Prepared for

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Hydrologic and Hydraulic Model Report

Active Water Management in Lake Mattamuskeet

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1. INTRODUCTION

This report was prepared by Geosyntec Consultants of NC, P.C. (Geosyntec) and Coastal Protection Engineering, LLC (CPE) for the North Carolina Coastal Federation, Inc. (federation) to summarize the results of a numerical model developed to evaluate potential engineering modifications in the Lake Mattamuskeet watershed. This study was funded by a grant award from the N.C. Clean Water Management Trust Fund (CWMTF) Project No. 2019-804, which awarded Hyde County funds to contract with federation and the selected engineering team to provide professional services and develop engineered plans for active water management within the Lake Mattamuskeet watershed. The purpose of the numerical modeling study conducted was to evaluate potential engineering modifications to the drainage network to both improve water management capabilities within the Lake Mattamuskeet watershed and reduce flooding.

Lake Mattamuskeet is located in the Mattamuskeet National Wildlife Refuge on the Albemarle-Pamlico Peninsula in Hyde County, North Carolina (see Figure 1). The 50,180-acre refuge was established in 1934 with the shallow, 40,000-acre Lake Mattamuskeet, North Carolina's largest natural lake, as its centerpiece. The lake is rich in history and beauty, and its recorded history dates back to July 11, 1585, when 60 English explorers from Sir Walter Raleigh's Roanoke Island expedition visited the lake (Lake Mattamuskeet Foundation, Inc.). The lake bed is a few feet below sea level and is effectively a fresh wetlands depression that fills with rainwater and runoff from the surrounding land. Currently, the Lake Mattamuskeet community faces flooding issues and declining water quality, which is threatening the ecology of the lake system. The Lake Mattamuskeet Watershed Restoration Plan (NCCF, 2018) speaks to the need to protect the way of life in Hyde County, reduce flooding (including actively managing the lake water level), and restore water quality. Therefore, the overarching goal of this study was to identify practical active water management options to reduce the risk of flooding within the watershed while improving the water quality and clarity in the lake.

1.1 Study Area

Lake Mattamuskeet is located in the Mattamuskeet National Wildlife Refuge on the Albemarle-Pamlico Peninsula in the outer coastal plain of eastern North Carolina. The normal average depth of Lake Mattamuskeet is three feet, and the lake bottom is below sea level, ranging from approximately -2.0 to -5.2 ft, North American Vertical Datum of 1988 (NAVD 88) (Moorman *et al.,* 2017). The bottom of the lake is also lower than average water levels in nearby Pamlico Sound. The lake is divided into two basins by NC Highway 94, which was completed in 1942; the east basin and west basin are connected by a series of box culverts under the road. The lake is managed by the U.S. Fish and Wildlife Service to attract and provide habitat for migratory waterfowl and other bird species utilizing the Atlantic Flyway. Surrounding the lake are several drainage districts that utilize pumps to move water from low lying land through canals to the Pamlico Sound; however, farming and residential communities within the lake watershed but outside of the drainage districts experience frequent hot spot flooding during both small and large rain events or wind-driven tides within the lake. Extensive hydromodification within the lake watershed over the years has also led to eutrophication and loss of natural hydrologic pathways.

Water inflow to the lake is primarily dominated by precipitation, and outflow from the lake is primarily dominated by evaporation. Additional inflows into the lake include agricultural drainage ditches connected to the lake. Outputs from the lake include four outflow canals. The Rose Bay Canal is located on the western basin of the lake; the other three canals (Outfall Canal, Lake Landing Canal and Waupoppin Canal) are in the eastern basin of the lake (Figure 1.1). Groundwater inputs and outputs are considered negligible based on work by Heath (1975). The four outfall canals connecting Lake Mattamuskeet to the Pamlico Sound are equipped with tide gates that have been installed and maintained by the U.S. Fish and Wildlife Service. The number and orientation of the tide gates (e.g., top-hinged vs. side-hinged) varies depending on the canal.All of the gates were designed to open as a result of positive head pressure when water levels are higher in the lake than the Pamlico Sound. The purpose of the tide gates is to prevent the inflow of saltwater from the Pamlico Sound to Lake Mattamuskeet, keeping the lake a primarily freshwater system (Moorman, 2018a).

1.2 Study Objectives

The primary objective of this study was to evaluate active water management options to reduce water levels within the lake watershed under both existing and relative sea level rise (RSLR) scenarios. During extreme storm events, the water level in the lake rises to flood stages, causing flooding around the lake shore and inhibiting drainage of adjacent properties to the lake. The lake then gradually draws down via the outflow canals, but the time for the lake water level to return to pre-storm levels can in some cases take over a month. This study evaluates engineered alternatives to accelerate the drawdown of the lake and reduce peak water levels.

A watershed-scale hydrologic and hydraulic (H&H) model was developed to simulate water elevations within the lake as well as the flow of surface water throughout the watershed. The scope of work required model calibration with named storms such as Hurricane Matthew and Hurricane Joaquin. These two storms were used for calibration. To evaluate the baseline hydrologic response of the lake without any engineering modifications, the calibrated model was simulated for a series of design storm events (2-, 5-, 10-, 25-, 50- and 100-year design storms) under existing and RSLR scenarios (NOAA, 2017). The model was then used to evaluate several potential modifications to the complex drainage network within the watershed. Eight design alternatives were screened using the 10-year design storm to understand their impact on the drawdown of lake water levels compared to the existing scenario. Based on these results, two preferred engineering alternatives were simulated for the full suite of design storm simulations.

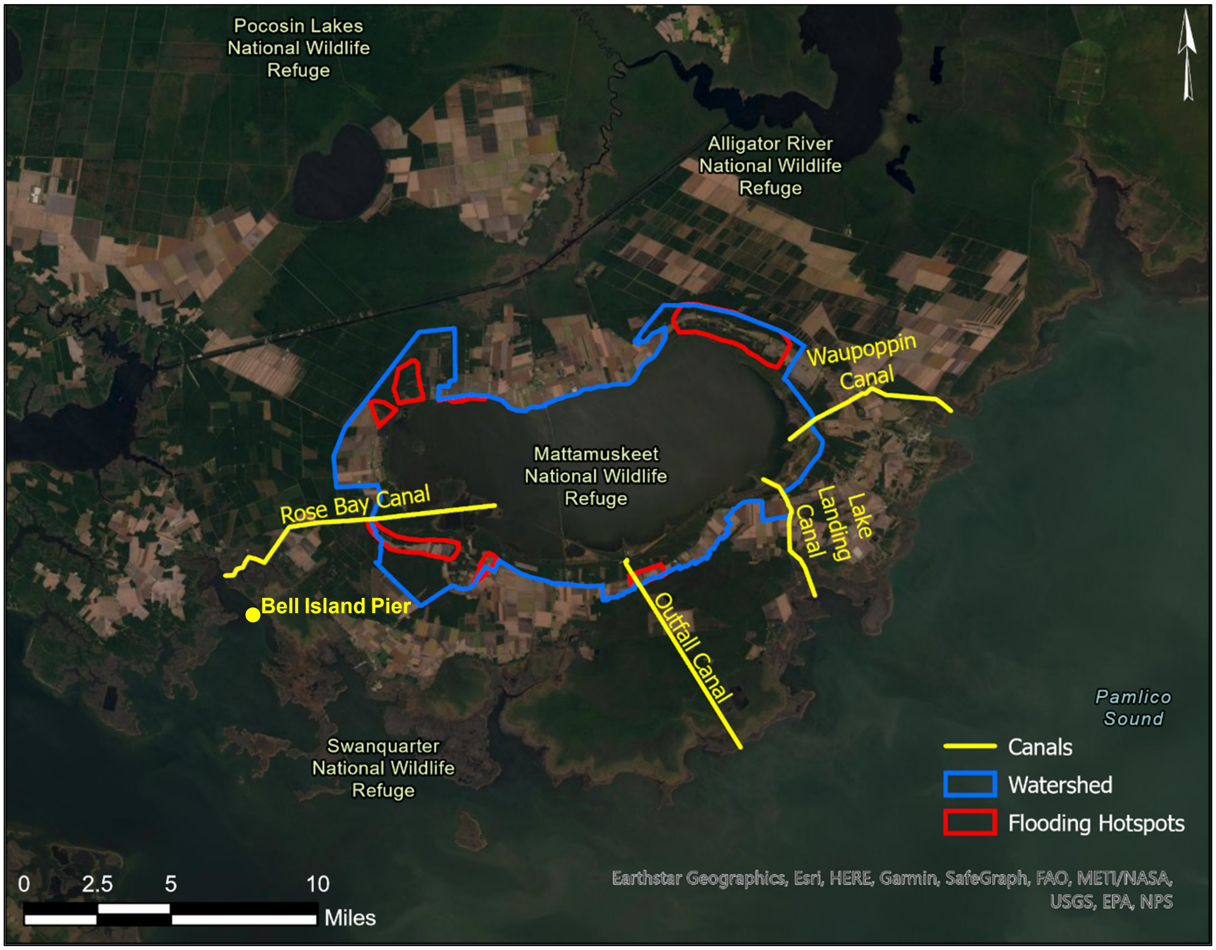

Figure 1.1 Lake Mattamuskeet study area.

2. NUMERICAL MODEL INPUTS, CALIBRATION, AND VALIDATION

As previously described, this modeling study focuses on the Lake Mattamuskeet watershed, which comprises a total area of 68,173 acres. The study utilizes Delft3D Flexible Mesh (FM), a hydrologic and hydrodynamic simulation program developed by TU Delft/Deltares (Deltares, 2021) which is part of an integrated model suite for multi-disciplinary 1D, 2D, and 3D computations for coastal, river, and estuarine areas. Delft3D FM incorporates hydrologic processes such as precipitation, infiltration, and evaporation and allows for implementation of hydraulic structures such as weirs, gates, and pumps. The set-up of the computational grids, input data used in the model, and model calibration are described in Sections 2.1 through 2.4.

2.1 Computational Grid

A large, unstructured, model grid was created to simulate the flow and circulation patterns in the lake and surrounding areas (Figure 2.1). This grid covers the entire watershed along with the four major outlet canals and has close to 300,000 computational cells. The grid varies in resolution for different areas of the domain. The resolution of the cells in the lake is around 650 ft where the elevation is consistent in the center of the Lake, in order to reduce computational time while still providing accurate representation of the lake water levels, whereas the resolution of the cells within the outlet canals ranges from 15-25 ft. The higher resolution in the outlet canals is necessary to resolve the canal topography properly in the model and estimate flow through the canals. The gates on the canals were modeled as gravity structures that open completely when there is a positive water level gradient between the lake and Pamlico Sound. The modeling domain includes the existing watershed boundary, including the current flooding hotspots, and Lake Mattamuskeet's four existing hydraulic connections to Pamlico Sound. The model grid on top of the interpolated DEM (Digital Elevation Model) is shown in Figure 2.2. A detailed view of the grid at the entrance of Rose Bay Canal is shown in Figure 2.3.

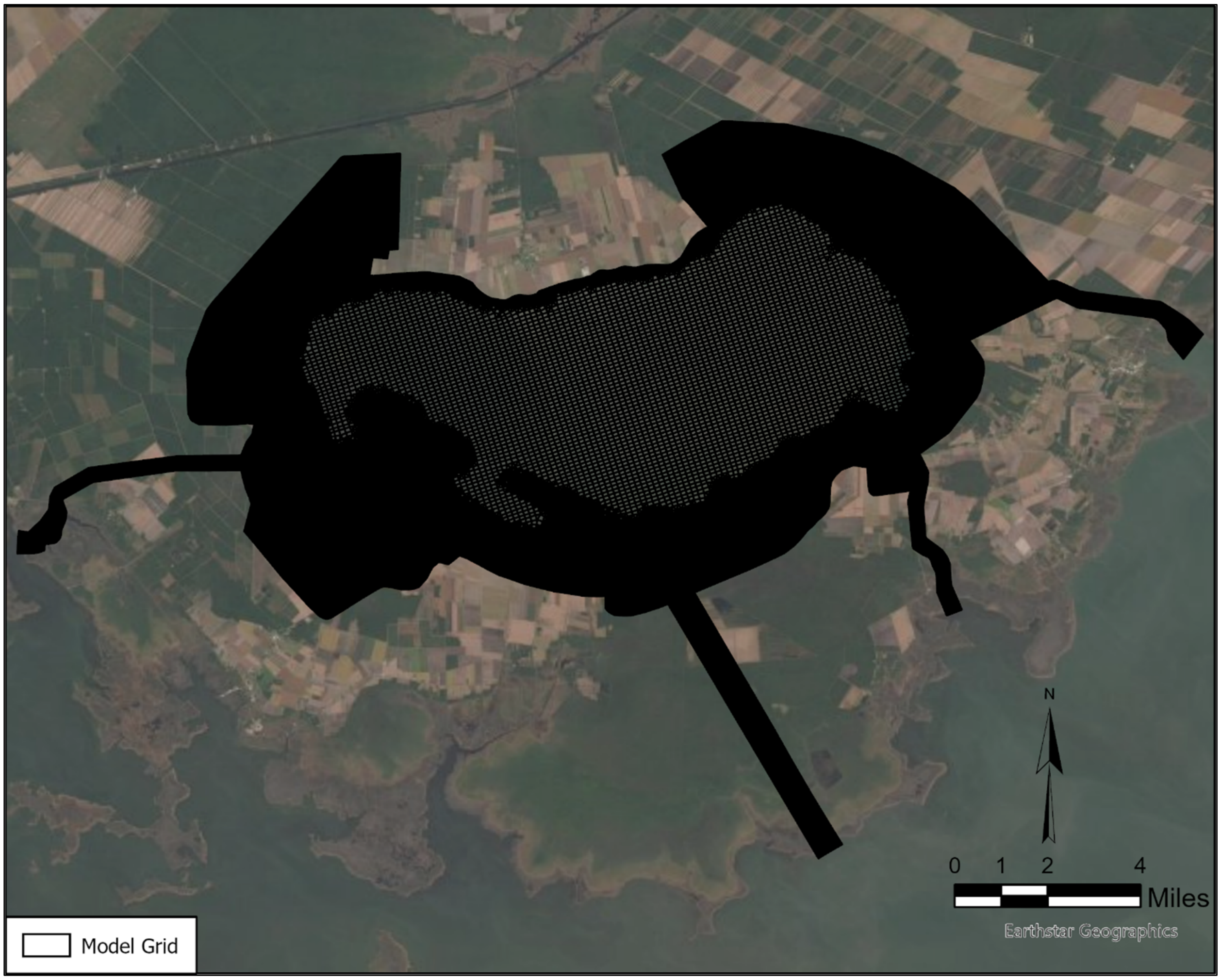

Figure 2.1 Lake Mattamuskeet model grid. Background is an aerial image of the study area. Black areas represent the portions of the model domain with a finer grid resolution; hatched areas represent the portions of the model domain with a coarser grid resolution.

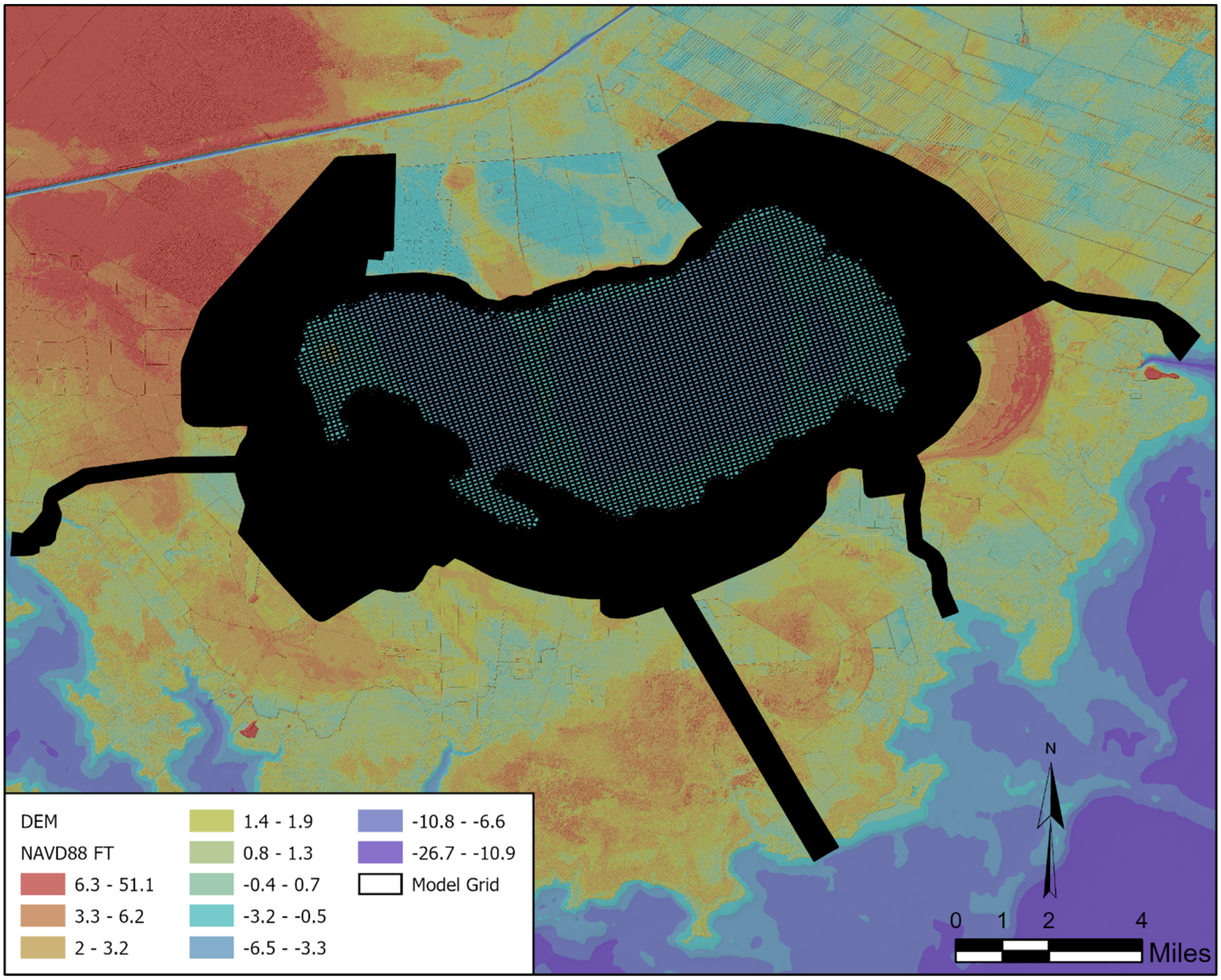

Figure 2.2. Lake Mattamuskeet model grid. Background is a color shaded relief DEM with cool colors representing low elevations and warm colors representing higher elevations.

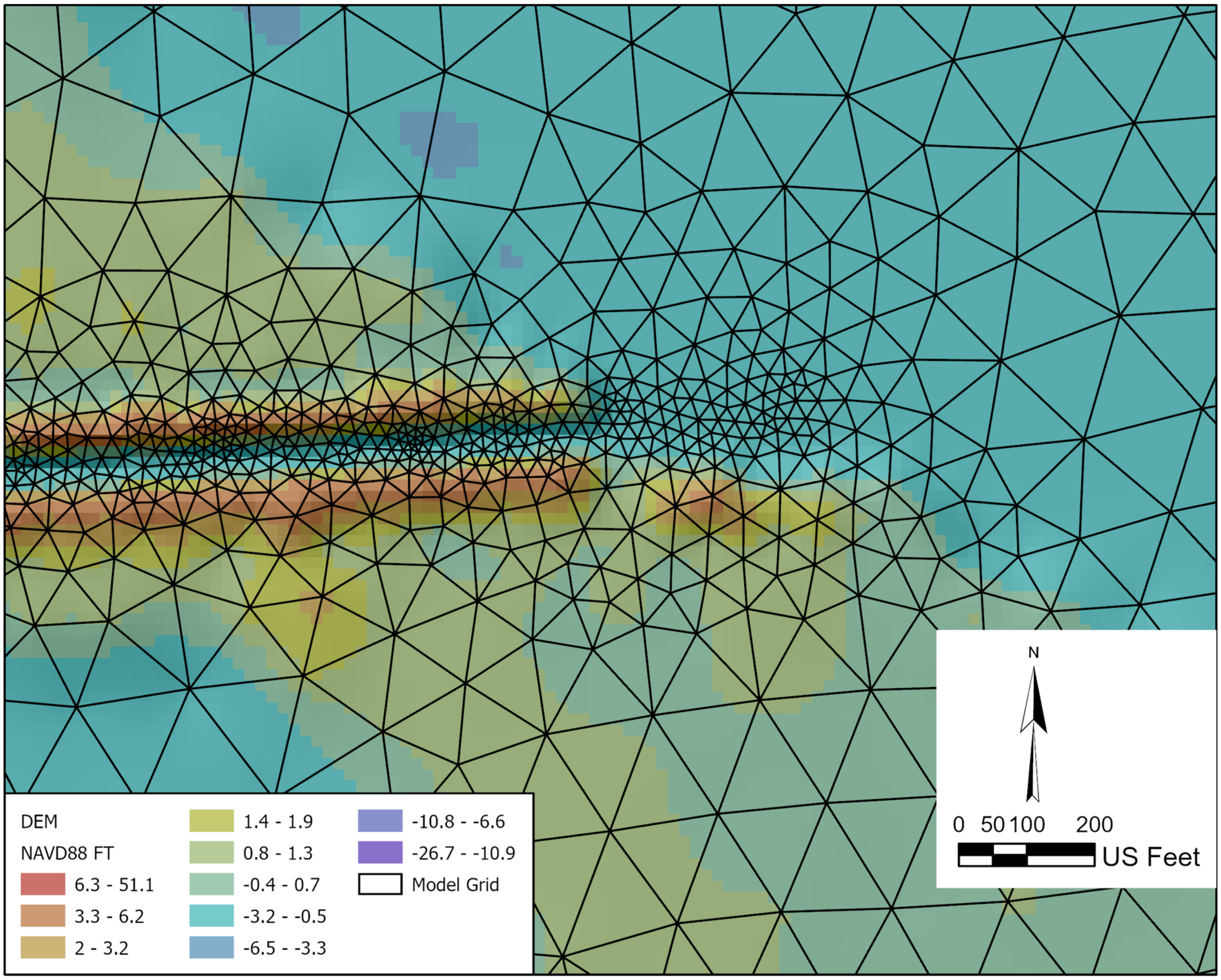

Figure 2.3. Detailed view of the model grid at the entrance of Rose Bay Canal overlaid on color shaded relief DEM.

2.2 Topography and Bathymetry

Topography and bathymetry data utilized in this study is relative to the NAVD 88 vertical datum. For consistency purposes, water level data are also presented relative to the same vertical datum. Water level measurements conducted by the United States Geological Survey (USGS) at the east and west basins of the lake are frequently presented relative to the USGS gauge datum, which is +2 ft above NAVD 88. Therefore, 0 ft NAVD 88 is equal 2 ft gauge, 1 ft NAVD 88 is equal to 3 ft gauge, -1 ft NAVD 88 is equal to 1 ft gauge and so on, as shown in Figure 2.4

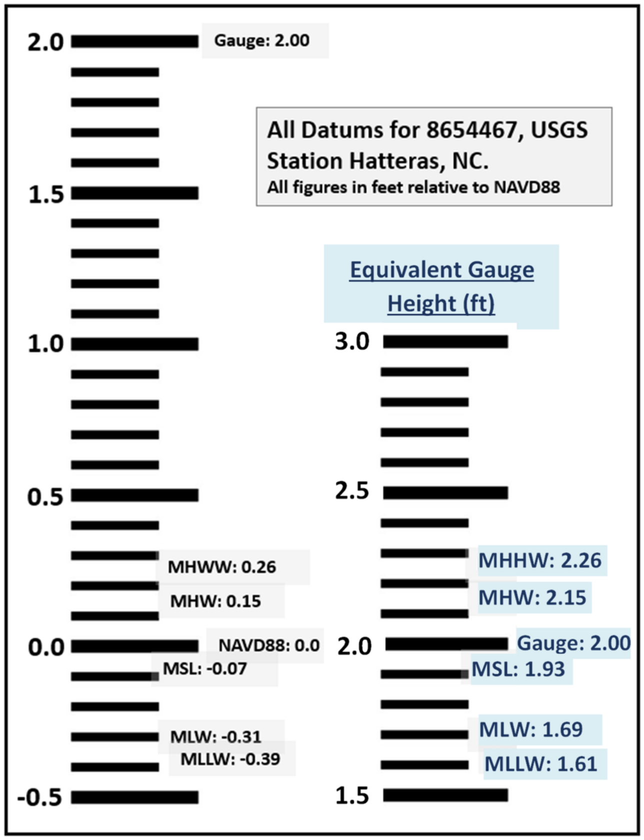

Figure 2.4. Comparison of NAVD 88 and gauge height datums for mean higher-high water (MHHW), mean high water (MHW), mean sea level (MSL), mean low water (MLW), and mean lower-low water.

The topographic and bathymetric (topobathy) digital elevation model (DEM) was created using a combination of bathymetric soundings and statewide topographic lidar. Bathymetry data were provided by the United States Geological Survey (USGS) using tidally corrected soundings, relative to NAVD 88, at more than 500 locations in the lake collected over multiple surveys between 2013 and 2016. Bathymetry data for major waterways including the Intracoastal Waterway and Pamlico Sound were derived from NOAA NCEI Continuously Updated Digital Elevation Model (CUDEM) ninth arc second data downloaded from NOAA's Digital Coast (NOAA, 2021). The North Carolina statewide lidar data were downloaded from North Carolina's Spatial Data Download (NCSDD, 2021). Lidar data were topographic. NC statewide lidar data were downloaded at 5-foot resolution. Water bodies were masked out by generating polygons based on the 0 contour. Small canals throughout the watershed where no bathymetry data existed were hydro flattened to represent bathymetric depths of -2 feet NAVD 88. The final DEM used in the model is shown in Figure 2.5.

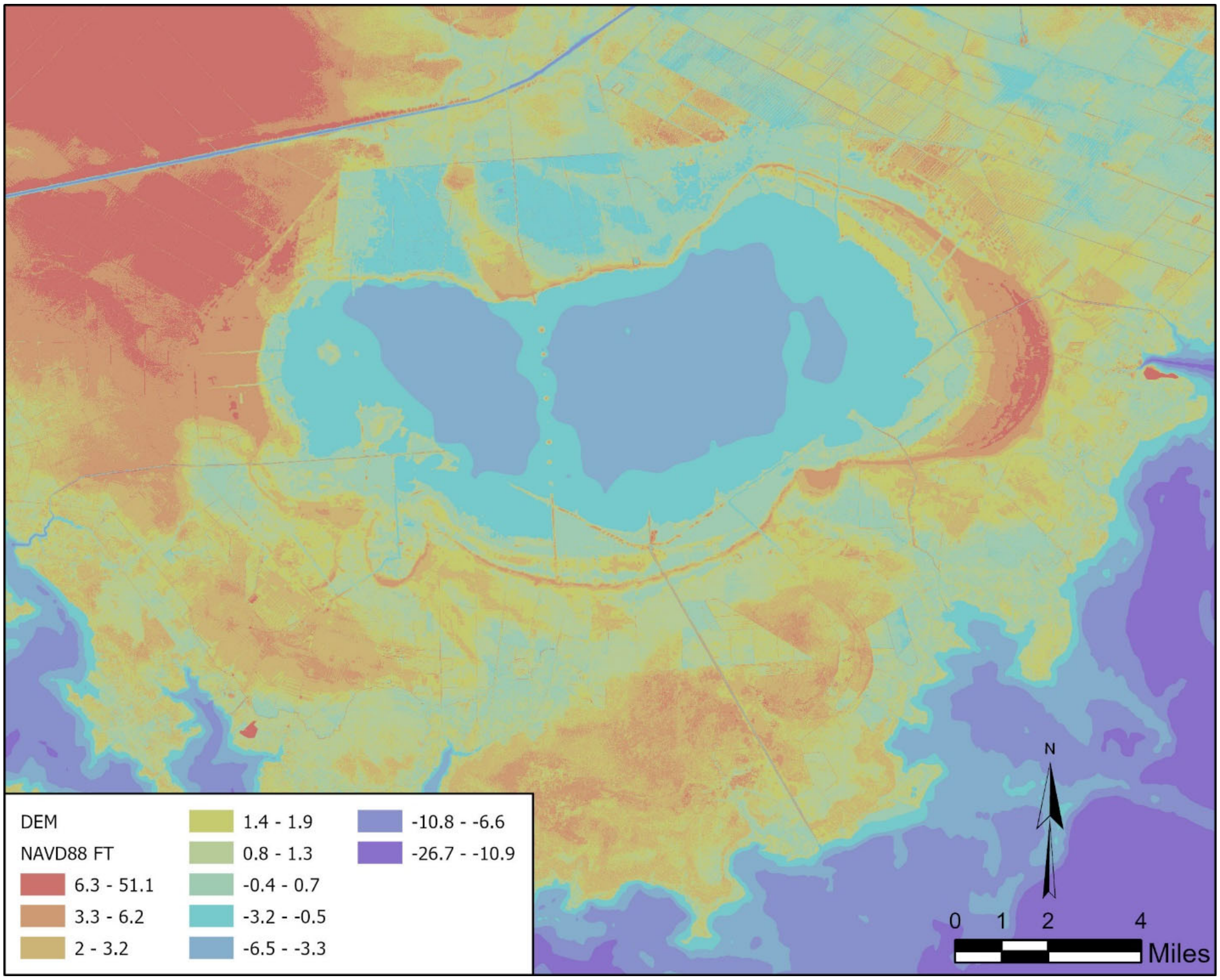

Figure 2.5. Color Shaded Relief DEM created using several sources with blue and red as low and high elevations, respectively.

2.3 Boundary Conditions

Input data for the model were obtained from publicly available sources. Existing data of water level, net precipitation, and wind speed were used as boundary conditions for the model.

Sources of input data are as follows:

- **Water level** - Water level boundary conditions for the four canals were obtained from the water levels recorded at Bell Island Pier near the Rose Bay canal outlet to Pamlico Sound (Figure 1.1). The measurement station is operated as part of the Refuge Inventory and Monitoring program through cooperation with Mattamuskeet NWR and NCSU University. The Bell Island pier water level data, provided to the study team by Dr. Randall Etheridge from East Carolina University, was selected because it was the only long-term dataset that had water level measurements on the sound side.

- **Net precipitation** - Net precipitation (precipitation minus evaporation) data were obtained at a time-step of 1 hour from National Center for Environmental Prediction (NCEP), version 2, coupled forecast system model (CFSv2) (Saha *et al.*, 2014). Net precipitation from the CFSv2 model was utilized due to the absence of long-term hourly precipitation and evaporation data at the study area.
- **Winds** - The model was forced with 10m hourly winds obtained from the Plymouth (PLYM) Tidewater Research Station. The station is part of the North Carolina Environment and Climate Observing Network (ECONet), run by the North Carolina State Climate Office/NC State University (NCSCO, 2021). Winds were utilized from this station as it is the closest station to the study area with long-term hourly wind measurements.

Infiltration and transpiration are considered negligible for the purposes of this study. The outlet structures in the outfall channels were modeled to be controlled by the hydraulic gradient between the lake water level, as measured at the USGS stations in the east and west basins of the lake, and the Pamlico Sound water level, as measured at the Bell Island Pier tide station. The outlet structures in the canals were modeled as a one-way valve structure that was completely open when the lake water levels were higher than sound water levels, and completely closed when the sound water levels were higher than lake water levels.

2.4 Model Calibration and Validation

2.4.1 Storm Events for Model Calibration

Two major storm events of the past decade, Hurricane Matthew and Hurricane Joaquin, were studied and used for model calibration and validation in this study. The storms, and their impact on the study area, are described below.

2.4.1.1 Hurricane Matthew

Hurricane Matthew passed by the coast of North Carolina on October 8, 2016. From October 2 to October 15, 2016, the USGS rain gage on Highway 94 recorded over 7 inches of rain (Figure 2.6). These large amounts of rain came following one of the wettest years on record with over 70 inches of rain falling since October 2015 (Figure 2.7). During the passing of Hurricane Matthew, USGS monitoring stations on the lake showed that lake levels rose from 1.1 to 1.7 ft above the NAVD 88 datum (Figure 2.8). These are some of the highest water levels observed in the lake based on a review of recorded lake water levels from Refuge archives (Moorman, 2018b).

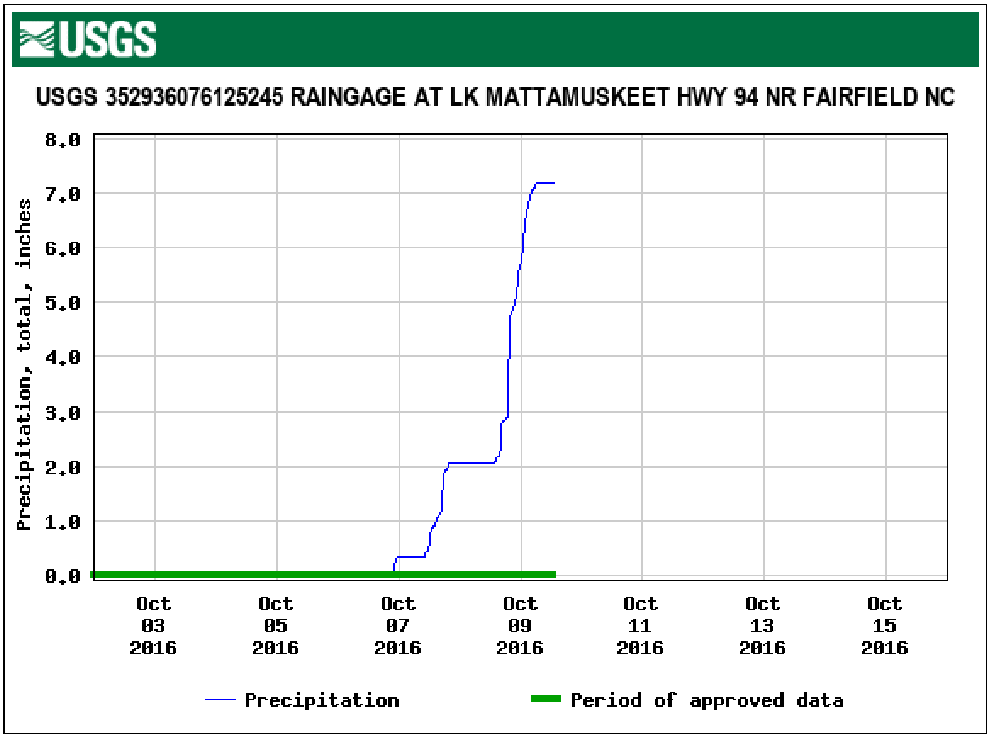

Figure 2.6. Rainfall measurement from October2 to October 15, 2016 at the USGS Hwy 94 rain gauge.

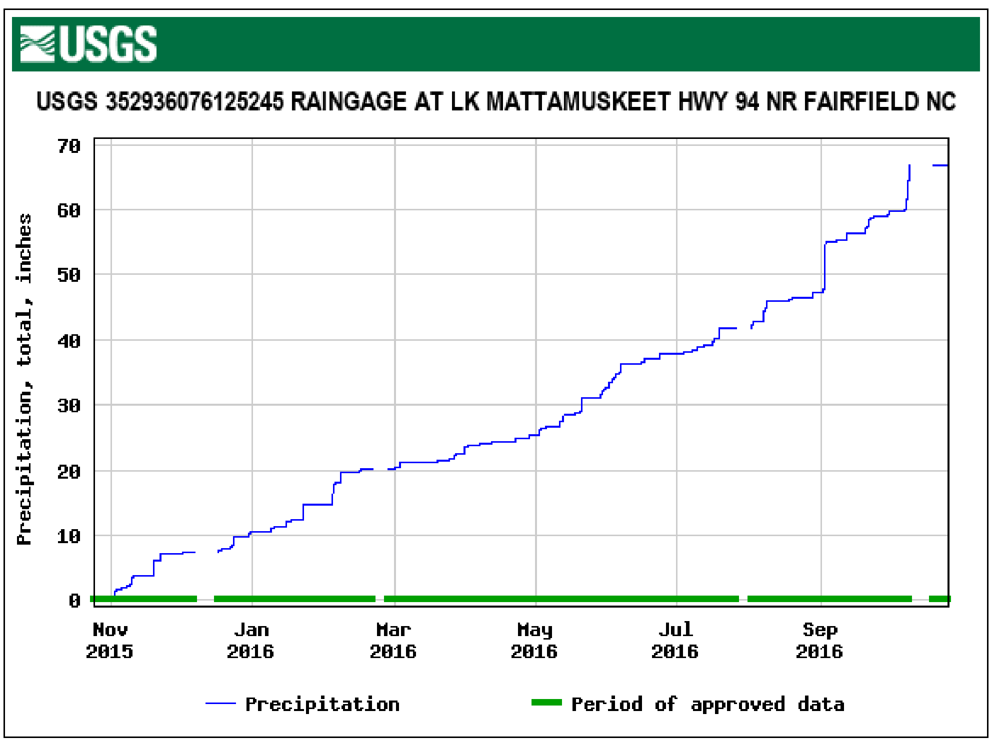

Figure 2.7. Rainfall totals at USGS rain gage between October 25, 2015 and October 25, 2016. 8 inches fell during Hurricane Matthew, this was in addition to the 70+ inches that has fallen since October 2015.

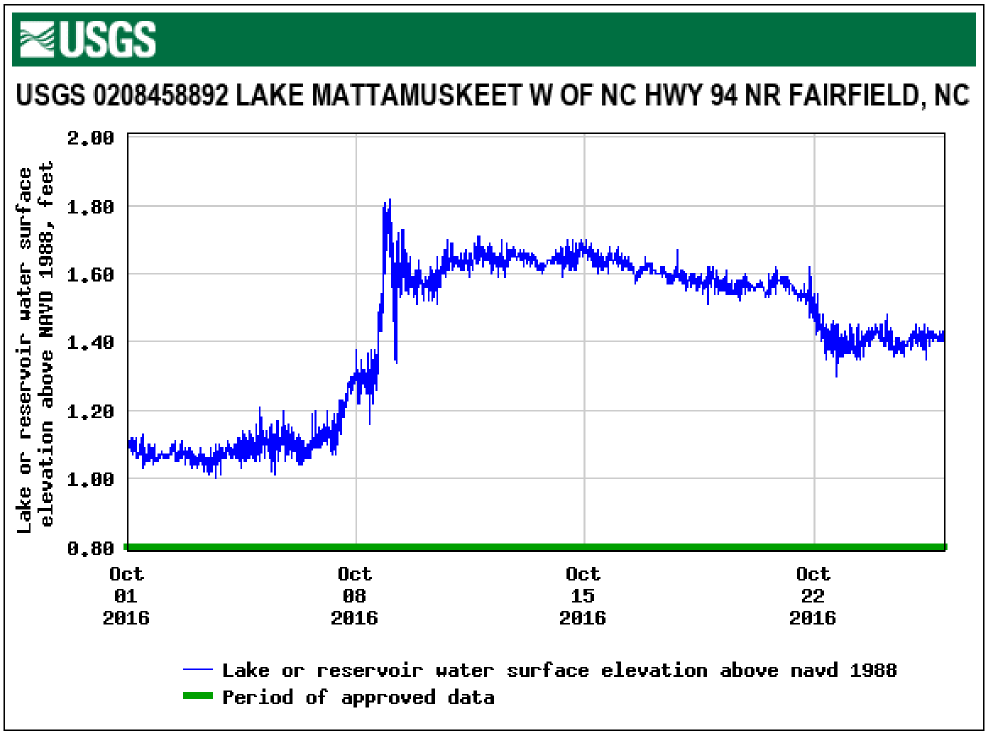

Figure 2.8. Lake Water Levels during Hurricane Matthew. Water levels peaked at 1.8 ft. during the storm.

