APPENDIX A
ENGINEERING APPENDIX

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ACRONYM LIST

ACI	American Concrete Institute
AGMA	American Gear Manufacturers Association
ALARP	As Low As Reasonably Practicable
APPZ	Avon Park Permeable Zone
ASCE	American Society of Civil Engineers
ASHRAE	American Society of Heating, Refrigeration, and Air-conditioning Engineers
ASPE	American Society of Plumbing Engineers
ASR	Aquifer Storage Recovery
AST	Above-ground Storage Tank
ASTM	American Society of Testing & Materials
ATR	Agency Technical Review
BEP	Backward Erosion Piping
BLS	Below Land Surface
BOR	Bureau of Reclamation
CCNA	Cypress Creek Natural Area
C&SF	Central & Southern Florida
CEPP	Central Everglades Planning Project
CERP	Comprehensive Everglades Restoration Plan
CPVC	Chlorinated Polyvinyl Chloride
DCM	Design Criteria Memorandum
DOT	Department of Transportation
DDR	Design Documentation Report
ECB	Engineering & Construction Bulletin
ECB	Existing Conditions Baseline
EIS	Environmental Impact Statement
EL	Elevation
EOC	End of Construction
EM	Engineering Manual
ER	Engineering Regulation
FAS	Floridan Aquifer System
FDEP	Florida Department of Environmental Protection
FDOT	Florida Department of Transportation
FEMA	Federal Emergency Management Agency
FOS	Factor of Safety
FTM	Flow-Through Marsh
FWO	Future Without
GWP	Grassy Waters Preserve
HEC-RAS	Hydrologic Engineering Center – River Analysis System
HEI	Heat Exchanger Unit
HH&C CoP	Hydrology, Hydraulics & Coastal Community of Practice
HI	Hydraulic Institute
HPC	Hazard Potential Classification
HSLCD	Hobe-St. Lucie Conservancy District
HTRW	Hazardous, Toxic, and Radioactive Waste
HVAC	Heating, Ventilation, and Air Conditioning

ICU	Intermediate Confining Unit
IEPR	Independent External Peer Review
ITID	Indian Trail Improvement District
KRASR	Kissimmee River Aquifer Storage Recovery system
LECSR-NP	Lower East Coast Sub-Regional Model – North Palm
LFRD	Load Resistance Factor Design
LiDAR	Light Detection and Ranging
LP	Liquid Propane
LRWRP	Loxahatchee River Watershed Restoration Project
LT	Long Term
MPSEG	Major Pump Station Engineering Guidelines
MWSL	Maximum Water Storage Level
NAD83	North American Datum 1983
NAVD88	North American Vertical Datum 1988
NFPA	National Fire Protection Association
NFSL	Normal Full Storage Level
NGVD29	National Geodetic Vertical Datum 1929
NOAA-NGS	National Oceanic and Atmospheric Administration – National Geodetic Survey
NRCS	Natural Resources Conservation Service
NWFLR	Northwest Fork of the Loxahatchee River
OMRR&R	Operation, Maintenance, Repair, Replacement, & Rehabilitation
PDT	Project Delivery Team
PEC	Probable Effect Concentration
PED	Pre-construction Engineering Design
PFMA	Potential Failure Mode Analysis
PFMs	Potential Failure Modes
PIR	Project Implementation Report
PMF	Potential Maximum Flood
PMP	Probable Maximum Precipitation
PLC	Programmable Logic Unit
PVC	Polyvinyl Chloride
QRA	Qualitative Risk Assessment
RCP	Reinforced Concrete Pipe
RCRA	Resource Conservation and Recovery Act
RDD	Rapid Draw Down
RTU	Remote Terminal Unit
SAJ	Jacksonville District
SAS	Surficial Aquifer System
SCADA	Supervisory Control and Data Acquisition
SCTL-LSW	Soil Cleanup Target Level - Leachability to Surface Water
SFWMD	South Florida Water Management District
SFWMM	South Florida Water Management Model
SMMS	Statewide Model Management System
SPLP	Synthetic Precipitation Leaching Procedure
SPT	Standard Penetration Test
SQAGs	Sediment Quality Assessment Guidelines
SQRA	Semi-Quantitative Risk Assessment

SWCTL	Surface Water Cleanup Target Level
TDH	Total Dynamic Head
TEC	Threshold Effect Concentration
UFA	Upper Floridan Aquifer
USACE-SAJ	US Army Corps of Engineers – Jacksonville District
USCS	Universal Soil Classification System
USGS	US Geological Survey
UST	Underground Storage Tank
UV	Ultraviolet
WAM	Wave Assessment Model
WCS	Water Control Structure
WMA	Wildlife Management Area
WMO	World Meteorological Organization
WRAP	Wetland Rapid Assessment Procedure
WTP	Water Treatment Plant

UNITS

AC-FT	acre-feet
CFS	cubic feet per second
CFS/LF	cubic feet per second per linear foot
CM	centimeter
CM/S	centimeter per second
CY	cubic yards
FT	foot, feet
FT/S	foot per second
HZ	Hertz
IN	inch
KV	kilovolt
MGD	million gallons per day
PCF	per cubic foot
PSF	per square foot
PSF/DEG	per square foot per degree
RPM	revolutions per minute
V	volt
VA	volt amperes

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A ENGINEERING DESIGN APPENDIX

The Engineering Appendix of the Project Implementation Report and Environmental Impact Statement (PIR-EIS) is a comprehensive record of the technical support given by the USACE SAJ Engineering Division to the Loxahatchee River Watershed Restoration Project (LRWRP), with technical information and analyses provided by the following engineering disciplines: Civil, Geotechnical, Hydrology and Hydraulics, Structural, Mechanical, and Electrical. This appendix presents the specific component features and engineering requirements, focusing primarily on the hydraulic designs.

This final Engineering Appendix is organized by technical discipline, with sub-sections within each discipline that describe the proposed structures and features that, when constructed, will improve the wetland and streamflow environments of the Loxahatchee Slough. Each discipline includes the following general information: an overview of the features of the LRWRP Recommended Plan; overview status of engineering design activities and analyses; discussion of general construction procedures; overview of preliminary civil site design information; overview of geotechnical considerations and analyses; overview of hydrologic and hydraulic design and analyses; and documentation of the hydrologic modeling. Note that all structures are subject to change in type and capacity following full analysis during Pre-construction Engineering and Design (PED). For the summary of costs, cost considerations and assumptions, refer to **Appendix B** of the PIR-EIS.

A.1 Loxahatchee River Watershed Restoration Project Overview

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.

The project features are designed to increase flows to the Loxahatchee River particularly in the dry season, to improve seasonal timing and volume surface water sheet flow and wetland hydroperiods, and to reduce damaging stormwater flows in tributaries leading to the Loxahatchee River. These objectives are accomplished by modifying, improving, and supplementing existing water control structures along three west-to-east flow-ways.

Major project features will include a 9,500 acre-foot reservoir augmented with four Aquifer Storage Recovery (ASR) wells, improvements to culverts and a new operable weir on Cypress Creek Canal, plugging ditches to improve sheet flow in the Nine Gems tract, and a flow-through marsh to capture canal and sheet flow from the west. The purpose of LRWRP as a part of Comprehensive Everglades Restoration Plan (CERP) is to improve the quantity, timing, and distribution of water flows to and within the Everglades system by addressing the problems created by the Central and Southern Florida (C&SF) Project. Existing conditions are summarized in **Section 2**, “Existing and Future Without Conditions,” of the PIR-EIS.

A.2 Supporting Analyses for the Recommended Plan

The LRWRP formerly was part of other CERP ecosystem restoration efforts conducted by the SFWMD and the USACE. As a result, some project components, such as the C-18W reservoir, have large existing datasets and analyses. This section describes analyses that were conducted to support this project.

A.2.1 Dam Safety Considerations

Two features of the LRWRP store and convey water in proximity to developed areas: the C-18W reservoir, and the Gulfstream West flow-through marsh. Both features were evaluated using Hydrologic Engineering Center-River Analysis System (HEC-RAS) model simulations. C-18W reservoir is considered a high-hazard facility and therefore a Potential Failure Mode Analysis (PFMA) and qualitative risk assessment (QRA) was completed for this feature. SAJ performed a PFMA and QRA on July 15 through July 19, 2019 to satisfy the requirements in Chapter 21 (*Dam Safety Policy for Planning and Pre-Construction Engineering and Design*) of ER 1110-2-1156 and draft *Engineering and Construction Bulletin (ECB) Interim Approach for Risk-Informed Designs for Dam and Levee Projects*. The draft PFMA and QRA report was reviewed initially following the Discipline Quality Control and Product Quality Control review processes. After revision, the draft document was submitted to the Agency Technical Review (ATR) team, with comments placed in the review database DrChecks. ATR comments were evaluated, the report was revised, and the ATR was closed out in October 2019.

A.2.1.1 C-18W Reservoir

The C-18W reservoir will impound water for months at a time during each wet season, and will serve as a basin for ASR recharge and recovery water supply. The C-18W reservoir embankment is considered a dam according to criteria in USACE Engineering Regulation (ER) 1110-2-1156 based on total water volume impounded at full pool and duration of hydraulic loading. The proposed dam is considered a high-hazard facility according to ER 1110-2-1156 and Design Criteria Memorandum 1 (DCM-1, Hazard Potential Classification) due to the potential for life loss if the facility were to fail. The reservoirs proposed by the CERP Yellow Book, which are consistent with the proposed C-18W reservoir, are somewhat unique in that they are off-river, pumped systems with no unregulated inflow except for direct rainfall occurring over the footprint of the reservoir.

Dam breach modeling was completed at a single location on each reach of the embankment (south, southeast, north, and southwest) at the C-18W reservoir. The breaches on the east and west reaches were placed closer to the south side of the reservoir due to the proximity of existing and proposed residential developments to the south and east, respectively. The reservoir will be relatively shallow, with Normal Full Storage Level (NFSL) at 7.5 ft in depth. Modeling results show that flooding from breach at NFSL would be shallow, with depths generally less than 2 ft in the developed areas. Although residential areas and roadways may be flooded from a breach, most of the structures would not be flooded because the slightly elevated building pads keep them above the flood elevation. Additionally, there is a new residential development under construction located southeast of the proposed reservoir. This development will likely increase population at risk around the proposed reservoir by several thousand. Breach modeling to the east shows that at NFSL, only low-lying depressional wetland areas would be inundated in the vicinity of the future development. Therefore, life loss potential from breach also will be low at this proposed development. Breach modeling also was performed using the Probable Maximum Flood (PMF) stage. Modeling of the breaches at the PMF show extensive flooding around the reservoir from direct rainfall. Breach simulations at PMF stage result in no significant incremental flooding. Therefore, incremental life loss potential is also considered low for breach at PMF stage.

All identified Potential Failure Modes (PFMs) appear to have been addressed by the preliminary design for the project, and the geologic setting of this reservoir. Backwards Erosion Piping (BEP) PFMs were estimated to have the highest contribution to total incremental risk at this project. Although these PFMs were considered the highest risk, they are still characterized as having low annual probability of failure

due to the low head of the reservoir, and use of interior embankment seepage control features such as an interior chimney drain and blanket drain. These interior drainage features are estimated to sufficiently control exit gradients through the embankment. Additionally, the proposed design has 12.5 ft of freeboard that would prevent overtopping during the Probable Maximum Precipitation (PMP) event. Assuming the project is designed and constructed in accordance with the current state of the practice for Dam Safety, the C-18W reservoir is expected to have a low annual probability of failure and incremental risk below the societal tolerable risk threshold (defined in ER 1110-2-1156). Recommendations were provided in the risk assessment that should be considered during PED to further reduce risk with respect to As Low As Reasonably Practicable (ALARP) principles.

Lastly, risk will continue to inform the designers of this project as it progresses from feasibility-level design into a constructed project. A Semi-Quantitative Risk Assessment (SQRA) will be performed during the design phase, after pre-final construction plans and specifications are developed. This risk assessment will include an additional PFMA based on detailed design and will be used to make additional risk based modifications to the project design as needed. This risk assessment will be reviewed further and revised as necessary after construction is complete and as-built records are available. Lastly, the reservoir will include a first filling plan, which will require a slow, deliberate filling of the reservoir, with increases in pool timed to allow steady state conditions to develop and detailed inspection before further increasing the reservoir pool elevation. The risk assessment also will be revised as needed after first filling of the reservoir. This approach allows for risk to be conveyed to Design, Construction, and Operation Divisions at each stage of the project. A First Filling Implementation Plan will be developed before the first filling of the reservoir in order to monitor critical areas based on breach modeling performed as part of the risk management and assessment.

All risk assessments are For Official Use Only and will not be released to the public. Specific dam-related documents and calculations are presented later in this document, particularly in **Annex A-1**, Hydraulic Design, **Annex A-3** HEC-RAS Modeling, and **Annex B-1**, Geotechnical Design and Investigations.

A.2.1.2 Gulfstream West Flow-Through Marsh

An analysis of the dam breach and potential downstream impacts was conducted as outlined in DCM-1 during the Feasibility phase to verify the classification of the Gulfstream West Flow-through Marsh (FTM) as a low-hazard potential facility. DCM-1 states “In many cases the Hazard Potential Classification (HPC) is not apparent from a field reconnaissance (e.g. the presence of commercial or residential property nearby, or the surrounding area consists entirely of agricultural land), and detailed studies including dam breach analyses of various flood flow conditions are required to evaluate the incremental effects of dam failure when compared to flow conditions without a dam failure. Dam breach analysis is used to assess the economic damage and life loss potential based on dam breach model results that include inundation maps of the area downstream of a project under various flow conditions.”

The objective of the Gulfstream West FTM dam breach analysis is to determine whether a breach on the western side of the FTM during an extreme event would increase incremental impacts to the adjacent neighborhoods or areas downstream. The HPC determination is important as it governs the inflow design flood, freeboard and other standard design applications. The Hobe St. Lucie Conservancy District (HSLCD) canal, two berms, and a road are located between the flow through marsh and the neighborhood. The dam breach was simulated at the maximum water storage level (MWSL), and the results of the dam breach show that the flood wave from the Gulfstream West FTM flows into the HSLCD canal. The flood wave does not affect the developed area west of the FTM. Therefore, the Gulfstream West FTM has a Low Hazard

Potential Classification following this additional dam breach analysis. A low HPC represents no expected loss of human life and generally low economic, environmental, and lifeline losses. ER 1110-2-1156 states that if a dam is within the recommended alternative, all supporting and necessary required documentation, including a PFMA, will be identified and scheduled for completion during the PED phase. Due to the low hazard potential classification, the Project Delivery Team (PDT) determined that these items would be completed in PED.

A.3 Recommended Plan

The overall project objectives are three-fold: 1) to restore and sustain the flow of freshwater to the federally designated "National Wild and Scenic" Northwest Fork of the Loxahatchee River (NWFLR), 2) to increase connectivity of hydrology, flora, and fauna between natural areas; and 3) to improve seasonal timing and distribution of water to restore drained wetlands that form the historic headwaters for the river. Among all alternatives considered, alternative 5R was chosen as the recommended plan **(Figure A-1)**.

Water is stored and conveyed along three geographic flow-ways, as described below. Water management, ecosystem restoration, and flood protection is facilitated along each flow-way through the addition or improvement of water control structures, canals, plugs, and levees.

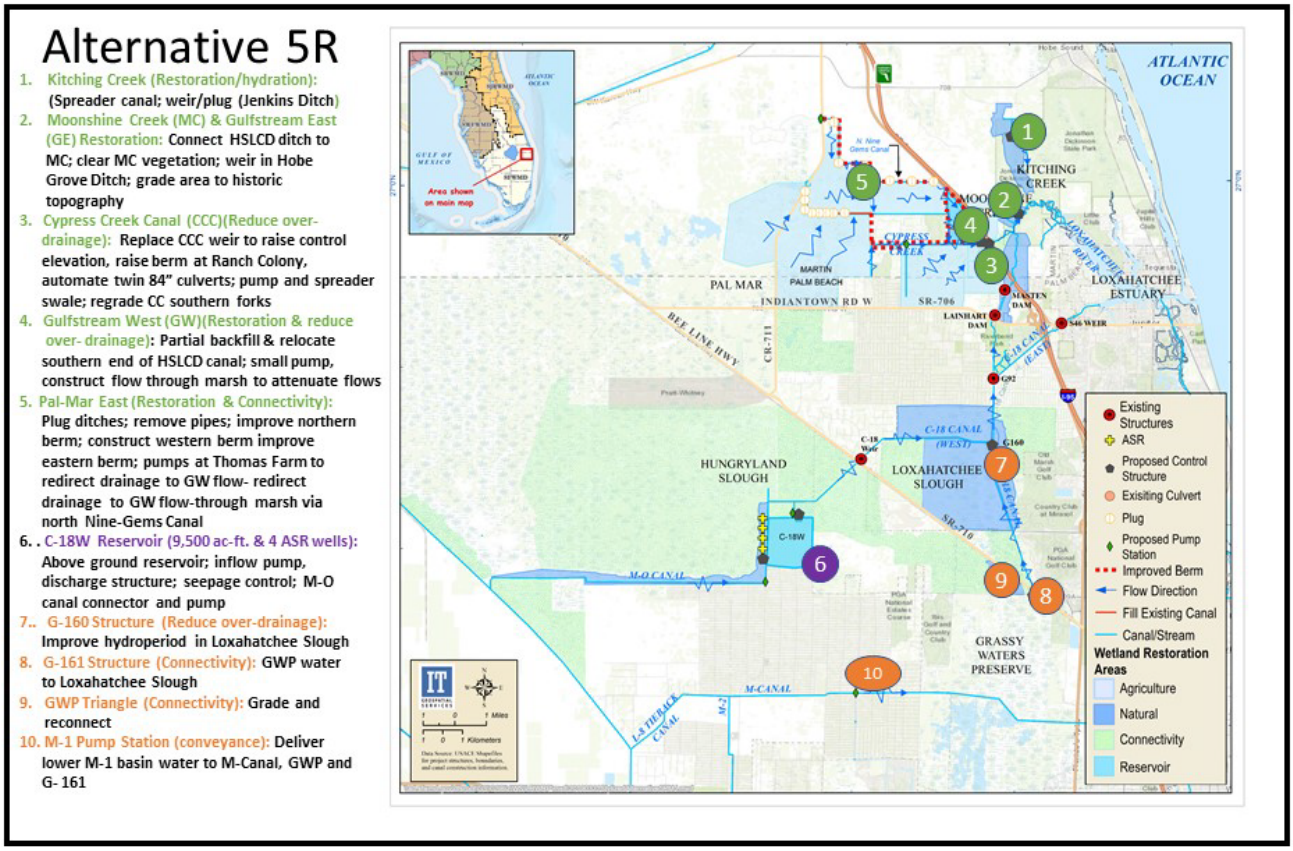


Figure A-1. Project overview map showing flow-way 1 (orange), 2 (purple), and 3 (green).

A.3.1 Flow-way 1

Flow-way 1 is located in the southeast and southernmost portion of the project area. This flow-way uses M-1 Basin canals, the M-Canal, and the C-18 Canal to route water from upstream project area basins to the Loxahatchee River **(Figure A-2)**.

The proposed M-1 pump station, S-100, is designed to bring water into the M-Canal and Grassy Waters Preserve (GWP), and provides a connection of watersheds (M-1 Lower to M-Canal). The S-100 pump station will deliver up to 75 cubic feet per second (cfs) to the M-Canal from Indian Trail Improvement District (ITID) Lower M-1 Basin when M-1 Basin Canal and M-Canal stage conditions allow. Operations of the M-1 Basin allow for water to be pumped to the M-Canal when stages in the M-1 Canal are above 17.0 ft NGVD29 (15.5 ft NAVD88) in the dry season and 15.0 ft NGVD29 (13.5 ft NAVD88) in the wet season.

The G-161 structure is an existing multi-purpose feature that facilitates hydroperiod restoration of GWP, maintain existing level of service of flood protection, and provide improved conveyance for the regional water system. Through the construction of G-161, excess water can be sent to the C-18 Canal and Northwest Fork of the Loxahatchee River from GWP. The G-161 structure will act as the gateway through which water will be transported from GWP through the system to the NWFLR. The structure is located west of the intersection of the Beeline Highway and Northlake Boulevard. Water will flow from GWP into the northern GWP triangle area, north of Northlake Boulevard. The operable structure will pass up to 150 cfs through two 60-in diameter culvert barrels with a total length of 240 ft.

The GWP Triangle is located northwest of the intersection of Beeline Highway (SR-710) and Northlake Boulevard. Hydrologic restoration of this area will be accomplished through earth work and strategic construction of a natural conveyance feature. These features include a shallow swale with gentle slopes running in a west to east direction, which will allow water discharged from G-161 to be spread westward to improve the hydroperiod in the area. The shallow swale will allow for a hydrologic connection between the western and eastern portions of the triangle. High water in the western triangle will hydrate an adjacent swale. This shallow swale will facilitate connectivity, provide some storage, and improve hydroperiod. Water will move from the GWP Triangle to the C-18 Canal under a railroad bridge and through an existing culvert beneath the Beeline Highway.

The purpose of the existing G-160 structure will achieve the following: 1) preserve sensitive wetlands in the Loxahatchee Slough, 2) maintain the existing level of service of flood protection; and 3) provide water to the Northwest Fork of the Loxahatchee River. G-160 will increase stages in the Loxahatchee Slough, improving hydroperiods that have been adversely impacted by the construction and historic operations of the C&SF Project, specifically the C-18 Canal and Structure 46 (S-46). With increased stages in the slough, G-160 can deliver additional flow to the Loxahatchee River as needed.

The G-160 structure is an existing reinforced concrete spillway designed to enhance delivery of the restoration flows to the NWFLR while maintaining water elevations within the Loxahatchee Slough. 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 2,000 cfs via two spillway bays, each 25 ft in length. The structure is operable to allow for the dual purposes of flood risk reduction 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.

Figure A-2. Flow-way 1 structures and flows.

A.3.2 Flow-way 2

Flow-way 2 is located in the west-central portion of the project area. Its primary canal conveyances are the M-O and C-18W Canals (**Figure A-3**). Features in Flow-way 2 capture water from ITID and the J.W. Corbett Wildlife Management Area (WMA) that may otherwise be delivered outside the Loxahatchee River watershed. Water is redirected northward to the NWFLR to achieve restoration flow targets. The C-18W reservoir and four co-located ASR wells will store water during wet season to be recovered during the dry season to establish a more natural seasonal delivery to the river.

A new M-O Connector Canal (C-101W) connects the eastern terminus of the M-O Canal to the C-18W reservoir via culvert S-105. C-101W will convey excess water from the M-O Canal (ITID upper basin runoff) to the C-18W basin. Due to local topography, a 150 cfs pump station (S-109) is required to pump from the M-O Canal to C-101W.

Flows from J.W. Corbett WMA will be conveyed beneath Seminole-Pratt Whitney Road to the C-18W reservoir through three 72-in gated culverts (S-107). S-107 will replace an existing sheet pile weir at the eastern edge of the J.W. Corbett WMA. Additionally, the Seminole-Pratt Whitney Road will be modified as L-101W to contain water within the J.W. Corbett WMA area during large rainfall events. The culverts have vertical slide gates at the inflow, and flap gates at the outflow. The operable structure will control flows into the reservoir seepage canal while simultaneously improving hydroperiod within the J.W. Corbett WMA. The seepage and inflow canal pump station, S-106, is located along the western embankment of the reservoir and has a capacity of 175 cfs. This pump draws water from the C-101W into the reservoir.

The C-18W reservoir is a 9,500 ac-ft above-ground reservoir on approximately 1,600 acres of a 1,920 acre former citrus grove. The reservoir will provide pumped diversion and storage of excess flows from the adjacent C-18W Canal, J.W. Corbett WMA, and from ITID upper basin via the M-O Canal. The reservoir will release water back to the C-18W Canal for delivery to Lainhart Dam and the Loxahatchee River downstream. The embankment height will be approximately 20.5 ft above natural ground elevation with a normal design pool depth of approximately 7.5 ft and a normal pool elevation of 28 ft NAVD88.

The main inflow/discharge canal for the C-18W reservoir, C-101N, is located between the C-18W Canal and the northern embankment of the reservoir. The L-101E feature will be constructed to retain water within the water storage feature that is currently existing north-east of the proposed reservoir. S-101A is a 150 cfs inflow pump station that will convey water into the reservoir from the C-18W Canal. The reservoir will have two main outflow structures 1) S-102, a gated culvert discharge structure for normal operations; and 2) S-101B, an auxiliary spillway. Both are along the northern side of the reservoir. S-102 includes dual 48-in diameter culverts that can discharge up to 300 cfs, depending on the stage within the reservoir. The auxiliary spillway has an 80 ft wide concrete crest at an elevation 28.1 ft NAVD88, with a design capacity of 1,960 cfs. The reservoir is surrounded by a seepage collection canal, C-101E and C-101W, which can discharge into the C-101N through two culverts, S-103W and S-103E.

A four-well Aquifer Storage and Recovery (ASR) system will be co-located at the C-18W reservoir to augment surface storage capacity and provide greater flexibility in reservoir operations. ASR wells will recharge from and recover into the C-18W reservoir. Each well will pump surface water into the Floridan Aquifer System at 5 million gallons per day (MGD). Water will be recovered at a rate of 5 MGD (8 cfs at each well) and discharged into the C-18W reservoir, for subsequent distribution into the C-18W Canal.

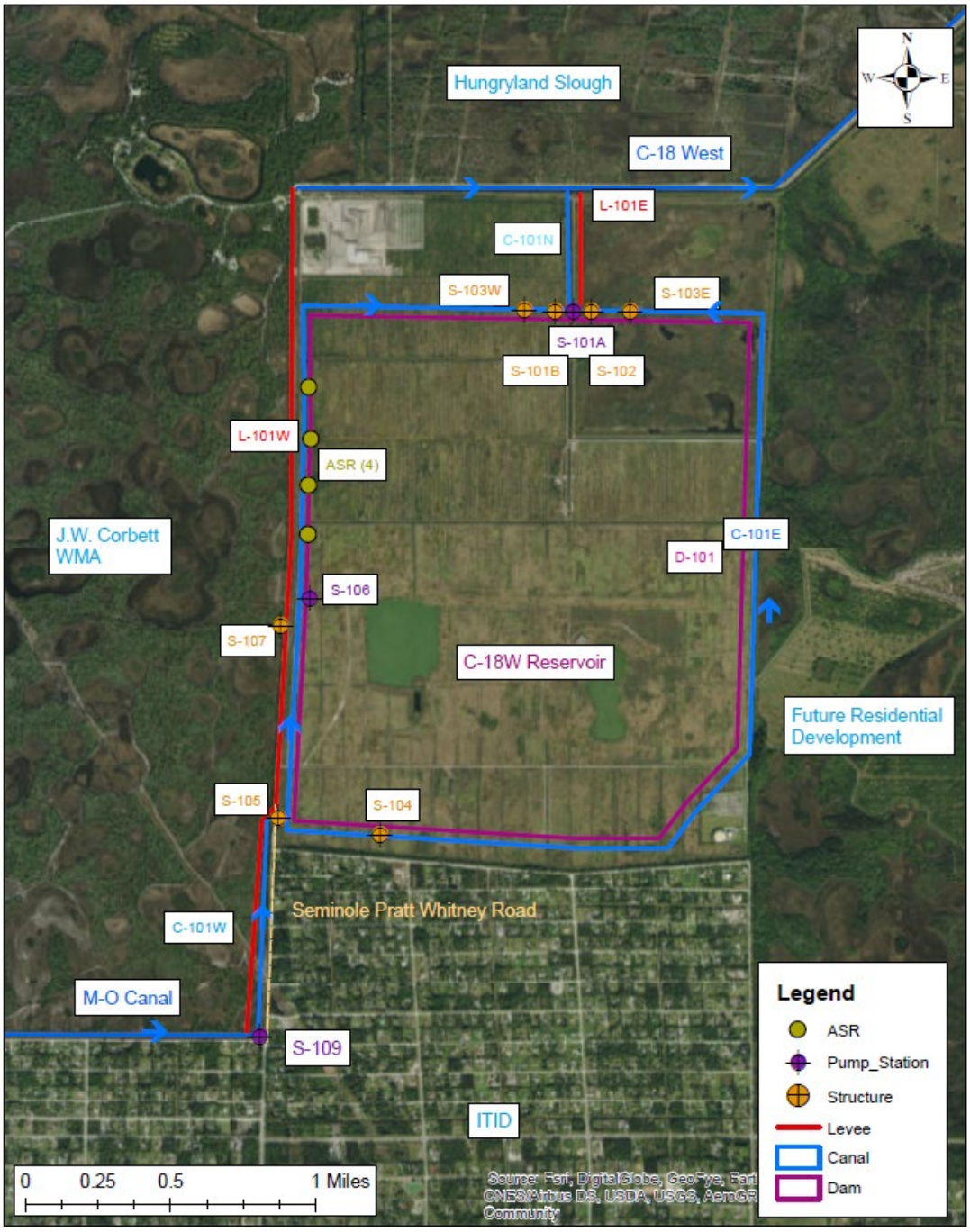


Figure A-3. Flow-way 2 structures and flows.

A.3.3 Flow-way 3

Flow-way 3 is located in the northern portion of the project area, within southern Martin County, and consists of the watersheds within the northern portion of the project study area **(Figure A-4, Figure A-5)**. Flow-way 3 actions include installing plugs, ditch backfill, and adding water control structures in canals and ditches to reduce over-drainage, restore water levels in semi-drained wetlands, and restore base flow

to the NWFLR. A flow-through marsh (within L-110) will capture pulsed discharges of water from northern and western agricultural areas and retain the water for on-site benefits and for improved timing of release to the river. Berm and channel improvements will ensure that nearby residential and agricultural areas will not be adversely impacted by the changed hydrology of the restored wetlands.

Pal-Mar East (Nine Gems)

The main purpose of the features proposed within the Nine Gems property is to help reduce over-drainage of the site, achieve rehydration and improve the hydroperiods and ecology of the area, as well as provide connectivity of Pal Mar East and the Nine Gems property.

On the Pal-Mar East property, internal drainage ditches will be plugged and small drainage pipes and culverts will be removed to reduce drainage and improve hydroperiods and ecology within the area. Improvements to the existing berms will be necessary at irregular intervals along the Pal-Mar East northern and eastern border to ensure water is held on site during larger storm events. This berm improvement will become L-111.

The existing canal along south side of Pal-Mar East (Southern Nine Gems Drainage Canal) will be partially plugged or backfilled. This will improve connectivity of Culpepper with Nine Gems and allow water to flow from Culpepper to Nine Gems rather than being captured in the existing canal. This canal provides drainage for Thomas Pepper Farm, so alternative drainage for the farm will be required. Additional discharge capability from the Nine Gems area will be accomplished through the S-114A, -B, and -C structures, which ultimately discharge out S-114D to the Cypress Creek Canal.

Thomas Pepper Farm

The Thomas pepper farm is located to the west of Pal-Mar East Property and Pratt Whitney Road (SR-711). Because the canal and ditch removals for restoration of Pal-Mar East will disrupt the existing drainage for the farm, an existing drainage ditch that runs along the northern boundary of Pal-Mar will be deepened and/or widened to handle additional flow from the Thomas pepper farm. A new pump will be required to re-route the farm's drainage under Seminole Pratt Whitney Road and into the newly expanded drainage canal. The pump location will likely be required to be on the site of the pepper farm (outside of project lands) and therefore will likely require to be turned over to the property owner for operation.

Cypress Creek Canal

The purpose of the modifications within the Cypress Creek Canal area are multi-purpose and, as stated in the previous section, will help improve the ecological conditions through re-hydration, while providing for greater operational flexibility within the Pal-Mar East (Culpepper) area as well within the Cypress Creek Canal.

Modifications to three Culpepper control structures, S-115A, S-115B and S-115C, include adding telemetry operated gates to allow the structures to achieve a more desirable hydroperiod within the Culpepper property of Pal-Mar East, while simultaneously reducing discharges into the Cypress Creek Canal.

Berm improvements along the Cypress Creek Canal and minor canal modifications (widening and deepening) along the eastern border of Pal-Mar East (western border of a developed area) will improve conveyance capacity to meet the design storm requirements. A new control structure near the eastern end of the Cypress Creek Canal will improve management of water elevations within the canal during the wet and dry seasons. Structure S-112 will be a two-bay concrete ogee spillway with telemetry operated vertical lift gates. Each bay will be 16 ft wide with a crest elevation at approximately 6.5 ft NAVD88. During

the dry season, the gates will hold additional water in the canal, assisting in the improvement of groundwater levels by reducing groundwater draw down in Pal-Mar East and the Cypress Creek Natural Area. During the wet season, the gates will help maintain restoration target stages in Pal-Mar East and the Cypress Creek Natural Area. The gates will open to allow high volume flows to pass without causing adverse flooding to nearby developed areas. The design maintains, and slightly improves the current flood protection for the surrounding developments, and reduces flashy discharges into the natural portion of Cypress Creek to the east.

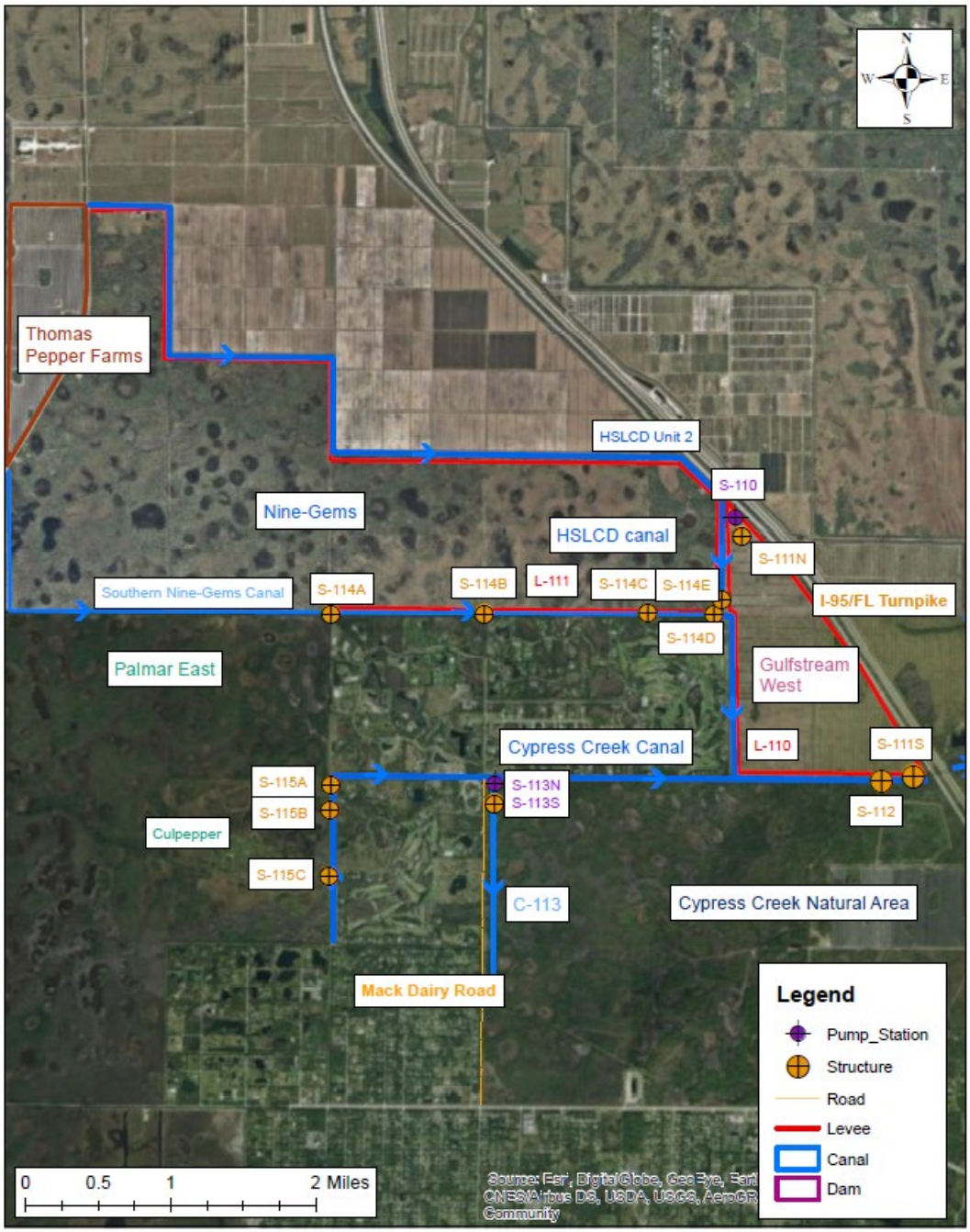


Figure A-4. Flow-way 3 structures and flows (western portion).

Mack Dairy Spreader Swale

The purpose of the Mack Dairy Road spreader is to assist in distributing water in a southerly and easterly direction to rehydrate the Cypress Creek Natural Area. This design for the distribution of water was to mimic historical flow patterns from the west (Culpepper area) that have been interrupted due to urban development.

The Mack Dairy Spreader Swale (C-114) will extend approximately 3,500 ft south from the Cypress Creek Canal, parallel to Mack Dairy Road. It will have an average depth of 3 ft, bottom width of 5 ft, 3H:1V side slopes, and the western edge of the spreader will be higher elevation than the eastern edge. A 20 cfs pump station (S-113N) will send water from the Cypress Creek Canal into the spreader swale. The discharge bay of the pump station will contain a concrete weir, S-113S, which will assist in distributing water in a southerly direction, then allowing the natural topography to cause the water to flow east across the Cypress Creek Natural Area toward the Loxahatchee River, restoring historical flow patterns. The easterly forks of Cypress Creek will be re-graded to maintain low flow velocities and promote native vegetation.

Gulfstream West

The purpose of the Gulfstream West flow-through marsh (FTM) are: 1) to divert source water from the HSLCD drainage canal into the marsh, thereby helping control discharge rates and attenuate flow; 2) to provide ecosystem benefits and water quality improvements; and 3) to reduce stages within the HSLCD Drainage Canal and Cypress Creek Canal. The FTM provides additional storage, and can re-route water from the HSLCD Canal to downstream of the newly proposed structure within Cypress Creek Canal, S-112.

The Gulfstream West FTM feature will pump water from the existing HSLCD drainage canal into a series of collection ditches and spreader berms that will promote sheet flow and re-hydration of the site. The site will be graded and existing drainage ditches will be removed to provide a more uniform topography and slight gradient to promote flow in a southerly direction. A perimeter levee will ensure water is held on-site. Pump station S-110 capacity is 150 cfs, and will discharge runoff from HSLCD and Pal-Mar East into the Gulfstream West flow-through marsh. The existing HSLCD discharge canal will be straightened and used as a bypass canal if runoff exceeds 150 cfs or when water elevations within the FTM exceed an average depth of 3 ft. The outflow structure, S-111S, is a notched weir and is designed to discharge a variable rate depending on the marsh depth. The weir will control discharge from the FTM, and is located downstream of S-112.

Gulfstream East and Moonshine Creek

The existing drainage ditches in Gulfstream East (**Figure A-5**) will be filled and the site will be re-graded to the pre-agricultural topographic condition. The Hobe Grove Ditch and Moonshine Creek are partially separated due to heavy vegetation and sediment. The proposed feature will connect the Hobe Grove Ditch and Moonshine Creek. Additionally, weir structure S-117 will be constructed at the eastern extent of the Hobe Grove Ditch to retain additional water within the ditch, improving the groundwater levels while helping to promote additional flow down the historic Moonshine Creek. The proposed weir crest elevation is 6 ft NAVD88. A pedestrian/equestrian bridge will be added to connect the Ocean-to-Lake trail across Moonshine Creek.

Kitching Creek

The main purpose of the features proposed in Gulfstream East and Kitching Creek is to improve the rehydration and ecology of these properties through re-grading and structure placement. Additionally, these features will help in the timing and distribution of flows to the Northwest Fork of the Loxahatchee River.

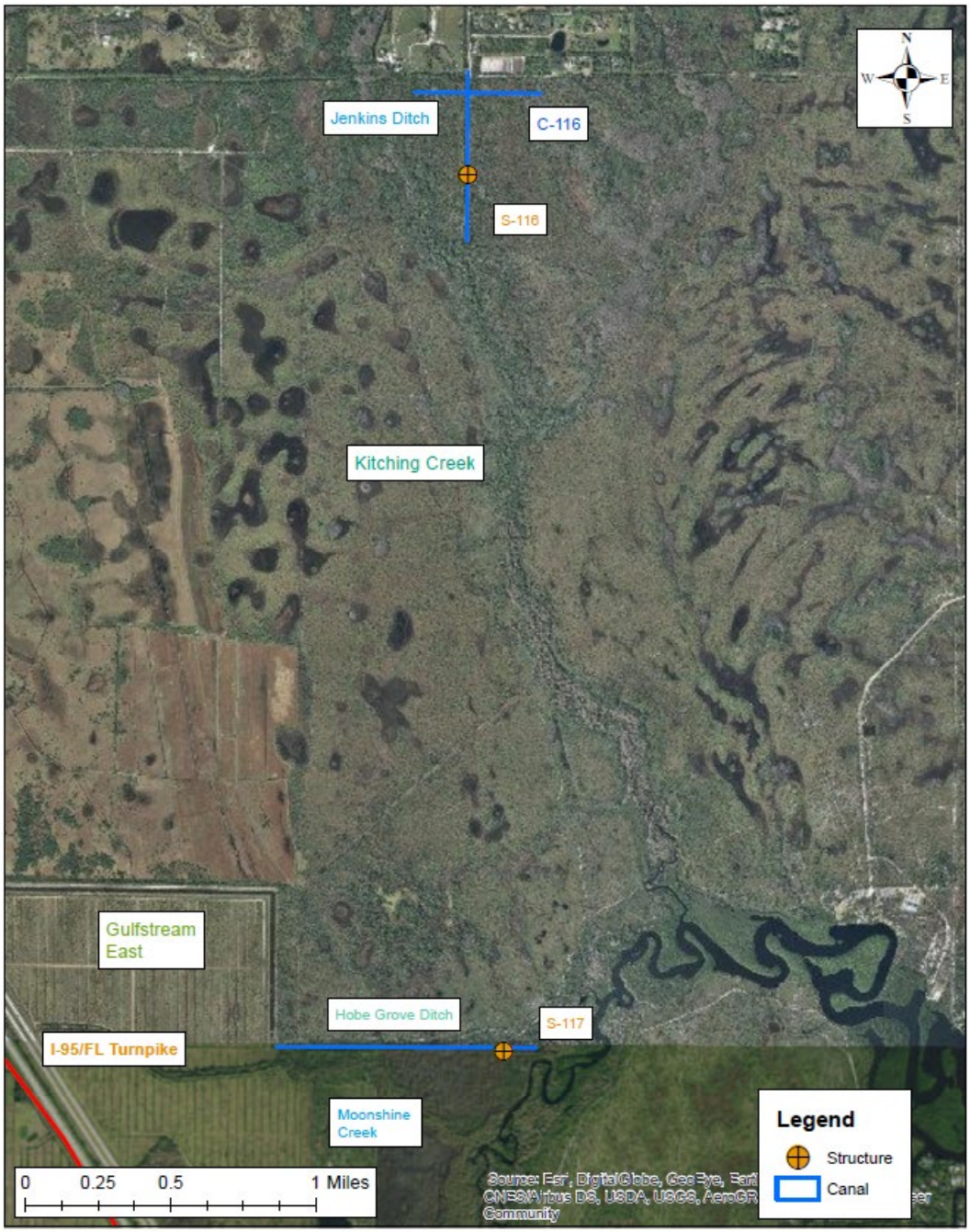


Figure A-5. Flow-way 3 structures and flows (eastern portion).

A spreader swale, C-116, will be constructed to the east and west from Jenkins Ditch at the north end of the Jonathan Dickinson State Park. This swale will help distribute flows to historic Kitching Creek channels instead of directly down the ditch. A gated culvert, S-116 will be constructed in the ditch upstream of the main Kitching Creek channel. A telemetry-operated gate on the culvert will allow for varying operation regimes. The gate may be fully opened to allow flood-discharge to exit through the Jenkins ditch, similar to existing conditions, or the gate may be close to increase water surface elevations upstream in the canal to aid in the dispersion of water into the spreader system.

A.3.4 Lands and Interests in Lands

The following real estate interests and lands identified below are required for the construction and operations, maintenance, repair, replacement, and rehabilitation (OMRR&R) of LRWRP. Approximately 81 parcels of land will be impacted by the project. See PIR-EIS **Appendix D**, Exhibit A, Real Estate Maps, page 8 of 8 for the list of impacted parcels. The following real estate interests identified below for each project feature are required for the construction and operations, maintenance, repair, replacement, and rehabilitation (OMRR&R) of LRWRP.

A.3.5 Project Features

Features proposed in the LRWRP are listed in **Table A-1**. Preliminary designs of these features are found in the plates **Annex C-1.**

A.3.6 Cost Estimates

Refer to the PIR main report **section 6.4.** Cost Estimates and **Appendix B** Cost for cost estimating methods and analysis for the project.

A.4 Status of Engineering Design Activities and Analyses

The following sections describe the status of engineering design activities and analyses.

A.4.1 Level of Design Efforts

Design Engineering Regulation ER 1110-2-1150, *Engineering and Design for Civil Works Projects*, provides guidance for feasibility-level design to accompany decision documents. Early during the LRWRP project scoping, risks that accompanied the expedited planning process were identified. The risks were presented in a project risk register.

The team identified work that would be deferred to PED. The up-front project risks recognized the potential for these design activities to significantly affect project costs. Due to the limited design, it is expected that higher risk based contingencies would be generated yet Cost Certification would still be achieved. This is in accordance with additional guidance from Engineering Construction Bulletin (ECB) 2012-18.

A.4.2 Recommendation for Design Completion

Features of the Recommended Plan have been identified according to available data, historic information, and best engineering judgment. All project components will be optimized during PED phase for cost

efficiency and performance, incorporating updated data and information as it becomes available. Design completion recommendations are provided by project element.

Table A-1. Summary of recommended plan features.

STRUCTURE NAME	STRUCTURE /FEATURE TYPE	LOCATION	PLATE NUMBER
C-101E	Canal - Seepage	Flow-way 2 C18W Reservoir	P-02, P-04
C-101N	Canal - Conveyance	Flow-way 2 C18W Reservoir Inflow/Outflow	P-02, P-05
C-101W	Canal - Conveyance	Flow-way 2 C18W Reservoir M-O Canal connector	P-02, P-05, P-06A
C-114	Canal - Distribution	Flow-way 3 Mack Dairy Rd spreader to Cypress Cr Natural Area	P-14
C-116	Canal - Distribution	Flow-way 3 Spreader to Kitching Creek	P-13
D-101	Embankment - Dam	Flow-way 2 C18W Reservoir Embankment	P-02, P-04
L-101E	Levee	Flow-way 2 C18W Reservoir Inflow/Outflow Canal (east)	P-02, P-05, P-06
L-101W	Levee	Flow-way 2 C18W Reservoir Seminole Pratt-Whitney Rd levee	P-02, P-05, P-06A
L-110	Embankment - Levee	Flow-way 3 Gulfstream West FTM perimeter	P-07, P-10
L-111	Embankment - Levee	Flow-way 3 Nine Gems Southern Boundary	P-12A
L-111	Plug canal	Nine Gems South Drainage Canal	P-11, P-12
S-100	Pump Station	Flow-way 1 M-Canal to Grassy Water Preserve	P-26, P-27
S-101A	Pump Station	Flow-way 2 C-18W reservoir pump station	P-02, P-06, P-16, P-17, P-18
S-101B	Spillway - Auxiliary	Flow-way 2 C-18W reservoir auxiliary spillway	P-02, P-06, P-47
S-102	Culvert - Gated	Flow-way 2 C-18W reservoir discharge	P-02, P-06, P-35, P-36
S-103E	Culvert - Gated	Flow-way 2 C-18W reservoir seepage canal	P-02, P-06, P-37, P-38
S-103W	Culvert - Gated	Flow-way 2 C-18W reservoir seepage canal	P-02, P-06, P-37, P-38
S-104	Culvert - Ungated	Flow-way 2 C-18W reservoir seepage canal	P-02, P-06A, P-15
S-105	Culvert - Gated	Flow-way 2 C-18W reservoir inflow from M-O canal	P-02, P-06A, P-40, P-41
S-106	Pump Station	Flow-way 2 C-18W reservoir Seepage Pump Station	P-02, P-19, P-20, P-21, P-22
S-107	Culvert - Gated	Flow-way 2 C-18W reservoir discharge from JW Corbett WMA	P-02, P-42, P-43
S-108	AVAILABLE	AVAILABLE	AVAILABLE
S-109	Pump Station	Flow-way 2 C-18W reservoir on the M-O Canal connector	P-02, P-23, P-24, P-25
S-110	Pump Station	Flow-way 3 Gulfstream West FTM	P-07, P-30, P31,P-32
S-111N	Weir – fixed crest	Flow-way 3 Gulfstream West FTM plunge pool weir	P-07, P-50
S-111S	Weir – fixed crest	Flow-way 3 Gulfstream West FTM discharge structure	P-07, P-48
S-112	Spillway - gated	Flow-way 3 Cypress Creek Canal	P-07, P-33, P-34
S-113N	Pump Station	Flow-way 3 Mack Dairy Rd spreader to Cypress Cr. Natural Area	P-14, P-28, P-29
S-113S	Weir – Fixed Crest	Flow-way 3 Mack Dairy Rd spreader PS plunge pool	P-14, P-28, P-29, P-49A
S-114A	Culvert - Gated	Flow-way 3 Nine Gems Drainage Canal	P-44, P-45
S-114B	Culvert - Gated	Flow-way 3 Nine Gems Drainage Canal	P-44, P-45
S-114C	Culvert - Gated	Flow-way 3 Nine Gems Drainage Canal	P-12, P-44, P-45, P-46, P-46A
S-114D	Culvert - Gated	Flow-way 3 Nine Gems Drainage Canal	P-12, P-44, P-45
S-114E	Culvert - Gated	Flow-way 3 Nine Gems Drainage Canal	P-11, P-44, P-45

Table A-1. Summary of recommended plan features – *completed.*

STRUCTURE NAME	STRUCTURE /FEATURE TYPE	LOCATION	PLATE NUMBER
S-115A	Culvert - gated	Flow-way 3 Cypress Creek Canal (aka WCS-1)	P-56
S-115B	Culvert – gated	Flow-way 3 Cypress Creek Canal (aka WCS-2)	P-57
S-115C	Culvert - gated	Flow-way 3 Cypress Creek Canal (aka WCS-2)	P-57
S-116	Culvert	Flow-way 3 culvert adjacent to spreader at Kitching Creek	P-13
S-117	Fixed crest weir	Flow-way 3 Hobe Grove Ditch eastern end	P-13, P-49
G-161	Gated culvert	C-18 Canal at northern end of Grassy Waters Preserve	As-Builts available
G-160	Gated spillway	C-18 Canal at Loxahatchee Slough	As-Builts available
ASR-1,-2, -3, -4	ASR well pumps	C-18W reservoir ASR system	P-54, P-55
Not numbered	Palm Beach Co	Replacement for WTP discharge pump station	No plate – see section XXX
Not numbered	Demolition plan	C-18W reservoir	P-03
Not numbered	Demolition plan	Gulfstream West Flow through marsh	P-08, P-09
Not numbered	SCADA Backbone	Add new Tower at Mack Dairy Spreader, Ops building	P-51, P-52
Not numbered	Pedestrian/Equestrian Bridge	Ocean to Lake Trail over Cypress Creek	P-53

A.5 General Construction Procedures Discussion

It is envisioned that the project will be constructed using conventional means and methods. The project features will be scoped by project elements and will be conceptually placed in contracts that maximize opportunities to realize benefits with features already in the existing system. To the fullest extent possible, the features/contracts will maximize the use of onsite material, reduce multiple handling scenarios, and maintain flood control operations and level of service provided by existing features.

Other schedule and construction assumptions included that all engineering design work would be completed by USACE with in-house resources. Independent External Peer Review (IEPR) may be performed using an external engineering firm. Beginning with investigative information gathering, multiple contracts would be awarded every year based on construction durations estimated from existing similar USACE construction projects. Adaptive Management will help with future development of the implementation and sequencing. During PED, detailed analyses, subsurface investigations, and site investigations will be conducted to prepare construction documents. A more detailed description of construction sequencing is found in **section 6.7.1** Implementation and Construction Sequencing in the PIR.

A.5.1 Flow-way 1

The only feature to be constructed in flow-way 1 is the M-1 canal pump station (S-100). This feature should be designed and constructed as one of the initial features of the LRWRP in order to maintain flood control in the ITID. Florida Power & Light transmission lines are adjacent to the proposed pump station location, but design may proceed without power line relocations.

A.5.2 Flow-way 2

The C-18W reservoir is the main component of flow-way 2. Design and construction of the C-18W reservoir will be initiated as soon as possible to develop storage capacity and subsequent benefits. Once the embankment alignment is confirmed during PED, one or more exploratory boreholes will be

constructed to characterize the hydrogeologic setting and hydrologic characteristics of the Floridan Aquifer System for ASR feasibility. Exploratory boreholes will be located west of the exterior embankment toe, on the wide 100-ft wide bench between the toe and the seepage canal. Site-specific hydrologic characteristics of the FAS will be evaluated for ASR feasibility in these boreholes. If testing results show that ASR is feasible, the exploratory borehole will be completed as ASR-1 well, and the remaining three ASR wells can be constructed, without impacting embankment construction.

A.5.3 Flow-way 3

Sequencing of design and construction of flow-way 3 components will proceed in an order to maintain flood control along Cypress Creek Canal and the HSLCD drainage canal, yet still provide ecosystem benefits. Generally, construction will proceed from east to west along flow-way 3. Construction of the Cypress Creek Canal gated spillway S-112 and the Gulfstream West flow-through marsh should be the first components to be designed and constructed.

A.6 Civil Site Design

The following subsections describe the civil site design of major features in the Recommended Plan for the Loxahatchee River Watershed Restoration Project.

A.6.1 General

The subsections below describe features that are common to all civil engineering components in the project. Features identified in the Recommended Plan have been designed to the level of detail necessary to provide a Class 3 cost estimate.

A.6.1.1 Aerial and Map Atlases

Aerials and maps for feasibility-level design are obtained from the Florida Department of Environmental Protection (FDEP), Division of State Lands, Bureau of Survey and Mapping. High-resolution imagery is most often 1-ft resolution and covers 5,000 ft x 5,000 ft on the ground. Occasionally imagery at 0.5-ft or 2-ft resolution is collected. Most counties are flown once every three years as part of a statewide cycle, with the flying season usually running from November through March.

A.6.1.2 Light Detection and Ranging (LiDAR) Survey Data

Survey data have been generated using airborne light detection and ranging (LiDAR), and these data were obtained from Palm Beach and Martin County governments, the US Geological Survey (USGS), and from surveys near Herbert Hoover Dike by the USACE. The LiDAR data sets were not collected in relation to this feasibility study, but are of sufficient detail in the project area to be used for planning purposes. Elevations will be shown in 0.1 ft (tenths of feet) and refer to the North American Vertical Datum of 1988 (NAVD88). Plan coordinates are based on the North American Datum of 1983 (NAD83) with the 2011 adjustment, Florida State Plane Coordinate System for the Florida East zone in U.S. feet.

Palm Beach County LiDAR project (2017) accuracy in non-vegetated terrain equaled 0.16 ft (4.9 cm) compared to the 10 cm specification from the USGS. The non-vegetated vertical accuracy of the classified LiDAR data was equal to 0.31 ft (9.4 cm). The vegetated vertical accuracy of the classified LiDAR data was equal to 0.59 ft (18 cm) compared with the 29.4 cm specification. For the Martin County LiDAR project (2016), the non-vegetated vertical accuracy was equal to 0.245 ft. The vegetated vertical accuracy was 0.642 ft. The expected horizontal accuracy was equal to +/- 3.28 ft (1 meter).

Survey data for the portions of the project located within Martin County, Florida were generated using airborne LiDAR in January 2016. The LiDAR data was not collected in relation to this planning project but is of sufficient detail in the project area to be used for planning purposes. The LiDAR was collected and processed to meet a maximum Nominal Post Spacing of 2.3 ft or 0.7 meter.

It is anticipated that the LiDAR data will be used more extensively in future design efforts. However, the current LiDAR data are not sufficiently accurate to generate the final design plans. During the PED phase for LRWRP, additional LiDAR data will be obtained and conventional land surveys will be conducted as appropriate for each structure as well as levee and canal alignment.

A.6.1.3 Surveying Mapping Geospatial Data

A topographic and boundary survey was completed for the C-18W reservoir footprint (formerly known as the Mecca Flow Equalization Basin; SFWMD, 2015a). This survey is supplemented by a survey of groundwater wells constructed as part of a seepage study on the C-18W reservoir footprint (SFWMD, 2015b), and boundary surveys for the shooting range and the wetland area (formerly known as Lantana Farms; SFWMD, 2014). A boundary and topographic survey was completed for the entire M-O Canal for berm improvements (SFWMD, 2013). Land survey requirements will be developed during the PED phase. These activities will include site-specific surveys of the project features, identification of utilities to be relocated, and internal canals to be filled.

A.6.1.4 Land Survey Data

Limited land survey data exist for some of the larger features in the LRWRP. A topographic and boundary survey was completed for the C-18W reservoir footprint (formerly known as the Mecca Flow Equalization Basin; SFWMD 2015a). This survey is supplemented by a survey of groundwater wells constructed as part of a seepage study on the C-18W reservoir footprint (SFWMD, 2015b), and boundary surveys for the shooting range and the wetland area (formerly known as Lantana Farms; SFWMD, 2014). A boundary and topographic survey was completed for the entire M-O Canal for berm improvements (SFWMD, 2013). Land survey requirements will be developed during PED. The cost for these surveys has not been estimated. The cost is included implicitly in the component cost estimate in Appendix B under the PED line item.

A.6.1.5 Datums Used in this Report

Hydrologic modeling of canal, river, and wetland stages for this project was performed using the National Geodetic Vertical Datum of 1929 (NGVD29). Operational data for surface water bodies currently is gathered in the NGVD29 datum. During design and construction of the project, the North American Vertical Datum of 1988 (NAVD88) will be used. Plates and other references to future project elements are related in NAVD88. Modeling and references to current conditions are related in NGVD29. Water gauges and other project elements containing elevations will be set during PED and construction and the exact conversion from NAVD88 to NGVD29 in those locations will be noted on the gauge. NOAA-NGS developed the VERTCON software to provide a conversion between NGVD29 and NAVD88. The VERTCON conversion ranges from NAVD88 equals NGVD29 minus 1.51 ft to minus 1.44 ft depending on the location within the project.

A.6.2 C-18W Reservoir

C-18W reservoir **(Plate P-02)** will have a water storage capacity of 9,500 ac-ft on a 1,590 acre property footprint. The reservoir will measure approximately 7,220 ft east to west and 8440 ft north to south. The overall crest perimeter of the Reservoir will total 28,740 ft (5.44 miles). The reservoir embankment **(Plate P-04)** will have side slopes of 3H:1V, a 14-ft crest width, and a vertical height of approximately 20 ft (EL 40.5 ft NAVD88). The design criteria followed for the reservoir embankment came from a combination of geotechnical requirements for slope stability for side slopes and crest width (drivability) and the DCM-2 for embankment height. The water source for C-18W reservoir will come from a combination of flows from the M-O Canal to the southwest via C-101W, the C-18W canal via C-101N and from the J.W. Corbett Wildlife Management Area (WMA) via S-107.

The entire reservoir will be surrounded with a seepage canal, C-101E. C-101E will serve as a seepage collection canal as well as a storm water collection route connecting C-101W to the C-18W reservoir. Pump station S-106 will pump water from C-101E into C-18W.

Due to the disruption of historic sheetflow across the original footprint during times of high rainfall, a levee (L-101W; **Plate P-05**) will be constructed along the western border of the property. The levee will run from the M-O canal, adjacent to proposed pump station S-109, and continue north until it connects to canal C-18W. L-101W will be constructed west of Seminole-Pratt Whitney Rd. until just north of structure S-105, and will then merge into Seminole-Pratt Whitney Rd. This will raise the elevation of the unpaved road portion of Seminole-Pratt Whitney Rd to an elevation of 25 ft NAVD88.

The construction of the C-18W reservoir will include a four-well ASR system. The ASR wells will be located along the northern portion of the western side of the reservoir. The wells will be located on a 100-ft wide bench between the seepage canal and the reservoir embankment **(Plate P-04)**.

A.6.2.1 Access

There will be two access roads to the C-18W reservoir. The first connects the northern portion of the reservoir to Halpatiokee Road along the northern side of the canal C-18W canal and intersects Seminole-Pratt Whitney Road **(Plate P-06)**. The second access road will connect Seminole Pratt-Whitney Road to the southwestern corner of C-18W reservoir **(Plate P-06A)**. The southwestern access road will serve as the access road to the Palm Beach County water treatment plant at the southeast corner of C-18W reservoir. The access road will run from Seminole Pratt Whitney Road and continue east to the plant.

The reservoir will also have two access roads that run both on top of the reservoir crest and along the reservoir bench **(Plates P-06, P-06A)**. Overbuilt access ramps will be evenly placed along the reservoir embankment to allow for transitioning between the crest access road and the toe access road. This toe access road will also allow for access to the ASR wells that are located along the northern half of the western side of the reservoir.

During the project, the unpaved portion of Seminole-Pratt Whitney Road that connects the neighborhood to the south with the gun range to the north, will be raised in elevation to create L-101W. This new levee will keep public access to the gun range while also protecting the subdivision from flood waters from the J.W. Corbett WMA. Public access to the gun range via Seminole Pratt-Whitney Road must be maintained during the construction of this project.

A.6.2.2 Material Balance and Disposal

Earthwork cut and fill quantities will be validated during PED to balance the on-site use of material to the extent possible. The data used for cut-and-fill estimated volumes for the C-18W reservoir were derived from the survey for an earlier design (Mecca Flow Equalization Basin; SFWMD 2015a). The C-18W reservoir embankment fill volumes can all be taken from on-site borrow locations. Borrow area dimensions will be 10-ft deep and will be located no closer than 100 ft off the interior toe of the reservoir embankment **(section A.7.9.2)**. Preliminary cut and fill calculations concluded that over 1,914,900 cubic yards of on-site borrow will be required to construct the reservoir embankment, D-101. Preliminary cut and fill calculations concluded that over 1,119,000 cubic yards of on-site cut will be generated during the excavation of the seepage canal C-101E. An interior borrow boundary of 100 ft off of the interior slope

of the reservoir has been established by seepage models. Maximum depths of the interior borrow are defined in the geotechnical section below.

Using existing boring data, most of the cut for C-101W can be utilized for the fill to construct L-101W. Any additional fill needed to construct L-101W can be taken from inside the interior borrow boundary associated with the adjacent reservoir. Preliminary cut and fill calculations concluded that 45,680 cubic yards of on-site fill will be required to construct L-101W. Preliminary cut and fill calculations concluded that over 175,300 cubic yards of on-site cut will be generated to construct canal C-101W. Additionally, preliminary cut and fill calculations concluded that over 140,500 cubic yards of on-site cut will be generated during canal C-101N construction.

Access roads along the reservoir bench and crown and proposed service roads will be constructed of 6-in of compacted limerock over compacted local subbase. Limerock will be brought in from an off-site source. Any required revetment for the C-18W reservoir will have to be brought in from off site. Reinforcement will be discussed in the geotechnical portion or completed during PED phase.

A.6.2.3 Site Utility Concerns

Buried electrical utilities appear to run along the east side of Seminole-Pratt Whitney Road near the gun range. These utilities are not sized to the needs of Pump Station S-106 and the ASR recharge and recovery pumps. The utilities will have to be upsized.

The eastern boundary of the reservoir is a right-of-way for a buried 16-in discharge pipe that runs north from the water treatment plant to the wetland area. This discharge pipe will be extended to discharge into the resized wetland area. A utility survey will be conducted during PED to identify any conflicts due to existing buried water, sewer, electrical or natural gas mains.

A.6.2.4 Aesthetics

Vegetation on the reservoir exterior embankment and the levee embankments will to be covered with sod. The reservoir bench between the seepage canal and reservoir embankment and all canal embankments will be seeded to combat erosion. The interior embankment of the reservoir will be covered with a soil cement, which will help armor the embankments from repeated wind driven wave action, and reservoir filling and release cycles. This soil cement hardening is described in **section A.8.8.** A vegetated screen is proposed along the southern property line to screen the 20-ft high embankment from residences located to the south.

A.6.3 Cypress Creek Canal Modifications

The Cypress Creek Canal **(Plate P-01)** is one of the major drains of the project area, and is a major contributor to increased flows to the northwest fork of the Loxahatchee River. Proposed modifications to the Cypress Creek Canal will maintain and improve current flood control in the canal, improve wetland hydroperiods to the west in the J.W. Corbett WMA, and reduce flashy discharges to Cypress Creek. Modifications to Cypress Creek Canal include widening the canal just south of the proposed Gulfstream West flow-through marsh, selective widening of the canal north of the Culpepper area, retrofitting three culverts with motorized gates, demolition of a bridge, construction of S-112 gated spillway, and the addition of the Mack Dairy Road spreader **(section A.6.3)**.

A.6.3.1 Access

Access to structures along the eastern reach of Cypress Creek Canal is through existing unpaved roads of the Cypress Creek Natural Area (CCNA). A gated unpaved road extends northward from western Indiantown Road (State Road 706) to the Cypress Creek Canal at a point south of the Gulfstream West flow-through marsh. An existing bridge allows passage over the canal to the north, but this bridge will be demolished as part of this project **(Plate P-08).** A vehicular crossing across Cypress Creek Canal will be restored by a service road across the proposed gated spillway structure S-112 **(Plates P-43** and **P-44).**

Access to structures along the western reach of Cypress Creek Canal, including S-115A, S-115B and S-115C **(Plate P-01)** is from Old Indiantown Road. Old Indiantown Road intersects at the southwestern most point of the north-south portion of Cypress Creek Canal.

