak data (in red circle) is estimated value only.

Table 14. Calibration of peak stage and flow at Rahway at Rahway during Irene

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	19.81	N/A
Model	19.78	9550

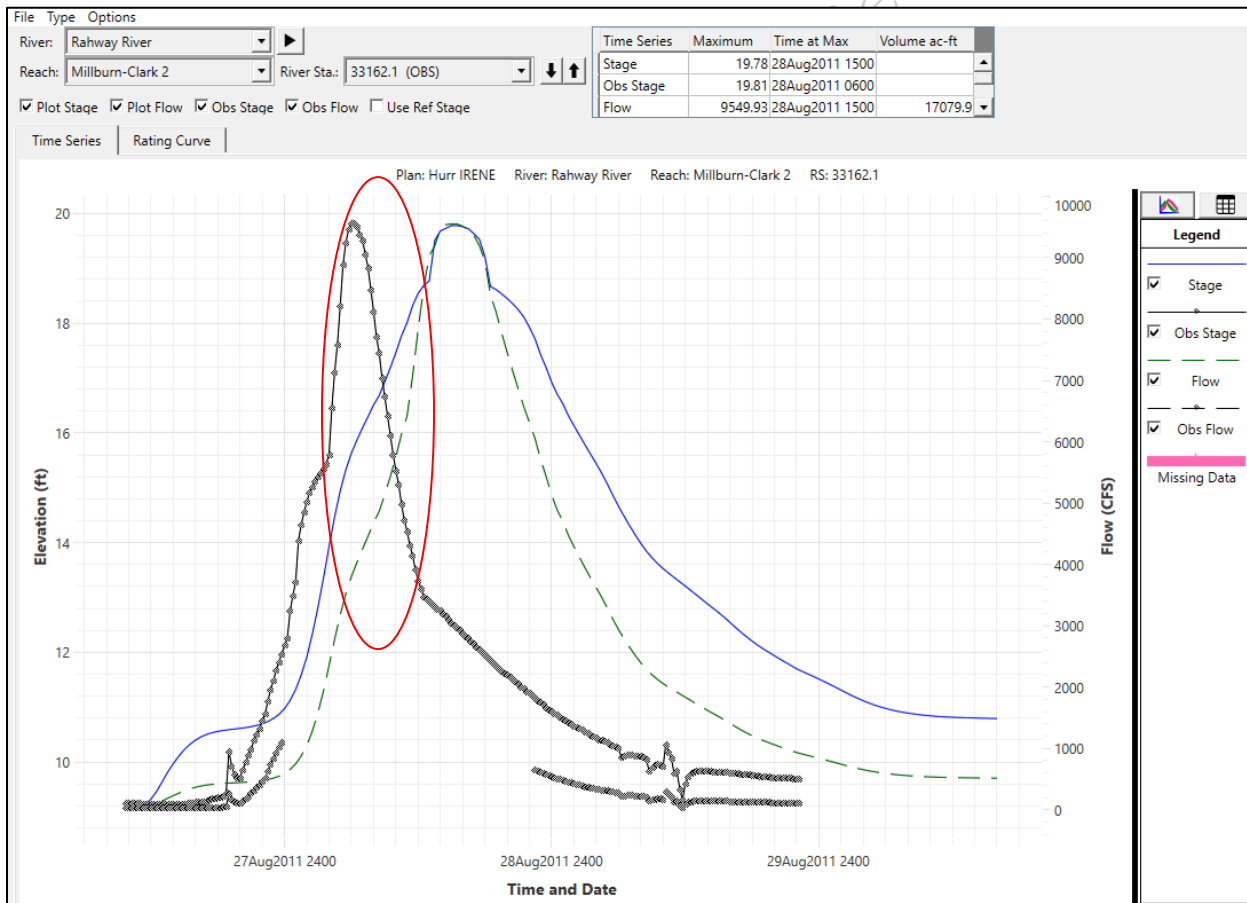


Figure 23. Hydrograph of Rahway at Rahway during Irene



Rahway at Rahway – Ida

As depicted in Table 15 and Figure 24, the discrepancy in peak stage, time to peak, and flow is due to gage failure during Hurricane Ida. Therefore, the observed peak data is estimated value only.

Table 15. Calibration of peak stage and flow at Rahway at Rahway during Ida

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	19.19	N/A
Model	18.36	7172

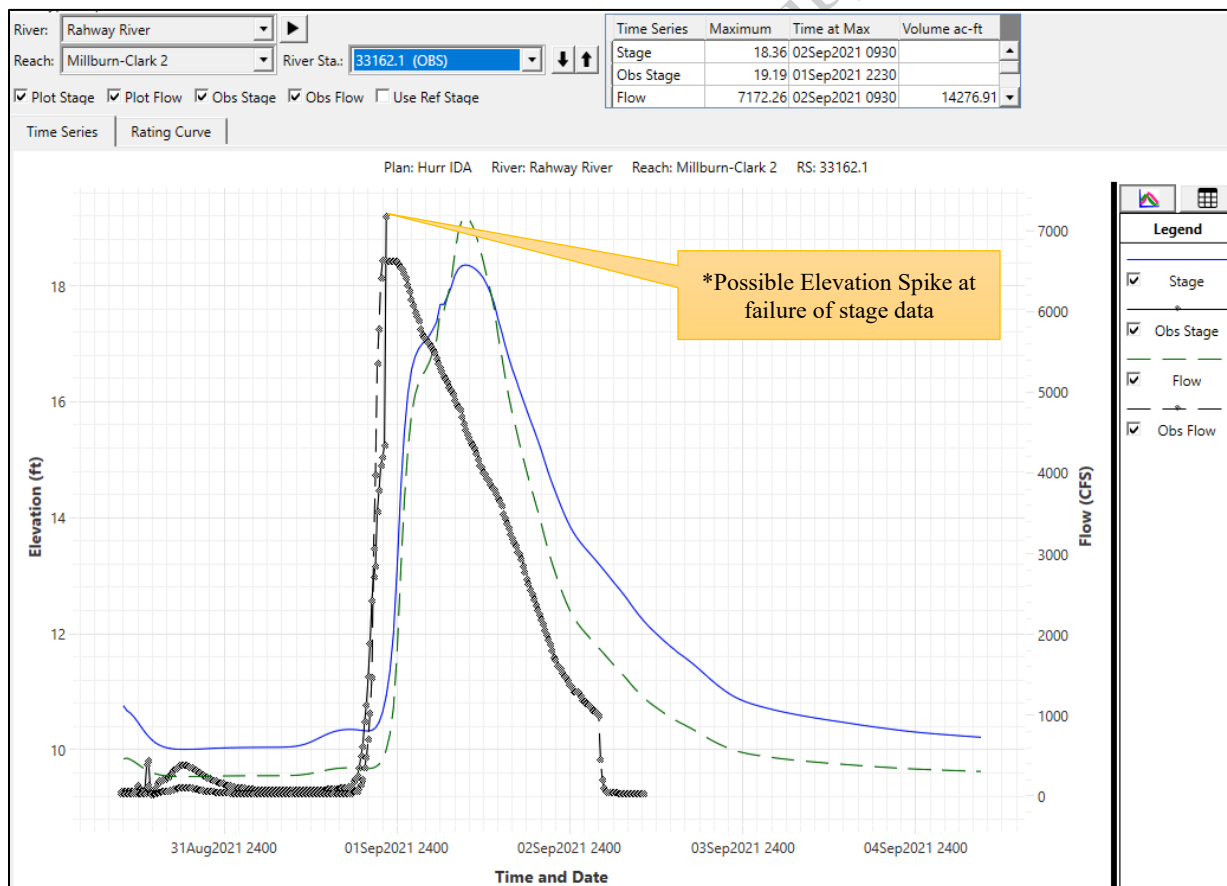


Figure 24. Hydrograph of Rahway at Rahway during Ida



Robinsons Branch – Irene

As shown in Table 16 and Figure 25, the Springfield gage has a long record length of continuous data, and the calibration of the hydrographs are reasonably good at this location. The peak stage differs by 0.09 ft.

Table 16. Calibration of peak stage and flow at Robinsons Branch during Irene

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	25.50	N/A
Model	25.41	4705

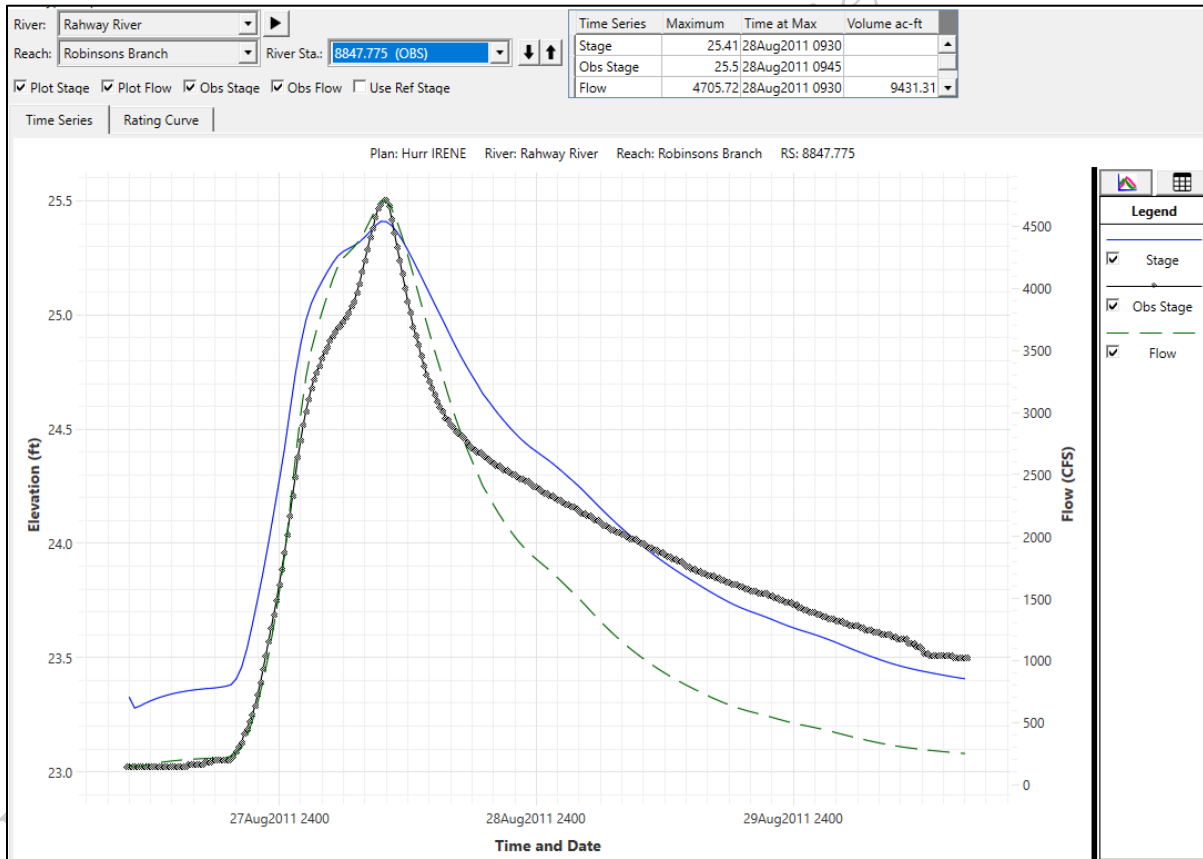


Figure 25. Hydrograph of Robinsons Branch during Irene



Robinsons Branch – Ida

As depicted in Table 17 and Figure 26, the calibrated model demonstrates a good approximation of peak stage, discharge, and timing compared to the gage data. The peak stage differs by 0.83 ft.

Table 17. Calibration of peak stage and flow at Robinsons Branch during Ida

Data	Peak Stage (ft NAVD88)	Peak Flow (cfs)
Observed	26.25	N/A
Model	26.40	7852

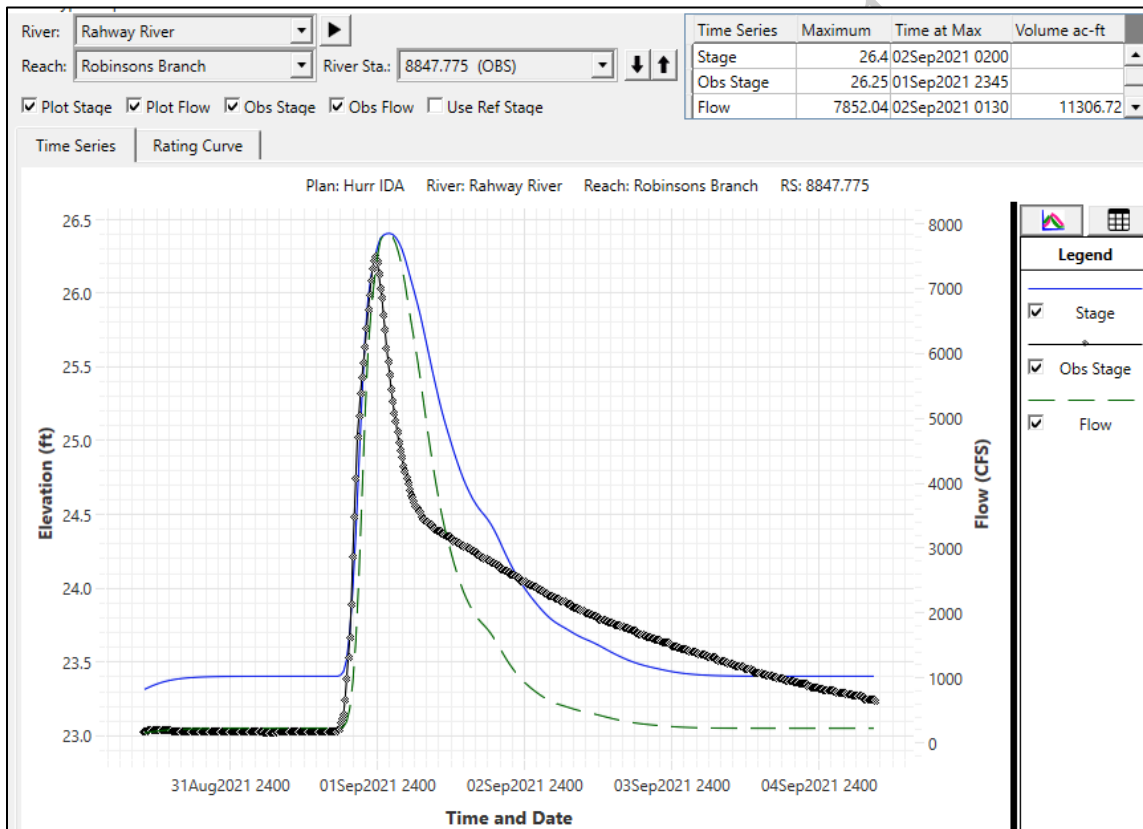


Figure 26. Hydrograph of Robinsons Branch during Ida



Historic Calibration Summary Table

Table 18. HEC-RAS calibration for Tropical Storm Irene

Gage Name	Gage Number	RAS Computed WSE (ft., NAVD88)	Observed WSE (ft., NAVD88)	Difference (ft.)	RAS Computed Peak Flow (cfs)	HMS Peak Flow (cfs)	Observed Peak Flow (cfs)
WB at Millburn	1394000	177.43	177.51	-0.08	2832	2620	
EB at Maplewood	1393890	122.55	122.18	0.37	1661	1455	
EB at Millburn	1393895	98.17	97.68	0.49	2478	2442	
Rwy. at Springfield	1394500	76.30	76.02	0.28	7791	7968	8620
Rwy. at Kenilworth	1394620	70.91	70.55	0.36	9049	5917	
Rwy. at Rahway	1395000	19.78	19.81	-0.03	9550	8027	7250
Robinsons Br	1396000	25.50	25.5	0.00	4706	5600	5600

Table 19. HEC-RAS calibration for Hurricane Ida

Gage Name	Gage Number	RAS Computed WSE (ft., NAVD88)	Observed WSE (ft., NAVD88)	Difference (ft.)	RAS Computed Peak Flow (cfs)	HMS Peak Flow (cfs)	Observed Peak Flow (cfs)
WB at Millburn	1394000	177.54	177.85	-0.31	3025	2725	
EB at Maplewood	1393890	123.40	123.74	-0.34	2268	2052	
EB at Millburn	1393895	99.18	99.23	-0.05	3111	2846	
Rwy. at Springfield	1394500	75.84	75.93	-0.09	6952	8274	8320
Rwy. at Kenilworth	1394620	69.94	69.76	0.18	7836	4710	
Rwy. at Rahway	1395000	18.36	*19.19	-0.83	7172	6475	6630
Robinsons Br	1396000	25.75	26.25	-0.50	5646	8728	

*Gage failed – peak flow estimated



USGS Surveyed Floodmarks

As depicted in Figure 27 and Figure 28, the calibrated model demonstrates a good match between the calculated peak stage and the USGS surveyed floodmark near the Kenilworth Gage. The peak stage differs by up to 0.2 ft at the floodmarks.



Figure 27. Comparison of peak stage to USGS surveyed floodmark during Ida at the downstream of Kenilworth gage



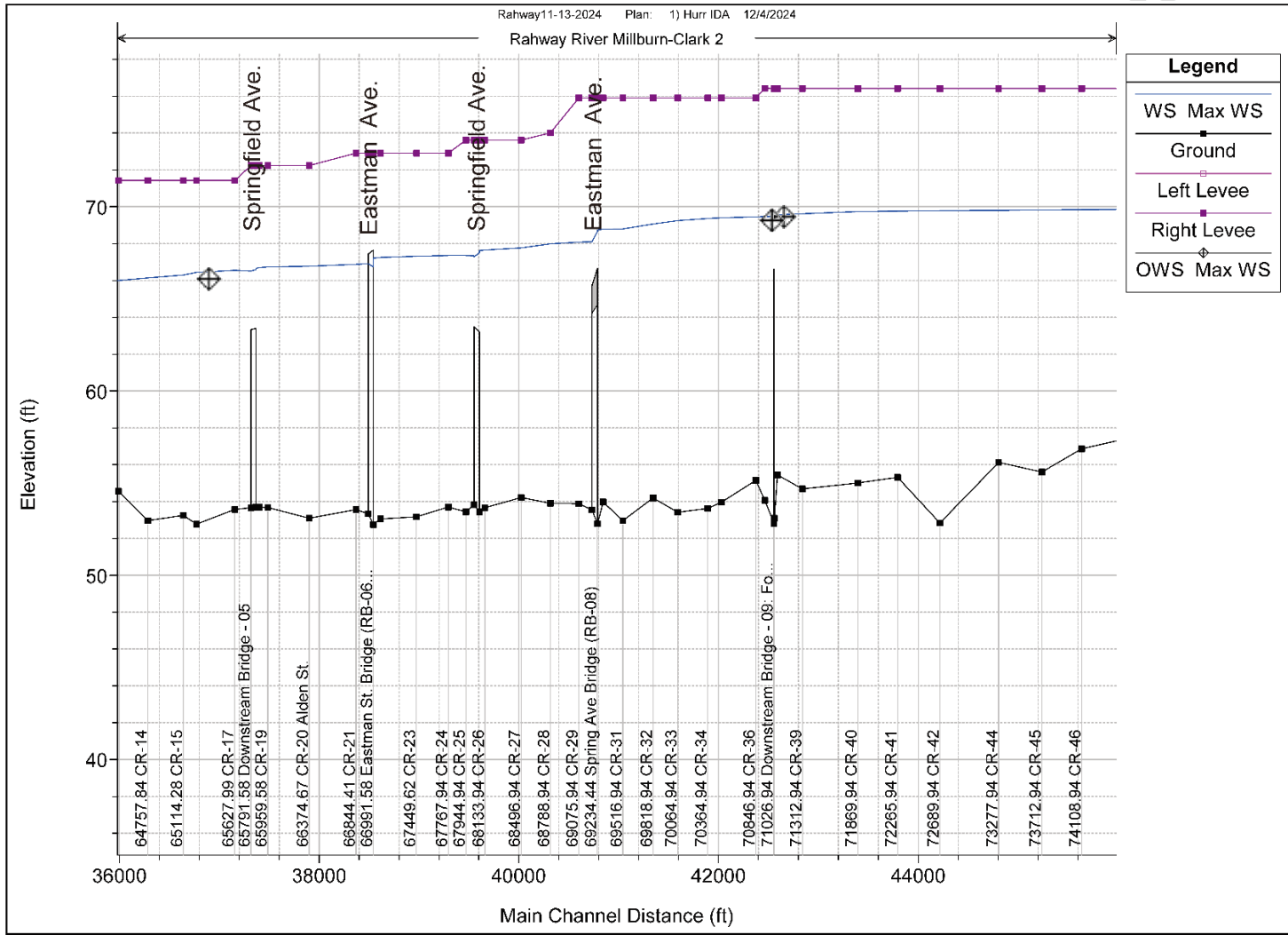


Figure 28. Computed water surface profile and observed HWMs for Hurricane Ida in the Rahway River in Cranford Township



As depicted in Figure 29 and Figure 30, the calibrated model varied in peak stage compared to the USGS surveyed floodmarks on Robyns Branch.

