ANNEX A-1

Loxahatchee River Watershed Restoration Project

Hydraulic Design

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ACRONYM LIST

ASR	Aquifer Storage Recovery
C&SF	Central & South Florida
CERP	Comprehensive Everglades Restoration Plan
DCM	Design Criteria Memorandum
EIS	Environmental Impact Statement
ER	Engineering Regulation
FDEP	Florida Department of Environmental Protection
LiDAR	Light Detection and Ranging
LRWRP	Loxahatchee River Watershed Restoration Project
NAD83	North American Datum 1983
NAVD88	North American Vertical Datum 1988
NGVD29	National Geodetic Vertical Datum 1929
NOAA-NGS	National Oceanic and Atmospheric Administration – National Geodetic Survey
PED	Pre-construction Engineering Design
PFMA	Potential Failure Mode Analysis
PIR	Project Implementation Report
QRA	Qualitative Risk Assessment
USGS	US Geological Survey

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A INTRODUCTION

All project features within the Loxahatchee River Watershed Restoration Project (LRWRP) were identified and summarized in the Hydraulic Design sections of the EN Appendix. Supplemental material, including further detailed design analyses, is included in this Hydraulic Design Annex. The intent of this Annex is to provide a more thorough explanation of design criteria, assumptions, and analysis. Further analysis will be conducted during Preconstruction Engineering Design (PED) phase in order to optimize all project features for performance and cost efficiency.

A.1 Location Maps

The project area includes undeveloped wetlands and former agricultural lands north and west of the Loxahatchee Slough and northwest fork of Loxahatchee River, in Martin and Palm Beach Counties, Florida. Cypress Creek, Moonshine Creek, and Kitching Creek are significant tributaries of the Loxahatchee River and are part of this restoration project. The study area is bounded on the north by the C-44 Canal, on the south by the C-51 Canal, on the west by the L-8 Canals and Lake Okeechobee, and on the east by the Loxahatchee River Estuary and Lake Worth Lagoon. Flow-ways are locations in the study area based on existing natural areas, topography, and associated canals, and are generally separated from each other by developed lands. Each flow-way represents a pathway for water to flow from the watershed to the Loxahatchee River. Flow-way 1 is located in the southernmost portion of the project area, bounded by L-8 Canal and M-Canal in the south and the G-92 spillway in the north. This flow-way uses the M-Canal and C-18 Canal to route water from upstream project area basins to the Loxahatchee River. Flow-way 2 is located in the central portion of the project area. Its primary canal conveyances are the M-O Canal and C-18W Canal. The watersheds that contribute to these canals include the Indian Trail Improvement District (ITID) basin via the M-O Canal, J.W. Corbett Wildlife Management Area (WMA), Hungryland Slough, “Mecca” property (area of proposed C-18W Reservoir), and natural area east of the North Palm Beach County Airport via the C-18W Canal. Flow-way 3 is located within southern Martin County and consists of the watersheds within the northern portion of the project study area. These watersheds contribute to the northwest fork of the Loxahatchee River via the Ranch Colony Canal and Cypress Creek, Moonshine Creek, Hobe Grove Ditch, and Kitching Creek tributaries. See **Figure 1** through **Figure 4** for the general locations of the three flow-ways, including the Recommended Plan features.

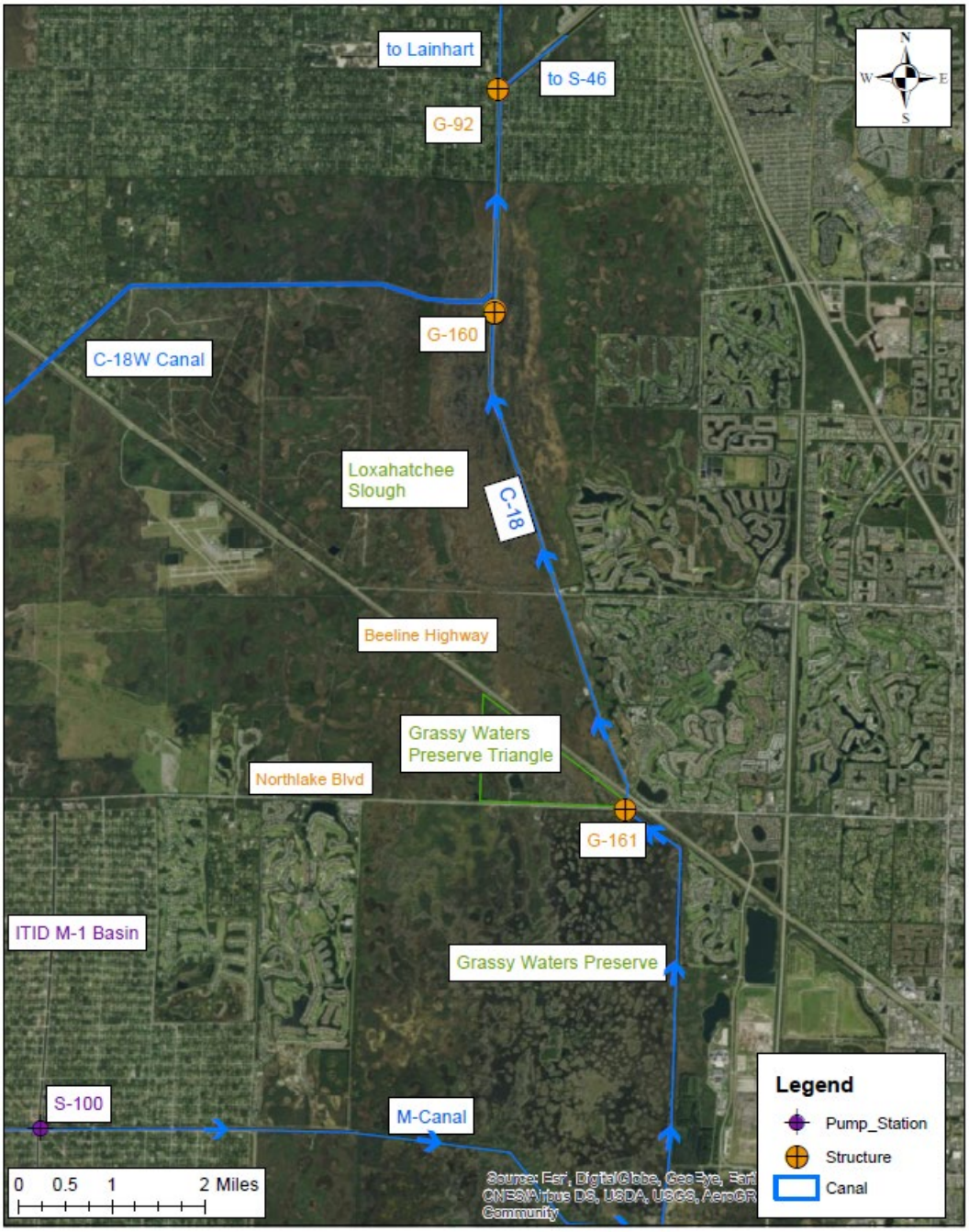

Figure 1. Location Map of Major Features in Flow-way 1.

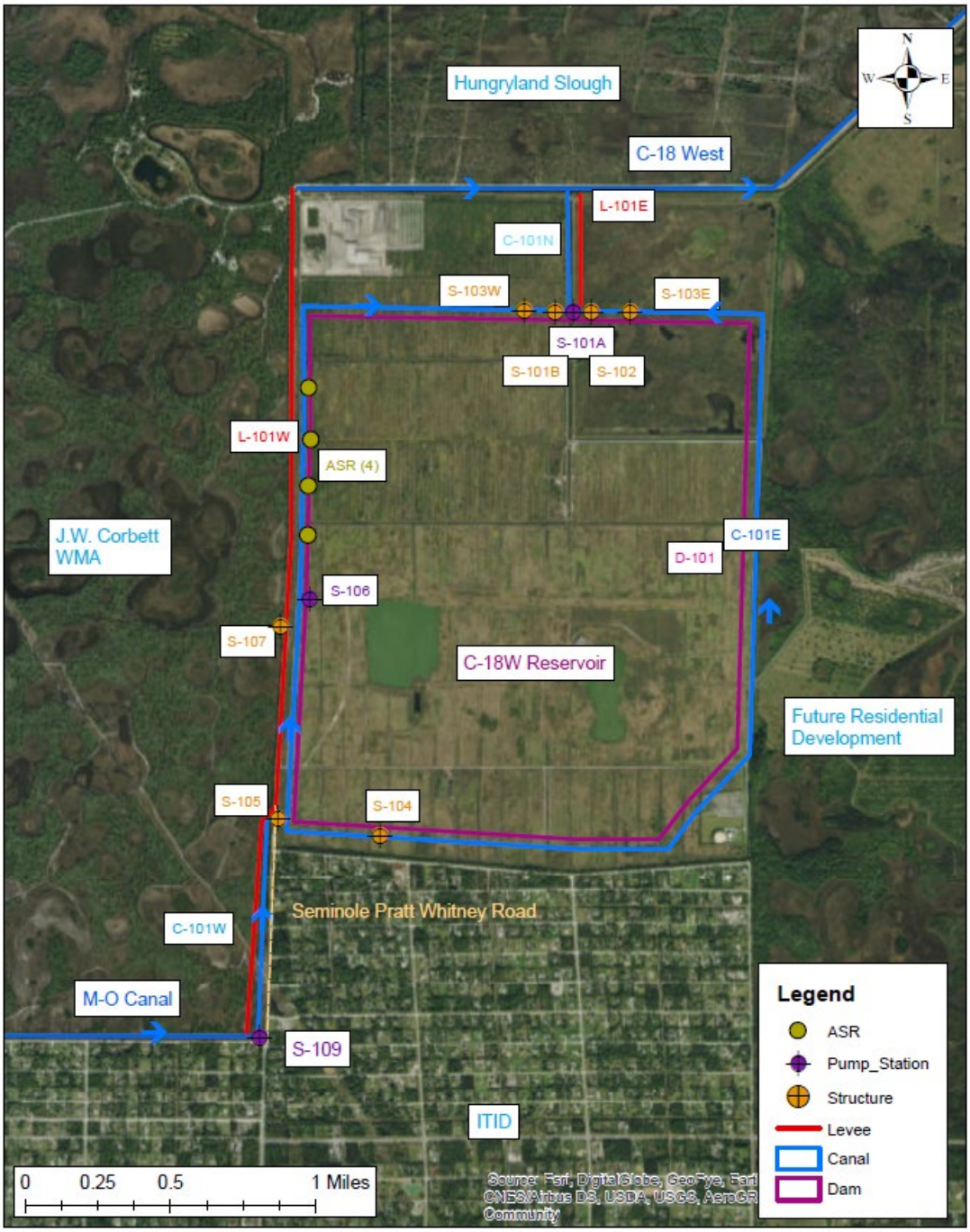

Figure 2. Location of Project Features for Flow-way 2.

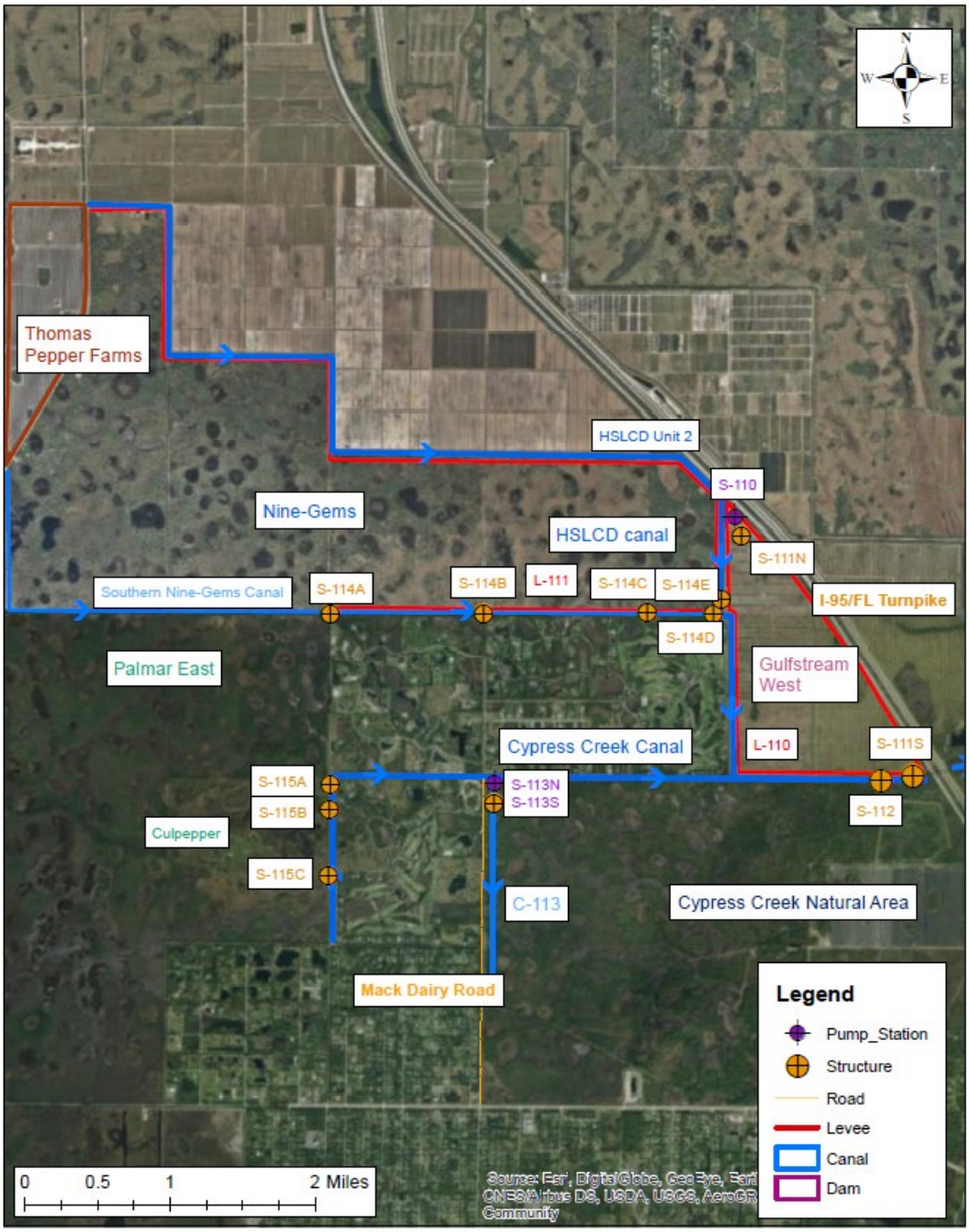

Figure 3. Location of Western Project Features of Flow-way 3.

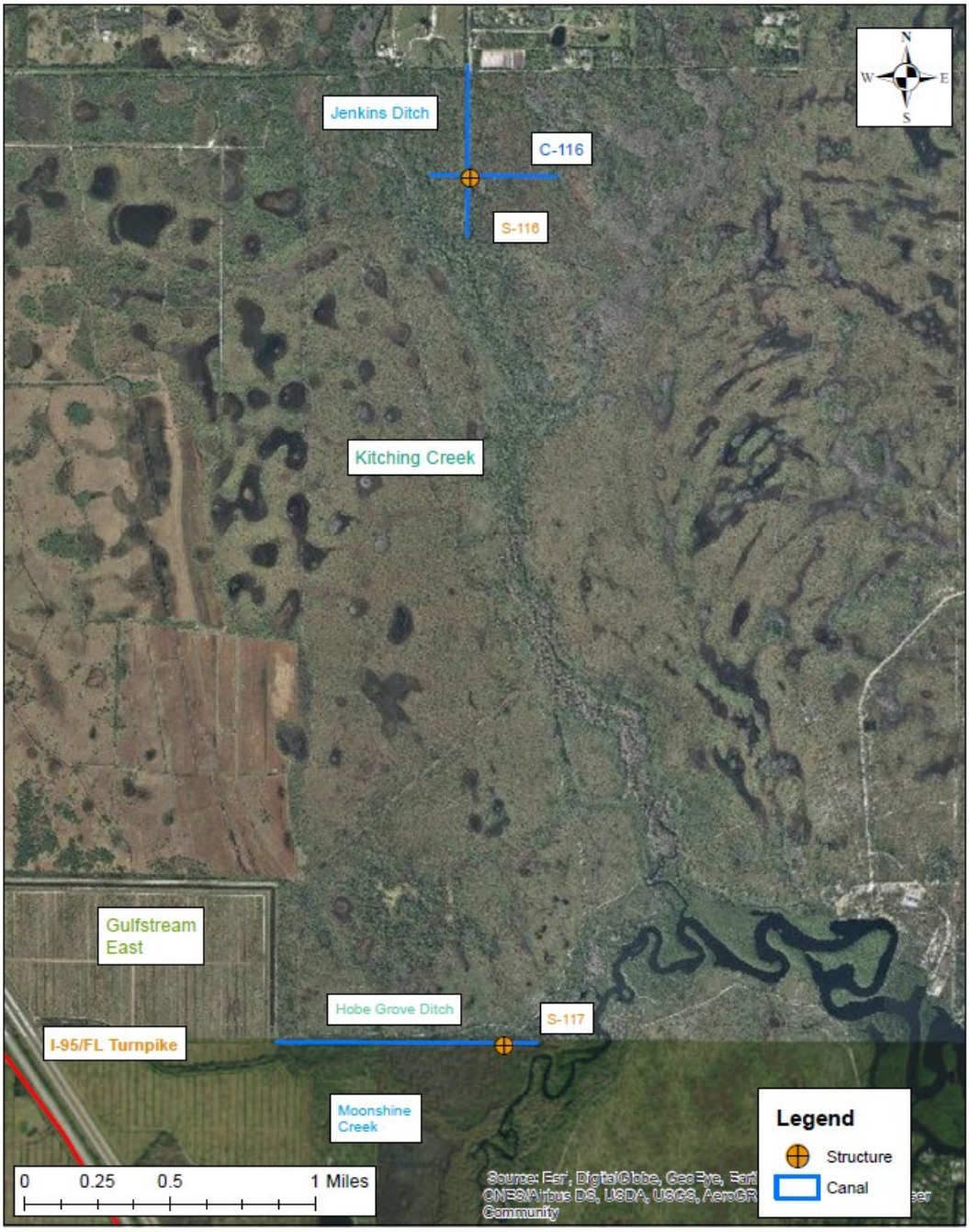

Figure 4. Location of Eastern Project Features of Flow-way 3.

B FLOW-WAY 1 FEATURES

B.1 Purpose

This intermediate hydraulic design is used solely to provide parameters for the calculation of quantities for the purpose of costing project features of the recommended plan. Final design will be performed in PED phase.

B.2 Objective

Structure sizing necessary to convey inflows was determined through analysis of the hydrologic modeling output from the South Florida Water Management Districts (SFWMD) Lower East Coast sub-Regional Model- North Palm (LECSR-NP) model output.

B.3 Assumptions/Constraints

It is important to note that the Comprehensive Everglades Restoration Plan (CERP) included the G-160 and G-161 components with the purpose of providing flows and enhancing hydroperiods in the Loxahatchee Slough, and to increase base flows to the Northwest Fork of the Loxahatchee River. The SFWMD recognized that the G-160 and G-161 water control structures, features of the CERP Plan, were necessary to provide connectivity between the river and its historic headwaters and essential to deliver necessary dry season restorative flows to the Northwest Fork of the Loxahatchee River. Therefore, in parallel with the USACE CERP planning process, the SFWMD moved ahead with the design and construction of G-160 - the Loxahatchee Slough Structure and G-161 - the Northlake Boulevard Structure to provide early and necessary benefits at lower costs to the Loxahatchee Slough and the Loxahatchee River, while allowing for delivery of additional water from and maintaining a more rainfall driven hydroperiod within the Grassy Waters Preserve (GWP). G-160 was constructed in 2004 and G-161 was constructed in 2007. The 2005 Feasibility Scoping Meeting Guidance Memorandum for the North Palm Beach County Part 1 study (predecessor study) approved their inclusion as measures for further evaluation in the with-project conditions. The Jacksonville District and SFWMD requested that G-160 and G-161 be included as measures for evaluation in the with-project conditions and that the study alternatives include the environmental lift/benefit associated with the features. Therefore, both structures have already been constructed and subsequent hydraulic design information is reflective of the previously completed design, with design information obtained from a design report by Stanley Consultants (Stanley, 2004) and as-built drawings. During the PED phase, determination regarding potential structure modification or replacement will be analyzed to ensure both structures are compliant with USACE standards.

The S-100 pump station is constrained by operating levels within the upstream (lower M-1 basin) and downstream (M-Canal). Structures have been designed and optimized in so far as possible based upon inflow parameters and expected operations as informed through the LECSR-NP Model. Additional refinement, as required, will occur during PED.

B.4 Structure Descriptions

B.4.1 G-160

The G-160 structure is a reinforced concrete spillway designed to enhance delivery of the restoration flows to the Northwest Fork of the Loxahatchee River while maintaining water elevations within the Loxahatchee Slough and conserving optimum upstream water control stages in Canal-18 (C-18). Discharges from G-160 are controlled by two stem-operated vertical lift gates. The design discharge rate, to maintain flood control capability, will be approximately 2000 cfs via two spillway bays, each 25 ft. in length. The structure is operable to allow for the dual purposes of flood risk management and environmental restoration. The operable gates allow for management of upstream stages to mimic natural slough recession and ascension in water depth between the wet and dry seasons.

B.4.2 G-161

The G-161 structure is a multi-purpose feature that will facilitate hydroperiod restoration of GWP, maintain existing level of service of flood protection, and provide improved conveyance for the regional water system. The design allows for variable flow rates between 0 and 150 cfs, adjustable as needed for conservation or flood risk management purposes. The structure will consist of two 60-in diameter culvert barrels with a total length of 240 ft. The barrels will be controlled by slide gates and have an invert elevation of 12.0 ft. NGVD29 (10.5 ft. NAVD88)

B.4.3 S-100

A pump station that will deliver up to 75 cfs, via 3-25 cfs pumps from the ITID Lower Basin into the M-Canal to augment GWP when G-161 is operating and reduce reliance upon the regional system. Operations of the M-1 Basin allow for water to be pumped to the M-Canal when stages are above 17.0 ft. NGVD29 (15.5 NAVD88) in the dry season and 15.0 ft. NGVD29 (13.5 NAVD88) in the wet season. Additionally, the M-Canal is to be maintained 0.2-0.3 feet below GWP stages and at a maximum elevation of 18.9 ft. NGVD29 (17.4 NAVD88).

B.5 Hydraulic Design Data Sheets (HDDS)

G-160
Gated Spillway

Design Conditions (optimum efficiency)			
	Discharge (cfs)	2,000	cfs
	Headwater Elevation	16.50	ft, NGVD29
	Tailwater Elevation	16.00	ft, NGVD29
Crest Data			
	Shape	Broad Crested	
	Design Head (Hd)	0.50	ft
	Net Crest Length	50.00	ft
	Crest Elevation	2.86	ft, NGVD29
	Approach Apron Elevation	-	ft, NGVD29
	Weir Control	Vertical Slide	
Gates			
	Number of Gates	2.00	
	Gate Width	25.00	ft
	Gate Height	15.00	ft
	Clearance Elevation	-	ft, NGVD29
	Breastwall Elevation (bottom)	-	ft, NGVD29
	Intermediate Pier Width	-	ft

Figure 5. G-160 HDDS

Culvert Calculator Report
G161

Solve For: Section Size

Culvert Summary					
Allowable HW Elevation	18.00	ft	Headwater Depth/Height	1.19	
Computed Headwater Eleva	17.97	ft	Discharge	150.00	cfs
Inlet Control HW Elev.	17.50	ft	Tailwater Elevation	17.50	ft
Outlet Control HW Elev.	17.97	ft	Control Type	Outlet Control	

Grades					
Upstream Invert	12.00	ft	Downstream Invert	12.00	ft
Length	160.00	ft	Constructed Slope	0.000000	ft/ft

Hydraulic Profile					
Profile	PressureProfile		Depth, Downstream	5.50	ft
Slope Type	N/A		Normal Depth	N/A	ft
Flow Regime	N/A		Critical Depth	2.45	ft
Velocity Downstream	3.82	ft/s	Critical Slope	0.003561	ft/ft

Section					
Section Shape	Circular		Mannings Coefficient	0.013	
Section Material	Concrete		Span	5.00	ft
Section Size	60 inch		Rise	5.00	ft
Number Sections	2				

Outlet Control Properties					
Outlet Control HW Elev.	17.97	ft	Upstream Velocity Head	0.23	ft
Ke	0.50		Entrance Loss	0.11	ft

Inlet Control Properties					
Inlet Control HW Elev.	17.50	ft	Flow Control	Unsubmerged	
Inlet Type	Square edge w/headwall		Area Full	39.3	ft²
K	0.00980		HDS 5 Chart	1	
M	2.00000		HDS 5 Scale	1	
C	0.03980		Equation Form	1	
Y	0.67000				

***HDDS was obtained from Design Report for Structure G-161, Elevation in NGVD29 (Stanley, 2004)**

Figure 6. G-161 HDDS

Loxahatchee River Watershed Restoration Project

Pumping Station 100 (S-100)

Palm Beach County, Florida

X Y Coordinate: 26.76681, 80.247278
Date Compiled (or revised) **27-May-19**

Design Condition		
Ecosystem Restoration (discharge, cfs)	75	cfs
Number of Pumps	3.00	
Pump Mix type and Size		
3 - 25 cfs		
Mix Criteria		
Pumping Station will have 3 pumps - The firm pumping capacity will be 75 cfs		
3 - 25 cfs - Submersible Electric Pumps for Resotration Flows		
Design Operating Heads (ft.)		
Normal	4.50	ft
Maximum	5.70	ft
Intake Water Surface Elevations		
Maximum Non-Pumping	19.00	ft, NAVD88
Maximum Pumping	18.00	ft, NAVD88
Start Pumping	14.5, 16.5	ft, NAVD88
Normal Drawdown	14, 16	ft, NAVD88
Minimum Drawdown	13.5, 15.5	ft, NAVD88
Minimum Non-Pump	13.5, 15.5	ft, NAVD88
Invert Elevation of Canal	6.00	ft, NAVD88
Discharge Water Surface Elevations		
Maximum Non-Pumping	19.20	ft, NAVD88
Maximum Pumping	18.90	ft, NAVD88
Normal Pumping	18	ft, NAVD88
Minimum Pumping	16.00	ft, NAVD88
Minimum Non-Pump	16.00	ft, NAVD88
Invert Elevation of Canal	10.00	ft, NAVD88

[1] Elevations are representative of wet season conditions, dry season elevations are approximately 2 ft. higher

Figure 7. S-100 HDDS

C FLOW-WAY 2 FEATURES

C.1 Purpose

This intermediate hydraulic design is used solely to provide parameters for the calculation of quantities for the purpose of costing project features of the recommended plan. Final design will be performed in PED phase.

C.2 Objective

Reservoir design and hydraulic structure sizing necessary to convey water was determined through hydraulic analysis of the proposed structures. Design standards as established in the Design Criteria Memorandum (DCM) and USACE regulations were used. Hydraulic modeling with HEC-RAS software version 5.0.7 was performed to support the hydraulic design of flow-way 2. The proposed hydraulic structures and the C-18W reservoir were included in the HEC-RAS model to determine the project performance under the design rainfall events.

C.3 Assumptions/Constraints

Design for the proposed hydraulic structures was developed for the 100-year recurrence, 72 hour duration rainfall event. Initial water surface elevation was based on preliminary operations and stage control in the seepage collection canal. **Figure 2** shows the proposed hydraulic structures location.

C.4 Structure Descriptions

C.4.1 S-101A

Structure S-101A is an inflow pump station located on the north side of the C-18W reservoir. It is designed to pump excess runoff water from the C-18W canal into the C-18W reservoir.

C.4.2 S-101B

Structure S-101B is a Service/Emergency spillway located on the C-18W embankment. It is designed as a dual structure: 1) service spillway to provide allowable discharges for the C-18 basin and 2) auxiliary (emergency) spillway to provide discharges during large flood events. It was designed for the PMP rainfall event.

C.4.3 S-102

Structure S-102 is an outflow structure located on the northern C-18W reservoir and discharges into the C-101N to the C-18W canal. It is a gate controlled box culvert that provides discharges from the C-18W reservoir during normal and drawdown operations. The structure is not designed to operate during flood events.

C.4.4 S-103E and –S-103W

Structures S-103E and S-103W are located in the north side of the reservoir and at the end each seepage collection canal. These structures are box culverts that are controlled via slide gates on the headwater side. Additionally, each culvert will contain a flap gate on the downstream end to help prevent back flows

from the C-18W canal. These culverts will provide outflow for excess runoff water captured in the seepage collection canal.

C.4.5 S-104

Structure S-104 is located south of the C-18W reservoir. It was designed as an ungated box culvert with bi-directional flow. The culvert is located on the seepage collection canal and provides drivable access to the C-18W reservoir.

C.4.6 S-105

Structure S-105 is located within the proposed levee L-101W. It is designed as an uncontrolled culvert with flap gates on the downstream side to prevent back flow from the seepage collection canal. S-105 will convey water from the C-101W canal to the seepage collection canal.

C.4.7 S-106

Structure S-106 is both an inflow and seepage control pump station. It is designed to pump water from the seepage collection canal into the C-18W reservoir. 175 cfs pump capacity is designed to capture runoff from upstream sources (including J.W. Corbett WMA and from S-109. A 30cfs pump station is provided for seepage control, with a redundant pump added to ensure seepage control can be maintained at all times. The 30 cfs pump provides seepage control, pumping seepage back into the C-18W reservoir.

C.4.8 S-107

Structure S-107 is located within levee L-101W, west of the C-18W reservoir. It is designed as a drop structure and culvert located at the Corbett WMA. The drop structure shall contain telemetry operated gates to aid in controlling water surface elevations within the Corbett WMA. Flap gates at the tailwater end control back flow from the seepage collection canal. The culvert is designed to convey excess runoff water from the Corbett WMA to the C-101W canal.

C.4.9 S-109

Pump station S-109 is located between the headwater of the existing M-O canal and the headwater of the proposed C-101W canal. It is designed to pump excess runoff water from the M-O canal into the C-101W canal.

C.5 Hydraulic Design Data Sheets (HDDS)

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Loxahatchee River Watershed Restoration Project
Pumping Station S-101A

Date Compiled (or revised)		**14-Jul-19**
Design Condition		
Design Condition	150	cfs
Design Capacity	150	cfs
Number of Pumps	4	
Pump Mix type and Size		
Electric 2 @ 25 cfs		
Electric 2 @ 50 cfs		
Mix Criteria		
Pumping Station will have 4 pumps - The firm pumping capacity will be 150 cfs		
Design Operating Heads		
Normal	17.50	ft, NAVD88
Maximum	19.00	ft, NAVD88
Intake Water Surface Elevations		
Maximum Non-Pumping	21.00	ft, NAVD88
Maximum Pumping	21.00	ft, NAVD88
Start Pumping	18.50	ft, NAVD88
Normal Drawdown	18.00	ft, NAVD88
Minimum Drawdown	16.00	ft, NAVD88
Minimum Non-Pump	16.00	ft, NAVD89
Invert Intake Canal	5.00	ft, NAVD88
Discharge Water Surface Elevations		
Maximum Non-Pumping	27	ft, NAVD88
Maximum Pumping	27	ft, NAVD88
Normal Pumping	25.00	ft, NAVD88
Minimum Pumping	23.00	ft, NAVD88
Minimum Non-Pump	23.00	ft, NAVD89
Invert Discharge Canal	20.50	ft, NAVD88

Figure 8. S-101A HDDS

S-101B
Service/Emergency Spillway

Revisions June 2019 - PIR

XY Coord TBD
Location C-18W Embankment
Purpose Service and Emergency Spillway

Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions

Service Spillway		
Discharge (CFS)	105	cfs
Headwater Elevation	29.3	ft,NAVD
Tailwater Elevation	19.5	ft,NAVD
Emergency Spillway		
Discharge (CFS)	1960.0	cfs
Headwater Elevation	32.7	ft,NAVD
Tailwater Elevation	19.5	ft,NAVD

Maximum Expected Stages

Headwater Elevation	32.7	ft,NAVD
Tailwater Elevation	19.5	ft,NAVD

Maximum Simultaneous Head Difference 13.17 ft

Weir Design Data

Service Spillway		
Weir Type	Broad-Crested	
Crest Elevation	28.1	feet
Weir Breadth	14.0	feet
Crest length	10.0	feet
Emergency Spillway		
Weir Type	Broad Crested	
Crest Elevation	29.3	feet
Weir Breadth	14.0	feet
Crest length	80.0	feet

Canal Section

C-18W Reservoir		
Side Slopes (Vert. on Hor.) 1 on	3	
Upstream Bottom Width	Pool	
Upstream Bottom Elevation	21.0	ft, NAVD
C-101N		
Side Slopes (Vert. on Hor.) 1 on	3	
Downstream Bottom Width	50.0	feet
Downstream Bottom Elevation	3.00	ft, NAVD

Apron / Riprap Requirements

Design Flow Velocity	5.00	fps
Design Riprap Velocity	5.00	fps
Downstream Length	50.00	feet
Downstream Width	100	feet

Figure 9. S-101B HDDS

S-102
Gated Culvert

Revisions June 2019 - PIR

XY Coord TBD
Location C-18W Embankment
Purpose C-18W Outflow Structure

Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	300	cfs
Headwater Elevation	28.00	ft,NAVD
Tailwater Elevation	17.70	ft,NAVD
Maximum Expected Stages		
Headwater Elevation	32.67	ft,NAVD
Tailwater Elevation	20.00	ft,NAVD
Minimum Expected Stages		
Headwater Elevation	20.50	ft,NAVD
Tailwater Elevation	16.00	ft,NAVD
Maximum Simultaneous Head Difference	12.67	ft
Culvert Data		
Number of Barrels	2	
Barrel Type	Concrete	
Barrel Height	6.0	feet
Barrel Width	4.0	feet
Barrel Length	50.0	feet
Barrel Invert Elevation (upstream)	21.00	ft, NAVD
Barrel Invert Elevation (downstream)	21.00	ft, NAVD
Type of Control (Upstream)		Slide Gate
Type of Control (Downstream)		Flap Gate
Canal Section		
Reservoir Pool		
Side Slopes (Vert. on Hor.) 1 on	3	
Upstream Bottom Width	Pool	
Upstream Bottom Elevation	21.00	ft, NAVD
C-101N		
Side Slopes (Vert. on Hor.) 1 on	3	
Downstream Bottom Width	50.00	
Downstream Bottom Elevation	5.00	ft, NAVD
Canal Riprap Requirements		
Design Flow Velocity	4.17	fps
Design Riprap Velocity	5.00	fps
Downstream Length	32.0	feet
Downstream Width	25.0	feet

Figure 10. S-102 HDDS

S-103E
Gated Culvert

Revisions June 2019 - PIR

XY Coord TBD
Location Seepage collection canal - C-101E
Purpose Convey water from C-101E canal to C-101N (Outflow Canal)

Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	500	cfs
Headwater Elevation	20.00	ft,NAVD
Tailwater Elevation	17.70	ft,NAVD
Maximum Expected Stages		
Headwater Elevation	22.00	ft,NAVD
Tailwater Elevation	20.00	ft,NAVD
Minimum Expected Stages		
Headwater Elevation	17.50	ft,NAVD
Tailwater Elevation	16.00	ft,NAVD
Maximum Simultaneous Head Difference	4.50	ft
Culvert Data		
Number of Barrels	1	
Barrel Type	Concrete	
Barrel Height	10.0	feet
Barrel Width	10.0	feet
Barrel Length	20.0	feet
Barrel Invert Elevation (upstream)	5.00	ft, NAVD
Barrel Invert Elevation (downstream)	5.00	ft, NAVD
Type of Control (Upstream)	Slide Gate	
Type of Control (Downstream)	Flap Gate	
Canal Section		
Side Slopes (Vert. on Hor.) 1 on	3	
Upstream Bottom Width	20.00	ft
Upstream Bottom Elevation	5.00	ft, NAVD
Side Slopes (Vert. on Hor.) 1 on	3	
Downstream Bottom Width	20.00	
Downstream Bottom Elevation	5.00	ft, NAVD
Canal Riprap Requirements		
Design Flow Velocity	5.00	fps
Design Riprap Velocity	5.00	fps
Downstream Length	80.0	feet
Downstream Width	62.0	feet

Figure 11. S-103E HDDS

S-103W
Gated Culvert

Revisions June 2019 - PIR

XY Coord TBD
Location Seepage collection canal - C-101E
Purpose Convey water from C-101W canal to C-101N (Outflow Canal)

Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	500	cfs
Headwater Elevation	20.00	ft,NAVD
Tailwater Elevation	17.70	ft,NAVD
Maximum Expected Stages		
Headwater Elevation	22.00	ft,NAVD
Tailwater Elevation	20.00	ft,NAVD
Minimum Expected Stages		
Headwater Elevation	17.50	ft,NAVD
Tailwater Elevation	16.00	ft,NAVD
Maximum Simultaneous Head Difference	4.50	ft
Culvert Data		
Number of Barrels	1	
Barrel Type	Concrete	
Barrel Height	10.0	feet
Barrel Width	10.0	feet
Barrel Length	20.0	feet
Barrel Invert Elevation (upstream)	5.00	ft, NAVD
Barrel Invert Elevation (downstream)	5.00	ft, NAVD
Type of Control (Upstream)	Slide Gate	
Type of Control (Downstream)	Flap Gate	
Canal Section		
Side Slopes (Vert. on Hor.) 1 on	3	
Upstream Bottom Width	20.00	ft
Upstream Bottom Elevation	5.00	ft, NAVD
Side Slopes (Vert. on Hor.) 1 on	3	
Downstream Bottom Width	20.00	
Downstream Bottom Elevation	5.00	ft, NAVD
Canal Riprap Requirements		
Design Flow Velocity	5.00	fps
Design Riprap Velocity	5.00	fps
Downstream Length	80.0	feet
Downstream Width	62.0	feet

Figure 12. S-103W HDDS

S-104
Ungated Culvert

Revisions June 2019 - PIR

XY Coord TBD
Location Seepage collection canal
Purpose Convey water in the C-101E canal

Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	500	cfs
Headwater Elevation	20.00	ft,NAVD
Tailwater Elevation	19.50	ft,NAVD
Maximum Expected Stages		
Headwater Elevation	22.00	ft,NAVD
Tailwater Elevation	22.00	ft,NAVD
Minimum Expected Stages		
Headwater Elevation	17.50	ft,NAVD
Tailwater Elevation	17.50	ft,NAVD
Maximum Simultaneous Head Difference	4.50	ft
Culvert Data		
Number of Barrels	1	
Barrel Type	Concrete	
Barrel Height	10.0	feet
Barrel Width	10.0	feet
Barrel Length	30.0	feet
Barrel Invert Elevation (upstream)	5.00	ft, NAVD
Barrel Invert Elevation (downstream)	5.00	ft, NAVD
Type of Control	none	
Canal Section		
C-101E		
Side Slopes (Vert. on Hor.) 1 on	3	
Upstream Bottom Width	20.00	ft
Upstream Bottom Elevation	5.00	ft, NAVD
C-101E		
Side Slopes (Vert. on Hor.) 1 on	3	
Downstream Bottom Width	Lateral	
Downstream Bottom Elevation	5.00	ft, NAVD
Canal Riprap Requirements		
Design Flow Velocity	5.00	fps
Design Riprap Velocity	5.00	fps
Downstream Length	80.0	feet
Downstream Width	62.0	feet

Figure 13. S-104 HDDS

S-105
Gated Culvert

Revisions June 2019 - PIR

XY Coord TBD
Location From C-101W (MO-Connector) to C-101E C-18W seepage collection canal
Purpose convey water to C-18W reservoir

Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	150	cfs
Headwater Elevation	18.00	ft,NAVD
Tailwater Elevation	17.50	ft,NAVD
Maximum Expected Stages		
Headwater Elevation	24.00	ft,NAVD
Tailwater Elevation	22.00	ft,NAVD
Minimum Expected Stages		
Headwater Elevation	17.50	ft,NAVD
Tailwater Elevation	17.50	ft,NAVD
Maximum Simultaneous Head Difference	6.50	ft
Culvert Data		
Number of Barrels	3	
Barrel Type	Concrete	
Barrel Diameter	6.5	feet
Barrel Length	79.5	feet
Barrel Invert Elevation (upstream)	11.00	ft, NAVD
Barrel Invert Elevation (downstream)	11.00	ft, NAVD
Type of Control	Flap Gate	
Canal Section		
C-101W		
Side Slopes (Vert. on Hor.) 1 on	2.5	
Upstream Bottom Width	10.00	ft
Upstream Bottom Elevation	10.25	ft, NAVD
C-101E		
Side Slopes (Vert. on Hor.) 1 on	3	
Downstream Bottom Width	Lateral	
Downstream Bottom Elevation	9.75	ft, NAVD
Canal Riprap Requirements		
Design Flow Velocity	2.10	fps
Design Riprap Velocity	5.00	fps
Downstream Length	44.0	feet
Downstream Width	35.0	feet

Figure 14. S-105 HDDS

Loxahatchee River Watershed Restoration Project
Pumping Station S-106

Date Compiled (or revised)		**14-Jul-19**
Design Condition		
Inflow Pump Design Condition	175	cfs
Inflow Pump Design Capacity	175	cfs
Seepage Pump Design Condition	30	cfs
Seepage Pump Design Capacity	60	cfs
Number of Pumps	7	
Pump Mix type and Size		
Electric 3 @ 25 cfs		
Electric 2 @ 50 cfs		
Electric 1 @ 30 cfs		
Electric 1 @ 30 cfs (backup)		
Mix Criteria		
Pumping Station will have 5 pumps - The firm pumping capacity will be 175 cfs		
2 - 30 cfs - Submersible Electric Pumps for Flood Control		
Design Operating Heads		
Normal	17.50	ft, NAVD88
Maximum	19.00	ft, NAVD88
Intake Water Surface Elevations		
Maximum Non-Pumping	21.00	ft, NAVD88
Maximum Pumping	21.00	ft, NAVD88
Start Pumping	18.50	ft, NAVD88
Normal Drawdown	18.00	ft, NAVD88
Minimum Drawdown	16.00	ft, NAVD88
Minimum Non-Pump	16.00	ft, NAVD89
Invert Intake Canal	5.00	ft, NAVD88
Discharge Water Surface Elevations		
Maximum Non-Pumping	27	ft, NAVD88
Maximum Pumping	27	ft, NAVD88
Normal Pumping	25.00	ft, NAVD88
Minimum Pumping	23.00	ft, NAVD88
Minimum Non-Pump	23.00	ft, NAVD89
Invert Discharge Canal	20.50	ft, NAVD88

Figure 15. S-106 HDDS

S-107
Gated Culvert

Revisions June 2019 - PIR

XY Coord
Location Corbette Area to canal C-101W
Purpose convey and control inflow water to C-101W canal

Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	300	cfs
Headwater Elevation	22.60	ft,NAVD
Tailwater Elevation	19.50	ft,NAVD
Maximum Expected Stages		
Headwater Elevation	22.60	ft,NAVD
Tailwater Elevation	22.00	ft,NAVD
Minimum Expected Stages		
Headwater Elevation	17.50	ft,NAVD
Tailwater Elevation	17.50	ft,NAVD
Maximum Simultaneous Head Difference	5.10	ft
Culvert Data		
Number of Barrels	3	
Barrel Type	Concrete	
Barrel Diameter	6.0	feet
Barrel Length	150.0	feet
Barrel Invert Elevation (upstream)	11.00	ft, NAVD
Barrel Invert Elevation (downstream)	10.50	ft, NAVD
Type of Control (Upstream)	Weir Gate	
Type of Control (Downstream)	Flap Gate	
Weir Data		
Crest Length	32	feet
Crest Elevation	19.52	
Maximum Head on Crest (feet)	22.60	ft
Shape	Broad Crested	
Weir Control	Weir Gate / Flap Gate	
Canal Riprap Requirements		
Design Flow Velocity	5.00	fps
Design Riprap Velocity	5.00	fps
Downstream Length	48.0	feet
Downstream Width	38.0	feet

Figure 16. S-107 HDDS

Loxahatchee, Florida
Pumping Station S-109

Date Compiled (or revised)		**14-Jul-19**
Design Condition		
Design Condition	150	cfs
Design Capacity	150	cfs
Number of Pumps	4	
Pump Mix type and Size		
Electric 2 @ 25 cfs		
Electric 2 @ 50 cfs		
Mix Criteria		
Pumping Station will have 4 pumps - The firm pumping capacity will be 150 cfs		
Design Operating Heads		
Normal	2.50	ft, NAVD88
Maximum	5.50	ft, NAVD88
Intake Water Surface Elevations		
Maximum Non-Pumping	19.00	ft, NAVD88
Maximum Pumping	19.00	ft, NAVD88
Start Pumping	15.5 - 16.5	ft, NAVD88
Normal Drawdown	15 - 16	ft, NAVD88
Minimum Drawdown	14.5 -15.5	ft, NAVD88
Minimum Non-Pump	14.5 -15.6	ft, NAVD89
Invert Intake Canal	7.50	ft, NAVD88
Discharge Water Surface Elevations		
Maximum Non-Pumping	20	ft, NAVD88
Maximum Pumping	19	ft, NAVD88
Normal Pumping	18.00	ft, NAVD88
Minimum Pumping	16.50	ft, NAVD88
Minimum Non-Pump	16.50	ft, NAVD89
Invert Discharge Canal	10.50	ft, NAVD88

Figure 17. S-109 HDDS

C.6 Spillway and Culvert Calculations

The flow capacity for the proposed structures was calculated using the energy balance equation for submerged uncontrolled flow.

$$\Delta H = K_e \frac{V^2}{2g} + K_o \frac{V^2}{2g} + L\left(\frac{Q * n}{1.486 * A * {R_h}^{2/3}}\right)^2$$

Where

ΔH is the head differential (ft.)
V is the velocity (fps)
Q is the flow (cfs)
L is the barrel length (ft.)
A is the cross sectional area (ft^2)
Rh is the hydraulics radius (ft.)
n is the Manning's roughness coefficient
g is the gravitational acceleration (ft/s^2)
Ke is the entrance loss coefficient
Ko is the exit loss coefficient.

Gated culvert structures provide flow control using sliding gates. The design discharges for the project's gated culverts will assume the gates are completely open, out of the water, and do not restrict flow through the structures. The conceptual designs of these structures will be standardized whenever possible to simplify design, construction, and future operations and maintenance.

Culverts are hydraulically short conduits that convey flow through embankments, levees, access roads or past some other obstruction to flow. They are constructed from a variety of materials and are available in many different shapes and configurations. They may be single-barrel or multiple-barrel structures. An exact theoretical analysis of culvert flow is extremely complex because the flow is usually non-uniform, with regions of varying flow. Hydraulic jumps often form inside or downstream of the culvert barrel. As the flow rate and tail water elevations change, the flow type within the barrel changes. An exact hydraulic analysis involves backwater and drawdown calculations, energy and momentum balance, and application of the results of hydraulic studies. An extensive hydraulic analysis is usually impractical and not warranted for the design of most culverts. Culvert design, therefore, is often based on empirical approximations. One of the first steps is to establish the type of control that applies.

The types of control used in the design of culverts are:

- Inlet Control – Inlet control occurs when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section of a culvert operating under inlet control is just inside the barrel entrance. Critical depth occurs at or near this location, and the flow regime immediately downstream is supercritical. Hydraulic characteristics downstream of the inlet control section do not affect the culvert capacity. The upstream water surface elevation and the inlet geometry represent the major flow

controls. The inlet geometry includes the inlet shape, inlet cross-sectional area, and the inlet configuration.

- Outlet Control – Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. The control section for outlet control flow in a culvert is located at the barrel exit or further downstream. Either subcritical or pressure flow exists in the culvert barrel under these conditions. All of the geometric and hydraulic characteristics of the culvert play a role in determining its capacity. These characteristics include all of the factors governing inlet control, the water surface elevation at the outlet, and the barrel characteristics.

For each type of control, different factors and formulas are used to compute the hydraulic capacity of a culvert. Under inlet control, the cross-sectional area of the culvert, inlet geometry, and elevation of headwater at entrance are of primary importance. Outlet control involves the additional consideration of the tail water elevation of the outlet channel and the slope, roughness and length of the culvert barrel. A discussion of these two types of control with charts for selecting a culvert size for a given set of conditions is included in the Federal Highway Administration (FHWA) Hydraulic Design Series No. 5, "Hydraulic Design of Highway Culverts."