2.4.1.2 Hurricane Joaquin

Hurricane Joaquin passed by the coast of North Carolina during the first week of October 2015. During this period, the USGS rain gage on Highway 94 recorded over 4.5 inches of rain (Figure 2.9). The weather station at Fairfield recorded 5.15 inches during that same period. During the passing of Hurricane Joaquin (Oct. 1-7, 2015), USGS monitoring stations on the lake showed that lake levels rose from 0.17 ft to 0.7 ft above the NAVD 88 datum on the east basin (Figure 2.10) and 0.37 ft to 0.9 ft above the NAVD 88 datum on the west basin (Figure 2.11). This six-inch increase in lake level added an estimated 16,800-acre feet of water to the lake as a result of Hurricane Joaquin rainfall. This rainfall was in addition to the 5 inches of rainfall that occurred in Hyde County, NC from the period of Sept. 23-Sept. 30, 2015 as a result of a low-pressure system off the coast. This September rain also increased lake water levels by 6 inches. During the last two weeks of September and first week of October, a total of approximately 10 inches of rain fell which raised lake levels by 1.1 feet on the east basin and 1.05 feet on the west basin and added approximately 33,600-acre feet of rainfall to the lake (Moorman, 2018c).

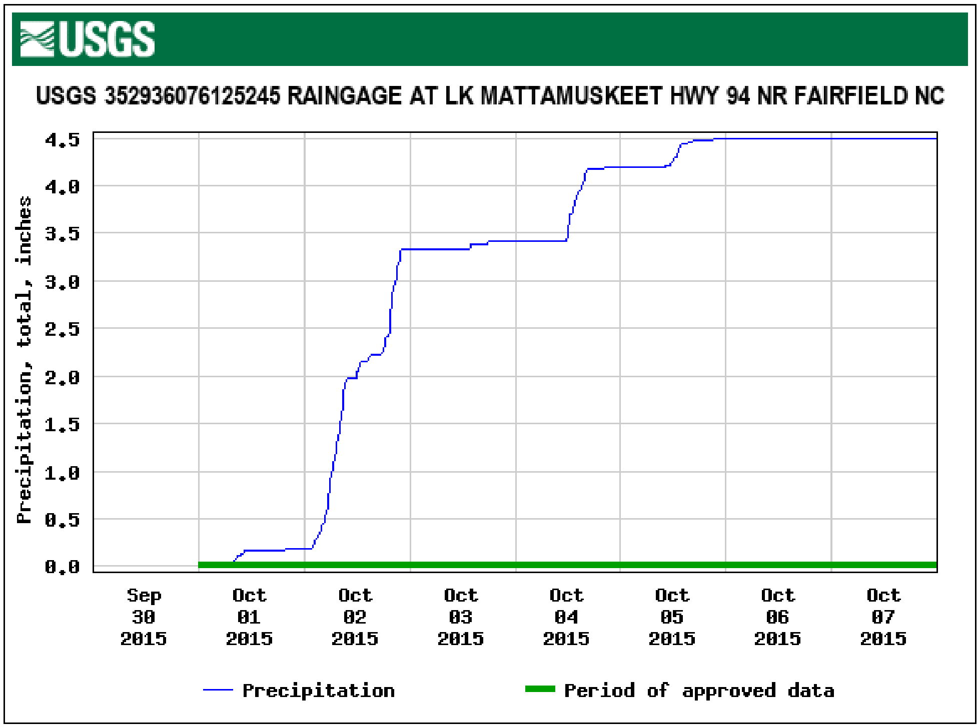

Figure 2.9. Rainfall measured on Lake Mattamuskeet during Hurricane Joaquin, Sept. 30, 2015 – Oct. 7, 2015.

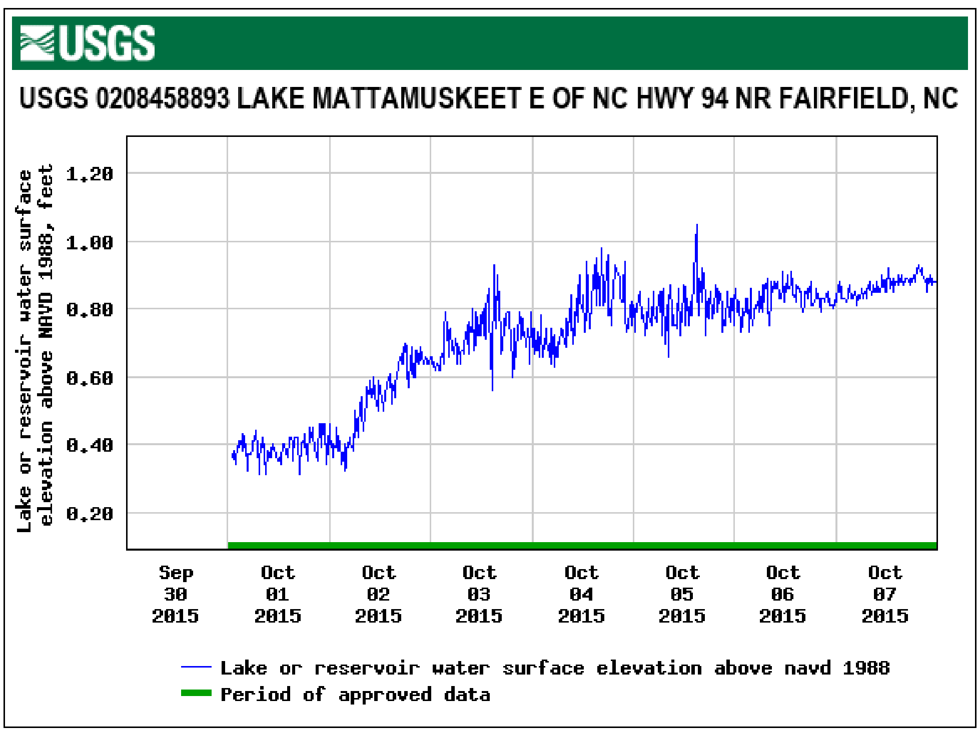

Figure 2.10. Water levels measured on the east side of Lake Mattamuskeet during Hurricane Joaquin, Sept. 30, 2015 – Oct. 7, 2015.

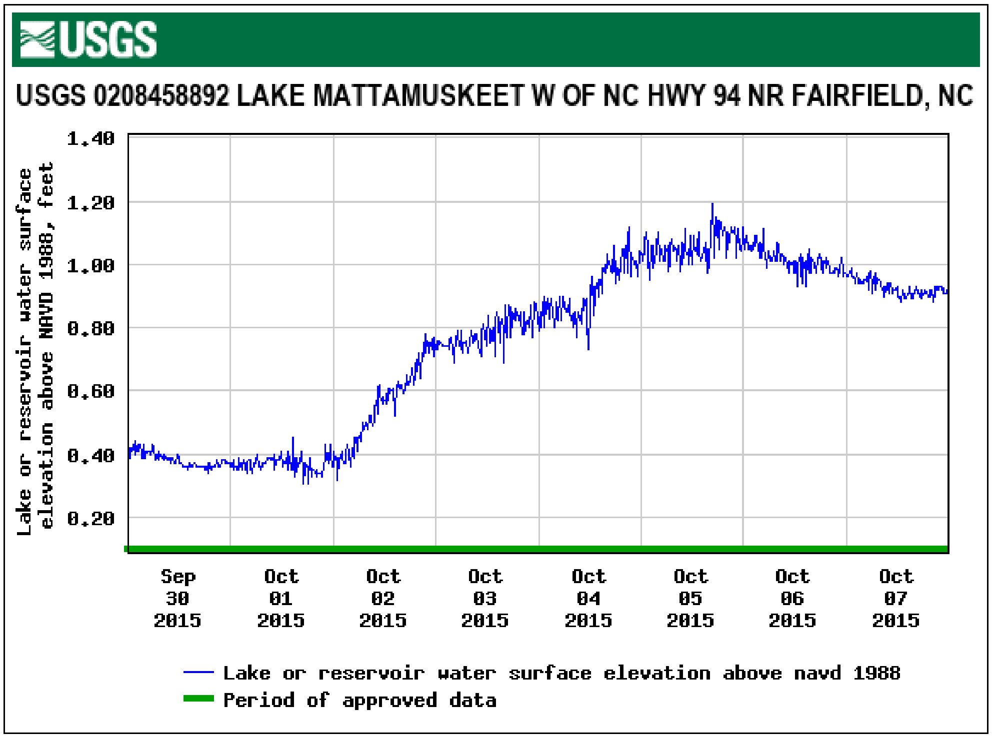

Figure 2.11. Water levels measured on the west side of Lake Mattamuskeet during Hurricane Joaquin, Sept. 30, 2015 – Oct. 7, 2015.

2.4.2 Evaluation of Model Calibration and Validation

Calibration and validation of the model were conducted by comparing simulated and measured water levels from the USGS gauges installed on the east and west side of the lake during Hurricane Matthew and Hurricane Joaquin. A two-month period starting from September 15, 2016 to November 15, 2016 was simulated for Hurricane Matthew. For Hurricane Joaquin, a one-month period from October 1, 2015 to October 31, 2015 was simulated. The calibration simulations consisted of different grid configurations and a sensitivity analysis of the bottom roughness parameter (Manning's coefficient). A higher Manning's coefficient value corresponds with a larger bottom roughness and thus a larger resistance to flow; conversely, a lower value corresponds with smaller roughness and less bed resistance. The Delft3D FM default Manning's coefficient value of 0.023 provided a reasonable agreement between measured and predicted water levels. Changes in the Manning coefficient value were evaluated in a series of calibration runs, and the best match between measurements and model was obtained with a Manning value of 0.015, which is Manning coefficient value typically used in vegetated floodplains (Arcement Jr., and Schneider, 1989). The comparison between measured and modeled water level during Hurricane Matthew for USGS east and west gauges are shown in Figure 2.12. The comparison between measured and modeled water level during Hurricane Joaquin, for USGS east and west gauges, are shown in Figure 2.13. The good agreement between measured and predicted water levels, as illustrated in the figures, demonstrates that the calibrated Delft3D FM model is able to simulate changes in the water level reasonably well within the study area. The small over/under predictions observed are generally order of 1/10th of a foot and are attributed to localized wind effects. Table 2.1 lists the root mean square error (RMSE) and Pearson correlation coefficient for the model calibration simulations. The calculated RMSEs and Pearson correlation coefficients demonstrate a strong correlation between the measured and modeled water levels.

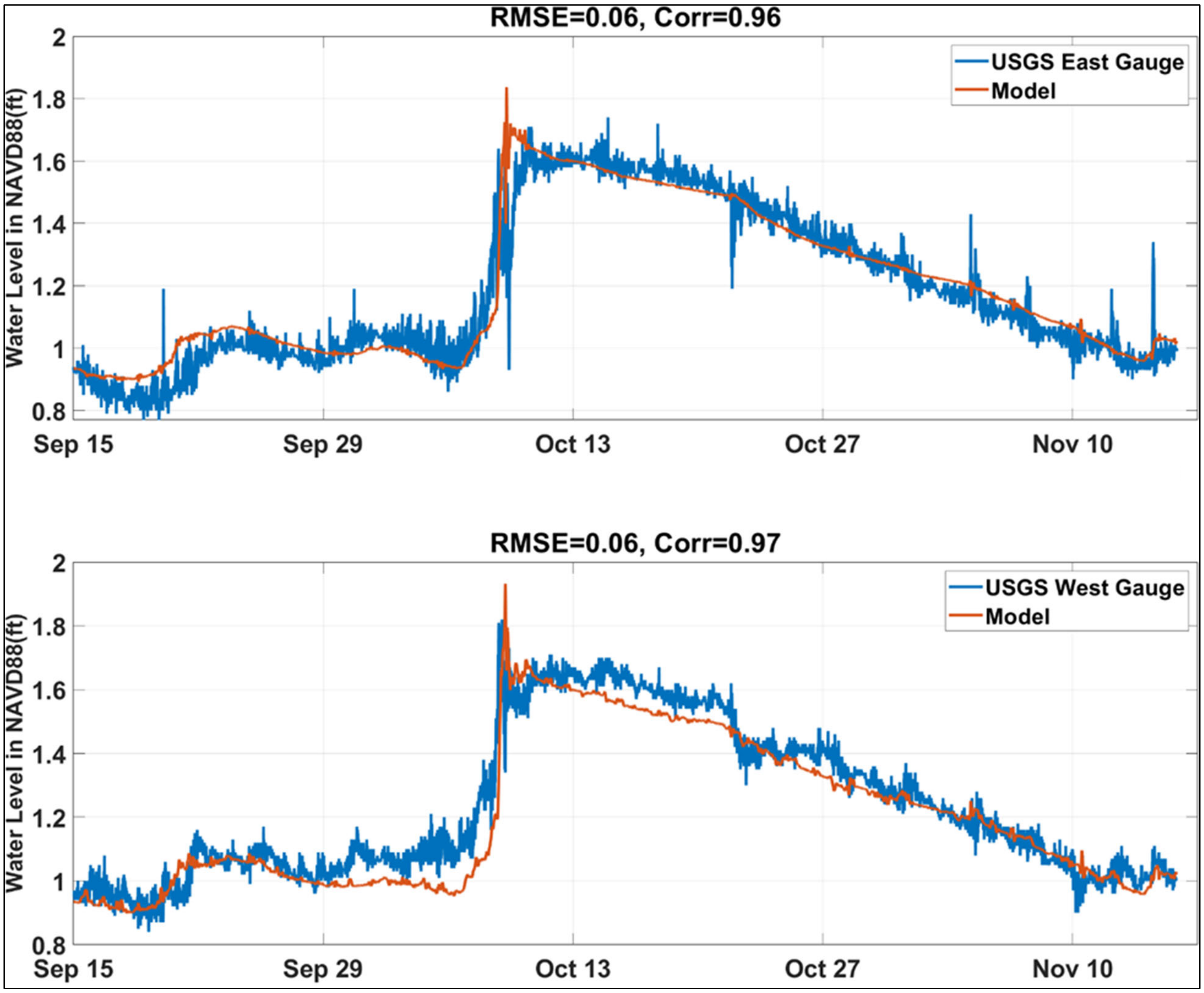

Figure 2.12. Comparison of measured and simulated water levels on the lake during Hurricane Matthew at the east (upper) and west (lower) stations.

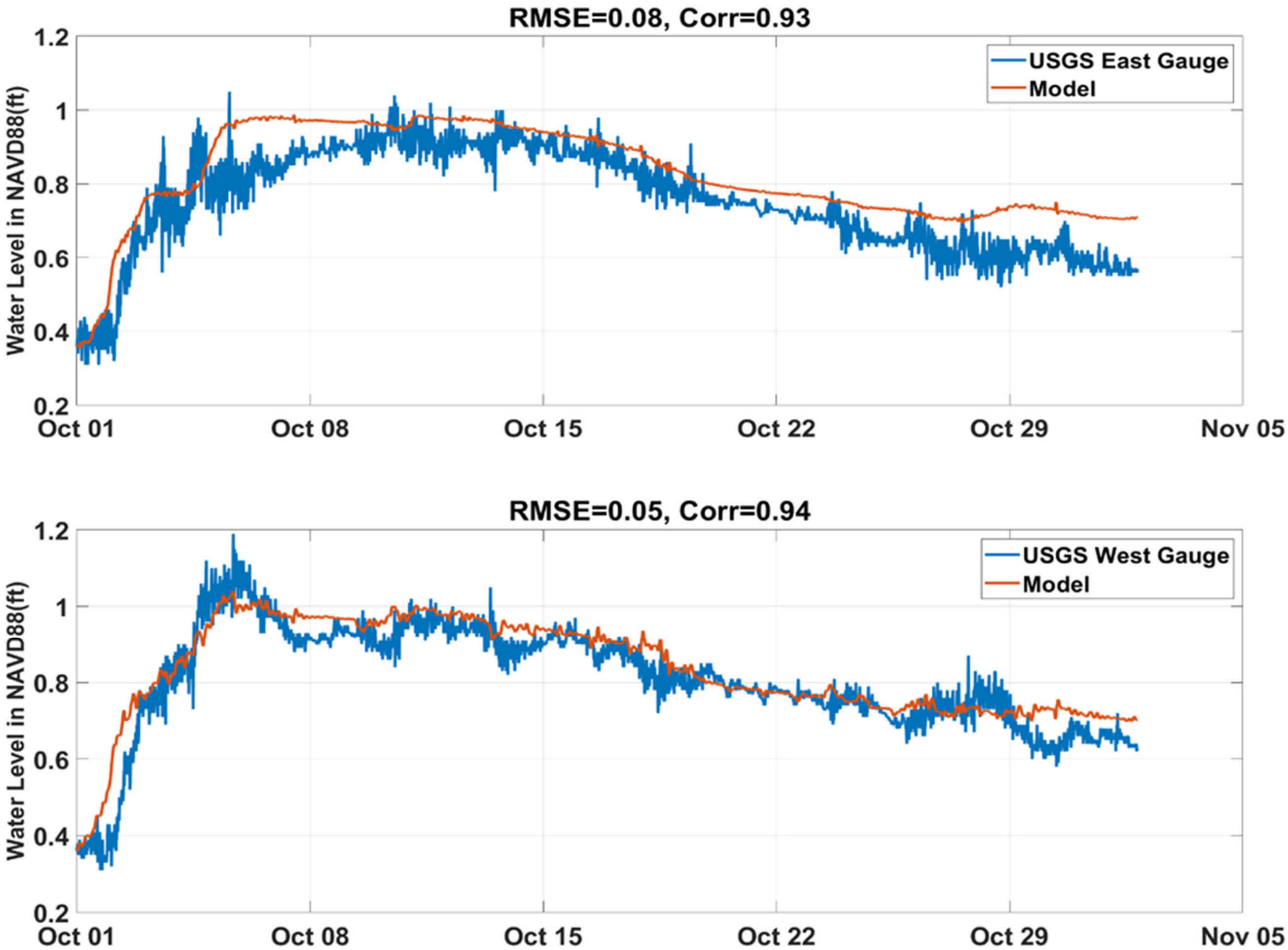

Figure 2.13. Comparison of measured and simulated water levels on the lake during Hurricane Joaquin at the east (upper) and west (lower) stations.

Table 2.1 RMSE and Pearson correlation coefficient for the two modeled storms.

Calibration Metric	Hurricane Matthew		Hurricane Joaquin	
	East	West	East	West
RMSE (ft)	0.06	0.06	0.08	0.05
Pearson correlation coefficient	0.96	0.97	0.93	0.94

Water depths during the peak of Hurricane Matthew are shown in Figure 2.14. The flooding hotspots previously identified by the stakeholders are illustrated as black dashed polygons. It can be observed that some of these flooding hotspots did not completely flood during Hurricane Matthew in the model simulations, but there was widespread flooding around all of the flooding hotspots. These hotspots did not completely flood because some sections of these hotspots lie on very high ground, above the peak water level elevation reached during Hurricane Matthew. Water depth on the lake watershed after the month-long drawdown post Hurricane Matthew is shown in Figure 2.15. One month after the storm passed most of the flooded areas dried out, except some extremely low-lying areas within the watershed.

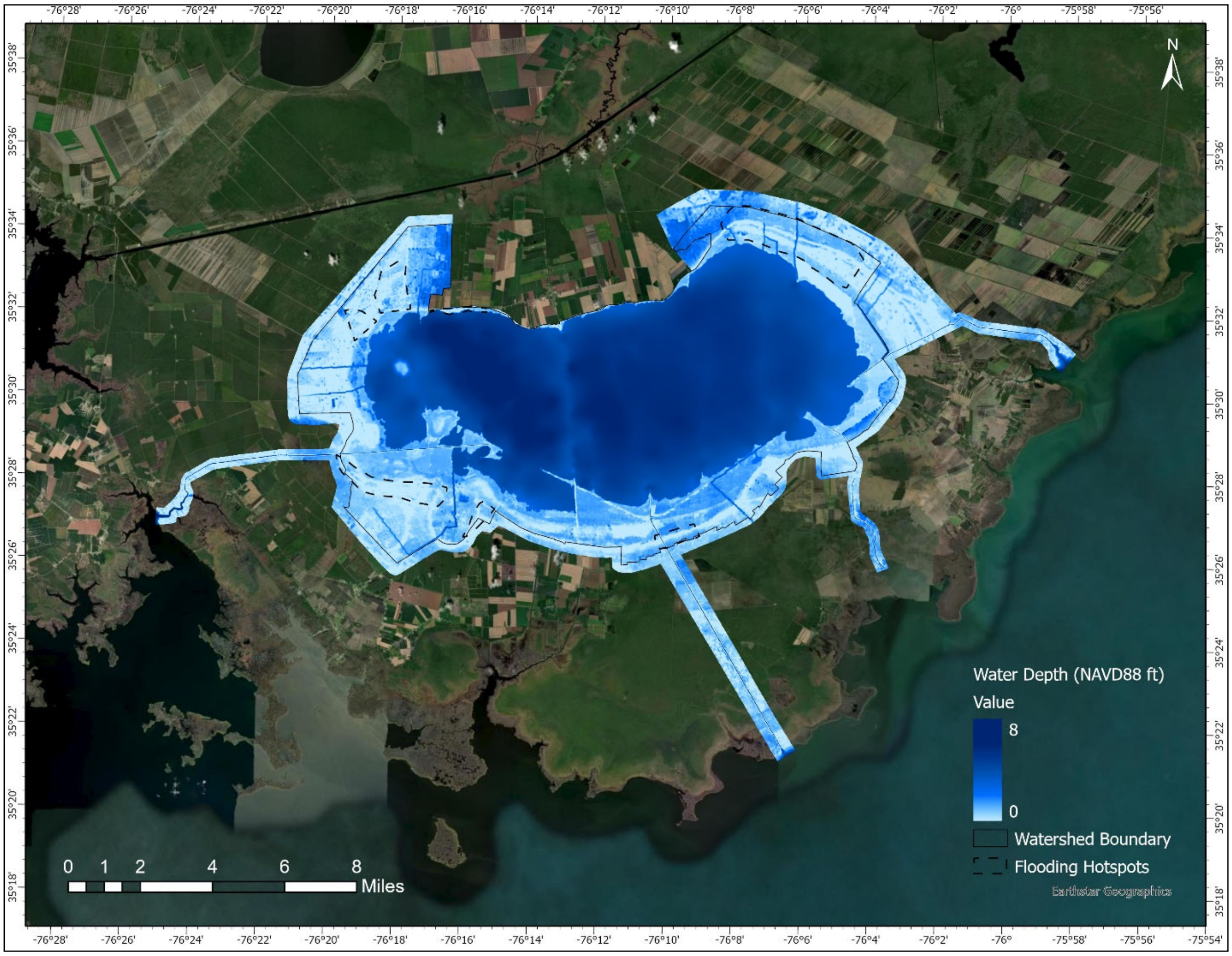

Figure 2.14. Water depth during the peak of Hurricane Matthew.

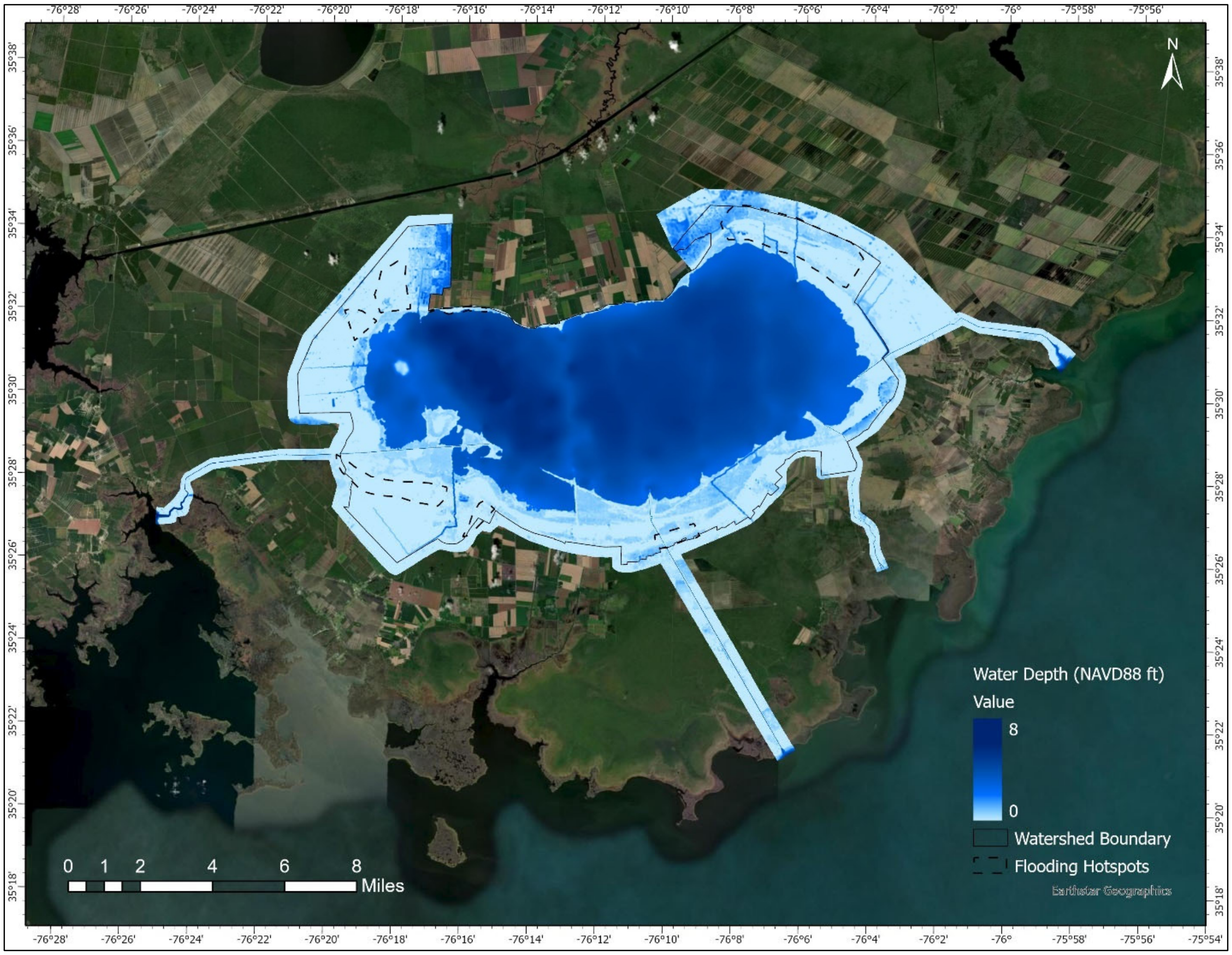

Figure 2.15. Water depth after the month-long drawdown after Hurricane Matthew.

3. SIMULATION OF DESIGN STORMS

After completion of numerical model calibration, boundary conditions for specific design storms were prepared, and simulations were conducted using the design storms. The purpose of the design storm simulations was to establish the response of lake water levels to various design storm scenarios without any action within the watershed (*i.e.*, baseline condition). The baseline conditions were used evaluate the efficiency of proposed engineering alternatives in reducing lake water levels and minimizing storm-driven flooding, described in subsequent sections.

3.1 Design Storms Time-Series

A thirty-seven-day input time-series for the soundside water level boundary condition and net precipitation was created for 2, 5, 10, 25, 50 and 100-year design storms. The time-series consisted of a 6-day run up period prior to the storm event, followed by a 2-day storm event, and a 29-day drawdown period. The simulation data for the run up and drawdown period of the design storm time-series were created using the October 2017 soundside water level data at Bell Island Pier and the averaged net precipitation data from the NCEP CFSv2 model from 2011-2020 for the month of October. The years with major storms during the month of October were not considered during this averaging exercise (i.e., 2015 and 2017). These years were excluded to remove any extreme values from the post-storm averaged time series. The month of October was specifically chosen because several major storms have passed this section of the North Carolina coast in the recent past during the month of October. Wind input was not used for design storm simulations. Since wind is a directional variable and is heavily dependent on storm tracks, to arbitrarily select a wind direction that could significantly affect the results would not be necessarily realistic. Furthermore, wind does not affect the overall lake drawdown timeframe, but only causes localized increased water levels in certain segments of the lake that are of short period. The alternatives are being measured in terms of days to drawdown overall lake water level, so the absence of wind in the design storm simulations does not significantly affect the evaluation of engineering alternatives.

The two-day storm time-series (between the run up and draw down period) of net precipitation and water level were created using two separate methods. The precipitation time-series was based on the NOAA Atlas 14 Point Precipitation frequency estimates for New Holland Station and was created using the Alternating Block method, a method for creating the temporal distribution of rainfall (i.e., design hyetograph) using the rainfall intensity-duration-frequency (IDF) curve. The design storm produced by this method specifies the rainfall depth occurring in "n" successive time intervals of duration (Δt) over a total duration ($T_d = n*\Delta t$). Based on the design return period, the rainfall intensity is extracted from the IDF curve/relation for each of the durations (Butler and John, 2011). The 2-day hyetograph generated using the Alternating Block Method for the 100-year design storm is shown in Figure 3.1. Since evaporation is negligible during storms, no evaporation was considered during the 48 hours when the storm precipitation occurred.

The soundside water level boundary condition time-series for different design storms was created based on Hurricane Matthew's surge curve with the peak surge value obtained from extreme value analysis of measured water levels at Bell Island Pier from 2013-2018. Extreme value analysis helps to predict the data trends by fitting a statistical distribution to measured data. Five different extreme value methods were utilized namely, Fisher Tippet-I, Weibull (0.75), Weibull (1.0), Weibull (1.4) and Weibull (2.0)**.** Curve fitting for the water level data and the different maximum water levels for the design storms obtained from the extreme value analysis methods are illustrated in Figure 3.2 and listed in Table 3.1. Peak surge values calculated using the Weibull 1.4 method were used as they are in the mid-range when compared to values calculated from other statistical methods. The complete 37-day net-precipitation time-series is shown in Figure 3.3. The net precipitation time-series for the storm duration is shown in Figure 3.4. The complete 37-day water level time-series is shown in Figure 3.5. The water level time-series for the storm duration is shown in Figure 3.6.

The design storms were simulated with an initial water level of 0.16 ft NAVD 88, which is the average lake water level during the month of October.

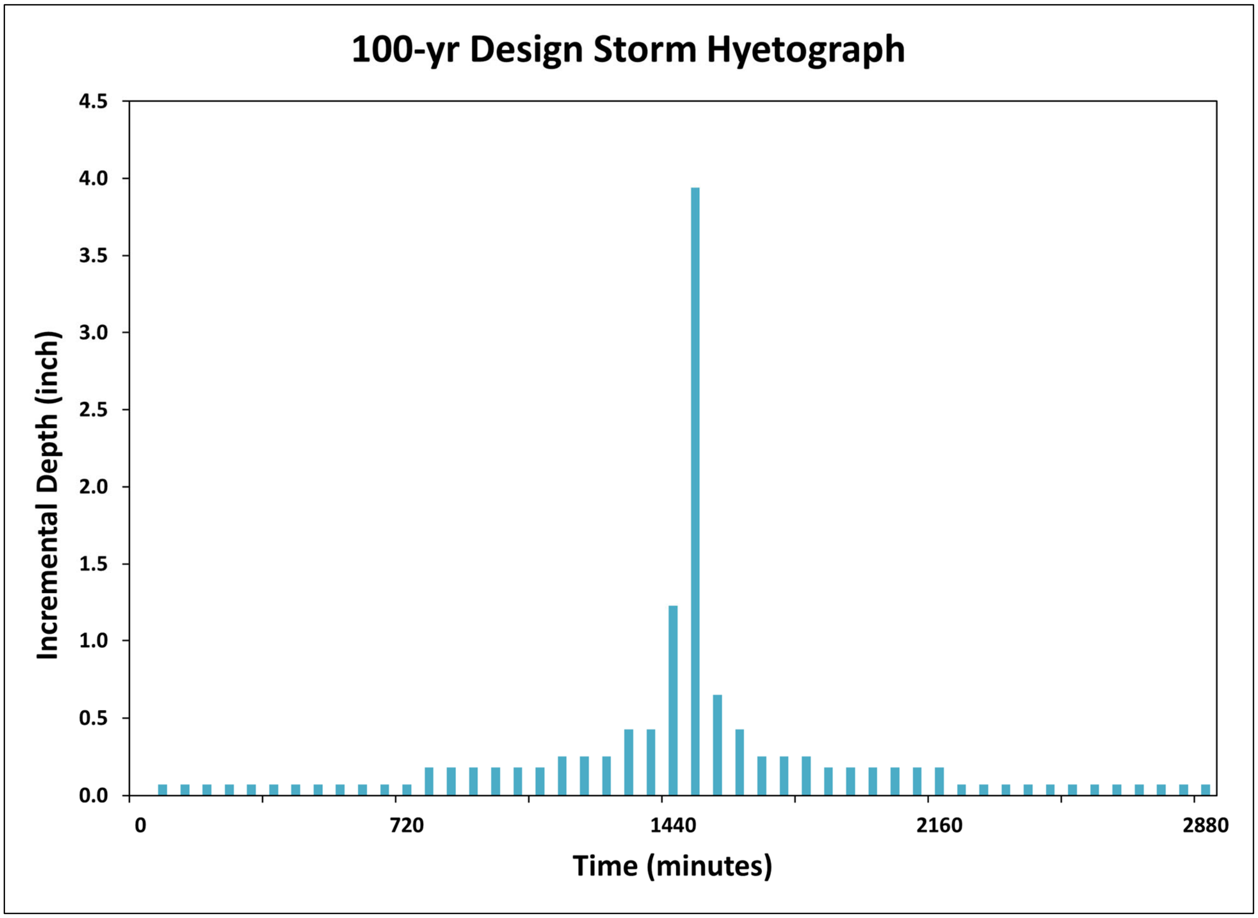

Figure 3.1. 100-year design storm hyetograph created using the Alternating Block method.

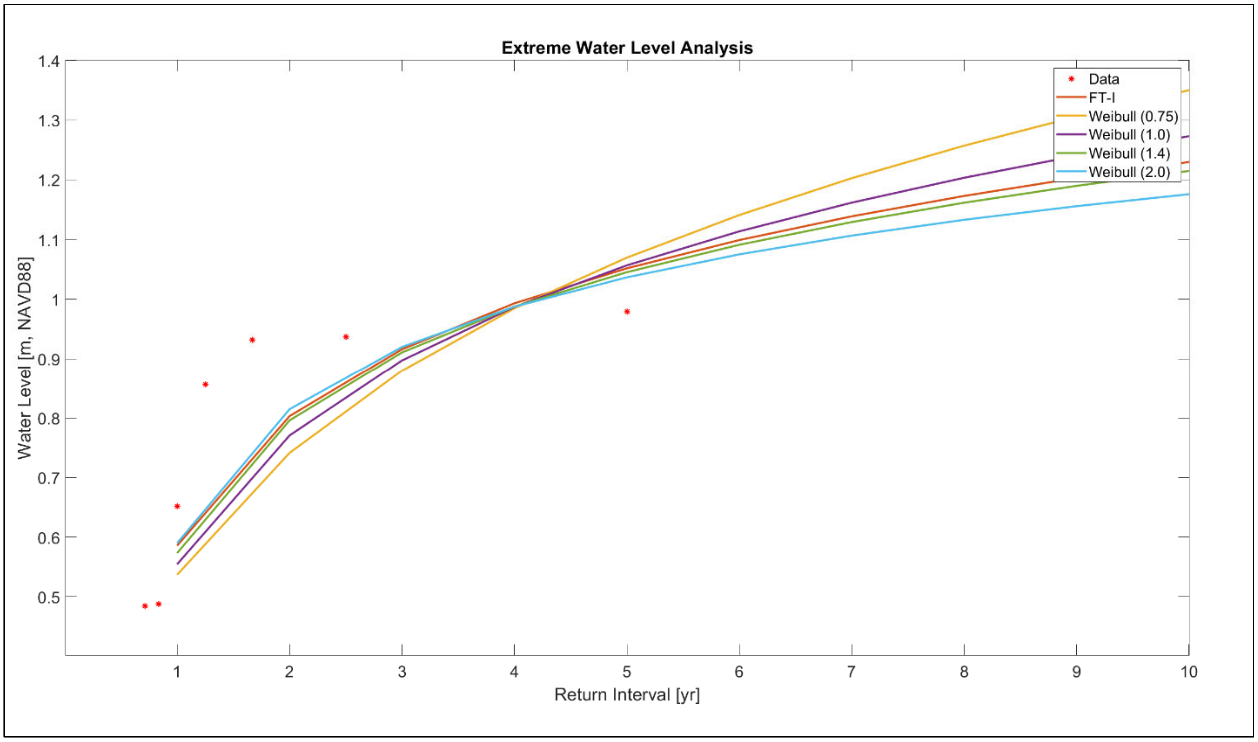

Figure 3.2. Curve fitting of extreme water levels at Bell Island Pier.

Table 3.1 Peak surge values for design storms utilizing different statistical models.

	Extreme Water Levels (ft, NAVD 88)				
Model	**Fisher Tippet-I**	**Weibull (0.75)**	**Weibull (1.0)**	**Weibull (1.4)**	**Weibull (2.0)**
2-year Event	2.63	2.43	2.53	2.61	2.67
5-year Event	3.45	3.51	3.47	3.43	3.40
10-year Event	4.03	4.43	4.18	3.99	3.86
25-year Event	4.79	5.76	5.11	4.67	4.39
50-year Event	5.36	6.84	5.82	5.15	4.75
100-year Event	5.92	7.97	6.53	5.62	5.08

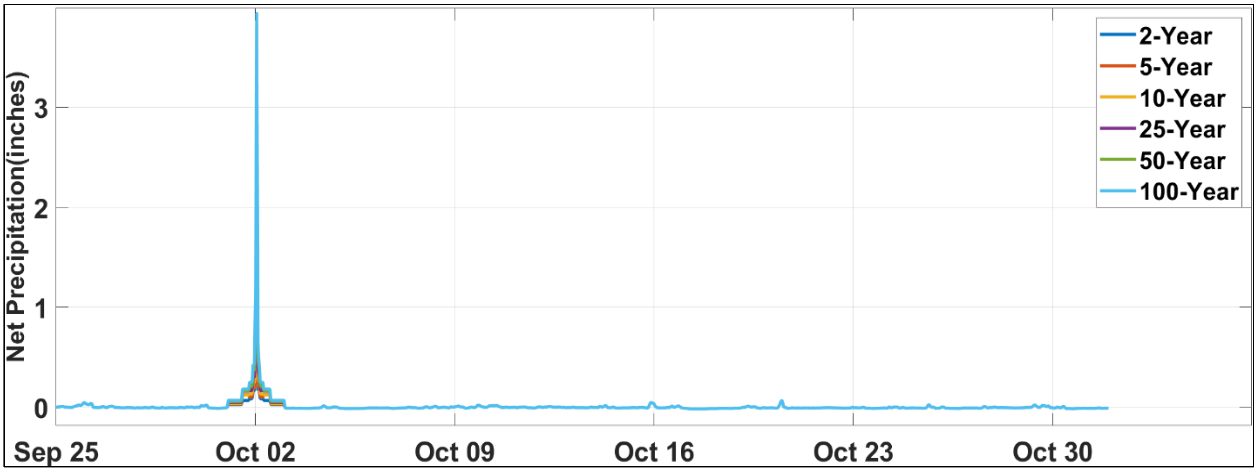

Figure 3.3. Design storms net precipitation time-series.

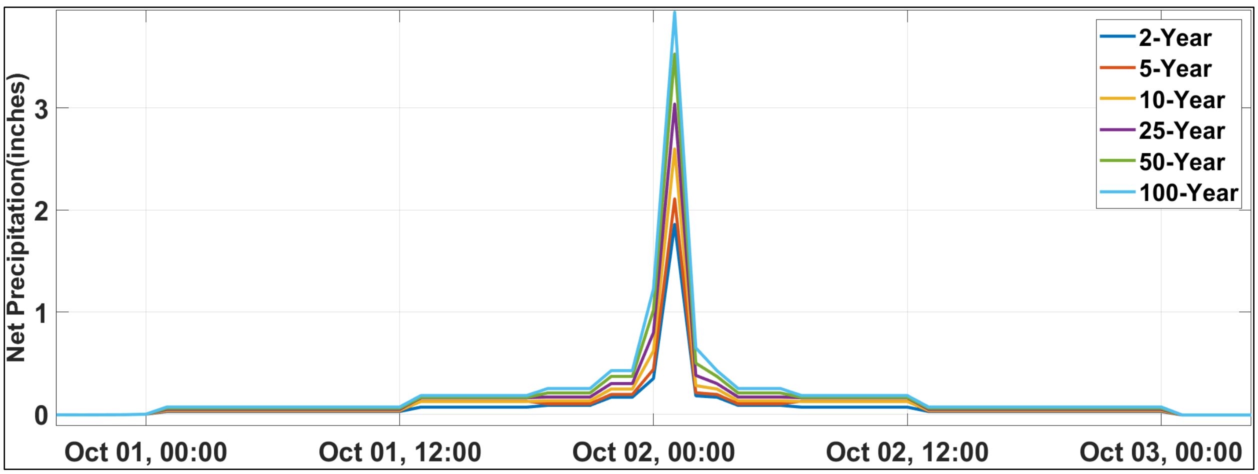

Figure 3.4. Detailed view of net-precipitation time-series during the design storms.

