FINAL

Appendix B – Storm Surge and Flood Risk Models

For the Feasibility Study of Rebuild by Design Meadowlands Flood Protection Project

May 2021

Boroughs of Little Ferry, Moonachie, Carlstadt, and Teterboro and the Township of South Hackensack, Bergen County, New Jersey

REBUILD BY DESIGN
MEADOWLANDS

Prepared by AECOM for the State of New Jersey Department of Environmental Protection

Table of Contents

Subappendix B1: Coastal Flood Risk Model

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Table of Contents

List of Figures

List of Tables

Acronyms and Abbreviations

2D	Two-dimensional
3D	Three-dimensional
ADCIRC	Advanced Circulation
DEM	Digital Elevation Model
DHI	Danish Hydraulic Institute
EIS	Environmental Impact Statement
FEMA	Federal Emergency Management Agency
FM	Flexible Mesh
GIS	Geographic Information System
HD	Hydrodynamic
HUD	Department of Housing and Urban Development
LOP	Line of Protection
NAVD 88	North American Vertical Datum of 1988
NFL	National Football League
NJDEP	New Jersey Department of Environmental Protection
NJSEA	New Jersey Sports and Exposition Authority
NOAA	National Oceanic and Atmospheric Administration
RAMPP	Risk Assessment, Mapping, and Planning Partners
RBD	Rebuild by Design
SLR	Sea level rise
SW	Spectral Wave

US	United States
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
WSEL	Water surface elevation

1.0 Introduction

The coastal area of New Jersey experiences frequent flooding problems induced by coastal storm surge and high-intensity rainfall/runoff events. The State of New Jersey is implementing the Rebuild by Design (RBD) Meadowlands Flood Protection Project (the Proposed Project), which includes a flood resilience development plan for the Meadowlands District to reduce the environmental, social, and economic losses that result from flooding. A comprehensive flood resiliency study, supported by the United States (US) Department of Housing and Urban Development (HUD), is being conducted for the Meadowlands District as a result of the RBD competition. The Proposed Project requires the preparation of an Environmental Impact Statement (EIS) and Feasibility Study that evaluate the alternatives for implementing the Proposed Project. Each alternative's feasibility will be weighed based on cost, flood benefits, and impacts, as determined by coastal hydrodynamic modeling.

A preliminary coastal model system was developed to better understand the areas that are vulnerable to flooding from coastal storm surge and waves, and to evaluate the effectiveness of the proposed alternatives to reduce flooding. The preliminary coastal model system, which primarily utilizes the Danish Hydraulic Institute's (DHI) MIKE 21 software (Hydrodynamic (HD) Flexible Mesh (FM) Flow Module and Spectral Wave (SW) Module) with assistance of Advanced Circulation (ADCIRC) (to provide necessary boundary conditions), evaluated existing flood risks and subsequently analyzed the effects a proposed Line of Protection (LOP) (i.e., Alternative 1) on the regional flooding conditions. The focus of this preliminary coastal model system is the MIKE 21 HD FM, which simulated the effects of coastal storm surge within the Meadowlands District study area — an area that includes the lower reaches of the Passaic River, the Hackensack River, and North Newark Bay.

This report summarizes the development of this preliminary coastal model system to date, including the model setup, model calibration (including sensitivity tests), validation and the evaluation of the effects of a proposed LOP (Alternative 1). The report also discusses the status of this model system and its potential paths going forward.

1.1 Study Area

Figure B1-1 shows the Meadowlands District study area (light green outline) and Pilot Area 1 (orange outline). The Meadowlands District coastal model study area includes parts of the Boroughs of Teterboro, Carlstadt, Little Ferry, and Moonachie, and the Township of South Hackensack in Bergen County, New Jersey. These municipalities are situated between the Passaic River and the Hackensack River, which both drain into North Newark Bay. Pilot Area 1 is the focus area for flood risk protection and is encompassed by the Meadowlands District study area, which is the broader area considered in the EIS. The Pilot Area 1 would become known as the "Project Area" of the Proposed Project, and is referred to as such in this report. The coastal model study area (pink outline) is considered the coastal modeling domain or mesh. Bound by high ground, the numerical domain covers approximately 84 square miles and extends into areas that could potentially be impacted by coastal storm surge. The physiographic characteristics of the area include flat terrain in a typical urban setting, as well as areas of open water and forested and wetland areas. The highest ground elevation is approximately 45 feet (NAVD88), located in the northern section of the study area. The study area includes one of the largest wetlands in a major urban area within the United States.

Figure B1-1: Pilot Area 1 / Project Area, Meadowlands District Study Area, and the Mesh Area

Based on the Land Use Map developed by the New Jersey Department of Environmental Protection (NJDEP), the predominant land use in the Meadowlands District study area is Industrial and Commercial Facilities, followed by Residential, Transportation, and Communication Facilities. **Figure B1-2** displays the land use distribution in the Meadowlands District study area for the year 2012 and **Table B1-1** lists the land use classes and their distribution across the study area.

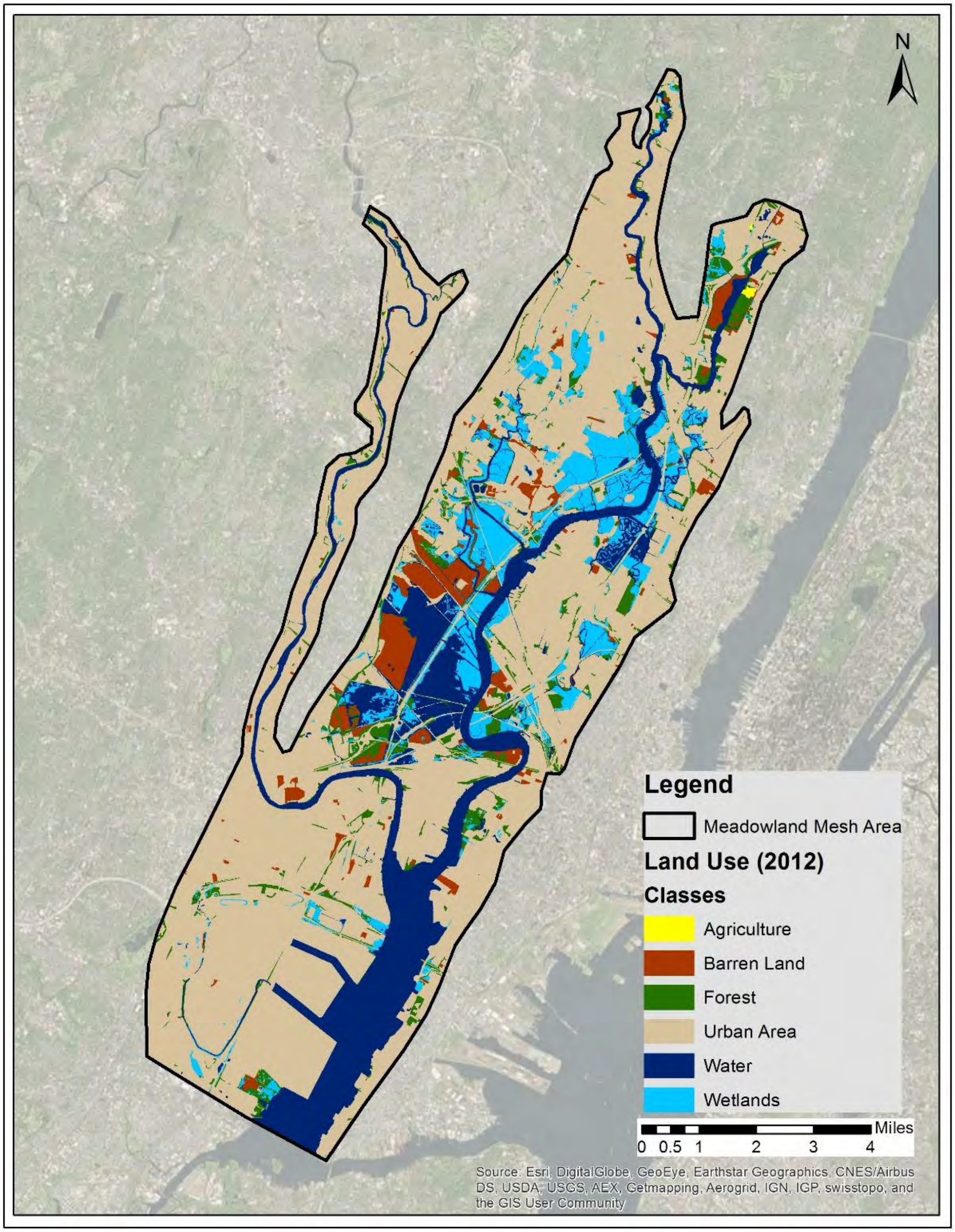


Figure B1-2: Land Use in Meadowlands District Study Area in the Year 2012 (source: NJDEP 2012 Land Use / Land Cover)

Table B1-1: Land Use Distribution in the Meadowlands District Study Area

Land Use Classification (assigned by NJDEP)	Area (square mile)	Percent (%) of Total Area
Residential	18.49	22
Industrial and Commercial Areas	19.60	23
Transportation and Communication Facilities	16.90	20
Forest	3.73	4
Barren land, Agriculture, and Cemetery	4.00	5
Wetland	7.04	8
Water	14.29	18
Total Area	**84.05**	**100**

1.2 Objectives

The primary objectives of the coastal modeling work are to develop a coastal model system to:

- Establish the extent and elevations of flooding for existing/baseline conditions;
- Assess the performance of Proposed Project alternatives under design storms; and
- Provide various support (such as flood risk assessment, drainage, structure, EIS, etc.) for the planning, analysis, design, and implementation of the Proposed Project alternatives.

The goal of this subappendix is to demonstrate that the coastal model system is correctly calibrated and validated and equipped to perform production runs.

2.0 Development of Coastal Model System

Numerical mathematical models are commonly used in engineering practice, as they provide a convenient and reliable method for comparing the Proposed Project alternatives to existing conditions (baseline) under different combinations of coastal storm surges, rainfall/runoff events, and sea level rise (SLR). For the Proposed Project, the preliminary coastal model system consists of a regional storm surge model ADCIRC and local models of MIKE 21 HD FM and SW Module. The development processes for this coastal model system are summarized in the subsections below.

2.1 Regional ADCIRC Coastal Storm Surge Model

Storm surge is a long-period wave caused by extreme wind and pressure forces. Water heights associated with storm surge are superimposed on water levels generated by tidal forcing. Past research and model experiences illustrate that domain size has considerable effects on the accuracy of storm surge predictions.

For the Proposed Project, the two-dimensional ADCIRC coastal storm surge model developed as part of the Federal Emergency Management Agency's (FEMA's) recently completed New York/New Jersey storm surge study to provide boundary conditions for the synthetic storms in terms of different return periods was utilized. The model domain extends from 97.85° to 60.04° W and from 7.90° to 45.83° N, encompassing the Western Atlantic, the Gulf of Mexico, and the Caribbean Sea (**Figure B1-3**). FEMA's data sources are outlined below:

- Topography Sources: US Geological Survey (USGS) Earth Resources Observation and Science Center, NGA, FEMA, and the New York City Department of Information Technology and Telecommunications.
- Bathymetry Sources: National Oceanic and Atmospheric Administration (NOAA) National Geophysical Data Center, NOAA Office of Coast Survey, US Army Corps of Engineers (USACE) New York and Philadelphia Districts, New York State Geographic Information System (GIS) Clearinghouse, and the NJDEP.

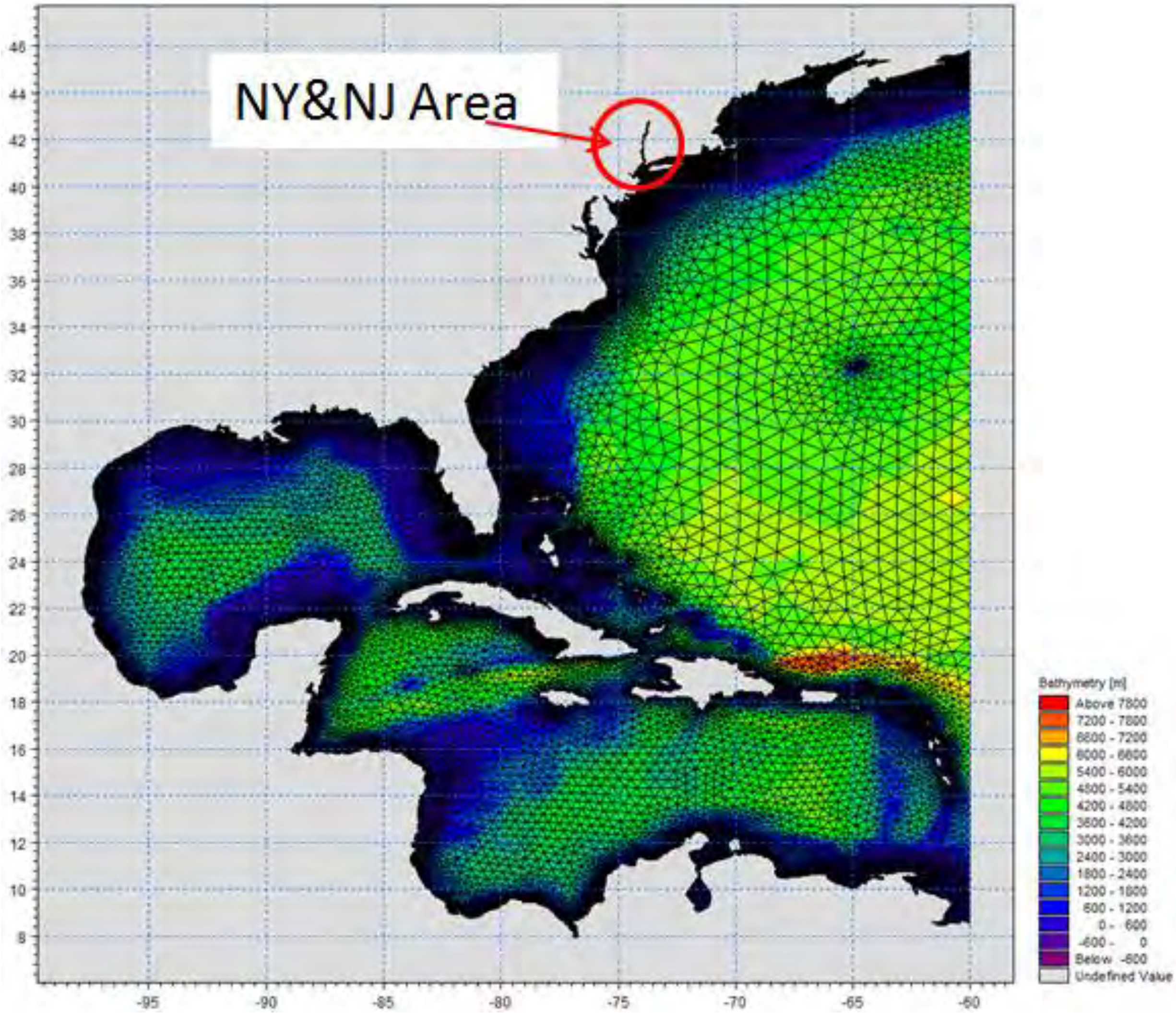


Figure B1-3: Numerical Grid of Model Domain of ADCIRC Model

ADCIRC is used to establish boundary conditions for the MIKE 21 models for storm events. Comparisons of ADCIRC model-predicted storm surge time series to measured surges at NOAA Bergen Point Tidal Station, The Battery Tidal Station, and Sandy Hook Tidal Station are shown in the **Figure B1-4**, **Figure B1-5**, and **Figure B1-6**. Hurricane Sandy was chosen as the validation storm because it is one of the most destructive storms in this region's history and the storm with the most reliable field records of the extent of flooding.

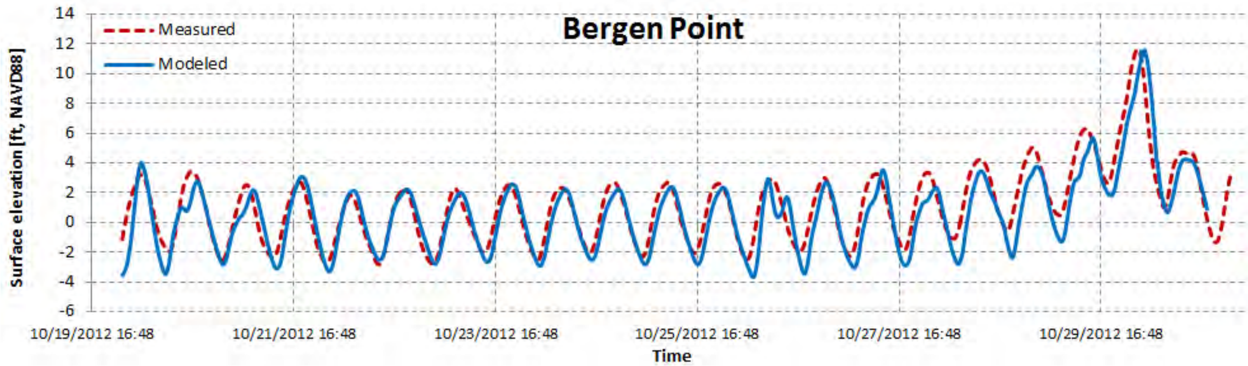


Figure B1-4: Comparison of ADCIRC Model Simulated Surge with Measured Surge during Hurricane Sandy at NOAA Bergen Point Tidal Station

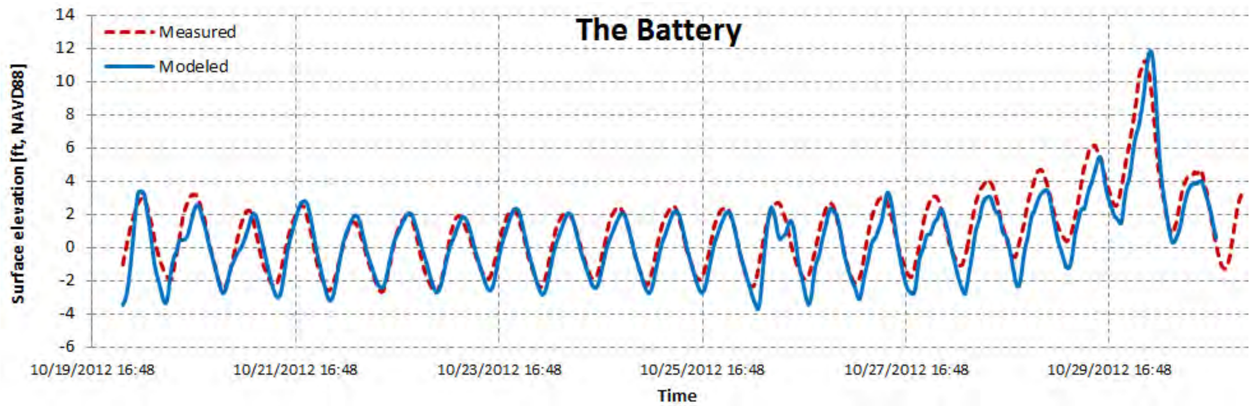


Figure B1-5: Comparison of ADCIRC Model Simulated Surge with Measured Surge during Hurricane Sandy at NOAA The Battery Tidal Station

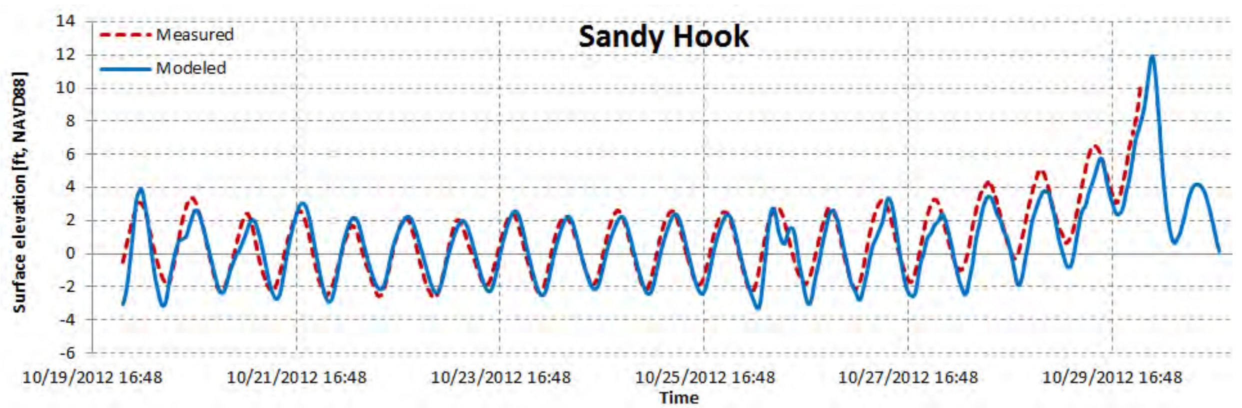


Figure B1-6: Comparison of ADCIRC Model Simulated Surge with Measured Surge during Hurricane Sandy at NOAA Sandy Hook Tidal Station

From **Figure B1-4**, **Figure B1-5**, and **Figure B1-6**, it can be seen that the model results show close agreement between model-predicted and observed storm surge elevations.

As noted above, for the Proposed Project, the simulation results from the ADCIRC model were used to provide boundary conditions of synthetic storms with different return periods for local MIKE 21 HD FM Model. The boundary conditions of the calibration with tide were based on the NOAA measurement of water level at Bergen Point and USGS measurement of discharge at Hackensack River and Passaic River.

2.2 MIKE 21 HD FM Model

The MIKE 21 HD FM, a two-dimensional (2D) hydrodynamic model package developed by DHI, is selected as the tool to simulate the storm surge water levels and the waves for the Proposed Project.

The existing FEMA ADCIRC model covers a much larger region and is only calibrated at major NOAA tidal stations (including NOAA Bergen Point Tidal Station). The region of the proposed model domain for the Proposed Project was not calibrated. The current FEMA ADCIRC model has 1.2 million elements; if the existing ADCIRC model is used and further refined at the study area, the computing power and time required for each run would dramatically increase and it would be unlikely that it could be completed within the Proposed Project schedule. MIKE 21 has powerful pre- and -post processing capability (time-saving) and the capacity to implement many hydraulic structure types in the model for flood assessment, whereas ADCIRC only has limited capability for structure type. The primary effort in this coastal model system work is the development of MIKE 21 HD FM, which includes model setup, model calibration, and model validation.

2.2.1 Model Setup

Topographic and bathymetric data are critical to the development of any hydrodynamic model. For the Proposed Project, significant efforts were made to employ the most up-to-date data available, which includes:

- Hackensack Meadowlands Digital Elevation Model (DEM) by Quantum Spatial, Inc. 2014;
- USACE North Atlantic Coast Comprehensive Study (NACCS) DEM, 2015;
- Topographic survey for hydraulic and hydrology model of Hackensack River by DIMCO, INC, 2000; and
- Berry's Creek DEM from Berry's Creek Remediation Group Section Survey at Berry's Creek.

Figure B1-7 and **Figure B1-8** below show an overview and closer view of the three-dimensional (3D) rendering of topographic and bathymetric data used in the model. It should be noted that the vertical dimension is scaled up with a factor of "5" for visualization purposes (the legend values in **Figure B1-7** and **Figure B1-8** remain the true elevations in North American Vertical Datum of 1988 (NAVD 88)).

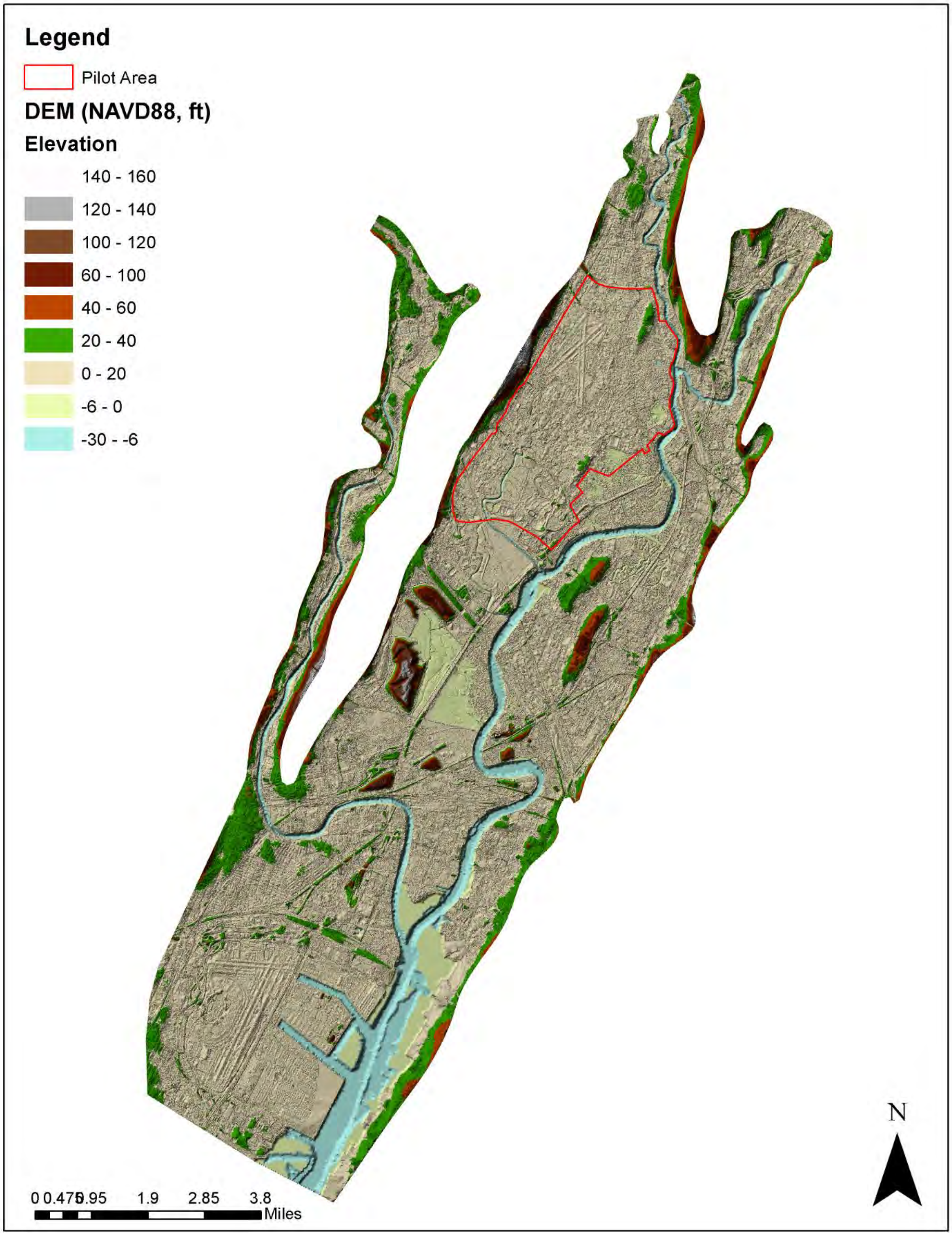


Figure B1-7: Overview of the 3D Render of Topographic and Bathymetric Data Used in the Model

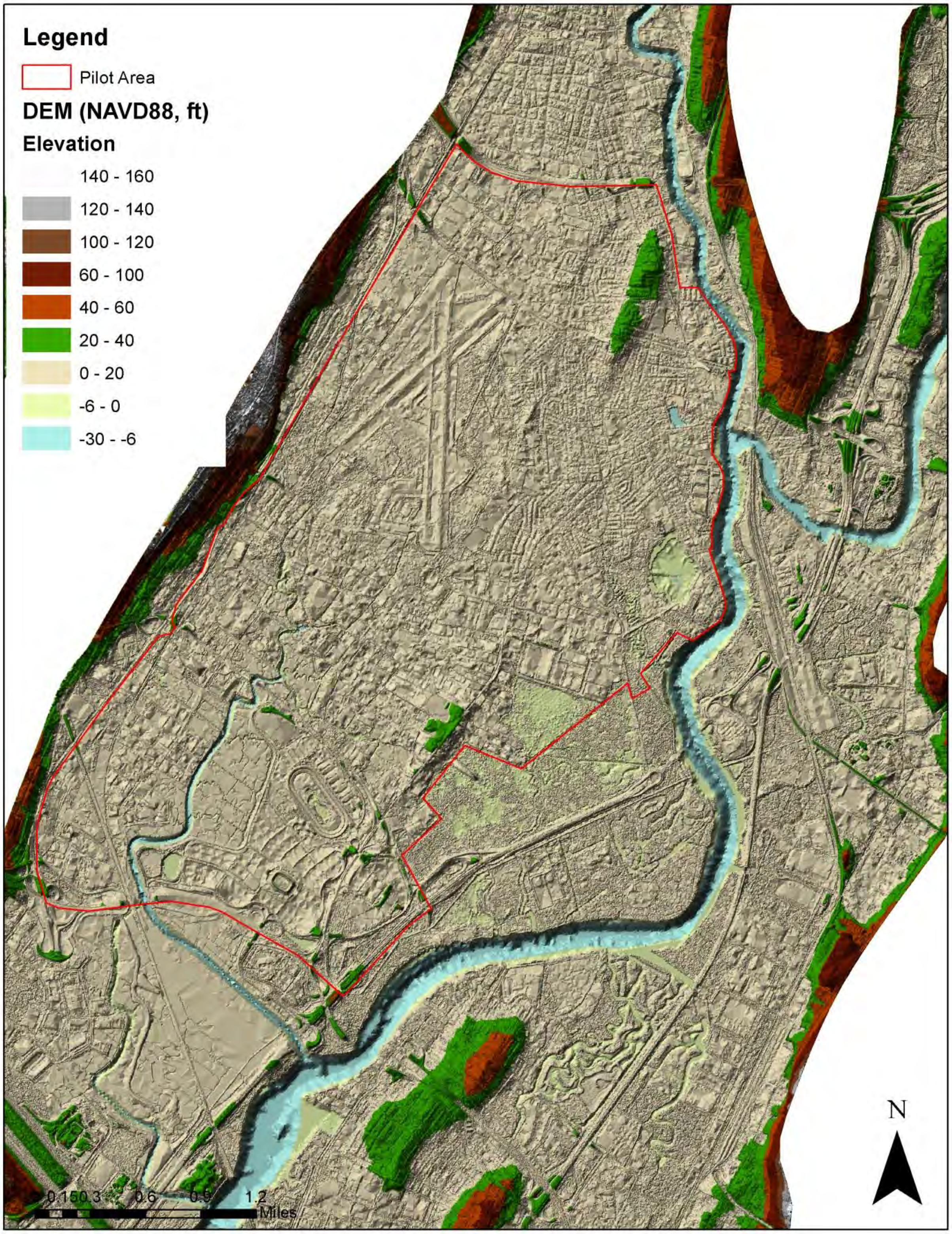


Figure B1-8: Closer View of the 3D Render of Topographic and Bathymetric Data Used in the Model

The preliminary coastal hydrodynamic model was prepared using a flexible mesh for the study area (shown in pink in **Figure B1-1**). **Figure B1-9** below shows the bathymetry map (with NAVD88 as the datum) used in the model. The model mesh was created to capture the required details of urban areas within the Meadowlands District study area and would become coarser in the periphery and for areas outside the Project Area (shown in orange in **Figure B1-1**). **Figure B1-10** shows the refined mesh in the Project Area of the Meadowlands District study area. The current model setup includes small creeks with tide gates.

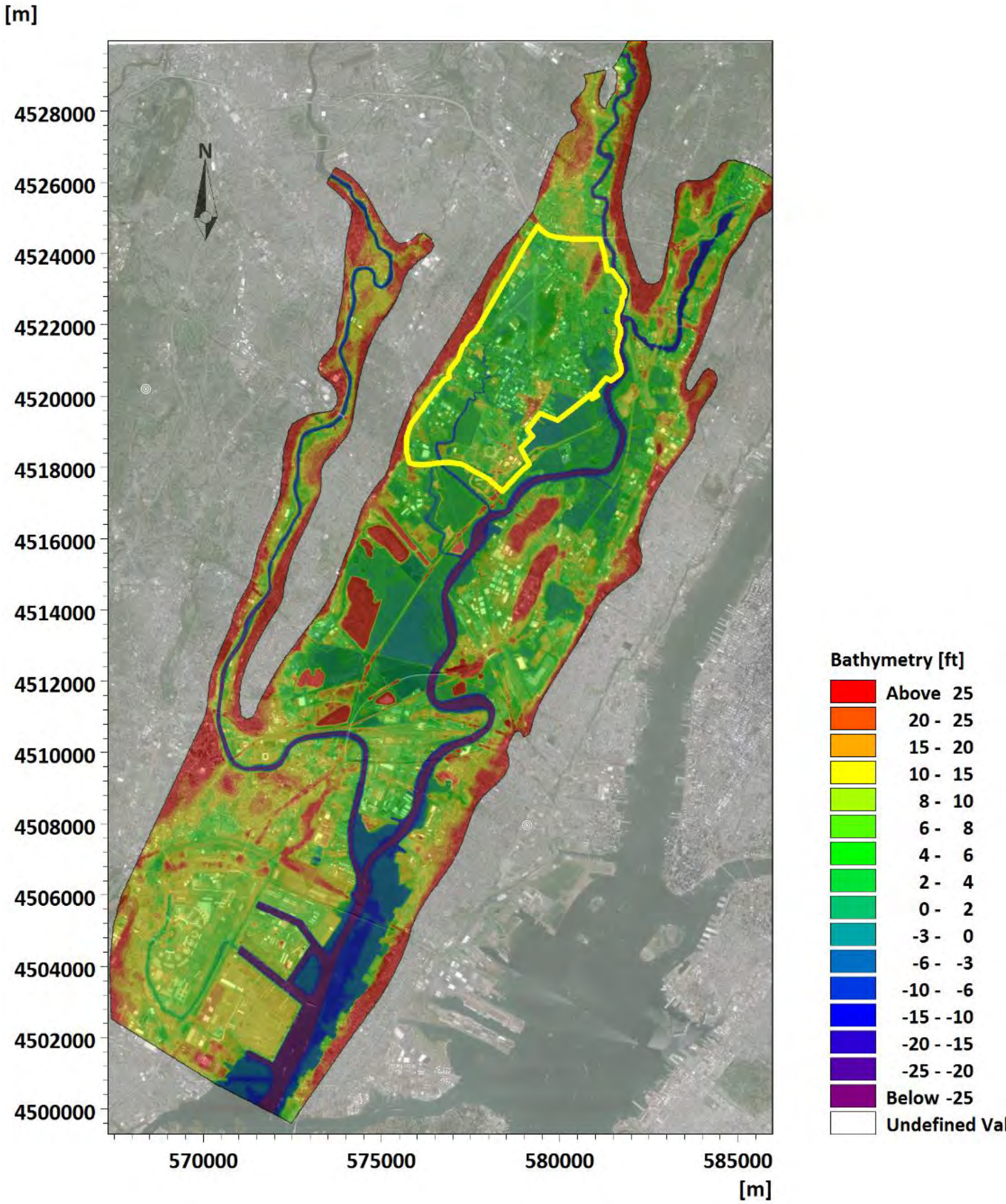


Figure B1-9: MIKE 21 HD FM Bathymetry Map

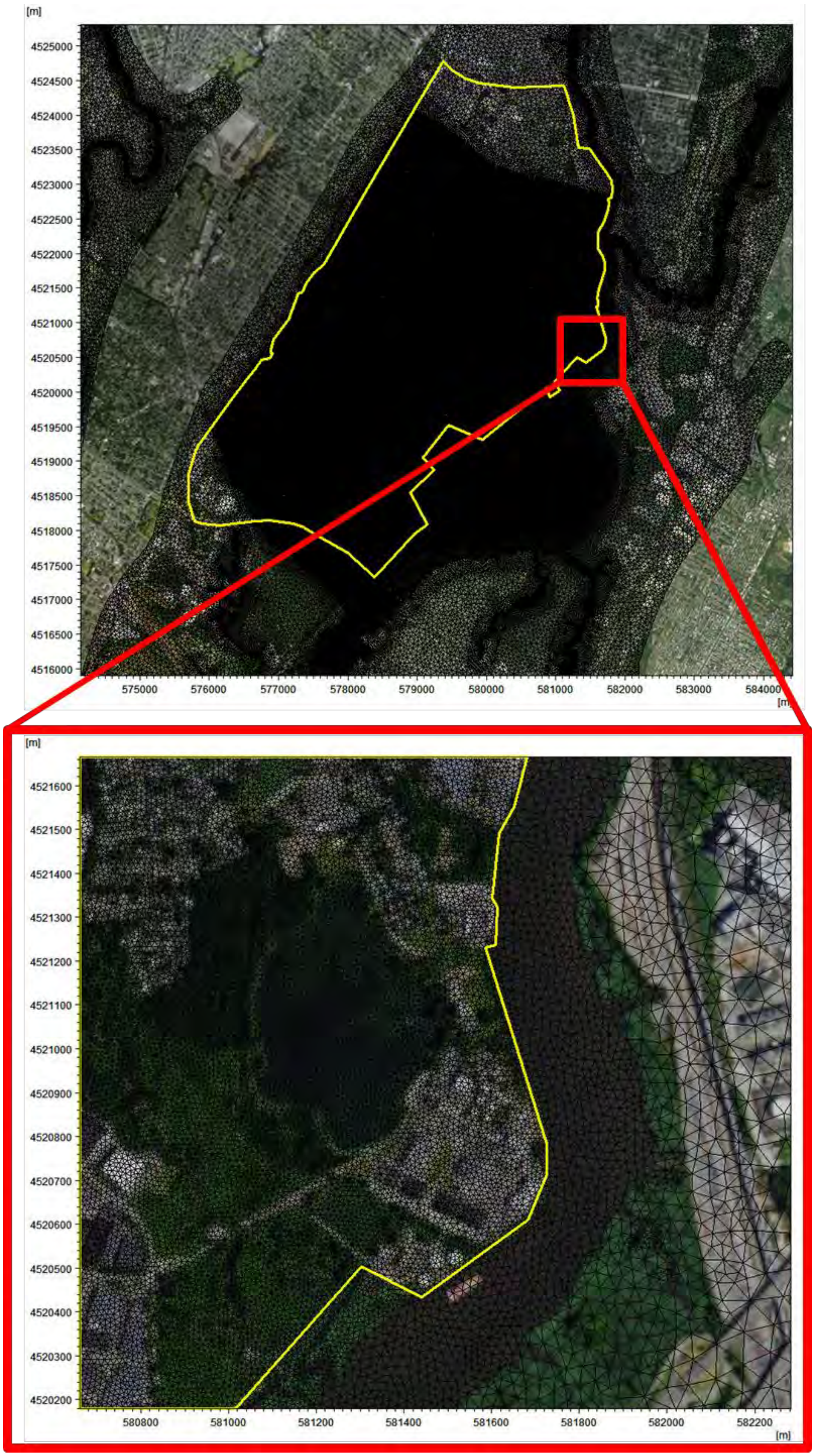


Figure B1-10: Refined Mesh in the Project Area of the Meadowlands District Study Area

The model setup parameters are summarized in **Table B1-2** below.

Table B1-2: Summary of Model Setup Parameters

Parameter	Value / Note
Study area for mesh	84 square miles
Model mesh	about 0.7 million elements; mesh is in the order from 3 m to 80 m
Model time step	Maximum time step interval: 30-second (frequency of output) Time step for hydrodynamic model: dynamic; each determined to satisfy stability criteria (CFL<0.8)
Boundary conditions	Downstream/ocean boundary: time series of surface water elevations; the ocean boundary conditions for tidal time series are based on measured time series at NOAA Bergen Point Tidal Station; for time series of storm events, the outputs of the ADCIRC model with 1-minute frequency are used as ocean boundary conditions Upstream boundary: discharges for the Passaic River and the Hackensack River by scaling down the real time-series of Hurricane Irene recorded at USGS gauge at Dundee Dam and USGS gauge 1378500 to the HMS-modeled 2-year discharge value
Flood and dry	Drying depth: 0.005 m (the element is removed from the simulation if its water depth is less than the drying depth) Flooding depth: 0.05 m (used to determine when an element is flooded and should re-enter the simulation) Wetting depth: 0.1 m (the element is considered wet and is included in the simulation if its water depth exceeds the wetting depth)
Density	Barotropic
Bed roughness	Manning's M (1/Manning's n); varying in domain
Horizontal eddy viscosity	Smagorinsky coefficient: 0.28 (determined through calibration with different values)
Coriolis forcing	Not included in the model due to domain size
Tidal potential	Not included in the model due to water depth and size of the domain
Precipitation-Evaporation	Not included in the model at this time
Wind forcing	Domain varying
Wave radiation	Not included in the model at this time because the waves present are too weak
Structures	Dikes and a tidal gate are included in the model at this time. The operation of a tide gate in the model is controlled by the water level at a control point set downstream of the tide gate. During a tidal cycle, when the water level at the control point rises above (drops below) 0 feet (NAVD88), the tide gate closes (opens). The open and close time intervals of the tide gate are both set to be 5 minutes. The dikes are implemented as thin dams with elevation matching the actual elevation of the structures, and overtopping in both directions is allowable depending on the upstream and downstream water levels.

Parameter	Value / Note
	Different types of structures (such as weir, culverts, etc.) can be implemented in the model as needed.
Salinity	Not included in the model because all the flood mitigation alignments are on land and should not cause long term changes in the salt distribution in the Hackensack River; there would be some changes in the salinity patterns during a surge event as flood waters are deflected by the alignments, but it would be short-lived (on the order of hours). If the alignments are in the Hackensack River itself, then salinity issues become notable and 3D baroclinic modeling using MIKE 31 as an example would be required.

During model setup, the bed roughness map was created using the Manning's n-values categorically assigned to the Land Use data downloaded from the NJDEP website. The NJDEP's Land Use GIS data were re-classified as nine different land use classifications to be used in the coastal model. The Manning's n-values corresponding to land use classification was assigned based on the literature and on published Manning's n-values from a coastal storm surge study conducted for FEMA by Risk Assessment, Mapping, and Planning Partners (RAMPP) (RAMPP 2014a, RAMPP 2014b, RAMPP 2014c). The Manning's n-values are further adjusted based on the overall comparison of modeled and measured water level. The Manning's n-values that are chosen are those that give the best overall comparison of modeled and measured water level and also stay within the reasonable range of empirical values. **Table B1-3** summarizes the land use classifications and Manning's n-values used in the model setup for the initial run.

Table B1-3: Land Use Classifications and Manning's n-Values for Initial Run

Land Use Name	Land Use Classification	Initial Value for Model Setup
Residential I	Residential High Density or Multiple Dwelling, Residential Single Unit Medium Density	0.10
Residential II	Residential Rural Single Unit, Residential Single Unit Low Density, Airport Facility, Other Built-up, Mixed Urban	0.05
Industrial & Commercial	Industrial and Commercial Complexes	0.15
Transportation & Communication Facilities	Transportation and Communication Facilities, Major Road, Railways, Recreational Land, Athletic Fields, Mixed Transportation Corridor, Upland Right of Ways, Stormwater Basin, Stadium Cultural Centre Zoo	0.025
Barren land	Barren Land	0.04
Other Agriculture	Other Agriculture, Cemetery	0.035
Forest	Forest	0.20
Wetland	Wetlands	0.06

Land Use Name	Land Use Classification	Initial Value for Model Setup
Water	Water	0.0313

Figure B1-11 shows the final Manning's n-values map used for the model runs (including model calibration and validation runs).

Model sensitivity tests were also conducted for related parameters in **Table B1-2** for model set-up and model performance assessment.

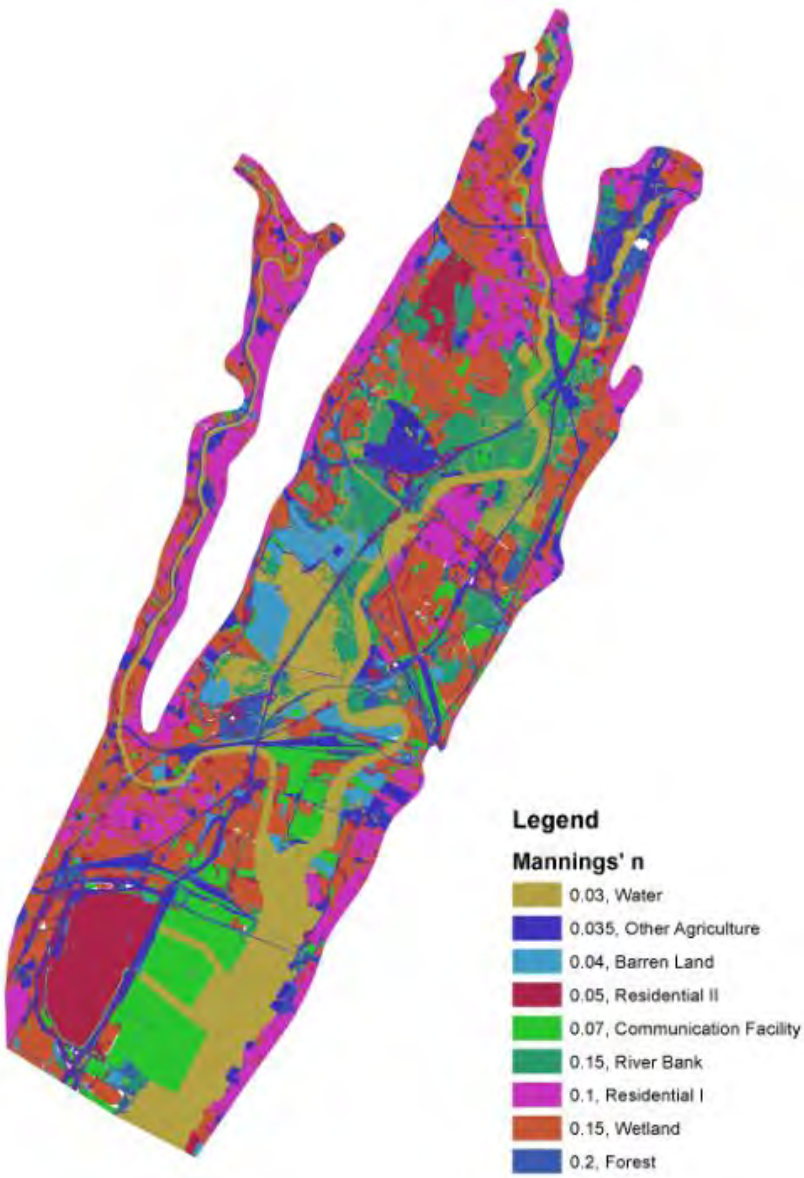


Figure B1-11: Final Manning's n-values Map

Significant efforts were made to optimize the bathymetry and mesh quality, which included:

- Further assessed and improved topographic and bathymetric data, such as incorporating new bathymetry data, topo, mitigation bank and tide gates, and checking hydraulic connection and hot spots;
- Conducted further mesh refinement and improvement;
- Assessed and re-assigned refined Manning roughness coefficients to the model domain;
- Performed sensitivity tests for different model parameters.

Figure B1-12 through **Figure B1-16** demonstrate some of the work completed during model calibration. Detailed bathymetry from the local survey in the vicinity of NFL facilities and the tide gates at East Riser, West Riser, Losen Slote, Moonachie, and Rutherford has been incorporated into the model. **Figure B1-12** shows the distribution of the local survey data at the East Riser and West Riser tide gates and the NFL facilities next to Berry's Creek. **Figure B1-13** presents the bathymetry adjustment required to represent the channel next to the Losen Slote tide gate. Kane Mitigation Bank is added to the model as a dike and its layout is shown in **Figure B1-14**. Tide gates at West Riser, East Riser, Peach Island Creek, Losen Slote, and DePeyster Creek are added to the model as dikes with the elevations shown in **Figure B1-14**.

The complexity of the bathymetry in the Meadowlands District study area shows in the existence of many small creeks. Correct representation of these small creeks in the model is critical since the creeks affect the drainage process after a storm peak passes. Failure to incorporate the creeks in the model may result in storm surge water trapped like a "pond" (see **Figure B1-15**), and consequently, the impact of a proposed Line of Protection (LOP) may be exaggerated. Efforts have been made in improving the incorporation of creeks in the model (like the small creek next to the Rutherford tide gate, as shown in **Figure B1-16**).

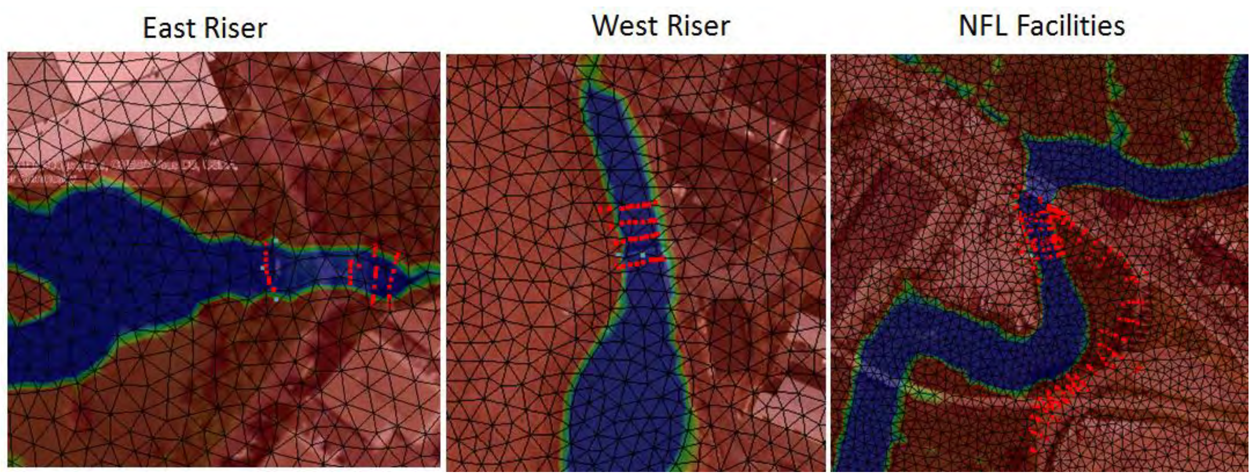


Figure B1-12: Bathymetry and Mesh Improvement at East Riser, West Riser, and NFL Facilities

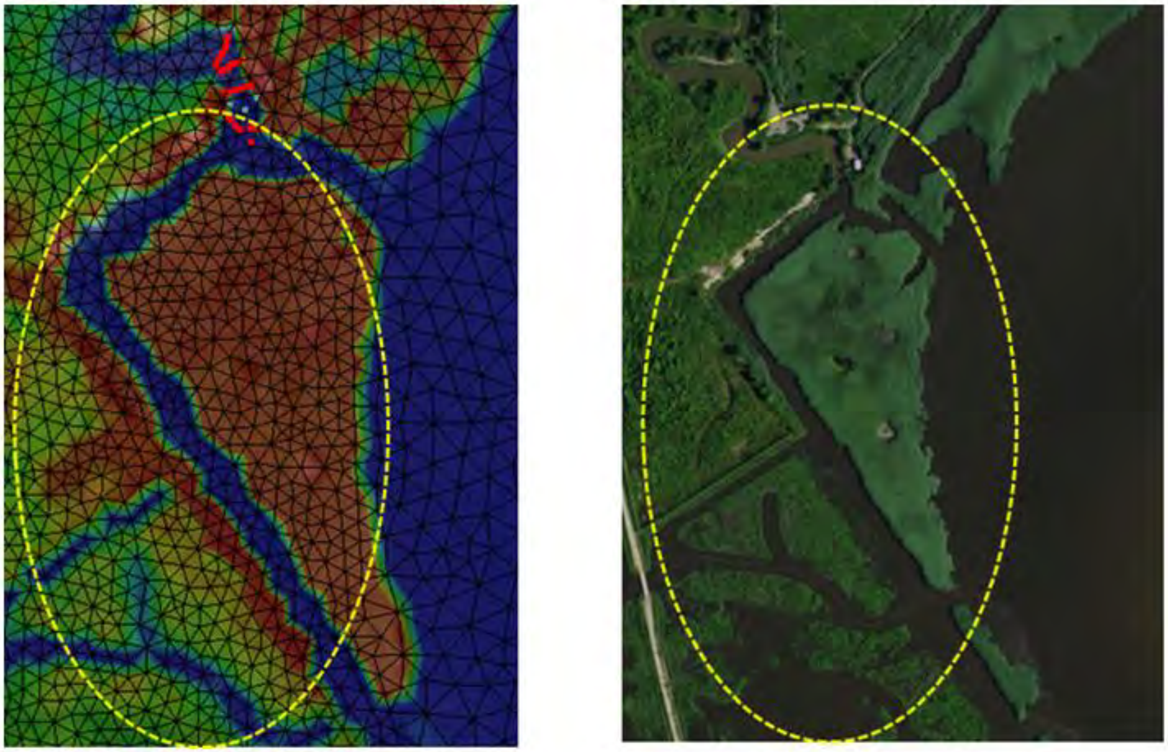
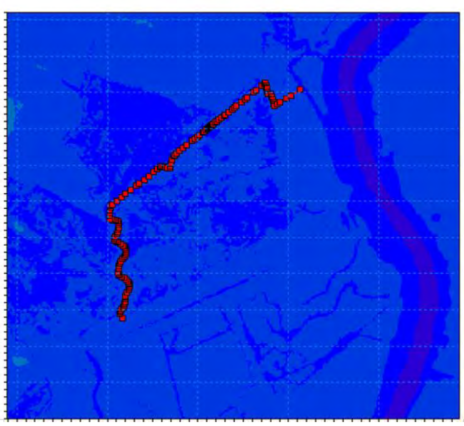


Figure B1-13: Bathymetry and Mesh Improvement in Losen Slote Area

Kane Mitigation Bank



Tide Gates

Tide Gate	Weir Elevation (ft - NAVD88)
West Riser	5
East Riser	6
Peach Island Creek	4
Losen Slote	11
DePeyster Creek	6

Figure B1-14: Incorporating Existing Structures

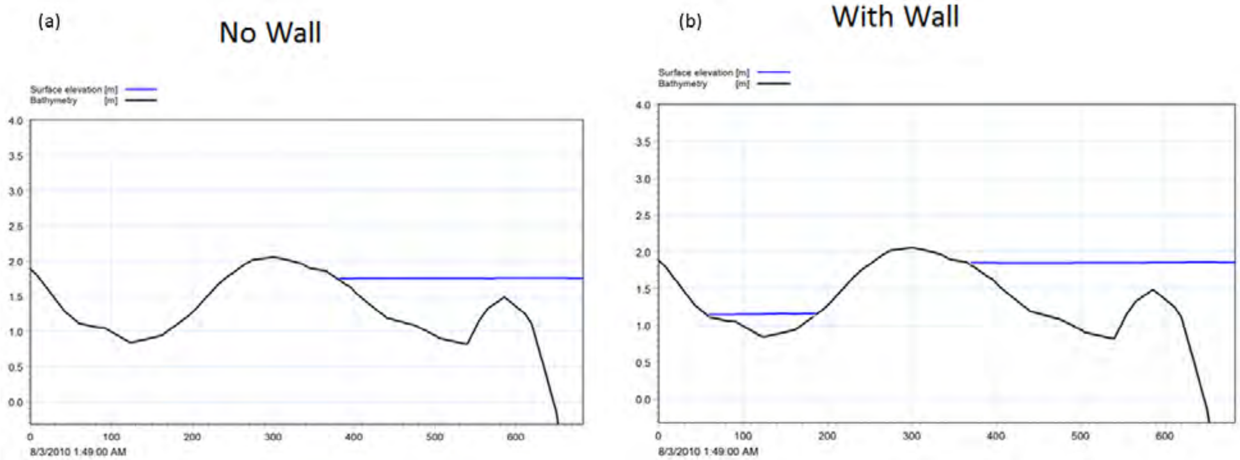


Figure B1-15: Hot Spot Check at Location near the Tide Gate at East Rutherford, NJ

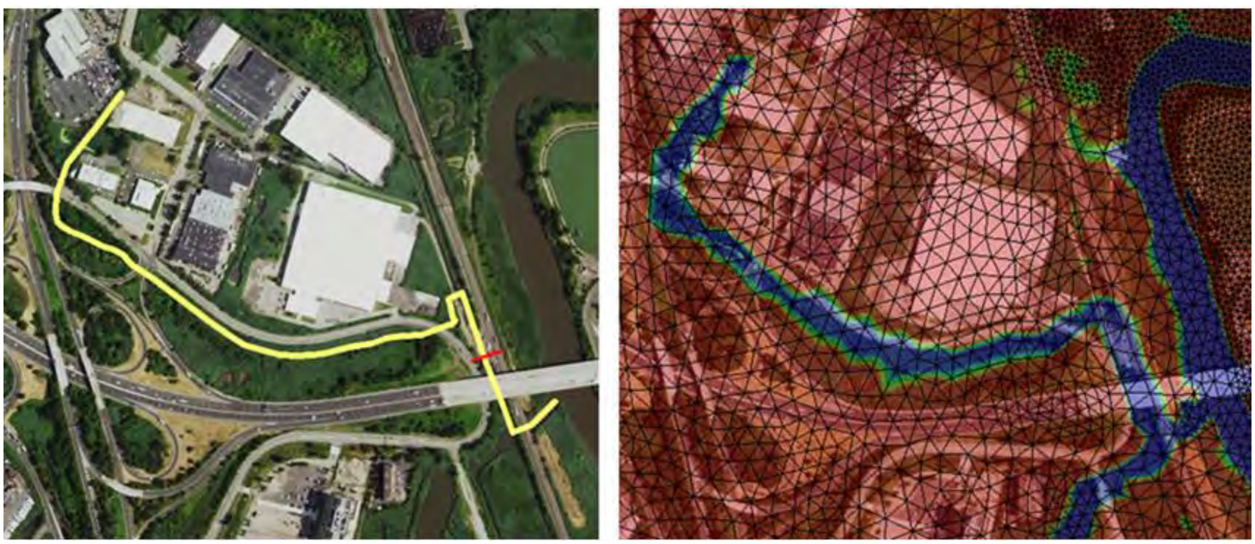


Figure B1-16: Model Refinement by Incorporating the Small Creek (yellow line) and Rutherford Tide Gate (red line) into Model

2.2.2 Available Water Level Measurement Stations for Model Calibration and Validation

The MIKE 21 HD FM model also requires calibration and validation before actual Proposed Project runs can be conducted.

The model calibration and validation involve the comparison of model-predicted time series of water levels with measured water levels for existing conditions (or Base Model Scenarios) at available tidal gauges/stations. **Figure B1-17** below shows the available water level measurement stations for model calibration and validation. The data used in the calibration and validation are from Meadowlands Environmental Research Institute (MERI) Environmental Monitoring Data, New Jersey Sports and Exposition Authority (NJSEA) real-time tide gate monitoring, and USGS Hurricane Sandy Mapper.

Figure B1-17: Available Water Level Measurement Stations for Model Calibration and Validation

2.2.3 Model Calibration

Figure B1-18 through **Figure B1-24** show the model calibration plots to compare the model-predicted time series of tidal water levels with measured water levels for existing conditions (or Base Model Scenarios) at available tidal gauges/stations during a 15-day tide cycle (from August 1, 2015 to August 15, 2015) which includes both spring and neap tides. The step-function-like time-series of measured water level at West Riser in **Figure B1-23** shows the real-time measurement at the West Riser tide gate.

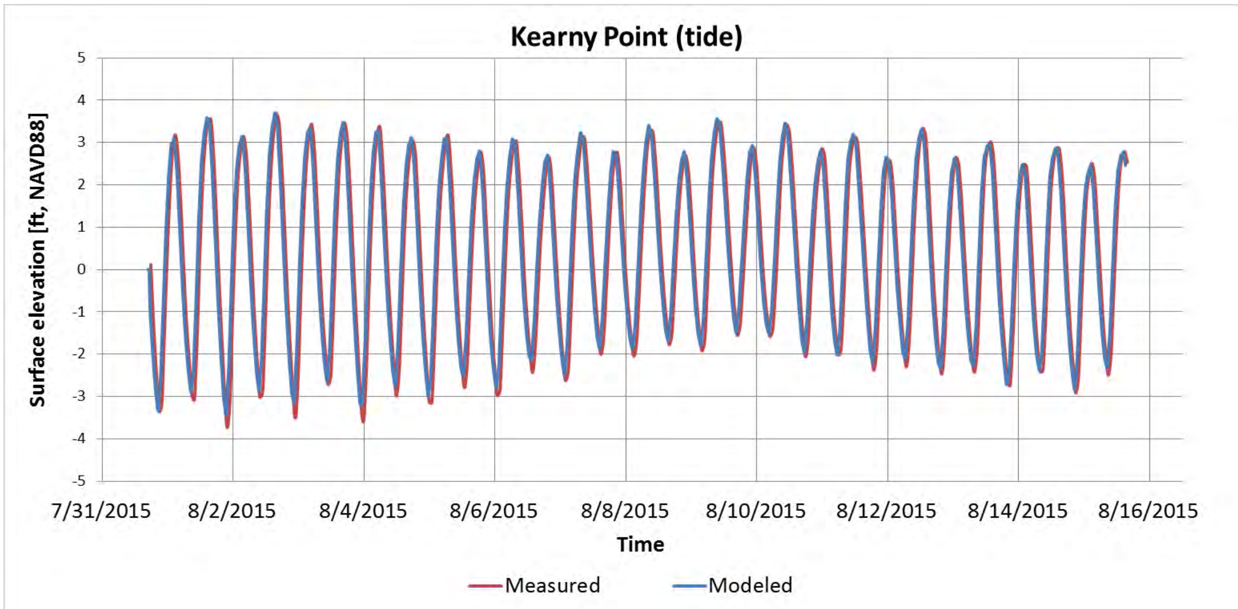


Figure B1-18: MIKE 21 HD FM Coastal Model: Tidal Calibration at Kearny Point

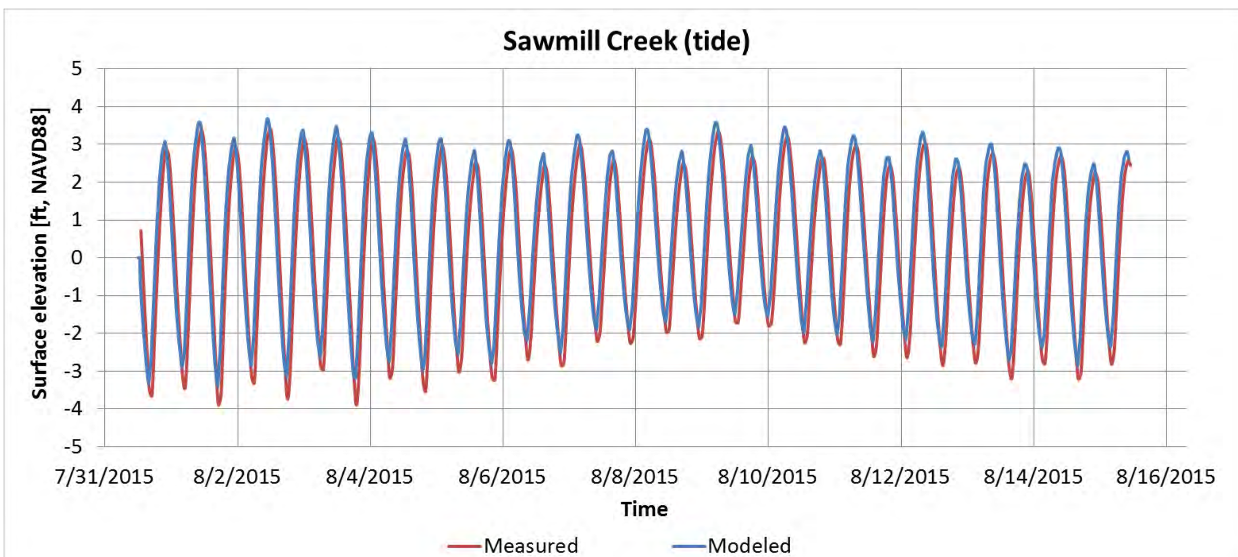


Figure B1-19: MIKE 21 HD FM Coastal Model: Tidal Calibration at Sawmill Creek

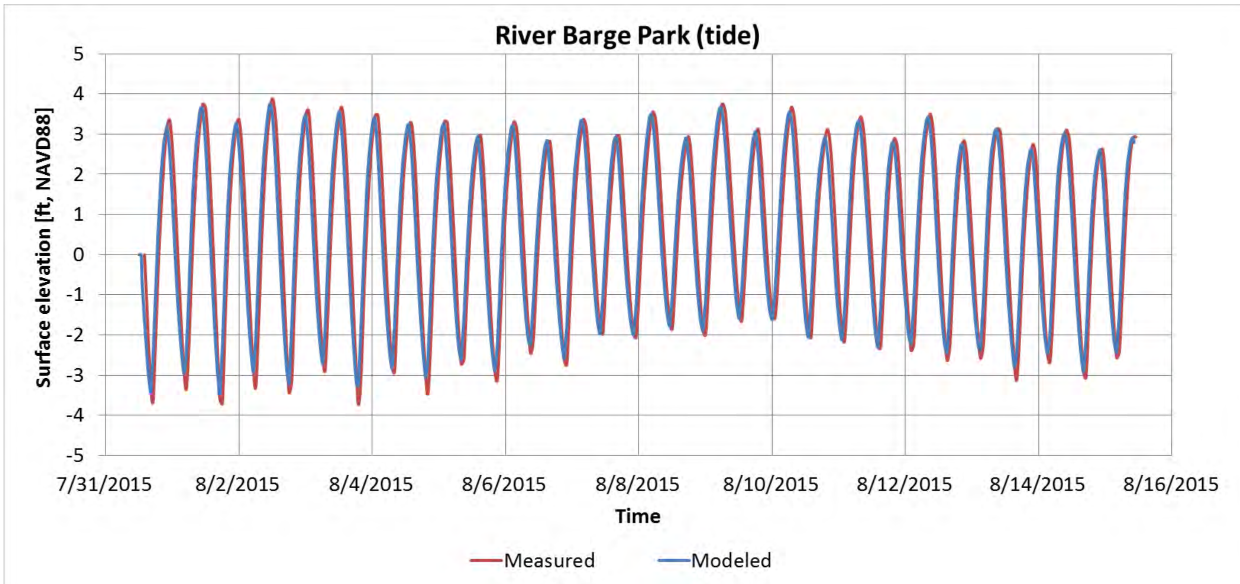


Figure B1-20: MIKE 21 HD FM Coastal Model: Tidal Calibration at River Barge Park

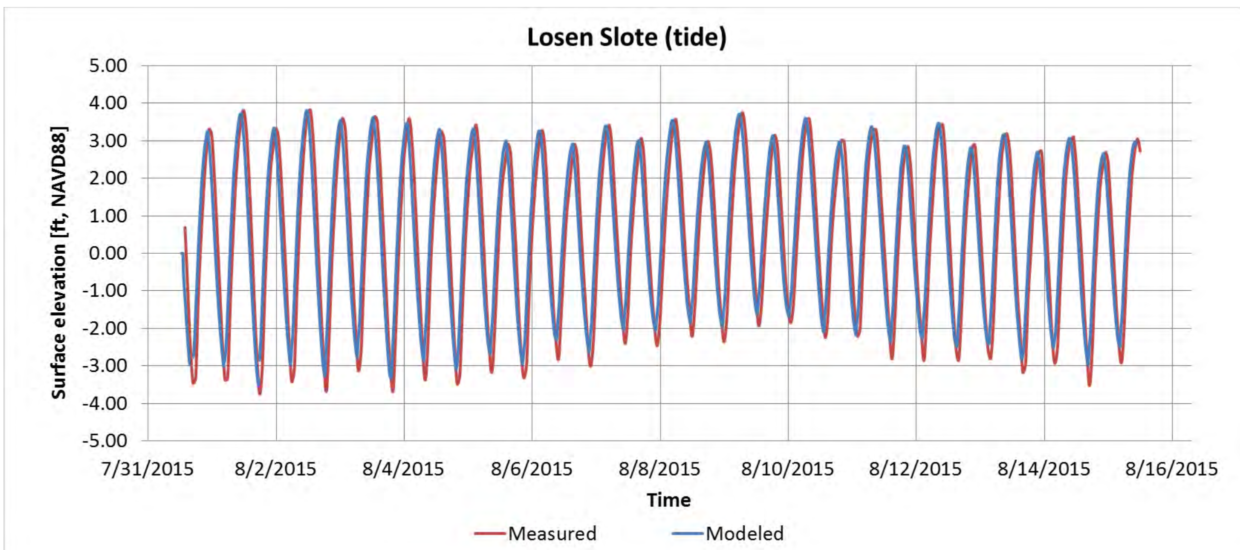


Figure B1-21: MIKE 21 HD FM Coastal Model: Tidal Calibration at Losen Slote

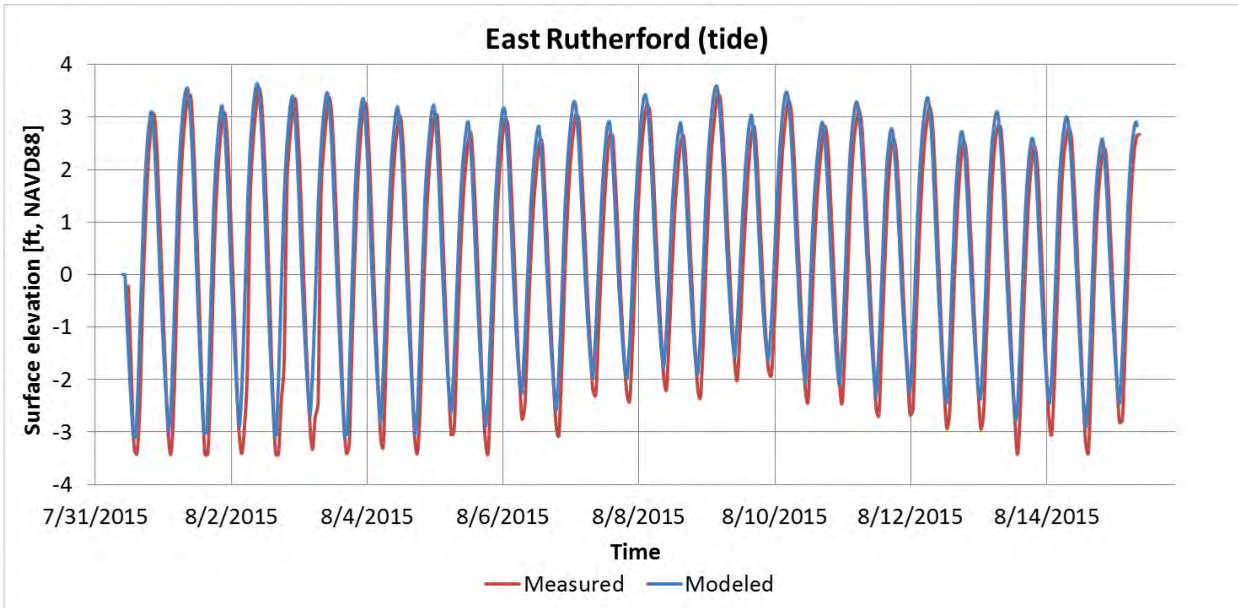


Figure B1-22: MIKE 21 HD FM Coastal Model: Tidal Calibration at East Rutherford

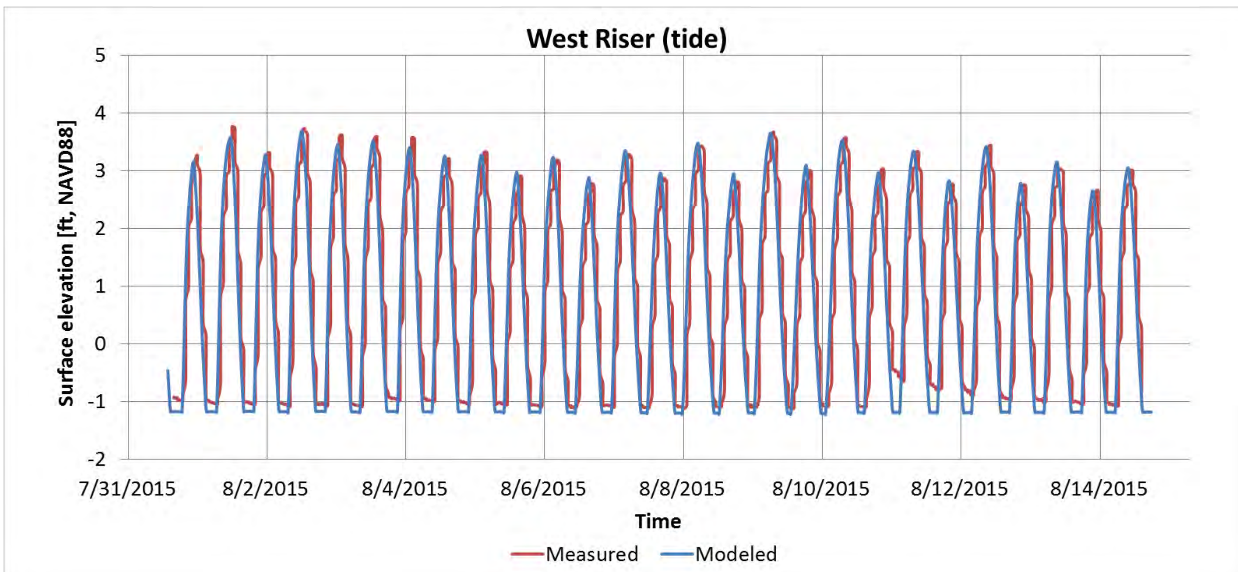


Figure B1-23: MIKE 21 HD FM Coastal Model: Tidal Calibration at West Riser

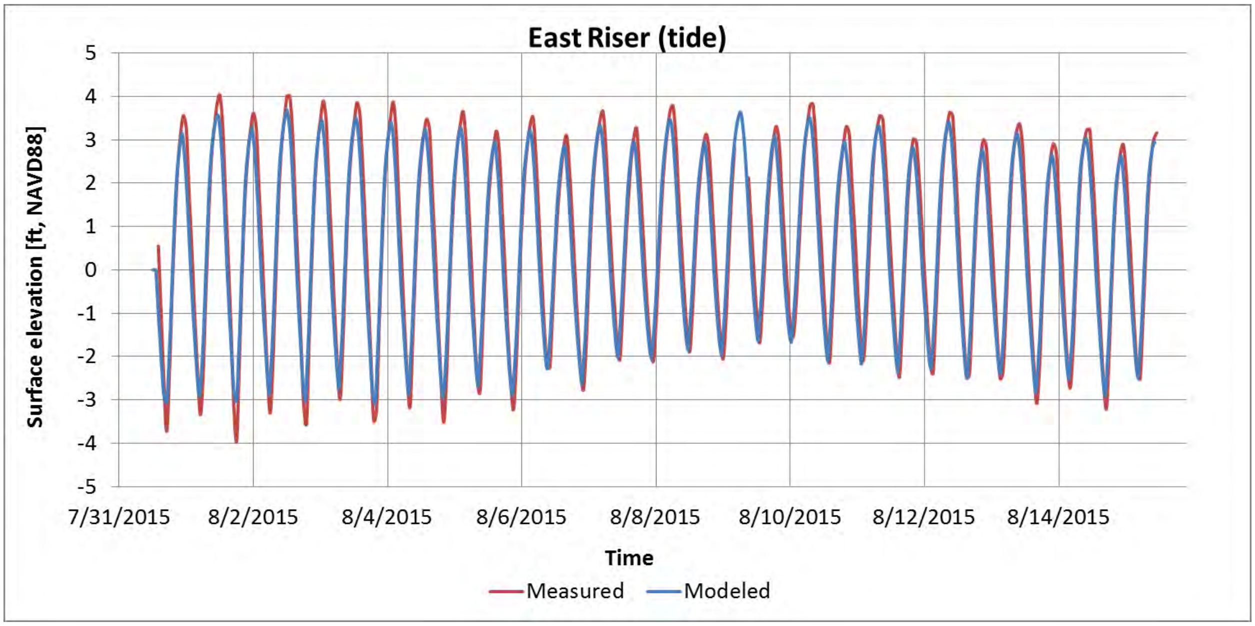


Figure B1-24: MIKE 21 HD FM Coastal Model: Tidal Calibration at East Riser

To further quantify these comparisons, **Table B1-4** shows the observed and simulated maximum water surface elevation (WSEL) in model calibration at seven stations, while **Figure B1-25** shows the correlogram of their data. **Table B1-5** shows the summary of the Root Mean Square Error (RMSE) analysis carried out for the model calibration. The RMSE for the model calibration run is 1.92 inches, which is approximately 4.21 percent of the highest WSEL and 4.45 percent of the lowest WSEL simulated in the model calibration run.

From the aforementioned figures and tables, it can be seen that the tidal calibration results generally show good agreement between model-predicted and measured tidal elevations.

Table B1-4: Observed and Simulated Maximum Water Surface Elevations (WSEL, feet NAVD88) in Model Calibration

Station	Observed Max WSEL (feet, NAVD 88)	Modeled Max WSEL (feet, NAVD 88)	Difference (simulated – measured) (feet)
Keary Point	3.68	3.54	-0.14
Sawmill Creek	3.39	3.67	0.28
River Barge Park	3.88	3.74	-0.14
Losen Slote	3.81	3.72	-0.09
East Rutherford	3.44	3.57	0.13
West Riser	3.76	3.68	-0.08
East Riser	4.03	3.68	-0.35

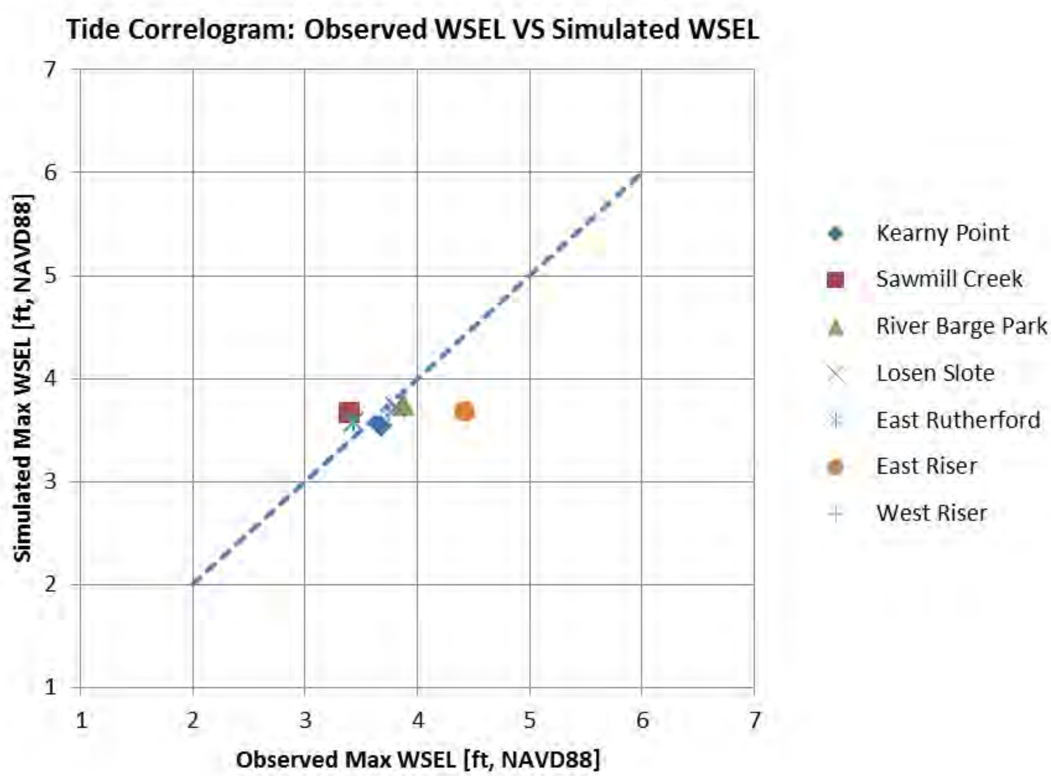


Figure B1-25: Comparison of Observed and Simulated WSEL for Calibration Model Run

Table B1-5: RMSE Analysis Summary for Calibration Model Run

Stations	RMSE Values
RMS error during Calibration Model Run for all stations	0.32 feet (3.84 inches)
Percent compared to highest WSEL simulated during Calibration Model Run	8.43 percent
Percent compared to lowest WSEL simulated during Calibration Model Run	8.91 percent

Tide constituent comparisons were also conducted at some measured water level stations, as shown in **Table B1-6**, **Table B1-7**, and **Table B1-8**.

Table B1-6: Comparison of Tide Constituent at Kearny Point (Admiralty Method)

Constituent Name	Modeled		Measured		Ratio of Amplitude Difference	Phase Difference (°)
	Amp. (feet)	Phase (°)	Amp. (feet)	Phase (°)		
M2	2.393	-109.26	2.523	-86.29	5%	22.97
S2	0.380	-84.63	0.400	-61.12	5%	23.51
K1	0.353	114.96	0.357	126.27	1%	11.31
O1	0.179	105.59	0.178	125.97	1%	20.38

Table B1-7: Comparison of Tide Constituent at Sawmill Creek (Admiralty Method)

Constituent Name	Modeled		Measured		Ratio of Amplitude Difference	Phase Difference (°)
	Amp. (feet)	Phase (°)	Amp. (feet)	Phase (°)		
M2	2.402	-101.66	2.520	-76.62	5%	25.04
S2	0.358	-77.00	0.390	-51.33	8%	25.67
K1	0.349	118.95	0.348	133.18	0.4%	14.23
O1	0.175	112.62	0.181	134.89	3%	22.27

Table B1-8: Comparison of Tide Constituent at River Barge Park (Admiralty Method)

Constituent Name	Modeled		Measured		Ratio of Amplitude Difference	Phase Difference (°)
	Amp. (feet)	Phase (°)	Amp. (feet)	Phase (°)		
M2	2.487	-93.76	2.612	-69.70	5%	24.06
S2	0.347	-68.10	0.390	-43.70	11%	24.40
K1	0.347	124.00	0.353	137.39	2%	13.39
O1	0.176	117.44	0.186	140.89	6%	23.45

As **Table B1-6** through **Table B1-8** show, the tide constituents generated by the model are reasonable. The phase difference is caused by the uncertainty of the wetland bathymetry. Besides the normal tide condition, the model is also calibrated against Hurricane Joaquin, a hurricane that occurred in 2015 with a return period less than 10-year. **Figure B1-26** through **Figure B1-32** show the model calibration plots that compare the model-simulated time series of water levels with measured water levels at functioning gauge stations during Hurricane Joaquin.

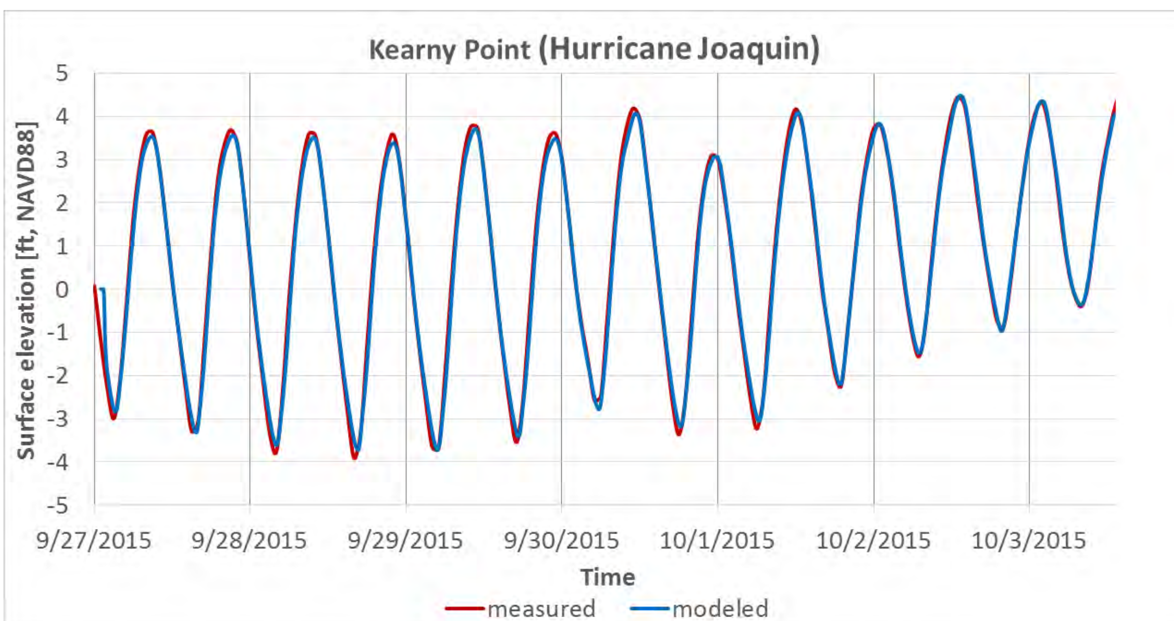


Figure B1-26: MIKE 21 HD FM Coastal Model: Calibration with Hurricane Joaquin at Kearny Point

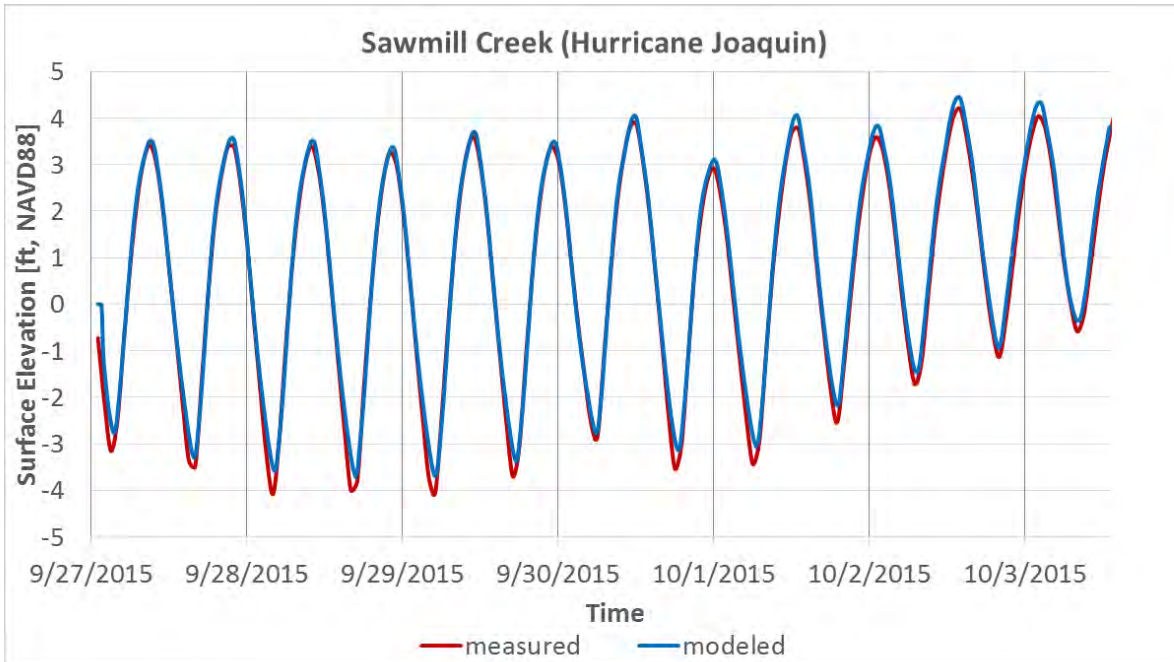


Figure B1-27: MIKE 21 HD FM Coastal Model: Calibration with Hurricane Joaquin at Sawmill Creek

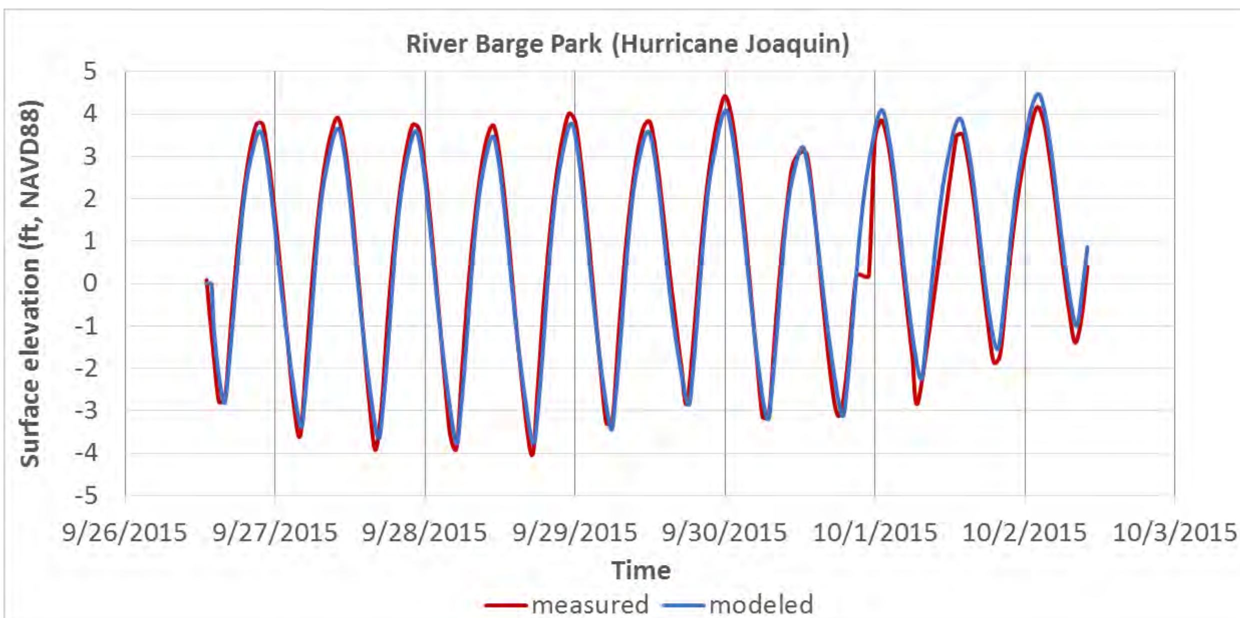


Figure B1-28: MIKE 21 HD FM Coastal Model: Calibration with Hurricane Joaquin at River Barge Park

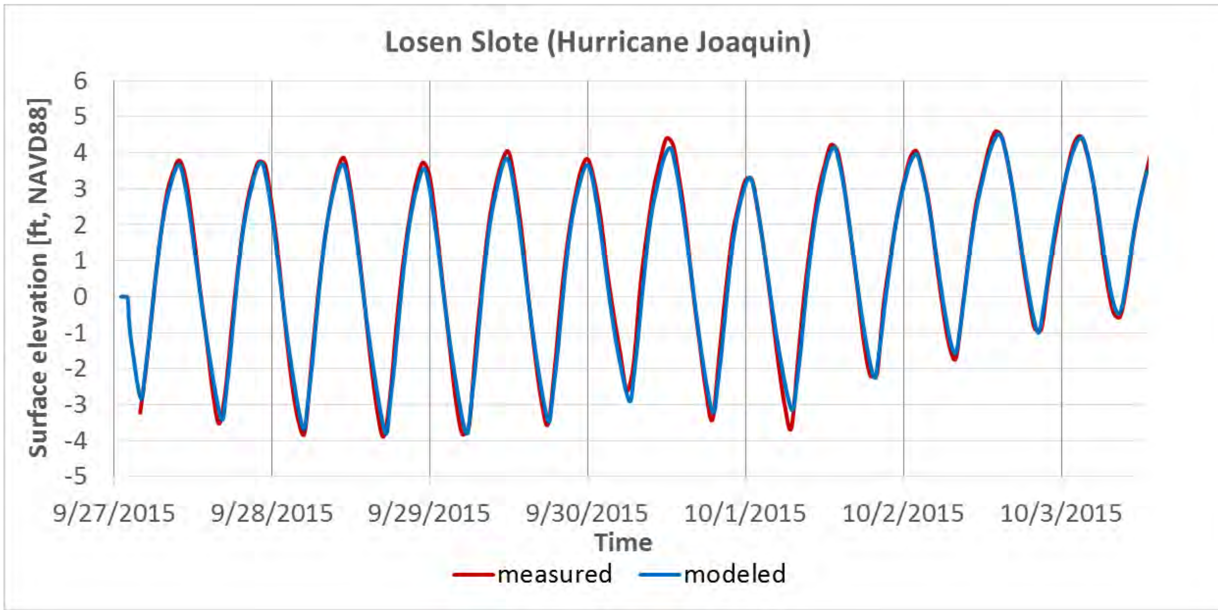


Figure B1-29: MIKE 21 HD FM Coastal Model: Calibration with Hurricane Joaquin at Losen Slote

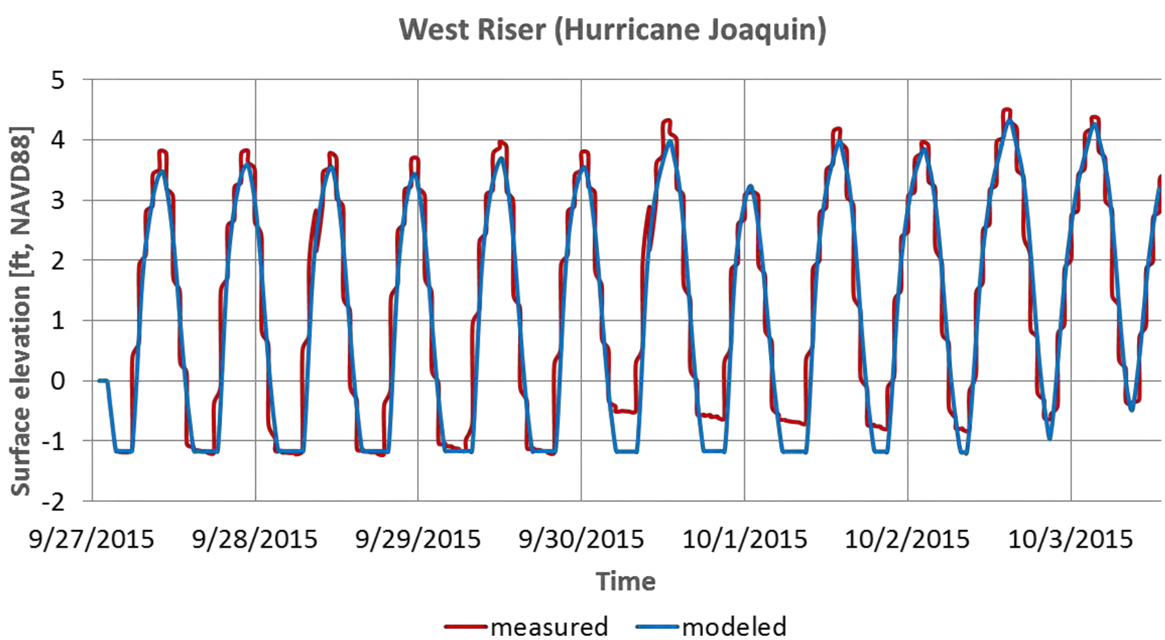


Figure B1-30: MIKE 21 HD FM Coastal Model: Calibration with Hurricane Joaquin at West Riser

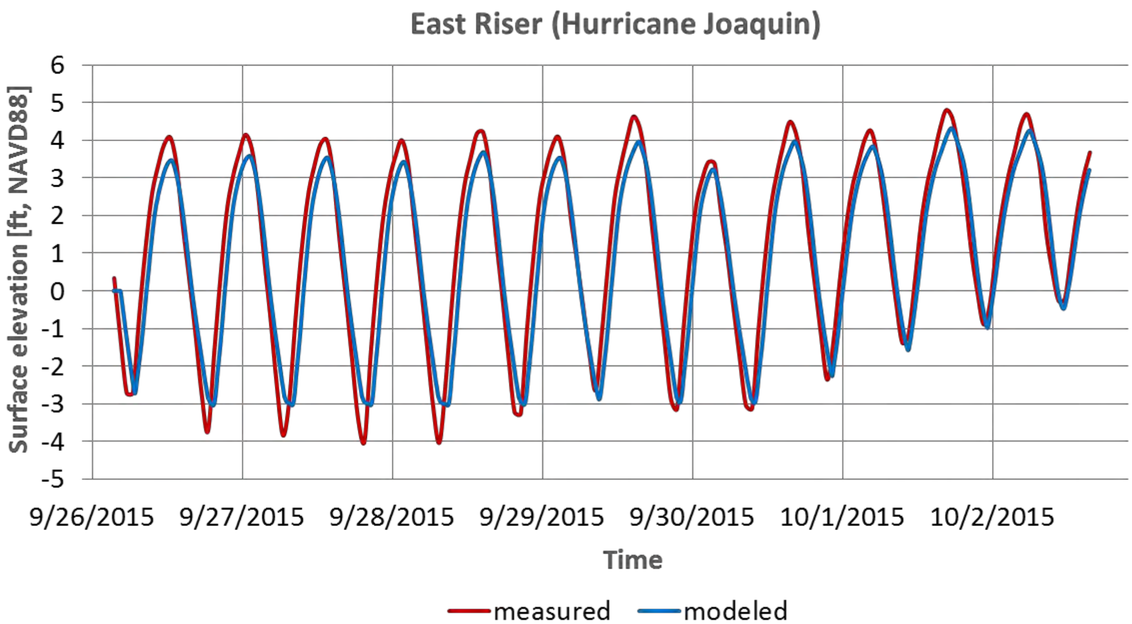


Figure B1-31: MIKE 21 HD FM Coastal Model: Calibration with Hurricane Joaquin at East Riser

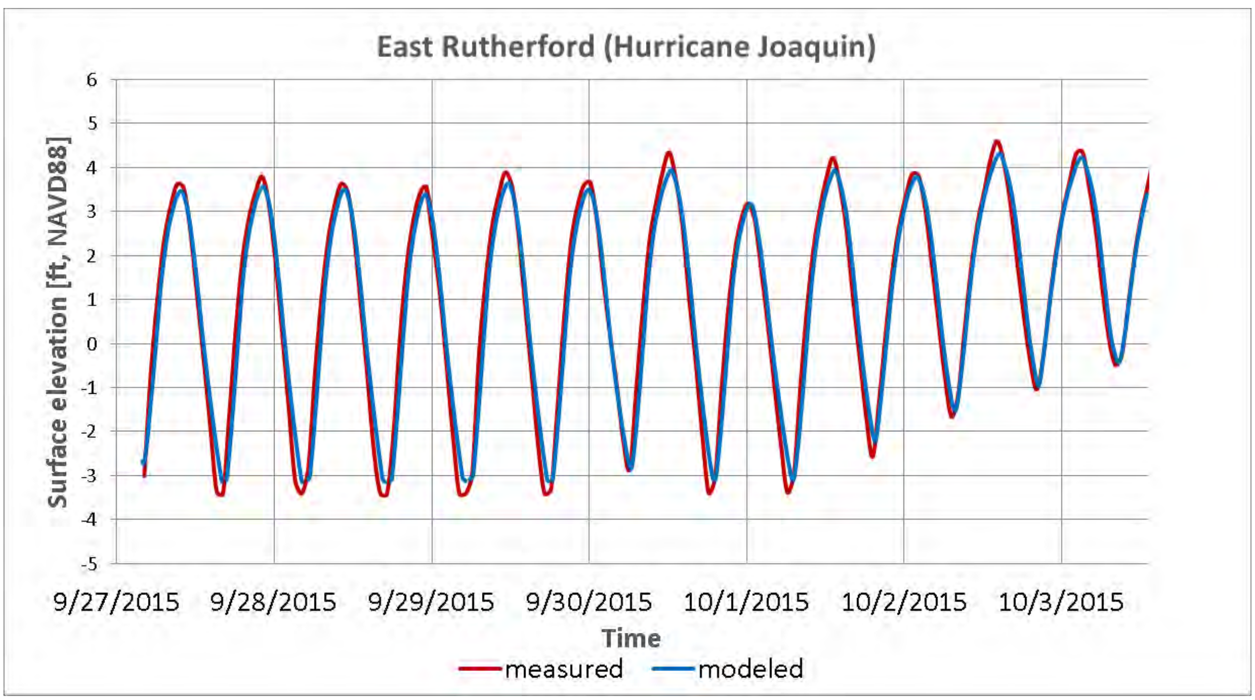


Figure B1-32: MIKE 21 HD FM Coastal Model: Calibration with Hurricane Joaquin at East Rutherford

The difference and correlogram between the observed and simulated maximum WSELs are also presented for the validation run of Hurricane Joaquin (see **Table B1-9** and **Figure B1-33**). The RMSE (see **Table B1-10**) is shown to be 2.28 inches, which is 4.37 percent of the highest WSEL and 4.59 percent of the lowest WSEL simulated.

Table B1-9: Observed and Simulated Maximum Water Surface Elevations (WSEL, feet NAVD88) in Model Validation with Hurricane Joaquin

Station	Observed Max WSEL [feet, NAVD88]	Simulated Max WSEL Base Model Run [feet, NAVD88]	Difference (Base Model Run - Observed) [feet]
Kearny Point	4.45	4.42	-0.03
Sawmill Creek	4.2	4.41	0.21
River Barge Park	4.42	4.46	0.04
Losen Slote	4.59	4.43	-0.16
East Rutherford	4.6	4.24	-0.36
West Riser	4.49	4.33	-0.16
East Riser	4.8	4.32	-0.48

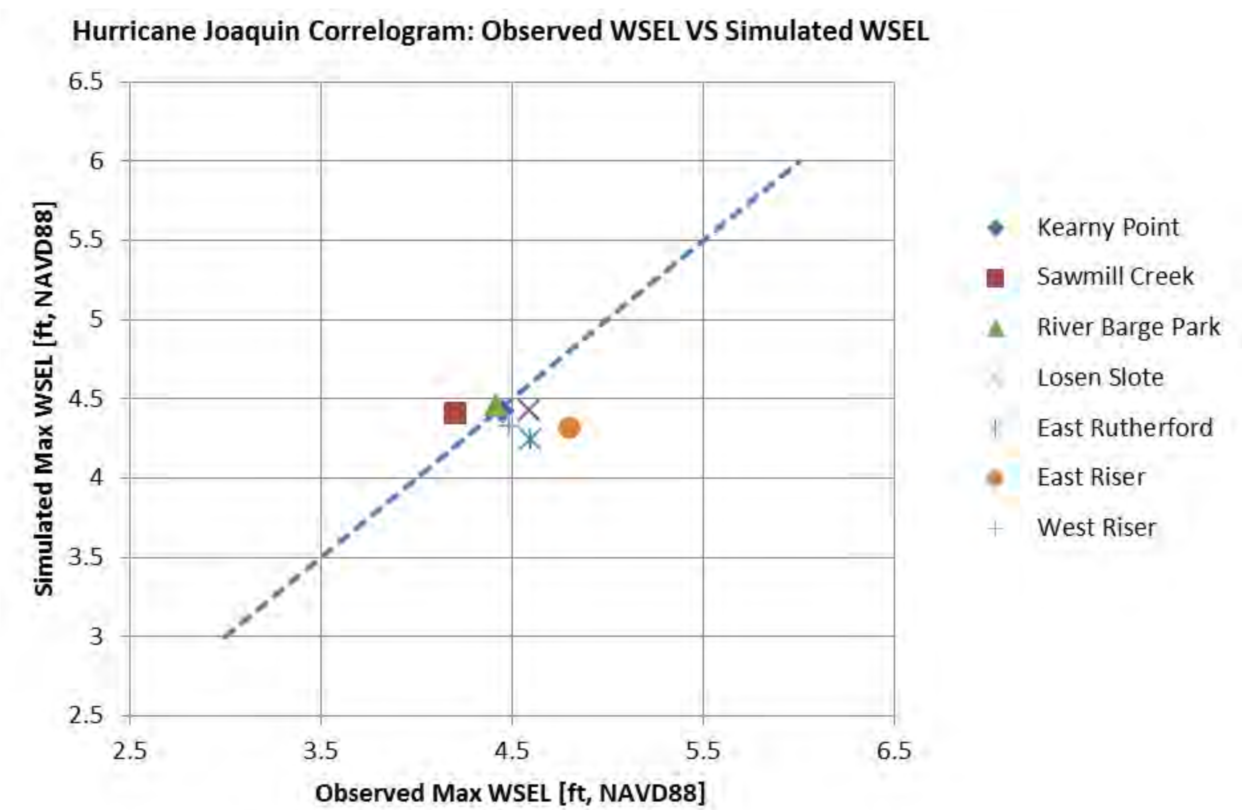


Figure B1-33: Comparison of Observed and Simulated WSEL for Calibration Model Run of Hurricane Joaquin

Table B1-10: RMSE Analysis Summary for Calibration Model Run of Hurricane Joaquin

Stations	RMSE Values
RMS error during Calibration Model Run for all stations	0.26 feet (3.12 inch)
Percent compared to highest WSEL simulated during Calibration Model Run of Hurricane Joaquin	5.73 percent
Percent compared to lowest WSEL simulated during Calibration Model Run of Hurricane Joaquin	6.03 percent

From the figures and tables shown above, it can be seen that the model calibration results with Hurricane Joaquin show good agreement between model-simulated and measured elevations.

2.2.4 Model Validation

Model validation involves the comparison of model-predicted surges with measured surges during storm events. For the Proposed Project, Hurricane Sandy, the superstorm that occurred in 2012 with a return period between 100-year and 500-year, was used for model validation.

Many water measurement stations were out of function during Hurricane Sandy, so there are relatively few stations with accessible measurement data. **Figure B1-34** and **Figure B1-35** show the model validation plots that compare the model-simulated time series of water levels with measured water levels at functioning gauge stations during Hurricane Sandy.

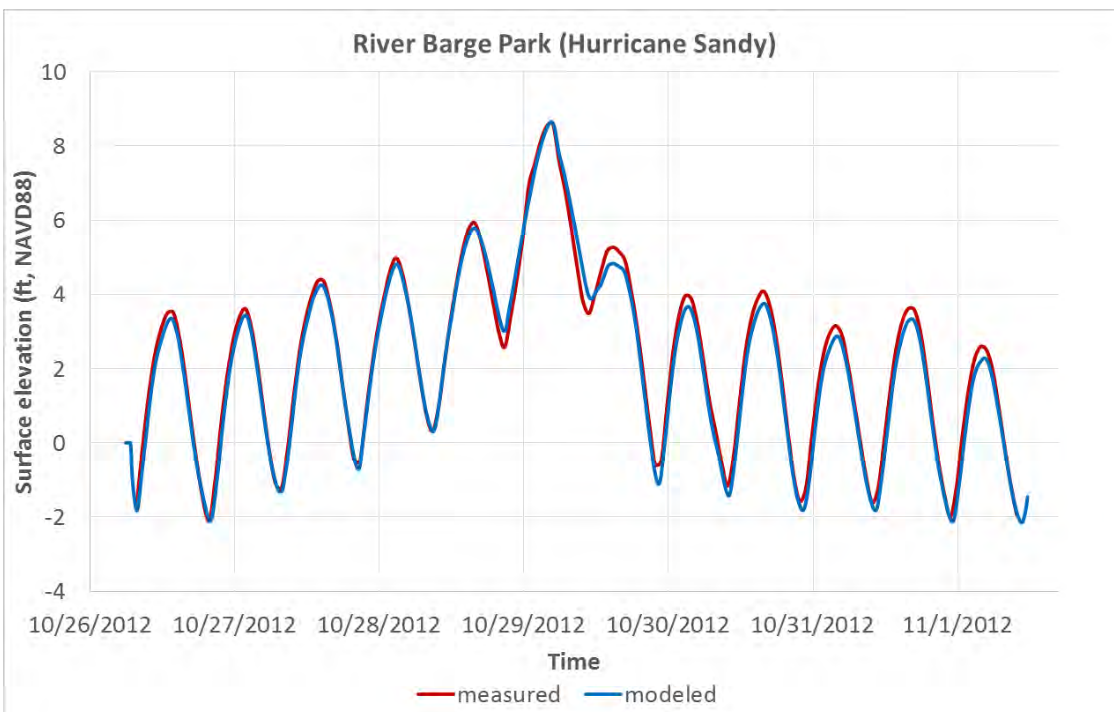


Figure B1-34: MIKE 21 HD FM Coastal Model: Validation with Hurricane Sandy at River Barge Park

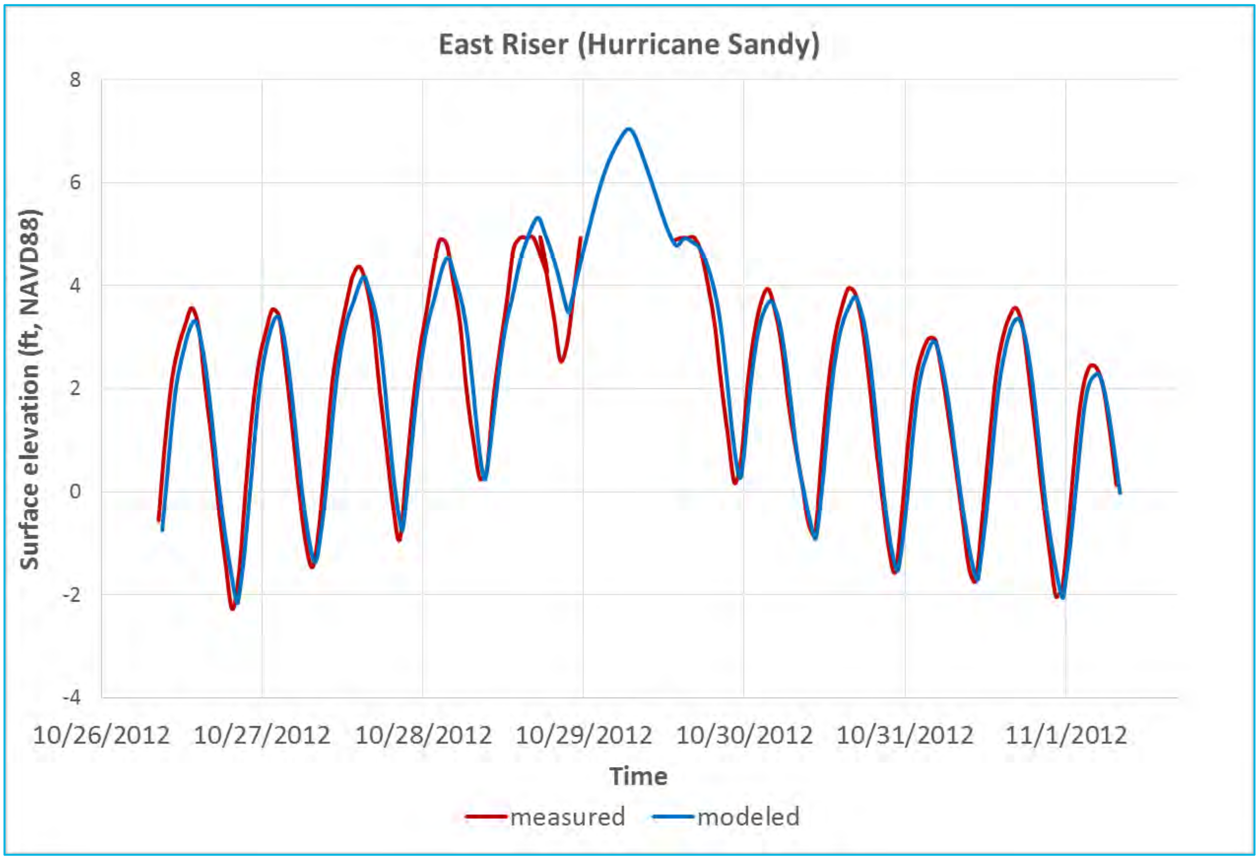


Figure B1-35: MIKE 21 HD FM Coastal Model: Validation with Hurricane Sandy at East Riser

Figure B1-36 shows the model validation plots that compare the model-simulated peak surge elevations with USGS-measured high water marks at eight locations during Hurricane Sandy.

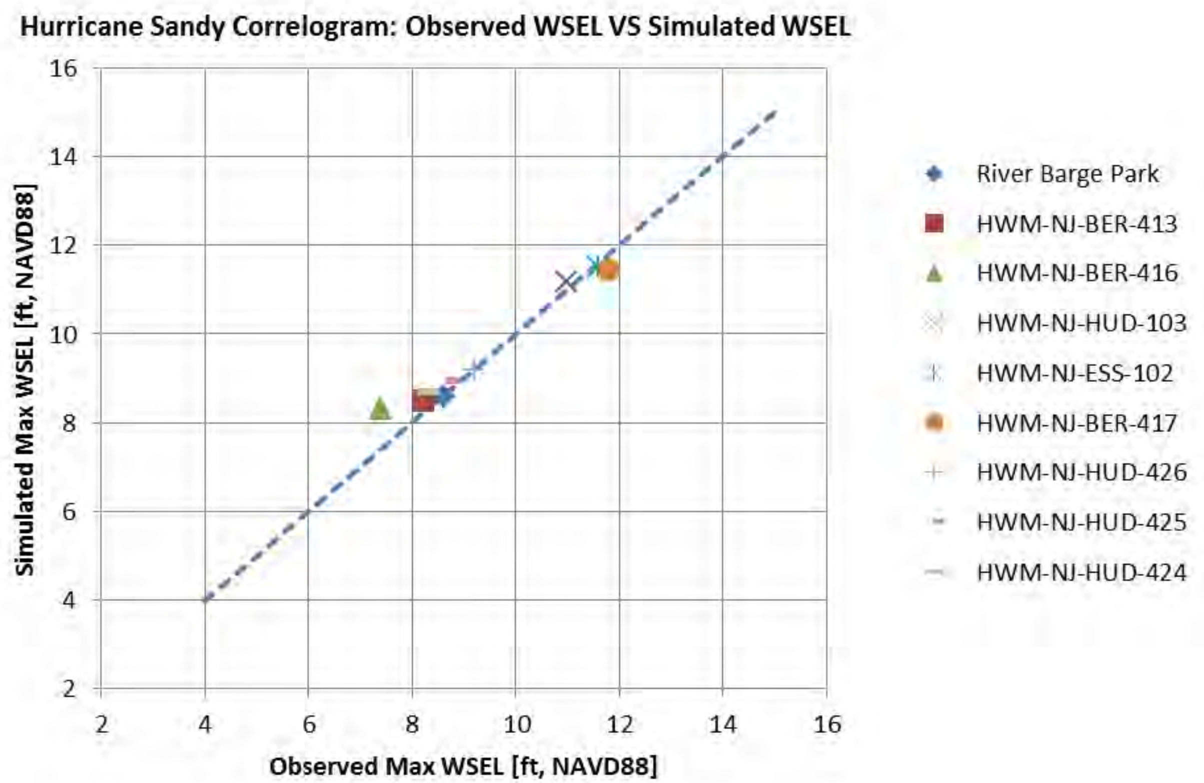


Figure B1-36: Comparison of Observed and Aimulated WSEL for Validation Model Run of Hurricane Sandy

Difference between the maximum observed and simulated WSELs are also presented in **Table B1-11**. The RMSE analysis (see **Table B1-12:**) shows that the RMSE in the validation simulation of Hurricane Sandy is 4.32 inches, which is 3.15 percent of the highest WSEL and 5.14 percent of the lowest WSEL simulated.

Table B1-11: Observed and Simulated Water Surface Elevations (WSEL, feet NAVD 88) in Model Validation with Hurricane Sandy

Station	Observed Max WSEL [feet, NAVD88]	Simulated Max WSEL Base Model Run [feet, NAVD88]	Difference (Base Model Run - Observed) [feet]
River Barge Park	8.61	8.57	-0.04
HWM-NJ-BER-413	8.2	8.5	0.3
HWM-NJ-BER-416	7.4	8.31	0.91
HWM-NJ-HUD-103	11	11.18	0.18
HWM-NJ-ESS-102	11.6	11.49	-0.11
HWM-NJ-BER-417	11.8	11.45	-0.35
HWM-NJ-HUD-426	9.2	9.18	-0.02
HWM-NJ-HUD-425	8.7	8.95	0.25
HWM-NJ-HUD-424	8.3	8.7	0.4

*The gauge at East Riser failed to capture the peak local WSEL during Hurricane Sandy

Table B1-12: RMSE Analysis in Model Validation with Hurricane Sandy

RMS Error during Validation Model Run of Hurricane Sandy for all stations	0.36 feet (4.32 inch)
Percent Compared to Highest WSEL simulated during Validation Model Run of Hurricane Sandy	3.15%
Percent Compared to Lowest WSEL simulated during Validation Model Run of Hurricane Sandy	5.14%

The figures and tables above demonstrate that the model validation results using Hurricane Sandy show generally close agreement between model-simulated and measured elevations.

2.3 MIKE 21 Spectral Wave Model SW

Besides the storm surge and current, the wave is also simulated because: (1) the wave may overtop the proposed Alternative 1 LOP and flood the area that is meant to be protected, and (2) the wave radiation stress affects the momentum of the water body. MIKE 21 SW wave models were set up for the Proposed Project to simulate the wave conditions during coastal storm events. The MIKE 21 SW wave model uses the same model domain and mesh as MIKE 21 HD FM model and the two models are dynamically coupled. The setup parameters of the MIKE 21 SW wave model are listed in **Table B1-13**. The temporally and spatially varying wind data used for the simulation of wind-generated waves are obtained from a FEMA flood study (RAMPP, 2014) and provided by Oceanweather Inc. Different wave generation mechanisms have been tested and it is found out that waves at the study area are mainly generated by the local wind instead of being transformed from the open ocean. The wind data correspond to the synthetic storm condition that produces the storm surge (with ADCIRC model) in terms of different frequencies of occurrence at Bergen Point.

Figure B1-37 and **Figure B1-38** show the simulated results of maximum significant wave height and peak wave period for a 10-year storm with 1.2 feet SLR (in 2075, as suggested by the NOAA Intermediate-Low projection and as described in **Section 3.2**) in the model domain. It can be seen that in the Project Area (indicated by the magenta line in **Figure B1-37**and **Figure B1-38**), the significant wave height is about 0.6 feet, while the peak wave period is about 2.0 seconds.

Table B1-13: Setup Parameters of the MIKE 21 SW model

Parameter	Value / Note
Frequency Discretization	Discretization type: logarithmic; Number of frequencies: 25 Minimum frequency: 0.055 Hz Frequency factor: 1.1
Directional Discretization	Discretization type: 360 degree rose; Number of direction: 16
Water Level	Water level variation from HD simulation
Bottom Friction	Nikuradse roughness with constant value of 0.035m in the domain
White Capping	Dissipation coefficient, Cdis = 4.5, DELTA dis = 0.5
Current	Not included
Wave-wave Interaction	Triad-wave interaction with transfer factor of 0.25
Wave Breaking	Included by using constant gamma of 0.8 and alpha of 1
Initial Condition	Jonswap fetch growth expression formula
Wind	Varying temporally and spatially in the domain. Obtained from a FEMA flood study (RAMPP 2014)and provided by Oceanweather Inc.

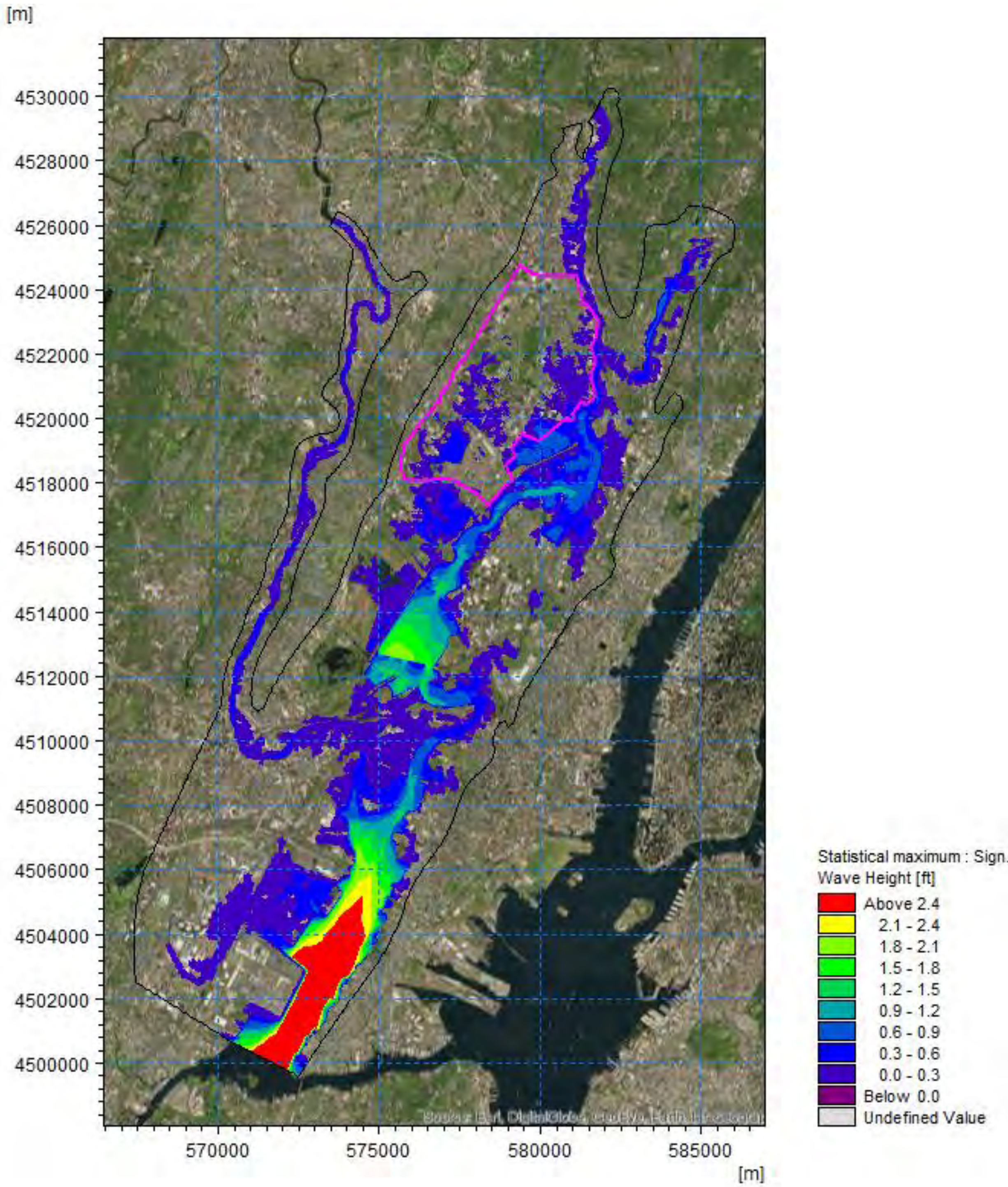


Figure B1-37: Distribution of Simulated Maximum Significant Wave Height for the 10-year Storm and 1.2 feet SLR

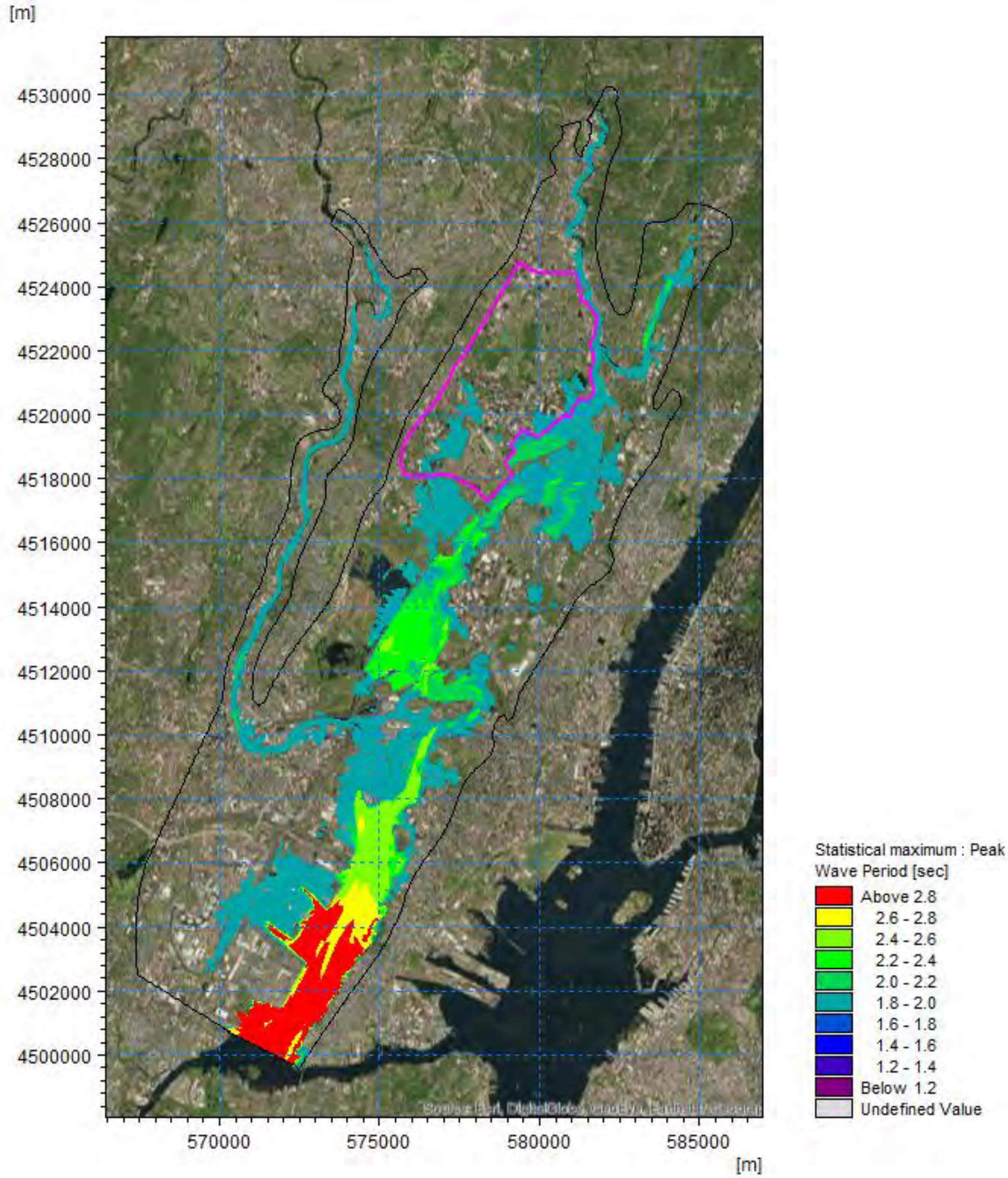


Figure B1-38: Distribution of Simulated Maximum Peak Wave Period for the 10-year Storm and 1.2 feet SLR

3.0 Coastal Flood Assessment

3.1 Identification of Coastal Storms

For the Proposed Project, the return period of the design storm event for the Alternative 1 LOP is 10-years, as suggested by NJDEP and considering the tradeoff between budget and risks. This 10-year storm refers to the 10-year storm surge elevation at NOAA Bergen Point Tidal Station. The stillwater elevation at NOAA Bergen Point Tidal Station for a 10-year coastal storm is 7.0 feet above NAVD88 (FEMA 2014, FEMA 2013). The storm surge elevations for different return periods of coastal storms at Bergen Point are summarized in **Table B1-14**. The NOAA Bergen Point Tidal Station was chosen because it is the only FEMA-validated station within the model domain.

Table B1-14: Stillwater Elevations for Different Return Periods of Storms at Bergen Point Tidal Station

Return Period (year)	Stillwater Elevation (feet, NAVD88)
10	7.0
50	9.7
100	10.9
500	13.8

The procedures for the identification of a coastal storm event for the selected return period are summarized below:

1. Based on FEMA studies at Bergen Point, water elevations for different return periods are established;
2. Based on previous model simulations for FEMA, design storm events which can generate water elevations for the corresponding return period are identified;
3. Wind and pressure fields for corresponding storm events are extracted; and
4. A simulation with driving forces, plus SLR, is performed.

The simulation results and the FEMA Stage Frequency analyses indicate that the still water levels at the project site are lower than at Bergen Point. It should be noted that only a limited number of storms were selected for simulation and that the selected storms each represent a combination of storm surge and tide conditions. Actual future storms could have other combinations of storm surge and astronomic tides, which could result in different flood responses. For example, a storm with an extremely long duration could result in higher flood elevations in the Project Area relative to Bergen Point than reflected in the current simulations.

3.2 Sea Level Rise

SLR is an important consideration in any flood resiliency planning and design for coastal regions due to its resulting increase in stillwater elevation and the compounding effect it has on storm surge. Uncertainties are inherent in the projection of SLR, so two NOAA projections at the year 2075 are incorporated for the Proposed Project: one with 1.2 feet of Intermediate-Low SLR projection and the other with 2.4 feet of Intermediate-High SLR projection.

3.3 Rainfall Storms

The rainfall-tide correlation analysis is conducted by analyzing the correlation of both tide residual and the water level with precipitation by using water level data from NOAA Station 8519483 – Bergen Point, NJ, in conjunction with precipitation data from NOAA Station 725025-94741 – Teterboro Airport, NJ and NOAA Station 725020-14734 – Newark Liberty International Airport, NJ. Detailed correlation analysis can be found in **Section 4** of **Subappendix B2**. The correlation is shown to be weak. Although there is a storm, such as Hurricane Irene (10-year event), that corresponds to a greater than 50-year rainfall event at Teterboro Airport, most of the significant storm surge events are weakly correlated with significant interior rainfall events, such as during Hurricane Sandy, when the maximum elevation at Bergen Point was 11.6 feet (greater than 100-year event), but the corresponding rainfall depth at Teterboro Airport was 1.4 inch (less than 2-year event). Additionally, it was observed that the timing of the peak of the rainfall event does not exactly coincide with the peak of the coastal storm surge event. Coastal storm tides dominate the peak storm surge elevations within the study area.

