Design Documentation Report for Basis of Design

Lower Kissimmee Basin Stormwater Treatment Area

OKEECHOBEE COUNTY, FLORIDA

Ecosystem Investment Partners

Lake Okeechobee Water Quality
An EIP Pay for Performance Solution

Brown AND Caldwell

SFWMD Contract No. 4600004527

February 2023

DESIGN DOCUMENTATION REPORT FOR BASIS OF DESIGN

LOWER KISSIMMEE BASIN STORMWATER TREATMENT AREA

Okeechobee County, Florida

Prepared For:

South Florida Water Management District
3301 Gun Club Road
West Palm Beach, FL 33406
Contract No. 4600004527 (PO 9500009421)

and

EIP Florida Water Quality, LLC
5550 Newbury Street, Suite B
Baltimore, MD 21209
Contact: Jeremy McBryan (jeremy@ecosystempartners.com)

Prepared By:

Brown and Caldwell
2301 Lucien Way, Suite 250
Orlando, FL 32751
Contact: Ann Redmond (aredmond@brwncald.com)

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List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AB	Allen Bradley
ac	acre(s)
ACI	American Concrete Institute
AISC	American Institute of Steel Constructions
ANSI	American National Standards Institute, Inc.
ASCE	American Society of Civil Engineers
ASHRAE	American Society of Heating Refrigerating and Air-Conditioning Engineers
ASTM	American Standard Test Method
ATS	automatic transfer switch
A/V	audio visual
AWS	American Welding Society
BA	Biological Assessment
bgs	below ground surface
BMAP	Basin Management Action Plan
BMP	best management practices
BMS	Building Management Systems
BO	Biological Opinion
CADD	computer-aided design and drafting
CERP	Comprehensive Everglades Restoration Plan
CFD	computational fluid dynamics
CFR	Code of Federal Regulations
cfs	cubic feet per second
CH	need soil classification
CHI	Computational Hydraulics International
CL	need soil classification
cm	centimeters
cm/d	centimeters per day
CPI	Consumer Price Index
CRAS	Cultural Resource Assessment Survey
DBHYDRO	SFWMD corporate environmental database that stores hydrologic, meteorologic, hydrogeologic and water quality data
DCM	Design Criteria Memoranda
DDR	Design Documentation Report
District	South Florida Water Management District
DMSTA2	Dynamic Model for Stormwater Treatment Areas (Version 2)
DNP	Distributed Network Protocol
DS	downstream
DTM	digital terrain model
EAP	Emergency Action Plan
EAV	emergent aquatic vegetation
ECB19	Existing Condition Base 2019
EIP	Ecosystem Investment Partners and EIP Florida Water Quality, LLC
EM	Engineering Manuals
EOC	end of construction
-	
ESA	Environmental Site Assessment
ESS	Electronic Safety and Security
ET	evapotranspiration
FAA	Federal Aviation Authority
F.A.C.	Florida Administrative Code
fps	feet per second
F.S.	Florida Statutes
FBC	Florida Building Code
FCC	Federal Communication Commission
FDACS	Florida Department of Agriculture and Consumer Services
FDEP	Florida Department of Environmental Protection
FDOT	Florida Department of Transportation
FEB	Flow Equalization Basin
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FFWCC	Florida Fish and Wildlife Conservation Commission
FPL	Florida Power & Light
ft	foot/feet
gal	gallon(s)
GDR	Geotechnical Data Report
GEDR	Geotechnical Engineering Design Report

H	horizontal
HDPE	high density polyethylene
HGL	hydraulic grade line
HHD	Herbert Hoover Dike
H&H	hydrology and hydraulic
hp	horsepower
HPC	Hazard Potential Classification
HQ	headquarters
HVAC	heating, ventilation and air conditioning HVHZ High Velocity Hurricane Zone
HW	Headwater
Hz	Hertz
IBC	International Building Code
in	inch(es)
g	peak ground acceleration
IATD	industry-accepted technical documentation
IBC	International Building Code
I&C	instrumentation and controls
IDF	inflow design flood
IEEE	Institute of Electrical and Electronics Engineers
IIOT	Industrial Internet of Things
I/O	input/output
ISM	industrial, scientific, and medical
IT	information technology
ITP	Incidental Take Permit
JD	Jurisdictional Delineation
KVA	kilovolt-amperes
kV	kilovolt
kW	kilowatt
Lake	Lake Okeechobee
lb/f^3	pounds per cubic foot
LED	light emitting diode
LF	linear foot/feet
LiDAR	light detection and ranging
LKBSTA	Lower Kissimmee Basin Stormwater Treatment Area
LOPP	Lake Okeechobee Protection Permit
LORS2008	Lake Okeechobee Regulation Schedule of 2008
LOSA	Lake Okeechobee Service Area
LOSOM	Lake Okeechobee System Operating Manual
LPG	liquified petroleum gas
LSIG	long time, short time, instantaneous, ground
mA	milliamp
Mb	megabyte
MHz	mega hertz
mi	mile(s)
MOPO	models for the construction process
mph	miles per hour
MSD	maximum storage depth
MTIF	Miccosukee Tribe of Indians of Florida
MTS	manual transfer switch
mt/yr	metric tons per year
mV	millivolt
MWSL	Maximum Water Storage Level
NA25f	LOSOM RSM 2025 Future without Condition
NAD	North American Datum
NAD83	North American Datum of 1983
NAVD	North American Vertical Datum
NAVD88	North American Vertical Datum of 1988
NEMA	National Electrical Manufacturers Association
NEPA	National Environmental Policy Act
NEEPA	Northern Everglades and Estuaries Protection Act
NEEPP	Northern Everglades and Estuaries Protection Program
NEC	National Electrical Code
NEMA	National Electrical Manufacturers Association
NEPA	National Environmental Policy Act
NFPA	National Fire Protection Association
NFSL	Normal Full Storage Level
NG	Natural Gas
NGVD29	National Geodetic Vertical Datum of 1929
NOA	Notice of Acceptance
NOAA	National Oceanic and Atmospheric Administration

NPSH	net positive suction head
NRCS	Natural Resources Conservation Service
NSRS	National Spatial Reference System
NVR	network video recorder
O&M	Operations and Maintenance
OSHA	Occupational Safety and Health Administration
OSTDS	onsite sewage treatment and disposal systems
P	phosphorus
pcf	pounds per cubic foot
PCSWMM	Personal Computer Stormwater Management Model
percent	%
PES	phosphorus elimination system
PLC	Programmable Logic Controller
PMP	Probable Maximum Precipitation
PoE	power over ethernet
POM	Project Operations Manual
PS	pump station
psf	pounds per square foot
psi	pounds per square inch
Project	Lower Kissimmee Basin Stormwater Treatment Area Project
PS	pump station
PTZ	pan-tilt-zoom
Q	flow
REC	Recognized Environmental Concern
RDD	rapid drawdown
RF	radio frequency
RMSE	Root Mean Square Error
RSM	Regional Simulation Model
RTDs	resistance temperature detectors
RTU	remote telemetry unit
RVSS	Reduced Voltage Soft Starters
SAS	Surficial Aquifer System
SCADA	supervisory control and data acquisition
SDC	Seismic Design Categories
SDI	SCADA design and installation
sec	second
Section 106	Section 106 of the Historic Preservation Act
SFWMD	South Florida Water Management District
SFWMD	South Florida Water Management District
SHPO	Florida State Historic Preservation Office
SIR	Subsidence Incident Reports
SLERA	Screening Level Ecological Risk Assessment
SMACNA	Sheet Metal and Air Conditioning Contractors' National Association
SP	shelly sand
SP-SC	sand with clay
SP-SM	slightly silty sands
SM	silty sands
SPT	standard penetration test
SQL	Structured Query Language
SR	State Route
SSL	Sovereign Submerged Lands
SSS	steady-state seepage
STA	stormwater treatment area
STOF	Seminole Tribe of Florida
SWMM	Stormwater Management Model
TCNS	Taylor Creek/Nubbin Slough
TFN	thermoplastic fixture wire nylon
THHN	thermoplastic, high-heat resistant, nylon coated wire
THWN	thermoplastic, high-heat resistant, water resistant, nylon coated wire
TIA	Telecommunications Industry Association
TM	Technical Memorandum
TMDL	total maximum daily load
TP	total phosphorus
TRAs	Targeted Restoration Areas
TRB	Technical Review Briefing
TW	Tailwater
T&E	Threatened and Endangered Species
UHF	ultra-high frequency
UL	Underwriters Laboratories, Inc.
UMAM	Uniform Mitigation Assessment Method
UPS	uninterruptable power supply
US	upstream

USBR	United States Bureau of Reclamation
U.S.	United States
USACE	U.S. Army Corps of Engineers
U.S.C.	U.S. Code
USCS	Unified Soil Classification System
USDA	U.S. Department of Agriculture
USDOI	U.S. Department of Interior
USEPA	U.S. Environmental Protection Agency
USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey
V	vertical
V	volt
VDC	volts direct current
VHF	very high frequency
WCS	water control structure(s)
WPI	weather protected type I
WSE	water surface elevation
WTRs	water treatment residuals
3D	three dimensional
401	Section 401 of the Clean Water Act
404	Section 404 of the Clean Water Act
408	Section 14 of the Rivers and Harbors Appropriation Act of 1899, as amended, and codified in 33 U.S. Code Section 408

Applicable Units and Datums	
Length	miles (mi), feet or foot (ft), inches (in)
Area	acres (ac), square feet (sf), square yards (sy)
Volume	acre-feet (ac-ft), million gallon (MG), cubic feet (cf), cubic yards (cy), gallon (gal)
Flow	acre-feet per day (ac-ft/d), million gallon per day (MGD), cubic feet per second (cfs)
Mass	metric tons per year (mt/yr)
Concentration	milligrams per liter (mg/L), micrograms per liter (µg/L), parts per billion (ppb)

The project generally utilizes English units and the elevation datum for the project is the NAVD88. The elevation datum for data from other sources, including the SFWMD, is the National Geodetic Vertical Datum of 1929 (NGVD29). For the design of the project, at the Project location, the conversion from NGVD29 to NAVD88 is -1.21 ft. Conversion factor identified in the Structure Information Verification (STRIVE) 2022 for the S-154 structure is -1.23 ft from NGVD29 to NAVD88. Following construction, as-builts will be created that will identify the datum conversion that is specific to each structure.

Executive Summary

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EXECUTIVE SUMMARY

The South Florida Water Management District (SFWMD or District) has engaged EIP Florida Water Quality, LLC (EIP) in a performance-based contract to deliver the Lower Kissimmee Basin Stormwater Treatment Area (LKBSTA) Project (Project) on EIP-owned property in Okeechobee County to maximize removal of total phosphorus (TP) loads from priority areas of the Lake Okeechobee watershed. The Project is proposed to be completed by EIP in two phases. Phase One consists of a Reconnaissance Study, a Design Documentation Report (DDR), and preliminary design activities, including preparation of initial permit applications. At the conclusion of Phase One, EIP will submit a proposal for Phase Two, which is anticipated to include final design activities, permitting, construction, land transfer, 5 years of productive operations, and Project turnover. This document is the DDR deliverable required by the above-mentioned contract and generally follows the typical DDR outline noted in SFWMD's Engineering Submittal Requirements (ESR), updated March 22, 2016 (SFWMD, 2016a). Modifications to the typical DDR outline in the ESR were coordinated in consultation with SFWMD and are summarized in Appendix 1.

As previously documented in the Reconnaissance Study Final Report submitted to SFWMD in September 2022, the EIP team identified an alternative (Alternative C) to be advanced to the DDR and preliminary design phases based on the results of an evaluation methodology that enabled an objective review and assessment of design elements, performance expectations, and construction complexity, among other issues. The identified alternative achieves the goals of reducing TP loads from the Taylor Creek/Nubbin Slough, Indian Prairie, and Lower Kissimmee subwatersheds as well as Lake Okeechobee, thereby helping the State of Florida achieve the Lake Okeechobee total maximum daily load goals and the goals and objectives of the Lake Okeechobee Basin Management Action Plan.

The alternative identified for the Project is approximately 2,500 acres (ac) of stormwater treatment area (STA) designed to treat stormwater runoff from the L-62 and C-38 canals and treat Lake Okeechobee water. A new inflow pump station (PS) will direct inflows into the Project from a rerouted L-62 canal; Project outflows will be conveyed to the C-38 canal through the existing Structure (S) S-154. Rerouting a portion of the L-62 canal facilitates improves configurations of the six STA cells and enables more efficient construction methods. The six STA cells are being designed to operate in parallel and are planned to be dominated by emergent aquatic vegetation. In addition, an innovative water quality treatment technology consisting of a vertical engineered media filtration system (referred to as a phosphorus elimination system or PES) is proposed and is being designed to allow both treatment of inflows independent from and in parallel with the STA cells and in series with and downstream of the STA cells. This approach enables both water quality treatment of inflows as well as additional treatment of STA outflows to mitigate the potential impacts of STA dry-out events that have been known to result in higher than desirable TP concentrations in STA outflows. The Project is proposed to be operated as a year-round, flow-through STA system that prioritizes the treatment water with the highest TP concentrations to maximize TP load reduction. During times when stormwater runoff flows in the L-62 canal are below the inflow PS capacity, available water from the C-38 canal will be treated and returned to the C-38 canal.

This DDR is intended to be a comprehensive document that describes Project goals, individual Project elements, and key design decisions and methods EIP will use in future design phases. This DDR is also intended to explain the applied design criteria, critical assumptions, and analytical methods to be used to complete the Project's design. Design optimization activities are ongoing and will continue during future design activities. This DDR will be updated during future design activities to both build upon and document the design process.

Modeling Completed to Inform Design

Several modeling analyses have been implemented to evaluate design concepts and assist in identifying and selecting design approaches. Hydrologic, hydraulic, water availability, and water quality modeling were performed to determine and validate the Project layout, optimize hydraulics and cell sizing, and develop operational strategies. Modeling analyses will continue to be updated throughout future design phases.

Hydrology

Due to the availability of robust historical flow and water level data in and around the Project site, hydrologic modeling activities were largely limited to estimating the offsite stormwater runoff flows that currently drain from the north into the Project site west of the L-62 canal via two culverts under State Road 70. A conservative approach to quantifying the flow rates entering the Project site from the north was used to determine preliminary canal and structure capacities needed to maintain historical drainage for these areas and convey the offsite flows to the L-62 canal to enable treatment by the Project. In addition, hydrologic modeling was conducted to estimate the offsite flows that currently drain from north of the Project site east of the L-62 canal and through proposed STA Cells 5 and 6. Under proposed conditions, offsite flows from east of the L-62 canal that currently enter the Project site will be collected via a proposed seepage canal located on the north and east sides of Cells 5 and 6, ultimately draining offsite to the south via a drainage easement located adjacent to and east of the Herbert Hoover Dike (HHD). The combination seepage and off-site drainage flows are less than the pre-project flows, therefore modification to the drainage easement is not anticipated. See Section 5 (Hydrology) for more details.

Water Availability

A water availability analysis was performed to assist in understanding the potential complexity and level of effort needed to obtain permit approvals, considering the regulatory water-supply-related constraints that exist when Lake Okeechobee water levels are below specific elevations. The water availability analysis evaluated the regulatory aspects of the Lake Okeechobee Service Area, Lake Okeechobee Regulation Schedule, and Lake Okeechobee System Operating Manual. The analysis also incorporated several scenarios to evaluate the potential benefits of incorporating a 500-ac flow equalization basin (FEB), with a maximum water depth of 4 feet (ft), upstream of the STA. While there were some slight reductions in the average number of days per year with low STA water depths and slight reductions in the number of low STA water depth events, the projected TP load reduction of the FEB+STA scenario was similar to the STA-only scenarios with similar areas. Therefore, the EIP team concluded that reducing the STA size to incorporate an FEB was not justified. See Section 6 (Water Availability Analysis) for more details.

Water Quality

Water quality modeling was performed to estimate long-term phosphorus (P) removal performance for the Project using the Dynamic Model for Stormwater Treatment Areas, Version 2 (DMSTA2), a spreadsheet-based platform that accounts for P and water mass balances. The EIP team is working to finalize the operational rules and constraints that will define the available water from each of the proposed sources, as well as refine the inflow and TP datasets and other DMSTA2 modeling assumptions. See Section 7 (Water Quality Model) for more details.

Hydraulics

A series of hydraulic models were prepared to define design parameters for the proposed rerouted L-62 canal, evaluate STA system hydraulics, assess the Project's Hazard Potential Classification (HPC), conduct wind set-up/wave run-up analyses, and ensure flood routing aspects of the design are appropriate. See Section 8 (Hydraulics) for more details.

Design Approach Overview and Summary of Key Design Criteria

The following sections provide an overview of the key design approaches and a summary of the key design criteria for the major Project features or elements (Figure ES-1).

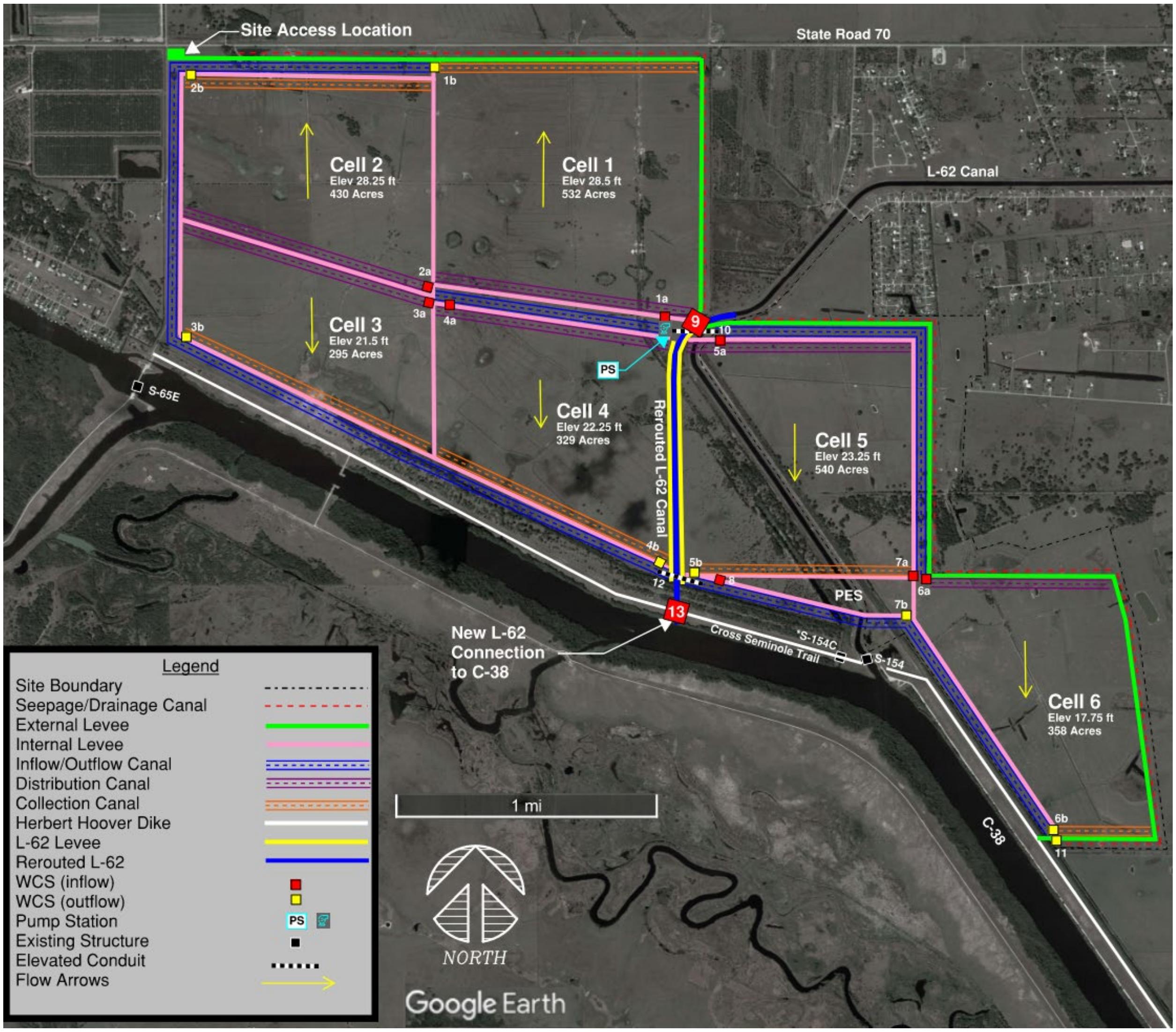

Figure ES-1. Project layout and major features

L-62 Canal Reroute

A portion of the L-62 canal is proposed to be relocated or rerouted to accommodate efficient construction of three major Project components (inflow PS, and water control structures [WCS] WCS-9 and WCS-13) outside of existing embankments and canals, enable improved STA cell configuration, and improve hydraulics of the Project's headworks system.

The approach of rerouting the L-62 canal is expected to minimize dewatering and bypass pumping needs during construction. Two new WCSs will be installed within the rerouted L-62 canal, one (WCS-9) located upstream of the inflow PS to control water levels within the L-62 canal and another (WCS-13) located near the confluence of the rerouted L-62 canal and the C-38 canal/HHD that is intended to reduce potential complexities and timelines associated with the authorization required by the U.S Army Corps of Engineers (USACE) to modify the HHD.

The rerouted L-62 canal also enables improved STA cell configuration within the Project site. The existing L-62 canal hooks through the LBKSTA site and creates two triangular-shaped properties on both the west and east sides of the L-62 canal. Irregular-shaped STA cells can result in areas with inconsistent hydraulic residence times and less-than-desirable treatment performance. STA treatment performance is best when water flow is laminar, relatively shallow, and uniformly distributed across treatment cells.

In addition, rerouting the L-62 canal is expected to improve the hydraulics of the Project's headworks system, as compared to the existing L-62 canal alignment. The existing L-62 canal alignment and the configuration of S-154 at the confluence of the C-38 canal were designed to facilitate the collection and discharge of S-154 basin drainage to the C-38 canal. The rerouted L-62 canal allows for efficient conveyance of stormwater runoff collected via the L-62 canal flowing south to the Project and enables efficient deliveries of available water from the C-38 canal flowing north to the Project. Rerouting the L-62 canal also eliminates the need to modify S-154 to accommodate bi-directional flow.

Inflow PS

The inflow PS's design capacity was identified based on statistical analyses of the water available for the Project. Upon review of the historical data (40-year record), a 500 cubic-ft-per-second (cfs) capacity inflow PS would be used over 90 percent of the period of record. The current design includes four 125-cfs vertical mixed flow electric motor-driven pumps, which would allow four inflow pumping rates (e.g., 125, 250, 375, and 500 cfs). The superstructure design of the inflow PS is expected to be similar to the Lake Hicpochee PS (G-725), which includes a shade structure over the pumps and a separate control/operation building. The pumped inflows would bifurcate downstream of the PS and be directed westward via the western inflow canal to STA Cells 1 through 4 and be directed eastward via an elevated conduit (co-located with WCS-9) and the eastern inflow canal to STA Cells 5 and 6 and the PES.

WCSs

Each STA cell has two WCSs, one inflow and one outflow. The design for these WCS consists of a double-barrel high-density polyethylene (HDPE) fused joint pipe, with a concrete gate structure with an operable gate for each barrel located immediately adjacent to the embankment access road. To allow for operational flexibility and provide redundancy, each WCS will be designed to convey 150 percent of the design flow capacity. In addition to sizing the outflow WCS to convey design flows through the STA cell, these structures will also have the capacity to convey rainfall that directly falls on the cells.

At the WCS inlet and outlets, erosion control measures will be installed to effectively manage the flow of water. Within the embankment, a concrete gate structure will contain two stainless-steel slide gates, one for each of the two barrels within the WCS. Operational flexibility is being designed into the inflow WCS for STA Cells 1 and 2 that allows reverse flow to enable these cells to temporarily serve as storage cells. As such, water within Cells 1 and 2 could either be conveyed to the outflow canal via the outflow WCS or conveyed back to the western inflow canal via the inflow WCS for distribution to STA Cells 3 through 6 and the PES.

The two WCSs to be constructed within the rerouted L-62 canal (WCS-9 and WCS-13) will be large-capacity structures with slide or roller gates, capable of passing the design flow of S-154 (1,000 cfs). In addition to conveying S-154 basin stormwater runoff collected by the L-62 canal, WCS-9 will manage the water levels within the L-62 canal upstream of the Project, while the new WCS within the HHD (WCS-13) will provide both flood protection along the HHD and allow 500 cfs to be conveyed from the C-38 canal north into the L-62 canal for pumping by the inflow PS.

Embankments

Earthen embankments will be constructed as part of the LKBSTA to separate various water-containing facilities (canals and STA cells) as well as provide vehicular access to Project elements. Embankments will be designed with a 14-ft top width to accommodate vehicular traffic access, sodded horizontal (H) to vertical (V) side slopes of 3H:1V, and a single 20-ft-wide horizontal maintenance bench alongside each adjacent canal for access and maintenance. Embankment heights will be designed assuming a low hazard potential classification and a maximum operating STA cell water depth of 24 inches.

The proposed embankments along the rerouted L-62 canal will be similarly designed; however, each side of the L-62 canal will have a 20-ft-wide maintenance bench.

Canals

The rerouted L-62 canal is located west of the existing location and will be designed to convey 1,000 cfs from the existing L-62 canal to the new connection with the C-38 canal. This new L-62 canal segment is proposed to have a 15-ft-wide bottom width and sodded side slopes of 3H:1V.

Two inflow canals, fed by the inflow PS, will direct water to the STA cells. The western inflow canal will convey water to STA Cells 1 through 4, and the eastern inflow canal will convey water to STA Cells 5 and 6 and the PES (Figure ES-2). Two outflow canals will convey treated water from all STA cells to a segment of the existing L-62 canal just north of S-154 (Figure ES-3).

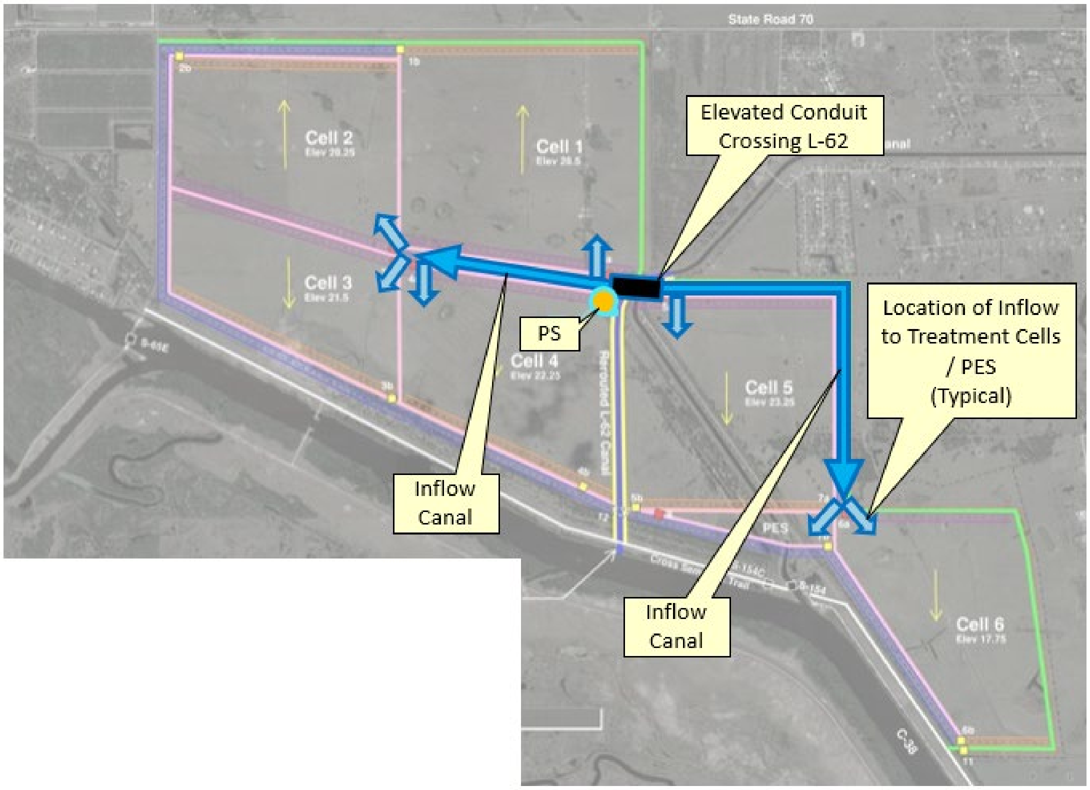

Figure ES-2. STA system inflow canals

Two seepage collection canals will be located along the perimeter of the Project site along the outside of the inflow canals or STA cell external embankments. These collection canals will be designed to convey both Project seepage and off-site drainage to one of three locations. All inflow, outflow, and seepage canals will have bottom widths ranging from 5 to 10 ft and sodded side slopes of 3H:1V.

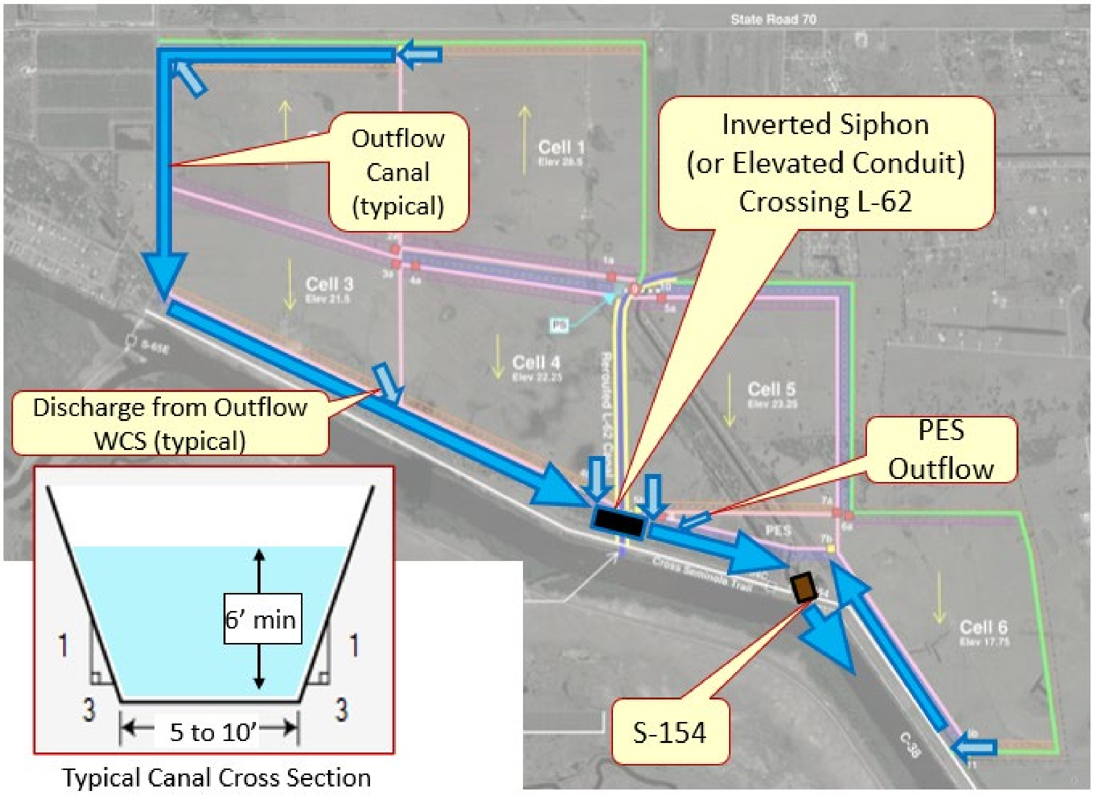

Figure ES-3. STA system outflow canals

Distribution and collection canals will be constructed within each STA cell at the upstream and downstream ends, respectively. The purpose of the distribution canals is to evenly distribute water over the width of the STA cell to promote uniform sheet flow across each cell's surface area. Collection canals collect water from each STA cell and convey it to the outflow WCS. Outflow WCSs control the flow of water from the collection canal to the outflow canal. Under normal operating conditions, there will be a maximum water depth of 2 ft above top of bank in both the distribution and collection canals. Distribution and collection canals are proposed to have a bottom width of 5 ft and a bottom elevation 6 ft below the STA cell bottom elevation.

PES

As a complementary and innovative technology, the PES will be designed to operate both independently of and in series with the STA cells. The PES includes four major elements: the flow distribution system, the treatment media, the stone and underdrain collection system, and vegetation. Upon entering the PES, flows are conveyed to individual cells through a system comprised of several perforated distribution pipes in a stone bed under the treatment media; the treatment media consists of a proprietary amendment, a sand aggregate matrix, and water treatment residuals, which are a waste product from water treatment plants. The inflows then rise through the media and spill over a weir that flows into a collection pipe system. The initial construction of 6 ac of PES cells (on 10 ac of land) is included in this DDR; however, an additional 40 ac is being reserved as part of the Project design to accommodate up to an additional 30 ac of PES cells or other innovative water quality treatment technologies.

Communications

All systems and facilities within the Project boundaries, as general practice, will be monitored and controlled from a local control system at the inflow PS. However, WCS-9 and WCS-13 will be designed and constructed to meet SFWMD requirements and be immediately turned over to the District to operate and maintain. As such, these WCSs will each employ a District-standard Motorola ACE remote terminal unit (RTU) with the standard peripherals (battery box, antenna surge protection, primary and secondary radio frequency paths), headwater and tailwater stilling well stage monitors, gate control panel(s), and a gate position sensor(s). All other Project elements will be operated and monitored by a local control system at the inflow PS. The inflow PS local control system will be a programmable logic controller. There will be an EIP RTU installed in the inflow PS control room that will be in operation for the duration of EIP's 5- to 7-year operations period. Monitoring and control will be available through the EIP RTU. Full monitoring capability will be made available to the District during EIP's 5- to 7-year operations period. After this period, and at Project turnover, a District-standard Motorola ACE RTU will replace the EIP RTU.

Operations

As noted above, the Project's overarching goal is to maximize removal of TP loads from priority areas of the Lake Okeechobee watershed. To achieve this goal, the EIP team has identified an operational strategy that involves two separate treatment facilities—the STA and the PES. Stormwater runoff from the L-62 canal will be prioritized for treatment due to its higher TP concentrations, followed by available water from the C-38 canal.

The operation of the Project is a function of flows and water levels in the L-62 and C-38 canals, Lake Okeechobee water levels and operations, and conditions within the STA cells. The inflow PS will direct water into the six treatment cells and PES system via an inflow canal system. STA cell and PES outflows will be collected by an outflow canal and conveyed via gravity back to the C-38 canal. The Project is being designed to accommodate various operational schemes which are described in the Draft Project Operations Manual (DPOM), provided in Appendix 5.

The Project is being designed to operate under a broad range of climatic conditions and therefore is expected to be resilient to many dynamic climate drivers. During preliminary design activities, approaches to evaluate the impacts of potential changes in climate drivers (e.g., rainfall, evapotranspiration, etc.) on the Project will be developed in close coordination with SFWMD's District Resiliency Officer.

The ability to recirculate a portion of STA outflows for additional treatment within the STA will be evaluated during preliminary design. The current recirculation concept being proposed includes an additional WCS to convey water from the outflow canal to the rerouted L-62 canal just north of the proposed location of WCS-13.

A 50-ac area south of Cell 5 is proposed to consist of a 6-ac PES facility (on 10-ac of land) as well as a 40-ac area reserved to allow for future expansion of the PES facilities or implementation of other innovative water quality treatment technologies. In the future, the entire 50-ac PES area could be converted into treatment wetlands and integrated into an expanded Cell 5. The Cell 5 outflow WCS is being designed and is located to accommodate outflows from an expanded Cell 5.

Deviations from District Standards

Generally, the Project's components will follow the standards established by the SFWMD. The operations strategy and contracting mechanism for the Project presents an opportunity to innovate to design and construct more cost-effective infrastructure. Where major innovations were considered, prior to inclusion in this report, EIP has coordinated closely with District staff to learn from their experience as managers of large water resources projects. The deviations proposed to date include, but are not limited to, the reduction of maintenance bench length, modification of the typical STA WCS, and incorporation of innovative PES technology. A list of the deviations from District standards is included in Appendix 2.

Design Optimization

Through the design process, a variety of value engineering decisions have been discussed with the SFWMD. During the discussion regarding these decisions the ease of construction and permitting were identified as well as the operations and maintenance drivers for each design component. Innovative ideas associated with potential design optimizations were discussed with SFWMD staff and include, but are not limited to, maximizing the STA acreage (and treatment capacity) by creating an STA cell layout that includes multiple flow options for enhanced operational flexibility, combining the outflow and seepage canals along the western property boundary, combining the off-site drainage management and seepage canals where possible, reducing the embankment height between cells, and incorporating a two new WCS along the L-62 canal. A list of the value engineering optimizations evaluated is included in Appendix 2.

Project Schedule Summary

The statement of work approved by SFWMD in December 2021 included estimated durations for the various phases of work and also recognized that the schedule may need to be adjusted to resolve concerns or if additional time is required during the review process due to the quantity and/or context of comments. Phase One A Task 1 (Reconnaissance Study) was completed in September 2022, approximately 4 months later than originally estimated, due to the need for additional coordination and to resolve SFWMD concerns related to proposed STA cell ground elevation topography, and to enable the development of more expansive technical evaluations of water availability and water storage. As a result, the DDR workshops were postponed by approximately 5 weeks and the Draft DDR was completed in November 2022, approximately 3 months later than originally estimated. Phase One A Task 2 (DDR) was completed in February 2023.

In an attempt to remediate delays to date, the EIP team initiated preliminary design activities immediately following completion of the Draft DDR, which include three-dimensional (3D) computational fluid dynamics modeling for several Project components, 3D groundwater modeling to understand the Project's potential impacts on adjacent areas, updating the water availability analyses and water quality modeling, and advancing various discipline designs. Coordination with Florida Power & Light is ongoing and will continue to identify the requirements for a new electrical connection to the LKBSTA. The final Phase One task, Phase One A Task 3 (Preliminary Design+), is scheduled to be completed during the final seven months of Phase One, consistent with the duration estimated in the approved statement of work. The Draft Preliminary Design+ deliverables are scheduled to be submitted in May 2023 and the Final Preliminary Design+ deliverables are anticipated to be submitted in July 2023.

At the conclusion of Phase One, EIP will submit a proposal for Phase Two, which is anticipated to include final design activities, permitting, construction, land transfer, 5 years of productive operations, and Project turnover. Once Phase Two is approved by SFWMD, EIP will initiate final design activities, which are expected to require 12-15 months, and will coordinate with SFWMD (as joint applicant) to submit state and federal permit applications. EIP began fieldwork related to wetlands, threatened and endangered species and archaeological resources in 2021 and has completed pre-application meetings with all relevant state and federal permitting agencies. State permits are expected to be able to be acquired within 15-18 months. Federal permit acquisition will likely be coupled with the USACE's authorization to alter a civil works project (e.g. HHD, L-62 Canal, etc.) via the USACE's Section 408 program, which could require 18-24 months after submittal of final design information. In an attempt to expedite USACE Section 408 authorization, pre-408 coordination between the SFWMD and the USACE began in October 2022, however, additional coordination will be essential to ensure timely USACE review and authorization of the Project. EIP will continue to evaluate potential efficiencies that could result in schedule compression during both preliminary and final design activities.

Items that Require Resolution

At the time of DDR delivery, no issues require resolution. As preliminary design proceeds and design deliverables are developed, the DDR components will be discussed in detail with SFWMD staff. An updated DDR will be delivered during preliminary design.

SECTION 1

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SECTION 1

INTRODUCTION

This section provides a description of the Lower Kissimmee Basin Stormwater Treatment Area (LKBSTA) Project (Project), including its purpose, goals, and objectives; describes the Project site and its existing facilities; includes a Project location map; and provides the proposed Project layout and details of the planned facilities. This section also includes background information, documents the purpose and scope of this Design Documentation Report (DDR) and summarizes the key design decisions reached to date. A summary of studies, reports and projects pertinent to the Project are also provided.

1.1 Project Description

The Project is an approximately 2,500-acre (ac) stormwater treatment area (STA) being designed to treat stormwater runoff collected by the L-62 and C-38 canals and treat Lake Okeechobee water. The Project includes rerouting a portion of the L-62 canal to the west of its existing alignment to facilitate improved STA cell configurations and construction methods. A new inflow pump station (PS) will direct inflows into the STA from the rerouted L-62 canal. Outflows will be conveyed to the C-38 canal through the existing S-154 structure. The STA includes six cells configured to operate in parallel and planned to be dominated by emergent aquatic vegetation (EAV).

In addition to the traditional STA cells, an innovative water quality treatment technology consisting of a vertical engineered media filtration system, referred to as a phosphorus elimination system (PES), is currently proposed within the Project limits, which creates a hybrid STA. The Project is being designed to allow the PES to treat inflows both independent from and in parallel with the STA cells, as well as in series with and downstream of the STA cells to enable additional treatment of STA outflows. This design approach mitigates the potential impacts of STA dry-out events that have been known to result in sediment phosphorus re-suspension and higher-than-desirable total phosphorus (TP) concentrations and loads in STA outflows.

The Project will be operated as a year-round, flow-through STA system that prioritizes the treatment of water with the highest TP concentration to maximize TP load reduction. During times when stormwater runoff flows are below the Project's inflow PS capacity, available water from the C-38 canal will be conveyed into the STA, treated, then conveyed back to the C-38 canal. Figure 1-1 provides the Project layout and planned facilities.

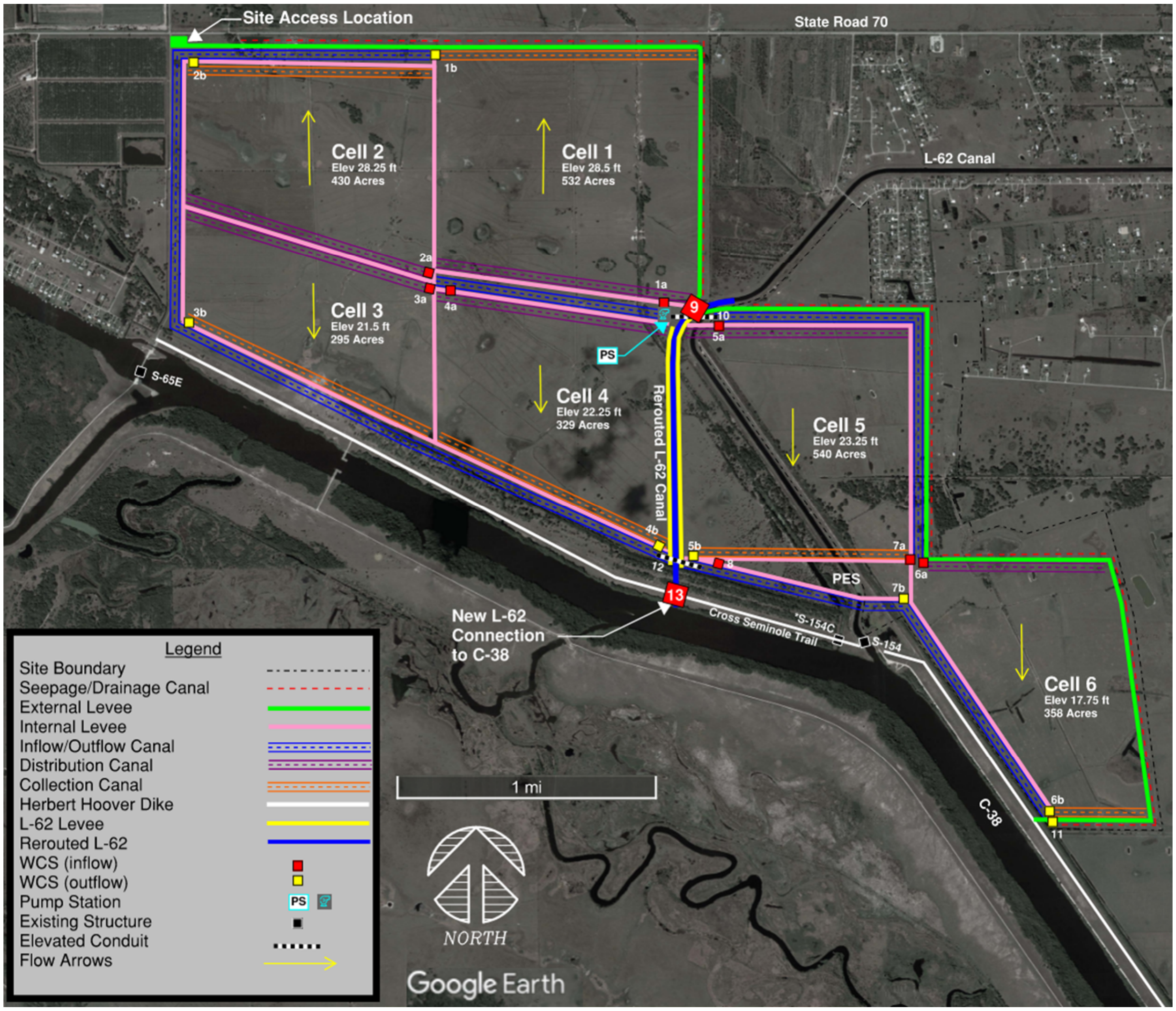

Figure 1-1. Project layout and planned facilities

1.2 Project Purpose, Goals, and Objectives

The LKBSTA is a South Florida Water Management District (SFWMD or District) water quality improvement project intended to reduce TP loads from priority areas of the Lake Okeechobee watershed, thereby helping the state achieve the Lake Okeechobee total maximum daily load (TMDL) goals and the goals and objectives of the Lake Okeechobee Basin Management Action Plan (BMAP). LKBSTA is identified as Project F-25 in the 2020 BMAP update, Table 51 (FDEP, 2020).

The primary objective of the Project is to reduce TP loads in stormwater runoff flows from the Taylor Creek/Nubbin Slough (TCNS), Indian Prairie, and Lower Kissimmee subwatersheds prior to discharge into Lake Okeechobee. As a secondary objective, due to its location, the Project will also be capable of treating Lake Okeechobee water.

1.2.1 Overview of Project Need

Lake Okeechobee is a shallow, eutrophic lake that provides habitat for fish, wading birds, and other wildlife and is a central component of the hydrology and environment of South Florida. Lake Okeechobee has been subject to long-term stressors, including excessive nutrient loads, extreme water level fluctuations, harmful algal blooms, and the rapid spread of exotic and nuisance plants in the littoral zone (Ollis *et al.*, 2022).

In 2001, the Florida Department of Environmental Protection (FDEP) adopted a TMDL for Lake Okeechobee of 140 metrics tons per year (mt/yr) of TP, of which 35 mt/yr are allocated to atmospheric deposition and 105 mt/yr are allocated to the 3.45-million-ac Lake Okeechobee watershed. In 2014, a BMAP, the framework for water quality restoration with projects and strategies to reduce pollutant loading, was adopted for Lake Okeechobee and subsequently updated in 2020. In 2016, the Northern Everglades and Estuaries Protection Program (NEEPP), originally adopted by the Florida Legislature in 2007, was amended to emphasize BMAPs for the Northern Everglades. The Northern Everglades include the Lake Okeechobee Watershed (Figure 1-2) and the Caloosahatchee River and St. Lucie River watersheds. NEEPP's intent is to protect and restore surface water resources and achieve and maintain compliance with water quality standards in the Northern Everglades through a phased, comprehensive, and innovative protection program that includes long-term solutions based on the state's TMDLs (Ollis *et al.*, 2022).

For Water Year 2016-2020, the TP load to Lake Okeechobee (not including atmospheric deposition) was calculated to be 540 mt/yr, which is 400 mt/yr above the TMDL target set by FDEP. Of that amount, the TCNS and Indian Prairie subwatersheds contributed 95 mt/yr and 80 mt/yr, respectively (Olson, 2022). The TCNS subwatershed includes the S-154C and S-154 basins, both of which, as a result of a robust technical and public process, SFWMD or District) selected in 2020 as Lake Okeechobee watershed focus areas. Additionally, the FDEP identified S-154C and S-154 basins as TP priority 1 Targeted Restoration Areas in the 2020 Lake Okeechobee BMAP update (Olson, 2022). In 2020, SFWMD identified several basins within the Indian Prairie subwatershed as Lake Okeechobee watershed focus areas, most of which FDEP also identified as TP priority 1 Targeted Restoration Areas in the 2020 Lake Okeechobee BMAP update.

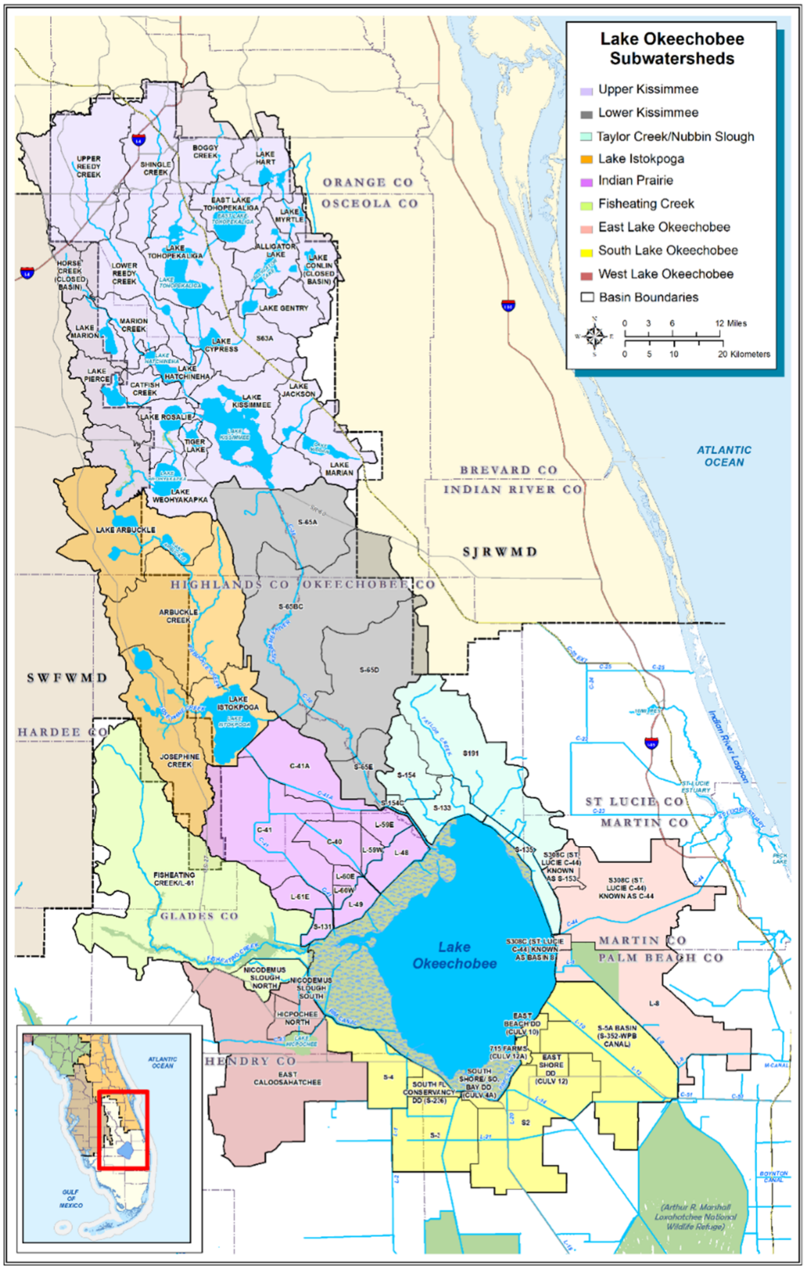

Figure 1-2. Subwatersheds and basins within the Lake Okeechobee watershed

(Olson, 2022)

1.3 Authorization for DDR

On Dec. 15, 2021, SFWMD authorized preparation of this DDR by executing Contract Number 4600004527 (Purchase Order Number 9500009421) with EIP Florida Water Quality, LLC (EIP).

1.4 Purpose and Scope of DDR

This DDR is intended to be a comprehensive document that describes Project goals, individual Project elements, and key design decisions and methods EIP will use. This DDR is also intended to explain the applied design criteria, critical assumptions, and analytical methods to be used to complete the Project's design. This DDR generally follows the outline noted in SFWMD's Engineering

Submittal Requirements, updated March 22, 2016 (SFWMD, 2016a). Any customization of the outline was in consultation with SFWMD. An updated DDR will be prepared during preliminary design and will incorporate results of preliminary design workshops held with SFWMD.

Decisions to be completed during the DDR, as stated in the Project's Statement of Work (Exhibit A of EIP's contract with SFWMD), are provided below, along with locations within this DDR where the information can be found.

1. Critical Assumptions

Sections 5 through 8 delineate the current assumptions used to model the LKBSTA Project. These models include hydrology, water availability, water quality, and initial system hydraulics. Sections 9 through 18 identify the current assumptions associated with the geotechnical, structural, civil, mechanical, electrical, instrumentation and controls, and architectural designs. A key critical design assumption is that inflows from the C-38 canal are available to maintain STA vegetation health during dry periods.

2. Analytical Methods

Sections 5 through 9 and Appendix 2 describe the analytical methods used to date to analyze the existing site characteristics, off-site impacts, and proposed project features. Throughout this DDR, proposed future analyses and design activities are described.

3. Deviations from Typical SFWMD Standards

Sections 10 through 18 identify deviations from the typical SFWMD standards. In summary, there are three key deviations proposed for the LKBSTA Project. First, the PS configuration includes a shade structure and flap gates. Second, the STA cell inflow and outflow water control structures are high-density polyethylene (HDPE) conduits. Third, the embankment design provides for maintenance benches along one side of each canal.

4. Operations Manual Specifics

Specifics include performance optimizations, monitoring requirements, normal/extreme/drought conditions, facility start-up period, and hydropattern restoration. Appendix 5 contains draft project operations manuals that describe the STA and PES operations. Hydropattern restoration is not applicable to this Project.

5. Ways Project will Connect with District's system

Sections 15 and 16 delineate the ways that the LKBSTA will connect with the District's electronic and controls system. In addition, Sections 10 and 13 identify how the physical improvements associated with the LKBSTA will connect to the District's infrastructure.

6. Ways Developer Team will Operate Project

Sections 15 and 16 and Appendix 5 describe how the EIP team intends to operate the Project, both electronically and physically.

7. PES Design Criteria

PES design information is interspersed throughout the DDR; subsections have been included to identify the various PES design criteria in each major section.

1.5 Project Background

In December 2021, SFWMD engaged EIP in a performance-based contract to maximize removal of TP loads from priority areas of the Lake Okeechobee watershed. The Project is proposed to be designed, constructed, and operated by EIP in two phases. The objective of Phase One is to complete the due diligence work necessary to determine project viability and analyze its feasibility. Phase One was divided into two subphases: Phase One A included a Reconnaissance Study (Task 1) and preparation of a DDR (Task 2). The Task 1 deliverable, the Reconnaissance Study Final Report, was submitted to SFWMD, on Sept. 30, 2022. The Task 2 deliverable is this DDR.

Phase One B consists of preliminary design activities and preparation of initial permit applications. At the conclusion of Phase One, EIP will prepare a Stipulated Payments and Deliverables Proposal for Phase Two, which is anticipated to include final design activities, permitting, construction, land transfer, operations, and project turnover. The Phase Two proposal will include a schedule of payments and deliverables and a total price for the successful delivery and 5 years of productive operations of the Project.

The objective of the Reconnaissance Study (EIP and Brown and Caldwell, 2022) was to analyze and document the Project's viability and feasibility to enable advancing the design process as efficiently as possible. During the study, the EIP team conceptualized numerous alternative designs that were further developed and refined in concert with SFWMD staff during collaborative technical workshops. As a result, three alternatives were developed and evaluated using a methodology organized into three major categories—cost, implementation schedule, and performance. The evaluation methodology developed and implemented by EIP enabled an objective review and assessment of design elements, performance expectations, construction complexity, schedule-related issues, and regulatory aspects, among other issues. As documented in the Reconnaissance Study Final Report, Alternative C was identified to advance to the DDR and preliminary design phases and is further documented in this DDR.

1.6 Project Location

The Project site is located approximately 7 miles (mi) west of the City of Okeechobee. The site is primarily located in unincorporated Okeechobee County, with a small portion located in Highlands County (Figure 1-3). The site is approximately 6 mi upstream of Lake Okeechobee on the C-38 canal, also known as the channelized Kissimmee River.

The Project site is located at the northernmost boundary of Lake Okeechobee waterbody[1] within or directly adjacent to areas that have historically had high TP concentrations in stormwater runoff flows (e.g., S-154C and S-154 basins). The site is outside of the proposed footprint of the recommended Tentatively Selected Plan for the Lake Okeechobee Watershed Restoration Project, a Comprehensive Everglades Restoration Plan (CERP) project north of Lake Okeechobee and complements the proposed CERP features by providing water quality treatment for runoff from the S-154 and S-154C basins, which CERP studies identified as two priority basins.

[1] The Lake Okeechobee waterbody includes the C-38 canal, which connects the main open water area of the lake to the S-65E structure, located on the C-38 canal approximately 1.5 mi southeast of State Road 70.

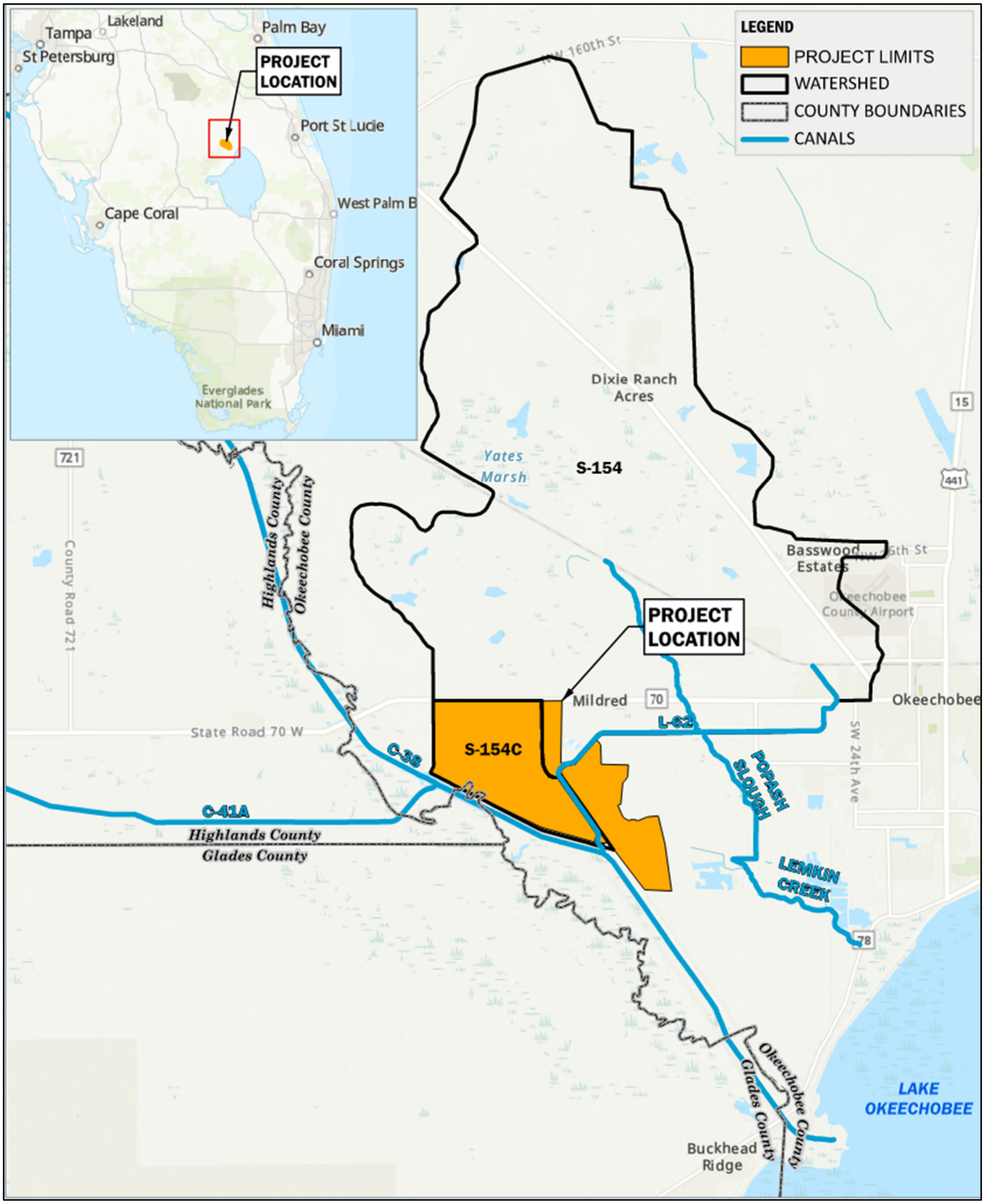

Figure 1-3. Project location

1.7 Project Site

The Project site is bisected by the L-62 canal, consists of approximately 3,400 ac of existing improved pasture, and has been zoned for agricultural land use for decades, primarily as improved pasture for cattle ranching (Figure 1-4). The site is bounded by SW 128th Avenue, a citrus grove, pastureland and residential properties to the west, State Road 70 (SR 70), the L-62 canal and single-family residential properties to the north, pastureland and a tree farm to the east, and the C-38 canal and pastureland to the south.

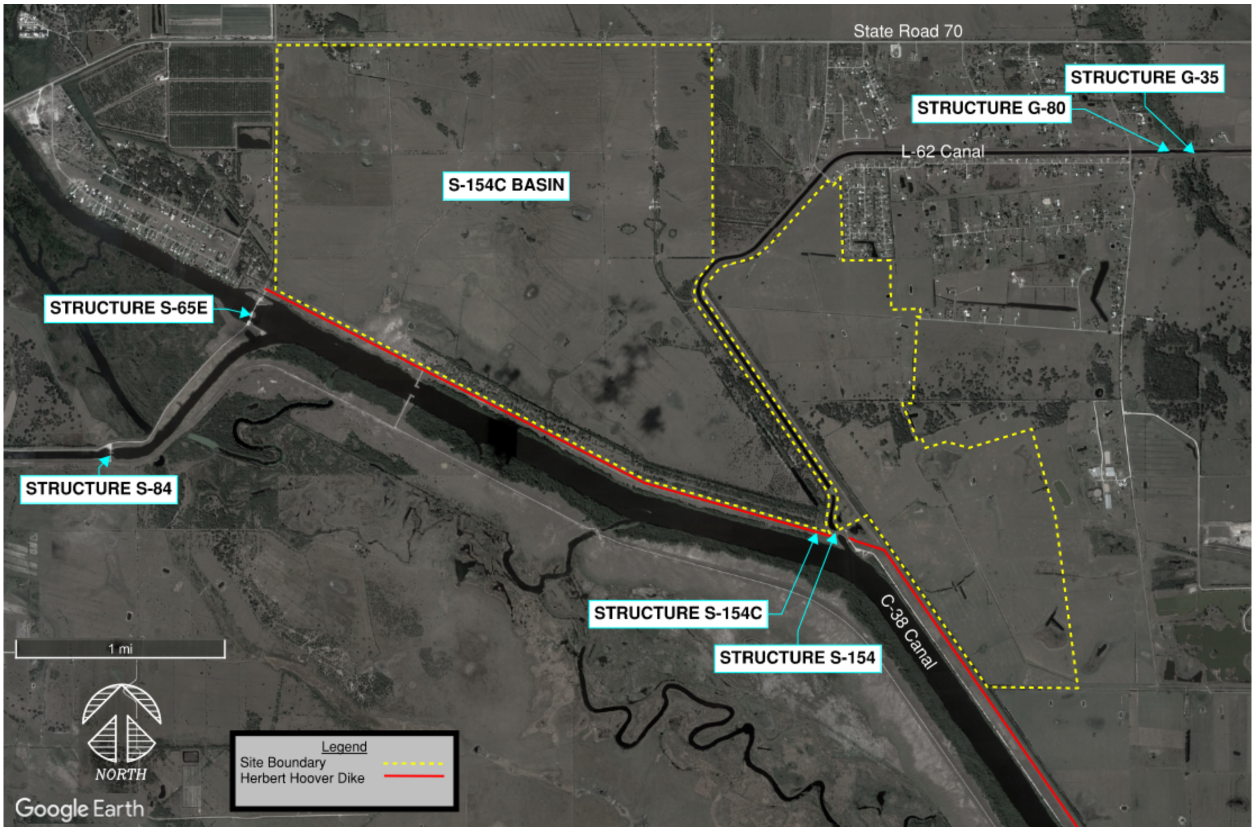

Figure 1-4. Project site

1.8 Existing Facilities

Existing facilities on the site include major drainage and surface water infrastructure, existing utilities, and various structures.