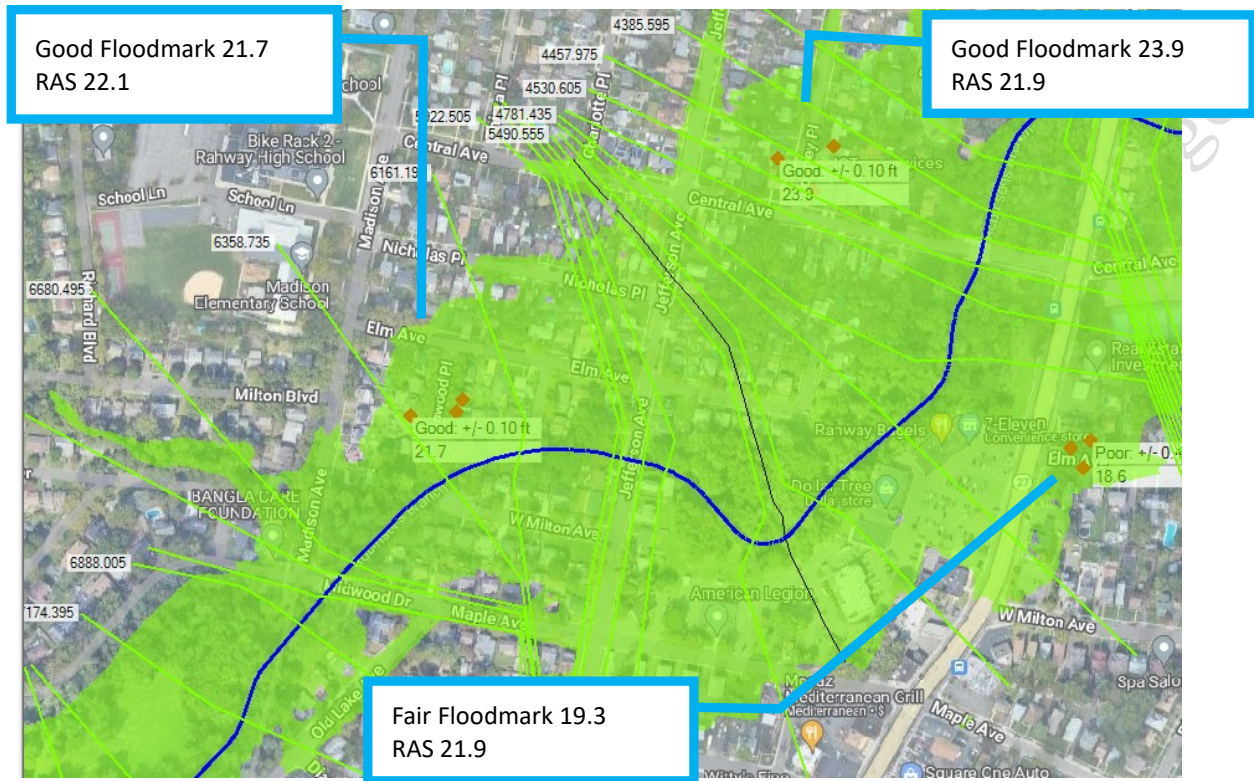


Figure 29. Comparison of peak stage to selected USGS surveyed floodmark during Ida at the Robyns gage



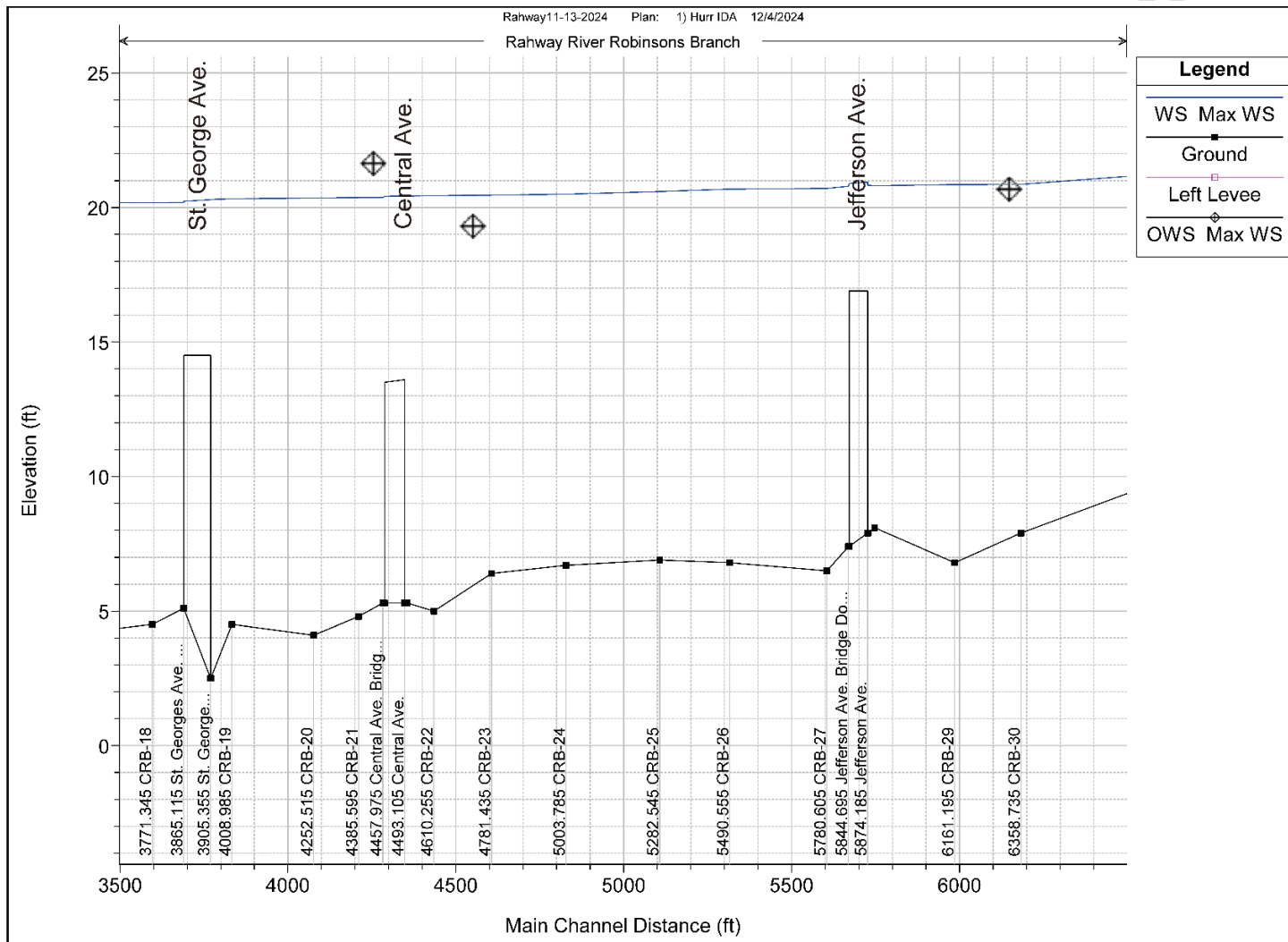


Figure 30. Computed water surface profile and observed HWMs for Hurricane Ida in Robinsons Branch



Historic Hydraulic Calibration Summary

The HEC-RAS model was calibrated to both Irene and Ida with a single geometry using adjustments to storage area stage volume curves and channel manning's n values. The calculated elevations are well within the desired 0.5-foot deviation from gage data for the historic storm events. The calibration was somewhat iterative between the historic calibration and the hypothetical calibration described in the text of the next section.



3.3 Boundary Conditions and Fluvial-Coastal Joint Probability

3.3.1 Boundary Conditions

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 fluvial-tidal correlation to establish the backwater elevations that may occur due to tide and surge during a typical fluvial event.

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 fluvial events greater than a 0.99 AEP (1-yr) and their corresponding maximum tidal stage were used in the correlation analysis. 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. Hurricane Ida was a significant fluvial event but did not coincide with a major tidal event. Therefore, the results of the correlation analysis remain the same. Figure 31 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 32 shows the frequency of significant fluvial events plotted with the frequency of the associated maximum coastal stage.

Based on the fluvial-coastal assessment in the 2016 fluvial study, it was determined that dominant fluvial storms are historically associated with high frequency fluvial events (low tides). Stream



flows associated for each fluvial frequency event were assigned a coincidental tide, which became the upstream and downstream boundary conditions, respectively.

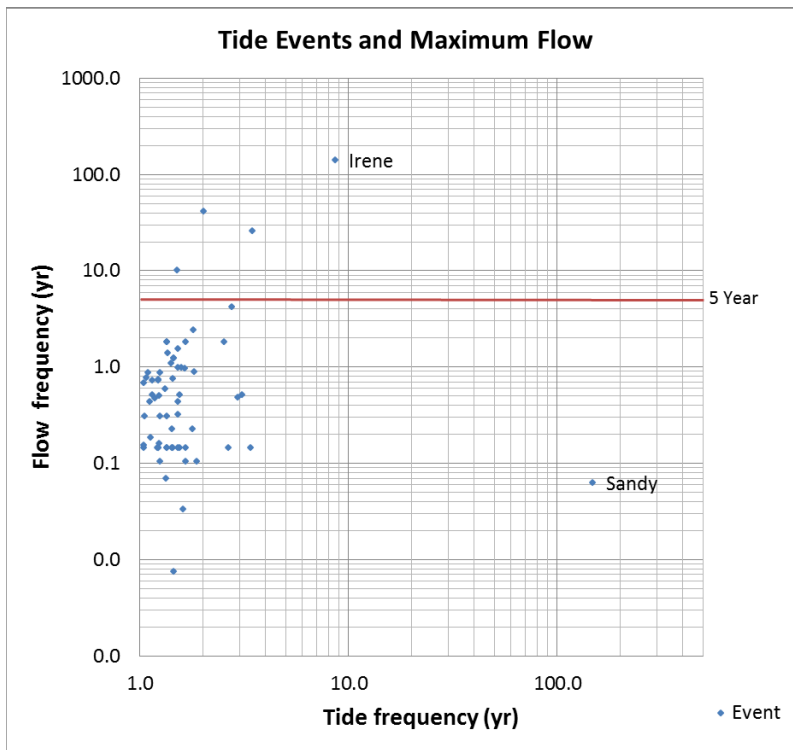


Figure 31. Coastal event and the maximum flow during the event



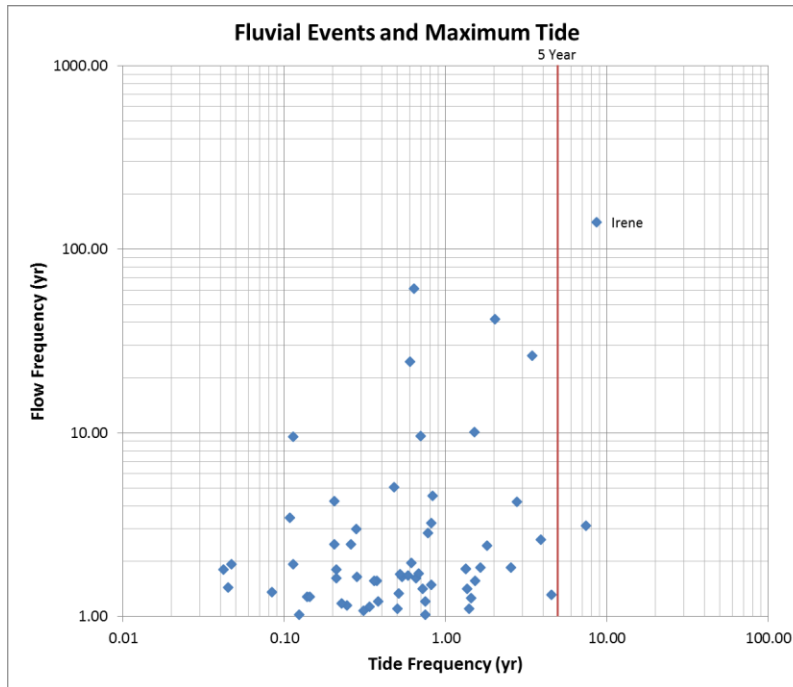


Figure 32. Fluvial event and the maximum coastal stage during the event

3.3.2 Coastal Stage Hydrographs and Downstream Boundary Condition

This study used the same downstream boundary conditions as the 2020 Feasibility Study. This section elaborates on the process of how the downstream boundary conditions were obtained.

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 Table 20 and Table 21. The coastal stage-frequency data from the 2013 FEMA Region II Storm Surge Project was included for reference purposes and, as depicted in Figure 33, 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 20) 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 21. The local tide has no effect on the final maximum stages at the mouth on the river, as the astronomical stages are lower in elevation than all NACCS AEP events. The USACE Survey Section at Caven Point, New Jersey provided the standard conversion at this gage which is MLLW at -2.95 ft NAVD88.

The duration of each hypothetical storm had previously been obtained for the Port Monmouth CSRSM study and it was reused for this study. In relation to the CSRSM 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 34 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 35 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 20. NACCS stage-frequency for Point ID 11659 for Year 1992 Epoch 1983-2001 Midpoint

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 21. Bergen Point Gage tide datum

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



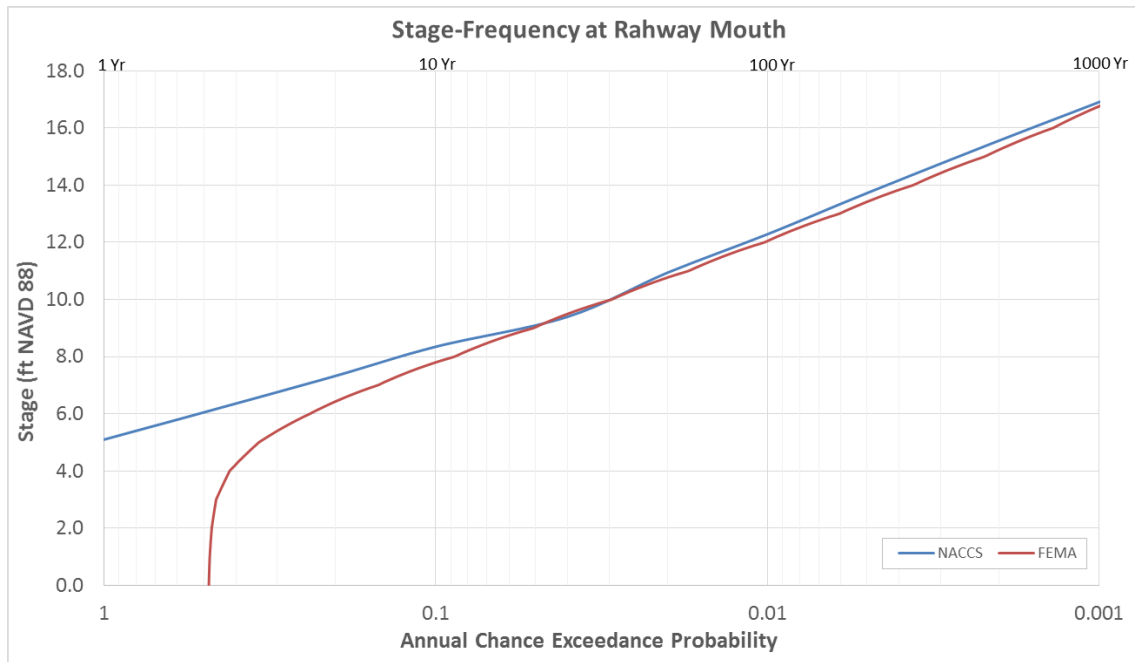


Figure 33. Stage-frequency curve at Rahway Mouth from NACCS and FEMA. *FEMA curve at Carteret (ID: 543829) approx. 0.5 mi. downstream of NACCS point on the Arthur Kill

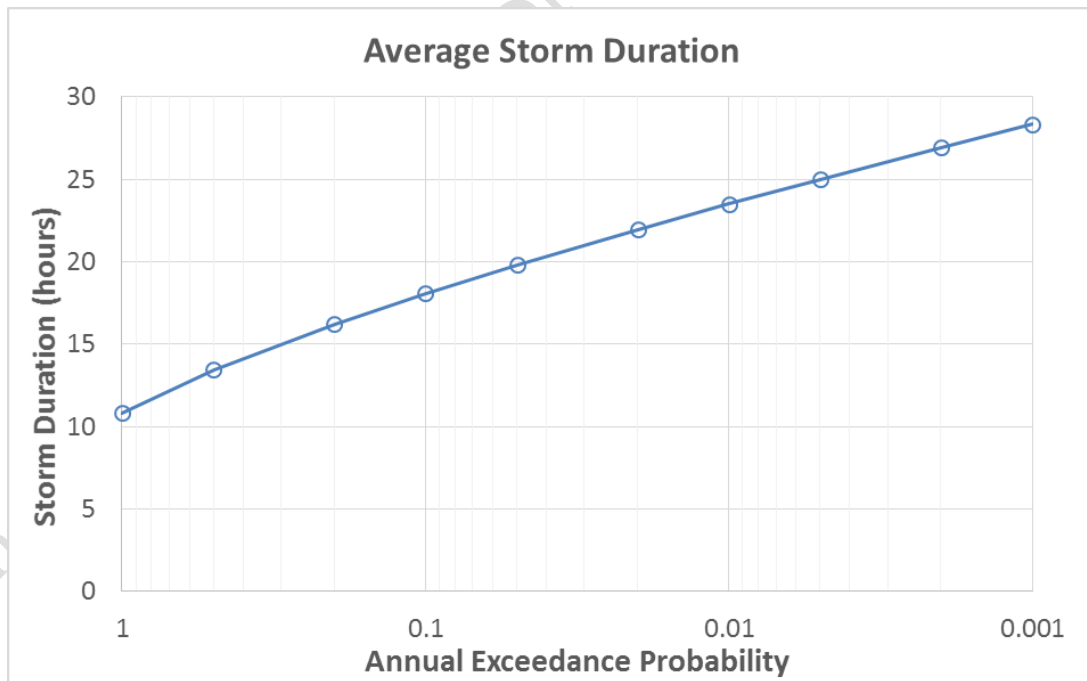


Figure 34. Storm duration curve from the NACCS study for Port Monmouth, NJ



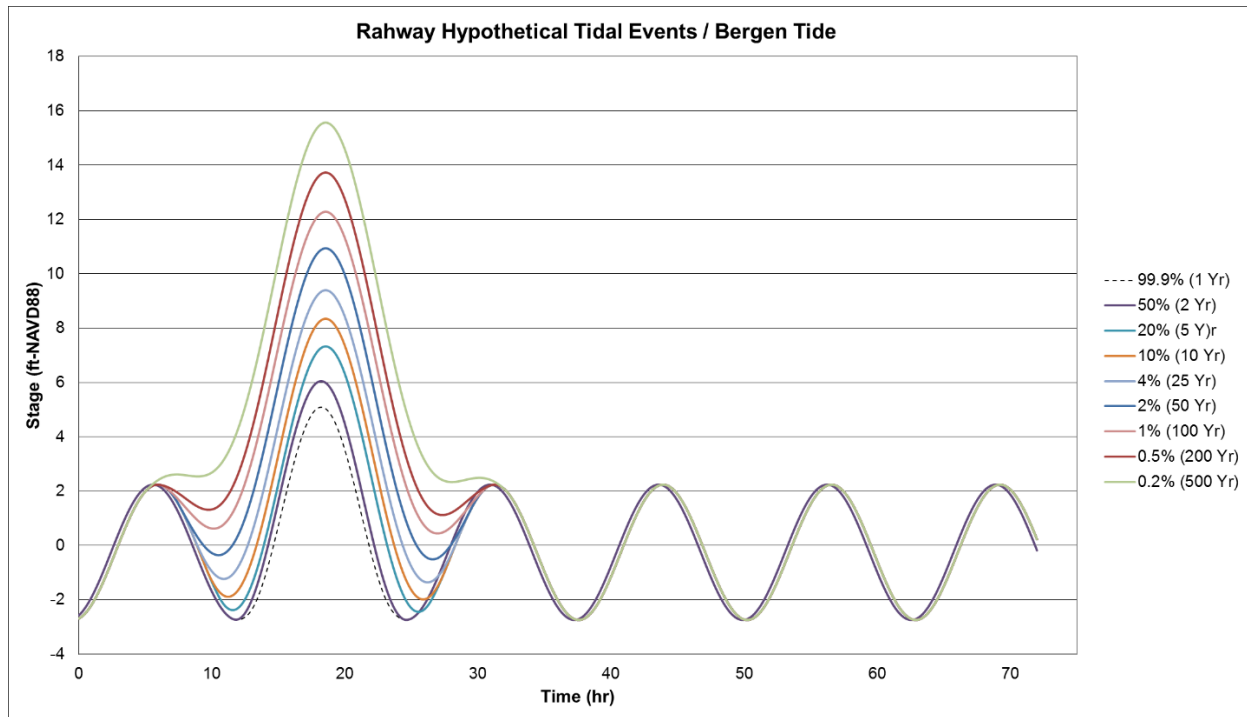


Figure 35. Stage hydrograph for hypothetical coastal events at the mouth of the Rahway River

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 22 to see the different plans that were run through the hydraulic model. Both the present (Year 2023) and future conditions (Year 2073) were analyzed. The intermediate coastal



scenarios was 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. Table 23 summarizes the cross-sections where a coincident frequency analysis was performed. Refer to Figure 36 through Figure 38 for joint probability analysis results at different locations of the Rahway River. The graphs compare the Rahway coastal stages obtained from NACSS versus the Rahway coastal stages with Rahway fluvial influence. Figure 39 depicts the computed water surface profile for the 1% AEP (100-yr) event in compound flood zone. The graph shows water surface profile 1% AEP (100-yr) event under tidal impact only, fluvial impact only, fluvial-coastal impact, respectively.

The joint probabilities account for the fact that: 1) the upstream 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, the lower portion of Robinson's Branch, and South Branch have risk from both coastal and fluvial flooding, and 3) Carteret and Linden are mainly flooded by coastal events. Figure 40 depicts the boundary of the compound flood zone.