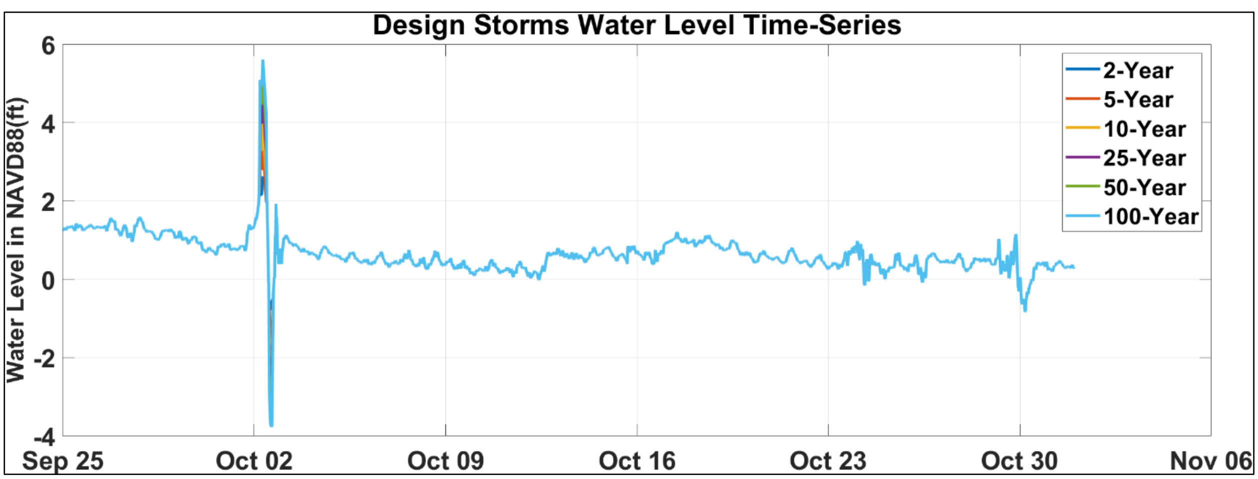

Figure 3.5. Design storms soundside water level time-series.

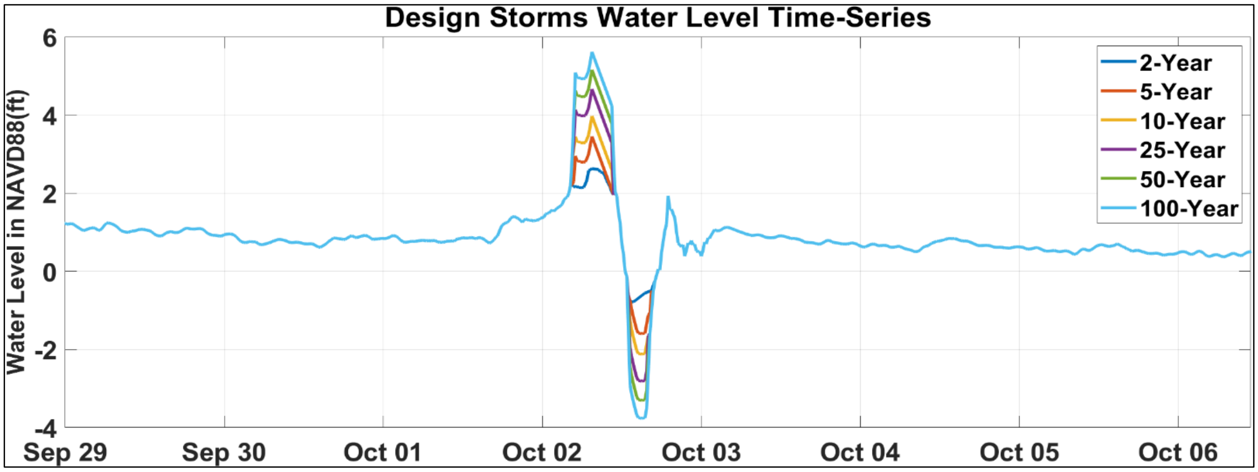

Figure 3.6. Detailed view of soundside water level time-series during the simulation period for the different design storms. The sound water level peaks as the storm is approaching and makes landfall in the region, after the storm passes a sound water level drawdown is observed as winds turn offshore drawing water away from the west sound shoreline.

3.1.2 Design Storm Results

The simulated lake water level increase and subsequent drawdown for the different design storms are shown in Figure 3.7 and summarized in Table 3.2. The estimated storm runoff volume for each design storm event is also described on Table 3.2. Due to similar water levels observed on the west and east basins, water level comparison is only shown for the west basin. Peak water level observed was highest for the 100-year storm (1.69 ft NAVD 88) and lowest for the 2-year design storm (0.69 ft NAVD 88). Observed drawdown was greater for the 100-yr storm and 50-yr storm over the 37-day simulation period (0.25 ft) when compared to the other design storms. The design storm runoff volume is the volume of water that was generated by the 2-day design storm; the design storm runoff volume was approximately three times greater for the 100-year design storm when compared to the 2-year design storm (Table 3.2).

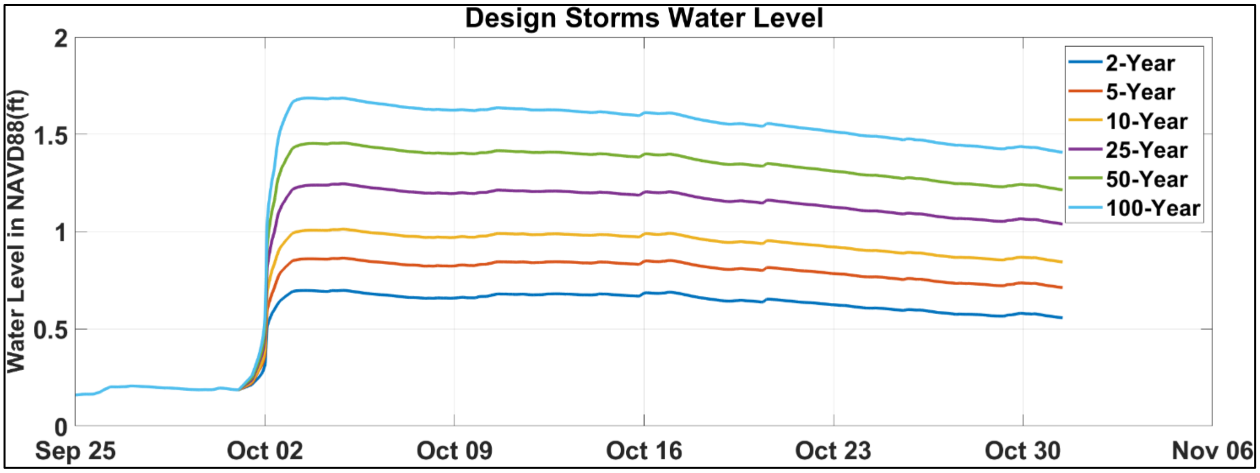

Figure 3.7. Simulated lake water levels for the design storms under existing conditions.

Table 3.2 Minimum water level, maximum water level, ending water level at simulation, drawdown over 37-day simulation period, and storm runoff volume for each design storm. Water level data is provided in ft relative to the NAVD 88 vertical datum.

Design Storm Event	Minimum Lake Water Level	Maximum Lake Water Level	Lake Water Level at End of Simulation	Drawdown Over 37-day Simulation Period	Design Storm Runoff Volume (ac-ft)
2-year	0.16	0.69	0.56	0.13	23,600
5-year	0.16	0.86	0.71	0.15	31,900
10-year	0.16	1.01	0.84	0.17	39,400
25-year	0.16	1.25	1.04	0.21	52,000
50-year	0.16	1.46	1.21	0.25	63,300
100-year	0.16	1.69	1.41	0.25	75,900

3.2 Simulation of Design Storms with Relative Sea Level Rise (RSLR)

Relative sea level rise can aggravate the impacts of extreme storm events in the study area. To incorporate RSLR into our estimates, the soundside water level time-series for the design storms was increased by relative sea level rise at the model's Pamlico Sound boundary. The NOAA intermediate low scenario of RSLR for the region was used as it has a high probability of occurrence and includes a slight acceleration on RSLR beyond measured rates (NOAA, 2017). The intermediate low projection is estimated to increase the sea level by 0.5 m (1.64 ft) by 2100.

The simulated lake water levels with RSLR added in the model is shown in Figure 3.8. Incorporating RSLR significantly decreased the ability of the lake to drain after the flooding event as demonstrated in Figure 3.8 and Table 3.3 when compared with the simulations under existing conditions (Figure 3.7, Table 3.2). For all the design storms simulated, the amount of lake water level drawdown over the 37-day simulation period, decreased by more than 50% due to RSLR (Table 3.2 and Table 3.3). This effect is due to the fact that RSLR increases the water level on the sound side, reducing the water level gradient between the lake and the sound, and directly affecting the drainage of the lake through the four outlet canals that connect Lake Mattamuskeet to the Pamlico Sound.

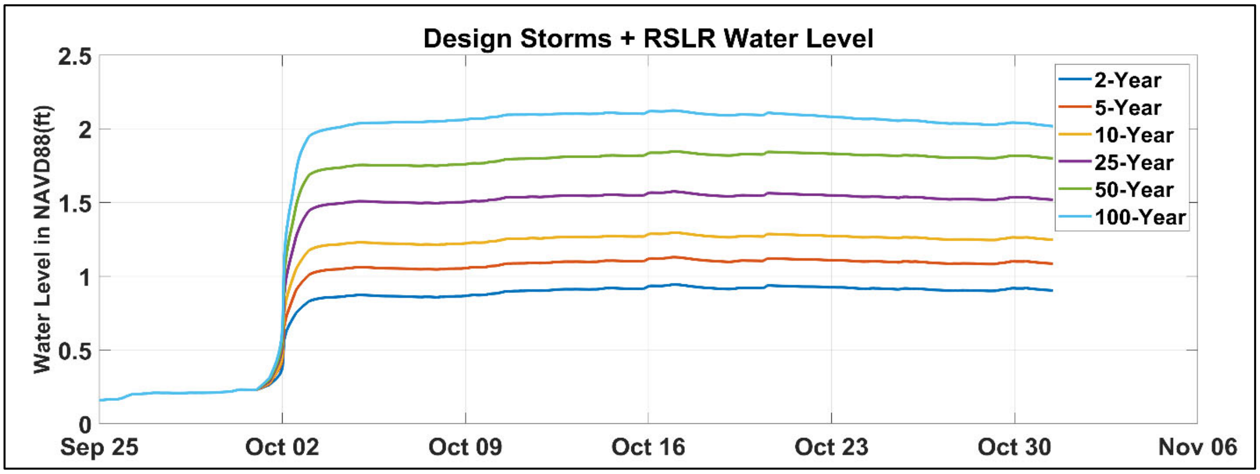

Figure 3.8. Simulated lake water levels for the design storms + RSLR.

Table 3.3 Minimum water level, maximum water level, ending water level at simulation, drawdown over 37-day simulation period for the design storms plus RLSR. Water level data is provided in ft relative to the NAVD 88 vertical datum.

Design Storm Event	Minimum Lake Water Level (ft)	Maximum Lake Water Level (ft)	Lake Water Level at End of Simulation (ft)	Drawdown Over 37-day Simulation Period (ft)
2-year	0.16	0.94	0.90	0.04
5-year	0.16	1.13	1.08	0.05
10-year	0.16	1.30	1.25	0.05
25-year	0.16	1.57	1.52	0.05
50-year	0.16	1.85	1.80	0.05
100-year	0.16	2.12	2.02	0.10

4. SIMULATION OF DESIGN ALTERNATIVES

Several engineered alternatives were evaluated in the H&H model to understand their impact on the drawdown of lake water levels compared to the existing scenario. Design alternatives were selected based on the alternatives proposed in the Lake Mattamuskeet Watershed Restoration Plan with additional input from the stakeholder and engineering teams. Eight alternatives were evaluated under simulation of the 10-year design storm as a screening exercise. Two preferred alternatives, as directed by the Hyde County Board of Commissioners, were simulated for a full suite of design storm scenarios under existing sea level and future sea level rise scenarios. The engineered alternatives are described in Section 4.1. The 10-year design storm screening evaluation is summarized in section 4.2; additional results from the 10-year design storm screening are provided in Appendix A. The results from the simulation of the two preferred design alternatives for all design storms are described in Section 4.4.

4.1 Engineered Alternatives and Design Basis

The eight engineered alternatives simulated in the model for the initial screening utilizing the 10-year design storm scenario are described in this section.

4.1.1 Centralized Pump Station Discharging to Intracoastal Waterway

There is an interest among LMWRP stakeholders in adding discharge capacity on the West Basin of the Lake, which is currently drained only by the Rose Bay Canal. For this engineered alternative, a pump station is proposed on the west basin of the lake that will discharge to an improved canal to the intracoastal waterway. A schematic of this concept is provided in Figure 4.1. A pumping capacity of 700,000 gallons per minute (gpm) was selected for this scenario, which would draw down the runoff volume produced from the 10-year storm in approximately 1.5 weeks. The pump was added in the model as a sink of water with a 700,000-gpm volume, in the location identified in Figure 4.1. The pump was turned on in the beginning of the simulation (10 days before the design storm peak) and stayed on for the duration of the entire simulation (37 days).

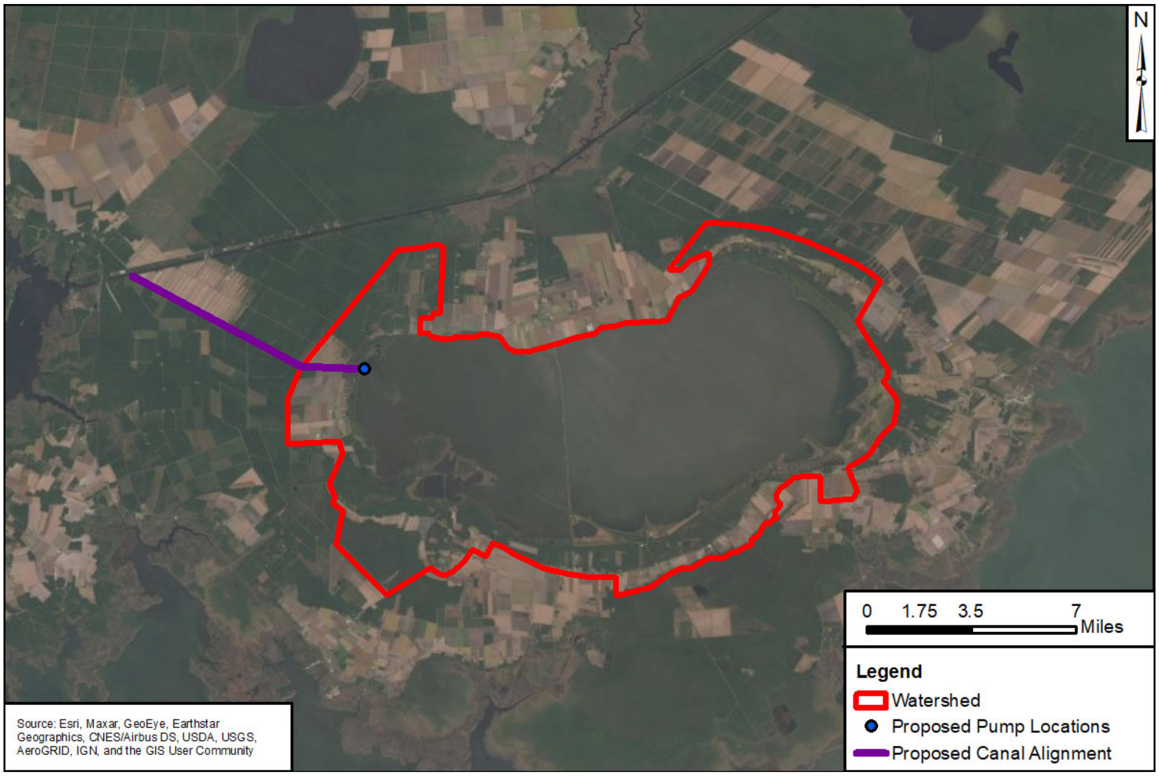

Figure 4.1. Schematic of centralized pump station discharging to intracoastal waterway.

4.1.2 Centralized Pump Station Discharging to Adjacent Drainage District

The Mattamuskeet Association is a private drainage district adjacent to the Lake Mattamuskeet watershed. This alternative includes pumping water from the east basin of the lake to the Mattamuskeet Association at a rate of 350,000 gpm. This pumping rate was selected to draw down the 10-year design storm in approximately 3 weeks. The pump was added in the model as a sink of water with a 350,00-gpm volume, in the location identified in Figure 4.2. The pump was turned on in the beginning of the simulation (10 days before the design storm peak) and stayed on for the duration of the entire simulation (37 days).

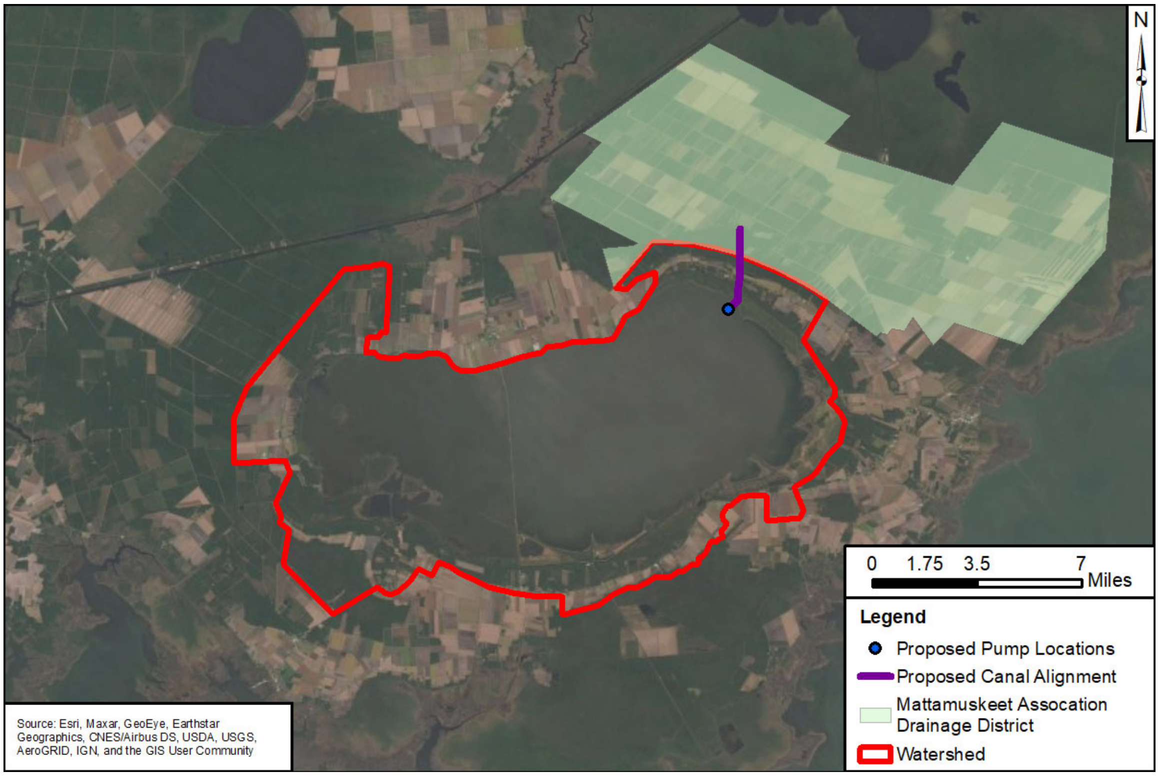

Figure 4.2 Schematic of centralized pump station discharging to adjacent drainage district.

4.1.3 Multiple Sheet Flow Sites

Prior to hydromodification in the Lake Mattamuskeet watershed, the natural hydrology of the lake flowed north towards the Alligator River. To restore natural hydrology of the lake, there is an interest in actively managing water to be conveyed north via pumping to constructed wetlands, where pumped water would be temporarily stored and sheet flow while drawing down over 48 hours. Under this scenario, dispersed pump stations are proposed to divert water from the lake or canals discharging to the lake to six sheet flow sites. Pumping ranges from 47,000 gpm to 190,000 gpm based on the storage capacity and discharge capacity of the potential sheet flow site. The total pumping capacity across the six stations was approximately 600,000 gpm. Cyclical pumping was modeled, which included the pumps being turned on for 24 hours and then off for 72 hours to allow the constructed wetland to draw down. More detail regarding the design basis for the sheet flow sites is provided in Appendix B. A schematic of the sheet flow sites concept is provided in Figure 4.3 Schematic of concept for multiple sheet flow sites.. The pumps were implemented in the model as multiple water sinks in the pump locations demonstrated in Figure 4.3.

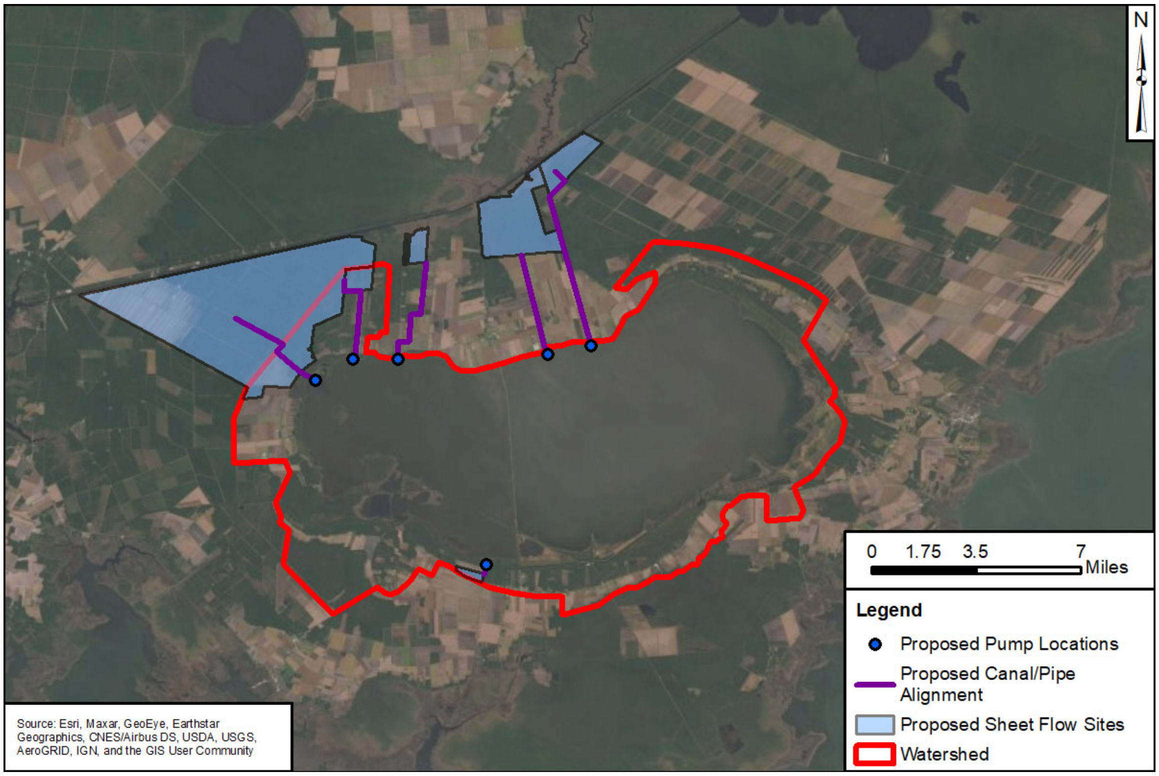

Figure 4.3 Schematic of concept for multiple sheet flow sites.

4.1.4 Dredge Existing Outlet Canals

There are four existing outlet canals that currently drain the lake, including one on the west basin (Rose Bay Canal) and three on the east basin (Outfall Canal, Lake Landing Canal, and Waupoppin Canal) (Figure 4.4). Over time, these outlet canals have become silted in, reducing their cross-section and overall discharge capacity. The outlet canals were originally implemented in the model using cross-sectional topobathy measurements provided by Dr. Randall Etheridge from East Carolina University. To evaluate the effect of dredging the outlet canals to their design cross-section on lake water levels, each canal was modified to the trapezoidal cross-sections listed in Table 4.1. The Outfall Canal dimensions were based on the description of the canal in Forrest (1999); the dimensions for the other three canals were obtained via email communication with Dr. Etheridge.

Table 4.1 Dimensions of design cross-section for each of the four outlet canals.

Canal	Top Width (ft)	Bottom Width (ft)	Depth (ft, NAVD 88)
Rose Bay	53	43	-9.5
Outfall	70	60	-10
Lake Landing	61	51	-7.5
Waupoppin	68	58	-6.5

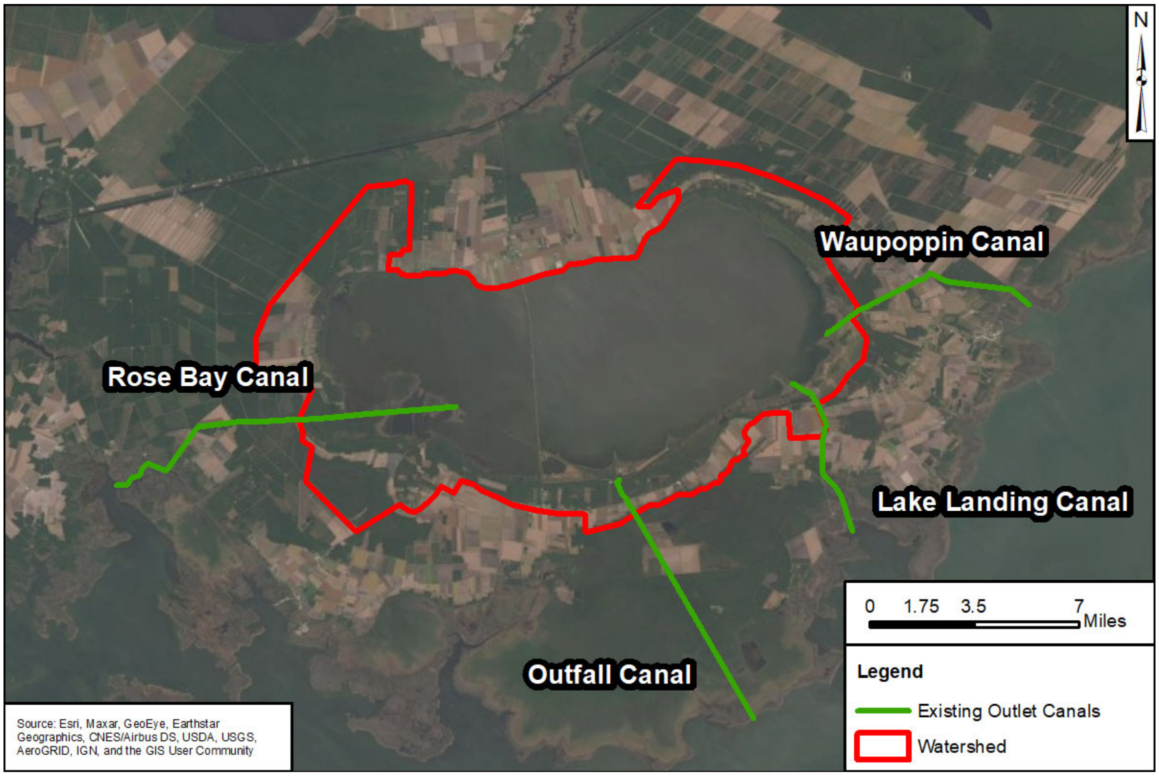

Figure 4.4 Existing outfall canals modeled as dredged to their design cross-section.

4.1.5 Removal of Tide Gates

The existing outfall canals are equipped with top hinged and side hinged tide gates that open when there is a positive hydraulic gradient between the lake water level and the downstream canal water level, which is controlled by the sound water level boundary in the model. The model was calibrated using a one-way valve that was assumed to be completely open when there was a positive hydraulic gradient between the lakeside and soundside of the outlet structures. To better understand the sensitivity of the model to the tide gate configuration, a simulation where the tide gates were completely removed was modeled.

4.1.6 Gravity Drained Canals to Adjacent Drainage Districts: 2 Canal Scenario

Another proposed engineered alternative that was evaluated included improving canals that formerly connected the Lake Mattamuskeet watershed to adjacent, privately-owned drainage districts, and therefore could be improved and used to lower the lake level via gravity drainage. The additional volume is proposed to discharge to the drainage network of the adjacent drainage districts and would be pumped out via upgraded or new pump stations at the existing drainage districts. Transfer between the Lake Mattamuskeet watershed and the private drainage districts would be controlled with an adjustable weir structure (e.g., sluice gate) that could be manually raised and lowered in accordance with lake management needs. Two smaller grids were prepared for the two new canals, and they were then merged with the original grid. A constant water level boundary condition of –3 ft NAVD 88 was set as the boundary condition at the end of the canals connecting to the drainage districts. This boundary condition added a hydraulic gradient for water to flow out of the lake through these canals. A weir structure was modeled at the end of the canal at -1 ft NAVD 88. Canal dimensions and slopes are provided in Table 4.2; canal locations are provided in Figure 4.5.

Table 4.2 Dimensions, slope, and depth of Jarvis and Burus gravity-drained canals.

Improved Canal	Top Width (ft)	Bottom Width (ft)	Slope (%)	Depth (ft, NAVD 88)
Jarvis Canal	70	60	0.03%	Varies (-2 ft to -6 ft)
Burus Canal	50	40	0.1%	Varies (-2 ft to -6 ft)

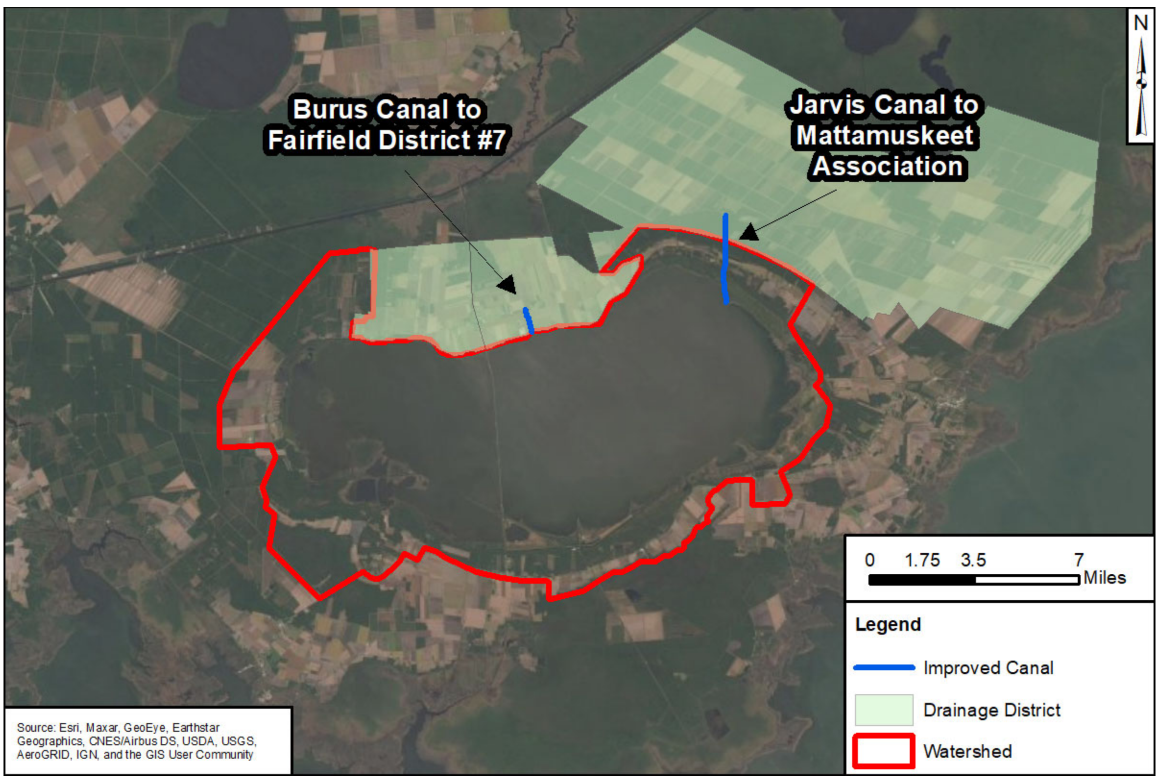

Figure 4.5 Location of gravity-drained canals - Burus Canal and Jarvis Canal.

4.1.7 Gravity Drained Canals to Adjacent Drainage Districts: 3 Canal Scenario

To increase drainage capacity after the initial model run with two improved canals, a third canal was added to the simulation in the west basin, as shown in Figure 4.6. Dimensions, slope, and depth of the additional gravity drained Swindells Canal, together with the two additional canals (Jarvis and Burus) are provided in Table 4.3.

Table 4.3 Dimensions, slope, and depth of Jarvis, Burus, and Swindells gravity-drained canals.

Improved Canal	Top Width (ft)	Bottom Width (ft)	Slope (%)	Depth (ft, NAVD 88)
Jarvis Canal	70	60	0.03%	-2.0 ft to -6.0
Burus Canal	50	40	0.1%	-2.0 ft to -6.0
Swindells Canal	40	30	0.3%	-2.0 ft to -4.5

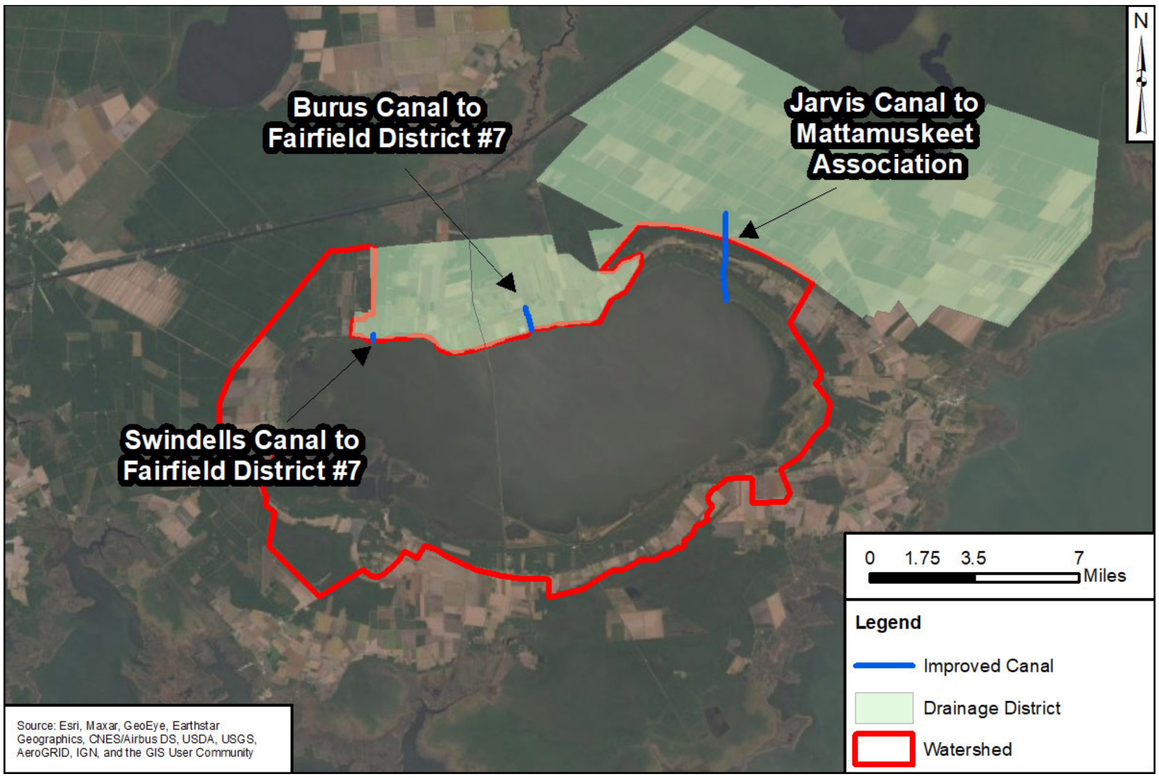

Figure 4.6 Location of gravity-drained canals - Burus, Jarvis and Swindells.

4.1.8 Optimized Pump Capacity (500,000 gpm)

The eighth and final alternative modeled was a pump station with an optimized pump capacity based on an annual operating budget. The optimized pump capacity was evaluated based on two scenarios: 1) assuming the entire watershed is assessed a $25/acre fee annually (500,000-gpm) and 2) assuming only areas outside the Refuge pay $25/acre (350,000-gpm).

Because the 350,000-gpm scenario had been modeled previously, the 500,000-gpm scenario was modeled for the optimized pump capacity with the pump located in the west basin. Additional details regarding the annual operating budget and the basis of the optimized pump capacity are provided in Appendix C.

4.2 10-Year Design Storm Screening Evaluation

To facilitate the screening evaluation of multiple alternatives for initial consideration, eight alternatives were simulated under the 10-year design storm event for comparison to the baseline 10-year scenario with no action. Each engineered alternative was simulated in the H&H model separately to evaluate its individual impact on lake water levels. The model simulations were based on previously described assumptions and best available data. The results of the evaluation would change if different assumptions were made.

Upon completion of the simulation, the following metrics were evaluated: 1) the peak water level measured in the lake during the simulation; 2) the number of days the pumps were turned on if the alternative includes pumping; 3) the number of days for the lake levels to return to the baseline elevation or the final water level at the end of the simulation, if that level is higher than the baseline elevation; 4) water level curves showing the water level of the no-action alternative and each engineering alternative graphed over the time of the simulation; and 5) change plot maps showing a comparison of water levels throughout the watershed comparing the no action alternative and the engineered alternative. A summary of the 10-year screening results is presented in this section. As discussed previously, due to similar water levels observed on the west and east basins, the peak water levels from the west basin only are presented and discussed. Additional results and maps for the ten-year screening simulations are provided in Appendix A.

Results from the simulations of pumping alternatives, including cyclical pumping to the sheet flow sites, and 350,000 gpm, 500,000 gpm and 700,00 gpm centralized pump station alternatives are shown in Figure 4.7 and summarized in Table 4.4. As expected, higher pumping capacity resulted in a greater reduction of peak water level during the 10-year storm and faster post-storm water level drawdown. For the 700,000 gpm pump, the peak lake water level during the storm was reduced from 1.01 ft NAVD 88 to 0.40 ft NAVD 88. Under this pump configuration the lake returned to the pre-storm water level of 0.16 ft NAVD 88 12.2 days after the peak water level modeled during the storm. The time for the lake to return to original pre-storm water level gradually increased to 17.8 days for the 500,00 gpm pump and 25.6 days for the 350,000 gpm pump. For the sheet flow sites alternative the lake never returned to the pre-storm water level of 0.16 ft NAVD 88 during the simulation period of 37 days. The sheet flow sites reduced the lake water level at the storm peak from 1.01 (existing conditions) to 0.89 ft. At the end of the 37-day simulation period, the lake water level under existing conditions was 0.84 ft NAVD 88 compared to 0.40 ft NAVD 88 for the sheet flow sites alternative; this resulted in a 0.44 ft reduction of flooding at the end of the simulation.

