



Model Documentation Report:

Central and Southern Florida Flood Resiliency
Study Broward Basins (C&SF Section 203)

South Florida Water Management District
Water Resources & Systems Modeling Bureau

Broward County, Florida
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List of Acronyms

AltRO – Resilience Optimized Alternative
ARF – Areal Reduction Factors
BBSEER – Biscayne Bay and Southeastern Everglades Restoration
C&SF – Central and Southern Florida
CEPP – Central Everglades Planning Project
Cfs – cubic feet per second
CHS – Coastal Hazard System
ECB - Existing Condition Baseline (2035 cond.)
ECWP-Baseline (2035 cond.) with project
FDEP – Florida Department of Environmental Protection
FRM- Flood Risk Management
FRM-PCX – Flood Risk Management Planning Center of Expertise
FWOPI- Future without project (2085) low sea level rise
FWOPI- Future without project (2085) intermediate sea level rise
FWOPh- Future without project (2085) high sea level rise
FWP – Future with project (2085)
GW - groundwater
H&H – Hydrology and Hydraulic
HW- headwater
LNWR – Loxahatchee National Wildlife Refuge
MDR – Model Documentation Report
MP – Markham Park
NAVD88 or NAVD – North American Vertical Datum of 1988
NGVD29 or NGVD – National Geodetic Vertical Datum of 1929
NNR – North New River
NSID – North Springs Improvement District
TW- tailwater
Q- discharge
RSM – Regional Simulation Model
20S25R – 20-year surge boundary with 25-year rainfall
2S25R – 2-year surge boundary with 25-year rainfall

35i – ECB year and intermediate sea level rise

85l, 85i, and 85h – Future year 2085 and low, intermediate, high sea level rise, respectively

RSLC- Relative Sea Level Change

SACS – South Atlantic Coastal Study

SL(C) – Sea Level, or Sea Level Change

SWL- Stillwater Level

TSP- Tentatively Selected Plan

WCA – Water Conservation Area

WOP – Without Project

WRDA – Water Resources Development Act (of 1986)

WS – Water Surface, referring to water surface elevation



1.0 Overview

Identification

South Florida Water Management District (SFWMD) is utilizing Section 203 of the Water Resources Development Act of 1986, as amended, to advance the Central and Southern Florida Flood Resiliency Study – Broward Basins (C&SF Section 203 Study) with support from the Florida Department of Environmental Protection, Broward County, and technical/federal assistance from the U.S. Army Corps of Engineers (USACE) – Jacksonville District, for inclusion in the Water Resources Development Act (WRDA) 2026. This Section 203 Study is an offshoot from a larger CS&F Flood Resiliency Section 216 Study (USACE, December 2024), which includes four reaches. The C&SF Section 203 Study area, corresponding to the Broward Basins, was originally identified as Reach A within the larger Section 216 Study which is currently targeting authorization in WRDA 2030. To expedite a portion of this larger effort, the C&SF Section 203 Study was pursued, which leverages the same modeling tools, assumptions and formulation framework of the originating 216 effort which was approved for use by the USACE Flood Risk Management Planning Center of Expertise (FRM-PCX).

The study seeks to enhance the C&SF system water control structure's functionality and capacity to reduce flood damages and improve resiliency caused by inundation and changed conditions over a 50-year period of analysis from 2035-2085. Study and project area boundaries were derived from the respective watersheds and contiguous urban areas where hydrologic, hydraulic and hydrodynamic modeling, including the SFWMD Flood Protection Level of Service Program and the USACE South Atlantic Coastal results, demonstrated highly vulnerable infrastructure in the C&SF system within the lower east coast of Florida in southern Palm Beach, Broward and Miami-Dade Counties. The project study area is shown in Figure 1 which consists of 9 watersheds and 8 structures (7 tidal structures and S37B upstream of S37A). Table 1 lists the managed watersheds and primary structures being studied under the C&SF Section 203 Study. This memorandum documents the Hydrology & Hydraulics (H&H) modeling analysis performed in support of the formulation efforts (baseline scenario development, alternative testing & identification of recommended plan components) in support of the overall C&SF Section 203 Study.

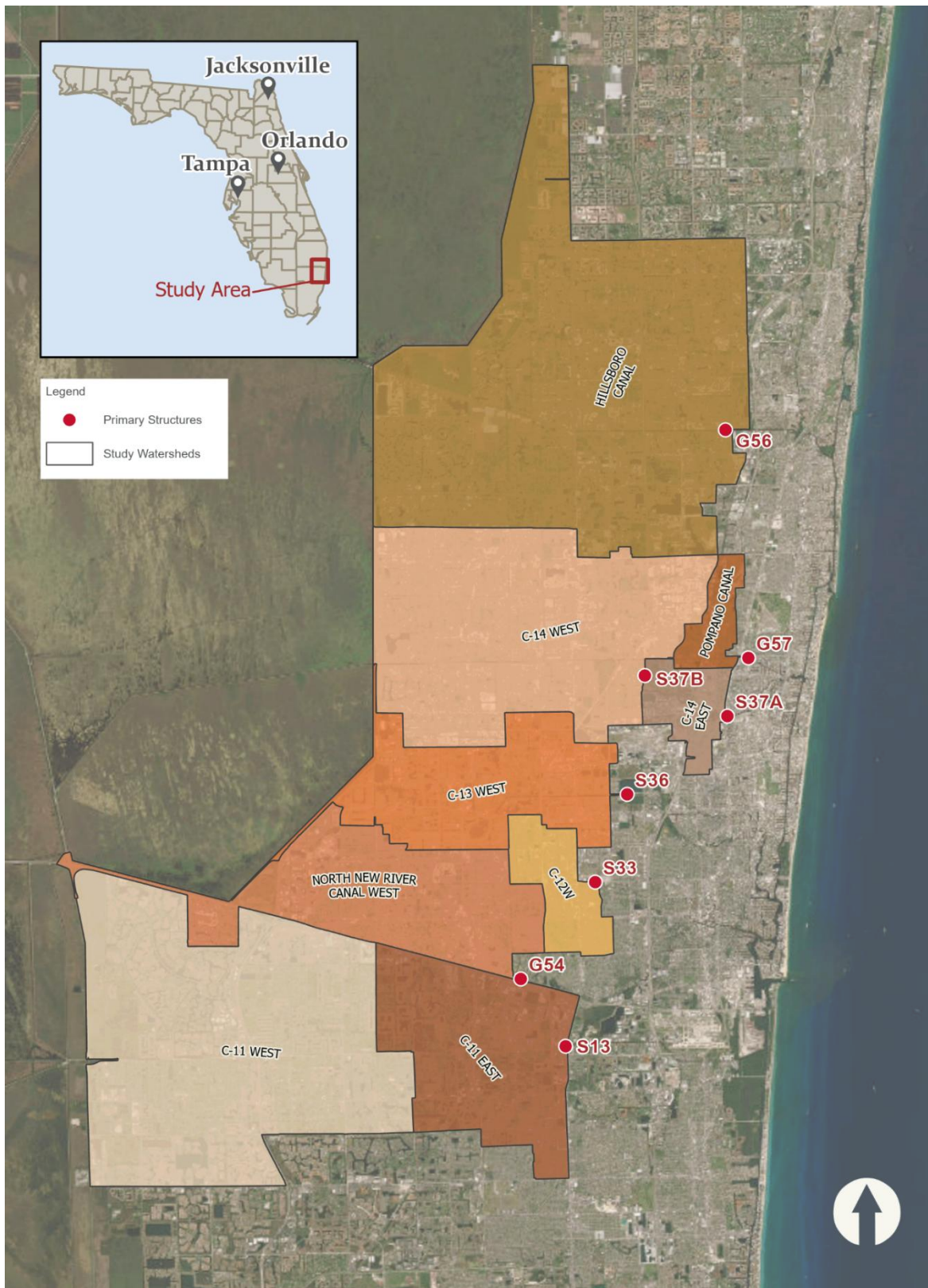


Figure 1 Study Watersheds in Broward County



Table 1 Project Study Area - Managed Drainage Basins

MANAGED BASIN	PRIMARY CANAL	COASTAL WATER CONTROL STRUCTURE
Hillsboro Canal Basin	Hillsboro (G-08) Canal	G56 Gated Spillway
Pompano Canal Basin	Pompano (G-16) Canal	G57 Gated Spillway
C-14 West Basin	C-14 Canal	S37B Gated Spillway
C-14 East Basin	C-14 Canal	S37A Gated Spillway
C-13 West Basin	C-13 Canal	S36 Gated Spillway
C-12 West Basin	C-12 Canal	S33 Gated Spillway
North New River Canal West Basin	North New River (G-15) Canal	G54 Gated Spillway
C-11 West & East Basins	C-11 Canal	S13 Pump Sta. & Gated Spillway

Scope and Objectives

Modeling support to the C&SF Section 203 Study was provided by a team comprised of modelers from the Water Resources & Systems Modeling Bureau of the SFWMD, and modeling consultants from Hazen and Sawyer. The project planning team consisted of SFWMD staff, SFWMD project contractor J-Tech and design consultants from Jacobs Engineering. The focus of the 203 effort is to identify flood risk management (FRM) recommendations to build flood resiliency, now and into the future, in the C&SF system of primary canals and water control structures within the lower southeast coast of Florida in southern Palm Beach, and Broward Counties.

Modeling support for the C&SF Section 203 Study focused on working with the project planning team and other interested parties to formulate and test project features leading to the ultimate identification of a Tentatively Selected Plan (TSP), later formalized as the Recommended Plan. An integrated/coupled surface-groundwater MIKE SHE/MIKE Hydro (Version 2019) model was previously created and calibrated by the SFWMD (Taylor Engineering, Inc. with Anclothe Consulting PLLC, 2021), and was refined and re-calibrated under the Broward County Risk Assessment and Resilience Plan effort (Hazen and Sawyer, April, 2024). The model is utilized to generate flood hazard information for use in

the flood damage and effects assessment. This 2D hydrologic and hydraulic (H&H) model covers the entire study area and extends past the tidal boundary condition to the east.

MIKE SHE | MIKE Hydro is an integrated surface water and groundwater hydrological & hydraulic modeling tool (DHI, 2022). MIKE SHE represents the hydrological model whereas MIKE Hydro which is coupled with MIKE SHE is where the hydraulic modeling is performed. The configuration used for C&SF Section 203 Study modeling is a 1D-2D coupled model where river reaches, cross-sections, and hydraulic structures are represented in the 1D (MIKE Hydro) hydraulic model and where floodplains, aquifers, saturated and unsaturated zones, precipitation, and evapotranspiration are modeled as 2D in MIKE SHE. MIKE SHE computes water depths/elevations, velocity, and flow (and several other variables) in the 2D gridded model. In other words, variables are calculated in a grid system which is defined by the user at the outset of the modeling process. Taking into account the interaction of the different modules (precipitation, evapotranspiration, unsaturated zone and saturated zone), MIKE SHE calculates the water depths/levels. Through the MIKE SHE - MIKE Hydro coupling, water depths/elevations are updated at river cross-sections as well as the 2D grid system. MIKE Hydro solves the Saint-Venant equations (higher order fully dynamic equations for this application) and the MIKE SHE solves the diffusive wave equations for cell-by-cell overland flow. These results can be accessed through the gridded output MIKE SHE results or through MIKE View using the .res1D result file. See Figure 2 for Reach A Model Domain, Mike 1D hydraulic branches, and the coastal water control structures.

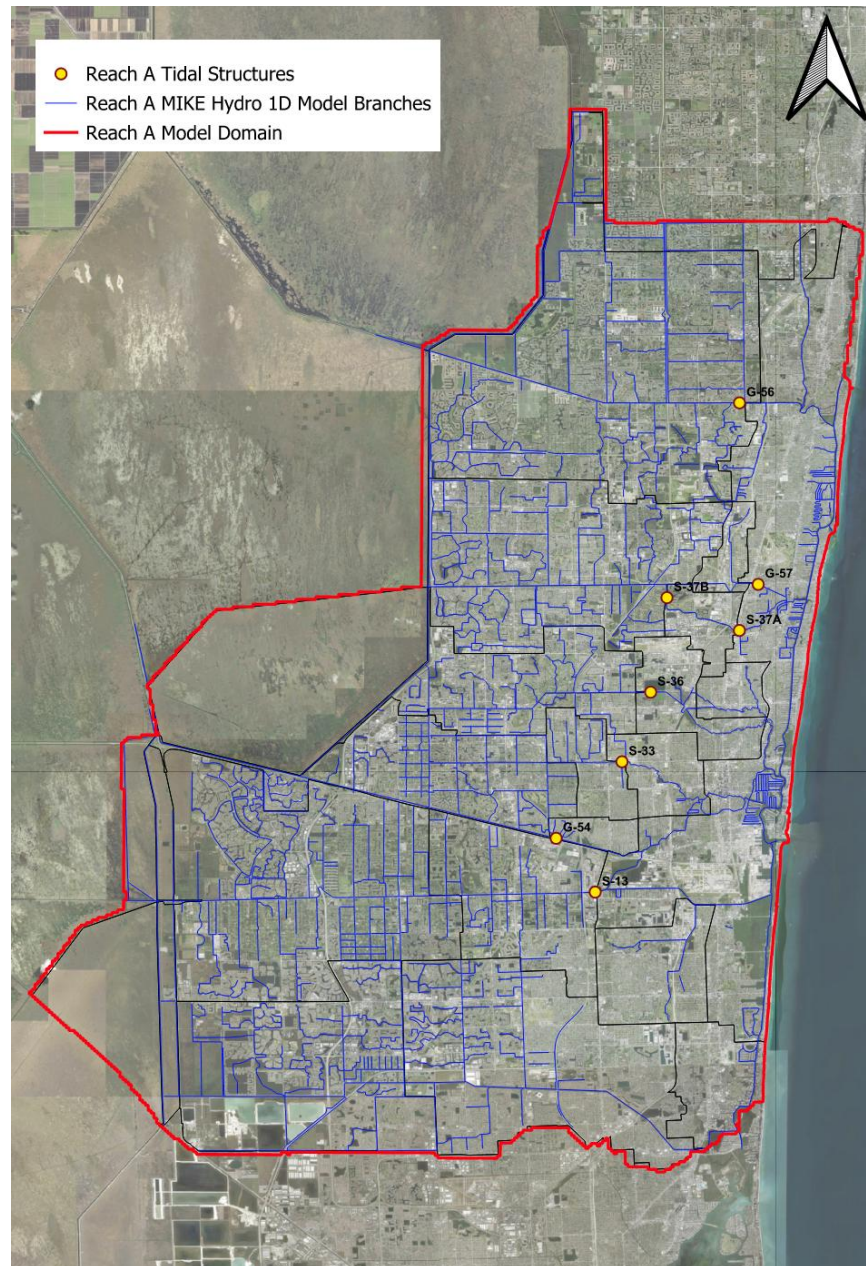


Figure 2 Reach A Model Domain, 1D branches, and tidal structures

This MIKE SHE/MIKE Hydro H&H modeling tool utilized for Reach A has a strong history as a flood modeling tool. From the original developed model in 2014 as a FEMA tool, the Broward County Mike model was updated and calibrated by consultants in 2020 to create future conditions flood maps for Broward County (Taylor Engineering, Inc., November, 2019). In 2022, under the SFWMD Flood Protection Level of Service effort, the model was further updated with 2018 LiDAR and surveyed cross sections, an extended model domain and additional secondary drainage system operations with the focus on evaluating the primary system. The model was re-calibrated to No-Name storm of June 2017 and

validated to the September, 2017 Hurricane Irma event (Taylor Engineering, Inc. with Anclote Consulting PLLC, 2021). For use by Broward Resiliency, the FPLOS model was then further updated with additional downstream detail (downstream of the tidal control structures), an extended eastern boundary, and other refinements including improvements to the hydraulic secondary, tertiary, and coastal drainage representation. The Broward Resiliency model was updated to version 2022 and re-calibrated (Hazen and Sawyer, April, 2024), (SFWMD, 2023), which is the model the project team started and applied to meet the objectives and requirements of the C&SF Section 216 and Section 203 studies. Further refinements to this model for this study is discussed in the project assumptions section of the report. The focus of these studies is the primary canal system; however, the model includes a high level of detail within the secondary and tertiary canal systems.

This Model Documentation Report (MDR) describes the assumptions, model implementation steps and observed outcomes associated with modeling representations of both the current and future without project conditions baseline model scenarios as well as the proposed project alternatives as simulated with the MIKE SHE/MIKE Hydro software for the C&SF Section 203 Study. These model runs were used as a basis of comparison and justification of features in support of plan formulation. This document will focus on the modeling details of these scenarios.

2.0 Basis

Project Assumptions

Four (4) scenarios or conditions were analyzed in the modeling: the Existing Condition Baseline for 2035 (ECB); the Existing Condition With Project for 2035 (ECWP); the Future Without Project for 2085 (FWOP); and the Future With Project for 2085 (FWP). Future scenarios include evaluation across three different sea level rise (SLR) scenarios: low, intermediate and high as identified using the USACE method and applied at the VACA KEY gauge shown in Figure 3 with rates and offsets of Table 2.

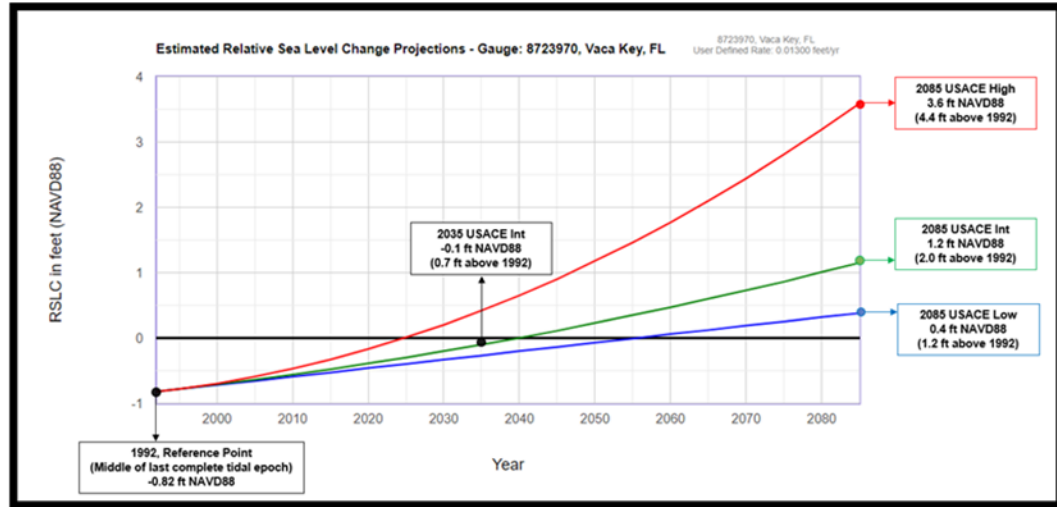


Figure 3 Sea Level Change Scenarios for the Selected Tidal Gauge (USACE, December 2024)

Table 2 Projected MSL elevations for Vaca Key Tidal Gauge using 0.013 ft/yr SLC rate (USACE, December 2024)

	Projected MSL Elevations (ft NAVD88)		
Year	USACE Low	USACE Int	USACE High
1992	-0.82	-0.82	-0.82
2023	-0.4	-0.3	-0.1
2035	-0.3	-0.1	0.4
2085	0.4	1.2	3.6
2135	1.0	2.9	8.6

Terminology used throughout the study and this report, in particular, is based on some acronyms and naming conventions presented in the Table of Acronyms at the beginning of the report. Eight design events were simulated for each of the scenarios which consisted of rainfall driven event simulations combined with downstream surge boundary conditions which were selected during the plan formulation process in the Section 216 study (USACE, December 2024) and documented in the USACE's April 4, 2024 Memorandum for Record (USACE, April 2024). For each scenario of features, both with and without project, the eight design events would be run for the 2035 intermediate sea

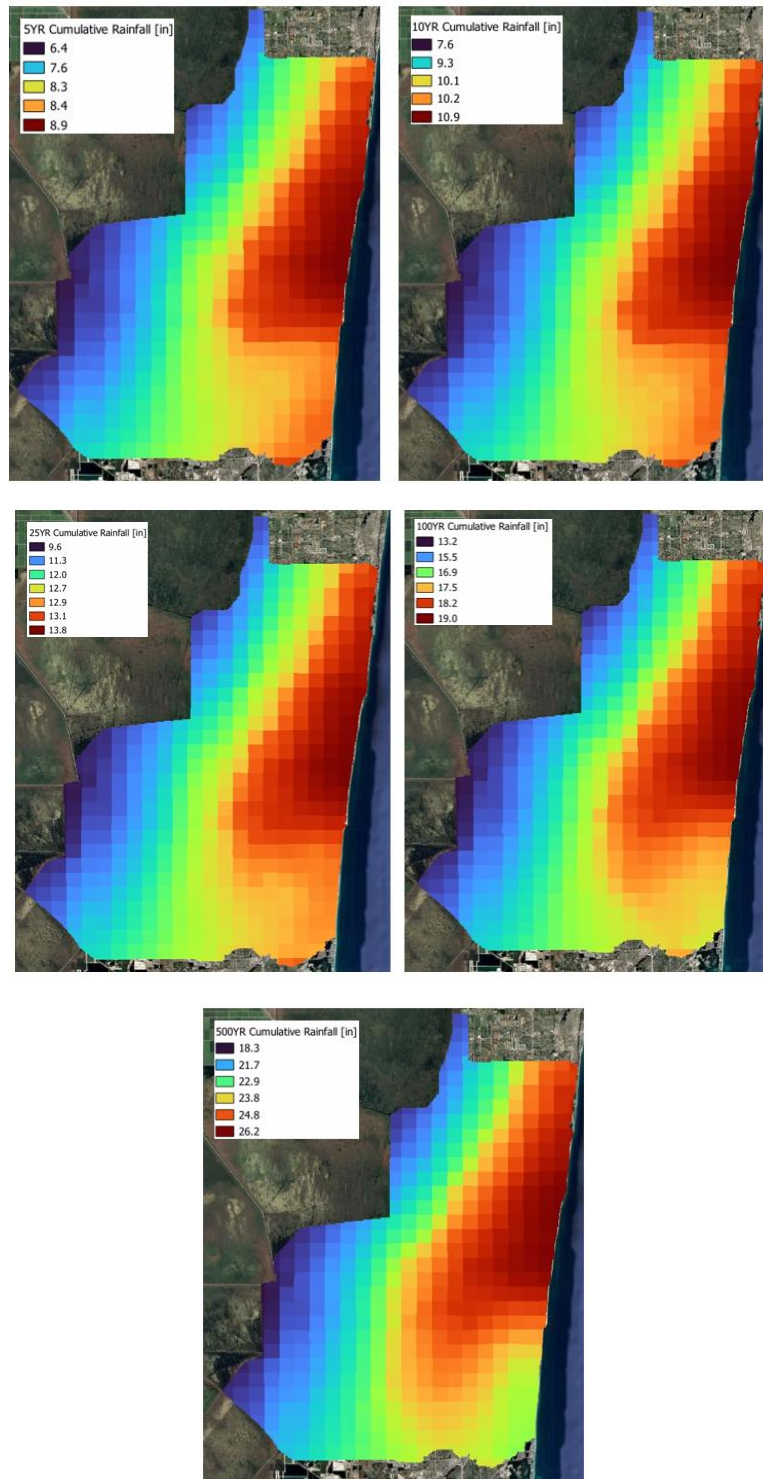
level and eight events each for the 2085 low, intermediate, and high sea level conditions. The eight design events are listed in Table 3 and the assumptions for determining the rainfall inputs and coastal water level tidal boundaries are described.

Table 3 C&SF Section 203 Study Design Events Simulated

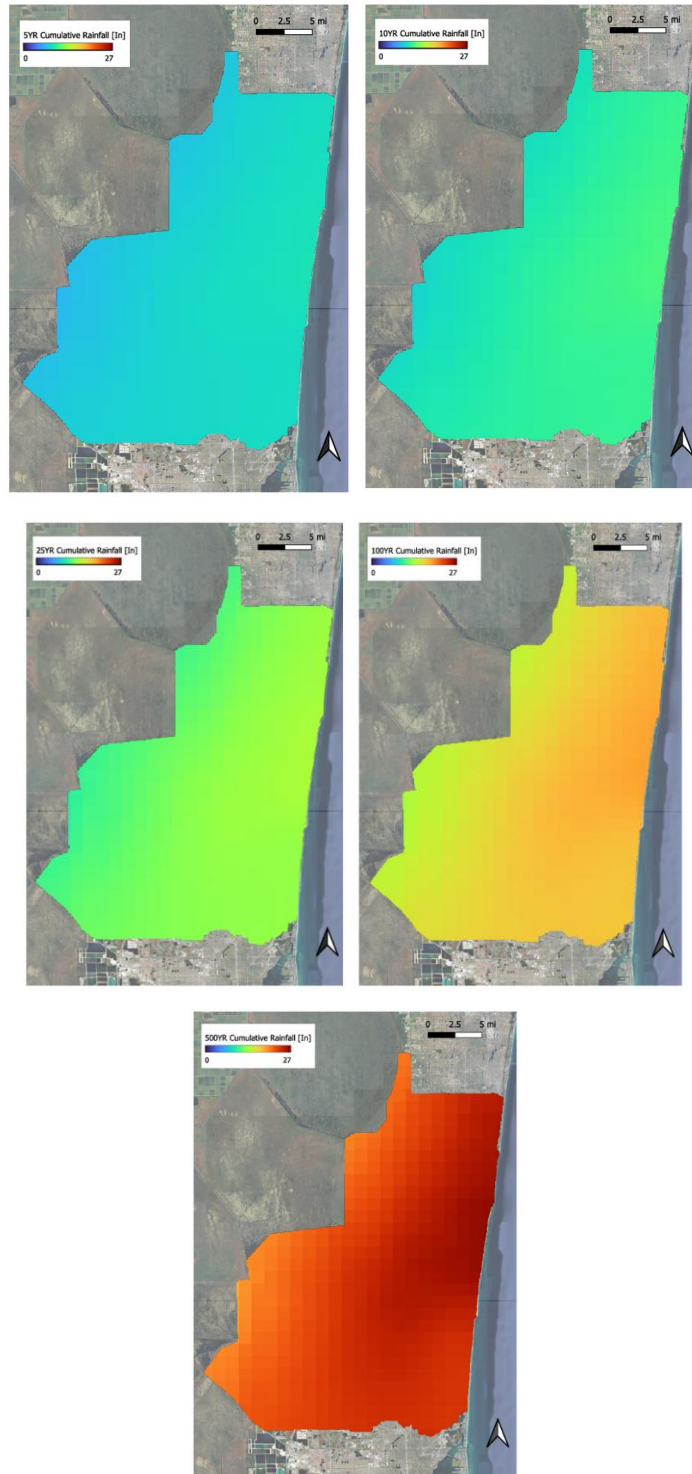
Coastal water level Return Period	Rainfall return period
2-year	5-year
2-year	10-year
10-year	10-year
2-year	25-year
20-year	25-year
2-year	100-year
100-year	100-year
2-year	500-year

For rainfall return period, modelling uses spatially distributed gridded input derived from NOAA Atlas 14 precipitation frequency estimates for 3-day rainfall using annual maximum time series (NOAA , 2023). Precipitation frequency estimates from the NOAA Atlas are based on point estimates and USACE guidance requires Areal Reduction Factors (ARF) to be applied for areas larger than 10 square miles (USACE, 1994). The reductions are determined based on the drainage area and the areal reduction factor region (Irizarry-Ortiz M. M., 2022). The reduction factors were relatively small for Reach A and are shown as blue shadow box numbers in Figure 4 within the red outlined model domain. The resulting gridded 3-day rainfall depths with areal reductions applied in the model domain are shown in Figure 5 for the 5-, 10-, 25-, 100-, and 500- year rainfall recurrence intervals. 5a shows rainfall variations across the modeled area for each recurrence interval and 5b shows a visual range between the 5- and 500- year rainfall recurrence intervals.





5a Showing rainfall variations across the modeled area for each recurrence interval



5b. Showing variations in rainfall between recurrence intervals

Figure 5 Gridded Cumulative 3-Day Design Rainfall Depths Applied

The applied rainfall was temporally distributed based on the SFWMD 3-day distribution as found in the District's Surface Water Environmental Resource Permit Manual (SFWMD, 2016). The distribution shown in Figure 6 is consistent with criteria used for permitting stormwater management systems within the study area and is widely used and accepted in flood studies carried out by county and city municipalities, and FEMA riverine flood studies in the region.

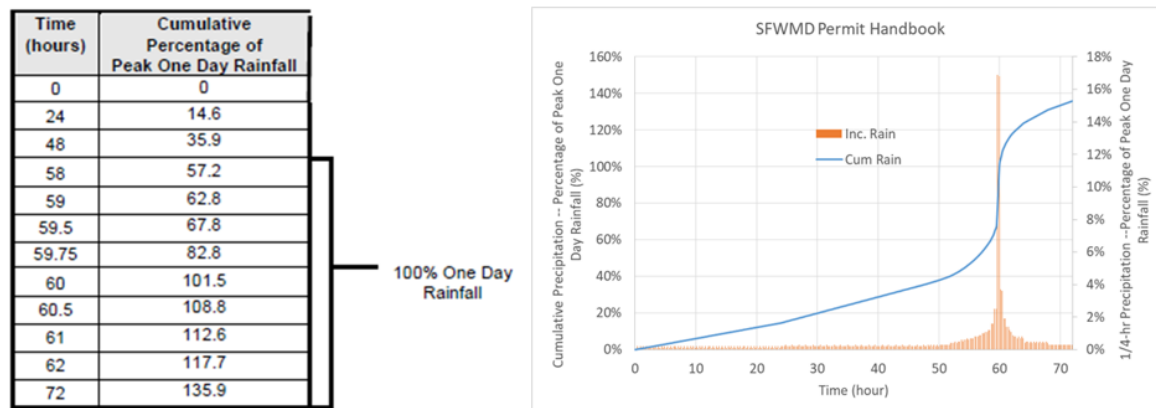


Figure 6 SFWMD Temporal Distribution of Rainfall (SFWMD, 2016)

Boundary tidal time series were provided by coastal engineers at the USACE and described in the referenced document (USACE, September, 2023). The hydrographs are a combination of tidal signals from historic predicted tides and storm still water levels (SWL). The SWL (peaks of the surge event) were based on South Atlantic Coastal Study (SACS) Coastal Hazard System (CHS) surge modeling and were pulled at locations along the coast which best represented the coastal variation in peaks referred to as savepoint locations. CHS savepoints were grouped based on peak still water levels to identify representative savepoints throughout each model reach and were selected by USACE Coastal Engineers. For Broward County Reach A, four savepoint locations (refer to Figure 7) were determined to adequately represent the coastline from north to south and approximate locations where the data was pulled from the surge model are shown in Figure 8. This is an improvement to previous Broward basin models where the eastern boundary was represented by just one or two boundary conditions.

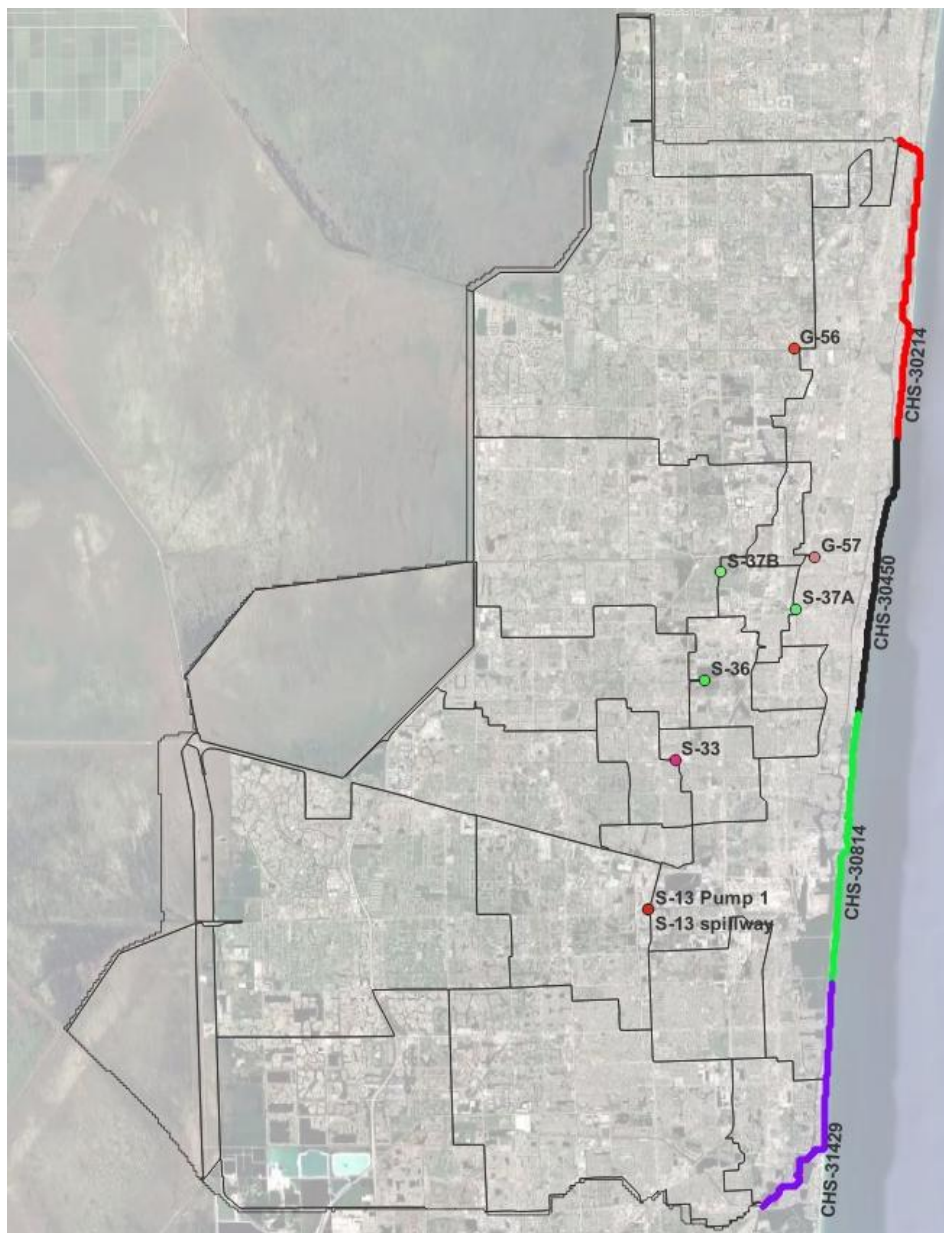


Figure 7 Boundary conditions applied to model (obtained from CHS savepoints)

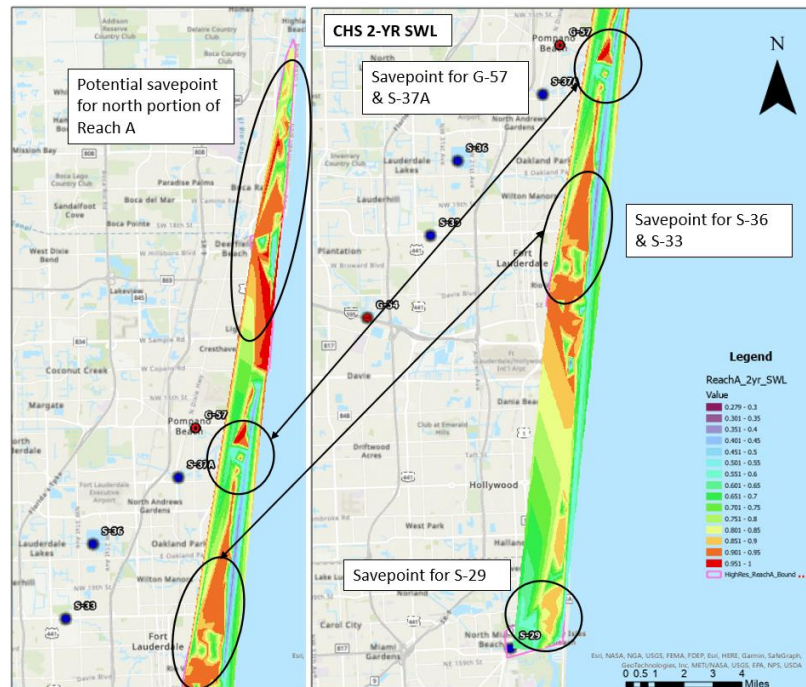


Figure 8 Example of CHS Savepoint Selection Process for Reach A

The SWL from CHS data was available for different sea level rise conditions but not for the same sea levels for study in this FRM study. For this study, SLC was linearly applied to the SWL from the coastal model to yield appropriate SWL under 203 study conditions. Tidal signal hydrographs were developed based on the NOAA gauges in the vicinity of the project area, which for Reach A were the Lake Worth Pier gauge and the Virginia Key gauge. Variations in peak tidal boundaries are tabulated for intermediate sea level in Table 4 for the four savepoint locations. Since the SLC was linearly applied, peaks for other sea level conditions can be determined by adding or subtracting differences in sea level. For high sea level, add 2.43 ft, and for 2035 intermediate, subtract 1.3 ft which are the differences in sea level as shown in Figure 3. In general, the peaks at the northernmost savepoint location, 30214, are the highest and the maximum difference occurs between 30214 and 30814 for the 100-year recurrence interval. Storms selected to create the data could be a factor and it is recommended to refer to the Hydrograph Metadata for additional information (USACE, September, 2023).



2085i Coastal Surge Return Period	CHS Savepoint Peak Elevations, ft NGVD29			
	30214	30450	30814	31429
2-year	6.74	6.70	6.57	6.35
10-year	7.89	7.59	7.44	7.51
20-year	8.34	8.04	7.88	8.18
100-year	9.46	9.03	8.84	9.26

Table 4 Variation in CHS Savepoint Peak Elevations for Intermediate SL

Example tidal boundaries are shown based on the northernmost save point location (CHS 30214) in Figure 9 and Figure 10. Figure 9 shows variation with sea level for the 2 yr recurrence interval surge, and Figure 10 presents the variation in surge peak with recurrence interval for 2085 intermediate sea level. The peaks and tidal signal are defined for each of 4 save point locations from north to south along the coast.

The Reach A model was built in the National Geodetic Vertical Datum of 1929 (NGVD29). CHS data and tidal signal hydrographs were converted to NGVD29 consistent with the model vertical datum using an SFWMD conversion raster file. The conversion values in the conversion raster file are based on the USACE Corpscon software.

Unless otherwise specified, the elevation of all model inputs and outputs are presented in NGVD29 datum which is the datum of Figure 9 and Figure 10.

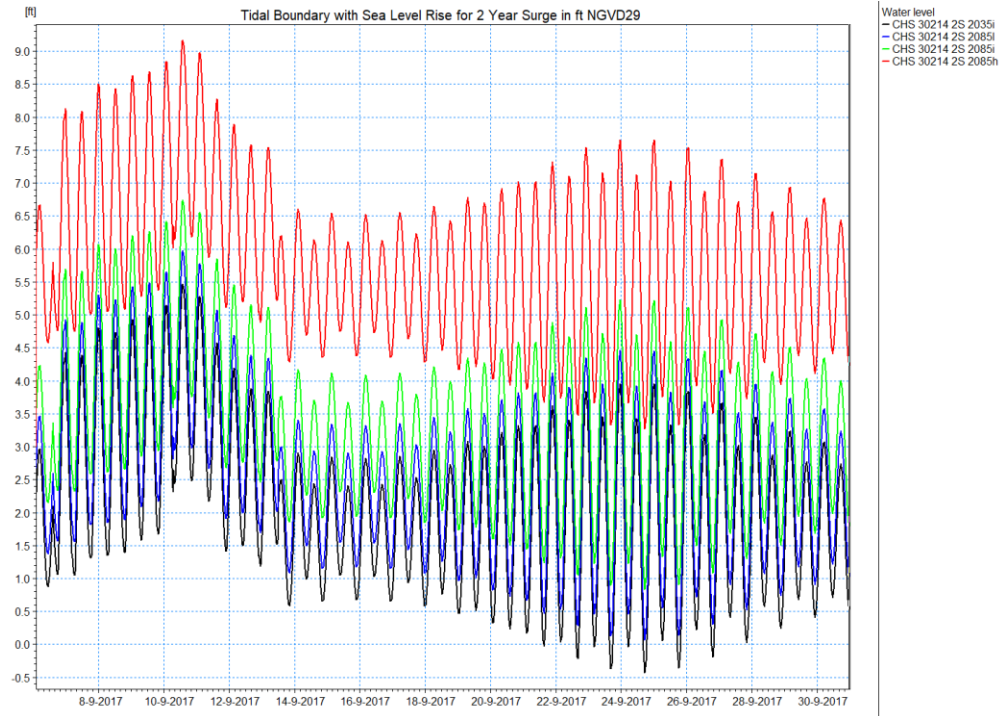


Figure 9 Example Tidal Boundary at 2035i, 2085i, 2085i, and 2085h for 2 Yr Surge

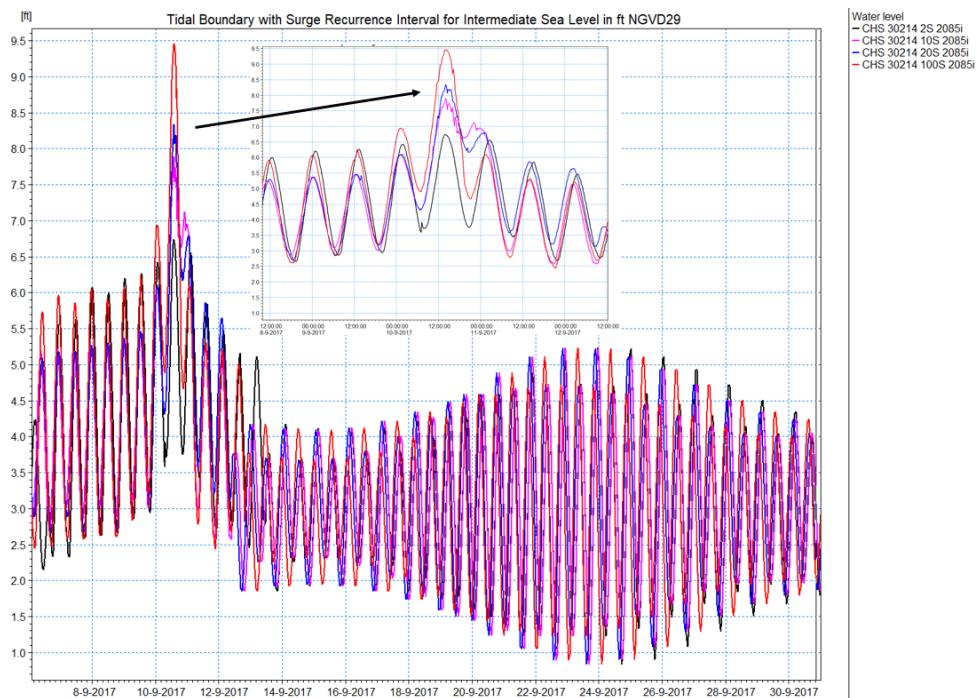


Figure 10 Example Tidal Boundary for 2, 10, 20, and 100 Surge Recurrence Interval at 2085i SL



Baseline Modeling

In order to evaluate the flood risk reduction from the various measures identified in this study, the project defines baseline conditions without the proposed measures, but with changes to the current conditions described in this section. The study evaluates projects over a fifty-year planning period starting with the year 2035. Four baseline conditions were analyzed in the modeling: the Existing Condition Baseline for 2035 (ECB), and the Future Without Project for 2085 (FWOP) under low, intermediate, and high sea level conditions. A Future Without Project (FWOP) model will be used to determine increased flood risk or damages resulting from changes to future conditions of land use, groundwater, rainfall, sea level rise, and tides contributing to flooding in the project area. For this study, the FWOP features were defined during the planning efforts for the 2023 Central and Southern Florida (C&SF) Flood Resiliency (Section 216) study and, therefore the ECB and FWOP modeling and plan formulation framework explained in this document follow the Section 216 guidance. The analysis of the FWOP condition involves the use of the hydrologic model tools to determine flood depth and duration as well as the economic tools to establish baseline damages for assessing damage reduction or avoidance from the proposed projects or measures.

Starting from current conditions, the project team identified known planned changes, changes already in process or that have been authorized and are reasonably expected to be completed by the future condition date. These changes include physical changes, structural changes, regulatory changes and operational changes approved within the study area. Future sea level, tides, and groundwater levels contributing to flooding are also included which ensure the model representation of the future condition is adequate and appropriate for use.

For the modeling changes, the project team developed ECB and FWOP models by updating current conditions models with applicable changes including future land use, future projects, future sea level rise and tidal boundaries, and increased initial groundwater elevations resulting from sea level rise.