Spreadsheet tools were used to perform initial calculations for the appropriate sizing and number of culvert barrels based on design flow rates, head water and tail water conditions, as well as losses due to entrance, exit and necessity of gates. HEC-RAS modeling was used to confirm and further refine the design of the Recommended Plan culverts. The design flow that is provided for a project culvert is for the entire structure, whether it is single- or multiple-barrel. Stilling basins and riprap will be added to many of the culvert designs to provide energy dissipation and erosion protection. Computational methods for culvert design are described further in EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works, EM 1110-2-2902, Conduits, Culverts and Pipes, and FHWA-HIF-12-026, Hydraulic Design of Highway Culverts.

The flow capacity for the proposed overflow spillway was calculated using the standard equation for weir flow.

$$Q = CLH^{1.5}$$

Where
C is the weir coefficient
L is the crest length
H is the head above the spillway

C.7 Rainfall & Reservoir Routing

The C-18W reservoir is the main component of flow-way 2. It is designed to store water and releases water when needed downstream. The Hazard Potential Classification (HPC) of the C-18W reservoir was determined following the guidance in the Design Criteria Memorandum number 1 (DCM-1) – Hazard Potential Classification (2005). The reservoir is located north of the Acreage residential community and due to the possibility of life loss during an unlikely breach event, the impoundment carries a "high" hazard HPC.

Following the guidance in the DCM-2 – Wind Precipitation Design Criteria for Freeboard - (2005), the design storm for the C-18W reservoir was selected to be the Probable Maximum Precipitation (PMP). The PMP was developed according to guidelines from the Hydrometeorological Report (HMR) No. 51 (Probable Maximum Precipitation Estimates, Eastern U.S.) and HMR No. 52 (Probable Maximum Storm Computation). A value of 55.7 inches was selected from the all season 72 hr, 10 mi^2 PMP chart.

C.7.1 Rainfall

Hydraulic structure design was performed for the 100-year event, 72 hour duration rainfall, totaling 16.3 inches. A synthetic hyetograph with no volume removed as initial abstraction was used as input in the model, ensuring a conservative approach. As specified previously, the design storm for the C-18W reservoir was selected to be the PMP.

C.7.2 Spillway Capacity & Reservoir Routing

The embankment height and freeboard were designed following the guidance in the DCM-2 and DCM-3 – Spillway capacity and Reservoir Drawdown Criteria (2006). A wind and wave study for Case 1 (PMP event combined with 100-yr wind condition) was performed to determine the embankment height. Over wash rate for the embankment height design was 0.1 cfs/lf. The selected over wash rate was coordinated with the SFWMD and was selected as a conservative estimate.

Routing of the design storm (PMP) was performed with antecedent flood conditions following the guidance in DCM-2:

- Routing started at normal full storage level
- Routed 30 percent of the 72-hour PMF (Time scale 0-hr to 72-hr): un-gated spillway flows, gated structures are in-operable.
- A 3-day dry interval with un-gated spillway flow, gated structures are operable (Time scale 72-hr to 144-hr).
- Routed 100 percent of the 72-hour PMF (Time scale 144-hr to 216-hr): un-gated spillway flows, gated structures are in-operable.
- A 10-day dry interval with un-gated spillway flow, gated structures are operable (Time scale 216-hr to 456-hr).
- Routed 30 percent of the 72-hour PMF (Time scale 456-hr to 528-hr): un-gated spillway flows, gated structures are in-operable.

The proposed spillway for the C-18W reservoir was designed as an uncontrolled overflow structure that has a dual function as a service spillway and auxiliary spillway. The service spillway crest elevation was set 0.1 ft. above the normal pool elevation, the length was designed to not exceed the allowable discharge of 41.6 cfs per square mile for the C-18 basin, as indicated in the “Basis of Review for Environmental Resource Application with SFWMD” (SFWMD, 2012). The auxiliary spillway crest elevation was set at the stage resulting from routing of the 100-year, 72 hour duration rainfall event. Auxiliary spillway length was designed to minimize possible adverse consequences downstream.

C.7.3 Wind and Wave Analysis

The wind and wave analysis was completed for the C-18W Reservoir. The purpose of the study is to apply wave run-up and corresponding over-wash rate estimates for a combination of water stage and design wind to determine recommended embankment heights for the Loxahatchee River Watershed Restoration Project (LRWRP). Water stage and design wind conditions were developed using methodologies consistent with DCM-2, which provides guidance for developing combined wind speed and precipitation events for freeboard determination of impoundments, reservoirs, and dams, specifically for application to projects located in South Florida. Wave run-up and over-wash rates, when combined with structural design and site specific geotechnical information, provide a key component in the determination of the optimum levee height for a given project location. Results from this analysis are presented as part of the larger investigation and do not constitute, in and of themselves, a recommendation for a final levee height design.

The LRWRP project covers a region of South Florida, east of Lake Okeechobee (**Figure 18**). This analysis focusses on the potential impoundment site, C-18W.

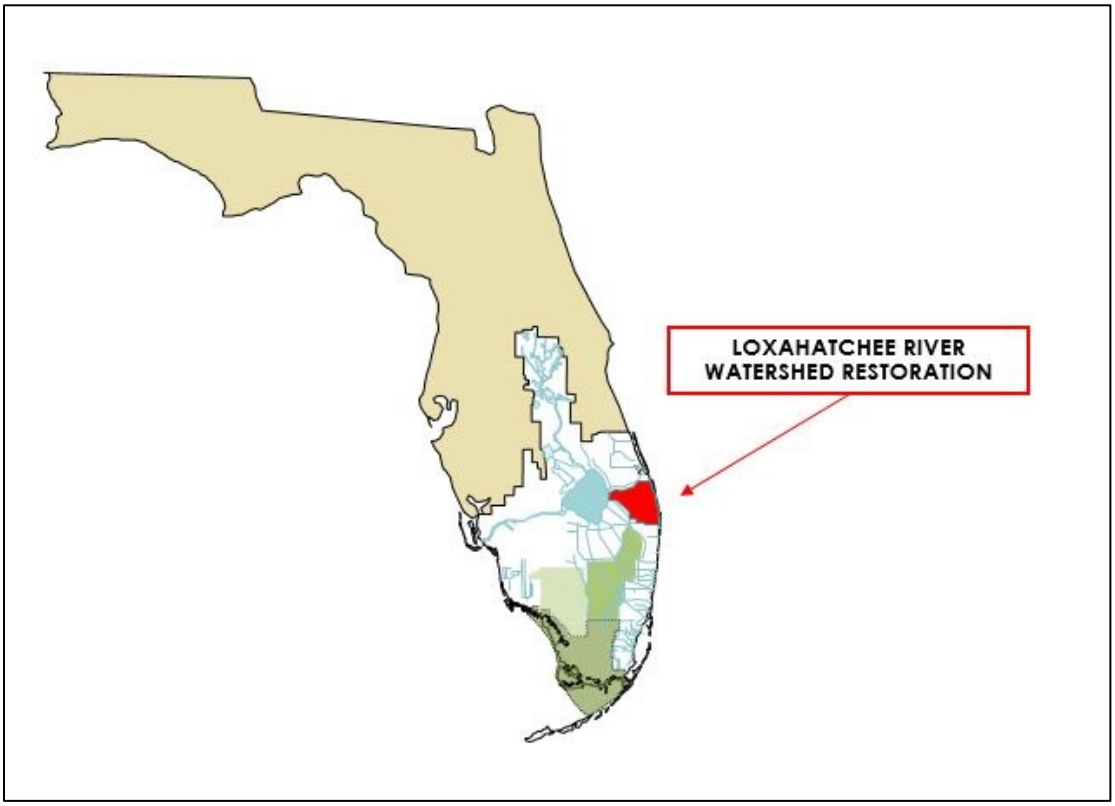


Figure 18. LRWRP Watershed Project Area

C.7.4 Methodology

Determination of design criteria requires combinations of starting water level (normal pool), precipitation, and wind conditions that can reasonably be expected to occur simultaneously. DCM-2 provides four design "Cases" for evaluation:

Case 1 – PMP event combined with a 100-year wind condition.

Case 2 – 100-year precipitation event combined with a Category 5 hurricane wind condition.

Case 3 – NFSL (normal full storage level) combined with a Probable Maximum Wind (PMW) condition.

Case 4 – Storm specific (historical) precipitation/wind event.

For this preliminary screening level analysis, only Case 1 (a PMP event of 55.7 inches) was investigated. This is acceptable for screening level evaluation as Case 1 has been the predominant governing case in previous South Florida investigations.

Wave heights due to wind-growth within each impoundment were estimated for range of water depths beginning with the average normal pool depth and increasing by half foot increments to the next highest incremental depth above the PMP water depth. This resulted in a depth range of 7.0 feet to 12.0 feet for C-18W. PMP water depth of the C-18 is 11.6 feet (corresponding to 31.6 ft-NAVD88).

Based on resulting wave conditions and predicted storm surge, wave run-up and over-wash were calculated using the US Army Corps of Engineers (USACE) Automated Coastal Engineering System (ACES) (USACE, 1992). Over-wash was estimated for a single levee configuration: a smooth, flat plate soil cement armor layer with a 1:3 slope.

C.7.5 Wave Heights and Periods

Wind waves, also known as oscillatory waves, are a key component in determining minimum freeboard. Wind waves are generated when wind blows over an open expanse of water. They grow in magnitude over time and, in a confined environment such as a reservoir or impoundment, will eventually encounter one of the surrounding embankments. As a result of this encounter water will rush up the embankment slope (wave run-up) and if the run-up elevation exceeds the elevation of the embankment crest it will produce over-wash.

Wind waves are most commonly defined by their height (crest to trough), length (crest to crest), and period (time between consecutive crests). In reality, wind driven waves are a highly complex mix of crests and troughs on the water surface, collectively referred to as a wave climate or wave field. Distribution of individual waves in a wave field can be closely approximated using the cumulative form of the Rayleigh distribution (USACE, 1984). Based on this distribution, the wave climate is most frequently represented by the significant wave height, Hs, (defined as the average height of the one-third highest waves) and the corresponding peak wave period. These parameters, as determined at a given location, are dependent on the following factors: the elevation of the pool (water depth), the velocity of the wind (wind speed), the distance over which the wind blows (fetch), the duration over which the wind blows, and the decay distance the wave travels after leaving the generating area (not relevant in a confined impoundment).

Due to the highly complex nature of wave field development, wave prediction is most accurately accomplished using computer based wave prediction models that can incorporate non-linear wave-wave interactions as well as wind inputs. However, when the level of project design is at a preliminary stage, a simplified analytical method may be employed to screen preliminary design scenarios. Evaluation of the wave climate for the final design should be completed using an appropriate numerical wave transformation model.

C.7.6 Wind Wave Prediction – Simplified Method

Several simplified formulas for wave prediction have been developed based on measured data (USACE, 1984). These include formulas for both deep-water wave prediction and shallow-water wave prediction. Deep water waves generally do not occur in reservoirs or impoundments unless they are exceptionally deep and the wave climate is low energy. Therefore, shallow-water wave prediction formulas are applicable to reservoirs and impoundments in south Florida.

Although there is no single theoretical method for determining the actual growth of waves generated by winds blowing over relatively shallow water, the forecasting method applied to this evaluation is consistent with DCM-2. The following formulas for the significant wave height (H_s) and shallow water wave period (T) are considered to be valid representations of wave development in South Florida impoundments:

$$\frac{gH_s}{U_A^2} = 0.283 \tanh\left[0.530\left(\frac{gd}{U_A^2}\right)^{\frac{3}{4}}\right] \tanh\left\{\frac{0.00565\left(\frac{gF}{U_A^2}\right)^{\frac{1}{2}}}{\tanh\left[0.530\left(\frac{gd}{U_A^2}\right)^{\frac{3}{4}}\right]}\right\}$$

$$\frac{gT}{U_A} = 7.54 \tanh\left[0.833\left(\frac{gd}{U_A^2}\right)^{\frac{3}{8}}\right] \tanh\left\{\frac{0.0379\left(\frac{gF}{U_A^2}\right)^{\frac{1}{3}}}{\tanh\left[0.833\left(\frac{gd}{U_A^2}\right)^{\frac{3}{8}}\right]}\right\}$$

Where,

H_s = Significant wave height
T = Wave Period
g= Acceleration due to gravity
U_A = Wind stress factor
d = Water depth
F = Fetch distance

By employing these equations using the maximum fetch condition, representative significant wave heights and corresponding wave periods can be determined. These heights and periods represent the expected significant wave height condition occurring at the up-wind embankment and are subsequently used in determining expected wave run-up and over-wash rates.

C.7.6.1 Wind Stress Factor, UA

The wind stress factor is essentially a wind speed that has been adjusted to account for drag. The wind stress factor is determined using the following formula:

$$U_A = 0.589\ U^{1.23}$$

Where U is the final design wind speed in miles per hour.

C.7.6.2 Design Wind

It is assumed that each wind-wave growth case begins with a still water surface. Following procedures outline in DCM-2, the 100-year (project specific Case 1) design wind was computed to be 104.6 mph for C-18W reservoir.

C.7.6.3 Fetch

Fetch is defined as a distance over which the wind speed and direction are reasonably constant. Fetches fall into two categories, open-water fetches, where wave growth is limited only by the incident meteorological conditions, and restricted fetches, where wave growth is limited by a confined geometry such as that of a lake, river, bay, or reservoir. The C-18W Reservoir is subject to a restricted fetch. The restricted fetch methodology applies the concept of wave development in off-wind directions and considers the shape of the basin. The fetch is defined as the radial average over an arc of 24 degrees centered on the wind direction. For this study, the wind direction is taken to be the direction corresponding to the maximum fetch distance. **Figure 19** show the 24-degree arc (divided into 3-degree intervals) that provides the maximum fetch for the conceptual design layout. Averaging the radial lengths over each arc gives average fetch length of 1.9 miles for C-18W.

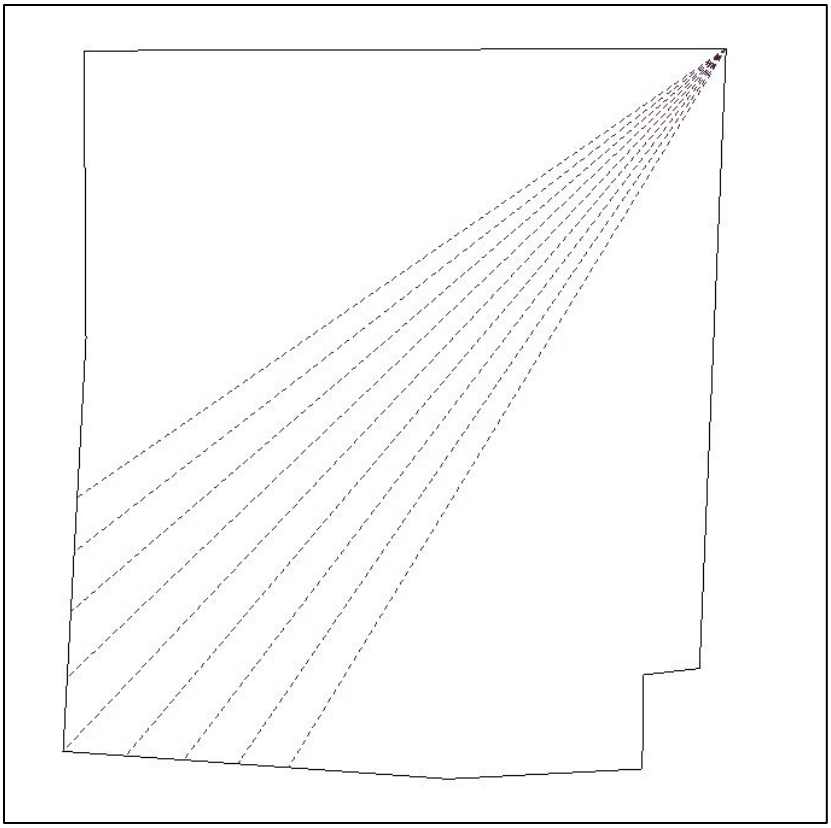


Figure 19. Fetch Determination for C-18

C.7.7 Wave Height Results

Wind waves, also known as oscillatory waves, are most commonly defined by their height, length, and period. These characteristics, as measured at a given location, are determined predominantly by the velocity of the wind (wind speed), the distance over which the wind blows (fetch), the duration over which the wind blows, the decay distance the wave travels after leaving the generating area (not applicable here), and the water depth.

The process of wind wave growth (assuming initial still water) begins with the motion of the air above the water disturbing the surface of the water. As wind begins to blow small perturbations form in the water surface. When the perturbations become large enough, a transfer of momentum and energy between the air layer and the water surface occurs, rapidly increasing wave heights. Wave heights build rapidly over a relatively short distance, increasing at a rate that varies dependent upon wind speed. Waves continue to grow until they reach either a fetch-limited condition (maximum wave height limited by the fetch distance) or a depth-limited condition (maximum wave height limited by the depth of the water). In each of the current design scenarios wave heights were found to be fetch-limited.

Table 1 presents the significant wave height, corresponding wave period, and wind direction for The C-18W Reservoir.

Table 1. Significant Wave Height and Peak Period

C-18W Reservoir		
Still Water Pool Depth (feet)	Significant Wave Height (feet)	Peak Wave Period (second)
7.00	4.12	3.44
7.50	4.28	3.46
8.00	4.44	3.48
8.50	4.58	3.50
9.00	4.71	3.52
9.50	4.84	3.53
10.00	4.96	3.54
10.50	5.07	3.56
11.00	5.17	3.57
11.50	5.27	3.58
12.00	5.36	3.59

C.7.8 Wind Set Up Analysis

Wind set up occurs when wind blows in a fairly constant direction over a body of water. Wind blowing across the water causes the surface layer to move in the direction of the wind due to shear stress development between the air and water. This causes water to “pile up” against the shore, creating the effect known as “wind setup”. A corresponding drop in water level will occur along the upwind shore.

A number of wind setup models exist, both computer based numerical models and analytical models. Due to the simple geometry of the reservoir, it was determined that an analytical model, based on a maximum effective fetch distance would be appropriate for this analysis. The Bretschneider method (Ippen, 1966) is a physics based model allowing for the different shear forces developed by the wind on the water surface and by return flow on the bottom bed. This model was originally solved numerically using Lake Okeechobee data and appears well founded on water bodies in South Florida. Bretschneider’s model was therefore selected as the model to calculate wind setup for C-18W.

C.7.8.1 Bretschneider

The numerical model presented by Bretschneider was derived under steady-state wind conditions, i.e. constant speed and direction along a channel axis. The differential form of the model, based on surface and bottom shear stresses as well as conservation of mass, determines wind setup as a function of the effective stress. The effective stress parameter employed in this method can be expressed as:

$$\frac{\kappa U^2 F}{g d^2}$$

where,

κ = wind drag coefficient

U = wind speed

F = fetch length

g = acceleration due to gravity

d = water depth

Based on the calculated value of this stress parameter, wind setup can be determined from tabulated data presented in Ippen (1966).

It should be noted that the wind drag coefficient, κ, is not a constant. Its value can be influenced by such factors as wind speed, wave steepness (eddy correlation), and salinity. For this study, a wind drag coefficient value of 3.3×10^{-6} was assigned. This value is based upon studies of Lake Okeechobee (Ippen, 1966) and is considered to be representative of conditions at C-18W.

C.7.8.2 Wind Set Up Results

Wind setup values, corresponding to each cell, are provided in **Table 2**. Note that wind setup increases as depth decreases. This is due to less space for upwind return flow along the bottom as well as increased friction on that return flow, allowing more water to "pile up" downwind.

C.7.9 Wave Run-Up and Over-Wash Analysis

C.7.9.1 Effective Depth

Although the generation of wind waves (and therefore wave run-up and over-wash) is influenced by the presence of wind setup, the relationship is highly complex and is not presently included in the wave equations. Therefore, in order to include the total water level increase due to wind conditions at the down-fetch face of the levee, wind setup is accounted for by adding the wind setup to the design water depth at the toe of the structure (**Table 2**). The resulting "effective depth" then becomes the water depth used during calculation of wave run-up and subsequent over-wash.

Table 2. Wind Setup Results

C-18W Reservoir		
Still Water Depth (feet)	Wind Setup (feet)	Effective Depth (feet)
7.0	1.64	8.64
7.5	1.54	9.04
8.0	1.45	9.45
8.5	1.37	9.87
9.0	1.30	10.30
9.5	1.24	10.74
10.0	1.18	11.18
10.5	1.13	11.63
11.0	1.08	12.08
11.5	1.04	12.54
12.0	0.99	12.99

C.7.9.2 Wave Run-Up

Wave run-up can be described as the resulting forward translation of water mass that is converted from wave energy as waves encounter a sloped surface. Water rushes up the slope resulting in the vertical rise above the still water line known as run-up (**Figure 20**). Run-up depends primarily on incident wave steepness, slope of the surface on which it is incident, and characteristics of the surface, including surface roughness and porosity. For a given fixed slope, run-up will increase with decreasing wave steepness. For a given incident wave height and period, run-up will increase with increasing slope steepness.

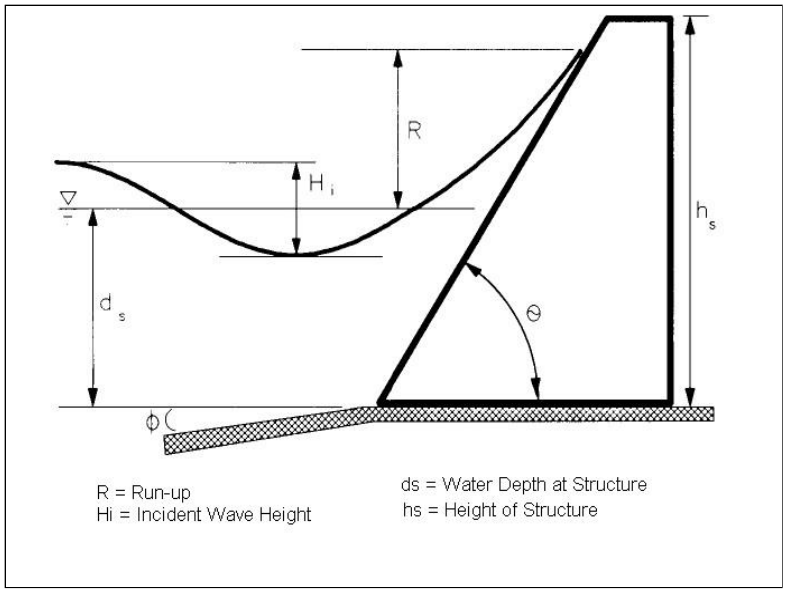


Figure 20. Wave Run-up Diagram

C.7.9.2.1 ACES

The USACE ACES software package contains an application that provides for the estimation of wave run-up on both rough and smooth impermeable surfaces. For this analysis only the smooth slope run-up formulation is presented

C.7.9.2.1.1 Smooth Slope Wave Run-Up

Smooth slope run-up, as presented in USACE ACES guidance (USACE, 1992), recommends the following general equation:

$$R = CH_i$$

where *R* is the run-up height, H_i is the incident wave height, and *C is a* coefficient characterized by the surf parameter:

$$\xi = \frac{\tan\theta}{\sqrt{\frac{H_i}{L_o}}}$$

(where θ is the angle between structure face and horizontal (18.435° = 1V:3H slope) and L_o is the deepwater wavelength.)

C is characterized according to three specific wave-structure scenarios:

$\xi \leq 2$ - waves plunging directly on the run-up slope.

$\xi \geq 3.5$ – wave conditions that are nonbreaking and are regarded as standing or surging waves.

$2 < \xi < 3.5$ – transition conditions where breaking characteristics are difficult to define.

Corresponding expressions for coefficient *C* for each scenario are detailed in the ACES technical guidance (USACE, 1992).

C.7.9.2.2 Results

Wave run-up values are presented in **Table 3**.

C.7.9.3 Over-wash

Over-wash occurs when wave run-up and wind setup levels combine to produce a water level greater than the height of the levee. Over-wash is an important design element both in terms of predicting backside flooding and safeguarding structural integrity of the levee. When waves over-wash a structure it occurs intermittently, not continuously, as individual high waves among a multitude of storm waves attack the face of the structure. Over-wash is typically calculated as a mean rate of over-wash volume per unit length over the duration of the storm event.

Table 3. Wave Run-up Values

C-18W Reservoir	
Still Water Depth (feet)	Wave Run-up (feet)
7.0	5.28
7.5	5.41
8.0	5.54
8.5	5.66
9.0	5.78
9.5	5.87
10.0	5.96
10.5	6.06
11.0	6.14
11.5	6.21
12.0	6.28

C.7.9.3.1 ACES

The USACE ACES software package provides an over-wash methodology that presents results according to a mean over-wash rate per unit length of structure. The ACES methodology contains two computations, one for monochromatic wave over-wash the other for irregular wave over-wash (USACE, 1992). Due to the irregular nature of the developed wave fields, the later method was selected for this analysis.

ACES estimates the over-wash rate by summing the over-wash contributions from each individual member of the run-up distribution:

$$Q = \frac{1}{199}\sum_{i=1}^{199} Q_i$$

Where Q is the volume rate of over-wash caused by irregular waves (cubic feet per second per linear foot - cfs/lf) and Q_i is the volume rate of over-wash caused by one run-up on the run-up distribution defined by:

$$= C_w \sqrt{gQ_o^*(H_{so})^3}\left(\frac{R_i + F}{R_i - F}\right)^{\frac{-0.1085}{\alpha}}$$

Where C_w is the wind correction factor, g is the gravitational acceleration, Q^*_o *and* α are empirical coefficients (USACE, 1984), H_{so} is the deepwater significant wave height, F is the freeboard (levee height – water depth), R_i is the run-up value having exceedance probability:

$$p = \sqrt{\frac{\ln\frac{1}{p}}{2}}R_s$$

Where R_s is run-up with a given significant wave height and period.

When the freeboard is greater than the significant run-up, larger waves in the distribution may still over-wash the structure. For these relatively high freeboards, the run-up distribution is broken into 999 elements, instead of 199, to better account for the effect of the higher run-ups. The over-wash equation for this larger distribution becomes:

$$Q = \frac{1}{999}\sum_{i=1}^{999} Q_i$$

C.7.9.4 Over-wash Flow rates

Preliminary levee heights can be determined by matching an embankment height to an allowable over-wash rate. The maximum allowable over-wash rate is a function of site-specific geotechnical information as well as the proposed structural design of the embankment. For this analysis three allowable over-wash rates were evaluated: 0.40cfs/lf (based on previous slope stability analyses conducted in South Florida), 0.10cfs/lf (generally accepted threshold for preliminary analysis), and 0.01cfs/lf (conservative threshold).

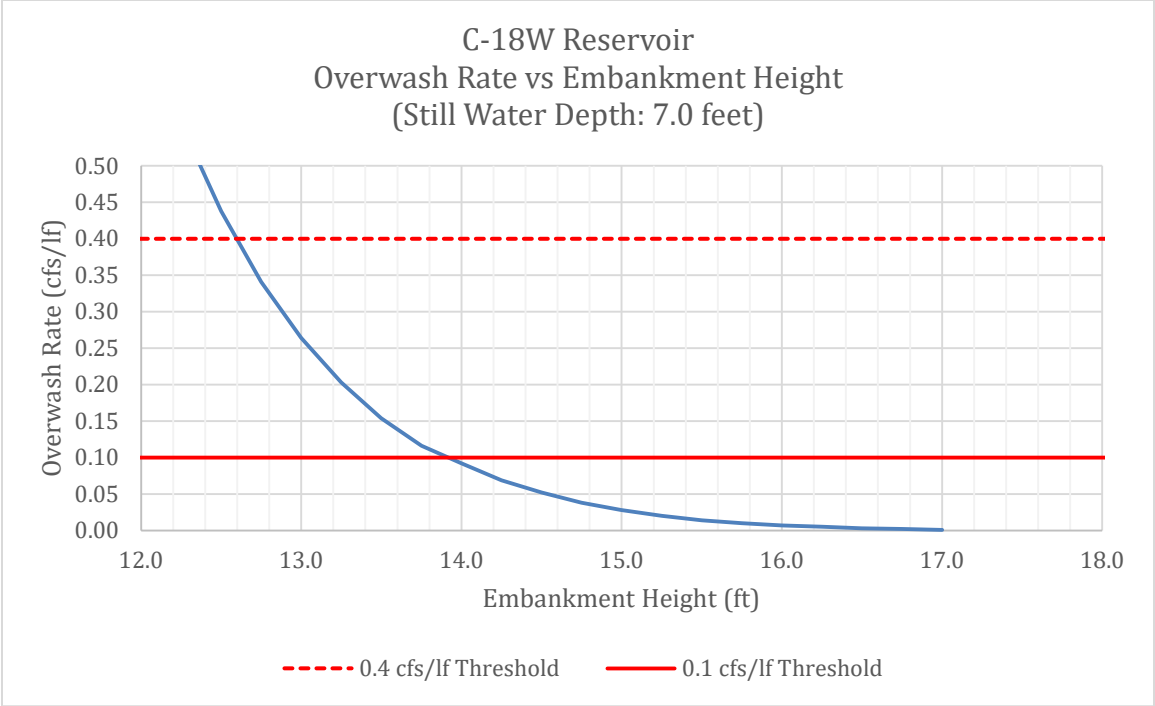
Table 4 provides mean rates of over-wash for wave conditions for C-18W Reservoir. Over-wash rates correspond to embankment heights shown in increments of 0.5 feet combined with specified still water levels (including precipitation). Allowable over-wash rates may fall between two consecutive embankment heights in the tables. Due to constructability resolution of the embankment heights is usually limited to 0.5 feet. Therefore, it must be determined is the final embankment heights is rounded up or down based on the corresponding over-wash rate.

Figure 21 to **Figure 31** show the mean rates of over-wash from **Table 4** graphically. Threshold over-wash rates are indicated on each figure.

Table 4. C-18W Reservoir Mean Over-wash Rates

Embankment Height	C-18 Average Overwash Rate (cfs/lf) by Still Water Depth (ft)										
(ft)	7.0ft	7.5ft	8.0ft	8.5ft	9.0ft	9.5ft	10.0ft	10.5ft	11.0ft	11.5ft	12.0ft
9.0	7.329	---	---	---	---	---	---	---	---	---	---
9.5	5.240	7.331	9.984	---	---	---	---	---	---	---	---
10.0	3.668	5.266	7.376	10.014	---	---	---	---	---	---	---
10.5	2.514	3.708	5.325	7.409	10.052	---	---	---	---	---	---
11.0	1.685	2.559	3.773	5.373	7.458	10.134	---	---	---	---	---
11.5	1.104	1.729	2.622	3.827	5.433	7.541	10.177	---	---	---	---
12.0	0.705	1.142	1.785	2.676	3.890	5.513	7.592	10.249	---	---	---
12.5	0.438	0.736	1.190	1.835	2.735	3.964	5.569	7.671	10.276	---	---
13.0	0.264	0.463	0.775	1.232	1.888	2.801	4.019	5.650	7.709	10.357	13.449
13.5	0.154	0.283	0.492	0.809	1.277	1.943	2.851	4.096	5.693	7.789	10.347
14.0	0.092	0.168	0.305	0.519	0.846	1.322	1.988	2.921	4.140	5.770	7.796
14.5	0.052	0.102	0.183	0.325	0.547	0.881	1.360	2.048	2.963	4.210	5.788
15.0	0.028	0.058	0.113	0.198	0.346	0.575	0.912	1.410	2.085	3.024	4.234
15.5	0.014	0.032	0.065	0.116	0.213	0.366	0.598	0.952	1.442	2.137	3.050
16.0	0.007	0.017	0.036	0.072	0.127	0.227	0.384	0.630	0.978	1.484	2.162
16.5	0.003	0.008	0.020	0.041	0.079	0.137	0.240	0.407	0.650	1.012	1.507
17.0	0.001	0.004	0.010	0.022	0.046	0.086	0.146	0.257	0.423	0.676	1.032
17.5		0.002	0.005	0.012	0.026	0.050	0.093	0.158	0.269	0.443	0.693
18.0			0.002	0.006	0.014	0.028	0.055	0.102	0.167	0.283	0.456
18.5				0.003	0.007	0.016	0.031	0.060	0.108	0.177	0.293
19.0				0.001	0.003	0.008	0.017	0.035	0.065	0.115	0.184
19.5					0.002	0.004	0.009	0.020	0.038	0.070	0.120
20.0						0.002	0.005	0.011	0.021	0.041	0.073
20.5							0.002	0.005	0.012	0.024	0.044
21.0								0.003	0.006	0.013	0.025
21.5								0.001	0.003	0.007	0.014
22.0									0.001	0.003	0.008
22.5										0.002	0.004
23.0											0.002

"---" Indicates Submersion of Structure



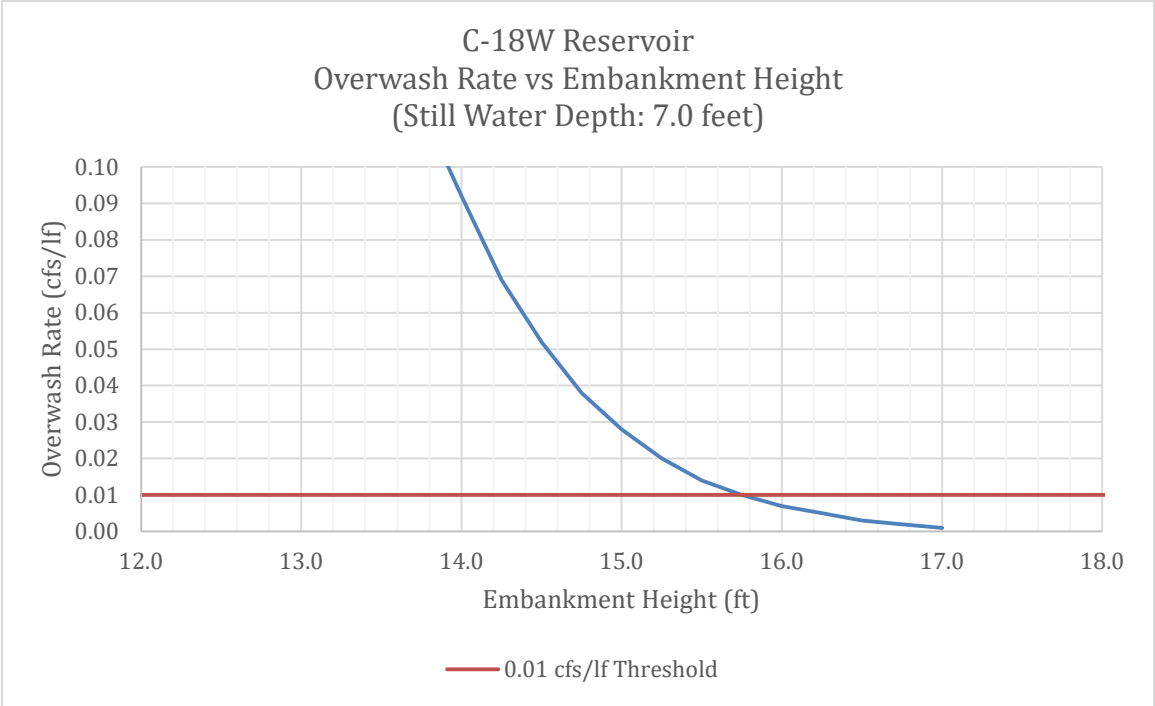
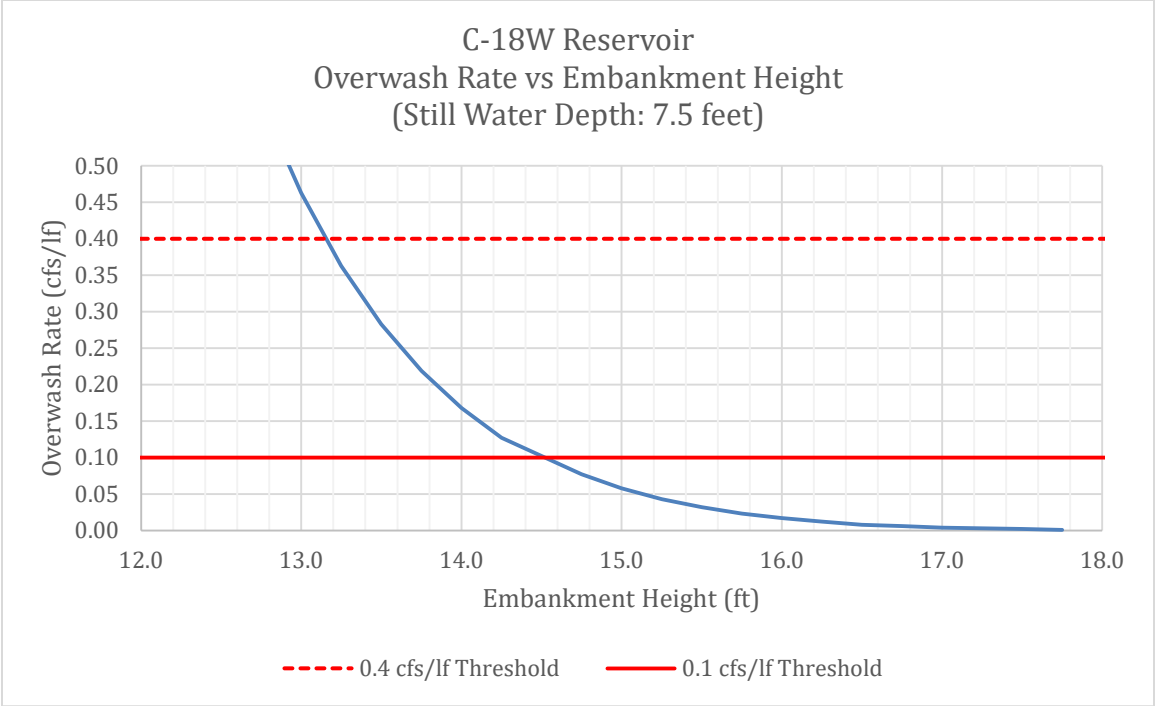


Figure 21. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 7.0ft)



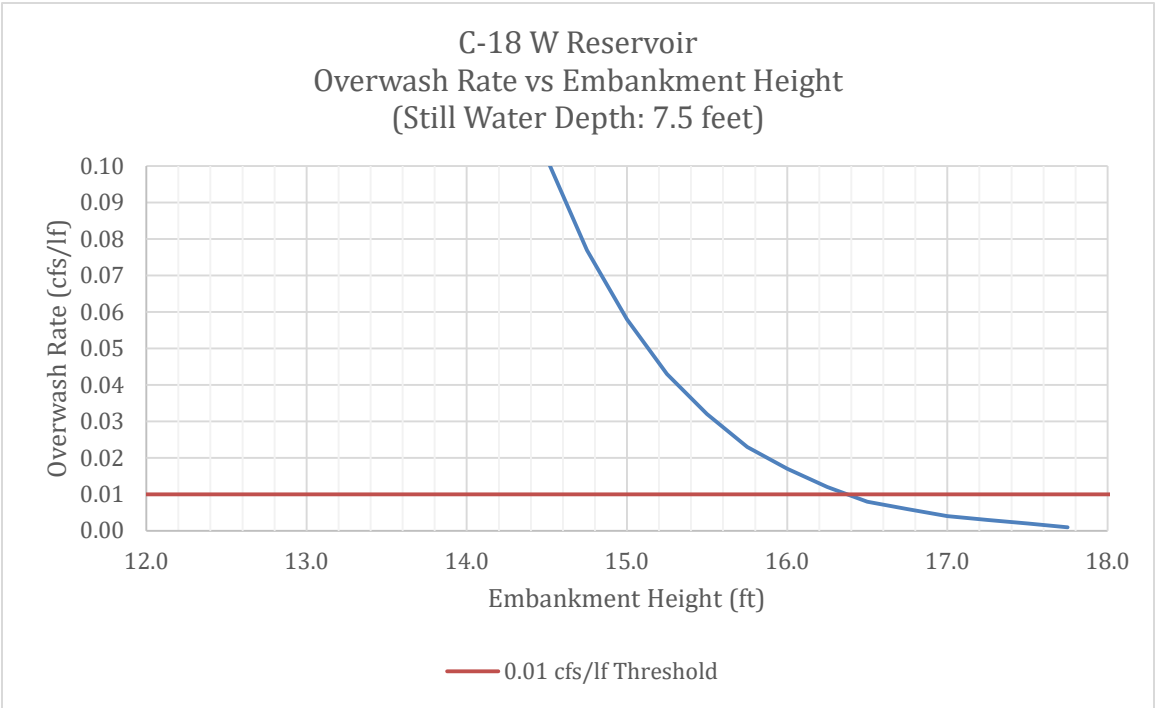
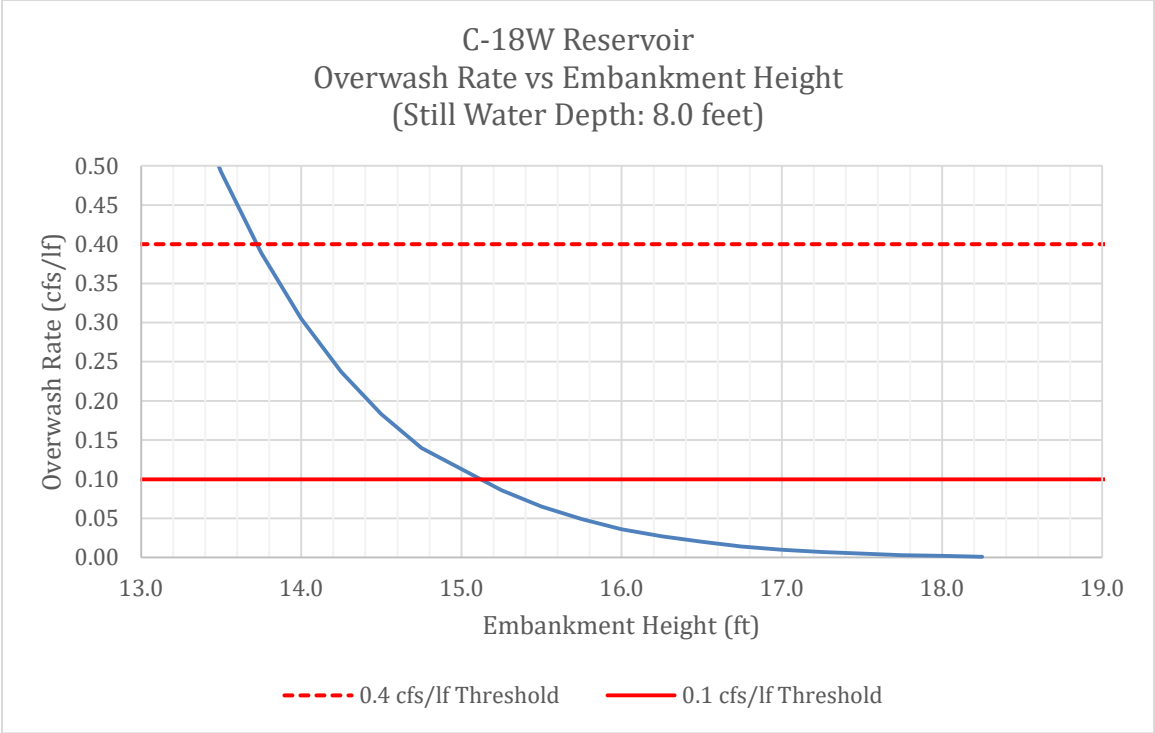


Figure 22. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 7.5ft)



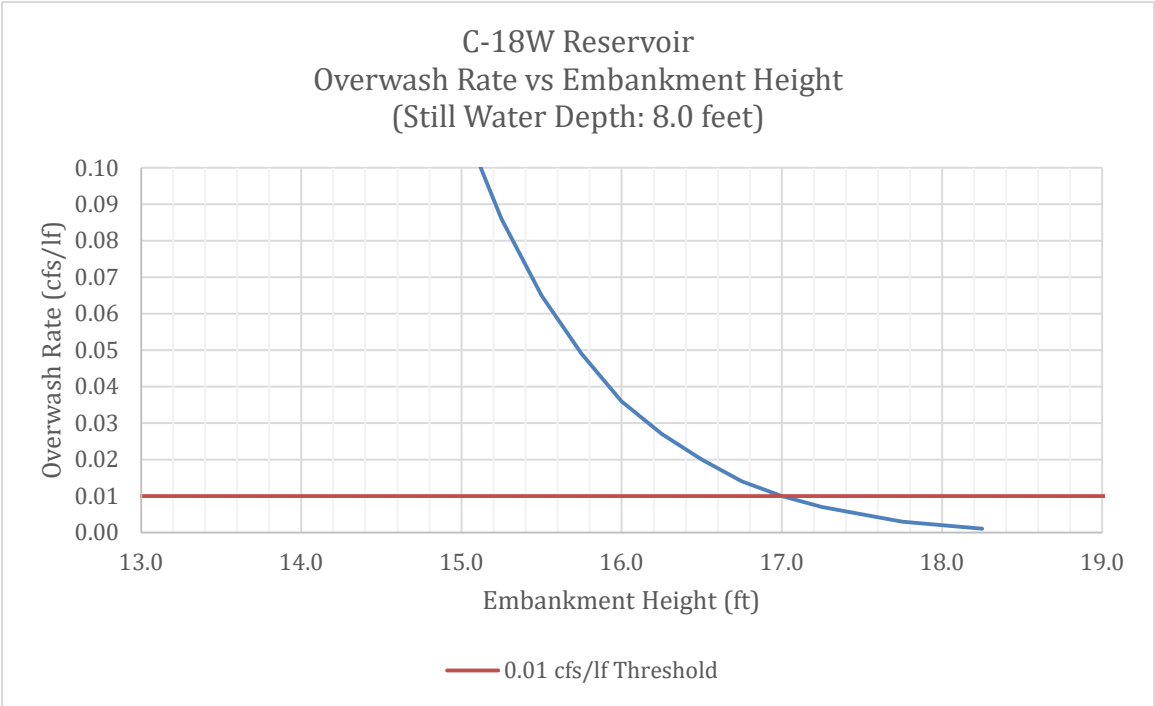
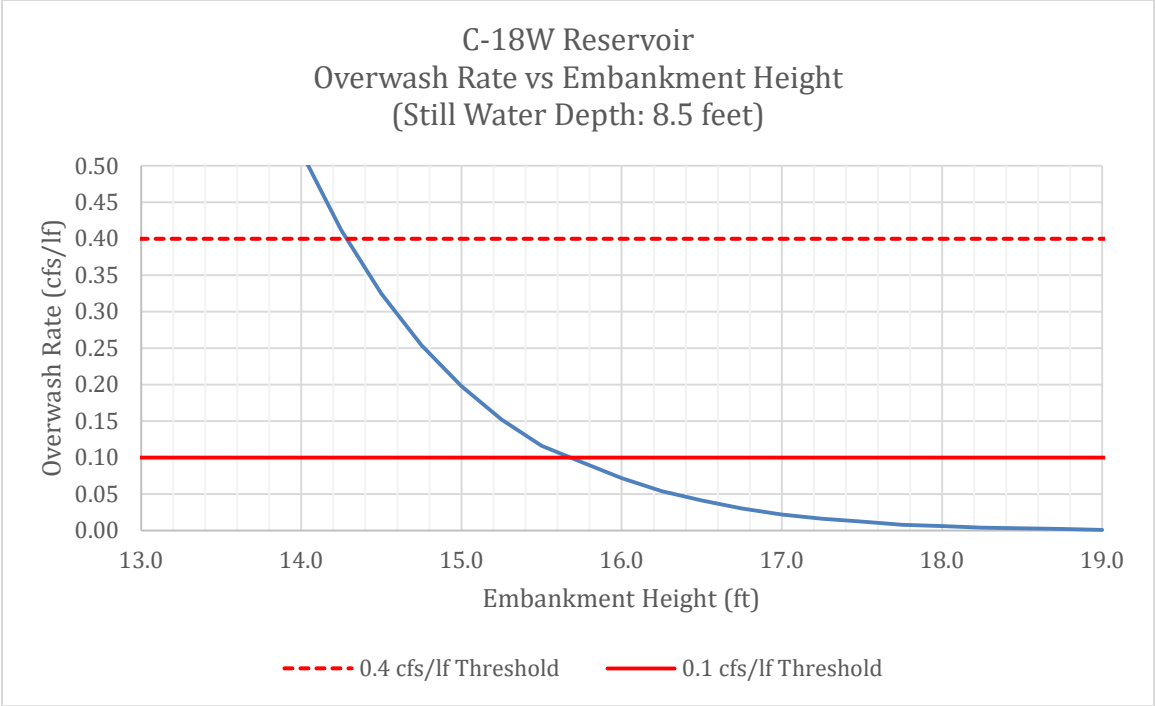