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 22. 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)



Model Plan #	Rahway Fluvial Storm Event	Rahway Coastal Storm 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)
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)



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Model Plan #	Rahway Fluvial Storm Event	Rahway Coastal Storm 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)
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)



Table 23. Cross-sections with coincident frequency analysis performed

Reach	Cross-Section
Rahway River Main Stem	25698.8
	27107.37
	27559.02
	27995.02
	28472.74
	29222
	30053
	30504
	30583.5
	31664
	32915.62
	33034
Robinsons Branch	175.4458
	564.14
	649.82
	721.8958
	777.8658
	880.7058
	962.5258
	1725.635
	2535.375
	3334.9
South Branch	210.7962
	3557.687
	6770.787
	7797.398
	11400.9



Joint Probability near Rahway and South Branch Confluence (XS 22885)

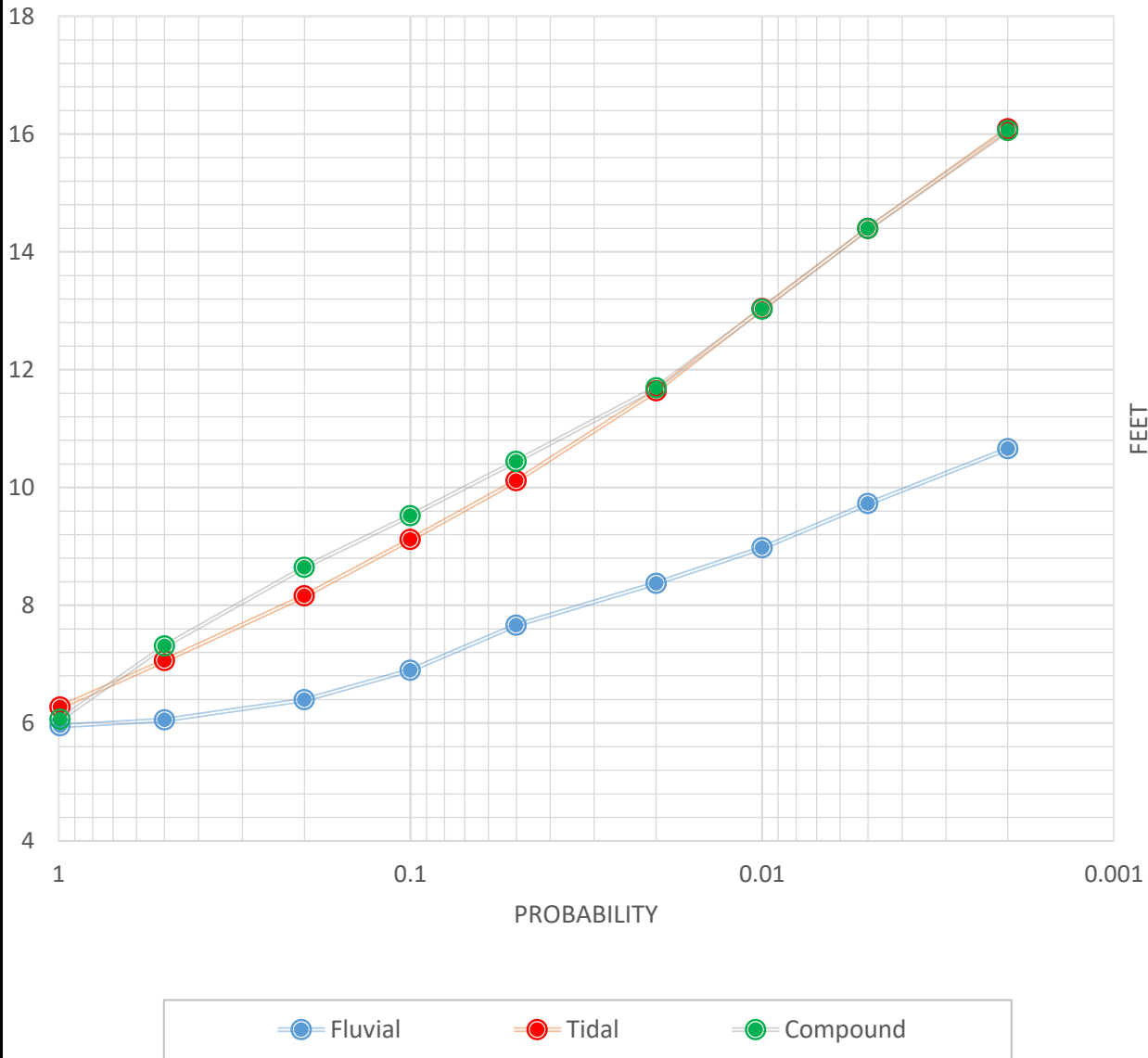


Figure 36. Joint probability at Rahway at Rahway Gage



Joint Probability at Rahway and Robinson's Branch Confluence (XS 27995)

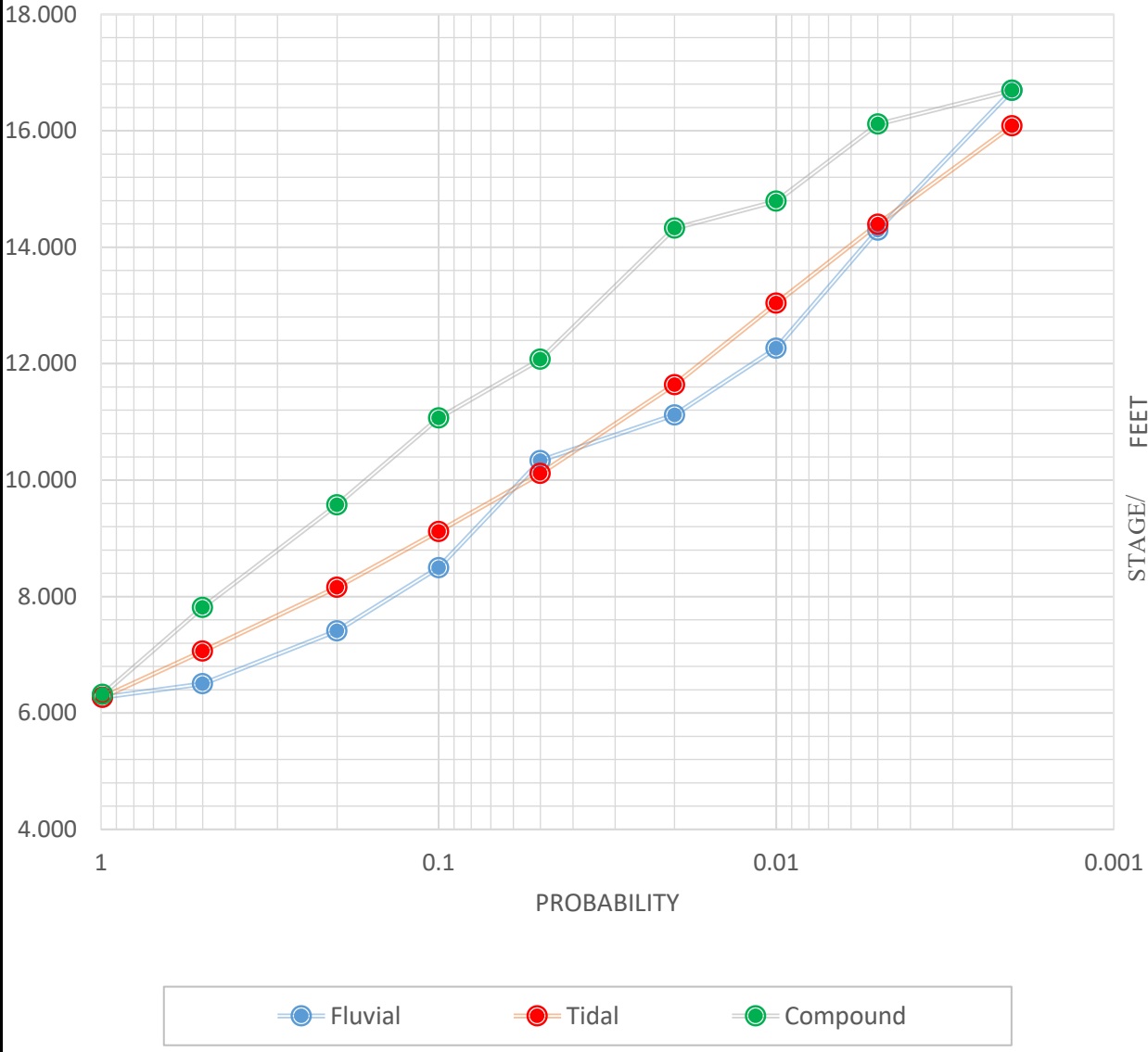


Figure 37. Joint probability at cross section 27995 - Rahway and Robinson's Branch Confluence



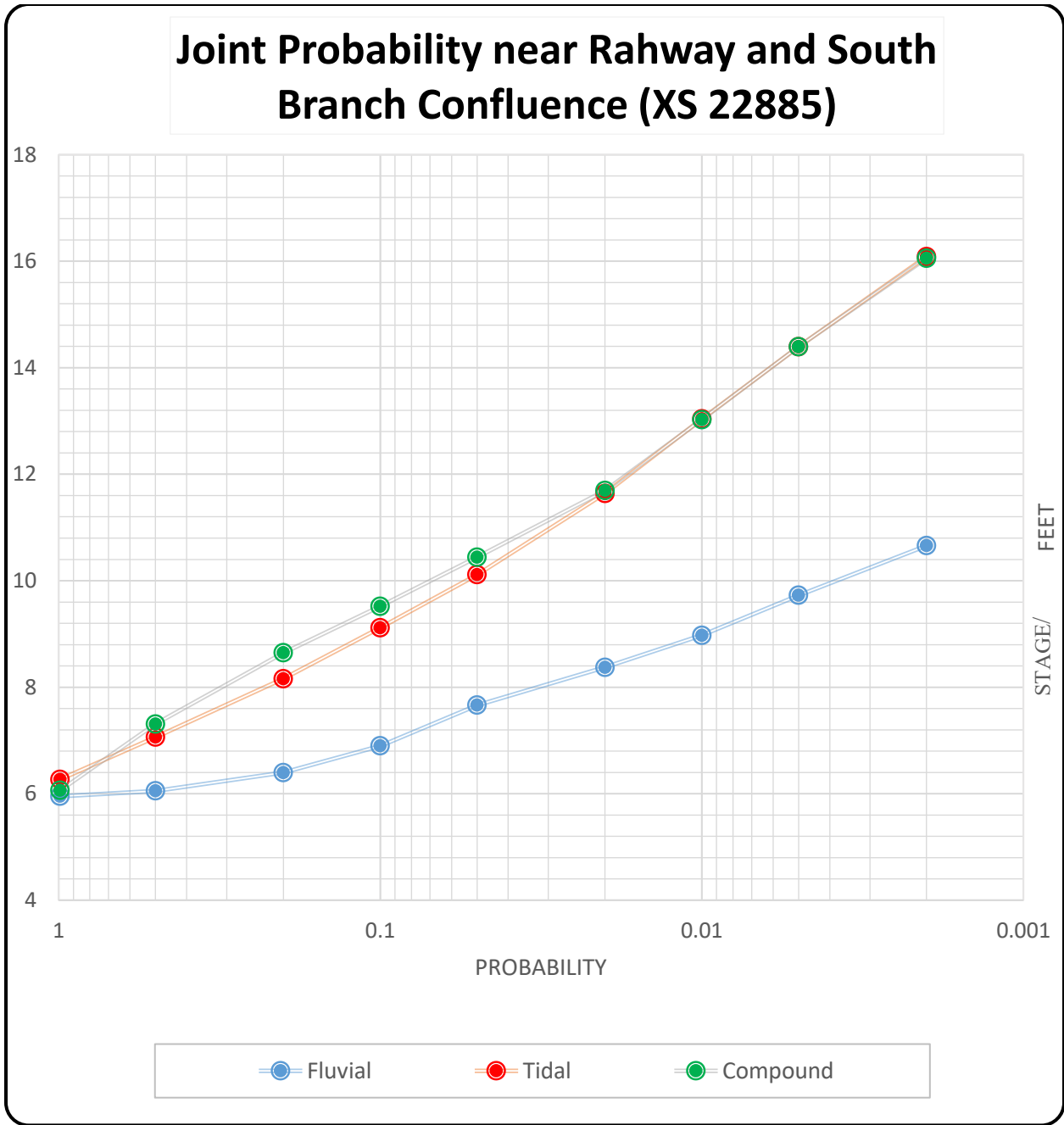


Figure 38. Joint probability in the Rahway River downstream of the confluence of South Branch and the Rahway River- cross section 22885



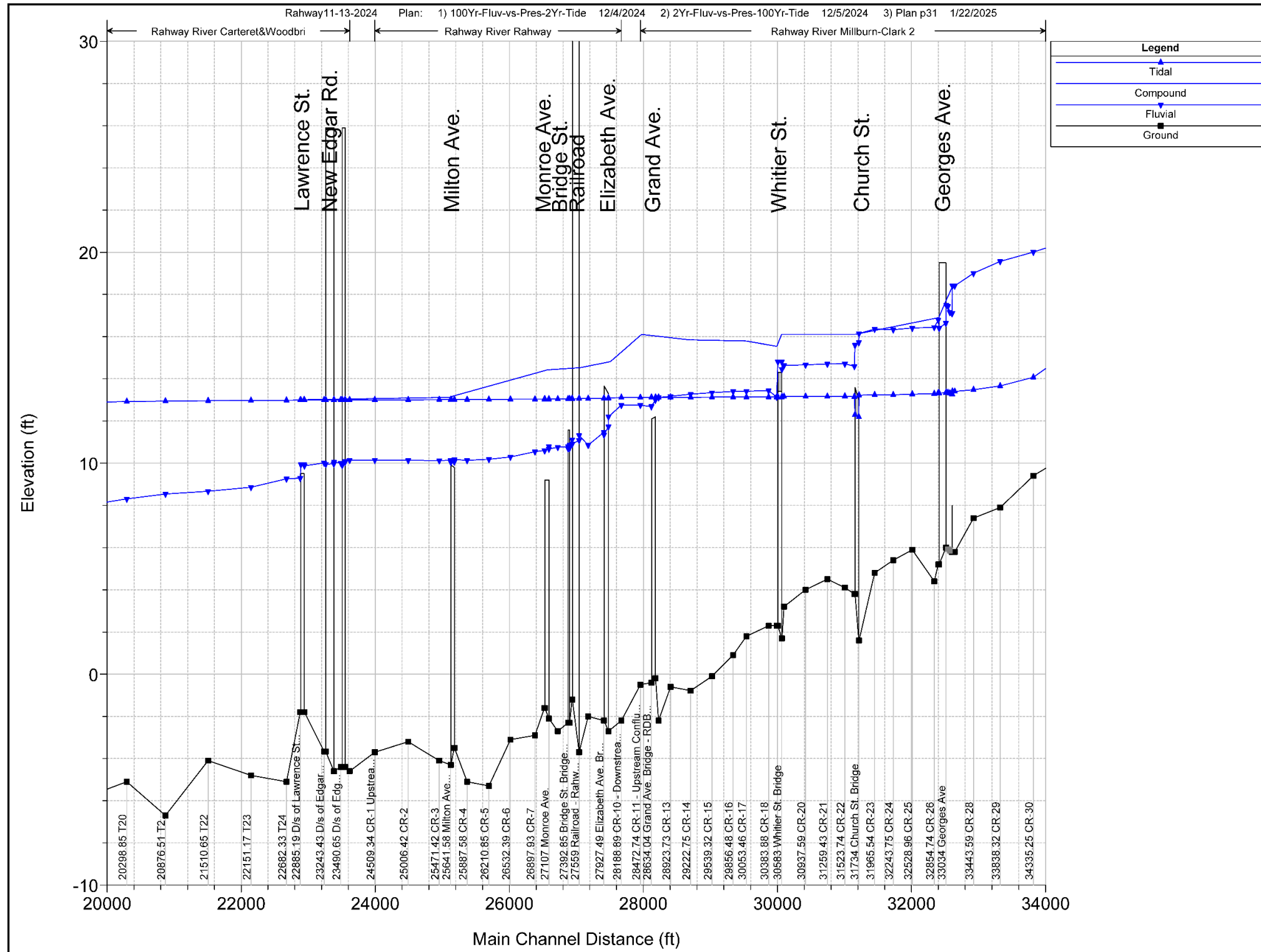


Figure 39. Without project condition computed water surface profile for the 1% AEP (100-yr) event in compound flood zone



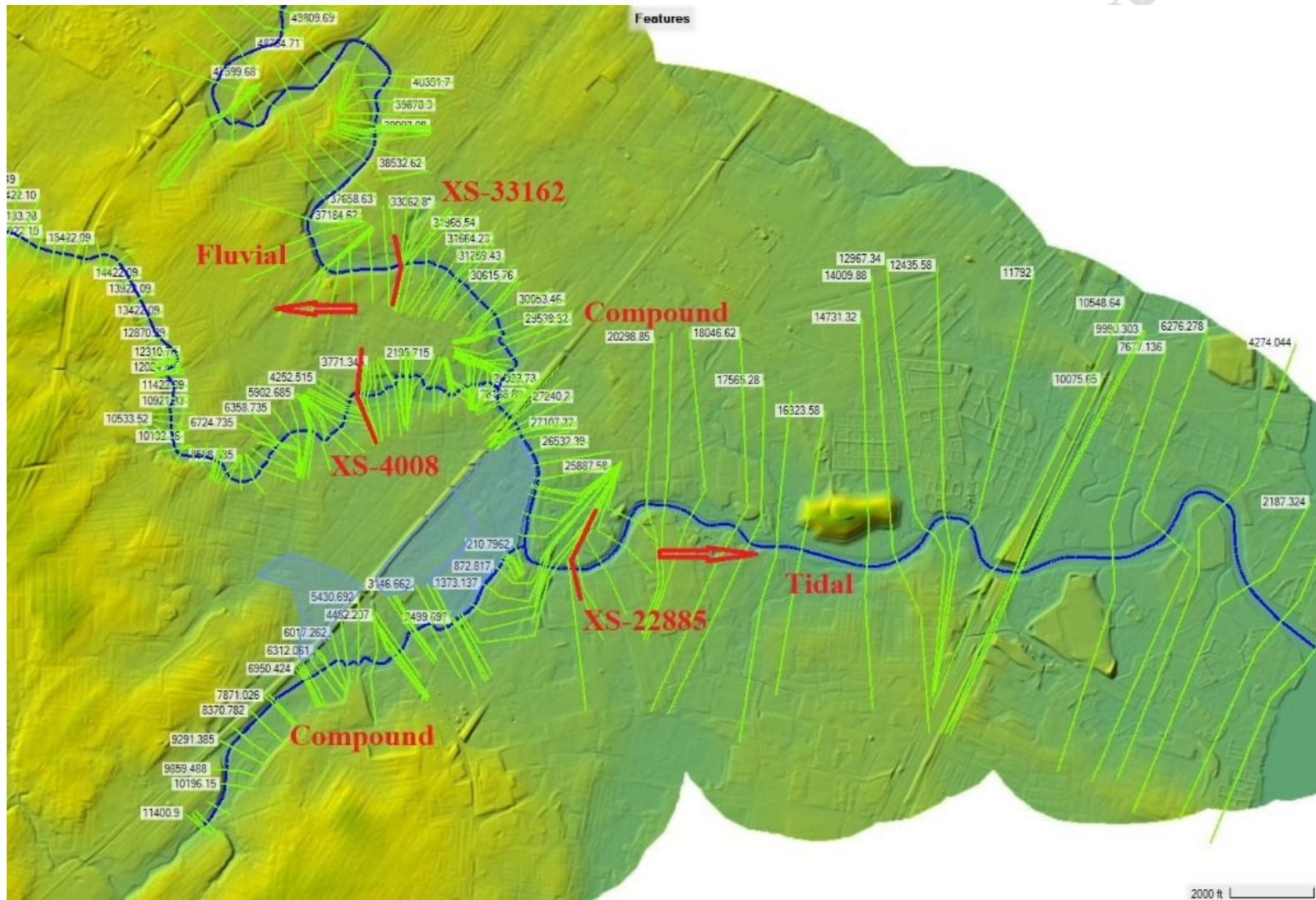


Figure 40. Boundary of compound flood zone



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 41 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) were also considered as a component of sea-level rise. The projected low, intermediate and high SLC scenarios are shown in Table 24 and Figure 42.

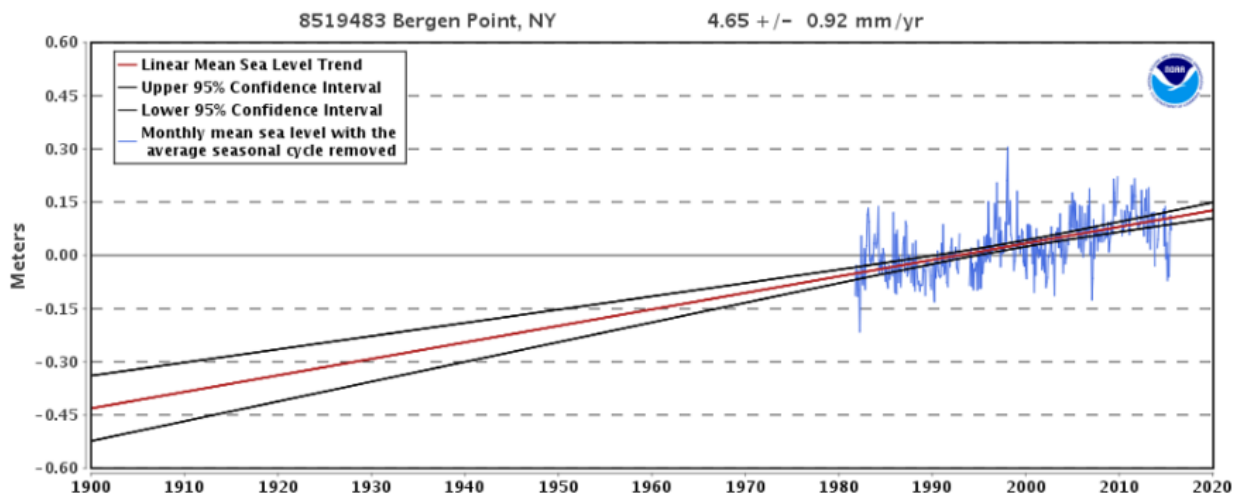


Figure 41. Sea level rise trends and monthly mean seal level at NOAA tide gage No. 8519483 at Bergen Point



Table 24. 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

Net Sea Level Rise Scenarios

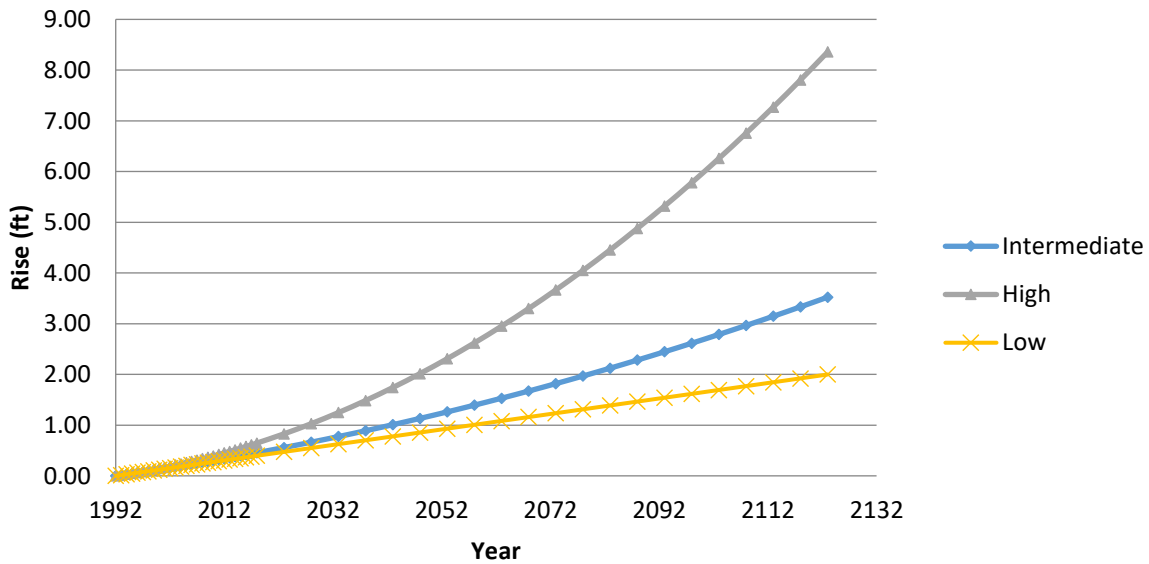


Figure 42. Projected SLC at Rahway for the local (low), NRC Type I (Intermediate), and NRC Type III (high) scenarios



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 is 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.