A.6.3.2 Material Balance and Disposal

Earthwork cut-and-fill quantities will be validated during PED to balance the on-site use of material to the extent possible. Preliminary cut-and-fill calculations showed that over 85,200 cubic yards of cut will be available during the widening of the Cypress Creek Canal. Further soil testing will be required to determine how much of this material will be suitable embankment fill for the Gulfstream West embankment. The canal will be widened to a 30 ft bottom width with 2H:1V side slopes.

The canal portion north of the Culpepper area will need selective excavations to increase conveyance capabilities. The selective locations are discussed the in the Hydraulic Design **section A.8**. Preliminary cut and fill calculations concluded that over 15,000 cubic yards of cut will be accumulated during the widening of the Cypress Creek Canal just north of the Culpepper area.

Any required reinforcement material for the Cypress Creek Canal will be brought in from off-site. Reinforcement will be discussed in the **section A.7.5.3** for the Gulfstream West flow-through marsh.

A.6.3.3 Site Utility Concerns

Pole-mounted electrical lines run along the east side of Mack Dairy Road. It does not appear that any electrical utilities will be moved during this project. The utilities will be installed to allow for power to run the telemetry at S-115A, -B and -C structures, Mack Dairy Spreader pump station, and Cypress Creek gated spillway **S-118.** The utilities will be required to allow for power to run the telemetry at S-115 structures and S-112 gated spillway.

A.6.3.4 Aesthetics

The Cypress Creek Canal embankments will be seeded to combat erosion where reinforcement material is not installed.

A.6.4 Mack Dairy Road Spreader

This feature will rehydrate wetlands of the CCNA east of Mack Dairy Road **(Plate P-01)**. Flows from Cypress Creek Canal will be diverted by a pump station into the spreader swale. Sheet flow will be conveyed through the CCNA toward the historic Cypress Creek tributary located west of I-95/Florida Turnpike.

A.6.4.1 Access

Access to the Mack Dairy Road spreader is from Mack Dairy Road that intersects the Cypress Creek Canal just south of SE Tailwinds Road. This intersection allows for access to the south side of the canal where the new S-113N pump station and S-113S weir will be constructed. The spreader swale extends southward from the pump station **(Plates P-14, P-28, P-29,** and **P-49A)**.

A.6.4.2 Material Balance and Disposal

Earthwork cut and fill quantities will be validated during PED to balance the on-site use of material to the extent possible. Due to limited borings, the minimal depth of the proposed spreader swale and the aerial imagery showing moderate vegetation in the area of the proposed spreader swale, it is assumed that a 40% loss of usable fill for the containment berm can be expected from the excavations of the swale and the plunge pool. This assumption will be validated during PED to confirm available fill.

The spreader swale will have a 5 ft bottom width with 1V:4H side slopes. The swale length will be 4,900 ft with a depth of approximately 2 ft below natural grade. Along the west side of the swale will be a containment berm with a 1V:3H side slope and a 7 foot top width that will tie into existing grade. The eastern side of the swale will require grading to allow for uniform flow across natural grade as much as possible. A plunge pool is required for the disbursement of the water from the new Mack Dairy Pump Station, S-113N. The plunge pool dimensions will be 10 ft wide by 20 ft long. Minor excavation of the south embankment of the Cypress Creek Canal may be required for the intake portions of the new pump station. The intake of the pump station and the embankment of the plunge pool will require rip-rap. Rip-rap material as well as fill material for the containment berm will be brought in from an off-site location. There will be some regrading of the historic Cypress Creek tributary located just south of Cypress Creek Canal to facilitate sheet flow between the spreader and the creek.

A.6.4.3 Utility Relocations

Pole mounted electrical lines run along the east side of Mack Dairy Road. It does not appear that any electrical utilities will have to be moved during this project. The utilities will have to be installed to allow for power to run the Mack Dairy Spreader pump station, S-113N, and the electrical needs listed in **section A.6.2.3.**

Buried water and sewer utilities were not observed but will likely be encountered along the Mack Dairy Road intersection with the canal. A utility survey will be conducted during PED to identify any conflicts due to existing buried water, sewer, electrical or natural gas mains.

A.6.4.4 Aesthetics

The Mack Dairy Road Spreader and Cypress Creek Natural Area will be re-vegetated after grading. The proposed spreader swale embankments and the berm will be seeded to combat erosion. Recruitment of terrestrial invasive species will likely be reduced with additional water flow. Expansion of emergent and floating aquatic invasive species are likely. Introduction of new invasive species to the area is likely. The spreader swale area will require maintenance. Diligent surveillance and control efforts will be required.

A.6.5 Gulfstream West Flow-Through Marsh

This Gulfstream flow-through marsh **(**FTM; **Plate P-07)** will divert flows from the HSLCD canal via a pump station into the Gulfstream West property. Grading and berms across the "triangle" will create a series of pools to reduce flow velocity, and enhance wetland rehydration of former citrus fields. The flow-through marsh (FTM) will have berms on all sides. The 90 degree canal bend portion of the HSLCD canal that is inside the Gulfstream West FTM area will be backfilled and rerouted southward along the western side of the FTM area. The HSLCD canal reroute is described in **section A.6.6**. Sheetflow will exit the FTM feature via a gated culvert into the eastern portion of Cypress Creek Canal.

A.6.5.1 Access

Access to the Gulfstream West FTM is limited to two routes. The first access would be the same dirt road access described in **section A.6.3.1**. The second access road is via an unimproved road that is along the north embankment of the Nine Gems Canal. This dirt road starts at Pratt-Whitney Road and proceeds south and east along the Nine Gems Drainage Canal until it runs into the Gulfstream West FTM site. The unpaved road crosses a three-barrel culvert whose load rating is unknown, and could be a limiting factor for heavy machinery access. A full access survey will be conducted during PED.

A.6.5.2 Material Balance and Disposal

Earthwork cut and fill quantities will be validated during PED to balance the on-site use of material to the extent possible. Preliminary cut and fill calculations concluded that over 10,600 cubic yards of excess on-site fill will be generated during the excavation of the pump station S-110 inlet bay. Preliminary cut and fill calculations also concluded that over 1,704,200 cubic yards of excess on-site fill will be generated during the excavation, regrading, and berm construction of the Gulfstream West FTM. A cumulative 1,710,800 cubic yards of excess on-site fill will be generated. The cut and fill calculations for the Gulfstream West FTM are based off the excavated minimum depth of 13.25 ft NAVD88. During the regrading effort, a portion of the HSLCD Canal will be back-filled, and the HSLCD canal will be redirected southward along the western portion of the Gulfstream West FTM.

Most of the excess on-site fill for Gulfstream West FTM can be utilized for the fill of the Mack Dairy Road Spreader berm, the HSLCD Canal plugs, and other Nine Gems fill requirements.

A.6.5.3 Site Utility Concerns

There are no known overhead or buried utilities. It does not appear that any electrical utilities will have to be moved during this project. New utilities will have to be installed to allow for power to run the new pump station S-110 and the gated spillway S-112. A utility survey will be conducted during PED to identify any conflicts due to existing buried water, sewer, electrical or natural gas mains.

A.6.5.4 Aesthetics

Vegetation on the exterior berm embankments will to be covered with sod. The interior embankment berms will be covered with or completely made of a soil cement, which will help armor the berms from repeated reservoir filling and release cycles. This soil cement berm hardening is described in **section A.7.8.5**. Gulfstream West will need a vegetation restoration plan during PED to identify which non-invasive species of plants should be planted inside the flow-through marsh area.

A.6.6 Nine Gems Property

The Nine Gems property **(Plate P-01)** includes five project features to aid in the increase of sheet flow from Palmar East and western Nine Gems to the east. These projects include plugging the Nine Gems Drainage Canal in three locations, Backfilling three north to south drainage canals, modifying the HSLCD canal along the western side of Gulfstream West, removing and constructing five water control structures (S-114A, S-114B, S-114C, S-114D, S-114E; **Plates P-12, P-44, P-45, P-46, P-46A**) along the Nine Gems Drainage Canal, and constructing a new levee, L-111 **(Plate P-10).**

A.6.6.1 Access

Access to the Nine Gems Property has several routes. The first would be the dirt road that is along the north embankment of the Nine Gems Drainage Canal. This dirt road starts at Pratt Whitney Rd and heads south along the Nine Gems drainage canal. It follows the canal along the north embankment as the canal turns eastward until it runs into the Gulfstream West flow-through marsh site. The unpaved road crosses a three-barrel culvert. The load rating of this culvert crossing is not known and maybe a limiting factor with heavy machinery access. A full access survey will have to be completed during PED phase.

A second access road would be the dirt road along the HSLCD Canal. This dirt road starts at Pratt Whitney Road and travels east along the HSLCD Canal. This dirt road is the berm that is going to be converted into the new Levee, L-111. L-111 will travel southeastward until it reaches the intersection of Gulfstream West and Nine Gems Drainage Canal. There L-111 will head westward until it ties into the Culpepper berm.

A third access road would be the same dirt road access route used for Gulf Stream West and the Cypress Creek Canal. The access road is via a dirt road that is just north of the W. Indiantown Road intersection with Jupiter Farms Road. This dirt road runs north until it intersects with the south side of the Cypress Creek Canal, just south of the proposed Gulfstream West flow-through marsh. The dirt road intersection with the Cypress Creek Canal is the location of the bridge that is to be demolished. Just to the east of the bridge will be S-112, Cypress Creek Canal gated spillway. S-112 will allow for vehicular maintenance traffic to access the north side of Cypress Creek Canal as well as the Gulfstream West FTM.

A.6.6.2 Material Balance and Disposal

Earthwork cut and fill quantities will be validated during PED to balance the on-site use of material to the extent possible. Preliminary cut and fill calculations concluded that a cumulative 1,710,800 cubic yards of excess on-site fill will be generated during the construction of the Gulfstream West FTM. Most of the excess on-site fill for Gulfstream West FTM can be utilized for the fill for the Nine Gems fill requirements.

The construction of L-111 **(Plate P-10)** will require significant amounts of fill. L-111 is approximately 46,000 ft long. The existing berm that is in places has a crest elevation ranging from 17 to 19 ft NAVD88. The crest elevation for L-111 will be 21.25 ft NAVD88. Due to limited available boring data, the material used to construct the current berm material is unknown.

To estimate material quantity for L-111, the existing berm north of the residential neighborhood will be completely removed and replaced with approved material. The remaining portion of L-111 will utilize the existing berm to reduce the fill material required for estimation purposes. During PED, if confirmation borings conclude that the berm is built of acceptable material, a reduction of 45 percent of required fill material will be available for reuse to construct L-111. Using available Lidar data, linear elevations were

utilized to generate cross sectional areas of L-111. Using the 45% reduction, preliminary cut and fill calculations concluded that a cumulative 185,300 cubic yards of fill will be required to construct L-111.

The Nine Gems drainage canal will be plugged in three locations **(Plates P-11 and P-12)**. The estimated fill requirements for each of the three plug locations is 400 cubic yards for a total of 1,200 cubic yards of fill. Each plug will include the berm removal on both sides of the canal and will fill approximately 50 feet of the canal. The final elevation at each plugged canal location should be approximately 19 feet NAVD88.

The Nine Gems area has three north-south drainage canals that are to be backfilled along with the removal of the berms on each side of the canals for a length of 2,500 ft **(Plates P-11 and P-12)**. Preliminary cut-fill calculations show that all three canals can be backfilled with the berm material on each side of the canals. Any extra fill needed can be taken from the excess from the construction of the Gulfstream West flow-through marsh.

The HSLCD canal that is along the west side of the Gulfstream West FTM needs to be redirected to flow southward towards the Cypress Creek Canal. During the construction of the Gulfstream West FTM, a portion of the HSLCD canal will be back filled. The HSLCD canal will need to be modified to continue running south. The HSLCD canal modification will lengthen the canal by 2,650 ft. The canal modification will include a 20 ft bottom width with 2.5H:1V side slopes. The starting invert elevation of the canal modification will be 3.3 ft NAVD88 and will end at elevation 2.0 ft NAVD88. With a cross sectional area of approximately 725 ft^2, it is expected that excavation for the new HSLCD canal modification will generate 71,200 CY of fill.

The intersection of the HSLCD canal modification with the Cypress Creek Canal will likely require rip-rap as well as the intake and discharge sides of each new structure in the Nine Gems area. Rip-rap material as well as fill material for the containment berm will be required to be brought in from an off-site location.

A.6.6.3 Site Utilities Concerns

There are no known overhead or buried utilities. It does not appear that any electrical utilities will have to be moved during this project. New utilities will have to be installed to allow for power to run the northern side of the Nine Gems Drainage Canal for structures with new telemetry. A utility survey will be conducted during PED to identify any conflicts due to existing buried water, sewer, electrical or natural gas mains.

A.6.6.4 Aesthetics

Vegetation on the levee L-111 embankments and crest will to be covered with sod. All canal embankments will be seeded to combat erosion.

A.6.7 Eastern Project Features: Hobe Grove Weir, Kitching Creek Spreader

The Eastern Project Features includes four projects east of the I-95/Florida Turnpike corridor, north of Cypress Creek **(Plates P-13, P-49)**. These projects include the re-grading of Gulfstream East, the construction S-117 Hobe Grove Ditch weir, the construction of a pedestrian/equestrian bridge over Cypress Creek, and the construction of culvert S-116, and spreader C-116 at the Kitching Creek culvert and spreader swale.

Gulfstream East is an approximately 450-acre former citrus field that will require re-grading and the filling of ditches to a more natural and historic grade. All of the agricultural ditches will be filled in with the adjacent farm berms. The removal of irrigation piping and other abandoned farm structures will be addressed during PED.

Construction of a fixed-crest weir at the eastern terminus of Hobe Grove ditch, S-117, is proposed. S-117 will helps divert discharge through historic Moonshine Creek while helping to control discharges into the northwest fork of the Loxahatchee River. A pedestrian/equestrian bridge over Cypress Creek will connect the Ocean-to-Lake Trail through this area. The exact location of this bridge will be determined during PED.

A telemetry operated gated box culvert and spreader, S-116, and C-116 are proposed near the headwaters of Kitching Creek in Jonathan Dickinson State Park north of the Loxahatchee River. This feature will slow and distribute flows in this tributary to the Loxahatchee River. S-116 will allow for water to freely flow under Kitching Creek Road to both the east and west 1,000 ft swales. The existing corrugated metal pipe culvert will be removed and replaced by S-116, and Kitching Creek Road will be resurfaced.

A.6.7.1 Access

Gulfstream East allows for access to the proposed structure S-117, the proposed pedestrian bridge, as well as the Gulfstream East area. Access to the Gulf Steam East property is limited to a few routes. The first access route would be via the underpass extending from the eastern Gulfstream West property beneath I-95 and the Florida Turnpike. This access route will greatly limit the size of machinery that can be transported based on the sizing of the underpass width and height. The second access route will be an unimproved road that travels perpendicular to I-95. This road is accessible from an unnamed southward road that is a half a mile east of I-95 off of SE Bridge Road. Traveling south 2.5 miles down the unnamed road, the dirt access road is located just off the unnamed road.

Access to the Kitching Creek structures S-116 and C-116 is via Kitching Creek Road. The culvert is located along the west side of Kitching Creek Road and the spreader swale is 1000 feet along both sides of Kitching Creek Road just north of the new gated box culvert, S-116.

A.6.7.2 Material Balance and Disposal

Gulfstream East will be graded to facilitate drainage to the historic Moonshine Creek. To accomplish this, farm ditches will be infilled. For estimation purposes, multiple cross-sections of ditches were measures north and south of the canal in the center of the property. A total of 46,000 cubic yards and 38,100 cubic yards of earth work will be required for both the north and south ditches respectively. Re-grading of the outlet of the canal along the eastern side of Gulf Stream West will also be needed. Primary earth work estimates determine that 5,000 cubic yards of material are required to fill ditches near the proposed S-117 structure.

A.6.7.3 Site Utility Concerns

A utility survey will be conducted during PED to identify any conflicts due to existing buried water, sewer, electrical or natural gas mains.

A.6.7.4 Aesthetics

Gulfstream East will need a vegetation restoration plan during PED to identify which non-invasive species of plants should be planted. All canal embankments around S-116 and S-117 as well as the spreader swale berms will be seeded to combat erosion.

A.7 Geotechnical Design

Aspects of geotechnical design of selected features of the alternatives and ultimately the Recommended Plan requires soils characterization, seepage/slope-stability analyses, bearing capacity and settlement analyses, cofferdam design, filter design, other foundation/excavation/ dewatering design analyses, and characterization of the hydrogeological setting for ASR. Seepage analysis was performed on two important structures of the Recommended Plan: the C-18W reservoir and the Gulfstream West flow-through marsh. Results of these analyses are presented in **section A.7.2.3** for the C-18W reservoir, and **section A.7.5.2** for the Gulfstream West flow-through marsh. Additional supporting materials for seepage and slope stability analyses are found in **Annex B-1 Geotechnical Design.** Existing soils data, geotechnical borings, and hydrogeologic data calculations are summarized in the Engineering Appendix and compiled in the Geotechnical **Annex C-1**. There is very little subsurface information available for most of the structures, canals, proposed embankment alignments, and earthworks in the project area. A more detailed drilling program plan will be required to inform the design of these features, and these investigations will be conducted during PED phase.

A.7.1 General

The subsections below describe features that are common to all geotechnical components in the project.

A.7.1.1 References

Geotechnical design of the project features is based on guidance found in U.S. Army Corps of Engineers Engineering Manuals, design standards (DS) from the U.S. Department of Interior Bureau of Reclamation (BOR), design criteria memoranda (DCM) from Acceler8, project design memoranda from Arcadis (2016), the American Society for Testing and Materials (ASTM), and other published guidance. This information was used insomuch as its direct applicability to the anticipated foundation conditions at each feature site and for the preliminary design of those features. Applicable guidance includes:

- American Concrete Institute (ACI) – Report on Soil Cement, ACI 230.1R09
- ARCADIS – Work Order No. 4600003011-WO01, Mecca Impoundment Design Documentation Report, Task 2.1 – Geotechnical Investigations (2017)
- ARCADIS – Work Order No. 4600003011-WO01, Mecca Flow Equalization Basin, Palm Beach County, Florida, Task 2.8 – Geotechnical Report (2016)
- ASTM C33/C33M 2016, Standard Specification for Concrete Aggregates
- DCM-4, Minimum Dimensions of Dams and Embankments
- DCM-9, Dam Safety Instrumentation and Monitoring
- DS No. 13, Embankment Dams, Chapter 4, “Static Stability Analysis,” Chapter 5, “Protective Filters,” and Chapter 8, “Seepage”
- EM 1110-2-1601, Hydraulic Design of Flood Control Channels
- EM 1110-2-1901, Seepage Analysis and Control for Dams
- EM 1110-2-1902, Slope Stability
- EM 1110-1-1904, Settlement Analysis
- EM 1110-2-1905, Bearing Capacity
- EM 1110-2-1908, Instrumentation of Embankment Dams and Levees
- EM 1110-2-1913, Design & Construction of Levees
- EM 1110-2-2100, Stability Analysis of Concrete Structures
- EM 1110-2-2300, Earth & Rock-Fill Dams General Design & Construction Considerations

- EM 1110-2-2504, Design of Sheet Pile Walls
- EM 1110-2-2906, Design of Pile Foundations
- EM 1110-2-2909, Conduits, Culverts, and Pipes
- ER 1110-2-1156. Safety of Dams – Policy and Procedures
- ER 1110-2-1806. Earthquake Design and Evaluation for Civil Works Projects. U.S. Army Corps of Engineers: 31 July 1995.
- ER 1165-2-132. Hazardous, Toxic and Radioactive Waste (HTRW) Guidance for Civil Works Projects
- FEMA - Filters for Embankment Dams, Best Practices for Design and Construction
- ER 1110-2-1806. Earthquake Design and Evaluation for Civil Works Projects. U.S. Army Corps of Engineers: 31 July 1995.
- FEMA – Technical Manual: Plastic Pipe Used in Embankment Dams
- Hunt, Roy. *Geotechnical Engineering Investigation Handbook*. 2nd Edition. Boca Raton, FL: CRC Press, 2005
- Spencer, E. *A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces*. Geotechnique, Vol 17, No. 1, pp 11-26. 1967

A.7.2 C-18W Reservoir and Adjacent Areas

Flow-way 2 comprises portions of J.W. Corbett WMA, the C-18W reservoir footprint, and the Loxahatchee Slough lands south of the C-18W Canal. Several geotechnical investigations were completed for lands at the C-18W reservoir footprint. Several iterations of geotechnical investigations were required because the proposed feature in this footprint evolved over time from a research facility (Palm Beach County Biotechnology Research Park; Tierra, Inc., 2004), to a flow-way (Mecca Farms, URS, 2008); to 7,200 ac-ft reservoir (Arcadis, 2016). Geotechnical investigations results are summarized below.

A.7.2.1 Soils

Surface soils (0 to 10 cm depth) in the area of the C-18W reservoir and adjacent areas are characterized mostly as SP-SM, or generally poorly graded, slightly silty sands, with silt content less than 12 percent by weight. Soils are mapped as drained or ponded Riviera fine sands (Natural Resources Conservation Service (NRCS), 2019; Tierra, Inc. 2004; URS, 2008). Surface soils show uniform saturated hydraulic conductivity estimates of approximately 9.2 x 10^{-3} centimeters per second (26 feet per day). **Figure A-6** shows the classification of surface soils at the C-18W reservoir and adjacent areas.

Most of the geotechnical data available for the shallow subsurface soils in flow-way 2 were obtained within the footprint of the C-18W reservoir. A preliminary geotechnical investigation at the site showed shallow subsurface soils to a depth of 25 ft below land surface (bls) in 24 core borings (Tierra, Inc., 2004). These borings showed soils interpreted as “silty clayey sand” (SM/SC-SM) at depths between 5 ft and 25 ft bls, interlayered with sand (SP) with shell or cemented sand.

A subsequent geotechnical investigation at the C-18W reservoir site (URS Corporation, 2008) showed shallow subsurface soils to a depth of 40 ft bls in eight Standard Penetration Test (SPT) borings, with analysis of selected samples to define geotechnical characteristics of shallow subsurface soils for preliminary slope stability evaluation and fill requirements. Shallow subsurface materials consisted primarily of sand (SP) with discontinuous layers or lenses of silty sand (SP-SM) or clayey sand (SC) and rarely a blue clay (CL) within the upper 20 ft bls. Occasional moderately dense limestone and shell fragments were also encountered in the upper 10 ft bls. Materials consisted primarily of SP sand from 20

ft to 30 ft bls in these borings. Four laboratory permeability measurements using remolded samples yielded moderate hydraulic conductivity values ranging between 9 x 10^{-5} cm/sec and 2 x 10^{-3} cm/sec (0.26 ft day to 5.7 ft/day; URS Corporation, 2008) over the depth range of 5 ft to 40 ft bls. Hydraulic gradients report for these tests was 19.67 +/- 0 for all four tests.

An expanded geotechnical investigation (Arcadis, 2016) was completed at the C-18W reservoir site as part of the Design Documentation Report (DDR). This DDR was required to characterize additional boring data, seepage, slope stability, and foundation characteristics because the storage feature was revised to reservoir, not a flow equalization basin. Twenty-four borings were completed around the periphery of the proposed impoundment, plus two interior borings completed as piezometers. Generally, surficial (0 ft to 15 ft bls) soils consist of interbedded poorly graded sand with silt (SP-SM). Deeper sediments are predominantly poorly graded sand with silt (SP-SM), silty sand (SM) and poorly graded sand (SP) with shell and occasional clayey sand (SC) layers. Limestone is encountered at depths below approximately 30 ft bls as discontinuous thin to massive beds.

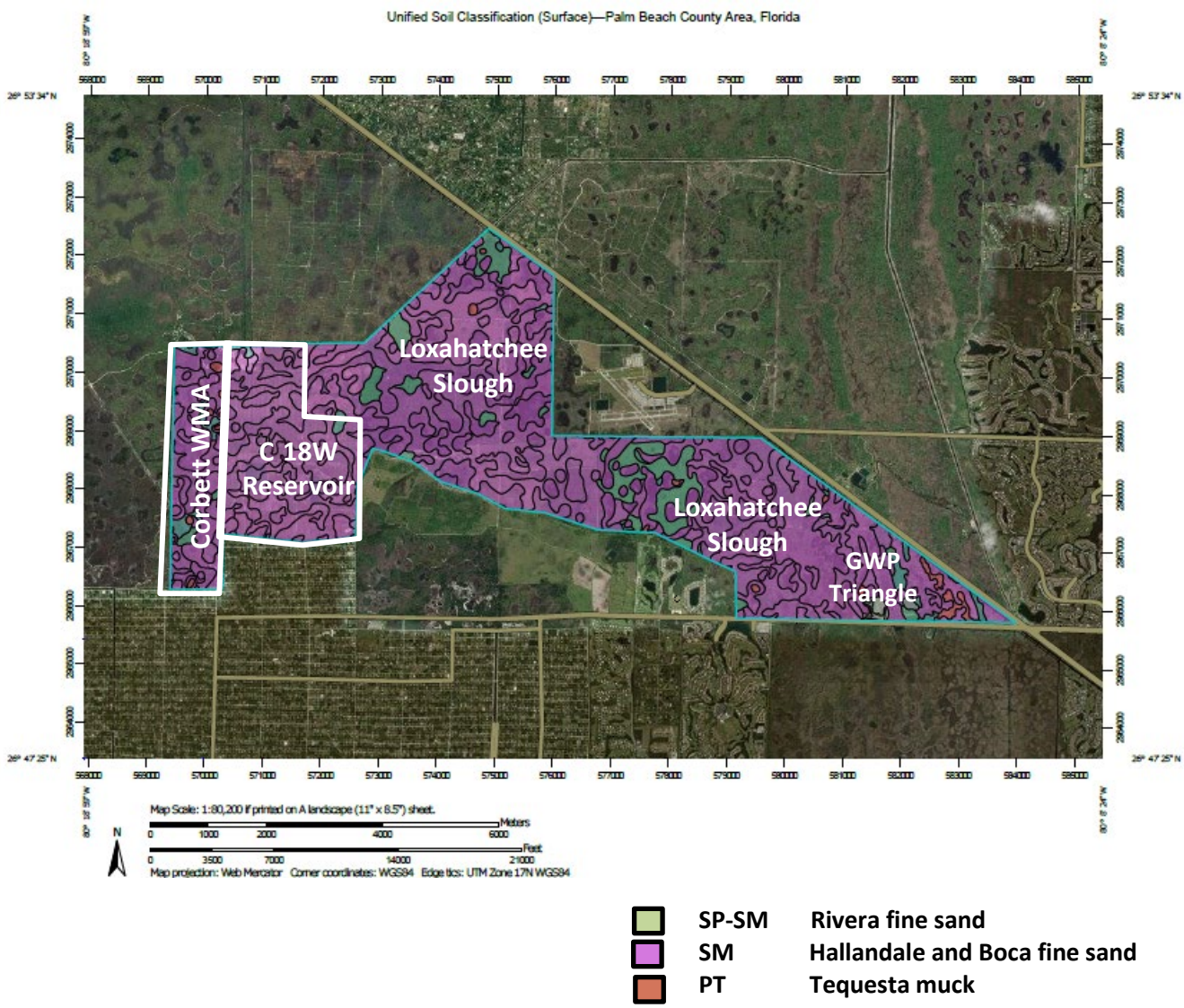


Figure A-6. Surface soil classification at C-18W reservoir footprint and adjacent areas.

A geotechnical and hydrogeological investigation was completed in the J.W. Corbett WMA (Gannett Fleming, Inc., 2016), which provided lithological and hydrogeological data for the undeveloped area in the western portion of flow-way 2. One standard penetration test (SPT) boring (to -80 ft NAVD88, 100 ft bls) was completed, and samples consisted of surficial peat (PT), and SP-SM (poorly graded sand with silt) and SP (poorly graded sand) at depth. Three groundwater monitor wells were constructed, with screened intervals at 95 to 100 ft bls, 34 to 39 ft bls, and 10 to 15 ft bls. A geophysical log was obtained from the deepest borehole, which showed an increase in fine-grained sediment at a depth range between 40 and 60 ft bls. Constant rate aquifer performance tests in each monitor well yielded hydraulic conductivity values that ranged between 1.41 x 10^{-4} cm/sec and 2.65 x 10^{-4} cm/sec (0.40 ft/day to 0.75 ft/day) using the Hantush solution.

A.7.2.2 Geology and Hydrogeology

Shallow subsurface characteristics and hydrogeologic settings of the surficial aquifer system and Floridan Aquifer System are discussed in the following subsections.

A.7.2.2.1 Shallow Subsurface Geotechnical Characterization

Arcadis (2016) conducted a subsurface exploration to support the design of a flow equalization basin at the C-18W reservoir site. Twenty-four boreholes were drilled, to depths of 100 ft bls (11), 50 to 60 ft bls (3), and 30 to 36 ft bls (10). Most borings showed marine sands (SP), sands with silt (SP-SM), and silty sand (SM) often with shell fragments. These sediments were deposited during Pleistocene high sea levels. Discontinuous indurated limestone layers having thicknesses of 1 to 10 ft are present at depths of 30 to 85 ft bls. The formation nomenclature for the Pleistocene marine sediment package is not clearly defined in this area. Reese and Wacker (2009) recognize four lithostratigraphic marker units that serve as the top of a distinct sedimentary package. In their framework, marine sands and limestone at C-18W reservoir at depths to 100 ft bls would correlate with the Fort Thompson Formation. Deeper Pliocene-aged units (Tamiami Formation) and Miocene-aged units (Hawthorn Group) were not encountered in the Arcadis (2016) subsurface exploration. The upper surface of the Hawthorn Group occurs at an approximate depth of 160 ft bls (Miller, 1987). Representative cross-sections at the proposed C-18W reservoir site can be found in **Annex G-1** to this Engineering Appendix.

A.7.2.2.2 Hydrogeologic Setting of the Surficial Aquifer System

The Surficial Aquifer System (SAS or water table aquifer) is included in the marine deposits that overlie Hawthorn Group sediments in the project area. Measurements of hydrologic characteristics (hydraulic conductivity, transmissivity, and storativity) were obtained from aquifer performance tests conducted for the DDR (Arcadis, 2016). Constant rate aquifer performance test results were collected at two locations within the reservoir footprint, at depths of less than 60 ft bls. The SAS is characterized by low to moderate transmissivity values ranging between 1,000 ft^2/day and 4,450 ft^2/day, and storage coefficients that range between 3.0 x 10^{-4} and 9.0 x 10^{-4}.

Constant head permeability tests also were conducted at 8 locations within the C-18W reservoir footprint, at three depths (10 to 15 ft bls, 40 to 45 ft bls; and 95-100 ft bls). Low to moderate hydraulic conductivity values are reported, ranging between 9.2 x 10^{-4} cm/sec and 1.7 x 10^{-2} cm/sec (10 to 15 ft bls); 7.8 x 10^{-4} cm/sec and 3.1 x 10^{-2} cm/sec (40 to 45 ft bls); and 4.2 x 10^{-4} cm/sec and 3.1 x 10^{-2} cm/sec (95 to 100 ft bls). Hydraulic conductivity values generally increase with depth; that is, there are more moderate values at depth. Constant rate aquifer performance tests in interior piezometers yielded hydraulic conductivity

values that ranged between 3.53 x 10^{-3} cm/sec and 1.59 x 10^{-2} cm/sec (10 ft/day to 450 ft/day) using the Cooper-Jacob solution for a leaky aquifer.

Groundwater quality in the upper portions of the SAS is fresh, because recharge infiltrates primarily from rainfall. Groundwater quality becomes increasingly saline towards the base of the SAS due to mixing of fresh groundwater with relict seawater (Reese and Wacker, 2009). Shallow SAS groundwater quality at C-18W reservoir shows chloride concentrations that range between 19 mg/L and 80 mg/L (PB-715, -58 ft NAVD88; SFWMD DBHYDRO; Reese and Wacker, 2009; Arcadis, 2016). Deeper SAS groundwater quality near C-18W reservoir site is brackish, showing chloride concentration of 1,250 mg/L (PB-830, approximately 70 ft bls; Reese and Wacker, 2009).

A.7.2.2.3 Hydrogeologic Setting of the Floridan Aquifer System

It is necessary to define the hydrogeologic setting of lithologic units and aquifers below the SAS because ASR is proposed to augment storage at the C-18W reservoir. A generalized hydrogeologic setting showing deeper subsurface geologic units that are applicable to ASR is shown in **Figure A-7**. Hydrostratigraphic setting is described below, starting from youngest to the oldest formations below the SAS.

Hawthorn Group (Late Oligocene-Miocene) sediments underlie Pliocene and Quaternary marine sediments throughout south Florida, including Palm Beach County (Miller, 1987; Reese and Memberg, 2000; Reese and Richardson, 2008). The Group consists of two formations, the Arcadia Formation and the overlying Peace River Formation. Hawthorn Group sediments are distinguished from underlying limestones by their high and variable siliciclastic and phosphatic content, gray-green coloration, and gamma-ray log response (Reese and Richardson, 2008). The basal Hawthorn unit is phosphate-rich, and shows pronounced gamma-ray log responses that contrast with low response in the phosphate-poor Ocala Limestone. Clays occur within the Arcadia Formation, and dolomite is the primary carbonate mineral (Scott, 1988). The Arcadia Formation/basal Hawthorn Group lies unconformably on the Ocala Limestone in the project area. The depth of the base of the Hawthorn Group near C-18W reservoir is 780 ft bls, and thickness is 320 ft (PB-1197; Reese and Memberg, 2000).

The Hawthorn Group sediments serve as confinement (Intermediate Confining Unit, or ICU) between the SAS and the FAS. The base of the ICU occurs at 575 ft bls at the L-8 site (PBF-15; Anderson, 2008), from 925 ft to 1,130 ft bls at the Town of Jupiter wellfield (RO-1 through RO-7; Stemle, Andersen & Associates, Inc., 1994), and 1,050 ft bls at PB-1197 (Reese and Memberg, 2000). The basal Hawthorn Group contact is a major unconformity with underlying limestones of Early Oligocene and Late Eocene age. A preferential flow zone often occurs at or slightly below this major unconformity (Reese and Richardson, 2008).

There is no agreement on the lithostratigraphic classification of the limestone unit that underlies the basal Hawthorn Group unconformity in Palm Beach County (Reese and Richardson, 2008). The Suwannee Limestone of early Oligocene age has been identified by some, or interpreted as absent by others. If present, the Suwannee Limestone will overlie the Ocala Limestone. The Ocala Limestone (Late Eocene) is a chalky to fossiliferous mud-rich to calcarenitic limestone. Where present, this formation occurs at depths of 900 to 1500 feet bls (Reese and Richardson, 2008), and thins to the east. The Ocala Limestone rests unconformably on the Avon Park Formation. A minor preferential flow zone often occurs near the base of the Ocala Limestone.

The uppermost permeable zone of the FAS is the upper Floridan aquifer (UFA), and occurs primarily in the Ocala Limestone in central Palm Beach County. The UFA is the shallowest storage zone used for ASR in south Florida, and occurs at depths of 850 ft to 1,250 ft bls at PBF-15 (PBF-15, Anderson, 2008; W-16182 Royal Palm Beach and PBF-3, in Reese and Richardson, 2008).

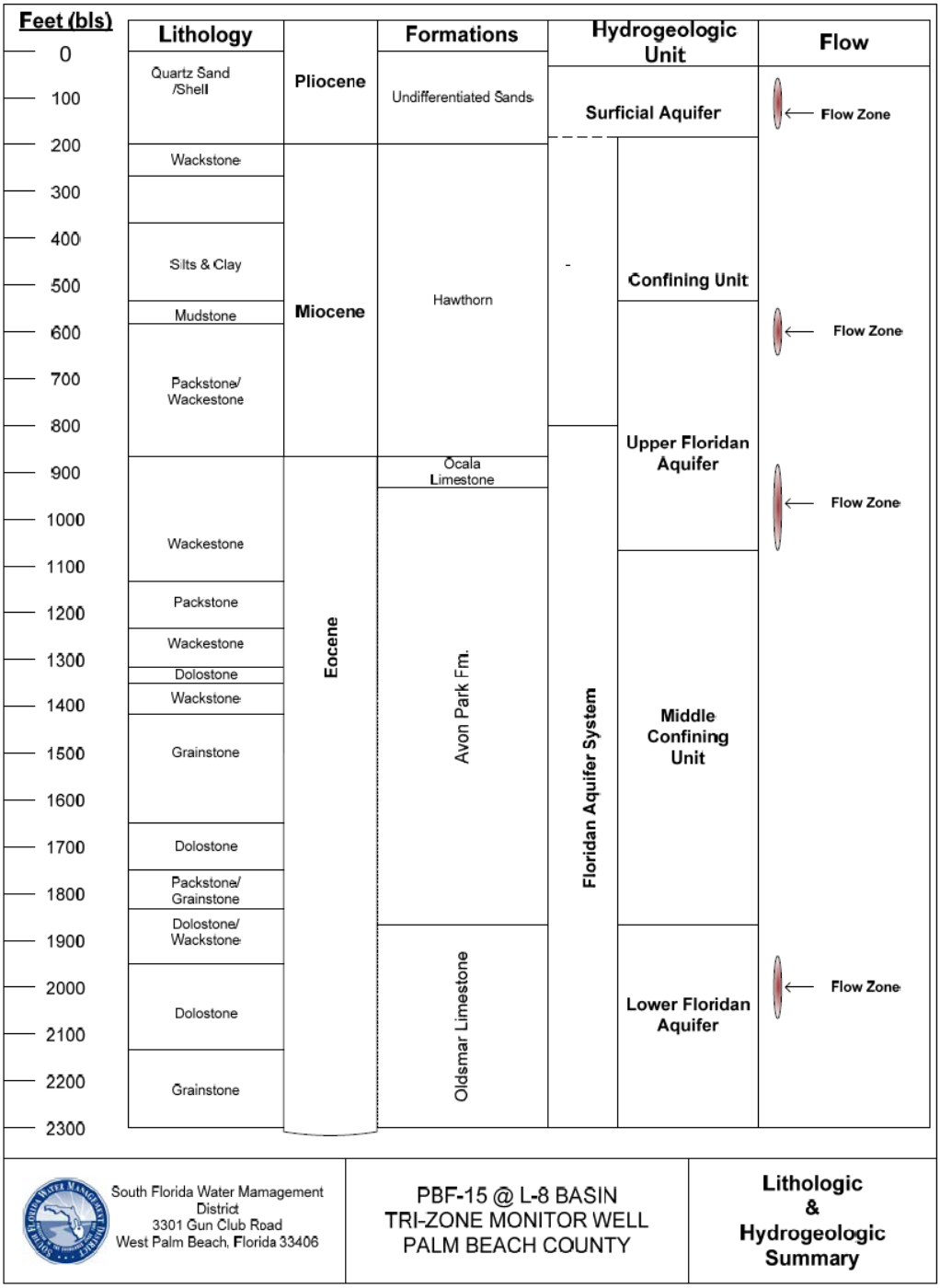


Figure A-7. Generalized hydrogeologic framework for southern Palm Beach County (PBF-15). From Anderson (2008).

The Avon Park Formation (Middle Eocene) is a thick, fine-grained micritic to fossiliferous limestone, dolomitic limestone, and dolostone (Reese and Richardson, 2008). Dolomitic portions of the middle to upper portions of the Avon Park Formation are fractured, resulting in high permeability. This permeable zone is known as the Avon Park Permeable Zone (APPZ). The APPZ is a thick, transmissive aquifer that occurs below the UFA throughout south Florida. The upper surface of the APPZ occurs at variable depths ranging between 1,100 ft and 1,500 ft bls; thickness ranges between 100 ft and 300 ft in Palm Beach County (Reese and Richardson, 2008). Although the APPZ has not been utilized previously for ASR in this area, it will be considered as a possible storage zone at the C-18W reservoir site.

Native groundwater quality in the UFA is fresh at interior locations, and becomes more saline toward the coast. At the Kissimmee River ASR (KRASR) system in Okeechobee County, native UFA groundwater quality is fresh, with chloride concentrations range between 150 mg/L and 281 mg/L (USACE, 2013). At Port Mayaca in western Palm Beach County, chloride concentrations of the UFA range between 578 mg/L and 740 mg/L (EXPM-1, MF-37U; SFWMD DBHYDRO). At the C-18 Test ASR well near Jupiter, chloride concentrations are brackish and range between 1,230 mg/L and 1,454 mg/L (990 ft to 1,280 ft; SFWMD DBHYDRO) but these values do not represent native groundwater.

Native groundwater quality in the APPZ generally is more saline than UFA groundwater. At KRASR, chloride concentrations (356 mg/L) indicates fresher water quality at the uppermost portion of the APPZ, and probably are not representative of the entire aquifer. At the Town of Jupiter Wellfield, the APPZ is interpreted to occur below 1,425 ft bls. Chloride concentrations in the APPZ were brackish (in 1994) and ranged between 1,230 mg/L and 2,000 mg/L (Stemle, Andersen & Associates, 1994). Groundwater quality of the APPZ varies considerably across the project area, but it is reasonable to expect that brackish water will be encountered at the C-18W reservoir site. ASR percent volume recoveries will be low during early ASR cycle tests, and large recharge volumes will be required to freshen the APPZ.

A.7.2.2.4 Feasibility of ASR at the C-18W Reservoir Site

A desk-top analysis was conducted using existing hydrogeologic data to evaluate the feasibility of ASR at the C-18W reservoir site and adjacent to the L-8 shallow storage feature. Surface water from the reservoir will recharge into the Floridan Aquifer System. Stored water will be recovered into the C-18W reservoir for subsequent distribution. For this evaluation, hydrogeologic data obtained from exploratory boreholes completed in western Palm Beach and Martin Counties are interpreted. Locations of existing exploratory boreholes that were converted into monitoring wells or deep injection wells are shown in **Figure A-8.**

For ASR implementation at the C-18W reservoir site, hydrogeologic data are available from the following subsurface investigations. These subsurface investigations indicate that portions of the Upper Floridan Aquifer are permeable, and can serve as ASR storage zones.

<u>Pratt & Whitney Injection Well</u>
The closest well that penetrates the entire Floridan Aquifer System (FAS) is located at the Pratt & Whitney injection deep well facility, approximately 4 miles north of the C-18W reservoir. The wells at this facility are completed within the Boulder Zone (approximately 3,000 feet bls). Information from the Pratt & Whitney investigation reveals that the top of the Upper Floridan Aquifer (UFA) occurs within the Suwannee Limestone at approximately 800 feet bls. Additionally, there are deeper, permeable dolomitic portions of the aquifer that may be also available for storage (CH2MHill, 1985).

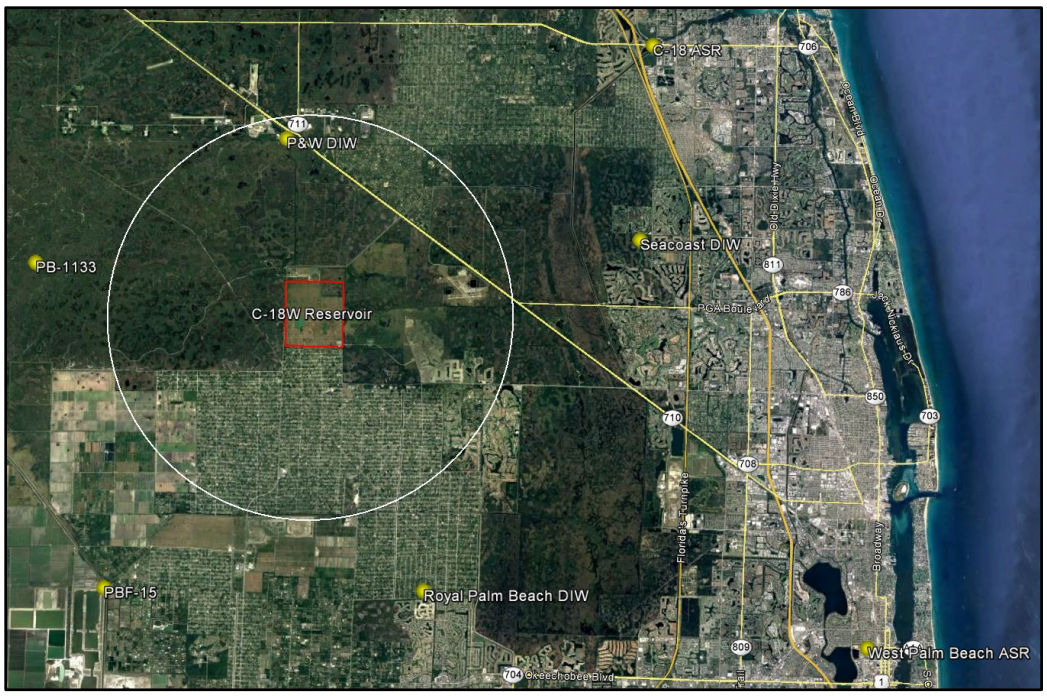


Figure A-8. FAS wells and boreholes near the C-18W reservoir. Circle shows a 5-mile radius around the C-18W reservoir.

Seacoast Injection Well

The stratigraphy at the Seacoast Utility deep injection well system, located 9 miles east of the C-18W reservoir footprint is similar to that in the Pratt & Whitney deep injection well system. This investigation shows that there are several potential storage zones within the upper portions of the FAS, at 900 feet bls, and in underlying dolomitic intervals to a depth of approximately 2,500 feet bls (CH2MHill, 1989).

C-18 Test ASR Well

During 1976, the Florida Department of Natural Resources constructed and tested an ASR system along the C-18 Canal, approximately 11 miles northeast of the proposed C-18W reservoir site. A 12-inch diameter test ASR well drilled within the upper FAS, underwent 4 short test cycles, at recharge rates of 3 million gallons per day (MGD) at relatively low pressures. During the fourth cycle, the system exhibited a recovery efficiency of 36 percent after recharging for only one month and a storage period of 120 days. The testing indicated that the Upper Floridan Aquifer was feasible. The limited cycle testing program (small recharge volumes stored for limited durations), is not representative of typical large volume, longer storage durations cycles that are typically permitted for ASR systems today. (Unpublished Report, 1977; Reese, 2006).

West Palm Beach ASR Well

The City of West Palm Beach is currently operating an ASR system approximately 11 miles southeast of the C-18W reservoir site. The ASR system is operating at recharge rates in excess of 7 MGD within the upper FAS, and is currently conducting test cycles using filtered surface water. The City has obtained a water quality criteria exemption from the FDEP that is allowing the ASR system to operate without a

disinfection treatment process. This regulatory mechanism may be available for pursuit at this project (CH2MHill, 1998).

US Sugar ASR Test Well

During 1992, United States Sugar Corporation constructed a test well system approximately 11 miles to the west of the C-18W reservoir footprint. A 6-inch test well, cased to the top of the FAS at a depth of 925 feet bls, with an open-hole extending to 1,690 feet bls. The well was hydraulically tested and indicated that the upper FAS exhibited a transmissivity of 540,000 gpd/ft and a corrected specific capacity of between 68 and 97 gallons per minute per foot of drawdown. These attributes indicate that a larger diameter ASR well at this location could be pumped at rate of 5 MGD while exhibiting reasonable drawdowns and recharge pressures. Additionally, the water within the upper FAS exhibited chloride concentrations of between 1,100 and 1,800 mg/L, which are similar to other ASR facilities that have experiences high recovery efficiencies in the lower east and west coasts of Florida (Missimer & Associates, 1993).

Royal Palm Beach Injection Well

The stratigraphy at the Royal Palm Beach deep injection well, located seven miles south of the C-18W reservoir footprint, confirms that the upper FAS is present at a depth of 900 feet bls, and artesian limestone and dolomitic intervals are present to a depth of over 2,500 feet bls. These findings indicate that multiple, vertically "stacked" zones may available for high capacity water recharge and storage in this area (CH2MHill, 1988).

PB-1133

In 1955, Amerada completed a petroleum exploration borehole to a depth of 11,000 ft in what is now J.W. Corbett WMA, immediately northwest of the proposed C-18W reservoir. The UFA was identified at depths between 685 and 1,172 feet bls, and was characterized as permeable (Miller, 1986).

PBF-15

A tri-zone monitor well was constructed by the SFWMD to measure groundwater levels continuously in FAS permeable zones. Two permeable zones suitable for storage were identified between depths of 890 to 1,100 feet bls. While drilling through this interval, the well drilling process experienced numerous lost circulation zones, indicating highly fractured or otherwise permeable strata within the upper FAS that would be capable of accommodating high-capacity recharge and recovery rates (Anderson, 2008).

A.7.2.3 C-18W Reservoir Seepage Analysis

For the preliminary design of the C-18W reservoir and adjacent seepage canal, SEEP/W was used to predict the resulting phreatic surface and seepage from the operation of the reservoir. The software was also used to verify that the embankment/canal will be safe with respect to uplift pressures and exit gradients. As such, the proposed embankments and adjacent canals were the primary focus of the modeling and not regional effects. Regional effects should be analyzed by 3D modeling prior to final design. SEEP/W is a 2D finite element program for analyzing groundwater seepage and pore-water pressure within porous soil and rock and Version 9.1.1.16749 was used for all analyses. SEEP/W inputs consist of cross sectional geometry, hydraulic conductivity, and boundary conditions for the flow domain.

Output results consist of phreatic surface, head distribution, hydraulic gradient, flow directions, and flow quantities within the flow domain.

A 2D seepage model was developed from the materials encountered by the borings performed for this portion of the Project. These materials, generally a mix of poorly graded sand (SP) and silty sand (SM), were separated into model layers based on constant head test results and/or pump test data. A section was modeled through the west, south, and east sides of the proposed reservoir with the understanding that further sections may be analyzed during the design phase of the project **(Figure A-9)**.

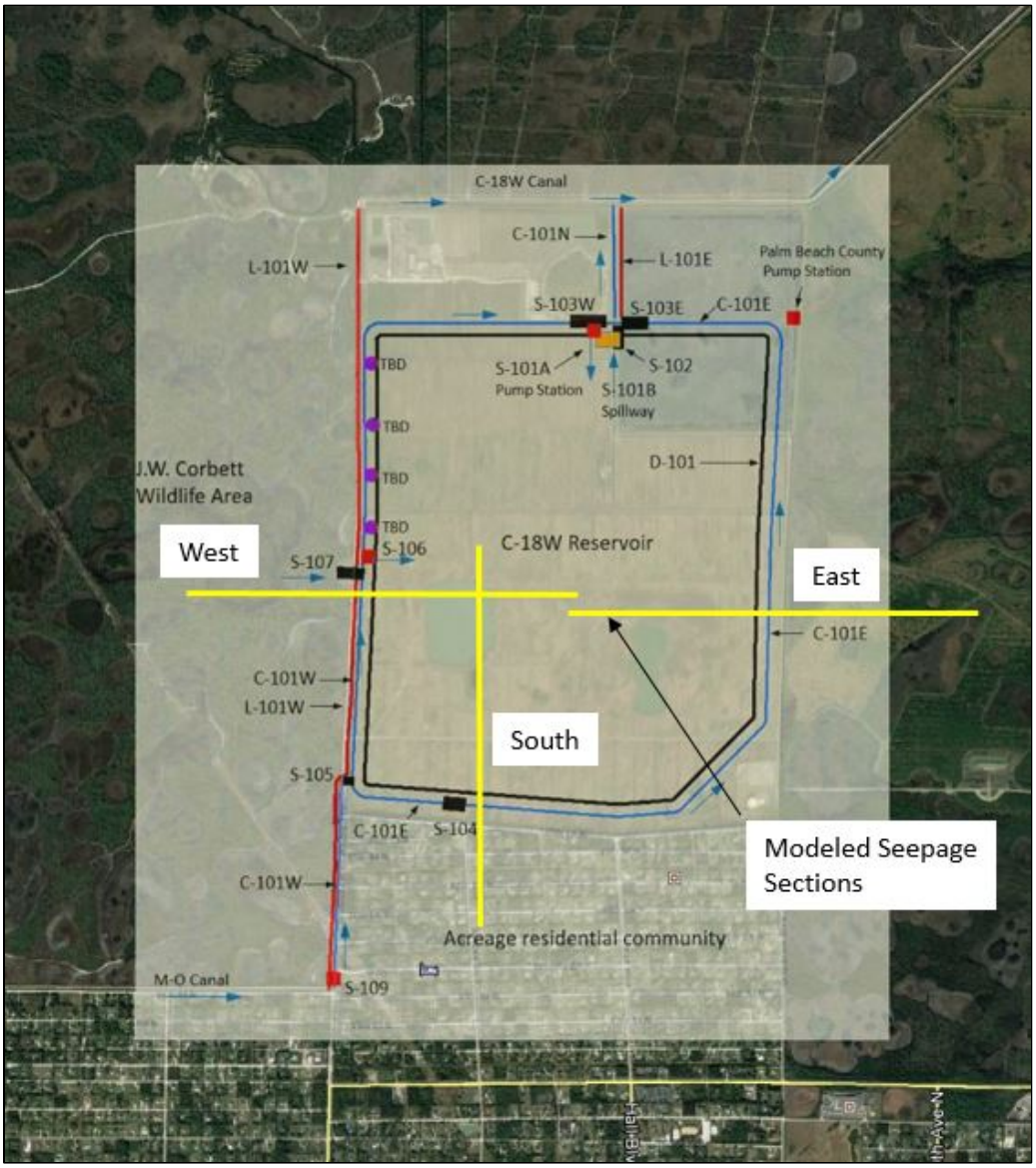


Figure A-9. Modeled seepage section locations at the C-18W reservoir.

A.7.2.3.1 Design Parameters for the 2D Seepage Analysis

The design parameters selected for the 2D seepage modeling are based on field data, laboratory test results, and published values for similar materials (see paragraph References). Anisotropy values (ky/kx) are based on Figures 8.3.2.3.1-1 and 8.3.2.3.3-1 in the Bureau of Reclamation (BOR) reference *Design*

Standards No. 13, Embankment Dams, Chapter 8, Seepage. Unless otherwise stated, all hydraulic conductivity values discussed herein are in the horizontal direction (i.e. Kx). **Figure A-10** shows some of the materials considered for the west section through the C-18W embankment.

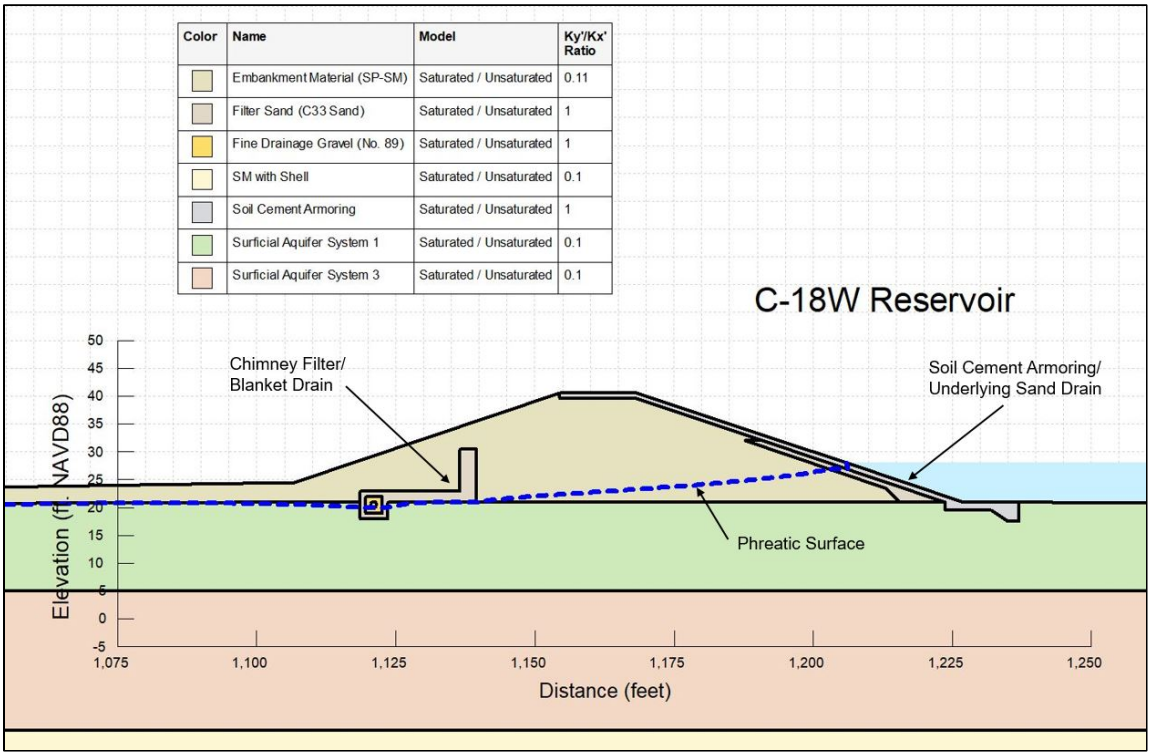


Figure A-10. West embankment section looking north.

A.7.2.3.2 West Section Values

<u>Embankment Material</u>

Based on borings performed for the Mecca Impoundment (i.e. C-18 West Reservoir), this material will likely be a blend of poorly-graded sand and silty sand characterized by a saturated hydraulic conductivity value of 1.0E-3 cm/s (estimated from published values for embankment fill by the BOR).

<u>Surficial Aquifer System 1</u>

Considering the constant head tests that were performed along the western side of the proposed reservoir footprint, the test results were organized by depth below land surface (bls) as shown in **Table A-2**. In viewing the results in this manner, three "groups" of similar hydraulic conductivity were chosen to represent the foundation strata along the western side of the reservoir **(Table A-3).** The first foundation layer, named Surficial Aquifer System 1 (SAS 1), was thus modeled from the ground surface to a depth of 16 ft and the average "K" value for this range (2.47E-4 cm/s) was used for the seepage modeling.

<u>Surficial Aquifer System 3</u>

This foundation layer was modeled from a depth of 16 ft to a depth of 41 ft bls and from a depth of 46 feet to a depth of 86 ft (bls). The average "K" value for this "group" (1.27E-3 cm/s) was used in the seepage modeling.

SM with Shell

Based on the constant head test results and the description of the materials encountered within the test intervals, the sand/silty-sand encountered from a depth of 41 ft to 46 ft bls is more permeable than that encountered directly above or below. This is true across the site with shell fragments being noted in the SE corner of the site for the same depth range. To be conservative, the constant head value of 6.58E-3 cm/s was used for this depth range and for a depth of 86 ft to 191 ft bls.

Filter Sand (ASTM C33 fine aggregate)

Based on permeability tests run on different aggregate sizes for the Herbert Hoover Dike Project, a value of 153 ft/d (5.39E-2 cm/s) was selected for the C33 sand that was modeled for the chimney filter material and the blanket drain material.

Fine Drainage Gravel (ASTM Size No. 89)

Based on permeability tests run on different aggregate sizes for the Herbert Hoover Dike Project, a value of 4.59E-1 cm/s was selected for the No. 89 aggregate that will surround the internal drainpipe within the C-18W embankment.

Soil Cement Armoring

Considering that the soil cement can be made from suitable material obtained from on-site sources (i.e. the seepage canal excavation and interior borrow operations), a silty sand with grain-size values close to that considered for the filter analysis (and with a cement content between 7 and 11 percent) was selected from Table 5.2 of ACI's 2009 Report on Soil Cement (2.1E-5 cm/s).

Table A-2. Constant head test results used for the west seepage section.

Soil Classification	Monitoring ID	Test Interval (ft, bgs)	Hydraulic Conductivity Kx (cm/s)	Group Designation*
SP-SM	MFET15-PZ2	5 - 10	1.76E-4	SAS 1
SC	MFET06-PZ1	6 - 8	2.47E-4	SAS 1
SP-SM	MFET12-PZ2	6 - 11	3.88E-4	SAS 1
SP-SM	MFET15-PZ1	10 - 15	1.06E-4	SAS 1
SP-SM	MFEB4-GW3	10 - 15	1.06E-3	SAS 3
SP-SM	MFET12-PZ1	11 - 16	7.41E-4	SAS 3
SM	MFET07-PZ2	20 - 25	1.69E-3	SAS 3
SP-SM	MFET11-PZ1	25 - 30	1.55E-3	SAS 3
SP-SM	MFEB4-GW2	40 - 45	6.60E-3	SM with Shell
SP-SM	MFET07-PZ1	65 - 70	1.23E-3	SAS 3
SW-SM/SP-SM	MFEB4-GW1	95 - 100	3.42E-3	SM with Shell

* SAS 2 was used in the South and East Seepage Sections

Table A-3. Hydraulic parameters for each material type for the west seepage section.

Material	Modeled Bottom Elev. (ft, NAVD88)	Kx (cm/s)	Kx (ft/s)	Ky/Kx
Embankment Material (SP-SM)	21.0	1.0E-3	3.28E-5	0.11
Surficial Aquifer System 1 (SP-SM)	5.0	2.47E-4	8.10E-6	0.1
Surficial Aquifer System 3 (SP-SM)	-20.0	1.27E-3	4.16E-5	0.1
SM with Shell	-25.0	6.58E-3	2.16E-4	0.1
Surficial Aquifer System 3 (SP-SM)	-65.0	1.27E-3	4.16E-5	0.1
SM with Shell	-170.0	6.58E-3	2.16E-4	0.1
Filter Sand (C33 Sand)	Varies	5.39E-2	1.77E-3	1
Fine Drainage Gravel (No. 89)	Varies	4.59E-1	1.50E-2	1
Soil Cement Armoring	Varies	2.1E-5	6.89E-7	1

A.7.2.3.3 South Section Values

Embankment Material

Same as discussed for the west section.

Surficial Aquifer System 1

In light of the constant head tests that were performed along the southern side of the proposed reservoir footprint, the test results were organized by depth bls as shown in **Table A-4.** Initially it was thought that four "groups" of similar hydraulic conductivity would be used to represent the foundation strata along the southern side of the reservoir. However, as pump test data was also available from three of the monitoring wells along this portion of the footprint, pump test results were used where applicable over the constant head results. The first foundation layer (SAS1) was modeled from the ground surface to a depth of 9.5 feet and the "K" value from the pump test with monitoring well MFEB3-GW3 (6.0E-3 cm/s) was used for the seepage modeling **(Table A-5)**.

Surficial Aquifer System 2

This foundation layer was modeled from a depth of 9.5 ft to a depth of 14.5 ft bls and the "K" value from the pump test with monitoring well MFEB3-GW3 (6.0E-3 cm/s) was used for the seepage modeling.

Surficial Aquifer System 3

This foundation layer was modeled from a depth of 14.5 ft to a depth of 39.5 ft bls and from a depth of 44.5 ft to a depth of 84.5 ft bls. The "K" value from the pump test with monitoring well MFEB3-GW2 (6.35E-3 cm/s) was used for the seepage modeling.

SM with Shell

Based on the constant head test results and the description of the materials encountered within the test intervals, the sand/silty-sand encountered from a depth of 39.5 ft to 44.5 ft bls is more permeable than that encountered directly above or below. This is true across the site with shell fragments being noted in the SE corner of the site for the same depth range. A conservative constant head value of 1.18E-2 cm/s was used for this depth range and for a depth of 84.5 ft to 190.5 ft bls.

Filter Sand (ASTM C33 fine aggregate)
Based on permeability tests run on different aggregate sizes for the Herbert Hoover Dike Project, a value of 5.39E-2 cm/s was selected for the C33 sand that was modeled for the chimney filter material and the blanket drain material.

Fine Drainage Gravel (ASTM Size No. 89)
Based on permeability tests run on different aggregate sizes for the Herbert Hoover Dike Project, a value of 4.59E-1 cm/s was selected for the No. 89 aggregate that will surround the internal drainpipe within the C-18W embankment.

Soil Cement Armoring
Same as that in the western section.

Table A-4. Constant-head and pump test values used for the south seepage section.

Soil Classification	Monitoring ID	Test Interval (ft, bgs)	Hydraulic Conductivity Kx (ft/d)	Group Designation
SP-SC	MFEB2-GW3	10 - 15	17*	SAS 1
SP-SM	MFEB3-GW3	10-15	17*	SAS 2
SP-SM	MFET11-PZ1	25-30	18*	SAS 3
SM	MFEB3-GW2	40-45	18*	SAS 3
SM	MFEB2-GW2	40-45	33.5	SM with Shell
SP-SM	MFET05-PZ1	40-60	18*	SAS 3
SM	MFET04-PZ1	55-60	18*	SAS 3
SW-SM	MFEB3-GW1	95-100	35**	SM with Shell
SW-SM/SP-SM	MFEB2-GW1	95-100	35**	SM with Shell

* From pump test data.

**From the constant head test result from monitoring well MFEB2-GW2.

Table A-5. Hydraulic parameters for each material type in the south seepage section.

Material	Modeled Bottom Elev. (ft, NAVD88)	Kx (cm/s)	Kx (ft/s)	Ky/Kx
Embankment Material (SP-SM)	19.5	1.0E-3	3.28E-5	0.11
Surficial Aquifer System 1 (SP-SM)	10.0	6.0E-3	1.97E-4	0.1
Surficial Aquifer System 2 (SP-SM)	5.0	6.0E-3	1.97E-4	0.1
Surficial Aquifer System 3 (SP-SM)	-20.0	6.35E-3	2.08E-4	0.1
SM with Shell	-25.0	1.18E-2	3.88E-4	0.1
Surficial Aquifer System 3 (SP-SM)	-65.0	6.35E-3	2.08E-4	0.1
SM with Shell	-171.0	1.18E-2	3.87E-4	0.1
Filter Sand (C33 Sand)	Varies	5.39E-2	1.77E-3	1
Fine Drainage Gravel (No. 89)	Varies	4.59E-1	1.50E-2	1
Soil Cement Armoring	Varies	2.1E-5	6.89E-7	1

* From pump test data. **From the constant head test result from monitoring well MFEB2-GW2.

A.7.2.3.4 East Section Values

The hydraulic conductivity values for the east section, shown in **Table A-6**, were determined in the same manner as that described for the west and the south sections.

Table A-6. Constant-head and pump test values used for the east seepage section.