Figure 23. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 8.0ft)



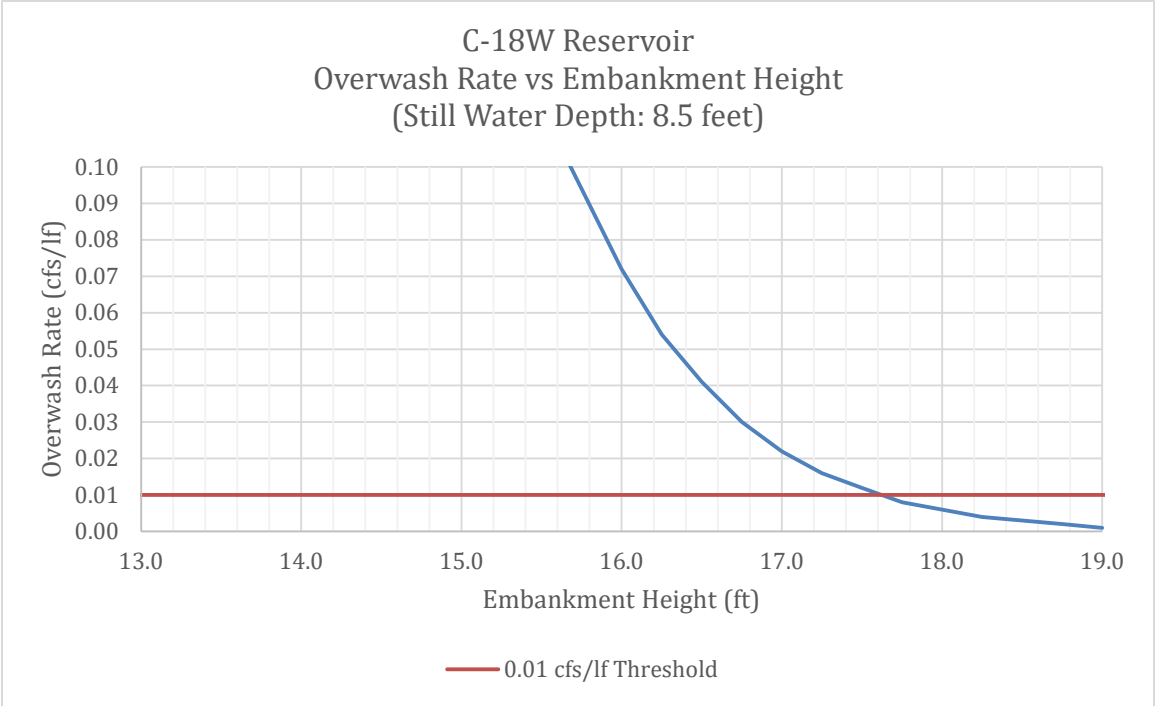
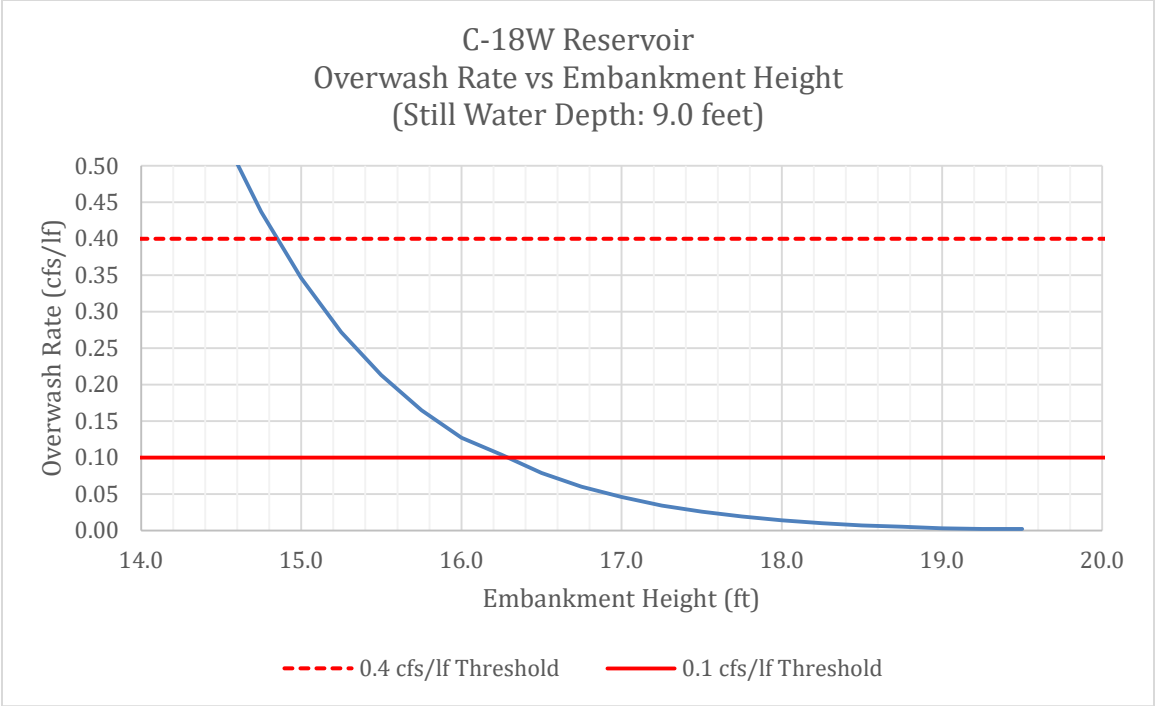


Figure 24. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 8.5ft)



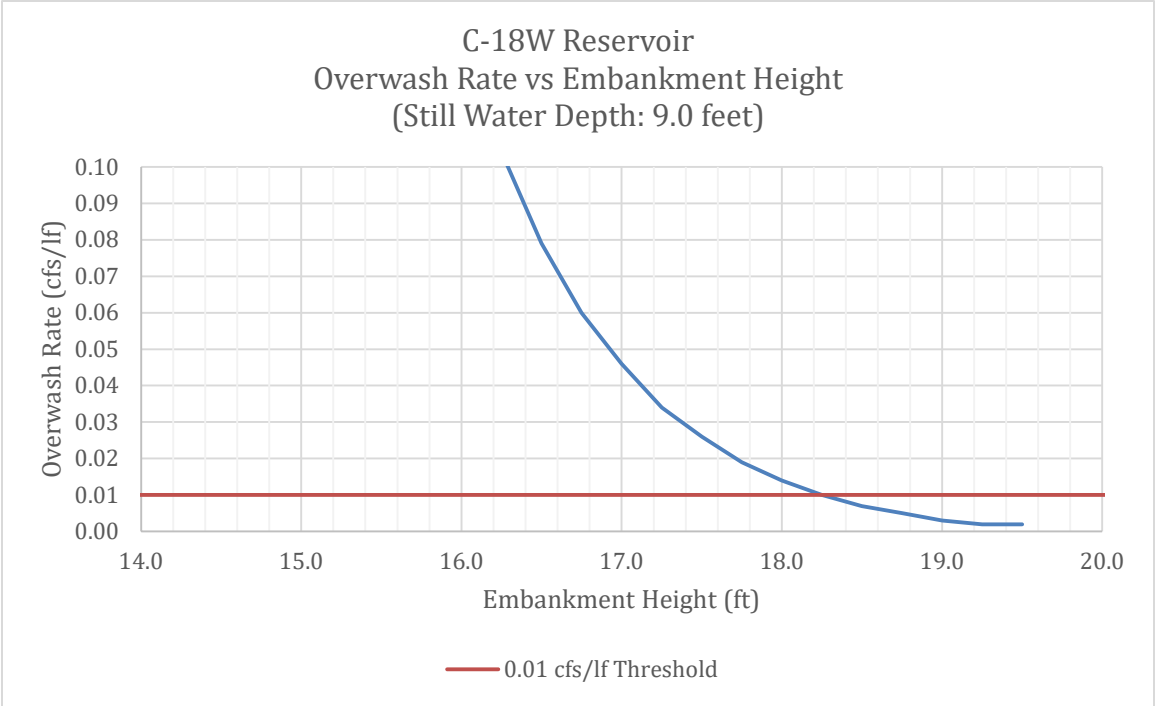
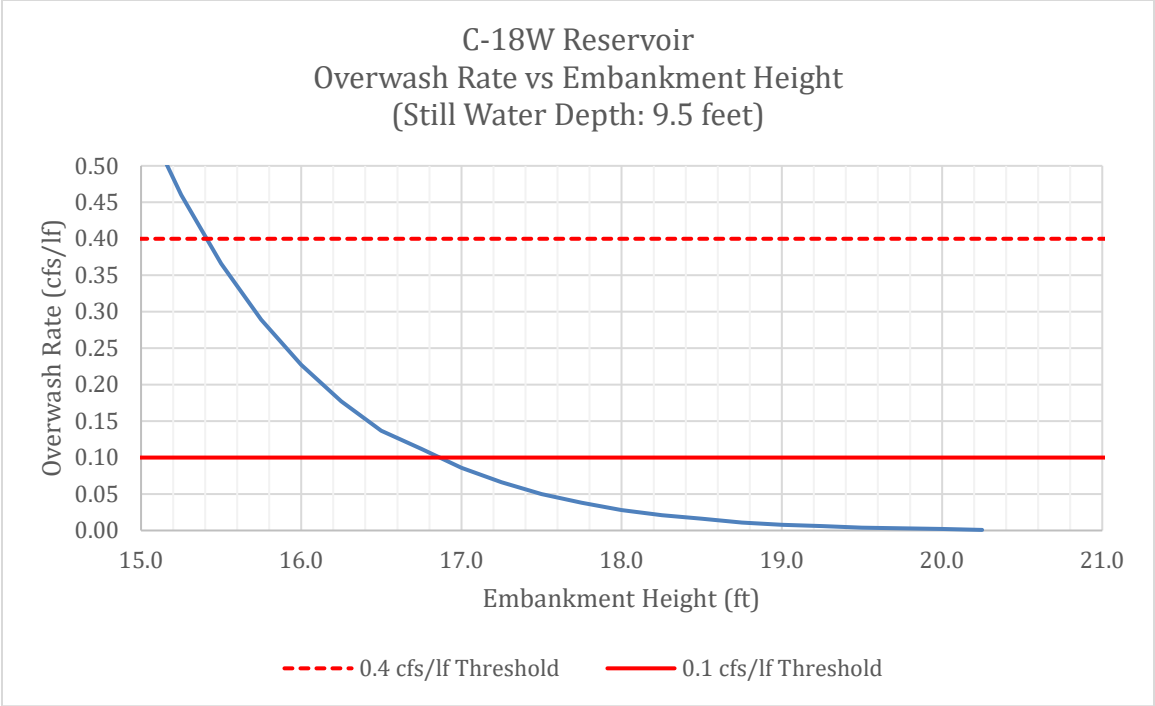


Figure 25. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 9.0ft)



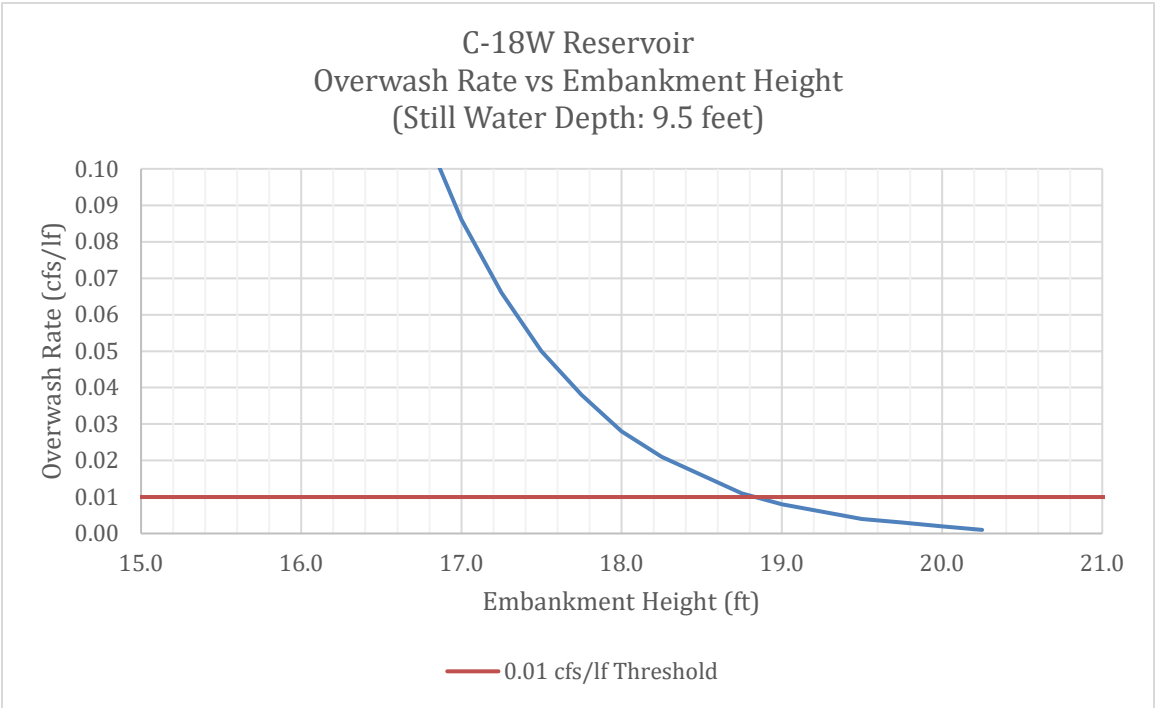
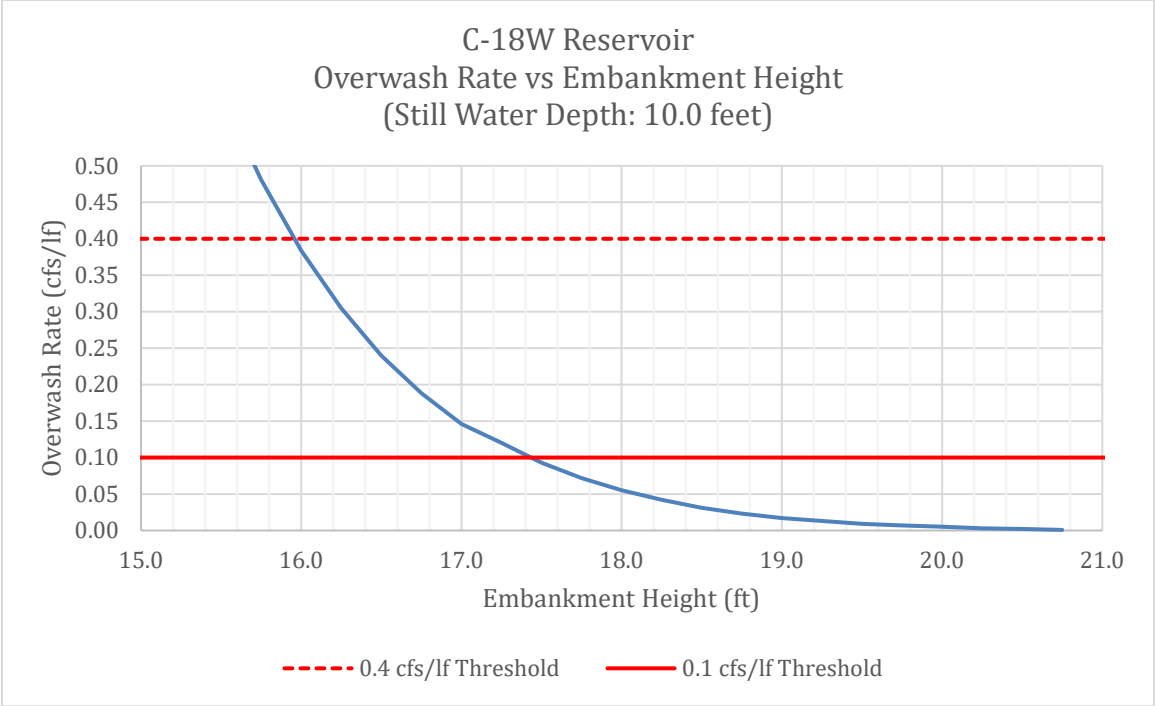


Figure 26. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 9.5ft)



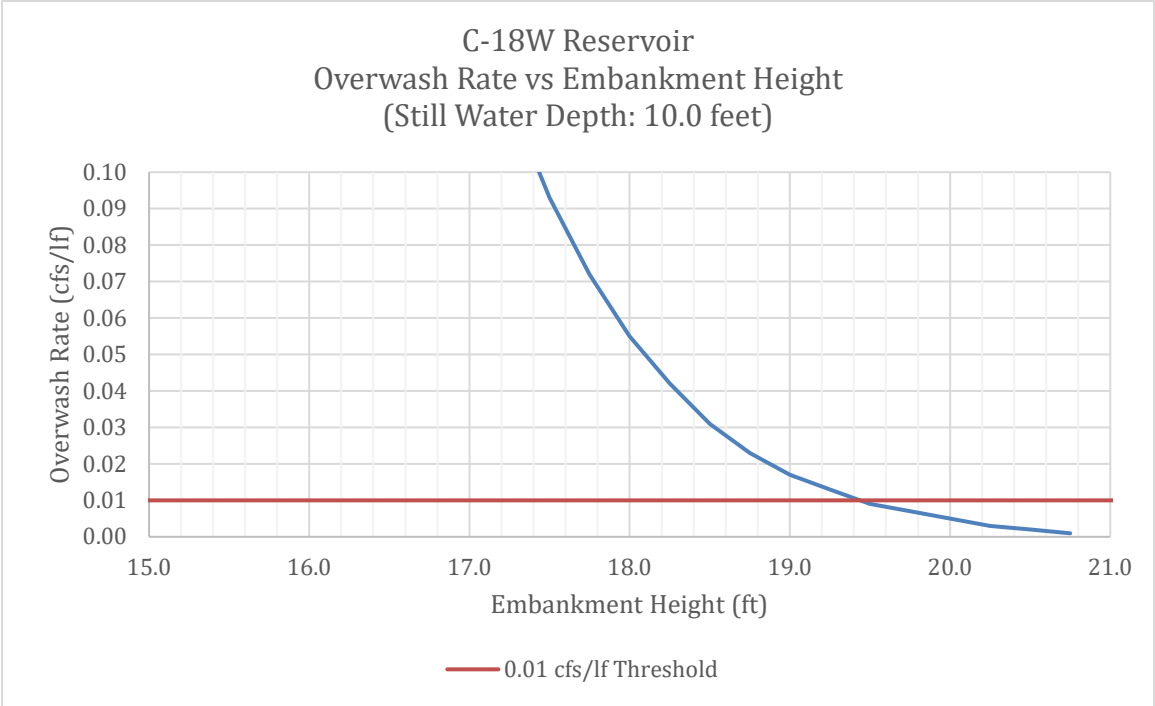
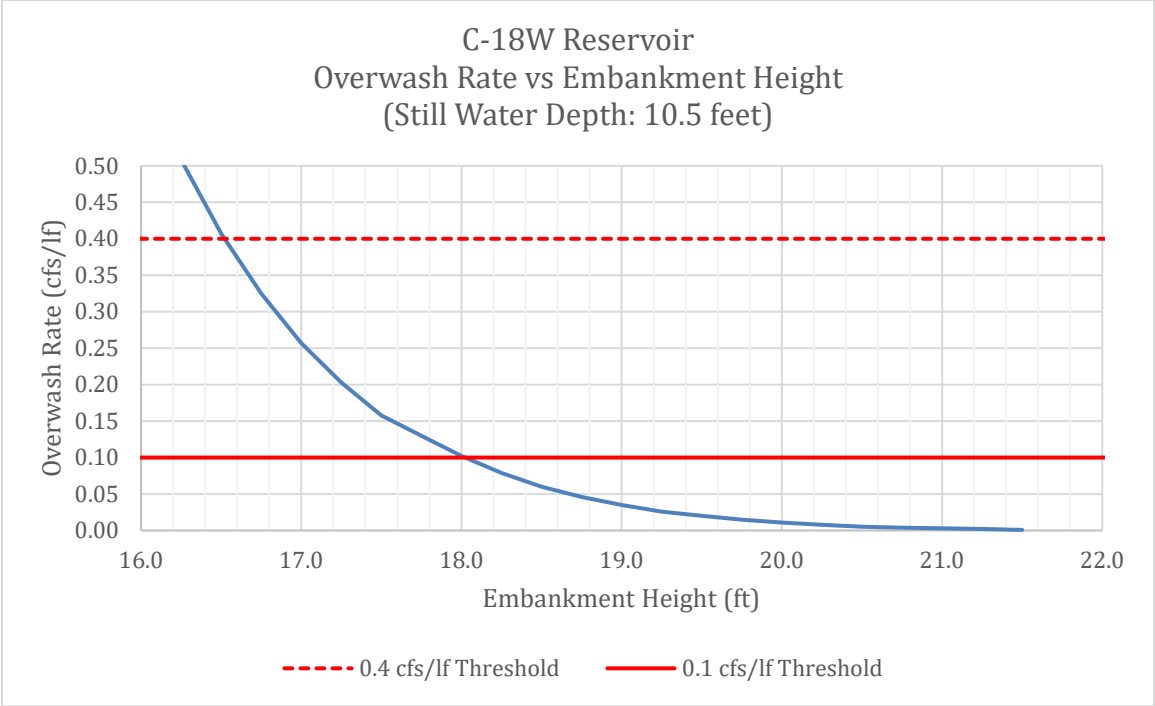


Figure 27. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 10.0ft)



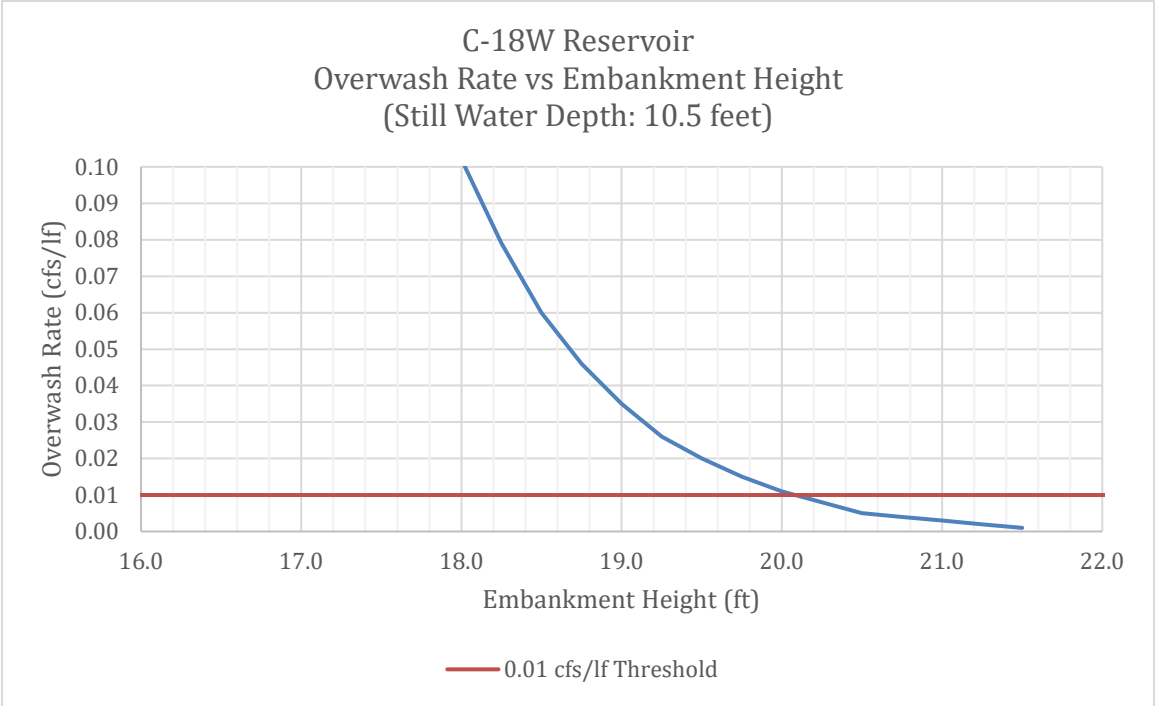
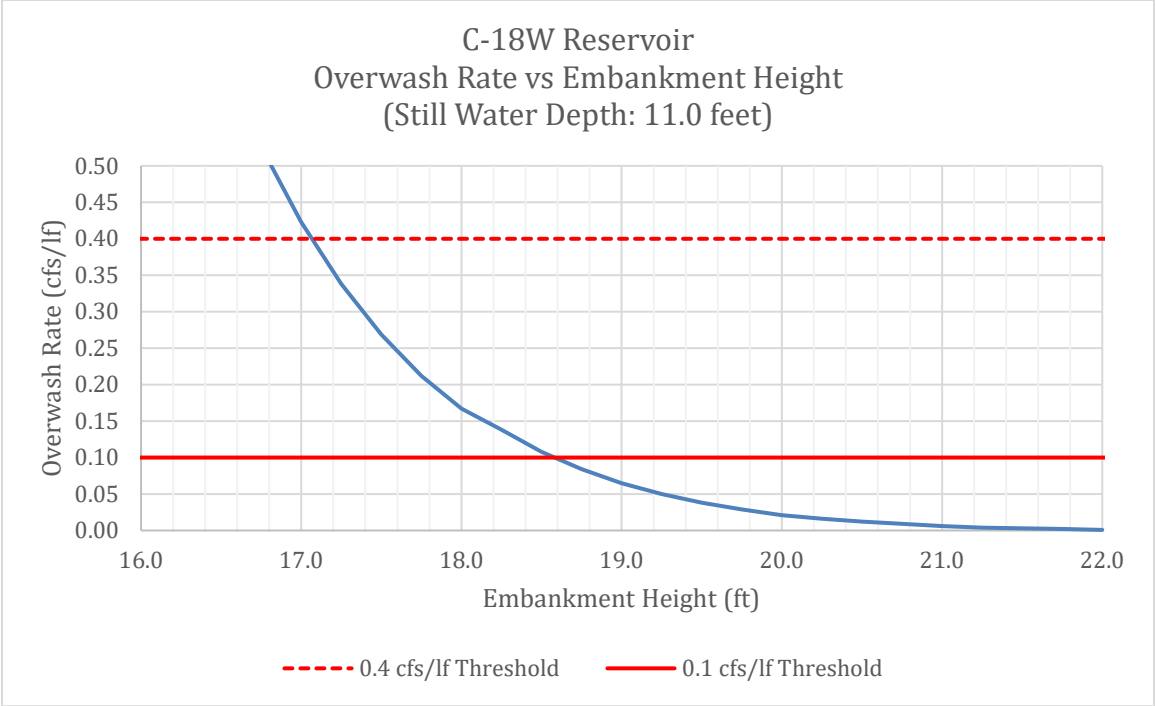


Figure 28. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 10.5ft)



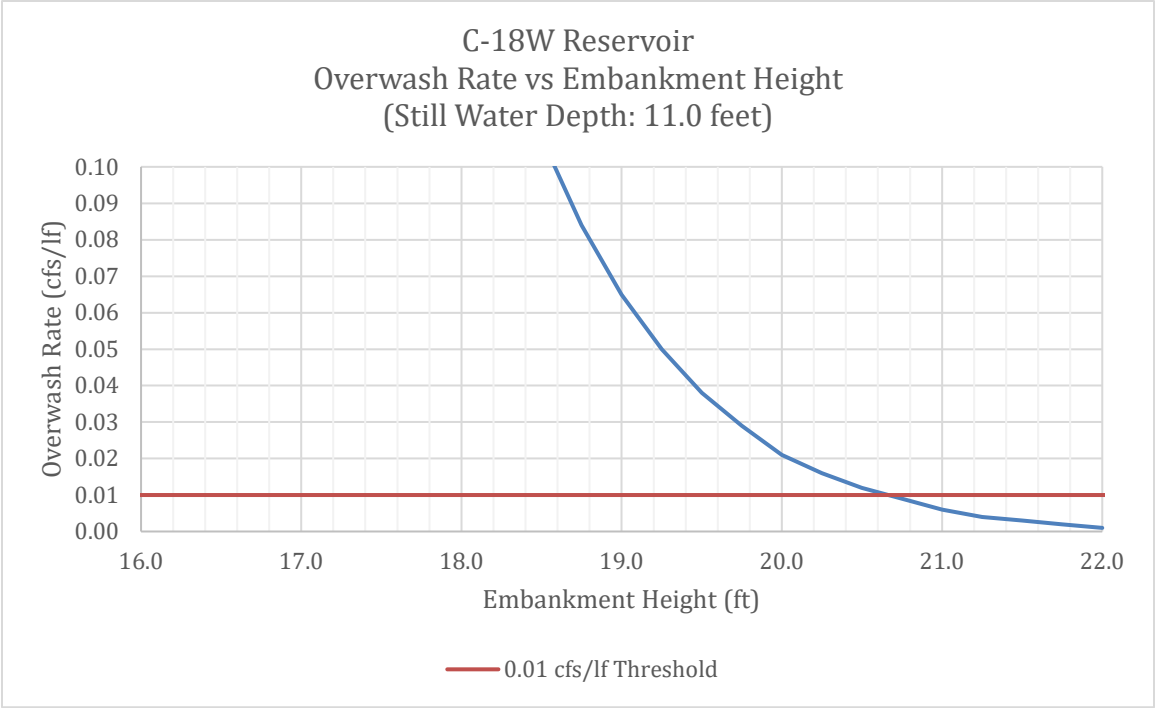
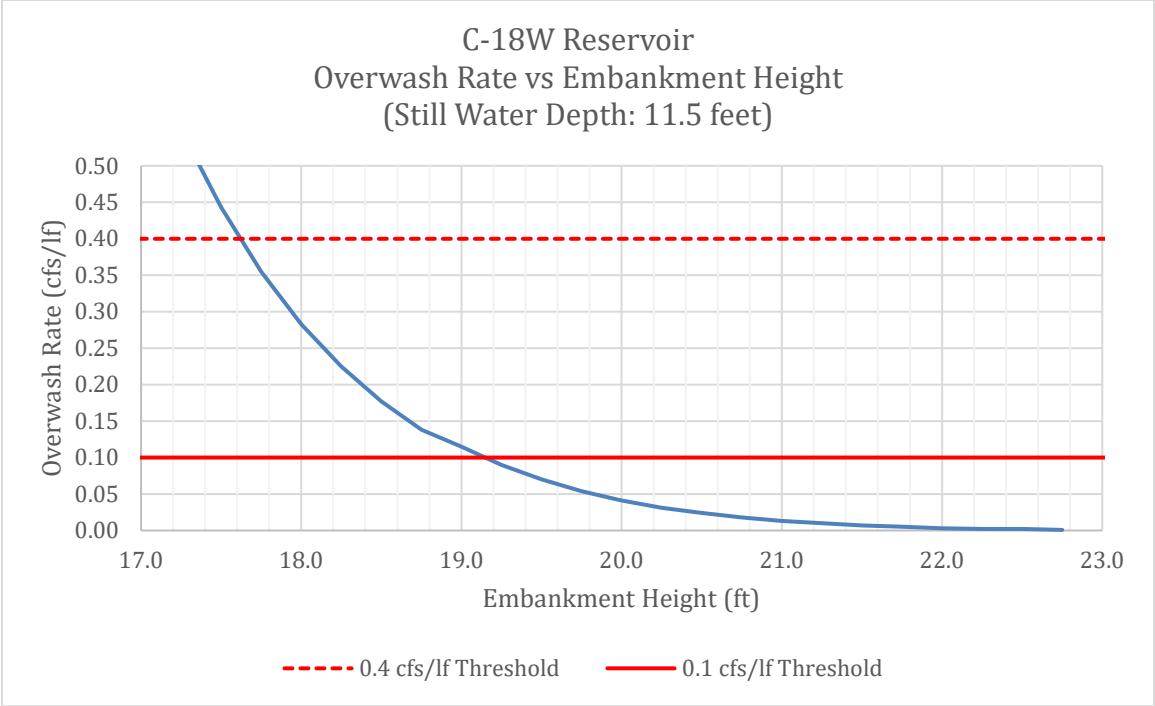


Figure 29. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 11.0ft)



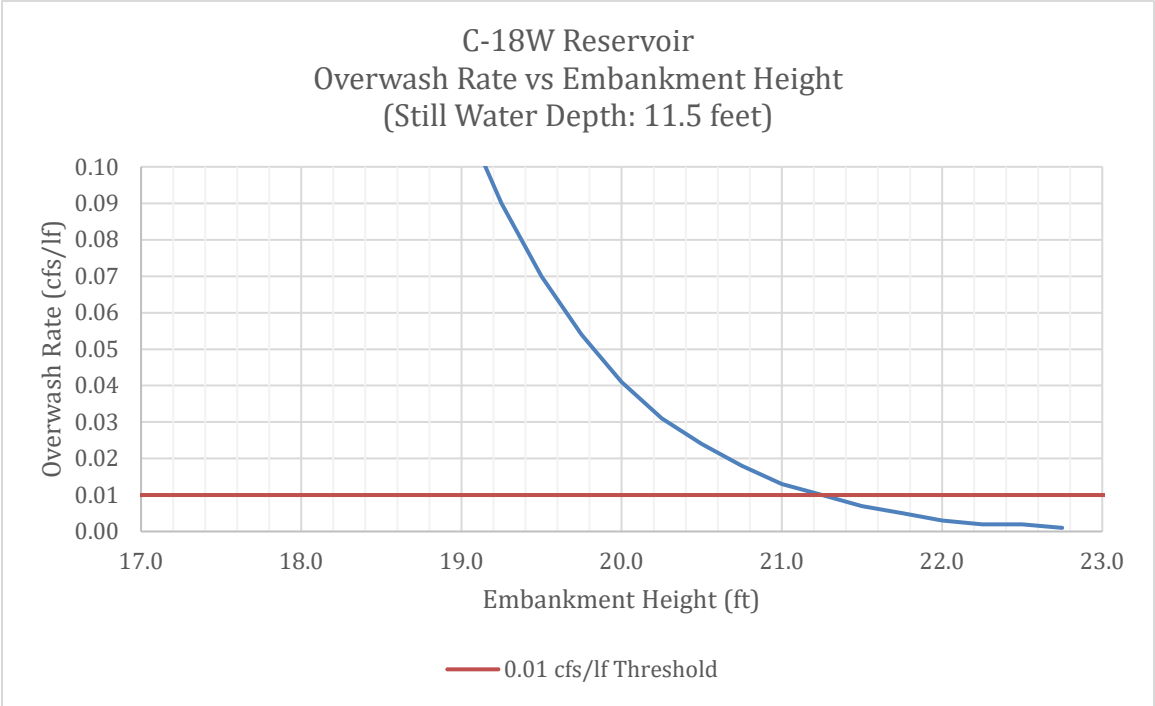
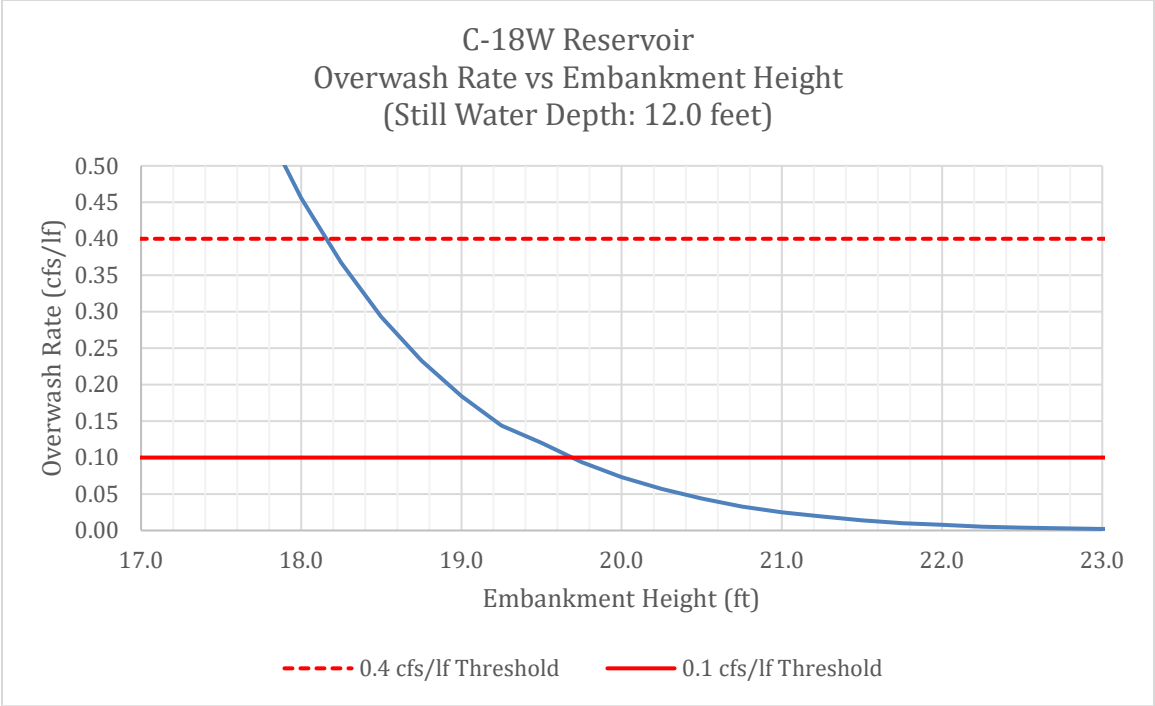


Figure 30. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 11.5ft)



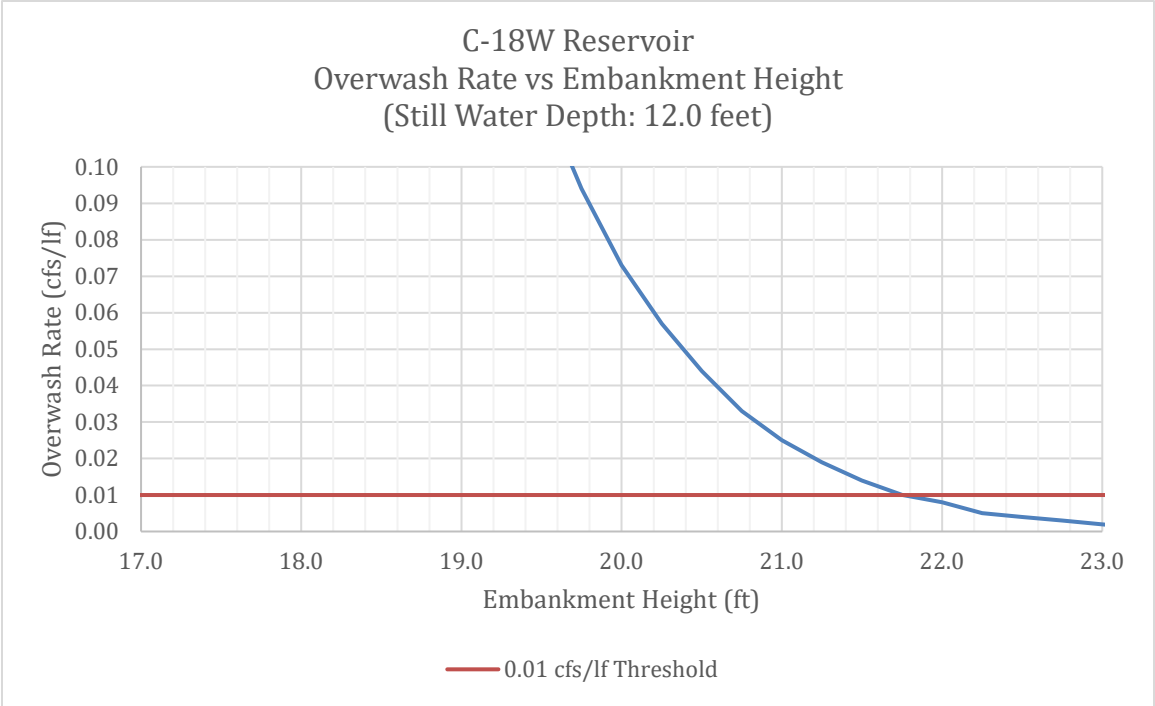


Figure 31. Over-wash Rate vs Embankment Height (C-18, Still Water Depth: 12.0ft)

C.7.10 Vegetation

Shallow water impoundments and reservoirs are ideal environments for aquatic vegetation. Both submerged and emergent aquatic vegetation are prevalent throughout South Florida. Vegetation (either submerged or emergent) not only provides wildlife habitat it can also act as a wave dampening feature which reduces wave energy (wave heights) which can then result in lower embankment crest elevation requirements.

For vegetation to be considered in embankment design, however, there must be reasonable assurance that the impoundment will contain enough water throughout the design life to sustain permanent vegetation. Should the impoundment experience dry out conditions such that the vegetation dies out, wave energy when the impoundment refills will no longer be dampened and the embankment may experience over-wash in excess of allowable rates. It is necessary to note also that the water depth (as well as dry out conditions) can prevent sustainable vegetation. Based on general observation in South Florida the limiting maximum depth for vegetation is about 6.0 feet. It is assumed that deeper normal pool depths will not support the density of vegetation necessary to dampen wave energy.

Additionally, water depth can inhibit the dampening effects of even sustained vegetation. In the event of significant, but relatively short term increases in water depth (such as a PMP event), vegetation, including previously emergent vegetation, may be too deeply submerged to interact with and dampen waves on the surface. Given these constraints, the C-18W Reservoir cannot be evaluated for possible damping effects due to vegetation.

C.7.11 Recommended Embankment Heights

Based on the allowable over-wash rates, corresponding embankment heights can be determined from either **Table 4 or Figure 21**. **Table 5.** C-18W Recommended Embankment Height summarizes the recommended embankment heights.

Table 5. C-18W Reservoir Recommended Embankment Heights

Still Water Depth (feet)	Embankment Height 0.4cfs/lf (feet)	Embankment Height 0.1cfs/lf (feet)	Embankment Height 0.01cfs/lf (feet)
7.0	13.0	14.0	16.0
7.5	13.5	14.5	16.5
8.0	14.0	15.5	17.0
8.5	14.5	16.0	18.0
9.0	15.0	16.5	18.5
9.5	15.5	17.0	19.0
10.0	16.0	17.5	19.5
10.5	17.0	18.0	20.0
11.0	17.5	19.0	21.0
11.5	18.0	19.5	21.5
12.0	18.5	20.0	22.0

D FLOW-WAY 3 FEATURES

D.1 Purpose

This intermediate hydraulic design is used to provide parameters for the calculation of quantities for the purpose of costing project features and to evaluate the savings clause performance of the recommended plan. Final design will be performed in PED phase.

D.2 Objective

Using HEC-RAS models, the structure size necessary to convey inflows advised through hydrologic modeling was determined.

D.3 Assumptions/Constraints

The LRWRP is for ecosystem restoration, therefore it is assumed that under extreme events when the Gulfstream West Flow-through marsh is at normal water surface elevation, the inflow pumping station will be turned off and the only additional inflow to the project will be direct rainfall. The structures have been designed and optimized in so far as possible based upon design inflow events as well as the expected operations to meet project objectives as informed through the SFWMD LECSR-NP Model. Additional refinement, as required, will occur during PED.

There are multiple existing structures proposed as part of the Recommended Plan that will become project structures, including S-115A, S-115B, S-115C, S-114D, and S-114E. The current modifications proposed to these existing structures include adding telemetry operated gates to S-115A, S-115B, S-115C and S-114D. The current assumption for the with-project condition is to keep S-114E as it was constructed. During the PED phase, determination regarding potential structure modification or replacement will be analyzed to ensure all structures are compliant with USACE standards. These water control structures do not have HDDS at this time as no HDDS information was available for the previously completed design (and construction). Additional effort including gathering necessary survey data, reviewing previous design documentation and as-built data will help determine the appropriate HDDS values.

D.4 Structure Descriptions

D.4.1 S-110

An inflow pump station for the Gulfstream West Flow-through marsh, which draws water from the HSLCD canal. The structure is intended to maintain an operational water surface elevation (from 14- 16, ft. NAVD88) within the marsh footprint.

D.4.2 S-111N

A plunge pool and weir structure which discharges pumped water into the Gulfstream West Flow-through marsh. This structure will assist in maintaining desired discharge rates and water surface elevations within the marsh.

D.4.3 S-111S

A notched weir structure that provides regulation of water surfaces within the Gulfstream West Flow-through Marsh. The weir will provide uncontrolled discharge capacity and was designed with varying notch elevations and widths for both ecological and flood control discharge capabilities.

D.4.4 S-112

Primary control structure along the Cypress Creek Canal. S-112 is a gated ogee spillway that will provide improved capability for management of water elevations within the canal during the wet and dry season while also maintain the current flood protection for the surrounding developments.

D.4.5 S-113N

An inflow pump station for the Mack Dairy Spreader Swale (C-114) which is designed to send water from the Cypress Creek Canal into Cypress Creek Natural Area.

D.4.6 S-113S

A plunge pool and weir structure which discharges pumped water into the C-114. This structure will assist in maintaining desired discharge rates and water surface elevations within the spreader swale and Cypress Creek Natural area.

D.4.7 S-114A, S-114B, and S-114C

Gated culvert structures that provides discharge capability from the Nine Gems natural area into the Nine Gems southern canal. The proposed structures will replace existing structures that are currently in a state of corrosion and are undersized to pass the necessary design flow. The new structures will provide improved operational control of water elevations for ecological improvement, while still allowing for conveyance offsite during larger storm events.

D.4.8 S-114D

Existing culvert structure located at the eastern terminus of the southern Nine Gems canal, allowing discharge into the HSLCD canal. The culvert provides regulation of the southern Nine Gems canal, while providing controlled discharge into HSLCD and ultimately the Cypress Creek Canal. Proposed modifications include adding telemetry operated gates as a means to control the headwater elevation within the southern Nine Gems canal.

D.4.9 S-114E

Existing uncontrolled culvert structure located on the HSCLD that helps control discharge for the upper portions of the HSLCD canal as well as provides drivable access across HSLCD. This structure is not proposed to be modified at this time but additional analysis will be required during PED.

D.4.10 S-115A, S-115B, S-115C

Existing outlet structures consisting of culverts with drop-structure inlet control. Modifications to the existing water control structures include telemetry operated gates to help maintain desired water surface

elevations on the Pal-Mar East property, while simultaneously providing additional control of discharges into the Cypress Creek Canal.

D.4.11 S-116

A gated culvert structure within Jenkins Ditch that will aid in dispersing water into the spreader system (C-116) for rehydration of Kitching Creek.

D.4.12 S-117

A fixed weir within Hobe Grove Ditch that will assist in promoting additional flow down the historic Moonshine Creek while improving surrounding groundwater elevations.

D.5 Hydraulic Design Data Sheets (HDDS)

Loxahatachee River Watershed Restoration Project
Pumping Station 110 (S-110)
Martin County, Florida

X Y Coordinate: 26.99493, 80.19862
Date Compiled (or revised) **27-May-19**

Design Condition		
Design Condition	150	cfs
Design Capacity	150	cfs
Number of Pumps	4	
Pump Mix type and Size		
Electric 2 @ 25 cfs		
Electric 2 @ 50 cfs		
Mix Criteria		
Pumping Station will have 4 pumps - The firm pumping capacity will be 150 cfs		
2 - 25 cfs and 2- 50 cfs, Submersible Electric Pumps for Resotration Flows		
Design Operating Heads (ft., NAVD88)		
Normal	5.00	ft, NAVD88
Maximum	7.50	ft, NAVD88
Intake Water Surface Elevations		
Maximum Non-Pumping	12.00	ft, NAVD88
Maximum Pumping	12.00	ft, NAVD88
Start Pumping	10.00	ft, NAVD88
Normal Drawdown	10.00	ft, NAVD88
Minimum Drawdown	9.50	ft, NAVD88
Minimum Non-Pump	9.50	ft, NAVD88
Invert Elevation of Canal	3.00	ft, NAVD88
Discharge Water Surface Elevations		
Maximum Non-Pumping	16.50	ft, NAVD88
Maximum Pumping	15.50	ft, NAVD88
Normal Pumping	14.5	ft, NAVD88
Minimum Pumping	13.00	ft, NAVD88
Minimum Non-Pump	13.00	ft, NAVD88
Invert Elevation of Canal	11.00	ft, NAVD88

Figure 32. S-110 HDDS

Loxahatachee River Watershed Restoration Project
S-111N
Non-Gated Broad Crested Weir

Revisions 30-May-19

XY Coord 26°59'41.6"N, 80°11'55.0"W

Location Northern tip of Gulfstream West Flow-through Marsh within Martin County, Florida
Purpose The plunge pool, with the spreader weir S-111N, is required as a discharge basin for flow dispersion into the flow-through marsh
Notes 1. Riprap requirements are estimated and have not been verified with Geotech.