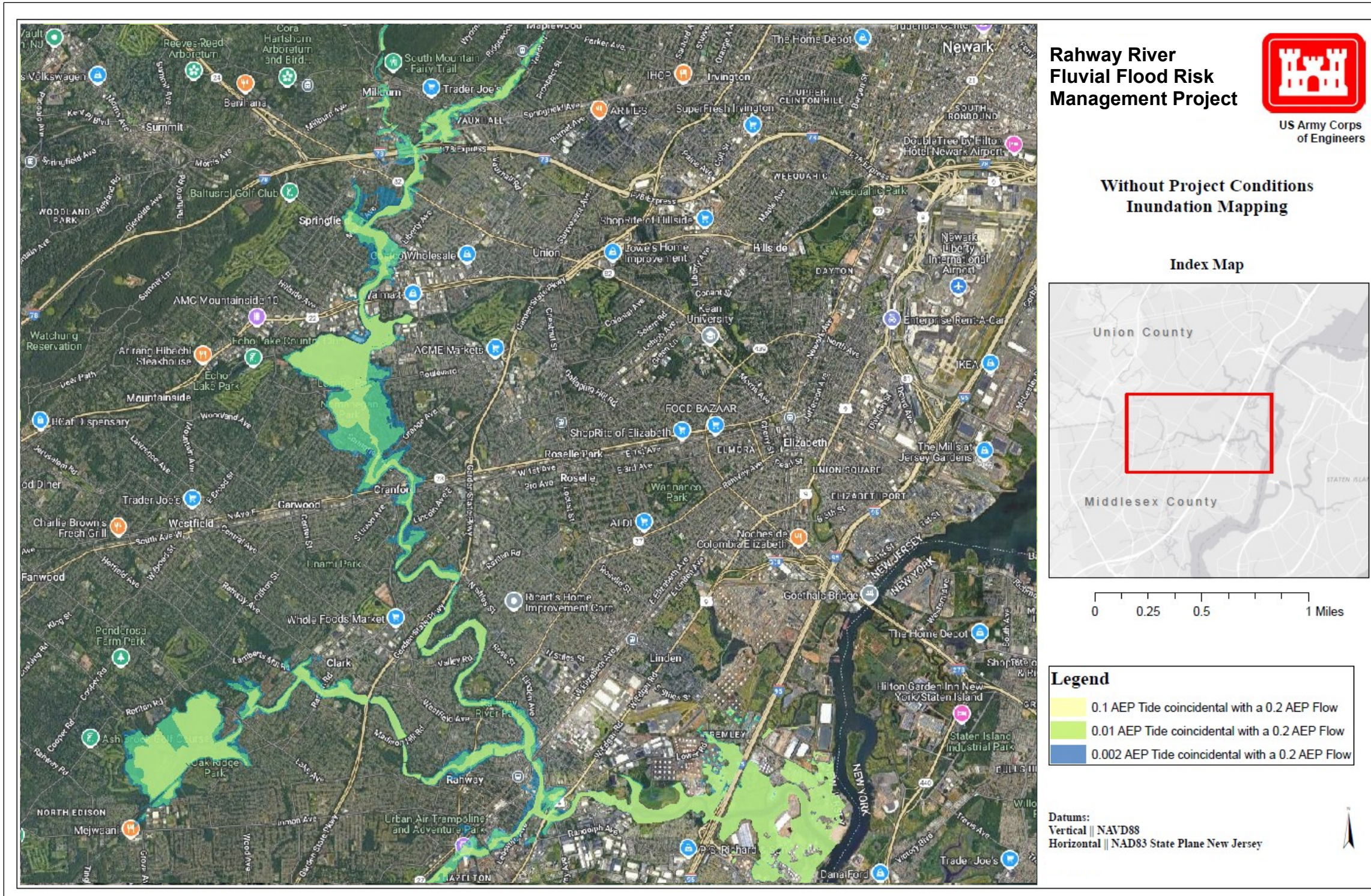
3.4 Hydraulic Profiles

3.4.1 Flow Line Computation

The calibrated HEC-RAS model of the Rahway River was used to determine the present “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 43.

Figure 44 through Figure 56 show the present “without project” WSE profiles for the Rahway River. The highlighted WSE profiles are the 0.5, 0.1, 0.04, 0.01, and 0.002 AEP (2, 10, 25, 100, and 500 year) events.





*Note: This is USACE-NACCS inundation of coastal events coincidental with 0.5 AEP fluvial flow. This does not represent joint-probability inundations.

Figure 43. “Without project” present condition inundation map for the 0.1, 0.01, and 0.002 AEP events



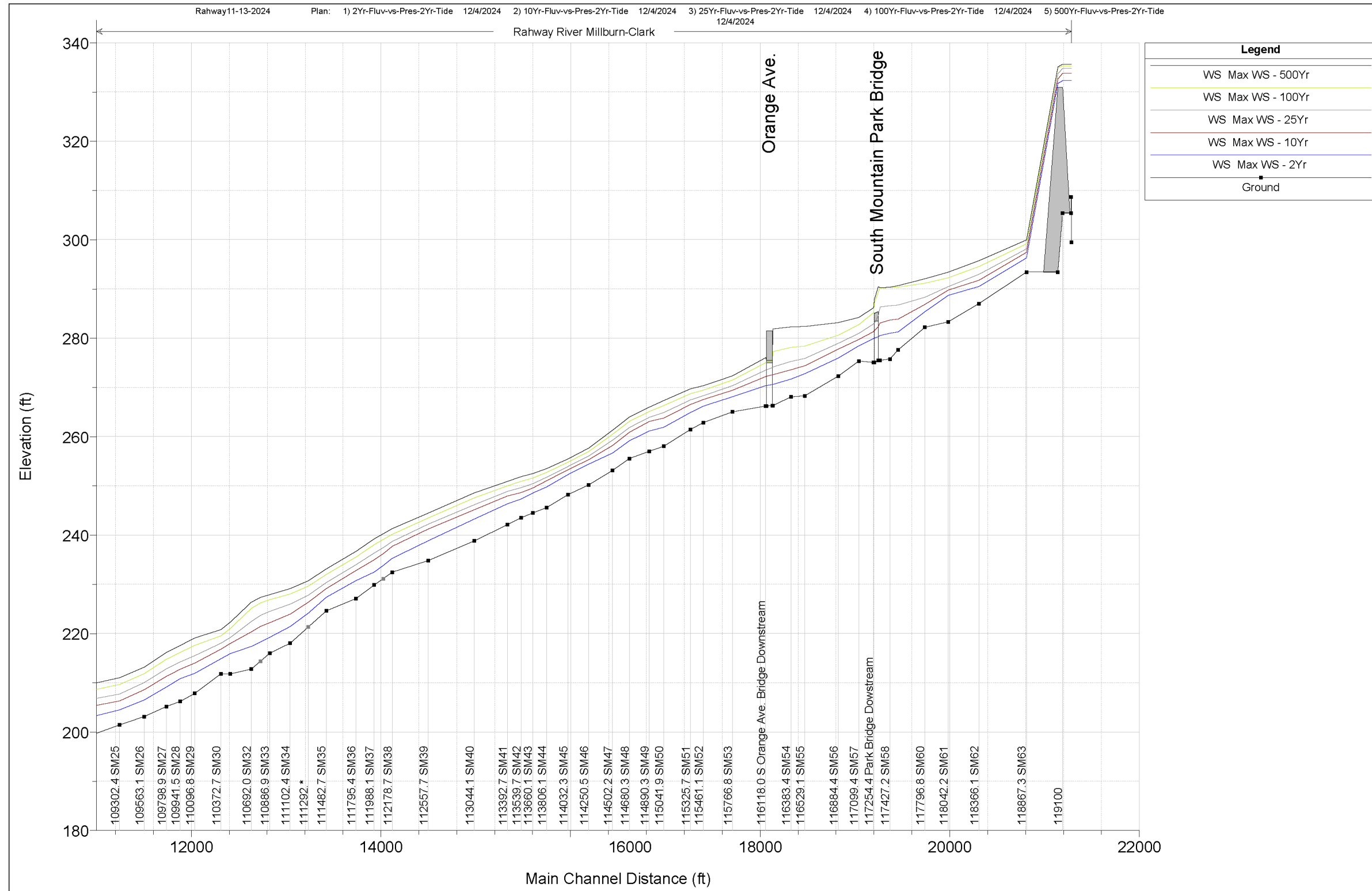


Figure 44. "Without project" condition computed water surface profile for West Branch (upstream section)



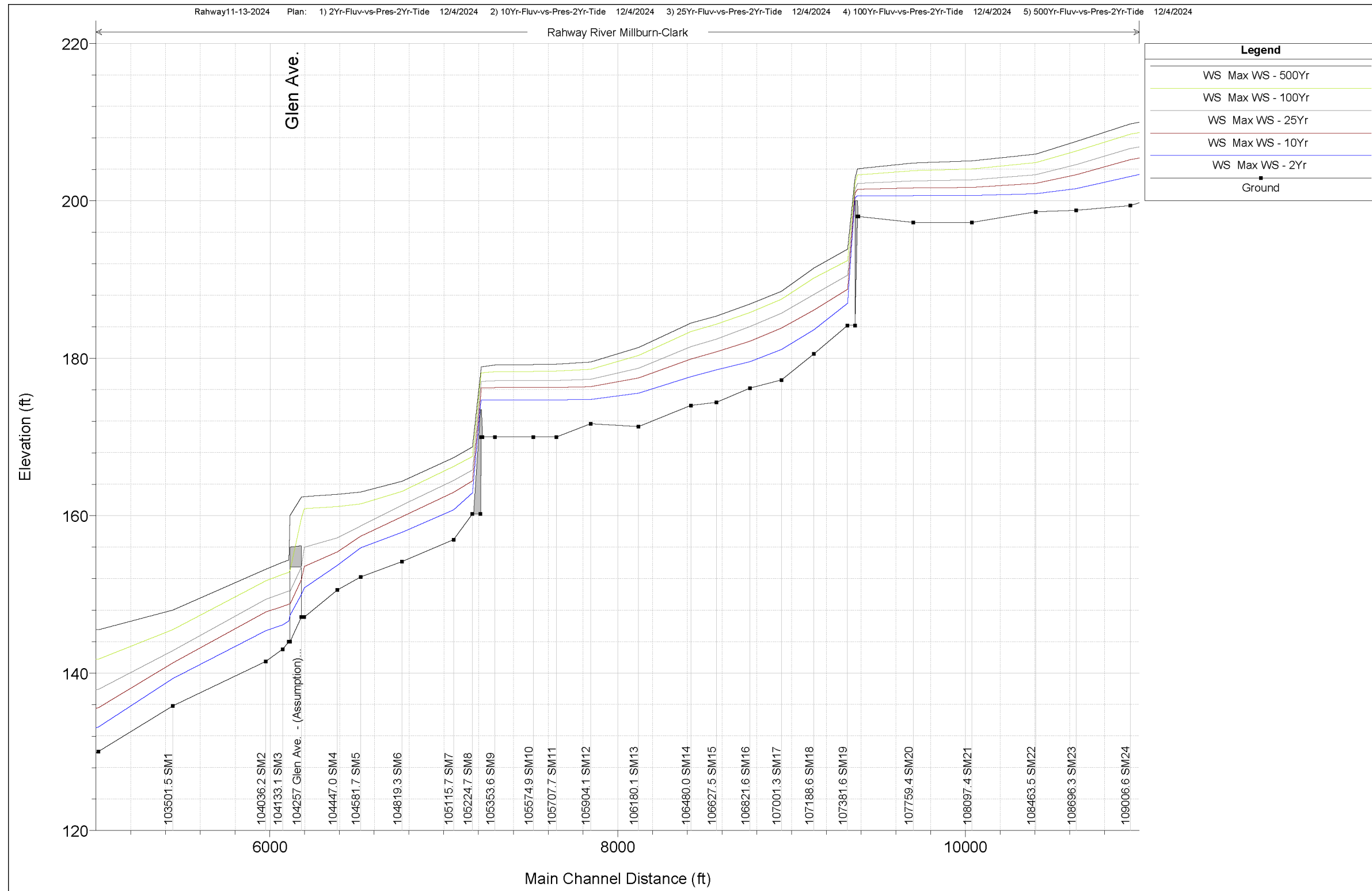


Figure 45. “Without project” condition computed water surface profile for West Branch (middle section)



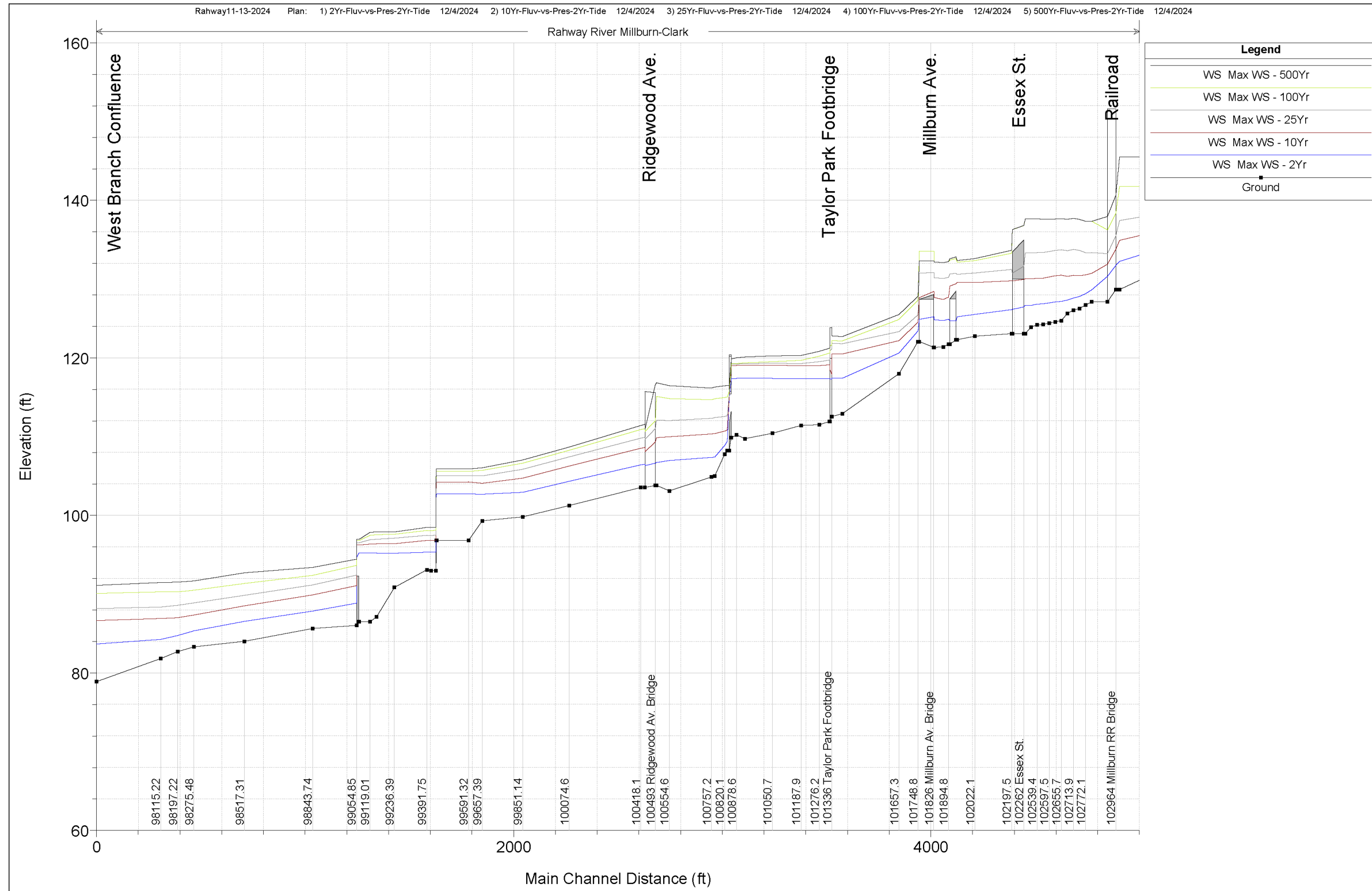


Figure 46. "Without project" condition computed water surface profile for West Branch (downstream section)



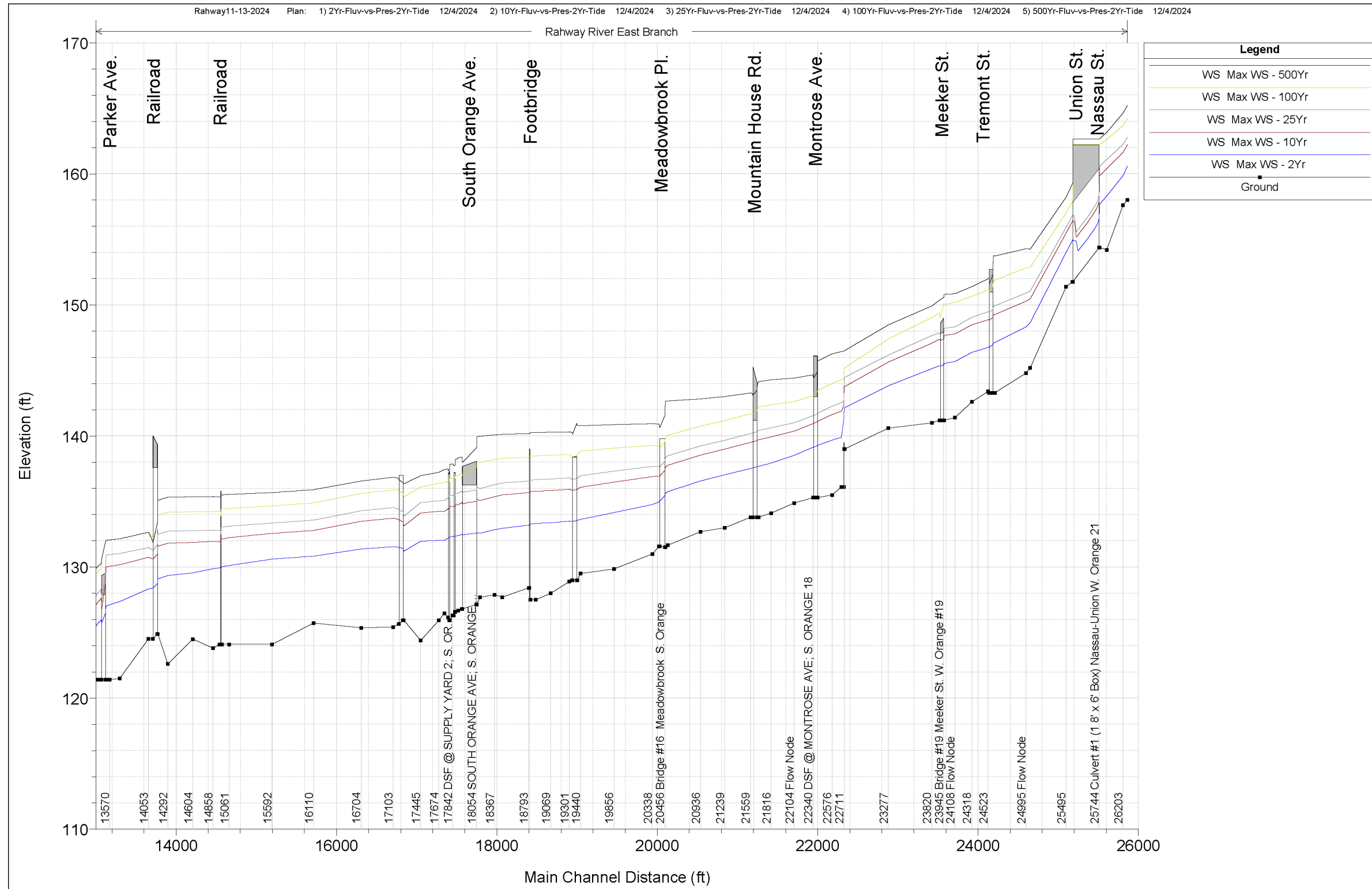


Figure 47. "Without project" condition computed water surface profile for East Branch (upstream section)



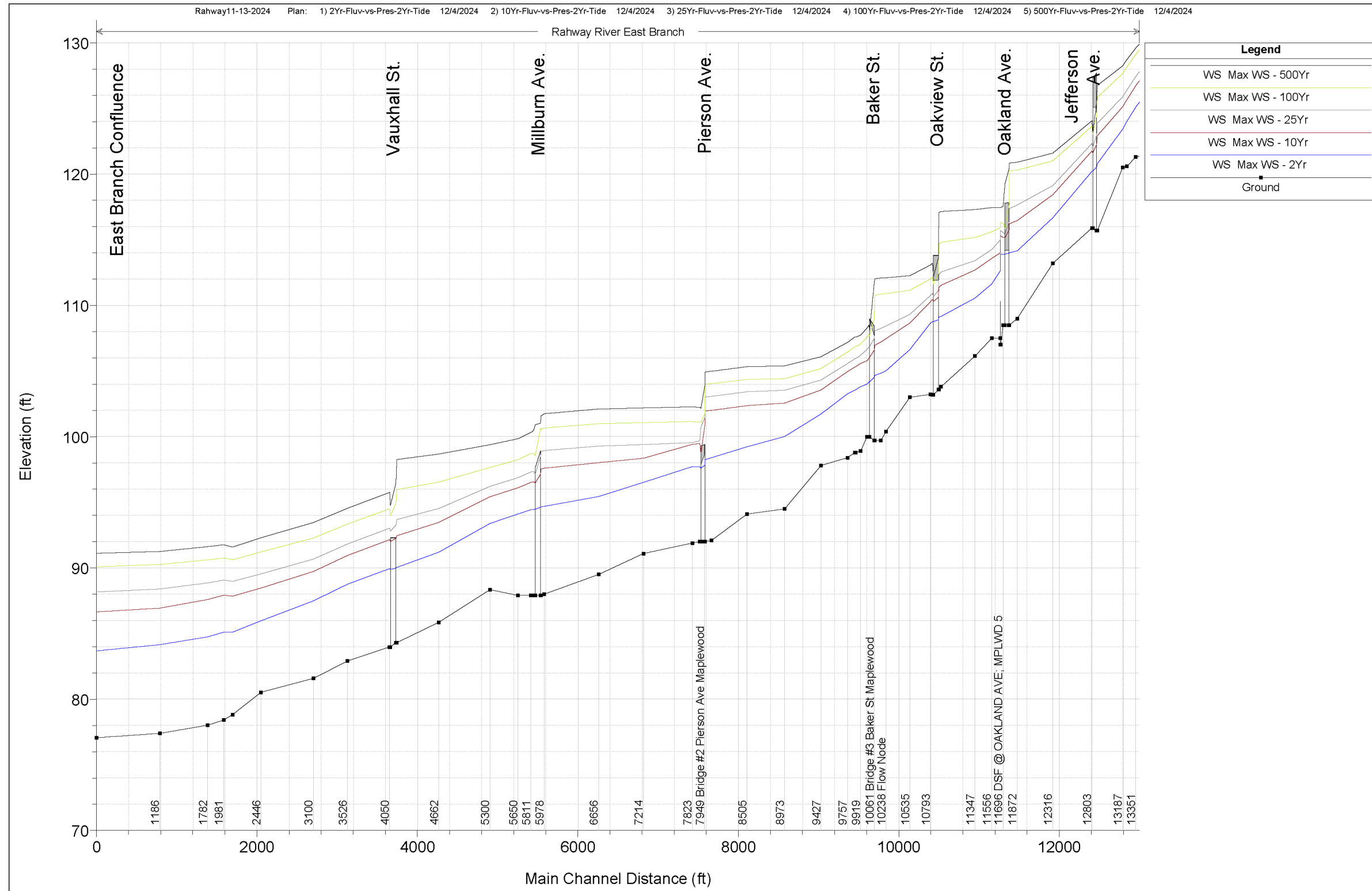


Figure 48. "Without project" condition computed water surface profile for East Branch (downstream section)