Soil Classification	Monitoring ID	Test Interval (ft, bgs)	Hydraulic Conductivity Kx (cm/s)	Group Designation
SM	MFET14-PZ2	5 - 10	2.12E-4*	SAS 1
SP	MFET13-PZ2	6 - 11	2.12E-4*	SAS 1
SP-SC	MFEB2-GW3	10 - 15	2.12E-4*	SAS 1
SM	MFET14-PZ1	10 - 15	2.12E-4*	SAS 1
SP-SC	MFET13-PZ1	11 - 16	2.12E-4*	SAS 1
SP-SM	MFET02-PZ1	10 - 30	1.20E-2**	SAS 2
SP-SC	MFET09-PZ1	20 - 25	1.20E-2**	SAS 2
SP-SM	MFET01-PZ1	20 - 25	1.20E-2**	SAS 2
SP-SM	MFET03-PZ1	25 - 30	1.20E-2**	SAS 2
SM	MFEB2-GW2	40 - 45	1.18E-2	SM with Shell
SP	MFET10-PZ1	45 - 50	2.54E-3	SAS 3
SW-SM/SP-SM	MFEB2-GW1	95 - 100	1.18E-2***	SM with Shell

* Average constant head value for the group.
**From pump test data.
***From the constant head test result from monitoring well MFEB2-GW2.

Table A-7. Hydraulic parameters for each material type in the east seepage section.

Material	Modeled Bottom Elev. (ft, NAVD88)	Kx (cm/s)	Kx (ft/s)	Ky/Kx
Embankment Material (SP-SM)	20.5	1.0E-3	3.28E-5	0.11
Surficial Aquifer System 1 (SP-SM)	5.0	2.12E-4	6.95E-6	0.1
Surficial Aquifer System 2 (SP-SM)	-20.0	1.20E-2	3.94E-4	0.1
SM with Shell	-25.0	1.18E-2	3.87E-4	0.1
Surficial Aquifer System 3 (SP-SM)	-65.0	2.54E-3	8.33E-5	0.1
SM with Shell	-170.0	1.18E-2	3.87E-4	0.1
Filter Sand (C33 Sand)	Varies	5.39E-2	1.77E-3	1
Fine Drainage Gravel (No. 89)	Varies	4.59E-1	1.50E-2	1
Soil Cement Armoring	Varies	2.1E-5	6.89E-7	1

A.7.2.3.5 Embankment/Seepage Canal General Geometries

The embankment will have a 14-ft crest width, 1V:3H side slopes, and a maximum vertical height of 21 ft (approximately). The adjacent exterior seepage canal will be trapezoidal in shape with 1V:3H side slopes, 20-ft bottom width, and a bottom elevation at 5 ft NAVD88. Optimization during the PED phase of the

project may decrease the size of the seepage canal – but any decrease in this feature will have to be made up with additional borrow from the interior of the reservoir.

A.7.2.3.6 West Embankment Internal/External Components

The west embankment will have a number of internal features designed to safely handle seepage entering the embankment during its service life **(Figure A-11)**. Starting on the interior (i.e. reservoir) side, the crown, side slope, and toe will be armored with 12 in of soil cement for the primary purpose of erosion control. This armoring will also reduce the rate at which seepage will eventually enter the embankment. Just below the soil cement will be a geotextile fabric designed to keep any embankment fill from being lost through the soil cement should a large enough crack occur in the soil cement. Below the geotextile will be a 12-in layer of non-calcareous, C33 sand, placed to provide a drainage path for water trapped within the embankment should a rapid drawdown of the pool be necessary. This layer will be placed up to elevation of 32 ft NAVD88. **Figure A-12** shows a concept view of the drainage outlet for this "under armor" drain. The pipes shown will be profile wall corrugated HDPE pipe and 6-in in diameter.

Closer to the downstream side of the embankment but still within the embankment proper, a vertical chimney filter (3 ft wide, top elevation at 30.5 ft NAVD88) will be placed to intercept any through seepage. Seepage entering the chimney will flow downwards and into a horizontal blanket drain. The material selected for the chimney and the blanket drain will be non-calcareous, C33 sand, which meets filter criteria for the surrounding soil types (see Filter Design in **Annex B-1** Geotechnical). The blanket drain will end at a perforated drainpipe oriented parallel to the embankment axis. This drainpipe, 12-in in diameter and of the same material for the under armor piping, will be connected to non-perforated pipes that will exit the embankment and drain into the adjacent seepage canal. These discharge pipes will be appropriately spaced and will be covered with at least 2-3 ft of fill to protect the pipes from traffic loading. A two-stage filter (No. 89 stone and C33 sand) will surround the perforated pipe and the materials will also be non-calcareous. Lastly, the exterior slope will be covered with sod.

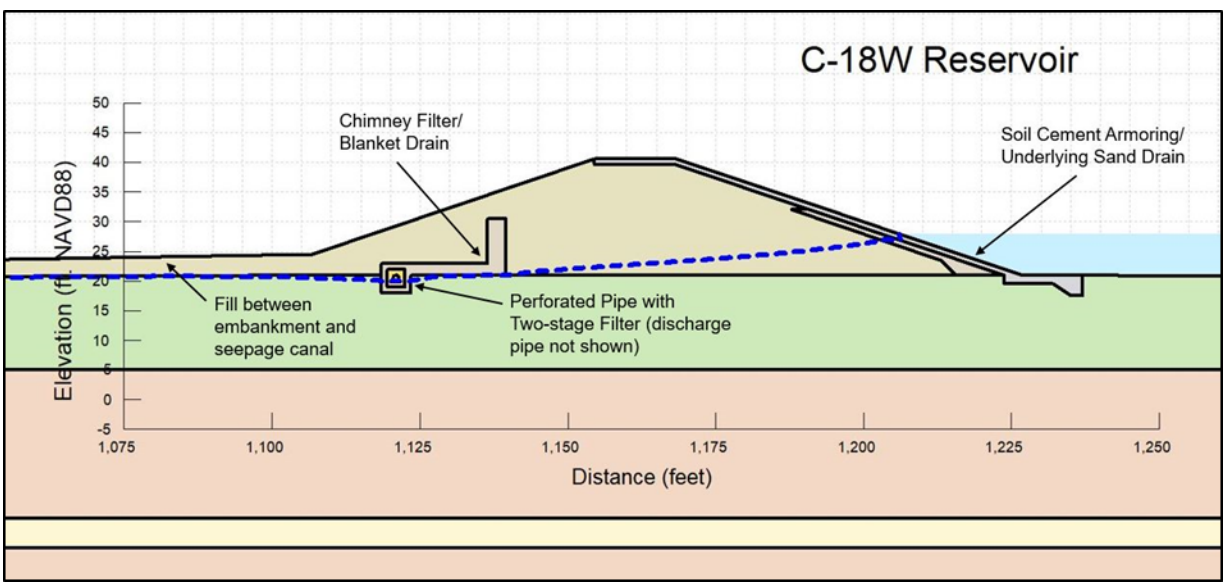


Figure A-11. West embankment typical cross-section.

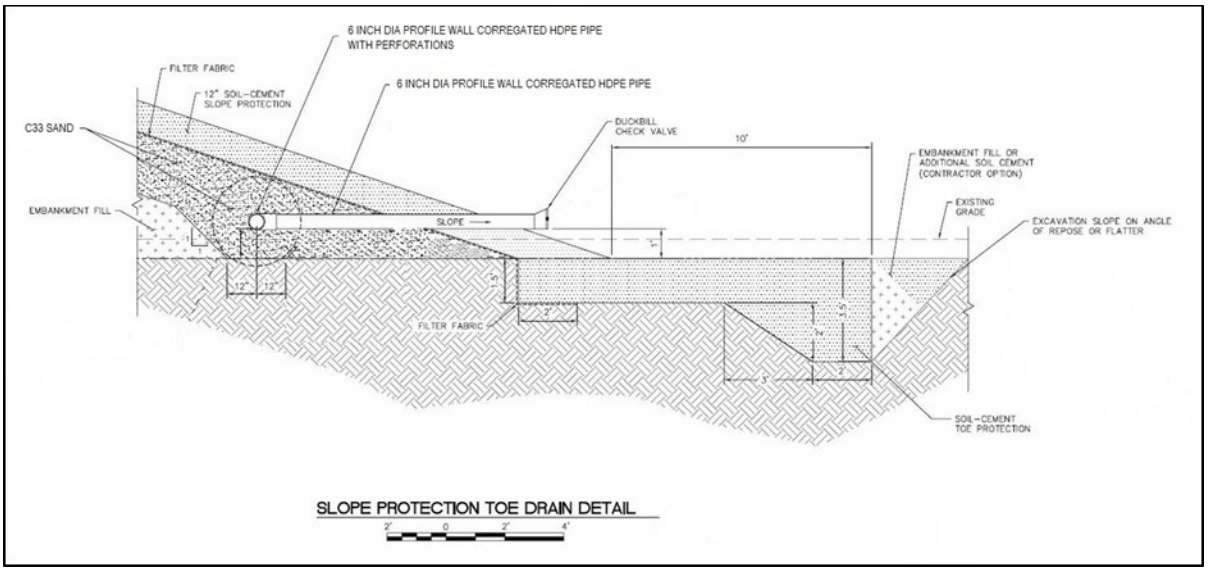


Figure A-12. C-18W reservoir, interior toe drain detail.

A.7.2.3.7 South Embankment Internal/External Components

With the exception of the drainage trench located below the downstream end of the internal drainage blanket, the south embankment will have the same internal/external components as the west embankment section. The drainage trench, filled with non-calcareous C33 sand, will be three feet wide and will extend down from the bottom of the drainage blanket to elevation 5 ft NAVD88 **(Figure A-13).**

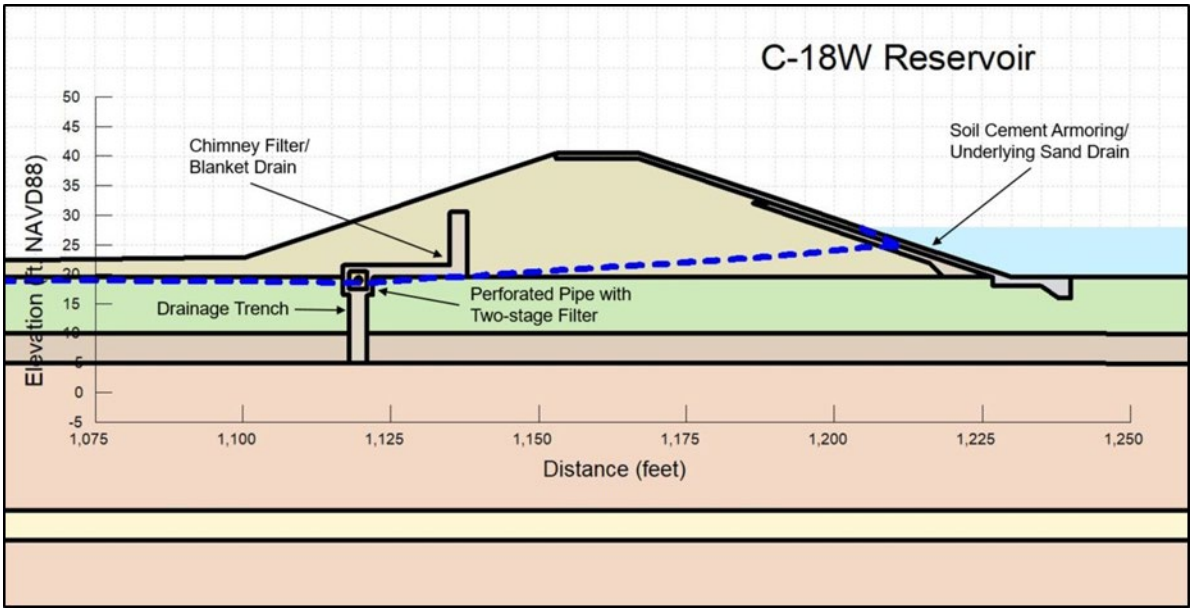


Figure A-13. South embankment typical cross-section.

A.7.2.3.8 East Embankment Internal/External Components

The east embankment will have the same internal/external components as the west embankment section.

A.7.2.3.9 Boundary Conditions

Three different types of boundary conditions were used in the seepage modeling: No Flow, Constant Head, and Potential Seepage Face.

- No Flow: Water cannot pass through the boundary and neither a total hydraulic head nor a flow rate has been specified.
- Constant Head: A total hydraulic head (in feet of water) has been specified at the boundary nodes. This boundary condition is applied at locations where the head value is known with reasonable confidence.
- Potential Seepage Face: Applied to nodes at the ground surface where seepage may, but does not necessarily, exit the model. When applied to a node, this boundary condition causes the SEEP/W software to determine which of the other two boundary conditions should be used at that node. If seepage exits a model face at the node having the potential seepage face boundary condition, that node is treated as having a constant head boundary using the node elevation as the constant head; otherwise the node is treated as having a no flow boundary.

These boundary conditions were applied as follows:

- A no-flow boundary was set along the bottom of the section, which was set at approximately 190 ft bls.
- With respect to the vertical face created by the right and left ends of each section, a no-flow boundary was also used. Each section was made long enough to have an equal distance inside/outside the reservoir.
- Where a water body was present (i.e. the reservoir pool, adjacent canal stage, etc.), a constant-head boundary was set to model that feature. These water surface elevations were provided by the Water Resources Engineering Branch.
- Where the above boundary conditions were not used, a potential seepage face boundary was used for most of the remaining nodes – which would allow the software to determine the water surface elevation at those locations.

A.7.2.3.10 Seepage Conditions Modeled

Two seepage conditions were modeled: Long-Term (steady-state seepage with a normal, average pool in the reservoir and a normal water level in the adjacent seepage canal) and Maximum Surcharge Pool (steady-state seepage with a probable maximum precipitation level pool in the reservoir and an elevated water level in the seepage canal).

A.7.2.3.11 Seepage Modeling Results at C-18W Reservoir

Whenever the exit gradient equals or exceeds the critical gradient of the material being analyzed, internal erosion could occur. From Design Standards No. 13, Chapter 8 (Seepage) by the Bureau of Reclamation (BOR), "It is worth reinforcing the concept that the internal gradient that might lead to the initiation of internal erosion may be as low as 0.02 to 0.08 for particularly susceptible soils. These internal (horizontal) gradients are much lower than the 'rule of thumb' critical gradient of 1.0 often assumed for exit (vertical) gradients." As a result, node pairs in each seepage section were examined to determine the likelihood of boiling/loss based on the magnitude and direction of seepage-flow velocity vectors at those locations. (In

SEEP/W, a large velocity vector indicates a higher flow velocity and therefore, is a good indicator of where higher exit gradients may be found.) Hydraulic exit gradients were calculated using the relationship:

$$ie = \frac{\Delta H}{L}$$

Where:
ie = average hydraulic exit gradient
ΔH = the change in total head between two node points
L = the distance between the nodes

The distance between node locations was generally 1 foot or more. Vertical exit gradients were compared to critical hydraulic gradients in the same direction to calculate a factor of safety (FOS) against boiling –

$$FOSboiling = \frac{icv}{iev}$$

Where:
icv = critical vertical gradient
iev = average hydraulic gradient in the vertical direction

The critical vertical gradient was calculated by dividing the effective unit weight of the soil through which the seepage is moving by the unit weight of water. For new dams, the BOR recommends a FOS of 4 against boiling (or heave). As this project feature will impound more than fifty acre-feet (ac-ft) under normal operating conditions, it is considered a dam by the definition found in ER 1110-2-1156.

The average hydraulic gradient in the horizontal direction was also calculated to analyze the potential for piping and erosion in terms of Bligh's creep ratio. Bligh's creep ratio (BCR) considers the base length of the hydraulic structure and the change in head across the structure. The equation used for this ratio is

BCR = L/H

where:

L = base length of the hydraulic structure (i.e. the line of creep)
H = change in head across the structure (i.e. headwater elevation – tailwater elevation)

From EM 1110-2-1901, Table 9-10, there are minimum values for BCR based on the foundation materials encountered. For very fine sand or silt (the primary foundation material encountered at the site), the minimum creep ratio is 18.

<u>West Section: Long Term</u>
For this condition, the difference in head between the normal pool (28 ft NAVD88) and the water surface in the canal (17.5 ft NAVD88) is 10.5 ft. Seepage exits the reservoir and flows towards the seepage canal and also down towards the more permeable layers modeled (i.e. the SM with Shell layer). The phreatic surface drops from the water surface elevation within the reservoir to the base of chimney filter, then proceeds horizontally and down into the perforated drainpipe located at the downstream end of the blanket drain. The perforated pipe receives approximately 1.85E-5 cfs of inflow (per linear foot of pipe) from the steady-state seepage. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the original ground surface between the exterior toe of the embankment and the seepage canal. This change in the groundwater elevation and the head values

driving it are shown in **Figure A-14**. If the fill between the downstream toe and the edge of the seepage canal spans the entire length of the west side of the reservoir (versus only to cover each drainpipe exiting the embankment), the mounding shown in **Figure A-14** will not be an issue with approximately 3 feet of cover.

Exit gradients were checked at three locations along the surface of the seepage canal – where the water surface elevation intersects the near slope of the canal, where the near slope meets the canal bottom, and where for far slope meets the canal bottom. Without any seepage control features in the canal (i.e. filters, geotextile, etc.), the calculated exit gradients (in the vertical direction) were 0.27, 0.39, and 0.36 respectively – resulting in a FOS against boiling of 3.1, 2.2, and 2.3, respectively. As these calculated values are less than that recommended by the Bureau of Reclamation, some form of seepage management is warranted. Additionally, given that the canal excavation will be through sandy soils that are also susceptible to erosion from weathering over time, lining the seepage canal with a geotextile and riprap is recommended and this work should be done in the dry. The exit gradients in the horizontal directions were 0.15, 0.18, and 0.15 respectively. For an embankment base width of 130 ft and a head differential of 7 ft, Bligh's creep ratio is 18.6, which is just above the minimum safe ratio of 18 for very fine sand or silt. Therefore, no further analyses were performed for this condition.

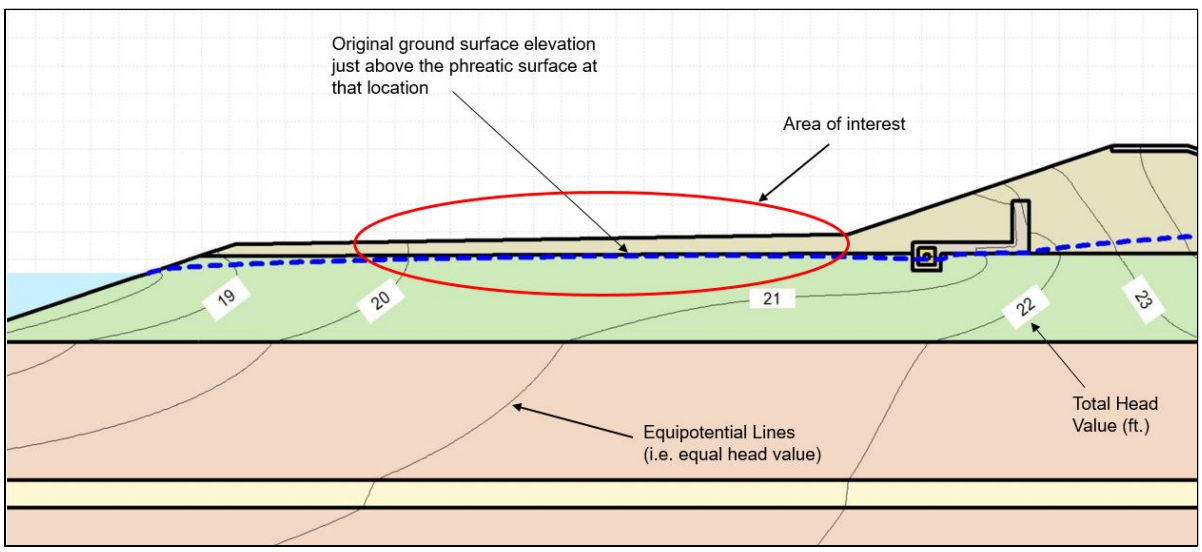


Figure A-14. Resulting head contours between the embankment toe and the seepage canal, west seepage section.

West Section: Maximum Pool

For this condition, the difference in head between the maximum pool (31.7 ft NAVD88) and the water surface in the canal (20.5 ft NAVD88) is 11.2 ft. Seepage exits the reservoir and flows towards the seepage canal and also down towards the more permeable layers modeled. The phreatic surface drops from the water surface elevation within the reservoir to the base of chimney filter, then proceeds horizontally and down into the perforated drainpipe located at the downstream end of the blanket drain. The perforated pipe receives approximately 4.75E-5 cfs of inflow (per linear foot of pipe) from the steady-state seepage. No seepage exits the downstream face of the embankment but rises to approximately 1.2 feet below the fill surface placed between the downstream toe of the embankment and the seepage canal. It should be noted that under a PMP event, it is very likely that the area between the embankment and Seminole Pratt

Whitney Road will be inundated and water will be against the exterior slope of the embankment. With a water surface elevation of 20.5 ft (NAVD88) in the canal and no seepage control features in the canal, the calculated exit gradients (in the vertical direction) were 0.20, 0.39, and 0.37 respectively – resulting in a FOS against boiling of 4.2, 2.2, and 2.3, respectively. Bligh's creep ratio for this steady state condition is 12.1, which falls below the minimum safe ratio of 18 for very fine sand or silt. Given the low probability of a maximum pool being sustained to steady-state conditions and the recommendation (for the normal pool condition) that the canal be lined with a geotextile and riprap, the concern over exit gradients and the initiation of piping has been dealt with.

South Section: Long Term

For this condition, the difference in head between the normal pool (28 ft NAVD88) and the water surface in the canal (17.5 ft NAVD88) is 10.5 ft. Seepage exits the reservoir and flows towards the seepage canal and also down towards the more permeable layers modeled. The phreatic surface drops from the water surface elevation within the reservoir to the base of chimney filter, then proceeds horizontally and down into the perforated drainpipe located at the downstream end of the blanket drain. The perforated pipe receives approximately 2.36E-4 cfs of inflow (per linear foot of pipe) from the steady-state seepage. No seepage exits the downstream face of the embankment but the phreatic surface rises to the original ground surface between the exterior toe of the embankment and the seepage canal. This change in the groundwater elevation and the head values driving it are shown in **Figure A-15**. If the fill between the downstream toe and the edge of the seepage canal spans the entire length of the south side of the reservoir (versus only to cover each drainpipe exiting the embankment), the mounding shown in **Figure A-15** will not be an issue.

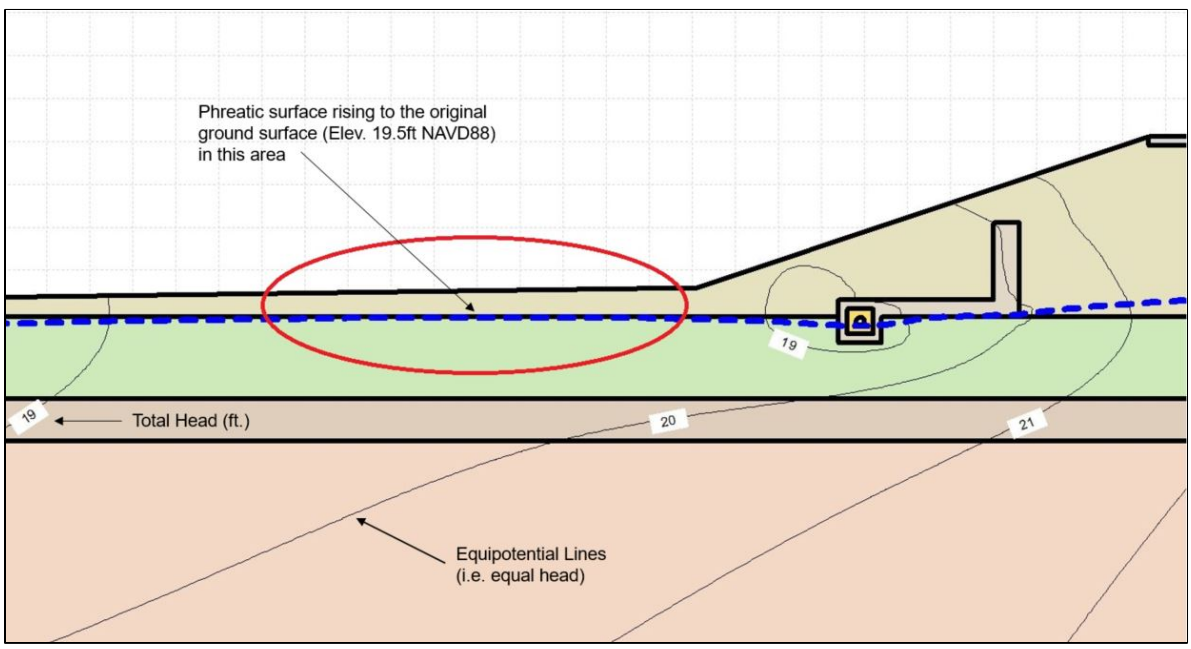


Figure A-15. Resulting head contours between the embankment toe and the seepage canal, long term condition, south seepage section.

Exit gradients were checked at three locations along the surface of the seepage canal: where the water surface elevation intersects the near slope of the canal, where the near slope meets the canal bottom, and where for far slope meets the canal bottom. Without any seepage control features in the canal (i.e. filters, geotextile, etc.), the calculated exit gradients (in the vertical direction) were 0.13, 0.15, and 0.11

respectively – resulting in a FOS against boiling of 6.3, 5.6, and 7.7, respectively. The exit gradients in the horizontal direction were 0.07, 0.04, and 0.02 respectively. For an embankment base width of 140 ft and a head differential of 8.5 ft, Bligh's creep ratio is 16.5, which falls below the minimum safe ratio of 18 for very fine sand or silt. Therefore, for long-term stability of the canal slope, seepage management, and maintenance considerations, lining the canal with a geotextile and riprap is recommended (the placement being done in the dry).

It should be noted that if the ground surface between the downstream toe of the embankment and the seepage canal is not elevated by fill placement, the phreatic surface will have to be lowered in order to prevent a saturated condition in this area and a zone of uncontrolled seepage. One method to accomplish this goal would be to add a drainage trench below the downstream end of the blanket drain **(Figure A-16).** With the drainage trench in place, a portion of the seepage flows into the drainage trench and upwards into the perforated drainpipe. Seepage not captured by the trench/drainpipe moves towards the seepage canal and also down towards the more permeable layers modeled. Beyond the trench, the phreatic levels off about 0.6 ft below the original ground surface before sloping downwards and into the seepage canal. The perforated pipe receives approximately 3.38E-4 cfs of inflow (per linear foot of pipe) from the steady-state seepage and no seepage exits the downstream face of the embankment.

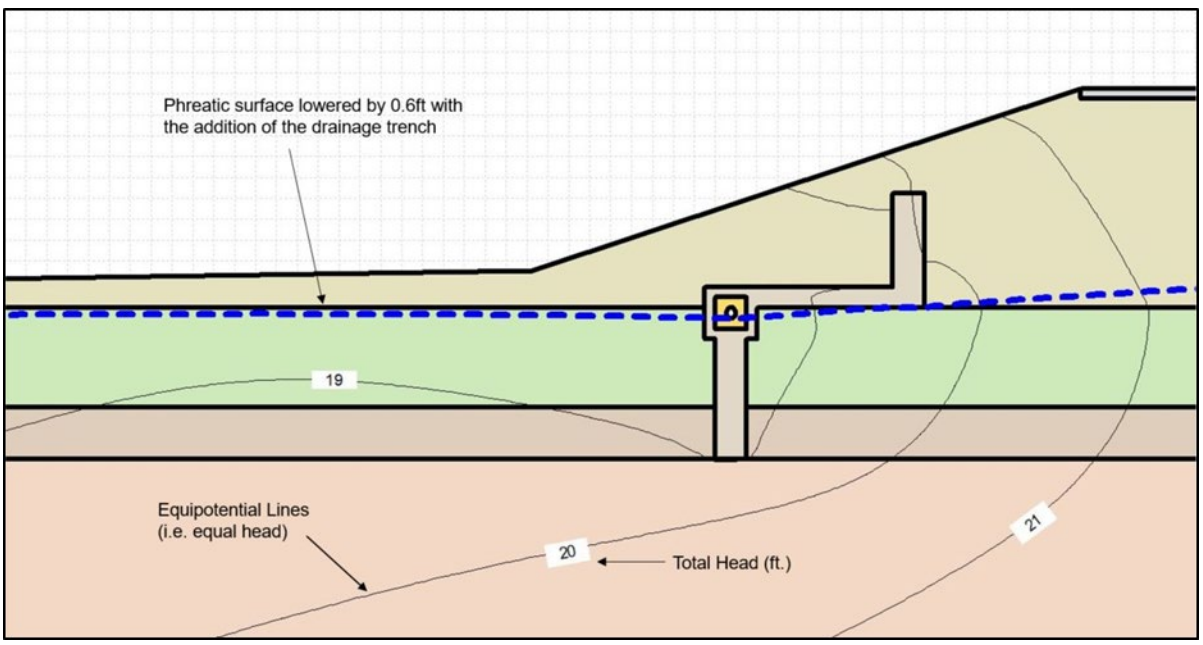


Figure A-16. Resulting head contours with drainage trench, long term condition, south seepage section.

Exit gradients were checked at three locations along the surface of the seepage canal: where the water surface elevation intersects the near slope of the canal, where the near slope meets the canal bottom, and where for far slope meets the canal bottom. With the drainage trench in place and no seepage control features in the canal (i.e. filters, geotextile, etc.), the calculated exit gradients were 0.10, 0.13, and 0.10 respectively – resulting in a FOS against boiling of 8.2, 6.5, and 8.4, respectively. The exit gradients in the horizontal direction were 0.06, 0.03, and 0.02 respectively.

South Section: Maximum Pool

For this condition, the difference in head between the maximum pool (31.7 ft NAVD88) and the water surface in the canal (20.5 ft NAVD88) is 11.2 ft. Seepage exits the reservoir and flows towards the drainage trench, seepage canal, and also downwards towards the more permeable layers modeled. The phreatic surface drops from the water surface elevation within the reservoir to the base of chimney filter, then proceeds horizontally and down into the perforated drainpipe located at the downstream end of the blanket drain. Beyond the drainage trench axis, the phreatic surface rises to a foot below the surface of the fill placed between the exterior toe of the embankment and the seepage canal. The perforated pipe receives approximately 6.37E-4 cfs of inflow (per linear foot of pipe) from the steady-state seepage. No seepage exits the downstream face of the embankment – but it should be noted, that under a PMP event, it is likely that the area between the embankment and the spoil berm could be inundated and that water will be against the exterior slope of the embankment. In checking the same locations discussed for the Long-Term Condition, the exit gradients/FOS against boiling were 0.04, 0.11, 0.09 and 23.2, 7.7, 9.4, respectively. In the horizontal direction the gradients were 0.02, 0.02, and 0.02. Bligh's creep ration for this steady-state condition is 11.5. As a result, lining the canal with a geotextile and riprap is recommended.

East Section: Long Term

For this condition, the difference in head between the normal pool (28 ft NAVD88) and the water surface in the canal (17.5 ft NAVD88) is 10.5 ft. Seepage exits the reservoir and flows towards the seepage canal and also down towards the more permeable layers modeled (i.e. SAS 2 and the SM with Shell layer). The phreatic surface drops from the water surface elevation within the reservoir to the base of chimney filter, then proceeds horizontally and down into the perforated drainpipe located at the downstream end of the blanket drain. The perforated pipe receives approximately 1.43E-5 cfs of inflow (per linear foot of pipe) from the steady-state seepage. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the original ground surface (0.7 ft) between the exterior toe of the embankment and the seepage canal. This change in the groundwater elevation and the head values driving it are shown in **Figure A-17**. If the fill between the downstream toe and the edge of the seepage canal spans the entire length of the east side of the reservoir (versus only to cover each drainpipe exiting the embankment), the mounding shown in **Figure A-17** will not be an issue.

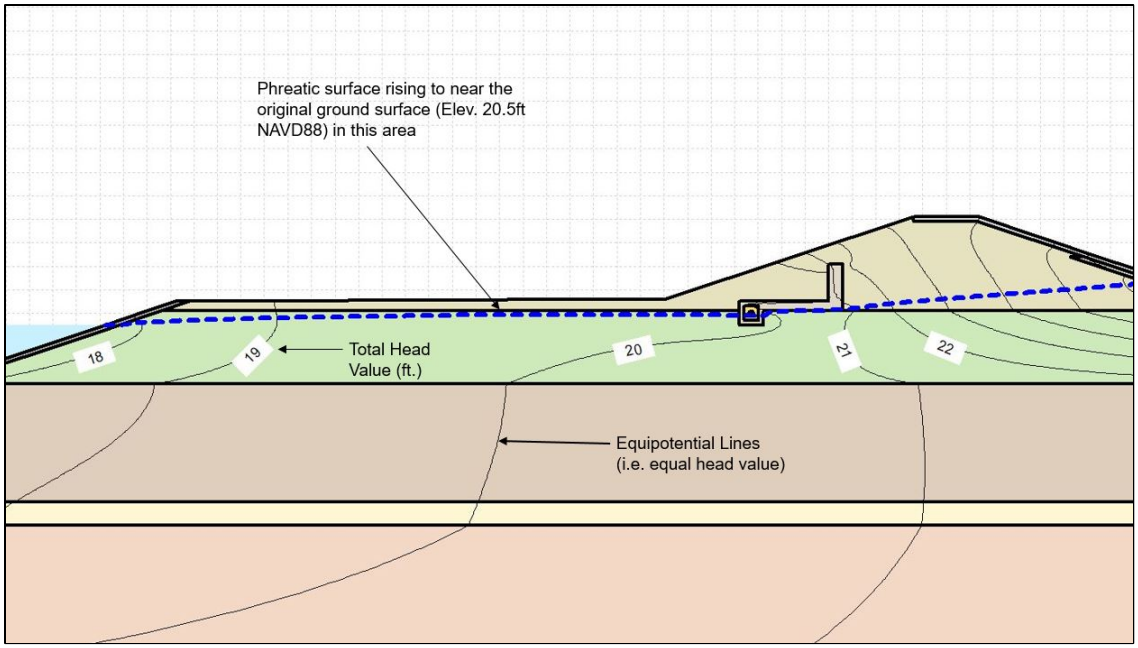


Figure A-17. Resulting head contours between the embankment toe and the seepage canal, east seepage section.

Exit gradients were checked at three locations along the surface of the seepage canal – where the water surface elevation intersects the near slope of the canal, where the near slope meets the canal bottom, and where for far slope meets the canal bottom. Without any seepage control features in the canal (i.e. filters, geotextile, etc.), the calculated exit gradients (in the vertical direction) were 0.24, 0.28, and 0.23 respectively – resulting in a FOS against boiling of 3.5, 3.0, and 3.7, respectively (all less than that recommended by the Bureau of Reclamation). The exit gradients in the horizontal direction were 0.14, 0.16, and 0.12 respectively. For an embankment base width of 136 feet and a head differential of 7.5 feet, Bligh's creep ratio is 17.8, which falls just below the minimum safe ratio of 18 for very fine sand or silt. Therefore, as discussed for the west seepage section, lining the seepage canal with a geotextile and riprap is recommended and this work should be done in the dry.

<u>East Section: Maximum Pool</u>

For this condition, the difference in head between the maximum pool (31.7 ft NAVD88) and the water surface in the canal (20.5 ft NAVD88) is 11.2 ft. Seepage exits the reservoir and flows towards the seepage canal and also down towards the more permeable layers modeled. The phreatic surface drops from the water surface elevation within the reservoir to the base of chimney filter, then proceeds horizontally and down into the perforated drainpipe located at the downstream end of the blanket drain. The perforated pipe receives approximately 4.4E-5 cfs of inflow (per linear foot of pipe) from the steady-state seepage. No seepage exits the downstream face of the embankment but rises to within 1 foot of the fill surface between the downstream toe of the embankment and the seepage canal. It should be noted that under a PMP event, it is very likely that the area between the embankment and the seepage canal will be inundated and water will be against the exterior slope of the embankment. With a water surface elevation of 20.5 ft (NAVD88) in the canal and no seepage control in the canal, the calculated exit gradients (in the vertical direction) were 0.19, 0.30, and 0.25 respectively – resulting in a FOS against boiling of 4.5, 2.8, and 3.4, respectively. Bligh's creep ratio for this steady state condition is 12, which falls below the minimum safe ratio of 18 for very fine sand or silt. However, given the low probability of a maximum pool

being sustained to steady-state conditions and the recommendation (for the normal pool condition) that the canal be lined with a geotextile and riprap, the concern over exit gradients and the initiation of piping has been dealt with.

A.7.2.4 Slope Stability Modeling

Slope stability analyses were performed using SLOPE/W, another component of the GeoStudio 2018 R2 software suite (Version 9.1.1.16749). The analyses considered the materials encountered by subsurface investigations for the C-18W reservoir. SLOPE/W's formulation is based on the general 2D limited equilibrium method and uses an iteration scheme to find the critical slip surface and corresponding minimum factor of safety. The method of analysis chosen to determine the factor of safety with respect to stability was Spencer's procedure (Spencer, 1967), which is the preferred method of the USACE because it fully satisfies static equilibrium for each slice within the failure area.

A.7.2.4.1 Preliminary Unit Weight and Strength Parameters for Slope Stability Analysis

Reservoir and published values for similar materials. With the exception of the phi angle for the embankment material and the strength parameters selected for the riprap/filter materials, all of the strength parameters listed in **Table A-8, Table A-9,** and **Table A-10** are the same as those found in Table 4 of the DDR by Arcadis (2016) – see **Annex B-1**.

Table A-8. Selected strength parameters for each material type, western seepage section.

Material	Total Unit Weight (pcf)	EOC Strength Parameters c & phi (psf/deg)	LT Strength Parameters c & phi (psf/deg)
Embankment Material (SP-SM)	115	0/34	0/34
Foundation Layer 1 (SP-SM)	115	0/32	0/32
Foundation Layer 2 (SP-SM)	115	0/30	0/30
Foundation Layer 3 (SP-SM)	121	0/35	0/35
Foundation Layer 4 (SP-SM)	121	0/35	0/35
Foundation Layer 5 (SP-SM)	130	0/35	0/35
Filter Sand (C33 Sand)	120	0/33	0/33
Fine Drainage Gravel (No. 89)	130	0/35	0/35
Soil Cement Armoring	125	0/35	0/35
R-20 Riprap	101	0/40	0/40

Table A-9. Selected strength parameters for each material type, southern seepage section.

Material	Total Unit Weight (pcf)	EOC Strength Parameters c & phi (psf/deg)	LT Strength Parameters c & phi (psf/deg)
Embankment Material (SP-SM)	115	0/34	0/34
Foundation Layer 1 (SP-SM)	115	0/31	0/31
Foundation Layer 2 (SP-SM)	115	0/31	0/31
Foundation Layer 3 (SP-SM)	115	0/33	0/33
Foundation Layer 4 (SP-SM)	121	0/29	0/29
Foundation Layer 5 (SP-SM)	121	0/31	0/31
Foundation Layer 6 (SP-SM)	130	0/32	0/32
Filter Sand (C33 Sand)	120	0/33	0/33
Fine Drainage Gravel (No. 89)	130	0/35	0/35
Soil Cement Armoring	125	0/35	0/35
R-20 Riprap	101	0/40	0/40

Table A-10. Selected strength parameters for each material type, east seepage section.

Material	Total Unit Weight (pcf)	EOC Strength Parameters c & phi (psf/deg)	LT Strength Parameters c & phi (psf/deg)
Embankment Material (SP-SM)	115	0/34	0/34
Foundation Layer 1 (SP-SM)	115	0/30	0/30
Foundation Layer 2 (SP-SM)	121	0/30	0/30
Foundation Layer 3 (SP-SM)	121	0/30	0/30
Foundation Layer 4 (SP-SM)	130	0/30	0/30
Foundation Layer 5 (SP-SM)	130	0/30	0/30
Filter Sand (C33 Sand)	120	0/33	0/33
Fine Drainage Gravel (No. 89)	130	0/35	0/35
Soil Cement Armoring	125	0/35	0/35
R-20 Riprap	101	0/40	0/40

A.7.2.4.2 Cross Sections Analyzed

The three sections analyzed for seepage were also used to evaluate the stability of the embankments and canals. In each analysis, either a piezometric line was assigned or the pore-water pressures computed by SEEP/W were imported into SLOPE/W and the Mohr-Coulomb strength model was used for all materials.

A.7.2.4.3 Cases Analyzed

For new embankment dams, EM 1110-2-1902 lists four conditions for analysis: End-of-Construction (EOC); Long-Term (LT - steady-state seepage, maximum storage pool); Maximum Surcharge Pool; and Rapid Drawdown (RDD). For this Appendix, EOC, LT, Max Surcharge Pool, and RDD were analyzed. (While the project site lies within a low seismic hazard zone, appropriate methods will be used during the design

phase to analyze the liquefaction potential and embankment deformation from a potential seismic event. As clean, loose sands exist within the project area capable of liquefaction, ER 1110-2-1806 will be used as a guide for this analysis).

Table A-11. Slope stability results for each design case for the C-18W Reservoir.

Design Case	Pool Elevation (ft NAVD88)	Slope	Required Factor of Safety	Calculated Factor of Safety
West Seepage Section				
End-of-Construction (EOC)	No Pool	Upstream	1.3	1.99
End-of-Construction (EOC)	No Pool	Downstream	1.3	2.02
End-of-Construction (EOC)	No Pool	Canal Slope	1.3*	1.69
Steady State-Normal Pool	28.0	Downstream	1.5	2.02
Steady State-Normal Pool	28.0	Canal Slope	1.5*	1.48
Steady State-Maximum Pool	31.7	Downstream	1.4	2.02
Steady State-Maximum Pool	31.7	Canal Slope	1.4*	1.61
Rapid Drawdown	28.0 to 21.0	Upstream	1.3	1.54
South Seepage Section				
End-of-Construction (EOC)	No Pool	Upstream	1.3	1.98
End-of-Construction (EOC)	No Pool	Downstream	1.3	1.90
End-of-Construction (EOC)	No Pool	Canal Slope	1.3*	1.64
Steady State-Normal Pool	28.0	Downstream	1.5	1.90
Steady State-Normal Pool	28.0	Canal Slope	1.5*	1.58
Steady State-Maximum Pool	31.7	Downstream	1.4	1.90
Steady State-Maximum Pool	31.7	Canal Slope	1.4*	1.82
Rapid Drawdown	28.0 to 19.5	Upstream	1.3	1.48
East Seepage Section				
End-of-Construction (EOC)	No Pool	Upstream	1.3	1.99
End-of-Construction (EOC)	No Pool	Downstream	1.3	2.33
End-of-Construction (EOC)	No Pool	Canal Slope	1.3*	1.58
Steady State-Normal Pool	28.0	Downstream	1.5	2.21
Steady State-Normal Pool	28.0	Canal Slope	1.5*	1.45
Steady State-Maximum Pool	31.7	Downstream	1.4	2.13
Steady State-Maximum Pool	31.7	Canal Slope	1.4*	1.56
Rapid Drawdown	28.0 to 19.5	Upstream	1.3	1.57

*Recommended but not required by EM 1110-2-1902.

A.7.2.4.4 Stability Modeling Results at C-18W Reservoir

For the embankment slopes modeled, the resulting factors of safety were all greater than the minimum required by EM 1110-2-1902 **(Table A-11)**, therefore no further analyses were required. As the seepage canal slopes are technically "other slopes" as defined by the EM, the "required" factors of safety for these slopes should be appropriate to the level of uncertainty associated with the parameters selected and the consequences of failure. With the exception of the FOS determined for the canal slope in the west and east embankment sections under normal pool conditions, all the resulting factors of safety were greater

than the minimum required for new earth and rock-filled dams (the difference between the recommended FOS and that determined is considered to be negligible in both cases). **Figure A-18** and **Figure A-19** shows some of the critical failure surfaces determined by SLOPE/W. Additional figures can be found in **Annex B-1.**

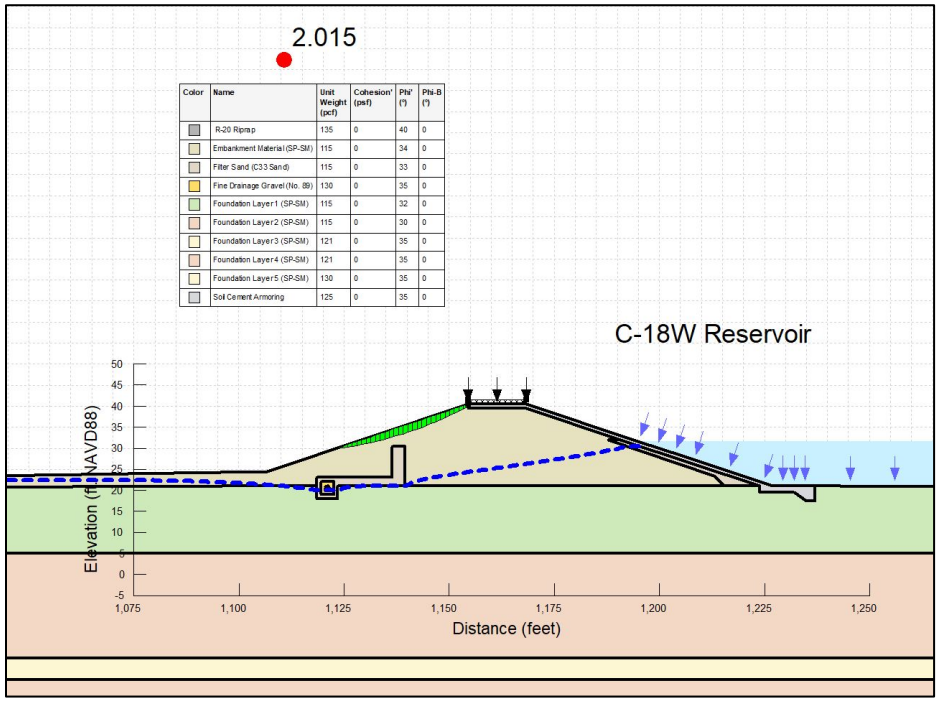


Figure A-18. Critical failure surface (D/S slope) from steady state conditions, normal pool, western seepage section.

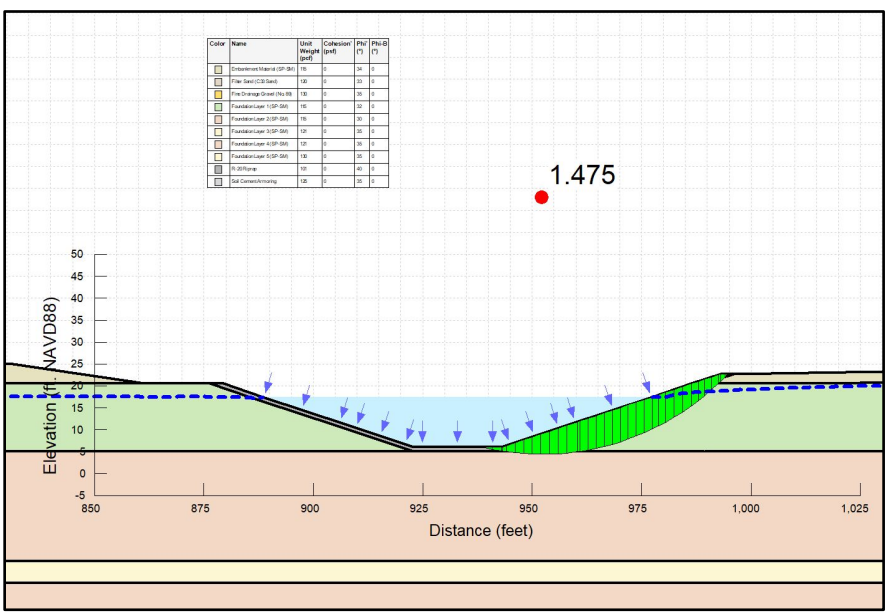


Figure A-19. Critical failure surface (near slope) from steady state condition, normal pool, western seepage section.

A.7.3 Cypress Creek Canal Modifications

The Cypress Creek Canal is one of the major drains of the project area, and is a major contributor to increased flows to the northwest fork of the Loxahatchee River. Proposed modifications to the Cypress Creek Canal will maintain and improve current flood control in the canal, improve wetland hydroperiods to the west in the J.W. Corbett WMA, and reduce flashy discharges to Cypress Creek. These modifications include the following:

- Conversion of Culpepper structures S-115A, B, and C to operable structures with telemetry (plates **P-56** and **P-57**)
- Removal of existing bridge across Cypress Creek Canal (plates **P-08** and **P-09**)
- Expansion of the eastern end of Cypress Creek Canal adjacent to the Gulfstream flow-through marsh (plate **P-09**)
- Construction of the Cypress Creek Canal weir and spillway (plates **P-48** and **P-33, P-34**)

A.7.4 Mack Dairy Road Spreader

The Mack Dairy Road spreader will function to rehydrate western portions of the Cypress Creek Natural Area, and will enhance sheet flow eastward towards the headwaters of the historic Cypress Creek.

A.7.4.1 Soils and Shallow Subsurface Geology

Surface soils (0 to 10 cm depth) in the area of the proposed Mack Dairy Road spreader and adjacent areas are characterized mostly as SP-SM, or generally poorly graded, slightly silty sands, with silt content less than 12 percent by weight. Soils are mapped as drained or ponded Riviera and Wabasso fine sands (NRCS, 2019), in a depressional wetland landscape. Surface soils show uniform saturated hydraulic conductivity estimates of approximately 9.2 x 10^{-3} cm per second (26 ft per day). The classification of surface soils at the Mack Dairy Road Spreader and adjacent areas is shown in **Figure A-20**. The shallow subsurface geology and hydrogeologic setting at the Cypress Creek Natural Area (CCNA) are expected to be similar to those at the C-18W reservoir **(section A.7.2.2).**

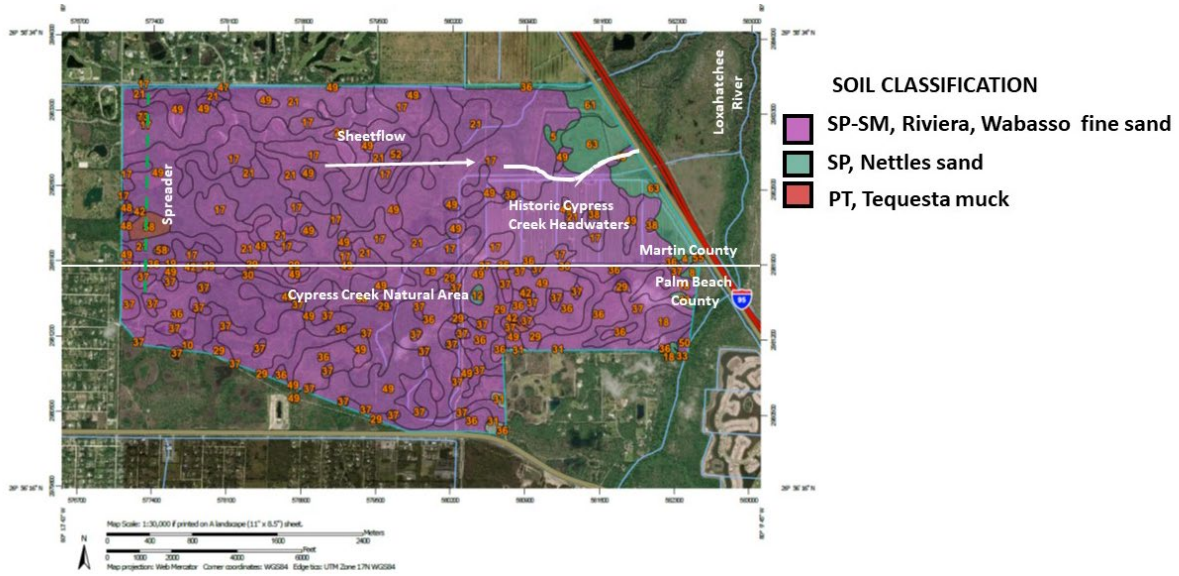


Figure A-20. Surface soil classification at Mack Dairy Road footprint and adjacent areas.

A.7.5 Gulfstream West Flow-Through Marsh

The Gulfstream West FTM is located west of the Florida Turnpike/I-95 corridor in Martin County. Surface water from the Hobe St. Lucie Conservancy District canals along the northern margin of the Nine Gems property will be pumped into the northern end of the Gulfstream West property. Sheet flow will pool adjacent to hardened berms that traverse the property, overtop, and continue to flow south toward Cypress Creek at an average depth of 2 ft. Water will exit the feature over a fixed-crest weir, discharging into Cypress Creek Canal.

A.7.5.1 Soils and Shallow Subsurface Geology

Surface soils (0 to 10 cm depth) in the area of the proposed Gulfstream West FTM are characterized mostly as SP-SM, or generally poorly graded, slightly silty sands, with silt content less than 12 percent by weight. Land use at the Gulfstream West property is consisted of citrus agriculture, and currently this area is range land. Soils are mapped as drained or ponded Piñeda-Riviera and Wabasso fine sands (NRCS, 2019). Surface soils show uniform saturated hydraulic conductivity estimates of approximately 9.2 x 10^{-3} centimeters per second (26 feet per day). **Figure A-21** below shows the classification of surface soils at the Gulfstream West property. The shallow subsurface geology and hydrogeologic setting at the Gulfstream West FTM are expected to be similar to those at the C-18W reservoir **(Section A.7.2.2).** The Gulfstream West feature is underlain by approximately 120 ft of fossiliferous marine sands and silt with discontinuous limestone layers. The upper surface of the Hawthorn Group is encountered at depths of approximately 120 ft to 150 ft bls (Lukasiewicz and Smith, 1996).

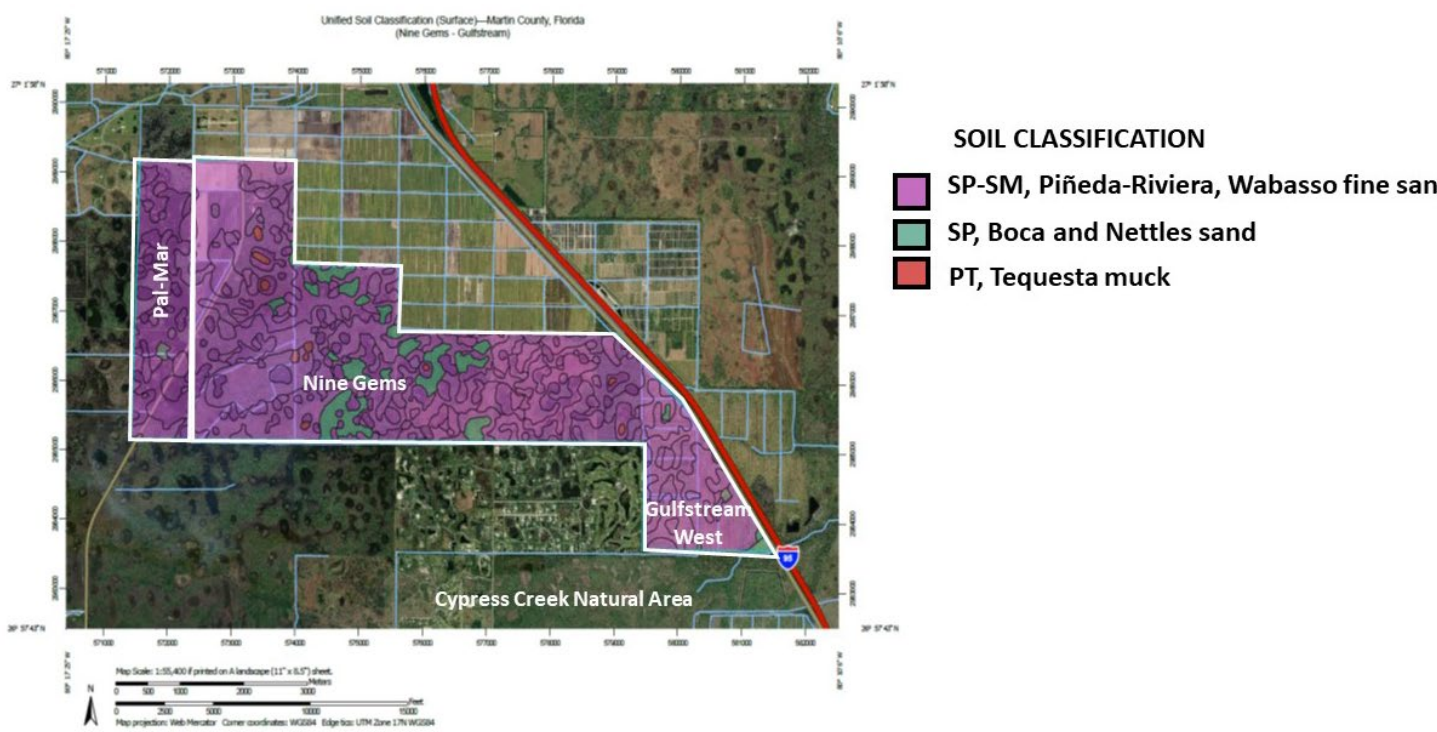


Figure A-21. Surface soil classification at the Gulfstream West and Nine Gems footprints.

A.7.5.2 Gulfstream West Seepage Analysis

For the preliminary design of the Gulfstream West exterior embankments, SEEP/W (Version 9.1.1.16749) was used to predict the resulting phreatic surface and seepage from the operation of the marsh. The software was also used to verify that the embankment will be safe with respect to uplift pressures and exit gradients and that there is minimal effect to existing canals and drainage ditches.

A 2D seepage model was developed from the materials encountered by the FDOT borings performed for the bridge over Cypress Creek, along I-95 in the general area of Gulfstream West, and a nearby weigh-in motion station. Representative borings used from these previous projects are included in **Annex B-1** Geotechnical. These materials were generalized into four conceptual model layers. Each were poorly graded sands (SP) with gradually increasing densities with depth. Due to the potential for undesirable exit gradients into the canals, limiting this seepage was considered by 89 stone liner to the canals. Due to the potential for undesirable exit gradients into the FDOT ditch, limiting this seepage was considered by constructing a sand-lined drainage ditch between the proposed berm and the FDOT ditch. During the design phase of the project, these options will either be carried forward, replaced with other solutions, or eliminated based on site specific data and testing. Other options for reducing gradients include using riprap and geotextile. Additionally, only those sections (i.e. S, E, and W) that represented a unique configuration were modeled with the understanding that further sections will be analyzed during the design phase of the project.

A.7.5.2.1 Preliminary Design Parameters for the 2D Seepage Analysis

The design parameters selected for the 2D seepage modeling are based on field data and published values for similar materials. Anisotropy values (ky/kx) are based on Figures 8.3.2.3.1-1 and 8.3.2.3.3-1 in the Bureau of Reclamation (BOR) reference *Design Standards No. 13, Embankment Dams, Chapter 8, Seepage*. These values will be updated during the design phase of the project with site-specific information. Preliminary hydraulic parameters for each material type are presented in **Table A-12.** The values in **Table A-12** were generally slightly higher than laboratory test data reported by Arcadis (2016).

<u>Fill</u>
This material will likely be a blend of poorly-graded sand and silty sand based on the FDOT borings nearby. This material will be placed on top of the Sand (0-5 ft) layer in order to achieve a crest elevation of 20 ft after stripping of topsoil and any replacement or treatment of any unsatisfactory foundation grade material. This material has a saturated hydraulic conductivity value of 3.3E-5 ft/s in the horizontal direction (estimated from published values referenced by the BOR).

<u>Sand Layer 1 (Sand, 0-5 ft)</u>
A 5-foot layer of poorly graded sand (SP), characterized by a saturated hydraulic conductivity value of 3.01E-5 ft/s in the horizontal direction. This value was estimated from saturated hydraulic conductivity values determined by the NRCS for the proposed flow-through marsh location (see **Figure G-1.5** in **Annex G-1**).

<u>Sand Layer 2 (Sand, 5-25 ft)</u>
A 20-foot layer of poorly graded sand (SP), characterized by a saturated hydraulic conductivity value of 7.93E-4 ft/s in the horizontal direction (estimated from published values referenced by the BOR). The layer thickness was estimated from the FDOT borings performed for the above reference projects.

Sand Layer 3 (Sand, 25-60 ft)
A 35-foot layer of poorly graded sand (SP), characterized by a saturated hydraulic conductivity value of 7.93E-4 ft/s in the horizontal direction (estimated from published values referenced by the BOR). The layer thickness was estimated from the FDOT borings performed for the above reference projects.

Sand Layer 4 (60-100 ft)
A 40-foot layer of poorly graded sand (SP), characterized by a saturated hydraulic conductivity value of 7.93E-4 ft/s in the horizontal direction (estimated from published values referenced by the BOR). The layer thickness was estimated from the FDOT borings performed for the above reference projects.

ASTM C33 Fine Aggregate (Sand)
Drainage sand will be used to line the eastern canal. The properties for this material were obtained using laboratory data from a similar project, and so will be applied here.

ASTM C33 No. 89 Stone
No. 89 stone will be used to line the southern and western canals due to the higher expected velocities and to assist with slope stability. The properties for this material were obtained using laboratory data from a similar project, and so will be applied here.

Table A-12 Preliminary hydraulic parameters for each material type.

Material	Modeled Bottom Elev. (ft, NAVD88)	Kx (cm/s)	Kx (ft/s)	Ky/Kx
1 – Fill	18.0	1.0E-3	3.3E-5	0.11
2 – Sand (0-5 ft)	13.0	9.17E-4	3.01E-5	0.1
3 – Sand (5-25 ft)	-7.0	2.42E-2	7.93E-4	0.1
4 – Sand (25-60 ft)	-42	2.42E-2	7.93E-4	0.1
5 – Sand (60-100 ft)	-82	2.42E-2	7.93E-4	0.1
6 – C33 Sand	---	5.39E-2	1.77E-3	1.0
7 – No. 89 Stone	---	0.457	1.50E-2	1.0

The values above were generally slightly higher than what was encountered by Arcadis in the laboratory tests performed for C-18W.

A.7.5.2.2 Embankment and Canal General Geometries

With respect to the exterior embankments, the side slopes will be 1V:3H, 14-ft crest width, and a crest elevation at 20 ft. **Figure A-22** shows locations of conceptual cross-sections depicted in

. The existing canal and proposed canal are trapezoidal in shape with 1V:2H and 1V:2.5H side slopes, respectively and bottom widths of 20 and 30 ft, respectively. These canals could be lined with 12 inches of filter sand to limit exit gradients into the canals. The existing FDOT ditch is approximately trapezoidal with a top width of 55 ft and a bottom width of 5 ft. The proposed drainage ditch on the east side of Gulfstream West has 3:1 side slopes and a bottom width of 10 ft. The interior separator berms will be approximately 1 to 2 ft in height, 4-ft crest width, and 1V:3H side slopes. These berms will be constructed out of soil cement as water from one cell to another will flow over them during the normal operation of the flow-through marsh.