The initial FWOP considerations for land use will be to apply changes to current conditions land use by identifying areas of change in each County's projections of future land use from their respective Future Land Use plans. The information on future land use projections was already made available and validated by the Counties. The Existing Condition Baseline is 2035 so both ECB and FWOP utilize the same future land use.

Future projects included are: CEPP Studies (approved and fully funded) and selected County, State, and Local Projects that have a high implementation certainty. Biscayne Bay and Southeastern Everglades Restoration (BBSEER) projects in early planning stages and Miami Back Bay Study projects will not be included. The Existing Condition Baseline is 2035 so both ECB and FWOP share many of the same future projects.

Future sea level rise and tidal conditions that are applied as downstream boundary conditions were derived from the South Atlantic Coastal Study (SACS) Coastal Hazard System (CHS). This system provides numerical and probabilistic modeling results for coastal forcings, including storm surge. A combination of eight (8) simulations were

conducted for various combinations of rainfall-based (pluvial) flooding and coastal hazards such as storm surge.

Appendix A-2.1.A contains assumption tables for ECB and FWOP, respectively. More details are included in 3.0 Simulation Model Setup for both Baseline Model (ECB) and Future Without Project (FWOP).

Screening & Initial Array Modeling

Formulation of possible management measures for the project was informed from several sources including 1) the C&SF Section 216 Study which compiled an initial management measures matrix, 2) The SFWMD Flood Protection Level of Service (FPLOS) evaluations which recommended management measures for further study, 3) the Broward County Resiliency Planning, and 4) Public comments Scoping and Kickoff Meetings. A master list was compiled and screened through the plan formulation process (USACE and SFWMD, 2024). Many of the screened features needed to be assessed separately to determine their effectiveness before combining into scenarios. Through iterative testing, the drainage basins were shown to be sensitive to features such as improved conveyance, pumping at tidal structures, relocating and/or adding tidal structures, and use of additional storage and inter-basin flow (where available). The testing also determined that drainage basins and their primary systems had different responses to the measures and there was not a 'one size fits all' approach. Combining the iterative process across seven drainage basins each with its own tidal structure and the process was substantial. Further refinements to features were also a part of the process considering details such as a) management operations, b) pump sizes, and c) effective canal and structure conveyance locations. Details are described in Section 3.0 Simulation under Preliminary Reach A Testing and Initial Array.

Final Array Modeling & Resilience Optimized Alternative (Alt RO)

The three final array alternatives were informed by the initial array and initial array refinement testing. The three alternatives increase in complexity starting with Alternative A which has the least number of features and expected lower cost, and move to Alternative B which adds additional complexity and project features, and finally to Alternative C which includes the most projects and therefore expected to be the most expensive. The Alternatives allow for the determination of which drainage areas are improved most by the additional cost and complexity so that the best projects for each drainage area can be carried forward to a fourth alternative which is the Resilience Optimized Alternative (Alt RO). Once identified, the Alt RO scenario is also compared back to the other alternatives as part of the comprehensive benefits analysis. Details are described in Section 3.0 Final Array of Alternatives.

Model Limitations and Intended Use of Results

The MIKE SHE model is a gridded fully integrated groundwater and hydrologic model with rainfall/ET, infiltration, recharge, overland flow and ponded drainage routines (stormwater conceptualization). The gridded mesh is linked to MIKE 1D with channel hydraulics and



complex structure definition and operational rules. Characteristics that are key to the model's suitability for the study are:

- Able to simulate the effects of different boundary conditions such as tidal and surge-influenced tailwater conditions with current and future sea level rise scenarios
- Comprehensive operational flexibility, can simulate structure gate operating rules and can use calibrated flow parameters for canal structures
- Simulates surface water and groundwater interactions which is important for the area where the water table is typically close to the ground surface
 - Allows for the accounting of rising water tables and reduced soil storage seen with rising mean sea levels
- Capable of outputting flood depth/stage across the project domain than can be input into economic models to establish the flood benefit of different scenarios

The fact that the model has seen extensive development and has been calibrated and validated combined with the attributes of the tool resulted in the software being "approved for use" in the study through the USACE engineering model certification process (Section 216 guidance – Justification Memo USACE, July 2023).

The user of the data should recognize that many assumptions of future conditions and alternative project features are being applied in this study. The data presented in the modeling report was prepared for the purpose of the C&SF Section 203 Study goals. The maps and data in the figures and the appendices were prepared for use by the Project Delivery Team (PDT) that is studying the C&SF complex system. They represent the results of data collection and modeling assumptions about future conditions designed to assist in evaluating flood risk over a large project footprint. The South Florida Water Management District modeling team has prepared modeling simulations and output at the planning level for the project and makes no representations to the suitability or accuracy of the data for any other purposes.

When utilizing the modeling data, it is important that the intended use of the data and the relationship between this use and the underlying model uncertainty be considered. Because numerical models operate at finite spatial and temporal resolution, small variations in simulated stages or durations can arise from grid scale, solver convergence, or rounding rather than real hydrologic change. Many small differences between scenarios arise from this model noise influenced by grid averaging, adaptive time-stepping, or the simplifications inherent in the solver (DHI, 2023). In some cases, where aggregate summaries are being used and there are limited model biases, it is appropriate to use precise model output to inform overall performance trends. In other cases, the concept of model-specific tolerances should be employed to emphasize focus on genuine physical signal rather than numerical noise, especially when acknowledging the low-relief topography of South Florida. For this model, it is generally recommended that for site-specific, stand-alone evaluation a stage tolerance of ± 0.10 ft be considered the primary threshold for interpreting scenario differences, with an expanded screening band of ± 0.20 ft in tidal or coastal cells (or other areas) where gradients are smallest, boundary forcing contribute additional uncertainty or highly complex physical interactions are

conceptualized in an abstract way in the model. It is also important to ensure that consideration of such tolerances should be made to addresses random and scale-related variability, not systematically correct for known model bias (USACE, 2019).

3.0 Simulation

Modeling Tools Used

MIKE SHE/MIKE Hydro was used to evaluate the C&SF Section 203 Study for the subject area in consideration of the low relief topography, unique hydrogeology, complex control structure operations, and high-water table including changing water table with future conditions of sea level. All simulations were performed using the MIKE Zero Release 2022 Update 1 platform (www.mikepoweredbydhi.com). The model topography uses vertical datum of ft NGVD29.

Model Set Up

Baseline Model Set Up

Western Boundaries: Western boundaries and the northern boundary of the model (L-38) are set based on representative storm conditions using the Regional Simulation Model Glades – LECSA (RSMGL) data from the Biscayne Bay and Southeastern Everglades Restoration (BBSEER) future without simulation. This RSMGL simulation incorporates the Central Everglades Planning Project (CEPP) features listed in the Appendix A-2.1.A Tables, which contains components identified for inclusion in both the C&SF Section 216 and C&SF Section 203 studies (USACE and SFWMD, 2024). The boundaries provided by RSMGL include time varying elevations at the west Miami Canal, WCA-3B, WCA-3A, WCA-2B, WCA-2A, and WCA1. The RSM data was provided for long term simulation period of 1965 – 2016. The data was reviewed for wet season elevations by looking at the October data average, 90th, and 10th percentile elevations. The variation in storm peaks was not found to be significant so a representative storm time series was selected as the model boundary which was 1999 Hurricane Irene, and the same boundary was applied for all design events. The timing of the western boundary conditions used for the water conservation areas as well as the northern boundaries align with that of the CHS boundary timeseries (surge boundaries). Additionally, in keeping with planned design of seepage canals to control seepage from the West to the urban areas of Broward, leakage coefficients are used to limit seepage at WCAs-2B and -3A/-3B.

Land Use: Using geospatial analysis, Land Use was updated within the model for areas of significance to the resolution of the model grid and appropriately account for parameterization changes due to changing land use. The update was performed under previous studies (Taylor Engineering, Inc. with Anclothe Consulting PLLC, 2021) and summarized here. The land use was updated by comparing undeveloped, agricultural, and low development areas within the model domain to future conditions land use maps for 2040 (see Appendix A-2.1.A Assumption Table). This resulted in about 7500 acres of land use change within the Broward County portion of the model not counting the impoundments which were modeled to design. The majority of the acreage was handled



by raising the topography to the FEMA Base Flood Elevation (BFE) and defining new land use for parameterization. Considering the scale of the model, areas of changed land use greater than 150 acres underwent additional adjustments appropriate to the project size which would include topographical changes to raise ground elevations to the FEMA BFE and compensation for storage onsite consistent with permitting requirements. There were five areas that were greater than 150 acres which were modeled explicitly with compensating storage (not including the impoundments). Since the baseline setup was for 2035, both baseline and future conditions set ups have the same land use formulation.

Future Projects: Future projects included in the ECB model were based on previous project considerations in the Section 216 C&SF Initial Draft Project Considerations for Baseline and FWO table prepared for the Draft Integrated Report (USACE and SFWMD, 2024). The guidance for inclusion of projects was that they were authorized and funded, in substantial design phase, and expected to be completed by 2035.

Projects which were already included in the model from previous studies (Taylor Engineering, Inc. with Anclore Consulting PLLC, 2021) are listed below and refer to the Appendix A-2.1.A for additional location and operational information:

- The Broward County Water Preserve Areas (BCWPA) which consist of the C-11 and C-9 Impoundments and seepage management areas.
- The Site 1 Impoundment Phase I features (Phase II not authorized and not included). The project is located north of the G-08 canal (Hillsboro Canal).
- Broward County Sample Road Recharge Line which is the addition of a gated culvert located at Sample Road and Military Trail in Deerfield Beach.
- North Springs Improvement District (NSID) Pump Station 3 located between the east of Meridian Drive North in Broward County, at the Hillsboro Canal (Figure 11). The pump has a discharge capacity of 25000 GPM and discharges to the Hillsboro Canal.

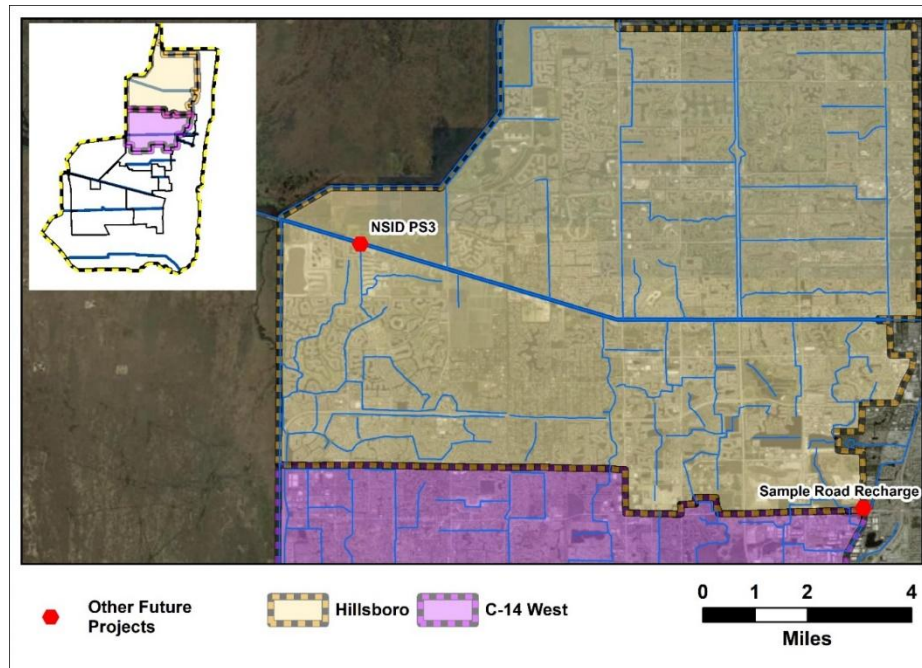


Figure 11 Location of NSID PS3 and Sample Road Recharge (Taylor Engineering, Inc. with Anclothe Consulting PLLC, 2021)

Projects which were added to the ECB model as a result of Section 216/203 project considerations are the following:

Broward County Canal Structure S-27 (Figure 12) was previously modeled as a rectangular culvert but was updated to two pipe culverts with flap gates (only positive flow allowed). The purpose of the new structure design was to prevent backflow of tidal water at high tide and storm surge but to release floodwater to C-13 during storms and is expected to be completed in 2026.

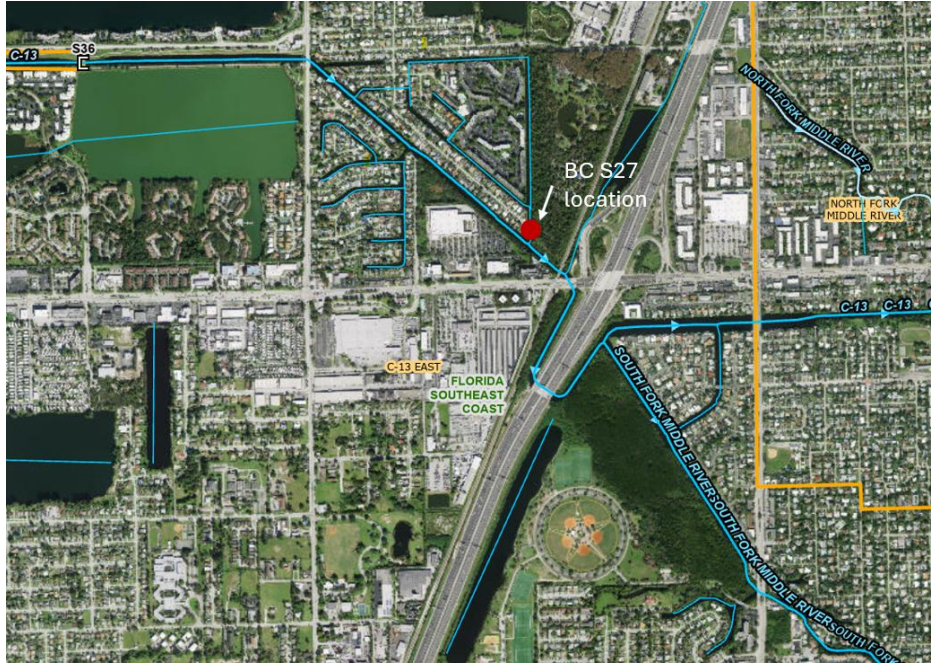


Figure 12. Location of Broward County Structure S27

- The Port Everglades Bulkhead Replacement Project is a FDEP Resilient Florida project focusing on seawall replacement to accommodate five feet of SLR. Figure 13 shows locations of raised seawalls which were accomplished in the modeling by raising the topography in model cells where the seawall replacement is proposed. Per the areas denoted in Figure 13, the following elevations were implemented in the raised topography with the asterisked exception. There was a discrepancy in the model and the Berth 1A, 1B, 1C, 1D was raised ~1 ft higher than the plan. The error does not affect the study outcomes but is acknowledged and noted for future modeling.
 - Berth 3: 5.0ft NAVD88 (6.591ft NGVD29)
 - Berth 2,1: 6.1ft NAVD88 (7.691ft NGVD29)
 - Berth 1A,1B,1C,1D: 5.1ft NAVD88 (6.691ft NGVD29)*
 - Bulkhead: varies from 4.5 to 8.0ft NAVD88 (between 6.091ft and 9.591ft NGVD29)

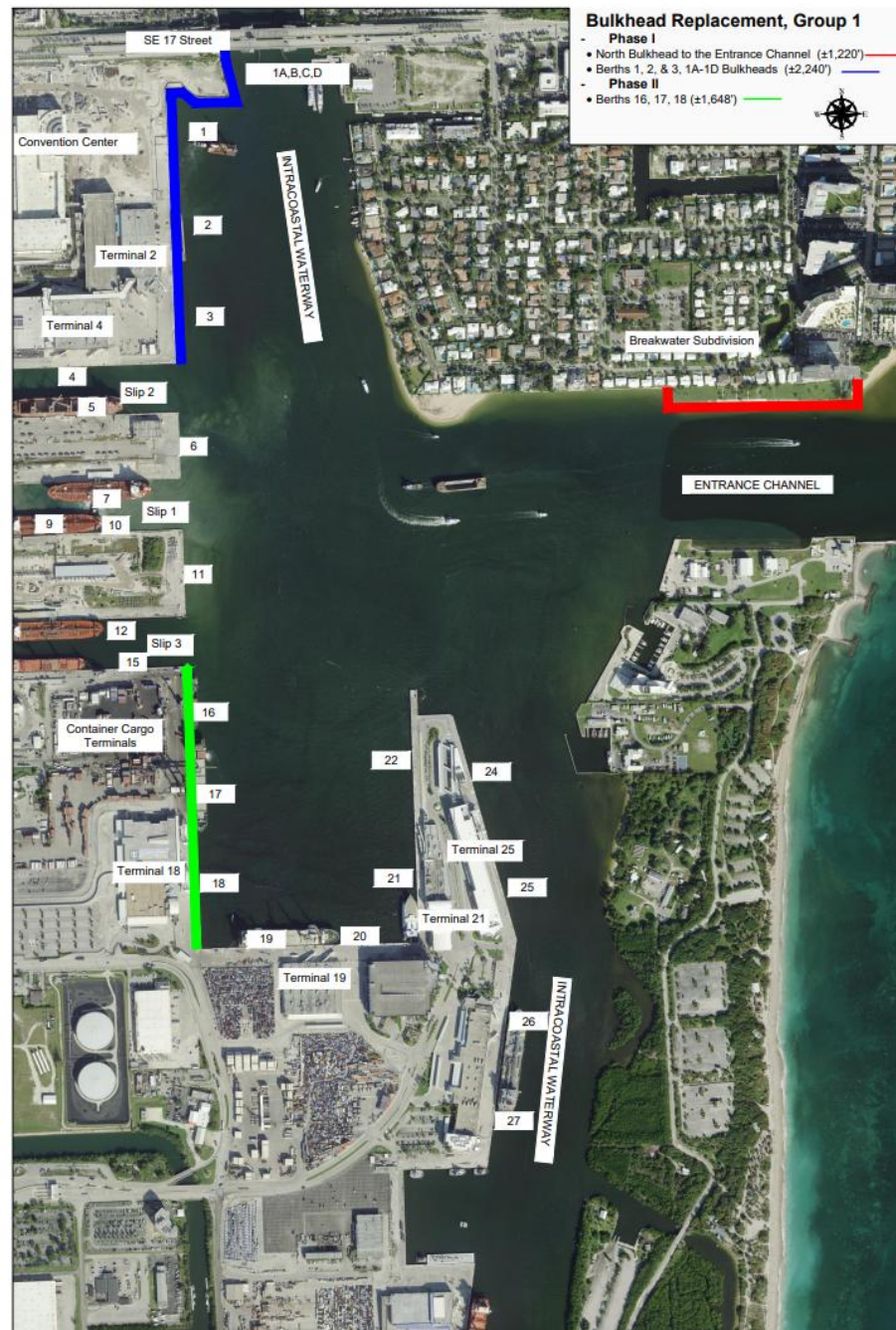


Figure 13 FDEP Resiliency (Environmental Resource License # DF22-1131)

Other FDEP Resilient Florida Projects were included in the study but when reviewed, it was determined that the drainage improvement projects focused on the Fort Lauderdale Hollywood International Airport internal drainage and storage improvements considered a tertiary drainage system and not explicitly modeled due to the small scale. The airport is located downstream of our study area and although they did not result in modifications to the model, the projects are listed here for awareness of consideration:



- FLL Ravenwood System Interconnection
- FLL System 1 Basin A Stormwater Interconnection Project
- FLL System 1 Basin B Stormwater Interconnection Project
- FLL Edgewood Park Stormwater Project

Tidal Structure Operations/ Canal management: For the design events in the study, the tidal structures were operated in low range operational mode prior to the start of the three-day (72 hr) rainfall event. Low range operation of the managed canal upstream of the structures was as defined in the SFWMD Water Control Operations Atlas (SFWMD, 2021). In addition to the low operational range established in the simulations, salinity rules were also applied to the structures. The salinity rules use a minimum head differential of 0.3 feet before a gate can be automatically opened by the gate control rules, and a value of 0.1 ft head differential is required for the rules to close the gate.

Initial Surface Water Elevation (1D Network): The existing baseline initial surface water elevations in the 1D network were based on the current conditions model which would apply normal operating range elevations to canal reaches and control elevations to water bodies within the study area. Operation of the system prior to the onset of rainfall would bring the canal reaches to the low operating range set by the rules in the model. This allowed for the model to determine the elevations prior to the event based on rules and downstream conditions.

Tidal Conditions: Tidal data with surge was implemented at the tidal boundary utilizing Coastal Hazard System (CHS) save point data with tidal signals developed as documented in USACE C&SF216 Hydrographs Metadata (USACE, September, 2023). ECB 2035 uses sea level change Intermediate Projection only. Tidal hydrographs are modified temporally to yield coincident peaks of rainfall and surge; sensitivity tests performed showed that coincident peaks of rainfall intensity and peak tailwater stages have the highest impact on stages and provide the most conservative case scenario for flooding depth and inundation extent.

Groundwater Initial Conditions: Average wet season groundwater assumptions were the goal of the project. Future conditions average wet season groundwater elevation maps from Broward County FPLOS model (Taylor Engineering, Inc. with Anclote Consulting PLLC, 2021) were the basis for the Broward Resilience model. These maps were utilized for the C&SF Section 203 Study and adjusted for USACE projected sea level conditions for 2035.

Future Without Project Model Set Up

Western Boundaries: (same as Baseline Model Set Up (ECB) and Appendix A-2.1.A)

Land Use: (same as ECB and Appendix A-2.1.A)

Future Projects: (same as ECB and Appendix A-2.1.A)

Tidal Structure Operations/ Canal management: As described in ECB set up, the tidal structures were operated in low range operational mode prior to the start of the three-day

(72 hr) rainfall event. Low range operation of the managed canal upstream of the structures was as defined in the SFWMD Water Control Operations Atlas (SFWMD, 2021). The difference is that for some canals, future sea level conditions in downstream reaches are too high to allow the tidal structures to maintain low range. The downstream water level may be higher than low range and not able to discharge by gravity. The initial conditions in the upstream canals are set to reflect the higher canal levels and the model adjusts to appropriate levels based on operations in the first twenty four hours of the simulation. Salinity rules were also applied to the structures as in the ECB set up. The salinity rules use a minimum head differential of 0.3 feet before a gate can be automatically opened by the gate control rules, and a value of 0.1 ft head differential is required for the rules to close the gate.

Initial Surface Water Elevation (1D Network): The future condition initial surface water elevations in the 1D network were based on the baseline conditions model but updated based on sea level conditions. For instance, if the average low tide under the various sea level rise scenarios was higher than the upstream initial water levels, then the initial water levels were increased. Essentially, it is impossible for gravity structures to discharge when the tailwater is always higher than the headwater, therefore, the initial headwater was set to approximate mean sea level on the tailwater side. With the updated initial conditions, the process would be the same as in the baseline - operation of the system prior to the onset of rainfall would bring the canal reaches to the operating range set by the rules in the model. The process, however, would result in higher stages in the canals as sea level increases above the low range. In other words, the low range operation would not be achieved with gravity structures for many of the canal reaches. The process allowed for the model to determine the elevations prior to the event based on rules and downstream conditions, and would naturally account for the increase in canal elevations based on system constraints.

Tidal Conditions: Determination of boundary tidal conditions were as described under Baseline Model Setup using the same CHS data sampling locations and NOAA tide gauge. The hydrographs were adjusted appropriately for each sea level condition modeled in 2085 - low, intermediate and high. (USACE, September, 2023)

Groundwater Initial Conditions: Average wet season groundwater assumptions were the goal of the project. Future conditions average wet season groundwater elevation maps from Broward County FPLOS model (Taylor Engineering, Inc. with Anclote Consulting PLLC, 2021) were the basis for the Broward Resilience model. These maps were utilized for the C&SF Section 203 Study and adjusted for USACE projected sea level conditions at 2085l, 2085i, and 2085h. The methodology used to adjust the groundwater initial condition files is documented in (SFWMD, Hazen and Sawyer, GW_InitialPotentialHead_Preparation, 2023). Refer to Table 1 in Page 4 of the above mentioned technical memorandum. The initial condition groundwater files are different for the three sea level conditions (2085l, 2085i, and 2085h).

Preliminary Reach A Testing and Initial Array

The compiled ideas for improving the water management system and initial testing results were combined into an “initial array” of alternatives to explore the performance of these



options. The initial array used a subset of the design events for assessment which were the 2s25r85i, 20s25r85i, and 20s25r85h events, and consisted of five modeled alternatives with specific features considered for the Final Array. Initial array alternatives modeled were:

Alternative 1: Gravity Conveyance Improvements to both the primary canal and tidal structures. This consisted of increasing tidal structure spillway widths or adding additional gated spillways, and also improving the conveyance of the canals which deliver the water to the structure. It should be noted that for conveyance improvements, the method was to decrease the roughness (Manning's n) in the canals to conceptually improve conveyance. At this testing phase, the cross sections of canals were not modified. The alternative included tie-back floodwalls and structure hardening for high tailwater conditions.

Alternative 2: Adding Pumps at tidal structures to discharge when tailwater is greater than headwater. For this alternative, the pumps were added to structures along with the gravity improvements. This alternative also included tie-back floodwalls and structure hardening for high tailwater conditions

Alternative 3: Consisted of removing all of the Coastal Structures and S37B for testing of 'what if' scenario of unmanaged canals.

Alternative 4: Relocating structures or adding auxiliary structures was modeled for this alternative. Relocation of structures downstream where topography would allow and combining with larger structures to handle additional runoff was modeled for all but two of the structures. Moving the structures further downstream served to protect more infrastructure from rising sea level and tidal influences during surge. For G54 and S33, an auxiliary structure was added downstream to manage tailwater at the existing tidal structure.

Alternative 5: Alternatives to Discharging East was explored in this alternative by adding storage features to the drainage basins or looking at interbasin connections. Features include:

- Hillsboro: Site 1 impoundment was operated per phase II which consisted of operating the 600 cfs pump during peak rain.
- C-14: The G65 structure which diverts C-14 West discharge from Pompano Canal south through the C-14 East watershed was open to move additional flood water through Pompano.
- Inter basin flow to/from the C-13 canal and North New River Canal in the western part of the basins was allowed by opening S-125 structure.
- Open the S13AW structure to connect C-11 East and West was allowed.
- Operation of the C-11 impoundment was modified to utilize the storage early in the event.

Alternatives were assessed by looking at changes in maximum flood depths in both overland gridded output and primary canal peak profiles. Structure performance was also reviewed as time series of structure flow, headwater and tailwater. Due to the desired

outcome of a positive benefit cost ratio, flood depth improvements were weighted heavily when evaluating features of the initial array. A formal process of evaluation was not performed on the initial array but the Alternative 2 showed promising results and was refined further and used as a test case for the economic tool. Examples of model results for the initial array are provided in Figure 14 through Figure 22 for various alternatives. The refinements to Alternative 2 which included pump testing are shown in Figure 23 and Table 5.

Figure 14 presents peak profile of Hillsboro Canal for the 20s25r85i event for all the initial array alternatives and shows how gravity improvements can lower the peak canal profile. Figure 15 illustrates how the increase in gravity at the structure and in the canal upstream and downstream can improve peak headwater and increase flow at the G56 structure. This is an example of a drainage basin (Hillsboro) which can be improved with gravity improvements. Not all drainage basins showed improvements due to some having more constraints at the structure such as low operating range of the canal with respect to tidal water elevation.

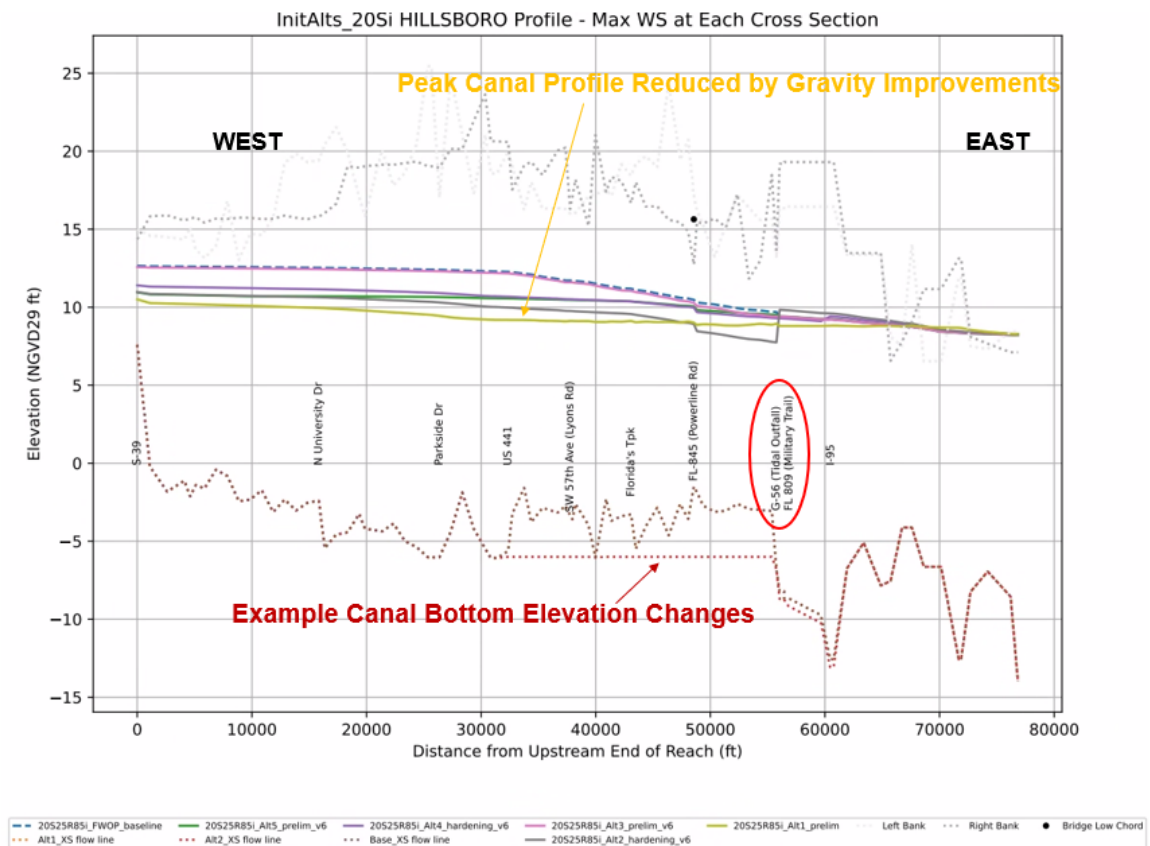


Figure 14 Initial Array Alt 1 Example Results at Hillsboro Canal

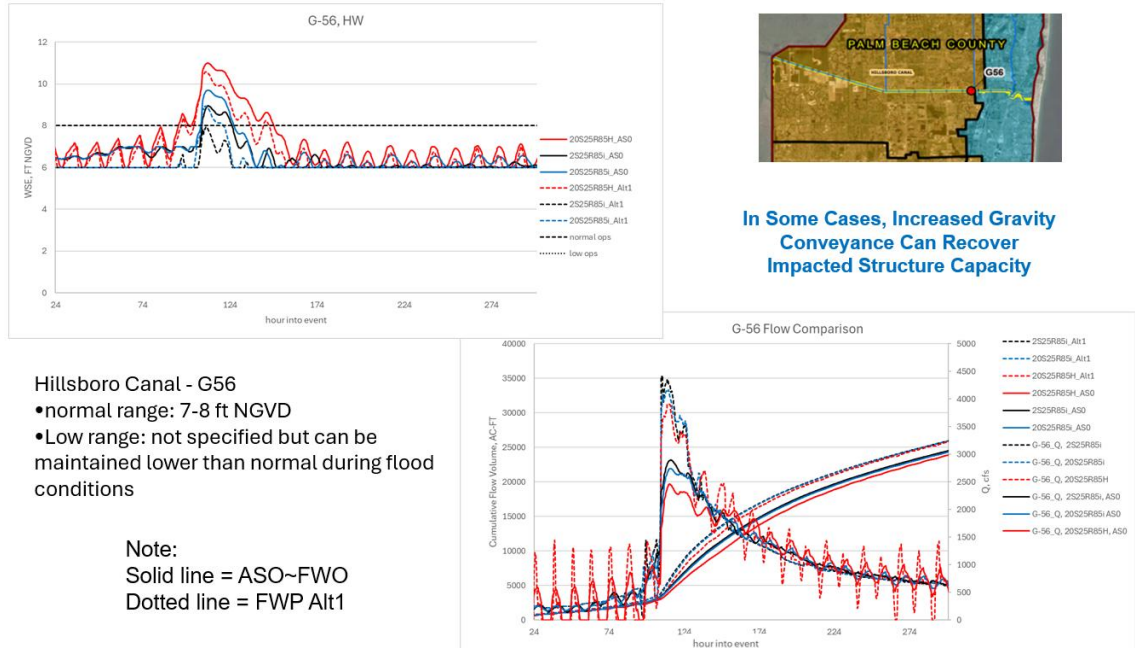


Figure 15 Initial Array Alt 1 Example Results- G56 Structure

Figure 16 is an example of a drainage basin which saw more improvement in peak canal profile with a pumped system due to the limitation to operate a gravity system under high downstream water elevation conditions.

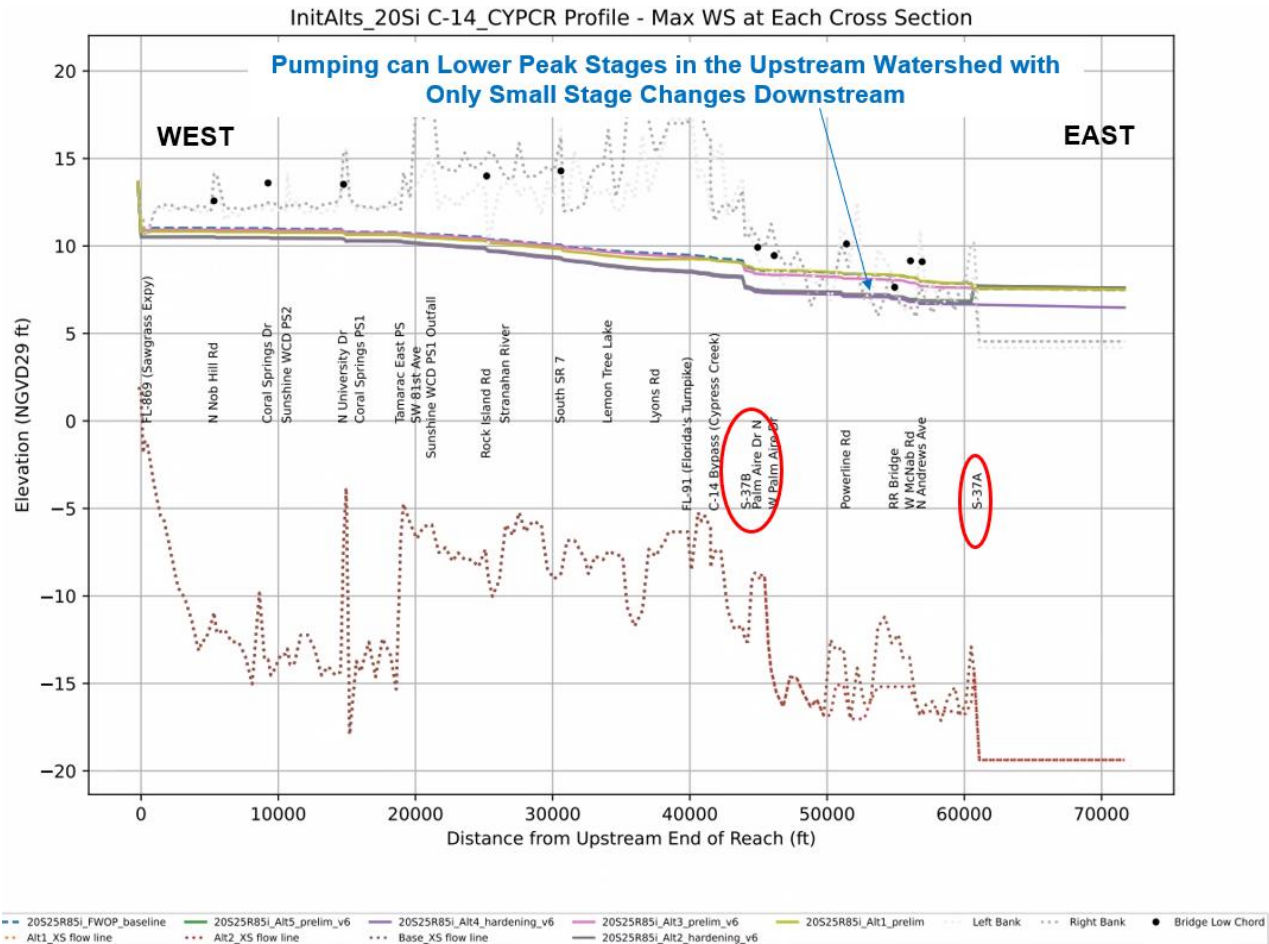


Figure 16 Initial Array Alt 2 Example Results

In order to determine whether the improvement in a canal translates into improvement in flood depths, the flood depth difference maps are created which present maximum depth for FWOP- maximum depth for FWP. Figure 17 presents the flood depth difference map for the C-14E watershed which the S37A structure serves and shows the improvement with the Alt 2 pumping. For flood depth improvement, a positive difference is desired which indicates the economic analysis can achieve positive benefits. Flood depth difference maps are shown in the report to present relative improvements between before and after project features within watersheds. They are shown for comparative purposes and are presented without high resolution background maps. In Section 4.0 with the presentation of results of the Alt RO, maps with high resolution background images are provided for select events in Appendix A-2.1.B. The Appendix A-2.1.B also has locations (ftp site) to additional difference maps for other events and individual watersheds for Alt RO.

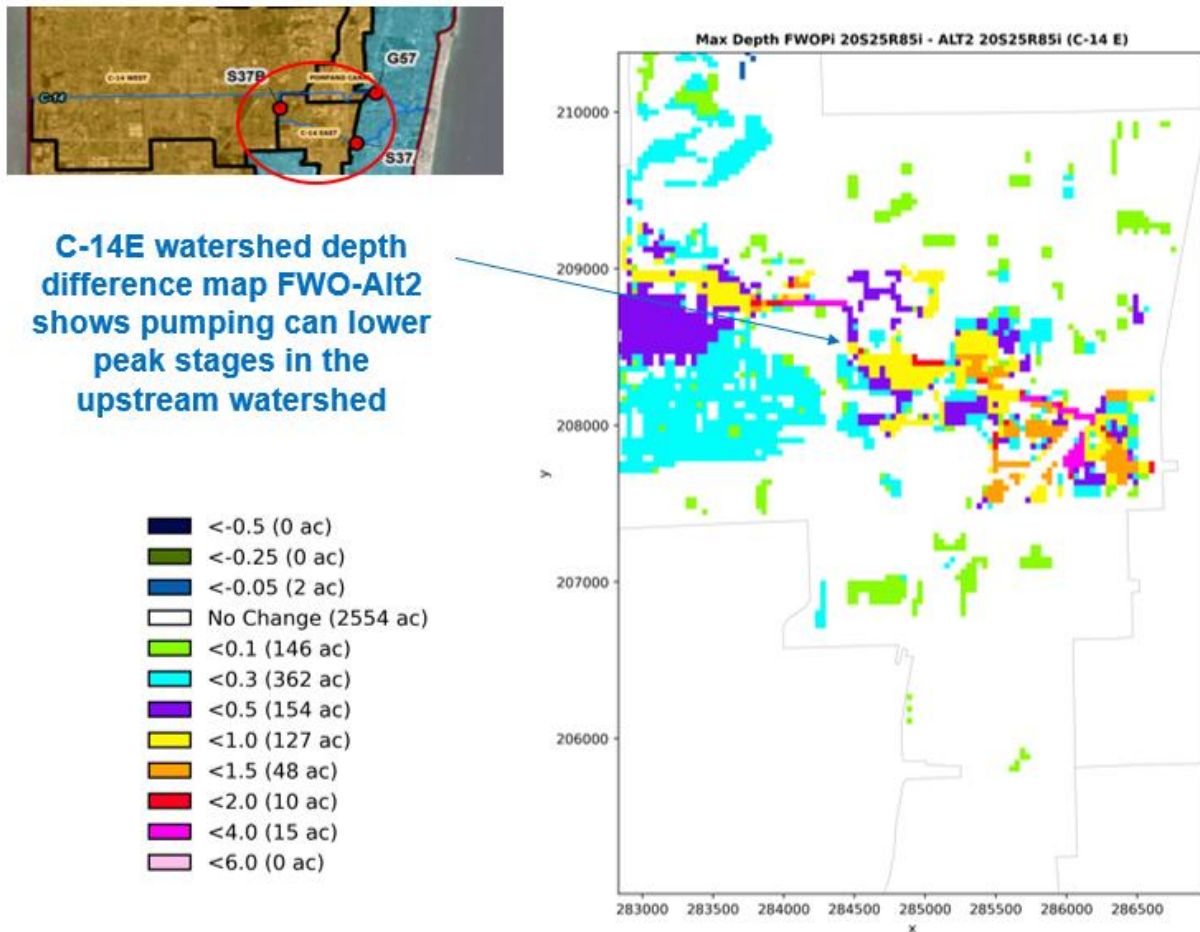


Figure 17 Initial Array Alt 2 Example Results in C-14E Watershed- map of difference in ft.

Removal of tidal structures of Alternative 3 can allow downstream water to reach the upstream watershed as shown in Figure 18. These results were evident in the C-11 canal due to the low operational range of the managed canal which cannot be maintained without the spillway/pump system. The divide structure, S13AW was not removed so the influence was not seen in the western part of the C-11.

Other drainage basins did not show as dramatic of results; however, the tidal structures' purpose of not over-draining during dry months and preventing saltwater intrusion eliminated the alternative from moving forward as a viable option.

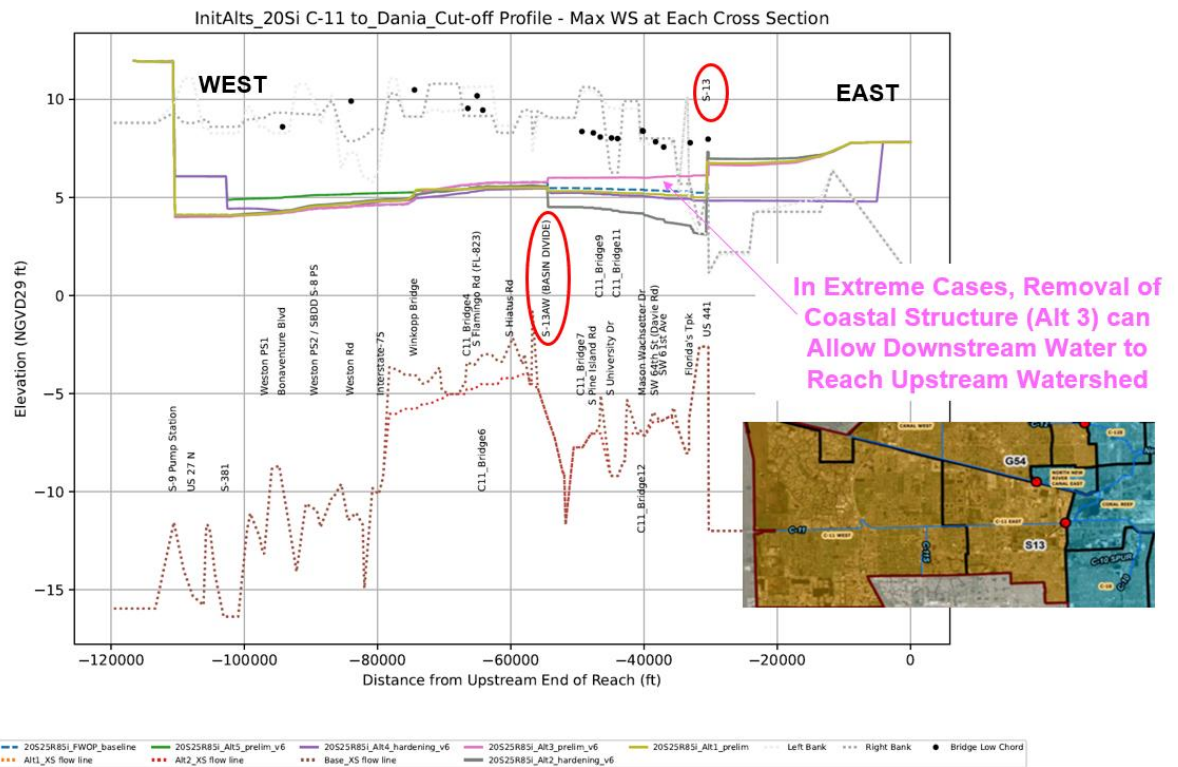


Figure 18 Initial Array Alt 3 Example Results

Using the C-11/S13 canal as an example again shows the effect of relocating or adding auxiliary structures as modeled in Alternative 4. In this alternative, the S13 structure was relocated to the Dania Cut-Off Canal downstream of the confluence with the C-10 Canal as shown in Figure 19. The hydrologic connection between the C-11 Canal and North New River Canal was also cut off to stop any influx from the North New River.