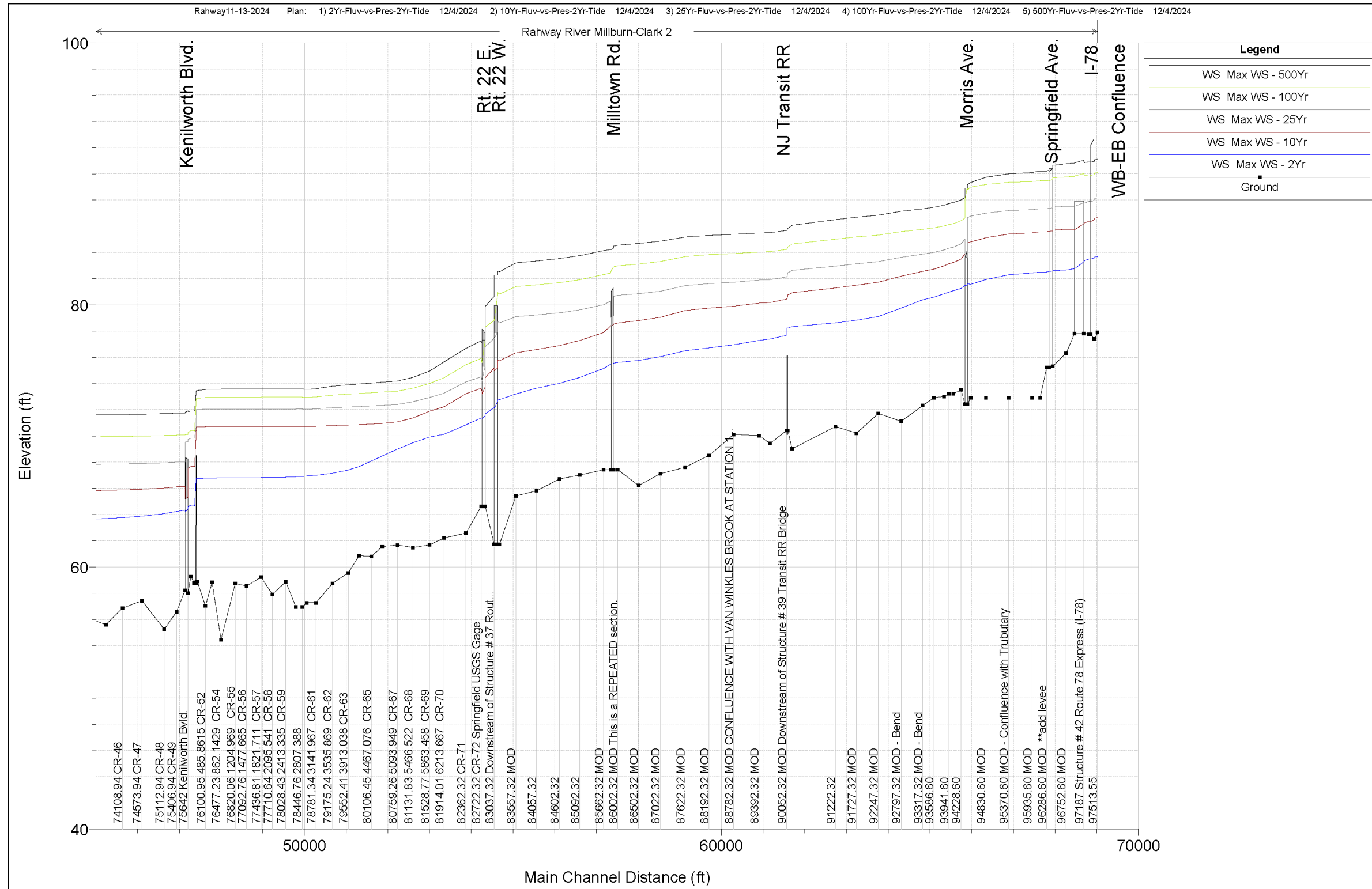


Figure 49. “Without project” condition computed water surface profile for Rahway River in Millburn, Springfield, and Kenilworth



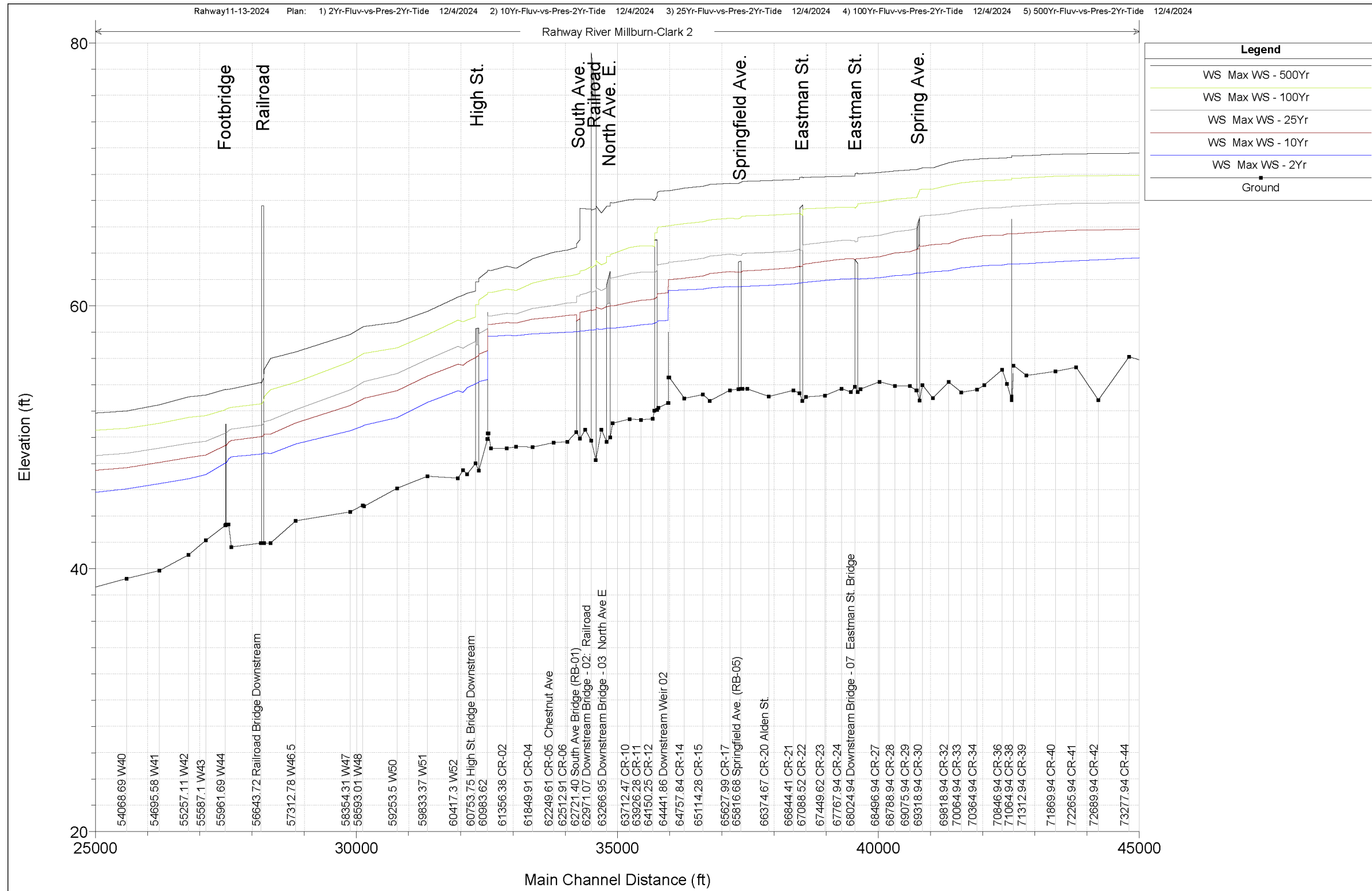


Figure 50. "Without project" condition computed water surface profile for Rahway River in Cranford



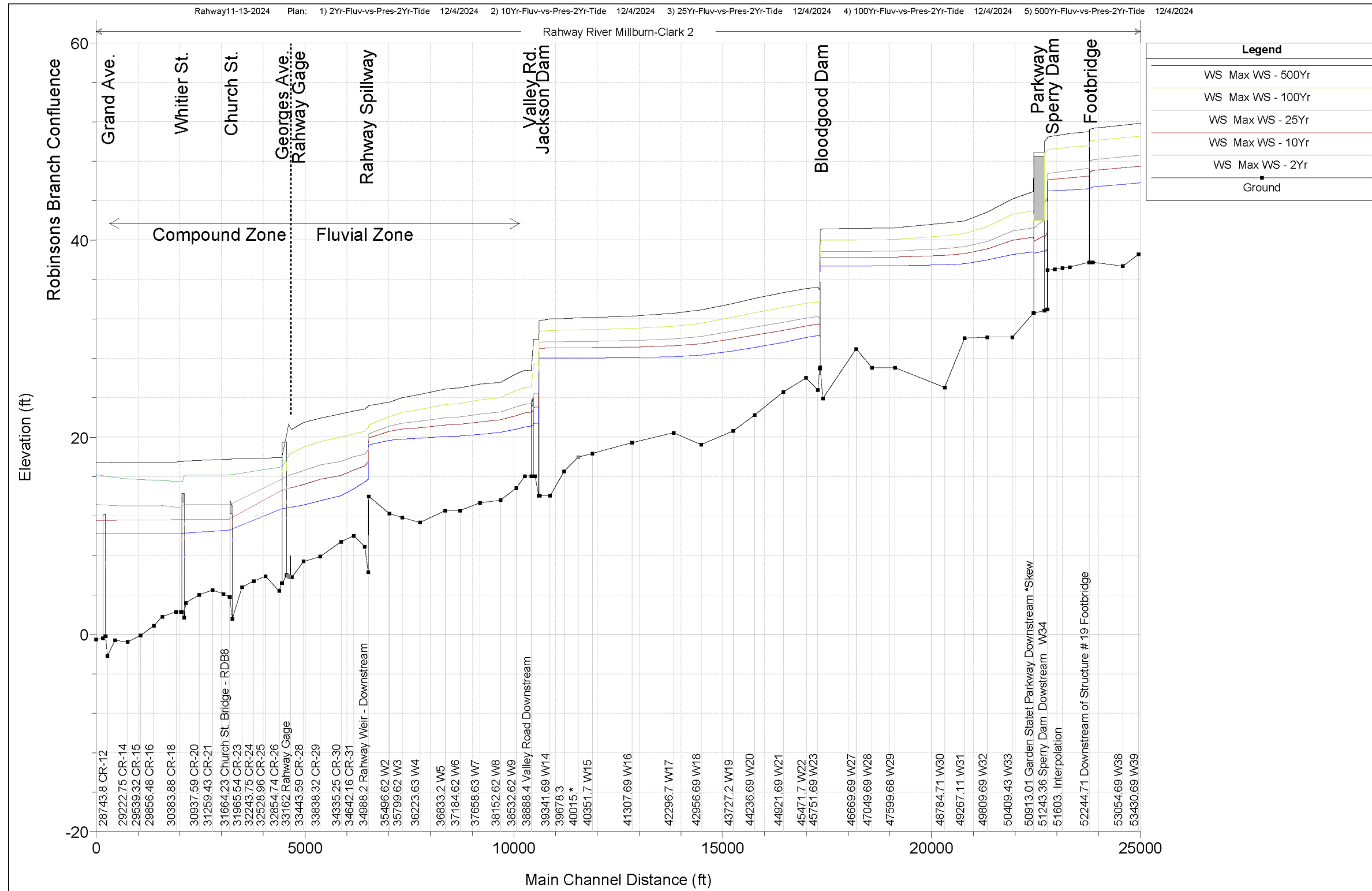


Figure 51. "Without project" condition computed water surface profile for Rahway River in Clark and Rahway



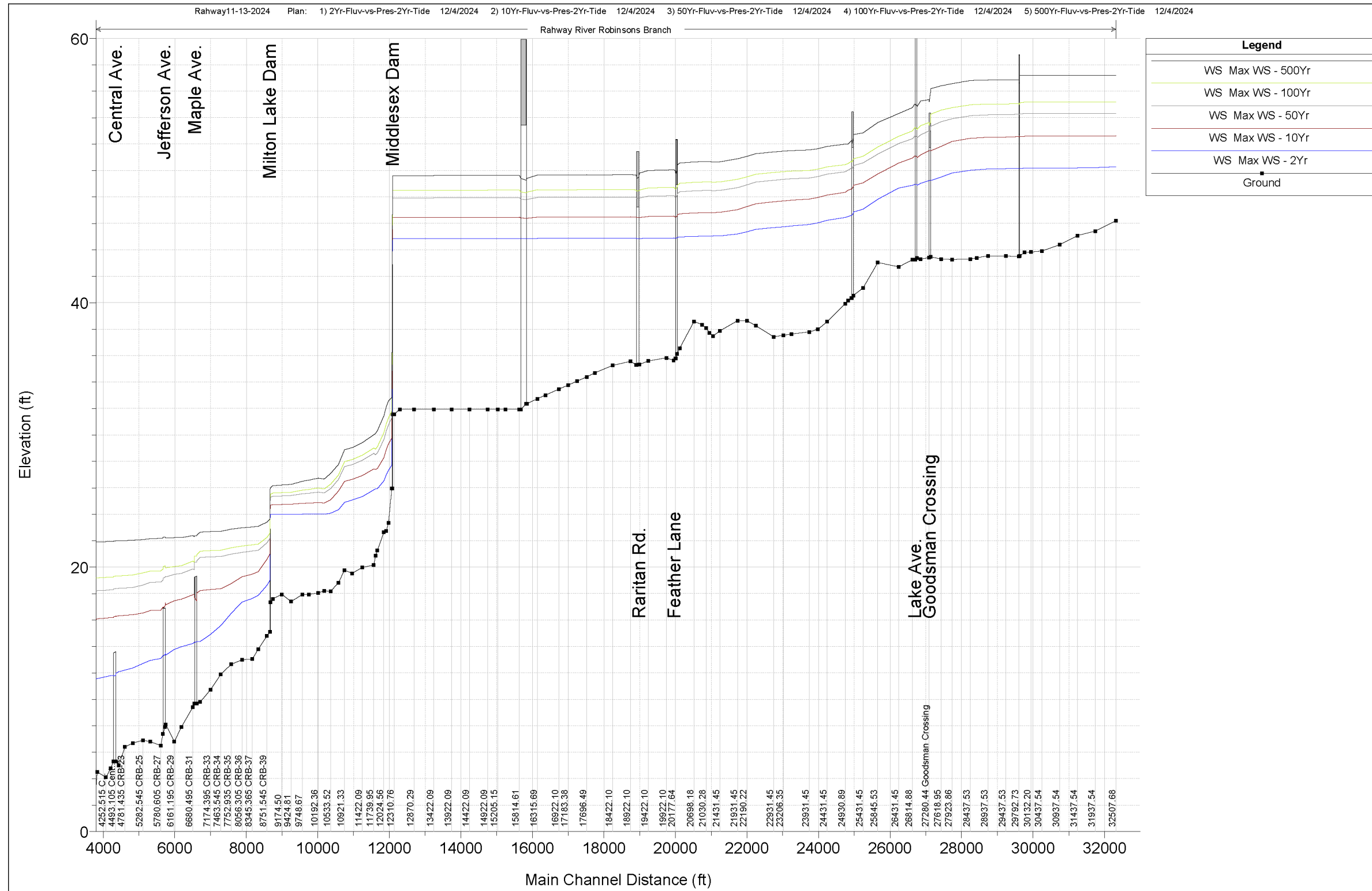


Figure 52. “Without project” condition computed water surface profile for Robinsons Branch (upstream section/fluvial zone)



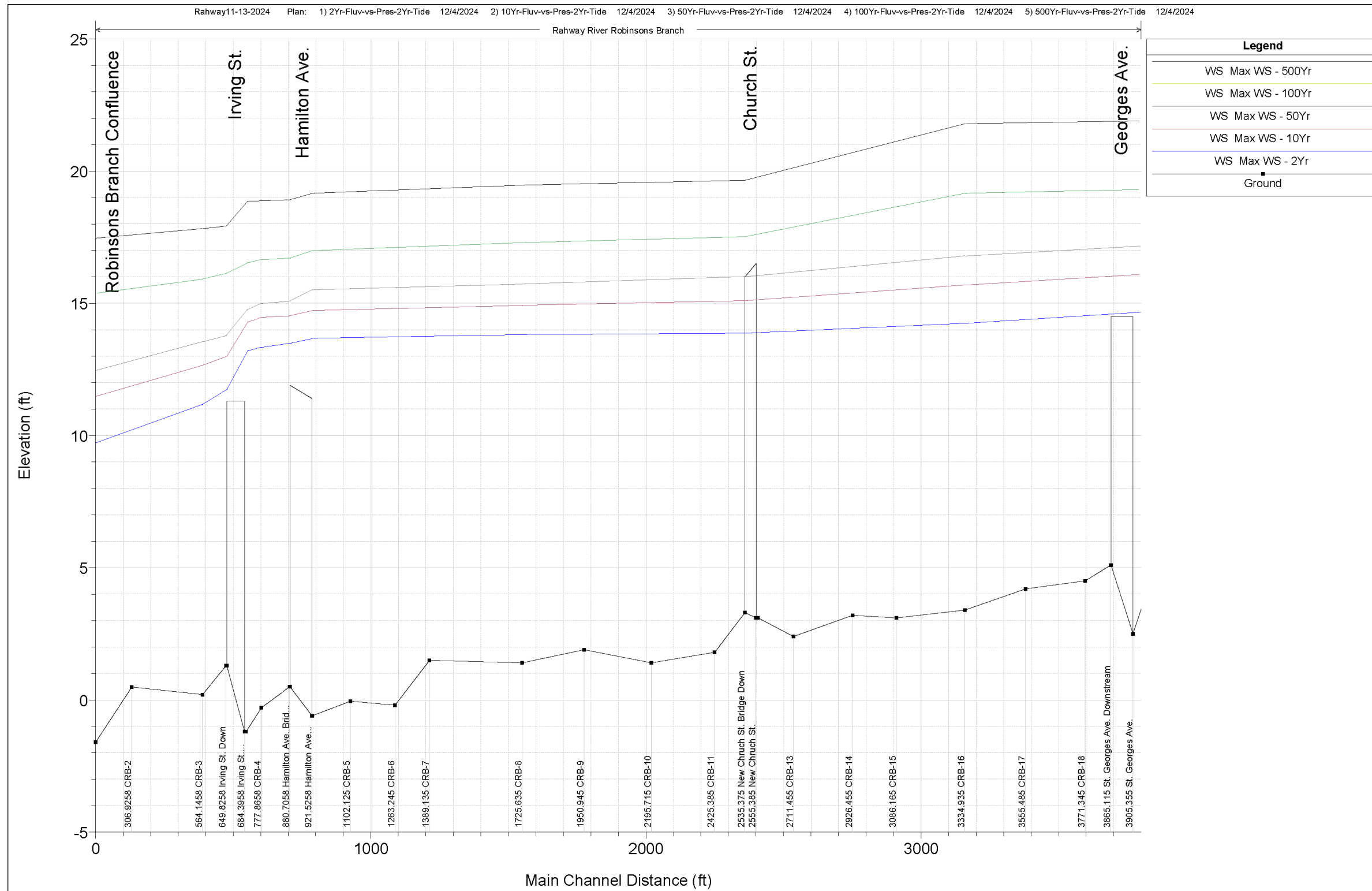


Figure 53. "Without project" condition computed water surface profile for Robyns Branch (downstream section/compound zone)



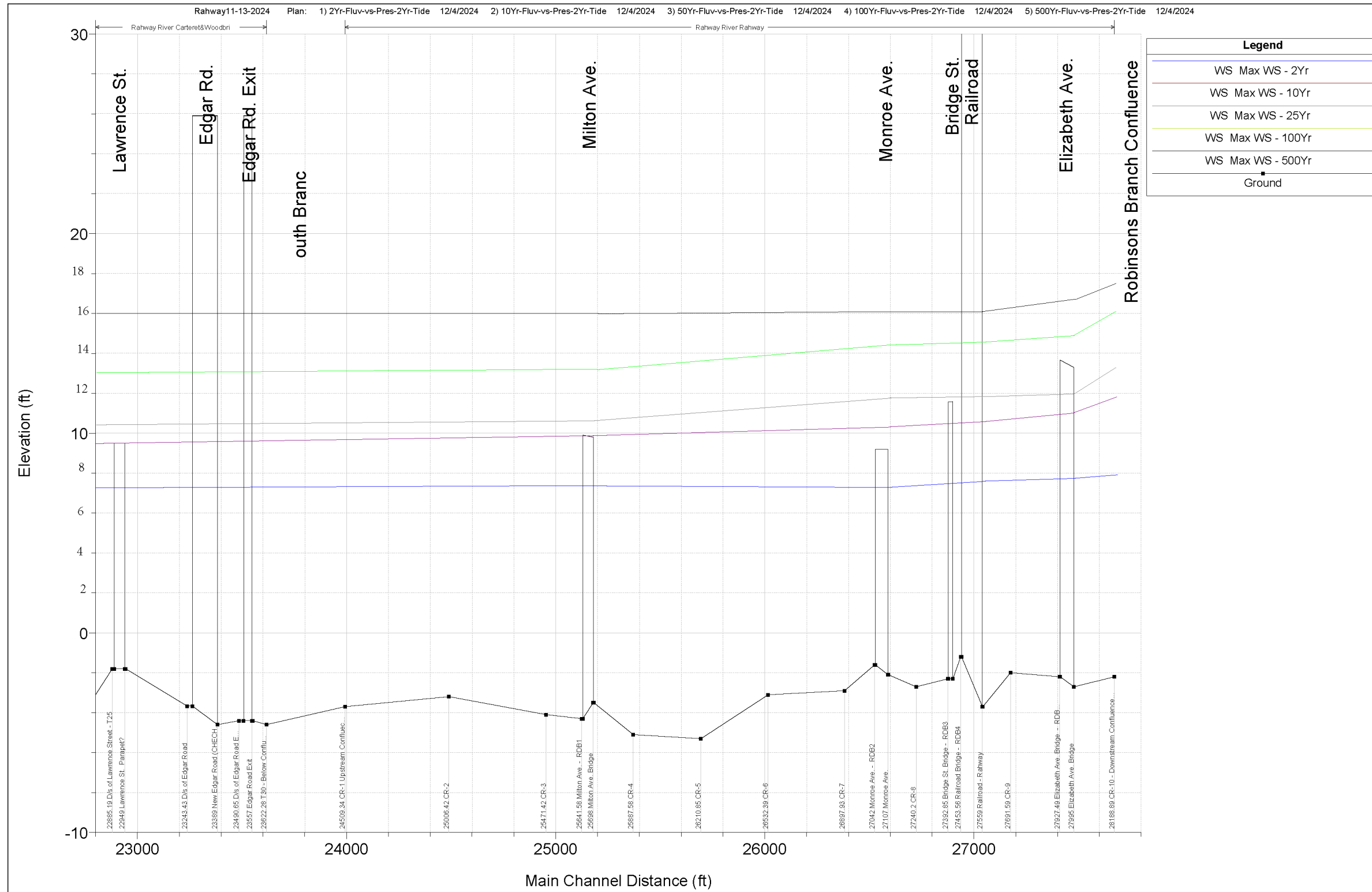


Figure 54. “Without project” condition computed water surface profile for Rahway River in Rahway (compound zone)