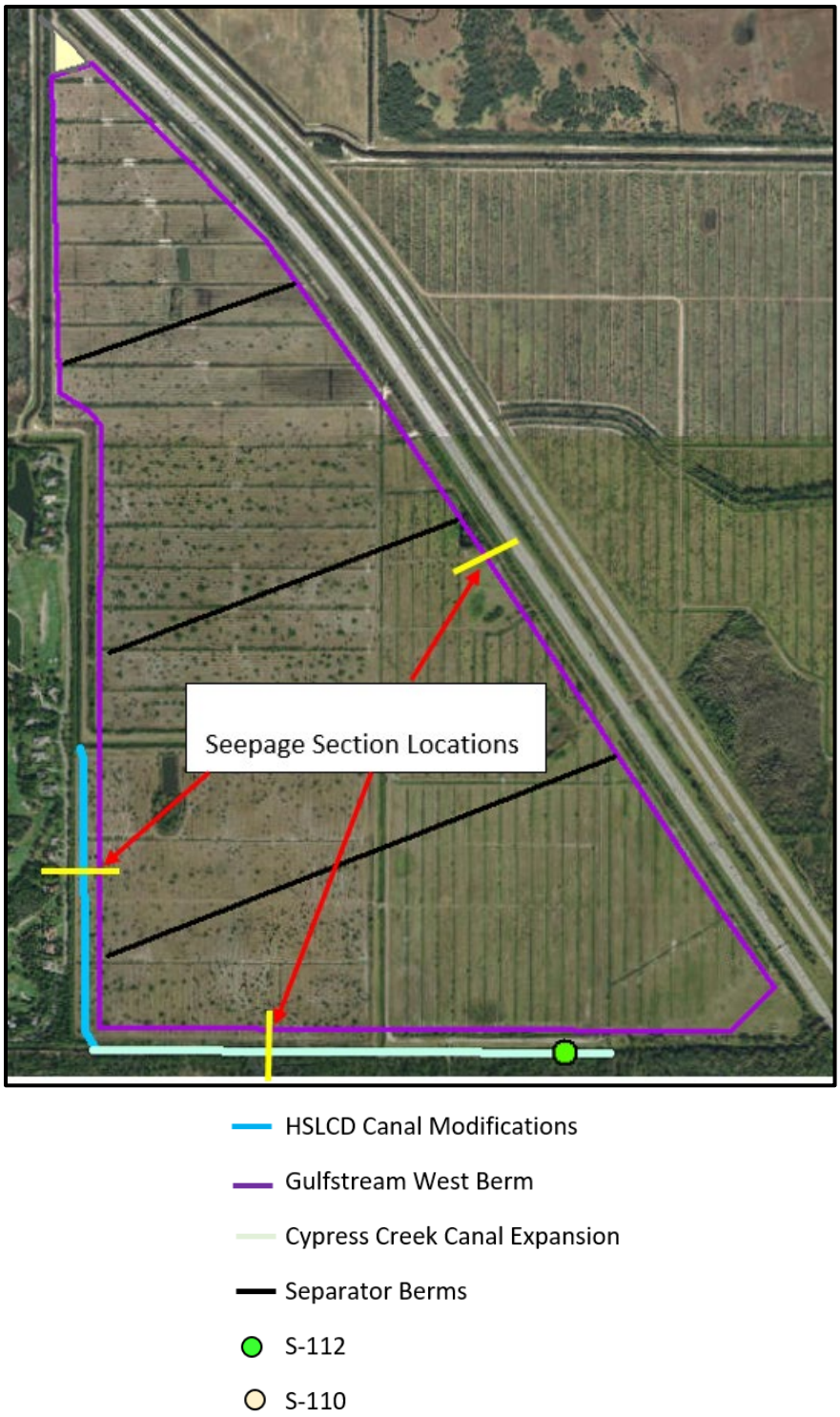
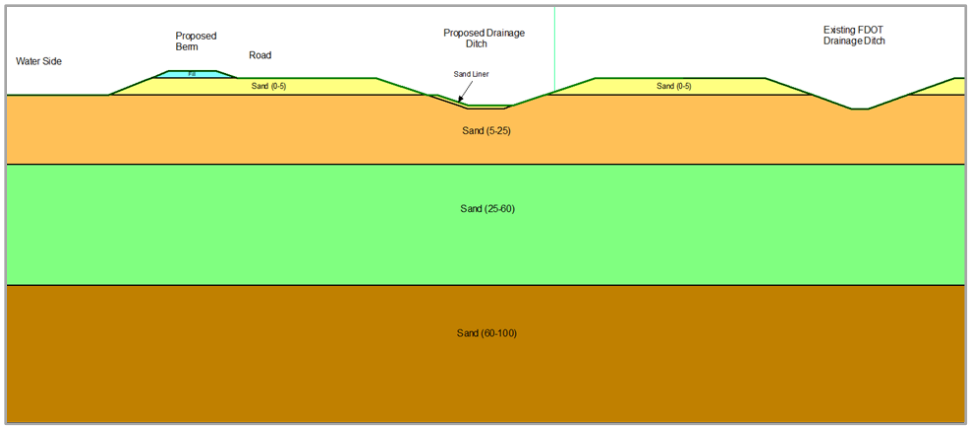
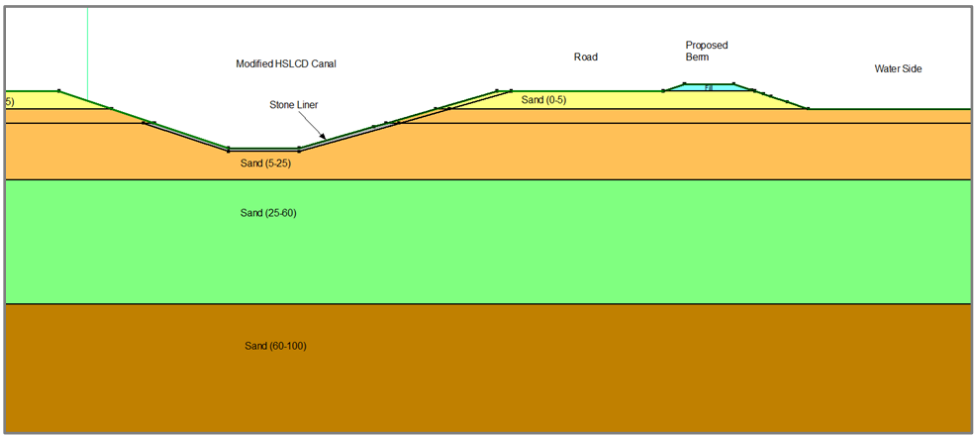


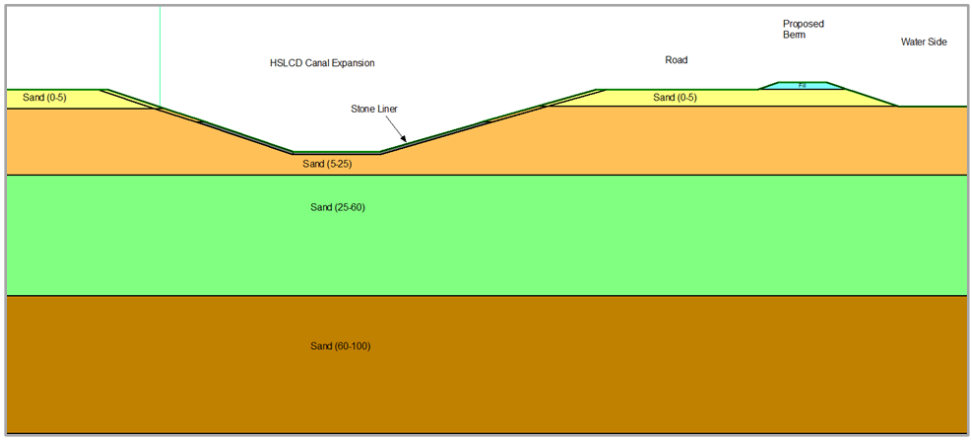
Figure A-22. Embankment locations for conceptual cross-sections (not to scale).



A.



B.



C.

Figure A-23. Conceptual cross-sections of Gulfstream West embankments. A. Eastern exterior embankment with adjacent drainage canal (looking north); B. Western exterior embankment with adjacent drainage canal (looking north); C. Southern exterior embankment with adjacent drainage canal (looking east).

A.7.5.2.3 Boundary Conditions

Three different types of boundary conditions were used in the seepage modeling: No Flow, Constant Head and Potential Seepage Face. These boundary conditions were applied as follows:

- Where a water body was present (the pool, adjacent canal, etc.), a constant-head boundary was set to model that feature. These water surface elevations were provided by the Water Resources Engineering Branch.
- Where the above boundary conditions were not used, a potential seepage face boundary was used for most of the remaining nodes – which would allow the software to determine the water surface elevation at those locations.
- No Flow: Water cannot pass through the boundary and neither a total hydraulic head nor a flow rate has been specified. This condition was set along the vertical sides and bottom of each model.

A.7.5.2.4 Seepage Conditions Modeled

Two seepage conditions were modeled: Long Term (steady-state seepage with a normal, average pool in the flow-through marsh and a normal water level in the canals and ditches) and Maximum Pool (steady-state seepage with a maximum, operational pool depth in the flow-through marsh, and a normal water level in the canals and ditches). Two typical cross-sections **(Figure A-24, Figure A-25)** showing normal pool conditions are presented below. Figures showing the remaining conditions are presented in **Annex B-1** Geotechnical.

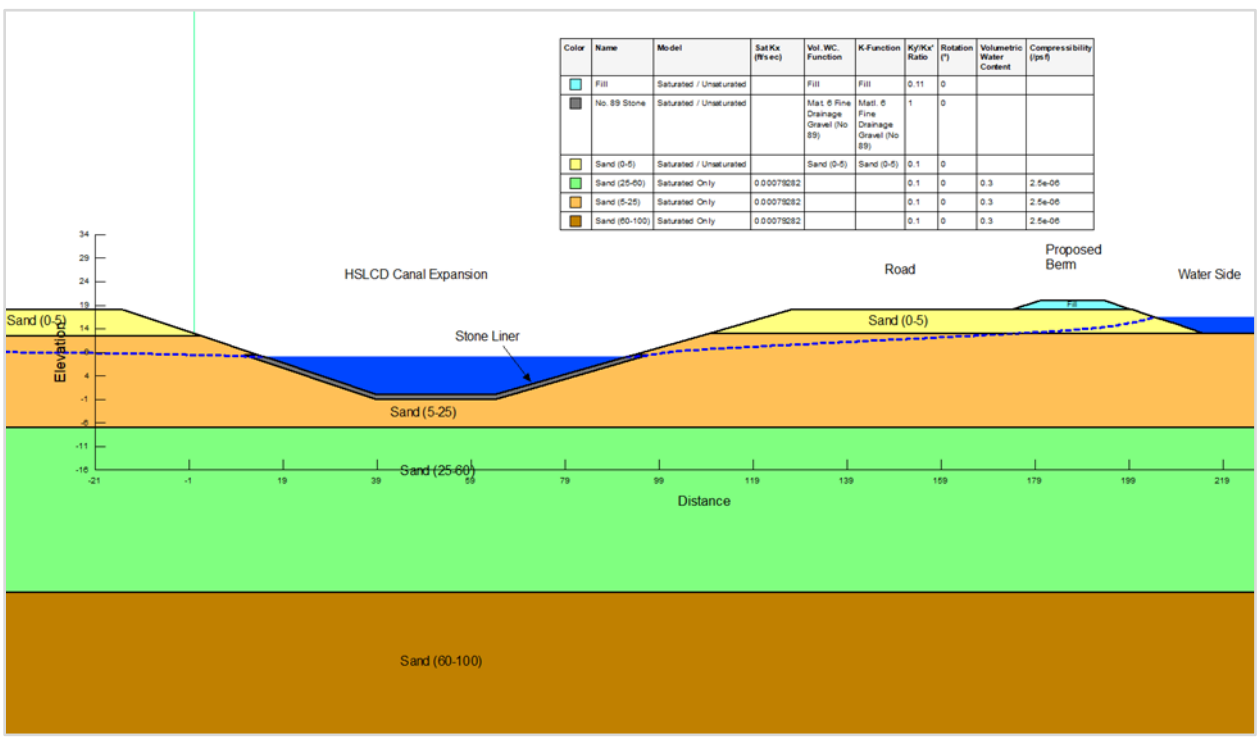


Figure A-24. Gulfstream West embankment, south section, long term condition, normal pool.

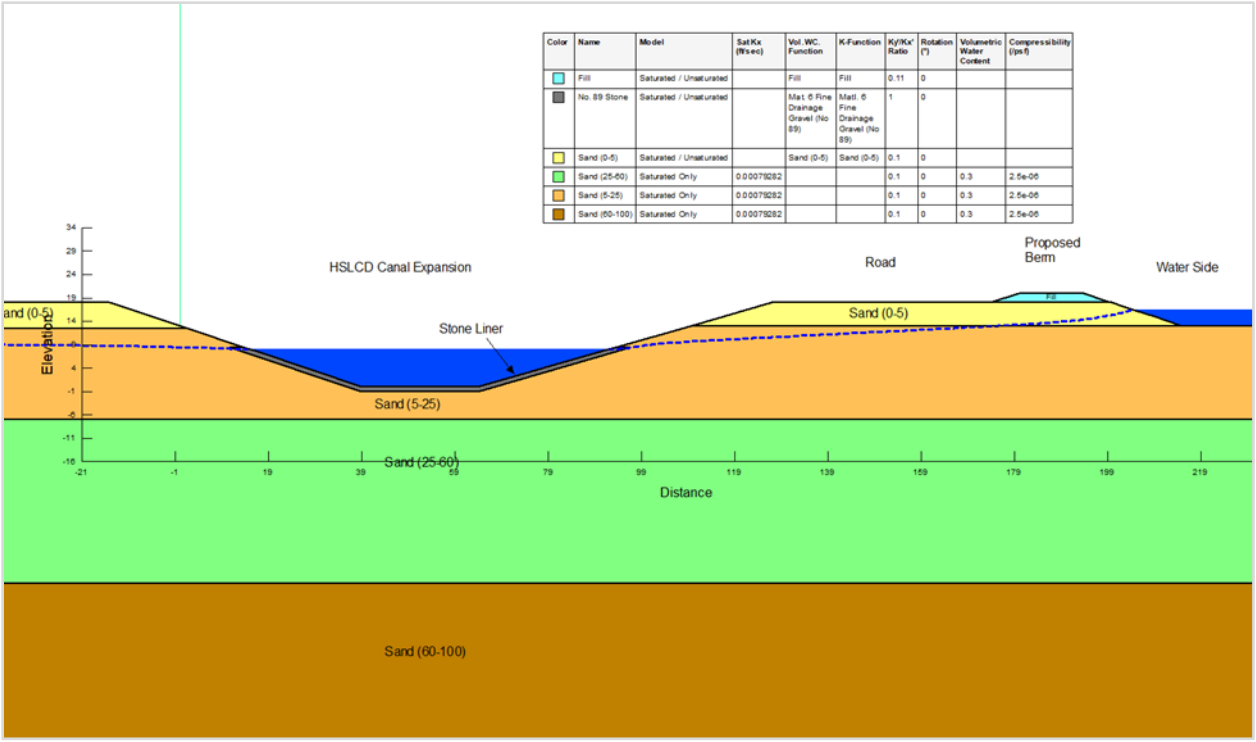


Figure A-25. Gulfstream West embankment, west section, long term condition, normal pool.

A.7.5.2.5 Seepage Modeling Results at the Gulfstream Marsh Flow-Through Marsh

<u>East Section (Drainage Canal): Long Term</u>

For this condition, the difference in head between the normal pool (16.5 ft NAVD88) and the water surface in the canal (10.0 ft NAVD88) is 6.5 ft. Seepage exits the STA and flows horizontally towards the seepage canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the seepage canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at one location along the surface of the seepage canal – where the water surface elevation intersects the canal bottom. With the addition of the sand filter layer, the calculated exit gradient (in the vertical direction) was 0.09 – resulting in a FOS against boiling of 8.6. The exit gradient in the horizontal direction was 0.13. For an embankment base width of approximately 105 ft and a head differential of 6.5 ft, Bligh's creep ratio is 16.1.

<u>East Section (Drainage Canal): Maximum Pool</u>

For this condition, the difference in head between the maximum pool (18.25 ft NAVD88) and the water surface in the canal (10.0 ft NAVD88) is 8.25 ft. Seepage exits the STA and flows horizontally towards the seepage canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the seepage canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at one location along the surface of the seepage canal – where the water surface elevation intersects the canal bottom. With the addition of the sand filter layer, the calculated exit gradient (in the vertical direction) was 0.12 – resulting in a FOS against boiling of 6.3. The exit gradient

in the horizontal direction was 0.17. For an embankment base width of approximately 105 ft and a head differential of 8.25 ft, Bligh's creep ratio is 12.7.

East Section (FDOT Ditch): Long Term

For this condition, the difference in head between the normal pool (16.5 ft NAVD88) and the water surface in the ditch (9.0 ft NAVD88) is 7.5 ft. Seepage exits the STA and flows horizontally towards the seepage canal. The phreatic surface just below the bottom of the canal and then towards the FDOT ditch. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the ditch. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at one location along the surface of the ditch – where the water surface elevation intersects the ditch bottom. The calculated exit gradient (in the vertical direction) was 0.25 – resulting in a FOS against boiling of 3.0. The exit gradient in the horizontal direction was 0.12. For an embankment base width of approximately 100 ft and a head differential of 1 ft between the drainage canal and ditch, Bligh's creep ratio is 100.

East Section (FDOT Ditch): Maximum Pool

For this condition, the difference in head between the maximum pool (18.25 ft NAVD88) and the water surface in the ditch (9.0 ft NAVD88) is 9.25 ft. Seepage exits the STA and flows horizontally towards the seepage canal. The phreatic surface just below the bottom of the canal and then towards the FDOT ditch. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the ditch. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at one location along the surface of the ditch – where the water surface elevation intersects the ditch bottom. The calculated exit gradient (in the vertical direction) was 0.31 – resulting in a FOS against boiling of 2.5. The exit gradient in the horizontal direction was 0.15. For an embankment base width of approximately 100 ft and a head differential of 1 ft between the drainage canal and ditch, Bligh's creep ratio is 100.

South Section: Long Term

For this condition, the difference in head between the normal pool (16.5 ft NAVD88) and the water surface in the canal (8.0 ft NAVD88) is 8.5 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.01 and 0.003, respectively – resulting in a FOS against boiling of greater than 10 where the water surface meets the near slope of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.04. For an embankment base width of approximately 125 ft and a head differential of 8.5 ft, Bligh's creep ratio is 14.7.

South Section: Maximum Pool

For this condition, the difference in head between the maximum pool (18.25 ft NAVD88) and the water surface in the canal (8.0 ft NAVD88) is 10.25 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just

below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.01 and 0.003, respectively – resulting in a FOS against boiling of greater than 10 where the water surface meets the near slope of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.05. For an embankment base width of approximately 125 ft and a head differential of 10.25 ft, Bligh's creep ratio is 12.2.

West Section: Long Term

For this condition, the difference in head between the normal pool (16.5 ft NAVD88) and the water surface in the canal (8.0 ft NAVD88) is 8.5 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.02 and 0.003, respectively – resulting in a FOS against boiling of greater than 10 where the water surface meets the near slope of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.03. For an embankment base width of approximately 120 ft and a head differential of 8.5 ft, Bligh's creep ratio is 14.1.

West Section: Maximum Pool

For this condition, the difference in head between the maximum pool (18.25 ft NAVD88) and the water surface in the canal (8.0 ft NAVD88) is 10.25 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.02 and 0.003, respectively – resulting in a FOS against boiling of greater than 10 where the water surface meets the near slope of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.04. For an embankment base width of approximately 120 ft and a head differential of 10.25 ft, Bligh's creep ratio is 11.7. The results of the seepage analysis are summarized in the table below **(Table A-13)**.

Table A-13. Results of the seepage analysis at the Gulfstream West flow-through marsh.

Design Case	Reservoir Pool Elevation (ft NAVD88)	Seepage Rate Into Canal/Ditch (cfs/lf)	Percent of Seepage Captured by Drainage Ditch/Canal	FOS Against Boiling	Bligh's Creep Ratio	Horizontal Exit Gradient
East Section (Drainage Canal)						
Steady-State - Normal Pool	16.5	7.52E-4	55%	8.6	16.1	0.13
Steady-State - Surcharge Pool	18.25	1.03E-3	52%	6.3	12.7	0.17
East Section (FDOT Ditch)						
Steady-State - Normal Pool	16.5	5.66E-4	45%	3.0	100*	0.12
Steady-State - Surcharge Pool	18.25	6.92E-4	48%	2.5	100*	0.15
South Section						
Steady-State - Normal Pool	16.5	1.83E-3	100%	> 10	14.7	0.4
Steady-State - Surcharge Pool	18.25	2.22E-3	100%	> 10	12.2	0.5
West Section						
Steady-State - Normal Pool	16.5	1.81E-3	100%	> 10	14.1	0.3
Steady-State - Surcharge Pool	18.25	2.20E-3	100%	> 10	11.7	0.4
* Calculated between the drainage canal and the FDOT canal.						

Each of the above factors of safety for boiling meet the recommended factor of safety of 4 as stated in the BOR with the exception of the FDOT ditch on the east section for both long term and maximum pool cases. The factors of safety for the FDOT ditch, however, should not be compared with the FOS from the BOR because this feature is not a new dam. These factors of safety for the FDOT ditch do fall into the acceptable range of 2.5 to 3 as stated by Cedergren (1989). Therefore, no further analyses are required

The above calculated Bligh's creep ratios fall in the range of 11 to 17 using equation 9-21 of EM110-2-1901. These creep ratios meet the minimum for fine to coarse sand conditions as shown on Table 9-10 of EM 1110-2-1901. Based on the soils that were encountered near the site, these creep ratios are slightly lower than what would be required. Due to this, the sand and stone filters will need to be installed as previously stated.

The horizontal exit gradients in **Table A-13** above, according to the DOR, are gradients that can possibly lead to the initiation of internal erosion. This is another reason why the sand and stone filters are to be installed into the canals. For the higher exit gradients into the FDOT ditch on the east section, laboratory tests will need to be conducted on the soils in the area to determine if further remediation is needed (such as moving the drainage canal further away).

A.7.5.3 Slope Stability Analysis at the Gulfstream West Flow-Through Marsh

Slope stability analyses were performed using SLOPE/W, Version 9.1.1.16749, and the analyses considered the soil types encountered at the Gulfstream West area, as shown in representative FDOT borings **(Annex G-1** Geotechnical**)**.

A.7.5.3.1 Preliminary Strength Design for Slope Stability Analysis

The preliminary design parameters used in the stability analysis are based on field data and published values for similar materials. These values will be refined as site-specific data is obtained during the design phase of the project. **Table A-14** summarizes the selected strength parameters for each soil type.

Fill

As the embankment fill will likely come from the excavation of the exterior canals and other on-site borrow sources, the unit weight for a SP material was estimated based on typical strength characteristics for compacted soils with this composition. From Table 3.39 (Hunt), the following parameters were selected: moist unit weight saturated unit weight of 110 pcf; cohesion c = 0 psf; phi angle = 29 degrees. These parameters were used for both the End-of-Construction (EOC) condition and the Long-Term (LT) condition.

Sand Layers

Unit weights and strengths were based on Hunt and BOR shear-strength tables for similar materials. For each material, the values selected are shown in **Table A-14.**

ASTM C33 Fine Aggregate (Sand)

The properties for this material were obtained using laboratory data from a similar project, and will be applied here.

ASTM C33 No. 89 Stone

The properties for this material were obtained using laboratory data from a similar project, and will be applied here.

Table A-14. Selected strength parameters for each material type at the Gulfstream West embankment.

Material	Unit Weight (pcf)	EOC Strength Parameters c & phi (psf/deg)	LT Strength Parameters c & phi (psf/deg)
1 – Fill	115	0/33	0/33
2 – Sand (0-5)	110	0/29	0/29
3 – Sand (5-25)	110	0/29	0/29
4 – Sand (25-60)	115	0/33	0/33
5 – Sand (60-100)	120	0/36	0/36
6 – C33 Sand	120	0/33	0/33
7 – No. 89 Stone	130	0/35	0/35

A.7.5.3.2 Cross-Sections Analyzed

The three sections analyzed for seepage were also used to evaluate the stability of the embankments and canals. In each analysis, either a piezometric line was assigned or the pore-water pressures computed by SEEP/W were imported into SLOPE/W and the Mohr-Coulomb strength model was used for all materials.

A.7.5.3.3 Cases Analyzed

For new embankment dams, EM 1110-2-1902 lists four conditions for analysis: End-of-Construction (EOC); Long-Term (LT - steady-state seepage, maximum normal pool); Maximum Surcharge Pool; and Rapid Drawdown (RDD). For this Appendix, EOC, LT, Max Surcharge Pool, and RDD were analyzed. (While the project site lies within a low seismic hazard zone, appropriate methods will be used during the design phase to analyze the liquefaction potential and embankment deformation from a potential seismic event. As clean, loose sands exist within the project area capable of liquefaction, ER 1110-2-1806 will be used as a guide for this analysis).

A.7.5.3.4 Stability Modeling Results

For the sections modeled, the resulting factors of safety were all greater than the minimum required per EM 1110-2-1902 **(Table A-15)**. The critical failure surfaces determined by SLOPE/W can be found in **Annex B-1**.

Table A-15. Slope stability results for each design case for the Gulfstream West embankment.

Design Case	FTM Pool Elevation (ft NAVD88)	Slope	Required Factor of Safety	Calculated Factor of Safety
East Section				
End-of-Construction (EOC)	GW at 9 ft*	Upstream	1.3	1.41
End-of-Construction (EOC)	GW at 9 ft*	Downstream	1.3	1.73
Steady-State - Normal Pool	16.5	Downstream	1.5	1.65
Steady-State - Surcharge Pool	18.5	Downstream	1.4	1.49
Rapid Drawdown	16.5 to 13	Upstream	1.3	1.41
South Section				
End-of-Construction (EOC)	GW at 9 ft*	Upstream	1.3	1.78
End-of-Construction (EOC)	GW at 9 ft*	Downstream	1.3	1.74
Steady-State - Normal Pool	16.5	Downstream	1.5	1.51
Steady-State - Surcharge Pool	18.5	Downstream	1.4	1.46
Rapid Drawdown	16.5 to 13	Upstream	1.3	1.77
West Section				
End-of-Construction (EOC)	GW at 9 ft*	Upstream	1.3	1.78
End-of-Construction (EOC)	GW at 9 ft*	Downstream	1.3	1.60
Steady-State - Normal Pool	16.5	Downstream	1.5	1.51
Steady-State - Surcharge Pool	18.5	Downstream	1.4	1.47
Rapid Drawdown	16.5 to 13	Upstream	1.3	1.76

* Groundwater level was obtained from FDOT borings from the Weigh-in-motion station and a boring at Sta. 523+80 on sheet 23 of 49.

A.7.5.3.5 Foundation Conditions for Structures

Based on the materials encountered by the FDOT borings performed for the bridge over Cypress Creek, along I-95 in the general area of Gulfstream West, and a nearby weigh-in motion station, the structures are likely to bear on poorly graded sand. As a result, it is anticipated that any settlement will be elastic (i.e. immediate) and that pile supported foundations will not be necessary. A list of the structures for this feature of work is presented in Error! Reference source not found..

A.7.6 Nine Gems Property and Adjacent Areas

The Nine Gems properties are located west of the Gulfstream West FTM, and east of the undeveloped Pal-Mar wetlands. North-south trending ditches have partially drained this area. Prior to drainage, the landscape consisted of depressional wetlands similar to those in the Pal-Mar area. An important project objective will be to plug ditches, improve the peripheral berms, leading to improved wetland hydroperiod throughout this property.

Ditching and draining, and subsequent development throughout much of the project area has reduced the magnitude and duration of wetland hydroperiods and also fragmented unaffected wetland areas. Over-drained wetlands result in the loss by oxidation of the surface organic layer, and exposure of sandy soils (Ecology and Environment, 2009).

A.7.6.1 Soils and Shallow Subsurface Geology

Surface soils (0 to 10 cm depth) in the area of the Nine Gems property are characterized mostly as SP-SM, or generally poorly graded, slightly silty sands, with silt content less than 12 percent by weight. Soils are mapped as drained or ponded Piñeda-Riviera and Wabasso fine sands, and Boca or Nettles sand (SP) (NRCS, 2019). Surface soils show uniform saturated hydraulic conductivity estimates of approximately 9.2 x 10^{-3} cm second (26 ft/day). The classification of surface soils at the Nine Gems property is shown in **Figure A-21.** The shallow subsurface geology and hydrogeologic setting at the Nine Gems property are expected to be similar to those at the Gulfstream West flow-through marsh **(Section A.7.5.1).**

A.7.6.2 Seepage Analysis at Nine Gems

For the preliminary design of the Nine Gems embankment, SEEP/W (Version 9.1.1.16749) was used to predict the resulting phreatic surface and seepage from the operation of the adjacent natural areas. The software was also used to verify that the embankment will be safe with respect to uplift pressures and exit gradients and that there is minimal effect to the adjacent canals.

A 2D seepage model was developed using the same material properties as used for the Gulfstream West analysis. Due to stability issues, the slopes of the existing canal nearest to the proposed berm were flattened to between 1V:3H and 1V:3.5H.

A.7.6.2.1 Preliminary Design Parameters for the 2D Seepage Analysis

The design parameters selected for the 2D seepage modeling are the same as that used for the Gulfstream West analysis. See **Table A-16** below for more information on the design parameters used for the seepage analysis.

Table A-16. Preliminary hydraulic parameters for each material type at Nine Gems.

Material	Modeled Bottom Elev. (ft, NAVD88)	Kx (cm/s)	Kx (ft/s)	Ky/Kx
1 – Fill	18.0	1.0-3	3.3E-5	0.11
2 – Sand (0-5 ft)	13.0	9.17E-4	3.01E-5	0.1
3 – Sand (5-25 ft)	-7.0	2.42E-2	7.93E-4	0.1
4 – Sand (25-60 ft)	-42	2.42E-2	7.93E-4	0.1
5 – Sand (60-100 ft)	-82	2.42E-4	7.93E-4	0.1

A.7.6.2.2 Embankment and Canal General Geometries

With respect to the exterior embankments, the side slopes will be 1V:3H on the upstream slope and 1V:3.5H on the downstream slope, 12-ft crest width, and a crest elevation raised to 21.25 ft (NAVD88). **Figure A-26** shows a conceptual section through the proposed embankment. The location of the seepage section is shown in **Figure A-27**. The existing canals are trapezoidal in shape. The canal nearest to the proposed embankment will have 1V:3.5H and 1V:3H side slopes and a bottom width of 5 ft. The existing canal further from the proposed berm will not be modified as it is not on the subject property. According to available LiDAR, this canal has side slopes between 1V:1.2H and 1V:1.5H with a bottom width of 15 ft.

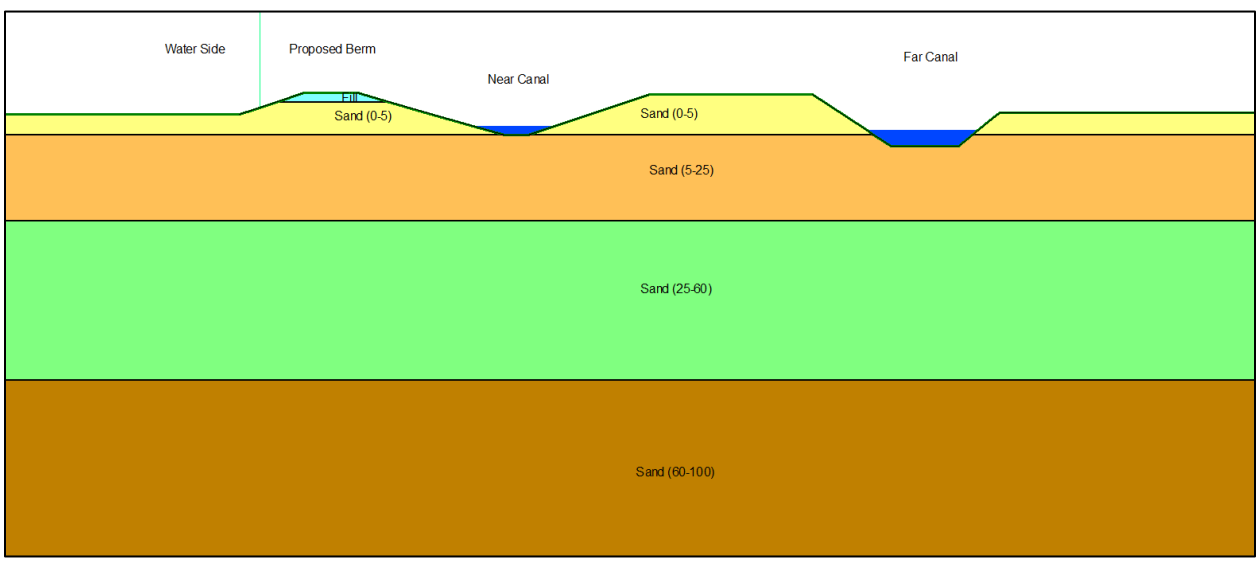
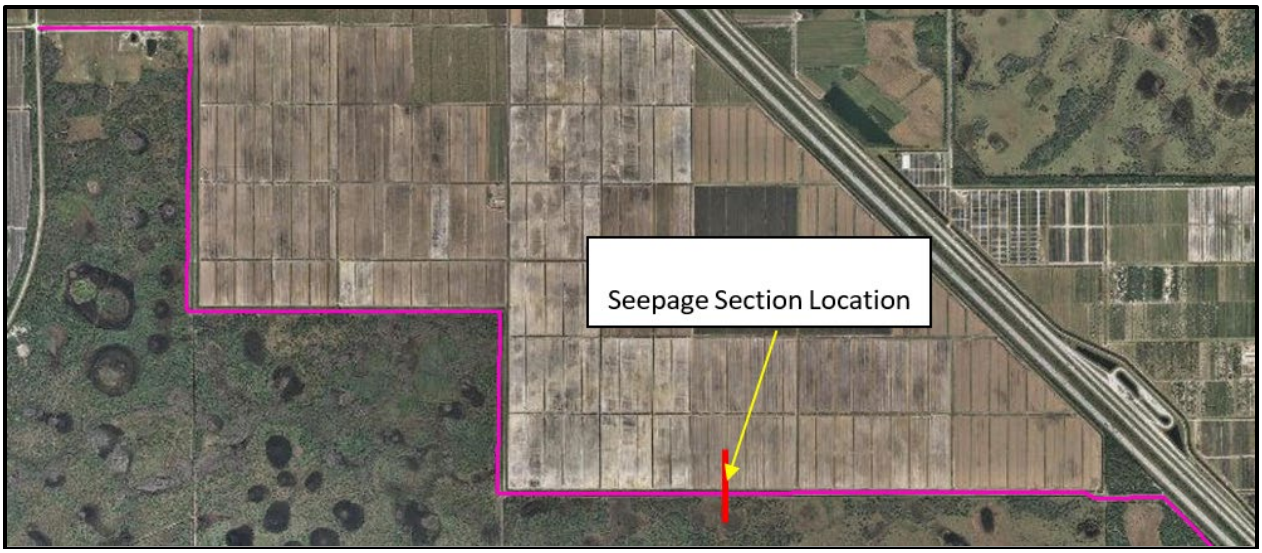


Figure A-26. Nine Gems exterior embankment (looking west).



Nine Gems Berm

Figure A-27. Seepage section location at Nine Gems (not to scale).

A.7.6.2.3 Boundary Conditions

Three different types of boundary conditions were used in the seepage modeling: Constant Head, Potential Seepage Face, and No Flow. These boundary conditions were applied as follows:

- Constant Head: Where a water body was present (the pool, adjacent canal stage, etc.), a constant-head boundary was set to model that feature. These water surface elevations were provided by the Water Resources Engineering Branch.
- Potential Seepage Face: Where the above boundary conditions were not used, a potential seepage face boundary was used for most of the remaining nodes – which would allow the software to determine the water surface elevation at those locations.
- No Flow: Water cannot pass through the boundary and neither a total hydraulic head nor a flow rate has been specified. This condition was set along the vertical sides and bottom of each model.

A.7.6.2.4 Seepage Conditions Modeled

Two seepage conditions were modeled: Long-Term (steady-state seepage with a normal, average pool in the rehydrated wetland, and a normal water level in the canals) and Maximum Pool (steady-state seepage with a maximum, operational pool depth in the rehydrated wetland and a normal water level in the canals and ditches). A typical cross-section **(Figure A-28)** showing normal pool conditions is presented below. Figures showing the remaining conditions modeled are presented in **Annex B-1** Geotechnical.

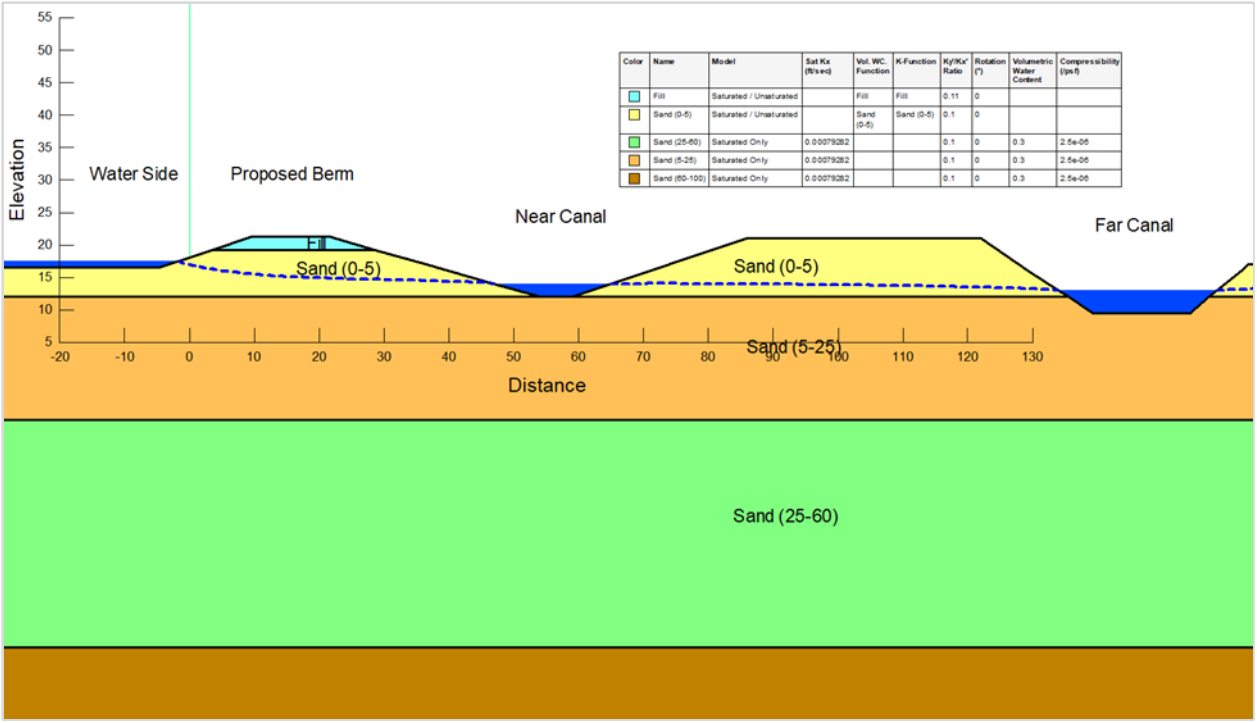


Figure A-28. Nine Gems embankment, long term condition, normal pool.

A.7.6.2.5 Seepage Modeling Results at Nine Gems

<u>Near Canal: Long Term</u>

For this condition, the difference in head between the normal pool (17.5 ft NAVD88) and the water surface in the near canal (14.0 ft NAVD88) is 3.5 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.17 and 0.14, respectively – resulting in a FOS against boiling of 4.5 where the water surface meets the near slope of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.08.

<u>Near Canal: Maximum Pool</u>

For this condition, the difference in head between the maximum pool (19.25 ft NAVD88) and the water surface in the near canal (14.0 ft NAVD88) is 5.25 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.31 and 0.28, respectively – resulting in a FOS against boiling of 2.5 where the water surface meets the near slope of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.16.

Far Canal: Long Term

For this condition, the difference in head between the normal pool (17.5 ft NAVD88) and the water surface in the near canal (13.0 ft NAVD88) is 4.5 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.11 and 0.14, respectively – resulting in a FOS against boiling of 5.4 where the near slope meets the bottom of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.06.

Far Canal: Maximum Pool

For this condition, the difference in head between the maximum pool (19.25 ft NAVD88) and the water surface in the near canal (13.0 ft NAVD88) is 6.25 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in **Annex B-1**.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.14 and 0.18, respectively – resulting in a FOS against boiling of 4.2 where the near slope meets the bottom of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.07. The results of the seepage analysis are summarized in the table below.

Table A-17. Seepage analysis results at Nine Gems.

Design Case	Reservoir Pool Elevation (ft NAVD88)	Seepage Rate Into Canal (cfs/lf)	Percent of Seepage Captured by Canal	FOS Against Boiling	Horizontal Exit Gradient
Near Canal					
Steady-State - Normal Pool	17.5	10.4E-4	20%	4.5	0.08
Steady-State - Surcharge Pool	19.25	2.11E-4	28%	2.5	0.16
Far Canal					
Steady-State - Normal Pool	17.5	4.06E-4	80%	5.4	0.06
Steady-State - Surcharge Pool	19.25	5.28E-4	72%	4.2	0.07

Each of the above factors of safety for boiling fall into the acceptable range of 2.5 to 3 as stated by Cedergren (1989). Therefore, no further analyses are required.

The horizontal exit gradients in **Table A-17** above, according to the BOR, are gradients that can possibly lead to the initiation of internal erosion. Laboratory tests will need to be conducted on the soils in the area to determine if further remediation is needed (such as moving the proposed berm further away or the installation of a liner).

Near Canal: Long Term

For this condition, the difference in head between the normal pool (17.5 ft NAVD88) and the water surface in the near canal (14.0 ft NAVD88) is 3.5 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in Annex B-1.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.17 and 0.14, respectively – resulting in a FOS against boiling of 4.5 where the water surface meets the near slope of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.08.

Near Canal: Maximum Pool

For this condition, the difference in head between the maximum pool (19.25 ft NAVD88) and the water surface in the near canal (14.0 ft NAVD88) is 5.25 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in Annex B-1.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.31 and 0.28, respectively – resulting in a FOS against boiling of 2.5 where the water surface meets the near slope of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.16.

Far Canal: Long Term

For this condition, the difference in head between the normal pool (17.5 ft NAVD88) and the water surface in the near canal (13.0 ft NAVD88) is 4.5 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in Annex B-1.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.11 and 0.14, respectively – resulting in a FOS against boiling of 5.4 where the near slope meets the bottom of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.06.

Far Canal: Maximum Pool

For this condition, the difference in head between the maximum pool (19.25 ft NAVD88) and the water surface in the near canal (13.0 ft NAVD88) is 6.25 ft. Seepage exits the STA and flows horizontally towards the canal. No seepage exits the downstream face of the embankment but the phreatic surface rises to just below the bottom of the canal. This change in the groundwater elevation and the head values driving it are shown in Annex B-1.

Exit gradients were checked at two locations along the surface of the canal – where the water surface elevation intersects the near slope of the canal and where the near slope meets the canal bottom. The calculated exit gradients (in the vertical direction) were 0.14 and 0.18, respectively – resulting in a FOS against boiling of 4.2 where the near slope meets the bottom of the canal (the critical location). At this critical location, the exit gradient in the horizontal direction was 0.07.

A.7.6.3 Slope Stability Analysis at Nine Gems

Slope stability analyses were performed using SLOPE/W, Version 9.1.1.16749, and the analyses considered the soil types encountered by the borings obtained by the FDOT.

A.7.6.3.1 Preliminary Strength Design for Slope Stability Analysis

The preliminary design parameters used in the stability analysis are the same that were used for the Gulfstream West analysis. See **Table A-18** below for more information on the soil parameters used for the stability analysis.

Table A-18. Selected strength parameters for each material type at Nine Gems.

Material	Unit Weight (pcf)	EOC Strength Parameters c & phi (psf/deg)	LT Strength Parameters c & phi (psf/deg)
1 – Fill	115	0/33	0/33
2 – Sand (0-5 ft)	110	0/29	0/29
3 – Sand (5-25 ft)	110	0/29	0/29
4 – Sand (25-60 ft)	115	0/33	0/33
5 – Sand (60-100 ft)	120	0/36	0/36

A.7.6.3.2 Cross-Sections Analyzed

The section analyzed for seepage was also used to evaluate the stability of the embankment and the nearest canal. In each analysis, either a piezometric line was assigned or the pore-water pressures computed by SEEP/W were imported into SLOPE/W and the Mohr-Coulomb strength model was used for all materials.

A.7.6.3.3 Cases Analyzed

For new levees, EM 1110-2-1913 lists three conditions for analysis: End-of-Construction (EOC); Long-Term (LT - steady-state seepage, maximum normal pool); and Rapid Drawdown (RDD). For this analysis, EOC, LT, and RDD were analyzed. (While the project site lies within a low seismic hazard zone, appropriate methods will be used during the design phase to analyze the liquefaction potential and embankment deformation from a potential seismic event. As clean, loose sands exist within the project area capable of liquefaction, ER 1110-2-1806 will be used as a guide for this analysis).

A.7.6.3.4 Stability Modeling Results

For the section modeled, the resulting factors of safety were all greater than the minimum required per EM 1110-2-1913 **(Table A-19)**. The critical failure surfaces determined by SLOPE/W can be found in **Annex B-1** Geotechnical.

Table A-19. Slope stability results for each design case at Nine Gems.

Design Case	Wetland Pool Elevation (ft NAVD88)	Slope	Required Factor of Safety	Calculated Factor of Safety
Nine Gems				
End-of-Construction (EOC)	GW at 9 ft*	Upstream	1.3	1.97
End-of-Construction (EOC)	GW at 9 ft*	Downstream	1.3	1.94
Steady-State - Normal Pool	17.5	Downstream	1.4	1.49
Rapid Drawdown	17.5 to 16.5	Upstream	1.2	1.80

* Groundwater level was obtained from FDOT borings from the Weigh-in-motion station and a boring at Sta. 523+80.

A.7.7 Eastern Project Features: Hobe Grove Weir and Kitching Creek Spreader

These project features are located east of the I-95/Florida Turnpike corridor, north of Jonathan Dickinson State Park and the Loxahatchee River. The Hobe Grove weir will reduce flashy discharge and sediment loading into the northwest fork of the Loxahatchee River, while maintaining "get-away" flood control for upstream HSLCD canals. The Kitching Creek spreader, along with a replacement culvert, well maintain existing surface water flow but will rehydrate the floodplain of Kitching Creek.

A.7.8 Geotechnical Analyses During PED

With respect to each of the anticipated components required for this project, a site-specific foundation design will be performed during PED that will include seepage/slope-stability analyses, bearing capacity and settlement analyses, cofferdam design, filter design, and other foundation/excavation/dewatering design analyses.

A.7.8.1 Settlement Analysis

Settlement analyses will be performed for each of the project embankments and structures during PED. Based on the limited field investigations performed to date, it is anticipated that the majority of the settlement will be elastic (i.e. immediate), with some consolidation settlement occurring in the clayey sand layers encountered by the borings.

These structures are to be founded on thick, wide mat foundations. The pump stations, in addition, are relatively lightly loaded. The predominant soils below the foundation grade of the structures are silty and clayey sands with occasional limerock or sandstone lenses. At each structure location, the foundation subgrade will be evaluated for bearing capacity and immediate settlement by core borings with continuous standard penetration testing. Although unlikely, should thick, soft silt and/or clay layers (or clayey/silty sand layers with over 40% fines) be encountered within the proposed structure footprint, consolidation settlement will be evaluated.

A.7.8.2 Revetment Design

Revetment for this project will be composed of graded, natural stone (riprap). The preliminary riprap design for each structure was performed using the following guidance using the velocities provided:

- EM 1110-2-1601 – *Hydraulic Design of Flood Control Channels (1994), Chapter 3 and Hydraulic Design Chart (HDC) 712-1*

- EM 1110-2-2300 – *General Design and Construction Consideration for Earth and Rockfill Dikes (2004)*
- ASTM D6092 – *Standard Practice for Specifying Standard Sizes of Stone for Erosion Control (2014)*
- ASTM C33/33M – *Standard Specification for Concrete Aggregates (2016)*

Riprap gradations for the zones immediately upstream and downstream of the proposed water control structures was designed using the D50 method for highly turbulent areas. "Local" rock (with a unit weight around 135 pcf) will be used for design velocities of 5 fps or less and "import" rock (with a unit weight around 165 pcf) will be used for design velocities greater than 5 fps. The recommended riprap sizes and volume computations are included in **Annex B-1**.

A.7.8.3 Dewatering

Given the sandy soils located throughout the project area, open pumping (for the shallowest of excavations), well points and/or wells, and steel cofferdams will likely be used to facilitate the work in the dry. Ultimately, the method selected will be based on the soils encountered, the required excavation depth, and if any adjacent water bodies exist at each location. Based on the borings performed for the C-18W Reservoir, the depth to groundwater varies from 1 ft to 13 ft below ground surface.

<u>Dam and Levee Embankments</u>
For the D-101 embankment at the C-18W reservoir, groundwater levels appear to be below the embankment foundation grade except for flooding conditions (Arcadis, 2016). Therefore, dewatering may not be required except during the wet season and where the embankment footprint crosses an existing canal or ditch.

<u>Pump Stations</u>
The deepest pumping stations have foundation at 30-35 ft bls. Where there are no constraints from existing features, open pit excavation can be accomplished with perimeter educator wells and well points, combined with foundation perimeter seepage trenches with sump pumps. It will be required that the groundwater surface within the foundation footprint be lowered to 3-to-4 ft below foundation grade.

<u>Culverts, Spillways, and Weir Structures</u>
In areas where very shallow pre-fabricated box structures can be placed in the wet such as in the middle of a canal, dewatering may not be necessary. Structures that have lateral constraints that preclude open pit excavation, such as within existing embankments or in flow ways, cellular cofferdams may be required to isolate the excavation area, protect existing features, and to minimize pumping quantities.

A.7.8.4 General Earthwork

Based on borings performed within the project limits, the foundation materials are unconsolidated and consist mainly of upland sand with silt. All of the project structures can be excavated with standard hydraulic excavators and loaders. It is anticipated that typical earthmoving equipment will be used and that rippers and blasting will not be necessary.

Embankments

Embankments (D-101) and berms will be built on grade after clearing/grubbing operations have been performed and any unsatisfactory foundation material is removed and backfilled with embankment fill. The excavations made for the water control structures will likely require a steel cofferdam to facilitate the construction of each structure in the dry. Any settlement will likely be elastic (i.e. immediate) given the free-draining, coarse-grained nature of the foundation materials encountered by the borings.

Canals and Swales

All of the proposed canals can be excavated with standard hydraulic excavators. Alternately, dragline excavation may also be used. Surficial vegetation and organic soils will be excavated and stockpiled for use as topsoil for areas to receive seed or sod. As the subgrade layers below the ground surface are predominantly sand with sporadic weak rock lenses, blasting will not be required. It is anticipated that pneumatic picks, ripper buckets or ripping claws will be used to remove any rippable rock.

Structures

Earthwork for the proposed structures can be excavated with standard hydraulic excavators. As the subgrade layers below the ground surface are predominantly sand with sporadic weak rock lenses, blasting will not be required. If necessary, it is anticipated that pneumatic picks, ripper buckets or ripping claws will be used to remove any rippable rock.

A.7.8.5 Soil-Cement

It is anticipated that 12-in of flat-plate soil cement will be required to armor the embankment crown, interior slope, and toe for the C-18W embankment. Additionally the interior separator berms within the Gulfstream West flow-through marsh will be made out of soil cement. The soil aggregate for the soil-cement production will come from the required excavations and interior borrow sources. Material will likely consist of sands classified as SP, SP-SM, and SM (per the Unified Soil Classification System). As such, it is anticipated that the cement content of the soil-cement mix will fall between 5 and 11 percent (by weight) of dry soil used. This will be verified by a soil-cement test mix program that will include test pits to obtain bulk samples and laboratory tests to determine the optimum cement content for long-term strength and durability requirements.

Uplift/stability analyses will be performed to determine the factor of safety against flotation/sliding during a controlled (i.e. emergency), rapid drawdown condition. Other conditions commonly analyzed for uplift are wind and wave set-up and breach with full pool loss. However, these conditions are considered to be too conservative and could lead to design features and armoring thicknesses that exceed what will adequately withstand the rigors of normal and emergency situation operation scenarios.

A.7.8.6 Anticipated Construction Techniques

Once the embankment footprint has been cleared of any organics, and stripped to a minimum depth of 6 inches, all embankment materials (excluding soil-cement armor) will be placed in horizontal lifts (6 to 12 inches loose lift prior to compaction) and compacted in a direction which is parallel to the embankment dam axis. Placement of embankment materials will not occur when the area is inundated, and will not be allowed on any portions of the embankment dam foundation until that portion has been proof-rolled, inspected, tested, and approved. All filter and drain materials (internal to the dam) will be from non-carbonate aggregate sources and will be washed to remove fines prior to placement. Mixing of materials

from adjacent embankment sections will not be allowed. Dumping ahead of the lift being placed will not be allowed. All spreading equipment will be required to have a "u" type blade, unless otherwise approved.

With respect to the chimney and blanket drain placement, some overbuild of these features is anticipated in order to meet the minimum line-and-grade requirements that will be shown on the drawings. Drain sand material will be placed such that the materials are always between 0.5 and 1.5 feet ahead of (higher than) the adjacent zone(s). Dumping ahead of the lift being placed will not be allowed. All spreading equipment will be required to have a "u" type blade, unless otherwise approved.

A.7.9 Geotechnical Investigations for the C-18W Reservoir During PED

The following approaches will be applied to embankment and structure design at the C-18W reservoir. A significant amount of geotechnical data exists for this site, as documented in the DDR (Arcadis, 2016) and other earlier surveys and investigations. Additional investigations will be conducted to inform the design during the PED phase.

A.7.9.1 Subsurface Explorations

Embankments and Canals

For the C-18W reservoir embankments and associated canals, Standard Penetration Test (SPT) borings will be performed every 500 to 1,000 feet along the proposed alignments and will be continuously sampled. These borings will be deep enough to adequately characterize the foundation conditions below each embankment/canal. Where cohesive materials are encountered, undisturbed Shelby tube samples will be obtained so that shear-strength and consolidation values can be determined. Along the proposed seepage/drainage canal alignments, test pits will also be performed so that bulk samples can be obtained for compaction tests and for soil-cement strength and durability tests. Additionally, laboratory tests to determine grain-size, moisture content, Atterberg limits, and specific gravity will be performed on boring/test-pit samples to further characterize the materials encountered.

Culverts, Spillways, and Weir Structures

Each structure will receive a minimum of two SPT borings and core barrel drilling in rock, where encountered. At least one boring will be deep enough to identify all suitable bearing layers and to establish the hydrogeological properties of underlying strata for modeling purposes. Where cohesive materials are encountered, undisturbed Shelby tube samples will be obtained so that shear-strength and consolidation values can be determined. Laboratory tests to determine grain-size, moisture content, Atterberg limits, and specific gravity will also be performed on boring samples to further characterize the materials encountered.

Pump Stations

All of these structures will be constructed, in part, below the groundwater table. Therefore, the field investigations will include groundwater level measurements to facilitate the design of the dewatering system at each structure location. At least four SPT borings will be performed for each pump station and core barrel drilling will be used if rock is encountered. One boring will be deep enough to identify all suitable bearing layers and to establish the hydrogeological properties of underlying strata for modeling purposes. Where cohesive materials are encountered, undisturbed Shelby tube samples will be obtained so that shear-strength and consolidation values can be determined. Laboratory tests to determine grain-size, moisture content, Atterberg limits, and specific gravity will also be performed on boring samples to further characterize the materials encountered.

A.7.9.2 Detailed Seepage Analyses

Preliminary 2-D seepage analyses have been completed here for the C-18W reservoir and the Gulfstream West FTM. The 2-D seepage simulations at the C-118W reservoir have the greatest detail due to the availability of subsurface lithological data and permeability data. However, a 3-D seepage model should be developed for the C-18W reservoir during PED, after geotechnical data from the embankment alignment are obtained. A preliminary 2-D seepage analysis was performed for the Gulfstream West FTM, which concluded with the addition of a seepage canal between the embankment and the I-95/Florida Turnpike corridor. However, this analysis was based on limited subsurface data. A 3-D seepage model should be developed for the Gulfstream West FTM during PED, after geotechnical data from the embankment alignment are obtained.

A.7.9.3 Potential Failure Mode Analysis and Semi-Quantitative Risk Assessment

A PFMA/Qualitative Risk Assessment was completed for the C-18W reservoir in July 2019. The level of design of embankments, canals, and structures is conceptual for a feasibility study, so the risk assessment is considered qualitative. As per ER 1110-2-1156, a more rigorous PFMA/Semi-quantitative Risk Assessment must be performed once the intermediate level of design has been completed for C-18W reservoir embankments, canals, and structures. The hazard potential classification of the Gulfstream West FTM also must be confirmed.

A.7.9.4 Potential Borrow and Disposal Sites

Embankment material for the C-18W reservoir will likely come from the excavation of the exterior seepage canal and the interior borrow sources. Interior borrow sources are expected to parallel the work and be no deeper than 10 ft below ground surface. Aggregate for soil-cement production will come from interior borrow sources. The construction of the seepage canal provides an adjacent borrow source for the reservoir embankment and multiple borrow sources for soil-cement production will limit the haul distance from any one borrow site to the interior face of the embankment. Disposal of suitable and unsatisfactory material will be in areas clearly indicated on the drawings.

Stockpiled materials from the canal and structure excavations will be used for embankment construction with minimal processing required to meet the suitability requirements in the earthwork specifications. Embankment fill should be free of organic material, trash and debris, and have a maximum particle size of 4” measured in any direction. Stockpiles of segregated materials will be blended to an extent that the embankment is nearly homogeneous in an effort to prevent gap grading. Following excavation to foundation grade, the foundation subgrade will be inspected for evidence of voids and the presence of peat, organic silts, and clays. Deep layers of peat do not appear within the limits of the C-18W embankment alignment, however, if such areas are encountered, foundation improvement recommendations will be implemented during the design phase. Shallow pockets of organic soils will be removed and replaced with suitable fill material. Embankment fill placement will be in loose lifts not to exceed 12-in. in thickness. Oversized borrow materials (if encountered), should be crushed for use as embankment fill or separated and reused as rockfill or riprap slope protection material. Embankment material quantity calculations for the C-18W reservoir are shown in **Annex B-2, Table B-2.1.** Cut-and-fill calculations to estimate materials volumes for the embankment are presented in **section A.6.2.2.**

A.7.9.5 Groundwater Monitoring

Monitoring wells open to shallow and intermediate depths of the SAS will be constructed around the periphery of the C-18W reservoir. These wells will be instrumented with pressure transducers to measure groundwater levels year round. The objectives of this groundwater monitoring program will be 1) to establish how groundwater levels vary seasonally prior to construction; 2) to document changes in groundwater levels during construction; and 3) to evaluate post-construction groundwater levels when water is stored in the reservoir. A separate monitoring plan for groundwater levels and quality will be required as part of the Underground Injection Control (UIC) permit application for the ASR system associated with the reservoir. A draft ASR groundwater monitoring program is found in **Annex D** Adaptive Monitoring Plan of the PIR-EIS.

A.7.10 Geotechnical Investigations at the Gulfstream West Flow-Through Marsh During PED

Few subsurface data exist at this feature. Borings for the I-95/Florida Turnpike design were obtained from FDOT, and these data were incorporated into the seepage model for this feature. It is expected that the subsurface explorations to support berms, canal expansion, structure, and pump station design will be similar to that defined in **section A.7.9.**

Construction of flow-way 3 features requires significant earthworks. Materials to plug ditches in the Nine Gems area, and modifying components of the Gulfstream West flow-through marsh will be obtained from grading the footprint of the flow through marsh. Cut-and-fill calculations to estimate materials volumes are presented in **section A.6.5.2.**

A.7.11 Hazardous, Toxic, and Radioactive Wastes

The Recommended Plan proposes two new features: the Gulfstream West flow-through marsh (1200 acres) and the C-18W reservoir (1916 acres). The Gulfstream West flow through marsh acreage has Phase I/II/III Environmental Site Assessments (as the Sunrise Boys property) conducted by Dames & Moore (1989, 1992, 1997), Foster Wheeler (2002), URS (2003, 2009), and Tetra Tech (2012). The Gulfstream West area was formerly used for citrus cultivation but has been fallow since 2000 due to a canker infestation. Completed corrective actions included the removal of a total of four above ground and two underground petroleum storage tanks, impacted soil, and contaminated groundwater. Six pump stations with associated AGSTs were also decommissioned/removed. FDEP concurred with the completed corrective actions and recommendation for No Further Action for these areas.

The C-18W reservoir has Phase I/II Environmental Site Assessments conducted by URS (2004) and Professional Service Industries (PSI; 2012) as the former Mecca Farms property. Mecca Farms was originally cultivated for citrus, but has been fallow since 2004. Surface water sampling and source removal activities were recommended in this report, but there is no record that these activities have been completed. The Mecca Farms chemicals of concern for a flooded condition are: barium (exceeded SQAGs TEC/PEC), chromium (exceeded SCTL-LSW and SPLP SWCTL), and selenium (exceeded SCTL-LSW, SPLP SWCTL, and USFWS Interim Screening Level), and TRPH (in soil and groundwater above CTLs). FDEP and USFWS requested current sampling of borrow pit water to evaluate actual sediment leaching of arsenic, barium, chromium, and selenium. If surface water concentrations meet the SW-CTLs, then no sampling at start-up would be needed. PSI recommended soil and groundwater remediation at the #2 Pump Station for the TRPH and testing for chromium and selenium in surface water at project start-up.

Aerostar (2017) summarized the results of historical Phase I and II Environmental Site Assessment (ESA) findings/conclusions (prepared by others for the SFWMD within the LRWRP project area. The purpose of this summary was to identify those areas with activities or conditions adverse to the project water quality. Numerous active chemical facilities (RCRA, ASTs, USTs) and cleanup sites were identified from these ESAs, but no facilities are suspected of negatively impacting the project water quality.

The SFWMD owns the land for both the Gulfstream West flow through marsh and the C-18W reservoir features. As SFWMD is charged with the remediation/corrective action of any environmental liabilities prior to USACE construction, 18-12 months before construction is scheduled to start, the SFWMD repeats desk-top screening and field reconnaissance to confirm no new environmental releases have occurred. This information is used to plan their final evaluation studies and to complete all remaining corrective actions of the Recommended Plan properties. USACE HTRW personnel will review the HTRW condition of the affected parcels and ensure that the proper due diligence is performed in accordance with ER 1165-2-132 prior to certifying lands for construction. Should remediation of HTRW contamination be required, it is the responsibility of the SFWMD, the non-federal sponsor and is not a creditable cost to the project.

A.8 Hydraulic Design

The following subsections provide a brief overview of the hydraulic design criteria, parameters, and intent/purpose of the major project features in the Recommended Plan. Features identified in the Recommended Plan have been designed to the level of detail necessary to provide a Class 3 cost estimate, as specified in ER 1110-2-1302 for the Feasibility project phase, and determine the feasibility of hydraulic design. A Class 3 cost required technical information approaching 10-60% quality of project definition and estimates must be supported by technical information. All components have been identified, sized appropriately according to available modeling data, historic information, and best engineering judgment. All project components will be optimized during PED phase for cost efficiency and performance, incorporating updated data and information as it becomes available. General hydraulic design of all identified components are described in the following sections. Detailed hydraulic design of individual components including hydraulic design data sheets can be found in **Annex A-1**.

A.8.1 General

Subsections below describe data and analyses that are common to many features of the project.

A.8.1.1 Design Criteria and Parameters

Due to the minimal relief in the topography between the upstream and downstream ends of each structure in the project area, the hydraulic head losses across many of the control structures are low, resulting in the design of larger structures (number and size of barrels, bays, etc.) than may typically be assumed for other regions. The use of pumps was avoided wherever possible to reduce operation and perpetual maintenance costs. During PED phase, USACE SAJ expects to optimize system operations and therefore structure sizes for cost and performance efficiencies.

Design flow rates for the pump stations were determined based on the Lower East Coast Sub-Regional Model- North Palm (LECSR-NP) model outputs. To capture cost impact adequately, structures and canals shall be designed for maximum capacity scenarios. Optimization of these features will be conducted during the PED phase for performance and cost efficiency.

A.8.1.2 Hydraulic Data

Hydraulic data were gathered from several sources, including canal and structure “as-built” drawings and flow and stage recorders. Refer to the **Annex A-1** for complete hydraulic design data information.

The project canals were assumed to be maintained to “as-built” specifications. Operations of the proposed project structures are detailed in **Annex C** to the PIR-EIS (Draft Project Operating Manual) which will also address the operations of related SFWMD and local water control district structures. Detailed design and construction will also require coordination with many agencies. The design and construction of project features will meet all applicable federal, state, and local regulatory requirements.

A.8.1.2.1 Canals

Current canal design capacities will be maintained in existing canals in the LRWRP area, unless modifications are required due to the Recommended Plan project features. Based on the Recommended Plan existing canals that have adequate capacity to convey water from the C-18W reservoir to the Northwest Fork of the Loxahatchee River include: C-18W Canal, M-Canal, M-O Canal, and C-18 Canal. Survey data, including bathymetry data of the C-14 Canal is needed before a determination regarding

possible canal modification can be made. It is assumed that during the PED phase survey data will be obtained and confirmation of canal capacity and potential modifications will be designed. New project canals associated with the C-18W reservoir include the C-101W canal connecting the M-O Canal to the C-18W reservoir seepage canal, C-101E seepage canal, and C-101N, the inflow/discharge canal to the C-18W.

Several local drainage canals will require relocation, including the southern portion of Hobe St. Lucie Control District (HSLCD) outfall canal and Thomas Pepper Farms outlet canal within the Nine Gems area (also called the southern Nine-Gems Canal). Additionally, new canals for a spreader swale within the Cypress Creek Natural area and Kitching Creek Natural Area, C-114 and C-116 respectively, were also designed as part of the Recommended Plan.

Preliminary design of diversion canals for the project or improvements to existing canals were estimated initially using spreadsheet models based on the Manning's equation. The hydraulic performance of the project canals, where necessary, were verified through additional analysis using HEC-RAS modeling. Cross-sectional area of proposed or modified canals will be sized such that velocities and associated scour or canal embankment erosion are minimized. Canals were designed for conveyance velocities necessary to avoid potential erosion damage with a target of under 2.5 ft/s. If, under design conditions, velocities exceeded the 2.5 ft/s than additional geotechnical analysis was performed to determine the appropriate canal lining necessary. It is expected that material excavated from canals is to be used to fill other canals as part of wetland restoration efforts, side cast along the canal bank, or if suitable, used to form berms around wetland perimeters or as fill for the proposed reservoir components.

A.8.1.2.2 Pump Station Design

The Recommended Plan includes multiple pump stations, as specified in **Table A-20.** The project pump stations include the S-100, S-109, S-106, S-101A, S-113N, and S-110. Pump station capacities were originally estimated from the LECSR-NP model, using historical knowledge of the conditions upstream and downstream of the proposed pump station, as well as the desired hydrologic outcome with the addition of the pump station. Prior to the final PIR, pump station capacities were further analyzed using the LECSR-NP average daily flow-frequency results. The flow-frequency curves were analyzed to determine both the suitable pump station capacity criteria as well as the appropriate pump mix to achieve the desired hydrologic response. The expected range of water surface elevations on both the intake and discharge sides of each pump station were determined through canal stage records, canal as-builts and survey data, local ground elevations from LiDAR, and operational controls determined from the hydrologic modeling LECSR-NP results. These elevations were used to define the normal pumping head differential and the maximum pumping head differential for each pump station. These head differentials provide mechanical engineers with the static head component of the total head value that the pumps must overcome. Pump mixes were suggested to provide flexible operations during varying headwater and tail water conditions, while minimizing pump cycling. Redundancy of pump sizes were utilized to reduce operations and maintenance costs. Redundancy and telemetry operations are required for pump stations that have a flood control component, as specified in the SFWMD Major Pump Station Engineering Guidelines. The preliminary designs of these structures were standardized whenever possible to simplify design, construction, and future operations and maintenance. Computational methods for pump station design are described further in EM 1110-2-3102, *General Principles of Pump Station Design and Layout*, EM 1110-2-3104, *Structural and Architectural Design of Pumping Stations*, and EM 1110-2-3105, *Mechanical and Electrical Design of Pump Stations*. The pump station design follows the SFWMD Major Pumping Station

Engineering Guidelines, 4th edition, as well. Additional details related to the pumping stations for this project can be found in **Annex A-1**.

Table A-20. Pump stations proposed in the Recommended Plan.