Design Conditions

Weir	
Discharge	150.00 cfs
Design Head	1.10 feet
Headwater Elevation	15.10 ft,NAVD88
Tailwater Elevation	14.00 ft,NAVD88
Maximum Expected Stages	
Headwater Elevation	16.00 ft,NAVD88
Tailwater Elevation	15.50 ft,NAVD88
Maximum Simultaneous Head Difference	
Maximum Headwater Elevation	N/A ft,NAVD88
Minimum Tailwater Elevation	N/A ft,NAVD88
Max Head Discharge Condition	N/A feet
Weir Design Data	
Weir Type	Broad Crested
Crest Elevation	14.50 ft,NAVD88
Weir Breadth	2.00 feet
Crest Length	50.00 feet
Minimum Tieback Elevation	20.00 ft,NAVD88
Weir Control	None
Canal Section	
Flow-Through Marsh	
Side Slopes (Vert. on Hor.) 1 on	N/A
Bottom Width	Lateral
Bottom Elevation	13.00 ft,NAVD88
Riprap Requirements	
Design Flow Velocity	2.00 fps
Design Riprap Velocity	4.00 fps
Downstream Length	50.00 feet
Downstream Armored Area	3000.00 ft^2

Figure 33. S-111N HDDS

Loxahatachee River Watershed Restoration Project
S-111S
Ungated Weir

Revisions August 2019 - PIR

XY Coord 26°58'18.26"N , 80°10'54.48"W
Location Martin County, at the south-east corner of the proposed Gulfstream West Flow-through Marsh.
Purpose Maintain stages within the Gulfstream West Flow-through Marsh for Ecosystem Restoration

Notes 1. Riprap requirements are estimated and have not been verified with Geotech.

Design Conditions

Weir		
Notch Discharge	30.00	cfs (normal conditions)
Design Discharge	250.00	cfs
Design Head	1.95	feet
Headwater Elevation	17.20	ft,NAVD88
Tailwater Elevation	12.40	ft,NAVD88

Maximum Expected Stages

Headwater Elevation	18.25	ft,NAVD88
Tailwater Elevation	13.00	ft,NAVD88

Minimum Expected Stages

Headwater Elevation	N/A	ft,NAVD88
Tailwater Elevation	N/A	ft,NAVD88

Maximum Simultaneous Head Difference	N/A	ft

Weir Design Data

Weir Type	Sheetpile, concrete cap	
Crest Elevation	15.25	ft,NAVD88
Notch Crest Length	15.00	feet
Weir Breadth	1.60	feet
Overall Weir Length	200.00	feet
1st Notch Crest Elevation	16.00	ft,NAVD88
1st Notch Crest Length	30.00	feet
2nd Notch Crest Elevation	17.50	ft,NAVD88
2nd Notch Crest Length	20.00	feet
3rd Notch Crest Elevation	20.00	ft,NAVD88
3rd Notch Crest Length	135.00	feet
Weir Control	None	

Canal Section

Cypress Creek Canal		
Side Slopes (Vert. on Hor.) 1 on	2.5	
Bottom Width	Lateral	
Bottom Elevation	-1.00	ft,NAVD88

Canal Riprap Requirements

Design Flow Velocity	4.90	fps
Design Riprap Velocity	10.00	fps
Downstream Length	150.00	feet
Downstream Width	200.00	feet

Figure 34. S-111S HDDS

Loxahatchee River Watershed Restoration Project
Cypress Creek Canal Structure
S-112

Date Compiled (or revised)		**28-Mar-19**
Latitude, Longitude	27° 18' 49.81"N, 81° 01' 25.03"W	
Design Conditions (optimum efficiency)		
Discharge (cfs)	2,150	cfs
Headwater Elevation	13.70	ft, NAVD88
Tailwater Elevation	10.80	ft, NAVD88
SPF Conditions		
Discharge (cfs)	2,450	cfs
Headwater Elevation	14.40	ft, NAVD88
Tailwater Elevation	11.00	ft, NGVD29
Maximum Head Difference (coincidental)		
Headwater Elevation	16.00	ft, NAVD88
Tailwater Elevation	8.00	ft, NAVD88
Crest Data		
Shape	Ogee	
Design Head (Hd)	7.40	ft
Net Crest Length	32.00	ft
Crest Elevation	6.50	ft, NAVD88
Approach Apron Elevation	0.50	ft, NAVD88
Weir Control	Vertical Slide	
Gates		
Number of Gates	2.00	
Gate Width	16.00	ft
Gate Height	10.50	ft
Clearance Elevation	17.00	ft, NAVD88
Breastwall Elevation (bottom)	17.00	ft, NAVD88
Intermediate Pier Width	3.25	ft
Stilling Basin		
Design Discharge (cfs)	2,150	cfs
Apron Elevation	-1.00	ft, NAVD88
Apron Width	35.25	ft
End Sill Elevation	0.00	ft, NAVD88
Baffle Block Elevation	0.83	ft, NAVD88
Dist. From crest toe to 1st row of blocks/2nd row	16.2/21.0	ft
Velocity over End Sill (at design TW)	5.65	ft/s
Training Wall Elevation	17.00	ft, NAVD88
Canal Data (Downstream)		
Invert - Thalweg	0.00	ft, NAVD88
Top of Bank	19.00	ft, NAVD88
Bottom Width	36.00	ft
Top Width	91.00	ft
Side Slope (L/R)	2.50	ft (h:1)
Revetment		
Riprap Extent (Downstream)	750	ft
Riprap Specific Weight	165.00	lb/ft3
Max Velocity Riprap can withstand (turbulent rough)	10.83	ft/s
Downstream Armored Area	17000.00	ft^2

Figure 35. S-112 HDDS

Loxahatachee River Watershed Restoration Project

Pumping Station 113 (S-113N)

Martin County, Florida

X Y Coordinate: 26.99493, 80.19862
Date Compiled (or revised) **27-May-19**

Design Condition		
Design Condition	20	cfs
Design Capacity	20	cfs
Number of Pumps	2	
Pump Mix type and Size		
Electric 2 @ 10 cfs		
Mix Criteria		
Pumping Station will have 2 pumps - The firm pumping capacity will be 20 cfs		
2 - 10 cfs, Submersible Electric Pumps for Resotration Flows		
Design Operating Heads (ft., NAVD88)		
Normal	7.50	ft, NAVD88
Maximum	11.00	ft, NAVD88
Intake Water Surface Elevations		
Maximum Non-Pumping	14.00	ft, NAVD88
Maximum Pumping	14.00	ft, NAVD88
Start Pumping	10.00	ft, NAVD88
Normal Drawdown	9.50	ft, NAVD88
Minimum Drawdown	9.00	ft, NAVD88
Minimum Non-Pump	9.00	ft, NAVD88
Invert Elevation of Canal	5.10	ft, NAVD88
Discharge Water Surface Elevations		
Maximum Non-Pumping	20.00	ft, NAVD88
Maximum Pumping	19.00	ft, NAVD88
Normal Pumping	16.00	ft, NAVD88
Minimum Pumping	15.50	ft, NAVD88
Minimum Non-Pump	15.50	ft, NAVD88
Invert Elevation of Canal	14.00	ft, NAVD88

Figure 36. S-113N HDDS

Loxahatachee River Watershed Restoration Project
S-113S
Ungated Weir

Revisions

XY Coord 26°58'17.01"N, 80°13'18.14"W

Location Approximately 400 feet of East of Mack Dairy Road in Martin County, Florida

Purpose The plunge pool, with the spreader weir S-113S, is required as a discharge basin for flow dispersion into the spreader swale.

Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions		
Weir		
Discharge	20.00	cfs
Design Head	0.50	feet
Headwater Elevation	17.00	ft,NAVD88
Tailwater Elevation	16.50	ft,NAVD88
Maximum Expected Stages		
Headwater Elevation	N/A	ft,NAVD88
Tailwater Elevation	N/A	ft,NAVD88
Minimum Expected Stages		
Headwater Elevation	N/A	ft,NAVD88
Tailwater Elevation	N/A	ft,NAVD88
Maximum Simultaneous Head Difference	N/A	ft,NAVD88
Weir Design Data		
Weir Type	Broad Crested	
Crest Elevation	16.50	ft,NAVD88
Weir Breadth	2.00	feet
Crest Length	20.00	feet
Minimum Tieback Elevation	19.00	ft,NAVD88
Weir Control	None	
Canal Section		
C-114 Canal		
Side Slopes (Vert. on Hor.) 1 on	4.00	
Bottom Width	5.00	ft
Bottom Elevation	15.00	ft,NAVD88
Riprap Requirements		
Design Flow Velocity	2.00	fps
Design Riprap Velocity	4.00	fps
Downstream Length	50.00	feet
Downstream Width	50.00	feet

Figure 37. S-113S HDDS

Loxahatachee River Watershed Restoration Project
S-114A, S-114B
Gated Culvert
Nine-Gems

Date Compiled (or Revised)	**30-Apr-19**
Latitude, Longitude	27°23'18"N, 81°11'02"W
Design Conditions	
Discharge (cfs)	105
Headwater Elevation	19.2 ft, NAVD88
Tailwater Elevation	17.2 ft, NAVD88
Maximum Expected Stages	
Headwater Elevation	N/A ft, NAVD88
Tailwater Elevation	N/A ft, NAVD88
Minimum Expected Stages	
Headwater Elevation	N/A ft, NAVD88
Tailwater Elevation	N/A ft, NAVD88
Maximum Simultaneous Head Difference	N/A ft
Culvert Data	
Type	RCP
Dimensions	6.0 ft
Number of Barrels	1.0
Length	60.0 ft
Invert elevation	13.5 ft, NAVD88
Inlet Condition (per plan drawings)	Headwall
Outlet Condition	Mitered End Sections
Type of Control	Vertical Lift Slide Gate(s)
Canal Data (Downstream)	
Invert - Thalweg	9.0 ft, NAVD88
Bottom Width	10.0 ft
Top of Bank	21.3 ft, NAVD88
Top Width	55.0 ft
Side Slope (h:v)	2.0 ft (h:1)
Canal Riprap Requirements	
Design Flow Velocity	3.7 fps
Design Riprap Velocity	6.0 fps
Downstream Length	50.0 feet

Figure 38. S-114A and S-114B HDDS

Loxahatachee River Watershed Restoration Project
S-114C
Gated Culvert
Nine-Gems

Date Compiled (or Revised)	**30-Apr-19**
Latitude, Longitude	26°59'10.5"N, 80°12'24"W
Design Conditions	
Discharge (cfs)	240
Headwater Elevation	19.2 ft, NAVD88
Tailwater Elevation	16.6 ft, NAVD88
Maximum Expected Stages	
Headwater Elevation	20.1 ft, NAVD88
Tailwater Elevation	17.5 ft, NAVD88
Minimum Expected Stages	
Headwater Elevation	N/A ft, NAVD88
Tailwater Elevation	N/A ft, NAVD88
Culvert Data	
Type	RCP
Dimensions	6.0 ft
Number of Barrels	2.0
Length	60.0 ft
Invert elevation	13.0 ft, NAVD88
Inlet Condition (per plan drawings)	Headwall
Outlet Condition	Mitered End Sections
Type of Control	Vertical Lift Slide Gate(s)
Canal Data (Downstream, Lateral)	
Invert - Thalweg	8.0 ft, NAVD88
Bottom Width	10.0 ft
Top of Bank	21.3 ft, NAVD88
Top Width	55.0 ft
Side Slope (h:v)	2.0 ft (h:1)
Canal Riprap Requirements	
Design Flow Velocity	4.2 fps
Design Riprap Velocity	7.0 fps
Downstream Length	50.0 Feet

Figure 39. S-114C HDDS

Loxahatchee River Watershed Restoration Project
S-116
Gated Culvert
Jenkins Ditch, Kitching Creek

Revisions

XY Coord 27° 1'25.7"N, 80° 9'56.9"W

Location Jenkins Ditch
Purpose To convey flow from Jenkins Ditch through a spreader system to the Northwest Fork of the Loxahatachee
Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions		
Discharge (CFS)	150	cfs
Headwater Elevation	11.80	ft,NAVD
Tailwater Elevation	11.30	ft,NAVD
Maximum Expected Stages		
Headwater Elevation	N/A	ft,NAVD
Tailwater Elevation	N/A	ft,NAVD
Minimum Expected Stages		
Headwater Elevation	N/A	ft,NAVD
Tailwater Elevation	N/A	ft,NAVD
Maximum Simultaneous Head Difference	N/A	ft
Culvert Data		
Number of Barrels	1	
Barrel Type	Concrete	
Barrel Dimensions	5 X 7	feet
Barrel Length	40.0	feet
Barrel Invert Elevation (upstream)	6.30	ft, NAVD
Barrel Invert Elevation (downstream)	6.20	ft, NAVD
Type of Control	Sluice Gate	
Canal Section		
Jenkins Ditch		
Side Slopes (Vert. on Hor.) 1 on	2	
Upstream Bottom Width	8.00	ft
Upstream Bottom Elevation	6.30	ft, NAVD
Canal Riprap Requirements		
Design Flow Velocity	4.30	fps
Design Riprap Velocity	7.00	fps
Downstream Length	75.00	feet
Downstream Width	50.00	feet

Figure 40. S-116 HDDS

Loxahatchee River Watershed Restoration Project
S-117
Ungated Weir

Revisions

XY Coord 26°59'9.1"N, 80° 9'49.4"W

Location Eastern end of Hobe Grove Ditch
Purpose Helps divert discharge through historic Moonshine Creek while helping to control discharges into the north-west fork of the Loxahatachee River
Notes 1. Riprap requirements have not been verified with Geotech.

Design Conditions

Weir		
Notch Design Discharge	400.00	cfs
Maximum Design Discharge	632.00	cfs
Design Head	5.40	feet
Design Headwater Elevation	8.50	ft,NAVD88
Design Tailwater Elevation	2.50	ft,NAVD88
Maximum Expected Stages		
Headwater Elevation	9.30	ft,NAVD88
Tailwater Elevation	3.90	ft,NAVD88
Maximum Simultaneous Head Difference	N/A	ft
Weir Design Data		
Weir Type	Broad Crested	
Crest Elevation	6.00	ft,NAVD88
Weir Breadth	2.00	feet
Crest Length	40.00	feet
Minimum Tieback Elevation	20.00	ft,NAVD88
Weir Control	None	
Canal Section		
Hobe Grove Ditch		
Side Slopes (Vert. on Hor.) 1 on	2.00	feet
Bottom Width	35.00	feet
Bottom Elevation	-3.00	ft,NAVD88
Canal Riprap Requirements		
Design Flow Velocity	3.50	fps
Design Riprap Velocity	7.00	fps
Downstream Length	100.00	feet
Downstream Armored Area	50.00	ft^2

Figure 41. S-117 HDDS

D.6 Spillway and Culvert Calculations

The uncontrolled weir (spillway) and culvert calculation approach for Flow-way 3 was similar to that described in section C.6.

Spreadsheet tools were used to preform initial calculations for the appropriate sizing for the Ogee spillway design based on design flow rates, head water and tail water conditions, as well as losses due to entrance, exit, crest elevation and width. Computational methods used in the spreadsheet analysis for the ogee spillway design are described further in EM 1110-2-1603, Hydraulic Design of Spillways. HEC-RAS modeling was used to confirm and further refine the design of the Recommended Plan.

D.7 Rainfall & Reservoir Routing

The Gulfstream West Flow-through Marsh through preliminary consequence analysis and criteria outlined in the Central Everglades Restoration Project (CERP), Design Criteria Memorandum 1 (DCM -1, Hazard Potential Classification) is determined to have a Hazard Potential Classification (HPC) of "Low". Water will either be pumped in at a rate up to the inflow pumping station's rated capacity, approximately 150 cfs, or by direct rainfall. Following the guidance in the DCM-2 – Wind Precipitation Design Criteria for Freeboard - (2005), the design storm for the Gulfstream West Flow-through Marsh was selected to be 100-year, 72-hour event. Additional description is included below.

D.7.1 Rainfall

Rainfall input was developed from National Oceanographic & Atmospheric Administration (NOAA) Atlas 14 Precipitation-Frequency Atlas of the United States (Volume 3, 2008) data for the 100-year, 72 hour storm. The precipitation estimate for the LRWRP project footprint for the Gulfstream Marsh Flow-through Marsh was 17.7 inches.

The PMP was developed according to guidelines from the Hydrometeorological Report (HMR) No. 51 (Probable Maximum Precipitation Estimates, Eastern U.S.) and HMR No. 52 (Probable Maximum Storm Computation). A value of 55.7 inches was selected from the all season 72 hour, 10 mi^2 PMP chart. A synthetic 72-hour PMP rainfall event was developed according to HMR-52 and input into the HEC-RAS model.

D.7.2 Spillway Capacity & Reservoir Routing

The DCMs provide recommended methodologies in use of design parameters and offer accepted avenues for the optimization of reservoir designs. Therefore, the Gulfstream West Flow-through Marsh embankment heights were designed using DCM-2 as the primary reference for all parameters and methodologies. The two most important design parameters in the design of embankment heights (excluding minimum freeboard requirements) are the selection of the Inflow Design Flood (IDF) and the design wind speed. Dependent on HPC, DCM-2 provides these two important design parameters. The minimum Federal standard freeboard on a reservoir with a Low HPC is three (3) feet above the surcharge pool determined by routing the 100-year 24-hour event in accordance with DCM-2. Since the design storm for the remaining project features within Flow-way 3 were designed for a 100-year, 72-hour event, the freeboard determination was also simulated using the 72-hour event to remain conservative. Additionally, since the Gulfstream West is a flow-through Marsh, it will contain sustained emergent vegetation which is assumed to impede the buildup of significant wave action. The reservoir is

also proposed to contain three (3) cross berms to promote sheet flow. These cross berms will aid in further dampening potential wind setup.

The HEC-RAS model was employed to determine the maximum pool (or surcharge pool) elevation and peak discharge. The model configuration consisted of a storage area equal to approximately 600 acres with an uncontrolled notched weir outlet. Since the invert of the discharge weir allows ponding and therefore the surface would be considered impermeable, all storm precipitation was readily available for routing without any loss. The initial pool stage used in the routing model for the marsh was the maximum normal pool stage of 16.25 ft. NAVD88. The IDF inflow, the 100 year frequency, 72-hour duration storm was simulated in HEC-RAS resulting in a peak water surface elevation of approximately 17 ft. NAVD88. This relatively minimal increase in water surface elevation is due to the discharge capacity of the notched weir design at higher water surface elevations. Therefore, the outflow spillway is designed to safely pass the 100-year frequency, 72-hour duration storm.

Although the Gulfstream West Impoundment carries a Low HPC, the PMP was also routed to compare the resulting maximum pool elevation against the design pool surcharge. Although the Impoundment is being designed for the 100-year frequency, 72-hour duration storm, the pool surcharge caused by the PMP remains contained within the proposed embankment height of 20.0 NAVD88, reaching a peak water surface elevation of 18.5 ft. NAVD88.

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ANNEX A-2

Loxahatchee River Watershed Restoration Project

Hydrologic Modeling

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1 Annex 2 Hydrologic Modeling

The Loxahatchee River Watershed Restoration Project (LRWRP) **Annex A-2 Hydrologic Modeling** includes a comprehensive overview of the Climate, Hydrology, Regional Management Operations and Flood Control aspects of the existing, future without project and future with project conditions within the study area. An assessment of these conditions at a regional hydrologic modeling scale was developed to assist the plan formulation process for the feasibility comprehensive analysis of LRWRP under SMART Planning. More detailed models were also applied to address specific questions related to water quality and hydraulic constraints. The more detailed hydraulic design information can be found in **Annex A-1 Hydraulic Design**. Detailed information about modeling scope developed for LRWRP can be found in **Annex A-3 Modeling Documentation Reports**.

1.1 Existing Conditions

The following sections provide a description of the existing conditions within the LRWRP study area.

1.1.1 Climate

The climate of the Loxahatchee River Watershed Restoration Project (LRWRP) area is considered subtropical with distinct wet and dry seasons, high evapotranspiration (ET) rates, and climatic extremes of floods, droughts, and hurricanes. Precipitation primarily occurs during the wet season from May through October when almost two-thirds of the annual rainfall occurs. During the wet season, thunderstorms that result from easterly tradewinds and land-sea convection patterns occur almost daily. Wet-season rainfall follows a bimodal pattern, with peaks during mid-May through June and September through mid-October. Tropical storms and hurricanes provide major contributions to wet season rainfall with a high level of interannual variability and low level of predictability. The balance of rainfall occurs in the dry season months, November through April. During the dry season (November through April), rainfall is governed by large-scale winter weather fronts that pass through the region approximately weekly. However, due to the variability of climate patterns (La Niña and El Niño), dry periods may occur during the wet season and wet periods may occur during the dry season. Multi-year high and low rainfall periods often alternate on a time scale approximately on the order of decades. These interannual extremes in rainfall result in frequent years of flood and drought (USACE 1999).

High evapotranspiration rates in LRWRP area roughly equal annual precipitation, with ET returning approximately 45 inches of water per year to the atmosphere (SFWMD 2002b). Evapotranspiration removes from 70% to 90% of the rainfall in undisturbed south Florida wetlands (Duever et. al. 1994). Evaporation from open water surfaces peaks annually in the late spring when temperatures and wind speeds are high and relative humidity is low. Evaporation is lowest during the winter when the temperatures and wind speeds are low (Duever et. al. 1994).

Global climate change and variability, particularly at regional levels, are not completely understood. Over the last two decades, South Florida Water Management District (SFWMD) scientists have investigated how natural, global climatic patterns such as the El Niño/La Niña Southern Oscillation and the Atlantic Multi-decadal Oscillation (AMO) are linked to South Florida's weather and climate.

Since 1900, there have been two cool phases and two warm phases of the AMO cycle; each phase lasted approximately 20-40 years. The exact year of the phase start and finish is an estimate as each phase goes through a "transition period" of a few years. South Florida was in a much drier regime from 1965 to the early 1990s when the AMO transitioned from the cool phase to the warm phase. South Florida experienced more droughts and dry weather during the cool phase. High-water events (some extreme) have been more frequent during the current warm phase. South Florida has been in a "wetter" regime since the early 1990s, mostly due to the AMO. With

AMO phases lasting typically 20-40 years, the current AMO warm phase has likely peaked. Thus, the generally wetter-than-normal conditions that Florida has experienced since the early 1990s should begin to slowly decline and we will see continually cooler conditions over the next 10-20 years, with an increase in dry years compared to wet years as we move into the next cool phase. However, low-frequency dry years can still occur during this warm phase due to other events such as La Niña, which can occur every 2-7 years, on average.

Regional climate studies of observed air temperature trends show an increase in temperature with a general consensus in an increase in minimum and maximum temperatures. Observed precipitation show no discernible trends in annual/seasonal precipitation but shows an increase in the frequency and intensity of extreme precipitation events. Heavy precipitation events in most parts of the United States have increased in both intensity and frequency since 1901. Extreme precipitation events are generally observed to increase in intensity by about 6% to 7% for each degree Celsius of temperature increase (USGCRP 2017). The annual frequency of hurricanes has remained relatively stable throughout the 20th and early 21st centuries; however, hurricane rainfall is expected to increase for Florida as the climate continues to warm. No trend in observed streamflow was found.

Sea-level change has been a persistent trend for decades in the United States and elsewhere in the world. Observed and reasonably foreseeable global sea level rise (SLR) means that local sea levels will continue to rise beyond the end of this century. In most locations, global SLR results in local relative SLR, which has already caused impacts such as flooding and coastal shoreline erosion to the nation's assets located at or near the ocean. These impacts will continue to change in severity. Along the U.S. Atlantic Coast alone, almost 60 percent of the land that is within a meter of sea level is planned for further development (USACE, 2018b).

1.1.2 Hydrology

The major characteristics of south Florida's hydrology are local rainfall, basin storage, evapotranspiration, gentle slope topography, and the highly permeable surficial aquifer underlying much of coastal Dade, Broward, and Palm Beach Counties. The management of surface water is generally conceived on a watershed level and in the project area, two watersheds have been delineated, the Loxahatchee River Watershed and the Lake Worth Lagoon Watershed. The Loxahatchee River Watershed includes the following basins: Kitching Creek, the Grove, Cypress Creek/Pal-Mar, Wild & Scenic River/Jupiter Farms, Loxahatchee Estuary, and the C-18/J.W. Corbett Wildlife Management Area (WMA). The L-8 Basin, C-17 Basin, the GWP Basin, and the C-51 Basin are part of the Lake Worth Lagoon Watershed.

Management within the project area is currently achieved by the East Coast Canals system (C&SF Restudy 1999). The East Coast Canals are the flood control and outlet works that extend from St. Lucie County southward through Martin, Palm Beach, and Broward counties to Miami- Dade County, a distance along the Atlantic Coast of approximately 170 miles. The East Coast Canals watersheds encompass the primary canals and water control structures located along the lower east coast of Florida and their hydrologic basins. Stormwater runoff in the project area is drained via canals and rivers to the St. Lucie River, the Loxahatchee River, and the Lake Worth Lagoon. Some of this water is also diverted to the Northwest Fork of the Loxahatchee River (NWFLR) for environmental water supply purposes and as a result of population growth, water use needs have necessitated that some of the runoff be retained for water supply. A better understanding of the importance of wetlands and surface waters to the health of the environment has also guided the decision to decrease the volume of water expeditiously drained off the land. The balancing of flood control, water supply needs, and the environment has resulted in a complex system of water management facilities within the project area.

During the dry season, gated control structures located near the coast (i.e., S-155, S-46) are typically closed to maintain adequate water levels in the canals, recharge the groundwater and prevent saltwater intrusion. In some

cases, the canals help recharge wellfields and thus it is important that adequate water is available. Currently, operations during the wet season pass excess stormwater through the canals and out to tide, rather than storing and using it during the next dry season. Without sufficient storage, it has been difficult to have water available during the dry season without causing flooding during the wet season. Due to urbanization, the existing surface water management system now has to handle greater peak flows than in the past (C&SF Restudy, 1999) and the peak stormwater discharges have adverse impacts on the estuaries.

Hydrologic modeling simulations of the existing condition baseline (ECB) and the LRWRP future without project condition (FWO) were developed using the Lower East Coast Subregional – North Palm (LECSR-NP) modeling tool, to provide baseline conditions for plan formulation, the assessment of LRWRP project benefits (comparisons against FWO), and the assessment of LRWRP alternative performance for the level-of-service for flood protection and water supply (comparisons against ECB). Detailed descriptions of the hydrologic models, can be found in **LRWRP Modeling Documentation** of this Annex. The ECB was developed to represent the system-wide infrastructure and operations that were in place at the time LRWRP plan formulation was initiated, approximately in the year 2014. Historical climate conditions were used for the simulation period, occurring from 1965-2005, and permitted allocations were used for the demands for all public water supply utilities. Features simulated in the existing condition base include the recent modifications to the G-92 structure, the North Lake Boulevard weir, wetland improvement areas constructed by Palm Beach County, regional system deliveries to the City of West Palm Beach, the east Corbett weir and the existing canal operations for the SFWMD canals, water control districts (298 Districts) and local developments.

1.1.2.1 Central & Southern Florida System Overview and Relationship with the Loxahatchee River Watershed Restoration Project

For most studies in central and south Florida, projects are part of the Central and Southern Florida (C&SF) Project, a larger system of interconnected water management features. The C&SF system is designed to capture, store, clean, and redistribute water to the south Florida ecosystem.

The C&SF Project was authorized by the U.S. Congress in 1948 in response to significant flooding in south Florida. With its complex, regional water management infrastructure, significant portions of the natural system in central and south Florida were altered. In response to the unintended impacts of the C&SF Project, the Water Resources Development Act (WRDA) 2000 approved the Comprehensive Everglades Restoration Plan (CERP), which is the framework for modification and operational changes to the Central and Southern Florida Project needed to restore, preserve, and protect the south Florida ecosystem while providing for other water-related needs of the region, including water supply and flood protection.

The Comprehensive Everglades Restoration Plan (CERP) is the largest environmental restoration program in history with the restoration of the 18,000 square mile south Florida ecosystem. CERP focuses on “getting the water right” in the south Florida ecosystem. The plan is composed of a series of projects designed to address four major characteristics of water flow: quantity, quality, timing, and distribution. In total, 68 individual components comprise more than 50 projects in the plan. Together, these projects aim to get the right amount of water, of the right quality, delivered to the right places, at the right times. Implementing projects that capture, store, clean, and redistribute water will restore natural water flow, enhance and protect habitats, and improve the ability to retain and utilize freshwater within the ecosystem (USACE, 2015b).

The CERP proposes to restore more natural sheetflow patterns by eliminating unnatural barriers to flow (i.e. canals/levees) that drain the current landscape. Prior to construction of the C&SF Project, the inundation pattern

of the pre-drainage system within the LRWRP area supported an expansive system of freshwater marshes. The LRWRP is part of the CERP. WRDA 2000 recognized that the CERP was conceptual in nature and that refinements would be required during further studies and implementation. Changed conditions since 1999, new scientific information, and learning from the first projects to be implemented all should influence the selection of specific measures and scales of later projects, such as LRWRP. LRWRP has one or more ecological objectives that correspond to each of the CERP ecological objectives

The purpose of the Loxahatchee River Watershed Restoration Project is to improve the timing and distribution of wet and dry season flows to the Northwest Fork of the Loxahatchee River floodplain to help restore native plant and animal species abundance and diversity in Loxahatchee River watershed natural areas, river, and estuary. Portions of the Loxahatchee River and Loxahatchee River Estuary are subject to tidal influences through the direct connection to the Atlantic Ocean from the Jupiter Inlet.

1.1.2.2 Hydrologic Basins

Most of the surface water runoff from the Loxahatchee Watershed historically drained through Hungryland and Loxahatchee Sloughs to the Northwest Fork where tidal interaction was limited. Flood control improvements that were implemented with the construction of the C&SF project canals and levees altered hydroperiods and water depths within the study area, and resulted in reduced flows to the Northwest Fork by diverting freshwater that historically flowed naturally to the Northwest Fork to the Southwest Fork and to tide. The altered drainage patterns and lowered groundwater levels due to canals, roads, and levees in many upper watershed natural areas further limited dry season flows. The permanent opening and stabilization of the Jupiter Inlet further compounded problems resulting from diminished flows by allowing more saltwater movement up the estuary. Significant reduction of the groundwater levels and lack of surface water storage in the tributary basins results in uncontrolled excess wet season flows, and limited availability of surface and ground water in the dry season.

These hydrologic alterations have substantially altered the species composition and structure of the native plant communities, reduced the abundance and diversity of animals, and facilitated the spread of non-native vegetation. The various drainage and flood control features dissected the landscape into a mosaic of variously-sized habitat patches. The canals adjacent to the project area generally create barriers to wildlife movement, interfering with or preventing life functions of many native wildlife species. The channelization of flows in the parts of the Pal-Mar and Groves basins over-drains area wetlands and results in large erosive stormwater discharges in Cypress Creek. Ditching has increased drainage and reduced connectivity of natural upstream wetlands, altering hydroperiods and degrading their structural integrity and function. The large pulses of freshwater followed by periods of insufficient freshwater inflow into the Loxahatchee River estuary result in an unnatural salinity oscillation, further degrading ecological communities requiring stable salinity regimes. Insufficient base flows to the NWFLR have resulted in extensive changes to the riparian river vegetation. Collectively, the hydrologic changes have promoted the upstream movement of saltwater. As a result, cypress and other freshwater vegetation intolerant of elevated salinity conditions have been replaced by mangroves and other estuarine plant communities. If freshwater dry season flows are not increased to improve riverine system resilience, the salinity cline will continue to extend further up river than under historical conditions, thereby converting more freshwater habitat to estuarine habitat; these effects are likely to be exacerbated by potential sea level rise effects.

The Loxahatchee River watershed is located within northeastern Palm Beach and southern Martin Counties, and discharges into the Atlantic Ocean through the Jupiter Inlet near Jupiter, Florida. This watershed is approximately 242 square miles, and includes Jonathan Dickinson State Park (JDSP), portions of the J.W. Corbett WMA, Cypress

Creek, Pal-Mar, Hobe Groves, C-18 Basin, and the Loxahatchee Estuary. The three forks of the Loxahatchee River (Northwest Fork, the North Fork, and the Southwest Fork), along with Kitching Creek, Cypress Creek, and the C-18 Canal make up the primary drainage network for this watershed. Tributary streams include Hobe Grove Ditch and the C-14 Canal or South Indian River Water Control District (SIRWCD). Surface waters generally flow from west to east and reviews of historic aerials indicate overland flows from the southwest to northeast as well.

There are 8 major sub-watersheds that were analyzed as part of this project are (see **Figure 1**):

- The Kitching Creek Basin is in the northeastern portion of the watershed, draining into the NWFLR. The southern portion of the 18 square miles of this basin contribute runoff from natural lands, such as Jonathan Dickinson State Park (JDSP).
- The Grove Basin consists of 17 square miles of predominately agricultural land that discharges into the NWFLR either via Cypress Creek or via Hobe Grove Ditch.
- The Pal-Mar Basin is a 43 square mile basin draining mostly wetlands in the western portions of the watershed. It includes the sparse residential communities of Ranch Colony, the Links and Old Trail, and is one of the major tributaries to the NWFLR.
- The Jupiter Farms Basin is over 16 square mile area with the majority of the land area comprising the South Indian River Water Control District (SIRWCD). This area is a rural, residential community with an extensive managed canal system that discharges primarily to the NWFWR via SIRWCD's Canal 14.
- The Historic Cypress Creek Basin is a more than 6 square mile area that includes the Wild and Scenic River reach of the Loxahatchee River.
- The Loxahatchee Estuary is a 2 square mile area of highly developed urban land use. This basin accepts runoff from the three forks (NW, SW and North) of the Loxahatchee River.
- The C-18/Corbett Basin encompasses approximately 103 square miles in northern Palm Beach County. This basin includes a portion of the J.W. Corbett Wildlife Management Area (WMA), the Loxahatchee Slough, and miscellaneous agricultural and residential communities. This basin's main drainage facility includes the two forks of the C-18 (C&SF) Canal. Water surface elevations within this canal system are controlled by three structures; C-18 weir at the west, G-92 culvert that diverts water to the NWFLR, and S-46 spillway that discharges to tide via the SW Fork.
- The L-8 Basin encompasses approximately 157 square miles in northwestern Palm Beach and southwestern Martin Counties. It includes a portion of J.W. Corbett WMA, Dupuis Wildlife and Environmental Area, Indian Trail Improvement District, and agricultural and residential lands. The L-8 Canal is connected to the City of West Palm Beach's M-Canal via the L-8 Tieback Canal and the Control 2 Pump Station. The south end of the L-8 Canal terminates at its intersection with the C-51 Canal.

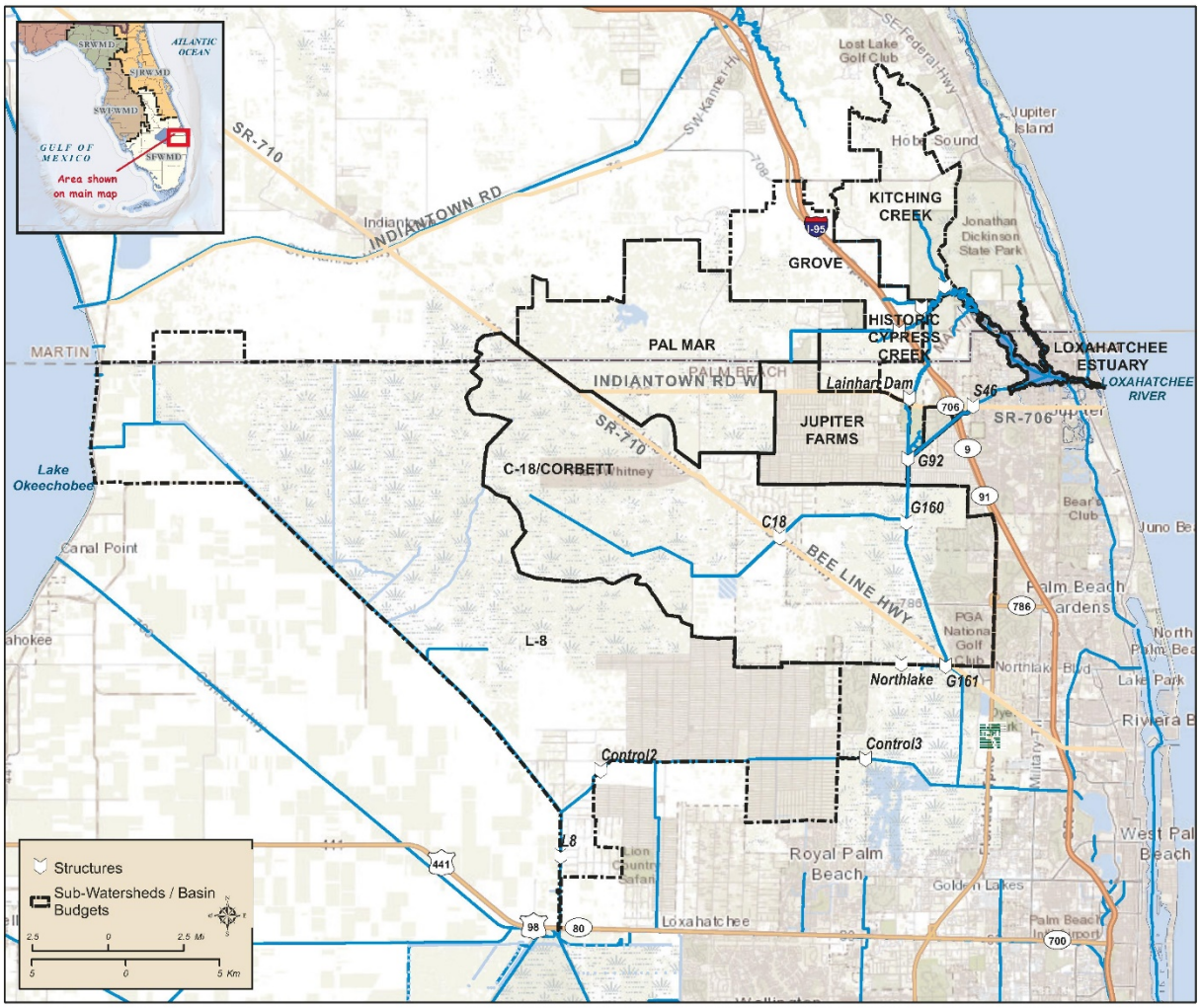

Figure 1. LRWRP sub-watersheds and structures.

Flow-ways are locations in the study area based on existing natural areas, topography, and associated canals, and are generally separated from each other by developed lands. Each flow-way represents a pathway for water to flow from the watershed to the Loxahatchee River. Flow-way 1 consists of portions of the L-8 and C-18 Basins. Flow-way 2 is located entirely within the C-18/Corbett watershed. Flow-way 3 is within multiple watersheds including Palmar, Grove, Kitching Creek, and the historic Cypress Creek. A generalized map of the various Flow-way locations is shown in **Figure 2**.

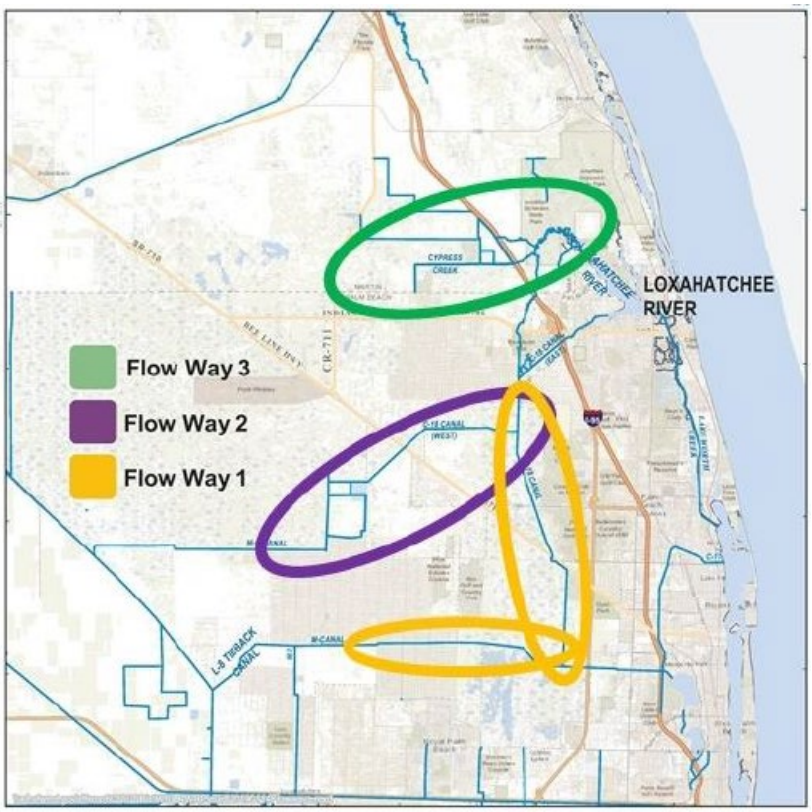

Figure 2. General Locations of LRWRP Flow-ways

1.1.3 Regional Water Management (Operations)

Stormwater runoff in the project area is drained via canals and rivers to the St. Lucie River, Lake Worth Lagoon, and the Loxahatchee River. Some of this water is also diverted to the Northwest Fork of the Loxahatchee River (NWFLR) for environmental water supply purposes and as a result of population growth, water use needs have necessitated that some of the runoff be retained for water supply. A better understanding of the importance of wetlands and surface waters to the health of the environment has also guided the decision to decrease the volume of water expeditiously drained off the land. The balancing of flood control, water supply needs, and the environment has resulted in a complex system of water management facilities within the project area.

During the dry season, gated control structures located near the coast (i.e. S-46) are typically closed to maintain adequate water levels in the canals, recharge the groundwater and prevent saltwater intrusion. In some cases, the canals help recharge wellfields and thus it is important that adequate water is available. Currently, operations during the wet season pass excess stormwater through the canals and out to tide, rather than storing and using it during the next dry season. Without sufficient storage, it has been difficult to have water available during the dry season. Due to urbanization, the existing surface water management system now has to handle greater peak flows than in the past (C&SF Restudy, 1999) and the peak stormwater discharges have adverse impacts on the estuaries.

The management of surface water is generally conceived on a watershed level and in the project area, three watersheds have been delineated: the St. Lucie River Watershed, the Loxahatchee River Watershed, and the Lake Worth Lagoon Watershed. Within the St. Lucie Watershed, some portions of the C-44 Basin and the south St. Lucie Basin are within the project area. All of the Loxahatchee River Watershed is within the project area and includes the following basins: Kitching Creek, the Grove, Cypress Creek/Pal-Mar, Wild & Scenic River/Jupiter Farms, Loxahatchee Estuary, and the C-18/J.W. Corbett Wildlife Management Area (WMA). The L-8 Basin, C-17 Basin, the GWP Basin, and the C-51 Basin are part of the Lake Worth Lagoon Watershed. While the St. Lucie River Watershed

is technically part of the project area, the focuses of the LRWRP project are the Loxahatchee River watershed.

The following sections provide a description of the existing regional operations within the Loxahatchee River Watershed Restoration Project (LRWRP) study area. More information about the Regional operations within LRWRP can be found in **Annex C, LRWRP Draft DPOM.**

1.1.3.1 Kitching Creek

The Kitching Creek Watershed contains areas of development and slough. Natural drainage patterns have been altered due to previously constructed roadways, small to medium residential developments, and the Kitching Creek Central Flow-way Restoration Project. Re-directed flow is routed to the Jenkins Ditch, which discharge uncontrolled into the NWFLR.

1.1.3.2 Grove

The Grove basin is predominately agricultural land. The property drains through 3-84" culverts, with an invert of 7.7' NGVD29 and a control elevation of 12.0' NGVD29 and a maximum permitted drainage of 4”/day. These culverts drain into the Hobe St. Lucie Canal that currently runs through the Gulfstream west property, and ultimately discharges to the Cypress Creek Canal.

1.1.3.3 Palmar Basin

The Cypress Creek canal provides drainage for Palmar East/Culpepper (including Nine Gems), Ranch Colony communities, the Gulfstream West property, and Hobe St. Lucie Conservancy District (HSLCD). The existing Cypress Creek structure has a control elevation of 2.0' NGVD29. The Ranch Colony Communities have existing control elevations ranging from 14.5-16.0 ft NGVD29.

1.1.3.4 Jupiter Farms

The G-92 is located on the north bank of the C-18 Canal and discharges into South Indian River Water Control District’s (SIRWCD) C-14 which in turn discharges to the NWFLR and Lainhart dam. The G-92 structure regulates the amount of water from the C-18 northward to the NWFLR and eastward to the S-46 for discharge to tide. The G-92 structure is a bi-directional structure can discharge up to 400 cfs to the NWFLR when stages within the C-14 and C-18 allow.

1.1.3.5 Historic Cypress Creek

The Lainhart and Masten Dams, located on the NWFLR and within the Historic Cypress Creek watershed, control and regulate upstream flow stages throughout the Northwest Fork and are used to prevent the migration of brackish water upstream, thereby alleviating the potential for adverse impacts upon the adjacent ecosystem. The Lainhart Dam is critical to the efficacy of the Northwest Fork, and serves as a stage and flow monitoring station. Staff gauges located upstream and downstream of the dam are used to continuously monitor stage elevations, and a stage-discharge relationship has been calibrated and utilized to estimate discharge quantities into the Northwest Fork. Both structures are uncontrolled fixed crest weirs.

1.1.3.6 Loxahatchee Estuary

There is no regional water management within the estuary.

1.1.3.7 C-18/Corbett

The C-18 Canal is a component of the Central and South Florida Flood Control System (C&SF) that was constructed in the 1950’s by the USACE to capture and remove runoff from the adjacent lands. The canal has an east leg which drains from south to north, and a west leg which drains from west to east. The two legs join and continue north and then east to discharge through the S-46 structure, located on the southwest Fork of the Loxahatchee River.

The S-46 structure was constructed for flood control purposes, and routes freshwater that historically flowed naturally to the Northwest Fork, away to the Southwest Fork. The S-46 can discharge up to 4000 cfs if canal stages within the C-18 exceed 15.0 ft NGVD29

The west leg of C-18, C-18W, is approximately 7.9 miles long with headwaters consisting of the C-18/J.W. Corbett Wildlife Management Area (WMA). Water surface elevations in the C-18 West leg are controlled by the C-18 Weir, which maintains water surface elevations west of the Beeline Highway at 17.6 ft., National Geodetic Vertical Datum (NGVD).

The J.W. Corbett WMA discharge is currently controlled through a 140' sheet pile weir at elevation 21.5' NGVD29 along the eastern side of Corbett. This weir allows uncontrolled discharge onto the "Mecca" property. Additionally, two water control structures within the southern containment berm provide discharge to the M-O Canal from J.W. Corbett when water surface elevations exceed the control invert.

1.1.3.7.1 M-Canal

The M-Canal provides a hydraulic connection between the L-8 Canal, via the L-8 Tieback Canal to the City of West Palm Beach. Water from the tie-back canal is pumped by the City of West Palm Beach's Control 2 Pump Station into the M-Canal. The M-Canal then flows eastward, through the Grassy Waters Preserve (GWP) marsh and ultimately to Lake Mangonia and Clear Lake. Discharges at Control 4 are not permitted when Clear Lake is above 12.5' NGVD29. The M-Canal conveyance is limited to 225 cfs and is normally maintained 0.2-0.3 feet below the GWP, which has an allowable permitted elevation of 17.6-19.2 ft NGVD29.

1.1.3.7.2 Indian Trail Improvement District (ITID) M-1 Basins

ITID has a permitted allowable peak discharge of 274 cfs for flood protection to the L-8 Canal. However, most discharges (up to 750 cfs) occur via the M-1 Canal to C-51 unless C-51 stages restrict such flows as specified in ITID permit. The M-1 Upper Basin control elevation is 17.0 ft. NGVD29 in the dry season (Jan-Apr, Nov-Dec) and 16.0 NGVD29 in the wet season (May-October). The M-1 Lower Basin control elevation is 17.0 ft. NGVD29 in the dry season (Jan-Apr., Nov-Dec) and 15.0 in the wet season (May-Oct).

1.1.3.8 L-8 Canal

This canal is located in western Palm Beach County and extends north from its intersection with the West Palm Beach Canal (C-51) for 4.0 miles then northwest for 17.9 miles then west for 3.9 miles to connect with C-10A at Lake Okeechobee, with flow being controlled by the S-76 structure. The purpose of the L-8 Canal is to protect the developed agricultural area around the southeastern shore of Lake Okeechobee from overflow by flood waters originating in the wide flat area located to the north of and east of the levee alignment. There is no single structure that serves to maintain the water surface elevation in the L-8 Canal. The simultaneous operation of a network of structures (at the C-51 intersection, Lake Okeechobee, and the L-8 Tieback Canal) is intended to maintain water levels in the range of 12.5 ft. NGVD29.