Figure 19 Alternative 4 Modeled Schematic at C-11/S13



Relocating the structure downstream lowered stages across a larger area including C-10 canal and watershed and would allow improved flooding but also causes questions to navigational concerns and waterfront property evaluations. The simulation was run and made available to the PDT but was not considered for the final array or TSP / Recommended Plan. Figure 20 shows reduced stages to the East in the Dania Cut Off portion of the canal and Figure 21 illustrates the improved flood depths in downstream watersheds not currently managed by the structure.

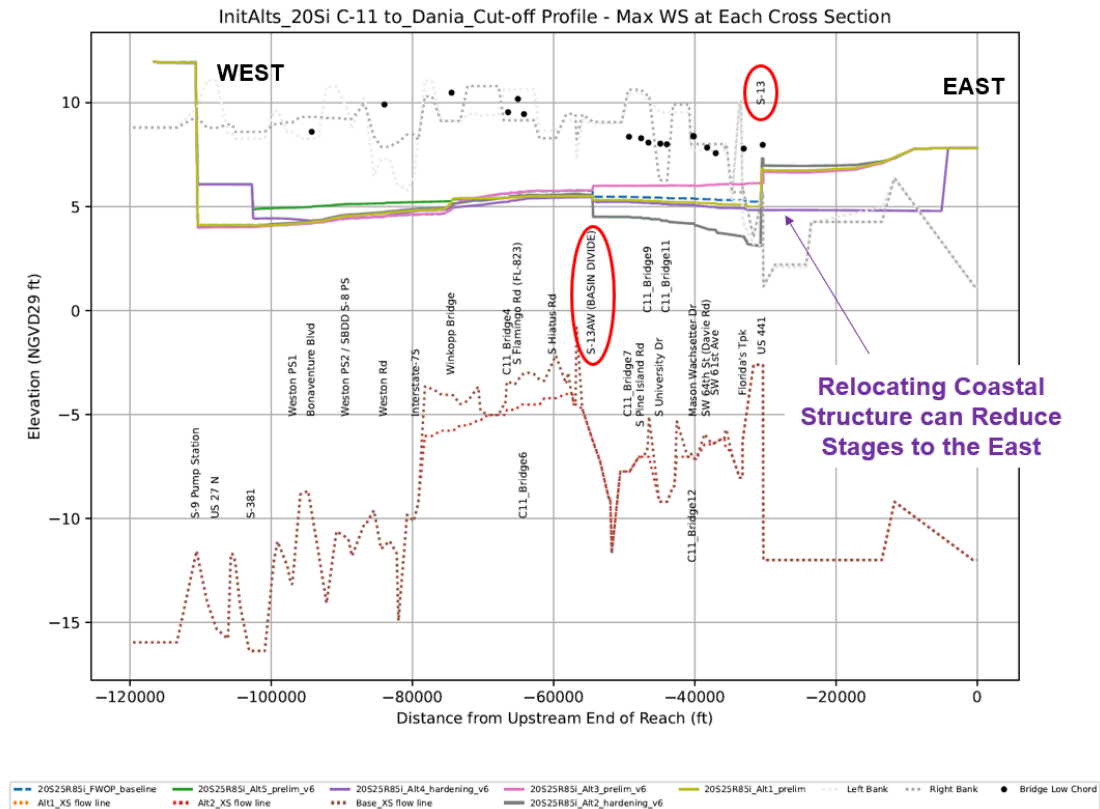


Figure 20 Initial Array Alt4 Example Results- peak profile

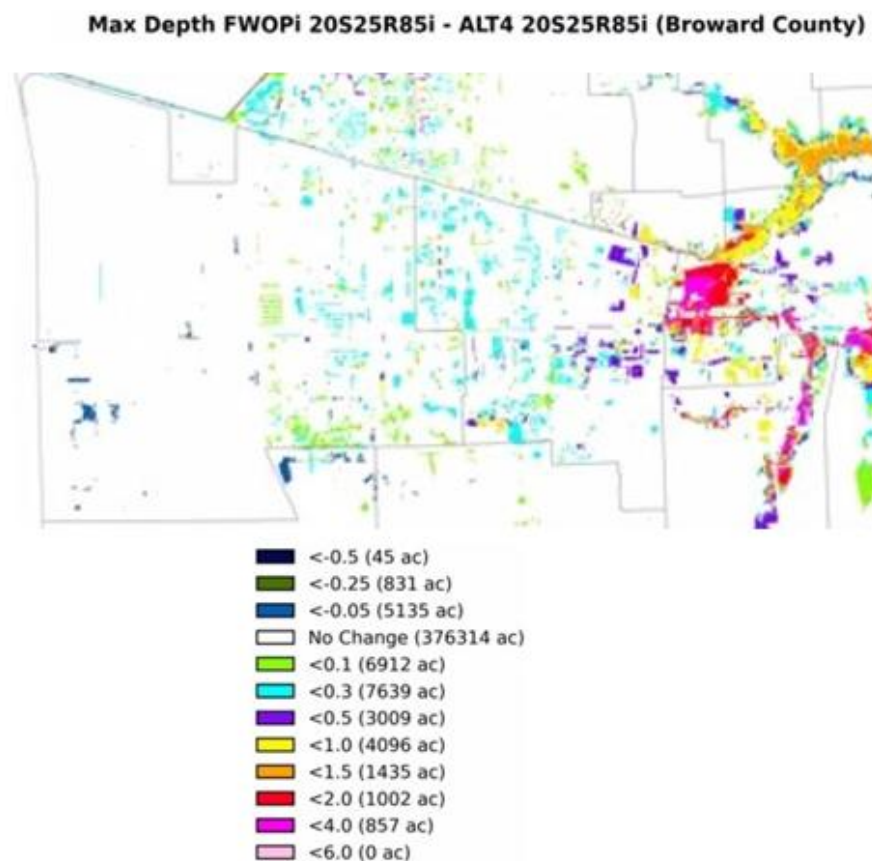


Figure 21 Initial Array Alt4 Example Results- map of difference in ft.

Features in Alternative 5, which were interbasin transfers and storage utilization had some level of improvement but most were not viable to move forward. The Site 1 impoundment operations did show some improvements in flood depths but is not authorized and screened out from moving forward to the final array. Changes in the operation of the C-11 impoundment was not effective and didn't move forward to the final array. Opening the S13AW structure and inter basin flow between the C-13 Canal and North New River at S125 which was tested in this alternative did not improve flood depths appreciably to justify further study. The use of the G65 structure to divert water through the Pompano canal was included in final array Alternative C as a way to improve the conditions in C-14E and at the S37A structure. Figure 22 presents example results for the 2S25R85i event.

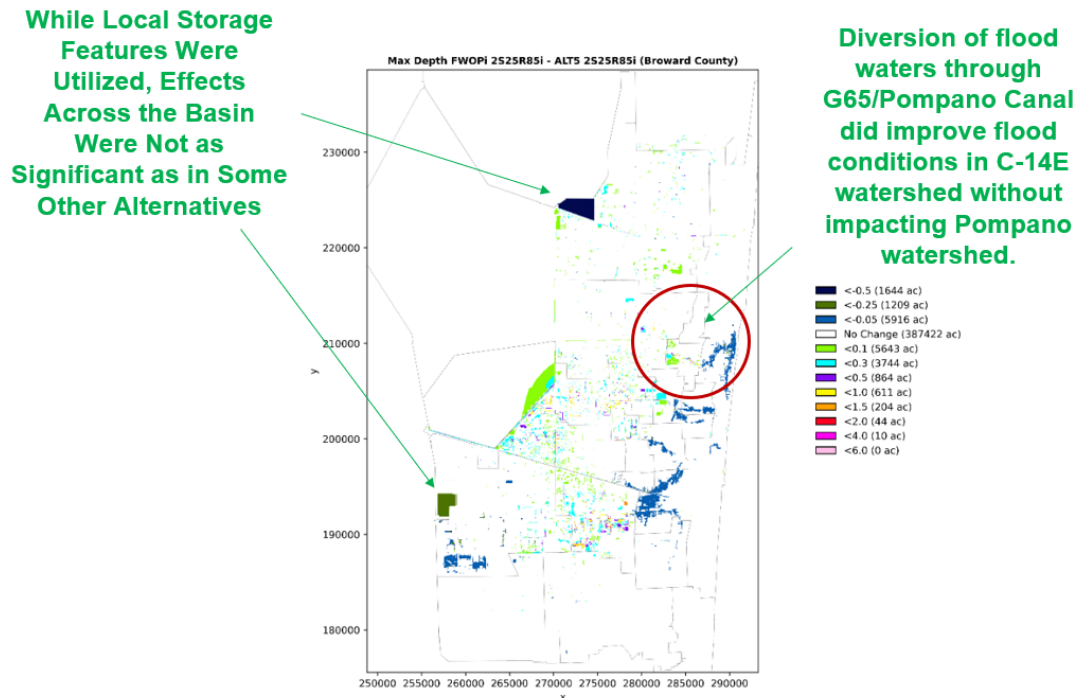


Figure 22 Initial Array Alt 5 Example Results – map of difference in ft.

Further testing using the pump Alternative 2 was performed to evaluate pre-storm drawdown, pump sizes, and sensitivity of the primary system to improved secondary drainage. Pre-storm drawdown is practiced in the field prior to a forecasted heavy rain event to create storage in the primary canals and the secondary system. Discussions with operations indicated lowering the canals to 0.5' below the low range can be initiated to help alleviate flooding during expected high intensity rain events. This was evaluated for use as an operational feature and found to be effective in decreasing peak stages in the watersheds.

Recognizing that with rising sea levels and a gravity system, the tidal structure may not be able to maintain low range in the canals nor lower, this operation was evaluated using the pump Alternative 2 and further used to help determine appropriate pump sizes for effective flood control. For pump size testing, the modeling team looked at flood depth maps (comparing with baseline), maximum water level profiles in the canals, peak tidal structure upstream water level (headwater, HW) differences among runs, and suitability of pump size to meet pre-storm drawdown requirements.

Figure 23 is an example of S37A upstream water elevation with varying pump sizes. The timeseries shows how increasing pump size can draw down the canal level during pre-storm drawdown phase which is occurring between 09-08 and 09-10 in the figure. The improved antecedent conditions can also lower peak stage seen at the headwater of the structure. The pump sizing for this example uses a high sea level condition to test the ability for the sized pumps to be effective with high tail water during the tide signal prior to

the onset of the storm. It may be noticed that the headwater for FWOP (fwop in the legend) remain high since the spillway cannot draw down due to high sea level tailwater conditions. With the pumping, iteration 1 and 2 (same pump size and same time series), and iteration three draw down to low range quickly. Iteration four and greater with higher pump capacities draw down almost immediately and not captured at the fifteen minute output resolution of the time series.

This methodology was used to inform the pump sizing for the Final Array simulations. Table 5 is an example of pump sizing results for each tidal structure but note that the testing was based on one scenario (20S25R85h) and results would vary with additional sea level and event scenarios.

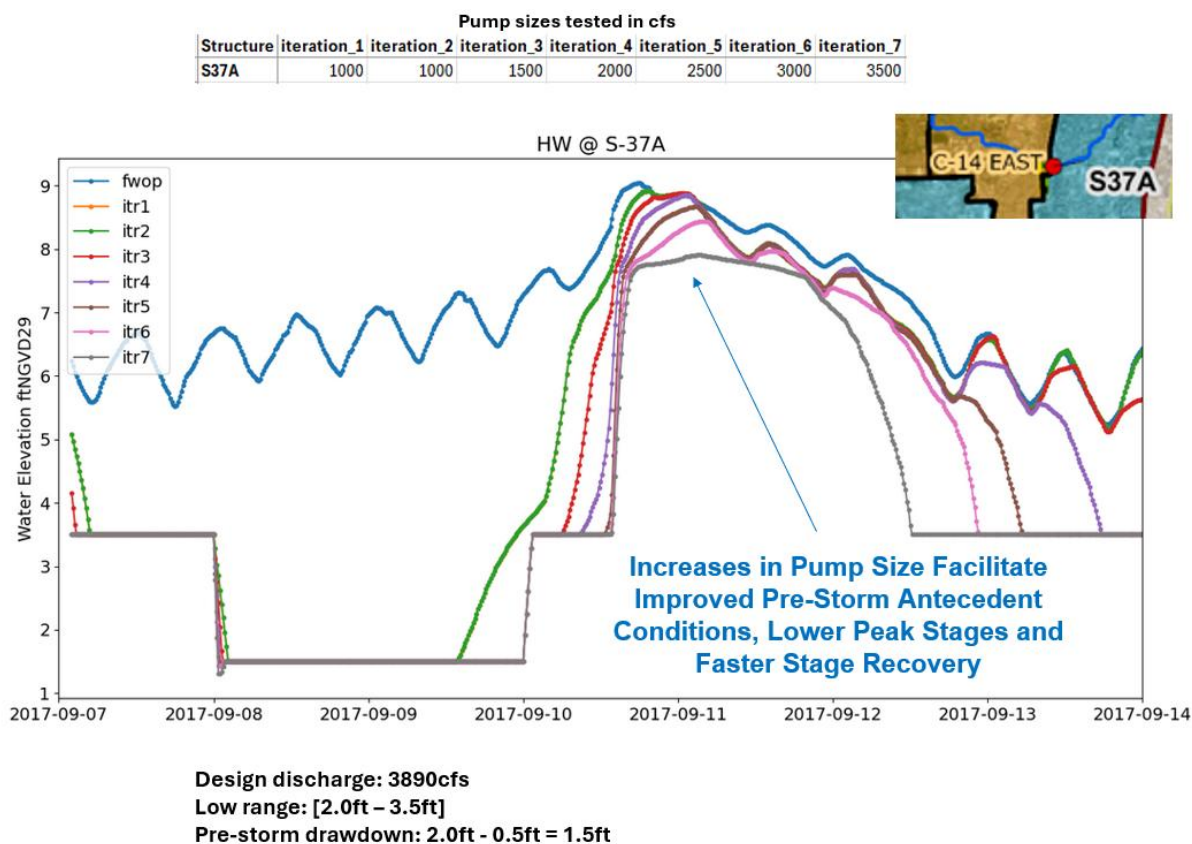


Figure 23 Initial Array Alt 2 Example Pump Sizing



**Table 5 Example Pump Testing and Recommended Sizes (green shade),
20S25R85h event**

Structure	Initial Testing Pump Size [cfs]	Pump Capacity [cfs]						
		iteration_1	iteration_2	iteration_3	iteration_4	iteration_5	iteration_6	iteration_7
G56	3600	0	500	1000	1500	2000	2500	3000
G57	375	0	0	0	0	0	0	0
S37A	4000	1000	1000	1500	2000	2500	3000	3500
S36	1100	0	0	500	500	600	600	800
S33	1000	300	400	500	600	700	800	900
G54	1500	500	500	600	600	800	800	1000
S13	1500	540	540	700	700	800	900	1000

With the improved ability to draw down prior to the storm afforded by increased capacity/pumping on the primary system, the model assumptions in the secondary system were revisited to appropriately improve secondary. While lowering pre-storm levels in the primary canals would naturally affect the secondary canals with an open connection to the primary system, that is not the case for much of the secondary which operates pumps to move water to the primary canals. For these pumped systems, the secondary pumps were also allowed to operate to pump below control prior and during the storm.

Consistent with expected maintained and improved secondary drainage by local concerns and municipalities, the movement of ponded water to the secondary system was assumed to be more efficient by increasing the maximum storage change rate of the ponded water to local depressions or drainage canals. The change was to sub-grid scale drainage features represented conceptually. The change only affects ponded drainage after infiltration and evapotranspiration and overland movement, and the delivery of water through the ponded drainage routine is always limited by head differential between the ponded water and the water body where drained. The change was made to take advantage of the additional capability of the primary system with the improvements.

The pre-storm draw down in the secondary and also the assumed efficiency improvements of the inflow from secondary to primary was carried forward in all of the Final Array of Alternatives in the next section. It should be noted, that the increase in delivery of the secondary water to the primary canal sometimes caused small increases in the peak canal profiles which may be seen in the Final Array results and the Tentatively Selected Plan. (See Figure 40)

Final Array of Alternatives

The Final Array of Alternatives were built in the Mike She/Mike 1D model starting from the FWO simulations. Each alternative contains features shown to be effective in improving flood levels in the seven upstream drainage areas of the study (a drainage area may include multiple watersheds- see study area map). The features consist of primary

structure modifications, primary C&SF Canal improvements, and storage and nature based proposed projects. Primary structure modifications may include increased gravity capacity by adding additional gates and/or pumps, hardening of the structures to block backflow during high tide conditions, and alternative operation of the structure with pre-storm drawdown and, where available, operation of pumps both pre- storm and during the storm to maintain lower canal water levels. It should be noted that operations for pre-storm drawdown also are applied to the secondary system due to improved primary canal stages and structure discharge capability.

Final Array features are listed in Table 6.

Table 6 Final Array of Alternatives Project Features*

Project Type	Project Type Detail	Alternatives		
		A	B	C
Primary Structure Modification	Gates	S-37A (4 x 25 ft) - C-14 East, S-36 (2 x 25 ft) - C-13 West	G-56 (5 x 20 ft) - Hillsboro S-37B (2 x 35 ft) - C-14 West, G-57 (2 x 21 ft) - Pompano, S-33 (2 x 20 ft) - C-12 West, G-54 (5 x 16 ft) - NNR West	G-56 (5 x 20 ft) - Hillsboro, S-37A (4 x 25 ft) - C-14 East, G-57 (2 x 21 ft) - Pompano, S-36 (2 x 25 ft) - C-13 West, S-33 (2 x 20 ft) - C-12 West, G-54 (5 x 16 ft) - NNR West
	Pump	S-37A (add new 1200 Cfs) - C-14 East, S-33 (add new 510 cfs) - C-12 West, S-13 (add'l 160 cfs) - C-11 East	S-37A (add new 1500 Cfs) - C-14 East, S-36 (add new 510 cfs) - C-13 West, S-33 (add new 510 cfs) - C-12 West, S-13 (add'l 540 cfs) - C-11 East	G-56 (add new 1000 cfs) - Hillsboro S-37A (add new 1200 cfs) - C-14 East, G-57 (add new 300 cfs) - Pompano, S-36 (add new 600 cfs) - C-13 West, S-33 (add new 700 cfs) - C-12 West, G-54 (add new 800 cfs) - NNR West, S-13 (add'l 960 cfs) - C-11 East
	Hardening	G-56 (Hillsboro), S-37A (Cypress Creek/C-14), G-57 (Pompano), S-36 (C-13 West), S-33 (C-12 West), G-54 (NNR West), S-13 (C-11 East)	G-56 (Hillsboro), S-37A (Cypress Creek/C-14), G-57 (Pompano), S-36 (C-13 West), S-33 (C-12 West), G-54 (NNR West), S-13 (C-11 East)	G-56 (Hillsboro), S-37A (Cypress Creek/C-14), G-57 (Pompano), S-36 (C-13 West), S-33 (C-12 West), G-54 (NNR West), S-13 (C-11 East)
Primary C&SF Canals	Canal Cross Section Modification		Hillsboro, C-14 West, C-14 East, C-13 West, C-12 West, NNR, C-11 West C-11 East	Hillsboro, C-14 West, C-14 East, C-13 West, C-12 West, NNR, C-11 West C-11 East
	Culverts/Bridges	Triple culvert u/s of G-57 - Pompano Canal	Triple culvert u/s of G-57 - Pompano Canal	G-65 - Pompano Canal, Triple culvert u/s of G-57 - Pompano Canal Winkopp Bridge - C-11 West
Storage & Nature Based			NNR West	Hillsboro, NNR West
Primary Structure Modification	Pre-storm Drawdown	All basins, all alternatives -except C-11 West, C-14 West	All basins, all alternatives - except C-11 West	All basins, all alternatives - except C-11 West
Assumed Inflows from Secondary to Primary	Pre-storm Drawdown	All basins, all alternatives		

*Note: If not listed, the structure retains its current geometry & capacity in the Alternative

Alternative A with the least number of features and lowest cost, included the addition of three pumps (S-33, S-13, and S-37A) and improving gravity capacity (adding additional



gates) at two tidal structures (S-36 and S-37A). During testing and initial array, gravity improvements were shown to have some benefit for the C-13W/S-36 drainage system so gravity improvements (without pump) were included in Alt A. S-33, however, was not shown to improve with initial testing of gravity only so Alt A for S-33 consisted of a pump at the structure. Similarly, initial testing at S-13 showed that increasing gravity capacity was not effective largely due to the low operational range of the upstream canal which could only be realized using a pumped system. Pumping and gravity improvements were incorporated for S-37A due to the improvements in flood depths particularly in the C-14E watershed. Both G56 and G57 show gravity structures can mostly maintain operational range of the upstream canal reaches during normal tide cycles so no pumps were proposed.

Drawdown in the primary canal was initiated two days prior to peak rainfall. Drawdown was allowed 0.5' below the bottom of low range. Depending on the tidal structure configuration, drawdown may or may not be achieved to the desired level. Once the drawdown period is complete, operations are maintained to low range setting for the respective structure.

Drawdown in the secondary system would naturally follow the drawdown of the primary systems for gravity flow into the primary. For pumped secondary system, the pumps were modeled to drawdown 0.5' below control elevations. The secondary operation is applied for the entire simulation period. Consistent with expected maintained and improved secondary drainage by local concerns and municipalities, inflows from the secondary system to the primary was assumed to be more efficient by increasing the maximum storage change rate of the ponded water to local depressions or drainage canals. The change was to sub-grid scale drainage features represented conceptually. The change was made to take advantage of the additional capability of the primary system with the improvements.

Hardening of the structures was included at all tidal structures. Model simulations raised top of gate and structure bypass (platforms/tie-in floodwalls) to accommodate no overtopping for all the design events/SLC conditions. S37B Note: no hardening of S-37B was modeled since it is upstream of S-37A and not a tidal structure- structure changes were made to S-37A to not allow any flow from tide to pass S-37A inland. Figure 24 shows a map of features included in Final Array Alternative A.

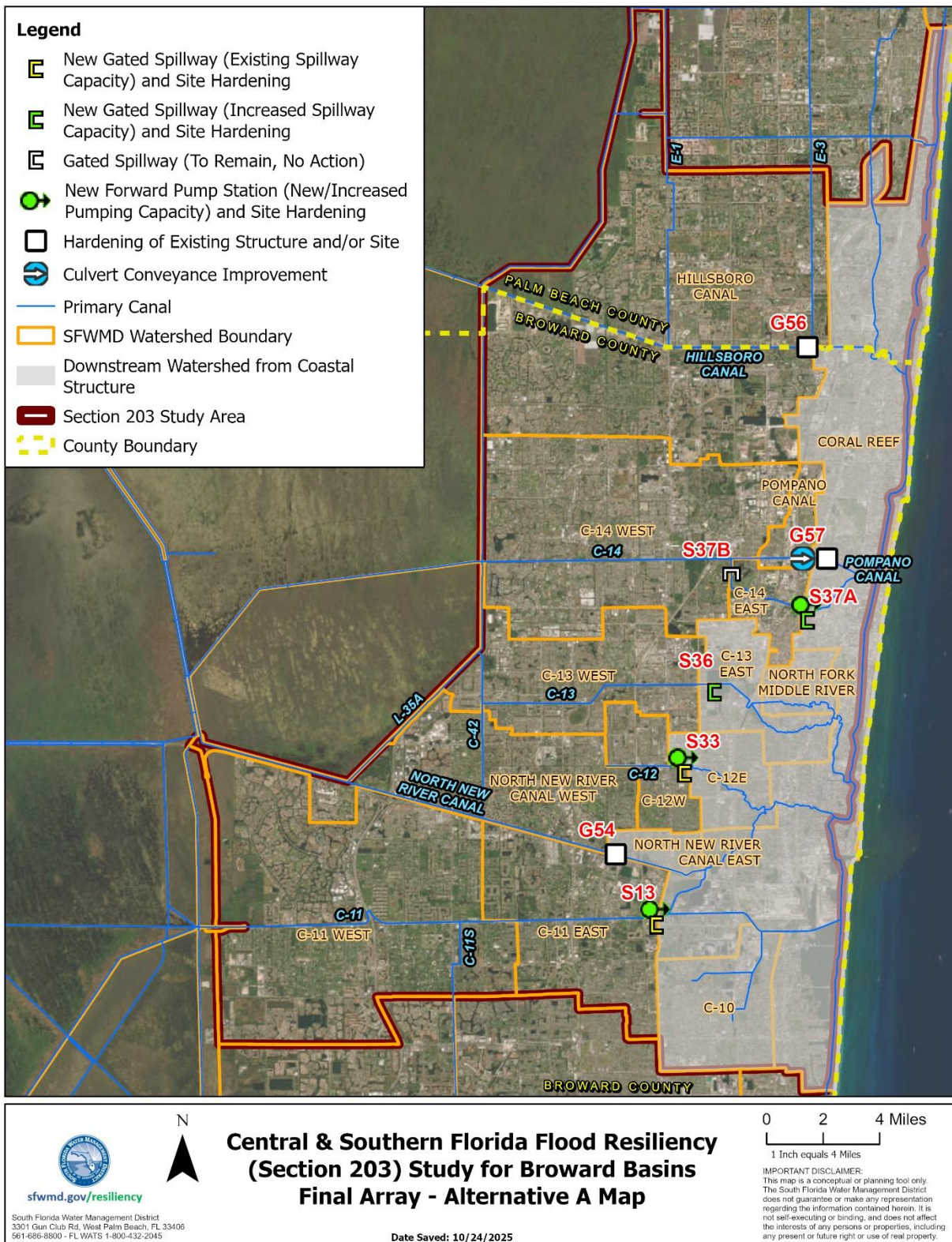


Figure 24 Map of Alternative A Final Array Features



Alternative B continues to build on the increase in pump capacity and gravity capacity and also adds some canal cross section modifications to the scenario. Pump capacity at S-37A is increased from 1,200 to 1,500 cfs and gravity capacity at S-37B upstream of S-37A is improved. Pumping at S-36 is added to Alternative B but assumes existing gate size to assess pump without gravity improvements. S-13 pump capacity is doubled, to 1,080 cfs.

The Alternative also adds additional gravity capacity/conveyance to G-56 structure and Hillsboro Canal, and S-37B structure and C-14 West Canal. G-57 increases gravity capacity at the structure to consider in combination with the upgrade to the underground culvert upstream of the structure. C-11 East cross sections are modified to improve the delivery of water to the S-13 pump which has doubled its capacity and to avoid local drawdown at the headwater and cycling of the pumps. Conveyance improvements were included in C-11 West to move water west to the S-9 and C-11 impoundment more efficiently.

Drawdown of the primary canal and secondary system operations, assumed improvements to inflows from secondary to primary systems, as well as hardening of the structures as described for Alternative A were the same for Alternative B.

Alternative B includes a nature-based storage feature which was identified by the plan formulation team. The feature utilizes Markham Park (MP) located in the western North New River West watershed to store water in ponded areas within the park. The modeling goal was to utilize existing storage as much as possible without impacting structures. To this goal, the modeling was set up to allow filling of MP up to 7.5 ft NGVD which was determined based on the existing Surface Water Management Permit No. 06-00311-S minimum road crown elevation, which was 1.5 ft below the minimum building floor elevation of 9.0 ft NGVD also stated in the permit. A 100 cfs pump was added to the model to move water from the North New River (NNR) canal to the Markham Park. Pump location is shown in Figure 25 and model schematic in Figure 26. Figure 27 presents an example of increased water depth in low lying areas for 2035 With Project conditions.





Operation of Markham Park (MP) as storage during rain event was modeled using the following rules:

- Add 100 cfs pump to move water from NNR to MP
- Add gate operation in place of CU01 (2.5 ft x 2.5 ft)
 - Pump to MP when NNR $\geq$ 5.5 ft
 - Close gate if MP $<$ 7.5 ft
 - Turn off pump & open gate if MP $\geq$ 7.5 ft
- Upgrade/remove 2 culverts upstream (Markham Park_CU03 & Markham Park_CU02)
- Enable overbank spilling

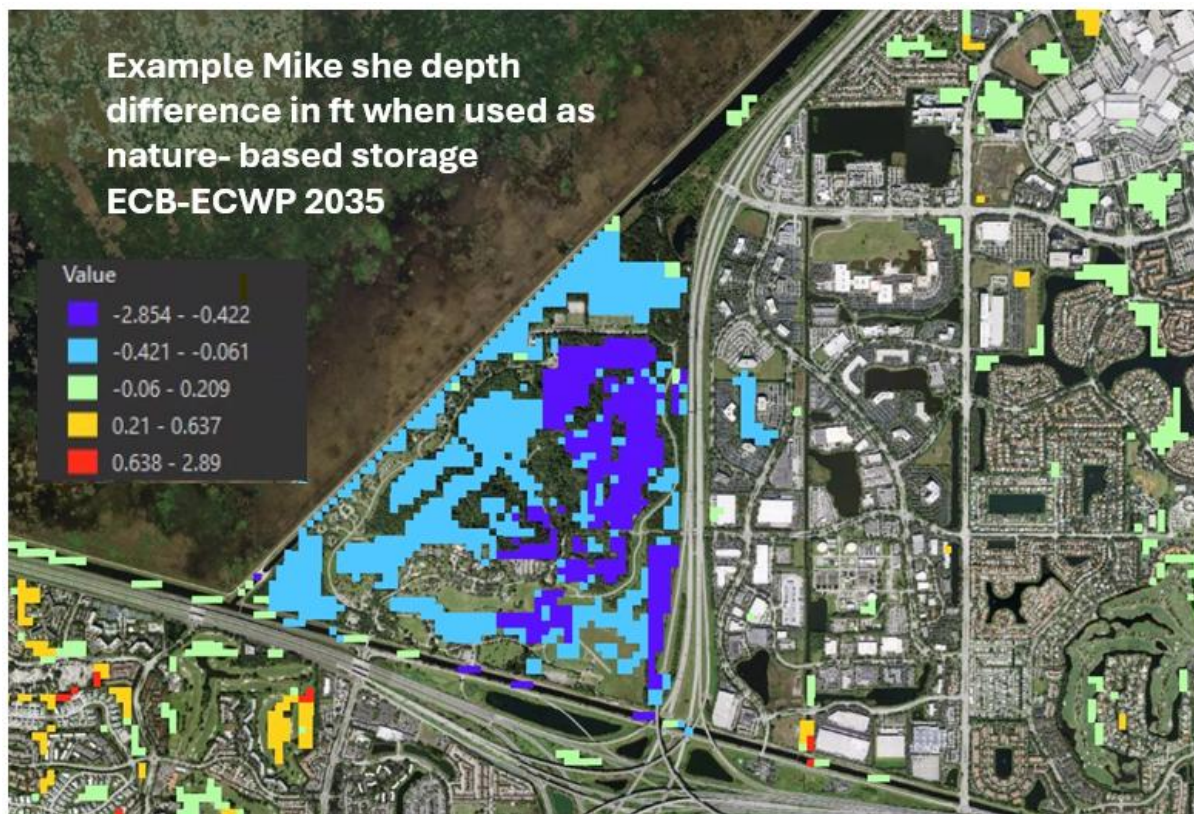


Figure 27 Example of 2035 With Project Conditions

Figure 28- shows a map of features included in Final Array Alternative B.

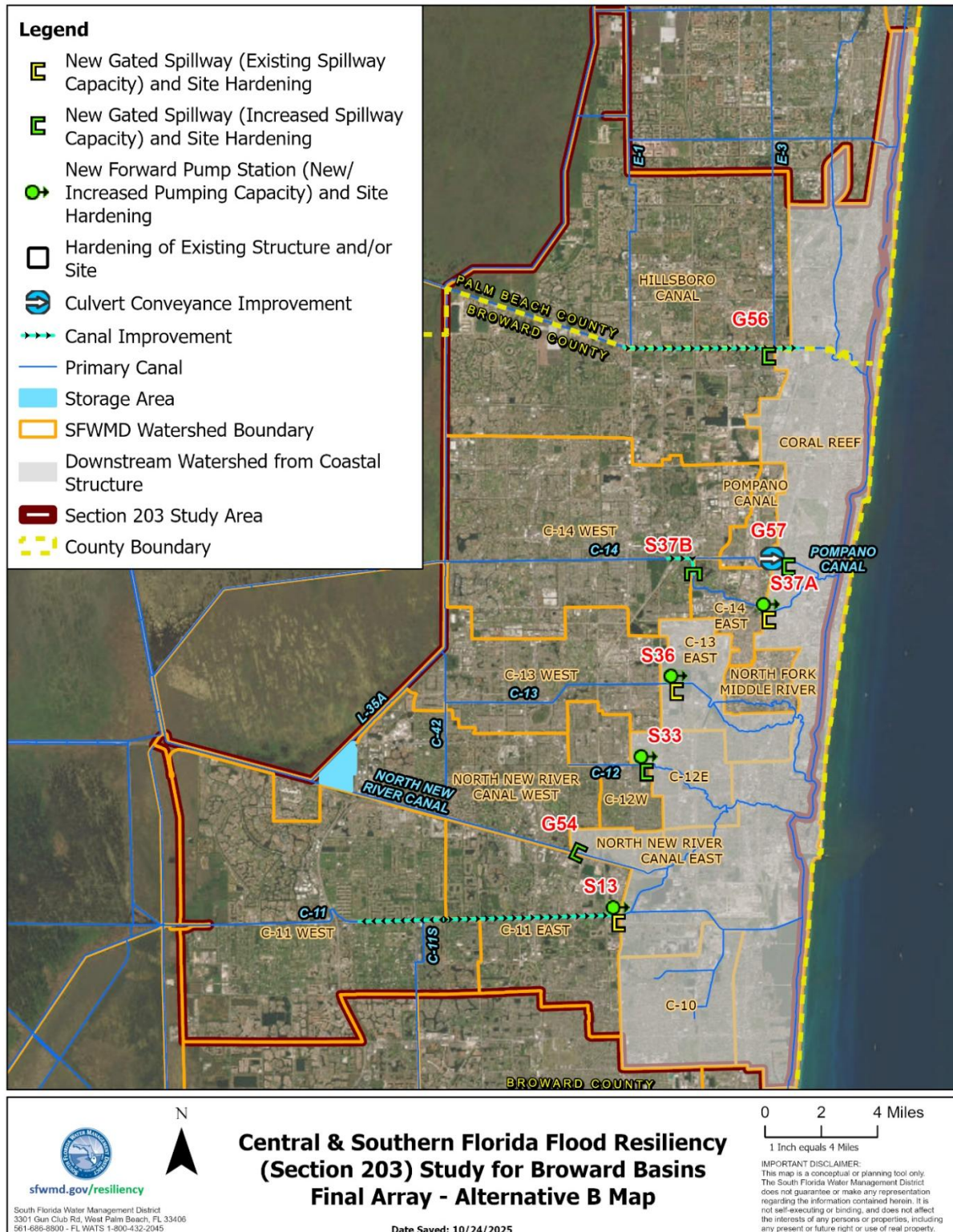


Figure 28 Map of Alternative B Final Array Features



Alternative C combines multiple features in drainage areas. G56 sees an increase in gate size and an added pump along with conveyance improvements in the Hillsboro canal. S-37A also sees both increase in gravity with pump addition of 1,200 cfs. Pumping is maintained at all 7 tidal structures (not at S-37B) and all but S-13 also see an increase in gate width. Alternative C is most likely to show improvement for high sea level more extreme events, but due to multiple features and complexity may not be cost effective.

To alleviate flooding and discharge through C-14 East, a project to open the G65 culvert during flood events was also explored to send more water through the Pompano Canal and out G57. G57 was also fitted with a pump to discharge additional flood water.

Markham Park is kept as a feature in Alternative C and an additional nature-based storage feature is included in the Hillsboro watershed at the Hendrix Farm site located adjacent and south of the G-08 Canal upstream of G56 as shown in Figure 29.

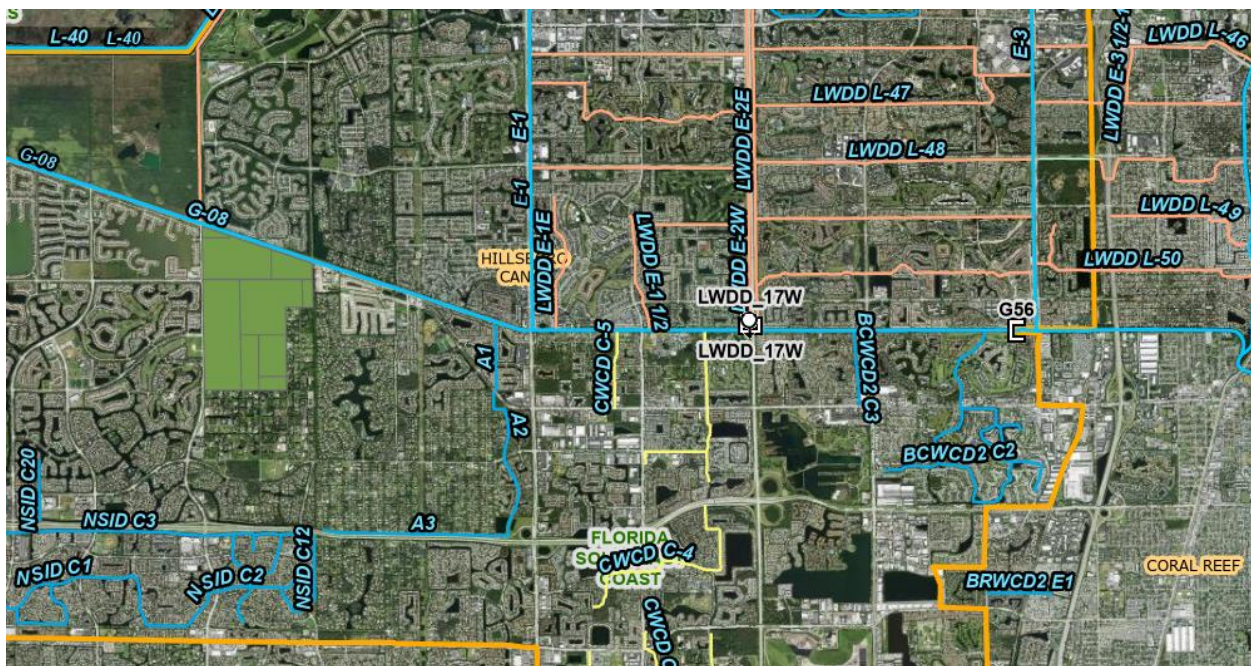


Figure 29 Location of Hendrix Farms nature-based storage in Alt C (green polygon)

For the Hendrix Farms site, the following model changes were performed:

- Topo was re-graded to 9 ft NGVD (above the average groundwater of ~8.1 ft NGVD).
- MIKE 1D branches and cross section were extended to cover the site with 1D features.
- A separated flow area was created of the site to prevent surface flow out of site to surrounding development (conceptual way to assume a berm with seepage canal to capture leakage).
- Flood codes were created of the site to prevent double accounting of storage (storage is accounted for in 1D branch).

The leakage coefficient was set to 0.0 which prohibits seepage from the Hendrix Farms site to the groundwater system which is an oversimplification resulting in a conservative analysis.

This is a conceptual way of representing a storage area with seepage canals and pumps to manage seepage without actually adding complexity to the modeling. Figure 30 presents a map of Alternative C features across the study area.

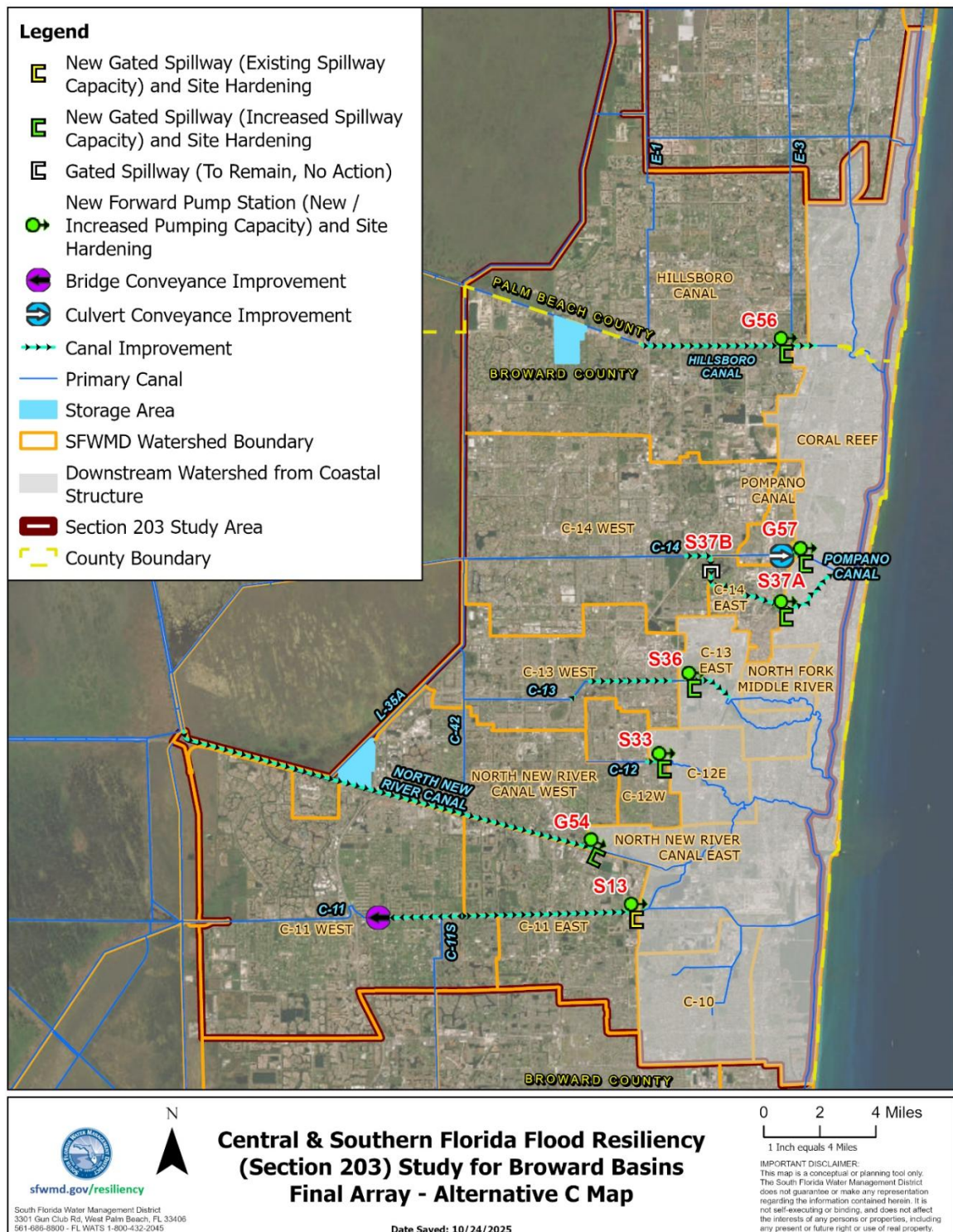


Figure 30 Map of Alternative C Final Array Features



Example results of the final array are shown in Figure 31 and Figure 32 for the C-13 Canal and C-13 West watershed. With the increasingly aggressive features at S36 between Alt A (increased spillway capability shown as red line) and Alt C (600 cfs pump added with increased spillway capability and canal conveyance- shown as orange line), the peak canal profile is lowered in Figure 31 and the depth difference maps show increasingly more areas of improvement for Alt A to Alt C in Figure 32.