Structure Number	Location	Purpose (from to); size	Flow-way	Provides Flood Control?
S-100	M-1 Basin	Transfer of water from the lower M-1 Basin (Lower ITID) to the M-Canal for deliveries to the River; 75 cfs	1	No
S-109	M-O Canal Connector (C-101W)	Connect the M-O Canal (ITID Basin) to the C-18W reservoir; 150 cfs	2	No
S-106	C-18W reservoir	Inflow to reservoir from M-O Canal connector canal and inflow/seepage management along western perimeter of proposed reservoir footprint; 175 cfs and 30 cfs, respectively	2	Yes, seepage pump only
S-101A	C-18W reservoir	Inflow to reservoir from C-18W Canal; 150 cfs	2	No
S-113N	Cypress Creek Canal	Mack Dairy Spreader – Deliver water from the Cypress Creek Canal to the Cypress Creek Natural area for rehydration; 20 cfs	3	No
S-110	Gulfstream West	Distribute water from the HSLCD Canal to the flow through marsh; 150 cfs	3	No

A.8.1.2.3 Aquifer Storage and Recovery System Pumps

ASR systems consist of a surface facility with water pre-treatment components, intake and outfall structures, one or more recharge/recovery (or ASR) wells, and groundwater monitoring wells. Surface water will enter the ASR well through an intake structure consisting of a wet well in which the recharge pump is submerged. Recharge pump specifications are:

- Vertical turbine pump
- 3,500 GPM (5 MGD) capacity
- 300 horsepower
- Stages
- Constant speed drive
- Nominal efficiency 95.4 percent

Pumped water passes through a filter to remove particulates, and is then conveyed by pipe to the ASR wells which are constructed to convey flow at a rate of 5 MGD (3,500 gallons per minute), resulting in velocities of 2.5 ft per second in the well bore. Wells have telescoping casings, with a 42-inch diameter steel pit casing, enclosing a 34-inch diameter steel surface casing through the surficial aquifer. A 24-inch steel production casing extends to the depth of the storage zone. Water is pumped into the limestone rock storage zone at an uncased, unscreened “open interval” at depths of approximately 800 ft to 1000 ft

below land surface (bls). Precise depths of each cased interval, well and final well construction specifications are determined during construction of the exploratory borehole.

Stored water is recovered using a separate recovery pump located on the ASR wellhead. Recovery pump specifications are:

- Vertical turbine pump
- 3,500 GPM (5 MGD) capacity
- 3 stages
- 150 horsepower
- Constant speed drive

Recovered water is then released into the C-18W reservoir via a pipe and outflow structure. The pipeline connecting the ASR well with the impoundment interior must traverse a high-head embankment. The design of the outlet structure will be defined further pending hydraulic analysis of flows and geotechnical evaluation of embankment construction.

A.8.1.2.4 Culverts Design

The proposed project includes several gated and un-gated culvert structures, as described in **Table A-21**. 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 were standardized whenever possible to simplify design, construction, and future operations and maintenance.

Culverts are hydraulically short conduits that convey flow through embankments or past some other obstruction to flow. They are constructed from a variety of materials and are available in many different shapes and configurations and 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.

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. Hydrologic Engineering Center River Analysis System (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*. Additional detail related to the culverts that are to be constructed for this project and the associated HEC-RAS modeling completed can be found in **Annex A-1.**

Table A-21. Culverts proposed in the Recommended Plan.

Structure Number	Location	Proposed Q (cfs)	Purpose	Length (ft)	Flow-way
G-161	GWP	75	Multipurpose, connect GWP to C-18 Canal	240	1
S-102	C-18W reservoir	300	Discharge Structure from Reservoir	50	2
S-104	C-18W reservoir	500	Connection of J.W. Corbett WMA to the C-18W reservoir Seepage Canal	30	2
S-103W and S-103E	C-18W reservoir	500	Discharge from C-18W reservoir Seepage Canal to C-18W Canal	20	2
S-107	J.W. Corbett WMA	300	Connection of J.W. Corbett WMA run-off to the C-18W reservoir Seepage Canal	150	2
S-105	Seminole-Pratt Whitney Road	150	Connection of C-101W (M-O canal connector) to C-101E (C-18W reservoir Seepage Canal)	80	2
S-115A-C	Cypress Creek Canal	Variable	Automate inlet control	Variable, 50 - 86	3
S-114A and S-115B	Nine Gems	105	Nine Gems discharge to Cypress Creek Canal	60	3
S-114C	Nine Gems	240	Nine Gems discharge to Cypress Creek Canal	60	3
S-114D	Nine Gems	500	Nine Gems discharge to Cypress Creek Canal	Approx., 50	3
S-114E	Nine Gems	500	Discharge from HSLCD to Cypress Creek Canal	Approx., 45	3

A.8.1.2.5 Spillway Design

The proposed project includes several gated and un-gated spillway structures. Gated spillway structures provide efficient flow control using sliding gates, while un-gated spillways provide flow control within the various project feature by allowing water to efficiently discharge when the water surface exceeds a pre-determined elevation. The Recommended Plan includes an ogee weir concrete spillway with steel vertical lift gates for both G-160 and S-112. Gated structures are proposed in key areas or where higher flows need to be more actively managed. The vertical gates allow for controlled discharge operations which are desired for operational flexibility and to facilitate adaptive management. Un-gated spillway structures are proposed for S-111 and the emergency discharge structure from the C-18W reservoir, S-101B. Additionally, two plunge pools with un-gated concrete sills are proposed for S-113S and S-111N. Additional detail related to the spillways that are to be constructed for this project can be found in **Annex A-1.**

Auxiliary Spillway at C-18W Reservoir

The C-18W reservoir is to be located directly north of the Acreage residential community. Due to the proximity of the proposed impoundment to the community, the impoundment requires an overflow auxiliary spillway, S-101B. Auxiliary spillways are non-gated non-mechanical structures that do not require human intervention for uncontrolled discharge operations. An auxiliary spillway allows for increase public safety, as excess water is able to be discharged from the spillway at a higher rate than the normal discharge structure would allow, thus lowering the maximum surcharge pool level. The S-101B structure will contain both a service and auxiliary spillway with design discharges of 105 cfs and 1,960 cfs, respectively.

A.8.1.2.6 Reservoir Design

The Recommended Plan includes one high hazard potential classification (C-18W reservoir) and one low hazard potential classification (Gulfstream West, Flow-through Marsh) feature per Design Criteria Memorandum (DCM-1) Hazard Potential Classification (SFWMD, 2005). This classification dictates additional reservoir design criteria presented in DCM-2 through DCM-5. This criteria as well as USACE criteria are presented in **Table A-22,** below and were used during the preliminary hydraulic design of both reservoirs.

Table A-22. USACE Engineering Manuals & DCM Reservoir Design Criteria.

Engineering Analysis	Description
Seepage	1110-2-1901 Seepage Analysis and Control from Dams
Slope Stability	1110-2-1902 Slope Stability
Settlement	1110-1-1904 Settlement Analysis
Canals	1110-2-1601 Hydraulic Design of Flood Control Channels
Embankments	1110-2-1614 Design of Coastal revetments, Seawalls, and Bulkheads
Hazard Classification	DCM 1 Hazard Potential Classification
Wind/Wave	DCM -2 Wind and Precipitation Design Criteria for Freeboard
Reservoir Spillway	DCM-3 Spillway Capacity and Reservoir Drawdown Criteria
Dam Embankments	DCM-4 Minimum Dimensions of Dams and Embankments
Pump Stations	DCM-5 Major Pump Station Engineering Guidelines

A.8.2 C-18W Reservoir

The C-18W reservoir is a 9,500 acre-feet above-ground reservoir, designed to be 7.5 feet deep and with an embankment height of 20 feet. The reservoir holds a High Hazard based on the Hazard Potential Classification (HPC) found in the DCM-1 – Hazard Potential Classification (SFWMD, 2005). The hazard potential classification results in the selection of the appropriate inflow design flood (IDF) and freeboard requirements, and is used to determine other design criteria for the dam and facilities.

The purposes of the reservoir are (1) capture excess flows from J.W. Corbett WMA, Indian Trail Improvement District (ITID) and the C-18W basin, 2) deliver water to meet the target restoration flows for the National Wild and Scenic Northwest Fork of the Loxahatchee River (NWFLR), and 3) change the timing of flow distribution and reduce waste to tide. The reservoir will release water back to the C-18W Canal as needed and available during low-flow periods, for delivery to Lainhart Dam and the Loxahatchee River

downstream. The C-18W reservoir will be designed to capture and store specific volumes of runoff that would otherwise be discharged to the C-18W Canal or through the ITID discharge structures (to L-8 Canal or C-51 Canal). This reservoir feature will have two inflow pump stations, a seepage pump station, an auxiliary overflow spillway, a service spillway, a drawdown structure/gated culvert, and seepage collection canal.

The embankment side slopes will be 3H:1V. Embankment side slopes are generally set at 3H:1V, 4H:1V, or 5H:1V depending on such factors as foundation conditions, borrow material characteristics, duration of high water, wind set-up and wave run-up effects, and maintenance. This slope will require the least amount of borrow material, while still allowing maintenance operations such as mowing.

The embankment will have an adjacent seepage collection canal to capture foundation seepage. Other seepage control measures will be utilized as necessary (see **Annex B-1** Geotechnical Annex). Material quantities (cut and fill) were calculated for the C-18W reservoir based on conceptual seepage canal cross-sections and lengths (cut) and conceptual embankment cross-sections and lengths (fill). Cut and fill quantities were based on the required amount of material necessary for the construction of an embankment as well as material available from excavation of the seepage canal. The embankment has a top elevation of 40.5 ft NAVD88, and an average ground elevation of 20.5 ft NAVD88. For cost estimation purposes, a compaction factor of 0.85 and a swell factor of 1.10 will be used, and 70 percent of the excavated material will be assumed usable. Shortfalls in fill material will be met with additional excavation within the project footprint. The material quantities calculations will be updated during detailed design, when more comprehensive topographic and geotechnical data will be available

The reservoir is intended to have outlet structures with invert elevations set to allow complete drainage for maintenance. Computational methods for above-ground storage design are described further in EM 1110-2-1420, *Hydrologic Engineering Requirements for Reservoirs*, EM 1110-2-1901, *Seepage Analysis and Control for Dams*, EM 1110-2-1913, *Design & Construction of Levees*, EM 1110-2-2220, *Gravity Dam Design*, EM 1110-2-2300, E*arth & Rock-Fill Dams General Design & Construction Considerations*, and ER 1110-8-2(FR), *Inflow Design Floods for Dams and Reservoirs*.

A.8.2.1 General Description of C-18W Reservoir Hydraulic Design

The main inflow/discharge canal for the C-18W reservoir is located between the C-18W Canal and the northern C-18W reservoir embankment. The inflow pump station has a 150 cfs capacity and is located south of the intersection of the northern reservoir embankment and the inflow canal. The pump can bring available water into the reservoir from the C-18W Canal. The sources of water that discharge to the C-18W Canal include the Hungryland Slough and several existing culvert connections to undeveloped lands south of the C-18W Canal and west of the North Palm Beach County Regional Airport.

The J.W. Corbett WMA currently discharges to the proposed C-18W reservoir footprint via a sheet-pile weir. The existing weir structure is to be replaced by an operable culvert structure, S-107, that will help control discharges to the seepage canal while simultaneously providing improved ecological conditions due to the operational flexibility during the wet and dry seasons. Within the J.W. Corbett WMA, elevated stages may cause an increase in the loading frequency and duration of the berm separating the J.W. Corbett WMA from the M-O Canal (at ITID), which will require further investigation during PED. The proposed operable structure will be three 72-in gated culvert structures located near or at the existing sheet-pile weir and will span under Seminole-Pratt Whitney Road. The gates will be telemetry operated and may require redundancy for flood control purposes. An operable structure will reduce the potential for flooding both upstream and downstream of the weir while allowing managers to better control and mitigate any flood impacts by providing the project with flexibility in timing and distribution of water from

both ITID and the J.W. Corbett WMA. Retaining the ability to optimize the gate opening following the construction of all project features provides considerable flexibility for ongoing management in the area. Berm improvements along the eastern perimeter of J.W. Corbett WMA will be required to help retain water to the design storm requirements. The berm improvements shall occur along Seminole Pratt Whitney Road and will be become a project Levee, L-101W. The levee will be constructed to a minimum crest elevation of 25.0 ft NAVD88 and top width of 25 ft.

A connector canal, C-101W, between the M-O Canal and the C-18W reservoir seepage canal is proposed to bring excess water from the ITID upper basin. Due to the topography of the area, this connector canal will require water to be pumped from the M-O Canal. The proposed pump station, S-109, will be located at the eastern terminus of the M-O Canal. The connector canal will be constructed adjacent to Seminole-Pratt Whitney Road, on the eastern border of the J.W. Corbett WMA. The new M-O connector canal can send up to 150 cfs to the C-18W reservoir seepage canal under the following conditions: 1) when C-18W reservoir has available capacity and ITID upper basin water is available; and 2) when ITID water stages are above wet (14.5 ft. NAVD88) or dry (15.5 ft. NAVD88) season control stages. The canal was designed using anticipated design flow conditions, headwater and tailwater elevations as well as maintaining the appropriate velocities to minimize erosion. A multi-barrel culvert containing three 60-in barrels, S-105, will carry the discharge from C-101W to the C-101E seepage canal under Seminole-Pratt Whitney Road (and L-101W).

The water collected from both S-107 and S-105 will be pumped from the seepage canal into the reservoir by S-106 at a rate of up to 175 cfs and will be located along the western perimeter of the C-18W reservoir.

Multiple general design requirements for the C-18W reservoir were considered in the design of the outflow structures, seepage management, and overall reservoir design. Due to the reservoir's location and storage being maintained above ground, the C-18W reservoir was classified as a "high" hazard impoundment using Design Criteria Memorandum 1: DCM-1 (SFMWD, 2005) due to the potential loss of life, lifeline losses, and loss of property being significant if a breach were to occur. This imposes a stricter and more robust set of design criteria, as outlined in the Design Criteria Memorandum 2: DCM-2 (SFWMD, 2006), for the reservoir. Therefore the highest, most current standards in design and construction were used for the reservoir as well as additional guidelines regarding a regular inspection program and emergency action plan will be followed once constructed.

There will be two main outflow structures at the reservoir: 1) a gated culvert discharge structure for normal operations; and 2) an auxiliary overflow spillway. The gated culvert structure, S-102, will be located along the northern embankment of the reservoir, closely situated to the inflow pump station. The design is for two 4ftX6ft gated box culverts that can discharge 300 cfs, depending on the stages within the reservoir. The proposed overflow spillway design is an 80 ft wide concrete spillway crest at an elevation of 29.3 ft NAVD88. It will be located adjacent to the culvert discharge structure and discharge into C-101N. The overflow spillway is designed to convey excess flood water to the C-18W Canal and away from the residential areas on the south side of the impoundment.

The seepage management system adjacent to the reservoir will maintain seepage through the use of seepage collection canals. The seepage canal collection system will be managed by a 30 cfs pump station located along the western embankment perimeter and will be co-located with the inflow pump stations. The seepage management system was evaluated for the anticipated head differences between the reservoir and the adjacent groundwater areas for verification of seepage rates. The seepage collection canals were designed with 3 horizontal to 1 vertical (3H:1V) side slopes, with a depth of approximately 17 ft and bottom width of 20 ft.

The main components of the reservoir design were the embankment design, erosion protection, and borrow material. Initial estimates for the embankment geometry were based on standard design requirements from DCM-2 for wind setup and wave run-up over that of the normal pool elevations. Additional requirements for freeboard included the potential probable maximum flood inflow volume (direct rainfall on site). These design consideration resulted in an embankment elevation of 40.5 ft NAVD88 with a normal design pool depth of approximately 7.5 ft or a normal pool elevation of 27.5 ft NAVD88. The embankment crest will be 14 ft wide and the exterior and interior slopes of the basin from the toe to crest are 3H:1V. Riprap lining of the intake/discharge structures and along the embankment side slope was included to help with erosion control either due to higher velocities near the structures or wave action for the interior side slopes.

A four-well ASR system is proposed beyond the western embankment toe of the C-18W Reservoir (L-101W). The ASR system will recharge from, and recover to the reservoir, which will store 9,500 ac-ft of surface water that is slowly (8 cfs, or 5 MGD) recharged into the aquifer during wet periods. Water will be recovered at the same rate into the reservoir. Sources of water for the reservoir will be excess flows from ITID (when available), seepage from the J.W. Corbett WMA, and overflow from a new weir (S-107), located along the eastern edge of J.W. Corbett WMA

A.8.2.2 Features of the C-18W Reservoir

The C-18W reservoir will have the features listed below. Additional detail regarding the hydraulic design of the various features for this project can be found in **Annex A-1**.

Canals

The C-18W reservoir will have a perimeter canal, composed by canals C-101E and C-101W, to collect seepage from the reservoir. An outlet canal, C-101N, is required to connect the C-18W Reservoir to the seepage collection canals, and the C-18W Canal. Canal C-101W also will be constructed to convey water pumped from the M-O Canal.

Culverts and Spillways

There will be several culverts and spillways required for the C-18W Reservoir. Two culvert structures, S-105 and S-107, will be required to convey flow to the C-18W Reservoir from the ITID basin and J.W. Corbett WMA area, respectively. There will be two main outflow structures at the reservoir, a gated culvert discharge structure for normal operations and an overflow spillway to provide permitted discharges and control the reservoir level during larger storm events. The seepage collection canal will contain two gated culvert structures, the S-103E and S-103W to control releases from the seepage collection canal when necessary. Additionally, both structures will provide additional discharge capability during larger events when the J.W. Corbett WMA discharges through S-107. One culvert structure, S-104, will be necessary to allow access across the seepage canal to the reservoir embankment.

Pump Stations

There will be several pump stations for the C-18W Reservoir. These will include a 175 cfs inflow pump on the western boundary of the C-18W Reservoir within the C-101E Seepage canal as well as a 150 cfs inflow pump on northern boundary at the C-18W canal. Additionally a 30 cfs seepage pump, also located on the western side of the reservoir within the seepage canal is necessary to return seepage water to the C-18W Reservoir.

Other Features

The ASR system consists of four ASR wells capable of pumping a total rate of 5 MGD. An emergency spillway will be provided for the C-18W reservoir.

A.8.3 Cypress Creek Canal Modifications

The Cypress Creek Canal is the main drainage canal within flow-way 3, conveying runoff from multiple natural watersheds and developed areas to the northwest fork of the Loxahatchee River. The canal's headwater is the Culpepper (Pal-Mar East) watershed to the east and multiple neighborhoods of the Ranch Colony area. As the canal flows eastward, additional drainage from the Tailwinds and Old Trail neighborhood, as well as runoff from the existing Gulfstream West property and HSLCD drainage canal enter the Cypress Creek Canal. Finally, discharge from the Cypress Creek Natural area discharges into the Cypress Creek Canal just upstream of I-95. The following sections describe the hydraulic design needs for this management measure.

A.8.3.1 General Description of the Cypress Creek Canal Hydraulic Design

Cypress Creek Canal modifications include modifying three existing Culpepper water control structures and adding a water control structure within the Cypress Creek Canal. Modifications to the existing water control structures to add telemetry operated gates are proposed for S-115A, S-115B, and S-115C to help maintain desired water surface elevations on the Pal-Mar East property. This modification will help achieve a more desirable hydroperiod within the Culpepper property, while simultaneously reducing discharges into the Cypress Creek Canal under certain conditions. The fourth water control structure within Culpepper, also known as the Jupiter Grade structure, cannot be modified to accommodate a telemetry operated gate due to structural constraints and therefore will remain operational through the use of flash-board risers.

The purpose of the proposed new water control structure, S-112, within the Cypress Creek Canal is to improve management of water elevations within the canal during the wet and dry season. The system experiences periods of high discharge during wet periods or large rain events, causing scour and erosion both within the canal and downstream. Additionally, during drier periods, the canal continues to intercept adjacent groundwater seepage due to the canal's low bottom elevation. The proposed structure is a two-bay concrete ogee spillway with telemetry operated vertical lift gates. Each bay will be 16 ft wide with a crest elevation at approximately 6.5 ft NAVD88. This design will allow for flexibility during the wet and dry season. During the dry season, the gates can help hold additional water in the canal, assisting in the improvement of groundwater levels by reducing groundwater draw down. The structure will be designed and operated to control discharge velocities associated with wet season releases that may cause downstream erosion. The design will maintain the current flood protection for the surrounding developments.

Additional modifications to ensure flood protection is maintained include proposed berm improvements along the Cypress Creek Canal as well as modifications to portions of the Cypress Creek Canal cross sectional area. Portions of the Cypress Creek Canal that will be modified include the expansion of approximately 4,600 ft of canal from the south-eastern corner of Old Trail neighborhood (southern terminus of newly constructed HSLCD canal southern leg) to approximately 400 ft downstream of S-112. Channel cross section modification to contain a 30 foot bottom width with 2H:1V side slopes. Existing side slopes are 2H:1V and channel modifications should tie into existing channel as practical. Cross sections shall be expanded to a 35 ft bottom width approximately 400 ft upstream and downstream of the proposed S-112. Additionally, the upstream extents of Cypress Creek Canal adjacent to the Culpepper berm will be modified to contain a bottom width of approximately 15 ft and canal invert of 6 ft NAVD88.

A.8.3.2 Features of the Cypress Creek Canal

Cypress Creek Canal will have the features listed below. Additional detail regarding the hydraulic design of the various features for this project can be found in **Annex A-1**.

Canals
Cypress Creek Canal will be improved at the eastern end in order to accommodate S-112 gated spillway.

Culverts and Spillways
The S-112 spillway is a dual-bay ogee spillway that will help control water levels and discharge within the Cypress Creek Canal. S-115A, S-115B, and S-115C are culverts of varying size and discharge capacity that will be modified for telemetry operated gates.

A.8.4 Mack Dairy Road Spreader and Cypress Creek Improvements

The Mack Dairy Road Spreader and Cypress Creek Natural Area modifications include adding a pump station and spreader system within the Cypress Creek Natural area and targeted areas of re-grading.

A.8.4.1 General Description of the Mack Dairy Road Spreader Hydraulic Design

The capacity of the spreader swale pump station, S-113N, is designed to send up to 20 cfs from the Cypress Creek Canal into the spreader swale. The swale will extend for approximately 4,900 ft, with an average depth below grade of 2 ft, 5 ft bottom width, and 4H:1V side slopes. The dredged material will be placed along the western edge of the swale as a small berm. This will assist in promoting an easterly flow direction while helping prevent back flow toward the Mack Dairy Road. C-114 and berm will tie into the existing grade with the left channel bank being constructed to follow a uniform contour elevation, when practical. Final swale alignment and determination of Geofabric use to underlay the channel and protect the berm face and the downslope lip from erosion will be confirmed during PED. A plunge pool, will be required as a discharge bay from S-113N to disperse the flow into the spreader swale via a spreader weir, S-113S. The plunge pool dimensions will be 10 ft wide (parallel), 20 ft long (perpendicular to flow). Plunge pool will tie into embankment, with riprap protecting both the upstream and downstream face of the constructed embankment as well as within the transition from spreader weir to swale.

To further improve the hydroperiod and flow conditions in the Cypress Creek Natural Area, re-grading of the easterly forks of Cypress Creek are proposed. This is to help maintain lower flow velocities, reduce scour, and promote natural vegetative growth and improved ecological conditions in the area.

A.8.4.2 Features of the Mack Dairy Road Spreader

The Cypress Creek Natural Area will have the features listed below. Additional detail regarding the hydraulic design of the various features for this project can be found in **Annex A-1.**

Canals
The C-114 is the spreader swale located within the Cypress Creek Natural Area.

Culverts and Spillways
The S-113S structure is a concrete weir.

Pump Stations
The S-113N structure has a discharge capacity of 20 cfs.

Other Features
Re-grading of the downstream portions of the Cypress Creek Natural Area.

A.8.5 Gulfstream West Flow-Through Marsh

The Gulfstream West parcel was historically a citrus grove, but is currently being used for cattle grazing. The HSLCD drainage canal extends along the western perimeter of the property before turning east, cutting through a portion of the property. The Gulfstream West area has been excessively drained due to the drainage canals and ditches from agricultural use as well as the low stages within the Ranch Colony Canal, to which the HSLCD canal drains.

The Gulfstream West property is a flow-through marsh. This feature will provide multiple benefits: 1) the flow-through marsh will source water from the HSLCD drainage canal, thereby helping control discharge rates and attenuate flow; 2) the flow-through marsh will provide ecosystem benefits and potential water quality improvements; and 3) potential reduction in stages within the HSLCD Drainage Canal and Cypress Creek Canal as the flow-through marsh provides additional storage and can re-route water from the HSLCD Canal to downstream of the newly proposed structure (S-112) within the Cypress Creek Canal.

A.8.5.1 General Description of the Gulfstream West FTM Hydraulic Design

The Gulfstream West Flow-through marsh feature will pump water from the existing HSLCD Drainage Canal into a series of collection cells connected by spreader berms that will promote sheet flow and re-hydration of the site. The design will require extensive earthwork, including the construction of a levee, L-110, to ensure water is maintained and held on-site. The levee will be constructed to 20 ft NAVD88 with 3H:1V side slopes. The site will be graded and existing drainage ditches will be backfilled to provide a more uniform topography and slight gradient to promote flow in a southerly direction. Three spreader berms will be installed perpendicular to flow to will help reduce preferential flow path development, while controlling discharge velocities for ecological benefits. The runoff from HSLCD and Pal-Mar East is discharged through the HSLCD canal and may enter the flow through marsh at the northern end of the property via an inlet bay and a 150 cfs pump station, S-110. S-110 discharges into a discharge basin bay (plunge pool) with a discharge weir, S-111N, 50 ft in length at elevation 14.5 ft NAVD88. The average depths within the marsh will be approximately 3 ft and uncontrolled discharge occurs from the site through S-111S. S-111S is a concrete capped sheetpile weir with a variable discharge rate depending on the marsh depth. Discharges of approximately 30 cfs occur when the marsh depth is 2.75 ft, which discharge capabilities increasing as the depth increases due to multi-notch design. All discharge from the flow-through marsh enters Cypress Creek Canal downstream of S-112.

If runoff exceeds 100 cfs, the by-pass canal (existing HSLCD canal) is used to route the excess runoff directly to the Cypress Creek Canal. The “dog-leg” at the southern end of the existing HSLCD Canal is removed and the newly constructed portion of the canal is approximately 2,700 ft in length and will continue north-south, along the west side of the property, with an open connection at the Cypress Creek Canal. The newly constructed canal will contain a 20 ft bottom width with 2.5H:1V side slope with invert elevation varying from 3.3 ft NAVD88 to 2.0 ft NAVD88.

A.8.5.2 Features of the Gulfstream West FTM

The Gulfstream West flow-through marsh will have the features listed below. Additional detail regarding the hydraulic design of the various features for this project can be found in **Annex A-1**.

<u>Canals</u>
Cypress Creek Canal and HSLCD canal with newly constructed southern-leg.

Culverts and Spillways

The S-111N structure is a concrete weir, located at the discharge bay downstream of S-110 pump station. The S-111S structure is an uncontrolled notched weir spillway that serves as the main discharge from the Gulfstream West Flow-through Marsh.

Pump Stations

The S-110 structure has a discharge capacity of 150 cfs.

Other Features

The flow-through marsh will be contained by L-110, and will require site grading and construction of interior berms. Additionally, an inlet-bay and discharge bay for S-110 is required.

A.8.6 Nine Gems Property and Adjacent Areas

The Nine Gems property is a natural area composed of uplands and lowlands that is proposed to be restored through the redirection of sheet flow, controlling of water surface elevations, and planned re-grading. The following sections describe the hydraulic design needs for this area.

A.8.6.1 General Description of Nine Gems Hydraulic Design

Multiple features within the Nine Gems property are proposed for the purpose of improving hydroperiods and ecology within the area while providing hydrologic connection between the southern portions of Pal Mar East. The first component is to fill the three western most internal drainage canals within Pal-Mar East to help promote sheet-flow in an easterly direction across the Nine Gems property. Approximately 2,500 ft of canal length is proposed to be filled within each canal to a target elevation of the approximate ground surface in the area. Additionally, the small uncontrolled drainage pipes (PVC) are to be removed and backfilled to further reduce the drainage of the site. Berm improvements will be necessary at irregular intervals along the Nine Gems perimeter to ensure water is held on site during larger storm events. This levee, L-111, was designed at an elevation of 21.25 ft NAVD88 with a 12 ft crest width and side slopes that tie into the existing berm side slopes.

The Thomas pepper farm is located to the west of Pal-Mar East property and Pratt Whitney Road (SR-711). Currently, the farm drains to the south through a canal that bisects the Pal-Mar East property and discharges into the existing HSLCD canal and eventually to the northwest fork of the Loxahatchee River. This southern Nine-Gems Canal has dredged/berm material adjacent to both canal banks, causing a disruption in sheet flow from the southern portion of Pal-Mar East flowing in a northerly direction. Due to the topography, portions of Pal Mar East property, north of Culpepper, may have periods of over-inundation. The proposed design is to backfill and remove the drainage canal at three locations approximately 50 ft in length each. The backfill locations were defined in areas that have similar elevations north and south of the canal, and will provide for a natural transition of flow from the southern portion of Pal Mar East to the Nine Gems property. The existing berm material (on both sides of canal) is used to fill in existing canal to an approximate elevations of 19 ft NAVD88. Additionally, the L-111 will be extended to connect the existing Culpepper berm (constructed by Martin County) to the rest of L-111, across the existing southern Nine Gems canal.

The portion of the southern Nine Gems canal located north of the neighborhoods will be maintained for drainage of Nine Gems during storm events. S-114D is located at the eastern terminus of the canal and discharges into the HSLCD canal. The current structure is to be maintained but structure refinements may be necessary in PED once survey and design data is obtained.

Finally, four water control structures drain the Nine Gems property under exiting conditions. The project will remove the western-most structure and plug the existing north-south canal at the structure location. The three remaining structures will be removed and replaced with new structures. S-114A and S-114B are 72-in culverts, each with telemetry operated gates. S-114C will contain two 72-in culverts each with a telemetry operated gate. These structures will allow for improved operational control of water elevations for ecological improvement, while still allowing for conveyance offsite to the Cypress Creek Canal during storm events.

The new drainage pattern from the Thomas Pepper Farm is to drain along the northern border of Pal-Mar East, between Pal-Mar East and the HSLCD agricultural land. A pump station may be required in order to re-direct the flow within the Farm. Negotiations with the landowner and with HSLCD will be required to ensure adequate drainage for the Farm, and enough discharge capacity in the HSLCD ditch. The proposed pump will likely be located on the Farm's site and will be owned, operated and maintained by those entities.

A.8.6.2 Features of the Nine Gems Property

The Nine Gems area will have the features listed below. Additional detail regarding the hydraulic design of the various features for this project can be found in **Annex A-1**.

<u>Canals</u>
The eastern portion of the southern Nine Gems canal will be maintained.

<u>Culverts and Spillways</u>
S-114A through C are proposed culvert structures within the L-111. Additionally, S-114D is a culvert structure that helps control discharge from the southern Nine Gems canal into the HSLCD canal.

<u>Pump Stations</u>
The L-111 is proposed to help contain water surface elevations within the Nine Gems property.

<u>Other Features</u>
Partial canal backfill and degrading of an existing berm along the western

A.8.7 Eastern Project Features: Hobe Grove Weir and Kitching Creek

The Gulfstream East property is approximately 450 acres of fallow citrus grove located to the east of the Florida Turnpike. Historically, the run-off of this property would flow east to the Moonshine Creek and ultimately to the Northwest Fork of the Loxahatchee River. Restoration of the Gulfstream East and Moonshine Creek is proposed as part of the Recommended Plan.

Kitching Creek is the northern tributary, which discharges into the north-west fork of the Loxahatchee River. Currently, Jenkins Ditch discharges flow from the northern portion of Kitching Creek via the Kitching Creek Preserve. The ditch also helps convey local runoff from surrounding residential properties. The natural area of Kitching Creek, just south of the residential areas, receives discharge from the ditch. The ditch has caused higher flow rates and a less natural run-off pattern to occur than historically observed. A spreader swale and water control structure are proposed to aid in the rehydration of Kitching Creek.

A.8.7.1 General Description of Eastern Project Features Hydraulic Design

Currently, the Gulfstream East property elevations range from approximately 16 ft to 11 ft NAVD88, with multiple north-south irrigation canals. The existing drainage ditches will be filled using a material from onsite (normally through pushing adjacent higher bed material into the canals) and the site will be re-graded to mimic site conditions closer to the historical topography. To the east of the Gulfstream East

property is the Unit 3 control structure, which discharges into the Hobe Grove Ditch. Approximately 250 ft downstream of the Unit 3 structure the Hobe Grove Ditch and Moonshine Creek are partially separated due to heavy vegetation and sediment. The proposed feature is to connect the Hobe Grove Ditch and Moonshine Creek through clearing of the historic channel and excavate to 4 ft NAVD88 for a length of approximately 1,500 ft to ensure the appropriate hydraulic gradient is achieved for diversion of flow. Additionally, a new weir, S-117, will be installed at the eastern extent of the Hobe Grove Ditch to help hold additional water within the ditch, improving the groundwater levels while helping to promote additional flow down the historic Moonshine Creek. The proposed weir length is 40 ft at an elevation of 6 ft NAVD88.

The proposed Kitching Creek spreader swale is to be constructed to the east and west from Jenkins Ditch at the north end of the Jonathan Dickinson State Park. This swale will help distribute flows to historic Kitching Creek channels instead of directly down the ditch. This distribution is meant to mimic historical conditions by reducing peak discharge rates and creating a more natural flow pattern, aiding in the overall rehydration of the area. S-116 is a gated 5 ft by 7 ft box culvert to be constructed in the ditch upstream of the main Kitching Creek channel with an invert elevation of approximately 6.3 ft NAVD88. The culvert will replace an existing un-gated elliptical culvert to provide continued access to the State Park. The culvert will contain telemetry operated gates that will may be closed during normal operations to aid in the dispersion of water into the spreader system. During flood conditions the gates may be opened to ensure the same level of flood protection that is currently being maintained. The spreader canal, C-116, will be approximately 2,000 ft in length with a 5 ft bottom width, approximately 2 ft in depth with a 4H:1V side slope.

A.8.7.2 Features of the Eastern Project Area

The Eastern project area will have the features listed below. Additional detail regarding the hydraulic design of the various features for this project can be found in **Annex A-1.**

<u>Canals</u>
A spreader swale, C-116, will be constructed to help facilitate sheet flow and rehydration of Kitching Creek.

<u>Culverts and Spillways</u>
S-116 is a gated box culvert within Jenkins Ditch. S-117 is a non-gated weir.

<u>Other Features</u>
Proposed earthwork in the Gulfstream West property includes filling of agricultural ditches and re-grading to promote sheet flow. Additional re-grading and minor channel improvements within the historic Moonshine Creek will promote a redirection of flow from the Hobe Grove Ditch into the historic Moonshine Creek.

A.8.8 Flow-way 1 Features: S-100 Pump Station, G-160, and G-161

The following sections describe the hydraulic design needs for the recommended plan features within the Flow-way 1 project area.

A.8.8.1 General Description of Flow-way 1 Hydraulic Design

The S-100 pump station will deliver up to 75 cfs to the M-canal from Indian Trails Improvement District (ITID) lower M-1 Basin when canal stage conditions allow. 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. The inflow pumping location within the M-Canal will be located within the area of the M-Canal that has been widened.

The proposed 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 2,000 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.

The G-161 structure will act as the gateway through which water will be transported from Grassy Waters Preserve (GWP) through the system to the Northwest Fork of the Loxahatchee River. Water will flow from GWP into the northern GWP Triangle area, north of Northlake Boulevard. A conveyance channel will carry the flow through the GWP Triangle between the G-161 structure and an existing culvert beneath the Beeline highway that connects GWP to the C-18 Canal. The conveyance channel also transverses under an existing railroad bridge. Flow will then pass through G-160 to G-92 structure and on to the Northwest Fork. 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 a flow line elevation of 11.1 ft NGVD29 (9.6 ft NAVD88).

A.8.8.2 Features of Flow-way 1

The flow-way 1 project area will have the features listed below. Additional detail regarding the hydraulic design of the various features for this project can be found in **Annex A-1**.

<u>Canals</u>
The proposed features are located on existing canals, including the M-Canal, M-1 Canal, and C-18 Canal.

<u>Culverts and Spillways</u>
G-161 is a gated culvert. G-160 is a gated dual bay reinforced concrete spillway.

<u>Pump Stations</u>
S-100 is a 75 cfs pump station.

A.8.9 Hydraulic Design Data Sheets

Hydraulic design data sheets of features contained within the Recommended Plan can be found in **Annex A-1.**

A.9 Structural Design

The following sections contain structural design information. Typical plans and sections for standard designs are shown in the Engineering Appendix plates. These drawings are to scale and illustrate primary design features.

A.9.1 General

Subsections listed below describe design data and criteria that are common to all structures.

A.9.1.1 Design Criteria

Structural design criteria is governed by multiple Engineering Manuals published by USACE as well as manuals from the American Society of Civil Engineers (ASCE) and American Concrete Institute (ACI). All designs use the Load Resistance Factor Design (LRFD) methodology which assigned factors to increase the ultimate loading from dead and live loads experienced by the structure. More detailed descriptions of the loading conditions and design can be found below.

A.9.1.2 Loading Conditions

Each standard design will be developed to withstand usual, unusual, and extreme loading conditions. Usual loading conditions include loads most frequently experienced by the structural system when performing its primary function throughout its normal service life. Unusual loading conditions, such as construction or maintenance operations, produce short duration loads, and their occurrence is not frequent. Extreme conditions such as the standard project floods, and hurricane wind force represent the worst-case scenario; extreme loads represent the widest deviation from the usual and unusual loading conditions.

A.9.1.3 Stability

EM 1110-2-2100, *Stability Analysis of Concrete Structures*, will be followed for sliding, flotation, and rotational (limits on resultant location) stability analyses and to determine the limits on the bearing capacity of the foundation materials. A factor of safety is required in sliding stability analyses to provide a suitable margin of safety between the loads that can cause instability and the strength of the materials along potential failure planes that can be mobilized to prevent instability. A factor of safety is required for flotation to provide a suitable margin of safety between the loads that can cause instability and the weights of materials that resist flotation. The required factor of safety is a product of a basic factor of safety, a loading condition factor to account for load condition probability, a structure importance factor to account for the different risk levels accepted for critical and normal structures, and a site information factor to account for the knowledge of the structure and foundation strength parameters. Rotational behavior is evaluated by determining the location of the resultant of all applied forces with respect to the potential failure plane. The allowable bearing capacity value is defined as the maximum pressure that can be permitted on soil or rock giving consideration to all pertinent factors with adequate safety against rupture of the soil or rock mass, or movement of the foundation of such magnitude that the structure is impaired.

A.9.1.4 Concrete

EM 1110-2-2104, *Strength Design for Reinforced Concrete Hydraulic Structures*, and ACI-318-14, *Building Code Requirements for Structural Concrete*, will be used for structural concrete design. According to EM 1110-2-2104, loads and loading combinations are divided into Usual, Unusual, and Extreme categories based on the duration and frequency. The Usual and Unusual categories are designed to meet serviceability limit state requirements of the structure and the Extreme category meets the strength limit state. If a structure element has limited fluid pressures, ACI 318 load factors can be used. The compressive strength of concrete is designed to resist the factored loads on the hydraulic structure. Additionally, the use of granitic aggregates, permeability reducing admixtures and/or pozzolans, or epoxy coatings below the water surface elevations will be considered to reduce the potential for dissolution of carbonate aggregates.

A.9.1.5 Steel Sheet Pile

Stability and design analyses for steel piling will be in accordance with EM 1110-2-2504, Design of Sheet Pile Walls. Steel sheet pile walls will be analyzed for the usual, unusual and extreme loading conditions and the steel piling will be designed using the allowable stress method.

A.9.1.6 Dewatering

Two types of dewatering systems have been assumed for this project: steel sheet pile cofferdam with underwater concrete seal and open excavation with well points and sump pumps.

A.9.1.7 Louvers

All louvers will be the aluminum, weather-resistant type, vertical blade, with removable stainless steel bird/insect screens, and made to withstand a wind load of not less than that specified on the structural drawings. Wall louvers will bear the Air Movement and Control Association (AMCA) certified ratings program seal for air performance and water penetration in accordance with AMCA 500-D and AMCA 511. The pressure drop through the louver will not exceed 0.40 inches w.g. at 2,000 feet per minute. The louvers will have an active Florida Product Approval Number as required by the Florida Building Code and an active Miami-Dade County Notice of Acceptance Number. The louvers will meet the Florida Building Code Testing Standards (TAS) TAS 100(A), TAS 201, TAS 202, and TAS 203. Mounting hardware will be of stainless steel.

A.9.1.8 Pump Stations

Pump stations will be designed to EM 1110-2-3102 *General Principals of Pump Station Design and Layout*, EM 1110-2-3104 *Structural and Architectural Design of Pump Stations* and SFWMD Pumping Station Engineering Guidelines. Structural stability analysis, applied loadings, and concrete designs will be designed to EM 1110-2-2100 *Stability Analysis of Concrete Structures*, EM 1110-2-2502 *Retaining and Flood Walls*, ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*, ACI 318-14 *Building Code Requirements for Structural Concrete* and EM 1110-2-2104 *Strength Design for Reinforced Concrete Hydraulic Structures.*

A.9.1.9 Spillways

Spillways will be designed to EM 1110-2-2400 *Structural Design and Evaluation of Outlet Works*. Structural stability analysis, applied loadings, and concrete and steel designs will be designed to EM 1110-2-2100 *Stability Analysis of Concrete Structures*, EM 1110-2-2502 *Retaining and Flood Walls*, ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*, ACI 318-14 *Building Code Requirements for Structural Concrete* and EM 1110-2-2104 *Strength Design for Reinforced Concrete Hydraulic Structures* and ETL 1110-2-584 *Design of Hydraulic Steel Structures.*

A.9.1.10 Culverts

Culverts will be designed to EM 1110-2-2902 Conduits, Culverts, and Pipes. Structural stability analysis, applied loadings, and concrete designs will be designed to EM 1110-2-2100 Stability Analysis of Concrete Structures, EM 1110-2-2502 Retaining and Flood Walls, ASCE 7-10 Minimum Design Loads for Buildings and Other Structures, ACI 318-14 Building Code Requirements for Structural Concrete, EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures and FEMA Technical Manual, Conduits Through Embankment Dams. Culverts will consist of cast-in-place concrete box or precast reinforced concrete pipe (RCP) type.

A.9.1.11 Weirs

Steel sheet pile weirs will be designed in accordance with EM 1110-2-2504 Design of Sheet Pile Walls.

A.9.1.12 References

The following additional design manuals were used in the development of standard designs:

- ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*
- ACI 318-14 *Building Code Requirements for Structural Concrete*
- AASHTO *LRFD Bridge Design Specifications*
- EM 1110-2-2104 *Strength Design for Reinforced Concrete Hydraulic Structures*
- ETL 1110-2-584 *Design of Hydraulic Steel Structures*
- EM 1110-2-2502 *Retaining and Flood Walls*
- EM 1110-2-2504 *Design of Sheet Pile Walls*
- EM 1110-2-3102 *General Principles of Pump station Design and Layout*
- EM 1110-2-3104 *Structural and Architectural Design of Pump Stations*
- EM 1110-2-2400 *Structural Design and Evaluation of Outlet Works*
- EM 1110-2-2902 *Conduits, Culverts, and Pipes*
- EM 1110-2-2007 *Structural Design of Concrete Lined Flood Control Channel*
- EM 1110-2-2400, *Structural Design and Evaluation of Outlet Works*
- FEMA Technical Manual, *Conduits Through Embankment Dams (Sept 2005)*

Structural design of pump stations, culverts, spillways and weirs will be completed during the design phase. During design phase the structural calculations will be completed after survey, hydraulic design, and geotechnical investigations are performed. The structural design will conform to the appropriate engineering manuals (EM), engineering regulations (ER), or design criteria memoranda (DCM).

A.9.2 S-100 M-1 Pump Station

S-100 pump station **(Plates P-26 and P-27)** will serve as the inflow pump station to divert water from the ITID lower M-1 Basin to the M-Canal. Main intent is to help divert excess water from the M-1 Basin to the M-canal and through G-161 to the river. Pump will not be operated for flood control purposes. S-100 will have three electric 25 cfs submersible pumps for the main intake design capacity of 75 cfs. The service bridge, service platform, operating floor, and substructure will be reinforced concrete. The service bridge over each intake bay will be designed to current AASHTO LRFD Bridge Design Specifications. A mechanical trash rake will be mounted to the service bridge in each bay to remove large debris from blocking the intake. A control building by the structures will house all electrical components for the structure.

A.9.3 G-160 Spillway

G-160 is an existing reinforced concrete spillway that is being incorporated into the project. G-160 is 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). The spillway consists of two bays and discharges are controlled by two vertical lift gates. The design discharge rate, to maintain flood control capability, will be approximately 2000 cfs.

A.9.4 G-161 Gated Culvert

G-161 is an existing culvert that is being incorporated into the project. G-161 is a multi-purpose structure that will facilitate hydroperiod restoration of Grassy Waters Preserve, maintain existing level of service of flood protection, and provide improved conveyance for the regional water system. G-161 is a dual barrel culvert structure that is designed for 150 cfs discharge. G-161 has two 60” diameter concrete culverts with slide gates on the downstream end.

A.9.5 C-18W Reservoir

The C-18W reservoir is a 9,455 ac-ft aboveground reservoir on approximately 1,600 acres of a 1,920 acre former citrus grove. The reservoir will provide pumped diversion and storage of excess flows from the adjacent C-18W Canal, J.W. Corbett WMA, and from ITID upper basin via the M-O Canal. The reservoir will release water back to the C-18W Canal for delivery to Lainhart Dam and the Loxahatchee River downstream. The embankment height will be approximately 20.5 ft above natural ground elevation with a normal design pool depth of approximately 7.5 ft and a normal pool elevation of 28 ft NAVD88.

A.9.5.1 S-101A Inflow Pump Station

S-101A pump station **(Plates P-16, P-17, and P-18)** will serve as the inflow pump station to divert water from the C-18W canal to the C-18W reservoir. The main intent is to help divert excess water from the C-18W basin into the reservoir. Pump station will not be operated for flood control purposes. S-101A will have two electric 25 cfs pumps and two electric 50 cfs pumps for the main intake design capacity of 150 cfs. The building superstructure, operating floor, service platform and substructure will be reinforced

concrete with a bridge crane supported by the building columns running the length of the station in order to perform maintenance on the pumps. Crane size will be dependent on the weight of the pumps. A service bridge and maintenance bridge over each intake bay will be designed to current AASHTO LRFD Bridge Design Specifications. A mechanical trash rake will be mounted to the service bridge in each bay to remove large debris from blocking the intake. Intake sheet pile wingwalls will serve as a retaining wall and extend from the concrete structure to the limits of the embankment.

A.9.5.2 S-106 Inflow Pump Station

S-106 pump station **(Plates P-19, P-20, P-21 and P-22)** will serve as the inflow pump station to divert water from the western seepage collection canal to the C-18W reservoir. Main intent is to help divert excess water from the J.W. Corbett WMA and ITID basin (via the M-O canal pump station) into the reservoir. Pump will not be operated for flood control purposes. S-106 will have three electric 25 cfs pumps, one electric and one diesel 30 cfs pump and two electric 50 cfs pumps for the main intake design capacity of 235 cfs. The building superstructure, operating floor, service platform and substructure will be reinforced concrete with a bridge crane supported by the building columns running the length of the station in order to perform maintenance on the pumps. Crane size will be dependent on the weight of the pumps. A service bridge and maintenance bridge over each intake bay will be designed to current AASHTO LRFD Bridge Design Specifications. A mechanical trash rake will be mounted to the service bridge in each bay to remove large debris from blocking the intake. Intake sheet pile wing-walls will serve as a retaining walls and extend from the concrete structure to the limits of the embankment.

A.9.5.3 S-109 M-O Canal (C-101W) Pump Station

S-109 pump station **(Plates P-23, P-24 and P-25)** will serve as the inflow pump station to divert water from the M-O Canal to the newly proposed connector canal to the C-18W reservoir. Main intent is to help divert excess water from the Indian Trail Improvement District (ITID) to the C-18W reservoir. Pump will not be operated for flood control purposes. S-109 will have two electric 25 cfs submersible pumps and two electric 50 cfs submersible pumps for the main intake design capacity of 150 cfs. The service bridge, service platform, operating floor, and substructure will be reinforced concrete. The service bridge over each intake bay will be designed to current AASHTO LRFD Bridge Design Specifications. A mechanical trash rake will be mounted to the service bridge in each bay to remove large debris from blocking the intake. A control building by the structure will house all electrical components for the structure.

A.9.5.4 S-101B Spillway

S-101B **(Plate P-47)** is an auxiliary overflow spillway on the crest of the C-18W embankment. Spillway S-101B is located adjacent to reservoir discharge structure S-102 on the north embankment of the C-18W reservoir. The emergency spillway allows for passive release of water over the embankment in high water storm event equal to their respective adjacent reservoir discharge structure. Spillways will be reinforced concrete lower than the embankment crest and extend down the embankment. Riprap will extend outward from the toe of the embankment to provide erosion protection. Concrete on the embankment crest will be designed to accommodate vehicle traffic traversing the dam.

A.9.5.5 Culvert S-102

Culvert S-102 **(Plates P-35** and **P-36)** is a double barrel cast-in-place reinforced concrete box culvert structure located on the northern end of the C-18W reservoir. S-102 is the C-18W reservoir outflow

structure and is designed for a 300 cfs discharge. Culvert structure S-102 contains 2 – 6' (H) x 4' (W) box culverts with flow controlled by stainless steel vertical slide gates on the upstream side. The U-framed inlet and outlet structures will be made of reinforced concrete headwalls and retaining walls that run parallel to the flow and match the slope of the embankment. Recesses in the inlet structure will provide for dewatering stop logs or stop gates. The slide gate actuators will be mounted on top of the inlet headwall. Box culverts will be designed to meet the requirements for best practices for dam safety. As such, cast-in-place box culvert monoliths will be limited to 12 to 16 foot sections with control joint interfaces. Control joints will contain water-stops, bond breaker and continuous longitudinal reinforcing across all control joints. Culvert monoliths will be constructed in an alternating pattern. Exterior wall surfaces of the culvert monoliths and inlet and outlet headwalls will be constructed with a 10 vertical on 1 horizontal batter to ensure that the fill will be compressed against the wall as consolidation takes place. The earth fill will also be ramped against the conduit on a slope of 6H:1V to help force the earth fill against the conduit. Conduit will contain geotechnical seepage control measures. A control building by the structure will house all electrical components for the structure.

A.9.5.6 Culverts S-103E and S-103W

Culverts S-103E and S-103W **(Plates P-37** and **P-38)** are a single barrel cast-in-place reinforced concrete box culvert structures located on the northern end of the C-18W reservoir. S-103E and S-103W convey water from the C-101E seepage canal to the C-18W reservoir inflow/outflow canal while allowing vehicle access across the seepage canal to the C-18W reservoir service road. The culvert structures are designed for a 500 cfs discharge. The culvert structures contain 1 – 10 x 10 foot box culvert with flow controlled by stainless steel flap gates installed on the downstream side. The inlet and outlet structures will be made of reinforced concrete headwalls.

A.9.5.7 Culvert S-104

Culvert S-104 **(Plate P-15)** is a single-barrel cast-in-place reinforced concrete box culvert structure located on the southwestern end of the C-18W reservoir. S-104 conveys water in the C-101E seepage canal while allowing vehicle access across the seepage canal to the C-18W reservoir service road. The culvert structure is designed for 500 cfs discharge. The culvert structure contain 1 – 10 x 10 ft ungated box culvert. The inlet and outlet structures will be made of reinforced concrete headwalls. Refer to plate

A.9.5.8 Culvert S-105

Culvert S-105 **(Plates P-40** and **P-41)** is a triple-barrel precast RCP pipe culvert structure that is positioned under Seminole Pratt Whitney Road and located outside the southwestern side of the C-18W reservoir. S-105 conveys water from M-O Connector Canal to the C-18W reservoir seepage canal. The culvert structure is designed for 150 cfs discharge. S-105 has three 78-in diameter RCP culverts with flap gates on the downstream end. The structure inlet and outlet structures will be made of reinforced concrete headwalls and retaining walls. Downstream third of culverts will contain seepage control measures.

A.9.5.9 Culvert S-107

Culvert S-107 **(Plate P-42** and **P-43)** is a triple-barrel precast reinforced concrete pipe (RCP) culvert structure that is positioned under Seminole Pratt Whitney Road. S-107 conveys and controls inflow water from the J.W. Corbett WMA to the C-101W seepage canal. The culvert structure is designed for 300 cfs discharge. S-107 has three 72-in diameter RCP culverts with flap gates on the downstream end. The upstream end of S-107 will consist of an intake drop pool controlled by stainless steel weir gates. The

rectangular drop pool inlet structure and the U-framed outlet structure will be made of reinforced concrete headwalls and retaining walls. The weir gate actuators will be mounted on top of the inlet headwall. Downstream third of culverts will contain seepage control measures. A control building by the structure will house all electrical components for the structure.

A.9.5.10 Palm Beach County Pump Station Replacement

There is an existing pump station owned by Palm Beach County, which serves the Palm Beach County water treatment plant located adjacent to the southeastern corner of the proposed C-18W reservoir. An existing pump station conveys water from the water treatment plant, into an adjacent SFWMD-owned wetland via a County-owned ditch. The pump station is located within the proposed C-18W reservoir footprint. This pump station will be demolished and replaced. The new pump station will be located outside the northeastern corner of the C-18W reservoir, and will discharge via the existing County-owned ditch into an existing wetland to the northeast of the C-18W reservoir on lands owned in fee by SFWMD. This pump station will be relocated as a replacement and need not conform to USACE design standards. The location of the C-18W reservoir embankment will not impact the Palm Beach County utility corridor or the existing County owned ditch.

A.9.5.11 Stilling Wells

Stilling wells will be designed in accordance with current SFWMD standards. Refer to plate **P-55.**

A.9.6 Cypress Creek Canal Modifications

The purpose of the modifications within the Cypress Creek Canal area are multi-purpose and, as stated in the previous section, will help in improving the ecological conditions through re-hydration, while providing for greater operational flexibility within the Pal-Mar East (Culpepper) area as well within the Cypress Creek Canal.

A.9.6.1 S-112 Gated Spillway on Cypress Creek Canal

S-112 **(Plates P-33** and **P-34)** is an ogee weir spillway located on the Cypress Creek Canal along the southeastern end of the Gulfstream West Flow-Through Marsh. This spillway is designed for a 2,150 cfs discharge. S-112 will have two 16-ft wide bays each with a lift gate that rests on the crest of the ogee weir. Downstream of the weir, concrete hydraulic jumps on the concrete surface provide energy dissipation. The foundation, weir, and superstructure of the spillways will be reinforced concrete in accordance with current design standards. A 16-ft wide service bridge will provide vehicle access across the structure and Cypress Creek. Sheet pile wing-walls will serve as retaining walls and extend from the concrete structure to the limits of the embankment.

A.9.6.2 Culverts S-115A, S-115B and S-115C Culpepper Outflow Culverts

Culverts S-115A, S-115B and S-115C **(Plates P-56** and **P-57)** drain from Culpepper area into the North-South leg of the Cypress Creek Canal. These are existing structures that are currently known as WS-1, WS-2 and WS-3 are being renamed to S-115A, S-115C and S-115B, respectively. S-115A, S-115B and S-115C will be upgraded to include steel platform grating, encasing existing steel sheet pile walls and adding weir gates on the inlet side of the structures. The weir gate actuators will be mounted on top of the modified inlet headwall. A control building by the structures will house all electrical components for the structure.

A.9.7 Mack Dairy Road Spreader

The purpose of the Mack Dairy Road spreader is to assist in distributing water in a southerly and easterly direction to rehydrate the Cypress Creek Natural Area. This design for the distribution of water was to mimic historical flow patterns from the west (Culpepper area) that have been interrupted due to urban development.

A.9.7.1 S-113N Pump Station

S-113N pump station **(Plates P-28 and P-29)** will serve as the inflow pump station to divert water from the Ranch Colony Canal to a spreader swale within the Cypress Creek Natural Area (CCNA). Main intent is to help rehydrate the CCNA when excess water is available within the Cypress Creek Canal. Pump will not be operated for flood control purposes. S-113N pump station will have two electric 10 cfs pumps for the main intake design capacity of 20 cfs. Pumps will be submerged and supported by a concrete platform. A service bridge over each intake bay will be designed to current AASHTO LRFD Bridge Design Specifications. A steel sheet pile wall with concrete cap supports the platform and extends outward from the pump station to form retaining walls along the embankment. Intake bays will each have a trash rack to block large debris from entering the pumps. A concrete retaining wall structure will be constructed on the downstream end of the pump station to serve as a plunge pool. A control building by the structure will house all electrical components for the structure.

A.9.7.2 S-113S Mack Dairy Spreader Weir

S-113S **(Plate P-49A)** is a non-gated broad crested weir structure located approximately 400 ft east of Mack Dairy Road in Martin County. The discharge basin plunge pool for pump station S-113N along with the S-113S spreader weir will convey dispersion flow into spreader swale. The structure contains a single notch with a crest elevation of EL 16.50 along the weir. The weir will be composed of a steel sheet pile system with a concrete cap along the rectangular notch in the center of the weir.

A.9.7.3 Stilling Wells

Stilling wells will be designed in accordance with current SFWMD standards. Refer to plate **P-55.**

A.9.7.4 Communications Facility

A project communications facility will be located near the Mack Dairy Road spreader **(Plates P-51 and P-52)**. The facility will provide a connection to the SFWMD backbone microwave communications system and will serve as a local communications hub for the project. The communications building facility will be a precast concrete building that contains electrical equipment for the communications facility, a backup liquid petroleum gas (LPG) generator, and a self-supported communications tower. The communications building and tower will be based on SFWMD standard designs adapted to the site. The tower and antenna heights will be determined by a path study from SFWMD.

A.9.8 Gulfstream West Flow-Through Marsh

The purpose of the Gulfstream West flow-through marsh are 1) to divert source water from the HSLCD drainage canal into the marsh, thereby helping control discharge rates and attenuate flow; 2) to provide ecosystem benefits and water quality improvements; and 3) to reduce stages within the HSLCD Drainage

Canal and Cypress Creek Canal. The flow-through marsh provides additional storage, and can re-route water from the HSLCD Canal to downstream of the newly proposed structure within Cypress Creek Canal.

A.9.8.1 S-110 Pump Station

S-110 pump station plates **(P-30, P-31, and P-32)** will serve as the inflow pump station to divert water from the Hobe St. Lucie Control District (HSLCD) Canal to a series of spreader swales within the Gulfstream West property. Main intent is to help rehydrate the Gulfstream West area and aid in attenuating peak flows from HSLCD through the formation of a shallow flow-through marsh. Pumps will not be operated for flood control purposes. S-109 will have two electric 25 cfs submersible pumps and two electric 50 cfs submersible pumps for the main intake design capacity of 150 cfs. The service bridge, service platform, operating floor, and substructure will be reinforced concrete. The service bridge over each intake bay will be designed to current AASHTO LRFD Bridge Design Specifications. A mechanical trash rake will be mounted to the service bridge in each bay to remove large debris from blocking the intake. A concrete retaining wall structure will be constructed on the downstream end of the pump station to serve as a plunge pool. A control building by the structure will house all electrical components for the structure.

A.9.8.2 S-111N Weir

S-111N **(Plate P-50)** is a non-gated broad crested weir structure located on the northern tip of the Gulfstream West Flow-through Marsh in Martin County. The discharge basin plunge pool for pump station S-110 along with the S-111N spreader weir will covey dispersion flow into spreader swale. The structure contains a single notch with a crest elevation of EL 14.50 along the weir. The weir will be composed of a steel sheet pile system with a concrete cap along the rectangular notch in the center of the weir.

A.9.8.3 S-111S Weir

S-111S is a non-gated weir structure located at the south-east corner of the Gulfstream West Flow-through Marsh in Martin County. S-111S will maintain stages within the Gulfstream West Flow-through Marsh for ecosystem restoration. The structure contains three notches with crest elevations at EL 15.25, EL 16.0 and EL 17.5 along the weir. The weir will be composed of a steel sheet pile system with a concrete cap along the rectangular notches. Refer to **Plate P-48**.

A.9.8.4 Stilling Wells

Stilling wells will be designed in accordance with current SFWMD standards. Refer to **Plate P-55.**

A.9.9 Nine Gems Property

The main purpose of the features proposed within the Nine Gems property is to help reduce over drainage of the site, achieve rehydration and improve the hydroperiods and ecology of the area, as well as provide connectivity of Pal Mar East and the Nine Gems property.

A.9.9.1 Outflow Culverts S-114A, S-11B, and S-114C

Culverts S-114A, S-114B, and S-115C **(Plates P-44, P-45, P-46** and **P-46A)** are existing structures that are located along the Nine Gems Drainage Canal. The existing culverts structures will be replaced with gated culverts. S-114A and S-114B culvert structures are designed for 105 cfs discharge. S-114C culvert structure is designed for 240 cfs discharge. S-114A and S-114B have one 72-in diameter RCP culvert with

slide gates on the upstream end. S-114C has 2-72-in diameter RCP culverts with slide gates on the upstream end. The U-framed inlet and outlet structures will be made of reinforced concrete headwalls and retaining walls. The slide gate actuators will be mounted on top of the inlet headwall. Downstream third of culverts will contain seepage control measures. A control building by the structures will house all electrical components for the structure.

A.9.9.2 Outflow Culverts S-114D and S-114E

Culverts S-114D and S-114E are existing culverts structures that are being incorporated into the project. No changes or modifications are currently being proposed for these culverts structures.

A.9.10 Eastern Project Features: Hobe Grove Weir and Kitching Creek Spreader

The main purpose of the features proposed Gulfstream East and Kitching Creek are to improve the rehydration and ecology of these properties through re-grading and structure placement. Additionally, these features will help in the timing and distribution of flows to the Northwest Fork of the Loxahatchee River.

A.9.10.1 Culvert S-116

Culvert S-116 **(Plates P-44** and **P-45)** is a single barrel culvert structure located in Jenkins Ditch. The culvert structure conveys flow from the Jenkins Ditch through a spreader system to rehydrate wetlands of Kitching Creek. S-116 culvert structure is designed for 150 cfs. S-116 contains one 5 ft by 7 ft box culvert with flow controlled by stainless steel vertical slide gates on the upstream side. The U-framed inlet and outlet structures will be made of reinforced concrete headwalls and retaining walls that run parallel to the flow and match the slope of the embankment. Recesses in the inlet structure will be provide for dewatering stop logs or stop gates. The slide gate actuators will be mounted on top of the inlet headwall. Downstream third of culverts will contain seepage control measures. A control building by the structure will house all electrical components for the structure.

A.9.10.2 S-117 Hobe Grove Ditch Weir

S-117 **(Plate P-49)** is a non-gated broad crested weir structure located on the eastern end of Hobe Grove Ditch. The weir helps divert discharge through historic Moonshine Creek while helping to control discharges into the northwest fork of the Loxahatchee River. The structure contains a single notch with a crest elevation of EL 6.00 along the weir. The weir will be composed of a steel sheet pile system with a concrete cap along the rectangular notch in the center of the weir.

A.9.10.3 Pedestrian Bridge

Bridge over Moonshine Creek at the Ocean to Lake Trail **(Plate P-53)** will be designed in accordance with AASHTO Guide Specification for Design of Pedestrian Bridges and will be designed to accommodate equestrian traffic.

A.10 Mechanical and Electrical Design

The LRWRP incorporates inflow pump stations on the M-1 Basin (S-100), M-O Canal Connector (S-110), Mack Dairy Road spreader (S-113N), Gulfstream West (S-109), an inflow pump station on the C-18W reservoir (S-101), and a combination inflow/seepage pump station on the C-18W reservoir (S-106). All of the mentioned pump stations are not designed or intended for flood control. The recommended plan also includes a nominal 20 million gallons per day (MGD) Aquifer Storage and Recovery (ASR) System. The system will be located at the C-18W reservoir site.

The proposed project includes either modifying or designing gated culvert structures. The existing culverts to be modified are Cypress Creek Canal culverts (S-115A, -B, and –C) and G-161. The culvert structures that require design are C-18W reservoir, J.W. Corbett WMA and Seminole-Pratt Whitney Road. The proposed project includes either modifying or designing gated culvert structures. The existing culverts to be modified are Cypress Creek Canal culvert and G-161. The culvert structures that require design are C-18W reservoir, J.W. Corbett WMA and Seminole-Pratt Whitney Road. The appropriate plates for each feature are cited, and all plates can be found on in **Annex C-1** Plates.

A.10.1 General

The following components of pump stations are common to the project. Pump stations will be designed in accordance with (but not limited to) the following:

- EM 1110-2-3102 General Principles of Pump station Design and Layout
- EM 1110-2-3105 Mechanical and Electrical Design of Pump Stations
- Hydraulic Institute (HI) Standards
- SFWMD's Major Pump Station Engineering Guidelines, 4th edition, (MPSEG)
- CERP Standard Design Manual

Although the capacity of these station are low enough that SFWMD's Major Pumping Station Engineering Guidelines is not applicable due to pump station sizes, pumping systems in which some of the features or components referenced in the manual are used shall have those items designed in accordance with the manual.

A.10.1.1 Pumping Station Mechanical Features

<u>Inlet Bays</u>
The inlet bays will serve as the approach for the pump intake, and a location for the trash rack. The depths of the supply canals will be determined by considering water surface elevation in the supply canal, minimum required submergence over the pump intakes, and minimum vertical clearance between the pump intakes and the floor of the sump. Minimum submergence for the pumps will be determined by using Hydraulic Institute Standard 9.8 and 2.3 (HI 9.8 & 2.3) and EM 1110-2-3105, Mechanical and Electrical Design of Pump Stations, for a suction bell.

<u>Discharge Arrangement</u>
The discharge for the inflow pump station will be located in the embankment with discharge pipes, air vent valves, and a flap gate. Two means of backflow prevention are necessary from the discharge basin back to the inlet bay. The invert of the discharge pipes for the inflow pumps will be set at an elevation 1-ft higher than the maximum tailwater level in the discharge basin. For the second means of backflow

prevention, the discharge piping for the inflow pump station will incorporate flap gates. Each inflow pump discharge pipe will have a vent-pipe for air to escape during filling.

Discharge Basin

Riprap or concrete apron will be provided downstream of the discharge flap gates for erosion protection.

Dewatering System

Bulkhead gates will be provided and can be installed in the inlet bays (bulkhead slots) so the pump suction area from just in front of the trash racks can be dewatered. Dewatering will be performed using portable pumps.