Results from the simulations of alternatives that do not including pumping within the model domain, including dredging of outlet canals, removal of the tide gates, and dredging of new gravity-drained canals are shown in Figure 4.8 and summarized in Table 4.4. Out of these alternatives, a return of lake water level to pre-storm level of 0.16 ft NAVD 88 during the 37-day simulation period was only observed for the alternative with three gravity-drained canals. In this alternative, the lake returned to pre-storm water levels 31.5 days after the peak water level modeled during the storm peak water. The second most efficient alternative was the two gravity-drained canals. Removal of existing tide gates from the outlet canals had no significant effect on lake water level drawdown and dredging of outlet canals had a relatively smaller effect on lake water level drawdown when compared to the gravity drainage canals. The effect of dredging was relatively small due to the lack of significant water level gradient between the lake and Pamlico Sound during the simulation period, which is typical for this time of the year (month of October) in the study area according to existing data.

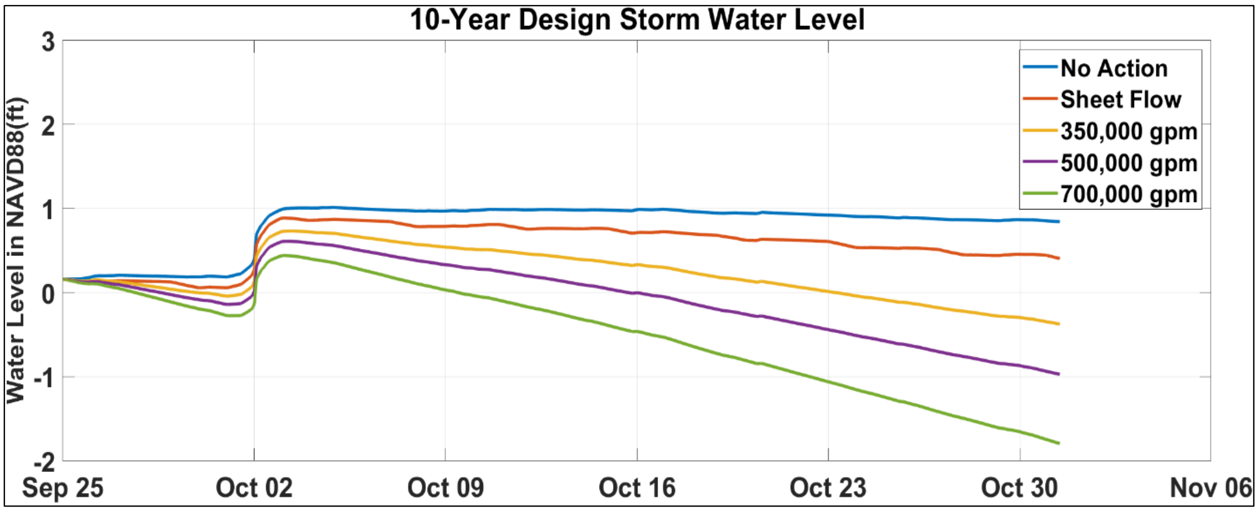

Figure 4.7 Lake water level increase and subsequent drawdown, ten-year design storm scenario, multiple pumping alternatives.

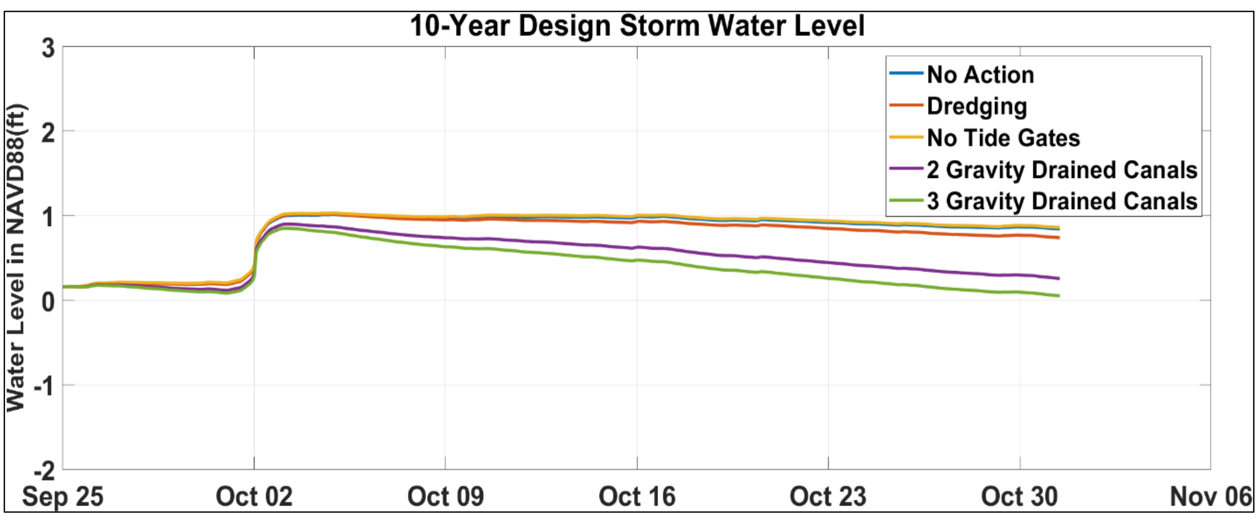

Figure 4.8 Lake water level increase and subsequent drawdown, ten-year design storm scenario, dredging, tide gate removal, and gravity drained canals.

Table 4.4 Peak water level, number of pumping days and number of days for the lake to return to the pre-storm water level of 0.16 ft NAVD 88. For the simulations where the lake water levels did not return to the original pre-storm level of 0.16 ft NAVD 88, the lake water level at the end of the 37-day simulation was provided.

Alternative	Peak Water Level (ft)	Number of Days of Pumping During Simulation (days)	Number of Days for lake water level to return to the pre-storm water level of 0.16 ft NAVD 88*
Existing Conditions (No Action)	1.01	-	0.84 ft in 37 days
700,000 gpm Pump	0.44	37	12.2 days
350,000 gpm Pump	0.73	37	24.6 days
Multiple Sheet Flow Sites	0.89	9	0.40 ft in 37 days
Dredge Existing Outlet Canals	1.01	-	0.74 ft in 37 days
Removal of Tide Gates	1.03	-	0.86 ft in 37 days
2 Gravity-Drained Canals	0.90	-	0.26 ft in 37 days
3 Gravity-Drained Canals	0.85	-	31.5 days
500,000 gpm Optimized Pump	0.61	37	17.8 days

* Scenarios where the lake never returned to the original water level of 0.16 ft NAVD 88 were presented in terms of final water level at end of the 37-day simulation period.

4.3 Simulation of Preferred Design Alternatives

After review of the results from the alternatives selected for 10-year design storm screening and the conceptual costs provided in Appendix C, the Hyde County Board of Commissioners selected the multiple sheet flow sites and the three gravity-drained canals as the preferred alternatives for additional simulations. These alternatives were selected based on a combination of stakeholder input, expected performance, relative cost-benefit, and potential for project funding and implementation. Model simulations under existing conditions and RSLR scenarios were performed for these two preferred alternatives for the 2-, 10-, 50-, and 100-year design storms and Hurricane Joaquin and Matthew. To evaluate the performance of the preferred alternatives the following metrics were used: 1) the peak water level measured in the lake during the simulation; 2) final water level at the end of the simulation; 3) drawdown over the simulation period; and 4) water level curves showing the water level of the no-action alternative compared to each preferred alternative during the simulation period. The results of the design storm simulations for the existing scenario and the two preferred alternatives under current sea level rise are shown in Figure 4.9 through Figure 4.14. The metrics described above are provided for each preferred alternative on Table 4.5 and Table 4.6.

Both alternatives showed a considerable improvement compared to the existing conditions under current sea level rise. The three gravity-drained canals alternative performed better than the sheet flow sites for all design storms simulated. The difference between alternatives was subtle for the 2-year design storm, but on the 10-year, 50-year, and 100-year design storms, the drawdown observed during the gravity-drained canals alternative was approximately twice the drawdown observed during the sheet flow site alternative (Figure 4.9 - Figure 4.12, Table 4.5 and Table 4.6). Both alternatives also increased lake drawdown during the Hurricane Joaquim and Hurricane Mathew conditions (Figure 4.14 and Figure 4.15). The gravity-drained canals also performed better than the sheet flow sites during the hurricane simulations, with subtler differences observed between the alternatives during Hurricane Joaquin compared to Hurricane Matthew. The gravity-drained canals significantly reduced peak lake water levels during Hurricane Matthew because the canals efficiently drained the lake prior to the storm, minimizing the flooding impact of this catastrophic event.

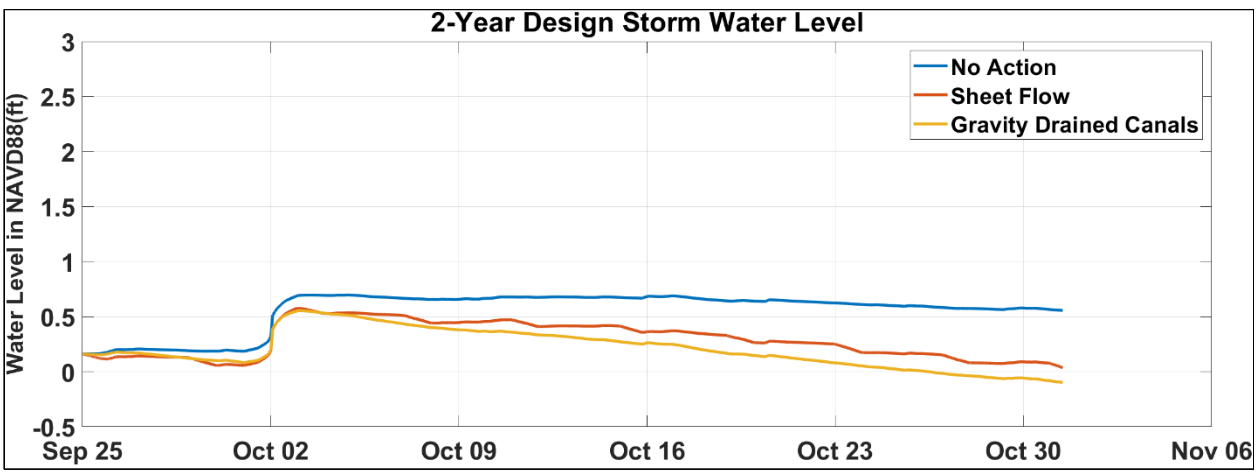

Figure 4.9 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, 2-year design storm.

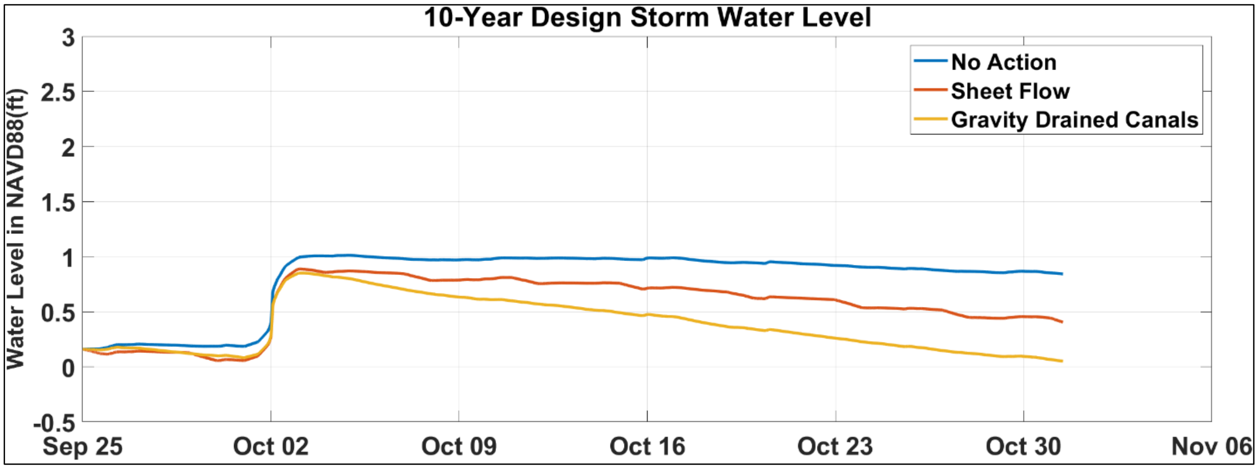

Figure 4.10 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, 10-year design storm.

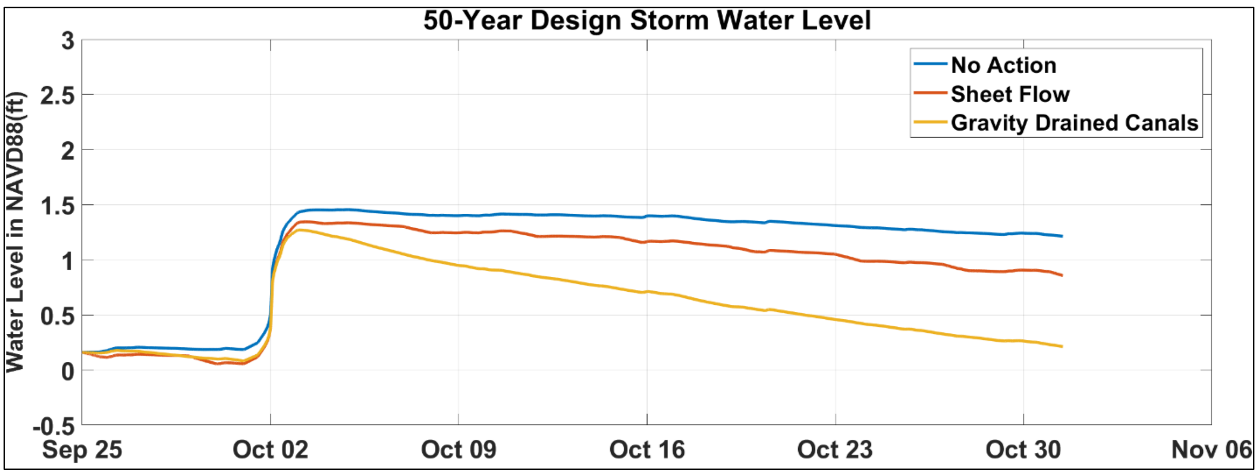

Figure 4.11 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, 50-year design storm.

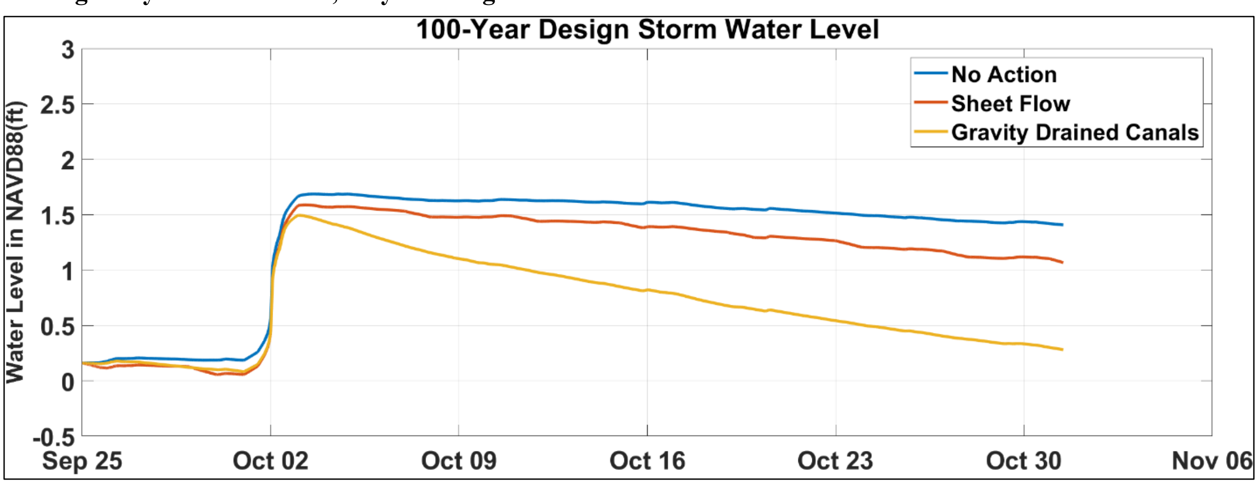

Figure 4.12 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, 100-year design storm.

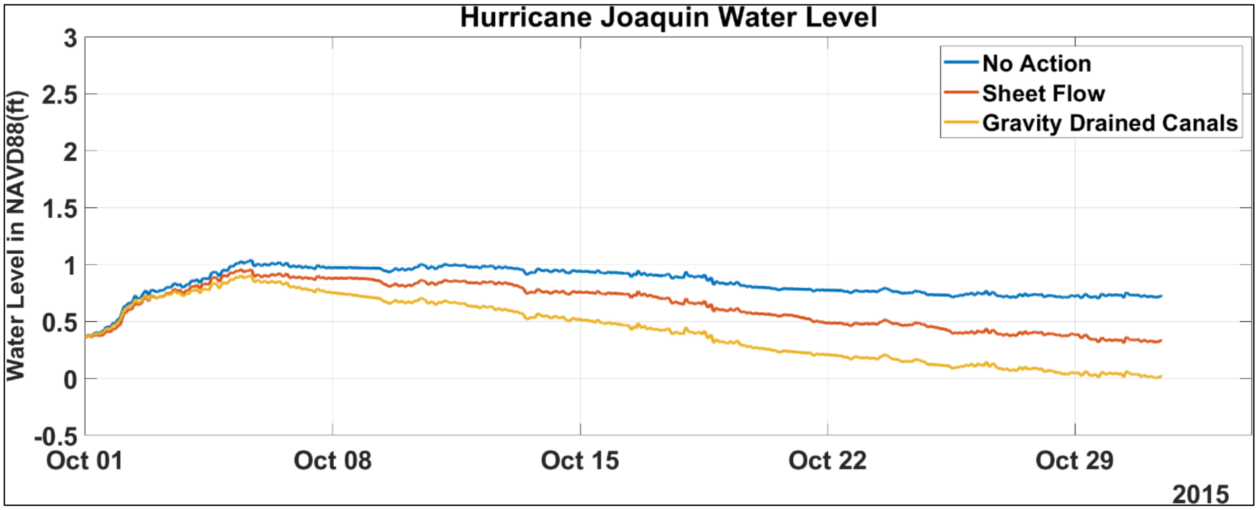

Figure 4.13 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, Hurricane Joaquin.

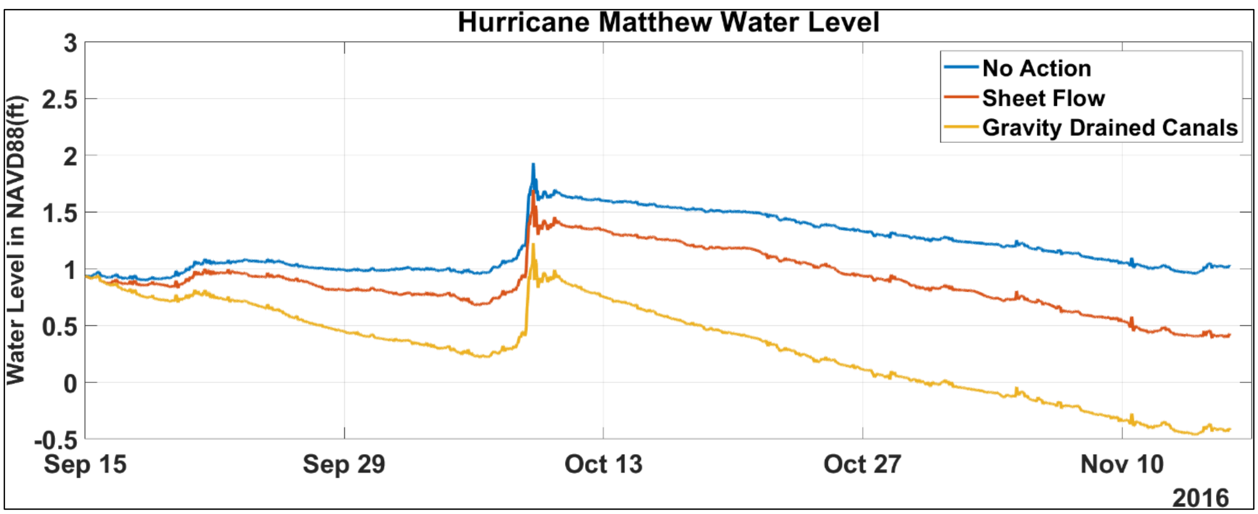

Figure 4.14 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity drainage canals, Hurricane Mathew.

Table 4.5 Maximum lake water level, lake water level at end of simulation and drawdown over simulation period for the multiple sheet flow sites alternative, multiple design storms, current sea level conditions. Water level data is provided in feet relative to the NAVD 88 vertical datum.

Design Storm Event	Starting Lake Water Level (ft)	Maximum Lake Water Level (ft)	Lake Water Level at End of Simulation (ft)	Drawdown over Simulation Period (ft)
2-year	0.16	0.58	0.04	0.54
10-year	0.16	0.89	0.40	0.49
50-year	0.16	1.34	0.86	0.48
100-year	0.16	1.59	1.07	0.52
Hurricane Joaquin	0.36	0.96	0.33	0.63
Hurricane Matthew	0.93	1.70	0.40	1.30

Table 4.6 Maximum lake water level, lake water level at end of simulation and drawdown over simulation period for the three gravity-drained canals alternative, multiple design storms, current sea level conditions. Water level data is provided in feet relative to the NAVD 88 vertical datum.

Design Storm Event	Starting Lake Water Level (ft)	Maximum Lake Water Level (ft)	Lake Water Level at End of Simulation (ft)	Drawdown over Simulation Period (ft)
2-year	0.16	0.55	-0.10	0.65
10-year	0.16	0.85	0.05	0.80
50-year	0.16	1.27	0.21	1.06
100-year	0.16	1.49	0.28	1.21
Hurricane Joaquin	0.36	0.90	0.01	0.89
Hurricane Matthew	0.93	1.23	-0.42	1.65

4.5. Simulation of Preferred Design Alternatives with RSLR

To estimate the performance of the two preferred alternatives under RSLR, the 2-, 10-, 50-, and 100-year design storms and Hurricane Joaquin and Matthew were simulated for the two preferred alternatives with RSLR added to the soundside water level boundaries at the existing outlet canals. Consistent with other model simulations incorporating RSLR, the intermediate low NOAA projection of RSLR was selected, with an estimated sea level increase of 1.64 ft by the year of 2100. To evaluate the performance of the preferred alternatives under RSLR, similar metrics as the simulations without RSLR were evaluated. The results of the simulations of the existing scenario with RLSR and the two preferred alternatives with RSLR are shown in Figure 4.15 through Figure 4.20. The metrics cited above are provided for the sheet flow sites alternative and the three gravity-drained canals alternative in Table 4.7 and Table 4.8, respectively. When RSLR is included in the simulation, the lake drawdown under existing conditions is greatly reduced as drainage through the existing 4 outlet canals is restricted by the higher soundside water levels. Similar to the simulations conducted under current sea level, in the simulations with RSLR, better performance is observed for the three gravity-drained canals when compared to the sheet flow sites; however, the relative difference in performance between these two preferred alternatives is greater when RSLR is incorporated, for example, for the 100-yr design storm, the lake drawdown over the simulation period was 2.2x greater for the gravity drained canals alternative compared to the sheet flow site in the simulation without RSLR. When RSLR was included, the drawdown from the gravity drained canals was 4.4x greater than the drawdown for the sheet flow sites (Table 4.5, Table 4.6, Table 4.7 and Table 4.8), these results indicate an increased benefit of the gravity-drained canals over the years.

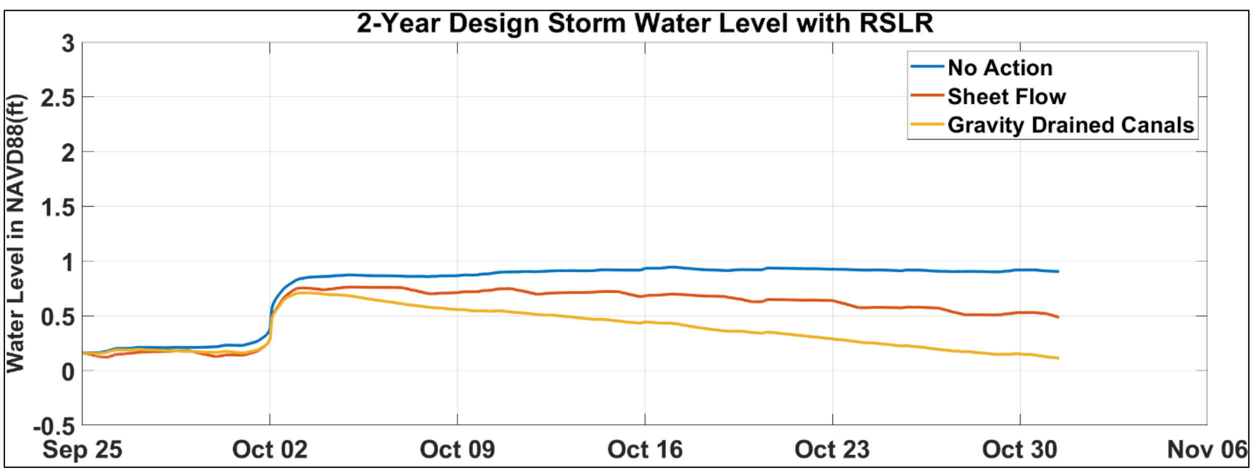

Figure 4.15 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, 2-year design storm with RSLR.

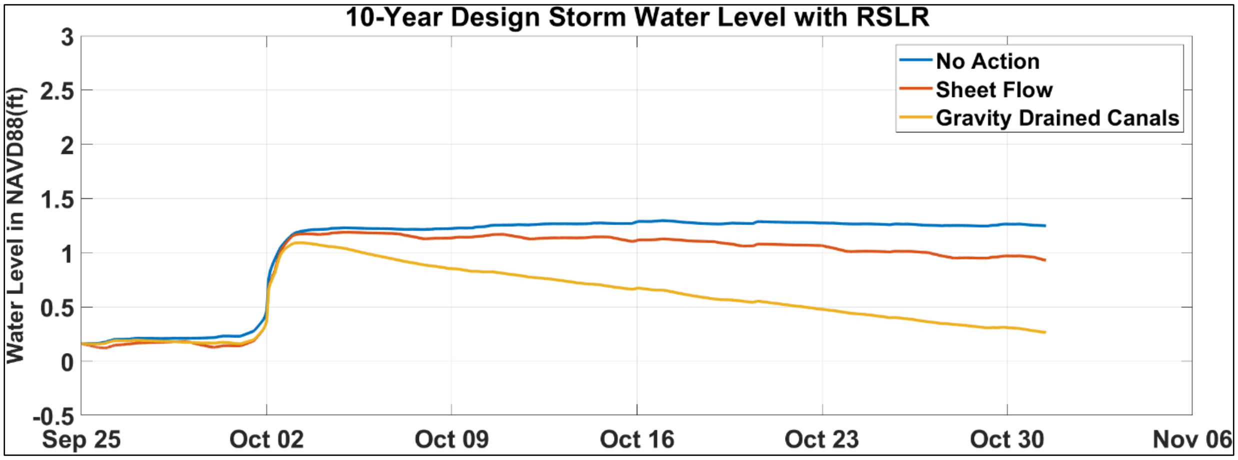

Figure 4.16 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, 10-year design storm with RSLR.

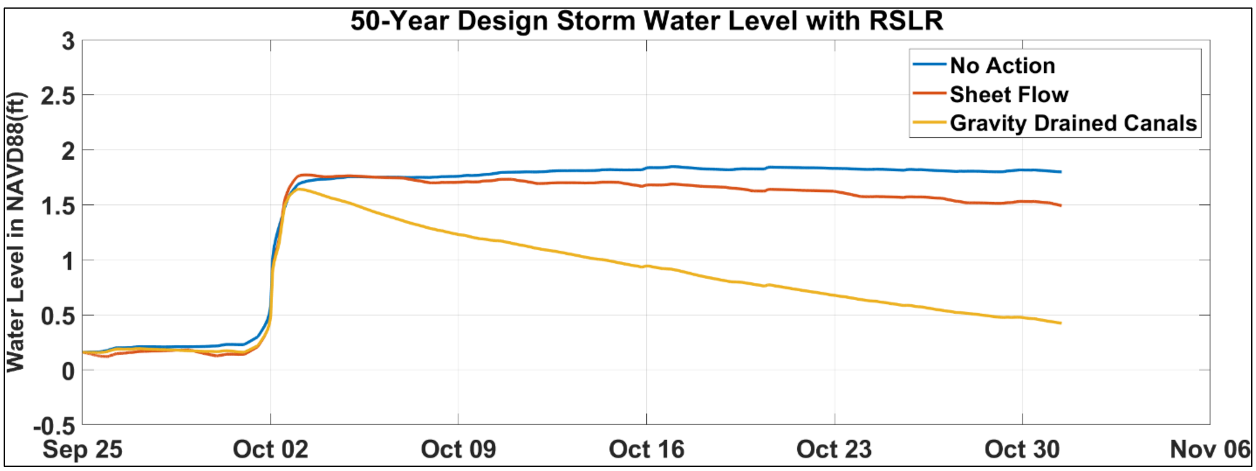

Figure 4.17 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, 50-year design storm with RSLR.

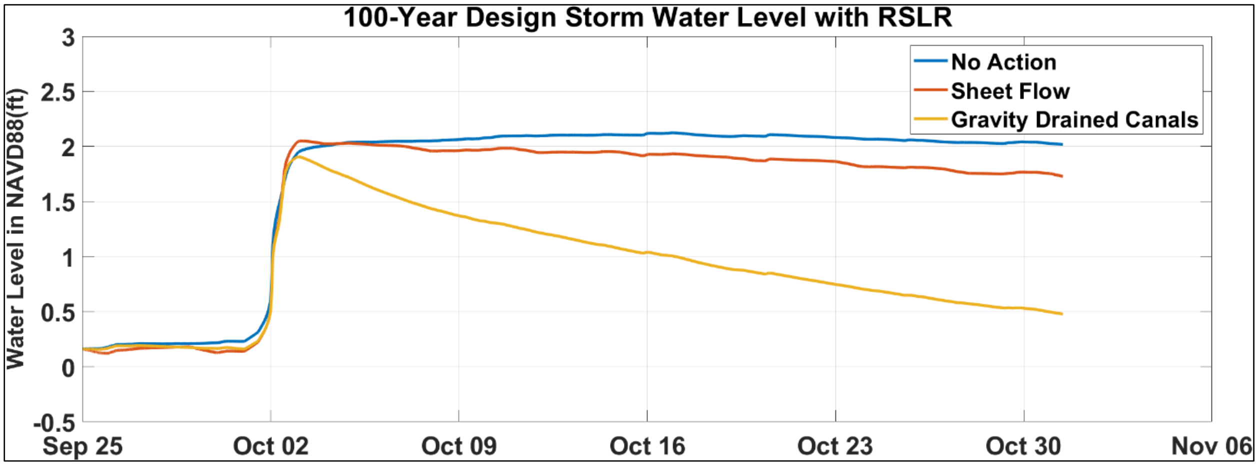

Figure 4.19 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, 100-year design storm with RSLR.

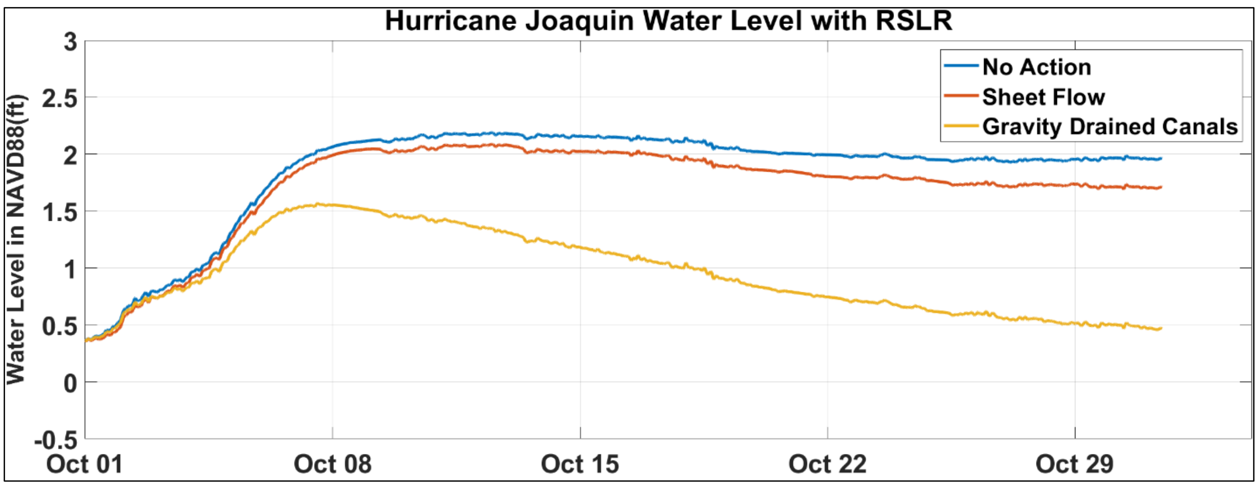

Figure 4.18 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, Hurricane Joaquin with RSLR.

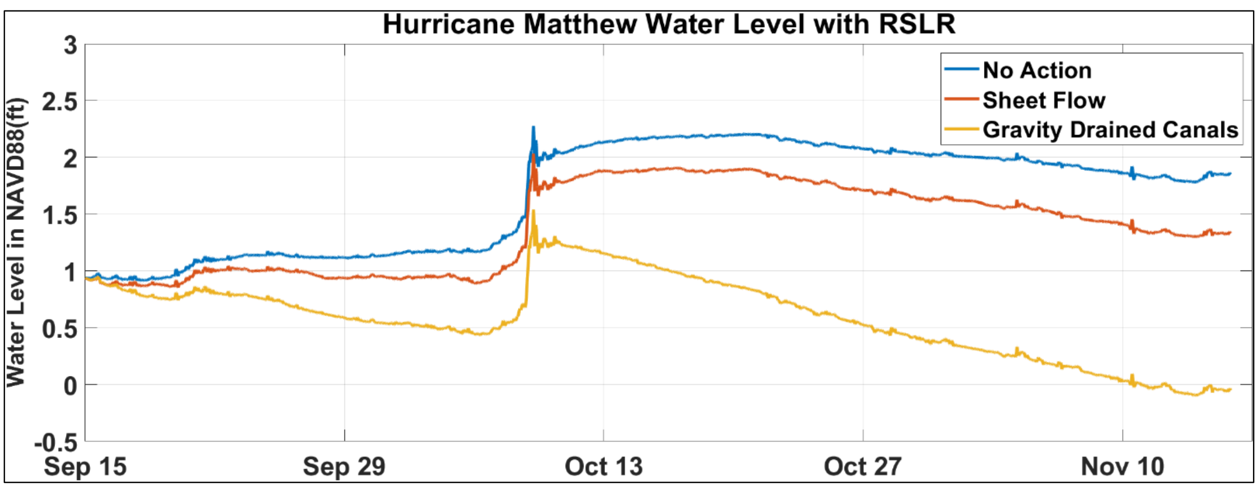

Figure 4.20 Results from the simulation of no-action and the two preferred alternatives, sheet flow sites and three gravity-drained canals, Hurricane Matthew with RSLR.

Table 4.7 Maximum lake water level, lake water level at end of simulation and drawdown over simulation period for the sheet flow sites alternative, multiple design storms, with RSLR. Water level data is provided in feet relative to the NAVD 88 vertical datum.

Design Storm Event	Starting Lake Water Level (ft)	Maximum Lake Water Level (ft)	Lake Water Level at End of Simulation (ft)	Drawdown over Simulation Period (ft)
2-year + RSLR	0.16	0.76	0.49	0.27
10-year + RSLR	0.16	1.19	0.93	0.26
50-year + RSLR	0.16	1.77	1.49	0.32
100-year + RSLR	0.16	2.05	1.73	0.32
Hurricane Joaquin + RSLR	0.36	2.09	1.71	0.38
Hurricane Matthew + RSLR	0.93	2.03	1.33	0.70

Table 4.8 Maximum lake water level, lake water level at end of simulation and drawdown over simulation period for the gravity drained canals alternative, multiple design storms, with RSLR. Water level data is provided in feet relative to the NAVD 88 vertical datum.

Design Storm Event	Starting Lake Water Level (ft)	Maximum Lake Water Level (ft)	Lake Water Level at End of Simulation (ft)	Drawdown over Simulation Period (ft)
2-year + RSLR	0.16	0.71	0.11	0.60
10-year + RSLR	0.16	1.09	0.26	0.83
50-year + RSLR	0.16	1.64	0.42	1.22
100-year + RSLR	0.16	1.91	0.48	1.43
Hurricane Joaquin + RSLR	0.36	1.57	0.47	1.10
Hurricane Matthew + RSLR	0.93	1.54	-0.05	1.59

5. CONCLUSIONS AND RECOMMENDATIONS

A watershed-scale hydrologic and hydraulic (H&H) model study utilizing the Delft3D-FM model was conducted to evaluate the effect of active water management strategies in reducing the frequency and magnitude of flooding within the lake watershed during extreme storms, under both existing and relative sea level rise (RSLR) scenarios. The model was successfully calibrated to the observed lake water levels observed during Hurricane Matthew and Hurricane Joaquin. The calibrated model was used to simulate engineering alternatives to manage lake water levels under multiple design storm scenarios, as well as Hurricanes Matthew and Joaquin. Eight initial design alternatives were screened using a 10-year design storm scenario. Based on the results from the screening, two preferred engineering alternatives were simulated for the full suite of design storm simulations. The conclusions from the study and recommendations for future work are summarized below:

- Comparisons between the simulations of existing conditions with and without RSLR demonstrate that RSLR significantly impacts the ability of Lake Mattamuskeet to drain after a storm event, which drastically reduces lake drawdown post storm. This effect is observed because RSLR increases the water level in the Pamlico Sound, decreasing the hydraulic gradient between the Lake Mattamuskeet and the sound, and greatly reducing the efficiency of the four outlet canals. These results indicate that natural lake drainage through the four outlet canals will reduce over the years, and that the need for active water level management strategies will increase.

- The screening of alternatives utilizing the 10-year design storm event demonstrated that active centralized pumping, cyclical pumping to sheet flow sites, and the dredging of new gravity-drained canals performed better than modifications to the existing tide gates and dredging of the existing four outlet canals. Based on these results, and considering other economic and social considerations, the Hyde County Board of Commissioners selected the sheet flow sites and gravity-drained canals for further evaluation under different design storm scenarios and RSLR.

- There was strong stakeholder involvement in this study by means of stakeholder meetings, where preliminary findings were presented and discussed. Some stakeholders voiced their preference towards the alternative of dredging the existing four outlet canals, but model results and engineering analyses demonstrated that the dredging alternative had limited benefit at a relatively high cost. Suggestions were made to evaluate dredging only small sections of the canals that are significantly clogged, instead of the entire canal, to reduce costs and potentially make this alternative more feasible. However, these suggestions could not be evaluated with the model due to the lack of continuous bathymetric data in the four outlet canals. If there is still stakeholder interest in further evaluating the dredging of existing outlet canals alternative, we recommend a detailed bathymetric survey of the entire extent of these four canals, as well as the immediate lake area that connects to these canals.

These additional bathymetric data will enable an evaluation of different dredging alternatives (*i.e.,* dredging selected areas that are more restricted instead of the entire canal), and accurate dredging volume calculations. These additional analyses may change the feasibility of the dredging alternative. It is worth noting, however, that the efficiency of dredging the existing outlet canals in controlling lake water levels will decrease with time, as RSLR increase water level in the Pamlico Sound will reduce the hydraulic head draining the lake into the sound.

- Both preferred alternatives caused a reduction in peak storm lake water level and an increase in post-storm lake drawdown compared to the existing condition. The three gravity-drained canals, however, demonstrated better overall performance in reducing the lake water levels compared to the sheet flow sites. The post-storm lake drawdown observed in the gravity-drained canal simulations was 1.2 to 2 times more than the drawdown observed in the sheet flow sites simulations. The difference between the alternatives increased for the larger and more intense storms. Therefore, the greatest differences between the alternatives were observed for the 100-year design storm and Hurricane Matthew. Both alternatives also contributed to reduce the peak water level associated with the design storms and hurricanes simulated, with greater reduction associated with the gravity-drained canals when compared to the sheet flow sites.