1.1.4 Flood Control

Flood protection is evaluated by a combination of best professional judgment interpreting model results and engineering analyses. Consistent with the Draft Guidance Memoranda, the same models and results used for plan formation (Lower East Coast Subregional – North Palm (LECSR-NP)) were applied for the LRWRP Savings Clause assessment. An analysis of the long period of record simulation was completed, focusing on the wet events included within the 1965–2005 simulation period. Stage duration curves for various areas of interest compare the

groundwater elevations of the TSP relative to the FWO and ground elevations to determine if the TSP may impact flood protection.

Flood protection within the study area were analyzed including:

- residential developments in Martin County (Ranch Colony, Old Trail and others),
- Hobe St. Lucie Conservancy District Unit 2 Agricultural Area,
- residential areas adjacent to Grassy Waters Preserve,
- upstream of Kitching Creek, and
- Jupiter Farms residential area.

Groundwater levels in the study area have been significantly impacted through the development of a secondary canal system with little or no control to manage canal flows and stages. Residential areas in Flow-way 3, the Unit 2 Agricultural Area, and the area of Pal-Mar to the west drain through the Cypress Creek canal system, resulting in extremely variable discharges to tide. In the existing condition, all the drainage to Cypress Creek flows unimpeded, resulting in excessive storm water flows and excessively reduced area groundwater levels. This also results in excessive discharges via the Cypress Creek Canal to the Cypress Creek Tributary in the wet season and little or no available water during the dry season. Complaints of depleted groundwater stages, resulting in excessive interior lake drawdowns in the dry season, are common. The Recommended Plan results in a more effective control of stormwater flows and an increase in groundwater levels, in some cases by greater than two (2) feet. Control elevations for the communities are variable, ranging from 14.5’ NGVD to 17.0’ NGVD.

Additional flood protection analysis was completed for the Flow-way 2 and Flow-way 3 project areas through the use of a hydraulic model, Hydrologic Engineering Center’s River Analysis System (HEC-RAS) 1D/2D. Event based model simulations for the 100-year, 72-hour storm were analyzed to understand the flood control conditions under existing conditions.

1.2 Future Without Project Condition

The following sections provide a description of the future without project conditions within the Loxahatchee River Watershed Restoration Project (LRWRP) study area.

1.2.1 Climate

During the period between the present and 2078, south Florida should experience a full multi-decadal cycle of Atlantic hurricane activity. Currently the area is in an active phase of this cycle that started in 1995. This active phase followed a 25-year period of low hurricane activity. This suggests that between the present and year 2078, the area would complete this active phase, pass through another low activity period and begin another active phase. There is now evidence of anthropogenic changes to global climate patterns that will likely have an impact on south Florida in terms of rainfall, evapotranspiration, and temperature (Intergovernmental Panel on Climate change 2007). Climatologists predict air temperatures will increase, with projections of summer temperatures being up to 3°F to 7°F warmer by 2100 (Twilley et.al. 2001, Union of Concerned Scientists 2008). Increases in air temperature, solar radiation, and water vapor deficit due to climate change are expected to increase evapotranspiration. Models used by Calanca et al. (2006) predict a 20 percent increase in evapotranspiration if summer temperatures increase from 4°F to 7°F.

Regional surface water storage systems (lakes, rivers, canals, reservoirs, water conservation areas) will most likely experience more rapid water loss when compared to current levels, ultimately impacting availability of water supplies. Increased evapotranspiration may increase water demand for irrigation and natural wetlands areas. In addition, accelerated evaporation losses from stormwater treatment areas could impact their phosphorus removal performance, increasing the need for supplemental water for these facilities.

The Florida Oceans Council (2009) predicts more frequent intense rainfall events will occur coupled with longer dry periods in between. SFWMD data indicate that there has been an increase in heavy downpours in many parts of the region, while the percentage of the region experiencing moderate to severe drought increased over the past three decades. While periodic heavy downpours may increase overall precipitation totals, much of the water may be runoff that is eventually lost to the Atlantic Ocean. The environmental impact of changes to floods and droughts depends on the relationship between the climate extremes. More droughts, without an increase in flooding conditions, pose a threat to the LRWRP area. They would likely cause large shifts in community structure due to saltwater intrusion into freshwater habitats within the Loxahatchee Estuary, drying and disappearance of inland wetlands, and an increase in frequency of fires (both terrestrial and wetland). Without the ability to maintain minimum flows and water levels in network of canal within LRWRP, agriculture and public water supply well fields may not be able to function as designed. In addition, well fields may be contaminated by saltwater intrusion and higher salt levels in coastal waters may limit the usefulness of currently installed desalinization plants. Larger precipitation events may impact the level of flood protection and cause larger freshwater pulses into the Estuary. Additionally, increased flooding conditions may cause over-inundation of wetlands and create more frequent water level reversals during critical wading bird foraging periods, thus causing further declines in nesting success for wading birds.

Current research indicates overall storm frequency may decrease, while the number of strong hurricanes (due to warmer temperatures) is expected to increase. Tropical storms and hurricanes provide huge amounts of rain for the area. The loss of storm-associated rainfall could have significant implications for the SFWMD regional water supplies. If a decrease in the number of storms does occur, there may be significant changes to the distribution of rainfall, which will affect the water supply and natural ecology of the LRWRP. Less rainfall may mean the region is under drought conditions more often. If tropical storms and hurricanes become more intense, the potential damage to levees, canals, and other water control structures may also increase – resulting in an increased likelihood of flooding on a local and regional scale.

Sea level change is one of the more certain consequences of climate change, and because it affects the land/ocean interface, it has the potential for environmental impacts on coastal areas. Various sites along the east coast of Florida indicate that the sea level is rising at a rate above the global average (Maul and Martin 1983). USACE sea level change projections from 1992 for the LRWRP period of analysis to 2125 for Miami Beach, Florida area for historic, intermediate and high rates of future sea level rise in mean sea level relative to NAVD88 are shown in **Figure 3.** The Miami Beach, Florida gage was selected as it was the closest compliant gage (long enough period of record) available for use within the USACE sea level change calculator tool (USACE, 2018)

The regional hydrologic models used to simulate with and without project conditions require climatic and tidal data as boundary conditions. Given the uncertainty in future climatic conditions, the historic climate conditions used in the period of record are assumed to represent conditions that are expected to occur in the study area in the future. Sea level change is not included in the FWO modeling boundary conditions for LRWRP in the LECSR-NP model. In order to evaluate the sea level rise impact on the restoration project, a curvilinear three-dimensional hydrodynamic model CH3D (Sheng, 1986), was applied to the Northwest Fork of the Loxahatchee River and the Loxahatchee River Estuary. To incorporate projected SLR, the tidal boundary condition offshore of the Jupiter Inlet was developed by combining tidal components and projected mean sea level (MSL), including SLR, a observed for

Miami Beach gage. Two alternatives were evaluated, the 2070 future without project (2070FWO) and the Recommended Plan at year 2070. The future with-project condition baseline was selected at year 2070 as it aligned with the hydrologic output from LECSR-NP. The impact of sea level change on project benefits is assessed for the FWO and with project conditions per USACE guidance ER 1110-2-8162, *Incorporating Sea Level Change in Civil Works Programs* and Engineer Technical ETL 1100-2-1, *Procedures to Evaluate Sea Level Change: Impacts, Responses, and Adaptation*. LRWRP is most vulnerable to climate change and at risk over the project life cycle (2020-2120) due to the following factors: increasing air temperatures, increases in extreme storm frequency and intensity, and rising sea-level elevations. SLR does affect the hydrologic boundaries governing the performance and operation of the LRWRP project features; however, the with-project condition provides better ecological conditions and resiliency for the river then the future without project condition under all SLR scenarios. In addition to ecosystem impacts, existing and proposed infrastructure within the LRWRP footprint are susceptible to sea level rise. Existing structures include the Lainhart Dam, Masten Dam, S-46 and Ranch Colony Canal and Hobe Grove Ditch water control structures within the Loxahatchee River project area. The Lainhart and Masten Dams are not being impacted by any future sea level rise scenario during the 100-year planning horizon. In general, future rates of sea level change are expected to result in impacts on coastal canals and communities, with loss of flood protection and increased saltwater intrusion being the primary effects. Additionally, coastal ecosystems and estuaries are expected to be adversely affected and require additional deliveries of freshwater to maintain desirable salinity patterns and healthy ecosystems. Inland hydrology and sea level change are discussed in more detail within **Annex H Climate Change Assessment** of the Main PIR.

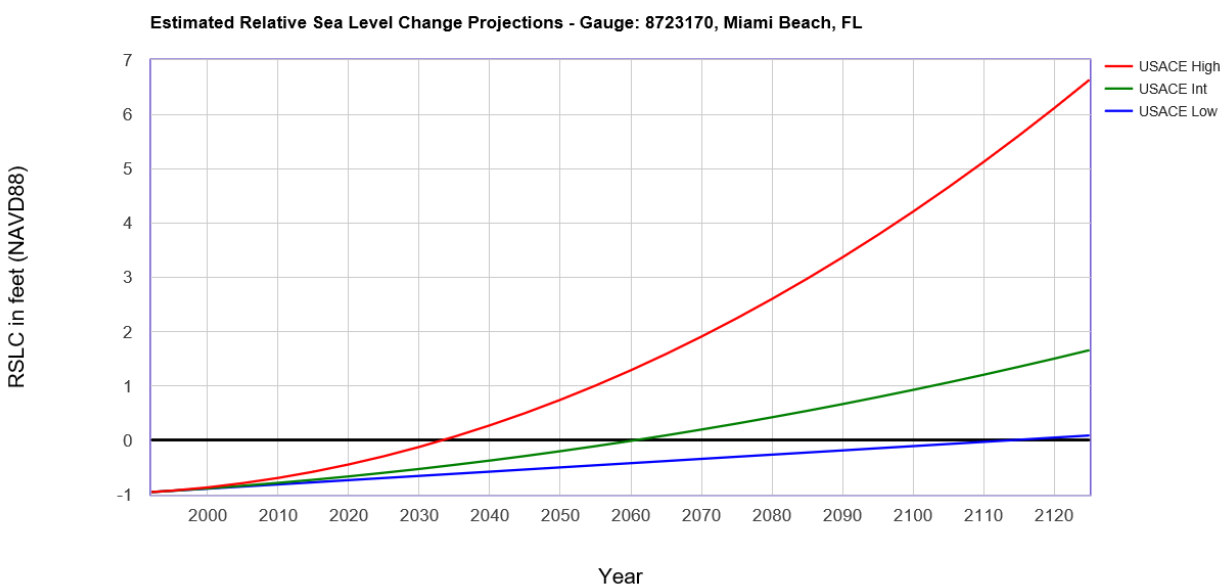

Figure 3. Estimated Sea Level Change for Miami Beach, FL Gauge

1.2.2 Hydrology

Hydrologic modeling simulations of the FWO condition includes all the existing condition base assumptions, except it is assumed that the L-8 Flow Equalization Basin (FEB) is operational and is receiving water from outside of the L-8 Basin. In addition, the future without project base includes the recent proposal for the Avenir property

which creates two wetland areas on the northern portion of the property and an urban development at the southern section of the property.

The LRWRP baselines as well as all alternatives hydrologic results are discussed in **Section 1.3.2 Hydrology of the Final Array.**

1.2.3 Regional Water Management (Operations)

The FWO for LRWRP assumes similar Regional Water Management Operations as specified in the ECB in Section 1.1.3 with the addition of the L-8 FEB, located in the L-8 Basin. Stages for L-8 FEB are input for the FWO conditions based on output from the Restoration Strategies DMSTA 2012 Project modeling. The project assumes that L-8 FEB water source is S-5A basin and C-51W via S-5A complex which is outside the LECSR-NP model boundary.

1.2.4 Flood Control

The negative effects associated with flooding are expected to increase during the period between the present and the year 2075. As agricultural and urban development continues, the volume, duration, and frequency of floodwaters may increase, and the actual levels of service for flood control may decline in some areas. If sea level change continues as predicted, it is foreseeable that the surficial aquifer is likely to experience greater intrusion of saltwater possibly rendering some of the current water supply well fields unusable due to contamination. Higher groundwater stages in the project area would reduce the ability of water managers to store rainfall runoff whether within wetlands or the surficial aquifer, resulting in increased intensity of stormwater discharges through the primary canals. Reduced water storage reduces the capacity of the flood control system to accommodate runoff and would likely lead to increased frequency of flooding events. Sea level change may also impact flood control effectiveness as rising tail water conditions at the coastal canal structures reduce the effective maximum discharge rates (assuming there is an attempt to maintain the existing headwater schedule). As additional information becomes available, these structures may be modified or replaced with pumps to ensure continued effective flood control. This may also require the implementation of forward pumping to maintain the existing level of flood protection in the future. An analysis of sea level change on the Recommended Plan is discussed in **Annex H Climate Change assessment**.

1.3 Future With Project Conditions: Alternatives

The following sections provide a description of the future with project conditions within the Loxahatchee River Watershed Restoration Project (LRWRP) study area.

1.3.1 Climate

Climate change is expected to alter rainfall and evapotranspiration patterns over the next 100 years. Implementation of any of the LRWRP alternatives would have a negligible effect on climate within the action area. Minor, localized and less than significant effects to microclimate may occur under all LRWRP alternatives as a result of redistribution of water and shifts in vegetation. Potential effects may include increases in evapotranspiration and temperature changes due to an increase in spatial extent of wetlands, changes in vegetative communities, and redistribution of water.

Since 1900, there have been two cool phases and two warm phases of the Atlantic Multi-decadal Oscillation (AMO) cycle with each of these phases lasting approximately 20-40 years each. The exact year of the phase start and finish is an estimate as each phase goes through a “transition period” of a few years. South Florida was in a much drier regime from 1965 to the early 1990s when the AMO transitioned from the cool phase to the warm phase. South Florida experienced more droughts and dry weather during the cool phase, with high-water events (some extreme) being more frequent during the current warm phase. South Florida has been in a “wetter” regime since the early 1990s mostly due to the AMO. With AMO phases lasting typically 20-40 years, the current AMO warm

phase has likely peaked. Thus, the generally wetter than normal conditions that Florida has experienced since the early 1990s should begin to slowly decline. After the peak, the warm phase wave will begin its gradual decline where we will see continually cooler anomalies over the next 10-20 years. As we approach the end of the cycle, Florida will experience an increase in dry years compared to wet years. Given the temporal stage of the current phase, conditions will continue to remain wetter than average for the next 10-20 years, but with a slow and gradual decline in intensity until this phase ends and a cool phase begins. However, low frequency dry years can still occur due to other events such as La Niña, which can occur on an average of every 2-7 years.

Global climate change and variability, particularly at regional levels, are not completely understood. Over the last two decades, South Florida Water Management District (SFWMD) scientists have researched how natural, global climatic patterns such as the El Niño/La Niña-Southern Oscillation and the Atlantic Multidecadal Oscillation are linked to South Florida's weather and climate. Based on this expanded experience and knowledge, the SFWMD has already adopted progressive measures to incorporate climate outlook into its planning and operations. The LRWRP features will be designed robustly to handle extreme wet and dry conditions, floods and droughts, and will be operated based on the climate outlook described above. Climate change is difficult as well as controversial to predict and our LRWRP assessment is not an exercise to predict what the climate change will be, but to ensure that the plan selected is robust enough to accommodate changing climatic conditions.

The regional hydrologic models used to simulate with- and without project conditions require climatic and tidal data as boundary conditions. Given the uncertainty in future climatic conditions, the historic climate data used in the period of record is assumed to be adequate for measuring the change in conditions that are directly attributable to the alternatives. The 41 year period of record (1965-2005) includes representative dry, average, and wet years as well as tropical storms, hurricanes, and droughts. The model tidal boundary used in the regional hydrologic model was developed using historic tidal data for the Jupiter Inlet.

The LRWRP team has conducted a Climate Change Assessment. This assessment includes an evaluation of Sea Level Change (SLC) as per the Engineering Regulation (ER) 1100-2-8162: Incorporating Sea Level Change in Civil Works Programs and Engineering Technical Letter (ETL) 1100-2-1: Global Changes Procedures to Evaluate Sea Level Change – Impacts, Responses, and Adaptation. USACE sea level change projections for the period from 1992 to 2125 for Miami Beach, Florida for historic, intermediate and high rates of future sea level rise were evaluated. The regional hydrologic models used to simulate with and without project conditions require climatic and tidal data as boundary conditions. In order to evaluate the sea level rise impact on the restoration project, a curvilinear three-dimensional hydrodynamic model CH3D (Sheng, 1986), was applied to the Northwest Fork of the Loxahatchee River and the Loxahatchee River Estuary. To incorporate projected SLR, the tidal boundary condition offshore of the Jupiter Inlet was developed by combining tidal components and projected mean sea level (MSL), including SLR, at Miami Beach. Two alternatives were evaluated, the 2070 future without project (2070FWO) and the Recommended Plan at year 2070. The future with-project condition baseline was selected at year 2070 as it aligned with the hydrologic output from LECSR-NP. The impact of sea level change on project benefits is assessed in **Annex H Climate Change Assessment** for the WFWO and with project conditions. Changes in hydrologic trends is also assessed and included in **Annex H**. The Engineering and Construction Bulletin (ECB) 2018-25: Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Work Studies, Design, and Projects describes a qualitative analysis that included tendencies for both past (observed) and future (projected) changes in hydrology.

1.3.2 Hydrology

Hydrologic modeling simulations of the ECB and FWO were developed with the LECSR-NP modeling tool, to provide baseline conditions for plan formulation, the assessment of LRWRP project benefits (comparisons against FWO), and the assessment of LRWRP alternative performance for the level-of-service for flood protection and water supply (comparisons against ECB). The ECB was developed to represent the system-wide infrastructure and operations that were in place in 2014. The FWO for LRWRP assumes the construction and implementation of

currently authorized CERP and non-CERP projects, and other Federal, state or local projects constructed or approved under existing governmental authorities that occur in the LRWRP study area. Selection of the TSP is conducted based on comparisons between the LRWRP action alternatives and the FWO. The reader should refer to **Section 2** of the **LRWRP PIR main report** and **Appendix C.1 Existing and Future Without Project Conditions** for additional documentation of the ECB and FWO conditions from the environmental point of view.

The following sections will discuss the modeling strategy, screening, scope of alternatives for the final array and the modeling results for all the tools used in LRWRP.

1.3.2.1 Preliminary Screening

The Plan Formulation Screening methodology for LRWRP can be found in detail on **Section 3, Plan Formulation**, of the Main Report. For the LRWRP, no hydrologic modeling strategy was applied during screening.

1.3.2.1.1 Screening Alternatives

Through the Plan Formulation process, the PDT identified the initial screening criteria needed in order to select alternatives that could allow the project to improve the timing and distribution of water to the Northwest Fork of the Loxahatchee River.

The management measures modeled to achieve the projects goals were a combination of reservoirs, aquifer storage and recovery wells, pump stations, weirs, canal backfills, canal modifications, gated structures, and culverts. The PDT identified conceptual locations for these management measures based on information gathered during project delivery team meetings with representatives from federal and non-federal agencies such as SFWMD, FDEP, City of West Palm Beach, Palm Beach County, Martin County, Florida Department of Transportation, ITID, SIRWCD, and various other water control districts.

These management measures were ranked and discussed among members of the PDT under the assistance of the Planning Formulation team. Then these management measures were combined into the first round of alternatives to be evaluated with the LECSR-NP hydrologic modeling tool.

1.3.2.1.2 Results

For the LRWRP, no hydrologic modeling strategy was applied during screening. Therefore, no preliminary screening results were developed for this stage of the LRWRP planning project.

1.3.2.2 Evaluation of the Final Array of Alternatives

The Final Array screening criteria for LRWRP can be found in detail on **Section 4, Plan Evaluation,** of the Main Report. However this section summarizes the hydrologic modeling strategy applied during the LRWRP final array.

1.3.2.2.1 Modeling Strategy and Tools: LECSR-NP

The LRWRP modeling tool, LECSR-NP, was jointly selected by the USACE Jacksonville District (SAJ) and the South Florida Water Management District (SFWMD) in 2016 based on the capability to provide adequate hydrologic information to conduct evaluations for the needs of the LRWRP. All tools used for this project are approved for use by the USACE Engineering software validation process, as further documented in **Section A.11 Hydrologic Modeling** of the **Engineering Appendix**.

The LECsR model was developed by the SFWMD as a regional water resources planning tool for the entire southeast coast of Florida. It was refined specifically for LRWRP in order to adequately address the proposed CERP project assumptions for plan formulation and evaluation. The resulting sub-model is referred to as LECSR-North Palm Version (NP) (Obeysekera et al, 2018, Giddings and Obeysekera 2018). The LECSR-NP utilizes USGS MODFLOW (Harbaugh and McDonald 1996, Harbaugh et al., 2000) computer code with independent subroutines called modules. The modules, in turn, have been grouped into packages which deal with a particular hydrologic

process or solution algorithm. Further details can be found in the **Model Calibration Report** (Obeysekera et al., 2018) and **Model Application Report** (Giddings et al., 2018) in **Annex A-3**. While there are limitations to the model (e.g., no canal conveyance capabilities, limited ability to handle complex water management operations), it provides reasonably good estimates of water stages and flows over/through structures over a period of record. To simulate surface water processes, several additional packages were implemented in the code to allow for routing of overland flow in wetland systems and operational packages to simulate canal structure operations and weir flow equations. The LECSR-NP should not be applied for detailed engineering analyses and results should only to be used comparatively for predictions (Obeysekera et al, 2018, Giddings and Obeysekera 2018). Refer to **Section A.11** of the **Engineering Appendix** for additional documentation of the LECSR-NP model.

It is expected that higher resolution hydrologic and hydraulic modeling tools will be required to further analyze localized and possibly regional-scale effects of specific components of the LRWRP recommended plan, with the scope of these analyses further identified during the Pre-Construction Engineering and Design (PED) phase of the project.

1.3.2.2.2 Alternatives Description

Table 1 highlights the key features proposed in the TSP and the final array of project alternatives. A description of the final four alternatives is found in multiple locations, including **Section 3** of the Main PIR, **Appendix A**, and within the LECSR-NP model Application report in **Annex A-3 Modeling Documentation Report.**

Table 1. Design features proposed for the RP and final array of alternatives

Project Feature	Feature Type	Alt5R/RP	Alt 2	Alt 10	Alt 13
Flow-way 1 - G-160	Conveyance	Yes	Yes	Yes	Yes
Flow-way 1 - G-161	Conveyance	Yes	Yes	Yes	Yes
Flow-way 1 - Grassy Waters Preserve Triangle	Conveyance/Storage	Yes	Yes	Yes	Yes
Flow-way 1 - M-1 Lower Pump Station	Pump Station	Yes	--	--	--
Flow-way 1 - C-51 Phase II Reservoir	Storage	--	--	44 kac-ft[1]	--
Flow-way 1 - Force Main	Conveyance	--	--	Yes	--
Flow-way 2 - C-18W Reservoir	Storage	9.5 kac-ft	7.2 kac-ft	7.2 kac-ft	--
Flow-way 2 - Aquifer Storage & Recovery (ASR)	Storage	4 wells	2 wells	--	4 wells
Flow-way 2 - M-O Canal Connector	Conveyance/Pump	Yes	Yes	Yes	Yes
Flow-way 2 - C-18WCanal Weir Modification	Conveyance	--	--	--	Yes
Flow-way 2 - L-8 Shallow	Storage	--	4.3 kac-ft	--	6.5 kac-ft
Flow-way 2 - C-18W Natural Storage	Storage	--	--	--	Yes
Flow-way 3 - Pal-Mar East	Conveyance/Storage	Yes	Yes	--	Yes
Flow-way 3 - Thomas Pepper Farm	Conveyance	Yes	Yes	--	Yes
Flow-way 3 - Ranch Colony Canal	Conveyance/Storage	Yes	Yes	Yes	Yes
Flow-way 3 - Gulfstream West	Conveyance/Storage	Yes	Yes	--	Yes

Flow-way 3 - Gulfstream East and Moonshine	Conveyance/Storage	Yes	Yes	Yes	Yes
Flow-way 3 - Kitching Creek	Conveyance/Storage	Yes	Yes	Yes	Yes
Mack Dairy Spreader Swale	Conveyance	Yes	--	--	Yes

1.3.2.2.3 Results: LECSR-NP

Modeling has been a central aspect of project planning for the Loxahatchee River Watershed Restoration Project. The objective was to demonstrate the extent to which each of the PDT proposed alternatives met specific ecological measures and performed relative to the project area's base condition. Modeling results increased the PDT's understanding of the existing hydrologic system and the range of expected responses of proposed project features. The final array of alternatives was evaluated for its effects on the environment. The alternatives were compared on how each alternative would perform with respect to each of the planning objectives (see **Section 1** of the Main PIR).

The LECSR-NP modeling output was analyzed using multiple types of performance measure graphics at key locations within the model domain. See **Appendix A** for a more detailed description of the LECSR-NP model output analysis.

1.3.2.2.3.1 Flow-way 1

The ECB and FWO condition assume that G-160 and G-161 are not constructed. All alternatives propose the G-160 and G-161 structures. Stage-Duration curves and structures flows results were analyzed to compare the FWO condition to the with-project alternatives. The G-160 structure aids in both the management of water levels within the C-18 Canals and the timing of water deliveries to Northwest fork of the Loxahatchee River during the wet and dry seasons. G-161 provides connectivity to the Flow-way 1 features, including the L-8 Basin, GWP, and C-18 Canal to the Northwest Fork of the Loxahatchee River. Compared to the 2070 FWO condition, the G-161 structure helps improve the hydrology over a longer duration during the simulation period, potentially leading to improved vegetation communities and additional freshwater availability to the Loxahatchee River.

Upstream of the forcemain, For Alternative 10, the C-51 Phase II Reservoir and force main within GWP are proposed features in Flow-way 1 for the purpose of supplying water to GWP and the upstream portion of the C-18 Canal. The force main allows a direct connection between the M-Canal (within GWP) and the C-18 Canal, without requiring an increase in stage or adversely affecting water quality within GWP. The benefits of the C-51 Reservoir and force main are realized as increased flow rates through the G-160 structure and flows to Lainhart Dam

The M-1 Pump station, proposed only in the TSP, was designed to bring extra water into the M-Canal from the M-1 Lower Basin to augment GWP when G-161 is operating to reduce reliance upon the regional system.

1.3.2.2.3.2 Flow-way 2

Flow-way 2 project features contained various configuration of reservoir(s), with varying storage volume capacity. Alternative 2 and 10 contained a 7,200 acre-ft reservoir on the proposed C-18W Reservoir site whereas the TSP proposed a 9,500 acre-ft reservoir. Alternative 13 proposed a natural storage feature instead of a reservoir component. Alternative 2 and 13 also contained the L-8 Basin shallow storage feature, with a capacity of 4,300 acre-ft and 5,400 acre-ft, respectively.

The construction of the C-18W Reservoir allows excess runoff from the C-18W basin, J.W. Corbett WMA, and inflows from the ITID upper basin via the M-O Canal to be collected and discharged when the Loxahatchee River requires additional water to achieve restoration targets. Additionally, the L-8 Shallow Storage feature was

designed as an additional means for water storage to supplement flows to the C-18W Reservoir or natural storage, depending on the alternative. The improved timing and flow quantities to the Loxahatchee River from the C-18W Reservoir can be seen from the C-18W weir average monthly flow rates, as shown in Figure 3. It can be noted that from the performance of Alternative 13, the natural storage does not perform as well at the C-18 Weir as excess flows cannot be captured and dispersed in an improved hydrologic manner without a storage feature. The C-18 Weir is located at the downstream extent of flow-way 2 and therefore captures the combined performance of all proposed features within the flow-way. The TSP (Alternative 5R) performance illustrates a discernable improvement over the FWO conditions for both the wet and dry season. Additionally, improvement over the other alternatives during the dry season is noted.

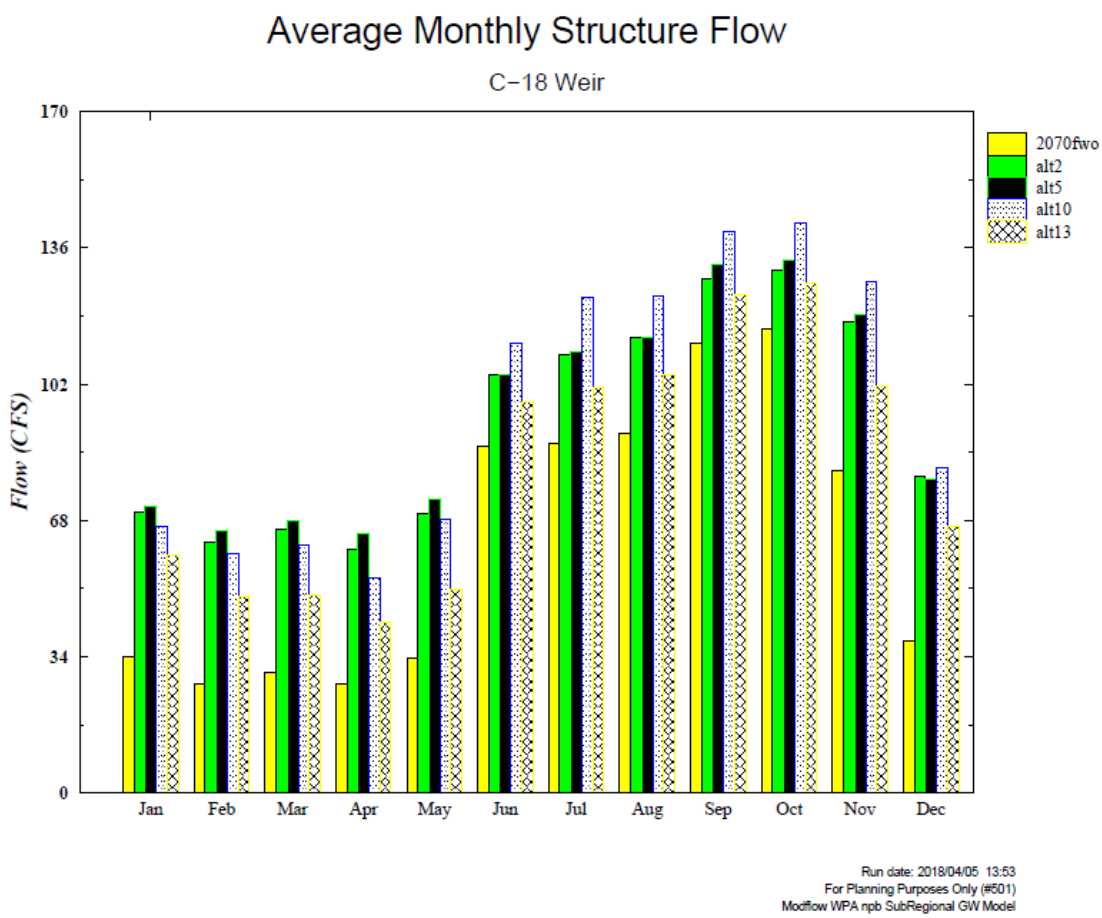

Figure 4. Average Monthly Structure Flow for C-18W Weir

1.3.2.2.3.3 Flow-way 3

The improvements within the Pal-Mar East area were proposed in Alternative 2, 5R (TSP), and 13. The improvements in Pal-Mar East were designed to improve the hydroperiod, mainly within the Nine Gems, by reducing the run-off capacity while improving the hydrology through re-connection of the Pal-Mar East parcels. Additional improvements within Pal-Mar East, Culpepper property, are realized through the addition of telemetry operated inlet controls (all alternatives), resulting in improved hydroperiod and storage within the Nine-Gems watershed. The proposed Cypress Creek Canal structure in all alternatives also achieved the desired project benefits by improving the surrounding groundwater (and surface water) elevations for a greater duration during

the simulation period. Further improvements to the water surface elevations within Cypress Creek Natural Area were realized with the addition of the Spreader system proposed in the TSP and Alternative 13.

The proposed flow through marsh within the Gulfstream West property in Alternatives 2, 13, and the TSP was designed to transform the existing overly drained agricultural land to marsh or wetland habitat, thereby increasing the natural extent of wetlands. The feature provided a drastic improvement in the hydroperiod for the Gulfstream West property as well reduced peak flows in the wet season and provided additional flow to the Cypress Creek Canal during drier periods.

The earthwork modifications to Gulfstream East, including the hydrologic connection to Moonshine Creek, demonstrate that the proposed improvements aid in restoring a more natural hydroperiod in the area. Similarly, the modifications on Jenkins Ditch in Kitching Creek demonstrates that a greater water surface elevation can be achieved correlating to improved floodplain inundation and hydrology regimes, aiding the restoration of wetland habitat.

1.3.3 Regional Water Management Operations

The main goal of the operational strategies is to complement LRWRP proposed management measures in order to meet the project goals, benefits and objectives. Goals include ecosystem restoration, preservation, and protection of the natural ecosystem, while providing for the other water related needs of the region, as well as meeting the requirements for protection of health and public safety. These goals, purposes, and benefits will not be fully realized until the completion of the construction and implementation of the LRWRP components. The components will be phased in as they become operational. The interim operations have not yet been developed. Reference **Annex C LRWRP Draft Project Operating Manual (DPOM)**.

It is important to understand that the DPOM will develop over time as the details of the detailed design of LRWRP components are developed and finalized. **Annex C Draft DPOM** presents the first draft with the recognition that multiple revisions will occur over the life of this project based on operational and other project updates.

As a general operational strategy, the LRWRP system will be operated to direct excess water into the storage features during the wet season and provide water during the dry season.

1.3.4 Flood Control

The LECSR-NP model results indicate that implementation of the TSP will not reduce the levels of service for flood protection within the areas affected by the project. Additionally, storm specific HEC-RAS modeling was performed to inform the design and operations of key Flow-way 3 and Flow-way 2 project features. Please refer to **Annex B** and **Annex A-3** for additional information regarding modeling results and associated level of service for flood protection analysis.

1.4 Recommended Plan

The Recommended Plan is similar to the Tentatively Selected Plan, Alternative 5R. Minor project feature refinements occurred after the selection of the TSP based on further analysis of the LECSR-NP model as well as the results of the HEC-RAS model. These refinements were minor and are not expected to affect the performance of the plan in terms of meeting project objectives.

1.4.1 Alternative Description

The purposes of LRWRP are to restore and sustain the flow of freshwater to the federally designated “National Wild and Scenic” NWFLR, and to provide flows to enhance hydroperiods in the Loxahatchee Slough; increase

connectivity of hydrology, flora, and fauna between natural areas; improve seasonal timing and distribution of water to improve hydropatterns in over-drained wetland areas throughout the historic headwaters for the river.

The LRWRP Recommended Plan, Alternative 5R, includes a surface storage reservoir, aquifer storage and recovery wells, flow-through marsh feature, multiple pump stations, and other structures to capture and store water that is currently lost to tide, and redistributes it to increase volume and improve timing of water deliveries to the NWFLR. The Recommended Plan will achieve 91% of the dry season restoration target flows and 98% of the wet season restoration target flows to the NWFLR measured at Lainhart Dam. These targets are based on the LECSR-NP modeling output, with results shown in **Annex A-3, LECSR-NP Model Documentation.**

The Recommended Plan also includes new pump stations and canals, ditch plugs, new and modified culverts and weirs, and other management measures to rehydrate over-drained wetlands in the watershed. These measures will restore more natural water deliveries to the Loxahatchee River, promote improved health and functionality of wetland and upland areas, restore hydrologic and ecological connectivity among natural areas, and provide increased quantity and quality of habitat available for wildlife and native vegetation. The TSP will restore 17,000 acres of former wetlands that had been converted to agriculture and an additional 10,000 acres of existing disturbed wetlands in the J.W. Corbett Wildlife Management Area (WMA), Loxahatchee Slough, Cypress Creek Natural Area, and Kitching Creek. These restored wetlands will connect with 51,000 acres of other wetland communities in the area, resulting in 78,000 acres of connected habitat.

The number and size of the features in each project element were optimized and presented in the Final Engineering Appendix. The shape and size of all features is subject to change during further analysis in Pre-Construction Engineering and Design phase.

1.4.2 Detailed Modeling: HEC-RAS

A total of four individual HEC-RAS models, each for a particular sub-watershed within the LRWRP area, were developed in support of the conceptual design for the Recommended Plan. A description and the results of these hydraulic models can be found in **Appendix A Engineering Appendix, Annex A-1 Hydraulic Design,** and **Annex A-3, HEC-RAS Model Documentation Report**. The boundary conditions used for each model were either extracted from the LECSR-NP model or based on a combination of storm event modeling and maximum permitted discharges.

The model simulations were used to ensure that the proposed project features within the Recommended Plan were appropriately sized and the design did not cause adverse impacts to the existing level of flood protection.

The HEC-RAS model accomplished the following:

- The HEC-RAS model simulated the topographic, drainage, and hydraulic processes that occur at a scale less than can be captured in a 704 by 704 ft cell used in the LECSR-NP model. These processes, such as flow velocity and water stages, were therefore captured in the evaluation with a greater level of accuracy.
- HEC-RAS was used as necessary to address hydraulic design considerations of potential canal conveyance and structural modifications of the Recommended Plan. HEC-RAS can be applied as a hydraulic design tool to aid with design of new gravity water control structures and pump stations. Additionally, HEC-RAS was applied to determine stages in the project area for the Recommended Plan versus the existing conditions.

- The HEC-RAS model was able to capture the local variability in hydraulic properties and operations at a resolution that is able to more accurately assess project impacts.

1.4.3 Saving Clause: Flood Control

The following sections summarize the programmatic regulations and preliminary analysis for the LRWRP.

1.4.3.1 Programmatic Regulation (33 CFR Part 385)

Section 385.37 of the CERP Programmatic Regulations requires that PIRs include an analysis of the project's impacts on levels of service for flood protection that existed on the date of enactment of WRDA 2000 (December 2000) and are in accordance with applicable law to demonstrate that the levels of service for flood protection will not be reduced by implementation of the project. Where appropriate and consistent with restoration of the natural system, opportunities to provide additional flood protection shall be considered. The conditions that existed on the date of enactment of WRDA 2000 are included in the Pre-CERP Baseline.

1.4.3.2 Analysis per Project Feature Area

Flood protection is evaluated by a combination of best professional judgment interpreting model results and engineering analyses. Consistent with the Draft Guidance Memoranda, the same models and results used for plan formation were applied for the LRWRP Savings Clause assessment of the current preliminary TSP. This varies from typical storm event analyses by using a long period of record simulation and focusing on the wet events included within the 1965–2005 simulation period.

The three areas that were preliminary analyzed are those near main management measures and that could experience increase in ponding. These include: 1) Flow-way 3 Western project features including Cypress Creek Canal, 2.) Flow-way 3, Kitching Creek and 3) C-18W Reservoir.

The results of the HEC-RAS modeling and resulting optimized design for the project features is included in **Annex A-1 Hydraulic Design** and **Annex A-3, HEC-RAS Model Documentation Report.**

1.4.3.3 Conclusion

The implementation of the Recommended Plan will not degrade the existing level of flood protection offered by various components of the C&SF Project for this area. Further, the Recommended Plan will ensure flood protection of the area through engineering design and construction following state of the practice methods for design and construction of pertinent features of the plan. U.S. Army Corps of Engineers, Engineering Regulation(s) 1110-2-1150 (Engineering and Design for Civil Works Projects) 1110-2-1156 (Engineering and Design Safety of Dams – Policy and Procedures) along with various other site/structure specific regulations will be adhered to prior to and during the PED phase.

ANNEX A-3

Loxahatchee River Watershed Restoration Project

HEC-RAS Model Documentation Report

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ACRONYM LIST

ALARP	As Low As Reasonably Practicable
ASR	Aquifer Storage Recovery
C&SF	Central & South Florida
CERP	Comprehensive Everglades Restoration Plan
DCM	Design Criteria Memorandum
EIS	Environmental Impact Statement
ER	Engineering Regulation
FDEP	Florida Department of Environmental Protection
LiDAR	Light Detection and Ranging
LRWRP	Loxahatchee River Watershed Restoration Project
NAD83	North American Datum 1983
NAVD88	North American Vertical Datum 1988
NGVD29	National Geodetic Vertical Datum 1929
NOAA-NGS	National Oceanic and Atmospheric Administration – National Geodetic Survey
PED	Pre-construction Engineering Design
PFMA	Potential Failure Mode Analysis
PIR	Project Implementation Report
QRA	Qualitative Risk Assessment
USGS	US Geological Survey

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A OVERVIEW

The Loxahatchee River Watershed Restoration Project (LRWRP) is a planning effort undertaken by the United States Army Corps of Engineers (USACE) with the South Florida Water Management District (SFWMD) as the local sponsor. Previous modeling efforts for the LRWRP include the Lower East Coast Sub-Regional Model – North Palm Version (LECSR-NP) (Obeysekera, et al., 2018), which supported the development of alternative modeling comparisons for the plan formulation and the evaluation phase of the LRWRP. Following the selection of the Recommended Plan, it was determined that additional analysis would be required to appropriately refine the project features to evaluate the Project Assurances, including flood protection, and to provide more refined design for Cost Certification purposes. The Hydrologic Engineering Center River Analysis System (HEC-RAS) software was developed by the U.S. Army Corps of Engineers and is designed to perform one-dimensional and two-dimensional hydraulic calculations for a full network of natural and constructed channels.

HEC-RAS models were constructed for certain areas within the LRWRP and assisted in accomplishing the following:

- Refine the topographic, drainage, and hydraulic processes that occur at a scale less than can be captured in a 704 by 704 ft cell used in the LECSR-NP model. These processes, such as flow velocity and water stages, were therefore captured at a greater level of accuracy within HEC-RAS than LECSR-NP.
- HEC-RAS was necessary to address hydraulic design considerations of potential canal conveyance and structural design refinements for preliminary features proposed as part of the Recommended Plan. HEC-RAS can be applied as a hydraulic design tool to aid with design of gravity water control structures, pump stations, or others. Additionally, HEC-RAS can be applied to determine stages in the project area for the Recommended Plan versus existing conditions.
- The HEC-RAS model aided in capturing local variability in hydraulic properties and operations at a resolution that is able to more accurately assess project impacts.

Modeling support to the LRWRP effort was provided by a team comprised of modelers from the Water Resources Branch of the Jacksonville District, USACE. The purpose of this report is to document the modeling process, assumptions, and outcomes for the refinement of project features proposed as part of the LRWRP Recommended Plan.

A.1 Study Area

The project area includes undeveloped wetlands and former agricultural lands north and west of the Loxahatchee Slough and northwest fork of Loxahatchee River, in Martin and Palm Beach Counties, Florida. Cypress Creek, Moonshine Creek, and Kitching Creek are significant tributaries of the Loxahatchee River and are part of this restoration project. The project area surrounds developed areas in Martin and Palm Beach Counties and is approximately 480,000 acres (753 square miles). The study area is bounded on the north by the C-44 Canal, on the south by the C-51 Canal, on the west by the L-8 Canals and Lake Okeechobee, and on the east by the Loxahatchee River Estuary and Lake Worth Lagoon. The Loxahatchee River discharges ultimately into the Atlantic Ocean near the town of Jupiter, Florida. The Northwest Fork of the Loxahatchee River, one of Florida's two federally designated National Wild and Scenic Rivers, is a natural river channel that originates in the Loxahatchee and Hungryland Sloughs. Downstream from these sloughs, the Northwest Fork receives additional input from the other major tributaries of the Loxahatchee River: Cypress Creek/Ranch Colony Canal, Hobe Grove Ditch, and Kitching Creek. The purpose of LRWRP

is to restore and sustain the overall quantity, quality, timing, and distribution of freshwaters to the federally designated "National Wild and Scenic" Northwest Fork of the Loxahatchee River for current and future generations. This project also seeks to restore, sustain, and reconnect the area's wetlands and watersheds that form the historic headwaters for the river.

There are three main Flow-ways within the LRWRP area and are referenced often throughout this report. Flow-ways are locations in the study area based on existing natural areas, topography, and associated canals, and are generally separated from each other by developed lands. Each flow-way represents a pathway for water to flow from the watershed to the Loxahatchee River. See **Figure 1** for the general locations of the three Flow-ways. Flow-way 1 is located in the southernmost portion of the project area, bounded by L-8 Canal and M-Canal in the south and the G-92 spillway in the north. This flow-way uses the M-Canal and C-18 Canal to route water from upstream project area basins to the Loxahatchee River. Flow-way 2 is located in the central portion of the project area. Its primary canal conveyances are the M-O Canal and C-18W Canal. The watersheds that contribute to these canals include the Indian Trail Improvement District (ITID) basin via the M-O Canal, J.W. Corbett Wildlife Management Area (WMA), Hungryland Slough, "Mecca" property (area of proposed C-18W Reservoir), and natural area east of the North Palm Beach County Airport via the C-18W Canal. Flow-way 3 is located within southern Martin County and consists of the watersheds within the northern portion of the project study area. These watersheds contribute to the northwest fork of the Loxahatchee River via the Ranch Colony Canal and Cypress Creek, Moonshine Creek, Hobe Grove Ditch, and Kitching Creek tributaries.

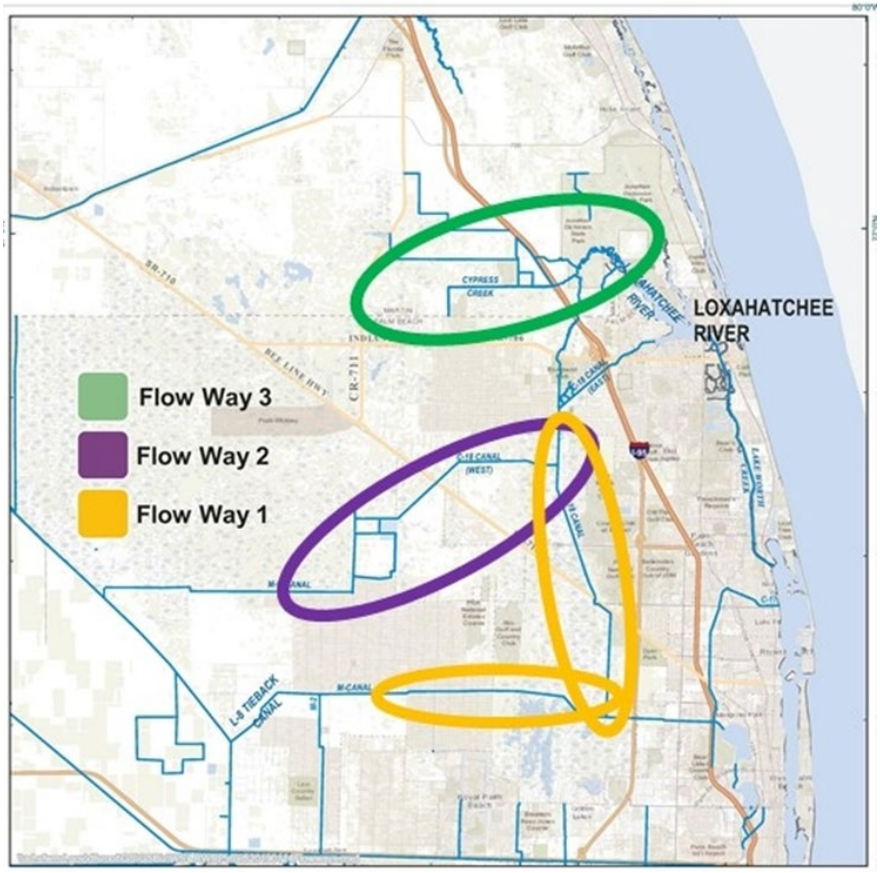

Figure 1. General Location of Each Flow-way

A.2 Model Scope

Model boundaries and extents were identified based on a common contributing area or downstream computation point. Therefore, four separate HEC-RAS models were created: three models in Flow-way 3 and one model in Flow-way 2. All project features within Flow-way 2 ultimately discharge to and from the C-18W Reservoir and therefore were required to be modeled as one system. Similarly, all features within the western portion of Flow-way 3 drain to the Cypress Creek Canal (under I-95) and were also modeled collectively. Two additional models were constructed for the eastern portion of Flow-way 3, one for the Gulfstream East and one for Kitching Creek.