Start-up

Large electric motors will have soft-start capability to ease power demand.

Inflow Pumps

The minimum pumping requirements provided by Jacksonville District (SAJ), manufacturer pump curves, and pumping system loss calculations to determine total dynamic head (TDH) will be used to determine required pump performance during PED.

The pumps will be vertical wet pit pumps with stainless steel impellers, shafts, shaft enclosing tubes, impeller bowls, and diffusers. Pump discharge head and column will be constructed of carbon steel. The pumps will be equipped with water-lubricated bearings lubricated with product (canal) water. The inflow pumps will be driven by large electric motors with in-line speed reducers.

Electric Motors

The inflow pumps are intended to be driven by electric motors, based on utility availability, but a value engineering study will be performed during PED to finalize the power supply. Electric motors, diesel, and natural gas will be analyzed. The required motor horsepower rating will be derived by examining the horsepower requirements when operating in the required operating range from the minimum static head (and corresponding minimum total dynamic head (TDH)), through the design point (design point static head and TDH), to the maximum static head and TDH in the priming state. For electric motor drives, the minimum horsepower rating required will be determined during PED.

Trash Rack/Rake

The pumps will be protected with trash racks to capture intake channel debris in each pump suction channel. The pump station will be provided with a trash rake for removal of the captured debris. A belt conveyor will be provided for the collection and removal of trash and vegetation debris from the intake channel flow and transport the trash to a disposal pit.

Bridge Crane

As per EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations, subsection 9.6, “Equipment Handling,” the crane capacity should be large enough to lift the completely assembled motor or pump, but not both at the same time. The crane/hoist shall be capable of lifting and moving loads up to the heaviest loads in the pumping station. The crane/hoist will handle pumping station equipment such as the diesel engine pump drives, reduction gears, and the pumps during initial installation as well as for general service thereafter. Based on weights submitted by various manufacturers, the size of the bridge crane will be determined during PED.

The clearance from finished floor to the bottom of the crane beam will be calculated. This clearance will not be sufficient to remove the whole pump assembly in one lift. This clearance calculation will allow the removal of a 12-foot long component assuming a 5-foot spreader beam/rigging assembly is used and the component is going to be lifted to a 4.5-foot tall flatbed truck. An additional 3-foot clearance will be added for maneuverability.

As per SFWMD MPSEG, the crane will be a top running, double girder bridge crane. The crane service shall be Class 1-C in accordance with the Crane Manufacturers Association of America. The design of the overhead crane shall be in accordance with ANSI B 30.2, Overhead and Gantry Cranes Top Running Bridge, Single or Multiple Girder Top Running Trolley Hoist. The bridge will be motor driven and travel the entire length of the station. The travel will have two speeds plus inching control to permit close positioning of the loads. The travel system will have an automatic brake system that is applied when bridge movement ceases or power is lost. The trolley will also have power driven travel and automatic brakes. The hoist mechanism will have two speeds plus inching control with an automatic brake system capable of safely supporting the design load.

Heating, Ventilation, and Air Conditioning (HVAC)

Site criteria for design of the ventilation systems will be as stipulated herein and based on ASHRAE climatic design information for West Palm Beach/IN, Florida (WMO#: 722030). The following design temperatures and ventilation rate criteria were used for system design parameters.

Outside design conditions for cooling are: 91.4° F dry bulb (ASHRAE, 2013 0.4% percent condition). Outside design conditions for heating are: 43.9° F dry bulb (ASHRAE 2019 99.6 percent condition). The ventilation system will be designed to achieve a design temperature inside the control room no more than 10ºF above the ASHRAE 0.4% cooling dry bulb design temperature.

The system will be a pull design utilizing an intake hood or intake lover on the wall and an exhaust fan located on the opposite wall located high above the floor to pull the heated air out. Exhaust fans will have missile barriers or louvers equipped with backdraft damper and bird screen. Inside design conditions are shown in **Table A-23.**

Table A-23. Inside design conditions for HVAC design.

Area	Heating	Cooling
Control Room, Break Room, Communications Room	68 degrees F	70 degrees F, 50% RH
Locker Room	68 degrees F	75 degrees F
Operating Room, Workshop	40 degrees F	101 degrees F

As per SFWMD Major Pumping Station Engineering Guidelines 4th Edition, the operator control/break room and communications room shall be mechanically cooled with a design temperature as shown above, and shall include humidity control to keep monitoring equipment cool and dry. The rooms shall be served by dedicated, commercial DX split systems, with air handlers inside and condensing units located outside the control building. The communications room shall incorporate a redundant system that will alternate

cooling/heating (lead/lag) between systems and act as a back-up in the case of one system failing. The workshop shall be (force) ventilated.

The humidity will be controlled with the unit's cooling coil during cooling season. The leaving coil temperature should be around 52˚F to 54˚F to achieve dehumidification and maintain the space between 50% to 55% relative humidity. Internal loads at the control building are shown in **Table A-24.**

Table A-24. Miscellaneous loads at the control building.

Room	Load	Remarks
Control Center, Upper Level	150W X 3	Computers, ASHRAE Fundamentals 2005
Telecom, Upper Level	2000W	Servers

The insulation values for the building envelope will be per the International Energy Conservation Code as seen in **Table A-25**.

Table A-25. Insulation values for the building envelope.

Location	Insulation Value (R)	Overall Heat Transfer Coefficient (U)
Roof	R-20	U-0.048
Walls Above Grade	R-5.7	U-0.142
Roll Up Doors	R-4.75	U-0.210

Ventilation

Ventilation will be provided in accordance with EM 1110-2-3102, General Principles of Pumping Station Design and Layout, SFWMD MPSEG, the Florida Building Code, and ANSI/ASHRAE Standard 62.1-2007.

Sump Ventilation

Open grating will be provided at the sump access points for ventilation considerations to comply with EM 1110-2-3102, *General Principles of Pumping Station Design and Layout, subsection 9-4*.

Pipe Gallery Ventilation

The pipe gallery will be ventilated at a rate of 1.0 cfm/ft^2, similar to an enclosed car garage application (0.75 cfm/ft^2), as stated on the International Mechanical Code, subsection 403.3, "Outdoor Airflow Rates."

Pump Operating Room Ventilation

Ventilation considerations as per SFWMD Major Pumping Station Engineering Guidelines 4th Edition:

- The ventilation system will be designed to achieve an operating floor design temperature inside the station no more than 10˚F above the ASHRAE 1% cooling dry bulb design temperature data from the climate station located closest to the project site.

- The ventilation design will be sized to have a minimum of 400 feet per minute of air velocity forced across the station.
- The system will be a push-pull design utilizing a set of filtered fans located on the floor in an enclosed building extension on the opposite side of the building from the engines to push air into the building; and fans on the opposite wall located high above the floor on the engine side to pull the heated air out.
- The fans will be equally spaced across the station to properly distribute the intake airflow.
- Fans that are wall-mounted will have louvers or missile barriers to withstand design wind forces and debris impact as required by Paragraph 2.7 and the local building code.
- Intake/supply fan louvers will be protected by drawing air through the floor.
- Exhaust fans will have missile barriers or louvers equipped with bug screens.

Operating floor will maintain a slight positive building pressure to prevent gases from entering the building during operations. Supply and Exhaust fans will be equipped with motor-operated dampers.

Plumbing

The water supply system will be designed using the Florida Building Code-Plumbing (2017), the International Plumbing Code (2017) and the American Society of Plumbing Engineers (ASPE) Plumbing Engineering Design Handbook, Vol. 2: Plumbing Systems.

Piping

Piping will be installed from the well to all fixtures requiring potable water within the facility. Underground water supply piping will be corrosion resistant material (CPVC, PVC, etc.) outside of the building footprint, and type K copper within the building footprint. Above ground water supply piping will be wither PVC, CPVC, or type L copper, depending on where it is located and its exposure, i.e. susceptibility to damage from people or the elements.

Additional piping will be installed to provide non-potable (well) water to hose bibbs around the station. The piping will be sized inside the building from the connection point outside the building. The contractor will be responsible to size the water line from the building connection point to the well connection point based on the location and depth of the well. Design Criteria: Fluid velocity within the piping will be kept below 5 fps as per FBC, IPC and ASPE guidelines.

Potable Water

A well will be drilled on the property to supply the water to the facility as shown on the site layout. This may require obtaining a permit as well as conducting water quality tests (contractor's responsibility). Subsequent to this, verification of the well capacity will be done (contractor's responsibility). Based on the results of the water quality tests, the contractor shall size and select the appropriate water purification system for the potable water system.

An eyewash station will be installed in front of the control building in front of the control/break room. Electric, chilled-water drinking fountains (including a bottle filler type) will be provided in the control/break room.

Non-Potable Water

Water supplied to the hose bibbs in the perimeter of the building for building cleaning operations will be untreated well water. Hose bibbs will be installed in locations as required/preferred by SFWMD. Typical installations include four hose bibbs provided within the pump operating room (one at each corner of the building) and three hose bibbs on the building perimeter (two bibbs located at the service bridge (trash rake side) and one provided at the back side of the building). Hose bibbs will also be provided inside the pipe gallery.

Domestic Hot Water

Potable water will be supplied to an electric-powered instantaneous water heater. The electric water heater will be sized according to the Department of Energy equipment sizing guidelines. The guidelines recommend a water demand of 2 gpm when selecting heating equipment to achieve 120° F at the fixture. Based on the hot water demand present for the building, the size will be adequate for the application. Any fixture used will reach the desired temperature with the equipment. If additional load is requested from the equipment the temperature will be distributed between both fixtures.

Sanitary Systems

Sanitary drainage and vent systems will be installed on all fixtures and equipment requiring drainage. Sanitary drainage from the building will be collected in a septic tank and effluent discharged to a drainfield.

Toilet facilities will be provided to meet the code requirements for the population in the pump station facility. Accessible facilities will be provided in accordance with the requirements of the ADA.

Water Closets will be wall hung vitreous china with flush valves. Wall hung, vitreous china urinals with flush valves will be provided in quantities required by code. Lavatories will be provided with faucets using tempered water. All lavatories will be ADA accessible.

Floor drains will be located in the restrooms. Also, floor drains will be installed in the pump operations room in locations outside of any fuel containment area. Locations for drains include condensate drain needs, near equipment that discharges water, and areas for station floor wash-down. The area around the generators, pump engines, and day tanks will be provided with a containment curbing, and there will be no floor drains will be located in these areas.

Fire Protection

A fire detection system will be provided for the entire facility. Fire suppression in the pump bays will not be required as the aggregate fuel storage is expected to stay below the 1,320-gallon maximum allowed in Section 6.3.2.3 of National Fire Protection Association (NFPA) 37. The SFWMD will review all design assumptions, criteria, and calculations. Verification with the SFWMD and the SFWMD insurance underwriter will be done for the fire protection system type. Portable fire extinguishers will be installed in accordance with NFPA 10. Dry chemical (ABC) extinguishers will be used in the operating floor area, and in the control and electrical rooms. Locations of extinguishers will be as shown on the drawings, and detailed as necessary for constructability.

Liquefied Petroleum Gas (LPG) Generator Sets

An LPG engine-driven generator will be required. These generators must provide sufficient power to operate all pre-determined instrumentation and monitoring equipment, continuously for as long as seven (7) days. Each generator will most typically be rated for 208/120 volts, 3 phase, and engine speeds not to exceed a maximum of 1,800 rpm. The engines will be 2 or 4 cycle with an electric starting system. Cooling

of the engine will be by means of a closed system consisting of keel coolers, overhead expansion tanks, and engine-driven jacket water and after-cooler water circulating pumps with proper heat balance maintained by the thermostatically controlled proportioning valves. The main lubricating oil pump for the engine will be driven directly by the engine.

A.10.1.2 Pump Station Mechanical Equipment

Alternate Pump Drive

Depending on lack of available/reliable electrical service in the area, the inflow pumps may be driven by diesel engines. The engines will be rated similarly to the electric motors as described above. The features for the diesel engines and associated auxiliary system are described below.

Diesel Engine Drives

The diesel engine will be a standard model full-diesel type, two or four cycle, EPA Tier 4 compliant with mechanical injection, exhaust after-treatment system, and cooling provided by keel coolers. Cooling of the engine will be by means of a closed system consisting of keel coolers, overhead expansion tanks, and engine-driven jacket water and after-cooler water circulating pumps with proper heat balance maintained by the thermostatically controlled proportioning valves. The main lubricating oil pump for the engine will be driven directly by the engine.

Speed Reduction Gear

Right-angle gear speed reducers will be provided for transferring power from the motor drives to the inflow pumps. Gear reducers will be designed in accordance with American Gear Manufacturers Association (AGMA) 11 with a minimum service factor of 2. Bearings will be rated for an L-10 life of 100,000 hours. The gear reducers will be equipped with a self-contained forced oil-lubrication system, including a primary mechanically driven oil pump, an auxiliary, electrically driven oil pump, and an oil cooler. The oil coolers will reject heat to the atmosphere. These oil coolers will be designed in accordance with the Heat Exchanger Institute (HEI) standards. All auxiliaries will be mounted directly to the gear reducers. The gear reducers will include anti-reverse rotation devices. The minimum transmission efficiency will be 97 percent. A minimum tooth hardness of Rockwell 58C will be required. Connection between reducer and engine will be by flexible coupling to compensate for misalignment and vibration or shock transmission. To prevent reverse rotation, the transmission would be fitted with an anti-reverse rotation clutch.

Fuel Oil Storage System and Supply

Above-ground Storage Tanks (ASTs) will be located at a safe distance from the station. ASTs shall be concrete-vaulted and have a dual containment feature. Multiple tanks may share the total capacity for the station. Fuel capacity should be for seven days, 24-hour/day continuous operation at maximum fuel consumption rate. The tanks will be filled from truck transport by means of lines extending to an unloading pump pit. The tanks will be connected to the station supply header. All fuel lines will consist of double-wall pipes. All fuel tanks and fuel pipes will be monitored for leaks. The fuel system for the engine will consist of a day tank to supply the diesel engine. The day tank will have automatic operation in sending and receiving fuel and controlling the level of the fuel inside of the day tank. A similar day tank will be provided for the engine generator set. Day tanks shall be provided and sized to supply adequately the diesel engine pump drives and generator within the limitations of NFPA 37.

Lube Oil System

ASTs will be located at a safe distance from the station. The lube oil system will consist of a new lube oil storage tank, a waste lube oil storage tank and a waste lube oil collection tank. The new and waste oil storage tanks will be concrete vaulted tanks and have a dual containment feature. Multiple tanks may share the total capacity for the station. Pumps will be provided for loading and unloading lube oil from delivery trucks. The new lube oil storage tank will be provided with a pump for sending oil to the engines. During an oil change, the oil from the engines will gravity drain into the collection tank. The tank is large enough to accommodate one oil change from all the engines. The oil is then pumped by a pump to the waste oil storage tank.

Exhaust After-Treatment System

The EPA Tier 4 regulations purpose is to reduce two primary exhaust pollutants, which are particulate matter and nitrates of oxygen or NOx. The after-treatment may consist on some or all of the following:

- exhaust gas recirculation (EGR)
- diesel particulate filter (DPF)
- diesel oxidation catalysts (DOC)
- selective catalytic reduction (SCR)

SCRs requires the use of diesel exhaust fluid (DEF). The DEF are stored in Underground Storage Tanks (USTs) and will be located at a safe distance from the station. USTs shall be high density polyethylene (HDPE), 304 or 316 stainless steel, or a combination of the two and also have a dual containment feature. Multiple tanks may share the total capacity for the station. DEF capacity should be for seven days, 24-hour/day continuous operation at maximum fuel consumption rate. The tanks will be filled from truck transport by means of lines extending to an unloading pump pit. The tanks will be connected to the station supply header. All fuel lines will consist of double-wall pipes. All DEF tanks and pipes will be monitored for leaks. The SCR will include a day tank to supply the DEF to the diesel engine. The SCR will be part of the diesel engine package.

Combustion and Cooling Air Systems

Combustion air will be taken from the inside of the pump station. This will reduce the noise outside of the pump station. Intake air-fans will be started each time a pump diesel engine or a back-up generator is started. These fans will provide combustion and cooling air for the engines and cooling air for the motors. Exhaust air-fans will be started each time a diesel engine or a back-up generator is started, to remove engine/motor heat from the pump station. The intake and exhaust fans will also start by manual switch and by thermostat. Each wall intake and exhaust fan opening will be provided with a hurricane rated louver (or hood), bird screen and motorized damper. Ventilation air intakes shall also incorporate roll filters. The intake and exhaust fan capacities will be sized to provide the building with a slight positive pressure.

A.10.1.3 Pump Station Electrical Features

Electric service will be coordinated with the local utility. The large electric driven pump station will require a significant utility power feed. Based on similar SFWMD projects, the local utility will need to construct a substation near the large electric driven pump station in order to provide adequate power. The electric substation will be constructed, owned, and operated by the local utility.

The service will likely be 5 kV three phase with 5 kV switchgear and motor controls located in a high voltage electrical room at the pump station to power the large electric pump motors. No pump stations

are for flood control. Therefore, no backup generation will be required for the pumps. A small backup generator may be required for auxiliary equipment in the event of a power outage.

The electrical distribution system will consist of motor control centers, switchboards, and panelboards. Interior and exterior lighting will be designed to conform to IES standards and operational requirements.

Interior Electrical Distribution

Switchgear rated for 480-volt, three phase with a main breaker will be connected to the incoming service and will feed engine control centers, motor control centers, lighting panels, power panels and station equipment defined in the **section A.10.1.1** above. Each engine control center will house starters and controls for auxiliary equipment for the engine unit. The main switchboard will also feed transformers to supply 208/120 volt loads as necessary.

Motor Control Center

Each motor control center will house starters and controls for auxiliary equipment for the pumping unit. It consists of multiple sections with each section containing circuit breaker, starter, and controls for respective motor. The motor control center is the central location for power distribution to the large horsepower motors, ventilation fans, trash rake, power panels and lighting panels.

Interior and Exterior Lighting

High intensity discharge, industrial high bay luminaries will be used for the main pumping station area with industrial fluorescent fixtures with electronic ballasts for office and general type areas. Exterior lighting for security purposes would be automatically controlled by photo-electric cells and contactors.

Wiring and Conduit

Insulated copper conductors will generally be installed in either PVC coated rigid galvanized steel conduit when exposed, or schedule 80 rigid plastic conduit when embedded. Conductors will be rated for 600 volt insulated types XHHW or XHHW-2. All wiring will conform to Unified Facilities Guide Specifications.

Instrumentation and Controls

The pumping station will have a centralized monitoring and control room. Each diesel engine pump drive will have a separate motor control center to supply power and house controls for the engine auxiliaries, such as jacket water pump, engine lube pump, fuel filter pump, etc. Each diesel engine will also be equipped with a separate instrument panel and will house engine start/stop controls and pressure and temperature indicators to indicate engine performance. Programmable logic controllers (PLCs) will be used to monitor and control the engine and station auxiliaries. An Ethernet network will connect the PLCs and station computer. Ethernet based IP cameras will also connect to the Ethernet network. The station computer will allow for operation of the station via SFWMD's preferred SCADA software. An LPG generator will be provided to power instrumentation and monitoring systems during commercial power failures.

SCADA and Telemetry

A PLC-based industrial control system with HMI panel interface will provide for local, automatic, and remote control of the structure. The controls systems shall include manual, automatic and telemetry capabilities for the pumps and auxiliary systems. The engine start/stop controls shall operate locally at each engine, remotely from the control room, and from the central control station. The automation components of all pumping stations and structures that will eventually be operated and maintained by SFWMD must conform to SFWMD standards in order to (1) achieve cost efficiency in design, construction,

and operation and maintenance, (2) meet safety, reliability, and performance requirements during routine and emergency operations. The automation components are broadly defined to include hardware, software, communications, and user interface elements.

A.10.1.4 Gated Spillway Mechanical Features

The gated spillways will have the following features:

Hoist Machinery

The number of gate hoist units will be determined during PED. The units will include wire rope drums, sheaves, hoisting wire rope, dogging slings, motors, brakes, gear boxes, electrical equipment, controls, shafts, couplings, pillow block bearings, limit switches, seals, equipment covers, and hoist assembly frame, and other necessary items, as shown. Dimensions shown on the drawings, including structural supports, will be estimated. The final hoist system design (drawings and calculations) will be performed, sealed, signed, dated, and submitted by a licensed Professional Engineer who is registered in the State of Florida.

Each vertical lift gate will be raised and lowered by an electrically operated hoist assembly. An electric motor will provide the power to one of two input shafts through the gear/speed reducer to turn the shaft and drum assembly. A brake will be installed on the other input shaft. A wire rope will be connected to the drum and to the gate through the sheave blocks. The drum will gather the wire rope connected to it as the gate is opened. A limit switch will prevent the stainless steel wire rope from developing slack should the gate jam while being lowered. A rotary limit switch will stop the gate at the fully-open and fully-closed positions. An upper-limit switch will be installed to stop the gate from raising if the rotary limit switch fails to stop the gate. The slack cable limit switch will also backup the rotary limit switch at the closed position. When not in operation, the gates will be suspended in the fully-open/maintenance operation position using dogging slings.

The entire gate hoist assembly will be furnished as a complete, frame-mounted unit capable of being installed and removed without breakdown into individual components. Anchor bolts will hold the hoist assembly frame to the operating platform. Hoist equipment will be furnished and installed as required. Operation of the gates will be from a local control panel. Each gate will operate at a speed of 6 inches per minute. The position of the gates will be determined visually with local wall gauges and electronically with a PLC housed in the control room. No remote position indication via telemetry will be provided.

Electric Motor-Drive, Brake, and Reduction Gear

The operating mechanism will consist of an electric motor-driven, worm type reduction gear equipped with a brake. The reduction gear provided will feature two parallel input shafts and two parallel output shafts. The (900/1800 rpm) electric motor will be mounted on one input shaft and the brake will be mounted on the other input shaft.

Wire Rope

The wire rope material will be stainless steel. This material will reduce maintenance and will provide reliable performance in the environment encountered at the project site.

Gate Position Transducer

Each gate will be provided with a gate position transducer to indicate the position of the gate in the PLC located in the control room as well as remotely via the internet.

Rotary Limit Switch
The rotary limit switch will control the required predetermined positions of the gate, i.e., the fully-closed and the fully-open/maintenance operation position. Other, intermediary positions will be controlled by pushing the stop button located in the support columns at the service bridge elevation, at the operating platform, or in the control room.

Slack Cable Sensor
Should the gate jam while in the process of being lowered, the wire rope can develop slack, which can cause the gate to drop suddenly when freed. As soon as the slack cable sensor detects a slight slack, downward movement of the gate will cease, preventing the slack to progress. Operation will not resume until the cause for the slack is removed and the sensor is back to its pre-set position.

Dogging Slings
There will be a set of two dogging slings per gate to dog a gate off at the fully-open/maintenance position of the gate.

Additional Components
Additional components of the gate hoist assembly include wire rope drums, drive shafts, flexible shaft couplings, pillow block bearings, gate upper-limit switches, and a structural steel base frame.

Telemetry
Each spillway or culvert site that requires remote automation will be equipped with an RTU compatible with the existing SFWMD telemetry system. RTU software will be in accordance with the SFWMD standard load set. The construction plans will contain plans for a fully functioning telemetry system capable of connecting to and communicating with the SFWMD existing system. Additional coordination during the development of plans and specifications will finalize the telemetry requirements.

A.10.1.5 Gated Culvert Mechanical Features

The culvert gates will consist of flush-bottom, medium-duty stainless steel slide gates per AWWA standard C-561 manufacture and design. Gates will be actuated by a single rising stem driven by an electric motor operated hoisting unit mounted on an operating pedestal at the top of the gate, or on the top yoke of the self-contained gate frame. The actuator will also be equipped with a hand wheel to permit manual operation in the event of power outage. The actuator will be capable of both open/close and modulating service. Integral motor controls and control accessories will be available to interface with MOSCAD Remote Control System used by the SFWMD. The actuator should be watertight for outside operation, and equipped with torque and rotary limits for up and down travel.

Gates will be fitted with gate position instrumentation mounted on the housing of the gate structure. A travel limit switch will be part of the actuator. The design operating speed of the gate with actuator is 6 inches per minute (+/- 10%).

- Gates will be fitted with gate position instrumentation mounted on the housing of the gate structure. A travel limit switch will be part of the actuator. The design operating speed of the gate with actuator is 6 inches per minute (+/- 10%).
- Gates will be 304 stainless steel, vertical lift (flush bottom) made of materials as below:

- Guides will be made of 304 stainless steel, designed for maximum rigidity, and will have high-density polymer or neoprene fixed, replaceable seals.
- Sliding member will be of 304 stainless steel plate reinforced with stainless steel members welded to the plate.
- Seals will prevent excessive leakage.
- Stem will be 304 or 316 stainless steel of suitable length and ample strength for the intended function.
- All parts of the gate will have a minimum thickness of ¼ inch.
- All necessary fasteners and anchor parts will be 316 stainless steel.

A.10.2 S-100 M-Canal Pump Station

The only project feature in Flow-way 1 is the S-100 pump station **(Plates P-26, P-27).** This pump station is designed and located to convey flows from the lower basin of the Indian Trail Improvement District southward into the M-Canal.

The inflow pump station will have a capacity of 75 cfs, with a pump mix of three 25 cfs pumps. The pump stations will consist of separate inlet bays with independent trash racks with a common trash rake, submersible axial-flow pumps, discharge piping, discharge flap gate, and accessories.

The pumping station S-100 pumps shall be driven by electrical motor rated at 460 volts minimum. External electrical distribution shall include area lighting, security cameras Internal electrical distribution includes but not limited to motor control center, power panel, lighting panel, various control stations, power receptacles and overhead lighting. The electrical load shall also be sufficient to cover any mechanical item that require electric power to operate, as well as to monitor auxiliary mechanical systems. The pump station security system shall include basic intrusion detection and keypad entry system.

The S-100 pumping station will receive commercial power for normal operation. The inflow pump station will not have a backup generator installed. An LPG generator will be installed to provide power for instrumentation and monitoring systems.

A.10.3 C-18W Reservoir

Pump stations will convey surface water into the reservoir from the M-O Canal Connector (C-101W, southeast portion of the reservoir) and also the C-18W Canal (north portion). Pump stations also will manage and convey seepage in the canal along the entire outer perimeter of the reservoir.

A.10.3.1 Pump Stations S-106 and S-101A

The inflow/seepage pump station S-106 will have an inflow capacity of 175 cfs, with a pump mix of two 50 cfs pumps and three 25 cfs pumps **(Plates P-02, P-19, P-20,** and **P-21)**. S-106 will also have an additional seepage capacity of 30 cfs, with a pump mix of two 30 cfs pumps, one will be used for backup **(Plate P-19 and P-22)**. The inflow pumps of pump station S-101A will have a capacity of 150 cfs, with a pump mix of two 50 cfs pumps and two 25 cfs pumps **(Plates P-02, P-16, P-17, and P-18)**

The pump stations will consist of an inflow supply canal for the inflow pump station, a seepage supply canal for the inflow/seepage pump station, separate inlet bays, trash/debris removal systems, vertical pumps, diesel engine drives with right angle gear speed reducers or electric motor drives with in-line speed reducers, discharge piping, discharge, and accessories. Life cycle cost analyses of alternatives for powering both the inflow and seepage pumps will be performed during PED. Both diesel and electric motors will be analyzed.

The pumping station S-101 and S-106 pumps shall be driven by electrical motor rated at 460 volts minimum. External electrical distribution shall include area lighting, security cameras Internal electrical distribution includes but not limited to motor control center, power panel, lighting panel, various control stations, power receptacles and overhead lighting. The electrical load shall also be sufficient to cover any mechanical item that require electric power to operate, as well as to monitor auxiliary mechanical systems. The pump station security system shall include basic intrusion detection and keypad entry system.

The S-101A and S-106 pumping station will receive commercial power for normal operation. The inflow pump station will not have a backup generator installed. A LPG generator will be installed to provide power for instrumentation and monitoring systems.

A.10.3.1.1 Pump Station Features

General features of the S-101A and S-106 pump station are described in **section A.10.1.1.** Features unique to S-101A and S-106 are described below.

<u>Discharge Arrangement</u>
The discharge structure for both pump stations will be buried along the prism of the embankment with discharge pipes, siphon break/air vent valves, and check valves. The two means of backflow prevention will be the invert of the discharge pipes for the inflow pumps will be set at an elevation 1-ft higher than the maximum tailwater level at the top of the embankment and check valves.

Two means of backflow prevention are necessary from the discharge basin back to the inlet bay. The invert of the discharge pipes for the inflow pumps will be set at an elevation 1-ft higher than the maximum tailwater level in the discharge basin or be submerged in water contained by a weir. For the second means of backflow prevention, the discharge piping for the inflow pump station will incorporate flap gates. Each inflow pump discharge pipe will have a vent-pipe for air to escape during filling.

An automated siphon break valve will be installed on the top of the inflow pump discharge piping to prevent reverse flow and allow the discharge pipe to drain when the pumps are stopped. To establish a siphon, the air release/vent valve on the pumps will open when the pumps are started to allow air to escape during filling. The valve will close when the pipe is full. The air release valve will have a manual override in case of automatic actuator failure. Once the siphon is established, the static head that the pumps will operate against will be the difference between the sump and discharge water surface elevations, which results in lower energy consumption by the pumps.

A.10.3.2 Pump Station S-109

Pump station S-109 **(Plates P-02, P-23, P-24, and P-25)** will move surface water from the M-O Canal northward to the C-18W reservoir. The pump station will have a capacity of 150 CFS, with a pump mix of

two 50 cfs and two 25 cfs pumps. This inflow pump station features are designed as described above in **section A.10.1**.

The pumping station S-109 pumps shall be driven by electrical motor rated at 460 volts minimum. External electrical distribution shall include area lighting, security cameras Internal electrical distribution includes but not limited to motor control center, power panel, lighting panel, various control stations, power receptacles and overhead lighting. The electrical load shall also be sufficient to cover any mechanical item that require electric power to operate, as well as to monitor auxiliary mechanical systems. The pump station security system shall include basic intrusion detection and keypad entry system.

The S-109 pump station will receive commercial power for normal operation. The inflow pump station will not have a backup generator installed. A LPG generator will be installed to provide power for instrumentation and monitoring systems.

A.10.3.3 ASR System at the C-18W Reservoir

Four ASR wells are proposed to augment storage at the reservoir. Water will be recharged to and recovered from these wells using the reservoir as a recharge and recovery basin. Reservoir water levels will be monitored at a stilling well **(Plate P-55).** ASR wells are not numbered structures, but are referred to as ASR-1 through -4 **(Plate P-54)**. An ASR system consists of the following components: a combined intake/discharge structure, recharge pump, filter, in-line ultraviolet (UV) disinfection system (if required), recovery pump on the ASR wellhead, four wells with inner and outer casings, appropriate piping to convey water between the wells and the reservoir. A SCADA system monitors system alarms, and is connected to the SCADA system at the intake pump station.

<u>Intake Structure</u>
Mechanical components of the intake structure include a type 304 stainless steel Tee-Screen flanged to an 18.5-in intake pipe. This intake structure, within the reservoir, is equipped with an air burst system to dislodge debris.

<u>Recharge Pump</u>
The recharge pump is mounted in the intake structure to pump water from the reservoir to the ASR wellhead. This pump is a three-stage vertical turbine pump, to deliver approximately 5 MGD. The pump is fitted with a 250-HP, 480-V, 1800-rpm three-phase electric motor, equipped with a soft start-stop feature. The recharge pump is equipped with alarms for automatic stop during low water levels.

<u>Filter</u>
Recharge water is filtered to remove particulates. The filtration system consists of automatic screen-type filters equipped with 80-micron screens that can accommodate the 5-MGD flow split among 8 filter units. Filter units have sensors to detect increased pressure, which triggers an automatic backwash cycle.

<u>In-line UV Disinfection System</u>
The disinfection system inactivates naturally occurring microorganisms in recharge water using UV radiation. Recharge water flows through the chamber within which are the UV lamps. Lamps are cleaned by an automatic wiper system. A system control center consists of a 480Y/277 V 3-phase, 4-wire plus ground at 50/60 HZ and 36 kVA. The power distribution center consists of one 120V, 1 phase, 2-wire plus ground at 60 Hz and 1.2 kVA.

Recovery Pump
The recovery pump is a 3-stage vertical turbine pump mounted on the wellhead, similar to the Recharge pump.

A.10.3.4 Gated Culverts S-102, S-103E, S-103W, S-105, S-107

Gated culverts at the C-18W reservoir function to convey water from C-101W (M-O canal connector), and to control water levels in the seepage canal. Refer to the following plates for reference: S-102 **(Plates P-02, P-06, P-35,** and **P-36)**; S-103E and S103W **(Plates P-02, P-06, P-37,** and **P-38)**; S-105 **(Plates P-02, P-06, P-40,** and **P-41)** and S-107 (**Plates P-02, P-42,** and **P-43).** Generalized features of culverts are defined in **section A.10.1.5**

A.10.3.5 Telemetry

An existing communications facility is located near the northwestern corner of the C-18W reservoir. The facility will provide a connection to the SFWMD backbone microwave communications system and will serve as a local communications hub for the project (**Plates P-51 and P-52)**.

Water level monitoring will occur at an instrumented stilling well **(Plate P-55)**. Power for the monitoring equipment will be provided by either commercial power or by solar power. Monitoring data will be transmitted by telemetry to the SCADA system.

A.10.3.6 Control Buildings

The C-18W reservoir will have prefabricated control buildings with 46-foot concrete poles for the SCADA and microwave, depending on SFWMD requirements **(Plates P-51** and **P-52)**. Control buildings will have either an emergency LP gas generator or standby power plug for a portable emergency generator. A LP gas generator will provide backup electrical power for the instrumentation/SCADA system network transmission equipment and communications room. Ventilation systems will be as described under Ventilation.

A.10.3.7 Palm Beach County Pump Station Replacement

The Palm Beach County pump station located within the C-18W reservoir footprint will be replaced-in-kind. The pump station is made up of one 50 cfs diesel-driven pump. Because the pump station is not a USACE design, it does not conform to USACE standards. The available auxiliaries are unknown. The discharge pipe diameter is estimated to be 30-inches, using USACE standards of keeping discharge velocities under 12 fps.

A.10.4 Cypress Creek Canal Modifications

Although there are no pump stations located directly on the Cypress Creek Canal, there are structures that have mechanical and electrical design components. Structure S-112 is a proposed gated spillway on the Cypress Creek Canal.

A.10.4.1 Gated Spillway S-112

The gated spillway S-112 **(Plates P-07, P-33,** and **P-34)** mechanical features are similar to those described in **section 10.1.4.**

A.10.4.2 Gated Culverts S-115A, S-115B, and S-115C

Gated culverts S-115A through C control sheet flow from the Culpepper property into the north-south oriented portion of the Cypress Creek Canal. All three culverts exist but will be replaced by operable structures **(Plates P-56** and **P-57).** The mechanical components for all the operable culverts will be the same. Both weir and sluice gates are operated using a stem driven by an electric motor actuator. Plates P-56 and P-57 show the weir gates for S-115A, B, and C with two actuating stems opposed to the single stems for the other operable culverts, however S-115B and S-115C weir gates with have a single stem.

A.10.4.3 Culvert S-115A

Culvert gates are similar to those described in **Section A.10.3.4 Gated Culverts S-102, S-103E, S-103W, S-105,** and **S-107** at C-18W reservoir. Culvert S-115A is larger than culverts S-115B and C.

A.10.5 Mack Dairy Road Spreader

Pump station S-113N **(Plates P-28 and P-29)** will draw surface water from the Cypress Creek Canal southward into the Mack Dairy Rd spreader, located at the western end of the Cypress Creek Natural Area. Pump station discharge will be controlled in a plunge pool at weir S-113S **(Plates P-14, P-28, P-29, P-49A)** prior to distribution into the spreader swale.

A.10.5.1 Pump Station S-113N

The pump station will have a capacity of 20 cfs, with a pump mix of two 10 cfs pumps. This inflow pump station will be designed similarly to the S-100 pump station described above in **A.10.1**, but flowing into a discharge weir (S-113S). Mechanical components of S-113N are similar to those described in **section A.10.1**

The pumping station S-113N pumps shall be driven by electrical motor rated at 460 volts minimum. External electrical distribution shall include area lighting, security cameras Internal electrical distribution includes but not limited to motor control center, power panel, lighting panel, various control stations, power receptacles and overhead lighting. The electrical load shall also be sufficient to cover any mechanical item that require electric power to operate, as well as to monitor auxiliary mechanical systems. The pump station security system shall include basic intrusion detection and keypad entry system.

The S-113N pumping station will receive commercial power for normal operation. The inflow pump station will not have a backup generator installed. A LPG generator will be installed to provide power for instrumentation and monitoring systems.

A.10.5.2 Fixed Crest Weir S-113S

Water level monitoring equipment will be provided on both sides of the weir. Power for the monitoring equipment will be provided by either commercial power or by solar power, depending on the final location of the weir.

A.10.5.3 Telemetry

A proposed project communications facility will be located near S-113N in the eastern portion of the Cypress Creek Natural Area **(Plates P-51** and **P-52)**. The facility will provide a connection to the SFWMD backbone microwave communications system and will serve as a local communications hub for the project. The communications building facility is a precast concrete building that contains electrical equipment for the communications facility, a backup LPG generator, and a self-supported 300 foot communications tower. The communications building and tower will be based on SFWMD standard designs adapted to the site. The tower and antenna heights will be determined by a path study from SFWMD.

A.10.6 Gulfstream West Flow-Through Marsh

The Gulfstream West flow-through marsh will serve to rehydrate former agricultural lands and control discharging sheet flow entering the Cypress Creek Canal.

A.10.7 Pump Station S-110

Pump station S-110 **(Plates P-30, P-31, and P-32)** will draw surface water from the HSLCD canal into the northern end of the Gulfstream West feature for subsequent hydration of the flow-through marsh. The pump station will have a capacity of 150 cfs, with a pump mix of two 50 cfs and two 25 cfs pumps. This inflow pump station will be designed similarly to the S-100 pump station described in section A.10.1, but flowing into a discharge weir.

The pump station S-110 pumps shall be driven by electrical motor rated at 460 volts minimum. External electrical distribution shall include area lighting, security cameras Internal electrical distribution includes but not limited to motor control center, power panel, lighting panel, various control stations, power receptacles and overhead lighting. The electrical load shall also be sufficient to cover any mechanical item that require electric power to operate, as well as to monitor auxiliary mechanical systems. The pump station security system shall include basic intrusion detection and keypad entry system.

The S-110 pumping station will receive commercial power for normal operation. The inflow pump station will not have a backup generator installed. A LPG generator will be installed to provide power for instrumentation and monitoring systems.

A.10.8 Weirs S-111N and S-111S

Fixed crest weirs S-111N and S-111S **(Plates P-07, P-48,** and **P-50)** are proposed to control sheet flow in the Gulfstream West FTM. Water level monitoring will be identical to that described in **section A.10.5.2** fixed-crest weir **S-113S.**

A.10.9 Nine Gems Property and Adjacent Areas

Most of the proposed water control structures in this area are passive. However, structures S-114 A through E are existing culverts will be replaced **(Plates P-11**, **P-12, P-44, P-45, P-46,** and **P-46A).** These structures will be operable and equipped with telemetry.

A.10.10 Gated Culverts S-114 A through E

Gated culverts S-114 A through E will control sheet flow discharging southward into the Nine Gems discharge canal. Mechanical and electrical components are similar to those described in **section 10.1.5,** and for structures S-115A, B, and C in **section 10.4.2.** Water level monitoring will be identical to that described in **section A.10.5.2.**

A.10.11 Eastern Project Features: Hobe Grove Weir and Kitching Creek Spreader

Most of the proposed water control structures are passive, with instrumentation to record and transmit water levels. Fixed-crest weir S-117 **(Plates P-13, P-49)** will reduce flashy discharge from the HSLCD Hobe Grove Ditch into the northwest fork of the Loxahatchee River. Kitching Creek consists of a two new ungated culverts S-116 to convey flow from an existing ditch, beneath a road to the spreader C-116 **(Plate P-13)**. Culvert S-116 design is similar to S-104 at the C-18W reservoir. Water level monitoring will be identical to that described in **section A.10.5.2** fixed-crest weir **S-113S.**

A.11 Hydrologic Modeling

The following sections describe the hydrologic and hydraulic modeling for the project. Detailed information about the hydrology of the project can be found in **Annex A-2** Hydrologic Modeling of the Engineering Appendix A.

A.11.1 Project and Modeling Objectives

The alternatives presented herein are evaluated based on their contribution to each of the five objectives using project specific performance measures. Performance measures (PM) are used to evaluate the hydrologic model output and ecosystem functions so that the restoration performance of each alternative can be quantified and compared to the 2070 Future Without Project base case condition. The ability for these objectives to be reached with the implementation of the Recommended Plan is analyzed using evaluation criteria (EC), which is a process to evaluate project effects and to establish a standard level of acceptance identified by law or stakeholders. The project objectives, as well as the associated evaluation and measurements using the PM's and EC's, are explained in greater detail within the main body of the PIR. An abbreviation of the project objectives and performance measures are provided below, as the evaluation of these criteria are dependent on the hydrologic model output.

A.11.1.1 Performance Objectives

- Restore the wet and dry season flows to the Northwest Fork of the Loxahatchee River floodplain.
- Restore and/or maintain oysters, sea grass and other estuarine communities in the Loxahatchee River Estuary.
- Increase natural area extent of wetlands.
- Restore connections between natural areas to improve the hydrology, sheet flow, hydroperiods, natural storage, and vegetation communities.
- Restore native plant and animal species abundance and diversity in Loxahatchee River watershed natural areas, river, and estuary.

A.11.1.2 Performance Measures

<u>Salinity and Flow, Performance Measure 1</u>
In the dry season (December–May): supplemental flows are suggested to maintain a mean monthly flow of 68 to 90 cubic feet per second (cfs) at Lainhart Dam.

In the wet season (June–November): mean daily flows of 115 cfs with a range of 110 to 130 cfs at Lainhart Dam. Mean monthly flow of 110 cfs for 120 days would result in the appropriate levels of wet season riverine floodplain stages.

<u>Watershed Hydrology, Performance Measure 4</u>
This performance measure is used to evaluate benefits to the watershed as measured by the Wetland Rapid Assessment Procedure, which helps measure the achievement of the appropriate depth, duration and frequency targets in the Loxahatchee watershed wetlands. General hydrologic requirements are defined for each of the major plant communities identified within the indicator regions.

Connectivity, Performance Measure 9

Performance measure qualitatively measures criteria for evaluating connectivity between watershed areas. Connectivity is measured by four criterion:

- Connection provides historic hydrologic linkage which contributes to the restoration of downstream areas and improved quantity, timing and distribution of water
- Connections cover majority area (>50%) of an existing or proposed greenbelt
- Connectivity promotes water quality improvements and protects water quality by allowing for only sheet flow across natural lands and natural flow ways.
- Connectivity contributes to expanded native habitats and the support of wildlife populations

A.11.2 Modeling Strategy and Tools

There were two primary applications for the modeling performed for the LRWRP, the first was for the assessment of regional-level hydrologic planning necessary for the identification of the Recommended Plan that achieves the goals and objectives of the study and the second was more detailed models that were used to address specific questions related to hydraulic design and the Savings Clause.

The PDT selected the LECSR-NP model (Giddings et al., 2006) to simulate regional hydrology and Hydrologic Engineering Center River Analysis System (HEC-RAS) for evaluating local canal stages for savings clause (Level of Service for Flood Protection) performance. 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). HEC-RAS 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. The current version supports steady and unsteady flow water surface profile calculations. The LECSR-NP and HEC-RAS model software are both approved by the USACE Hydrology, Hydraulics & Coastal (HH&C) Community of Practice. **Table A-26** provides a summary of the models used and their application. Historic planning level hydrology and hydraulic modeling efforts in the LRWRP project area are available at the Statewide Model Management System (SMMS). http://apps.sfwmd.gov/smmsviewer .

Table A-26. Summary of key hydrology and hydraulics models for current LRWRP study.

Model	Application
LECSR-NP	Regional hydrology (stages and flows over through structures) – SFWMD
HEC-RAS	Canal routing and changes in canal stages - USACE

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 identified during the PED phase of the project. Modifications to the current HEC-RAS models may be necessary to address additional hydraulic design considerations for potential canal conveyance and structural modifications of the Recommended Plan during PED. The optimization and refinement of features is possible in LECSR-NP and HEC-RAS. However, optimization simulations have been deferred to the PED phase.

A.11.2.1 Overview of USACE Model Validation Process and LRWRP Approach

There are several processes and requirements set within the Corps while pursuing USACE model certification. In Engineering Division, models used by the Water Resources Engineering Branch fall under the Hydrology, Hydraulics, and Coastal Community of Practice (HH&C CoP) for model validation **(Table A-27)**. The HH&C CoP process establishes that both the District and Division need to recommend the software for evaluation. The recommendation should state whether the software will be used nationally, regionally, or locally, and should include why the software is needed, an explanation as to what it does and how it does it, why any of the other corporate software already on the list does not meet the needs, who within the Corps has knowledge of this software, what type of peer review has it received, what Area of Expertise (AoE) software list should it be included with, and what documentation, training and support can be found. The goal of the Science and Engineering Technology initiative is to manage the number of pieces of software so the Corps does not have to support multiple pieces of software that do roughly the same thing. The USACE should use "well-known and proven" software unless a new piece of software does something one of the "validated" pieces of software does not. It is important to mention that the HH&C CoP cautions the user that the Agency Technical Review (ATR) must include a much more thorough review of those models that have not been pre-validated. This thorough review should look at the inner workings of the model, as the basic assumptions, equations, and output used or created. An Independent External Peer Review (IEPR) could also be required to ensure the piece of software is implemented successfully. The Project Review Plan reflects and details technical requirements for individuals reviewing these tools.

The LECSR model was reviewed through the Hydrology, Hydraulics, and Coastal Community of Practice (HH&C CoP) validation process for engineering software, as part of the Central Everglades Planning Project (CEPP) project.

The use of LECSR has been noted as "Allowed for Use" by the HH&C Executive Advisory Committee. The Committee approved LECSR for one time use and use only on LRWRP. If the Jacksonville District determines that LECSR is needed to be used on another project, the district will be required to submit a new software validation request and WAM will have to undergo review for all functional areas of the model including groundwater, water quality, reservoir operations, surface water hydrology, and river hydraulics.

The Hydrologic Engineering Center River Analysis System (HEC-RAS), developed by the USACE HEC, has been previously reviewed and classified as a "CoP Preferred" in the hydraulic design and river hydraulics modeling tool category. The expectation is to use HEC-RAS as needed for hydraulic design during PED.

Table A-27. Model classification status for LRWRP models.

Name	Brief Description of the Model & How It Will Be Applied in the Study	Approval Status
Evaluation of Alternatives		
LECSR-NP	The LECSR-NP utilizes USGS MODFLOW 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. 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 model provided output for water stages at cells located throughout the study area as well as flows at key locations.	HH&C CoP Validation Status: Approved for Use
Refinement of Features for Design and Savings Clause		
HEC-RAS	The Hydrologic Engineering Center's River Analysis System (HEC-RAS) program provides the capability to perform one-dimensional steady and unsteady flow river hydraulics calculations. The most recent release of HEC-RAS (version 5.0.4) includes capabilities that allow the model to apply complex operation of gated structures and pump stations. Such operations can change in time or water level conditions anywhere in the system. A new feature in HEC-RAS will allow the 1-dimensional channel flow to interact with 2-dimensional floodplain flow allowing for more accurate floodplain mapping. In areas where the interaction of open channel flow and aquifer groundwater needs to be explicitly modeled, a new integrated tool based on the original HEC-RAS and MODFLOW models can now be used to accurately simulate the aquifer/canal flow exchange.	HH&C CoP Validation Status: Preferred Model

A.11.3 Phases of the Evaluation in LRWRP

Modeling support for LRWRP focused on working with the larger project planning team and other interested parties to formulate and test project features leading to the ultimate identification and refinement of the Recommended Plan. Modeling products were developed at the appropriate level of detail to support the detailed representation of project features and to provide information to all necessary evaluations required for plan development and documentation in the Project Implementation Report (PIR). The phases are: 1) preliminary screening 2) baseline modeling and focused array of alternatives; 3) design refinements of the Recommended Plan; and 4) optimization. The report also includes a snapshot of the expectations of the project during PED.

A.11.3.1 Preliminary Screening

Preliminary screening of the initial array of alternatives identified during the LRWRP Plan Formulation phase was completed without updated hydraulic and hydrologic modeling tools or results. The PDT assigned scores for each of the alternatives using screening criteria based on the planning objectives using professional judgment and prior modeling results from past studies. The results of the first rounds of screening analyses yielded four alternatives, 2, 5, 10, and 13, that were carried forward for further analysis.

A.11.3.2 Evaluation of Baseline Conditions and Alternatives

This section provides a summary description of the results of the existing and future without project conditions within the study area from the LECSR-NP model results. Detailed documentation of existing and future without project conditions is further provided in Annex A-3, Modeling Documentation. Baselines are necessary to identify the changes between the existing conditions and the future without project. Information gained from the alternatives evaluations were compared among the baselines until

the best plan that fits the project objectives and criteria was selected. In order to reach the best plan, several steps were required. These steps are described in the subsequent sections.

A.11.3.2.1 Modeling Tools Overview: Lower East Coast Sub-Regional Model-North Palm (LECSR-NP)

In August 2016, the LRWRP PDT selected the LECSR-NP hydrologic model to simulate and compare the performance of four alternatives relative to a future without project and existing base condition. The LECsR model boundaries were changed and refinements were made for the LRWRP project during the development of the sub-model, LECSR-NP (Obeysekera et al., 2018 and Giddings and Obeysekera, 2018). During the 2016 LECSR-NP model implementation, multiple significant model updates were completed, including:

- Incorporated updated topography and land use as well as additional hydrogeologic information.
- Revised control structure operations and weir elevations in Martin County and portions of Palm Beach County.
- Improve simulation of coastal well fields.
- Revised active model domain and wetland assumptions.
- Included multiple model refinements in the Flow-way 3 area.
- Revised evapotranspiration, rainfall and runoff calculations
- Revised calibration period 2006-2014, includes a number of additional wetland and groundwater monitoring sites recently installed

A.11.3.2.2 Conceptual Model

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). 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. 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). The project boundary and model domain are shown in **Figure A-29**.

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 MOD-FLOW packages used are described in **Table A-28.**

The LECSR-NP Model is a three-dimensional numerical model consisting of 704 x 704 foot cells with 3 vertical layers. There are 292 rows and 408 columns which cover portions of southern Martin County and northern Palm Beach County. Temporal discretization is applied as a daily time step in a daily stress period which allows for an adequate level of sub-regional accuracy. Additional information on model conceptualization can be found in Giddings and Obeysekera (2018) and Obeysekera et al. (2018).

Table A-28. MOD-FLOW packages used in the LECSR-NP model for the LRWRP.

Package	Application
River (RIV)	Used to simulate rivers and canals that can contribute water or drain water from the groundwater aquifer. Head values are specified in the river package for each cell.
Drain (DRN)	Used to simulate the effects of existing drainage canals and ditches. Removes water from the model when the elevation is above the control elevation for the drain.
Reinjection Drain-flow (RDF)	Similar to the Drain package except that it allows water to be redirected to another location in the model instead of being permanently removed from the model
Diversion (DIV)	Simulates the effects of water control structures (e.g., pumping stations, gravity flow drains, weirs) on water levels
Wetland (WTL)	Simulates overland flow in wetlands using the uppermost model layer and barriers to flow.

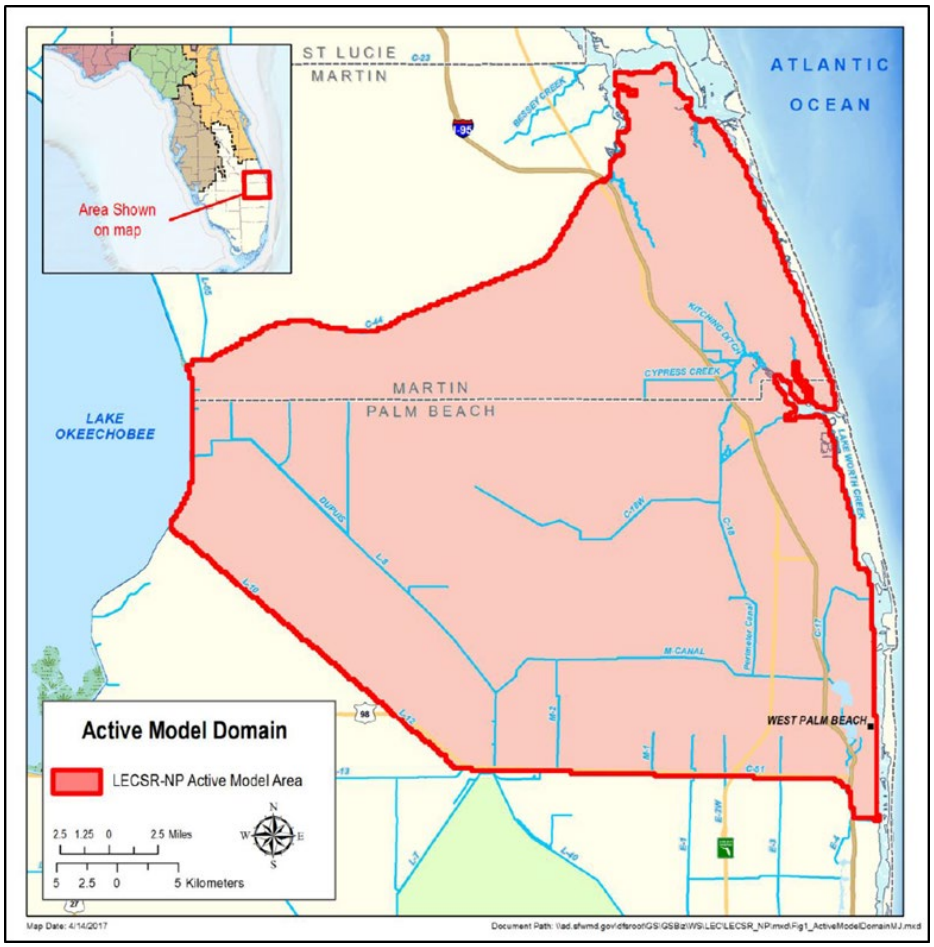


Figure A-29. Project boundary and LECSR-NP model domain.

The LECSR-NP Model uses historical climatic conditions to represent rainfall and reference ET across the model domain. Tidal stages are also represented by historical conditions. For model calibration, canal stages for Lake Okeechobee, L-8, C-44, and C-51 are governed by historical conditions, while model application uses canal stages from the South Florida Water Management Model (SFWMM). Additionally, during model calibration historical flows from the Control 2 structure along the M-Canal to the City of West Palm Beach are used, while the model application uses flows from SFWMM.

The LECSR-NP model provided output for water stages at cells located throughout the study area as well as flows at key locations such as Lainhart Dam, C-18W weir, S-46, G-92, G-160, and G-161. The model also tracked reservoir stages to determine the storage capacity and water supply availability at any given time. Output was in many forms including, but not limited to, stage-duration curves, stage hydrographs and flow-frequency curves. The modeling results are to be evaluated comparatively (i.e., evaluating the relative difference between two simulations); results from a particular simulation should not be taken as absolutes.

A.11.3.2.3 Baseline Condition Modeling

This section describes the assumptions, model implementation steps and observed outcomes associated with modeling representations of the Existing Condition Baseline (ECB) and Future without Project Baseline (FWO) condition baseline model scenarios. These model runs were predominantly used as a basis of comparison among the alternatives to evaluate performance. LRWRP Plan Formulation Baselines were a 2014 ECB, representing the starting scenario, and a 2070 FWO condition. Historical climate conditions were used for the simulation period, and permitted allocations were used for the demands for all public water supply utilities. The ECB and FWO project bases include current system infrastructure assumptions and current operational practices (projects constructed by 2014, with some exceptions. The existing features that are not simulated in either of the base case conditions are the G-160 and G-161 structures and the project culverts along the southern leg of the C-18 Canal. Features simulated in the ECB 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 J.W. Corbett WMA weir and the existing canal operations for the SFWMD canals, water control districts (298 Districts) and local developments. The main change from ECB not presently observed is the Public Water Supply Utility demands which are based upon the SFWMD permitted allocation and not upon recently observed usage. The FWO project base includes all the ECB assumptions, except it is assumed that the L-8 Flow Equalization Basin is operational and is receiving water from outside of the L-8 Basin. In addition, the FWO 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. All alternatives include the components of the FWO project base assumptions unless otherwise specified.

The model simulation period is 14,975 days, or 41 years, and includes rainfall, evapotranspiration, infiltration, runoff, groundwater withdrawals, canal operations, land use, and other factors affecting surface and groundwater flow in south Florida. Output from the model includes daily structure flow, groundwater/surface levels and numerous flow budget terms, for example, groundwater seepage into the Loxahatchee River. Please refer to Giddings and Obeysekera (2018) for additional details regarding the LECSR-NP model application, assumptions, and results. Detailed information of the model table of assumptions can be found in **Annex A-2**, Hydrologic Modeling.

A.11.3.2.4 LECSR-NP Model Calibration

To ensure the model simulates the proper hydrologic processes, the model is calibrated to the observed conditions during the period from 2006 to 2014. This period included both wet (tropical or subtropical events) and dry conditions. The primary goal of the calibration process is to meet both the qualitative and quantitative criteria and targets identified. The modeled error, as measured by statistical analysis, should be minimized by the calibration process. Both flow and water level stages (groundwater and surface water) were used for quantitative calibration criteria for overall performance whereas a qualitative analysis was performed for the water budgets and the Wetland Rapid Assessment Procedure (WRAP) cells. The quantitative analysis was performed using statistical calibration criteria and targets, as specified in **Table A-29**.

Table A-29. Calibration and validation statistical analysis.

Criteria	Analysis	Description
Coefficient of Determination (R^2)	Structure Flow	Coefficient of determination measures the goodness of fit. The target is a R^2 value greater than 0.4.
Nash-Sutcliffe (NS)	Structure Flow	Nash-Sutcliffe is a model efficiency coefficient that indicates the predictive power of models. The target is a NS value greater than 0.4.
Deviation of Volume (DV%)	Structure Flow	Deviation of volume measures the difference between historical and simulated flow volumes. Positive values indicate that the model is under-predicting, negative values indicate that the model is over-predicting. The target is a DV within + 15%.
±Range Error Target (% of time target met)	Water Level Elevations (groundwater and surface water)	Percentage of time that simulated head lies within a plus or minus range error target (ft) of the observed head. Each individual calibration location has its own criteria based on 20% of the absolute difference of minimum and maximum observed values during the calibration period.
Mean Error (ft)	Water Level Elevations (groundwater and surface water)	The mean error is the mean difference between measured and simulated heads.
Mean Absolute Error (ft)	Water Level Elevations (groundwater and surface water)	The mean absolute error is the mean of the absolute value of the differences in measured and simulated heads.
Root Mean Squared Error (ft)	Water Level Elevations (groundwater and surface water)	The root mean squared error is the average of the root of the squared differences in measured and simulated heads.

Each water level calibration location has its own criteria. The acceptance range error target for mean error, mean absolute error, and root mean squared error also are based on the 20% range of the absolute difference of minimum and maximum observed values. The calibration statistics for the flow locations (structure flow) are shown in **Table A-30** and the overall calibration statistic for the water level elevations are in

Table A-31. The wetland and groundwater monitor well calibration gage locations are shown in **Figure A-30.** Calibration gage locations within the model domain for wetland gages and groundwater monitoring well gages

Table A-30. Calibration statistics for structure flow, 2006 to 2014.

Monitoring Station	R^2	DV (%)	NS
C-18 Weir	0.57	-1	0.57
Lainhart Dam	0.82	-9	0.80
S-46	0.85	-5	0.82
G-92	0.74	2	0.74
G-160	0.76	-5	0.75
Cypress Creek and Hobe Grove Ditch	0.73	-4	0.70
Kitching Creek	0.79	-6	0.78

Table A-31. Calibration statistics for groundwater and surface water (wetland) elevations.

Location	Overall Mean Absolute Error (ft)
Groundwater Monitor Wells	0.56
Wetland Gages	0.68

Manual sensitivity analysis was conducted by SFWMD to determine which model input parameters were most sensitive within the LECSR-NP model domain. Both simulated heads and simulated flows were checked during the sensitivity analysis. The tested parameters include vertical hydraulic conductivity, horizontal hydraulic conductivity, river, drain, and general head boundary conductance, specific yield, groundwater recharge, and saturated zone et. Additional details and results of the sensitivity analysis can be found in the calibration report from Obeysekera et al. (2018).

The model was considered reasonably calibrated and could be used to compare between alternatives and base conditions for the purpose of identifying a Tentatively Selected Plan. Multiple agencies participated in the discussion during calibration, including the Florida Department of Environmental Protection, Interagency Modeling Center, U.S. Army Corps of Engineers, and South Florida Water Management District. Calibration and verification results were presented to PDT in meeting on 13 December 2017. A further discussion of the packages and the overall calibration processes and results of the LECSR-NP Model can be found in Obeysekera et al. (2018).

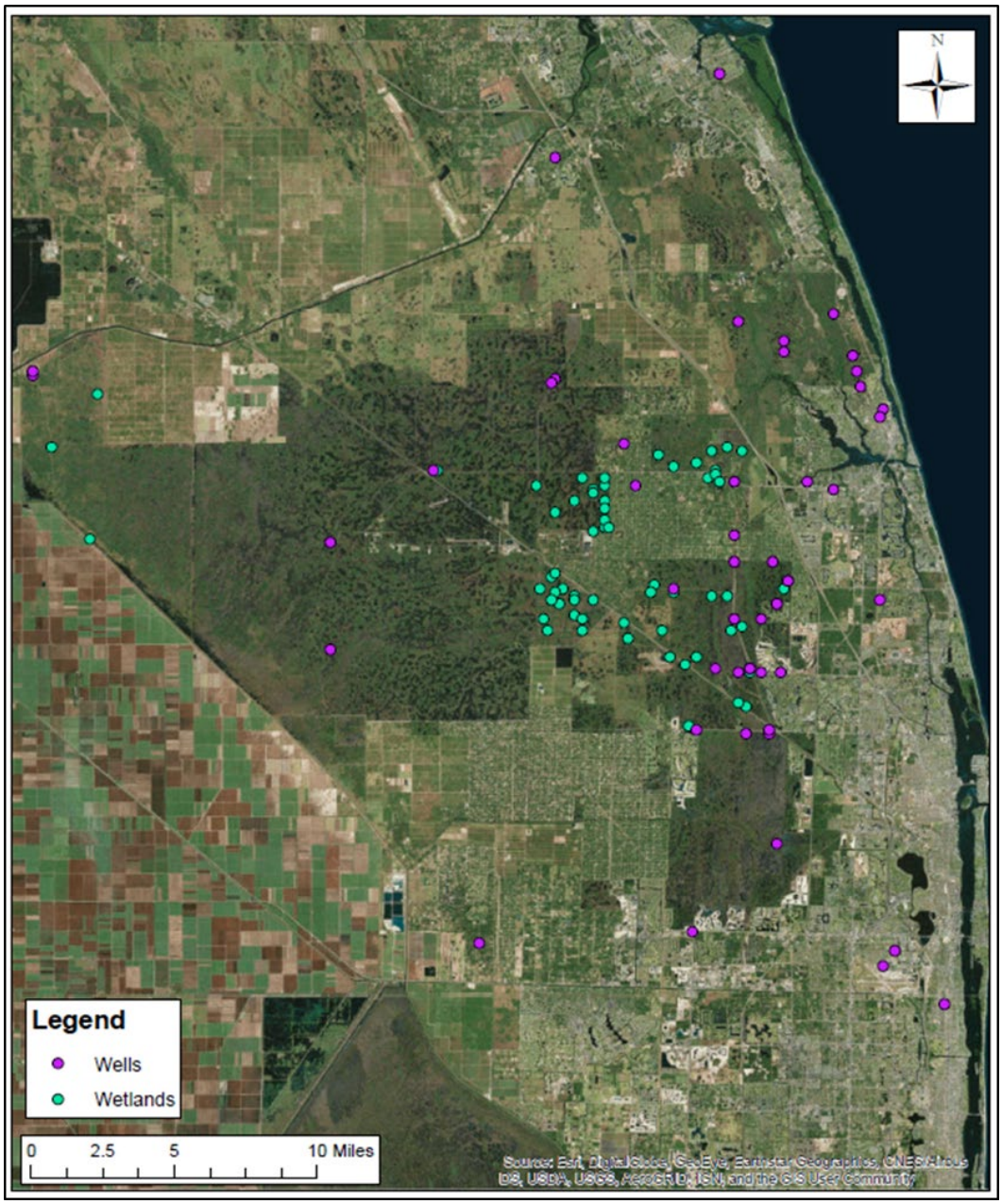


Figure A-30. Calibration gage locations within the model domain for wetland gages and groundwater monitoring well gages.

A.11.3.2.5 LECSR-NP Model Verification

Model Verification is the process to demonstrate that the calibrated model matches a set of field data independent of what was used to calibrate the model. The model verification period was from 2000-2005, as compared to the calibration period that was from 2006-2014. Minor differences in the verification and calibration period are included below:

- G-160 and G-161 structures were not operational during the verification period
- Significant restoration areas undertaken by Palm Beach County in recent years had not occurred or were just beginning
- Large parcels of State owned lands in the project area had not be acquired or recently acquired during the period and had not undergone significant alterations

Similar to the model calibration, statistical analysis were used to compare the model performance against observed data. The validation statistics for the structure flows and water surface elevations are included in **Table A-32** and **Table A-33**, respectively.

Table A-32. Overall verification statistics for structure flow, 2000 to 2005.

Monitoring Station	R^2	DV (%)	NS
C-18 Weir	0.71	3	0.71
Lainhart Dam	0.80	-6	0.71
S-46	0.85	6	0.82
G-92	0.60	2	0.56
Cypress Creek and Hobe Grove Ditch	0.88	7	0.87
Kitching Creek	0.83	3	0.81

Table A-33. Overall calibration and verification statistics for groundwater and surface water (wetland) elevations.