The major drainage feature located within the site is the L-62 canal, which divides the east and west portions of the property. The L-62 canal flows west and then bends south toward and through the Herbert Hoover Dike (HHD) to the C-38 canal; its discharges are controlled via the S-154 structure. On the north side of the western portion of the site, two culverts are located under SR 70. The eastern culvert under SR 70 flows through an existing drainage easement southward through the western portion of the site before entering the L-62 canal via culverts. The western culvert under SR 70 acts as an equalization structure, which allows water to transfer between low-elevation areas on the north and south sides of the road. A channel running west to east along the northern toe of the HHD drains most of the site, including the eastern SR 70 culvert flow, by flowing through the HHD and S-154C structure into the C-38 canal. The eastern portion of the site generally flows south toward a channel flowing southeast along the eastern toe of the HHD. Within the southern limits of the site's eastern portion, a drainage easement exists to drain properties farther east toward this southeastern channel. Property encumbrances are identified in the Draft Boundary Survey, which was provided in Appendix 13 of the Reconnaissance Study Final Report.

The G-80 structure, located on the north side of the L-62 canal, east of the site, drains Popash Slough into the L-62 canal. The G-35 structure, located approximately 500 feet (ft) east of the G-80 structure on the south side of the L-62 canal, conveys flows south to Popash Slough. Southwest of the site, the C-38 canal flows through the S-65E structure. To the southwest of S-65E, the C-41A canal flows to the C-38 canal via the S-84 structure.

Existing site utilities include electrical, water wells, irrigation systems, and stormwater culverts. Electrical lines were observed in both the east and west portions of the site near homes, barns, and outbuildings.

Structures were observed on both portions of the site. The west portion has structures used to store farming equipment related to current cattle ranching practices, and two residential structures just south of SR 70 in the site's northwest corner. The east portion has a residential structure and horse stable located near the entrance to the property from Southwest 67th Drive/Granada Avenue.

1.9 Planned Facilities

The following sections provide summaries of the Project's planned facilities. The Project layout with graphical representations of planned facilities was provided previously on Figure 1-1.

1.9.1 Headworks System

The headworks system includes the L-62 canal reroute, water control structure (WCS) WCS-9, WCS-13, and the inflow PS.

1.9.1.1 L-62 Canal Reroute

The Project consists of a 7,000-linear-foot (LF) north-south reroute of the L-62 canal west of the existing L-62 canal alignment to facilitate improved hydraulics of the headworks system, improved STA cell configurations, and more efficient construction methods for three major Project components (WCS-9, WCS-13, and inflow PS).

1.9.1.2 WCS-9

WCS-9, a new L-62 canal divide structure, is proposed to be located near the intersection of the existing and rerouted L-62 canal segments and is designed to maintain current operations and functionality of the existing L-62 canal, which is currently provided by structure S-154. WCS-9 is needed to manage L-62 canal stages upstream of WCS-9 during STA inflow operations, particularly when the source of STA inflows is the C-38 canal.

1.9.1.3 WCS-13

WCS-13, another new L-62 canal divide structure, is proposed to be located at the proposed confluence of the rerouted L-62 canal and the C-38 canal and coincide with the HHD alignment. WCS-13 is being proposed to avoid the need to extend the HHD along both sides of the rerouted L-62 canal and through the inflow PS and WCS-9.

1.9.1.4 Inflow PS

Project inflows will be pumped from the rerouted L-62 canal by a single inflow PS located northwest and adjacent to the rerouted L-62 canal and just downstream of WCS-9. Inflows are pumped into the western inflow canal, which can convey inflows to Cells 1 through 4. Pumped inflows can also be conveyed across the L-62 canal to the eastern inflow canal via an elevated conduit located near WCS-9. Inflows in the eastern inflow canal can be directed to Cells 5 and 6 and the PES.

1.9.2 Inflow Canal System

The inflow canal system includes eastern and western inflow canals that are being designed to accept flows pumped by the inflow PS. The western inflow canal is an east-west canal located west of the L-62 canal in the middle of the site between Cells 1 and 4 and is designed to convey inflows to Cells 1, 2, 3, and 4. The eastern inflow canal is located on the east side of the rerouted L-62 canal and is adjacent to the north side of Cell 5, then turns south and is adjacent to the east side of Cell 5 until it terminates at the northwest corner of Cell 6. The eastern inflow canal is being designed to convey inflows to Cells 5 and 6 and the PES.

1.9.3 STA System

The STA consists of six treatment cells configured to operate independently and in parallel to maximize TP load reduction. The STA cells will be graded flat and have perimeter embankments to contain the water being treated and enable flow-through. The flow direction within Cells 1 and 2 is south to north; the flow direction within Cells 3 through 6 is north to south. Each STA cell will have one inflow and one outflow WCS. The STA cells will be graded flat, and each cell will include a distribution canal on the upstream end and a collection canal on the downstream end to encourage uniform sheet flow distribution within the cells.

1.9.4 PES

A PES facility is proposed to be located east of the L-62 reroute between Cell 5 and the outflow canal. The Project is being designed to allow both treatment of inflows by the PES independent from and in parallel with the STA cells and in series with and downstream of the STA cells to enable additional treatment of STA outflows. This design approach mitigates the potential impacts of STA dry-out events, which have been known to result in sediment phosphorus re-suspension and higher-than-desirable TP concentrations and loads in STA outflows.

1.9.5 Outflow System

Outflows from Cells 1 and 2 will be conveyed to an outflow canal located north of Cell 2. The outflow canal then turns south and runs along the western limits of the site (west of Cells 2 and 3) before turning southeast and paralleling the HHD (south of Cells 3 and 4). Outflows from Cells 3 and 4 will be conveyed to the portion of outflow canal that is parallel to the HHD. Outflows from Cells 1, 2, 3, and 4 will be conveyed across the rerouted L-62 canal via an inverted siphon or elevated conduit. Outflows from Cells 5 and 6 will be conveyed northwest and west to the outflow canal, which parallels the HHD. The outflow canal will connect to a segment of the existing L-62 canal, and Project outflows will be directed to the C-38 canal via the existing S-154 structure.

1.9.6 Seepage Collection and Management System

The Project's seepage collection and management system is proposed to consist of two separate continuous canal systems (one on each side of the rerouted L-62 canal). Seepage will be collected and routed either back to the inflow PS (for additional STA treatment), routed to the L-62 canal upstream and/or downstream of WCS-9, or directed to the outflow canal. In some areas of the Project, the design water surface elevations of the outflow canal may eliminate the need to include a seepage canal exterior of the outflow canal (e.g., west of Cells 2 and 3, south of Cells 3-5, and west of Cell 6).

1.9.7 Public Access

To date, space accommodations have been incorporated into the Project design assuming that the District would lead the design and construction of public use and recreation facilities, separate from this Project. EIP is currently evaluating the information provided by the District regarding public use and recreation facilities and/or infrastructure that would typically be constructed for this type of Project (parking areas, informational kiosks, picnic shelters, signage, etc.) and will coordinate with the District regarding next steps.

1.10 Other Studies, Reports, and Projects

Due to Lake Okeechobee's importance to the region's water and environmental resources, numerous studies related to its operations, ecology, water quality, and the water quality characteristics of contributing and downstream watersheds have been conducted. In addition, SFWMD and others have been thoroughly researched and studied STAs. A list of key studies, reports, publications, and information that assisted the team during the DDR are:

- Total Maximum Daily Load for Total Phosphorus, Lake Okeechobee, Florida, prepared by FDEP, August 2001
- Lake Okeechobee Basin Management Action Plan, prepared by FDEP, January 2020
- Lake Okeechobee Basin Management Action Plan, prepared by FDEP, December 2014
- Chapter 8A: Northern Everglades and Estuaries Protection Program – Annual Progress Report, In: 2022 South Florida Environmental Report – Volume I, Prepared by SFWMD and FDEP, March 1, 2022
- Appendix 8B-2: Water Year 2021 Lake Okeechobee Watershed Focus Area Assessments, In: South Florida Environmental Report – Volume I, Prepared by SFWMD and FDEP, March 1, 2022
- Chapter 4: Northern Everglades and Estuaries Protection Program Projects, In: 2022 South Florida Environmental Report – Volume III, Prepared by SFWMD and FDEP, March 1, 2022
- Phosphorus Flux in the Taylor Creek Stormwater Treatment Area: Potential Causes and Recommended Control Strategies, Prepared by SFWMD, April 2016
- Appendix 5C-1: Evaluation of Inundation Depth and Duration Threshold for Typha domingensis (Cattail) Sustainability: Test Cell Study, In: 2022 South Florida Environmental Report – Volume I, Prepared by SFWMD and FDEP, March 1, 2022
- Lake Okeechobee Watershed Construction Project, Phase II Technical Plan, Prepared by FDEP, SFWMD and the Florida Department of Agriculture and Consumer Services (FDACS), February 2008
- Treatment Wetlands, 2nd Edition, by Robert H. Kadlec and Scott D. Wallace, 2009
- Development of Design Criteria for Stormwater Treatment Areas in the Northern Lake Okeechobee Watershed, Prepared by Wetland Solutions, Inc., October 2009
- Restoration Strategies Regional Water Quality Plan, Prepared by SFWMD, April 27, 2021

SECTION 2

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SECTION 2

SITE CONDITIONS

General information related to existing conditions at the proposed Project site is presented in this section. Information regarding climate, geology and soils, topography, environmental conditions, and communications is provided.

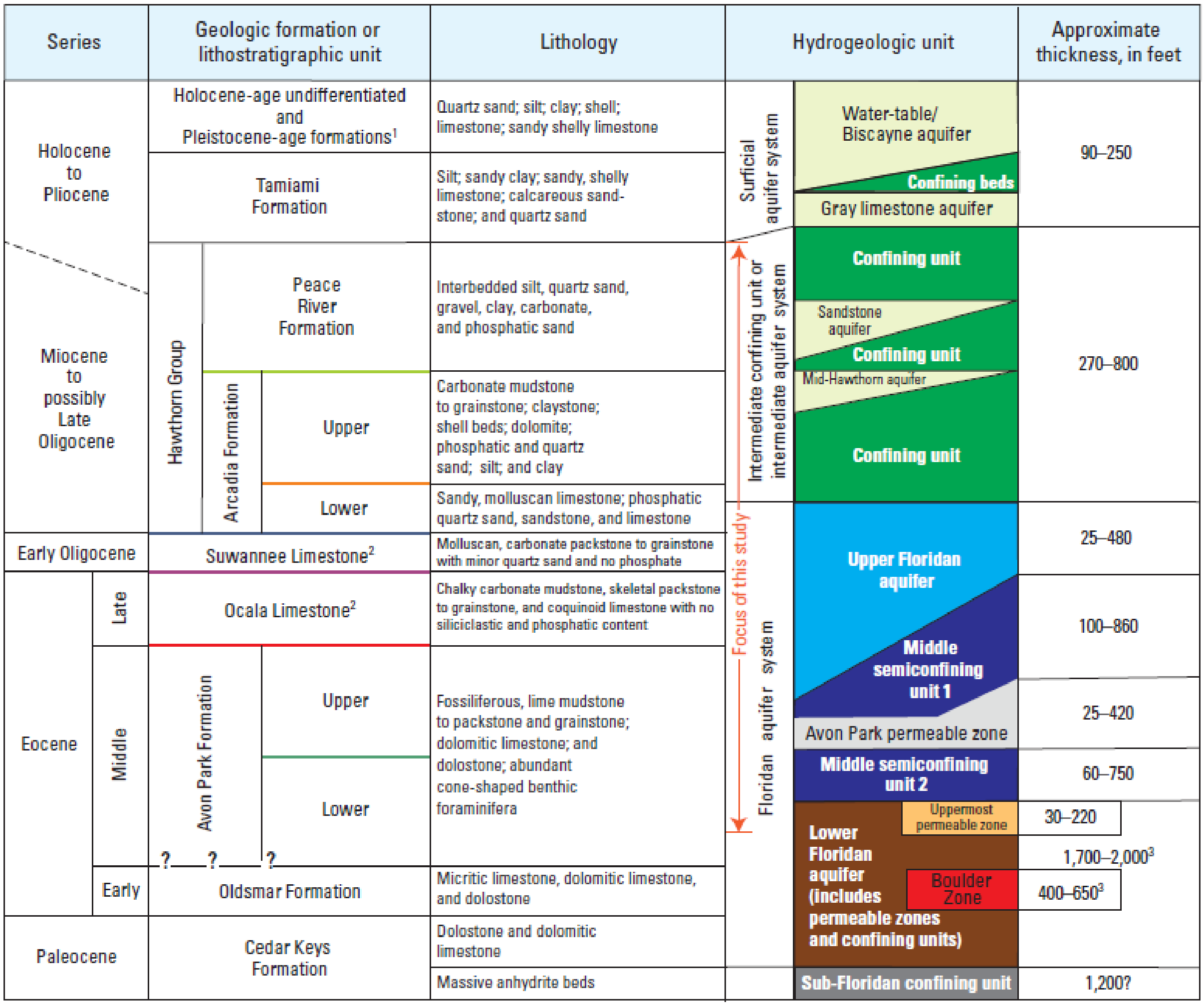
2.1 Regional and Local Climate

The Lower Kissimmee Basin and surrounding region has a subtropical climate with average annual rainfall of 51 inches (in). Approximately 72 percent of the rainfall is in the wet season (May through October). The remaining 28 percent occurs during the dry season (November through April). Wet season precipitation is mainly convective, while dry season precipitation is primarily associated with frontal systems. The mean daily temperature from May to October is 80 degrees Fahrenheit (°F), and from November to April it ranges from 60°F to 70°F (Abtew, 1992).

2.2 Regional and Local Geology and Soils

2.2.1 Regional Geology

A review of geologic/geotechnical literature (Reese, 2014) indicates that Okeechobee County is underlain by up to 13,000 ft of shallow marine sedimentary deposits. Deposition of these sediments was the result of changes in paleoclimate and tectonic events that caused sea level fluctuations of various magnitudes. These sea level fluctuations also caused the current land surface topography to be formed. A chart with descriptions and characteristics of the hydrogeologic and geologic soils of Okeechobee County is provided in Figure 2-1.



[1] **Pleistocene-age formations in southeastern Florida**–Pamlico Sand, Miami Limestone, Anastasia Formation, Fort Thompson Formation, Key Largo Limestone

[2] **Geologic unit missing in eastern parts of study area**

[3] **Thicknesses are from the southeastern Florida part of the study area**

Figure 2-1. Hydrogeologic and geologic units of Okeechobee County

(Reese, 2014)

The regional geology of Okeechobee County for the Quaternary and upper Tertiary systems range in age from recent to Pleistocene Age to Miocene Age sediments. The Recent to late-Pleistocene-Age sediments are undifferentiated and cover the county with a range of 2 to 100 ft in thickness of fine to medium quartz sand, shells, and organic soils. From the west coast to Lake Okeechobee, the sand sediments thicken, and thin organic-rich soils appear below. Below the sands and organic soils, the mid to early Pleistocene Anastasia Formation is present. The Anastasia differs in composition as a whole from a coquina to pure sand, but in Okeechobee County, it is composed of semi-consolidated fine quartz and carbonate sands, shell beds, and thin discontinuous layers of sandy limestone or sandstone. The Anastasia ranges in thickness from 200 ft at the coast to 20 ft at Lake Okeechobee, where it then merges with the Fort Thompson Formation. The Fort Thompson is prevalent in Okeechobee County and its composition varies throughout, but is primarily composed of marine sands, shell beds, limestone, or sandstone. The formation ranges in thickness from 50 to 20 ft. These deposits lie unconformably with the upper Miocene/Pliocene sediments of the Caloosahatchee and Tamiami formations. The Caloosahatchee and Tamiami sediments are comprised of shelly quartz sand, silty shelly sand, and indurated clayey sand with occasional thin interbedded limestone and sandstone. These sediments vary in thickness from 10 to 140 ft throughout the region and have an overall trend that thins to the west. The Surficial Aquifer System (SAS) occurs within this shallow stratigraphic section, which is generally between 200 and 250 ft thick. Clayey sediments of the Peace River Formation of the Hawthorne Group form the base of the SAS.

The Hawthorn Group sediments of middle Miocene Age lies unconformably below the upper Miocene/Pliocene sediments, which underlie all of Okeechobee County. The Hawthorn Group is composed of dark green to white phosphatic clay containing silt and quartz sands interbedded with layers of sandy limestone and chert. Its formational contact ranges from 100 to 200 ft below ground surface near the lake and dips to 400 ft near the coast. The thickness of the Hawthorn Group ranges from 250 to 800 ft.

2.2.2 Sinkhole Analyses

A sinkhole is a landform that occurs due to the subsidence or collapse of sediment or rock as underlying limestone or dolostone layers are dissolved by slightly acidic groundwater. Sinkhole activity is common in some areas of peninsular Florida; however, sinkhole development is not a common geological event in Okeechobee County.

Sinkhole type, development, and distribution in Florida (Sinclair and Stewart, 1985) categorizes Florida into four area types (Areas I through Area IV) based on thickness of overburden, frequency of occurrence, and dominant type and morphology of sinkholes. The project site is located in Area II, where sinkholes are few, generally shallow and broad, and develop gradually. Cover-subsidence sinkholes evidenced by surface ground depressions are the dominant type within this area. Cover-subsidence sinkholes tend to develop gradually over geologic time where the covering sediments are permeable and contain sand. In areas where cover material is thicker or sediments contain more clay, cover-subsidence sinkholes are relatively uncommon and therefore may not be seen frequently. They are smaller and thus may go undetected for long periods. In cover-subsidence events, the void in the deeper rock is filled by sand sediments raveling downward from above. Eventually, the ground surface often shows a gentle circular depression. If a relatively thick layer of impermeable sediments covers the limestone, there may not be a surface expression of a subsurface collapse.

Geographic data (FGS, 2022) from the Florida Geological Survey (FGS) showing Subsidence Incident Reports (SIR) in the vicinity of the Project site were reviewed. Subsidence incidents include both sinkhole activity and other subterranean events formed by other mechanisms, such as expansive clay, organic layers, and anthropogenic events. The data indicates there was one SIR within a 10-mi radius of the Project site. The FGS subsidence incident data contain only those events reported to the FGS. There was no information available for review regarding the SIR near the Project site.

The probability of a sinkhole developing in the vicinity of the Project site is considered low.

2.2.3 Site Soil Survey

Soil data from the United States Department of Agriculture (USDA) Natural Resources Conservation Service's (NRCS) soil surveys of Okeechobee, Highlands, and Glades counties and the Web Soil Survey website (NRCS, 2018a, 2018b, 2022) were reviewed as part of the investigation. The portions of the Highlands and Okeechobee counties' soils map for the Project area is presented in Figure 2-2.

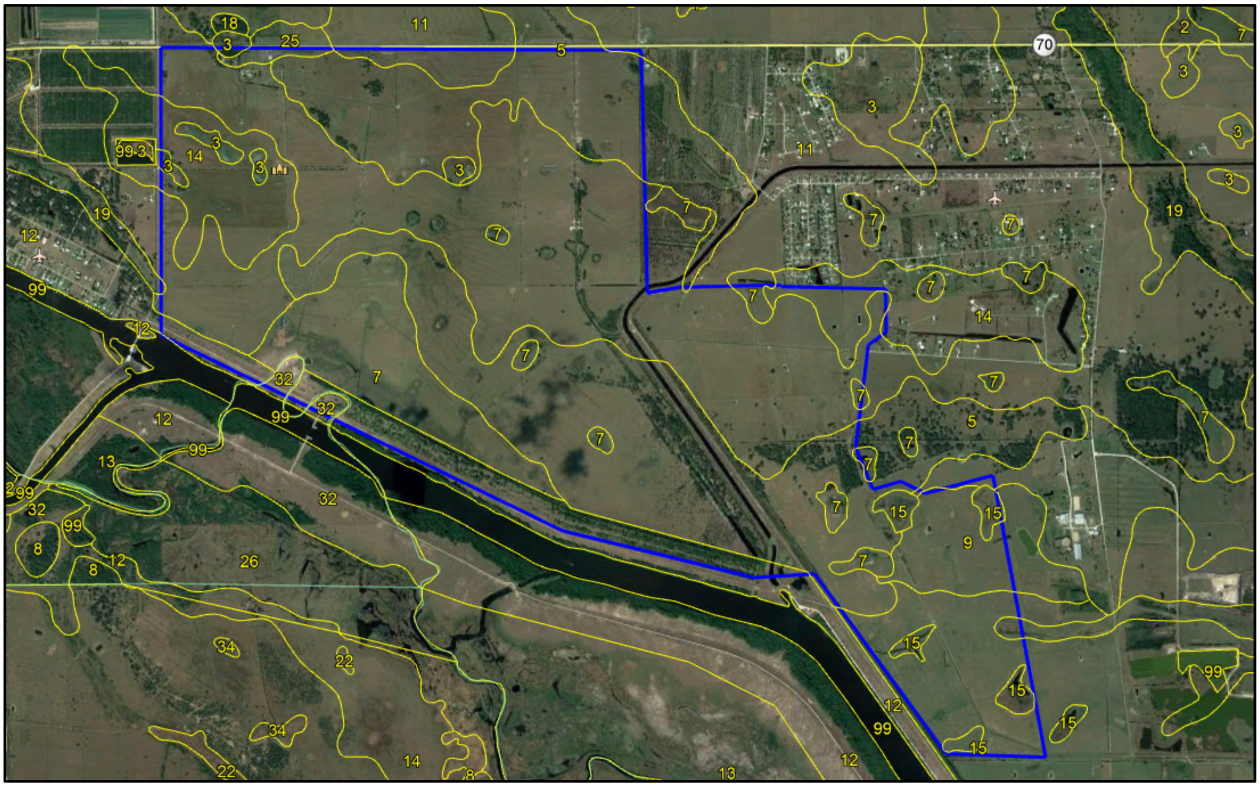


Figure 2-2. Soil surveys of Glades, Highlands, and Okeechobee counties

The mapped surficial soil units that will impact construction at the Project site were identified as Basinger and Placid soils (Figure 2-2 map symbol 3); Valkaria fine sand (map symbol 5); Floridana, Riviera, and Placid soils (Map Symbol 7); Riviera fine sand (map symbol 9); Immokalee fine sand (map symbol 11); Myakka fine sand (map symbol 14); Okeelanta muck (map symbol 15); Wabasso fine sand (map symbol 25); and Arents (map symbol 32). Brief descriptions of the surficial soil types to be encountered are:

- Basinger and Placid soils (map symbol 3): This soil is very poorly drained and occurs in swamps, marshes, and low-lying areas. Typically, the surface layer is very dark gray fine sand about 2 in thick. The subsurface layer, which extends to a depth of about 18 in, is light gray fine sand. The subsoil, which extends to a depth of about 36 in, is brown and light brownish gray fine sand and brown fine sand having yellowish brown mottles. The substratum is light brownish gray fine sand to a depth of 80 in.
- Valkaria fine sand (map symbol 5): This soil is poorly drained and occurs in sloughs, on low flats, in depressions, and in poorly defined drainageways. Typically, the surface layer is very dark gray fine sand about 6 in thick. The subsurface layer, which extends to a depth of about 19 in, is grayish brown fine sand. The subsoil, which extends to a depth of about 46 in, is fine sand. In the upper part, it is brownish yellow and has dark yellowish-brown mottles. In the lower part, it is yellowish brown and has very pale brown, dark grayish brown, and dark yellowish-brown mottles. The substratum is white fine sand to a depth of 80 in.
- Floridana, Riviera, and Placid soils (map symbol 7): This soil is very poorly drained and occurs in freshwater swamps and marshes and in low lying areas. Typically, the surface layer is fine sand 7 in thick. It is black in the upper part and gray in the lower part. The subsurface layer, which extends to a depth of about 22 in, is also fine sand. In the upper part, it is light brownish-gray and has brownish-yellow mottles. In the next part, it is light gray and has brownish yellow mottles. In the lower part, it is very pale brown and has brownish-yellow mottles. The subsoil,

which extends to a depth of 40 in, is fine sandy loam. In the upper part, it is light gray and has tongues of very pale brown sand. In the lower part, it is mixed light gray and gray and has yellowish-brown mottles. The substratum to a depth of 80 in is grayish brown sandy loam.

- Riviera fine sand (map symbol 9): This soil is poorly drained and occurs in broad, low areas of flatwoods, in sloughs, and in poorly defined drainageways. Typically, the surface layer is fine sand about 7 in thick. It is black in the upper part and gray in the lower part. The subsurface layer, which extends to a depth of about 22 in, is also fine sand. In the upper part, it is light brownish-gray and has brownish-yellow mottles. In the next part, it is light gray and has brownish-yellow mottles. In the lower part, it is very pale brown and has brownish-yellow mottles. The subsoil extends to a depth of 40 in. In the upper part, it is light gray fine sandy loam and has tongues of very pale brown sand. In the lower part, it is mixed light gray and gray fine sandy loam that has yellowish-brown mottles. The substratum is grayish-brown sandy loam to a depth of 80 in.
- Immokalee fine sand (map symbol 11): This soil is poorly drained and occurs in broad areas of flatwoods. Typically, the surface layer is very dark gray fine sand about 6 in thick. The subsurface layer, which extends to a depth of 35 in, is fine sand. In the upper part, it is gray and has gray and dark gray mottles. In the lower part, it is light gray. The subsoil, which extends to a depth of 54 in, is fine sand. In the upper part, it is black grading to dark reddish-brown. In the lower part, it is dark reddish-brown. The substratum to a depth of 80 in is dark brown fine sand that has pale brown and light gray mottles.
- Myakka fine sand (map symbol 14): This soil is poorly drained and occurs in broad areas of flatwoods. Typically, the surface layer is black fine sand about 4 in thick. The subsurface layer, which extends to a depth of 27 in, is fine sand. It is gray in the upper part and white in the lower part. The subsoil, which extends to a depth of about 46 in, is a mix of very dark grayish-brown and black fine sand. The substratum is brown fine sand to a depth of 80 in.
- Okeelanta muck (map symbol 15): This soil is very poorly drained and occurs in small depressions and large freshwater marshes and is subject to ponding much of the year. Typically, the surface layer is muck (i.e., highly organic soils) 28 in thick. It is black in the upper part and very dark brown in the lower part. The substratum to a depth of 80 in is gray sand that has dark gray mottles.
- Wabasso fine sand (map symbol 25): This soil is poorly drained and occurs in areas of flatwoods, on flood plains, and in depressions. Typically, the surface layer is very dark gray fine sand 4 in thick. The subsurface layer, which extends to a depth of about 16 in, is gray fine sand. The subsoil, which extends to a depth of 32 in, is fine sand. In the upper part, it is dark reddish brown and has reddish-brown and gray mottles. In the lower part, it is very pale brown and has light brownish-gray mottles. The upper part of the substratum is light gray fine sandy loam that has pockets of yellowish-brown, the next part is yellowish-brown fine sandy loam that has gray mottles, and the lower part to a depth of 80 in is light gray and gray fine sand that has brownish-yellow and olive-brown mottles.
- Arents (map symbol 32): This soil consists of unconsolidated soil material that has been excavated from major canals and deposited alongside the channel. This map unit is primarily along the edge of the HHD, Harney Pond canal, and flood control canals along the Kissimmee River. The texture and thickness of the layers of the Arents are highly variable. Typically, the surface layer is olive gray fine sand about 2 in thick. Below this are various layers of fine sand or loamy material from former natural soil horizons. Colors vary from black, gray, and olive brown to white. Some layers contain various amounts of shell fragments.

The USDA and NRCS soil classifications are based on an interpretation of aerial photographs and widely spaced hand-auger borings. Borders between mapping units are considered approximate, and the transition between soil types may be very gradual. Areas of dissimilar soils can occur within a mapped unit; however, the soil survey provides a good basis for an initial evaluation of shallow soil conditions in the area and can provide an indication of changes that may have occurred due to land filling, excavation, and other activities at the site. See Section 9 (Geotechnical Investigation and Design) for further information regarding the site soils.

2.3 Seismic Environment

Seismic Hazard Maps produced by the United States Geological Survey (USGS) (USGS, 2022) display earthquake ground motions for various probability levels. Figure 2-3 indicates approximate peak ground acceleration (g) for the LKBSTA project of 0.02 g to 0.04 g (for 1 hertz spectral acceleration) with a 2-percent-in 50-years probability of exceedance at the Project. This value is very low in comparison to seismically active areas in the U.S., and no active faults are identified in the southeastern Florida region.

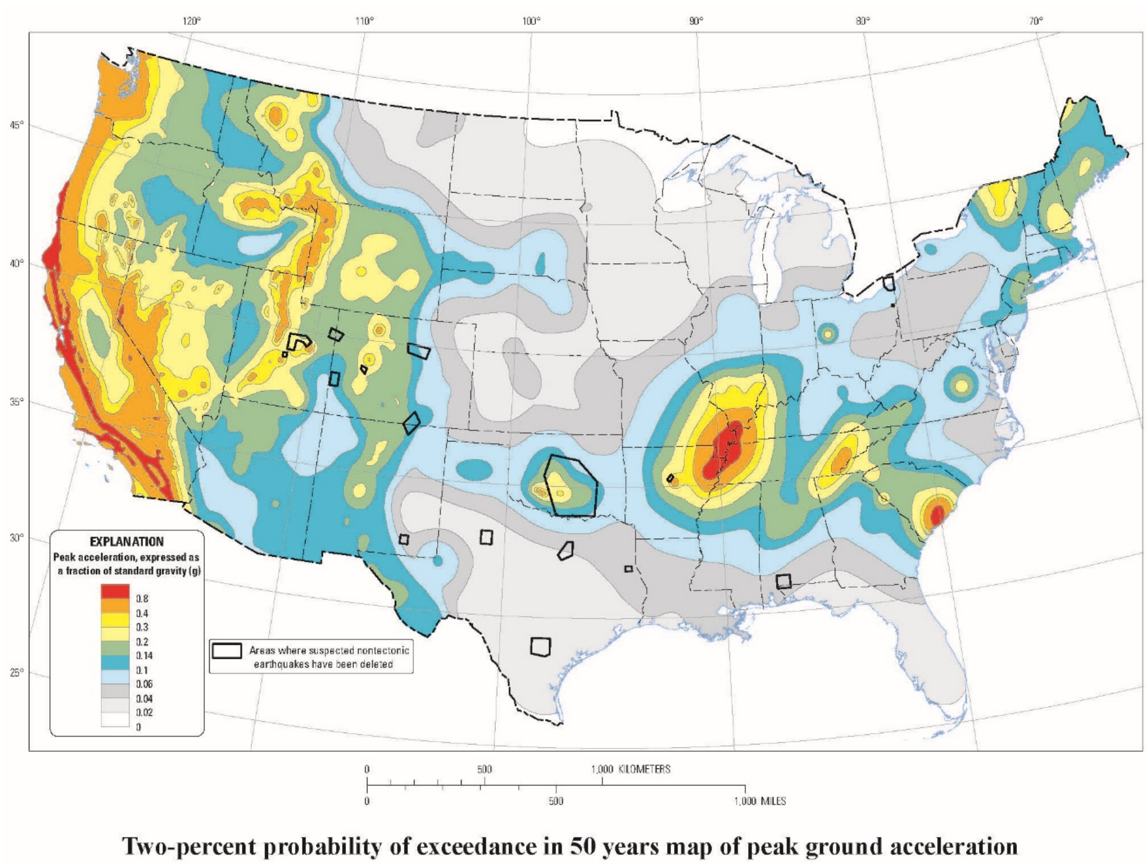


Figure 2-3. Earthquake ground motion map

(USGS, 2022)

In addition, the Florida Bureau of Geology (Lane, 1983), indicates that Okeechobee County is in Zone 0, which is an area with no reasonable expectancy of earthquake damage.

The Federal Emergency Management Agency (FEMA) (FEMA, 2022) and International Building Code (IBC) classify areas by Seismic Design Categories (SDC), which reflect the likelihood of experiencing earthquake shaking of various intensities. The southeast Florida area has an SDC ‘A’ classification, which is described as “very small probability of experiencing damaging earthquake effects.”

The risk of an earthquake large enough to cause structural damage occurring in the vicinity of the Project site is considered extremely low.

2.4 Topography and Survey

General site topography is flat with a slight fall to the south in both the eastern and western portions of the Project site. Site surface elevations vary from 15 to 30 ft North American Vertical Datum of 1988 (NAVD88), as shown on Figure 2.4. For reference, relative to historical elevation data commonly provided in National Geodetic Vertical Datum of 1929 (NGVD29), the conversion from NGVD29 to NAVD88 is -1.21 ft for onsite elevations.

In 2019, EIP acquired an aerial light detection and ranging (LiDAR) topographic survey of the Project site with the goal of collecting LiDAR data to enable the development of map products with absolute horizontal accuracies of 0.3 ft Root Mean Square Error (RMSE) (calculated as 1/10,000 x Altitude) or better and absolute vertical accuracies of 0.15 ft RMSE or better (typically 0.25 ft or better at the 95% confidence interval). LiDAR and digital color imagery was acquired on October 14, 2019. The total LiDAR survey area was approximately 5.5 square mi, with a nominal buffer of 300 ft outside of the Project boundary for the imagery.

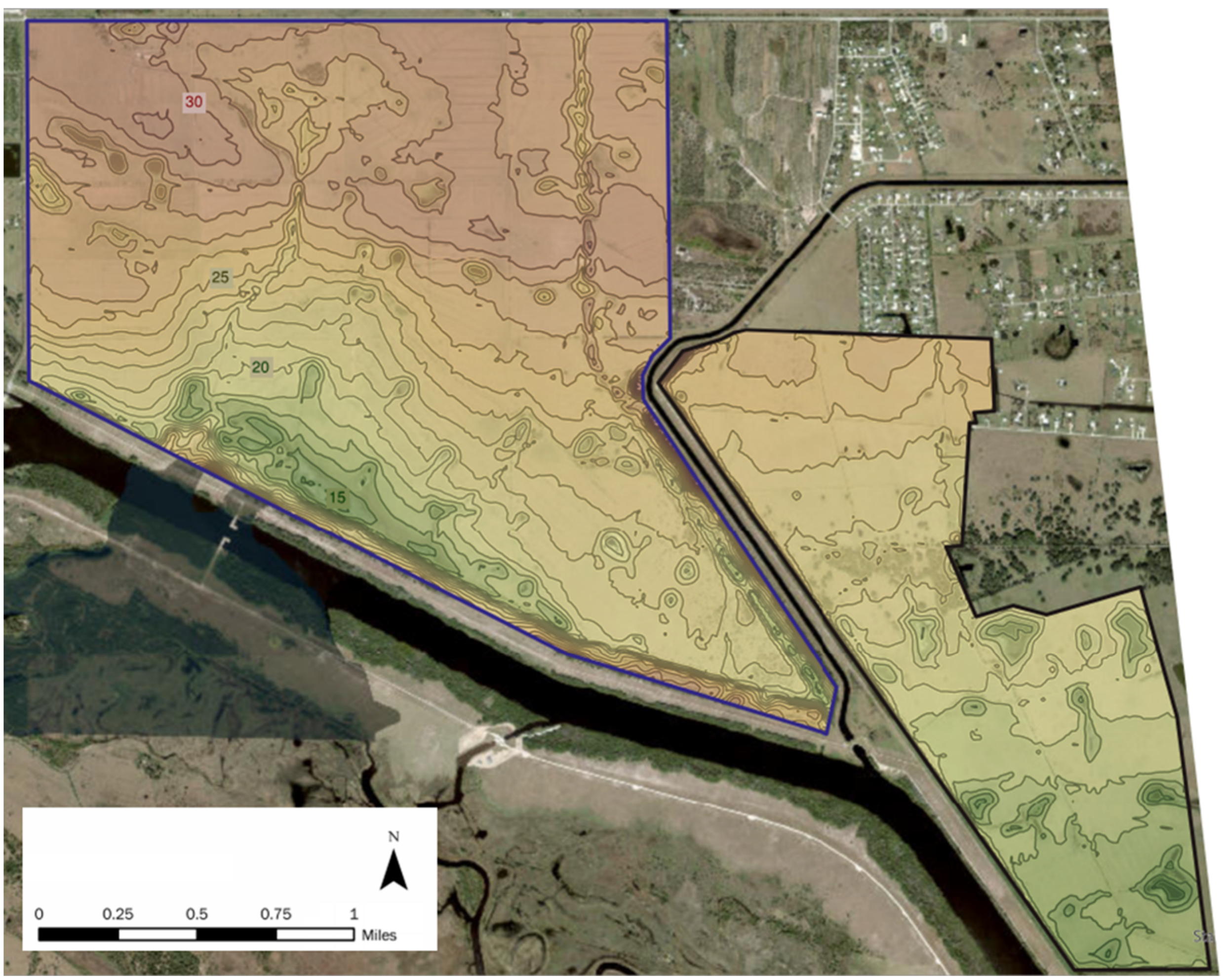


Figure 2-4. General site topography (1-ft contours)

Source: Pickett Surveying 2021 LiDAR

In 2021, channel cross sections and a bathymetric survey of the areas currently underwater within the portion of the L-62 canal adjacent to the Project site were conducted. The bathymetric data for the underwater areas was then integrated with the light detection and ranging (LiDAR) data to generate half-foot contours for the entire Project site. The topographic survey of the Project site (including LiDAR and bathymetry point files) and the corresponding LiDAR survey report were delivered to SFWMD in May 2022 with the Reconnaissance Study Final Report and are also included in Appendix 2. 2.

2.5 Surface Water Hydrology

The surface water hydrology for the Project area is comprised of flow contributions from two basins: S-154 and the S-154C. Data from the Atlas of the Lower Kissimmee River and Istokpoga Surface Water Management Basins (Abtew, 1992) indicate that the S-154 basin has an area of 49.4 square mi. The L-62 canal is an interceptor dike and canal on the west side of the basin and begins southwest of the Okeechobee County airport at the CSX railroad approximately 3,800 ft west of U.S. Highway 98. It is approximately 7 mi long and has a crown width of 10.0 ft. The elevation of the levee ranges from approximately 29 ft to 33 ft (NAVD), and has side slopes of 1 vertical to 3 horizontal per the L-62 construction drawings (USACE, 1973). Water flows to the west, and the design discharge at this end is 1,000 cubic feet per second (cfs) from the drainage area. The S-154 structure is the major WCS in the S-154 basin. Its purpose is to maintain upstream water control stages of 22.1 ft (NAVD) (SFWMD, 2022d) and pass the design flood without creating channel-eroding velocities downstream. Structure S-154 also prevents back flow from Lake Okeechobee during excessive stages in the lake. The design discharge for S-154 is 1,000 cfs, which is 30 percent of the Standard Project Flood (3,333 cfs) as listed in the SFWMD Structures Books (SFWMD, 2022d).

The entire S-154C basin (3.4 square mi) is within the project area. The major WCS in this basin is S-154C, and its purpose is to maintain optimum upstream water stages of 14.8 ft (NAVD).

Based on the 40-year DBHYDRO data record, the maximum daily flow recorded for S-154 was 1,378 cfs on October 7, 2004. The maximum daily flow recorded for S-154C was 163 cfs on May 18, 2018. The maximum stage of Lake Okeechobee was 17.3 ft (NAVD) in October, 1995.

The annual average rainfall is 51 in (Abtew, 1992) and, according to the technical paper by Abtew *et.al*., (2003), the annual evapotranspiration range for the area is between 50 in and 52 in.

The site received offsite flows on the west and east sides of the L-62 canal. On the west side, flows enter the site from the north. On the east side, flows enter the site from the north and east. For the 100-year 3-day storm event, the peak flow entering the west and east sides of the site is 650 cfs and 590 cfs, respectively. See Section 5 (Hydrology) for additional information. Flows will be routed around the proposed site via seepage canals. A portion of the flow on the west side of the site will be designed to be pumped into the STA system. See Section 8 (Hydraulics) for additional information.

2.6 Ground Water Hydrology

Okeechobee County, including the project area, is underlain by two aquifer systems: the upper SAS and the underlying Florida Aquifer System, which is separated by the intermediate confining unit (Reese, 2014). SAS within Okeechobee County consists of undifferentiated sediments that overlie the clayey sands of the intermediate confining unit (Hawthorn Formation). The SAS consists of a relatively thin unit of undifferentiated sand, silt, and shell of Pleistocene to Holocene ages overlying more permeable zones composed of thin layers of sand, silt, and clay interbedded with layers of limestone, coarse sand, and shell of Pliocene age. In some places, the permeable beds are absent and wells are drilled into the deeper sand, shell, and silt layers of Late Miocene Age. These deeper wells also are classified as SAS wells. Gamma-ray logging and lithologic description of test wells in

Okeechobee County indicates that the thickness of the SAS in Okeechobee County increases southward from about 150 ft to more than 250 ft but is generally between 200 ft and 250 ft thick. The underlying Floridan aquifer system consists of limestone and dolomitic limestone and ranges from 2,700 ft to 3,000 ft in thickness in Okeechobee County.

2.7 Environmental Conditions

Current environmental conditions of the site are described below.

2.7.1 Air Quality

Current air quality conditions are normal and typical for rural peninsular Florida. Any environmental air permits needed to meet all air quality standards for proposed PS will be acquired.

2.7.2 Noise

Minor, short-term increases in noise during construction are expected. PS construction and operation would result in long-term, negligible increases in noise; however, the PS location in the center of this large landholding is expected to ameliorate potential for offsite noise effects.

2.7.3 Vegetation

Site vegetation consists predominantly of pasture improved with bahia grass. Sabal palm trees are scattered throughout the site, particularly in the south on the west side of the L-62 canal. Sabal palm and oak trees are found on the sidecast of the ditches and along the berms of the L-62 canal. The disturbed wetlands contain a variety of species often associated with cattle grazing, including grasses, sedges, and rushes. Exotic species and nuisance species are prevalent, including cattail, Bermuda grass, and smut grass. Brazilian pepper and Australian pine were also present along the edges of the wetlands and ditches. Exotic species will be either eliminated during construction or treated post-construction as a condition of the permits.

2.7.4 Fish and Wildlife

Wildlife species observed on the site are sparse due to the intensive agricultural practices. A variety of songbirds and wading birds have been observed, as well as a few duck, raptor, and several bat species. = Utilization by terrestrial species is low due to the cattle grazing; traces of raccoons, armadillos, coyotes, and wild hogs are the only species that have been noted. Within the wetlands, alligators, turtles, and frogs are present, albeit in low densities due to the low quality of the wetlands and ditches onsite. Very few fish species, such as mosquitofish, have been observed in the wetlands and/or ditches because these features are sometimes dry during the year.

2.7.5 Threatened/Endangered Species

Although Okeechobee and Highlands counties have the potential for a variety of state and/or federal threatened and/or endangered species, surveys over the past 2 years have identified the few species that occur or have the potential to occur onsite. These species include the bald eagle, Audubon's crested caracara, gopher tortoise, and Florida burrowing owl. Several species have a high potential to occur onsite, including the eastern indigo snake, American alligator, osprey, and a variety of listed wading birds. The West Indian manatee has been observed adjacent to the site in the C-38 canal and could potentially utilize the L-62 canal. A more detailed discussion of each species is provided in Section 19 (Special Considerations).

2.7.6 Socioeconomics

The current socioeconomic circumstances of the site are focused on cattle ranching as the primary economic driver. After construction, the primary economic driver will be STA operation, with some potential recreational uses as deemed appropriate by District Land Use.

The current dominant land use is improved pasture for cattle ranching with the associated improvements (e.g., drainage ditches, wells, buildings to house equipment, etc.) and two single-family homes and associated improvements. The Project includes within its boundary the L-62 canal.

2.7.7 Recreation Resources

The current recreational capacity of the site is limited to passive recreational activities, such as bird watching (particularly along the berms of the canals), horseback riding, camping, and potentially hunting (e.g., feral hogs or ducks).

2.7.8 Aesthetics

The current aesthetic value of the site is based on open space associated with rural agricultural land uses. This component will continue to be investigated as the Project progresses.

2.7.9 Cultural Resources

A cultural resource assessment survey (CRAS) has been completed for the site (ACI, 2022). One new archaeological site was identified, a bone midden (8OB00383) that is not considered eligible for listing in the National Registry of Historic Places (NRHP); however, the project has been redesigned to avoid this archaeological resource. Several buildings more than 50 years old were identified; none of these buildings is eligible for listing with the NRHP. Two features associated with the HHD, including the L-62 canal and associated berms and borrow areas, are potentially eligible for listing with the NRHP because these features will be 50 years old in 2023 (before the Project's expected construction start) and may be adversely affected by the project design. These features will be addressed during the permitting process.

2.7.10 Hazardous, Toxic, and Radioactive Waste

A Phase 1 Environmental Site Assessment (ESA), provided to the District on May 24, 2022, was completed in general conformance with the scope and limitations of the American Standard Test Method International (ASTM) Standard E 1527-13 to identify any Recognized Environmental Concerns (REC) in connection with the LKBSTA site. This assessment included the presence, or likely presence, of any hazardous substances or petroleum products on the site under conditions that indicate an existing or past release or a material threat of release into the ground, groundwater, surface water, or structures on the site. The assessment included an evaluation to the extent practicable of the past and present land uses at the site and on adjacent properties.

The Phase 1 ESA revealed evidence of RECs in connection with the LKBSTA site. The storage building on the West Property contains various fuels, chemicals, and oils. Some of these substances were observed to be improperly stored both inside the building and outside under an awning. Some evidence of historical staining was observed within the building. In addition, historical aerial photography of the eastern property showed evidence of row-cropping activities. Soil sampling and analysis is underway to determine if any environmental impacts occurred from these activities. There was no evidence of controlled RECs (past release that has been addressed but material allowed to remain on site) in connection with the LKBSTA site.

The results of the Phase 1 ESA identified the following environmental concerns, housekeeping, and/or developmental conditions that will need to be addressed via removal or closure.

- Site reconnaissance revealed evidence of solid waste on the eastern, northern, and central portions of the East Property in the form of discarded tractor tires, irrigation hosing, a metal casing, scrap wood, and corrugated plastic pipe. A series of at least 15 water wells is present on the site. Three septic systems were also observed, and a monitoring well was reported to be located on the East Property but was not observed during field reconnaissance.
- The site reconnaissance also revealed evidence of aboveground storage tanks (AST) on the East and West properties. The ASTs included plastic and metal containers that contained molasses, diesel fuel, and unknown contents with no labels present. The site reconnaissance also revealed some 55-gallon (gal) drums containing household waste, of which two were labeled as hydraulic fluid, and another labeled as antifreeze, which appeared full.

Follow-up site work is underway that will be reported in a Phase 2 ESA Report expected to be completed by March 2, 2023.

2.8 Proximity of Project to SFWMD Communications Network Infrastructure

At least one steel microwave tower exists near the LKBSTA site. It is regarded as “in-sight,” thus allowing line-of-sight communication. The existing microwave tower and base station at S-65E appears to be a good candidate for line-of-site communications. The operation and capacity of S-65E to carry LKBSTA data will be confirmed with SFWMD information technology and microwave groups during detailed design. A secondary microwave tower and base station will be identified and confirmed during that same time.

2.9 SFWMD Communications Network Infrastructure Integration

A 60-ft concrete pole with primary and secondary radio frequency (RF) path, primary to a tower that is closer to the site and the secondary to the next closest tower, will integrate the PS to the SFWMD supervisory control and data acquisition (SCADA) system.

2.10 Conditions of Existing Communications Facilities

It is assumed the existing communications base stations and towers considered for the Project are operational and that they will accommodate connection with the Project.

2.11 Proposed Future Activities

2.11.1 Preliminary and Final Design

Site assessment activities will continue, as needed, through progression of the Project to final design. These activities may include additional topographic survey work, geotechnical exploration, and refinement of data used for modeling.

Several topics will continue to be investigated during preliminary design, in particular, permitting requirements for air quality and threatened and endangered species, as well as socioeconomic circumstances and site aesthetic values.

The Phase 2 ESA fieldwork is complete and all lab results have been received. The team is completing its initial data assessment and Screening Level Ecological Risk Assessment (SLERA), including assessing the need to analyze select provisional samples to supplement the assessment, for the False Negative Assessment. If a False Positive Assessment is needed, a subset of the provisional samples will require analysis, which would include completing any additional field sampling that may be appropriate. The draft Phase 2 ESA Report will be drafted and provided to the

District for review and discussion. At this stage, a determination will be made whether soil mitigation or remediation is recommended. The final Phase 2 ESA Report will incorporate District comments and be submitted to FDEP and U.S. Fish and Wildlife Service (USFWS) in March 2023. If deemed necessary, additional work may be needed, specifically a SLERA (a more comprehensive Ecological Risk Assessment for sediments that exceeded screening-level criteria), a Draft Soil Management Plan, a Draft Corrective Action Plan, and a Draft Corrective Action Implementation Report.

SECTION 3

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SECTION 3

GENERAL DESIGN REQUIREMENTS

The LKBSTA Project includes the design of approximately 27 miles of canals, one PS, an estimated 14 gated WCSs for controlling inflow to and outflow from the STA and PES treatment cells, two large gated concrete structures on the proposed rerouted L-62 canal, ungated cast-in-place culvert structures, gated HDPE pipe conduits with cast-in-place or precast concrete headwalls, and support structural features such as precast concrete electrical buildings, generator buildings, gate control buildings, slabs, retaining walls, etc. This section summarizes general design criteria, including survey standards, design service life, units of measurement, and industry codes and standards that will be used throughout the design process for all discipline work associated with designing these elements.

3.1 Project Limits and Site Datums

The Project site is primarily located in unincorporated Okeechobee County with a small portion along the C-38 canal within the footprint of the historic Kissimmee River alignment located in Highlands County. The property has primarily been used as improved pasture for cattle ranching and is zoned for agricultural use. The proposed discharge point at S-154 is located approximately 6 mi upstream of Lake Okeechobee on the C-38 canal, also known as the channelized Kissimmee River (Figure 3-1).

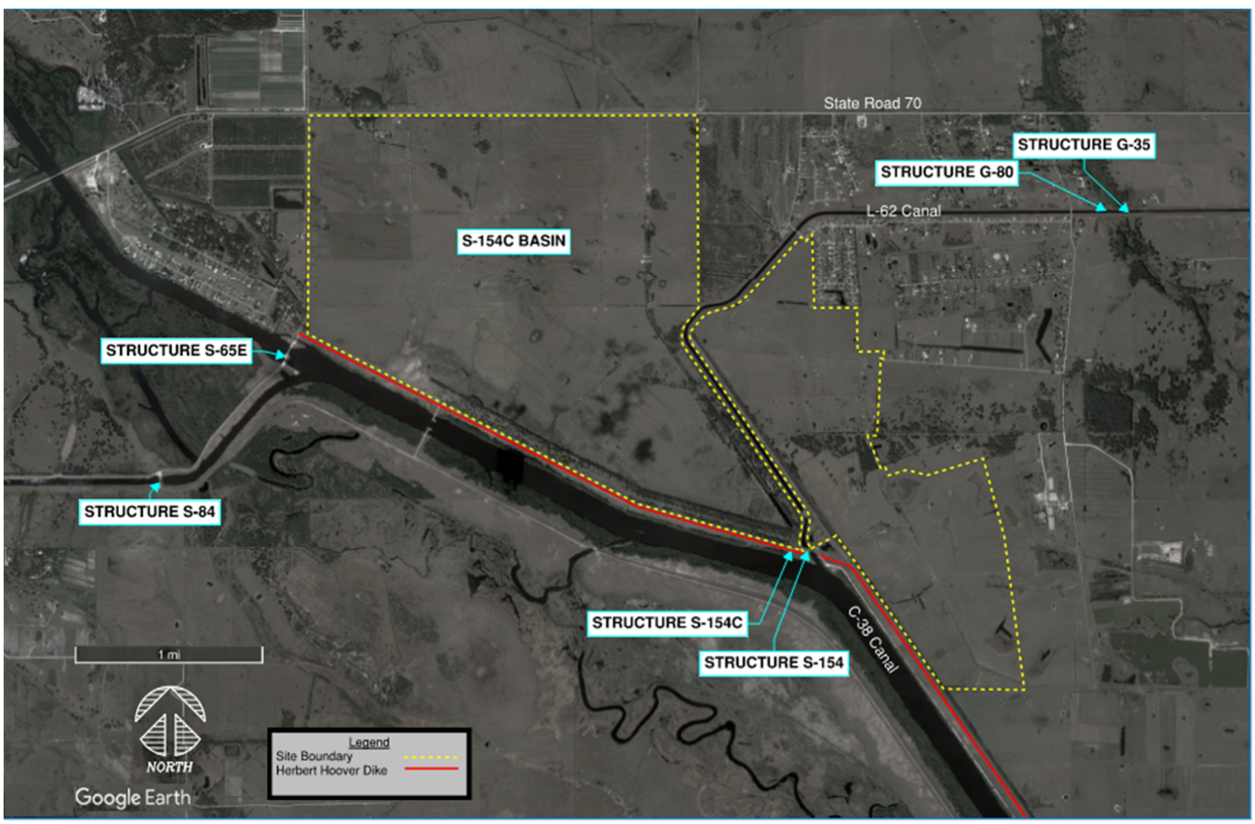

Figure 3-1. Project limits

The site is bisected by the existing L-62 canal and covers approximately 3,400 ac of existing improved pasture. It is bounded by SW 128th Avenue to the west; SR 70, the L-62 canal, and single-family residential properties to the north; pastureland and a tree farm to the east; and the C-38 canal and pastureland to the south.

Existing structures are located on both portions of the site, i.e., west and east of the existing L-62 canal. The west portion has structures related to current cattle ranching practices that house various pieces of farming equipment and two residential structures just south of SR 70 in the northwest corner of the site. The east portion of the site has a residential structure and horse stable located near the entrance to the property on Southwest 87th Terrace.

Existing site utilities include electrical, water wells, irrigation systems, and stormwater culverts. Electrical lines were observed in both the east and west portions of the site near homes, barns, and outbuildings.

3.1.1 Project Site Datums

The governing horizontal survey site datum for the Project is the National Spatial Reference System (NSRS 11) Florida State Plane, East Zone, United States Foot coordinate system. The vertical site datum used for the Project is NAVD88. Several data sources used for reference are based on a horizontal datum of North American Datum of 1983 (NAD83) and a National Geodetic Vertical Datum of 1929 (NGVD29). All shifts to updated coordinate systems have been performed according to National Geodetic Standards Coordinate Conversion and Transformation Tool.

3.2 Project Operational Requirements

Appendix 5 contains the Draft Project Operations Manual, which outlines the preliminary operational requirements for the Project.

3.3 Design Service Life

According to USACE Engineering Manuals (EM) 1110-2-3104, EM 1110-2-3102, and Major Pumping Station Engineering Guidelines, the design life for the new PS and WCSs will be a minimum of 50 years. With proper maintenance, this design life can be achieved by following the guidance in these documents. Mechanical equipment will require rehabilitation or replacement over the design life. The pumps and bar screens will operate continuously and will require regular maintenance. The engine generators will only operate intermittently and thus should not require a major overhaul during the design life. The slide and roller gates will be constructed out of 316 stainless steel to resist corrosion, but gate seals and actuators will require intermittent maintenance. The architectural and structural design of the PS and WCSs will include elements that will require minimum maintenance and repair over the design life. The design elements for the structural, civil, mechanical, electrical, instrumentation and control, architectural, plumbing, and heating, ventilation, and air conditioning (HVAC) are further detailed in Section 14 (Mechanical Design) and Section 17 (Architectural).

3.4 Units and System of Measurement

Although some of the formulas used for wind set-up and wave run-up calculations use numerical equations that use metric units, the majority of information presented in this DDR is based on English units and the English system of measurement.

3.5 Codes and Standards

Given that LKBSTA will ultimately be owned and operated by SFWMD and will affect existing USACE facilities, the Project will adhere to both USACE and SFWMD design standards. The following is a list of organizations, codes and standards that may be applicable in the design of this project. Additional design-discipline specific codes and standards are listed under each individual design discipline section.

- SFWMD
  - SFWMD Engineering Standards, most recent version
  - SFWMD Engineering and Construction Bureau Design Standards, most recent version
  - SFWMD Pump Station Design Guidelines, latest edition
- Design and Construction of Levees, EM 1110-2-1913, USACE, April 2000
- Hydraulic Design Criteria, USACE, 18th Issue, Revised November 1987
- Design of Small Dams, Bureau of Reclamation, 3rd Edition, 1987
- American Association of State Highway and Transportation Officials (AASHTO)
- American National Standards Institute, Inc. (ANSI)
- ASTM
- Uniform Federal Accessibility Standard
- State of Florida, Florida Department of Transportation (FDOT) Standard Specifications for Road and Bridge Construction, Latest Edition
- Florida Building Code (FBC), Latest Edition
- Steel Sheet Piling Design Manual, United States Steel Corp., dated September 1970
- Hydraulic Institute of Hydraulics, latest edition
- Applicable Florida Power & Light (FPL) requirements

3.6 Proposed Future Activities

3.6.1 Preliminary and Final Design

Design requirements may continue to be refined as the Project is progressed through preliminary and final design phases.

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SECTION 4

REGULATORY CONSIDERATIONS

This section addresses the environmental permits and authorizations necessary for the Project at the federal, state, and local level. The final composition of these permits and authorizations will depend on consultation with the agencies and results of field data collection. It is anticipated that EIP and the District will be co-applicants for all federal and state permits. For the USACE Section 408 authorization, the District will submit all required documents and/or applications, with support provided by the EIP team. It is not anticipated that any FDOT permitting for road access will be needed. The construction contractor will be responsible for obtaining all local county permits, as necessary.

4.1 Environmental Permitting

4.1.1 List of Regulatory Agencies with Jurisdiction Over the Project

- FDEP – state-regulated waters and wetlands, assumed federally regulated wetlands, stormwater, flood zone, water quality, air quality
- SFWMD – water use
- USACE – retained federally regulated wetlands, navigable waters, HHD
- USFWS – federally threatened and endangered species
- Florida Fish and Wildlife Conservation Commission (FFWCC) – state-listed species
- Florida State Historic Preservation Office (SHPO) – archaeological and historical resources

4.1.2 Applicable Regulations Affecting the Project

- NEEPP – Chapter 373.4595 Florida Statutes (F.S.)
- State Wetland Permitting – Chapter 373, F.S.; Chapter 62-330, (Florida Administrative Code (F.A.C.)
- State Wetland Delineation – Chapter 62-340, F.A.C.
- State Wildlife Regulations – Chapter 379, F.S., FFWCC - Division 68A, F.A.C., T&E Species Rule 68A-27, F.A.C.
- State Sovereign Submerged Lands – Chapter 253, F.S.; Chapter 18-21, F.A.C.
- State Historical Preservation Regulations – Chapter 267, F.S.; Chapter 1A-46, F.A.C.
- Section 401 of the Clean Water Act – 33 United States Code (U.S.C.) 1341
- Section 404 of the Clean Water Act – 33 U.S.C. 1344
- Section 14 of the Rivers and Harbors Appropriations Act of 1899, as amended, and codified in 33 U.S.C. 408 (aka USACE Section 408 authorization)
- Section 10 Rivers and Harbor Act – 33 U.S.C. 403
- Section 106 of the Historical Preservation Act – 36 Code of Federal Regulations (CFR) Part 800
- Section 7 of the Endangered Species Act – 7 U.S.C. 136

4.1.3 Summary of Permits and Approvals Required for the Project

- Lake Okeechobee Protection Permit (LOPP)

This permit will address wetland impacts and mitigation, stormwater, flood zone, and water quality monitoring. Based on pre-application meetings with FDEP, the wetlands are expected to be considered self-mitigating. Stormwater and flood zone effects will be addressed in the design. Water quality monitoring is included in this permit. Through this permitting process, archaeological and historical resource impacts will be coordinated with SHPO, as will sovereign submerged land authorization.

- Section 404 Permit

  This permit will address impacts to federally jurisdictional wetlands. At this time, it has not been determined whether FDEP will issue a 404 permit for the assumed wetlands, or whether USACE will issue the 404 permit since the project connects to retained waters (the C-38 canal). It may be possible to "split" the 404 permits such that FDEP issues a 404 permit for the assumed waters and USACE issues a 404 permit for the retained waters. A pre-application meeting has been requested with USACE.

- Section 408 Authorization

  This authorization will address the effects of construction in association with any modification to the existing federal features. SFWMD will submit the application to USACE, unlike all of the other permits where EIP will be a co-applicant with SFWMD.

- Section 10 Rivers and Harbors Act

  This permit will address effects of the connection to the C-38 canal on navigable waters. It is anticipated that this permit will be processed in conjunction with the USACE 404 permit.

- Section 7 Endangered Species Act Incidental Take Permit

  This permit will be issued by USFWS but will be coordinated by either FFWCC if FDEP processes the 404 permit, or by USACE if the 404 is retained by USACE. This permit will address the impacts and mitigation, if necessary, to federally listed species.

- FFWCC Incidental Take Permit

  This permit will address the impacts and mitigation to state-listed species and may be coordinated through FDEP if they process the 404 permit.

4.1.4 Environmental Project Design Constraints

Project design constraints related to environmental considerations such as wetlands, listed wildlife species, and archaeological and historical resources were evaluated during the Reconnaissance Study.

The onsite wetlands are extremely low quality due to the ongoing agricultural operations; therefore, avoidance of these wetlands was not necessary. Furthermore, because the wetlands within the stormwater treatment cells will be much higher quality, the project is considered "self-mitigating" from the perspective of wetland function. A preliminary wetland determination has been completed (see map in Appendix 2). This wetland determination will be further refined prior to permit application submittal to include the vast network of agricultural ditches (surface waters).

The project footprint has been adjusted to avoid impacting crucial nesting habitat (i.e., nest trees) for the bald eagle and the Audubon's crested caracara. Wildlife surveys are continuing for territorial usage by caracaras, which will be used during the permitting process to quantify the amount of take, if any. Conservation measures and compensatory mitigation, as appropriate, will be determined in the future. Other listed wildlife species whose habitat cannot be avoided, such as the gopher tortoise, will be relocated to a protected area offsite. The results of the listed wildlife surveys that have been completed to date are shown on the maps in Appendix 2.

The only archaeological resource identified onsite, a bone midden, has been avoided and will be permanently protected. The figure in Appendix 2 shows the bone midden and its 50-ft buffer. The historical structures (i.e., buildings) have been determined to be not significant, and these structures will be removed. The Project will affect the HHD, including the L-62 canal and associated dikes and borrow areas, the impacts of which will be addressed during the permitting process.

4.2 Construction Permitting and Coordination

4.2.1 Communications Tower Regulations

Applicable federal, state, and local codes will be reviewed as detailed design progresses. Potential federal agency regulations may include the Federal Aviation Authority (FAA) and Federal Communication Commission (FCC).

4.2.1.1 FAA

It is not anticipated that FAA review will be required for project communications tower(s)/pole(s). Pending final design of these systems, a review of applicable regulations will be completed.

4.2.1.2 FCC

It is anticipated that the Project's communications system will be integrated into the existing SFWMD communications system, for which SFWMD already holds the appropriate FCC licenses and approvals. Existing protocols will be reviewed during detailed design and incorporated as applicable.

4.2.2 Radio Frequency Operation Regulations

RF operations will be designed within the existing framework of the SFWMD system. Microwave, RF, and two-way radio are potential technologies to be evaluated.

4.2.2.1 Microwave Systems

It is not anticipated that new microwave systems will be included. Instead, Project communications will be integrated into the SFWMD microwave system under existing protocols.

4.2.2.2 RF Telemetry SCADA Systems

It is likely that a communications pole of a yet-to-be-determined height will be needed for RF communications. RF communications will follow SFWMD standards and be integrated into the District's existing system.

4.2.2.3 Voice Two-way Radio Systems

Two-way radio communications systems are not anticipated; however, applicable regulatory requirements will be evaluated if these systems are incorporated into the Project.

4.3 Land Use/Zoning

Planning and zoning approvals will be required for the STA cells and the PS. The land use change involves standard rural land uses (agricultural, cattle ranching) and would not represent significant changes to land use patterns. The STA will be constructed in two counties—Okeechobee and Highlands. The PS will be located in Okeechobee County. The time required to obtain these permits is yet to be determined but is not expected to be more than several months.

4.3.1 Communications Tower

The communications tower/pole zoning modifications/approval will take place concurrently with the STA cells and associated infrastructure.