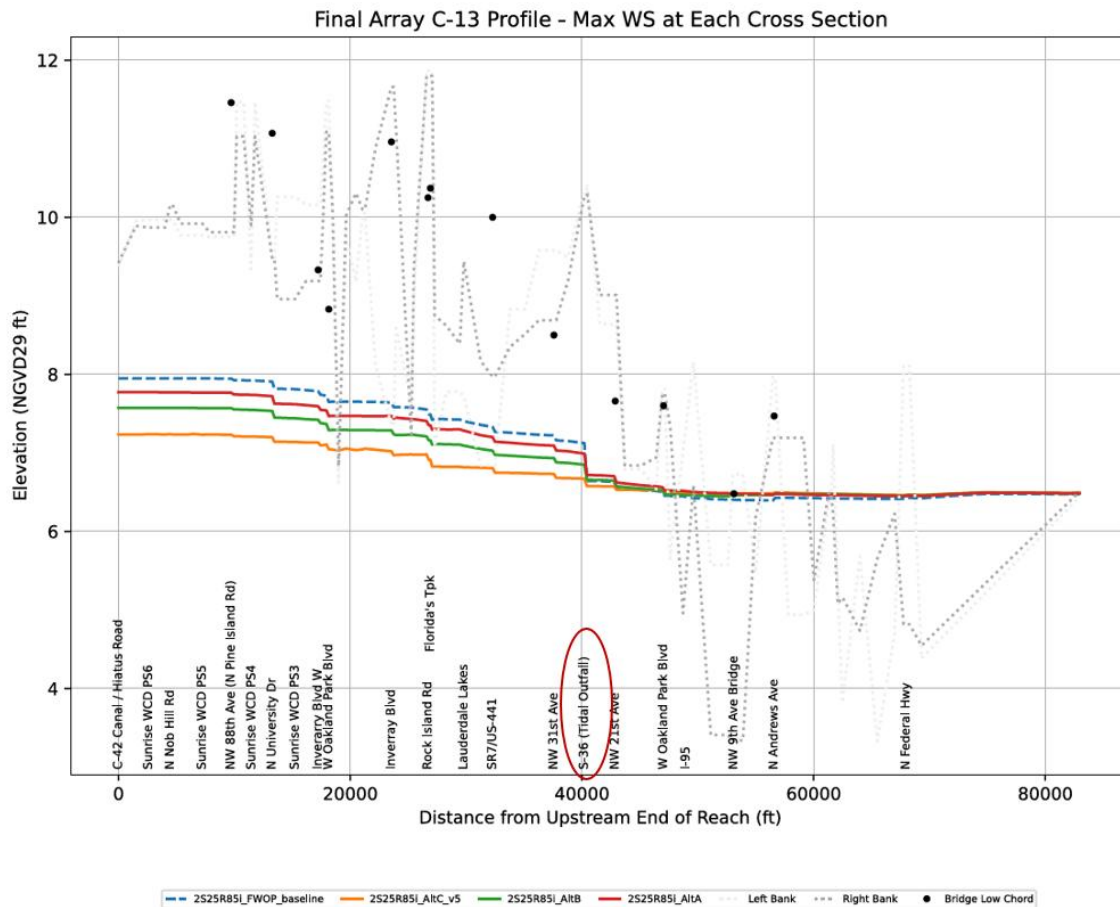


Figure 31 Final Array Example Model Output: Peak Canal Stage Profile

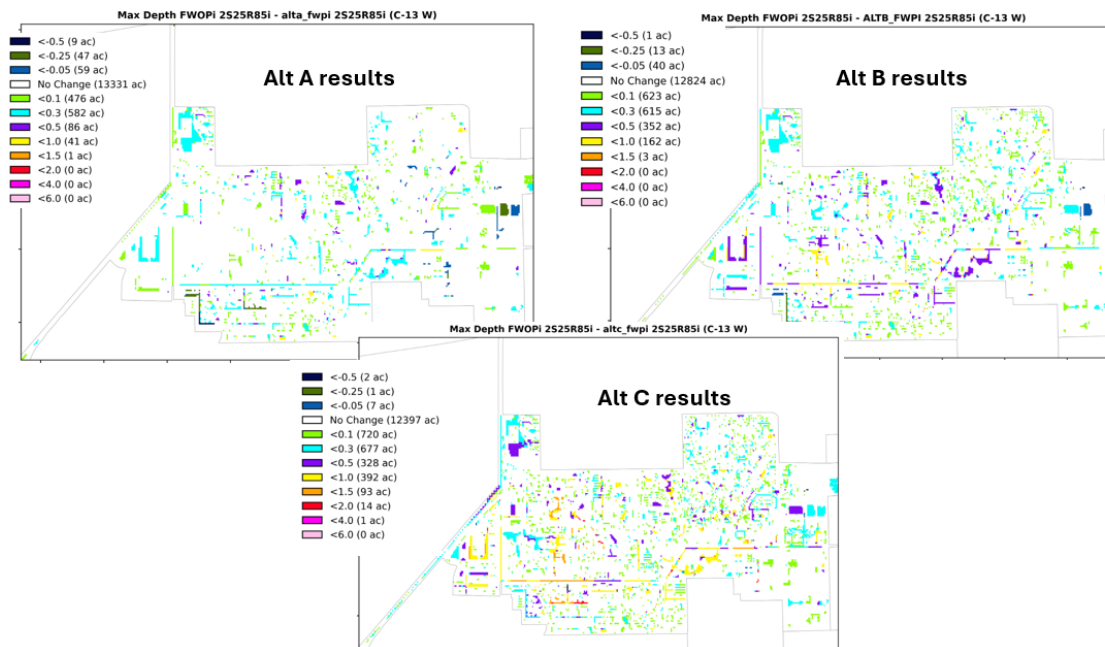


Figure 32 Final Array Example Model Output: Peak Stage Difference Maps in ft.

Model output post processed results were provided to the PDT team to help inform the Resilience Optimized Alternative, or Alt RO. The type of information provided was flood depth maps at the model multi-cell resolution which is at 125' by 125'. These were used by the HEC FDA model to calculate cost benefits of each of the alternatives by drainage basin. Project features were also provided to the design team to determine initial designs and project costs. Modeling output focused more heavily on flood depth difference maps than it did on flood duration maps due to the use of the HEC FDA tool which doesn't have a flood duration component. However, both flood depth maps and flood duration maps were provided to the comprehensive benefits team to perform analysis for each of the three alternatives. This output also helped inform the PDT of the alternative's relative performance. Flood duration difference maps for select events of Alt RO are included in Appendix A-2.1.C with high resolution background maps for the interested reader. The Appendix A-2.1.C also includes an ftp location for additional duration difference maps by watershed.

The PDT team collectively determined features for each drainage basin to be modeled for the Alt RO. The PDT team identified several important considerations when identifying elements of the final alternative, including:

- Comprehensive benefits absolute performance
- Benefit to cost efficiency – i.e. Are the additional benefits worth the additional investment? Are there features that add cost but don't improve performance or resiliency?
- Plan completeness and robustness in the face of uncertain and changing conditions



- Environmental effects
- Downstream effects

Downstream impacts became an important consideration which led to additional testing of strategies to minimize the impacts to downstream caused by the project. Operational refinements were studied for a 'pre-optimized' test case to impose constraints on the operation of the pumps and additional gravity structures during the peak of the storm when downstream water levels were above flood levels. The operation restricts discharge during the peak of the storm, but still allows for pre-storm drawdown and release of water prior to the peak creating additional storage in the primary system.

Model setup included determining a location in the downstream tidal reach where an elevation sensor could be installed and monitored using tidal structure programmed rules in the model. When the tidal reach as determined by the sensor reached flood levels (sensor cut off), the rules would stop pumping until the sensor level dropped back below the sensor cutoff level.

Initially the testing used a downstream sensor cut off consistent with the Broward ordinance for sea wall levels in 2035 (4.0 ft NAVD) and 2050 (5.0 ft NAVD) (Figure 33). In order to not cause impacts downstream by the project, the testing determined that a lower sensor cutoff level was needed.



impermeable water barrier to tidal flooding.

Tidally influenced area means the real property adjacent to, or affected by, a waterway with water level changes in response to the daily tide.

Sec. 39-407. Minimum elevations for coastal infrastructure within tidally influenced areas.

(a) All new or substantially repaired or substantially rehabilitated banks, berms, green-grey infrastructure, seawalls, seawall caps, upland stem walls, or other similar infrastructure shall be designed and constructed to perform as tidal flood barriers. Tidal flood barriers shall have a minimum elevation of five (5) feet NAVD88. Applications for new or substantially repaired or substantially rehabilitated tidal flood barriers submitted prior to January 1, 2035, may be permitted a minimum elevation of four (4) feet NAVD88, if designed and constructed to accommodate a minimum elevation of five (5) feet NAVD88 by January 1, 2050.

Figure 33 Initial 'test' was based on planned minimum sea wall elevations

It should be noted that the modeling was aimed at not increasing flood depths and flood duration downstream due to the project. The project was not aimed at downstream impacts already caused by sea level rise and surge, but rather to minimize or remove worsening of conditions downstream caused by the proposed project features.

Figure 34 is an example of how the model imposed the use of downstream sensor elevations to turn off pumps at S37A. The modified Alt B depth difference map (FWOPi-FWP) in Figure 35 presents improvements seen in the downstream watersheds with the operations for the 20S25R85i conditions. Although these results seem to show desired downstream goals with the initial sensor cutoff test of 6.58 ft, some other events were more sensitive to the pumping. The rainfall with low level surge conditions, such as the 2s25r and 2s100r were more sensitive and determined the ultimate numbers for sensor cut off.

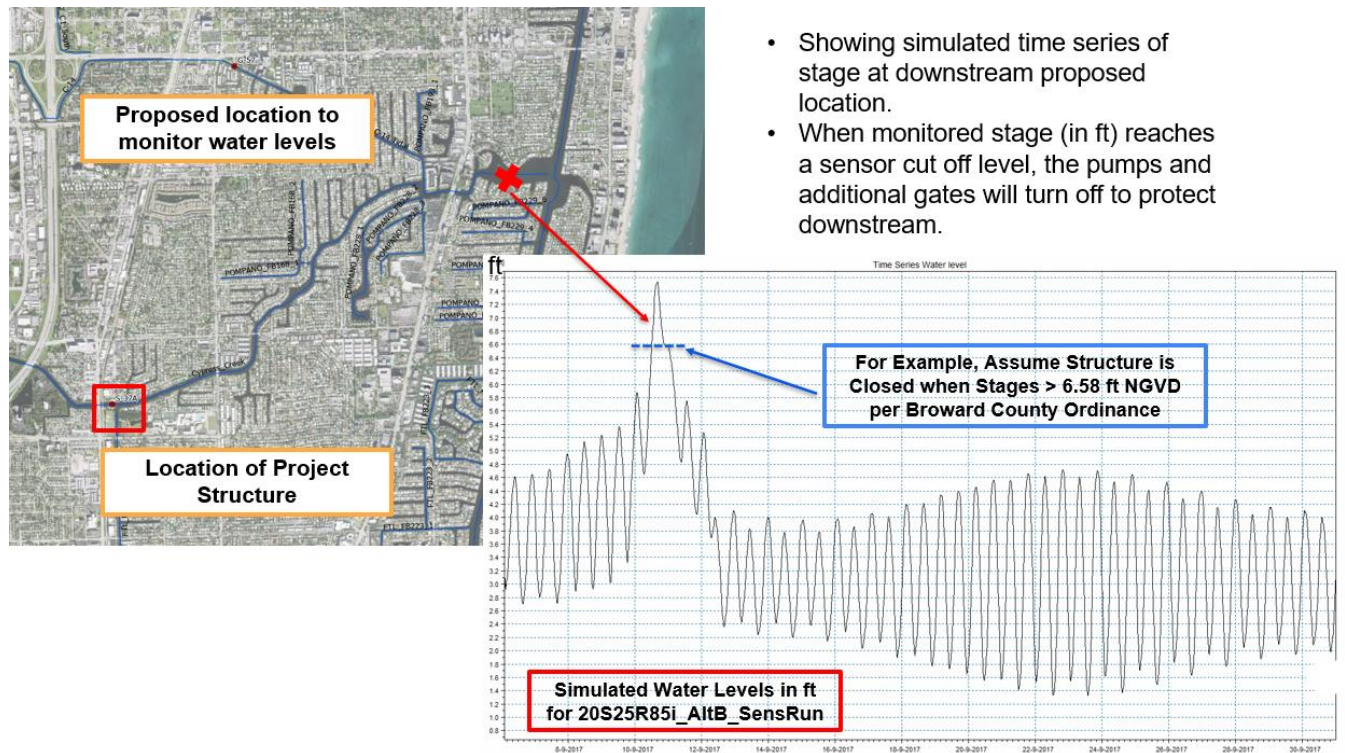


Figure 34 Example Operations Refinement of Selected Plan

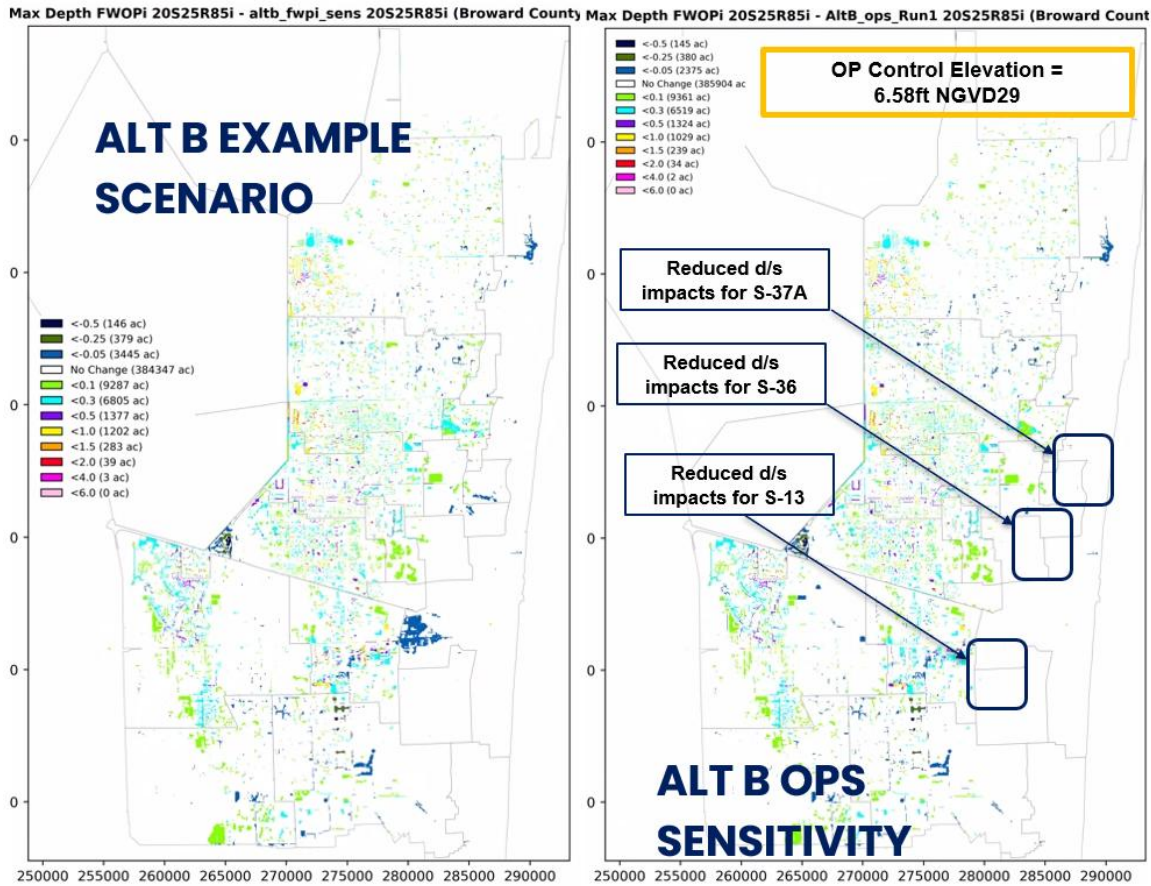


Figure 35 Example Improvement to Downstream with Operation Refinement – difference maps in ft.

Resilience Optimized Alternative (Alt RO)

Following review of the Final Array of Alternatives A, B, and C, a resilience optimized alternative was identified by the project team based on considerations of performance, cost, and implementation. The resilience optimized alternative was based mostly on features of Alternative B, but with some modifications. The following are the modifications:

- The C-14 East watershed discharges through the S-37A structure based on Alternative C features without the C-14 East canal improvements.
- The North New River West discharges through a G54 gated spillway consistent with Alternative B but was modified to also add the 810 cfs pump capacity at this location.
- The Markham Park nature-based storage originally part of the Alternative B in the North New River watershed was not carried forward to the Alt RO and was removed.

- In order to minimize downstream impacts with the addition of increased gated capacity and forward pumps, sensor cutoff rules were imposed for the Alt RO as discussed in the previous section. During the peak of the storm, the sensor elevation was utilized to cut off both pumping and to close any additional gates that were added as a result of the project. Final sensor cut off levels carried forward to the Alt RO are tabulated in Table 7.

Table 7 Sensor Cut Off Levels for Alt RO

Structure	additional gate	pump	sensor cutoff level (ft NGVD)			
	(close add'l gate at sensor cutoff)	(turn off pumps at sensor cutoff)	2035i	2085l	2085i	2085h
S13	none	yes	5.0	5.0	5.5	7.4
G54	1 add'l gate	yes	5.0	5.0	5.5	7.4
S33	1 add'l gate	yes	5.0	5.0	5.5	7.4
S36	none	yes	5.0	5.0	5.5	7.4
S37A	1 add'l gate	yes	5.0	5.0	6.0	7.8
G56	1 add'l gate	no	6.0	6.0	7.0	8.8
S37B	none	no	n/a	n/a	n/a	n/a

The Alt RO features modeled are listed in Table 8 along with Existing features for reference. The map of Alt RO Features is shown in Figure 36. This alternative aligns closely with the final recommended plan although some features were modified during engineering and design decisions presented in the Engineering Appendix (Appendix A).



Table 8 Alt RO Features

Project Type	Project Type Detail	Alternative RO	Existing
Primary Structure Modification	Gates	G-56 (4 x 25 ft) - Hillsboro, S-37A (3 x 25 ft) - C-14 East, G-57 (2x21ft) S-33 (2 x 20 ft) - C-12 West, G-54 (4 x 20 ft) - NNR West, S-36 (2x16), S-13 gate (2x14)	G-56 (3 x 20 ft) - Hillsboro, S-37A (2 x 25 ft) - C-14 East, S-37B (2 x 25 ft) - C-14 West, G-57 (2 x 14 ft) - Pompano, S-33 (1 x 20 ft) - C-12 West, G-54 (3 x 16 ft) - NNR West, S-36 (1 x 25 ft), C-13 West, S-13 (1 x 16 ft)
	Pump	S-37A (add new 1200 cfs) - C-14 East, S-36 (add new 510 cfs) - C-13 West, S-33 (add new 510 cfs) - C-12 West, G-54 (add new 810 cfs) - NNR West, S-13 (add add'l 540 cfs) - C-11 East	S-13 (540 cfs) - C-11 East
	Hardening	G-56 (Hillsboro), S-37A (Cypress Creek/C-14), G-57 (Pompano), S-36 (C-13 West), S-33 (C-12 West), G-54 (NNR West), S-13 (C-11 East)	n/a
Primary C&SF Canals	Canal Cross Section Modification	Hillsboro (AltB) C-11 West (AltB) C-11 East (AltB) C-14 at S37B	n/a
	Culverts/Bridges	Triple culvert u/s of G-57 - Pompano Canal	10' dia RCP, ~ 1474' long
Storage & Nature Based		None	
Secondary Structure Modification	Pump	allow to prestorm drain	
Primary Structure Modification	Pre-storm Drawdown	All basins, all alternatives - except C-11 West	
Assumed Inflows from Secondary to Primary	Pre-storm Drawdown	All upstream basins	
Other		turn off pump & add'l gravity based on tidal reach sensors	

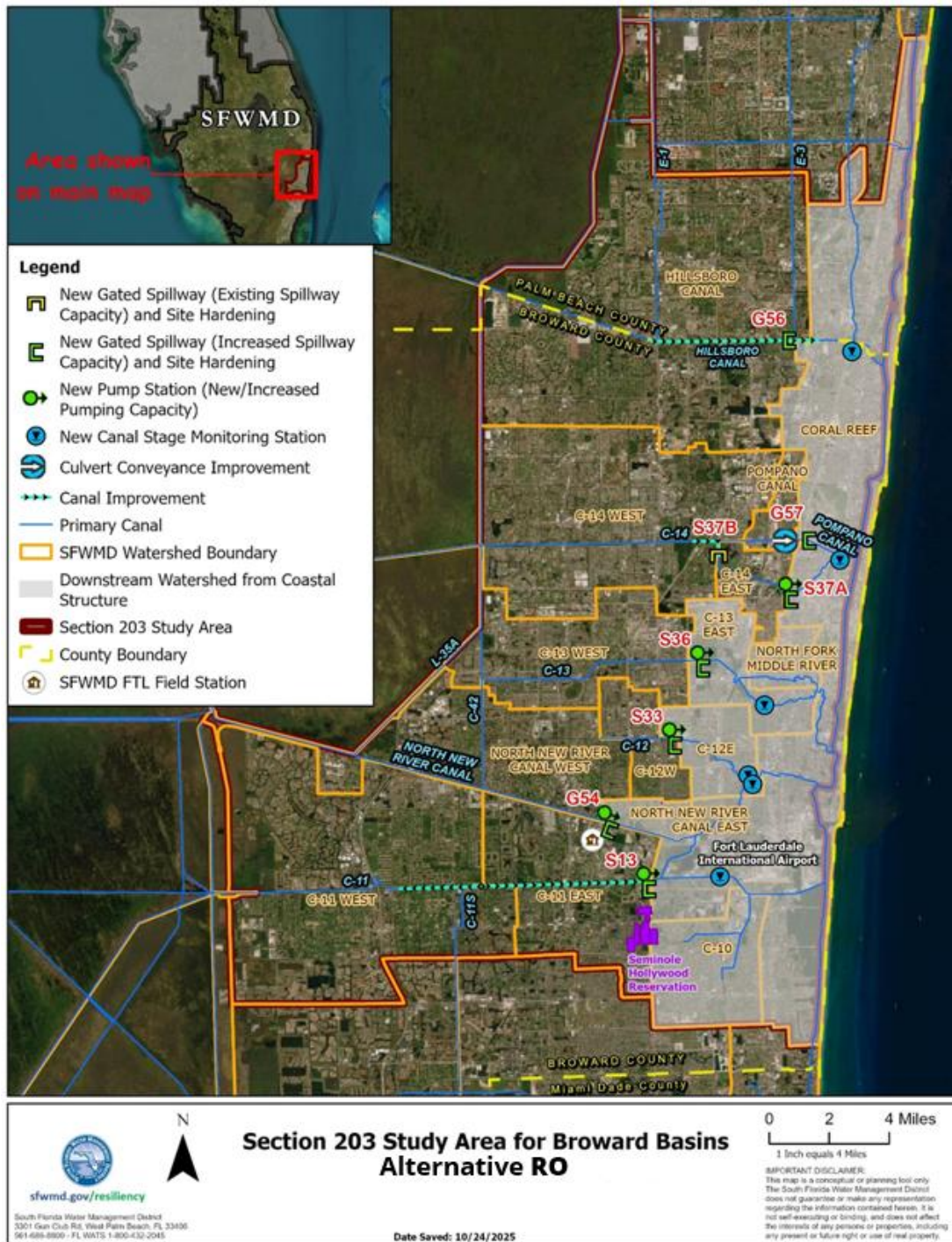


Figure 36 Map of Alt RO Features



4.0 Results

Multiple rounds of modeling and evaluation have culminated in an optimized plan which incorporates best performing features for resiliency. The C&SF Section 203 Study is focused on improving the primary system infrastructure which will be complemented by other efforts at the regional, county or local level. The modeling focused on improving the primary structures and canal operation to meet the challenges of future conditions of sea level rise and land use changes, and considering multiple flood drivers such as rainfall, rising groundwater, and surge conditions.

The resilience optimized alternative (Alt RO) was developed by the PDT based on multiple considerations which included public project scoping, project management measures and cost effectiveness, as detailed in the main report. Model outputs used to determine cost effectiveness were gridded flood depths across the study area provided to the economic team for use in the HEC FDA modeling tool. They were provided at 125' by 125' model resolution for eight design events both with and without project for 2035 intermediate (baseline), and 2085 low, 2085 intermediate, and 2085 high sea level. The model files and select results are located at the District server shown in the table. At the conclusion of the project, they will be uploaded to the Statewide Model Management System (SFWMD, 2025), but are currently published through ftp. The link can be copied into the user's browser for access : <ftp://ftppub.sfwmd.gov/outgoing/BC203/>

Simulation Suite	Location
ECB_runs	\\ad.sfwmd.gov\dfsroot\data\BC203\12_ECB_runs
FWOPI_runs	\\ad.sfwmd.gov\dfsroot\data\BC203\15_FWOPI_runs
FWOPi_runs	\\ad.sfwmd.gov\dfsroot\data\BC203\13_FWOPi_runs
FWOPh_runs	\\ad.sfwmd.gov\dfsroot\data\BC203\14_FWOPh_runs
AltRO_runs:	
ECWP	\\ad.sfwmd.gov\dfsroot\data\BC203\21_TSP_runs\02_ECWP
FWPI	\\ad.sfwmd.gov\dfsroot\data\BC203\21_TSP_runs\04_FWPI
FWPi	\\ad.sfwmd.gov\dfsroot\data\BC203\21_TSP_runs\01_FWPi
FWPh	\\ad.sfwmd.gov\dfsroot\data\BC203\21_TSP_runs\03_FWPh

Maps of features are presented in each drainage area, but the following features are also included in all upstream managed watersheds and/or their control structures:

- Hardening at all tidal structures
- Pre-storm drawdown in primary canal and secondary system. Drawdown in the canals was initiated two days prior to peak rainfall design event simulation day of 9/10/2017. Drawdown was allowed 0.5' below the bottom of low range in the primary canal and 0.5' below control elevation in the secondary pumped systems. Depending on the tidal structure configuration, drawdown may or may not be achieved to the desired level. Once the two days pre-storm drawdown period is complete, operations are maintained to low range setting for the respective structure.
- Assumed improvements to the stormwater drainage system within the secondary and tertiary system where conceptualized

- Operations to discontinue pumping and/or close additional gates when downstream sensor elevations indicate impacted flood levels

In this results section for the optimized plan, modeling results will be shown by drainage areas and consist of visuals of selected events which vary in order to tell the story. Model result visuals included for each drainage area are structure time series of HW, TW, and flow, canal peak profiles, and watershed flood depth difference maps.

In addition to the model result visuals for selected events, structure performance tables are presented for all events. As sea level increases, the ability of the structure to deliver baseline flows at baseline headwater is decreased. Typically, the flow is decreased and/or the headwater at which the flow can be delivered is increased. The measures incorporated in the Alt RO have improved the structure performance when acting against these obstacles and are compared using three structure performance metrics which are peak 12 hour moving average flow (average flow during a tide cycle), peak 12 hour moving average HW seen at the canal, and flow volume discharged through the structure during the 72 hour rainfall event. The performance metrics are tabulated for each structure/drainage area and compared to the baseline and additionally as a table of normalized values. Normalizing the values with respect to the baseline (ECB) flow, headwater, and volume yield a measure of how well the Alt RO features can maintain 2035 intermediate sea level performance under future conditions.

The performance tables are prepared for flow and headwater stage as follows:

1. Tabulate the 15-minute time series of flow and head water stage for every event of every case,
2. Calculate the maximum 12-hour rolling average headwater stages and 12-hour rolling average flow excluding the initial 24 hours of simulation. Determine the maximum values of 12-hour stage and flow from these 12 hour averages.
3. Tabulate values in the top table and normalize the values for the bottom table (for example Table 9 and Table 10)
4. To normalize flow, calculate scores of each case/event as either 1.0 if the flow is as great or greater than the corresponding ECB event's flow, or a normalized value if less than 1.0 determined by dividing the event value by the ECB event's flow
5. To normalize HW, calculate scores of each case/event as either 1.0 if the headwater stage is not greater than the corresponding ECB event's stage, or a normalized value less than 1.0 if the headwater stage is greater than ECB
 - a. Using the maximum rolling average value as a reference value, calculate as (difference between reference value and event value)/ (difference between reference stage and ECB event value)
6. The total volumes of flow are calculated started at the beginning of the 3 day event for 3 days. The event starts on September 8, 2017 2 a.m. The normalized values are determined the same method as the normalized flow values.

Based on the detailed review of these modeling results (as included below) and reflecting the rigorous model setup and QAQC process, the modeling effort for the C&SF Section 203 Study is deemed to be technically correct & robust for the intended use by the project to evaluate relative differences in project features across a range of sea level and climate conditions.



Hillsboro Canal/ G56

The main Alt RO features in the Hillsboro watershed are gravity improvements at the G56 structure and canal improvements for more efficient delivery of the runoff to the tidal structure and downstream. Improvements are prudent downstream to augment the ability of the tidal reach to absorb the additional flow volume from G56 while not adversely impacting the ability of the E-3 canal to continue discharging. Figure 37 presents a map showing Alt RO features for Hillsboro Canal and watershed.

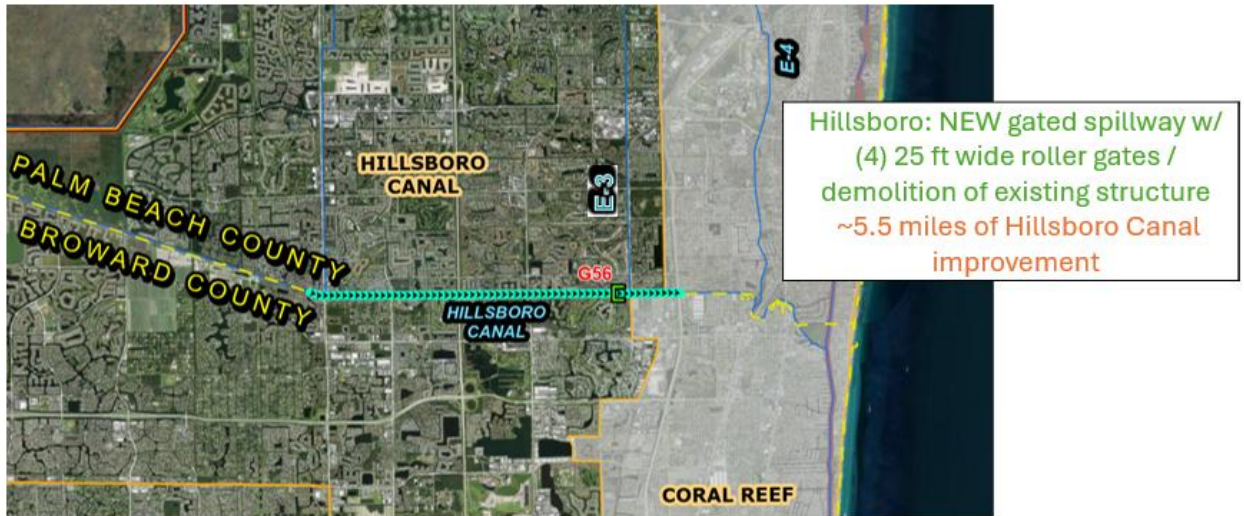


Figure 37 Map of Hillsboro Canal/ G56 Alt RO Features

The G56 structure operates the upstream canal reach at a higher operational range than some of the other tidal structures in the County. Because of this, gravity improvements were shown to be effective even at high sea level. For the Alternative, Pre-storm drawdown was realized with a gravity structure as shown in the structure time series plots for G56 in Figure 38 and Figure 39. The gravity structure was operated to maintain the upstream canal reach below the normal range (below 8 ft NGVD29) prior to and after the storm surge under normal high tide conditions. It should be noted that early adjustments in stage and flow are due to initial water levels in the canal. As explained in Future Without Project Model Setup initial conditions, the initial elevations were set high to allow model operations in the first hours of the simulation to determine canal levels which result from high sea level downstream.

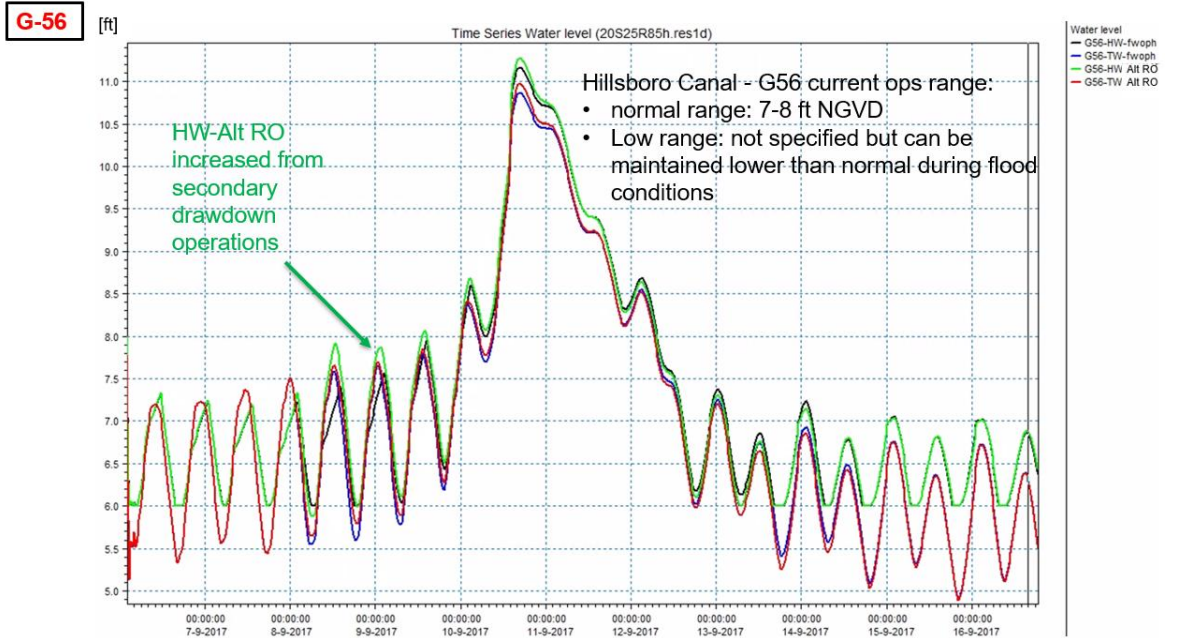


Figure 38 HW & TW at G56 for 20S25R85h (High Sea Level) - FWOP vs. Alt RO

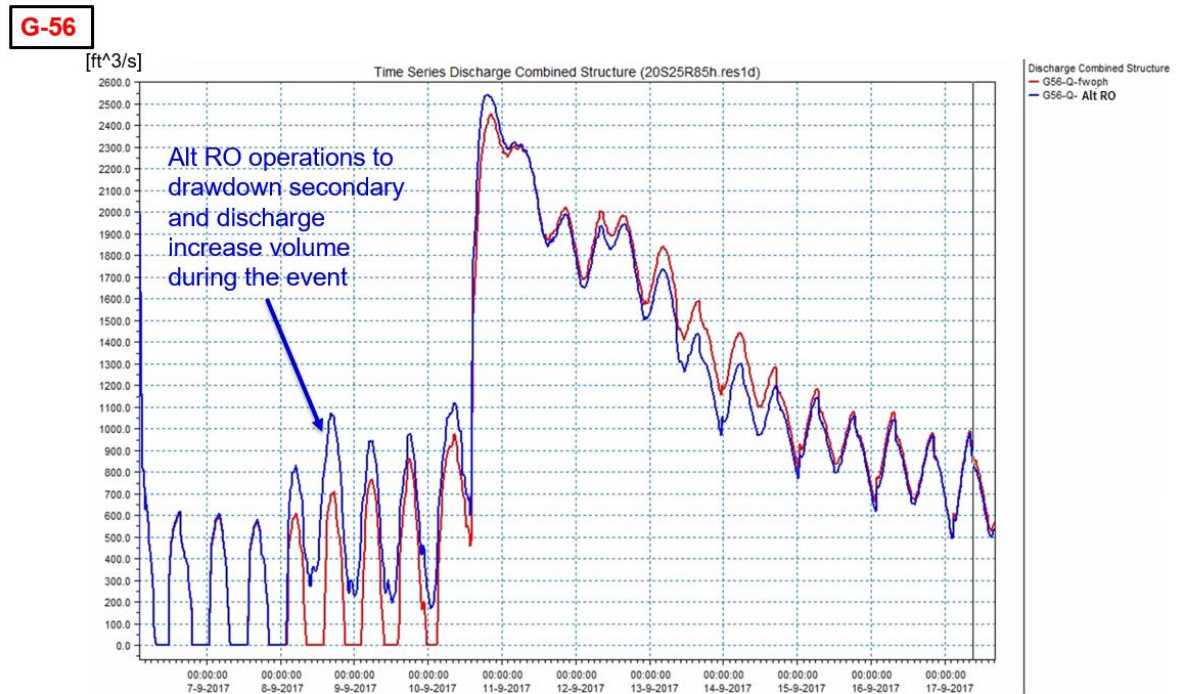


Figure 39 Flow at G56 for 20S25R85h (High Sea Level) - FWOP vs. Alt RO

Peak profile plots for three events are shown in Figure 40. The peak stages along the Hillsboro canal stay within bank upstream of G56. The Alt RO peaks within the canal are



slightly higher than the FWOP for the three events even though the capacity of the structure is increased. This is due to the modeled improved efficiency of the secondary to move water to the primary system. The operational changes and G56 capacity increase result in more volume of water being discharged during the 3-day event.

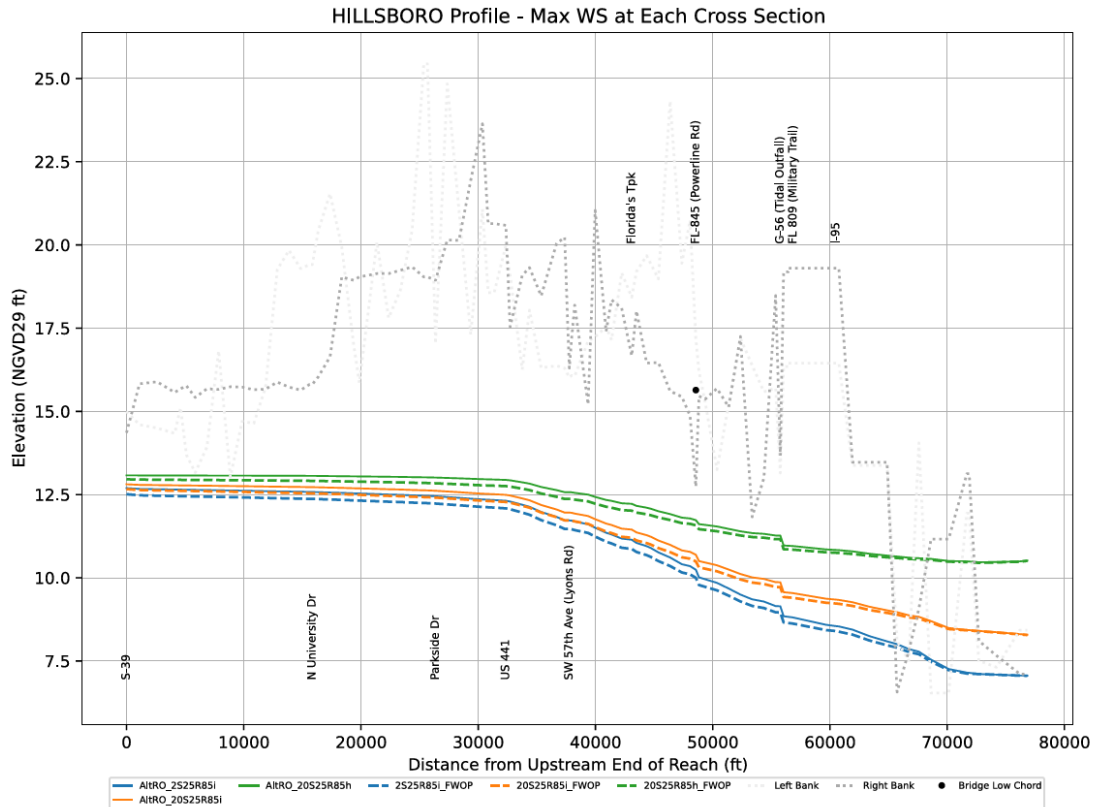


Figure 40 Hillsboro Canal Peak Profile - Alt RO vs. FWOP

Improving the ability of the primary system to move more volume of water during the event is realized by pre-storm drawdown operations and improvements at the structure and canal. The improvements combined with the primary system improvements to operations and capacity are what yield benefits. Two improvements are very important in the Hillsboro watershed for benefits – pre-storm drawdown not only in the primary system but also in the secondary system, and efficient movement of secondary water to the primary system.

Table 9 presents the peak moving average 12-hour flow (averaged over the 12 hr tide cycle), and shows only nominal improvements to flow are experienced with the upgrade when compared to FWO. Table 10 shows the peak 12-hour HW to mostly increase for Alt RO. These results would indicate no improvement to the primary system was seen with the changes. However, if you look at the volume of water discharged during the event (Table 11), the data shows there is more water moving through the primary system for the entirety of the 3-day event for all scenarios. The increase in volume discharged results in an improvement in the flood depths. Along with the volumes, the drainage area for the Hillsboro Canal watershed is included, however the drainage area upstream of G56 may

vary by several square miles as the stages in the Lake Worth Drainage District canals vary. The canal levels in the lateral canals draining to the E3 canal (see map of Figure 37) can affect how much water is drained downstream of G56.

Table 9 G56 Structure Performance Table - Peak 12 hr Average Q (CFS)*

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	2199	2277	2202	2270	2163	2238	1870	1978
2S10R	2505	2571	2486	2552	2420	2489	2135	2241
2S25R	2855	2932	2828	2907	2781	2863	2539	2634
10S10R	2447	2509	2406	2462	2326	2372	1962	2036
20S25R	2778	2845	2735	2803	2665	2731	2338	2414
2S100R	3385	3464	3359	3443	3313	3394	3071	3178
100S100R	3253	3327	3204	3283	3123	3195	2793	2882
2S500R	3940	4088	3922	4073	3894	4046	3715	3811
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	0.98	1.00	0.85	0.90
2S10R	1.00	1.00	0.99	1.00	0.97	0.99	0.85	0.89
2S25R	1.00	1.00	0.99	1.00	0.97	1.00	0.89	0.92
10S10R	1.00	1.00	0.98	1.00	0.95	0.97	0.80	0.83
20S25R	1.00	1.00	0.98	1.00	0.96	0.98	0.84	0.87
2S100R	1.00	1.00	0.99	1.00	0.98	1.00	0.91	0.94
100S100R	1.00	1.00	0.99	1.00	0.96	0.98	0.86	0.89
2S500R	1.00	1.00	1.00	1.00	0.99	1.00	0.94	0.97

*TSP/Recommended Plan = Alt RO



Table 10 G56 Structure Performance Table - Peak 12 hr Average HW (ft NGVD29)*

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	7.11	6.99	7.21	7.22	7.39	7.54	9.24	9.35
2S10R	7.47	7.63	7.64	7.79	7.90	8.06	9.56	9.68
2S25R	8.29	8.45	8.43	8.60	8.71	8.86	10.05	10.16
10S10R	7.97	8.12	8.24	8.38	8.79	8.91	10.44	10.52
20S25R	8.81	8.96	9.09	9.23	9.51	9.63	10.90	10.97
2S100R	9.60	9.68	9.72	9.80	9.91	10.01	10.87	10.92
100S100R	10.23	10.31	10.44	10.51	10.74	10.78	11.86	11.92
2S500R	11.22	11.19	11.30	11.26	11.40	11.37	11.97	11.85
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	0.98	0.98	0.95	0.93	0.64	0.62
2S10R	1.00	0.97	0.97	0.94	0.92	0.89	0.62	0.60
2S25R	1.00	0.97	0.97	0.93	0.91	0.88	0.63	0.60
10S10R	1.00	0.97	0.95	0.92	0.84	0.81	0.51	0.49
20S25R	1.00	0.96	0.93	0.90	0.83	0.81	0.50	0.48
2S100R	1.00	0.98	0.96	0.94	0.91	0.88	0.63	0.61
100S100R	1.00	0.97	0.93	0.90	0.81	0.80	0.41	0.39
2S500R	1.00	1.00	0.96	0.98	0.90	0.92	0.58	0.65

*TSP/Recommended Plan = Alt RO

Table 11 G56 Structure Performance Table – Volume Discharged During the 3 Day Event* (ac-ft) **

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	3604	5013	3598	4995	3541	4932	3043	4371
2S10R	4094	5530	4097	5522	4039	5444	3541	4891
2S25R	4894	6362	4908	6354	4850	6285	4396	5769
10S10R	3976	5406	3930	5339	3817	5220	3163	4559
20S25R	4760	6215	4702	6173	4610	6041	4017	5406
2S100R	6421	7885	6440	7875	6382	7793	5855	7225
100S100R	6131	7574	6060	7490	5910	7321	5124	6508
2S500R	8445	9937	8469	9929	8418	9847	7826	9140
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	0.98	1.00	0.84	1.00
2S10R	1.00	1.00	1.00	1.00	0.99	1.00	0.86	1.00
2S25R	1.00	1.00	1.00	1.00	0.99	1.00	0.90	1.00
10S10R	1.00	1.00	0.99	1.00	0.96	1.00	0.80	1.00
20S25R	1.00	1.00	0.99	1.00	0.97	1.00	0.84	1.00
2S100R	1.00	1.00	1.00	1.00	0.99	1.00	0.91	1.00
100S100R	1.00	1.00	0.99	1.00	0.96	1.00	0.84	1.00
2S500R	1.00	1.00	1.00	1.00	1.00	1.00	0.93	1.00

*TSP/Recommended Plan = Alt RO** Hillsboro canal drainage area 90 sq. miles, but can vary due to E3 canal operation discharging downstream of G56 (SFWMD, 2021)

The maps of Figure 41 illustrate improvements in the Hillsboro basin flood depths for 3 selected events. The maps are meant to qualitatively show improvements to depths, but the quantification of benefits from the improvements are determined using the HEC FDA tool (SFWMD, November, 2025)

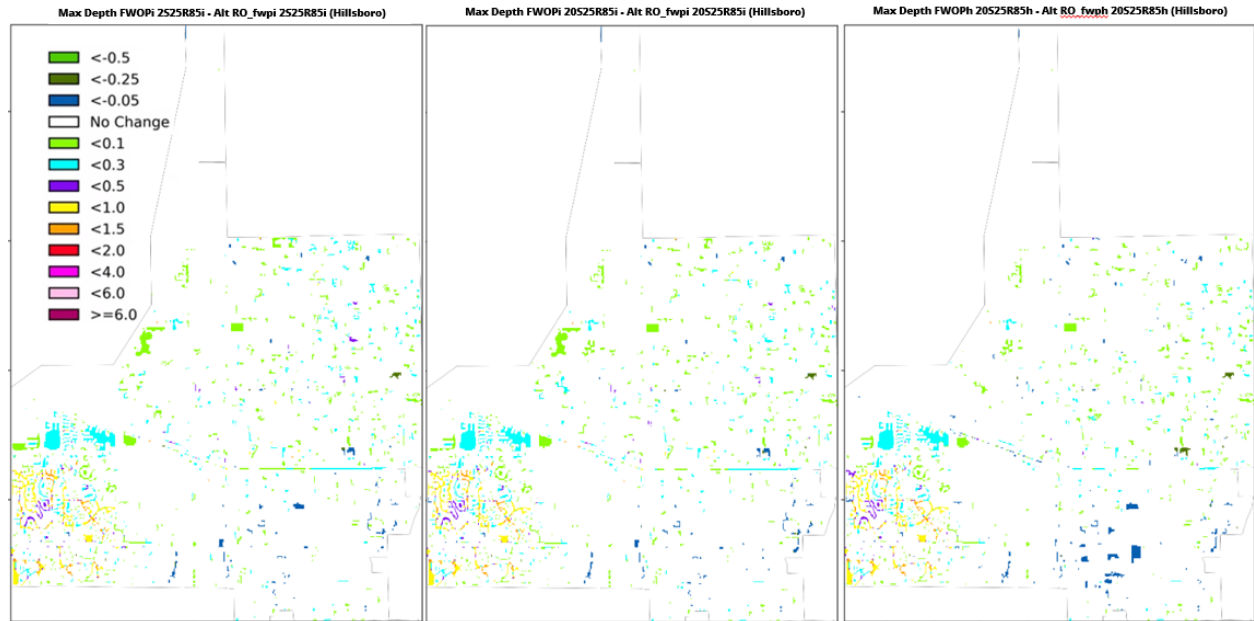


Figure 41 Hillsboro Drainage Area Flood Depth Difference Maps in ft.