3.4 Sensitivity Tests of the MIKE 21 HD FM Model

In addition to sensitivity tests on model parameters during the model calibration phase, additional sensitivity tests were carried out for the MIKE 21 HD FM model to assess its performance under different conditions. These include the test on model domain size and test on the effects of wind and waves on storm surge.

3.4.1 Model Domain Size

The MIKE 21 HD FM model domain size should be large enough so that the effects of the proposed Alternative 1 LOP alternatives on the model boundary conditions can be neglected.

The sensitivity test of model domain size is carried out by simulating a 10-year storm plus the 1.2 feet Intermediate-Low SLR condition with and without the proposed Alternative 1 LOP, and comparing the time-series of WSEL at Bergen Point between these two conditions. The model sensitivity test shows that the size of the numerical model domain is large enough and that the effect of the Alternative 1 LOP would not reach the southern boundary at Bergen Point. Surface elevation at Point 1 (located on the Bergen Point boundary) and Point 2 (a short distance away from the Bergen Point boundary) is plotted in **Figure B1-39** for the baseline scenario (without the LOP) and the Alternative 1 scenario (with the LOP). It can be clearly observed that the existence of the Alternative 1 LOP does not affect the WSEL at the Bergen Point boundary. Therefore, the model domain is proven to be sufficiently large for the Proposed Project objectives.

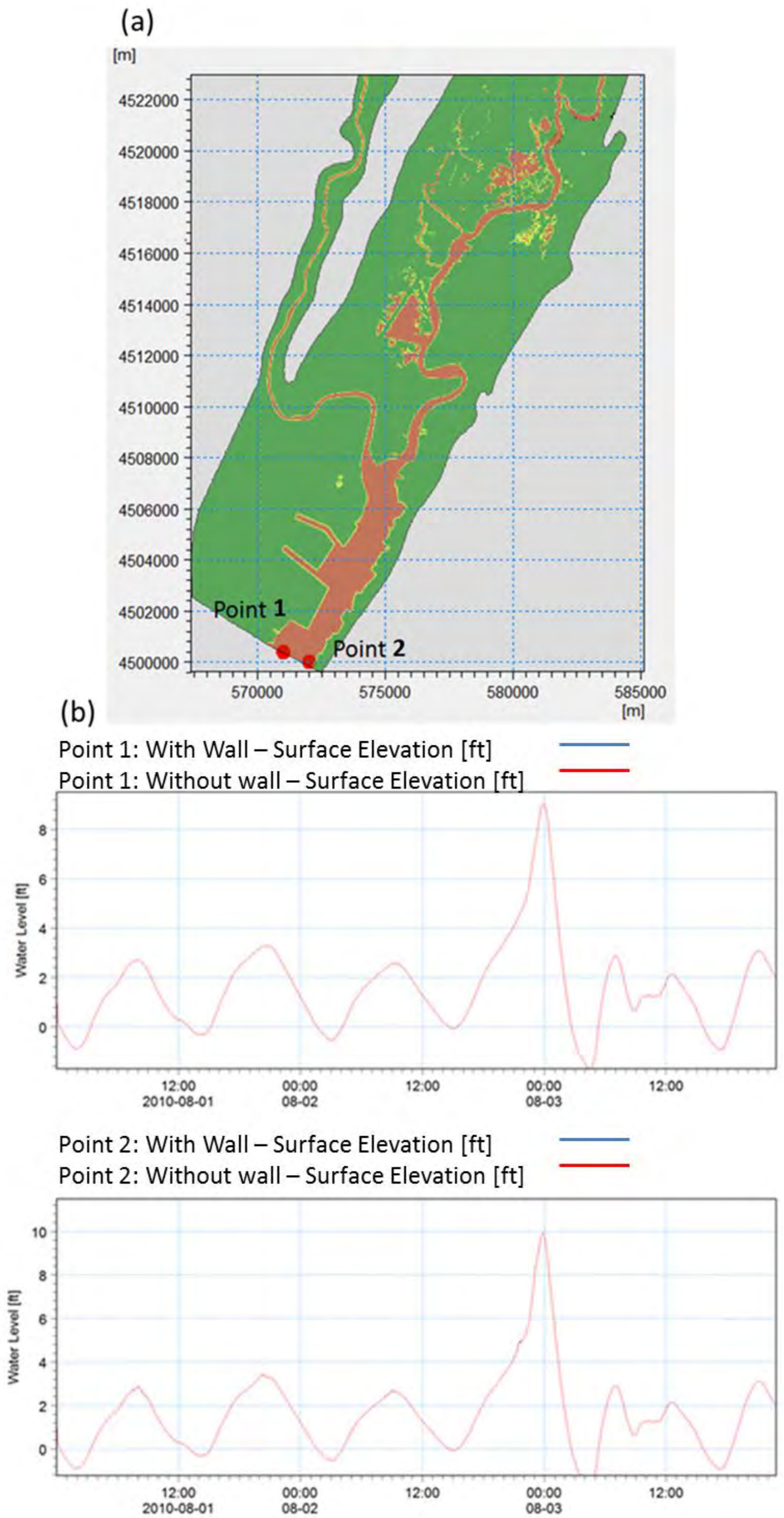


Figure B1-39: Effect of the Alternative 1 LOP on the Domain Boundary at Bergen Point

3.4.2 Effects of Wind on MIKE 21 HD FM Model Storm Surge Simulation

Assessment of wind effects on storm surge for the MIKE 21 HD FM model is performed by carrying out simulations with the wind in a 10-year storm surge condition and without wind condition. The wind time-series are the same one that produces the 10-year-return-period storm surge at Bergen Point with a time interval of 15 minutes. This wind data was obtained from a FEMA flood study (RAMPP, 2014) and provided by Oceanweather Inc.. The difference in maximum surface elevation between with and without wind simulations is presented in **Figure B1-40**. The effect of wind on this local hydrodynamic model is very limited because it only shows up in a wetland area that is a distance from the study area; the difference in maximum surface elevation is generally 2 inches. This difference within the study area is small due to small fetch; the effect of wind field on the storm surge of the study area can be neglected.

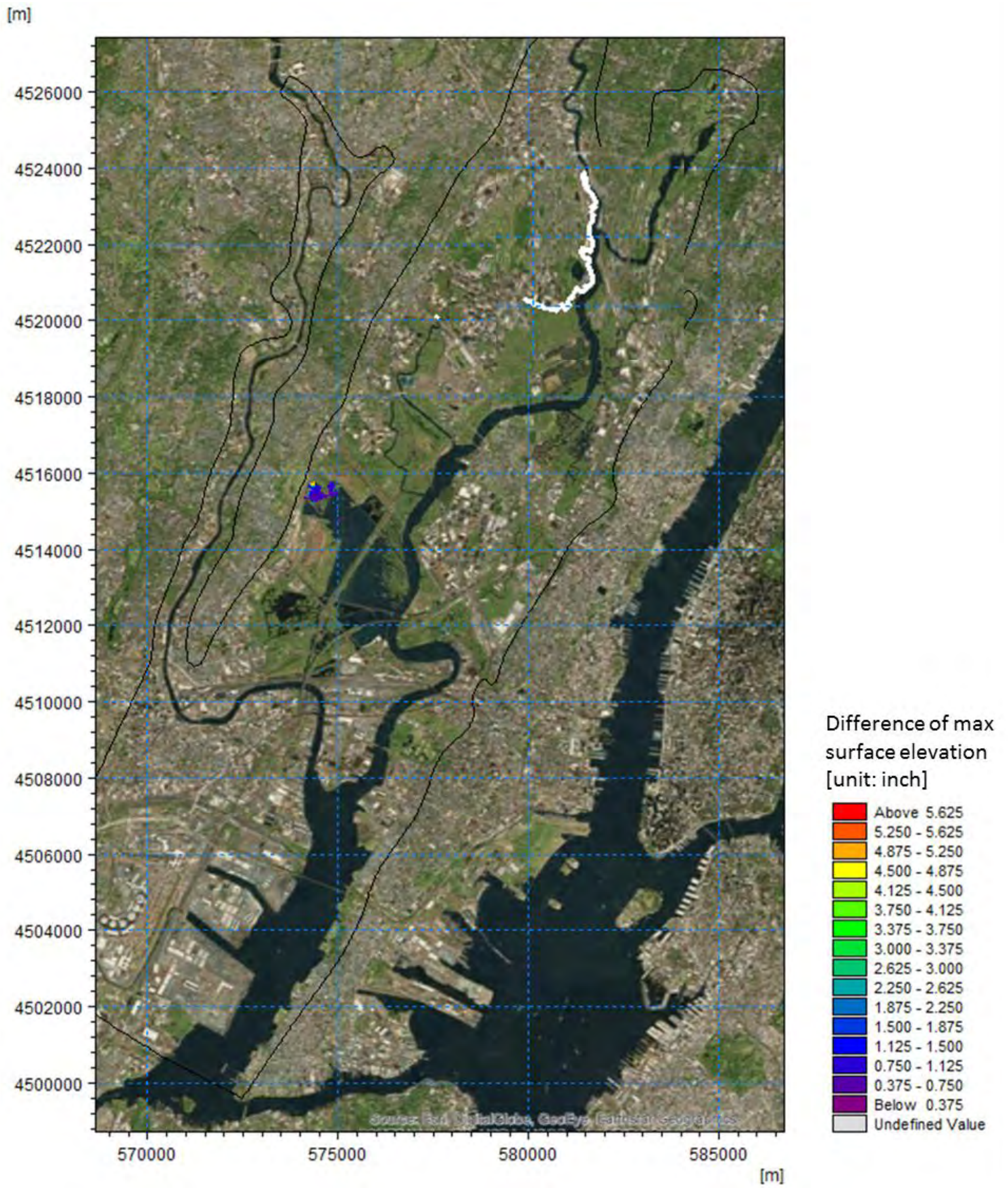


Figure B1-40: Difference of Maximum WSEL in the Alternative 1 Model Simulations with 10-year Wind and Without Wind

3.5 Performance Assessment of the Line of Protection

The MIKE 21 HD FM model is utilized as the primary model for the coastal flood assessment. At this phase, two basic scenarios are simulated and compared. One scenario is the simulation of coastal flooding for existing conditions (without the LOP; a baseline condition). The other scenario is the simulation of coastal flooding with a LOP implemented within the Project Area. At this phase, installation of the proposed Alternative 1 LOP is planned along the west bank of the Hackensack River and at the intersection of New Jersey Route 120 and Berry's Creek Canal (see the white lines in **Figure B1-42**, **Figure B1-43**, **Figure B1-45**, **Figure B1-46**, **Figure B1-48**, **Figure B1-49**, **Figure B1-51**, and **Figure B1-52**) to protect the Project Area in the west (the area enclosed by the magenta line in **Figure B1-** through **Figure B1-52**). The crest elevation of the LOP is 7 feet NAVD 88 at the inland ties back to the high ground and other locations which are not subject to wave impacts. More information about Alternative 1 can be found in **Subappendix C2**. With the 1.2 feet SLR scenario (the 2075 NOAA Intermediate-Low projection), there is anticipated to be approximately a 10 percent annual chance of exceedance of the 7 feet NAVD 88 crest elevation at the end of the design period. The overtopping frequency is a median value and does not include freeboard. When flood elevations exceed elevation 7 feet NAVD 88, the storm surge would begin to flow over and around the LOP. In areas subject to wave impacts, an additional 1 foot is added to the crest of the LOP to reduce wave overtopping rates that would compromise the stability of the structure and could reasonably be accommodated by the interior drainage facilities.

The impact of the proposed Alternative 1 LOP is analyzed by comparing the simulated maximum WSEL between the cases with and without the LOP. For this report, a 10-year coastal storm and 100-year coastal storm plus SLR are simulated.

Figure B1-41 to **Figure B1-43** present the model simulation results and impact assessment for a 10-year coastal storm with 1.2 feet SLR under baseline conditions (existing conditions without a LOP) and conditions with the Alternative 1 LOP implemented. As shown in **Figure B1-41** and **Figure B1-42**, the proposed LOP acts effectively as a flood barrier since flooding in the Project Area is significantly reduced. **Figure B1-43** shows that, outside the Project Area, the difference in maximum WSEL caused by the installation of the Alternative 1 LOP is around 2 inches. Inside the Project Area, the difference is slightly greater, but its magnitude remains below 7 inches.

Figure B1-44 to **Figure B1-46** present the model simulation results and impact assessment for the same 10-year coastal storm but with 2.4 feet SLR (the 2075 NOAA Intermediate-High projection) under baseline conditions and conditions with the Alternative 1 LOP implemented. It can be observed that, although there is more flooding in the baseline conditions due to the higher SLR (see **Figure B1-44**), the resulting impact with the Alternative 1 LOP implemented (see **Figure B1-45**) is still much less than that in the conditions without a LOP.

Figure B1-47 to **Figure B1-49** present the model simulation results and impact assessment under a 100-year coastal storm with 1.2 feet SLR for baseline conditions (existing conditions without a LOP) and conditions with the Alternative 1 LOP implemented. **Figure B1-47** and **Figure B1-48** show that, although the proposed Alternative 1 LOP designed with 10-year storm crest elevation is overtopped under the 100-year storm, it still reduces the flooding in the Project Area in terms of the total area of inundation and the depth of inundation. **Figure B1-49** shows that, outside of the Project Area, the impact area is more widespread than in a 10-year storm, but the increased maximum surface elevation is still around 2 inches. Inside the Project Area, the biggest impact of approximately 12 inches occurs near the Meadowlands Race Track and the area between the Alternative 1 LOP at Berry's Creek and the west edge of the Project Area. The Teterboro Airport area also experiences a difference of about 10 inches. The increase of water level in the Project Area is caused by local bathymetry. For example, the increase of water level next to the Meadowlands Race Track is caused by the flooding and local bathymetry. The proposed storm surge barrier at Berry's Creek causes flooding in the area nearby, and because the ground elevation at that spot is very low (like a pond), water is trapped there and results in a 6 feet difference. As for the increased water level at Teterboro Airport in **Figure B1-49**, it is caused by the water overtopping the LOP when the peak of storm surge comes. However, after the peak time, the water is retained behind the LOP in the Project Area instead of draining into Hackensack River as it does in conditions without the proposed LOP. Additionally, local bathymetry at Teterboro Airport causes flood water to be trapped there like a pond, which is difficult to drain and results in increased water level at Teterboro Airport.

Figure B1-50 to **Figure B1-52** present the model simulation results and impact assessment under a 100-year coastal storm with 2.4 feet SLR for baseline conditions and conditions with the Alternative 1 LOP implemented. Despite the higher SLR condition, reduction of flooding is observed with the LOP implemented (see **Figure B1-50** and **Figure B1-51**). Under this condition, water elevation difference at the Teterboro Airport is no longer present because, with the higher SLR, this area is inundated regardless of the LOP implementation.

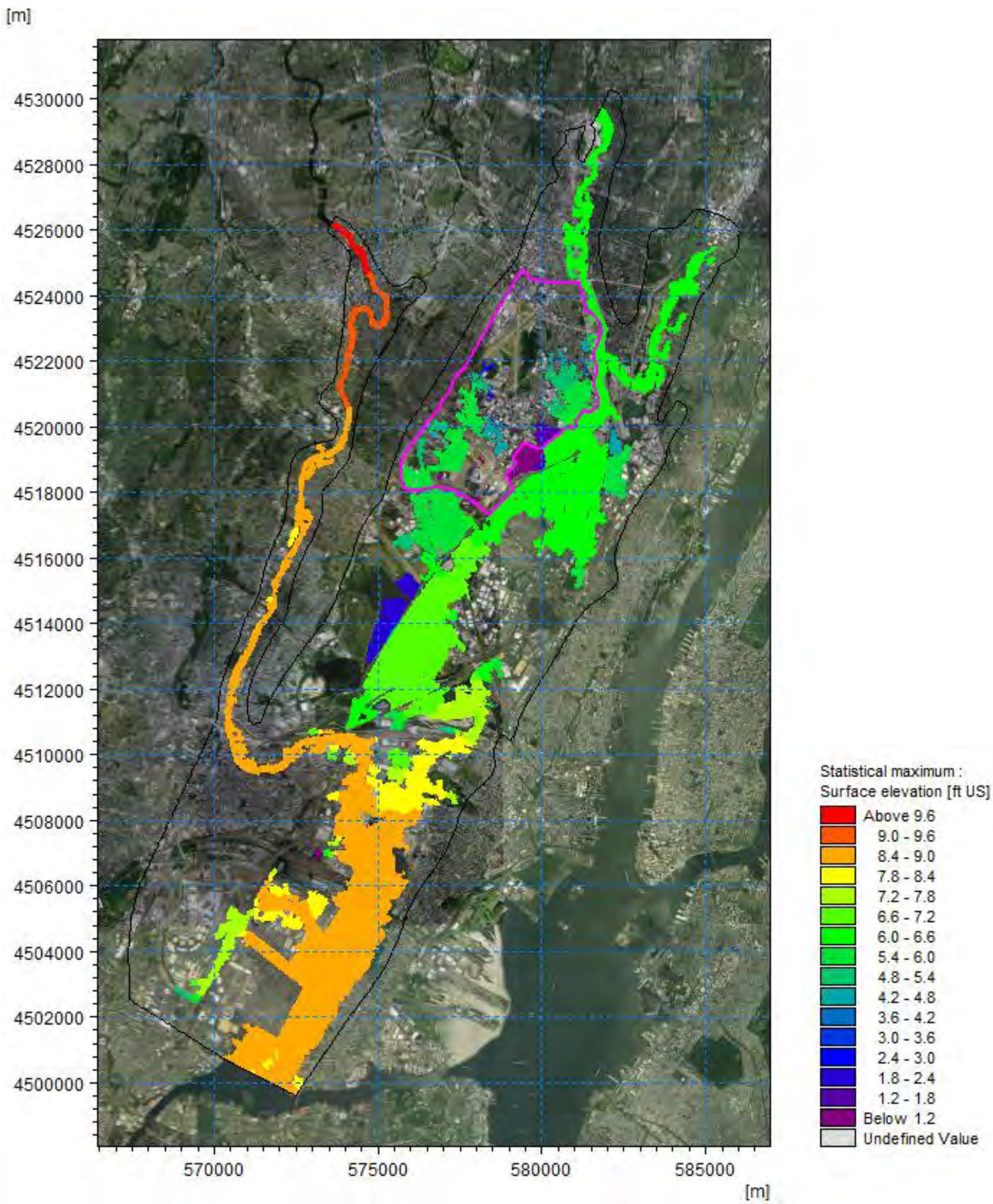


Figure B1-41: Maximum WSEL (NAVD 88) Caused by the 10-year Storm and 1.2 feet SLR Baseline Simulation (Without Alternative 1 LOP)

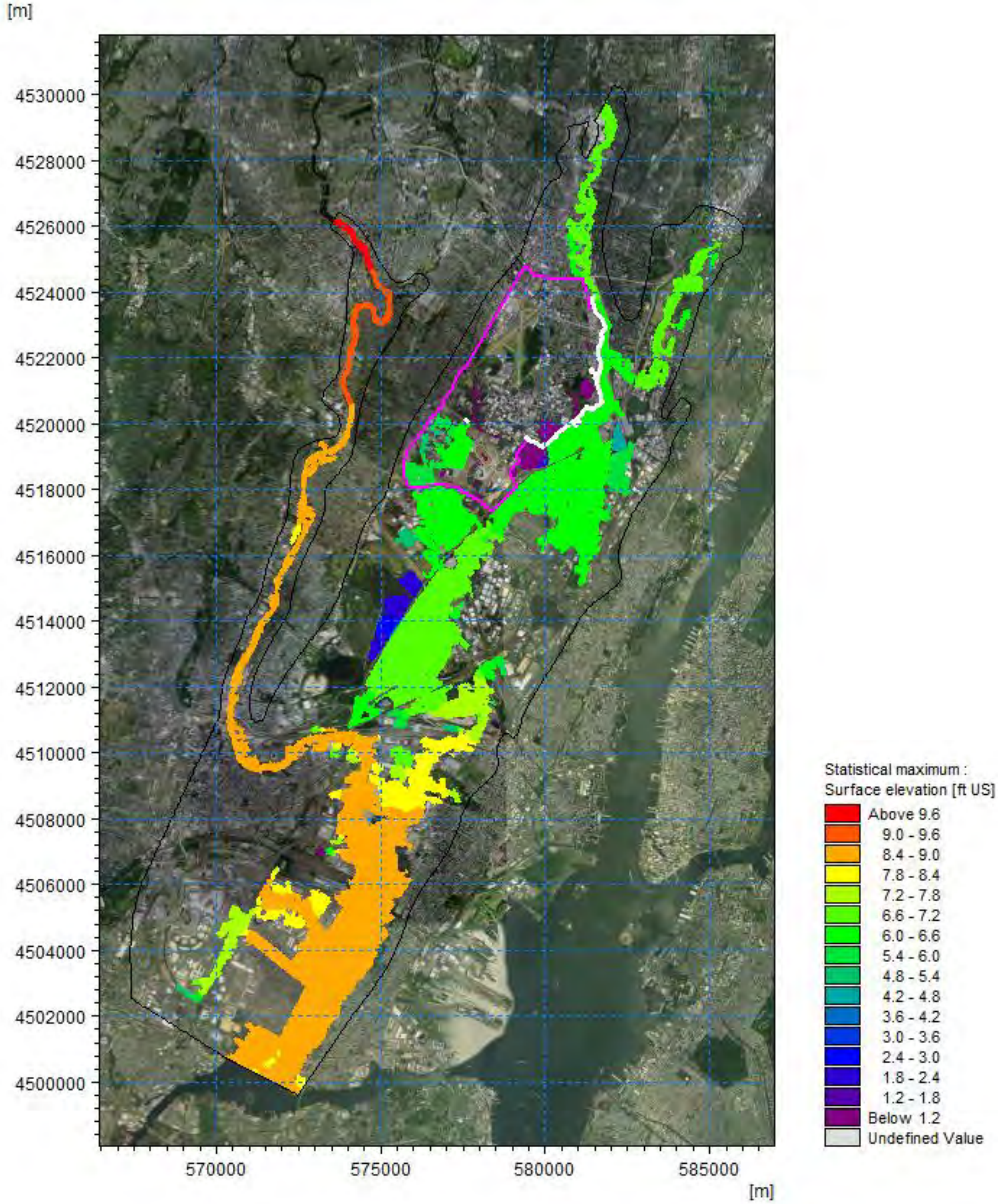


Figure B1-42: Maximum WSEL (NAVD 88) Caused by the 10-year Storm and 1.2 feet SLR Simulation with Alternative 1 LOP

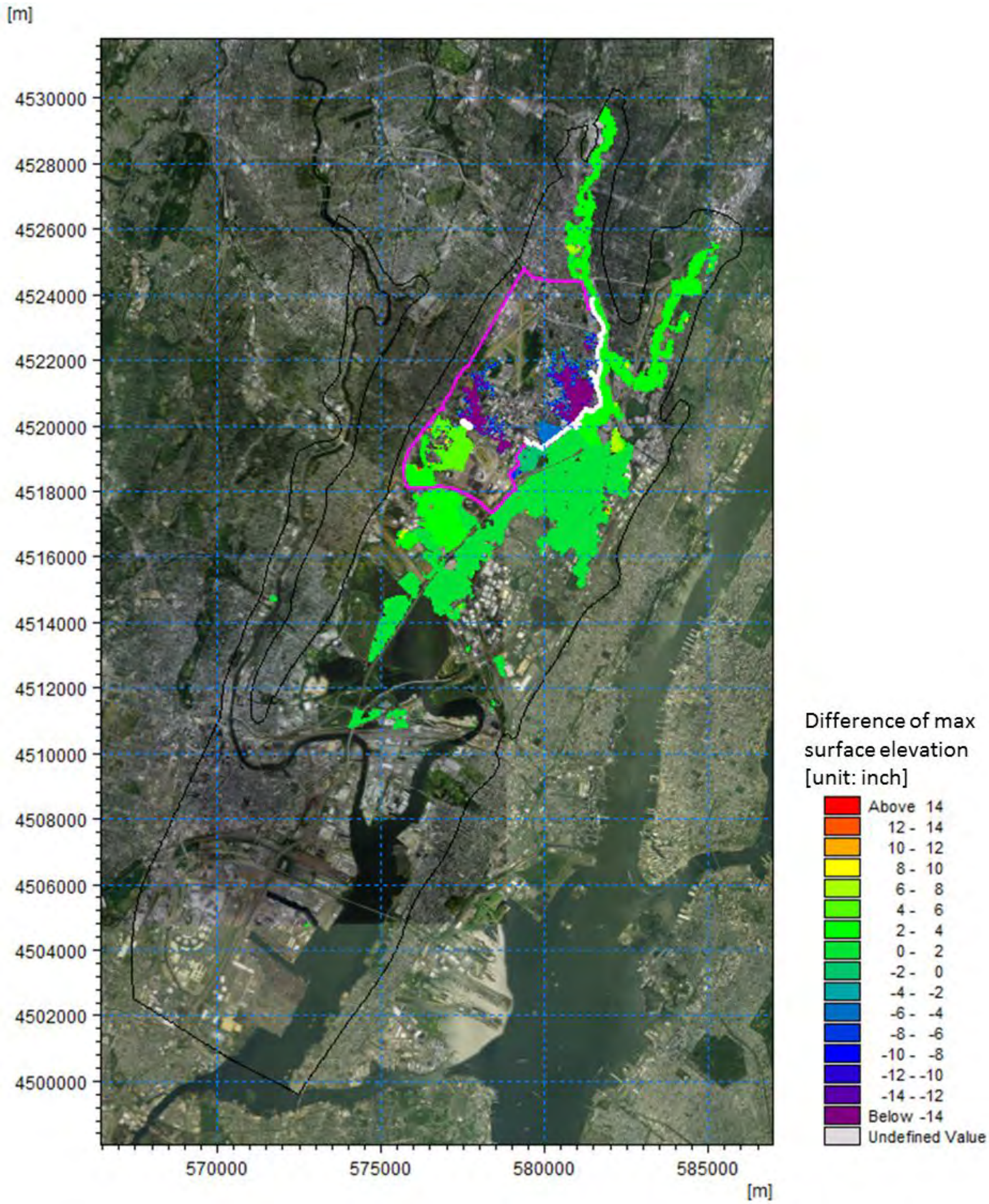


Figure B1-43: Impact of Alternative 1 LOP Represented by Difference of Maximum WSEL between the 10-year Storm and 1.2 feet SLR Simulations With and Without Alternative 1 LOP

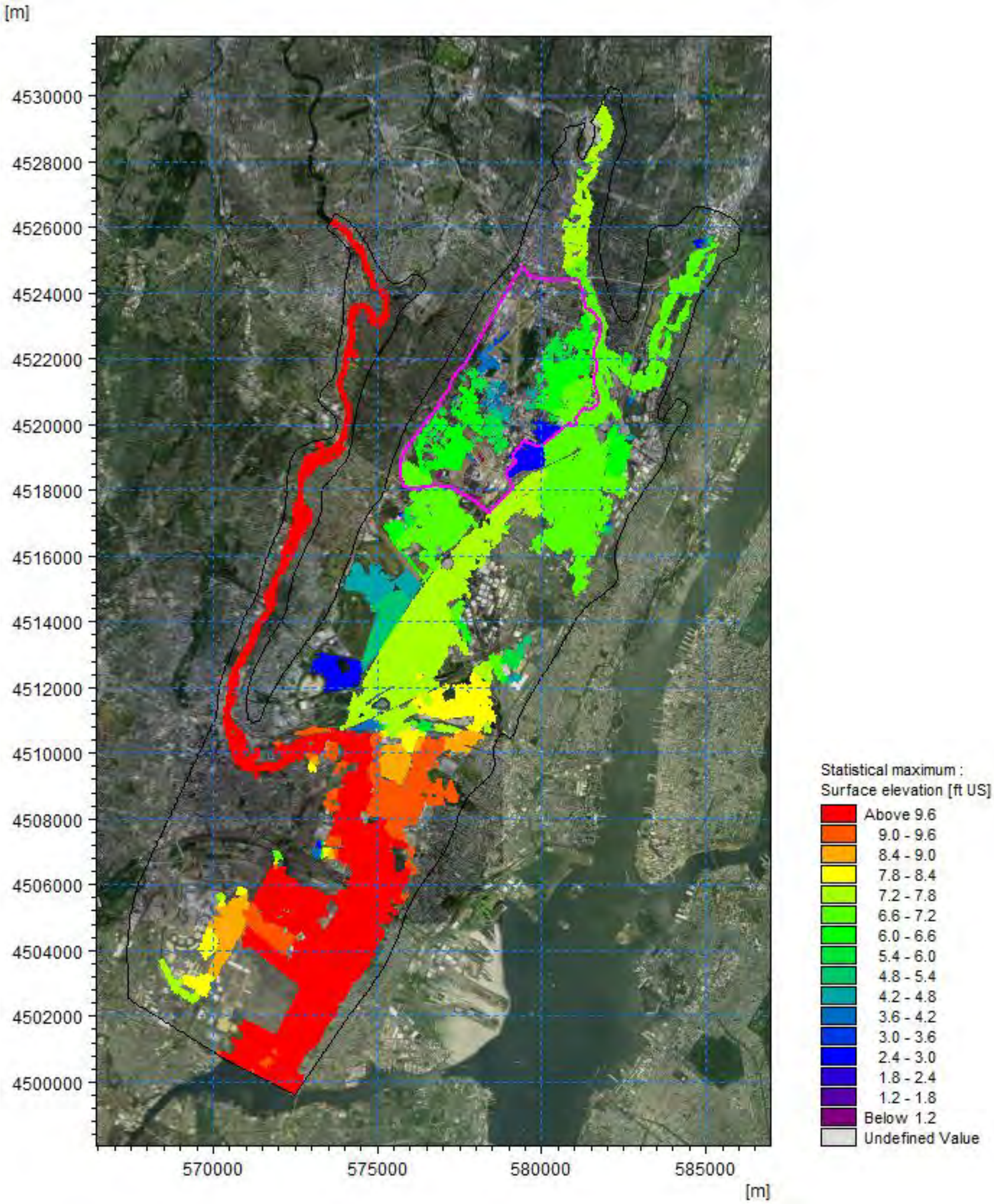


Figure B1-44: Maximum WSEL (NAVD 88) caused by the 10-year Storm and 2.4 feet SLR Baseline Simulation (Without Alternative 1 LOP)

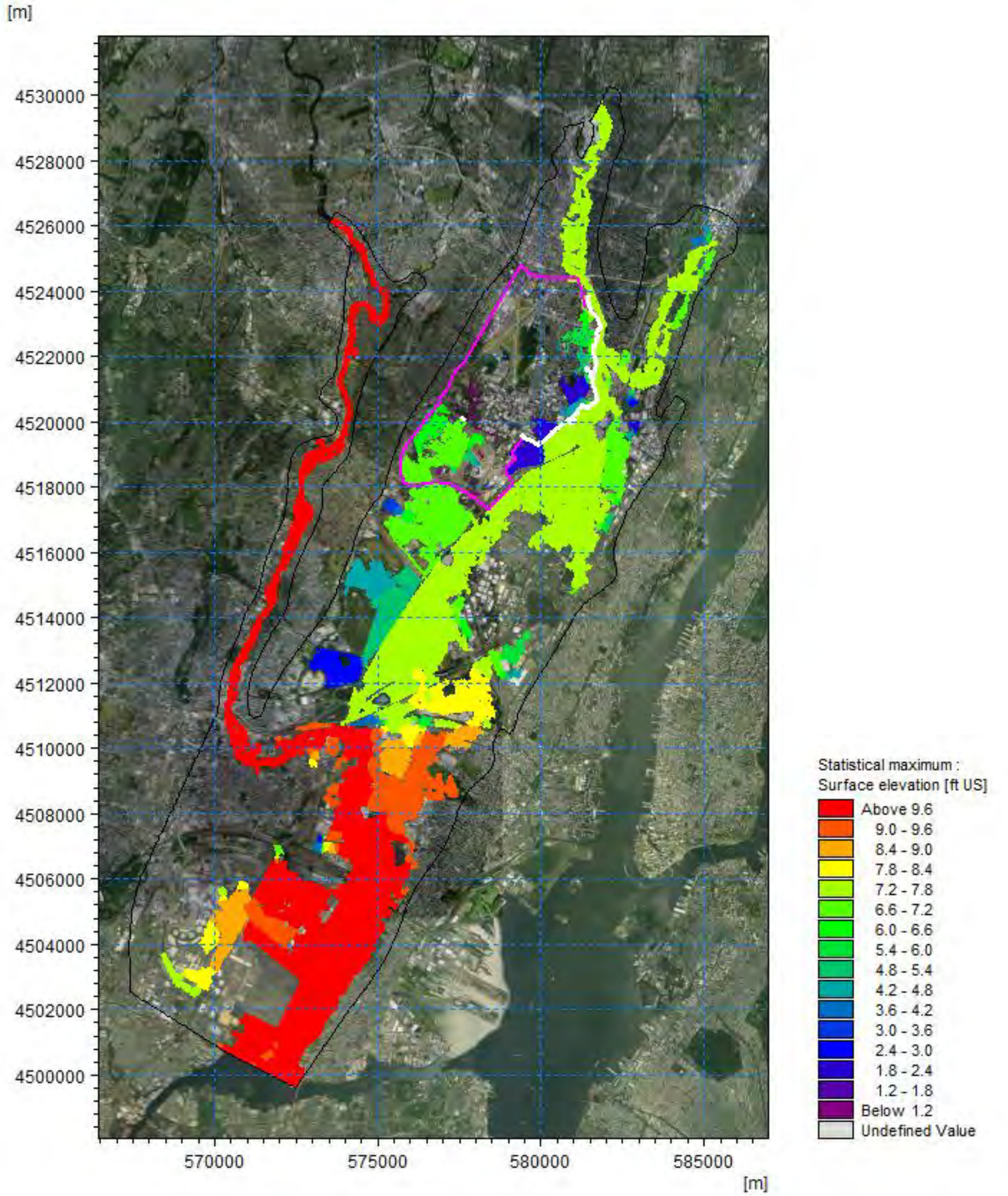


Figure B1-45: Maximum WSEL (NAVD 88) Caused by the 10-year Storm and 2.4 feet SLR Simulation With Alternative 1 LOP

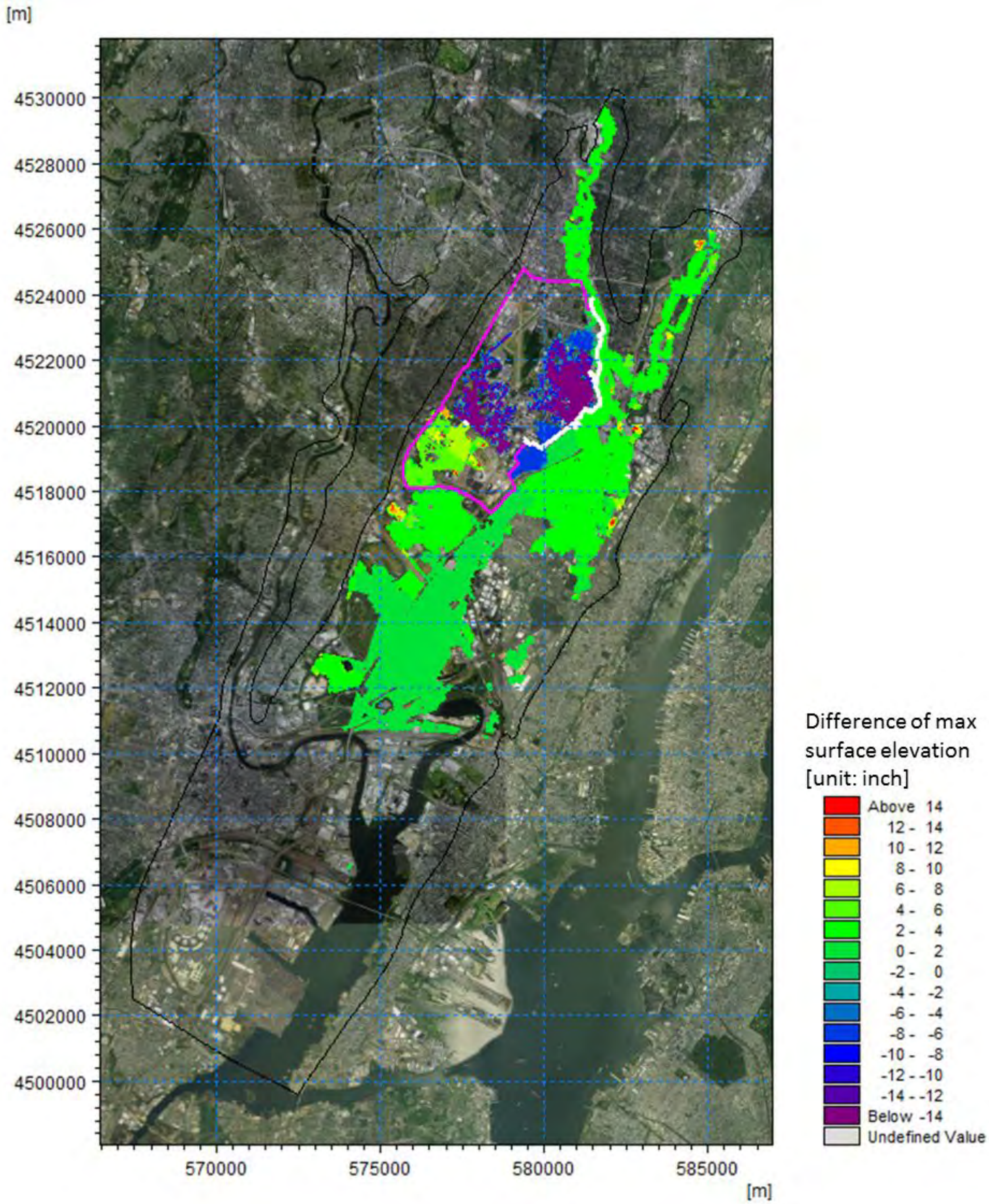


Figure B1-46: Impact of Alternative 1 LOP Represented by Difference of Maximum WSEL between the 10-year Storm and 2.4 feet SLR simulations With and Without Alternative 1 LOP

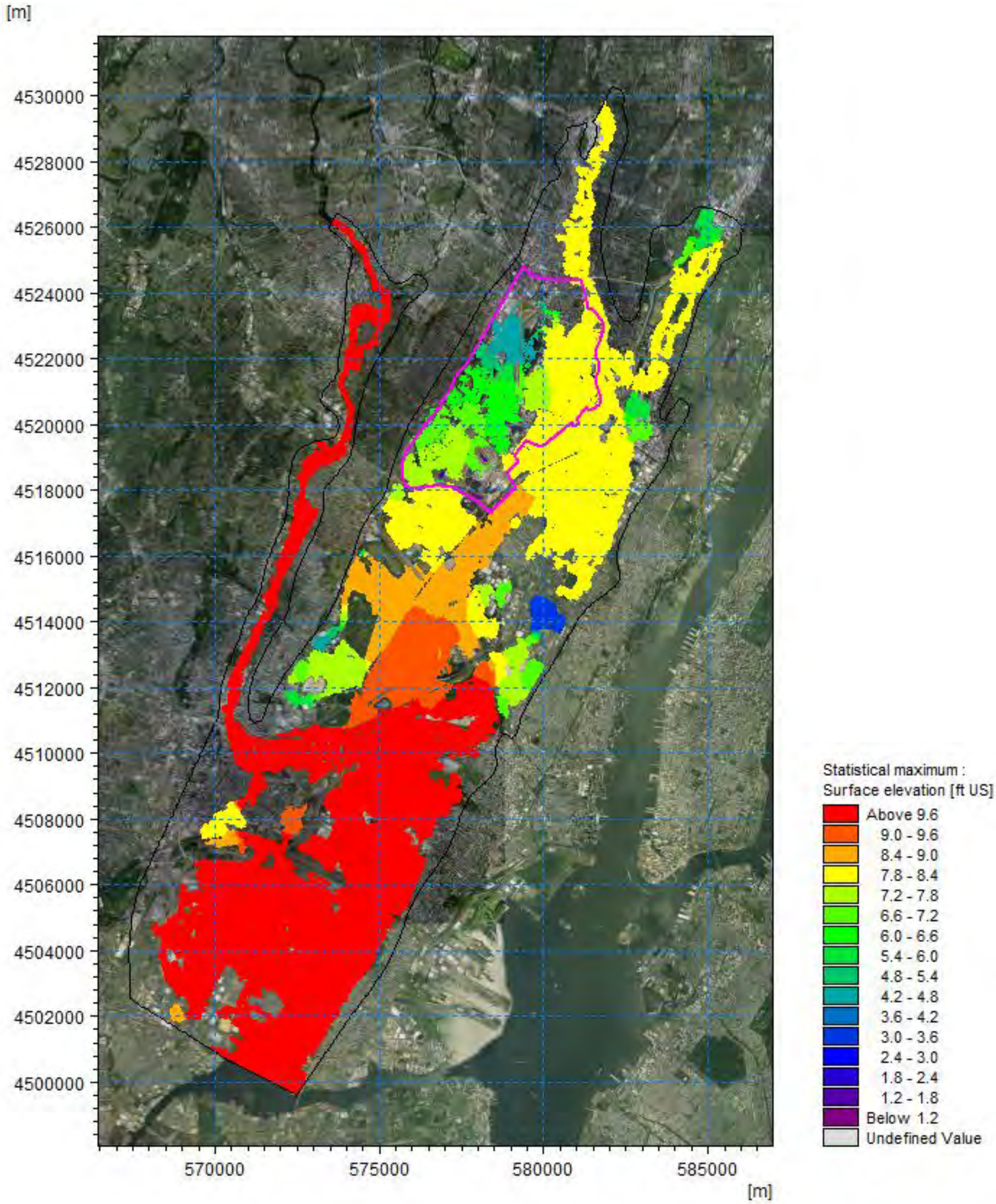


Figure B1-47: Maximum WSEL (NAVD 88) Caused by the 100-year Storm and 1.2 feet SLR Baseline Simulation (Without Alternative 1 LOP)

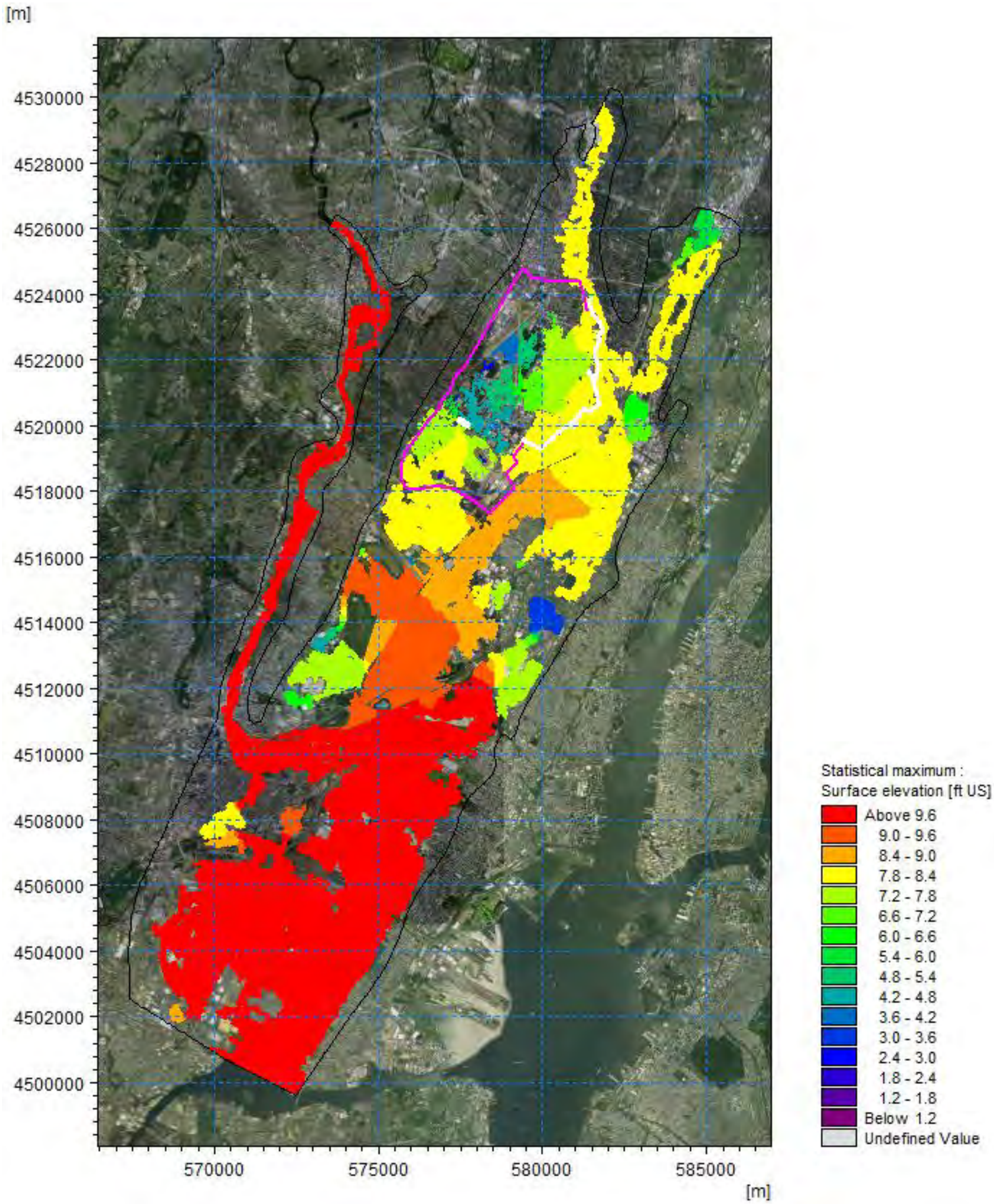


Figure B1-48: Maximum WSEL (NAVD 88) Caused by the 100-year Storm and 1.2 feet SLR Simulation With Alternative 1 LOP

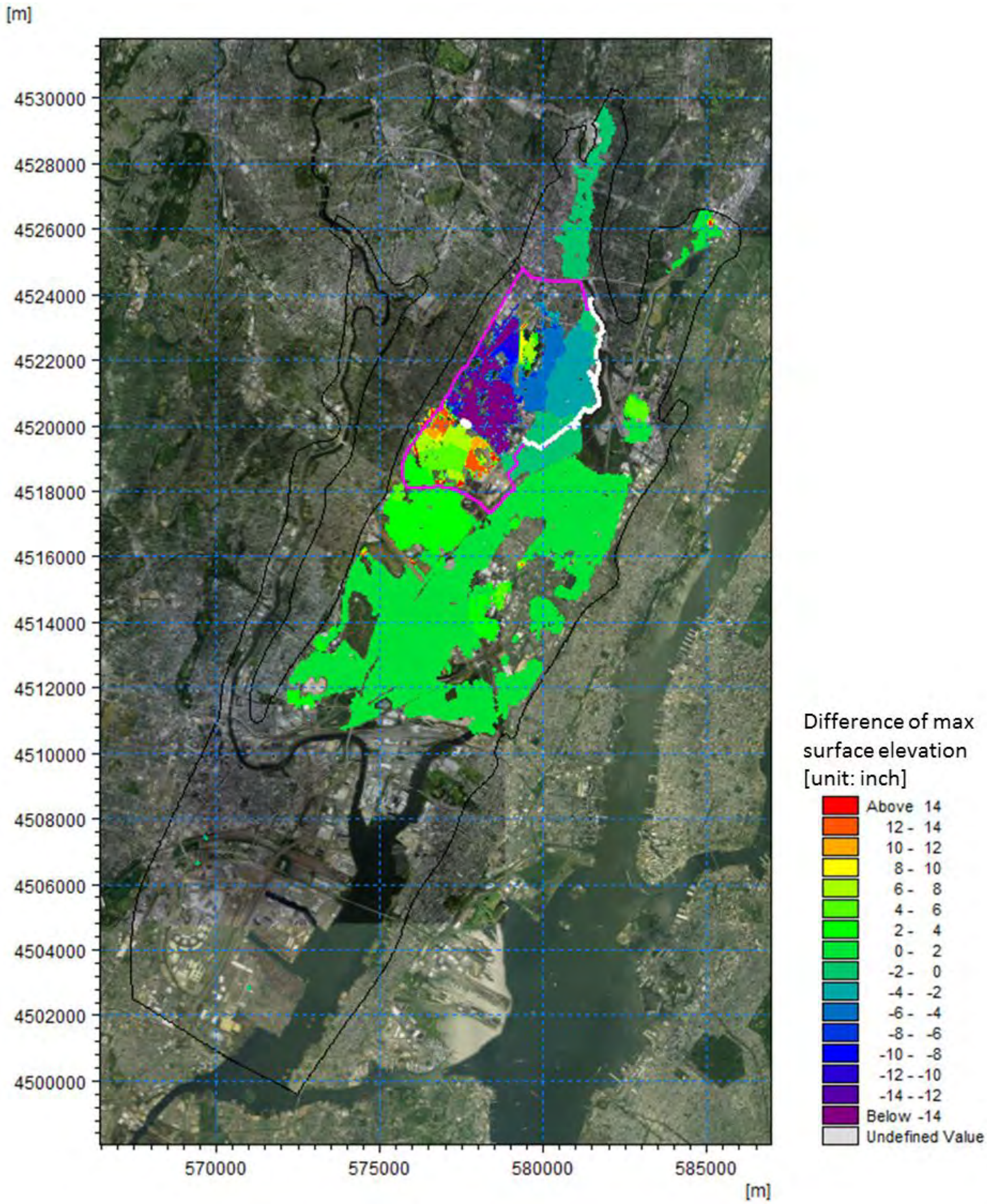


Figure B1-49: Impact of Alternative 1 LOP Represented by Difference of Maximum WSEL between the 100-year Storm and 1.2 feet SLR Simulations With and Without Alternative 1 LOP

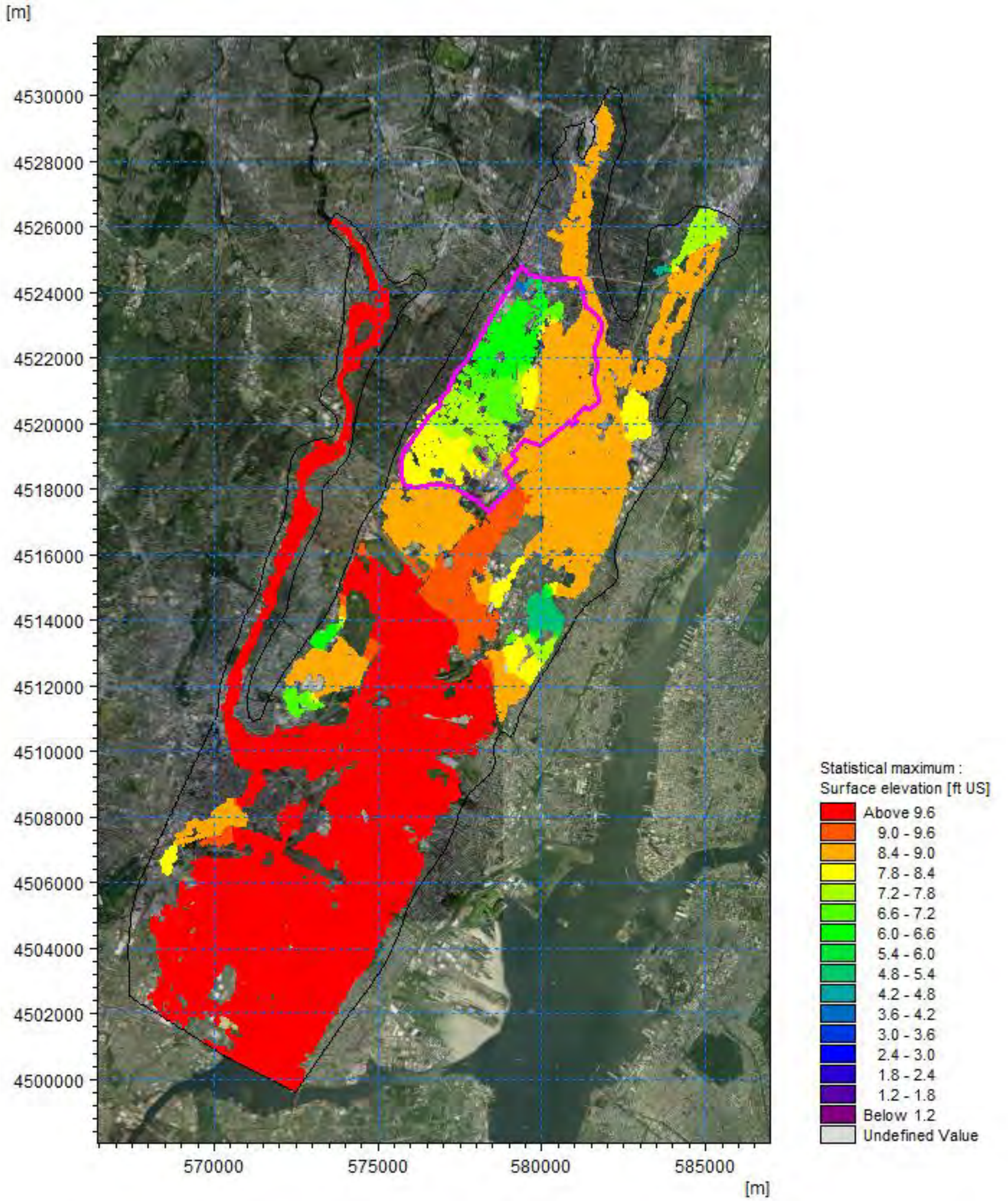


Figure B1-50: Maximum WSEL (NAVD 88) Caused by the 100-year Storm and 2.4 feet SLR Baseline Simulation (Without Alternative 1 LOP)

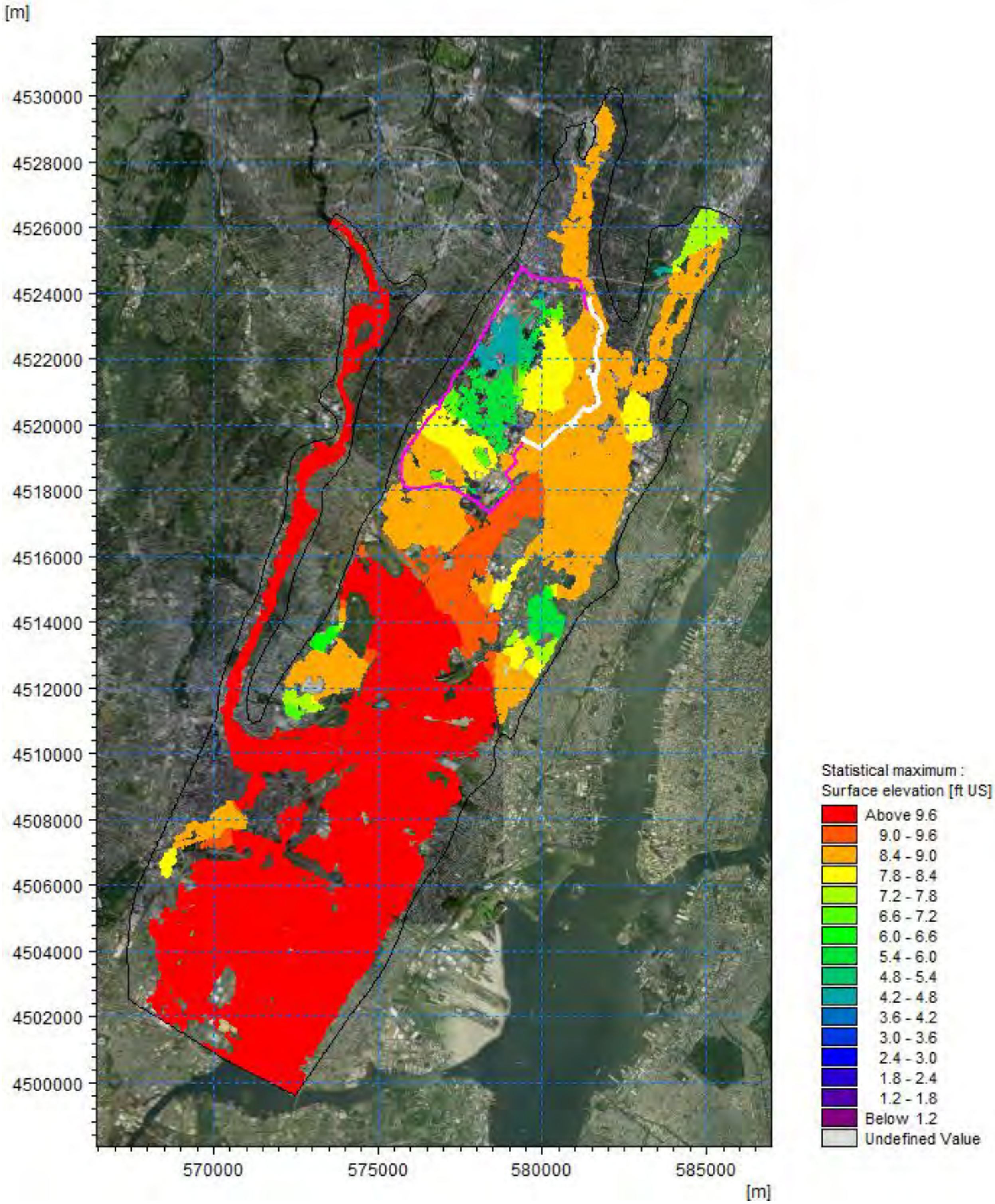


Figure B1-51: Maximum WSEL (NAVD 88) Caused by the 100-year Storm and 2.4 feet SLR Simulation With Alternative 1 LOP

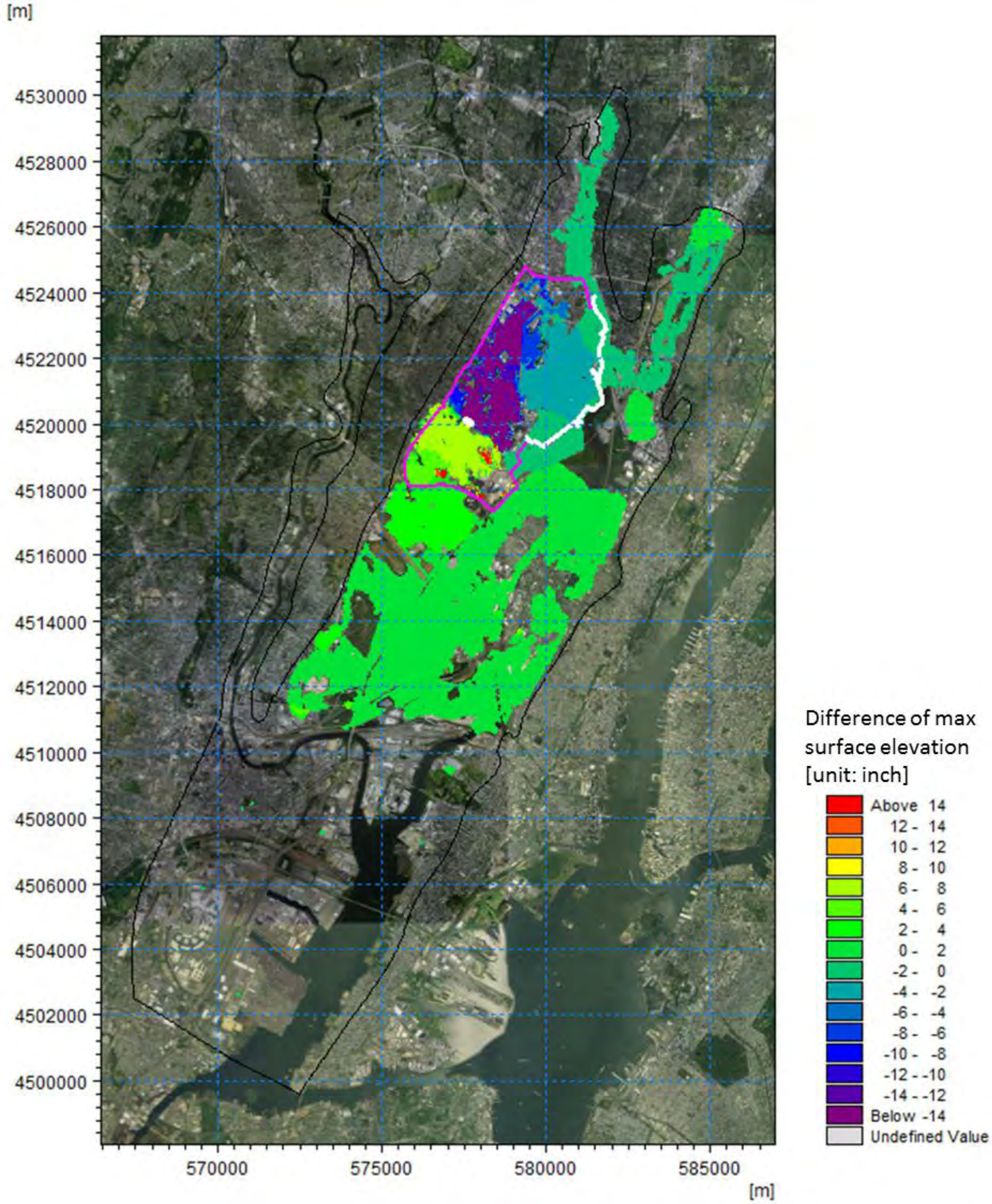


Figure B1-52: Impact of Alternative 1 LOP Represented by Difference of Maximum WSEL between the 100-year Storm and 2.4 feet SLR Simulations With and Without Alternative 1 LOP

4.0 Conclusion and Recommendation

The major conclusions from developing a coastal model system and coastal modeling for flood assessment are summarized below:

- Significant efforts were made in the development process of this coastal modeling system. From model calibration, it can be demonstrated that model simulation results generally have good agreement between model-predicted and measured tidal elevations. From model validation, it can be demonstrated that storm surge time series have generally a close agreement between model-simulated and measured elevations.
- Preliminary flood assessment was performed for the proposed Alternative 1 LOP by comparing the distributions of the maximum WSEL with and without this LOP. Under 10-year storm conditions, the impact in terms of increased maximum WSEL is around 2 inches outside of the Project Area, and can reach 7 inches in limited regions inside the Project Area. Under 100-year storm conditions, the proposed Alternative 1 LOP designed with 10-year storm crest elevation fails to protect the Project Area, with the water overtopping the LOP and flooding the majority of the Project Area. The impact outside the Project Area expands, but remains about the same value (2 inches) as in the 10-year storm conditions, while the impact inside the Project Area is slightly larger (12 inches maximum) and wider (i.e., Teterboro Airport is also affected).
- The developed present coastal model system is ready for production runs to investigate alternatives to the LOP and is competent as the valuable numerical tool for the Proposed Project objectives.
- Building obstruction is not included in the current feasibility level of study. Model refinement, including building obstruction, may be potential in a more detailed design phase in the future.
- Further bathymetry survey in certain areas (e.g., the wetlands) and the modification and refinement of this coastal model system could be made in response to further design needs and needs of future Proposed Project phases.
- Since the Meadowlands District study area is affected by both hurricanes and Nor’easters, a study of Nor’easter’s effect would be carried out in the future by scaling up and down the historical Nor’easters to Nor’easter with 10-year return period.

5.0 References

FEMA. *Preliminary Flood Insurance Study – Bergen County.* Bergen County, New Jersey, 2014.

—. *Preliminary Flood Insurance Study – Hudson County.* Hudson County, New Jersey, 2013.

RAMPP. *Region II Coastal Storm Surge Study Overview.* FEMA, 2014a.

—. *Region II Storm Surge Project - Development of Wind and Pressure Forcing in Tropical and Extratropical Storms.* FEMA, 2014c.

—. *Region II Storm Surge Project - Spatially Varying Nodal Attribute Parameters.* FEMA, 2014b.

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Subappendix B2: Hydrology and Rainfall Tide Correlation Analysis

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Table of Contents

List of Tables

List of Figures

Acronyms and Abbreviations

ARI	Annual Recurrence Interval
BCUA	Bergen County Utilities Authority
cfs	Cubic feet per second
CN	Curve number
D	Diversion/flow split
DEM	Digital elevation model

DA	Drainage Area
EC	Engineering Circular
FEMA	Federal Emergency Management Agency
GIS	Geographic Information System
HEC-HMS	Hydrologic Engineering Center - Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center - River Analysis System
HUD	Department of Housing and Urban Development
IPCC	Intergovernmental Panel on Climate Change
J	Flow Junction
LiDAR	Light Detection and Ranging
mm/year	Millimeters per year
MACA	Multivariate Adaptive Constructed Analogs
NAVD 88	North American Vertical Datum of 1988
NJDEP	New Jersey Department of Environmental Protection
NJGIN	New Jersey Geographic Information Network
NJSEA	New Jersey Sports and Exposition Authority
NOAA	National Oceanic and Atmospheric Administration
NRC	National Research Council
NRCS	Natural Resources Conservation Service
P	Ponding area
PFDS	Precipitation Frequency Data Server
R	Reach routing
RBDM	Rebuild by Design Meadowlands
RSLC	Relative sea level change
SLC	Sea level change
SLR	Sea level rise
SSURGO	Soil Survey Geographic Database
Tc	Time of concentration
TR-55	Technical Release No. 55
US	United States
USACE	United States Army Corps of Engineers
WSEL	Water surface elevation

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1.0 Introduction

This appendix documents the hydrologic analysis of the Rebuild by Design Meadowlands (RBDM) Flood Protection Project Area. This program is managed by the New Jersey Department of Environmental Protection (NJDEP) and the United States (US) Department of Housing and Urban Development (HUD). The Proposed Project is scheduled to be constructed by September 2022. Typical flood mitigation features include walls, levees, road raising, pump stations, and improved outfalls. Details on the program are available at the NJDEP website.

The hydrologic analyses were performed with a computer model with inputs of Geographic Information System (GIS)-derived physical characteristics as land topography, channel slope, soil hydrologic group and land use / land cover. Also analyzed in this document are the tailwater tidal waveforms with storm surge and the correlation with fluvial storm events.

2.0 Project Area

The Project Area lies along the Hackensack River and encompasses four boroughs and one township: the Borough of Carlstadt, the Borough of Moonachie, the Township of South Hackensack, the Borough of Little Ferry, and the Borough of Teterboro. The Project Area is exposed to approximately nine miles of river bank extending along the Hackensack River (see **Figure B2-1**). The northern-most section of river bank begins at the municipal boundary between the Borough of Little Ferry and the City of Hackensack. The southern-most section of river bank ends at the municipal boundary between the Boroughs of Carlstadt and East Rutherford. The approach to the Hackensack River from deep water in the ocean is through the Lower and Upper New York Bay and then through Newark Bay.

2.1 Physical Characteristics

The interior drainage area is approximately 7 square miles. The majority of the topography is flat or somewhat flat, especially near the Hackensack River. The area west of Route 17 rises up rather steeply. Elevations vary from 211 feet (North American Vertical Datum of 1988 (NAVD 88)) in the Paterson Plank Road Bridge drainage area, the highest elevation area, to -8 feet (NAVD 88) at the lowest elevation area around Willow Lake with Berry's Creek and the Hackensack River at about -2 feet (NAVD 88). The majority of the area is urban, consisting of residential, commercial, and industrial establishments. Most of the residential areas are within the Borough of Little Ferry. A portion of the Project Area is occupied by the Teterboro Airport, which has experienced extensive flooding.

The floodplain of Berry's Creek, a flat tidal creek, is populated by mostly commercial or industrial developments.

Route 46 cuts through the upper third of the Project Area and runs east and slightly south in the downhill direction. The north and south of the Project Area are lined with two other major roads, Route 80 in the north and Route 120 in the south. Moonachie Avenue is another major auto route, which runs roughly parallel to, and south of, Route 46. These, along with other roads, act as barriers that concentrate surface runoff to channels and closed conduits through the low-lying areas of the Project Area to the Hackensack River. The major storm sewers that lie under some of these roads serve to convey upland interior runoff to the Hackensack River.

2.2 Sources of Flooding

Flooding in the Project Area can result from either coastal storm surges via the Hackensack River or Berry's Creek or rainfall runoff that drains to the Hackensack River or Berry's Creek, but exceeds the capacity of the existing drainage system. The Project Area is partially protected from storm surges by existing high ground, constructed berms, tide gates, and pump stations. These existing features provide relief from tidal surges during high frequency storm events (e.g., a 2-year coastal storm event), but for higher surge levels, large and low-lying portions of the inland area become inundated causing extensive property damages and risks to life-safety.

The frequency of inland inundation will continue and increase as sea level is projected to rise. Relative sea level in the Project Area has been rising at an average of 0.01 feet per year (Horton, et al. 2015). It is also anticipated that continued development and fill placement will occur within the floodplain. As new construction is elevated above the base interior flood elevation, the fill would reduce storage for interior runoff and may exacerbate interior flooding conditions during high intensity rainfall events.

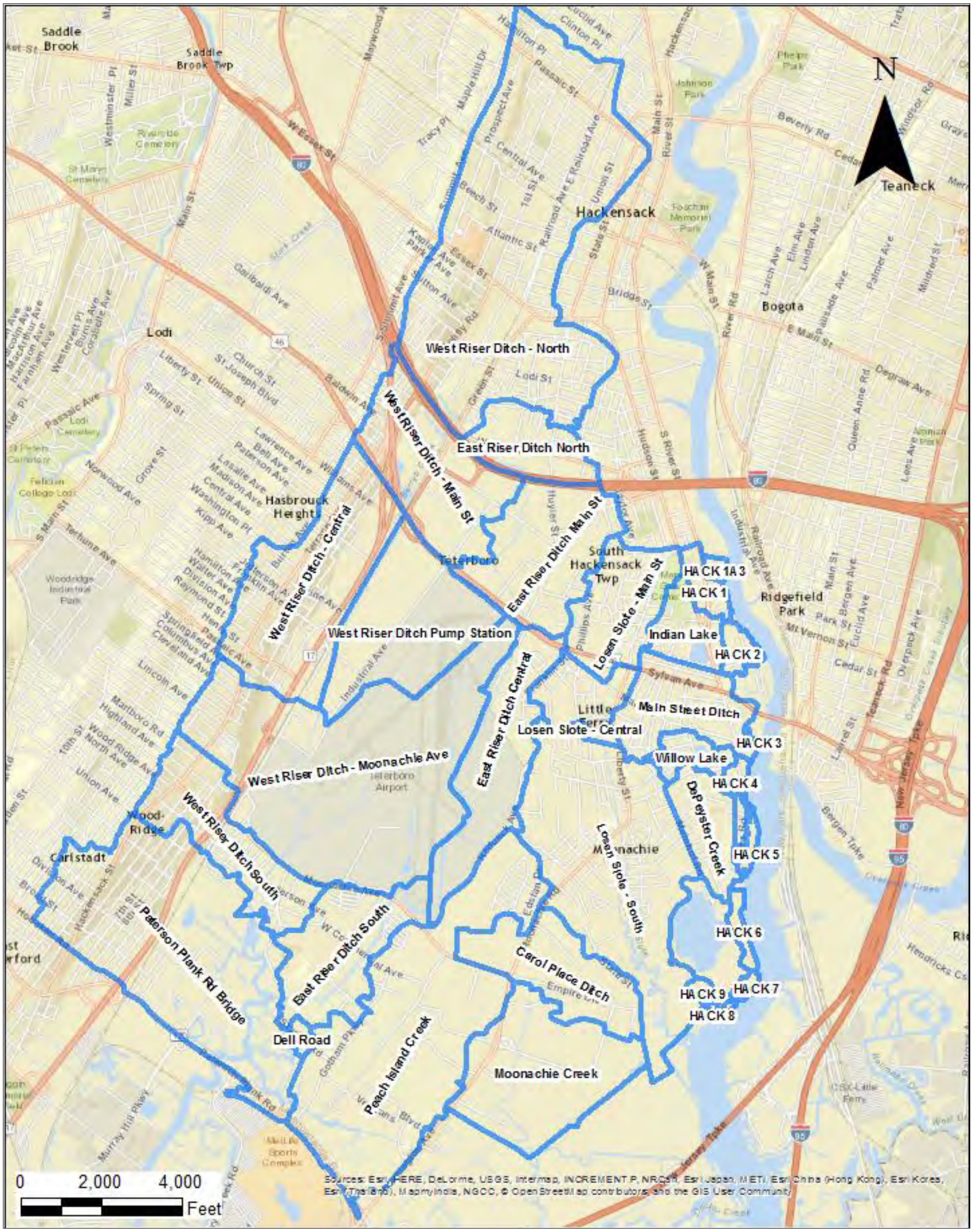

Figure B2-1: RBDM Drainage Areas

3.0 Local Hydrologic and Tidal Conditions

This section presents the rainfall data for the fluvial events and the tidal surge data for marine events for the present rainfall data and rainfall data scaled to future conditions.

3.1 Present Precipitation Data

All specific frequency hypothetical point rainfall depths were from National Oceanic and Atmospheric Administration's (NOAA's), National Weather Service Precipitation Frequency Data Server (PFDS) centered on the Borough of Moonachie as downloaded in October 2016 and unchanged as of March 2018 (NOAA 2017). This data was download Hypothetical point rainfall depths for the 1- through 500-year storms are shown in **Table B2-1**:. Future precipitation data is presented in **Section 5.0**.

Table B2-1: Specific Frequency Hypothetical Point Rainfall Depths in Inches

Duration	Return Period							
	2-year Rainfall [in]	5-year Rainfall [in]	10-year Rainfall [in]	25-year Rainfall [in]	50-year Rainfall [in]	100-year Rainfall [in]	250-year Rainfall [in]	500-year Rainfall [in]
5 min	0.40	0.48	0.53	0.61	0.66	0.71	0.77	0.82
15 min	0.79	0.95	1.06	1.19	1.29	1.39	1.50	1.59
1 hour	1.35	1.69	1.95	2.3	2.57	2.85	3.18	3.50
2 hours	1.66	2.1	2.44	2.91	3.29	3.69	4.20	4.67
3 hours	1.85	2.34	2.73	3.26	3.7	4.15	4.74	5.29
6 hours	2.39	3.01	3.5	4.2	4.78	5.38	6.17	6.93
12 hours	2.94	3.72	4.35	5.27	6.05	6.87	7.99	9.07
24 hours	3.31	4.23	5.01	6.18	7.18	8.28	9.80	11.30
48 hours	3.87	4.95	5.85	7.18	8.32	9.57	11.25	13.0

A 48-hour hypothetical storm was used to allow for Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) interior inflow routing against the exterior time-varying marigrams (astronomic tide plus storm surge) through two tide cycles.

3.2 Hypothetical Storm Surge Data

For storm events (tropical events such as hurricanes and extratropical events such as nor'easters), a storm hydrograph was developed to simulate surge levels during storm conditions. Two main assumptions were made to develop the storm hydrograph: (1) the peak elevation of the storm would occur at high tide; and (2) the duration of the storm would be approximately two days. Storm hydrographs were developed for return periods from 2-year to 500-year and the peak elevation for each return period was developed as described in **Subappendix B1**. Hypothetical tide marigrams (hydrographs) used in this interim study for the exterior stages are plotted in **Figure B2-2**. The storm surge data utilizes stage frequency curves from the Federal Emergency Management Agency's (FEMA's) Region 2 coastal storm surge modeling for New York and New Jersey. The data for this area is presented in the Bergen County Preliminary Flood Insurance Study (FEMA 2014).

The relationship between rainfall / runoff (including river flow) and storm surge is uncertain and may have a significant impact on interior stages. Uncertainty was incorporated into the analysis by routing the interior storm events against an exterior typical tidal condition to establish a lower bound of interior flood levels, and routing the interior events against exterior 10-year tidal surge conditions to establish a reasonable upper bound of interior flood levels. This methodology was then applied with a 2-year external surge level to create the “Most Likely” interior flood levels. The three conditions: “Most Likely” (design), “lower bound,” and “upper bound” were then incorporated into the economic analysis using a triangular probability distribution.

3.3 Storm Surge Duration

While storms with longer surge duration are possible, multiple peak conditions have a significantly lower probability of occurring. Accordingly, a 48-hour storm surge event was used for all cases.

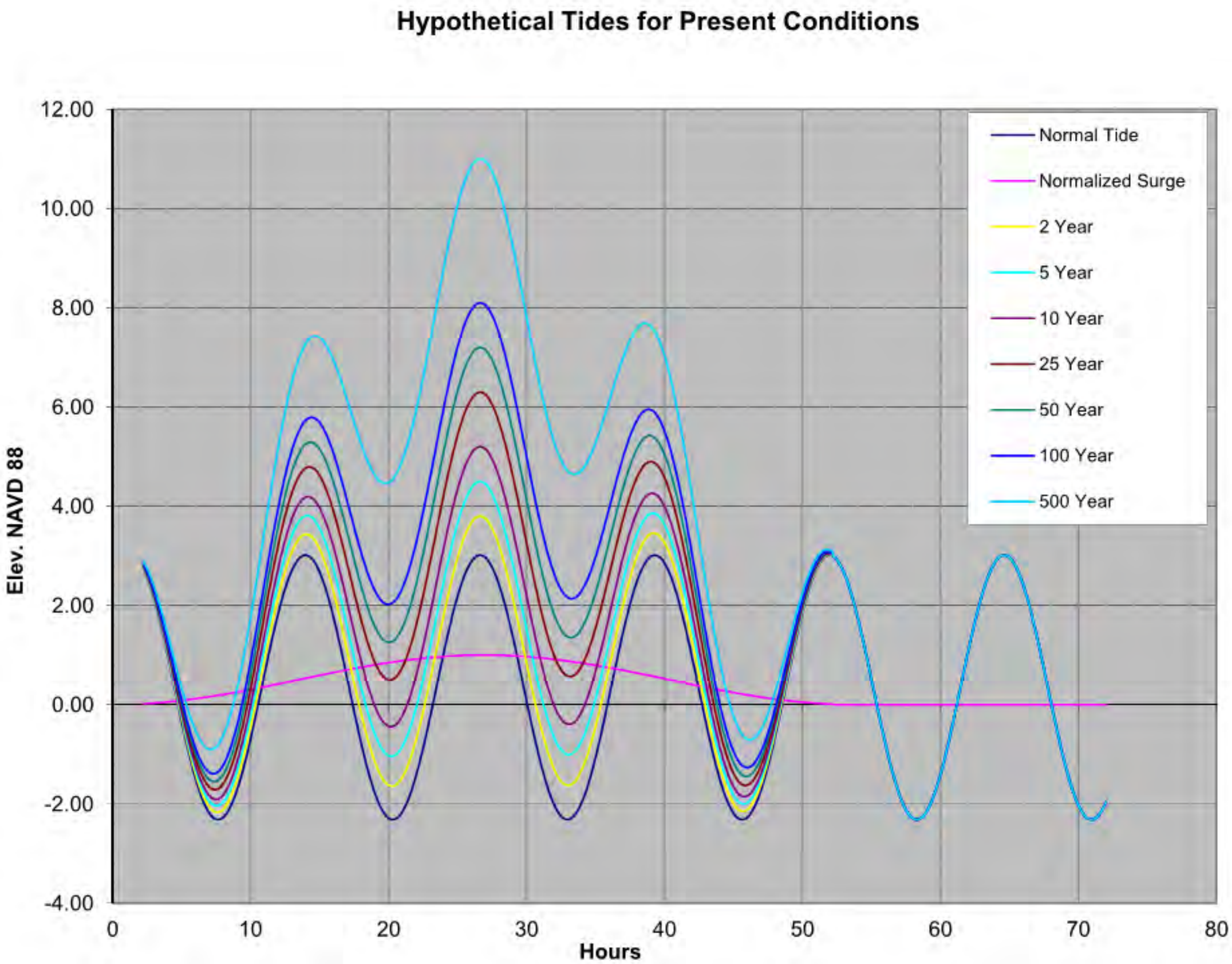

Figure B2-2: Present Condition Marigrams Applied as Exterior Stages in Models

4.0 Rainfall and Tidal Surge Correlation Analysis

For the “Without” and “With” the Proposed Project conditions, the exterior stage (still water elevation within Hackensack River) is an important factor in the drainage of the interior precipitation runoff. The exterior stage is controlled by the tide cycle and storm surge elevations during storm events. Inland, the interior surface runoff is conveyed out into the Hackensack River through the existing high ground via stormwater outfalls. In the “Without” condition, these outfalls cease to operate when the exterior stage (tide/ storm surge level) rises above the outfall opening because they rely on gravity to facilitate the transport of interior surface runoff. Similarly, if a new coastal storm risk management structure is introduced (the “With” condition) to reduce the risk of storm surge entering the Project Area, the existing outfalls, under high exterior (tailwater) stage conditions, would not be able to leave through gravity flow. Therefore, it is important to develop an understanding of whether there is a relationship between interior surface runoff and exterior tidal events in both the Without and With conditions.

To understand the relationship between the interior and exterior stage conditions, if any, a correlation analysis was performed. The correlation of both tide residual and water level with precipitation was evaluated using water level data from NOAA Station 8519483 – Bergen Point, NJ in conjunction with precipitation data from NOAA Station 725025-94741 – Teterboro Airport, NJ and NOAA Station 725020-14734 – Newark Liberty International Airport, NJ **(Figure B2-3)**. In accordance with EM 1110-2-1413, the correlation analysis included a data analysis of the correlation, dependence, and coincidence of the interior and exterior stage relationship. Tide residual / storm surge at Bergen Point does not correlate directly to precipitation in the Project Area and its peak stage is unpredictable, but could coincide with peak interior discharges. A summary of the results of the correlation analysis performed are provided in this section and its subsections.

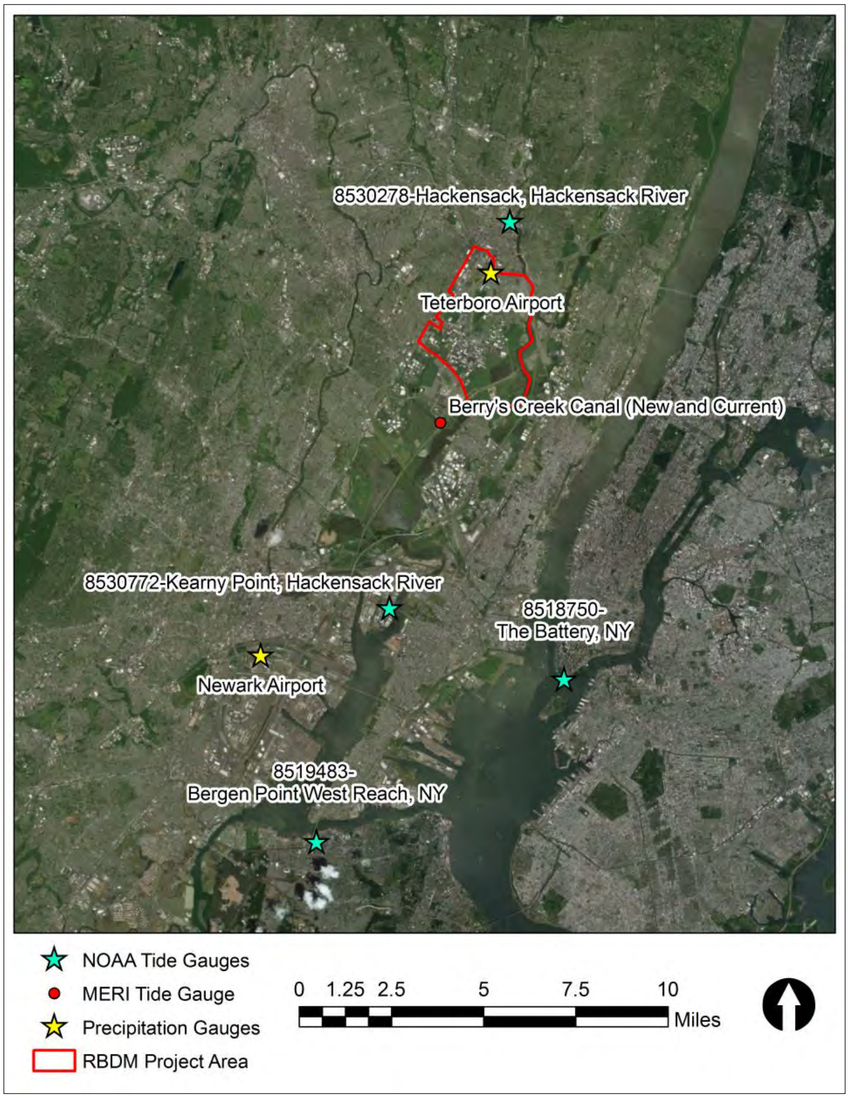

Figure B2-3: RBDM Locations used for Correlation Analysis

4.1.1.1 Correlation

Correlation is a measure of the linear relationship between two datasets. The Pearson correlation coefficient was calculated using large periods of continuous daily and hourly water level and precipitation data from 1994 through 2013 as shown in **Table B2-2**. None of the correlation coefficients calculated were considered significant (greater than 0.5); however, the strongest relationships were between daily tide residual and precipitation.

Table B2-2: Pearson Correlation Coefficients for Large Continuous Data Periods at Newark Airport and Teterboro Airport

	Newark Airport		Teterboro Airport	
	Tide Residual – Precipitation	*Water Level - Precipitation*	*Tide Residual - Precipitation*	*Water Level - Precipitation*
Analysis Period	2/17/1994 - 2/1/2007		1/2/1997 - 2/1/2007	
Correlation Coefficient (Daily)	0.33	0.20	0.31	0.18
Correlation Coefficient (Hourly)	0.10	0.03	0.16	0.05
Analysis Period	3/24/2010 - 12/23/2013		3/24/2010 - 12/23/2013	
Correlation Coefficient (Daily)	0.35	0.26	0.36	0.29
Correlation Coefficient (Hourly)	0.13	0.03	0.19	0.06

The Pearson correlation coefficient was also calculated for 14 separate tropical storm events with continuous hourly water level and precipitation data for the period from 1994 through 2015. As shown in **Table B2-3**, the mean correlation coefficients of the events analyzed are somewhat greater than the correlations observed using a large continuous period though the correlations are still weak. It is interesting to note that some individual events produced significant correlations between tide residual and precipitation.

Table B2-3 shows the event with the highest correlation coefficient of those analyzed.

Table B2-3: Pearson Correlation Coefficients for Tropical Events at Newark Airport and Teterboro Airport

	Newark Airport		Teterboro Airport	
	Tide Residual - Precipitation	*Water Level - Precipitation*	*Tide Residual - Precipitation*	*Water Level - Precipitation*
Mean Correlation Coefficient (Hourly)	0.12	0.06	0.31	0.08
Maximum Correlation Coefficient (Hourly)	0.52	0.21	0.85	0.47
Maximum Correlation Event	Hurricane Irene (2011)	Hurricane Irene (2011)	Hurricane Irene (2011)	Hurricane Floyd (1999)

The Pearson correlation coefficient was also calculated for 13 separate extratropical storm events with continuous hourly water level and precipitation data for the period from 1994 through 2015. As shown in

Table B2-4, the mean correlation coefficients of the events analyzed are greater than the correlations observed using both a large continuous period and a sample of tropical storm events though the correlations are still quite weak. It is interesting to note that, as with the tropical events, some individual events produced significant correlations between tide residual and precipitation.

Table B2-4 shows the event with the highest correlation coefficient of those analyzed.

Table B2-4: Pearson Correlation Coefficients for Extratropical Events at Newark Airport and Teterboro Airport

	Newark Airport		Teterboro Airport	
	Tide Residual - Precipitation	*Water Level - Precipitation*	*Tide Residual - Precipitation*	*Water Level - Precipitation*
Mean Correlation Coefficient (Hourly)	0.28	0.12	0.41	0.18
Maximum Correlation Coefficient (Hourly)	0.49	0.36	0.61	0.47
Maximum Correlation Event	Unnamed (11/2014)	Unnamed (2/1998)	Unnamed (2/2014)	Unnamed (11/2005)

4.1.1.2 Dependence

Tide levels depend on astronomical bodies, primarily the Moon and Sun, and are independent of atmospheric processes by definition. Precipitation and tide residual / storm surge are dependent on atmospheric processes as they are directly caused by them. Thus, it can be informative to consider the nature of a storm when trying to quantify potential precipitation and surge magnitudes. For example, longer duration events can create surge that lasts multiple tide cycles and is more likely to coincide with high tides as well as high precipitation. As shown in **Table B2-2** to **Table B2-4**, the greatest positive correlation was found between tide residual and precipitation. This relationship is expected as both storm surge / tide residual and precipitation are dependent on the atmospheric processes that are associated with storms while water levels are primarily determined by tide levels.

Of the events analyzed, extratropical storm events had on average consistently stronger correlation between hourly precipitation and both tide residual and water level. However, the highest correlation for any single event analyzed was for Hurricane Irene, a tropical event. Thus both tropical and extratropical storms can exhibit increased correlation without a direct dependence / causal relationship between rainfall and surge. In other words, precipitation, surge, and water level are all independent processes, but during some storm events precipitation and surge are more likely to occur together. Further study would be necessary to fully characterize the relationships and causes of high correlation events such as Hurricane Irene in contrast with low correlation events like Hurricane Sandy.

4.1.1.3 Coincidence

Three types of coincidence will be considered in this analysis—the coincidence of high precipitation events with high storm surge events, the coincidence of peak precipitation intensity and peak water level within storm events, and the coincidence of peak water level and peak storm surge within storm events.

Forty five events between 1997 and 2016 were analyzed in order to evaluate the timing associated with high precipitation and high storm surge events. The majority of events with relatively high water levels and/or high surge did not occur simultaneously with high precipitation events. Most notably, Hurricane Sandy - the largest water level and surge event in the analyzed record - occurred simultaneously with less than 2-year Annual Recurrence Interval (ARI) precipitation. However, it is important to note that several major storms (e.g., Hurricane Floyd, Tropical Storm Tammy and Subtropical Depression 22, and Hurricane Irene) did create high water levels, surge, and precipitation in various combinations of intensity.

In order to further describe the coincidence of high precipitation, surge, and water levels, all available daily data from 1997 to 2015 was plotted. **Figure B2-4** shows the total daily precipitation versus the corresponding maximum daily tide residual, while **Figure B2-5** shows the total daily precipitation versus the corresponding maximum daily water level. ARI values are included as dashed redlines in both plots for reference.

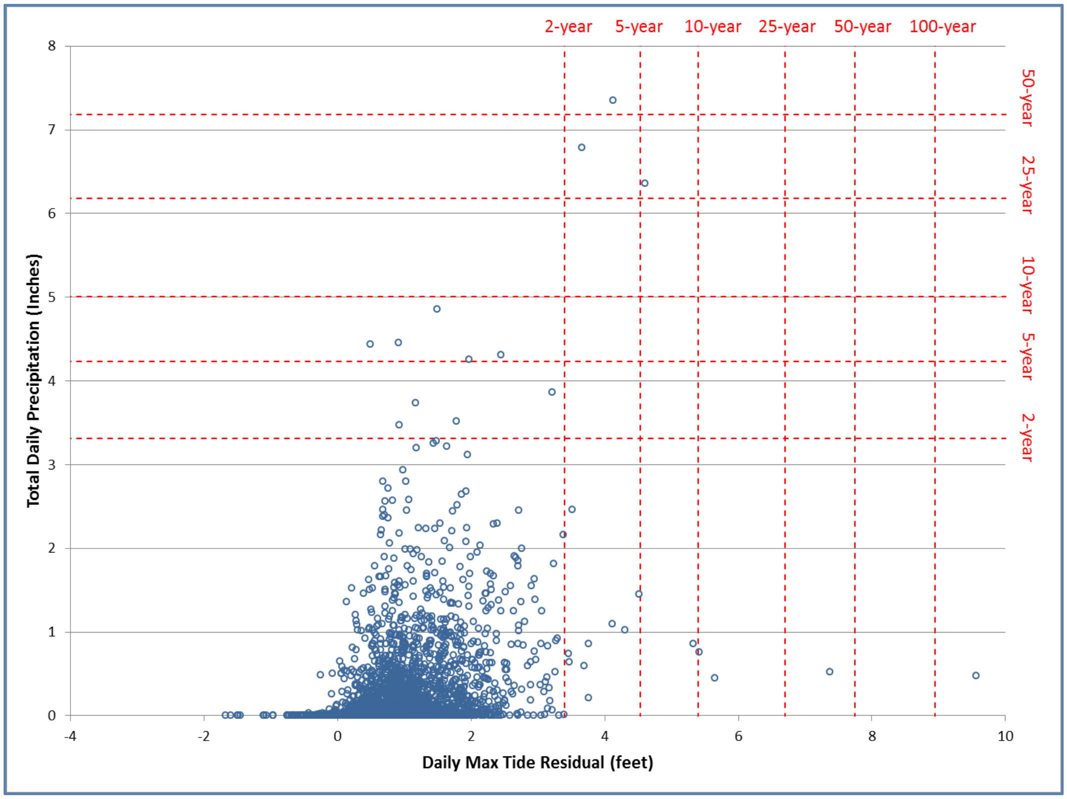

Figure B2-4: Total Daily Precipitation vs. Maximum Daily Tide Residual

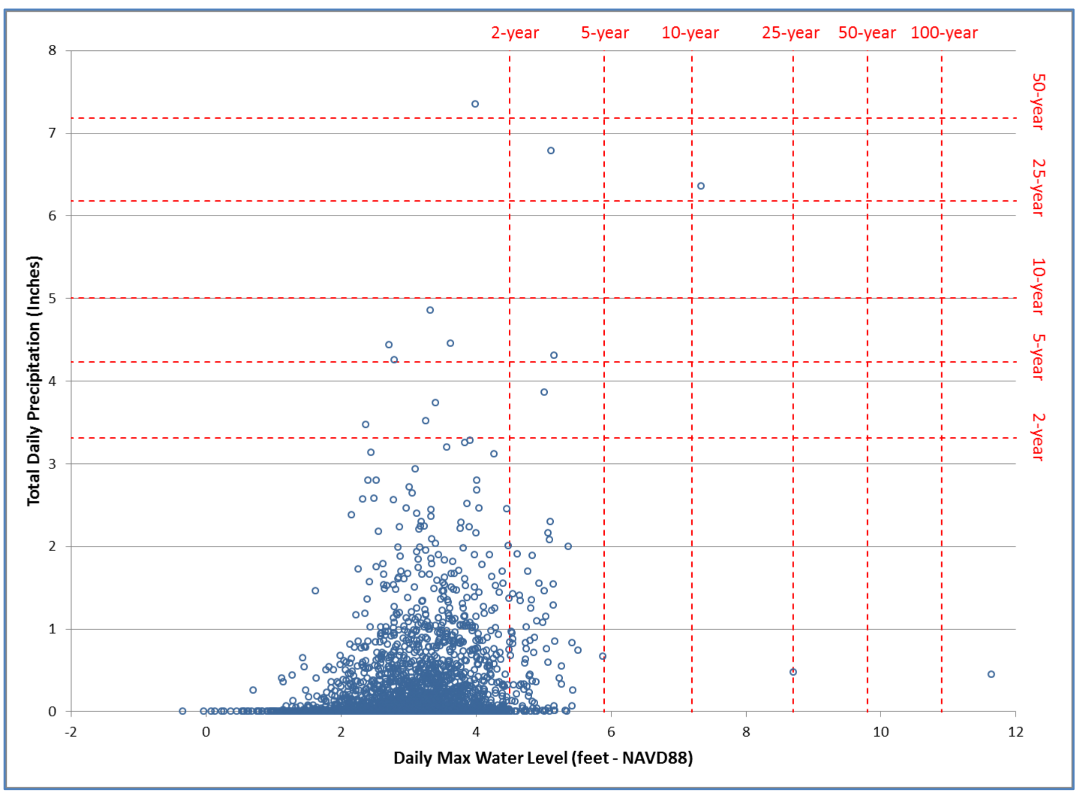

Figure B2-5: Total Daily Precipitation vs. Maximum Daily Water Level

The behavior of the tides generally does not influence the behavior of surge as the physics governing the tides is separate from that which governs storm events. However, there are some key interactions between tides and surge that have been established. (Horsburgh 2007) showed that surge production when caused by wind shear can be modulated by water depth, thus making it is possible for low tide to indirectly result in amplified storm surge. Secondly, surge can cause a phase shift in the tidal wave, resulting in advancement of high water and explaining the tendency for peak surge to avoid maximum high water and low water ((Rossiter 1961); (Horsburgh 2007)). Outside of these interactions, the magnitude of storm surge is independent from the magnitude of the tidal water level, and thus the two can occur in any combination. As with surge, precipitation functions independently of the tides and may occur at any time relative to high tide. While there appears to be a slight tendency for peak precipitation to precede peak water level, the sample size analyzed is not large enough to discern a clear trend.

4.1.1.4 Modeling in the Study

There is weak correlation between rainfall/ runoff events (interior condition) and tidal flooding events (exterior condition), but, because there is a minor dependence between the two, it is considered most likely that only limited surface runoff would coincide with severe storm surge and significant storm surge would coincide with only moderate rainfall. Historic data indicates that the majority of interior runoff events would coincide with a storm surge level less than or equal to a 2-year storm. Similarly, the majority of significant storm surge events are likely to coincide with runoff equivalent to a 2-year event or less.

The interior stage analysis was conducted for events with eight recurrence intervals: the 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 250-year, and 500-year frequency events. In order to develop a stage-frequency relationship, the interior events were routed against exterior tidal marigrams. For the most likely flooding scenarios, the eight interior storm events were routed against a 2-year exterior tide, and a 2-year interior storm event was routed against the six exterior events, the astronomical, 2-, 5-, 10-, 25-, and 50-year tide. **Table B2-5** presents the different interior and exterior runs analyzed and the risk condition assigned to each.

Table B2-5: Recommended Analysis Approach - Combination of Interior and Exterior Conditions

Varied Interior Condition		Varied Exterior Condition		Risk Condition
Interior Flow	Exterior Stage	Interior Flow	Exterior Stage	
2-year	Typical	N/A	N/A	Lower Bound
5-year	Typical	N/A	N/A	Lower Bound
10-year	Typical	N/A	N/A	Lower Bound
25-year	Typical	N/A	N/A	Lower Bound
50-year	Typical	N/A	N/A	Lower Bound
100-year	Typical	N/A	N/A	Lower Bound
250-year	Typical	N/A	N/A	Lower Bound
500-year	Typical	N/A	N/A	Lower Bound

Varied Interior Condition		Varied Exterior Condition		Risk Condition
Interior Flow	Exterior Stage	Interior Flow	Exterior Stage	
2-year	2-year	2-year	2-year	Most Likely
5-year	2-year	2-year	5-year	Most Likely
10-year	2-year	2-year	10-year	Most Likely
25-year	2-year	2-year	25-year	Most Likely
50-year	2-year	2-year	50-year	Most Likely
100-year	2-year	2-year	N/A	Most Likely
250-year	2-year	2-year	N/A	Most Likely
500-year	2-year	2-year	N/A	Most Likely
2-year	10-year	10-year	2-year	Upper Bound
5-year	10-year	10-year	5-year	Upper Bound
10-year	10-year	10-year	10-year	Upper Bound
25-year	10-year	10-year	25-year	Upper Bound
50-year	10-year	10-year	50-year	Upper Bound
100-year	10-year	10-year	N/A	Upper Bound
250-year	10-year	10-year	N/A	Upper Bound
500-year	10-year	10-year	N/A	Upper Bound

As demonstrated in the Risk Condition column of **Table B2-5**, uncertainty was incorporated into the analysis by establishing lower and upper coincidental frequency bounds. For the lower bound, the interior storm events were routed against a typical exterior tidal condition and for the upper bound the interior events were routed against a 10-year external tide. The maximum water surface elevation (WSEL) of corresponding coincidental frequencies (e.g., 2-year interior and 10-year exterior, or 10-year interior and 2-year exterior) was identified as the most damaging flood level for the coincidental frequency. In the "With" the Proposed Project analysis, the 25- and 50-year exterior events were found to be more damaging than its corresponding reversed condition frequency because it would overtop the proposed seawall/armored levee. The analysis was performed for both the current and future conditions to include the impacts of sea level rise (SLR) and increased intensity of fluvial storms.

5.0 Climate Change and Future Conditions

In accordance with the US Army Corps of Engineers (USACE) Engineering and Construction Bullitian 2016-25, "Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs and Projects," a consideration of past trends and projected changes to relevant hydrologic variables is presented in this section (USACE 2016). Although projected changes may have major impacts to the region, it is unlikely that increases to precipitation and water levels will significantly impact the interior drainage of the proposed Line of Protection (LOP). Accordingly, no formal quantitative assessment of climate change effects was performed for this study.

5.1 Historical Trends

Global sea level change (SLC) averaged between +0.5 and +0.7 inches per decade for the majority of the 20th century with increases to approximately +1.3 inches per decade starting in 1993 (Horton, et al. 2015). Along the New Jersey coastal plain, SLC rates have been much higher due to land subsidence. For example, water levels at Atlantic City have risen at a rate of +1.5 inches per decade for the period between 1912 and approximately 2012 (Broccoli 2013).

On average, annual precipitation has increased for New Jersey by approximately 4.1 inches (9 percent) since 1895 though due to naturally broad decadal variation in annual precipitation, it is unclear exactly how precipitation is changing over time (Broccoli 2013). However, in the National Climate Assessment, the Northeast is reported to be experiencing the greatest recent increase in extreme precipitation observed for any region in the United States with increases of over 70 percent in precipitation due to very heavy events (top 1 percent of daily events) (Horton, et al. 2014).

5.2 Projected Sea Level Rise Scenarios

The science of projecting future SLR is constantly evolving as researchers gather the most up-to-date data on key climate and physical variables that govern this complex rate of change. In 1987, the National Research Council (NRC) completed a study on SLC and published Responding to Changes in Sea Level: Engineering Implications (NRC 1987). The report reviewed relative sea level change (RSLC) and presented a range of possible global mean SLC scenarios. Global mean SLC rates were presented up to 2100 (from a starting year of 1986) for three different scenarios (typically referred to as NRC Curve I, NRC Curve II, and NRC Curve III).

When the NRC report was published, the estimate of global mean SLC was 1.2 millimeters per year (mm/year). By the time the Intergovernmental Panel on Climate Change (IPCC) released the 4th Assessment Report (AR4) on climate change in 2007, the rate increased to 1.7 mm/year (IPCC 2007). In the USACE Engineering Circular (EC) 1165-2-212 published in 2011 (USACE 2011), the 1987 NRC curves were modified to: account for this increased global mean SLC reported by the IPCC, incorporate the local rate of vertical land movement, and start in 1992 (the midpoint of the current National Tidal Datum Epoch from 1983 to 2001). The 1987 NRC curves were renamed the Modified NRC Curve I, Modified NRC Curve II, and the Modified NRC Curve III. The recommendation from the USACE's EC 1165-2-212 was to use the local historical rate of SLC for a USACE Low scenario, the Modified NRC Curve I as the USACE Intermediate scenario, and the Modified NRC Curve II as the USACE High scenario. These curves were further modified in 2013 when the USACE released Engineering Regulation ER 1100-2-8162 (USACE 2013).

In 2012, NOAA published its own global mean SLC estimates (High, Intermediate-High, Intermediate-Low, Low) as a part of the National Climate Assessment. The Intermediate-High Scenario is based on an average of published high end semi-empirical, global SLR projections and includes statistical relationships between observed global SLC, recent ice sheet loss, and air temperature. The Intermediate-Low Scenario is based on the upper end of IPCC Fourth Assessment Report (AR4) global SLR projections resulting from climate models using the B1 emissions scenarios and primarily captures the effects of ocean warming (IPCC 2007). The Intermediate-Low Scenario corresponds with the USACE Intermediate Scenario (Modified NRC Curve II), while the Low Scenario corresponds with the USACE Low Scenario (Modified NRC Curve I). These rates were updated in 2013 to include regional vertical land movement where data was available (NOAA 2013) and again in 2017 to include two additional scenarios (Intermediate and Extreme), as well as updated climate science for the upcoming National Climate Assessment due in 2018 (NOAA 2017).

The New York City Panel on Climate Change (Horton, et al. 2015) published estimates of local RSLC in 2013 that were then updated in 2015. Projections are relative to the 2000–2004 baseline period and based on a six-component approach that incorporates both local and global factors. For each of the components of SLC, the 10th, 25th, 75th, and 90th percentiles of the distribution were calculated and the sum of all components at each percentile is assumed to give the aggregate sea level rise projection (Horton, et al. 2015).

5.3 Selected Future Low and Future High Sea Level Rise Scenarios

All of the above SLC curves at Battery Park, NY are shown in **Figure B2-6** except for the NOAA 2017 scenarios. The two tidal SLR scenarios applied to the hydraulic models designated as Future Low and Future High are noted in **Figure B2-6** as NOAA INT-LOW and NOAA INT-HIGH, respectively.

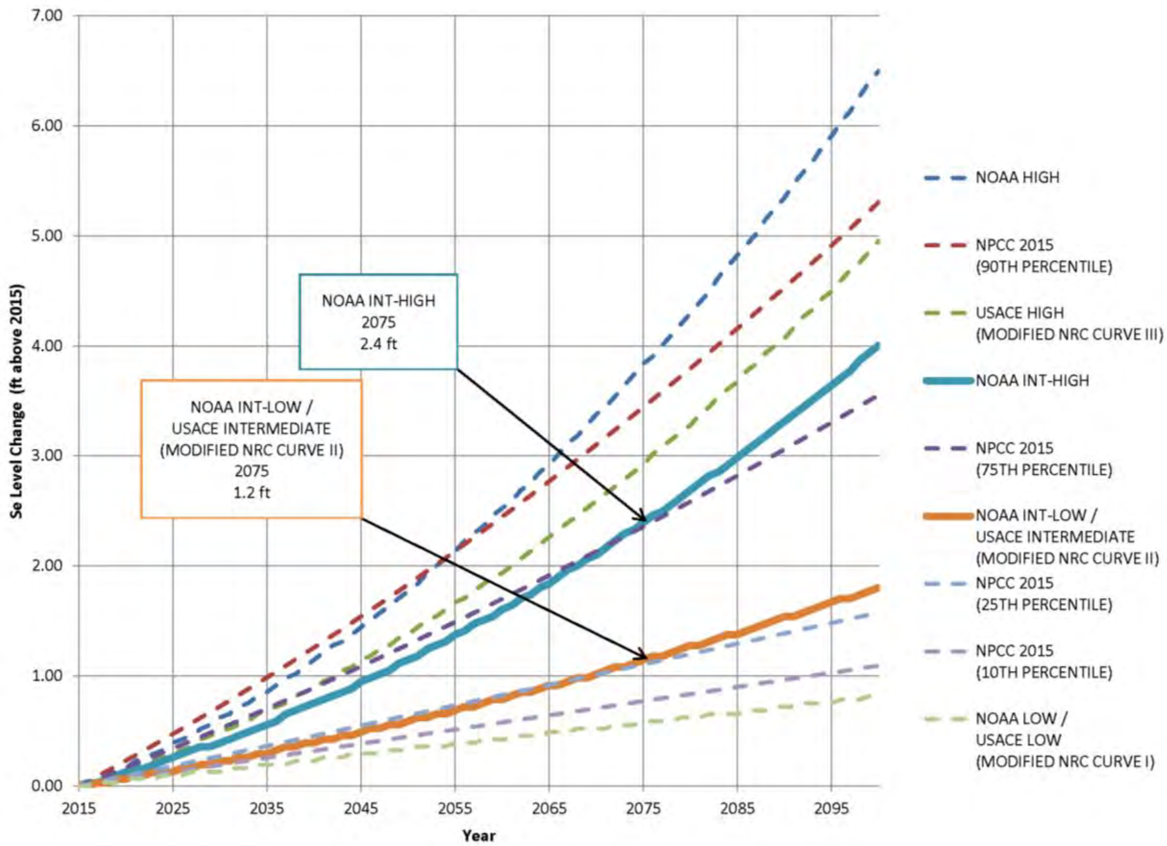

Figure B2-6: Sea Level Change Estimates for the Battery, NY (Gauge: 8518750)

Note: NOAA values are regionally corrected to include regional vertical land movement as documented in the NOAA report on Estimating Vertical Land Motion from Long-Term Tide Gauge Records (NOAA 2013).

5.4 Projected Precipitation Increases

Although regional projections of future precipitation exist, quantitative Project-Area-specific projections of changes to return period events are not readily available. Therefore, AECOM performed a future conditions frequency analysis using statistically downscaled general circulation model output to generate projected changes to the different return period events for the year 2075. Details of the future conditions frequency analysis are detailed in **Subappendix B9**.

Results from the downscaled model output from the University of Idaho were selected for use in future conditions modeling and precipitation increase factors are provided in **Table B2-6**. Percent changes from 2016 to 2075 were applied to the existing NOAA Atlas 14 frequency analysis used for existing conditions modeling. The factor for the 100-year return period was also applied to all greater return periods as insufficient model output is available to calculate these values without significant uncertainty. The University of Idaho values were the most conservative (greatest change) of the downscaled data sources evaluated and have been shown to possibly capture extremes in precipitation more realistically than some of the other downscaling methods considered. These values will be used for planning purposes only as projections of future precipitation continue to be significantly uncertain, but they will provide an important window into the possibility of future precipitation trends for the Project Area.

Table B2-6: Future Conditions Precipitation Changes for RBDM

Return Period	2-year	5-year	10-year	25-year	50-year	100-year plus
University of Idaho (MACA Method)	10%	12%	16%	23%	32%	45%

5.5 Project Future Precipitation Data

The Present Rainfall data shown previously in **Table B2-1**: was scaled up with the projected rainfall increases to create the Future Rainfall data for the future conditions hydrologic and hydraulic models.

The University of Idaho Multivariate Adaptive Constructed Analogs (MACA) method precipitation increase estimate values were selected for use in future conditions modeling (see **Subappendix B9**). Specifically, the MACA year-2075 value means were chosen for precipitation increase factors and are provided in **Table B2-7**. The factor for the 100-year return period was also applied to all greater return periods. The MACA values were the most conservative (greatest change) and have been shown to possibly capture extremes in precipitation more realistically. These values will be used for planning purposes only as they continue to be significantly uncertain, but they will provide an important window into the possibility of future precipitation trends for the Project Area.

Future hypothetical point rainfall depths for the 1- through 500-year storms based on the above projected seasonal changes are shown in **Table B2-7**.

Table B2-7: Future Conditions Hypothetical Point Rainfall Depths in Inches

Duration	Return Period							
	2-year Rainfall [in]	5-year Rainfall [in]	10-year Rainfall [in]	25-year Rainfall [in]	50-year Rainfall [in]	100-year Rainfall [in]	250-year Rainfall [in]	500-year Rainfall [in]
5 min.	0.44	0.54	0.62	0.74	0.87	1.03	1.11	1.19
15 min.	0.87	1.06	1.23	1.46	1.7	2.02	2.17	2.31
1 hour	1.49	1.89	2.26	2.83	3.39	4.13	4.62	5.08
2 hours	1.83	2.35	2.83	3.58	4.34	5.35	6.08	6.77
3 hours	2.04	2.62	3.17	4.01	4.88	6.02	6.87	7.67
6 hours	2.63	3.37	4.06	5.17	6.31	7.8	8.95	10.05
12 hours	3.23	4.17	5.05	6.48	7.99	9.96	11.58	13.15
24 hours	3.64	4.74	5.81	7.6	9.48	12.01	14.21	16.39
48 hours	4.26	5.54	6.79	8.83	10.98	13.88	16.31	18.85

6.0 Drainage Area Delineation

The drainage areas were identified and characterized using the best available GIS data. Drainage areas were delineated by identifying the contributing areas to the various stormwater discharge pipes and open channels exiting to the Hackensack River or Berry's Creek. Surface overflow paths that flow when channels or conduits are overwhelmed were identified and modeled as weirs.

6.1 Delineation Source Data

Interior drainage basins and sub-basins were delineated using contours derived from the 2014 Hackensack Meadowlands Light Detection and Ranging (LiDAR) dataset collected by Quantum Spatial for the New Jersey Meadowlands Commission. A hydro-flattened bare earth digital elevation model (DEM) was derived from the LiDAR dataset. GIS point and polyline datasets depicting stormwater manholes, catch basins and mains, and sanitary sewer manholes, prepared by the New Jersey Sports and Exposition Authority (NJSEA) and dated as of 2016, as supplemented and updated by stormwater and utility surveys conducted by New World Engineering for the RBDM Proposed Project in 2016 in certain designated portions of the Project Area were used to further refine the drainage divides, where storm sewer and surface runoff divides did not coincide and where a distinct surface runoff divide was unapparent. All elevation data is in the NAVD 88 vertical datum.

6.2 Drainage Areas

The locations and naming of major interior drainage areas are split into two main groups: Berry's Creek Group and Hackensack River Group. The Hackensack is further split into two groups, one for the larger inner areas away from the river and one for the areas immediately adjacent to the river. Each is named after a significant feature that is found within the drainage area. The areas along the Hackensack River, however, are named from North to South starting with "HACK 1" and ending with "HACK 9." "HACK 1A3" is the only exception at the most northeast area along the river bank.

6.2.1 Hackensack Inner Drainage Areas

These areas drain to the Hackensack River, but are generally much larger and extend inland further than the many small drainage areas immediately adjacent to the Hackensack River. Losen Slote, the largest of the inner areas, consists of three sub drainage areas and is conveyed via the Losen Slote stream to the Hackensack River. Below are descriptions of each area as shown in **Figure B2-7**. A summary of this information is provided with the remainder of the Hackensack drainage areas in **Table B2-8**.

6.2.1.1 Carol Place Ditch Drainage Area

Carol Place Ditch is located in the southeast portion of the Project Area, encompassing Empire Boulevard off of Moonachie Road; Carol Place Ditch is tributary to Losen Slote.

6.2.1.2 DePeyster Creek Drainage Area

DePeyster Creek is an 84-acre drainage area that stretches north and south from Maple Street to Maiden Lane along the east side of the Project Area.

6.2.1.3 Indian Lake Drainage Area

Indian Lake is a recreational lake with a 64-acre drainage area located in the northeast section of the Project Area where River Street meets Bergen Turnpike.

6.2.1.4 Losen Slote Drainage Area

Losen Slote is a creek that was analyzed with three sub-basins: Main Street, Central, and South. Losen Slote – Main Street is a 144-acre drainage area. Losen Slote – Central is an 80-acre drainage area, and Losen Slote – South is a 492-acre drainage area. Losen Slote – Main Street is the northern most sub-basin of the three. Stormwater flows from Losen Slote - Main Street into Losen Slote – Central, which is directly south of it. Then stormwater flows into Losen Slote – South, which is a large drainage area in the vicinity of where the creek angles towards the Hackensack River.

6.2.1.5 Main Street Ditch Drainage Area

Main Street Ditch is a 141-acre drainage area located along the east side of the Project Area surrounding Route 46.

6.2.1.6 Moonachie Creek Drainage Area

Moonachie Creek is a 294-acre drainage area located at the southeast most corner of the Project Area.

6.2.1.7 Willow Lake Drainage Area

Willow Lake has a 37-acre drainage area is located in the east-central portion of the Project Area along Washington Avenue.

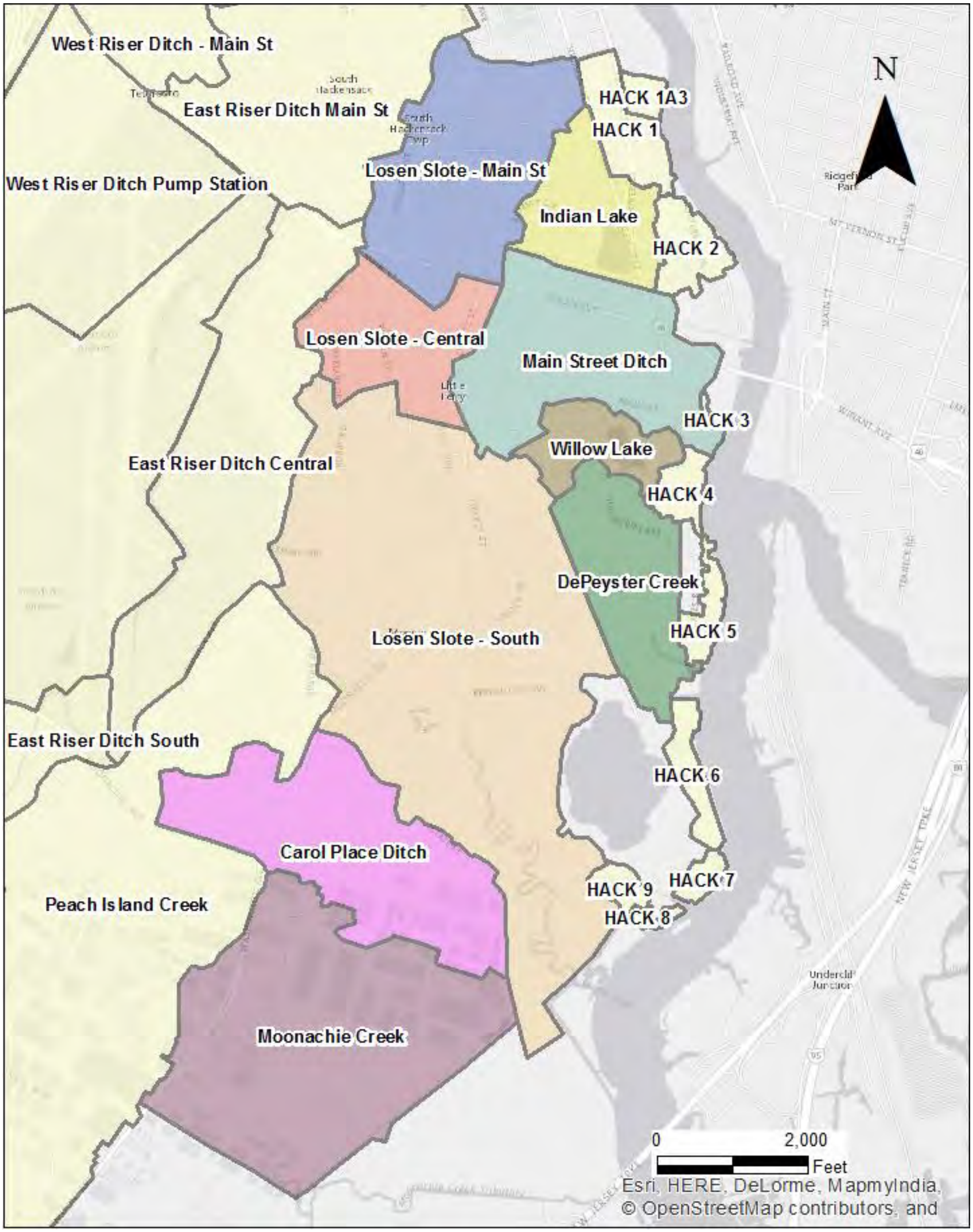

Figure B2-7: Hackensack Inner Drainage Areas

6.2.2 Hackensack Drainage Areas

The Hackensack Drainage Areas are many small drainage areas that lie directly adjacent and drain to the Hackensack River. These areas will pond against a line of protection if one is installed within any or all of these areas. Below is a description of each area. **Figure B2-8** shows these areas on a map and

Table B2-8 provides a summary of each area.

6.2.2.1 HACK 1A3 Drainage Area

HACK 1A3 is a 6-acre drainage area located in the northern most section along the Hackensack River.

6.2.2.2 HACK 1 Drainage Area

HACK 1 is a 30-acre drainage area located along the Hackensack River right above where River Street connects with the Bergen Turnpike.

6.2.2.3 HACK 2 Drainage Area

HACK 2 is a 21-acre drainage area located along the Hackensack River encompassing River Street Extension.

6.2.2.4 HACK 3 Drainage Area

HACK 3 is a 1-acre drainage area located along the Hackensack River between Route 46 and the Bergen Turnpike.

6.2.2.5 HACK 4 Drainage Area

HACK 4 is a 13-acre drainage area located along the Hackensack River, south of Bergen Turnpike.

6.2.2.6 HACK 5 Drainage Area

HACK 5 is a 12-acre drainage area located along the Hackensack River near Gates Road.

6.2.2.7 HACK 6 Drainage Area

HACK 6 is a 15-acre drainage area located along the Hackensack River, south of McCabe Court in Bergen County Utilities Authority.

6.2.2.8 HACK 7 Drainage Area

HACK 7 is a 7-acre drainage area that is located Along the Hackensack River, south of HACK 6, in the Bergen County Utilities Authority (BCUA).

6.2.2.9 HACK 8 Drainage Area

HACK 8 is a 3-acre drainage area located south of HACK 7 in the BCUA.

6.2.2.10 HACK 9 Drainage Area

HACK 9 is a 7-acre drainage area located northwest of HACK 8 in the BCUA.

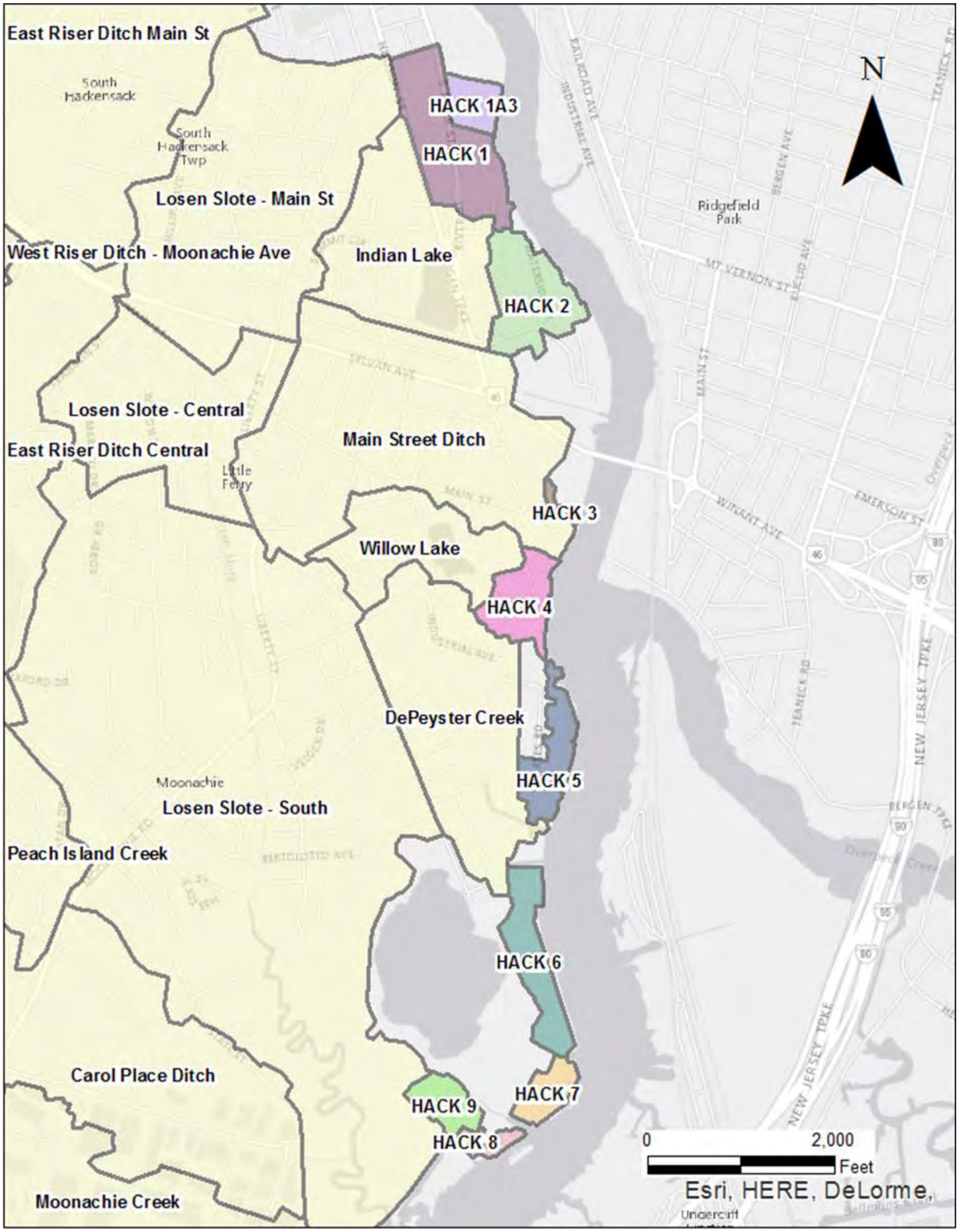

Figure B2-8: Hackensack Drainage Areas

Table B2-8: Hackensack Drainage Areas and Existing Outlet Features

Interior Areas Draining to Hackensack River - all elevations (el.) in feet (NAVD 88)				
Interior Area	**Drainage Area (Acres)**	**Gravity Outlets**	**Surface Overflow (weir)**	**Pump Station**
DePeyster Creek	84	3.5-foot flap gate	50-foot wide at el. 6.0	3 x 2 cfs pumps
Indian Lake	64	None	100-foot wide at el. 6.0	2 x 55 cfs pumps 2 x 6 cfs pumps
Losen Slote - Carol Place Ditch	167	via ditch to Losen Slote South	-	-
Losen Slote - Main Street	144	flows to Losen Slote Central	-	-
Losen Slote – Central	80	flows to Losen Slote South	-	-
Losen Slote – South	492	3 x 5-foot x 5-foot flap gates to Hackensack River	2,500-foot at el. 6.5	3 x 96 cfs pumps
Main Street Ditch	141	Ditch	50-foot at el. 5.5	2 x 18 cfs pumps
Moonachie Creek	294	2 x 8-foot diameter flap gates	-	-
Willow Lake	37	none	30-foot at el. 5.0	2 x 12 cfs pumps
HACK 1A3	6	2 x 2-foot diameter pipes	35-foot at el. 5.0	-
HACK 1	30	3 x 2-foot diameter pipes	20-foot at el. 5.5	-
HACK 2	21	4 x 3-foot diameter pipes	60-foot at el. 6.5	-
HACK 3	1	Surface flow	65-foot at el. 5.0	-
HACK 4	13	2 x 2-foot diameter pipes	47-foot at el. 4.0	-
HACK 5	12	3 x 2-foot diameter pipes	197-foot at el. 4.0	-
HACK 6	15	Suface flow	190-foot at el. 5.0	-
HACK 7	7	1 x 3-foot diameter external pipe	250-foot at el. 5.0	-
HACK 8	3	1 x 3-foot diameter external pipe	250-foot at el. 5.0	-
HACK 9	7	Surface flow	105-foot at el. 3.0	-

6.2.3 Berry's Creek Drainage Areas

Figure B2-9 shows the drainage areas of Berry's Creek as described below.

6.2.3.1 Dell Road Drainage Area

Dell Road is a 10-acre drainage area located in the southern portion of the Project Area at the intersection of Dell Road and Starke Road.

6.2.3.2 East Riser Ditch Central Drainage Area

East Riser Ditch Central is a 211-acre drainage area located in the center of the Project Area, along Redneck Avenue and Franklin Street. A portion of the outflows into East Riser Ditch Central are diverted to West Riser Ditch – Moonachie. The rest outflows to East Riser Ditch South via one 4.5 foot by 12 foot box culvert and two spillways.

6.2.3.3 East Riser Ditch South Drainage Area

East Riser Ditch South is a 189-acre drainage area located in the southern portion of the Project Area below Moonachie Avenue. Stormwater flows out of East Riser Ditch South to Paterson Plank Bridge.

6.2.3.4 Peach Island Creek Drainage Area

Peach Island Creek is a 571-acre drainage area located in the southeast portion of the Project Area. Stormwater flows out to Paterson Plank Bridge.

6.2.3.5 West Riser Ditch South Drainage Area

West Riser Ditch South is a 183-acre drainage area located in the southwest portion of the Project Area, and encompasses a portion of Moonachie Avenue. The ditch flows downstream through Paterson Plank Bridge.