4.4 Environmental Assessment

A stand-alone Environmental Assessment (EA) document prepared to National Environmental Policy Act (NEPA) standards is typically not generated because the federal permitting process (i.e., 401, 404, 408, etc.) includes these components. All federal actions must comply with NEPA; therefore, these federal permit applications include the information necessary to complete the NEPA compliance review. The information typically contained in an EA can be found in this DDR in Section 2 (Site Conditions), Section 4 (Regulatory Considerations), and Section 19 (Special Considerations). Other sections will also include relevant information.

4.5 Proposed Future Activities

4.5.1 Preliminary Design

The Project's permitting process has already been initiated via pre-application meetings with several of the regulatory agencies. Other pre-application meetings have been scheduled but not yet occurred, and the pre-application meeting with USACE has been requested but not yet scheduled. Outstanding permit-related items include determining the lead agency for the 404 permit (FDEP or USACE, or both if the 404 permit is split).

The wetland determination/delineation is ongoing and will be completed prior to permit submittal. Discussions with USFWS and FFWCC are ongoing regarding continued species monitoring, impacts, and mitigation alternatives; these discussions will continue before, during, and after (during construction) the permits have been obtained.

Table 4-1 presents anticipated permits, including major components of those permits, the time frame for initiation and completion of each permit, as well as the Responsible Party for permit application preparation; a list of the table's acronyms follows.

Table 4-1. Permit Timing					
Permit No.	Permit/Approval Name	Lead Agency	Approach	Timeframe	Responsible Party
1	NEEPA/LOPP and State 404	FDEP	Complete FDEP application for consolidated NEEPA/LOPP and State 404 program (for assumed waters in L-62), which includes subcomponents of JD, T&E, SHPO, SSL, UMAM, 401, stormwater, flood zone, soils.	Initial Application 6/2023; Complete Permit 12/2025	EIP
1a	Jurisdictional Wetland Delineation	FDEP	Complete delineation of all wetlands and surface waters (including ditches) within the project area, which could include wetlands/surface waters outside ownership boundary, such as L-62 canal, etc.	Initial Application 6/2023; Complete Permit 12/2025	EIP
1b	USFWS Threatened and Endangered Species Consultation	USFWS	Initiate informal consultation with USFWS regarding BA for eventual formal consultation for BO and potential ITP for Eastern Indigo Snake and Audubon's Crested Caracara.	Informal consultation 11/2022; Initial Application 6/2023; Complete Permit 12/2025	EIP
1c	SHPO/Section 106	SHPO-DHR	Initiate informal consultation with SHPO regarding CRAS for avoidance of archaeological resources (midden) and impact of historical resources (HHD and ditches).	Informal consultation 11/2022; Initial Application 6/2023; Complete Permit 12/2025	EIP/SFWMD
1d	SSL Authorization	FDEP	Complete FDEP application for SSL authorization; submit with NEEPA/LOPP/404.	Initial Application 6/2023; Complete Permit 12/2025	EIP
1e	UMAM Analysis	FDEP	Complete UMAM analysis of all wetlands and surface waters (including ditches) with the project area (which could include wetlands/surface waters outside ownership boundary such as L-62 canal, etc.).	Initial Application 6/2023; Complete Permit 12/2025	EIP
1f	401 Water Quality Certification	FDEP	Complete 401 Water Quality Certification as component of NEEPA/LOPP/404 permit.	Initial Application 6/2023; Complete Permit 12/2025	EIP
1g	Stormwater Analysis	FDEP	Complete Stormwater Analysis as component of NEEPA/LOPP/404 permit.	Initial Application 6/2023; Complete Permit 12/2025	EIP
1h	Flood Zone Compensation Analysis	FDEP	Complete Flood Zone Compensation Analysis as component of NEEPA/LOPP/404 permit.	Initial Application 6/2023; Complete Permit 12/2025	EIP
1i	Soil Sampling	FDEP	Complete Soil Sampling Analysis as component of NEEPA/LOPP/404 permit.	Initial Application 6/2023; Complete Permit 12/2025	EIP

Table 4-1. Permit Timing					
Permit No.	Permit/Approval Name	Lead Agency	Approach	Timeframe	Responsible Party
2	408 Authorization by USACE	USACE	Complete Review Plan (submitted by SFWMD) for 30% complete construction plans to initiate review by USACE. Include Backfill Plan for GT relocation from federal levees (L-62).	Review Plan Submittal 1/2023; Complete Application 6/2023; Complete Permit 12/2025	EIP/SFWMD
2a	H&H Analysis	USACE	Complete H&H Analysis as component of 408 Authorization.	Review Plan Submittal 1/2023; Complete Application 6/2023; Complete Permit 12/2025	EIP
3	FFWCC State-Listed Species ITP	FFWCC	Initiate informal consultation with FFWCC regarding eventual ITP for gopher tortoise and Florida Burrowing Owl.	Informal consultation 11/2022; Initial Application 6/2023; Complete Permit 12/2025	EIP

Acronyms

BA	biological assessment
BO	biological opinion
CRAS	cultural resource assessment survey
ESA	Endangered Species Act
FDEP	Florida Department of Environmental Protection
FFWCC	Florida Fish and Wildlife Conservation Commission
GT	gopher tortoise
H&H	hydrology and hydraulics
ITP	Incidental Take Permit
JD	jurisdictional determination
LOPP	Lake Okeechobee Protection Permit
NEEPA	Northern Everglades and Estuaries Protection Act
Section 106	Section 106 of the Historic Preservation Act
Section 7	Section 7 of the Endangered Species Act
SFWMD	South Florida Water Management District
SHPO	State Historic Preservation Office
SSL	sovereign submerged lands
T&E	threatened and endangered species
UMAM	Uniform Mitigation Assessment Method
USFWS	United States Fish and Wildlife Service
401	Section 401 of the Clean Water Act
404	Section 404 of the Clean Water Act
408	Section 14 of the Rivers and Harbors Appropriation Act of 1899, as amended, and codified in 33 U.S. Code Section 408

4.5.2 Final Design

Following preliminary design, permit applications will be submitted to the various agencies for consideration and review. Processing of all necessary permits will be completed prior to the conclusion of final design.

SECTION 5

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SECTION 5

HYDROLOGY

This section presents the design criteria, analysis methodology, analysis results, and future work related to hydrology. Because this Project is strictly a water quality treatment system controlled by pumping and will not function as a flood control facility, limited hydrologic analyses have been completed. These include:

- Historical flow and gate operation records from DBHYDRO, which is SFWMD's corporate environmental database that stores hydrologic, meteorologic, hydrogeologic and water quality data, have been used for assessing flows in the L-62 canal.
- Hydrologic analysis, which has been completed for offsite drainage that flows into the Project site.
- Direct rainfall runoff associated with the Project site will be incorporated into flood routing.
- Hazard Potential Classification (HPC) assessment, which includes use of rain depth when determining hazard class and resultant embankment height.

5.1 Design Criteria

The hydrologic analyses generally conform to applicable standards and reference manuals. The following list of standards and reference manuals are applicable to hydrologic analyses:

- Environmental Resource Permit Information Manual, SFWMD
- SFWMD Design Criteria Memoranda (DCM)
  - DCM-1 – Hazard Potential Classification
  - DCM-2 – Wind and Precipitation Design Criteria for Freeboard
  - DCM-3 – Spillway Capacity and Reservoir Drawdown Criteria
- SFWMD Engineering Design Standards for Water Resource Facilities – Design Guidelines
- USACE Engineering Manuals (EM), including:
  - EM 1110-2-1415 – Hydrologic Frequency Analysis
  - EM 1110-2-1417 – Flood-Runoff Analysis
  - EM 1110-2-1420 – Engineering and Design Hydrologic Engineering Requirements for Reservoirs

5.1.1 L-62 Canal and WCS-9

SFWMD's DBHYDRO flow data, design flow references from the Structures Books (SFWMD, 2022d) and the Central and Southern Florida Project (USACE, 1963; USACE, 1969) were used to identify original and current design criteria for the L-62 canal and the S-154 structure, including design and flood flows. No separate hydrologic modeling was completed for the drainage areas contributing flows to the L-62 canal and WCS-9.

5.1.2 STA System

The STA system is intended to operate as an offline system from SFWMD drainage infrastructure. Flows into the STA system will be conveyed via inflow PS. The inflow pumping rate is based on general water availability and water quality treatment performance, not storm hydrology. Pumping rates are further discussed in Section 8 (Hydraulics).

Note that direct precipitation will still produce runoff that will need to be routed through the system. Direct runoff is considered precipitation that falls on all inflow canals, treatment cells, outflow canals, and portions of embankments that drain into inflow canals, treatment cells, and outflow canals. Direct runoff from small storms may be able to be routed through the WCSs already provided for treatment flows, as those structures will have capacity beyond the treatment flow rates. During large storms (e.g., 100-year recurrence interval), overflow weirs will be provided as part of the outlet for each treatment cell to convey storm flows to the outflow canals. The outflow canals will be sized to convey the storm flow to S-154. Storm flow will then be conveyed through the S-154 to C-38 canal. In addition, outflow canals may be sized to convey dam-breach flows if determined to meet the Hazard Potential Classification, discussed below.

5.1.3 Hazard Potential Classification

The DCM-1 methodology is used to determine the HPC. The HPC results in the selection of an inflow design flood (IDF). If the HPC is low, the IDF is the 100-year/24-hour storm event. If the HPC is Significant or High, the IDF is the Probable Maximum Precipitation (PMP) event.

A review of rainfall data for the Project area (National Oceanic and Atmospheric Administration [NOAA] Atlas 14) (NOAA, 2013) defines rainfall from the 100-year, 24-hour storm event to be 9.2 in (0.77 ft) for a low hazard classification. NOAA Hydrometeorological Report No. 51 (NOAA, 1980) defines the PMP rainfall depth to be 55.7 in (4.64 ft).

HPC analyses to determine embankment freeboard requirements are described in Section 8 (Hydraulics). Future work during preliminary design will verify the HPC.

5.1.4 Wind Set-up/Wave Run-up and Freeboard

The DCM-2 methodology is used to determine the Wind Set-up/Wave Run-up and Freeboard. Like DCM-1 and the HPC, the IDF used in Wind Set-up/Wave Run-up and Freeboard is dependent on the HPC selected. The same storm events listed in the preceding section apply to the Wind Set-up/Wave Run-up and Freeboard: If the HPC is low, the IDF is the 100-yr/24-hour storm event. If the HPC is Significant or High, the IDF is the PMP event.

Wind set-up and wave run-up analyses to determine embankment freeboard requirements are described in Section 8 (Hydraulics).

Future work during preliminary design will verify the HPC, which may prompt updates to the Wind Set-up/Wave Run-up and Freeboard analysis.

5.1.5 Offsite Flows

Offsite stormwater flows that currently drain onto the Project site during a flood storm event were modeled to determine peak and total flows that will need to be routed around the site. Hydrologic modeling of two design storm events were completed. The 100-year, 3-day event was modeled to quantify flood flows to be used in the conveyance capacity analysis presented in Section 8 (Hydraulics). Both rain depth (10.27 in) and storm distribution were based on the requirements referenced in the SFWMD Environmental Resource Permit Applicants Handbook. This criterion applies to seepage canals on both the east and west sides of the L-62 canal.

The 2-year 1-day event was also modeled to quantify peak flows from offsite that will be routed to the STA system for treatment. This recurrence interval is commonly used to estimate flows for stormwater quality treatment facilities. Specific to this project, it corresponds to the flow generated from offsite areas west of the L-62 canal that would be collected by the seepage canal and conveyed to the PS intake. This routing is further discussed in Section 8 (Hydraulics). The 2-year 1-day storm event depth of 3.95-in was obtained from NOAA Atlas 14 (NOAA, 2013); distribution is based on the Florida Type II event. Both storm distributions are presented on Figure 5-1.

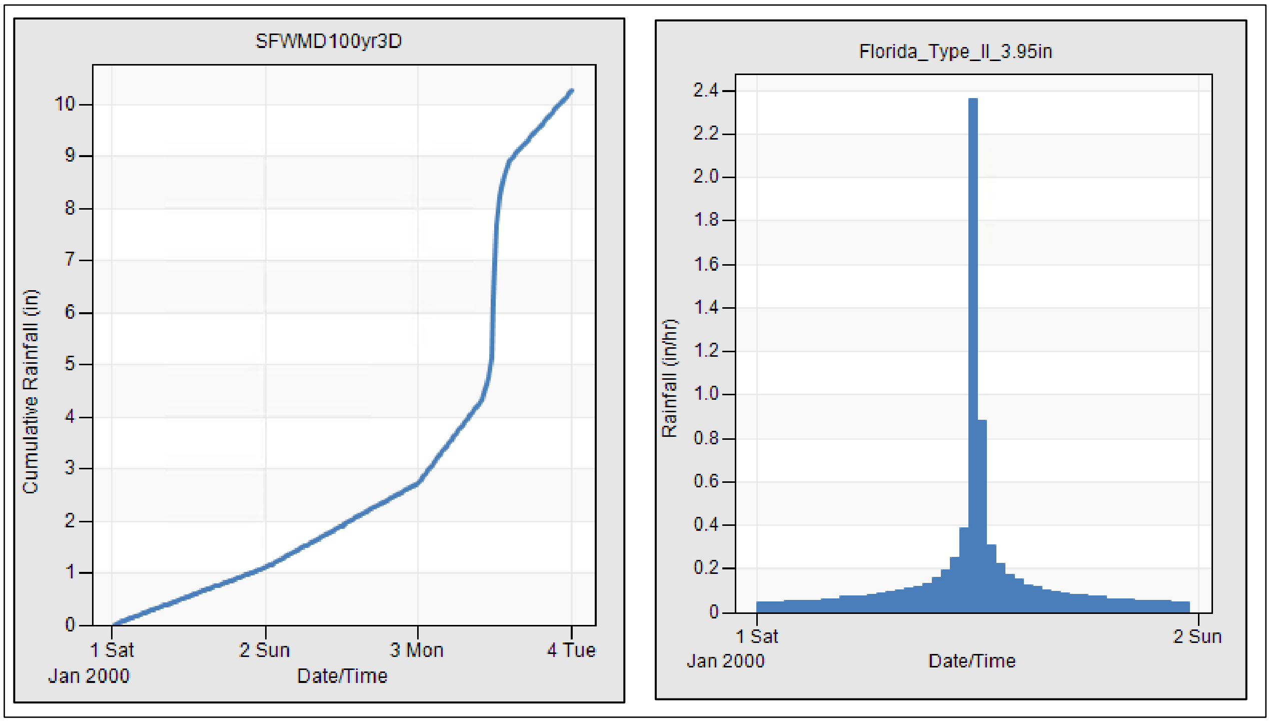

Figure 5-1. 100-year/3-day and 2-year/1-day storm distributions for LKBSTA

5.2 Analysis Methods

5.2.1 L-62 Canal, WCS-9, and WCS-13

SFWMD's DBHYDRO flow data, and design flow references from the Structures Book (SFWMD, 2022d) and the Central and Southern Florida Project (USACE, 1963; USACE, 1969) were used to identify original and current design criteria for the L-62 canal and the S-154 structure, including design and flood flows. No separate hydrologic modeling or analysis was completed for the drainage areas contributing flows to the L-62 canal and WCS-9. The data were used for hydraulic analyses of the L-62 canal and WCS-9 to confirm that conveyance capacity criteria are being met as presented in Section 8 (Hydraulics).

5.2.2 STA System

During preliminary design, methodologies in the Personal Computer Stormwater Management Model (PCSWMM) will be used to route direct precipitation falling on the STA system. Precipitation will be routed through the treatment cells and through WCSs and overflow weirs that will be sized and located in preliminary design. Flows will then be routed downstream through outflow canals and discharged into the C-38 canal.

PCSWMM, which is developed and supported by Computational Hydraulics International (CHI), uses the Environmental Protection Agency's Stormwater Management Model (SWMM) engine to complete hydrologic modeling. The NRCS Unit Hydrograph Method was used to estimate infiltration.

Resultant hydrographs for the selected storm events were then routed through the hydraulic portion of the PCSWMM model; see Section 8 (Hydraulics).

For SWMM information beyond that included in this summary document, the Open SWMM website provides links to all SWMM reference manuals and user's manual (CHI, 2022a).

5.2.3 Hydraulic Structure Flow Capacity Analysis

An exercise was completed to demonstrate that the resultant combination of existing and proposed hydraulic structures can provide the same or additional conveyance and management capacity as existing conditions.

Under proposed conditions, WCS-9 will provide the same function as S-154 currently does as described in the structure book (SFWMD, 2022d), primarily maintaining a normal pool elevation between 21.6 and 22.6 ft (NAVD) in the L-62 canal upstream of the site and conveying the design flow of 1,000 cfs (SFWMD, 2022d) in L-62 to the C-38 canal. The proposed WCS-13 structure, downstream of WCS-9, will also be sized to convey the 1,000 cfs design flow.

The existing S-154C structure will no longer be needed as its entire drainage area becomes part of the STA area. The STA area will then discharge through the S-154 structure, which is effectively being repurposed as the outlet from the STA into the C-38 canal.

The limits of the STA area east of the existing L-62 canal currently drains south via the LD-4 ditch. In addition, some offsite area east of the site drains onto the site, and then south across the site and to the LD-4 ditch. As a result of STA construction, the STA portion of this drainage area will be effectively rerouted into the STA and discharged out to the C-38 canal vis the repurposed S-154 structure. As a result, the net flow into the LD-4 canal will be reduced.

Figures 5-2a and 5-2b graphically present the existing and proposed conditions as it relates to drainage areas and hydraulic structures. The combination of the WCS-9/WCS-13 plus the repurposed S-154 structure provided additional capacity when compared to existing conditions that have only the S-154 structure. Further refinement of the WCS-9 and WCS-13 structures and hydrologic and hydraulic calculations demonstrating added capacity will be included in Preliminary Design.

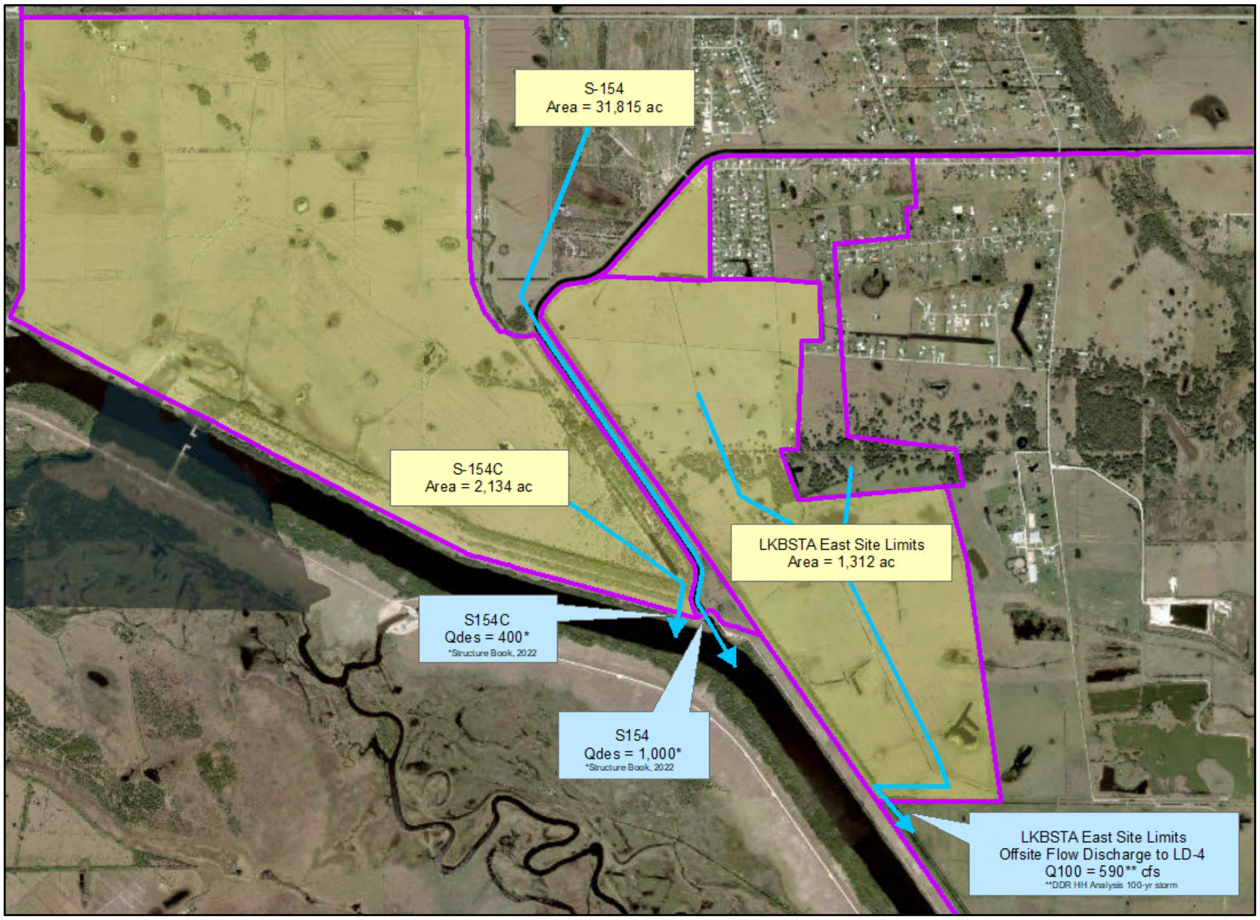

Figure 5-2a. Existing Drainage Area / Hydraulic Structure Configuration

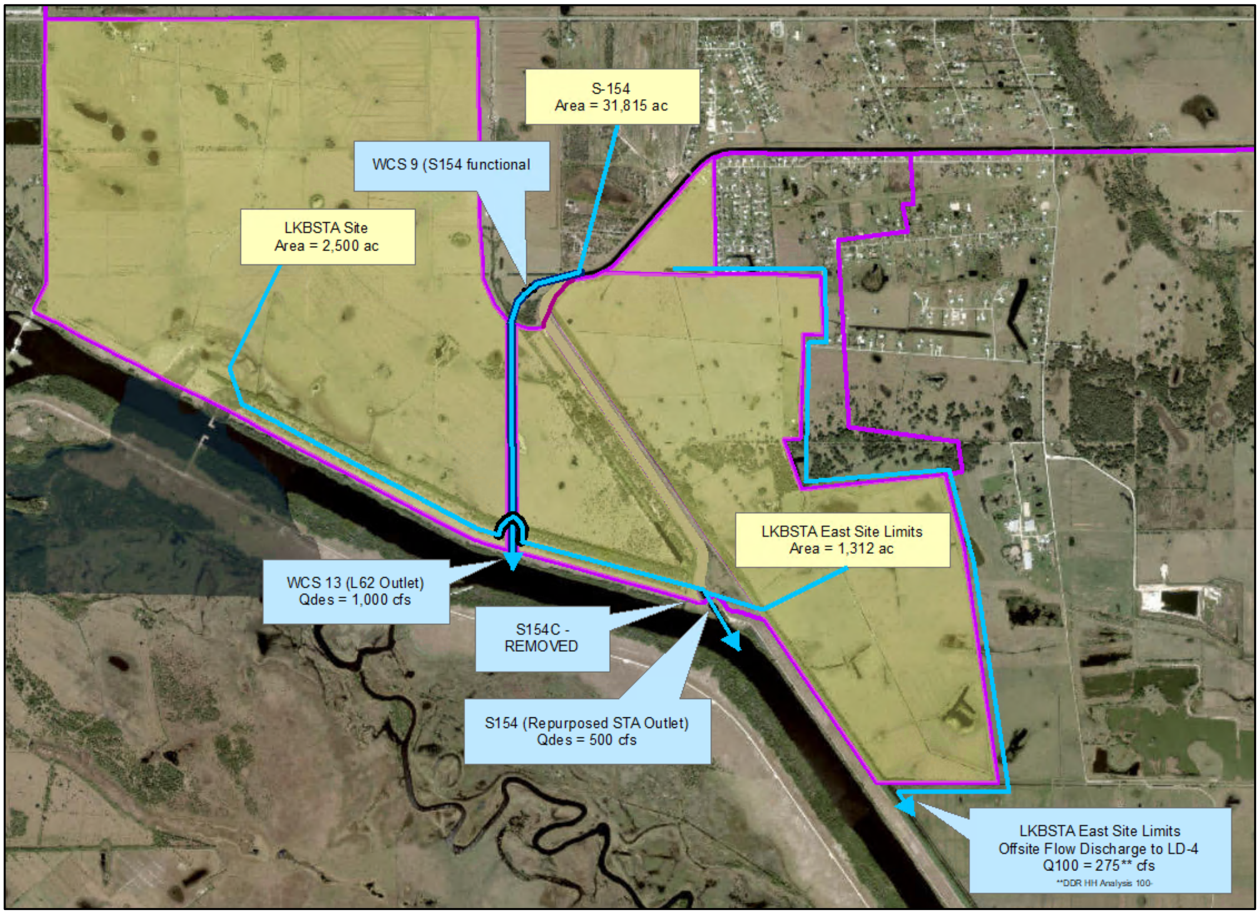

Figure 5-2b. Proposed Drainage Area / Hydraulic Structure Configuration

5.2.4 HPC

The HPC methodology is described in Section 8 (Hydraulics).

5.2.5 Wind Set-up/Wave Run-up and Freeboard

The Wind Set-up/Wave Run-up and Freeboard analyses are described in Section 8 (Hydraulics).

5.3 Results

The PCSWMM model files, along with a list of the file names and associated modeling scenarios, are included in Appendix 4. The modeling results presented in the following subsections are taken from these model files.

5.3.1 Offsite Flows West of L-62 Canal

Offsite flows currently drain into the STA site from six subcatchments north of SR 70. Runoff from these subcatchments crosses under SR 70 via two culverts, as shown on Figure 5-2. One small subcatchment is adjacent to and drains to culvert 1, a 24-in-diameter culvert located 0.35 miles east of SW 128th Avenue. The remaining five subcatchments ultimately drain to culvert 2, which is a 13-ft by 6-ft box culvert located 1.75 miles east of SW 128th Avenue, also as shown on Figure 5-2. Flows were routed through the five subcatchment drainage system via drainage ditches that are based on existing topography (SFWMD LiDAR). In addition, localized depressional areas were accounted for by developing stage/storage rating curves based on site topography. Two flow splits were incorporated into the model as shown on Figure 5-2. At these locations, a portion of runoff may be diverted out of the initial drainage system and into a separate drainage system that does not contribute to the Project site.

A conservative approach was used to quantify the stormwater flow rates entering the STA site from the north. The existing culverts' capacity was not limited, and flow is assumed to drain freely from the north side of SR 70 to the south side. The existing flow at culvert 2 is assumed to continue south in an existing drainage channel across the proposed Project site. The peak flow is limited based on the capacity of this existing onsite drainage channel, and not limited by culvert capacity. The same approach was used for culvert 1 but on a much smaller scale.

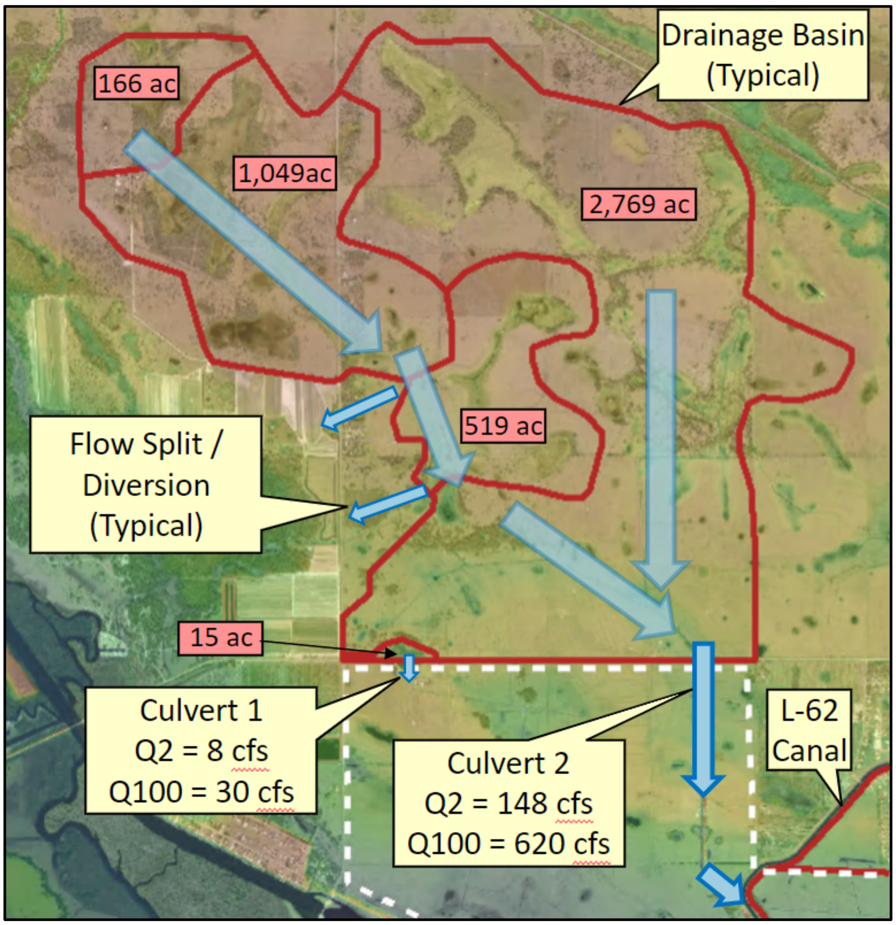

Figure 5-3. Offsite flow routing west of L-62 canal

5.3.2 Offsite Flows East of L-62 Canal

East of the L-62 canal, offsite flows currently drain from north and east of the STA site and through the area with proposed Cells 5 and 6, as shown on Figure 5-3.

Under proposed conditions, offsite flows currently entering the site will be routed around the STA site via a proposed seepage canal that drains west to east along the north site limit and then north to south along the east site limit, ultimately draining offsite to a drainage easement that runs southeast adjacent to the outside slope of the HHD, as shown on Figure 5-4.

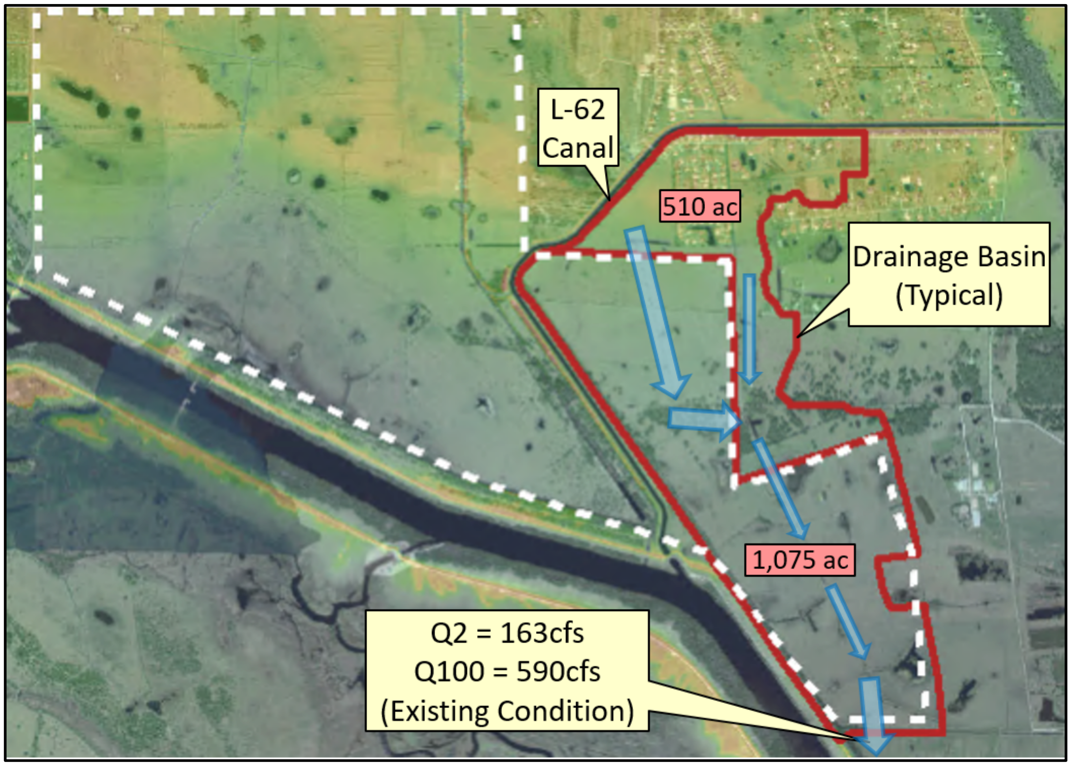

Figure 5-4. Offsite flow routing east of L-62 canal - existing conditions

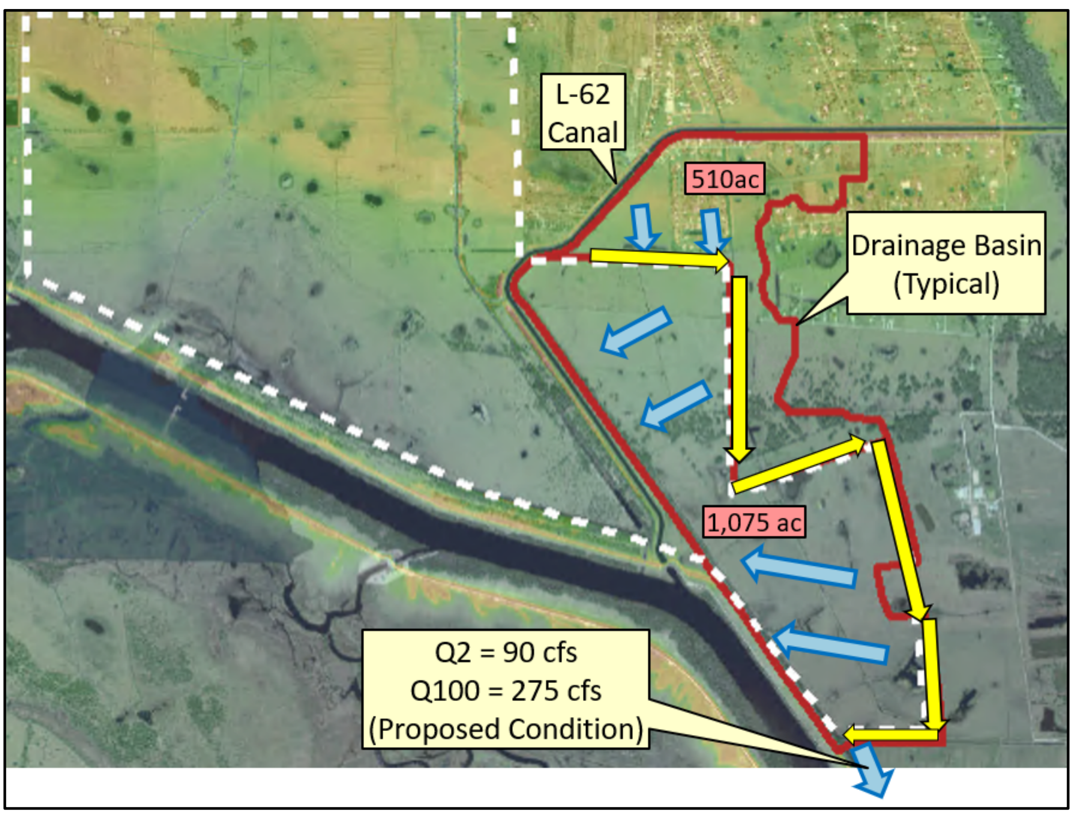

Figure 5-5. Offsite flow routing east of L-62 canal - proposed conditions

5.4 Proposed Future Activities

5.4.1 Preliminary Design

The Project is being designed to operate under a range of climatic conditions and is expected to be resilient to climate variability. During preliminary design, approaches to evaluate the impacts of potential changes in climate drivers (e.g., rainfall, evapotranspiration, etc.) on the Project will be developed in coordination with SFWMD's District Resiliency Officer.

As the design evolves, changes that require modifications to the hydrologic analyses will be completed, as necessary.

5.4.2 Final Design

During final design, the approaches developed during preliminary design will be implemented to evaluate the impacts of potential changes in climate drivers.

There are no other specific items to be completed during final design; however, as the design evolves, modifications will be completed on the hydrologic analyses, as needed.

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SECTION 6

WATER AVAILABILITY ANALYSIS

A water availability analysis was performed to help understand the potential complexity and level of effort needed to obtain permit approvals, considering the regulatory water-supply-related constraints that exist when Lake Okeechobee water levels are below specific elevations. The water availability analysis evaluated the regulatory aspects of the Lake Okeechobee Service Area (LOSA), Lake Okeechobee Regulation Schedule of 2008 (LORS2008), and Lake Okeechobee System Operating Manual (LOSOM). The analysis also incorporated several scenarios to evaluate the potential benefits of incorporating a 500-ac flow equalization basin (FEB), with a maximum water depth of 4 ft, upstream of the STA. While there were some slight reductions in the average number of days per year with STA water depths less than 0.5 ft and 0.0 ft (i.e., ground surface) and slight reductions in the number of events below these water depth thresholds, the EIP team concluded that reducing the STA size to incorporate an FEB was not justified. As such, none of the final alternatives included an FEB.

6.1 Design Criteria

A daily water budget spreadsheet model was developed to estimate the availability of source water to be used by the proposed STA. Based on available information, the period of analysis was selected to be 42 years (1980 through 2021). This period includes several drought years and several extreme wet years to ensure there is a representative timeframe to evaluate water availability for the STA.

6.1.1 Lake Okeechobee Stage Information

The following three Lake Okeechobee (Lake) stages were used for the analysis:

- **Historical** – daily Lake stage data (from station L OKEE) were obtained from DBHYDRO (DBKEY: 15611).
- **ECB19** – daily Lake stage data was obtained from the LOSOM's Regional Simulation Model (RSM) Existing Condition Base 2019 (ECB19) from January 1980 through April 2008. The **Historical** daily Lake stage was used from May 2008 through December 2021. This blended Lake stage was developed because the existing Lake schedule adopted in 2008 (i.e., LORS2008) manages the Lake lower than previous schedules. The **ECB19** blended schedule uses the simulated existing condition base run from LOSOM, which simulates LORS2008 that manages the Lake lower than the Historical Lake stage. Using only **Historical** daily Lake stage data would overestimate the amount of water available for the Project.
- **NA25f** – daily stage obtained from the LOSOM RSM 2025 Future without Condition (NA25f) for the period of 1980-2016. This Lake stage information was selected to evaluate the inclusion of the Kissimmee River Headwaters Revitalization Project, which when completed will increase the stages in the Upper Kissimmee Basin (Lakes Kissimmee, Hatchineha, Tiger, and Cypress) approximately 1.5 ft during the wet season. This is expected to store an additional 100,000 ac-ft of water that can be released during the dry season to improve water management and better approximate historical natural flows.

6.1.2 Rainfall and Evapotranspiration

Daily rainfall and pan evapotranspiration (ET) data was obtained from the LOSOM RSM ECB19 model from 1980–2016 and DBHYDRO (station S65DWX; DBKEY(s): VN308, S1491) for 2016–2021.

6.1.3 Canal Flow

Daily flow records for S-154, S-84, and S-65E were obtained from DBHYDRO for the **Historical** and **ECB19** simulations. The **NA25f** simulations used the flows generated from the **NA25f** model. S-154C records were not included since the entire S-154C basin is proposed to be an STA.

6.1.4 Assumptions for Evaluating Water Availability

The following are the assumptions used for the analysis:

- Source Basin Priority – The STA uses water from the S-154 basin flows until there was no discharge, then uses S-84 flows until there was no discharge, and then uses S-65E flows until there was no discharge, at which point water would be withdrawn directly from Lake Okeechobee.
- Lake Schedule Cutoff – No STA inflows from S-84, S-65E, and Lake Okeechobee when the Lake stage is below (see Figure 6-1):
  - The Low sub-band for designated scenarios
  - The Base Flow sub-band for designated scenarios
- A Depth of Deep Root Zone of 5.0 ft was used in the model as well as a porosity of 0.2. This represents the bottom of the deep root zone where ET loss becomes zero. This value is based on the South Florida Water Management Model (SFWMM Version 5.5, November 2005) for Land Use Type 10 (Wetland/stormwater treatment area and above-ground reservoir). In the model, daily ET occurs to a water depth of 1 ft below ground surface (bgs) and then decreases linearly to a rate of zero ET from 1 ft to 5 ft bgs. The 1 ft bgs is assumed to be the bottom of the shallow root zone. Models, such as MODFLOW, typically assume a linear reduction of ET from the maximum ET rate at the bottom of the shallow root zone to zero at the bottom of the deep root zone. Daily RSM-BN ET was used from 1980 to 2016, and from DBHYDRO station S65DWX from 2017 to 2021.
- A multiplier of 0.75 was applied to the daily pan ET rates from the RSM and DBHYDRO. This ET rate applies when the STA water level is above 1 ft bgs; from 1 ft bgs to 5 ft bgs, the ET rate declines linearly to a rate of 0.

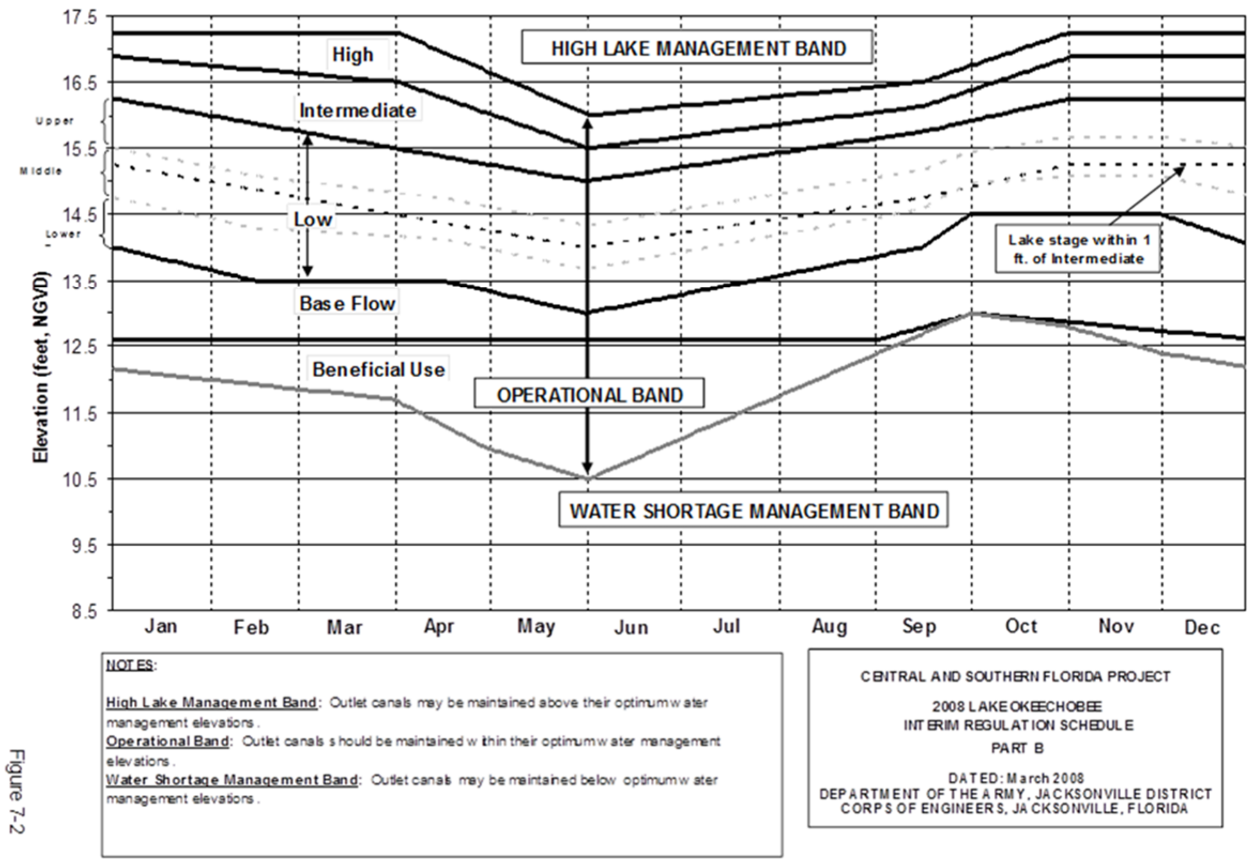

Figure 6-1. LORS2008 schedule

6.2 Analysis Methods

6.2.1 Alternatives Evaluated for Estimating Water Availability

The alternatives include an STA only and a FEB and STA combination to see the change resulting from using some of the STA area for water storage for when the Lake stage was below the Low or Base Flow sub-bands. The following parameters are the same for all STA-only scenarios:

- STA size = approximately 2,700 ac
- STA inflow capacity = 500 cfs
- STA target depth = 16.5 in. This is the STA maximum water depth for all inflows
- Net seepage loss = 2.8 cfs
- STA outflow start = (400 cfs @ 15-in STA water depth) – Below 15-in, STA outflow = 0 cfs
- STA outflow max = (600 cfs @ 18-in STA water depth) – Flow linearly increases from 400 cfs at 15 in deep to 600 cfs at 18 in deep.

For the FEB-STA configurations, FEB and STA outflow control parameters were adjusted such that the STA's average hydraulic loading rate is comparable to that of the corresponding STA-only scenarios. The following parameters are the same for all FEB and STA alternatives:

- FEB = 500 ac
- FEB inflow capacity = 500 cfs
- FEB max depth = 48 in
- FEB outflow normal = 460 cfs
- FEB outflow when Lake Okeechobee below Low or Base Flow sub-band = 25 cfs
- Net seepage loss FEB ~ 3.5 cfs pending further studies
- STA = 2,200 ac
- STA max depth = 16.5 in
- STA outflow start = (300 cfs @ 15 in water depth) – Below 15 in, STA outflow = 0 cfs
- STA outflow max = (500 cfs @ 18 in STA water depth) – Flow linearly increases from 300 cfs at 15 in deep to 500 cfs at 18 in deep
- Net seepage loss for STA = 2.8 cfs

The scenarios evaluated are provided in Table 6-1. Note that the Scenario Name begins with the Lake stage used (H for Historical, ECB19 and NA25f); followed by the STA or FEB and STA combination; followed by the Lake Schedule Cutoff sub-band.

Table 6-1. LKBSTA Alternatives Evaluated for Estimating Water Availability				
Scenario Name	Lake Stage	Lake Schedule Cutoff	STA Only	FEB and STA
A - H_STA_Low	Historical	Low	X	
B - H_FEBSTA_Low	Historical	Low		X
C - H_STA_Base	Historical	Base Flow	X	
D - H_FEBSTA_Base	Historical	Base Flow		X
E - ECB19_STA_Low	ECB19	Low	X	
F - ECB19_FEBSTA_Low	ECB19	Low		X
G - ECB19_STA_Base	ECB19	Base Flow	X	
H - ECB19_FEBSTA_Base	ECB19	Base Flow		X
I - NA25f_STA_Low	NA25f	Low	X	
J - NA25f_FEBSTA_Low	NA25f	Low		X
K - NA25f_STA_Base	NA25f	Base Flow	X	
L - NA25f_FEBSTA_Base	NA25f	Base Flow		X

6.3 Results

A summary of the water availability analysis results is provided in Tables 6-2 and 6-3. Note that Table 6-2 defines the headings in Table 6-3. Additional information is provided in Appendix 4 (Applicable Modeling Results).

The results of this water availability analysis, along with dynamic water quality model simulations, provided technical information to evaluate the potential impacts of incorporating an FEB with an STA on the Project site. Some slight reductions in the average number of days per year with low water depths and the number of low water depth events were predicted, while the projected TP load reduction of the FEB+STA scenario was similar to STA-only scenarios with similar areas. Therefore, the EIP team concluded that reducing the STA size to incorporate an FEB into the Project was not justified.

Table 6-2. LKBSTA Table 6-3 Header Definitions

	Average Annual Inflow Volume from Water Sources						STA Performance Metrics												
Header	**Total cfs** kacft/ year	**S-154 cfs** % of total	**S84 cfs** % of total	**S65E cfs** % of total	**Lake O. cfs** % of total	**STA Outflow cfs** kacft/ year	**HLR in/day** cm/day	**Mean Depth in** cm	**Max Depth in** cm	**days/year** % of time **Depth < 0.5 feet**	**days/year** % of time **Depth < 0 feet**	**< 0 feet Events 1-30** >=1 **days**	**< 0 feet Events 30-60** >30 **days**	**< 0 feet Events 60-90** >60 **days**	**< 0 feet Events 90-120** >90 **days**	**< 0 feet Events 120-180** >120 **days**	**< 0 feet Events 180-240** >180 **days**	**< 0 feet Events 240-300** >240 **days**	**< 0 feet Events >300 days**
Description	Total STA inflow for the period of record	STA inflow from S154 for the period of record	STA inflow from S84 for the period of record	STA inflow from S65E for the period of record	STA inflow from Lake O. for the period of record	Total STA outflow for the period of record	Mean STA hydraulic loading rate for the period of record	Mean STA depth for the period of record	Max STA depth for the period of record	Mean duration when STA depth<0.5 ft for the period of record	Mean duration when STA depth<0 ft for the peiod of record	# of dry events (depth<0) of 1-30 days duration for the period of record	# of dry events (depth<0) of 30-60 days duration for the period of record	# of dry events (depth<0) of 60-90 days duration for the period of record	# of dry events (depth<0) of 90-120 days duration for the period of record	# of dry events (depth<0) of 120-180 days duration for the period of record	# of dry events (depth<0) of 180-240 days duration for the period of record	# of dry events (depth<0) of 240-300 days duration for the period of record	# of dry events (depth<0) of >300 days duration for the period of record
Black Color Entry	cfs	cfs	cfs	cfs	cfs	cfs	in/day	in	in	days per year	days per year	1-30 day dry events	30-60 day dry events	60-90 day dry events	90-120 day dry events	120-180 day dry events	180-240 day dry events	240-300 day dry events	>300 day dry events
Blue Color Entry	kacft per year	kacft per year	kacft per year	kacft per year	kacft per year	kacft per year	cm/day	cm	cm	% of time	% of time	Sum of >= 1 day dry events	Sum of >30 day dry events	Sum of >60 day dry events	Sum of >90 day dry events	Sum of >120 day dry events	Sum of >180 day dry events	Sum of >240 day dry events	

Table 6-3. LKBSTA Water Availability Results

	Average Annual Inflow Volume from Water Sources						STA Performance Metrics												
Scenario	Total cfs kacft/ year	S-154 cfs % of total	S84 cfs % of total	S65E cfs % of total	Lake O. cfs % of total	STA Outflow cfs kacft/ year	HLR in/day cm/day	Mean Depth in cm	Max Depth in cm	days/year % of time Depth < 0.5 feet	days/year % of time Depth < 0 feet	< 0 feet Events 1-30 >=1 days	< 0 feet Events 30-60 >30 days	< 0 feet Events 60-90 >60 days	< 0 feet Events 90-120 >90 days	< 0 feet Events 120-180 >120 days	< 0 feet Events 180-240 >180 days	< 0 feet Events 240-300 >240 days	< 0 feet Events >300 days
A	303.9	27.6	93.6	130.1	52.6	302.2	2.7	12.8	25.4	41.0	22.2	8	1	0	1	2	2	0	0
	220.0	9%	31%	43%	17%	218.8	6.8	32.5	64.6	11%	6%	14	6	5	5	4	2	0	--
B	252.8	27.5	79.4	106.5	39.5	247.4	2.7	13.8	25.4	30.0	17.0	7	0	1	0	1	1	1	0
	183.0	11%	31%	42%	16%	179.1	6.9	35.1	64.6	8%	5%	11	4	4	3	3	2	1	--
C	374.5	29.3	107.4	161.6	76.1	372.7	3.3	14.2	23.9	25.3	12.7	7	0	2	0	1	1	0	0
	271.1	8%	29%	43%	20%	269.8	8.4	36.0	60.7	7%	3%	11	4	4	2	2	1	0	--
D	310.9	28.9	91.3	133.8	56.9	305.1	3.3	15.1	21.7	19.0	7.8	0	0	0	0	1	1	0	0
	225.1	9%	29%	43%	18%	220.9	8.4	38.3	55.1	5%	2%	2	2	2	2	2	1	0	--
E	236.3	24.1	82.5	97.9	31.8	234.6	2.1	12.0	25.4	50.6	24.3	8	2	0	1	2	2	0	0
	171.1	10%	35%	41%	13%	169.8	5.3	30.4	64.6	14%	7%	15	7	5	5	4	2	0	--
F	197.3	24.6	69.7	79.5	23.4	192.2	2.1	13.3	25.4	36.5	19.4	7	0	2	0	1	1	1	0
	142.8	12%	35%	40%	12%	139.1	5.3	33.7	64.6	10%	5%	12	5	5	3	3	2	1	--
G	335.5	28.4	103.3	141.9	61.9	333.7	3.0	13.4	23.9	30.4	17.8	3	1	1	1	2	1	0	0
	242.9	8%	31%	42%	18%	241.6	7.5	34.1	60.7	8%	5%	9	6	5	4	3	1	0	--
H	278.9	28.1	87.8	116.9	46.0	273.3	3.0	14.6	21.5	24.5	11.4	3	2	0	0	1	1	0	0
	201.9	10%	31%	42%	17%	197.8	7.6	37.0	54.5	7%	3%	7	4	2	2	2	1	0	--
I	224.0	23.0	63.6	103.8	33.6	221.9	2.0	11.7	24.4	51.7	26.9	8	2	0	1	2	2	0	0
	162.1	10%	28%	46%	15%	160.7	5.0	29.7	62.0	14%	7%	15	7	5	5	4	2	0	--
J	187.2	23.7	54.2	84.3	25.0	181.9	2.0	13.0	21.6	40.1	22.0	7	0	2	0	1	1	1	0
	135.5	13%	29%	45%	13%	131.7	5.1	33.0	55.0	11%	6%	12	5	5	3	3	2	1	--
K	327.9	28.1	82.2	150.1	67.6	325.9	2.7	13.2	24.4	34.0	19.5	3	2	0	1	2	1	0	0
	237.4	9%	25%	46%	21%	235.9	6.8	33.5	62.0	9%	5%	9	6	4	4	3	1	0	--
L	272.6	27.8	70.5	123.3	50.9	266.8	2.9	14.3	21.9	25.7	12.9	3	2	0	0	1	1	0	0
	197.4	10%	26%	45%	19%	193.1	7.4	36.4	55.5	7%	4%	7	4	2	2	2	1	0	--

6.4 Proposed Future Activities

Based on recent discussions with and guidance from the District, the following are proposed activities EIP will perform as part of preliminary design:

- Incorporate the ability to implement low-level inflows from the C-38 canal to maintain STA vegetation during dry periods.
- Refine STA target and maximum water depths consistent with STA embankment design.
- Continue to develop and pre-screen operational strategies and prepare inflow datasets for DMSTA2 modeling.
- Implement and document a model sensitivity analysis.

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SECTION 7

WATER QUALITY MODEL

This section describes the modeling approach used to estimate long-term phosphorus removal performance for the Project. The modeling was prepared using a dynamic spreadsheet-based platform that accounts for phosphorus and water mass balances with phosphorus removal primarily controlled by a first-order rate coefficient that has been calibrated to operational data from other large-scale Florida STA projects. Model output includes daily estimates of STA inflow and outflow volumes, inflow and outflow phosphorus concentrations, inflow and outflow phosphorus mass loads, and water depth.

7.1 Design Criteria

Required input data include STA cell dimensions (area, length, width, and depth) and a time series file containing daily inflow phosphorus concentrations, inflow rates, rainfall, and ET values.

7.1.1 Cell Dimensions and Configuration

As noted in Section 1, the design alternative includes six cells that will be operated in parallel. Table 7-1 summarizes the surface area and mean width for each cell based on the current layout. These values may be modified as the design progresses. DMSTA2 inputs for these dimensional parameters are entered with units of square kilometers (km^2) for area and kilometers (km) for width.

Table 7-1. LKBSTA Cell Dimensions

Cell Number	Area (km^2)	Area (ac)	Width (km)	Width (ft)
1	2.20	543	1.54	5,040
2	1.67	412	1.52	5,000
3	1.18	293	1.64	5,370
4	1.51	374	1.34	4,410
5	1.98	490	1.48	4,860
6	1.58	390	1.28	4,190

7.1.2 Inflow Volumes and Concentrations

Section 6 describes the basis for developing inflow time series for STA flows, rainfall, and ET as a function of various operational rules and constraints. The operating rules and water availability analyses have not been finalized and will be included in DMSTA2 modeling updates during the design phase.

The EIP team (MacVicar, 2022) summarized TP concentration data by month for the various water sources (S-154, S-84, S-65E, and Lake Okeechobee) for the 10-year period from 2012 through 2021 (Table 7-2, Figure 7-1). During preliminary DMSTA2 modeling efforts, these mean monthly TP concentrations were paired with each day's flow from each source to estimate the flow-weighted mean concentration delivered to the STA. During the design phase, the EIP team will confirm the suitability of using the 2012–2021 mean monthly TP concentrations and will re-evaluate the approach of pairing these concentrations with their respective C-38 canal inflow sources (S-84, S-65E, and Lake Okeechobee).

Table 7-2. LKBSTA Mean Monthly Total Phosphorus Concentrations by Source (2012-2021)

Month	TP Concentration (micrograms per liter, µg/L)			
	S-154	S-84	S-65E	Lake Okeechobee
January	365	96	78	89
February	428	82	78	81
March	346	84	80	95
April	335	73	76	95
May	804	102	96	92
June	686	91	152	126
July	545	83	148	130
August	562	88	125	123
September	568	113	146	116
October	570	86	119	116
November	439	89	102	95
December	393	75	74	79

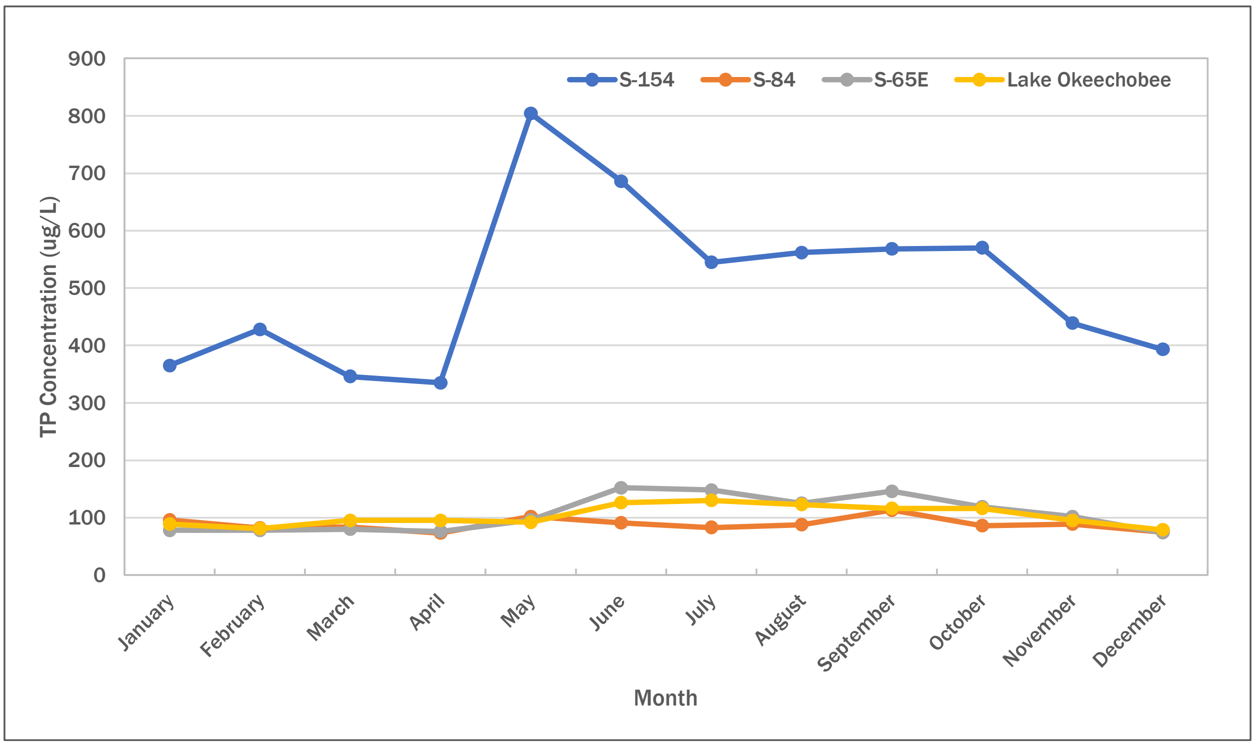

Figure 7-1. LKBSTA Mean Monthly Total Phosphorus Concentrations by Source (2012-2021)

7.1.3 Model Parameterization

DMSTA2 requires user inputs for a suite of model parameters before the code can be executed and results generated. The parameter values summarized in Table 7-3 are expected to be used for all simulations as the design phase progresses. It is likely that the first several years of each inflow time series file will be used to “ramp-up” the model and stabilize phosphorus storage changes. Input parameters C0, C1, C2, Z1, Z2, and Z3 were consistent with the default parameters for the emergent vegetation calibration (EMG_3, Walker and Kadlec, 2008). Additional calibration efforts are not anticipated. Initial values for the following parameters were generally consistent with values used for other South Florida STA DMSTA2 simulations:

- Rainfall P concentration,
- Atmospheric P load (dry),
- Initial water column concentration,
- Initial P storage per unit area, and
- Initial water column depth.

Values for the net settling rate varied as described below in Section 7.4.1.2.1.

Table 7-3. DMSTA2 Parameterization for the LKBSTA Project

Parameter	Units	Value
Rainfall P Conc	µg/L	10
Atmospheric P Load (Dry)	mg/m^2-yr	20
Initial Water Column Conc	µg/L	50
Initial P Storage Per Unit Area	mg/m^2	500
Initial Water Column Depth	cm	40
C0 = Conc at 0 g/m2 P Storage	µg/L	3
C1 = Conc at 1 g/m2 P storage	µg/L	22
C2 = Conc at Half-Max Uptake	µg/L	300
K = Net Settling Rate at Steady State	m/yr	Varies
Z1 = Saturated Uptake Depth	cm	40
Z2 = Lower Penalty Depth	cm	100
Z3 = Upper Penalty Depth	cm	200

cm = centimeters; m^2 = square meter; m/yr = meters per year; mg/m^2-yr = milligrams per square meter per year; µg/L = micrograms per liter

STA cells are assumed to receive inflows in proportion to their individual surface areas. STA cell outflow hydraulic parameters will be calibrated to match the output from the hydraulic modeling described in Section 8. Seepage will be included based on the results summarized in Section 11.

7.2 Analysis Methods

DMSTA2 was developed to estimate the TP removal performance of shallow reservoirs and treatment wetlands (Walker and Kadlec, 2008). The specific model file used for this project is labeled as dmsta2c2b_2010v2b.xlsx.

7.3 Results

As noted above, the EIP team is working with the District to finalize the operational rules and constraints that will define the volumes available from each of the proposed water sources. Accordingly, modeling results cannot be presented for the proposed STA layout in this DDR. DMSTA2 modeling updates will be prepared when the inflow data sets are finalized. Anticipated simulations will include the following considerations:

- Updated STA cell dimensions and configuration
- Updated STA cell target and maximum water depths based on embankment design refinements that allow flexibility to store additional water

- Updated seepage estimates
- Effect of operating cells 1 and 2 as upstream storage compartments for Cells 3 through 6 on projected TP load reduction
- Effect of recirculation of STA discharges on projected TP load reduction

Reported tabular and graphical results will quantify and compare the following performance metrics:

- Inflow and outflow volumes
- Inflow and outflow TP concentrations
- Inflow, outflow, and removed TP loads
- TP load reduction efficiency
- Mean and maximum water depths
- Simulation period-of-record water depth frequency distributions
- Mean and maximum hydraulic loading rates

7.4 Proposed Future Activities

This section identifies the water quality modeling tasks that will be completed during preliminary design.

7.4.1 Preliminary Design

7.4.1.1 Updated Water Quality Modeling

As noted in Section 7.3 above, DMSTA2 modeling will be updated for the refined STA layout that will be included in preliminary design documents.

7.4.1.2 Sensitivity Analyses

Sensitivity analyses will be performed to bracket the expected range of performance for the proposed STA. The specific sensitivity analyses that will be conducted are described below.

During the Reconnaissance Study the EIP team evaluated the sensitivity of varying the number of tanks-in-series (N) assigned to each STA cell. Each STA cell was initially assigned a value of N=3.0, which implies that each cell has the hydraulic mixing behavior of three completely stirred tanks that operate in series. Simulations were also conducted for N=6, with the resulting average annual TP load removal improving by about 2 percent. Using conceptual DMSTA2 modeling, the EIP team (Wetland Solutions, Inc., 2009) showed that increasing the value of N is important when target outflow concentrations approach the irreducible background (C*) but has minimal effect when the objective is to maximize load reduction.