C-14, Pompano Canal/ S37A & S37B and G57

The main Alt RO features in the C-14/Pompano drainage area are shown in Figure 42 and consist of both gravity and conveyance improvements along with pumping at the most vulnerable structure S-37A.



Figure 42 Map of C-14 and Pompano Canal Alt RO Features

G57 spillway gravity capacity was slightly increased and the conveyance on the canal upstream was improved by increasing the size of an upstream culvert. Hardening of the structure by raising the top of gate and the overtopping elevation at the structure stopped the backflow at the gate during high tailwater conditions and especially at high sea level conditions where backflow was experienced during normal tide cycles as shown in Figure 43 and Figure 44.



G-57

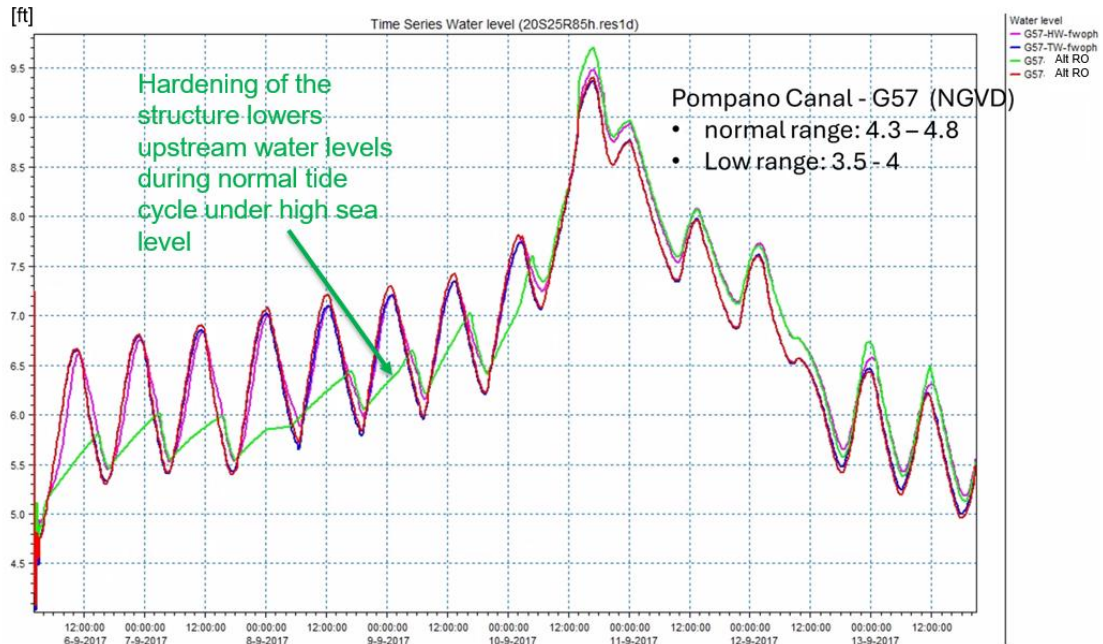


Figure 43 HW & TW at G57 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

G-57

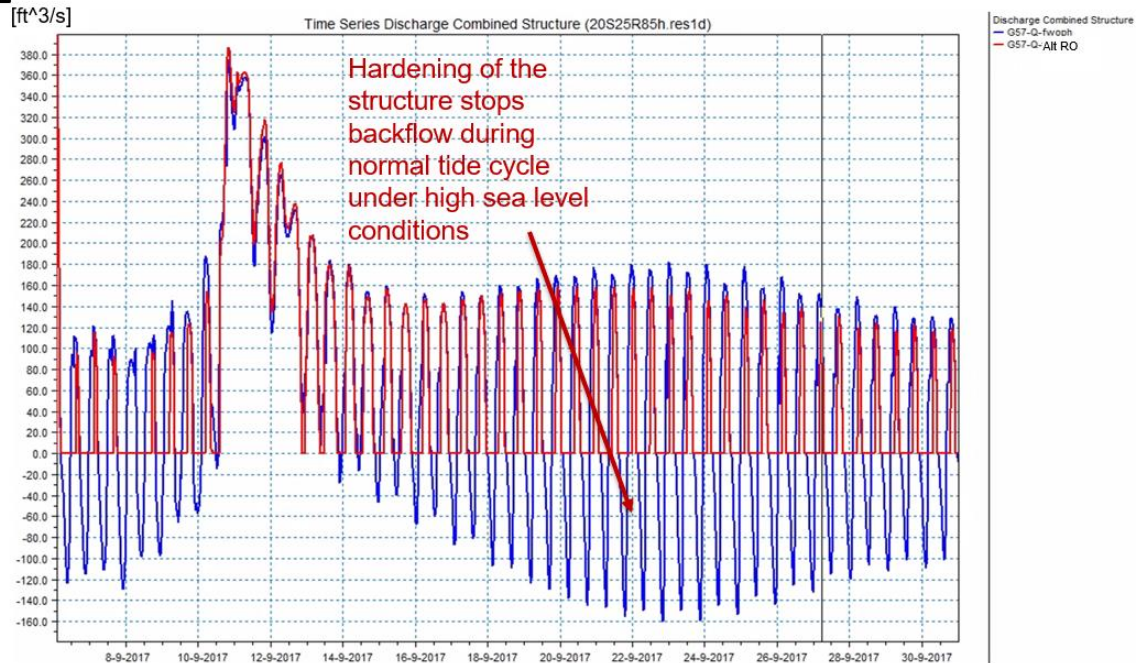


Figure 44 Flow at G57 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

Peak profile plots for 3 events are shown in Figure 45. The Pompano watershed drains between the G-65 structure and the G57 structure on the profile plot. The peak stages along this section of the canal stay within the banks even for high sea level event. The Alt RO peaks within the canal are slightly higher than the FWOP for the three events even though the capacity of the structure is increased. This is due to the modeled increase in efficiency of the secondary to move water to the primary system which will be realized with the improvements in the primary system. The operational changes and G57 capacity increase results in more volume of water being discharged during the 3-day event.

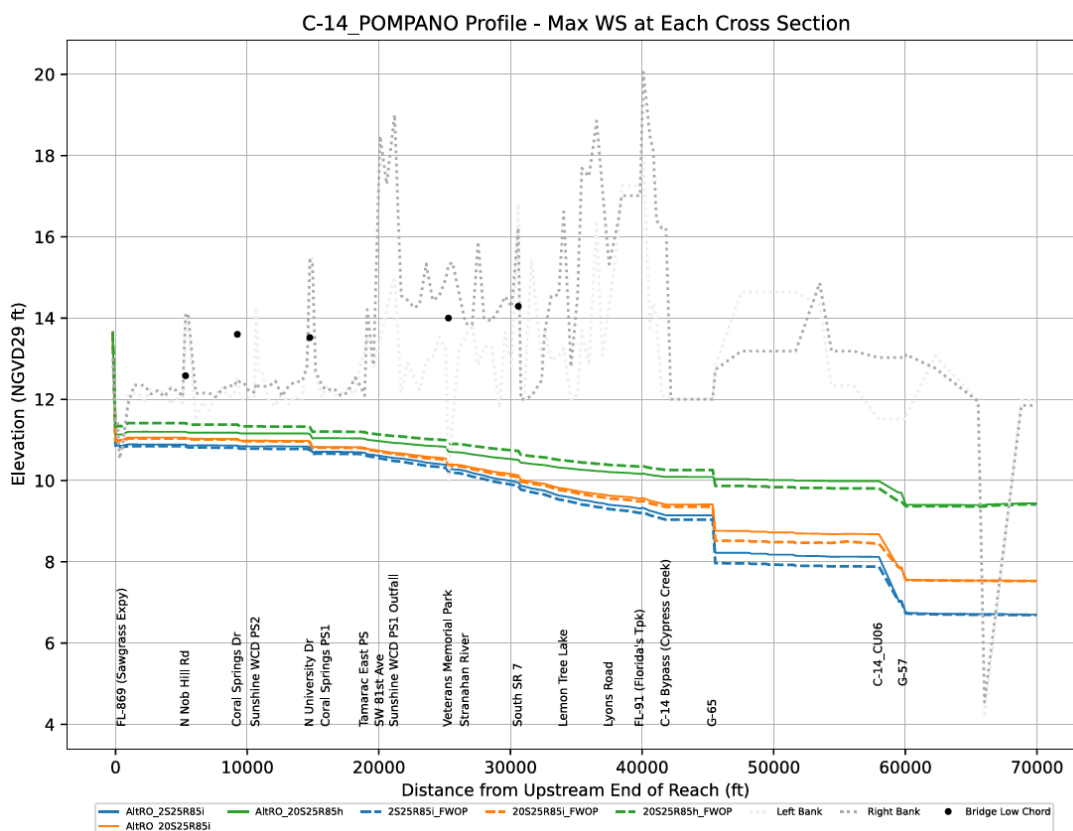


Figure 45 C-14/ Pompano Canal Peak Profile - Alt RO vs. FWOP

Table 12 presents the peak moving average 12-hour flow (averaged over the 12-hour tide cycle), and shows only nominal improvements to flow with the upgrade when compared to FWO. Table 13 shows changes to the peak 12-hour HW when comparing Alt RO to FWOP. The headwater at the structure is not improved to ECB levels and are very similar to FWOP elevations. The improvement to an upstream culvert likely allowed improved conveyance to the upstream of the structure causing higher headwater at the structure. When looking at the volume of water discharge throughout the 3-day event, the numbers increase slightly except for high sea level volumes (Table 14). For high sea level conditions, the FWOP discharge would include backflow water which the Alt RO doesn't have due to hardening. The lower volumes in Alt RO would be consistent with less backflow coming into the watershed which for FWOP would then need to be discharged.



The maps of Figure 46 illustrate improvements in the Pompano basin flood depths for three selected events.

Table 12 G57 Structure Performance Table - Peak 12 hr Average Q (CFS)*

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	208	222	213	229	219	233	227	230
2S10R	275	296	285	308	287	313	292	303
2S25R	390	422	405	439	413	446	382	400
10S10R	265	289	272	297	270	294	248	259
20S25R	383	414	387	420	379	405	346	357
2S100R	604	638	609	641	595	624	532	554
100S100R	540	577	535	566	518	546	468	485
2S500R	800	807	793	800	758	768	634	654
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2S10R	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
2S25R	1.00	1.00	1.00	1.00	1.00	1.00	0.98	1.00
10S10R	1.00	1.00	1.00	1.00	1.00	1.00	0.94	0.98
20S25R	1.00	1.00	1.00	1.00	0.99	1.00	0.91	0.93
2S100R	1.00	1.00	1.00	1.00	0.99	1.00	0.88	0.92
100S100R	1.00	1.00	0.99	1.00	0.96	1.00	0.87	0.90
2S500R	1.00	1.00	0.99	1.00	0.95	0.96	0.79	0.82

*TSP/Recommended Plan = Alt RO

Table 13 G57 Structure Performance Table - Peak 12 hr Average HW (ft NGVD29)*

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	4.60	4.57	5.07	5.08	5.77	5.77	7.83	7.92
2S10R	4.72	4.74	5.21	5.23	5.88	5.91	7.91	8.00
2S25R	4.91	4.98	5.39	5.46	6.05	6.10	8.03	8.11
10S10R	5.80	5.83	6.24	6.28	6.87	6.93	8.82	8.93
20S25R	6.16	6.19	6.60	6.64	7.19	7.24	9.06	9.16
2S100R	5.32	5.37	5.75	5.76	6.39	6.40	8.32	8.25
100S100R	7.30	7.26	7.67	7.64	8.14	8.21	10.04	10.16
2S500R	5.87	5.83	6.25	6.18	6.88	6.77	8.64	8.44
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	0.93	0.92	0.82	0.82	0.50	0.48
2S10R	1.00	1.00	0.92	0.92	0.82	0.81	0.49	0.48
2S25R	1.00	0.99	0.92	0.91	0.81	0.80	0.49	0.47
10S10R	1.00	1.00	0.92	0.91	0.79	0.78	0.42	0.40
20S25R	1.00	0.99	0.91	0.90	0.79	0.78	0.40	0.38
2S100R	1.00	0.99	0.92	0.92	0.81	0.81	0.47	0.48
100S100R	1.00	1.00	0.90	0.91	0.77	0.75	0.26	0.23
2S500R	1.00	1.00	0.92	0.94	0.80	0.82	0.46	0.50

*TSP/Recommended Plan = Alt RO

Table 14 G57 Structure Performance Table - Volume Discharged During the 3 Day Event* (ac-ft) **

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	465	487	467	484	450	456	356	276
2S10R	565	593	573	596	554	568	463	379
2S25R	732	771	743	778	724	748	616	528
10S10R	521	538	524	547	503	509	354	266
20S25R	682	716	686	719	657	669	493	399
2S100R	1037	1077	1039	1071	1004	1021	873	799
100S100R	876	895	853	866	809	803	571	466
2S500R	1376	1407	1370	1400	1319	1345	1162	1082
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	0.97	0.98	0.77	0.59
2S10R	1.00	1.00	1.00	1.00	0.98	1.00	0.82	0.67
2S25R	1.00	1.00	1.00	1.00	0.99	1.00	0.84	0.72
10S10R	1.00	1.00	1.00	1.00	0.97	0.98	0.68	0.51
20S25R	1.00	1.00	1.00	1.00	0.96	0.98	0.72	0.59
2S100R	1.00	1.00	1.00	1.00	0.97	0.98	0.84	0.77
100S100R	1.00	1.00	0.97	0.99	0.92	0.92	0.65	0.53
2S500R	1.00	1.00	1.00	1.00	0.96	0.98	0.84	0.79

*TSP/Recommended Plan = Alt RO

** Pompano Canal drainage area = 3.9 sq. miles (SFWMD, 2021)

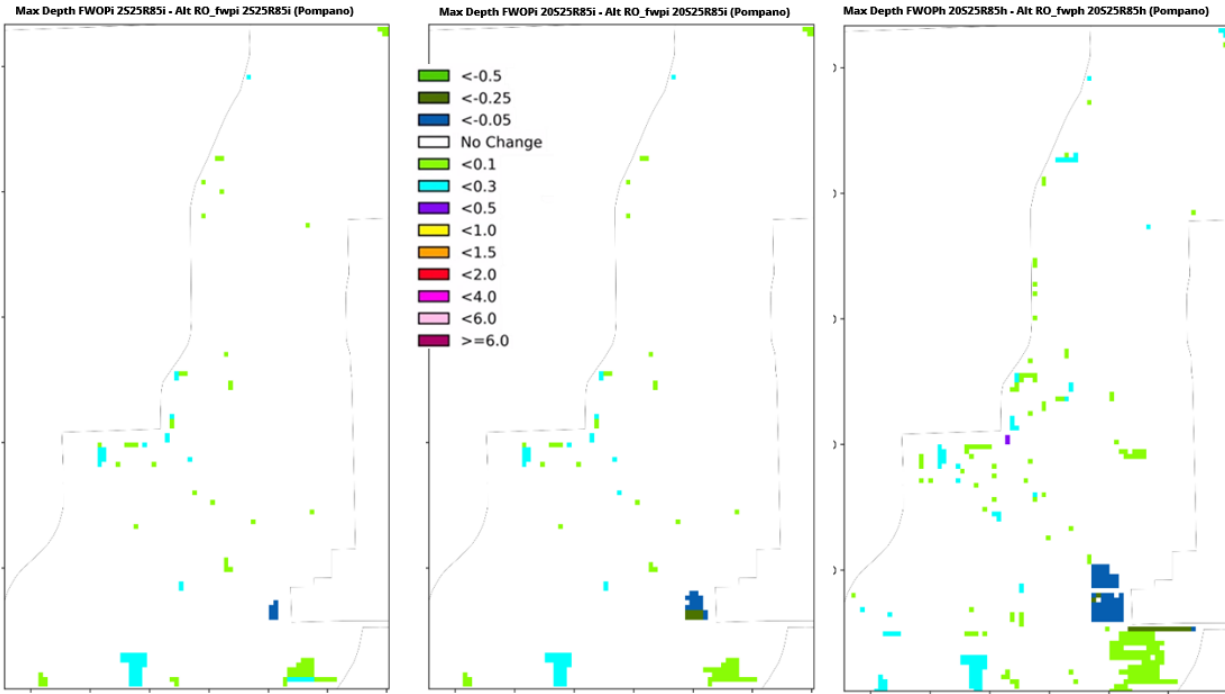


Figure 46 Pompano Watershed Flood Depth Difference Maps in ft

S37A spillway gravity capacity was increased by adding a third 25 ft spillway gate, and pumping capability was added with 1,200 cfs capacity. Hardening of the structure by raising the top of gate and the overtopping elevation at the structure stopped the backflow at the gate during high tailwater conditions and especially at high sea level conditions. The hardening at this structure also had to include raising canal cross section bank elevations.

Pumps were necessary at this structure to achieve pre storm drawdown which was found to alleviate flooding in the C-14 E watershed. Figure 47 illustrates the pump drawdown of the Alt RO HW under high sea level conditions (20S25R85h event). The figure also shows how the structure would not be able to operate the upstream canal in the normal range without the help of a pump if high sea level conditions are experienced. Figure 48 flow comparison with FWOP shows how hardening was included to stop structure backflow. Cycling of the pump at the peak would occur because of the downstream sensor cut off elevation.

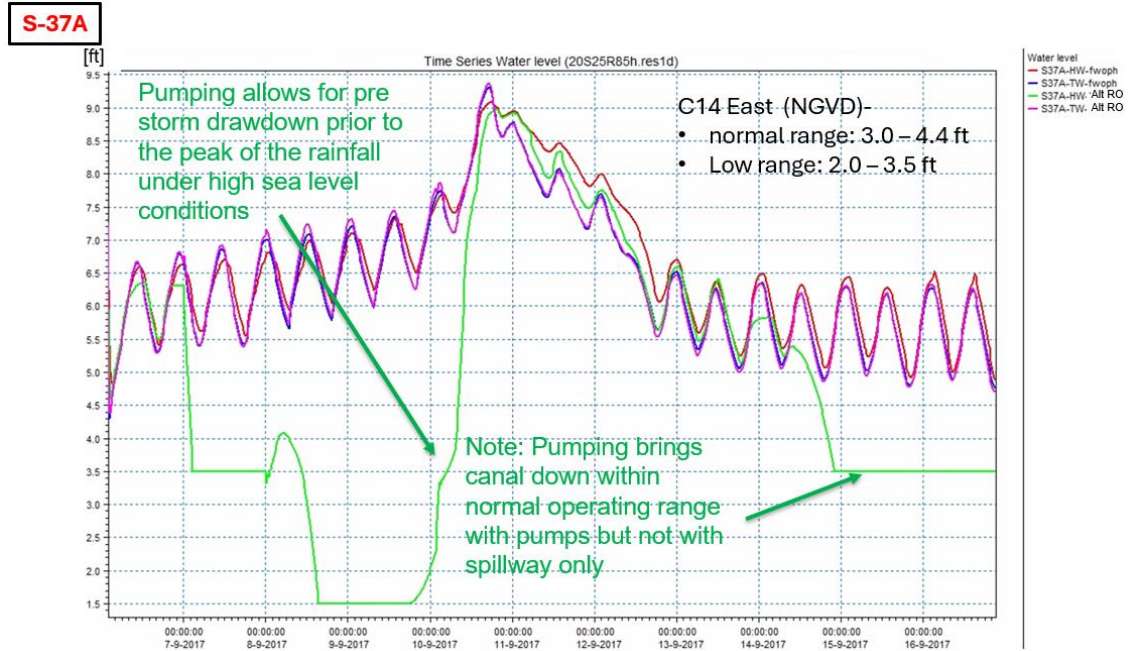


Figure 47 HW & TW at S37A for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

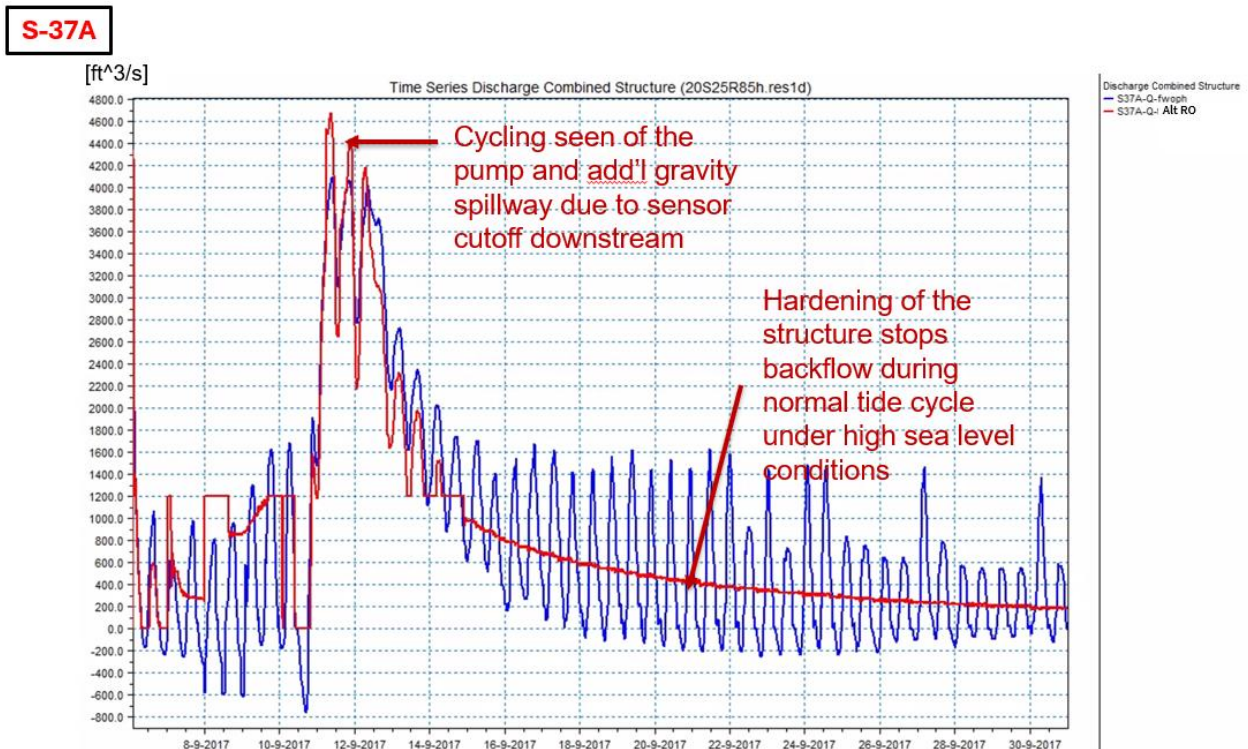


Figure 48 Flow at S37A for 20S25R85h (High Sea Level) – FWOP vs. Alt RO



Peak profile plots for three events are shown in Figure 49. The C-14E watershed drains between the S37B structure and the S37A structure on the profile plot. There is a lot of out of bank flow and the railroad bridge is impacted even for the 2S25R85i event. The peak stages for 20S25R85h conditions impact several bridges within the C-14E canal reach. Operational changes and S37A capacity increase results in more volume of water being discharged during the 3-day event.

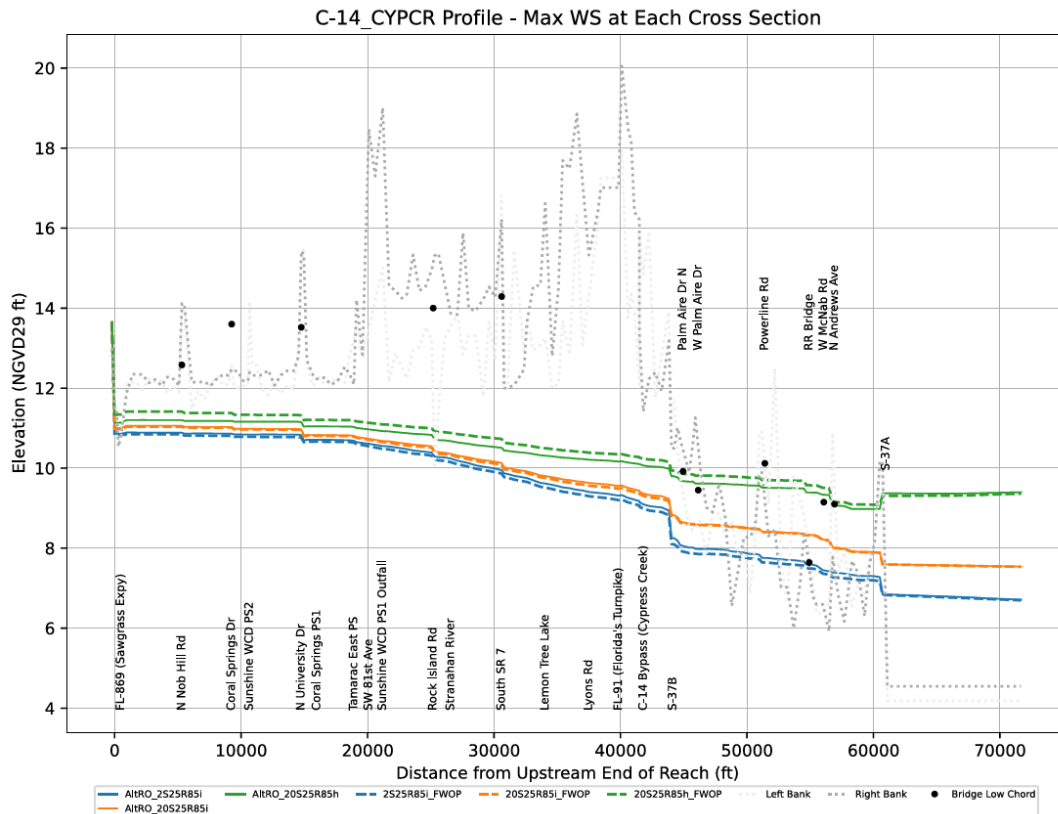


Figure 49 C-14 Canal Peak Profile - Alt RO vs. FWOP

Table 15 presents the peak moving average 12 hour flow (averaged over the 12-hour tide cycle), and generally shows improvements to flow with the upgrade when compared to FWO. Table 16 shows changes to the peak 12-hour HW when comparing Alt RO to FWOP. The headwater at the structure also generally shows improvement with Alt RO except for some of the low recurrence interval surge events. The surge levels could trigger the downstream sensor cutoff which would impact the HW. When looking at the volume of water discharge throughout the three day event, all events are able to discharge more runoff with the Alt RO (Table 17). The improvement in runoff discharged is particularly noticed for high sea level conditions. The maps of Figure 50 illustrate improvements in the C-14E basin flood depths for three selected events.

Table 15 S37A Structure Performance Table - Peak 12 hr Average Q* (cfs)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	3127	3257	3141	3241	3099	3133	2778	2314
2S10R	3588	3750	3605	3729	3572	3669	3177	2988
2S25R	4218	4446	4235	4398	4184	4416	3558	3662
10S10R	3579	3594	3597	3565	3555	3524	3085	2823
20S25R	4271	4388	4278	4376	4325	4480	3765	3865
2S100R	5181	5513	5149	5408	5012	5353	4113	4411
100S100R	5201	5382	5200	5366	5126	5609	4568	5232
2S500R	6042	6626	5956	6357	5650	6236	4659	5173
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	0.99	1.00	0.89	0.74
2S10R	1.00	1.00	1.00	1.00	1.00	1.00	0.89	0.83
2S25R	1.00	1.00	1.00	1.00	0.99	1.00	0.84	0.87
10S10R	1.00	1.00	1.00	1.00	0.99	0.98	0.86	0.79
20S25R	1.00	1.00	1.00	1.00	1.00	1.00	0.88	0.91
2S100R	1.00	1.00	0.99	1.00	0.97	1.00	0.79	0.85
100S100R	1.00	1.00	1.00	1.00	0.99	1.00	0.88	1.00
2S500R	1.00	1.00	0.99	1.00	0.94	1.00	0.77	0.86

*TSP/Recommended Plan = Alt RO



Table 16 S37A Structure Performance Table - Peak 12 hr Average HW * (ft NGVD29)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	5.21	4.66	5.52	5.14	6.03	5.75	8.01	7.78
2S10R	5.48	4.97	5.73	5.45	6.24	5.98	8.17	7.92
2S25R	5.84	5.41	6.06	5.86	6.60	6.31	8.38	8.12
10S10R	6.17	6.12	6.60	6.67	7.24	7.27	8.82	8.68
20S25R	6.71	6.80	7.10	7.18	7.60	7.45	8.96	8.87
2S100R	6.46	6.06	6.78	6.52	7.23	6.77	8.58	8.45
100S100R	7.73	7.76	8.04	8.07	8.49	8.50	9.82	9.58
2S500R	7.27	6.98	7.44	7.43	7.71	7.54	8.78	8.68
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	0.93	1.00	0.83	0.89	0.41	0.46
2S10R	1.00	1.00	0.95	1.00	0.83	0.89	0.41	0.46
2S25R	1.00	1.00	0.95	1.00	0.82	0.89	0.39	0.45
10S10R	1.00	1.00	0.89	0.87	0.72	0.71	0.31	0.34
20S25R	1.00	0.97	0.88	0.86	0.73	0.77	0.32	0.34
2S100R	1.00	1.00	0.91	0.98	0.78	0.91	0.40	0.44
100S100R	1.00	0.99	0.86	0.85	0.67	0.66	0.08	0.19
2S500R	1.00	1.00	0.94	0.94	0.84	0.90	0.45	0.48

*TSP/Recommended Plan = Alt RO

Table 17 S37A Structure Performance Table - Volume Discharged During the 3 Day Event * (ac-ft) **

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	5011	7020	5064	7006	5074	6924	3473	6242
2S10R	5985	7958	6055	7929	6040	7854	4176	6864
2S25R	7420	9417	7466	9362	7424	9332	5069	7785
10S10R	5718	7629	5629	7446	5311	7126	2411	5317
20S25R	7007	8867	6872	8644	6336	8415	2719	5664
2S100R	9862	11907	9845	11822	9544	11770	6723	9035
100S100R	8125	9949	7762	9513	6848	8419	1603	5007
2S500R	12714	14931	12497	14269	12052	14185	8544	10491
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	1.00	1.00	0.69	1.00
2S10R	1.00	1.00	1.00	1.00	1.00	1.00	0.70	1.00
2S25R	1.00	1.00	1.00	1.00	1.00	1.00	0.68	1.00
10S10R	1.00	1.00	0.98	1.00	0.93	1.00	0.42	0.93
20S25R	1.00	1.00	0.98	1.00	0.90	1.00	0.39	0.81
2S100R	1.00	1.00	1.00	1.00	0.97	1.00	0.68	0.92
100S100R	1.00	1.00	0.96	1.00	0.84	1.00	0.20	0.62
2S500R	1.00	1.00	0.98	1.00	0.95	1.00	0.67	0.83

*TSP/Recommended Plan = Alt RO

** C-14 drainage area = 53 sq. miles (SFWMD, 2021)

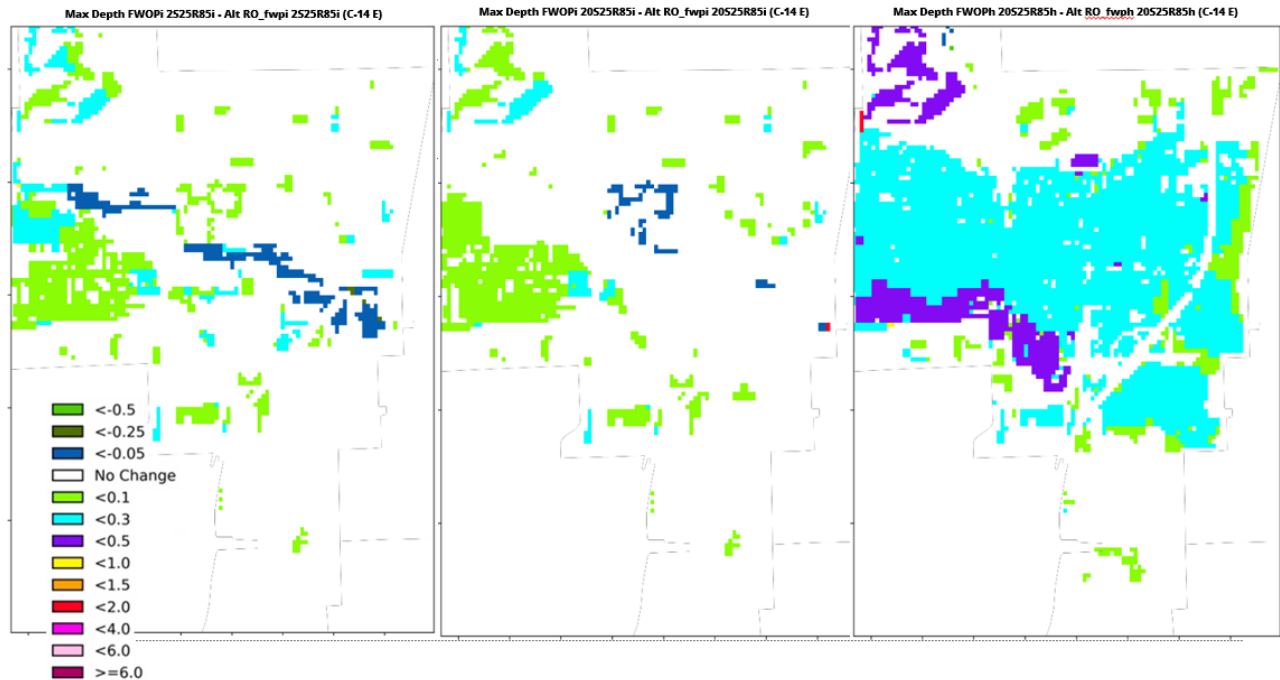


Figure 50 C-14 E Watershed Flood Depth Difference Maps in ft.

S37B spillway gravity capacity was not increased but improvements at S37A in discharging more volume also affected discharge at S37B. Similar drawdown operations were incorporated at the S37B structure and in the secondary of C-14W as part of the Alt RO and some improvements were made within the canal in the vicinity of the S37B to facilitate efficient discharge at S37B. The S37B structure operates the upstream canal reach at a higher operational range than most of the tidal structures in the County. Operational improvements including pre storm drawdown were achievable at S37B because of the drawdown at S37A which lowered S37B downstream water level as shown in Figure 51. Canal operational ranges were maintained prior to and after the storm surge under normal high tide conditions as shown in Figure 51 and Figure 52.

S-37B

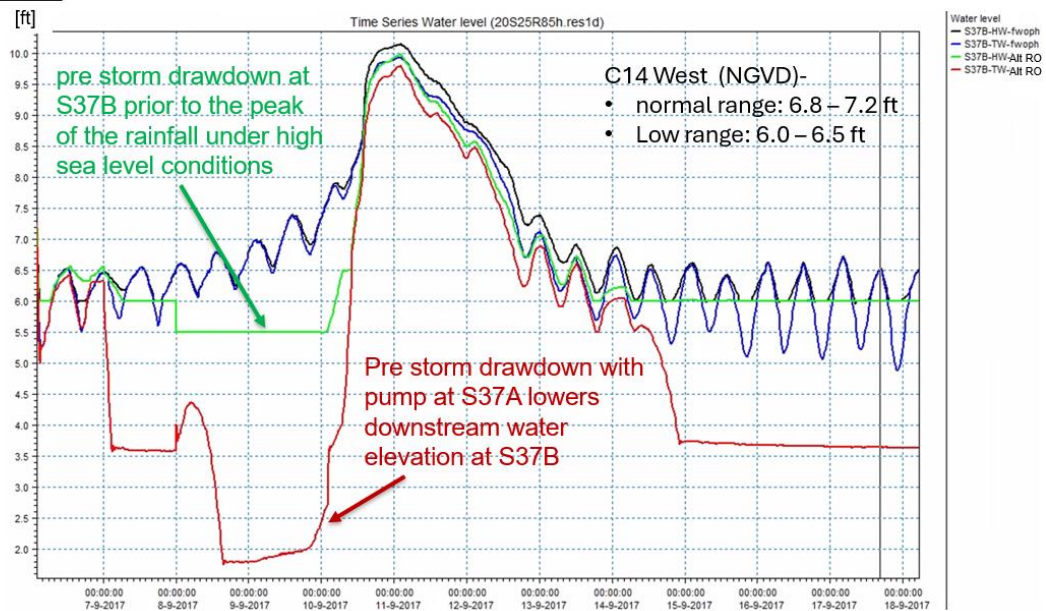


Figure 51 HW & TW at S37B for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

S-37B

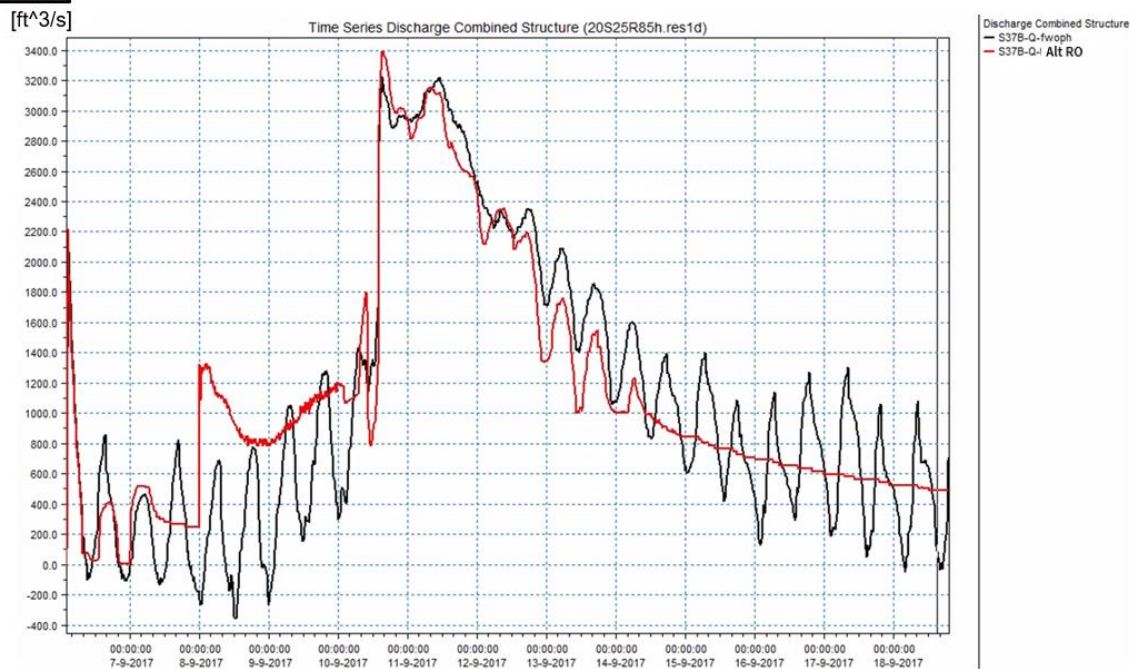


Figure 52 Flow at S37B for 20S25R85h (High Sea Level) – FWOP vs. Alt RO



Peak profile plots for three events are shown in Figure 49. On the Figure, the S37B structure is located upstream of the S37A tidal structure. The C-14W watershed drains upstream of the S37B structure on the profile plot. Modeling predicts no out of bank flow in the C-14 Canal upstream of S37B and no bridges are impacted by high water levels.

Table 18 presents the peak moving average 12-hour flow (averaged over the 12-hour tide cycle), and shows similar peak flows to FWO. Table 19 shows changes to the peak 12-hour HW when comparing Alt RO to FWOP. The headwater at the structure shows improvement with Alt RO for high sea level conditions. When looking at the volume of water discharge throughout the three day event, all events are able to discharge more runoff with the Alt RO (Table 20). The improvement in runoff discharged is particularly noticed for high sea level conditions where for FWOPh the flow volume through the structure had decreased from ECB. The maps of Figure 53 illustrate improvements in the C-14E basin flood depths for three selected events.

Table 18 S37B Structure Performance Table – Peak 12 hr Average Q * (CFS)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	2910	2971	2899	2935	2826	2815	2613	2379
2S10R	3314	3387	3305	3353	3232	3250	3014	2846
2S25R	3828	3949	3808	3889	3701	3822	3407	3420
10S10R	3266	3222	3216	3131	3081	2993	2743	2645
20S25R	3743	3742	3677	3650	3559	3594	3101	3067
2S100R	4418	4557	4378	4457	4299	4426	3873	3942
100S100R	4095	4087	4032	4038	3895	3905	3275	3343
2S500R	4783	4903	4707	4720	4627	4709	4087	4176
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	0.97	0.97	0.90	0.82
2S10R	1.00	1.00	1.00	1.00	0.98	0.98	0.91	0.86
2S25R	1.00	1.00	0.99	1.00	0.97	1.00	0.89	0.89
10S10R	1.00	0.99	0.98	0.96	0.94	0.92	0.84	0.81
20S25R	1.00	1.00	0.98	0.98	0.95	0.96	0.83	0.82
2S100R	1.00	1.00	0.99	1.00	0.97	1.00	0.88	0.89
100S100R	1.00	1.00	0.98	0.99	0.95	0.95	0.80	0.82
2S500R	1.00	1.00	0.98	0.99	0.97	0.98	0.85	0.87

*TSP/Recommended Plan = Alt RO

Table 19 S37B Structure Performance Table – Peak 12 hr HW * (ft NGVD29)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	7.02	6.86	7.20	7.09	7.47	7.39	8.84	8.52
2S10R	7.56	7.38	7.69	7.61	7.98	7.90	9.17	8.95
2S25R	8.25	8.11	8.38	8.33	8.63	8.52	9.60	9.42
10S10R	7.91	7.92	8.19	8.23	8.54	8.53	9.63	9.41
20S25R	8.66	8.72	8.83	8.88	9.05	9.00	10.08	9.87
2S100R	9.21	9.12	9.32	9.28	9.51	9.38	10.27	10.19
100S100R	9.77	9.78	9.89	9.90	10.11	10.13	10.92	10.78
2S500R	10.29	10.26	10.38	10.40	10.48	10.45	11.03	10.97
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	0.96	0.99	0.91	0.93	0.64	0.70
2S10R	1.00	1.00	0.97	0.99	0.91	0.92	0.64	0.69
2S25R	1.00	1.00	0.97	0.98	0.90	0.93	0.64	0.69
10S10R	1.00	1.00	0.93	0.92	0.85	0.85	0.58	0.63
20S25R	1.00	0.98	0.95	0.93	0.88	0.90	0.57	0.64
2S100R	1.00	1.00	0.96	0.97	0.89	0.94	0.62	0.65
100S100R	1.00	0.99	0.94	0.94	0.85	0.84	0.49	0.55
2S500R	1.00	1.00	0.95	0.94	0.89	0.91	0.57	0.60

*TSP/Recommended Plan = Alt RO



Table 20 S37B Structure Performance Table – Volume Discharged During the 3 Day Event *(Ac-ft) **

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	4796	6747	4808	6724	4824	6681	4183	6544
2S10R	5689	7581	5712	7542	5712	7511	5053	7312
2S25R	6985	8868	6992	8810	6963	8787	6164	8439
10S10R	5548	7372	5490	7254	5343	7159	4751	7003
20S25R	6758	8545	6717	8479	6613	8465	5756	8130
2S100R	9096	10974	9085	10913	8984	10858	8008	10060
100S100R	8518	10333	8440	10216	8252	9895	6893	9126
2S500R	11667	13493	11603	13317	11486	13159	10262	11801
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	1.00	1.00	0.87	1.00
2S10R	1.00	1.00	1.00	1.00	1.00	1.00	0.89	1.00
2S25R	1.00	1.00	1.00	1.00	1.00	1.00	0.88	1.00
10S10R	1.00	1.00	0.99	1.00	0.96	1.00	0.86	1.00
20S25R	1.00	1.00	0.99	1.00	0.98	1.00	0.85	1.00
2S100R	1.00	1.00	1.00	1.00	0.99	1.00	0.88	1.00
100S100R	1.00	1.00	0.99	1.00	0.97	1.00	0.81	1.00
2S500R	1.00	1.00	0.99	1.00	0.98	1.00	0.88	1.00

*TSP/Recommended Plan = Alt RO

** C-14 West drainage area = 50.3 sq. miles (SFWMD, 2021)



Figure 53 C-14 W Watershed Flood Depth Difference Maps in ft.