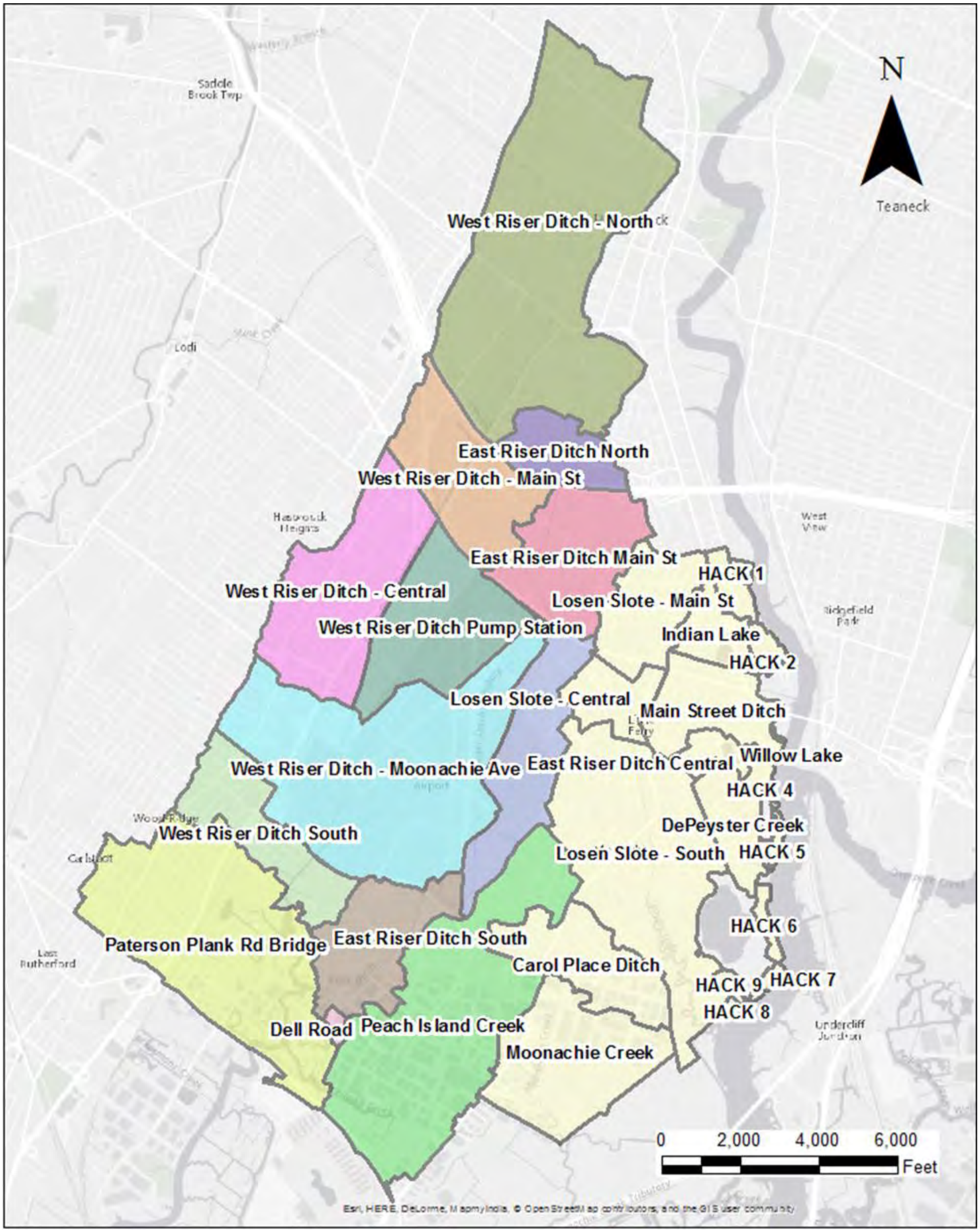

Figure B2-9: Berry's Creek Drainage Areas

6.2.3.6 Paterson Plank Road Bridge Drainage Area

Paterson Plank Road Bridge is a 571-acre drainage area located at the southwestern most corner of the Project Area. Stormwater flows under Paterson Plank Road Bridge on Berry's Creek with three overflows on and adjacent to the bridge.

6.2.3.7 West Riser Ditch Pump Station Drainage Area

West Riser Ditch Pump Station is a 255-acre drainage area located northwest of the Teterboro Airport. A pump station was built for West Riser Ditch with one 100-cubic feet per second (cfs) pump and three 25-cfs pumps that pump water out to West Riser Ditch Moonachie.

6.2.3.8 East Riser Ditch at Route 80 Drainage Area

East Riser Ditch at Route 80 is a 102-acre drainage area located just north of Route 80. Stormwater flows out of this area to East Riser Ditch Central via a 4-foot by 6-foot box culvert.

6.2.3.9 West Riser Ditch at Moonachie Avenue Drainage Area

West Riser Ditch at Moonachie Avenue is a 673-acre drainage area located at the northwest quadrant of the Project Area and stretching down to Moonachie Avenue. Stormwater flows to West Riser Ditch South via two arch culverts and three overflow locations modeled as spillways.

Table B2-9 provides a summary of Berry's Creek drainage areas and existing outlet features.

Table B2-9: Berry's Creek Drainage Areas and Existing Outlet Features

Interior Areas Draining to Berry's Creek - all elevations (el.) in feet NAVD 88				
Interior Area	**Drainage Area (Acres)**	**Gravity Outlets**	**Surface Overflow (weir)**	**Pump Station**
Dell Road	10	2 x 4-foot x 4-foot flap gates to Berry's Creek	30-foot wide at el. 4.0 120-foot wide at el. 5.0 90-foot wide at el. 6.0	-
East Riser Ditch – Central (East Riser Ditch C)	211	Some diversion to West Riser Ditch Moonachie, remainder to East Riser Ditch South via 1 x 4.5-foot x 12-foot culvert	100-foot wide at el. 5.5 107-foot wide at el. 6.0	
East Riser Ditch – South (East Riser Ditch S)	189	4 x 5.5-foot x 7.5-foot flap gates to Berry's Creek	120-foot wide at el. 6.0	-
Peach Island Creek	571	3 x 3-foot diameter flap gates to Berry's Creek	35-foot wide at el. 4.0 159-foot wide at el. 5.0 391-foot wide at el. 6.0 123-foot wide at el. 7.0	-
West Riser Ditch South (West Riser Ditch S)	183	4 x 8-foot x 5.5-foot flap gates to Berry's Creek	200-foot wide at el. 5.0 & 40-foot wide at el. 6.0 **To Berry's Creek** and 100-foot wide at el. 4.0 160-foot wide at el. 4.5 & 240-foot wide at el. 5.5 **to East Riser Ditch S**	2 x 200 cfs pumps
West Riser Ditch Pump Station	255	-	150-foot wide at el. 5.2	1 x 100 cfs and 3 x25 cfs pumps
East Riser Ditch at Route 80	102	1 x 4-foot x 6-foot box culvert	-	-
West Riser Ditch at Moonachie Avenue	673	2 x 7.1-foot x 12.67-foot box culverts	322-foot wide at el. 4.0 149.5-foot wide at el. 4.5 394.5-foot wide at el. 5.0	-
Paterson Plank Road Bridge	571	Bridge Openings – 14-foot x 53-foot 17-foot x 62-foot 11.46-foot x 53-foot	140-foot wide at el. 6.0 100-foot wide at el. 6.5 & 10-foot wide at el. 8.0	-

7.0 Drainage Area Characterization and HMS Model

The drainage areas identified were each characterized using an overlay of soil hydrologic group and land cover data. Topographic data was used to calculate the flow path slopes for input data to determine flow travel time and hydrologic time of concentration for each drainage area. This information was used to develop a computer based hydrologic model.

7.1 HEC-HMS Model Hydrologic Parameters

The HEC-HMS computer model, developed for the interior drainage areas of the Meadowlands District, is described in **Subappendix B3**. Basic input parameters developed for the hydrologic models include: surface area, hypothetical precipitation storm data (2- to 500-year return periods), runoff curve number, and time of concentration (Tc). This information is derived from extracting characterization information from GIS. Natural Resources Conservation Service (NRCS) runoff curve numbers, NRCS unit hydrograph lag times, routing reach travel times, and hydrograph combinations and diversions were used to define the interior basin response to the specific frequency hypothetical rainfall. Each input parameter is described in more detail in the subsequent sections.

7.2 NRCS Runoff Curve Numbers

The NRCS runoff curve number (CN) procedure (as outlined in NRCS Technical Release No. 55 (TR-55), Urban Hydrology for Small Watersheds) was used to define the rainfall-loss-excess (or runoff) behavior of the interior drainage sub-basins in the HEC-HMS model (NRCS 1986). The runoff CN relate total accumulated excess to total accumulated precipitation and are based on factors such as hydrologic soil group, land use, ground cover, quality of vegetative cover, and antecedent moisture conditions.

Soil Survey Geographic Database (SSURGO) soils maps (2016) were downloaded from the New Jersey Geographic Information Network (NJGIN) website (NJGIN n.d.). Hydrologic Soil Codes were updated using the NRCS Web Soil Survey (NRCS 2017).

The land use shapefile (2012) was downloaded from the NJDEP Bureau of GIS website (NJDEP Bureau of GIS 2018).

7.3 Time of Concentration

The longest hydraulic path for each sub-basin was identified. The travel times of surface runoff along the longest hydraulic paths were then computed incrementally between the 1 foot contour lines. This was done by first computing the slope within the first one-hundred feet. Then, the slope was calculated for the remainder of the hydraulic path. An average slope was used for these segments by taking the ending elevation and subtracting it from the beginning elevation and dividing by the distance between them. The slopes and lengths are then inputted into TR-55. A surface Manning's "n" value must be selected based on the type of ground cover. The drainage area and weight curve number must also be entered. Then TR-55 will calculate the time of concentration for that area.

7.4 NRCS Dimensionless Unit Hydrograph

The NRCS dimensionless unit hydrograph is based on a dimensionless table of discharge per unit area versus time, normalized to the peak discharge and time of concentration respectively. The actual sub-basin unit hydrograph is created within HEC-HMS when supplemented with a specific drainage area and a lag time. The lag time is the time from the center of mass of excess rainfall to the time of the peak discharge of the unit hydrograph.

The lag time was calculated to be 60 percent of the time of concentration (Tc) as recommended by the NRCS. Sub-basin lag times were estimated from TR-55 utilizing the longest hydraulic flow path, its slope, and the local CN. A minimum lag time of 10 minutes was used.

7.5 HEC-HMS Model

Figure B2-10 through **Figure B2-12** provide the HEC-HMS schematic diagrams of the hydrologic models for the Hackensack Inner Drainage Areas, the Hackensack Drainage Areas, and the Berry's Creek Drainage Areas, respectively. Generally, at the beginning of the hydrologic element names in the HEC-HMS models, the following uppercase letters have the listed meanings:

- DA = Drainage Area (drainage basin)
- R = Reach routing
- P = Ponding area
- D = Diversion/flow split
- J = flow Junction

Table B2-10 and **Table B2-11** provide summaries of the HEC-HMS input parameters for the Hackensack Drainage Areas and the Berry's Creek Drainage Areas basins, respectively.

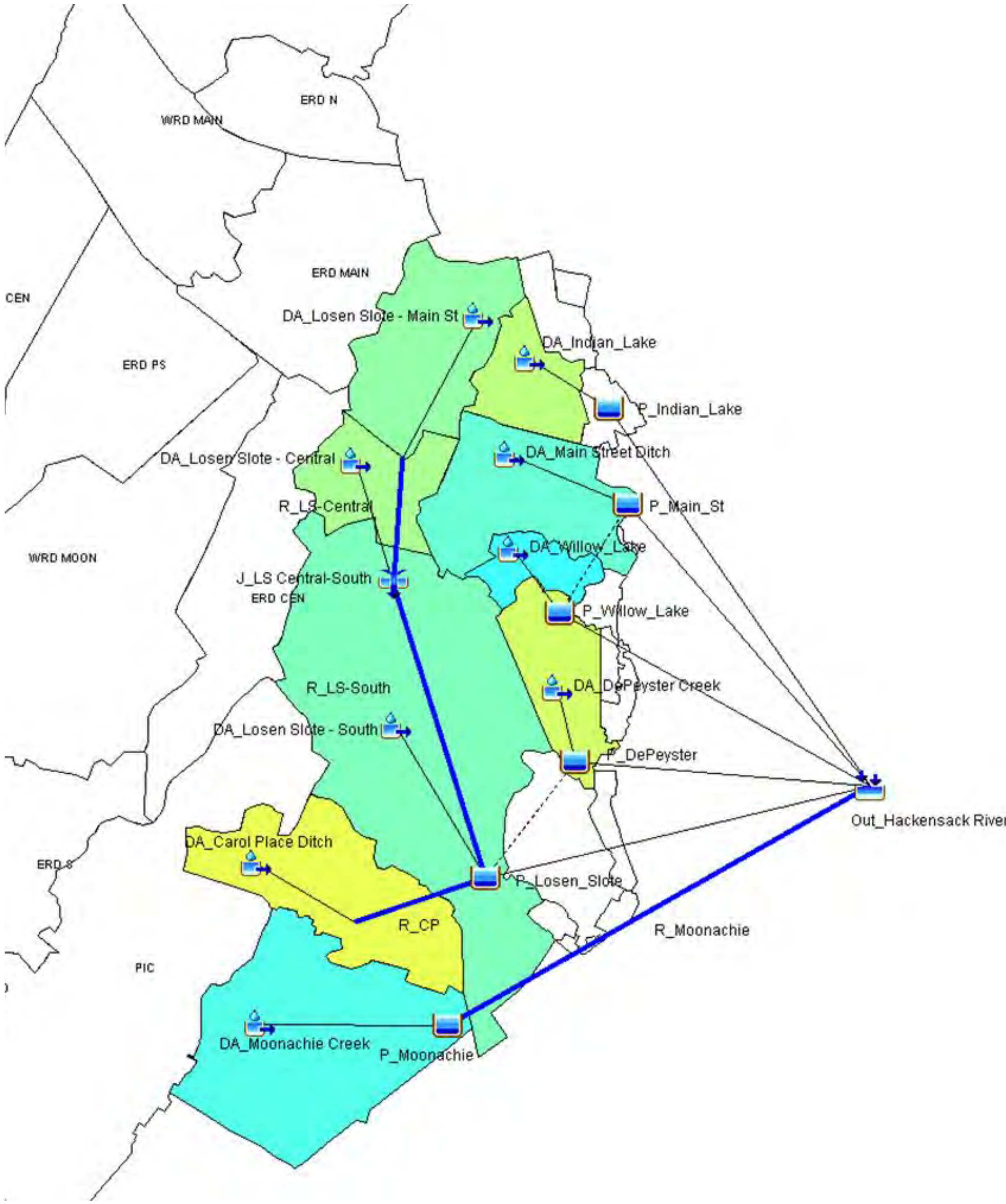

Figure B2-10: Hackensack Inner Drainage Areas – HMS Diagram

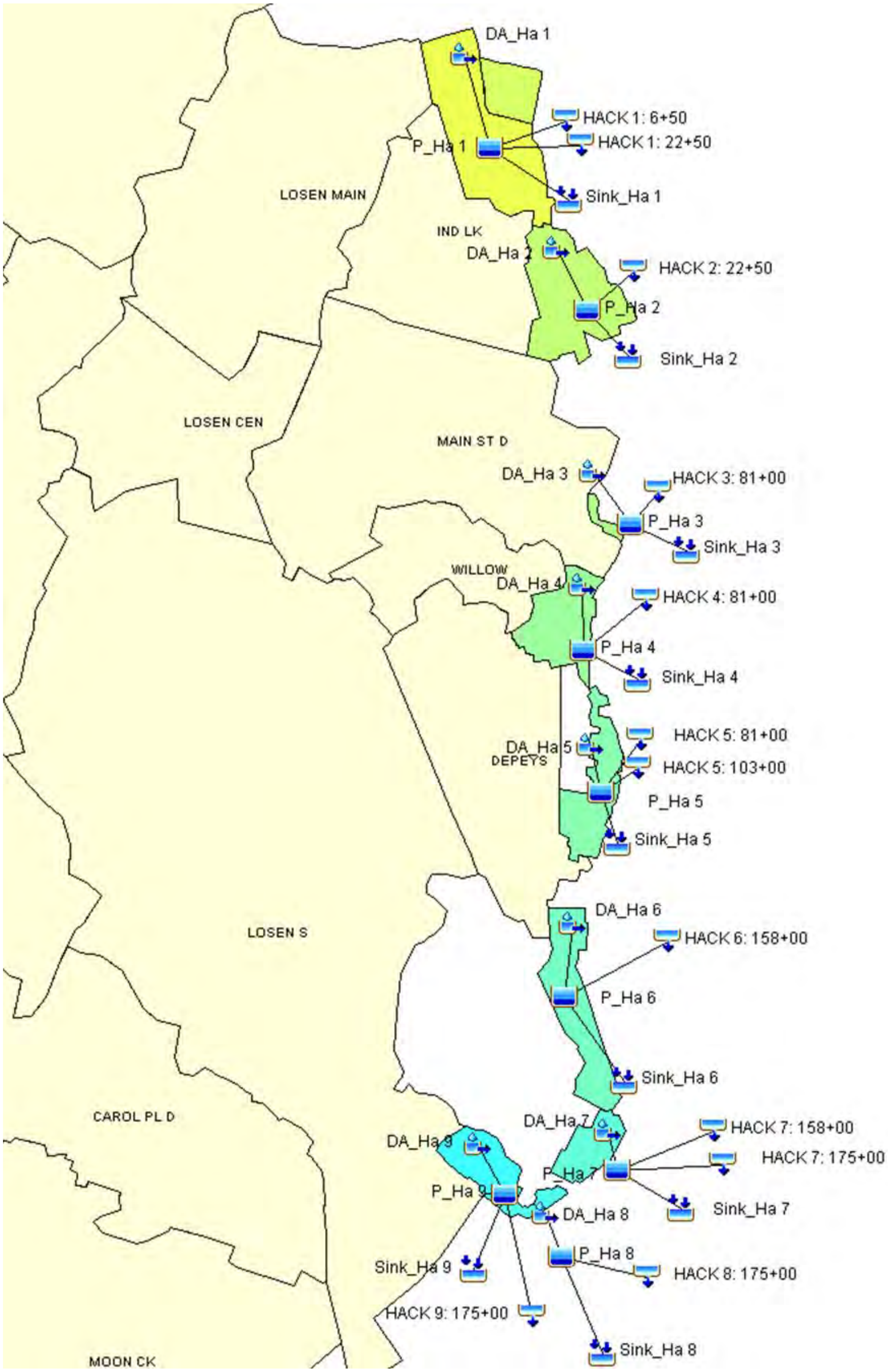

Figure B2-11: Hackensack Drainage Areas - HMS Diagram

Figure B2-12: Berry's Creek Drainage Areas - HMS Diagram

Table B2-10: HEC-HMS Model Sub-Basin Data – Hackensack River

Drainage Area	Sub-Basin (HEC-HMS Hydrologic Element)	Drainage Area Square Miles	Runoff Curve No. CN	Longest Length, Feet	Average Slope of Longest Length	NRCS Unit Hydrograph Lag, Minutes
DePeyster Creek	-	0.1323	88	3465	0.0058	40.9
Indian Lake	-	0.0993	79	1510	0.0193	11
Losen Slote	Carol Place Ditch	0.2599	94	5700	0.0024	3.5
Losen Slote	Losen Slote – Main Street	0.2249	87	3925	0.0087	26.4
Losen Slote	Losen Slote – Central	0.1243	89	3140	0.0029	45.5
Losen Slote	Losen Slote – South	0.7684	90	12490	0.0059	47.6
Main Street Ditch	-	0.2199	89	4350	0.0161	45.1
Moonachie Creek	-	0.4206	94	5200	0.0073	24.2
Willow Lake	-	0.0586	87	1410	0.008	17.8
HACK 1A3	-	0.0101	96	610	0.006	10.0
HACK 1	-	0.0415	90	750	.0031	13.4
HACK 2	-	0.0304	95	362	.0038	10.0
HACK 3	-	0.0019	97	406	.0082	11.7
HACK 4	-	0.0140	88	305	.0049	10.0
HACK 5	-	0.0040	94	624	.0095	10.3
HACK 6	-	0.0166	87	1097	.0005	37.7
HACK 7	-	0.0085	90	490	.0051	10.3
HACK 8	-	0.0024	92	309	.0096	10.0
HACK 9	-	0.0115	95	429	.0046	10.0

Table B2-11: HEC-HMS Model Sub-Basin Data – Berry's Creek

Drainage Area	Sub-Basin (HEC-HMS Hydrologic Element)	Drainage Area Square Miles	Runoff Curve No. CN	Longest Length, Feet	Average Slope of Longest Length	NRCS Unit Hydrograph Lag, Minutes
Dell Road		0.015787	94.8	930	0.005	6.3
East Riser Ditch – Central	East Riser Ditch – Central	0.329428	90.2	8675	0.0031	51.6
East Riser Ditch – Central	East Riser Ditch – Main Street	0.346037	93.6	4420	0.0024	46.2

Drainage Area	Sub-Basin (HEC-HMS Hydrologic Element)	Drainage Area Square Miles	Runoff Curve No. CN	Longest Length, Feet	Average Slope of Longest Length	NRCS Unit Hydrograph Lag, Minutes
East Riser Ditch South	-	0.294902	93.7	5865	0.0081	29.4
Peach Island Creek	-	0.892791	93.1	10810	0.0037	50.3
West Riser Ditch South	-	0.286324	89.4	6850	0.0316	25.2
Paterson Plank Road Bridge	-	0.892738	91.5	10270	0.0348	22.4
West Riser Ditch Pump Station	-	0.398407	90.8	6350	0.0060	37
East Riser Ditch at Route 80	East Riser Ditch – North	0.159826	93.4	3400	0.0045	64.3
West Riser Ditch at Moonachie Avenue	West Riser Ditch at Moonachie Avenue	1.052338	89.9	6510	0.1588	50.2
West Riser Ditch at Moonachie Avenue	West Riser Ditch - North	1.294934	87.3	12330	0.0384	87.1
West Riser Ditch at Moonachie Avenue	West Riser Ditch – Main Street	0.368335	93	3470	0.0218	12.1
West Riser Ditch at Moonachie Avenue	West Riser Ditch – Central	0.511415	87.6	7310	0.0296	34.9

7.6 Reach Routing and Travel Time

For some sub-basin reaches, it was appropriate to calculate routing reach travel times with the aforementioned velocity vs. slope chart plots. Modified Puls storage-outflow routing data equivalent to these travel times was then input to HEC-HMS for the routing reach. By definition, the reach storage divided by its corresponding outflow is the travel time through the reach, at that outflow. Storage in acre-feet divided by outflow in cfs, multiplied by 12.1, gives the reach travel time in hours, accounting for unit conversion. Modified Puls routing was used to allow reach storage to have the maximum effect of hydrograph peak inflow attenuation that would result from interior flood runoff spreading out over the sidewalks, lawns and lots of residential streets, and over the floodplains of natural channels. For other reaches, it was appropriate to enter the reach length, slope, channel and overbank Manning "n" values, and scaled-off typical eight-point cross section into HEC-HMS via the normal depth routing option. HEC-HMS computes a table of storage-outflow-elevation, which is used to perform hydrograph routing. Values from the HEC-HMS routing reach data are summarized in **Table B2-12**. For the flattest, most spread-out, and most irregularly defined routing reaches, the models did not include the rising flat pool storage that would be encountered by an incoming hydrograph accumulating behind the tentatively selected plan alignment.

Table B2-12: HEC-HMS Model Kinematic Routing Reach Data

Basin	Routing Reach Hydrologic Element	From	To	Length in feet	Slope	Manning's "n"
Losen Slote	R_LS-Central	DA_Losen Slote –Main Street	J_LS Central-South	1545	0.0004	0.011
Losen Slote	R_LS-South	J_LS Central-South	P_Losen Slote	10835	0.00055	0.022
Carol Place Ditch	R_CP	DA_Carol Place Ditch	P_Losen Slote	550	0.00135	0.03
Moonachie	R_Moonachie	DA_Moonachie Creek	P_Moonachie	3170	0.0004	0.03
West Riser Ditch – Main Street	R_WR-Main	WRD to ERD Rt80	J_WR-Main-West	2360	0.0004	0.03
West Riser Ditch – Main Street	R_WR-West	J_WR-Main-West	J_WR PS	5455	0.00041	0.03
West Riser Ditch – Pump Station	R_WR-Moonachie	J_WR PS	J_WR Moonachie	4635	0.0004	0.03
West Riser Ditch - Moonachie	R_WR South	P_WRD Moonachie	J_WR South	1580	0.0004	0.03
West Riser Ditch - South	R_BC-1	P_WRD South	J_BC-2	2990	0.00216	0.03
East Riser Ditch – RT 80	R_ER-Main St	P_ERD RT80	J_ERD Main St	4600	0.0004	0.03
East Riser Ditch - Central	R_ER-South	P_ERD Central	J_ER Tide Gate	5262	0.0004	0.03
East Riser Ditch - South	R_BC-2	P_ERD South	J_BC-2	450	0.00216	0.03
East Riser Ditch - South	R_BC-2-3	J_BC-2	J_BC-3	1900	0.00216	0.03
Peach Island Creek	R_BC-4	P_PIC	J_BC-3	1860	0.00216	0.03
Dell Road	R_BC-3	P_Dell Road	J_BC-3	1900	0.00216	0.03
Dell Road	R_BC-3-4	J_BC-3	P_PatersonPlankBr	915	0.00216	0.03

7.7 Diversion, Lag and Routing of Major Storm Sewer Hydrographs

Diversion, reach lag, reach-routing and confluences/junctions of flow hydrographs to simulate interior drainage behavior in local major storm sewers through the storm sewers were modeled in HEC-HMS (**Table B2-13**). The routing through storm sewers was estimated to be pure lag because of their minimal storage capacity. Pure lag routing indicates no change in hydrograph shape, only a single time delay for the reach travel time along all its ordinates.

Most direct runoff hydrographs computed at the outlets of the interior sub-basins have the potential to divert before reaching a storage area, pond, or pump system. Part of the runoff may move towards the Hackensack River or Berry's Creek through the existing storm sewer system via intakes or catch basins. Once the storm sewer system is charged to capacity, the remainder of the runoff will overflow through streets and open areas as open channel flow or overland flow.

Lag times for the major storm sewer reaches were found by computing their capacities using the Manning formula with roughness “n” = 0.03 with the cross-section and slope. Average velocity was found by dividing capacity by the cross-sectional area of flow. Appropriate dimensional conversions were made prior to calculating the average velocity. Reach travel times were computed as length divided by velocity and rounded to the nearest five minute mark in order to input them as an integer number of translation routing steps into the HEC-HMS models.

The diversion functions in the HEC-HMS model input file are also based on the storm sewer capacities computed as described above. Diversion functions assign labels to the diverted and residual hydrographs and pair inflow with diverted flow. Zero inflow is paired with zero diverted flow. Diverted flow equals inflow up to, and including, storm sewer capacity. Diverted flow then remains constant at this maximum value of storm sewer capacity. No matter how high inflow becomes (10,000 cfs was evaluated in HEC-HMS to cover all size floods up to and including the 500-year flood) the diverted flow remains the same and the remaining residual flow is then routed downstream as open channel flow.

One of the major diversions is from East Riser Ditch towards West Riser Ditch. East Riser Ditch overflows during extreme events across the Teterboro Airport runway into the West Riser Ditch area. The flooding WSEL between East and West Riser Ditch almost equalizes to the same elevation (within ½ foot), but East Riser Ditch remains slightly higher.

Table B2-13: HEC-HMS Model Diversion Data

Hydrologic Element	Capacity, cfs (diverted flow)	From	To	Via
West Riser Ditch to East Riser Ditch at Route 80	Not applicable	West Riser Ditch – North	West Riser – Main Street	East Riser Ditch at Route 80
West Riser Pump Station 36” RCP	57	West Riser Ditch at Moonachie Ave	West Riser at Moonachie Avenue - Junction	West Riser Ditch Pump Station
East Riser Ditch to West Riser Ditch	Not applicable	East Riser Ditch Central - Junction	East Riser Ditch - Central	West Riser at Moonachie Ave

Notes: The figure in the “capacity” column is the diverted flow. All inflow up to and including this value of “capacity” is diverted. For inflows above this value, the diverted flow remains constant at “capacity”.

7.8 Tide Gate Gages and Validation

Most drainage areas in the Project Area do not have gage stations monitoring either water levels or flow; therefore, calibrating the rainfall-runoff model to previous precipitation events was not performed for these drainage areas. The following tide gates monitor water levels upstream and downstream of their respective control structures:

- West Riser Ditch Tide Gate;
- East Riser Ditch Tide Gate;
- Moonachie Creek Tide Gate; and
- Losen Slote Tide Gate.

Limited validation runs were performed to confirm the model results were realistic. Hurricane Joaquin (2015) was selected as a representative event to validate model performance. **Figure B2-13**, **Figure B2-14**, **Figure B2-15**, and **Figure B2-16** compare HEC-HMS model results to tide gage data locally and at the River Barge gage, as well as to precipitation depths from the Teterboro Airport gage. The model generally performs well in all locations. The largest differences are noted at West Riser Tide Gate due likely to blockages in the channel which slow discharge as characterized by the elevated water levels in the channel during low tide.

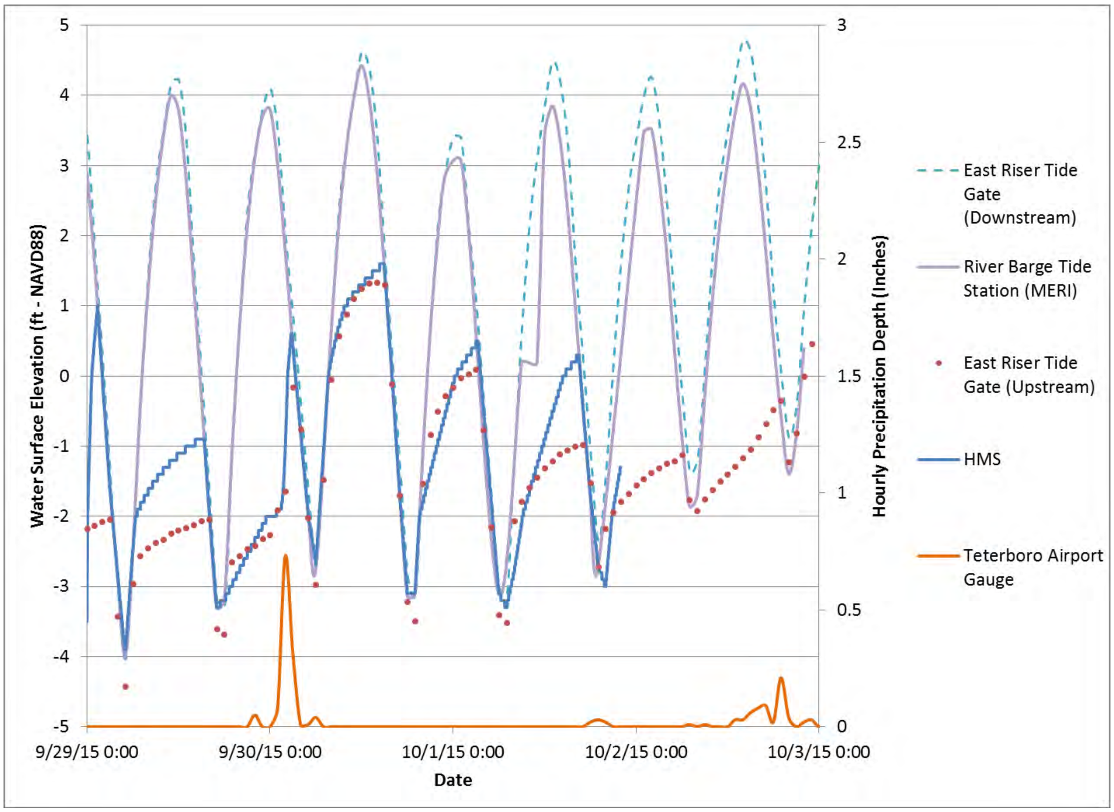

Figure B2-13: Hurricane Joaquin Validation Run Comparison (East Riser)

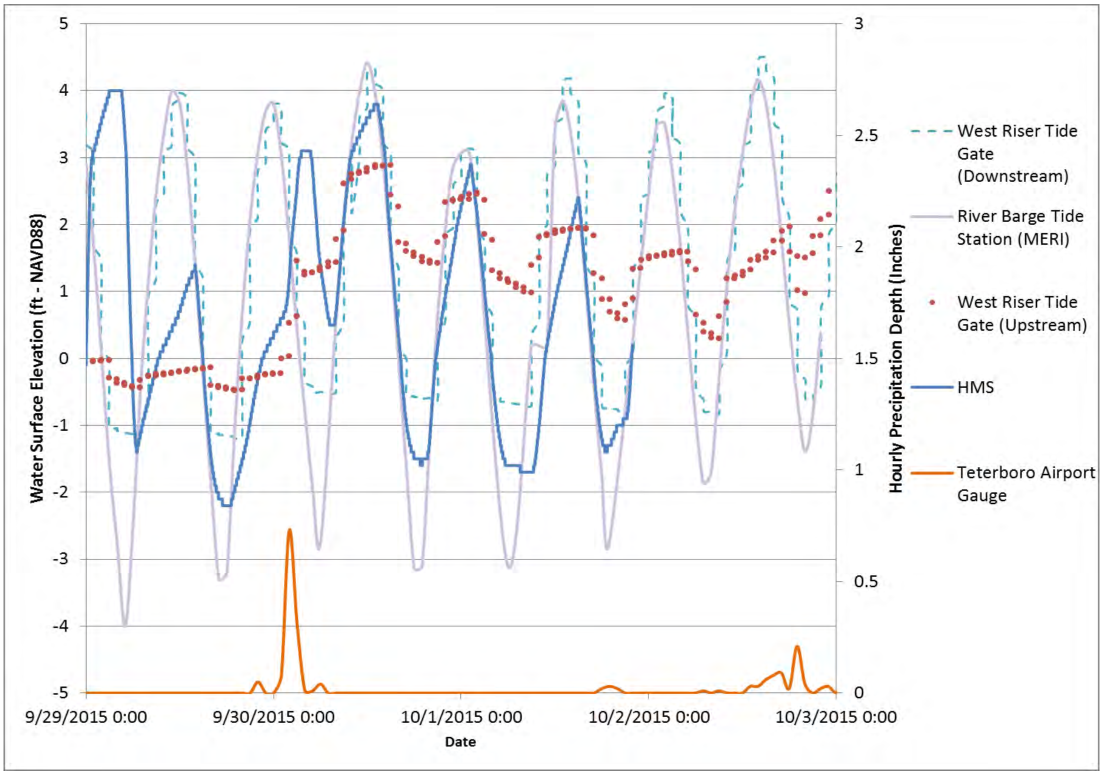

Figure B2-14: Hurricane Joaquin Validation Run Comparison (West Riser)

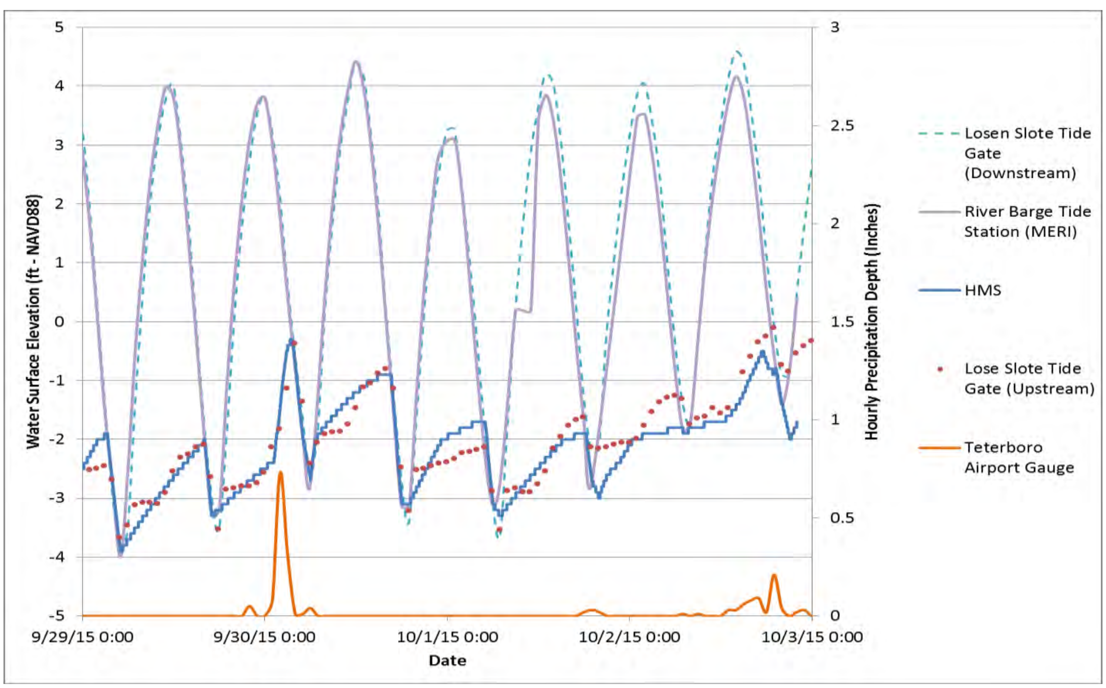

Figure B2-15: Hurricane Joaquin Validation Run Comparison (Losen Slote)

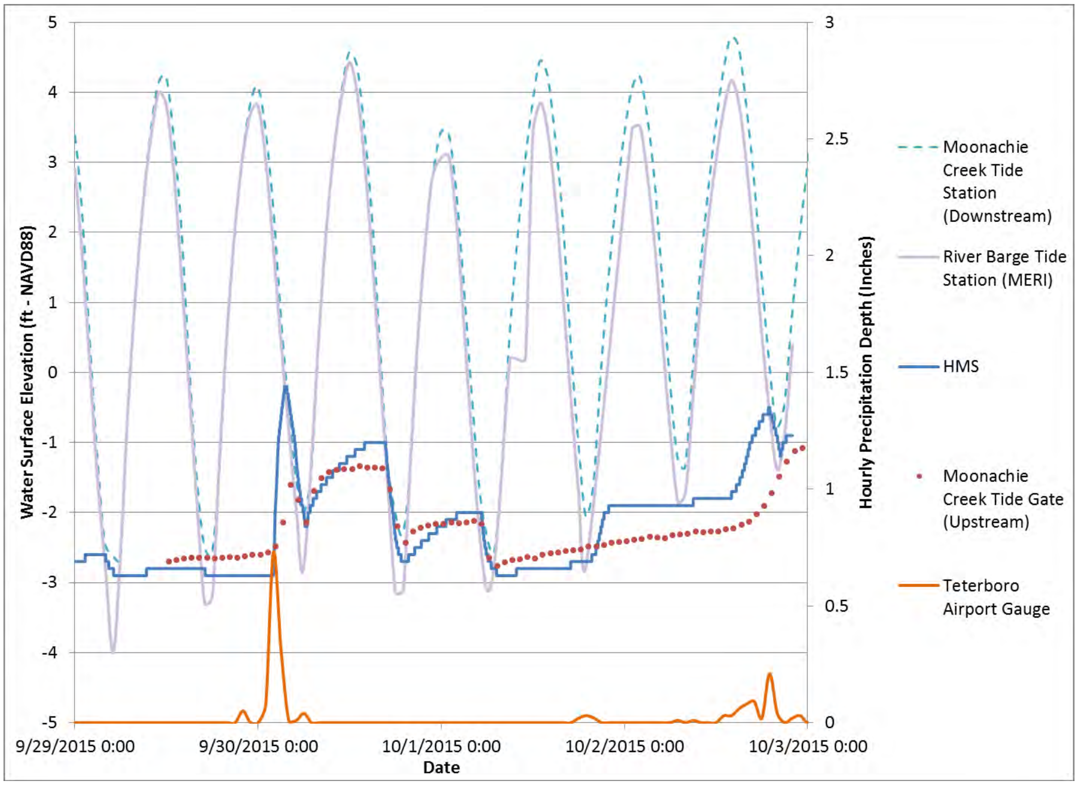

Figure B2-16: Hurricane Joaquin Validation Run Comparison (Moonachie Creek)

Available water level data was somewhat limited and at times suspect, but several historic events were run using observed rainfall data from NOAA Station 725025-94741 – Teterboro Airport, NJ and observed water level data from MERI's River Barge water level gauge for the upstream and downstream boundary conditions respectively.

8.0 Drainage Area Peak Flows

A summary of the Hackensack River and Berry's Creek drainage area existing condition peak flow results from the HEC-HMS models is provided in this section.

8.1 Peak Flow Results

Table B2-14 provides the peak flow results in cfs for the modeled series of fluvial events for the Hackensack River drainage areas. **Table B2-15** presents the peak flow results for the Berry's Creek drainage areas.

Table B2-14: Interior Drainage Area Peak Flows - Hackensack River

Interior Drainage Areas	Peak Flows (cfs) per Fluvial Event							
	2-year	5-year	10-year	25-year	50-year	100-year	250-year	500-year
DePeyster Creek	64	86	103	126	145	164	187	209
Indian Lake	39	57	72	93	109	126	147	166
Losen Slote	558	751	900	1126	1276	1456	1701	1912
MainStreet Ditch	98	133	159	196	224	255	292	327
Moonachie Creek	201	271	310	377	454	492	581	634
Willow Lake	62	90	115	133	143	155	168	181
HACK 1A3	7	9	10	12	13	15	17	19
HACK 1	24	32	38	46	53	59	67	75
HACK 2	20	26	30	36	40	45	51	56
HACK 3	1	2	2	2	3	3	3	4
HACK 4	8	10	12	15	17	20	22	25
HACK 5	3	3	4	5	5	6	7	7
HACK 6	8	11	13	16	19	21	24	27
HACK 7	5	7	8	10	11	12	14	15
HACK 8	1	2	2	3	3	3	4	4
HACK 9	8	10	11	14	15	17	19	21

Table B2-15: Interior Drainage Area Peak Flows - Berry's Creek

Interior Drainage Areas	Peak Flows (cfs) per Fluvial Event							
	2-year	5-year	10-year	25-year	50-year	100-year	250-year	500-year
Dell Road	34	42	48	54	59	64	70	75
East Riser Ditch Central	210	216	221	224	226	227	228	279
East Riser Ditch South	382	498	581	744	901	1058	1296	1579
Peach Island Creek	608	784	919	1098	1238	1383	1557	1720
West Riser Ditch South	632	824	1047	1410	1673	1929	2416	2858
Paterson Plank Road Bridge	1134	1420	1634	1935	2157	2607	3032	3324
West Riser Ditch Pump Station	371	466	539	634	708	784	873	956
East Riser Ditch at Route 80	93	120	146	299	450	621	802	944
West Riser Ditch at Moonachie Avenue	1798	2327	2860	3505	4004	4498	5091	5592

8.2 Rainfall Excess

The excess rainfall is that which remains on the ground surface contributing to runoff and not percolating or absorbed into the ground. Rainfall excess hydrographs in digital format were imported from the HEC-HMS digital storage files (HEC-DSS) into Hydrologic Engineering Center - River Analysis System (HEC-RAS) for two-dimensional analysis.

9.0 Summary

The analysis in this subappendix was used for two different types of analyses. The inflow hydrographs were used as the boundary conditions for the unsteady HEC-RAS models and as the basis for the HEC-HMS models. The discussion and results of which can be found in **Subappendix B3**.

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Subappendix B3: Interior Drainage Analysis for Alternative 1

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Table of Contents

List of Tables

List of Figures

Acronyms and Abbreviations

1D	One-dimensional
2D	Two-dimensional
AAD	Average Annual Damages
EAD	Equivalent Annual Damages
HEC-HMS	Hydrologic Engineering Center - Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center - River Analysis System
LOP	Line of Protection
NAVD 88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
NRC	National Research Council
RBDM	Rebuild by Design Meadowlands
SLR	Sea level rise
USACE	United States Army Corps of Engineers
WSEL	Water surface elevation

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1.0 Introduction

The Rebuild by Design Meadowlands (RBDM) Flood Protection Project (the Proposed Project) includes the construction of flood risk reduction measures designed to address the impacts of coastal and systemic inland flooding on the quality of the physical, natural, cultural, and socioeconomic environment due to both storm hazards and sea level rise (SLR) within the Project Area. The purpose of the Proposed Project is to reduce flood risk and increase the resiliency of the communities and ecosystems within the Project Area, thereby protecting critical infrastructure, residences, businesses, and recreational areas from the more frequent and intense flood events anticipated in the future.

The Project Area is within the Meadowlands District and is situated in a valley or “bowl” with ridges on its sides that run parallel in a southwest to northeast direction. In some locations, these ridges are over 100 feet above sea level. Comprised of mostly flat terrain, elevations within the Meadowlands District do not exceed 10 feet above sea level, with elevations in most areas less than 6 to 7 feet above sea level. Flow of water within the Project Area is greatly affected not only by local topography, but also by patterns of urbanization and development. In addition, historic construction of dikes and tide gates in an attempt to control and reduce flooding events has further altered the hydrologic and hydraulic characteristics of the Project Area.

Three alternatives were developed for the Proposed Project. Alternative 1 is the Structural Flood Reduction Alternative, which relies on floodwalls, levees, flap gates, and pump stations to provide a level of protection from both inland and tidal storm surge flooding. Alternative 2 is the Stormwater Drainage Improvement Alternative, which applies a series of stormwater drainage projects aimed at reducing the occurrence of higher frequency, small- to medium-scale inland and tidal storm surge flooding events. Alternative 3, or the Hybrid Alternative, uses a strategic, synergistic blend of new infrastructure and local drainage improvements to reduce flood risk from both inland and tidal storm surges in the Project Area.

This subappendix documents the conditions and hydraulic analysis ‘With’ and ‘Without’ proposed Alternative 1 interior drainage facilities for the Project Area. The analysis herein represents the results of the interior drainage facility performance assessment.

This subappendix has been organized to provide the reader with an overview of the hydraulic models with computed results and economic damage analysis. The evaluation effort incorporates an analysis of varying types and sizes of interior drainage facilities to determine the plan which adequately meets the design criteria of each area.

1.1 Without Interior Drainage Facilities

Existing interior drainage facilities (i.e., without the proposed interior drainage facilities under Alternative 1) lay upland of the marshlands and the river banks that run along the Hackensack River and Berry’s Creek. The existing structures and landforms form a line of protection (LOP) to an elevation of approximately 5 feet North American Vertical Datum of 1988 (NAVD 88).

Presently a total of five pump stations and multiple drainage outlets with tide gates are located along the existing LOPs within the Project Area. The tide gates that are equipped with the pump stations include: Main Street Ditch, Willow Lake, DePeyster Creek, Indian Lake, and Losen Slote. These tide gates function to allow outflow when the interior stormwater elevation is higher than the exterior or tidal elevations.

1.2 With Interior Drainage Facilities

Alternative 1 (i.e., the ‘With’ condition) uses structural measures to form a LOP in order to reduce flood levels in the Project Area. It consists of a system of flood walls, levees, road raisings, tie-ins to existing high ground, and other features to provide protection up to 7 feet NAVD 88. These structural measures are broken into two overall groups, the Hackensack River and Berry's Creek.

This interior drainage analysis for Alternative 1 includes Hackensack drainage areas HACK 1A3, HACK 1 through HACK 9, DePeyster Creek, Indian Lake, Losen Slote, Main Street Ditch, Moonachie Creek, and Willow Lake. Berry's Creek drainage areas consist of Dell Road, East Riser Ditch South, Peach Island Creek, West Riser Ditch South, and Paterson Plank Road Bridge.. See **Figure B3-1** for the locations of these RBDM drainage areas.

Various options were considered for Alternative 1:

- The alignment along the Hackensack River was split into three sections; Northern, Central, and Southern. Five alignment options were considered for the Northern section. NE 3, which was proposed to run along the Hackensack River and extends to the Hackensack Riverwalk, was selected through prior screening. Therefore, only NE 3 was modeled for the Northern section. There was only one option for the Central section, which was modeled. There were three main options considered for the Southern section. SE 2, which has the proposed LOP adjacent to Commerce Boulevard, was selected through prior screening and was modeled.
- Three options were considered for Berry's Creek area. Option 1 consists of a surge barrier on Berry's Creek just downstream of Paterson Plank Road Bridge. Option 2 consists of localized floodwalls and pump stations along the east bank of Berry's Creek. Option 3 consists of localized floodwalls and pump stations along the east and west banks of Berry's Creek. Option 1 and 3 were modeled. Option 2 was dismissed during prior screening due to its similarity in nature to Option 3 and the lack of protection of the west side of Berry's Creek without showing offsetting cost reductions.

The proposed Hackensack River facilities would consist of a series of levees, flood walls, tide gates, and outlets to create a LOP along the bank of the river. Berry's Creek Option 1 would consist of a surge barrier with a pump station located just downstream of the Paterson Plank Road Bridge.

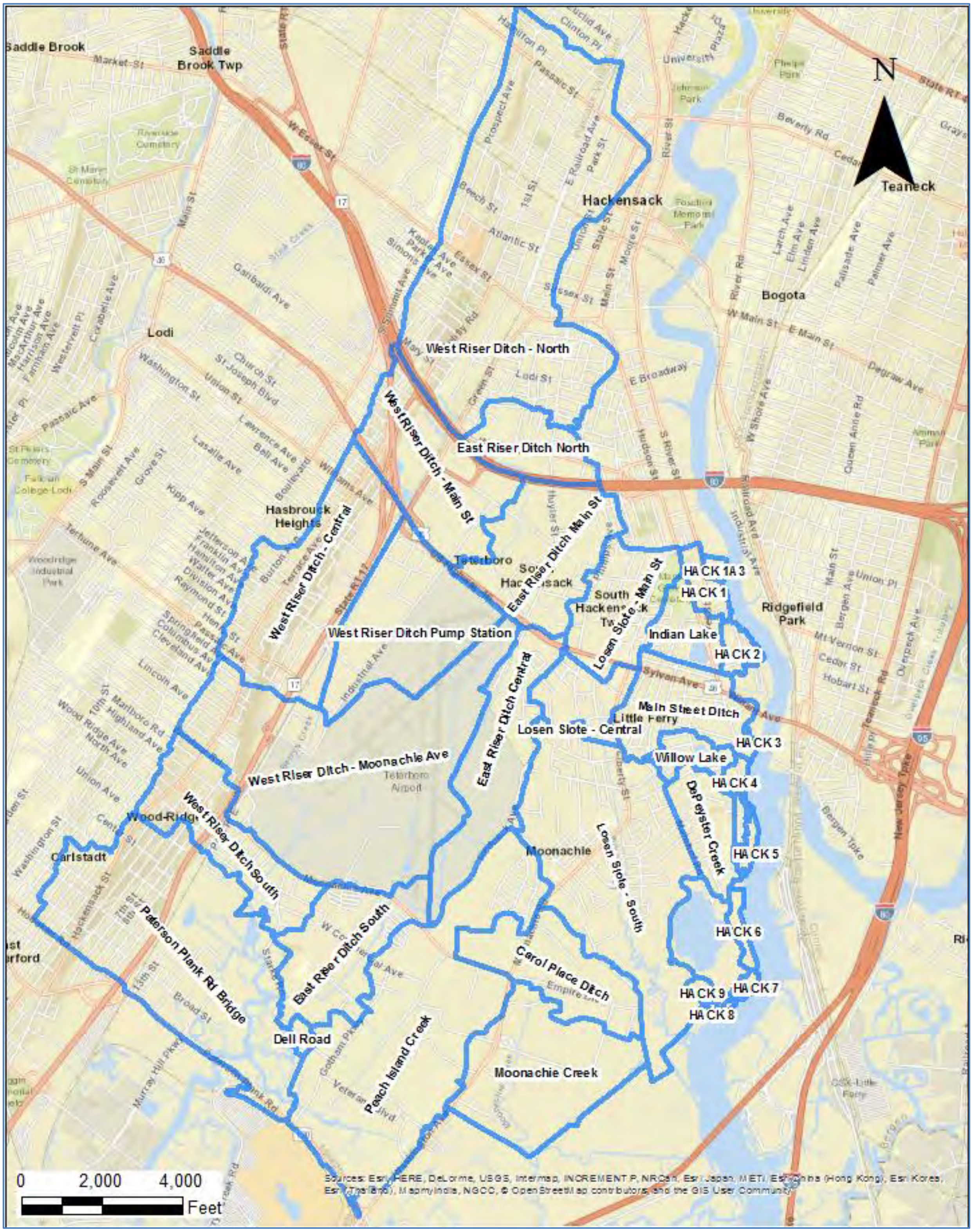

Figure B3-1: RBDM Drainage Areas

1.2.1 Gravity Outlet Retrofits

The proposed floodwalls, levees, and surge barrier under Alternative 1 would trap stormwater behind the LOP that would otherwise naturally flow out of the drainage areas to the Hackensack River or Berry's Creek, causing residual flooding. To prevent residual flooding, the existing stormwater outlets were analyzed for each interior drainage area to determine if additional outlets are required to prevent residual flooding. All existing gravity outlets are either fitted, or would be fitted, with flap gates to prevent tidal or high river backflow. These outlet improvements would allow the drainage areas to drain as if the LOP was not in place using only the current and supplemental gravity outlets. This was used as the baseline evaluation. If the design objective was not met with the outlet improvements, then the addition of pumps was evaluated. The target event for analysis of the gravity outlets is the 10-year fluvial storm with the 2-year tide.

1.2.2 Analysis of Additional Plan Options

The gravity outlet improvement strategy was the baseline from which various plan options were considered. The benefits accrued from alternatives are attributable to the reduction in the residual flood damages that would have remained under the With condition. For an alternative to be justified, it must be implementable and reasonably maximize benefits versus the cost required for its construction, operation, and maintenance.

From the gravity outlet analysis, it was concluded that all Hackensack River gravity outlets retrofits provided adequate drainage sufficient to avoid any significant induced damages. Therefore, no further alternatives were analyzed for the Hackensack River drainage areas. Berry's Creek drainage areas did require further analysis past the gravity outlet retrofits. Three options were considered for Berry's Creek area, all requiring additional drainage features beyond the gravity outlet retrofits.

1.3 Without and With Future Conditions

After the plan was selected and analyzed based on the present climate conditions, the Without and With conditions were analyzed with estimated future condition parameters. Future conditions incorporate estimated SLR and increases in rainfall runoff, both resulting from climate change. Two future conditions of SLR were considered: (1) National Oceanic and Atmospheric Administration (NOAA) Intermediate-Low / United States Army Corps of Engineers (USACE) Intermediate (Modified National Research Council (NRC) Curve II), and (2) NOAA Intermediate-High. These two conditions will further be referred to as Future Low SLR (approximately 1.2 feet rise) and Future High SLR (approximately 2.4 feet rise), respectively. Rising sea levels and increased rainfall runoff reduce the effectiveness of gravity outfalls to drain low-lying areas and increase the probability that tidal surges overtop lines of protection. Details on the determination of future tide and interior storm events can be found in **Subappendix B2**.

2.0 Interior Drainage Evaluation

The interior drainage areas, behind and upstream of the LOP, were analyzed with hydrologic and hydraulic models to determine the residual flooding impacts of the LOP. The existing drainage pipes that cross through the proposed LOP are assumed to be furnished with flap gates. All existing or proposed pump stations are also incorporated in the models for analysis. The target event is the 10-year rainfall fluvial event versus a typical tide with a 2-year surge, however, a range of various events was run to understand the sensitivity to various fluvial versus tidal event combinations. The results of the hydraulic analyses are the peak interior flood elevations for the various events modeled. These peak interior flood elevations are then input to an economic model to determine the flood damages for each location.

2.1 Interior Drainage Simulation Models

The USACE Hydrologic Engineering Center (HEC)'s hydrologic and hydraulic modeling software Hydrologic Modeling System (HEC-HMS) and River Analysis System (HEC-RAS) were used for the modeling of interior drainage models. Modeling approach for both HEC-HMS and HEC-RAS is described in detail in the following sections.

2.1.1 HEC-HMS Hydrologic Model

HEC-HMS version 4.2 is a hydrologic modeling computer program with limited hydraulic modeling capabilities designed to both compute runoff and to route floods through interior drainage facilities to adjacent rivers, estuaries, or oceans accounting for variable tailwater conditions. This program was utilized to simulate the surface runoff response of the interior basins to precipitation while taking into account both the hydrologic and hydraulic components of these basins. One of the limitations of HEC-HMS is that outflows cannot dynamically respond to variable tailwater conditions and, therefore, models with interconnected ponds may not be routed accurately.

2.1.2 HEC-RAS Hydraulic Model

HEC-RAS version 5.0.3 is a hydraulic modeling computer program originally designed to compute flood water surface elevations (WSELs) on rivers with steady flow computations. Present capabilities include one-dimensional (1D) and two-dimensional (2D) unsteady flow simulations, sediment transport computations, and water quality analysis. Weirs, pump stations, storage areas, and programmable logic control gate operations allow for modeling of complex hydraulics including interior drainage facilities with tide gates on, or adjacent to, rivers, estuaries, or oceans, accounting for variable tailwater conditions. HEC-RAS can model bi-directional tailwater flows, as well as complex logic-controlled tide and sector gate operations, 1D river flow, 2D surface flow, storage elements, and various interconnected pipes and weir elements. HEC-RAS links to runoff hydrograph and excess precipitation data computed by the HEC-HMS program in a database to provide hydrologic input for the hydraulic analysis. This program was utilized to compute WSELs in complex interactive tailwater conditions, such as the Berry's Creek Option 1.

HEC-HMS uses the hydrology conditions to calculate the inflows for each drainage area. Then, HEC-HMS or HEC-RAS use those flows determine how much of that flow can exit through the hydraulic structures of an area and how much of it ponds inside the proposed LOP. The Alternative 1 design was based on the 'Likely' condition (2-year tidal surge with the 10-year interior storm event).

2.2 Hydrologic and Tidal Surge Parameters

Multiple storms and surge events were analyzed for the Project Area in order to understand the sensitivity of the strong tidal surges and large rainfall events. Storm hydrographs were developed for eight returns periods and tidal hydrographs were developed for six. Since there is uncertainty in the relationship between the tidal events and the storm surges, various combinations of the two were run in the models in order to capture the uncertainty into the economic damages analysis. HEC-HMS and HEC-RAS then used these storm events to create an interior flow of surface runoff that either exits the area towards the Hackensack River in the Without condition; ponds behind currently existing high land or the proposed LOP; or flows out through Without or With hydraulic structures. The details and assumptions of the hydrologic storm and tidal events are described below and in **Subappendix B2**.

2.2.1 Storm Surge Development

For storm surge events (tropical events such as hurricanes and extratropical events such as nor'easters), a storm hydrograph was developed to simulate surge levels during storm conditions. Two main assumptions were made to develop the storm hydrograph: (1) the peak elevation of the storm will occur at high tide, and (2) the duration of the storm is approximately two days. A typical synthetic tide waveform was developed and various storm surges were added to the waveform to create the various tidal surge events. The 2-year, 5-year, and 10-year surge events were applied to the various hydraulic models.

2.2.2 Stormwater Runoff

Subappendix B2 describes the hydrologic models and sources of data applied to the hydrologic models to provide runoff data to the hydraulic model. Storm hydrographs were developed for eight return periods; 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-years; and the peak elevation for each return period was developed.

2.2.3 Combination of Fluvial Runoff and Storm Surge Events with Uncertainty

In order to develop a stage-frequency relationship and capture the uncertainty of the relationship between rainfall runoff and tidal storm surges, the interior events were routed against exterior tidal hydrographs. For the most likely flooding scenarios, the eight interior storm events were routed against a 2-year exterior tide and a 2-year interior storm event was routed against the six exterior events (the typical, 2-, 5-, 10-, 25-, and 50-year tidal surge). The same process was applied for the upper and lower bounds with the 10-year and typical tide, respectively. **Table B3-1** presents the array of interior and exterior model runs analyzed and the risk condition associated with each.

Table B3-1: Selected Analysis Approach - Interior versus Exterior Conditions

Varied Interior Condition		Varied Exterior Condition		Risk Condition
Interior Flow	Exterior Stage	Interior Flow	Exterior Stage	
2-yr	Typical Tide	NA	NA	Lower Bound
5-yr	Typical Tide	NA	NA	Lower Bound
10-yr	Typical Tide	NA	NA	Lower Bound
25-yr	Typical Tide	NA	NA	Lower Bound
50-yr	Typical Tide	NA	NA	Lower Bound
100-yr	Typical Tide	NA	NA	Lower Bound
250-yr	Typical Tide	NA	NA	Lower Bound
500-yr	Typical Tide	NA	NA	Lower Bound
2-yr	2-yr Surge	2-yr	2-yr Surge	Most Likely
5-yr	2-yr Surge	2-yr	5-yr Surge	Most Likely
10-yr	2-yr Surge	2-yr	10-yr Surge	Most Likely
25-yr	2-yr Surge	2-yr	25-yr Surge	Most Likely
50-yr	2-yr Surge	2-yr	50-yr Surge	Most Likely
100-yr	2-yr Surge	2-yr	NA	Most Likely
250-yr	2-yr Surge	2-yr	NA	Most Likely
500-yr	2-yr Surge	2-yr	NA	Most Likely
2-yr	10-yr Surge	10-yr	2-yr Surge	Upper Bound
5-yr	10-yr Surge	10-yr	5-yr Surge	Upper Bound
10-yr	10-yr Surge	10-yr	10-yr Surge	Upper Bound
25-yr	10-yr Surge	10-yr	25-yr Surge	Upper Bound
50-yr	10-yr Surge	10-yr	50-yr Surge	Upper Bound

100-yr	10-yr Surge	10-yr	NA	Upper Bound
250-yr	10-yr Surge	10-yr	NA	Upper Bound
500-yr	10-yr Surge	10-yr	NA	Upper Bound

As demonstrated in the Risk Condition column of **Table B3-1**, uncertainty was incorporated into the analysis by establishing lower and upper coincidental frequency bounds. The maximum WSEL of corresponding coincidental frequencies (e.g., 2-year interior and 10-year exterior, or 10-year interior and 2-year exterior) was identified as the most damaging flood level for the coincidental frequency. In the With analysis, the 25- and 50-year exterior events were found to be more damaging than its corresponding reversed condition frequency because it would overtop the proposed LOP. The three conditions, Likely, Lower Bound, and Upper Bound, were then incorporated into the economic analysis using a triangular probability distribution. The analysis was performed for both the present and two future conditions to include the impacts of SLR and increased intensity of fluvial storms.

2.3 Hydraulic Parameters

In addition to the development of hydrologic data, the analysis of interior drainage facilities required additional input to describe the physical and operational characteristics of gravity outlet retrofits and other alternatives. Input requirements consisted of potential storage volumes, outlet sizes, and pumping rates for both the Without and With conditions. No single hydraulic measure is effective in all situations and typically no single hydraulic measure is effective by itself. The most cost effective approach for Alternative 1 to reduce interior flooding stages is likely to be a combination of hydraulic measures. HEC-HMS and HEC-RAS programs were utilized to evaluate the effects of Without or With hydraulic structures by routing interior fluvial flood events through the ponding areas. The assumptions, criteria, and descriptions of hydraulic structures used to inform the models are described below.

2.3.1 Ponding Storage Volume

In order to evaluate the stormwater storage capacity behind the proposedLOP, elevation and storage relationships in the ponding areas were developed. Using Proposed Project mapping and commencing with the lowest elevation at the natural ponding site behind the proposed LOP, the planimetric area enveloped by a particular elevation was computed. The conic volume method was used to compute the volume by elevation in either in HEC-HMS or HEC-RAS.

2.3.2 Gravity Outlets

Gravity outlets use the driving head to drain interior runoff. The driving head of runoff outflow from the protected areas is the elevation difference between two water surfaces: (1) the elevation of runoff that is accumulated landward of the plan alignment (headwater), and (2) the elevation of the surge seaward of the plan alignment (tailwater). Flap gates would be installed onto gravity outlets to prevent backflow from the tailwater when headwaters are low. The minimum head to open the flap gates for gravity outlet operation through the levees and floodwalls was estimated to be 0.2 feet.

HEC-HMS is not capable of modeling flow in two directions through an outlet. Therefore, it only models flow from the headwater to the tailwater. HEC-HMS assumes zero outflows when the tailwater is higher than headwater. HEC-RAS, however, can model two-directional flow through outlets. All proposed outlets are modeled with flap gates, which would make those outlets function as they do in HMS, with one directional flow from headwater to tailwater and zero flow for larger tailwater events. The surge barrier is the exception as it would allow flow inward if the gate is open. Taking a conservative approach, the surge barrier gate is being modeled to function like a flap gate, where it closes when the tailwater is higher than the headwater.

Gravity outlets, typically the least expensive drainage measure, function best during high rainfall coupled with low tide events, when there is sufficient head for gravity discharge. Gravity outlets also work well when the current grade landward of the plan alignment is higher, again providing additional head. Conversely, gravity outlets are ineffective during high tide events when the tailwater elevations are higher than the interior elevations. During these events, outlets are effectively blocked and thus the gravity discharge is zero. Gravity outlets do not function well with large, low-lying natural flood storage areas such as freshwater wetlands, where even a moderate tide can prevent gravity discharge.

2.3.3 Tidal Surge Barrier

A tidal surge barrier provides protection in-line on rivers where large fluvial flows and tidal surges are experienced. Surge barriers are closed just prior to anticipated tidal surges and typically use pump stations to evacuate the fluvial flows to the marine side of the barrier.

2.3.4 Pumping

Pumping is usually the most costly option in initial construction, as well as operation and maintenance, and is therefore typically considered the “last resort.” Pumping is most effective during higher exterior stages when gravity outlets are blocked and there is insufficient natural flood storage area landward of the plan alignment. Pumping can be used to reduce the volume of a ponding area, or it can be used to handle the peak runoff. The construction of a pump station creates additional capital costs and also increases annual maintenance and operation costs. Capital expenditures affected by the addition of pump stations include mechanical equipment, associated housing, and any new outfalls. Increases in the cost of Proposed Project operation and maintenance include power consumption, equipment operation, inspection and testing, maintenance, and replacement.

3.0 Interior Drainage Evaluation Results

In this section, the results of the hydraulic analyses are presented in tables of peak interior flood elevations and include a count of the number of structures flooded for each area. The Without and With tables are presented with results for the Present, Future Low SLR, and Future High SLR hydrologic conditions, respectively. For each drainage area, the first results table provides a detailed summary of the WSEL and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. The second table for each drainage area provides a summary of the With conditions in the same format.

Each area includes a description of existing and proposed gravity drainage outlets, either an open channel or closed conduit pipes and culverts, and specify surface overflows that occur when pipes are blocked or at capacity. These surface overflows are modeled as spillway weirs with a separate spillway for each continuous elevation in the computer model. Each ‘spillway’ is identified with a length and an elevation.

3.1 Hackensack River Drainage Areas

There are two general sets of the Hackensack River drainage areas. One set consists of small areas that border the banks of the Hackensack River, designated as the ‘HACK’ group. The HACK group includes HACK 1A3 and HACK 1 through HACK 9. The other set consists of much larger areas, extending more to the inland interior and designated as the Hackensack Interior group. This group includes DePeyster Creek, Indian Lake, Losen Slote, Main Street Ditch, Moonachie Creek, and Willow Lake. All Hackensack River drainage areas were modeled using HEC-HMS. Since the Hackensack Interior group is further inland than the HACK group, most of the areas do not see significant effects from the proposed LOP. These areas were further analyzed for other Proposed Project alternatives. **Table B3-2** provides a summary of the Hackensack River drainage areas and associated outlet features.

Table B3-2: Hackensack Interior Areas with Existing and Proposed Outlet Features

Interior Areas Draining to Hackensack River - all elevations (el.) in feet (NAVD 88)				
Interior Area	**Drainage Area (Acres)**	**Gravity Outlets**	**Surface Overflow (weir)**	**Pump Station**
DePeyster Creek	84	Existing 3.5-foot flap gate, 2-foot flap gate to be added	50-foot wide at eEl. 6.0	3 x 2 cfs pumps
Indian Lake	64	None	100-foot wide at el. 6.0	2 x 55 cfs pumps 2 x 6 cfs pumps
Losen Slote - Carol Pl Ditch	167	via ditch to Losen Slote South	-	-
Losen Slote - Main Street	144	flows to Losen Slote Central	-	-
Losen Slote – Central	80	flows to Losen Slote South	-	-
Losen Slote – South	492	3 x 5-foot x 5-foot flap gates to Hackensack River	2,500-foot at el. 6.5	3 x 96 cfs pumps
Main Street Ditch	141	1 x 2-foot flap gate to be added	50-foot at el. 5.5	2 x 18 cfs pumps
Moonachie Creek	294	2 x 8-foot diameter flap gates	2,500-foot at el. 6.0	-
Willow Lake	37	None	30-foot at el. 5.0	2 x 12 cfs pumps
HACK 1A3	6	2 x 2-foot diameter pipes	35-foot at el. 5.0 (to be blocked)	-
HACK 1	30	3 x 2-foot diamerter pipes	20-foot at el. 5.5	-
HACK 2	21	4 x 3-foot diameter pipes	60-foot at el. 6.5 (to be blocked)	-
HACK 3	1	None existing, 1 x 2-foot dia. flap gate to be added	65-foot at el. 5.0	-
HACK 4	13	2 x 2-foot diameter pipes	47-foot at el. 4.0 (to be blocked)	-
HACK 5	12	3 x 2-foot diameter pipes	197-foot at el. 4.0 (to be blocked)	-
HACK 6	15	None existing, six 2-foot diameter flap gates to be added	190-foot at el. 5.0 (to be blocked)	-
HACK 7	7	1 x 3-foot diameter external pipe, 3 x 3-foot diameter. flap gates to be added	250-foot at el. 5.0 (to be blocked)	-
HACK 8	3	1 x 3-foot diameter external pipe, 3 x 3-foot diameter flap gates to be added	250-foot at el. 5.0 (to be blocked)	-
HACK 9	7	1 x 2-foot diameter flap gates to be added	105-foot at el. 3.0 (to be blocked)	-

3.1.1 DePeyster Creek Drainage Area

Under the Without conditions, DePeyster Creek flows to the Hackensack River via a 3.5 foot diameter tide gate. Stormwater surface overflows out to the Hackensack River are represented in the model as a 50-foot long spillway at elevation 6.0 feet (NAVD 88). If the proposed LOP is constructed, the current 3.5-foot diameter tide gate would be the only means for runoff to flow through the LOP, which would lead to an increase in interior water levels. Stormwater is currently pumped out through the tide gate via three 2-cfs pumps. The structures needed to address the increase of water level and duration of flooding is provided in the following paragraphs.

Gravity outlet improvements would include an additional sluice gate structure with a 2-foot diameter outlet to increase outflow. (Refer to the Alternative 1 Plan Set for typical details of the gates.) Drainage swales would be constructed along the landward side of the coastal storm risk management structure to direct runoff toward the outlets. All outlets that would run through the proposed LOP would be fitted with flap gates.

Table B3-3 provides a summary of the WSELs and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-4** provides a summary of the With conditions in the same format.

Table B3-3: DePeyster Creek Drainage Area Results – Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.09	0	0	3.71	9	0	4.14	18	1
5 yr	3.28	2	0	4.02	14	1	4.32	24	2
10 yr	3.45	6	0	4.14	18	1	4.46	37	5
25 yr	3.84	12	1	4.25	21	2	4.65	55	6
50 yr	4.04	14	1	4.31	23	2	4.80	69	9
100 yr	4.14	18	1	4.43	36	5	4.92	81	11
250 yr	4.27	22	2	4.56	44	5	5.01	87	14
500 yr	4.41	36	5	4.69	60	7	5.07	90	14
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.82	12	1	4.13	18	1	4.46	37	5
5 yr	4.05	14	1	4.30	23	2	4.62	51	5
10 yr	4.17	18	1	4.46	37	5	4.71	61	7
25 yr	4.41	36	5	4.74	65	7	4.96	84	13
50 yr	4.62	51	5	4.94	84	13	5.08	91	14
100 yr	4.89	81	11	5.05	87	14	5.19	99	15
250 yr	5.02	87	14	5.11	94	15	5.25	102	18
500 yr	5.07	90	14	5.17	98	15	5.30	105	19
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.40	35	5	4.41	36	5	4.63	53	5
5 yr	4.40	35	5	4.42	36	5	4.69	60	7
10 yr	4.48	38	5	4.63	53	5	4.75	66	9
25 yr	4.74	65	7	4.92	81	11	4.96	84	13
50 yr	4.97	87	14	5.04	87	14	5.08	91	14
100 yr	5.06	88	14	5.14	94	15	5.19	99	15
250 yr	5.12	94	15	5.21	99	15	5.25	102	18
500 yr	5.18	99	15	5.26	102	17	5.31	105	19

Table B3-4: DePeyster Creek Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.02	0	0	3.71	9	0	4.14	18	1
5 yr	3.17	1	0	3.93	12	1	4.32	24	2
10 yr	3.28	2	0	4.14	18	1	4.46	37	5
25 yr	3.52	6	0	4.25	21	2	4.64	55	6
50 yr	3.78	11	1	4.25	21	2	4.78	68	9
100 yr	4.03	14	1	4.31	23	2	4.90	81	11
250 yr	4.14	18	1	4.43	36	5	5.00	87	14
500 yr	4.26	21	2	4.55	43	5	5.05	87	14

Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.81	12	1	4.11	15	1	4.41	36	5
5 yr	3.98	13	1	4.26	21	2	4.62	51	5
10 yr	4.08	15	1	4.41	36	5	4.67	58	7
25 yr	4.29	22	2	4.66	57	7	4.96	84	13
50 yr	4.49	39	5	4.88	79	11	5.08	91	14
100 yr	4.75	66	9	5.02	87	14	5.18	99	15
250 yr	4.93	83	12	5.08	91	14	5.25	102	18
500 yr	5.02	87	14	5.13	94	15	5.30	105	19

Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.40	35	5	4.41	5	5	4.63	53	5
5 yr	4.40	35	5	4.42	5	5	4.64	55	6
10 yr	4.44	36	5	4.63	9	5	4.72	64	7
25 yr	4.69	60	7	4.91	14	11	4.96	84	13
50 yr	4.91	81	11	5.04	87	14	5.08	91	14
100 yr	5.03	87	14	5.13	94	15	5.19	99	15
250 yr	5.09	93	15	5.19	99	15	5.25	102	18
500 yr	5.14	94	15	5.24	101	17	5.31	105	19

3.1.2 Indian Lake Drainage Area

Indian Lake is a man-made recreational lake. Under the Without conditions, surface runoff upstream of the lake is able to be conveyed through the existing lake outlet controls. The surface water elevation of the lake is maintained at 6.0 feet (NAVD 88) with two 55-cfs pumps, two 6-cfs pumps, and a 100-foot long overflow spillway at 6.0 feet (NAVD 88). The surface water elevation remains unchanged by all model conditions run. Since Indian Lake is further inland, it is not directly affected by the construction of the proposed LOP. Therefore, no improvements are proposed for Indian Lake.

3.1.3 Losen Slote Drainage Area

Under the Without conditions, the surface runoff from Losen Slote is able to be conveyed to the Hackensack River via three 5-foot by 5-foot tide gates and, when the river is high, stormwater is pumped out from Losen Slote South to the Hackensack River via three 96-cfs pumps. Interior stormwateralso overflows to the Hackensack River via overland flows at an elevation of 6.5 feet (NAVD 88) over and is modeled as a 2,500 foot long spillway.

If the proposed LOP is constructed, the tide gates (three 5-foot x 5-foot box culverts) and three 96-cfs pumps would be the only means for interior runoff to flow through the LOP, which would lead to an increase in interior water levels. The proposed structures needed to address the increase of water level and duration of flooding is provided in the following paragraphs.

Gravity outlet retrofits would include four additional sluice gate structures, each with 2-foot diameter outlets to increase outflow. Drainage swales would be constructed along the landward side of the coastal storm risk management structure to direct runoff toward the structures. All culverts that would run through the LOP would be fitted with flap gates.

Table B3-5 provides a summary of the WSELs and number of structures impacted under the Without conditions for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-6** provides a summary of the With conditions in the same format.

Table B3-5: Losen Slote Drainage Area Results - Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	1.34	0	0	1.43	0	0	2.18	0	0
5 yr	1.72	0	0	1.95	0	0	2.27	0	0
10 yr	1.98	0	0	2.18	0	0	2.27	0	0
25 yr	2.32	0	0	2.53	2	1	2.61	2	1
50 yr	2.57	2	1	2.72	4	2	2.97	13	4
100 yr	2.78	6	3	3.03	16	4	3.28	31	7
250 yr	3.12	22	6	3.35	39	9	3.67	70	13
500 yr	3.38	43	9	3.62	67	13	4.01	121	21
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	1.61	0	0	1.60	0	0	2.57	2	1
5 yr	2.11	0	0	2.17	0	0	2.58	2	1
10 yr	2.44	1	1	2.57	2	1	2.62	2	1
25 yr	2.98	15	4	3.14	23	6	3.26	30	6
50 yr	3.33	36	8	3.53	55	11	3.75	80	15
100 yr	3.89	104	18	4.09	137	25	4.31	192	35
250 yr	4.25	178	33	4.42	221	37	4.71	320	50
500 yr	4.53	255	43	4.71	320	50	5.03	469	70
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	1.61	0	0	1.62	0	0	2.58	2	1
5 yr	2.16	0	0	2.13	0	0	2.59	2	1
10 yr	2.57	2	1	2.58	2	1	2.62	2	1
25 yr	3.15	23	6	3.24	29	6	3.28	31	7
50 yr	3.53	55	11	3.69	70	13	3.75	80	15
100 yr	4.09	137	25	4.23	174	32	4.33	196	35
250 yr	4.43	224	37	4.60	272	46	4.74	332	53
500 yr	4.73	330	53	4.95	425	61	5.05	478	70

Table B3-6: Losen Slote Drainage Area Results - With Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
Present Conditions									
Event	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	1.32	0	0	1.43	0	0	2.18	0	0
5 yr	1.68	0	0	1.95	0	0	2.27	0	0
10 yr	1.98	0	0	2.18	0	0	2.27	0	0
25 yr	2.32	0	0	2.51	2	1	2.61	2	1
50 yr	2.56	2	1	2.69	3	1	2.97	13	4
100 yr	2.77	5	3	3.00	16	4	3.27	31	7
250 yr	3.07	20	6	3.30	34	8	3.66	69	13
500 yr	3.29	32	7	3.56	58	12	3.99	118	21
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	1.61	0	0	1.60	0	0	2.57	2	1
5 yr	2.10	0	0	2.17	0	0	2.59	2	1
10 yr	2.43	1	1	2.57	2	1	2.64	3	1
25 yr	2.94	12	4	3.13	22	6	3.26	30	6
50 yr	3.28	31	7	3.50	49	10	3.75	80	15
100 yr	3.81	88	16	4.05	128	24	4.31	192	35
250 yr	4.17	156	29	4.37	209	35	4.70	313	49
500 yr	4.45	231	40	4.66	302	48	5.02	467	70
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	1.61	0	0	1.63	0	0	2.59	2	1
5 yr	2.17	0	0	2.13	0	0	2.59	2	1
10 yr	2.58	2	1	2.59	2	1	2.62	2	1
25 yr	3.13	22	6	3.24	29	6	3.26	30	6
50 yr	3.50	49	10	3.69	70	13	3.75	80	15
100 yr	4.05	128	24	4.23	174	32	4.33	196	35
250 yr	4.38	212	35	4.58	265	45	4.74	332	53
500 yr	4.67	303	48	4.92	404	59	5.04	472	70

3.1.4 Main Street Ditch Drainage Area

Under the Without conditions, the surface runoff from Main Street Ditch drainage area flows to the Hackensack River without any restrictions. A pump station serves Main Street Ditch with two 18-cfs pumps that pump stormwater out to the Hackensack River. Stormwater also overflows overland at elevation 5.5 feet (NAVD 88) and this is modeled as a 50-foot long spillway. Under the With conditions, this overflow stormwater flow would be blocked by the proposed LOP.

If the proposed LOP is constructed, the water would no longer be able to flow freely over the ground to the Hackensack River, which would lead to an increase in interior water levels. However, water would still be able to flow into Willow Lake drainage area, and then out to the Hackensack River. The water is currently pumped out of the area via two existing 18-cfs pumps. The proposed structures needed to address the increase of water level and duration of flooding is provided in the following paragraphs.

Gravity outlet retrofits would include one additional sluice gate structure with a 2-foot diameter outlet. Drainage swales would be constructed along the landward side of the coastal storm risk management structure to direct runoff toward the outlet structures. All outlets, including the sluice gate structure and pump outlets, that would run through the LOP would be fitted with flap gates.

Table B3-7 provides a summary of the flood levels and number of structures impacted under the Without conditions for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-8** provides a summary of the With conditions in the same format.

Table B3-7: Main Street Ditch Drainage Area Results – Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	0.44	0	0	0.44	0	0	2.36	0	0
5 yr	1.49	0	0	1.49	0	0	2.40	0	0
10 yr	2.36	0	0	2.36	0	0	2.40	0	0
25 yr	3.34	0	0	3.34	0	0	3.34	0	0
50 yr	4.06	2	1	4.06	2	1	4.06	2	1
100 yr	4.32	4	1	4.32	4	1	4.32	4	1
250 yr	4.67	9	3	4.67	9	3	4.67	9	3
500 yr	5.00	27	7	5.00	27	7	5.00	27	7
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	0.76	0	0	0.76	0	0	3.13	0	0
5 yr	2.21	0	0	2.21	0	0	3.16	0	0
10 yr	3.13	0	0	3.13	0	0	3.18	0	0
25 yr	4.23	3	1	4.23	3	1	4.24	3	1
50 yr	4.75	14	3	4.75	14	3	4.76	14	3
100 yr	5.15	46	10	5.15	46	10	5.16	48	10
250 yr	5.37	85	16	5.37	85	16	5.37	85	16
500 yr	5.59	130	21	5.59	130	21	5.59	130	21
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	0.76	0	0	0.77	0	0	3.16	0	0
5 yr	2.21	0	0	2.22	0	0	3.18	0	0
10 yr	3.13	0	0	3.16	0	0	3.19	0	0
25 yr	4.23	3	1	4.24	3	1	4.25	3	1
50 yr	4.75	14	3	4.76	14	3	4.77	15	4
100 yr	5.15	46	10	5.16	48	10	5.16	48	10
250 yr	5.37	85	16	5.37	85	16	5.38	87	16
500 yr	5.59	130	21	5.59	130	21	5.60	133	21

Table B3-8: Main Street Ditch Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	0.44	0	0	0.44	0	0	2.36	0	0
5 yr	1.49	0	0	1.49	0	0	2.36	0	0
10 yr	2.35	0	0	2.36	0	0	2.36	0	0
25 yr	3.15	0	0	3.31	0	0	3.34	0	0
50 yr	3.86	1	1	4.01	2	1	4.04	2	1
100 yr	4.14	3	1	4.18	3	1	4.26	3	1
250 yr	4.43	5	1	4.47	6	1	4.59	9	3
500 yr	4.73	12	3	4.78	15	4	4.90	21	6
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	0.76	0	0	0.76	0	0	3.13	0	0
5 yr	2.21	0	0	2.21	0	0	3.16	0	0
10 yr	3.12	0	0	3.13	0	0	3.17	0	0
25 yr	4.12	2	1	4.15	3	1	4.23	3	1
50 yr	4.54	9	3	4.60	9	3	4.71	11	3
100 yr	5.06	36	8	5.08	37	9	5.13	44	10
250 yr	5.25	58	13	5.27	60	14	5.33	76	15
500 yr	5.44	100	17	5.46	103	17	5.52	112	18
Future High SLR Conditions 10									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	0.76	0	0	0.77	0	0	3.16	0	0
5 yr	2.21	0	0	2.22	0	0	3.18	0	0
10 yr	3.13	0	0	3.16	0	0	3.19	0	0
25 yr	4.16	3	1	4.19	3	1	4.25	3	1
50 yr	4.60	9	3	4.68	9	3	4.74	12	3
100 yr	5.09	38	9	5.12	42	10	5.14	45	10
250 yr	5.28	62	14	5.31	68	14	5.34	79	16
500 yr	5.47	104	18	5.50	106	18	5.53	117	18

3.1.5 Moonachie Creek Drainage Area

Under the Without conditions, stormwater runoff from Moonachie Creek drainage area is conveyed to the Hackensack River via two 8-foot diameter tide gates.

If the proposed LOP is constructed, the tide gates would be the only means for stormwater to flow through the LOP. No additional structures would need to be added in order to reduce residual flooding. The existing tide gate structure keeps water elevations below the design criteria.

Table B3-9 provides a summary of the flood levels and number of structures impacted under the Without conditions for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions.

Table B3-10 provides a summary of the With conditions in the same format.

Table B3-9: Moonachie Creek Drainage Area Results - Without Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
Present Conditions									
2 yr	0.89	0	0	0.93	0	0	1.26	0	0
5 yr	1.11	0	0	1.14	0	0	1.33	0	0
10 yr	1.22	0	0	1.26	0	0	1.40	0	0
25 yr	1.39	0	0	1.44	0	0	1.61	0	0
50 yr	1.53	0	0	1.62	0	0	1.82	0	0
100 yr	1.64	0	0	1.77	0	0	1.98	0	0
250 yr	1.82	0	0	1.99	0	0	2.19	0	0
500 yr	1.97	0	0	2.15	0	0	2.35	0	0
Future Low SLR Conditions									
2 yr	1.01	0	0	1.10	0	0	1.48	0	0
5 yr	1.21	0	0	1.29	0	0	1.56	0	0
10 yr	1.40	0	0	1.48	0	0	1.59	0	0
25 yr	1.68	0	0	1.73	0	0	1.92	0	0
50 yr	1.96	0	0	2.02	0	0	2.29	0	0
100 yr	2.27	0	0	2.38	0	0	2.67	1	1
250 yr	2.53	0	0	2.67	1	1	2.98	2	2
500 yr	2.67	1	1	2.85	1	1	3.20	2	2
Future High SLR Conditions									
2 yr	1.09	0	0	1.11	0	0	1.56	0	0
5 yr	1.28	0	0	1.35	0	0	1.59	0	0
10 yr	1.45	0	0	1.56	0	0	1.59	0	0
25 yr	1.72	0	0	1.86	0	0	1.94	0	0
50 yr	2.01	0	0	2.16	0	0	2.29	0	0
100 yr	2.37	0	0	2.48	0	0	2.69	1	1
250 yr	2.66	1	1	2.77	1	1	3.01	2	2
500 yr	2.84	1	1	2.98	2	2	3.21	3	3

Table B3-10: Moonachie Creek Drainage Area Results - With Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
Present Conditions									
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	0.89	0	0	0.93	0	0	1.26	0	0
5 yr	1.11	0	0	1.14	0	0	1.33	0	0
10 yr	1.22	0	0	1.26	0	0	1.40	0	0
25 yr	1.39	0	0	1.44	0	0	1.61	0	0
50 yr	1.53	0	0	1.62	0	0	1.82	0	0
100 yr	1.64	0	0	1.77	0	0	1.98	0	0
250 yr	1.82	0	0	1.99	0	0	2.19	0	0
500 yr	1.97	0	0	2.15	0	0	2.35	0	0
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	1.01	0	0	1.10	0	0	1.48	0	0
5 yr	1.21	0	0	1.29	0	0	1.56	0	0
10 yr	1.40	0	0	1.48	0	0	1.60	0	0
25 yr	1.68	0	0	1.73	0	0	1.93	0	0
50 yr	1.96	0	0	2.02	0	0	2.29	0	0
100 yr	2.27	0	0	2.38	0	0	2.67	1	1
250 yr	2.53	1	1	2.67	1	1	2.99	2	2
500 yr	2.67	1	1	2.85	1	1	3.20	2	2
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	1.09	0	0	1.11	0	0	1.56	0	0
5 yr	1.28	0	0	1.35	0	0	1.59	0	0
10 yr	1.45	0	0	1.56	0	0	1.60	0	0
25 yr	1.72	0	0	1.86	0	0	1.94	0	0
50 yr	2.01	0	0	2.16	0	0	2.29	0	0
100 yr	2.37	0	0	2.48	0	0	2.69	1	1
250 yr	2.66	1	1	2.77	1	1	3.01	2	2
500 yr	2.84	1	1	2.98	2	2	3.21	3	3

3.1.6 Willow Lake Drainage Area

Under the Without conditions, the surface runoff from Willow Lake drainage area is conveyed to the Hackensack River via a pump station, with two 12-cfs pumps, which pumps stormwater out to the Hackensack River. A gravity outlet for this lake does not exist and during extreme events and stormwater will eventually overflow at elevation 5.0 feet (NAVD 88) by gravity; this is modeled as a 30-foot long spillway.

Since Willow Lake is further inland, it is not directly affected by the proposed Alternative 1 LOP. The LOP does not directly impact Willow Lake, and, therefore, no additional structures are proposed. However, Willow Lake does benefit from a slight reduction in flood levels because Main Street Ditch drainage area overflows into Willow Lake. Since Main Street Ditch would have a LOP added to it under Alternative 1, there would be less overflow from Main Street Ditch into Willow Lake under the With condition.

Table B3-11 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-12** provides a summary of the With conditions in the same format.

Table B3-11: Willow Lake Drainage Area Results - Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.02	19	4	5.02	19	4	5.02	19	4
5 yr	5.02	19	4	5.02	19	4	5.02	19	4
10 yr	5.02	19	4	5.02	19	4	5.02	19	4
25 yr	5.02	19	4	5.02	19	4	5.02	19	4
50 yr	5.27	43	8	5.27	43	8	5.27	43	8
100 yr	5.51	64	13	5.51	64	13	5.51	64	13
250 yr	5.70	79	15	5.70	79	15	5.70	79	15
500 yr	5.81	83	16	5.81	83	16	5.81	83	16
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.02	19	4	5.02	19	4	5.02	19	4
5 yr	5.02	19	4	5.02	19	4	5.02	19	4
10 yr	5.02	19	4	5.02	19	4	5.02	19	4
25 yr	5.38	55	11	5.38	55	11	5.38	55	11
50 yr	5.70	79	15	5.70	79	15	5.70	79	15
100 yr	5.84	86	18	5.84	86	18	5.84	86	18
250 yr	5.88	87	20	5.88	87	20	5.88	87	20
500 yr	5.97	91	21	5.97	91	21	5.97	91	21
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.02	19	4	5.02	19	4	5.02	19	4
5 yr	5.02	19	4	5.02	19	4	5.02	19	4
10 yr	5.02	19	4	5.02	19	4	5.02	19	4
25 yr	5.38	55	11	5.38	55	11	5.39	55	11
50 yr	5.70	79	15	5.70	79	15	5.70	79	15
100 yr	5.84	86	18	5.84	86	18	5.84	86	18
250 yr	5.88	87	20	5.88	87	20	5.88	87	20
500 yr	5.97	91	21	5.97	91	21	5.97	91	21

Table B3-12: Willow Lake Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.74	0	0	2.75	0	0	3.75	0	0
5 yr	3.07	0	0	3.22	0	0	3.89	0	0
10 yr	3.56	0	0	3.75	0	0	4.04	0	0
25 yr	4.03	0	0	4.15	0	0	4.43	0	0
50 yr	4.34	0	0	4.46	0	0	4.71	2	1
100 yr	4.63	1	1	4.79	4	1	5.03	20	4
250 yr	5.00	19	4	5.09	27	5	5.22	36	6
500 yr	5.15	32	6	5.25	40	6	5.34	50	10
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.57	0	0	3.58	0	0	4.18	0	0
5 yr	3.58	0	0	3.73	0	0	4.34	0	0
10 yr	4.04	0	0	4.18	0	0	4.45	0	0
25 yr	4.57	0	0	4.70	2	1	5.00	19	4
50 yr	5.10	28	5	5.16	32	6	5.25	40	6
100 yr	5.31	47	9	5.36	54	11	5.48	62	13
250 yr	5.41	57	12	5.48	62	13	5.62	72	15
500 yr	5.51	64	13	5.59	70	14	5.78	81	15
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.40	0	0	4.41	0	0	4.43	0	0
5 yr	4.40	0	0	4.42	0	0	4.43	0	0
10 yr	4.40	0	0	4.43	0	0	4.45	0	0
25 yr	4.71	2	1	4.89	11	2	5.01	19	4
50 yr	5.16	32	6	5.22	36	6	5.26	41	7
100 yr	5.37	54	11	5.42	59	13	5.50	64	13
250 yr	5.48	62	13	5.56	69	14	5.65	74	15
500 yr	5.60	71	15	5.70	79	15	5.80	82	16

3.1.7 HACK 1A3 Drainage Area

Under the Without conditions, the surface runoff from HACK 1A3 drainage area is conveyed to the Hackensack River via two 2-foot culverts with surface overflows modeled as a 35-foot long spillway at elevation 5.0 feet (NAVD 88).

If the proposed LOP is constructed, the two existing 2-foot diameter outlets would be the only means for stormwater runoff to flow through the LOP. The modeling results indicate, however, that the current tide gate structure keeps water elevations below the design criteria. Therefore, no additional structures would be added.

Table B3-13 provides a summary of the flood levels and number of structures impacted under the Without conditions for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-14** provides a summary of the With conditions in the same format.

Table B3-13: HACK 1A3 Drainage Area Results - Without Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
Present Conditions									
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.32	0	0	4.32	0	0	5.09	0	0
5 yr	4.45	0	0	4.45	0	0	5.09	0	0
10 yr	4.55	0	0	5.09	0	0	5.10	0	0
25 yr	4.66	0	0	5.09	0	0	5.11	0	0
50 yr	4.75	0	0	5.09	0	0	5.11	0	0
100 yr	4.83	0	0	5.09	0	0	5.12	0	0
250 yr	4.93	0	0	5.09	0	0	5.13	0	0
500 yr	5.01	0	0	5.09	0	0	5.15	0	0
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.12	0	0	5.12	0	0	5.76	0	0
5 yr	5.15	0	0	5.27	0	0	5.89	0	0
10 yr	5.17	0	0	5.76	0	0	5.89	0	0
25 yr	5.20	0	0	5.83	0	0	6.10	0	0
50 yr	5.23	0	0	6.15	0	0	6.43	0	0
100 yr	5.27	0	0	6.15	0	0	6.43	0	0
250 yr	5.30	0	0	6.15	0	0	6.43	0	0
500 yr	5.32	0	0	6.15	0	0	6.43	0	0
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.12	0	0	5.44	0	0	5.78	0	0
5 yr	5.15	0	0	5.75	0	0	6.01	0	0
10 yr	5.17	0	0	5.78	0	0	6.08	0	0
25 yr	5.20	0	0	6.14	0	0	6.22	0	0
50 yr	5.23	0	0	6.22	0	0	6.44	0	0
100 yr	5.27	0	0	6.22	0	0	6.44	0	0
250 yr	5.30	0	0	6.22	0	0	6.44	0	0
500 yr	5.32	0	0	6.22	0	0	6.44	0	0

Table B3-14: HACK 1A3 Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.32	0	0	4.32	0	0	5.11	0	0
5 yr	4.45	0	0	4.45	0	0	5.12	0	0
10 yr	4.55	0	0	5.11	0	0	5.13	0	0
25 yr	4.66	0	0	5.71	0	0	6.04	0	0
50 yr	4.75	0	0	6.17	0	0	6.38	0	0
100 yr	4.83	0	0	6.17	0	0	6.38	0	0
250 yr	4.93	0	0	6.17	0	0	6.38	0	0
500 yr	5.01	0	0	6.17	0	0	6.38	0	0
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.47	0	0	5.47	0	0	5.77	0	0
5 yr	5.59	0	0	5.59	0	0	5.93	0	0
10 yr	5.70	0	0	5.77	0	0	5.95	0	0
25 yr	5.85	0	0	6.31	0	0	6.48	0	0
50 yr	6.00	0	0	7.99	1	1	8.05	1	1
100 yr	6.09	0	0	7.99	1	1	8.05	1	1
250 yr	6.17	0	0	7.99	1	1	8.05	1	1
500 yr	6.26	0	0	7.99	1	1	8.05	1	1
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.47	0	0	5.52	0	0	5.79	0	0
5 yr	5.59	0	0	5.87	0	0	6.02	0	0
10 yr	5.70	0	0	5.87	0	0	6.10	0	0
25 yr	5.85	0	0	7.29	1	1	7.34	1	1
50 yr	6.00	0	0	8.36	1	1	8.44	1	1
100 yr	6.09	0	0	8.36	1	1	8.44	1	1
250 yr	6.17	0	0	8.36	1	1	8.44	1	1
500 yr	6.26	0	0	8.36	1	1	8.44	1	1

3.1.8 HACK 1 Drainage Area

Under the Without conditions, the surface runoff from HACK 1 drainage area is conveyed to the Hackensack River through three 2-foot diameter outlets, with potential surface overflow at elevation 5.5 feet (NAVD 88) which is modeled with a 20-foot long spillway.

The current outlet structures keep water elevations below the design criteria and, therefore, no additional structures are proposed. **Table B3-15** provides a summary of the flood levels and number of structures impacted under the Without conditions for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-16** provides a summary of the With conditions in the same format.

Table B3-15: HACK 1 Drainage Area Results - Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.51	0	0	3.76	0	0	5.14	0	0
5 yr	3.76	0	0	4.27	0	0	5.18	0	0
10 yr	3.93	0	0	5.14	0	0	5.22	0	0
25 yr	4.10	0	0	5.78	1	0	6.13	1	0
50 yr	4.27	0	0	6.16	1	0	6.43	1	0
100 yr	4.45	0	0	6.16	1	0	6.43	1	0
250 yr	4.71	0	0	6.16	1	0	6.43	1	0
500 yr	4.94	0	0	6.16	1	0	6.43	1	0
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.57	0	0	4.59	0	0	5.88	1	0
5 yr	4.83	0	0	5.34	0	0	5.92	1	0
10 yr	5.05	0	0	5.88	1	0	5.95	1	0
25 yr	5.33	0	0	6.02	1	0	6.22	1	0
50 yr	5.61	0	0	6.24	1	0	6.67	2	0
100 yr	5.84	1	0	6.24	1	0	6.67	2	0
250 yr	5.97	1	0	6.24	1	0	6.67	2	0
500 yr	6.05	1	0	6.24	1	0	6.67	2	0
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.68	0	0	5.51	0	0	5.89	1	0
5 yr	4.83	0	0	5.87	1	0	6.09	1	0
10 yr	5.05	0	0	5.89	1	0	6.19	1	0
25 yr	5.34	0	0	6.22	1	0	6.38	1	0
50 yr	5.62	0	0	6.31	1	0	6.69	2	0
100 yr	5.84	1	0	6.31	1	0	6.69	2	0
250 yr	5.97	1	0	6.31	1	0	6.69	2	0
500 yr	6.05	1	0	6.31	1	0	6.69	2	0

Table B3-16: HACK 1 Drainage Area Results - With Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
Present Conditions									
2 yr	3.51	0	0	3.76	0	0	5.14	0	0
5 yr	3.76	0	0	4.27	0	0	5.18	0	0
10 yr	3.93	0	0	5.14	0	0	5.22	0	0
25 yr	4.10	0	0	5.79	1	0	6.13	1	0
50 yr	4.27	0	0	6.21	1	0	6.45	1	0
100 yr	4.45	0	0	6.21	1	0	6.45	1	0
250 yr	4.71	0	0	6.21	1	0	6.45	1	0
500 yr	4.94	0	0	6.21	1	0	6.45	1	0
Future Low SLR Conditions									
2 yr	4.57	0	0	4.59	0	0	5.88	1	0
5 yr	4.83	0	0	5.35	0	0	5.92	1	0
10 yr	5.05	0	0	5.88	1	0	5.95	1	0
25 yr	5.33	0	0	6.02	1	0	6.23	1	0
50 yr	5.62	0	0	6.31	1	0	6.75	2	0
100 yr	5.84	1	0	6.31	1	0	6.75	2	0
250 yr	5.97	1	0	6.31	1	0	6.75	2	0
500 yr	6.05	1	0	6.31	1	0	6.75	2	0
Future High SLR Conditions									
2 yr	4.68	0	0	5.51	0	0	5.90	1	0
5 yr	4.83	0	0	5.87	1	0	6.09	1	0
10 yr	5.06	0	0	5.90	1	0	6.20	1	0
25 yr	5.34	0	0	6.25	1	0	6.39	1	0
50 yr	5.62	0	0	6.34	1	0	6.76	2	0
100 yr	5.84	1	0	6.34	1	0	6.76	2	0
250 yr	5.97	1	0	6.34	1	0	6.76	2	0
500 yr	6.05	1	0	6.34	1	0	6.76	2	0

3.1.9 HACK 2 Drainage Area

Under the Without conditions, the surface runoff from HACK 2 drainage area is conveyed to the Hackensack River via four 3-foot diameter culverts. Surface overflows can occur at elevation 6.5 feet (NAVD 88) and are modeled as a 60-foot long spillway.

If the proposed LOP is constructed, the current outlet structures would be the only means for runoff to flow through the LOP. The current outlet structures maintain water elevations below the design criteria and, therefore, no additional structures are proposed.

Table B3-17 provides a summary of the flood levels and number of structures impacted under the Without conditions for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-18** provides a summary of the With conditions in the same format.

Table B3-17: HACK 2 Drainage Area Results - Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.73	0	0	3.75	0	0	5.02	0	0
5 yr	3.74	0	0	4.20	0	0	5.09	0	0
10 yr	3.83	0	0	5.02	0	0	5.10	0	0
25 yr	3.94	0	0	5.25	0	0	5.50	0	0
50 yr	3.99	0	0	5.27	0	0	5.54	0	0
100 yr	4.07	0	0	5.27	0	0	5.54	0	0
250 yr	4.16	0	0	5.27	0	0	5.54	0	0
500 yr	4.25	0	0	5.27	0	0	5.54	0	0
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.05	0	0	5.05	0	0	5.33	0	0
5 yr	5.14	0	0	5.18	0	0	5.44	0	0
10 yr	5.22	0	0	5.33	0	0	5.53	0	0
25 yr	5.34	0	0	5.41	0	0	5.77	0	0
50 yr	5.47	0	0	5.51	0	0	5.87	0	0
100 yr	5.63	0	0	5.63	0	0	5.91	0	0
250 yr	5.75	0	0	5.75	0	0	6.00	0	0
500 yr	5.86	0	0	5.86	0	0	6.07	0	0
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.05	0	0	5.21	0	0	5.41	0	0
5 yr	5.14	0	0	5.32	0	0	5.54	0	0
10 yr	5.22	0	0	5.41	0	0	5.63	0	0
25 yr	5.34	0	0	5.52	0	0	5.78	0	0
50 yr	5.47	0	0	5.63	0	0	5.97	0	0
100 yr	5.63	0	0	5.76	0	0	6.02	0	0
250 yr	5.75	0	0	5.85	0	0	6.10	0	0
500 yr	5.86	0	0	5.94	0	0	6.18	0	0

Table B3-18. HACK 2 Drainage Area Results - With Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
Present Conditions									
2 yr	3.66	0	0	3.75	0	0	5.02	0	0
5 yr	3.80	0	0	4.20	0	0	5.09	0	0
10 yr	3.80	0	0	5.02	0	0	5.09	0	0
25 yr	3.85	0	0	5.26	0	0	5.50	0	0
50 yr	3.92	0	0	5.40	0	0	5.66	0	0
100 yr	4.00	0	0	5.40	0	0	5.66	0	0
250 yr	4.05	0	0	5.40	0	0	5.66	0	0
500 yr	4.13	0	0	5.40	0	0	5.66	0	0
Future Low SLR Conditions									
2 yr	5.27	0	0	5.27	0	0	5.45	0	0
5 yr	5.37	0	0	5.37	0	0	5.54	0	0
10 yr	5.45	0	0	5.45	0	0	5.63	0	0
25 yr	5.58	0	0	5.58	0	0	5.75	0	0
50 yr	5.70	0	0	5.83	0	0	6.06	0	0
100 yr	5.84	0	0	5.84	0	0	6.06	0	0
250 yr	5.94	0	0	5.94	0	0	6.06	0	0
500 yr	6.02	0	0	6.02	0	0	6.10	0	0
Future High SLR Conditions									
2 yr	5.27	0	0	5.35	0	0	5.54	0	0
5 yr	5.37	0	0	5.45	0	0	5.62	0	0
10 yr	5.45	0	0	5.54	0	0	5.70	0	0
25 yr	5.58	0	0	5.65	0	0	5.99	0	0
50 yr	5.70	0	0	5.88	0	0	6.19	0	0
100 yr	5.84	0	0	5.88	0	0	6.19	0	0
250 yr	5.94	0	0	5.96	0	0	6.19	0	0
500 yr	6.02	0	0	6.03	0	0	6.26	1	0

3.1.10 HACK 3 Drainage Area

Under the Without conditions, the surface runoff from HACK 3 drainage area is conveyed to the Hackensack River without any restrictions. There are no existing outlets in this area, and stormwater flows out to the Hackensack River via overland flows modeled as a 65-foot long overflow spillway at elevation 5.0 feet (NAVD 88).

The proposed improvements incorporated under Alternative 1 include the construction of a sluice gate structure with a 2-foot diameter outlet fitted with a flap gate. Drainage swales would be constructed along the landward side of proposed LOP to direct runoff toward the sluice gate structure.

Table B3-19 provides a summary of the flood levels and number of structures impacted under the Without conditions for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-20** provides a summary of the With conditions in the same format.

Table B3-19: HACK 3 Drainage Area Results - Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.03	0	0	5.03	0	0	5.08	0	0
5 yr	5.04	0	0	5.22	0	0	5.23	0	0
10 yr	5.04	0	0	5.22	0	0	5.23	0	0
25 yr	5.05	0	0	5.22	0	0	5.23	0	0
50 yr	5.05	0	0	5.22	0	0	5.23	0	0
100 yr	5.05	0	0	5.22	0	0	5.23	0	0
250 yr	5.06	0	0	5.22	0	0	5.23	0	0
500 yr	5.06	0	0	5.22	0	0	5.23	0	0

Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.03	0	0	5.03	0	0	5.22	0	0
5 yr	5.04	0	0	5.22	0	0	5.23	0	0
10 yr	5.04	0	0	5.22	0	0	5.23	0	0
25 yr	5.05	0	0	5.22	0	0	5.23	0	0
50 yr	5.06	0	0	5.22	0	0	5.23	0	0
100 yr	5.07	0	0	5.22	0	0	5.23	0	0
250 yr	5.07	0	0	5.22	0	0	5.24	0	0
500 yr	5.08	0	0	5.22	0	0	5.24	0	0

Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.03	0	0	5.22	0	0	5.22	0	0
5 yr	5.04	0	0	5.22	0	0	5.23	0	0
10 yr	5.04	0	0	5.22	0	0	5.23	0	0
25 yr	5.05	0	0	5.22	0	0	5.23	0	0
50 yr	5.06	0	0	5.23	0	0	5.23	0	0
100 yr	5.07	0	0	5.23	0	0	5.23	0	0
250 yr	5.07	0	0	5.24	0	0	5.24	0	0
500 yr	5.08	0	0	5.24	0	0	5.24	0	0

Table B3-20: HACK 3 Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.66	0	0	3.75	0	0	5.08	0	0
5 yr	3.73	0	0	4.19	0	0	5.08	0	0
10 yr	3.77	0	0	5.08	0	0	5.08	0	0
25 yr	3.82	0	0	5.22	0	0	5.22	0	0
50 yr	3.86	0	0	5.22	0	0	5.23	0	0
100 yr	3.90	0	0	5.22	0	0	5.23	0	0
250 yr	3.95	0	0	5.22	0	0	5.23	0	0
500 yr	3.99	0	0	5.22	0	0	5.23	0	0
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.02	0	0	5.02	0	0	5.22	0	0
5 yr	5.03	0	0	5.22	0	0	5.23	0	0
10 yr	5.04	0	0	5.22	0	0	5.23	0	0
25 yr	5.04	0	0	5.22	0	0	5.23	0	0
50 yr	5.05	0	0	5.22	0	0	5.23	0	0
100 yr	5.06	0	0	5.22	0	0	5.23	0	0
250 yr	5.06	0	0	5.22	0	0	5.24	0	0
500 yr	5.07	0	0	5.22	0	0	5.24	0	0
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.02	0	0	5.22	0	0	5.22	0	0
5 yr	5.03	0	0	5.22	0	0	5.23	0	0
10 yr	5.04	0	0	5.22	0	0	5.23	0	0
25 yr	5.04	0	0	5.23	0	0	5.23	0	0
50 yr	5.05	0	0	5.23	0	0	5.23	0	0
100 yr	5.06	0	0	5.23	0	0	5.23	0	0
250 yr	5.06	0	0	5.24	0	0	5.24	0	0
500 yr	5.07	0	0	5.24	0	0	5.24	0	0

3.1.11 HACK 4 Drainage Area

Under the Without conditions, the surface runoff from HACK 4 is conveyed to the Hackensack River via two existing 2-foot diameter outlets. Surface overflows from this area are modeled as a 47-foot long spillway at elevation 4.0 feet (NAVD 88).

If the proposed LOP is constructed, the two existing outlet structures would be the only means for runoff to flow through the LOP, however, the existing outlet structures keep water elevations below the design criteria and, therefore, no additional structures would be required.

Table B3-21 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-22** provides a summary of the With conditions in the same format.

Table B3-21: HACK 4 Drainage Area Results - Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.06	0	0	4.06	0	0	5.08	0	0
5 yr	4.10	0	0	4.18	0	0	5.08	0	0
10 yr	4.13	0	0	5.08	0	0	5.08	0	0
25 yr	4.16	0	0	5.40	1	0	5.47	1	0
50 yr	4.19	0	0	5.41	1	0	5.80	2	0
100 yr	4.21	0	0	5.41	1	0	5.80	2	0
250 yr	4.24	0	0	5.41	1	0	5.80	2	0
500 yr	4.27	0	0	5.41	1	0	5.80	2	0
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.15	0	0	4.46	0	0	5.19	0	0
5 yr	4.19	0	0	5.17	0	0	5.34	1	0
10 yr	4.22	0	0	5.19	0	0	5.48	1	0
25 yr	4.27	0	0	5.48	1	0	5.83	3	1
50 yr	4.31	0	0	5.62	1	0	6.00	5	2
100 yr	4.36	0	0	5.62	1	0	6.00	5	2
250 yr	4.39	0	0	5.62	1	0	6.00	5	2
500 yr	4.42	0	0	5.62	1	0	6.00	5	2
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.58	0	0	5.18	0	0	5.44	1	0
5 yr	4.59	0	0	5.32	1	0	5.47	1	0
10 yr	4.66	0	0	5.44	1	0	5.49	1	0
25 yr	4.66	0	0	5.48	1	0	5.99	5	2
50 yr	4.66	0	0	5.62	1	0	6.09	5	2
100 yr	4.66	0	0	5.62	1	0	6.11	5	2
250 yr	4.66	0	0	5.62	1	0	6.19	5	2
500 yr	4.68	0	0	5.62	1	0	6.20	5	2

Table B3-22: HACK 4 Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.02	0	0	3.75	0	0	5.10	0	0
5 yr	3.16	0	0	4.22	0	0	5.11	0	0
10 yr	3.27	0	0	5.10	0	0	5.12	0	0
25 yr	3.39	0	0	5.45	1	0	5.63	1	0
50 yr	3.49	0	0	6.08	5	2	6.23	5	2
100 yr	3.58	0	0	6.08	5	2	6.23	5	2
250 yr	3.70	0	0	6.08	5	2	6.23	5	2
500 yr	3.80	0	0	6.08	5	2	6.23	5	2
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.35	1	0	5.35	1	0	5.62	1	0
5 yr	5.49	1	0	5.49	1	0	5.66	2	0
10 yr	5.62	1	0	5.62	1	0	5.78	2	0
25 yr	5.80	2	0	5.80	2	0	6.05	5	2
50 yr	5.96	5	2	6.35	6	3	6.56	9	4
100 yr	6.07	5	2	6.35	6	3	6.56	9	4
250 yr	6.15	5	2	6.35	6	3	6.56	9	4
500 yr	6.22	5	2	6.35	6	3	6.56	9	4
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.35	1	0	5.43	1	0	5.68	2	0
5 yr	5.50	1	0	5.57	1	0	5.78	2	0
10 yr	5.62	1	0	5.68	2	0	5.86	4	2
25 yr	5.80	2	0	6.16	5	2	6.33	6	3
50 yr	5.96	5	2	6.44	8	4	6.61	9	4
100 yr	6.07	5	2	6.44	8	4	6.61	9	4
250 yr	6.15	5	2	6.44	8	4	6.61	9	4
500 yr	6.22	5	2	6.44	8	4	6.61	9	4

3.1.12 HACK 5 Drainage Area

Under the Without conditions, the surface runoff from HACK 5 is conveyed to the Hackensack River via three existing 2-foot diameter outlets, and overflow via a 197-foot long spillway at elevation 4.0 feet (NAVD 88).

If the proposed LOP is constructed, the three existing outlet structures would be the only means for stormwater runoff to flow through the LOP, which would lead to an increase in interior water ponding elevations. Proposed improvements to mitigate an increase in water levels include the construction of a sluice gate structure with a 2-foot diameter outlet fitted with a flap gate. Drainage swales would be constructed along the landward side of the LOP to direct runoff toward the structure.

Table B3-23 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-24** provides a summary of the With conditions in the same format.

Table B3-23. HACK 5 Drainage Area Results - Without Alternative 1

Event	Lower Bound Interior WSEL ft. NAVD88	Lower Bound Structures Flooded Above Ground	Lower Bound Structures Flooded Above Main Fl.	Most Likely Interior WSEL ft. NAVD88	Most Likely Structures Flooded Above Ground	Most Likely Structures Flooded Above Main Fl.	Upper Bound Interior WSEL ft. NAVD88	Upper Bound Structures Flooded Above Ground	Upper Bound Structures Flooded Above Main Fl.
Present Conditions									
2 yr	4.01	0	0	4.01	0	0	5.08	0	0
5 yr	4.03	0	0	4.21	0	0	5.08	0	0
10 yr	4.04	0	0	5.08	0	0	5.08	0	0
25 yr	4.06	0	0	5.47	0	0	5.52	0	0
50 yr	4.07	0	0	5.62	0	0	6.03	0	0
100 yr	4.07	0	0	5.62	0	0	6.03	0	0
250 yr	4.09	0	0	5.62	0	0	6.03	0	0
500 yr	4.10	0	0	5.62	0	0	6.03	0	0
Future Low SLR Conditions									
2 yr	4.06	0	0	4.46	0	0	5.31	0	0
5 yr	4.08	0	0	5.27	0	0	5.49	0	0
10 yr	4.09	0	0	5.31	0	0	5.64	0	0
25 yr	4.10	0	0	5.69	0	0	5.89	0	0
50 yr	4.12	0	0	5.72	0	0	6.15	0	0
100 yr	4.14	0	0	5.72	0	0	6.15	0	0
250 yr	4.15	0	0	5.72	0	0	6.15	0	0
500 yr	4.16	0	0	5.72	0	0	6.15	0	0
Future High SLR Conditions									
2 yr	4.58	0	0	5.28	0	0	5.44	0	0
5 yr	4.66	0	0	5.43	0	0	5.61	0	0
10 yr	4.66	0	0	5.44	0	0	5.64	0	0
25 yr	4.66	0	0	5.70	0	0	6.12	0	0
50 yr	4.66	0	0	5.89	0	0	6.22	0	0
100 yr	4.66	0	0	5.89	0	0	6.22	0	0
250 yr	4.66	0	0	5.89	0	0	6.22	0	0
500 yr	4.69	0	0	5.89	0	0	6.22	0	0

Table B3-24: HACK 5 Drainage Area Results - With Alternative 1

Event	Lower Bound Interior WSEL ft. NAVD88	Lower Bound Structures Flooded Above Ground	Lower Bound Structures Flooded Above Main Fl.	Most Likely Interior WSEL ft. NAVD88	Most Likely Structures Flooded Above Ground	Most Likely Structures Flooded Above Main Fl.	Upper Bound Interior WSEL ft. NAVD88	Upper Bound Structures Flooded Above Ground	Upper Bound Structures Flooded Above Main Fl.
Present Conditions									
2 yr	4.05	0	0	4.05	0	0	5.10	0	0
5 yr	4.17	0	0	4.23	0	0	5.11	0	0
10 yr	4.26	0	0	5.10	0	0	5.12	0	0
25 yr	4.37	0	0	5.55	0	0	5.60	0	0
50 yr	4.45	0	0	6.33	0	0	6.48	1	1
100 yr	4.53	0	0	6.33	0	0	6.48	1	1
250 yr	4.62	0	0	6.33	0	0	6.48	1	1
500 yr	4.70	0	0	6.33	0	0	6.48	1	1
Future Low SLR Conditions									
2 yr	4.97	0	0	4.97	0	0	5.40	0	0
5 yr	5.06	0	0	5.30	0	0	5.57	0	0
10 yr	5.15	0	0	5.40	0	0	5.73	0	0
25 yr	5.28	0	0	5.90	0	0	6.05	0	0
50 yr	5.40	0	0	6.73	1	1	7.00	2	2
100 yr	5.55	0	0	6.73	1	1	7.00	2	2
250 yr	5.66	0	0	6.73	1	1	7.00	2	2
500 yr	5.76	0	0	6.73	1	1	7.00	2	2
Future High SLR Conditions									
2 yr	4.97	0	0	5.35	0	0	5.55	0	0
5 yr	5.07	0	0	5.47	0	0	5.71	0	0
10 yr	5.15	0	0	5.55	0	0	5.75	0	0
25 yr	5.28	0	0	6.34	1	1	6.45	1	1
50 yr	5.40	0	0	6.77	2	2	7.04	3	3
100 yr	5.55	0	0	6.77	2	2	7.04	3	3
250 yr	5.66	0	0	6.77	2	2	7.04	3	3
500 yr	5.76	0	0	6.77	2	2	7.04	3	3

3.1.13 HACK 6 Drainage Area

Under the Without conditions, the surface runoff from HACK 6 is conveyed to the Hackensack River only via overland flows which are modeled as a 190-foot long spillway at elevation 5.0 feet (NAVD 88). There are no existing pipe outlets in this area.

With the Alternative 1 in place, the LOP would block the surface overland flows and stormwater would drain via proposed outlets to the Hackensack River. Proposed improvements include six sluice gate structures each with 2-foot diameter outlets, fitted flap gates, draining to the river. Drainage swales would be constructed along the landward side of the LOP to direct runoff toward the structures.

Table B3-25 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-26** provides a summary of the With conditions in the same format.

Table B3-25: HACK 6 Drainage Area Results - Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.07	2	2	5.07	2	2	5.10	2	2
5 yr	5.08	2	2	5.08	2	2	5.10	2	2
10 yr	5.10	2	2	5.10	2	2	5.10	2	2
25 yr	5.11	2	2	5.24	2	2	5.41	2	2
50 yr	5.12	2	2	5.26	2	2	5.42	2	2
100 yr	5.13	2	2	5.26	2	2	5.42	2	2
250 yr	5.14	2	2	5.26	2	2	5.42	2	2
500 yr	5.16	2	2	5.26	2	2	5.42	2	2
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.07	2	2	5.07	2	2	5.19	2	2
5 yr	5.09	2	2	5.14	2	2	5.25	2	2
10 yr	5.11	2	2	5.19	2	2	5.27	2	2
25 yr	5.13	2	2	5.28	2	2	5.50	2	2
50 yr	5.15	2	2	5.35	2	2	5.63	2	2
100 yr	5.17	2	2	5.35	2	2	5.63	2	2
250 yr	5.19	2	2	5.35	2	2	5.68	3	3
500 yr	5.20	2	2	5.35	2	2	5.78	3	3
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.07	2	2	5.14	2	2	5.27	2	2
5 yr	5.09	2	2	5.20	2	2	5.39	2	2
10 yr	5.11	2	2	5.27	2	2	5.40	2	2
25 yr	5.13	2	2	5.34	2	2	5.51	2	2
50 yr	5.15	2	2	5.42	2	2	5.63	2	2
100 yr	5.17	2	2	5.44	2	2	5.63	2	2
250 yr	5.19	2	2	5.44	2	2	5.71	3	3
500 yr	5.20	2	2	5.44	2	2	5.81	3	3

Table B3-26: HACK 6 Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.86	1	1	3.86	1	1	4.47	1	1
5 yr	3.98	1	1	4.20	1	1	4.66	1	1
10 yr	4.05	1	1	4.47	1	1	4.81	1	1
25 yr	4.15	1	1	4.93	2	2	5.15	2	2
50 yr	4.23	1	1	5.15	2	2	5.28	2	2
100 yr	4.30	1	1	5.15	2	2	5.28	2	2
250 yr	4.39	1	1	5.15	2	2	5.28	2	2
500 yr	4.46	1	1	5.15	2	2	5.28	2	2
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.35	1	1	4.47	1	1	4.62	1	1
5 yr	4.48	1	1	4.68	1	1	4.99	2	2
10 yr	4.59	1	1	4.68	1	1	4.99	2	2
25 yr	4.74	1	1	5.03	2	2	5.25	2	2
50 yr	4.87	2	2	5.29	2	2	5.56	2	2
100 yr	5.01	2	2	5.29	2	2	5.56	2	2
250 yr	5.06	2	2	5.29	2	2	5.56	2	2
500 yr	5.12	2	2	5.29	2	2	5.56	2	2
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Gro1und	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.49	1	1	4.76	1	1	5.01	2	2
5 yr	4.62	1	1	5.01	2	2	5.18	2	2
10 yr	4.65	1	1	5.01	2	2	5.23	2	2
25 yr	4.76	1	1	5.12	2	2	5.40	2	2
50 yr	4.88	2	2	5.30	2	2	5.58	2	2
100 yr	5.01	2	2	5.36	2	2	5.65	3	3
250 yr	5.08	2	2	5.46	2	2	5.71	3	3
500 yr	5.14	2	2	5.50	2	2	5.78	3	3

3.1.14 HACK 7 Drainage Area

Under the Without conditions, the surface runoff from HACK 7 is conveyed to the Hackensack River through one 3-foot diameter existing outlet. Overflows from this area begin at elevation 5.0 feet (NAVD 88) and are modeled as a 250-foot long overflow spillway.

If the proposed LOP is constructed, the current outlet structure would be the only means for stormwater runoff to pass through the LOP, which would lead to an increase in interior water levels if no improvements are constructed. Proposed improvements include three additional sluice gate structures each with 3-foot diameter outlets fitted with flap gates. Drainage swales would be constructed along the landward side of the coastal line of protection to direct runoff toward the three structures.

Table B3-27 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-28** provides a summary of the With conditions in the same format.

Table B3-27:. HACK 7 Drainage Area Results - Without Alternative 1

Present Conditions

Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.12	0	0	3.75	0	0	5.04	0	0
5 yr	3.27	0	0	4.19	0	0	5.08	0	0
10 yr	3.37	0	0	5.04	0	0	5.08	0	0
25 yr	3.50	0	0	5.29	0	0	5.47	0	0
50 yr	3.60	0	0	5.30	0	0	5.65	0	0
100 yr	3.70	0	0	5.30	0	0	5.65	0	0
250 yr	3.81	0	0	5.30	0	0	5.65	0	0
500 yr	3.91	0	0	5.30	0	0	5.65	0	0

Future Low SLR Conditions

Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.03	0	0	5.03	0	0	5.28	0	0
5 yr	5.04	0	0	5.26	0	0	5.42	0	0
10 yr	5.05	0	0	5.28	0	0	5.53	0	0
25 yr	5.06	0	0	5.54	0	0	5.83	0	0
50 yr	5.07	0	0	5.66	0	0	6.02	0	0
100 yr	5.08	0	0	5.66	0	0	6.02	0	0
250 yr	5.09	0	0	5.66	0	0	6.02	0	0
500 yr	5.09	0	0	5.66	0	0	6.02	0	0

Future High SLR Conditions

Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.03	0	0	5.27	0	0	5.44	0	0
5 yr	5.04	0	0	5.38	0	0	5.53	0	0
10 yr	5.05	0	0	5.44	0	0	5.53	0	0
25 yr	5.06	0	0	5.54	0	0	6.01	0	0
50 yr	5.07	0	0	5.66	0	0	6.13	0	0
100 yr	5.08	0	0	5.66	0	0	6.13	0	0
250 yr	5.09	0	0	5.66	0	0	6.17	0	0
500 yr	5.09	0	0	5.66	0	0	6.20	0	0

Table B3-28: HACK 7 Drainage Area Results - With Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
Present Conditions									
2 yr	3.17	0	0	3.75	0	0	5.08	0	0
5 yr	3.24	0	0	4.18	0	0	5.08	0	0
10 yr	3.28	0	0	5.08	0	0	5.08	0	0
25 yr	3.34	0	0	5.48	0	0	5.49	0	0
50 yr	3.39	0	0	6.40	0	0	6.45	0	0
100 yr	3.43	0	0	6.40	0	0	6.45	0	0
250 yr	3.49	0	0	6.40	0	0	6.45	0	0
500 yr	3.53	0	0	6.40	0	0	6.45	0	0
Future Low SLR Conditions									
2 yr	5.16	0	0	5.16	0	0	5.30	0	0
5 yr	5.24	0	0	5.28	0	0	5.34	0	0
10 yr	5.30	0	0	5.30	0	0	5.34	0	0
25 yr	5.38	0	0	5.89	0	0	6.02	0	0
50 yr	5.46	0	0	7.04	2	2	7.04	2	2
100 yr	5.55	0	0	7.04	2	2	7.04	2	2
250 yr	5.61	0	0	7.04	2	2	7.04	2	2
500 yr	5.66	0	0	7.04	2	2	7.04	2	2
Future High SLR Conditions									
2 yr	5.16	0	0	5.31	0	0	5.53	0	0
5 yr	5.24	0	0	5.42	0	0	5.63	0	0
10 yr	5.30	0	0	5.53	0	0	5.74	0	0
25 yr	5.38	0	0	6.44	0	0	6.58	1	1
50 yr	5.46	0	0	7.44	1	1	7.59	3	3
100 yr	5.55	0	0	7.44	1	1	7.59	3	3
250 yr	5.61	0	0	7.44	1	1	7.59	3	3
500 yr	5.66	0	0	7.44	1	1	7.59	3	3

3.1.15 HACK 8 Drainage Area

Under the Without conditions, the surface runoff from HACK 8 is conveyed to the Hackensack River through an existing 3-foot diameter outlet structure. Overland surface overflows can occur beginning at elevation 5.0 feet (NAVD 88) over a width of 210-foot.

With Alternative 1 in place, the LOP would block the surface overflows and the existing 3-foot diameter outlet would not have sufficient capacity to meet design conditions and improvements are required. Proposed improvements include two additional sluice gate structures each with 3-foot diameter outlets fitted with flap gates. Drainage swales would be constructed along the landward side of the coastal storm risk management structure to direct runoff toward the two structures.

Table B3-29 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-30** provides a summary of the With conditions in the same format.

Table B3-29: HACK 8 Drainage Area Results - Without Alternative 1

Event	Lower Bound Interior WSEL ft. NAVD88	Lower Bound Structures Flooded Above Ground	Lower Bound Structures Flooded Above Main Fl.	Most Likely Interior WSEL ft. NAVD88	Most Likely Structures Flooded Above Ground	Most Likely Structures Flooded Above Main Fl.	Upper Bound Interior WSEL ft. NAVD88	Upper Bound Structures Flooded Above Ground	Upper Bound Structures Flooded Above Main Fl.
Present Conditions									
2 yr	3.01	0	0	3.75	0	0	5.06	0	0
5 yr	3.12	0	0	4.18	0	0	5.08	0	0
10 yr	3.20	0	0	5.06	0	0	5.08	0	0
25 yr	3.20	0	0	5.29	0	0	5.47	0	0
50 yr	3.20	0	0	5.30	0	0	5.60	0	0
100 yr	3.22	0	0	5.30	0	0	5.60	0	0
250 yr	3.29	0	0	5.30	0	0	5.60	0	0
500 yr	3.34	0	0	5.30	0	0	5.60	0	0
Future Low SLR Conditions									
2 yr	5.02	0	0	5.02	0	0	5.26	0	0
5 yr	5.03	0	0	5.23	0	0	5.38	0	0
10 yr	5.03	0	0	5.26	0	0	5.48	0	0
25 yr	5.04	0	0	5.51	0	0	5.83	0	0
50 yr	5.05	0	0	5.62	0	0	5.92	0	0
100 yr	5.05	0	0	5.62	0	0	5.92	0	0
250 yr	5.06	0	0	5.62	0	0	5.92	0	0
500 yr	5.06	0	0	5.62	0	0	5.92	0	0
Future High SLR Conditions									
2 yr	5.02	0	0	5.24	0	0	5.44	0	0
5 yr	5.03	0	0	5.35	0	0	5.48	0	0
10 yr	5.03	0	0	5.44	0	0	5.48	0	0
25 yr	5.04	0	0	5.51	0	0	5.91	0	0
50 yr	5.05	0	0	5.62	0	0	6.06	0	0
100 yr	5.05	0	0	5.62	0	0	6.06	0	0
250 yr	5.06	0	0	5.62	0	0	6.08	0	0
500 yr	5.06	0	0	5.62	0	0	6.18	0	0

Table B3-30: HACK 8 Drainage Area Results - With Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
Present Conditions									
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.01	0	0	3.75	0	0	5.06	0	0
5 yr	3.02	0	0	4.18	0	0	5.08	0	0
10 yr	3.07	0	0	5.06	0	0	5.08	0	0
25 yr	3.07	0	0	5.47	0	0	5.63	0	0
50 yr	3.07	0	0	7.09	0	0	7.09	0	0
100 yr	3.08	0	0	7.09	0	0	7.09	0	0
250 yr	3.12	0	0	7.09	0	0	7.09	0	0
500 yr	3.15	0	0	7.09	0	0	7.09	0	0
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.02	0	0	5.03	0	0	5.12	0	0
5 yr	5.07	0	0	5.24	0	0	5.29	0	0
10 yr	5.11	0	0	5.24	0	0	5.29	0	0
25 yr	5.17	0	0	6.25	0	0	6.25	0	0
50 yr	5.22	0	0	7.76	0	0	7.76	0	0
100 yr	5.29	0	0	7.76	0	0	7.76	0	0
250 yr	5.33	0	0	7.76	0	0	7.76	0	0
500 yr	5.36	0	0	7.76	0	0	7.76	0	0
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	5.03	0	0	5.25	0	0	5.61	0	0
5 yr	5.07	0	0	5.43	0	0	5.72	0	0
10 yr	5.12	0	0	5.61	0	0	5.83	0	0
25 yr	5.17	0	0	7.04	0	0	7.04	0	0
50 yr	5.23	0	0	7.97	1	1	8.05	1	1
100 yr	5.29	0	0	7.97	1	1	8.05	1	1
250 yr	5.33	0	0	7.97	1	1	8.05	1	1
500 yr	5.36	0	0	7.97	1	1	8.05	1	1

3.1.16 HACK 9 Drainage Area

Under the Without conditions, there are no existing subsurface outlets and the surface runoff from HACK 9 is conveyed to the Hackensack River via overland flows. Overland flow begins at an elevation of 3.0 feet (NAVD 88) over a width of 105 feet and is modeled as a spillway.