7.4.1.2.1 Net Settling Rate

Simulations will be run with net phosphorus settling rates of 12.4 meters per year (m/yr), 16.0, m/yr, and 20.9 m/yr, which represent the 10th percentile, median (50th percentile), and 90th percentile values for emergent STA vegetation (http://wwwalker.net/dmsta/testing/calib_summary.htm) after adjustment for a 95 percent duty-cycle factor.

7.4.1.2.2 Target Average and Maximum Water Depths

Simulations will be run with the following water depths:

- Target Depth: 1 ft (~30 cm); 1.5 ft (~46 cm)
- Maximum Depth: 2 ft (61 cm); 4 ft (122 cm)

7.4.1.2.3 Inflow Concentrations

Simulations will be run to explore the effects of variability in inflow TP concentrations. These simulations will use the DMSTA2 concentration scale multiplier parameter to increase or decrease each daily TP concentration by the following factors:

- 0.75
- 1.0
- 1.25

7.4.2 Final Design

Following completion of preliminary design, updated water quality modeling will be conducted to reflect any changes to the Project layout or water availability assumptions that may occur as final design nears completion. Modeling scenarios will match the operational strategies described in the Project Operations Manual (POM). Reported results will include:

- Inflow and outflow volumes
- Inflow and outflow TP concentrations
- Inflow, outflow, and removed TP loads
- TP load reduction efficiency
- Mean and maximum water depths
- Simulation period-of-record water depth frequency distributions
- Mean and maximum hydraulic loading rates

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SECTION 8

HYDRAULICS

This section presents the design criteria, analysis methodology, analysis results, and future work related to Project hydraulics. This information is divided into the five main design elements of the LKBSTA:

- **Headworks**, which includes the hydraulic analyses of the proposed L-62 canal realignment and associated L-62 canal WCSs
- **STA system**, which includes the hydraulic analyses of the internal system from pump discharge through inflow canals, internal WCSs, treatment cells, PES, and outflow canals, to discharge into the C-38 canal
- **HPC assessment**, which determines the HPC and the corresponding Project design requirements, including embankment height
- **Wind setup/Wave run-up analysis**, which assesses the impacts wind and waves have on cell embankments and resultant embankment design height
- **Flood routing analysis**, which includes sizing water conveyance systems and structures to safely route flood flows through the STA

This section also lists the future work that is anticipated to further advance the hydraulic analyses.

8.1 Design Criteria

The design criteria for the Project were developed through collaboration with SFWMD staff, review of previously completed SFWMD STA projects, and relevant standards and reference manuals. The following list of standards and reference manuals are applicable to hydraulic analyses:

- Environmental Resource Permit Information Manual, SFWMD
- SFWMD Design Criteria Memoranda
  - DCM-1 – Hazard Potential Classification
  - DCM-2 – Wind and Precipitation Design Criteria for Freeboard
  - DCM-3 – Spillway Capacity and Reservoir Drawdown Criteria
- SFWMD Engineering Design Standards for Water Resource Facilities – Design Guidelines
- USACE EMs, including:
  - EM 1110-2-1601 – Hydraulic Design of Flood Control Channels
  - EM 1110-2-1603 – Hydraulic Design of Spillways

8.1.1 Headworks Model

The design criteria for the STA system are summarized in the following sub-sections organized by project element.

8.1.1.1 Boundary Conditions

The headworks model was refined to incorporate changes associated with the capacity of WCS-9 and the re-route of L-62 canal under dry and wet weather conditions.

- WCS-9

- Under wet weather conditions, the structure should pass the design flood of 1,000 cfs (30 percent of the Standard Project Flood) as defined in the Structures Book (SFWMD, 1997). Under this condition, the headwater elevation is 20.1 ft (NAVD) and the tailwater elevation is 17.9 ft (NAVD).
- A recorded high-flow condition of 1,150 cfs was identified from DBHYDRO, which is near the design flood and with a water surface elevation (WSE) of 12.1 ft (NAVD) at C-38 canal, which is slightly higher than the 60th percentile of the future LOSOM water level.
- Under dry weather conditions it is envisioned that WCS-9 will remain closed.

- C-38 canal tailwater/re-route of L-62 canal
  - Wet weather – The realigned L-62 canal will discharge into the C-38 canal through WCS-13. The flood condition tailwater elevation, as defined in the Structures Book (SFWMD, 1997), is 17.9 ft (NAVD). Water will be conveyed downstream from the S-154 basin through WCS-9 and the rerouted L-62 canal.
  - Dry weather – Water will be conveyed upstream from the C-38 canal through the L-62 canal to the PS intake. Under this condition, the low operational tailwater elevation, when STA system flow is 500 cfs, is 10.7 ft (NAVD), which is based on a water level that is near the 80th percentile of the Future LOSOM water level. Under vegetation survival flow rate/drought conditions, the low operational tailwater elevation is 9.5 ft (NAVD), which corresponds to the 90th percentile of the Future LOSOM water level.

8.1.1.2 WCS-9 and WCS-13

- WCS-9 and WCS-13 should be capable of producing similar (or improved) WSEs upstream of the structure under select high-flow conditions (i.e., no increased WSE). The WSE should not exceed the WSE at G-80 of 22 ft under design flow and high flow of 1,000 cfs and 1,150 cfs, respectively.
- Submerged flow conditions at the gates are desired for better flow measurement computation.
- Velocity within the control structure should be less than 8 ft per second (fps) for the 1,000 cfs design flow.
- Energy dissipation is required at the discharge point of WCS-9 to lower the velocity to 2 fps. Modeling will be advanced during preliminary design to identify appropriate energy-dissipation features.
- It is anticipated that an additional WCS will be included on the L-62 canal just upstream of the confluence with the C-38 canal and built into the HHD. Other sections of this DDR refer to this structure as WCS-13. This structure will be analyzed during preliminary design.

8.1.1.3 L-62 Canal

- Identify the capacity of the rerouted L-62 canal under dry and wet weather conditions to determine if it is capable of conveying the design flow to C-38 canal.
- The velocity in the unarmored portion of the canal should be less than 2.0 fps. The velocity is selected based on feedback from District staff, which is consistent with published values for earthen channels with similar soil textural class (initial silty sand soil per preliminary geotechnical investigation).
- The desired minimum water depth is 8 ft to prevent excessive vegetation growth and maintain a clear area to convey inflow to the PS.

8.1.1.3.1 Wet Weather Condition

- Identified flow conditions for a design flow of 1,000 cfs and a high flow of 1,150 cfs

- Table summary of depths and velocities

8.1.1.3.2 Dry Weather Condition

- Provide 500 cfs conveyance capacity of L-62 canal from C-38 canal to PS intake when C-38 canal water elevation is as low as 10.7 ft (NAVD).
- Provide 125 cfs conveyance capacity of L-62 canal from C-38 canal to PS intake when C-38 canal water elevation is as low as 9.5 ft (NAVD). Also, check results of pumping at the normal operational flow rate (500 cfs) when C-38 canal water elevation is as low as 9.5 ft (NAVD). The 125-cfs rate is based on one of four 125-horsepower pumps running for 6 hours per day to yield an average daily flow rate of approximately 30 cfs, which is the Project vegetation survival flow rate.

8.1.2 STA System

The design criteria for the STA system are summarized in the following sub-sections organized by project element.

8.1.2.1 Boundary Conditions

There are two main boundary conditions considered in the STA System hydraulic modeling:

- Inflow rate – The inflow boundary conditions set by the pumping rate from the PS. Two pumping operation conditions are applied to the hydraulic model:
  - The normal operation pumping rate (i.e., flow into the STA system under normal conditions) is 500 cfs. This is based on separate water quality and water availability analyses (Sections 6 and 7).
  - The second operation pumping rate is the vegetation survival flow rate. This is a scenario where limited water is available in a dry weather condition, but water is provided at a rate to assist in vegetation survival. In this scenario, there is no surface water discharged from the STA system (outflow gates closed) and there is minimal pumping into the STA system to maintain wet bottom conditions. This rate is equal to the sum of ET and seepage loss from the system, currently estimated as 30 cfs. With the anticipated pump configuration (four 125-cfs pumps), it would be anticipated that one pump would have a daily 6-hour duty cycle to result in an average daily rate of approximately 30 cfs in the cells. This rate and modeling of the duty cycle will be refined during preliminary design.
- Tailwater/S154 Outlet Structure – The S154 structure will become the STA system outlet structure. The S154 structure connects the outflow canal to the C-38 canal via a 24-ft-wide weir (crest elevation 9.8 ft (NAVD)) followed by two submerged 8 ft x 10 ft culverts with gates on the upstream side (invert elevation 3.8 ft +/-(NAVD)). The gates will have the ability to be closed in vegetation survival operations, drought conditions, or during maintenance activities; however, the gates will remain completely open during normal operations, and therefore the tailwater elevations in the C-38 canal will be a boundary condition. There are three C-38 canal tailwater conditions that are considered in the hydraulic model:
  - The average tailwater elevation in the C-38 canal is 13.4 ft (NAVD), which is the average water level of Lake Okeechobee (https://w3.saj.usace.army.mil/h2o/reports/r-oke.html). The PCSWMM 2D STA system model used a tailwater elevation of 14.0 ft (NAVD), slightly above the lake's average water level. This is the primary tailwater condition used in the STA system modeling.
  - The flood condition (3,333 cfs) tailwater elevation is 17.9 ft (NAVD) (see Section 8.1.1). This tailwater boundary condition was modeled in a limited number of simulations for key

treatment cells (3 and 6). Development of this condition will be advanced during preliminary design.

The low operational tailwater elevation is 10.7 ft (NAVD) (see Section 8.1.1). Because this tailwater condition would not limit the outflow conveyance capacity of normal operations, it was not simulated for this DDR. This condition will be simulated during preliminary design to determine anticipated depths in the outflow canal and potential gate operations of the S154 structure.

8.1.2.2 PS

There are two pumping operation conditions, as discussed in Section 8.1.2. The normal operation pumping rate is 500 cfs. The dry weather pumping rate is 125 cfs for 6 hours per day to yield the average daily vegetation survival flow rate of 30 cfs.

As it relates to the STA system, the water discharge head was determined through STA modeling by determining the head needed at the PS discharge to "push" water through the STA system. Based on modeling, a head elevation of approximately 31 ft (NAVD) to 32 ft (NAVD) is needed to distribute the anticipated flow rates. Coordination with the mechanical design was completed to select pumps that meet the PS performance requirement.

8.1.2.3 Inflow Canals

- Flow Capacity – The inflow canals should have the capacity to convey the proportional flow, through their respective inflow WCSs to each treatment cell, at the desired cell WSE.
- Manning's roughness coefficient (n) for internal canals is 0.03.
- Velocities in canals should be less than 2.0 fps. If localized velocities are greater, armoring should be provided.
- The desired minimum water depth in the canals is 6 ft to prevent excessive vegetation growth (applies to all canals).
- Canal side-slopes are 3H:1V or flatter.

8.1.2.4 Inflow/Outflow WCSs

- Pipes will be smooth wall HDPE. Inflow and outflow WCSs should be fully submerged under the anticipated STA operating ranges. To achieve this, the following design criteria apply:
  - Inflow and outflow pipes from WCSs will be set 2 ft to 3 ft above the bottom of adjacent canals.
  - Gates will be set with gate bottom elevations equal to the invert elevation of the adjacent outflow pipe.
- Consistent and/or similar inflow and outflow control structure sizes should be used across the STA.
  - Inflow and outflow pipes are currently sized with a diameter of between 3 ft and 3.5 ft, with square gates having 3 ft or 3.5 ft side dimensions.
- The maximum flow velocity at control structures is 8 fps.
- Channel areas adjacent to the pipe inlet and outlet will be armored as necessary to protect banks until velocities are 2.0 fps or less.
- Manning's roughness coefficient (n) for pipes is 0.01 (based on use of HDPE pipe material).
- The following local head loss coefficients are assumed at internal WCSs:
  - C_{ENT} = 0.5
  - C_{EXT} @ gate = 0.2

- C_{EXT} @ spreader canal = 0.5
- C_{EXT} @ outflow canal = 0.5

- The discharge coefficient used for the orifice equation to model internal WCS gates is 0.65.
- Treatment cells
  - Flow rates to the treatment cells are distributed on an area-weighted basis.
- Spreader canals
  - Canal depth is 6 ft below the treatment cell bottom elevation
  - Canal bottom width is 5 ft

8.1.2.5 STA Cells

- The Manning's roughness coefficient ranges from n = 0.32 to 0.45 under normal operations.
- Under normal operations, the maximum water depth in the cells is 2 ft with expected depths between 1.25 and 1.75 feet.
- Velocities in the STA cells should be less than or equal to 0.1 fps.
- Spillways/outflow WCSs – Analysis methodology and standards for spillways and outflow WCSs will generally follow the guidance from CERP DCM 3 (CERP, 2006), which quantifies and routes the 100-year/3-day storm through treatment cells.

8.1.2.6 Outflow Canal

- Flow Capacity
  - Normal operations
    - Convey flow from normal operations to the outlet at S154.
    - The outflow canal from the Cell 2 outflow WCS to the Cell 3 outflow WCS will also function as a seepage canal along the western site limit. The canal will be designed to generally maintain pre-project ground water levels along the western property line.
  - Vegetation survival operations – Under this condition it is anticipated that the outflow WCSs will be closed and there would be no flow or negligible flow routed through the outflow canals.
  - Flood conditions flood routing – Analysis methodology and standards for flood routing will generally follow DCM-3 guidance using the 100year/3-day storm. The system will be able to safely route the 100-year/3-day storm through an outflow canal and ultimately to the outlet at S154.
  - Overall criteria
    - Manning's roughness coefficient (n) for internal canals is 0.03.
    - Velocities in canals should be less than 2.0 fps.
    - The minimum water depth in the canals will be 6 ft.
    - S154 Outlet - The S154 structure will become the outlet of the STA system. The gates at S154 will remain completely open during normal operations. The gates will have the ability to close during vegetation survival operations or drought conditions.

8.1.2.7 Offsite Flows

Offsite flows around the Project site will be conveyed by perimeter seepage canals. The seepage canals will be sized to collect and convey the 100-year/3-day peak flow (See Section 5) from the offsite drainage areas, in addition to seepage flows, while maintaining the minimum freeboard requirements of adjacent levees.

8.1.2.8 Seepage Canals

The seepage canal running west to east, on the north side of the site, would be sized to collect and convey the 100-year/3-day peak flow from the offsite area to the north (see Section 5 Hydrology). The canal flows east to the northeast corner of the site, and then south in the seepage canal running along the eastern site limit.

Based on preliminary geotechnical investigation (Section 9), the seepage rate and resultant conveyance rate in the seepage canals is insignificant relative to the stormwater conveyance capacity. Seepage rates and their ultimate impact on seepage canal sizing will be refined during fseepage management requirements.

In addition, the total conveyance capacity of WCS-9 and WCS-13 will include the flow from the worst-case seepage contributions if the pump station is offline and unable to pump seepage flows back into the STA system. As stated above, the seepage rate is relatively small compared to the stormwater conveyance capacity but will be accounted for in refined WCS sizing during preliminary design.

8.1.3 HPC

The HPC is based on the DCM-1 (CERP, 2005). Based on feedback from Project partners and experience with similar projects, the LKBSTA is initially assumed to be classified as a low hazard facility.

The recommendation is based on evaluation of the four categories of potential impacts identified in DCM-1:

- Direct Loss of Life
- Lifeline Losses
- Property Losses
- Environmental Losses

Dam-break analyses are planned on the Project's northern and western boundaries to confirm the Project meets the criteria of a low hazard HPC during design.

DCM-1 guidelines help in determining low, significant, and high hazard potential associated with each of the four categories of impacts identified above. Both USACE and FEMA have guidelines for these categories, which are summarized in Table 8-1. The primary difference between the two sets of guidelines is the way the two agencies define high-hazard potential with respect to loss of human life.

Table 8-1. USACE and FEMA Guidelines for Evaluating DCM-1 Hazard Potential Classification per DCM-1

Hazard Potential Classification	Agency Interpretation	Loss of Human Life	Lifeline Losses	Economic Losses	Environmental Losses
Low	USACE	None expected (due to rural location with no permanent structures for human habitation)	No disruption of services (rapid or cosmetic repairs)	Private agricultural lands, equipment and isolated buildings impacted	Minimal incremental damage
	FEMA	None expected	Low and generally limited to owner	Low and generally limited to owner	Low and generally limited to owner

Significant	USACE	Uncertain (rural with few residences or industrial facilities)	Disruption of essential services and access	Major public and private facilities impacted	Major mitigation required
	FEMA	None expected	Yes	Yes	Yes
High	USACE	Certain	Disruption of critical services and access	Extensive public and private facilities	Extensive mitigation cost or impossible to mitigate
	FEMA	Probable	Yes	Yes	Yes

8.1.4 Wind setup/Wave run-up and Freeboard

Wind setup, wave run-up, and freeboard analyses generally follow the guidance from DCM-2 (CERP, 2006) and the Coastal Engineering Manual (USACE, 2003a).

8.1.5 Flood Routing/Spillways

Analysis methodology and standards for flood routing and spillways will generally follow the guidance from DCM-3. It is anticipated that the 100-year/3-day storm will need to be safely conveyed through and/or around the STA system. Overflow spillways through the STA cell levees will likely be required. The flood event modeling and the design of spillways will be initiated in preliminary design.

8.2 Analysis Methods

PCSWMM is the primary modeling tool used to perform steady and unsteady hydraulic modeling for both the headworks and STA systems. PCSWMM can integrate 2D free-surface flow with the fully dynamic 1D approach. For additional information on PCSWMM, refer to this Project's "PCSWMM Write-up" (Appendix 4), previously submitted to SFWMD in February 2022.

Computational Hydraulics International also has documentation on SWMM (CHI, 2022a) and 2D-specific background (CHI, 2022b).

8.2.1 Headworks System

This section presents the approach for assessing the hydraulic conditions of the proposed WCS-9 and the rerouted L-62 canal.

Modeling of the proposed L-62 canal and associated WCSs is used to size proposed elements and confirm the proposed system will achieve the required hydraulic performance criteria in a variety of conditions. Figure 8-1 shows a plan view of the headworks system.

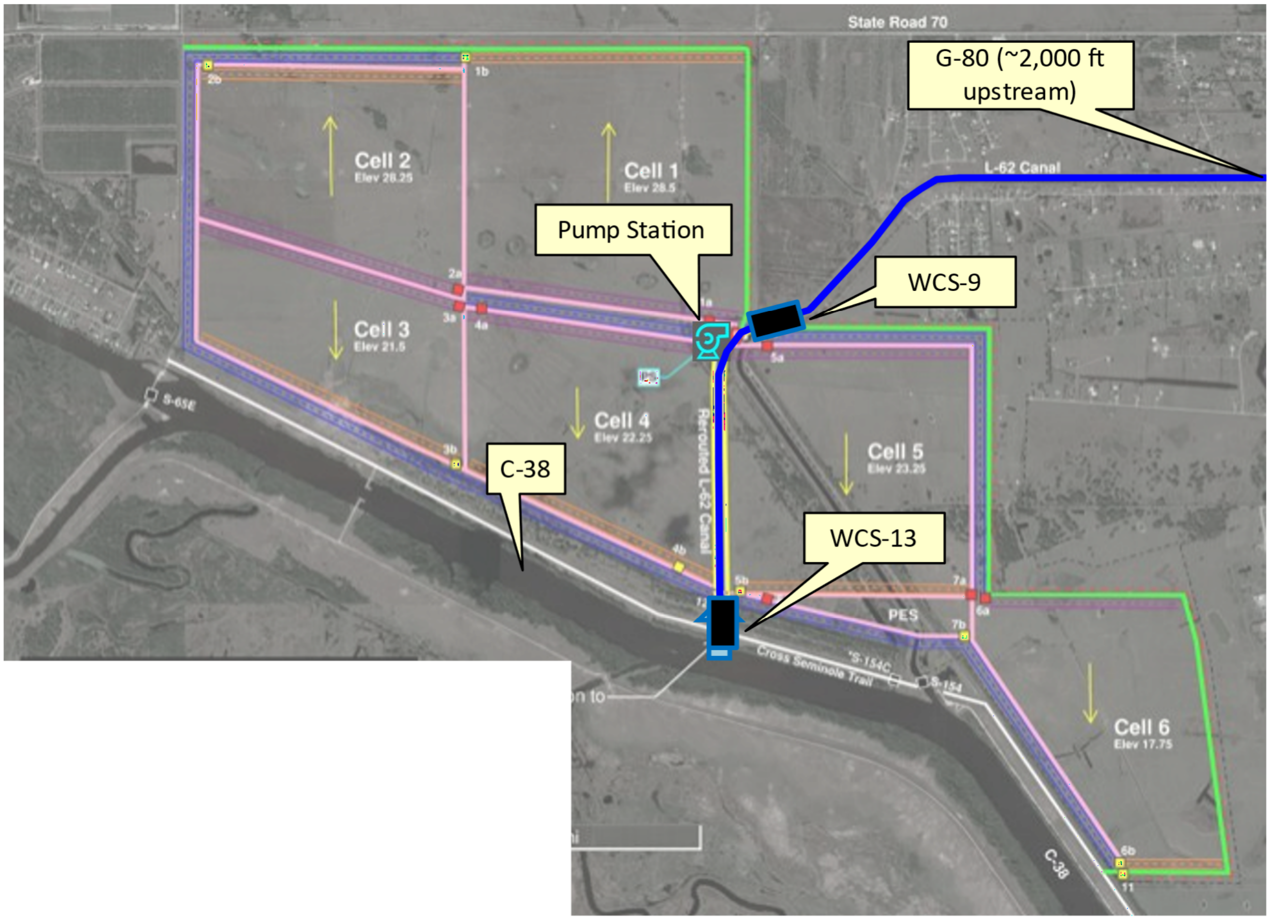

Figure 8-1. Plan view of headworks system

8.2.1.1 WCS-9 and WCS-13

The hydraulic modeling includes steady-state simulations for:

- Sizing of the in-canal water control structure, WCS-9, and WCS-13
- Flow velocity evaluation downstream of the structures
- Water depth evaluation upstream and downstream of the structures

Future work will include computational fluid dynamics (CFD) modeling of the two structures. This modeling will provide greater detail on hydraulic performance and confirm the 1D modeling results.

8.2.1.2 L-62 Canal

The hydraulic evaluation during steady state conditions includes:

- Sizing the canal reach between WCS-9 and the confluence with the C-38 canal
- A velocity evaluation of the canal reach between WCS-9 and the confluence with the C-38 canal
- A depth evaluation of the canal reach between WCS-9 and the confluence with the C-38 canal
- CFD modeling of the L-62 canal confluence, to be completed during preliminary design
- A fully open WCS-13

8.2.1.2.1 Wet Weather

Three wet weather conditions were simulated. Table 8-2 tabulates the prescribed boundary flow conditions established at the upstream end of the 1D model at the G-80 structure and tailwater at the downstream end of the 1D model at the C-38 canal confluence.

The minimum C-value for the existing S154 structure, for various head differentials and different gate openings, was 0.7. The results were compared with the type 4 or 5 flow conditions identified in the Reconnaissance Study. This C-value was also applied to the proposed WCSs 9 and 13.

Table 8-2. Prescribed Flow and Tailwater Wet Weather Conditions for the Headworks Model

Flow Condition	Flow (cfs)	Tailwater at C-38 (ft)	Gates State
High Flow	1,150	12.1	Fully Open
Design Flow	1,000	17.9	Fully Open
Standard Project Flood	3,333	17.9	Fully Open

8.2.1.2.2 Dry Weather

Under dry weather conditions, two flow conditions were identified for water being convey/pumped upstream from the C-38 canal to the pump station. Table 8-3 tabulates the prescribed boundary flow conditions established, the WCS-9 gate position (closed) and at the water surface elevation downstream end of the 1D model at the C-38 canal confluence.

Table 8-3. Prescribed Flow and Tailwater Dry Weather Conditions for the Headworks Model

Flow Condition	Flow (cfs)	Tailwater Elevation at C-38 (ft-NAVD)	Gates State
High Flow	500	10.7	Fully Closed
Survivability Flow	125	9.5	Fully Closed

8.2.1.3 PS

Under dry weather conditions, a constant pump flow rate was used to withdraw 500 cfs and 125 cfs from the C-38 canal. For wet weather (storm) simulations, the pump element was not used to assess only the flood routing capacity of the L-62 canal and associated WCSs.

8.2.2 STA System

A systemwide PCSWMM 1D model of the STA System was developed for the Reconnaissance Study. The 1D model was subsequently updated with dual inflow and outflow WCSs and updated treatment cell elevations from design refinements.

Modeling of the treatment cells was refined by converting these areas from simple 1D conveyance conduits into 2D treatment areas in 2D models. To balance model run time with connectivity of other system elements, a separate model was developed for each of the six treatment cells. The limits of each 2D model included the spreader canals, the treatment cell, and the collection canal. The remaining parts of the systemwide model are included in each cell model as 1D elements.

8.2.2.1 Inflow Canals

The inflow canals were modeled as 1D elements in the hydraulic model from the PS discharge to the respective inflow WCSs for each treatment cell and the PES. Figure 8-2 presents the extents of the inflow canals in the model.

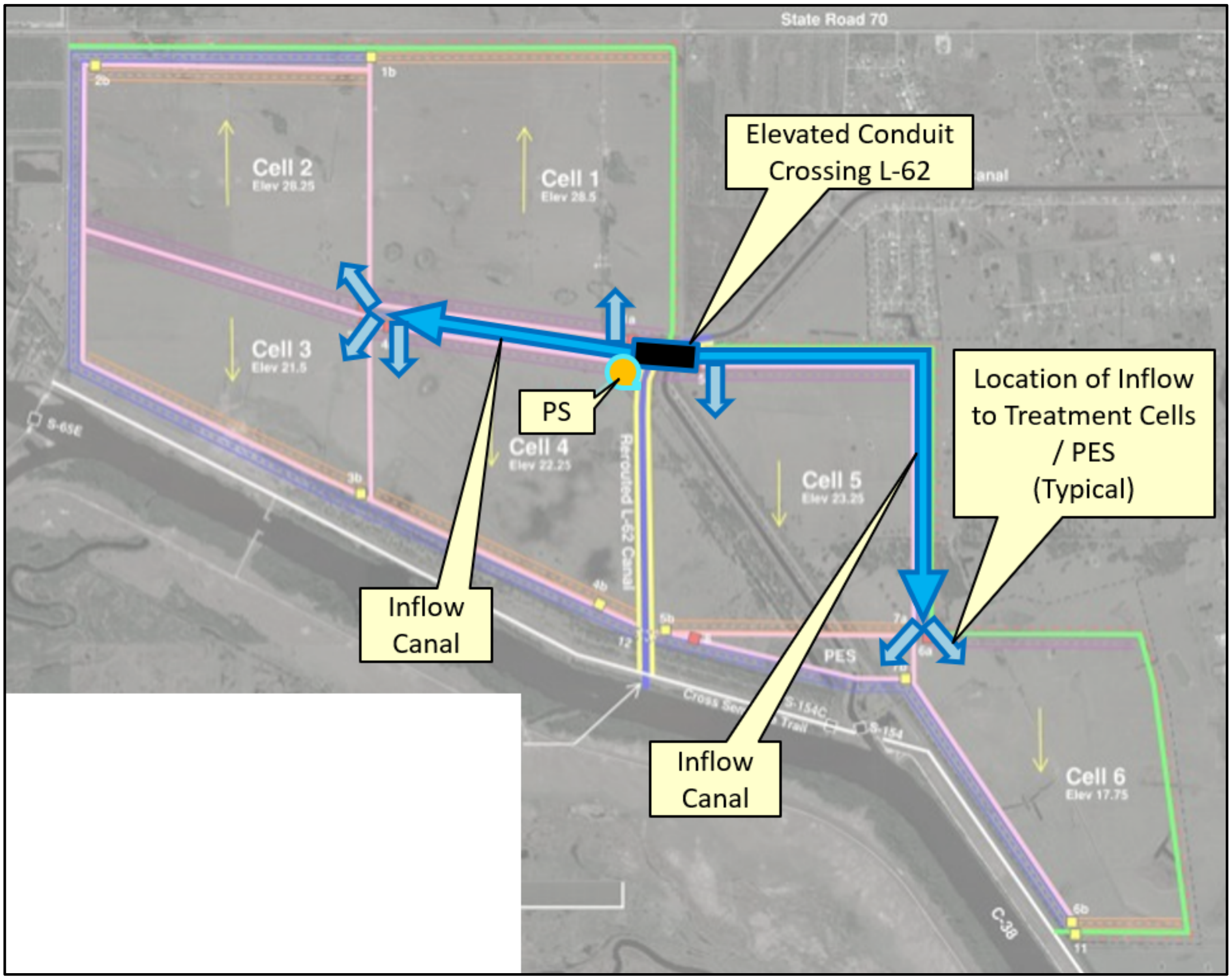

Figure 8-2. STA system inflow canals

The PS discharge is on the west side of the L-62 canal. From this point, the inflow canal flowing west serves Cells 1 through 4. To convey flow to the east side of the L-62 canal, a 7 ft x 7 ft elevated conduit built into the WCS structure will convey flow over the L-62 canal. The eastern inflow canal delivers flow to the Cell 5 WCS, and then to Cell 6 and the PES. A profile of the inflow canals, a typical canal section, and initial model results for normal operations, are presented on Figure 8-3.

PCSWMM was used to determine the inflow canal's water depths, flow velocities, and head loss. Through an iterative approach, canal depths and adjacent inflow WCSs were adjusted to produce a connected system that meets the desired design criteria.

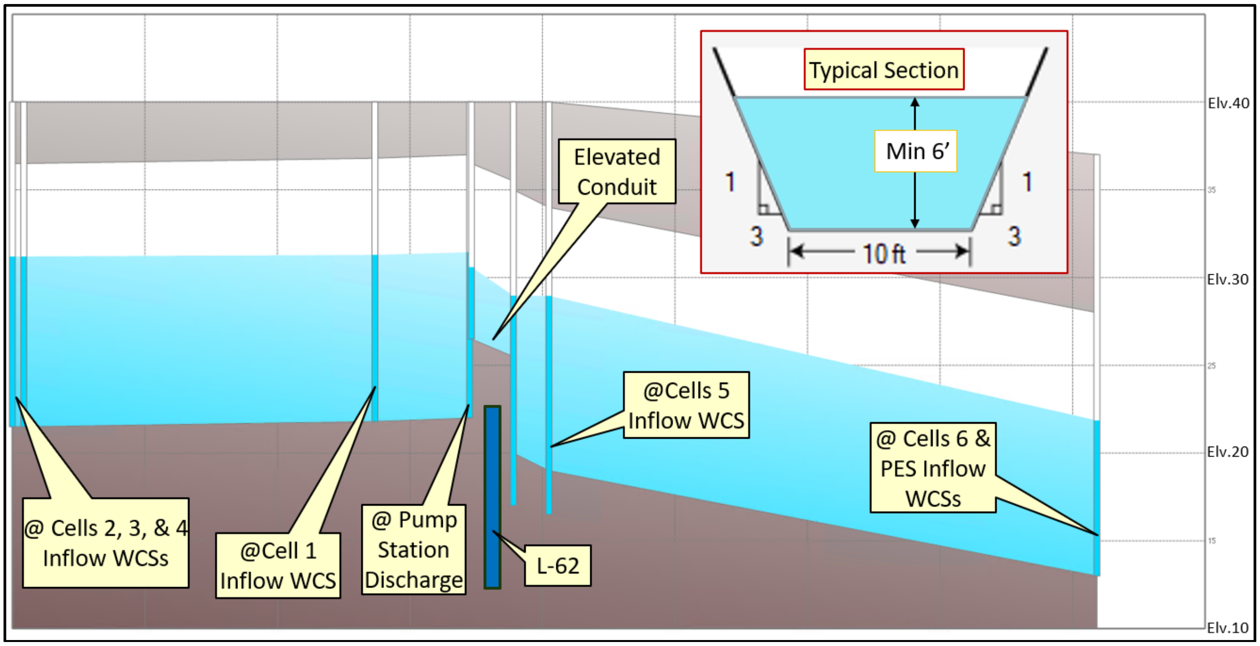

Figure 8-3. STA system inflow canal profile

8.2.2.2 Inflow/Outflow WCSs

Dual parallel inflow and outflow WCSs are proposed for the STA. This will allow the system to operate while one gate is taken offline. Each of the two (dual) structures will have:

- One segment of upstream pipe from inflow canal to gate (inflow WCS) or spreader canal to gate (outflow WCS). The diameter of these pipes ranges from 3 ft to 3.5 ft.
- Square gates between the upstream and downstream pipes. The width and height dimensions of each gate is equal to the diameter of the adjacent pipes.

PCSWMM was used to determine the WCSs gate openings, water depths, flow velocities, and head loss. Through an iterative approach, WCS gate openings were adjusted until flow rates and depths met the desired design criteria. A typical inflow WCS profile and section is presented on Figure 8-4.

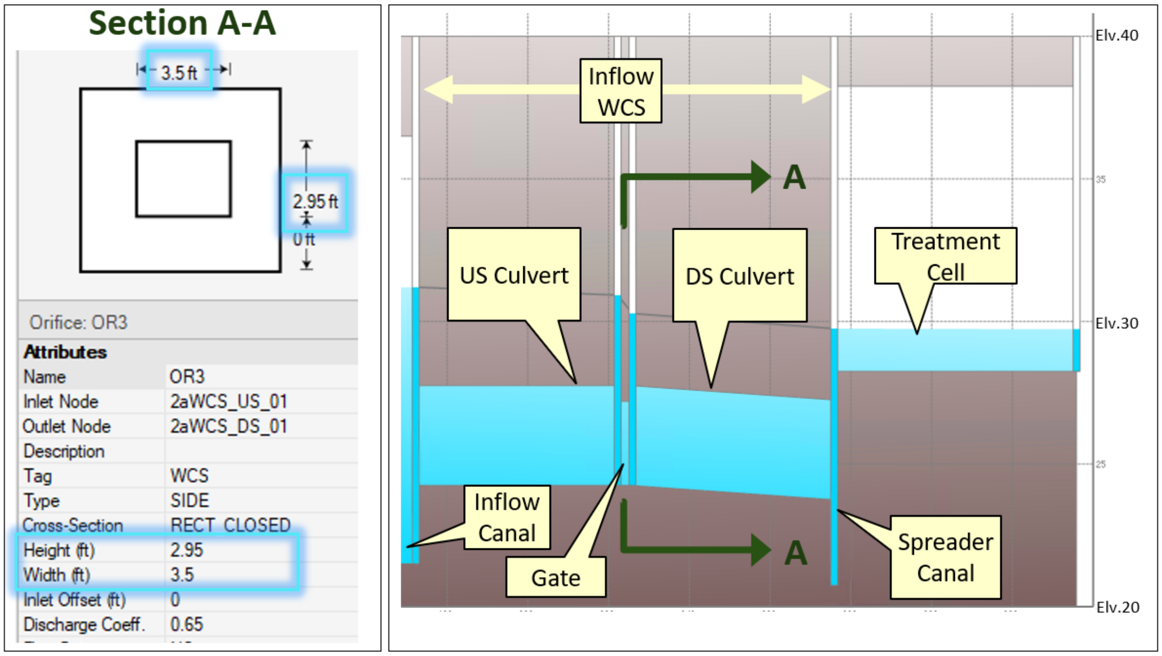

Figure 8-4. Typical treatment cell inflow WCS

8.2.2.3 Treatment Cells

To construct the 2D portions of the STA System models, a digital terrain model (DTM) was constructed in AutoCAD Civil3D. The DTM is a digital representation of the ground surface for the entire treatment cell, including spreader and collection canals. The DTM was imported into PCSWMM, which has a GIS interface able to import, read, and modify DTMs. Then, a mesh representation of the DTM was created in PCSWMM.

Because of the relative uniformity of the proposed treatment cells and the relative variability of the spreader and collections cells (multiple grade breaks from 3:1 side-slopes to flat), three 2D mesh resolutions were created. A mesh resolution is the equivalent distance between 2D nodes for each cell of the 2D surface. Each link connecting 2D nodes has a length and width that yields a link area that, when all links areas are summed, produce an area equal to the total 2D surface area. The spreader and collection canals, including a transition area from the flat treatment area, were modeled with a resolution of 25 ft. The main treatment area is modeled with a 1,000-ft resolution. The third mesh, a transitional mesh, lies between the 1,000-ft and 25-ft resolution meshes, with a resolution of 300 ft to prevent modeling instability that could occur by moving directly from a high-resolution mesh to a low-resolution mesh.

PCSWMM allows for different types of mesh shapes. Hexagonal mesh was selected, as it is typically recommended for flat expansive areas. Figure 8-5 presents an example of the mesh developed for 2D modeling (Cell 2).

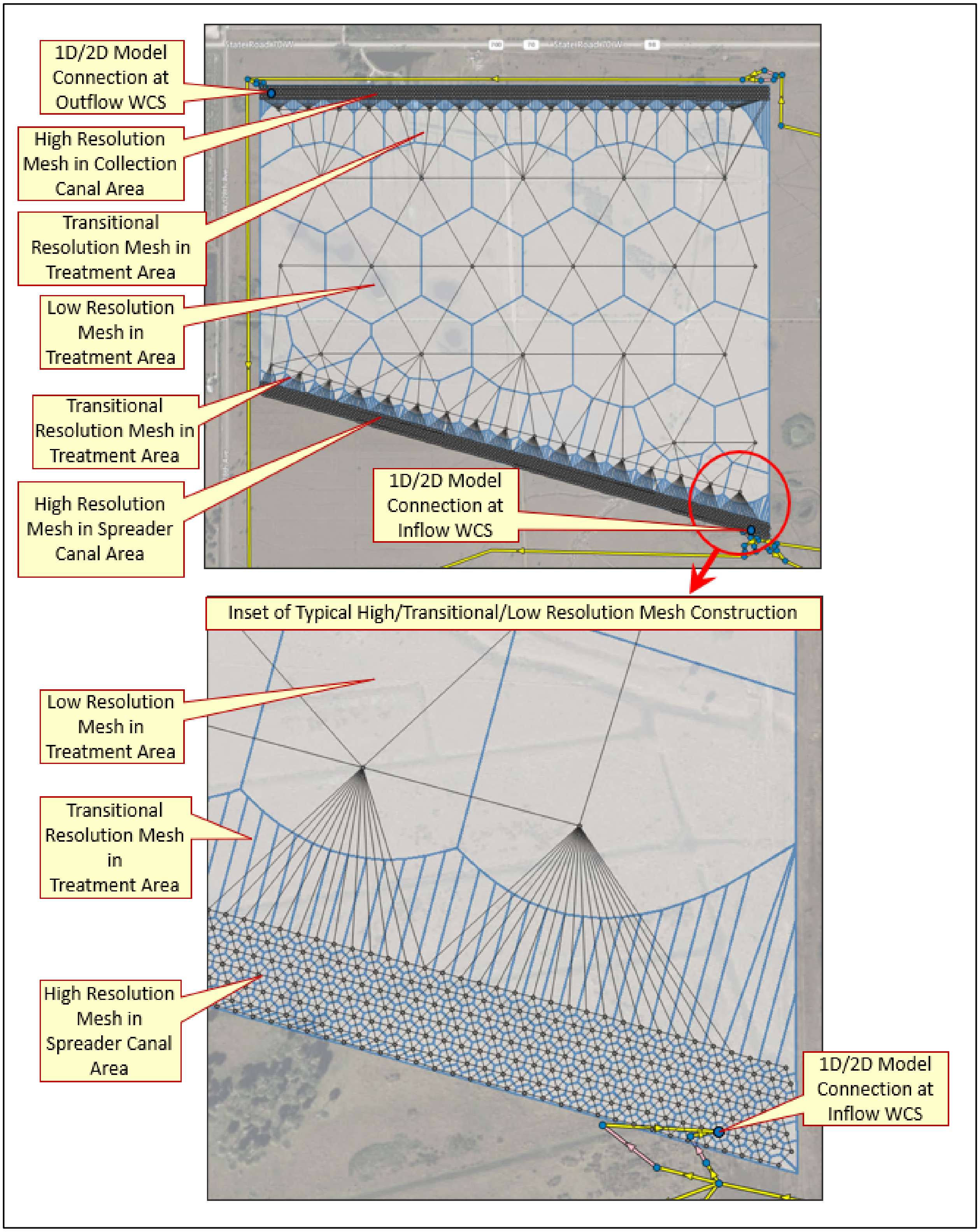

Figure 8-5. Typical 2D treatment cell mesh construction (Cell 2)

Once the mesh for each cell was developed, a network of 2D model nodes and links was generated. A node was created at the center (or centroid) of each mesh polygon with a link to each adjacent node. Figure 8-6 presents the same extents as shown in the inset from Figure 8-5. The first view includes a color spectrum view of the DTM with the model mesh; the second view includes the node/link network overlaid.

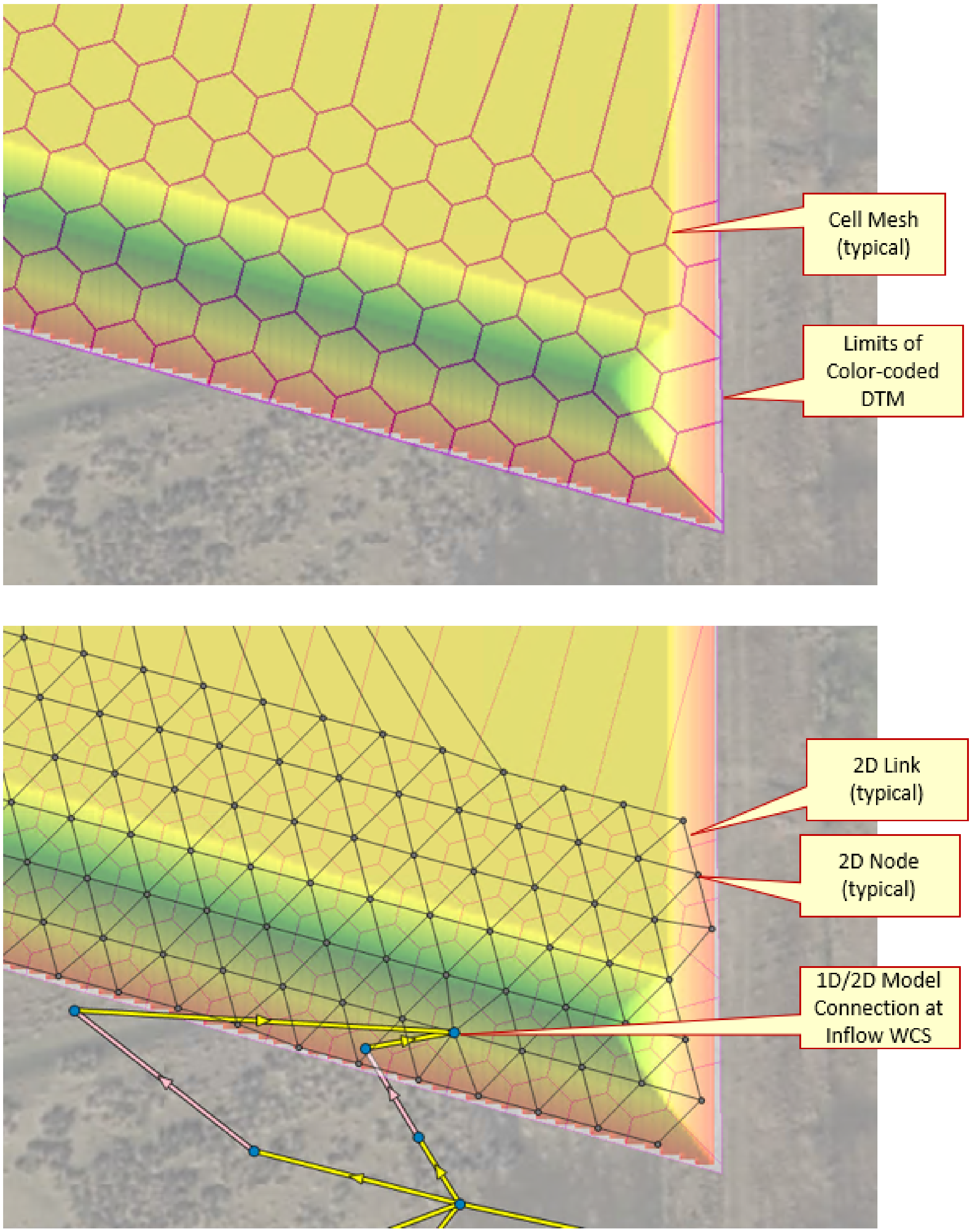

Figure 8-6. Typical 2D treatment cell DTM and 2D nodes/links

The invert elevation of each 2D node was set to the average elevation within its associated mesh polygon. Each 2D link was constructed as a rectangular channel with a width based on the length of the polygon line segment it crosses.

Two potential Manning's roughness coefficients for the treatment cells were established from the Power Law analysis completed during the Reconnaissance Study, where roughness can vary with water depth (see Figure 8-7).

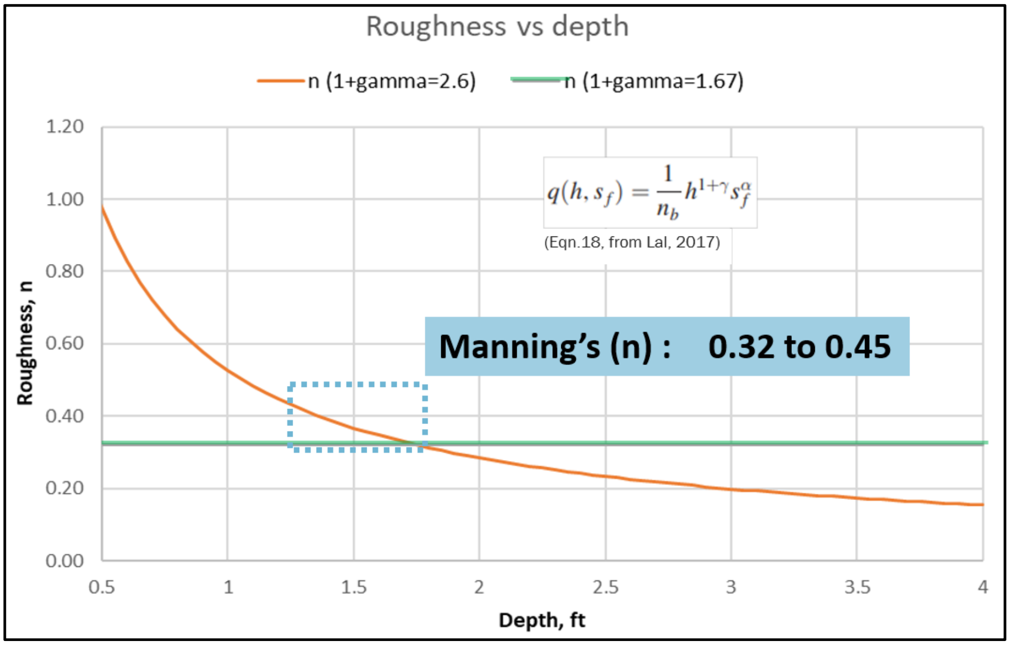

Figure 8-7. Power Law equation analysis (from Reconnaissance Study)

The Manning's roughness coefficient ranged from n = 0.32 to 0.45. This range was selected in a process described in the Reconnaissance Study. For purposes of the DDR analysis, the Manning's roughness coefficient for each link was set to n = 0.32. A sensitivity analysis of using a roughness coefficient of n = 0.45 was performed for Cell 2 showing small increases in depth of less than 0.05 ft, with the same gate openings as the simulation with n = 0.32. Additional sensitivity analysis of roughness will be completed in preliminary design.

8.2.2.4 Outflow Canals

The outflow canals are modeled as 1D elements in the hydraulic model. The first begins at the Cell 1 outflow WCS and flows west to the northwest corner of the site where the Cell 1 outflow WCS discharges, then south along the western property line to the southwest corner, then along the southern limit of the site across the L-62 canal, and ultimately to the S154 outlet structure. Outflow WCSs for Cells 3, 4, and 5 discharge into the outflow canal along the Project southern boundary. The canal is preliminarily sized with a 10-ft-wide bottom and may be adjusted in preliminary design for additional flows, such as dam-break flows. Between the outflow WCSs for Cells 4 and 5, the outflow canal crosses the L-62 canal. The modeling currently assumes this is an inverted siphon crossing preliminarily sized as an 8 ft x 8 ft box. During preliminary design, this crossing will be reevaluated and potentially revised to an elevated conduit.

The second canal begins at the Cell 6 outflow WCS and flows northwest to the S154 outlet structure. Because of the smaller flow rate, the preliminary bottom width of this canal is 5 ft. The outflow from the PES discharges into the outflow canal along this route. The outflow canals then combine and are routed through the S154 outlet structure to discharge into the C-38 canal. Locations of the outflow canals are presented on Figure 8-8. A profile of the outflow canal from Cell 1 to S154 is shown on Figure 8-9, and from Cell 6 to S154 on Figure 8-10.

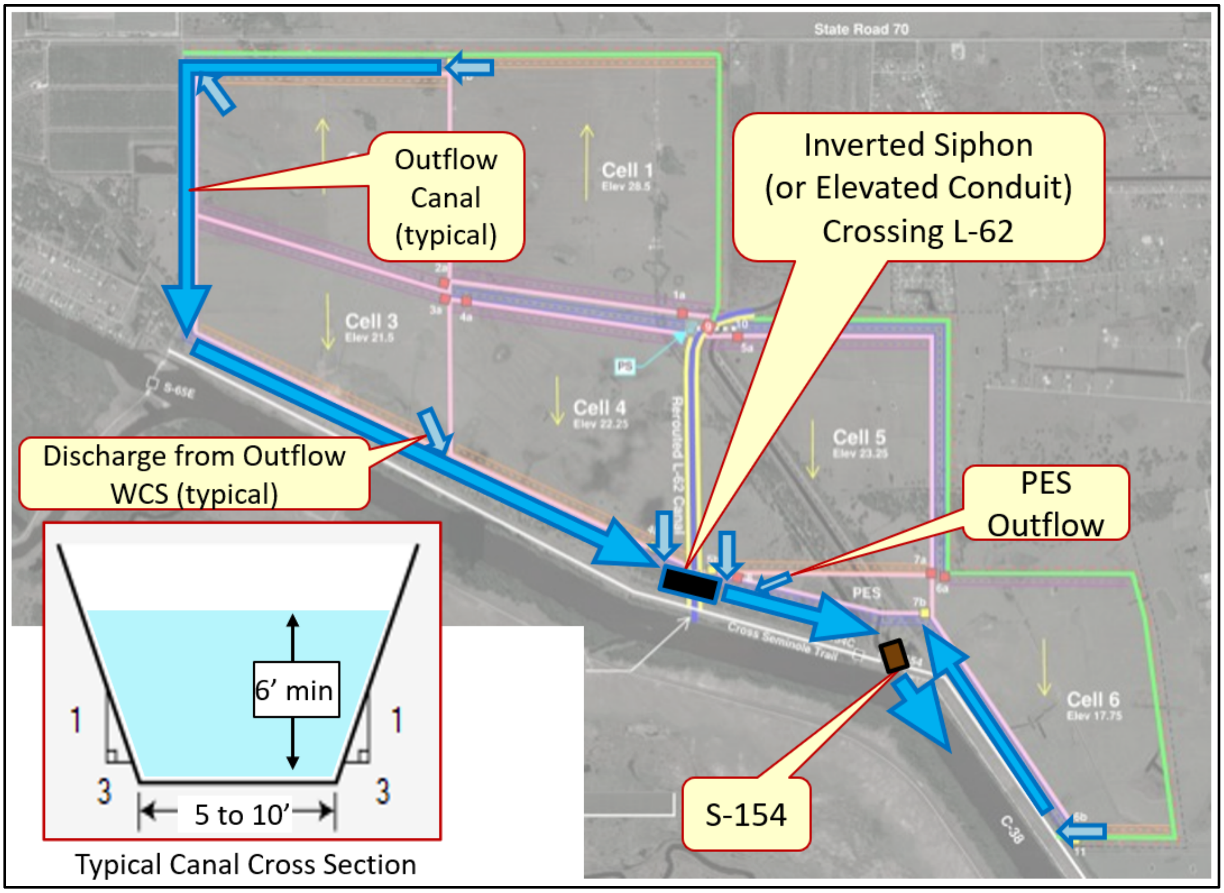

Figure 8-8. STA system outflow canals

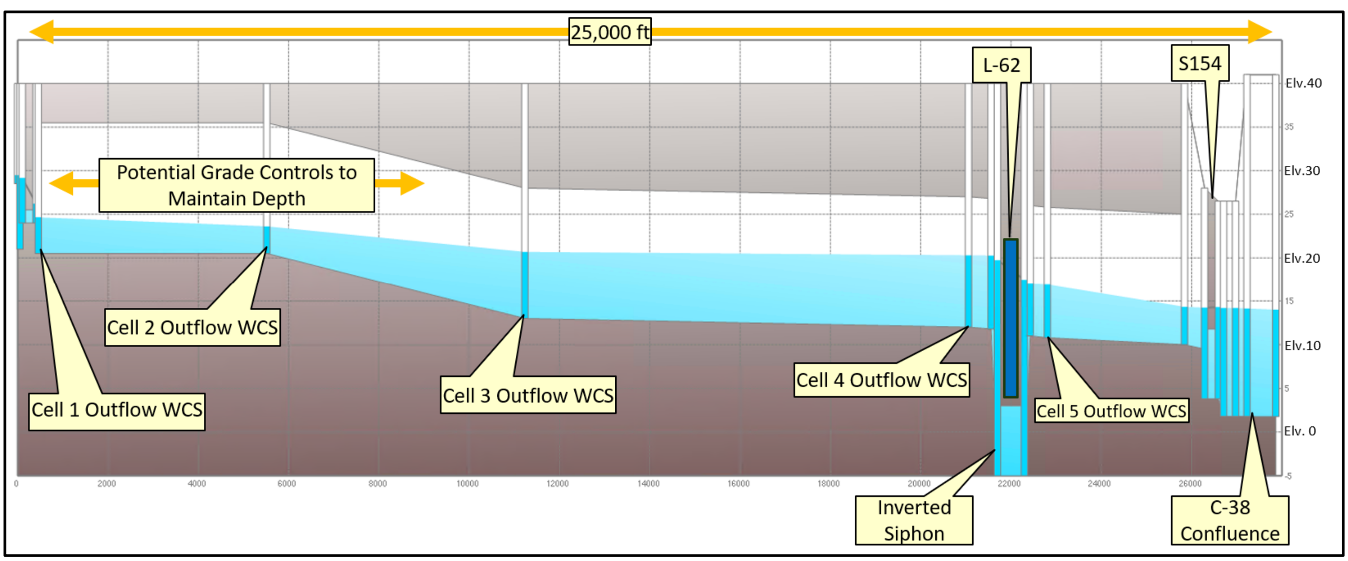

Figure 8-9. STA system outflow canal profile, Cell 1 to S154

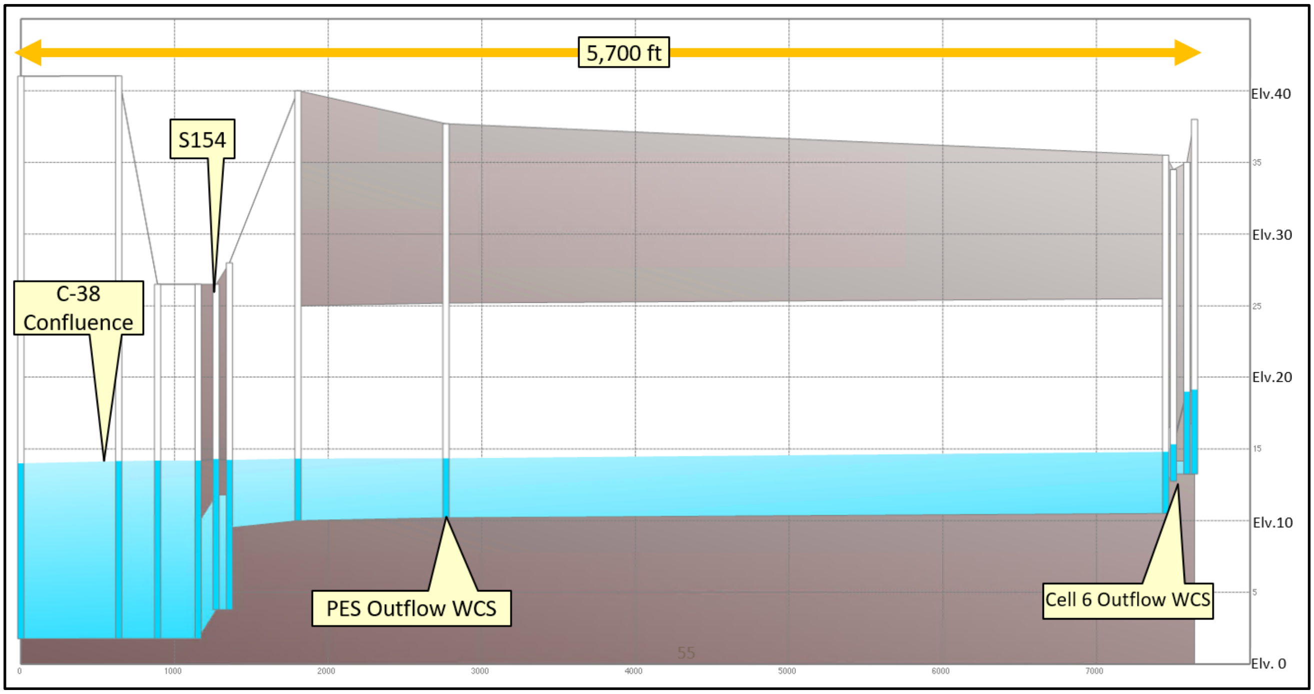

Figure 8-10. STA system outflow canal profile, Cell 6 to S154

PCSWMM was used to determine water depth, flow velocities, and head loss in the outflow canals. It will be used in future phases to confirm the capacity and sizing to route flood flows.

8.2.2.5 Offsite Flows

Offsite flows that currently traverse the site will be routed around the site via the perimeter seepage canals. Offsite flows on the west side of the L-62 canal will be discharged into the L-62 canal and pumped into the STA system. The design flow rates are estimated using methodology described in Section 5 (Hydrology). See sub-section 8.3.2.6 for the flow routing analysis methodology.

8.2.2.6 Seepage Canals

Seepage canals will be used to:

- Manage project seepage and groundwater levels to avoid impacts to adjacent properties.
- Route offsite flows around the Project site.
- Potentially route dam-break flows around the Project site. This will be confirmed during preliminary design.

PCSWMM, using the previously described methodology, was used to size the seepage canals. Canal dimensions and elevations were entered into the model, and flows, discussed in Section 5, were routed through the canals. Resulting water depth, flow velocities, and head loss were determined for the seepage canals. Routing offsite flows controlled the size of the canals, given seepage rates are anticipated to be relatively insignificant compared to the storm flows.

8.2.2.6.1 Seepage West of the L-62 Canal

The runoff from drainage areas north of SR 70 (see Section 5) are currently conveyed under the highway by two culverts (Figure 8-11). Culvert 1 is a 24-in-diameter culvert located 0.35 miles east of SW 128th Avenue. Culvert 2 is a 13 ft x 6 ft box culvert located 1.75 miles east of SW 128th Avenue. Drainage from these culverts currently runs south across the site. To document that the proposed

Project would not reduce the current conveyance capacity, an existing condition model was developed. The model was used to determine the peak flows reaching the existing culverts on their upstream (northern) side, just north of SR 70. As a conservative assumption, the existing culvert's capacity was ignored, and flow was assumed to flow freely from the north side of SR 70 to the south side. For Culvert 1, this was used as the peak flow. For Culvert 2, flow continues south in an existing drainage channel across the proposed Project site which has some constraint on conveyance capacity. The flow estimated in this drainage channel, ignoring culvert capacity limitation, was the estimated peak flow at the Culvert 2 crossing.

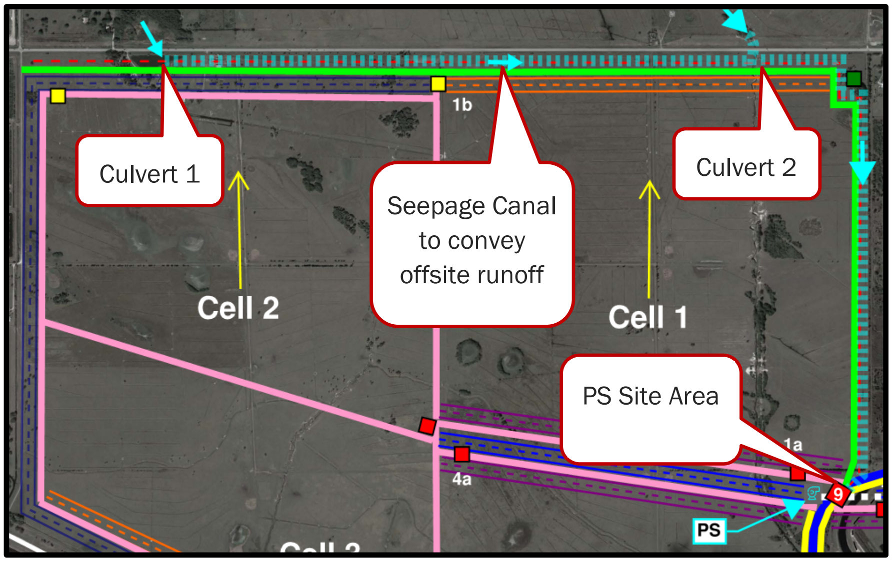

Figure 8-11. West offsite flow management

Under proposed conditions, flows from these culvert crossings will be collected by the seepage canal. The seepage canal is anticipated to have a 10-ft-wide bottom, 3:1 side slopes, and a normal water depth of 6 ft. It will extend west to east along the north site limit, parallel to SR 70, and convey runoff east to the northeast corner of the site, and continue south along the eastern site limit to the PS site area (Figure 8-11). At this location, runoff will be conveyed by a culvert that will run under the PS site area, and discharge into the L-62 canal adjacent to the PS intake, just downstream of WCS-9. This location was selected to allow the water to be pumped into the STA system for treatment. The discharge culvert will be sized to convey flows up to the peak 2-year/1-day storm flow rate. Flows in the seepage canal above the 2-year/1-day storm will overflow across the PS site area and through an overflow weir into the L-62 canal, upstream of WCS-9.

Note that the outflow canal from the Cell 1 outflow WCS to S154 will be deep enough to act as a seepage canal, in addition to conveying discharge from treatment cells.

8.2.2.6.2 Seepage East of the L-62 Canal

The runoff from offsite drainage areas east of the L-62 canal will be collected by the seepage canal located around the north, east, and south sides of the site. The seepage canal begins on the north side of Cell 5 (identified generally as Junction 1 (J1) on Figure 8-12). Some runoff will be collected as dispersed runoff; however, an existing drainage ditch enters the site north of Cell 6, and extends south across the Cell 6 area before continuing south of the site, parallel to the HHD. This drainage ditch will be intercepted by the seepage canal north of Cell 6 (J2 on Figure 8-12). Drainage will then be conveyed around the eastern site of Cell 6 and rejoin the existing ditch south of Cell 6 (J3 on Figure 8-12). A WCS may be added at the downstream end of this seepage canal which will be assessed and advanced during preliminary design. The seepage canal is anticipated to have a 10-ft bottom width and 3:1 side slopes. For modeling purposes, the channel is assumed to function as a ditch with a dry bottom during dry weather.