C-13 Canal/ S36

S36 spillway gravity capacity was increased from one 25 ft gate to two 16 ft gates, and pumping capability was added with 510 cfs capacity (Figure 54). Incorporating two gates into the design was for redundancy to facilitate maintenance of a gate. Hardening of the structure by raising the top of gate and the overtopping elevation at the structure stopped the backflow at the gate during high tailwater conditions and especially at high sea level conditions. The hardening at this structure did not need to include raising canal cross section bank elevations close to the structure.

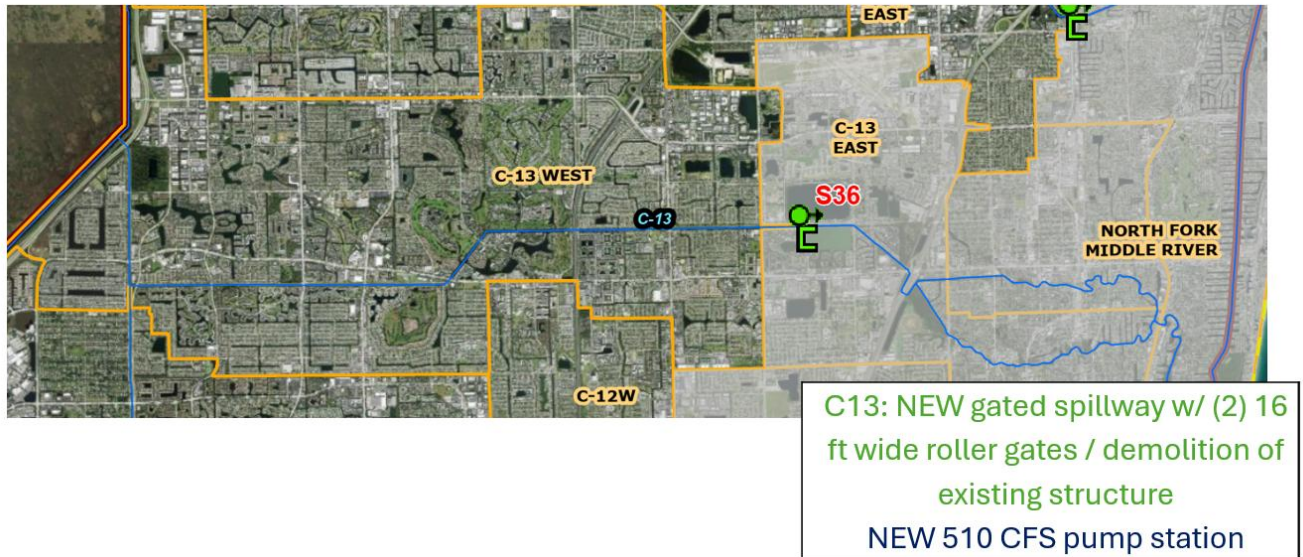


Figure 54 Map of C-13 Canal Alt RO Features

Pumps were able to achieve pre storm drawdown even during high sea level conditions. Figure 55 illustrates the pump drawdown of the Alt RO HW under high sea level conditions (20S25R85h event). The figure also shows how the structure would not be able to operate the upstream canal in the normal range without the help of a pump if high sea level conditions are experienced and canal operating ranges are not increased. Figure 56 shows a decrease in peak flow from FWOP but a change in timing with more flow earlier in the event due to pumping. For this structure, peak HW is decreased and so is peak flow. Cycling of the pump at the peak because of the downstream sensor cut off elevation would also decrease flow at the peak of the storm.

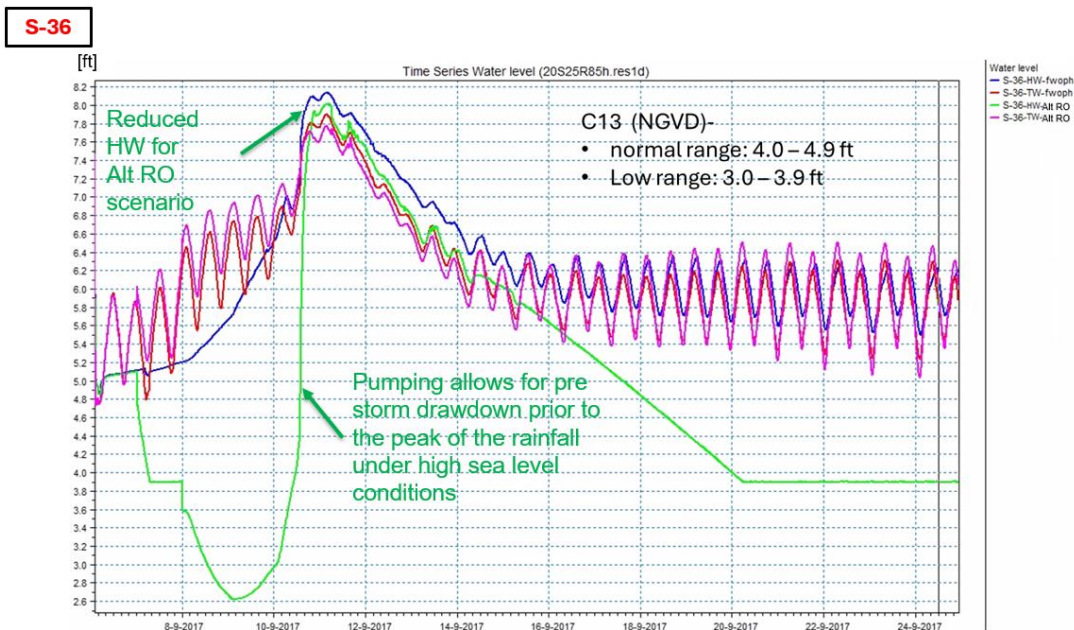


Figure 55 HW & TW at S36 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

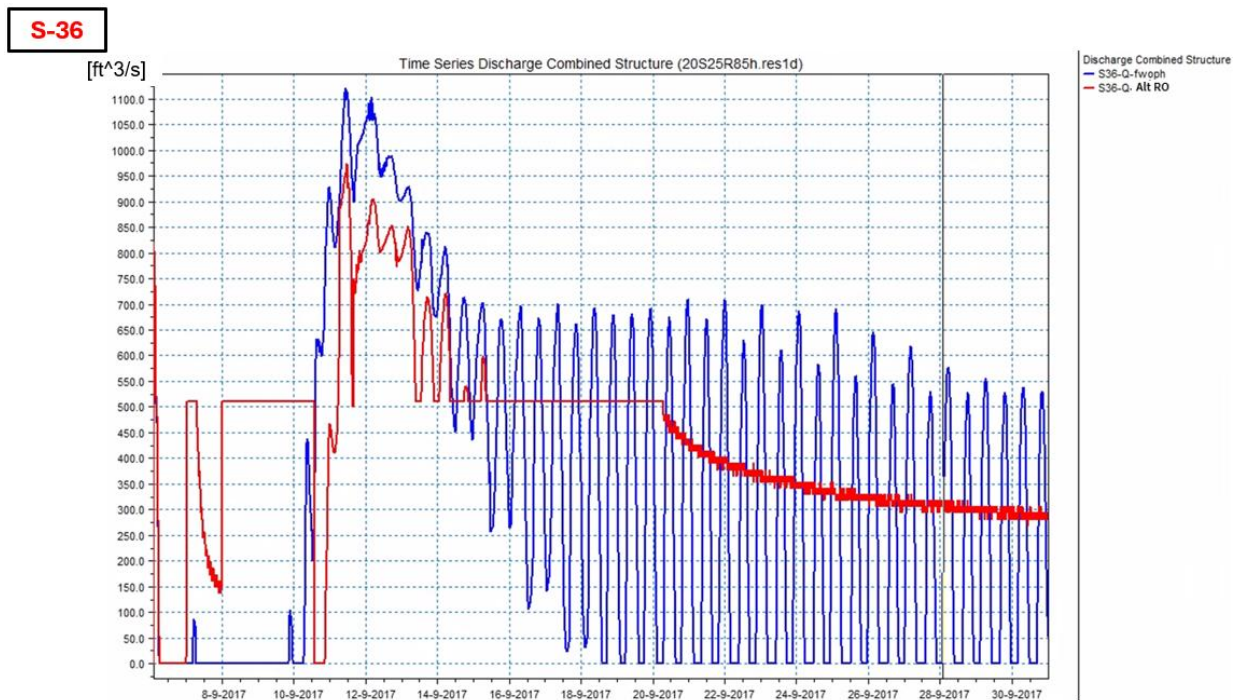


Figure 56 Flow at S36 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

Peak profile plots for three events are shown in Figure 57. The peak stages are improved from FWOP for all three events shown, and peak stage is slightly lowered just downstream



of the tidal structure, particularly noticeable under high sea level conditions for 20S25R85h. This is consistent with decrease in peak flows achieved by changing the timing of the discharge of runoff as illustrated in Figure 56. There is not a lot of out of bank flow and the modeling does not show impact to the bridge low chord elevations upstream of the S36 tidal structure for the events shown.

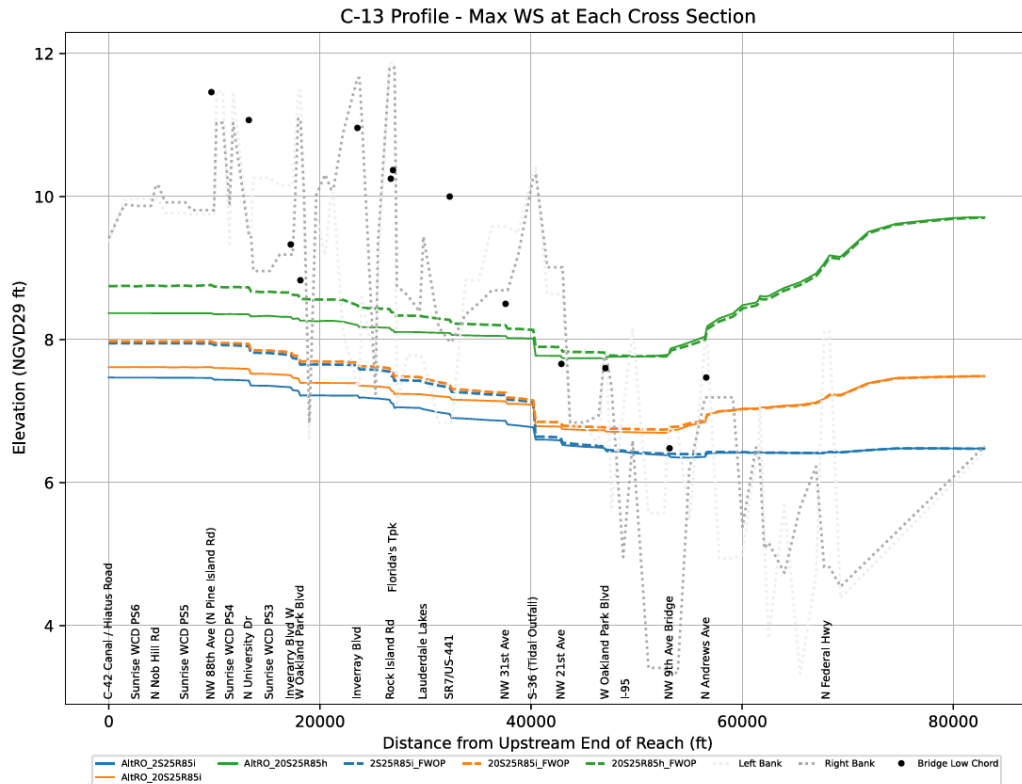


Figure 57 C-13 Canal Peak Profile – Alt RO vs. FWOP

Table 21 presents the peak moving average 12-hour flow (averaged over the 12-hour tide cycle). For more frequent events, 12-hour average flow decreases. Table 22 shows changes to the peak 12-hour HW when comparing Alt RO to FWOP and shows a decrease in headwater for all events with Alt RO features applied. When looking at the volume of water discharged throughout the three day event, all events are able to discharge more runoff with the Alt RO (Table 23). The improvement in runoff discharged is particularly noticed for high sea level conditions. The maps of Figure 58 illustrate improvements in the C-13W watershed flood depths for three selected events.

Table 21 S36 Structure Performance Table – Peak 12 hr Average Q * (CFS)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	754	635	744	608	720	598	607	510
2S10R	952	875	940	865	895	787	796	510
2S25R	1201	1260	1176	1212	1131	1083	976	685
10S10R	1008	943	975	921	947	926	767	578
20S25R	1291	1366	1262	1302	1231	1213	1054	853
2S100R	1514	1630	1499	1607	1481	1580	1137	1034
100S100R	1464	1605	1414	1531	1380	1435	1213	1208
2S500R	1835	2074	1812	1997	1731	1890	1336	1342
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	0.84	0.99	0.81	0.96	0.79	0.81	0.68
2S10R	1.00	0.92	0.99	0.91	0.94	0.83	0.84	0.54
2S25R	1.00	1.00	0.98	1.00	0.94	0.90	0.81	0.57
10S10R	1.00	0.94	0.97	0.91	0.94	0.92	0.76	0.57
20S25R	1.00	1.00	0.98	1.00	0.95	0.94	0.82	0.66
2S100R	1.00	1.00	0.99	1.00	0.98	1.00	0.75	0.68
100S100R	1.00	1.00	0.97	1.00	0.94	0.98	0.83	0.83
2S500R	1.00	1.00	0.99	1.00	0.94	1.00	0.73	0.73

*TSP/Recommended Plan = Alt RO

Table 22 S36 Structure Performance Table – Peak 12 hr HW * (ft NGVD29)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	5.20	4.81	5.59	5.29	6.15	5.81	7.60	6.53
2S10R	5.66	5.26	6.04	5.64	6.54	6.13	7.76	7.43
2S25R	6.36	5.80	6.63	6.21	7.01	6.50	7.99	7.84
10S10R	6.26	5.95	6.58	6.36	6.95	6.80	8.22	7.98
20S25R	6.57	6.20	6.82	6.59	7.12	6.81	8.09	7.94
2S100R	7.26	6.88	7.45	7.13	7.62	7.14	8.36	8.23
100S100R	7.69	7.53	7.85	7.71	8.07	7.97	8.94	8.90
2S500R	7.97	7.76	8.03	7.86	8.12	7.98	8.71	8.62
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	0.90	0.98	0.75	0.84	0.37	0.65
2S10R	1.00	1.00	0.89	1.00	0.74	0.86	0.37	0.47
2S25R	1.00	1.00	0.90	1.00	0.75	0.95	0.38	0.44
10S10R	1.00	1.00	0.88	0.96	0.75	0.80	0.28	0.37
20S25R	1.00	1.00	0.90	0.99	0.78	0.90	0.37	0.44
2S100R	1.00	1.00	0.89	1.00	0.79	1.00	0.37	0.44
100S100R	1.00	1.00	0.88	0.99	0.72	0.79	0.04	0.07
2S500R	1.00	1.00	0.94	1.00	0.85	0.99	0.28	0.37

*TSP/Recommended Plan = Alt RO



Table 23 S36 Structure Performance Table – Volume Discharged During the 3 Day Event * (Ac-ft)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1388	2646	1401	2536	1118	2420	427	2784
2S10R	1652	2895	1647	2797	1365	2662	665	2808
2S25R	2079	3356	2039	3194	1719	3295	1010	2820
10S10R	1439	2668	1367	2495	1092	2360	269	2415
20S25R	1874	3109	1860	3028	1621	3060	894	2663
2S100R	2730	3922	2657	3734	2399	3900	1490	3203
100S100R	2380	3364	2204	3074	1902	2805	383	1905
2S500R	3582	4665	3513	4468	3179	4216	2066	3432
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	0.81	1.00	0.31	1.00
2S10R	1.00	1.00	1.00	1.00	0.83	1.00	0.40	1.00
2S25R	1.00	1.00	0.98	1.00	0.83	1.00	0.49	1.00
10S10R	1.00	1.00	0.95	1.00	0.76	1.00	0.19	1.00
20S25R	1.00	1.00	0.99	1.00	0.87	1.00	0.48	1.00
2S100R	1.00	1.00	0.97	1.00	0.88	1.00	0.55	1.00
100S100R	1.00	1.00	0.93	1.00	0.80	1.00	0.16	0.80
2S500R	1.00	1.00	0.98	1.00	0.89	1.00	0.58	0.96

*TSP/Recommended Plan = Alt RO

** C-13 West drainage area = 23.9 sq. miles (SFWMD, 2021)



Figure 58 C-13 W Watershed Flood Depth Difference Maps in ft.



C-12, North Fork New River Canal/ S33

S33 spillway gravity capacity was increased from one 20 ft gate to two 20 ft gates, and pumping capability was added with 510 cfs capacity (Figure 59). Hardening of the structure by raising the top of gate and the overtopping elevation at the structure stopped the backflow at the gate during high tailwater conditions and especially at high sea level conditions. The hardening at this structure did not need to include raising canal cross section bank elevations at the structure.



Figure 59 Map of C-12 (North Fork New River) Canal Alt RO Features

Pumps were able to achieve pre storm drawdown even during high sea level conditions. Figure 60 illustrates the pump drawdown (green line) of the Alt RO HW under high sea level conditions (20S25R85h event). The figure also shows how the structure would not be able to operate the upstream canal in the normal range without the help of a pump if high sea level conditions are experienced and canal operating ranges are not increased. Figure shows the pump operating early in the storm when the spillway couldn't open under FWOP conditions of upstream and downstream water elevations. For this structure, instantaneous peak HW and flow is not significantly different. Cycling of the pump at the peak because of the downstream sensor cut off elevation would also decrease flow at the peak of the storm.

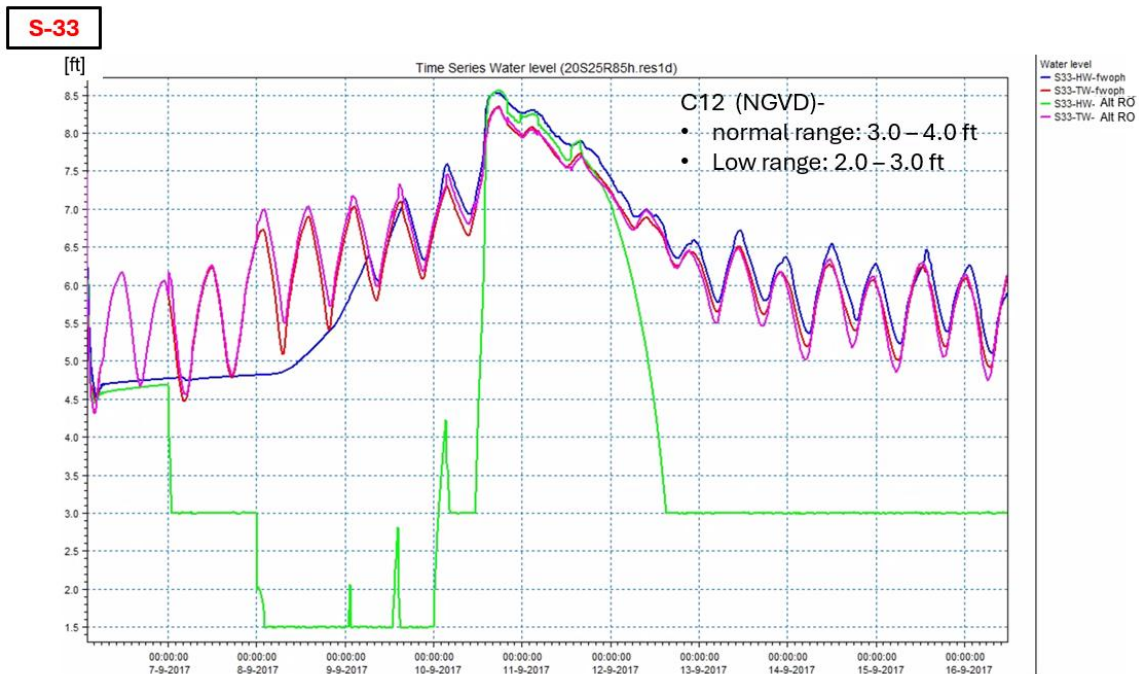


Figure 60 HW & TW at S33 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

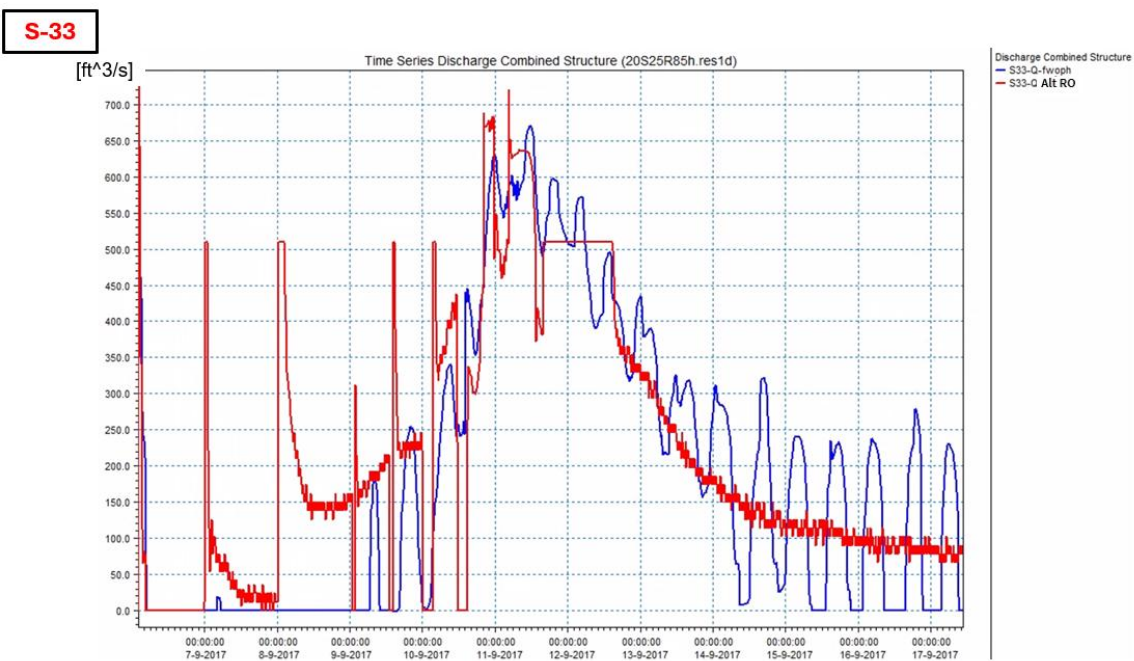


Figure 61 Flow at S33 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO



Peak profile plots for three events are shown in Figure 62. The peak stages are not significantly changed for all three events shown, although there is a slight decrease in peak stage upstream of S-33 for the intermediate sea level events.

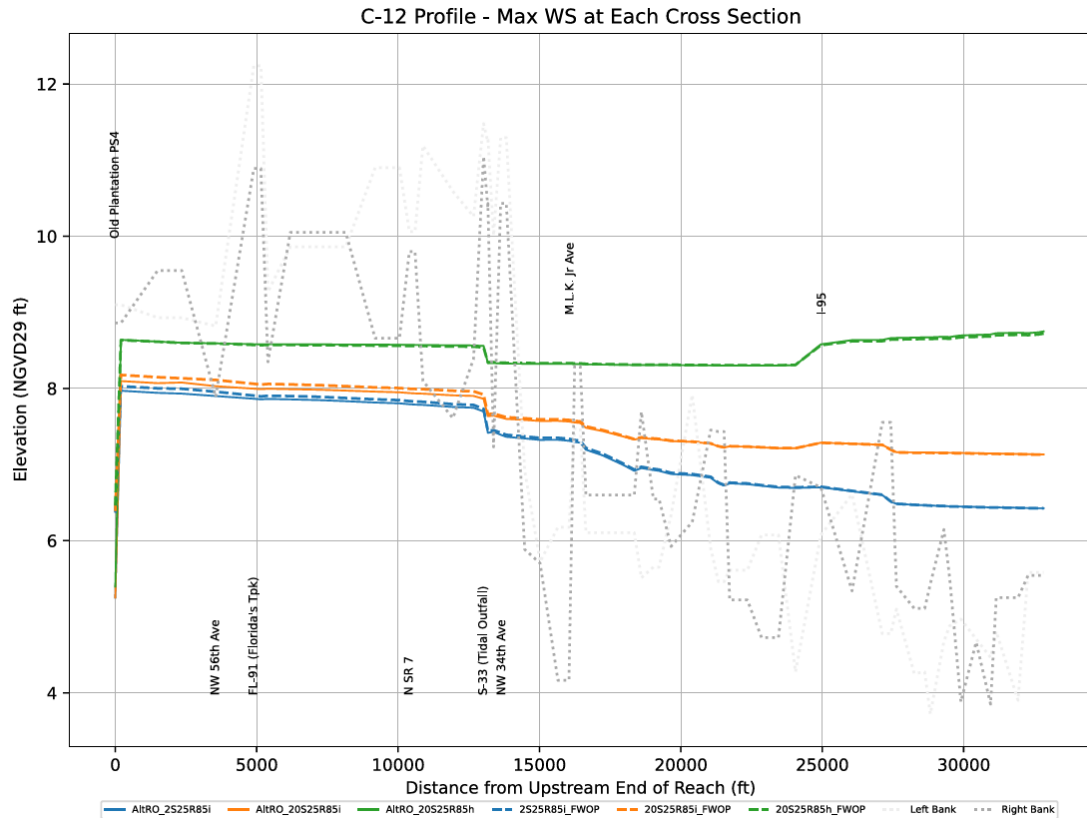


Figure 62 C-12 Canal Peak Profile – Alt RO vs. FWOP

Table 24 presents the peak moving average 12-hour flow (averaged over the 12-hour tide cycle), and Table 25 the peak 12-hour HW - both comparing Alt RO to FWOP and ECB. These numbers show small improvements in flow and water elevations and are not that significant. When looking at the volume of water discharged throughout the three day event, all events are able to discharge more runoff with the Alt RO (Table 26). The improvement in runoff discharged is particularly noticed for high sea level conditions. The results indicate that pre-storm drawdown operations create available storage in the watershed which alleviates flooding. The drawdown can occur with the operational flexibility of a pump that can discharge under conditions that would not allow gravity discharge. The maps of Figure 63 illustrate improvements in the C-12 W watershed flood depths for three selected events.

Table 24 S33 Structure Performance Table – Peak 12 hr Average Q * (CFS)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	556	605	562	582	558	556	449	366
2S10R	680	706	682	703	666	675	538	454
2S25R	772	819	767	813	749	755	600	572
10S10R	653	684	644	667	613	612	423	494
20S25R	767	807	757	790	731	752	600	591
2S100R	819	913	836	889	833	900	635	651
100S100R	777	839	756	803	725	745	640	644
2S500R	882	991	862	963	825	918	734	720
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	1.00	1.00	0.81	0.66
2S10R	1.00	1.00	1.00	1.00	0.98	0.99	0.79	0.67
2S25R	1.00	1.00	0.99	1.00	0.97	0.98	0.78	0.74
10S10R	1.00	1.00	0.99	1.00	0.94	0.94	0.65	0.76
20S25R	1.00	1.00	0.99	1.00	0.95	0.98	0.78	0.77
2S100R	1.00	1.00	1.00	1.00	1.00	1.00	0.77	0.79
100S100R	1.00	1.00	0.97	1.00	0.93	0.96	0.82	0.83
2S500R	1.00	1.00	0.98	1.00	0.94	1.00	0.83	0.82

*TSP/Recommended Plan = Alt RO

Table 25 S33 Structure Performance Table – Peak 12 hr Average HW * (ft NGVD29)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	5.67	5.57	6.02	6.02	6.53	6.48	7.89	7.78
2S10R	6.23	6.18	6.52	6.50	6.91	6.87	8.06	7.97
2S25R	6.79	6.76	7.01	7.00	7.32	7.28	8.25	8.18
10S10R	6.75	6.76	7.02	7.06	7.40	7.41	8.45	8.44
20S25R	7.07	7.06	7.30	7.30	7.58	7.50	8.39	8.35
2S100R	7.54	7.57	7.59	7.78	7.81	7.74	8.49	8.45
100S100R	7.93	8.00	8.08	8.15	8.29	8.31	9.07	9.05
2S500R	8.41	8.26	8.47	8.36	8.55	8.45	8.81	8.77
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	0.92	0.92	0.80	0.81	0.49	0.51
2S10R	1.00	1.00	0.92	0.93	0.82	0.83	0.52	0.54
2S25R	1.00	1.00	0.93	0.93	0.83	0.85	0.54	0.57
10S10R	1.00	1.00	0.92	0.90	0.80	0.80	0.48	0.48
20S25R	1.00	1.00	0.92	0.92	0.83	0.85	0.55	0.56
2S100R	1.00	0.99	0.98	0.90	0.89	0.92	0.61	0.63
100S100R	1.00	0.97	0.93	0.90	0.82	0.82	0.45	0.46
2S500R	1.00	1.00	0.96	1.00	0.91	0.97	0.75	0.77

*TSP/Recommended Plan = Alt RO



Table 26 S33 Structure Performance Table – Volume Discharged During the 3 Day Event * (ac-ft) **

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	716	1219	731	1207	703	1195	567	1106
2S10R	905	1425	914	1422	886	1409	741	1279
2S25R	1138	1681	1147	1681	1112	1688	925	1539
10S10R	841	1352	834	1344	792	1301	510	1028
20S25R	1093	1620	1094	1618	1050	1631	844	1433
2S100R	1434	2071	1470	2043	1453	2129	1214	1857
100S100R	1334	1913	1318	1882	1237	1797	543	1115
2S500R	1782	2500	1792	2438	1749	2384	1674	2163
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	0.98	1.00	0.79	1.00
2S10R	1.00	1.00	1.00	1.00	0.98	1.00	0.82	1.00
2S25R	1.00	1.00	1.00	1.00	0.98	1.00	0.81	1.00
10S10R	1.00	1.00	0.99	1.00	0.94	1.00	0.61	1.00
20S25R	1.00	1.00	1.00	1.00	0.96	1.00	0.77	1.00
2S100R	1.00	1.00	1.00	1.00	1.00	1.00	0.85	1.00
100S100R	1.00	1.00	0.99	1.00	0.93	1.00	0.41	0.84
2S500R	1.00	1.00	1.00	1.00	0.98	1.00	0.94	1.00

*TSP/Recommended Plan = Alt RO

** C-12 West drainage area = 7.5 sq. miles (SFWMD, 2021)

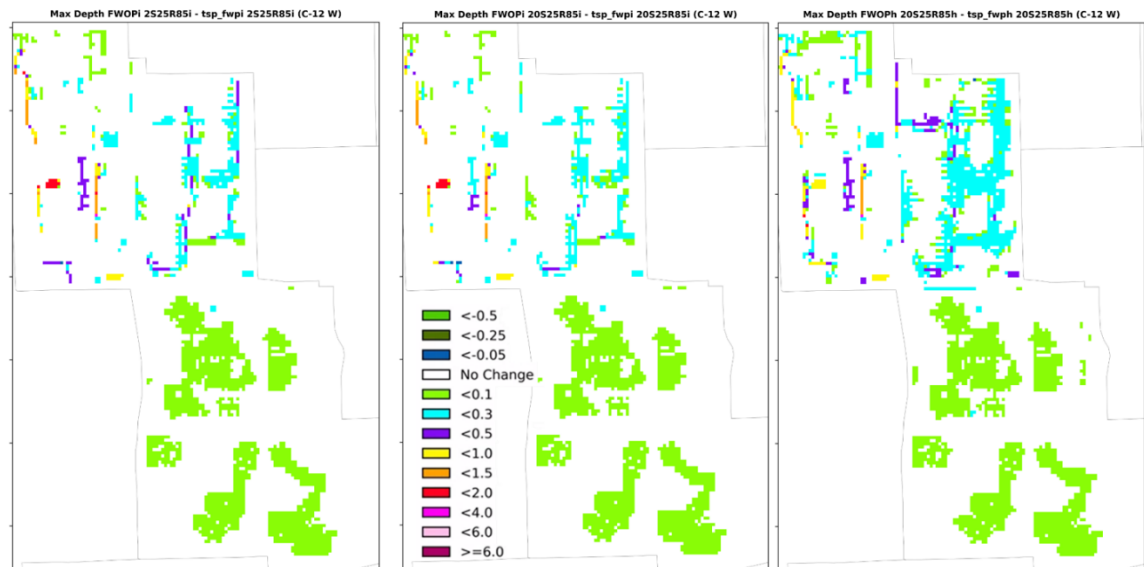


Figure 63 C-12 W Watershed Flood Depth Difference Maps in ft.

North New River Canal/ G54

G54 spillway gravity capacity was increased from three 16 ft gates to four 20 ft gates, and pumping capability was added with 810 cfs capacity (Figure 64). Hardening of the structure by raising the top of gate and the overtopping elevation at the structure stopped the backflow at the gate during high tailwater conditions and especially at high sea level conditions. The hardening at this structure also had to include raising canal cross section bank elevations in the vicinity of the structure.



Figure 64 Map of North New River Canal Alt RO Features

Pumping was able to achieve pre storm drawdown even during high sea level conditions. Figure 65 illustrates the pump drawdown of the Alt RO HW under high sea level conditions (20S25R85h event). The figure also shows how the structure would not be able to operate the upstream canal in the normal range without the help of a pump if high sea level conditions are experienced and canal operating ranges are not increased. Peak instantaneous HW is decreased under Alt RO conditions. Figure 66 shows the 810 cfs pump operating early in the storm when the spillway couldn't open under FWOP conditions of upstream and downstream water elevations. In Figure 66, FWOP flow at G54 also shows backflow occurring prior to the peak of the storm for FWOP, but no backflow is occurring after hardening of the structure in the Alt RO. Cycling of the pump occurs because of the downstream sensor cut off elevation.

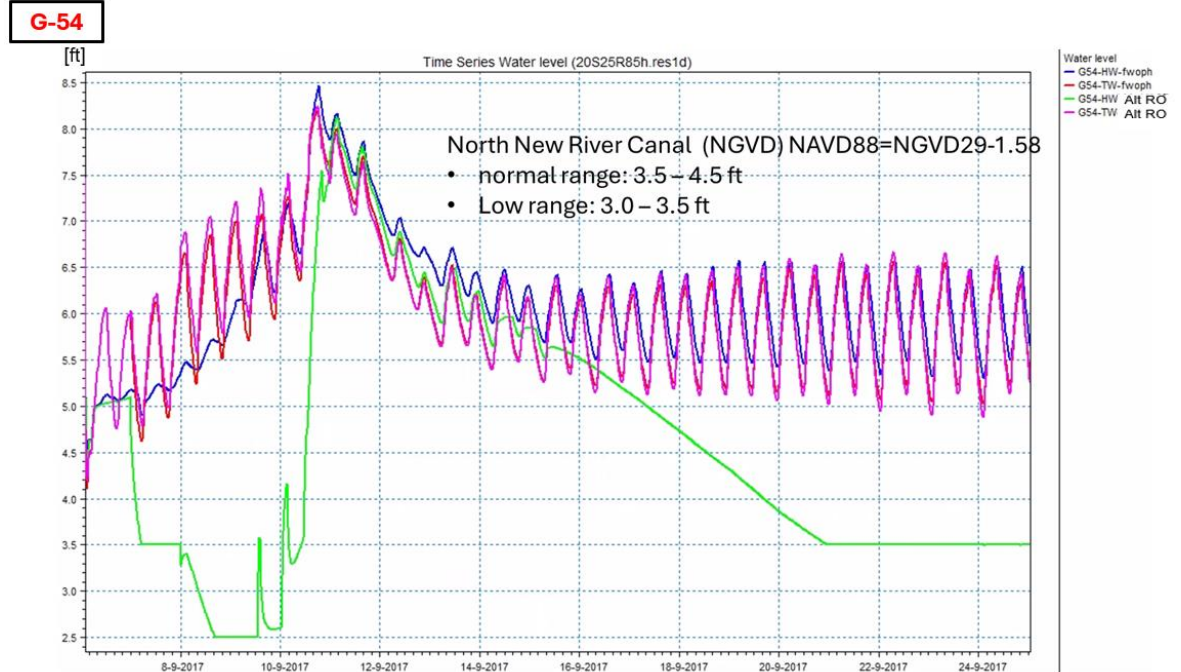


Figure 65 HW & TW at G54 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

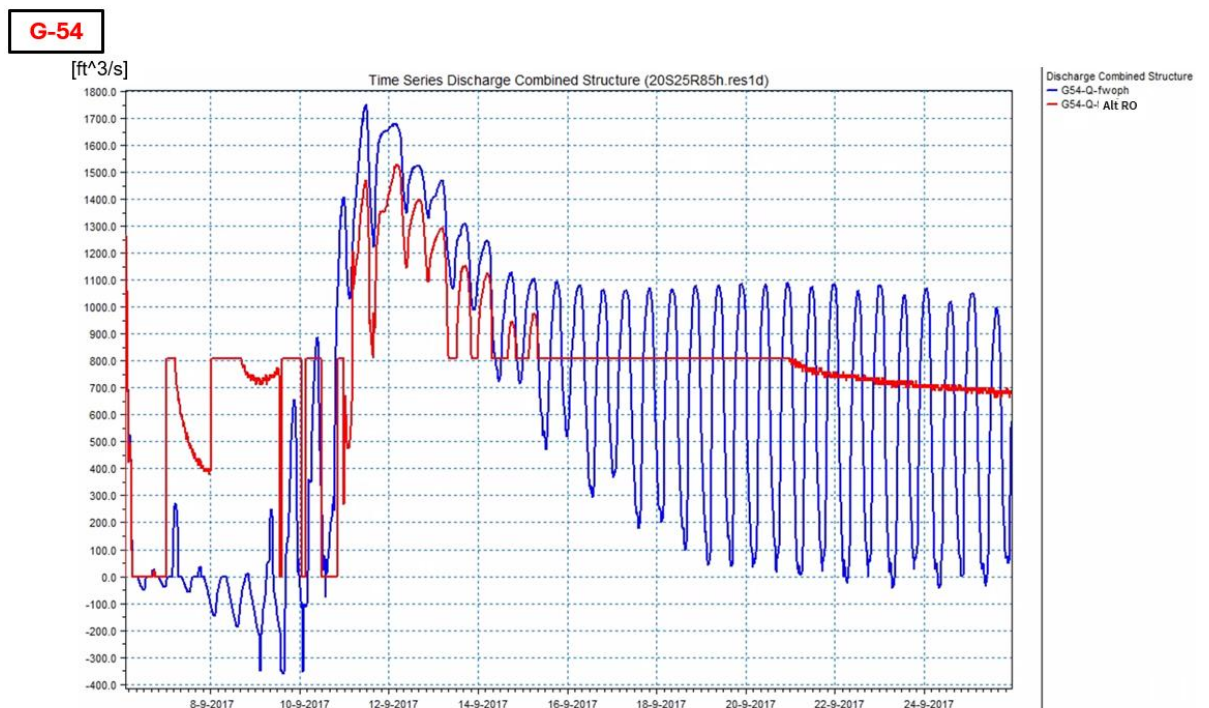


Figure 66 Flow at G54 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

Peak profile plots for three events are shown in Figure 67. The peak stages are improved upstream of the G54 structure for all three events shown. No low chord of bridges are impacted by the events shown.

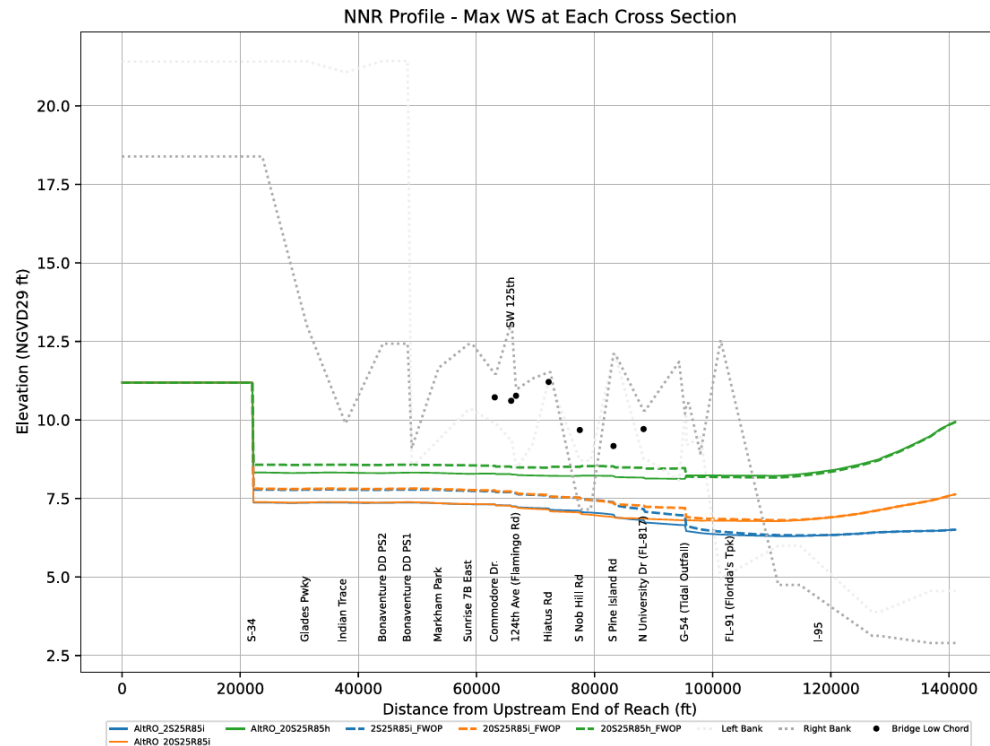


Figure 67 North New River Peak Profile – Alt RO vs. FWOP

Table 27 presents the peak moving average 12-hour flow (averaged over the 12-hour tide cycle), both comparing Alt RO to FWOP and ECB. The peak 12-hour flow is often lower than the same event Without Project due to the timing of the discharge – the pump can discharge earlier during the event to drawdown and create storage. Peak 12-hour flow can be affected by cycling of the pump when downstream sensors reach cutoff levels. Table 28Table 16 shows the peak 12 hour HW where Alt RO generally improves the headwater of the same event without project, but for intermediate and high sea levels does not produce headwater as low as the ECB. The only exception is the 20S25R85i event. When looking at the volume of water discharged throughout the three day event, all events are able to discharge more runoff with the Alt RO (Table 29) than for FWOP. Only two events, 100S100R85h and 2S500R85h discharge volume less than the ECB simulation. The maps of Figure 68Figure 50 illustrate improvements in the North New River Canal West watershed flood depths for three selected events.