Element	Overall Mean Absolute Error (ft)
Groundwater Monitor Wells	0.57
Wetland Gages	0.65

A.11.3.2.6 Final Array Modeling

This section describes the assumptions, model implementation steps and observed outcomes associated with modeling representations of the with-project condition scenarios for the Final Array of Alternatives. The Final Array of Alternatives consists of four alternatives: 2, 5, 10, and 13. Each alternative was conceptualized to restore wet and dry season flows to the Northwest Fork of the Loxahatchee River, improve ecological communities, increase wetland extent and restore hydrologic connection throughout the project area. This was accomplished by using evaluation criteria and performance measures. For more details, please see the main body of the PIR as well as **Appendix A**, **Annex A-3**, Model Documentation Report.

The following sections describe the modeling assumptions for Alternatives 2, 5, 10, 13 of the project. For more details on the modeling assumptions per alternative, please see **Appendix A, Annex A-3**, LECSR-NP Model Application Report.

Alternative 2

Alternative 2, shown in **Figure A-31**, consists of the C-18W Reservoir with a capacity is 7,200 acre/ft to provide environmental deliveries to the Loxahatchee River via the Lainhart Dam. The reservoir receives direct runoff from ITID upper basin, inflows from the L-8 Shallow Impoundment, inflow from the C-18W canal and J.W. Corbett WMA, and ASR. The ASR system consists of 2 ASR wells with inflow and outflow capacity limited to 15 cfs. The L-8 Shallow Impoundment is a 4,500-acre reservoir that receives inflow from L-8 basin runoff via the L-8 canal. The G-160 structure is operational and maintains a dry season control of 14.0 ft NAVD88 (15.5 ft NGVD29) and a wet season control of 15.4 ft NAVD88 (16.9 ft NGVD29). It provides environmental deliveries to the Loxahatchee River. The G-161 structure can supply up to 20 cfs from Grassy Waters Preserve if the stage within Grassy Waters Preserve is greater than 16.9 ft NAVD88 (18.4 ft NGVD29). Other features include the Kitching Creek spreader swale which is a spreader swale along the northern edge of Jonathan Dickinson State Park and is designed to distribute flow coming into Kitching Creek across the northern wetlands. The Moonshine Creek and Gulfstream East restoration includes adding a weir within the Hobe Grove Ditch Weir to an elevation of 6.0 ft NAVD88 (7.5 ft NGVD29) and removing all infrastructure in the Gulfstream East grove area except for the HSLCD discharge canal. Replace and increase the control elevation of the Cypress Creek Canal weir to 7.5 ft NAVD88 (9.0 ft NGVD29). The Gulfstream West restoration removes all infrastructure in the Gulfstream West grove and replaces it with a flow-through marsh system. The Palmar East restoration involves the removal of all interior ditches and a section of the main southern canal on the Nine Gems property. In addition, the Thomas farms runoff is routed east through HSLCD Unit 2 canal.

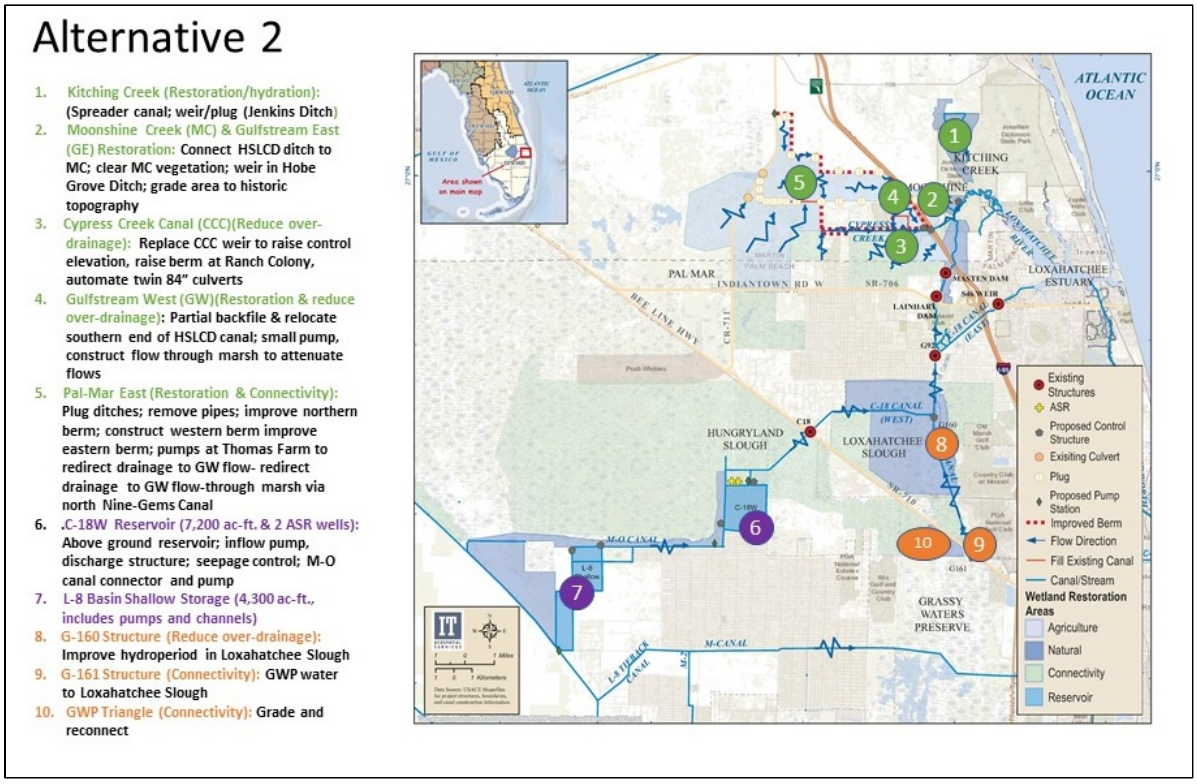


Figure A-31. Alternative 2 Project Components

Alternative 5

Alternative 5, shown in **Figure A-32**, consists of the C-18W Reservoir with a capacity of 9,500 acre/ft to provide environmental deliveries to the Loxahatchee River via the Lainhart Dam is constructed as a reservoir to provide environmental deliveries to the Loxahatchee River via Lainhart Dam. The reservoir receives direct runoff from ITID upper basin, inflow from the C-18W canal and J.W. Corbett WMA, and ASR. The ASR system consists of 4 ASR wells with an inflow and outflow capacity limited to 30 cfs. Construction and operation of both the G-160 and G-161 structures are included. The G-160 structure is operational and maintains a dry season control of 14.0 ft NAVD88 (15.5 ft NGVD29) and a wet season control of 15.5 ft NAVD88 (16.9 ft NGVD29). It provides environmental deliveries to the Loxahatchee River. The G-161 structure can supply up to 50 cfs from Grassy Waters Preserve if the stage within Grassy Waters Preserve is greater than 16.9 NAVD88 (18.4 ft NGVD29) as well as when other operational conditions are met. The Grassy Waters Preserve triangle receives water mainly from the North Lake Weir. The M-1 pump station delivers ITID Lower Basin water into the M-Canal to augment Grassy Water Preserve when G-161 is operating and reduce reliance upon the regional system. Project features for Kitching Creek, Moonshine Creek, Gulfstream East and West, Palmar East, and Thomas Pepper Farms as described for Alternative 2 (section above) were also included for Alternative 5.

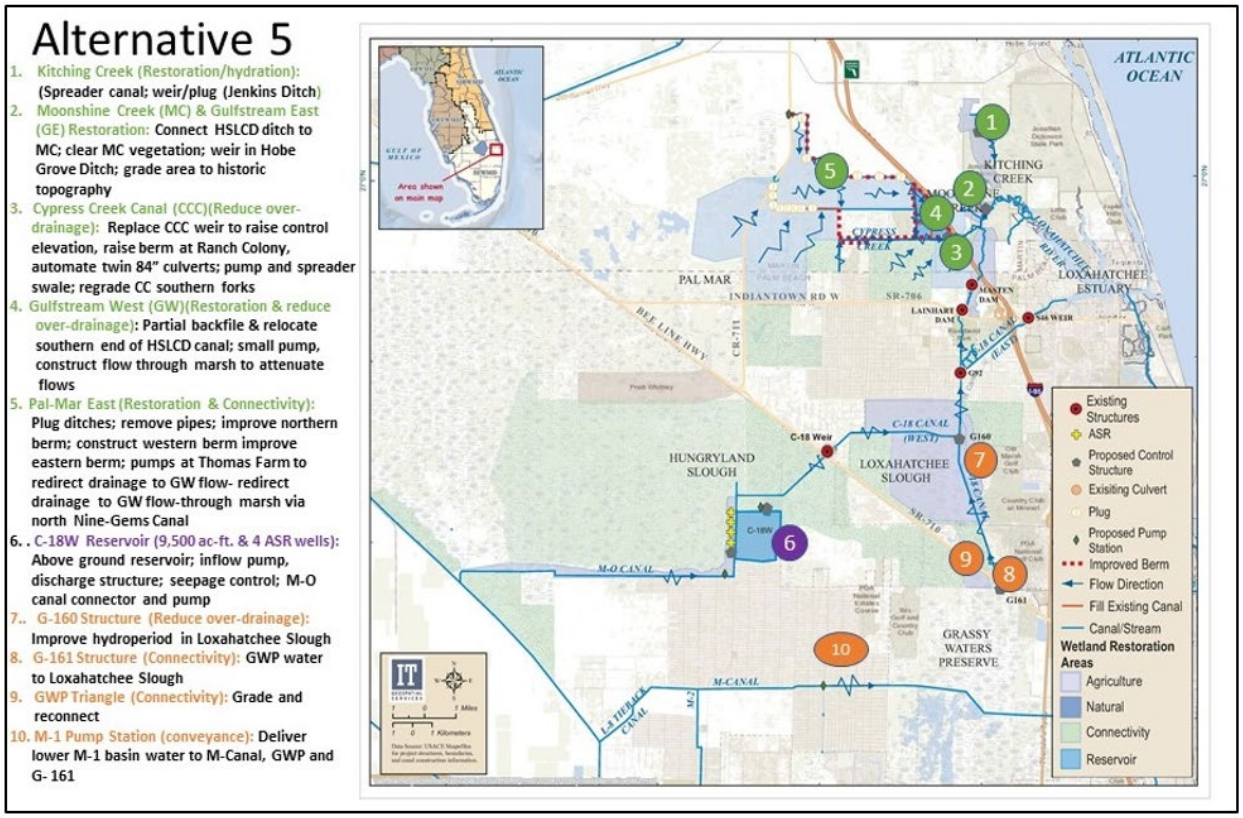


Figure A-32. Alternative 5 Project Components

Alternative 10

The Alternative 10, shown in **Figure A-33**, components include two reservoirs, The C-51 Phase II in-ground reservoir is an approximately 44,000 acre/feet facility whereas the C-18W Reservoir is an above-ground

reservoir with 7,200 acre/ft of storage. The C-51 Phase II receives runoff from the western L-8 Basin as well as from ITID. It is designed to minimize the use of regional system water by the City of West Palm Beach and to provide direct flows, via a force main through G-161, to the Loxahatchee River to meet the wet and dry season targets. The C-18W Reservoir receives direct runoff from ITID upper basin, inflow from the C-18W canal and J.W. Corbett WMA. Other features include Kitching Creek spreader swale, the Moonshine Creek and Gulfstream East restoration and the Cypress Creek Canal weir to 7.5 ft NAVD88 (9.0 feet NGVD29). The G-160 structure is operational and maintains a dry season control of 14 ft NAVD88 (15.5 ft NGVD29) and a wet season control of 15.5 ft NAVD88 (16.9 ft NGVD29). It provides environmental deliveries to the Loxahatchee River. The G-161 structure can supply up to 50 cfs from the C-51 Phase II reservoir, via a force main from the M-Canal to the G-161 structure, if certain conditions are met.

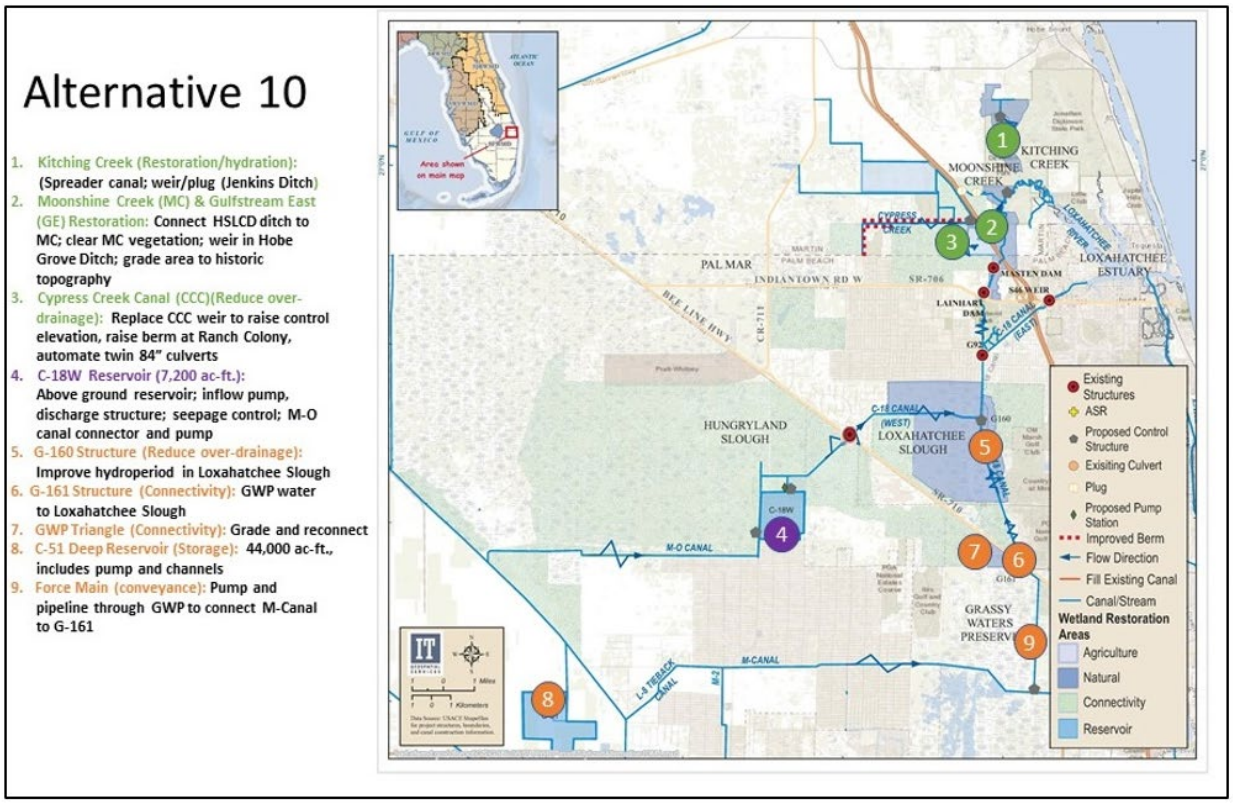


Figure A-33. Alternative 10 Project Components

<u>Alternative 13</u>

Alternative 13, shown in **Figure A-34**, includes the C-18 West Natural Storage feature for wetland restoration that takes water from the L-8 Shallow Impoundment and spreads it from the C-18 W impoundment (footprint) area eastward to the western portion of the Loxahatchee slough. The L-8 Shallow Impoundment is an approximately 5,500-acre reservoir that receives inflow from the ITID upper basin runoff via the M-O canal. It provides water to the C-18 West natural storage area when needed. Alternative 13 includes replacement of the existing-18W to the proposed location just upstream of the existing railroad and Beeline Highway to an elevation of 17.1 ft NAVD88 (18.6 ft NGVD29). Construction and operation of both the G-160 and G-161 structures are included. The G-160 structure is operational and maintains a dry season control of 14.0 ft NAVD88 (15.5 ft NGVD29) and a wet season control of 15.4 ft NAVD88 (16.9 ft NGVD29). It provides environmental deliveries to the Loxahatchee River. The G-161 structure can supply up to 20 cfs from Grassy Waters Preserve if the stage within Grassy Waters Preserve

is greater than 16.9 ft NAVD88 (18.4 ft NGVD29) and other conditions are met. The Grassy Waters Preserve triangle receives water mainly from the North Lake Weir.

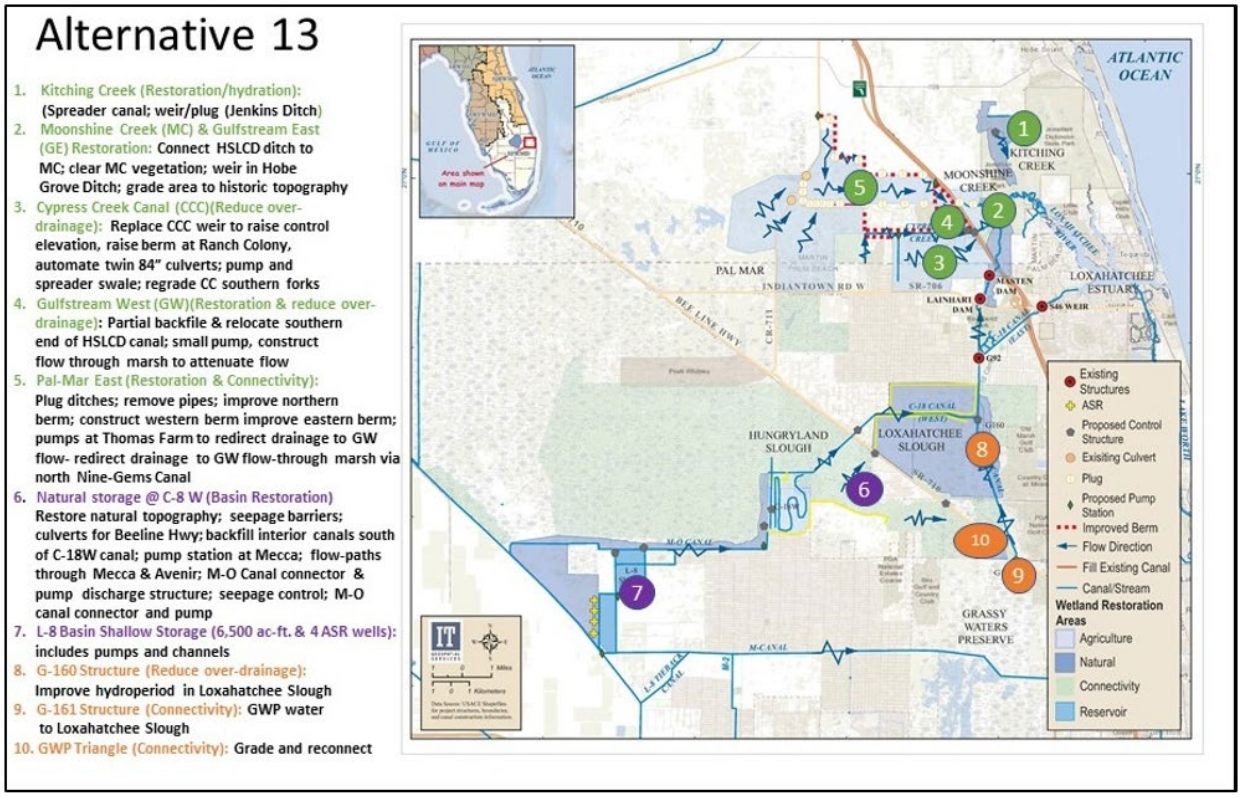


Figure A-34. Alternative 13 Project Components

A.11.3.2.7 Summary of Results

This section summarizes the modeling results for the baselines (ECB and FWO) as well as the alternatives (2, 5, 10, 13). In summary, the four alternatives modeled for LRWRP adequately represent the planning conditions and provide a reasonable basis for evaluation when compared against the proposed LRWRP project baselines. The alternatives performance compared to the baselines, specifically to the FWO, is crucial in determining and claiming benefits for the project.

The LECSR-NP modeling output was analyzed using multiple types of performance measure graphics at key locations within the model domain. The types of performance measure graphics evaluated included storage reservoirs, profile points, indicator regions, other indicators, profiles, difference maps, flow transects, ASR, structure flows, water supply, and water budget. These performance graphics were utilized by the entire PDT to evaluate the model performance for the base case conditions and the various proposed Alternative conditions. Although all performance measure graphics provide beneficial insight to the model's performance and potential project benefits, the main subset of graphics used for model evaluation, from an engineering design perspective, are discussed below. There are approximately 1,500 performance measure graphics produced from the LECSR-NP output, therefore only a small subset of graphics will be included within this report to aid in the understanding of the analysis and evaluation process performed to ensure the project objectives were met and justified for the Recommended Plan.

See Giddings and Obeysekera (2018) for further description of modeling application and discussion of results.

A.11.3.2.8 Flow-way 1

G-160 and G-161

The G-160 structure supports project objectives and delivers project benefits by providing additional control and management of water surface elevations within the C-18 Canal and the Loxahatchee Slough. This is illustrated in the stage-duration curve for location LS-2 in **Figure A-35.** The targeted operations for the G-160 structure are included below:

- December to April: Gate opens at 16.2 ft. NGVD and closes at 15.5 ft. NGVD
- November and May gate opens at 17.1 ft. NGVD and closes at 16.2 ft. NGVD
- June to October (wet season); gate opens at 17.5 ft. NGVD and closes at 17.1 ft. NGVD

The G-160 allowable discharge is up to 150 cfs for environmental deliveries, and to maintain flood control capability as necessary for non-environmental deliveries. The modeling results illustrate that the proposed G-160 structure creates the desired effect, namely improvement in the ecology through hydrologic lift of the Loxahatchee Slough area. Additionally, the G-160 structure aids in both the management of water levels within the C-18 and C-18E Canals and the timing of water deliveries to Northwest fork of the Loxahatchee River during the wet and dry seasons. The average monthly structure flow rates for the proposed G-160 structure can be seen in **Figure A-36**.

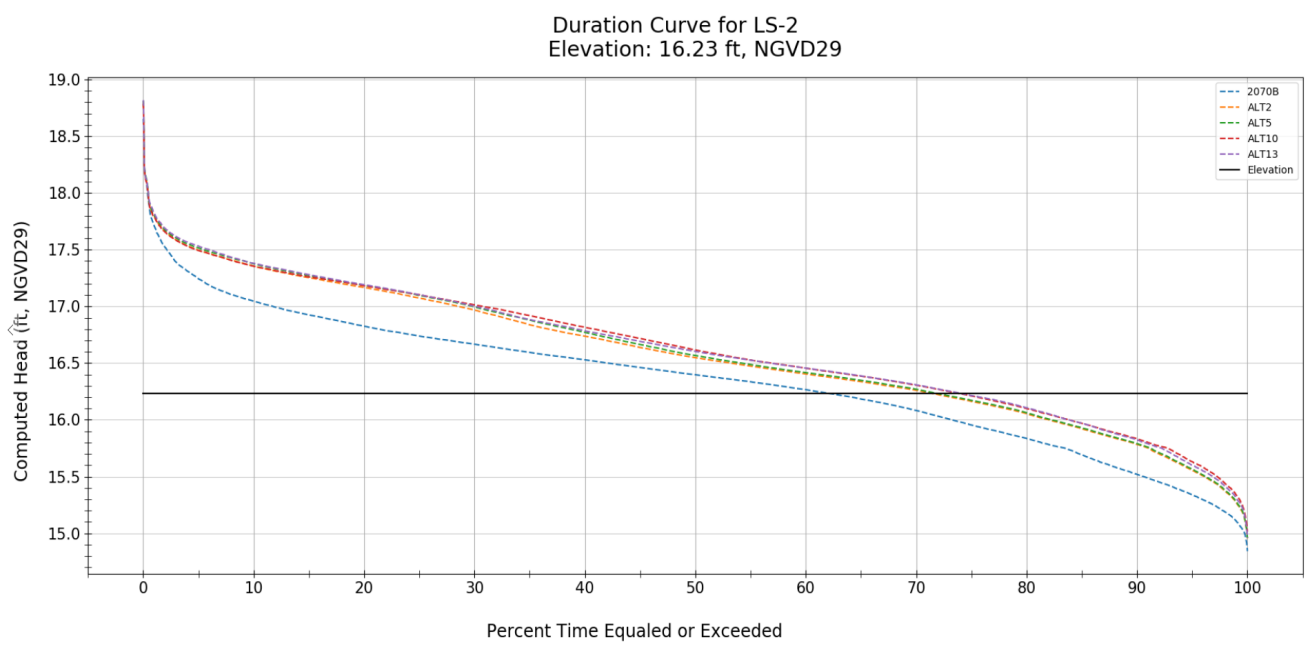


Figure A-35. Loxahatchee Slough stage-duration curve.

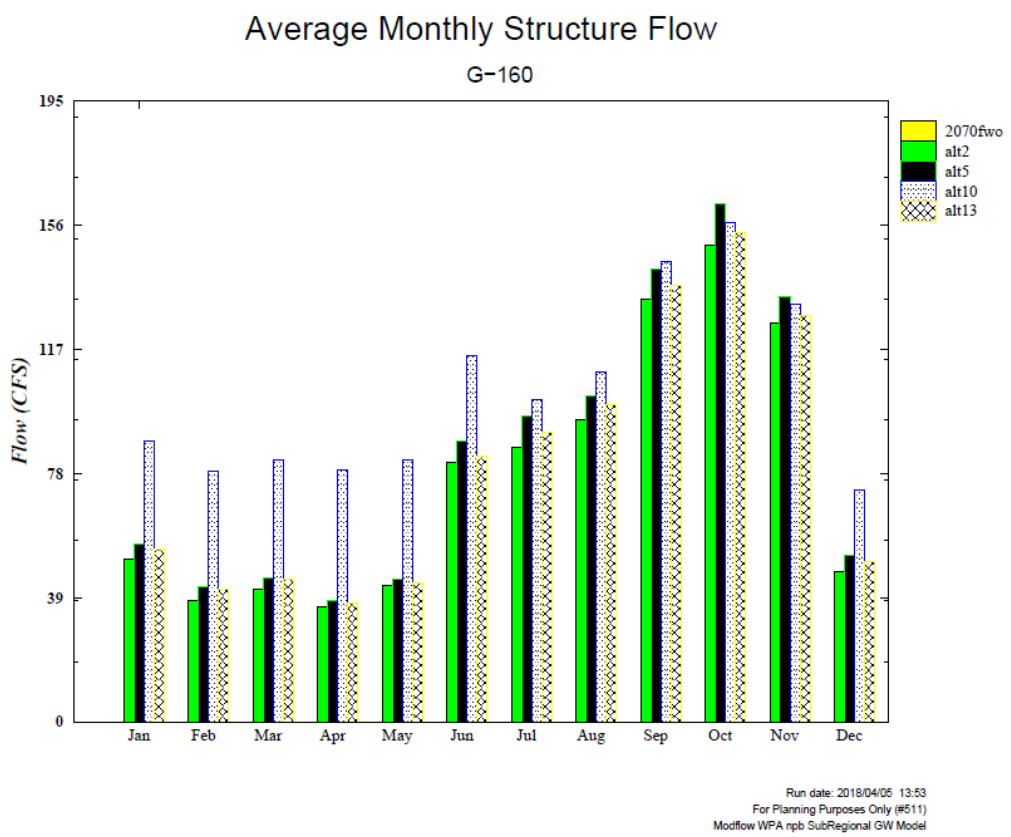


Figure A-36. Average monthly structure flow for the proposed G-160 structure.

The G-161 structure can aid in achieving the target stages in the Grassy Waters Preserve area while simultaneously sending excess water to G-160 and onto the Northwest Fork of the Loxahatchee River. G-161 provides connectivity to the Flow-way 1 features, including GWP, the C-18 Canal and the Northwest Fork of the Loxahatchee River. For Alternatives 2, 5, and 13, up to 20 cfs can move from the northern area of GWP into the C-18 Canal north of the G-160 structure if northern GWP is above 18.4 feet NGVD. Water only moves into the C-18 Canal south leg when stages are below G-160 wet/dry seasonal control elevations described in the section above and if replacement water is available. For Alternative 10, flows through G-161 are up to 50 cfs via the force main. Although the average monthly structure flow in **Figure A-37** illustrates a relatively minor amount of flow being sent through G-161, it is assumed that current model operational assumptions limit the availability of water that can be sent through the G-161 structure. Additional model optimization, through modification of upstream or downstream constraints, may further increase flow rates through the structure. The G-161 structure also provided additional lift within the GWP triangle area, as illustrated in the stage-duration curve in **Figure A-38**. 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.

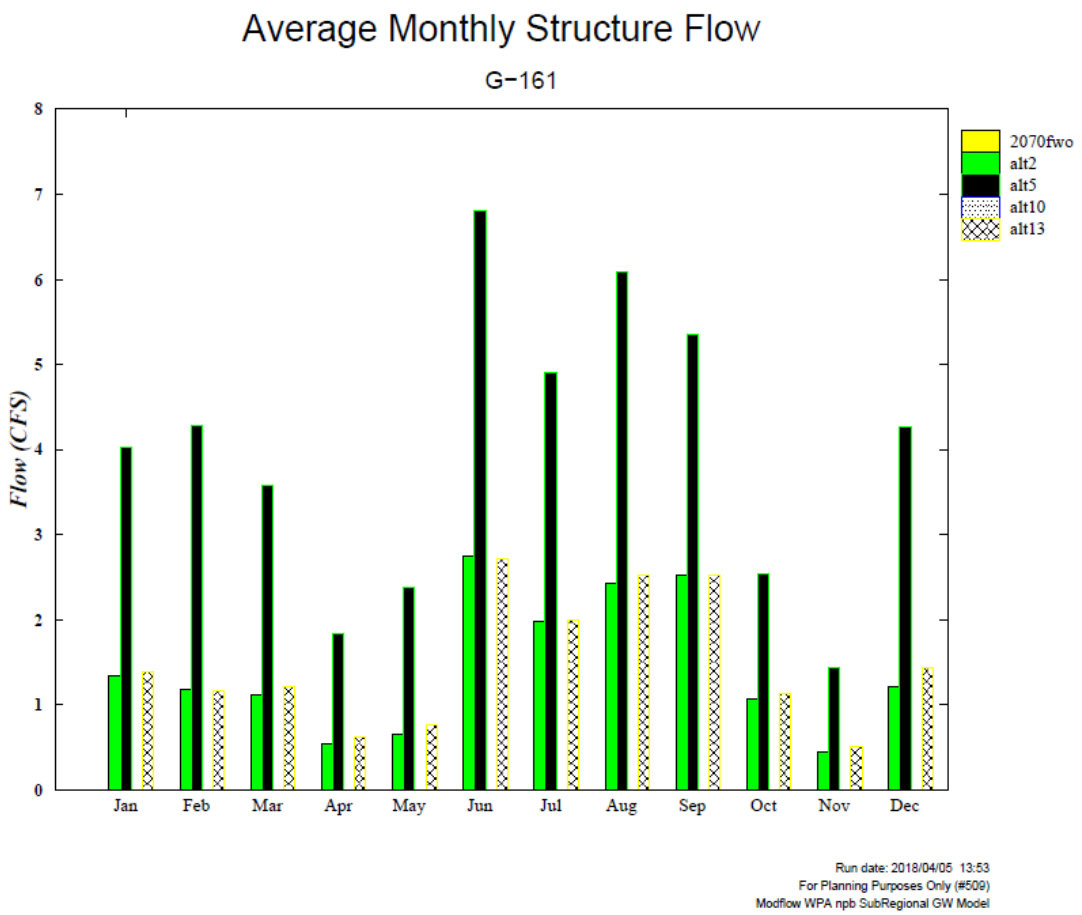


Figure A-37. Average monthly structure flow for the proposed G-161 structure.

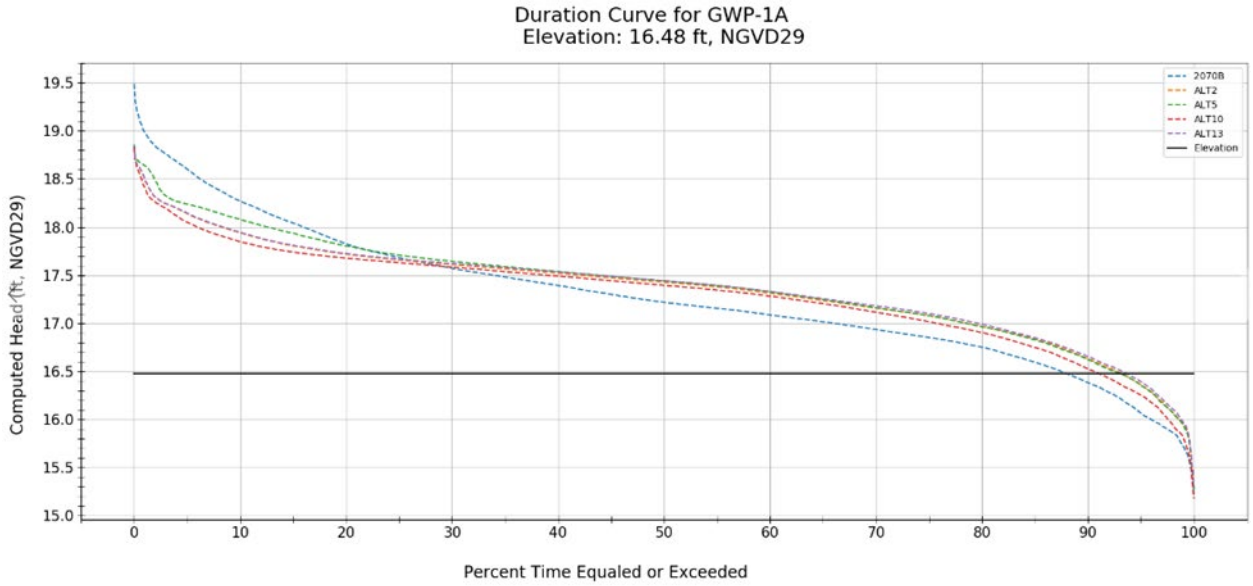


Figure A-38. Grassy Waters Preserve triangle stage-duration curve.

M-1 Pump Station (S-100)

The proposed M-1 Pump station, was designed to bring water into the M-Canal from the ITID lower M-1 Basin. When conditions allow, up to 75 cfs is simulated into the M-Canal downstream of Control 2, beyond the canal segment where flow is limited to 225 cfs. Water is used to offset G-161 plus replacement water flows for Grassy Waters. If G-161 flow is not needed, water is used to reduce regional system demands. Through the construction of G-161, excess water can be sent to the C-18 Canal and Northwest Fork of the Loxahatchee River from GWP. **Figure A-39** demonstrates that for Alternative 5 (Recommended Plan), the only alternative with the proposed M-1 pump station, the average daily flow frequency to the GWP is improved over the 41-year simulation. This increase in flow is also seen in Figure 14, where Alternative 5R (Recommended Plan) continuously sends more water through G-161 than the other proposed alternatives.

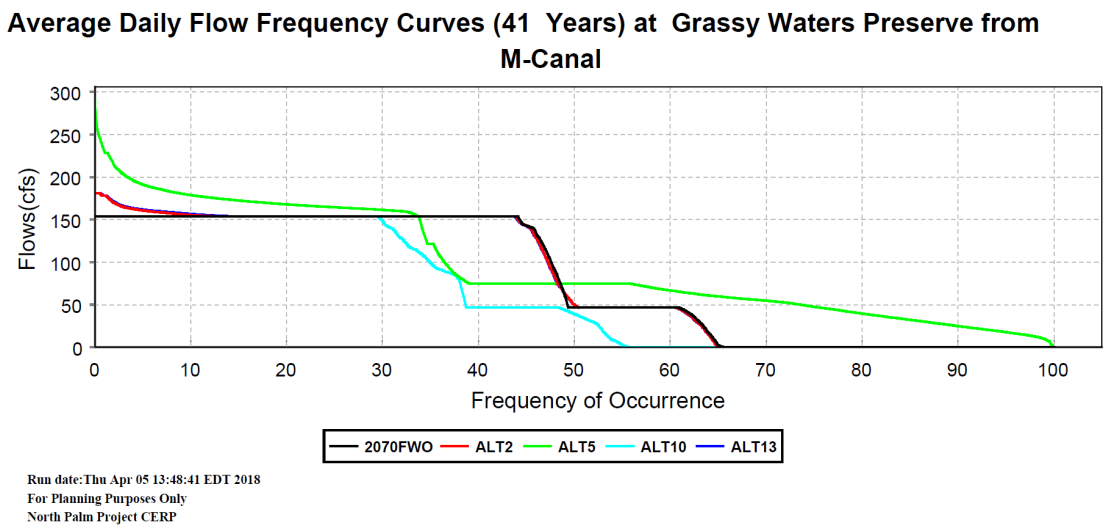


Figure A-39. Average daily flow frequency curve for M-Canal at Grassy Waters Preserve.

C-51 Phase II Reservoir and Force Main

The C-51 Phase II Reservoir and force main are proposed features in Alternative 10 only 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, as seen in **Figure A-36**, above. Increased flows to Lainhart are important especially during the dry season, when the quantity of flow is difficult to maintain. **Figure A-40** illustrates the average daily flow frequency for the C-51 phase II reservoir to the force main, whereas **Figure A-41** shows the flow frequency for the C-51 Phase II Reservoir total outflow. The excess discharge of water from the C-51 Phase II Reservoir is directed to the City of West Palm Beach for water supply purposes. Therefore, it is important to note per the graphics, that the reservoir discharges to the force main at a much lower flow rate and frequency than it does for water supply. Water supply is not an identified project objective and therefore cannot be claimed as a project benefit. Due to the relatively high construction and operation and maintenance costs of the C-51 Phase II Reservoir and force main, the efficiency, or project benefits versus project costs, was analyzed and it was determined that the features should not be carried forward as part of the Recommended Plan.

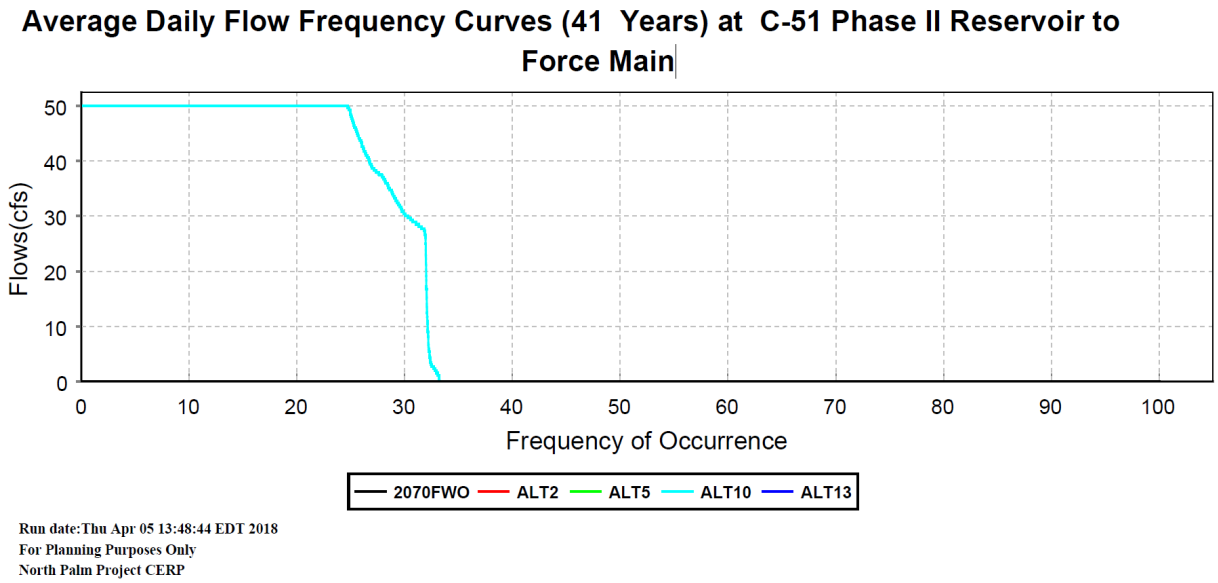


Figure A-40. Average daily flow frequency curves for the force main (from C-51 Reservoir).

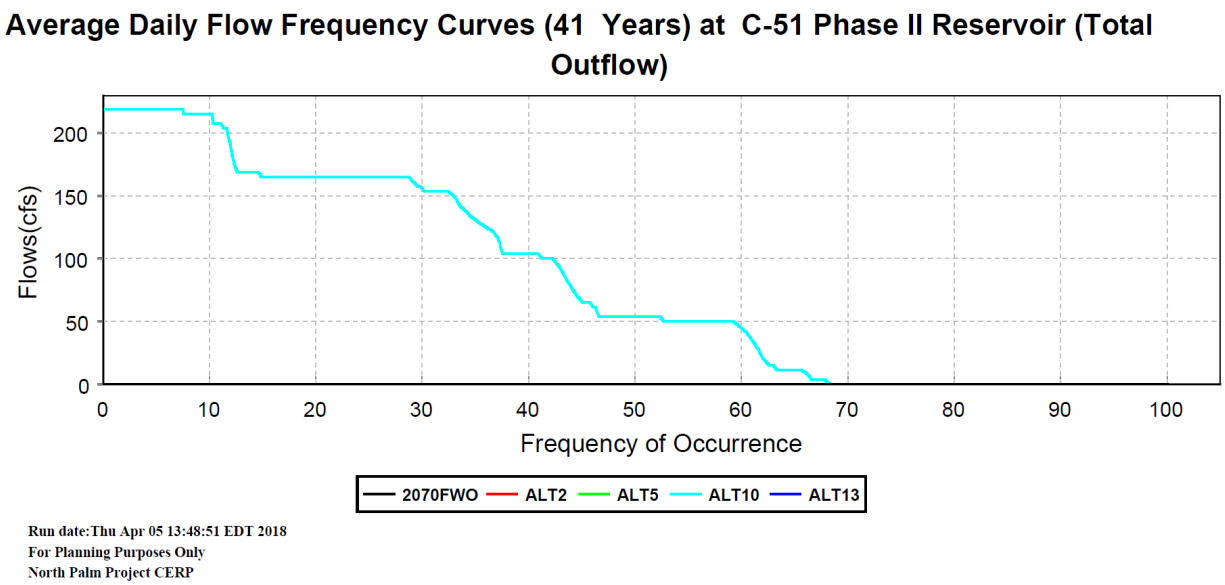


Figure A-41. Average daily flow frequency curves for C-51 reservoir total outflow.

A.11.3.2.9 Flow-way 2

<u>C-18W Reservoir</u>

A configuration of an above-ground reservoir, the C-18W Reservoir, was proposed in three alternatives, 2, 5, and 10. 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. Without the

proposed C-18W Reservoir, excess flows cannot be captured and dispersed in a hydrologically improved manner. During wetter periods, much of the runoff from C-18W basin and J.W. Corbett WMA may potentially be lost to tide through the S-46 structure because there is no storage facility to capture and store the water. Similarly, flows from the ITID upper basin may be lost to the C-51 and/or L-18 to the Lake Worth Lagoon (also to tide). 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 the **Figure A-42**. The C-18W weir is located downstream of the reservoir within the C-18W Canal. The Recommended Plan (Alternative 5) performance illustrates a discernable improvement over the FWO conditions for both the wet and dry season. Alternative 5 shows additional improvement over the other alternatives in **Figure A-43**. This graphic shows that the larger reservoir size coupled with the addition of the ASR wells provides a greater overall C-18W outflow capability. This additional capacity allows for greater flexibility in reservoir operations and additional robustness in providing flows to the river during the dry season or periods of drought.

The C-18W natural storage feature in Alternative 13 does not include a reservoir component but is designed to pump water into a more natural flow path for the purpose of rehydrating and restoring historical wetlands and providing additional water to the C-18W Canal. The below figures demonstrate the shortcomings of the natural storage feature due to the decreased deliveries to the C-18W Canal and to the river. The increased flow provided by the above ground storage in Alternatives 2, 5, 10 aid in sending greater flow volumes to the C-18W Canal year-round.

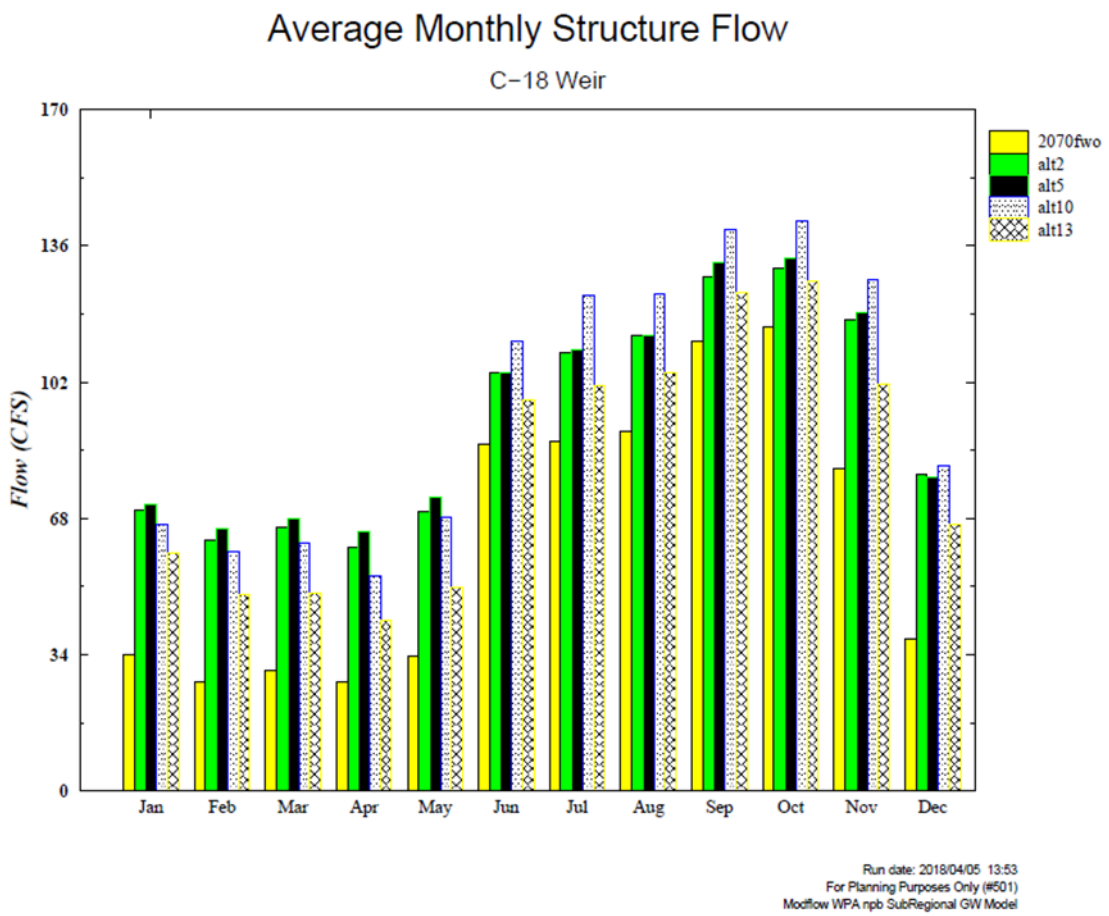


Figure A-42. Average monthly structure flow at the C-18W weir.

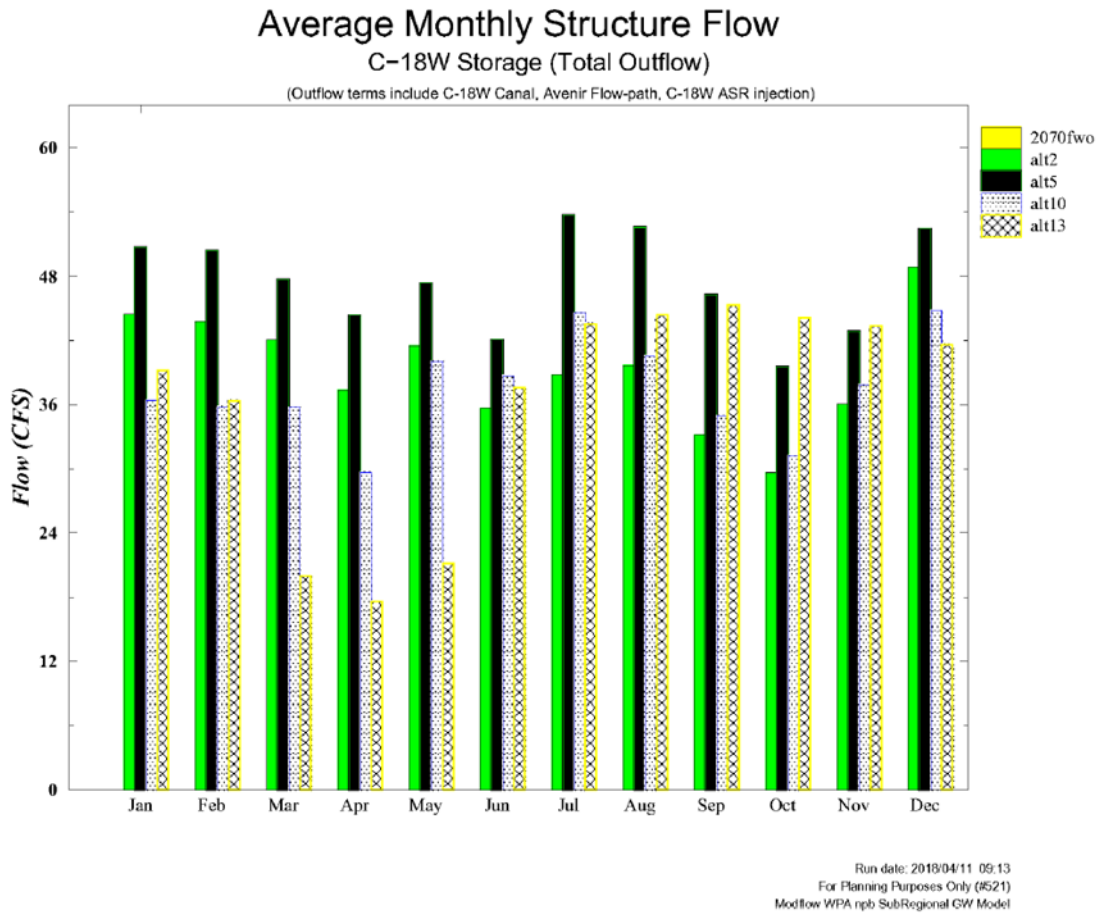


Figure A-43. Average monthly flow for the total outflow of the C-18W reservoir.

The M-O Canal connection and pump station deliver excess water from the ITID basin to the C-18W Reservoir. **Figure A-44** shows that on average, the ITID basin supplies the C-18W Reservoir between 25 and 60 cfs, which accounts for a large percentage of the total inflow into the reservoir. The ASR performance is shown in **Figure A-45** and highlight the performance of the 4 ASR well systems for the C-18W Reservoir. It is important to note that the ASR is modeled using the diversion package within the LECSR-NP model. Recovery efficiency is not based on the flows that can be injected or withdrawn into the ASR wells (that is limited to 30 cfs per the project assumptions) but is based on the volume held within the bubble. For example, if the maximum storage volume is 30,000 acre-feet, then only 70% of that volume would be available for withdrawal (21,000 acre-feet). The ASR wells are able to recover during the dry season an approximate monthly average of 13.4 cfs. From the months of March to May, when the dry season flows can be at their lowest, the average recovery is 15.5 cfs, which is accounting for approximately 23 percent of the flow for the dry season river targets (15.5/68 cfs). This illustrates the importance of the ASR wells and their contribution to the river during the dry season.

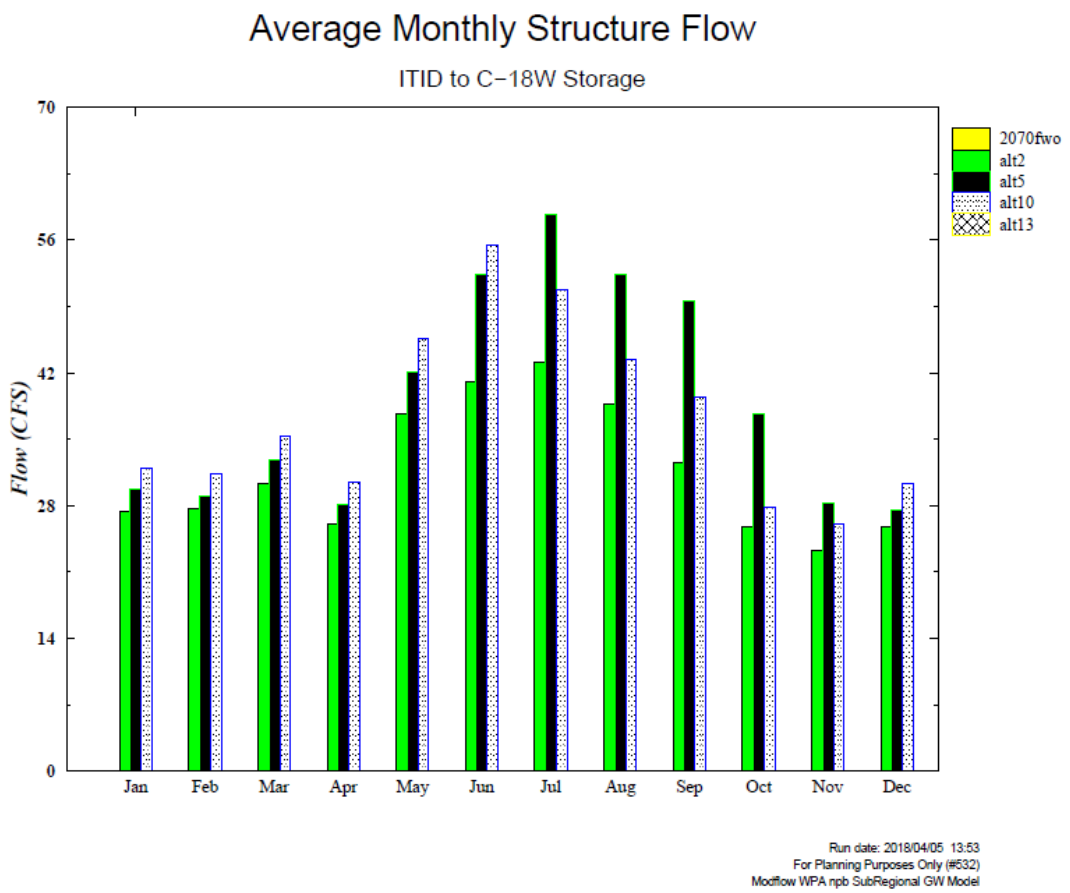
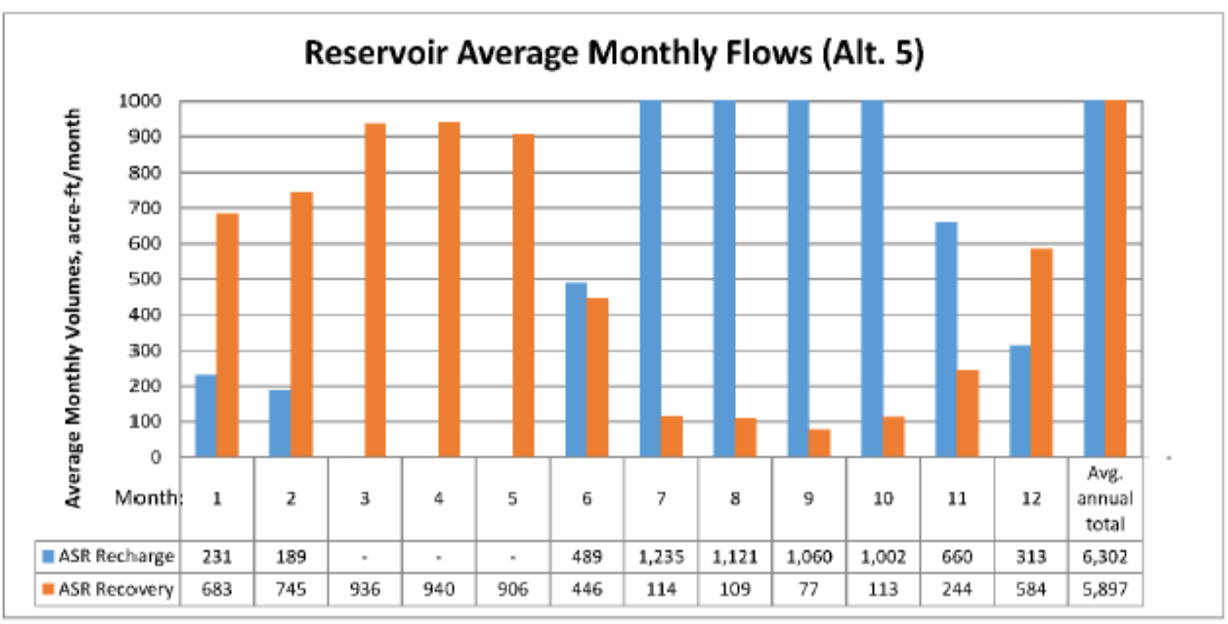


Figure A-44. Average monthly structure flow to C-18W reservoir from ITID basin.



Month:	1	2	3	4	5	6	7	8	9	10	11	12	Avg. annual total
ASR Recharge	231	189	-	-	-	489	1,235	1,121	1,060	1,002	660	313	6,302
ASR Recovery	683	745	936	940	906	446	114	109	77	113	244	584	5,897

Figure A-45. ASR-reservoir average monthly flows for Alternative 5

L-8 Basin Shallow Storage

The L-8 Shallow Storage feature, proposed in Alternatives 2 and 13 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 model results with respect to the L-8 shallow storage feature indicated multiple problems including losses due to seepage, operational complexities, and minimal outflow to the C-18W Reservoir or natural storage feature. The losses due to seepage were noticed when reviewing the difference maps, which are maps that highlight the change in water levels between two model runs (with alternative against the base-case). **Figure A-46** illustrates the average monthly flow discharged from the L-8 Shallow Storage feature, which is minimal. The operational complexities arise from both the stage constraints in the M-O Canal as well as the distribution of water from the L-8 Shallow Storage since there are multiple transfers before flows reach the Loxahatchee River. Connectivity from the L-8 shallow storage to the river requires the water to be transported through the M-O Canal, M-O Canal Connector, C-18W Reservoir (or natural storage feature), C-18W Canal and the C-14 Canal. At each location the water may be subject to losses either by drainage canals, via the M-O Canal, or through seepage and evaporation.

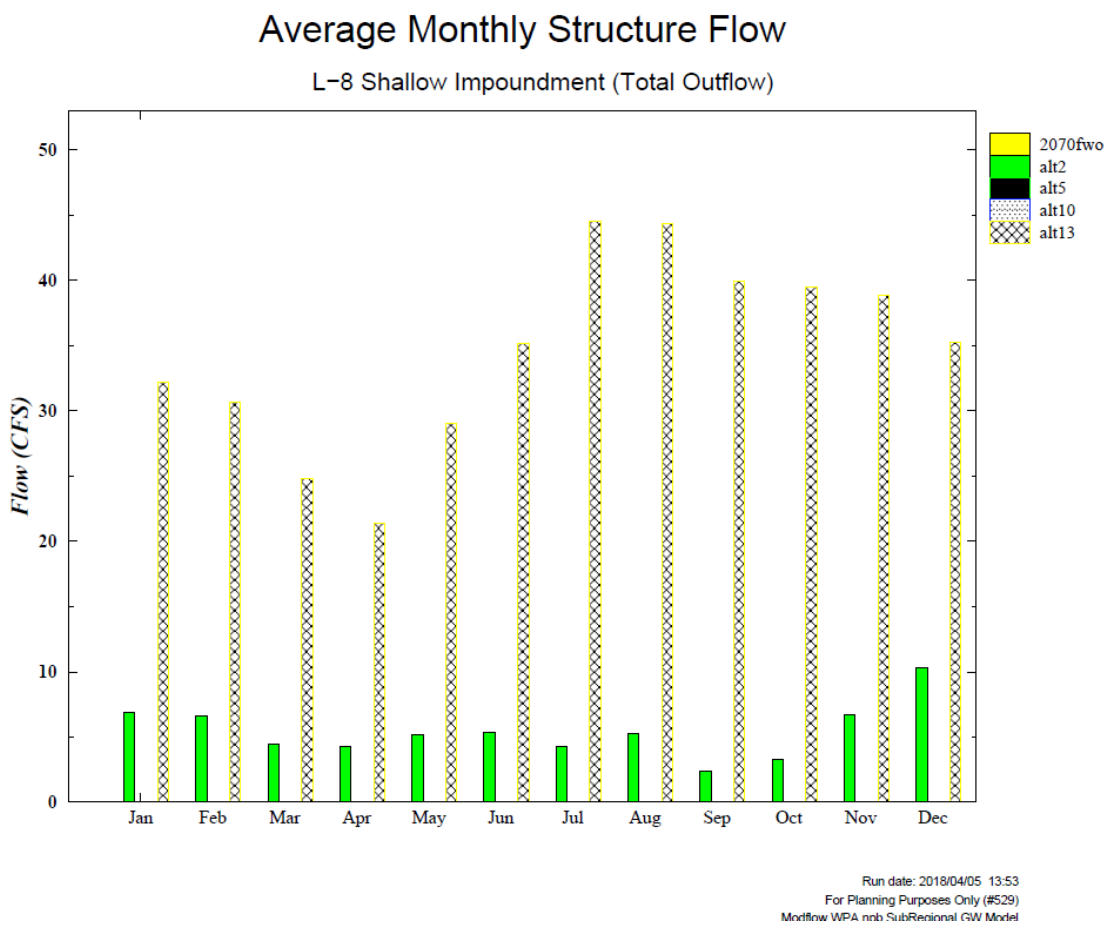


Figure A-46. Average monthly outflow of the L-8 shallow storage.

A.11.3.2.10 Flow-way 3

Pal-Mar East (Nine Gems)

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. The stage-duration curve, shown in **Figure A-47**, reflects this improvement of the hydrology within the Nine Gems area, which results in improved ecological benefits including increased wetland habitat. The PM-1.1 cell is located within the middle of the Nine Gems area, north of the existing Thomas Pepper Farms drainage canal. The resulting increase in stage at this location, and the associated project benefits, are assumed to derive from the applied alternative measures in this area. With the removal of the existing Thomas Pepper Farms drainage canal (currently bisecting Pal-Mar East), hydrologic re-connection of flow occurs between the two parcels, further improving the wetland structure and function within the area. It is important to note that the modifications in Pal-Mar East were proposed in Alternative 2, Alternative 5 and Alternative 13. Therefore, there is no discernable difference between the results of these alternatives and the resulting graphical display of the results are overlapping and the results shown in the graphic for Alternative 13 can also be assumed for Alternative 2 and 5. Similarly, Alternative 10 and the FWO condition do not contain any features within Pal-Mar East so the resulting lines on the stage-frequency graphic are also overlapping.

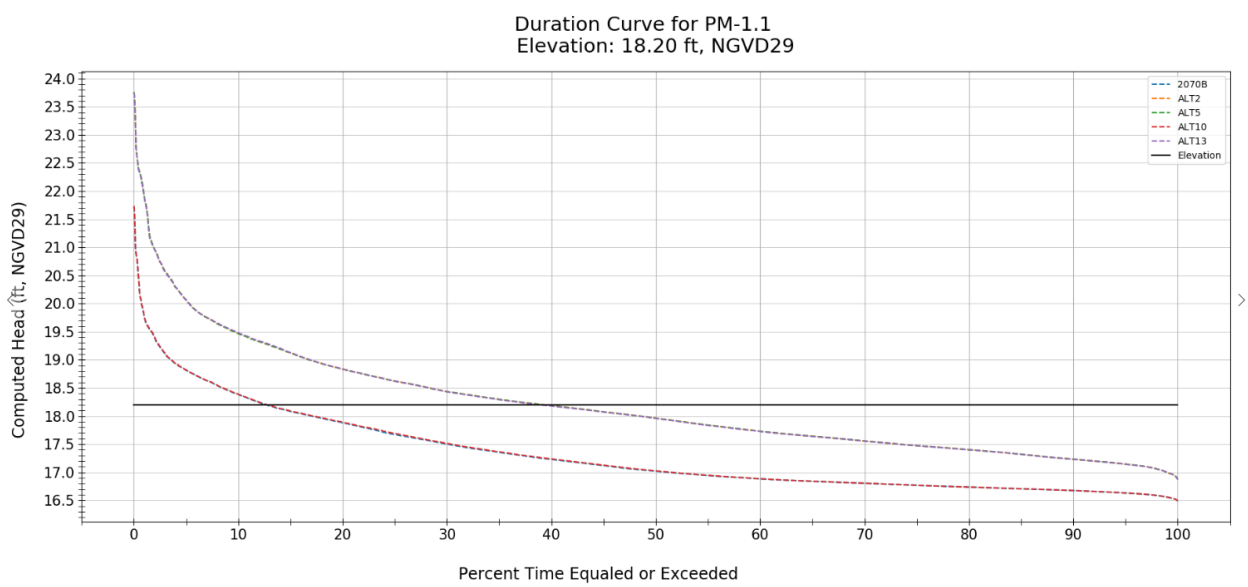


Figure A-47. Stage-duration curve for Pal-Mar East.

Cypress Creek Canal

The proposed Cypress Creek Canal improvements include modifications to the existing water control structures at Culpepper as well as adding an additional control structure within the canal. The improved hydroperiod and natural storage within Culpepper is shown in **Figure A-49**, and is the product of the proposed modification to the WCS 1-4 inlet elevations. It is estimated that further improvements of the stage-duration curve are expected if the structures are operated specifically for wet, dry, and possible transition periods to assist in achieving the most desirable hydroperiod for wetland habitat. This optimization of an operation schedule will occur during PED. The proposed Cypress Creek Canal structure also achieved the desired project benefits by improving the surrounding groundwater (and surface water)

elevations for a greater duration during the simulation period. As can be seen from **Figure A-49**, all alternatives create hydroperiod improvements within the Cypress Creek Natural Area, with CC-4.1 representing a cell within the north-central portion of the property. Though Alternative 5 shows a relatively large improvement, Alternative 13 shows greater improvement due to the Mack Dairy Spreader System, which was a feature that was added to the Recommended Plan (Alternative 5R). Therefore, the final hydrologic response expected from the implementation of the alternative features for the Recommended Plan are expected to mirror that of Alternative 13 in this area. The addition of the Mack Dairy Spreader system helps improve the hydroperiod and ecological restoration of the Cypress Creek Natural Area. Additionally, it allows excess water within the Ranch Colony Canal to be distributed to the natural areas, further improving the timing, quantity, and potentially water quality of deliveries from the Ranch Colony Canal to the Northwest Fork of the Loxahatchee River.