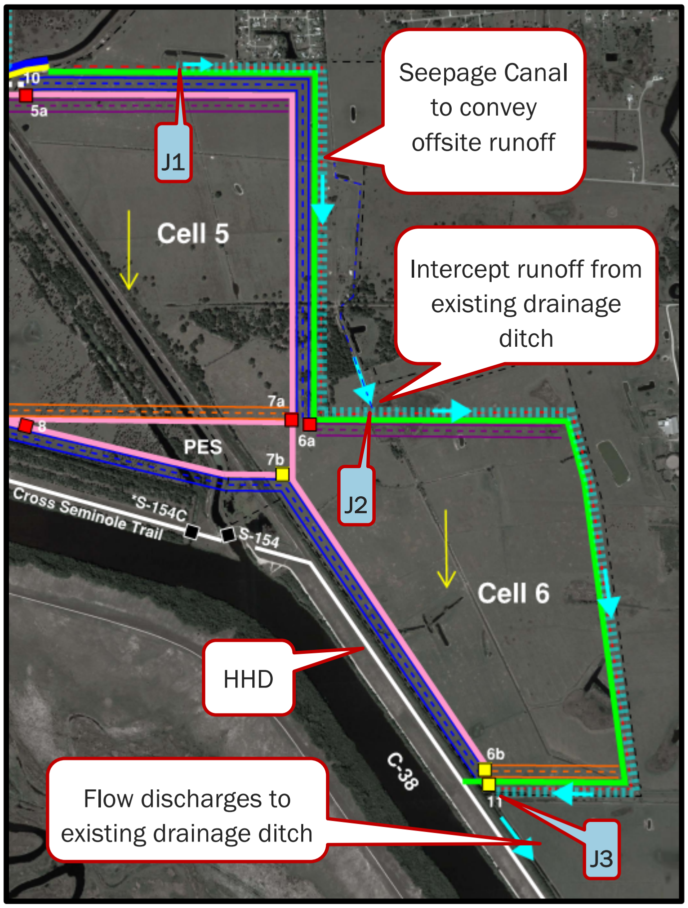

Figure 8-12. East offsite flow management

8.2.3 Hazard Potential Classification

The HPC is based on the DCM-1 guidelines. It is recommended that the LKBSTA be classified as a low hazard facility based on the evaluation of four categories of potential impacts identified in DCM-1.

A dam-break analysis will be completed during preliminary design to substantiate the classification; however, the hazard potential evaluation completed to date considered the four categories of potential impacts per DCM-1 as presented in the following sub-sections.

8.2.3.1 Direct Loss of Life

Hazard Potential: Low – Direct loss of life not expected

The closest structures intended for human habitation are houses located on the west, north, and east sides of the Project. The lowest ground elevation of these properties is 29.7 ft (NAVD), which is slightly below the estimated maximum STA water storage level (MWSL) of 31.0 ft. The outflow canal along the western boundary is expected to convey any water volume from a potential breach on the west side of the Project. Water discharged from a perimeter embankment breach on the north and east sides of the STA would discharge into the seepage canals or the outflow canal. Therefore, direct loss of life from a breach of the embankment along the west, north, and east side of the STA is not expected based on the work to date. This will be assessed further during preliminary design. An overview of the structures around the STAs are shown on Figure 8-13.

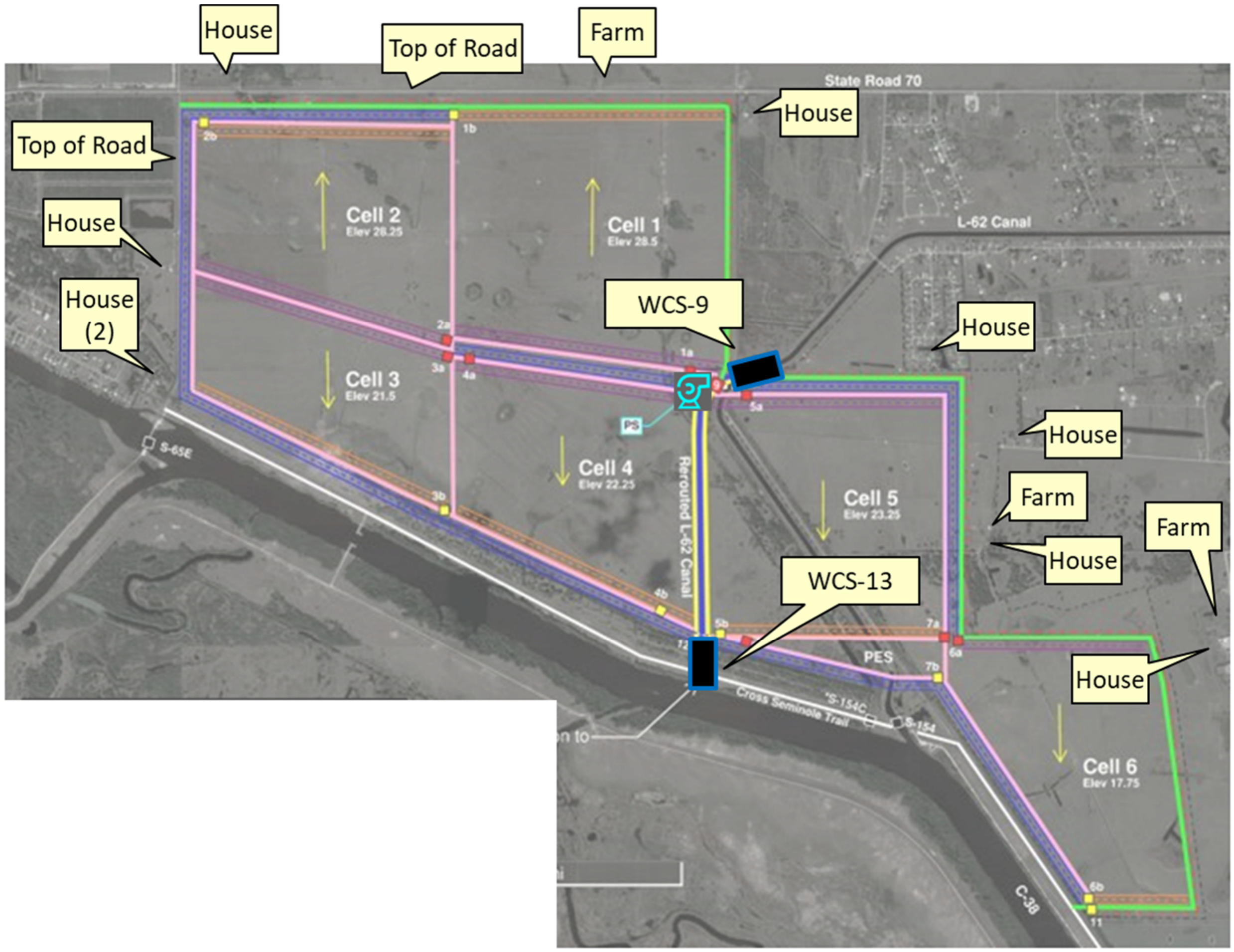

Figure 8-13. Hazard potential overview map

8.2.3.2 Lifeline Losses

Hazard Potential: Low – No disruption of services

There is one primary means of egress from the STA site: SR 70, located on the site's north side. In an emergency, the recommended route is SR 70 east to the City of Okeechobee, where emergency medical facilities are available. SR 70 in the vicinity of the Project site ranges in elevation from about 30.6 ft (NAVD) to about 31.6 ft (NAVD), with the lowest elevation below the assumed MWSL in the STA of 31.0 ft (NAVD). However, water discharged from a breach of a perimeter embankment on the north or east sides of the STA would discharge into the seepage canal or the outflow canal. Potential impacts to vehicular site access resulting from an STA perimeter embankment breach will be evaluated further in preliminary design.

In the unlikely event that access to SR 70 along the perimeter levee system is not possible due to a catastrophic event, it should still be possible to exit the site and travel along SR 70 west toward US Highway 27.

8.2.3.3 Economic Losses

Hazard Potential: Low – Agricultural land, equipment, isolated buildings impacted

Land immediately north, east, and west of the LKBSTA is mainly in agricultural production. EIP owns a portion of the land to the east. Land to the north and west of the STA is privately owned. The probability of impacts to agricultural land north or west of the STA are low, but an embankment breach along the north or west side of the STA could have short-term adverse impacts (crop loss) on privately owned agricultural land to the north. Impacts to agricultural land, equipment, and isolated buildings are anticipated to be classified as low hazard potential based on DCM-1. This will be evaluated further in preliminary design.

There is also a low probability of agricultural lands to the east or west being impacted by a breach since the STA seepage and outflow canals would limit any water discharged from the STA from moving further south to the C-38 canal.

FPL overhead power distribution lines run along SR 70. These lines will provide low-voltage power to the PS located about 1 mile south of the proposed FPL entry point to the site, at the northeast corner of the STA. These lines also provide power to privately owned agricultural PSs, farms, and households along SR 70. Because these lines are all above grade and located along SR 70 at approximately elevation 31 ft (NAVD), there is low probability that they would be impacted by a breach of the perimeter embankment along the STA's north side. It is anticipated that any flow from such a breach would leave the STA via seepage or outflow canal and be routed south to the C-38 canal. This will be evaluated further in preliminary design.

Similarly, a breach along the south side of the STA would leave the STA via the outflow canal and be routed south to the C-38 canal. As previously stated, a dam-break analysis will be completed in preliminary design.

8.2.3.4 Environmental Losses

Hazard Potential: Low – Minimal incremental impact

The LKBSTA will be surrounded by canals on all four sides. The capacity of these canals to absorb water discharged from the STA due to a breach of a perimeter embankment will be assessed during preliminary design. All LKBSTA perimeter levees will have top-of-levee elevations well above the assumed MWSL. Due to the Project area's remote location with respect to nearby wildlife management areas, the probability of impact to wildlife management areas is low. This will be evaluated further in preliminary design.

8.2.4 Wind Set-up/Wave Run-up and Freeboard

Based on the Coastal Engineering Manual (USACE, 2003a), the freeboard calculation is a 3-step method that involves calculating wave characteristics, the wind setup, and the wave runup. The wind setup is defined as the upward buildup of water resulting from the wind pushing over the water surface, as shown on Figure 8-14. The wave runup is the maximum vertical extent of the wave uprush on a structure (e.g., beach, levee) above the still water level. The wave runup calculation uses wave characteristics such as wave height, wave length, and wave period.

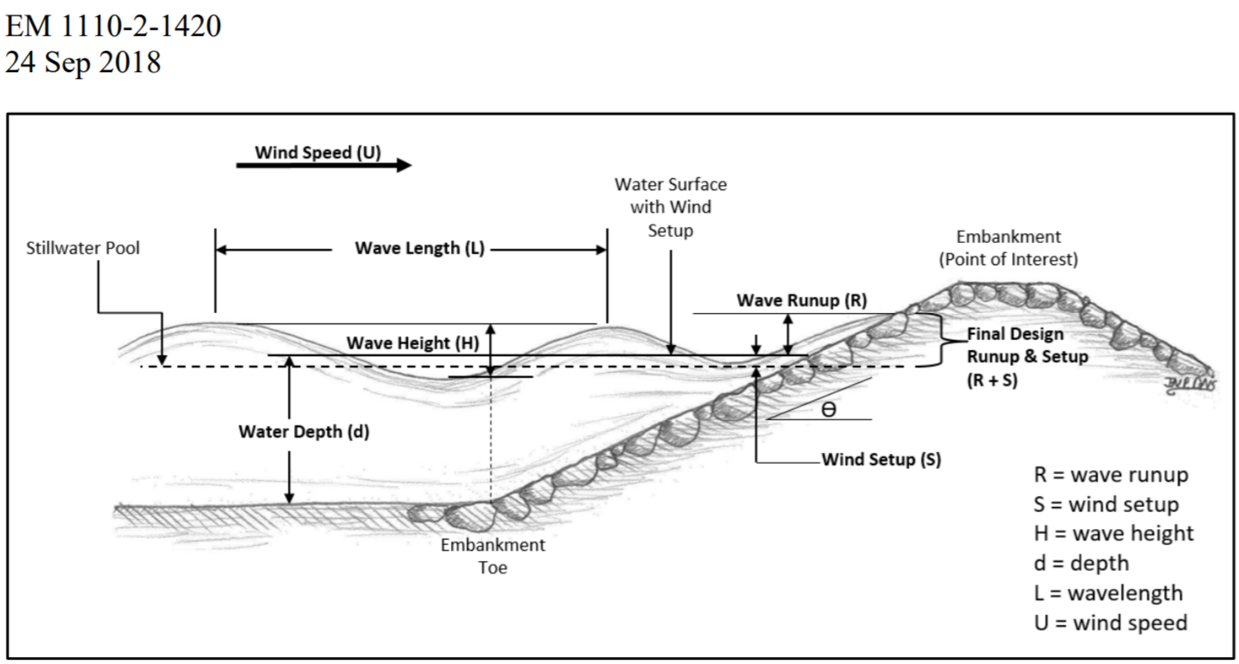

Figure 8-14. Wave run-up and wind setup from EM 1110-2-1420

Cell 1 and a combined supercell of Cells 1 and 2 were analyzed in the freeboard calculations with a maximum treatment cell water depth of 2.77 ft which accounts for the maximum operating depth of 2 ft plus the runoff for the 100-year, 24-hour storm which equates to a depth of 0.77 ft (9.2 in) of direct rainfall in the cell. Due to its geometry, Cell 1 was identified as the largest cell with the potential to generate the highest wind setup and wave run-up. All calculations were based on the conservative assumption that a lower Manning's roughness coefficient of 0.32 is prevalent among the emergent aquatic vegetation. This vegetation assumption allowed for simplified calculations and conservative results, as emergent vegetation is more likely to dampen waves.

8.2.4.1 Wind Setup

Wind stress acting on the surface of a water body increases the water's surface elevation in a process known as wind setup. Three models were tested to evaluate the wind setup height on top of the still water depth: the Zeider Zee model, the Sibul model and the Bretschneider model. The Zeider Zee model determines the wind setup height according to the following equation (USACE, 1997, page 15-3):

$$S = \left[\frac{U^2 F}{1400\ d} \right]$$

where S is the wind setup height on top of the still water depth [ft], U is the average wind speed [miles per hour], F is the total fetch distance [miles], and d is the average (i.e., normal full storage) water depth along the fetch [ft]. The total fetch is the longest distance across open water the potential wind blows to generate waves and was calculated by measuring the distance from the southeast corner to the northwest corner of each cell. The Zeider Zee model is commonly used in deep and large reservoirs that are substantially deeper than the Project site. As a result, it tends to overestimate the wind setup in shallow impoundments.

An alternative is the Sibul model, which is an empirical expression developed by Brater (Brater *et. al*., 1996) per the equation:

$$S = 2.44\times10^{-5}\left(\frac{F}{d}\right)^{1.66}\left(\frac{U^2}{F\,g}\right)^{2.02\left(\frac{F}{d}\right)^{-0.0768}}$$

where S is the wind setup height on top of the still water depth [ft], F is the total fetch length [ft], U is the wind speed [ft/second], g is the gravitational constant [ft/second^2], and d is the average water depth along the total fetch [ft]. The Sibul model tends to underestimate the wind setup.

For water depth less than 16 ft, the recommended wind setup model is the mass-balance-based Bretschneider's model (Bretschneider, 1966). The trapezoidal rule was used to calculate the cumulative volumes of setup and setdown along the fetch and the total mass balance between the initial water volume and the final water volume. Note that the wind setup calculation did not account for any friction due to emergent and submerged vegetation, which is accounted for in wave generation. The wind setup estimations of the Zeider Zee and Sibul models were used to check and validate the Bretschneider's model. Refer to Appendix 4 for the spreadsheets containing the calculations.

8.2.4.2 Wave Generation and Propagation

Determining wave height requires using the effective fetch, whereas the total fetch (i.e., longest fetch in each cell) suffices for the wind setup calculation. The effective fetch length is based on A Guide for Design and Layout of Vegetative Wave Protection for Earth Dam Embankments (USDA, 1974). Computation of the maximum effective fetch for the external embankment is essentially a trial-and-error process for reservoirs with irregular shapes. In the case of the LKBSTA, where Cells 1 through 6 have near-rectangular shapes, the trial-and-error process is simplified when accounting for dominant wind direction. The windrose plot on Figure 8-13 produced by Iowa State University (Iowa State University, 2022) using Okeechobee 2007–2022 data provides frequencies of wind direction and wind speed. Based on Figure 8-15, wind was assumed to blow from the east or northeast across the proposed LKBSTA water surface. Wind from the northeast was selected since it led to a greater effective fetch than when the east direction was selected for Cell 1. Refer to Appendix 4 for maps illustrating the effective fetch measurements based on wind direction.

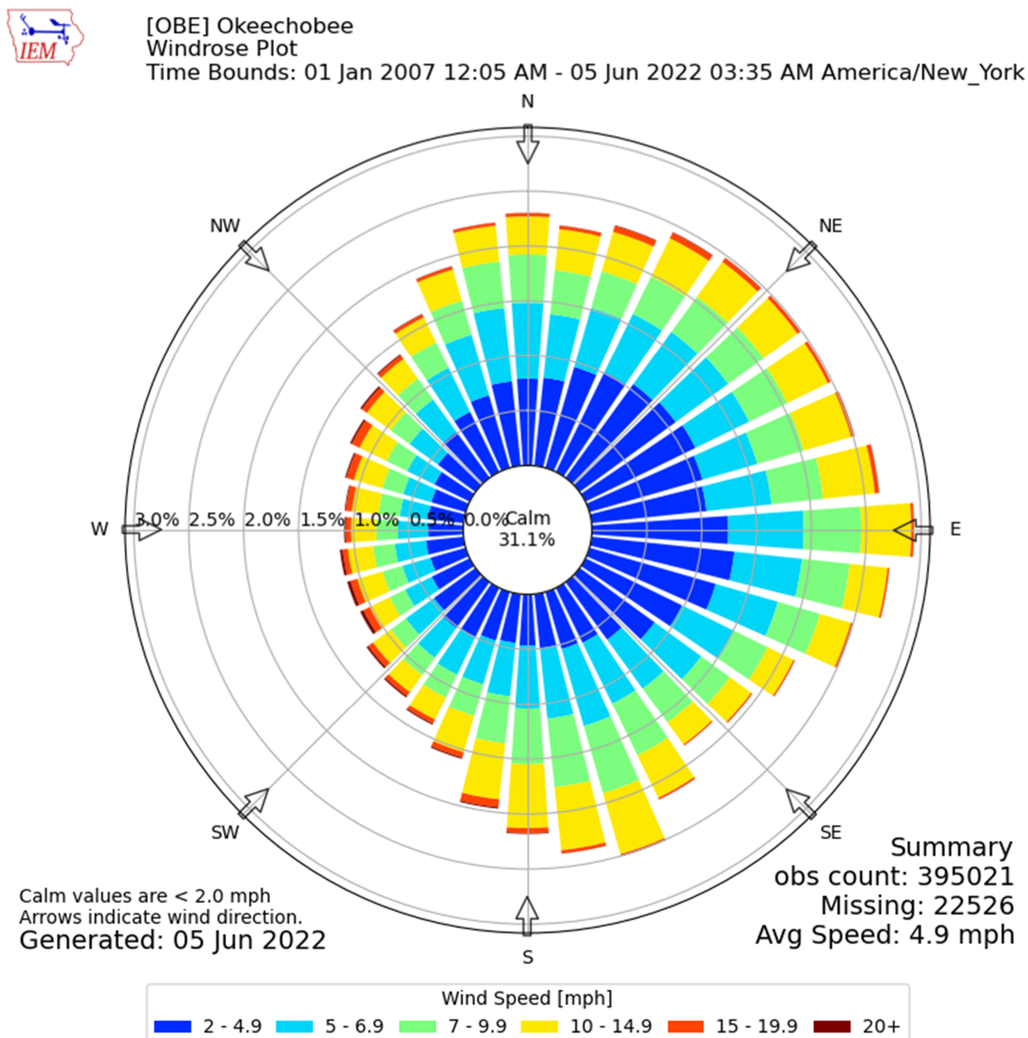

Figure 8-15. 2007-2022 Okeechobee windrose analysis

Source: Iowa State University Environmental Mesonet

The Coastal Engineering Manual (USACE, 2003a) describes the dependency of wave height and period on fetch length and wind speed assuming a fetch-limited condition. All the cells in the Project site were found to be fetch limited.

Waves propagating over rough or highly vegetated areas dissipate their energy through bottom friction and breaking. This energy dissipation greatly reduces the wave height at the end of a given fetch. Neglecting wave breaking and assuming shallow water with a constant depth (both conservative assumptions), (Dean and Dalrymple, 1991) as well as (USACE and WES, 1993a) developed a conservation of energy approach that accounts for wave energy dissipation due to bottom friction. The bottom friction coefficient includes Manning's n. A Manning's roughness coefficient (previously shown on Figure 8-9) of 0.32 was used to account for the vegetation expected in each cell, with EAV included for reference, as referenced in the Reconnaissance Study (EIP, 2022).

The Coastal Engineering Manual (USACE, 2003a) gives the following governing equation for the zeroth-moment wave height [m]:

$$\frac{d(H_{mo})^2}{dx} = \frac{1}{gc_g(x)}\left[\left(4.13\times10^{-2}\right)^2 u_*^2 c_g(x) - \frac{f}{6\pi}\left(\frac{H_{mo}\sigma(x)}{\sinh(kh)}\right)^3\right]$$

This equation was solved numerically using the 4th order Runge-Kutta's method (RK). The performance of the RK scheme depends on the spatial discretization of the governing equation. Therefore, an iterative solution was implemented that produces a reasonable answer with a convergence tolerance of 1 percent. The output of the numerical scheme yields to the zeroth-moment wave height generated by a given wind over a given effective fetch with energy dissipation due to real bottom conditions described by the friction coefficient. Also, the Coastal Engineering Manual (USACE, 2003a) recommends a breaker index (H/h) of 0.6 in shallow water conditions. Refer to Appendix 4 for the spreadsheets containing the calculations.

8.2.4.3 Wave Run-up

The wave run-up height was calculated by taking the average of two commonly used models: Ahrens and Heimbaugh model and the de Waal and van der Meer model. The Ahrens and Heimbaugh model (Ahrens and Heimbaugh, 1988) (USACE, 1995, page 2-5) is suitable for irregular wave distribution reaching rough surfaces and is described with the following equation:

$$\frac{R_{\max}}{H_{mo}} = \frac{a\xi}{1+b\xi}$$

where R_{max} is the maximum runup [m], H_{mo} is the zeroth-moment wave height [m], and a and b are regression parameters equivalent to 1.022 and 0.247, respectively. x is a dimensionless surf parameter defined as:

$$\xi = \frac{\tan\theta}{\sqrt{\dfrac{2\pi H_{mo}}{g(T_p)^2}}}$$

where Θ is the angle between the structure slope and the horizontal plane and Tp is the peak period of the irregular wave distribution [s]. The de Waal and Van der Meer model (de Waal and van der Meer, 1992) (USACE, 2006, page VI-5-10) is adequate for smooth revetment surface and is described as:

$$\frac{R_{u2\%}}{H_s} = 1.5\xi_{op} \; for \; 0.5 < \xi_{op} \le 2$$

$$\frac{R_{u2\%}}{H_s} = 3.0 \; for \; 2 < \xi_{op} \le 3-4$$

where x_{op} is a dimensionless surf parameter, H_s is the significant wave height [m], and $R_{u2\%}$ is the vertical runup distance exceeded by 2 percent of runups. Both x_{op} and H_s are expressed as:

$$\xi_{op} = \frac{\tan\theta}{\sqrt{\dfrac{2\pi H_s}{g(T_p)^2}}}$$

$$H_s = H_{mo} \exp^{[C_0\left(\frac{d}{g(T_p)^2}\right)^{-C1}]}$$

where C_o and C1 are regression constants equal to 0.00089 and 0.834, respectively. The wave runup is based on a 3H:1V embankment slope and a more conservative value of 0.00136 for the coefficient C1 as recommended by Hughes and Borgman (Hughes and Borgman, 1987). Refer to Appendix 4 for the spreadsheets containing the calculations.

8.2.5 Flood Routing

The STA system will operate as an offline system with the primary function of water treatment, specifically phosphorus reduction. Therefore, flood routing is limited to routing direct rainfall runoff. Quantifying and routing this runoff will be completed in PCSWMM using the previously described numerical methods. This analysis will occur during preliminary design.

8.3 Results

Electronic model files are provided separately. A list of the electronic file names and associated modeling scenario is included in Appendix 4.

8.3.1 Headworks System

8.3.1.1 WCS-9 and WCS-13

The WCS-9 and WCS-13 structure sizes were determined based on the proposed flow conditions and variations of structure elevation while meeting the design criteria. Higher structure elevations for WCS-9 were initially proposed but caused a higher head loss under high flow conditions that eventually exceeded the WSE design criteria of 22 ft (NAVD) at G-80. Tables 8-4 and 8-5 tabulate the depths, velocities, and WSEs upstream (US) and downstream (DS) of WCS-9 and WCS-13 under wet weather conditions. For the Standard Project Flood flow scenario, the gate generates a WSE at G-80 of 30.0 ft (NAVD), which produces a lower WSE compared to the WSE produced by the existing S154 structure at the same elevation of 30.6 ft (NAVD).

Therefore, a preliminary structure size for WCS-9 is 15 ft wide by 8 ft high. The proposed structure has a bottom elevation of 7 ft (NAVD)with a weir crest elevation at 10 ft (NAVD).

A preliminary size for WCS-13 structure is 12 ft high and 12 ft wide. The proposed gate structure elevation of 1 ft (NAVD)with double rectangular barrel culverts with identical dimensions at an invert elevation of –2 ft (NAVD).

Electronic model files are provided separately. A list of the electronic file names and associated modeling scenario is included in Appendix 4.

Table 8-4. Hydraulic Parameters for WCS-9 Under Wet Weather Conditions									
Flow Condition	Flow (cfs)	Tailwater at C-38 (ft)	US WSE (ft)	US depth² (ft)	DS WSE (ft)	DS depth² (ft)	Velocity (fps)¹	WSE G-80 (ft)	Gates State
High Flow	1,150	12.1	18.8	8.8	13.1	3.1	9.7	22.0	Fully Open
High Flow	1,150	12.1	18.8	8.8	13.1	3.1	10.1	22.0	Partially Open 54%
Design Flow	1,000	17.9	19.3	9.3	18.5	8.5	3.4	21.6	Fully Open
Standard Project Flood	3,333	17.9	28.6	18.6	23.0	13.0	7.8	30.0	Fully Open

1: Velocity at the structure
2: US and DS depths near the structure

Table 8-5. Hydraulic Parameters for WCS-13 Under Wet Weather Conditions								
Flow Condition	Flow (cfs)	Tailwater at C-38 (ft)	US WSE (ft)	US depth² (ft)	DS WSE (ft)	DS depth² (ft)	Velocity¹ (fps)	Gates State
High Flow	1,150	12.1	12.5	11.5	12.1	11.1	4.2	Fully Open
High Flow	1,150	12.1	12.5	11.5	11.5	11.1	4.0	Fully Open
Design Flow	1,000	17.9	18.4	17.4	17.9	16.9	3.5	Fully Open
Standard Project Flood	3,333	17.9	22.7	21.7	17.9	16.9	11.5	Fully Open

1: Velocity at the structure
2: US and DS depths near the structure

Figure 8-16 shows the profile of WCS-9 for high-flow conditions with both gates partially open to 54 percent (4.3 ft open of the 8 ft high gates). By closing the gates, the flow condition is characterized as controlled. Despite the gates being partially constricted, the WSE at G-80 is less than 22 ft which is identical to the existing S154 structure for the same flow condition.

Figure 8-17 depicts the profile of WCS-13 for the same high-flow conditions. Both WCS-9 and WCS-13 have gates in the open position and the flow condition is characterized as uncontrolled submerged.

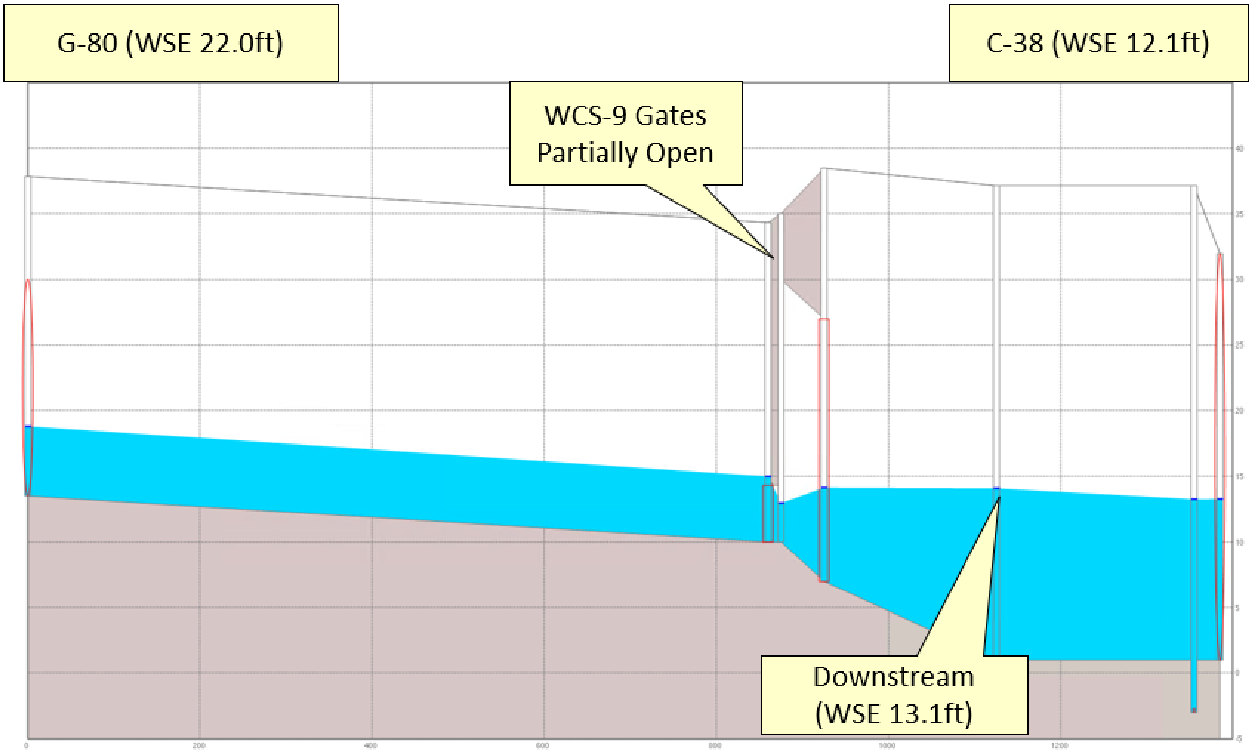

Figure 8-16. WCS-9 profile under high-flow conditions (1,150 cfs)

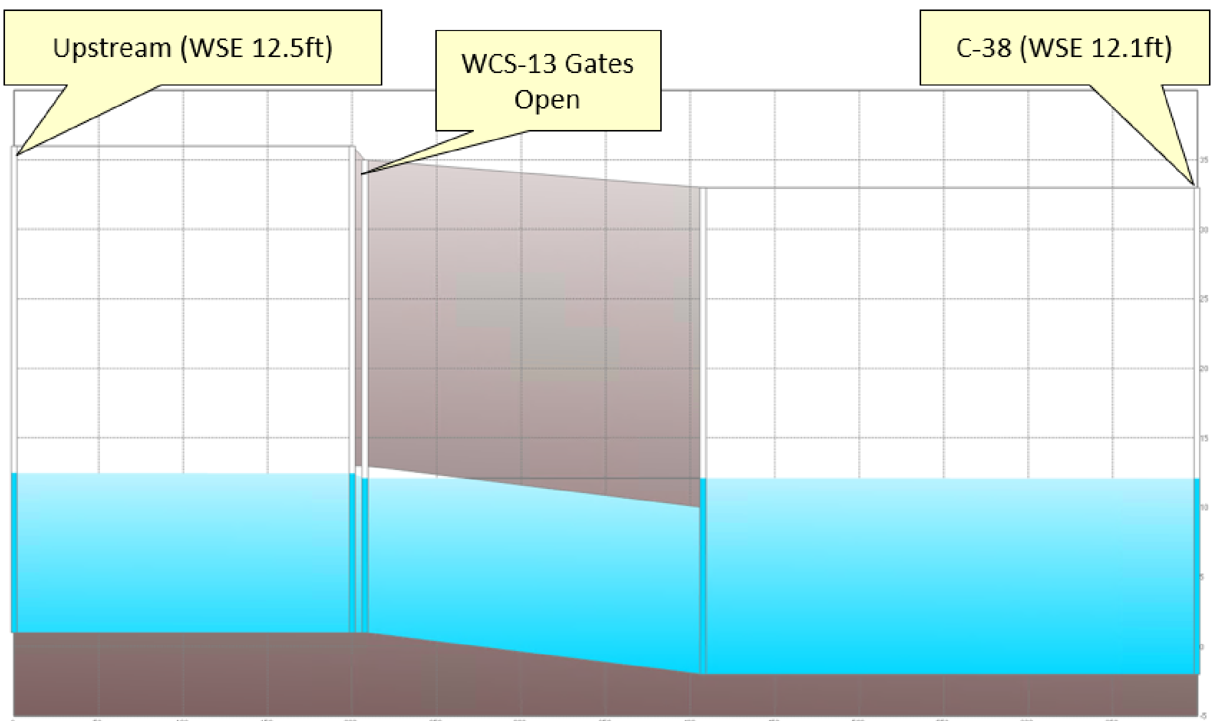

Figure 8-17. WCS-13 profile under high-flow conditions (1,150 cfs)

Figure 8-18 depicts the profile for WCS-9 under design-flow conditions. The flow condition is characterized as controlled submerged with the gates 100 percent open. Figure 8-19 depicts the profile for WCS-13 under design-flow conditions with gates open, also characterized as controlled submerged.

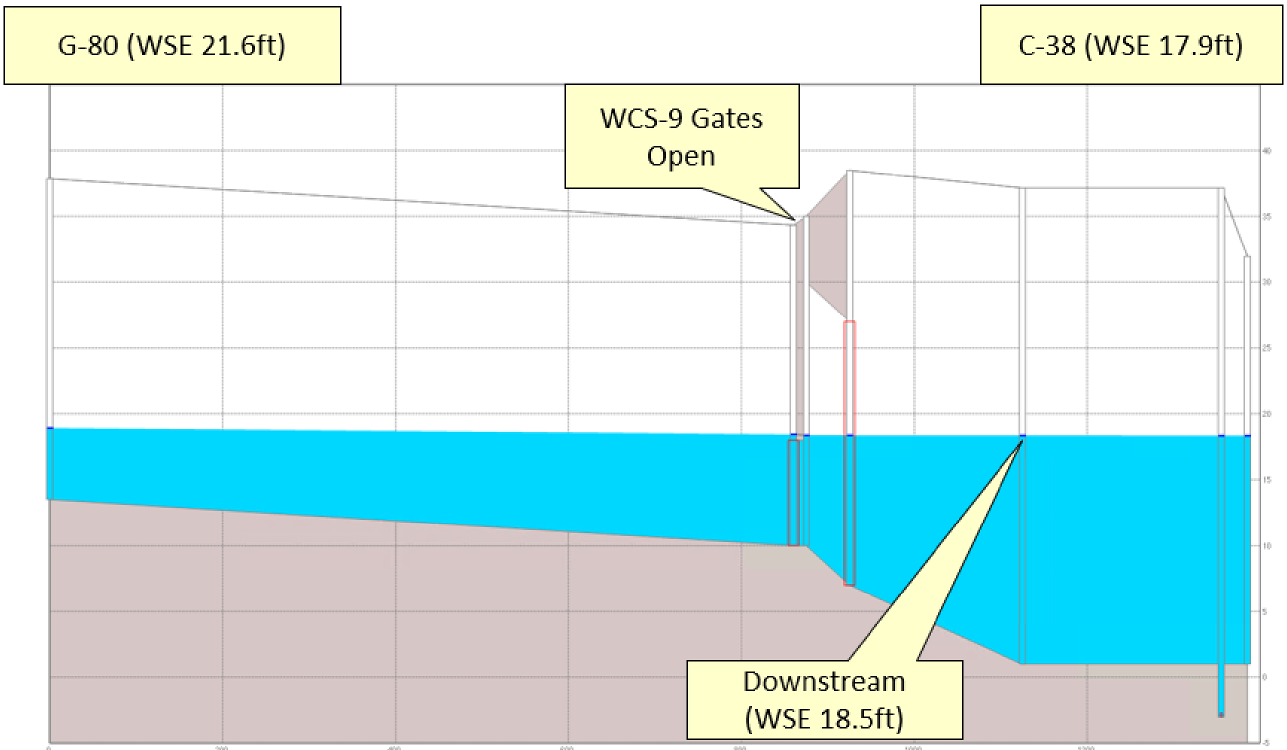

Figure 8-18. WCS-9 profile under design-flow conditions (1,000 cfs)

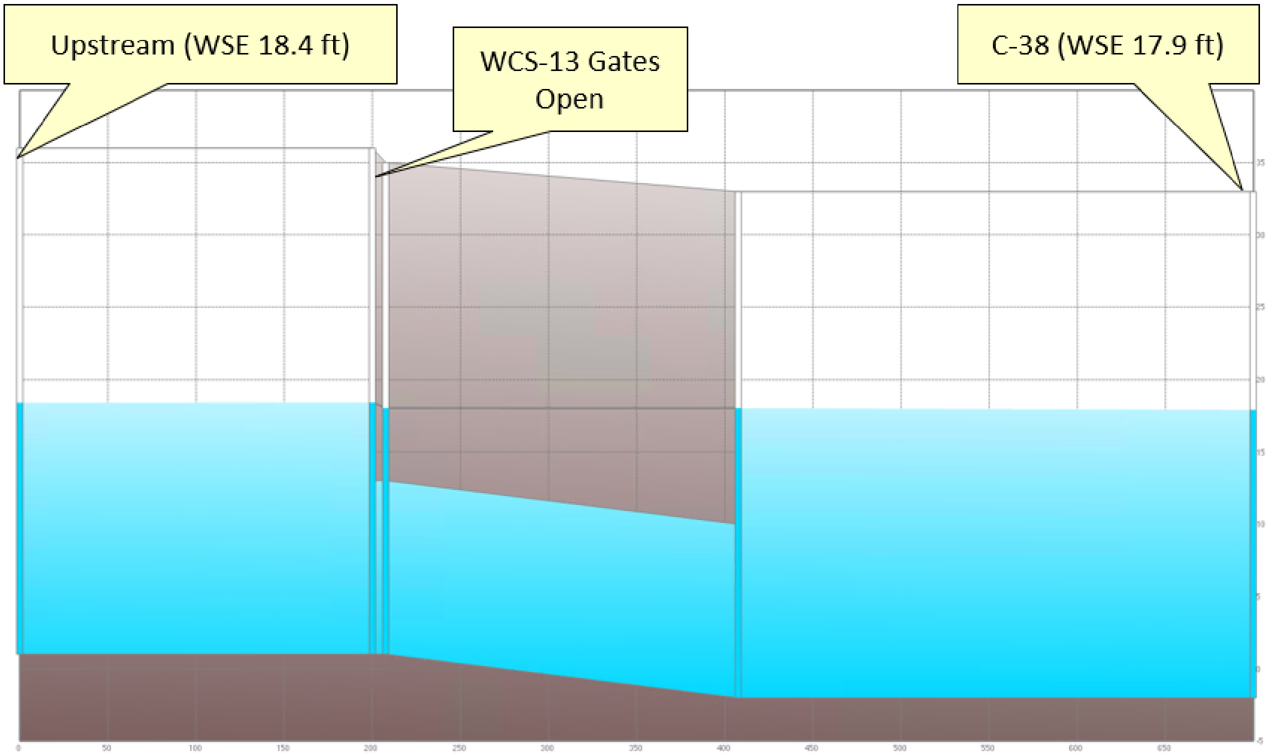

Figure 8-19. WCS-13 profile under design-flow conditions (1,000 cfs)

An additional simulation was performed to assess the impact of routing the Standard Project Flood condition. Under this extreme high-flow condition, 3333 cfs, the flow is characterized as controlled submerged for both WCS-9 and WCS-13. The expected WSE at G-80 generated by WCS-9 is lower than the WSE from the existing S154 structure model (Figure 8-20). Figure 8-21 shows the profile for WCS-13 under the Standard Project Flood condition.

A headwater-tailwater chart will be provided for the preliminary design phase.

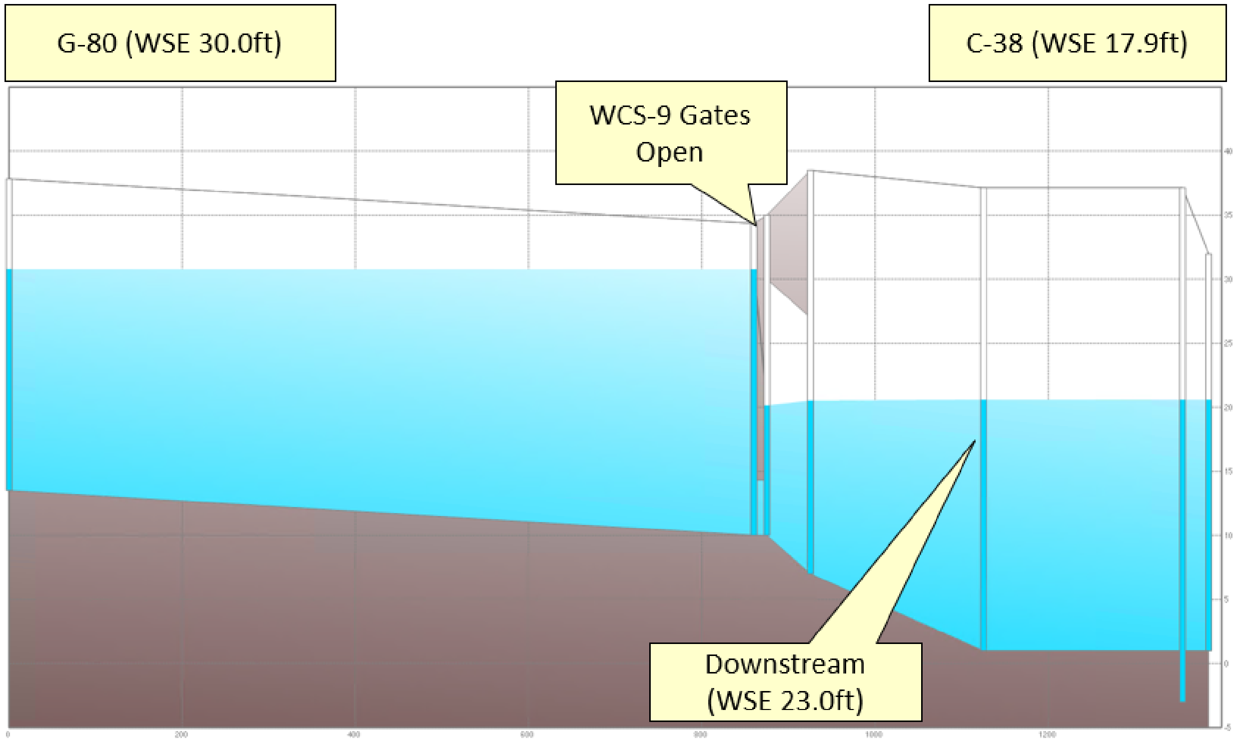

Figure 8-20. WCS-9 profile under Standard Project Flood flow conditions (3,333 cfs)

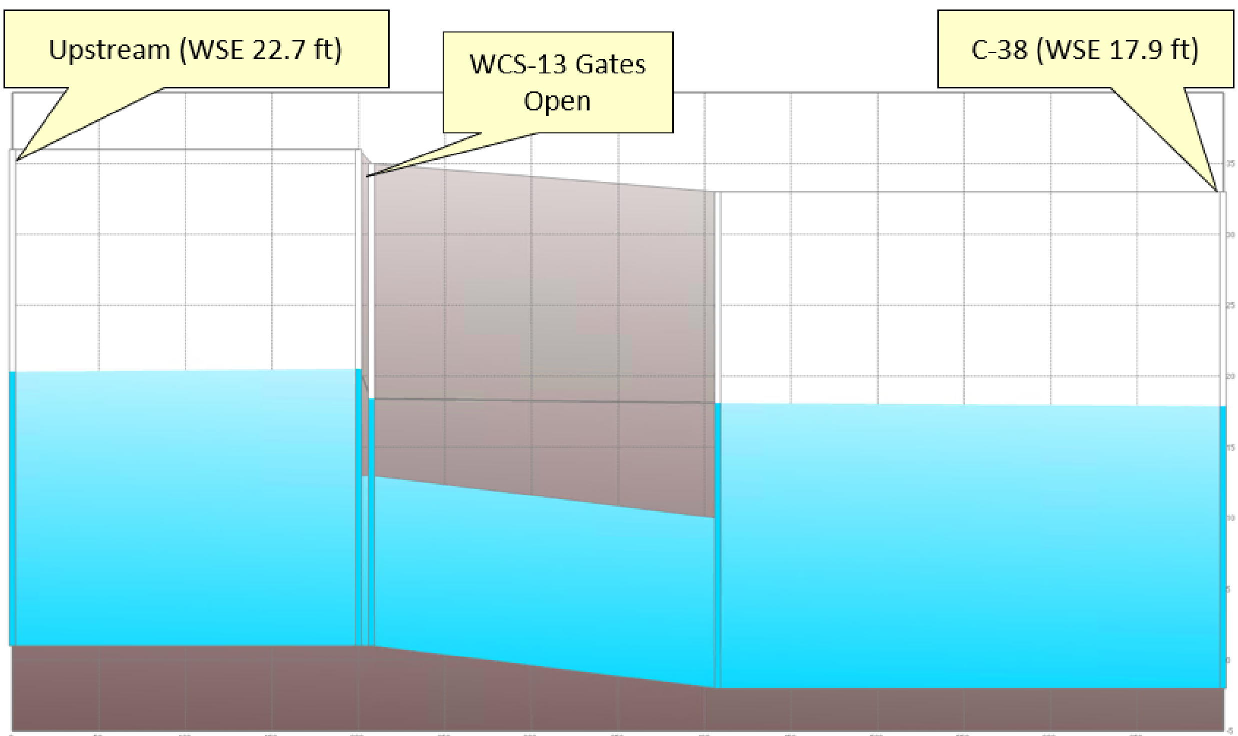

Figure 8-21. WCS-13 profile under Standard Project Flood flow conditions (3,333 cfs)

8.3.1.2 L-62 Canal to PS

Under steady-state conditions, a constant flow of 500 cfs or 125 cfs will be drawn from the realigned section of the L-62 canal under the dry weather flow scenarios. A conservative approach of zero flow was imposed at these pump links when modeling wet weather flow scenarios.

8.3.1.3 L-62 Canal to C-38 Canal

The realigned reach of the L-62 canal is expected to provide bidirectional flow to a) convey the flow from the S-154 basin downstream to C-38 canal under high-flow conditions and b) to maintain the vegetation under dry weather conditions by conveying flow from C-38 canal upstream to the PS. The reroute of the L-62 canal is sized to limit velocities to 2.0 fps under high-flow conditions and a minimum depth of 8 ft under low-flow conditions. Thus, a sensitivity analysis was performed changing a) the bottom elevation of the canal and b) the bottom width of the canal while maintaining the banks at a 3:1 slope. The resulting L-62 canal has a flat bottom elevation of 1 ft (NAVD) with a bottom width of 15 ft. During high-flow conditions, the velocities in the new section of the canal did not exceed the 2.0 fps criterion. Table 8-6 tabulates the hydraulic parameters associated with the realigned reach of the L-62 canal under wet weather conditions.

Table 8-6. Hydraulic Parameters for Re-routing L-62 Canal Under Wet Weather Conditions

Flow Condition	Flow (cfs)	Tailwater at C-38 (ft)	Manning's Roughness	Depth[1] (ft)	Velocity (fps)
High Flow	1,150	12.1	0.03	12.1	1.8
Design Flow	1,000	17.9	0.03	17.5	0.9

1: Depth and velocity near the vicinity of the PS.

For the high flow with the proposed canal dimensions, the water depth near the PS is 12.1 ft with a velocity of 1.8 fps. The velocity is lower than the 2.0 fps criterion. This is for a tailwater elevation of 12.1 ft (NAVD) at the C-38 canal. Figure 8-22 illustrates the channel profile for the high flow condition.

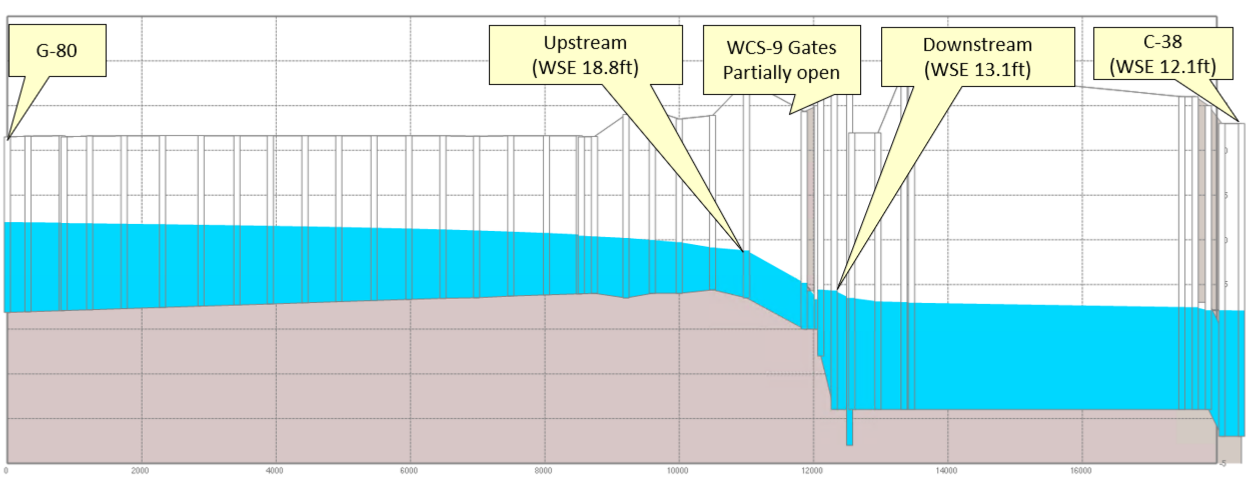

Figure 8-22. L-62 Canal to C-38 Canal profile under high-flow conditions (1,150 cfs)

Figure 8-23 depicts the canal profile under the design-flow conditions from Table 8-6. Given that the tailwater elevation at the C-38 canal is higher, the 0.9 fps velocity near the PS is lower than the high flow condition results.

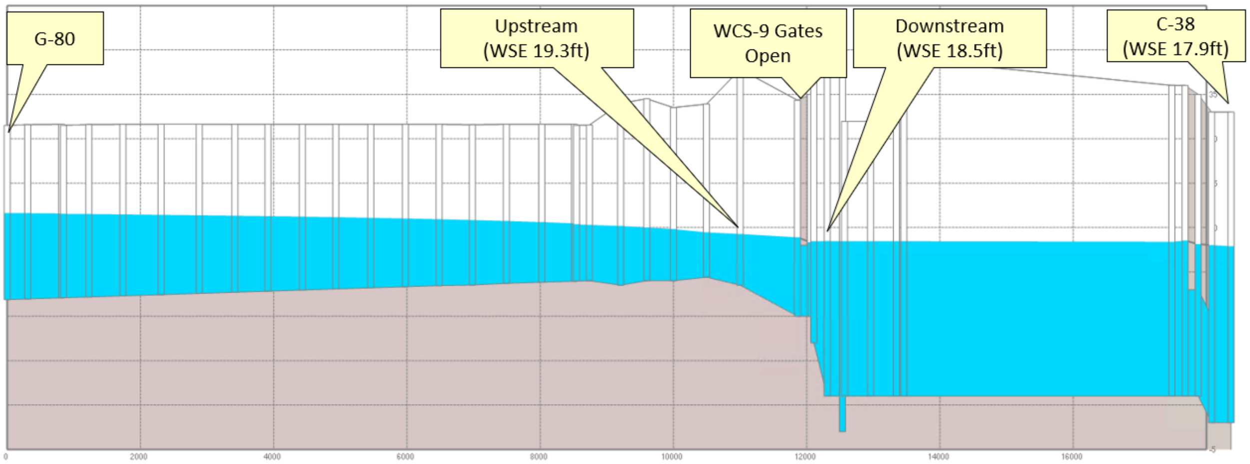

Figure 8-23. L-62 Canal to C-38 Canal profile under design-flow conditions (1,000 cfs)

Table 8-7 tabulates the hydraulic parameters associated with the realigned reach of the L-62 canal under dry weather conditions. The minimum depths attained under the proposed dry weather conditions did not decrease the minimum 8-ft depth criterion. Under dry weather conditions, the gates are closed and the canal is sized to convey the required flows of 500 cfs and 125 cfs for normal and survivability operations, respectively.

To size the canal, the typical Manning's roughness coefficient of 0.03 was applied. A second scenario was modeled to represent vegetation within the canal, using a roughness of 0.05, which is typically used for the overbank.

Table 8-7. Hydraulic Parameters for Re-routing L-62 Canal Under Dry Weather Conditions

Flow Condition	Flow (cfs)	Tailwater at C-38 Canal (ft)	Manning's Roughness	Depth[1] (ft)	Velocity (fps)
High Flow	500	10.7	0.03	9.3	1.2
High Flow / Low Stage	500	9.5	0.03	7.7	1.7
Survivability Flow	125	9.5	0.03	8.5	0.4
High Flow	500	10.7	0.05	8.5	1.4
Survivability Flow	125	9.5	0.05	8.4	0.4

[1]*Depth in the vicinity of the PS*

Figure 8-24 depicts the L-62 canal WSE profile when taking 500 cfs from C-38 canal, with a tailwater elevation of 10.7 ft (NAVD) at the C-38 canal, and a roughness of 0.05. Under this condition, the velocity near the PS is 1.4 fps and the water depth is 8.5 ft, which meets the canal minimum depth criterion of 8-ft.

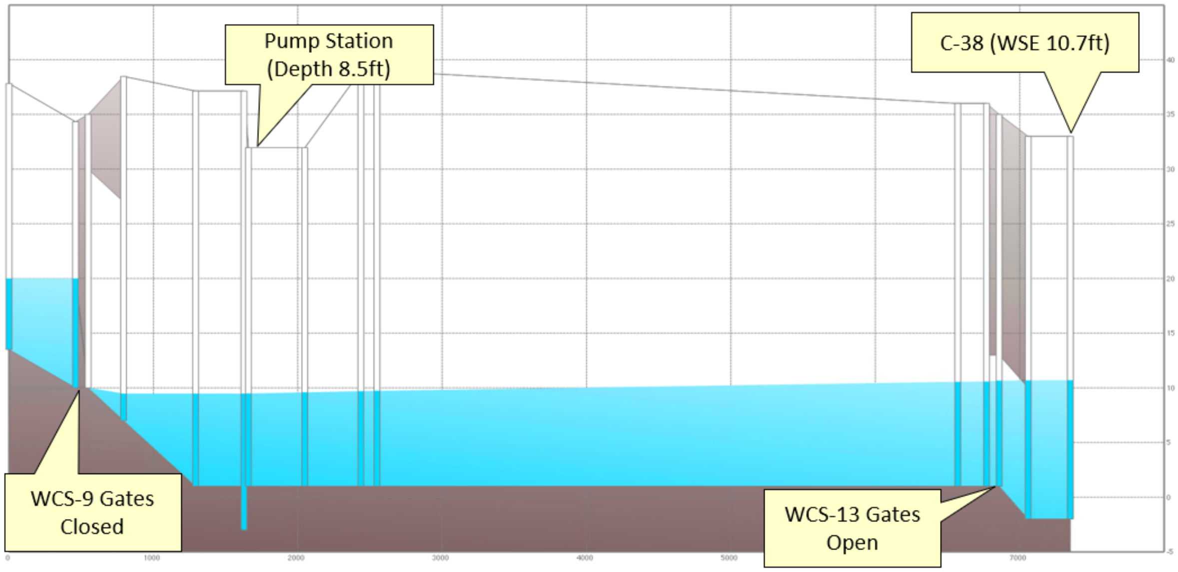

Figure 8-24. L-62 Canal to C-38 Canal profile under high-flow dry weather conditions [Q=500 cfs, n=0.05]

An extreme scenario was also assessed that pulled 500 cfs from the C-38 canal at a tailwater elevation of 9.5 ft. (NAVD) Based on the proposed re-routed L-62 canal dimensions, the system can convey 500 cfs with a roughness of 0.03; however, the minimum depth near the intake of the PS is approximately 7.7 ft. This is slightly lower than the 8-ft-minimum depth requirement. At 400 cfs PS withdraw, the depth near the PS intake is approximately 8.0 ft, at the same C-38 canal tailwater elevation of 9.5 ft (NAVD).

Figure 8-25 depicts the canal WSE when only one pump is in operation pulling 125 cfs from C-38 canal (9.5-ft tailwater condition) with a roughness of 0.05. Under this condition, the velocity near the PS is 0.4 fps and the water depth is 8.4 ft, which exceeds the minimum 8-ft depth criterion.

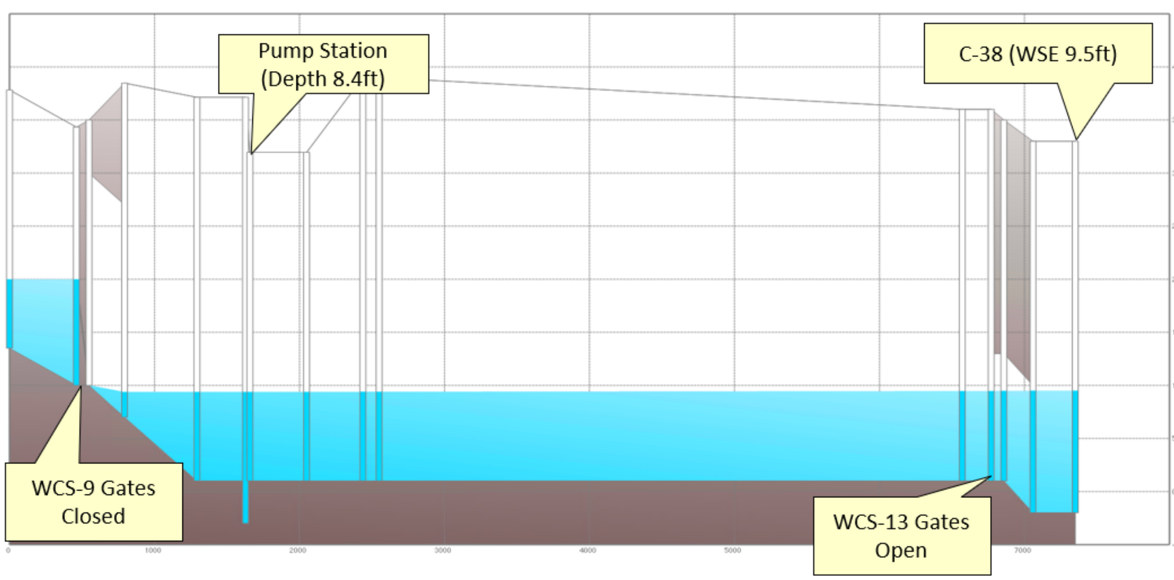

Figure 8-25. L-62 Canal to C-38 Canal profile under survivability flow conditions under dry weather conditions [Q=125 cfs, n=0.05]

It is anticipated that additional analyses for both WCS-9 and WCS-13 structures will be performed during preliminary design.

8.3.2 STA Cells

The system consists of six STA cells—four treatment cells to the west of the re-routed L-62 canal, and two treatment cells east of the canal. Each cell includes gated WCSs at both the upstream and downstream ends, a spreader canal, a collection canal, and the treatment cell itself, which is graded flat.

8.3.2.1 Model Scenarios

To minimize model run-time, six models were created in which each STA cell being evaluated was modeled in 2D while the remaining five cells were modeled with 1D conduits. Flows were entered into the models at the PS discharge node using a time series that began with no flow and ramped up to a steady state of 500 cfs over the course of a day and a half. After all model nodes and conduits were filled and achieved a steady state, the WSE at the PS discharge node was around 31.5 ft.

8.3.2.2 Inflow Canals

The system has two inflow canals, one west of the PS that feeds STA Cells 1 through 4, and one east of the PS that feeds Cells 5 and 6. The east inflow canal includes an elevated conduit crossing of the L-62 canal.

8.3.2.2.1 Inflow Canal from PS to STA Cells 1 to 4

The inflow canal from the PS to STA Cells 1 to 4 is approximately 5,600 ft long and is modeled as a trapezoidal channel (see Section 8.3.2.1). Table 8-8 summarizes the inflow canal model results for flow, velocity, and WSEs under normal operating conditions.

Table 8-8. Inflow Canal from PS to STA Cells 1-4 Model Results Summary

	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
PS to STA Cells 1 to 4	5,600	22	21.5	314	0.8	31.8	31.7

8.3.2.2.2 Inflow Canal Elevated Conduit

The inflow canal elevated crossing over the L-62 canal for STA cells 5 and 6 was initially modeled with dimensions of 7 ft wide by 10 ft high. On the upstream face of the conduit is a weir that is 1.5 ft above the bottom of the conduit. The conduit has an upstream invert elevation of 26.5 ft (NAVD), a downstream invert elevation of 25.5 ft (NAVD), and drops into the inflow canal with a node invert elevation of 17 ft. Through modeling, the conduit was determined to have a maximum water depth of less than 4 ft. The conduit was subsequently sized as a 7 ft x 7 ft box conduit, which will provide more-than-adequate capacity for the operational flow. A gate, stop logs, or needle beams may be incorporated with the 1.5-ft-high weir to provide additional flow control. This will be considered during preliminary design. The elevated conduit was shown previously on Figure 8-2. Table 8-9 summarizes the modeling results for the elevated conduit.

Table 8-9. Inflow Canal Elevated Conduit Model Results Summary								
Size	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Design Q (cfs)	Model Q[a] (cfs)	Velocity (fps)	US WSE[a] (ft)	DS WSE[a] (ft)
1.5' H x 7' W Weir	2	28.0		186	186	7.6	31.8	30.3
7'W x 7'H Conduit	650	26.5	25.5	186	186	6	30.3	27.8

[a]*From Cell 5 2D Model*

8.3.2.2.3 Inflow Canal from Elevated Conduit to Cell 6 and PES

From the elevated conduit to STA Cell 6 and the PES the inflow canal is approximately 8,900 ft long and is modeled as a trapezoidal channel (see Section 8.3.2.1). Table 8-10 summarizes the inflow canal model results for flow, velocity, and WSEs.

Table 8-10. Inflow Canal to Cell 6/PES Model Results Summary							
	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Q (cfs)	Velocity (fps)	US WSE[a] (ft)	DS WSE[b] (ft)
Elevated Conduit to STA Cell 6 / PES	8,900	22.0	13.0	186	0.2-0.4	27.8	23.9

[a]*From Cell 5 2D Model*

[b]*From Cell 6 2D Model*

The downstream WSE at the Cell 6 and PES inflow WCS listed above is slightly different than used in the independent PES model. During preliminary design, the two modeling efforts will be refined (and potentially combined) and will use the same values.

8.3.2.3 STA Cell 1

The following sub-sections present model input and results for simulating STA Cell 1 under normal operations.

8.3.2.3.1 Inflow WCS

The Cell 1 inflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-11 summarizes information for the Cell 1 inflow WCS.

Table 8-11. Cell 1 Inflow WCS Model Results Summary								
	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.5-ft-diameter US Pipe	50	24.5	24.5	N/A	92.0	4.8	31.9	31.5
3.5 x 3.5 ft Gate	N/A	24.5	24.5	2.9	92.0	N/A	31.5	30.8
3.5-ft-diamater DS Pipe	50	24.5	24.0	N/A	92.0	4.8	30.8	30.2

8.3.2.3.2 Spreader Canal

The spreader canal was modeled as described in Section 8.3.2.3 with 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the spreader canal are:

- Flow Velocity: 0.006 fps to 0.098 fps

- WSEs: 30.14 ft to 30.18 ft

8.3.2.3.3 Treatment Cell

The treatment cell was modeled as described in Section 8.3.2.3 with 1,000-ft 2D model mesh resolution. Velocities and WSEs across the treatment cell were estimated as presented in Table 8-12.

Table 8-12. Cell 1 Treatment Cell Model Results Summary

Hydraulic Loading Rate (cm/day)	Upstream Flow Velocity Range (fps)	Downstream Flow Velocity Range (fps)	Upstream Water Depth (ft)	Downstream Water Depth (ft)	Cell Residence Time (days)
10.8	0.008-0.034	0.006-0.019	1.6	1.5	3.9

8.3.2.3.4 Collection Canal

The collection canal was modeled as described in Section 8.3.2.3 with 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the collection canal are:

- Flow Velocity: 0.003 fps to 0.109 fps
- WSEs: 30.07 ft to 30.13 ft

8.3.2.3.5 Outflow WCS

The Cell 1 outflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-13 summarizes information for the Cell 1 outflow WCS.