Table 27 G54 Structure Performance Table – Peak 12 hr Average Q * (cfs)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1144	950	1141	906	1133	871	952	810
2S10R	1400	1149	1402	1160	1399	1092	1160	811
2S25R	1745	1631	1737	1654	1702	1454	1353	1012
10S10R	1448	1305	1425	1309	1390	1259	1375	1123
20S25R	1865	1780	1851	1814	1811	1618	1655	1437
2S100R	2083	2177	2044	2129	1981	2029	1559	1462
100S100R	2132	2283	2082	2252	2023	2160	1887	1791
2S500R	2288	2519	2240	2429	2197	2316	2000	1975
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	0.83	1.00	0.79	0.99	0.76	0.83	0.71
2S10R	1.00	0.82	1.00	0.83	1.00	0.78	0.83	0.58
2S25R	1.00	0.93	0.99	0.95	0.98	0.83	0.77	0.58
10S10R	1.00	0.90	0.98	0.90	0.96	0.87	0.95	0.78
20S25R	1.00	0.95	0.99	0.97	0.97	0.87	0.89	0.77
2S100R	1.00	1.00	0.98	1.00	0.95	0.97	0.75	0.70
100S100R	1.00	1.00	0.98	1.00	0.95	1.00	0.89	0.84
2S500R	1.00	1.00	0.98	1.00	0.96	1.00	0.87	0.86

*TSP/Recommended Plan = Alt RO

Table 28 G54 Structure Performance Table – Peak 12 hr Average HW * (ft NGVD29)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	4.96	4.37	5.37	4.96	5.98	5.58	7.63	6.75
2S10R	5.38	4.79	5.77	5.24	6.33	5.84	7.78	7.54
2S25R	6.01	5.20	6.34	5.63	6.74	6.15	7.93	7.81
10S10R	6.05	5.58	6.38	0.00	6.85	6.61	8.40	7.99
20S25R	6.20	5.57	6.52	5.93	6.89	6.13	8.14	7.77
2S100R	6.56	5.89	6.79	6.25	7.05	6.64	8.13	8.02
100S100R	7.24	7.07	7.47	7.39	7.87	7.81	8.86	8.78
2S500R	7.11	6.50	7.18	6.76	7.40	7.12	8.43	8.32
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	0.90	1.00	0.75	0.85	0.34	0.56
2S10R	1.00	1.00	0.89	1.00	0.74	0.87	0.34	0.40
2S25R	1.00	1.00	0.89	1.00	0.76	0.95	0.36	0.40
10S10R	1.00	1.00	0.89	1.00	0.73	0.81	0.20	0.34
20S25R	1.00	1.00	0.89	1.00	0.75	1.00	0.31	0.44
2S100R	1.00	1.00	0.91	1.00	0.80	0.97	0.36	0.40
100S100R	1.00	1.00	0.87	0.92	0.65	0.68	0.08	0.13
2S500R	1.00	1.00	0.96	1.00	0.85	0.99	0.30	0.36

*TSP/Recommended Plan = Alt RO



Table 29 G54 Structure Performance Table – Volume Discharged During the 3 Day Event * (ac-ft) **

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	2484	3897	2250	3705	1807	3560	340	3661
2S10R	2761	4142	2536	3932	2101	3782	656	3680
2S25R	3185	4520	2944	4325	2506	4561	1041	3776
10S10R	2485	3782	2222	3598	1798	3429	41	3121
20S25R	3011	4234	2857	4189	2427	4471	978	3826
2S100R	3704	5064	3488	4808	3107	5049	1623	3937
100S100R	3101	4295	2817	3888	2278	3467	-329	2307
2S500R	4553	5803	4389	5578	4030	5286	2405	4126
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	0.91	1.00	0.73	1.00	0.14	1.00
2S10R	1.00	1.00	0.92	1.00	0.76	1.00	0.24	1.00
2S25R	1.00	1.00	0.92	1.00	0.79	1.00	0.33	1.00
10S10R	1.00	1.00	0.89	1.00	0.72	1.00	0.02	1.00
20S25R	1.00	1.00	0.95	1.00	0.81	1.00	0.32	1.00
2S100R	1.00	1.00	0.94	1.00	0.84	1.00	0.44	1.00
100S100R	1.00	1.00	0.91	1.00	0.73	1.00	-0.11	0.74
2S500R	1.00	1.00	0.96	1.00	0.89	1.00	0.53	0.91

*TSP/Recommended Plan = Alt RO

** North New River Canal West drainage area = 27.9 sq. miles (SFWMD, 2021)

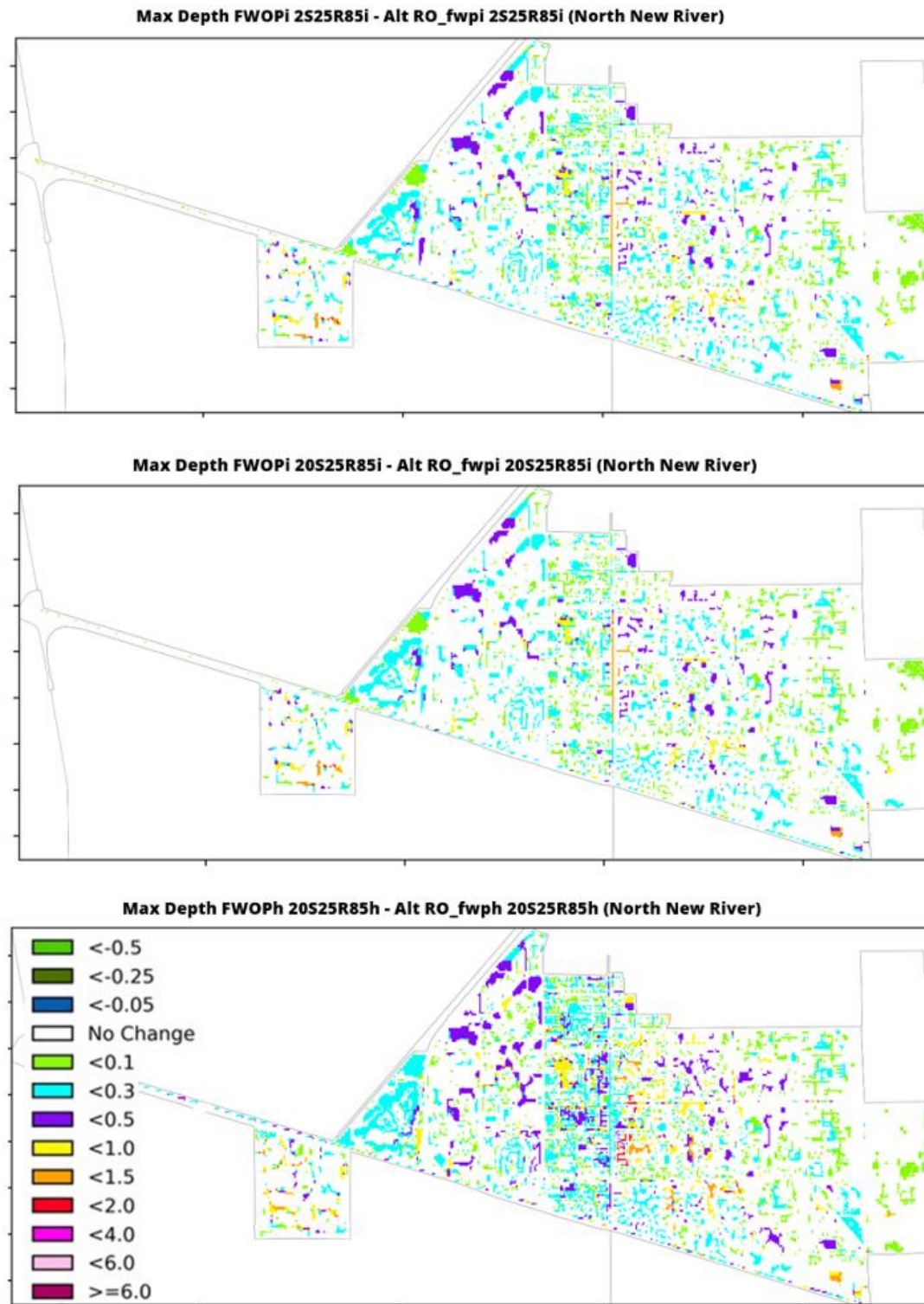


Figure 68 North New River Canal West Watershed Flood Depth Difference Maps in ft.



C-11 Canal/ S13

S13 tidal structure pumping capacity was increased from 540 cfs to 1080 cfs and spillway was changed from one 16 ft gate to two 14 ft gates (Figure). Hardening of the structure by raising the top of gate and the overtopping elevation at the structure stopped the backflow at the gate during high tailwater conditions and especially at high sea level conditions. To stop backflow and flow around the structure, the cross-section banks had to be elevated in the vicinity of the structure.

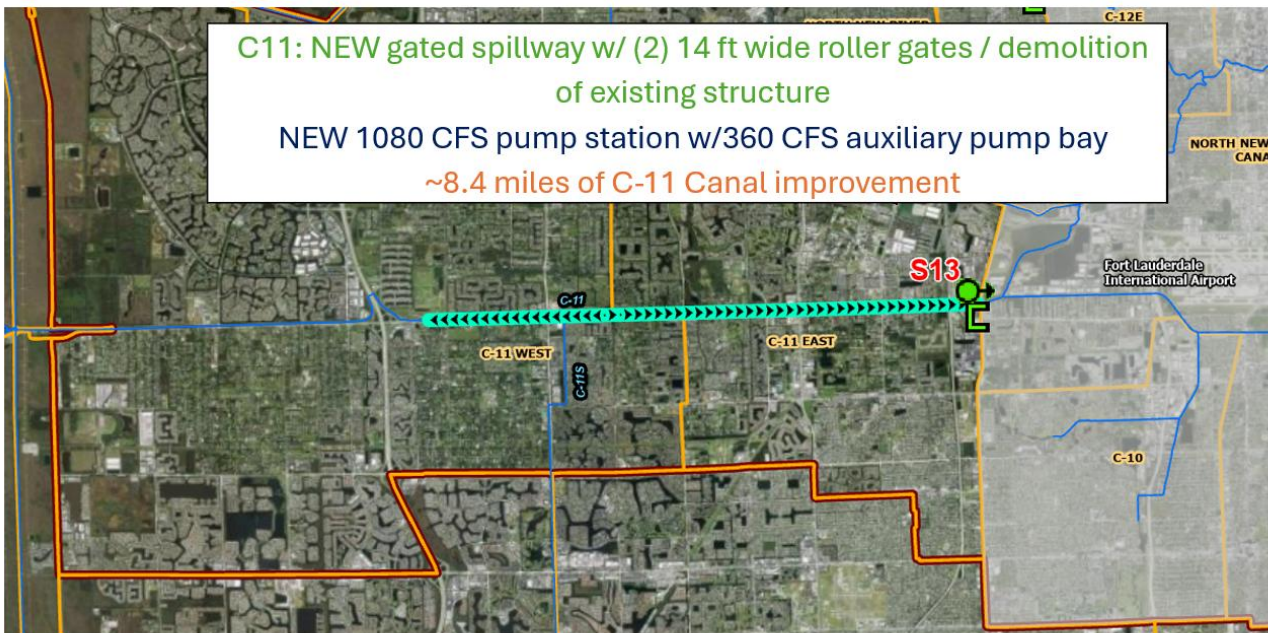


Figure 69 Map of C-11 Canal Alt RO Features

The S13 structure discharges runoff from only the C-11 E watershed. The S13AW is a divide structure that was modeled closed for events. (In one of the initial array alternatives (Alternative 5) the structure was open and benefits were not seen). The canal operational range between S13 and S13AW is relatively low and to maintain the range, the structure already has a pump to augment the spillway. Figure 70 shows the existing pump in FWOP can maintain low canal levels prior to the rising tides, but with runoff combined with rising tailwater, the pumped system struggles to maintain low range, and after the storm, returning to normal range takes roughly two weeks. Increasing the pump capacity allowed for drawdown prior to the storm and quicker recovery after the storm to reach canal operating range as illustrated for the high sea level event of Figure 70. Figure 71 shows that before hardening (FWOP) the watershed would see backflow under normal operating conditions and only positive net flow during the peak of the storm. For this structure,

instantaneous peak HW and flow is significantly different during high sea level conditions when comparing with and without project.

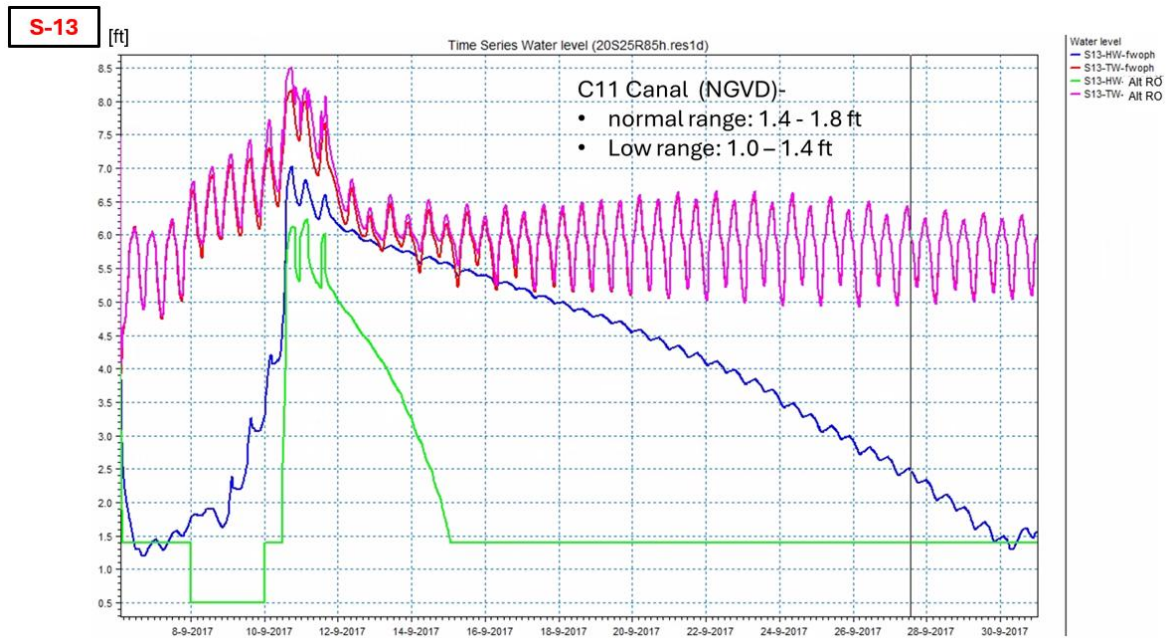


Figure 70 HW & TW at S13 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO

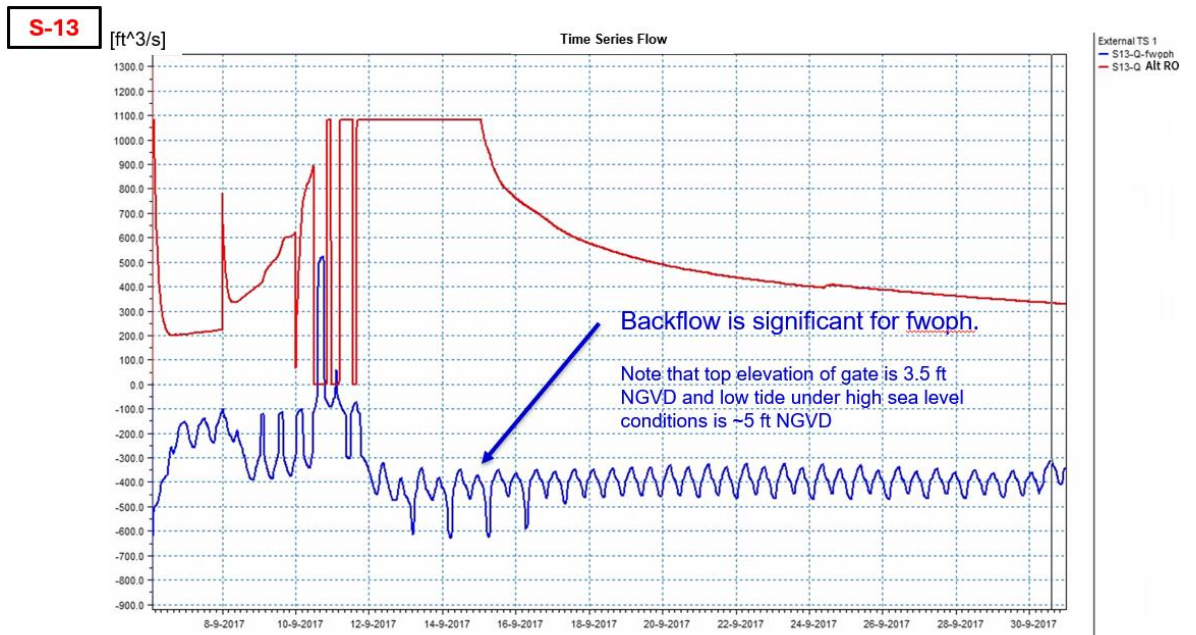


Figure 71 Flow at S13 for 20S25R85h (High Sea Level) – FWOP vs. Alt RO



Peak profile plots for three events are shown in Figure 72. Under high sea level conditions, the peak profile for FWOP is higher since it is experiencing inflow over the top of the gate from high tailwater conditions. The existing configuration of this structure already includes a pump so the water is continuously being pumped against the backflow entering the watershed, but the pump is not effective in drawing down to low range (Figure 70). For intermediate sea level, more water is moved to the primary system with assumed improvements to secondary but the pumps are shut off based on downstream sensor cut offs. Overall, the watershed experiences benefits when analyzed through the economic tool HEC FDA. The modeling turned the pumps off based on the sensor cut off, even the existing pump. This operation could be refined.

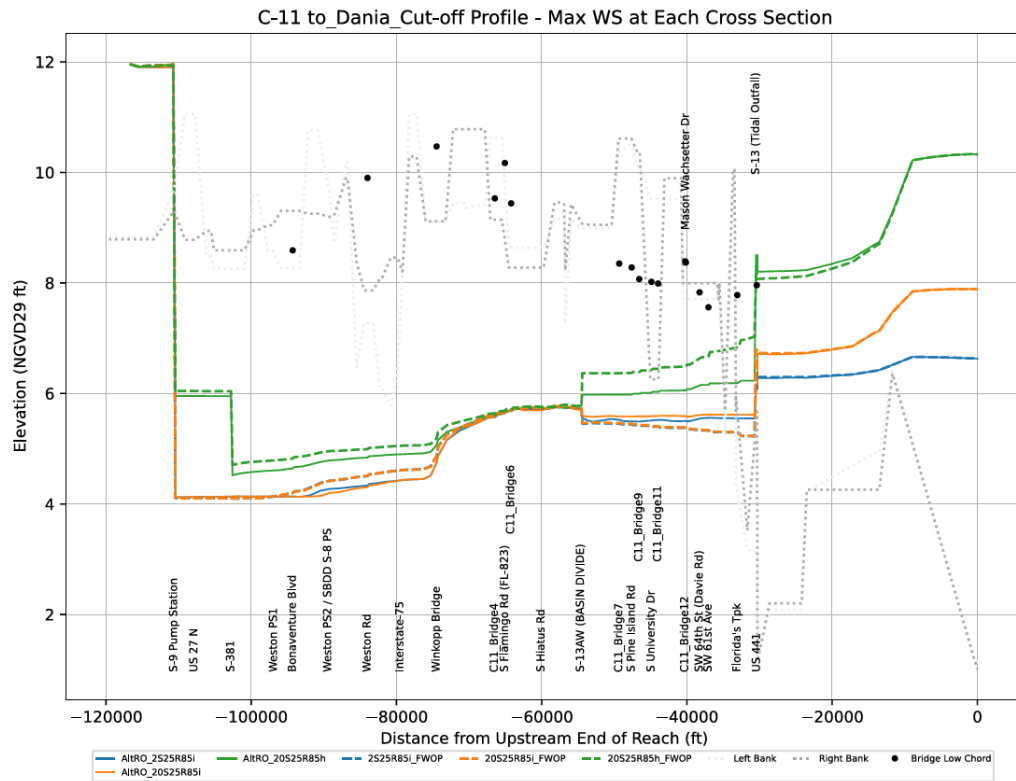


Figure 72 C-11 Canal Peak Profile – Alt RO vs. FWOP

Table 30 S13 Structure Performance Table – Peak 12 hr Average Q * (CFS)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPI	TSP	FWOPH	TSP
2S5R	533	984	531	788	536	817	412	1080
2S10R	600	1043	568	905	539	929	429	1080
2S25R	722	1080	691	1080	691	1080	491	1080
10S10R	800	1080	805	1080	728	1080	486	1080
20S25R	1027	1080	1011	1080	923	1080	472	1080
2S100R	986	1151	999	1141	1015	1080	626	1080
100S100R	1190	1368	1226	1316	1266	1336	781	1080
2S500R	1357	1503	1363	1465	1347	1499	884	1080
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPI	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	1.00	1.00	0.77	1.00
2S10R	1.00	1.00	0.95	1.00	0.90	1.00	0.72	1.00
2S25R	1.00	1.00	0.96	1.00	0.96	1.00	0.68	1.00
10S10R	1.00	1.00	1.00	1.00	0.91	1.00	0.61	1.00
20S25R	1.00	1.00	0.98	1.00	0.90	1.00	0.46	1.00
2S100R	1.00	1.00	1.00	1.00	1.00	1.00	0.63	1.00
100S100R	1.00	1.00	1.00	1.00	1.00	1.00	0.66	0.91
2S500R	1.00	1.00	1.00	1.00	0.99	1.00	0.65	0.80

*TSP/Recommended Plan = Alt RO



Table 31 S13 Structure Performance Table – Peak 12 hr Average HW * (ft NGVD29)

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	3.67	2.24	3.90	3.15	4.21	3.52	6.14	4.79
2S10R	4.18	3.25	4.45	3.91	4.70	4.19	6.35	5.23
2S25R	4.60	4.22	4.90	4.64	5.16	3.92	6.65	5.74
10S10R	4.48	4.45	4.66	5.25	4.88	5.37	7.23	6.39
20S25R	4.80	4.55	4.95	4.96	5.16	4.41	6.72	5.94
2S100R	5.18	5.10	5.43	5.43	5.79	4.93	7.09	6.35
100S100R	5.83	6.09	6.03	6.23	6.49	6.42	8.16	7.64
2S500R	5.89	5.86	6.08	6.15	6.29	6.36	7.55	7.01
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	0.96	1.00	0.90	1.00	0.54	0.79
2S10R	1.00	1.00	0.95	1.00	0.89	1.00	0.55	0.78
2S25R	1.00	1.00	0.93	0.99	0.87	1.00	0.54	0.74
10S10R	1.00	1.00	0.96	0.83	0.91	0.80	0.39	0.58
20S25R	1.00	1.00	0.96	0.96	0.91	1.00	0.54	0.73
2S100R	1.00	1.00	0.93	0.93	0.84	1.00	0.50	0.69
100S100R	1.00	0.92	0.94	0.87	0.79	0.81	0.26	0.43
2S500R	1.00	1.00	0.94	0.92	0.87	0.85	0.47	0.64

*TSP/Recommended Plan = Alt RO

Table 32 S13 Structure Performance Table – Volume Discharged During the 3 Day Event * (ac-ft) **

Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1404	2142	1497	2084	1586	2122	1325	2755
2S10R	1592	2298	1649	2218	1727	2258	1289	2887
2S25R	1911	2506	1885	2409	1899	2998	1201	3069
10S10R	1478	1748	1559	1406	1633	1506	735	2086
20S25R	1748	2215	1789	2086	1832	2688	1185	2665
2S100R	2352	2873	2311	2763	2113	3442	1054	3238
100S100R	1913	1849	1837	1807	1722	1774	37	2054
2S500R	2717	3451	2526	3132	2352	2973	875	3300
Event	2035i		2085L		2085i		2085h	
	ECB	TSP	FWOPL	TSP	FWOPi	TSP	FWOPH	TSP
2S5R	1.00	1.00	1.00	1.00	1.00	1.00	0.94	1.00
2S10R	1.00	1.00	1.00	1.00	1.00	1.00	0.81	1.00
2S25R	1.00	1.00	0.99	1.00	0.99	1.00	0.63	1.00
10S10R	1.00	1.00	1.00	0.95	1.00	1.00	0.50	1.00
20S25R	1.00	1.00	1.00	1.00	1.00	1.00	0.68	1.00
2S100R	1.00	1.00	0.98	1.00	0.90	1.00	0.45	1.00
100S100R	1.00	0.97	0.96	0.94	0.90	0.93	0.02	1.00
2S500R	1.00	1.00	0.93	1.00	0.87	1.00	0.32	1.00

*TSP/Recommended Plan = Alt RO

** C-11 East drainage area = 26.5 sq. miles

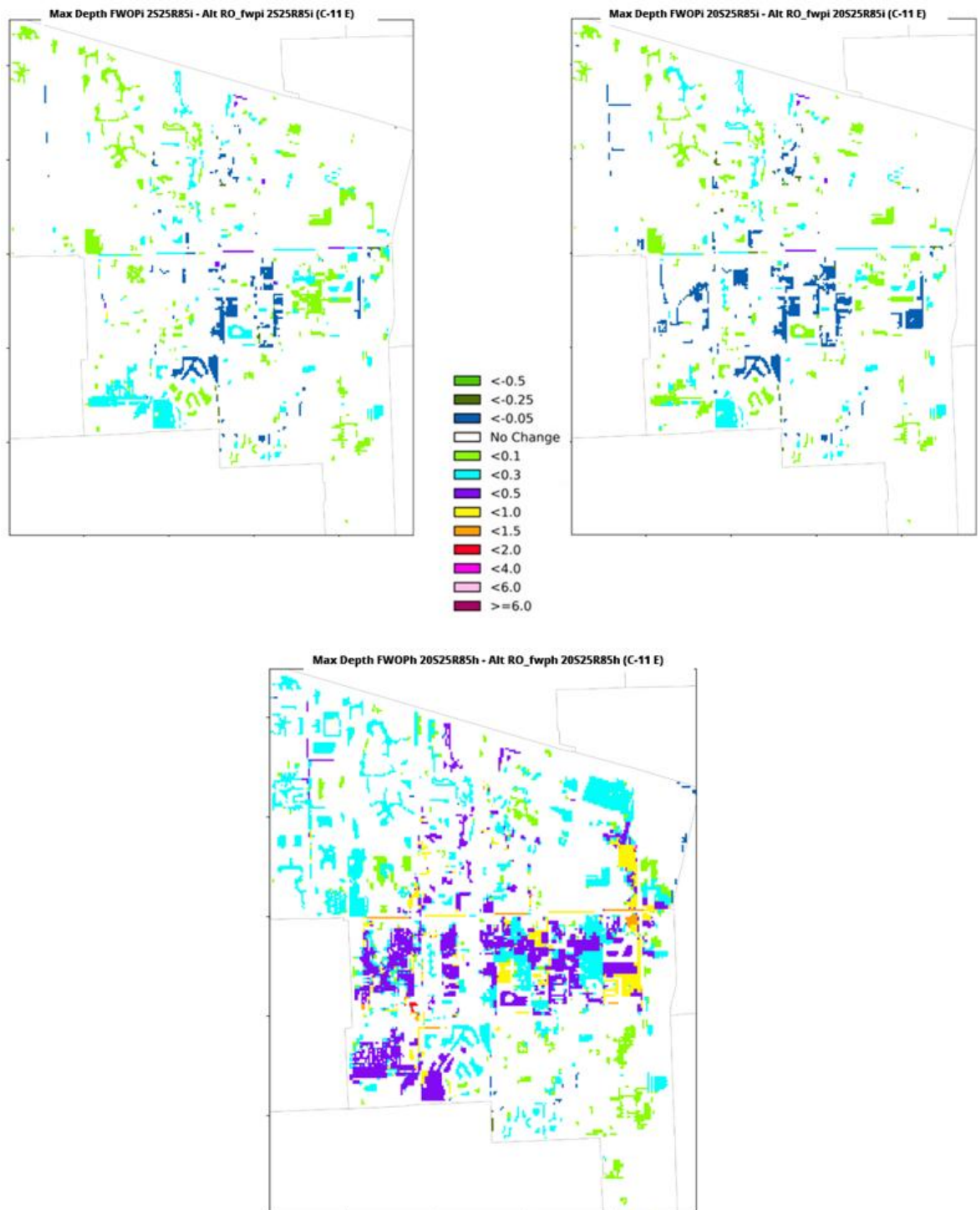


Figure 73 C-11 E Watershed Flood Depth Difference Maps in ft.

Combining and summarizing results of each structure's performance metrics yielded a measure of effectiveness as presented in Table 33 for all eight structures. The process normalizes the values to ECB – meaning a value of 1.0 for the alternatives maintains the ECB performance. Less than 1.0 indicates the metric is degraded from ECB performance. Table 33 uses the normalized values averaged over the eight events for each Alternative's score rather than reporting for each event. For each event, see the scores in the Performance Tables for each drainage area (Table 9 through Table 32).

The final Structure Effectiveness Table tabulates by Volume (volume discharged during the 3-day event), and by Max Avg HW (peak 12-hour average HW) with Combined being their average score. The peak 12-hour average Q as presented for each drainage area was not included in the Effectiveness Table because not all structures showed improvements in peak 12-hour flow. The metric was not considered a high indicator of effectiveness since the peak 12-hour average flow was influenced by sensor cut off operations of the pumps and any additional gravity features which were turned off sometimes at the peak of the storm.

Normalizing by Volume and by Max Avg HW to ECB yields a measure of how well the alternative features can maintain 2035 intermediate sea level performance under future conditions. In general, the Table shows increasing effectiveness from Alt A to Alt C and the TSP/Recommended Plan (Alt RO) to be an optimized improvement over the FWOP, as multiple project considerations allows.

Table 33 Structure Effectiveness Table for All Reach A Structures, Normalized to ECB (TSP/Recommended Plan = AltRO)

Structure	Metric	2035i					2085L					2085i					2085h				
		ECB	TSP	AltA	AltB	AltC	FWOPL	TSP	AltA	AltB	AltC	FWOPI	TSP	AltA	AltB	AltC	FWOPH	TSP	AltA	AltB	AltC
G-54	by Volume	1.00	1.00	1.00	1.00	1.00	0.93	1.00	1.00	0.99	1.00	0.78	1.00	0.98	0.81	1.00	0.24	1.00	0.45	0.20	1.00
	by Max Avg HW	1.00	0.95	1.00	1.00	1.00	0.93	0.91	0.94	0.98	0.99	0.82	0.84	0.83	0.88	0.96	0.48	0.38	0.49	0.50	0.71
	Combined	1.00	0.98	1.00	1.00	1.00	0.93	0.96	0.97	0.98	1.00	0.80	0.92	0.90	0.84	0.98	0.36	0.69	0.47	0.35	0.85
G-56	by Volume	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	1.00	1.00	1.00	1.00	0.87	1.00	1.00	1.00	1.00
	by Max Avg HW	1.00	0.98	0.94	0.95	0.94	0.96	0.94	0.89	0.92	0.90	0.88	0.86	0.82	0.85	0.84	0.56	0.56	0.53	0.55	0.57
	Combined	1.00	0.99	0.97	0.98	0.97	0.98	0.97	0.95	0.96	0.95	0.93	0.93	0.91	0.92	0.92	0.71	0.78	0.77	0.77	0.78
G-57	by Volume	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.96	0.98	1.00	1.00	1.00	0.77	0.65	0.73	0.69	1.00
	by Max Avg HW	1.00	1.00	0.98	0.99	0.99	0.92	0.92	0.90	0.91	0.91	0.80	0.80	0.78	0.79	0.81	0.44	0.43	0.41	0.42	0.51
	Combined	1.00	1.00	0.99	0.99	0.99	0.96	0.96	0.95	0.95	0.96	0.88	0.89	0.89	0.89	0.90	0.60	0.54	0.57	0.55	0.75
S-13	by Volume	1.00	1.00	1.00	1.00	1.00	0.98	0.99	1.00	1.00	1.00	0.96	0.99	1.00	1.00	1.00	0.54	1.00	1.00	1.00	1.00
	by Max Avg HW	1.00	0.99	1.00	1.00	1.00	0.95	0.94	0.99	1.00	1.00	0.87	0.93	0.97	1.00	1.00	0.47	0.67	0.74	0.86	0.92
	Combined	1.00	0.99	1.00	1.00	1.00	0.96	0.96	0.99	1.00	1.00	0.92	0.96	0.99	1.00	1.00	0.51	0.84	0.87	0.93	0.96
S-33	by Volume	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.97	1.00	1.00	1.00	1.00	0.75	0.98	1.00	1.00	1.00
	by Max Avg HW	1.00	0.99	0.99	0.99	1.00	0.93	0.93	0.95	0.95	0.97	0.84	0.86	0.87	0.87	0.94	0.55	0.57	0.66	0.67	0.84
	Combined	1.00	1.00	0.99	1.00	1.00	0.97	0.96	0.97	0.97	0.99	0.90	0.93	0.93	0.94	0.97	0.65	0.77	0.83	0.83	0.92
S-36	by Volume	1.00	1.00	1.00	1.00	1.00	0.98	1.00	1.00	1.00	1.00	0.83	1.00	1.00	1.00	1.00	0.39	0.97	0.48	1.00	1.00
	by Max Avg HW	1.00	1.00	1.00	1.00	1.00	0.90	0.99	0.95	0.99	1.00	0.77	0.89	0.83	0.87	0.97	0.31	0.41	0.31	0.41	0.66
	Combined	1.00	1.00	1.00	1.00	1.00	0.94	1.00	0.98	1.00	1.00	0.80	0.95	0.91	0.93	0.98	0.35	0.69	0.40	0.70	0.83
S-37B	by Volume	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.98	1.00	1.00	1.00	1.00	0.87	1.00	1.00	1.00	1.00
	by Max Avg HW	1.00	1.00	1.00	1.00	1.00	0.95	0.96	0.98	1.00	1.00	0.88	0.90	0.89	0.99	0.95	0.59	0.64	0.67	0.74	0.70
	Combined	1.00	1.00	1.00	1.00	1.00	0.98	0.98	0.99	1.00	1.00	0.93	0.95	0.94	1.00	0.98	0.73	0.82	0.83	0.87	0.85
S-37A	by Volume	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00	1.00	1.00	0.95	1.00	0.97	1.00	1.00	0.55	0.89	0.93	0.99	0.92
	by Max Avg HW	1.00	1.00	1.00	1.00	1.00	0.91	0.94	0.98	0.98	0.99	0.78	0.83	0.82	0.84	0.87	0.34	0.40	0.43	0.45	0.43
	Combined	1.00	1.00	1.00	1.00	1.00	0.95	0.97	0.99	0.99	0.99	0.86	0.91	0.90	0.92	0.94	0.45	0.64	0.68	0.72	0.67

Notes: Scores are average of scores for all eight surge/storm event combos for a given column.
Scores in each row are generated in relation to ECB. ECB is always set to 1.
by Volume score uses total/accumulated discharged during the simulated period from September 8th at 2am to September 11th at 2am. Sept 8th is when pre-storm draw down begins (for the alternatives) and Sept 11th 2am is 12 hours after the
by Max Avg HW score uses the maximum from 12 hour rolling averages of HW (excluding the first 24 hours of simulated time).



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Appendix A-2.1.A

Tables of Assumptions

ECB 2035

FWOP 2085

MIKE SHE/MIKE HYDRO – Reach A Watershed Model Design Event Modeling for C&SF Section 203 Study 2035 Existing Conditions (ECB) Baseline Table of Assumptions

Consideration /Feature	Description																																																
Meteorological Data	The following 8 Design Event combinations of tidal surge and rainfall will be simulated: Surge - Rainfall 2YR - 5YR 2YR - 10YR 10YR - 10YR 2YR - 25YR 20YR - 25YR 2YR - 100YR 100YR - 100YR 2YR - 500YR Design Event Rainfall from NOAA Atlas 14 3-day rainfall depths using spatially distributed gridded input. The rainfall depths obtained from NOAA are point values and require adjustments per USACE Engineering Manual 1110-2-1417, Section 13-4. Areal reduction factors are applied using USGS methodology described in the referenced publication: (Irizarry-Ortiz & Stamm, 2022)SFWMD 72-hr rainfall distribution as found in the SFWMD Environmental Resource Permit Applicant’s Handbook Volume II dated 2016. The reference ET is spatially uniform and based on minimum three-day wet season ET values for year 2017 developed from USGS reference ET data (USGS Reference and Potential Evapotranspiration, accessed 2018)																																																
Topography	SFWMD mosaic 2.5 ft Digital Elevation Model (DEM) created from the following data sources: <table><tr><th>Data Set ID</th><th>Group Name</th><th>Product Name</th><th>Acquisition Year</th><th>Resolution (ft)</th><th>Source Agency</th></tr><tr><td>BR_2018_1_6FT</td><td>3DEP</td><td>Broward_2018</td><td>2018</td><td>1.6</td><td>USGS</td></tr><tr><td>COASTAL_2017_5FT</td><td>Project</td><td>COASTAL_2017</td><td>2017</td><td>5</td><td>FEMA</td></tr><tr><td>ENP_2017_1_6FT</td><td>3DEP</td><td>ENP_2017</td><td>2017</td><td>1.6</td><td>USGS</td></tr><tr><td>HAED_2007_100FT</td><td>Project</td><td>HAED_2007</td><td>2007</td><td>100</td><td>USGS</td></tr><tr><td>MD_2015_5FT</td><td>County</td><td>MiamiDade_2015</td><td>2015</td><td>5</td><td>Miami-Dade County</td></tr><tr><td>MD_2018_5FT</td><td>County</td><td>MiamiDade_2018</td><td>2018</td><td>5</td><td>Miami-Dade County</td></tr><tr><td>PB_2017_2_5FT</td><td>3DEP</td><td>PalmBeach_2017</td><td>2017</td><td>2.5</td><td>USGS</td></tr></table>	Data Set ID	Group Name	Product Name	Acquisition Year	Resolution (ft)	Source Agency	BR_2018_1_6FT	3DEP	Broward_2018	2018	1.6	USGS	COASTAL_2017_5FT	Project	COASTAL_2017	2017	5	FEMA	ENP_2017_1_6FT	3DEP	ENP_2017	2017	1.6	USGS	HAED_2007_100FT	Project	HAED_2007	2007	100	USGS	MD_2015_5FT	County	MiamiDade_2015	2015	5	Miami-Dade County	MD_2018_5FT	County	MiamiDade_2018	2018	5	Miami-Dade County	PB_2017_2_5FT	3DEP	PalmBeach_2017	2017	2.5	USGS
Data Set ID	Group Name	Product Name	Acquisition Year	Resolution (ft)	Source Agency																																												
BR_2018_1_6FT	3DEP	Broward_2018	2018	1.6	USGS																																												
COASTAL_2017_5FT	Project	COASTAL_2017	2017	5	FEMA																																												
ENP_2017_1_6FT	3DEP	ENP_2017	2017	1.6	USGS																																												
HAED_2007_100FT	Project	HAED_2007	2007	100	USGS																																												
MD_2015_5FT	County	MiamiDade_2015	2015	5	Miami-Dade County																																												
MD_2018_5FT	County	MiamiDade_2018	2018	5	Miami-Dade County																																												
PB_2017_2_5FT	3DEP	PalmBeach_2017	2017	2.5	USGS																																												



Consideration /Feature	Description
	- Converted to NGVD using District GIS Layer with vertical datum elevation offsets calculated using CORPSCON Program Version 6.0.1 - Reach A is modeled in NGVD29 datum
Tidal Data	- Tidal data with surge was implemented at the tidal boundary utilizing Coastal Hazard System (CHS) save point data with tidal signals developed as documented by USACE (USACE, September, 2023). - Sea level change ECB 2035 uses Intermediate Projection only - Tidal signals were 28 days. - Tidal hydrographs are modified temporally to yield coincident peaks of rainfall and surge; sensitivity tests performed showed that coincident peaks of rainfall intensity and peak tailwater stages have the highest impact on stages and provide the most conservative case scenario for flooding depth and inundation extent.
Land Use and Land Cover	- The land use data in the Reach A model was based on the land use data from the 2019 Broward County Current Conditions model, which was developed from the SFWMD Land Use Land Cover data with minor changes based on satellite imagery from 2015 with 2018. The raw SFWMD 2014-2016 Land Use dataset was used to define land use conditions in the newly added area in the model domain. - The future conditions land use map was developed by modifying the current conditions land use map to reflect projected future changes. Areas of future change were identified by comparing undeveloped, agricultural, and low development areas (such as low density residential) to future conditions land use maps for 2040 from the Broward County Planning Council, Palm Beach County Planning, Zoning and Building Department, and Miami-Dade County (2030 condition). - The Broward Resilience models make use of the recently modified land use which was modified by using a combination of infrared images and aerial photographs. Areas are classified as impervious or pervious to accurately depict the runoff in the model. Classification was performed based on unique spectral characteristics (with the use of machine learning)
Water Control Districts (WCDs)	- Much of the local drainage data was gathered from numerous stakeholders during the development of the Reach A model in 2019 by Broward County. (Taylor Engineering, Inc., November, 2019) - Broward County and South Broward Drainage District structures operations from the 2019 Broward County Current Conditions model which has operating criteria inherited from the 2014 FEMA model and verified/updated based on stakeholder data and the SBDD Facilities Report, 2013, GIS database and Water Control Plan (Taylor Engineering, Inc., November, 2019)

Consideration / Feature	Description
Duration	- Longer duration (assuming total to include 20 days after peak rainfall) will be run for comparison with Alt RO for comp benefits calculation – flood duration. - Shorter duration runs were made for some initial testing during FWP assessments to determine peak water levels for FDA
Initial Conditions	- Average wet season groundwater assumptions were the goal of the project. Future conditions average wet season groundwater elevation maps from Broward County FPLOS model (Taylor Engineering, Inc. with Anclote Consulting PLLC, 2021) were the basis for the Broward Resilience model and utilized for the C&SF Section 203 study. The initial groundwater was adjusted for USACE projected sea level conditions for 2035. - For Reach A MIKE 1D initial conditions the following methodology has been adopted: the CHS timeseries closest to the primary canals in the Reach has been used to extract the average of the daily maximum values. For each CHS boundary this is calculated. This single value is then used as the initial condition for the primary canals in Reach A. Initial conditions for secondary and tertiary canals haven't been modified.
Western Boundary Condition	Western boundaries are set based on representative storm conditions using Regional Simulation Model Glades-LECSA (RSMGL) data from the Biscayne Bay and Southeastern Everglades Restoration (BBSEER) future without (FWOi) simulation which incorporates the Central Everglades Planning Project (CEPP) features affecting western boundaries of all Reaches.
City of Fort Lauderdale Seawall Master Plan	Per the Seawall Master Plan, city owned seawalls will be raised which includes some seawalls in the New River Canal tidal reach. The information only refers to the locations of the larger city owned seawalls which is only about 5.8 mi of roughly 165 miles of waterways, and therefore incompletely addressed all planned raising of seawalls which could skew results of out of bank flooding. It is acknowledged that raising of seawalls could have a local effect on out of bank flooding, and it is suggested to consider raising seawalls at all locations experiencing out of bank flooding in the alternatives.
Site 1 Impoundment Phase 1	- The Phase I features will reduce the amount of seepage loss from the adjacent Loxahatchee National Wildlife Refuge (LNWR), also known as Water Conservation Area 1. This will help increase the amount of water that remains in the natural system, especially during dry periods. Maintaining additional water will allow for ecological habitat improvements in the LNWR. - Modeling Assessment: This project has already been included in the Broward FPLOS model and will be carried over to the ECB/FWOP models. The operation of the project will be turned off consistent with including Phase I only.



Consideration /Feature	Description
Broward County Water Preserve Areas (BCWPA)	This project which is the C9/C11 impoundment is included in the model according to the design/documentation and according to the following information: Although the C-9 and C-11 Impoundment projects are being designed to operate together in the future, for the purposes of this study they are operated independently. For Reach A, the C-11 Impoundment is operated without diverting water to the C-9 Impoundment, as this represents a reasonable worst-case scenario for the C-11 Basin. It is important to note that plans for the two impoundments do not include the ability to divert water from the C-9 Impoundment to the C-11 Impoundment, as it is only intended to be transferred the opposite way. Therefore, the worst-case scenario for C-11 Impoundment is that it becomes full and is unable to divert water away to C-9 Impoundment. (Taylor Engineering, Inc. with Anclote Consulting PLLC, 2021)
Broward County: Sample Road Improvement Project (C1-C2 Canal Interconnect) Sample Road Recharge Line	The Sample Road Recharge Line is the addition of a gated culvert located at Sample Road and Military Trail, in Deerfield Beach. This new structure would release water from Crystal Lake (also known as Sailboat Lake) to the 2_C1 Canal, which runs along the western side of I-95 and outfalls about 1-mile downstream of Structure G-56 on the Hillsboro Canal. For the purposes of this study, this gated culvert was opened at the start of the design storm rainfall.
Broward & Hillsboro Basins: North Springs Improvement District (NSID) Pump Station 3	- This project is included in the model based on the Conceptual Environmental Resource Permit (# 06-00073-S dated 11/9/2017) - This pump is located between the east of Meridian Dr N in Broward County, at the Hillsboro Canal. During on-peak operation (assumed to be during the rainfall event and up to 24-hours later), the pump has a discharge capacity of 25,000 GPM and discharges to the Hillsboro Canal whenever the upstream stage (water level in the secondary system) is greater than 8.7 ft NGVD29 (7.1 ft NAVD88) and turns off when upstream stage falls below 8.5 ft NGVD29 (6.9 ft NAVD88). During off-peak operation (assumed to be more than 24-hours after rainfall event), the pump has a discharge capacity of 50,000 GPM and discharges to the Hillsboro Canal whenever the upstream stage is greater than 8.7 ft NGVD29 (7.1 ft NAVD88) and the stage in the Hillsboro Canal (in the proximity of the pump) is less than 10.0 ft NGVD29 (8.4 ft NAVD88) and turns off when the upstream stage falls below 8.5 ft NGVD29 (6.9 ft NAVD88).
Central Everglades Planning Project (CEPP).	This project has components that influence all Reaches but are not included within the model domain of Reach A. Distribution conveyance features include: - Conveyance improvements to Miami and North New River canals. - Diversion of L-6 flows, infrastructure, and L-5 canal improvements. - Remove western approximately 2.9 miles of L-4 levee west of S-8.