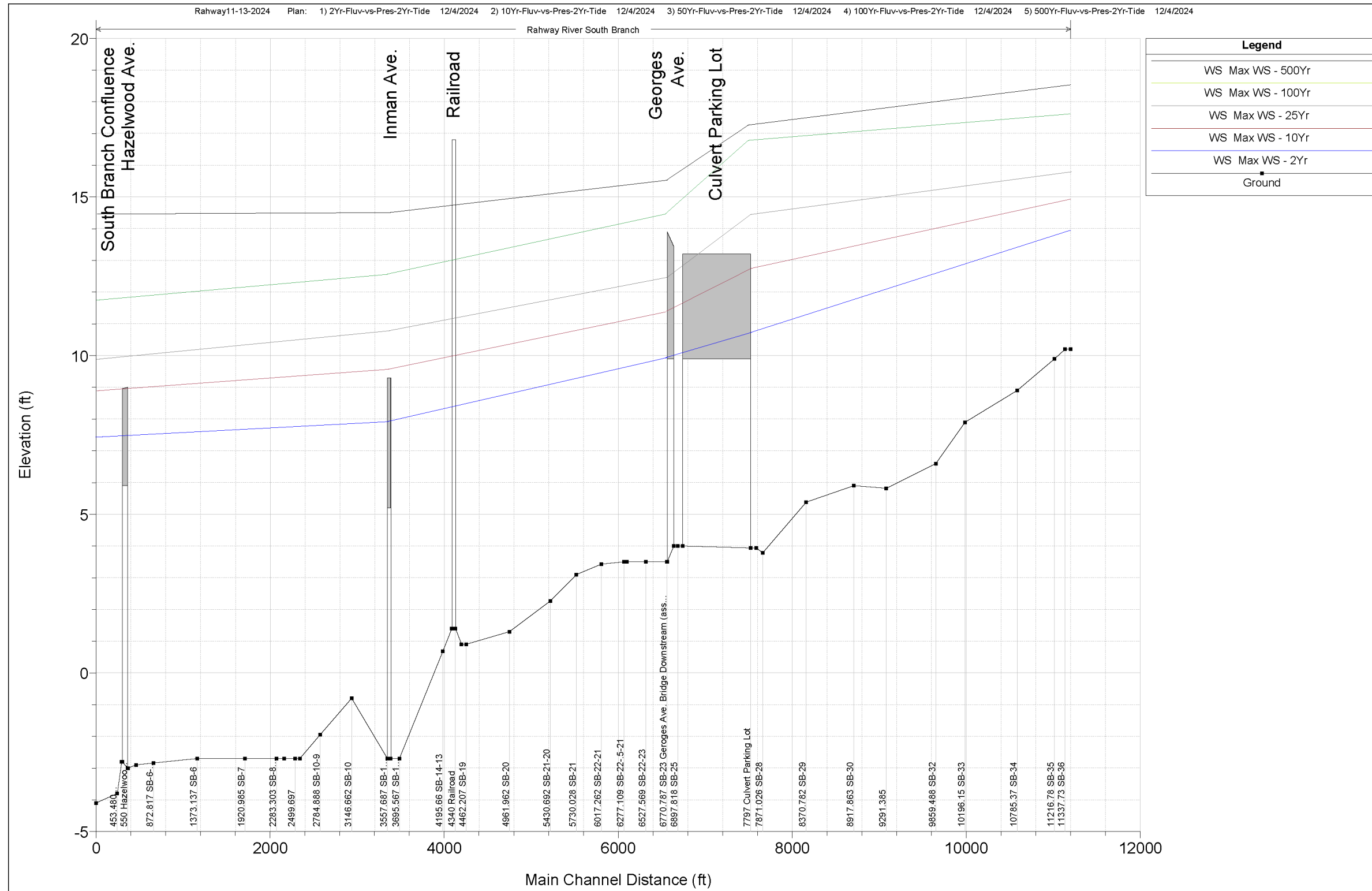


Figure 55. "Without project" condition computed water surface profile for South Branch (compound zone)



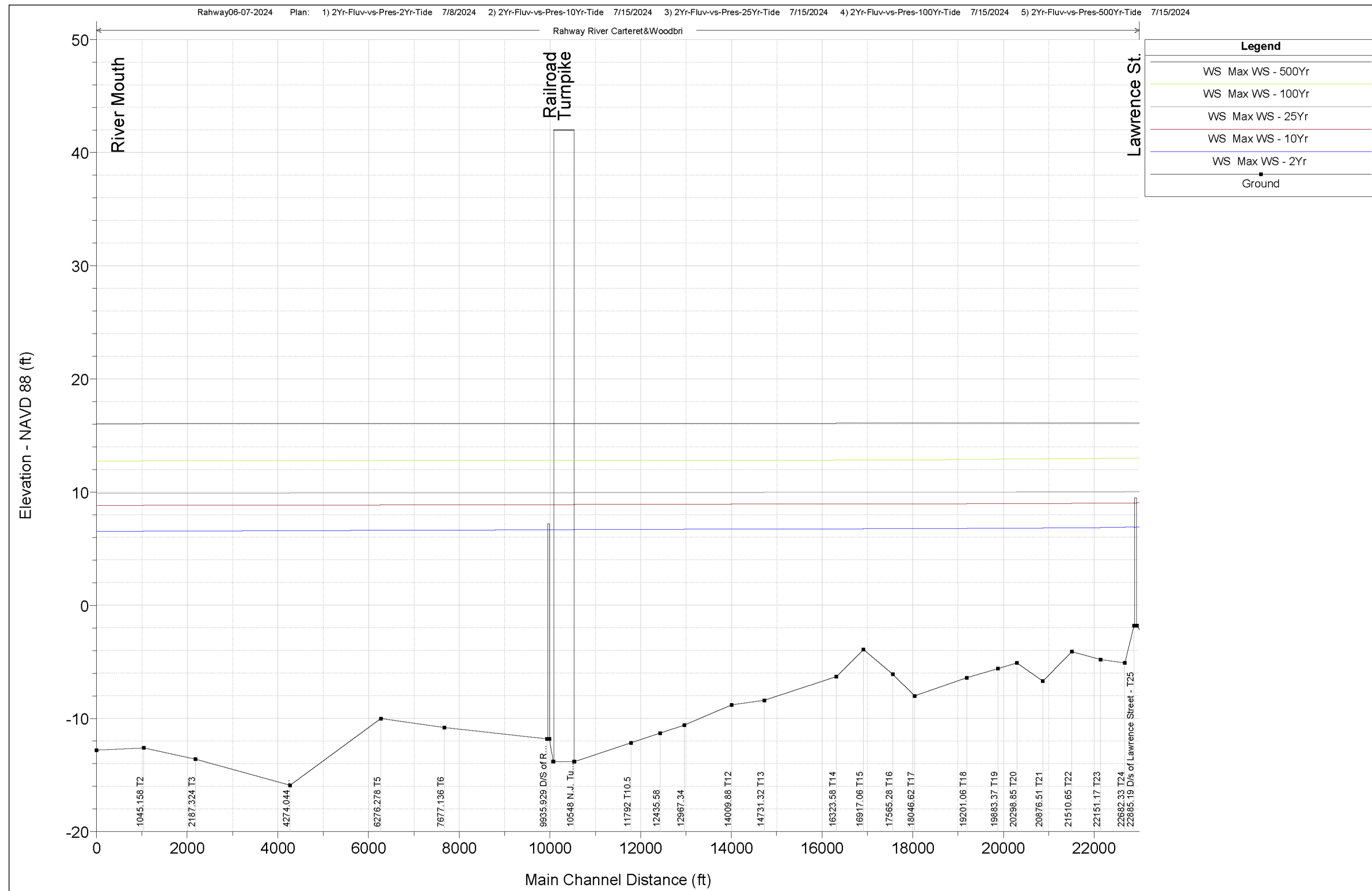


Figure 56. "Without project" condition computed water surface profile for Rahway River in Carteret (tidal zone)



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 evaluated alternatives include channel work, levees, floodwalls, reservoirs detention, nonstructural, and/or a combination of the above. The alternatives were focused on reducing flood risk in the Townships of Millburn and Cranford and City of Rahway on the Robinson's Branch. The alternatives evaluated can be classified as No Action (same as Future without Project Conditions), Structural, and Nonstructural alternatives. The Structural alternatives involve channel work, levees, floodwalls, tide gates, and/or a combination of the above. Nonstructural 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 nonstructural 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#1

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 #2: Upstream Detention*

4.3.1.1 Alternative #2 – Summary and Features

This structural alternative is to provide detention along the West Branch in the South Mountain Reservation upstream of Campbell Pond Dam. As shown in Figure 57, the proposed 60-foot-tall and 500-foot-wide earthen dam was based on the design event of the 50-yr flood. The 300-foot-wide spillway was sized based on containing the full 50-yr pool below this spillway. The outlet size of 5-foot by 5-foot was based on maintaining peak discharge at the spillway level below the damaging elevations in the Township of Millburn. Based on the potential downstream loss of life in the event of dam failure, the Inflow Design Flood (IDF) was identified as the Probable Maximum Flood (PMF). The PMF storm was developed based on HMR52 Probable Maximum Storm. The spillway length and flow over the spillway were determined based on the available geometry at the dam site. The spillway is located on the leftover bank, where the subsurface rocks subsurface rock is basalt and is more competent to maintain the concrete spillway. This results in a side flow spillway of 300 feet and a flow depth over the spillway of 12.5 feet to pass the PMF. Overall, the dam height at the spillway is 42 1/2 feet, with an additional 17.5 feet to pass the PMF flood with five feet of freeboard to the non-overflow sections.

The details of each component are listed below:

- 60-foot-tall, 500 ft. wide earthen dam located to minimize fill
- Crest elevation: 260 ft.
- Outlet structure: 5ft. x 5ft. concrete box
- Weir: elevation 242 ft., 300 ft. wide
- 300-foot-wide spillway located based on terrain, rock 10-foot below existing surface
- Spillway chute: concrete, 100 ft. width

The plan layout is pending USACE input in the Engineering Appendix.



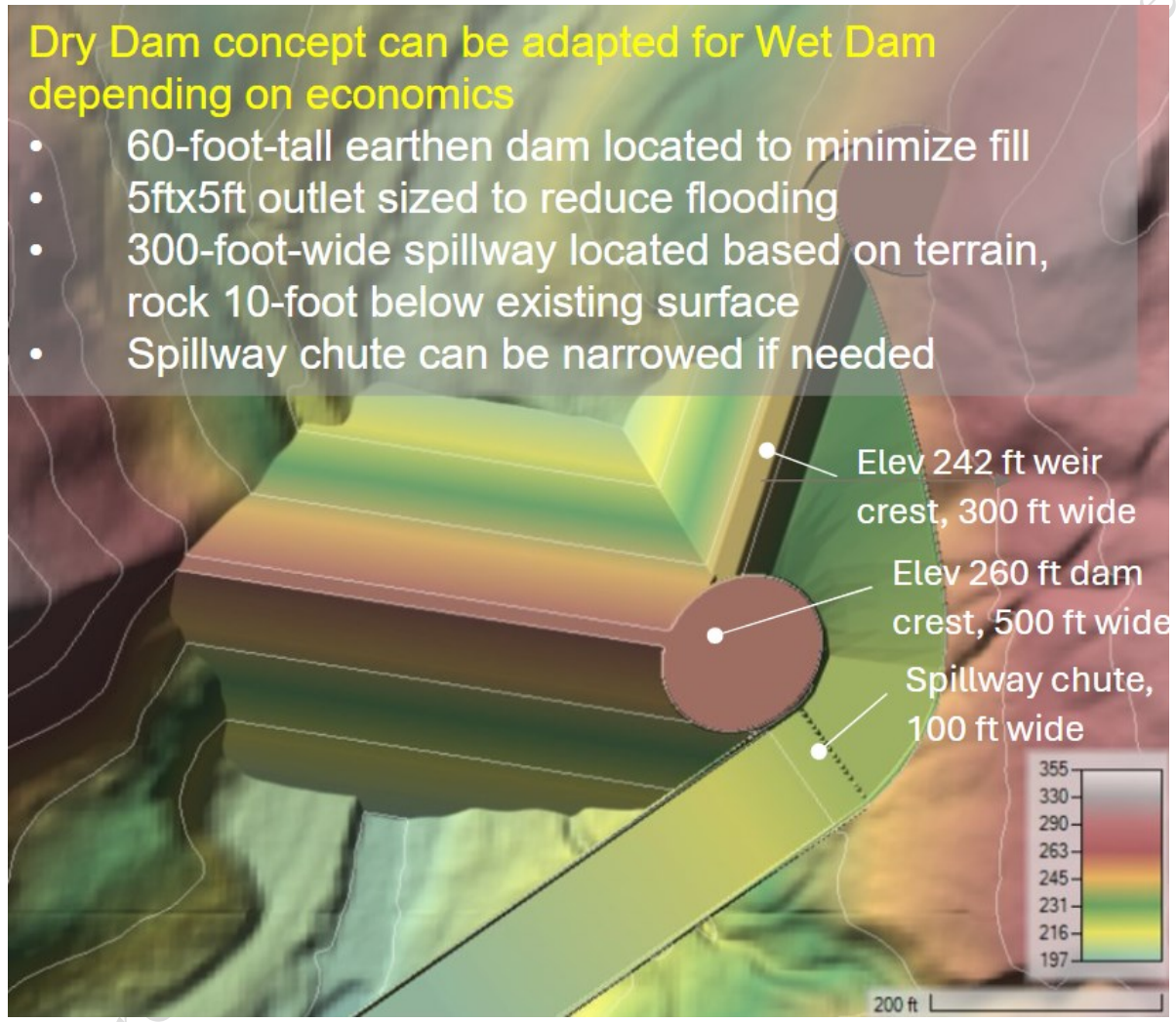


Figure 57. Dry dam proposed in Alternative #2: Upstream Detention



4.3.1.2 Alternative #2 – Hydraulic Analysis

This alternative has an upstream detention feature that mitigates for the downstream induced damages. The proposed South Mountain Reservation Dry Dam would produce benefits to the Townships of Millburn and Springfield. Figure 58 shows the computed water surface profile along West Branch in the Township of Millburn under the 2% AEP (50-yr) event. Reduction in WSEs up to 7 ft. are expected. Figure 59 and Figure 60 delineate the computed inundation map along West Branch downstream of the proposed dry dam and in the Township of Millburn, respectively. Significant flood reduction is observed at the south end of the South Mountain Reservation, near Millburn Avenue, and Main Street.

4.3.1.3 Alternative #2 – Assessment of Modified Alignments

The alternative alignment presented above is considered the most cost-efficient location and layout to reduce downstream flood damages. If the Alternative is selected as part of the TSP, the outlet size, spillway elevation, and alignment should be optimized with consideration of additional geotechnical data. As part of the plan development, the inclusion of a small permanent pool to provide enhanced recreation experience and reduce the need for parkland diversion and replacement was considered. Because this layout was found to provide substantially less flood damage reduction and would likely have a higher construction cost, it was dropped from further consideration.

Another layout considered was a multiple dam alignment developed by the non-Federal interests in Millburn. The conceptual layouts of these dams were reviewed, and it was noted that the dam alignments did not include an emergency spillway capable of passing the IDF/PMF. Assuming each location could be modified to include a similar spillway to the alignment described above, the effective storage at each dam would be reduced by about 18 feet in depth. This would result in available flood storage volumes comparable to the alignment described above but would require construction of multiple spillways and energy dissipation structures, increasing design, construction, and O&M costs. Accordingly, the multiple dam alignment was not included in detailed modeling or design.



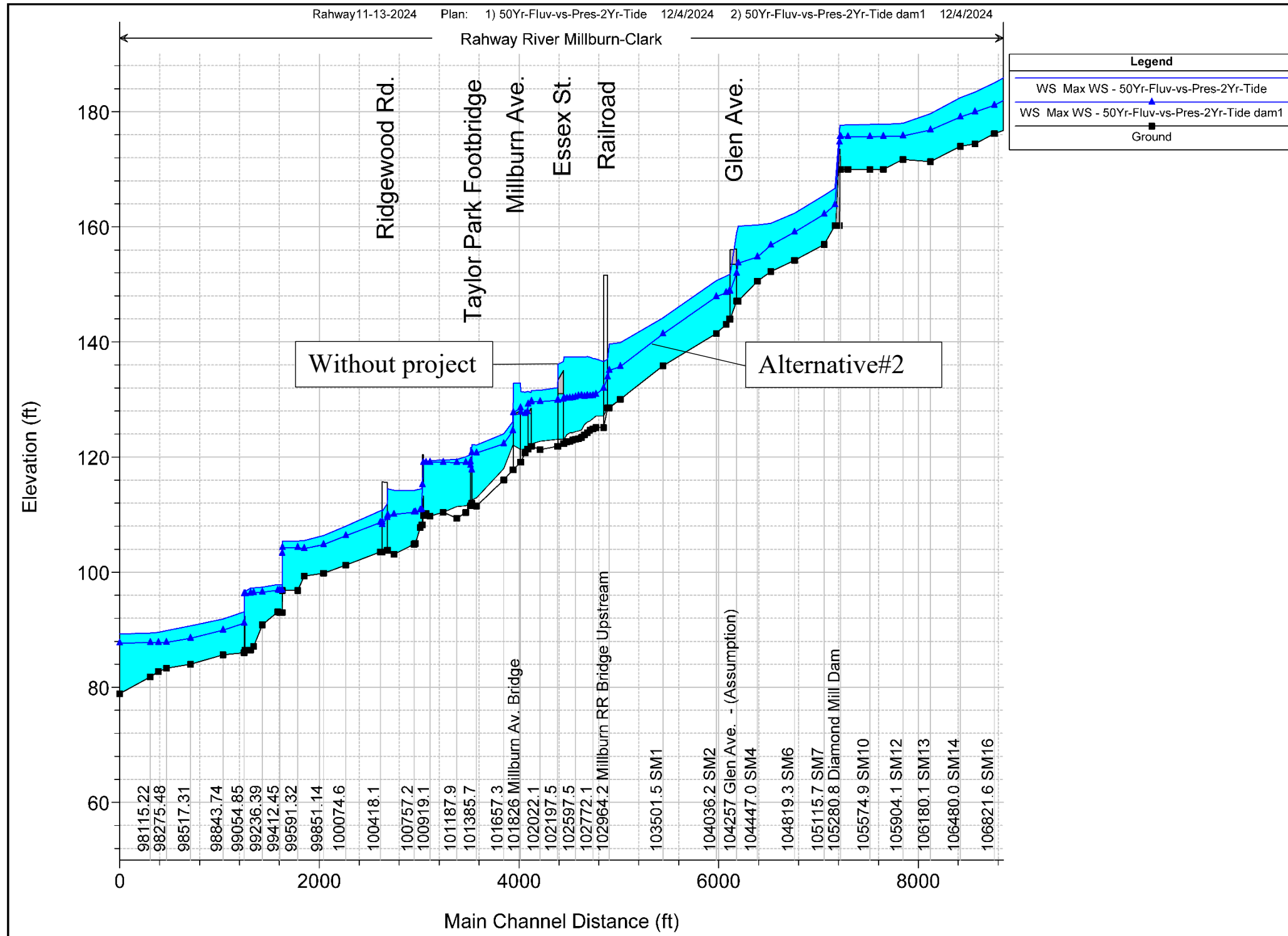


Figure 58. Computed water surface profile for Alternative #2: West Branch in the Township of Millburn



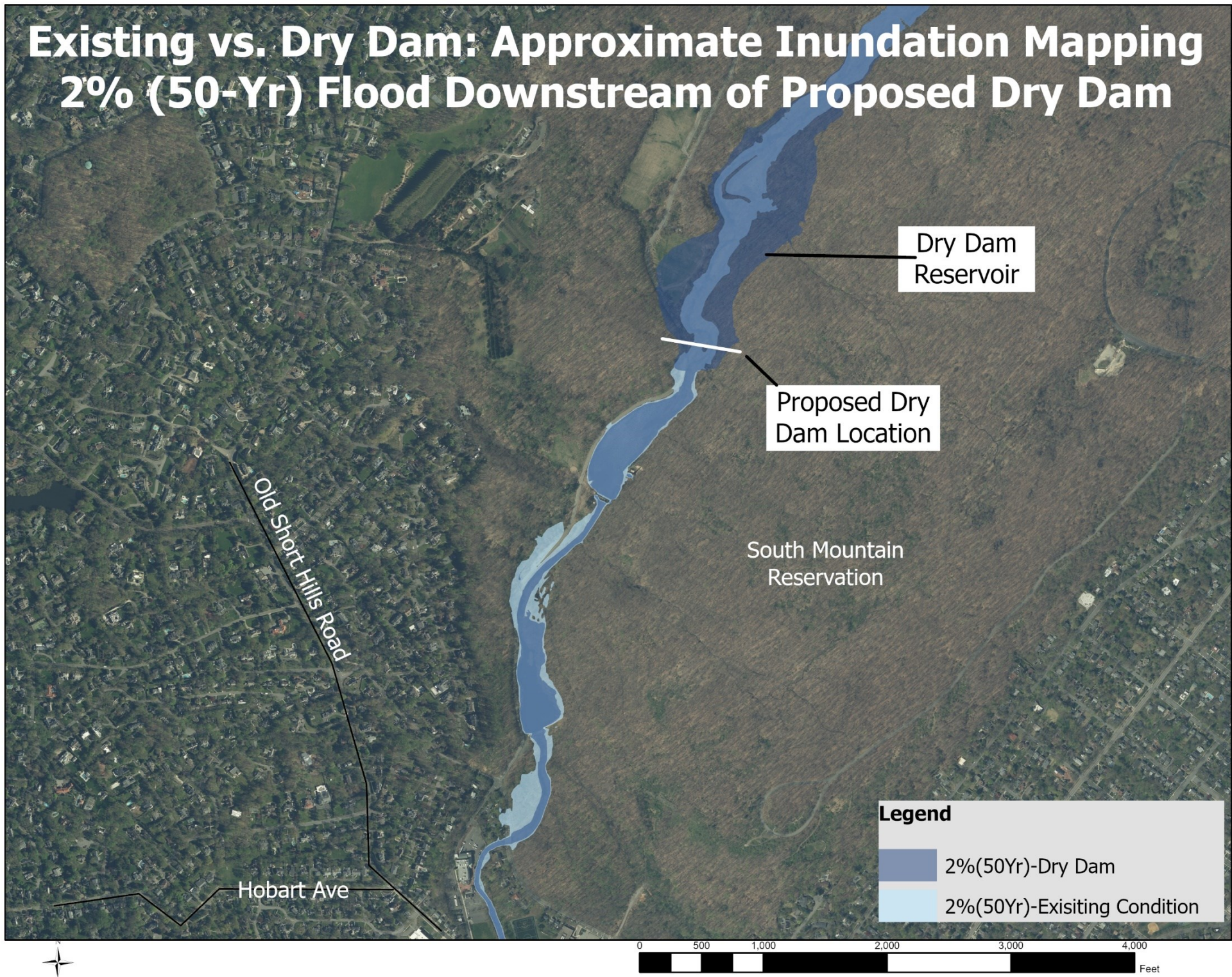


Figure 59. Computed inundation map for Alternative #2: West Branch downstream of proposed dry dam