Table 8-13. Cell 1 Outflow WCS Model Results Summary

	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.5-ft-diameter US Pipe	50	27.5	24	N/A	92.0	5.2	30.1	29.7
3.5 x 3.5 ft Gate	N/A	24	24	1.35	92.0	N/A	29.7	26.2
3.5-ft-diameter DS Pipe	50	24	23.5	N/A	92.0	9.1	26.2	24.6

8.3.2.4 STA Cell 2

The following sub-sections present model input and results for simulating STA Cell 2 under normal operations.

8.3.2.4.1 Inflow WCS

The Cell 2 inflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-14 summarizes information for the Cell 2 inflow WCS.

Table 8-14. Cell 2 Inflow WCS Model Results Summary								
	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.5-ft-diamater US Pipe	50	24.25	24.25	N/A	76.3	4.0	31.7	31.4
3.5 x 3.5 ft Gate	N/A	24.25	24.25	2.05	76.3	N/A	31.4	30.4
3.5-ft-diamater DS Pipe	50	23.75	23.25	N/A	76.3	4.0	30.4	30.0

8.3.2.4.2 Spreader Canal

The spreader canal was modeled as described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSE across the spreader canal are:

- Flow Velocity: 0.007 fps to 0.031 fps
- WSEs: 29.94 ft to 29.99 ft

8.3.2.4.3 Treatment Cell

The treatment cell was modeled as described in Section 8.3.2.3 with the 1,000-ft 2D model mesh resolution. Estimated velocities and WSEs across the treatment cell are presented in Table 8-15.

Table 8-15. Cell 2 Treatment Cell Model Results Summary					
Hydraulic Loading Rate (cm/day)	US Flow Velocity Range (fps)	DS Flow Velocity Range (fps)	US Water Depth (ft)	DS Water Depth (ft)	Cell Residence Time (days)
11.4	0.006-0.030	0.008-0.034	1.7	1.7	3.3

8.3.2.4.4 Collection Canal

The collection canal was modeled as described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the collection canal are:

- Flow Velocity: 0.003 fps to 0.136 fps
- WSEs: 29.9 to 29.95

8.3.2.4.5 Outflow WCS

The Cell 2 outflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-16 summarizes information for the Cell 2 outflow WCS.

Table 8-16. Cell 2 Outflow WCS Model Results Summary								
	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.5-ft-diameter US Pipe	50	26.75	23.75	N/A	76.3	4.0	29.9	29.7
3.5 x 3.5 ft Gate	N/A	23.75	23.75	2.05	76.3	N/A	29.7	25.7
3.-ft-diamater DS Pipe	50	23.75	23.25	N/A	76.3	8.7	25.7	23.6

8.3.2.5 STA Cell 3

The following sub-sections present model input and results for simulating STA Cell 3 under normal operations.

8.3.2.5.1 Inflow WCS

The Cell 3 inflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-17 summarizes information for the Cell 3 inflow WCS.

Table 8-17. Cell 3 Inflow WCS Model Results Summary

	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.0-ft-diameter US Pipe	50	24.55	24.55	N/A	58.5	5.96	31.7	31.1
3.0 x 3.0 ft Gate	N/A	24.55	24.55	0.75	58.5	N/A	31.1	24.1
3.0-ft-diameter DS Pipe	50	24.55	17	N/A	58.5	6.99	24.1	23.2

8.3.2.5.2 Spreader Canal

The spreader canal was modeled was described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the spreader canal are:

- Flow Velocity: 0.0001 fps to 0.051 fps
- WSEs: 23.23 ft to 23.24 ft

8.3.2.5.3 Treatment Cell

The treatment cell was modeled as described in Section 8.3.2.3 with the 1,000-ft 2D model mesh resolution. Estimated velocities and WSEs across the treatment cell are presented in Table 8-18.

Table 8-18. Cell 3 Treatment Cell Model Results Summary

Hydraulic Loading Rate (cm/day)	US Flow Velocity Range (fps)	DS Flow Velocity Range (fps)	US Water Depth (ft)	DS Water Depth (ft)	Cell Residence Time (days)
11.1	0.0003-0.019	0.0003-0.029	1.7	1.6	5.0

8.3.2.5.4 Collection Canal

The collection canal was modeled as described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the collection canal are:

- Flow Velocity: 0.002 fps to 0.089 fps
- WSEs: 23.22 ft to 23.24 ft

8.3.2.5.5 Outflow WCS

The Cell 3 outflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-19 summarizes information for the Cell 3 outflow WCS.

Table 8-19. Cell 3 Outflow WCS Model Results Summary

	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.0-ft-diameter US Pipe	50	17	17	N/A	58.42	3.04	23.11	22.98
3.0 x 3.0 ft Gate	N/A	17	17	1.28	58.42	N/A	22.98	20.72
3.0-ft-diameter DS Pipe	50	17	16.5	N/A	58.42	3.04	20.72	20.55

8.3.2.6 STA Cell 4

The following sub-sections present model input and results for simulating STA Cell 4 under normal operations.

8.3.2.6.1 Inflow WCS

The Cell 4 inflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-20 summarizes information for the Cell 4 inflow WCS.

Table 8-20. Cell 4 Inflow WCS Model Results Summary

	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD))	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.5-ft-diamater US Pipe	50	24.65	24.65	N/A	80.82	4.2	31.7	31.4
3.5 x 3.5 ft Gate	N/A	24.65	24.65	1.05	80.82	N/A	31.4	27.0
3.5-ft-diameter DS Pipe	50	24.65	24.15	N/A	80.82	7.9	27.0	23.9

8.3.2.6.2 Spreader Canal

The spreader canal was modeled as described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the spreader canal are:

- Flow Velocity: 0.0008 fps to 0.003 fps
- WSEs: 23.84 ft to 23.89 ft

8.3.2.6.3 Treatment Cell

The treatment cell was modeled as described in Section 8.3.2.3 with the 1,000-ft 2D model mesh resolution. Estimated velocities and WSEs across the treatment cell are presented in Table 8-21.

Table 8-21. Cell 4 Treatment Cell Model Results Summary					
Hydraulic Loading Rate (cm/day)	US Flow Velocity Range(fps)	DS Flow Velocity Range (fps)	US Water Depth (ft)	DS Water Depth (ft)	Cell Residence Time (days)
12.5	0.007-0.044	0.009-0.024	1.6	1.6	2.8

8.3.2.6.4 Collection Canal

The collection canal was modeled as described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the collection canal are:

- Flow Velocity: 0.006 fps to 0.036 fps
- WSEs: 23.80 ft to 23.84 ft

8.3.2.6.5 Outflow WCS

The Cell 4 outflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-22 summarizes information for the Cell 4 outflow WCS.

Table 8-22. Cell 4 Outflow WCS Model Results Summary								
	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.5-ft-diameter US Pipe	50	17.75	17.75	N/A	79.62	4.14	23.80	23.56
3.5 x 3.5 ft Gate	N/A	17.75	17.75	1.30	79.62	N/A	23.56	20.75
3.5-ft-diameter DS Pipe	50	17.75	17.25	N/A	79.62	4.45	20.75	20.40

8.3.2.7 STA Cell 5

The following sub-sections present model input and results for simulating STA Cell 5 under normal operations.

8.3.2.7.1 Inflow WCS

The Cell 5 inflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-23 summarizes information for the Cell 5 inflow WCS.

Table 8-23. Cell 5 Inflow WCS Model Results Summary								
	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.5-ft-diameter US Pipe	50	19.25	19.25	N/A	103	5.35	27.80	27.42
3.5 x 3.5 ft Gate	N/A	19.25	19.25	2.25	103	N/A	27.42	25.84
3.5-ft-diameter DS Pipe	50	19.25	18.38	N/A	103	5.35	25.84	25.1

8.3.2.7.2 Spreader Canal

The spreader canal was modeled as described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the spreader canal are:

- Flow Velocity: 0.003 fps to 0.040 fps
- WSEs: 25.07 ft to 25.09 ft

8.3.2.7.3 Treatment Cell

The treatment cell was modeled as described in Section 8.3.2.3 with the 1,000-ft 2D model mesh resolution. Estimated velocities and WSEs across the treatment cell are presented in Table 8-24.

Table 8-24. Cell 5 Treatment Cell Model Results Summary

Hydraulic Loading Rate (cm/day)	UP Flow Velocity Range (fps)	DS Flow Velocity Range (fps)	US Water Depth (ft)	DS Water Depth (ft)	Cell Residence Time (days)
12.5	0.007-0.033	0.004-0.028	1.8	1.8	4.0

8.3.2.7.4 Collection Canal

The collection canal was modeled as described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the collection canal are:

- Flow Velocity: 0.0009 fps to 0.016 fps
- WSEs: 25.04 ft to 25.06 ft

8.3.2.7.5 Outflow WCS

The Cell 5 outflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-25 summarizes information for the Cell 5 outflow WCS.

Table 8-25. Cell 5 Outflow WCS Model Results Summary

	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.5-ft-diameter US Pipe	50	18.25	18.75	N/A	102.98	5.35	25.04	24.64
3.5 x 3.5 ft Gate	N/A	18.75	18.75	1.5	102.98	N/A	24.64	21.11
3.5-ft-diameter DS Pipe	50	18.75	18.25	N/A	102.98	9.39	21.11	16.94

8.3.2.8 STA Cell 6

The following sub-sections present model input and results for simulating STA Cell 6 under normal operations.

8.3.2.8.1 Inflow WCS

The Cell 6 inflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-26 summarizes information for the Cell 6 inflow WCS.

Table 8-26. Cell 6 Inflow WCS Model Results Summary								
	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.0-ft-diameter US Pipe	50	15	15	N/A	64	4.52	23.86	23.56
3.0 x 3.0 ft Gate	N/A	15	15	1.1	64	N/A	23.56	20.11
3.0-ft-diameter DS Pipe	50	15	14.25	N/A	64	4.52	20.11	19.56

8.3.2.8.2 Spreader Canal

The spreader canal was modeled as described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the spreader canal are:

- Flow Velocity: 0.002 fps to 0.063 fps
- WSEs: 19.55 ft to 19.56 ft

8.3.2.8.3 Treatment Cell

The treatment cell was modeled as described in Section 8.3.2.3 with the 1,000-ft 2D model mesh resolution. Estimated velocities and WSEs across the treatment cell are presented in Table. 8-27.

Table 8-27. Cell 6 Treatment Cell Model Results Summary					
Hydraulic Loading Rate (cm/day)	US Flow Velocity Range (fps)	SD Flow Velocity Range (fps)	US Water Depth (ft)	DS Water Depth (ft)	Cell Residence Time (days)
9.5	0.005-0.012	0.002-0.019	1.8	1.8	3.9

8.3.2.8.4 Collection Canal

The collection canal was modeled as described in Section 8.3.2.3 with the 25-ft 2D model mesh resolution. Estimated velocities and WSEs across the collection canal are:

- Flow Velocity: 0.001 fps to 0.031 fps
- WSEs: 19.54 ft to 19.55 ft

8.3.2.8.5 Outflow WCS

The Cell 6 outflow WCS will have 3.5-ft-diameter piping and a slide gate measuring 3.5 ft x 3.5 ft. Table 8-28 summarizes information for the Cell 6 outflow WCS.

Table 8-28. Cell 6 Outflow WCS Model Results Summary								
	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Gate Vertical Opening (ft)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
3.0-ft-diameter US Pipe	50	13.25	13.25	N/A	64.06	4.53	19.54	19.24
3.0 x 3.0 ft Gate	N/A	13.25	13.25	1.05	64.06	N/A	19.24	15.44

3.0-ft-diameter DS Pipe	50	13.25	12.75	N/A	64.06	5.99	15.44	14.80

8.3.2.9 Outflow Canal

The system has two outflow canals, one from the Cell 1 outflow WCS to the S154 discharge structure (Outfall Canal 1), and a second from the Cell 6 outflow WCS to the S154 discharge structure (Outfall Canal 2).

Outflow Canal 1 is approximately 25,000 ft long and is modeled as a trapezoidal channel (see Section 8.2.2.4). The canal collects the outflow from treatment cells 1, 2, 3, 4, and 5. In addition, it includes an inverted siphon to convey flows below the L-62 canal.

Outflow Canal 2 is approximately 5,700 ft long and is modeled as a trapezoidal channel (see Section 8.3.2.4). The canal collects the outflow from Cell 6 and the PES which is conveyed through an open S154 outlet structure to the C-38 canal.

Tables 8-29 and 8-30 present a summary of the outflow canal model results. Table 8-31 presents a summary of the model results for the S154 outlet structure under normal operating conditions with a typical tailwater elevation in the C-38 canal of 14.0 ft (NAVD).

Table 8-29. Outflow Canal 1 Model Results Summary[a]

	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
Cell 1 Outflow WCS to Cell 2 Outflow WCS STA	5,100	20.5	20.5	95	1.2	24.7	23.6[b]
Cell 2 Outflow WCS to Cell 3 Outflow WCS STA	11,300	20.5	12.4	175	0.9 – 1.5	23.6[b]	20.6
Cell 3 Outflow WCS to Cell 4 Outflow WCS STA	4,200	12.4	12.0	233	1.1	20.6	20.4
Cell 4 Outflow WCS to Inverted Siphon	500	12.0	11.8	313	1.6	20.4	20.4
Inverted Siphon	900	11.8 / -5.0	-5.0 / 11.0	313	4.9	20.4	17.0
Inverted Siphon to Cell 5 Outflow WCS STA	380	11.0	10.8	314	1.8	17.0	16.9
Cell 5 Outflow WCS to S154	3,000	10.8	10.0	417	3.1	16.7	14.3

[a]*From Cell 3 2D Model*

[b]*Water depth is less than 6 ft. Grade controls or other features will be considered during preliminary design to increase water depth.*

Table 8-30. Outflow Canal 2 Model Results Summary[a]

	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
Cell 6 Outflow WCS to PES	4,700	10.5	10.2	64	0.9	14.8	14.5
PES to S154	1,000	10.2	10.0	83	1.1	14.5	14.3

[a]*From Cell 3 2D Model*

Table 8-31. S154 Outflow Summary[a]							
	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
Dual 8 ft x10 ft Pipes @ S154	120	4	4	500	3.1	14.2	14.0

[a]*From Cell 3 2D Model*

8.3.2.10 Seepage Canals

The following sub-sections present model input and results for simulating the Project seepage canals.

8.3.2.10.1 Seepages Canals West of L-62 Canal

Following the methodologies presented in Section 8.3, the peak flows and peak water elevations were determined for existing conditions at the existing SR 70 cross-culverts 1 and 2. Table 8-32 summarizes model results at the culverts for the 100-year/3-day storm.

Table 8-32. SR70 Cross Culverts Model Results Summary (100-year/3-day storm)		
	Peak Flow (cfs)	Peak WSE @ Culvert Crossing (ft)
Culvert 1	30	27.7
Culvert 2	620	29.0

Under proposed conditions, flows from these culvert crossings will be collected by the seepage canal and flow west to east along the north site limit to the northeast corner of the site, and then south along the eastern site limit. For the 100-year/3-day storm, modeling indicates the seepage canal water depth will increase by 4.5 ft to 5.5 ft. The canal will be sized in preliminary design to provide adequate freeboard. Results of modeling the 100-year/3-day storm are presented in Table 8-33.

Table 8-33. Seepage Canal West of L-62 Modeling of Offsite Flows (100-year/3-day storm)								
ID	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	NWSE (ft)	Design Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
North Channel (Culvert 1 to NE Site Corner)	1,300	14.0	14.0	20.0	780	1.5	26.2	26.1
East Channel	5,100	14.0	13.0	20.0	780	1.5	26.1	25.6

8.3.2.10.2 Seepages Canals East of L-62 Canal

The runoff from offsite drainage areas east of the L-62 canal will be collected by the seepage canal running around the north, east, and south sides of the Project as described in Section 8.3. Preliminary modeling indicates this seepage canal will have a 10 ft bottom width and 3:1 side slopes. For the 100-year/3-day storm, the canal water depths will range from 2 ft to 6 ft. Results of routing the 100-year/3-day storm through this seepage canal are presented in Table 8-34.

Table 8-34. Seepage Canal East of L-62 Canal Modeling of Offsite Flows (100-year/3-day storm)							
ID	Length (ft)	US Bottom Elevation (ft-NAVD)	DS Bottom Elevation (ft-NAVD)	Design Q (cfs)	Velocity (fps)	US WSE (ft)	DS WSE (ft)
Channel J1 to J2	7,500	22.0	16.0	73	0.8	24.5	22.6
Channel J2 to J3	11,000	16	14	220	1.9	22.6	16.8

As a result of the construction of Cells 5 and 6, a significant portion of the drainage area contributing to the existing drainage ditch (ultimately runs south offsite and parallel to the HHD) will now be captured by these treatment cells and routed to the outflow canal. A summary of pre- and post-project hydrologic modeling results for the area draining to this ditch is presented in Section 5.

8.3.3 Hazard Potential Classification

Based on site conditions in and around the vicinity of the LKBSTA Project and evaluation of the proposed LKBSTA design as described in DCM-1, the potential impacts and hazards associated with the LKBSTA Project reasonably support its classification as a low hazard facility. This is supported by the following:

- STA embankment designs will be designed in accordance with the requirements of DCM-2, DCM-4 and other applicable SFWMD and USACE design standards.
- The height of the STA embankments will be relatively low, typically 5 ft to 7 ft above existing grade.
- The nearest town is the City of Okeechobee located 8 miles east of the STA. The STA is located in a rural area surrounded by land in agricultural production. The nearest permanent structures where human activity occurs on a regular basis are houses located on the west, north and east sides of the Project area. The lowest ground elevation of these properties is 29.7 ft (NAVD), which is slightly below the estimated MWSL of 31.0 ft in the STA; however, the outflow canal in the western boundary of the property is expected to convey any water volume from a potential breach. Therefore, direct loss of life from a breach of the embankment along the west, north, or east side of the STA would not be expected. Water discharged from a breach of a perimeter embankment on the north, east or west sides of the STA are not expected to impact structures located in the vicinity of the Project area.
- Dam-break analyses will be completed for the northern, eastern, and western boundaries of the Project as part of the preliminary design.
- The STA will be surrounded by canals on all four sides. Critical peak flow rates resulting from an embankment failure will be estimated from the dam-break analysis. The canals will be sized to convey these flows to the S154 outlet without overflow being discharged offsite; therefore, the probability of a discharge onto adjacent property is low. The intent of sizing canals for these flow rates is maintaining a low HPC.
- Because the STA will be surrounded by canals on all four sides, all with embankment elevations several feet higher than the expected MWSL in the STA, the potential for lifeline, economic, and/or environmental losses from a breach of an embankment around the perimeter of the STA is low.

Future design efforts, including wind setup and wave run-up calculations to determine embankment freeboard requirements, are expected to be based on the LKBSTA being classified as a low hazard facility.

8.3.4 Wind Setup/Wave Run-Up and Freeboard

Results of the Wind Setup/Wave Run-Up and Freeboard analyses presented in this section are based on the output included in Appendix 4. Analyses were performed on Cells 1 through 6, and a supercell (Cells 1 and 2 combined) to account for the longest fetch and the potential for a higher wind setup when Cells 1 and 2 are combined. The maximum storage depth (MSD) of 2.77 ft was used which includes the LKBSTA anticipated normal full storage water depth during typical operation (2.0 ft) and the design rainfall from a 100-year, 24-hour storm event (9.2 inches, 0.77 ft). Table 8-35 summarizes the depths of each of the assessed cells when MSD equals 2.77 ft.

Table 8-35. Summary of LKBSTA Maximum Storage Depth Conditions

Conditions	Cell 1	Cell 2	Cell 3	Cell 4	Cell 5	Cell 6	Supercell
Normal Full Storage Depth (ft)	2.00	2.00	2.00	2.00	2.00	2.00	2.00
Design Rainfall from 100-year, 24-hour storm event (ft)	0.77	0.77	0.77	0.77	0.77	0.77	0.77
Maximum Storage Depth (ft)	2.77	2.77	2.77	2.77	2.77	2.77	2.77

Table 8-36 summarizes the wave run-up and wind setup calculations performed at the maximum storage depth for Cells 1 through 6 and the supercell. Table 8-37 contains the minimum top of levee elevations recommended for Cells 1 to 6 based on the maximum water depth. Average ground surface elevations varied from 17.75 ft (NAVD) to 28.50 ft (NAVD)depending on the cell (refer to Appendix 4).

Table 8-36. Summary of Wind Setup and Wave Run-up Calculations at MSD of 2.77 ft

Conditions	Cell 1	Cell 2	Cell 3	Cell 4	Cell 5	Cell 6	Supercell 1 & 2
Maximum Storage Depth (ft)	2.77	2.77	2.77	2.77	2.77	2.77	2.77
Wind Speed (mph)	60	60	60	60	60	60	60
Manning's roughness coefficient, n	0.32	0.32	0.32	0.32	0.32	0.32	0.32
Wave Effective Fetch (ft)	5,787	4,846	3,363	4,593	5,203	4,127	7,303
Wave Height (ft)	0.29	0.30	0.32	0.30	0.29	0.30	0.28
Wave Height (ft)	0.30	0.31	0.33	0.31	0.30	0.31	0.29
Wave Period (seconds)	2.1	2.0	1.8	2.0	2.0	1.9	2.3
Wave Run-up (ft)	0.70	0.71	0.73	0.71	0.70	0.72	0.69
Wind Setup Total Fetch (ft)	7,458	6,678	6,863	7,052	6,479	6,926	10,960
Wind Setup (ft)	0.89	0.81	0.73	0.85	0.78	0.84	1.28
Wind Setup + Wave Run-up (ft)	1.59	1.51	1.56	1.56	1.48	1.56	1.97
Mass Balance Error for Wind Setup/Total (%)	<1%/<1%	<1%/<1%	<1%/<1%	<1%/<1%	<1%/<1%	<1%/<1%	<1%/<1%

Table 8-37. Summary of Top of Levee Elevations at MSD of 2.77 ft

Conditions	Cell 1	Cell 2	Cell 3	Cell 4	Cell 5	Cell 6	Supercell 1 & 2
Average Ground Elevation (ft-NAVD)	28.50	28.25	21.5	22.25	23.25	17.75	28.25

Maximum Storage Depth (ft)	2.77	2.77	2.77	2.77	2.77	2.77	2.77
Water Surface Elevation (ft-NAVD)	31.27	31.02	24.27	25.02	26.02	20.52	31.02
Wave Run-up + Wind Setup (ft)	1.59	1.51	1.56	1.56	1.48	1.56	1.97
Minimum Freeboard (ft)	3.00	3.00	3.00	3.00	3.00	3.00	3.00
Minimum Top of Levee (ft NAVD) *	34.27	34.02	27.27	28.02	29.02	23.52	34.02

**Minimum top levee elevation is the greater value between the (WSE + wind setup and wave run-up) or the (WSE + minimum freeboard).*

The sum of wave run-up and wind setup ranges from 1.48 ft to 1.97 ft, which correspond to Cell 5 and the supercell. Maximum wave run-up plus wind setup for the six individual cells is 1.59 ft for Cell 1.

The probability of overtopping the levee is less than 2 percent since the 3 ft minimum freeboard is greater than the wind setup plus wave run-up for all cases. Wave run-up was calculated by taking the average of the Ahrens and Heimbaugh result, and the vertical run-up distance exceeded by 2 percent of run-ups ($R_{u2\%}$). If only the $R_{u2\%}$ value was used for the wave run-up calculation, the run-up height would increase by 0.2 ft, at most. If 0.2 ft was added to each wave run-up and wind setup sum, the total would still be less than the 3 ft minimum freeboard for all cells. Therefore, the probability of overtopping the levee is still less than 2 percent.

As shown in Table 8-36, the highest top of levee elevation is Cell 1 at 34.27 ft (NAVD). Although the combined wave run-up and wind setup is less than 3 ft, DCM-2 freeboard requirements for low hazard potential classification storage impoundments are “the greater between the described minimum 3 ft or the combination impact of wind setup plus wave run-up due to a 60-mph 1-hour wind condition with the MWSL”. A minimum freeboard of 3 ft is required in the case of shallow water depth and sustained emergent vegetation. Cell 1 minimum top of levee elevation was selected as the baseline reference.

Additional conditions that could affect levee height were identified for possible further evaluation during preliminary design. Two conditions were selected for consideration and two were not due to low probability as follows:

- Embankment settlement to be determined during preliminary design.
- Road base on top of the levee to be determined during preliminary design.
- Landslide-induced wave potential from the proposed embankments or other features surrounding the LKBSTA is low and not included in the minimum top of levee calculation. It is assumed that the probability for an earthquake or sinkhole (potentially causing a landslide) is low in the proximity of the LKBSTA.
- In the event of a cell discharge structure malfunction, the structure’s second barrel is designed to convey 75% of the design flow. Further consideration is not needed due to the redundancy provided.

A summary of the levee height conditions to be considered during preliminary design is included in Table 8-38.

Table 8-38. Summary of Levee Height Conditions for Preliminary Design	
Conditions	**Value (ft-NAVD)**
Cell 1 Minimum Top of Levee Elevation	34.27
Embankment Settlement	TBD
Road Base	TBD
Minimum Top of Levee Elevation with Provisions	TBD

The freeboard and minimum embankment elevation analyses for Cells 1 through 6 indicate these cells comply with the DCM-2 requirements. This analysis was based on a 1-hour, 60-mph wind from the northeast. The wave characteristics, wind setup heights, and wave run-up were determined under fetch-limited conditions. The numerical methods (Runge-Kutta's method and the trapezoidal rule) were used with stringent tolerance criteria to estimate the wave characteristics and the wind setup and wave run-up heights. The minimum freeboard recommended is 3 ft for each cell, which is greater than the calculated wind setup and wave run-up sums.

The embankment elevations are conservative for the following reasons:

- The supercell wind setup plus wave run-up height is less than 2 ft. and the recommended minimum freeboard above the MSD is 3 ft.
- A low roughness coefficient of 0.32 was used in the calculations.
- Each cell is expected to have a maximum depth of 2.77 ft. Unlike a storage impoundment, the LKBSTA is intended to operate at a water depth lower than the maximum storage depth, which reduces the risk of waves overtopping the levees.

Additional provisions should be included for embankment settlement to properly estimate the maximum top of levee height. Nevertheless, the minimum height for the external embankments should be at least 5.77 ft. The internal berms between Cells 1 and 2 and Cells 3 and 6 should have a minimum height of 5 ft given that the external embankment for the supercell does not exceed the minimum freeboard of 3 ft.

8.3.5 Flood Routing

Analysis methodology and standards for flood routing and spillways will generally follow the guidance from DCM-3. This analysis will be completed in preliminary design.

8.4 PES

8.4.1 Model Results

The hydrology and hydraulic (H&H) modeling objectives for the PES were to analyze and assess how water will be conveyed from inflow gate WCS-7A through the pretreatment flow-way and distribution channel, into the PES cells, through the cells, and subsequent discharge. The PCSWMM model selected was for the PES hydraulic design. Based on the PES future buildout design including 36 ac, Sustainable Water Infrastructure Group (SWIG) used a typical six-cell PES module of six treatment acres. SWIG prepared a technical memo entitled "Technical Memorandum: PES Hydrology and Hydraulics" (H&H TM), which provides modeling results that show the system can treat the inflow volumes required to meet the PES treatment performance. These volumes are based on a flow rate of 3 cfs/ac, or a total flow of 18 cfs for the typical 6-ac module. Refer to Figures 2 and 3 of the H&H TM for more information on the layout of the initial six cells and the PES buildout phasing, attached as Appendix 4 to this DDR. Initial model results will be verified by a pilot system being implemented at the SWIG Research facility in Orange Park, Florida, and potentially at the Project site, over the next few months.

8.5 Proposed Future Activities

8.5.1 Preliminary Design

The following items are anticipated to be completed during preliminary design:

- Modeling of the STA System will be advanced, including a sensitivity analysis of varying applicable input criteria as follows:
  - Low-flow modeling with 125 cfs pumping on a daily 6-hour duty cycle for an average daily flow rate of 30 cfs to the treatment cells
  - Modeling with high and low tailwater conditions, including consideration of Lake Okeechobee storm surge condition
  - Dam-break analysis in support of HPC determination
  - Routing of direct runoff flood flows in STA System modeling
  - Higher condition Manning's roughness coefficient (n)
  - Incorporating a gate, stop logs, or needle beams with the 1.5-ft-high weir for crossing the inflow canal over the L-62 canal
  - Modeling with an elevated conduit along the outflow canal crossing over the L-62 canal. A recirculation of flow, which could be accomplished by pump or gravity from the elevated conduit into the L-62 canal, may also be incorporated into this model. It will be further evaluated in the future,
  - Inclusion of grade control structures or other features in applicable reaches of the outflow canals to achieve minimum canal depths of 6 ft
  - Advancing seepage canal modeling and integration with STA system
  - Combining hydraulic modeling of the STA and PES systems
- Headworks system modeling refinement, including a sensitivity analysis of varying applicable input criteria:
  - Refining structure elevation, weir elevation, and structure size for both WCS 9 and WCS 13
  - Consideration of system conditions with a Lake Okeechobee storm surge condition
  - Energy dissipation for areas immediately downstream (and upstream) of WCSs
- CFD modeling of:
  - WCS-9
  - PS
  - Internal high-head WCS
  - Confluence of L-62 canal with C-38 canal and WCS-13
- Refinement of levee height based on HPC and wind/wave analysis

8.5.2 Final Design

During final design, it is anticipated that minimal modeling will be required. It may include further refinement of the modeling due to final design modifications.

SECTION 9

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SECTION 9

GEOTECHNICAL INVESTIGATION AND DESIGN

This section presents a summary of the Project's preliminary geotechnical investigation and design information. Additional geotechnical information is provided in the memorandums in Appendix 3.

9.1 Applicable Standards and Publications

- Florida Bureau of Geology
- Florida Building Code
- Florida Department of Transportation Design Manuals
- NRCS National Engineering Handbook Part 631 (NRCS, 2012)
- Occupational Safety and Health Administration (OSHA) 29 CFR part 1926 (Subpart P, Excavations)
- Principles of Geotechnical Engineering (Das and Sobhan, 2014)
- Soil Mechanics (Lambe and Whitman, 1969).
- USACE EM 1110-2-2300, General Design and Construction Considerations for Earth and Rock-Fill Dams
- USACE EM 1110-1-1804, Geotechnical Investigations
- USACE EM 1110-2-1901, Seepage Analysis and Control for Dams
- USACE EM 1110-2-1902, Slope Stability
- USACE EM 1110-1-1904, Settlement Analysis
- USACE EM 1110-1-1905, Bearing Capacity of Soils
- USACE EM 1110-2-2100, Stability Analysis of Concrete Structures
- USACE EM 1110-2-2504, Design of Sheet Pile Walls
- USDA NRCS Soil Survey Mapping – Okeechobee County, Florida
- USGS Seismic Hazard Mapping

9.2 Geotechnical Exploration

The EIP team conducted preliminary field investigations for the LKBSTA project to gather information on the site subsurface stratigraphy and associated engineering properties. This information was used to develop and analyze typical design sections of the proposed embankments, canals, structures, and pump stations. The results of the preliminary investigation are documented in the Geotechnical Engineering Memorandum provided in Appendix 3.

The field investigation included:

- Completing 26 standard penetration test (SPT) borings along the proposed embankment/canal alignments to depths ranging from 30 to 50 ft bgs in accordance with ASTM D 1586.
- Installing piezometer clusters at eight locations along the proposed embankment/canal alignment during the investigation. Three to four piezometers were installed at each location at different elevations to facilitate performance of permeability tests of the in situ soil strata using constant head permeability testing procedures.

- Installing 26 piezometers along the proposed alignment of the proposed perimeter embankments. The piezometers were installed such that the screen intervals were placed within the different subsurface strata to measure the in situ horizontal permeability using the constant head test method as described below.
- Laboratory testing of soil samples, including visual soil classification, determination of index properties, modified proctor tests, laboratory permeability tests, and corrosion testing

The preliminary results discussed herein were used to support the Reconnaissance Study and this DDR. A comprehensive geotechnical investigation program will be conducted in general accordance with USACE EM 1110-1-1804 Engineering and Design *Geotechnical Investigations* (USACE, 2001). A Geotechnical Data Report (GDR) and Geotechnical Engineering Design Report (GEDR) will be subsequently prepared for the project.

9.3 Stratigraphy

In general, the project site is underlain by sands and shell sands Unified Soil Classification System (USCS) SP, slightly silty (SP-SM) and silty sands (SM) from the ground surface to the termination depth of the deepest borings at approximately 50 ft bgs. Deeper layers of clay (CL & CH) and silt (MH) were encountered in several of the borings below an elevation of -10 ft NAVD. In addition, a deep layer of limestone was encountered in one of the borings at an elevation of -10 ft NAVD.

Table 9-1. General Soil Descriptions

Stratum No.	General Description (USCS)
1	SHELLY SAND/ SAND WITH SILT/ SILTY SAND (SP/ SP-SM/ SM)
2	SAND WITH CLAY (SP-SC)
3	CLAYEY SAND/ SILTY CLAY (SC/ CL-ML)
4	CLAY (CL/CH)
5	SILT (MH)
6	MODERATELY HARD LIMESTONE
7	SHELLY SAND (SP)

For additional information, review the boring logs and subsurface profiles in the Geotechnical Engineering Memorandum provided in Appendix 3.

9.4 Laboratory Test Results

The EIP team logged soil samples in the field using typical logging procedures to have consistent descriptions of subsurface strata in accordance with the USCS and ASTM D 2488. Representative samples from the borings were tested for index properties to help classify borings for engineering purposes (ASTM D 2487). The following laboratory tests were performed in accordance with the applicable cited ASTM standardized procedures:

- Seventy-six (76) moisture content tests (per ASTM D 2216)
- Forty-one (410 tests for percent of material passing through the No. 200 US Standard Sieve Size (per ASTM D 1140)
- Twenty-four (24) mechanical grain size analyses (per ASTM D 6913)
- Eleven (11) Atterberg limits tests (per ASTM D 4318)

The laboratory test results are included in the Geotechnical Engineering Memorandum provided in Appendix 3.

9.5 Hydro-stratigraphy

The project site slopes downward from north to south with ground elevations on the order of 30 ft NAVD along the northern perimeter of the site to 15 ft NAVD along the southern perimeter of the site, just north of the C-38 Canal HHD. The groundwater is generally within 2 to 6 ft of the ground surface at the site. Groundwater levels will fluctuate with the seasonal precipitation variations. Groundwater levels are also expected to be influenced by nearby canals and the stormwater management systems for the adjacent undeveloped properties in the area. Based on a review of the soil survey (Soil Survey Staff, 2022), the pre-drained seasonal high-water table was at the surface to 24 inches above the surface from June to March. During the remainder of the year, it was typically at the surface to a depth of 12 inches and receded below a depth of 12 inches during extended dry periods.

Eight of the shallow piezometers at the site have been instrumented with water level monitoring equipment to establish pre-construction groundwater conditions. Based on the first couple months of data, local groundwater fluctuations on the order of up 2 ft have been observed after periods of heavy rainfall. Groundwater level fluctuations have typically been less than 1 ft. Groundwater monitoring results will be included in the GDR.

Due to seasonal fluctuations, the seasonal high-water table is assumed at existing grade for this project; however, note that the post-construction groundwater level at the Project will effectively be based on the elevation of the water within the treatment cells.

9.6 Seismicity

Seismic hazard maps produced by USGS (USGS, 2022) display earthquake ground motions for various probability levels. Figure 9-1 indicates the approximate peak ground acceleration (g) for the LKBSTA project of 0.02 g to 0.04 g (for 1 hertz spectral acceleration) with a 2-percent-in-50-years probability of exceedance at the Project site. This value is very low in comparison to seismically active areas in the U.S., and no active faults are identified in the southeastern Florida region.

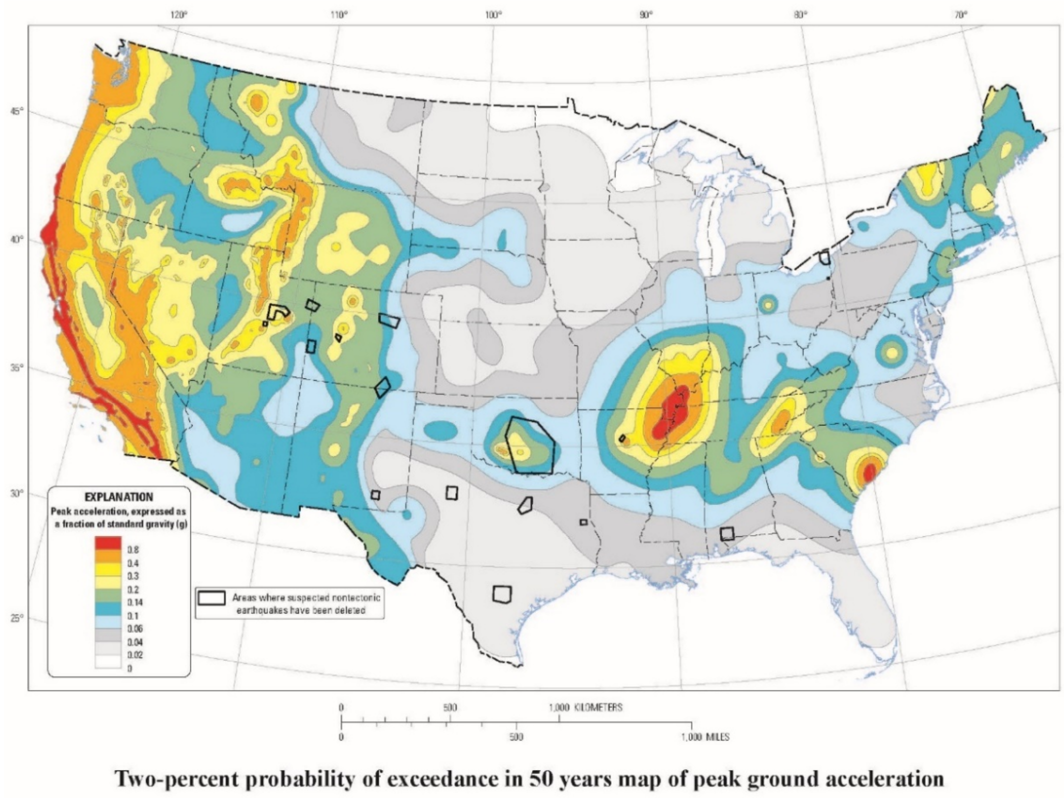

Figure 9-1. Earthquake ground motion map

(USGS, 2022)

In addition to the above, the Florida Bureau of Geology (Lane, 1983) indicates that Martin County, FL is in Zone 0, which is an area with no reasonable expectancy of earthquake damage.

FEMA (FEMA, 2022) and IBC classify areas by seismic design categories (SDC), which reflect the likelihood of experiencing earthquake shaking of various intensities. The southeast Florida area has an SDC ‘A’ classification, which is described as “very small probability of experiencing damaging earthquake effects.”

The risk of an earthquake large enough to cause structural damage occurring in the vicinity of the Project site is considered extremely low.

9.7 Borrow

Borrow for the project construction will primarily come from the excavation of the various inflow, outflow, and seepage collection canals and the re-routed L-62 canal. A diligent attempt will be made to balance cut-and-fill quantities to prevent on-site long-haul trucking of borrow fill.

9.8 Excavations

The soils encountered in the borings generally consist of relatively clean sands (shelly sand/sand with silt [SP and SP-SM]). Occupational Safety and Health Administration (OSHA) 29 CFR part 1926 (Subpart P, Excavations) defines such soils as Type C soils. Any temporary sheeting designs will be provided by the construction contractor and will be signed and sealed by a structural engineer registered in the State of Florida.

Blasting will not be required to facilitate excavations.

In addition to the various inflow, outflow and seepage collection canals, and the re-routed L-62 canal, significant site excavations will be required to construct the PS and WCS's. It is anticipated that structure excavations will be conducted in the dry within dewatered, sheet-piled cofferdam excavations.

9.9 Foundations

Project structures include one PS, an estimated 13 gated WCSs for controlling inflow to and outflow from the STA treatment cells, ungated cast-in-place culvert structures, gated HDPE pipe culverts with cast-in-place or precast concrete headwalls, and related structural features such as precast concrete electrical buildings, generator buildings, gate control buildings, slabs, retaining walls, etc. Foundation materials will be assumed to have an allowable bearing value of 3000 pounds per square foot (psf) until determined during preliminary design for hydraulic structures with a factor of safety equal to or greater than 3.0 with respect to shear failure in the subsurface material. This is consistent with the requirements of USACE EM 1110-2-2100 and ECB 2017-2. See Section 12.0 – Structural Design for further discussion on foundations.

9.10 Pipelines/Trenches

The sands encountered in the borings are expected to provide good support for utility pipelines without the need for additional bedding when the invert elevations are at least 24 in above the groundwater level (natural or pre-drained by dewatering). Any organics or other deleterious materials encountered at or below the pipe invert shall be considered compressible and unsuitable for pipe support. These soils will be over-excavated and replaced with compacted clean sand or FDOT No. 57 coarse aggregate.

The bedding surface will be uniformly compacted to a density of not less than 95 percent of the maximum dry density in accordance with ASTM D 1557, the Modified Proctor Method. Select backfill should consist of clean, granular materials that are free of debris, cinders, combustibles, and organic-laden materials. The fines content (i.e., material passing U.S. Standard Number 200 sieve) should not be more than 15 percent by weight. Particle sizes larger than 3 in in any direction shall not be allowed, and the organic content should not exceed 3 percent by dry weight. Select fill should meet the USCS (ASTM D2487) designations as SW, SP, SP-SM, SC (<15% fines) or SM (<15% fines).

9.11 Design Parameters

Analyses of the geological investigation presented in the Geotechnical Memorandum in Appendix 3 were used to determine design parameters for the strata detailed in Section 9.3.

9.11.1 Unit Weight

Based on the NRCS' National Engineering Handbook Part 631 (NRCS, 2012), the general maximum dry density for the following soil classifications are provided:

- SP ranges from 100 to 120 pounds per cubic foot (pcf),
- SP-SM ranges from 105 to 123 lb/ft3,
- SM ranges from 110 to 25 lb/ft3,
- SP-SC ranges from 103 to 123 lb/ft3,
- SC ranges from 105 to 125 lb/ft3,
- MH ranges from 70 to 95 lb/ft3,
- CH ranges from 75 to 105 lb/ft3, and

- CL-ML ranges from 95 to 120 lb/ft3.

Principles of Geotechnical Engineering (Das and Sobhan, 2014) references the following:

- typical uniform sand dry unit weight as 92 lb/ft^3 in a loose condition to 115 lb/ft^3 in a dense condition;
- silty sands from 102 lb/ft^3 in a loose condition to 121 lb/ft^3 in a dense condition; and
- clays from 73 lb/ft^3 in a soft condition to 108 lb/ft^3 in a stiff condition.

Compressive strength testing was not performed on the limestone layers; however, based on experience with other projects in South Florida, limestone is expected to have unit weights on the order of 115 to 130 lb/ft^3. Moist and saturated densities were estimated using natural moisture contents from laboratory testing on jar samples. The recommend unit weight parameters listed in Table 9-2 were estimated based on SPT-N Values and engineering judgement.

Table 9-2. Unit Weight Parameters

Stratum No.	General Description (USCS)	Relative Density		Unit Weight (pcf)	
		Average N_{AUTO}	Average N_{ES}	Moist	Submerged
1	SHELLY SAND/ SAND WITH SILT/ SILTY SAND (SP/ SP-SM/ SM)	11	14	98	48
2	SAND WITH CLAY (SP-SC)	5	6	94	45
3	CLAYEY SAND/ SILTY CLAY (SC/ CL-ML)	6	7	95	45
4	CLAY (CL/CH)	6	7	95	45
5	SILT (MH)	8	10	96	47
6	MODERATELY HARD LIMESTONE	14	17	108	60
7	SHELLY SAND (SP)	10	12	97	48

9.11.2 Specific Gravity

Specific gravity testing was not performed. Principles of Geotechnical Engineering (Das and Sobhan, 2014) indicates that sand, which is mostly quartz, has a specific gravity of approximately 2.65, whereas silty and clays soils have specific gravities that vary between 2.6 and 2.9. The recommended specific gravity parameters listed in Table 9-3 were estimated based on documentation and engineering judgement.

Table 9-3. Specific Gravity Parameters

Stratum No.	General Description (USCS)	Relative Density		Specific Gravity
		Average N_{AUTO}	Average N_{ES}	
1	SHELLY SAND/ SAND WITH SILT/ SILTY SAND (SP/ SP-SM/ SM)	11	14	2.65
2	SAND WITH CLAY (SP-SC)	5	6	2.65
3	CLAYEY SAND/ SILTY CLAY (SC/ CL-ML)	6	7	2.68
4	CLAY (CL/CH)	6	7	2.70
5	SILT (MH)	8	10	2.70
6	MODERATELY HARD LIMESTONE	14	17	2.71
7	SHELLY SAND (SP)	10	12	2.65

9.11.3 Shear Strength

Undisturbed samples of fill material were not obtained during the geotechnical investigation as the material is generally non-cohesive. Shear strength parameters were estimated based on SPT-N values and engineering judgement. The effective friction angle was estimated based on USACE EM 1110-1-1905 for SPT resistance values (N). The N values were adjusted to an average energy ratio of 60 percent (N60) based on methodology of Seed and Skempton (Das and Sobhan, 2014).

Fill material used in constructing the new levees or berms are assumed be of similar material of existing materials; however, the material is expected to be more densely compacted in lifts during construction. Therefore, for new fill material, a friction angle of 35 degrees is assumed. The new fill is assumed classify as sand, and therefore 0 psf cohesion is assumed.

Strength parameters for limestone were based on field SPT values. The effective friction angle was estimated based on USACE EM 1110-1-1905 and N60 parameters as described prior in this section. The effective friction angle for limestone based on the average of SPT tests within limestone is 37 degrees. Limestone cohesion was estimated based on the method described in Estimating Mohr-Coulomb Friction and Cohesion Values from the Hoek-Brown Failure Criterion (Hoek, 1990); however, strength tests were not performed. Based on our experience with the Florida limestone, a cohesion value on the order of 300 to 1,000 psf is expected for competent moderately to well-cemented limestones. For weakly cemented limestones, a range of 50 to 100 psf is anticipated.

The selected strength parameters are presented in Table 9-4.

Table 9-4. Shear Strength Parameters

Stratum No.	General Description (USCS)	Relative Density		Effective Stress	
		Average N_{AUTO}	Average N_{ES}	Φ' (Degrees)	c' (psf)
1	SHELLY SAND/ SAND WITH SILT/ SILTY SAND (SP/ SP-SM/ SM)	11	14	31	0
2	SAND WITH CLAY (SP-SC)	5	6	30	0
3	CLAYEY SAND/ SILTY CLAY (SC/ CL-ML)	6	7	30	50
4	CLAY (CL/CH)	6	7	30	300/500
5	SILT (MH)	8	10	30	0
6	MODERATELY HARD LIMESTONE	14	17	37	200
7	SHELLY SAND (SP)	10	12	31	0

9.11.4 Elastic Deformation

Material properties for elastic deformation are based on SPT data gathered during the geotechnical investigation with the exception of limestone. The analysis of the SPT is based on USACE EM 1110-1-1904. Poisson's ratio was estimated from typical range of values obtained from Principles of Geotechnical Engineering (Das and Sobhan, 2014), and USACE 1110-1-1904. Limestone deformation parameters were based on empirical correlations from Deere and Miller (Deere and Miller, 1966) and SPT values. The selected parameters are shown in Table 9-5.

Table 9-5. Elastic Deformation Parameters

Stratum No.	General Description (USCS)	Relative Density		Modulus of Elasticity (Es) (psf)	Poisson's Ratio (v_s)
		Average N_{AUTO}	Average N_{ES}		
1	SHELLY SAND/ SAND WITH SILT/ SILTY SAND (SP/ SP-SM/ SM)	11	14	110000	0.27

2	SAND WITH CLAY (SP-SC)	5	6	50000	0.28
3	CLAYEY SAND/ SILTY CLAY (SC/ CL-ML)	6	7	60000	0.30
4	CLAY (CL/CH)	6	7	40000	0.40
5	SILT (MH)	8	10	40000	0.32
6	MODERATELY HARD LIMESTONE	14	17	560000	0.30
7	SHELLY SAND (SP)	10	12	100000	0.27

9.11.5 Hydraulic Conductivity

Twenty-six piezometers were installed at the Project site. The piezometers were installed such that the screen intervals were placed within the different subsurface strata to measure the in situ permeability using the constant head test method as described below.

Constant head permeability tests were performed by pumping water into the piezometers at a measured constant but slightly variable flow rate to maintain the water level constant near the top of the piezometer casing while measuring and recording the cumulative water flow rate (volume pumped during each consecutive minute). Cumulative flow rates were recorded with an in-line flow meter and then subsequently discretized to flow rates per minute. Following an initial stabilization period, the flow rates were monitored and recorded during a 10-minute period for each test. Test data is provided in the Geotechnical Memorandum in Appendix 3. Permeabilities were estimated using Hvorslev's formulation, as presented in Soil Mechanics (Lambe and Whitman, 1969). The values for constant head in situ permeability tests are summarized in Table 9-6.

Table 9-6. Summary of In Situ Permeability Test Results					
Piezometer ID		Test Depth (bgs) (ft)	Stratum Designation(s)	Horizontal Hydraulic Conductivity - Constant Head, kh/kv=2	
				ft/day	cm/sec
PZ-1 (B-26)	#1	5 – 15	1	1.154	4.07E-04
	#2	20 – 30	1	0.398	1.41E-04
	#3	35 - 45	1	1.429	5.04E-04
PZ-2 (B-3)	#1	8 - 13	1	0.675	2.38E-04
	#2	20 – 25	1	1.746	6.16E-04
	#3	32 - 42	2,3,4	1.644	5.80E-04
PZ-3 (B-6)	#1	5 – 15	1	0.611	2.16E-04
	#2	32 – 36	1	1.734	6.12E-04
	#3	44 - 50	1	2.789	9.84E-04
PZ-4 (B-8)	#1	5 – 15	1	0.384	1.35E-04
	#2	20 – 27	1	2.832	9.99E-04
	#3	28 - 34	3,4,5	1.759	6.20E-04
PZ-5 (B-11)	#1	4 – 10	1,7	0.688	2.43E-04
	#2	15 – 25	1	1.125	3.97E-04
	#3	32 - 38	1	1.820	6.42E-04
PZ-6 (B-18)	#1	10 – 20	1	0.583	2.06E-04
	#2	20 - 28	1	1.247	4.40E-04
	#3	28 – 38	3,4	1.542	5.44E-04
PZ-7 (B-14)	#1	8 – 16	7	0.476	1.68E-04
	#2	18 – 26	1	1.254	4.42E-04
	#3	28 – 34	7	1.605	5.66E-04
	#4	35 - 42	1	2.748	9.69E-04
PZ-8 (B-23)	#1	4 – 8	1	1.325	4.68E-04
	#2	8 – 18	1, 2	0.954	3.37E-04
	#3	18 – 28	1	1.532	5.40E-04
	#4	32 - 42	4	1.895	6.69E-04

9.12 Additional Geotechnical Investigations and Design

The geotechnical investigation associated with this DDR is detailed in the Geotechnical Engineering Memorandum provided in Appendix 3. To complete the preliminary and final design, the work completed to date must be supplemented with additional subsurface investigation, testing, and design analysis. To complete the preliminary design submittal, additional soil borings will be conducted at the location of all potential PSs, WCSs, and along the alignment of the re-routed L-62 canal. The explorations needed for final design will be determined as the design progresses; however, a comprehensive geotechnical investigation program will be conducted in general accordance with USACE EM 1110-1-1804. The results of the geotechnical investigation will be documented in the GDR, which will be submitted as part of preliminary design and updated as part of final design.

Also as part of preliminary design, slope stability analyses will be performed on typical embankment/canal sections to verify that slope stability requirements are met for the various foundation conditions in accordance with SFWMD DCMs or USACE EM 1110-2-1902.

Preliminary seepage analyses were conducted to support the surface water modeling for the conceptual design. Note that this modeling was performed on preliminary cross sections and preliminary water levels and was conservatively used to support the surface water modeling. A comprehensive seepage analysis will be conducted in general accordance with USACE EM 1110-2-1901 and USACE EM 1110-2-1902.

In addition, settlement evaluations will be performed on typical earthwork sections to identify immediate (elastic), long-term, and secondary settlement potential for the proposed alignment. Elastic and consolidation settlement of the embankment will be estimated in accordance with SFWMD DCMs or USACE EM 1110-1-1904.

The data gathered from the soil borings and laboratory testing will be used to develop conceptual foundation support recommendations, as well as seepage mitigation measures for water impounding structures and/or structures that will penetrate proposed levee embankments. Development of the foundation type for the proposed structures will be based on the results of the field investigation and laboratory testing results. Foundation seepage and the potential for piping of subsurface materials will be evaluated for major structures. Foundation analysis and design will be completed in accordance with pertinent USACE design manuals. The allowable bearing capacity of the soils underlying the structures will be computed in accordance with USACE EM 1110-1-1905. Additionally, structure settlement will be estimated in accordance USACE EM 1110-1-1904.

The results of the geotechnical engineering evaluations will be documented in the GEDR, which will be submitted as part of preliminary design and updated as part of final design.

9.13 PES

The EIP team will provide the final cross-sections and latest elevations for the PES for evaluation during preliminary design. Given the location of the inflow flow-way and inflow channel, the former will intercept groundwater from Cell 5, which will bring the groundwater elevation to approximately 23.0 ft NAVD and decline to 21.7 ft NAVD as flows proceed down the flow-way. The inflow channel at elevation 21.6 ft NAVD will set the groundwater elevation between the northern and southern cell modules where the initial PES module would be located. Interpolating between these values, the groundwater elevation at the northern cells would be approximately 22.3 ft NAVD at the eastern end, and 21.7 ft NAVD at the western end where the initial 6-ac module would be located.

The stone bottom elevation of the PES is 16.8 ft NAVD, with a discharge elevation of 17.0 ft NAVD, means there will be a 4.7-ft hydraulic gradient driving groundwater into the stone. This is a similar hydraulic gradient to the seepage analysis of Cell 3 with 4 ft of water as calculated by the "Geotechnical Engineering Memorandum-Preliminary Seepage Modeling-Lower Kissimmee Basin STA-Okeechobee County, Florida, RADISE Project No: 210910." In that document, the water surface elevation differences were considerably greater than 4.7 ft. Yet the seepage flows varied from only 3.2 to 25.6 cf/day per linear foot of canal. Converted to inflow per square foot, an inflow of 10 cf/day applied to a 5.5-ft-deep seepage canal 5 ft wide would amount to slightly more than 3 in per day. Compared to the 6 ft/day applied to the PES cells, this is an insignificant amount, comprising little more than 4 percent of the total inflows.

Through detailed computations, the EIP team will verify the extent of groundwater inflow as the design is finalized. Since these flows are likely to be but a fraction of the 18 cfs applied, its effect on PES hydraulic modeling is considered minimal. The EIP team will review and update the PES design PES according to the final seepage analysis. Refer to the H&H TM for associated figures describing the system and its hydraulic grades. The TM is attached in Appendix 2 of the DDR.

SECTION 10

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SECTION 10

EMBANKMENTS

Embankments will be located along the perimeter and interior of the site to manage flows and separate STA cells. Preliminary design includes approximately 45 miles of embankment construction throughout the site. Embankments will be constructed in accordance with SFWMD DCM-4 and USACE EM 1110-2-1913 using suitable fill, as defined by ongoing geotechnical analyses, generated from onsite excavation activities. Details regarding embankment analyses and design are presented in this section.

10.1 Embankment Design

Embankments will be designed following SFWMD's minimum standard top width of 14 ft, cross-sloped at a minimum 2 percent grade for drainage. There will be a 1-ft shoulder on both sides of the road base, and embankments will be designed to allow vehicular traffic access to all structures and appurtenances throughout the site.

Embankment construction will be accomplished by conventional means using techniques employed for existing STAs in the vicinity of the Project area. Embankment construction material is proposed to be excavated during construction of adjacent canals to the extent practical. Excavated material will be processed onsite to meet geotechnical requirements and placed in 1-ft (maximum) lifts at the required compaction specifications. Levee slopes 3H:1V and steeper will be sodded per SFWMD's specifications.

A 20-ft wide, horizontal maintenance bench will be provided on an embankment on a minimum of one side of each adjacent canal to allow for canal access and maintenance. Access ramps to and from maintenance benches will be provided from the levee roads at the top of the embankments.

All embankment top elevations will be set to maintain required freeboard as defined by hydrologic and hydraulic modeling described in Section 8.4.4.

10.2 Stability

As part of preliminary design, slope stability analyses will be performed on typical embankment sections to verify that slope stability requirements are met for the various foundation conditions. The analyses will be performed using the Limit Equilibrium Approach adapted to computer solution using the SLOPE/W computer program to identify the margin of safety against slope failure. SLOPE/W analyzes circular sliding surfaces via Spencer's Methods for circular sliding surfaces. It also includes wedge-type mechanisms to determine the margin of safety against slope failure. Embankments will be analyzed for shallow and deep sliding surfaces under the range of operating water head levels. Stability analyses will be performed to evaluate embankment performance under typical operating conditions reasonably expected during the facility's operating life. These analyses are to include:

- The end-of-construction (EOC) condition, which simulates the embankment condition immediately after completion of the perimeter embankment. Pore pressures in the underlying foundation soils usually reach their maximum values when the embankment reaches maximum height. The upstream and downstream slope stability is critical for EOC. A water level at the ground surface was used to analyze this condition.
- Steady-state seepage (SSS), which occurs when the water storage pool area and downstream canals are filled with water for a long period. Pore pressures are determined by SSS conditions

where gravitational flow conditions govern. The downstream slope stability is critical for the SSS conditions. Porewater pressures generated from the SSS analyses were used to analyze the SSS slope stability condition.

- Rapid drawdown (RDD) simulation. RDD primarily occurs when the upstream impoundment water level is lowered rapidly from the maximum storage level to a low or empty water level. Under such an assumed RDD condition, the water level in the upstream embankment side slope is modeled as remaining at the maximum storage pool level. The upstream slope stability is critical for the RDD condition.

Recommended minimum factors of safety for long term (SSS at maximum storage pool elevation) as well as short-term EOC and RDD conditions and are provided in Table 10-1. These values are provided as recommended guidance in USACE's EM 1110-2-1902, Engineering and Design, *Slope Stability* (USACE, 2003b), however final values will be selected on each individual case analysis as part of the engineering evaluation for the project.

Table 10-1. Embankment Slope Stability Analyses

Analysis Condition	Required Minimum Factor of Safety	Slope Face
EOC	1.3	Upstream/Downstream
SSS - Maximum Storage Pool -Maximum Surcharge Pool	 1.5 1.4	Downstream
RDD*	1.1 - 1.3*	Upstream

**Note: The 1.1 Factor of Safety 1 applies to RDD from the maximum surcharge pool.*

In addition, if evaluation of wind-generated waves and storm surge indicates impact on STA performance, operating pool "set-up/set-down" due to sudden wind direction changes will also be included. The operational drawdown rate and the set-up/set-down rate will be provided by the hydraulic modelers.

Settlement calculations will be performed to estimate settlement of the embankment foundation during and after construction, and settlement of the embankment fill after construction. Elastic deformation of sands, consolidation of clays and long-term creep will be considered, and recommendations for construction and operation to address the predicted affects of settlement on embankments and structures will be provided. Embankment settlement will be estimated in accordance with SFWMD DCMs or USACE EM 1110-1-1904 criteria, Engineering and Design, *Settlement Analysis* (USACE, 1990).

The preliminary slope design will consider variations in embankment side slope, embankment height, embankment material, bench distances between embankment toe of slope and canal top of bank, and operating water levels. Slope stability analysis will be performed in accordance with SFWMD DCMs or USACE EM 1110-2-1902, Engineering and Design, *Slope Stability* (USACE, 2003b).

10.3 Seepage Control

Preliminary seepage analyses were conducted to support the surface water modeling for conceptual design. Note that this modeling was performed on preliminary cross-sections and preliminary water levels and was used to support surface water modeling. A comprehensive seepage analysis will be conducted in general accordance with USACE EM 1110-2-1901 Engineering and Design *Seepage Analysis and Control for Dams* (USACE, 1993b) and USACE EM 1110-2-1902, Engineering and Design, *Slope Stability* (USACE, 2003b).

Preliminary seepage analysis was performed using GEOSLOPE SEEP/W 2020 computer modelling software. SEEP/W is a finite element software for modelling groundwater flow. The software is capable of simulating steady-state and transient conditions using 2D analysis. Preliminary sensitivity analysis of the seepage models was performed using existing conditions and borehole permeability testing data. These values were used to estimate the phreatic surface in and around the modelled cross sections.

The EIP team identified and evaluated 19 preliminary, representative design cross-sections. The cross sections are presented in Geotechnical Engineering Memorandum – Preliminary Seepage Modeling in Appendix 3.

Seepage analysis model boundaries were extended 1,000 ft exterior to the centerline of the embankment/canal section to minimize potential seepage-reduction impacts due to close proximity model boundary conditions. For seepage modeling purposes, normal canal operating water levels across the embankment/canal section were determined from hydraulic modeling data. Hydraulic modeling data is located in Section 8. A project-specific 3D groundwater model is being developed to assist in evaluating the potential impacts to adjacent lands and optimizing the design and operation of the seepage collection and management systems.

10.4 Erosion Protection

The embankment materials to be produced for the Project will consist of a well-graded mixture of sand with some gravel, silt, and cobbles that forms a durable surface that is resistant to erosion by wind-generated waves. Preliminary analyses indicate that erosion protection is not required for all embankments; however, evaluation will continue throughout the design to determine if and where additional protection is required.

Erosion protection should be compatible with the design embankment slopes, embankment soil types, surface water levels, and freeboard requirements and should be in accordance with the District's Applicant's Handbook Part IV: Erosion and Sediment Control. The District allows riprap as a means to reduce the force of waves and to protect land from erosion. Riprap should consist of predominantly angular unconsolidated boulders, rocks, or clean concrete rubble with no exposed reinforcing rods or similar protrusions. Erosion control measures (such as riprap and concrete-lined channels) should provide proposed velocities that are non-erosive and sufficient to safely convey flows. Information on design and performance standards to achieve storage and conveyance requirements are in Volume II of the Handbook, specific to the geographic area covered by each District. The stone size for protecting embankment slopes and canals upstream and downstream of structures should be selected in accordance with District and/or USACE criteria (USACE EM 1110-2-1601 and USACE HDC Sheet 712-1).

Where flow velocities are greater than 2.5 ft/sec or nearby control structures are at the inflow and discharge points for the PS slope protection will be included that will consist primarily of rip rap. Other slope protection measures, such as articulated concrete block, could also be used.

10.5 PES

The PES embankments are being designed in accordance with the standard criteria outlined above. Design details and associated figures are outlined in the H&H TM 4 – Additional Modeling Results provided in Appendix 4. The EIP team will revise the design as needed based on feedback and any changes to the PES layout.

10.6 Proposed Future Activities

10.6.1 Preliminary Design

The following embankment-related activities are proposed for preliminary design:

- EIP will develop draft preliminary plans and specifications. As related to the embankments, the draft POM, DDR, and regulatory plan will be updated, and a construction schedule and stipulated price proposal will be provided.
- Geotechnical analyses will continue, and embankment configurations may be adjusted to meet geotechnical requirements.
- In close coordination with SFWMD staff, a project-specific 3D groundwater model will be prepared to assist in evaluating the potential impacts to adjacent lands and optimizing the design and operation of the seepage collection and management systems. The technical approach will include key assumptions, summaries of data availability and gaps, and performance measures.
- Embankment configurations may be adjusted as site access and maintenance needs are evaluated as the design progresses.

10.6.2 Final Design

The following activities are proposed for final design:

- EIP will advance plans and specifications

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SECTION 11

CANALS

Conveyance canals for the Project will be designed and constructed in general accordance with previous SFWMD design criteria and sized to provide capacity for modeled flows in each area of the site as described in Section 8. The Project will include six canal configurations, each serving a specific purpose as described in Section 11.1. The six canal types are L-62 canal reroute, inflow, distribution, collection, outflow, and seepage; their proposed locations are shown on Figure 11-1.