Consideration / Feature	Description
	- Construct pump station (S-630) at western terminus of L-4 levee removal. - Backfill Miami Canal and Spoil Mound Removal from approx. 1.5 miles south of S-8 to I-75. Seepage management features include: - Construct S-356E pump station. - Construct 4.2-mile partial-depth seepage barrier south of Tamiami Trail (along L-31N). - G-211 operational refinements; use coastal canals to convey seepage. Storage and Treatment features include: - Construct the EAA reservoir and stormwater treatment area and integrate with A-1 FEB operations. - Lake Okeechobee operational refinements. CEPP is located west of the C&SF flood resiliency study reaches, project area. Water deliveries south of Lake Okeechobee, water management including seepage management will influence all the study reaches. These projects are informed through boundary conditions on the west side of model domain. Western boundaries are set based on representative storm conditions using Regional Simulation Model Glades-LECSA (RSMGL) data from the Biscayne Bay and Southeastern Everglades Restoration (BBSEER) future without (FWOi) simulation which incorporates the Central Everglades Planning Project (CEPP) features affecting western boundaries of all Reaches.
Broward County: FLL Ravenwood System Interconnection	This Project will interconnect the Northern Stormwater Management System ("Northern System") to the Ravenswood System, to the west of I-95, and will be constructed in 2 phases. The first phase includes a proposed 72-inch equalizer stormwater pipe to interconnect the Northern System and the Ravenswood System. It will also involve expanding the existing stormwater management area in the Ravenswood System. This improvement will provide additional storage volume for stormwater runoff and improve flood attenuation while increasing water quality treatment capacity. The second phase includes retrofitting a proposed stormwater pump station within the interconnection between the 2 systems. This improvement will mitigate the impacts of future sea level rise by maintaining a consistent discharge flow via the interconnection. Design info obtained from Exhibits 8.8 through 8.12 in the Fort Lauderdale Stormwater Master Plan Update (Chen Moore and Associates , 2021) Modeling assessment: This project's design documents were reviewed which showed it to be part of a set of four drainage improvement initiatives



Consideration / Feature	Description
	in FLL. It focuses on enhancing internal conveyance and optimizing internal storage efficiency. This project will be documented but will not be included in the model due to the relatively small scale.
Broward County: FLL Edgewood Park Stormwater Project	The proposed Project will improve the stormwater management system within Edgewood Park, located just north of FLL. The recommended improvements include the upsizing of drainage culverts and restoration of the original cross sections which consist of open drainage ditches at Edgewood Park. Completion of the Project will ensure that a consistent stormwater discharge rate is maintained from the existing stormwater management system into Osceola Creek, located just north of Edgewood Park and will prepare FLL for potential impacts of future sea level rise Design info obtained from Exhibits 8.8 through 8.12 in the Fort Lauderdale Stormwater Master Plan Update (Chen Moore and Associates , 2021) Modeling assessment: This project's design documents were reviewed which showed it to be part of a set of four drainage improvement initiatives in FLL. It focuses on enhancing internal conveyance and optimizing internal storage efficiency. This project will be documented but will not be included in the model due to the relatively small scale.
Broward County: FLL System 1 Basin A Stormwater Interconnection Project	The master stormwater management system at FLL is subdivided into 7 primary drainage basins with independent positive outfalls that discharge stormwater runoff off-site. The Project will include improvements to the master stormwater management system which will interconnect the Northern Stormwater Management System ("Northern System") and the System 1 Basin A ("Basin A") within the North Airfield at FLL. The improvements provide flood protection and include the installation of a new control structure and 48-inch stormwater pipe between the Northern System and Basin A. The interconnectivity will maximize the effectiveness and discharge capacity from each of the existing primary drainage basins. Design info obtained from Exhibits 8.8 through 8.12 in the Fort Lauderdale Stormwater Master Plan Update (Chen Moore and Associates , 2021) Modeling assessment: This project's design documents were reviewed which showed it to be part of a set of four drainage improvement initiatives in FLL. It focuses on enhancing internal conveyance and optimizing internal storage efficiency. This project will be documented but will not be included in the model due to the relatively small scale.
Broward County: FLL System 1 Basin B Stormwater Interconnection Project	The master stormwater management system at FLL is subdivided into 7 primary drainage basins with independent positive outfalls that discharge stormwater runoff off-site. The Project will include improvements to the master stormwater management system which will interconnect the Northern Stormwater Management System ("Northern System") and the System 1 Basin B ("Basin B") within the North Airfield at FLL. The improvements provide flood protection and include the installation of a new 24-inch

Consideration / Feature	Description
	stormwater pipe connection between the Northern System and Basin B. The interconnectivity will maximize the effectiveness and discharge capacity from each of the existing primary drainage basins. Design info obtained from Exhibits 8.8 through 8.12 in the Fort Lauderdale Stormwater Master Plan Update (Chen Moore and Associates , 2021) Modeling assessment: This project's design documents were reviewed which showed it to be part of a set of four drainage improvement initiatives in FLL. It focuses on enhancing internal conveyance and optimizing internal storage efficiency. This project will be documented but will not be included in the model due to the relatively small scale.
Broward County: Broward County Canal Structure S-27 Resilience Improvements	This Project will enhance the reliability and strength of the Broward County S-27 structure to withstand sea level rise pressures, minimize tidally controlled water from contaminating freshwater canals, and protect existing vegetation in the adjacent County-owned Easterlin Park. The project will be active year-round holding freshwater inland during dry times, releasing water to the tidal side of the State's District's C-13 canal during wet times, and preventing the backflow of saline water at high tide and during storm surges. This project will prevent saline water from the tidal section of the C-13 canal from contaminating inland fresh water sources. Design info obtained from Water Control Structure #27 Refurbishment Design Drawings (URS, 2015) To view location go to: https://geohub-bcgis.opendata.arcgis.com/apps/bf33feac7c2c4c3cbd5fd0f1a6730f7c/explore The structure location is at these coordinates: 26.16792, -80.162897. Modeling Assessment: This project has been included in the model in the form of two pipe culverts with flap gates to prevent saline water backflow. A gate with operation schedule has also been included to control the flow through the culverts. A weir with "only positive flow" has also been added according to the design documents provided.
Port Everglades: Bulkhead Replacement Project (Only Bulkhead Replacement Project, Group 1,	North Bulkhead to the Entrance Channel ($\pm 1,220'$). Designed to accommodate five feet of SLR after the design date (~2022). Design info located in Environmental Resource License DF22-1131Final_drawings Modeling assessment: This project has been added to the model by updating the DEM cells where the seawall replacement is proposed.



Consideration / Feature	Description
Project 1 is currently active).	
Time period	- Time series data – simulation time step is variable (SHE Vs Hydro (GW vs. Surface). Simulation timestep may change during simulation based on courant condition. Input timestep is identified as a Maximum time step. The maximum allowed timestep for overland, unsaturated zone and saturated zone in MIKE SHE is 0.1 hrs. whereas the MIKE 1D has a fixed timestep of 4sec. - Time series output timestep is 15 minute for 1d time series, and 1.0 hr for groundwater (head elevation in saturated zone). - Time series output timestep for overland is 1 hour. In addition the model catalogs maximums in simulation time step.
Pumpage and Irrigation	No water supply pumping and irrigation is included in the Reach A model for the storm event modeling
Water Control Structure Operations	- C&SF system and operating rules for SFWMD structures from (SFWMD, 2021) - Broward County and South Broward Drainage District structures operations from the 2019 Broward County Current Conditions model which has operating criteria inherited from the 2014 FEMA model and verified/updated based on stakeholder data and the SBDD Facilities Report, 2013, GIS database and Water Control Plan (Taylor Engineering, Inc., November, 2019) - Assumes high rainfall and pre-storm drawdown operations by operating tidal structures in low range to meet control elevations in the primary canals. - Tidal structures assume salinity operations (spillway gates close when tailwater at the structure is higher than headwater for the prevention of saltwater intrusion) - SFWMD structure flow parameters from SFWMD flow rating analysis reports and the SFWMD Atlas of Flow Computations (The Hydraulic Design Unit, 2015)
Contributing Canal Configuration	- The focus of this study is on the primary system with a high level of detail placed on the secondary/tertiary canal systems, as they are both a major source of discharge into the primary system and storage prior to discharging into the primary system. - Many of the secondary/tertiary canal systems were setup to simulate the connectivity between lakes and other discontinuous (from DEM) water bodies, which are connected through a series of hydraulic structures. - Water bodies that are not explicitly represented via a branch may still be connected to the 1D river network through the use of flood codes This C&SF Section 203 Study Reach A model is based on the Broward Resilience model which has an updated network (particularly in the intercoastal area) since the Broward FPLOS model.

Consideration / Feature	Description
Model Domain Assumptions	• The model domain includes all of urban Broward County and portions of Palm Beach County in the North, Miami-Dade County in the South, and Water Conservation Areas in the West. • A computational grid size of 250-ft is used and coupled with the multi-cell overland feature at a 125-ft grid size. This higher resolution feature further refines the storage and conveyance characteristics of each computational grid cell. Although the model computations are based on a 250-ft grid cell, the conveyance and storage characteristics of each cell are calculated based on the finer 125-ft grid. This local refinement provides a higher level of topographic detail and overland storage while attaining reasonable model runtimes.

Notes



MIKE SHE/MIKE HYDRO – Reach A Watershed Model Design Event Modeling for C&SF Section 203 Study 2085 Future Without Project (FWOP) Table of Assumptions

Consideration /Feature	Description																																																
Meteorological Data	- The following 8 Design Even combinations of tidal surge and rainfall will be simulated: <u>Surge - Rainfall</u> 2YR - 5YR 2YR - 10YR 10YR - 10YR 2YR - 25YR 20YR - 25YR 2YR - 100YR 100YR - 100YR 2YR - 500YR - Design Event Rainfall from NOAA Atlas 14 3-day rainfall depths using spatially distributed gridded input. The rainfall depths obtained from NOAA are point values and require adjustments per USACE Engineering Manual 1110-2-1417, Section 13-4. Areal reduction factors are applied using USGS methodology described in the referenced publication: (Irizarry-Ortiz & Stamm, 2022) - SFWMD 72-hr rainfall distribution as found in the SFWMD Environmental Resource Permit Applicant’s Handbook Volume II dated 2016 - The reference ET is spatially uniform and based on minimum three-day wet season ET values for year 2017 developed from USGS reference ET data (USGS Reference and Potential Evapotranspiration, 2018)																																																
Topography	- SFWMD mosaic 2.5 ft Digital Elevation Model (DEM) created from the following data sources:<table><tr><th>Data Set ID</th><th>Group Name</th><th>Product Name</th><th>Acquisition Year</th><th>Resolution (ft)</th><th>Source Agency</th></tr><tr><td>BR_2018_1_6FT</td><td>3DEP</td><td>Broward_2018</td><td>2018</td><td>1.6</td><td>USGS</td></tr><tr><td>COASTAL_2017_5FT</td><td>Project</td><td>COASTAL_2017</td><td>2017</td><td>5</td><td>FEMA</td></tr><tr><td>ENP_2017_1_6FT</td><td>3DEP</td><td>ENP_2017</td><td>2017</td><td>1.6</td><td>USGS</td></tr><tr><td>HAED_2007_100FT</td><td>Project</td><td>HAED_2007</td><td>2007</td><td>100</td><td>USGS</td></tr><tr><td>MD_2015_5FT</td><td>County</td><td>MiamiDade_2015</td><td>2015</td><td>5</td><td>Miami-Dade County</td></tr><tr><td>MD_2018_5FT</td><td>County</td><td>MiamiDade_2018</td><td>2018</td><td>5</td><td>Miami-Dade County</td></tr><tr><td>PB_2017_2_5FT</td><td>3DEP</td><td>PalmBeach_2017</td><td>2017</td><td>2.5</td><td>USGS</td></tr></table> - Converted to NGVD using District GIS Layer with vertical datum elevation offsets calculated using CORPSCON Program Version 6.0.1 - Reach A is modeled in NGVD29 datum	Data Set ID	Group Name	Product Name	Acquisition Year	Resolution (ft)	Source Agency	BR_2018_1_6FT	3DEP	Broward_2018	2018	1.6	USGS	COASTAL_2017_5FT	Project	COASTAL_2017	2017	5	FEMA	ENP_2017_1_6FT	3DEP	ENP_2017	2017	1.6	USGS	HAED_2007_100FT	Project	HAED_2007	2007	100	USGS	MD_2015_5FT	County	MiamiDade_2015	2015	5	Miami-Dade County	MD_2018_5FT	County	MiamiDade_2018	2018	5	Miami-Dade County	PB_2017_2_5FT	3DEP	PalmBeach_2017	2017	2.5	USGS
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Consideration / Feature	Description																								
Tidal Data	- Tidal data with surge from CHS (USACE, September, 2023) - Hydrographs developed based on NOAA’s predicted tides, based on harmonic constituents. - The months of September through November were analyzed to capture steric effects. - Tidal signals were 28 days. - The recommended plan is evaluated under low, intermediate, and high SLC scenarios to determine the overall project performance and potential adaptation measures - The nearest compliant NOAA tidal gauge, Vaca Key, FL, was used to estimate the RSLC rate Estimated Relative Sea Level Change Projections - Gauge: 8723970, Vaca Key, FL <table><thead><tr><th>Year</th><th>USACE Low (ft NAVD88)</th><th>USACE Int (ft NAVD88)</th><th>USACE High (ft NAVD88)</th></tr></thead><tbody><tr><td>1992</td><td>-0.82</td><td>-0.82</td><td>-0.82</td></tr><tr><td>2023</td><td>-0.4</td><td>-0.3</td><td>-0.1</td></tr><tr><td>2035</td><td>-0.3</td><td>-0.1</td><td>0.4</td></tr><tr><td>2085</td><td>0.4</td><td>1.2</td><td>3.6</td></tr><tr><td>2135</td><td>1.0</td><td>2.9</td><td>8.6</td></tr></tbody></table> - Low-lying coastal structures utilize a sub-daily tailwater timestep to limit structure discharge when high tides would cause these structures to close or experience reduced discharge capacity. - Coincident peaks of rainfall and surge; sensitivity tests performed showed that coincident peaks of rainfall intensity and peak tailwater stages have the highest impact on stages and provide the most conservative case scenario for flooding depth and inundation extent.	Year	USACE Low (ft NAVD88)	USACE Int (ft NAVD88)	USACE High (ft NAVD88)	1992	-0.82	-0.82	-0.82	2023	-0.4	-0.3	-0.1	2035	-0.3	-0.1	0.4	2085	0.4	1.2	3.6	2135	1.0	2.9	8.6
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Land Use and Land Cover	The land use data in the Reach A model was based on the land use data from the 2019 Broward County Current Conditions model, which was developed from the SFWMD Land Use Land Cover data with minor changes based on satellite imagery from 2015 with 2018. The raw SFWMD 2014-2016 Land Use dataset was used to define land use conditions in the newly added area in the model domain.																								



Consideration / Feature	Description
	- The future conditions land use map was developed by modifying the current conditions land use map to reflect projected future changes. Areas of future change were identified by comparing undeveloped, agricultural, and low development areas (such as low density residential) to future conditions land use maps for 2040 from the Broward County Planning Council, Palm Beach County Planning, Zoning and Building Department, and Miami-Dade County(2030 condition). - The Broward Resilience model modifies the gridded impervious area percentages by using a combination of infrared images and aerial photographs. This modified impervious area fraction is used for the ponded drainage runoff. Classification was performed based on unique spectral characteristics (with the use of machine learning)
Water Control Districts (WCDs)	- Much of the local drainage data was gathered from numerous stakeholders during the development of the Reach A model in 2019 by Broward County. (Taylor Engineering, Inc., November, 2019) - Broward County and South Broward Drainage District structures operations from the 2019 Broward County Current Conditions model which has operating criteria inherited from the 2014 FEMA model and verified/updated based on stakeholder data and the SBDD Facilities Report, 2013, GIS database and Water Control Plan (Taylor Engineering, Inc., November, 2019)
Duration	- Longer duration (assuming total to include 20 days after peak rainfall) will be run for comparison with Alt RO for comp benefits calculation - (flood duration) - Shorter duration runs were made for some initial testing during FWP assessments to determine peak water levels for FDA
Initial Conditions	- Average wet season groundwater assumptions were the goal of the project. Future conditions average wet season groundwater elevation maps from Broward County FPLOS model (Taylor Engineering, Inc. with Anclote Consulting PLLC, 2021) were the basis for the Broward Resilience model and utilized for the C&SF Section 203 study. The initial groundwater was adjusted for USACE projected sea level conditions for 2085l, 2085i, and 2085h. - For Reach A MIKE 1D initial conditions the following methodology has been adopted: the CHS timeseries closest to the primary canals in the Reach has been used to extract the average of the daily maximum values. For each CHS boundary and each of the four SLR conditions (FWOPI, FWOPi, and FWOPh) this is calculated. This single value is then used as the initial condition for the primary canals in Reach A. Initial conditions for secondary and tertiary canals haven't been modified.
Western Boundary Conditions	Western boundaries are set based on representative storm conditions using Regional Simulation Model Glades-LECSA (RSMGL) data from the Biscayne Bay and Southeastern Everglades Restoration (BBSEER) future without (FWOi) simulation which incorporates the Central

Consideration / Feature	Description
	Everglades Planning Project (CEPP) features affecting western boundaries of all Reaches.
City of Fort Lauderdale Seawall Master Plan	Per the Seawall Master Plan, city owned seawalls will be raised which includes some seawalls in the New River Canal tidal reach. The information only refers to the locations of the larger city owned seawalls which is only about 5.8 mi of roughly 165 miles of waterways, and therefore incompletely addressed all planned raising of seawalls which could skew results of out of bank flooding. It is acknowledged that raising of seawalls could have a local effect on out of bank flooding, and it is suggested to consider raising seawalls at all locations experiencing out of bank flooding in the alternatives.
Site 1 Impoundment Phase 1	- The Phase I features will reduce the amount of seepage loss from the adjacent Loxahatchee National Wildlife Refuge (LNWR), also known as Water Conservation Area 1. This will help increase the amount of water that remains in the natural system, especially during dry periods. Maintaining additional water will allow for ecological habitat improvements in the LNWR. - Modeling Assessment: This project has already been included in the Broward FPLOS model and will be carried over to the ECB/FWOP models. The operation of the project will be turned off consistent with including Phase I only.
Broward County Water Preserve Areas (BCWPA)	This project which is the C9/C11 impoundment is included in the model according to the design/documentation and according to the following information: Although the C-9 and C-11 Impoundment projects are being designed to operate together in the future, for the purposes of this study they are operated independently. For Reach A, the C-11 Impoundment is operated without diverting water to the C-9 Impoundment, as this represents a reasonable worst-case scenario for the C-11 Basin. It is important to note that plans for the two impoundments do not include the ability to divert water from the C-9 Impoundment to the C-11 Impoundment, as it is only intended to be transferred the opposite way. Therefore, the worst-case scenario for C-11 Impoundment is that it becomes full and is unable to divert water away to C-9 Impoundment. (Taylor Engineering, Inc. with Anclothe Consulting PLLC, 2021)
Broward County: Sample Road Improvement Project (C1-C2 Canal Interconnect) Sample Road Recharge Line	The Sample Road Recharge Line is the addition of a gated culvert located at Sample Road and Military Trail, in Deerfield Beach. This new structure would release water from Crystal Lake (also known as Sailboat Lake) to the 2_C1 Canal, which runs along the western side of I-95 and outfalls about 1-mile downstream of Structure G-56 on the Hillsboro Canal. In this study, this gated culvert was assumed opened at the start of the design storm rainfall.



Consideration / Feature	Description
Broward & Hillsboro Basins: North Springs Improvement District (NSID) Pump Station 3	• This project is included in the model based on the Conceptual Environmental Resource Permit (# 06-00073-S dated 11/9/2017) • This pump is located between the east of Meridian Dr N in Broward County, at the Hillsboro Canal. During on-peak operation (assumed to be during the rainfall event and up to 24-hours later), the pump has a discharge capacity of 25,000 GPM and discharges to the Hillsboro Canal whenever the upstream stage (water level in the secondary system) is greater than 8.7 ft NGVD29 (7.1 ft NAVD88) and turns off when upstream stage falls below 8.5 ft NGVD29 (6.9 ft NAVD88). During off-peak operation (assumed to be more than 24-hours after rainfall event), the pump has a discharge capacity of 50,000 GPM and discharges to the Hillsboro Canal whenever the upstream stage is greater than 8.7 ft NGVD29 (7.1 ft NAVD88) and the stage in the Hillsboro Canal (in the proximity of the pump) is less than 10.0 ft NGVD29 (8.4 ft NAVD88) and turns off when the upstream stage falls below 8.5 ft NGVD29 (6.9 ft NAVD88).
Central Everglades Planning Project (CEPP).	This project has components that influence all Reaches but are not included within the model domain of Reach A. Distribution conveyance features include: • Conveyance improvements to Miami and North New River canals. • Diversion of L-6 flows, infrastructure, and L-5 canal improvements. • Remove western approximately 2.9 miles of L-4 levee west of S-8. • Construct pump station (S-630) at western terminus of L-4 levee removal. • Backfill Miami Canal and Spoil Mound Removal from approx. 1.5 miles south of S-8 to I-75. Seepage management features include: • Construct S-356E pump station. • Construct 4.2-mile partial-depth seepage barrier south of Tamiami Trail (along L-31N). • G-211 operational refinements; use coastal canals to convey seepage. Storage and Treatment features include: • Construct the EAA reservoir and stormwater treatment area and integrate with A-1 FEB operations. • Lake Okeechobee operational refinements. CEPP is located west of the C&SF flood resiliency study reaches, project area. Water deliveries south of Lake Okeechobee, water management including seepage management will influences all the study reaches. These projects are informed through boundary conditions on the west side of model domain.
Broward County: FLL Ravenwood System	This Project will interconnect the Northern Stormwater Management System ("Northern System") to the Ravenswood System, to the west of I-95, and will be constructed in 2 phases. The first phase includes a proposed 72-inch equalizer stormwater pipe to interconnect the Northern System and the Ravenswood System. It will also involve expanding the existing stormwater management area

Consideration / Feature	Description
Interconnection	in the Ravenswood System. This improvement will provide additional storage volume for stormwater runoff and improve flood attenuation while increasing water quality treatment capacity. The second phase includes retrofitting a proposed stormwater pump station within the interconnection between the 2 systems. This improvement will mitigate the impacts of future sea level rise by maintaining a consistent discharge flow via the interconnection. Design info obtained from Exhibits 8.8 through 8.12 in the Fort Lauderdale Stormwater Master Plan Update (Chen Moore and Associates , 2021) Modeling assessment: This project’s design documents were reviewed which showed it to be part of a set of four drainage improvement initiatives in FLL. It focuses on enhancing internal conveyance and optimizing internal storage efficiency. This project will be documented but will not be included in the model due to the relatively small scale.
Broward County: FLL Edgewood Park Stormwater Project	The proposed Project will improve the stormwater management system within Edgewood Park, located just north of FLL. The recommended improvements include the upsizing of drainage culverts and restoration of the original cross sections which consist of open drainage ditches at Edgewood Park. Completion of the Project will ensure that a consistent stormwater discharge rate is maintained from the existing stormwater management system into Osceola Creek, located just north of Edgewood Park and will prepare FLL for potential impacts of future sea level rise Design info obtained from Exhibits 8.8 through 8.12 in the Fort Lauderdale Stormwater Master Plan Update (Chen Moore and Associates , 2021) Modeling assessment: This project’s design documents were reviewed which showed it to be part of a set of four drainage improvement initiatives in FLL. It focuses on enhancing internal conveyance and optimizing internal storage efficiency. This project will be documented but will not be included in the model due to the relatively small scale.
Broward County: FLL System 1 Basin A Stormwater Interconnection Project	The master stormwater management system at FLL is subdivided into 7 primary drainage basins with independent positive outfalls that discharge stormwater runoff off-site. The Project will include improvements to the master stormwater management system which will interconnect the Northern Stormwater Management System (“Northern System”) and the System 1 Basin A (“Basin A”) within the North Airfield at FLL. The improvements provide flood protection and include the installation of a new control structure and 48-inch stormwater pipe between the Northern System and Basin A. The interconnectivity will maximize the effectiveness and discharge capacity from each of the existing primary drainage basins. Design info obtained from Exhibits 8.8 through 8.12 in the Fort Lauderdale Stormwater Master Plan Update (Chen Moore and Associates , 2021) Modeling assessment: This project’s design documents were reviewed which showed it to be part of a set of four drainage improvement initiatives in FLL. It focuses on enhancing internal conveyance and optimizing internal storage efficiency. This project will be documented but will not be included in the model due to the relatively small scale.



Consideration / Feature	Description
Broward County: FLL System 1 Basin B Stormwater Interconnection Project	The master stormwater management system at FLL is subdivided into 7 primary drainage basins with independent positive outfalls that discharge stormwater runoff off-site. The Project will include improvements to the master stormwater management system which will interconnect the Northern Stormwater Management System (“Northern System”) and the System 1 Basin B (“Basin B”) within the North Airfield at FLL. The improvements provide flood protection and include the installation of a new 24-inch stormwater pipe connection between the Northern System and Basin B. The interconnectivity will maximize the effectiveness and discharge capacity from each of the existing primary drainage basins. Design info obtained from Exhibits 8.8 through 8.12 in the Fort Lauderdale Stormwater Master Plan Update (Chen Moore and Associates , 2021) Modeling assessment: This project’s design documents were reviewed which showed it to be part of a set of four drainage improvement initiatives in FLL. It focuses on enhancing internal conveyance and optimizing internal storage efficiency. This project will be documented but will not be included in the model due to the relatively small scale.
Broward County: Broward County Canal Structure S-27 Resilience Improvements	This Project will enhance the reliability and strength of the Broward County S-27 structure to withstand sea level rise pressures, minimize tidally controlled water from contaminating freshwater canals, and protect existing vegetation in the adjacent County-owned Easterlin Park. The project will be active year-round holding freshwater inland during dry times, releasing water to the tidal side of the State’s District’s C-13 canal during wet times, and preventing the backflow of saline water at high tide and during storm surges. This project will prevent saline water from the tidal section of the C-13 canal from contaminating inland fresh water sources. Design info obtained from Water Control Structure #27 Refurbishment Design Drawings (URS, 2015) To view location go to: https://geohub-bcgis.opendata.arcgis.com/apps/bf33feac7c2c4c3cbd5fd0f1a6730f7c/explore The structure location is at these coordinates: 26.16792, -80.162897. Modeling Assessment: This project has been included in the model in the form of two pipe culverts with flap gates to prevent saline water backflow. A gate with operation schedule has also been included to control the flow through the culverts. A weir with “only positive flow” has also been added according to the design documents provided.
Port Everglades: Bulkhead Replacement Project	North Bulkhead to the Entrance Channel (±1,220'). Designed to accommodate five feet of SLR after the design date (~2022). Design info located in Environmental Resource License DF22-1131Final_drawings (URS, 2015)

Consideration / Feature	Description
(Only Bulkhead Replacement Project, Group 1, Project 1 is currently active).	Modeling Assessment: This project has been added to the model by updating the DEM cells where the seawall replacement is proposed.
Time period	- Time series data – simulation time step is variable (SHE Vs Hydro (GW vs. Surface). Simulation timestep may change during simulation based on courant condition. Input timestep is identified as a Maximum time step. The maximum allowed timestep for overland, unsaturated zone and saturated zone in MIKE SHE is 0.1 hrs. whereas the MIKE 1D has a fixed timestep of 4sec. - Time series output timestep is 15 minutes for 1d time series, and 1.0 hr for groundwater (head elevation in saturated zone). - Time series output timestep for overland is 1 hour. In addition the model catalogs maximums in simulation time step.
Pumpage and Irrigation	No water supply pumping and irrigation is included in the Reach A model for the storm event modeling
Water Control Structure Operations	- C&SF system and operating rules for SFWMD structures from Eastern Broward County Water Control Operations Atlas (SFWMD, 2021) - Broward County and South Broward Drainage District (SBDD) structures operations from the 2019 Broward County Current Conditions model which has operating criteria inherited from the 2014 FEMA model and verified/updated based on stakeholder data and the SBDD Facilities Report, 2013, GIS database and Water Control Plan. (Taylor Engineering, Inc., November, 2019) - Assumes high rainfall and operates tidal structures in low range to meet control elevations in the primary canals. - Tidal structures assume salinity operations (spillway gates close when tailwater at the structure is higher than headwater for the prevention of saltwater intrusion) - SFWMD structure flow parameters from SFWMD flow rating analysis reports and the SFWMD Atlas of Flow Computations (The Hydraulic Design Unit, 2015)
Contributing Canal Configuration	- The focus of this study is on the primary system with a high level of detail placed on the secondary/tertiary canal systems, as they are both a major source of discharge into the primary system and storage prior to discharging into the primary system. - Many of the secondary/tertiary canal systems were setup to simulate the connectivity between lakes and other discontinuous (from DEM) water bodies, which are connected through a series of hydraulic structures. - Water bodies that are not explicitly represented via a branch may still be connected to the 1D river network through the use of flood codes



Consideration / Feature	Description
	Broward Resilience models' updated network (particularly in the intercoastal area) is included in the Reach A model.
Model Domain Assumptions	- The model domain includes all of urban Broward County and portions of Palm Beach County in the North, Miami-Dade County in the South, and Water Conservation Areas in the West. - A computational grid size of 250-ft is used and coupled with the multi-cell overland feature at a 125-ft grid size. This higher resolution feature further refines the storage and conveyance characteristics of each computational grid cell. Although the model computations are based on a 250-ft grid cell, the conveyance and storage characteristics of each cell are calculated based on the finer 125-ft grid. This local refinement provides a higher level of topographic detail and overland storage while attaining reasonable model runtimes.

Notes



Appendix A-2.1.B

Alt RO Maximum Depth Difference Maps with high resolution aerial

Maps included in this Appendix are for select events shown in the report– 2s25r85i, 20s25r85i, 20s25r85h

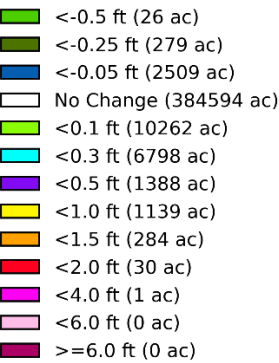
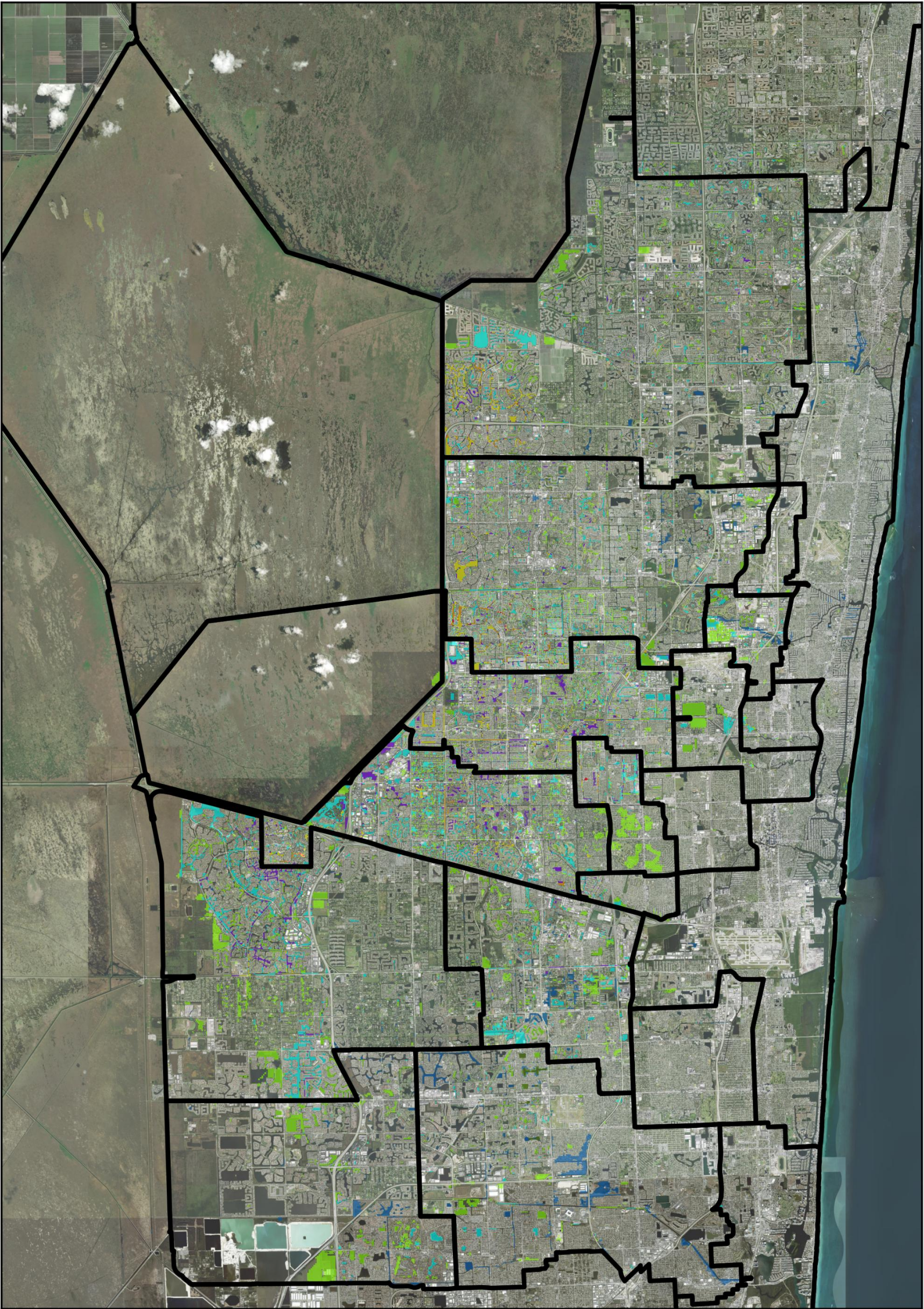
Each map's legend delineates difference in feet, with area of delineated difference in parentheses.

Additional event maps including individual watershed maps are included in the ftp data site which can be accessed by

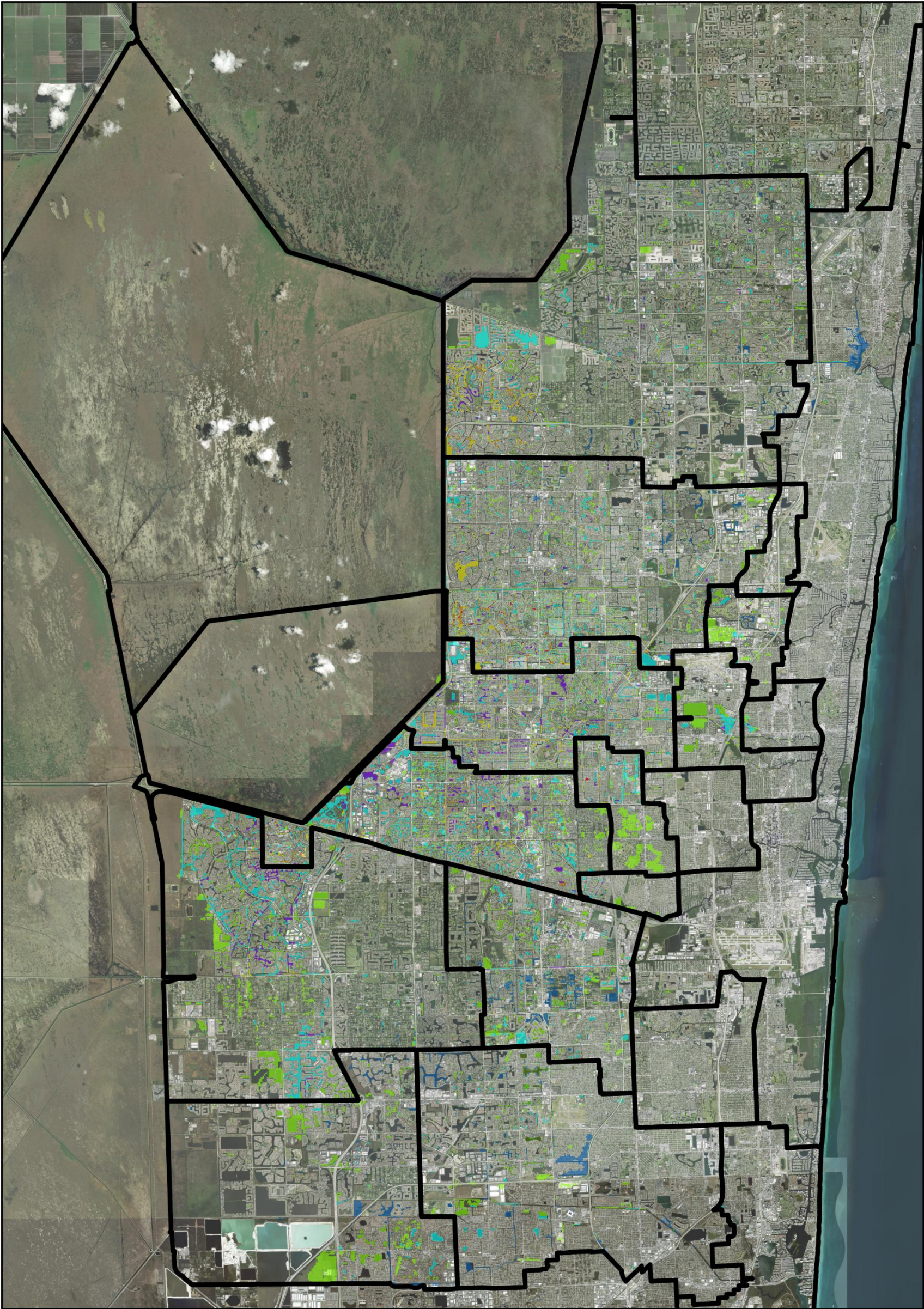
copying the following to a browser

<ftp://ftppub.sfwmd.gov/outgoing/BC203/>

Max Depth FWOPi 2S25R85i - AltRO 2S25R85i (Broward County)

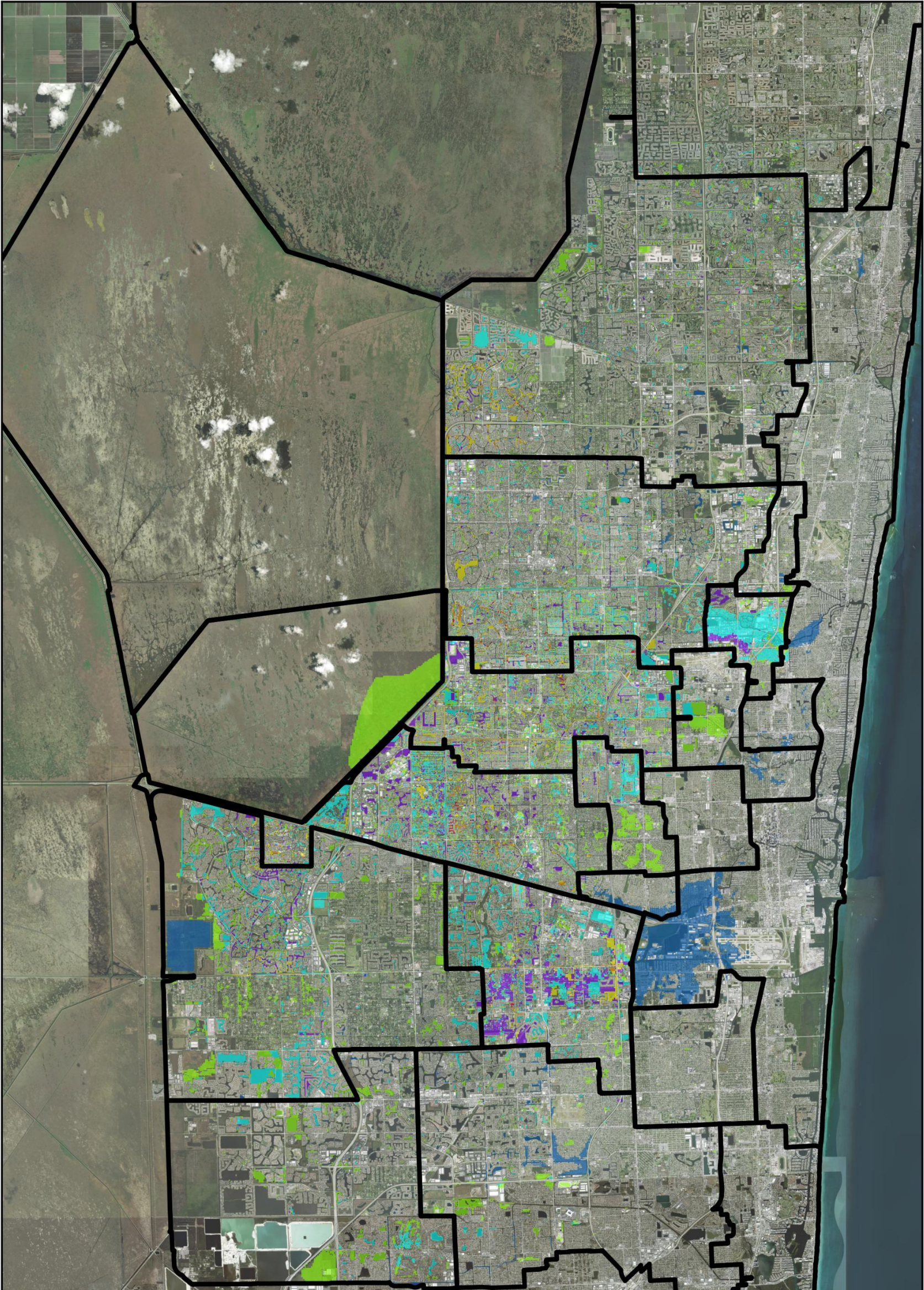


Max Depth FWOPi 20S25R85i - AltRO 20S25R85i (Broward County)



- <-0.5 ft (80 ac)
- <-0.25 ft (286 ac)
- <-0.05 ft (2413 ac)
- No Change (385224 ac)
- <0.1 ft (9914 ac)
- <0.3 ft (6723 ac)
- <0.5 ft (1275 ac)
- <1.0 ft (1087 ac)
- <1.5 ft (272 ac)
- <2.0 ft (34 ac)
- <4.0 ft (2 ac)
- <6.0 ft (0 ac)
- >=6.0 ft (0 ac)

Max Depth FWOPh 20S25R85h - AltRO 20S25R85h (Broward County)



- <-0.5 ft (16 ac)
- <-0.25 ft (121 ac)
- <-0.05 ft (7364 ac)
- No Change (367425 ac)
- <0.1 ft (13746 ac)
- <0.3 ft (12019 ac)
- <0.5 ft (3980 ac)
- <1.0 ft (2091 ac)
- <1.5 ft (454 ac)
- <2.0 ft (84 ac)
- <4.0 ft (10 ac)
- <6.0 ft (1 ac)
- >=6.0 ft (0 ac)



Appendix A-2.1.C

Alt RO Duration Difference Maps with high resolution aerial

Maps included in this Appendix are for select events shown in the report– 2s25r85i, 20s25r85i, 20s25r85h.

Each map's legend delineates difference in hours for flood duration of Future without Project minus the Alt RO. The maps show duration differences assuming depths greater than 0.0 ft and passing a threshold of 0.1 ft before falling back to 0.0 ft.

Individual watershed maps are included in the ftp data site which can be accessed by copying the following to a browser

<ftp://ftppub.sfwmd.gov/outgoing/BC203/>