A.2.1 Model Baseline Conditions

The following modeling scenarios were run:

- 2014 Existing Condition Baseline
- LRWRP Proposed Recommended Plan, Alternative 5R

A.2.2 Model Limitations

Model simulations were performed to aid in the design refinement of features. HEC-RAS has extensive capabilities to model hydraulic processes, but is limited when performing hydrologic processes. Therefore, the use of the HEC-RAS models described in this report should be limited to design storms, or event based simulations. If longer-term or continuous simulation modeling is needed, additional efforts will be necessary to capture the hydrologic cycle, or water budget, in a separate modeling tool such as the Hydrologic Modeling System (HEC-HMS).

B HYDRAULIC MODEL DESCRIPTION AND DEVELOPMENT

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) 1-D/2-D, Version 5.0.5 (Flow-way 3) and 5.0.7 (Flow-way 2) modeling software was used for analysis and mapping. HEC-RAS 2-D flow modeling advantage and capabilities include:

1. The ability to perform 1-D, 2-D only, or combined 1-D and 2-D modeling.
2. The 2-D equation solver uses an Implicit Finite Volume algorithm.
3. Can solve either 2-D Diffusion Wave or 2-D Full Saint Venant Equations.
4. The 1-D and 2-D solution algorithms are tightly coupled on a time step by time step basis (or even iteration by iteration).
5. The software was designed to use Unstructured and Structured Computational Meshes. The outer boundary is defined with a multi-point polygon.
6. The underlying terrain and the computational mesh are pre-processed in order to develop detailed Hydraulic Property Table of the Cells and the Cell Faces.
7. Mapping of the 1-D/2-D inundation area and animations of the flooding can be done inside of RAS using RAS-Mapper.
8. The 2-D flow computations take advantage of multi-processors.
9. 64 Bit and 32 Bit Computation Engines are used.

The HEC-RAS model combines 1-D and 2-D unsteady-flow hydrodynamic routing. 2-D unsteady-flow routing used the Diffusion Wave equations. 2-D flow modeling is accomplished by adding 2-D flow area elements into the model similar to adding a storage area. A structured 2-D computational mesh of nominal

cell size is first generated. Break lines for levees, roads and other features are then used to further define the mesh. Boundary cells vary in shape and size to follow the detailed polygon boundary. Interior cells can also vary in shape and size, as for the cells around the break lines. The computational cells can be triangles, rectangles, or elements with up to eight sides.

The 2-D computation mesh is preprocessed into an elevation–volume curve for each cell, and a series of hydraulic property curves for each cell face (elevation vs. wetted perimeter, area, and roughness). These relationships are derived from the details of the underlying terrain used for the model. Each grid cell face is like a detailed cross section so the flow of water into, through, and out of a cell is controlled by the details of these face properties and the cell elevation-volume relationship. The benefit of this approach is increased hydraulic details at the cell level instead of models that use a single elevation for each cell and face. With HEC-RAS, users can have much larger cells, but still retain significant hydraulic detail within a cell. HEC-RAS cells can be partially wet so water does not have to cover the entire cell and can move through a portion of the cell.

B.1 Data Compilation

No new or additional data was collected as a part of the modeling effort for the Project Implementation Report (PIR). Data was compiled from existing sources including Martin and Palm Beach Counties, South Florida Water Management District, scientific agencies (i.e. USGS, NOAA, etc.), and others. It is noted that there was a lack of sufficient data/information for much of the project area. Analysis of available information and best professional judgment were used to make informed modeling assumptions. It is recommended that confirmation of relatively old or missing data with new survey data is completed during the Preconstruction Engineering and Design (PED) phase and models are updated as necessary.

B.1.1 Digital Terrain Data

Surface elevation used in the Flow-way 2 (C-18W Reservoir) model was obtained from high-accuracy Light Detention and ranging (LiDAR) for the Palm Beach County, Florida. The LiDAR dataset covered an area of approximately 1,994 mi^2 with a grid size of 4x4 ft. Metadata for the LiDAR data indicated that the aerial acquisition survey was performed from December 2016 to March 2017. The mean non-vegetated vertical accuracy (NVA) is 0.07 ft and vegetated vertical accuracy (VVA) is 0.20 ft.

Surface elevation for Flow-way 3 was obtained from LiDAR for Martin County, Florida. The dataset covered approximately 566 mi^2 and was collected in January 2016. The mean NVA was 0.26 ft and VVA was 0.64 ft.

HEC-RAS has the ability to modify terrain based on specific geometry files. Geometry files that represent the modeled Recommended Plan were created and used to modify the terrain of the Loxahatchee area to include the new channel improvement features and levees.

B.1.1.1 Coordinate Reference System

The horizontal datum used in the model development is the North American Datum (NAD) of 1986 with the 2011 adjustment (NAD 83 (2011)). The coordinate system used is Florida State Plane. Vertical datum corresponds to North American vertical Datum of 1988 (NAVD88). All the units were in U.S. ft.

B.1.2 Aerial Images

The modeling team used the map layer digital photos in ArcGIS and Google Earth to locate and verify important physical features such as roads and levees and other geographic features (landmarks identified in the field, structures, etc.).

B.1.3 Land Use Data

The United States Geologic Survey (USGS) 2011 National Land Cover Dataset (NLCD) for Martin and Palm Beach County, Florida were used as a base map for determining land use types. It was concluded that the 2011 NLCD resolution was sufficient to capture the desired detail within the project area for the planning-level HEC-RAS model creation.

B.1.4 Precipitation Data

The 24-hour and 72-hour basin average rainfall was developed from NOAA Atlas 14 Precipitation Frequency Estimates for the 100-year storm event. The Atlas 14 rainfall data can either be downloaded as a singular point or as a spatially varying dataset (raster coverage). Due to the size of the model domain and the HEC-RAS requirement of one precipitation input per 2D area, the spatially varying datasets were used to compute basin average rainfall totals for each model. These calculations were performed externally to HEC-RAS, through ArcGIS Spatial Analyst tool. Two different basin average rainfall totals were created, one for each Flow-way. The PMP was developed according to guidelines from the Hydrometeorological Report (HMR) No. 51 (Probable Maximum Precipitation Estimates, Eastern U.S.) and HMR No. 52 (Probable Maximum Storm Computation). A value of 55.7 inches was selected from the all season 72 hr, 10 mi^2 PMP chart.

B.1.5 Stream flow Data

Limited stream flow data is available within the model domains. Daily stage and flow measurements are available in Flow-way 3 from USGS station # 265818080111900 Cypress Creek Canal Below Gulfstream Bridge, FL. The period of record was from October 2002 to September 2004. The gage records are noted to be poor, with buildup of debris occurring at the bridge piling and the downstream structure. Flow-way 2 does not contain any stream flow data in the immediate vicinity of the proposed C-18W Reservoir but extensive flow and stage records are available downstream at the C-18W weir.

C MODEL DEVELOPMENT

C.1 Flow-way 2, C-18W Reservoir and Associated Features

The C-18W reservoir is the main component of Flow-way 2. The reservoir is designed as an above ground impoundment with a normal pool volume of 9,500 acre-ft, and a normal depth of 7.5 ft. Following Design Criteria Memorandum (DCM)-1, the C-18W reservoir was classified as a high hazard impoundment due to its location near residential communities. The hazard classification dictated the controlling conditions, design criteria and recommendations that were followed in the modeling effort described below.

Modeling developed for Flow-way 2 includes several existing and proposed hydraulic features, as shown in **Figure 2**. The C-18W reservoir is surrounded by the proposed embankment D-101 and seepage collection canals, C-101W and C-101E. Two outlet structures provide discharge from the C-18W reservoir. A gated culvert structure, S-102, provides discharges during normal operations. An uncontrolled spillway

structure, S-101B, provides outflow for permitted discharge to comply with the SFWMD's regulations, and also provides discharge during large flood events. The discharges are conveyed by the proposed canal, C-101N, which drains into the existing C-18W canal, located north of the reservoir.

Inflow to the reservoir is from rainfall runoff from various watersheds including ITID, J.W. Corbett Wildlife Area, and the C-18W Basin. During normal operations, water is pumped into the reservoir from the seepage collection canal using the proposed western pump station S-101A and also from S-106, which is located within C-101N.

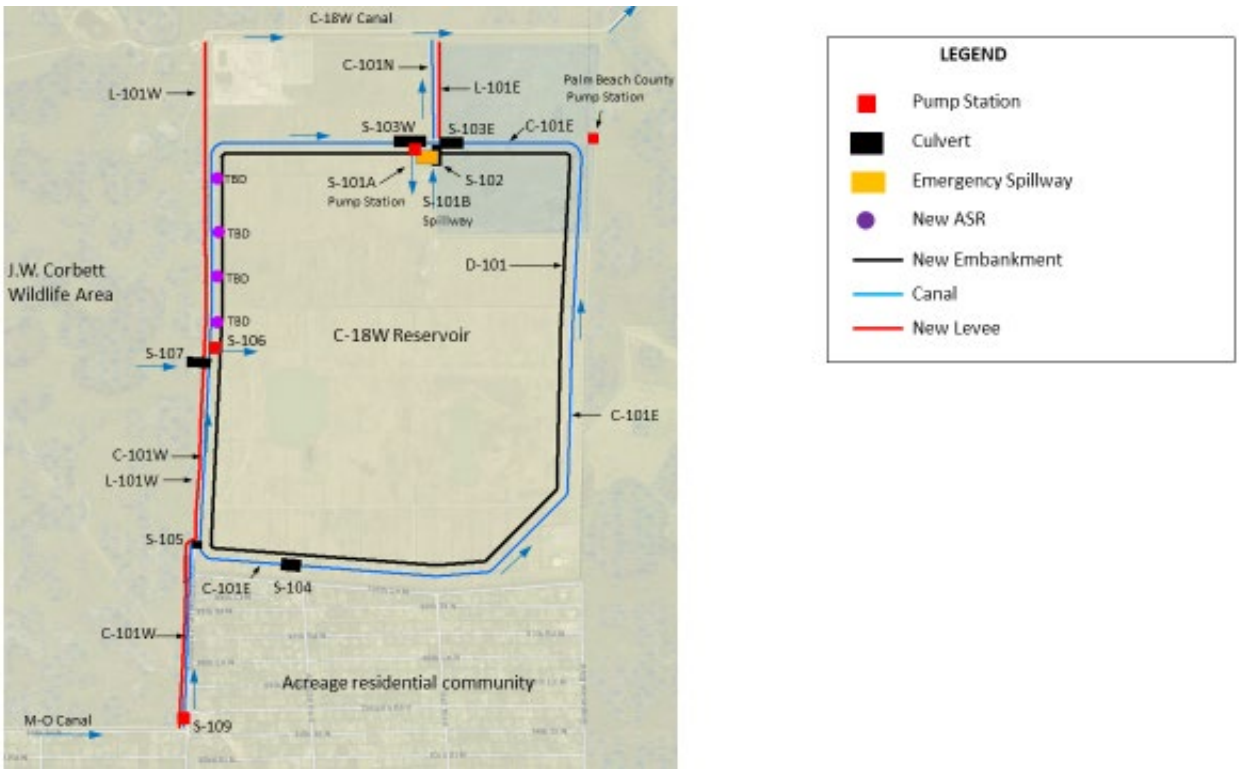

Figure 2. Flow-way 2 Proposed Hydraulic Features

C.1.1 Model Domain

The model domain extension was selected to include areas that could be influenced by the proposed features in Flow-way 2. **Figure 3** shows the model domain limits included in the HEC-RAS model. The model boundaries include State Road 710 to the north, Indian Trail Improvement District (ITID) residential areas to the south, the watershed divide for the J.W Corbett WMA and levee L-8 West to the west and the Sweetbay Natural area to the east. Surface water within the model domain drains from north to south and west to east. The model includes the existing C-18W canal, draining northeast.

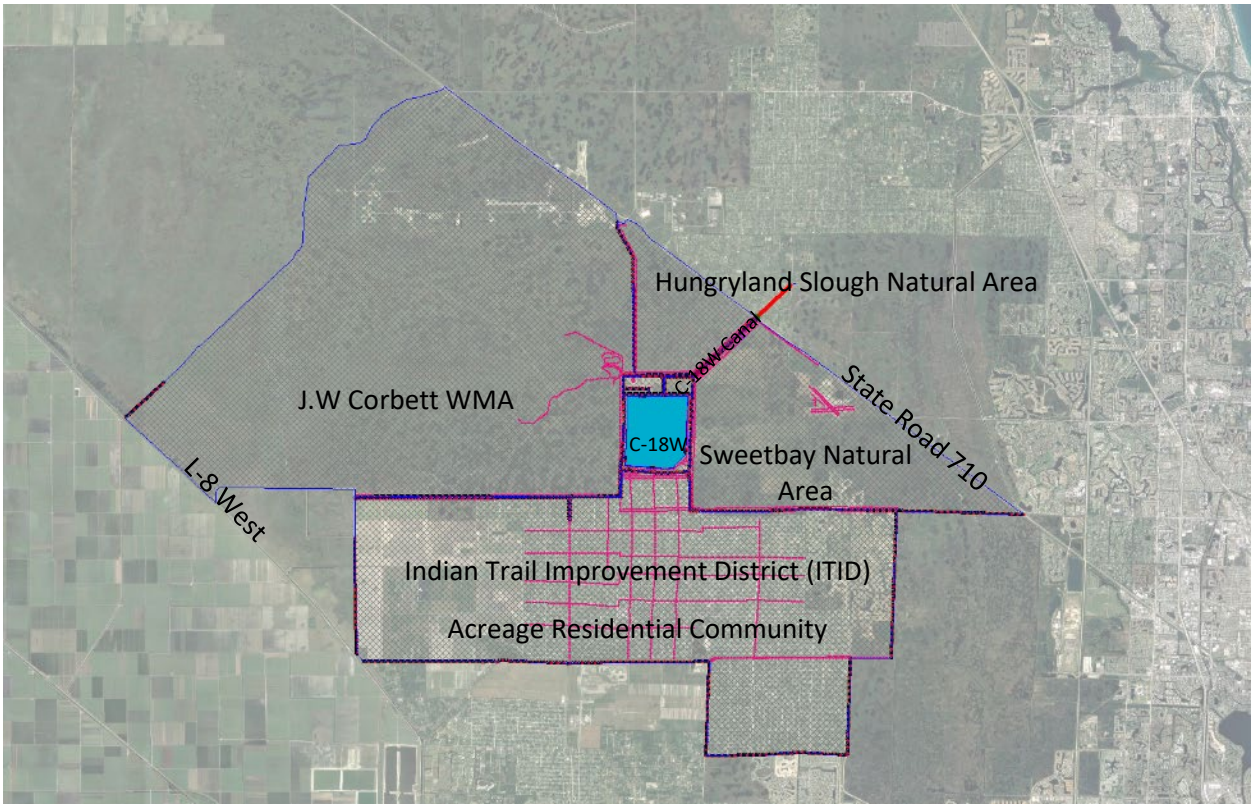

Figure 3. HEC-RAS Model Domain for Flow-way 2

C.1.1.1 Model Mesh

The model mesh was developed from a terrain model developed from high-accuracy Light Detection and ranging (LiDAR) for the Palm Beach County, Florida. The proposed project features, including the reservoir embankment, canals and levees were added to the terrain post-processing the design and LiDAR data in ArcGIS.

The proposed C-18W reservoir was included in the model as a 1-D storage area feature. Areas surrounding the reservoir were simulated as 2-D areas in the HEC-RAS model. Initial cell size of 50x50 ft was used for the 2-D areas including the seepage collection canal and an undeveloped area located north of the reservoir. The J.W Corbett WMA and the area including the Sweetbay natural area, composed mostly by wetlands, were simulated with a cell size of 200x200 ft. Residential areas including the acreage residential community as part of the Indian Trail Improvement District (ITID) and the existing gun range were simulated using a cell size of 100x100 ft. A larger size cell of 500x500 ft was used for the Hungryland Slough natural area. **Figure 4** shows details for the 2-D areas included in the model for the surrounding C-18W reservoir.

Breaklines to define existing roads and canals were added to provide more topographic definition in some areas. The 2-D areas were hydraulically connected using the 2-D areas connection tool in HEC-RAS. Proposed culverts, the reservoir spillway, the embankments and levees were added as part of the 2-D area connection.

The existing C-18W canal located north of the project and the existing M-O canal located southwest of the C-18W reservoir, were included in the model as 1-D reaches.

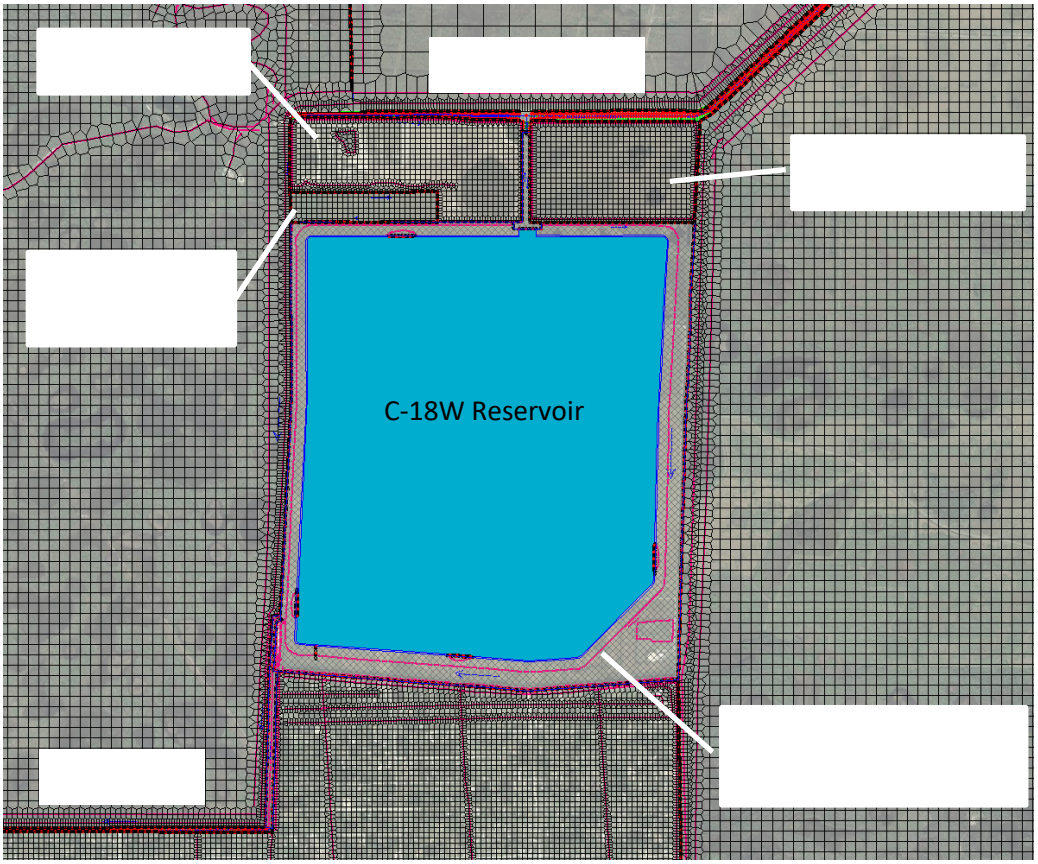

Figure 4. HEC-RAS Model Mesh for Flow-way 2

C.1.2 Boundary Conditions

The HEC-RAS model was developed for unsteady flow conditions. Different boundary conditions were used in the model. Direct rainfall for the 100-year event for a 72 hour duration, totaling 16.3 inches, was applied to the 2-D areas and the C-18W reservoir. A synthetic hyetograph with no volume removed as initial abstraction was used as input in the model for the 2-D areas, ensuring a conservative approach. Stage data using the maximum stage on record for the C-18W canal at the C-18 Weir was used as boundary condition for the 1-D reach.

No data was available on the model perimeter domain, outflow boundary conditions using normal depth were used in the project perimeter. A table of the boundary condition type and associated location within the model is included in **Table 1.**

Table 1. C-18W HEC-RAS Boundary Conditions for 100-year, 72-hour Design Storm

Select Location in table then select Boundary Condition Type				
	River	**Reach**	**RS**	**Boundary Condition**
1	canal_C-18W_ds	C-18W_ds	698	Stage Hydrograph
2	MO	MO Canal	5688	Stage Hydrograph
3	MO	MO Canal	5675	Stage Hydrograph
Storage/2D Flow Areas				**Boundary Condition**
1	Airport BCLine: northlake_blvd			Normal Depth
2	Airport BCLine:			Precipitation
3	C-101N BCLine:			Precipitation
4	C-101W BCLine:			Precipitation
5	C-101W_&_C101E BCLine:			Precipitation
6	C-18W BCLine:			Precipitation
7	Corbette			Precipitation
8	Gun Range			Precipitation
9	Hungryland Sloug			Precipitation
10	ITID BCLine: M-Canal			Normal Depth
11	ITID BCLine: 40th_st_N			Normal Depth
12	ITID BCLine: 140th_Ave_N			Normal Depth
13	ITID BCLine: west_boundary			Normal Depth
14	ITID BCLine: southwest			Normal Depth
15	ITID BCLine: north_state_rd			Normal Depth
16	ITID BCLine: 110th_Ave_N			Normal Depth
17	ITID			Precipitation
18	MO Canal			Precipitation
19	SFWMD undevelope			Precipitation
20	wetland_north			Precipitation
SA/2D Area Conns				**Boundary Condition**
1	L-101W & S-107			T.S. Gate Openings

C.1.3 Model Parameters

C.1.3.1 Channel Cross-Section and Profile

Surface elevation used in the Flow-way 2 model was obtained from high-accuracy Light Detention and ranging (LiDAR) for the Palm Beach County, Florida. Cross section data for the existing C-18W canal, the M-O Canal, and bridge data was gathered from previous modeling effort performed by an A/E contractor (Arcadis, 2016).

C.1.3.2 Canals

The hydraulic model includes the proposed seepage collection canals C-101E and C-101W, and the inflow/discharge canal, C-101N. Existing canals include the M-O canal, located southwest of the C-18W reservoir, and the C-18W canal located north.

The proposed C-101W canal was designed to convey pumped water from the M-O canal during normal operations, as well as collect seepage from then C-18W reservoir at the west side. The C-101W canal is hydraulically connected to the east seepage collection canal, C-101E. Both structures can discharge into the proposed C-101N canal. The canal flows from south to north and eventually drains into the existing C-18W canal, which drains west to east. Existing C-18W canal and part of the M-O canal were included in HEC-RAS as 1D reaches, all the other canals are part of the 2-D mesh.

C.1.3.3 Manning's "n" Values

Surface roughness coefficients (Manning's n value) were estimated using the National Land Cover Database (NLCD) 2011 for the state of Florida. **Figure 5** shows the NLCD land use coverage. Aerial images of the project area shows no significant changes in land cover from 2011 to 2017, therefore it was assumed that the 2011 land coverage is a good representation of the project conditions. The Manning's n values were changed in the proposed seepage collection canal to represent the proposed condition. A representative value of 0.035 was used. **Table 2** shows the Manning's n values used for each land cover.

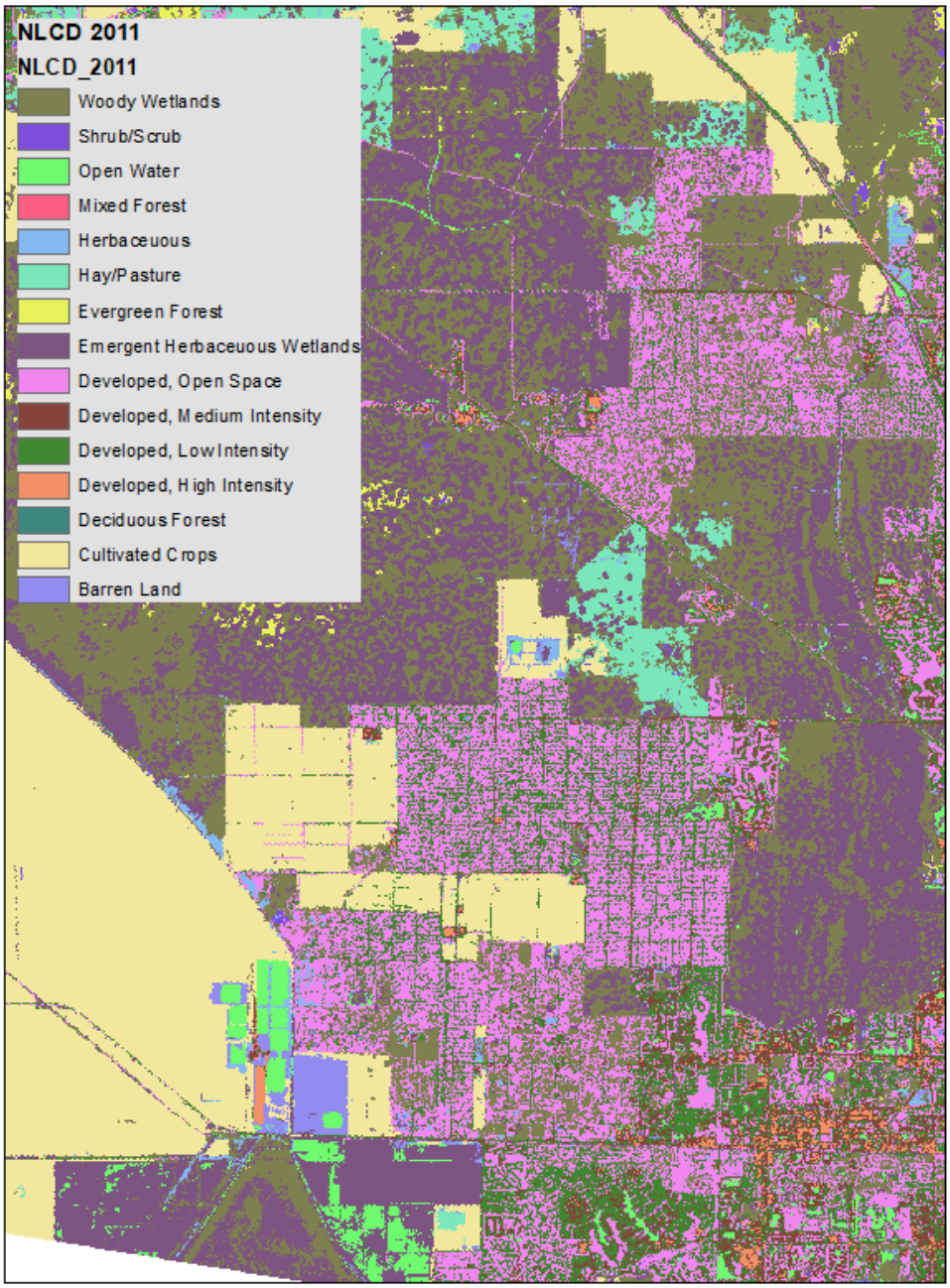

Figure 5. National Land Cover Database (NLCD) Coverage for Flow-way 2 (2011)

Table 2. Land Covers Manning's n Coefficient

Land Cover	Manning's n coefficient
Barren land	0.025
Cultivated Crops	0.04
Deciduous Forest	0.12
Development, High Intensity	0.15
Development, Medium Intensity	0.08
Development, Low Intensity	0.08
Development, Open Space	0.04
Emergent Herbaceous Wetland	0.07
Evergreen Forest	0.12
Grassland	0.035
Mixed Forest	0.12
Open Water	0.04
Pasture	0.035
Shrub	0.1
Woody Wetlands	0.12

C.1.3.4 Structures

C.1.3.4.1 Bridges

Two existing bridges are located over the existing C-18W canal. A rail road bridge and the State Road 710 bridges were included in the model as part of the C-18W 1-D reach. Bridge data was acquired from the SFWMD and previous studies performed by Arcadis (2016).

C.1.3.4.2 Culverts

Several culverts are proposed for the Flow-way 2 within the C-18W reservoir areas. These culverts are designed to convey water through the seepage collection canal and convey it to the existing C-18W canal.

Located in the southwest corner, culvert S-105 is designed to convey water that will be pumped from the M-O canal into C-101W. There are three (3) culverts located in the seepage collection canal. The S-103W and S-103E are gated control structures located at the northern terminus of the seepage canal, which discharge water from the seepage collection canal to the outflow canal, C-101N. Culvert S-104, located south of the C-18W reservoir, is an ungated culvert that provides a crossing to access the reservoir embankment area. Runoff from the J.W. Corbett WMA discharges via the S-107 drop structure, which is composed of an inlet controlled weir with gated culverts. **Table 3** is a summary of the culverts included in Flow-way 2.

Table 3. Flow-way 2 Culvert Descriptions

Structure Number	Culvert Location	Control	Size
S-102	C-18W Embankment	Gated	(2) 6 ft x 4 ft
S-103E	Seepage collection canal (C-101W)	Gated	(1) 10 ft x 10 ft
S-103W	Seepage collection canal (C-101W)	Gated	(1) 10 ft x 10 ft
S-104	Seepage collection canal (C-101E)	Ungated	(1) 10 ft x 10 ft
S-105	Levee L-101W	Gated	(3) 6.5 ft
S-107	Levee L-101W	Gated	(3) 6.0 ft

C.1.3.4.3 Pumps

Three (3) pump stations are proposed for the project area: S-101A, S-106 and S-109. S-101A and S-106 pump water into the reservoir to be stored for environmental releases as needed. S-106 also contains seepage control pumps, which aid in the recovery of seepage back into the reservoir. Pump station S-109 brings water from the M-O canal into the C-101W canal. All pump station capacities were determined through analysis of the flow-frequency curves from the LECSR-NP long-term hydrologic model simulation. Pump mix criteria were also determined through analysis of the flow frequency curves as well as USACE and SFWMD pump mix design criteria. Additional details regarding the pump station capacities and mix designs can be found in **Appendix A, Engineering Appendix.**

Pump station S-101A, located on the north side of the C-18W reservoir, was designed to provide a maximum discharge of 150 cfs. The station was designed for two (2) 25 cfs and two (2) 50 cfs pumps.

Pump station S-106, located in the west side of the reservoir was designed to provide a maximum discharge of 175 cfs. The station was designed for three (3) 25 cfs and two (2) 50 cfs pumps. Additionally, the station will have (2) 30 cfs pumps to seepage control.

Pump station S-109, was designed to provide a maximum discharge of 150 cfs. The station was designed for two (2) 25 cfs and two (2) 50 cfs pumps.

For modeling purposes, the pump stations were assumed to not be operating for the evaluated 100-yr flood event. For large flood events water in the seepage collection canal can discharge using the proposed culverts S-103E and S-103W. In addition, no water will be pumped into the reservoir during flood events.

C.1.3.4.4 Lateral Structures

Lateral structures were added to the 1-D canals to connect to the 2-D areas included in the model. The C-18W and M-O canal reaches include lateral structures at the right and left banks of the canals. Lateral structures connect the canal banks to the adjacent 2-D areas to simulate bank overflow. Lateral weir coefficients were assigned based guidance given in the HEC-RAS 2-D modeling manual (2016). Coefficient values ranged from 1.0 to 2.6 for levees and roadways and from 0.2 to 1.0 for natural high ground.

C.1.3.4.5 Inline Structures

The existing C-18W weir located in the existing C-18W canal was included in the HEC-RAS model as an inline structure in the C-18W canal 1-D reach. C-18 weir is an uncontrolled sheet pile weir located east of the project and downstream of the existing bridge over State Road 710. The inline structure was simulated as a broad crested weir with a weir coefficient of 2.6.

C.1.3.5 Levees

The proposed features for Flow-way 2 include two levees. Levee L-101W is located west of the reservoir, and was designed to contain runoff in the J.W Corbett WMA. Levee L-101E is located north of the C-18W, to the right of and adjacent to C-101W canal. Levees were included in the model as 2-D storage area connections.

C.1.3.6 Storage Areas

The C-18W reservoir was included in the HEC-RAS model as a storage area. The reservoir has two outlet structures on the proposed D-101 embankment. Structure S-101B has a dual function as a service spillway for permitted discharges and auxiliary spillway for emergency discharges. During the regular operation (no flood events), the C-18W reservoir discharges using the S-102 gated culvert. Culvert S-102 was not included in the modeling since this structures is not designed to operate during flood events. The S-101B spillway and the D-101 embankment were included in the model as storage area connections.

C.1.3.7 Additional Detail

C.1.3.7.1 Calculation Options and Tolerances

Unsteady computation options and tolerances were adjusted to provide more stability to the model. Tolerance for 1-D water surface elevations and storage areas were set to 0.2 ft. Flow calculation for 2-D areas was set as a combination of full momentum and diffusion wave equations, maximum water surface tolerance for 2-D areas was set to 0.2 ft and volume tolerance to 0.1 ft. Cross sections HTab parameters were adjusted to 0.2 increments and 100 points.

C.1.3.7.2 Numerical Stability

The target for numerical stability for the developed HEC-RAS model was to minimalize the model mass balance error. The hydraulic model was developed to have mass balance error lower than 1 percent.

C.1.3.7.3 Ineffective Flow and Levees

Ineffective flow areas were added to bridge cross sections to account for areas when the flow is restricted due to the bridge embankment. These areas are used to describe a portion of the cross section in which the water velocity is close to zero.

C.1.3.7.4 Junctions

Junctions are used in 1-D reaches to combine reaches that come together at specific points. One junction was used in the C-18W reaches; the junction was added to combine the outflow canal C-101N with the existing C-18W canal. Computation for junctions was the force equal to water surface elevation.

C.1.4 Model Calibration and Validation

There is no observed hydrologic data in the areas nearby to Flow-way 2 that can be used for model calibration. The HEC-RAS model developed for Flow-way 2 was not calibrated. Best engineering practices and judgment were used for the selection of the model parameters in order to obtain adequate model results. Standard values for Manning's roughness dependent on land use were used. Breaklines and cell spacing adjustments occurred to reduce errors based on terrain data.

C.1.5 Model Application for Design

The HEC-RAS model determined water surface elevations in the areas nearby to the Flow-way 2 and sizes for the proposed hydraulic structures to ensure the adequate capacity for the proposed project objectives. The design storm, 100-year, 72-hour rainfall event was input as a rainfall hyetograph onto the 2D areas. The simulation was run to determine peak water surface elevations and flow rates at the proposed structures to help inform the proper hydraulic design.

C.1.6 Model Results

The HEC-RAS modeling results are used to support the hydraulic design of the proposed Flow-way 2 features. Water surface elevation and velocities for the design event (100-year, 72 hour) were to determine hydraulic performance of the project with the structures in places. **Figure 6** to **Figure 11** show the water surface elevations in the proposed canals.

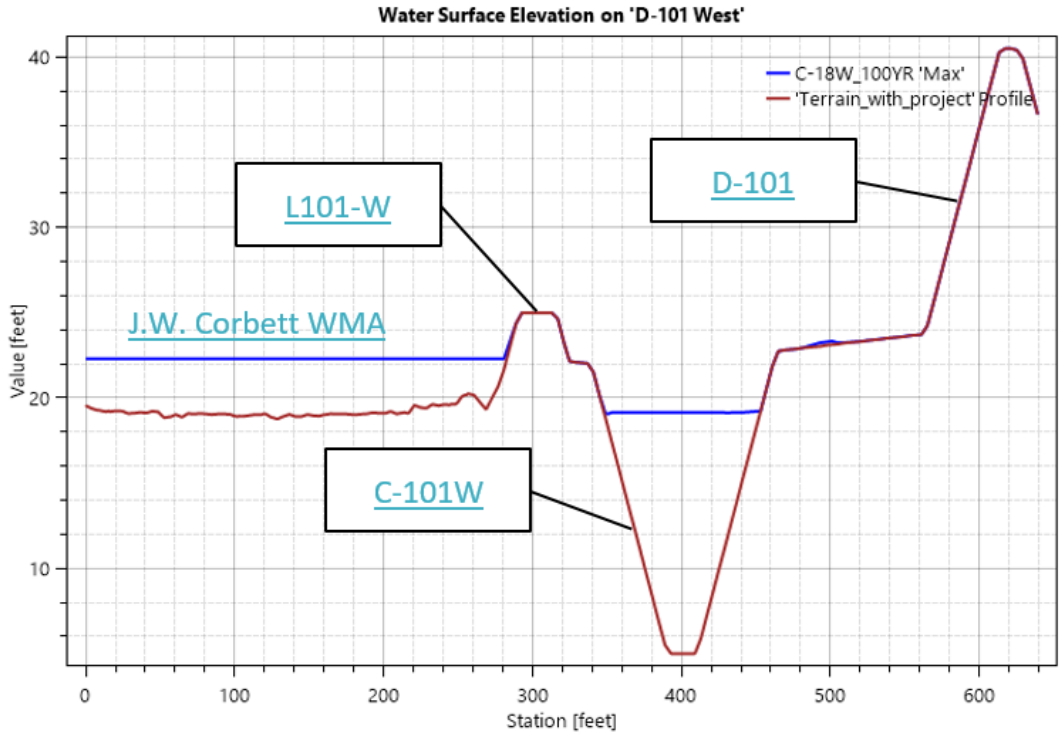

Figure 6. Maximum Water Surface Elevation For the 100-year Event - West Side of the C-18W Embankment

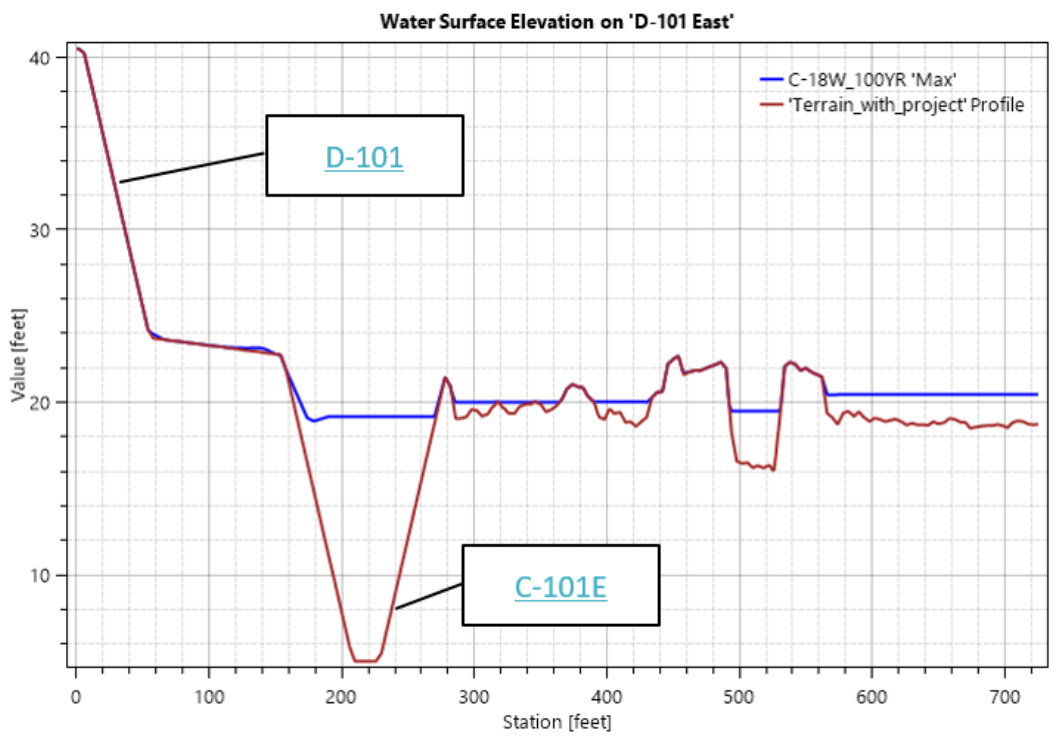

Figure 7. Maximum Water Surface Elevation For the 100-year Event - East Side of the C-18W Embankmen

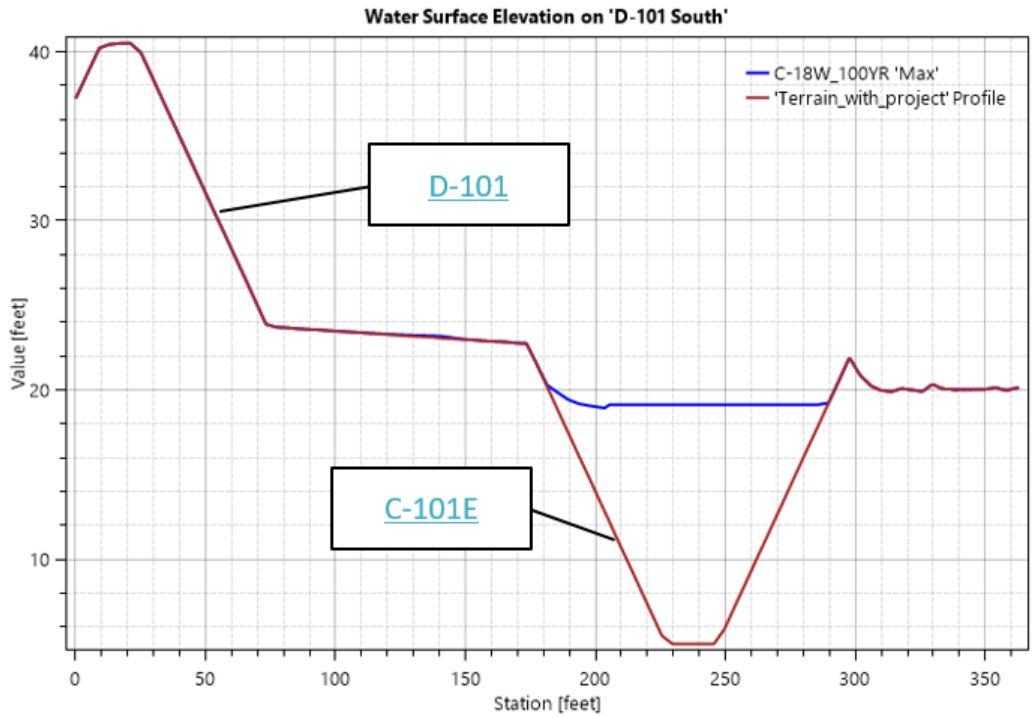

Figure 8. Maximum Water Surface Elevation For the 100-year Event - South Side of the C-18W Embankment

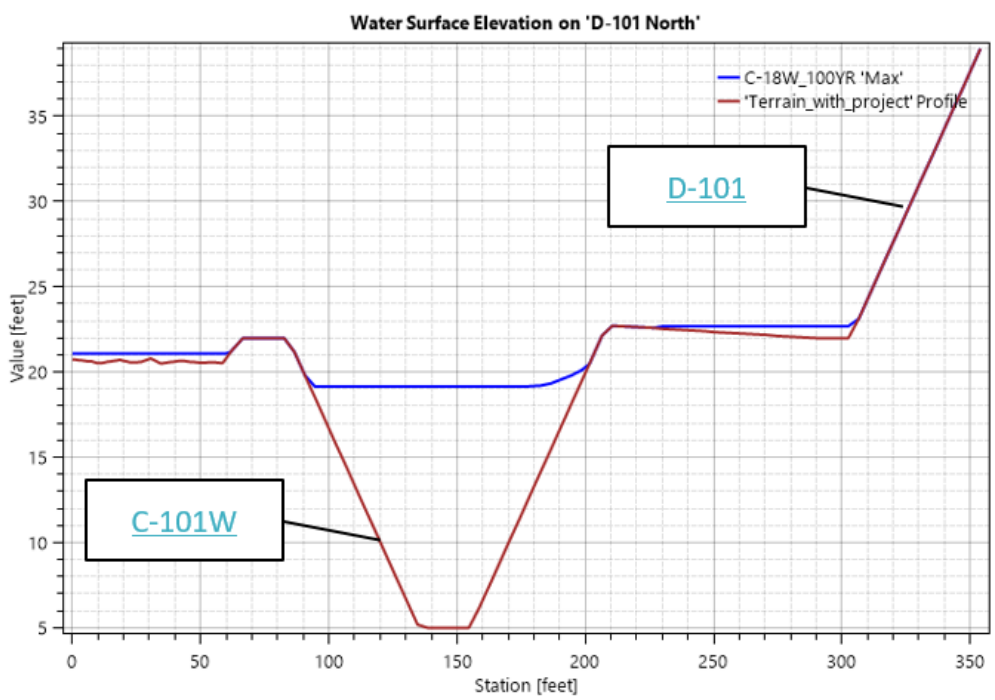

Figure 9. Maximum Water Surface Elevation For the 100-year Event - North Side of the C-18W Embankment

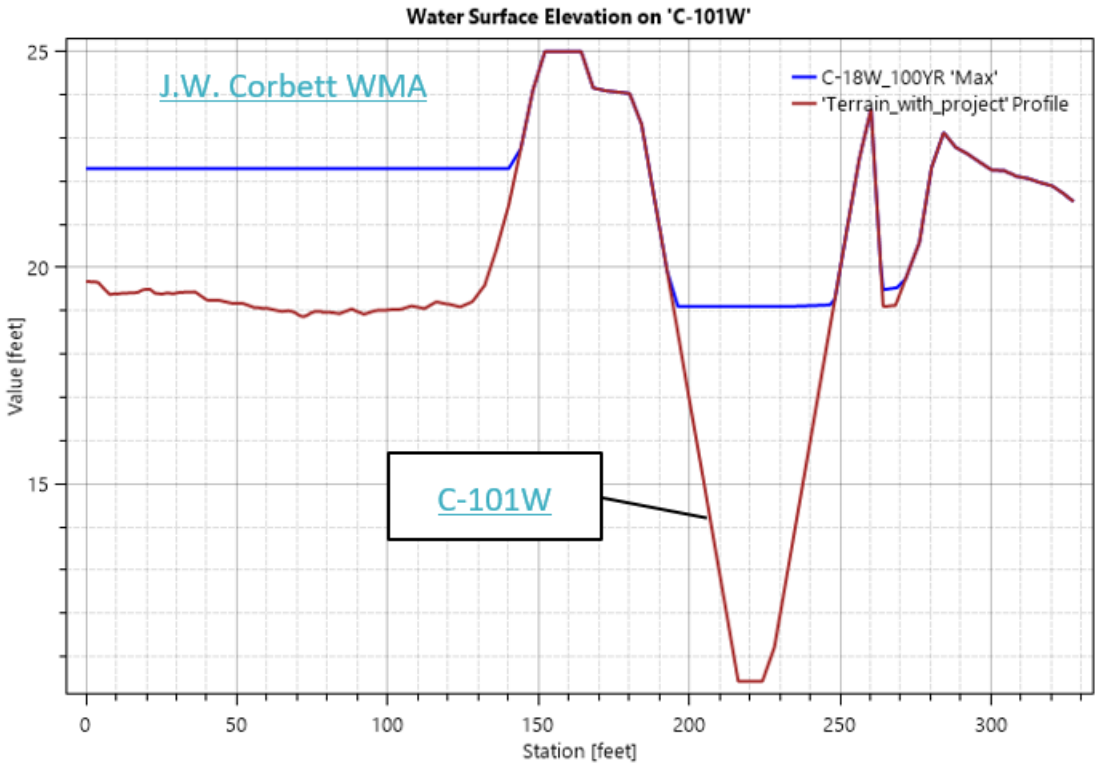

Figure 10. Maximum Water Surface Elevation For the 100-year Event – South C-101W Canal

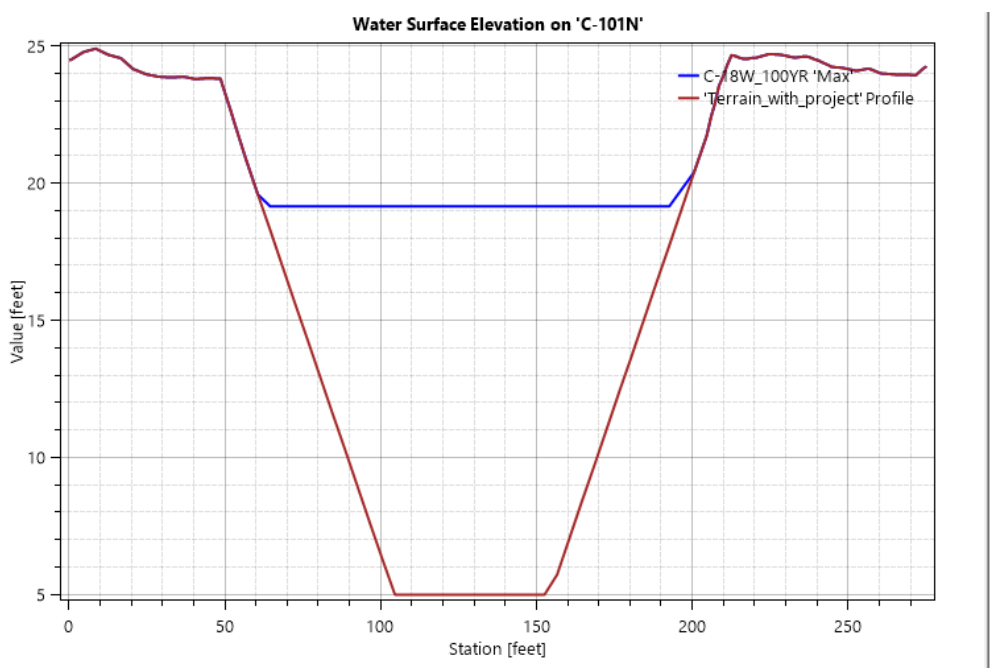

Figure 11. Maximum Water Surface Elevation For the 100-year Event – South C-101N Canal

C.2 Flow-way 3, Western Project Features

C.2.1 Model Domain

The model domain lies within southeastern Martin County, and covers the watersheds that contribute to the northwest fork of the Loxahatchee River via the Cypress Creek Canal. The extent of the model is Pratt Whitney Road (SR-711) to the west, Florida Turnpike to the east, West Indiantown Road to the south and the Nine Gems northern canal to the north. The model includes the Pal-Mar East watershed, in which the Nine Gems area is a microcosm of the larger Pal-Mar area. Nine Gems is bordered by Pratt Whitney Road to the west and the Hobe St. Lucie Conservancy District (HSLCD) Unit 2 to the north, where it is separated by two, parallel medium sized drainage canals. The canal immediately adjacent of Nine Gems conveys flow from a farm outside of the model domain, known as the Thomas Pepper Farm, as well as drainage from Nine Gems. Immediately to the east, the property is bordered by a large canal, labeled the HSLCD canal, which separates it from the Gulfstream West Property. The Gulfstream West Property is an existing 700 acre agricultural tract vegetated with abandoned orange groves. To the southeast, Nine Gems is bordered by the Ranch Colony Development and to the southwest by undeveloped Pal-Mar lands. The length of the southern border is separated from adjacent areas by the southern Nine Gems Canal. The south eastern portion of Pal-Mar East (also known as "Culpepper") currently drains through four water control structures into the Cypress Creek Canal. The Cypress Creek Canal begins at the south-eastern corner of Culpepper and extends north and east between residential communities (The Links and Ranch Colony). Additionally, the HSLCD drainage canal (combined discharge from Nine Gems and HSLCD Unit 2) discharges into the downstream portion of the Cypress Creek Canal. The combined flow discharges uncontrolled under I-95 and the Florida Turnpike eventually reaching the northwest fork of the Loxahatchee River, downstream of Lainhart Dam. The Cypress Creek Canal is bordered by the Cypress Creek Natural Area to the south, which is an area of uplands and sloughs. The aforementioned watershed areas and canals, as well as additional features including berms and hydraulic structures, were included within the model and are shown in **Figure 12**.