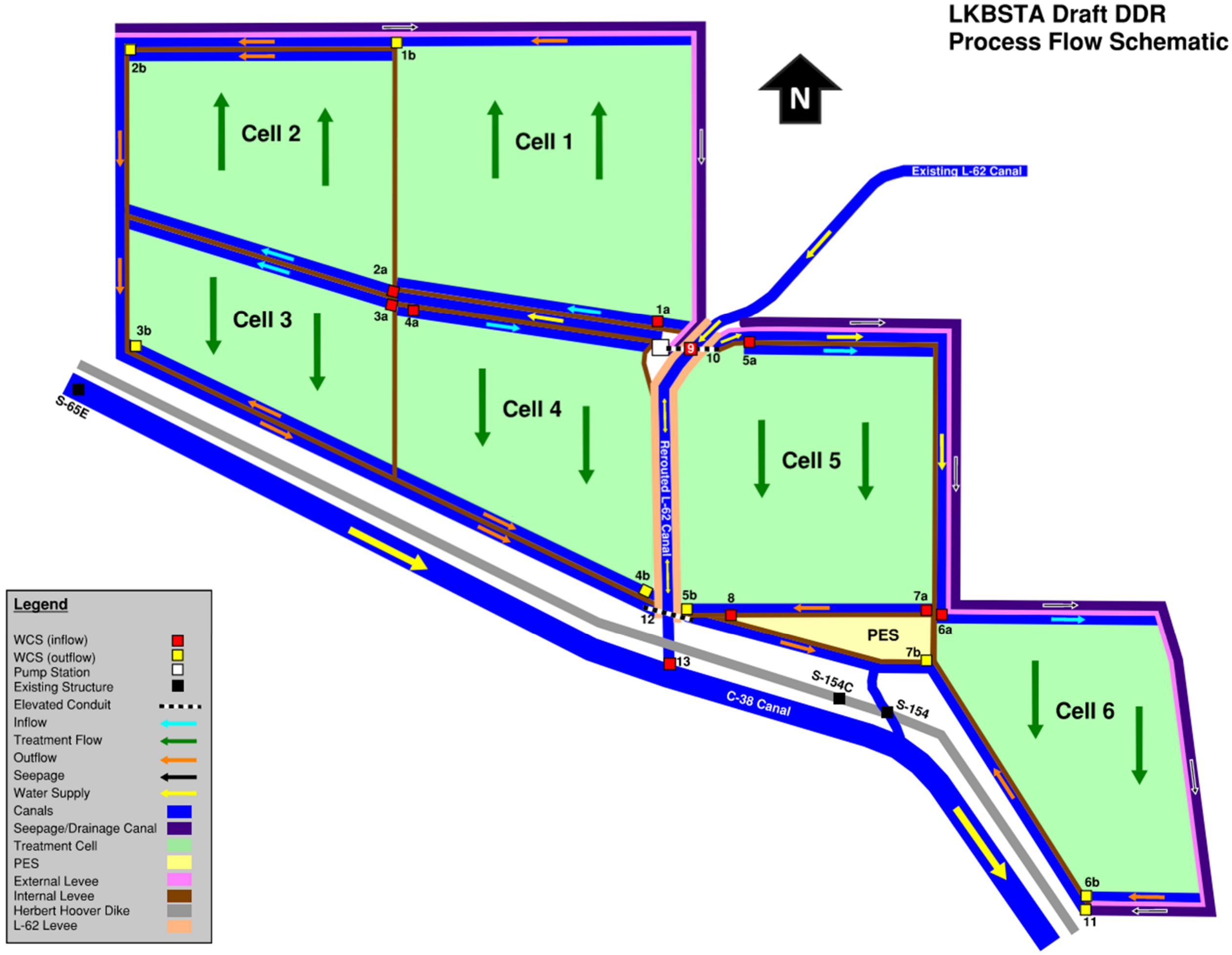

Figure 11-1. Process flow schematic of canals

11.1 Canal Design

Canal configurations may vary depending on location and adjacent structures, but all canals will be trapezoidal and have minimum side slopes of 3H:1V and be sloped to maintain a minimum operating depth of 6 ft to inhibit vegetation growth. Each canal will include a 20-ft bench on at least one side to provide access by a CAT-336L long-reach excavator.

L-62 Canal Reroute – The purpose of the L-62 canal reroute, located west of the existing alignment, is to redirect flows from the existing canal to the location of the inflow PS to provide the primary raw water source for treatment. The approximately 7,000-ft L-62 canal reroute will continue to convey flow from the existing L-62 canal into the C-38 canal through a new WCS (WCS-13) at the HHD. At the northern end, WCS-9 will maintain current operations and functionality of the existing L-62 canal and function as a replacement for the current S-154 structure. The design capacity for the L-62 canal reroute is 1,000 cfs. Details regarding WCS-9 and WCS-13 design parameters are provided in Sections 8 and 12.

The L-62 canal will be designed with a bottom width of 15 ft and a bottom elevation of 1.0 ft NAVD. 20-ft-wide maintenance benches will be located on both sides and sloped at 2 percent toward the canal for drainage. The top of berm elevations are set at a minimum of 30.5 ft NAVD on either side, with top berm widths of 14 ft on each.

A typical cross-section of the proposed L-62 canal reroute is provided on Figure 11-2.

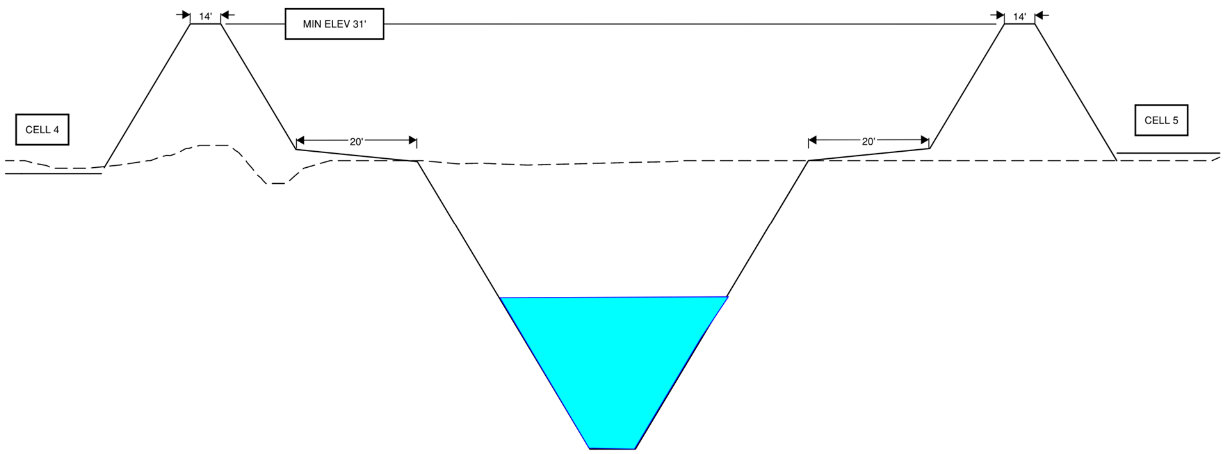

Figure 11-2. L-62 canal reroute – typical section

Inflow Canals – Two inflow canals, fed by the inflow PS, will route raw water to STA cells on the west and east sides of the L-62 canal reroute. The west inflow canal will feed raw water to Cells 1 through 4; the east inflow canal will feed raw water to Cells 5 and 6 and the PES.

The west inflow canal is proposed to be approximately 4,900 ft long and will be routed from the PS west between Cells 1 and 4, terminating at the embankment confluence between Cells 1, 2, 3, and 4. Four STA WCSs (1a, 2a, 3a, and 4a) will feed each of the respective four cells.

The east inflow canal will receive raw water discharges from an elevated conduit connected to the PS discharge stilling area. The raw water discharges will be routed approximately 9,050 ft along the north and east sides of Cell 5 and terminate at the northwest corner of Cell 6. Two STA WSCs (5a and 6a) will discharge raw water from the canal into Cells 5 and 6, respectively. A seventh WSC (7a) will be located at the termination point of the canal, adjacent to WSC 6a, for raw water discharges into the PES as required based on operations.

Inflow canals will have bottom widths of 5 to 10 ft, and slopes will be shallow with grade control structures as required to maintain a 6-ft depth. Bottom elevations will be set approximately 3 ft below STA WCS inverts at each cell inflow location.

Outflow Canals – Two outflow canals will route treated water from all STA cells to a common discharge point at the original L-62 canal outflow location through the S-154 structure. Like the inflow canals, the outflow canals are categorized as “west” or “east” based on their location west or east of the L-62 canal reroute.

The west outflow canal will collect treated water from Cells 1 through 5; the east outflow canal will route treated water from Cell 6, the PES system, and a portion of the seepage canal along Cells 5 and 6. Both outflow canals will receive treated flows from each STA cell through the respective outflow STA WCSs and from the PES through STA WCS 7b.

The west outflow will be approximately 25,600 ft long and will be routed west from the north end of the interior embankment between Cells 1 and 2, turning south and running parallel to 128th Avenue on the west side of Cells 2 and 3. At the point where the canal meets the HHD on the southwest corner of Cell 3, the canal then will parallel the HHD flowing southwest below Cell 4, cross the rerouted L-62 canal via an elevated conduit, and run along the south side of Cell 5 and the PES before discharging upstream of S-154. Five STA outflow WSCs (1b, 2b, 3b, 4b, and 5b) will discharge to the west outflow canal.

The east outflow canal will be approximately 5,750 ft long and located parallel to the HHD, flowing generally northwest along the west side of Cell 6 and the southeast corner of the PES. The canal will receive treated flows from the Cell 6 STA outflow WCS (6b), the PES outflow WCS (7b), and seepage canal WSC 11.

Outflow canals will have bottom widths of 5 to 10 ft, and slopes will be shallow with grade control structures as required to maintain a 6-ft depth. Bottom elevations will be set approximately 3 ft below STA WCS inverts at each discharge location.

Seepage Canals – Two seepage collection canals will be located on the perimeter of the project site along the outside of the STA cell external embankments and will be designed to convey STA cell seepage and off-site drainage, as needed, to one of three discharge locations. All seepage collection canals will be trapezoidal with 3H:1V side slopes and varying bottom widths and elevations depending on required capacity. Passive grade control structures will be implemented to control flow depth (6 ft minimum to inhibit vegetation growth) and account for elevation drops from north to south across the site.

The west seepage canal will be located along Cells 1 and 2 and extend from the northwest corner of Cell 2, east along SR 70 north of Cell 1. There are two notable offsite stormwater drainage (run-on) sources along SR 70 that the canal will collect in addition to seepage along this section. The canal then will be routed south along the east side of Cell 1 to a discharge point at the inflow PS site. The total length of the west seepage canal will be approximately 16,000 ft.

Under normal seepage control operations, low flows in the west seepage canal will be routed through a conduit discharging into the rerouted L-62 canal downstream of WCS-9. During storm events, when the WSC will be conveying up to 900 cfs of flows for a 100-year event, the discharge system will be designed to allow first-flush flows of up to 150 cfs through the conduit downstream of WCS-9 while excess flows are routed over a spillway to the L-62 canal upstream of WCS-9. This allows the system to capture the highest concentration of runoff from the drainage area upstream of SR 70 to be treated.

The east seepage canal collects seepage from Cells 5 and 6 and will be routed along the north and east sides of Cells 5 and 6, terminating at WCS-11 on the southwest corner of Cell 6. The total length of the canal is approximately 20,150 ft.

There are no seepage collection canals to be constructed along the west sides of Cells 2 and 3 based on extremely low modeled values of less than 3 cfs for the entire section. It is anticipated that the outflow canal along this section will be sufficient to also capture the small amount of seepage. There are no seepage controls proposed along any portions of the project boundary adjacent to the HHD as they are unnecessary given that the C-38 canal is on the opposite side of the HHD embankment.

A cross-section showing the typical configuration for canals and embankments at a cell discharge point is provided on Figure 11-3. Note that this configuration is not drawn to scale and will differ by cell based on the design flow at each specific location.

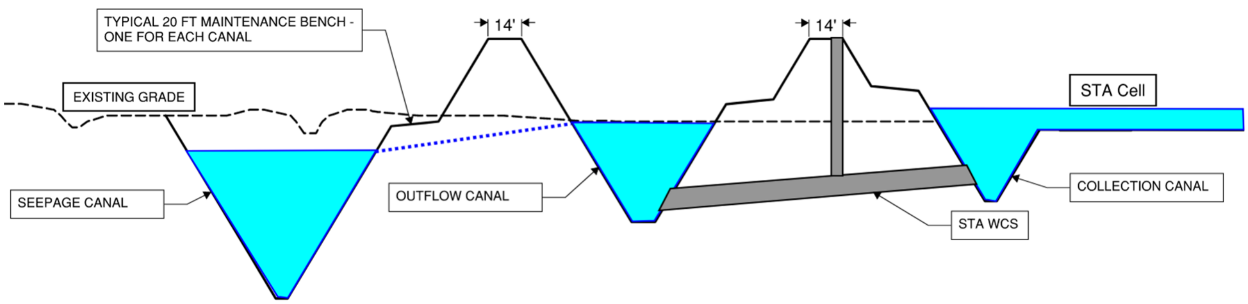

Figure 11-3. Typical canal/embankment section at cell discharge area

Distribution Canals – Each of the six treatment cells in the STA will include a dedicated distribution canal on its inflow side that receives water from the inflow canal through an STA WCS. The purpose of the distribution canals is to evenly distribute water over the width of the treatment cell to promote uniform sheet flow across the cell's surface area. During preliminary design, a gradual transition between the distribution canal and cell will be evaluated.

Collection Canals – Similarly designed collection canals are located at the opposite end of each cell to provide a common conveyance path of treated flow, i.e., routing to the cell outflow STA WCS. Collection canals convey water to STA WCSs connecting to outflow canals.

Under normal operating conditions, there will be a maximum water depth of 2 ft above STA cell bottom in both distribution and collection canals. Distribution and collection canals are proposed to have a bottom width of 5 ft and a bottom elevation 6 ft below the STA cell bottom elevation, which translates to a top width of 53 ft.

11.2 Stability

As part of preliminary design, slope stability analyses will be performed on typical embankment sections to verify that slope stability requirements are met for the various foundation conditions. The analyses will be performed using the Limit Equilibrium Approach adapted to a computer solution using the SLOPE/W computer program to identify the margin of safety against slope failure. SLOPE/W analyzes circular sliding surfaces via Spencer's Methods for circular sliding surfaces. It also includes wedge-type mechanisms to determine the margin of safety against slope failure. Embankments will be analyzed for shallow and deep sliding surfaces under the range of operating water head levels. Stability analyses will be performed to evaluate embankment performance under typical operating conditions reasonably expected during the facility's operating life.

The analyses to be performed are to include:

- The EOC condition, which simulates the embankment condition immediately after completion of the perimeter embankment. Pore pressures in the underlying foundation soils usually reach their maximum values when the embankment reaches maximum height. The upstream and downstream slope stability is critical for the EOC condition. A water level at the ground surface was used to analyze this condition.
- SSS, which occurs when the water storage pool area and downstream canals are filled with water for a long period. Pore pressures are determined by SSS conditions where gravitational flow conditions govern. The downstream slope stability is critical for SSS conditions. Porewater

pressures generated from the SSS analyses were used to analyze the SSS slope stability condition.

- RDD simulation. RDD primarily occurs when the upstream impoundment water level is lowered rapidly from the maximum storage level to a low or empty water level. Under such an assumed RDD condition, the water level in the upstream embankment side slope is modeled as remaining at the maximum storage pool level. The upstream slope stability is critical for the RDD condition.

Review of USACE's Engineering Manual EM 1110-2-1902 indicates that the recommended factors of safety for long term (SSS at maximum storage pool elevation), the short-term EOC, and RDD conditions are presented as follows. The factors of safety listed in EM 1110-2-1902 provide guidance for levee slope stability, but the values listed are not required. Recommended minimum factors of safety for slope conditions across upstream and downstream faces are presented in Table 11-1.

Table 11-1: Embankment Slope Stability Analyses Factors of Safety

Analysis Condition	Recommended Minimum Factor of Safety	Slope Face
EOC	1.3	Upstream/Downstream
SSS - Maximum Storage Pool -Maximum Surcharge Pool	 1.5 1.4	Downstream
RDD*	1.1 - 1.3*	Upstream

**Note: A Factor of Safety of 1.1 applies to RDD from maximum surcharge pool*

In addition, if evaluation of wind-generated waves and storm surge indicate impact on STA performance, operating pool "set-up/set-down" due to sudden wind direction changes will also be included. The operational drawdown rate and the set-up/set-down rate will be provided by the hydraulic modelers.

Settlement calculations will be performed to estimate settlement of the embankment foundation during and after construction, and settlement of the embankment fill after construction. Elastic deformation of sands, consolidation of clays and long-term creep will be considered, and recommendations for construction and operation to address the predicted effects of settlement on embankments and structures will be provided. Settlement of the embankment will be estimated in accordance with SFWMD DCMs or USACE EM 1110-1-1904 criteria, Engineering and Design, *Settlement Analysis* (USACE, 1990).

The preliminary slope design will consider variations in embankment side slope, embankment height, embankment material, bench distances between embankment toe of slope and canal top of bank, and operating water levels. Slope stability analysis will be performed in accordance with SFWMD DCMs or USACE EM 1110-2-1902, Engineering and Design, *Slope Stability* (USACE, 2003b).

11.3 Seepage Control

Preliminary seepage analyses were conducted to support the surface water modeling for conceptual design. Note that this modeling was performed on preliminary cross-sections and preliminary water levels and was used to support the surface water modeling. A comprehensive seepage analysis will be conducted in general accordance with USACE EM 1110-2-1901 Engineering and Design *Seepage Analysis and Control for Dams* (USACE, 1993b) and USACE EM 1110-2-1902, Engineering and Design, *Slope Stability* (USACE, 2003b).

Preliminary seepage analysis was performed using GEOSLOPE SEEP/W 2020 computer modelling software. SEEP/W is a finite element software for modelling groundwater flow. The computer software is capable of simulating steady-state and transient conditions using 2D analysis. Preliminary sensitivity analysis of the seepage models was performed using existing conditions and borehole permeability testing data. These values were used to estimate the phreatic surface in and around the modelled cross sections.

The EIP team identified and evaluated 19 preliminary, representative design cross-sections. The cross sections are presented in Geotechnical Engineering Memorandum – Preliminary Seepage Modeling in Appendix 3.

Seepage analysis model boundaries were extended 1,000 ft exterior to the centerline of the embankment/canal section to minimize seepage reduction impacts due to close model boundary conditions. For seepage modeling purposes, normal canal operating water levels across the embankment/canal section were determined from hydraulic modeling data. Hydraulic modeling data is located in Section 8. If determined to be necessary, a 3D groundwater model will be developed to assist in evaluating the potential impacts to adjacent lands and optimizing the design and operation of the seepage collection and management systems.

11.4 Erosion Protection

Erosion protection will be provided where modeling indicates flow velocities greater than 2.5 ft/sec, near WCSs, at elevated conduits, and at the inflow and discharge points for the PS. Erosion protection will consist primarily of rip rap or articulated concrete block.

11.5 PES

Section 9.11 of this DDR addresses the canal elevations and how they set the hydraulic gradient into the initial 6-ac PES module. This was done to estimate the amount of groundwater likely to enter the cells. Since inflows were projected to be minimal relative to the design inflows, such changes have been ignored in this preliminary analysis.

The PES canals are being designed in accordance with the standard criteria outlined above. The PES H&H TM describes the geometry of the inflow flow-way, distribution canal and how the geometry is to be designed Due to vegetative retardance, the flow-way slope has been designed to maintain a 2-ft-deep channel along its length. Design details and associated figures are outlined in the PES H&H TM located in Appendix 4. The EIP team will revise the design as needed based on feedback and any changes to the PES layout.

11.6 Proposed Future Activities

11.6.1 Preliminary Design

The following canal-related activities are proposed for preliminary design:

- EIP will develop draft preliminary plans and specifications. Related to the canals, the draft POM, DDR, and regulatory plan will be updated, and a construction schedule and stipulated price proposal will be provided.
- All canal sizing will be finalized in coordination with ongoing modeling efforts.
- Distribution and collection canals will be optimized to provide effective spreading of flows across the STA cells.
- Evaluation of canal geometry will continue to limit erosion at critical transition points, such as from the cell to distribution and collection canals and inflow/discharge locations.

11.6.2 Final Design

The following activities are proposed for final design:

- EIP will advance plans and specifications.
- Fine grading of canal corners to limit shear from erosive velocities and implementation of erosion protection measures.

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STRUCTURAL DESIGN

The LKBSTA Project includes the structural design of one PS, an estimated 14 gated WCSs for controlling inflow to and outflow from the STA and PES treatment cells, two large gated concrete structures on the proposed rerouted L-62 canal, ungated cast-in-place culvert structures, gated HDPE pipe conduits with cast-in-place or precast concrete headwalls, and project support structural features such as precast concrete electrical buildings, generator buildings, gate control buildings, slabs, retaining walls, etc. A general description of the structural design criteria and basis for design of these structural elements is presented in this section.

Preliminary design of structures associated with the LKBSTA Project is based on design criteria and loading factors established by the American Concrete Institute (ACI) and are consistent with the Florida Building Code (FBC). The structural design must also comply with the requirements of the USACE's EMs and any SFWMD criteria.

12.1 Structural Design Criteria and Codes

The LKBSTA Project is being designed and built by EIP with review by the SFWMD. The project will be generally designed to SFWMD standards and guidelines, taking into account applicable federal, state, and local codes, standards, and regulations. The structural design will generally conform to the following applicable editions of standards and codes:

- USACE, U.S. Army Corps of Engineers EMs
- 2020 FBC, Building, 7th Edition
- ASCE 7-16, Minimum Design Loads for Buildings and Other Structures Engineers
- ACI 318-14, Building Code Requirements for Structural Concrete and Commentary
- ACI 315-04 Details and Detailing of Concrete Reinforcement
- American Institute of Steel Construction (AISC) 360-16, Specifications for Structural Steel Buildings
- ADM1-15, Aluminum Design Manual, The Aluminum Association, Inc.
- SFWMD Engineering Design Standards for Water Resource Facilities – Design Guidelines
- Precast Concrete Institute Design Handbook: Precast and Prestressed Concrete (MNL 120-10), Seventh Edition
- ASTM specified standards
- American Welding Society (AWS) D1.1/D1.1M:2010, Structural Welding Code - Steel
- AWS D1.4/D1.4M:2005, Structural Welding Code – Reinforcing Steel
- OSHA 1910, Occupational Safety and Health Standards
- Sheet pile walls (designed by others) are in accordance with USACE EM 1110-02-2504.
- The EIP team will develop the GEDR as part of preliminary design and it will be included in a future DDR appendix when completed.

12.2 Design Loads

Structures will be designed to meet the requirements of Risk Category III-IV for wind speeds as required by the FBC. Structures designed under this Project will be based on the design loads contained in this report.

12.2.1 Dead Loads

Dead loads consist of the weight of materials of construction incorporated into the structure and include, but not limited to, walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding, other similarly incorporated architectural and structural items, and fixed service equipment. Structural material dead loads will be based on the material unit weights indicated herein. Other dead loads will be based on the actual weight of the material and components.

- Concrete – 150 pounds per cubic foot (pcf)
- Steel – 490 pcf
- Aluminum – 170 pcf

12.2.2 Live Loads

The live loads that will be used for structural design of different project areas are listed below. Other live loads will be based on the requirements of ASCE 7 and FBC Table 1607.1.

- Access Walkways – 100 psf uniform
- Operating Floor – 250 psf uniform or actual equipment weight (over its footprint) plus 100 psf uniform, whichever controls
- Storage and Generator/Electrical Room Floor– 250 psf uniform or actual equipment weight (over its footprint) plus 100 psf uniform, whichever controls
- Control Building Floor – 150 psf uniform
- Stairs and Landings – 100 psf uniform, 300 pounds concentrated
- Ladder and Rungs – 40 psf uniform, 300 pounds concentrated
- Standard AASHTO Live loading, AASHTO HL-93

12.2.3 Snow Loads

Exempt in accordance with the FBC. The ground snow load is zero in accordance with the FBC per Section 1608 and Figure 1608.2 for this particular project location.

12.2.4 Seismic Loads

Exempt in accordance with the FBC, as stated in the “Analysis of Changes for the 7th Edition (2020) Florida Building Code” published by the Florida Building Code. “Note: Seismic loading and snow loading provisions in the code are not reserved (deleted) in the 7th Edition (2020) FBCB, even though they do not apply in the State of Florida. While there are some changes to some of these sections and provisions, they are not shown here in this Analysis because they do not apply to construction in the State of Florida.”

12.2.5 Wind Loads

Structures that are at or below grade will not have wind load applied. Structures above finished grade will have wind loads applied in accordance with the requirements of ASCE 7-16. The following criteria are applicable for wind loads:

- Ultimate Design Wind Speed, Vult: 168 miles per hour (mph)

- Nominal Design Wind Speed, Vasd: 130 mph
- Exposure Category: C
- Internal Pressure Coefficients GCPi will be taken as +/- 0.55 for partial enclosed buildings
- Per SFWMD Design Guideline DG-S001, latest edition, a 1.02 factor will be applied to increase the ultimate wind speed noted above for a 200-year mean recurrence interval.

12.2.6 Flood Loads

The majority of the LKBSTA Project site is located within a flood hazard area based on a FEMA Flood Insurance Rate Map (Panels 12093C0455C, 12093C0460C). The majority of the flood hazard areas are classified as Zone A (undetermined base flood elevation [BFE]). The southern end of the location where treatment cell 6 is proposed is in Zone AE, with a BFE of 16 (NAVD). As structures on the LKBSTA site will be designed for maximum water surface elevations at or higher than this base flood elevation, flood loading is not a factor in structural design.

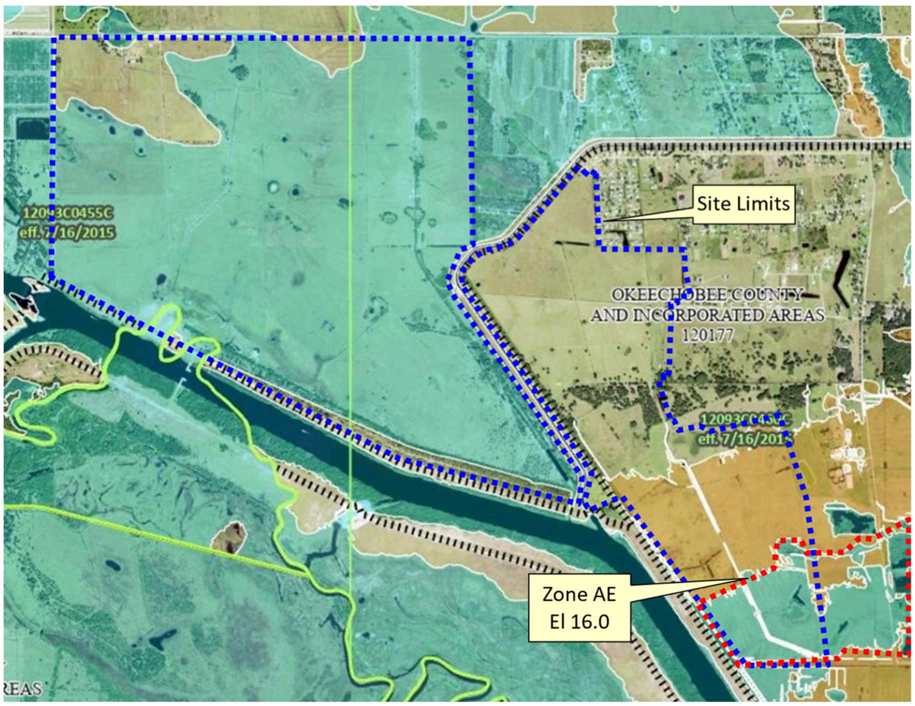

Figure 12-1. Extents of flood hazard areas

12.2.7 Soil Loads and Foundation Requirements

Soil loads and foundation design criteria will be consistent with the recommendations presented in the future GEDR prepared by the EIP team and developed as part of preliminary design. Additionally, a live load surcharge of 500 psf will be applied to below-grade structures, sheeting, and retaining walls. The surcharge earth pressure will have an applicable lateral earth pressure coefficient applied. Live load surcharge will be applied directly to the top of the slab without reduction for soil depth over the structure. Retaining walls will be designed to resist lateral soil pressures and surcharge loads, assuming the hydraulic structure is fully dewatered. Groundwater is known to be within a few feet of the surface. Due to seasonal fluctuations, groundwater will be assumed to be at grade or at maximum water level in the canal for the design of flow-control structures. Until the GEDR is completed, the following preliminary soil parameters will be used:

- Moist Weight (backfill soil): 110 pcf

- Buoyant Weight (backfill soil): 53 pcf
- Allowable bearing capacity: 3,000 psf for shallow foundations
- At-Rest Soil Pressures coefficient: 0.47
- Active Soil Pressures coefficient: 0.31
- Passive Soil Pressures coefficient: 3.25

Foundation materials will be assumed to have an allowable bearing value of 3,000 psf until determined in a future GEDR for hydraulic structures, with a factor of safety equal to or greater than 3.0 with respect to shear failure in the subsurface material. This is consistent with the requirements of USACE EM 1110-2-2100 and ECB 2017-2.

12.2.8 Impact Loads

For elements supporting reciprocating or rotating equipment and monorails, elements will be designed for impact in addition to other loads. The following minimum impact loads will be used:

- Rotating equipment: 20 percent of the total machine weight
- Reciprocating equipment: 50 percent of the total machine weight; consideration to be given to the deflection of beams supporting reciprocating and rotating machines

12.3 Serviceability

In addition to the design requirements in the FBC and the codes and standards for specific materials, additional design requirements to enhance structural serviceability will be used. Materials will be chosen to provide durability, limit deterioration, and minimize structure maintenance.

12.4 Stability

The stability of structures must meet or exceed the factors of safety against sliding, flotation, and overturning specified in Table 12-1. A stability analysis will be performed to comply with the requirements of USACE Manual EM 1110-2-2100. Based on the EM 1110-2-2100 site information classification, this site can be categorized as an ordinary site.

Table 12-1. Stability Criteria and Loading Categories for Structural Stability Analyses

Stability Criteria	Structure Classification	Loading Category		
		Usual	Unusual	Extreme
Sliding Factor of Safety	Normal	1.5	1.3	1.1
Sliding Factor of Safety	Critical	2.0	1.5	1.1
Floating Safety Factor	All	1.3	1.2	1.1
Resultant Location	All	100% Base in Compression	75% of Base in Compression	Resultant within Base

Loading categories, as defined in USACE Manual EM 1110-2-2100, are presented below:

Usual: Usual loads refer to loads and loading conditions that are related to the primary function of a structure and can be expected to occur frequently during the structure's service life. A usual event is a common occurrence, and the structure is expected to perform in the linearly elastic range. Usual load condition categories have annual probability of greater than or equal to 0.10 and return periods of less than or equal to 10 years.

Unusual: Unusual loads refer to operating loads and load conditions that are of infrequent occurrence. Construction and maintenance loads are also classified as unusual loads because risks can be controlled by specifying the sequence or duration of activities or by monitoring performance. For an unusual event, some minor non-linear behavior is acceptable. Unusual load condition categories have annual probabilities of less than 0.10 but greater than or equal to 0.0033 and return periods of greater than 10 years but less than or equal to 300 years.

Extreme: Extreme loads refer to events that are highly improbable and can be regarded as emergency conditions. Such events may be associated with major accidents involving impacts or explosions and natural disasters due to earthquakes or flooding that have a frequency of occurrence that greatly exceeds the service life of the structure. The structure is expected to accommodate extreme loads without experiencing a catastrophic failure. Extreme load condition categories have annual probabilities of less than 0.0033 and return periods greater than 300 years.

12.5 Design Criteria for Materials

12.5.1 Concrete Design Criteria

Concrete structures will be designed in accordance with FBC requirements and the ACI 318 strength design method and as indicated below; they will be shown on the design drawings. The load factors and cover requirements for hydraulic structures will be designed in accordance with USACE EM 1110-02-2104.

Typical parameters for concrete design will be:

- Concrete for cast-in-place structures
  - 4,500 pounds per square in (psi) @ 28 days with a maximum water/cement ratio = 0.40
- Precast concrete
  - 5,000 psi @ 28 days
- Steel reinforcement
  - 60,000 psi (ASTM A615, Grade 60)
- Welded wire reinforcement
  - ASTM A1064

12.5.2 Steel Design Criteria

Steel design will be based on AISC Manual of Steel Construction. Load combinations will be in accordance with the FBC.

Typical design parameters for steel will be:

- Wide flange shapes: ASTM A992
- Other rolled shapes: ASTM A572
- Plates: ASTM A36
- Anchor bolts: ASTM F1554
- High strength bolts: ASTM A325
- Pipe: ASTM A53, Grade B
- Hollow structural shapes: ASTM A500, Grade B
- Welding electrode: AWS Series E70
- Coating details: Society for Protective Coatings

12.5.3 Aluminum Design Criteria

Aluminum structural members will be designed in accordance with the requirements of the FBC and the Aluminum Association's Aluminum Design Manual, latest edition. The design of aluminum structures and members will be based on aluminum alloy 6061-T6. Design of aluminum railing will be based on aluminum alloy 6063. Other alloys will be used for specific requirements as applicable.

12.6 WCS

There will be an estimated 13 gated WCSs designed as part of this Project to introduce untreated water into the STA treatment cells and PES treatment cell and to convey treated water out of the treatment cells into the STA outflow canals. All 11 minor WCSs are double-barrel box culverts. Two other structures, WCS-9 and WCS-13, will also be constructed. WCS-9, located at the north-central area of the STA, will discharge treated water from the STA project as a whole to the new L-62 canal connection to the C-38 canal. WCS-9 is a dual-channel, roller-gate-equipped box culvert. The EIP team will optimize the WCS-13 at the C-38 canal during preliminary final design to use box culverts and slide gates. The WCSs will be designed as cast-in-place, reinforced-concrete construction. Foundations for these structures are mat foundation systems, designed for consistency with the recommendations contained in the future GEDR prepared by the EIP team. Dimensional parameters for the 11 gated WCSs,WCS-9, and WCS-13 at the C-38 canal will be presented in a future table when designed.

Retaining/wing walls for all of these structures will be steel sheet pile walls with concrete pile caps. The sheet pile incorporates an anchor and tie-rod system for support. When necessary to dewater the culverts to perform maintenance on the control gates, stop gates will be installed at the upstream and downstream ends of each inflow and outflow control structure. The stop gates will permit full dewatering of the culvert barrel and control gate well.

12.7 PS

There will be only one PS in the LKBSTA project. The PS will be intended for use only within the LKBSTA. The structural design of this PS includes support of the vertical axial flow pumps, concrete slabs for above-ground equipment, and a discharge end-wall structure. This PS will include SFWMD standard intake channels and an optimized suction bay for each pump. The PS will have a pre-engineered canopy over the pumps that uses pre-engineered galvanized steel frames with roof hatches at the pumps and galvanized steel decking components to create a canopy structure. It will have a preliminary size of 20 ft wide by 60 ft long with a 20- to 25-ft ceiling height. The EIP team will optimize these dimensions during preliminary final design. This pump canopy will be supported by reinforced cast-in-place concrete piers and an elevated slab. This PS will include cast-in-place concrete channels, stop gates for channel isolation, trash racks, screenings storage pads, pump sumps, and discharge structures, as well as steel sheet pile seepage and wing walls with concrete caps. A drive-across bridge at the PS influent channels will allow service of the stop gates, screens, and debris conveyor. The PS will have a mat foundation system, designed for consistency with the recommendations contained in the future GEDR prepared by the EIP team.

Ancillary structural elements for the PS include:

- Concrete slab-on-grade crane pad for pulling pumps
- Concrete slab-on-grade for stop gate storage
- Miscellaneous sidewalks from buildings to PS

12.8 PS Electrical and Control/Operations Buildings

The PS Electrical and Control/Operations Buildings will be two buildings that use pre-engineered precast building components to create a building structure. These buildings will be built on the embankments instead of over top of the pumping bays and will house electrical components for the adjacent PS. The Electrical Building will have a single 277/480-volt (V), 3-phase equipment room. The Control/Operations Building with three rooms consisting of the control room, low-voltage room, and generator room. They will have a preliminary size in compliance with SFWMD's current standard width of 14 ft, with 10- to 12-ft ceiling height and lengths to be approximately 42 ft for the Electrical Building and 34 ft for the Control/Operations Building. The EPI team will optimize these dimensions during design. The precast shells will consist of integral slab/vertical wall/roof segmental elements. The buildings will be supported by reinforced cast-in-place concrete footings and/or slab-on-grade.

12.9 Precast Control/Operations Building

The Control/Operations Building will house electrical and control components for the inflow and outflow WCS, WCS-9, and WCS-13. The Control Buildings will be supported by reinforced cast-in-place concrete footings and/or slab-on-grade.

Control Buildings for the 12 gated inflow/outflow structures will be SFWMD standard control buildings without standby generators and will be optimized to be 10 ft by 10 ft. The Control Building for WCS-9 will be SFWMD standard control building with standby power coming from the PS complex and will be optimized to be 14 ft by 14 ft. The Control Building for WCS-13 at the C-38 canal will be an SFWMD standard control building with standby generator and optimized to be 14 ft by 22 ft.

12.10 WCS-9

WCS-9 will be a cast-in-place concrete structure with a separate service bridge level and operation level for gate operation. WCS-9 will be designed with two stainless-steel roller gates operated via a mechanical hoist system. Each roller gate will be provided in three sections to facilitate delivery to the site and to allow SFWMD to remove them for maintenance using existing crane equipment. The gate sections will be bolted together to form complete operable gates. Flat channel slabs and interior piers will be based on SFWMD standard design details. Each channel will include a provision for isolating and dewatering individual bays using stop gates or needle beams.

12.11 PES

The only structures in the PES are the inflow slide gate box culverts and the outflow drain manholes. These standard structures will be designed in accordance with the structural standards outlined above. Design details and associated figures are outlined in the H&H TM located in Appendix 4.

12.12 Proposed Future Activities

12.12.1 Preliminary Design

The following activities are proposed for preliminary design:

- EIP will develop draft preliminary plans and specifications. Related to the structural design, the draft POM, DDR, and regulatory plan will be updated, and a construction schedule and stipulated price proposal will be provided.

- Develop strategy for accessing pumps through the PS canopy structure (access hatches, removable roof panels, or other).
- Develop strategy for protecting facilities from random arms fire.
- Determine best approach for isolating and dewatering the PS and large WCS bays (needle beams vs. stop logs vs. stop gates).
- Perform geotechnical investigations and develop GEDR.

12.12.2 Final Design

- No additional items are anticipated during final design.

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SITE CIVIL DESIGN

Civil design for the LKBSTA Project includes the layout of six STA treatment cells, one inflow pump station, an estimated 14 gated WCSs for controlling inflow to and outflow from the STA treatment cells, canals, and embankments. A general description of the civil design criteria and basis for design of these civil elements is presented in this section.

13.1 Site Access and Roadways

General access to the Project and associated structures will be limited to District and EIP staff and their guests during EIP ownership. EIP and District staff will have access to the project area to maintain Project assets, including all STA cells, WCSs, the PES, canals, embankments, and other appurtenances. The design will allow for space for future public access to the site through designated public access points.

Main site access will be from the northwest corner of the project off 128th Avenue. In general, all embankments will be designed to have a minimum crest width of 14 ft with a stabilized surface to allow for vehicular traffic along the top of the embankments, with access ramps and pullout areas (for turnaround and passing maneuvers) provided at the required intervals per DCM-4, along with cell access via boat ramps.

WCS-9 will include a vehicular access path to allow crossing of the L-62 canal from the western portion of the site to the eastern portion. A vehicular access path over WCS-13 will maintain continuous access along the northern HHD once construction is complete.

Preliminary cell layouts incorporate a minimum 120-ft turning radius for adequate travel of a semi-truck with a long-bed lowboy trailer along exterior embankments dedicated to main site access such as WCS-9, WCS-13, and the PES. Interior embankments will be designed with a minimum 60-ft turning radius to allow smaller maintenance vehicles, excavators, and dump trucks access to STA cells, STA WCSs, and appurtenances, such as airboat ramps and overflow spillways. Specific design vehicles based on anticipated needs will be evaluated as preliminary design progresses to address adequate movement throughout the site.

13.2 Site Layout

The Project will include a north-south reroute of the L-62 canal west of the existing alignment. At the southern end of the reroute at the tie into C-38 canal, a new WCS will allow the HHD to remain contiguous. A new WCS (WCS-9), PS, and two inflow canals feeding each of the six treatment cells and PES will be located near the intersection of the existing and relocated L-62 canal systems. On the west side of the reroute, an inflow canal located in the center portion of the STA runs from east to west to distribute flow to Cells 1 through 4, while a second inflow canal on the east side of the reroute runs from west to east and then south to feed influent to the PES and Cells 5 and 6.

The interior cell layout has Cells 1 and 2 routing treated flows from the center of the site to the north side of the STA, and Cells 3 through 6 routing flow to the south side of the site.

A Project site layout with graphical representations of the key project features is provided on Figure 13-1. Designs of embankments and canals are covered in Sections 10 and 11, respectively.

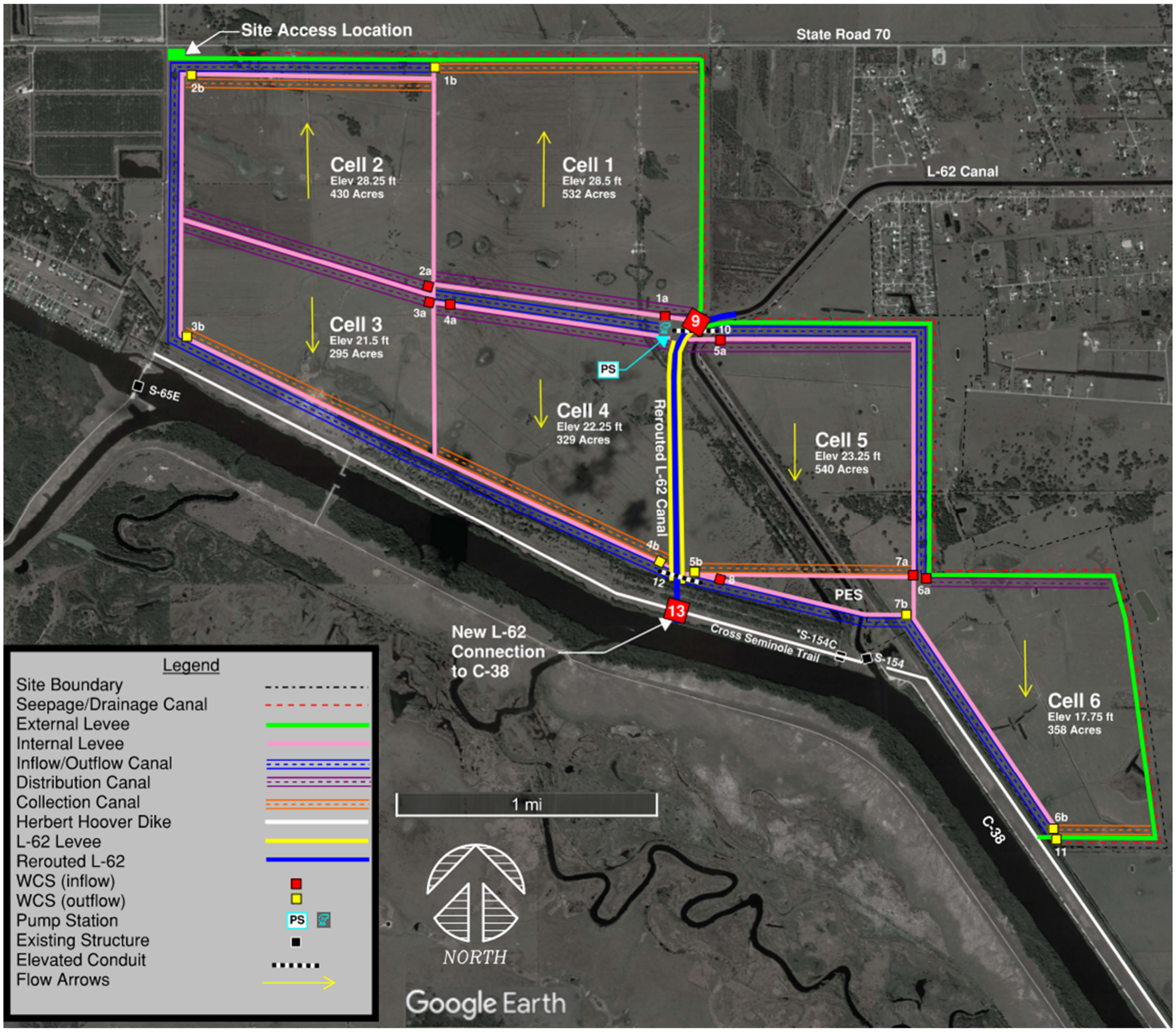

Figure 13-1. Project site layout

13.2.1 Headworks System

System influent is obtained from the relocated L-62 canal downstream of the new WCS-9 and distributed by the PS into two inflow canals fed by the PS discharge area. The western inflow canal provides influent for Cells 1 through 4, while the eastern inflow canal directs flow to Cells 5 and 6 via a conduit from the PS discharge area over the relocated L-62 canal. Flow from the western cells' seepage canal would be routed downstream of WCS-9 by gated conduit or overflow upstream of WCS-9 via a seepage structure.

The new WCS-9, located near the point of intersection with existing and rerouted L-62 canal, is designed to maintain current operations and functionality of the existing L-62 canal.

A draft layout of the PS and WCS-9 area with rerouted L-62 canal is shown on Figure 13-2.

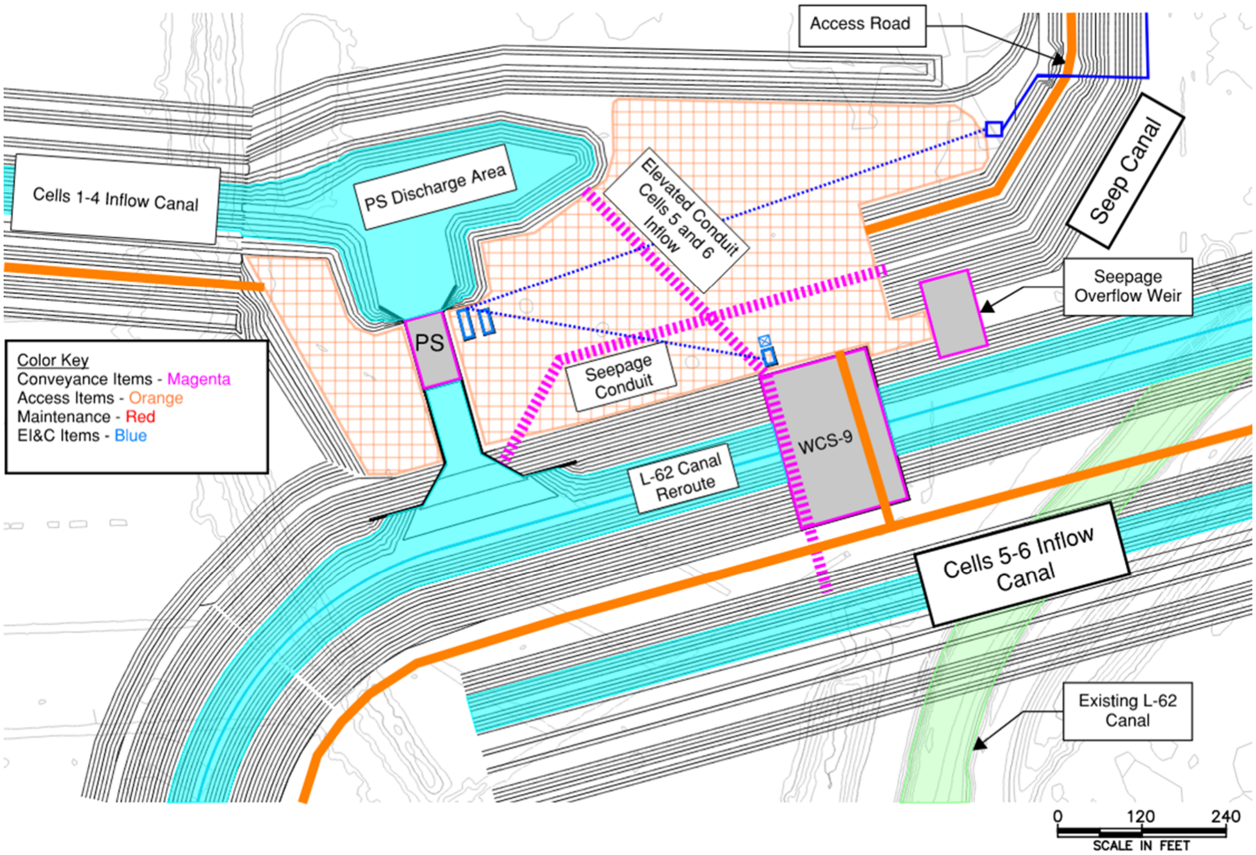

Figure 13-2. Proposed headworks system site layout

13.2.2 STA System

Between inflow and outflow canals, each STA cell includes a gated inflow WCS, a distribution canal, a graded cell, a collection canal, and a gated outflow WCS. The six cells are graded flat at the elevations shown previously on Figure 13-1 with the cell bottom starting at the edge of the distribution canal and ending at the edge of the collection canal. The grades shown represent a planned balance of material within each cell limit.

The PES is located east of the L-62 canal reroute between Cell 5 and the outflow canal.

13.2.3 Discharge System

Northern Cells 1 and 2 flow into an outflow canal running east to west along the northwestern side of the site. This outflow canal runs south along the western limits of the site before turning southeast and paralleling the HHD. Southern Cells 3 and 4 flow into this stretch of the outflow canal before flowing to the outflow canal east of rerouted L-62 canal via a conduit through the canal. Eastern Cells 5 and 6 flow into this outflow canal, which parallels the HHD with all outflow discharging into the C-38 canal through the existing S-154.

13.2.4 Seepage Control System

Seepage management for this layout requires seepage collection along the western, northern, and eastern perimeters of the project. The western cells seepage is controlled by two canals. Off the west side of Cell 2, a dual-seepage/outflow canal routes seepage to the Project discharge. The other western seepage canal runs from the north side of Cell 2 along the north and east side of Cell 1 back to the PS location for discharge downstream of WCS-9 by gated conduit or overflow upstream of

WCS-9 via a seepage overflow structure. Eastern cell seepage is controlled by a seepage canal running along the northern, eastern, and southern project limits and will discharge on the southern end of Cell 6 to the outflow canal via a WCS.

13.3 Stormwater Control/Site Drainage

The size and nature of this Project requires that stormwater be managed during construction. A conceptual Stormwater Pollution Prevention Plan (SWPPP) will be required as a part of the construction contract documents. The objective of the SWPPP is to prevent erosion where construction activities are occurring, prevent pollutants from mixing with stormwater, and prevent pollutants from being discharged by containing them onsite before they can affect the receiving waters. Contractors will be required to prepare and submit a comprehensive SWPPP tailored to their sequence of construction. Contractors will be provided conceptual plans, guidelines, and criteria so that detailed drainage plans for all phases and sequences of construction can be prepared.

The site grading around the PS area will include provisions for capturing and treating, where necessary, stormwater runoff. Stormwater calculations and facilities will be designed to comply with local and state guidelines and regulations. In the PS and WCS-9 area, stormwater piping and a stormwater treatment pond are anticipated with the pond outflows being conveyed to Cell 1. Outside of the PS and WCS-9 area, all levees and benches will be graded to direct stormwater runoff toward STA cells and canals.

13.4 Utilities

Required site utilities will be evaluated and included as part of preliminary design. Proposed site utilities include provisions for potable and non-potable water, fire protection, sanitary and solid waste disposal and power and communications as described in Section 15.

13.5 Communications Appurtenances

A communications tower will be located near the PS, with telemetry poles located near all other WCSs. See Section 15 – Instrumentation and Controls for more information.

13.6 PES

The site layout can be found on Figure 13-1. In addition to staging and blending areas near the PES, an area for temporary diesel pumps placement will be designed. Further information on the monitoring setup is addressed in Section 16 of this DDR. Electric power provided to WCS-7A will be extended to the slide gates and drain control valves, which have low-current requirements. The EIP team will revise the design as needed based on feedback and any changes to the PES layout.

13.7 Proposed Future Activities

13.7.1 Preliminary Design

The following activities are proposed for preliminary design:

- EIP will develop draft preliminary plans and specifications. Related to the site civil design, the draft POM, DDR, and regulatory plan will be updated, and a construction schedule and stipulated price proposal will be provided.
- Design vehicles will be finalized for turning radius analysis and access design will be finalized.
- Stormwater management design for internal sites will be advanced.
- Site perimeter buffers will be finalized based on updated property boundary information.

- Site utility options will be further evaluated and designed in accordance with SFWMD standards and local requirements.

13.7.2 Final Design

The following activities are proposed for final design:

- EIP will advance plans and specifications.
- Fine grading and specifications for the stabilization of the site will be completed.
- Stormwater management design for internal sites will be advanced.
- Site perimeter buffers will be finalized based on updated property boundary information.

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MECHANICAL DESIGN

Mechanical design for the LKBSTA Project will focus on moving and controlling flow throughout the project site. This section describes the design of water control facilities including STA inflow and outflow structures, WCS-9, WCS-13, seepage gates, and PES inflow and outflow structures; and two PSs including the main project PS and a temporary series flow PS used during startup. A general description of the design criteria and codes that will be used during design is included in this section and future activities are identified to progress the Mechanical Design to the Final Design.

14.1 Mechanical Design Criteria and Codes

The LKBSTA Project will be designed to the latest SFWMD standards and guidelines, taking into account current federal, state and local codes and regulations applicable to the project. Applicable standards, publications, and guidelines include but are not limited to the following:

- PS Engineering Guidelines, South Florida Water Management District, January 2021
- SFWMD PS Design Criteria Memoranda
- SFWMD PS Design Guideline Drawings
- SFWMD Design Standards
- USACE Engineering Manuals and Regulations EM1110-2-3102 General Principals of Pumping Station Design and Layout
- USACE Engineering Manuals and Regulations EM1110-2-3105 Mechanical and Electrical Design of PSs
- USACE ETL 1110-2-584 Design of Hydraulic Steel Structures
- ANSI/HI 9.6.1 Rotodynamic Pumps Guideline for NPSH Margin
- ANSI/HI 9.6.3 Rotodynamic (Centrifugal and Vertical) Pumps – Guidelines for Allowable Operating Region
- ANSI/HI 9.6.6 Rotodynamic Pumps for Pump Piping
- ANSI/HI 9.8 Pump Intake Design
- NFPA 54 – National Fuel Gas Code (2018)
- AWWA C561 – Fabricated Stainless Steel Slide Gates
- ASTM A36, A148, A240, A, 276, A314, A380, A564, A743, B22, D395 D412, D471, D2000, D2240 F593, F594
- AWS D1.1, D1.5, D1.6
- AISC Manual of Steel Construction
- ANSI B1.1, B46.1

14.2 Water Control Facilities

The design objective of the water control facilities is to control the flow of water into and through the project site. Several different types of structures will be used to accomplish this goal depending on the flow rate being conveyed, the amount of differential head required across the control structure, and if the flow is uni or bidirectional. The water control facilities will be designed in accordance with

SFWMD standards as much as possible. Hydraulic sizing for these structures is based on the peak flows as described in the Hydraulics Section of this report. The structure locations are shown in Figure 14-1 below. Structural design is as described in Section 12.

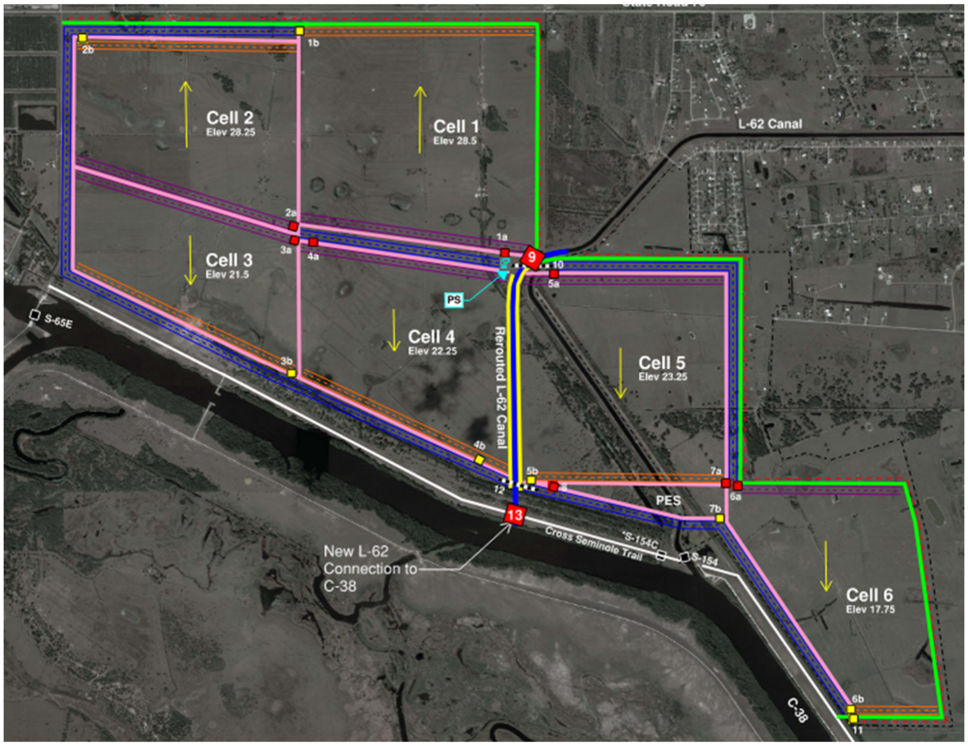

Figure 14-1 Locations of Project WCSs

14.2.1 STA Inlet and Outlet Water Control Facilities

Each STA cell will have a dual barrel and gated inlet and outlet WCS used to maintain constant stages through the STA cells. A typical plan and section of the inlet and outlet water control facility is shown in Figures 14-2 and 14-3. The structures will be designed to pass the design flow without exceeding the upstream flood design stage and will restrict downstream velocities to prevent downstream canal damage. They will also be used to prevent backflow during excessive stages. When operating, the gates will open from the bottom up allowing the water to flow under the gate opening in a fully submerged condition. The slide gates will be mounted on a center wall in a reinforced concrete box structure constructed on the treatment cell berms and will either be to allow water from the distribution canal into the treatment cells, or to allow the water from the treatment cells into the canal. The WCS box will provide the ability to dewater and access the slide gates from either side of the slide gate mounting wall in order to perform ongoing gate maintenance. Double barrel high density polyethylene pipe conduits will link the gate structure with the distribution and discharge canals. Each gate will be fitted with an electric actuator and position indicator that can be read from both the deck and the control building.

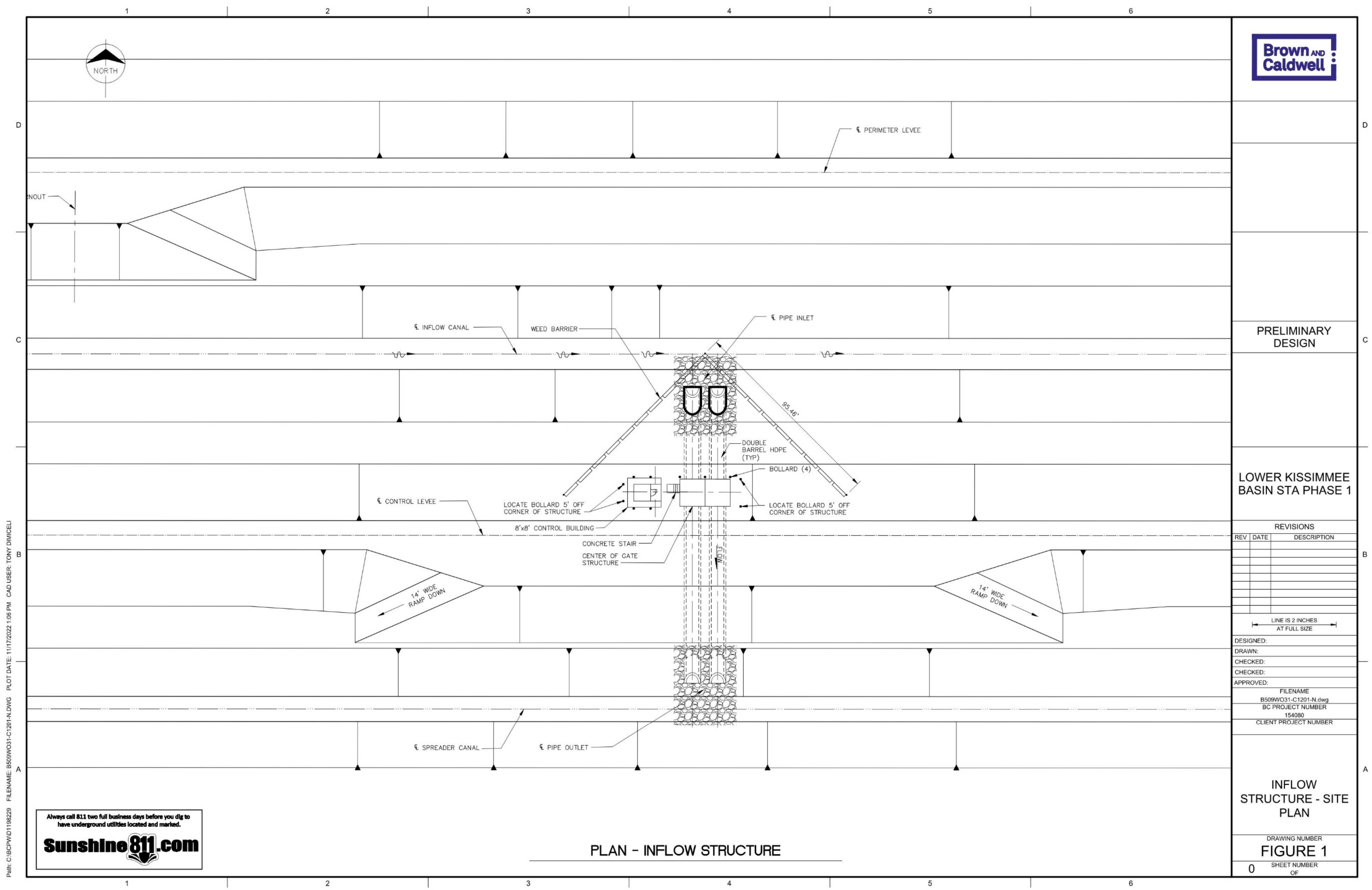

Figure 14-2. Inflow Structure – Site Plan

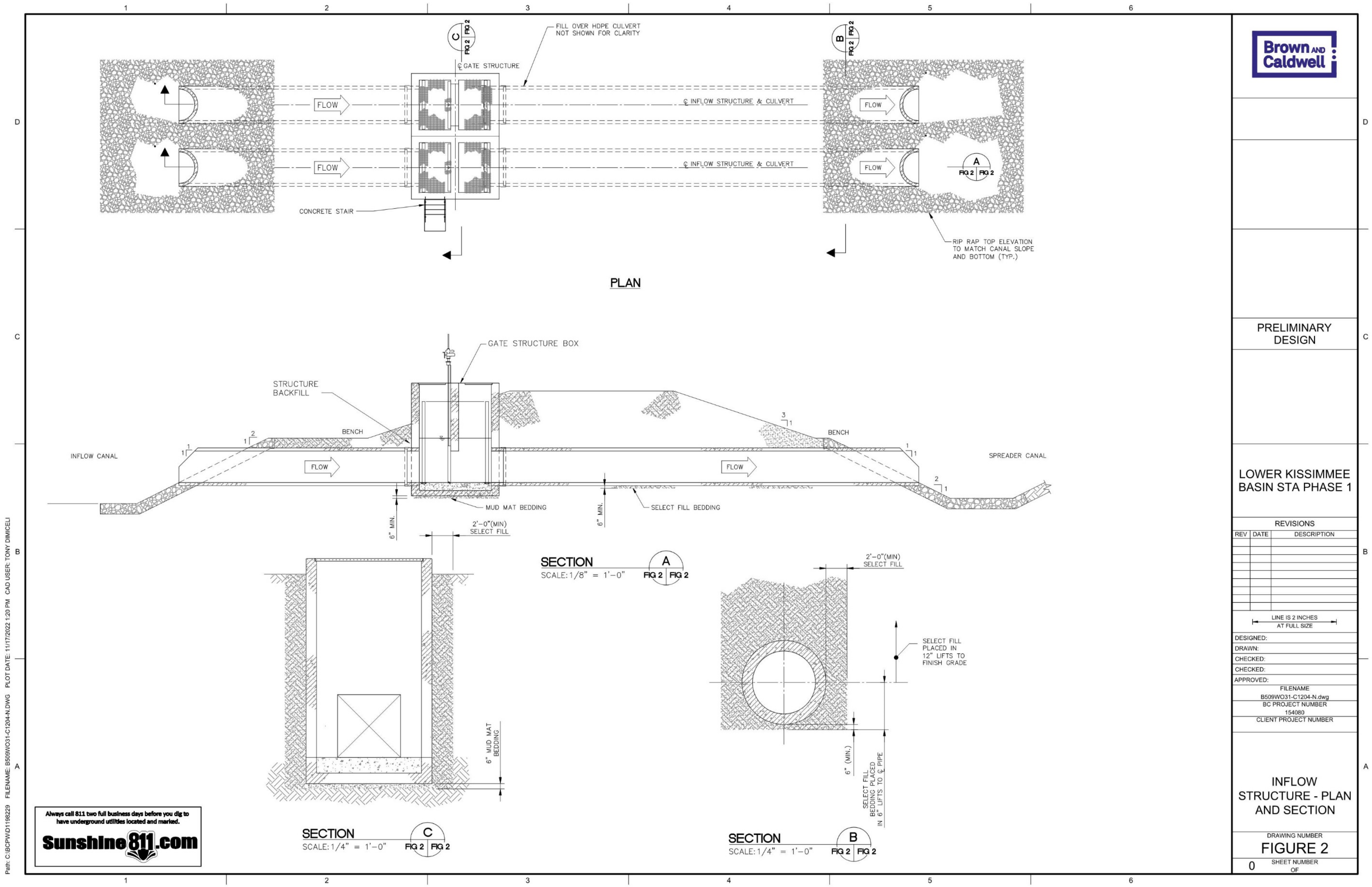

Figure 14-3. Inflow Structure – Plan and Section

The design conditions for the STA inlet and outlet water control facilities are outlined in Table 14-1 and are based on the results of the hydraulic evaluation in Section 8 of this report.