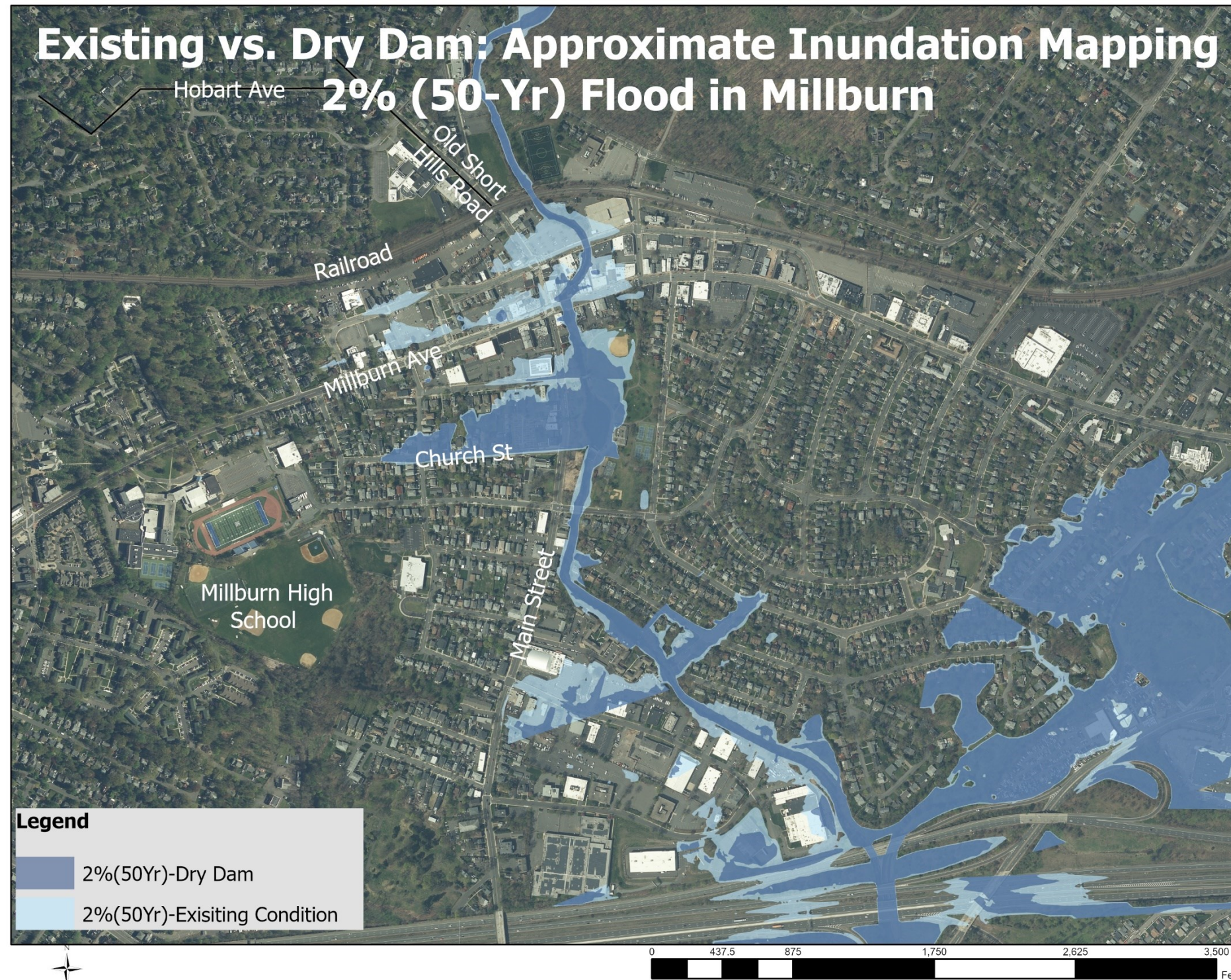


Figure 60. Computed inundation map for Alternative #2: West Branch in the Township of Millburn



4.3.2 *Alternative #3: Combination Plan*

4.3.2.1 Alternative #3 – Summary and Features

In the high damage area near the Township of Millburn, the cause of the flooding is predominantly lack of channel capacity and restrict and the presence of restricted bridges. This alternative identified two bridges for replacement at Oakland Rd. and at Essex St. In addition, there are several areas of channel improvement. In the Township of Cranford, there are currently some existing levees that provide protection of extremely low reliability. These levees are tied into existing levees downstream of Nomahegan park. This alternative proposes new levees throughout the Cranford reach. In areas where there is limited space, the levees are replaced by floodwalls (primarily T-walls). In addition to the levee and floodwall considerations, there are several dam modifications and removals proposed. These include removal of the Sperry Dam, modification of the Jackson Dam, and modification of the Droschers Dam. To avoid induced flooding with this combination plan, there is an area of new offline storage proposed in the area of Rt-78, downstream of the channel and bridge improvements in Millburn but upstream of the Lenape Park Detention Dam and the proposed levees in Cranford. The offline detention area effectively mitigates the increased flows from the channel and bridge improvements in the Township of Millburn. In addition, it reduces the total inflow to the Lenape park detention area. Therefore, with more available storage during the peak flood stages, the existing detention facilities in Lenape Park more effectively reduces flows through the Cranford area and avoids induced flooding from the proposed levy and flood wall improvements.

The details of each component are shown in Table 25.



Table 25. Descriptions of features in Alternative #3

Feature	Cross-Section Station	Description
Raise Bridges	102262 11736	<ul style="list-style-type: none"> •Essex St: raise low chord by 1ft, elev. 131 ft •Oakland Road: raise low chord by 1ft, elev. 115.2 ft
Improve Channel	102964.2-100494.1 5843-397	<ul style="list-style-type: none"> •2,470 feet removal of existing concrete lining, deepen channel by 1 ft; channel slope of 0.0112 ft/ft •Deepen channel by 1 ft; channel slope of 0.025 ft/ft (Existing vertical concrete channel walls would not be modified)
Offline Storage	97310.76-97187.60	Storage between EB-WB I-78 embankment, provides 30 ac-ft. A 5-ft-diameter pipe will connect the basins within the storage area
Levees/Floodwalls	75840.77-61533.50	Incorporates pump stations for interior drainage. Levee embankment section has a 15-ft crest width, sheetpile seepage cutoff wall, and 3H:1V and 3H:1V landside and waterside slopes. Floodwalls are concrete T-walls.
Dams/Weir modifications	64452.44 39093 60983.62	<ul style="list-style-type: none"> •Sperry Dam: Remove existing weir; lower channel invert to elev. 55 ft •Jackson Dam: Modification to saw toothed weir, lower crest elevation by 0.6 ft (elev. 24.5). Weir length is 280 ft. •Droeschers Dam: Modification to saw toothed weir, maintain crest elevation of 55 ft. Weir length is 250 ft.

The plan layout of each component IS pending USACE input in the Engineering Appendix.



4.3.2.2 Alternative #3 - Hydraulic Analysis

This alternative would produce flood mitigation to the Townships of Millburn, Maplewood, and Cranford. Figure 61 shows the computed water surface profile along West Branch in the Township of Millburn under the 2% AEP (50-yr) event. Reduction in WSEs of 2 to 8 ft. are observed from the Millburn Railroad to Ridgewood Ave. Figure 62 shows the computed water surface profile along East Branch in the Township of Maplewood under the 2% AEP (50-yr) event. Reduction in WSEs up to 2 ft. are expected near Oakland Ave. and from Pierson Ave. to Vauxhall St. Figure 63 and Figure 64 depict the computed inundation map and water surface profile along Rahway in the Township of Millburn, respectively. Significant flood reduction is observed from Lenape Park Dam to South Ave. E. where levees, floodwalls, and a pump station are installed to address interior drainage. Reduction in WSEs up to 4 ft. are expected along Rahway River at the “U” turn just upstream of the Garden State Parkway.



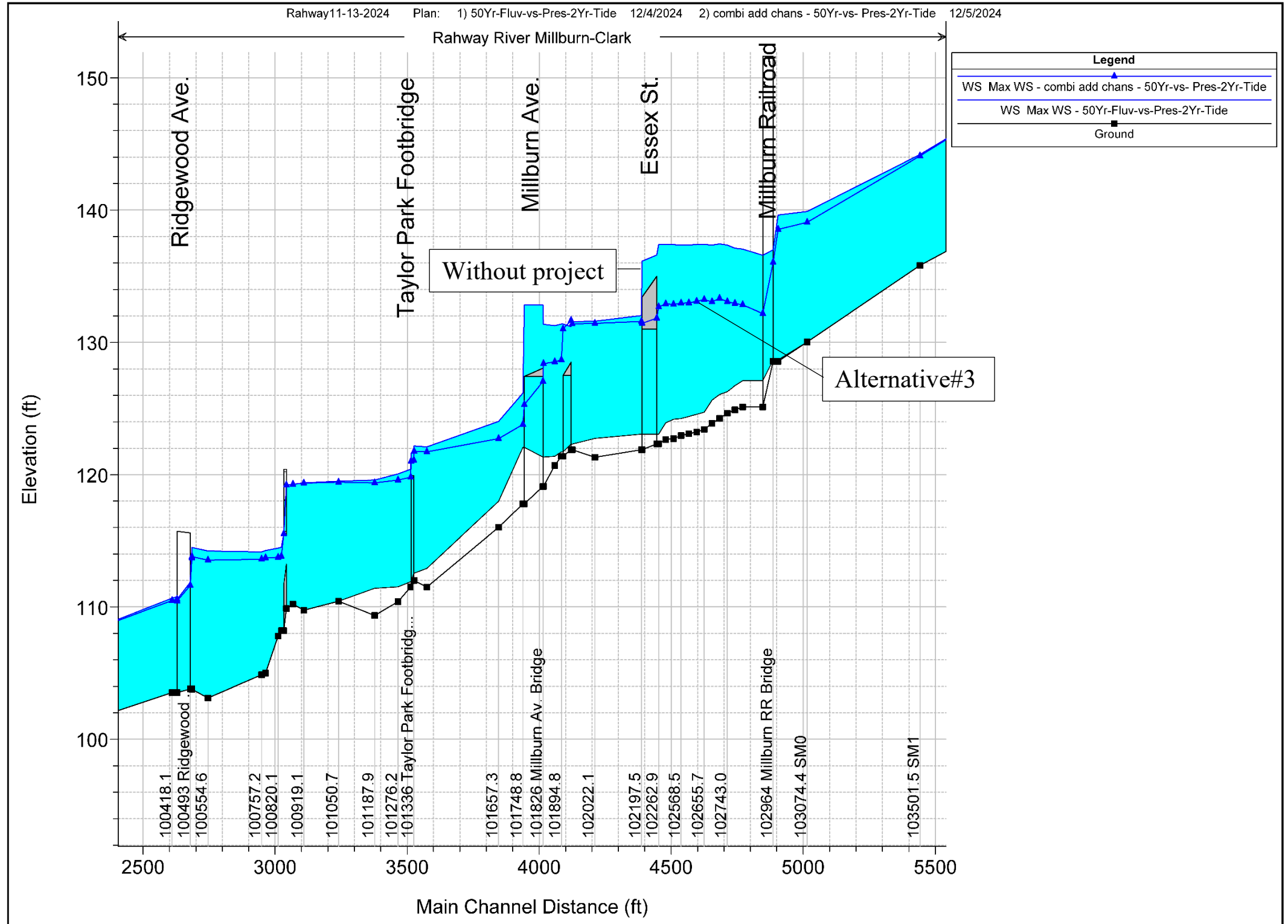


Figure 61. Computed water surface profile for Alternative #3: West Branch in the Township of Millburn



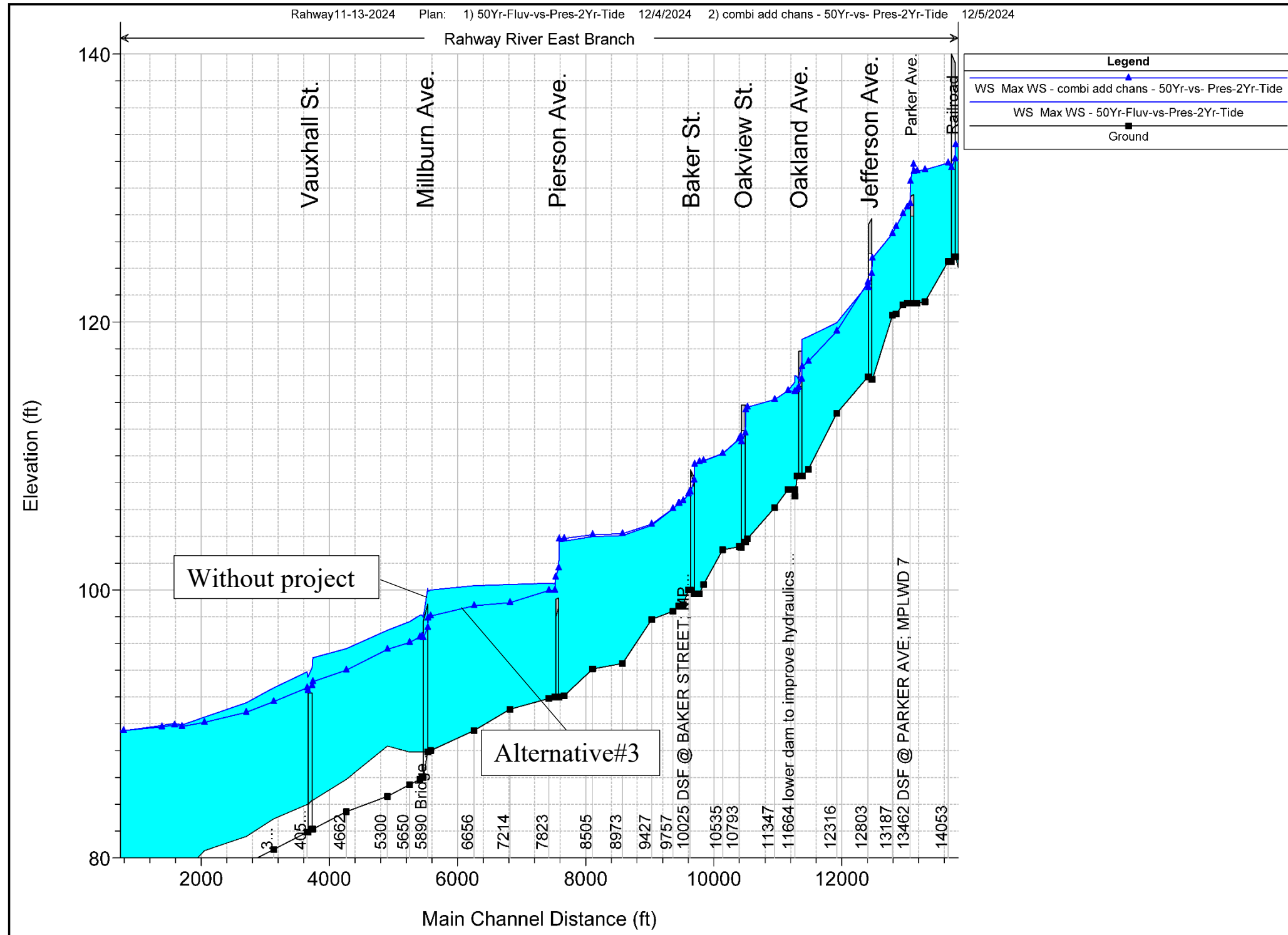


Figure 62. Computed water surface profile for Alternative #3: East Branch in the Township of Maplewood



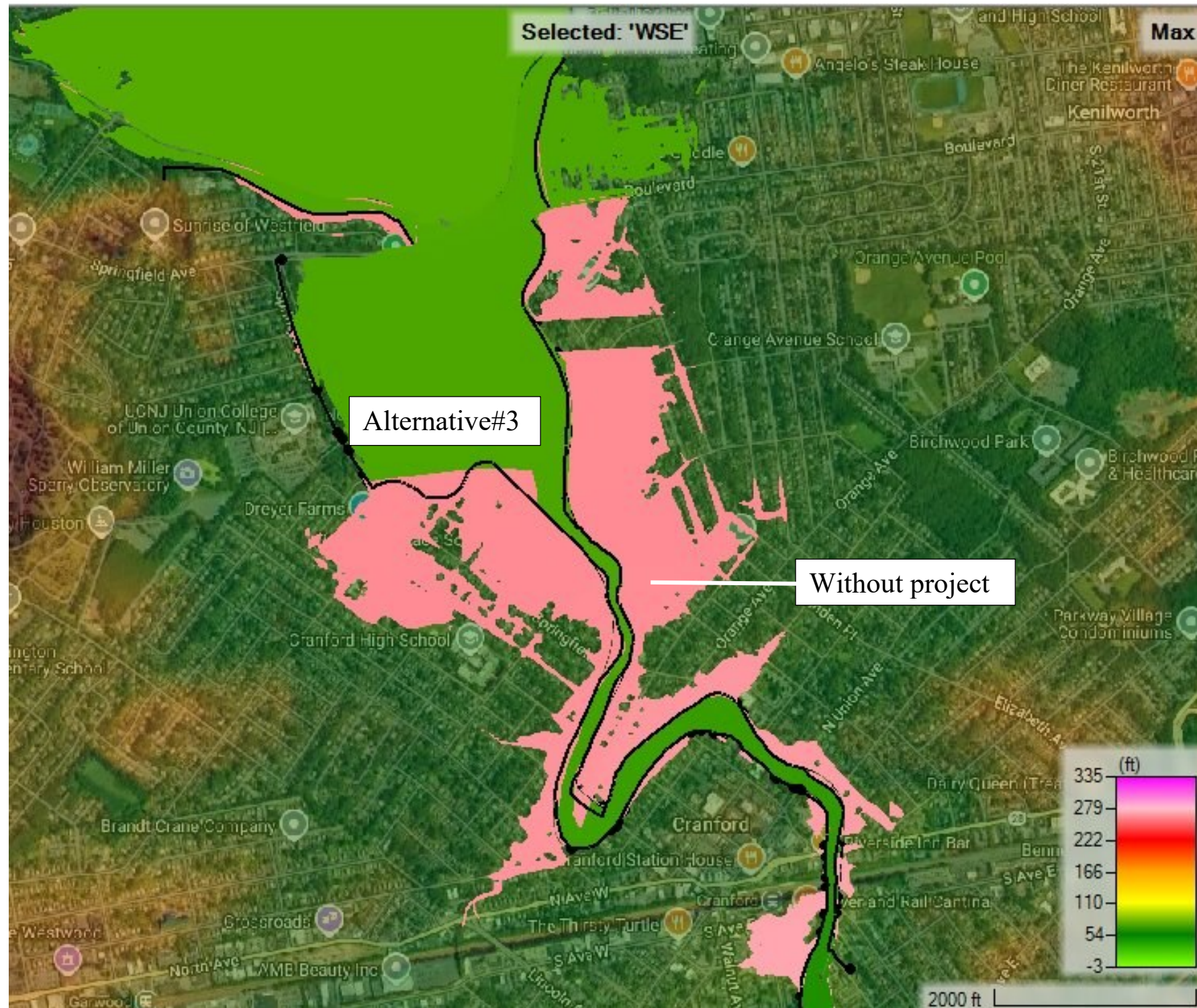


Figure 63. Computed inundation map for Alternative #3: Rahway River in the Township of Cranford

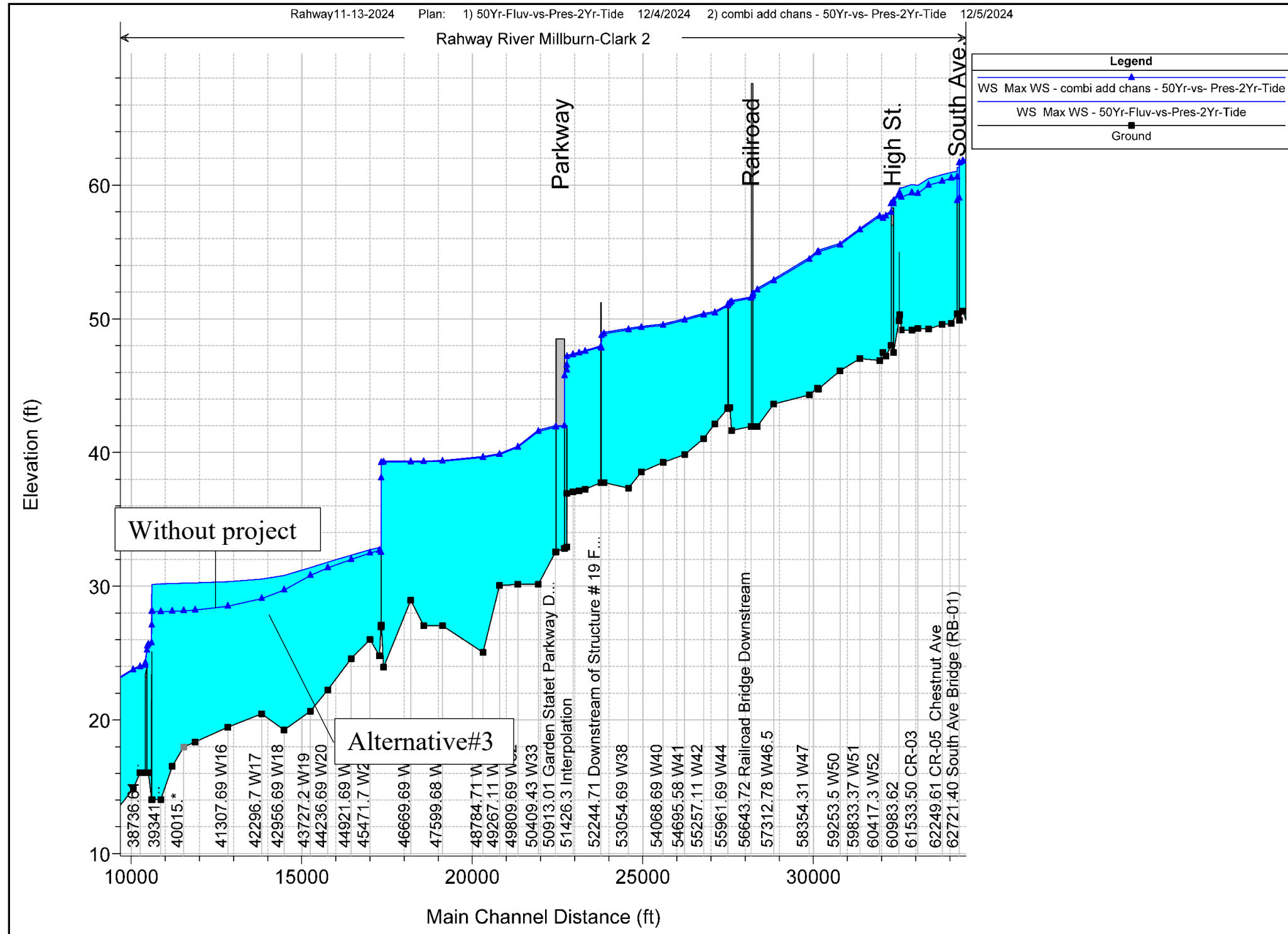


Figure 64. Computed water surface profile for Alternative #3: Rahway River in the Township of Cranford



4.3.3 *Alternative #5: Lenepe Park Detention*

4.3.3.1 Alternative #4 - Summary and Features

The details of this alternative are pending USACE input in the Engineering Appendix.

4.3.3.2 Alternative #5 - Hydraulic Analysis

The details of this alternative are pending USACE input in the Engineering Appendix.

In-Progress Review Draft, Subject to Change



4.4 Nonstructural Alternatives

4.4.1 *Alternative #4: Nonstructural Plan*

4.4.1.1 Alternative #4 - Summary and Features

The details of this alternative are pending USACE input in the Engineering Appendix.