With Alternative 1 in place, the LOP would block the overflow. Proposed improvements include the addition of one sluice gate structure with one 2-foot diameter outlet fitted with a flap gate. Drainage swales would be constructed along the landward side of the coastal storm risk management structure to direct runoff toward the structure.

Table B3-31 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-32** provides a summary of the With conditions in the same format.

Table B3-31: HACK 9 Drainage Area Results - Without Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
Present Conditions									
2 yr	3.07	NA	NA	3.15	NA	NA	3.26	NA	NA
5 yr	3.08	NA	NA	3.25	NA	NA	3.38	NA	NA
10 yr	3.09	NA	NA	3.26	NA	NA	3.40	NA	NA
25 yr	3.11	NA	NA	3.31	NA	NA	3.48	NA	NA
50 yr	3.12	NA	NA	3.34	NA	NA	3.55	NA	NA
100 yr	3.12	NA	NA	3.34	NA	NA	3.62	NA	NA
250 yr	3.14	NA	NA	3.39	NA	NA	3.71	NA	NA
500 yr	3.15	NA	NA	3.44	NA	NA	3.80	NA	NA
Future Low SLR Conditions									
2 yr	3.50	NA	NA	3.50	NA	NA	3.50	NA	NA
5 yr	3.50	NA	NA	3.50	NA	NA	3.50	NA	NA
10 yr	3.50	NA	NA	3.50	NA	NA	3.54	NA	NA
25 yr	3.50	NA	NA	3.57	NA	NA	3.70	NA	NA
50 yr	3.50	NA	NA	3.70	NA	NA	3.86	NA	NA
100 yr	3.50	NA	NA	3.72	NA	NA	3.92	NA	NA
250 yr	3.56	NA	NA	3.72	NA	NA	4.05	NA	NA
500 yr	3.63	NA	NA	3.72	NA	NA	4.18	NA	NA
Future High SLR Conditions									
2 yr	4.00	NA	NA	4.00	NA	NA	4.00	NA	NA
5 yr	4.00	NA	NA	4.00	NA	NA	4.00	NA	NA
10 yr	4.00	NA	NA	4.00	NA	NA	4.00	NA	NA
25 yr	4.00	NA	NA	4.00	NA	NA	4.00	NA	NA
50 yr	4.00	NA	NA	4.00	NA	NA	4.00	NA	NA
100 yr	4.00	NA	NA	4.00	NA	NA	4.00	NA	NA
250 yr	4.00	NA	NA	4.00	NA	NA	4.00	NA	NA
500 yr	4.00	NA	NA	4.00	NA	NA	4.00	NA	NA

Table B3-32: HACK 9 Drainage Area Results - With Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
Present Conditions									
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.07	NA	NA	3.15	NA	NA	3.26	NA	NA
5 yr	3.08	NA	NA	3.25	NA	NA	3.38	NA	NA
10 yr	3.09	NA	NA	3.26	NA	NA	3.40	NA	NA
25 yr	3.11	NA	NA	3.32	NA	NA	3.48	NA	NA
50 yr	3.12	NA	NA	3.38	NA	NA	3.56	NA	NA
100 yr	3.12	NA	NA	3.38	NA	NA	3.62	NA	NA
250 yr	3.14	NA	NA	3.39	NA	NA	3.71	NA	NA
500 yr	3.15	NA	NA	3.44	NA	NA	3.80	NA	NA
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.50	NA	NA	3.50	NA	NA	3.57	NA	NA
5 yr	3.50	NA	NA	3.50	NA	NA	3.57	NA	NA
10 yr	3.50	NA	NA	3.57	NA	NA	3.57	NA	NA
25 yr	3.50	NA	NA	3.57	NA	NA	3.57	NA	NA
50 yr	3.50	NA	NA	3.70	NA	NA	3.69	NA	NA
100 yr	3.50	NA	NA	3.72	NA	NA	3.69	NA	NA
250 yr	3.56	NA	NA	3.72	NA	NA	3.69	NA	NA
500 yr	3.63	NA	NA	3.72	NA	NA	3.69	NA	NA
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.40	NA	NA	4.41	NA	NA	4.43	NA	NA
5 yr	4.40	NA	NA	4.42	NA	NA	4.43	NA	NA
10 yr	4.40	NA	NA	4.43	NA	NA	4.43	NA	NA
25 yr	4.40	NA	NA	4.44	NA	NA	4.44	NA	NA
50 yr	4.40	NA	NA	4.47	NA	NA	4.47	NA	NA
100 yr	4.40	NA	NA	4.47	NA	NA	4.47	NA	NA
250 yr	4.40	NA	NA	4.47	NA	NA	4.47	NA	NA
500 yr	4.40	NA	NA	4.47	NA	NA	4.47	NA	NA

3.2 Berry's Creek Drainage Area

Three options were analyzed for Berry's Creek drainage area. Option 1 involves a surge barrier on Berry's Creek near Paterson Plank Road Bridge. This surge barrier would then be combined with floodwalls and road raisings that tie the barrier into high ground. Option 1 protects both the east and west banks of Berry's Creek. Option 2 and 3 involve a combination of short, local sections of floodwall and levee with multiple pump stations. Option 2 only protects the east bank of Berry's Creek. Option 3 protects both banks of Berry's Creek by including additional floodwalls and levees from the east side to the west side of Berry's Creek. The concept of a gravity outlet retrofits plan, which was analyzed for the Hackensack River, is not appropriate for Berry's Creek as a complete system including pump stations and all LOP elements would be required in order to form a functioning project. Ultimately, Option 1 was the proposed option carried forward for Alternative 1 since it protects both sides of Berry's Creek, is cost-effective, and allows for flexibility for increased protection in future.

3.2.1 Option 1 (Surge Barrier)

Option 1 consists of a surge barrier on Berry's Creek just downstream of the Paterson Plank Road Bridge. The proposed surge barrier would be constructed to an elevation of 10.0 feet NAVD 88 and approximately 118 feet wide. The surge barrier would consist of two side by side gates on Berry's Creek to block tidal surges and would be closed just prior to anticipated surge events. A pump station with a capacity of 1,000-cfs is estimated to be required to pump interior river flows downstream to Berry's Creek outside of the closed barrier. The pump station capacity was determined by a preliminary HEC-HMS analysis stepping through a series of pump station capacities. A series of flap gates installed in the face of the surge barrier is recommended to improve the efficiency of the surge barrier system by allowing fluvial outflows in addition to pumped outflows when the upstream head differential increases above approximately 0.2 feet. The simulations run for Berry's Creek Option 1 included four 8-foot by 8-foot flap gates.

Levees, floodwalls, and other features would connect the surge barrier to existing high ground on both banks of Berry's Creek to maintain a top of LOP elevation of 7.0 feet NAVD 88. The surge barrier would be constructed to an elevation of 10.0 feet NAVD 88 to allow for future elevating of the LOP tie-off features without having to replace the surge barrier. Accessory features to the surge gate include a 400-fot long floodwall that would be about 2 feet high along Paterson Plank Road and the exit ramp to Plaza A Road, just east of the proposed surge barrier; road-raising of portions of Paterson Plank Road and Murray Hill Parkway northwest of the surge barrier; a small floodwall and regrading just east of the Rutherford Commons shopping center, and a closure gate over adjacent railroad tracks.

The areas described below are a part of the Berry's Creek drainage system. Each section will provide further detail on their respective water levels and damages associated with the various storm and risk conditions.

3.2.1.1 Dell Road Drainage Area

Under the Without conditions, stormwater flows out of Dell Road drainage area via two 4-foot by 4-foot box culverts to Paterson Plank Road Bridge, which then flows into Berry's Creek. Stormwater can also overflow overland and is modeled as three spillways with staged overflow elevations. One spillway is 30 feet long at 4.0 feet (NAVD 88), one is 120 feet long at 5.0 feet (NAVD 88), and the highest one is 90 feet long at 6.0 feet (NAVD 88).

With Option 1 in place, stormwater would no longer be able to flow out of Dell Road via the three spillways, therefore, all of the water would flow out through the two box culverts.

Table B3-33 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present , Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-34** provides a summary of the With conditions in the same format.

Table B3-33: Dell Road Drainage Area Results - Without Alternative 1

Event	Lower Bound Interior WSEL ft. NAVD88	Lower Bound Structures Flooded Above Ground	Lower Bound Structures Flooded Above Main Fl.	Most Likely Interior WSEL ft. NAVD88	Most Likely Structures Flooded Above Ground	Most Likely Structures Flooded Above Main Fl.	Upper Bound Interior WSEL ft. NAVD88	Upper Bound Structures Flooded Above Ground	Upper Bound Structures Flooded Above Main Fl.
Present Conditions									
2 yr	3.15	0	0	4.47	0	0	4.93	0	0
5 yr	3.18	0	0	4.71	0	0	4.94	0	0
10 yr	3.21	0	0	4.93	0	0	4.95	0	0
25 yr	3.25	0	0	4.93	0	0	4.96	0	0
50 yr	3.29	0	0	4.93	0	0	4.97	0	0
100 yr	3.34	0	0	4.93	0	0	4.98	0	0
250 yr	3.39	0	0	4.93	0	0	5.00	0	0
500 yr	3.44	0	0	4.93	0	0	5.02	0	0
Future Low SLR Conditions									
2 yr	4.01	0	0	5.18	0	0	5.18	0	0
5 yr	4.04	0	0	5.43	0	0	5.43	0	0
10 yr	4.06	0	0	5.62	0	0	5.62	0	0
25 yr	4.11	0	0	5.62	0	0	5.62	0	0
50 yr	4.15	0	0	5.62	0	0	5.62	0	0
100 yr	4.22	0	0	5.62	0	0	5.62	0	0
250 yr	4.27	0	0	5.62	0	0	5.62	0	0
500 yr	4.36	0	0	5.62	0	0	5.62	0	0
Future High SLR Conditions									
2 yr	4.89	0	0	6.21	1	1	6.21	1	1
5 yr	4.90	0	0	6.38	2	2	6.38	2	2
10 yr	4.91	0	0	6.47	2	2	6.47	2	2
25 yr	4.94	0	0	6.47	2	2	6.47	2	2
50 yr	4.97	0	0	6.47	2	2	6.47	2	2
100 yr	5.02	0	0	6.47	2	2	6.47	2	2
250 yr	5.05	0	0	6.47	2	2	6.47	2	2
500 yr	5.10	0	0	6.47	2	2	6.47	2	2

Note: Red values indicate a computed interior water surface or exterior tide elevation higher than the existing line of protection elevation. In this case, the peak exterior tide elevation is shown instead.

Table B3-34: Dell Road Drainage Area Results – With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	1.79	0	0	1.91	0	0	2.68	0	0
5 yr	2.33	0	0	2.42	0	0	2.71	0	0
10 yr	2.58	0	0	2.68	0	0	2.72	0	0
25 yr	2.82	0	0	2.96	0	0	3.00	0	0
50 yr	3.01	0	0	3.20	0	0	3.22	0	0
100 yr	3.18	0	0	3.41	0	0	3.45	0	0
250 yr	3.34	0	0	3.69	0	0	3.72	0	0
500 yr	3.46	0	0	3.89	0	0	3.96	0	0
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.05	0	0	2.06	0	0	2.92	0	0
5 yr	2.55	0	0	2.62	0	0	2.93	0	0
10 yr	2.84	0	0	2.92	0	0	2.94	0	0
25 yr	3.20	0	0	3.31	0	0	3.34	0	0
50 yr	3.60	0	0	3.67	0	0	3.70	0	0
100 yr	3.94	0	0	4.20	0	0	4.30	0	0
250 yr	4.19	0	0	4.78	0	0	4.94	0	0
500 yr	4.47	0	0	5.01	0	0	5.18	0	0
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.08	0	0	3.57	0	0	3.57	0	0
5 yr	2.60	0	0	3.57	0	0	3.57	0	0
10 yr	2.88	0	0	3.57	0	0	3.57	0	0
25 yr	3.28	0	0	3.57	0	0	3.57	0	0
50 yr	3.66	0	0	3.73	0	0	3.76	0	0
100 yr	4.11	0	0	4.40	0	0	4.54	0	0
250 yr	4.84	0	0	5.13	0	0	5.12	0	0
500 yr	5.00	0	0	5.48	0	0	5.53	0	0

3.2.1.2 East Riser Ditch – South Drainage Area

Under the Without conditions, the surface runoff from East Riser Ditch South is conveyed to Berry's Creek via four 5.5 foot by 7.5 foot box culverts and one surface overflow modeled as a 120-foot long spillway at elevation 6.0 feet (NAVD 88).

With Option 1 in place, water would no longer overflow on the ground surface. **Table B3-35** provides a summary of the flood levels and number of structures impacted under the Without condition for the Present, Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-36** provides a summary of the With conditions in the same format.

Table B3-35: East Riser Ditch South Drainage Area Results - Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.41	90	1	3.98	179	8	4.17	208	11
5 yr	3.49	99	2	4.07	195	11	4.21	216	11
10 yr	3.55	114	6	4.17	208	11	4.24	222	11
25 yr	3.65	129	6	4.26	225	12	4.38	244	12
50 yr	3.71	136	6	4.34	239	12	4.47	267	13
100 yr	3.79	154	7	4.42	254	13	4.55	282	14
250 yr	3.89	170	7	4.51	275	14	4.65	296	14
500 yr	4.01	187	11	4.72	307	14	5.02	356	19
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.87	166	7	5.18	371	21	5.18	371	21
5 yr	4.00	183	8	5.43	385	37	5.43	385	37
10 yr	4.08	198	11	5.62	388	52	5.62	388	52
25 yr	4.21	216	11	5.62	388	52	5.62	388	52
50 yr	4.35	240	12	5.62	388	52	5.62	388	52
100 yr	4.80	323	17	5.62	388	52	5.62	388	52
250 yr	5.62	388	52	5.62	388	52	5.62	388	52
500 yr	5.62	388	52	5.62	388	52	5.62	388	52
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.05	193	11	6.21	395	114	6.21	395	114
5 yr	4.18	212	11	6.35	396	134	6.35	396	134
10 yr	4.32	233	12	6.47	396	145	6.47	396	145
25 yr	4.49	268	13	6.47	396	145	6.47	396	145
50 yr	4.63	292	14	6.47	396	145	6.47	396	145
100 yr	5.40	383	33	6.47	396	145	6.47	396	145
250 yr	6.47	396	145	6.47	396	145	6.47	396	145
500 yr	6.47	396	145	6.47	396	145	6.47	396	145

Note: Red values indicate a computed interior water surface or exterior tide elevation higher than the existing line of protection elevation. In this case, the peak exterior tide elevation is shown instead.

Table B3-36: East Riser Ditch South Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.61	6	0	2.68	8	0	3.13	64	1
5 yr	2.97	42	0	3.01	55	1	3.14	64	1
10 yr	3.08	63	1	3.13	64	1	3.14	64	1
25 yr	3.25	76	1	3.29	78	1	3.31	78	1
50 yr	3.39	89	1	3.46	95	2	3.45	91	1
100 yr	3.54	114	6	3.59	120	6	3.61	124	6
250 yr	3.71	136	6	3.81	156	7	3.86	163	7
500 yr	3.83	158	7	4.36	241	12	4.52	279	14
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.78	14	0	2.81	19	0	3.26	76	1
5 yr	3.06	60	1	3.08	63	1	3.27	76	1
10 yr	3.23	73	1	3.26	76	1	3.27	76	1
25 yr	3.46	95	2	3.51	109	6	3.52	111	6
50 yr	3.76	146	6	3.85	163	7	3.94	175	7
100 yr	4.71	307	14	5.08	360	20	5.17	369	21
250 yr	5.79	391	67	6.02	395	93	6.05	395	94
500 yr	6.37	396	136	6.48	396	146	6.46	396	143
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.81	19	0	2.94	34	0	3.27	76	1
5 yr	3.07	62	1	3.09	58	1	3.27	76	1
10 yr	3.25	76	1	3.27	76	1	3.27	76	1
25 yr	3.50	103	3	3.47	97	2	3.53	114	6
50 yr	3.78	152	7	4.01	187	11	4.00	183	8
100 yr	4.96	347	19	5.24	375	24	5.28	379	26
250 yr	5.92	394	80	6.07	395	98	6.08	395	99
500 yr	6.36	396	135	6.49	396	148	6.53	396	151

3.2.1.3 Peach Island Creek Drainage Area

Under the Without conditions, the surface runoff from Peach Island Creek is conveyed to Paterson Plank Bridge via three 3-foot diameter culverts and overflow on the ground surface to the river. The existing ground surface overflows are modeled as four spillways at stepped elevations to replicate existing conditions. The spillways are configured as 35 feet long at 4.0 feet (NAVD 88), 159 feet long at 5.0 feet (NAVD 88), 391 foot long at 6.0 feet (NAVD 88), and 123 foot long at 7.0 feet (NAVD 88).

With Option 1 in place, water would no longer overflow via the ground surface. **Table B3-37** provides a summary of the flood levels and number of structures impacted under the Without condition for the Present, Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-38** provides a summary of the With conditions in the same format.

Table B3-37: Peach Island Creek Drainage Area Results - Without Alternative 1

Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
Present Conditions									
2 yr	3.02	5	2	3.25	6	3	4.01	64	20
5 yr	3.41	10	4	3.68	28	7	4.05	67	20
10 yr	3.74	30	7	4.01	64	20	4.09	70	21
25 yr	4.08	68	20	4.22	87	25	4.30	97	30
50 yr	4.25	92	29	4.39	103	31	4.47	113	36
100 yr	4.42	107	32	4.57	118	40	4.66	125	41
250 yr	4.64	123	41	4.81	132	46	4.90	136	49
500 yr	4.85	133	47	5.04	147	56	5.11	151	58
Future Low SLR Conditions									
2 yr	3.30	7	3	5.18	159	61	5.18	159	61
5 yr	3.79	34	7	5.43	188	69	5.43	188	69
10 yr	4.11	73	21	5.62	206	79	5.62	206	79
25 yr	4.42	107	32	5.62	206	79	5.62	206	79
50 yr	4.75	130	44	5.62	206	79	5.62	206	79
100 yr	5.14	157	60	5.62	206	79	5.62	206	79
250 yr	5.42	186	69	5.62	206	79	5.62	206	79
500 yr	5.62	206	79	5.62	206	79	5.62	206	79
Future High SLR Conditions									
2 yr	3.52	14	7	6.21	292	120	6.21	292	120
5 yr	4.02	66	20	6.35	305	126	6.35	305	126
10 yr	4.22	87	25	6.47	320	133	6.47	320	133
25 yr	4.55	117	39	6.47	320	133	6.47	320	133
50 yr	4.88	135	48	6.47	320	133	6.47	320	133
100 yr	5.26	163	63	6.47	320	133	6.47	320	133
250 yr	5.54	197	73	6.47	320	133	6.47	320	133
500 yr	5.80	232	91	6.47	320	133	6.47	320	133

Note: Red values indicate a computed interior water surface or exterior tide elevation higher than the existing line of protection elevation. In this case, the peak exterior tide elevation is shown instead.

Table B3-38: Peach Island Creek Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.57	2	2	2.58	2	2	3.54	14	7
5 yr	3.16	6	3	3.18	6	3	3.55	17	7
10 yr	3.48	13	6	3.54	14	7	3.55	17	7
25 yr	3.90	41	8	4.00	55	12	4.01	64	20
50 yr	4.13	76	21	4.17	81	24	4.18	84	25
100 yr	4.30	97	30	4.36	103	31	4.37	103	31
250 yr	4.55	117	39	4.60	120	41	4.62	122	41
500 yr	4.78	132	46	4.84	133	47	4.86	134	48
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.75	3	2	2.79	3	2	3.92	45	9
5 yr	3.42	11	4	3.42	11	4	3.93	48	11
10 yr	3.88	40	8	3.92	45	9	3.93	48	11
25 yr	4.25	92	29	4.28	95	29	4.29	96	30
50 yr	4.59	119	41	4.62	122	41	4.63	122	41
100 yr	5.02	146	56	5.06	148	56	5.07	149	57
250 yr	5.33	173	66	5.36	178	68	5.37	179	68
500 yr	5.64	207	80	5.68	212	81	5.69	213	81
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.81	3	2	2.89	3	2	3.93	48	11
5 yr	3.43	11	4	3.41	10	4	3.93	48	11
10 yr	3.90	41	8	3.93	48	11	3.93	48	11
25 yr	4.28	95	29	4.30	97	30	4.30	97	30
50 yr	4.61	120	41	4.63	122	41	4.64	123	41
100 yr	5.04	147	56	5.08	150	57	5.08	150	57
250 yr	5.35	178	68	5.38	180	68	5.40	184	68
500 yr	5.66	208	80	5.70	216	82	5.71	217	83

3.2.1.4 West Riser Ditch – South Drainage Area

Under the Without conditions, the surface runoff from West Riser Ditch South is conveyed to Paterson Plank Bridge via four 8-foot by 5.5-foot box culverts and also ground surface overflows to Berry's Creek. A portion of this interior stormwater also diverts out to East Riser Ditch South via overland surface flow.

The existing surface overflow from West Riser Ditch to Paterson Plank Road is modeled as two spillways at successively higher elevations; one spillway is 200 feet long at elevation 5.0 feet (NAVD 88) and another spillway is 40 feet long at elevation 6.0 feet (NAVD 88).

Additional existing surface overflow out to East Riser Ditch South is modeled with three spillways at successively higher elevations. One spillway is 100 feet long at 4.0 feet (NAVD 88); one spillway is 160 feet long at 4.5 feet (NAVD 88); and one spillway is 240 feet long at 5.5 feet (NAVD 88).

With Option 1 in place, water would no longer be able to flow out to Paterson Plank Road Bridge via the two ground surface overflows. Two 200-cfs pumps to Berry's Creek would be constructed to maintain lower interior WSELs.

Table B3-39 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present, Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-40** provides a summary of the With conditions in the same format.

Table B3-39: West Riser Ditch South Drainage Area Results - Without Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.15	0	0	4.47	16	7	4.93	28	13
5 yr	3.18	0	0	4.71	21	8	4.94	28	13
10 yr	3.21	0	0	4.93	28	13	4.95	28	13
25 yr	3.25	0	0	4.93	28	13	4.96	28	13
50 yr	3.29	0	0	4.93	28	13	4.97	28	13
100 yr	3.34	0	0	4.93	28	13	4.98	28	13
250 yr	3.39	0	0	4.93	28	13	5.00	28	13
500 yr	3.44	0	0	4.93	28	13	5.02	28	13
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.01	9	2	5.18	34	15	5.18	34	15
5 yr	4.04	9	2	5.43	42	20	5.43	42	20
10 yr	4.06	9	2	5.62	49	22	5.62	49	22
25 yr	4.11	10	2	5.62	49	22	5.62	49	22
50 yr	4.15	11	2	5.62	49	22	5.62	49	22
100 yr	4.22	13	2	5.62	49	22	5.62	49	22
250 yr	4.27	13	2	5.62	49	22	5.62	49	22
500 yr	4.36	14	3	5.62	49	22	5.62	49	22
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	4.89	27	12	6.21	70	32	6.21	70	32
5 yr	4.90	27	12	6.38	77	34	6.38	77	34
10 yr	4.91	27	12	6.47	79	35	6.47	79	35
25 yr	4.94	28	13	6.47	79	35	6.47	79	35
50 yr	4.97	28	13	6.47	79	35	6.47	79	35
100 yr	5.02	28	13	6.47	79	35	6.47	79	35
250 yr	5.05	30	13	6.47	79	35	6.47	79	35
500 yr	5.10	32	14	6.47	79	35	6.47	79	35

Note: Red values indicate a computed interior water surface or exterior tide elevation higher than the existing line of protection elevation. In this case, the peak exterior tide elevation is shown instead.

Table B3-40: West Riser Ditch South Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.77	0	0	2.81	0	0	3.52	1	0
5 yr	3.20	0	0	3.28	0	0	3.56	1	0
10 yr	3.42	0	0	3.52	1	0	3.56	1	0
25 yr	3.66	2	0	3.81	4	1	3.86	5	1
50 yr	4.22	13	2	4.33	13	2	4.38	15	4
100 yr	4.72	21	8	4.84	25	11	4.89	27	12
250 yr	5.15	34	15	5.24	35	16	5.27	36	17
500 yr	5.41	41	20	5.48	44	21	5.50	46	21
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.61	2	0	3.63	2	0	3.72	3	1
5 yr	3.63	2	0	3.64	2	0	3.73	3	1
10 yr	3.68	3	1	3.72	3	1	3.74	3	1
25 yr	4.32	13	2	4.42	16	5	4.51	18	7
50 yr	5.04	29	13	5.16	34	15	5.17	34	15
100 yr	5.49	45	21	5.55	47	22	5.57	47	22
250 yr	5.70	51	23	5.78	55	25	5.80	55	25
500 yr	5.92	57	27	6.00	62	30	6.00	62	30
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.62	2	0	4.50	16	7	4.50	16	7
5 yr	3.64	2	0	4.50	16	7	4.50	16	7
10 yr	3.70	3	1	4.50	16	7	4.50	16	7
25 yr	4.33	13	2	4.59	20	7	4.64	20	7
50 yr	5.12	32	14	5.20	35	16	5.21	35	16
100 yr	5.52	46	21	5.59	49	22	5.59	49	22
250 yr	5.75	55	24	5.82	55	25	5.82	55	25
500 yr	5.96	62	29	6.08	67	31	6.13	68	31

3.2.1.5 Paterson Plank Road Bridge Drainage Area

Under the Without conditions, Paterson Plank Road Bridge conveys the flow of Berry's Creek via 3 outlets.

Stormwater flows under Paterson Plank Road Bridge on Berry's Creek with three overflow spillways which represent flows over the roadway. In the initial hydraulic model, the flow under the bridge is represented as culvert flow: one culvert is 14 feet by 53 feet; another one is 17 feet by 62 feet; and one is 11.46 feet by 53 feet. The HEC-RAS unsteady model for Option 1 uses bridge modeling computations. The overflow spillways are: 140-feet long at elevation 6.0 feet (NAVD 88), 100-feet long at elevation 6.5 feet (NAVD 88); and 10 feet long at elevation 8.0 feet (NAVD 88).

Table B3-41 provides a summary of the flood levels and number of structures impacted under the Without condition for the Present, Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions. **Table B3-42** provides a summary of the With conditions in the same format.

Table B3-41: Paterson Plank Road Bridge Drainage Area Results - Without Alternative 1

Event	Lower Bound Interior WSEL ft. NAVD88	Lower Bound Structures Flooded Above Ground	Lower Bound Structures Flooded Above Main Fl.	Most Likely Interior WSEL ft. NAVD88	Most Likely Structures Flooded Above Ground	Most Likely Structures Flooded Above Main Fl.	Upper Bound Interior WSEL ft. NAVD88	Upper Bound Structures Flooded Above Ground	Upper Bound Structures Flooded Above Main Fl.
Present Conditions									
2 yr	3.15	1	1	4.47	8	6	4.93	14	11
5 yr	3.18	1	1	4.71	11	9	4.94	15	12
10 yr	3.21	2	1	4.93	14	11	4.95	16	12
25 yr	3.25	2	1	4.93	14	11	4.96	16	12
50 yr	3.29	2	1	4.93	14	11	4.97	16	12
100 yr	3.34	2	1	4.93	14	11	4.98	16	12
250 yr	3.39	2	1	4.93	14	11	5.00	16	12
500 yr	3.44	2	1	4.93	14	11	5.02	17	12
Future Low SLR Conditions									
2 yr	4.01	4	2	5.18	21	16	5.18	21	16
5 yr	4.04	4	2	5.43	29	22	5.43	29	22
10 yr	4.06	4	2	5.62	32	26	5.62	32	26
25 yr	4.10	4	2	5.62	32	26	5.62	32	26
50 yr	4.14	4	2	5.62	32	26	5.62	32	26
100 yr	4.21	4	2	5.62	32	26	5.62	32	26
250 yr	4.25	5	3	5.62	32	26	5.62	32	26
500 yr	4.34	5	3	5.62	32	26	5.62	32	26
Future High SLR Conditions									
2 yr	4.88	14	11	6.21	44	32	6.21	44	32
5 yr	4.90	14	11	6.35	46	34	6.35	46	34
10 yr	4.91	14	11	6.47	49	37	6.47	49	37
25 yr	4.93	14	11	6.47	49	37	6.47	49	37
50 yr	4.96	16	12	6.47	49	37	6.47	49	37
100 yr	5.01	16	12	6.47	49	37	6.47	49	37
250 yr	5.04	17	12	6.47	49	37	6.47	49	37
500 yr	5.09	18	13	6.47	49	37	6.47	49	37

Note: Red values indicate a computed interior water surface or exterior tide elevation higher than the existing line of protection elevation. In this case, the peak exterior tide elevation is shown instead.

Table B3-42: Paterson Plank Road Bridge Drainage Area Results - With Alternative 1

Present Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	2.75	0	0	2.77	0	0	2.78	0	0
5 yr	2.75	0	0	2.78	0	0	2.78	0	0
10 yr	2.75	0	0	2.78	0	0	2.78	0	0
25 yr	2.79	0	0	2.96	0	0	2.99	1	1
50 yr	3.02	1	1	3.20	2	1	3.21	2	1
100 yr	3.16	1	1	3.41	2	1	3.45	2	1
250 yr	3.34	2	1	3.68	3	1	3.72	3	1
500 yr	3.46	2	1	3.88	3	1	3.95	3	1
Future Low SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.61	3	1	3.63	3	1	3.64	3	1
5 yr	3.61	3	1	3.64	3	1	3.64	3	1
10 yr	3.61	3	1	3.64	3	1	3.64	3	1
25 yr	3.61	3	1	3.64	3	1	3.64	3	1
50 yr	3.61	3	1	3.67	3	1	3.70	3	1
100 yr	3.93	3	1	4.20	4	2	4.30	5	3
250 yr	4.18	4	2	4.78	12	10	4.94	15	12
500 yr	4.46	8	6	5.00	16	12	5.17	21	16
Future High SLR Conditions									
Event	Lower Bound			Most Likely			Upper Bound		
	Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded		Interior WSEL ft. NAVD88	Structures Flooded	
		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.		Above Ground	Above Main Fl.
2 yr	3.62	3	1	4.49	8	6	4.49	8	6
5 yr	3.62	3	1	4.49	8	6	4.49	8	6
10 yr	3.62	3	1	4.49	8	6	4.49	8	6
25 yr	3.62	3	1	4.49	8	6	4.49	8	6
50 yr	3.66	3	1	4.49	8	6	4.49	8	6
100 yr	4.10	4	2	4.49	8	6	4.54	8	6
250 yr	4.84	12	10	5.13	19	14	5.12	19	14
500 yr	5.00	16	12	5.47	29	22	5.52	30	24

3.2.2 Option 2 (East Bank of Berry's Creek)

Option 2 consists of sections of floodwall/levee that border the east side of Berry's Creek. The floodwall/levee structures, with additional pumping, would create localized protection in the Berry's Creek area, as opposed to the surge barrier under Option 1 that protects widely by blocking any tidal surge from entering the Berry's Creek area. Option 2 would only protect the east side, leaving the west side exposed to the flood waters that naturally occur, and to the flood waters that could no longer flow over the east drainage areas.

Option 3 is very similar to Option 2, except it includes additional protection on the west side of Berry's Creek; Option 3 was developed prior to Option 2 due to their similarity in nature. Since Option 1 proved to be more feasible in terms of costs, benefits, and flexibility than Option 3, Option 2 was not analyzed due to its loss of benefits from lack of protection on the west side of Berry's Creek without equally offsetting the costs.

3.2.3 Option 3 (East and West Bank of Berry's Creek)

As mentioned in **Section 3.2.2**, Option 3 would consist of numerous sections of floodwall/levee and smaller pumps surrounding both the east and west side of Berry's Creek. Details of the Option 3 structures and water levels caused by various storm and risk conditions are described in the below sections, separated by the drainages areas along Berry's Creek.

3.2.3.1 Dell Road

With Option 3 in place, water would no longer be able to flow out to Berry's Creek via the spillways due to the addition of a wall/levee along the flow paths. No additional outlet structures would be required to meet the design goal to avoid induced flooding for the 10-year storm against the 2-year tide, which is the Likely risk condition. **Table B3-43** provides a summary of the flood levels and number of structures impacted under Option 3 for the Present, Future Low SLR, and Future High SLR with the Most Likely, Lower Bound, and Upper Bound conditions.

Table B3-43: Dell Road Drainage Area Results - With Berry's Creek Option 3

Present Conditions			
Event	Interior WSEL ft. NAVD88		
	Lower Bound	Most Likely	Upper Bound
2 yr	3.02	3.80	4.19
5 yr	3.02	4.07	4.46
10 yr	3.03	4.19	4.66
25 yr	3.04	4.19	4.92
50 yr	3.04	4.19	5.06
100 yr	3.05	4.19	5.14
250 yr	3.06	4.19	5.20
500 yr	3.07	4.19	5.22

3.2.3.2 East Riser Ditch – South Drainage Area

With Option 3 in place, water would no longer be able to flow out to Berry's Creek via the spillway due to the addition of a wall/levee over the flow path. Option 3 would include the addition of a pump station with two 35-cfs pumps to help drain the interior flooding. Drainage swales would be constructed along the landward side of the coastal storm risk management structure to direct runoff toward the structure. Additionally, all outlets that run through the proposed LOP would be fitted with a flap gate.

Table B3-44 provides a summary of the flood levels and number of structures impacted under Option 3 for the Present, Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions.

Table B3-44: East Riser Ditch South Drainage Area Results - With Berry's Creek Option 3

Present Conditions			
Event	Interior WSEL ft. NAVD88		
	Lower Bound	Most Likely	Upper Bound
2 yr	2.83	3.11	3.57
5 yr	3.01	3.39	3.77
10 yr	3.08	3.57	4.01
25 yr	3.16	3.77	4.26
50 yr	3.21	3.88	4.45
100 yr	3.26	3.97	4.65
250 yr	3.31	4.05	4.86
500 yr	3.43	4.10	5.02

3.2.3.3 Peach Island Creek Drainage Area

With Option 3 plan in place, water would no longer be able to flow out through the spillways due to the addition of a wall/levee across the flow paths. Option 3 would include the addition of a pump station with two 70-cfs pumps to drain the interior flooding. Drainage swales would be constructed along the landward side of the coastal storm risk management structure to direct runoff toward the structure. Additionally, all outlets that run through the proposed LOP would be fitted with a flap gate.

Table B3-45 provides a summary of the flood levels and number of structures impacted under Option 3 for the Present, Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions.

Table B3-45. Peach Island Creek Drainage Area Results - With Berry's Creek Option 3

Present Conditions			
Event	Interior WSEL ft. NAVD88		
	Lower Bound	Most Likely	Upper Bound
2 yr	2.74	2.32	3.17
5 yr	2.74	2.86	3.22
10 yr	3.09	3.17	3.26
25 yr	3.43	3.53	3.66
50 yr	3.75	3.85	4.00
100 yr	4.05	4.11	4.19
250 yr	4.30	4.36	4.47
500 yr	4.55	4.62	4.76

3.2.3.4 West Riser Ditch – South Drainage Area

With the Option 3 plan in place, water would no longer be able to flow out towards Paterson Plank Road Bridge through the two spillways due to the addition of wall/levee across the flow paths. The Option 3 plan would include the addition of a pump station with two 200-cfs pumps to drain the interior flooding. Drainage swales would be constructed along the landward side of the coastal storm risk management structure to direct runoff toward the structure. Additionally, all outlets that run through the proposed LOP would be fitted with flap gates.

Table B3-46 provides a summary of the flood levels and number of structures impacted under Option 3 for the Present, Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions.

Table B3-46: West Riser Ditch South Drainage Area Results - With Berry's Creek Option 3

Present Conditions			
Event	Interior WSEL ft. NAVD88		
	Lower Bound	Most Likely	Upper Bound
2 yr	3.39	3.85	4.40
5 yr	4.11	4.14	4.44
10 yr	4.35	4.40	4.49
25 yr	4.69	4.74	4.80
50 yr	4.91	4.97	5.02
100 yr	5.10	5.16	5.20
250 yr	5.32	5.39	5.44
500 yr	5.52	5.61	5.66

3.2.3.5 Paterson Plank Road Bridge Drainage Area

The Option 3 plan would not have any direct effects on the Without drainage conditions. Therefore, no additional structures would be required. However, some drainage areas that drained to the Paterson Plank Road Bridge drainage area would be unable to with the Option 3 plan in place. Therefore, Paterson Plank Road Bridge would experience some water level reductions.

Table B3-47 provides a summary of the flood levels and number of structures impacted under Option 3 for the Present, Future Low SLR, and Future High SLR conditions with the Most Likely, Lower Bound, and Upper Bound conditions.

Table B3-47: Paterson Plank Road Bridge Drainage Area Results - With Berry's Creek Option 3

Present Conditions			
Event	Interior WSEL ft. NAVD88		
	Lower Bound	Most Likely	Upper Bound
2 yr	3.03	3.82	4.33
5 yr	3.04	4.17	4.41
10 yr	3.05	4.33	4.47
25 yr	3.07	4.33	4.53
50 yr	3.09	4.33	4.58
100 yr	3.11	4.33	4.61
250 yr	3.13	4.33	4.66
500 yr	3.15	4.33	4.72

4.0 Economic Damage Assessments

4.1 Conditions

Analysis of benefits and costs for formulation of interior drainage plans is conducted using an interest rate of 7 percent applied over a 50 year period-of-analysis. Baseline conditions consider the current sea level; future conditions consider the two SLR conditions as previously stated (Future Low SLR and Future High SLR).

4.2 Costs

Interior drainage consists of various structures required to maintain current drainage and avoid induced flood-damage. Structures were selected based on interior drainage improvements that are economically justified based on a comparison of benefits and costs. These costs consist of first construction costs and annual operation and maintenance expenses, as described in the sections below.

4.2.1 First Construction Costs

First construction costs for interior drainage facilities may include primary and secondary outlets, intake structures and outlet gates, pump stations, and new outfalls.

4.2.2 Operation and Maintenance

Annual costs attributed to the operation and maintenance of interior drainage facilities may consist of, but are not limited to, labor charges for the inspection, care and cleaning of pond areas, outlets and pump stations, as well as anticipated energy charges and annualized replacement costs.

4.3 Damages

Flood damage reduction benefits for interior drainage facilities are calculated as the difference between the Without damages and the residual damages associated under the With condition (i.e., with Alternative 1 implemented).

4.3.1 Interior Flood Damage

As described in **Appendix E**, the Likely damage to each structure was calculated for the required range of flooding depths. These damages were then aggregated to determine composite stage versus damage relationships for each interior area.

4.3.2 Annual Damages

Annual damage was calculated using a risk based simulation technique. The stage frequency and discharge frequency relationships calculated in HEC-HMS and HEC-RAS were input into HEC-FDA. The HEC-FDA model calculates the Average Annual Damages (AAD) for both the base and future conditions (with sea level change). Equivalent Annual Damages (EAD) for the 50 year period of analysis was also calculated.

4.3.3 Alternative 1 Damages

As noted in **Section 1.2.1** and **Section 1.2.2**, the gravity outlet retrofits are the baseline for evaluating interior drainage alternatives. The magnitude of these damages helps to guide decisions on the type and scale of interior flood risk management measures to consider. **Table B3-48** provides a summary of the Expected Annual Damage and EAD for each of the Hackensack River drainage area interior areas. The majority of the interior damages occur in DePeyster Creek and Losen Slote. **Table B3-49** provides a corresponding summary for each of the Berry's Creek drainage area interior areas.

Table B3-48: Annual Damage of Hackensack River Drainage Area - With Alternative 1

Interior Drainage Area	Expected Annual Damage			Equivalent Annual Damage*	
	Base Year	Future Low SLR**	Future: High SLR***	Future: Low SLR**	Future: High SLR***
DePeyster Creek	$231,100	$549,800	$831,300	$314,700	$381,500
Indian Lake	$6,100	$6,100	$6,100	$6,200	$6,100
Losen Slote	$227,800	$616,500	$693,000	$326,000	$344,300
Main Street Ditch	$23,900	$107,000	$113,300	$44,700	$46,300
Moonachie Creek	$29,200	$49,900	$55,900	$34,300	$35,900
Willow Lake	$24,100	$76,100	$155,900	$37,100	$57,100
HACK 1A3	$400	$19,000	$32,000	$5,000	$8,300
HACK 1	$3,500	$14,500	$20,500	$6,300	$7,800
HACK 2	$1,000	$11,200	$12,700	$3,600	$3,900

Interior Drainage Area	Expected Annual Damage			Equivalent Annual Damage*	
	Base Year	Future Low SLR**	Future: High SLR***	Future: Low SLR**	Future: High SLR***
HACK 3	$400	$2,400	$2,600	$900	$900
HACK 4	$9,300	$93,900	$107,700	$30,500	$33,900
HACK 5	$2,400	$11,900	$16,000	$4,700	$5,800
HACK 6	$21,800	$29,900	$37,600	$23,200	$25,700
HACK 7	$100	$1,100	$2,500	$400	$700
HACK 8	$0	$200	$400	$100	$100
HACK 9	$0	$0	$0	$0	$0

*7 percent Discount Rate
**NOAA Intermediate-Low / USACE Intermediate (Modified NRC Curve II)
***NOAA Intermediate-High

Table B3-49: Annual Damages of the Berry's Creek Drainage Area - With Alternative 1

Interior Drainage Area	Expected Annual Damage			Equivalent Annual Damage*	
	Base Year	Future: Low SLR**	Future: High SLR***	Future: Low SLR**	Future: High SLR***
Dell Road	$1,200	$2,800	$8,700	$1,800	$3,700
East Riser Ditch South	$1,414,000	$3,781,400	$3,700,800	$2,210,000	$2,167,600
Peach Island Creek	$5,181,900	$8,652,300	$8,857,600	$6,348,800	$6,550,800
West Riser Ditch South	$254,800	$604,100	$1,094,300	$372,200	$535,600
Paterson Plank Road Bridge	$591,900	$2,189,000	$3,623,300	$1,128,900	$1,702,800

*7 percent Discount Rate
**NOAA Intermediate-Low / USACE Intermediate (Modified NRC Curve II)
***NOAA Intermediate-High

5.0 Conclusion

The Alternative 1 LOP recommended in the RBDM Flood Protection Project Draft Feasibility Study Report would be the first line of defense against significant coastal surge and wave action. However, if the design was implemented in absence of any interior drainage measures, the plan would not meet the Proposed Project objectives and the Project Area would still experience extensive damage to properties, and would have experienced increased WSELs in some interior drainage areas. For some drainage areas, the local flooding damage experienced in the range of studied storm frequency events was severe enough to justify the cost of the construction of pump stations to effectively lower WSELs.

The Alternative 1 interior drainage plan would aid in the discharge or controlled storage of interior floodwaters during low frequency precipitation events. Together with the Alternative 1 LOP plan, this complimentary system would provide coastal storm risk management in the Project Area for the two most common forms of severe storm events (i.e., hurricanes and nor'easters).

The specified design criteria, however, would not eliminate all coastal flooding or interior drainage flooding within the study limits of the Project Area. There could still be some localized flooding behind the LOP alignment after implementation of Alternative 1.

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Subappendix B4: Integrated Modeling Approach

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Table of Contents

List of Tables

List of Figures

Acronyms and Abbreviations

1D	One-dimensional
2D	Two-dimensional
HEC-HMS	Hydrologic Engineering Center - Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center - River Analysis System
ICM	Integrated Catchment Modeling
NAVD 88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
RBDM	Rebuild by Design Meadowlands
WSEL	Water surface elevation

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1.0 Introduction

The Rebuild by Design Meadowlands (RBDM) project team implemented an integrated numerical modeling approach to evaluate potential drainage improvements in the Project Area under normal tidal conditions. Due to the large area involved (approximately 5,500 acres), the complexity of the existing storm drainage system, and the need to address isolated areas of flooding and system-wide improvements, a phased modeling approach utilizing multiple models was implemented. This subappendix does the following:

- Describes the general drainage system configuration in the Project Area;
- Identifies the models use to characterize different parts of the system;
- Describes the overall phased modeling approach applied; and
- Defines the how the model outputs and inputs were linked.

This subappendix is intended to provide a framework for understanding the overall modeling effort applied for the development, screening, and evaluation of drainage improvement concepts. Detailed discussions of each separate modeling effort are included in the RBDM Feasibility Study Report and associated appendices.

2.0 Project Drainage System Configuration

The drainage system conveys rainfall runoff to water bodies within and around the Project Area and services approximately 6.85 square miles. The majority of the topography is flat or somewhat flat, especially near the Hackensack River. Some of the inland areas have topography that is slightly steep. Elevations vary from 211 feet North American Vertical Datum of 1988 (NAVD 88) in the Paterson Plank Road Bridge drainage area, to -8 feet NAVD 88 at the lowest area in Willow Lake. The majority of the Project Area is urban, consisting of residential, commercial, and industrial establishments. Most of the residential areas are located in the Borough of Little Ferry. A portion of the Project Area is occupied by Teterboro Airport. Precipitation falling in the Project Area drains to surface ditches and subsurface pipes and reports to the Hackensack River through the following main drainage conveyances:

- West Riser Ditch;
- East Riser Ditch;
- Little Ferry Circle Drainage System and Outfall;
- Main Street Collection and Discharge;
- Depeyster Creek;
- Losen Slote; and
- Carlstadt Collection and Discharge.

Figure B4-1 identifies these areas, the associated drainage sub-basins, and the existing discharge locations to the Hackensack River.

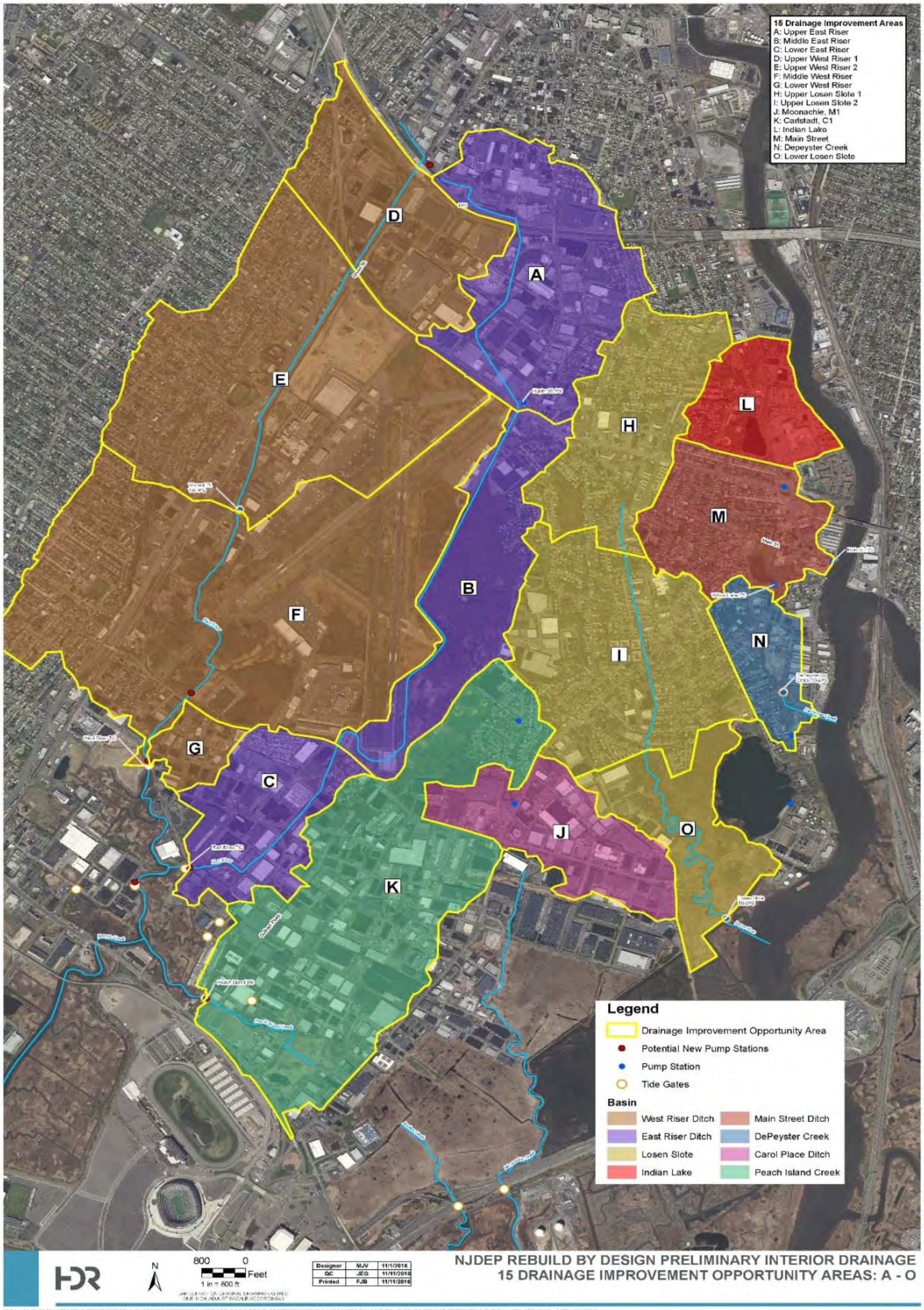

Figure B4-1: Drainage Improvement Sub-Basins in the Project Area

Flooding in the Project Area can result from either high storm surges from the Hackensack River or Berry's Creek, or interior precipitation runoff that cannot be conveyed to the Hackensack River or Berry's Creek through the existing drainage system under normal tidal conditions. Coincident heavy rainfall and storm surge events exacerbate the fluvial drainage problems that occur under heavy rainfall events during normal tidal conditions.

The selection of models to be applied in the Project Area was influenced by the need to model a large and complex, tidally influenced set of subsurface storm drainage networks, ditches, and streams.

3.0 Models Selected For Fluvial Drainage Evaluations

The RBDM project team selected the models summarized in **Table B4-1** to evaluate fluvial drainage improvement in the Project Area. The RBDM project team identified a range of potential drainage system improvements prior to actual concept development, including channel dredging, existing pipe upgrades, and pump station upgrades and new installation. For areas where the focus was primarily on increasing open channel conveyance via dredging and pump station installation, such as the East and West Riser, Hydrologic Engineering Center - River Analysis System (HEC-RAS), a widely recognized open channel and floodplain model, was applied. Areas including dense networks of subsurface pipes, such as the Main Street and the Little Ferry area, were modeled with InfoWorks Integrated Catchment Modeling (ICM), which can accommodate all candidate measures for drainage improvement (i.e., channel modifications, subsurface pipe upgrades, pumps, etc.).

Table B4-1: Models Selected for Fluvial Evaluations

Model	Primary Purpose	Comments
Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS)	Simulate interior runoff in the Project Area	Applied to all sub-basins on **Figure B4-1** Uses runoff curve numbers based on land use and soil types and includes drainage system diversions to translate rainfall to runoff
HEC-RAS one-dimensional (1D) and two-dimensional (2D)	Initial simplified evaluation of effectiveness of channel improvements (1D) and pumping Detailed evaluation of channel improvements and pumping	Applied to sub-basins A through I, N, and O Used to understand effects of potential proposed improvements and include changes in water surface elevation (WSEL) in stream conveyances and overbank areas Applied to major streams receiving interior drainage from pipe and ditch networks
InfoWorks ICM	Initial simplified evaluation of effectiveness of subsurface pipe, channel, and pumping concepts Detailed evaluation of effectiveness of the same measures	Applied to sub-basins L, M, K, and J Applied to sub-basins M, N, H, I, and O

Model	Primary Purpose	Comments
MIKE 21	Evaluate coastal surge in the Project Area Provide tidal boundary conditions for fluvial models	-

4.0 Phased Model Evaluations

The RBDM project team applied the models summarized in **Table B4-1** in a phased fashion. Simplified HEC-HMS, HEC-RAS 1D, and InfoWorks models were developed to facilitate screening of the first set of 30 drainage improvement concepts. The initial 30 concepts were screened down to six based on hydraulic improvements and other criteria. More detailed models, or more detailed versions of the models noted above, were developed for the second screening of concepts and for the final feasibility evaluations. For the HEC-HMS and InfoWorks models, the general difference between the simplified and detailed models was the sub-basin resolution (HEC-HMS) and the number of subsurface structures included in the models (InfoWorks). For HEC-RAS, the simplified models were developed as 1D steady state models run agains tidal datums at the downstream boundary condition. The detailed models were unsteady HEC-RAS 2D models containing the final complement of channel and structure survey data and run against a dynamic downstream boundary condition. A more detailed description of the differences between the simplified and detailed models can be found in the appendices of the RBDM Feasibility Study Report; see **Subappendices B6**, **B7**, and **B8**.

5.0 Model Linkages

The models summarized in **Table B4-1** were applied to simulate existing and proposed fluvial drainage conditions in the Project Area. Outputs from some models were used as inputs to others, as described below and summarized in **Figure B4-2** and **Figure B4-3**. Simplified and detailed modeling was completed as part of the drainage improvement and screening process. This section is organized to describe these linkages.

Simplified HEC-HMS, HEC-RAS 1D, and InfoWorks Modeling (Figure B4-2)

The phase 1 HEC-HMS model was run to produce sub-basin discharge hydrographs. Those hydrographs were routed through the streams in noted sub-basins to evaluate existing WSELs and changes resulting from dredging, culvert upgrades, and pumping. Rainfall data applied in HEC-HMS was applied in InfoWorks for the sub-basins noted in **Table B4-1** to evaluate existing WSELs and changes affected by subsurface pipe, culvert and ditch upgrades, and pumping. The downstream boundary condition for both the HEC-RAS 1D and the InfoWorks model runs were National Oceanic and Atmospheric Administration (NOAA) tidal datums translated to the appropriate locations.

Detailed HEC-HMS, HEC-RAS 2D and InfoWorks Modeling (Figure B4-3)

The phase 2 HEC-HMS model was run to produce excess precipitation recorded in inches per unit time. This metric represents the incident rainfall translated into runoff. The excess rainfall was applied to the HEC-RAS 2D model to route the flows through the 2D terrain to evaluate the effects of dredging, culvert upgrades, and pumping. The rainfall applied in the HEC-HMS model was applied in the InfoWorks model to evaluate effects of subsurface pipe, culvert and ditch upgrades, and pumping. The MIKE 21 model was run to produce time stage time series at the downstream boundary condition for the HEC-RAS 2D and InfoWorks models. Where appropriate, the HEC-RAS 2D model provided interior boundary conditions for the InfoWorks models.

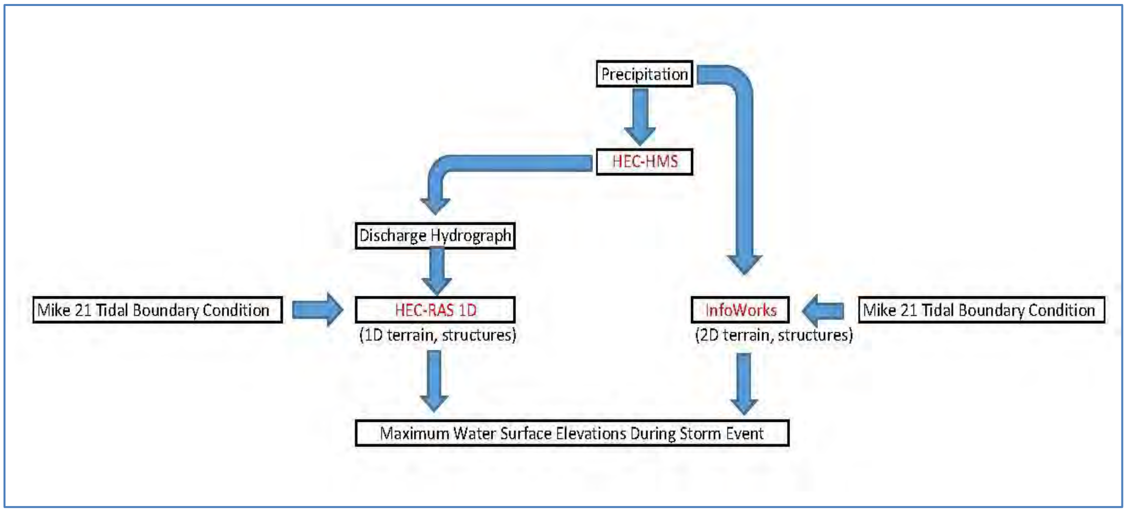

Figure B4-2: Simplified Modeling Linkages

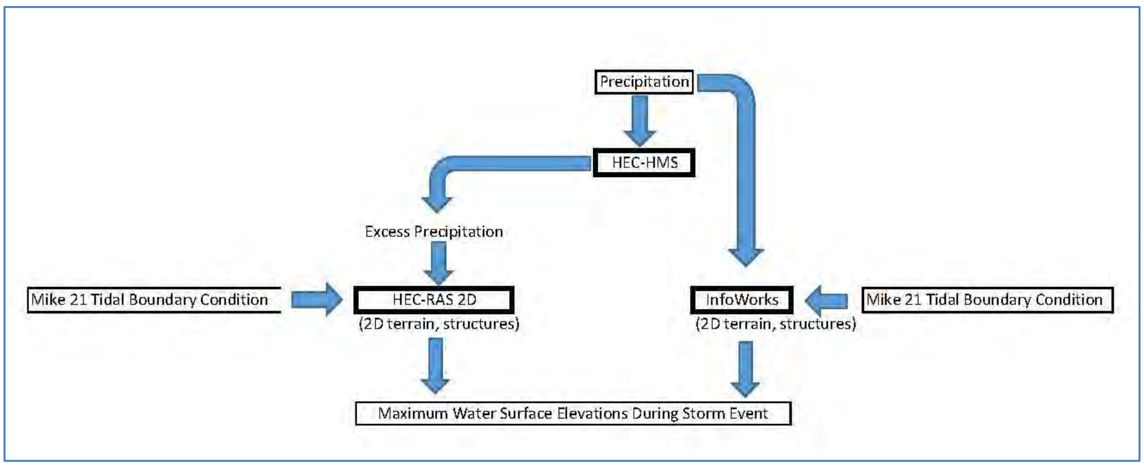

Figure B4-3: Detailed Modeling Linkages

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Subappendix B5: Drainage Capacity Evaluation Criteria

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Table of Contents

List of Tables

List of Figures

Acronyms and Abbreviations

1D	One-dimensional
2D	Two-dimensional
CPK	Central Park NOAA gauge
DEM	Digital elevation model

HEC-HMS	Hydrologic Engineering Center - Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center - River Analysis System
ICM	Integrated Catchment Modeling
LiDAR	Light Detection and Ranging
MHW	Mean High Water
MHHW	Mean Higher High Water
MLW	Mean Low Water
MLLW	Mean Lower Low Water
MSL	Mean Sea Level
NAVD 88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
PFE	Precipitation Frequency Estimates
QA/QC	Quality Assurance / Quality Control
SLR	Sea level rise
TET	Teterboro Airport NOAA gauge
WSEL	Water surface elevation

1.0 Introduction

This subappendix provides information for the Proposed Project fluvial modeling efforts as follows:

1) To identify validation rainfall events for model accuracy for Validation Scenarios for InfoWorks Integrated Catchment Modeling (ICM) and Hydrologic Engineering Center - River Analysis System (HEC-RAS) two-dimensional (2D) models;
2) To establish boundary conditions for InfoWorks and HEC-RAS models for tidal and rainfall data (these boundary conditions will be used for model runs for the scenarios Without (i.e., existing conditions) and With (i.e., proposed conditions) the Proposed Project); and
3) To identify water surface elevation (WSEL) reductions associated with flood damage reduction.

To identify potential validation rainfall events, HDR reviewed data for a 22-year period (1995 to 2017) from multiple sources near the Project Area. **Figure B5-1** displays the data stations that HDR has identified as credible sources to determine validation events. To qualify a storm for a validation event, normal tidal conditions had to prevail throughout the event and there needed to be visual observations or metered data to document the systems response (i.e., levels of in-basin flooding).

2.0 Tide Data

2.1 Validation Events Tidal Datum

The Battery, NY tidal data was used to determine if the tidal level was within normal tidal conditions. The Battery, NY gauge was chosen as the basis because it is the closest long-term reliable gauge to the Project Area (eight miles southeast of the Project Area, see **Figure B5-1**). Although it is not ideal to characterize the tidal conditions at the Project Area, any offshore high tidal events observed at the Project Area should be observed at the Battery in NY. The Battery is available to download in sub-hourly time steps from the National Oceanic and Atmospheric Administration (NOAA) for the past 22 years of concern. Other stations that are closer to the Project Area, such as Kearny Point, Hackensack River, and Bergen Point West Reach, were not used because they are not available for download and verified by NOAA.

Mean Higher High Water (MHHW), Mean High Water (MHW), Mean Sea Level (MSL), Mean Low Water (MLW), and Mean Lower Low Water (MLLW) are displayed in the **Table B5-1** for the Battery. Normal tidal conditions are defined as between MHHW and MLLW. These values were calculated during the last Epoch (1983 to 2001). A full list of reference datum's can be found in **Appendix 1** to **Subappendix B5**. The datum provided by the Battery, NY are provided relative to the local station datum and were converted to North American Vertical Datum of 1988 (NAVD 88).

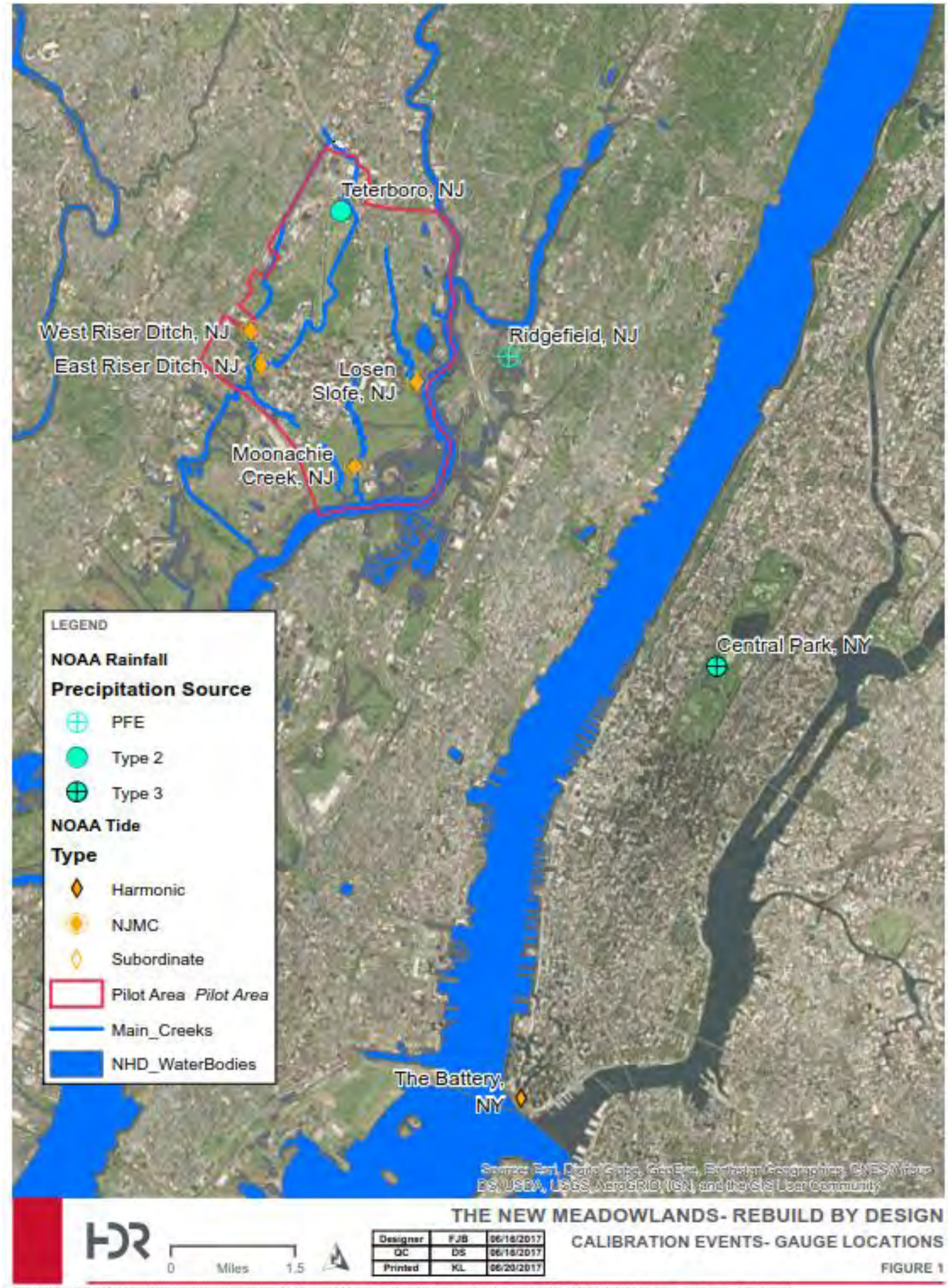

Figure B5-1: Gauge Locations for Calibration Events

Table B5-1: The Battery Tidal Datums

Datum	Battery, NY (feet NAVD 88)
MHHW	2.28
MHW	1.96
MSL	-0.2
MLW	-2.57
MLLW	-2.77

2.2 Tidal Boundary Conditions for Without and With Scenarios

AECOM performed two separate analyses in order to determine the tidal datums to be used for East Riser Ditch, West Riser Ditch, Losen Slote Creek, and DePeyster Creek. Seven months of data from East Riser Ditch was used to compute tidal datums, as per NOAA's Tidal Datums Handbook's Modified Range Ratio Method (NOAA 2003). Below describes some of the key limitations:

- Raw data was biased. To compensate, 1.2 feet were added to each of the data points in order to match the lows on the East Riser upstream gauge.
- Seven consecutive months of continuous data were not available. These available seven months were selected:
    - March, April, and May 2014;
    - August and September 2015; and
    - January and February 2016.
- Numerous gaps observed in the data were filled as relevant and necessary.

Linear relationships between tidal datums and distance along the Hackensack River were established based on the available NOAA stations. Since data is available at points upstream and downstream of the locations of interest, the datums were interpolated between them to identify the appropriate tidal datums.

AECOM provided the MHHW, MSL, and MLLW data. These are displayed in the **Table B5-2** for the four locations within the Project Area. MHW and MLW were calculated from data for the last two years of data since the data had fewer missing records and was more consistent.

Table B5-2: Tidal Datums for Water Bodies

Datum	East/West Riser Ditch	DePeyster Creek	Losen Slote
	Level (feet NAVD 88)		
MHHW	3.07	3.01	2.98
MHW	2.85	2.69	2.66
MSL	0.55	-0.02	-0.03
MLW	-2.49	-3.17	-3.14
MLLW	-2.69	-3.45	-3.42

3.0 Evaluation of Validation Event Rainfall and Storm Return Period

3.1 Precipitation Data Sources

Teterboro Airport (TET) is a NOAA gauge located within the Project Area (see **Figure B5-1**) which has data available from 1997 to the present. TET is a Type 2 data record. NOAA's Type 2's are not verified by NOAA and does not complete their Quality Assurance / Quality Control (QA/QC) process. The Central Park (CPK) NOAA gauge, located 7 miles east of the Project Area, can be used when TET data is not available. Type 3 data are verified by NOAA and complete their QA/QC process before being shared with the public. Rainfall hyetographs for validation events (**Section 4.0**) are available in **Appendix 6** to **Subappendix B5**.

3.2 Design Storms for Without and With Scenarios

A summary of the available rainfall data from each gauge is provided in **Table B5-3**. CPK also provides Precipitation Frequency Estimates (PFE), as does a gauge in the Borough of Moonachie. PFE's are calculated for both precipitation and intensity. 24-hour precipitation and 1-hour maximum intensity are shown in **Table B5-4**. Systems are overwhelmed by both the total volume and large bursts of high intensity rainfall. The effect on the system is different and varies from system to system. **Appendix 2** to **Subappendix B5** contains the complete PFE's for Central Park and Moonachie gauges.

Table B5-3: Available Rainfall Data

Gauge	Period of Records	Type
Moonachie	-	PFE[1]
TET	1997 - Present	Type 2
CPK	1948 - Present	Type 3, PFE

1. The PFE for Moonachie was developed by NOAA using methods described in NOAA Atlas 14 "Precipitation-Frequency Atlas of the United States" (NOAA 2006).

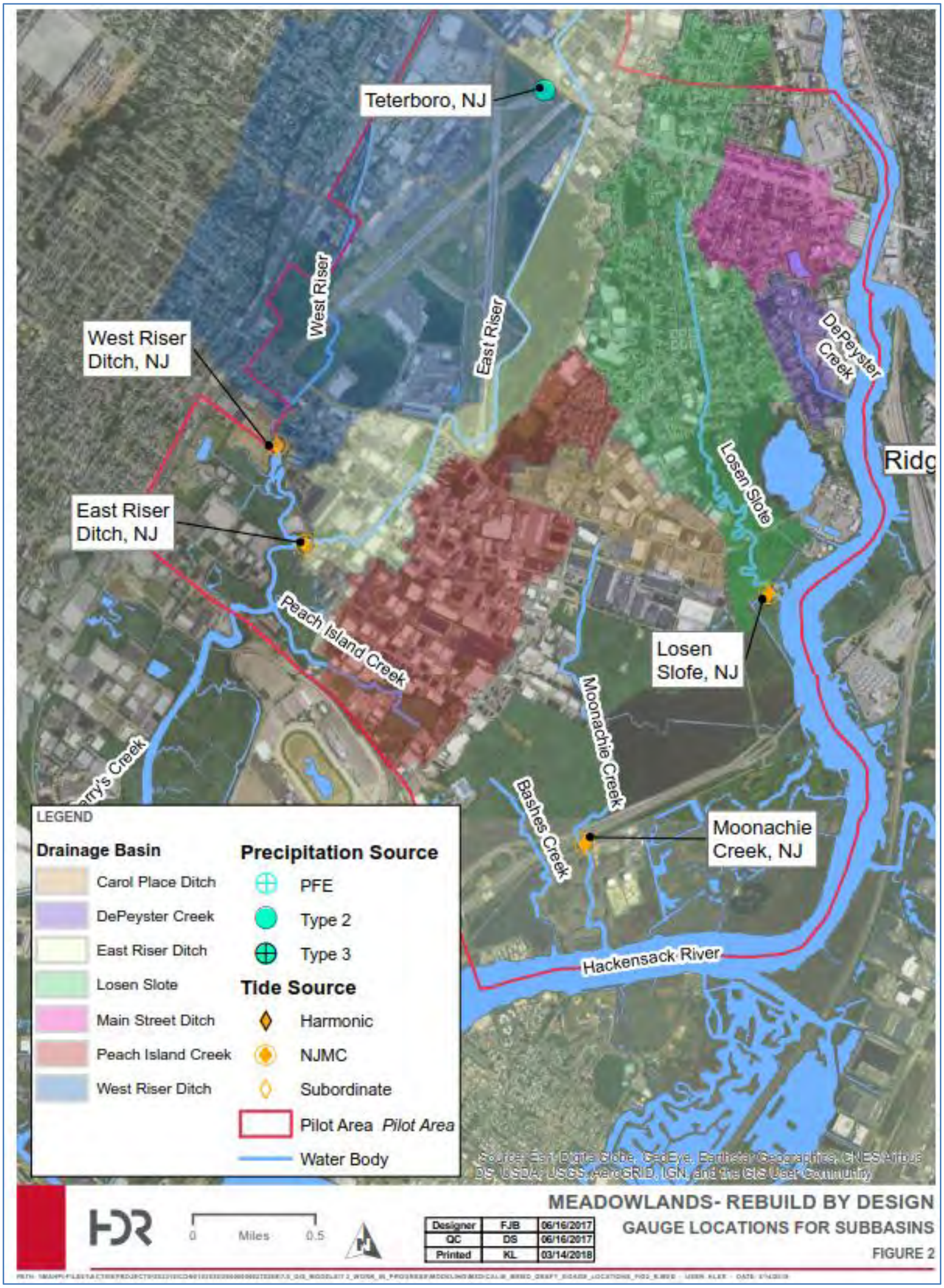

Figure B5-1: Gauge Locations for Sub-basins

Table B5-4: 24 hour Precipitation Frequency Estimates

Return Period (years)	24-Hour Precipitation (inches)	
	Moonachie / TET	CPK
2	3.31	3.60
5	4.23	4.70
10	5.01	5.61
25	6.18	6.86
50	7.18	7.83
100	8.28	8.79

The Project Area is located within a geographic region, consisting of the Gulf of Mexico area and Atlantic coastal areas, which is characterized by NRCS Type 3 rainfall distribution, or an area that experiences tropical storms that bring large 24-hour rainfall amounts (NRCS 1986). This storm pattern will be used for the Without and With scenarios.

4.0 Visual Observations and Data Recorded

Visual observations from photographs, newspapers, field visits during or after rainfall events, and other forms of media have been reviewed for the dates and locations shown in **Table B5-5**.

Figure B5-3 presents the locations for observations 1 through 29; photos showing flooding at these locations are included in **Appendix 3** to **Subappendix B5**. Based on the photos, estimates of flood elevations and the areal extent of inundation were made. The available photos were closely reviewed to determine the location and extent of the inundation shown in the photo; Google Street View and Bing Streetside were also used to locate the photo and identify the general location of the WSEL from the photo. A high resolution Light Detection and Ranging (LiDAR), which was derived from a digital elevation model (DEM) developed for the Meadowlands District from LiDAR flown in 2014, was then used to estimate the WSEL for each location based on the photos. The DEM was then used to show an area of inundation by shading all elevations less than or equal to the estimated elevation.

For observations 30 and 31, figures from the Draft “Berry’s Creek Study Area Remedial Investigation” were abstracted and assembled in **Appendix 4** to **Subappendix B5** (BCSA Cooperating PRP Group 2016).

For observations 32, refer to local news reporting from the Borough of Little Ferry on May 1, 2014, accessible at: http://abc7ny.com/weather/flooding-in-little-ferry-nj/39965/ (Freeze 2014).

For observations 33 and 34, refer to the Federal Emergency Management Agency National Flood Insurance Program claims in **Appendix 5** to **Subappendix B5**.

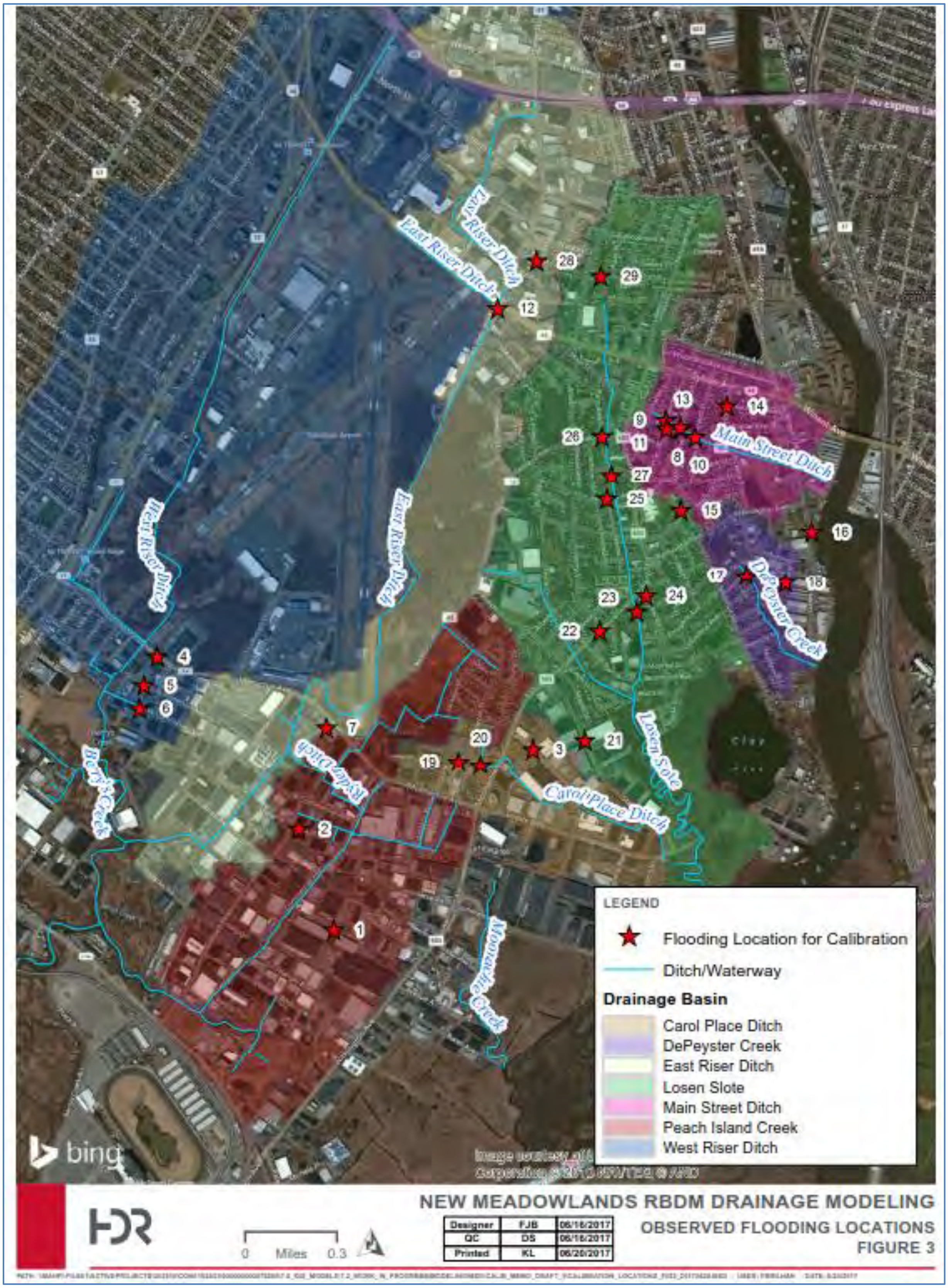

Figure B5-3: Map of Observed Flooding Locations in the Project Area for Calibration

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Table B5-5 - Flooding Observations

Observation Number	Location	Event Date	Total Rainfall (inches)[1]	Storm Length (hours)	Return Frequency Total Rainfall (years)[2]	Maximum Intensity (inches/hour)[1]	Return Frequency Max. Intensity (years)[2]	Peak Tide Level (feet NAVD 88)[3]	Model Area	Estimated Flooding Elevation (feet NAVD 88)	Source
1	140 Kero Road	7/8/2005	1.42	24	<1	0.40	<1	2.97	Peach Island Creek	4.5	A
2	110 Asia Place	6/10/2005	1.08	3	<1	0.71	<1	2.52	Peach Island Creek	3.75	A
3[4]	1 Carol Place	10/12/2005	3.87	24	2-5	0.55	<1	4.84	Carol Place	4.8	A
4	Grand Street and Moonachie Avenue	10/14/2005	7.14	77	10-25	0.55	<1	4.84	West Riser	3.4	A
5	Grand Street and Anderson Avenue	10/14/2005	7.14	77	10-25	0.55	<1	4.84	West Riser	3.4	A
6	Grand Street and Christina Avenue	10/14/2005	7.14	77	10-25	0.55	<1	4.84	M2	3.4	A
7	Avenue A and Moonachie Avenue	10/14/2005	7.14	77	10-25	0.55	<1	4.84	East Riser Ditch	4.5	A
8	Brandt Street	8/31/2014	0.77	6	<1	0.74	<1	2.55	Main Street	4.5	B
9	Garden Street	8/31/2014	0.77	6	<1	0.74	<1	2.55	Main Street	4.5	B
10	Grand Street	8/31/2014	0.77	6	<1	0.74	<1	2.55	Main Street	4.5	B
11	John Street	8/31/2014	0.77	6	<1	0.74	<1	2.55	Main Street	4.5	B
12	Riser Road	4/6/2017	0.83	13	<1	0.44	<1	4.28	East Riser Ditch	4.7	C
13	Main Street and Brandt Street	4/6/2017	0.83	13	<1	0.44	<1	4.28	Main Street	4.7	C
14	Kaufman Avenue & Frederick Street	4/6/2017	0.83	13	<1	0.44	<1	4.28	Main Street	4.7	C
15	Katherine Street	4/6/2017	0.83	13	<1	0.44	<1	4.28	DePeyster	4.7	C
16	Riverside Avenue	4/6/2017	0.83	13	<1	0.44	<1	4.28	DePeyster	4.7	C
17	Hartwick Street	4/6/2017	0.83	13	<1	0.44	<1	4.28	DePeyster	4.7	C
18	Industrial Avenue	4/6/2017	0.83	13	<1	0.44	<1	4.28	DePeyster	4.7	C
19	Parking lot, off Washington Place	4/6/2017	0.83	13	<1	0.44	<1	4.28	Carol Place	3.75	C
20	Parking lot, off Moonachie Road	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Carol Place	4.5	C
21	State Street	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Losen Slote Creek	4.5	C
22	Moonachie Road between West Park Street and Broad Street	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Losen Slote Creek	4.5	C
23	Adams Street	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Losen Slote Creek	2.5	C
24	Eckel Road	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Losen Slote Creek	4	C
25	Sabina Street	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Losen Slote Creek	4	C
26	Redneck Avenue and Paroubek Street	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Losen Slote Creek	4.75	C
27	William Street	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Losen Slote Creek	4.25	C
28	East Grove Street (1)	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Losen Slote Creek	4.75	C
29	East Grove Street (2)	5/5/2017	3.36	24	2-5	1.22	1-2	0.88	Losen Slote Creek	4.75	C
30	East and West Riser flow data	8/14/2011	3.62	27	2-5	0.69	<1	3.53	East and West Riser	-	D

Table B5-5 - Flooding Observations

Observation Number	Location	Event Date	Total Rainfall (inches)[1]	Storm Length (hours)	Return Frequency Total Rainfall (years)[2]	Maximum Intensity (inches/hour)[1]	Return Frequency Max. Intensity (years)[2]	Peak Tide Level (feet NAVD 88)[3]	Model Area	Estimated Flooding Elevation (feet NAVD 88)	Source
31	East and West Riser flow data	7/14/2014	1.90	25	<1	0.61	<1	3.85	East and West Riser	-	
32	Video - multiple locations	6/6/2013	3.48	31	2-5	0.43	<1	3.43	-	-	E
33	Multiple locations	9/16/1999	8.06	37	25-50	1.31	1-2	3.59	-	-	F
34	Multiple locations	4/15/2007	7.55	47	25-50	0.80	<1	4.93	-	-	

Notes:
1. Total rain at Teterboro Airport Point Gauge (TET).
2. Return frequency for storm length and peak intensity (1 hour) is based on point return frequency estimate for Moonachie, NJ.
3. Peak tide level at The Battery, NY.
4. Observation occurred in the middle of storm event. Total rainfall shown is at the end of 10/12/2005.

Source:
A. Hackensack Meadowlands Floodplain Management Plan (NJSEA 2005)
B. Main Street Drainage Study; Borough of Little Ferry, Bergen County, NJ; Drainage Evaluation Report (T&M Associates 2014)
C. HDR field visits
D. Draft Berry's Creek Study Area Remedial Investigation (BCSA Cooperating PRP Group 2016)
E. ABC 7 News (Freeze 2014)
F. Federal Emergency Management Agency National Flood Insurance Program Claims

5.0 Validation Event Recommendations

The five events shown in **Table B5-6** are potential validation events. The rainfall amounts are based on rainfall at Teterboro Airport. They were identified because they had associated visual observations of in-basin flooding and were not impacted by a large coastal surge. **Appendix 6** to **Subappendix B5** displays the observed tides, with tidal datums for reference. All storms fall within normal tidal condition and are not influenced by coastal storm conditions.

Table B5-6 - Recommended Validation Events

Storm ID	Date	Total Rainfall (inches)	Storm Length (hours)	Return Frequency (years)	Maximum Intensity (inches/ hour)	Return Frequency (years)	Area of Visual Observation
1	8/14/2011	3.57	24	2-5	0.69	<1	West/East Riser Ditch flow data
2	7/14/2014	1.90	25	<1	0.61	<1	West/East Riser Ditch flow data
3	6/6/2013	3.48	31	2-5	0.43	<1	Main Street
4	8/31/2014	0.77	6	<1	0.74	<1	Main Street
5	4/6/2017	0.79	11	<1	0.44	<1	Main Street, DePeyster Creek, East Riser Ditch

6.0 Fluvial Modeling Scenarios

Several conditions for which drainage improvements would be assessed were identified: varying magnitude rainfall events, existing coastal conditions and those affected by sea level rise (SLR), and rainfall events with coincident coastal storm surge. A set of rainfall return periods were selected to evaluate system performance under high frequency / low magnitude and low frequency / high magnitude events. The following rainfall return periods were selected: 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year. SLR associated with 2075, as estimated by NOAA, was selected for the future coastal condition and the 2-year coincident coastal storm surge was selected. A value of 1.2 feet was applied at the outer coastal modeling boundary to simulate the effects of SLR and to develop a value appropriate for the fluvial downstream boundary locations immediately adjacent to the Project Area. See **Appendix B1** for an explanation of the amount of SLR incorporated into the modeling efforts.

Together, these conditions translate to 48 existing and proposed conditions for each concept evaluated, as shown on the **Table B5-7**.

Table B5-7 - Model Scenarios Runs for Evaluating Effectiveness for Drainage Improvements

	Existing Condition						Proposed Condition					
General Condition	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
2016 Meteorology, Coastal Conditions and Normal Tide	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
2075 Meteorology, Coastal Conditions and Normal Tide	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
2016 Meterology, Coastal Conditions and Storm Surge	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
2075 Meterology, Coastal Conditions and Storm Surge	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√
	√	√	√	√	√	√	√	√	√	√	√	√

7.0 Rainfall and Tidal Boundary Conditions for Drainage Improvement Modeling

7.1 2016 Rainfall

All specific frequency hypothetical point rainfall depths were taken from NOAA Atlas 14 “Precipitation-Frequency Atlas of the United States” (NOAA 2006). Hypothetical point rainfall depths for the 1 through 500 year storms are shown in **Table B5-8**. A 48-hour hypothetical storm was used to allow for Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) interior inflow routing against the exterior time-varying marigrams (astronomic tide plus storm surge) through six tide cycles.

7.2 2075 Rainfall

All specific frequency hypothetical point rainfall depths were taken from NOAA Atlas 14 “Precipitation-Frequency Atlas of the United States” (NOAA 2006). Hypothetical point rainfall depths for the 1 through 500 year storms are shown in **Table B5-9**.

7.3 Tidal Boundary Conditions

Tidal datums and model-generated tide stage time series were applied as part of the fluvial modeling effort. Tidal datums were used for the simplified modeling effort, and the preliminary detailed model runs. Stage time series generated from the MIKE 21 model were used for the final detailed fluvial modeling efforts. **Table B5-10** provides a summary of the tidal data applied in these efforts and associated sources.

Table B5-8 - Specific Frequency Hypothetical Point Rainfall Depths in Inches

Duration	Return Period							
	2 Year Rainfall [in]	5 Year Rainfall [in]	10 Year Rainfall [in]	25 Year Rainfall [in]	50 Year Rainfall [in]	100 Year Rainfall [in]	250 Year Rainfall [in]	500 Year Rainfall [in]
5 min.	0.40	0.48	0.53	0.61	0.66	0.71	0.77	0.82
15 min.	0.79	0.95	1.06	1.19	1.29	1.39	1.50	1.59
1 hour	1.35	1.69	1.95	2.3	2.57	2.85	3.18	3.50
2 hours	1.66	2.1	2.44	2.91	3.29	3.69	4.20	4.67
3 hours	1.85	2.34	2.73	3.26	3.7	4.15	4.74	5.29
6 hours	2.39	3.01	3.5	4.2	4.78	5.38	6.17	6.93
12 hours	2.94	3.72	4.35	5.27	6.05	6.87	7.99	9.07
24 hours	3.31	4.23	5.01	6.18	7.18	8.28	9.80	11.30
48 hours	3.87	4.95	5.85	7.18	8.32	9.57	11.25	13.0

Table B5-9 - Specific Frequency Hypothetical Point Rainfall Depths in Inches - Future

Duration	Return Period							
	2 Year Rainfall [in]	5 Year Rainfall [in]	10 Year Rainfall [in]	25 Year Rainfall [in]	50 Year Rainfall [in]	100 Year Rainfall [in]	250 Year Rainfall [in]	500 Year Rainfall [in]
5 min.	0.44	0.54	0.62	0.74	0.87	1.03	1.11	1.19
15 min.	0.87	1.06	1.23	1.46	1.7	2.02	2.17	2.31
1 hour	1.49	1.89	2.26	2.83	3.39	4.13	4.62	5.08
2 hours	1.83	2.35	2.83	3.58	4.34	5.35	6.08	6.77
3 hours	2.04	2.62	3.17	4.01	4.88	6.02	6.87	7.67
6 hours	2.63	3.37	4.06	5.17	6.31	7.8	8.95	10.05
12 hours	3.23	4.17	5.05	6.48	7.99	9.96	11.58	13.15
24 hours	3.64	4.74	5.81	7.6	9.48	12.01	14.21	16.39
48 hours	4.26	5.54	6.79	8.83	10.98	13.88	16.31	18.85

Table B5-10 - Tidal Data Applied in Fluvial Modeling (All Elevations Reference NAVD 88)

HDR				
HEC-RAS one-dimensional (1D) and InfoWorks				
Use	**Datum**	**Value Used**	**Modelled Location**	**Source**
Simplified Modeling	MHW	3.03	East Riser Ditch, West Riser Ditch, Peach Island Creek	Woods Hole Group, 2007 (NJMC Station 10 - Observed)[1]
	MHW	2.86	Losen Slote Creek, DePeyster Creek	Woods Hole Group, 2007 (NJMC Station 9 - Observed)[1]

HDR				
HEC-RAS one-dimensional (1D) and InfoWorks				
Use	**Datum**	**Value Used**	**Modelled Location**	**Source**
	MLW	-2.48	East Riser Ditch, West Riser Ditch, Peach Island Creek	Woods Hole Group, 2007 (NJMC Station 10 - Observed)[1]
	MLW	-2.77	Losen Slote Creek, DePeyster Creek	Woods Hole Group, 2007 (NJMC Station 9 - Observed)[1]
HEC-RAS 2D and InfoWorks				
Detailed Modeling	MHW	2.85	East Riser Ditch, West Riser Ditch	AECOM - Modified Range Ratio Method[2]
	MHW	2.66	Losen Slote Creek	AECOM - Linear Interpolation of NOAA station datums
	MHW	2.69	DePeyster Creek	AECOM - Linear Interpolation of NOAA station datums
	MSL	0.55	East Riser Ditch, West Riser Ditch	AECOM - Modified Range Ratio Method[2]
	MSL	-0.03	Losen Slote Creek	AECOM - Linear Interpolation of NOAA station datums
	MSL	-0.02	DePeyster Creek	AECOM - Linear Interpolation of NOAA station datums
	MLW	-2.49	East Riser Ditch, West Riser Ditch	AECOM - Modified Range Ratio Method[2]
	MLW	-3.14	Losen Slote Creek	AECOM - Linear Interpolation of NOAA station datums
	MLW	-3.17	DePeyster Creek	AECOM - Linear Interpolation of NOAA station datums

1. (Woods Hole Group 2007)
2. (NOAA 2003)

8.0 Thresholds for Evaluation of Effectiveness of Drainage Improvements

Two thresholds were defined to evaluate the effectiveness of drainage improvements when concepts were being developed and screened: (1) incidence of floodwater contacting the foundation of a structure, and (2) incidence of the WSEL exceeding the first floor elevation of a structure. The Proposed Project's cost/benefit analysis reported in the Draft Feasibility Study Report contains additional information on how these thresholds were utilized.

9.0 References

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Appendix 1 to Subappendix B5

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(http://tidesandcurrents.noaa.gov/datums.html?id=8518750#)

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OFS (http://tidesandcurrents.noaa.gov/ofs/ofs_station.shtml?stname=The Battery&ofs=ny&stnid=8518750&subdomain=0)

Datums for 8518750, The Battery NY

Elevations on Station Datum

Station: 8518750, The Battery, NY
Status: Accepted (Nov 19 2012)
Units: Feet
T.M.: 75
Epoch: (http://tidesandcurrents.noaa.gov/datum_options.html#NTDE) 1983-2001
Datum: STND

Datum	Value	Description
MHHW (http://tidesandcurrents.noaa.gov/datum_options.html#MHHW)	8.34	Mean Higher-High Water
MHW (http://tidesandcurrents.noaa.gov/datum_options.html#MHW)	8.02	Mean High Water
MTL (http://tidesandcurrents.noaa.gov/datum_options.html#MTL)	5.76	Mean Tide Level
MSL (http://tidesandcurrents.noaa.gov/datum_options.html#MSL)	5.86	Mean Sea Level
DTL	5.82	Mean Diurnal Tide

(http://tidesandcurrents.noaa.gov/datum_options.html#DTL)		Level
MLW (http://tidesandcurrents.noaa.gov/datum_options.html#MLW)	3.49	Mean Low Water
MLLW (http://tidesandcurrents.noaa.gov/datum_options.html#MLLW)	3.29	Mean Lower-Low Water
NAVD88 (http://tidesandcurrents.noaa.gov/datum_options.html)	6.06	North American Vertical Datum of 1988
STND (http://tidesandcurrents.noaa.gov/datum_options.html#STND)	0.00	Station Datum
GT (http://tidesandcurrents.noaa.gov/datum_options.html#GT)	5.06	Great Diurnal Range
MN (http://tidesandcurrents.noaa.gov/datum_options.html#MN)	4.53	Mean Range of Tide
DHQ (http://tidesandcurrents.noaa.gov/datum_options.html#DHQ)	0.32	Mean Diurnal High Water Inequality
DLQ (http://tidesandcurrents.noaa.gov/datum_options.html#DLQ)	0.21	Mean Diurnal Low Water Inequality
HWI (http://tidesandcurrents.noaa.gov/datum_options.html#HWI)	0.84	Greenwich High Water Interval (in hours)
LWI (http://tidesandcurrents.noaa.gov/datum_options.html#LWI)	7.21	Greenwich Low Water Interval (in hours)
Maximum	17.33	Highest Observed Water Level
Max Date & Time	10/30/2012 01:12	Highest Observed Water Level Date and Time
Minimum	-1.00	Lowest Observed Water Level
Min Date & Time	02/02/1976 21:30	Lowest Observed Water Level Date and Time
HAT (http://tidesandcurrents.noaa.gov/datum_options.html#HAT)	9.64	Highest Astronomical Tide
HAT Date & Time	10/16/1993 13:12	HAT Date and Time
LAT (http://tidesandcurrents.noaa.gov/datum_options.html#LAT)	1.90	Lowest Astronomical Tide

LAT Date & Time	01/21/1996 20:06	LAT Date and Time

Tidal Datum Analysis Periods

01/01/1983 - 12/31/2001

Showing datums for

Data Units Feet
Meters

Epoch Present (1983-2001)
Superseded (1960-1978)

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Show nearby stations

Products available at 8518750 The Battery, NY

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OFS product page for The Battery (http://tidesandcurrents.noaa.gov/ofs/ofs_station.shtml?stname=The Battery&ofs=ny&stnid=8518750&subdomain=0)

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Revised: 10/15/2013
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Web site owner: Center for Operational Oceanographic Products and Services
<p></p>

Appendix 2 to Subappendix B5

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NOAA Atlas 14, Volume 2, Version 3
Location name: Moonachie, New Jersey, USA*
Latitude: 40.8396°, Longitude: -74.0556°
Elevation: 4.52 ft**
* source: ESRI Maps
** source: USGS

POINT PRECIPITATION FREQUENCY ESTIMATES

G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.336 (0.307-0.368)	0.401 (0.366-0.440)	0.478 (0.435-0.525)	0.533 (0.484-0.584)	0.605 (0.547-0.664)	0.658 (0.591-0.722)	0.708 (0.633-0.779)	0.755 (0.670-0.832)	0.819 (0.717-0.908)	0.867 (0.752-0.966)
10-min	0.532 (0.486-0.584)	0.636 (0.580-0.698)	0.757 (0.689-0.831)	0.843 (0.766-0.925)	0.950 (0.858-1.04)	1.03 (0.924-1.13)	1.10 (0.986-1.21)	1.17 (1.04-1.29)	1.26 (1.11-1.40)	1.33 (1.15-1.48)
15-min	0.661 (0.603-0.725)	0.793 (0.723-0.871)	0.947 (0.862-1.04)	1.06 (0.959-1.16)	1.19 (1.08-1.31)	1.29 (1.16-1.42)	1.39 (1.24-1.53)	1.48 (1.31-1.63)	1.59 (1.39-1.76)	1.66 (1.44-1.85)
30-min	0.895 (0.817-0.981)	1.09 (0.989-1.19)	1.33 (1.21-1.46)	1.51 (1.37-1.66)	1.74 (1.57-1.91)	1.91 (1.72-2.10)	2.09 (1.86-2.29)	2.25 (2.00-2.48)	2.47 (2.16-2.74)	2.62 (2.28-2.92)
60-min	1.11 (1.01-1.22)	1.35 (1.23-1.48)	1.69 (1.54-1.86)	1.95 (1.77-2.14)	2.30 (2.08-2.52)	2.57 (2.31-2.82)	2.85 (2.54-3.13)	3.12 (2.77-3.44)	3.50 (3.06-3.88)	3.79 (3.29-4.22)
2-hr	1.37 (1.24-1.51)	1.66 (1.51-1.83)	2.10 (1.91-2.32)	2.44 (2.21-2.69)	2.91 (2.62-3.21)	3.29 (2.95-3.63)	3.69 (3.28-4.07)	4.10 (3.61-4.53)	4.67 (4.06-5.18)	5.12 (4.40-5.71)
3-hr	1.53 (1.39-1.68)	1.85 (1.69-2.04)	2.34 (2.13-2.58)	2.73 (2.48-3.00)	3.26 (2.94-3.59)	3.70 (3.32-4.07)	4.15 (3.69-4.57)	4.63 (4.08-5.10)	5.29 (4.60-5.85)	5.82 (5.00-6.47)
6-hr	1.98 (1.81-2.17)	2.39 (2.19-2.63)	3.01 (2.75-3.30)	3.50 (3.19-3.83)	4.20 (3.80-4.59)	4.78 (4.29-5.23)	5.38 (4.79-5.89)	6.02 (5.31-6.61)	6.93 (6.03-7.63)	7.67 (6.59-8.48)
12-hr	2.43 (2.21-2.68)	2.94 (2.68-3.25)	3.72 (3.38-4.10)	4.35 (3.94-4.79)	5.27 (4.73-5.79)	6.05 (5.39-6.63)	6.87 (6.06-7.54)	7.77 (6.78-8.54)	9.07 (7.77-10.0)	10.1 (8.57-11.2)
24-hr	2.74 (2.53-2.99)	3.31 (3.06-3.62)	4.23 (3.90-4.61)	5.01 (4.60-5.45)	6.18 (5.63-6.71)	7.18 (6.49-7.79)	8.28 (7.42-8.99)	9.50 (8.43-10.3)	11.3 (9.87-12.4)	12.9 (11.1-14.1)
2-day	3.20 (2.94-3.52)	3.87 (3.55-4.27)	4.95 (4.52-5.44)	5.85 (5.33-6.42)	7.18 (6.50-7.87)	8.32 (7.48-9.12)	9.57 (8.53-10.5)	10.9 (9.65-12.1)	13.0 (11.2-14.4)	14.7 (12.5-16.4)
3-day	3.37 (3.10-3.69)	4.08 (3.76-4.46)	5.19 (4.77-5.67)	6.12 (5.61-6.68)	7.49 (6.81-8.16)	8.64 (7.81-9.42)	9.90 (8.88-10.8)	11.3 (10.0-12.4)	13.3 (11.6-14.7)	15.0 (12.9-16.6)
4-day	3.54 (3.27-3.85)	4.29 (3.96-4.66)	5.44 (5.02-5.91)	6.40 (5.89-6.94)	7.79 (7.12-8.45)	8.97 (8.14-9.73)	10.2 (9.23-11.1)	11.6 (10.4-12.7)	13.6 (12.0-14.9)	15.3 (13.3-16.9)
7-day	4.16 (3.87-4.50)	5.01 (4.65-5.41)	6.24 (5.79-6.74)	7.26 (6.72-7.83)	8.74 (8.04-9.41)	9.97 (9.11-10.7)	11.3 (10.2-12.2)	12.7 (11.4-13.7)	14.7 (13.0-16.0)	16.4 (14.3-17.9)
10-day	4.75 (4.44-5.12)	5.68 (5.31-6.12)	6.98 (6.51-7.51)	8.05 (7.48-8.65)	9.58 (8.85-10.3)	10.8 (9.96-11.7)	12.2 (11.1-13.1)	13.6 (12.3-14.7)	15.6 (13.9-16.9)	17.2 (15.2-18.8)
20-day	6.42 (6.02-6.85)	7.62 (7.15-8.13)	9.11 (8.54-9.72)	10.3 (9.62-11.0)	11.9 (11.1-12.7)	13.1 (12.2-14.0)	14.4 (13.3-15.3)	15.6 (14.4-16.7)	17.3 (15.8-18.6)	18.7 (16.9-20.2)
30-day	8.02 (7.56-8.51)	9.48 (8.93-10.1)	11.1 (10.5-11.8)	12.3 (11.6-13.1)	14.0 (13.1-14.8)	15.2 (14.2-16.2)	16.4 (15.3-17.5)	17.6 (16.3-18.7)	19.1 (17.6-20.5)	20.3 (18.6-21.8)
45-day	10.2 (9.65-10.8)	12.0 (11.3-12.7)	13.9 (13.1-14.7)	15.3 (14.4-16.2)	17.1 (16.1-18.1)	18.5 (17.4-19.6)	19.8 (18.6-21.0)	21.1 (19.7-22.4)	22.7 (21.1-24.1)	23.9 (22.1-25.5)
60-day	12.2 (11.6-12.9)	14.4 (13.6-15.1)	16.4 (15.6-17.3)	18.0 (17.0-18.9)	19.9 (18.8-21.0)	21.3 (20.1-22.5)	22.7 (21.3-23.9)	23.9 (22.4-25.3)	25.5 (23.8-27.0)	26.5 (24.7-28.2)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

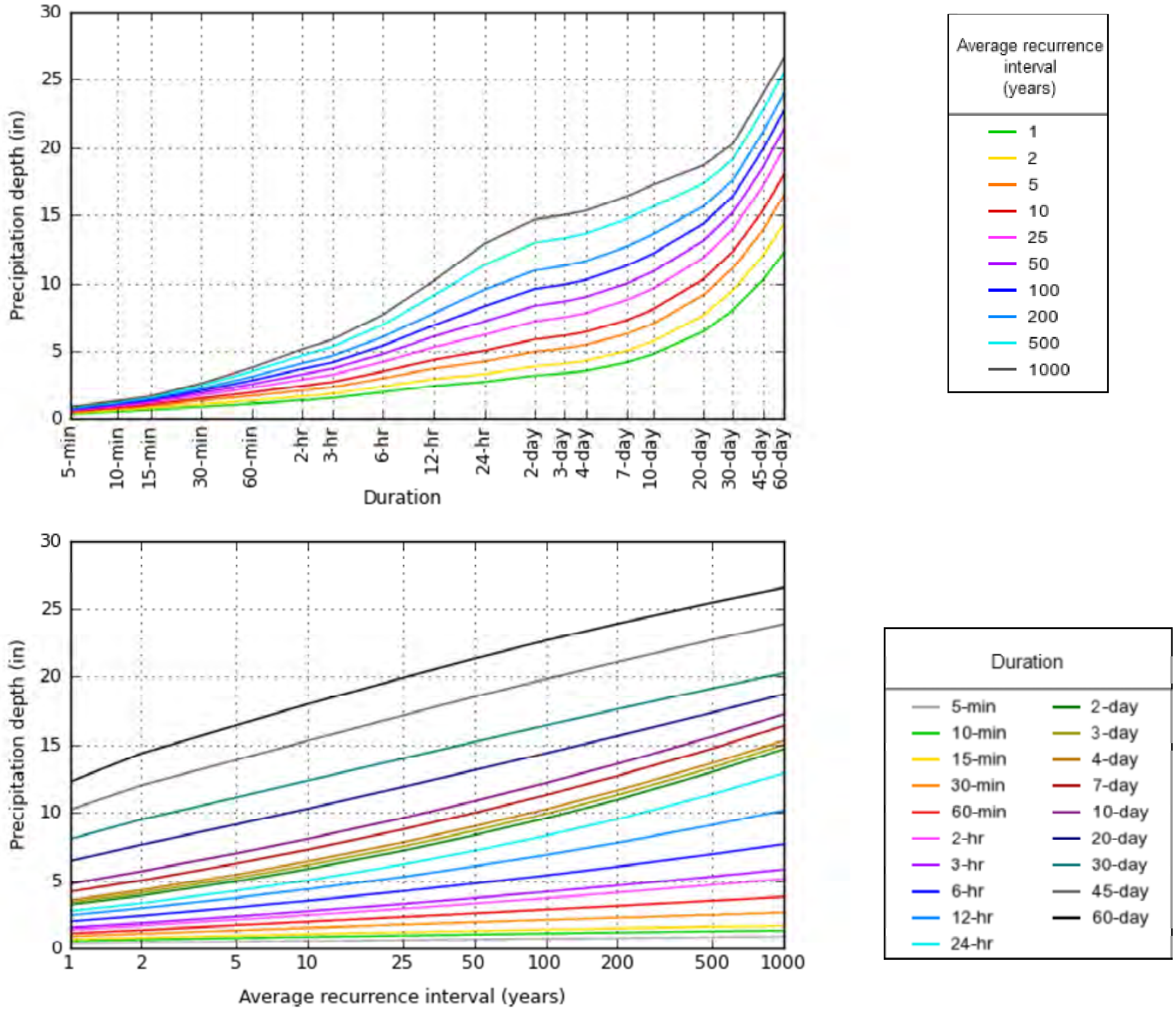
Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

Back to Top

PF graphical

PDS-based depth-duration-frequency (DDF) curves
Latitude: 40.8396°, Longitude: -74.0556°



NOAA Atlas 14, Volume 2, Version 3 Created (GMT): Mon Oct 10 15:10:06 2016

Back to Top

Maps & aerials

Small scale terrain

Large scale terrain

Large scale map

Large scale aerial

Back to Top

US Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service
National Water Center
1325 East West Highway
Silver Spring, MD 20910
Questions?: HDSC.Questions@noaa.gov

Disclaimer

Appendix 3 to Subappendix B5

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Kero Rd, Observation #1

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
7/8/2005	24	1.42	<1	0.41	<1	3.12

MEADOWLANDS RBDM DRAINAGE MODELING

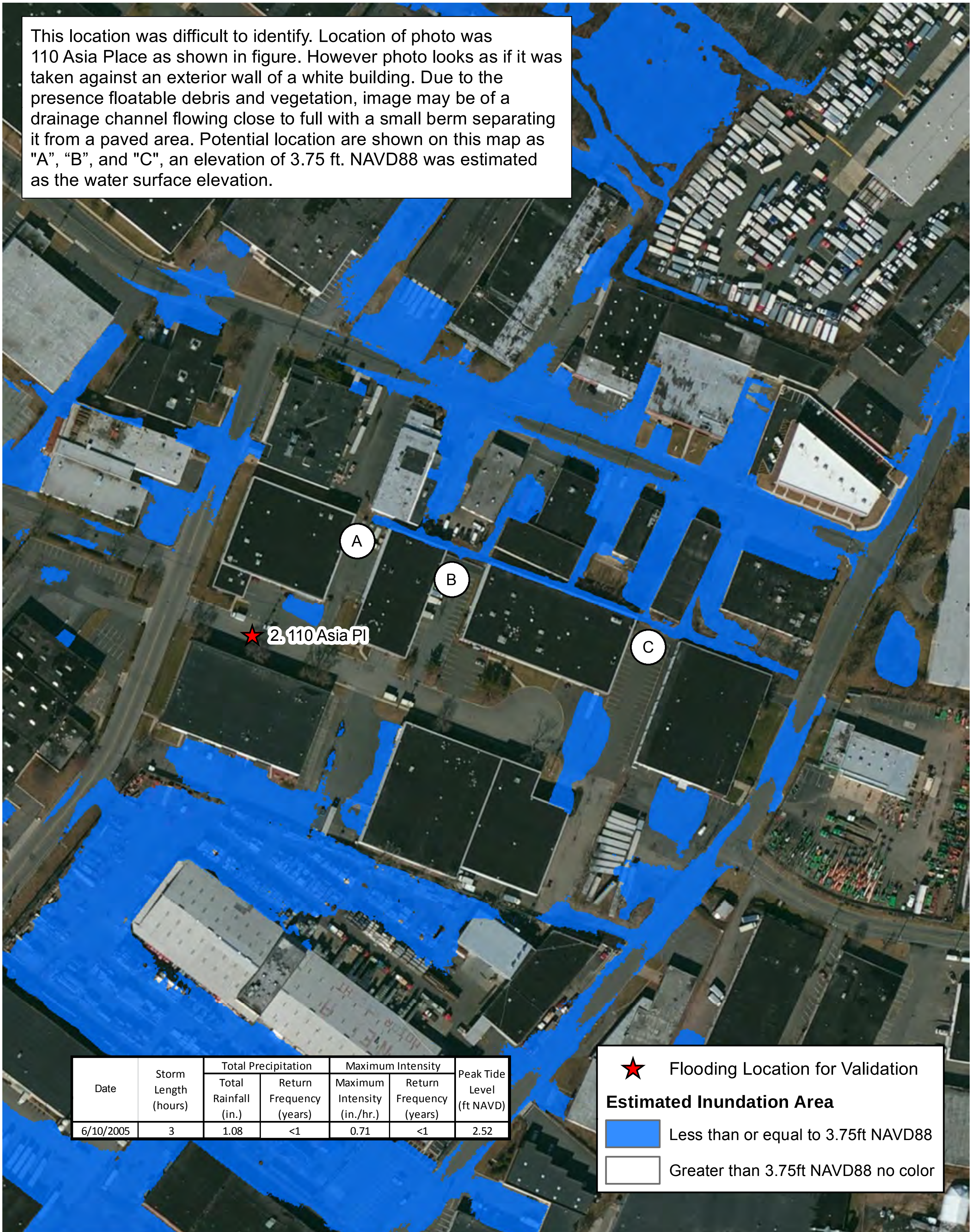
OBSERVED FLOODING LOCATION
1. 140 KERO RD CARLSTADT

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_1_KERORD.MXD - USER: KLEE - DATE: 6/23/2017

Asia Pl, Observation #2

This location was difficult to identify. Location of photo was 110 Asia Place as shown in figure. However photo looks as if it was taken against an exterior wall of a white building. Due to the presence floatable debris and vegetation, image may be of a drainage channel flowing close to full with a small berm separating it from a paved area. Potential location are shown on this map as "A", "B", and "C", an elevation of 3.75 ft. NAVD88 was estimated as the water surface elevation.



Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
6/10/2005	3	1.08	<1	0.71	<1	2.52

★ Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 3.75ft NAVD88

Greater than 3.75ft NAVD88 no color

HDR

0 Feet 200

N

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATION
2. 110 ASIA PL CARLSTADT

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_2_ASIAPL.MXD - USER: KLEE - DATE: 6/23/2017

View of flooding in the parking lot at 1 Carol Place (10/12/05)

View of flooding at 1 Carol Place (10/12/05)

Carol Pl, Observation #3

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
10/12/2005	24	3.87	2-5	0.55	<1	4.84

0 Feet 100

MEADOWLANDS RBDM DRAINAGE MODELING

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

OBSERVED FLOODING LOCATION
3. 1 CAROL PL MOONACHIE

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\0000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_3_CAROLPL.MXD - USER: KLEE - DATE: 6/23/2017

. Flooding at the intersection of Grand Street and Moonachie Road (10/14/05)

Grand St Moonachie Ave, Observation #4

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
10/14/2005	77	7.14	10-25	0.55	<1	4.84

★ Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 3.4ft NAVD88

Greater than 3.4ft NAVD88 no color

0 Feet 100

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATION
4. GRAND ST AND MOONACHIE AVE MOONACHIE

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\0000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_4_GRANDST&MOONACHIEAVE.MXD - USER: KLEE - DATE: 6/23/2017

Observation #5

Flooding at the intersection of Grand Street and Anderson Avenue (10/14/05)

Flooding at the intersection of Grand Street and Anderson Avenue (10/14/05)

Observation #6

Flooding at the intersection of Grand Street and Christiana Avenue (10/14/05)

Grand St and Christina Ave at Anderson Ave, Observations #5 and #6

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
10/14/2005	77	7.14	10-25	0.55	<1	4.84

HDR

0 Feet 100

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING
OBSERVED FLOODING LOCATIONS MOONACHIE
5. GRAND ST AND ANDERSON AVE
6. GRAND ST AND CHRISTINA AVE

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_5&6_GRANDST&ANDERSONAVE_CHRISTINA.MXD - USER: KLEE - DATE: 6/23/2017

Photos for Avenue A & Moonachie Avenue, Moonachie

1. Looking southwest from the entrance to the mobile home area (10/14/05)

3. Flooding condition along the edge of the mobile home area (10/14/05)

2. Flooding condition at Southeast of the mobile home area (10/14/05)

4. Flooding over the roadway in the mobile home area (10/14/05)

Avenue A & Moonachie Ave, Observation #7

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
10/14/2005	77	7.14	10-25	0.55	<1	4.84

0 Feet 100

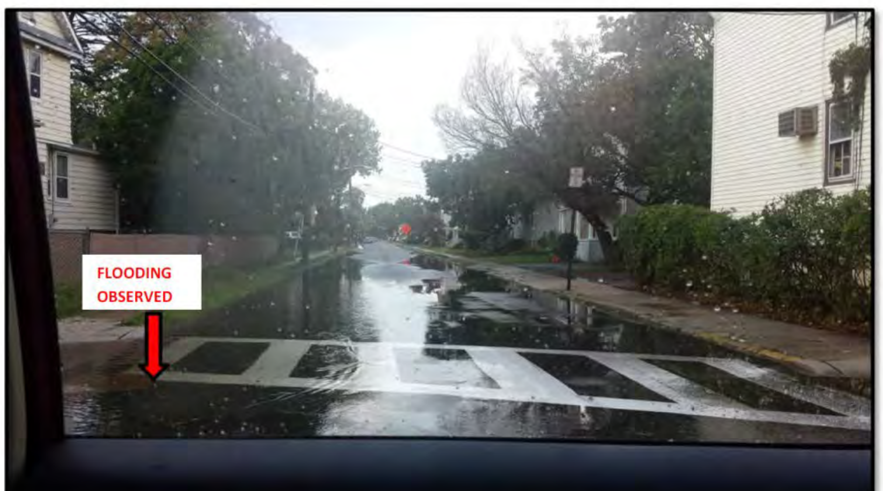
Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATION
7. AVENUE A AND MOONACHIE AVE MOONACHIE

Observation #8 **Flooding Observed on Brandt Street after Aug 31, 2014 Rainfall Event**

Observation #9 **Flooding Observed on Garden Street after Aug 31, 2014 Rainfall Event**



Observation #10 **Flooding Observed on Grand Street after Aug 31, 2014 Rainfall Event**

Observation #11 **Flooding Observed on John Street after Aug 31, 2014 Rainfall Event**

Brandt St, Garden St, Grand St, John St, at Main St, Observations #8, #9, #10, and #11

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
8/31/2014	6	0.77	<1	0.74	<1	2.55

★ Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 4.5ft NAVD88

Greater than 4.5ft NAVD88 no color

0 Feet 150

N

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
8-11. MAIN ST LITTLE FERRY

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_8-11_MAINST_LITTLEFERRY_R.MXD - USER: KLEE - DATE: 6/23/2017

Riser Rd, East Riser, Observation #12

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
4/6/2017	13	0.83	<1	0.44	<1	4.28

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
12. RISER RD LITTLE FERRY

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_12_RISERRD.MXD - USER: KLEE - DATE: 6/23/2017

Main St & Brandt St, Observation #13

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
4/6/2017	13	0.83	<1	0.44	<1	4.28

★ Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 4.7ft NAVD88

Greater than 4.7ft NAVD88 no color

0 Feet 150

N

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
13. MAIN ST & BRANDT ST LITTLE FERRY

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_13_MAINST&BRANDTST.MXD - USER: KLEE - DATE: 6/23/2017

Kaufman Ave & Frederick St, Observation #14

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
4/6/2017	13	0.83	<1	0.44	<1	4.28

Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 4.7ft NAVD88

Greater than 4.7ft NAVD88 no color

0 Feet 150

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS

14. KAUFMAN AVE & FREDERICK ST LITTLE FERRY

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_14_KAUFMANAVET.MXD - USER: KLEE - DATE: 6/23/2017

Katherine St, Observation #15

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
4/6/2017	13	0.83	<1	0.44	<1	4.28

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
15. KATHERINE ST LITTLE FERRY

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\0000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_15_KATHERINEST.MXD - USER: KLEE - DATE: 6/23/2017

Riverside Ave, Observation #16

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
4/6/2017	13	0.83	<1	0.44	<1	4.28

★ Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 4.7ft NAVD88

Greater than 4.7ft NAVD88 no color

0 Feet 150

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
16. RIVERSIDE AVE LITTLE FERRY

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\0000000002702888\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_16_RIVERSIDE.MXD - USER: KLEE - DATE: 6/23/2017

Hartwick St, Observation #17

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
4/6/2017	13	0.83	<1	0.44	<1	4.28

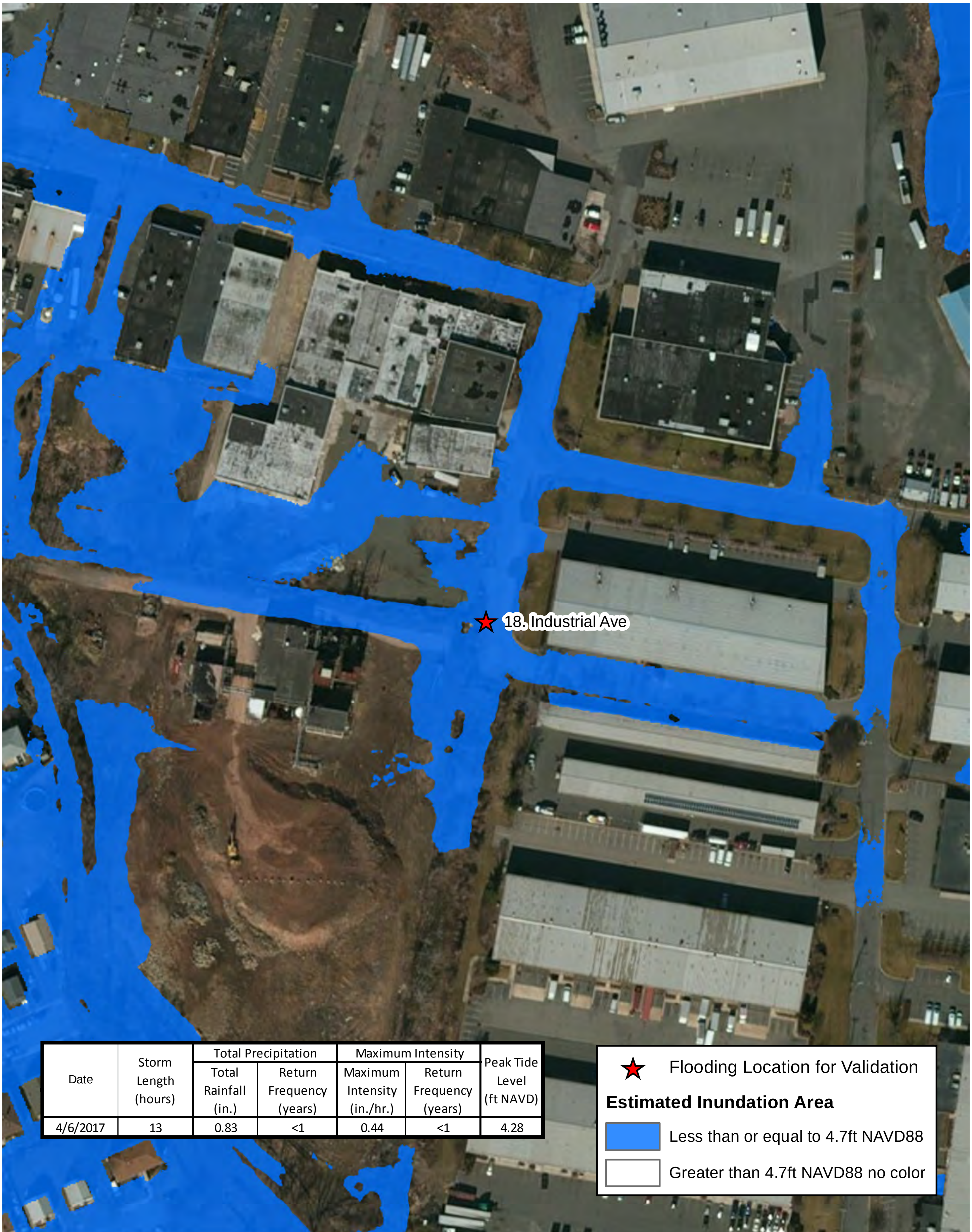
MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
17. HARTWICK ST LITTLE FERRY

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\0000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_17_HARTWICKST.MXD - USER: KLEE - DATE: 6/23/2017

Industrial Ave, Observation #18



Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
4/6/2017	13	0.83	<1	0.44	<1	4.28

★ Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 4.7ft NAVD88

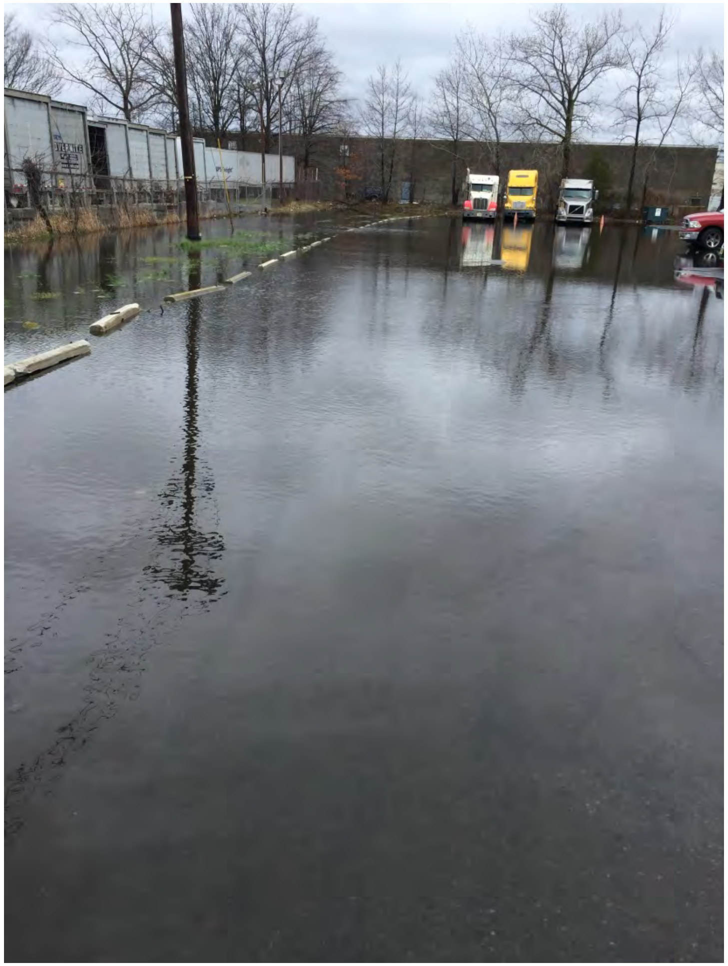
Greater than 4.7ft NAVD88 no color

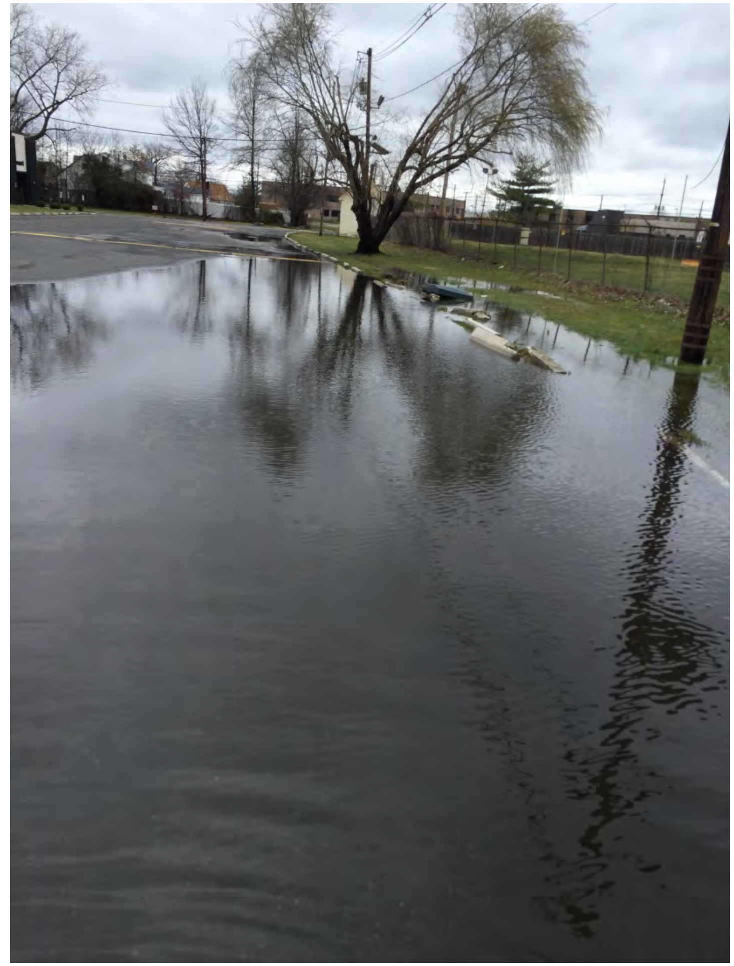
0 Feet 150

MEADOWLANDS RBDM DRAINAGE MODELING

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

OBSERVED FLOODING LOCATIONS
18. INDUSTRIAL AVE LITTLE FERRY





Parking lot, off Washington Pl, Observation #19

Date	Storm Length (hours)	Total Precipitation: Total Rainfall (in.)	Total Precipitation: Return Frequency (years)	Maximum Intensity: Maximum Intensity (in./hr.)	Maximum Intensity: Return Frequency (years)	Peak Tide Level (ft NAVD)
4/6/2017	13	0.83	<1	0.44	<1	4.28

0 Feet 150

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS

19. PARKING LOT, OFF WASHINGTON PL MOONACHIE

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_19_WASHINGTONST.MXD - USER: KLEE - DATE: 6/23/2017

Parking lot, off Moonachie Rd, Observation #20

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

HDR

0 Feet 100

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS

20. PARKING LOT, OFF MOONACHIE RD MOONACHIE

State St., Observation #21

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

★ Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 4.5ft NAVD88

Greater than 4.5ft NAVD88 no color

0 Feet 100

MEADOWLANDS RBDM DRAINAGE MODELING

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

OBSERVED FLOODING LOCATIONS
21. STATE ST MOONACHIE

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_21_STATEST.MXD - USER: KLEE - DATE: 6/23/2017

Moonachie Rd, Between West Park St and Broad St, Observation #22

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

★ Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 4.5ft NAVD88

Greater than 4.5ft NAVD88 no color

HDR

N

0 Feet 100

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

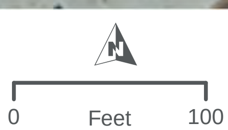
OBSERVED FLOODING LOCATIONS

22. MOONACHIE RD MOONACHIE

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_22_MOONACHIERD.MXD - USER: KLEE - DATE: 6/23/2017

Adams St, Observation #23

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88



Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
23. ADAMS ST LITTLE FERRY

Eckel Rd, Observation #24

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
24. ECKEL RD LITTLE FERRY

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\0000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_24_ECKELRD.MXD - USER: KLEE - DATE: 6/23/2017

Sabina St, Observation #25

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
25. SABINA ST LITTLE FERRY

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_25_SABINAST.MXD - USER: KLEE - DATE: 6/23/2017

Redneck Ave and Paroubek St, Observation #26

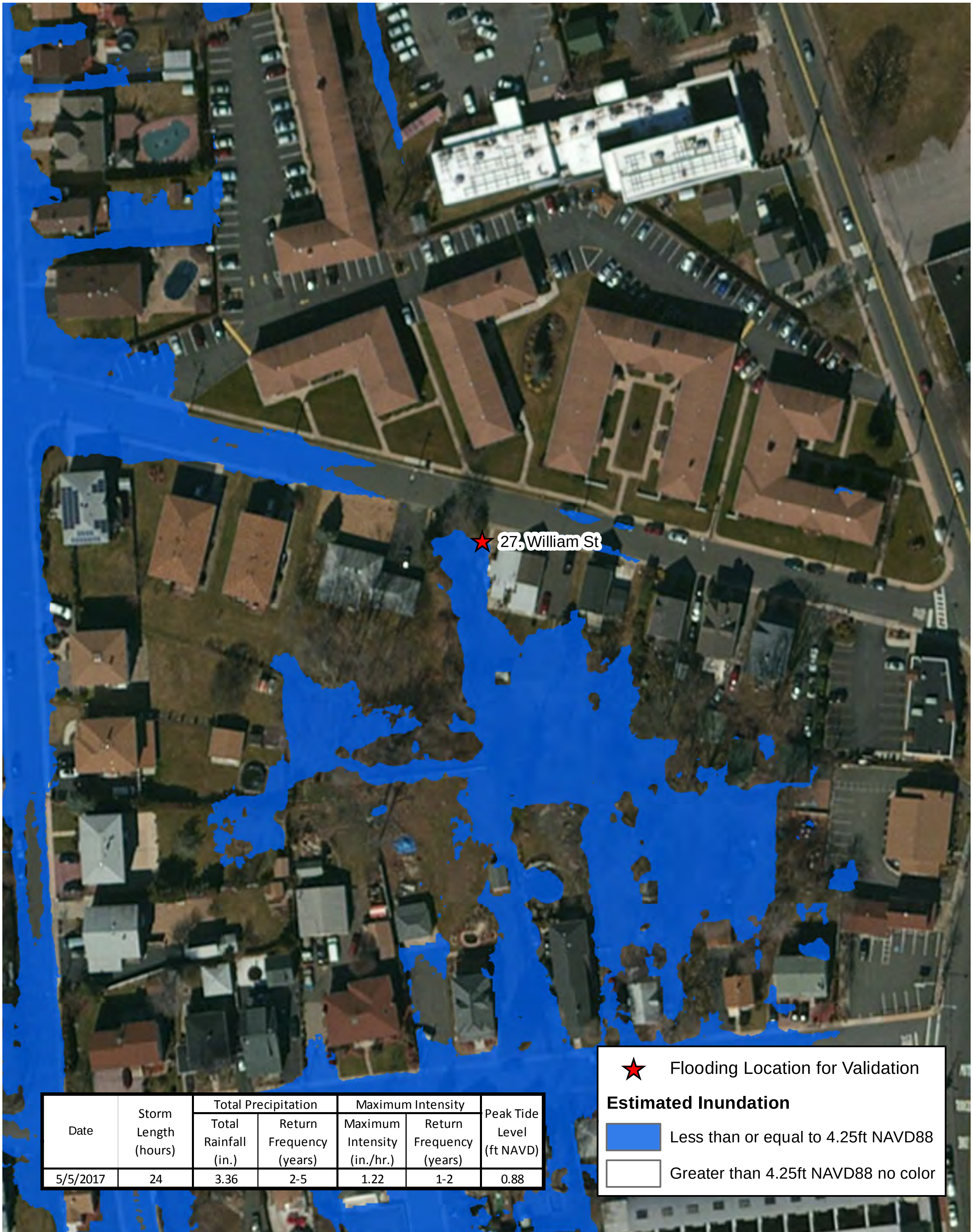
Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING
OBSERVED FLOODING LOCATIONS
26. REDNECK AVE AND PAROUBEK ST LITTLE FERRY

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\0000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_26_REDNECKAVE.MXD - USER: KLEE - DATE: 6/23/2017

William St, Observation #27



Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

★ Flooding Location for Validation

Estimated Inundation

Less than or equal to 4.25ft NAVD88

Greater than 4.25ft NAVD88 no color

0 Feet 100

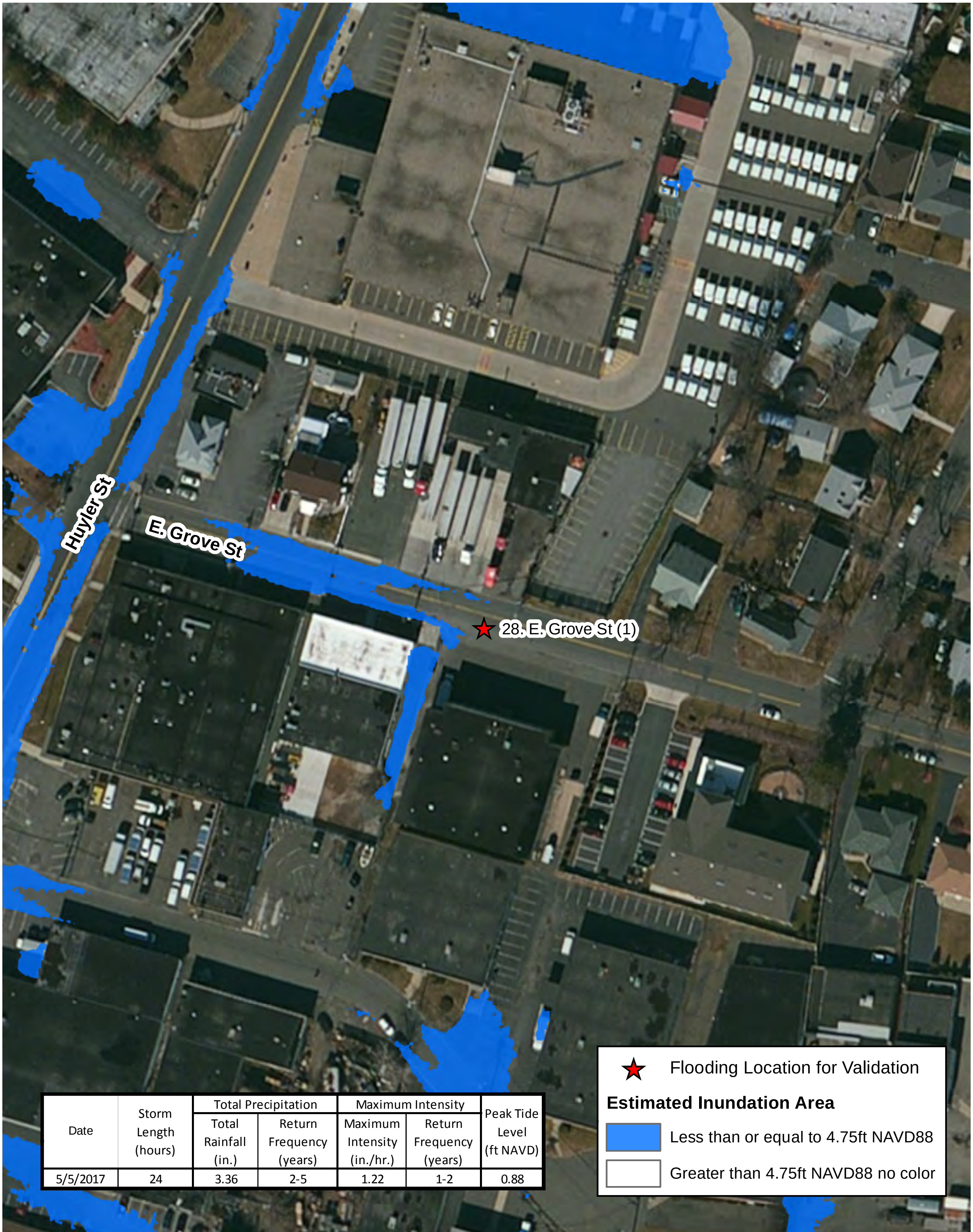
MEADOWLANDS RBDM DRAINAGE MODELING

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

OBSERVED FLOODING LOCATIONS
27. WILLIAM ST LITTLE FERRY

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\000000000270288\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_27_WILLIAMST.MXD - USER: KLEE - DATE: 6/23/2017

East Grove St (1), Observation #28



Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

0 Feet 100

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS
28. E. GROVE ST (1) SOUTH HACKENSACK

East Grove St (2), Observation #29

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

★ Flooding Location for Validation

Estimated Inundation Area

Less than or equal to 4.75ft NAVD88

Greater than 4.75ft NAVD88 no color

0 Feet 100

Designer	FJB	06/16/2017
QC	DS	06/16/2017
Printed	KL	06/23/2017

MEADOWLANDS RBDM DRAINAGE MODELING

OBSERVED FLOODING LOCATIONS

29. E. GROVE ST (2) SOUTH HACKENSACK

PATH: \\MAHPI-FILE01\ACTIVEPROJECTS\202310\CON0103633\0000000002702888\7.0_GIS_MODELS\7.2_WORK_IN_PROGRESS\MODELING\MXD\CALIB_MEMO_DRAFT_5\CALIBRATION_LOCATION_29_GROVE_2.MXD - USER: KLEE - DATE: 6/23/2017

Appendix 4 to Subappendix B5

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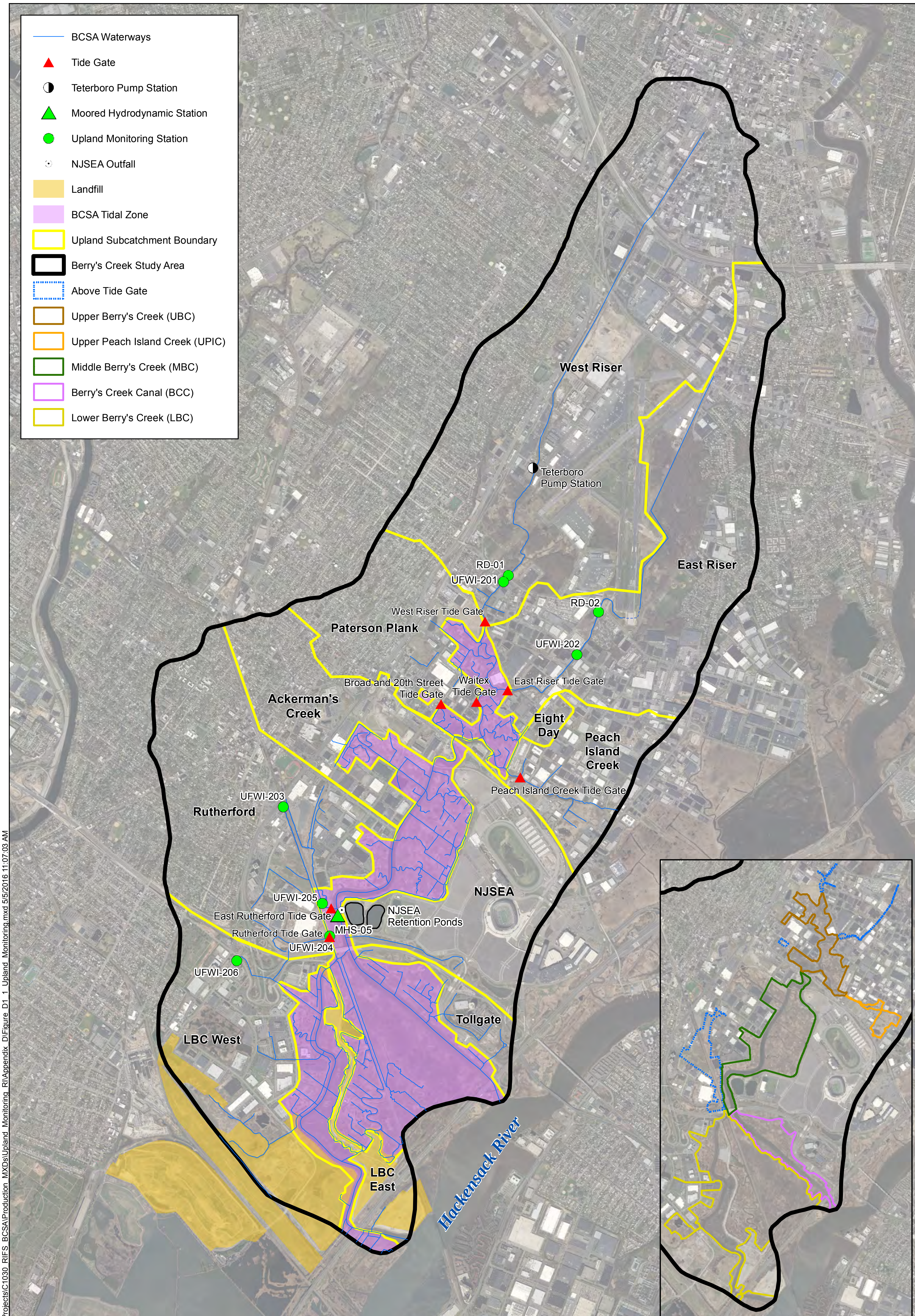

Basemap Source: NJ Imagery, Natural, 2012.
Hydrography Source: U.S. Geological Survey
National Hydrography Dataset (NHD), 2014
NJSEA = New Jersey Sports and Exposition Authority
LBC = Lower Berry's Creek

0 0.25 0.5 1 Miles

N

DRAFT

BCSA Watershed Upland Subcatchment Drainage Areas and Upland Monitoring Locations

Appendix D: Urban Hydrology
Berry's Creek Study Area Remedial Investigation

Figure
A4-1

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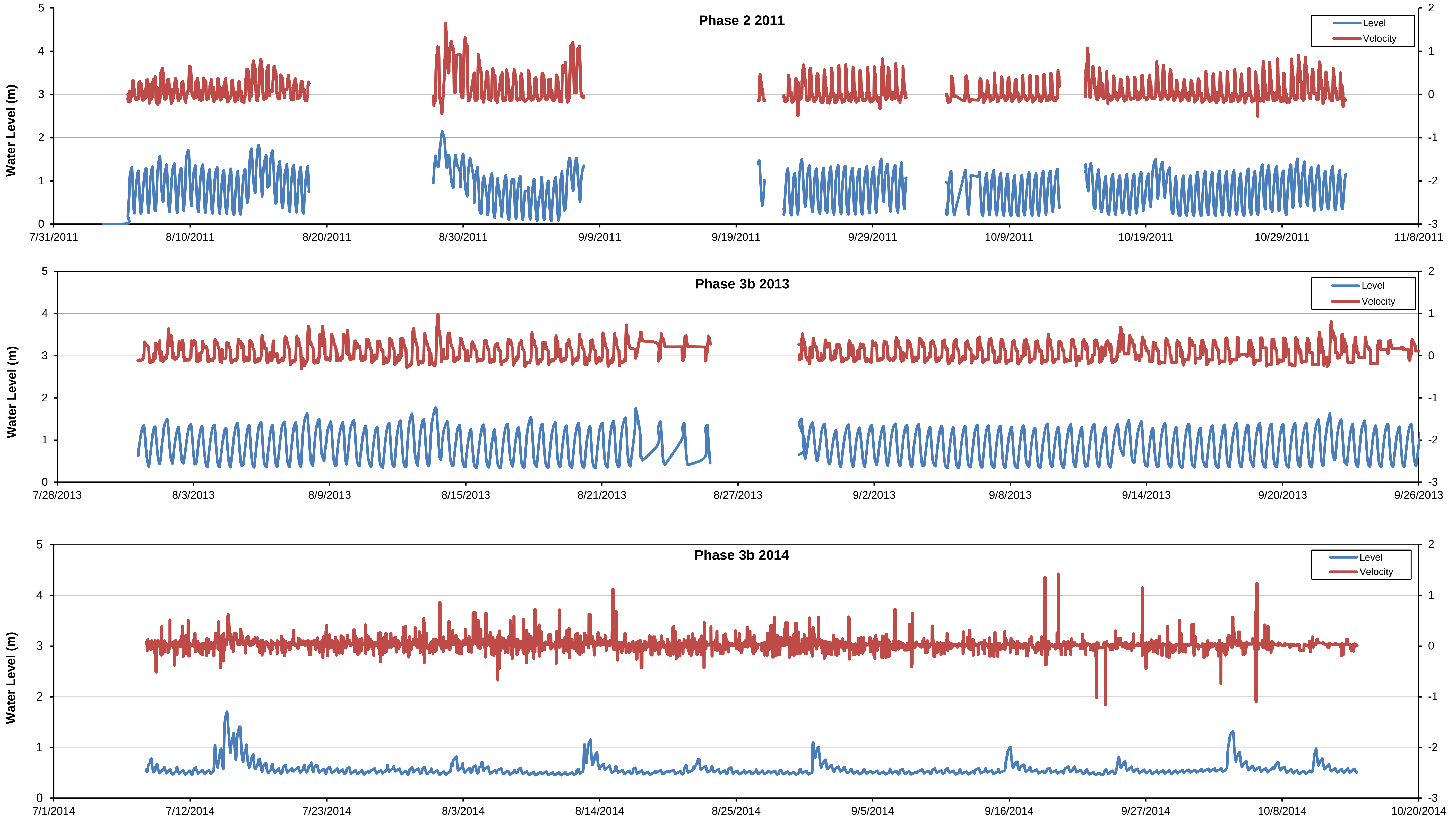


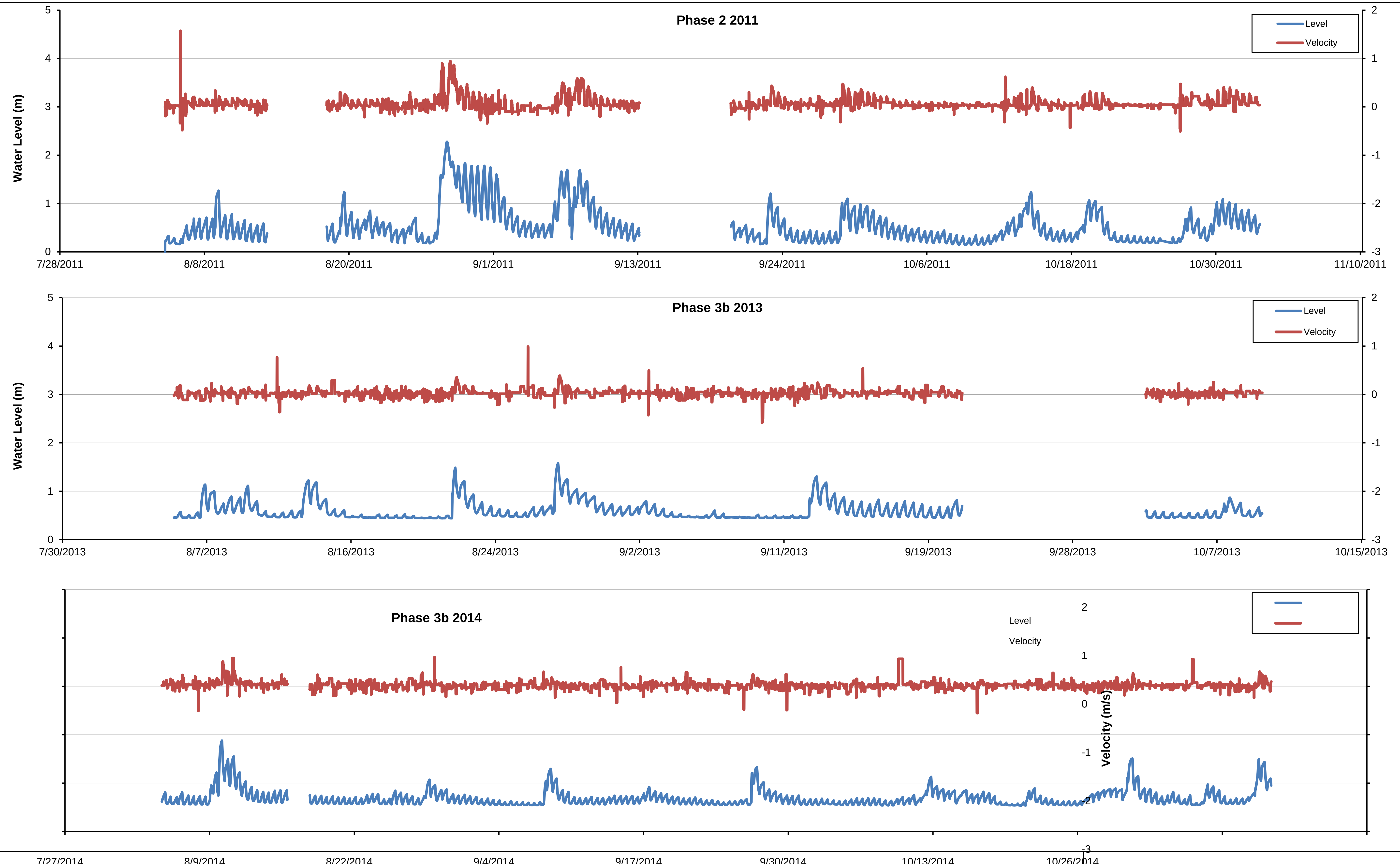
Notes:
Gaps in data reflect periods when instruments were non-operational (downloads, malfunction and repair, etc.).

DRAFT

Measured Level and Velocity
West Riser Ditch (UFWI-201)

Attachment D1: Velocity and Depth Data
Urban Hydrology
Berry's Creek Study Area Remedial Investigation

Figure
A4-2



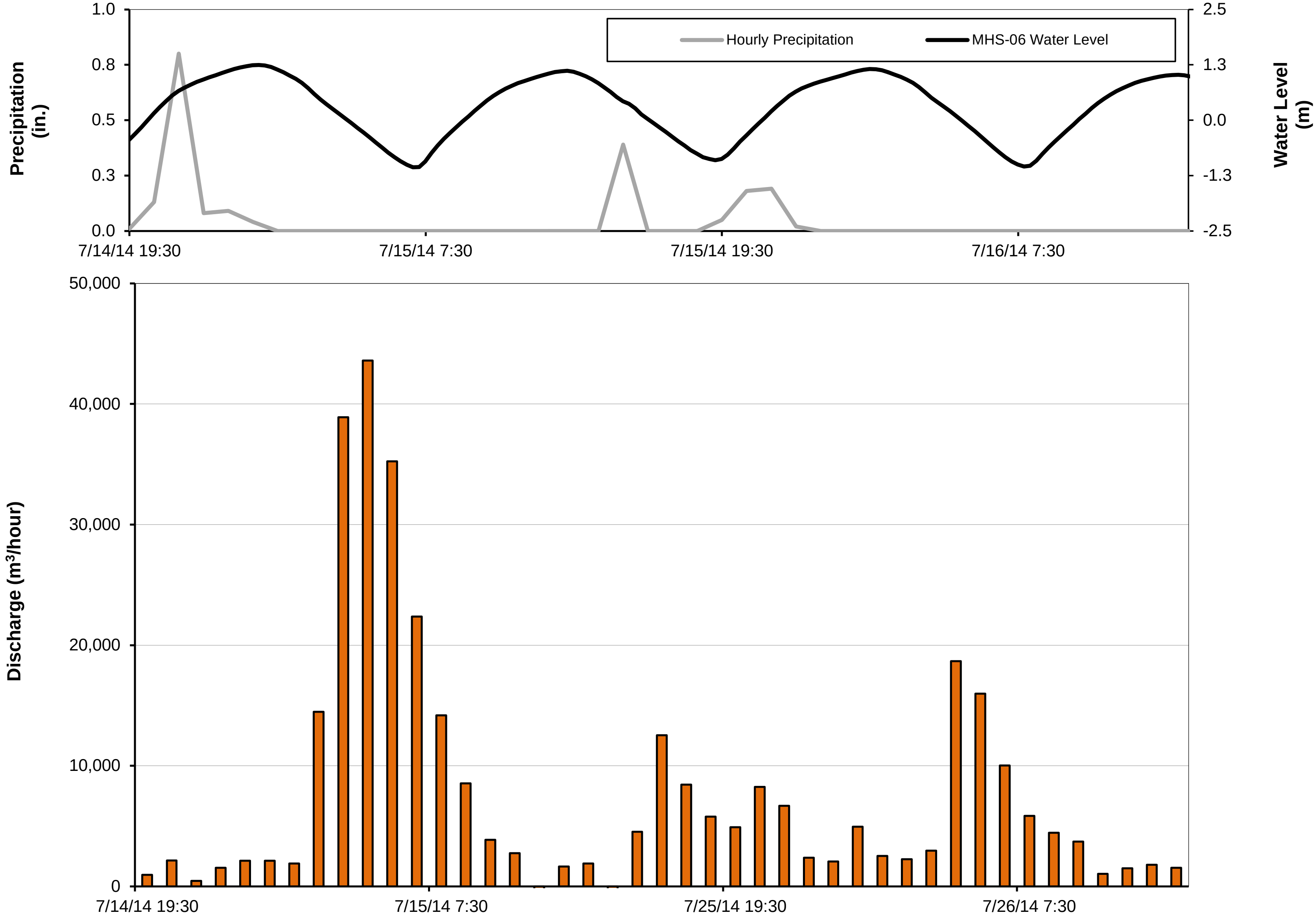
s were non-operational (downloads, malfunction and repair, etc.).

DRAFT

Measured Level and Velocity
East Riser Ditch (UFWI-202)

Attachment D1: Velocity and Depth Data
Urban Hydrology
Berry's Creek Study Area Remedial Investigation

Figure **A4-3**



Notes:
UFWI = upland freshwater input

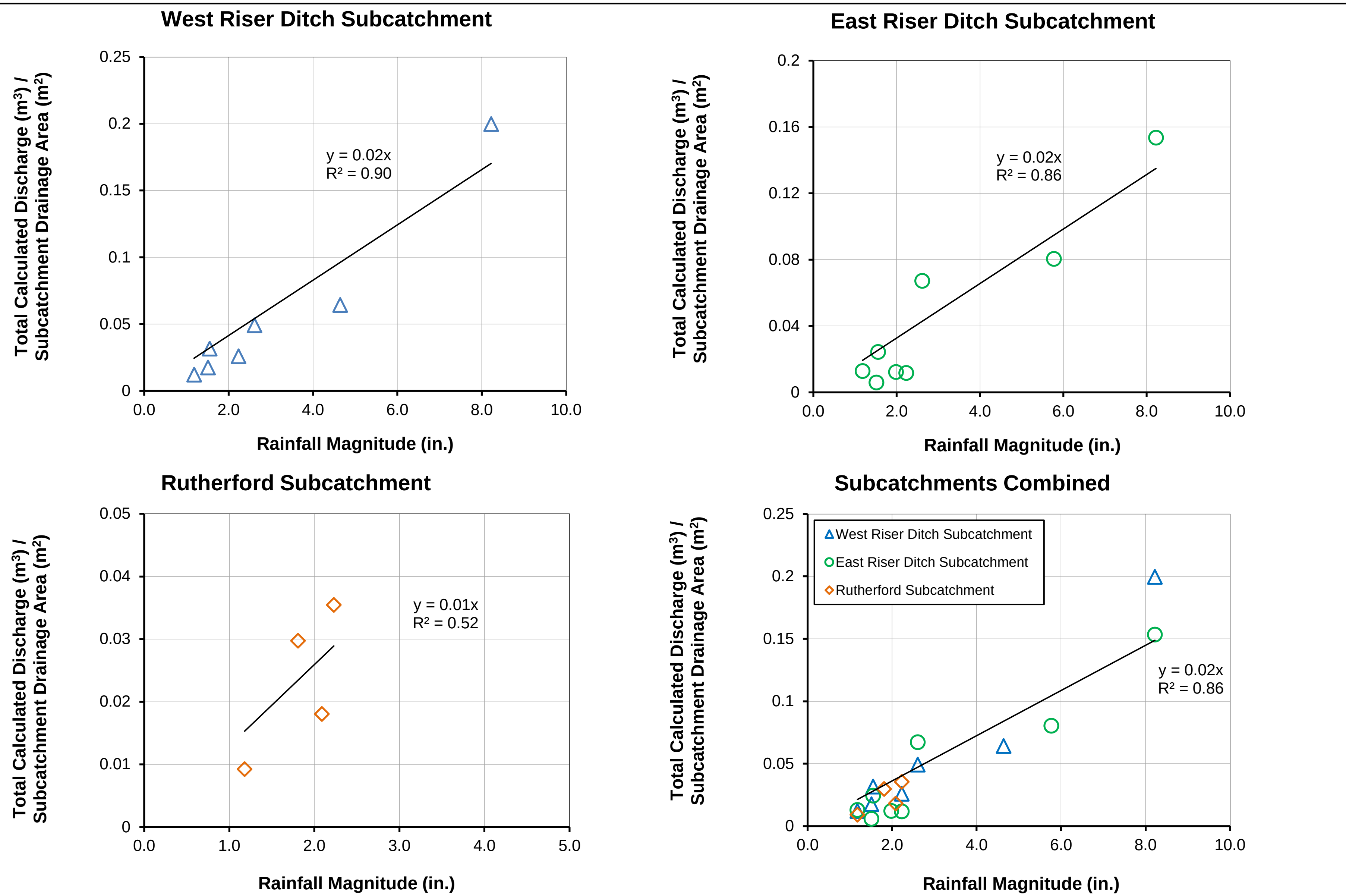
Source:
Hourly precipitation data from Lyndhurst, New Jersey (NJMERI, 2015).

DRAFT

Storm Hydrograph for the West Riser Ditch (UFWI-201) for 2.5-inch Storm Event, July 14 to 15, 2014

Appendix D: Urban Hydrology
Berry's Creek Study Area Remedial Investigation

Figure
A4-4

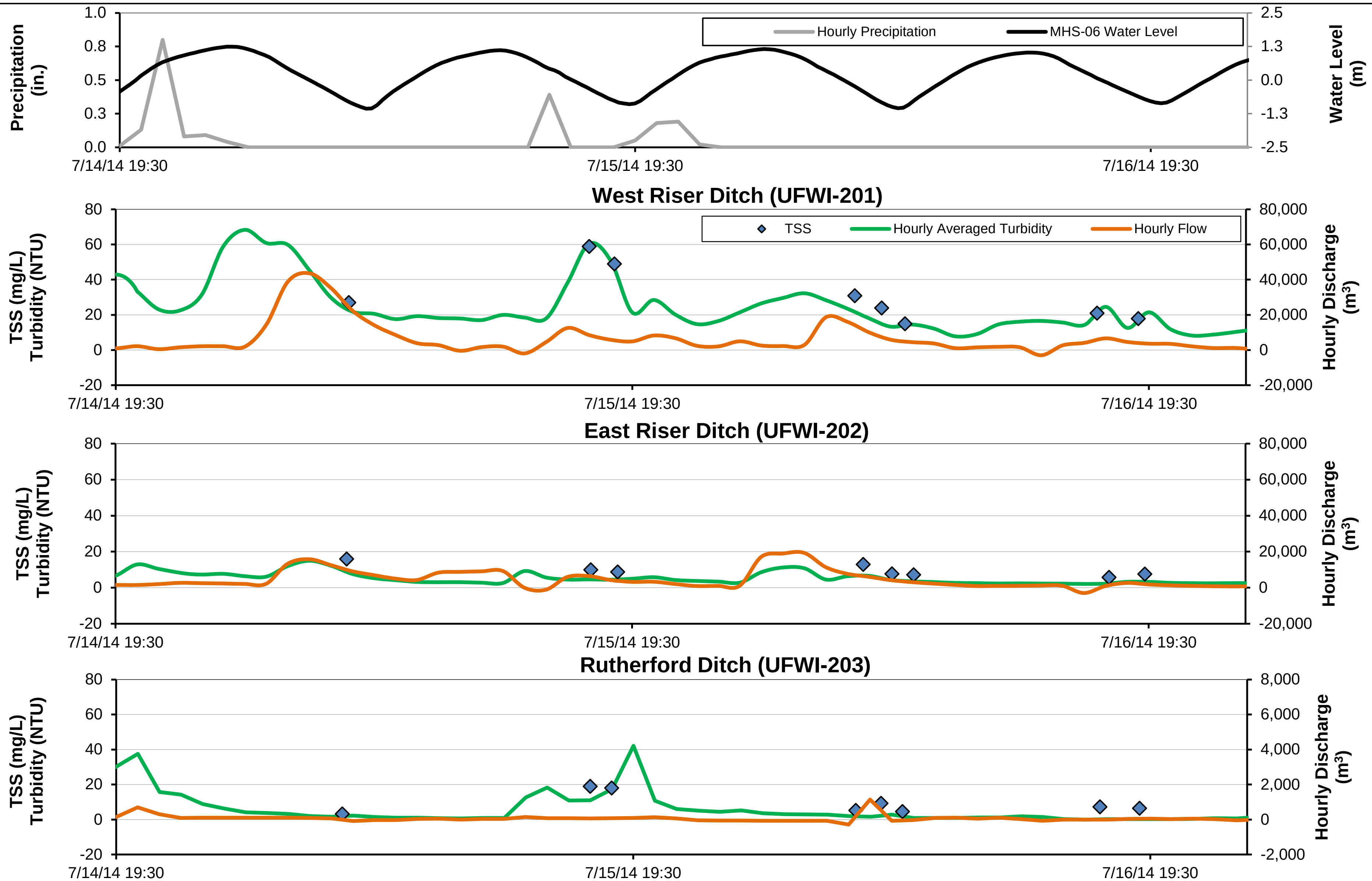


Calculated Storm Flow Runoff Normalized to Subcatchment Drainage Area as a Function of Rainfall Magnitude

DRAFT

Appendix D: Urban Hydrology
Berry's Creek Study Area Remedial Investigation

Figurea A4-5



Notes:
TSS = total suspended solids
NTU = nephelometric turbidity units
UFWI = upland freshwater input
Source:
Hourly precipitation data from Lyndhurst, New Jersey (NJMERI, 2015).

DRAFT

Discharge, Total Suspended Solids, and Turbidity Recorded at Stations UFWI-201, UFWI-202, and UFWI-203 during the July 14 to 15, 2014 Storm Event

Appendix D: Urban Hydrology
Berry's Creek Study Area Remedial Investigation

Figure
A4-6

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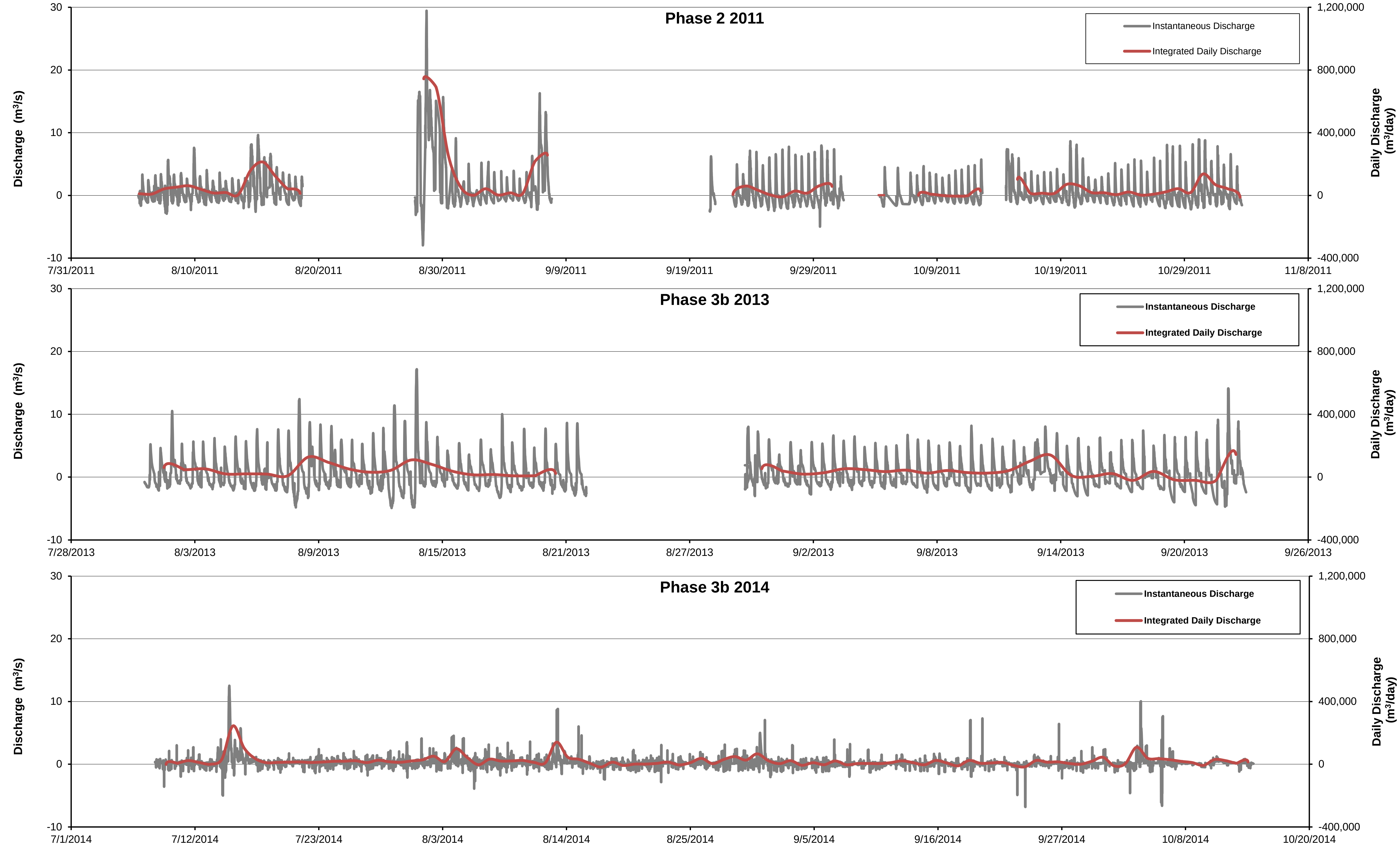


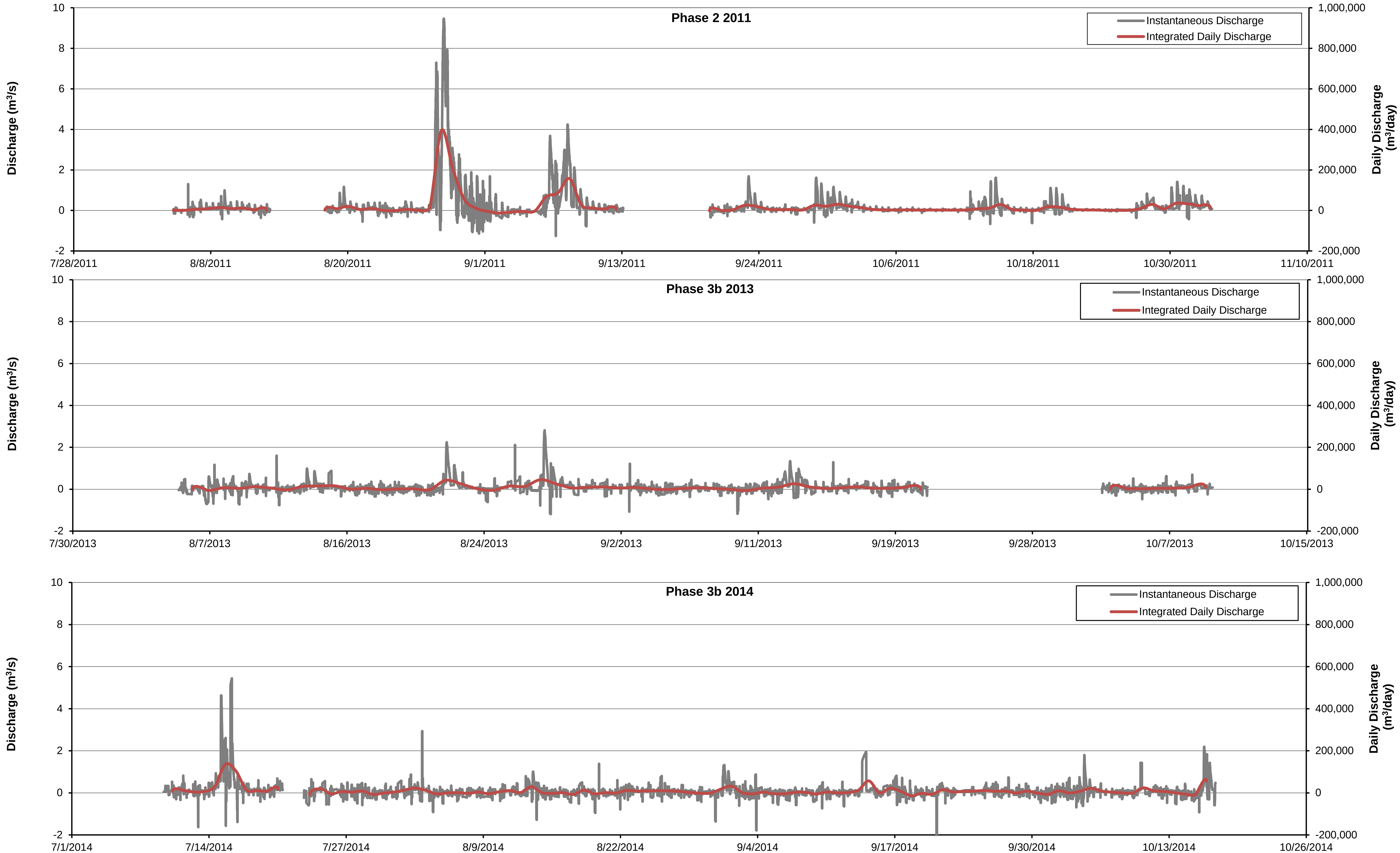
Notes:
Gaps in data reflect periods when instruments were non-operational (downloads, malfunction and repair, etc.).

DRAFT

Freshwater Discharge during Phase 2 and Phase 3b Monitoring, West Riser Ditch (UFWI-201)

Attachment D3: Calculated Freshwater Discharge
Urban Hydrology
Berry's Creek Study Area Remedial Investigation

Figure
A4-7



Notes:
Gaps in data reflect periods when instruments were non-operational (downloads, malfunction and repair, etc.).

DRAFT

Freshwater Discharge during Phase 2 and Phase 3b Monitoring, East Riser Ditch (UFWI-202)

Attachment D3: Calculated Freshwater Discharge
Urban Hydrology
Berry's Creek Study Area Remedial Investigation

Figure **A4-8**

Appendix 5 to Subappendix B5

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Meadowlands NFIP Claims
Note: 2007 April (90 Claims) and
1999 August/September (42 Claims) Events
Legend
1999 August/September Event
2007 April Event
Municipality Boundaries
N
0 1,000 2,000 4,000 Feet
ELMWOOD PARK BORO
ROCHELLE PARK TWP
MAYWOOD BORO
SADDLE BROOK TWP
Hackensack
TEANECK TWP
LODI BORO
GARFIELD CITY
Bogota
BOGOTA BORO
SOUTH HACKENSACK TWP
HASBROUCK HEIGHTS BORO
Teterboro
TETERBORO BORO
RIDGEFIELD PARK VILLAGE
WALLINGTON BORO
WOOD-RIDGE BORO
Little Ferry
LITTLE FERRY BORO
PALISADES PARK BORO
Carlstadt
Moonachie
MOONACHIE BORO
East Rutherford
RIDGEFIELD BORO
CARLSTADT BORO
FAIRVIEW BORO
EAST RUTHERFORD BORO
RUTHERFORD BORO
NORTH BERGEN TWP
LYNDHURST TWP
SECAUCUS TOWN
GUTTENBERG TOWN
Secaucus


Meadowlands NFIP Claims
in Little Falls and Moonachie
Note: 2007 April (90 Claims) and
1999 August/September (42 Claims) Events
SOUTH HACKENSACK TWP
TETERBORO BORO
RIDGEFIELD PARK VILLAGE
Little Ferry
LITTLE FERRY BORO
Moonachie
MOONACHIE BORO
RIDGEFIELD BORO
CARLSTADT BORO
SOUTH HACKENSACK TWP
Legend
1999 August/September Event
2007 April Event
Municipality Boundaries
N
0
300
600
1,200 Feet

Appendix 6 to Subappendix B5

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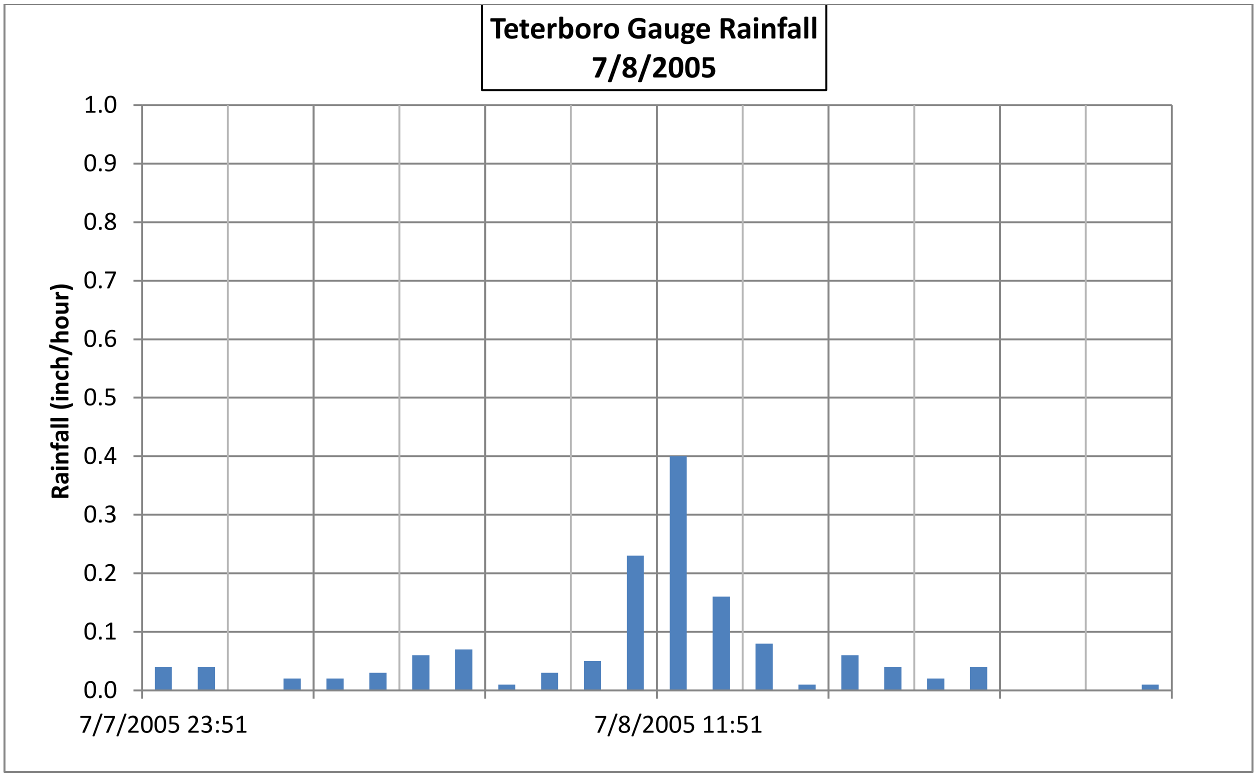
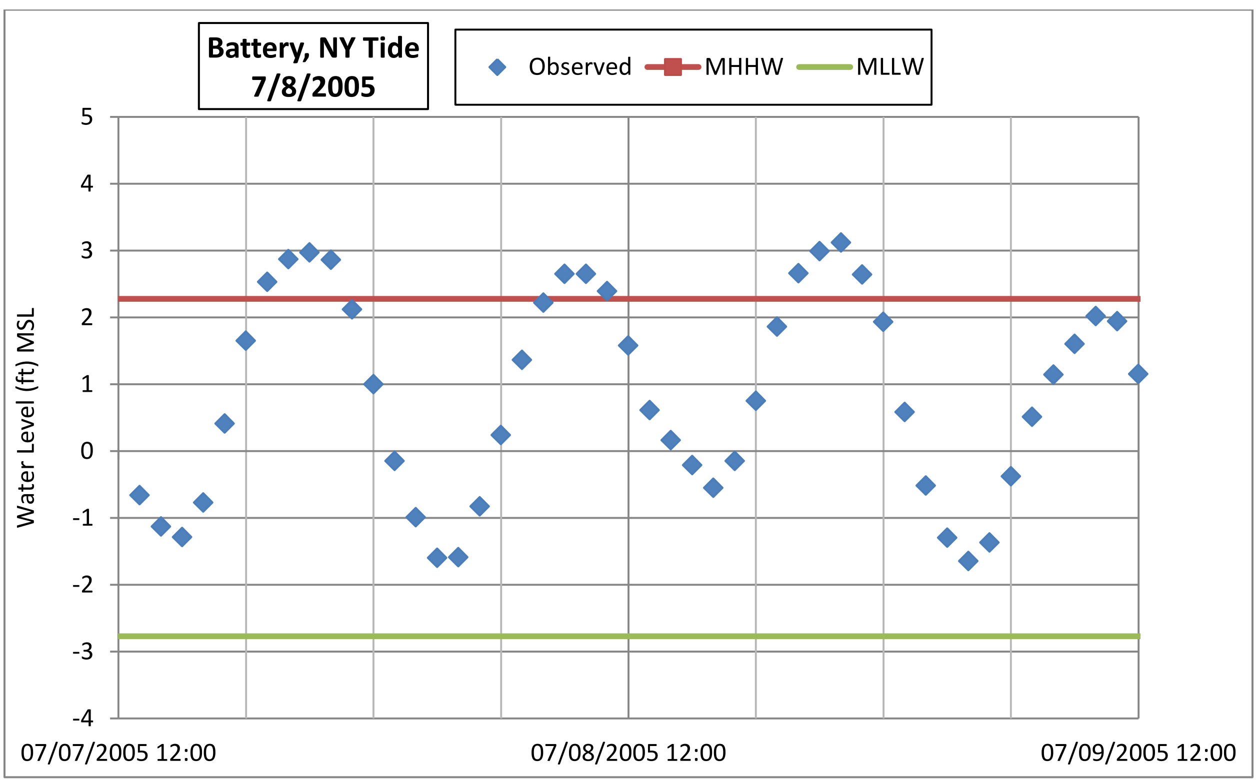
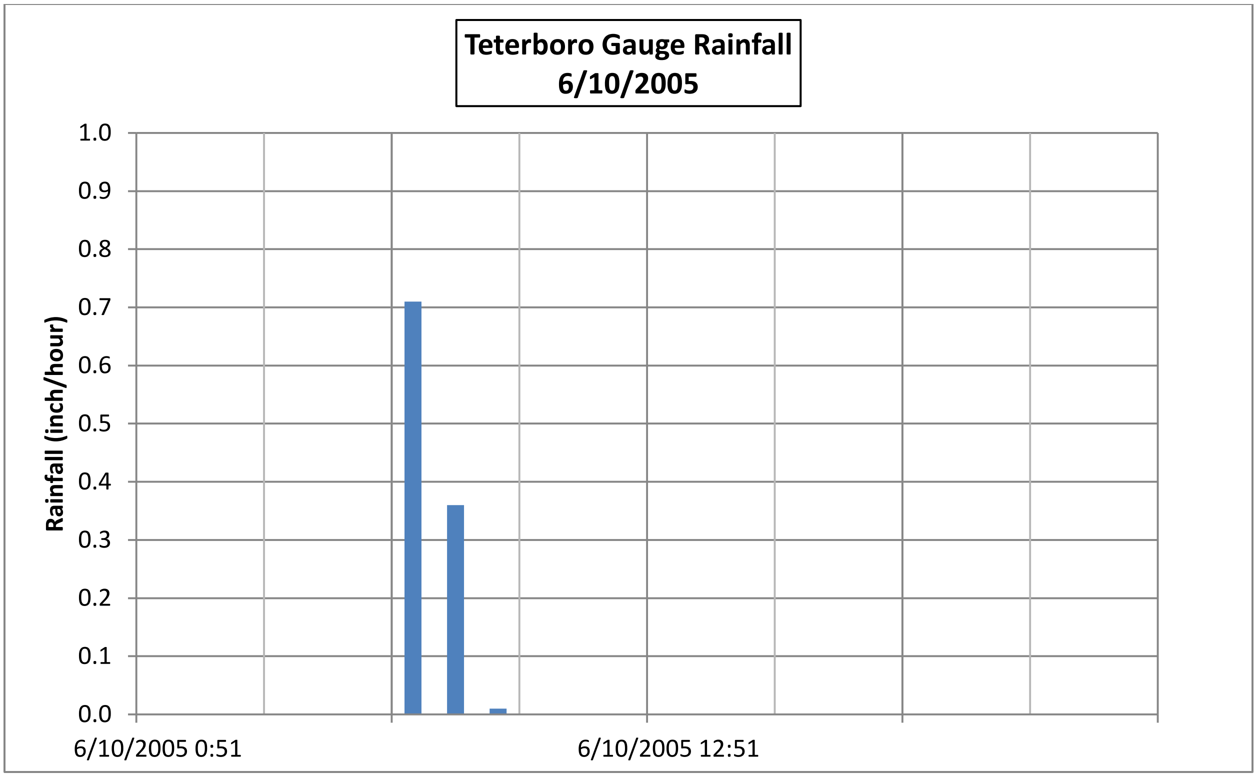


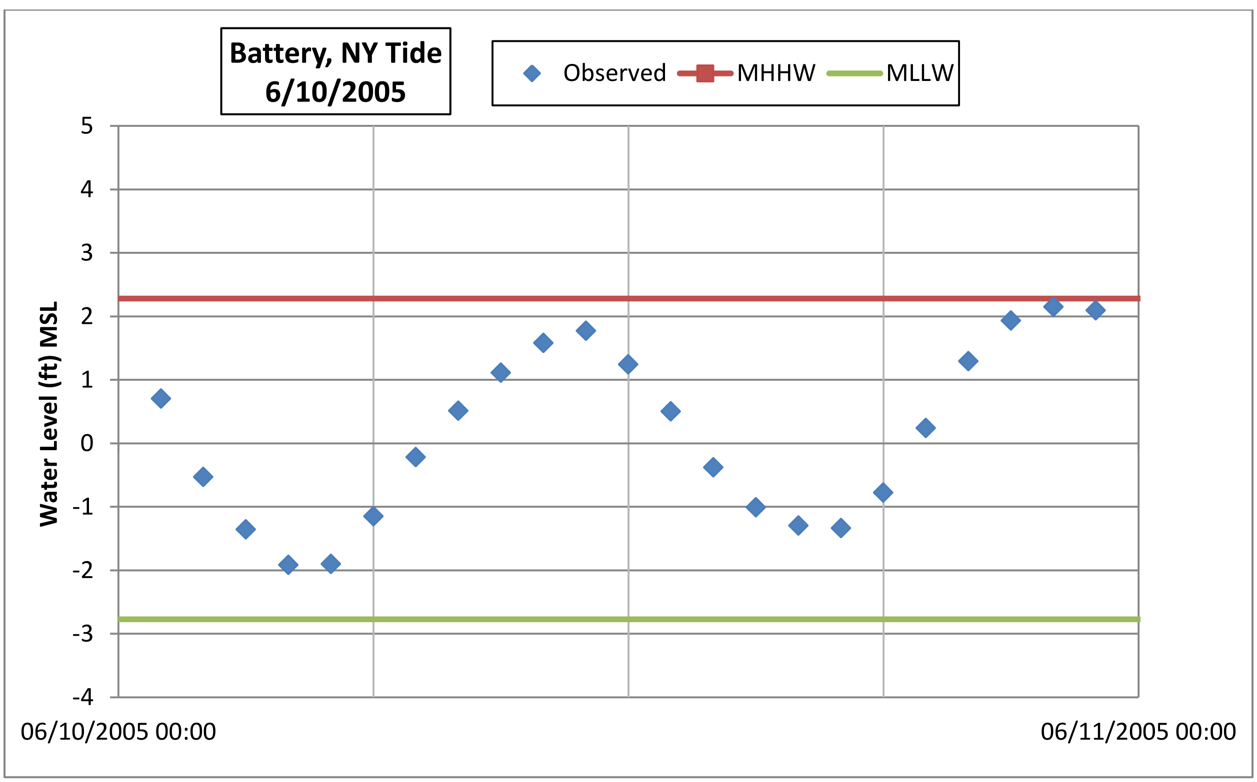
Teterboro Gauge Rainfall
7/8/2005
Rainfall (inch/hour)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
7/7/2005 23:51
7/8/2005 11:51



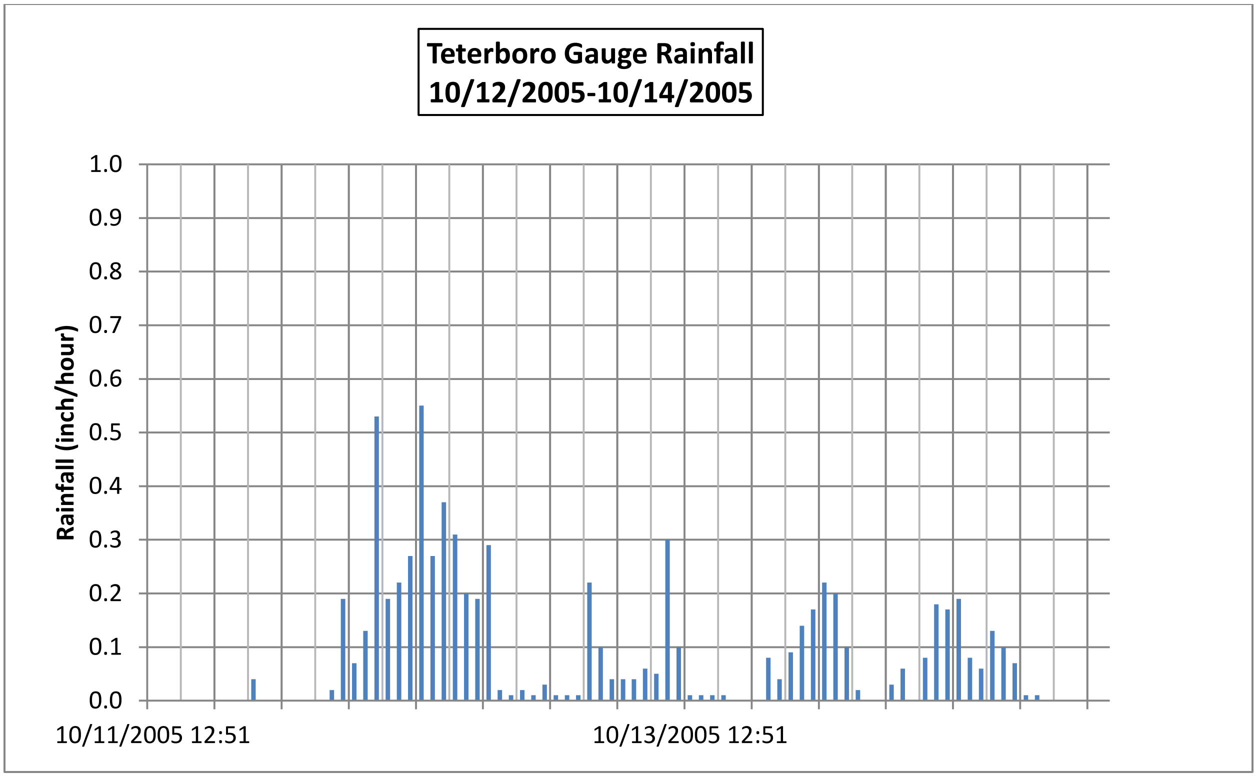
Battery, NY Tide
7/8/2005
Observed
MHHW
MLLW
Water Level (ft) MSL
5
4
3
2
1
0
-1
-2
-3
-4
07/07/2005 12:00
07/08/2005 12:00
07/09/2005 12:00

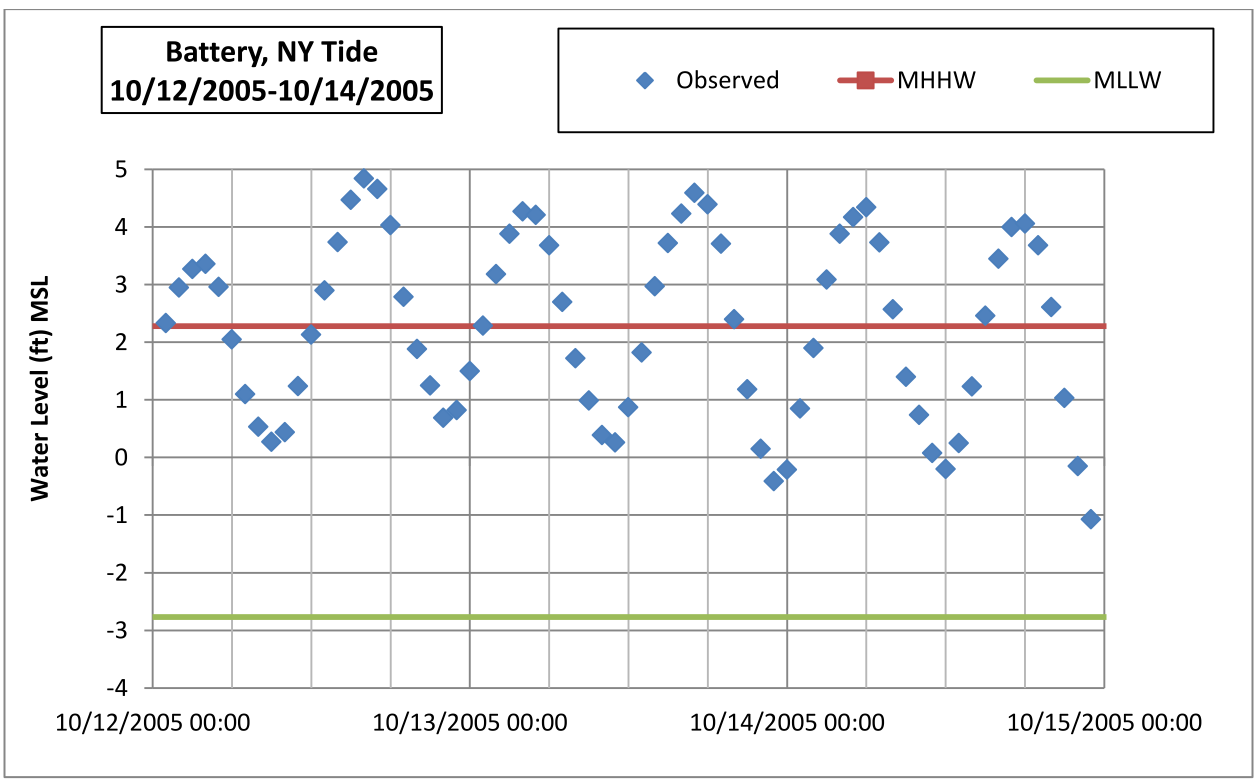


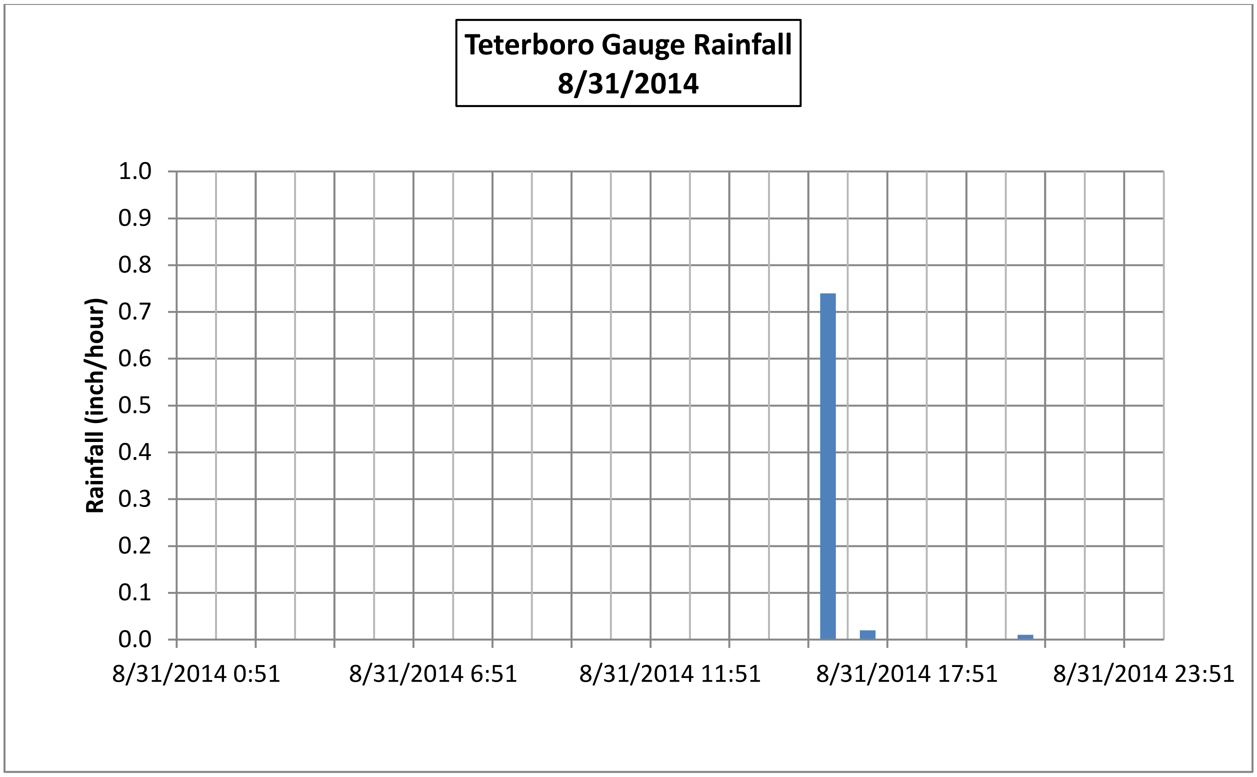
Teterboro Gauge Rainfall
6/10/2005
Rainfall (inch/hour)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
6/10/2005 0:51
6/10/2005 12:51

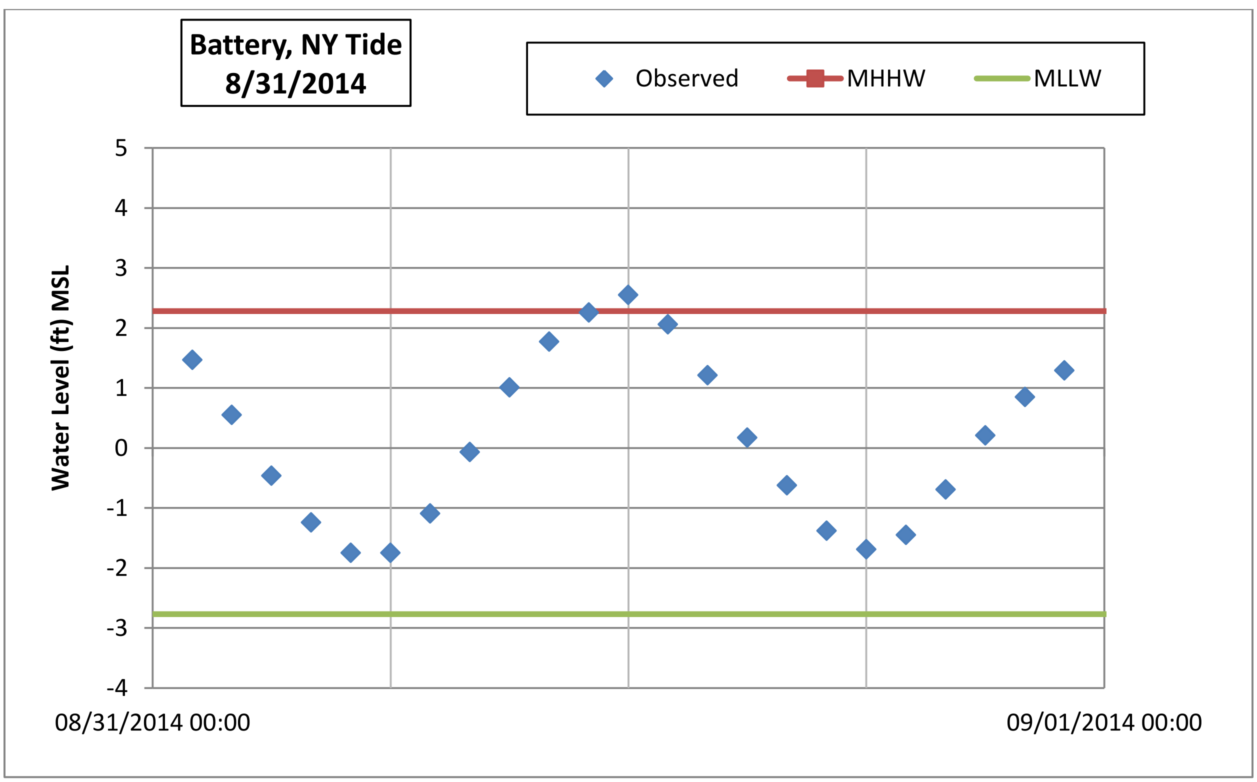


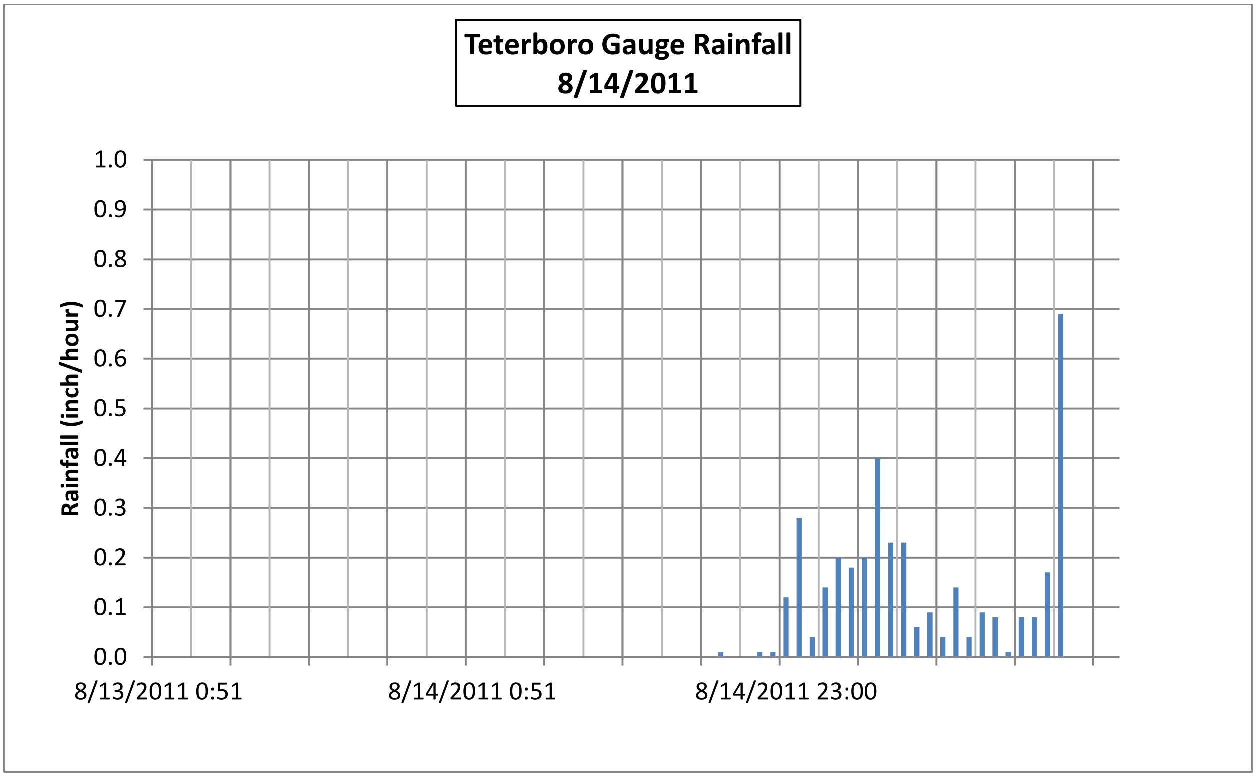
Battery, NY Tide
6/10/2005
Observed
MHHW
MLLW
Water Level (ft) MSL
5
4
3
2
1
0
-1
-2
-3
-4
06/10/2005 00:00
06/11/2005 00:00

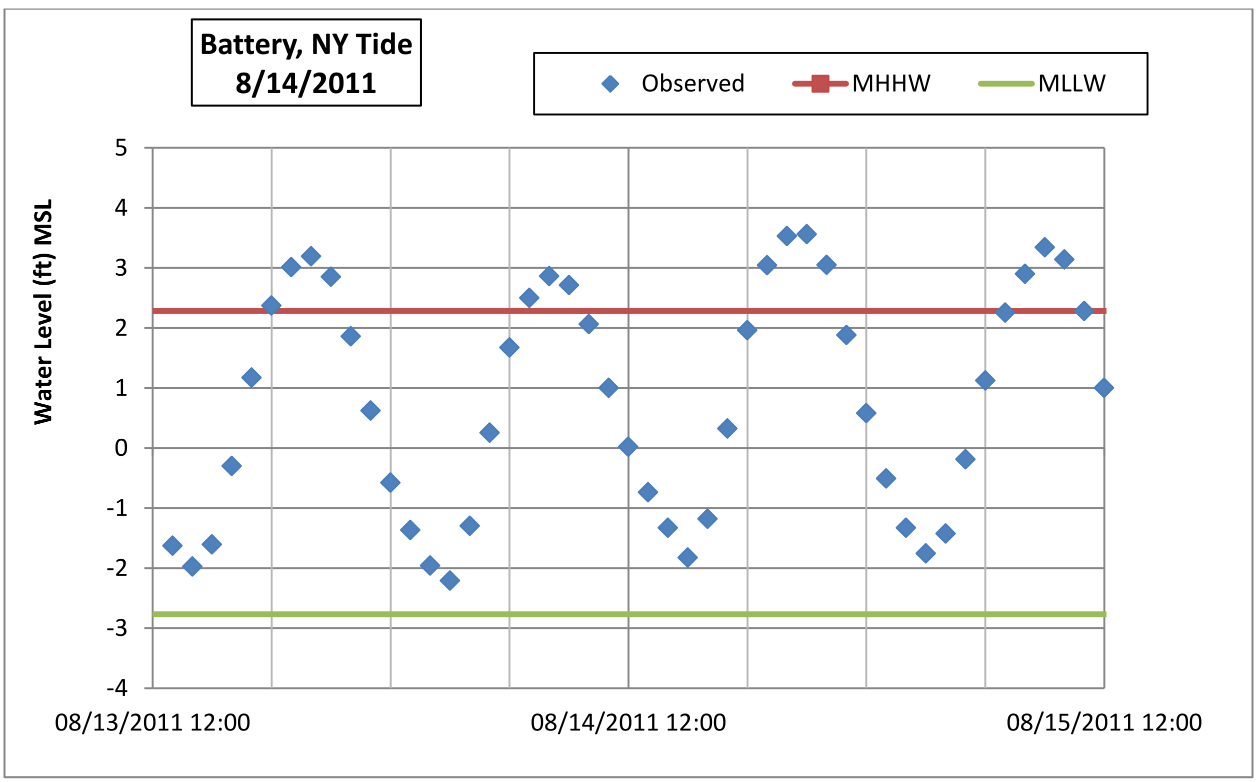

Teterboro Gauge Rainfall
10/12/2005-10/14/2005
Rainfall (inch/hour)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
10/11/2005 12:51
10/13/2005 12:51

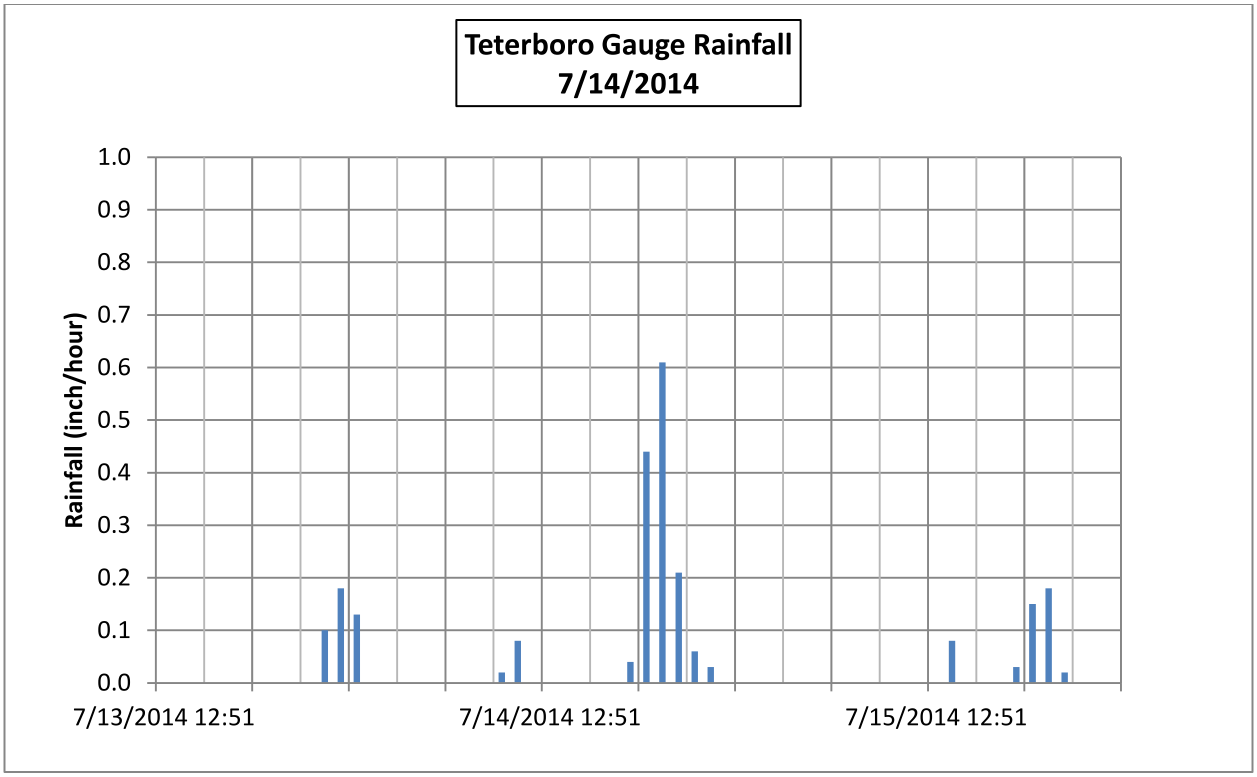

Battery, NY Tide
10/12/2005-10/14/2005
Observed
MHHW
MLLW
Water Level (ft) MSL
5
4
3
2
1
0
-1
-2
-3
-4
10/12/2005 00:00
10/13/2005 00:00
10/14/2005 00:00
10/15/2005 00:00


Teterboro Gauge Rainfall
8/31/2014
Rainfall (inch/hour)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
8/31/2014 0:51
8/31/2014 6:51
8/31/2014 11:51
8/31/2014 17:51
8/31/2014 23:51


Battery, NY Tide
8/31/2014
Observed
MHHW
MLLW
Water Level (ft) MSL
5
4
3
2
1
0
-1
-2
-3
-4
08/31/2014 00:00
09/01/2014 00:00


Teterboro Gauge Rainfall
8/14/2011
Rainfall (inch/hour)
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
8/13/2011 0:51
8/14/2011 0:51
8/14/2011 23:00


Battery, NY Tide
8/14/2011
Observed
MHHW
MLLW
Water Level (ft) MSL
5
4
3
2
1
0
-1
-2
-3
-4
08/13/2011 12:00
08/14/2011 12:00
08/15/2011 12:00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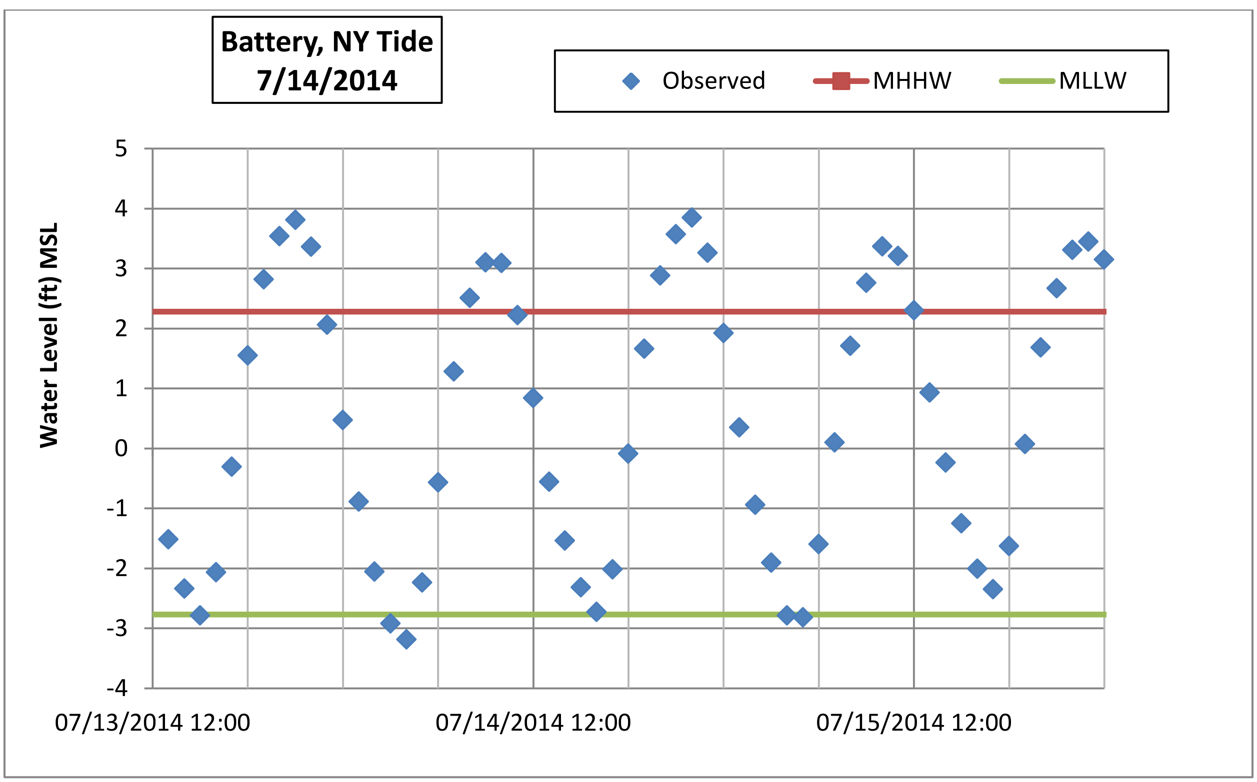
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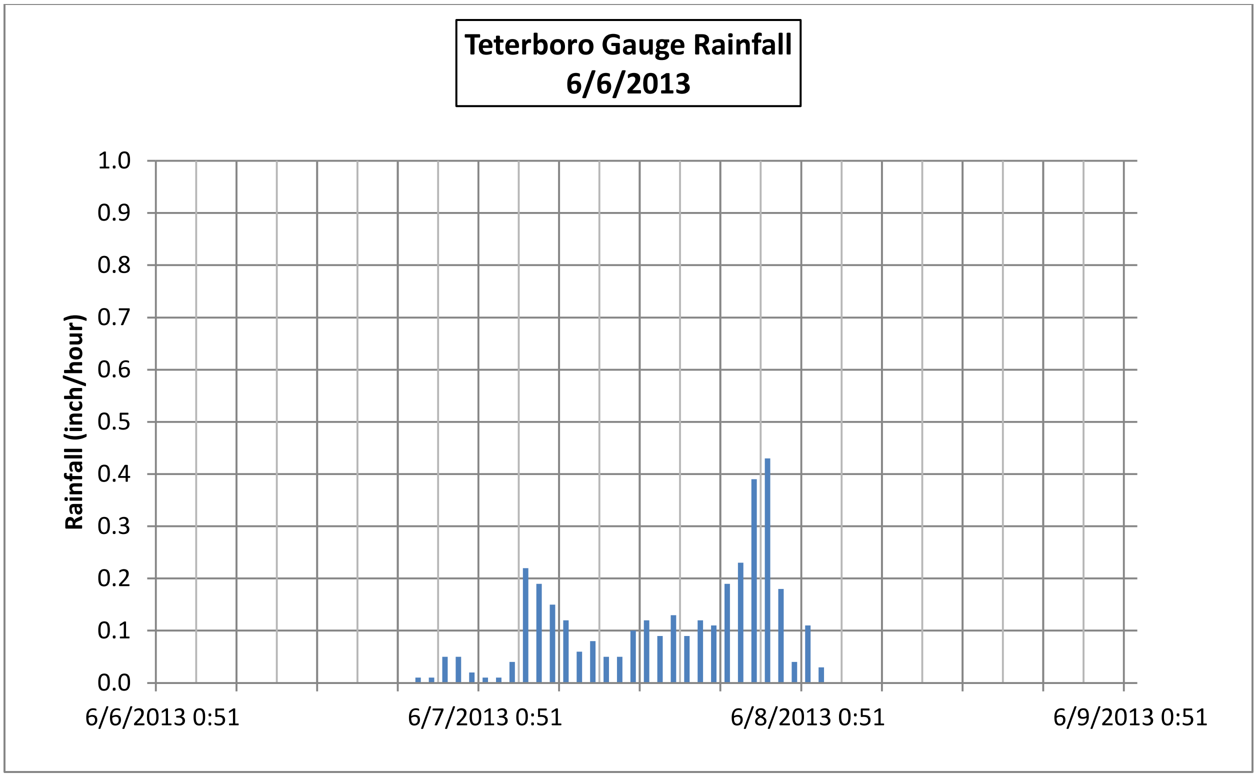

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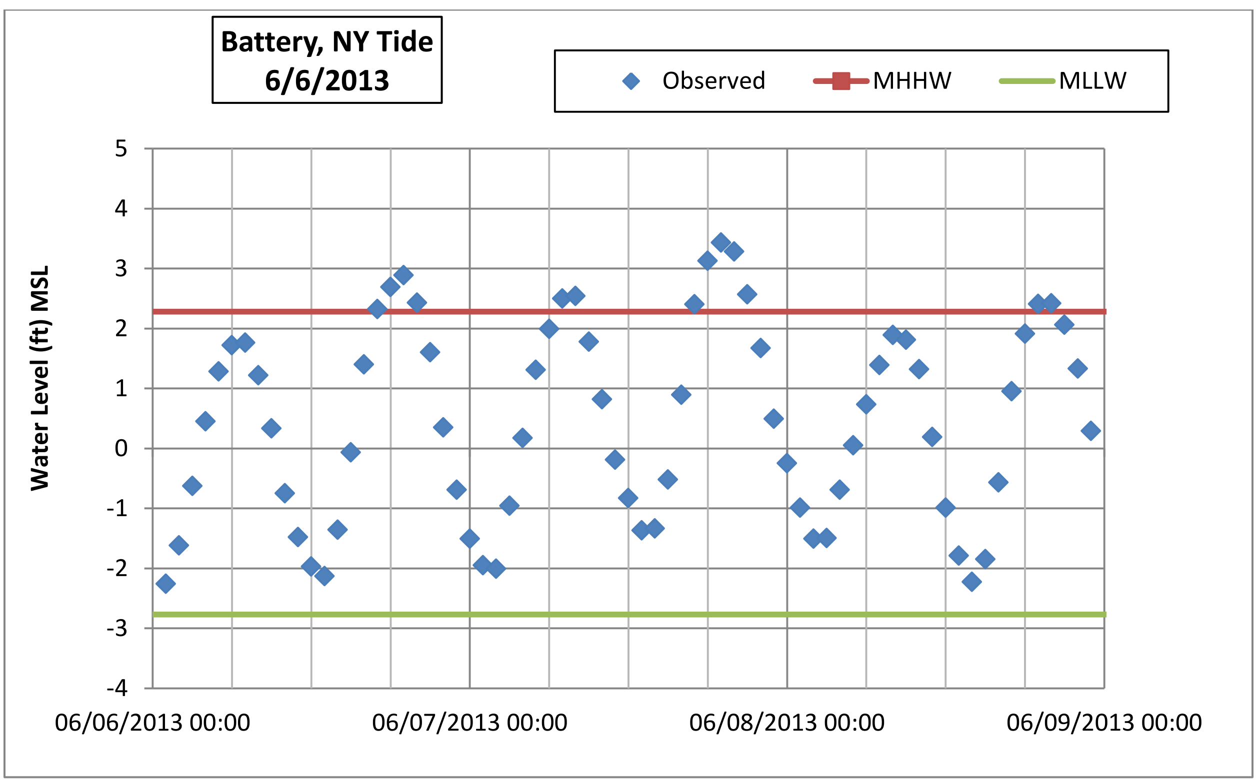

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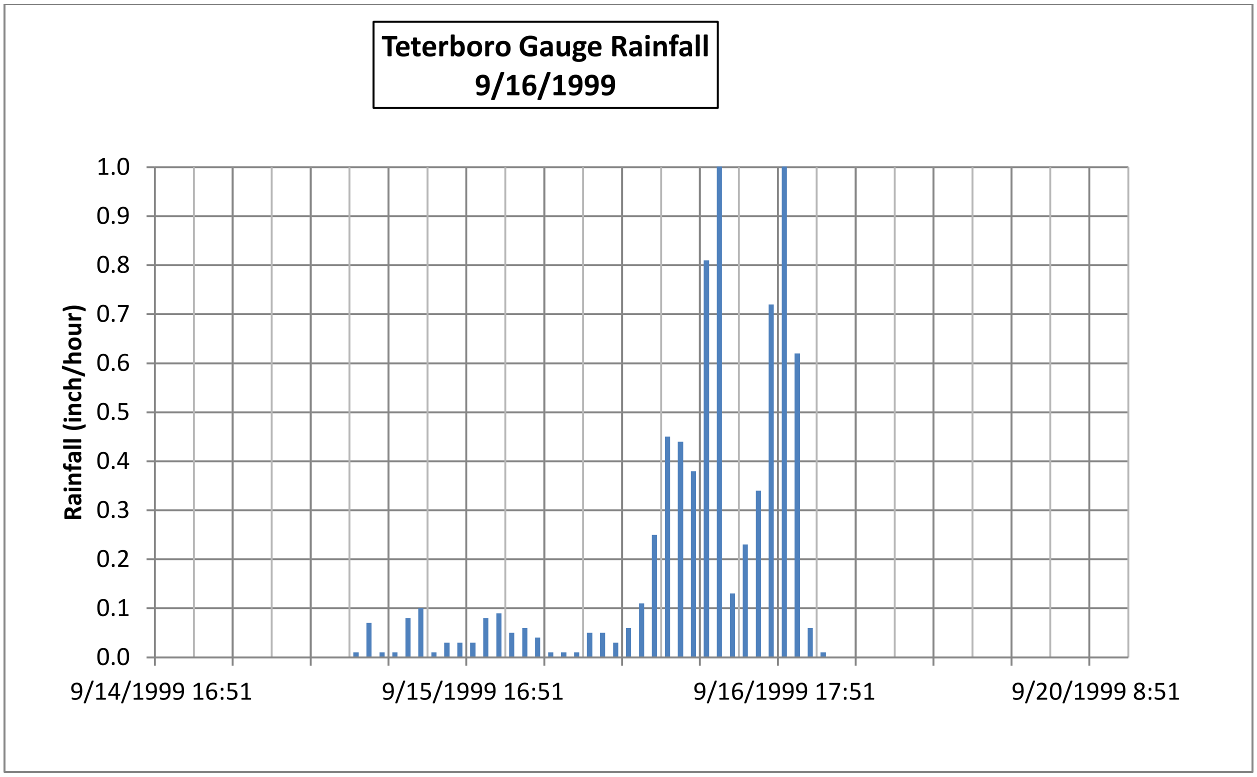
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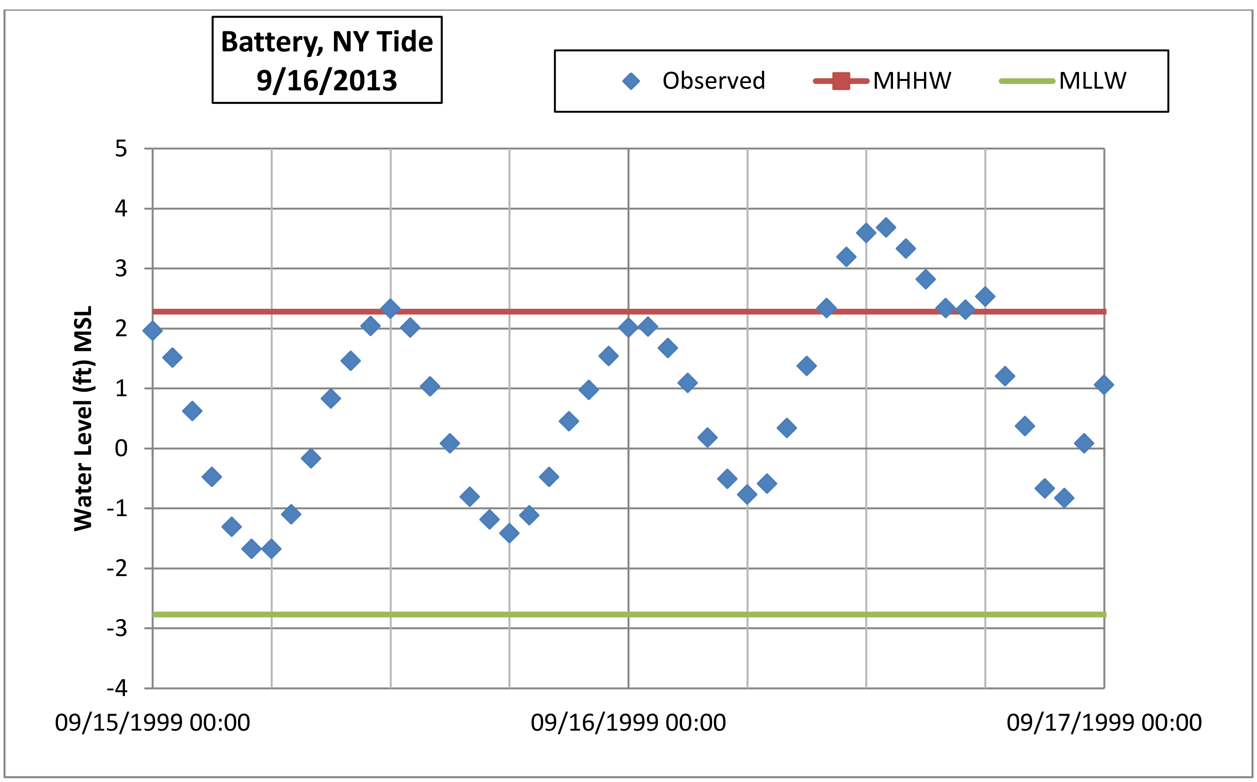
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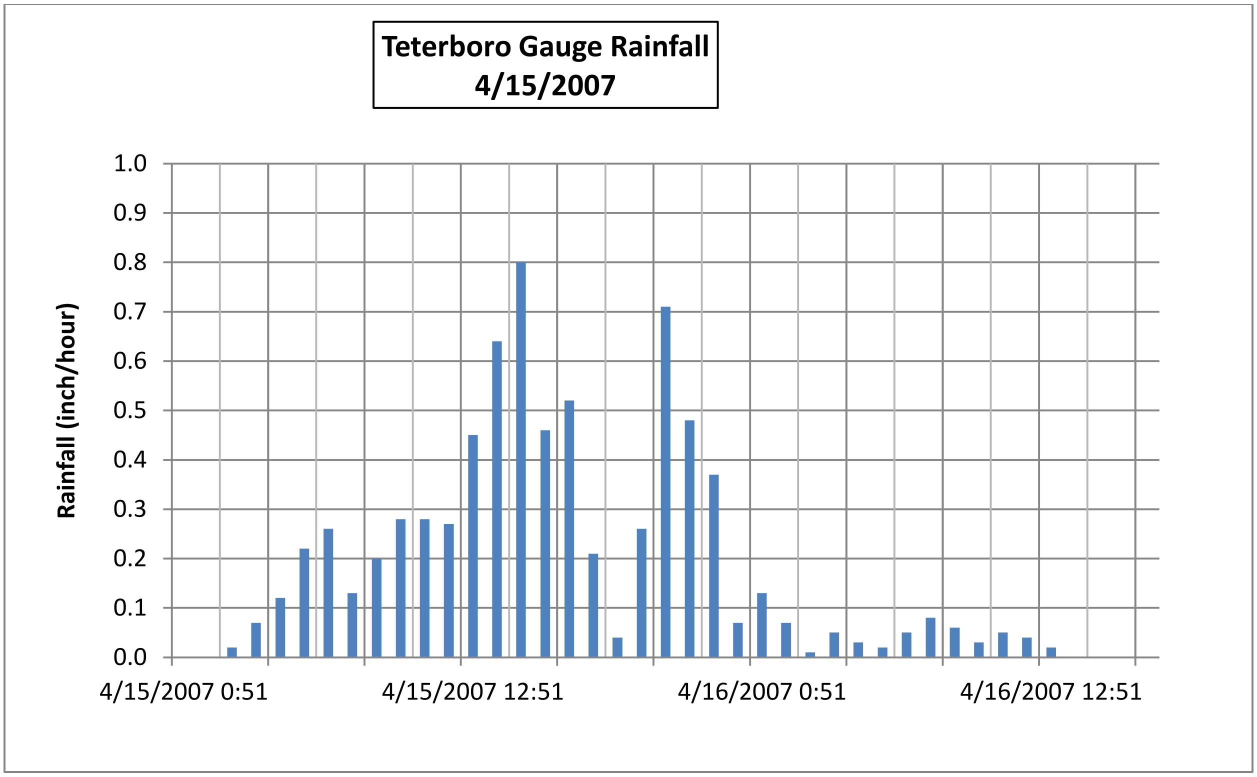
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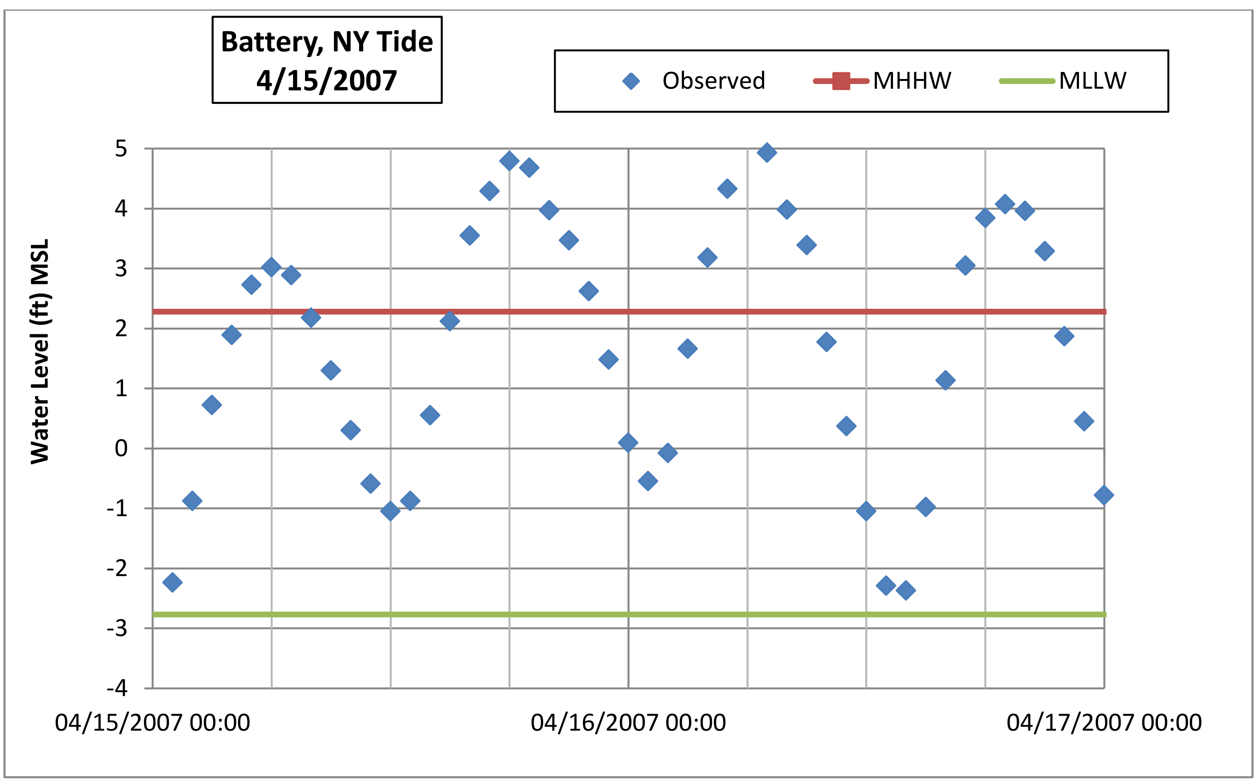
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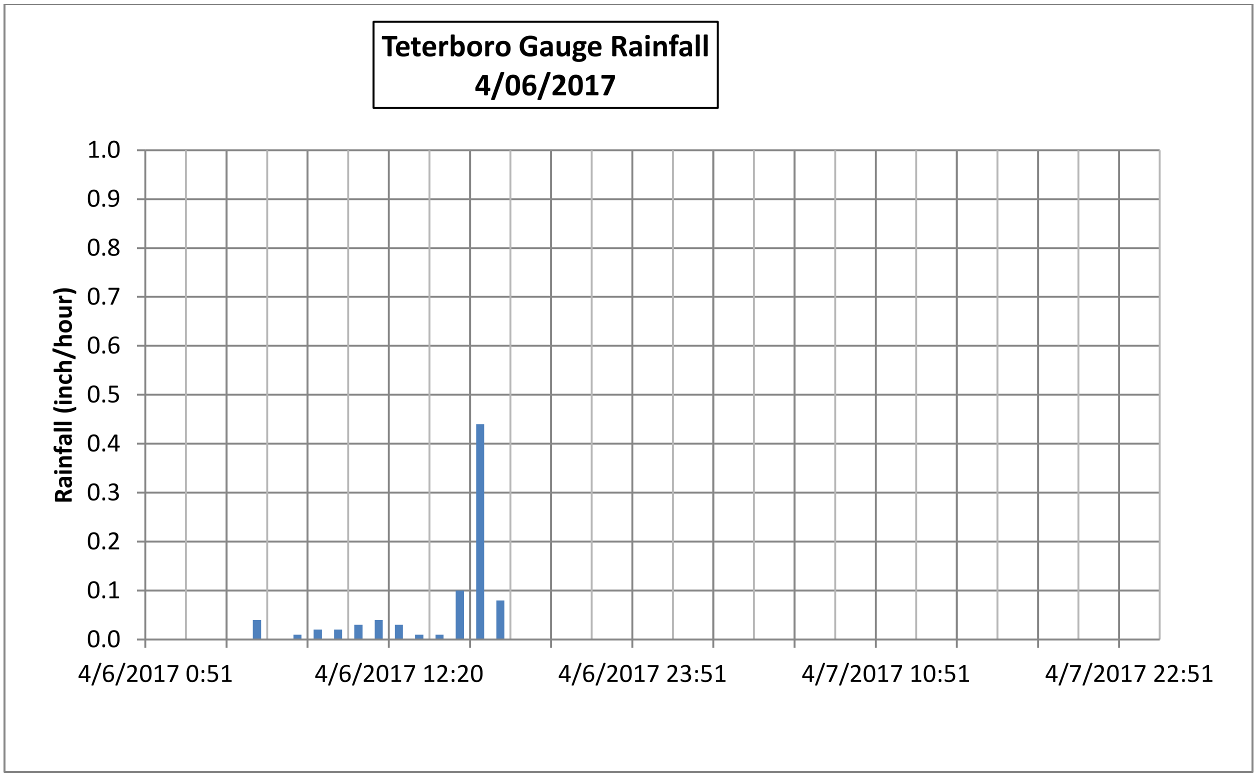
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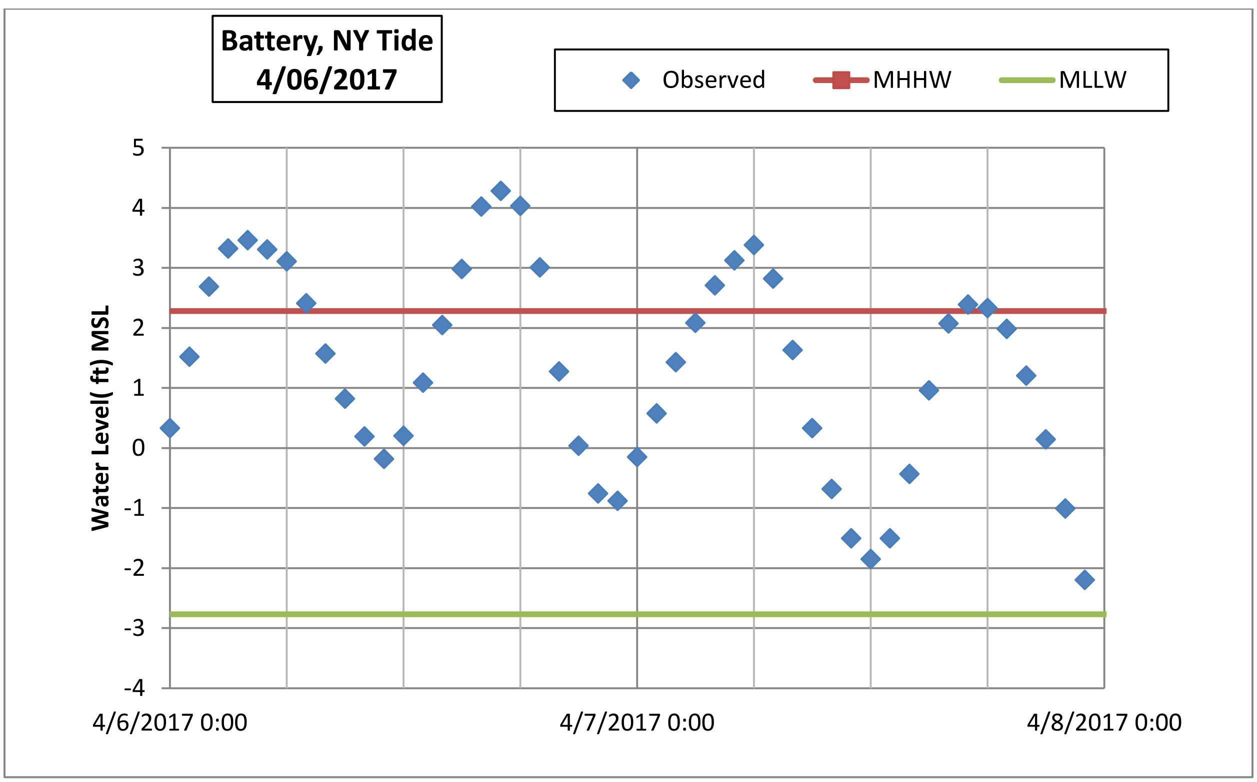
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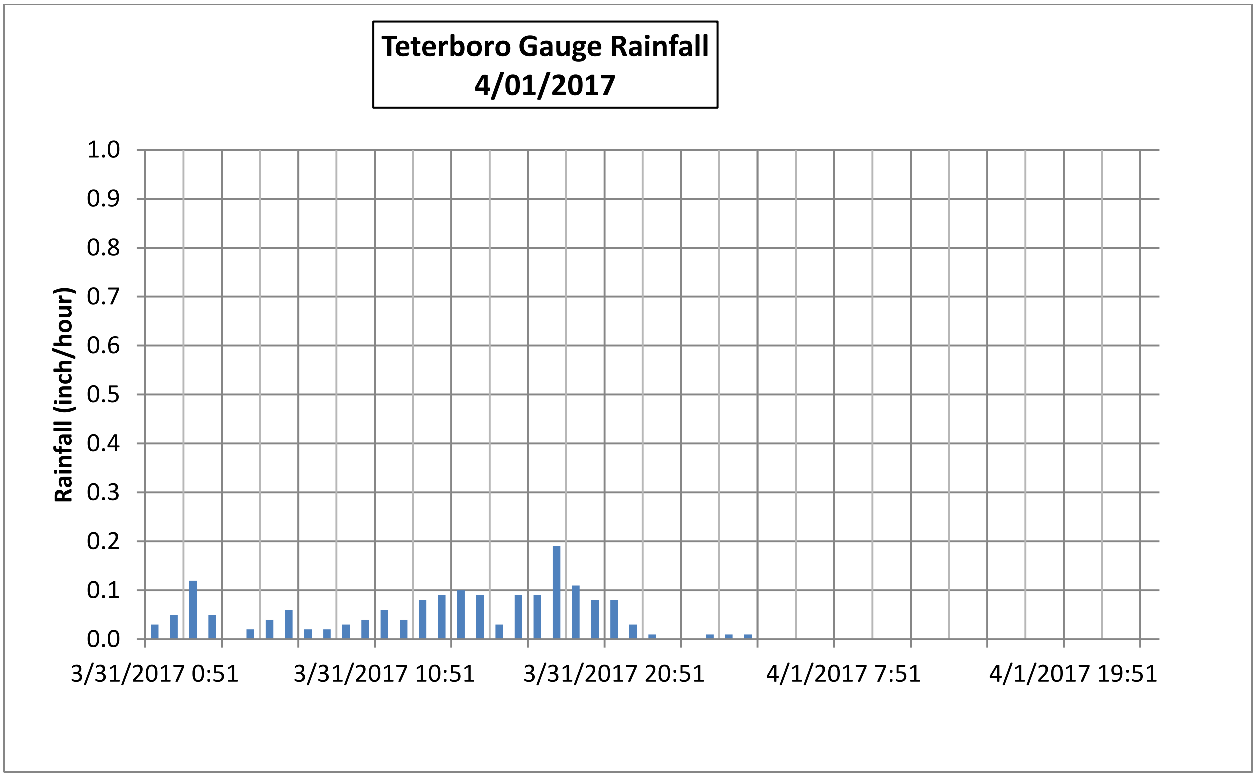
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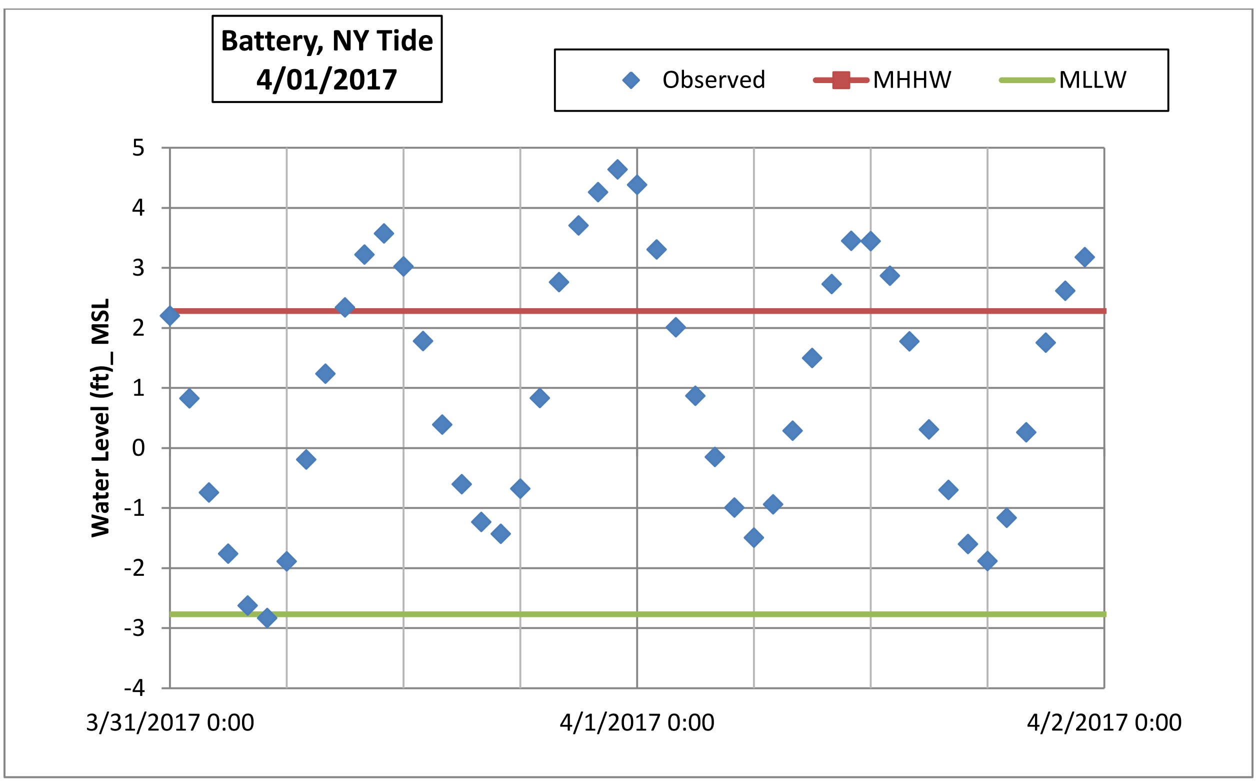
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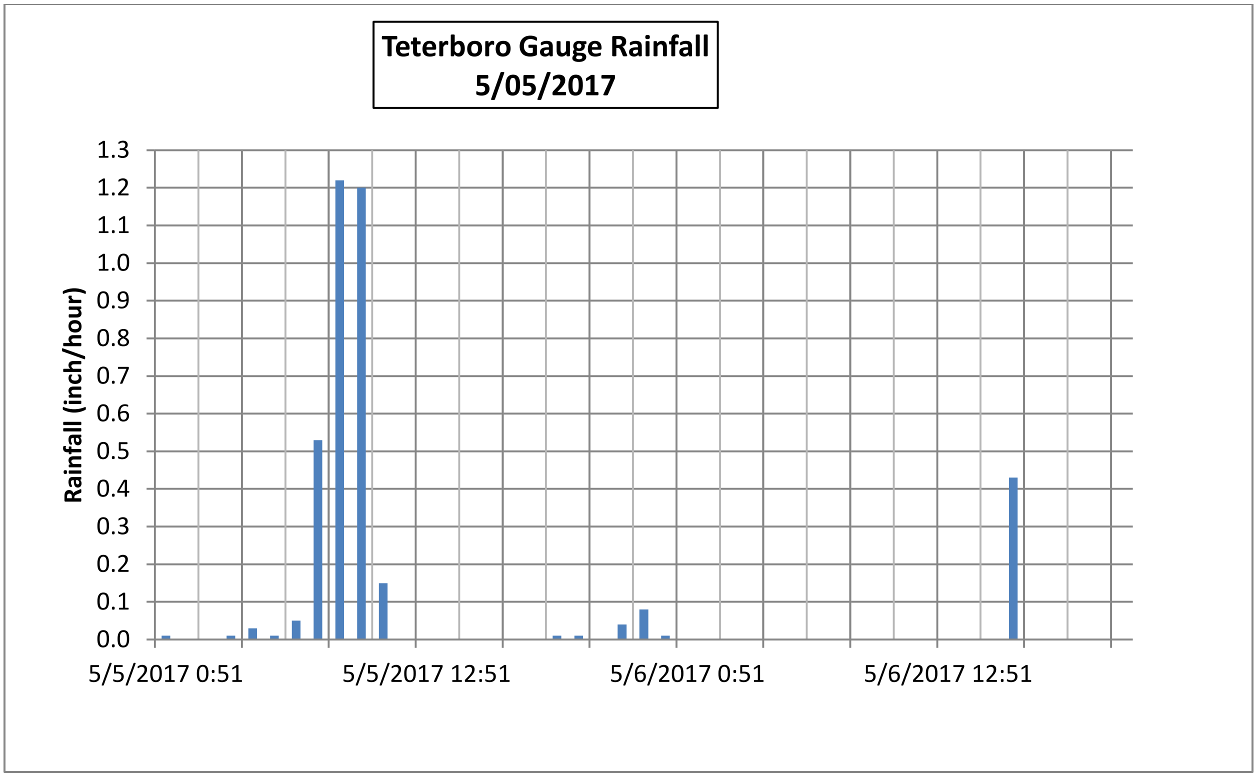
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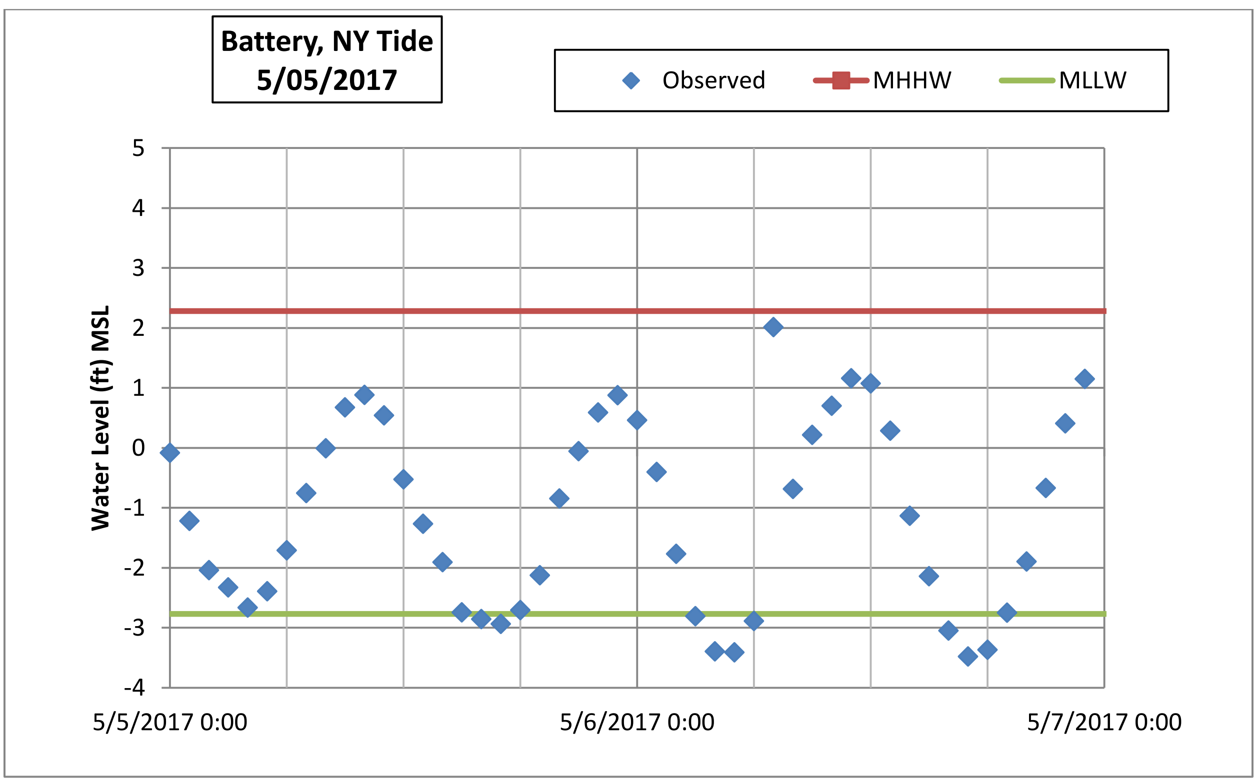
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Subappendix B6: Preliminary Screening Hydraulic Modeling Report

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Table of Contents

List of Figures

List of Tables

Acronyms and Abbreviations

1-D	One-dimensional
2-D	Two-dimensional
CAG	Citizen Advisory Group
cfs	Cubic feet per second
CMP	Corrugated metal pipe
DEM	Digital Elevation Model
DEP	DePeyster Creek
DIA	Drainage Improvement Area
ERD	East Riser Ditch
gpm	Gallons per minute
HEC-HMS	Hydrologic Engineering Center - Hydrologic Modeling System
HEC-RAS	Hydraulic Engineering Center - River Analysis System
H&H	Hydrology and Hydraulics
H:V	Horizontal to Vertical slope
LiDAR	Light Detection and Ranging
LOD	Level of Development
LOS	Losen Slote
MLW	Mean Low Water
MHW	Mean High Water
NAD 83	North American Datum of 1983
NAVD 88	North American Vertical Datum of 1988
NCCHE	National Center for Computation Hydroscience and Engineering
NJSEA	New Jersey Sports and Exposition Authority
NJT	New Jersey Transit
PIC	Peach Island Creek
RBDM	Rebuild by Design Meadowlands
RCP	Reinforced concrete pipe
RS	River Station
US	United States
USEPA	US Environmental Protection Agency
USGS	US Geological Survey
TIN	Triangular Irregular Network
WRD	West Riser Ditch
WSEL	Water Surface Elevation

1.0 Introduction

As a part of the Proposed Project alternative development, The Rebuild by Design Meadowlands (RBDM) team (referred to in this subappendix as “The Design Team”) is tasked to identify a screening process through which flood risk reduction concepts will be screened systematically to select the preferred alternative at the end of the screening process.

As a first step, using the readily available desktop data and local input, The Design Team identified thirty flood risk reduction concepts that can help achieve the Proposed Project goals. During the first phase of the screening, the Preliminary Screening Phase, The Design Team identified preliminary criteria including hydraulic feasibility of the concepts to screen 30 flood risk reduction concepts down to 15. To develop the metrics towards hydraulic feasibility, a simplified modeling approach using Hydrologic Engineering Center - River Analysis System (HEC-RAS) one-dimensional (1D) Steady State models (“models”) was developed. Hydraulic models were constructed for West Riser Ditch (WRD), East Riser Ditch (ERD), Losen Slote (LOS), DePeyster Creek (DEP), and Peach Island Creek (PIC) streams, within the Meadowlands District.

This report focuses on the simplified modeling approach for the five streams. For each stream, Drainage Improvement Areas (DIAs) were identified to locate flood risk reduction opportunities. These DIAs were derived using drainage area delineations represented in the hydrologic modeling effort. The goal of the simplified hydraulic modeling effort is to evaluate the flood risk reduction concepts in each DIA and screen them using the metrics developed in the Preliminary Screening Phase. In addition, simplified modeling approach also prioritizes concepts that need to be evaluated via detailed methods to further refine the concepts in the subsequent screening phases. These models described in this report are part of the Phase 1 analysis shown in **Figure B6-1**. The results of simplified models provide input to further refine the evaluation process to be completed in Phase 2 of this project.

Figure B6-2 shows all the DIAs in the Project Area. Only 16 out of 30 concepts were analyzed using the HEC-RAS simplified modeling approach. The remaining 14 were analyzed using Infoworks modeling.

Each of the following sections describe specific modeling methodologies for establishing existing and proposed conditions, as well as assumptions and limitations for each stream specific hydraulic model. Stream-specific information including general stream details such as location, overview of hydrology, and hydraulic features and a description of the data inputs and gaps, assumptions used to establish existing conditions, and scenarios representing the potential proposed conditions, evaluation of results, and selection criteria were included for each stream. Proposed conditions models reflect the 16 concepts evaluated in this modeling approach. These concepts generally include dredging, channelization, increased conveyance capacity of bridges and culverts, and pumping.

The simplified modeling results are not intended for design or cost estimating purposes.

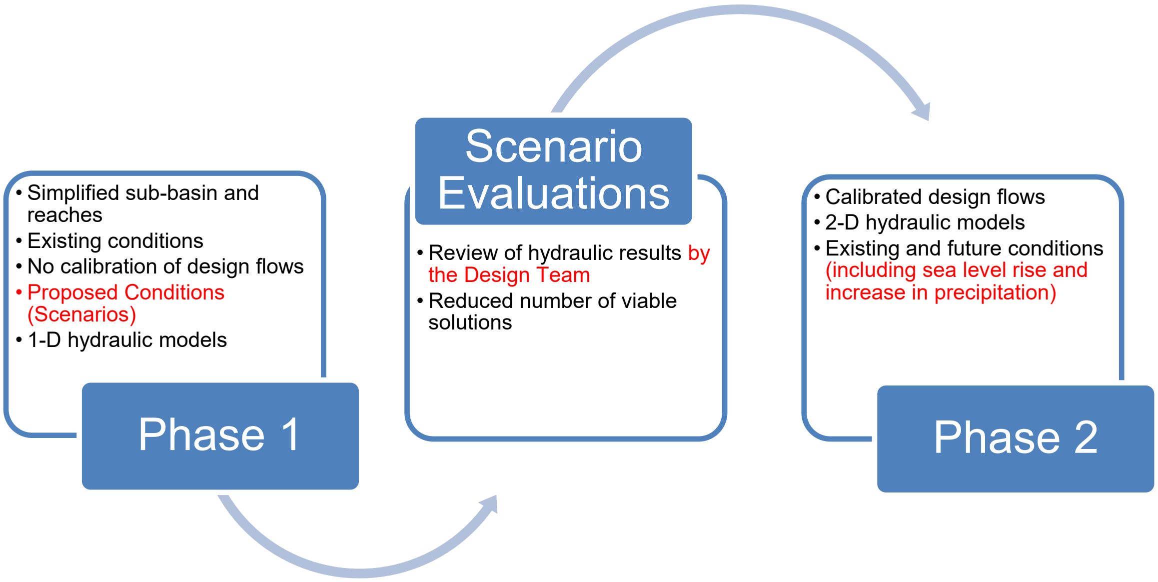

Figure B6-1: Model Process

Note: This report describes the hydraulic modeling performed in Phase 1.

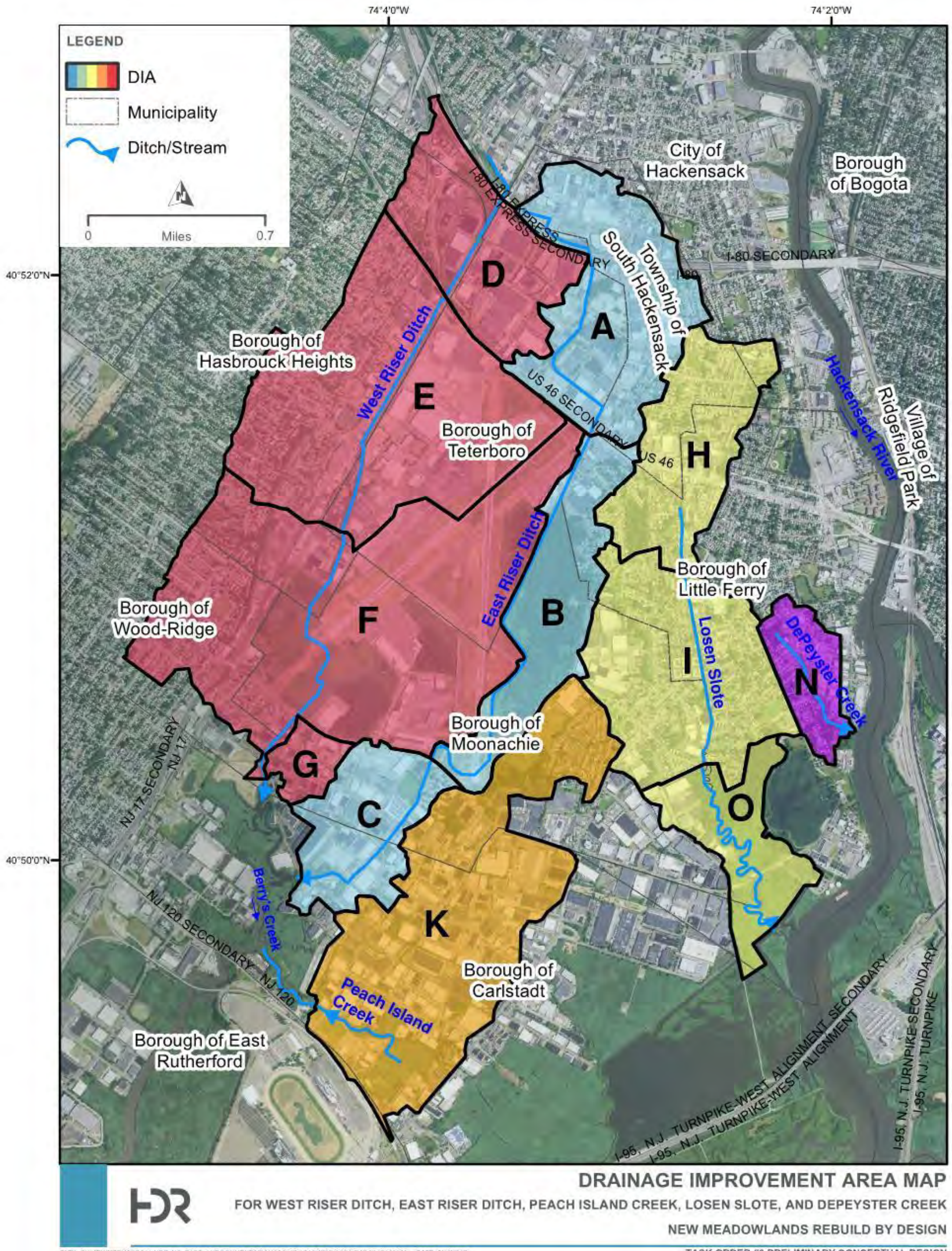

Figure B6-2: Meadowlands District Overview

2.0 West Riser Ditch

West Riser Ditch (WRD) is located in the City of Hackensack, Borough of Hasbrouck Heights, Borough of Teterboro, Borough of Wood-Ridge, Township of South Hackensack, and Borough of Moonachie. The length of WRD is approximately three miles, beginning north of Route 80 in Teterboro, and ending downstream of the existing WRD Tide Gate, where the stream joins with the East Riser Ditch (ERD), and transitions to Berry's Creek. Most of the Teterboro Airport is located in the WRD drainage area. WRD was divided into four DIA, namely D, E, F, and G, where proposed changes were evaluated for flood management benefits. These DIA are shown in **Figure B6-3**.

Most of the WRD drainage area in the City of Hackensack, above Interstate 80, is not within a defined drainage improvement area. While this area contributes flow to the WRD, flood management benefits are not evaluated in this area because historical studies reviewed and information gathered from the Citizen Advisory Group (CAG) did not identify flood-prone locations. The Green Street area was identified and modeled as part of this effort. Drainage Improvement Area “D” is located between Interstate 80 and US Route 46. The northern boundary of DIA "E" is US Route 46. The east side of this area is drained by a parallel drainage ditch within the Teterboro Airport. The airport drainage ditch enters the WRD via the Vincent Place pump station, which is the southern boundary of DIA “E.” DIA "F" is located between Vincent Place pump station and West Riser tide gate. DIA "G" is a developed area between Moonachie Avenue and West Riser tide gate. **Table B6-1** provides a summary of each DIA.