Several engineering alternatives were evaluated in this study, and preferred alternatives were identified for potential future implementation to actively manage Lake Mattamuskeet water levels and reduce impacts from flooding during extreme rainfall events. Alternatives excluded after the screening process may also be revisited in the future as new information becomes available. It is understood that before any of the alternatives evaluated here are implemented, the environmental, economic, and social considerations also need to be evaluated. The alternatives evaluated in this study have varying levels of cost, benefit, and impact. Simulations with increased sea level demonstrated that the need for active lake water level management will increase over time. It is possible that alternatives that are not as efficient, but have a lower cost (*e.g.,* sheet flow sites, selective dredging of small areas), are implemented to start the process of active management of the lake water level. As the sea level rises, there may be a point reached in which the low-cost alternatives are no longer effective, and more efficient, but higher cost alternatives, such as additional gravity drainage canals and centralized pumping are likely to be needed.

6. REFERENCES

Arcement Jr., G.J., and Schneider, V. R. 1989. Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains. United States Geological Survey, Water Supply Paper No 2339, 44 pg.

Butler, D. and Davies, J. 2011. Urban Drainage, Chapter 7. System component and design. Taylor & Francis.

Cooperative Institute for Research in Environmental Sciences (CIRES) at the University of Colorado, Boulder. 2014: Continuously Updated Digital Elevation Model (CUDEM) - 1/9 Arc-Second Resolution Bathymetric-Topographic Tiles. NOAA National Centers for Environmental Information. https://doi.org/10.25921/ds9v-ky35. Accessed 8 May 2020.

Deltares. 2021. Delft3D Flexible Mesh Suite: 1D/2D/3D Modelling suite for integral water solutions. D-Flow Flexible Mesh Computational Cores and User Interface, User Manual, Released for Delft3D-FM Suite 2021. June 2nd, 2021, 570 pg. Available at: https://content.oss.deltares.nl/delft3d/manuals/D-Flow_FM_User_Manual.pdf.

Heath, R.C. 1975; Hydrology of the Albemarle–Pamlico Region North Carolina; A preliminary report on the impact of agricultural developments. Raleigh, North Carolina: U.S. Geological Survey. Water-Resources Investigations 9-75 (see Supplemental Material, Reference S7, http://dx.doi.org/10.3996/082016-JFWM-068.S19); also available: https://pubs.er.usgs.gov/publication/wri759.

Moorman, M.C., Augspurger, T., Stanton, J.D., Smith, A. 2017. Where's the Grass? Disappearing Submerged Aquatic Vegetation and Declining Water Quality in Lake Mattamuskeet. Journal of Fish and Wildlife Management, 1 December 2017; 8 (2): 401–417. doi: https://doi.org/10.3996/082016-JFWM-068.

Moorman, M.C. 2018a. Mattamuskeet Hydrology, 9 April 2018 (Unpublished article).

Moorman, M.C. 2018b. Hydrologic Information from Hurricane Matthew, 1 January 2018 (Unpublished article).

Moorman, M.C. 2018c. Monitoring Hydrology at Mattamuskeet NWR During Hurricane Joaquin, 1 January 2018 (Unpublished article).

National Oceanic Atmospheric Association (NOAA). 2017. Global and Regional Sea Level Rise Scenarios for the United States. NOAA Technical Report NOS CO-OPS 083. Downloaded at:https://tidesandcurrents.noaa.gov/publications/techrpt83_Global_and_Regional_SLR_Scenarios_for_the_US_final.pdf . Accessed June, 2020.

National Oceanic Atmospheric Association (NOAA). 2021. NOAA NCEI Continuously Updated Digital Elevation Model (CUDEM). Downloaded at: https://chs.coast.noaa.gov/htdata/raster2/elevation/NCEI_ninth_Topobathy_2014_8483/. Accessed May 8, 2020.

North Carolina Spatial Data Download (NCSDD). 2021. Quality Level 2 Lidar Data downloaded at: https://sdd.nc.gov. Accessed May 2020.

North Carolina Coastal Federation (NCCF). 2018. Lake Mattamuskeet Watershed Restoration Plan. November 30, 2018. Available at: www.nccoast.org/lakemattamuskeet.

North Carolina State Climate Office (NCSCO). 2021. ECONet 10m Wind Speeds for PLYM-Tidewater Research Station. Data download available at: https://products.climate.ncsu.edu/.

OCM Partners, 2020: North Carolina Statewide Lidar DEM 2014 Phase 1 from 2010-06-15 to 2010-08-15. NOAA National Centers for Environmental Information, https://www.fisheries.noaa.gov/inport/item/49411. Accessed 14 July 2020.

Saha, S. et al. 2014. The NCEP Climate Forecast System Version 2. Journal of Climate, 27(6): 2185-2208.

APPENDIX A
Detailed 10-Year Design Storm Screening Results

APPENDIX A: DETAILED 10-YEAR DESIGN STORM SCREENING RESULTS

1. INTRODUCTION

This appendix provides detailed results for individual water level curves and change plot maps for each alternative simulated in the 10-year design storm screening of eight initial alternatives. Additional discussion about the 10-year screening results is provided in the main report.

2. 10-YEAR DESIGN STORM SCREENING RESULTS

2.1 Centralized Pump Station to Intracoastal Waterway (700,000 gpm)

2.1.1 Centralized Pump Station to Intracoastal Waterway (700,000 gpm) with Pump Starting at Beginning of Simulation

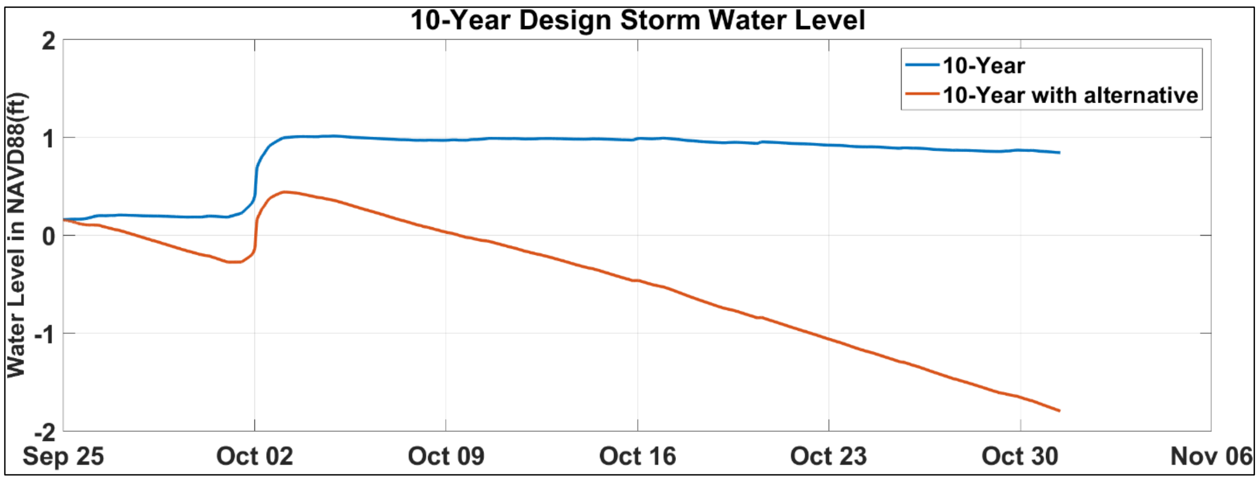

Figure 1. Lake water level increase and subsequent drawdown, 10-year design storm scenario, 700,000 gpm pump on the west basin, pump starts at beginning of simulation.

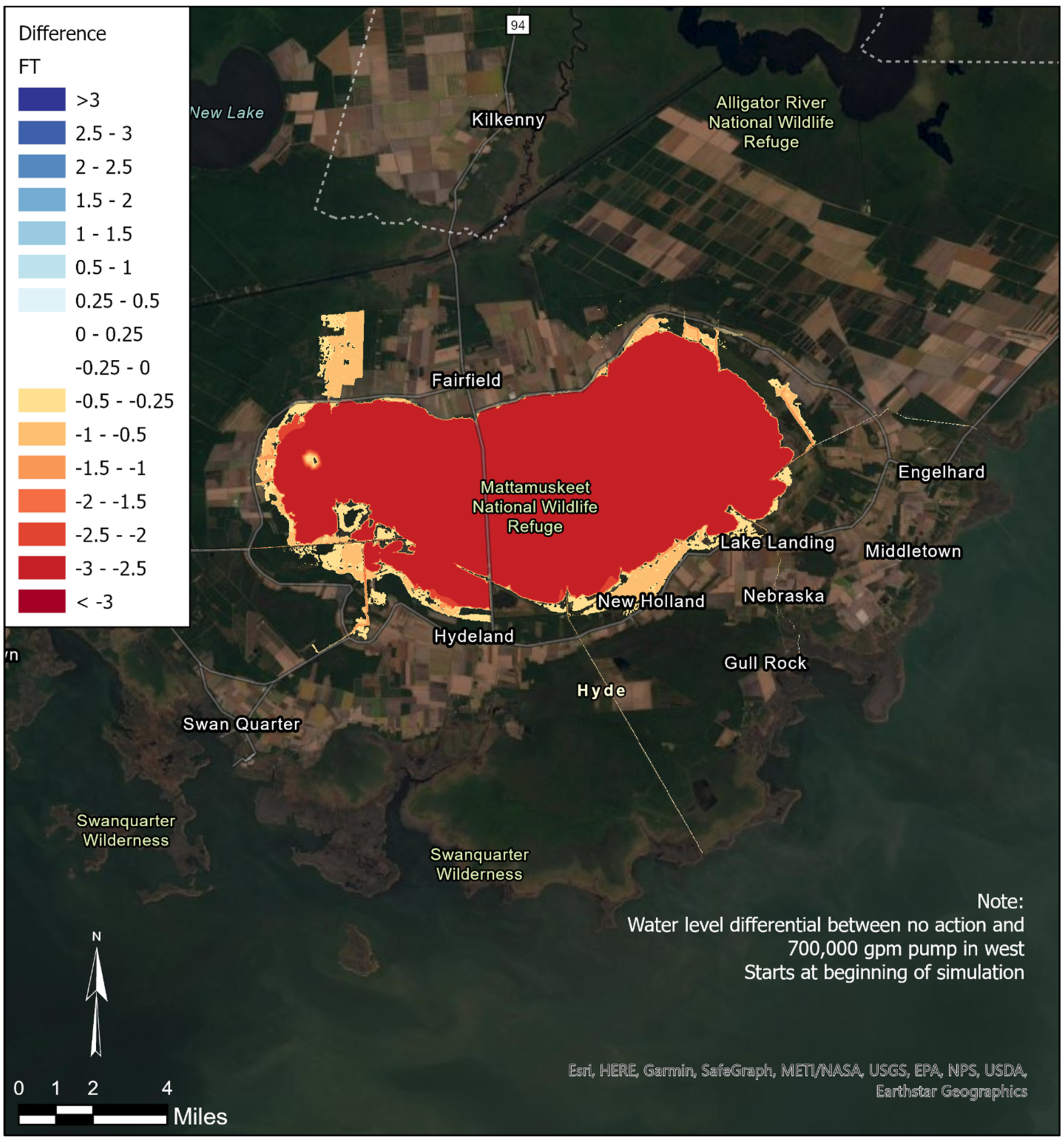

Figure 2. Water level differential between no action and 700,000 gpm pump on the west basin, pump starts at beginning of simulation.

2.1.2 Centralized Pump Station to Intracoastal Waterway (700,000 gpm) with Pump Starting at the Peak Water Level

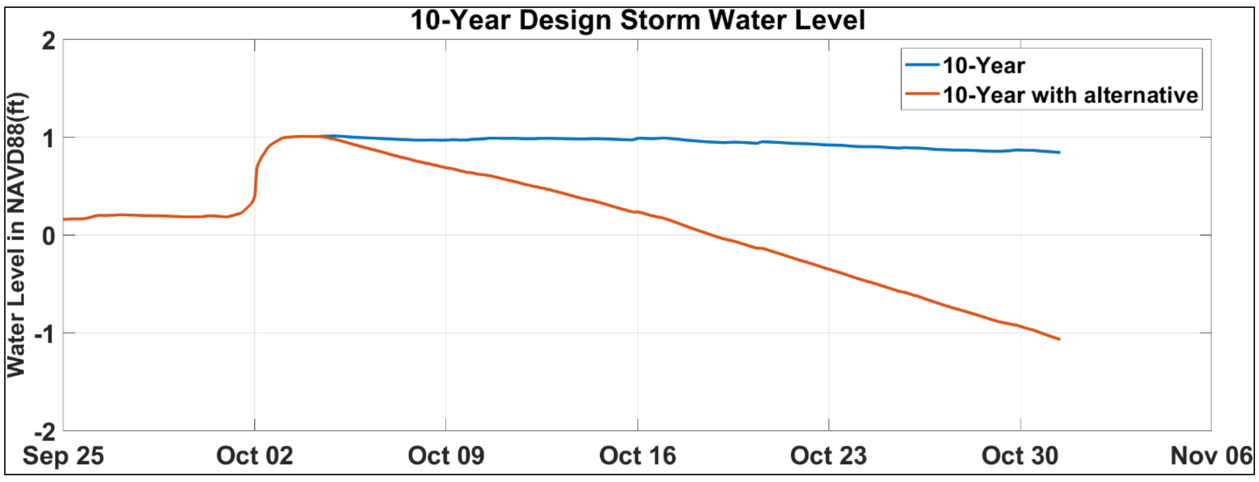

Figure 3. Lake water level increase and subsequent drawdown, 10-year design storm scenario, 700,000 gpm pump on the west basin, pump starts at peak water level.

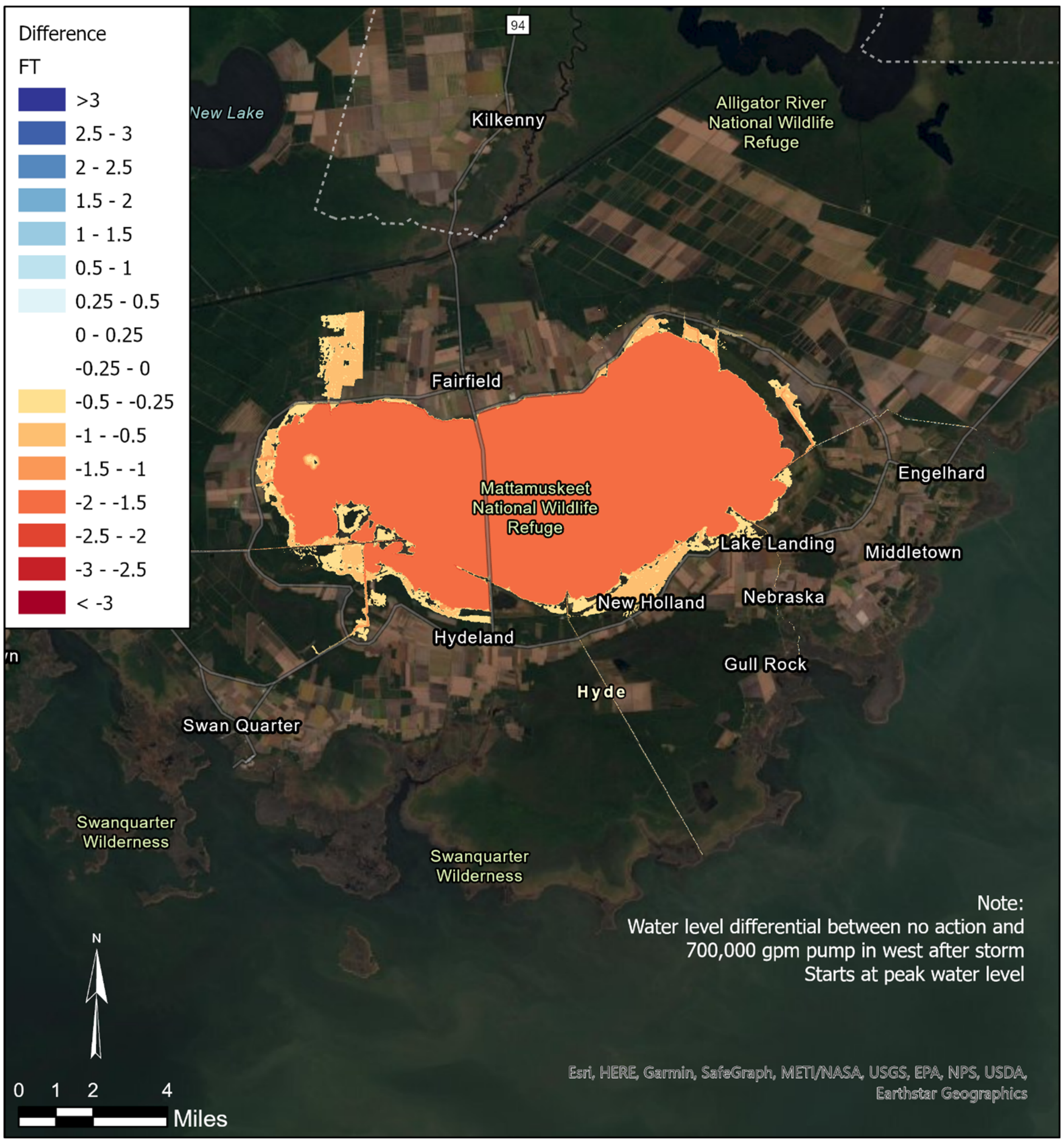

Figure 4. Water level differential between no action and 700,000 gpm pump on the west basin, pump starts at peak water level.

2.2 Centralized Pump Station to Adjacent Drainage Districts (350,000 gpm)

2.2.1 Centralized Pump Station to Adjacent Drainage Districts (350,000 gpm) with Pump Starting at Beginning of Simulation

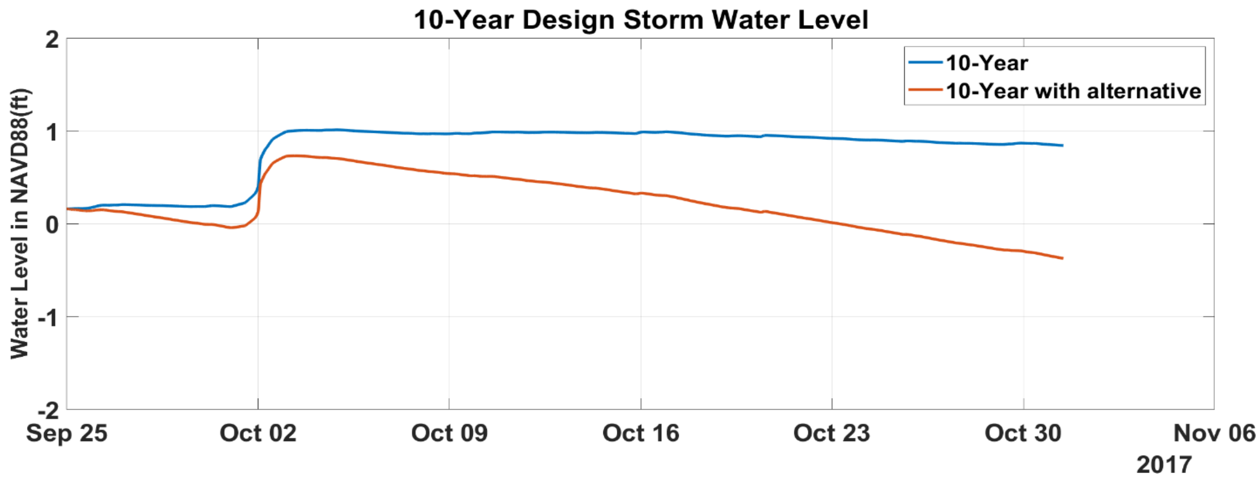

Figure 5. Lake water level increase and subsequent drawdown, 10-year design storm scenario, 350,000 gpm pump on the east basin, pump starts at beginning of simulation.

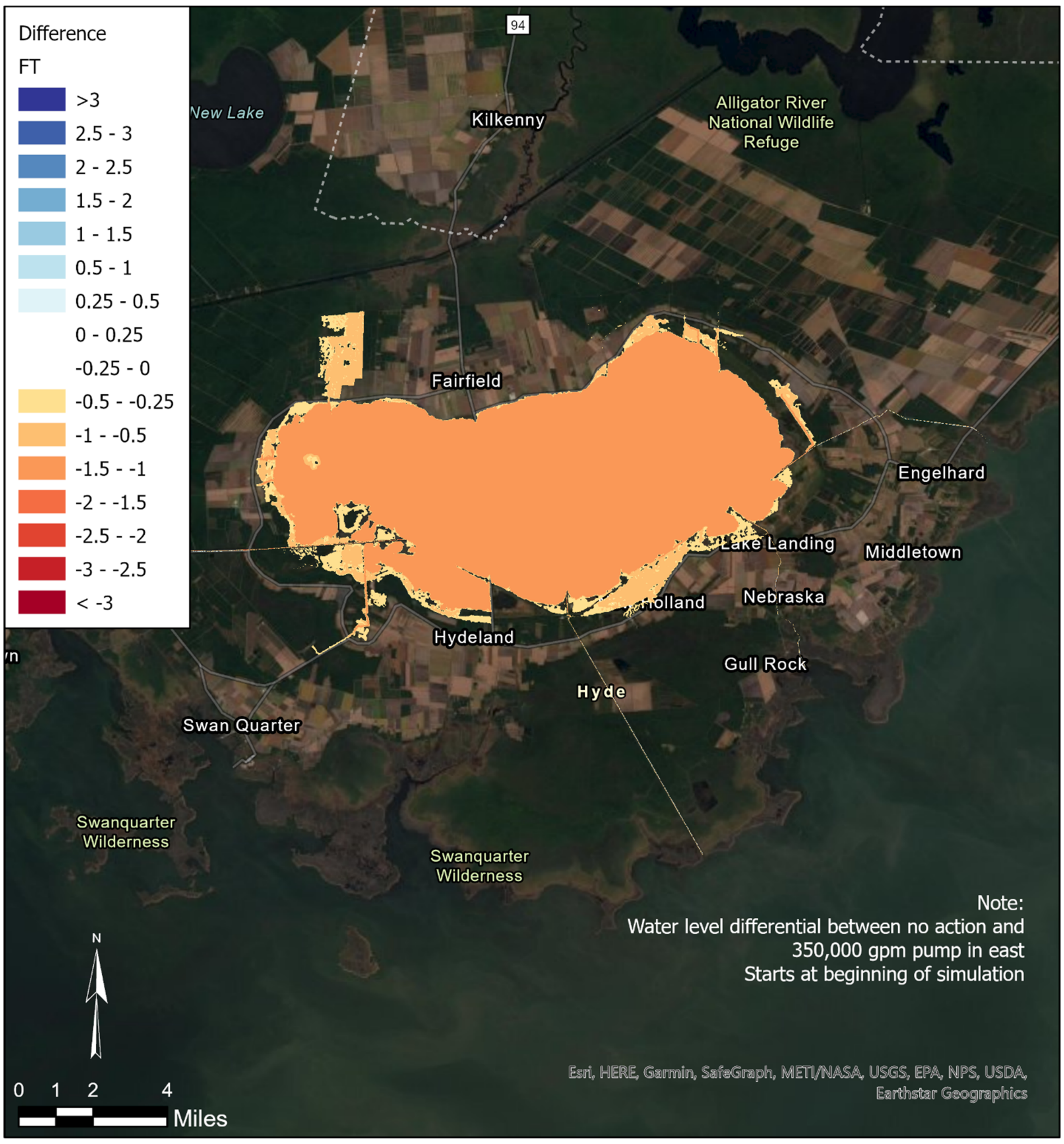

Figure 6. Water level differential between no action and 350,000 gpm pump on the east basin, pump starts at beginning of simulation.

2.2.2 Centralized Pump Station to Adjacent Drainage Districts (350,000 gpm) with Pump Starting at the Peak Water Level

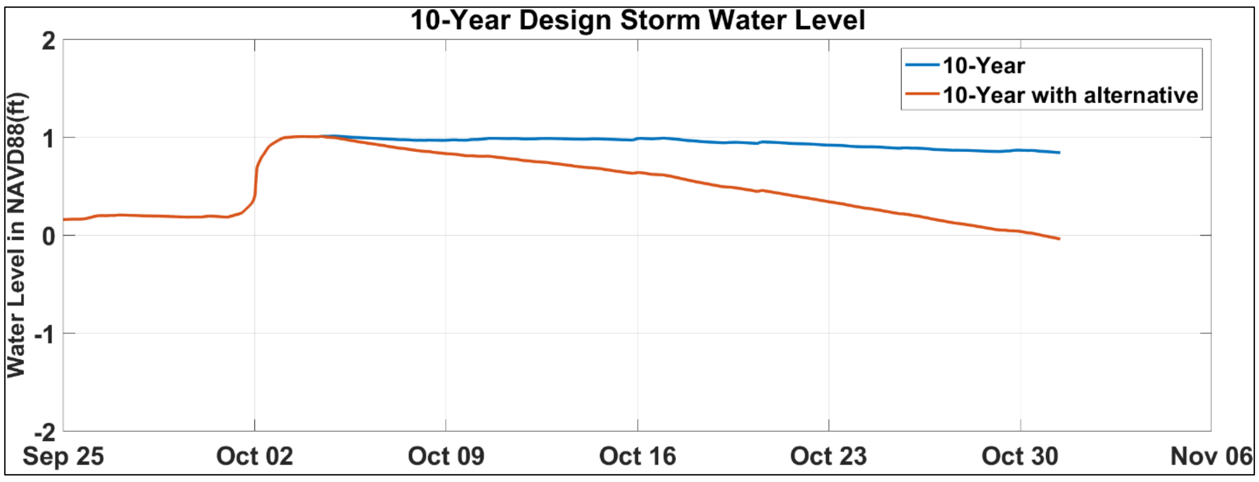

Figure 7. Lake water level increase and subsequent drawdown, 10-year design storm scenario, 350,000 gpm pump on the east basin, pump starts at peak water level.

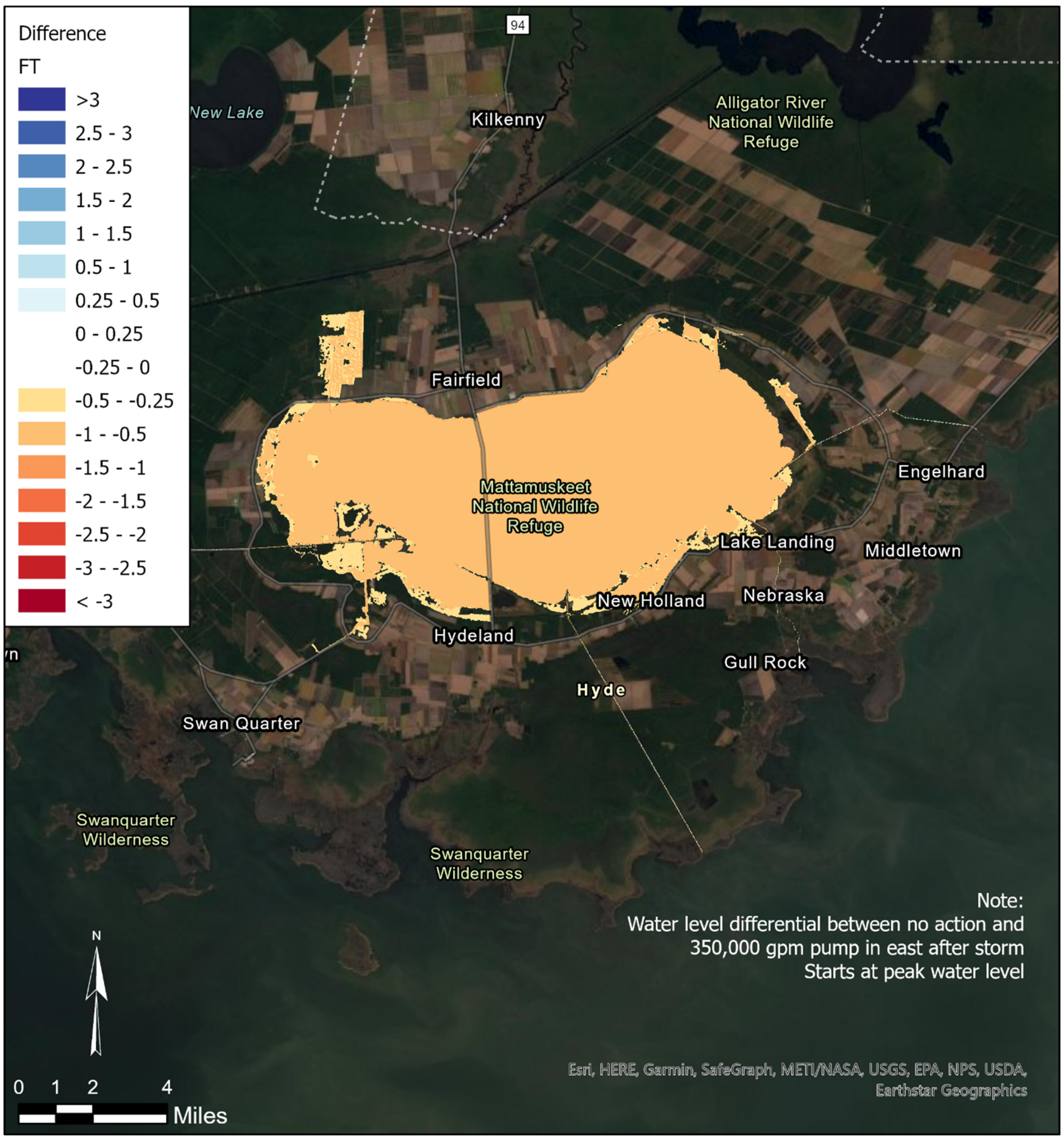

Figure 8. Water level differential between no action and 350,000 gpm pump on the east basin, pump starts at peak water level.

2.3 Multiple Sheet Flow Sites

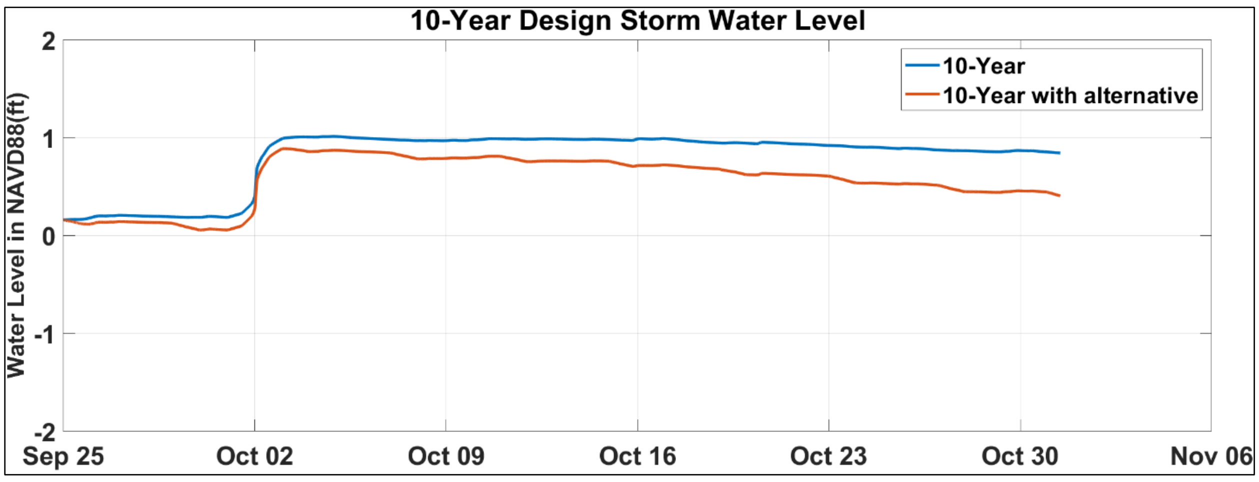

Figure 9. Lake water level increase and subsequent drawdown, 10-year design storm scenario, pumping to multiple sheet flow sites alternative, pump starts at beginning of simulation.

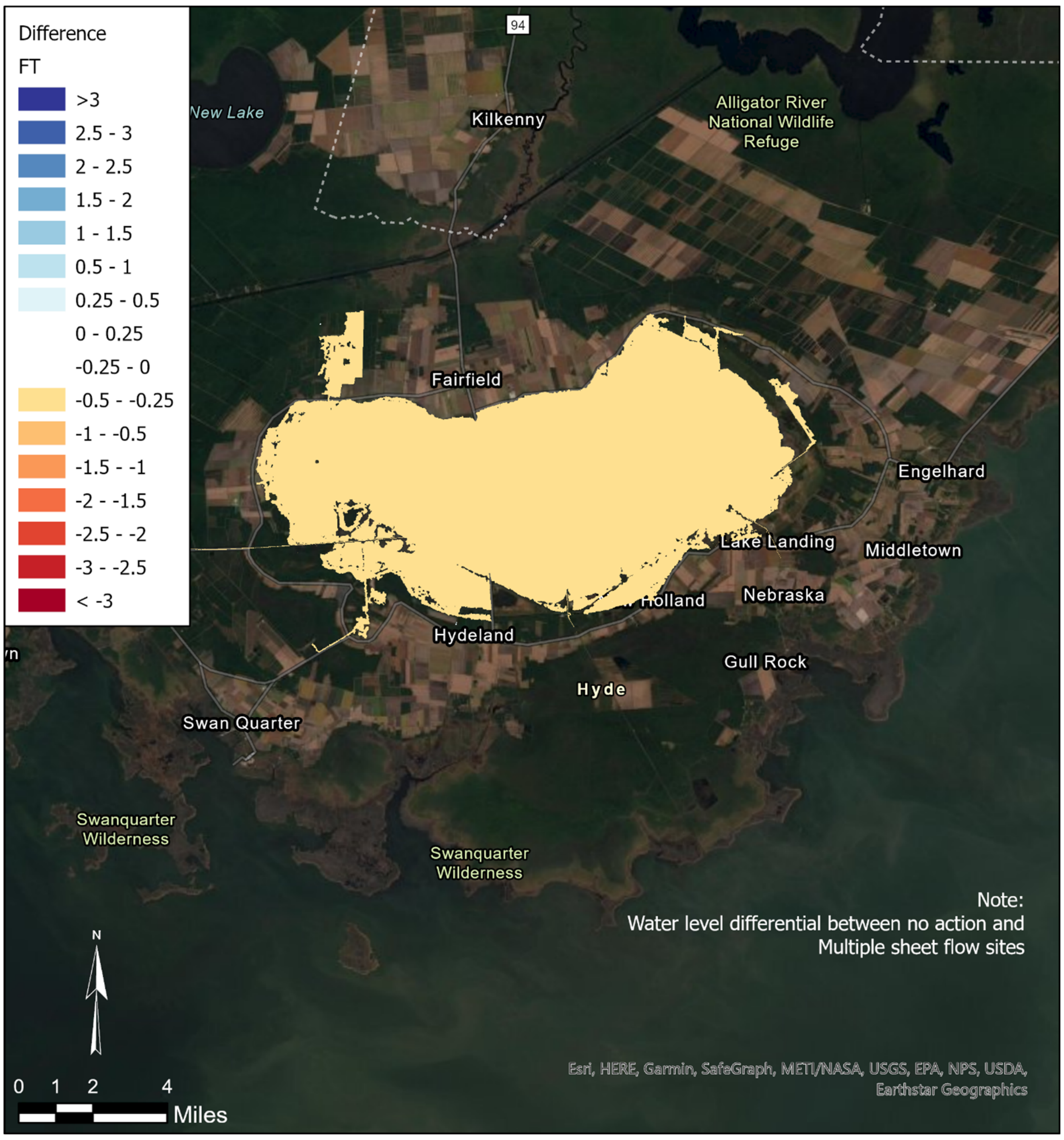

Figure 10. Water level differential between no action and pumping to multiple sheet flow sites alternative, pump starts at beginning of simulation.

2.4 Dredging Existing Outlet Canals

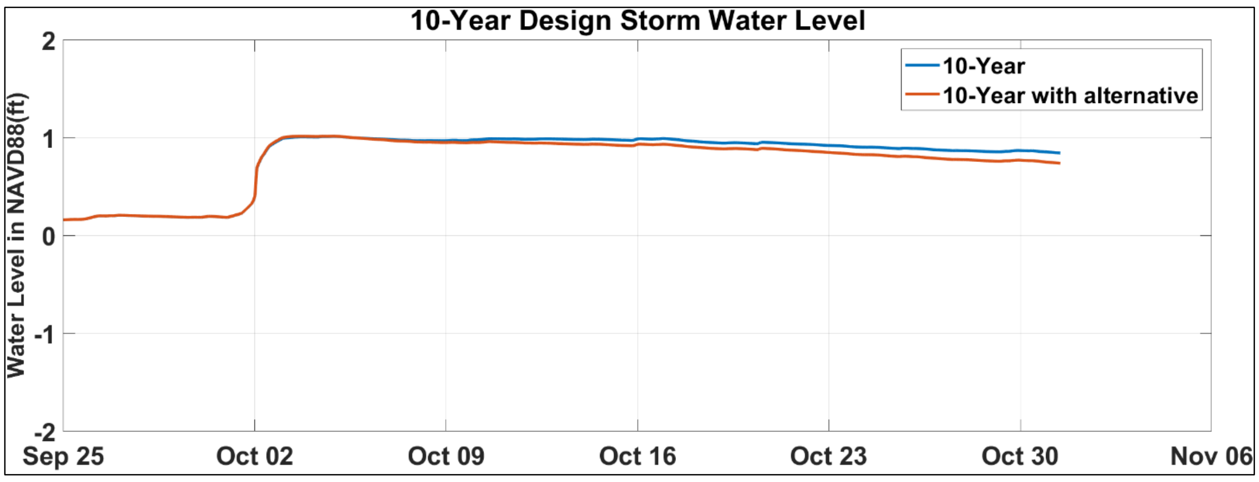

Figure 11. Lake water level increase and subsequent drawdown, 10-year design storm scenario, dredging existing outlet canals.

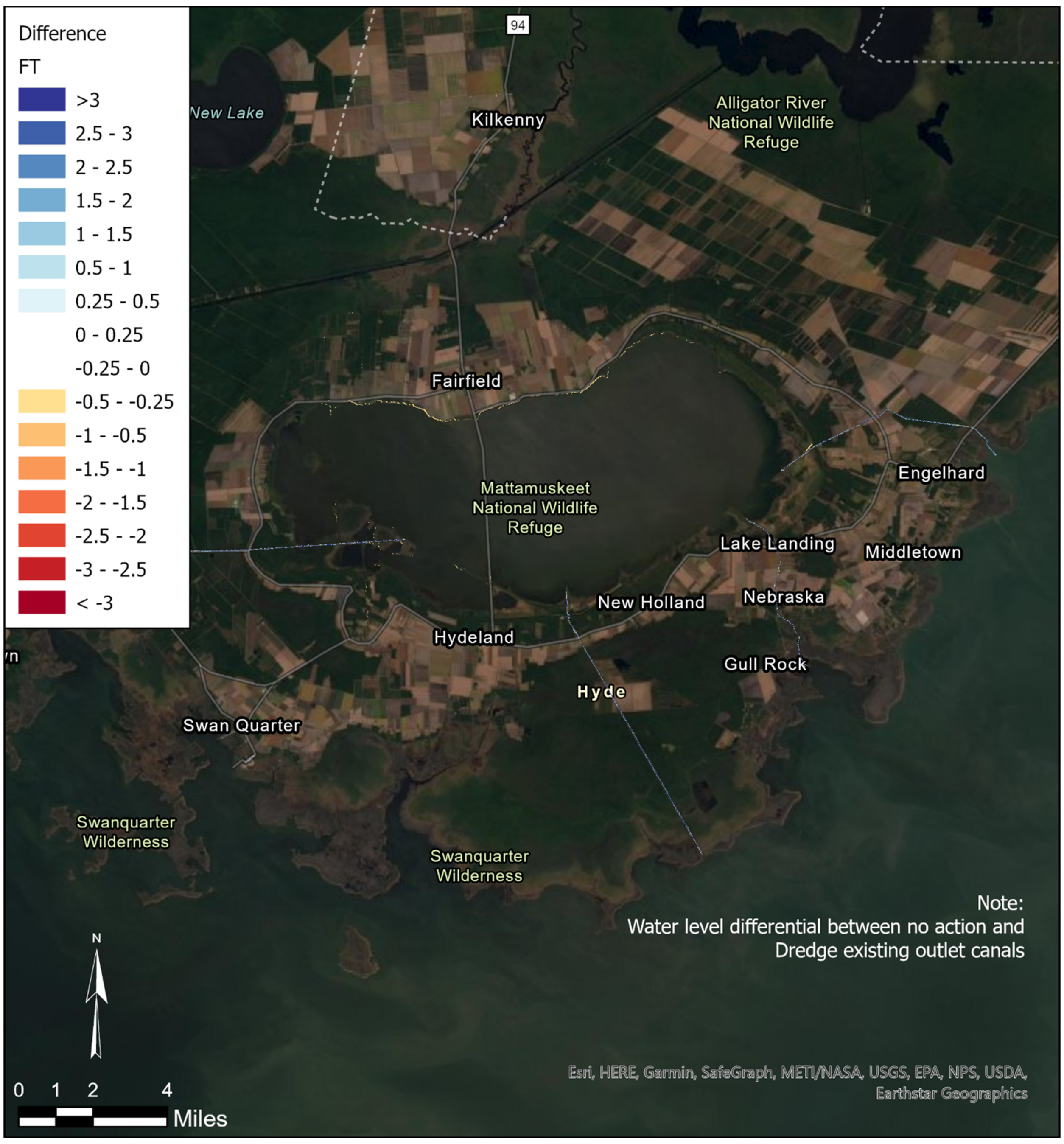

Figure 12. Water level differential between no action and dredging existing outlet canals alternative.