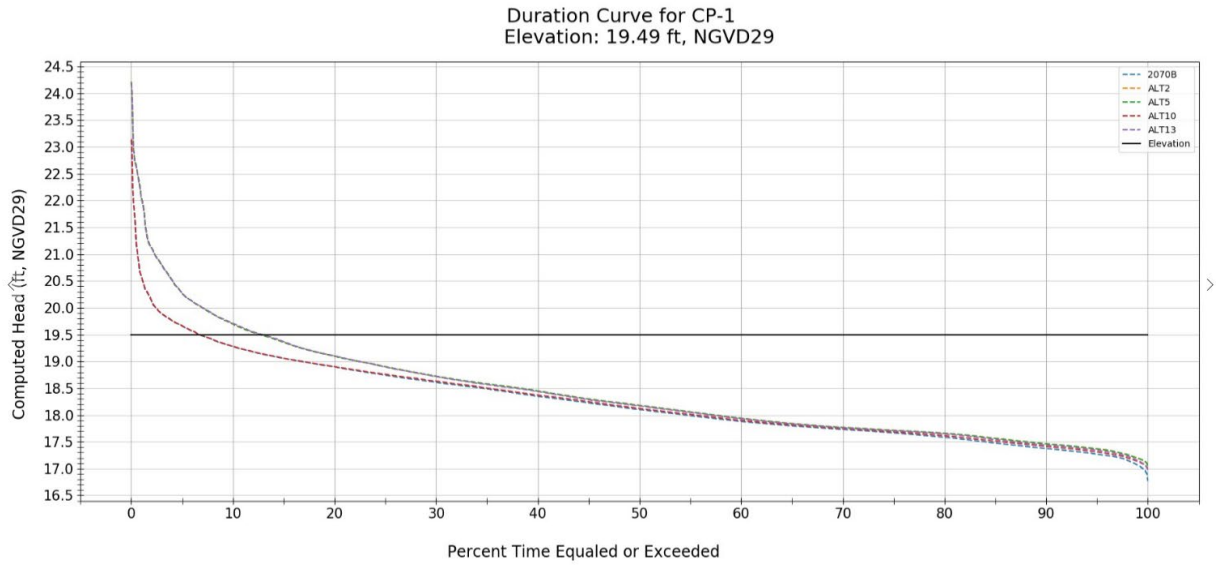


Figure A-48. Stage-duration curve for Pal-Mar East, Culpepper property.

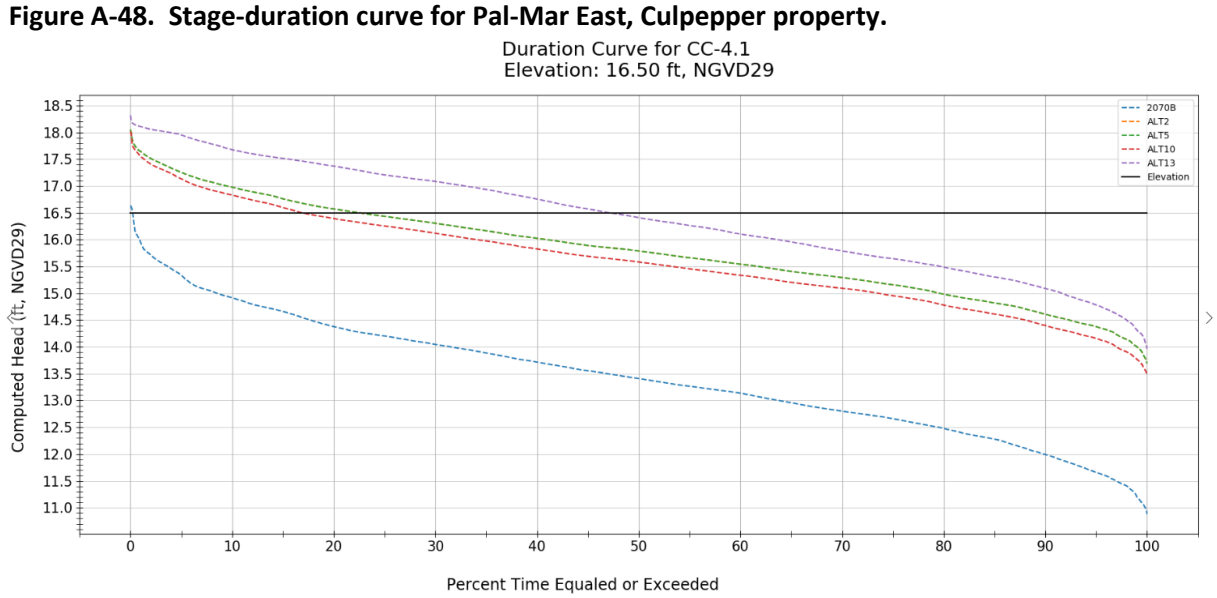


Figure A-49. Stage-duration curve for Cypress Creek Natural Area.

Gulfstream West

The proposed flow through marsh within the Gulfstream West property was designed to transform the existing overly drained agricultural land to marsh or wetland habitat, thereby increasing the natural extent of wetlands. A drastic improvement in the hydroperiod for the Gulfstream West property can be seen in **Figure A-50**. Similar to other graphics, Alternatives 2, 5, and 13 have similar results and therefore the lines may overlap on the graphic. This feature allows excess water from HSLCD to be stored, reducing peak inflows into the Ranch Colony Canal while providing natural storage, improved wetland function, and potential incidental water quality improvements. Additionally, the purpose of the proposed features west of I- 95 (Pal-Mar East, Ranch Colony, Cypress Creek, and Gulfstream West) was to assist in reducing peak flows in the wet season and provide additional flow during drier periods. As shown in **Figure A-51**, the average monthly flow rates, measured at the confluence of the Cypress Creek Natural area and Ranch Colony Canal, are reduced in the wet season and either reasonably maintained or improved in the dry season.

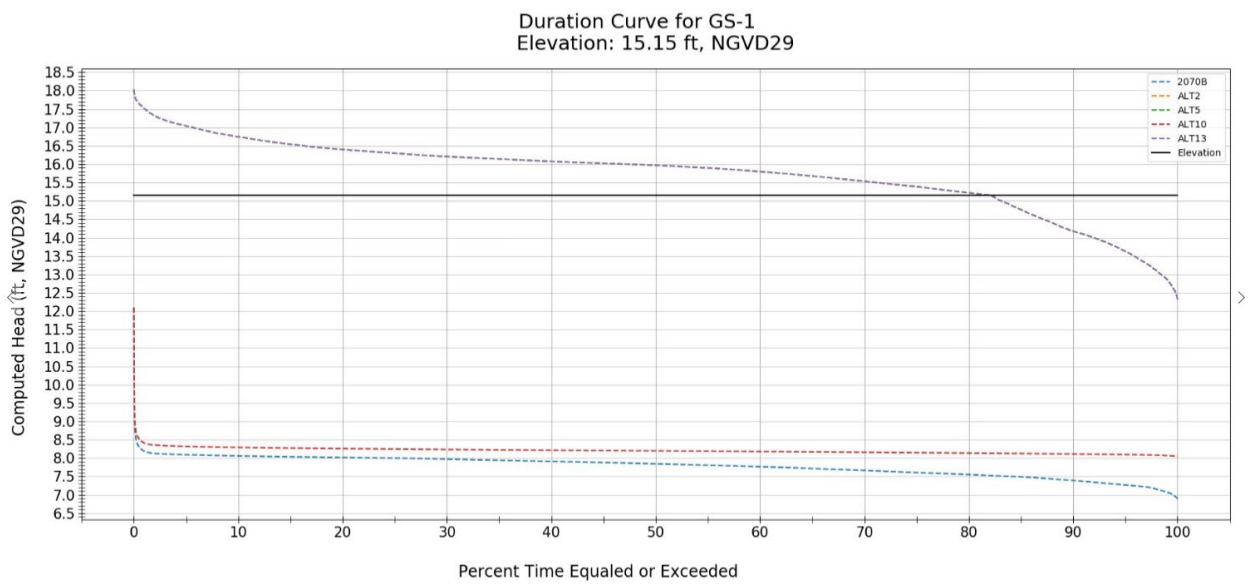


Figure A-50. Stage-duration curve for Gulfstream West.

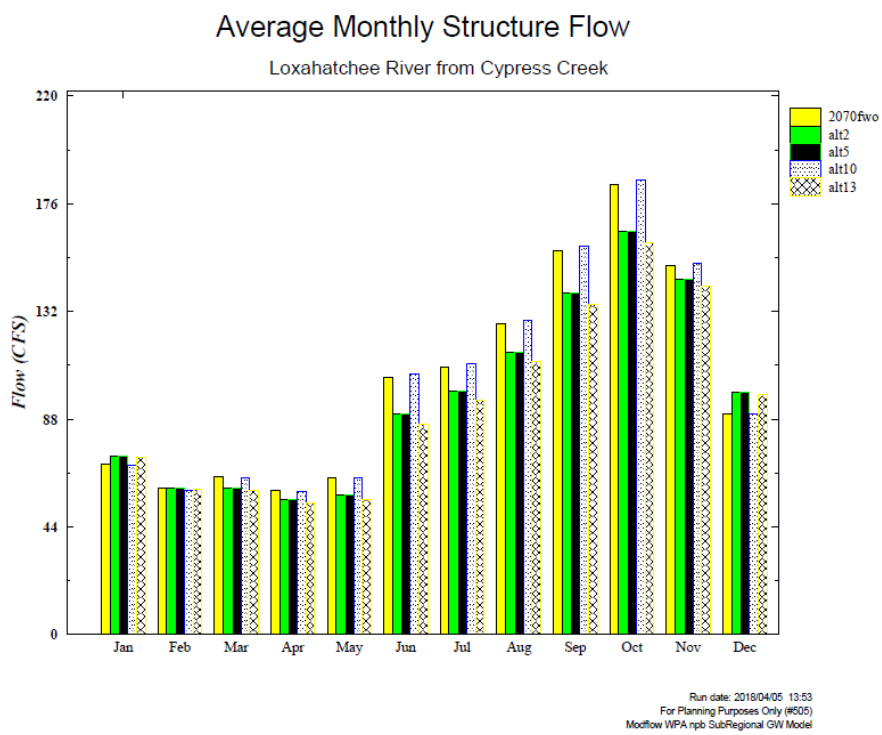


Figure A-51. Average monthly structure flow downstream of Cypress Creek Canal.

Gulfstream East and Moonshine Creek

The earthwork modifications to Gulfstream East, including the hydrologic connection to Moonshine Creek, were proposed to help improve hydroperiods within the area. **Figure A-52** and **Figure A-53** representing Gulfstream East and Moonshine Creek, respectively, demonstrate that the proposed improvements aid in restoring a more natural hydroperiod necessary for ecosystem restoration. Both locations experience a large increase in the depth and duration of inundation compared to the FWO project conditions.

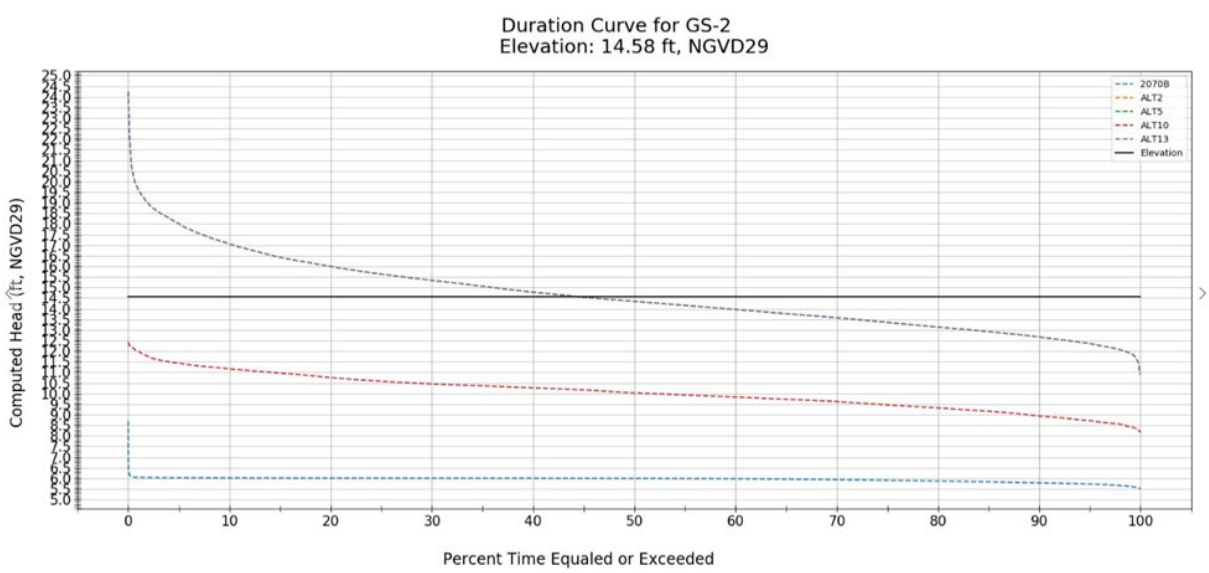


Figure A-52. Stage-duration curve for Gulfstream East.

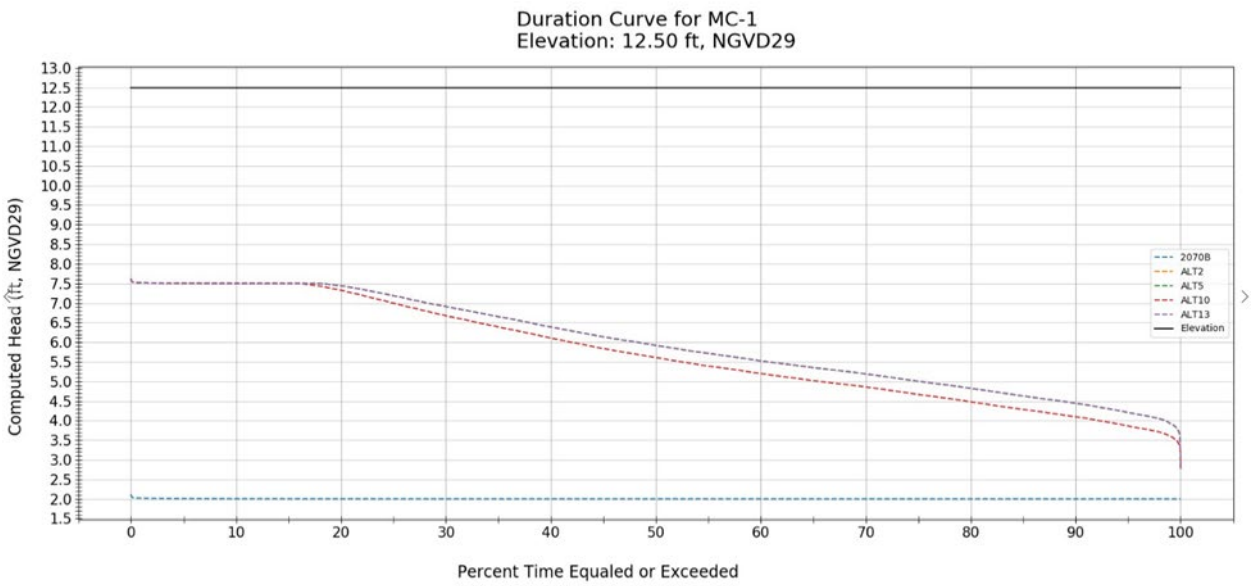


Figure A-53. Stage-duration curve for Moonshine Creek.

Kitching Creek

Similar to other locations within Flow-way 3, the Kitching Creek modifications were proposed to help rehydrate areas surrounding Jenkins Ditch and create a more natural hydroperiod for the upstream portions of Kitching Creek. The cell for KC-1.2, shown in **Figure A-54** represents the surrounding area of Jenkins ditch and demonstrates that a greater water surface elevation can be achieved throughout almost the entire simulation period due to the features proposed in the Recommended Plan (also proposed in Alternative 2 and 13). This increase in stage correlates to improved floodplain inundation and hydrology regimes, aiding the restoration of wetland habitat.

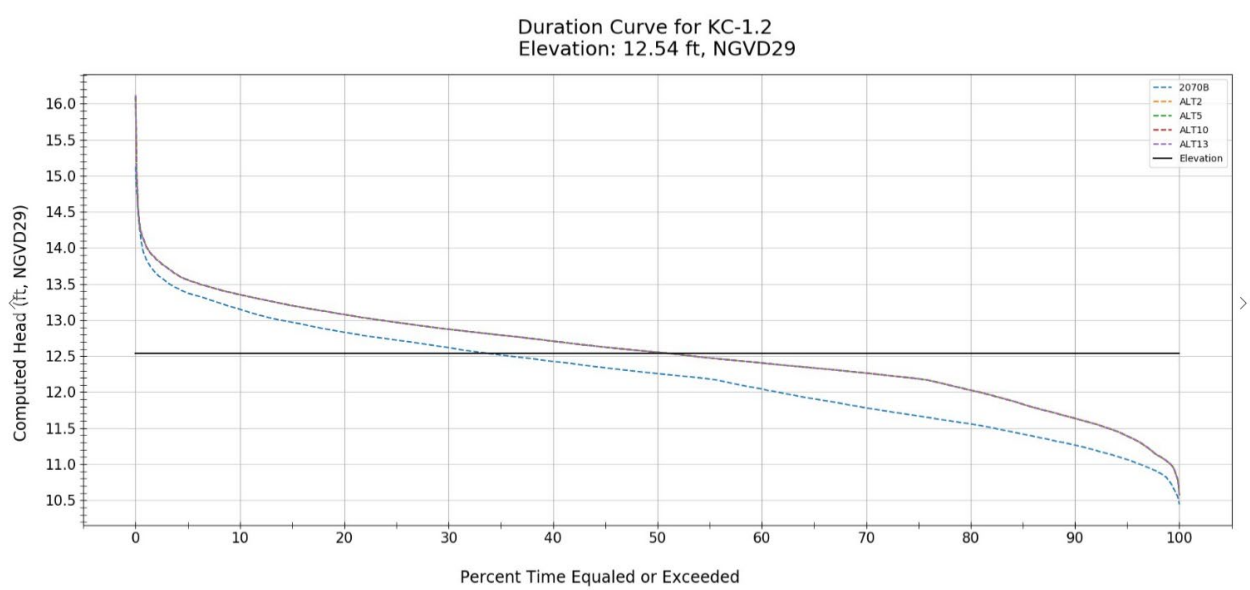


Figure A-54. Stage-duration curve for Kitching Creek.

A.11.3.2.11 Key Performance Measure Locations

As specified in **Section A.1** Project Objectives, each alternative is evaluated based on how well it met the performance measures, evaluation criteria, and other metrics identified by the PDT. Locations of importance within the model domain that are either directly or indirectly related to the PM's and EC include the following locations: Lainhart Dam, S-46, and the total contributions to the Northwest Fork of the Loxahatchee River.

Lainhart Dam was identified as a location that preferred restoration flow targets shall be met as part of the project objectives and specifically to meet performance measure 1. The performance at Lainhart Dam can be seen in **Figure A-55**, which illustrates the improvement of flows over the future without condition for the various alternatives. Alternative 10 shows the greatest improvement in flow, likely due to the C-51 outflow and force main combination. Alternative 5 and 2 preform similarly, with Alternative 5 showing minor improvement over that of Alternative 2 during lower flow conditions due to the increased reservoir size and addition of ASR wells. Alternative 13 consecutively performs worse than the other alternatives but does show improvement over that of the FWO condition.

Table A-34 compares the percentage of time the wet and dry season target at Lainhart are met for the ECB, FWO, and Alternatives 2, 5, 10, and 13. The table illustrates that Alternative 5 and 10 are the best performing alternatives for achieving the desired Lainhart flow rates during the wet and dry season.

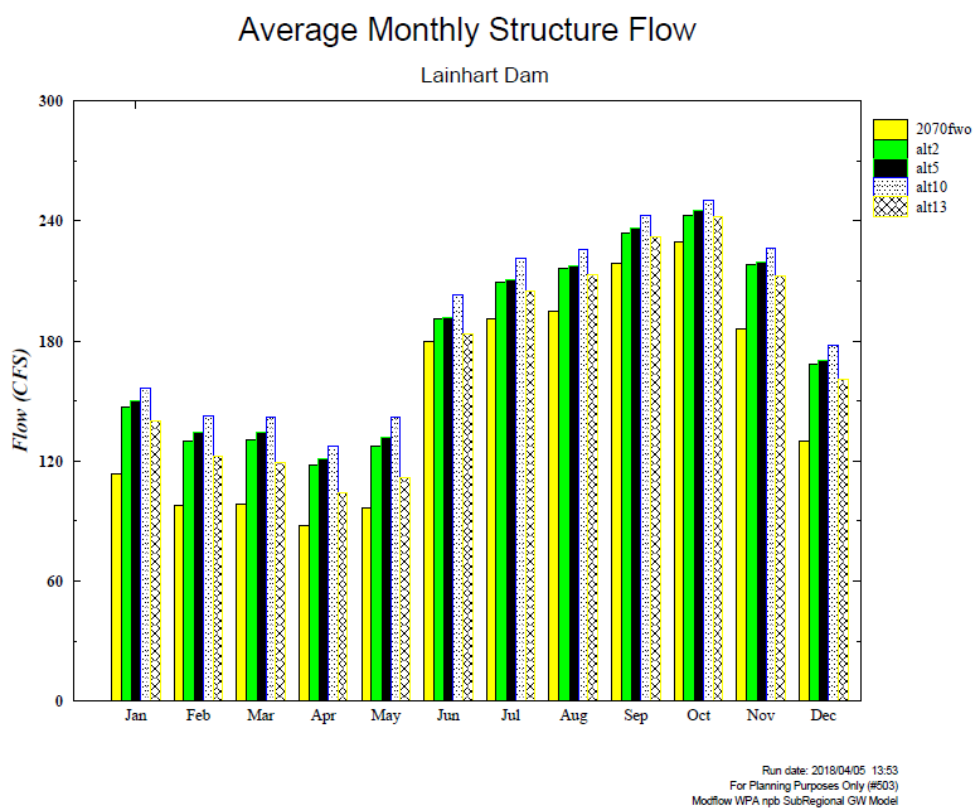


Figure A-55. Average monthly structure flow for Lainhart Dam.

Table A-34. Percentage of time wet and dry season targets are met.

Season	ECB	FWO	Alt 2	Alt 5	Alt 10	Alt 13
Wet season	76%	78%	98%	98%	100%	93%
Dry Season	65%	65%	87%	91%	95%	75%

The construction of the C&SF Project S-46 Discharge Structure for flood control has resulted in increased discharges into the Loxahatchee Estuary. Lack of storage in the tributary basins results in periods of excessive freshwater inflow, followed by periods of insufficient freshwater inflow to the estuary. The net result is an unnatural oscillation of salinity conditions. Large pulsed releases cause impacts to seagrasses, shellfish populations, and other fish and invertebrate communities residing in the estuary. The percent reduction in frequency and duration of S-46 structure peak flow discharges that relate to salinity threshold criteria for seagrasses is directly related to the planning objectives. Additionally, a more regulated release or fewer large releases from S-46 correlates to a more controlled system upstream or that additional water is being sent to the Northwest fork of the Loxahatchee River. **Figure A-56** illustrates that the flows sent to S-46 are similar between the Alternative conditions and the FWO condition. Additional optimization of the features may be required to realize additional benefits at the S-46 structure.

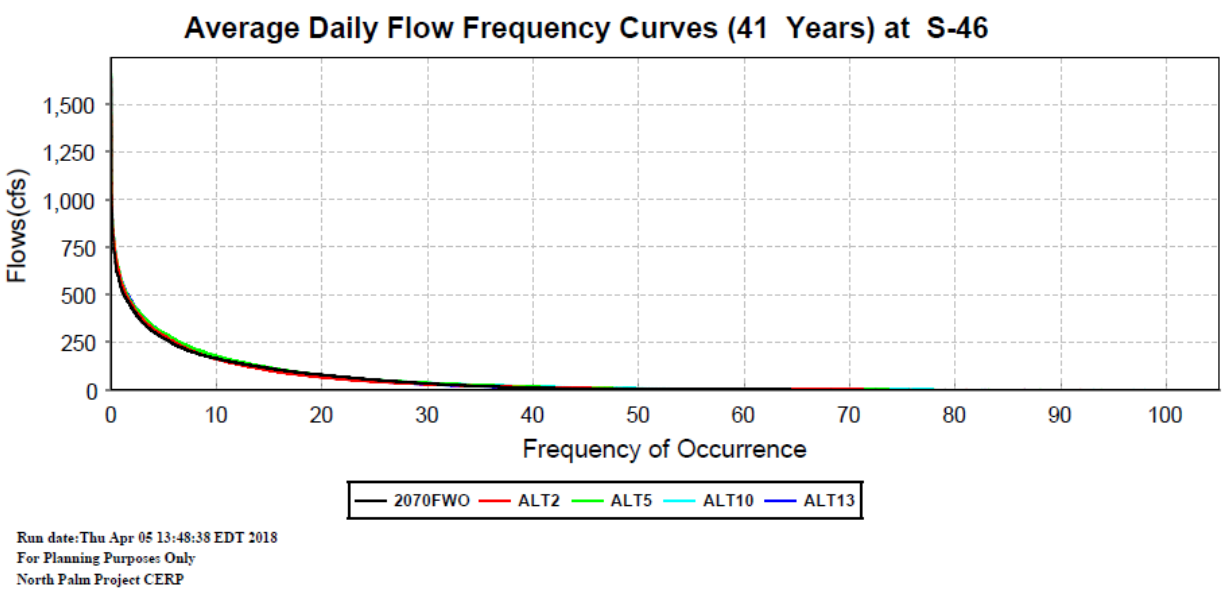


Figure A-56. Average daily flow frequency curve for S-46.

Finally, **Figure A-57** shows the average daily contribution for Lainhart Dam, Cypress Creek, Hobe Grove Ditch, and Kitching Creek to the Northwest Fork of the Loxahatchee River. From Lainhart, average flow values are expected to increase due the implementation of the alternatives. Cypress Creek, Kitching Creek, and Hobe Grove display lower daily flow values reaching the Northwest fork due to the water being dispersed to aid in increasing the natural extent of the wetlands while improving the hydrology and hydroperiods within these areas. Decreased flows from these areas are a hydrologic improvement as these tributaries are currently very flashy in nature with high uncontrolled discharge rates that can cause adverse impacts to the river.

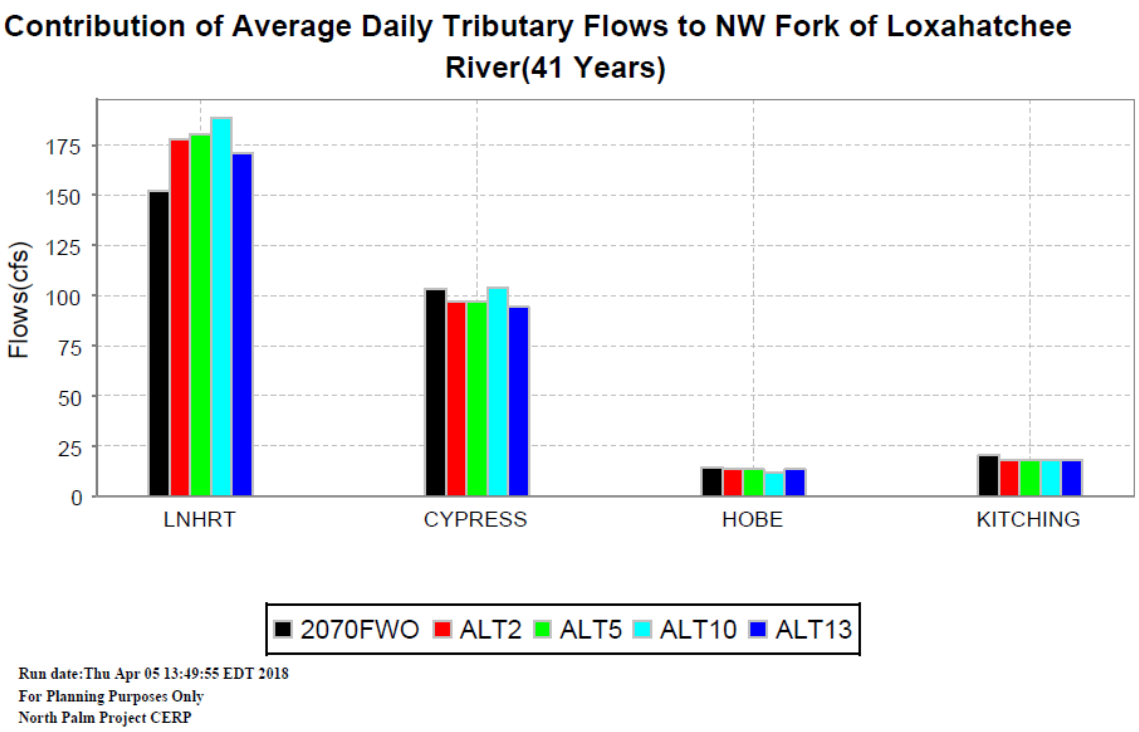


Figure A-57. Total average daily flow to the Northwest Fork of the Loxahatchee River

A.11.4 Tentatively Selected Plan

The LRWRP team identified Alternative 5 as the Tentatively Selected Plan and ultimately the Recommended Plan. Stakeholders requested the team look at revisions to Alt 5 to improve its performance, thus additional wetland features (Mack Dairy Spreader and re-grading of Cypress Creek Natural Area), were included as an additional Alternative iteration, Alternative, 5R. For more information about the analysis that resulted in the selection of Alternative 5R as the Recommended Plan, see Plan Formulation Section in the LRWRP Main Report. Due to the additional coordination involved and expedited plan formulation process, the LECSR-NP model was not re-run for Alternative 5R. Performance of the revised alternatives were inferred from the performance of the original, modeled alternatives Alt 5R has a 10.2% increase of average annual watershed wetland benefits for only a 5.5% increase of average annual cost, therefore it was determined to be the most cost effective alternative and was selected as the Recommended Plan.

A.11.5 Design Refinements of the Recommended Plan

The modeling software tool, HEC-RAS, was used to aid in the design of hydraulic features, evaluate conveyance and structural modifications, and aid in the determination of Savings Clause for features identified with the LRWRP Recommended Plan. Detailed documentation of the LRWRP hydraulic design methods and results are discussed in the Hydraulic Design, **Annex A-1** and Hydrologic and Hydraulic Modeling, **Annex A-2**.

A.11.5.1 HEC-RAS Model

The HEC-RAS modeling system is designed to perform one-dimensional and two-dimensional hydraulic calculations for multiple types of river analysis. HEC-RAS models were constructed for the Flow-way 2 and Flow-way 3 areas and assisted in accomplishing the following:

- The HEC-RAS model was used to 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 used as 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 other. Additionally, HEC-RAS can be applied to determine stages in the project area for the Recommended Plan versus the ECB and FWO condition.

- 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.

Due to the limitations of LECSR-NP model in the Flow-way 3 area, this is the primary alternative means for evaluating Savings Clause (flood protection) to residential communities resulting from project improvements. Additionally, the HEC-RAS modelling software was also used to perform a dam break scenarios as well as preform design refinements within Flow-way 2 for the above ground reservoir. HEC-RAS modeling was not necessary in Flow-way 1.

For a more detailed description of the HEC-RAS model set-up, performance, and associated results see **Annex A-3**, HEC-RAS Model Documentation.

A.11.5.1.1 Conceptual Model

<u>Flow-way 2</u>
A 1D/2D HEC-RAS model was developed for Flow-way 2. The model domain extension was selected to include areas that could influenced by the proposed features in Flow-way 2. The northeast limit of the model coincided with State Road 710, east and south limits include the ITID residential areas, and west limit extends to the L-8 West and the watershed divide for the J.W Corbett WMA.

The proposed C-18W reservoir was included in the model as a storage area feature. Areas surrounding the reservoir were simulated as 2D areas in the HEC-RAS model. The 2D areas include proposed canals and levees, the J.W Corbett WMA, Sweetbay natural area, the ITID, and Hungryland slough natural area. Existing C-18W canal and part of the M-O canal were included in HEC-RAS as 1-D reaches.

The HEC-RAS model was developed for unsteady flow conditions. Direct Rainfall for the 100-year flood event for 72 hour duration was applied to the 2D areas and the C-18W reservoir, no initial abstraction was calculated, as a more conservative approach. A synthetic hyetograph was used as input in the model for the 2D areas with a 100-year rainfall total of 16.3 inches.

Flow-way 3

The model domain includes the proposed western features of Flow-way 3, including the Cypress Creek Canal structure, Gulfstream West modification for the Flow-through Marsh, Water Control structure modifications near Culpepper, Nine Gems area, and the Mack Dairy Spreader system. The purpose of the model is to refine the engineering design of the proposed features to ensure that when implemented together, increases within the Cypress Creek Canal are not exceeded over that of existing conditions. Stages within Cypress Creek Canal are of importance because overtopping of the canal can cause both flooding and neighborhood drainage issues, as the outlet structures for the neighborhood depend on a particular tail-water stage within the canal. The model was constructed using both 1D/2D for existing conditions and the with-project condition. Model runs were simulated for the 100-year, 72 hour storm event for design purposes, utilizing the maximum permitted discharges for the urban areas and HSLCD drainage and direct rainfall (representing the design frequency) for the remaining project areas, all of which were constructed as a 2D grid.

Two additional HEC-RAS models were built for the eastern portion of Flow-way 3 for the Gulfstream East and Kitching Creek area to aid in the determination of appropriate hydraulic structure design and earthwork needs. The Gulfstream East model included the downstream portions of HSLCD unit 3 canal and discharge structure, Hobe Grove Ditch, and the historic Moonshine Creek area to the south. The Kitching Creek model domain included a downstream reach of Jenkins Ditch, two existing culverts, and the Johnathan Dickenson State Park.

A.11.5.1.2 Baseline Condition Modeling

Flow-way 2

The Flow-way 2 model was developed to simulate the design storm condition for the 100-year rainfall, 72 hour duration event. Computed water surface elevations, velocities and flows were used to support the hydraulic design of the proposed structures, the levee superiority, and channel and structures protection features.

Flow-way 3

The baseline conditions within the three HEC-RAS models for Flow-way 3 were representative of 2014 conditions and were used as a basis of comparison to evaluate against with-project performance, similar to what was performed in the LECSR-NP model. The models were constructed using LiDAR from Martin County (2016), as-builts, survey data, permitted discharge information and other data as necessary. The baseline model includes current system infrastructure assumptions and current permitted allocations.

The models were designed to simulate design storm conditions and do not include the necessary hydrologic and climatological conditions for long-term simulations. Therefore, the model simulation period was approximately four to five days, three days for the design storm with the necessary time after to observe the receding limb of the hydrograph throughout the modeled area.

Under the baseline conditions, the HEC-RAS models computed the expected water surface elevations, velocities, and flow-rates for the selected design storm. These simulations are necessary to compare how the system response changes with the project features to determine the appropriate hydraulic design of structures and sizing of hydraulic features to ensure, at minimum, the same level of service for flood protection is maintained.

A.11.5.1.3 Calibration and Validation

Flow-way 2

The HEC-RAS model for Flow-way 2 was not calibrated to existing conditions as there are minimal to no data available for calibration.

Flow-way 3

Limited observed data is available within the proposed Flow-way 3 area. Calibration of the western Flow-way 3 model was performed using daily stage and flow measurements from gage 265818080111900 Cypress Creek Canal below Gulfstream Bridge, FL. The period of record was from October 2002 to September 2004. This duration coincided with a large tropical event, Hurricane Jeanne, to which the model was calibrated. Additionally, observed direct rainfall from the Jonathan Dickinson State Park Weather Station was applied to the 2-D areas within the model domain. To validate the model performance, an additional analysis for the design storm condition was run and compared with a previous modeling effort completed in 2016 by Tomasello Consulting Engineers, Inc. The purpose of the 2016 modeling effort was to evaluate operation of the Pal Mar/Ranch Colony Canal water management system. The Tomasello model was run using the 100-year, 72-hour storm event for the design of a proposed weir within the Cypress Creek Canal. The performance of the HEC-RAS model was compared to the S2DMM model at a specific location within the Cypress Creek Canal.

The HEC-RAS models representing the Gulfstream East and Kitching Creek Areas were not capable of being calibrated or validated as no stage or flow data is available. Boundary condition inflows were representative of maximum permitted discharge for HSLCD unit 3 inflow and an estimated bank full flow from Jenkins Ditch.

A.11.5.1.4 Recommended Plan Modeling

Flow-way 2

Proposed features for Flow-way 2 were included in the HEC-RAS model in order to simulate the proposed condition and to design hydraulic structures with sufficient capacity for the project function and avoid adverse impact in adjacent areas.

Proposed canals features for Flow-way 2 model includes levees, canals, pump stations and pump stations. These features are designed to control and convey runoff waters from the M-O canal to the proposed C-18W reservoir and the existing C-18W canal.

The C-18W Reservoir is the main component of Flow-way 2 and is designed to store water and release, when needed, downstream. Hazard Potential Classification (HPC) of the C-18W Reservoir was determined following the guidance in the DCM-1 – Hazard Potential Classification (SFWMD, 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 - (SFWMD, 2006), 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.

The embankment height, and freeboard were design following the guidance in the DCM-2 and DCM-3 – Spillway capacity and Reservoir Drawdown Criteria, respectively (2006). A wind and wave study for case 1 (PMP event combined with 100-yr wind condition) was performed to determine the embankment height. The selected over wash for the embankment height design was 0.1 cfs/lf.

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 of the 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 of 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 of 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 discharges 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.

Flow-way 3

The Recommended Plan components modeled in Flow-way 3 included all proposed features except for the earthwork in the lower portions of the Cypress Creek natural area and within the Gulfstream East. The earthwork estimates within these portions was completed using a targeted elevation for ecosystem restoration and civil site calculations were performed.

The preliminary design features for the Recommended Plan model were incorporated into the HEC-RAS model through manipulation of the terrain map layer, by modifying/adding structures, and by modifying canal cross sections. These features, as proposed in the LECSR modeling, were refined through multiple model iterations to ensure all the proposed project features functioned collectively during the design event and adverse impacts above that experienced in existing conditions, was not realized. Modeling the entire system and performing multiple model iterations was important as modifications to certain project features may cause impacts upstream or downstream.

The design for the flow-through marsh in Gulfstream West was incorporated into the model by smoothing the existing "dog-leg" of the HSLCD canal and extending a design cross-sectional area to create the new

north-south extension. The cross-sectional dimensions were similar to the existing canal capacity upstream but was enlarged slightly to ensure the appropriate amount of canal freeboard was maintained adjacent to the neighborhood. The flow-through marsh design was governed by two major features, the necessary embankment height and the discharge structure. The flow-through marsh water surface elevations were designed to be between approximately 2-3 feet in depth, with inflow occurring either from direct rainfall or the inflow pump station. The outflow structure was designed as notched weir able to discharge a variable rate depending on the marsh depth. The design of the embankment was as a Low Hazard Potential, as according to DCM-2 Low HPC (Hazard Potential Classification) impoundments. For this design a minimum of 3 ft of freeboard was required, in addition to the 100 yr-24 hour rainfall. Since the design storm selected for the remainder of the project was a 100 yr-72 hour event, it was also used for the embankment design of the flow-through marsh and is a more conservative approach. The spreader berms within the interior of the sites, as well as the inflow and discharge basins of the pump intake & discharge were also included in the terrain modifications. Additional refinements to the various interior spreader berms and pump discharge structure may be necessary during PED to ensure environmental constraints regarding depths and velocities are met.

Similar terrain adjustments were made for the Mack Dairy pump intake/discharge areas and spreader system. The spreader system geometry was modified to ensure the pump discharge rate would be dispersed through the spreader and into the Cypress Creek Natural area with minimal velocity.

Modifications to the southern Nine Gems canal and determination of locations and lengths of backfill along the canal (between Nine Gems and the rest of the Palmar) was also completed. Modeling of the Nine Gems area under existing conditions also illustrated uncontrolled discharge of the property over the existing containment berm to the southern Nine Gems canal. Uncontrolled discharge into this canal could cause failure of the containment berm and potential flooding impacts to the residents to the south. Therefore, a containment levee with new discharge structures to the remaining portion of the southern Nine Gems canal was proposed based on the modeling results.

Finally, the proposed design for the Cypress Creek Canal water control structure and canal modifications were also designed through multiple model iterations. The water control structure was modeled as an inline structure, with the spillway geometry being optimized through the use of spreadsheet models.

The model simulations for the Gulfstream East area included running multiple weir elevation and width scenarios to determine the appropriate configuration that did not impact the tailwater elevation of the HSLCD unit 3 discharge structure. Additionally, terrain modifications for the reconnection of the historic Moonshine Creek was completed using an estimated cross sectional area, invert, and slope necessary to help re-direct flow from the Hobe Grove ditch.

The proposed operable structure and spreader swale were modeled by running multiple gated culvert geometries that would achieve the following: cause no increase in water surface elevation adjacent to the residential properties, be able to pass the bankfull condition of Jenkins ditch, be located such that the elevations required for dispersion into the spreader swale would not cause adverse impacts to the drainage capability of Jenkins ditch.

Additional details regarding the modeling effort, results, and overall design refinements of the Recommended Plan can be seen in **Annex A-3**, HEC-RAS Model Documentation.

A.11.6 Summary of Results

HEC-RAS model simulations are presented in the following subsections.

A.11.6.1 Flow-way 2

The proposed embankment and spillway for the C-18W reservoir were designed for the PMP rainfall event following guidance in the DCM joint design guidance. The HEC-RAS modeling was used to design the proposed hydraulic structures with capacity for the 100-year, 72-hour duration rainfall event. Description of proposed features are included below.

Reservoirs

- The only reservoir included in Flow-way 2 is the C-18W reservoir. The reservoir is designed as an above the ground reservoir with a normal pool depth of 7.5 ft (28.0 ft, NAVD88) and volume of 9,500 acre-ft.
- The proposed D-101 embankments has a design elevation of 40.5 ft NAVD88 with an average toe elevation of 20.5 ft NAVD88. The reservoir was designed with two outlet structures, uncontrolled spillway S-101B and gated culvert S-102.
- Spillway S-101B has a dual function as a service spillway for permitted discharges and auxiliary spillway for auxiliary discharge (emergency). Service spillway elevation was set to 28.1 ft NAVD88 with a length of 10 ft and maximum discharge capacity of 105 cfs. The auxiliary spillway elevation was set to 29.3 ft NAVD88 with a length of 80 ft with, and maximum discharge capacity of 1,960 cfs.
- During regular reservoir operations (non-flood event), the C-18W Reservoir discharges using the S-102 gated culvert. It is composed by two (2) 4 ft x 6 ft box culvert. This structures was designed to not operate during flood events and it was not included in the HEC-RAS modeling.

Canals

- Proposed canals include the seepage collection canals C-101E and C-101W and the inlet/outlet canal C-101N. Existing canals included the M-O canal located southwest of the C-18W reservoir and the C-18W canal, located north of the reservoir.
- The proposed C-101W canal was designed to convey pumped water from the M-O canal during normal operations, it also collects 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 canals drain into the proposed C-101N canal. The normal direction of flow within the seepage collection canals is from south to north and eventually drain into the existing C-18W canal.
- Seepage collection canal C-101E was designed with an invert elevation of 5 ft NAVD88, a bottom width of 20 ft and channel slopes 3H:1V. The south portion of the C-101W canal, which conveys water from the M-O canal, was designed with a bottom width of 10 ft and channel slopes 2.5H:1V.
- The C-101N canal was designed with an invert elevation of 5 ft NAVD88, a bottom width of 50 ft and channel slopes 3H:1V.

Culverts

- Several culverts are proposed for the Flow-way 2 within the C-18W Reservoir area. 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 canal C-101W. The S-103W and S-103E culverts, located at the northern terminus of the seepage canals, are gated controlled structures that discharge to the outflow canal C-101N. Culvert S-104, located south of the C-18W reservoir, is an ungated culvert that provides access to the reservoir area. Runoff from the J.W. Corbett WMA is drained using the S-107 structure, which is composed of a gated weir connected to culverts. Additional culvert data are shown below **(Table A-35)**.

Table A-35. Design characteristics for proposed culverts at the C-18W Reservoir.

Structure Number	Culvert Location	Control	Size	Design Capacity
S-102	C-18W Embankment	Gated	(2) 4 ft x 6 ft	300 cfs
S-103E	Seepage collection canal (C-101W)	Gated	(1) 10 ft x 10 ft	500 cfs
S-103W	Seepage collection canal (C-101W)	Gated	(1) 10 ft x 10 ft	500 cfs
S-104	Seepage collection canal (C-101E)	Ungated	(1) 10 ft x 10 ft	500 cfs
S-105	Levee L-101W	Flap Gate	(3) 6.5 ft	150
S-107	Levee L-101W	Gated	(3) 6.0 ft	300

Pump Stations

- Three (3) pump stations are proposed for the project area, pump station S-101A, S-106 and S-109. Pumps S-101A and S-106 pump water into the C-18W reservoir to be store for releases when needed. Pump S-106 also contains pumps for seepage control from the C-18W Reservoir. Pump station S-109 transfers' water from the M-O canal into the C-101W canal. The pump stations were assumed to not be operating for the evaluated flood events.

Levees

- The proposed features for flow-way 2 include two levees. Levee L-101W is located west of the reservoir and is designed to control runoff discharges from the J.W Corbett WMA. Levee L-101E is located north in the right embankment of canal C-101N. The levee was designed to have a top elevation of 25 ft NAVD88 with slopes 3:1 (H:V), providing a superiority of 2 ft.

A.11.6.2 Flow-way 3

The modeling results for all portions of Flow-way 3 helped determine the appropriate design to pass either the 100-yr, 24-hour event or existing upstream discharge rates (permitted or otherwise). A brief

description of the project components, as determined through the HEC-RAS modeling effort is included below.

Gulfstream West Flow-through Marsh

- Inflow pump station (S-110) will contain two 25 cfs and two 50 cfs pumps
- Construct embankment (L-110), to a crest elevation of 20 ft NAVD88, 14 ft top width and 3H:1V side slopes. Embankment will be approximately 25,000 feet in length.
- Re-grade interior portion of the flow through marsh by filling existing agriculture ditches and overturning any citrus mounds. The purpose of the re-grading is to promote uniform sheet flow from the pump station discharge structure to the outlet structure. The re-grading and cut/fill volume calculations for balancing construction material needs in the area generated the average design interior elevation of approximately 13.25 ft NAVD88.
- The existing "dog-leg" of HSLCD canal will be filled. The existing cross section area is estimated at approximately 700 ft^2 with a length of 5,300 ft.
- Three interior spreader berms shall be constructed at elevation between 14-15 ft NAVD88 to reduce the formation of preferential flow-paths and help redistribute flow throughout the marsh.
- The pump station will discharge into a concrete plunge pool basin (S-111N) and will be 10 feet wide (parallel) and 50 feet long (perpendicular to flow). The bottom elevation will be at 12.5 ft. NAVD88 whereas the top of plunge pool (weir) will be at 14.5 ft NAVD88. Concrete or riprap at the bottom of the plunge pool as well as in the transition from spreader weir to flow-through will be required.
- Discharge structure (S-111S) is a notched weir that will discharge a variable rate up to 30 cfs in the first notch and 330 cfs within the second notch. The third notch was designed for extreme events above the 100-year, 72-hour storm.

New southern HSLCD Canal (replaces existing "dog-leg" portion)

- The design cross sectional parameters are the following: 20 ft bottom width, 2.5H:1V side slopes, 2,650 ft in length, starting invert elevation of channel is 3.3 ft NAVD88, downstream elevation at confluence with Cypress Creek Canal 2.0 ft NAVD88.
- It is necessary to fill existing drainage canal adjacent to the neighborhood to EL 17.0 ft NAVD88 with a minimum 10 ft top width. All excess material should be used in the filling of this drainage ditch or for the Gulfstream West Flow-through Marsh.

Mack Dairy Spreader (C-114), Pump Station (S-113N), and Discharge Structure (S-113S)

- Pump station will consist of two 10 cfs pumps
- Spreader swale approximately 4,900 feet in length will be constructed with the following cross sectional geometry: 5 ft bottom width, 4H:1V side slopes, bottom elevation is 15 ft NAVD88 near pump station discharge structure and 14.5 ft NAVD88 at the last cross section (0.001 ft/ft channel slope). Termination of the swale should occur over a length of approximately 100 feet to natural grade. Average depth of swale is 2 feet below natural grade.

- Containment berm shall have a 3H:1V side slope, 7 foot top width and top elevation of 19 ft NAVD88.
- Swale and berm will tie into the existing grade with the left channel bank being constructed to follow a uniform contour elevation, when practical. Final swale alignment will be confirmed during PED.
- A plunge pool will be required for pumping into as a discharge basin to disperse the flow into the spreader swale via a spreader weir. Plunge pool dimensions will be 10 feet wide (parallel), 20 feet long (perpendicular to flow). Plunge pool will tie into embankment, with riprap protecting both the upstream and downstream face of the constructed embankment. Rip-rap is required at the bottom of the plunge pool as well as in the transition from spreader weir to swale.
- Modification of approximately 150 ft of Cypress Creek Canal Berm for inlet bay to pump station.

Nine Gems Modifications (L-111, and new control structures S-114A, B and C)

- Backfill existing southern Nine Gems canal and remove berm at three locations, approximately 50 feet in length each. Existing berm material (on both sides of canal) shall be used to fill in canal. These locations were selected based on topography and trying to re-establish sheet flow across lower lying wetland areas. Final berm removal and canal backfill should be graded to match wetland elevations, approximately at 19.0 ft NAVD88 elevation.
- Replace existing water control structures along the three eastern most Nine Gems canals (that run in a North-South direction). Removal of the existing culverts will be necessary due to their degradation and replacement with larger, telemetry operated structures (S-114A through C) is necessary.
- Backfill and berm removal to occur within the central portion, 2,500 ft in length, of the three western North-South Nine Gems drainage canals. Target elevation for each backfill shall match the existing topography in the area to promote sheet flow from west to east.
- A new berm (L-111) shall be constructed along the existing road/containment berm to an elevation of approximately 21.3 ft NAVD88, with 12 ft top width and 3H:1V side slopes. This new berm shall also cross the Nine Gems canal and tie into the Culpepper berm.

Cypress Creek Canal Expansion and Water Control Structure (S-112)

- Expansion of approximately 4,600 ft of canal necessary between new HSLCD canal and downstream of S-112. Channel cross section will contain a 30 foot bottom width with 2.5H:1V side slopes. Existing side slopes are 2H:1V and channel modifications should tie into existing channel as practical. Cross section shall be expanded to a 35 ft bottom width approximately 400 feet upstream and downstream of the proposed Cypress Creek Discharge Structure, S-112.
- The S-112 structure was designed as a two-bay ogee spillway with a crest elevation of 6.5 ft NAVD88 and net crest length of 32 ft.
- Existing bridge shall be removed and S-112 will provide access from the Cypress Creek Natural area to the Gulfstream West property.
- Additional expansion of the upstream portions of Cypress Creek Canal will also be necessary to ensure flood protection from the design storm is maintained.

Hobe Grove Ditch Weir (S-117)

- A sheetpile weir (S-117), at elevation 6.0 ft NAVD88 with a 40 ft length is to be constructed at the downstream end of Hobe Grove Ditch. To establish reconnection between Moonshine creek and Hobe Grove Ditch, the connection should be excavated to 4 ft NAVD88 for approximately 1,500 ft in length at an approximately 5 ft bottom width and 3H:1V side slopes.

Jenkins Ditch Control Structure (S-116) and Spreader System (C-116) for Kitching Creek

- The new water control structure (S-116) will be a 5 ft by 7 ft box culvert and will replace an existing elliptical corrugated metal pipe culvert.
- The spreader swale will be approximately 2,000 ft in length with a 5 ft bottom width, 2 ft depth, and 4H:1V side slopes.

Additional design details and results of the hydraulic modeling effort can be found in **Annex A-1**, Hydraulic Design and **Annex A-3**, Model Documentation Reports.

A.11.7 Saving Clause: Flood Control

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. Refer to Annex B Savings Clause WRDA 2000 for more details.

A.11.8 Identification of Additional Hydrologic Modeling for Pre-Construction Engineering and Design

The PDT determined LECSR-NP was acceptable to support the preliminary screening, alternative formulation, and evaluation of LRWRP alternatives. Additionally, the HEC-RAS model has provided additional details regarding the hydraulic design of the proposed structures. However, more detailed physically-based modeling with higher resolution including hydrologic and hydraulic coupling, updated survey data, and additional data collection 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. A semi-quantitative dam risk assessment will be performed during PED. A quantitative dam risk assessment will be performed following PED before first fill of the C-18W Reservoir and completion of the Emergency Action Plan (EAP). Below, please find a minimum list of project components that may require further hydrologic and hydraulic analyses and/or modeling during PED:

- Intake canals and seepage canals;
- Culverts, weirs, and spillways;
- Pump stations;
- Wetland areas.

All proposed hydraulic structure, canals, and pump stations will require optimization during PED for all Flow-ways. A specific list of tasks that should occur during PED include:

Flow-way 1

1. Survey data collection within the M-1 Canal and M-Canal at the M-1 Pump Station proposed location.

Flow-way 2

1. Survey data within the C-18W Canal
2. Stage recordings within the M-O eastern canal extent.
3. Coupled Hydrologic and Hydraulic model for the J.W. Corbett WMA for further design refinements of the L-101W and S-107.

Flow-way 3

1. Coupled Hydrologic and Hydraulic model for the Pal-Mar East watershed for further design refinements of the L-111, S-114A, S-114B, S-114C.
2. Additional Calibration/Validation of the HEC-RAS Flow-way 3 Model (western project features).
    a. Collection of flow and/or stage data, at a sub-daily time step, within the Flow-way 3 project area, specifically within the HSLCD drainage canal downstream of the Unit 2 structure and S-114D (Southern nine-gems canal) as well as within the Cypress Creek Canal near the propose S-112 structure.
    b. Survey data collection within the HSLCD canal and Cypress Creek Canal, as current HEC-RAS Model geometry is based off a combination of LiDAR and survey data from 2000.
    c. Survey data of existing structures within the HSLCD canal, including S-114D and S-114E.
3. Calibration data for the HEC-RAS Flow-way 3 Model (eastern project features).
    a. Defined operational conditions of the HSLCD Unit 3 water control structure and permitted discharge or design discharge conditions within the Jenkins Ditch.
        i. Or measured flow and/or stage data within both the Hobe Grove and Jenkins Ditch.
    b. Survey data for the Hobe Grove and Jenkins Ditch cross sections.
    c. Survey data for the Jenkins Ditch water control structures.

A.12 Operations and Maintenance

Operation, Maintenance, Repair, Replacement and Rehabilitation (OMRR&R) begins after project construction and Operational Testing and Monitoring is complete and generally includes all operation activities and maintenance needed to keep the project features functioning as intended. OMRR&R for the LOWRP project will occur for all new facilities constructed as a result of the project, and as an increase to the OMRR&R for State Facilities that LRWRP will use. The OMRR&R costs are included in the main report. The location of boat ramps and access ramps for O&M purposes will be determined during PED.

Activities included in the OMRR&R costs will be:

- Pump and facility maintenance which are per manufacturer's recommendations and schedules.
- Repair and rehabilitation of pumps, drivers, and switchgear are assumed to be rehabilitated or replaced once during the 50-year life.
- Erosion control to make sure banks and areas around culverts and other structures are not compromised by weather, plants, or animals.
- Mowing to maintain grass areas for a neat and clean appearance and also to make sure there are no other maintenance issues being hidden by high grass vegetation. Mowing also reduces the ability of woody plants to gain a foothold and lead to larger issues.
- All monitoring, required by permit, USFWS Incidental Take Statement, and/or needed to adaptively manage the Project.
- Invasive, exotic, native, and nuisance vegetation control. Vegetation control is done both to control underwater infestations and surface infestations.
- Adaptive Management (AM) measures needed to ensure project benefits or avoid violating one or more project constraints.

A.12.1 State Facilities Used by LRWRP

The future OMRR&R costs of operating the system without LRWRP features once LRWRP is constructed and operational will be based on new water flows through the state facilities as a portion of the overall water flows through the state facilities. Please see **Section 6**, Recommended Plan, of the main report for cost-sharing information.

Table A-36. List of state-associated infrastructure.

Structure	Design Capacity	Description
G-160	2,000 cfs	Two-bay gated spillway
G-161	150 cfs	Gated culvert with two 60-inch diameter barrels and total length of 240 ft.

A.13 Cost Engineering

The certified Cost Estimate is shown in **Appendix B** of the PIR-EIS. This cost estimate is based on the elements and structures described in this Engineering Appendix and **Annex C-1** Plates.

A.14 Cost Schedule and Risk Analysis

A Cost Schedule & Risk Analysis Workshop for LRWRP was conducted on July 10, 2019. A Value Engineering Study will not be performed during the feasibility phase of this project.

A.15 Engineering Plates

Engineering plates are provided in **Annex C-1** to this Engineering Appendix.

A.16 Engineering Appendix Support Documents

Engineering Appendix support documents are provided in **Annexes A-1** through **A-3**, Hydrologic and Hydraulic Modeling and Design, **Annex B-1,** Geotechnical Investigations**,** and **Annex G-1**, Geological Investigations**.**

A.17 References

Anderson, S., 2008. Hydrogeologic Investigation of the Floridan Aquifer System L-8 (PBF-15), Palm Beach County, Florida. SFWMD Technical Publication WS-25, 136 p.

Arcadis Design and & Consultancy, 2016. Mecca Impoundment Palm Beach County, Florida. Work Order No. 4600003011-WO01. Final Design Documentation Report. Variously paginated.

Cedergren, H.R. (1989) Seepage, Drainage, and Flow Nets. Wiley-InterScience, John Wiley & Sons, New York.

CH2MHill, 1985. Drilling and testing of the deep injection and monitoring wells, Pratt & Whitney wastewater treatment Plant. Engineering Report submitted to United Technologies, Pratt & Whitney, West Palm Beach, FL. 505 p.

CH2MHill, 1988. Drilling and testing of the deep injection and well System at the Village of Royal Palm Beach. Engineering Report dated August 1988 submitted to Craig A. Smith & Associates, 430 p.

CH2MHill, 1989. Drilling and testing of the deep injection well and monitor well. Engineering report dated June 1989 submitted to Seacoast Utilities Authority, Palm Beach Gardens, Florida. 190 p.

CH2MHill, 1998. Construction and testing of the aquifer storage and recovery facility at the West Palm Beach Water Treatment Plant. Engineering report dated February 1998 for the City of West Palm Beach, variously paginated.

Dames & Moore, 1989. Phase I Environmental Site Assessment, Sunrise Boys

Dames & Moore, 1992. Phase II Environmental Site Assessment, Sunrise Boys

Dames & Moore, 1997. Phase III Environmental Site Assessment, Sunrise Boys

Ecology and Environment, Inc., 2009. Nine Gems Restoration: Field Studies and Recommendations Report. Report prepared for SFWMD dated February 2009, 29 p.

Foster Wheeler Environmental Corporation, 2002. Phase I/II Environmental Site Assessment for Sunrise Boys, LLC and Indiantown Realty LLP Properties.

Gannett Fleming, Inc., 2016. Final Geotechnical Data Report for the J.W. Corbett Wildlife Management Area Hydrologic Assessment. Report prepared for South Florida Engineering and Consulting, LLC, dated 28 January 2016, 71 p.

Giddings, J.B, Obeysekera, A.N. (2018) Model Application Report for Loxahatchee River Watershed Restoration Project. South Florida Water Management District.

Harbaugh, A.W. and M.G. McDonald. 1996. Documentation for MODFLOW-96, and Update to the U.S. Geological Survey Modular Finite-Difference Ground Water Flow Model. Techniques of Water-Resources Investigations of the USGS. Open-File Report 96-485, USGS, Reston, VA.

Hunt, Roy. 2005. Geotechnical Engineering Investigation Handbook. 2nd Edition. Boca Raton, FL: CRC Press.

Kuebler, L.L., 2010. Model Documentation for the Application of the Northern Palm Beach County Version of the Lower East Coast sub-Regional Model (LECsR-NP). South Florida Water Management District, West Palm Beach, Florida.

Lukasiewicz, J., and Smith, K.A., 1996. Hydrogeologic data and information collected from the surficial and Floridan Aquifer systems, Upper East Coast Planning Area –Part 1 Text. South Florida Water Management District Technical Publication 96-02, 244 pages.

Miller, J.A., 1986. Hydrogeologic framework of the Floridan aquifer system in Florida, and in Parts of Georgia, Alabama, and South Carolina. U.S. Geological Survey Professional Paper 1403-B, 91 p. plus 33 plates.

Miller, W.L., 1987. Lithology and base of the surficial aquifer system in Palm Beach County, Florida. USGS Water Resources Investigations Report 86-4067, 1 page.

Missimer & Associates, 1993. US Sugar Corp. ASR test well construction report. Report dated January 1993 submitted to South Florida Water Management District, 131 p.s

Montoya, A., Kuebler, L., Radin, H., Mullen, V. (2010) Model Implementation of the Northern Palm Beach County Version of the Lower East Coast Subregional Model (LECsR-NP). South Florida Water Management District.

NRCS (Natural Resources Conservation Service), 2019. Soil Maps of C-18W reservoir footprint, Gulfstream West and Flow-way 3 defined using the Web Soil Survey App accessed at https://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx on 13 June 2019.

Obeysekera, A.N., Giddings, J.B, and Bandara, D.U. (2018). Model Calibration Report for Lower East Coast Subregional Model - North Palm Version. South Florida Water Management District.

Professional Service Industries, 2012. Phase II Environmental Site Assessment, Mecca Farms Property.

Reese, R.S., 2004. Hydrogeology, water quality, and distribution and sources of salinity in in the Floridan Aquifer System, Martin and St. Lucie Counties, Florida. USGS Water-Resources Investigation Report 03-4242, 96 p. plus appendices.

Reese, R.S., 2006. Hydrology and Aquifer Storage and Recovery performance in the Upper Floridan Aquifer, southern Florida. US Geological Survey Scientific Investigations Report 2006-5239, 110 p.

Reese, R.S. and Memberg, S.J., 2000. Hydrogeology and the distribution of salinity in the Floridan Aquifer System, Palm Beach County, Florida. US Geological Survey Water-Resources Investigations Report 99-4061, 52 p. plus appendices.

Reese, R.S. and Richardson, E., 2008. Synthesis of the hydrogeologic framework of the Floridan Aquifer System and delineation of a major Avon Park Permeable Zone in central and southern Florida. US Geological Survey Scientific Investigations Report 2007-5207, 60 p.

Reese, R.S. and Wacker, M.A., 2009. Hydrogeologic and hydraulic characterization of the surficial aquifer system, and origins of high salinity groundwater, Palm Beach County, Florida. US Geological Survey Scientific Investigations Report 2009-5113, 83 p.

Scott, T.M., 1988. The lithostratigraphy of the Hawthorn Group (Miocene) of Florida. Florida Geological Survey Bulletin No. 59, 148 p.

SFWMD, 2013. Corbett Wildlife Management Area, M-O Canal. Report submitted to SFWMD by Creech Engineers, Inc., 38 p.

SFWMD, 2014. Boundary Survey, northern portion of Palm Beach County Biotechnology Research Park. Survey submitted to SFWMD by Bowman Consulting Group, LTC. 13 pages.

SFWMD, 2015a. Boundary and Topographic Survey of the Mecca Flow Equalization. Appendix D, Final Design Documentation Report dated June 30, 2016. Report submitted to SFWMD by Arcadis Design & Consultancy, 25 p.

SFWMD, 2015b. Control Survey and Well Survey Report for the Mecca Farms Equalization Basin. Survey report submitted to SFWMD by Arcadis Design and Consultancy, 37 p.

Spencer, E. 1967. "A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces," Geotechnique, Volume 17, No. 1, pp 11-26.

Stemle, Andersen & Associates, Inc., 1994. Floridan Aquifer Wellfield Deepening Preliminary Hydrogeologic Report. Report dated December 1994 submitted to the Town of Jupiter, FL, 21 p.

Tetra Tech, 2012. Loxahatchee River Floway 3 Environmental Summary Report; Nine Gem, Culpepper & Sunrise Boys Properties, prepared for SFWMD.

Tierra, Inc., 2004. Preliminary Geotechnical Engineering Services Report for the Palm Beach County Biotechnology Research Park, Palm Beach County, Florida. Report prepared for Miller, Legg and Associates, Inc., dated August 2, 2004, 23 p.

Tomasello Consulting Engineers, (2016). Cypress Creek Modeling of Operations. Report dated December 2016 prepared for Martin County Board of County Commissioners, 48 p.

Unpublished Report, 1975. Report of freshwater injection-recovery study, Palm Beach County (PB-747). 12 p.

USACE, 2013. Comprehensive Everglades Restoration Plan Aquifer Storage and Recovery Pilot Project, Final Technical Data Report. Report prepared by the US Army Corps of Engineers dated December 2013, 340 p. plus Appendices.

URS Corporation, 2003. Phase III Environmental Site Assessment and Ecological Risk Assessment, Sunrise Boys/Indiantown Realty.

URS Corporation, 2004. Phase I/II Environmental Site Assessment, Mecca Farms Property.

URS Corporation, 2009. Underground Storage Tank and Aboveground Storage Tank Removal/Closure Assessment Report.

URS Corporation, 2008. Geotechnical Investigation & Soils Evaluation, Mecca Farms Flow Way, Palm Beach County, Florida. Report prepared for RCT Engineering dated January 15, 2008, 64 p.

US Department of Transportation, 2019. Pipeline and Hazardous Materials Safety Administration, National Pipeline Mapping System Public Viewer. Searched Palm Beach and Martin Counties at https://pvnpms.phmsa.dot.gov/PublicViewer/ on 12 June 2019.