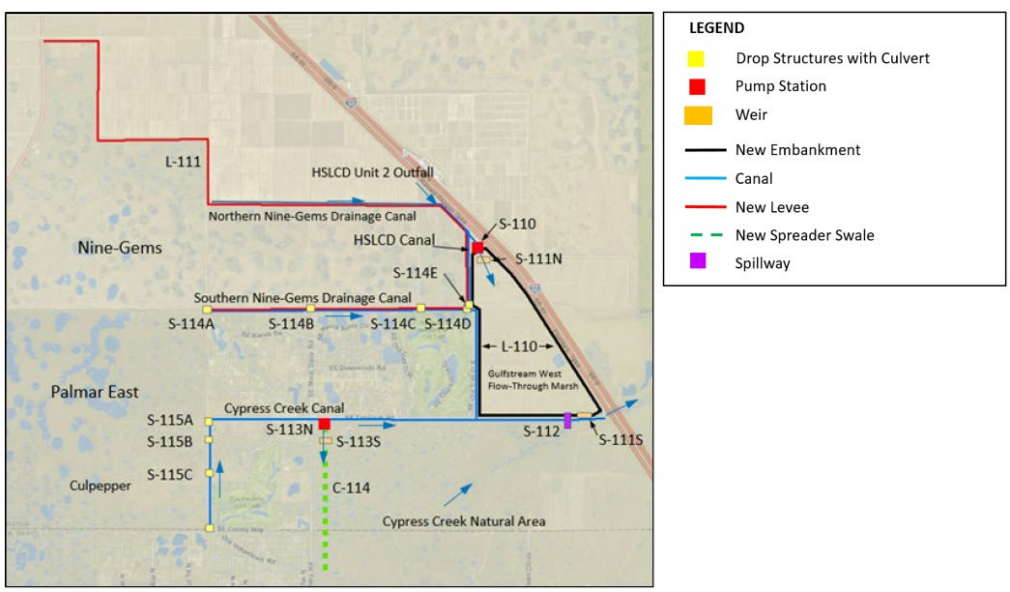

Figure 12. Flow-way 3 Proposed Hydraulic Features

The model domain between the existing and with-project condition is maintained but interior features are modified. Figure 13. Model Domain for Flow-way 3, Western Project Area **Figure 13** shows the HEC-RAS model domain for the Flow-way 3 Western project features. The gridded areas within the figure represent 2-D coverage of watersheds whereas the 1-D river reaches (canals) are shown as blue lines. Surrounding areas that contribute runoff but are not explicitly shown as a 1-D or 2-D area include the residential communities, HSLCD unit 2 agricultural area, and Thomas Pepper Farms runoff, which were input as boundary conditions to the model.

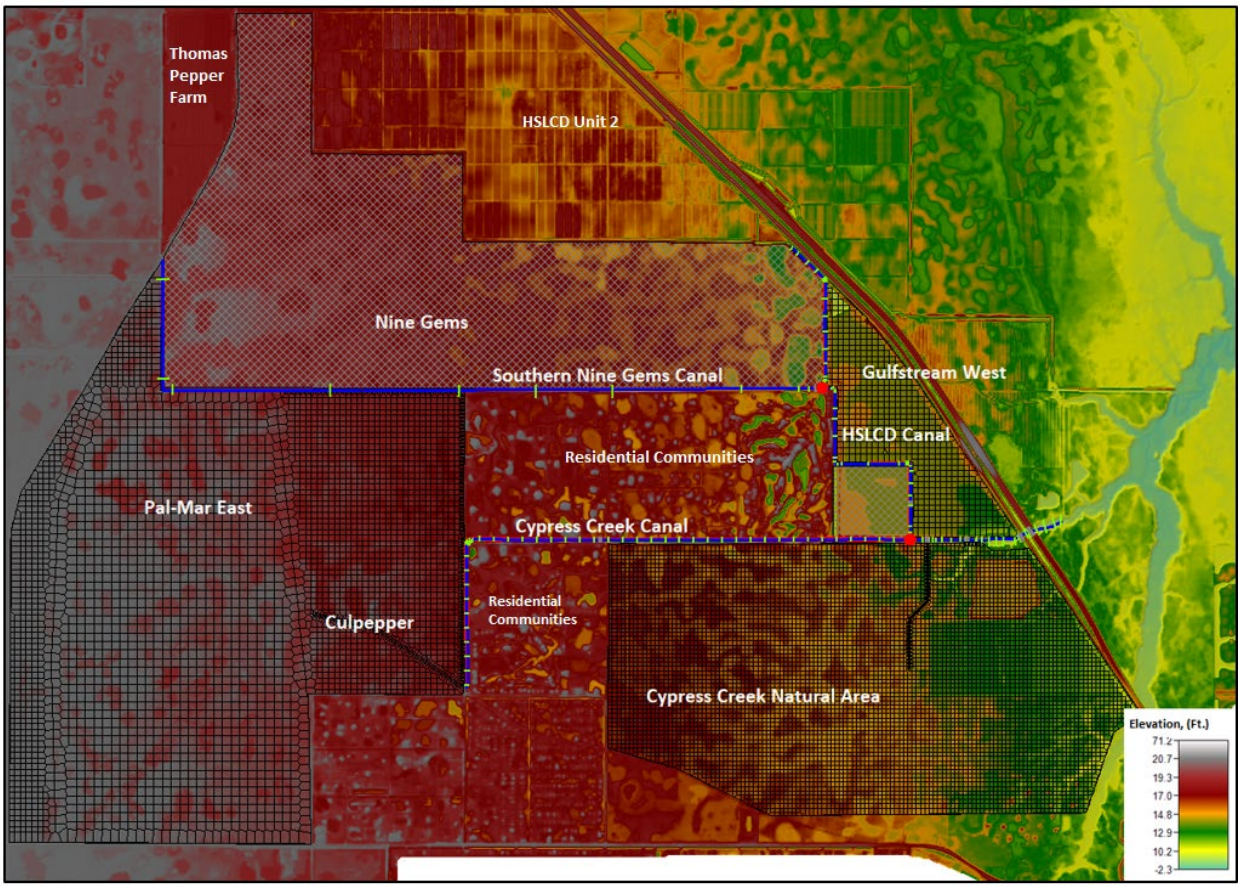

Figure 13. Model Domain for Flow-way 3, Western Project Area (Existing Conditions)

C.2.1.1 Model Mesh

The model mesh was developed from a terrain model developed from the LiDAR data for Martin County, Florida. The proposed project features, including the natural areas and flow-through marsh, are included in the terrain as 2D areas. These areas were constructed in contributing watershed areas to help represent the potential hydrologic runoff occurring from the design storm. The cell spacing varied for each watershed area. The cell spacing is a tradeoff of considering topographic affects and features, anticipated hydraulic turbulence, proximity to the flood wave, total number of cells in the model, and reasonable numerical accuracy while minimizing simulation run times. As simulations were run the grid mesh was further refined and improved with breaklines for levees, roads and other features requiring smaller cell sizes. The model mesh sizes varied between 25-75 ft in refined areas, and 150-250 ft within the greater watershed areas.

C.2.2 Boundary Conditions

Boundary conditions within HEC-RAS are necessary to perform the hydraulic calculations and are the main driver of the model simulation. Additionally, they establish the upstream and downstream starting water surface elevations and are necessary to perform the calculations. Boundary conditions were entered into the model as flow hydrographs, lateral inflow hydrographs, precipitation and normal depth. Flow-hydrographs were specified as upstream boundary conditions to the 1-D reaches. The lateral inflow hydrographs were used as internal boundary conditions along the 1-D river reach at the appropriate cross section. Direct precipitation was input as the boundary condition for all 2-D areas. The normal depth was specified only as a downstream boundary condition for the 1-D reaches and applicable 2-D areas using an estimated slope of the energy grade line. An example of the boundary conditions used in the design storm, 100-year, 72-hour event, including all proposed features of the Recommended Plan, is included in **Table 4**. Additional details and specific model boundary conditions for the Calibration event, 2014 Existing Conditions model, and Recommended Plan model are included in sections C.2.4 and C.2.5.

Table 4. Flow-way 3 West HEC-RAS Boundary Conditions for 100-year, 72-hour Design Storm

	River	Reach	RS	Boundary Condition
1	HSLCD	Reach1	13689.11	Flow Hydrograph
2	NineGemsSouth	Reach 1	12649	Flow Hydrograph
3	RanchColonyCana	Reach1	24500	Flow Hydrograph
4	RanchColonyCana	Reach1	24400 LS	T.S. Gate Openings
5	RanchColonyCana	Reach1	23500	Lateral Inflow Hydr.
6	RanchColonyCana	Reach1	16000	Lateral Inflow Hydr.
7	RanchColonyCana	Reach1	13500	Lateral Inflow Hydr.
8	RanchColonyCana	Reach1	10500	Lateral Inflow Hydr.
9	RanchColonyCana	Reach1 - Lower	2200 IS	T.S. Gate Openings
10	RanchColonyCana	Reach1 - Lower	-1991	Normal Depth
11	spreader_swale	Reach1	5721.837	Flow Hydrograph
12	spreader_swale	Reach1	84.35181	Normal Depth
	Storage/2D Flow Areas			**Boundary Condition**
1	CCC_2d	BCLine: ccc_southernboun		Normal Depth
2	CCC_2d			Precipitation
3	Culpepper			Precipitation
4	FTM2			Precipitation
5	FTM_2D			Precipitation
6	gulfstreamW			Precipitation
7	GulfstreamW2			Precipitation
8	NineGems			Precipitation
9	Spreader_Pump			Precipitation

C.2.3 Model Parameters

Channel and floodplain information required by the HEC-RAS model was extracted from LiDAR digital terrain elevation data and aerial imagery data using HEC-GeoRAS (HEC, 2006). HEC-GeoRAS is an ArcGIS tool that creates geometric input for HEC-RAS by utilizing user-defined GIS layers to extract information from the terrain data. The version of HEC-RAS, 5.0.5, contains the RAS mapper feature that allows pre-processing of data from various sources to create geometry files for RAS modeling. Additional data was entered into the HEC-RAS model based on information gathered from available documentation and/or

from using standard engineering equations to estimate model parameters. The following paragraphs describe both GIS generated data and additional model parameter estimation used in the construction of the HEC-RAS model.

C.2.3.1 Channel Cross-Section and Profile

The cross-sections cut from the DEM data did not contain information below the water surface. Cross section spacing was set at approximately 500 ft for Cypress Creek Canal and varied for the HSLCD canal. Cross-sectional survey data performed by Martin County from 2000 was available for portions of the Cypress Creek Canal west of Mack Dairy road. Channel depth and width were based on estimation when no survey data was available. Tools in HEC-RAS were used to modify the cross-sections to include the channel below the water. Channel invert elevations were also compared to elevation data presented in a 2016 modeling effort by Tomasello Consultants (Tomasello, 2016).

C.2.3.1.1 Canals

Two canals were modeled in the Calibration model geometry, the HSLCD canal and Cypress Creek Canal. For the 2014 Existing Conditions model, the southern Nine Gems canal was added to represent runoff from the Nine Gems and Thomas Pepper Farm Area. This canal had previously been represented in the Calibration model as a boundary condition, using discharge values computed by the LECSR-NP model. The Recommended Plan model geometry included the new Mack Dairy Spreader Swale (C-114) as well the Cypress Creek Canal, HSLCD canal, and southern Nine Gems canal, all of which had been modified due to the proposed project features.

C.2.3.2 Manning's "n" Values

Manning's n values within the 1D cross sections were represented using horizontally varying manning's value for the channel, right and left overbank. Manning's n-values from "Open-Channel Hydraulics" (Chow, 1959) were used and based on field observations and aerial imagery. For the Calibration and 2014 Existing Baseline, Manning's values for the channel were, on average, between 0.035-0.045, with an increase up to 0.065, representing increased channel blockage from debris, which was observed in historical photographs. The left and right overbank Manning's values were on average around 0.045, increasing to 0.075. Portions of the Cypress Creek Canal and the HSLCD canal contain vegetated banks, thus requiring a higher Manning's value. The Manning's n values were modified slightly for the with-project condition as areas of the channel have proposed improvements and/or widening and therefore will have decreased vegetation coverage, resulting in a lower Manning's n value.

Spatially varying Manning's n values were used for the 2D areas and were estimated based on land use coverage. The USGS 2011 NLCD coverage was imported into the RAS Mapper and associated Manning's values were identified. The Manning's n-value spatial coverage for the 2-D flow areas of the model are similar to those used in the Flow-way 2 model and are shown in **Table 2**.

C.2.3.3 Structures

C.2.3.3.1 Bridges

Four bridges exist within the Cypress Creek Canal, including the Mack Dairy Road, canal crossing from Cypress Creek Natural Area to Gulfstream West, Florida Turnpike and I-95. Data for the Mack Dairy Road

and Canal crossing bridges was estimated using the DEM and field observations. Data for the I-95 and Florida Turnpike were obtained from the Florida Department of Transportation. Bridge inputs required for the model included low chord elevations, high chord elevations, pier diameters, and pier spacing. Bridge data was not modified between the Calibration, 2014 Existing Condition Baseline and Recommended Plan model runs with the exception of removal of the Canal crossing bridge in the Recommended Plan.

C.2.3.3.2 Culverts

Multiple culverts were identified for inclusion into the geometry file for the Calibration, 2014 Existing Conditions and the Recommended Plan. Culvert as-built data was only available for the Water Control Structures (WCS) within the Culpepper Berm, which were constructed in 2013. The Calibration contained geometry reflective of conditions in 2004. All other existing culverts were estimated based on data received, field observations, or measurements. The Nine Gems Discharge culverts are located at the southern terminus of the north-south bisecting canals within Nine Gems and discharge into the southern Nine Gems Canal. **Figure 14** illustrates the various culvert locations and with corresponding labels in **Table 5**, which contains additional information and identification of model inclusion.

The southern Nine Gems canal culverts 1-4 and the Eastern Nine Gems culvert were not simulated in the calibration run as the LECSR-NP model results were used to represent the hydrologic response in this area and input into the model as a boundary condition. Within the existing and with-project conditions, these culverts were simulated to capture the hydraulic response of the design storm event. The Culpepper WCS 1 and 2 were replaced and WCS 3 and the Jupiter Grade structure were added in 2013, therefore the Calibration and Existing/Recommended Plan project conditions vary for these structures. The HSLCD 2 and Gulfstream West project culverts are removed in the Recommended Plan and therefore only simulated in the Calibration and Existing conditions simulation. The HSLCD 1 is modeled in all simulations.

Figure 14. Culvert Locations and Names

Table 5. Flow-way 3, Western Area Culvert Description

Culvert Location, Name	Control	Size	Model Simulation
Culpepper Berm, WCS 1	Inlet Control, Drop Structure	2-84”	Existing, Recommended Plan
Culpepper Berm, WCS 2	Inlet Control, Drop Structure	48”	Existing, Recommended Plan
Culpepper Berm, WCS 3	Inlet Control, Drop Structure	60”	Existing, Recommended Plan
Jupiter Grade	Inlet Control, Drop Structure	48”	Existing, Recommended Plan
Culpepper Berm, WCS 1	None	2-84”	Calibration
Culpepper Berm, WCS 2	None	48”	Calibration
HSLCD 1	None	3-84”	All model Simulations
HSLCD 2	None	4-72”	Calibration & Existing
Southern Nine Gems Canal Eastern Discharge	Inlet Control, Drop Structure	2-96”	Existing, Recommended Plan
Southern Nine Gems Canal 1-4	Inlet Control, Drop Structure	Variable, 3- 48” and 1-60”	Existing
Southern Nine Gems Canal 2-4	Gated Culvert	4-72”	Recommended Plan
Gulfstream West	None	1-48’, 1-36’	Calibration & Existing

C.2.3.3.3 Pumps

No pumps were present in the Calibration or 2014 Existing Conditions model conditions. For the Recommended Plan, pumps were sized using the LECSR-NP model output flow frequency curves and were not further refined in HEC-RAS. Maximum pumping rates were used as upstream boundary conditions for the Mack Dairy Spreader Swale (C-114) and for the Gulfstream Flow-through Marsh.

C.2.3.3.4 Lateral Structures

Lateral structures in the model are located along 1-D canal reaches and connect to 2-D flow areas. Lateral structures were included to represent existing or proposed berms that may or may not contain structures. For the Calibration and 2014 existing condition model, two lateral structure were used to represent drainage culverts from Gulfstream West to the HSLCD canal. The Culpepper berm and four WCS were also

included as a lateral structure for all model simulations. An additional lateral structure was added to represent the Gulfstream West discharge structure (S-111S) in the Recommended Plan model simulations and was modeled as a sharp crested weir with a coefficient of 3.1. The lateral structures representing berms were simulated as a broad crested weir with typical weir coefficient of 2.6, based on guidance given in the HEC-RAS 2-D reference manual (2016). Additionally, a lateral structure was necessary to connect the 2D area for Cypress Creek Natural Area to the Cypress Creek Canal in all simulations and was simulated using a Lateral weir coefficient of 0.9, also identified as an appropriate coefficient within the HEC-RAS 2-D reference manual.

C.2.3.3.5 Inline Structures

The existing Cypress Creek Canal Weir structure was added as an inline structure in the Calibration and 2014 existing condition model. Replacement of the Cypress Creek Canal Weir with the proposed ogee spillway structure (S-112) was included in the Recommended Plan model. Additional inline structures included existing culverts along HSLCD and the southern Nine Gems Canal. The existing Cypress Creek Canal weir structure was modeled using a weir coefficient of 2.6 due to the presence of debris buildup. The S-112 structure was modeled with a weir coefficient of 3.8, which was computed based on the spillway approach height and design energy head. The inline structure containing culverts were simulated as a broad crested weir shape with coefficient of 2.6.

C.2.3.4 Levees

The Recommended Plan's Gulfstream West Levee, L-110, was burned into the with-project digital terrain model.

C.2.3.5 Storage Areas

There are multiple 2D flow areas constructed within the model, each representing a watershed or contributing area. For the Calibration event, the Gulfstream West property, Cypress Creek Natural area, and Culpepper watersheds were simulated as 2D flow areas. An additional flow area representing the Nine Gems property was added for the Existing and Recommended Plan simulations.

C.2.3.6 Additional Details

C.2.3.6.1 Calculation Options and Tolerances

The calculation options and tolerances were not modified from the default settings. Hydraulic property table (HTab) parameters were adjusted for better performance in the unsteady flow model, specifying an increment of 0.1 ft with 100-250 points, depending on the cross section. HTab parameters were adjusted at bridge crossings to better define the HTab curves for the bridges.

C.2.3.6.2 Numerical Stability

Unsteady flow testing yielded some numerical instabilities near junctions and where inline and lateral structures were placed. These instabilities were reduced through reducing the computation time step interval, refining the 2D mesh, and decreasing the HTab increment. The target for numerical stability for the developed HEC-RAS model was to minimalize the model mass balance error. All simulations (Calibration, Existing, and Recommended Plan) were developed to have mass balance error lower than 3 percent.

C.2.3.6.3 Ineffective Flow

Ineffective flow areas were used in the HEC-RAS cross sections at bridges and inline structures that prevents HEC-RAS from using the entire cross section for active flow calculations. These areas are used to describe a portion of the cross section in which the water velocity is close to zero.

C.2.3.6.4 Junctions

Junction computation mode was selected as force equal water surface elevations for all simulations. The appropriate length across the Junction was input based on the appropriate downstream reach length.

C.2.4 Model Calibration

Limited observed data is available within the proposed Flow-way 3 area. Gage 265818080111900 Cypress Creek Canal below Gulfstream Bridge, FL was the only gage located along the modeled reach and reported daily stage and flow values. The gage was located approximately 0.5 miles west of I-95 within the Cypress Creek Canal and contained a period of record of October 2002 to September 2004. This duration coincided with a large tropical event in September 2004, Hurricane Jeanne, to which the model was calibrated. The gage records are noted to be poor, with buildup of debris occurring at the bridge piling and the downstream structure. **Figure 15** shows the location of the gage in relation to the rest of the model domain. This gage is located near the downstream boundary of the model, therefore capturing the majority of the modeled area's response.

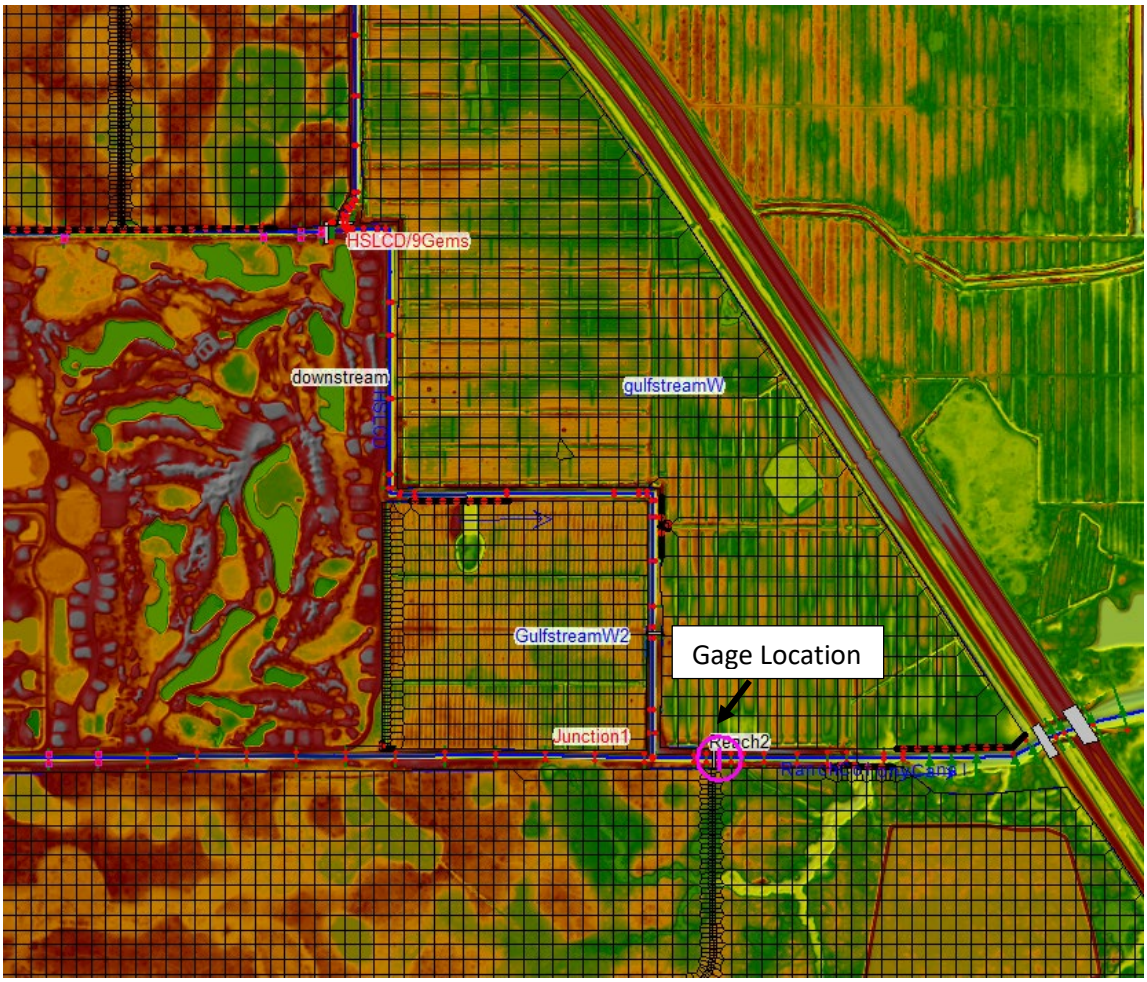

Figure 15. USGS Gage Cypress Creek Canal Location

The model calibration simulation time window was selected as September 17-29, 2004. The LECSR-NP model output (see **Appendix A** for LECSR-NP description) was used to represent the hydrologic response of areas outside of, or not computed within, the model domain and were input as boundary conditions into the HEC-RAS model. Where applicable, an inflow time series from LECSR-NP model was used as a flow hydrograph or lateral inflow hydrograph in the 1D river reaches within HEC-RAS. The upstream boundary condition for the HSLCD canal was specified as a flow hydrograph and was representative of a combination of HSLCD Unit 2 and the northern Nine Gems canal discharge. A lateral inflow hydrograph was also specified along the appropriate cross-section within the HSLCD reach to represent discharge from the Nine Gems southern canal discharge. Similarly, lateral inflow hydrographs were used to represent drainage from the surrounding neighborhoods and input at the applicable cross sections along Cypress Creek Canal. All aforementioned LECSR-NP simulation results were from the corresponding model calibration simulation time window and were at a daily time step.

Additional boundary conditions included a flow hydrograph within the Cypress Creek Canal, which included a representative initial starting condition within the canal as baseflow. A normal depth of 0.005

ft/ft was used to represent downstream boundary conditions for the Cypress Creek Canal and Cypress Creek Natural Area. Observed direct precipitation data from the Jonathan Dickinson State Park Weather Station, Pratt and Whitney Station, and the S-46 Station were used as they were each located approximately 5-6 miles from the centroid of the HEC-RAS model domain. An average rainfall total from the gage records was computed and applied as direct precipitation to the 2D flow areas at a daily time step.

The geometric model data for the Calibration effort is shown below in **Figure 16**. The geometric data is representative of project features present in 2004, which, with the exception of the Culpepper water control structures, is similar to existing conditions in the area. It was important to simulate the 2004 features as these conditions were present during the calibration simulation time window.

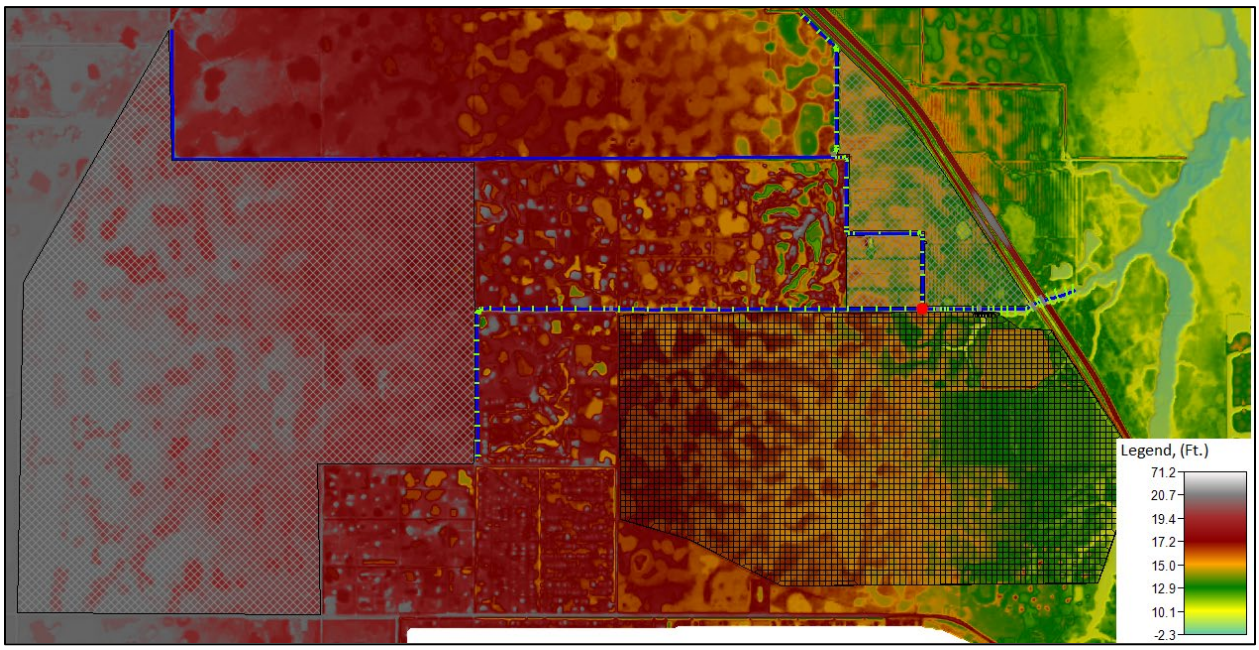

Figure 16. Calibration Model Geometric Data View

The primary goal of the calibration process is to match the simulation results to the observed stage and flow gage data as closely as possible. The modeled error, as measured by statistical analysis, should be minimized by the calibration process.

Calibration of the model is achieved by running the HEC-RAS simulations, comparing the results to the observed data, adjusting parameter values within their reasonable ranges, and re-simulating until the calibration goals are met. The goal of the model calibration is to achieve a balance between the "goodness of fit" statistics and the physical meaning of the parameters values. The calibration parameters are varied during the initial calibration process to begin comparison between model simulation results and observed data. Those parameters that seem to produce a relatively close approximation to the observed data remain the same, while other parameters are modified to reproduce the results as closely as possible.

Calibration was achieved through modification of Manning's N values and multiplication of the LECSR-NP inflow hydrographs, which were believed to be underestimating peak daily flow for the event. Manning's N values were modified approximately +0.01-0.02 in the 1D cross sections near the downstream portions

of Cypress Creek Canal, close to the gage. This modification was necessary due to the presence of heavy vegetation noted in aerial imagery from 2004.

Although the LECSR-NP model was calibrated in the Flow-way 3 area, it has limited hydraulic routing capabilities and may not perform adequately during intense rainfall events. In the Model Calibration report for LECSR-NP (Obeysekera, et al., 2018), it is stated that the "Cypress Creek area appears to be simulating drier conditions than the field observed estimates". The LECSR-NP simulated versus historical cumulative flow curves illustrate that the model simulated lower flows than historical conditions during the 2004 time period. Therefore, the inflow hydrograph from LECSR-NP for the HSLCD inflow and southern Nine Gems canal was modified by applying a uniform multiplication value of two for the entire hydrograph time series. Additionally, the urban area hydrograph from the LECSR-NP was multiplied by three. This was necessary to achieve a closer match to the observed flow data.

Observed discharge and simulation data allows a direct comparison to be made to evaluate the performance of the model. This comparison is completed using the statistical measures Nash-Sutcliffe (NSE) as well as the correlation coefficient (r^2). These were used to quantitatively compare the hydraulic simulation results to determine if the models were adequately calibrated and validated. The statistical thresholds for calibration was to achieve above a NSE and r^2 value greater than 0.5. The NSE and r^2 for stage were computed as 0.58 and 0.88, respectively. The NSE and r^2 for flow were computed as 0.83 and 0.92, respectively.

Calibration results are shown below in **Figure 17** and **Figure 18**. Calibration results illustrate the modeled stage data is approximately 1-2 ft lower than observed data, but the data trends are being matched relatively well. Therefore, although the magnitude is off, the simulated system response is similar to the observed data. Potential explanations for the discrepancy between the modeled and observed data are: 1.) the channel geometry below the recorded LiDAR data water levels is estimated and modeled channel inverts may contain errors, additional channel survey will be necessary for PED efforts 2.) Hurricane Jeanne made landfall just north of the project area, potentially causing storm surge and varying tailwater conditions within the Loxahatchee Estuary which were not captured in the Normal Depth boundary (no observed tidal data available for use) 3.) Debris within the channel could cause potential channel blockage thereby decreasing the cross sectional area for flow and impacting stages 4.) Rainfall variability and uncertainty.

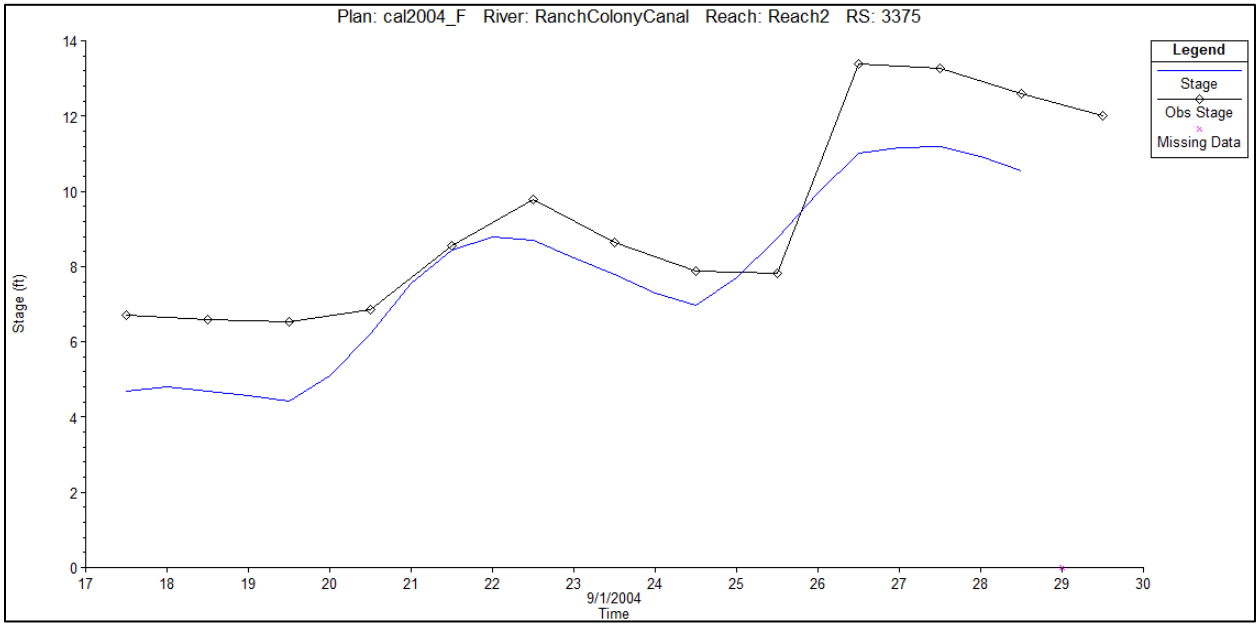

Figure 17. Calibrated vs. Observed Stage data

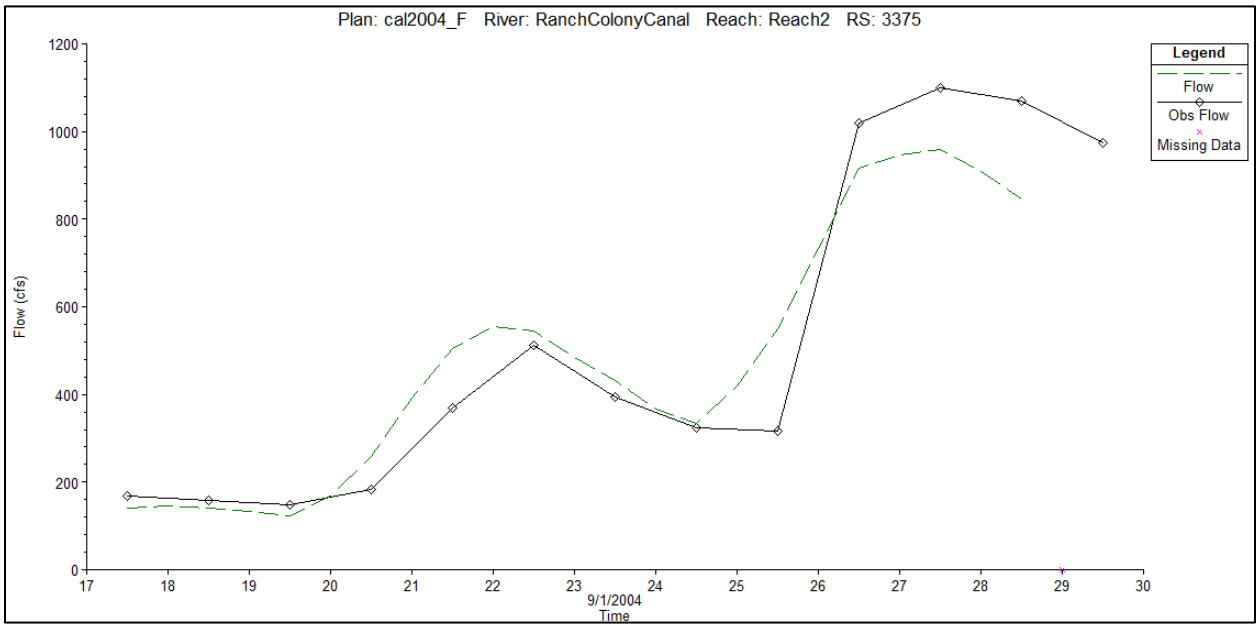

Figure 18. Calibrated vs. Observed Flow data

There was no additional data available to validate the model. Additional data collection will be necessary to complete additional calibration and validation model simulations for further model refinement during the PED phase.

C.2.5 Model Application for Design

The calibration model was used as the 2014 Existing Conditions baseline to provide an estimate of existing stages and flows in the project area during the design storm, the 100-year, 72 hour event.

The proposed features within the Recommended Plan were included within the HEC-RAS model geometric file and include canal and berm improvements, a spreader swale, multiple water control structures, and a flow-through marsh (shallow reservoir) feature. These features were incorporated by performing multiple iterative modifications to the geometric file for the purpose of refining feature sizes and location and are shown in **Figure 19**. The proposed features and model simulations performed are described below.

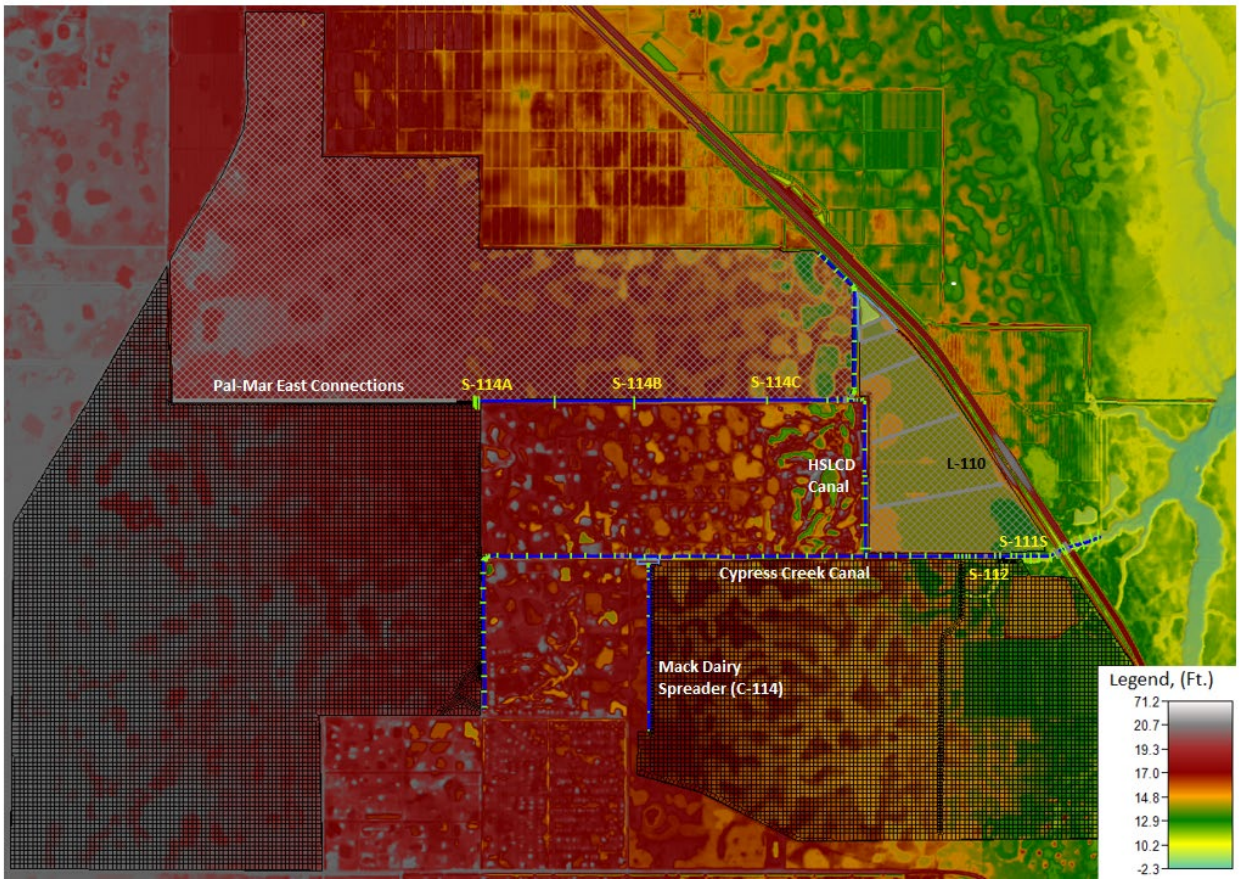

Figure 19. Recommended Plan Model Geometric Data

Mack Diary spreader swale (C-114): The spreader swale cross section was determined by simulating an upstream boundary condition equal to the maximum pump (S-113N) discharge rate. The cross section of the swale was modified until various objectives including: achieving flow velocities of less than 1 ft/s within the swale, maintaining appropriate overflow elevations above the swale into the depressional wetlands between 0.1 to 0.5 ft, and minimizing overflow velocities to maintain vegetation coverage.

Pal-Mar East connection with Nine Gems: The Pal-Mar East and Nine Gems connection was represented by a 2D flow area connection. Various configurations for connection were simulated to determine an

appropriate balance between earthwork quantities, hydrologic reconnection and redistribution of flow from Pal-Mar East to Nine Gems. The hydrologic reconnections were selected at locations where the southern Nine Gems canal has cut-off areas of historical lower lying wetlands within the Nine Gems and Pal-Mar East properties. These locations were selected to help promote sheetflow from Pal-Mar East to Nine Gems to facilitate rehydration and were sized to facilitate the transfer of volume without over-inundation of Nine Gems from Pal-Mar during larger storm events. Velocities during the 100-year, 72 hour simulation were limited to approximately 1.5 ft/s upstream and downstream of the connection.

Nine-Gem Culverts (S-114A, S-114B, and S-114C): The sizing and number of barrels located along the southern Nine Gems canal (remainder) were sized to help discharge flood waters and minimize peak water surface elevations within Nine Gems while maintaining an appropriate amount of discharge as to not impact downstream conditions within the HSLCD canal and Cypress Creek Canal. The downstream structure, S-114D, has an estimated maximum discharge capacity of 500 cfs and therefore the combined design discharge of S-114A, S-114B, and S-114C was 450 cfs, to allow additional discharge capacity for seepage and direct precipitation into the remaining portion of the southern Nine Gems canal.

Nine Gems levee (L-111): It was determined that the existing conditions could not appropriately contain the 100 year-72 hour storm event on site and uncontrolled discharge would likely occur from the site into adjacent canals (Southern Nine Gems canal or HSLCD canal). This is not appropriate as it could cause flooding to adjacent neighborhoods. Therefore, L-111 was proposed to retain water on site that will be discharged in an appropriate and controlled manner through the S-114A through C structures. The elevation of the proposed L-111 was designed to be two ft in excess of the modeled maximum water surface elevation resulting from the design storm, within the 2D flow area.

Canal work: The cross sectional template for the new southern HSLCD canal and modification to the Cypress Creek Canal was determined based on maintaining appropriate canal velocities (less than 2.5 ft/s), freeboard (3 ft) and flow capacity while minimizing amount of earthwork required. Flow velocities within the Cypress Creek Canal were noted as having velocities up to 3.5 ft/s within the central portion of the channel, and therefore the appropriate geotechnical material was added to the channel bottom and side slopes for erosion protection.

Gulfstream Flow-through Marsh Features (L-110, S-111S): The embankment design and discharge structure were initially performed outside of HEC-RAS following design guidance for low hazard impoundments, as outlined in the DCM-2 (SFWMD, 2006). Initial geometries were input and simulated in HEC-RAS to determine the appropriate relationship of the stage-storage and discharge capabilities. Refinements were made to ensure the discharge structure capabilities included both environmental and flood discharge capabilities. The environmental targets were specified as 30cfs whereas the flood discharge capabilities were designed to minimize the peak water surface elevation as reasonably as possible while ensuring an appropriate discharge amount from the spillway as to not cause adverse impacts downstream (targeting approximately 250 cfs maximum discharge). Additional model simulations were performed to determine the appropriate embankment height and freeboard above the maximum water surface elevation resulting from the design storm.

Cypress Creek Discharge Structure (S-112): The structure was modeled as an inline structure with ogee weir crest. The spillway geometry was modified to ensure that upstream water surface elevations were either improved or not impacted above existing conditions. Model simulations results and ogee spillway geometries were confirmed using external spreadsheet models based on formulas found in Engineering Manual (EM) 1110-2-1603, Hydraulic Design of Spillways.

C.2.6 Model Results

HEC-RAS model simulations were run to compare the water surface elevations and flow rates for the existing condition and the with-project condition for the entire Flow-way 3 western area. There was concern that the addition of a control structure within the Cypress Creek Canal and other associated project features could cause stages within the project area to rise above that of existing conditions. Special consideration was given to the performance within Cypress Creek Canal due to the proximity of the neighboring communities. The model simulations evaluated varying structure invert elevations and sill widths to ensure that the proposed spillway structure did not cause adverse impacts. The modeling results are shown below at varying locations of interest.

Multiple locations along the Cypress Creek Canal were used for comparison between the existing and with-project conditions. Two cross section graphics are included to illustrate the difference between the existing condition and the Recommended Plan maximum water surface elevations. **Figure 20** illustrates resulting maximum water surface elevations for the existing condition and Recommended plan at Cypress Creek Canal cross section 19000, which is located just downstream of WCS #1. Similarly, **Figure 21** shows maximum water surface elevations for both simulations at cross section 7000, which is located just upstream of the new proposed HSLCD canal connection. As can be seen from the figures, the with-project condition (pink line) does not cause adverse flooding impacts within Cypress Creek Canal. Water surface elevations show a decrease of approximately 1+ ft in water surface elevation from existing conditions.