2.5 Removal of Tide Gates

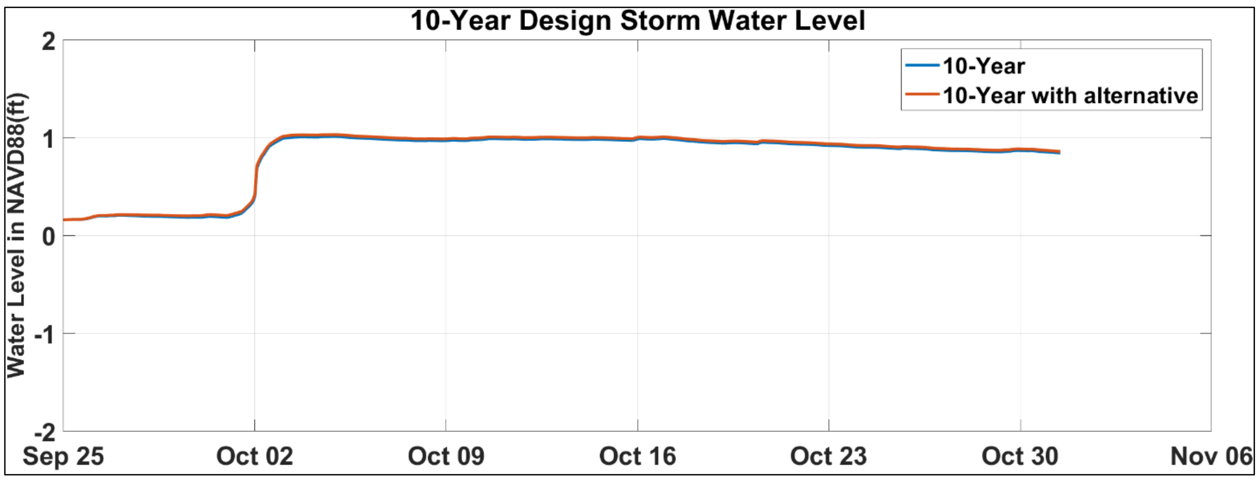

Figure 13. Lake water level, 10-year design storm scenario, removal of tide gates.

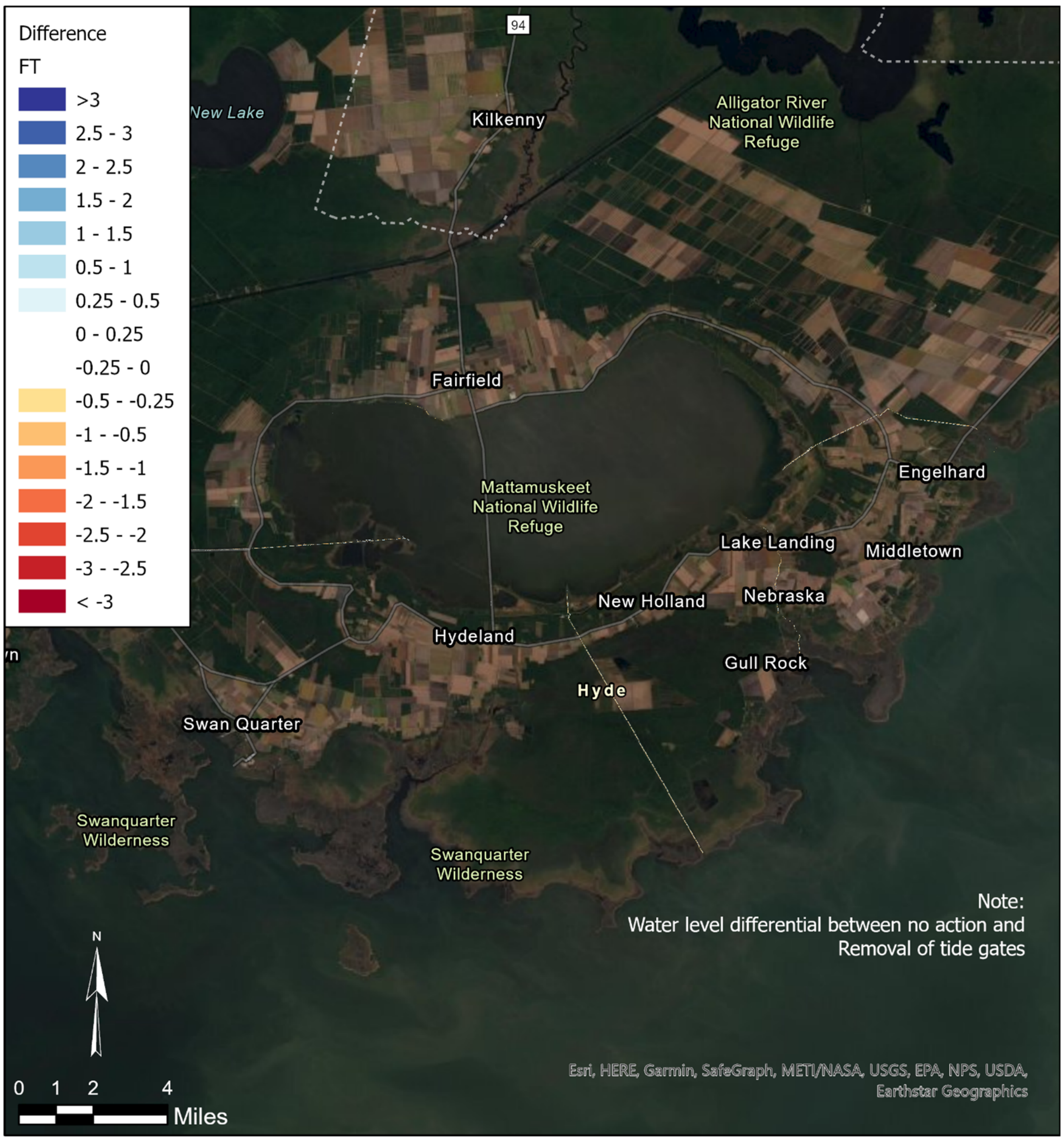

Figure 14. Water level differential between no action and removal of tide gates alternative.

2.6 Gravity Drained Canals to Adjacent Drainage Districts: Two Canals

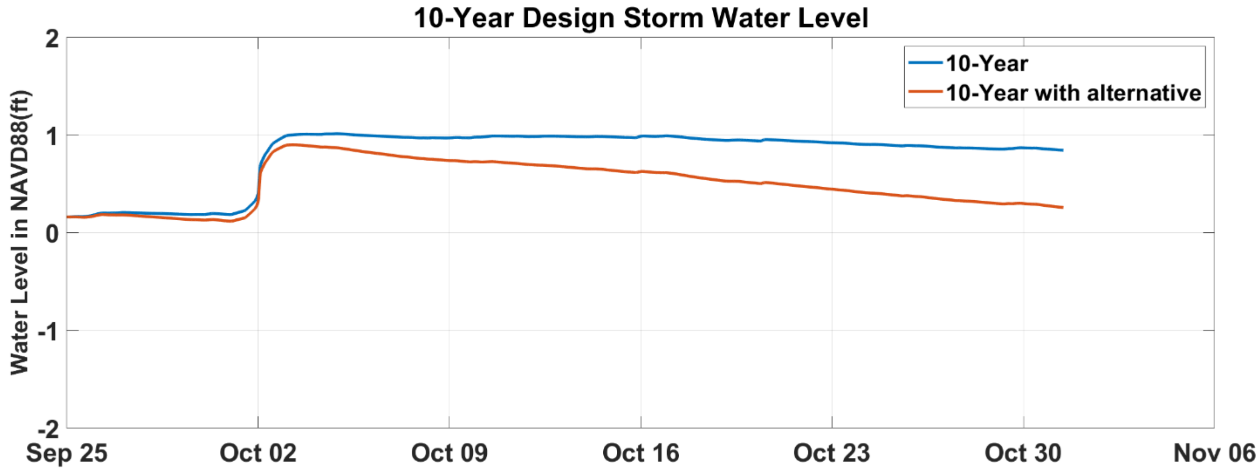

Figure 15. Lake water level increase and subsequent drawdown, 10-year design storm scenario, 2 gravity drained canals.

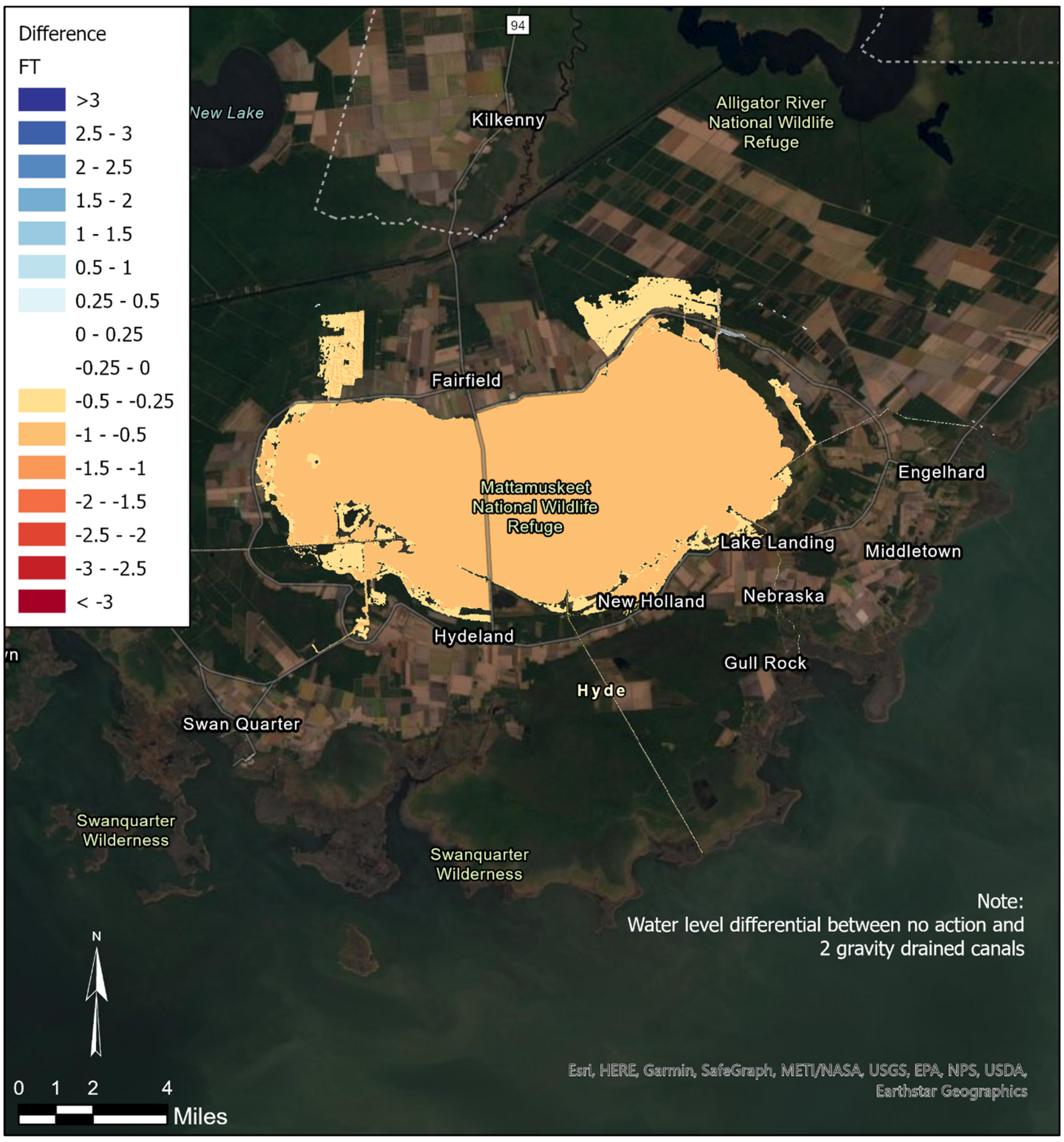

Figure 16. Water level differential between no action and 2 gravity drained canals alternative.

2.7 Gravity Drained Canals to Adjacent Drainage Districts: Three Canals

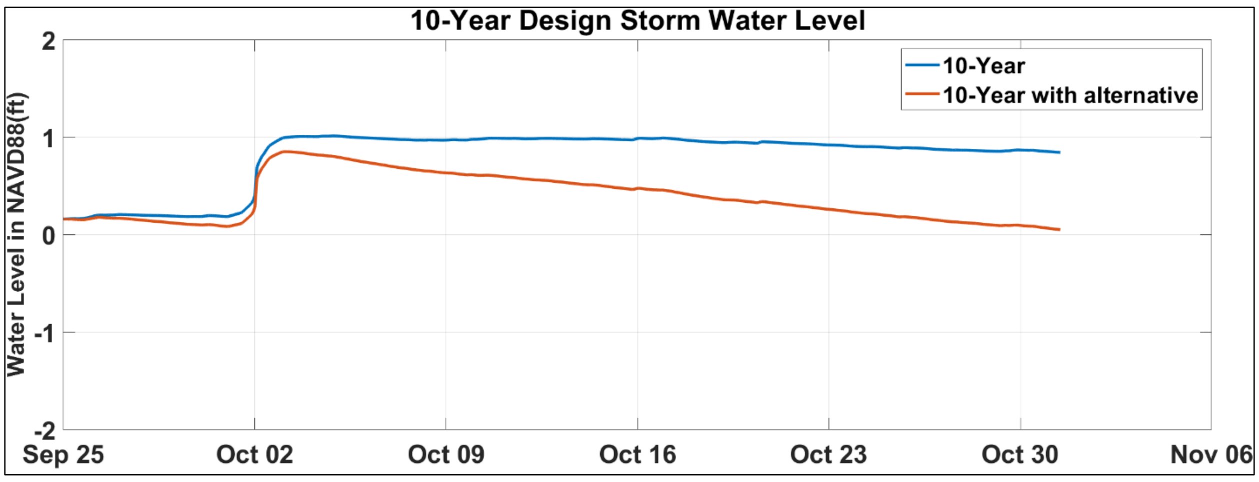

Figure 17. Lake water level increase and subsequent drawdown, 10-year design storm scenario, 3 gravity drained canals.

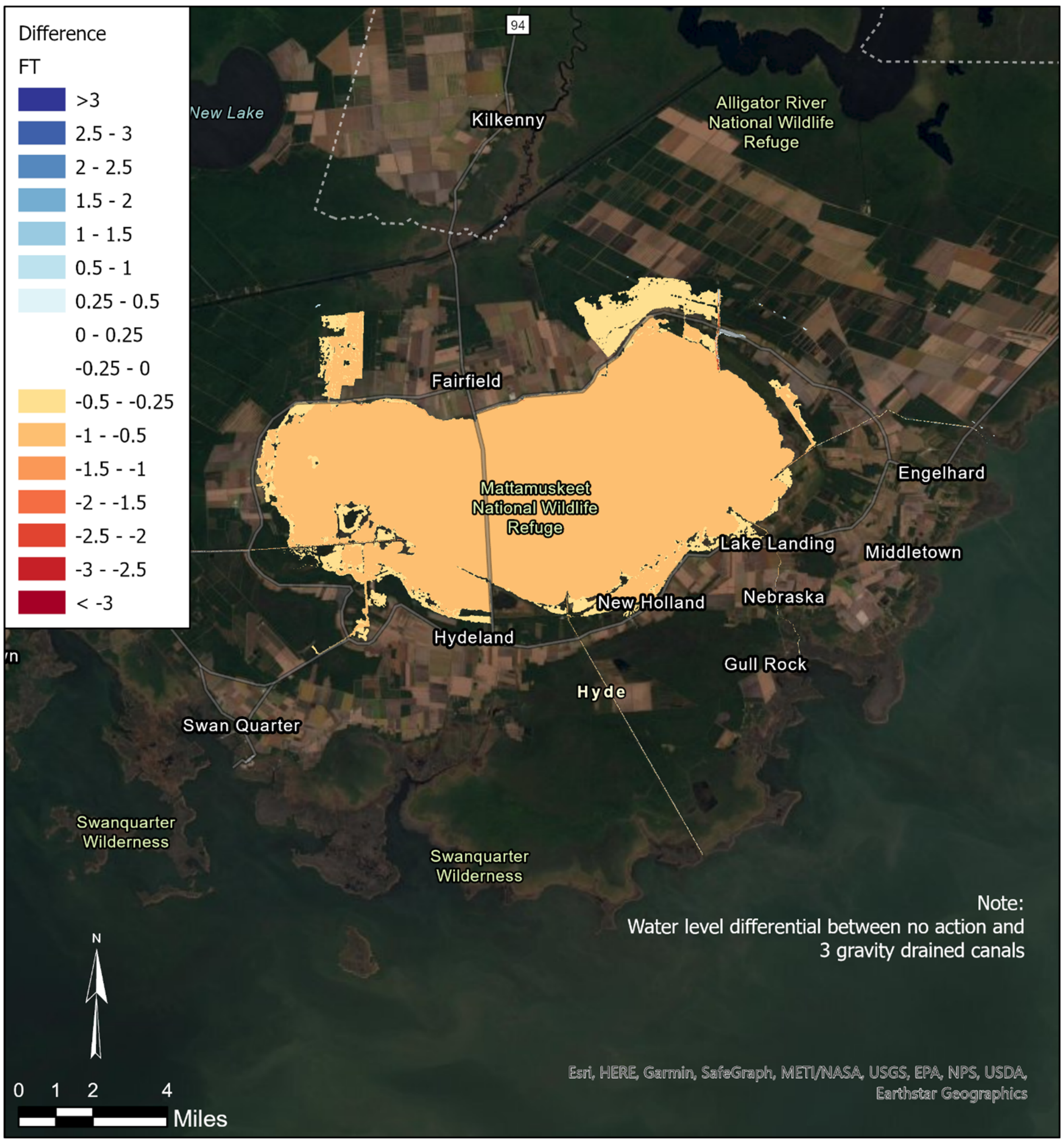

Figure 18. Water level differential between no action and 3 gravity drained canals alternative.

2.8 Centralized Pump Station to Intracoastal Waterway with Optimized Pump Capacity (500,000 gpm)

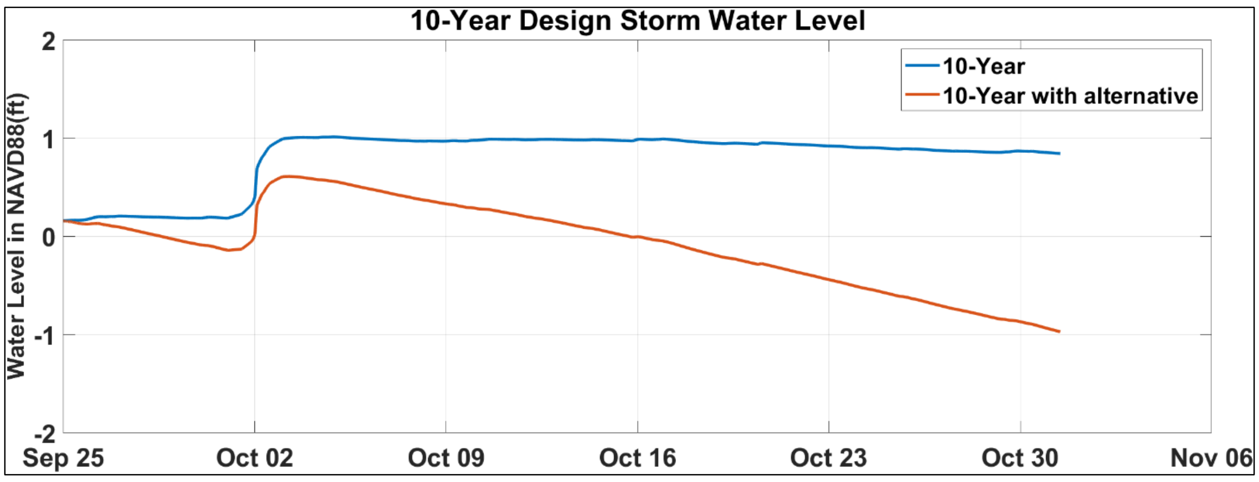

Figure 19. Lake water level increase and subsequent drawdown, 10-year design storm scenario, 500,000 gpm optimized pump.

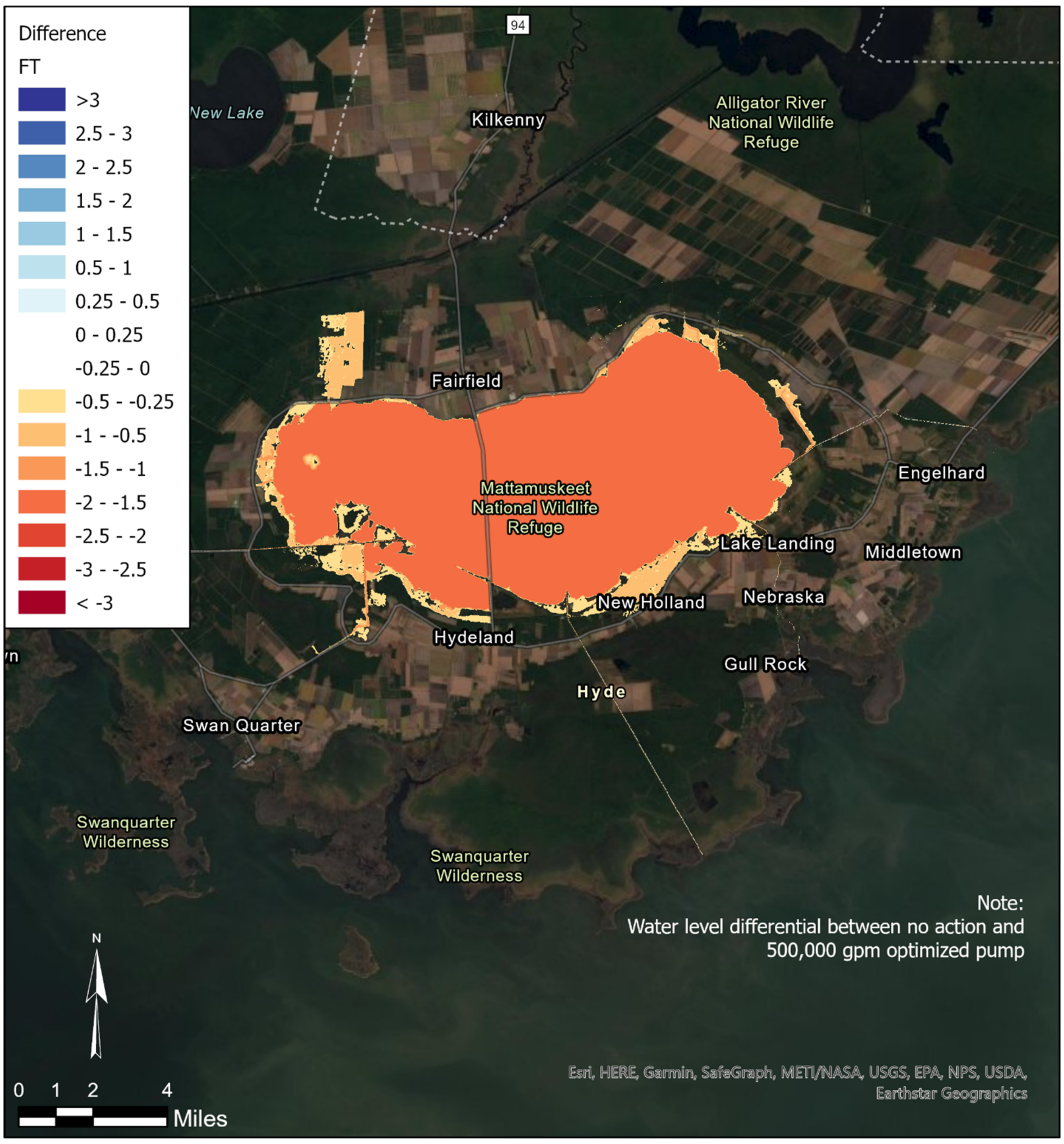

Figure 20. Water level differential between no action and 500,000 gpm optimized pump alternative.

APPENDIX B
Wetland Siting and Capacity Analysis

APPENDIX B: WETLAND SITING AND CAPACITY ANALYSIS

1. INTRODUCTION

This appendix summarizes the evaluation of six potential sheet flow sites for their suitability to both: 1) manage runoff volume from design storm events and 2) provide temporary storage to facilitate the drawdown of Lake Mattamuskeet and achieve desired seasonal water levels. Prior to hydromodification within the Lake Mattamuskeet watershed, the natural hydrology of Lake Mattamuskeet flowed north towards the Alligator River (NCCF, 2018); the lake is now drained via four outlet canals along the southern border of the lake shore. Therefore, there is interest in restoring that natural flow of water from the lake towards the north by actively diverting water from the lake or lake watershed to constructed wetlands north of the lake towards the Alligator River. These constructed wetlands, or "sheet flow sites", would serve as both temporary storage for flood mitigation and improve water quality of the diverted water.

This appendix summarizes the wetland siting and capacity analysis that was conducted to evaluate and prioritize sheet flow sites for their suitability and use in actively managing water within the Lake Mattamuskeet Watershed. This appendix also summarizes the design parameters, including optimized pumping rates, that were developed to support modeling the sheet flow sites in the comprehensive hydrologic and hydraulic (H&H) model, Delft 3D Flexible Mesh (FM) (Deltares, 2021), as described in the main body of the report.

2. METHODS

2.1 Storage Capacity Needs for Managing Water

To estimate the volume of water that would need to be managed in the potential sheet flow sites to achieve active water management goals of both seasonally lowering the lake and managing water generated from storm events, the volume generated from design storms and the storage required to achieve a range of desired lake levels were both assessed; these methods are described in Sections 2.1.1 and 2.1.2, respectively.

2.1.1 Design Storm Runoff Volumes

The volumes generated from the 2-, 5-, 10-, 25-, and 100-year design storms were obtained from the results of the H&H model, as described in the main body of the report. A stage-storage curve was generated in 0.25 ft increments from the digital elevation model (DEM) developed for the

H&H model as described in Section 2.2 of the main report (Table 1). The volume generated from each design storm event was approximated based on the volumetric difference between the minimum and maximum lake levels predicted from the existing conditions model simulations. Volumes were interpolated from Table 1.

Table 1. Stage-storage curve for Lake Mattamuskeet based on the digital elevation model used in the comprehensive H&H model.

Elevation (ft, NAVD88)	Volume (ac-ft)
-2.00	47,597
-1.75	57,035
-1.50	66,764
-1.25	76,563
-1.00	86,379
-0.75	96,211
-0.50	106,072
-0.25	116,017
0.00	126,025
0.25	136,273
0.50	147,247
0.75	159,015
1.00	171,508
1.25	184,566
1.50	197,994
1.75	211,762
2.00	225,834
2.25	240,146
2.50	254,637
2.75	269,278
3.00	284,051

Notes:
1. ac-ft = acre-feet
2. ft = feet
3. NAVD88 = North American Vertical Datum of 1988

2.1.2 Storage Required to Achieve Desired Lake Levels

Seasonal water levels within the lake were evaluated for the available period of record using gauge-height data measured at United States Geological Survey (USGS) Station No. 0208468892 (representing the east basin of Lake Mattamuskeet) and USGS Station No. 0208058893 (representing the west basin of Lake Mattamuskeet). Data were downloaded from the USGS Current Water Data for the Nation interface (USGS, 2020). The East Basin station period of record began September 20, 2012, and the west basin station period of record began October 1, 2013. The lake water level datasets were evaluated through November 2020, resulting in an approximate period of record of 8 years for the east basin and 7 years for the west basin.

It is noted that the gauge height is equivalent to 2 feet higher than the North American Vertical Datum of 1988 (NAVD88 datum) at these stations (e.g., a 0 ft gauge height corresponds to -2 ft NAVD88 and a 2 ft gauge height corresponds to 0 ft NAVD88) as shown in Figure 1. Minimum, maximum, and average gauge heights were evaluated for the period of record in each basin based on the following approximate seasonal designations: Winter (January, February, March), Spring (April, May, June), Summer (July, August, September) and Fall (October, November, December).

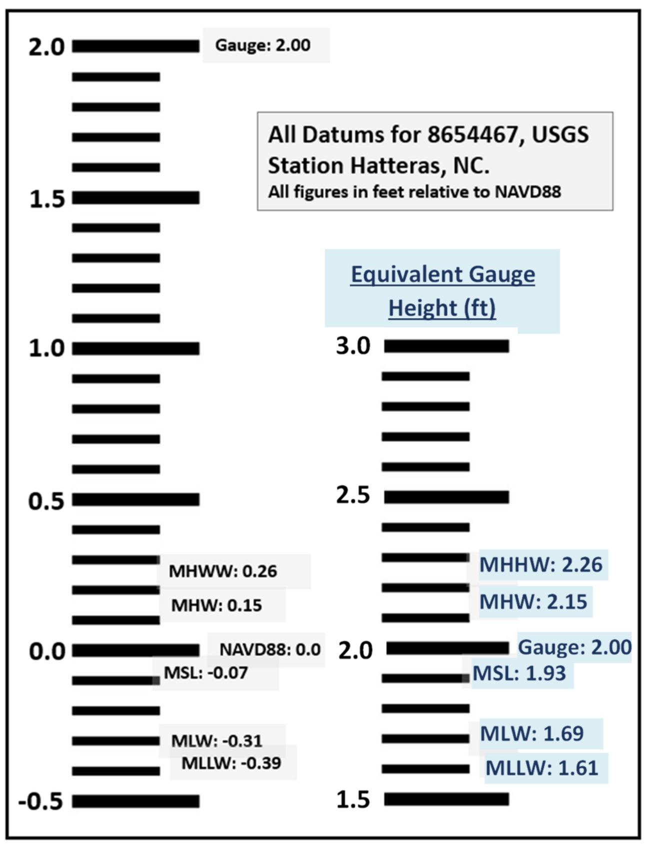

Figure 1. Comparison of NAVD 88 and gauge height datums for mean higher-high water (MHHW), mean high water (MHW), mean sea level (MSL), mean low water (MLW), and mean lower-low water (MLLW).

Desired lake levels were obtained from the Mattamuskeet Technical Working Group in October 2020. After consultation with stakeholders, the Mattamuskeet Technical Working Group indicated the desired lake levels ranged from 1.0 ft to 2.5 ft gauge height, dependent upon the season. Lower water levels were desired during growing season (March – early June) and higher water levels were desired during October to January (up to 2.5 ft) for recreational purposes. Therefore, the volume of storage that would be required to achieve desired lake levels was approximated on 0.5-ft increments (1.0 ft, 1.5 ft, 2.0 ft, and 2.5 ft gauge height) and based on the volumetric difference between the average seasonal water levels in the lake and the desired lake water levels.

2.2 Potential Sheet Flow Sites

Six sites were evaluated for their suitability to be converted to constructed wetlands and accept water that is pumped or diverted from the watershed (e.g., "sheet flow sites") (Figure 2). Four of the potential sheet flow sites that were evaluated were initially identified in the Lake Mattamuskeet Watershed Restoration Plan (NCCF, 2018), including the Ben Simmons/Joey Ben Williams property, Gull Rock Game Land Carter Tract, the Kelly Davis property, and the Pat Simmons property. Two additional sites, the Tierney property and White Tail Farms, were selected for evaluation based on input from the project team and stakeholders. Characteristics of the evaluated sheet flow sites are provided in Table 2.

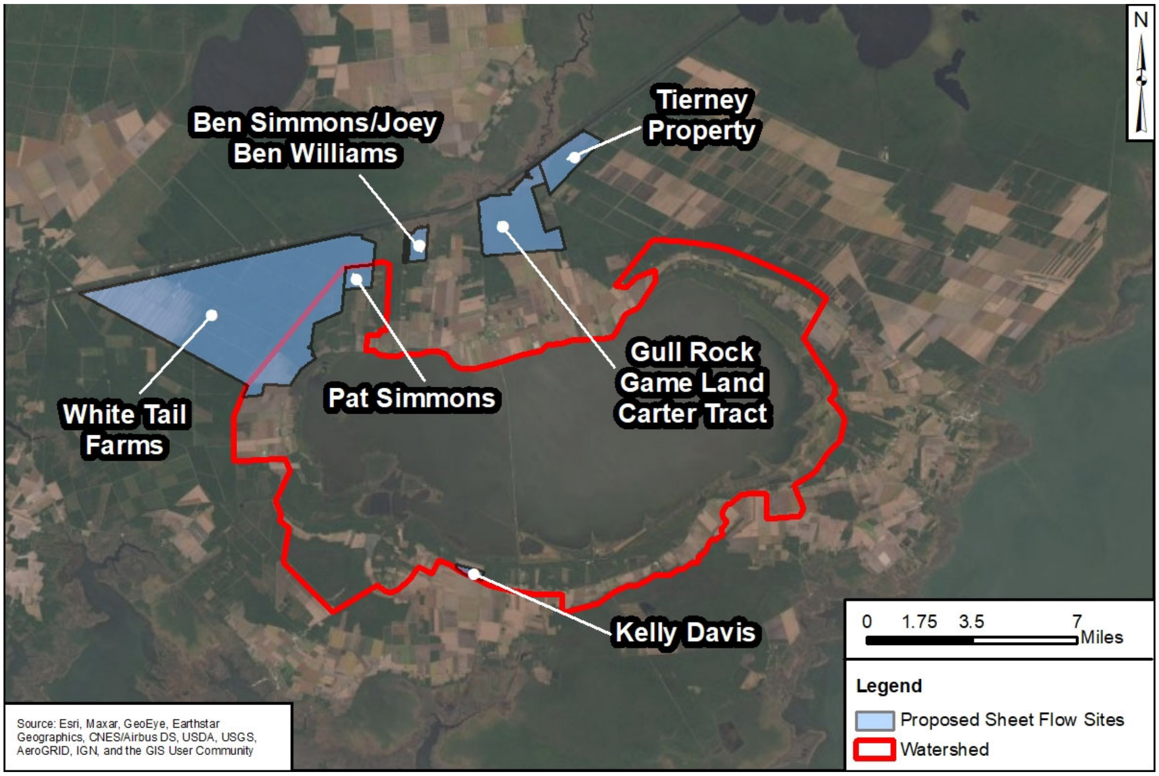

Figure 2. Location of potential sheet flow sites.

Table 2. Characteristics of Potential Sheet Flow Sites

Potential Sheet Flow Site	Area (ac)	Minimum Elevation (ft, NAVD88)	Maximum Elevation (ft, NAVD88)	Average Elevation (ft, NAVD88)	Shortest Distance to Lake Shoreline (mi)	Inside/Outside the Lake Mattamuskeet Watershed
Ben Simmons/ Joey Ben Williams[1]	338	-4.3	3	0.9	2.9	Outside
Gull Rock Game Land Carter Tract	2,139	-5.6	6.5	1.1	2.6	Outside
Kelly Davis	95	0.2	4.3	1.4	0.5	Inside
Pat Simmons	294	-2.2	3.7	0.6	1.8	Inside

Potential Sheet Flow Site	Area (ac)	Minimum Elevation (ft, NAVD88)	Maximum Elevation (ft, NAVD88)	Average Elevation (ft, NAVD88)	Shortest Distance to Lake Shoreline (mi)	Inside/Outside the Lake Mattamuskeet Watershed
Tierney Property	791	-5.5	5.7	1.2	4.1	Outside
White Tail Farms	10,792	-0.8	15.7	6.3	0.8	Primarily Outside

Notes:

1. The Ben Simmons property and Joey Ben Williams properties were evaluated as one because they are contiguous. The range of elevations and distance to lake
2. ac-ft = acre-feet
3. ft = feet
4. NAVD88 = North American Vertical Datum of 1988

2.3 Wetland Siting Suitability Matrix

To assess the suitability of each potential site for conversion to constructed wetlands, the following factors were evaluated: potential storage capacity, suitability of soils, presence of environmental features, flood risk, constructability, and permitting. These assessments are described in further detail in Sections 2.3.1 through 2.3.6.

2.3.1 Potential Storage Capacity

The potential storage capacity of each sheet flow site was estimated based on a standardized constructed wetland design. The standardized constructed wetland design assumed a 12-inch temporary storage zone with minimal grading at the site (NCDEQ, 2020). The temporary storage zone was assumed to be created by installing water control structures at the outlet canals or drainage features at each site to control the normal pool of the constructed wetland. The crest elevation of the weir was assumed to be similar to the average elevation of the site and would create flooded areas in the portions of the site that are lower than the weir elevation. To create temporary storage, it was assumed a perimeter berm around the site would be constructed at a minimum height of 12 inches above the weir elevation. The temporary storage volume, or potential storage capacity, is therefore the volume available in the 12 inches between the crest of the perimeter berm and the crest of the weir.

The surface volume tool in ArcGIS was used to create stage-storage curves for each wetland. The potential storage capacity was computed by evaluating the volumetric difference between the temporary storage elevation and the weir elevation. Due to the large size and topographic relief on White Tail Farms, the parcel was divided into five zones to maximize potential storage for this analysis; four of the five zones were located outside the Lake Mattamuskeet watershed and therefore included in the potential storage capacity. Sheet flow sites with more temporary storage were considered more suitable for wetland siting.

2.3.2 Suitability of Soils

The soil types present on each potential sheet flow site were assessed using the NRCS gridded Soil Survey Geographic Database (SSURGO) (Soil Survey Staff, 2020). Soils that were primarily mucky and frequently flooded were considered to be more suitable for use as a sheet flow site because these soils are appropriate for a wetlands application; soils that were sandy, loamy, or rarely flooded were considered to be less suitable for use as a sheet flow site, in part due to the conversion of the ecosystem type.

2.3.3 Presence of Environmental Features

Each potential sheet flow site was evaluated for the presence of environmental features through review of the USGS National Hydrography Dataset (NHD). The NHD is a comprehensive set of spatial data that identifies naturally occurring and constructed bodies of surface water (e.g., ponds, swamps, marshes, and lakes) and drainage features and waterways (e.g., canals, ditches, streams, and rivers). Sheet flow sites with a large presence of stream and wetland features on site were considered less suitable for a constructed wetland due to the potential environmental impact of disturbing existing features.

2.3.4 Flood Risk

The flood risk for each potential sheet flow site was assessed using Federal Emergency Management Agency (FEMA) flood maps. The approximate percentage of the site in the AE flood zone (areas that present a 1% annual chance of flooding) was evaluated. Sheet flow sites with a higher percentage of area in the flood zone were considered less suitable for wetland siting because those sites are at a higher probability of being inundated with floodwaters during larger storm events, and therefore their potential storage capacity would not be available for diversion of water. Offsite increases in volume and flow rates were not assessed, but should be considered in the detailed design phase.