Figure B6-4 shows the key hydraulic features of the WRD. WRD passes underneath an overpass at Interstate 80. The overpass contains the WRD, the New Jersey Transit (NJT) rail, and Green Street. A foot bridge is located underneath US Route 46, followed by culverts at Williams Street. The NJT rail runs parallel to the east of WRD until just north of Malcolm Avenue. The WRD passes underneath the rail embankment through culverts and then underneath Malcolm Avenue. Flows from the Teterboro Airport interior drainage ditch are pumped into the WRD at Vincent Place. South of Vincent Place, WRD passes under Industrial Avenue, an airport access road, Moonachie Avenue, and a rail bridge just upstream of the West Riser tide gate.

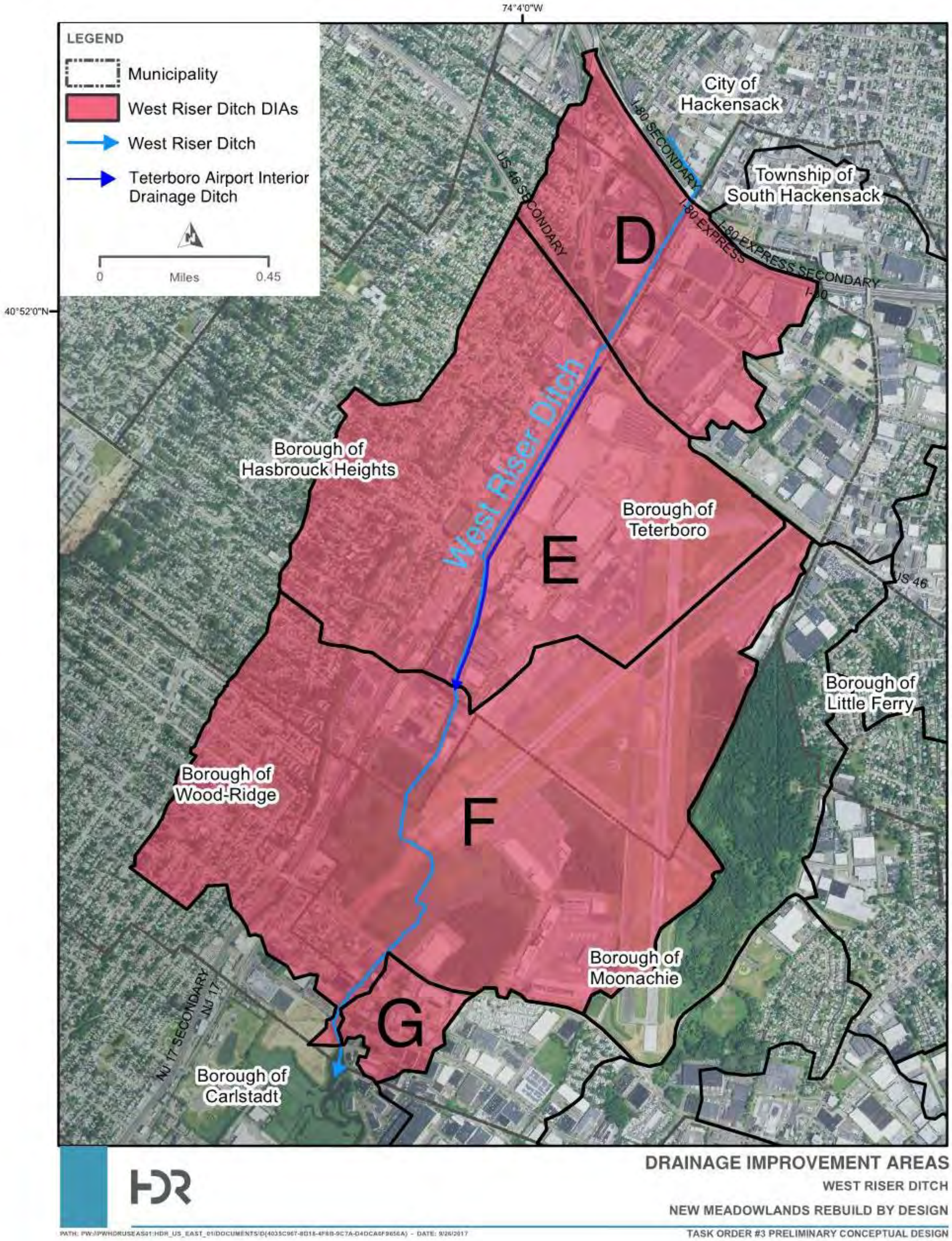

Figure B6-3: WRD Drainage Improvement Areas

Table B6-1: Summary Statistics of WRD Drainage Improvement Areas

Drainage Improvement Area	Location	Drainage Area [square miles]
D	Between Interstate 80 and US Route 46	0.37
E	Between US Route 46 to Vincent Place pump station	0.91
F	Vincent Place pump station to West Riser tide gate	1.27
G	Southeast drainage area between Moonachie Avenue and West Riser tide gate	0.07
Total Drainage Area		**3.91**

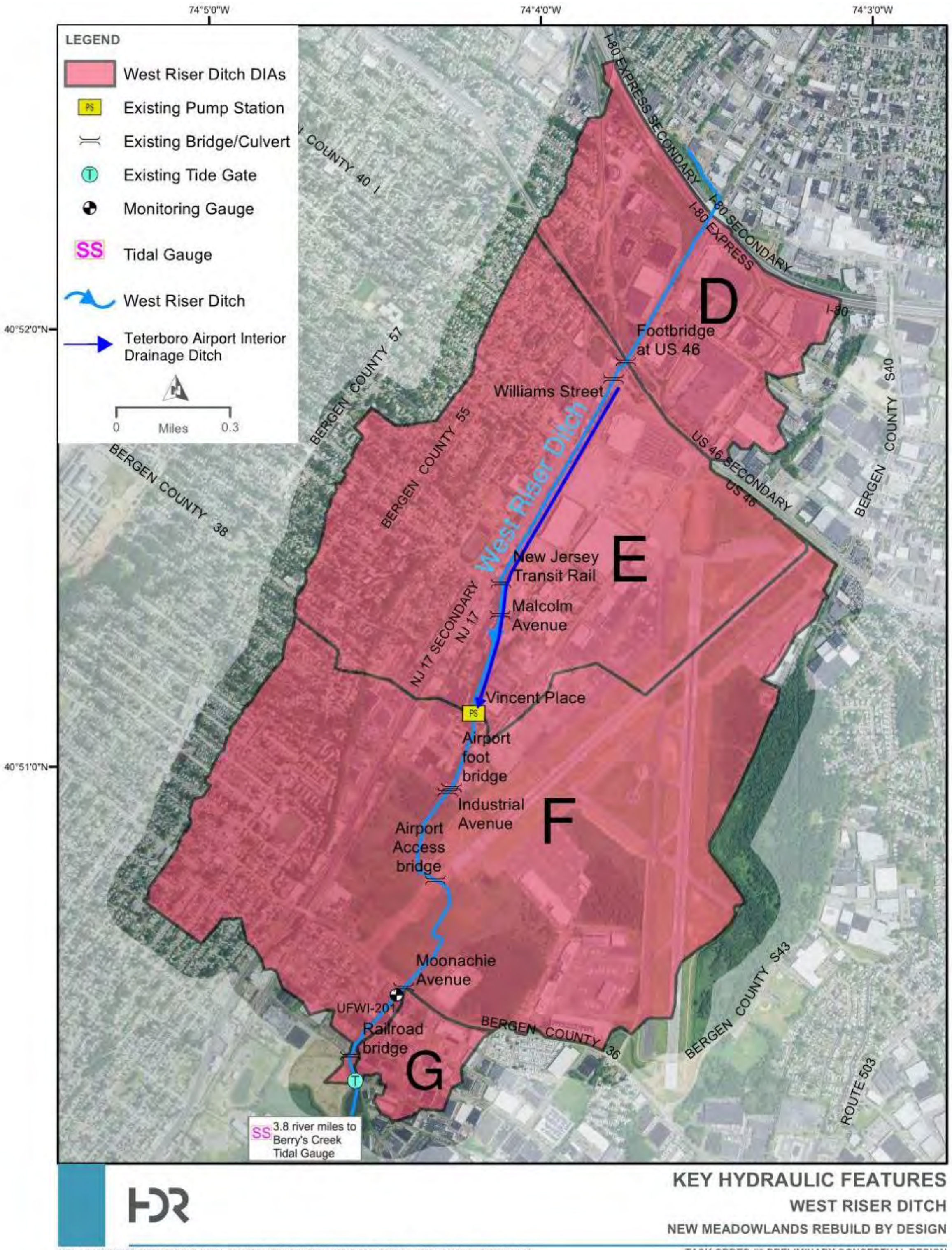

Figure B6-4: WRD Key Hydraulic Features

2.1 Data Sources

2.1.1 Survey

The survey of ditch cross sections and structure information was performed by Matrix New World Engineering. The survey for WRD was conducted between November 7, 2016 and November 15, 2016. Field survey gathered information at 17 cross sections and 4 structures within DIA “D” and “E” (see **Figure B6-5** for locations). Surveyed cross sections were used to describe channel conditions. No surveys were conducted between Williams Street and the NJT railroad culverts due to site access restrictions by NJT. Data from a separate hydraulic study by Jacobs Engineering was used downstream of Malcom Avenue; no survey data was collected in the areas covered by this study (Jacobs Civil Consultants 2013).

2.1.2 LiDAR

A LiDAR DEM with 2-foot pixel resolution was supplied by the New Jersey Sports and Exposition Authority (NJSEA). Quantum Spatial collected the near-infrared data in April 2014 and the LiDAR was derived by using TIN processing. The horizontal datum is NAD83 (2011) and the data was projected to the New Jersey State Plane coordinate system. Elevation units are feet in the NAVD 88. The LiDAR terrain data was used for the overbank cross sections with survey data used to define the channel areas.

2.1.3 Design Flows

The design flows for the WRD were sourced from the “Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) Model.” The Proposed Project specifications used 48-hour durations for the 2-year (50 percent annual exceedance probability) to 100-year (1 percent annual exceedance probability) reoccurrence intervals.

2.1.4 Tidal Conditions

Downstream tidal conditions were obtained from a study by the Woods Hole Group commissioned by the New Jersey Sports and Exposition Authority (NJSEA) (Woods Hole Group 2007). Model boundary conditions downstream of the West Riser tide gate were defined as the Mean Low Water (MLW) condition from the 1983 to 2001 epoch. The nearest tidal gauge to the WRD is the Berry’s Creek Canal, located approximately 3.8 river miles downstream of the West Riser tide gate. The MLW is -2.48 feet NAVD 88, while the Mean High Water (MHW) is 3.03 feet NAVD 88.

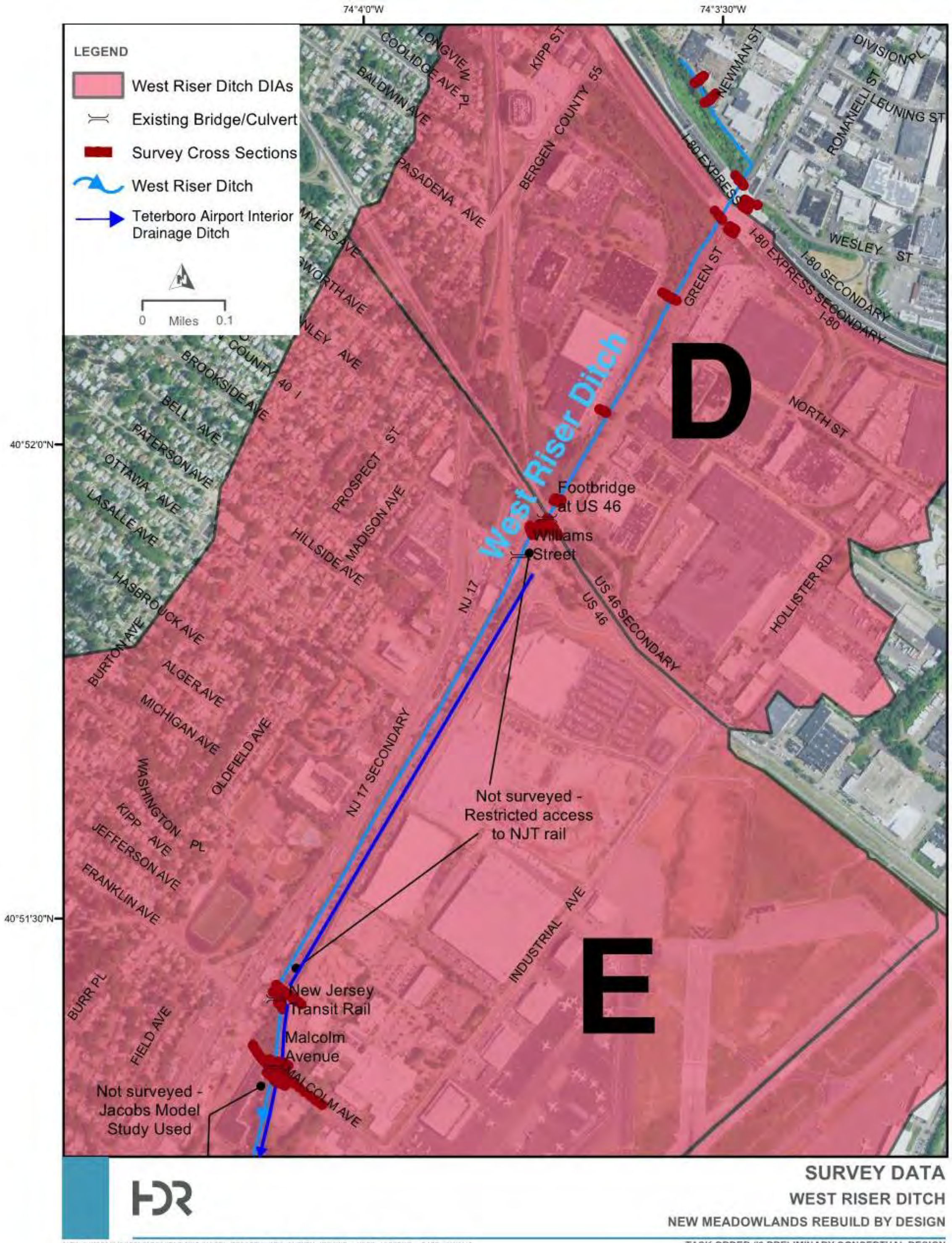

Figure B6-5: WRD Surveyed Cross Section Locations

2.1.5 Tide Gates and Pump Stations

The West Riser tide gate was replaced in 2014. Survey of the West Riser tide gate was not completed at the time the simplified model was being developed, although a description of the new gate was found in a

New Jersey Department of Community Affairs press release (New Jersey Department of Community Affairs 2014):

> *"The original West Riser Tide Gate was constructed in 1977 by Bergen County to prevent high tides from reaching upstream properties in Carlstadt, Moonachie, Teterboro and Wood-Ridge. The all metal structure was in need of replacement.*
>
> *The new West Riser Tide Gate is comprised of corrosion-resistant sheet piling with four (4) 6-feet wide by 7 feet long openings with composite flap valves attached on the downstream side. The new openings are larger than the old structure's to allow for better flow release at low tide.*
>
> *Additionally, the new structure was built to one-foot above the 25-year flood elevation to add additional storage capacity for storms. A corrosion-resistant trash rack system was installed upstream to prevent debris from flowing through the openings. Finally, a catwalk system was installed on top of the structure for maintenance access and includes four winch systems to open the gates for inspection."*

The Vincent Street pump station combines the airport drainage ditch with the WRD. The pump station has a maximum flow rate of 175 cubic feet per second (cfs), although field notes indicate an operational capacity of 112 cfs (Jacobs Civil Consultants 2013).

2.1.6 Other Studies and Models

A HEC-RAS model from Jacob's Engineering was appended to the area downstream of Malcom Avenue to downstream of the West Riser tide gate (Jacobs Civil Consultants 2013). No survey data was available to verify the supplemental HEC-RAS models from others.

2.2 Model Limitations

2.2.1 Survey

Submerged structure geometry, type, and, in some cases, locations were assumed as the surveyor was not scoped to perform underwater survey. The surveyor noted (Personal communication between Matrix New World surveyors and Pratik Desai (HDR Engineering, Inc.) 2016):

> *"We were to survey these areas the best we could using our methods but with limited sight line and several feet of silt and debris that we can't see under the water line we are limited in giving accurate numbers."*

The modeling assumptions were made for submerged or partially submerged structures, see **Table B6-2** for survey assumptions.

Table B6-2: Survey Assumptions for Structures along WRD

Structure	Notes/Limitations
Foot bridge (RS12155.76)	Only trash rack was visible. Top of trash rack on inlet side from survey point 2148. Inlet invert from survey point 2150. On outlet side, top of opening from survey points 2119 and 2117; opening width from these same points. Invert of outlet from survey point 2121.
Williams Avenue (RS11843.47)	On the inlet side, the top of pipe is from survey point 2480. The pipe was partially submerged. The surveyor estimated the culvert size as "Possibly 36 [inch] Pipe." The surveyor could not obtain permission to access the outlet side from NJT. The outlet pipe geometry was assumed the same as the inlet geometry. The outlet invert was assumed as the downstream boundary cross section thalweg elevation. Note that the downstream cross sections from here to the next crossing were not surveyed, also due to access permissions from NJT.
Railroad crossing, 480 feet upstream of Malcolm Avenue (RS8573.54)	Survey point 2414 on the downstream side provides the low chord. The width of the opening is from survey points numbers 2412 and 2415, which are separated by about 6.3 feet. The low chord on the upstream side was not measured; it is assume to be the same elevation as the downstream side. The upstream opening width is provided by survey point numbers 5067 and 5055.
Malcolm Avenue (RS8105.586)	On the upstream side, the low chord is defined by survey points: 2243, 2244, 2242, 2241, 2240 and 2372, 2369, 2368. The high chord on the upstream side is defined by survey points 2262, 2261, 2234, 2233 and 2380, 2379, 2382, 2381, 2384, 2383. On the downstream side the low chord is defined by survey points 2197, 2198, 2200, 2201 and 2322, 2323, 2328. The downstream side high chord is defined by survey points 2166, 2168, 2171 and 2350, 2351, and 2353.
Foot Bridge upstream of Industrial Ave Bridge (RS5593.253)	Not surveyed, from Jacobs HEC-RAS model station 5635
Industrial Ave Bridge (RS5507.788)	Not surveyed, from Jacob HEC-RAS model station 5550
Airport Access Road Bridge (RS4009.538)	Not surveyed, from Jacobs HEC-RAS model station 4048; slight (0.1 ft) adjustment to deck geometry at opening to resolve fatal HEC-RAS error "The GUI sent a vertical wall that has no width. The wall goes up and then down (or down and up) without moving sideways."
Moonachie Ave (RS2090.202)	Not surveyed, from Jacobs HEC-RAS model station 2131
Railroad Bridge, about 390 feet upstream of tide gate (RS860.7379)	Not surveyed, from Jacobs HEC-RAS model station 921.5

2.3 Existing Conditions

An existing conditions HEC-RAS model was created for WRD running from north of Interstate 80 to just downstream of the West Riser tide gate. This model acts as a baseline conditions for which Proposed Project scenarios were compared.

2.3.1 Development of Existing Condition Cross Sections

For the upstream-most 7,350 feet, within DIA “D” and a portion of DIA “E,” cross sections were co-located with field surveyed cross sections described in **Section 2.1.1**. For the remaining downstream 7,150 feet, within a portion of DIA “E” and DIAs “F” and “G,” cross section locations were co-located with Jacobs cross sections (Jacobs Civil Consultants 2013). For all cross sections, overbank areas were extracted from LiDAR data. The channel area was adjusted based on survey or the Jacobs model (Jacobs Civil Consultants 2013). The HEC-RAS cross section beginning at the most downstream section of WRD is located at River Station (RS) 456.9081 and the most upstream cross section ends at RS 15413.91.

Figure B6-6, **Figure B6-7**, and **Figure B6-8** show the cross sections defined in the HEC-RAS model.

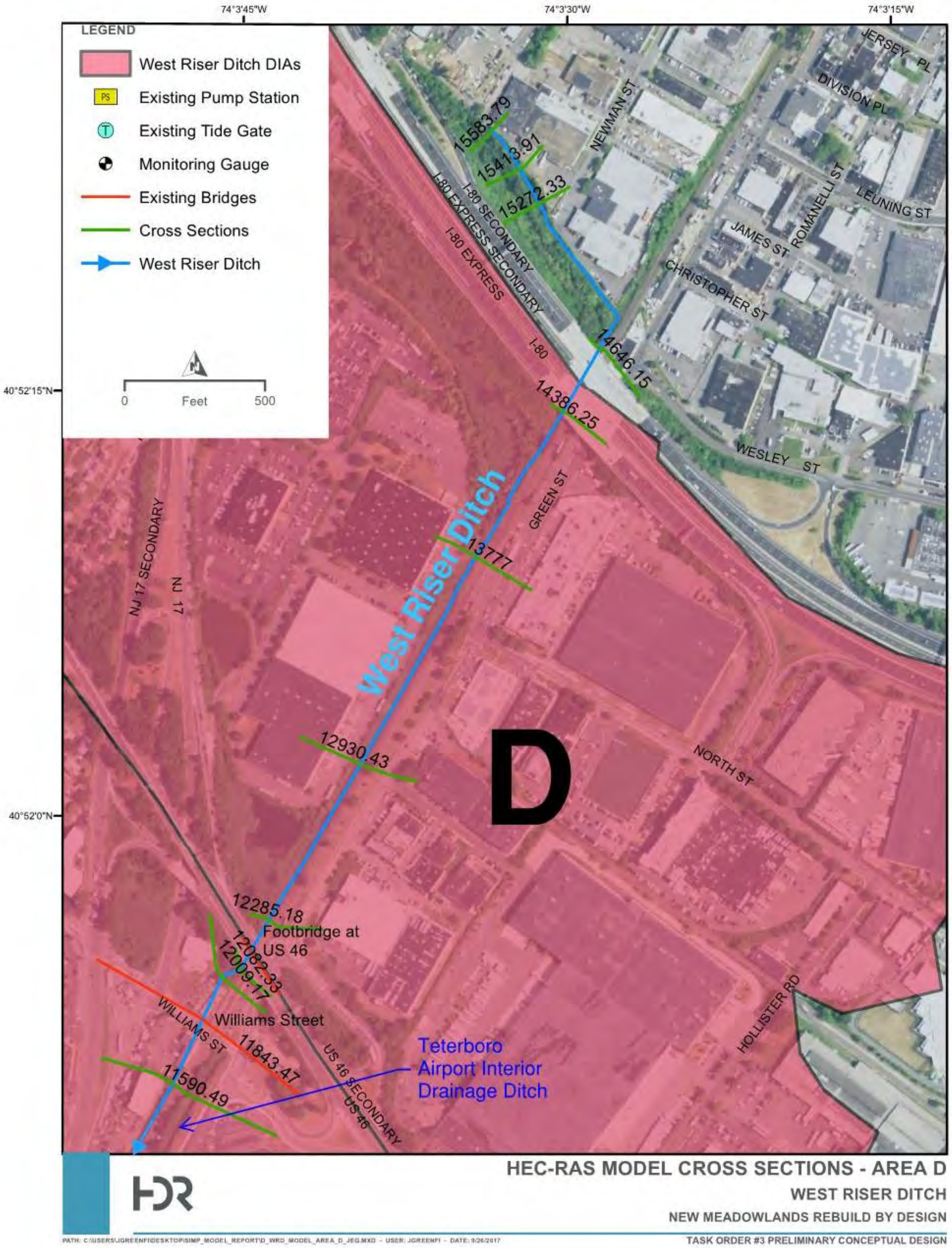

Figure B6-6: WRD HEC-RAS Model Cross Sections for DIA 'D'

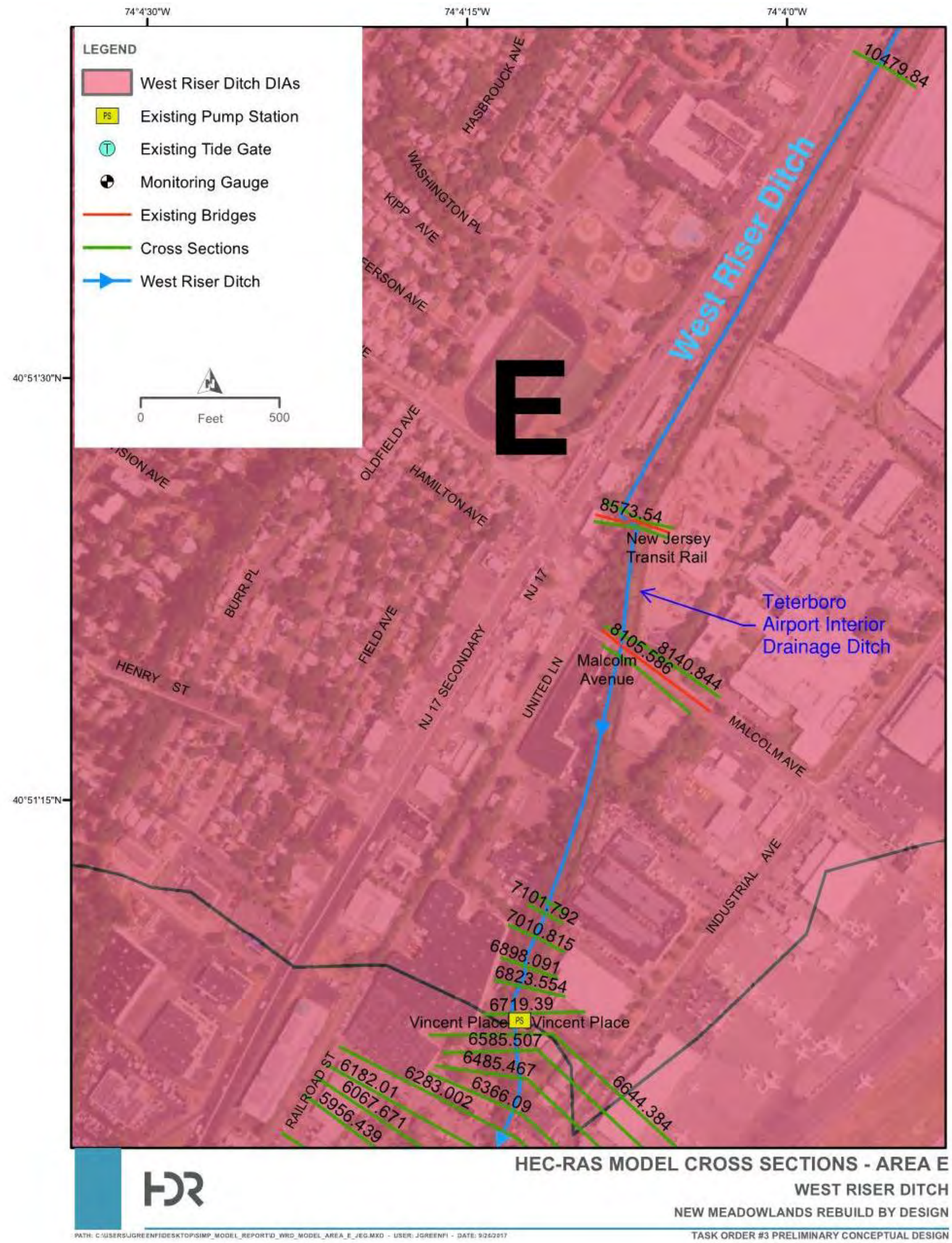

Figure B6-7: WRD HEC-RAS Model Cross Sections for DIA 'E'

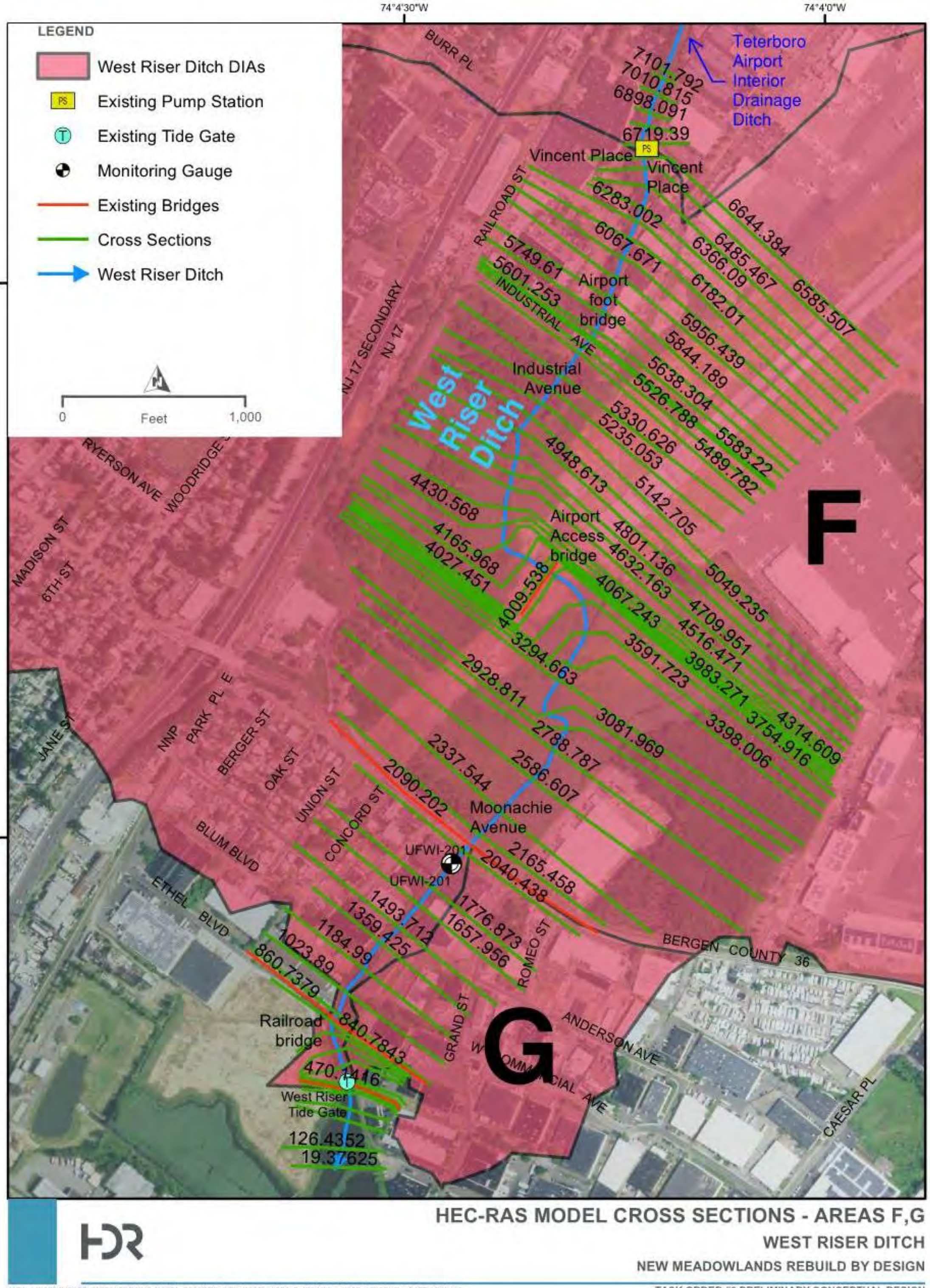

Figure B6-8: WRD HEC-RAS Model Cross Sections for DIAs ‘F’ and ‘G’

2.3.2 Steady Flow Data

2.3.2.1 Flow Breakout Locations

The flow rates beginning at the 2-year reoccurrence interval (50 percent annual chance exceedance) overtopped the NJT railroad embankment separating the WRD and Teterboro Airport interior drainage flows. For all flow events considered, it was assumed that the WRD and airport interior drainage ditch flows were combined, and the Vincent Place pump station did not have any influence on the system.

2.3.2.2 Assignment of HEC-HMS Flows to RAS Cross Sections

Additional flow change locations were developed using a drainage area ratio applied to the flow reporting locations in the Phase 1 HEC-HMS model. See **Table B6-3** below for additional details.

Table B6-3: Drainage Area Ratios for WRD

HEC-HMS Reach Flows	+	Drainage Area Portion…	…Of HEC-HMS Sub-basin	= flow at HEC-RAS Cross Section Station
---	+	100%	West Riser Ditch - North	15583
R_WR-Main	+	67%	West Riser Ditch - Main Street	12285
R_WR-Main	+	100%	West Riser Ditch - Main Street	11590
R_WR-West	+	71%	West Riser Ditch – Central	8630
		100%	West Riser Ditch – PS	
R_WR-West	+	100%	West Riser Ditch – Central and West Riser Ditch – PS	6719
P_WRD_PS+ R_WR-South	+	6%	West Riser Ditch - South	5526
P_WRD_PS+ R_WR-South	+	51%	West Riser Ditch - South	4027
P_WRD_PS+ R_WR-South	+	100%	West Riser Ditch - South	2165
R_WR-Grand	+	100%	West Riser Ditch - Grand	896

Note: Between HEC-RAS cross sections 8630 and 6719, the WRD and airport interior drainage flows are combined under the assumption that the intervening rail road embankment is overtopped.
Not intended for construction.

2.3.3 Ineffective Flow Areas

Ineffective widths were set based on the distance between the upstream or downstream cross section and the structure, and a ratio of 1:1 (channel length: ineffective width) upstream or 2:1 downstream. Ineffective elevations upstream of a structure were initially set 0.1 feet higher than the upstream deck low point. At the cross section downstream of a structure, ineffective flow heights were based on the average of the highest low chord and lowest roadway surface elevation. Through applying a number of discharges to the system, ineffective flow heights were iteratively adjusted such that both upstream and downstream ineffective flow elevations were exceeded at a consistent discharge.

2.3.4 Culverts and Bridge Openings

There are eight structures (culverts and bridges) along the WRD reach length. Four of the structures were defined based on site specific topographic and structure survey. Deck elevations were extracted from the LiDAR datasets and survey data. Bridge low chords, culvert sizes, pipe inverts, and upstream and downstream cross section were extracted from surveyed data. The field surveyors noted uncertainty in some structure measurements, as detailed in **Table B6-2**. The geometries for the remaining four structures were obtained from the existing Jacobs model (Jacobs Civil Consultants 2013).

Bridges and culverts were assumed perpendicular to the direction of the flow. The pressure and/or weir equations were used to model the structures. Contraction and expansion coefficients were set to 0.3 and 0.5, respectively, upstream and downstream of culverts, bridges, or tide gates. The remaining cross section contraction and expansion coefficients were set to 0.1 and 0.3, respectively.

The following structures and their data sources included in the WRD model are listed in **Table B6-4**. Information for the structures that were not surveyed was obtained from the existing Jacobs model (Jacobs Civil Consultants 2013). These structures were not field validated.

Table B6-4: WRD – Existing Structures and Data Sources

Structure Location (RS)	Description	Source
12155.76	Foot Bridge	Survey
11843.47	William's Avenue Crossing	Survey
8573.54	Railroad Crossing	Survey
8105.586	Malcolm Avenue Crossing	Survey
5593.253	Foot Bridge	Jacobs
5507.788	Industrial Avenue Bridge	Jacobs
4009.538	Access Road Bridge	Jacobs
2090.202	Moonachie Avenue Crossing	Jacobs
860.7379	Railroad Bridge	Jacobs

Not intended for construction.

2.3.5 Pump Station

Drainage from the Teterboro Airport is collected in an internal drainage ditch between RS 11590.49 to RS 6719.39. This internal drainage ditch runs parallel to the WRD, separated by the NJT rail embankment. Vincent Place pump station is located at RS 6719.39, which pumps the internal drainage ditch flows into the WRD. The HEC-HMS model provided the pumping station flows, along with potential for overbank storage on the airport property. The potential for breakout flows between the WRD and airport interior drainage ditch was evaluated in **Section 2.3.2.1**.

2.3.6 Tide Gate Configuration

The West Riser tide gate was modeled as an inline structure with gates. The gates were assumed fully open for the MLW condition. A fixed water surface elevation (WSEL) equal to the MLW condition was used as the boundary condition downstream of the tide gate.

2.3.7 Manning's Roughness Coefficients

Manning's roughness coefficients were allocated based on year 2015 aerial imagery, as noted in **Table**

B6-5.

Table B6-5: WRD - Manning's Roughness Coefficients

Manning's n value	Description
0.03	Developed: buildings/pavement
0.035	Straight ditch segments
0.04	Sinuous ditch segments; grass overbanks
0.05	Mix of trees and grass
0.06	Wetlands
0.07	Dense trees, isolated areas
0.10	Dense trees, large areas

2.4 Proposed Conditions

2.4.1 Description of Proposed Project Scenarios

Three Proposed Project scenarios were simulated, as detailed in **Table B6-6**. Proposed conditions included additional pump stations, channel excavation, and increases in culvert sizes. Various combinations of these potential improvements were simulated in the different DIAs to evaluate system response.

Table B6-6: WRD Proposed Project Scenarios

Scenario Number	HEC-RAS Plan Name	Description
1	PRP_MLW_S1GALLYN YN0HS	Proposed pump station at tide gate
2	PRP_MLW_S2DEFALLYN0HS	DIA D: Excavate channel, increase culvert size DIA E: Excavate channel, increase culvert size DIA F: Excavate channel, increase culvert size
3	PRP_MLW_S3DEFGALLYN0HS	DIA D: Excavate channel, increase culvert size DIA E: Excavate channel, increase culvert size DIA F: Excavate channel, increase culvert size DIA G: Pump station at tide gate

Not intended for construction.

2.4.2 Proposed Conditions: Channel Modifications

Channels were modified in DIAs D, E, and F for Scenarios 2 and 3. No channel modifications were proposed for Scenario 1. Generally, existing cross sections were excavated to increase conveyance capacity. The proposed channel cross sections were defined by trapezoidal shape, maximum channel width, channel side slope, and channel invert. The top channel width was based on an assumed right-of-way that started 15 feet from the edge of existing buildings and parking lots. Where the WRD top widths were less restricted by existing infrastructure, top widths were assumed to increase by a maximum of 150 percent. **Table B6-7** shows the proposed channel cross section for each DIA.

The channel inverts were lowered in areas to promote a positive slope. **Figure B6-9** shows the change in proposed channel inverts. Note that filling was generally not implemented, so areas of likely scour holes were left as is.

Table B6-7: WRD Proposed Channel Modifications

Drainage Improvement Area	Proposed Channel Side Slopes
D	2H:1V
E	2H:1V
F	3H:1V

Not intended for construction.

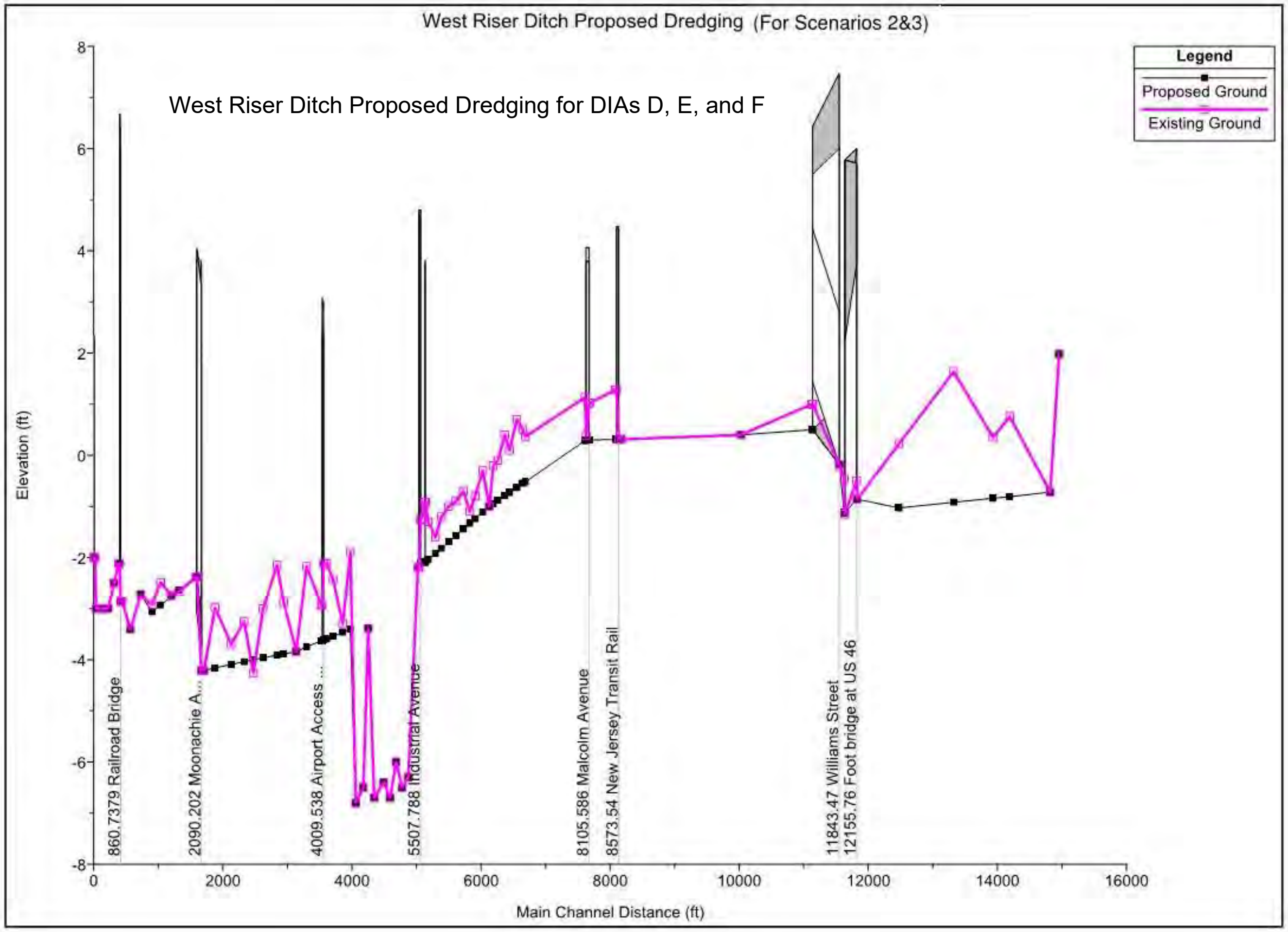

Figure B6-9: WRD Channel Profile

Not intended for construction.

2.4.3 Proposed Conditions: Culvert/Bridge Opening Modifications

Proposed culvert replacements in WRD used a concrete box culvert configuration. The maximum size of the replacement culvert required one foot of fill at the sides of the culvert and 18 inches of fill over the top of the culvert. **Table B6-8** provides details on culvert and bridge modifications. Ineffective flow area locations and elevations were adjusted to represent the modified openings.

Table B6-8: WRD - Proposed Culvert and Bridge Modifications

Location	Existing Configuration	Proposed Configuration
RS 12155.76 (footbridge)	Assumed low chord 2.25 feet above channel bottom	Increase low chord by 2 feet
RS 11843.47	36” corrugated metal pipe (CMP)	4’ x 5’ box culvert
RS 8573.54	6.3' x 4.2' bridge	No change
RS 8105.586	15’ x 3’ bridge	16' x 3.5' bridge
RS 5593.253 (footbridge)	40’ x 4.75’ bridge	No change
RS 5507.788	36’ x 5.3’ bridge	No change
RS 4009.538	21.7’ x 5’ bridge	No change
RS 2090.202	2 barrels 12.67' x 7.1' arch	24' x 5.5' box culvert
RS 860.7379	48' x 5.9' bridge	No change

Not intended for construction.

2.4.4 Proposed Conditions: Pump Stations

The maximum pumping size is assumed to be based on the existing maximum pump station capacity in the WRD area. The existing Teterboro Airport pump station (Vincent Place) has a capacity of 175 cfs (operationally limited to 112 cfs) (Jacobs Civil Consultants 2013) and has a drainage area of 255 acres[1]. Based on the 723 acre drainage area of the ERD, the proportionally assumed proposed maximum pumping capacity is 500 cfs. This same capacity was applied to the proposed pump station at the West Riser tide gate for Scenarios 1 and 3.

2.5 Model Results

2.5.1 Validation or Comparisons with other Studies

A US Environmental Protection Agency (USEPA) study for Berry’s Creek provided monitored stage and flow data that was used to validate the HEC-RAS existing condition (USEPA 2016). The USEPA monitoring location in the WRD was UFWI-201, located downstream of Moonachie Avenue near HEC-RAS model cross section RS 2040.

Peak flows and stages were digitized from **Attachment D1, Figure 1** and **Attachment D3, Figure 1** of the USEPA report (USEPA 2016). The pairing of this data produced a rating curve that could be compared with the simulated model rating curve. There was variability in the rating curve data, presumably due to variations in the tidal boundary conditions. An average rating curve was fitted with the monitoring data. The stage was converted to a WSEL based on the cross section RS 2040 invert elevation assumed as stage of 0.0 feet.

[1] Drainage area from the Phase 1 HEC-HMS model (dated October 31, 2016) for DA-WestRD-PS sub-basin.

$$E = 0.62226169 * Q^{0.34805167} - 2.38$$

(Where E is the elevation (NAVD 88) in feet, and Q is the flow in cfs.)

The digitized monitored rating curve data and comparison to the HEC-RAS existing condition model is in **Figure B6-10**. The peak stage monitored by USEPA did not exceed the channel bank.

2.5.2 HEC-RAS Modeling Results

Results showing the change in WSEL for the WRD Proposed Project scenarios for the 2-,5-, and 10-year return periods compared to existing conditions are displayed in **Table B6-9**. The lower return period storm events were considered more realistic targeted reductions for the WRD.

2.5.3 Proposed Project Scenarios Evaluation

Scenario 3 was selected for further evaluation because it represented the maximum WSEL reduction. Profile plots for the 2, 5, and 10-year storm events comparing existing and Scenario 3 WSEL are shown in **Figure B6-11**.

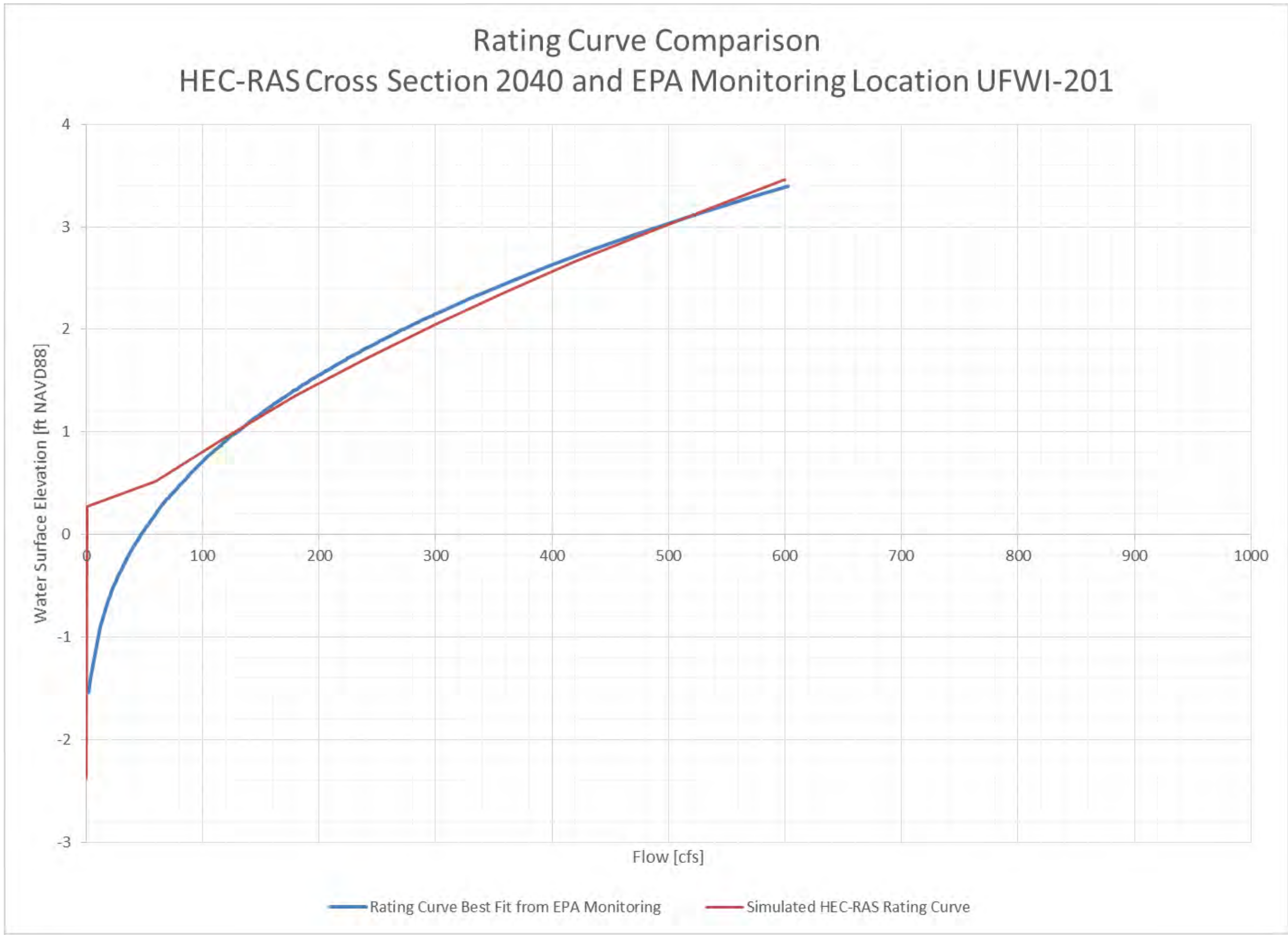

Figure B6-10: Validation of WRD at Moonachie Avenue from USEPA Based Rating Curve

Not intended for construction.

Table B6-9: Representative WRD Existing and Proposed WSELs

RS	Event	Existing WSEL [feet NAVD 88]	Change from Existing WSEL [feet]		
			Scenario 1	Scenario 2	Scenario 3
Upper Portion of Area D					
14386.25	2-yr	9.47	0.01	-0.19	-0.19
	5-yr	9.81	0.03	-0.17	-0.16
	10-yr	10.12	-0.04	-0.17	-0.24
Lower Portion of DIA “D”					
12285.18	2-yr	9.42	0.00	-0.19	-0.19
	5-yr	9.74	0.02	-0.17	-0.15
	10-yr	10.05	-0.06	-0.18	-0.25
Upper Portion of DIA “E”					
12082.33	2-yr	9.14	0.01	-0.07	-0.08
	5-yr	9.40	0.04	-0.05	-0.03
	10-yr	9.68	-0.07	-0.06	-0.15
Lower Portion of DIA “E”					
6719.39	2-yr	7.04	-0.61	-0.03	-0.68
	5-yr	8.24	-0.30	-0.01	-0.32
	10-yr	9.05	-0.23	-0.02	-0.24
Midway of DIA “F”/Upper Portion of DIA “G”					
2040.438	2-yr	6.83	-0.73	0.00	-0.74
	5-yr	8.24	-0.32	-0.01	-0.33
	10-yr	9.04	-0.23	0.00	-0.24
860.7379	**Railroad Bridge**				
840.7843	2-yr	4.80	-0.87	0.00	-0.87
	5-yr	5.60	-0.65	0.00	-0.65
	10-yr	6.16	-0.58	0.00	-0.58
476.5974	2-yr	4.55	-1.00	0.00	-1.00
	5-yr	5.33	-0.70	0.00	-0.70
	10-yr	5.87	-0.61	0.00	-0.61
470	**West Riser Tide Gate**				

Not intended for construction.

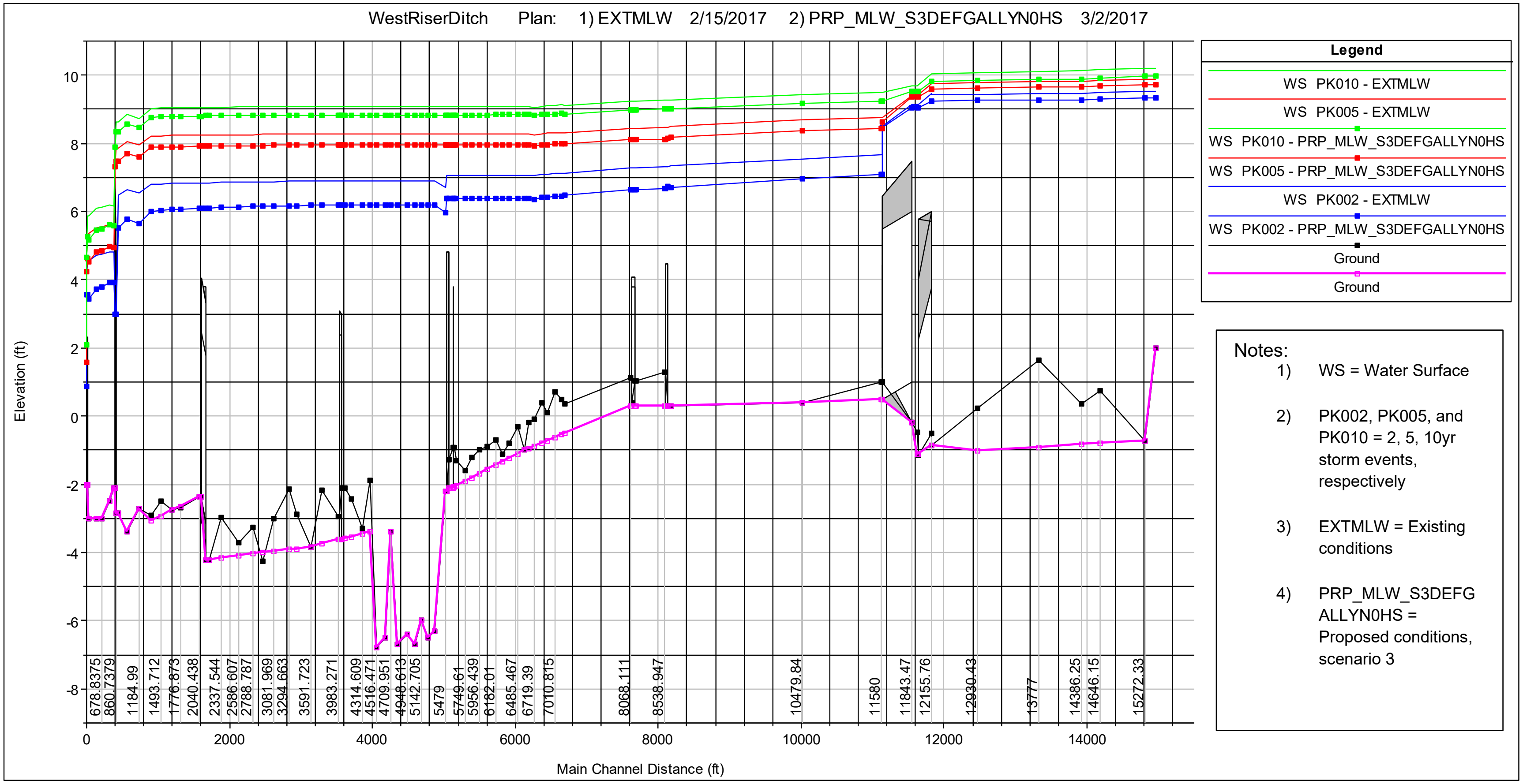

Figure B6-11: WRD WSEL Comparison between Existing and the Selected Proposed Scenario 3 for the 2, 5, and 10-Year Storm Events

3.0 East Riser Ditch

East Riser Ditch (ERD) is located within the Township of South Hackensack, the Borough of Teterboro, the Borough of Little Ferry, the Borough of Moonachie, and the Borough of Carlstadt. The watershed is divided into three DIAs, shown in **Figure B6-12**. DIA ‘A’ is located upstream of Interstate 80 primarily in the Township of South Hackensack. DIA ‘B’ starts at Interstate 80 and continues to US Route 46. This area is split between the Township of South Hackensack and the Borough of Teterboro. DIA ‘C’ is located between US Route 46 and the East Riser tide gate. Through the Teterboro Airport reach, the drainage area is constrained by the airport levees to the west and Redneck Avenue / Bergen County Highway S43 to the east. **Table B6-10** provides a summary of each DIA.

Table B6-10: Summary Statistics of ERD Drainage Improvement Areas

Drainage Improvement Area	Location	Drainage Area [square miles]
A	Upstream of US Route 46	0.51
B	Between US Route 46 and Moonachie Avenue	0.36
C	Downstream of Moonachie Avenue	0.27
Total Drainage Area		**1.14**

Figure B6-13 shows the key hydraulic features of the ERD. Modeling of the ERD begins approximately 0.3 miles upstream of the Interstate 80 crossing. ERD passes through two culverts associated with the entrance and exit ramps from Wesley Street, after which the ditch runs parallel to the Interstate. After passing through a culvert at Interstate 80, the ditch enters an underground pipe through a former railroad embankment. The ditch passes through culverts at North Street and then through two smaller crossings. At Central Avenue the ditch again enters an underground pipe. The pipe runs along Central Avenue prior to joining a larger pipe at Huyler Street / Bergen County S40. The ditch daylights downstream of US Route 46 and, at Huyler Street, a pump station provides flow to the ditch from 123 acres (Jacobs Civil Consultants 2009).

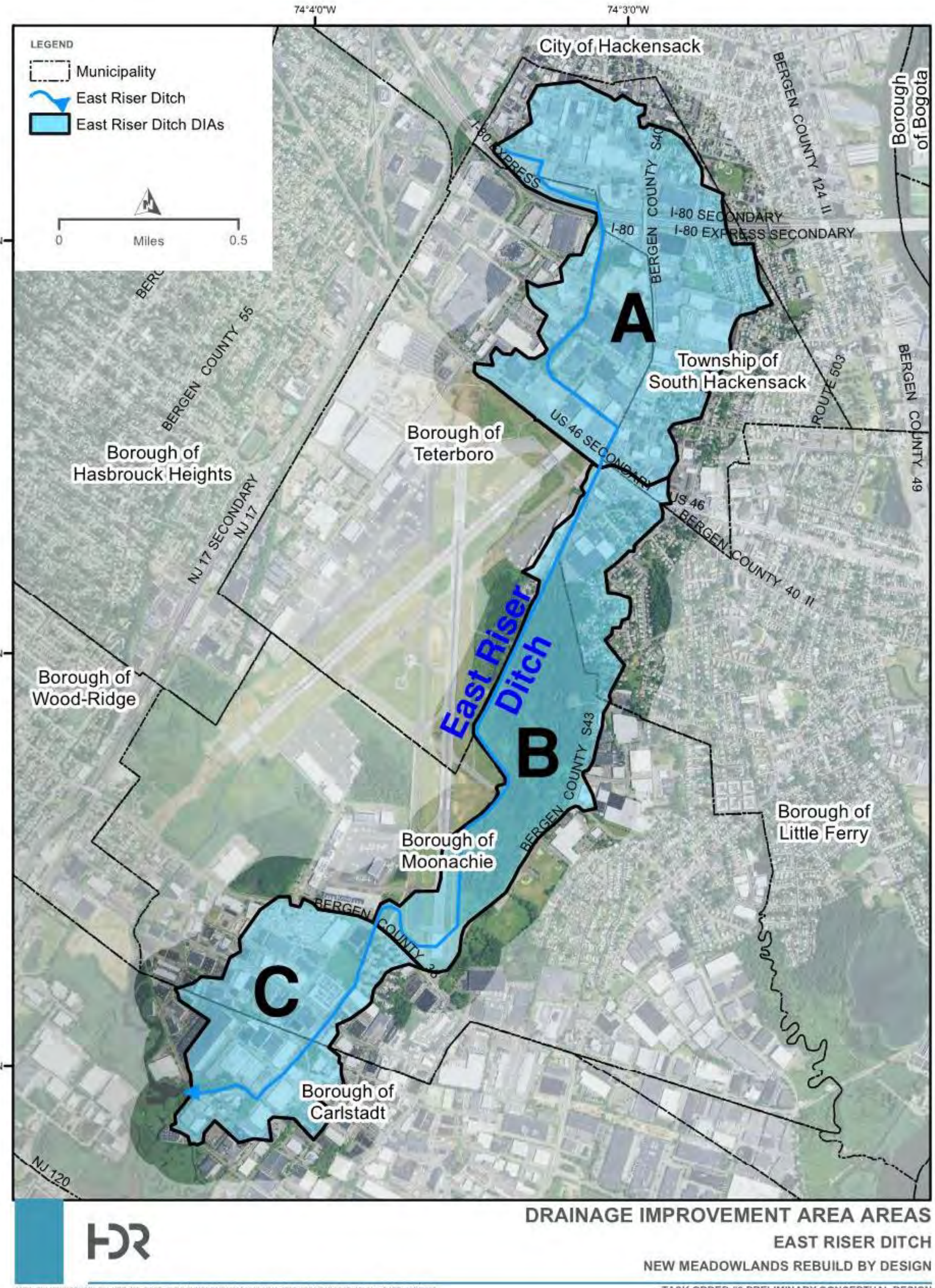

Figure B6-12: ERD Drainage Improvement Areas

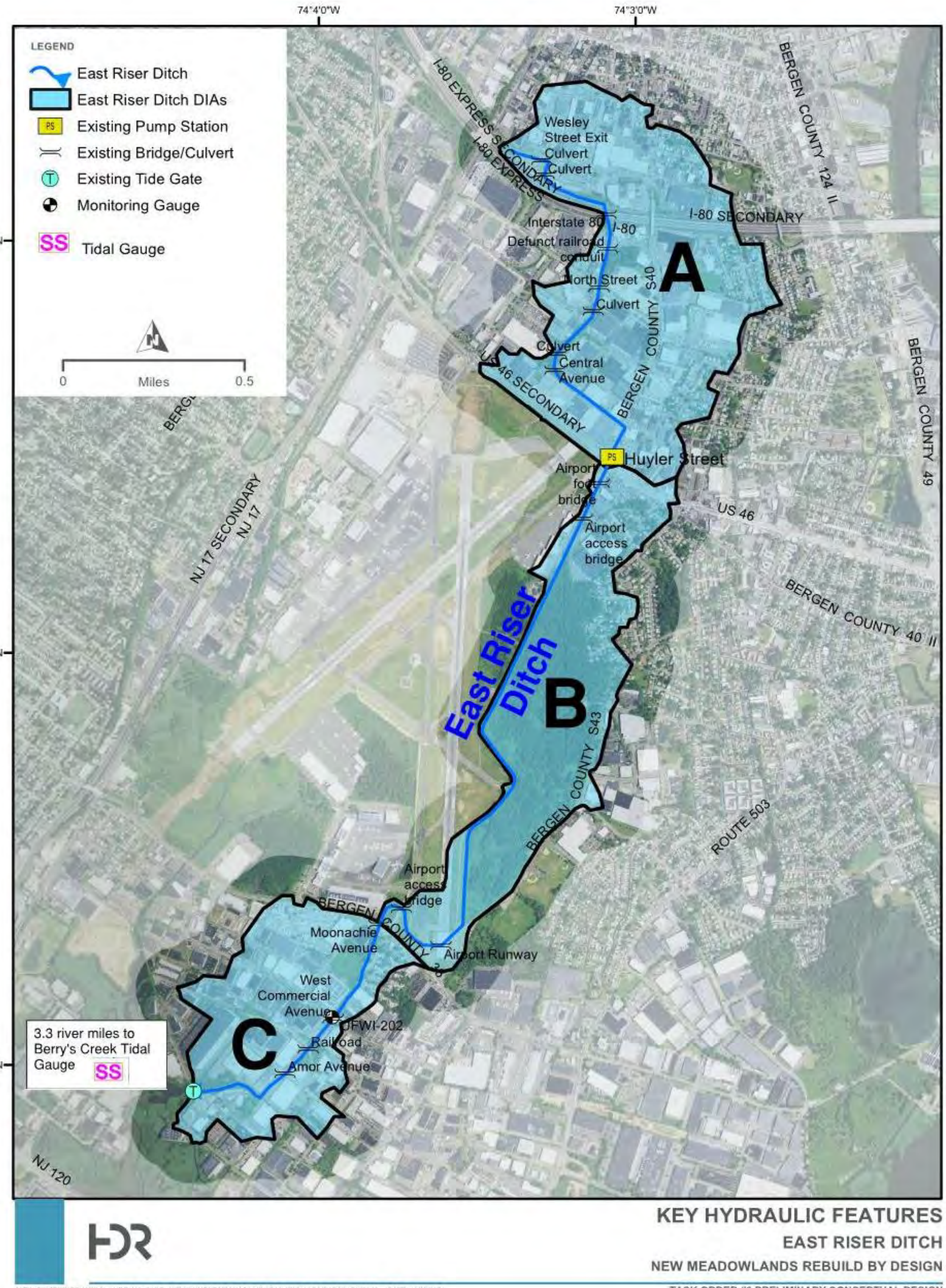

Figure B6-13: ERD Key Hydraulic Features

Through the Teterboro Airport reach, the ditch passes through two airport access crossings (a foot bridge and then a roadway access). The ditch continues without interruption for approximately 1.3 miles, with the airport levees to the west and a wooded area bounded by Redneck Avenue to the east. Exiting the airport reach, the ditch passes through a large culvert underneath the south-north runway and then an airport access road bridge.

On the south end of the ditch, ERD passes under Moonachie Avenue, West Commercial Avenue, a railroad, and Amor Avenue. The USEPA briefly operated a stage and flow measurement gauge at West Commercial Avenue. The ERD ends at the East Riser tide gate, which is located under Starke Road.

3.1 Data Sources

3.1.1 Survey

The survey of ditch cross sections and structure information was performed by Matrix New World Engineering. The survey for ERD was conducted between October 18, 2016 and November 15, 2016. Field survey was taken at 68 cross sections and 10 structures. **Figure B6-14** shows the locations of the survey. Surveyed cross sections were used to describe channel conditions. No surveys were conducted between US Route 46 and up to and including Moonachie Avenue. Data from a separate hydraulic study by Jacobs Engineering, a portion of which was later revised by URS, was used in this area (Jacobs Civil Consultants 2009, URS 2013).

3.1.2 LiDAR

A LiDAR DEM with 2-foot pixel resolution was supplied by the New Jersey Sports and Exposition Authority (NJSEA). Quantum Spatial collected the near-infrared data in April 2014 and the LiDAR was derived by using TIN processing. The horizontal datum is NAD83 (2011) and the data was projected to the New Jersey State Plane coordinate system. Elevation units are feet in the NAVD 88. The LiDAR terrain data was used for the overbank cross sections with survey data used to define the channel areas.

3.1.3 Design Flows

The design flows for the ERD were sourced from Phase 1 of the HEC-HMS model (“HEC-HMS Model”). The Proposed Project specifications used 48-hour durations for the 2-year (50 percent annual exceedance probability) to 100-year (1 percent annual exceedance probability) reoccurrence intervals.

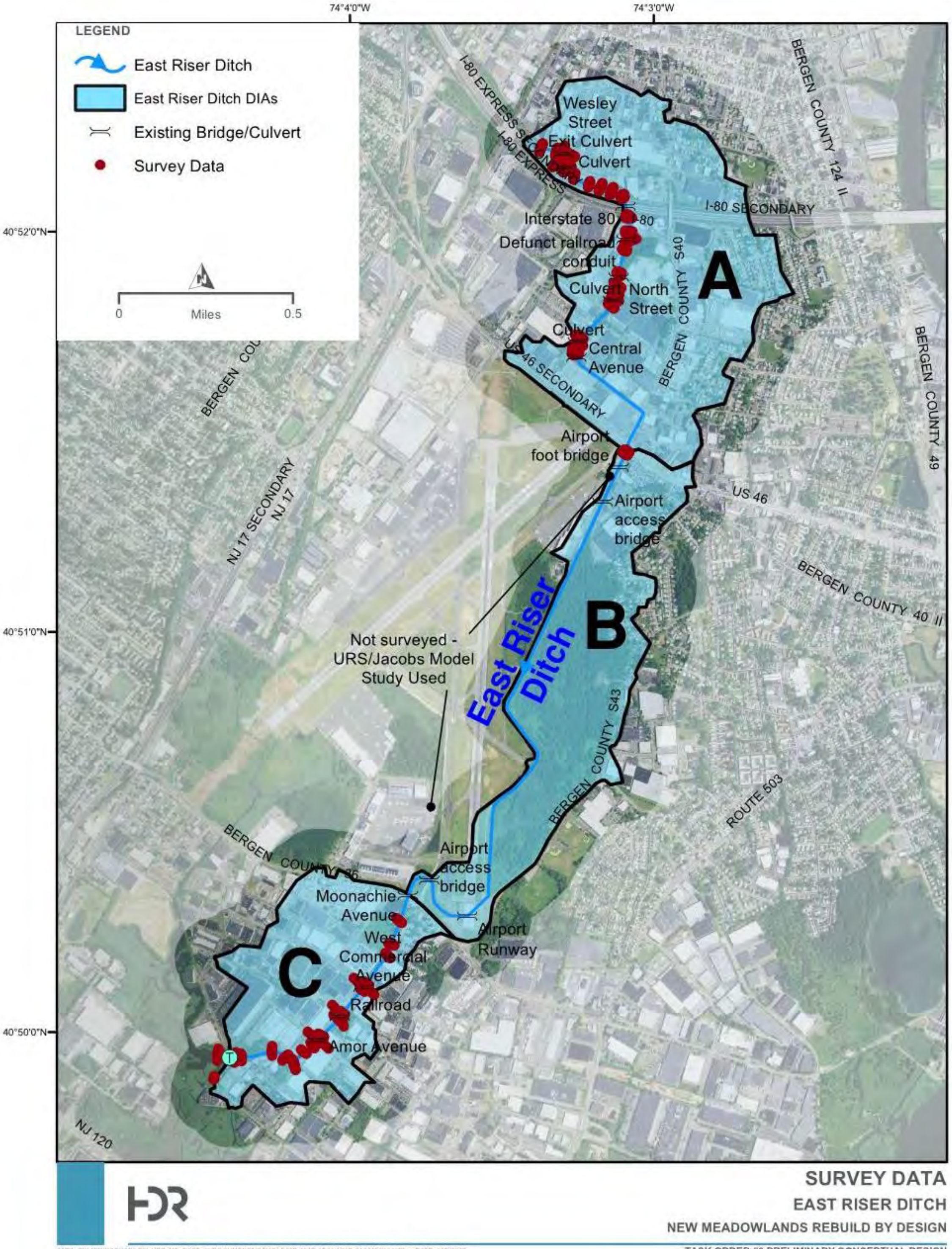

Figure B6-14: ERD Surveyed Cross Section Locations

3.1.4 Tidal Conditions

Downstream tidal conditions were obtained through a study by the Woods Hole Group (Woods Hole Group 2007). The nearest tidal gauge to the ERD is the Berry's Creek Canal, located approximately 3.3 river miles downstream of the East Riser tide gate. The MLW is -2.48 feet NAVD 88 (1983 to 2001 epoch) and the MHW is 3.03 feet NAVD 88.

3.1.5 Tide Gates and Pump Stations

The East Riser tide gate was surveyed by AECOM (AECOM 2016). The survey noted four gates sized 8.7-feet by 6.0-feet. The inverts of these gates were noted by HDR with top of deck elevations from LiDAR of Starke Road (HDR Engineering 2016). The maximum pumping capacity of the Huyler Street pump station was noted as 89.1 cfs (HDR Engineering 2016).

3.1.6 Other Studies and Models

A HEC-RAS model from Jacobs Engineering and URS was appended to the Teterboro Airport reach between US Route 46 and Moonachie Avenue (Jacobs Civil Consultants 2009, URS 2013). No survey data was available to verify the supplemental HEC-RAS models from others.

3.2 Model Limitations

3.2.1 Survey

Submerged structure geometry, type, and, in some cases, locations were assumed as the surveyor was not scoped to perform underwater survey. The surveyor noted (Personal communication between Matrix New World surveyors and Pratik Desai (HDR Engineering, Inc.) 2016):

> *"We were to survey these areas the best we could using our methods but with limited sight line and several feet of silt and debris that we can't see under the water line we are limited in giving accurate numbers."*

The modeling assumptions were made for submerged or partially submerged structures, see **Table B6-11** for survey assumptions.

Table B6-11: Survey Assumptions for Structures along ERD

Structure	Notes/Limitations
Adjacent to Wesley Street (RS 19755)	Structure inlet inverts and geometry from survey point numbers 710 and 711. The surveyed point 584 is interpreted as a different culvert through Wesley Street. The structure outlets did not appear to be surveyed. The outlet inverts were assumed to be the same as the inlet inverts, as the cross section thalweg elevation on the outlet bounding section is higher than on the inlet bounding cross section.
Interstate 80 (RS 18271)	Survey field notes that structure was submerged and "pipe size and invert are beat [sic] guess fits." Structure outlet geometry was assumed to be the same as the inlet geometry. Structure outlet invert was calculated from the surveyed top of culvert (survey point 821) minus 4 feet rise.
Crossing (RS 17756)	Structure was not surveyed; it was noted as an underground pipe in survey field notes. structure inverts and geometry of two barrels of 36" pipe from Boswell Engineering as-builts (stations 2+70 to 5+20)
North Street (RS 17176)	Survey field notes that inlet pipes were submerged and could not be located. Due to survey uncertainty, Geometry and inverts from Boswell plans ERD (stations 9+20 to 12+00).
Foot Bridge about 300 feet downstream from North Street (RS 16815)	Used Boswell plans ERD (stations 12+90 to 13+40)
Foot bridge about 300 feet upstream of inlet to Central Avenue (RS 15975)	Survey field notes that the structure was submerged and that the inverts and pipe sizes are "best guess fit." Used Boswell plans ERD (stations 21+00 to 22+10)

Structure	Notes/Limitations
Central Avenue (RS 15725)	Survey notes inlets as two barrels of 39"x54" reinforced concrete pipe (RCP). Outlet geometry and inverts from survey point numbers 1008 to 1017. Survey field notes outlet as 9'x42" RCP box culvert. Due to survey uncertainty used Boswell plans ERD (station 28+30) for inlet.
Teterboro Airport Runway (RS 5756)	Structure was not surveyed. Structure geometry was obtained from URS "Figure 4 East Riser Ditch Lateral Weir and Cross-Section Locations" Teterboro Airport Airport Traffic Control Tower Facility Flood Hazard Area Individual Permit Engineering Report East Riser Ditch. December 20, 2013.
Moonachie Avenue (RS 4307)	Structure was not surveyed. Jacobs model data was from "Bridge 147 Moonachie Ave Culvert - 12'-8" x 8'-1" CMPA Bit. Coated Assume culvert cleaned and no debris."
West Commercial Avenue (RS 2750)	Structure geometry from survey field note sketch. Inlet and outlet inverts from survey points 475 and 506.
Amor Avenue (RS 1679)	Structure geometries from survey field notes. Inverts using top of pipe elevations (survey points 367 and 372 and points 312, 313) minus estimated pipe rise.

3.3 Existing Conditions

An existing conditions HEC-RAS model was created for ERD running from north of Interstate 80 to just downstream of the East Riser tide gate. This model acts as a baseline conditions for which Proposed Project scenarios were compared. The MLW condition was used for the boundary condition.

3.3.1 Development of Existing Condition Cross Sections

For areas outside of the Teterboro Airport reach, cross sections were co-located with field surveyed cross sections described in **Section 3.1.1**. For the airport reach cross section, locations were co-located with Jacobs or URS cross sections (Jacobs Civil Consultants 2009, URS 2013). For all cross sections, overbank areas were extracted from LiDAR data. The channel area was adjusted based on survey or the Jacobs or URS models (Jacobs Civil Consultants 2009, URS 2013).

Figure B6-15, **Figure B6-16**, and **Figure B6-17** show the cross sections defined in the HEC-RAS model.

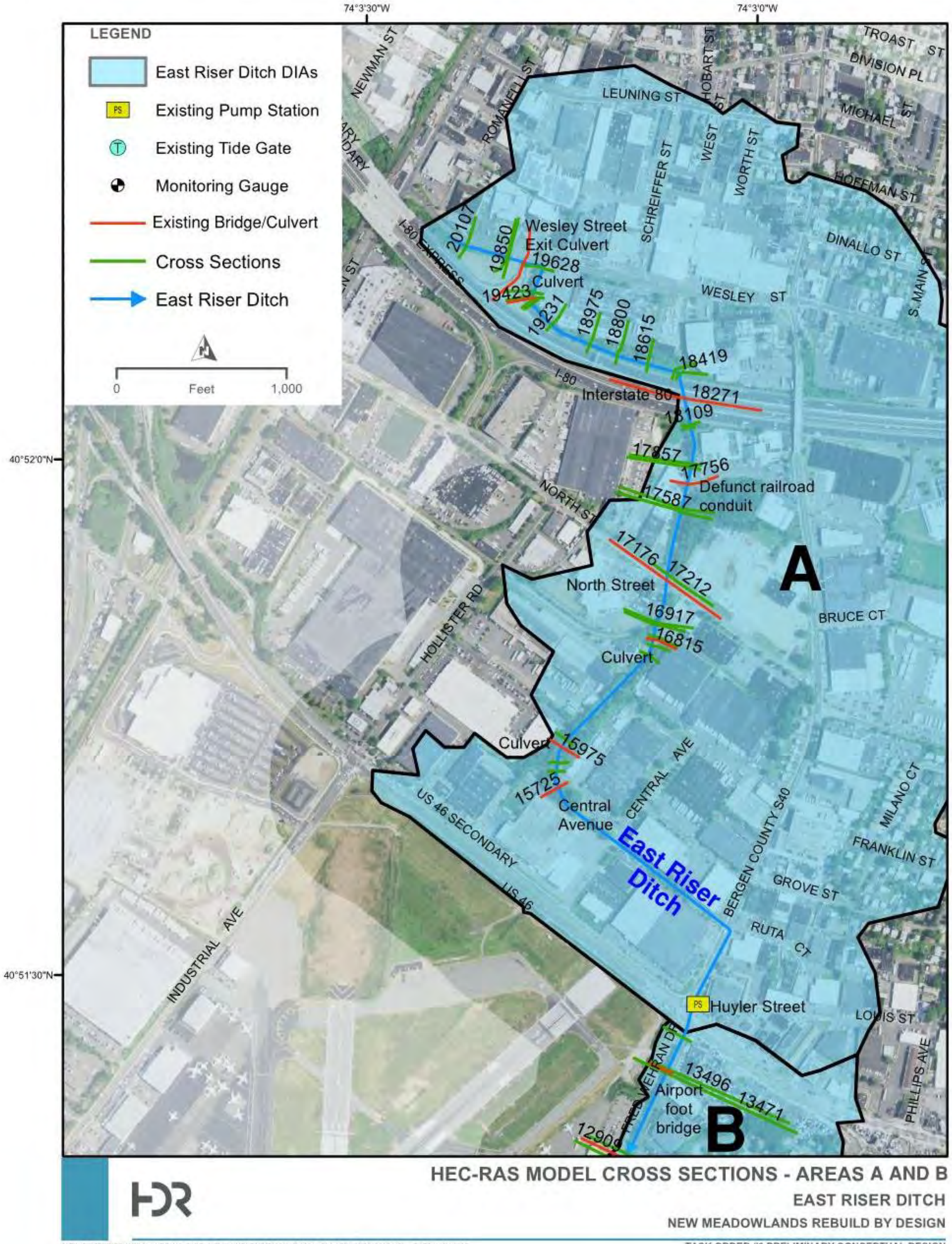

Figure B6-15: ERD HEC-RAS Model Cross Sections for DIAs ‘A’ and ‘B’

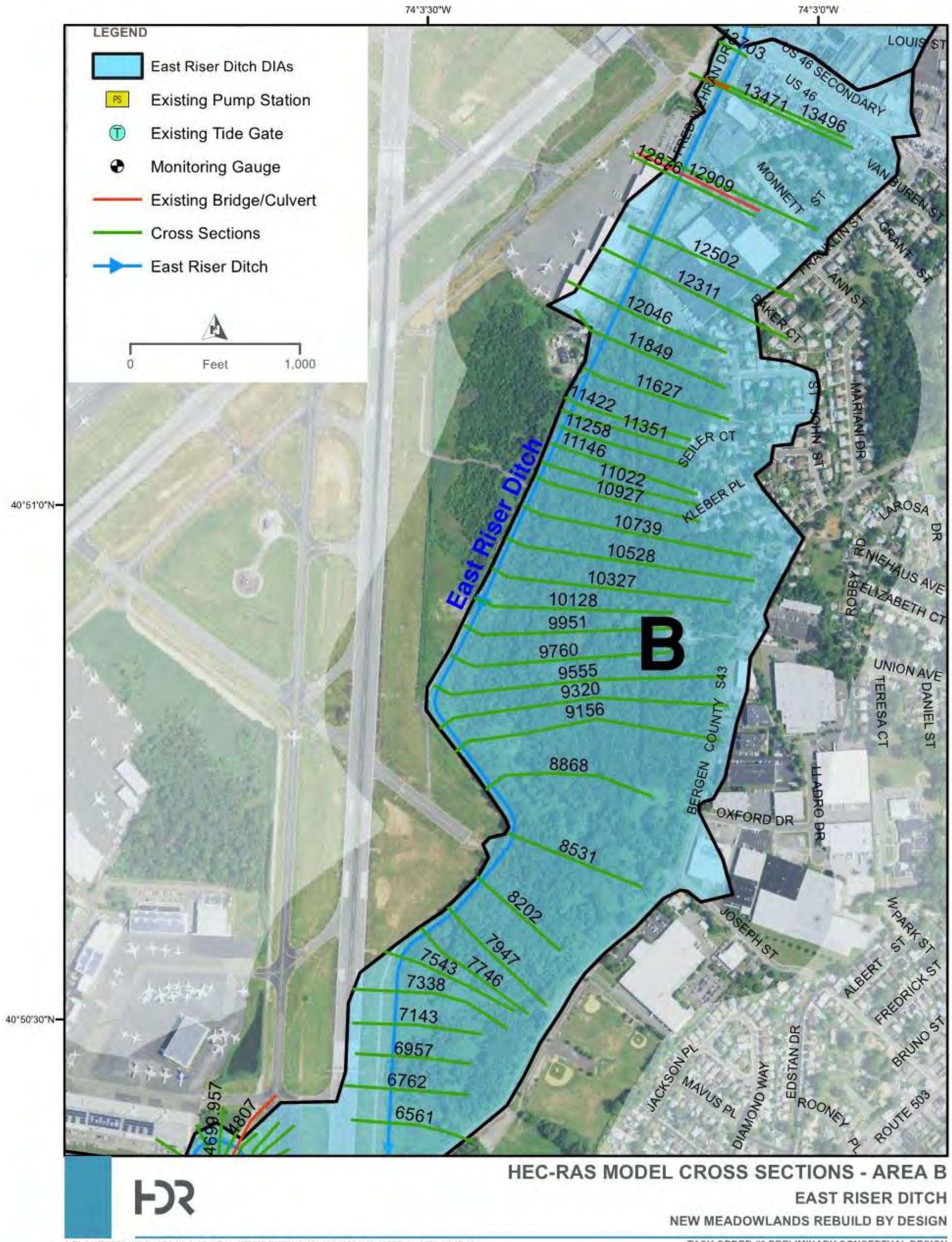

Figure B6-16: ERD HEC-RAS Model Cross Sections for DIA ‘B’

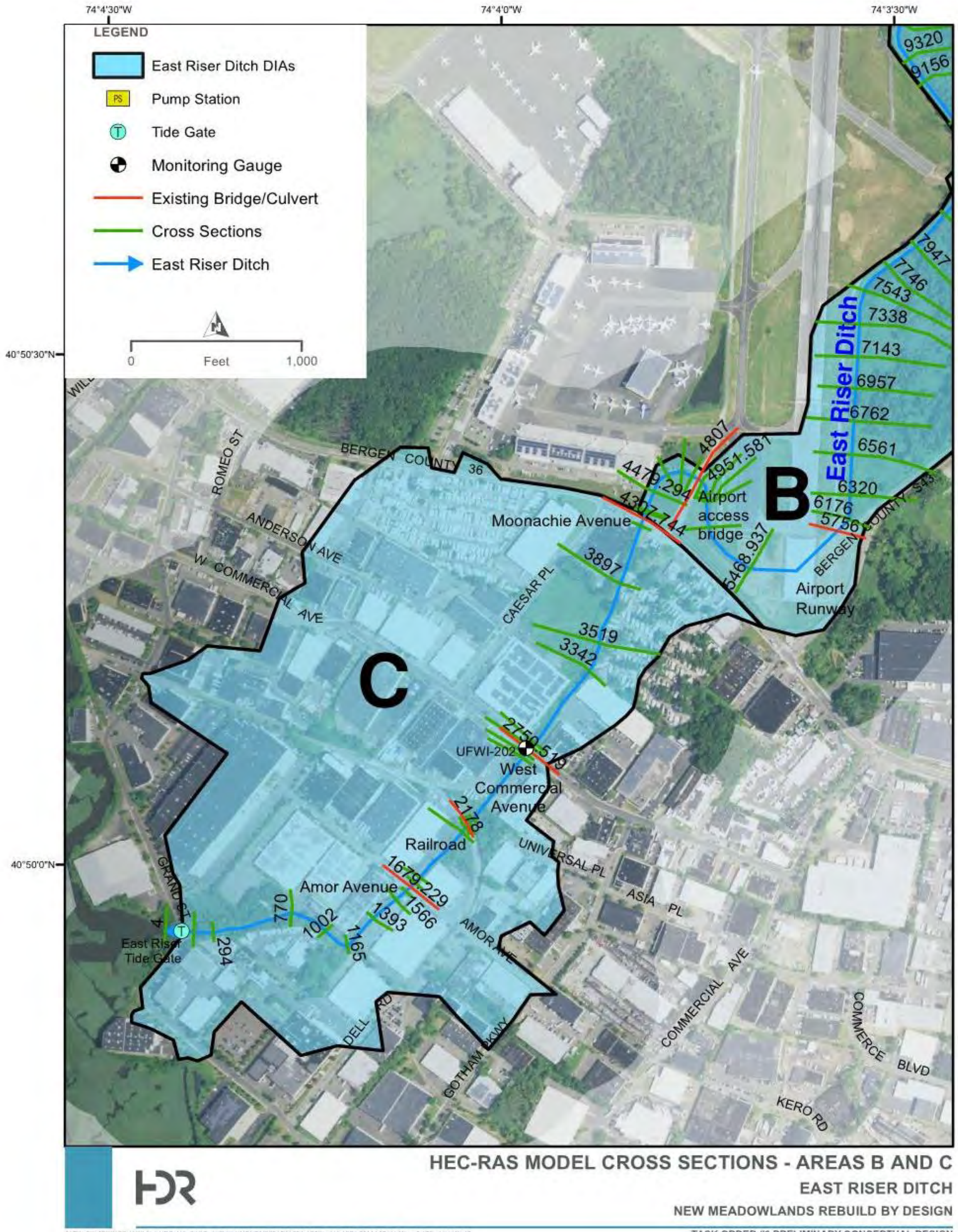

Figure B6-17: ERD HEC-RAS Model Cross Sections for DIAs ‘B’ and ‘C’

3.3.2 Steady Flow Data

3.3.2.1 Flow Breakout Locations

Flows from the ERD can potentially overtop the Teterboro Airport levee system or Redneck Avenue within DIA ‘B.’ The Jacobs-URS model assumed that flows overtopping these areas were lost from the ERD watershed (Jacobs Civil Consultants 2009, URS 2013). It is not clear that this occurs, as flow breakouts might re-enter the ERD lower in the system, such as upstream or downstream of Moonachie Avenue. For the existing and proposed conditions, it is assumed that no breakout flows are lost from the ERD.

3.3.2.2 Assignment of HEC-HMS Flows to RAS Cross Sections

Additional flow change locations were developed by HDR using a drainage area ratio applied to the flow reporting locations in the Phase 1 HEC-HMS model. See **Table B6-12** for additional details.

Table B6-12: Drainage Area Ratios for ERD

HEC-HMS Reach Flows	+	Drainage Area Portion…	…Of HEC-HMS Sub-basin	= flow at HEC-RAS Cross Section Station
---	+	55%	East Riser Ditch – North	20105
---	+	62%	East Riser Ditch – North	19626
---	+	100%	East Riser Ditch – North	18418
R_ER-Central	+	33%	East Riser Ditch - Main Street	17211
R_ER-Central	+	80%	East Riser Ditch - Main Street	13701
R_ER-South	+	24%	East Riser Ditch	11020
R_ER-South	+	25%	East Riser Ditch	9153
R_ER-South	+	31%	East Riser Ditch	6174
R_ER-South	+	75%	East Riser Ditch	4344
R_ER-South	+	79%	East Riser Ditch	2772
R_ER-South	+	91%	East Riser Ditch	2195
R_ER-South	+	91%	East Riser Ditch	1716
R_ER-South	+	100%	East Riser Ditch	177

Note: At cross section station 13701, the remaining 20% assumed to be part of Huyler Street pump station. This contributes at a fixed rate from pumping from the non-contributing drainage area.

Not intended for construction.

The Phase 1 HEC-HMS model did not include ERD’s existing interior pump station or non-contributing areas that could be served with proposed pump stations. HDR revised the HEC-HMS model to include a preliminary version of these pump stations. The pump stations are:

- Existing Huyler Street pump station that provides flows from a non-contributing area into ERD;
- A proposed pump station in the upper watershed near Green Street that would service a non-contributing area with flows into ERD; and
- A proposed pump station that would transfer flows into the Main Street watershed.

The non-contributing drainage areas were estimated by subtracting the hydrologically filled elevation dataset from the original elevation dataset. The drainage area and storage volume of the resulting sinks were compiled. For Huyler Street, the non-contributing area was estimated at 0.07 square miles. The non-contributing area by Green Street within the ERD drainage area is 0.04 square miles. There was a second non-contributing area by Green Street and Lodi Street within the WRD drainage area of 0.16 square miles. It was assumed that only the area within the ERD drainage would be served by a future pump station.

The “East Riser Ditch – North” sub-basin and “East Riser Ditch - Main Street” sub-basin were subdivided into the non-contributing areas for the Green Street and Huyler Street pump areas, respectively. For Green Street, the non-contributing storage becomes contributing through a 55 feet wide roadway at an elevation of 5.78 feet NAVD 88. Huyler Street pumping was modeled at 89.1 cfs based on available survey data of the pump station (HDR Engineering 2016). The Green Street proposed pumping rate was assumed at 30 cfs, based on HEC-HMS simulated inflow rates for the 25-year storm event. The proposed Main Street pumping rate was the same as the proposed Green Street pumping rate, as it is assumed that the Main Street pumping is to mitigate the introduction of Green Street flows.

3.3.3 Ineffective Flow Areas

Ineffective widths were set based on the distance between the upstream or downstream cross section and the structure, and a ratio of 1:1 (channel length: ineffective width) upstream or 2:1 downstream. Ineffective elevations upstream of a structure were initially set 0.1 feet higher than the upstream deck low point. At the cross section downstream of a structure, ineffective flow heights were based on the average of the highest low chord and lowest roadway surface elevation. Through applying a number of discharges to the system, ineffective flow heights were iteratively adjusted to such that both upstream and downstream ineffective flow elevations were exceeded at a consistent discharge.

3.3.4 Culverts and Bridge Openings

Deck elevations were extracted from the LiDAR datasets and survey data. Bridge low chords, culvert sizes, pipe inverts, and upstream and downstream cross section were extracted from surveyed data. The field surveyors noted uncertainty in some structure measurements, as detailed in **Table B6-11**.

No survey data was available for the Teterboro Airport reach between model cross sections 13946 to 4260.561. These structures were described by the existing Jacobs or URS model, but were not field validated (Jacobs Civil Consultants 2009, URS 2013). A summary of the ERD structures and the source of data supplemented with LiDAR are included in **Table B6-13**.

Bridges and culverts were assumed perpendicular to the direction of the flow. The pressure and/or weir equations were used to model the structures. Contraction and expansion coefficients were set to 0.3 and 0.5, respectively, upstream and downstream of culverts, bridges, or tide gates. The remaining cross section contraction and expansion coefficients were set to 0.1 and 0.3, respectively.

Table B6-13: ERD – Existing Structures and Data Sources

Structure Location (RS)	Description	Source
19755	Wesley Street Exit Culvert	Survey
19423	Culvert	Survey
18271	Interstate 80	Survey
17756	Defunct Railroad Conduit	Boswell McClave Plans
17176	North Street	Boswell McClave Plans
16815	Culvert	Boswell McClave Plans
15975	Culvert	Boswell McClave Plans
15725	Central Avenue	Boswell McClave Plans
13483	Airport Foot Bridge	Jacobs/URS
12909	Airport Access Bridge	Jacobs/URS
5756	Airport Runway	Jacobs/URS
4807.49	Airport Access Bridge	Jacobs/URS
4307	Moonachie Avenue	Jacobs/URS
2750	West Commercial Ave	Survey
2178	Railroad	Survey
1679	Amor Ave	Survey

Not intended for construction.

3.3.5 Pump Station

Huyler Street pump station is located at RS 13703. The effects of the pump station were modeled as changes in flow at this location. The HEC-HMS model provided the pumping station flows, based on a drainage area ratio.

3.3.6 Tide Gate Configuration

East Riser tide gate structures were modeled as an inline structure with gates. The gates were assumed fully open for the MLW condition. A fixed WSEL equal to the MLW condition was used as the boundary condition downstream of the tide gate.

3.3.7 Manning's Roughness Coefficients

Manning's roughness coefficients were allocated based on year 2015 aerial imagery, as noted in **Table B6-14**.

Table B6-14: ERD - Manning's Roughness Coefficients

Manning's n value	Description
0.013 to 0.03	Developed: buildings/pavement
0.035	Straight ditch segments
0.04	Sinuous ditch segments; grass overbanks
0.05 to 0.055	Mix of trees and grass
0.06 to 0.065	Wetlands
0.08	Dense trees, isolated areas
0.105 to 0.11	Dense trees, large areas

3.4 Proposed Conditions

3.4.1 Description of Proposed Project Scenarios

Five Proposed Project scenarios were simulated, as detailed in **Table B6-15**. Proposed conditions included additional pump stations, channel excavation, and increases in culvert sizes. In Scenario 4, a new proposed pump station is added to the upper area in DIA 'A.' This proposed pump station is intended to alleviate localized flooding in the Green Street area; this mitigation tactic in turn increases flows in the ERD. In Scenario 5, the effects of the new pumping station at Green Street are partially mitigated by installing a pump station near the existing Huyler Street pump station. This new pump station removes flows from ERD and pipes this outside of the ERD watershed to the Main Street area. This transferred water is simulated in an InfoWorks model (not described in this report).

The MLW condition was used for the boundary condition.

Table B6-15: ERD Proposed Project Scenarios

Scenario Number	Plan Name	Description
1	PRP_MLW_S1ABCALLYN0HS	Proposed pump station at tide gate
2	PRP_MLW_S2ABCALL	DIA A: Excavate channel, increase culvert size DIA B: Excavate channel DIA C: Excavate channel
3	PRP_MLW_S3ABCALLYN0HS	DIA A: Excavate channel, increase culvert size DIA B: Excavate channel DIA C: Excavate channel + pump station at tide gate
4	PRP_MLW_S4ABCALLYN0HS	DIA A: Excavate channel, increase culvert size, pump station at Green Street DIA B: Excavate channel DIA C: Excavate channel, pump station at tide gate
5	PRP_MLW_S5ABCALLYN0HS	DIA A: Excavate channel, increase culvert size, pump station at Green Street, diversion to Main Street DIA B: Excavate channel DIA C: Excavate channel, pump station at tide gate

Not intended for construction.

3.4.2 Proposed Conditions: Channel Modifications

Channels were modified in DIAs 'A,' B,' and 'C' for Scenarios 2 through 5. No channel modifications were proposed for Scenario 1. Generally, existing cross sections were excavated to increase conveyance capacity. The proposed channel cross sections were defined by a trapezoidal shape, maximum channel width, channel side slope, and channel invert. The top channel width was based on an assumed right-of-way that started 15 feet from the edge of existing buildings and parking lots. The side slopes of the proposed ditch were assumed at a Horizontal to Vertical slope (H:V) of 2H:1V. Some cross sections in DIAs 'A' and 'B' could be expanded further, as top widths were assumed to increase by a maximum of 150 percent with a 3H:1V side slope. **Table B6-16** shows the proposed channel cross section for each DIA.

The channel inverts were lowered in areas to promote a positive slope. Scour holes along a small reach near North Avenue was filled to match the culvert invert elevations. **Figure B6-18** shows the change in proposed channel inverts.

Table B6-16: East Riser Ditch Proposed Channel Modifications

Drainage Improvement Area	Proposed Channel Side Slopes
A	2H:1V and 3H:1V
B	2H:1V
C	2H:1V and 3H:1V

Not intended for construction.

3.4.3 Proposed Conditions: Culvert/Bridge Opening Modifications

In ERD, Boswell McClave Engineering provided proposed culverts within DIA ‘A’ ("Boswell McClave project area") downstream of the Interstate 80 bridge and upstream of the Central Avenue crossing (Boswell McClave Engineering 2012). These proposed culverts were be implemented in this area for Scenarios 2, 3, 4, and 5. Some of the culverts in the Boswell McClave project area are noted as “remain-in-place.” It is assumed that, for purposes of this channel capacity feasibility assessment, the remain-in-place culverts will be replaced with larger capacity culverts. Additionally, it is assumed that the Interstate 80 bridge culvert can also be replaced.

Proposed culvert replacements in other areas of ERD used a concrete box culvert configuration. The maximum size of the replacement culvert was limited, by providing one foot of fill at the sides of the culvert and 18 inches of fill over the top of the culvert. See **Table B6-17** for details on culvert and bridge modifications. Ineffective flow area locations and elevations were adjusted to represent the modified openings.

3.4.4 Proposed Conditions: Pump Stations

The proposed pump station at the ERD tide gate was modeled as a change in flow. The maximum pumping size is assumed to be based on the existing maximum pump station capacity in the WRD area. The existing Teterboro Airport pump station (Vincent Place) has a capacity of 175 cfs (operationally limited to 112 cfs) (Jacobs Civil Consultants 2013) and has a drainage area of 255 acres[2]. Based on the 723 acre drainage area of the ERD, a proportionally assumed proposed maximum pumping capacity is 500 cfs.

[2] Drainage area from the project Phase 1 HEC-HMS model (dated October 31, 2016) for DA-WestRD-PS sub-basin.

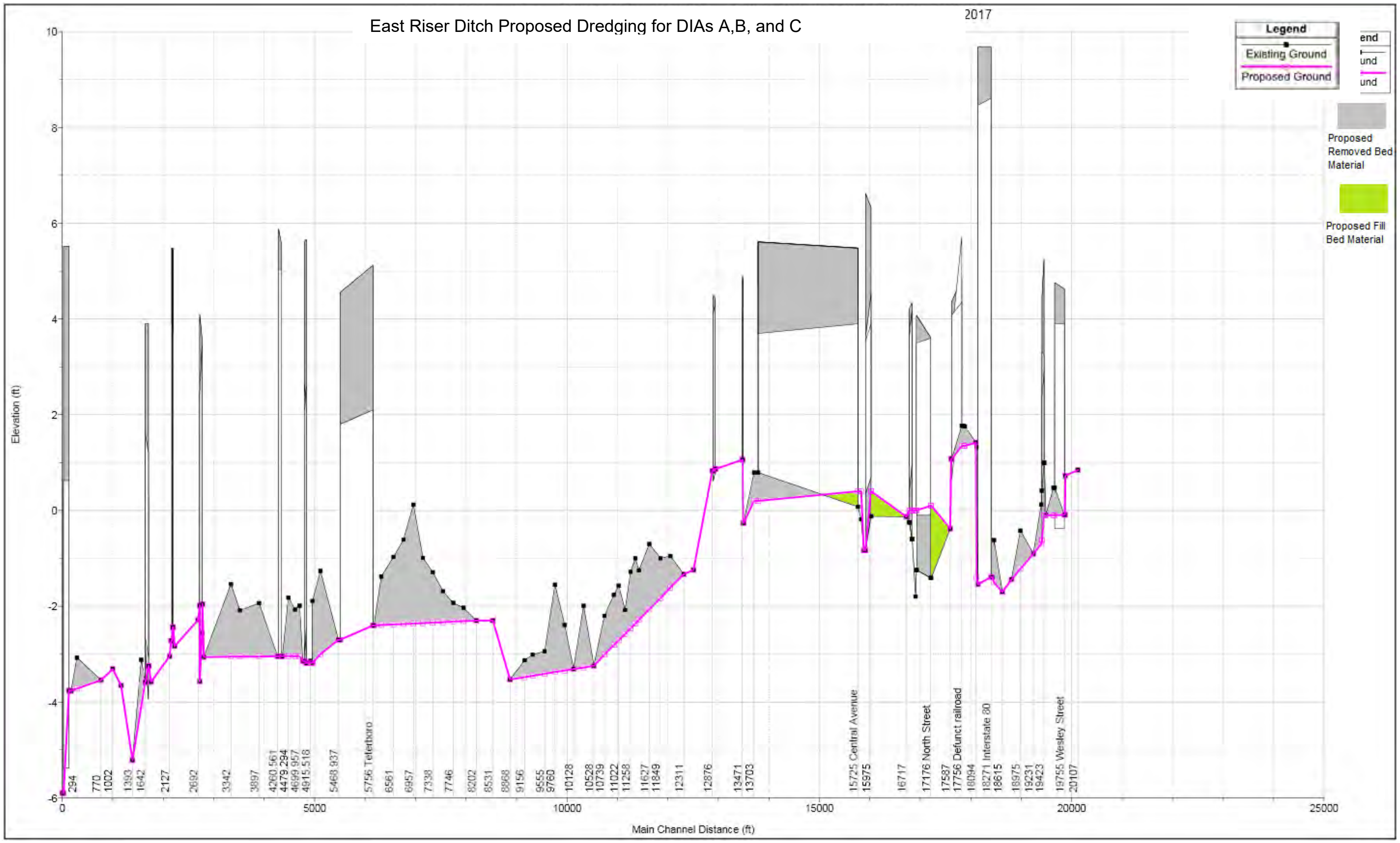

Figure B6-18: ERD Channel Profile

Not intended for construction.

Table B6-17: ERD - Proposed Culvert and Bridge Modifications

Location	Existing Configuration	Proposed Configuration
RS 19755 Wesley Street	36" RCP	10' x 4' box culvert
RS 19423 Culvert	3.33' RCP	3' x 3' box culvert
RS 18271 Interstate 80	6' x 4' ellipse RCP	10' x 10' box culvert
RS 17756 Defunct Railroad	2 barrels of 36" RCP	5' x 3' box culvert
RS 17176 North Street	2 barrels of 5' x 3.2' ellipse RCP	8' x 3.5' box culvert
RS 16815 Culvert	2 barrels of 5' x 3.2' ellipse RCP	2 barrels of 5' x 3.2' ellipse RCP
RS 15975 Culvert	2 barrels of 5' x 3.2' ellipse RCP	2 barrels of 5' x 3.2' ellipse RCP
RS 15725 Central Avenue	2 barrels of 48" RCP on inlet 9' x 3.9' box culvert on outlet	10' x 3.5' box culvert on inlet 10' x 3.5' box culvert on outlet
RS 13485 Culvert	10' x 4' box culvert	10' x 4' box culvert
RS 12909 Culvert	14' x 3.5' box culvert	14' x 3.5' box culvert
RS 5756 Teterboro Runway Culvert	12' x 4.5' box culvert	12' x 4.5' box culvert
RS 4807.490 Culvert	2 barrels of 60" RCP	2 barrels of 60" RCP
RS 4307 Moonachie Avenue	12.7' x 8.08' arch CMP	12.7' x 8.08' arch CMP
RS 2750 West Commercial Avenue	12' x 6' arch CMP	12' x 6' arch CMP
RS 2178 Railroad Bridge	18.8' x 6.2' bridge	18.8' x 6.2' bridge
RS 1679 Amor Avenue	2 barrels of 7.75' x 4.25' ellipse RCP	2 barrels of 7.75' x 4.25' ellipse RCP

Not intended for construction.

3.5 Model Results

3.5.1 Validation or Comparisons with other Studies

A USEPA study for Berry's Creek provided monitored stage and flow data that was used to validate the HEC-RAS existing condition (USEPA 2016). The USEPA monitoring location in the ERD was UFWI-202, located downstream of West Commercial Avenue near HEC-RAS model cross section RS 2629.

Peak flows and stages were digitized from **Attachment D1, Figure 2** and **Attachment D3, Figure 2** of the USEPA report (USEPA 2016). The pairing of this data produced a rating curve that could be compared with the simulated model rating curve. There was variability in the rating curve data, presumably due to variations in the tidal boundary conditions. An average rating curve was fitted with the monitoring data. The stage was converted to a WSEL based on the cross section RS 2629 invert elevation assumed as stage of 0.0 feet.

$$E = 4.1267949 * (1.4130651 - e^{-0.015724585*Q}) - 4.0$$

(Where E is the elevation (NAVD 88) in feet, and Q is the flow in cfs.)

The digitized monitored rating curve data and comparison to the HEC-RAS existing condition model is in **Figure B6-19**. The peak stage monitored by USEPA did not exceed the channel bank.

3.5.2 HEC-RAS Modeling Results

Results showing the change in WSEL for the ERD Proposed Project scenarios for the 2-, 5-, and 10-year return periods compared to existing conditions are displayed in **Table B6-18**. The lower return period storm events were considered more realistic targeted reductions for the ERD.

3.5.3 Proposed Project Scenarios Selected for Further Evaluation

The scenario which represented the maximum WSEL reduction and was selected for further evaluation during Level of Development 2 includes Scenario 5. Scenario 5 consists of six remove and replace culvert upgrades in DIA A, channel dredging in DIAs A, B, and C, two proposed pump stations at the existing ERD tide gate and Green Street flooding location, and a proposed force main in conjunction with the Green Street pump station to divert flow to Main Street. Profile plots for the 2, 5, and 10-year storm events comparing existing and Scenario 5 WSEL are shown in **Figure B6-20**.

3.5.4 Proposed Project Scenarios Eliminated from Further Evaluation

The remaining Scenarios 1 to 4 were considered less favorable and not evaluated further.

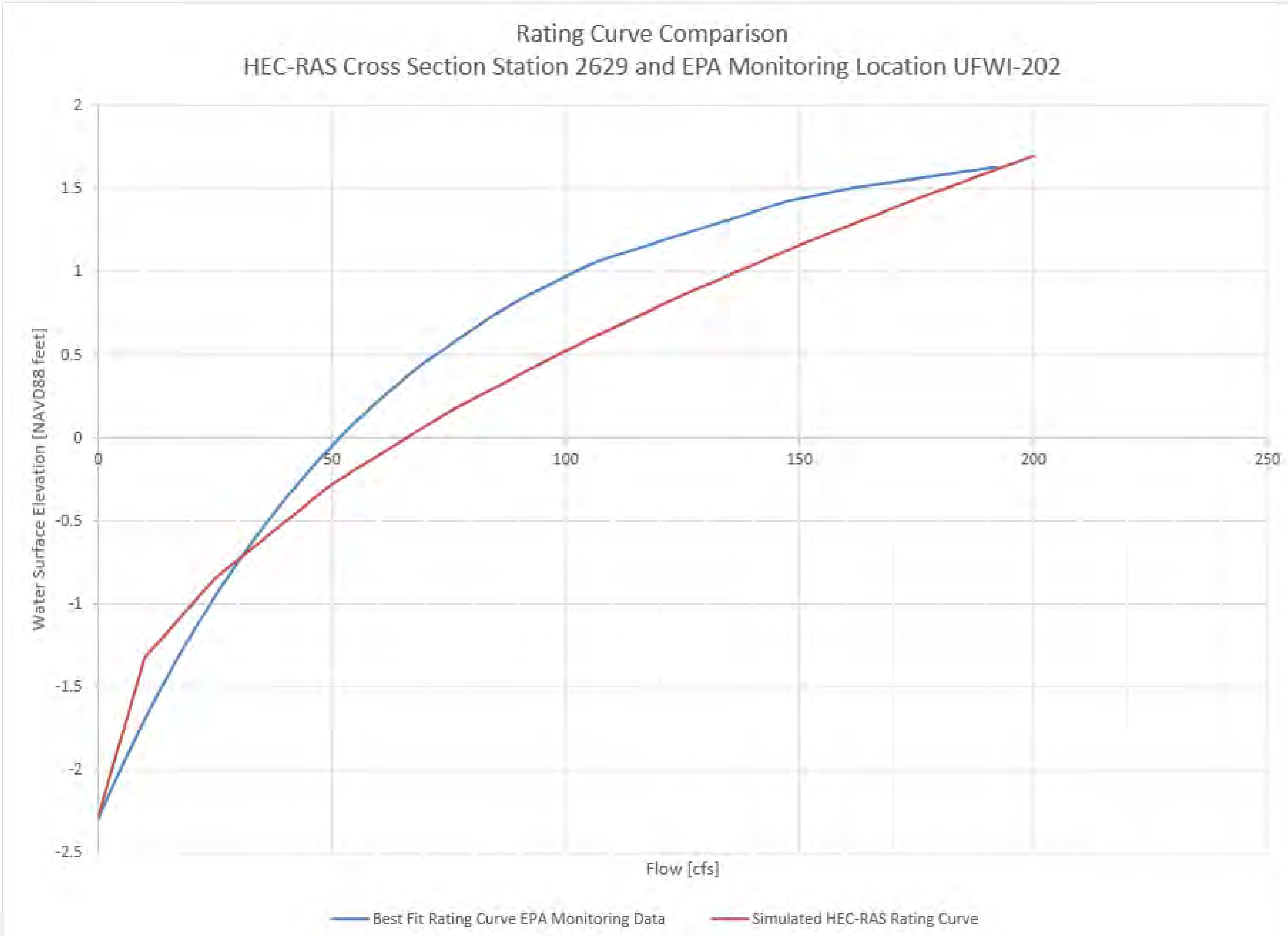

Figure B6-19: Validation of ERD at Moonachie Avenue from USEPA Based Rating Curve

Not intended for construction.

Table B6-18: Representative ERD Existing and Proposed WSELs

RS	Event	Existing WSE [feet NAVD 88]	Change from Existing WSEL [feet]				
			Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5
Upper portion of Drainage Improvement Area A							
20107	2-yr	8.88	0.00	-1.89	-1.90	-1.80	-2.01
	5-yr	10.05	0.00	-2.75	-2.75	-2.72	-2.76
	10-yr	10.23	0.01	-2.76	-2.77	-2.75	-2.78
18419	2-yr	8.88	-0.01	-1.91	-1.92	-1.82	-2.03
	5-yr	10.04	0.01	-2.76	-2.76	-2.73	-2.77
	10-yr	10.22	0.01	-2.78	-2.79	-2.76	-2.79
Interstate 80							
18109	2-yr	7.00	-0.01	-0.06	-0.06	0.03	-0.18
	5-yr	7.27	0.00	-0.03	-0.03	-0.01	-0.04
	10-yr	7.41	-0.01	-0.03	-0.04	-0.01	-0.04
16025	2-yr	6.97	-0.01	-0.06	-0.07	0.02	-0.19
	5-yr	7.21	0.00	-0.03	-0.03	0.00	-0.04
	10-yr	7.34	-0.01	-0.03	-0.03	-0.01	-0.04
15975	**Culvert**						
15876	2-yr	6.66	-0.02	-0.19	-0.21	-0.11	-0.77
	5-yr	7.01	0.00	-0.11	-0.11	-0.07	-0.24
	10-yr	7.22	-0.01	-0.11	-0.12	-0.10	-0.21
Lower Portion of DIA ‘A’/Upper Portion of DIA ‘B’							
13703	2-yr	6.54	-0.03	-0.18	-0.20	-0.12	-0.90
	5-yr	6.87	0.00	-0.08	-0.08	-0.05	-0.28
	10-yr	7.07	-0.01	-0.09	-0.09	-0.07	-0.23
4346.286	2-yr	5.81	-0.27	-0.58	-0.66	-0.49	-2.23
	5-yr	6.38	-0.01	-0.04	-0.06	-0.03	-0.42
	10-yr	6.64	0.00	-0.03	-0.03	-0.02	-0.23
4307	**Moonachie Avenue**						

RS	Event	Existing WSE [feet NAVD 88]	Change from Existing WSEL [feet]				
			Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5
4260.561	2-yr	5.34	-0.20	-0.50	-0.58	-0.42	-2.01
	5-yr	5.89	-0.02	-0.09	-0.11	-0.09	-0.45
	10-yr	6.19	-0.02	-0.08	-0.09	-0.07	-0.30
1717	2-yr	3.75	-0.25	-0.60	-0.71	-0.59	-1.93
	5-yr	4.79	-0.07	-0.20	-0.27	-0.23	-0.96
	10-yr	5.11	-0.07	-0.17	-0.21	-0.20	-0.44
1679	**Amor Avenue**						
1642	2-yr	1.63	-0.26	-0.76	-1.08	-1.05	-1.54
	5-yr	2.16	-0.31	-0.73	-1.05	-1.02	-1.38
	10-yr	2.5	-0.30	-0.74	-1.02	-1.00	-1.27
178	2-yr	-1.87	-0.60	0.00	-0.60	-0.60	-0.60
	5-yr	-1.58	-1.02	0.00	-1.02	-1.05	-0.90
	10-yr	-1.39	-1.06	0.00	-1.06	-1.04	-1.32

Not intended for construction.

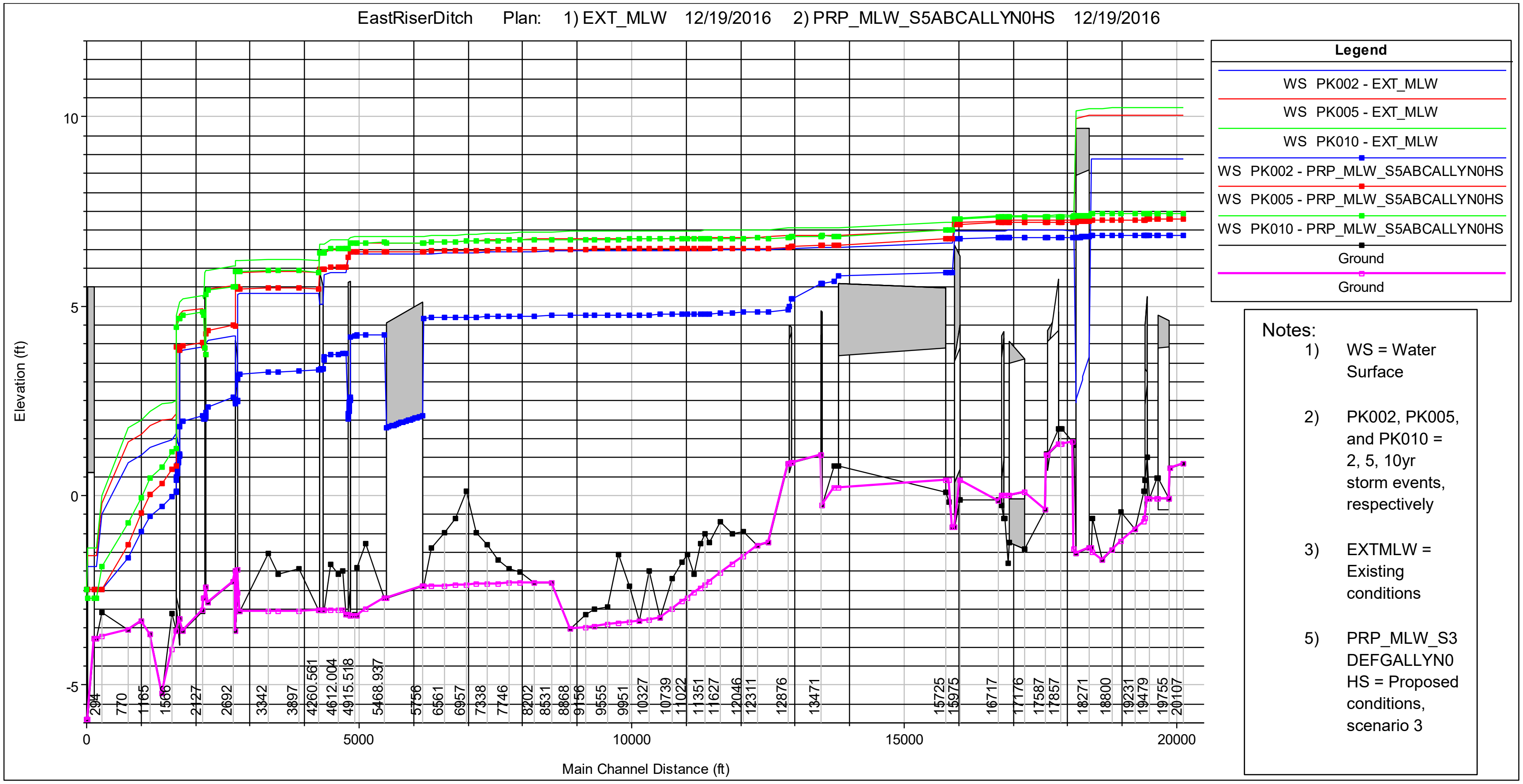

Figure B6-20: ERD WSEL Comparison between Existing and Selected Proposed Scenario 5 for the 2, 5, and 10-Year Storm Events

Not intended for construction.

4.0 Peach Island Creek

Peach Island Creek (PIC) is located in the Borough of Carlstadt and the Borough of Moonachie. The contributing drainage area is shown in **Figure B6-21**. The drainage area extends from Bergen County S43 / Redneck Avenue and Joseph Street in the North to the existing PIC tide gate and Route 503 in the south. The total drainage area is 0.89 square miles. **Table B6-19** provides a summary of the single DIA along PIC.

Table B6-19: Summary Statistics of Peach Island Drainage Improvement Areas

Drainage Improvement Area	Location	Drainage Area [square miles]
K	From Bergen County S43 / Redneck Avenue and Joseph Street in the North to the PIC tide gate and Route 503 in the south	0.89
Total Drainage Area		**0.89**

The modeling effort evaluated flood management benefits within a reach in the southern portion of the Borough of Carlstadt. **Figure B6-22** shows the key hydraulic features of the portion of PIC under study. The reach evaluated extended from 0.4 miles upstream of the Gotham Avenue Bridge to the PIC tide gate. The creek originates in a wetland west of Route 503, discharging into Berry's Creek at the downstream end. The Gotham Avenue Bridge and PIC tide gate are the only structures included in the model. There is a third structure located downstream of the tide gate, but this was considered outside of the Project Area, which terminated at the tide gate. It was assumed that this structure is not a controlling factor on the downstream boundary condition and the tide gate.

A proposed pumping scenario was evaluated to route water upstream of the tide gate to downstream to reduce flood prone areas documented along PIC. The drainage network along Gotham Parkway was modeled using the program InfoWorks. The Infoworks model used the WSEL from the PIC model upstream of the tide gate as the downstream boundary condition. Pumping was assumed to be a potential improvement that minimally disturbs channel bed material and the existing contamination in PIC and Berry's Creek.

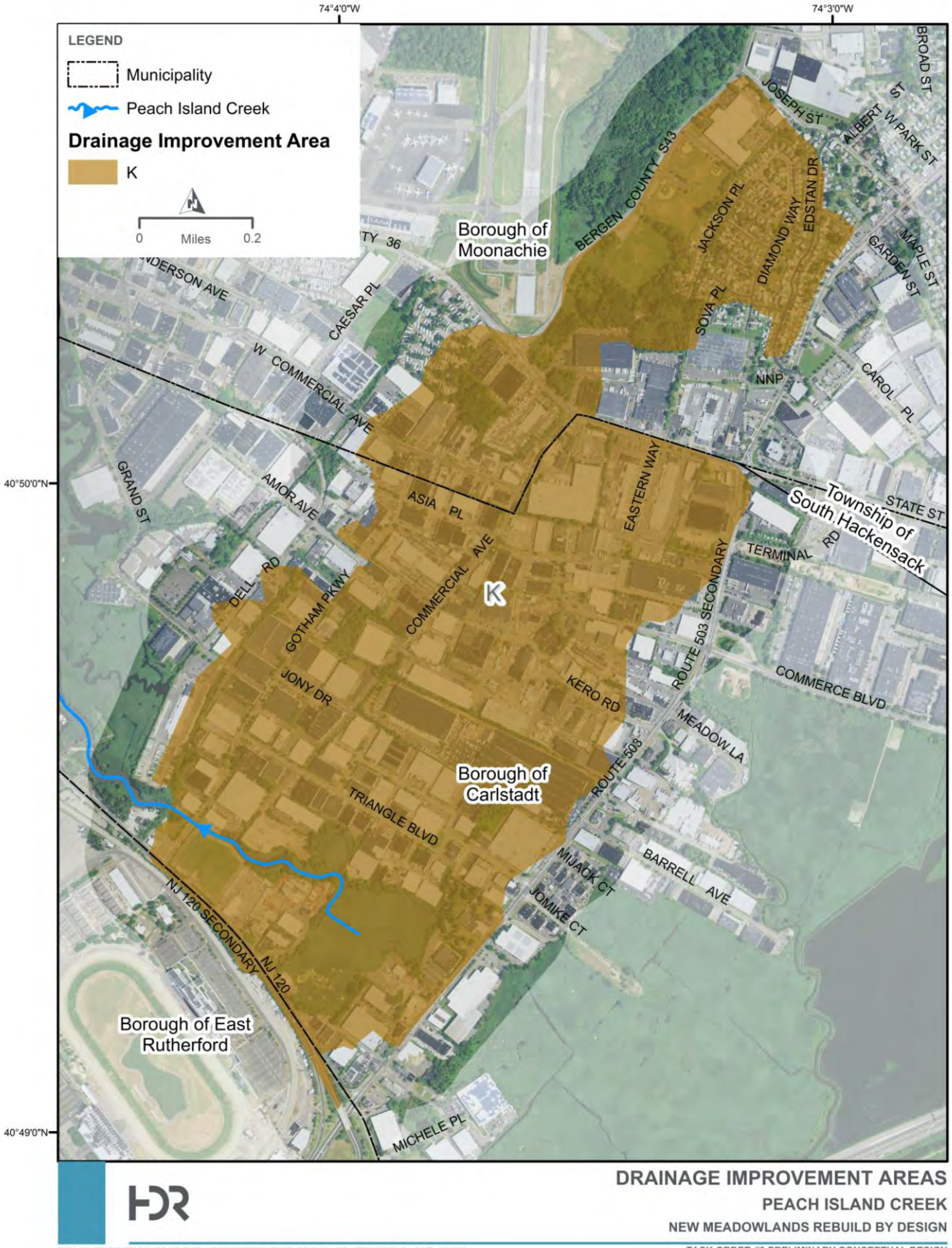

Figure B6-21: PIC Drainage Improvement Area

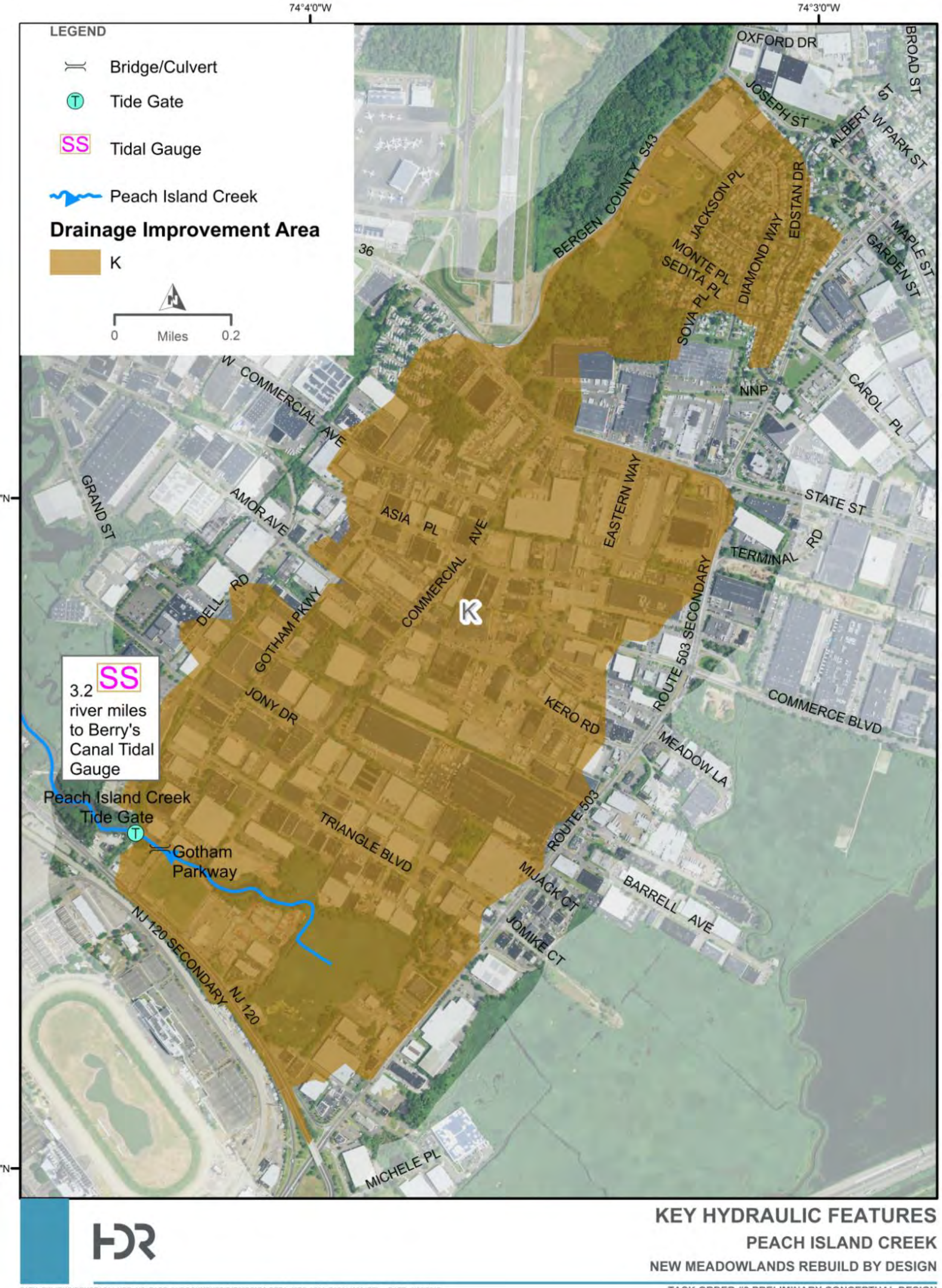

Figure B6-22: PIC Key Hydraulic Features

4.1 Data Sources

4.1.1 Survey

The survey of ditch cross sections and structure information was performed by AECOM survey personnel between September and October 2016 (AECOM 2016). Survey of the PIC tide gate was performed by Robinson Aerial Survey, Inc. Field surveyed sections were taken at 18 cross sections and 1 structure (see **Figure B6-23** for locations). Surveyed cross sections were used to describe channel conditions. Generally, the stream sections called out were surveyed, with exception of the sections immediately upstream and downstream of the tide gate.

The size of the gate flow through structure is unclear from survey, and AECOM team members reported 3-feet diameter from hand measurements in the field. Missing or incomplete surveyed features include: tide gate invert, shape, and dimensions of downstream flow through structure and invert & dimensions of upstream flow through structure and bottom of tide gate elevation.

4.1.2 LiDAR

A LiDAR DEM with 2-foot pixel resolution was supplied by the New Jersey Sports and Exposition Authority (NJSEA). Quantum Spatial collected the near-infrared data in April 2014 and the LiDAR was derived by using TIN processing. The horizontal datum is NAD83 (2011) and the data was projected to the New Jersey State Plane coordinate system. Elevation units are feet in the NAVD 88. The LiDAR terrain data was used for the overbank cross sections with survey data used to define the channel areas.

4.1.3 Design Flows

The design flows for the PIC HEC-RAS model were sourced from Phase 1 of the HEC-HMS model (“HEC-HMS Model”). The Proposed Project specifications used 48-hour durations for the 2-year (50 percent annual exceedance probability) to 100-year (1 percent annual exceedance probability) reoccurrence intervals. PIC was represented by a single sub-basin within the HEC-HMS model, therefore, only one flow value was assumed for the entire modeled reach.

4.1.4 Tidal Conditions

Downstream tidal conditions were obtained from a study by the Woods Hole Group (Woods Hole Group 2007). The nearest tidal gauge is the Berry’s Creek Canal, located approximately 3.2 river miles downstream of the PIC tide gate. The MLW is -2.48 feet NAVD 88 (1983 to 2001 epoch) while the MHW is 3.03 feet NAVD 88.