4.4.1.2 Alternative #4 - Hydraulic Analysis

The details of this alternative are pending USACE input in the Engineering Appendix.



5.0 OPTIMIZATION OF THE TSP

5.1.1 *The Tentatively Selected Plan (TSP)*

This section is pending USACE input.

5.1.2 *Optimization of Alternative*

This section is pending USACE input.

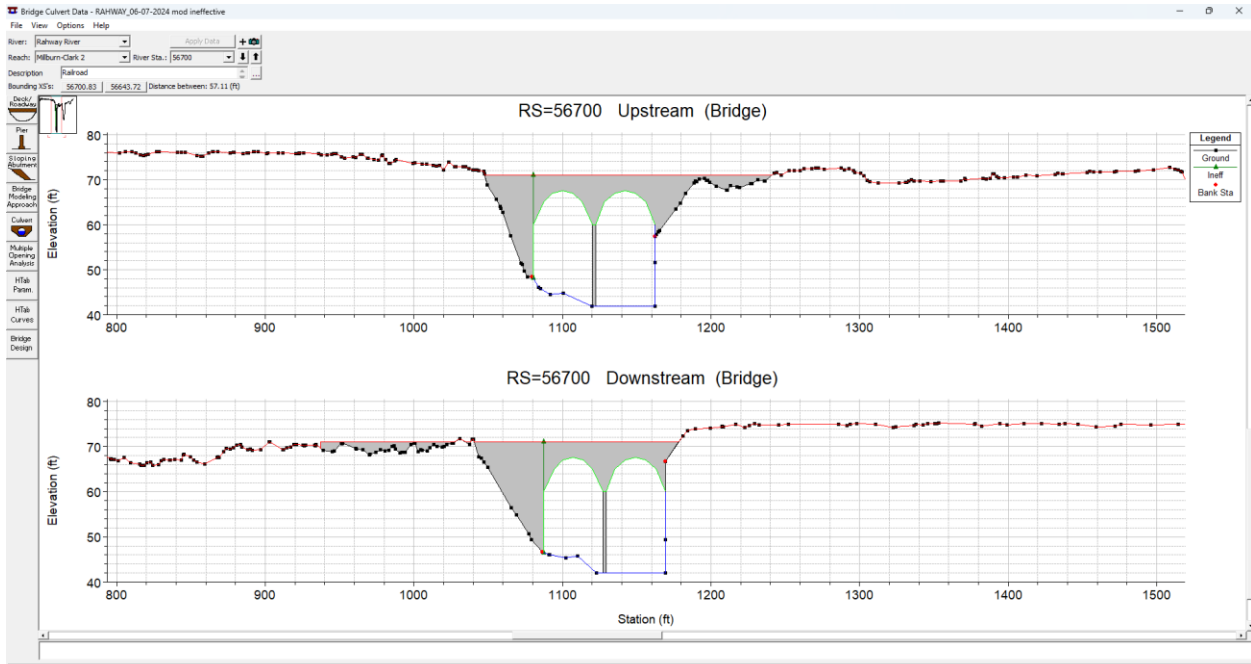


6.0 ATTACHMENTS

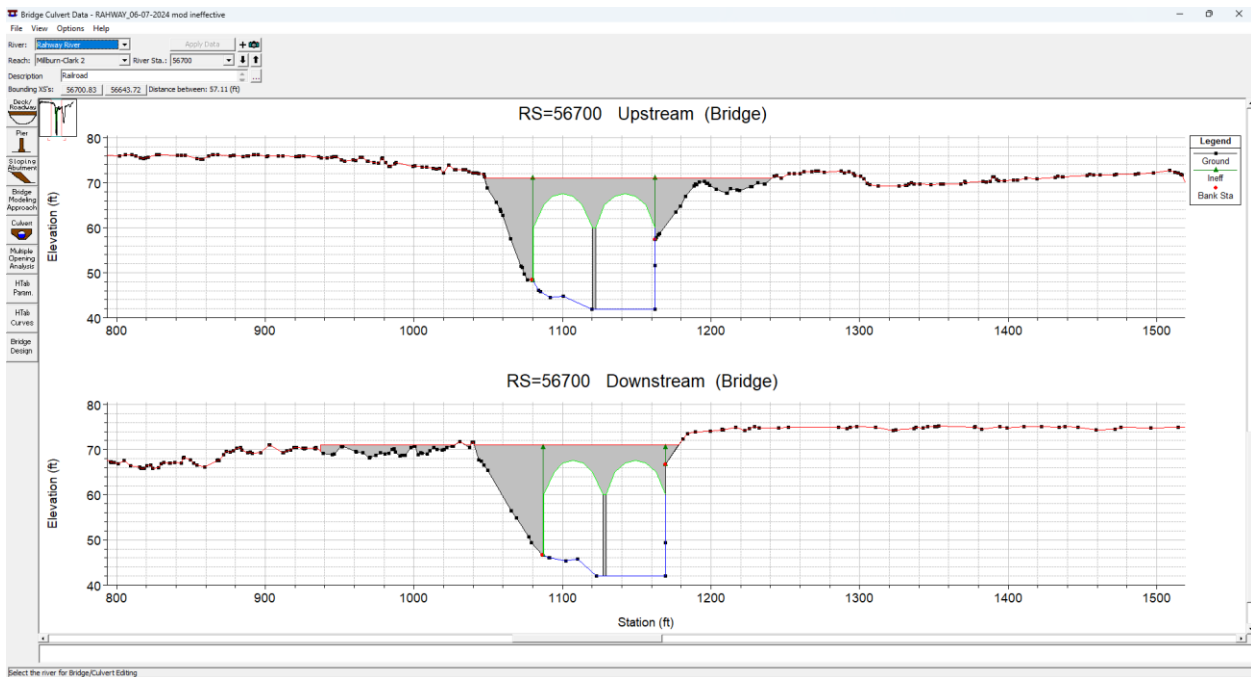
Table 26. Descriptions of plans in HEC-RAS model

Plan	Event/Alternative
Hurr XXX	Historical Event
XXYr-Fluv-vs-Pres-XXYr-Tide	Alternative 1 - No Action (Without Project)
XXYr-Fluv-vs-Pres-XXYr-Tide dam 1	Alternative 2 - Upstream Detention (Dry Detention)
XXYr-Fluv-vs-Pres-XXYr-Tide wet dam 2	Alternative 2 - Upstream Detention (Permanent Pool)
combi add chans - XXYr-vs-Pres-XXYr-Tide	Alternative 3 - Combination Plan





(a) Cross section of original ineffective areas



(b) Cross section of revised ineffective areas

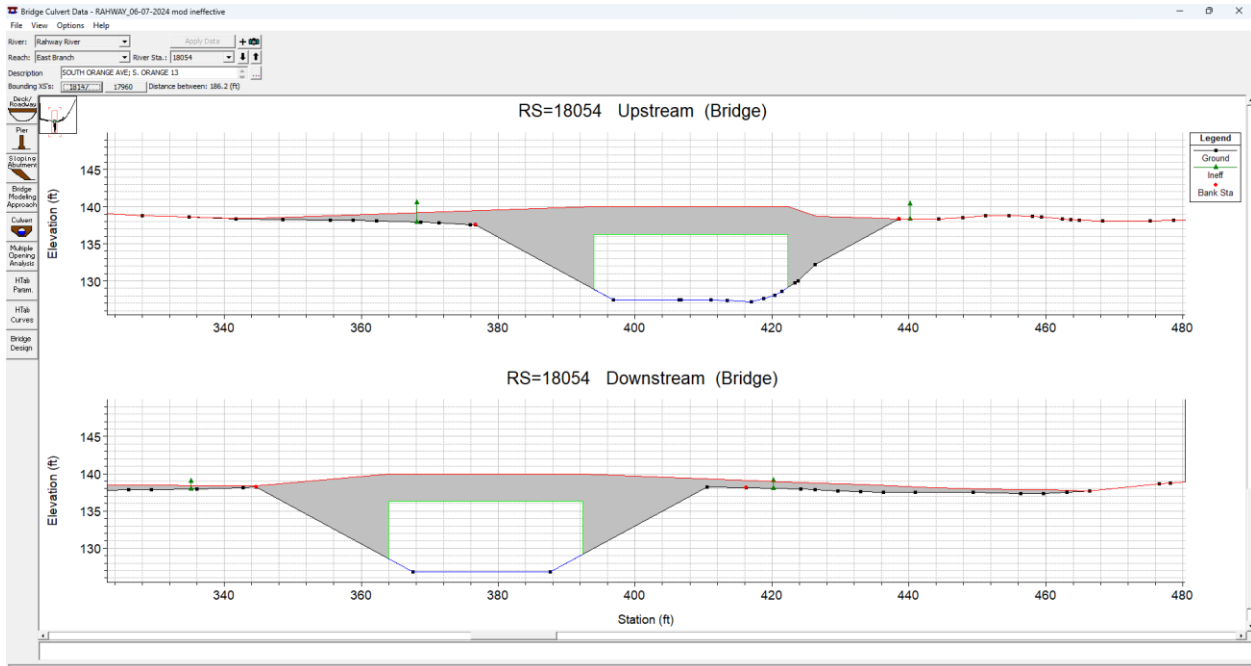


Millburn-Clark 2	58619.69	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6876.42	56.38
Millburn-Clark 2	58619.69	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6885.17	56.39
Millburn-Clark 2	58593.01	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6877.07	56.34
Millburn-Clark 2	58593.01	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6885.81	56.34
Millburn-Clark 2	58354.31	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6882.43	55.75
Millburn-Clark 2	58354.31	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6891.35	55.75
Millburn-Clark 2	57312.78	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6904.73	54.18
Millburn-Clark 2	57312.78	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6913.94	54.18
Millburn-Clark 2	56828.84	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6913.95	53.61
Millburn-Clark 2	56828.84	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6923.42	53.62
Millburn-Clark 2	56700.83	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6916.93	53.08
Millburn-Clark 2	56700.83	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6926.44	53.08
Millburn-Clark 2	56700			Bridge	
Millburn-Clark 2	56643.72	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6916.45	52.54
Millburn-Clark 2	56643.72	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6925.93	52.55
Millburn-Clark 2	56076.7	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6928.18	52.27
Millburn-Clark 2	56076.7	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6937.84	52.27
Millburn-Clark 2	56026.13	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6928.99	52.23
Millburn-Clark 2	56026.13	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6938.80	52.23
Millburn-Clark 2	56026			Bridge	
Millburn-Clark 2	55961.69	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6928.53	52.08
Millburn-Clark 2	55961.69	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6938.26	52.08
Millburn-Clark 2	55587.1	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	6934.39	51.64
Millburn-Clark 2	55587.1	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	6944.21	51.64

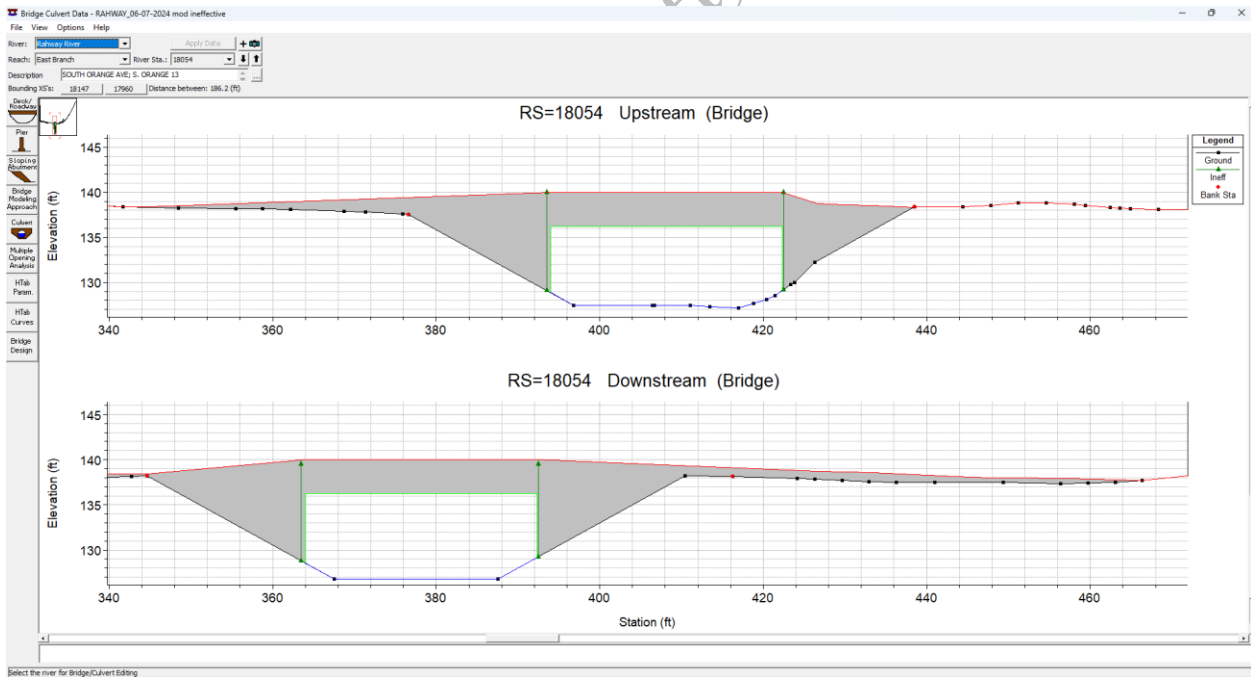
(c) Comparisons of WSE with and without modifications to ineffective areas.

Figure 65. Cross sections and WSE of Bridge 56700 on main stem: (a) cross section of original ineffective areas; (b) cross section of revised ineffective areas; (c) comparisons of WSE with and without modifications to ineffective areas





(a) Cross section of original ineffective areas



(b) Cross section of revised ineffective areas



East Branch	18793	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1464.74	138.39
East Branch	18793	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1478.72	138.31
East Branch	18465	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1507.94	138.30
East Branch	18465	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1522.63	138.20
East Branch	18367	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1520.56	138.20
East Branch	18367	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1535.44	138.10
East Branch	18188	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1543.75	137.99
East Branch	18188	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1558.95	137.88
East Branch	18147	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1549.08	138.05
East Branch	18147	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1564.37	137.79
East Branch	18054			Bridge	
East Branch	17960	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1494.07	137.18
East Branch	17960	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1511.30	136.99
East Branch	17915	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1496.36	136.99
East Branch	17915	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1515.06	137.03
East Branch	17881	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1497.43	136.95
East Branch	17881	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1516.08	136.98
East Branch	17862			Bridge	
East Branch	17842	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1497.43	136.78
East Branch	17842	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1516.08	136.81
East Branch	17810	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide	1498.29	136.76
East Branch	17810	Max WS	100Yr-Fluv-vs-Pres-2Yr-Tide ineff mod	1517.00	136.79

(c) Comparisons of WSE with and without modifications to ineffective areas

Figure 66. Cross sections and WSE of Bridge 18054 on East Branch: (a) cross section of original ineffective areas; (b) cross section of revised ineffective areas; (c) comparisons of WSE with and without modifications to ineffective areas



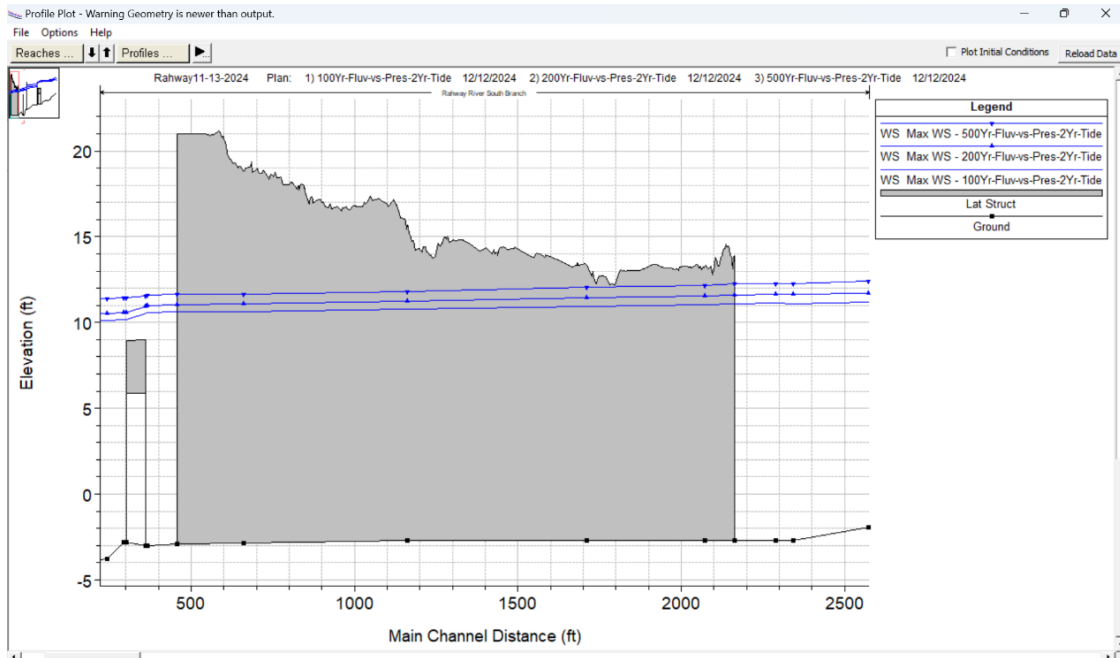


Figure 67. Profiles of East Branch lateral weir and water surface elevations

The weir on East Branch is not overtopped even under the 500-Yr fluvial conditions. Therefore, the weir coefficient does not have an impact.

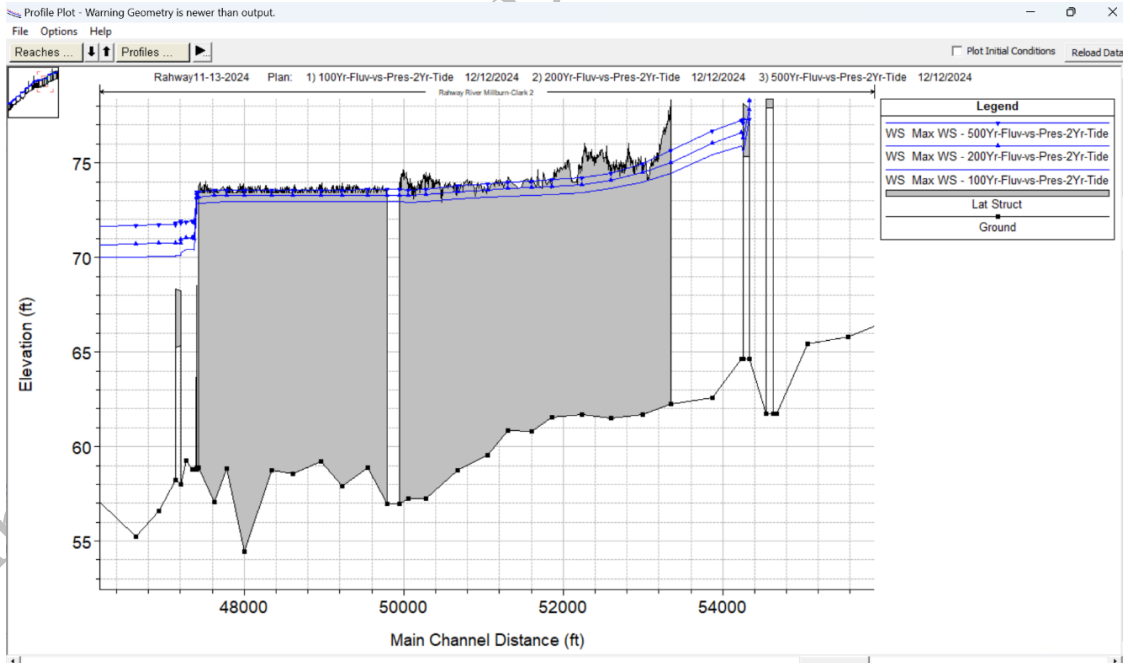


Figure 68. Profiles of the lateral weir in Kenilworth and water surface elevations



The weir in Kenilworth is not overtopped under the 100-Yr fluvial condition. The overtopping is partial and limited under the 200-Yr and 500-Yr fluvial conditions. Therefore, the weir coefficient has a minimal impact.

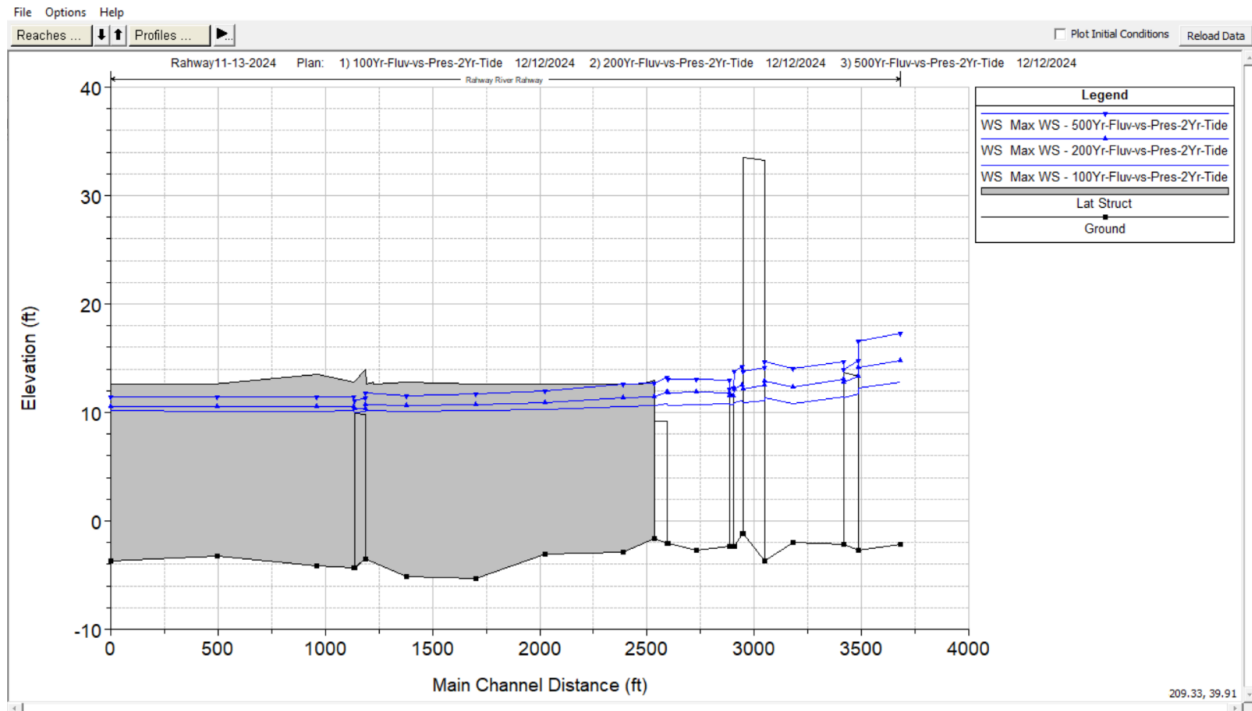


Figure 69. Profiles of the lateral weir in Rahway and water surface elevations

The weir in Rahway is not overtopped even under the 500-Yr fluvial conditions. Therefore, the weir coefficient does not have an impact.