2.3.5 Constructability

The evaluation of the ease of constructability of the sheet flow sites considered two primary factors: distance from the lake and topography. Sheet flow sites that were farther from the lake would incur more canal improvements or require a longer distance of pipe installation to divert water from the lake watershed to the sheet flow site and were therefore considered less desirable. Sites that had less topographic relief were considered more suitable because site access and berm construction would be easier. Sites with greater topographic relief were considered less suitable because these sites would likely require the construction of check dams or varying berm elevations to maximize potential storage capacity.

2.3.6 Permitting

For the permitting evaluation, most of the sheet flow sites present similar challenges from a permitting perspective regarding the likely need for Section 404 Individual Permits and Coastal Area Management Area Management Act (CAMA) Major Permits. However, one differentiator that was evaluated was the potential need for an Interbasin Transfer (IBT) certification. Large surface water transfers between river basins were regulated in 1993 by General Statute G.S. §143-215.22I as part of An Act to Regulate Interbasin Transfers (Session Law 1993-348). In general, transfer certificates are required for a new, hard-piped transfers of 2 million gallons per day or more between river basins. The IBT line separating the Tar River Basin and Pasquotank River Basin arbitrarily crosses several of the sheet flow sites (Figure 3), and therefore water that is pumped from the Lake Mattamuskeet watershed to the sheet flow site and crosses the IBT line could require the IBT certification, which is a lengthy process requiring approval from the Environmental Management Commission. If the property is entirely in the Tar Basin, the diversion of water would not require an IBT certification and permitting was therefore considered “feasible”. If the property straddled the Tar and Pasquotank Basin, permitting was considered “possible” because water could be pumped to a point within the Tar Basin and naturally flow across the Pasquotank Basin. Properties that were entirely in the Pasquotank Basin were considered to have “difficult” permitting as these sites would require a hardened pipe crossing the IBT line to transfer water from the Lake Mattamuskeet watershed to the potential site.

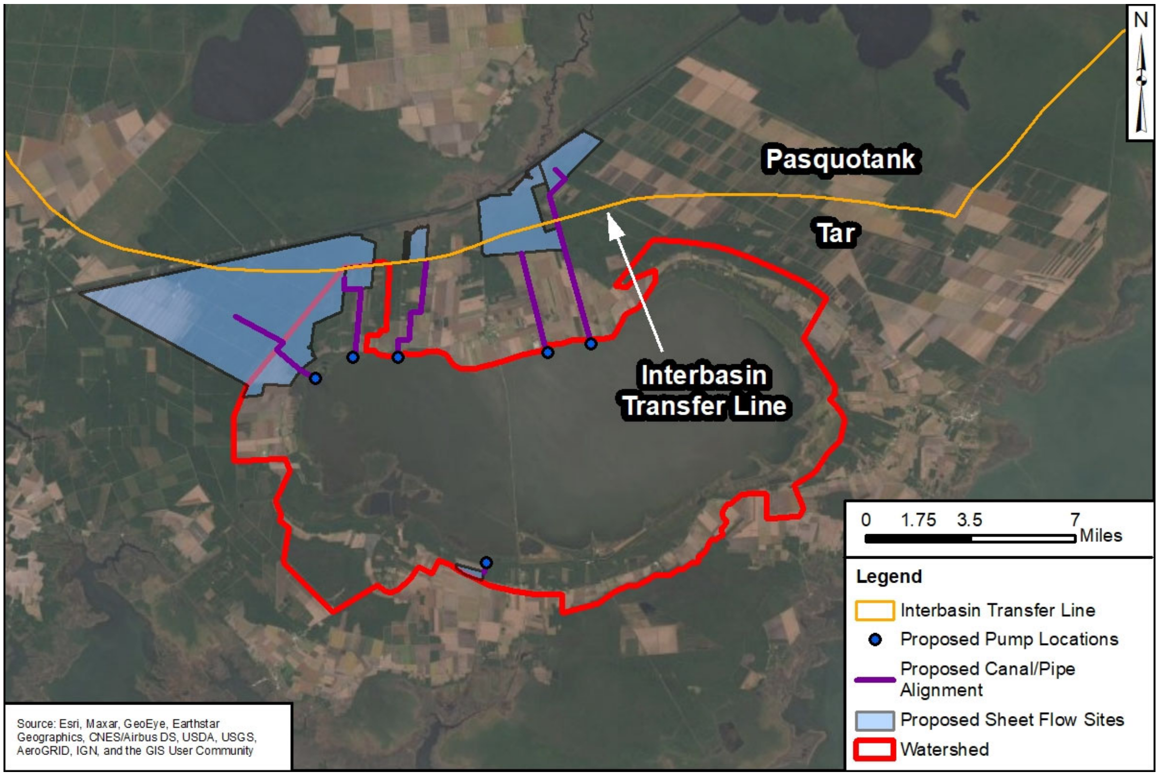

Figure 3. Location of interbasin transfer line.

2.4 Evaluation of Pumping Needs for Model Simulation

To evaluate the effect of diverting water from the lake watershed to the sheet flow sites on lake water levels in the comprehensive H&H model, an optimized pumping rate was calculated for each potential sheet flow site. This evaluation required a three-step process: 1) computing an initial pumping rate based on the temporary storage volume, 2) evaluating the discharge capacity of the site to assess whether the volume of water pumped to the site could drain in a reasonable amount of time, and 3) optimizing the pumping rate for sheet flow based on results from the first two steps. These steps are described in Sections 2.4.1 through 2.4.3. This optimized pumping rate was assessed preliminarily for inclusion in the H&H model and would be further refined if the sheet flow sites management option were selected as the preferred alternative to advance to the conceptual design phase.

2.4.1 Temporary Storage Volume and Initial Pumping Rate

An initial pumping rate was selected for each sheet flow site based on the available temporary storage volume in approximately 12 inches of storage without any significant grading, as discussed in Section 2.2.1. The initial pumping rate was selected to transfer a water volume equivalent to the temporary storage volume in 24 hours.

2.4.2 Discharge Capacity

The standardized design of a constructed wetland assumes the volume diverted to the constructed wetland circulates through the site and is treated and discharged from the site within 2 to 5 days (NCDEQ, 2020). To assess whether the temporary storage volume pumped to each site could be discharged within 48 hours, the discharge capacity was evaluated assuming each existing drainage feature or outlet canal on the site that discharged to the Alligator River, Intracoastal Waterway, or other drainage pathway would be equipped with a water control structure with a weir elevation set at the normal pool of the constructed wetland. The width of each outlet canal was assessed based on aerial imagery; it was assumed the weir length was equivalent to the outlet canal width. The discharge capacity was calculated using the weir equation (Eq. 1) with the approximate total discharge weir length at the site and a maximum head of 12 inches.

The weir equation is as follows:

$$Q = 2.6 * L * H^{1.5} \tag{1}$$

where Q = discharge rate of weir (cubic feet per second, cfs)
L = total weir length (feet, ft)
H = weir head, assumed to be a maximum of 1 ft (feet, ft)

The maximum volume that could be discharged from each site over 48 hours based on this discharge rate was then calculated. A 48-hour drawdown time was selected to restore the available storage capacity in the sheet flow site more quickly while still meeting drawdown requirements for water quality treatment. If the volume pumped to the sheet flow site over 24 hours as evaluated in Section 2.4.1 was greater than the maximum discharge volume over 48 hours, the pumping rate to the site was reduced such that the volume pumped to the site in 24 hours was no greater than the maximum discharge volume in 48 hours (i.e., the pumping rate for pumping into the wetland site was set to be no greater than twice the maximum discharge rate for flow out of the wetland).

2.4.3 Optimized Pumping Rate for Sheet Flow

To select the optimized pumping rate for sheet flow, results from the first two steps were combined to confirm that sheet flow (e.g., a velocity of less than 0.5 feet per second [ft/s]) was achievable based on the topography of the site and the rate of pumping to the site. Lower velocities both minimize erosion and result in a more suitable flow regime for water quality treatment. Manning's equation (Eq. 2) was used to back-calculate the depth of flow and, subsequently, the velocity across the site based on the maximum pumping rate identified in Section 2.4.2.

Manning's equation is as follows:

$$Q = \frac{1.49}{n} * A * R^{2/3} * S^{1/2} \quad (1)$$

where Q = sheet flow rate (cfs)
n = Manning's n coefficient, selected as 0.40 for sheet flow (unitless)
A = cross-sectional area of sheet flow, equivalent to the sheet flow depth multiplied by the sheet flow width (ft^2)
R = hydraulic radius for the rectangular section of sheet flow (unitless)
S = average slope across the site, evaluated in ArcMap (ft/ft)

The sheet flow width for each site was assumed to be the average width of the site perpendicular to the outlet canals. The sheet flow depth was back-calculated from the known sheet flow width and maximum pumping rate. For each optimized pumping rate, the sheet flow depth was confirmed to be less than 12 inches, and the sheet flow velocity was confirmed to be less than 0.5 ft/s.

3. RESULTS AND DISCUSSION

Results and discussion from the wetland siting and capacity analysis are described in Sections 3.1 through 3.3.

3.1 Design Storm and Seasonal Storage Needs

The runoff volumes generated by the 2-, 5-, 10-, 25-, 50-, and 100-year design storms are presented in Table 3. The runoff volume generated by a design storm event is an estimate of the volume of water that would need to be stored or managed after a given storm to return the lake to the water level prior to the storm. Approximate runoff volumes ranged from 23,600 ac-ft to 75,900 ac-ft for the 2-year and 100-year design storms, respectively.

Table 3. Summary of Approximate Design Storm Runoff Volumes

Design Storm Event	Approximate Design Storm Runoff Volume	
	ac-ft	**Million gallons**
2-year	23,600	6,610
5-year	31,900	8,940
10-year	39,400	11,000
25-year	52,000	14,600
50-year	63,300	17,700
100-year	75,900	21,200

Notes:
1. ac-ft = acre-feet

The recorded gauge heights at the east basin and west basin were evaluated for seasonal variation and are summarized in Table 4. In general, average gauge heights were lowest in the summer and highest in the winter, which is consistent with the management goals of stakeholders within the watershed. However, average gauge heights in the summer (1.92 ft gauge height in the East Basin and 2.03 ft gauge height in the west Basin) were considerably higher than the desired lake level of 1.0 – 1.5 ft gauge height. Average gauge heights in the winter (2.33 ft in the east basin and 2.40 ft in the west basin) were also lower than the desired lake level of 2.5 ft gauge height.

Table 4. Summary of recorded gauge heights in the east and west basins of Lake Mattamuskeet.

Season	East Basin[1]			West Basin[2]		
	Minimum Gauge Height (ft)	**Average Gauge Height (ft)**	**Maximum Gauge Height (ft)**	**Minimum Gauge Height (ft)**	**Average Gauge Height (ft)**	**Maximum Gauge Height (ft)**
Winter	0.96	2.33	3.40	1.20	2.40	3.63

Season	East Basin[1]			West Basin[2]		
	Minimum Gauge Height (ft)	Average Gauge Height (ft)	Maximum Gauge Height (ft)	Minimum Gauge Height (ft)	Average Gauge Height (ft)	Maximum Gauge Height (ft)
(January, February, March)						
Spring (April, May, June)	1.10	2.11	2.97	1.27	2.14	2.99
Summer (July, August, September)	0.85	1.92	3.33	1.22	2.03	3.18
Fall (October, November, December)	0.52	2.19	3.74	0.76	2.24	3.82

Notes:
1. Data summarized from USGS Station No. 0208468892 with a period of record of 9/20/2012 – 11/02/2020.
2. Data summarized from USGS Station No. 0208058893 with a period of record of 10/01/2013 – 11/02/2020.
3. Lake water levels presented in gauge height, which corresponds to 2 ft above NAVD88.
4. ft = feet.

The storage volume required to achieve desired lake levels based on average summer and winter gauge heights is summarized in Table 5. The summary provides a range of volumes that would be required to achieve different lake levels; note the average presented is the average of the combined east and west basins. Up to 38,800 ac-ft of storage volume would be required to lower the lake from the average summer gauge height (1.98 ft gauge height) to the desired lake level of 1.0 ft gauge height. These volumes are useful for providing context on whether the six potential sheet flow sites could provide the required storage capacity to achieve these management goals.

Table 5. Storage required to achieve desired lake levels.

Desired Lake Level Gauge Height (ft)	Estimated Volumetric Difference Between Desired Lake Level and Average Lake Level (ac-ft)	
	Average Winter Gauge Height (2.37 ft)	Average Summer Gauge Height (1.98 ft)
1.0	55,200	38,800
1.5	35,500	19,200
2.0	15,500	1,230
2.5	-5,710	-22,000

Notes:
1. Desired lake levels obtained from Mattamuskeet Technical Working Group and vary based on season.
2. Lake water levels presented in gauge height, which corresponds to 2 ft above NAVD88.
3. Average lake levels presented represent the average of both the east and west Basins.
4. ac-ft = acre-feet
5. ft = feet.

3.2 Wetland Siting Suitability Matrix

A summary of the factors evaluated to assess wetland siting suitability for each site is provided in Table 6. Supporting documentation for the evaluation of these factors is provided in Attachment A.

The Gull Rock Game Land Carter Tract and White Tail Farms sheet flow sites provided the most storage capacity and therefore were categorized as "more suitable" overall, as this is the most critical factor for improving the drawdown of lake levels. The total storage capacity across all six sheet flow sites is approximately 8,500 ac-ft; Gull Rock Game Land Carter Tract and White Tail Farms provide approximately 85% of the total storage capacity.

The total storage capacity is approximately one-third of the volume generated by the 2-year design storm (Table 3). To effectively use the sheet flow sites for active water management, cyclical pumping to these sites would be needed to manage the estimated volumes of water described in Section 3.1. As an example, pumping water to all six sheet flow sites for 24 hours, allowing the water in the temporary storage zone to draw down for 48 – 72 hours, and then repeating this pumping cycle, could lower the lake from 1.98 ft gauge height (average summer gauge height) to 1.50 ft gauge height. Similarly, three cycles of pumping for 24 hours and allowing the water level in the sheet flow site to draw down for 48 – 72 hours would manage the volume of runoff generated by the 2-year design storm. The storage capacity provided by the sheet flow sites will be more useful in managing runoff volumes from smaller storms or lowering the lake levels when it is not raining compared to managing runoff volumes from larger storms such as hurricanes.

The other four sites (Ben Simmons/Joey Ben Williams, Kelly Davis, Pat Simmons, and Tierney Property) were all categorized as "suitable" overall. The Pat Simmons property was categorized as "suitable" overall due to the presence of suitable soil types, minimal presence of environmental features, and feasible constructability and permitting; however, the site has the second smallest potential storage capacity and would therefore potentially have a lower cost-benefit value. The Ben Simmons/Joey Ben Williams and Tierney properties present design challenges from a permitting perspective but could be conceivably used to manage water from the lake watershed. Similar to the Pat Simmons property, the Kelly Davis property is overall listed as "suitable" due to its limited storage capacity and therefore potential for having a lower cost-benefit value; however, it is still a feasible project due to ease of construction and permitting.

Table 6. Wetland siting suitability matrix.

Potential Sheet Flow Site	Storage Capacity (ac-ft)	Soil Type	Environmental Features	Flood Risk	Constructability	Permitting	Overall
Ben Simmons/Joey Ben Williams	295	Most Suitable	Large Presence	~ 100%	Feasible	Difficult	Suitable
Gull Rock Game Land Carter Tract	1,707	Most Suitable	Large Presence	~ 99%	Possible	Possible	More Suitable
Kelly Davis	79	Suitable	Large Presence	~ 30%	Feasible	Feasible	Suitable
Pat Simmons	220	Most Suitable	Minimal Presence	~ 100%	Feasible	Feasible	Suitable
Tierney Property	572	Most Suitable	Some Presence	~ 100%	Possible	Difficult	Suitable
White Tail Farms	5,575	Suitable	Minimal Presence	~ 5%	Difficult	Possible	More Suitable

Notes:

1. ac-ft = acre-feet

3.3 Optimized Pumping Rate for Sheet Flow

The design parameters used to evaluate the optimized pumping rate are summarized in Table 7. The optimized pumping rate that was simulated for each sheet flow site in the comprehensive H&H model is summarized in Table 8. As discussed in Section 3.2, cyclical pumping is proposed for the sheet flow sites to maximize the use of the available storage capacity. In the H&H model, pumping was simulated for 24 hours on and then 72 hours off. The optimized pumping rate for each sheet flow site varied between 18,000 gpm (Kelly Davis property) and 190,000 gpm (Gull Rock Game Land Carter Tract). If each sheet flow site were constructed and in operation, a total pumped rate of approximately 600,000 gpm dispersed across the watershed can be achieved.

In most cases, the optimized pumping rate resulted in a pumped volume within 24 hours that was less than the available storage capacity due to limitations of the discharge capacity. If the six sheet flow sites are selected as the preferred alternative to address to conceptual design, additional modeling and analysis would be conducted to refine the design of these sites.

Table 7. Design parameters used to evaluate optimized pumping rate.

Potential Sheet Flow Site	Weir Elevation	Temporary Storage Elevation	Temporary Storage Volume	No. of Outlet Canals	Outlet Canal Width	Total Weir Length	Maximum Discharge Capacity
	ft	ft	ac-ft	-	ft	ft	cfs
Ben Simmons/Joey Ben Williams	1.0	2.0	295	1	20	20	52
Gull Rock Game Land Carter Tract	1.0	2.0	1,707	4	30	120	312
Kelly Davis	1.5	2.5	79	1	15	15	20
Pat Simmons	0.5	1.5	220	2	20	40	56
Tierney Property	1.0	2.0	572	3	30	90	143
White Tail Farms	4.0 – 7.5[1]	5.0 – 8.5[1]	5,575	10	25	250	650

Notes:
1. Due to the large size of and topographic relief present at White Tail Farms, the site was divided into five zones to maximize potential storage at this site. Four of the five zones were located outside the Lake Mattamuskeet watershed and therefore included in the temporary storage volume.
2. ac-ft = acre-feet
3. cfs = cubic feet per second

4. ft = feet

Table 8. Summary of optimized pumping rate and subsequent sheet flow depth and velocity.

Potential Sheet Flow Site	Optimized Pumping Rate		Pumped Volume in 24 hours	Sheet Flow Depth	Sheet Flow Velocity
	gpm	cfs	ac-ft	ft	ft/s
Ben Simmons/Joey Ben Williams	47,000	103	206	0.20	0.08
Gull Rock Game Land Carter Tract	190,000	418	853	0.27	0.11
Kelly Davis	18,000	40	78	0.36	0.15
Pat Simmons	50,000	110	219	0.26	0.13
Tierney Property	130,000	286	571	0.43	0.17
White Tail Farms	180,000	396	796	0.17	0.13

Notes:
1. ac-ft = acre-feet
2. cfs = cubic feet per second
3. ft = feet
4. ft/s = feet per second
5. gpm = gallons per minute

4. CONCLUSIONS

The wetland siting and capacity analysis identified the storage required to manage both runoff volumes from various design storms and to achieve desired lake levels based on seasonal average lake levels. These volumes exceeded the potential storage capacity available in the six sheet flow sites that were evaluated; therefore, an optimized management strategy of cyclical pumping (e.g., 24 hours on, 72 hours off) is recommended to allow the sheet flow sites a minimum of 48 hours to draw down and restore the available storage capacity. Using this management strategy, the sheet flow sites could be used to draw down the lake to desired levels over several pumping cycles. Optimized pumping rates were identified for each site based on the available storage capacity and maximum discharge capacity of each site assuming a standardized constructed wetland design. To understand the maximum effect of the use of sheet flow sites on the drawdown of lake water levels, all six sheet flow sites were simulated in the comprehensive H&H model using the optimized pumping rates.

Each potential site was also evaluated for its suitability as a sheet flow site based on six factors: potential storage capacity, suitability of soils, presence of environmental features, flood risk, constructability, and possible permitting. The Gull Rock Game Land Carter Tract and White Tail

Farms properties were identified as the most suitable sites for constructed wetlands due to their available storage capacity, suitable soils, and permitting. The Pat Simmons property was also identified as most suitable due to its suitable soils, minimal presence of environmental features, and feasible permitting and constructability. Two of the other three sites (Ben Simmons/Joey Ben Williams and Tierney properties) present design challenges from a permitting perspective but should still be considered for future projects to maximize storage options. The Kelly Davis property has minimal storage capacity but otherwise is an attractive, suitable site due to ease of construction and permitting.

5. REFERENCES

Deltares, 2021. Delft3D Flexible Mesh Suite: 1D/2D/3D Modelling suite for integral water solutions. D-Flow Flexible Mesh Computational Cores and User Interface, User Manual, Released for Delft3D-FM Suite 2021. June 2nd, 2021, 570 pg. Available at: https://content.oss.deltares.nl/delft3d/manuals/D-Flow_FM_User_Manual.pdf.

North Carolina Coastal Federation (NCCF). 2018. *Lake Mattamuskeet Watershed Restoration Plan.* November 30, 2018. Available at: www.nccoast.org/lakemattamuskeet.

North Carolina Department of Environmental Quality (NCDEQ). 2020. *NCDEQ Stormwater Design Manual. Chapter C-4 Stormwater Wetland.* November 20, 2020. Available at: https://deq.nc.gov/about/divisions/energy-mineral-and-land-resources/stormwater/stormwater-program/stormwater-design

Soil Survey Staff (2020). Gridded Soil Survey Geographic Database (SSURGO) for the United States of America and the Territories, Commonwealths, and Island Nations served by the USDA-NRCS. U.S. Department of Agriculture, Natural Resources Conservation Service, Web Soil Survey. Available online at http://datagateway.nrcs.usda.gov/. Accessed October 1, 2020.

United States Geological Survey (2020). Current Water Data for the Nation. Available at: http://waterdata.usgs.gov/nwis. Accessed November 3, 2020.

ATTACHMENT A: SUPPORTING DOCUMENTATION FOR WETLAND SITING SUITABILITY MATRIX

A.1 WETLAND SITING SUITABILITY MATRIX

To assess the suitability of each potential site for conversion to constructed wetlands, the following factors were evaluated: potential storage capacity, suitability of soils, presence of environmental features, flood risk, constructability, and permitting. These assessments are described in further detail in Appendix A Sections 2.3.1 through 2.3.6 and Section 3.2. Supporting documentation for the assessments are provided in this attachment.

A.2 POTENTIAL STORAGE CAPACITY

A.2.1 Stage-Storage Curves

Table 1. Stage-storage curve for the Ben Simmons/Joey Ben Williams property.

Elevation	Volume
ft, NAVD88	yd^3
0.00	814
0.50	3,374
1.00	100,959
1.50	320,158
2.00	577,946
2.50	837,307
3.00	1,096,725

Notes:
1. Volume estimated using surface volume tool in ArcGIS. Surface volume tool estimates the volume below a surface at a specified elevation.
2. ft = feet
3. yd^3 = cubic yard

Table 2. Stage-storage curve for the Gull Rock Game Land Carter Tract.

Elevation	Volume
ft, NAVD88	yd^3
0.00	5,099
0.50	57,989
1.00	472,271
1.50	1,683,790

Elevation	Volume
ft, NAVD88	**yd^3**
2.00	3,225,546
2.50	4,893,230
3.00	6,569,551

Notes:

1. Volume estimated using surface volume tool in ArcGIS. Surface volume tool estimates the volume below a surface at a specified elevation.
2. ft = feet
3. yd^3 = cubic yard

Table 3. Stage-storage curve for the Kelly Davis property.

Elevation	Volume
ft, NAVD88	**yd^3**
0.00	0
0.50	57
1.00	4,258
1.50	39,327
2.00	97,374
2.50	166,223
3.00	238,148

Notes:

1. Volume estimated using surface volume tool in ArcGIS. Surface volume tool estimates the volume below a surface at a specified elevation.
2. ft = feet
3. yd^3 = cubic yard

Table 4. Stage-storage curve for the Pat Simmons property.

Elevation	Volume
ft, NAVD88	yd^3
0.00	6,132
0.50	59,557
1.00	202,633
1.50	414,948
2.00	638,644
2.50	868,077
3.00	1,098,576

Notes:
1. Volume estimated using surface volume tool in ArcGIS. Surface volume tool estimates the volume below a surface at a specified elevation.
2. ft = feet
3. yd^3 = cubic yard

Table 5. Stage-storage curve for the Tierney property.

Elevation	Volume
ft, NAVD88	yd^3
0.00	12,220
0.50	23,224
1.00	132,312
1.50	525,710
2.00	1,055,268
2.50	1,656,757
3.00	2,277,143

Notes:
1. Volume estimated using surface volume tool in ArcGIS. Surface volume tool estimates the volume below a surface at a specified elevation.
2. ft = feet
3. yd^3 = cubic yard

Table 6. Stage-storage curve for White Tail Farms by individual zone.

White Tail Farms – Zone 1		White Tail Farms – Zone 2		White Tail Farms – Zone 3		White Tail Farms – Zone 4	
Elevation	Volume	Elevation	Volume	Elevation	Volume	Elevation	Volume
ft, NAVD88	yd^3	ft, NAVD88	yd^3	ft, NAVD88	yd^3	ft, NAVD88	yd^3
0.00	0	0.00	1	0.00	14	0.00	0
0.50	42	0.50	6	0.50	71	0.50	1
1.00	8,010	1.00	20	1.00	195	1.00	6
1.50	73,635	1.50	774	1.50	430	1.50	35
2.00	223,625	2.00	6,411	2.00	839	2.00	112
2.50	463,777	2.50	22,979	2.50	1,498	2.50	370
3.00	776,101	3.00	55,570	3.00	2,576	3.00	1,039
3.50	1,221,537	3.50	108,146	3.50	5,503	3.50	3,143
4.00	1,820,122	4.00	182,089	4.00	13,399	4.00	10,184
4.50	2,642,024	4.50	291,190	4.50	31,442	4.50	31,587
5.00	3,678,640	5.00	458,011	5.00	74,032	5.00	80,238
5.50	4,980,578	5.50	722,734	5.50	167,770	5.50	178,719
6.00	6,378,093	6.00	1,063,404	6.00	313,089	6.00	338,787
6.50	7,964,815	6.50	1,495,377	6.50	538,711	6.50	612,769
7.00	9,735,437	7.00	2,029,712	7.00	948,362	7.00	1,086,861
7.50	11,784,718	7.50	2,762,058	7.50	1,769,329	7.50	1,922,535
8.00	13,823,657	8.00	3,635,812	8.00	2,777,231	8.00	2,990,134
8.50	15,972,179	8.50	4,722,126	8.50	3,917,318	8.50	4,147,272
9.00	18,234,491	9.00	6,042,094	9.00	5,174,814	9.00	5,325,654
9.50	20,766,436	9.50	7,664,206	9.50	6,628,205	9.50	6,588,745

Notes:

1. Volume estimated using surface volume tool in ArcGIS. Surface volume tool estimates the volume below a surface at a specified elevation.
2. White Tail Farms was split into five zones for analysis due to its size and the topographic relief on the site. Only four zones were included in the storage capacity analysis because the fifth zone was located entirely inside the watershed.
3. ft = feet
4. yd^3 = cubic yard

Table 7. Total stage-storage curve for White Tail Farms.

White Tail Farms - Total	
Elevation	Volume
ft, NAVD88	yd^3
0.00	15
0.50	120
1.00	8,231
1.50	74,874
2.00	230,987

White Tail Farms - Total	
Elevation	**Volume**
ft, NAVD88	**yd^3**
2.50	488,626
3.00	835,286
3.50	1,338,329
4.00	2,025,793
4.50	2,996,244
5.00	4,290,921
5.50	6,049,800
6.00	8,093,373
6.50	10,611,672
7.00	13,800,371
7.50	18,238,640
8.00	23,226,834
8.50	28,758,895
9.00	34,777,054
9.50	41,647,592

Notes:
1. Volume estimated using surface volume tool in ArcGIS. Surface volume tool estimates the volume below a surface at a specified elevation.
2. White Tail Farms was split into five zones for analysis due to its size and the topographic relief on the site. Only four zones were included in the storage capacity analysis because the fifth zone was located entirely inside the watershed.
3. ft = feet
4. yd^3 = cubic yard

A.2.2 Summary of Potential Storage Capacity Assessment

Table 8. Summary of potential storage capacity assessment.

Potential Sheet Flow Site	Summary of Site Characteristics and Design Parameters					Temporary Storage Volume	
	Min. Elevation	**Max. Elevation**	**Average El.**	**Weir El.**	**Temporary Storage El.**	**yd^3**	**ac-ft**
Ben Simmons/Joey Ben Williams	-4.3	3	0.7 - 0.9	1.0	2.0	476,987	295
Gull Rock Game Land Carter Tract	-5.6	6.5	1.1	1.0	2.0	2,753,275	1,707
Kelly Davis	0.2	4.3	1.4	1.5	2.5	126,896	79
Pat Simmons	-2.2	3.7	0.6	0.5	1.5	355,391	220
Tierney Property	-5.5	5.7	1.2	1.0	2.0	922,956	572
White Tail Farms (Total)	-0.8	15.7	6.3	Varies	Varies	8,995,567	5,575

Potential Sheet Flow Site	Summary of Site Characteristics and Design Parameters					Temporary Storage Volume	
	Min. Elevation	Max. Elevation	Average El.	Weir El.	Temporary Storage El.	yd^3	ac-ft
White Tail Farms Zone 1	0.3	12.9	5.6	5.5	6.5	2,984,237	1,850
White Tail Farms Zone 2	-0.7	13.2	7.6	7.5	8.5	1,960,068	1,215
White Tail Farms Zone 3	-0.7	13.7	7.4	7.5	8.5	2,147,989	1,331
White Tail Farms Zone 4	-0.2	12.8	6.8	7.0	8.0	1,903,273	1,180

Notes:
1. Volume estimated using surface volume tool in ArcGIS. Surface volume tool estimates the volume below a surface at a specified elevation.
2. White Tail Farms was split into five zones for analysis due to its size and the topographic relief on the site. Only four zones were included in the storage capacity analysis because the fifth zone was located entirely inside the watershed.
3. yd^3 = cubic yard
4. ac-ft = acre-feet

A.3 SUITABILITY OF SOILS

A.3.1 Summary of Soil Type Assessment

Table 9. Summary of soil type assessment.

Potential Sheet Flow Site	Soil Types Present at Site	Suitability of Soils Assessment
Ben Simmons/Joey Ben Williams	Approximately half of soils are Belhaven muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: BmA) and half of soils are Dorovan muck, 0 to 2 percent slopes, frequently flooded (map unit symbol: DoA)	Most Suitable
Gull Rock Game Land Carter Tract	55 acres of Udorthents, sandy, rarely flooded (map unit symbol: Ud); 197.4 acres of Longshoal mucky peat, 0 to 1 percent slopes, very frequently flooded (map unit symbol: LfA); 800.9 acres of Pungo muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: PuA); 810.6 acres of Ponzer muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: PnA); 146.6 acres of Scuppernong muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: ScA); 99.3 acres of Wasda muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: WaA); 10.7 acres of Newholland mucky loamy sand, 0 to 2 percent slopes, rarely flooded (map unit symbol: NeA);	Most Suitable

Potential Sheet Flow Site	Soil Types Present at Site	Suitability of Soils Assessment
	26.8 acres of Stockade mucky sandy loam, 0 to 2 percent slopes, rarely flooded (map unit symbol: StA); 1.9 acres of Gullrock muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: GuA)	
Kelly Davis	28.7 acres of Wysocking very fine sandy loam, 0 to 3 percent slopes, rarely flooded (map unit symbol: WyA); 58.8 acres of Belhaven muck, 0 to 2 percent slopes, frequently flooded (map unit symbol: BnA); 8.3 acres of Fortescue silt loam, 0 to 2 percent slopes, rarely flooded (map unit symbol: Foa)	Suitable
Pat Simmons	Majority of soils are Pungo muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: PuA)	Most Suitable
Tierney Property	33 acres of Longshoal mucky peat, 0 to 1 percent slopes, very frequently flooded (map unit symbol: LfA); 60.9 acres of Ponzer muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: PnA); 689.5 acres of Pungo muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: PuA); 15.3 acres of Udorthents, sandy, rarely flooded (map unit symbol: Ud)	Most Suitable
White Tail Farms	42.1 acres of Conaby muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: CoA); 664.8 acres of Portsmouth mucky sandy loam, 0 to 2 percent slopes, rarely flooded (map unit symbol: PoA); 41.8 acres of Fork fine sandy loam, 0 to 2 percent slopes, rarely flooded (map unit symbol: FkA); 203.7 acres of Yonges loam, 0 to 2 percent slopes, rarely flooded (map unit symbol: YoA); 15.2 acres of Argent loam, 0 to 2 percent slopes, rarely flooded (map unit symbol: ArA); 779.6 acres of Pettigrew muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: PeA); 270.9 acres of Wasda muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: WaA); 18.6 acres of Bolling loamy fine sand, 0 to 3 percent slopes, rarely flooded (map unit symbol: BoA); 789.4 acres of Belhaven muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: BmA); 353.6 acres of Ponzer muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: PnA); 33.3 acres of Udorthents, sandy, rarely flooded (map unit symbol: Ud);	Suitable

Potential Sheet Flow Site	Soil Types Present at Site	Suitability of Soils Assessment
	7240.6 acres of Pungo muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: PuA); 218.5 acres of Scuppernong muck, 0 to 2 percent slopes, rarely flooded (map unit symbol: ScA); 3.3 acres of Newholland mucky loamy sand, 0 to 2 percent slopes, rarely flooded (map unit symbol: NeA); 225.1 acres of Dorovan muck, 0 to 2 percent slopes, frequently flooded (map unit symbol: DoA); 43.9 acres of Acredale silt loam, 0 to 2 percent slopes, rarely flooded (map unit symbol: AcA)	

A.4 PRESENCE OF ENVIRONMENTAL FEATURES

A.4.1 Environmental Features Figure

See Figure 1.

A.4.2 Summary of Environmental Features Assessment

Table 10. Summary of environmental features assessment.

Potential Sheet Flow Site	Environmental Features Present at Site	Environmental Features Assessment
Ben Simmons/Joey Ben Williams	100% of site is swamp. Alligator River runs along north boundary. Canal ditches run along south and east boundaries.	Large Presence
Gull Rock Game Land Carter Tract	Swamp located throughout site. Intracoastal Waterway runs along north boundary. Alligator River runs throughout northern part. Carters Canal runs along the east boundary. Burus Canal runs along middle of site. Fairfield Canal runs along west boundary.	Large Presence
Kelly Davis	Majority of site is swamp. Canal ditches run along south and west boundaries of property.	Large Presence
Pat Simmons	Canal ditches run along all boundaries of property. Few environmental features present on site	Minimal Presence
Tierney Property	Approximately 60% of site is swamp.	Some Presence
White Tail Farms	Swamp is located in northeast corner of property. Intracoastal Waterway runs along north boundary. Alligator	Minimal Presence

Potential Sheet Flow Site	Environmental Features Present at Site	Environmental Features Assessment
	River runs slightly in north east corner. Canal ditches run throughout site in multiple areas.	

A.5 FLOOD RISK

A.5.1 FEMA Flood Map Figures

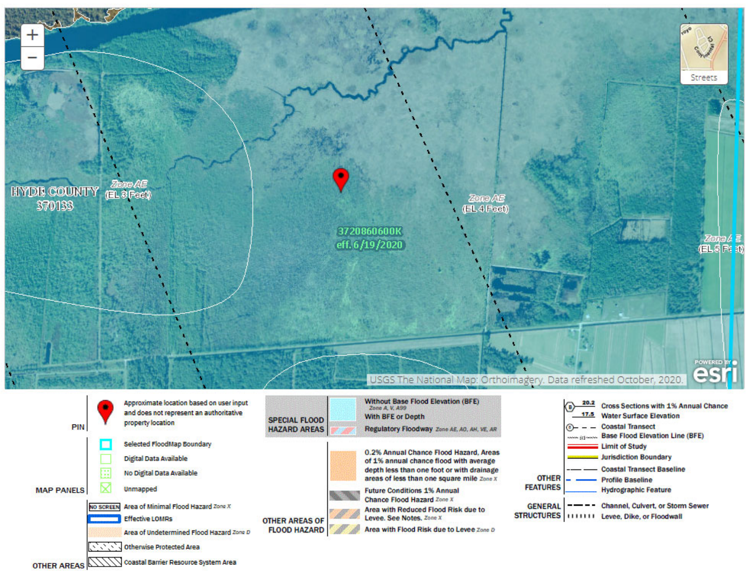

Figure 2. FEMA special flood hazard area in the vicinity of the Ben Simmons/Joey Ben Williams property.

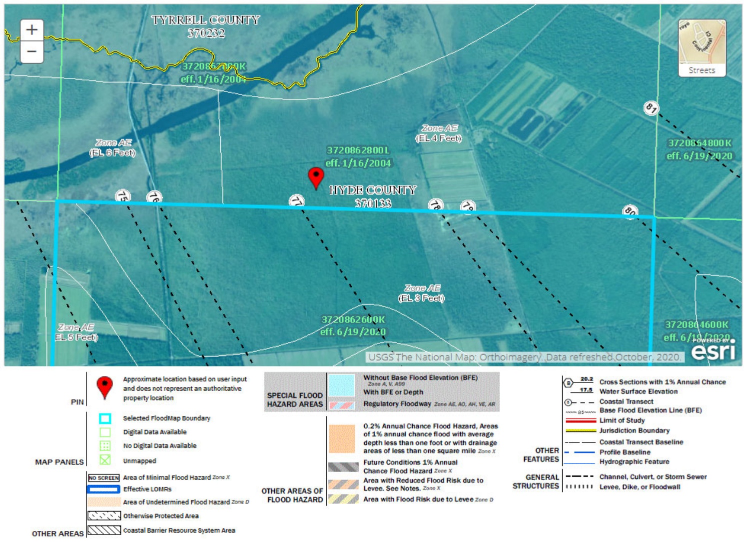

Figure 3. FEMA special flood hazard area in the vicinity of the Gull Rock Game Land Carter Tract.

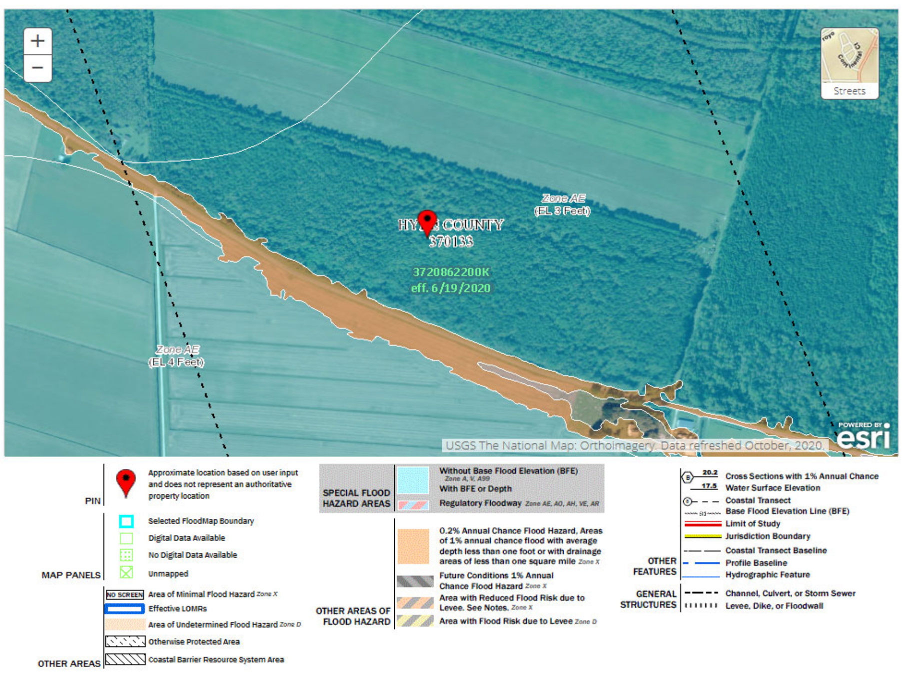

Figure 4. FEMA special flood hazard area in the vicinity of the Kelly Davis property.