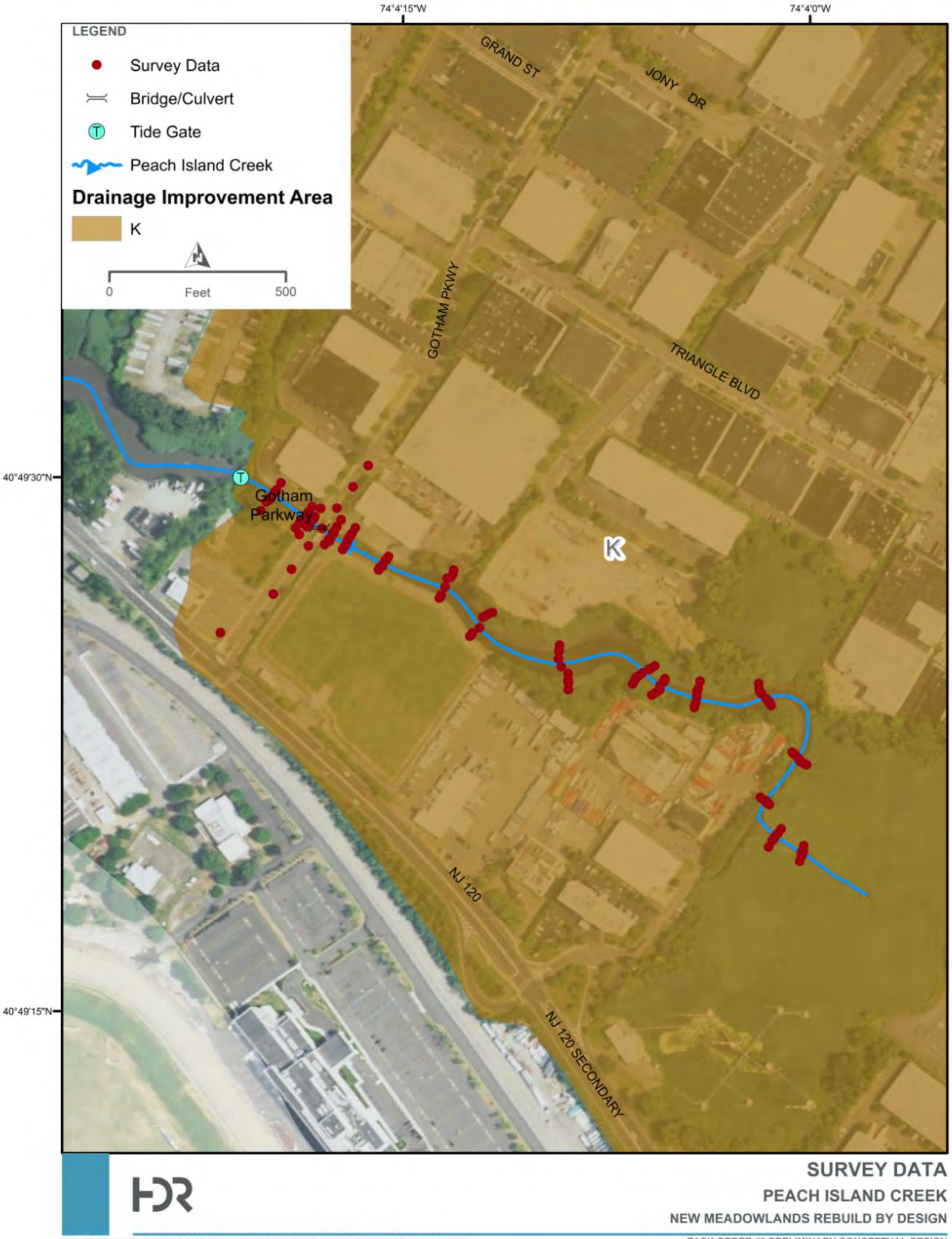

Figure B6-23: PIC Surveyed Cross Section Locations

4.1.5 Tide Gate

The PIC tide gate was described by AECOM as having four gate bays (AECOM 2016). One of these bays was blocked with a metal plate bolted to the structure. The remaining three bays had circular flap gates with a diameter of 36 inches.

4.2 Model Limitations

4.2.1 Contamination and Hazardous Materials

According to the Hackensack Meadowlands Floodplain Management Plan, there is a Superfund site located directly upstream of the tide gate, posing hazardous substance threats (NJSEA 2005). Due to the existing contamination throughout the reach and downstream into Berry's Creek, the proposed scenarios were limited to minimizing disturbance of contaminated material.

4.3 Existing Conditions

An existing conditions HEC-RAS steady-state model was created for PIC running 0.4 miles upstream of the Gotham Avenue Bridge to the PIC tide gate. This model acts as a baseline conditions for which proposed project scenarios were compared.

4.3.1 Development of Existing Condition Cross Sections

Cross sections were developed from the field surveyed cross sections described in **Section 4.1.1** Overbank areas were extracted from LiDAR data. The channel area was adjusted based on survey.

Figure B6-24 shows the cross sections defined in the HEC-RAS model. **Figure B6-25** provides the channel profile.

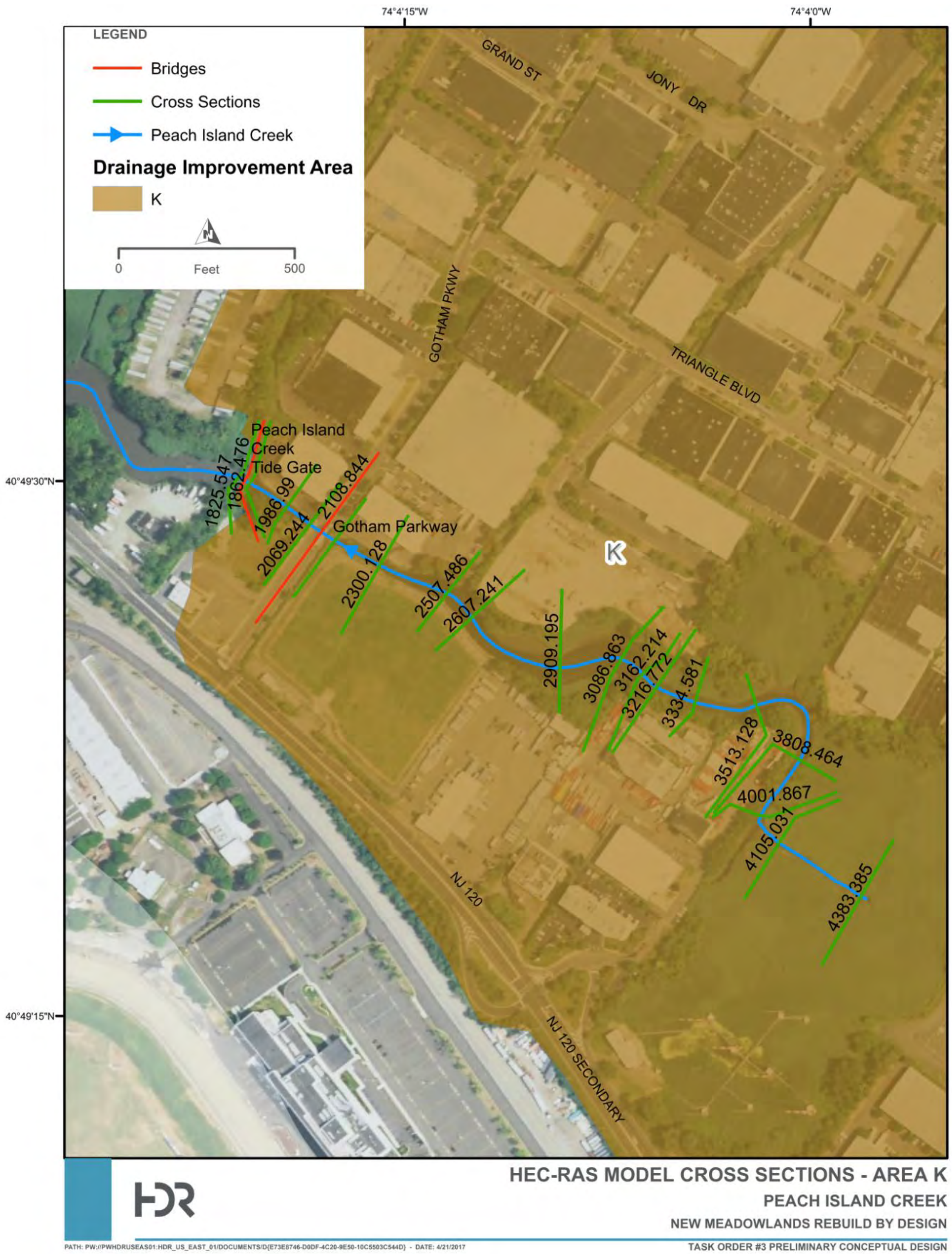

Figure B6-24: PIC HEC-RAS Model Cross Sections for DIA ‘K’

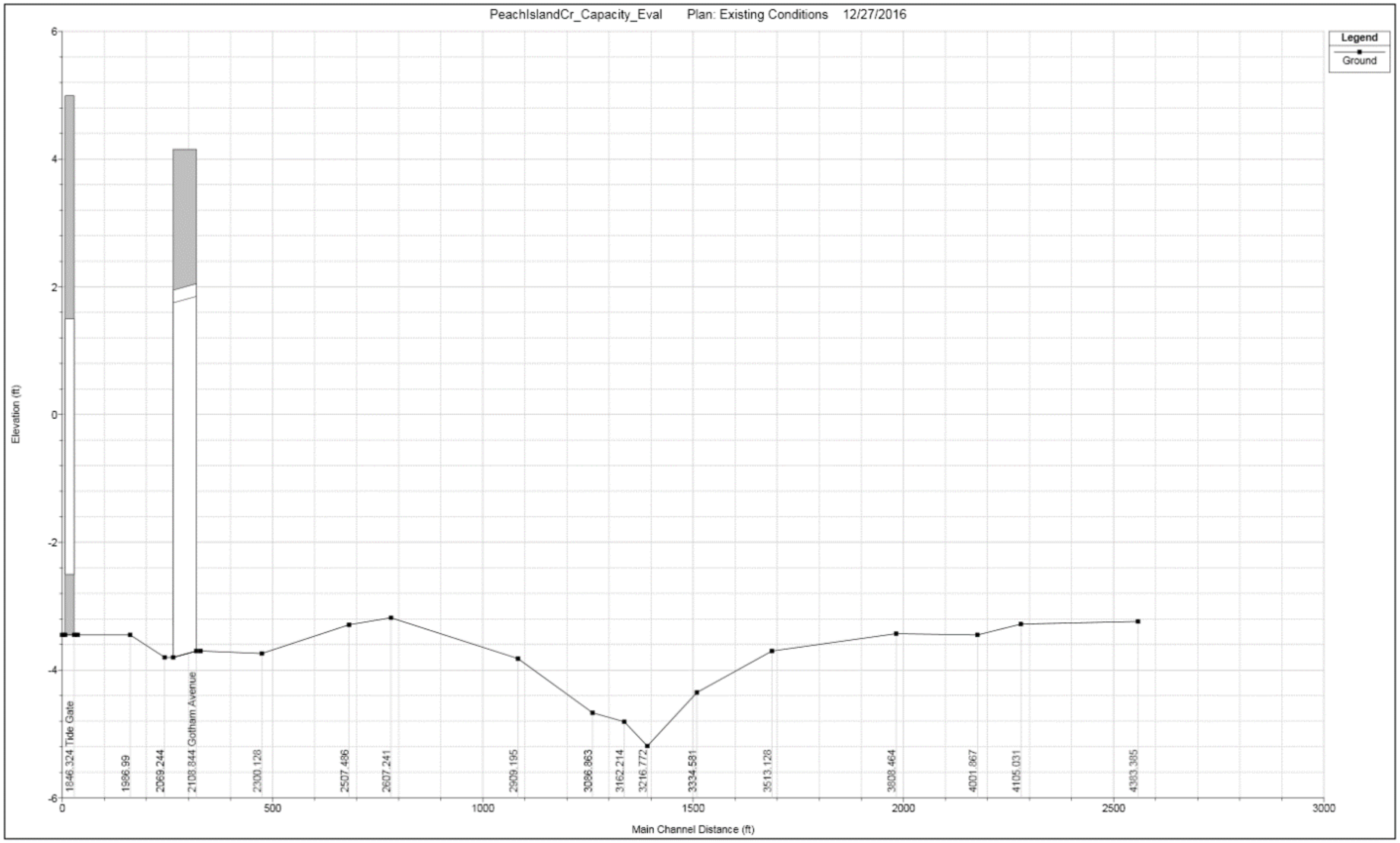

Figure B6-25: HEC-RAS PIC Channel Profile

Not intended for construction.

4.3.2 Steady Flow Data

4.3.2.1 Assignment of HEC-HMS Flows to RAS Cross Sections

The entire drainage area was assumed to contribute through all of the PIC reach.

4.3.3 Culverts and Bridge Openings

Deck elevations for the Gotham Avenue Bridge and the PIC tide gate were extracted from the LiDAR datasets and survey data. Bridges and culverts were assumed perpendicular to the direction of the flow. The pressure and/or weir equations were used to model the structures. Contraction and expansion coefficients were set to 0.3 and 0.5, respectively, upstream and downstream of culverts, bridges, or tide gates. The remaining cross section contraction and expansion coefficients were set to 0.1 and 0.3, respectively.

4.3.4 Tide Gate Configuration

The PIC tide gate was modeled as an inline structure with gates. The gates were assumed fully open for the MLW condition. A fixed WSEL equal to the MLW condition was used as the boundary condition downstream of the tide gate. The three operational gates are assumed to be free of debris and fully functioning. The culverts are set to “flaps prevent negative flow.”

The gates were modeled as 2.7-feet by 2.7-feet square gates (no culverts) due to limitations in steady state HEC-RAS (steady flow data, set opening heights of gates, restricted to the rectangular and square shaped gates). The area is approximately the same as 3-feet diameter pipes, assuming negligible differences in friction losses and flow due to manning’s roughness values and perimeter.

The sections upstream and downstream of the tide gate structure were not surveyed (copied from upstream surveyed section).

4.3.5 Manning’s Roughness Coefficients

Manning’s roughness coefficients were assigned based on year 2015 aerial imagery, as noted in **Table B6-20**. A building complex under development at Palmer Terrace in 2015 was assumed to be developed.

Table B6-20: PIC - Manning’s Roughness Coefficients

Manning’s n value	Description
0.013	Paved area
0.035	Partially paved and grass areas
0.04	Lawn
0.045	Channel
0.05	Isolated trees
0.06	Unmaintained grass
0.08	Wetland
0.09 to 0.1	Dense trees

4.4 Proposed Conditions

4.4.1 Description of Proposed Project Scenario

One Proposed Project scenario was simulated. The proposed scenario considered a 400 cfs proposed pump station located approximately 175 feet upstream of the Gotham Avenue bridge, transferring flows directly to Berry's Creek Canal. Dredging was not evaluated due to the existing contamination.

4.4.2 Proposed Conditions: Pump Station

The proposed pump station upstream of the Gotham Avenue bridge was modeled as a change in flow. The maximum pumping size is assumed to be 400 cfs.

4.5 Model Results

4.5.1 Validation or Comparisons with Other Studies

No validation was performed in this phase of the simplified modeling.

4.5.2 HEC-RAS Modeling Results

The PIC proposed pumping scenarios were compared to the existing conditions displayed in **Table B6-21.**

Table B6-21: Representative PIC Existing and Proposed WSELs

RS	Event	Existing WSEL [feet NAVD 88]	Change from Existing WSEL [feet]
			Proposed Pumping Scenario
4383.385	25-yr	6.50	-1.23
	100-yr	7.07	-0.80
2154.545	25-yr	6.33	-1.45
	100-yr	6.89	-0.89
2108.844	**Gotham Avenue**		
2069.244	25-yr	6.26	-3.42
	100-yr	6.80	-0.87
1986.99	25-yr	6.28	-3.37
	100-yr	6.81	-0.86
1862.476	25-yr	6.28	-3.55
	100-yr	6.82	-0.87
1846.324	**PIC Tide Gate**		

Not intended for construction.

4.5.3 Proposed Project Scenarios Selected for Further Evaluation

The Design Team evaluated the simplified modeling results and choose not to proceed with further evaluation of concepts within PIC.

4.5.4 Proposed Project Scenarios Eliminated from Further Evaluation

The proposed pumping scenario was screened out by The Design Team. WSEL profiles were not included as the model was put on hold and ultimately dismissed due to the contamination concern.

5.0 Losen Slote

Losen Slote (LOS) is located in the City of Hackensack, Township of South Hackensack, Borough of Little Ferry, and Borough of Moonachie. LOS is divided into three DIAs, where proposed changes were evaluated for flood management benefits. These DIAs are shown in **Figure B6-26**.

DIA ‘H’ is the most upstream DIA in LOS, and extends into the Township of South Hackensack in the north and is bounded primarily by Kavrick Street in the Borough of Little Ferry to the south. During this level of modeling effort, it was assumed that a portion of the flow from DIA ‘H’ is routed through the culvert beneath Main Street, discharging to the Hackensack River. Potentially, some of this flow could enter the LOS. This area was not surveyed and there was little supplemental information to evaluate this flow breakout during the simplified modeling effort. The majority of DIA ‘H’ was not considered a heavily flood prone area and opportunities along LOS for flood risk reduction were focused primarily on DIAs ‘I’ and ‘O.’ DIA ‘I’ is the largest area and contains the majority of the developed residential and industrial areas along LOS. DIA ‘O’ extends from the upstream portion of the downstream marshy tidal wetlands and continues to the LOS tide gate and the levees located along the Hackensack River. **Table B6-22** provides sizes of each of these DIAs.

Table B6-22: Summary Statistics of LOS Drainage Improvement Areas

Drainage Improvement Area	Location	Drainage Area [Square Miles]
H	Township of South Hackensack to the north and Kavrick Street in the Borough of Little Ferry to the south	0.35
I	Between Kavrik Street and the marshy tidal wetland area	0.54
O	Marshy tidal wetland area to the LOS tide gate	0.24
Total Drainage Area		**1.13**

Figure B6-27 shows the key hydraulic features of the LOS. LOS flows primarily in a North to South orientation. The HEC-RAS model was initiated at the most upstream surveyed structure outfall, just south of Main Street. In the upstream developed areas, much of the reach runs underground through extended subsurface piping. Only a small portion of the channel is open upstream of Redneck Avenue prior passing underground, resurfacing just downstream of Williams Avenue. The channel then runs parallel to Liberty Street passing under bridges at Union Avenue and Moonachie Road. LOS enters the tidal marsh area, defined by low slope channels and undeveloped expansive floodplains. The stream meanders under a single span bridge access road to a waste water treatment facility and to the LOS tide gate.

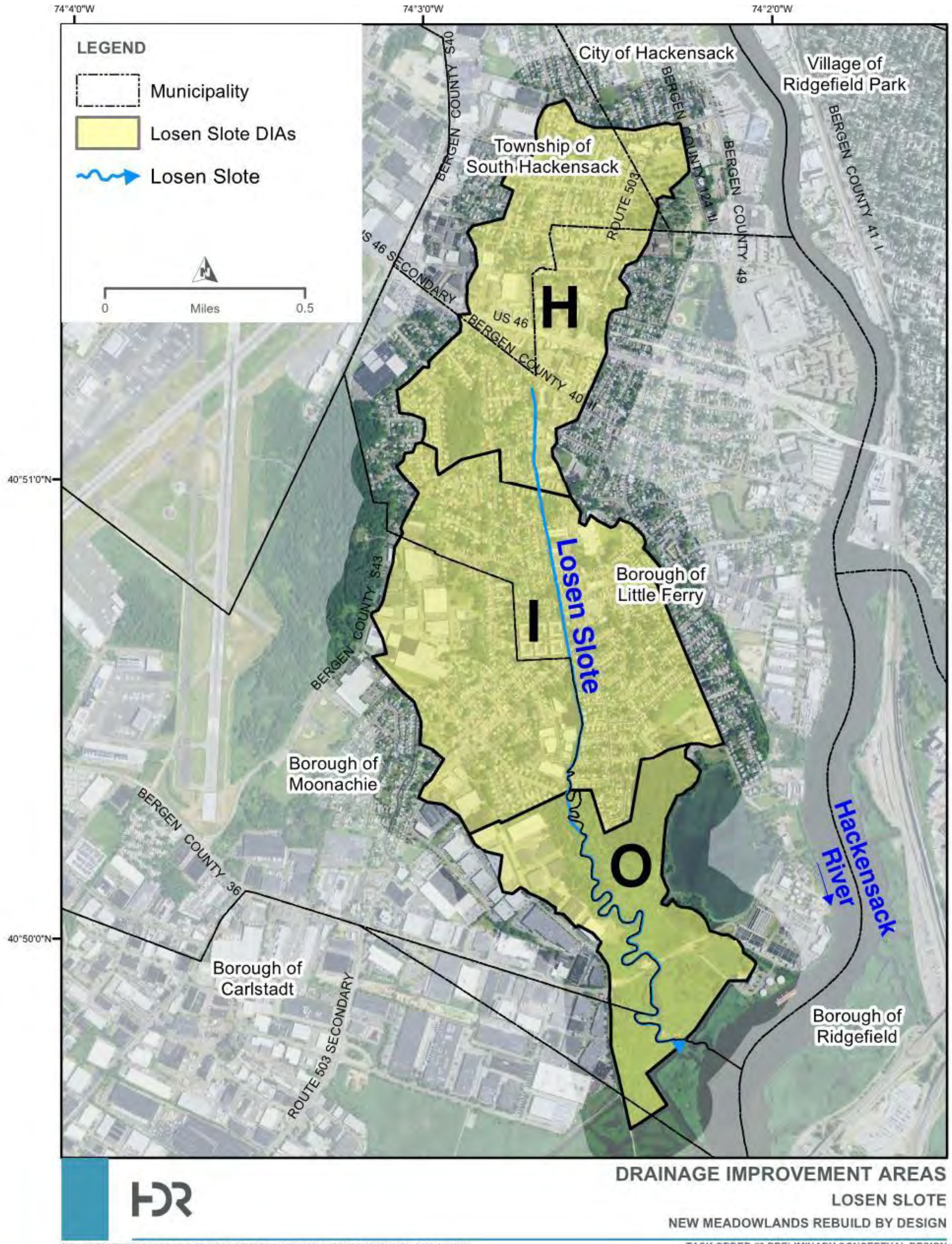

Figure B6-26: LOS Drainage Improvement Areas

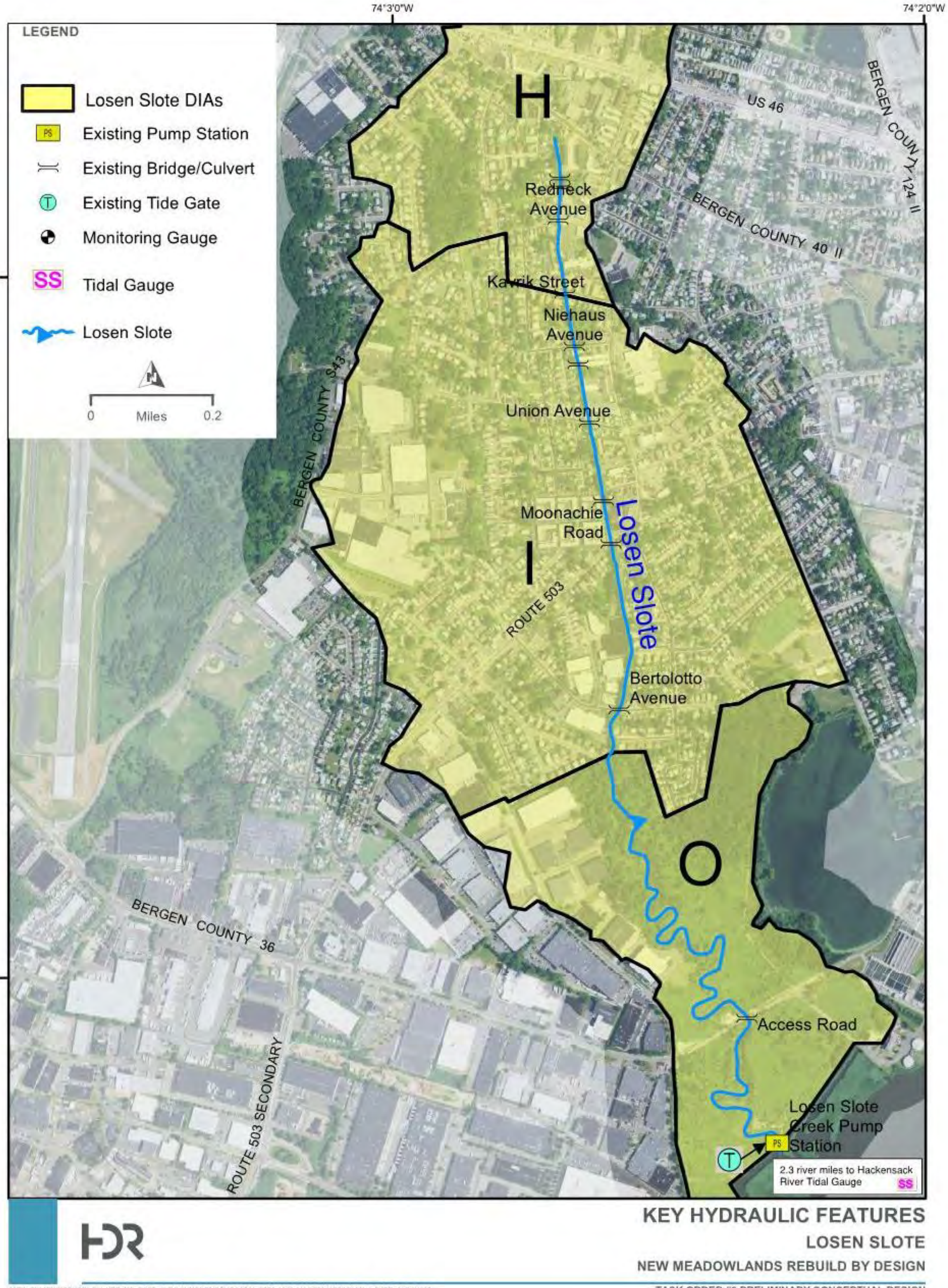

Figure B6-27: LOS Key Hydraulic Features

5.1 Data Sources

5.1.1 Survey

The survey of ditch cross sections and structure information was performed by AECOM survey personnel (AECOM 2016). The survey for LOS was conducted in September 2016. Field surveyed sections were taken at 23 cross sections and 10 structures along the northern 1.0 miles of LOS (see **Figure B6-28** for locations). Surveyed cross sections were used to describe channel conditions. No surveys were conducted upstream of Main Street as the major flow pathway open channel sections were assumed to start south of Main Street. No surveys were taken in the 1.3 miles of downstream marshy tidal wetlands, this includes the access road structure at RS 2104.251 and the tide gate at RS 90. Data from a separate hydraulic study by URS was used in the downstream marshy tidal wetlands (URS 2014). Information from as-built plans of the tide gate and pump station were used to model this structure. Incomplete channel survey includes:

- Survey of structure at RS 2104.251; and
- Upstream and downstream tide gate stream channel sections (RS 75.24606 and RS115.7447).

5.1.2 LiDAR

A LiDAR DEM with 2-foot pixel resolution was supplied by the New Jersey Sports and Exposition Authority (NJSEA). Quantum Spatial collected the near-infrared data in April 2014 and the LiDAR was derived by using TIN processing. The horizontal datum is NAD83 (2011) and the data was projected to the New Jersey State Plane coordinate system. Elevation units are feet in the NAVD 88. The LiDAR terrain data was used for the overbank cross sections with survey data used to define the channel areas.

5.1.3 Design Flows

The design flows for the LOS were sourced from Phase 1 of the HEC-HMS model (“HEC-HMS Model”). The Proposed Project specifications used 48-hour durations for the 2-year (50 percent annual exceedance probability) to 100-year (1 percent annual exceedance probability) reoccurrence intervals.

A portion of the flow within DIA ‘H’ was not included into the LOS model, as it was assumed to flow down Main Street and out of the LOS domain. The LOS analysis existing and proposed WSELs act as the downstream boundary condition for the flooding in the vicinity of Carol Place in the eastern section of Moonachie within an additional Infoworks model.

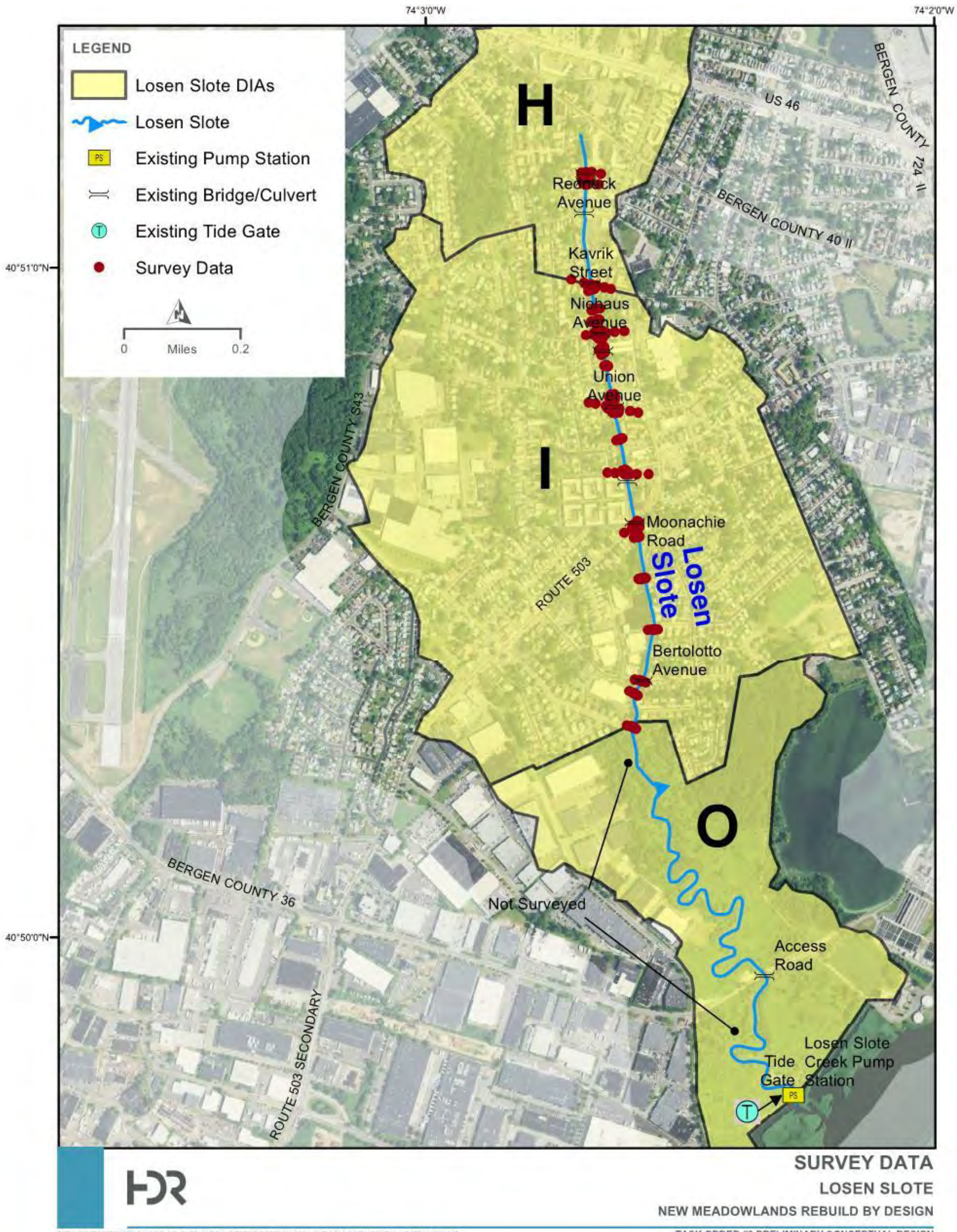

Figure B6-28: LOS Surveyed Cross Section Locations

5.1.4 Tidal Conditions

Downstream tidal conditions were obtained from a study by the Woods Hole Group commissioned by the NJSEA (Woods Hole Group 2007). Model boundary conditions downstream of the LOS tide gate were defined as the MLW condition from the 1983 to 2001 epoch. The nearest tidal gauge to the LOS is the Mill Creek Canal, located approximately 2.3 river miles downstream of the LOS tide gate along the Hackensack River. The MLW is -2.77 feet NAVD 88 while the MHW is 2.86 feet NAVD 88.

5.1.5 Tide Gates and Pump Stations

The LOS tide gate was replaced in 1999. The LOS tide gate was not surveyed. Dimension for the structure were obtained from as-built plans prepared by Kenneth Job (Job 1999). It was interpreted from the plans that the tide gates consist of three 5-feet tall rectangular gate openings, likely with circular flow through structures. According to a National Center for Computation Hydroscience and Engineering (NCCHE) report, three pumps with typical 43,000 gallons per minute (gpm) (95.7 cfs) discharge, and a maximum discharge capacity of 129,000 gpm (287 cfs), discharge water from LOS just downstream of the tide gate to the Hackensack River (NCCHE 2014).

5.1.5 Other Studies and Models

The channel sections from a HEC-RAS model developed by URS was appended to the 2014 LiDAR for the overbanks in the downstream marshy tidal wetland area (URS 2014). No additional field topographic survey data was available to verify the supplemental HEC-RAS models from others. According to the report associated with this model, the channel was surveyed (dates of survey unknown).

5.2 Model Limitations

5.2.1 Survey

Submerged structure geometry, type, and, in some cases, locations were assumed. The modeling assumptions were made for submerged or partially submerged structures. There were locations that were not surveyed. Engineering judgment and supplemental information had to be used to make appropriate assumptions where there was missing or incomplete channel and structure surveyed sections.

5.2.2 Ongoing Channel Improvements

There is ongoing dredging occurring in the marshy tidal wetland area. The current dredging is not taken into account in the HEC-RAS simulations.

5.3 Existing Conditions

An existing conditions HEC-RAS model was created for LOS running from north Main Street to just downstream of the LOS tide gate. This model acts as a baseline conditions for which Proposed Project scenarios were compared.

5.3.1 Development of Existing Condition Cross Sections

LOS open channel flow was assumed to begin at the outlet of the most upstream culvert surveyed RS 12002.46. Upstream of this location, the drainage system was assumed to be piped. For the upstream-most 5,430 feet, cross sections were developed from the field surveyed cross sections described in **Section 5.1.1**. For the remaining downstream 6,575 feet located in the marshy area, cross section locations were co-located with URS cross sections (URS 2014). For all cross sections, overbank areas were extracted from LiDAR data. The channel area was adjusted based on survey of the URS model (URS 2014).

Figure B6-29 and **Figure B6-30** shows the cross sections defined in the HEC-RAS model.

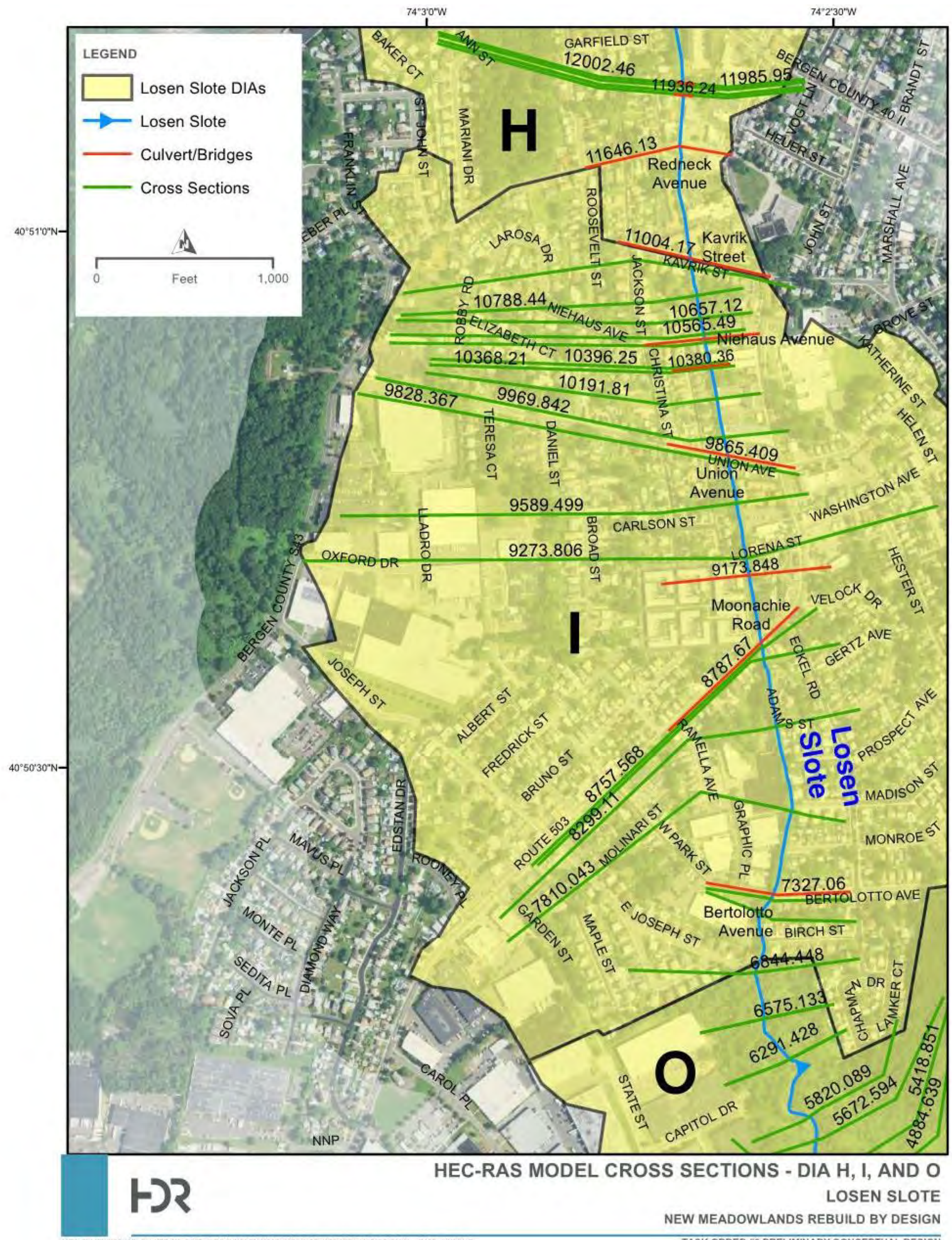

Figure B6-29: LOS HEC-RAS Model Cross Sections for DIAs H, I, and O

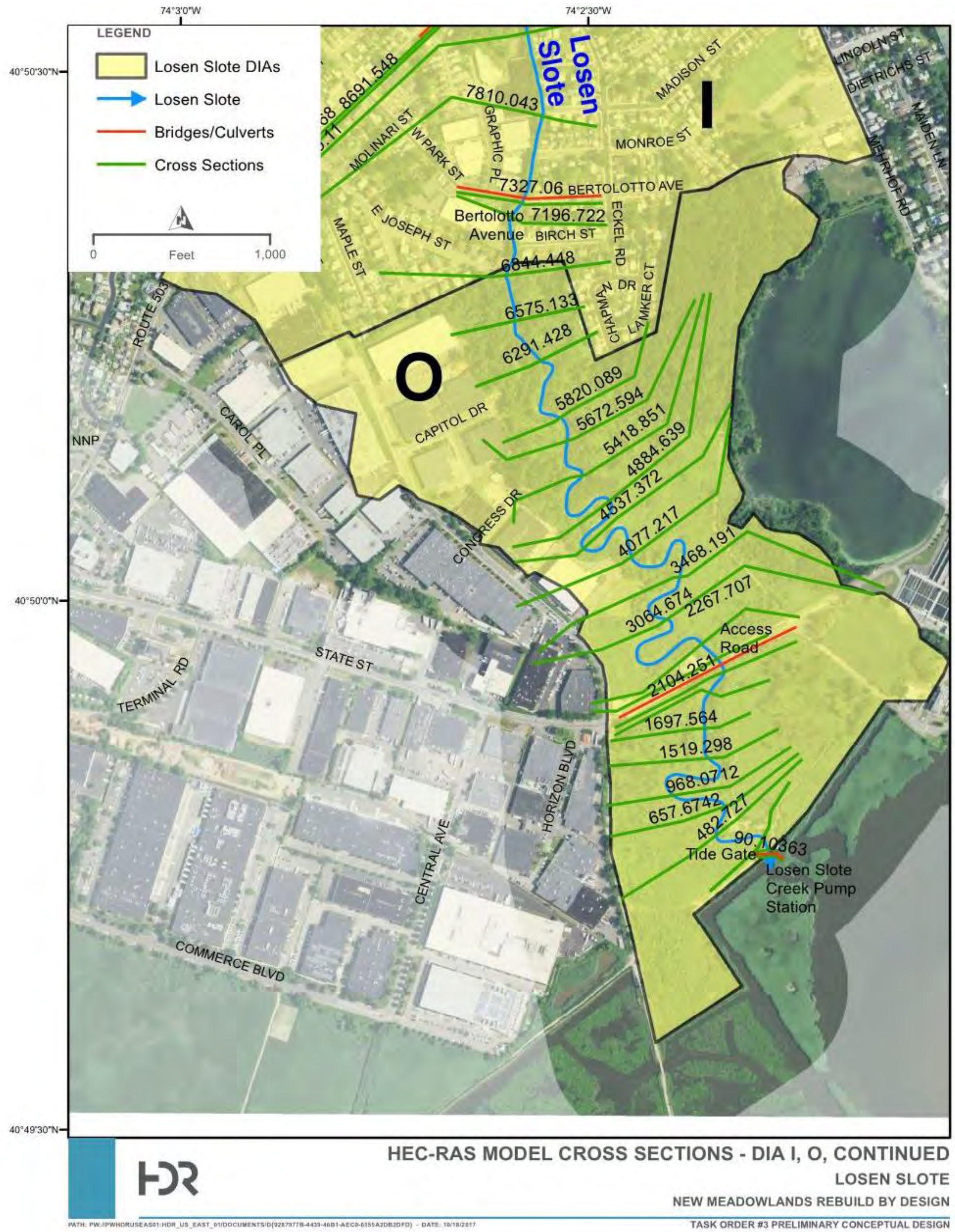

Figure B6-30: LOS HEC-RAS Model Cross Sections for DIAs I and O

5.3.2 Steady Flow Data

5.3.2.1 Assignment of HEC-HMS Flows to RAS Cross Sections

It was assumed that a portion of the flow from DIA 'H' is routed through the culvert beneath Main Street, discharging to the Hackensack River. The remaining steady flow data was obtained from the HEC-HMS model node locations and applied at appropriate HEC-RAS cross section locations.

5.3.3 Ineffective Flow Areas and Blocked Obstructions

Due to the highly developed nature of the floodplain in the upper reach, the ineffective flow locations typically corresponded with blocked obstruction areas. Buildings were modeled as blocked obstructions. Areas of dense residential development were assumed to have little flow conveyance or storage capacity and were simulated as blocked obstructions.

5.3.4 Culverts and Bridge Openings

There are six culverts or bridges along the LOS reach length. Deck elevations were extracted from the LiDAR datasets and survey data. Bridge low chords, culvert sizes, pipe inverts, and upstream and downstream cross sections were extracted from surveyed data, with the exception of the structure at RS 2104.251. The field surveyors noted uncertainty in some structure measurements, as detailed in **Section 5.1.1**.

The most upstream culvert, which outlets at RS 12002.46, was not simulated as it was assumed the remaining upstream network was piped out of the LOS drainage area.

No survey data was available for the access road to the waste water treatment facility (RS 2104.251). This structure was described by the existing URS model, but was not field validated (URS 2014). The following structures and their data sources included in the LOS model are listed in **Table B6-23**. Information for the structures that were not surveyed was obtained from the existing URS model (URS 2014).

Table B6-23: LOS – Existing Structures and Data Sources

RS	Description	Source
11445	Redneck Avenue	Survey
10537.65	Niehaus Avenue	Survey
10380.36	Foot Bridge	Survey
9865.409	Union Avenue	Survey
9173.848	Moonachie Road	Survey
2104.251	Access Road	URS Model

Not intended for construction.

Bridges and culverts were assumed perpendicular to the direction of the flow. The energy and pressure and/or weir equations were used to model the structures. Contraction and expansion coefficients were generally set to 0.3 and 0.5, respectively, upstream and downstream of culverts or bridges. The remaining cross section contraction and expansion coefficients were set to 0.1 and 0.3, respectively.

5.3.5 Pump Station

The existing LOS pump station is located at RS 90. The effects of the pump station were modeled as a change in flow at this location. Pumped water was simulated as discharge into the Hackensack River and assumed to be lost from the system. The pump station was assumed to be fully functional and running at maximum capacity (287 cfs).

5.3.6 Tide Gate Configuration

LOS tide gate structures were modeled as an inline structure with three 5-feet by 5-feet square gates (Job 1999). Tide gate openings were assumed to be square as to allow steady state flow simulation including the "gate height opening" option not allowed with circular openings. The gates were assumed fully open for the MLW condition. A fixed WSEL equal to the MLW condition was used as the boundary condition downstream of the tide gate.

5.3.7 Manning's Roughness Coefficients

Manning's roughness coefficients were allocated based on year 2015 aerial imagery and guidance from the Wetland Restoration Project for the downstream marshy section, as noted in **Table B6-24** (NOAA 2009). The coefficients used in the URS 2014 model were not incorporated.

Table B6-24: LOS - Manning's Roughness Coefficients

Manning's n Value	Description
0.013 – 0.03	Developed: buildings/pavement, some grass
0.045	Straight ditch segments
0.05-0.06	Unmowed grass - some trees
0.02-0.1	Wetlands
0.07	Dense trees, isolated areas
0.10	Dense trees, large areas

5.4 Proposed Conditions

5.4.1 Description of Proposed Project Scenarios

Two Proposed Project scenarios were simulated using HEC-RAS, as detailed in **Table B6-25**. Proposed conditions included an additional pump station, reach bypass, and channel excavation.

Table B6-25: LOS Ditch Proposed Project Scenarios

Scenario Number	Plan Name	Description
2	PRP_S2I	DIA I : Local pump station discharging into Lower LOS
3	PRP_S3IO	DIA I: Local pump station discharging into Lower LOS DIA O: Excavate channel

5.4.2 Proposed Conditions: Channel Modifications

Channels were modified in the downstream 6,575 feet of LOS within DIA 'O' for Scenario 3. No channel modifications were proposed for Scenario 2. Generally, existing cross sections were excavated to increase conveyance capacity. The proposed channel cross sections were defined by a trapezoidal shape, maximum channel width, a channel side slope, and channel invert. In the marshy tidal wetlands, the cross section top widths were not influenced by existing infrastructure, top widths and bottom widths were assumed to increase by a maximum of 150 percent, and generally no fill was added. The top channel width was based on an assumed right-of-way that started 15 feet from the edge of existing buildings and parking lots. The side slopes of the proposed ditch were assumed at 3H:1V or milder. Assumed dredging was not influenced by constructability, this was to be evaluated in the next phase of development.

The channel inverts were lowered in areas to promote a positive slope. **Figure B6-31** shows the change in proposed channel inverts.

5.4.3 Proposed Conditions: Culvert/Bridge Opening Modifications

Culvert and bridge openings were not directly adjusted. Increased flow conveyance was indirectly achieved for the structure at RS 2104.251 through modifications to the upstream and downstream channel cross sections described in **Section 5.4.2**.

5.4.4 Proposed Conditions: Pump Stations

The proposed localized pump station described in Scenario 3 was modeled as a change in flow. The pump station was assumed to be located at RS 12002.46 at an undeveloped location along the LOS and would discharge flow back into the channel at RS 6291.428. The localized pumping was intended to provide flow relief along the heavily developed upstream areas with structures of undersized conveyance (RS 12002.46 to RS 6844.448) in DIAs 'H' and 'I,' and discharging into the predominantly undeveloped marshy tidal wetlands in DIA 'O.' The pumping capacity was modeled as a maximum capacity of 125 cfs.

5.5 Model Results

5.5.1 Validation or Comparisons with Other Studies

The LOS hydraulic model was not directly calibrated or validated. Reported flood prone areas were aggregated from measured and anecdotal data (NJSEA 2005, NCCHE 2014, Bergen County Office of Emergency Management 2015, CAG Meeting 2016, Meeting with the Borough of Little Ferry 2016) . The developed residential area in the upper section of LOS was targeted as a potential improvement area and assumed to be flood prone due to proximity of the buildings to LOS and likely undersized hydraulic structures. Input from the municipality and CAG meetings was also considered in evaluating water surface reductions in the developed portion of the LOS reach.

5.5.2 HEC-RAS Modeling Results

Results showing the change in WSEL for the LOS Proposed Project scenarios for the 2-, 5-, and 10-year return periods compared to existing conditions are displayed in **Table B6-26**. The lower return period storm events were considered more realistic targeted reductions for LOS.

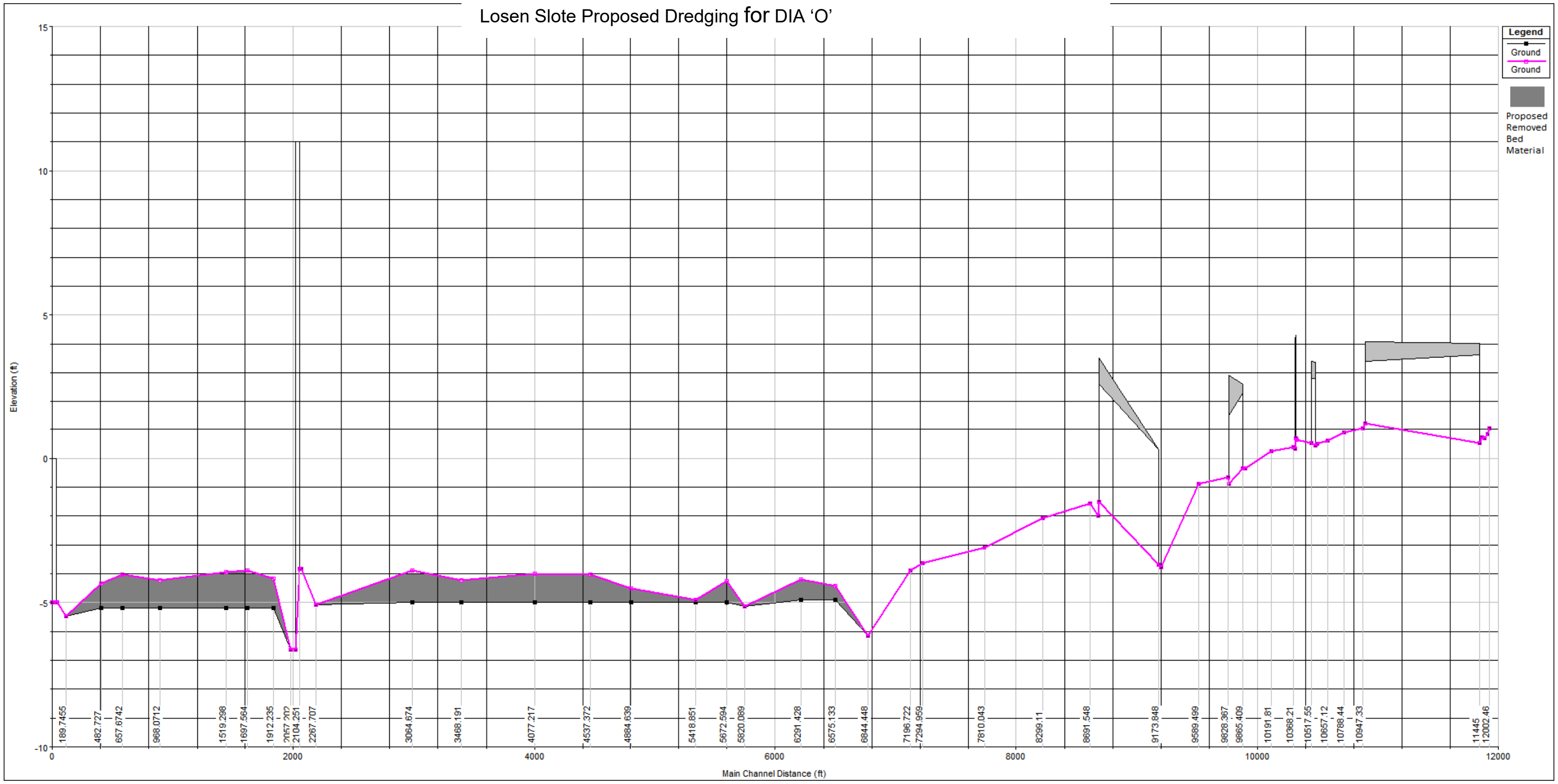

Figure B6-31: HEC-RAS LOS Channel Profile

Not intended for construction.

Table B6-26: Representative LOS Existing and Proposed WSELs

RS	Event	Existing WSEL [feet NAVD 88]	Change from Existing WSEL [feet]	
			Scenario 2	Scenario 3
12002.46	2-yr	4.64	-3.04	-3.04
	5-yr	4.96	-3.36	-3.36
	10-yr	5.18	-3.18	-3.18
Lower Portion of DIA "H" and Upper Portion of DIA "I"				
10396.25	2-yr	4.14	-3.08	-3.08
	5-yr	4.63	-3.57	-3.57
	10-yr	4.86	-3.5	-3.50
Bridge at RS 10380.36				
10368.21	2-yr	3.89	-2.90	-2.90
	5-yr	4.20	-3.21	-3.21
	10-yr	4.40	-3.12	-3.12
9273.806	2-yr	2.06	-3.15	-3.24
	5-yr	2.45	-2.79	-3.00
	10-yr	2.69	-2.4	-2.53
Bridge at RS 9173.85				
8757.568	2-yr	1.87	-2.97	-3.06
	5-yr	2.23	-2.57	-2.78
	10-yr	2.43	-2.15	-2.28
5820.089	2-yr	-1.17	0.00	-0.54
	5-yr	-0.36	0.00	-0.21
	10-yr	0.26	0.00	-0.13
Middle Portion of Drainage DIA "O"				
1519.298	2-yr	-1.57	0.00	-0.22
	5-yr	-0.57	0.00	-0.05
	10-yr	0.11	0.00	-0.02
90	Losen Slote Tidal Gate			

Not intended for construction.

5.5.3 Proposed Project Scenarios Selected for Further Evaluation

Scenario 3 was selected by The Design Team for further evaluation. Scenario 3 represents channel excavations in DIA 'O,' and a localized pump station diverting flow from the high density residential areas to the marsh wetland area. This scenario represented the maximum WSEL reduction and was carried forward for further evaluation. Water surface profiles between existing and proposed conditions Scenario 3 were compared in **Figure B6-32**.

5.5.4 Proposed Project Scenarios Eliminated from Further Evaluation

The remaining Scenario 2 was not carried forward for further assessment.

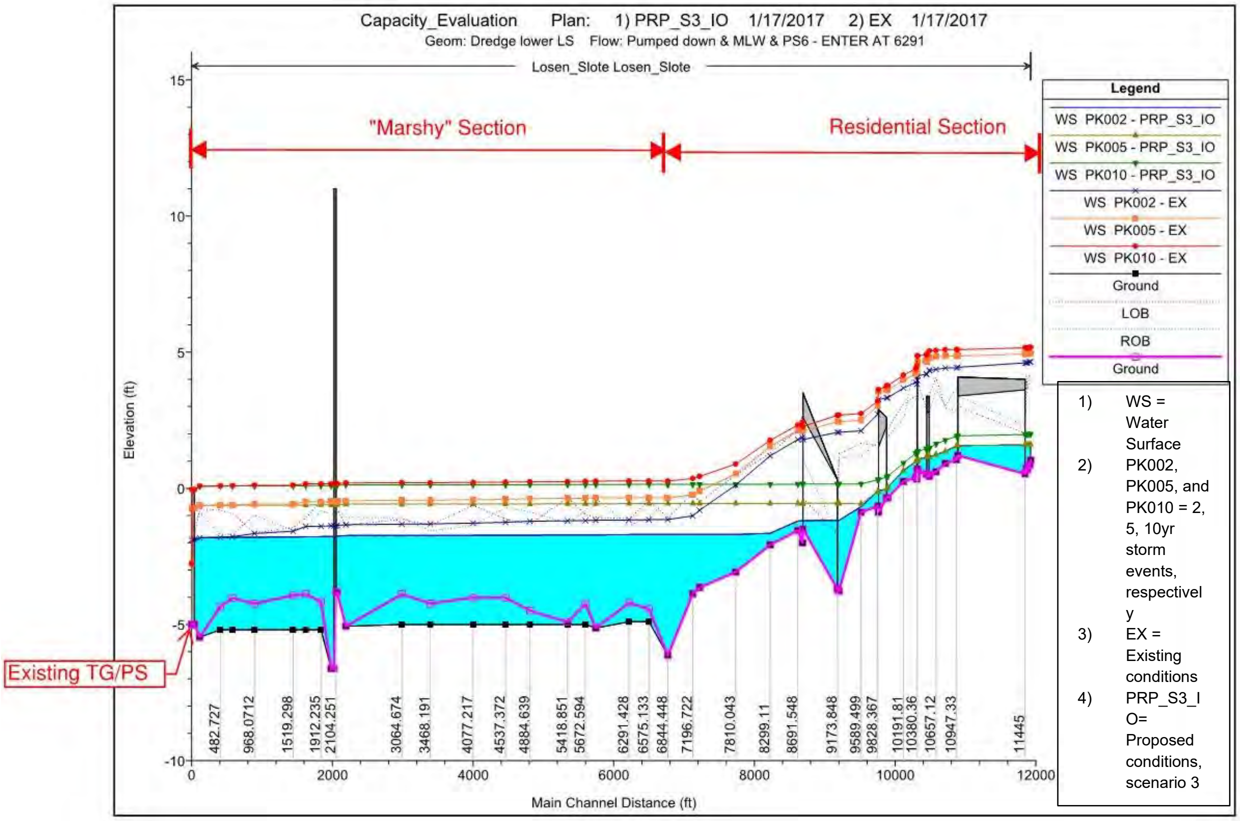

Figure B6-32: LOS WSEL Comparison between Existing and Selected Proposed Scenario 3 for the 2, 5, and 10-Year Storm Events

Not intended for construction.

6.0 DePeyster Creek

DePeyster Creek (DEP) is located within the Borough of Little Ferry. The contributing drainage area is shown in **Figure B6-33**. Flood management benefits are evaluated in a single DIA N, covering the entire drainage area. The drainage area extends from Washington Avenue and Maple Street in the north to the Hackensack River and Industrial Avenue in the south, totaling 0.13 square miles (see **Table B6-27** for details).

Table B6-27: Summary Statistics of DEP Drainage Improvement Areas

Drainage Improvement Area	Location	Drainage Area [square miles]
N	Between Washington Avenue to Industrial Avenue	0.13

Figure B6-34 shows the key hydraulic features of DEP. The reach evaluated extended from 400 feet upstream of Louis Street to the confluence with Hackensack River and just south of Industrial Avenue. The creek originates in a forested area south of Washington Avenue. The DEP tide gate, pump station, and the culvert north of Louis Street are the only structures within the modeling domain.

6.1 Data Sources

6.1.1 Survey

The survey of ditch cross sections and structure information was performed by AECOM survey personnel (AECOM 2016). The channel and structure survey for DEP was conducted in September 2016. Field surveyed sections were taken at 9 cross sections and 1 structure along the 0.5 miles of DEP (see **Figure B6-35** for locations). Surveyed cross sections were used to describe channel conditions. Survey taken at the tide gate was part of a separate survey performed by Robinson Aerial Surveys, Inc. Missing or incomplete channel/structure survey included the channel sections upstream and downstream of the tide gate. Missing tide gate and pump station survey included dimensions and inverts of upstream flow-through structure, elevation of top of tide gate, and intake pump size.

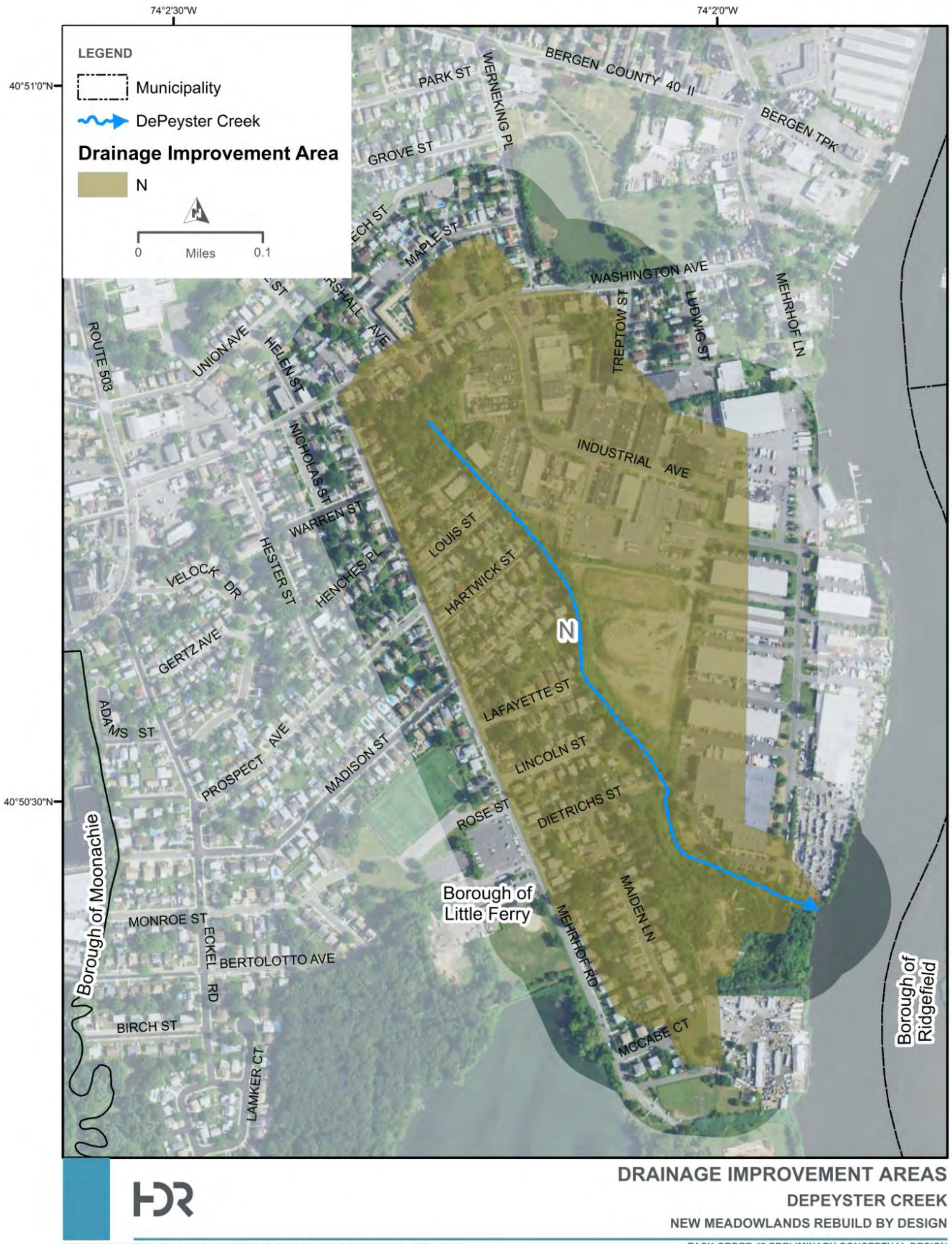

Figure B6-33: DEP Drainage Improvement Areas

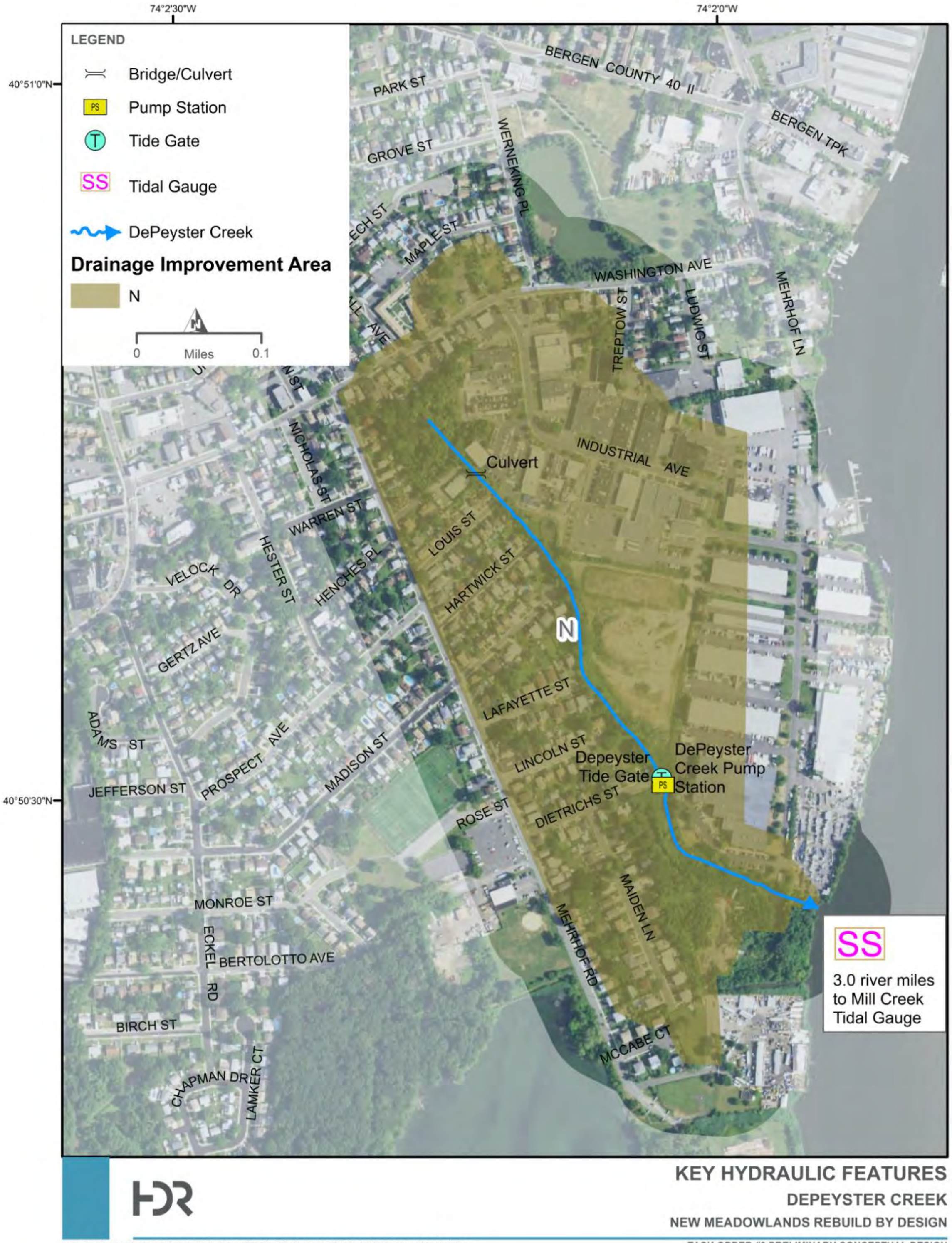

Figure B6-34: DEP Key Hydraulic Features

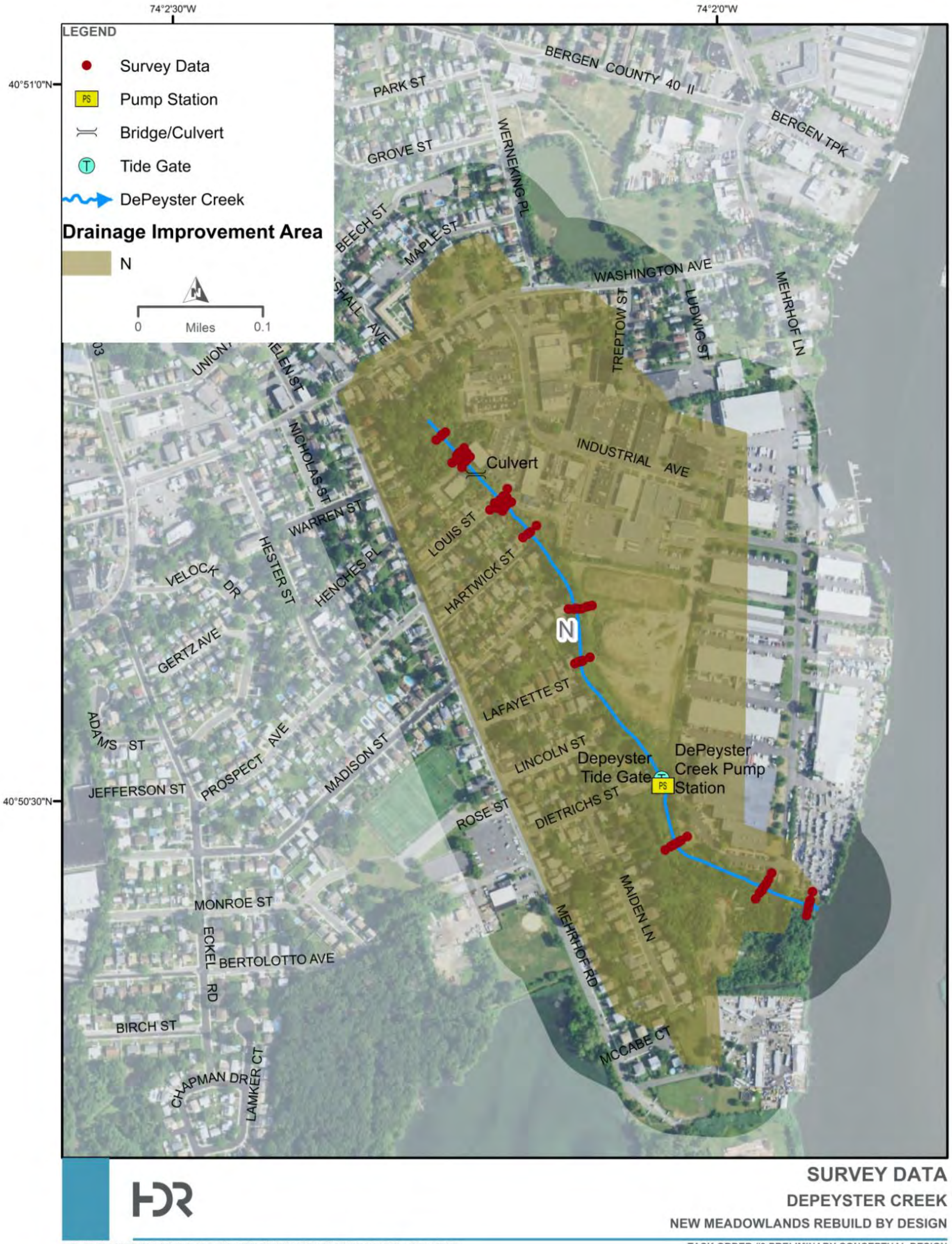

Figure B6-35: DEP Surveyed Cross Section Locations

6.1.2 LiDAR

A LiDAR DEM with 2-foot pixel resolution was supplied by the New Jersey Sports and Exposition Authority (NJSEA). Quantum Spatial collected the near-infrared data in April 2014 and the LiDAR was derived by using TIN processing. The horizontal datum is NAD83 (2011) and the data was projected to the New Jersey State Plane coordinate system. Elevation units are feet in the NAVD 88. The LiDAR terrain data was used for the overbank cross sections with survey data used to define the channel areas.

6.1.3 Design flows

The design flows for the DEP were sourced from Phase 1 of the HEC-HMS model ("HEC-HMS Model"). The Proposed Project specifications used 48-hour durations for the 2-year (50 percent annual exceedance probability) to 100-year (1 percent annual exceedance probability) reoccurrence intervals. DEP was represented by a single sub-basin within the HEC-HMS model, therefore, only one flow value was assumed for the entire modeled reach.

6.1.4 Tidal Conditions

Downstream tidal conditions were obtained from a study by the Woods Hole Group commissioned by the NJSEA (Woods Hole Group 2007). Model boundary conditions downstream of the DEP tide gate were defined as the MLW condition from the 1983 to 2001 epoch. The nearest tidal gauge to the DEP is the Mill Creek Canal, located approximately 3.0 river miles downstream of the DEP tide gate. The MLW is -2.77 feet NAVD 88 while the MHW is 2.86 feet NAVD 88.

6.1.5 Tide Gates and Pump Stations

The DEP tide gate is located 900 feet upstream of the DEP confluence with the Hackensack River. The DEP tide gate was not surveyed. Dimensions for the tide gate were obtained from Hydraulic & Hydrologic Calculations for Proposed Stormwater Management Systems (Uni Reality, LLC 2007). Available plans that were available but not incorporated into the model include "preliminary" and "not for bid" for DEP pump station upgrades. Since there was no as-built information, the Hydraulic & Hydrologic report, AECOM survey, and various other reports, such as the NCCHE report, were used.

Based on the AECOM survey, Hydraulic & Hydrologic report, and NCCHE report the gate structure consisted of one, 3.5-feet by 4.4-feet circular opening with a flap gate preventing back flow (Uni Reality, LLC 2007, NCCHE 2014, AECOM 2016). Three 1,100 gpm (2.45 cfs) pumps with a maximum discharge capacity of 3,300 gpm (7.3 cfs) moves flows from the upstream side of the DEP tide gate to the downstream side flowing to the Hackensack River during flooding episodes.

6.2 Existing Conditions

An existing conditions HEC-RAS model was created for DEP running from south of Washington Avenue to the confluence with the Hackensack River. This model acts as a baseline conditions for which proposed project scenarios were compared.

6.2.1 Development of Existing Condition Cross Sections

Cross sections were developed from the field surveyed cross sections described in **Section 6.1.1**. For all cross sections, overbank areas were extracted from LiDAR data. **Figure B6-36** shows the cross sections defined in the HEC-RAS model.

Figure B6-36: DEP HEC-RAS Model Cross Sections for Area N

6.2.2 Steady Flow Data

6.2.2.1 Assignment of HEC-HMS Flows to RAS Cross Sections

The entire drainage area was assumed to contribute through all of the DEP reach.

6.2.3 Flood Prone Areas

The following information regarding flood prone areas in the vicinity of DEP from the Hackensack Meadowlands Floodplain Management Plan helped establish potential improvements (NJSEA 2005). The report cites that water greater than 4.1 feet (25-year storm) leaves the watershed and enters LOS. Cross basin flow was not accounted for in the HEC-HMS and HEC-RAS models at this level of development.

The report noted the following flooding issues:

- Localized flooding in the vicinity of Hartwick Street in the Borough of Little Ferry due to backup of a nearby drainage ditch;
- National Flood Insurance Program Repetitive Loss properties;
- Inundation of DEP pump station causes flooding; and
- During high intensity storm events, the drainage ditch leading to the pump station overflows due to lack of pumping capacity.

6.2.4 Culverts and Bridge Openings

There is one culvert along the DEP reach length. Deck elevations, culvert size, pipe inverts, and upstream and downstream cross sections were extracted from surveyed data.

The culvert was assumed perpendicular to the direction of the flow. The pressure and/or weir equations were used to model the structure. Contraction and expansion coefficients were set to 0.3 and 0.5, respectively, upstream and downstream of the culvert and tidal gate. The remaining cross section contraction and expansion coefficients were set to 0.1 and 0.3, respectively.

6.2.5 Pump Station

DEP pump station is collocated with the tide gate at RS 915. The effects of the pump station were modeled as changes in flow at this location. The pump station was assumed to be fully functional and operating at maximum capacity (7.3 cfs). The pumped water was assumed to be discharged approximately 700 feet downstream of the tide gate and pump station, reintroducing the flow back into the system.

6.2.6 Tide Gate Configuration

DEP tide gate structures were modeled as an inline structure with gates. One 4.38-feet by 3.8-feet fully open rectangular gate represented the opening. The tide gate opening was assumed to be rectangular as to allow steady state flow simulation including the “gate height opening” option not allowed with circular openings. A fixed WSEL equal to the MLW condition was used as the boundary condition downstream of the tide gate.

6.2.7 Manning’s roughness coefficients

Manning’s roughness coefficients were allocated based on year 2015 aerial imagery, as noted in **Table B6-28**.

Table B6-28: DEP - Manning's Roughness Coefficients

Manning's n value	Description
0.013-0.022	Developed: buildings/pavement
0.035-0.045	Grass overbanks
0.04	Straight ditch segments
0.05	Mix of trees and grass
0.065	Dense trees, isolated areas

6.3 Proposed Conditions

6.3.1 Description of Proposed Project Scenarios

Five Proposed Project scenarios were simulated, as detailed in **Table B6-29**. Proposed conditions included stand alone and combined improvements including increasing pump capacity, channel excavation, berm construction, and an increase in culvert size.

Table B6-29: DEP Proposed Project Scenarios

Scenario Number	Plan Name	Description
1	PRP_S1N	Dredge channel
2	PRP_S2N	Dredge channel and increase existing pump station capacity to 75 cfs
3	PRP_S3N	Dredging channel, 75 cfs, and increase culvert capacity
4	PRP_S4N	Dredging channel, 200 cfs, and increase culvert capacity
5	PRP_S5N	Dredging channel, pump at 75 cfs, increase culvert capacity, berms

Not intended for construction.

6.3.2 Proposed Conditions: Channel Modifications

Channels were modified through the basin represented by DIA N for all five scenarios. Existing cross sections were excavated to increase conveyance capacity. The proposed channel cross sections were defined by a trapezoidal shape, maximum channel width, channel side slope, and channel invert. The top channel width was based on an assumed right-of-way that started 15 feet from the edge of existing buildings and parking lots. Where the DEP top widths were less restricted by existing infrastructure, top widths were assumed to increase by a maximum of 150 percent. In areas confined by existing infrastructure, a 2H:1V side slope was needed to fit the channel within the setback limits.

The channel inverts were lowered in areas to promote a positive slope. **Figure B6-37** shows the change in proposed channel inverts.

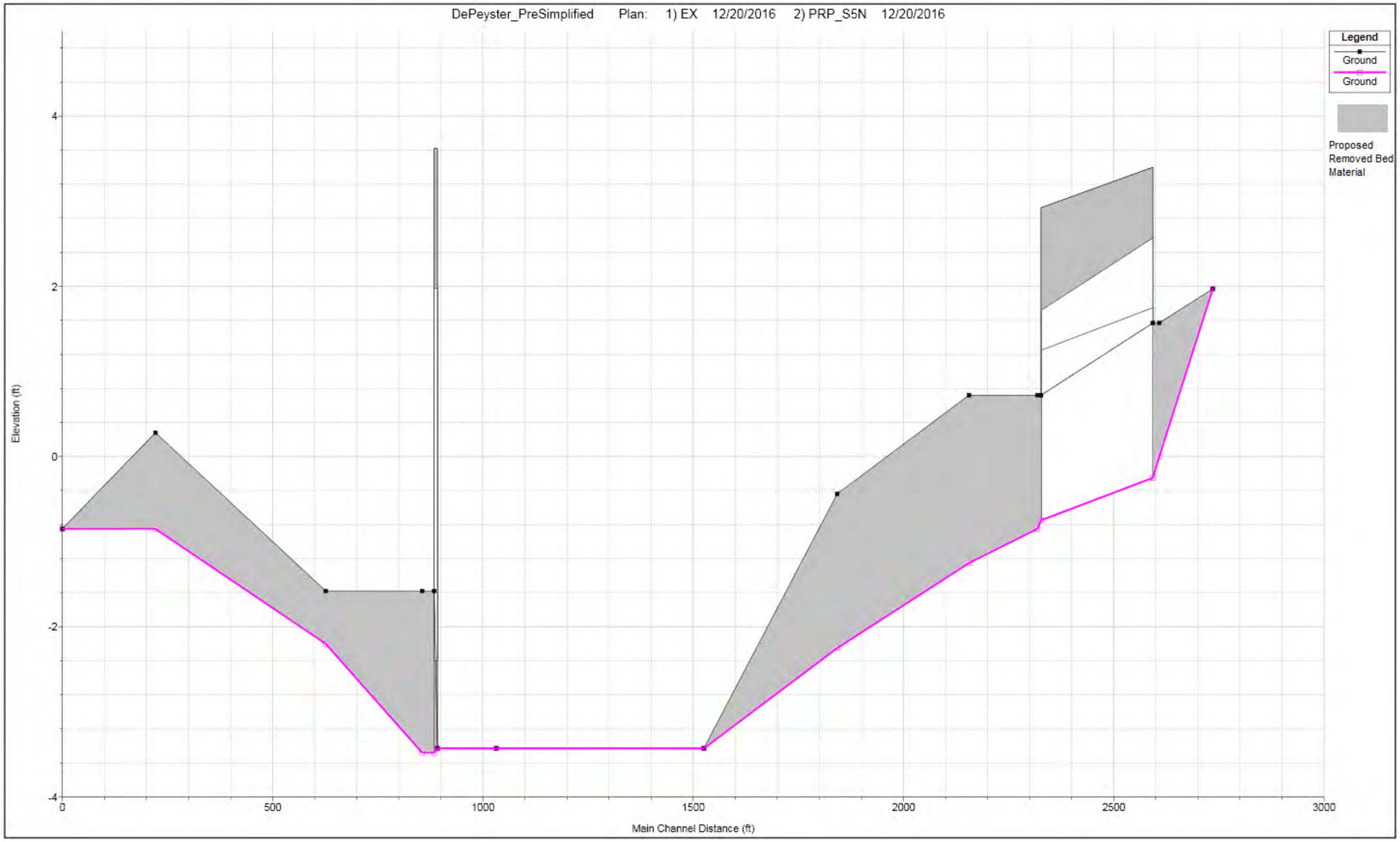

Figure B6-37: DEP Channel Profile

Not intended for construction.

6.3.3 Proposed Conditions: Culvert/Bridge Opening Modifications

The proposed culvert replacement in DEP used a concrete box culvert configuration. The maximum size of the replacement culvert was limited by providing one foot of fill at the sides of the culvert and 18 inches of fill over the top of the culvert, while leaving the roadway elevation as is. **Table B6-30** provides details on culvert and bridge modifications. Ineffective flow area locations and elevations were adjusted to represent the modified openings.

Table B6-30: DEP - Proposed Culvert and Bridge Modifications

Location	Existing Configuration	Proposed Configuration
RS 2511	1.5’ x 1’ elliptical culvert	6’ x 2’ box culvert

Not intended for construction.

6.3.4 Proposed Conditions: Pump Stations

The proposed pump station improvement at the DEP tide gate was modeled as a change in flow. The proposed pump station was assumed to be operating at a maximum capacity of 75 cfs, after iteratively selecting different discharges and looking at the effects on the WSELs. Pump station upgrade recommendations are documented in the flood mitigation study by NCCHE (NCCHE 2014).

6.4 Model Results

6.4.1 Validation or Comparisons with Other Studies

No measured flow or stage information was available within the watershed for validation.

6.4.2 HEC-RAS Modeling Results

Results showing the change in WSEL for the DEP Proposed Project scenarios for the 2-, 5-, and 10-year return periods compared to existing conditions are displayed in **Table B6-31**. The lower return period storm events were considered more realistic targeted reductions for the DEP.

Table B6-31: Representative DEP Existing and Proposed WSELs

RS	Event	Existing WSEL [feet NAVD 88]	Change from Existing WSEL [feet]				
			Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5
Upper Portion of Area N							
2631.09	2-yr	4.15	-0.03	-0.03	-0.36	-0.36	-0.32
	5-yr	4.86	-0.68	-0.68	-0.86	-0.87	-0.75
	10-yr	5.44	-1.20	-1.20	-1.38	-1.35	-1.17
Culvert at RS 2511.15							
2341.1	2-yr	4.04	-0.37	-0.37	-1.91	-1.93	-1.91
	5-yr	4.82	-1.00	-1.00	-2.26	-2.32	-2.26
	10-yr	5.42	-1.52	-1.52	-2.57	-2.67	-2.57
1054.92	2-yr	3.93	-2.14	-2.59	-2.60	-2.68	-2.60
	5-yr	4.76	-2.27	-2.99	-2.99	-3.21	-2.99
	10-yr	5.38	-2.26	-3.28	-3.28	-3.63	-3.28
DEP Tide Gate							
878.57	2-yr	3.28	-1.95	-2.03	-2.03	-2.04	-2.03
	5-yr	3.58	-1.93	-2.01	-2.01	-2.04	-2.01
	10-yr	3.79	-1.93	-2.00	-2.00	-2.05	-1.99
Bridge at RS 9173.85							
244	2-yr	3.04	-1.86	-1.86	-1.87	-1.87	-1.87
	5-yr	3.28	-1.82	-1.82	-1.82	-1.82	-1.82
	10-yr	3.44	-1.79	-1.79	-1.79	-1.79	-1.79

Not intended for construction.

6.4.3 Proposed Project Scenarios Selected for Further Evaluation

Scenario 5 was selected for further evaluation by The Design Team. Scenario 5 represents channel excavations with berms in select locations, increased culvert sizing, and an increase in pump station capacity at the existing tidal gate. This scenario represented a combination of the most realistic flood mitigation approach with the maximum WSEL reduction and was carried forward for further evaluation. Water surface profiles of existing conditions as compared to proposed conditions Scenario 5 were compared in **Figure B6-38**.

6.4.4 Proposed Project Scenarios Eliminated from Further Evaluation

The remaining Scenarios 1, 2, 3, and 4 were not carried forward for further assessment. The methodology followed incorporated additional improvements until the reduced WSELs, especially within the flood prone areas, was negligible. Scenarios 1, 2, 3, and 5 represented incremental aggregations of flood mitigation approaches within DEP. Scenario 4 included an unrealistic pumping capacity, testing the upper bound of WSEL reduction. Scenario 3 included channel dredging, increased culvert capacity, and increased pump capacity, which produced a lower WSEL than Scenario 5, which was the same as Scenario 3 with an included berm. The berm increased in-stream WSELs, while providing additional out of bank protection, and was selected to be further evaluated.

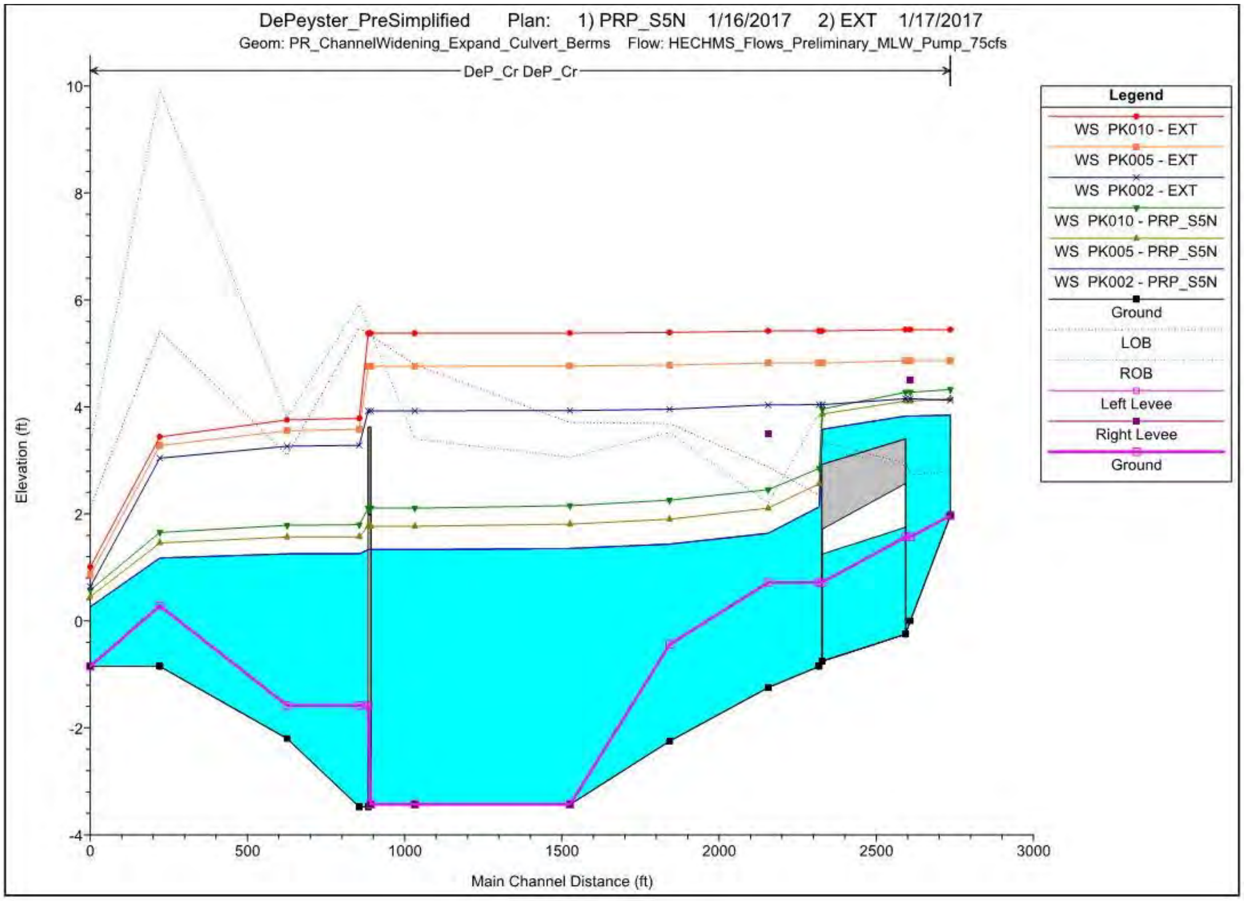

Figure B6-38: DEP WSEL Comparison between Existing and Selected Proposed Scenarios

Not intended for construction.

7.0 Summary

7.1 Model Results

This subappendix describes preliminary hydraulic modeling and results that were used to screen 30 concepts. The simplified modeling have significant sources of uncertainty, including: details of submerged structures, simplified pumping simulation, limitations of channel survey, uncalibrated steady state flow and stage, static tidal conditions, no simulation of breakout flows between drainage areas, and simplified overland flow. Submerged structure bridge and culvert geometry, type, and, in some cases, locations were assumed. Other studies and models were used as available to supplement survey data.

Based on the simplified hydraulic model results, a total of 20 concepts (**Table B6-32**) were selected by The Design Team for further evaluation. The detailed modeling in Phase 2 will provide further feasibility analysis to screen the alternatives. Flood risk reduction benefits, cost opinions, and other pertinent information will be developed during the feasibility phase.

Table B6-32: Selected Concepts

Drainage Improvement Area	Concept
West Riser Ditch	
D	Excavate channel
D	Increase culvert size
E	Excavate channel
E	Increase culvert size
F	Excavate channel
F	Increase culvert size
G	Pump station at tide gate
East Riser Ditch	
A	Excavate channel
A	Increase culvert size
A	Pump station at Green Street
A	Diversion to Main Street
B	Excavate channel
C	Excavate channel
C	Pump station at tide gate
Peach Island Creek	
K	No concepts for Phase 2
Losen Slote	
I	Local pump station discharging into Lower Losen Slote
O	Excavate channel
DePeyster Creek	
N	Excavate channel

Drainage Improvement Area	Concept
N	Increase capacity of existing pump station
N	Increase culvert size
N	Berms

8.0 References

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Subappendix B7: HEC-RAS Detailed Modeling Summary

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Table of Contents

List of Tables

List of Figures

Acronyms and Abbreviations

1D	One-dimensional
2D	Two-dimensional
cfs	Cubic feet per second
DEM	Digital elevation model
GIS	Geographic Information System
HEC-FDA	Hydrologic Engineering Center - Flood Damage Reduction Analysis
HEC-HMS	Hydrologic Engineering Center - Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center - River Analysis System
LiDAR	Light Detection and Ranging
NJDEP	New Jersey Department of Environmental Protection
SLR	Sea level rise
WSEL	Water surface elevation

1.0 Project Objective

The purpose of this subappendix is to provide feasibility-level modeling and civil planning and design information needed to develop a defensible benefit to cost ratio for two drainage improvement concepts. The concepts, if designed and built, are intended to lower water surface elevations (WSELs) in the East Riser Ditch and West Riser Ditch, and the adjacent and upstream drainage areas under normal tidal conditions, which are characterized as tidal fluctuations that vary between Mean High Water and Mean Low Water elevations.

1.1 Tools Used for the Economic Analysis and Data Required to Support these Analyses

Hydrologic Engineering Center - Flood Damage Reduction Analysis (HEC-FDA) will be used to perform benefit-cost analysis for the drainage improvement concepts. Performance of the conditions Without (i.e., existing conditions) and With (i.e., proposed conditions) the Proposed Project will be tested against six rainfall frequency events with a duration of 48 hours: 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year. Conditions will be under normal tidal conditions (2016 precipitation and 2075 precipitation), and during a 2-year coastal storm surge event (2016 precipitation and 2075 precipitation). The storm surge event is estimated to be the 2-year frequency storm. Hydrologic Engineering Center - River Analysis System (HEC-RAS) results will be provided in the form of maximum WSEL rasters, as well as stage and flow hydrographs, for each recurrence interval analyzed.

1.2 Model Use and Level of Accuracy

The HEC-RAS model will only be used to provide feasibility-level results for existing and proposed conditions, to be used in an HEC-FDA analysis[1]. Due to limited availability of hydraulic loading and system response data, it is not possible to determine whether the models meet an accuracy requirement, even if one was defined. That leaves accuracy to be determined based on a close examination of model inputs and responses to the various loading conditions. If those inputs are judged to be consistent with the industry standard of care, and the model responses to be reasonable, the models are considered to meet the model accuracy requirements. However, results from one validation run are presented and indicate a favorable comparison between modeled and observed WSELs in the Project Area.

2.0 Modeling

2.1 Description of Construction of the Model

The two dimensional modeling domain boundary was developed by merging the East Riser Ditch and West Riser Ditch watershed boundaries that were developed by AECOM for use in their created Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) model. The model domain is shown in **Figure B7-1**. This boundary was not refined for use in the HEC-RAS modeling.

The model geometry uses hydraulic structure dimensions and elevations taken from previous HEC-RAS one-dimensional (1D) models developed by other HDR modelers. Portions of those models were developed using survey data, while other portions were based upon HEC-RAS models developed by other engineering firms. See **Appendix D** for extents of the newly surveyed portions of the Project Area, and those taken from prior modeling studies. The 1D cross sections in the downstream portion of the West Riser Ditch were developed for this modeling effort, and are based on the existing conditions terrain described in **Section 2.3**.

[1] That effort is being completed by AECOM and is not part of HDR's contract.

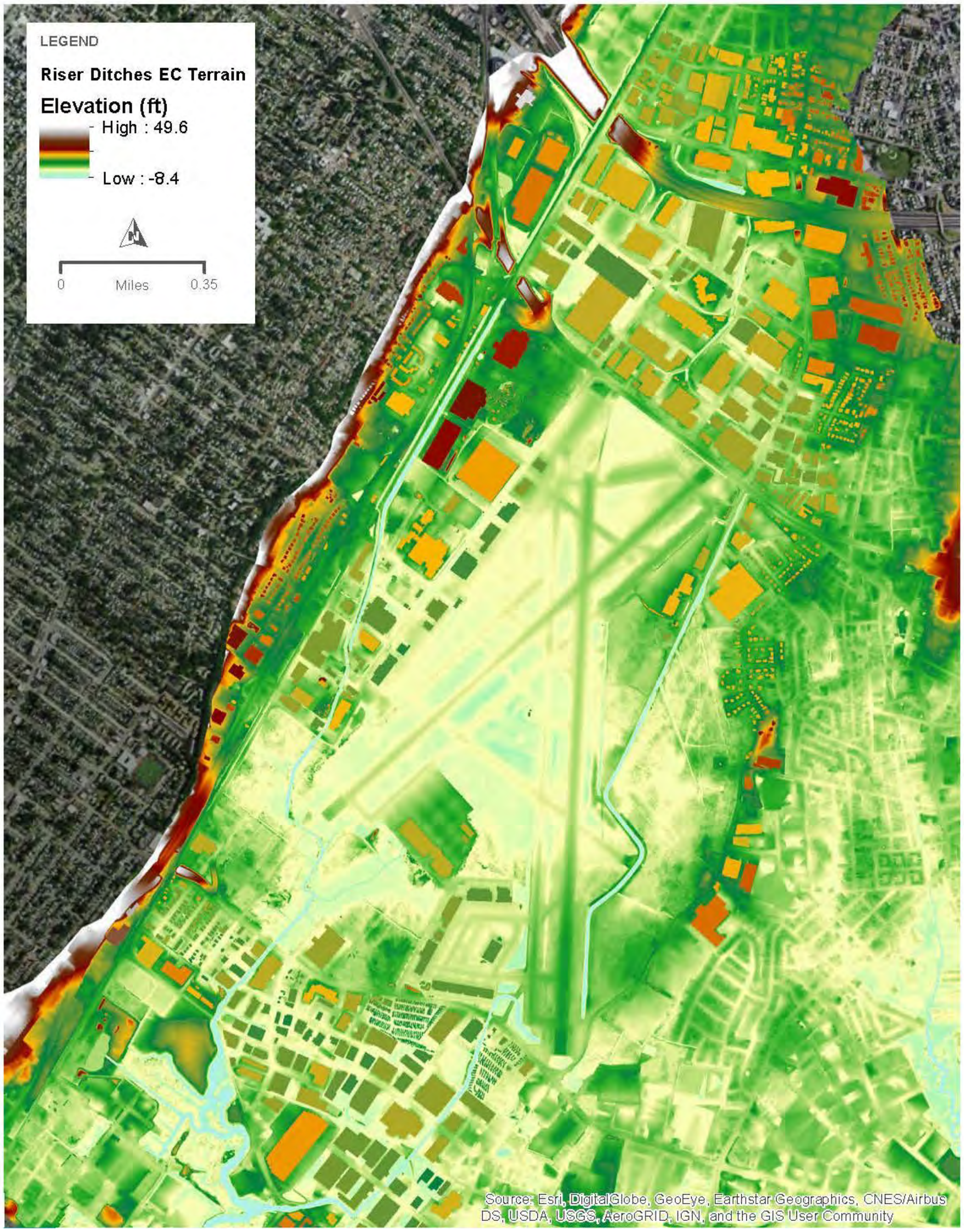


Figure B7-1: Terrain Model for Detailed HEC-RAS 2D Modeling

The model uses a 50 feet by 50 feet cell size for the two-dimensional (2D) flow area, with break-lines being added along building edges, along roadways, and along other important terrain features that would be likely to impact flow conditions. A relatively small grid cell size was chosen for desired level of detail; no grid cell sensitivity was conducted.

At the edge of the model domain along the Berry's Creek estuary, the tidal stage time series provided by AECOM is specified as the model boundary condition. At any inland locations where runoff reaches the model domain, flow is not allowed to leave the domain. This assumes no interaction with the regions outside the model domain, and may result in elevated WSELs in these areas.

Land use coverage was based on data acquired from the New Jersey Department of Environmental Protection (NJDEP), with roadways being added based on local parcel data and some refinements being made in the open channel regions. Several Manning's n roughness value override polygons were added to the domain along steep roadways, with a higher n-value assigned to these areas in order to reduce model instabilities in these areas. The roughness values used are shown in **Table B7-1**.

Table B7-1: Land Use Data

Land Use Type	Manning's n Roughness Value
airport facilities	0.015
altered lands	0.025
artificial lakes	0.03
artificial lakes	0.035
athletic fields (schools)	0.035
bridge over water	0.015
cemetery	0.025
commercial/services	0.15
deciduous brush/shrubland	0.08
deciduous brush/shrubland	0.06
deciduous forest (>50% crown closure)	0.08
deciduous forest (10-50% crown closure)	0.08
deciduous scrub/shrub wetlands	0.08
deciduous wooded wetlands	0.08
disturbed wetlands (modified)	0.08
disturbed wetlands (modified)	0.055
disturbed wetlands (modified)	0.045
disturbed wetlands (modified)	0.03
herbaceous wetlands	0.04
industrial	0.15
managed wetland in maintained lawn greenspace	0.045

Land Use Type	Manning's n Roughness Value
mixed deciduous/coniferous brush/shrubland	0.08
mixed urban or built-up land	0.12
mixed urban or built-up land	0.045
natural lakes	0.03
old field (<25% brush covered)	0.045
other urban or built-up land	0.04
other urban or built-up land	0.035
other urban or built-up land	0.03
other urban or built-up land	0.025
parking lot	0.018
phragmites dominate coastal wetlands	0.035
phragmites dominate coastal wetlands	0.025
phragmites dominate interior wetlands	0.035
phragmites dominate interior wetlands	0.055
phragmites dominate interior wetlands	0.08
phragmites dominate old field	0.055
railroads	0.025
recreational land	0.04
residential, high density or multiple dwelling	0.14
residential, high density or multiple dwelling	0.18
residential, single unit, low density	0.12
residential, single unit, medium density	0.12
residential, single unit, medium density	0.1
road	0.015
saline marsh (high marsh)	0.03
stadium, theaters, cultural centers and zoos	0.025
streams and canals	0.03
tidal rivers, inland bays, and other tidal waters	0.03
tidal rivers, inland bays, and other tidal waters	0.035
transitional areas	0.04

Land Use Type	Manning's n Roughness Value
transitional areas	0.03
transitional areas	0.035
transportation/communication/utilities	0.03

2.2 Plans Being Run

A total of 24 existing conditions plans will be run. The precipitation input consists of six rainfall frequency events with a single duration of 48 hours (2-year, 5-year, 10-year, 25-year, 50-year, and 100-year) based on 2016 precipitation and the same recurrence interval events based on 2075 precipitation. Each set of precipitation data will be analyzed with two sets of downstream tidal boundary stage hydrographs, one for "normal" non-storm conditions, and one for storm surge conditions. The 2075 tidal condition assumes a 1.2 feet sea level rise (SLR) above the 2016 levels. The storm surge boundary condition was developed by an AECOM team using a MIKE 21 model to simulate the impact of a 2-year coastal storm upon WSEL within the Hackensack River. **Appendix C** contains a summary of all existing and proposed condition runs.

2.3 Terrain Development

The overbank areas of the model domain are based upon Light Detection and Ranging (LiDAR) data provided by AECOM. This data was collected in April 2014 by Quantum Spatial. The buildings were removed from the surface when it was provided by the LiDAR contractor.

A building footprints shapefile was provided to HDR by the local county Geographic Information System (GIS) department. The buildings were assigned a high Manning's n-value of 200 to act as blocked obstructions in the model.

The West Riser Ditch and East Riser Ditch open channels were represented by creating interpolation surfaces based on 1D HEC-RAS models provided by the HDR 1D HEC-RAS modeling team, and integrating survey data where available.

The terrain in the area of culvert and bridges inlets and outlets was adjusted to match or be slightly lower than the structure invert in order to allow HEC-RAS to run successfully.

3.0 Model Data Sources

3.1 Terrain

As mentioned in **Section 2.3**, the overbank areas of the model domain are based upon LiDAR data provided by AECOM and collected in April 2014 by Quantum Spatial. The digital elevation model (DEM) was provided to the HEC-RAS 2D modeling team as a bare-earth geotiff, with buildings removed from the terrain. This DEM uses a 2 feet pixel size. The LiDAR report of survey from Quantum Spatial states that one of the products that was produced along with the DEM was a set of 1-foot contours. It is assumed that the DEM is accurate to that same level. Other layers used in creating the final terrain are described in **Section 2.3**.

3.2 Inflows

The HEC-RAS 2D model does not incorporate any inflow hydrographs. Rainfall excess hyetographs derived from the HEC-HMS model are applied at each grid cell in the model. Runoff is then determined hydrodynamically by the HEC-RAS model.

3.3 Documentation of Historic Flooding Conditions and Availability of Rainfall and Observed Water Surface Elevation, Flow and Velocity Data

Anecdotal flooding WSEL data was available, as summarized in **Appendix C**. Photographs from historical flooding events were used to approximate flooding WSELs in the Project Area. The type and distribution of data needed to properly calibrate the HEC-RAS model was not available for this study.

Validation of the model was accomplished in the form of comparing model WSELs during one flooding event at four locations with WSELs interpreted from photographs and the GIS terrain model of the Project Area. That comparison is presented in **Figure B7-2** and **Figure B7-3**.

A second flooding event occurring under normal tidal conditions was identified as a candidate for model validation, but there was a low level of confidence in the system response data (i.e., flow 'level' data in the East Riser Ditch). Since the modeling team was unable to obtain a confirmation of whether the 'level' data represented depth or WSEL, that candidate validation event was dismissed from further consideration.

3.4 Precipitation

Precipitation excess hyetographs used to represent the various storm events simulated were developed in HEC-HMS by AECOM and provided to HDR. Because the HEC-RAS model uses only one 2D flow area encompassing several of the HEC-HMS sub-basins, the HEC-HMS sub-basin which produced the highest peak precipitation excess was used as the input for the entire HEC-RAS model 2D Area domain.

3.5 2D Area Boundaries

The 2D Area domain boundary was developed by merging the East Riser Ditch and West Riser Ditch watershed boundaries developed by AECOM for use in their created HEC-HMS model. This boundary was refined at the downstream end to include Peach Island Creek and the "line of defense" areas for use in the HEC-RAS modeling. The goal of the boundary revision was to utilize tidal stage time series as downstream boundary conditions.

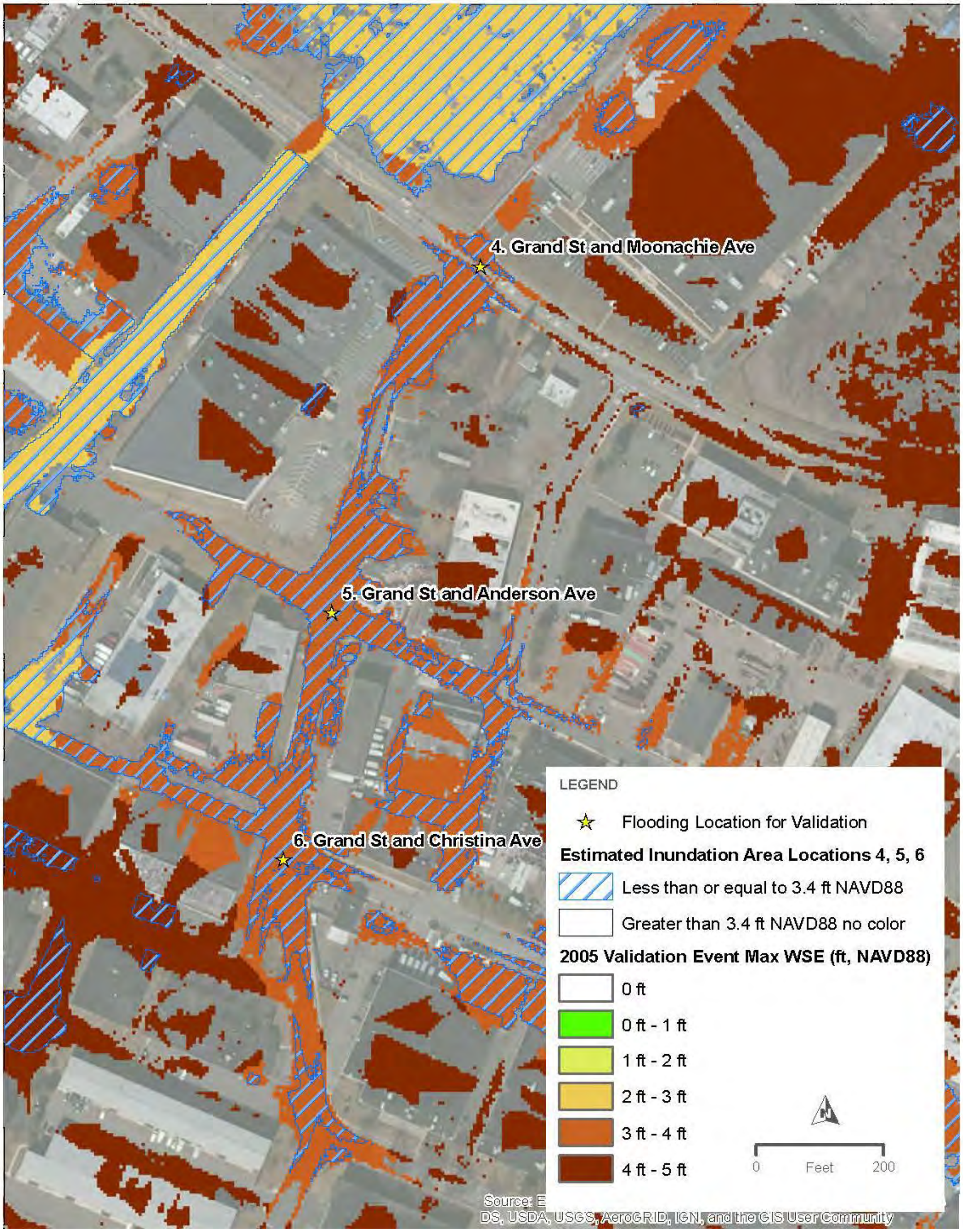
1) 

Figure B7-2: HEC-RAS 2D Validation, Comparison of Maximum Water Surface Elevations Predicted by Model with those Estimated from Photographs and LIDAR Analysis, October 13-14, 2005 Event

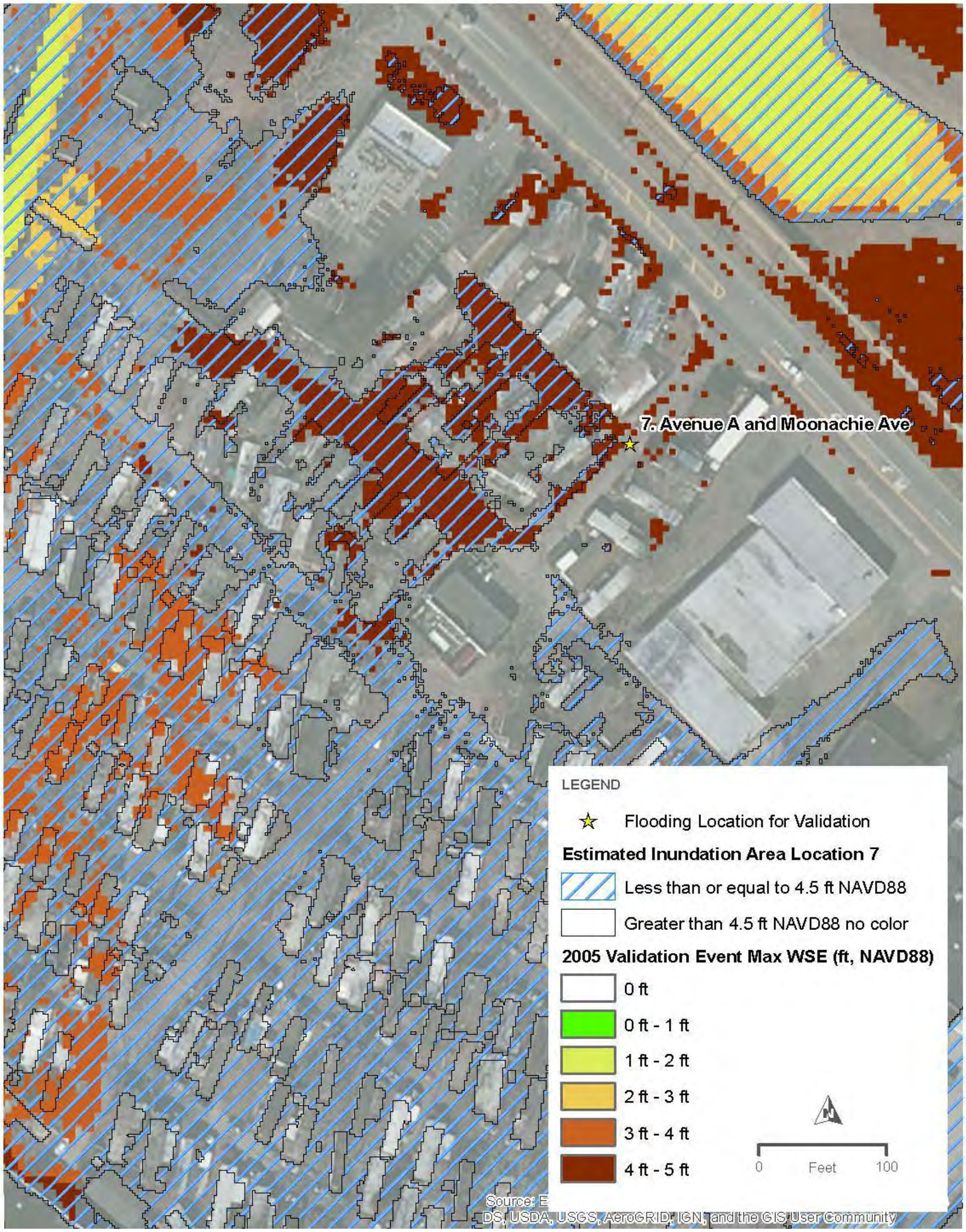


Figure B7-3: HEC-RAS 2D Validation, Comparison of Maximum Water Surface Elevations Predicted by Model with those Estimated from Photographs and LIDAR Analysis, October 13-14, 2005 Event

3.6 Boundary Conditions

Tidal boundary condition stage hydrographs were provided to HDR by AECOM. These stage hydrographs were assigned to boundary condition lines created along the open channels downstream of the East Riser Ditch and West Riser Ditch tide gates.

Four tidal time series were analyzed:

1. 2016 "normal", which represents no SLR or storm surge impact on tidal elevations;

2. 2075 "normal", which assumes 1.2 feet of SLR above 2016 tidal elevations, with no storm surge impact on tidal elevations;

3. 2016 "surge", which represents no SLR, but does include the storm surge impact on tidal elevations of a 2-year coastal storm event; and

4. 2075 "normal", which assumes 1.2 feet of SLR above 2016 tidal elevations, and includes the storm surge impact on tidal elevations of a 2-year coastal storm event.

3.7 Pumps

Two versions of the existing West Riser Ditch and East Riser Ditch model were developed: one including an existing pump station at Teterboro Airport, and one without it. This facility is located near the southwestern edge of the Teterboro Airport and is known as the Vincent Street pump station. The Vincent Street facility pumps from the Airport Ditch and discharges directly into the West Riser Ditch. The maximum discharge capacity of this facility is 175 cubic feet per second (cfs), but is limited to 112 cfs by local decree, as discharges higher than 112 cfs have caused flooding along the West Riser Ditch below the pump station. The pump in the model pumps a maximum of 102 cfs based on direction from an HDR pump specialist. The pump station wet well dimensions and pump operating characteristics were extracted from plans and summarized by Stephen McKelvie, and provided in spreadsheet format to the HEC-RAS 2D modeling team.

There is also an existing pump station at Huyler Street, which is not represented in the model.

The model containing the Vincent Street pump station did not meet the 1 percent continuity threshold deemed by the modeling team to be necessary for model output use on the project by the Benefit-Cost Analysis team. Numerous refinements were attempted, some of which improved model continuity, but none of which lowered model continuity to the noted level. The likely cause of the continuity errors were model instabilities associated with the inclusion of the pump station and associated stream reaches as 1D model elements. As a result, a primarily 2D model that excluded both the Vincent Street and Huyler Street pump stations was utilized and a separate set of evaluations to assess what effect those exclusions might have on the end use of the model data was preformed.

Results from the original combined 1D/2D HEC-RAS model that included the Vincent Street pump station were compared to results from the 2D model that does not include the pump station. Peak discharge in the West Riser Ditch channel immediately downstream of the Vincent Street pump station outfall was found to be significantly higher in the model which included the pump station, but WSELs were quite similar. See **Table B7-2** for a comparison of peak WSEL and discharge for 2016 existing conditions, normal tide runs. It was concluded that WSELs in the West Riser Ditch system were not significantly sensitive to the increased flows associated with the Vincent Street pump station, and the decision was made to utilize the 2D model which excludes that component.

Table B7-2: Comparison of Existing Condition Results WSEL and Flows in West Riser Ditch Downstream of Vincent Street Pump Station

Plan	WSEL in 1D-2D model (feet)	WSEL in 2D Model (feet)	Peak Discharge in 1D/2D Model (cfs)	Peak Discharge in 2D Model (cfs)
2016 Normal Tide 2 year	4.01	4.08	143.89	52.35
2016 Normal Tide 5 year	4.4	4.45	190.88	86.2
2016 Normal Tide 10 year	4.69	4.64	248.12	131.7
2016 Normal Tide 25 year	5.01	4.82	301.82	140.5
2016 Normal Tide 50 year	5.21	5	330.47	182
2016 Normal Tide 100 year	5.37	5.11	368	220

The Huyler Street pump station was not simulated in an HEC-RAS model due to the inability of HEC-RAS to represent the complex system of storm drains delivering runoff to the pump station. In addition, very little information about these storm drains is available to inform a detailed hydraulic model. A hydrologic model of the approximate area that contributes flow to the Huyler Street pump station was constructed by AECOM using HEC-HMS. This model does include the Huyler Street pump station, utilizing simplifying assumptions about the storm drain network in order to produce approximate discharge rates from the pump station. The HEC-HMS model does not calculate stage, only discharge. A stage-discharge rating curve was developed from the HEC-RAS model for a cross section that includes the East Riser Ditch downstream of Route 46, and also includes the overland discharge that enters the Teterboro Airport from the area of the Huyler Street pump station. This is the location where outflow from the Huyler Street pump station would surface as open channel flow. The rating curve shows marked looping, indicating that more than one WSEL value can occur for a given flow rate, depending on the timing of the measurement during the storm event. This rating curve was used to extract an expected WSEL for the HEC-HMS peak discharge values, reporting the highest WSEL for a given discharge rate. This analysis was only performed for the 2-year through 10-year events, as the HEC-RAS peak discharge equals or exceeds the HEC-HMS flows for the larger magnitude events. For the 2- year through 10-year events, the HEC-HMS peak flows are very similar, resulting in a single WSEL estimate for all of these events from the HEC-RAS rating curve. Peak WSEL values were extracted from the HEC-RAS model at the East Riser Ditch channel just downstream of Route 46. The HEC-RAS results indicate very little difference between the peak WSEL for these lower magnitude events, and the estimated WSEL for the HEC-HMS peak discharges is within 0.1 foot of the peak WSE values extracted from the HEC-RAS model. The insensitivity of the HEC-RAS model to peak discharge indicates that the lack of the Huyler Street pump station in the HEC-RAS model is unlikely to result in a large variation from actual conditions that would occur during significant storm events.

It should also be noted that the purpose of this modeling effort was to provide relative WSEL impacts due to proposed improvements. Both the Vincent Street and Huyler Street pump stations were excluded from the existing conditions models as well, providing a reasonable comparative analysis.

4.0 Simplifying Model Assumptions

4.1 Storm Drains

It is assumed that the amount of water contained in the subsurface storm drainage systems is approximately the same for Without and With conditions. Because the HEC-RAS 2D model does not include the subsurface storm drainage system, it is viewed as approximately equivalent to a model whose subsurface system is full of water at the beginning of each storm event and, therefore, it is considered a reasonable approximation of a worst-case scenario regarding inundation conditions it predicts.

4.2 Geometric

Buildings were represented using elevated Manning's n roughness values in order to prevent overland flow from moving through the building footprint areas. Due to HEC-RAS 2D modeling constraints, any bridges within the 2D model area were represented using culverts. Upstream and downstream inverts of the culverts were matched back to the 1D models. Culvert rise was set to result in a match to the upstream low chord of the 1D models. Culvert span was adjusted to result in a close match to the open area seen for the bridge in the 1D model.

4.3 Pump Stations

It is assumed that the effects the Vincent Street and Huyler Street pump stations have on Without and With WSELs are approximately equivalent for a given storm event and, therefore, are of relatively little consequence regarding predicted reductions in flood damages.

4.4 Boundary Conditions

At the edge of the model domain along the Peach Island Creek estuary, the tidal stage time series provided by AECOM is specified as the model boundary condition. No other boundary condition lines were added to the model domain. Any flow that reaches the domain boundary in areas away from the Berry's Creek estuary will not be allowed to leave the system, but will collect against that edge.

4.5 Options and Tolerances

The Diffusion Wave equation set was utilized for this modeling effort, due to the fact there are not significant flow contraction or expansions taking place within the domain, and because velocities seen within the model results are relatively low. It was assumed that even during storm surge tidal events, there was very little flow which makes it over the line of defense barrier into the 2D modeling domain. This assumption allows for the use of the Diffusion Wave equation set.

4.6 Results

It is assumed that the relative differences between WSELs predicted by Without and With models is adequate for estimating reduction in flood damages.

4.7 Desired Modeling Products

HEC-RAS results will be provided in the form of maximum WSEL rasters, as well as stage and flow hydrographs, for each recurrence interval analyzed. The economic analysis team will use the HEC-RAS results to perform an HEC-FDA benefit-cost analysis for the drainage improvement concepts.

5.0 Conclusions

5.1 Description of the Results of the Modeling

Appendix C contains figures showing WSEL reductions for 25-year rainfall events: 2016 and 2075 normal tide, and 2016 and 2075 with a 2-year coastal storm surge. A compact disc containing all model output provided to the Benefit-Cost Analysis team is included with this subappendix.

5.2 Validation Run

The validation run simulates a rainstorm that took place from October 13 to 14, 2005. Precipitation excess hyetographs and tidal stage hydrographs for the October 2005 storm were provided to HDR by AECOM. Maximum WSELs at four locations were estimated using photographs taken during the event and LIDAR data from the flooded areas. See **Appendix C** for details. Only the date of the photographs were available. It was assumed that the photographs reflected the maximum WSELs that occurred at the noted locations during the event. **Figure B7-2** and **Figure B7-3** compare those water surface estimate values with maximum values recorded for that day of the model validation simulation run. In general, these comparisons show a good agreement at the comparison locations between the HEC-RAS validation model results and the inundation extents based on the estimated water surface elevations.

5.3 Level of Confidence in the Results

Volume continuity tracking is a parameter that is used to assess the level of error occurring within a given HEC-RAS model. Volume error within the model results was generally low, with all model plans reporting continuity error less than 0.5 percent.

Model stability varies throughout the domain. Some structures within the model show rapid variation in flow, indicating some level of model instability, but stage hydrographs for these structures are generally stable. Many of the instability issues seen in the model results are due to very shallow overland flow resulting from precipitation being applied to a very low slope terrain. Examination of model results in RAS Mapper indicates that flow direction in some reaches of the East Riser Ditch and West Riser Ditch varies rapidly from downstream to upstream, depending on overland runoff patterns. These conditions are realistic, given the extremely low slopes seen in these drainage channels. Numerous efforts were made to reduce the model instabilities seen, but not all instabilities could be eliminated.

The hydraulic models used for this analysis do not include sub-surface features, such as ancillary storm drains and internal pump stations. The base topographic data used was based on LiDAR data, with only limited channel bathymetry available to supplement the channel regions of the terrain. These models are intended to produce results that would represent the impacts of the Proposed Project features being simulated. Due to the limitations of the software used and the data available, model results may vary from the actual stage and flow conditions that would occur within the model domain during the events being simulated, particularly in the overbank regions. However, the comparison of the Without and With results does provide a reasonable approximation of the change in WSEL that would occur within the model domain, given the specified Proposed Project features.

The final design phase of the East Riser Ditch and West Riser Ditch elements will include development of hydrologic and hydraulic model(s) that use software capable of simulating both open channel and subsurface pipe flow.

Subappendix B8: InfoWorks Modeling Data, Methods, and Results

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Table of Contents

List of Tables

List of Figures

Acronyms and Abbreviations

1D	One-dimensional
2D	Two-dimensional
BCA	Benefit-cost analysis
C1	Carlstadt Area 1
CB	Catch basin
GIS	Geographic Information System
GPS	Global Positioning System
HEC-FDA	Hydrologic Engineering Center – Flood Damage Reduction Analysis
HEC-RAS	Hydrologic Engineering Center - River Analysis System
ICM	Integrated Catchment Modeling
M1	Moonachie Area 1
M2	Moonachie Area 2
MH	Manhole
NJDEP	New Jersey Department of Environmental Protection
NJSEA	New Jersey Sports and Exposition Authority
RBDM	Rebuild by Design Meadowlands
TIN	Triangular irregular network
USDA	United States Department of Agriculture
WSEL	Water surface elevation

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1.0 Proposed Project Background

The New Jersey Department of Environmental Protection (NJDEP), as the Proposed Project sponsor, plans to complete Preliminary Planning and Design for the Rebuild by Design Meadowlands (RDDM) Flood Protection Project. The Project Area includes the Boroughs of Little Ferry, Moonachie, Carlstadt, Teterboro, and the Township of South Hackensack in Bergen County, New Jersey.

As part of Alternative 2 (Fluvial/Local Drainage Protection), this technical memorandum documents the assumptions and methods used in development for the simplified and detailed hydrologic and hydraulic models. InfoWorks Integrated Catchment Modeling (ICM) simplified models ("simplified models") are constructed for four sub-watersheds within the Project Area: Moonachie Area 1 (M1), Moonachie Area 2 (M2), Carlstadt Area 1 (C1), and Main Street. As the second phase of the InfoWorks modeling task, detailed ICM models ("detailed models") are constructed for four sub-watersheds within the Project Area: Main Street including Indian Lake (Main Street), Losen Slote Concept D (Concept D), Losen Slote – Carol Place (Carol Place), and DePeyster Creek (DePeyster). The intent of this subappendix is to demonstrate if and to what extent proposed options for stormwater infrastructure, including dredging/channelization, off-channel storage, and pumping, would increase hydraulic capacity (and decrease water surface elevations (WSEL)).

2.0 Simplified and Detailed Modeling

Simplified models were developed for rapid prototyping of the simplified modeling framework for the selected model areas as the expedited approach for preliminary screening. Detailed modeling required expanded parameterization, and survey data implementation of the sewer system and stream bathymetry. Specifics of parameterization and survey details are described in the later sections of this report. Detailed modeling was performed in five selected sub-watersheds. Drainage area of both simplified and detailed models is shown in the **Figure B8-1** and **Figure B8-2**, respectively. Some of the models have overlapping drainage areas due to dynamic nature of the boundary where flooding level determines the direction of the flow.