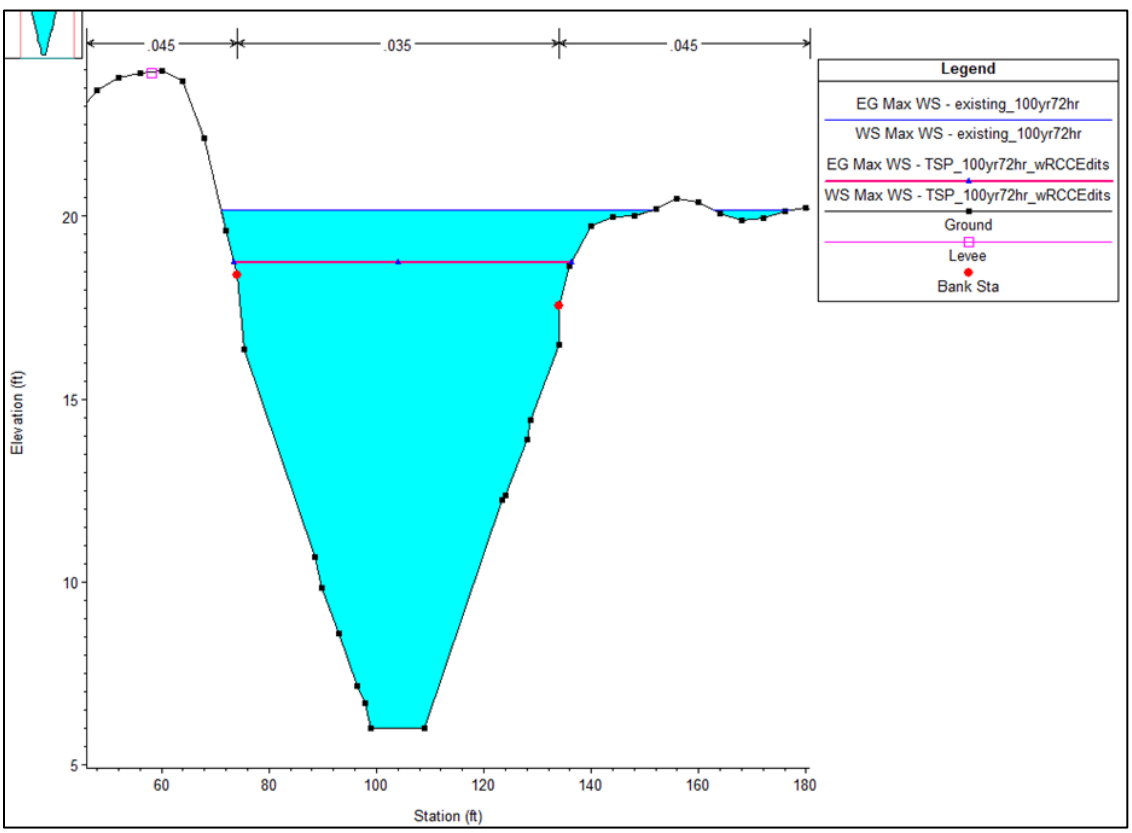

Figure 20. Cross Section 19000 in Cypress Creek Canal Comparing Existing and With-Project Water Surface Elevations

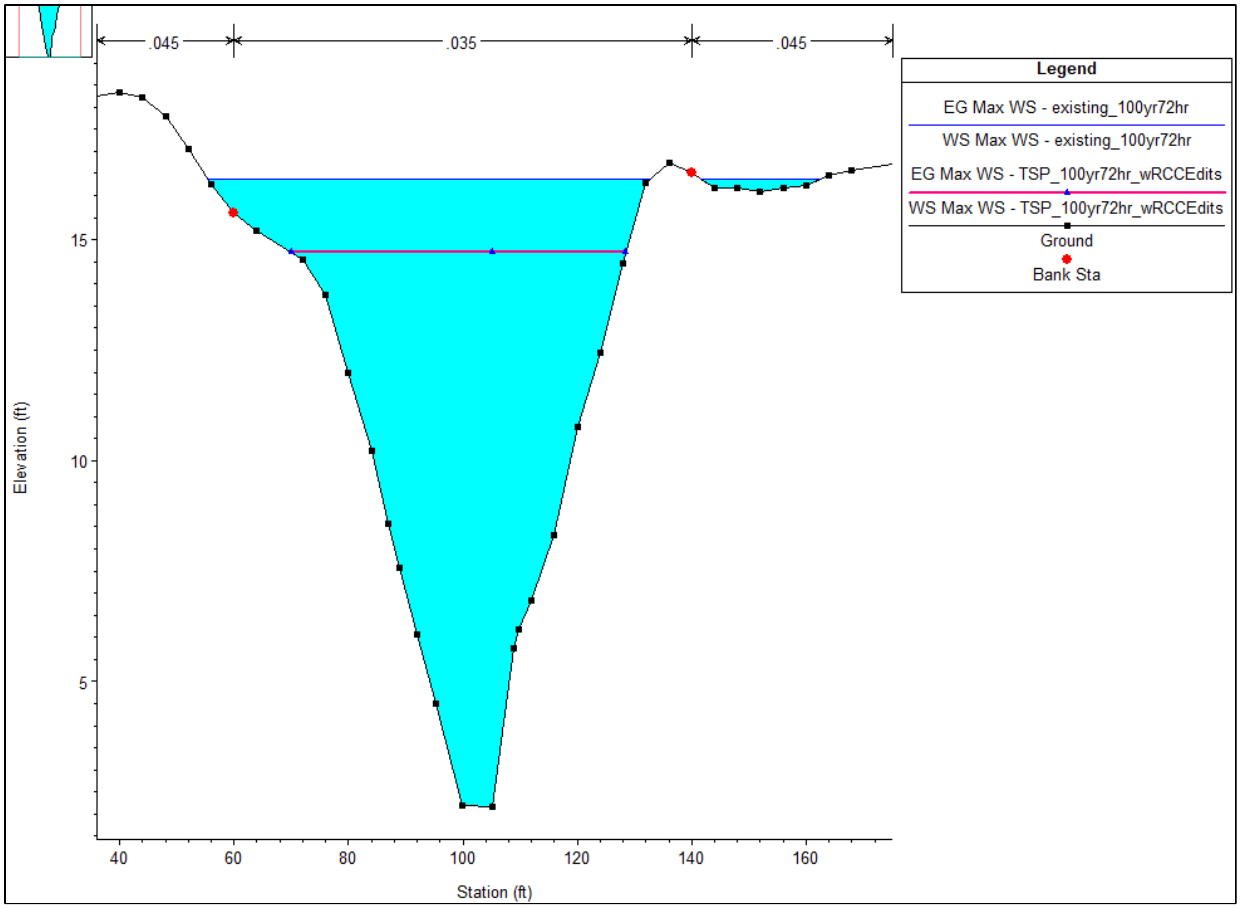

Figure 21. Cross Section 7000 in Cypress Creek Canal Comparing Existing and With-Project Water Surface Elevations

The Recommended plan included modification of the HSLCD southern "dog-leg" portion of the canal due to the proposed Gulfstream West Flow-through Marsh. The cross section and resulting water surface elevation for the newly proposed canal is shown in **Figure 22**.

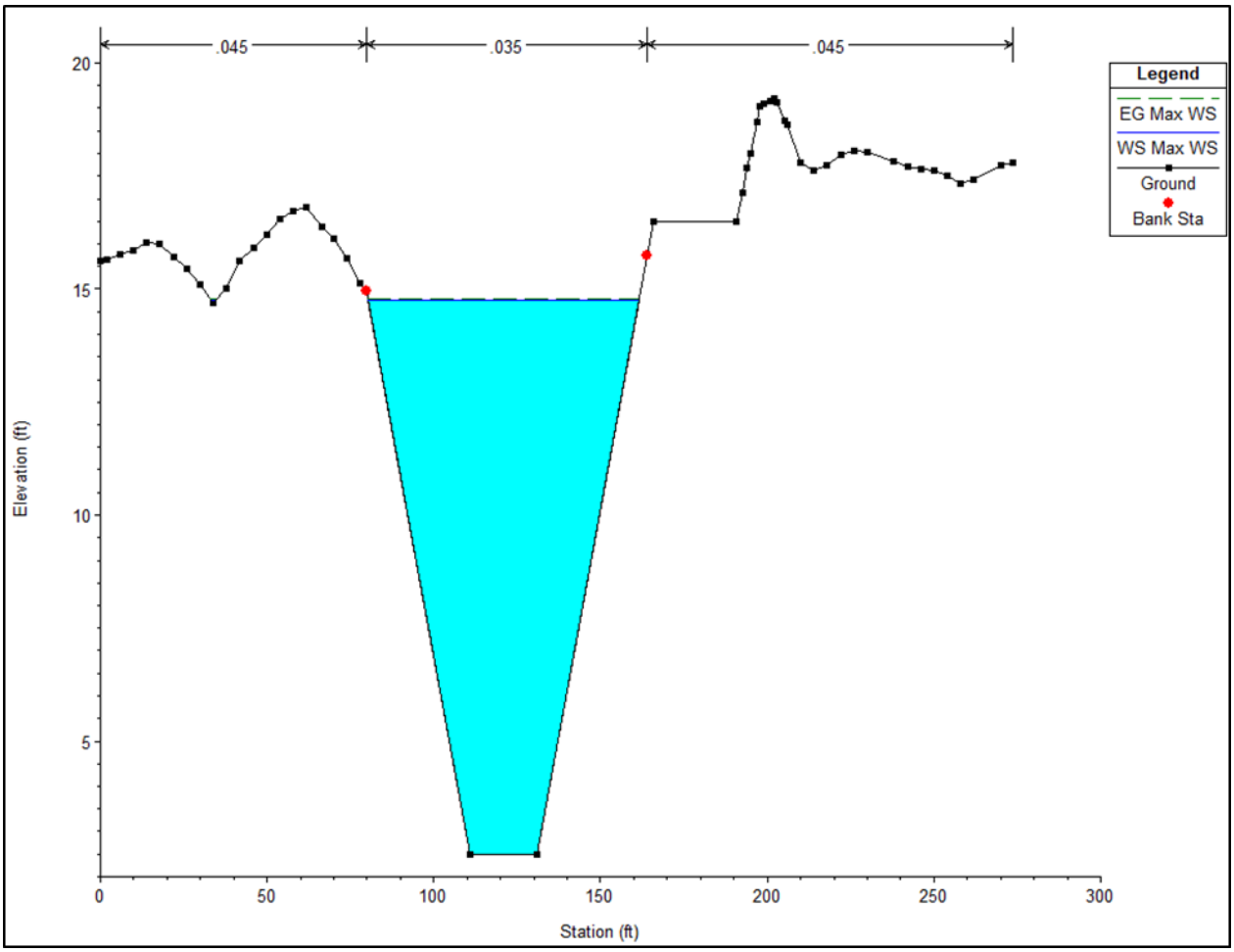

Figure 22. Recommended Plan Cross Section for Southern HSLCD Canal

The stages within the Pal-Mar East (Culpepper and Nine Gems) area were analyzed using the RAS Mapper function to determine the maximum water surface elevation within the grid cells of interest. The existing conditions model resulting water surface within Culpepper was approximately 21.0 ft NAVD88 and 18.6 NAVD88 within Nine Gems. In the Recommended Plan model simulation, three 2-D connections, approximately 50 ft in length each, were simulated to represent the proposed connection between the Nine Gems area and the rest of Pal Mar East. These connections resulted in additional water flowing from Culpepper to Nine Gems. The resulting maximum water surface elevations were 20.8 ft NAVD88 and 19.1 ft NAVD88 in Culpepper and Nine Gems, respectively.

The Gulfstream west project feature was simulated using a 2D flow area with an assumed starting water surface representing the full pump capacity operations. Therefore, the starting water surface elevation was determined by equating the maximum pump inflow equal to the associated head water elevation of the discharge structure. The design storm was then simulated and a maximum water surface elevation was determined. Per the Design Criteria Memorandum 2 and other design guidance, 3 ft of freeboard was added above the maximum water surface elevation.

C.3 Flow-way 3, Eastern Project Features

C.3.1 Model Domain

The model domain lies within southeastern Martin County, and contains two small man-made canals, Hobe Grove Ditch and Jenkins Ditch, which both contribute to the northwest fork of the Loxahatchee River.

The Hobe Grove Ditch is located downstream of HSLCD Unit 3 outfall, east of I-95 and Gulfstream East property. The Gulfstream East property is a former agricultural land that is proposed to be re-graded to promote sheetflow and rehydration of the property while providing additional water to the historic Moonshine Creek. The historic Moonshine Creek, south of Hobe Grove Ditch, was also represented. The model domain for the Hobe Grove Ditch related features can be seen in **Figure 23**.

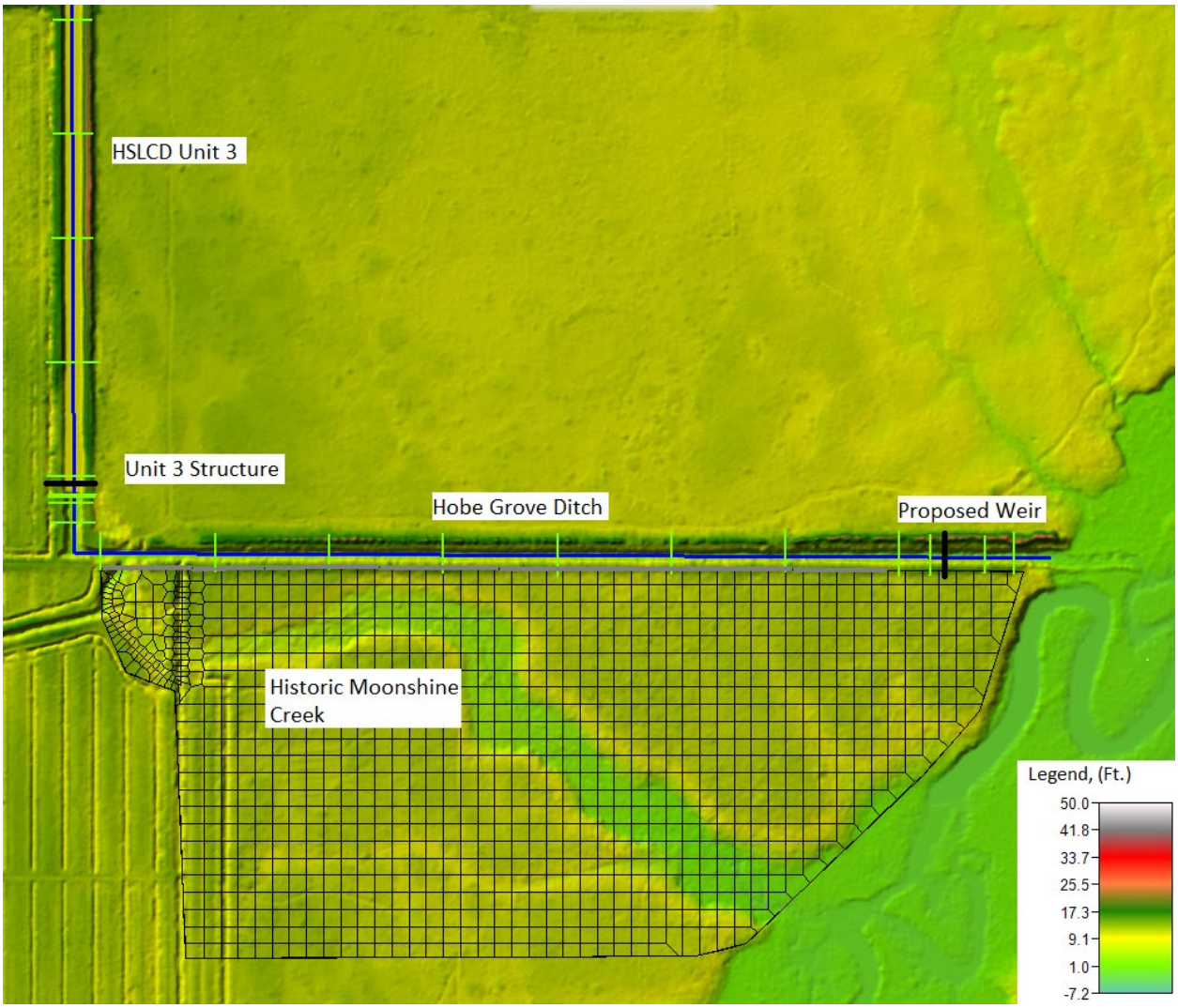

Figure 23. Flow-way 3 Eastern Features, Hobe Grove Ditch and Related Areas Model Domain

Jenkins Ditch is located within the Kitching Creek watershed and aids in conveying flow through a developed portion of the watershed. Jenkins Ditch receives flow from the northern portion of the watershed, including the Kitching Creek Central Flow-way Restoration Project. The model domain, shown in **Figure 24**, contains a portion of the Jenkins Ditch, south of the residential area until its termination into the natural Kitching Creek.

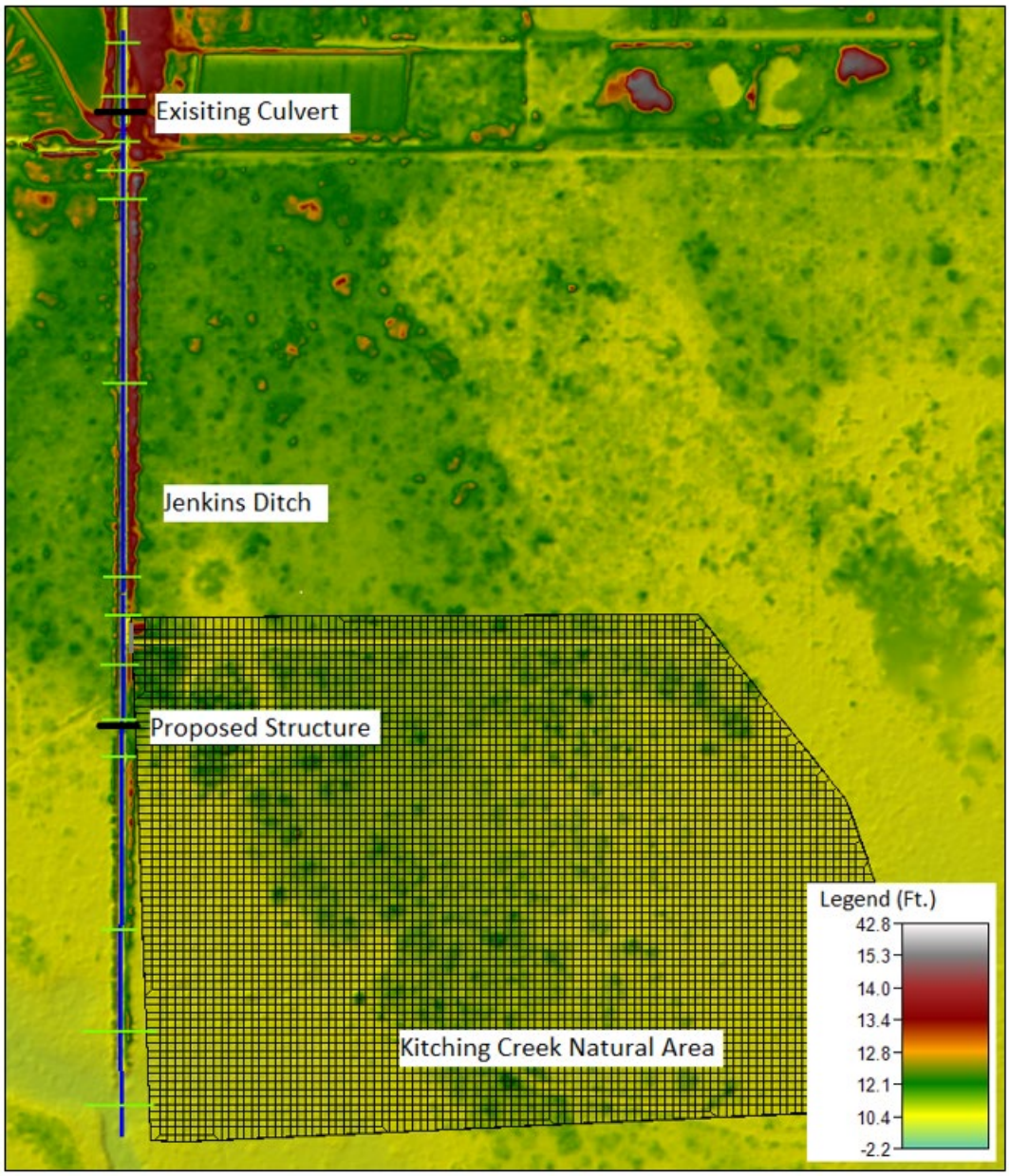

Figure 24. Flow-way 3 Eastern Features, Jenkins Ditch Model Domain

C.3.1.1 Model Mesh

The model mesh was developed from a terrain model developed from the LiDAR data for Martin County, Florida. The proposed project features added into the terrain as 2D areas were the historic Moonshine creek and the proposed spreader berm adjacent to Jenkins Ditch. The 2D area within Moonshine had a cell spacing of 75 ft and breakline near the Moonshine connection with Hobe Grove as 25-50 ft spacing. The cell size for the spreader system in Kitching Creek was selected as 25 ft.

C.3.2 Boundary Conditions

The upstream boundary condition for the Hobe Grove Ditch was the maximum permitted discharge from HSLCD Unit 3. No information was available regarding the appropriate maximum permitted discharge or design capabilities of the Jenkins Ditch, therefore the upstream boundary condition was assumed to be bank full flow at 150 cfs. The downstream boundary condition for both models is free flowing and was set as a normal depth, specified using an estimated friction slope. The normal depth for the Jenkins Ditch and Hobe Grove Ditch were 0.0005 ft/ft and 0.001 ft/ft, respectively, and were estimated using the LiDAR data.

C.3.3 Model Parameters

C.3.3.1 Channel Cross-Section and Profile

Similar to the Western Flow-way 3 model, the cross-sections cut from the DEM data did not contain information below the water surface. Channel depth and width were estimated using best professional judgment as no survey or bathymetry data was available. Cross section spacing was set at approximately 500 ft for Hobe Grove Ditch and varied for the Jenkins Ditch.

C.3.3.1.1 Canals

The Hobe Grove Ditch and Jenkins Ditch were represented using 1D cross sections. The recommended plan included construction of a swale, C-116, splitting off from the Jenkins Ditch. The swale geometry was estimated based on upstream inflow rates, associated velocity constraints and known topographic conditions. The cross section of the swale was modified until various objectives including achieving flow velocities of less than 1 ft/s within the swale and maintaining appropriate overflow elevations above the swale into the depressional wetlands between 0.1 to 1.0 ft, with overflow velocities minimized to maintain vegetative growth. This geometry was estimated as a 1D cross section and then input into the terrain data using the RAS mapper function, ultimately becoming a part of the 2D mesh for representation in the model, as shown in **Figure 25.**

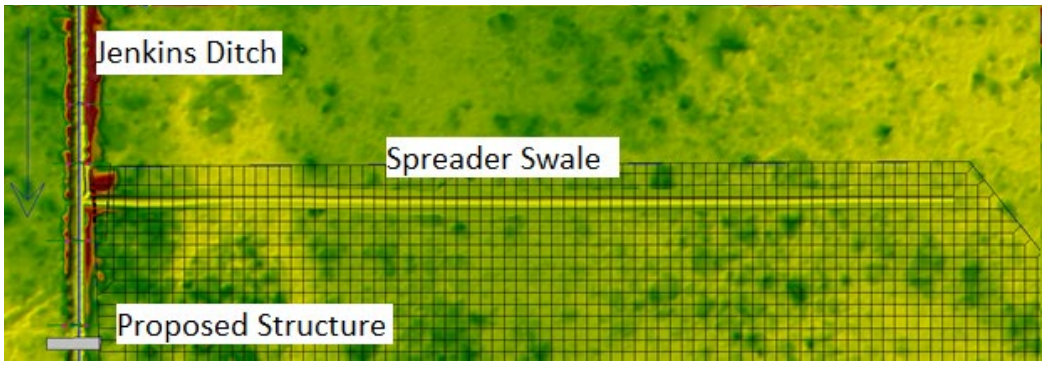

Figure 25. Spreader Swale in Jenkins Ditch, Modified Terrain Data

C.3.3.2 Manning's "n" Values

Manning's n values within the 1D cross sections of Hobe Grove Ditch and Jenkins Ditch were represented using horizontally varying Manning's values based on n-values from Chow, 1959. Field observations and aerial imagery were used to estimate channel values between 0.03 and 0.05, depending on the vegetation presence. Additionally, the overland areas were represented using a default Manning's value of 0.07 representative of Emergent Herbaceous Wetland land use coverage. The swale was also associated with a Manning's value of 0.07.

C.3.3.2.1 Bridges

No bridges were required in either model.

C.3.3.2.2 Culverts

Two culverts were included in the Jenkins Ditch model geometry as existing structures, using field observations and measurements as input. The northern culvert was estimated as a 6 x 4 ft ellipsoid culvert, 35 ft in length at an invert of 6.2 ft NAVD88. This culvert was modeled in both the existing and with-project condition. The second culvert structure is located approximately 2000 ft south of the first, and is used for access to the Johnathan Dickinson State Park. Under existing conditions, the culvert was estimated to be an 8.0 by 5.5 ft ellipsoid culvert structure, 35 ft in length with an invert of 6.2 ft NAVD88. Both culverts under existing conditions are corrugated metal, with some debris located along the bottom of each pipe, each having no headwall (pipe projects from fill). Therefore, the Manning's value of 0.024 and entrance head loss of 0.9 was selected. During the with-project conditions this culvert was replaced with a 7 by 5 ft gated concrete box culvert, S-116.

C.3.3.2.3 Pumps

No pumps were required in either model.

C.3.3.2.4 Lateral Structures

A lateral structure was used in each model to provide connection along the 1-D canal reaches to connect to the 2-D flow areas. The lateral structure in the Hobe Grove Ditch model connected the canal to the historic Moonshine Creek to allow flow from the canal to enter the 2D area. Similarly, the Jenkins Ditch lateral structure allowed flow to propagate down the spreader system when water stages allowed. These

lateral structure were set to match the average elevation of the surrounding ground including the designed overtopping area for each connection (i.e. earthwork necessary to connect the proposed features). A broad crest weir coefficient of 0.9 was used, based on guidance given in the HEC-RAS 2-D reference manual (2016).

C.3.3.2.5 Inline Structures

The culverts within Jenkins Ditch were added using inline structures for the existing and Recommended Plan Project conditions (description of culvert sizes included in Section C.3.3.2.2). The northern culvert embankment was modeled using a weir coefficient of 2.6, width of 25 ft and elevation of 13.3 ft NAVD88. The southern embankment was modeled at a width of 25 ft, weir coefficient of 2.6 and elevation of 12.0 ft NAVD88. The southern culvert geometry was modified while the embankment geometry was kept similar for the existing and Recommended Plan model simulations.

For the Hobe Grove Ditch model, inline structures were used to represent the Unit 3 control structure and Recommended Plan proposed un-gated weir, S-117. The Unit 3 control structure has four concrete discharge bays, each 4.5 wide, with a sheet pile wall on both sides to the canal banks. The discharge bay sill elevation is at 2.5 ft NAVD88 whereas the sheet pile overflow weir is at 10.5 ft NAVD88. Stop logs may be added to the discharge bays to vary the upstream water surface elevation but no stop logs were simulated within this modeling application.

C.3.3.3 Levees

No levees were included in either model. Areas of higher elevation from spoil, or otherwise, were included in the 1D cross sectional geometry and were estimated using the terrain data.

C.3.3.4 Storage Areas

The Hobe Grove Ditch model simulated the historic Moonshine Creek as a 2D area. The Jenkins Ditch model used a 2D flow area to represent the portion of Kitching Creek that was influenced by the spreader system.

C.3.3.5 Additional Detail

C.3.3.5.1 Calculation Options and Tolerances

No changes to calculation options and tolerances were applied. The 1D/2D Unsteady Flow option parameters were not adjusted for better performance in the unsteady flow model.

C.3.3.5.2 Numerical Stability

Unsteady flow testing yielded some numerical instabilities near the inline structures but no significant errors or warnings were reported for the simulations. These instabilities were reduced through reducing the computation time step interval and decreasing the HTab increment. The target for numerical stability for the developed HEC-RAS model was to minimalize the model mass balance error. All simulations for both models were developed to have mass balance error lower than or equal to 1 percent.

C.3.3.5.3 Ineffective Flow and Levees

Ineffective flow areas were used in the HEC-RAS cross sections at bridges and inline structures that prevents HEC-RAS from using the entire cross section for active flow calculations. These areas are used to describe a portion of the cross section in which the water velocity is close to zero.

C.3.3.5.4 Junctions

No Junctions were necessary in either model.

C.3.4 Model Calibration

There is no observed hydrologic data, including stage or flow, which can be used for model calibration. The HEC-RAS model developed was not calibrated. Modeling assumptions followed standard modeling practice, including using published Manning's values, using geometric data from either published reports or through field observation and measurements.

C.3.5 Model Application for Design

The 2014 Existing Conditions baseline model was simulated to provide an estimate of existing stages and flow in the project area to ensure the features within the Recommended Plan caused no adverse impacts to surrounding areas. The proposed features within the Recommended Plan were included within the HEC-RAS model geometric file and include a new weir and earthwork for the Hobe Grove Ditch and a new water control structure and spreader swale for Jenkins Ditch. These features were incorporated by performing multiple iterative modifications to the geometric file for the purpose of refining feature sizes and location. The proposed features and model simulations performed are described below.

Jenkins Ditch Water Control Structure (S-116): The water control structure within Jenkins Ditch, downstream of the proposed spreader swale, was sized by simulating an unsteady inflow hydrograph with a maximum bank full flow and comparing upstream water surface elevations against the existing conditions model. Since the S-116 is a dual purpose structure, it was designed to provide additional control of water surface elevations within the canal through the addition of a telemetry operated gate as well as have adequate capacity to discharge during larger storm events. Structural design refinements were performed to ensure upstream water surface elevations did not increase over existing conditions. Therefore, the geometric properties of S-116 were sized such that the capacity was slightly larger than the existing structure, resulting in a 7 ft by 5 ft box culvert. The S-116 culvert design is a concrete box culvert with upstream and downstream wing walls, thereby having an entrance loss coefficient of 0.5 and estimated Manning's value of 0.013.

Kitching Creek Spreader Swale (C-116): The spreader swale cross section was determined using an iterative modeling approach ensuring the expected upstream water surface elevations within the Jenkins Ditch would flow into the spreader, while minimizing flow velocities and earth work necessary. The cross section of the swale was modified until various objectives including achieving flow velocities of less than 1 ft/s within the swale and maintaining appropriate overflow elevations above the swale into the depressional wetlands between 0.1 to 0.75 ft with overflow velocities less than 1.0 ft/s during the storm event.

Hobe Grove Ditch Weir (S-117): The S-117 structure was proposed to help increase water surface elevations upstream of the ditch to promote the redistribution of flow into the historic Moonshine Creek and decrease peak discharges into the northwest fork. The structure was modeled as an inline structure with broad weir crest, using a weir coefficient of 2.6. The design objective was to maximize the weir elevation while ensuring that the water surface elevations or the discharge capacity upstream of the HSLCD Unit 3 structure, were not impacted above existing conditions. The final weir geometry was a 40 ft wide notched weir at elevation 6.0 NAVD88.

Historic Moonshine Creek: A 1D cross section and river reach was estimated based on the necessary bottom elevation and cross sectional area to promote a redirection of flow from the Hobe Grove Ditch into Moonshine Creek. The cross sectional area design under the design discharge was to limit flow velocities to 1.5 ft/s with target depths of 2-3 ft in the Moonshine Creek area. Once the 1D cross sections were finalized, RAS mapper was used to modify the base terrain to a terrain version that represents the proposed earthwork for the 2D flow area.

C.3.6 Model Results

The Hobe Grove Ditch modeling results are shown in **Figure 26** through **Figure 28**. The cross section upstream of the HSLCD unit 3 control structure is shown in **Figure 26** and illustrates that there is no appreciable difference in water surface elevations between the existing condition and Recommended Plan condition (14.12 and 14.11 ft NAVD88, respectively). **Figure 27** demonstrates that the maximum depth of Moonshine Creek under existing conditions is less than 1 ft in most of the natural area. The purpose of restoring a depth of 1-3 ft throughout the creek with the Recommended Plan can be seen in **Figure 28**, which illustrates the hydrologic reconnection and improved depth in most of the natural area with the Recommended Plan earthwork improvements.

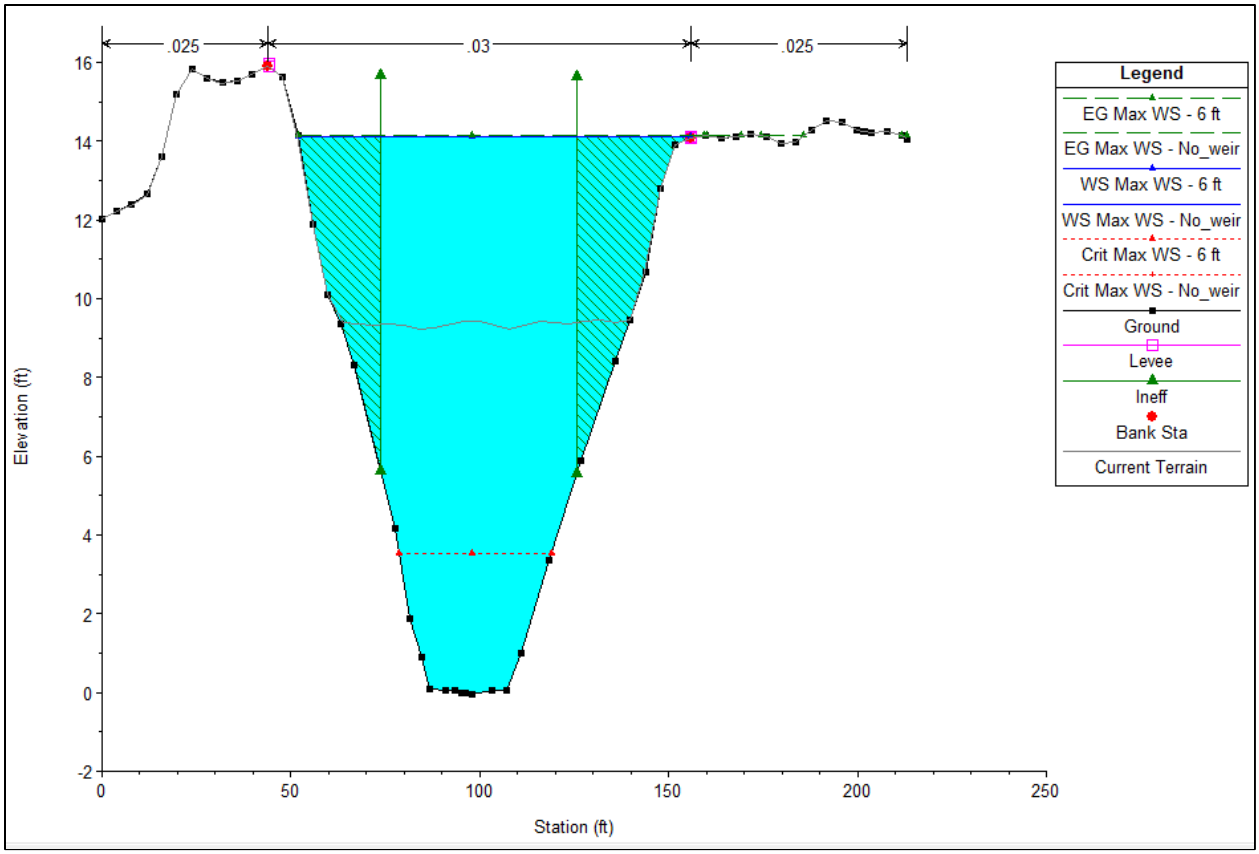

Figure 26. Hobe Grove Ditch, Upstream of HSLCD Unit 3 Control Structure Comparing Existing and With-Project Water Surface Elevations

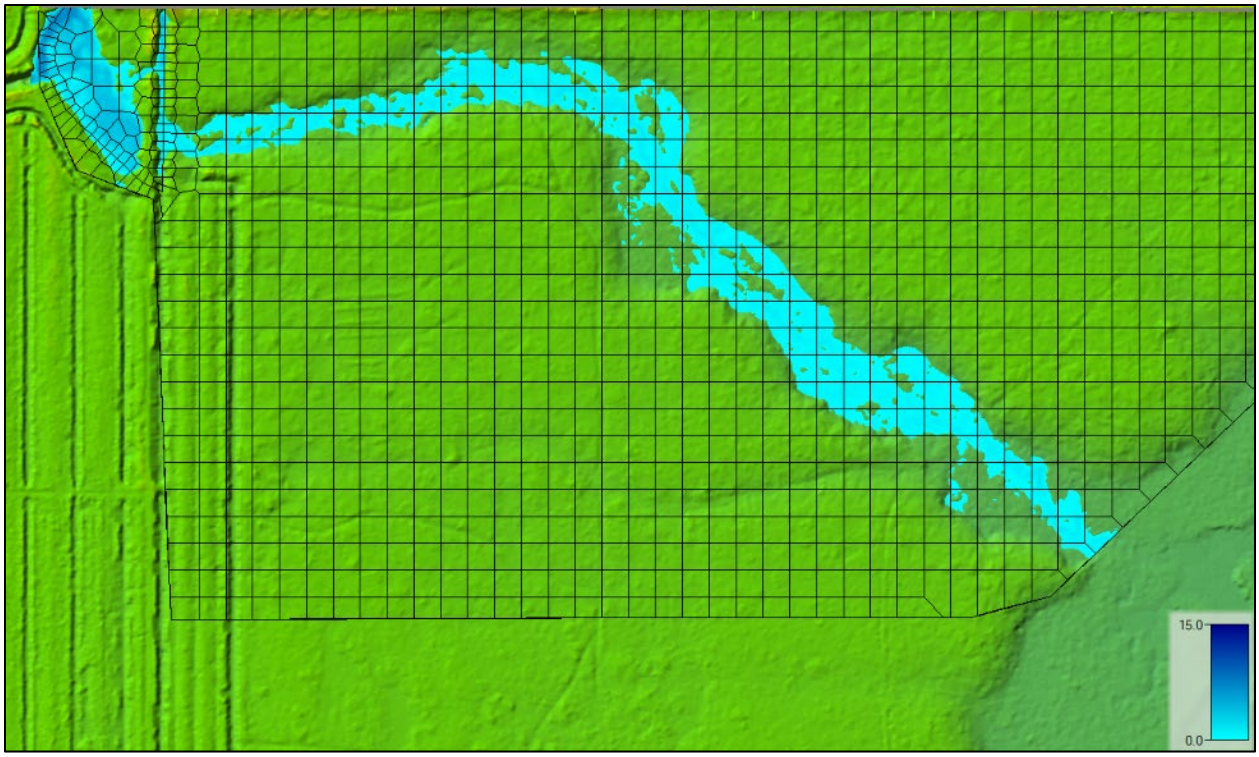

Figure 27. Moonshine Creek Existing Conditions Depths

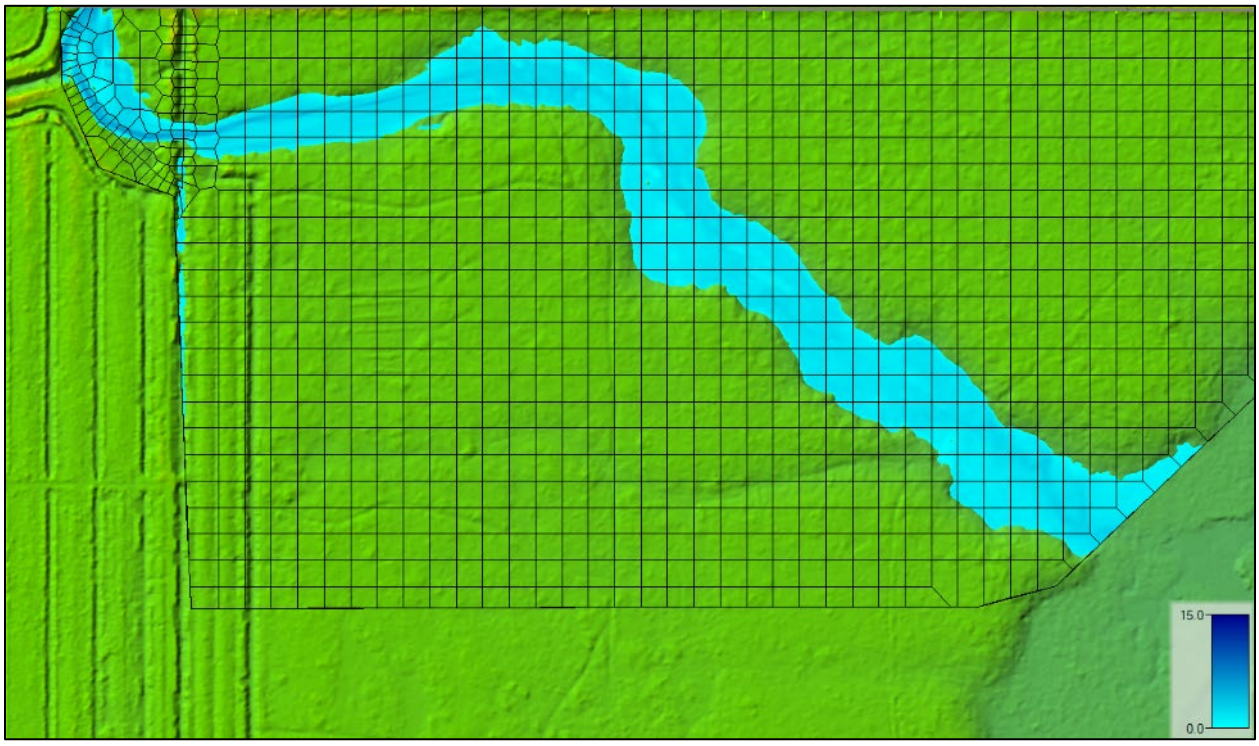

Figure 28. Moonshine Creek Recommended Plan Conditions Depths

The Jenkins Ditch modeling results, shown in **Figure 29**, represents the cross section located upstream of the Recommended Plan water control structure, S-116, which is a gated box culvert. During the simulation runs the gate was fully open, allowing maximum discharge through the structure. The figure represents a decrease in water surface elevation for the with-project condition (in pink) over the existing conditions model (blue line) because a portion of the volume enters the spreader canal system, as illustrated in **Figure 30**.

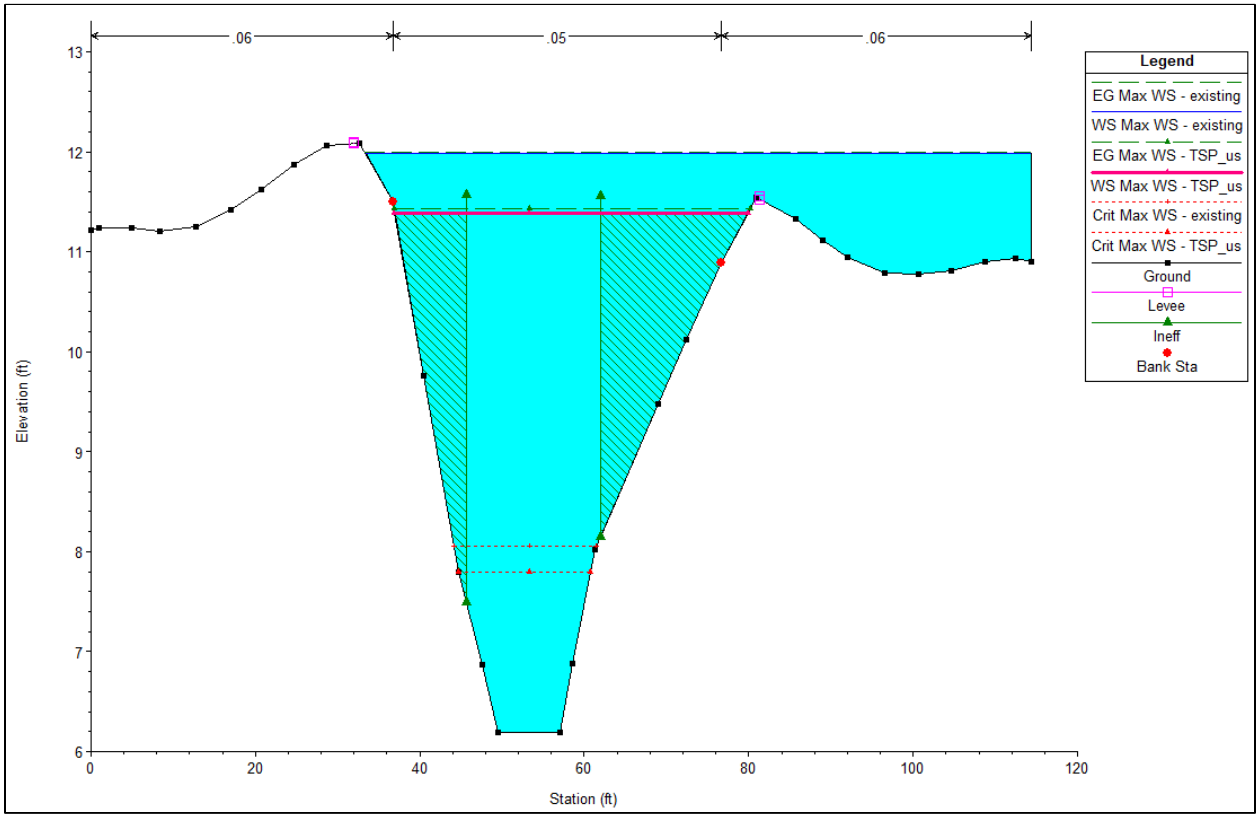

Figure 29. Jenkins Ditch Cross Section Upstream of S-116 Comparing Existing and With-Project Water Surface Elevations

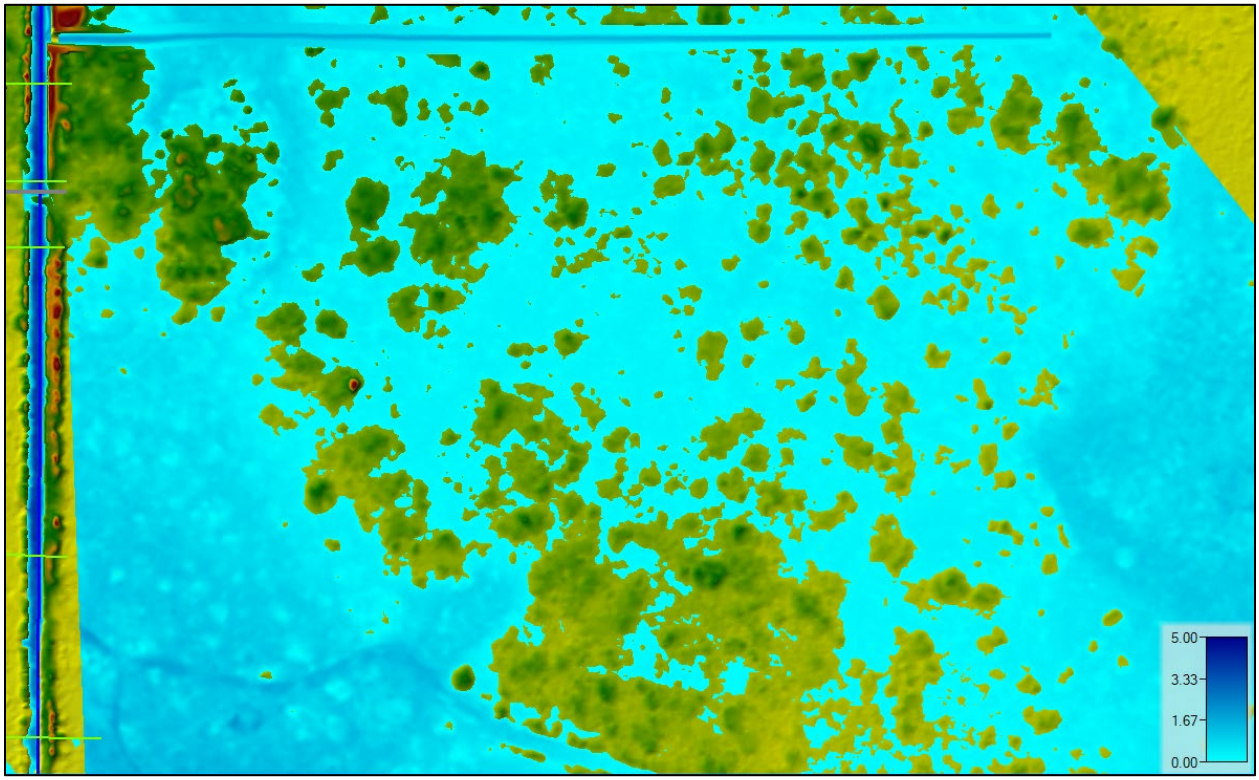

Figure 30. Spreader Swale and Maximum Associated Water Surface Depths within Kitching Creek

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