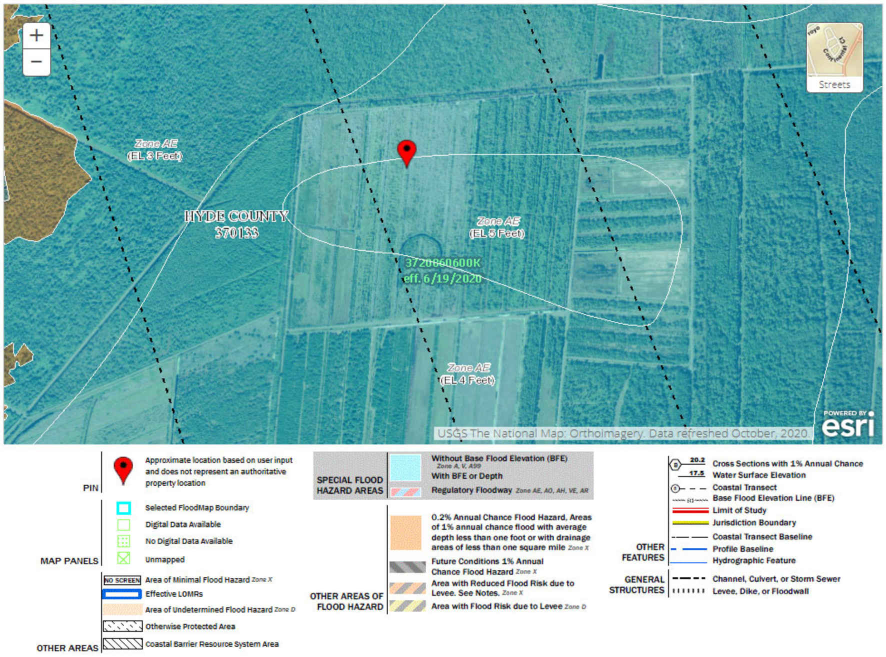

Figure 5. FEMA special flood hazard area in the vicinity of the Pat Simmons property.

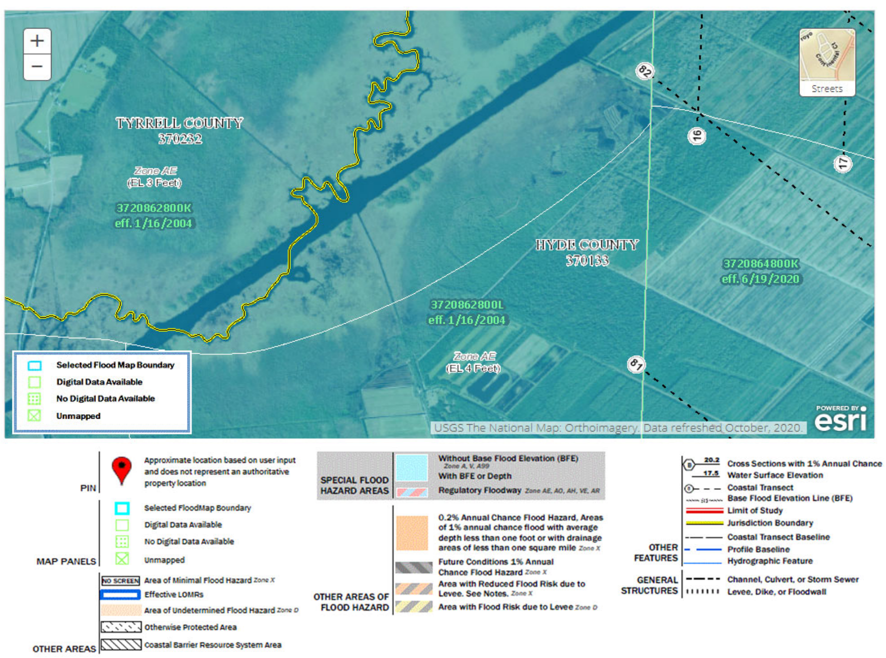

Figure 6. FEMA special flood hazard area in the vicinity of the Tierney property.

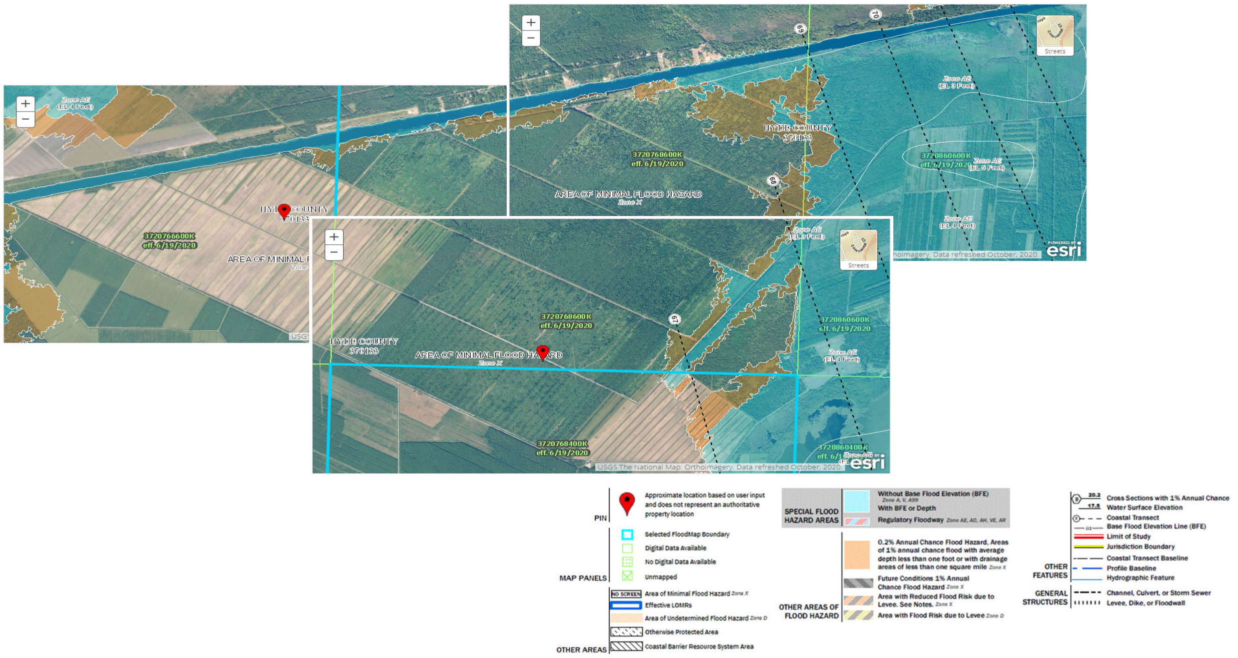

Figure 7. FEMA special flood hazard area in the vicinity of the White Tail Farms property.

A.5.2 Flood Risk Assessment

Table 11. Summary of flood risk assessment.

Potential Sheet Flow Site	Approximate percentage of property within AE flood zone
Ben Simmons/Joey Ben Williams	~ 100%
Gull Rock Game Land Carter Tract	~ 99%
Kelly Davis	~ 30%
Pat Simmons	~ 100%
Tierney Property	~ 100%
White Tail Farms	~ 5%

A.6 CONSTRUCTABILITY

A.6.1 Summary of Constructability Assessment

Table 12. Summary of constructability assessment.

Potential Sheet Flow Site	Distance to Site (mi)	Topographic Relief (ft)	Constructability Assessment
Ben Simmons/Joey Ben Williams	1.8 – 2.9	7.3	Feasible
Gull Rock Game Land Carter Tract	2.6	12.1	Possible
Kelly Davis	0.5	4.1	Feasible
Pat Simmons	1.8	5.9	Feasible
Tierney Property	4.1	11.2	Possible
White Tail Farms	0.8	16.5	Difficult

Notes:
1. mi = miles
2. ft = feet

A.7 PERMITTING

A.7.1 Interbasin Transfer Line Figure

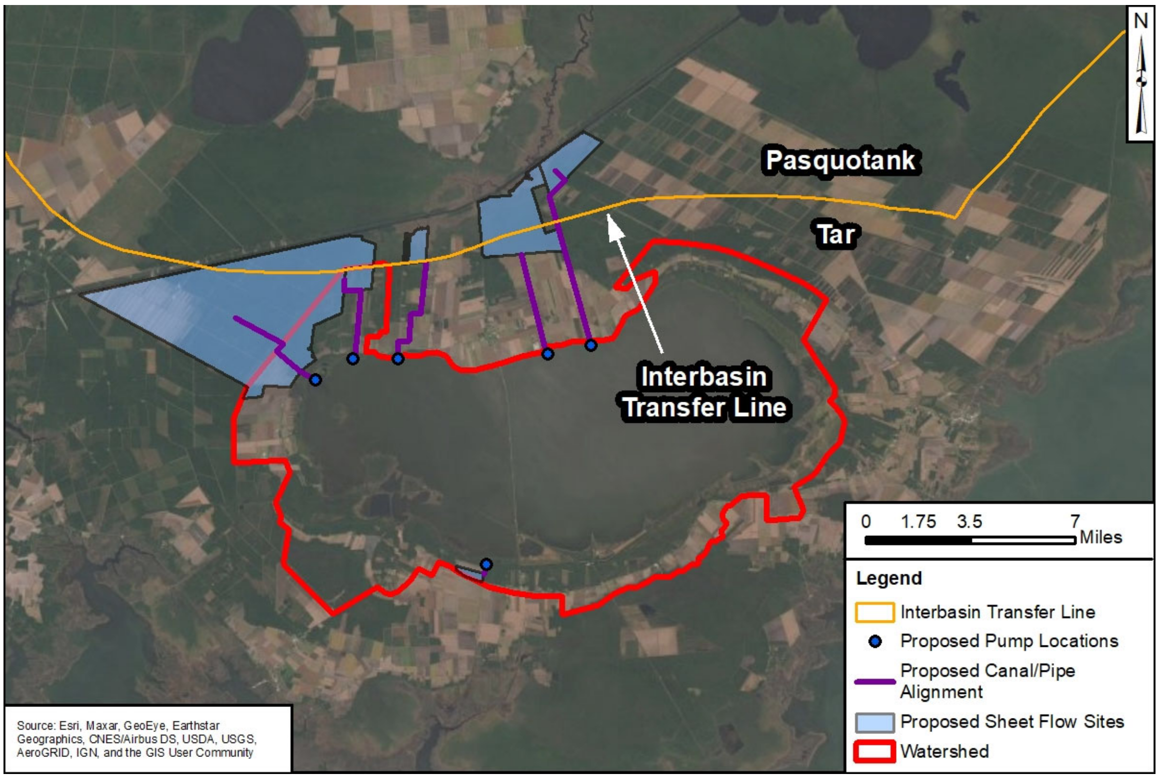

Figure 8. Location of interbasin transfer line.

A.7.2 Summary of Permitting Assessment

Table 13. Summary of permitting assessment.

Potential Sheet Flow Site	Location of Sheet Flow Site	Permitting Assessment
Ben Simmons/Joey Ben Williams	Pasquotank Basin	Difficult
Gull Rock Game Land Carter Tract	Tar Basin and Pasquotank Basin; interbasin transfer line straddles site	Possible
Kelly Davis	Tar Basin	Feasible
Pat Simmons	Tar Basin	Feasible

Potential Sheet Flow Site	Location of Sheet Flow Site	Permitting Assessment
Tierney Property	Tar Basin and Pasquotank Basin; interbasin transfer line straddles site	Possible
White Tail Farms	Tar Basin and Pasquotank Basin; interbasin transfer line straddles site	Possible

A.8 SUMMARY OF WETLAND SITING SUITABILITY

Table 14. Wetland siting suitability matrix.

Potential Sheet Flow Site	Storage Capacity (ac-ft)	Soil Type	Environmental Features	Flood Risk	Constructability	Permitting	Overall
Ben Simmons/Joey Ben Williams	295	Most Suitable	Large Presence	~ 100%	Feasible	Difficult	Suitable
Gull Rock Game Land Carter Tract	1,707	Most Suitable	Large Presence	~ 99%	Possible	Possible	More Suitable
Kelly Davis	79	Suitable	Large Presence	~ 30%	Feasible	Feasible	Suitable
Pat Simmons	220	Most Suitable	Minimal Presence	~ 100%	Feasible	Feasible	Suitable
Tierney Property	572	Most Suitable	Some Presence	~ 100%	Possible	Difficult	Suitable
White Tail Farms	5,575	Suitable	Minimal Presence	~ 5%	Difficult	Possible	More Suitable

Notes:

1. ac-ft = acre-feet

Legend

- Proposed Sheet Flow Sites
- Perennial
- Intermittent
- Artificial Path
- Canal Ditch
- Coastline
- Connector
- Pipeline
- Underground Conduit
- Estuary
- Ice Mass
- LakePond
- Playa
- Reservoir
- SwampMarsh

0 3 Miles

Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community
USGS TNM – National Hydrography Dataset. Data Refreshed April, 2021.

Environmental Features Present on Potential Sheet Flow Sites

Lake Mattamuskeet
Hyde County, North Carolina

Geosyntec consultants
Geosyntec Consultants of NC, P.C.

Coastal Protection Engineering

Figure 1

Raleigh, North Carolina | June 2021

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APPENDIX C
Conceptual Costs Analysis

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Table 1. AACE Class 5 Capital Costs for
Three Gravity-Drained Canals and Associated Pump Stations

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	Cost ($) Low	Cost ($) High
Drainage canal improvements and dredging	CY	$4	$10	205,000	$820,000	$2,050,000
Hauling & disposal of dredged material	CY	$0	$10	205,000	$0	$2,050,000
Water control structures	LS	$200,000	$300,000	3	$600,000	$900,000
Pump Stations						
Pump + Installation (54" axial flow pumps at 350 hp and 70,000 gpm pump capacity)	LS	$276,250	$425,000	9	$2,486,250	$3,825,000
Diesel generators (400 kW)	LS	$100,000	$120,000	6	$600,000	$720,000
Double-walled fuel storage tank including foundation (20,000 gallons)	LS	$50,000	$60,000	3	$150,000	$180,000
Secondary containment structure for loading and unloading	LS	$0	$75,000	3	$0	$225,000
Pump structure	LS	$25,000	$30,000	3	$75,000	$90,000
54" pipe to discharge point (installed)	LF	$300	$400	120	$36,000	$48,000
Electrical controls + electrical room	LS	$50,000	$60,000	3	$150,000	$180,000
Pump station site work	LS	$40,000	$50,000	3	$120,000	$150,000
Miscellaneous site work including prep and grading	LS	$75,000	$100,000	3	$225,000	$300,000
Total					**$5,262,250**	**$10,718,000**
30% Contingency					**$6,840,925**	**$13,933,400**

Notes:
1. This cost analysis should be considered in line with a Class 5 Estimate that the Association for the Advancement of Cost Engineering (AACE) recognizes as having an uncertainty range of -20% to 100%.
2. This cost analysis is based on conceptual costs only and unit costs from a wide variety of sources.
3. This cost analysis is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.
4. The estimate of costs for canal improvements based on use of long-reach excavators.
5. The estimate of dredged material is based on the approximate cut volume estimated for dredging of the Burus, Jarvis, and Swindell canals to the dimensions identified in the report. Estimates are based on topobathy used in H&H model and not detailed survey.
6. The low end of the hauling disposal item assumes no hauling or disposal is required and material can be cast off adjacent to canal.
7. The water contol structures assume a sluice gate type structure.
8. The pump stations assume three pumps installed at each canal for a total capacity of 210,000 gpm per canal.
9. This estimate assumes pumps and associated infrastucture are fueled by diesel generators. The diesel generators assume two generators per canal.
10. Secondary containment is only needed if there is a fuel transfer operation.
11. Canal alignment based on Figure 4.6 of report.
12. This cost analysis considers capital costs only and does not include annual operational costs.

Acronyms:
CY = cubic yard
gpm = gallons per minute
hp = horsepower
kW = kilowatt
LS = lump sum
No. = number

Table 2. AACE Class 5 Capital Costs for Centralized Pump Station with Capacity of 350,000 gpm

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	Cost ($) Low	Cost ($) High
Drainage canal improvements and dredging	CY	$4	$10	300,000	$1,200,000	$3,000,000
Hauling & disposal of dredged material	CY	$0	$10	300,000	$0	$3,000,000
Centralized Pump Station						
Pump + Installation (48" axial flow pumps at 250 hp and 50,000 gpm pump capacity)	LS	$211,250	$325,000	7	$1,478,750	$2,275,000
Diesel generators (400 kW)	LS	$100,000	$120,000	4	$400,000	$480,000
Double-walled fuel storage tank including foundation (20,000 gallons)	LS	$40,000	$60,000	4	$160,000	$240,000
Pump structure	LS	$50,000	$100,000	1	$50,000	$100,000
48" pipe to discharge point (installed)	LF	$240	$400	10,000	$2,400,000	$4,000,000
Electrical controls + electrical room	LS	$60,000	$100,000	1	$60,000	$100,000
Pump station site work	LS	$75,000	$100,000	1	$75,000	$100,000
Miscellaneous site work including prep and grading	LS	$75,000	$100,000	1	$75,000	$100,000
Total					**$5,898,750**	**$13,395,000**
30% Contingency					**$7,668,375**	**$17,413,500**

Notes:
1. This cost analysis should be considered in line with a Class 5 Estimate that the Association for the Advancement of Cost Engineering (AACE) recognizes as having an uncertainty range of -20% to 100%.
2. This cost analysis is based on conceptual costs only and unit costs from a wide variety of sources.
3. This cost analysis is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.
4. The estimate of costs for canal improvements based on use of long-reach excavators.
5. The estimate of dredged material is based on siting of pump station in west basin and discharging to the intracoastal waterway. Assumes widening of canal at southwest boundary of White Tail Farms to 60 feet. Estimates are based on topobathy used in H&H model and not detailed survey.
6. The low end of the hauling disposal item assumes no hauling or disposal is required and material can be cast off adjacent to canal.
7. The pump stations assume seven pumps installed near lakeshore for a total capacity of 350,000 gpm per canal.
8. The use of diesel generators was assumed; however, there may be an option to tie into electrical grid if sited on western basin.
9. Pump location and canal alignment based on Figure 4.1 of report.
10. This cost analysis considers capital costs only and does not include annual operational costs

Acronyms:
CY = cubic yard
gpm = gallons per minute
hp = horsepower
kW = kilowatt
LS = lump sum
No. = number

Table 3. AACE Class 5 Capital Costs for Disperesed Pump Stations to Sheet Flow Sites

Item	Unit	Unit Cost ($)		Ben Simmons/Joey Ben Williams		
		Low	High	No. of Units	Low Sub-Cost ($)	High Sub-Cost ($)
Drainage canal improvements	LF	$4	$10	16,152	$64,608	$161,520
Water control structures	LS	$20,000	$40,000	1	$20,000	$40,000
Distributed Pump Stations						
Pump + Installation (24" axial flow pumps at 250 hp and 13,333 gpm pump capacity)	LS	$120,000	$200,000	3	$360,000	$600,000
Diesel generators (250 kW)	LS	$50,000	$75,000	3	$150,000	$225,000
Double-walled fuel storage tank including foundation (10,000 gallons)	LS	$15,000	$25,000	2	$22,500	$37,500
Electrical controls + electrical room	LS	$8,000	$10,000	1	$8,000	$10,000
24" pipe to discharge point (installed)	LF	$120	$200	200	$24,000	$40,000
Berm grading at sheet flow sites	CY	$4	$10	20,000	$80,000	$200,000
Other sheet flow sites improvements (plantings, etc.)	LS	$25,000	$75,000	1	$25,000	$75,000
Total					**$754,108**	**$1,389,020**
30% Contingency					**$980,340**	**$1,805,726**

Item	Unit	Unit Cost ($)		Ben Simmons/Joey Ben Williams		
		Low	High	No. of Units	Low Sub-Cost ($)	High Sub-Cost ($)
Annual Leasing Fees	AC	$60	$73	338	$20,280	$24,505

Notes:

1. This cost analysis should be considered in line with a Class 5 Estimate that the Association for the Advancement of Cost Engineering (AACE) recognizes as having an uncertainty range of -20% to 100%.
2. This cost analysis is based on conceptual costs only and unit costs from a wide variety of sources.
3. This cost analysis is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.
4. The estimate of costs for canal improvements based on use of long-reach excavators, minimal improvements, priced per linear foot. No hauling.
5. The low end of the hauling disposal item assumes no hauling or disposal is required and material can be cast off adjacent to canal.
6. The pump stations assume 13,333 gpm pumps to meet approximate pumping rate in model
7. This estimate assumes pumps and associated infrastucture are fueled by diesel generators.
8. Berm grading assumed a 10 ft perimeter berm at approximately 5 feet high.
9. Annual leasing rate based on National Resources Conservation Service (NRCS) Wetland Reserve Program at $1,800 - $2,175 per acre for 30-year easement.
10. Pump locations and canal/pipe alignment based on schematic in Figure 4.3 of report.
11. This cost analysis considers capital costs only and does not include annual operational costs

Acronyms:
AC = acre
CY = cubic yard
gpm = gallons per minute
hp = horsepower
kW = kilowatt
LF = linear foot
LS = lump sum
No. = number

Table 3. AACE Class 5 Capital Costs for Disperesed Pump Stations to Sheet Flow Sites

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	Kelly Davis Low Sub-Cost ($)	Kelly Davis High Sub-Cost ($)
Drainage canal improvements	LF	$4	$10	1,237	$4,948	$12,370
Water control structures	LS	$20,000	$40,000	1	$20,000	$40,000
Distributed Pump Stations						
Pump + Installation (24" axial flow pumps at 250 hp and 13,333 gpm pump capacity)	LS	$120,000	$200,000	1	$120,000	$200,000
Diesel generators (250 kW)	LS	$50,000	$75,000	1	$50,000	$75,000
Double-walled fuel storage tank including foundation (10,000 gallons)	LS	$15,000	$25,000	1	$7,500	$12,500
Electrical controls + electrical room	LS	$8,000	$10,000	1	$8,000	$10,000
24" pipe to discharge point (installed)	LF	$120	$200	200	$24,000	$40,000
Berm grading at sheet flow sites	CY	$4	$10	11,097	$44,387	$110,967
Other sheet flow sites improvements (plantings, etc.)	LS	$25,000	$75,000	1	$25,000	$75,000
Total					**$303,835**	**$575,837**
30% Contingency					**$394,985**	**$748,588**

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	Kelly Davis Low Sub-Cost ($)	Kelly Davis High Sub-Cost ($)
Annual Leasing Fees	AC	$60	$73	95	$5,700	$6,888

Notes:

1. This cost analysis should be considered in line with a Class 5 Estimate that the Association for the Advancement of Cost Engineering (AACE) recognizes as having an uncertainty range of -20% to 100%.
2. This cost analysis is based on conceptual costs only and unit costs from a wide variety of sources.
3. This cost analysis is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.
4. The estimate of costs for canal improvements based on use of long-reach excavators, minimal improvements, priced per linear foot. No hauling.
5. The low end of the hauling disposal item assumes no hauling or disposal is required and material can be cast off adjacent to canal.
6. The pump stations assume 13,333 gpm pumps to meet approximate pumping rate in model
7. This estimate assumes pumps and associated infrastucture are fueled by diesel generators.
8. Berm grading assumed a 10 ft perimeter berm at approximately 5 feet high.
9. Annual leasing rate based on National Resources Conservation Service (NRCS) Wetland Reserve Program at $1,800 - $2,175 per acre for 30-year easement.
10. Pump locations and canal/pipe alignment based on schematic in Figure 4.3 of report.
11. This cost analysis considers capital costs only and does not include annual operational costs

Acronyms:
AC = acre
CY = cubic yard
gpm = gallons per minute
hp = horsepower
kW = kilowatt
LF = linear foot
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Table 3. AACE Class 5 Capital Costs for
Disperesed Pump Stations to Sheet Flow Sites

Item	Unit	Unit Cost ($)		Gull Rock Game Land Carter Tract		
		Low	High	No. of Units	Low Sub-Cost ($)	High Sub-Cost ($)
Drainage canal improvements	LF	$4	$10	18,252	$73,008	$182,520
Water control structures	LS	$20,000	$40,000	4	$80,000	$160,000
Distributed Pump Stations						
Pump + Installation (24" axial flow pumps at 250 hp and 13,333 gpm pump capacity)	LS	$120,000	$200,000	12	$1,440,000	$2,400,000
Diesel generators (250 kW)	LS	$50,000	$75,000	12	$600,000	$900,000
Double-walled fuel storage tank including foundation (10,000 gallons)	LS	$15,000	$25,000	6	$90,000	$150,000
Electrical controls + electrical room	LS	$8,000	$10,000	2	$16,000	$20,000
24" pipe to discharge point (installed)	LF	$120	$200	200	$24,000	$40,000
Berm grading at sheet flow sites	CY	$4	$10	60,550	$242,200	$605,500
Other sheet flow sites improvements (plantings, etc.)	LS	$25,000	$75,000	1	$25,000	$75,000
Total					**$2,590,208**	**$4,533,020**
30% Contingency					**$3,367,270**	**$5,892,926**

Item	Unit	Unit Cost ($)		Gull Rock Game Land Carter Tract		
		Low	High	No. of Units	Low Sub-Cost ($)	High Sub-Cost ($)
Annual Leasing Fees	AC	$60	$73	2,139	$128,340	$155,078

Notes:
1. This cost analysis should be considered in line with a Class 5 Estimate that the Association for the Advancement of Cost Engineering (AACE) recognizes as having an uncertainty range of -20% to 100%.
2. This cost analysis is based on conceptual costs only and unit costs from a wide variety of sources.
3. This cost analysis is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.
4. The estimate of costs for canal improvements based on use of long-reach excavators, minimal improvements, priced per linear foot. No hauling.
5. The low end of the hauling disposal item assumes no hauling or disposal is required and material can be cast off adjacent to canal.
6. The pump stations assume 13,333 gpm pumps to meet approximate pumping rate in model
7. This estimate assumes pumps and associated infrastucture are fueled by diesel generators.
8. Berm grading assumed a 10 ft perimeter berm at approximately 5 feet high.
9. Annual leasing rate based on National Resources Conservation Service (NRCS) Wetland Reserve Program at $1,800 - $2,175 per acre for 30-year easement.
10. Pump locations and canal/pipe alignment based on schematic in Figure 4.3 of report.
11. This cost analysis considers capital costs only and does not include annual operational costs

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Table 3. AACE Class 5 Capital Costs for Disperesed Pump Stations to Sheet Flow Sites

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	Pat Simmons Low Sub-Cost ($)	High Sub-Cost ($)
Drainage canal improvements	LF	$4	$10	13,865	$55,460	$138,650
Water control structures	LS	$20,000	$40,000	2	$40,000	$80,000
Distributed Pump Stations						
Pump + Installation (24" axial flow pumps at 250 hp and 13,333 gpm pump capacity)	LS	$120,000	$200,000	3	$360,000	$600,000
Diesel generators (250 kW)	LS	$50,000	$75,000	3	$150,000	$225,000
Double-walled fuel storage tank including foundation (10,000 gallons)	LS	$15,000	$25,000	2	$22,500	$37,500
Electrical controls + electrical room	LS	$8,000	$10,000	1	$8,000	$10,000
24" pipe to discharge point (installed)	LF	$120	$200	200	$24,000	$40,000
Berm grading at sheet flow sites	CY	$4	$10	16,104	$64,418	$161,044
Other sheet flow sites improvements (plantings, etc.)	LS	$25,000	$75,000	1	$25,000	$75,000
Total					**$749,378**	**$1,367,194**
30% Contingency					**$974,191**	**$1,777,353**

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	Pat Simmons Low Sub-Cost ($)	High Sub-Cost ($)
Annual Leasing Fees	AC	$60	$73	294	$17,640	$21,315

Notes:

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3. This cost analysis is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.
4. The estimate of costs for canal improvements based on use of long-reach excavators, minimal improvements, priced per linear foot. No hauling.
5. The low end of the hauling disposal item assumes no hauling or disposal is required and material can be cast off adjacent to canal.
6. The pump stations assume 13,333 gpm pumps to meet approximate pumping rate in model
7. This estimate assumes pumps and associated infrastucture are fueled by diesel generators.
8. Berm grading assumed a 10 ft perimeter berm at approximately 5 feet high.
9. Annual leasing rate based on National Resources Conservation Service (NRCS) Wetland Reserve Program at $1,800 - $2,175 per acre for 30-year easement.
10. Pump locations and canal/pipe alignment based on schematic in Figure 4.3 of report.
11. This cost analysis considers capital costs only and does not include annual operational costs

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AC = acre
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Table 3. AACE Class 5 Capital Costs for
Disperesed Pump Stations to Sheet Flow Sites

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	Tierney Property Low Sub-Cost ($)	Tierney Property High Sub-Cost ($)
Drainage canal improvements	LF	$4	$10	27,215	$108,860	$272,150
Water control structures	LS	$20,000	$40,000	3	$60,000	$120,000
Distributed Pump Stations						
Pump + Installation (24" axial flow pumps at 250 hp and 13,333 gpm pump capacity)	LS	$120,000	$200,000	10	$1,200,000	$2,000,000
Diesel generators (250 kW)	LS	$50,000	$75,000	10	$500,000	$750,000
Double-walled fuel storage tank including foundation (10,000 gallons)	LS	$15,000	$25,000	5	$75,000	$125,000
Electrical controls + electrical room	LS	$8,000	$10,000	2	$16,000	$20,000
24" pipe to discharge point (installed)	LF	$120	$200	200	$24,000	$40,000
Berm grading at sheet flow sites	CY	$4	$10	30,059	$120,236	$300,589
Other sheet flow sites improvements (plantings, etc.)	LS	$25,000	$75,000	1	$25,000	$75,000
Total					**$2,129,096**	**$3,702,739**
30% Contingency					**$2,767,824**	**$4,813,561**

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	Tierney Property Low Sub-Cost ($)	Tierney Property High Sub-Cost ($)
Annual Leasing Fees	AC	$60	$73	791	$47,460	$57,348

Notes:
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3. This cost analysis is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.
4. The estimate of costs for canal improvements based on use of long-reach excavators, minimal improvements, priced per linear foot. No hauling.
5. The low end of the hauling disposal item assumes no hauling or disposal is required and material can be cast off adjacent to canal.
6. The pump stations assume 13,333 gpm pumps to meet approximate pumping rate in model
7. This estimate assumes pumps and associated infrastucture are fueled by diesel generators.
8. Berm grading assumed a 10 ft perimeter berm at approximately 5 feet high.
9. Annual leasing rate based on National Resources Conservation Service (NRCS) Wetland Reserve Program at $1,800 - $2,175 per acre for 30-year easement.
10. Pump locations and canal/pipe alignment based on schematic in Figure 4.3 of report.
11. This cost analysis considers capital costs only and does not include annual operational costs

Acronyms:
AC = acre
CY = cubic yard
gpm = gallons per minute
hp = horsepower
kW = kilowatt
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Table 3. AACE Class 5 Capital Costs for Disperesed Pump Stations to Sheet Flow Sites

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	White Tail Farms Low Sub-Cost ($)	White Tail Farms High Sub-Cost ($)
Drainage canal improvements	LF	$4	$10	7,065	$28,260	$70,650
Water control structures	LS	$20,000	$40,000	10	$200,000	$400,000
Distributed Pump Stations						
Pump + Installation (24" axial flow pumps at 250 hp and 13,333 gpm pump capacity)	LS	$120,000	$200,000	12	$1,440,000	$2,400,000
Diesel generators (250 kW)	LS	$50,000	$75,000	12	$600,000	$900,000
Double-walled fuel storage tank including foundation (10,000 gallons)	LS	$15,000	$25,000	6	$90,000	$150,000
Electrical controls + electrical room	LS	$8,000	$10,000	2	$16,000	$20,000
24" pipe to discharge point (installed)	LF	$120	$200	7,000	$840,000	$1,400,000
Berm grading at sheet flow sites	CY	$4	$10	124,503	$498,013	$1,245,033
Other sheet flow sites improvements (plantings, etc.)	LS	$25,000	$75,000	1	$25,000	$75,000
Total					**$3,737,273**	**$6,660,683**
30% Contingency					**$4,858,455**	**$8,658,888**

Item	Unit	Unit Cost ($) Low	Unit Cost ($) High	No. of Units	White Tail Farms Low Sub-Cost ($)	White Tail Farms High Sub-Cost ($)
Annual Leasing Fees	AC	$60	$73	10,792	$647,520	$782,420

Notes:
1. This cost analysis should be considered in line with a Class 5 Estimate that the Association for the Advancement of Cost Engineering (AACE) recognizes as having an uncertainty range of -20% to 100%.
2. This cost analysis is based on conceptual costs only and unit costs from a wide variety of sources.
3. This cost analysis is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.
4. The estimate of costs for canal improvements based on use of long-reach excavators, minimal improvements, priced per linear foot. No hauling.
5. The low end of the hauling disposal item assumes no hauling or disposal is required and material can be cast off adjacent to canal.
6. The pump stations assume 13,333 gpm pumps to meet approximate pumping rate in model
7. This estimate assumes pumps and associated infrastucture are fueled by diesel generators.
8. Berm grading assumed a 10 ft perimeter berm at approximately 5 feet high.
9. Annual leasing rate based on National Resources Conservation Service (NRCS) Wetland Reserve Program at $1,800 - $2,175 per acre for 30-year easement.
10. Pump locations and canal/pipe alignment based on schematic in Figure 4.3 of report.
11. This cost analysis considers capital costs only and does not include annual operational costs

Acronyms:
AC = acre
CY = cubic yard
gpm = gallons per minute
hp = horsepower
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Table 3. AACE Class 5 Capital Costs for Disperesed Pump Stations to Sheet Flow Sites

Item	Unit	Unit Cost ($)		Total Cost for All Six Sites	
		Low	High	Low Cost ($)	High Cost ($)
Drainage canal improvements	LF	$4	$10	$335,144	$837,860
Water control structures	LS	$20,000	$40,000	$420,000	$840,000
Distributed Pump Stations					
Pump + Installation (24" axial flow pumps at 250 hp and 13,333 gpm pump capacity)	LS	$120,000	$200,000	$4,920,000	$8,200,000
Diesel generators (250 kW)	LS	$50,000	$75,000	$2,050,000	$3,075,000
Double-walled fuel storage tank including foundation (10,000 gallons)	LS	$15,000	$25,000	$307,500	$512,500
Electrical controls + electrical room	LS	$8,000	$10,000	$72,000	$90,000
24" pipe to discharge point (installed)	LF	$120	$200	$960,000	$1,600,000
Berm grading at sheet flow sites	CY	$4	$10	$1,049,253	$2,623,133
Other sheet flow sites improvements (plantings, etc.)	LS	$25,000	$75,000	$150,000	$450,000
Total				**$10,263,897**	**$18,228,493**
30% Contingency				**$13,343,067**	**$23,697,041**

Item	Unit	Unit Cost ($)		Total Cost for All Six Sites	
		Low	High	Low ($)	High ($)
Annual Leasing Fees	AC	$60	$73	$866,940	$1,047,553

Notes:
1. This cost analysis should be considered in line with a Class 5 Estimate that the Association for the Advancement of Cost Engineering (AACE) recognizes as having an uncertainty range of -20% to 100%.
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3. This cost analysis is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.
4. The estimate of costs for canal improvements based on use of long-reach excavators, minimal improvements, priced per linear foot. No hauling.
5. The low end of the hauling disposal item assumes no hauling or disposal is required and material can be cast off adjacent to canal.
6. The pump stations assume 13,333 gpm pumps to meet approximate pumping rate in model
7. This estimate assumes pumps and associated infrastucture are fueled by diesel generators.
8. Berm grading assumed a 10 ft perimeter berm at approximately 5 feet high.
9. Annual leasing rate based on National Resources Conservation Service (NRCS) Wetland Reserve Program at $1,800 - $2,175 per acre for 30-year easement.
10. Pump locations and canal/pipe alignment based on schematic in Figure 4.3 of report.
11. This cost analysis considers capital costs only and does not include annual operational costs

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Table 4. Annual Operating Costs
Based on Various Annual Budgets

Operational Category	Variable	Unit	Potential Annual Operating Budget of $1,725,000 based on $25/acre assessment for entire watershed	Potential Annual Operating Budget of $475,000 based on $25/acre assessment for areas within the watershed but outside the Refuge only
Diesel Fuel Consumption	48 inch pump flow rate[1]	gpm	50,000	50,000
	Number of pumps	-	10	7
	Total flow rate	gpm	500,000	350,000
	Horsepower of each pump[1]	hp	250	250
	Power required to operate one pump (kW)	kW	186	186
	Number of 400 kW generators required to operate all pumps	-	5	4
	Diesel fuel consumption per hour (full load)[2]	gal/hr	28.0	28
	Hours of operation per year for one pump	hrs/yr	3,000	600
	Annual diesel consumption for one generator[3]	gal/yr	84,000	16,800
	Annual diesel cost for one generator at $3.15/gallon[4]	$	$264,600	$52,920
Estimate of annual diesel costs for all generators[5]		**$**	**$1,323,000**	**$211,680**
Annual Maintenance Costs (5% of Capital Costs)[6]	Capital cost for one pump	LS	$180,000	$180,000
	Capital cost for one generator	LS	$120,000	$120,000
	Capital cost for all pumps	$	$1,800,000	$1,260,000
	Capital cost for all generators	$	$600,000	$480,000
	10% of capital costs for all pumps	$	$180,000	$126,000
	10% of capital costs for all generators	$	$60,000	$48,000
Estimate of annual maintenance costs[7]		$	**$240,000**	**$174,000**
Contingency Fund for Replacement (5% of Capital Costs)[8]	Capital cost for one pump	LS	$180,000	$180,000
	Capital cost for one generator	LS	$120,000	$120,000
	Capital cost for all pumps	$	$1,800,000	$1,260,000
	Capital cost for all generators	$	$600,000	$480,000
	5% of capital costs for all pumps	$	$90,000	$63,000
	5% of capital costs for all generators	$	$30,000	$24,000
Estimate of contingency fund costs[9]		$	**$120,000**	**$87,000**
Total Operating Cost Per Year + Contingency Fund[10]		**$**	**$1,683,000**	**$472,680**

Summary of Performance for Each Scenario	Unit	Potential Annual Operating Budget of $1,725,000 based on $25/acre assessment for entire watershed	Potential Annual Operating Budget of $475,000 based on $25/acre assessment for areas within the watershed but outside the Refuge only
Operational Budget	$	$1,725,00	475,000
Total Pump Capacity	gpm	500,000	350,000
Annual operational hours	hrs	3,000	600
Number of days to draw down lake 0.5 ft	days	10	14
Annual drawdown volume	ac-ft	276,219	38,671

Notes:

1. Assumes 48-inch axial flow pumps at 250 hp with a pump capacity of 50,000 gpm. Assumes pumps and associated infrastructure are fueled by diesel generators.
2. Based on diesel fuel consumption estimate from: https://www.generatorsource.com/Diesel_Fuel_Consumption.aspx
3. Annual diesel consumption is equivalent to the diesel fuel consumption per hour multiplied by the hours of operation per year.
4. Estimate of annual diesel costs for one generator is equivalent to the annual diesel consumption for one generator multiplied by $3.15.
5. Estimate of annual diesel costs for all generators is equivalent to the annual diesel cost for one generator multiplied by the number of 400 kW generators required to operate all pumps.
6. The annual maintenance costs consider 10% of the capital costs for the pumps and generators only and does not include installation costs.
7. Estimate of the annual maintenance costs includes 10% of all pump capital costs and 10% of all generator capital costs.
8. The contingency fund assumes 5% of capital costs for pumps and generators are reserved each year to replace pumps and generators every 20 years.
9. Estimate of the contingency fund costs includes 5% of all pump capital costs and 5% of all generator capital costs to replace pumps and generators every 20 years.
10. The total operating cost + contingency includes diesel costs, maintenance costs, and replacement fund costs. Operational hours were optimized to meet the annual operating budget.
11. This annual operating budget is based on conceptual costs only and unit costs from a wide variety of sources.
12. This annual operating budget is not based on detailed plans, specifications, or expectations, the benefit of bidding, or abnormal market conditions.

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kW = kilowatt
LS = lump sum
yr = year