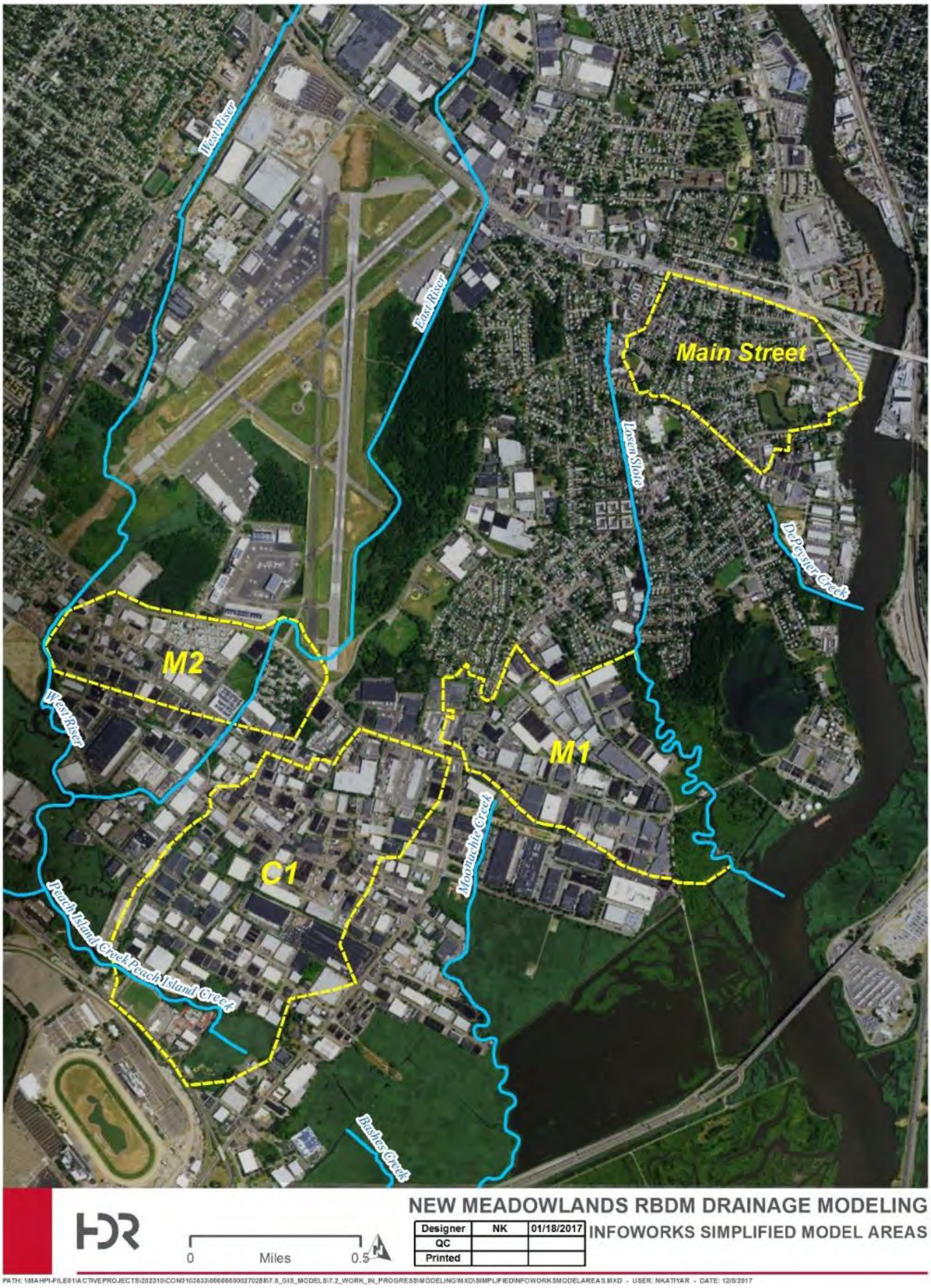


Figure B8-1: Simplified Model Drainage Areas

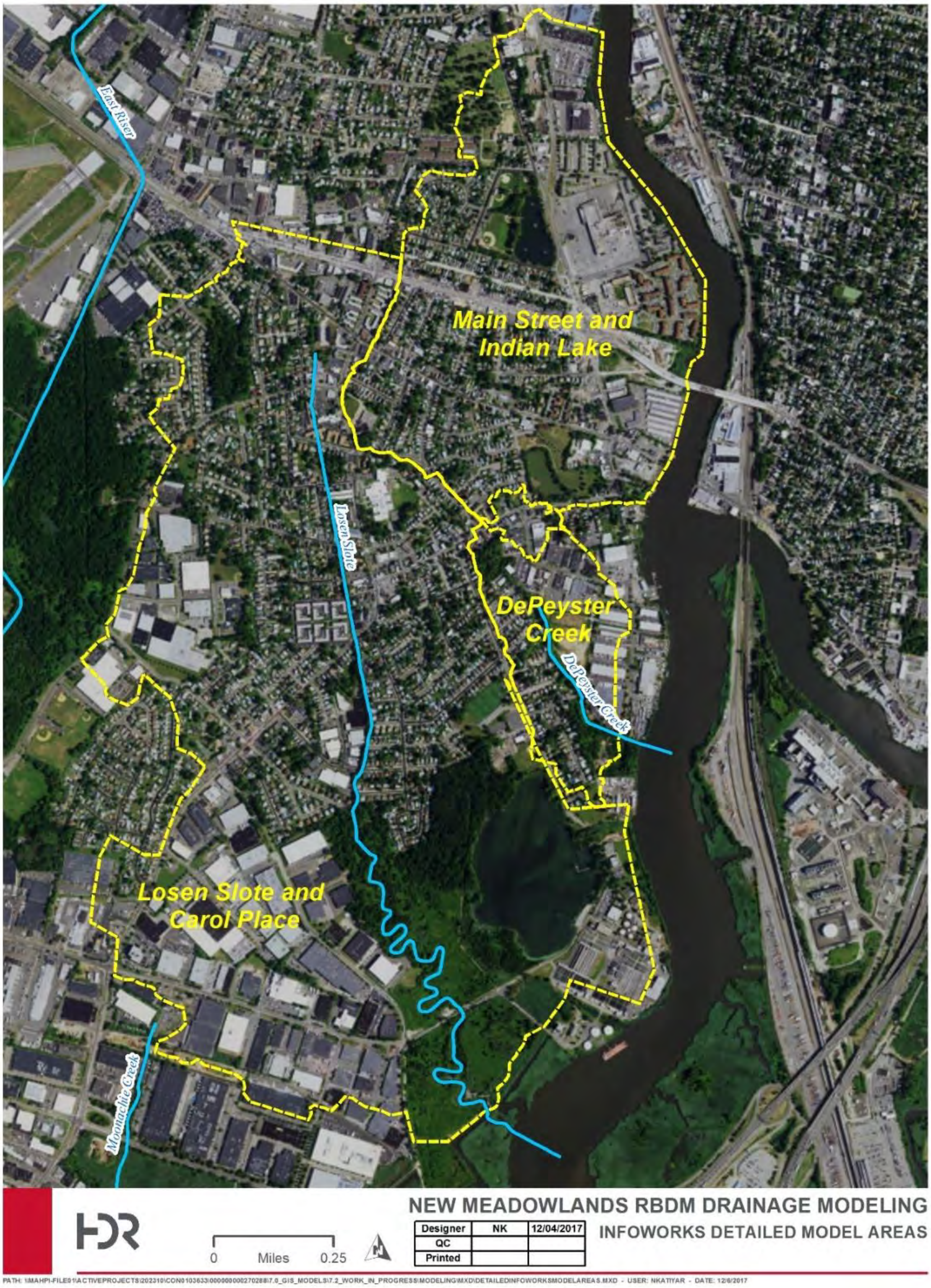


Figure B8-2: Detailed Model Drainage Areas

3.0 Anecdotal Evidence of Flooding

In the absence of flow monitoring data in conduits and ditches, anecdotal information regarding flooding location and severity is collected for the Project Area.Data is obtained from various sources, such as site visits performed during storm events, anecdotal flooding hotspot information input from community representatives, published reports, and flooding reported in newspapers and local television news channel. Details of these data sources and information collected is documented in **Appendix A** and **Appendix C**. This collected information was utilized at both model development and verification of the completed models. An example of collected information is shown in **Figure B8-3** where location of past flooding incidents reported in the New Jersey Sports and Exposition Authority (NJSEA) Hackensack Meadowlands Floodplain Management Plan within and in the vicinity of the Simplified C1 model area (NJSEA 2005). In **Figure B8-3**, the Simplified C1 model area is bounded by red line, with past flood incident locations highlighted with purple polygons.

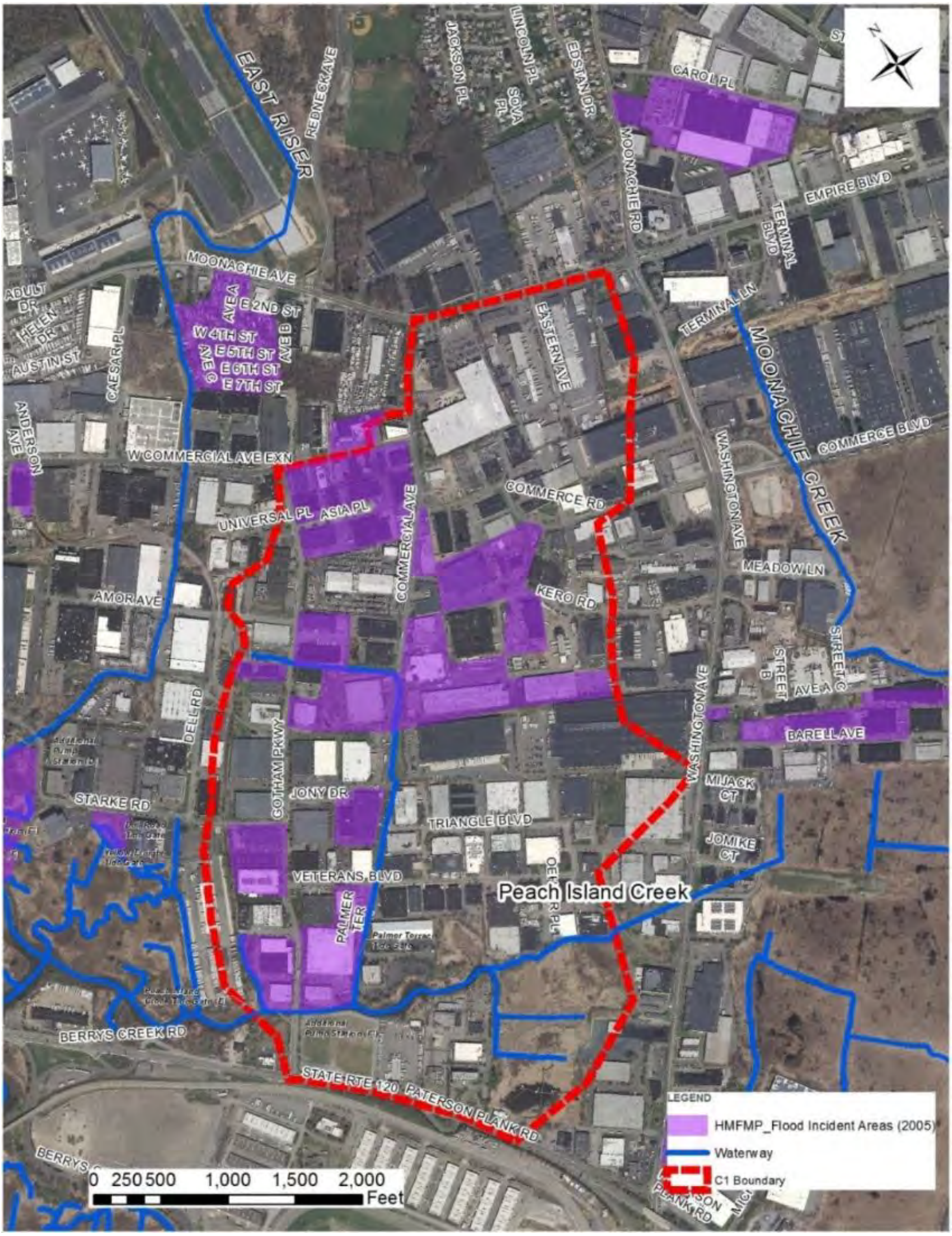


Figure B8-3: Simplified Model Area C1

4.0 Model Development

4.1 Modeling Software Selection

A mathematical model of various watersheds in the Project Area can assist in characterizing the land surfaces' responses to rain events, and simulating the consequent peak and volumetric flows that cause flooding. Routing of flows through stream channels (and floodplain) with hydraulic structures such as conduits, pumps, and bridges can be well captured in a model to account for potential restrictions and allowed conveyance. High tidal boundary condition can not only detain the fluvial runoff in the local conveyance system, such as conduits and ditches, but also can infiltrate in this confined watershed if the tide water level is high enough. Modeling for such conditions requires complex flow and volume interactions between overland and conduit flows with the dynamic tidal boundary conditions. Due to complexity of the existing condition, modeling components, and the Proposed Project's alternatives, the modeling software selected for this analysis is InfoWorks ICM 7.5.

5.0 Hydrologic and Hydraulic Parameters

5.1 Subcatchment Delineation

The runoff is produced on subcatchments and is assigned to one-dimensional (1D) model domain directly. The 1D modeled conduits and ditches surcharge when the capacity is reached and two-dimensional (2D) system starts to route this runoff volume. Subcatchment delineation and resolution extent is determined by three primary data sources: (1) manholes / catch-basins location, (2) anecdotal flooding locations, and (3) the Proposed Project alternative.

5.2 Mesh Domain

The extent of 2D triangular irregular network (TIN) mesh domain includes areas where subcatchments are created, and the domain is extended to include neighboring areas where inflow to the sub-watershed can potentially occur during high water surface level conditions from other sub-watersheds and waterbodies. Rainfall outside the subcatchment domain is directly assigned to the mesh.

5.3 Infiltration

The Horton infiltration method and corresponding parameters are assigned to pervious area components in subcatchments and in 2D mesh simulation polygons. Using a digitized impervious cover shapefile developed for this modeling, pervious and impervious polygons in the model drainage area were separated. Furthermore, we used United States Department of Agriculture's (USDA's) soil vector data layer to assign a hydrologic soil group category to various sub-polygons. Horton infiltration rate parameterization is based on these hydrologic group categories (as listed in **Table B8-1**). Four surfaces, one for each hydrologic group (HSG-A, HSB-B, HSG-C, and HSGD), and four corresponding 2D surface identifications (IDs) are assigned in the model networks.

Table B8-1: Horton Infiltration Rate Parameters

Minimum Infiltration Rate, f_c	(Musgrave, 1955)
Hydrologic Soil Group	**fc (in/hr)**
A	0.45 – 0.30
B	0.30 – 0.15
C	0.15 – 0.05
D	0.05 – 0

Maximum/Initial Infiltration Rate, f_0	(Reference)
a.	DRY soils (with little or no vegetation): Sandy soils: 5 in/hr Loam soils: 3 in/hr Clay soils: 1 in/hr
b.	DRY soils (with dense vegetation): Multiply values given in a. by 2
c.	MOIST soils Soils which have drained but not dried out (i.e., field capacity): divide values from a and b by 3. Soils close to saturation: choose value close to min. infiltration rate. Soils which have partially dried out: divide values from a and b by 1.5 - 2.5.

Decay Rate and Drying Time (Reference)	
Decay Const.	Infiltration rate decay constant for the Horton curve (1/hours). Typical values range between 2 and 7.
Drying Time	Time in days for a fully saturated soil to dry completely. Typical values range from 2 to 14 days.

5.4 Manhole / Catch-Basin Connectivity

Survey data for manhole (MH) and catch-basin (CB) is imported from a geographic information system (GIS) database created using data from the field survey performed. Details of the field survey are provided in **Appendix D**. Naming convention of the nodes is based on unique Survey IDs assigned in the GIS database. Modeled nodes include two types of features: (1) where the detailed structure survey is performed, and (2) the location, identified from other datasets, such as from a global positioning system (GPS) survey and NJSEA GIS data. While incorporating nodes data, sanitary MHs and CBs from both surveyed and previously identified nodes were removed based on survey results and other available site specific information. Other relevant parameterization of the nodes is summarized below:

- Structures flags indicate the nodes' data source, as specified below:
  - SV – Surveyed and incorporated as recorded
  - IF – Inferred using other sources
  - DR - From drawings
  - ES – Estimated
- Ground/Rim elevation is inferred from TIN mesh, unless elevation is applied using values obtained during detailed survey.
- Only CBs are implemented as flood type 2D with the appropriate opening and flooding discharge coefficients (0.50).

- MHs are modeled as sealed 1D structures. As observed during limited wet-weather field visits that were performed, none of the MHs were open during or after a rain event caused by system surcharged.
- Sediment levels are applied at all connecting pipes in the model as recorded during the detailed surveys. If ponded water is observed in the CB or MH during dry weather period, the sediment depth is estimated to be equal to the observed water depth above invert level. High water depth is indicator of blocked pipe, either at the CB/MH or at the downstream conduit, caused by stagnant water backing-up in the upstream sewer. If during a survey the sediment levels in any particular structure is found to be 100 percent of the conduit's cross-section, then 80 percent of the cross-sectional depth is used as sediment levels in those pipes since InfoWorks does not allow 100 percent blocked conduits.
- The CB inlet area (shaft area) used for CBs is 3.39 square feet, as calculated for New York City type A (typical) CBs. This number can be modified when information is received. Roof elevation is assigned to be equal to ground elevation at the node location.
- The chamber area default value used for MHs and CBs is 5 by 5 square feet.

5.5 Conduit Parameters

Sediment information from the surveyed manholes is expanded over the entire model area. Median sediment level is calculated as a percentage obstruction of the cross-section area and assigned to the remaining conduits in the model area. The Manning's roughness n-value range, with initial values in parenthesis, is provided in the **Table B8-2**.

Table B8-2: Pipe Material Manning's Roughness

Pipe Material	N-value
Corrugated metal (default)	0.019 – 0.032 (0.025)
Concrete pipe	0.011 – 0.013 (0.013)
Cast Iron	0.012 – 0.014 (0.013)
Brick	0.014 – 0.017 (0.015)
Clay	0.012 – 0.014 (0.013)

5.6 Open Channel Parameters

Open channels in the model are implemented as a 1D cross-section in conjunction with any structural information surveyed and/or recorded by Hydrologic Engineering Center - River Analysis System (HEC-RAS) 1D and 2D teams. The banks of these channels act as a 1D/2D boundary and define 2D mesh floodplains. The range of Manning's roughness n-values of the channel are tabulated in **Table B8-3**, including the initial value assigned in the parenthesis (Chow 1959).

Table B8-3: Open Channel Manning's Roughness

Open Channel Type	N-value
Concrete unfinished	0.014 – 0.020 (0.017)
Excavated or dredged channels - Earth, straight and uniform	0.016 – 0.033 (0.022)
Excavated or dredged channels - Overgrown with vegetation	0.040 – 0.140 (0.080)
Natural streams – Flood plains, pasture	0.025 – 0.050 (0.033

5.7 Sluice Gates / Weir Discharge Coefficients

InfoWorks model's discharge coefficients are slightly different than those from the weir or orifice equation because they already include a conversion factor. Coefficient values used in the model are:

- Weir first coefficient = 0.49, second coefficient =0.92; and
- Orifice first coefficient = 0.92, second coefficient = 0.49.

Table B8-4 summarizes the simplified model components. Since simplified models are developed for preliminary screening only, majority of parameters for nodes, conduits, ditches, and subcatchments are based on initial values, and no adjustment is performed on model components during model verification process. Detailed models, however, are developed for benefit-cost analysis (BCA) WSEL reduction assessments of the Proposed Project's alternatives in the four abovementioned model areas. Due to the overlapping boundary for the two detailed model areas, the model extent for Losen Slote Concept D (Concept D) and Losen Slote – Carol Place (Carol Place) is same. This results in effectively three existing condition detailed modeling domains and corresponding components, as listed in **Table B8-5**.

Table B8-4: Simplified Model Components

Model Component	Main Street	M1	M2	C1
Manhole / Catch Basins	100 Structures	117 Structures	154 Structures	275 Structures
Link	96 Conduits	113 Conduits 7 Open channels	152 conduits	226 Conduits 62 Open channels
Simulation Zone	205 acres	327 acres	186 acres	567 acres


Main Street


M1


M2


C1

Table B8-5: Detailed Model Components

Model Component	Losen Slote and Carol Place	Main Street and Indian Lake	DePeyster Creek
Manhole / Catch Basins	741 Structures	491 Structures	2 Structures
Link	738 Conduits 29 Open channels	491 Conduits 5 Open channels	2 Conduits 7 Open channels
Simulation Zone	843 acres	331 acres	82 acres


Losen Slote and Carol Place


Main Street and Indian Lake


DePeyster Creek

6.0 Model Verification

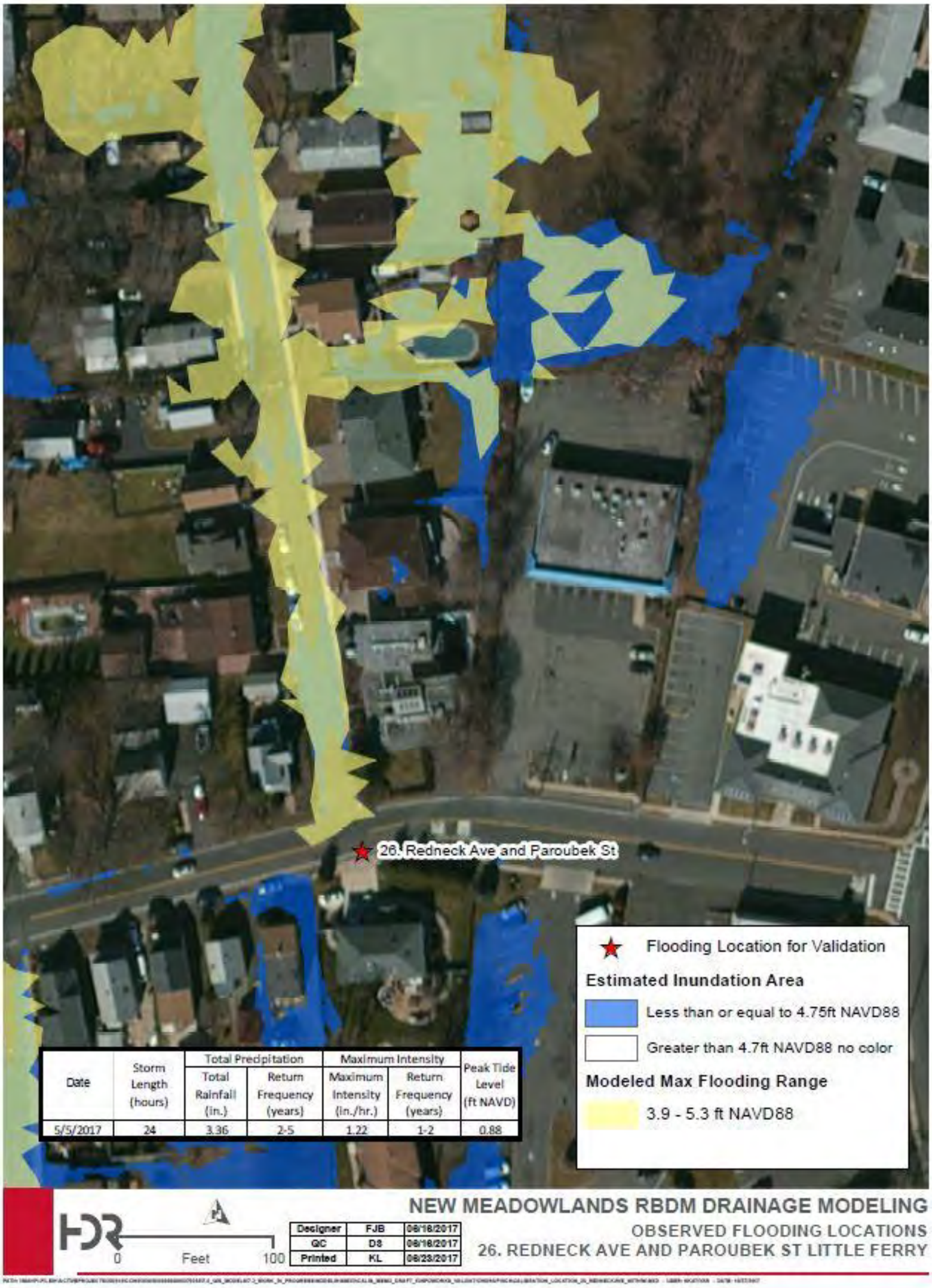
In the absence of calibration data, model results are verified using data from **Appendix C** and **Appendix A** against a model simulation with actual, observed storm event and tidal boundary conditions. Limited verification is performed for selected models at both simplified and detailed modeling stages. Example of one such verification process is shown in **Figure B8-4** to **Figure B8-7**. The observed flooding location and the photograph(s) is recorded and converted into the GIS flooding extent for verification. This flooding extent is based on masking the elevations below estimated flooded WSEL from the photograph. This estimated WSEL is highlighted with blue color in **Figure B8-5** to **Figure B8-7**. Model results corresponding to the same May 5, 2017 rain events and tidal boundary conditions are also shown on **Figure B8-5** to **Figure B8-7**. The model results WSELs are shows as ranges because the flood polygons in each figure contain disconnected pockets of flooding extents, each having different WSELs. In some cases, the model verification process resulted in model network adjustment, primarily changing the mesh and void orientations and a parameters adjustment.

Figure B8-4: Field Observations Made During Rain Event on May 5, 2017

Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

Figure B8-5: Verification Graphic Comparing Modeled and Observed Flooding Extent at State Street Location

Figure B8-6: Verification Graphic Comparing Modeled and Observed Flooding Extent at Sabrina Street Location



Date	Storm Length (hours)	Total Precipitation		Maximum Intensity		Peak Tide Level (ft NAVD)
		Total Rainfall (in.)	Return Frequency (years)	Maximum Intensity (in./hr.)	Return Frequency (years)	
5/5/2017	24	3.36	2-5	1.22	1-2	0.88

Figure B8-7 - Verification Graphic Comparing Modeled and Observed Flooding Extent at Redneck Avenue Location

7.0 Model Results

Hydrologic Engineering Center – Flood Damage Reduction Analysis (HEC-FDA) will be used to perform the BCA for the drainage improvement concepts. Performance of existing conditions and conditions with the Proposed Project in place will be assessed against six rainfall frequency design events: 2-year, 5-year, 10-year, 25-year, 50-year, and 100-year. A 48 hour duration for 2016 and 2075 scenarios under normal tidal conditions (2016 and 2075 sea level rise (SLR)) and during a 2-year coastal storm surge event (2016 and 2075 SLR) will be implemented. The storm surge event is estimated to be the 2-year frequency storm. InfoWorks results are provided in the form of maximum WSEL TIN polygons for the mesh zone and maximum WSEL at 1D stream cross-sections. Mesh zone 2D flood reduction graphics (showing the Proposed Project condition clean conduits WSEL, minus the existing condition clean conduits WSEL), are prepared for five Proposed Project condition model scenarios, namely DePeyster Creek, Losen Slote Concept D, Losen Slote Alternative 3 *Build Plan*, Losen Slote/Carol Place, and Main Street. These exhibits for the 25-year 48-hour 2016 and 2075 rain events, combined with normal 2016 and 2075 SLR with and without coastal surge tidal conditions, are prepared and shown in **Figure B8-8** to **Figure B8-27**.

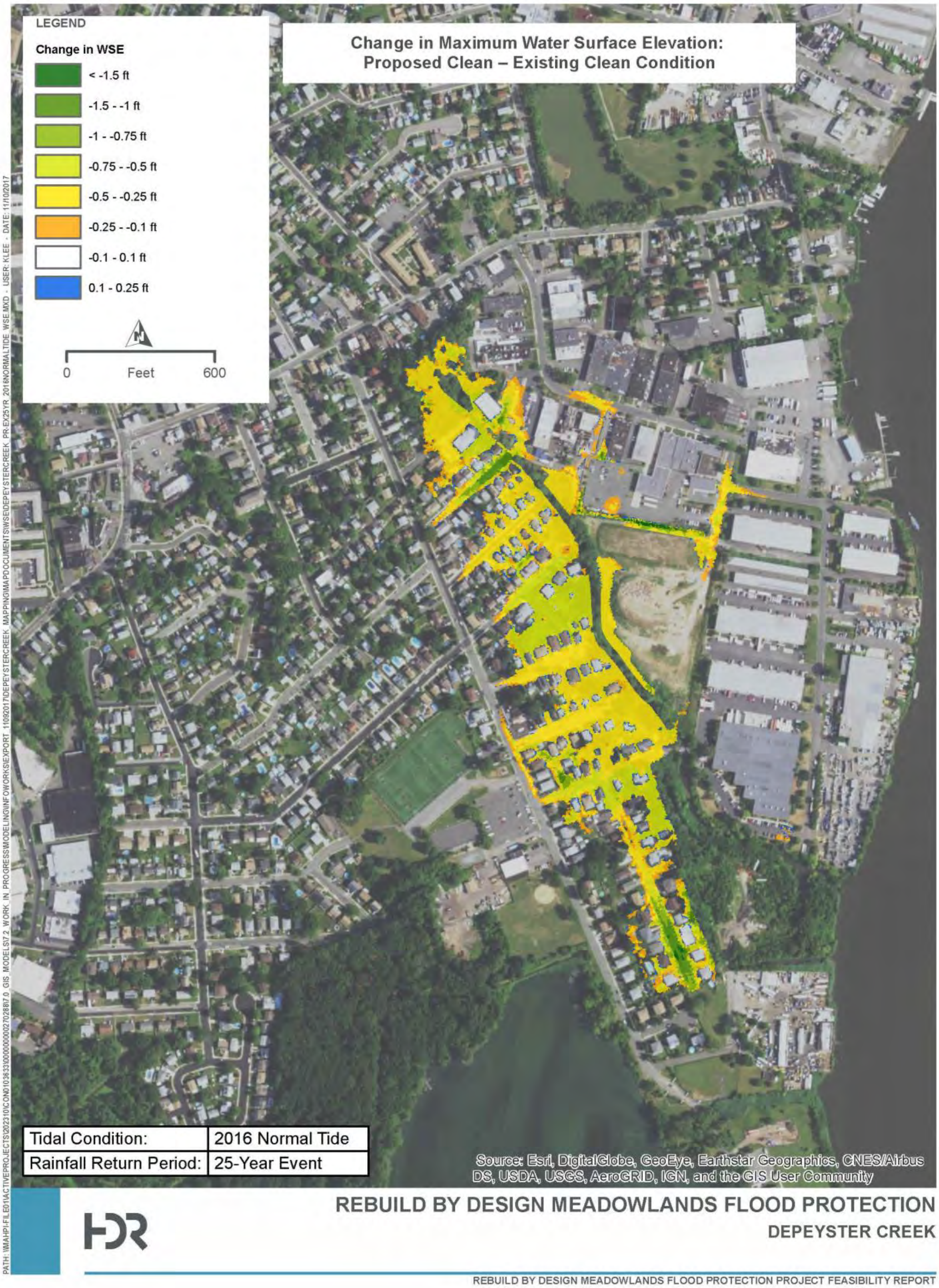


Figure B8-8 - Change in Water Surface Elevation for DePeyster Creek Model in Response to 25-Year 48-Hour Rainfall and 2016 Normal Tide Condition

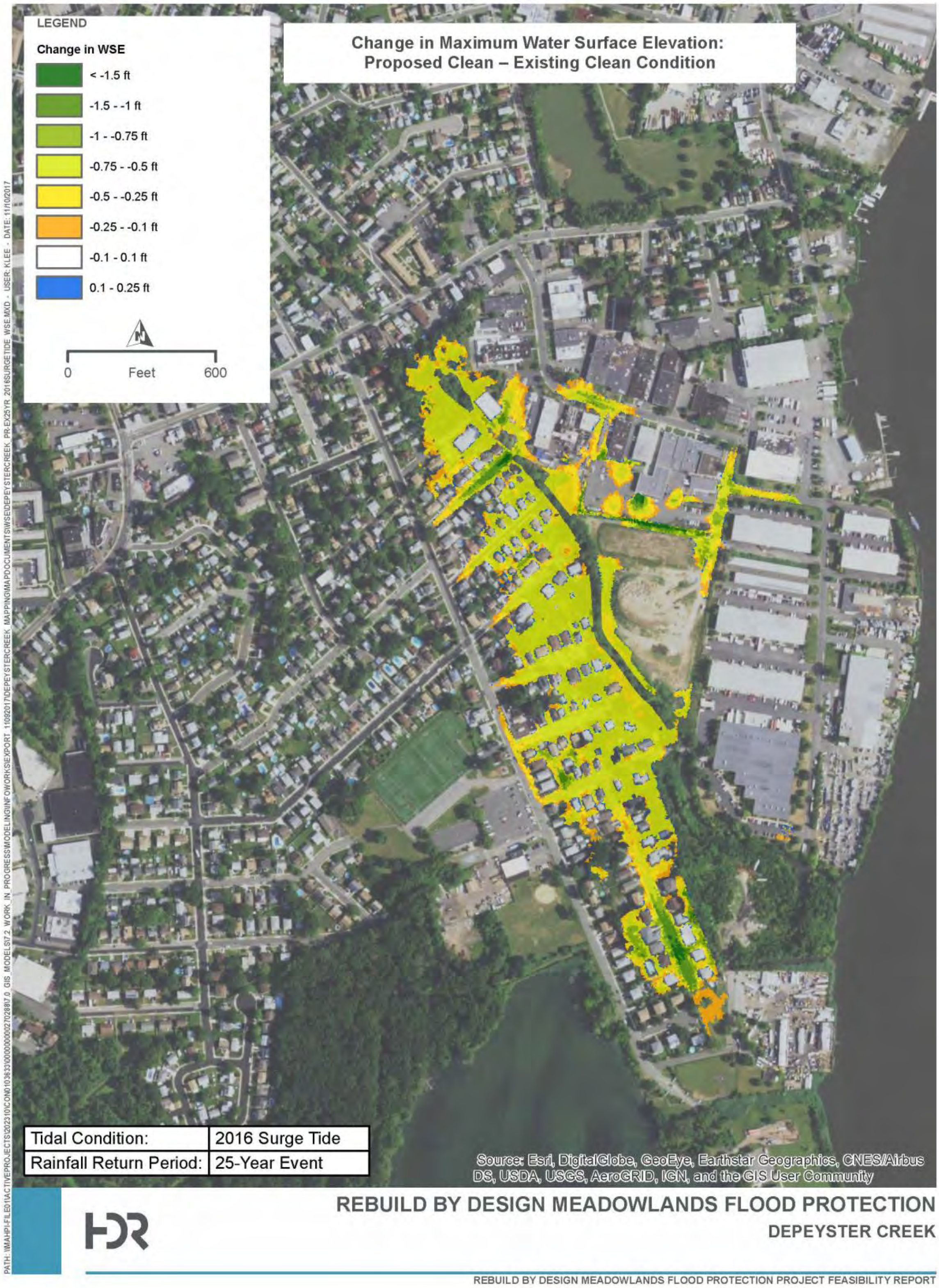


Figure B8-9 - Change in Water Surface Elevation for DePeyster Creek Model in Response to 25-Year 48-Hour Rainfall and 2016 Surge Tide Condition

Figure B8-10 - Change in Water Surface Elevation for DePeyster Creek Model in Response to 25-Year 48-Hour Rainfall and 2075 Normal Tide Condition

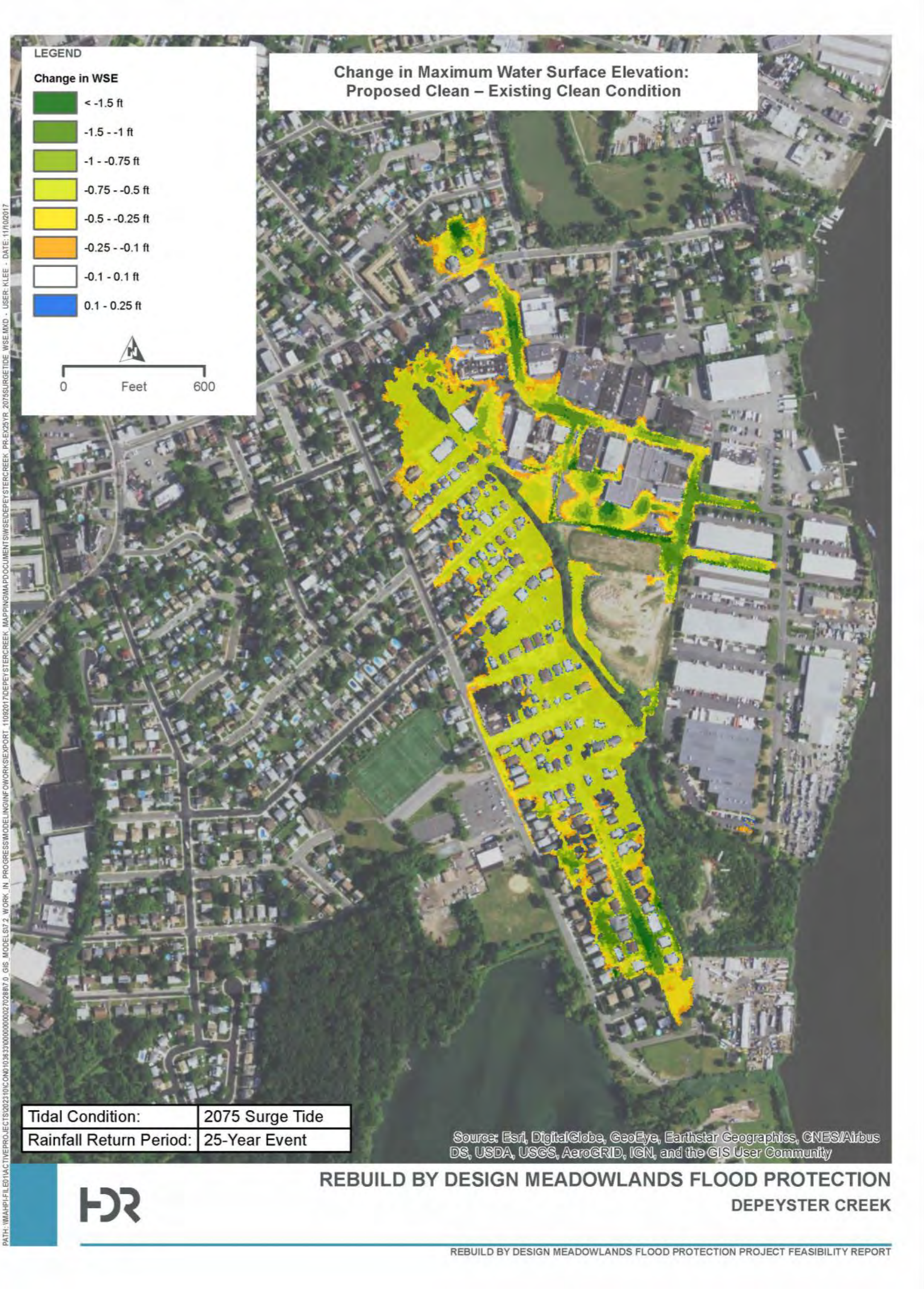


Figure B8-11 - Change in Water Surface Elevation for DePeyster Creek Model in Response to 25-Year 48-Hour Rainfall and 2075 Surge Tide Condition

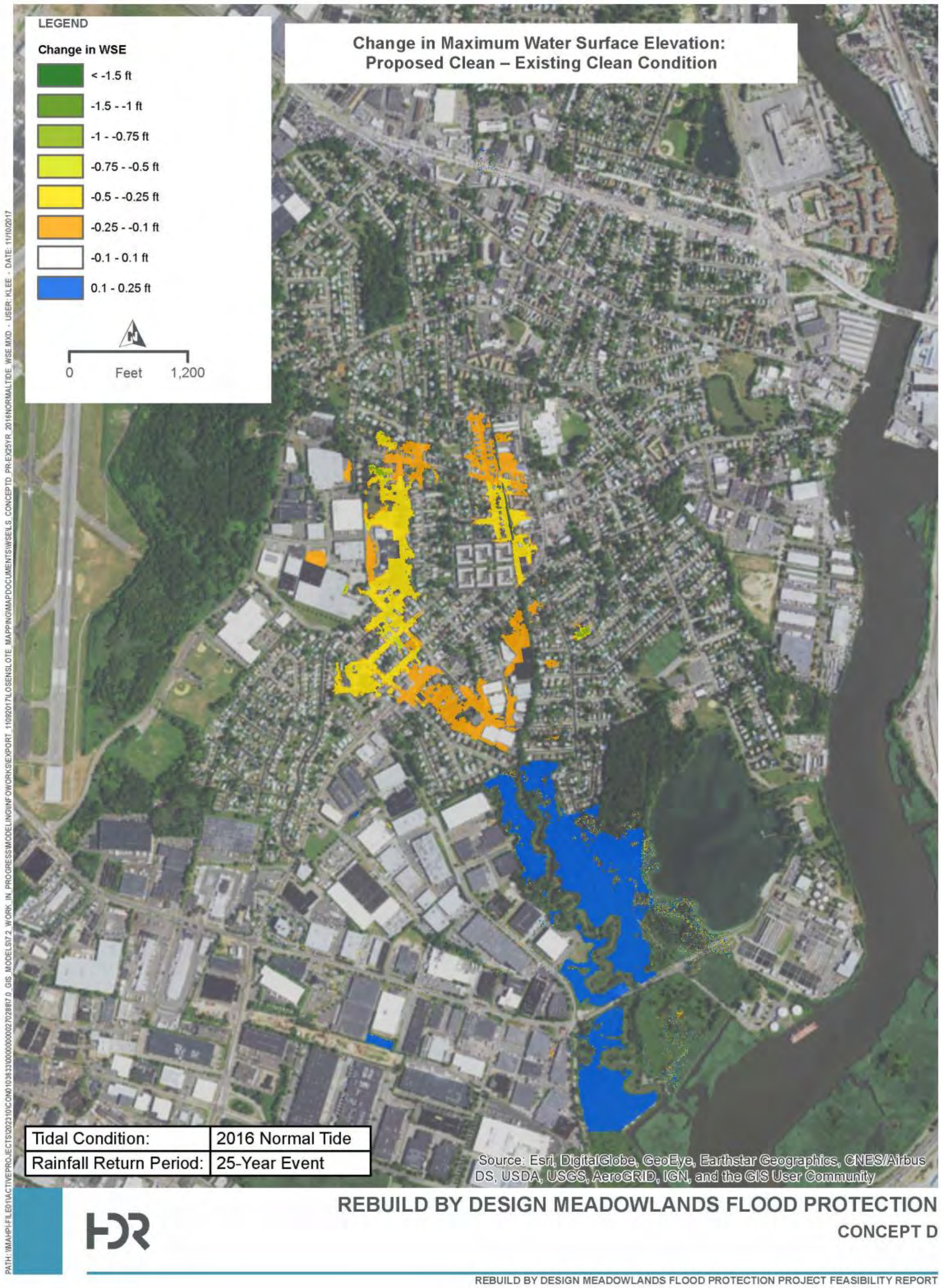


Figure B8-12 - Change in Water Surface Elevation for Losen Slote Concept D Model in Response to 25-Year 48-Hour Rainfall and 2016 Normal Tide Condition

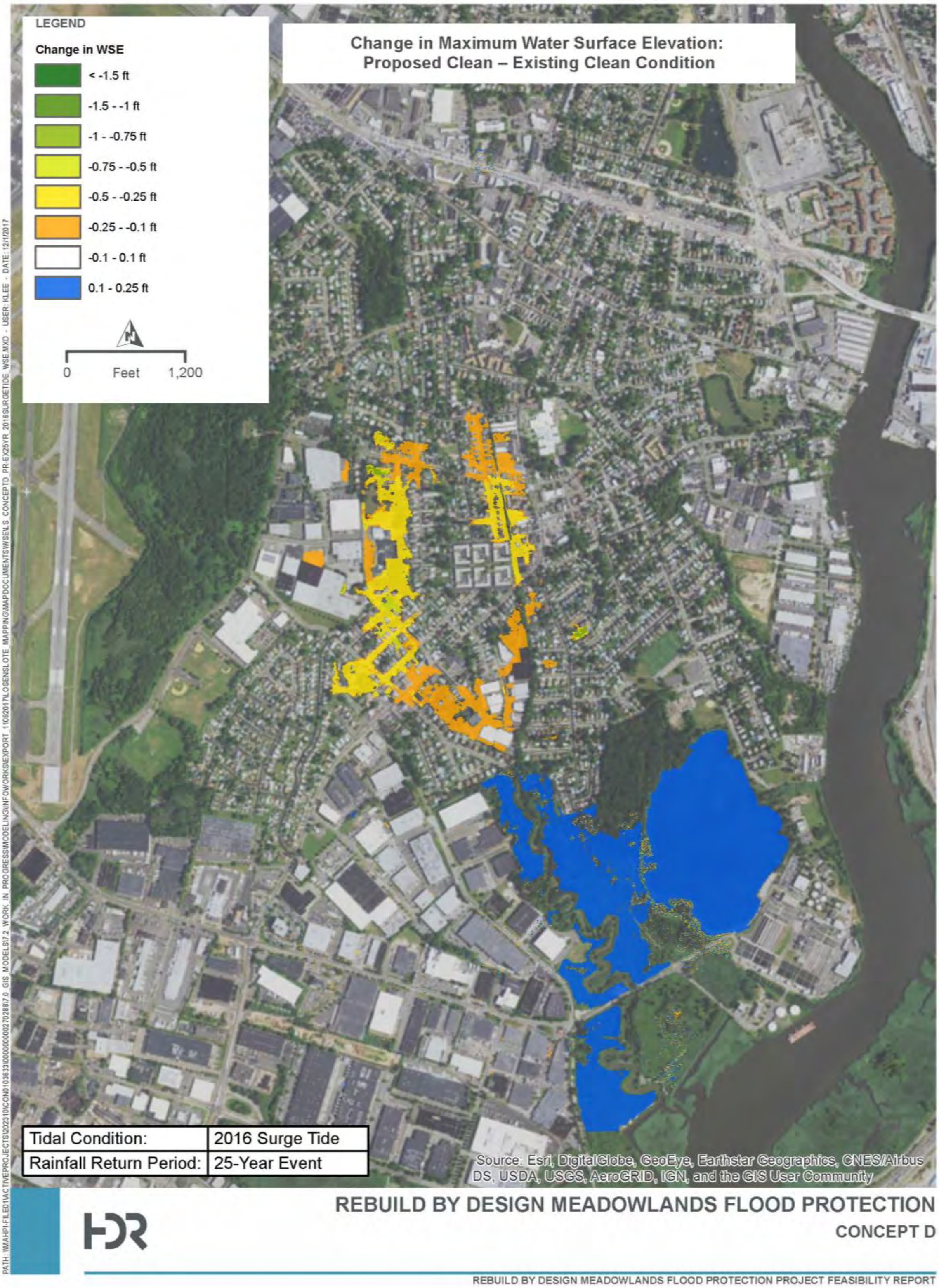


Figure B8-13 - Change in Water Surface Elevation for Losen Slote Concept D Model in Response to 25-Year 48-Hour Rainfall and 2016 Surge Tide Condition

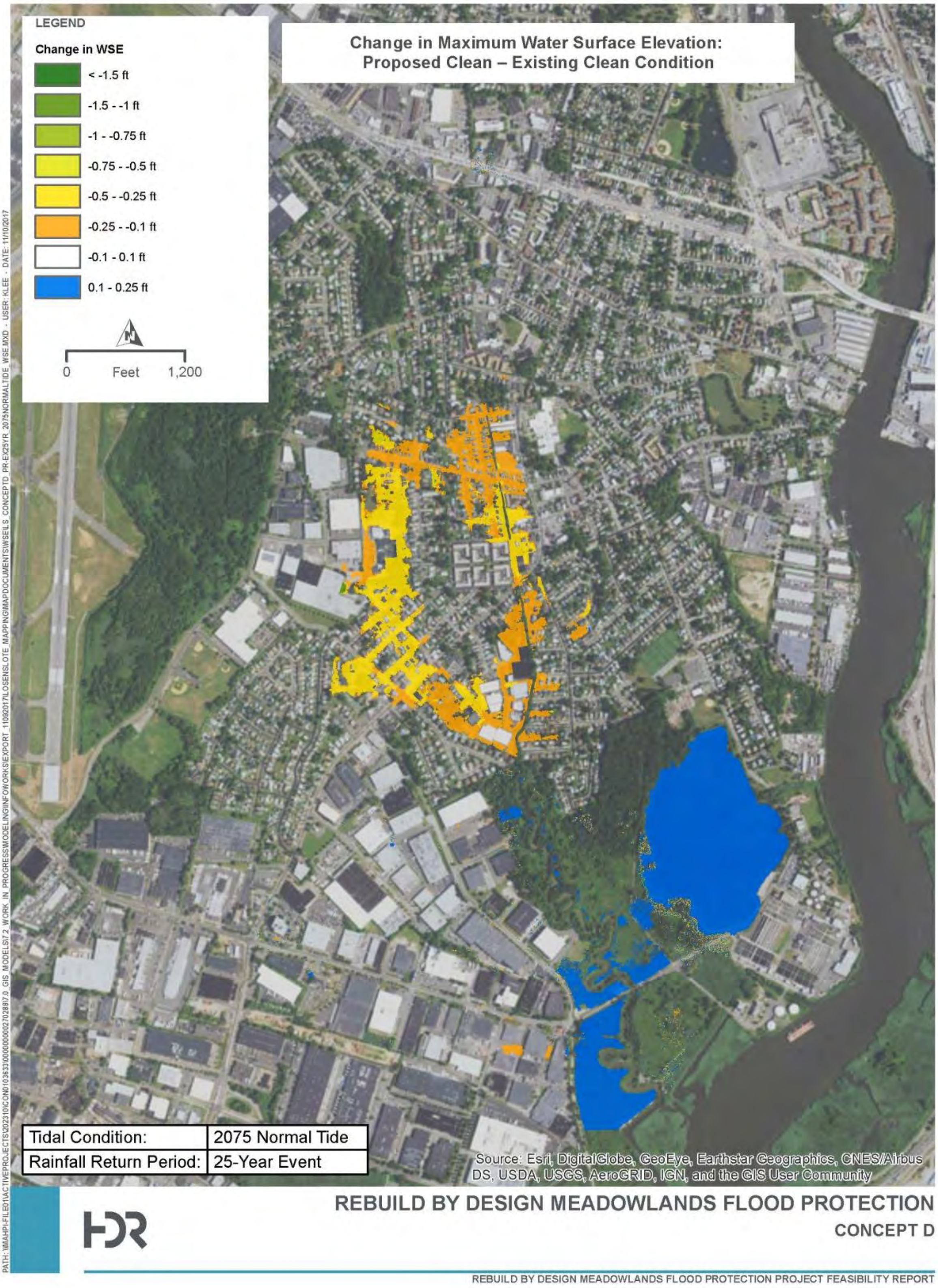


Figure B8-14 - Change in Water Surface Elevation for Losen Slote Concept D Model in Response to 25-Year 48-Hour Rainfall and 2075 Normal Tide Condition

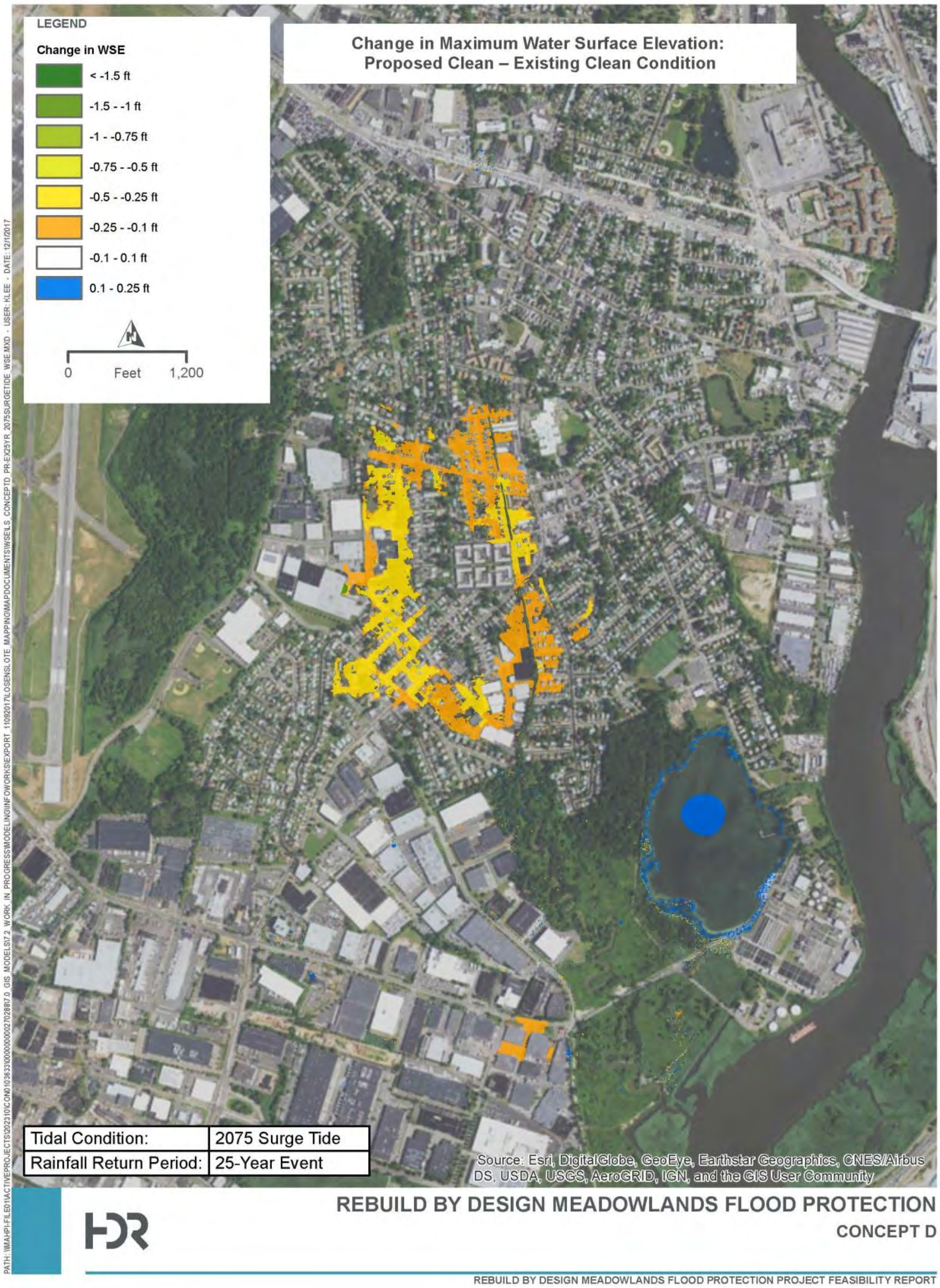


Figure B8-15 - Change in Water Surface Elevation for Losen Slote Concept D Model in Response to 25-Year 48-Hour Rainfall and 2075 Surge Tide Condition

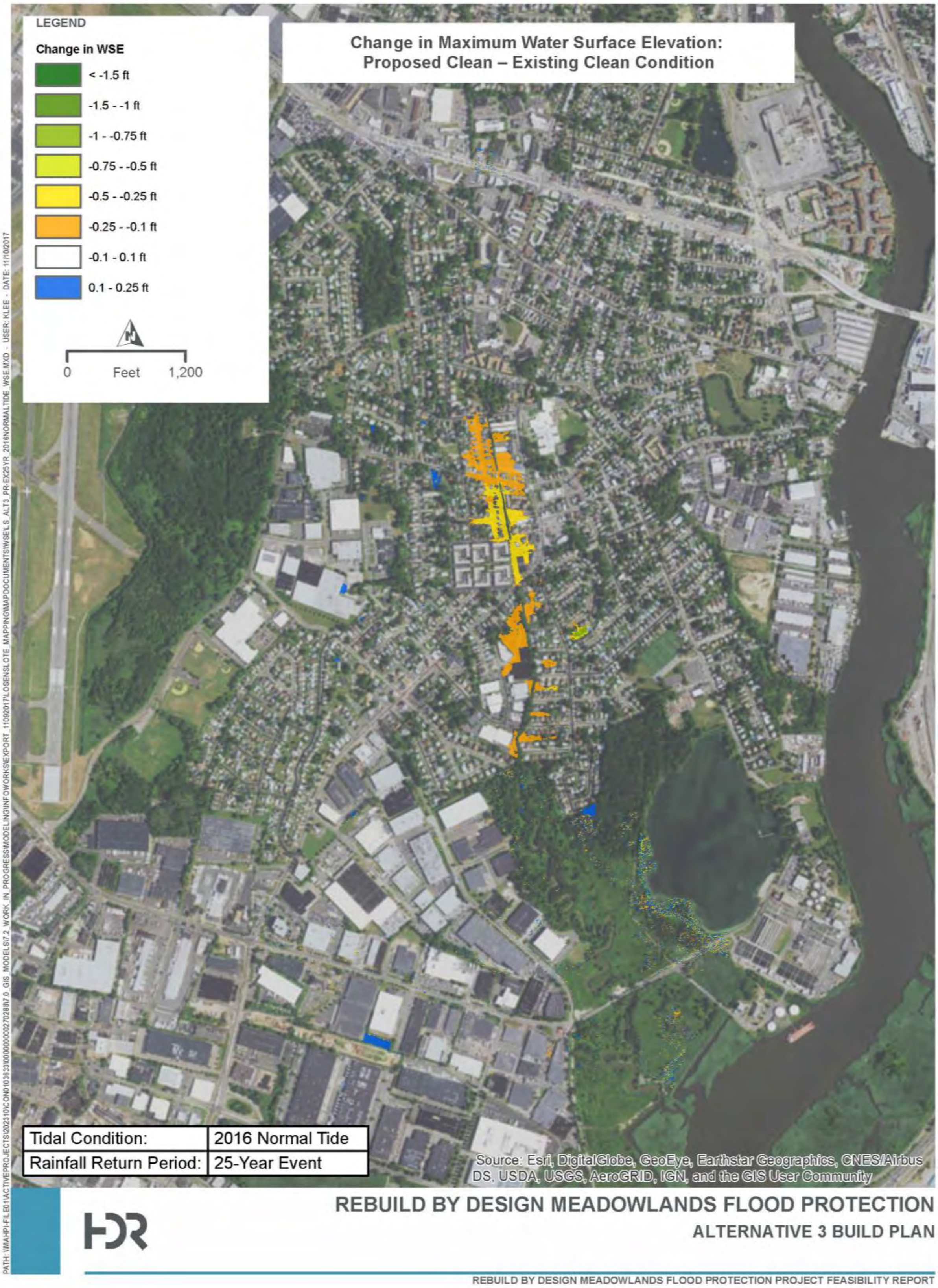


Figure B8-16 - Change in Water Surface Elevation for Losen Slote Alternative 3 Model in Response to 25-Year 48-Hour Rainfall and 2016 Normal Tide Condition

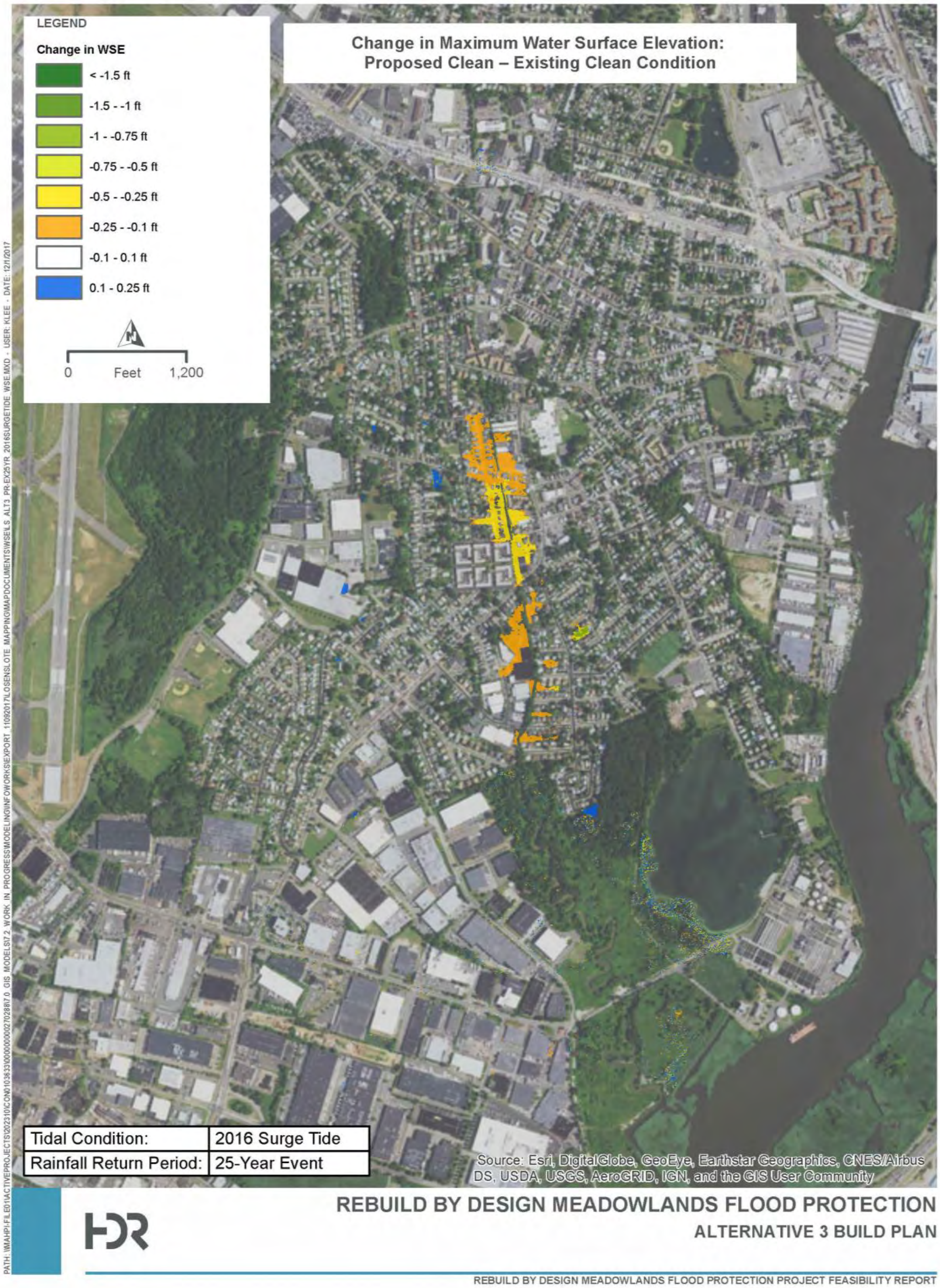


Figure B8-17 - Change in Water Surface Elevation for Losen Slote Alternative 3 Model in Response to 25-Year 48-Hour Rainfall and 2016 Surge Tide Condition

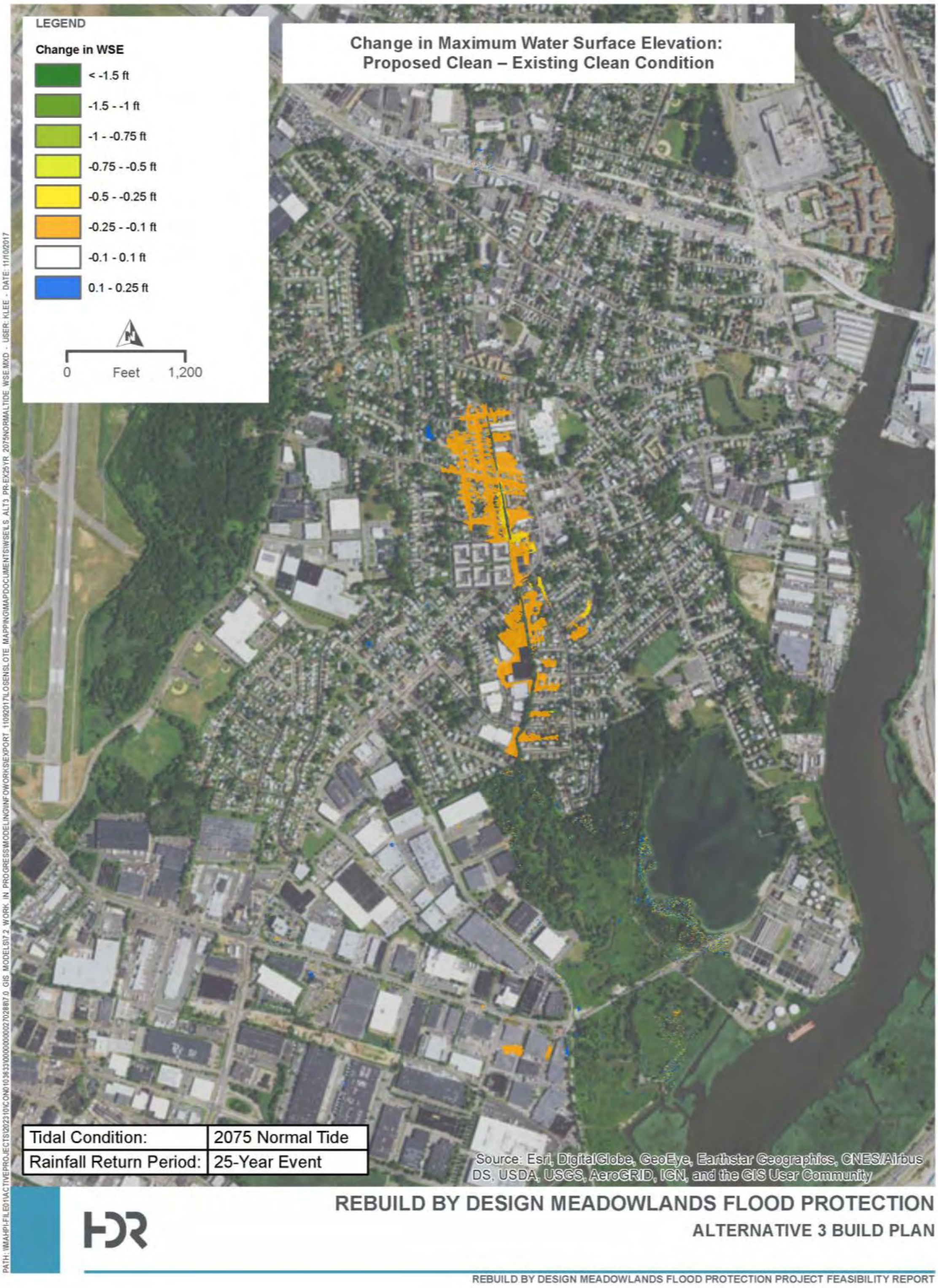


Figure B8-18 - Change in Water Surface Elevation for Losen Slote Alternative 3 Model in Response to 25-Year 48-Hour Rainfall and 2075 Normal Tide Condition

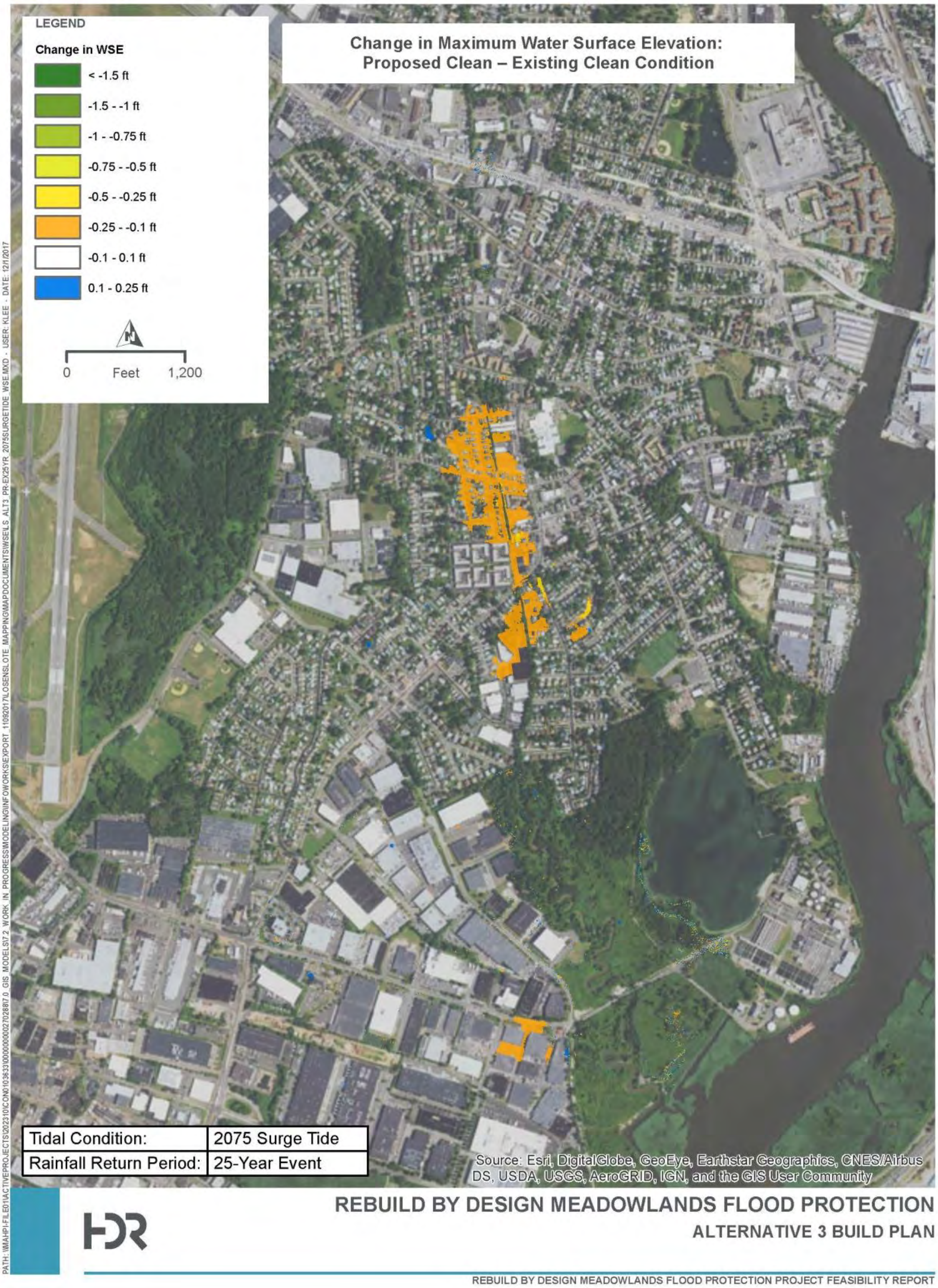


Figure B8-19 - Change in Water Surface Elevation for Losen Slote Alternative 3 Model in Response to 25-Year 48-Hour Rainfall and 2075 Surge Tide Condition

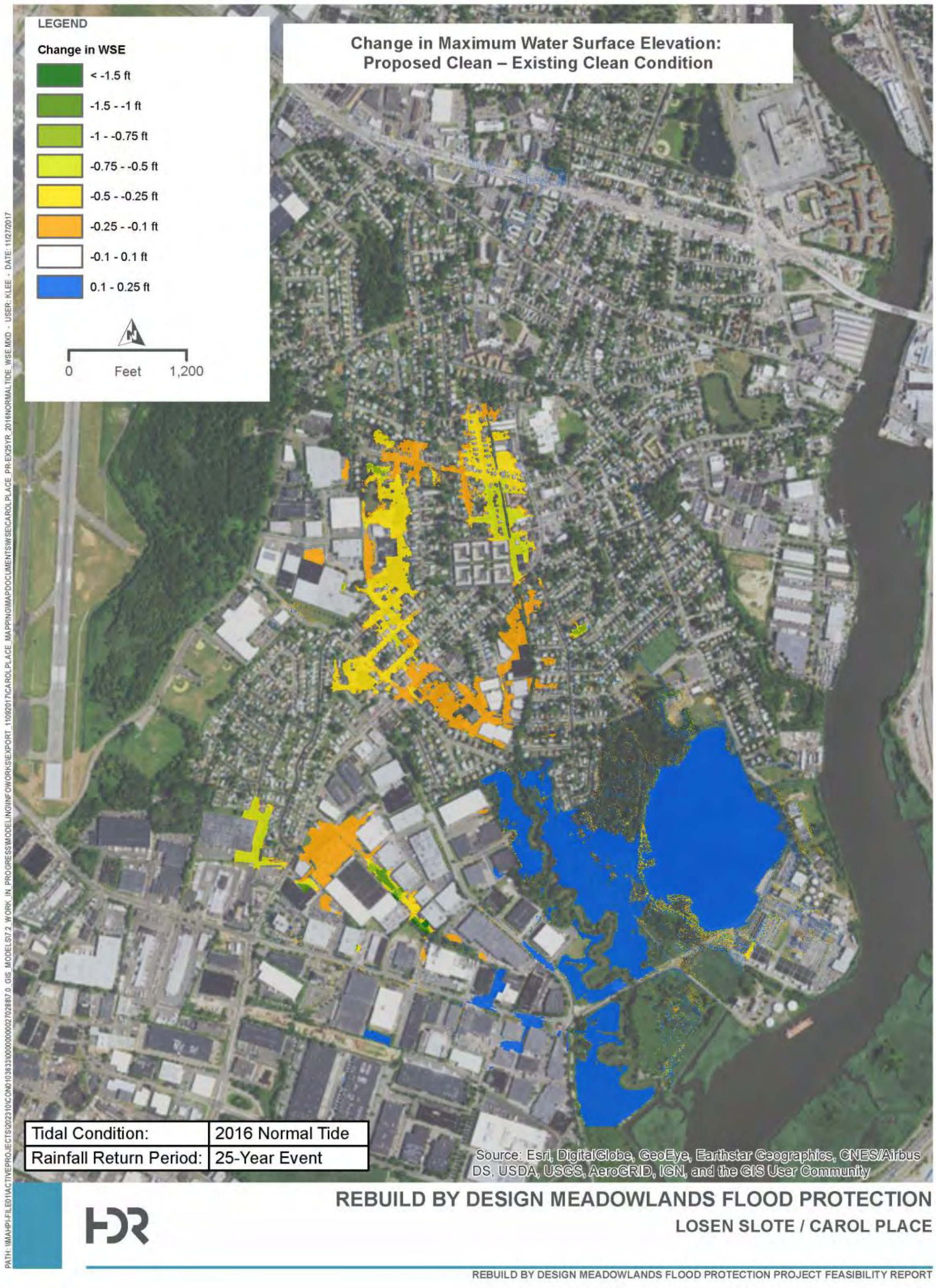


Figure B8-20 - Change in Water Surface Elevation for Losen Slote / Carol Place Model in Response to 25-Year 48-Hour Rainfall and 2016 Normal Tide Condition

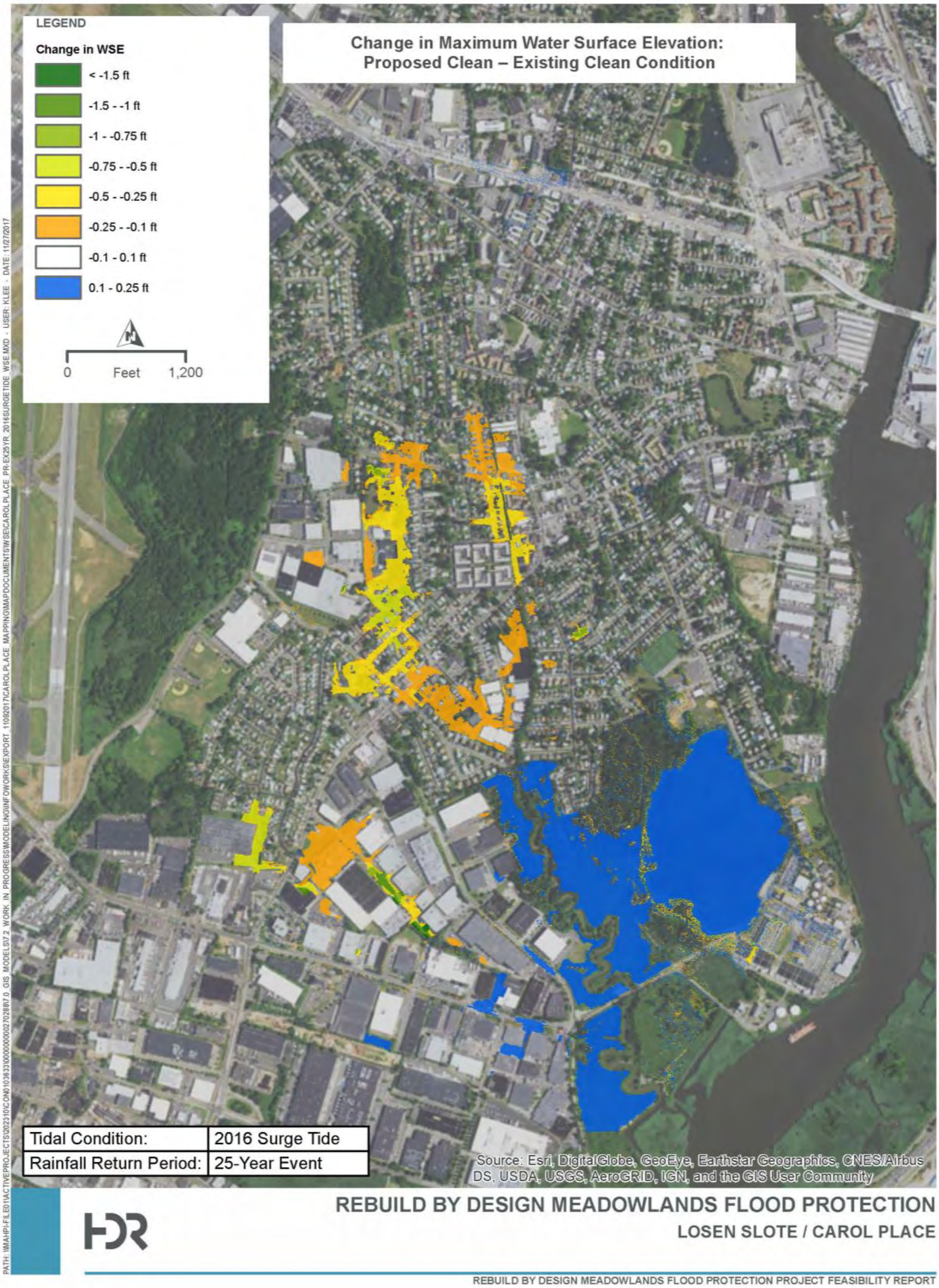


Figure B8-21 - Change in Water Surface Elevation for Losen Slote / Carol Place Model in Response to 25-Year 48-Hour Rainfall and 2016 Surge Tide Condition

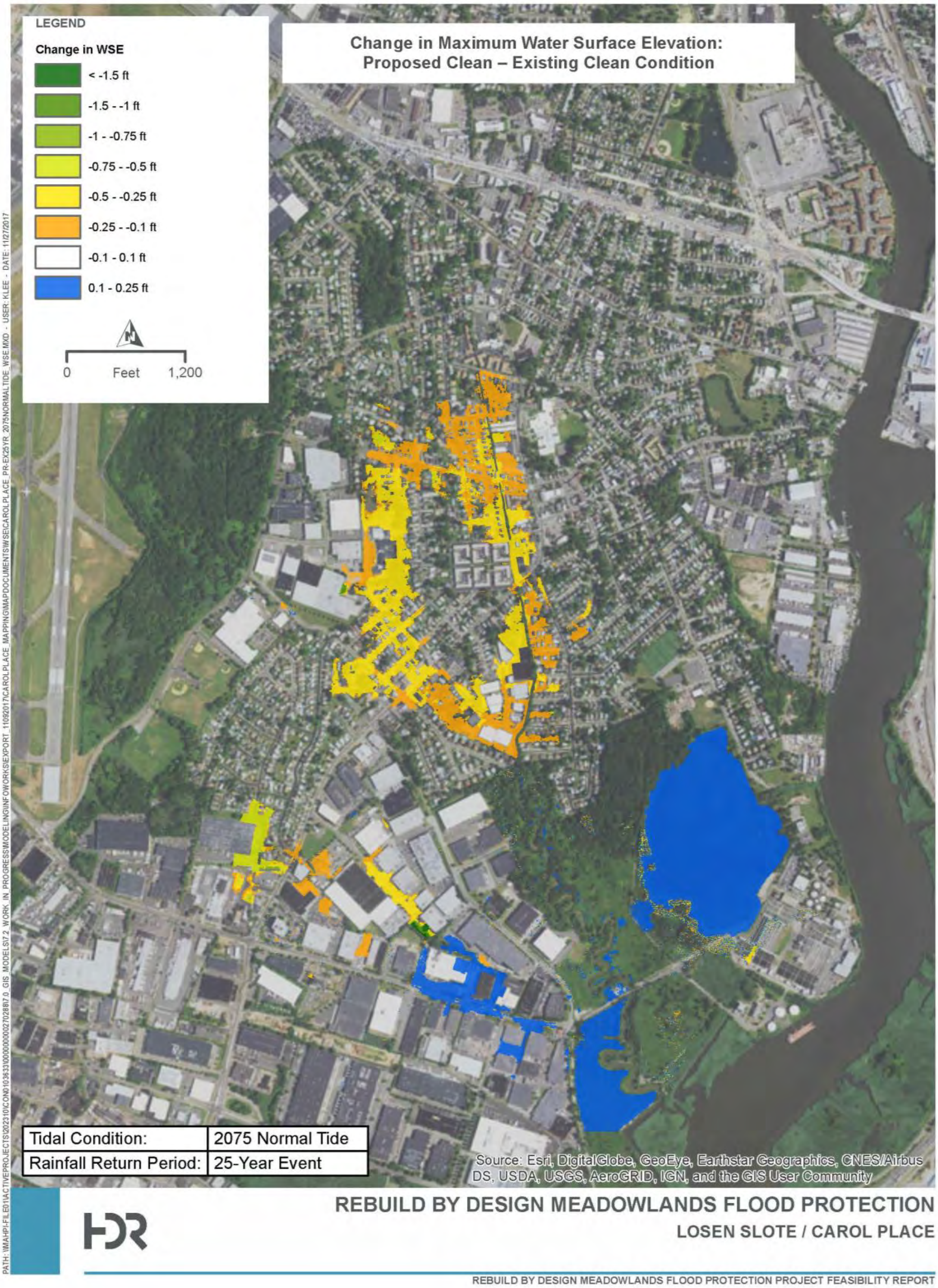


Figure B8-22 - Change in Water Surface Elevation for Losen Slote / Carol Place Model in Response to 25-Year 48-Hour Rainfall and 2075 Normal Tide Condition

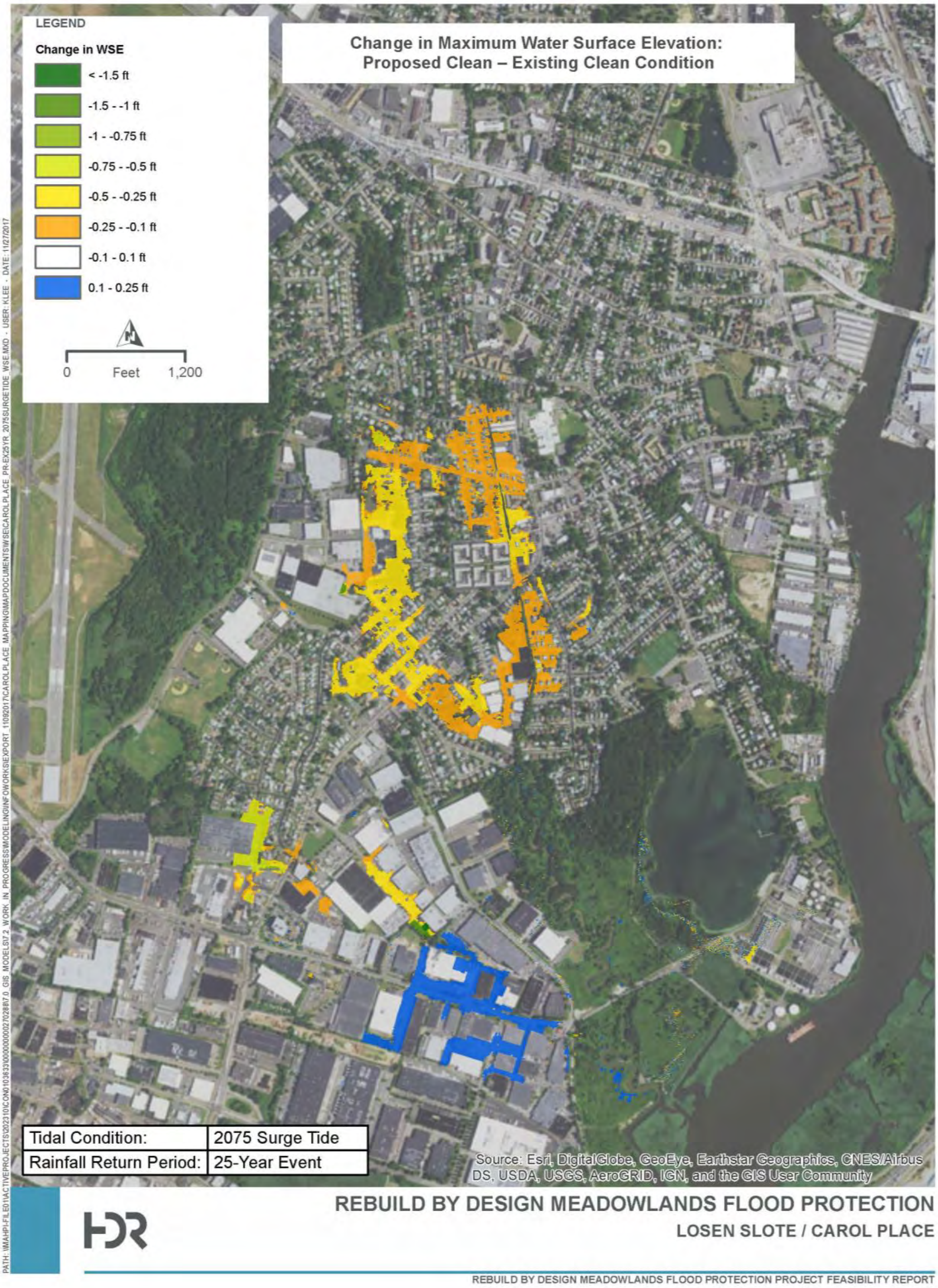


Figure B8-23 - Change in Water Surface Elevation for Losen Slote / Carol Place Model in Response to 25-Year 48-Hour Rainfall and 2075 Surge Tide Condition

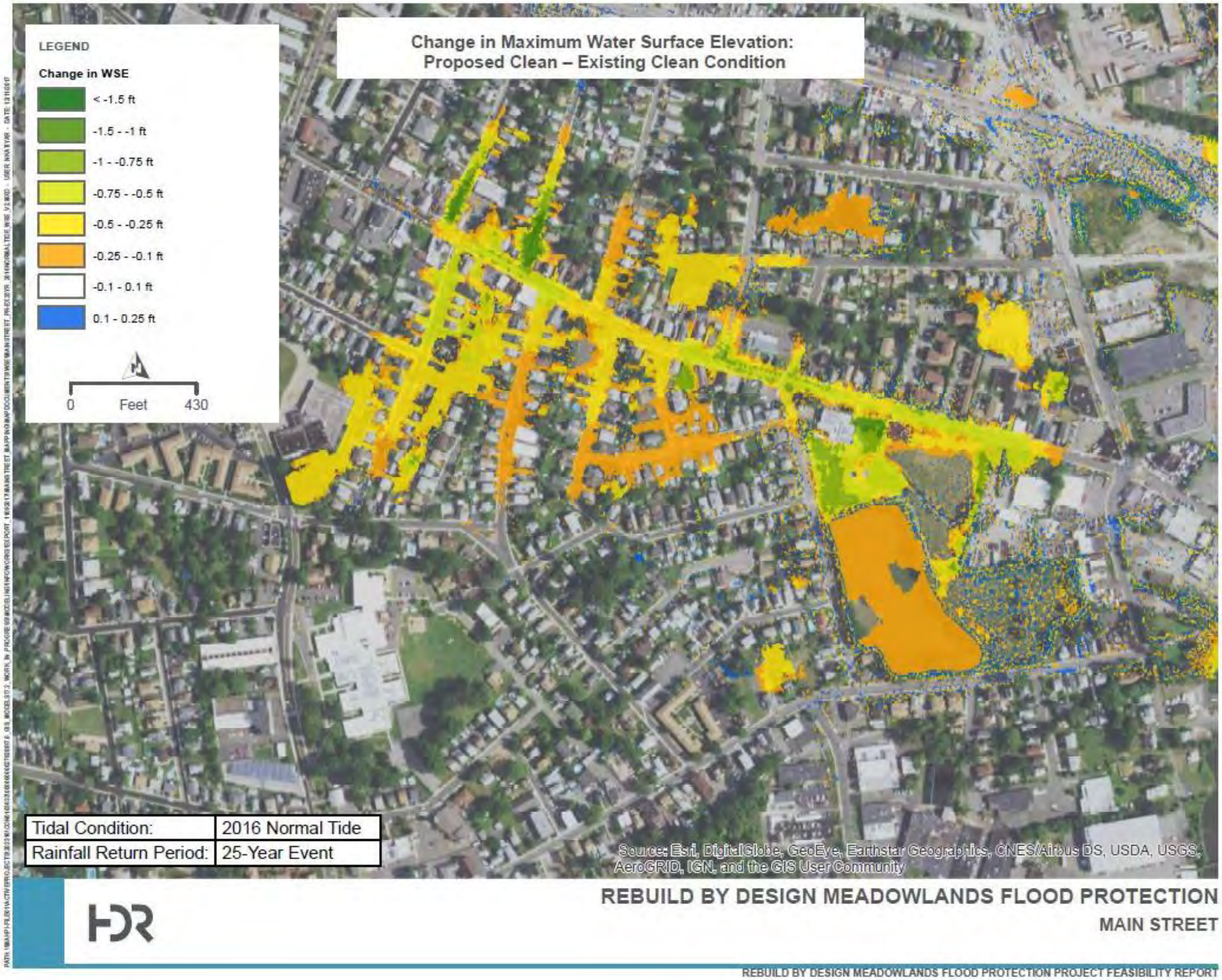


Figure B8-24 - Change in Water Surface Elevation for Main Street Model in Response to 25-Year 48-Hour Rainfall and 2016 Normal Tide Condition

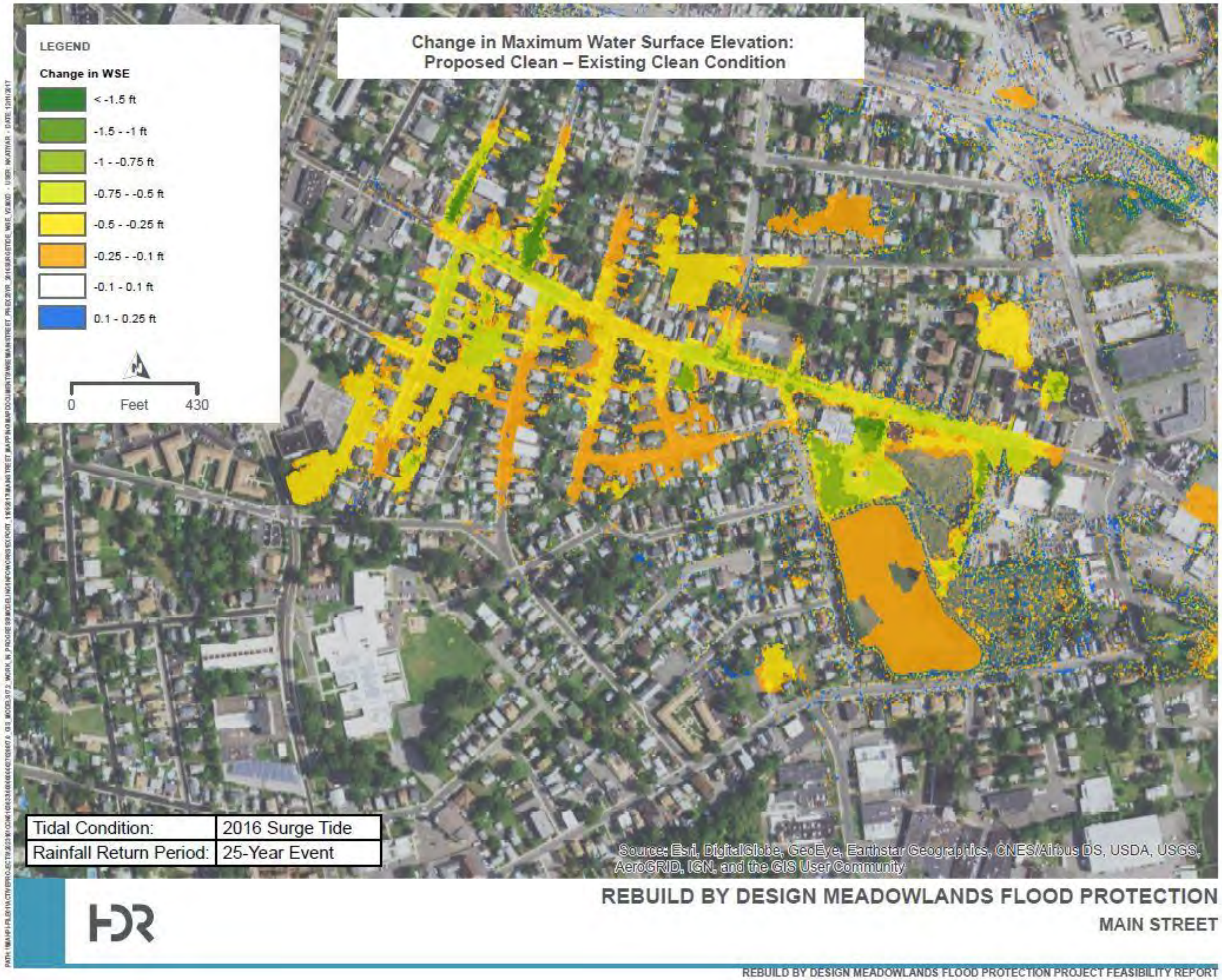


Figure B8-25 - Change in Water Surface Elevation for Main Street Model in Response to 25-Year 48-Hour Rainfall and 2016 Surge Tide Condition

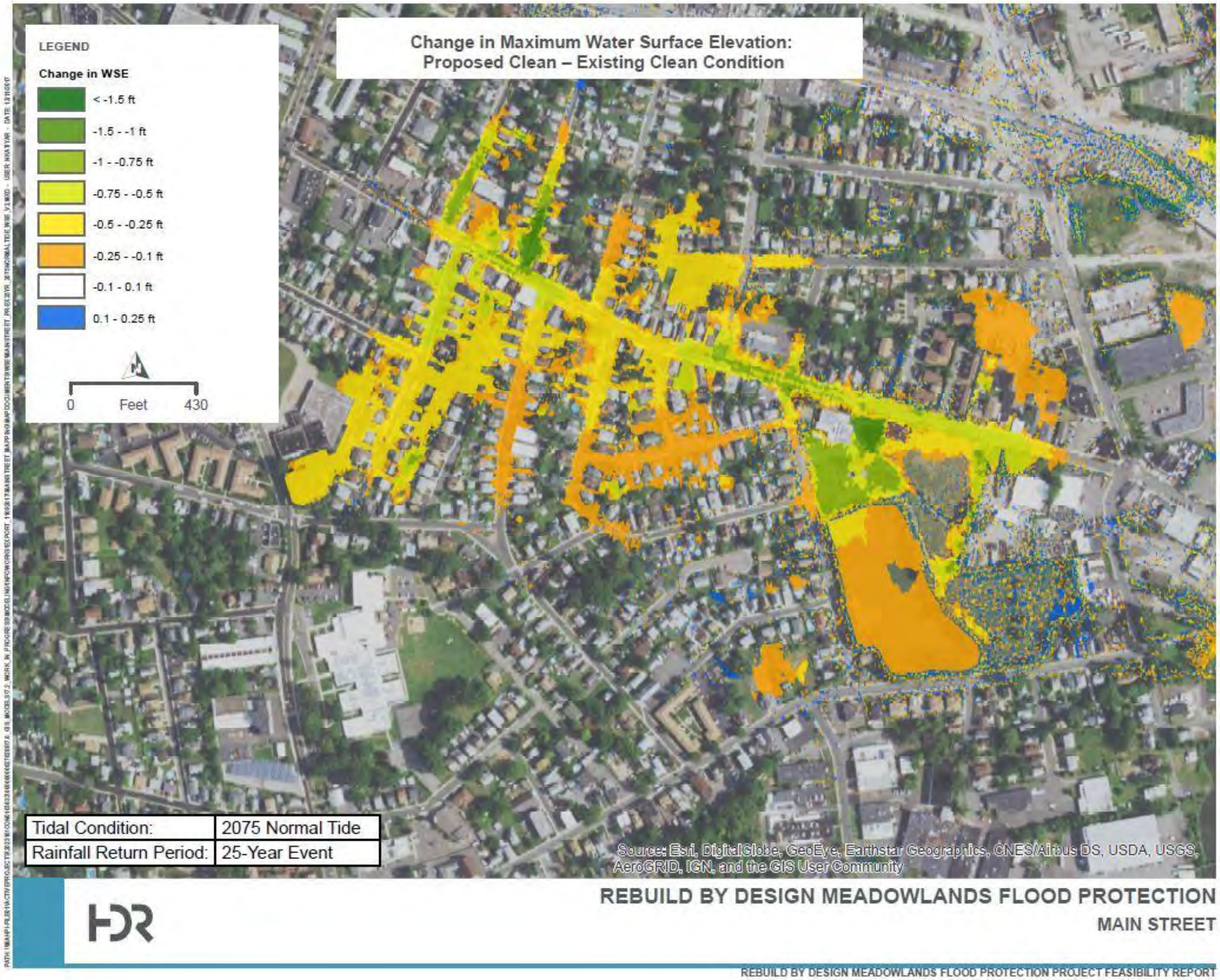


Figure B8-26 - Change in Water Surface Elevation for Main Street Model in Response to 25-Year 48-Hour Rainfall and 2075 Normal Tide Condition

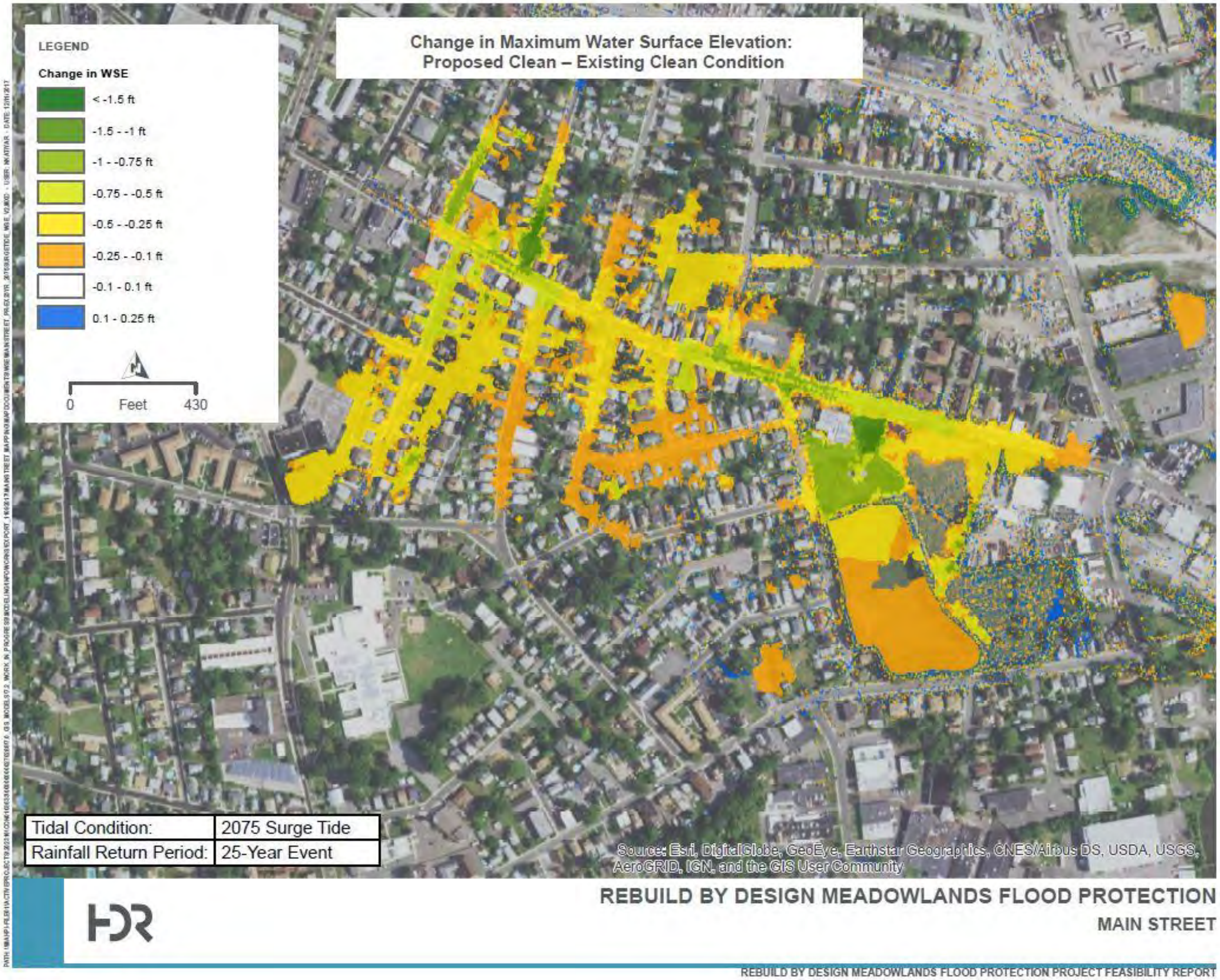


Figure B8-27 - Change in Water Surface Elevation for Main Street Model in Response to 25-Year 48-Hour Rainfall and 2075 Surge Tide Condition

8.0 References

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Subappendix B9: Global Climate Change and Sea Level Change

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Table of Contents

List of Tables

List of Figures

Acronyms and Abbreviations

BCCA	Bias-Corrected Constructed Analogs
BoR	Bureau of Reclamation
CEQ	Council on Environmental Quality
CMIP5	Coupled Model Intercomparison Project
FLEx	Forecasting Local Extremes
GCM	General circulation models
GHG	Greenhouse gas
IPCC	Intergovernmental Panel on Climate Change
km	Kilometers
LOCA	Localized Constructed Analogs
MACA	Multivariate Adaptive Constructed Analogs
mm/year	Millimeters per year
NEPA	National Environmental Policy Act
NJDEP	New Jersey Department of Environmental Protection
NPCC	New York City Panel on Climate Change
NRC	National Research Council
PDS	Partial duration series
RBDM	Rebuild by Design Meadowlands
RCP	Representative concentration pathway
RSLC	Relative sea level change
SLC	Sea level change
SLR	Sea level rise
UoI	University of Idaho
US	United States

1.0 Global Climate Change and Sea Level Change

Global climate change is an important environmental challenge facing the world today, and human activity is one of the drivers affecting it. Research on this topic has been well documented in reports by the United Nations Intergovernmental Panel on Climate Change (IPCC), United States (US) Climate Change Science Program's Science Synthesis and Assessment Products, and the US Global Change Research Program. In addition, the Council on Environmental Quality (CEQ) issued updated Draft Guidance on Considering Climate Change in National Environmental Policy Act (NEPA) Reviews, which provides Federal agencies with direction on when and how to consider the effects of greenhouse gas (GHG) emissions and climate change in their evaluation of proposed Federal actions (CEQ 2016).

1.1 Meadowlands Climate Science Review and Analysis

The purpose of this subappendix is to evaluate changes to key flood-related climate variables for the Meadowlands District as predicted by the most recent scientific literature and model output available. First, the Project Area is described in **Section 1.1.1**. **Section 1.2** focuses on projected changes to sea level, while **Section 1.3** focuses on projected changes to precipitation.

1.1.1 Project Area

The Project Area is located in the State of New Jersey approximately four miles northwest of New York City and includes either the entirety or portions of the Boroughs of Carlstadt, Moonachie, Little Ferry, Hasbrouck Heights, Teterboro, Wood-Ridge, Rutherford, East Rutherford, and the City of Hackensack and the Township of South Hackensack,. The Project Area was modified from the original Rebuild by Design Meadowlands (RBDM) concept to span the area west of the Hackensack River between Interstate-80 (I-80) and Berry's Creek Canal, as shown in **Figure B9-1**.

1.1.2 Existing Flood Hazards

A majority of the Project Area is relatively low-lying and experiences flooding from three main sources: stormwater, fluvial, and coastal. The most frequent flooding occurs due to inadequate or deteriorated stormwater infrastructure that stores and conveys local runoff from precipitation events. This complex network of drainage ditches, tide gates, pump stations, and other structures present a formidable challenge in addressing this flood hazard for the Project Area as a whole.

Figure B9-1: RBDM Flood Protection Project Area

The Hackensack River is controlled upstream of the Project Area by the Oradell Reservoir Dam and flows south into Newark Bay. The river is tidally influenced, and as a result, coastal flood events typically have a greater impact on flooding the Project Area than fluvial events. Hurricanes and nor'easters can create substantial storm surges, which reach up the Hackensack River to the Project Area and can overtop existing flood protection along Berry's Creek and the Hackensack River. Stormwater flooding can occur in combination with coastal flooding which presents additional stress on the stormwater infrastructure's ability to discharge runoff; however, high precipitation events have not historically been accompanied by high coastal storm surge.

1.2 Sea Level Change

1.2.1 Description and Timeline of Key Sea Level Change Estimates

The science of projecting future sea level rise (SLR) is constantly evolving as researchers gather the most up-to-date data on key climate and physical variables that govern this complex rate of change. In 1987, the National Research Council (NRC) completed a study on sea level change (SLC) and published Responding to Changes in Sea Level: Engineering Implications (NRC 1987). The report reviewed relative sea level change (RSLC) and presented a range of possible global mean SLC scenarios. Global mean SLC rates were presented up to 2100 (from a starting year of 1986) for three different scenarios (typically referred to as NRC Curve I, NRC Curve II, and NRC Curve III).

When the NRC report was published, the estimate of global mean SLC was 1.2 millimeters per year (mm/year). By the time IPCC released the fourth Assessment Report (AR4) on climate change in 2007, the rate increased to 1.7 mm/year (IPCC 2007). In the U.S. Army Corps of Engineers' (USACE) Engineering Circular (EC 1165-2-212) published in 2011, the 1987 NRC curves were modified to: account for this increased global mean SLC reported by the IPCC, incorporate the local rate of vertical land movement, and start in 1992 (the midpoint of the current National Tidal Datum Epoch from 1983 to 2001) (USACE 2011). The 1987 NRC curves were renamed the Modified NRC Curve I, Modified NRC Curve II, and the Modified NRC Curve III. The recommendation from the USACE's EC 1165-2-212 was to use the local historical rate of SLC for a USACE Low scenario, the Modified NRC Curve I as the USACE Intermediate scenario, and the Modified NRC Curve II as the USACE High scenario. These curves were further modified in 2013 when the USACE released Engineering Regulation ER 1100-2-8162 (USACE 2013).

In 2012, the National Oceanic and Atmospheric Administration (NOAA) published its own global mean SLC estimates (High, Intermediate-High, Intermediate-Low, Low) as a part of the National Climate Assessment (NOAA 2012). The Intermediate-High Scenario is based on an average of published high-end semi-empirical, global SLR projections and includes statistical relationships between observed global SLC, recent ice sheet loss, and air temperature. The Intermediate-Low Scenario is based on the upper end of IPCC Fourth Assessment Report (AR4) global SLR projections resulting from climate models using the B1 emissions scenarios and primarily captures the effects of ocean warming (IPCC 2007). The Intermediate-Low Scenario corresponds with the USACE Intermediate Scenario (Modified NRC Curve II) while the Low Scenario corresponds with the USACE Low Scenario (Modified NRC Curve I). These rates were updated in 2013 to include regional vertical land movement where data was available (NOAA 2013).

Most recently, the New York City Panel on Climate Change (NPCC) published estimates of local RSLC in 2013 that were then updated in 2015. Projections are relative to the 2000–2004 baseline period and based on a six-component approach that incorporates both local and global factors. For each of the components of sea level change, the 10th, 25th, 75th, and 90th percentiles of the distribution were calculated and the sum of all components at each percentile is assumed to give the aggregate sea level rise projection (Horton, et al. 2015).

All of the above SLC curves at Battery Park, NY are shown in **Figure B9-2**.

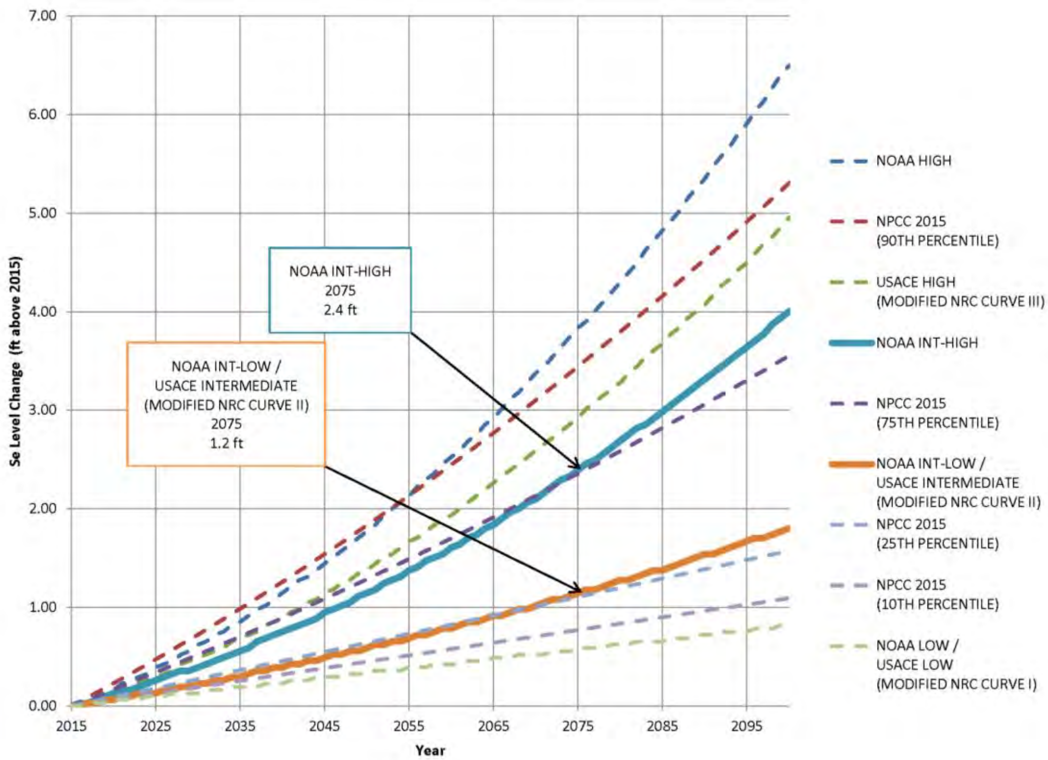

Figure B9-2: Sea Level Change Estimates for the Battery, NY (Gauge: 8518750)

Note: NOAA values are regionally corrected to include regional vertical land movement as documented in the NOAA report on Estimating Vertical Land Motion from Long-Term Tide Gauge Records (NOAA 2013).

1.3 Precipitation

In addition to SLC, existing and future changes to precipitation could have significant impacts to stormwater and fluvial flooding in the Meadowlands District. As in the case of SLC, precipitation changes occur as a result of many influencing factors (anthropogenic and natural) that govern the water cycle on local, regional, and global scales. While fundamentally increasing trends in future sea levels and global temperature are relatively certain, precipitation trends are much more uncertain (Kunkel, et al. 2013). This uncertainty is at least partially due to the fact that the general circulation models (GCMs) used in these analyses do not have sufficient spatial resolution to resolve some of the most important physical process that cause precipitation (Wehner, et al. 2010, Sillmann, et al. 2013).

1.3.1 Regional Trends

In the National Climate Assessment, the Northeast is reported to be experiencing the greatest recent increase in extreme precipitation observed for any region in the United States with increases of over 70 percent in precipitation due to very heavy events (top 1 percent of daily events) (Horton, et al. 2014). Similarly, NOAA's more recent assessment of updated GCM output found that while mean annual precipitation is projected to increase by 10 to15 percent by mid-century (2050) in the representative concentration pathway (RCP[1]) 8.5 scenario extreme precipitation in the same scenarios is projected to increase by 70 to 80 percent in the Northeast (Sun, et al. 2015).

There are two key factors that contribute to projected changes to precipitation in the Northeast and globally. Firstly, it is well established that as atmospheric temperatures increase the atmosphere can hold greater amounts of moisture (Kharin, et al. 2013, Allen and Ingram 2002). Although exactly when precipitation will occur is influenced by many variables, the fundamental increase in available atmospheric moisture suggests that precipitation itself will likely increase as well. Secondly, changes to atmospheric circulation patterns will affect where increased precipitation occurs (Groisman, Knight and Zolina 2013). Kevin Trenberth (Trenberth 2011) explains:

> "There is a direct influence of global warming on precipitation. Increased heating leads to greater evaporation and thus surface drying, thereby increasing the intensity and duration of drought. However, the water holding capacity of air increases by about 7% per 1°C warming, which leads to increased water vapor in the atmosphere. Hence, storms, whether individual thunderstorms, extratropical rain or snow storms, or tropical cyclones, supplied with increased moisture, produce more intense precipitation events that are widely observed to be occurring, even in places where total precipitation is decreasing: "it never rains but it pours!" In turn this increases the risk of flooding."

1.3.2 Local Trends for the Meadowlands

In addition to a review of regional trends, it is important to examine precipitation changes on a local scale. Precipitation is inherently a local-scale process and can vary significantly throughout a region. Results from analyses performed by the NPCC and AECOM are presented in **Section 1.3.2.1** and **Section 1.1.1.1**, respectively.

1.3.2.1 The New York City Panel on Climate Change

The NPCC performed an analysis of future changes to precipitation for the New York City area. This analysis uses output from 35 GCMs from the most recent Coupled Model Intercomparison Project phase (CMIP5). Two scenarios—RCP 4.5 and RCP 8.5—are included and equally weighted in creating probability distributions for the analysis. The baseline period from 1971 to 2000 was used for comparison with future periods. Future periods were calculated using 30-year periods centered on the decade of interest (e.g., 2050s uses model output from 2040 to 2069). Changes to seasonal precipitation (**Table B9-2**) as well as changes to mean annual precipitation and temperature (**Table B9-2**) are calculated.

[1] RCP 8.5 is a worst-case scenario out of four GHG concentrations used in the most recent IPCC report (Fifth Assessment Report) in 2013 (IPCC 2013). The RCP 8.5 scenario represents unabated emissions without implementing any mitigation measures, due to a world characterized by rapid economic growth and high CO_2 equivalent concentrations in 2100.

Table B9-1: NPCC Projected Seasonal Precipitation Changes

	Low Estimate (10th percentile)	Middle Range (25th to 75th percentile)	High Estimate (90th percentile)
2020s			
Winter	-3%	+1% to +12%	+20%
Spring	-3%	+1% to +9%	+15%
Summer	-5%	-1% to +11%	+15%
Fall	-5%	-2% to +7%	+10%
2050s			
Winter	+2%	+7% to +18%	+24%
Spring	-1%	+3% to +12%	+18%
Summer	-9%	-5% to +11%	+18%
Fall	-2%	+1% to +10%	+14%
2080s			
Winter	+4%	+10% to +25%	+33%
Spring	-1%	+4% to +15%	+21%
Summer	-10%	-5% to +18%	+23%
Fall	-7%	-1% to +11%	+18%

Table B9-2. NPCC Mean Annual Changes to Precipitation and Temperature

Temperature			
Baseline (1971-2000) 54°F	**Low Estimate (10th percentile)**	**Middle Range (25th to 75th percentile)**	**High Estimate (90th percentile)**
2020s	+1.5°F	+2.0 to +2.9°F	+3.2°F
2050s	+3.1°F	+4.1 to +5.7°F	+6.6°F
2080s	+3.8°F	+5.3 to +8.8°F	+10.3°F
2100	+4.2°F	+5.8 to +10.4°F	+12.1°F
Precipitation			
Baseline (1971-2000) 50.1 in	**Low Estimate (10th percentile)**	**Middle Range (25th to 75th percentile)**	**High Estimate (90th percentile)**
2020s	-1%	+1% to +8%	+10%
2050s	+1%	+4% to +11%	+13%
2080s	+2%	+5% to +13%	+19%
2100	-6%	-1% to +19%	+25%

1.3.2.2 Analysis of Downscaled GCM Output

Additional analysis was performed by AECOM using the proprietary Forecasting Local Extremes (FLEx) tool in order to further evaluate local changes to precipitation. Statistically downscaled CMIP5 GCM output for a single scenario (RCP 8.5) was obtained from the Bureau of Reclamation (BoR) and the University of Idaho (UoI). Two datasets were obtained from the BoR. First, output from 20 GCMs downscaled to 1/8 degree (approximately 12 kilometers (km)) spatial resolution (**Figure B9-3**) using the Bias-Corrected Constructed Analogs (BCCA) method and an observed baseline period from 1950-2000. Second, output from 20 GCMs downscaled to 1/16 degree (approximately 6 km) spatial resolution (**Figure B9-4**) using the Localized Constructed Analogs (LOCA) method and an observed baseline period from 1950 to 2000. The UoI data source downscales output from 20 GCMs to 1/24 degree (approximately 4 km) spatial resolution (**Figure B9-5**), but using the Multivariate Adaptive Constructed Analogs (MACA) method and an observed baseline period from 1979 to 2012. Precipitation statistics were calculated using a baseline period from 1950 to 2015. Three future periods were evaluated using 70-year periods: 1980 to 2050, 2005 to 2075, and 2030 to 2100. Seasonal precipitation statistics as well as a frequency analysis are calculated. The frequency analysis fits a partial duration series for each period to a Generalized Pareto distribution.

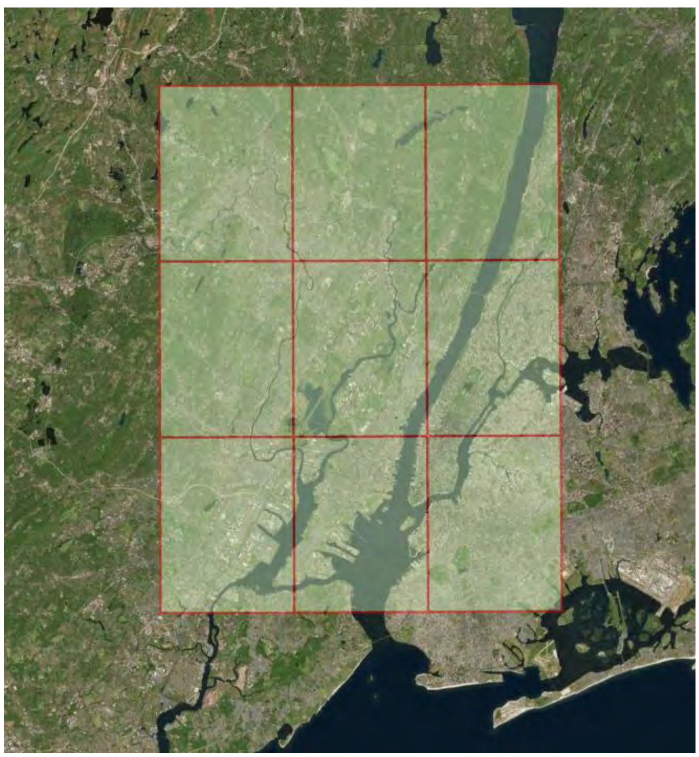

Figure B9-3: Bureau of Reclamation Grid (BCCA Method)

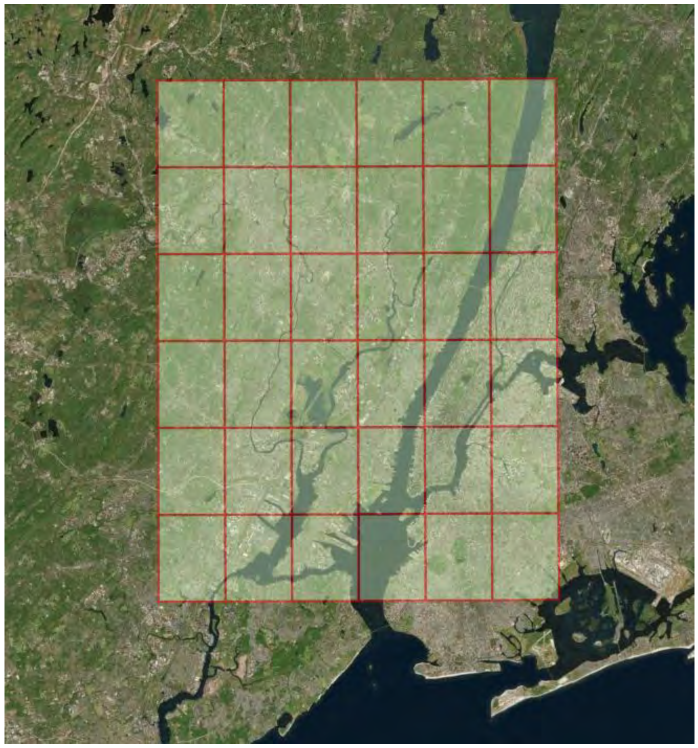

Figure B9-4: Bureau of Reclamation Grid (LOCA Method)

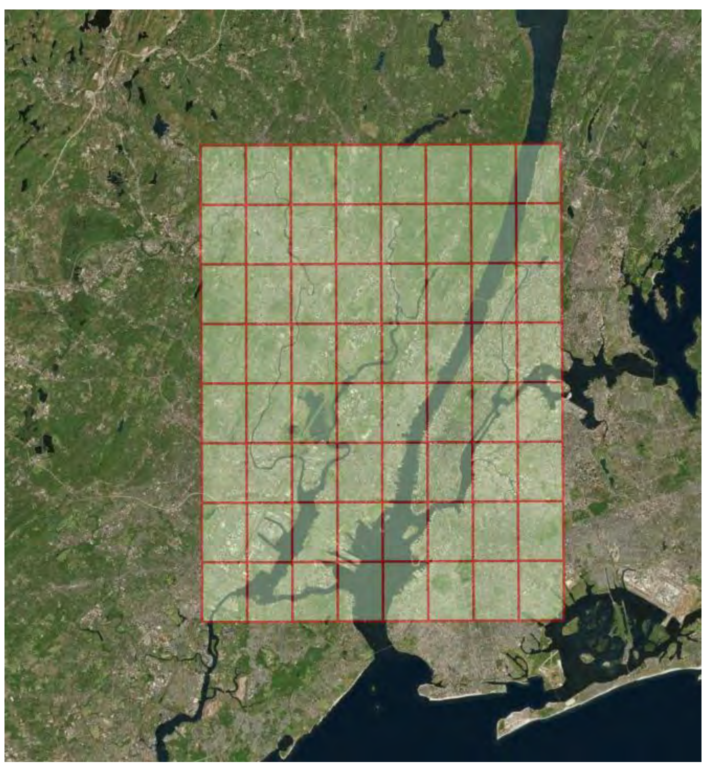

Figure B9-5: University of Idaho Grid (MACA Method)

1.3.2.3 Seasonal Precipitation

A comparison of projected changes to average total seasonal precipitation is shown in **Table B9-3**. A comparison of the NPCC statistics to the FLEx Tool results for the UoI and BoR data sources yields similar results. Increases in precipitation are greatest in the Winter and Spring (+3 percent to +25 percent) while Summer and Autumn are projected to experience small increases or possibly decreases in total seasonal precipitation (-5 percent to +18 percent).

Table B9-3: Comparison of Projected Percent Changes to Seasonal Precipitation

Century	Season	NPCC	BoR (BCCA)	BoR (LOCA)	UoI
		25th to 75th Percentile	Ensemble Mean	Ensemble Mean	Ensemble Mean
Mid-Century	Autumn	+1% to +10%	3%	2%	2%
	Winter	+7% to +18%	6%	5%	4%
	Spring	+3% to +12%	5%	4%	5%
	Summer	-5% to +11%	4%	2%	3%
Late Century	Autumn	-1% to +11%	5%	5%	5%
	Winter	+10% to +25%	16%	17%	13%
	Spring	+4% to +15%	13%	13%	11%
	Summer	-5% to +18%	9%	5%	5%

1.3.2.4 Frequency Analysis

Frequency analyses were performed using the BoR and UoI data sources. Partial duration series (PDS) from Baseline (1950 to 2015), Mid-Century (1980 to 2050), and Late Century (2030 to 2100) epochs were developed by selecting the top N events within the selected date range where N is equal to the number of years. The PDS were fit to the Generalized Pareto distribution. Only 24-hour data is available; thus, all return period storms considered in this analysis have 24-hour duration. **Table B9-4** shows the range (+/- one standard deviation of models) of percent changes in precipitation intensity from the Baseline epoch for Mid-Century and Late Century.

Table B9-4: Comparison of Percent Change Ranges (+/- one Standard Deviation of models) from Baseline for Return Period Storms

2050						
Return Period (Years)	2	5	10	25	50	100
Bureau of Reclamation (BCCA Method)	-4% to +11%	-12% to +15%	-11% to +13%	-12% to +14%	-20% to +20%	-37% to +34%
Bureau of Reclamation (LOCA Method)	-7% to +13%	-12% to +15%	-11% to +13%	-9% to +12%	-12% to +15%	-19% to +23%
University of Idaho (MACA Method)	-4% to +13%	-3% to +15%	+2% to +15%	+6% to +23%	0% to +44%	-21% to +90%
2075						
Return Period (Years)	2	5	10	25	50	100
Bureau of Reclamation (BCCA Method)	0% to +11%	-9% to +17%	-8% to +15%	-9% to +16%	-19% to +27%	-46% to +56%
Bureau of Reclamation (LOCA Method)	-6% to +22%	-11% to +26%	-9% to +23%	-10% to +23%	-16% to +28%	-29% to +42%
University of Idaho (MACA Method)	+4% to +15%	+6% to +17%	+11% to +20%	+15% to +31%	+13% to +50%	0% to +89%
2100						
Return Period (Years)	2	5	10	25	50	100
Bureau of Reclamation (BCCA Method)	+6% to +11%	-3% to +16%	-4% to +15%	-6% to +16%	-15% to +22%	-33% to +37%
Bureau of Reclamation (LOCA Method)	+8% to +18%	+8% to +18%	+7% to +17%	+5% to +16%	-2% to +21%	-14% to +30%
University of Idaho (MACA Method)	+8% to +16%	+9% to +19%	+13% to +22%	+17% to +32%	19% to +44%	14% to +68%

The BCCA method yields the lowest increases while the MACA the greatest. It has been shown that BCCA generally underpredicts precipitation as well as extreme events (Gutmann, et al. 2014). Additionally, the MACA method used to downscale the UoI model output has been shown to better simulate precipitation, particularly extreme events (Pierce, Cayan and Thrasher 2014). However, all data sources confirm a positive trend in future precipitation as well as increased uncertainty for higher return period events.

As with any attempt to predict future events, there is significant uncertainty in all of the results presented in this subappendix. Moreover, extreme events (i.e., higher return period storms) are less certain in any frequency analysis. **Figure B9-6** provides a graphical representation of the relationships between the intensity of precipitation events and their frequency for existing conditions, as well as Mid-Century and Late Century for the MACA method results. One standard deviation on either side of the mean values are represented by the shaded areas as a measure both of the spread associated with the different models considered as well as the implied uncertainty.

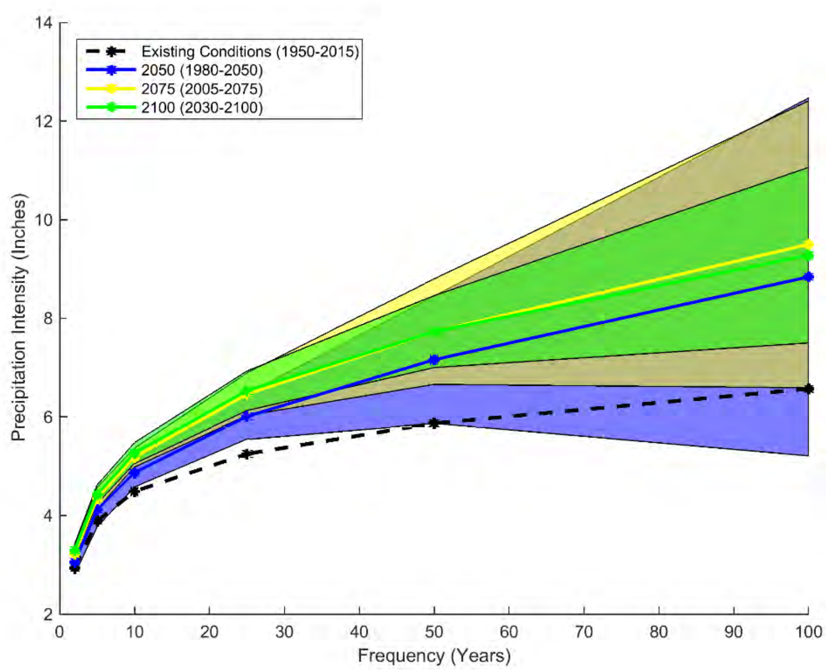

Figure B9-6: UoI (MACA Method) Frequency Analysis Results with Error Bands (+/- Standard Deviation)

1.4 Recommendations

1.4.1 Sea Level Change Estimates for the Meadowlands District

As described in **Section 1.1.2**, the Hackensack River is strongly influenced by the tides and, thus, will be affected by rising sea levels. Although the projections presented in **Section 1.2.1** are for the Battery tide station in Lower Manhattan, it is conservatively assumed in this report that the Hackensack River will experience equivalent sea level changes to the Battery. While this is a reasonable assumption for planning, in actuality, the changes experienced in the Project Area could be somewhat muted or amplified relative to the Battery. Hydrodynamic modeling would be necessary to determine the exact nature of this difference and this is currently outside of the scope of this analysis.

NOAA's Intermediate-High and Intermediate-Low curves were selected by the New Jersey Department of Environmental Protection (NJDEP) for consideration in the Project Area in order to account for SLC, and 2075 was selected as the relevant planning time horizon. **Table B9-5** shows the projected SLC values relative to 2015 for the year 2075; the years 2050 and 2100 are also included to provide some context for the different SLC rates represented by each curve. The grey rows were not selected for consideration in this study.

Table B9-5: Sea Level Change Estimates Relative to 2015

Source	2050	2075	2100
NOAA HIGH	1.8 ft	3.8 ft	6.5 ft
NPCC 2015 90TH PERCENTILE	1.8 ft	3.4 ft	5.3 ft
USACE HIGH (MODIFIED NRC CURVE III)	1.4 ft	2.9 ft	5.0 ft
NPCC 2015 75TH PERCENTILE	1.3 ft	2.4 ft	3.6 ft
NOAA INTERMEDIATE-HIGH	1.1 ft	2.4 ft	4.0 ft
NOAA INTERMEDIATE-LOW / USACE INTERMEDIATE (MODIFIED NRC CURVE II)	0.6 ft	1.2 ft	1.8 ft
NPCC 2015 25TH PERCENTILE	0.6 ft	1.1 ft	1.6 ft
NPCC 2015 10TH PERCENTILE	0.5 ft	0.8 ft	1.1 ft
NOAA LOW / USACE LOW (MODIFIED NRC CURVE I)	0.3 ft	0.6 ft	0.8 ft

Note that the selected curves are generally towards the middle of the range of estimates available as shown in **Figure B9-2** and **Table B9-5** and are quite similar to the NPCC's 25th and 75th percentile estimates for the region, particularly in 2075. While higher and lower estimates are available, the range of selected SLC values is a good representation of the most likely changes projected to occur in the Project Area.

1.4.2 Precipitation Estimates for the Meadowlands District

The MACA method values were selected for use in future conditions modeling. The MACA values were the most conservative (greatest change) and have been shown to possibly capture extremes in precipitation more realistically. These values will be used for planning purposes only as they continue to be significantly uncertain, but they will provide an important window into the possibility of future precipitation trends for the Project Area (see **Table B9-6**).

Table B9-6. Future Conditions Precipitation Changes for RBDM

Return Period	2-year	5-year	10-year	25-year	50-year	100-year
University of Idaho (MACA Method)	10%	12%	16%	23%	32%	45%

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Subappendix B10: Fluvial Model Output

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FINAL

Subappendix B10 – Alternative 2 and Alternative 3 Model Output

For the Feasibility Study of Rebuild by Design Meadowlands Flood Protection Project

May 2021



Boroughs of Little Ferry, Moonachie, Carlstadt, and Teterboro and the Township of South Hackensack, Bergen County, New Jersey

REBUILD BY DESIGN
MEADOWLANDS

Prepared by AECOM for the State of New Jersey Department of Environmental Protection

Subappendix B10 –

Alternative 2 and Alternative 3 Model Output

See Disc Submitted along with the Feasibility Report