Table 14-1. Design Conditions

Structure Name	Flow Direction	HW Elevation (ft-NGVD)	TW Elevation (ft-NGVD)	Headloss (ft)	Required Gate Opening (ft)	Calculated Flow (cfs) per Structure	Comments
WCS-1A	Inflow	31.85	31.19	1.66	2.9	92.02	From Inflow Canal to STA Cell 1
WCS-1B	Outflow	30.07	24.62	5.45	1.35	92.02	From STA Cell 1 to Discharge Canal
WCS-2A	Inflow	31.65	29.99	1.66	2.05	76.30	From Inflow Canal to STA Cell 2
WCS-2B	Outflow	29.89	23.62	6.27	1.05	76.30	From STA Cell 2 to Discharge Canal
WCS-3A	Inflow	31.65	23.26	8.39	0.75	58.46	From Inflow Canal to STA Cell 3
WCS-3B	Outflow	23.22	20.77	2.45	1.31	58.46	From STA Cell 3 to Discharge Canal
WCS-4A	Inflow	31.66	23.90	7.76	1.05	80.66	From Inflow Canal to STA Cell 4
WCS-4B	Outflow	23.8	20.4	3.40	1.30	80.66	From STA Cell 4 to Discharge Canal
WCS-5A	Inflow	27.80	25.10	2.70	2.25	103.00	From Inflow Canal to STA Cell 5
WCS-5B	Outflow	25.04	19.94	5.10	1.50	103.00	From STA Cell 5 to Discharge Canal
WCS-6A	Inflow	23.86	19.56	4.30	1.10	64.00	From Inflow Canal to STA Cell 6
WCS-6B	Outflow	19.54	14.80	4.74	1.05	64.00	From STA Cell 6 to Discharge Canal

14.2.2 WCS-9 Facility

WCS-9 will include dual roller gates used to restrict the flow from the L-62 canal to the C-38 canal. The structure will have the ability to be seated by the design differential water pressure head in both directions. Each gate will be fitted with a position indicator that can be read from both the bridge deck and the control building. The gate channel will be fitted with a trapezoidal weir above the main channel.

The design conditions for WCS-9 are outlined in Table 14-2 below and are based on the results of the hydraulic evaluation in Section 8 of this report.

Table 14-2. Design Conditions							
Structure Name	Flow Direction	HW Elevation (ft-NGVD)	TW Elevation (ft-NGVD)	Headloss (ft)	Required Gate Opening (ft)	Calculated Flow (cfs) per structure	Comments
WCS-9	Outflow	22 (max at G-80)	17.9 - Wet weather 9.5 - Dry Weather	4.1 - Wet Weather 12.5 - Dry Weather	8 - Fully Open – Wet Weather Fully Closed – Dry Weather	1,000 – Wet Weather 0 – Dry Weather	From L62 Canal to PS

14.2.3 WCS-13 Facility

WCS-13 will be a structure with a slide gate located on the C-38 Canal downstream of WCS-9 on the HHD. The primary function of this structure will be to avoid extending the HHD up to WCS-9. The structure will have the same capacity as the existing S154 but will be closed for normal operations. The structure will consist of a concrete box with two concrete box conduits built directly on the dike each with a design capacity of 500 cfs. The structure will be capable of conveying a flow of 3,333 cfs without exceeding a water level of 30 ft immediately upstream of the structure. Each gate will be fitted with an electric actuator and position indicator that can be read from both the deck and the control building.

14.2.4 Seepage Gated Conduits

Seepage from the treatment system cells will be collected via the Seepage canals running along the site perimeter. Seepage captured on the west side of the of the L-62 canal will be collected in a concrete structure from which a gated conduit will convey flows to a point downstream of WCS-9 and as close as possible to the intake of the project PS. The seepage conveyance system will have a capacity of approximately 30 cfs, with excess flows discharging via an overflow weir upstream of WCS-9.

14.2.5 STA Inlet Elevated Crossing

The STA Elevated Conduit will be designed to convey water from the inflow canal to treatment cells 5 and 6. Flow to these cells will cross over the L-62 canal via an elevated crossing which will be 7 ft wide and supported from WCS-9. The elevated crossing will have slide gates for isolation.

14.2.6 PES Water Control Facilities

PES Water control facilities will regulate the flow in and out of the PES. These will consist of two inflow structures; WCS-7a will convey flows from the inflow canal to cells 5 and 6, and WCS-8 will convey flows from the discharge canal from cells 1 to 4. WCS-7b will allow for discharge flows from the PES and ultimately discharge into the C-38 canal.

The Design Conditions for the for the PES water control facilities are outlined in Table 14-3 below and are based on the results of the hydraulic evaluation in Section 8 of this report.

Table 14-3. Design Conditions							
Structure Name	Flow Direction	HW Elevation (ft-NGVD)	TW Elevation (ft-NGVD)	Headloss (ft)	Required Gate Opening (ft)	Calculated Flow (cfs) per structure	Comments
WCS-7A	Inflow	23.86	14.35	9.51	1.75	19.00	From Inflow Canal to PES Facility

14.2.7 Slide Gates

The design criteria for slide gates shall conform to requirements outlined in Tables 14-4, 14-5 and 14-6 and below.

Table 14-4. Gate – Structure and Geometry					
Structure Name	Number of Gates	Conduit Diameter (in)	Gate Dimensions (ft)	Invert EL (ft-NGVD)	Max Flow Per Conduit (cfs)
WCS-1A	2	42	3.5 x 3.5	24.5	78
WCS-1B	2	42	3.5 x 3.5	24.0	78
WCS-2A	2	42	3.5 x 3.5	24.25	78
WCS-2B	2	42	3.5 x 3.5	23.75	78
WCS-3A	2	36	3 x 3	24.55	56
WCS-3B	2	36	3 x 3	17.00	56
WCS-4A	2	42	3.5 x 3.5	24.65	78
WCS-4B	2	42	3.5 x 3.5	17.75	78
WCS-5A	2	42	3.5 x 3.5	19.25	78
WCS-5B	2	42	3.5 x 3.5	18.75	78
WCS-6A	2	36	3 x 3	15.00	56
WCS-6B	2	36	3 x 3	13.25	56

Table 14-5. Gate – Structure and Geometry (WCS-13)					
Structure Name	Number of Gates	Conduit Dimensions (ft)	Gate Dimensions (ft)	Invert EL (ft-NGVD)	Max Design Flow Per Conduit (cfs)
WCS-13	2	12 x 12	12 x 12	-2	500

Table 14-6. Slide Gates – Materials and Requirements	
Criteria	**Value**
Gate Type	Upward Opening Sluice Gate
Gate Assembly	Self-Contained
Gate Materials (Slide, Frame, Stem, Fasteners, Yoke, and Enclosure)	316 Stainless Steel L
Frame Mounting Type	Flange Back or Channel Mount
Frame Design Safety Factor	5 - regarding Ultimate Tensile, compressive and shear strength
Seating Faces and Seals	Flush Bottom UHMW PE
Maximum Allowable Velocity	8 ft/s
Maximum Allowable Head	4.8 inches at design flow
Maximum Allowable Leakage Rate	Seating - 0.05 gpm per linear ft Unseating – 0.10 gpm per linear ft
Actuation	Gate travel – Minimum of 6 inches per minute at 250 RPM Maximum Pull Force – 40 lbs or 80 ft-lb torque after gate is unseated. (When manual operation is required)
Stop gates	Gates shall have embedded stop gate frames for secondary isolation.
Coatings	When required, all gate materials shall be factory coated for outdoor weather exposure

14.2.8 Knife Gate Valves

Knife gate valves used to control the flow into and out of the PES will meet the requirements specified on Table 14-7 below:

Table 14-7. Knife Gates – Materials and Requirements	
Criteria	**Value**
Gate Type	Knife gate
Gate materials (Body, Gate)	316 Stainless Steel
Packing	PTFE impregnated synthetic fiber
Seat Rings	Buna N, FKM, Neoprene or EDPM
Stem	316 Stainless Steel
Actuator	Motor Operated, Bonnetless rising stem
Size	30 inches

14.2.9 Roller Gates

Roller gates used to restrict the flow through the L-62 canal shall be made of stainless steel and operated via a mechanical hoisting system mounted on an elevated platform. The gates will be fitted with a cable attachment point on each side to allow the cables to be anchored for opening and closing.

The design criteria for roller gates shall conform to requirements outlined in Tables 14-8 and 14-9 and below.

Table 14-8. Gate – Structure and Geometry						
Structure Name	Number of Gates	Channel Elevation (ft-NGVD)	Gate Dimensions (ft x ft)	Weir Crest EL (ft-NGVD)	Max Flow (cfs) Total	Max Flow Velocity (ft/s)
WCS-9	2	7	15 x 8	10	3,300	8

Table 14-9. Roller Gates – Materials and Requirements	
Criteria	Value
Gate Type	Downward Opening Roller Gate, sealable in either direction, embedded type
Gate materials (Rollers Guide Rails, Disc Plate, Disc Frame, Sealing Surfaces)	316 Stainless Steel L
Gate Materials (Studs, Anchors, and Assembly Bolts)	316 Stainless Steel
Gate Sections	Gate sections shall not be more than 10 ft in height and the weight shall not exceed 14,000 lbs
Gate Storage	Gate Slots below hoist deck
Gate Closing	Self-Closing with sufficient ballast under all operating conditions
Operator Type	Wire Rope/Cable Drum Hoist Operable from a control panel. Gate shall be operable under unbalanced conditions
Gate Section Height	Max 10 ft
Seals	Molded Neoprene Rubber (Both on the gate and on the embedment)
Roller	Removable axle or shaft
Maximum Allowable Leakage Rate	0.01 gpm per linear ft in either direction
Actuation	Motor Operated Gate travel – Minimum of 6 inches per minute at 250 RPM Maximum Pull Force – 40 lbs or 80 ft-lb torque after gate is unseated (when manual operation is required)

14.2.10 PS

14.2.10.1 Design Objective

One PS will be constructed as part of the LKBSTA project. The pump equipment will be designed for continuous service but also capable of immediate automatic or manual start-up and intermittent operation. The pump equipment including auxiliaries will be designed and constructed for a minimum service life of 25 years excluding normal wear parts. The estimated average annual operating time should average approximately 7,000 hours. The characteristics of flow to the pumps includes storm water that is expected to contain sand, silt, and floating or transported debris capable of passing the trash rack. Water temperature is expected to be in the range of 50 to 100 degrees F.

The pump will be designed to facilitate routine and heavy maintenance. ANSI/HI 14.4 provides guidance for the installation, operation, and maintenance of vertical pumps. Major parts, such as the bowl components, will be designed and manufactured to ensure accurate alignment on reassembly. The pumps in this facility will operate through formed suction inlets, requiring maintenance to be performed from the pump deck. The PS will be located on the rerouted portion of the L-62 canal to direct water from the L-62 canal to the STA. The PS will be designed to convey up to 500 cfs of water from the L-62 canal to the STA Inflow/Outflow canal.

14.2.10.2 Basis of Design

The current basis of design for the PS includes four identical pumps, each with a rated capacity of 125 cfs. All four pumps will be in operation during peak flow conditions. Backup pumping capability will not be provided.

As the design progresses the number of pumps and need to include a low flow pump will be evaluated.

A physical model study conforming to the requirements of ANSI-HI 9.8 will be conducted to confirm the intake design.

Basis of Design information for the PS is presented in Table 14-10.

Table 14-10. Mechanical Basis of Design for PS

Parameter	
Pumps	
Number	4
Type	Vertical mixed flow, electric motor driven
Potential Manufacturers	Flygt/Xylem, Morrison
Flow at primary design point, each pump	125 cfs
TH at primary design point	27.8 ft
NPSHr at primary design point	17 ft
NPSHa at primary design point	34 ft
Bowl efficiency at design point	87%
Motor, HP	600
Preferred operating range (POR)	85% to 115% of BEP
Intake Bar Screen/belt Conveyor	
Manufacturer	Duperon, D&J Machinery
Type	Mechanical, front-cleaning/front return
Bar Spacing	3 inches
Flow	125 cfs
Headloss, max	1 inch
Screen Motor, HP	1
Conveyor Motor, HP	7.5
Backflow Control	Flapgate, Dewatering Needle

The pumps will be constant speed, vertical mixed flow pumps. The pumps will each have a separate 11-ft wide inlet bay and will discharge through a 48-inch diameter stainless steel flap gate into a 11-ft wide discharge channel. Provision for installation of dewatering needles in the inlet bay and discharge channel will be included in the design as a means of secondary backflow control.

The bottom of the PS inlet bay will be set at -5 ft NAVD88, approximately 9-ft below the bottom of the rerouted L-62 canal. The inlet layout to each pump will be based on the USACE Type 10 Formed Suction Intake.

A self-cleaning, mechanical bar screen will be provided at the entrance to each pump intake channel to remove debris 3-inches and larger from the water prior to pumping. The screen will span the full width of the channel and extend to grade. A single belt conveyor will be provided to receive screenings from each of the bar screens and convey them to a containment area for dewatering, storage, and removal. The bar screens will be of the front-cleaning/front-return (Flex Rake) type as manufactured by Duperon, or similar as manufactured by D&J Machinery, without lower sprockets or bearings below the liquid level. Belt conveyor will be provided by the same manufacturer of the bar screen.

A canopy will be provided over the PS to provide shade and cover from rain. The canopy will extend over the pumps and include hatches above each pump to allow for pump removal.

As noted above, the purpose of the PS is to deliver water to the distribution canal feeding the STA. Boundary conditions in the L-62 canal are as follows:

- Low water level: +7 ft NAVD88
- High water level: +15 ft NAVD88

The boundary conditions in the STA distribution canal are as follows:

- Low water level: +29.9 ft NAVD88
- High water level: +32 ft NAVD88
- Flooded STA water level: +34.25 ft NAVD88

The design pool to pool water elevation will be based on the difference between the WSE in the intake bay and the WSE in the distribution canal plus piping and valve headlosses.

The WSEs in the intake bay are estimated to range from a low of about +6.5 ft NAVD88 to a high of about +15 ft NAVD88. The low-level elevation is based on 6 inches of head loss through a partially blinded screen when the WSE in the L-62 canal upstream of the PS is at +7 ft NAVD88. The high-level elevation is equal to a WSE of +15 ft NAVD88 in the L-62 canal upstream of the PS with 0.0 ft of head loss through a clean screen.

The WSEs in the inflow canal are estimated to range from a low of 29.9 ft NAVD88 to a high of about 32 ft NAVD88. A flooded STA condition may also occur during periods of low water availability to store additional water, during these periods the WSE in the inflow canal will be approximately 2.25 ft above the high water level.

The primary design condition is based on a low water level in the L-62 canal and high water level in the inflow canal. The primary design condition will be rated at 500 cfs.

The primary design condition lands within the POR, as defined by ANSI/HI 9.6.3, for preliminary pump selections supplied by Flygt/Xylem and Morrison.

14.2.10.3 Standby Power and Fuel Systems

The PS will not be provided with emergency standby power to run the pumps during power outages but will be equipped with a 45-kW generator and liquified petroleum gas (LPG) fuel system to power SCADA, lights, and HVAC for the electrical and control buildings. The generator will also provide emergency standby power to WCS-9 for operation of the gate and SCADA, lights, and HVAC for the control building. The LPG generator fuel system will consist of a buried storage tank and associated piping, valves, instrumentation, and appurtenances. Basis of Design information is presented in Table 14-11.

A 1,500 gallon below grade cylindrical LPG storage tank will be provided with sufficient capacity for 7 days of generator operation at 100% load. A fill connection port will be located at the tank for fuel delivery by truck. The fuel pressure inside the tank will be 70.6 psig at 45° F and will be regulated down to the SFWMD standard 10 psig, which will provide a workable pressure through the system.

Exhaust Piping: Generator exhaust will be conveyed from the building via 3" Schedule 10s 304 SST pipe with blanket insulation. The pipe material was selected to withstand the high heat loads (1,376°F) produced by the engine exhaust. A critical grade silencer, insulated wall thimble, condensate drains, and rain cap will be provided for the exhaust system.

Table 14-11. LPG Fuel System Equipment Table

Equipment	Description
Bulk Fuel Tank	
Type	Below Ground, Cylindrical, Steel
Volume (gal)	1,500
Generator	
Rating	45 kW
Fuel Consumption (scfh)	187.8
Exhaust Gas Flow (cfm)	375.2
Maximum System Back Pressure (kPa)	4.0
Voltage	120/206V, 3PH
Exhaust Silencer	
Size (in)	3
Noise Attenuation (dB A)	25-35
Pressure Loss (in W.C.)	4

14.3 PES (WCS-7A and 7B)

The only mechanical devices associated with PES are the inlet and outlet WCS discussed above and in Section 8.5. These WCS will be designed in accordance with the mechanical standards outlined above and required to control flow into and out of the PES system.

14.4 Temporary Series Flow PS

During Preliminary Design, our team will evaluate the value of providing the capability to pump STA effluent through the PES system as a polishing step during startup activities to minimize the discharge of any phosphorus during startup or re-startup activities. To accomplish this desire, an additional WCS would be provided in the STA effluent canal just south of the PES system. Additionally, a flat, hardscaped temporary diesel pump staging area would be provided to set up temporary diesel pumps, pump fuel supply system and suction and discharge piping and valves. STA effluent would be intercepted at the proposed additional WCS and pumped to the PES inlet canal. Other improvements may be required in the east STA effluent canal system in order to alleviate treated effluent short circuiting in the system during the series pumping process. It is estimated the pumping system would be designed to pump approximately 30 cfs of STA effluent flow. The temporary pumps and piping systems would be procured as part of the project and then ultimately turned over to the District to use when needed.

14.5 Proposed Future Activities

14.5.1 Preliminary Design

Items to be completed during the Preliminary Design are identified below:

- In the Preliminary Design phase, EIP will develop draft preliminary plans and specifications. In addition to these technical specifications, related to the mechanical design, the draft POM, DDR, and regulatory plan will be updated, and a construction schedule and stipulated price proposal will be provided.
- Confirm pump sizing and make a final decision on the need for a low flow pump.
- Finalize PS hydraulics and select pumps.
- Confirm sizing for all STA WCS.
- Confirm sizing for WCS-9.
- Confirm sizing and design for PES WCS.
- Determine the design requirements for WCS-13.
- Determine the design requirements for seepage flow control structures.
- Determine the design requirements for west to east STA inlet flow control conduit.
- Consider implementation of overflow slide gates at specific WCS.
- Coordinate with CFD modelers on PS and WCS modeling and design.
- Security fencing extents, location and size of access gates, and general public restrictions around the PS.
- Determine if a series flow temporary pumping system is desired/necessary.
- Determine on-site storage requirements for maintenance items such as diesel pumps and associated piping and valves, stop logs, needle beams, spare parts, etc.

14.5.2 Final Design

Items to be completed during the Final Design are identified below:

- In the Final Design phase, EIP will advance plans and specifications.
- Physical model study in accordance with ANSI/HI 9.8 for the selected pumps.

SECTION 15

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SECTION 15

ELECTRICAL DESIGN

The sub-sections below cover the power system distribution and components that will be procured by EIP; 69 kilovolt (kV) will be distributed throughout the site. FPL will run power initially overhead and or below grade to the WCS; coordination with FPL is ongoing. A few WCSs will use a single transformer to sub-feed power to, at most, three WCSs for a more cost-effective and simpler power distribution scheme. The PS, being the largest load, will use a 3,000-kilovolt-amperes (KVA) transformer for primary power. Further details behind the power distribution for the major loads, namely the 600-horsepower (hp) pumps, the WCS-9 structure, lighting, SCADA, and other house loads, as well as the generator back-up power, are covered in the sections below. The pumps use solid-state reduced-voltage soft starters (RVSS) each with a full National Electrical Manufacturers Association (NEMA)-rated full-bypass starter. The PS features an automatic transfer switch (ATS) to switch to backup power for the life safety (i.e., HVAC), lighting, and SCADA remote telemetry unit (RTU). The PS also features a manual transfer switch (MTS) to switch off utility power and onto a portable generator. The PS control and electrical buildings will be of pre-cast construction featuring a 480V electrical room (main disconnect, switchboard, pump RVSSs), control room (bar screen control panel, PLC control panel, RTU), generator room, and IT room (air-conditioned space with an operator workstation computer). The sub-sections also cover the WCS control rooms' gate control panels, RTU, and auxiliary plug receptacle for a portable generator. Additional details regarding the electrical installation are covered in the sub-sections below.

15.1 Design Criteria

15.1.1 Applicable Standards and Publications, and Codes

The facility's electrical system design will conform, but not be limited, to the latest editions of the codes, applicable standards, and recommended practices of the following organizations:

- Institute of Electrical and Electronics Engineers (IEEE)
- American National Standards Institute (ANSI)
- Underwriters Laboratories, Inc. (UL)
- National Electrical Manufacturers Association (NEMA)
- Florida Fire Prevention Code (FPC)
- National Fire Protection Association (NFPA):
  - NFPA 70 – National Electrical Code (NEC) 2020
  - NFPA 101 - Life Safety Code
  - NFPA 110 –Standard for Emergency and Standby Power Systems

15.1.2 Electrical

The Project's electrical design includes one PS and 13 WCSs to be serviced directly by the electrical utility. WCS-9 will be powered from the PS. The electrical designs for the PS and control structures are based on SFWMD Standards and Details modified to meet the individual requirements of each of the PS and control structures. The electric PS design is based on SFWMD's Pumping Station Engineering Guidelines. The control structures, depending on the size and location, are either based on electric slide gates or vertical lift roller gates. Electrical service calculations, conduit sizing, and cable sizing are based on NEC requirements. The new PS will be serviced by the electrical utility from new FPL overhead or below-grade power distribution lines located along the PS and levee and/or embankment access road. At the PS, FLP will provide the high-voltage cables from its service pole to an FPL-supplied step-down pad-mounted transformer to provide the 480V, three-phase electrical service to the new PS. FPL utility metering shall be via current transformers (CT) located inside the low-voltage wiring compartment of the utility transformer with the utility meter mounted adjacent to the utility transformer.

The distribution system will be serviced by FPL at 69 kV, three-phase, 60 Hertz (Hz) with primary metering. This FPL primary will be routed from the existing FPL 69 kV overhead line located near SR 70. It will extend overhead and/or below grade; EPI is presently coordinating with FPL on the installation. The utility line will be routed along the levee and/or embankment access road and at the PS; FPL will provide the high-voltage cables from the utility service pole to an FPL- supplied step-down pad-mounted transformer to provide the 480V, three-phase electrical service to the new PS. FPL utility metering shall be via current transformers located inside the low-voltage wiring compartment of the utility transformer with the utility meter mounted adjacent to the utility transformer. The FPL-owned 69 kV to 480V step-down, pad-mounted transformer will provide 480V, three-phase, 60 Hz power to the PS. In general, station equipment voltages will be specified to operate as follows:

- Motors rated 600 hp 480V, 3 phase *
- Motors rated 0.5 hp to 600 hp 480V, 3 phase
- Motors less than 0.5 hp 120V, 1 phase
- Lighting 120V, 1 phase
- Convenience receptacles 120V, 1 phase

**At or > 600 hp will require use of 4160 volts alternating current (VAC) – to be finalized during preliminary design*

15.2 Gate/Valve Operators and Controls

Each WCS has two electric-motor-operated gates. Each WCS will be serviced directly by the electrical utility. WCS-9 will be powered from the PS. The electrical design will be based on SFWMD Standards for slide gate and roller gate structures applying the standard SFWMD details modified to meet the requirements of the control structure. Electrical service calculations, conduit sizing, and cable sizing are based on NEC requirements.

15.2.1 Electrical Service

WCSs will be serviced directly by FPL from a new distribution substation with new utilities installed. High-voltage power distribution lines will be located along the control structure/levee and/or embankment access road, either overhead or underground, as coordinated with FPL. At each WCS, FPL will connect the high-voltage power distribution lines to an FPL-supplied step-down transformer to provide the 120/240V, single-phase electrical service to the WCS. The utility metering shall be a n FPL-standard NEMA 3R meter can for single-phase 120/240V service.

15.2.2 Structure Electrical System

The 120/240V, single-phase electrical system for each WCS consists of a service-entrance-rated fused MTS, fed from the utility meter. The MTS feeds the structure distribution panel, which provides power for the slide gate motors, RTU, exhaust fans, lighting, and receptacles.

15.2.3 Standby Power

Each slide gate water control structure is equipped with an MTS with a generator receptacle to connect a portable backup emergency generator in the event power is lost.

15.2.4 Lighting

All control structure interior and exterior lighting fixtures shall be LED type. All interior lighting shall be controlled via light switches. All exterior lighting shall be controlled via light switches and lighting fixture integral photocells. Interior and exterior lighting levels shall be per SFWMD Requirements

15.2.5 Controls

The WCS gate operators will have line-powered drive motor and limit switches; control power transformer; open, stop, and close pushbuttons; and local-remote selector switch. The operator voltage and number of phases will be determined based on the available power at each site. A local-mounted safety disconnect switch pushbutton station will be provided near each operator.

The valve operators will be powered with 240V, single-phase power and have an integral reversing starter; limit switches; control power transformer; open, stop, and close pushbuttons; and local-remote selector switch. A local-mounted safety disconnect switch will be provided near each valve motor actuator.

15.3 WCS-9 and WCS-13

WCS-9 has two electric-motor-operated roller gates. WCS-9 will be powered from the PS with 120/240V, single-phase, 60 Hz power.

WCS-13 will be powered from FPL and will include a standby generator. FPL power will be 120/240V, 60 Hz power.

The electrical design will be based on SFWMD Standards for roller and/or slide gate structures and standard SFWMD details modified to meet the requirements of the control structure. Electrical service calculations, conduit sizing, and cable sizing are based on NEC requirements.

15.3.1 Electrical Service

WCS-9 will receive power from the nearby PS panelboard. WCS-13 will receive power from FPL from a pad-mounted transformer. At WCS-9 and WCS-13, the incoming power will be 120/240V, single-phase electrical service to the WCS.

15.3.2 Structure Electrical System

The 120/240V, single-phase electrical system for the WCS consists of a service-entrance-rated fused disconnect switch. The service disconnect switch feeds an MTS. The MTS feeds the structure MTS and the distribution panel, which provides power for the roller gate motor, RTU, exhaust fans, lighting, and receptacles.

15.3.3 Standby Power

The roller gate WCS-9 receives 120/240V power from the PS, which is also backed by a propane generator. WCS-13 will have a propane generator connected to an ATS. Each WCS has an MTS connected to incoming power.

15.4 PS Engineering Guidelines

15.4.1 Station Electrical System

The 480V, three-phase PS electrical system consists of a service-entrance-rated main breaker fed from the utility transformer; this main breaker feeds a switchboard. The switchboard is equipped with circuit breakers that feed four electric pump soft starters, trash screens, a trash conveyor, WCSs, a lighting step-down transformer from 480V to 120/208V, 3-phase, and a transient voltage surge suppression device. The service-entrance-rated main breaker and selected circuit breakers in the switchboard are equipped with adjustable settings for long- short-, instantaneous-, and ground-fault (LSIG) amperage pick up. The LSIG breakers shall be adjusted and set per the electrical device coordination study to provide a coordinated electrical system.

The lighting step down transformer feeds the lighting panel via an automatic transfer switch that is connected to an emergency backup generator. The lighting panel feeds auxiliary support systems for the PS like the exhaust fans, emergency backup generator support systems, RTU panel, lighting, and receptacles.

The auxiliary support systems for the PS have either localized disconnect switches to control panels, or combination disconnect and motor starters near the respective equipment.

15.4.2 Standby Power

The PS will not be provided with permanent back-up power to run the pumps under power outages. The PS will be equipped with permanent generator power for life-safety and SCADA purposes.

15.4.3 Lighting

All PS interior and exterior lighting fixtures shall be LED type. All interior lighting shall be controlled via light switches. All exterior lighting shall be controlled via a lighting contactor with photocell. Interior and exterior lighting levels shall be per SFWMD Requirements.

15.4.1 PS Switchboard

A switchboard consisting of circuit breakers will be provided to distribute 480 volts of power to various loads, including but not limited to the following equipment:

- Pump motor starters
- Screen control panels
- Miscellaneous loads

The switchboard for miscellaneous loads will supply power to individual motor starters that are not part of a vendor-supplied package and other loads as indicated below. The following listing of equipment is typical and tentative, and subject to change during final design:

- Building supply and exhaust fans
- Trash rakes
- Trash rake conveyor
- Lighting panel transformer (480V/ 120/208V, 3 phase)
- The 120/208V, 3-phase lighting panel will power:
  - WCS-9
  - Lighting, interior and exterior
  - Receptacles

- SCADA equipment
- RTU
- Air conditioning
- Security system
- Any other miscellaneous items

15.4.2 PS Building System

Various criteria for the building electrical system are:

- Lightning protection system will include air terminals on the roof interconnected with aluminum conductors.
- Grounding system will include a ground ring installed around the PS consisting of #4/0 bare copper stranded cable and ground rods to establish a resistance of 5 ohms or less. The building's steel columns, steel rebar in the footing, water piping, lightning protection system, motors, panels, transformers, etc., will be connected to the ground ring in accordance with NEC.
- The fire alarm system will be a zoned, supervised fire detection and alarm system with ionization-type smoke detectors used in the electrical, control, IT, and generator rooms. To protect against false alarms, the detectors in these rooms will be cross-zoned so that two detectors must be initiated before an alarm is sounded.

15.5 Motors

Motors below 150 hp will be totally enclosed fan cooled and of premium efficiency. Motors 200 hp and above shall be housed within weatherproof enclosures (weather protected type I). All outdoor motors will have integral space heaters. Indoor motors 5 hp and larger will have integral space heaters.

15.6 Conduits and Wiring

Above-grade conduits will be galvanized rigid steel. Conduits below grade will be polyvinyl chloride (PVC) Schedule 80 pipe. Underground conduits, in general, will be direct bury. Liquid-tight flexible metal conduit will be used at all motors, transformers, instruments, and any other equipment that can vibrate or move. Cable tray will be reviewed for use in the PS during final design. Wire for 480V power applications will be XHHW insulation type with stranded copper conductors. The minimum size wire for power will be 12 gauge. Wire for control and alarm circuits will be multi-conductor type THHN or THWN insulation, with stranded copper conductors, with a nylon jacket suitable for installation in either a tray or conduit. The minimum size wire will be 14 gauge. Wire for milliamp (mA)/millivolt (mV) circuits, typical for signals and instrumentation, will be single-pair shielded instrument cable, TFN-type insulation, 300V rating, with stranded copper conductors, and a nylon jacket suitable for installation in either a tray or conduit. The minimum size wire will be 16 gauge.

15.7 Backup Systems

The PS will not be provided with permanent backup power to run the pumps under power outages. The PS will be equipped with a permanent generator power for SCADA purposes, HVAC equipment, and WCS-9.

WCS-9 will be on standby power from the PS generator for gate operation, lighting, ventilation, and SCADA purposes.

15.8 Radio Path Study for Off-Site Communications

There are two WCSs that will communicate by radio to the District, WCS-9 and WCS-13. The EIP team will work with the District to develop the radio path study for these two sites. This will be done during preliminary and final design.

15.9 PES

The PES is being designed in accordance with the electrical standards outlined above. Design details and associated figures are outlined in the PES H&H TM located in Appendix 4. The only electrical demand is to operate the slide gates and outlet control valves. As these devices draw less than a kilowatt each, the total draw would be less than 12 kW.

15.10 Proposed Future Activities

15.10.1 Preliminary Design

The following future activities are proposed for preliminary design:

- EIP will develop draft preliminary plans and specifications. Related to the electrical design, the draft POM, DDR, and regulatory plan will be updated, and a construction schedule and stipulated price proposal will be provided.
- Develop power requirements for PES system.
- Coordinate with FPL for utility power.
- Determine overhead and underground power routing.
- Determine drop locations to power WCS and PS.

15.10.2 Final Design

The following activities are proposed for:

- EIP will advance plans and specifications.

SECTION 16

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SECTION 16

INSTRUMENTATION AND CONTROLS

This section covers the SCADA system components that EIP will procure, as well as the electronic safety and security system (ESS) features, including door position switches and cameras focused on strategic locations. There will be two approaches taken to instrumentation and controls (I&C) on this Project. The first approach is for EIP to initially own and operate facilities that they will then turn over to the District after the proving period (5 to 7 years). The second approach is for facilities to be immediately owned and operated by the District. The section below summarizes how these approaches will be implemented so all facilities meet District standards in the long term and EIP's in the short term. Additional details regarding the instrumentation and SCADA communications are also covered in this section.

16.1 Design Criteria

This section defines the Project's I&C design criteria. Since the Project will be turned over to the District after the proving period (5 to 7 years), all systems will be designed in accordance with SFWMD standards.

Applicable standards and publications used throughout design not covered under the District's Standards but used for electrical and equipment include:

- NEMA 250, Enclosures for Electrical Equipment (1,000 VAC Maximum), latest edition
- NFPA 70 National Electrical Code
- Telecommunications Industry Association (TIA): TIA 232-F, Interface Between Data Terminal Equipment and Data Circuit Terminating Equipment Employing Serial Binary Data Interchange

All systems and facilities within the project boundaries, as a general practice, will be monitored and controlled from a local control system in the PS. The PS local control system will be a PLC-based. The PLC under consideration is a District Standard Allen Bradley (AB) of the ControlLogix family of processors. Coordination with the District's Operations, Engineering and Construction Department is ongoing to determine if a smaller PLC family, such as the CompactLogix family of processors, can be considered instead. The PLC will be installed in a PS master control panel. This PLC will control and monitor the process equipment in the PS for the life of the project i.e., during the EIP operations period and after turn-over to the District. An EIP RTU will be installed in the PS control room. It will be in operation for the first 5- to 7-year EIP operations period. Monitoring and control will be available through the EIP RTU. Full monitoring capability will be made available to the District during the EIP operations period. After this period, and at project turn-over, a District-standard Motorola ACE RTU will replace the EIP RTU. As is standard with the District, the AB PLC will employ a communication card to connect to the District-standard Motorola ACE RTU. This hardware configuration, which is standard to the District, enables communications between the PS PLC and the SFWMD SCADA system.

Operation of the STA requires control and monitoring of all the equipment. All WCSs and the PES will employ an EIP RTU to monitor signals from and deliver commands to the equipment. Monitoring signals and control commands will be communicated to the PS through point-to-point radio telemetry transceivers.

A secure, one-way, read-only means of acquiring real-time data is planned to the existing SFWMD SCADA system for monitoring of existing upstream WCSs owned and operated by District. This monitoring is required for LKBSTA operations. It is important to note the connection mentioned is not direct to the District SCADA system, but rather one of four possible indirect solutions discussed with District SCADA operations that achieve the same result. In general, the Project I&C will include the following features:

- The AB master control PLC will have an uninterruptable power supply included in the control panel. Packaged systems for the Project will be provided with stand-alone AB ControlLogix PLCs communicating over Ethernet-IP.
- Monitoring and control of remote sites, including gated spillways, gated culverts, and monitoring stations will be over cellular radios during EIP's operational period. Equipment capable of control will be controlled by the site's RTU.
- Pumps and gated structures will have control from EIP's SCADA system. SFWMD will have full monitoring capabilities. The PS IT Room will also provide the ability to monitor and control the Project.
- This real-time data monitoring of multiple existing WCS sites is required for LKBSTA operations. The following four approaches are proposed to enable this acquisition of SCADA data without involving a direct connection to the B-1 District SCADA system. The approach that meets EIP requirements and receives District approval will be implemented:
  - Approach 1. Enterprise Data Solutions (EDS): The Emerson system, i.e., Ovation, is the District's SCADA system. It features a read-only extension of the SCADA system, known as the EDS. This extension allows secure access to near-real-time process information (trending, graphics, alarm display), but does not allow control. This near-real-time (under 1 minute delay) version of the Ovation SCADA system, which includes the same graphics available to SFMWD's Water Managers, is essentially a replication of SCADA but with read-only access. It is for these reasons that a connection to the Emerson Ovation EDS is proposed; however, access to this solution is pending approval by IT Security, SCADA Operations, and Water Managers.
  - Approach 2. Data Depot: An Oracle database table containing raw data from District SCADA is proposed for acquiring data from multiple WCSs. The raw data reaches Data Depot via middleware and database query language (i.e., SQL).
  - Approach 3. Database installed on an EIP server: An Oracle database installed on the EIP virtual server, external to District, would be populated in real-time with the data that is of interest to EIP. This would be implemented using middleware, SQL querying, or similar, with mapping that allows only the needed sites identified by EIP.
  - Approach 4. DBHydro: DBHydro does not involve any approval from the other parties (Security and Water Managers) and is readily available. Another benefit is the data it contains has been quality checked. A disadvantage, however, is that the quality-check process requires time, which causes a significant delay before data is available.
- Analog control signals will be 24 volts direct current (VDC), 4-20 mA. Discrete input signals will be 24 VDC. Interposing relays shall be used where necessary to provide isolation and conversion to 24 VDC. Discrete output signals will interface with field devices through interposing relays. Surge suppression shall be provided for all instrumentation. SFWMD design details will be followed.

A proposed network block diagram of the envisioned SCADA network architecture is referenced on Figure 16-1.

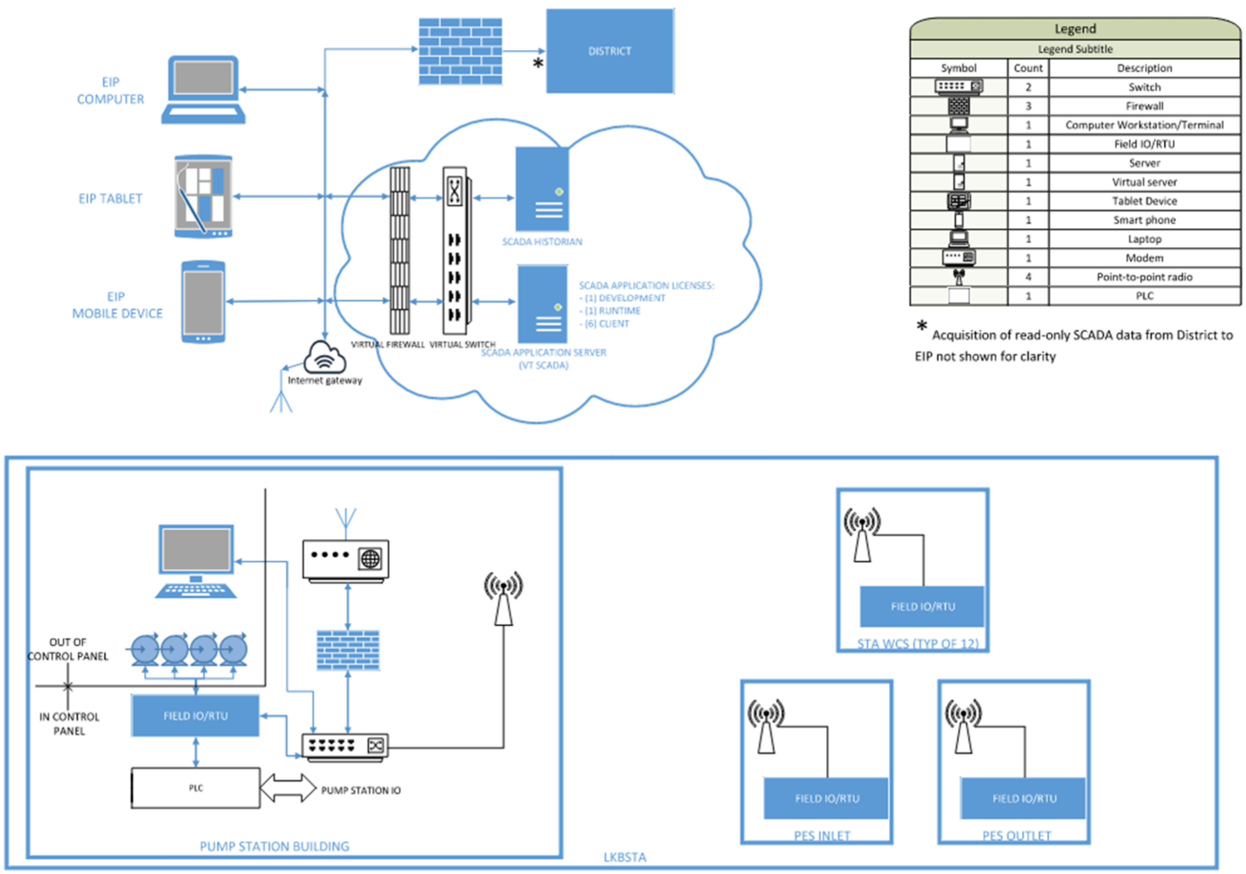

Figure 16-1: Proposed SCADA network block diagram

16.2 WCS-9 and WCS-13

WCS-9 and WCS-13 will be designed and constructed to meet SFWMD requirements and be immediately turned over to the District to own and operate. As such, these WCSs will each employ a District-standard Motorola ACE RTU with the standard peripherals (battery box, antenna surge protection, and primary and secondary RF paths), headwater and tailwater stilling-well stage monitors, gate control panel, and gate position sensor.

EIP will monitor WCS-9 and WCS-13 through one of the four approaches detailed in the previous sub-section. The chosen approach will enable a read-only monitoring channel for these and other structures.

16.3 Water Control Facilities

An EIP-provided RTU will be strategically placed at each of the WCSs, including the L-62 canal system gated conduits and gated spillways, inverted syphons, and inflow/outflow structures. These RTUs, which will allow monitoring and control for EIP, will be stand-alone units for exclusive use by EIP for the EIP operations period. Each RTU will capture and process the field input/output (IO) as is typical (e.g., gather instrument signals through hard-wired connections, provide output for commands from EIP SCADA through hard-wired connections) through provided and installed electrical infrastructure that fully adheres to District standards.

Each RTU will communicate wirelessly to the PS using point-to-point transceivers. A cellular modem at the PS will link the site to EIP's cloud-based SCADA system. Depending on distance and topography, an omni-directional or directional antenna will be employed at the PS to allow communication between the modem and nearest cellular tower. The resulting private network will securely connect the RTUs and the SCADA application server to allow data (commands and status signals) to be passed back and forth from the RTUs to EIP SCADA and from EIP SCADA to the RTUs. Full monitoring capability of all the assets in real-time will be available to SFWMD through EIP's SCADA system during the EIP operations period.

During the design phase, provisions will be made specifically for the District-standard Motorola ACE RTU platform for compatibility with the existing RTUs already installed at other SFWMD locations. These provisions will allow a relatively short cut-over for the Project and allow its assets to be turned over to the District after the EIP proving period. The provisions include supply and installation of electrical infrastructure (e.g., conduits, wireways, termination boxes, and enclosures) such that they allow the District RTU to fully integrate with the installed infrastructure and replace the EIP RTU subpanel. As a specific example, the RTU enclosure will be the same as the Districts or supplied by the District. This will allow the infrastructure (wiring from discrete and analog signals from District-standard instrumentation and equipment) to be re-terminated for sole use by the District RTU platform when ready by simply installing the District-standard RTU subpanel and re-terminating the wires.

The resulting RTU system and SCADA system will be the District-standard Motorola ACE, with two RF transceivers and two antennas providing two redundant communication paths (primary and secondary), in accordance with District standards, for increased robustness. The RF transceivers will operate on licensed VHF, UHF, or un-licensed 900 MHz ISM/Spread Spectrum frequencies, with the site-specific transceiver model determined by SCADA design and installation (SDI) (District SCADA). The transceiver radios will be part of the owner-furnished RTU backplanes, which the District will mount into the EIP-provided enclosures. EIP‘s system will be completely decommissioned and removed. Gate control will be either manual or from the control system, based on headwater and tailwater levels.

Upstream (headwater) and downstream (tailwater) stilling wells with water level transmitters provide analog water level data of the approach and discharge channels. The water level transmitters provide a proportional 4-20 mA signal. The stilling well water level transmitters are normally continuously powered from a 24 VDC power supply.

16.4 PS

PS control and monitoring can vary from the simple manual operation of an agricultural-style station to the more complicated automatic or remote operation of a typical SFWMD station. There are also varying degrees of complications for remote station operation. Electric motor drivers have far fewer auxiliary systems than a diesel-engine-driven PS and, therefore, a much simpler control and monitoring system. Electric-driven PSs are also typically not used for flood control applications due to the possibility of power outages during a storm event. The PS at LKBSTA is an electrically driven small-sized PS.

Accurate headwater- and tailwater-level measurements are essential for the desired control of the PS. The District's standard instrument for electronic level measurement is the Digital Shaft Encoder by Waterlog. This transmitter measures water level by monitoring the position of a float and a brass pulley using a magnetic sensor.

Each Digital Shaft Encoder will be installed in a stilling well located in specific waterways. The encoder produces a 4-20 mA output signal, which represents the range of the water level within the stilling well. The signals for all the encoders will be routed to the PLC in the local control panel for the automatic control of the inflow pumps. This level signal will also be transmitted back to the EIP

SCADA system via the connection to the RTU described in later sections. District-standard detail drawings for platform-mounted stilling wells will be incorporated into the Project.

A float switch will be used to disable each pump in the event the water level falls in each pump bay due to a clogged trash rake. The float adjustments are set manually. The float switches will be located on the down-flow side of the screens. The low setting will be used to disable a pump with a low water level to prevent pump cavitation. This disable signal will be set about 6 inches below the normal flow cut-off of each pump. This low-level signal will be hardwired directly to the pump starter and will lock out the pump when the pump is controlled either locally, or remotely through the RTU, and will work even if the RTU fails. The float switch will be mounted in a stilling well made of PVC pipe. When locked out due to low water, an operator will have to manually reset the starter locally before the pump can be put back into operation.

The hardwired low-level signal discussed above will be repeated and made available to EIP SCADA. The provision will stay in place at turn-over for the District to use, abandon in place, or disconnect and remove.

16.4.1 Operation

Pumps typically have four modes of operation: local/manual (based on hardwired controls), local/auto (based on PLC controls from the local control panel within the facility), remote/auto (based on PLC controls from the other human-machine interfaces within the facility) and remote/auto (same as local/auto but based on station/central key switch permissive in the main station control panel – set to Central). Not all four modes of operation are expected to be implemented or used. Remote operation also consists of operating the pumps via the EIP SCADA system. Remote or manual startup and shutdown, as well as motor protection alarm shutdowns for the pump motor, are automatically controlled by the pump controls.

16.4.2 PLC

The microprocessor-based station PLC shall be, as described above, an AB model, no equal, for machine level control applications requiring limited I/O quantities and limited communications capabilities. The PLC shall be mounted in the control room control panel and shall provide control and monitoring of the pumps, screens, and generator systems. The PLC will be network-linked through an RS232 connection with the RTU (Motorola ACE) via a Modbus over serial converter. The PLC will communicate with EIP SCADA using Ethernet IP through a cellular modem connection.

16.4.3 Interface Module (Modbus/RS232)

A Modbus over serial communication converter module, previously mentioned, will be provided and installed within the main control panel enclosure to connect the PS PLC to the Motorola ACE RTU. The module converts the Modbus network digital output to the serial communication signal that is required by the RTU. The Modbus communication module shall be a Prosoft Modbus master/slave communications interface for the Control Logix family of processors, part number MVI-56-MCME, or equivalent.

16.4.4 Surge Suppressor

A surge suppressor will be provided that will protect the PLC’s I/O modules from lightning-induced surges, electrical fast transients, and electromagnetic or radio interference noise. The surge suppressor shall meet or exceed the highest class severity level of IEC 1000-4-4 and 1000-4-5. The suppressor shall be UL- 497B listed. The surge protector will be Circuit Components, Inc.’s “Surge Control SAB Series” or SFWMD-approved equal.

16.4.5 Temperature Monitors

Temperature instrumentation will be as required by the pump manufacturer. Instrumentation will be in the form of motor-stator embedded temperature switches, or resistance temperature detectors.

16.4.6 Level Switches

Level switches shall be provided to signal a low-level condition. The level switches shall incorporate floatless, pressure-sensitive, diaphragm-actuated sensors that activate a corresponding switch contact.

16.4.7 Monitoring Instrumentation

Depending on a variety of causative factors, some structures will require monitoring for water quality, which may or may not entail the use of autosamplers. In either case, flow measurement will be needed at select structures and, in the case of structures with autosamplers, telemetry units capable of triggering the autosampler for flow-proportional sampling will be required. The installation of autosamplers and associated telemetry will also necessitate the need for routine support and access infrastructure, including but not limited to stairs, platforms, housings, conduit, power supply, and stage gauges. The design and installation of all monitoring equipment and appurtenances will be coordinated with the SFWMD Water Quality Monitoring Section and built to SFWMD Standards.

16.4.8 Station Standby Power

The LKBSTA PS would require electric power backup for lighting, HVAC, and SCADA RTUs in the event of a power outage. Process equipment, including the pumps and trash racks, are not backed by this generator. Typically, an adequately sized engine generator is provided for backup station service during the time that utility power is lost. Redundant systems are not considered since the PS is not used for flood control. Controls are provided in the ATS to either manually or automatically start the liquified petroleum gas engine generator. An RTU output command will remotely stop the engine generator should the unit unnecessarily start. During intermittent power outages, an uninterruptible power supply will provide backup power to the PLC to allow communications to SCADA.

16.4.9 Stage Monitors

Upstream (headwater) and downstream (tailwater) stilling wells with water level transmitters provide analog water level data of the approach and discharge channels. A stilling well with a water level transmitter is also provided in each pump intake to monitor low water levels. The water level transmitters provide a proportional 4-20 mA signal. The stilling well water level transmitters are normally continuously powered from a 24 VDC power supply.

16.4.10 Telemetry and SCADA

As described earlier in this section, an EIP-provided RTU will be located in the control room. The enclosure will house the EIP RTU for the EIP operations period. The PS EIP RTU will be similar in form and function to the permanent District standard RTU. It will be mounted to a backplane/subpanel to enable a swap out at turn-over. Upon turnover, the EIP RTU will be removed and the same enclosure will house a District RTU. The EIP RTU will be installed such that it is as close as possible to the external antenna pole. The antenna pole type (concrete pole-mounted or side of building steel mast-mounted) will be determined during design.

An EIP-provided main control panel will house the PS PLC. This control panel will be located in the PS building control room as descried earlier. The EIP RTU will communicate with the PS PLC to read and write PS data to/from it. The plan is to read/write data in one location (a Modbus register), similar to the District’s current approach. This allows the RTU to efficiently interface with the PS PLC for all the

PSs data. The EIP RTU will also wirelessly communicate with the WCSs and PES RTUs for those IO signals, as well as communicate with the EIP cloud-based SCADA system via cellular modem.

At the time of project turn-over to the District, after the EIP operations period, the permanent PS RTU backplane (i.e., the District-provided sub-panel) will replace the EIP RTU backplane.

The PS will have the capability of being remotely controlled and monitored through EIP's SCADA system. After EIP's proving period, the PLC and RTU will communicate with District B-1 headquarters only.

16.4.10.1 Cloud-based SCADA

For this Project, it was decided to follow an Industrial Internet of Things (IIOT) approach but implement a traditional SCADA system. After additional discussions on project needs and expectations, a decision was made in favor of a traditional SCADA system over an actual IIOT-architected solution. The decision was based on the ability to monitor assets, animate and have a certain level of graphics sophistication beyond premade "widgets," generate alarms, generate reports, graph data, historize or store the data for future analysis, control equipment (issue commands to open and close several gates, start and stop minimum four pumps), and process an estimated maximum of 1,000 IO points. As a result, the IO point count, scalability, and the need for reporting pointed to a traditional SCADA system being the more appropriate system to implement over an IIOT-based system.

The EIP SCADA system will have a virtual/cloud-based architecture that will give EIP full remote monitoring and control. The decision to implement such a system was based on a comparison between physical and cloud-based SCADA servers with two similar SCADA platforms. As shown on the SCADA network block diagram (Figure 16-1), a physical firewall will be installed at the PS and a virtual firewall implemented in the cloud environment for cyber-security reasons. In addition to monitoring and control for EIP, the SCADA system will offer a secure, one-way channel for monitoring by the District. This channel will meet District IT Security standards and requirements and will enable full monitoring capability for District staff.

The system of EIP RTUs, including the PS PLC/RTU, will communicate with the VTSCADA application wirelessly using cellular technologies (e.g., 4G/LTE modems). A preliminary survey of the cellular coverage maps, published by the three main cellular providers (Verizon, AT&T, T-Mobile) shows coverage in the LKBSTA area. One of the three providers will be considered, and confirmation of service will be formally made during preliminary design. With wireless communication established between EIP RTU and EIP SCADA, a domain or private network will be created that contains the RTUs and the SCADA application server. This network will allow data and information to pass back and forth securely between the EIP RTU to EIP SCADA and from EIP SCADA back to EIP RTU.

The SCADA system will be based on Trihedral's VTSCADA application that will be hosted by a cloud-based service. A monthly/quarterly/yearly subscription to the cloud-based service will be required in addition to the cost of the VTSCADA application. The service will be used during the proving period, then completely disconnected and de-commissioned for project turnover and use by District and its standard SCADA system. The decision to implement the Trihedral system was based on a cost and feature comparison of two similar SCADA application candidates on a physical server, versus the same two candidates on a virtual/cloud-based server.

16.4.10.2 Microwave Communication to District

The SFWMD SCADA system is proprietary and consists of an RTU and an antenna. SFWMD's SCADA system will replace EIP's after the proving period. The EIP system will be disconnected to allow the SFWMD system to take its place. An existing District SCADA microwave tower will be identified during design to allow the District RTUs to connect to the District SCADA.

16.5 Electronic Safety and Security Systems

The Project and all associated ESS features will follow the security guidelines of the SFWMD and the U.S. Department of Homeland Security.

16.5.1 Design Criteria

The LKBSTA is in a remote rural area. It is anticipated only the PS area and other critical infrastructure be surrounded by a perimeter chain-link fence for protection. As such, ESS systems will be concentrated in vehicular, PS entrance, and critical infrastructure.

16.5.2 Design Analysis

A closed-circuit television surveillance system will be employed for security at the proposed LKBSTA PS. The details are covered in the Security section herein. Electrically powered ESS equipment shall operate on 120V, 1-phase, 60 Hz AC sources. Equipment shall be able to tolerate variations in the voltage source of plus or minus 10 percent, and variations in the line frequency of plus or minus 2 percent with no degradation of performance.

16.5.3 Monitoring and Control (Video, Access Control, Alarm)

The proposed PS will have controlled access through fences and gates. Manual gates will be provided at vehicle access points located where needed. Electric gates and locks are not anticipated. Items that will be considered when monitoring and controlling access to buildings will include:

- Door position switches
- Access control system

All security features and elements will be coordinated with SFWMD during preliminary design.

16.5.3.1 Functional Operations Requirements

A system will be integrated so critical infrastructure can be operated by a single or multiple operator(s). The system shall provide communication of totally automatic status changes, commands, field-initiated interrupts, and any other communications required for proper system operation. The system will be such that it does not require operator initiation for the system to respond. The system’s communication is to return to normal after any partial or total network interruption, including power loss or transient upset. The system will also automatically annunciate communication failures to the operator with identification of the communication link that has experienced a partial or total failure.

16.5.3.2 Local Communications

Any required local communications will be established for the ESS system during detailed design either by networking the system components via a managed network switch, or through simpler means. A networked approach would use a switch with small form-factor pluggable Ethernet ports to allow fiber optic connection, and feature protocols and configurations that allow a ring network for simplicity in network wiring. The switch will use 24 VDC nominal input voltage, be layer 3 capable, have the capability of configuring in a star or self-healing ring, and with power over Ethernet (PoE) to all ports. The switch will be capable of using a layer 3 (routed) port to connect to a gateway port for Internet and web access. Further consideration for a networked approach or a simpler approach will be determined at during preliminary and detailed design.

16.5.3.3 Wide-area Communications

The EIP SCADA system will receive the security data statuses. Security video will be made available through a separate system and connection or through the same SCADA system connection using a feature on the VTSCADA application for video streaming; the approach will be established during detailed design. Upon turn-over to SFWMD, its microwave network will carry all signals to/from the PS to the District's B-1 headquarters.

16.5.4 Life Safety

Life Safety aspects of the project will be discussed here as they impact the PS. The PS fire alarm system will be evaluated and integrated into the systemwide SCADA and communications system per applicable codes and SFWMD standards.

1. Fire alarm
    a. General Scope: A fire alarm system that conforms to SFWMD standards will be provided at the PS to provide alarm management. This will include, but not be limited to:
        i. Ionization-type smoke detectors located in the electrical building and control building.
        ii. The electrical room will have fixed heat detectors strategically placed and connected to the fire alarm system.
        iii. In addition to smoke detectors and heat detectors, the overhead door opening and the office egress will have manual pull stations.
        iv. In addition to signaling to the control building, the fire alarm panel will drive horn/strobes that are provided throughout the structure.
        v. The fire alarm signal will also be transmitted over EIP SCADA to an on-call operator.

16.5.5 Security

Aspects related to securing the Project's facilities are discussed here as they impact the PS and critical infrastructure. The security systems at the PS will be evaluated and integrated into the systemwide SCADA and communications system per applicable codes and SFWMD standards. The design scope of this project involves electronic security systems. These include electronic access control and network security systems. To be specific, the project scope requires the design of:

a. Electronic door locks: access control to the PS IT room.
b. Intrusion alarm system: Provide intrusion alarm for station access man doors and overhead doors. This alarm involves the door position switch.
c. Network security: Incorporate existing District network security systems into the design of the PS IT room SCADA/Enterprise systems. The security systems shall be designed to use the same make and model hardware and software as that currently in use by the District's security office.
d. Video surveillance system: A closed-circuit video surveillance system will be considered for security at the proposed LKBSTA PS. Cameras will be proposed in the following locations: One external, pole-mounted or wall-mounted security camera in or near the entrance/access area of the PS; one external, pole-mounted security camera at the trash rakes; and one strategically located camera within the IT room, partly focused on the control room entrance door. The cameras are anticipated to be high resolution (day or night), designed to work in harsh outdoor environments (external cameras), and network enabled. Outdoor cameras will have integrated pan-tilt-zoom (PTZ) or panoramic capabilities with varifocal auto-iris lenses and IR illumination. Indoor cameras are anticipated to be fixed, PTZ, or panoramic as required. The cameras will operate over a voltage range required for PoE IEEE 802.3 and be compatible with network video recording devices.

2. Network security (short term): The communication links (to the cloud where the data processing takes place) need to meet security standards. The EIP SCADA system will employ hardware- and software-implemented firewalls. The hardware firewall will be installed at the PS upstream of the cellular/4G LTE modem before the data exits/enters the system, and configured to only allow ports, applications, and services that EIP is expecting to use for access and communications. This firewall will protect the PS control system. A second, software-implemented firewall will be configured in the cloud system that houses the EIP SCADA application to protect the VTSCADA application from unwanted incoming traffic from the cloud environment.
3. Network security (long term): The SCADA system can have potential multiple network connections. For this project the potential interconnection of these networks occurs at the pump station control room. These potential interconnections require a security appliance, also known as a firewall, to provide access control. For similar projects, District standards have suggested a device from the Juniper Networks company which supports capabilities such as intrusion prevention, application visibility and control, and content security features that include anti-virus, anti-spam, and enhanced web filtering. The District IT Security section will review the design package for compliance to IT Security standards for SCADA Systems. The device model and manufacturer, from Juniper Networks, or District-approved equal, will be confirmed during detailed design.

16.6 Audio Visual Systems

The video surveillance system described earlier in this report involves video cameras to transmit a signal to a specific, limited set of monitors. It is typical for a video surveillance system to use a pole-mounted video camera to project a magnified image onto a video monitor. The PS IT room will be the primary location for video monitoring, with ability to monitor events live and/or record. Monitoring remotely will also be available through EIP's SCADA system.

16.6.1 Design Criteria

Video surveillance systems may operate continuously or only as required to monitor a particular event. The objective for the video surveillance system is to assist with safety during remote operations of a facility and allow observation of anomalies or abnormalities that may occur during normal operation. The number of cameras will be limited to that considered necessary for operations to observe areas of the station during pumping and that are necessary for security. The number of cameras and their locations, as identified in the Security section of this report, will be connected to a network video recorder (NVR). The NVR will feed the video through a network connection to the EIP cloud-based SCADA system. Upon turn-over of the project to SFWMD the EIP link will be disconnected. The video will then be reconnected such that it is sent back to the District Headquarters' security office.

16.6.2 Design Analysis

The required storage capacity for the NVR storage device will be determined during detailed design. The recording length is currently proposed to be 30 days.

16.6.2.1 IT Room Functional Operational Requirements

The control room will be designed with an area for a chair next to or part of the workstation desk, and a monitor at which to view security camera video.

16.6.2.2 A/V Devices

Audio visual (A/V) devices comprising the audio/visual system, in the context of security monitoring, are proposed to be a video monitor for displaying video and images, and an NVR. The monitor will display camera feeds live or recorded, and contain video time and date and camera alphanumeric identifier. At minimum, the monitor shall feature signal input connectors of the high-definition multimedia interface (HDMI) type. The monitor's minimum diagonal viewing angle should be 24 inches. The NVR shall be capable of providing encoding, recording, and multiscreen viewing modes simultaneously, with video loss detection on all channels and motion-based recording.

16.6.3 Ergonomics

Operators will have a chair to use when monitoring the security video stream. The chair and associated workspace will have a high degree of adjustability with the goal of enhancing work conditions for a wide range of different users, in order to reduce the risk of repetitive strain injuries, muscle pains, body discomforts, and fatigue.

16.6.3.1 Lighting

The lighting in the IT room is proposed to be dimmable to allow easy viewing when needed of the video stream.

16.6.3.2 Local Communications

Direct access from the PC workstation to the NVR will be via web browser interface. All functions are to be accessible through HTML commands from a user's web browser.

16.6.3.3 Wide Area Communications

An NVR that can provide all the viewing modes and features through a network connection will be considered during preliminary and detailed design.

16.6.3.4 Local Data Processing

A system that features, at a minimum, pictures to be available for attachment via a user-provided SMTP-based email transport system will be considered during preliminary and detailed design. Consideration will also be given to a system that includes capability for multiple users (number to be determined), three user-access levels (admin, control, and user), and an integral programmable switcher with programmable dwell time and camera order that automatically switches multiple camera images to enable sequential spot monitoring and simultaneous field recording.

16.6.3.5 Wide Area Data Processing

The remote, or "wide-area," processing of the video streaming and other features related to the NVR are currently being considered and will be further conceptualized and subsequently designed during preliminary and detailed design. The current plan is to have NVR recording, review, playback, camera control, and setup to be available via an internally mounted Network Interface. To realize this, a 1,000 megabits per second gigabit Ethernet connection for record review, camera view, and control will be required on a personal computer equipped with Internet browser software.

16.6.4 Information Flow

The flow of the photos and video made available from the NVR system is envisioned to be from the site to the end user's device.

16.7 Communications Systems

A SCADA system consists of several RTUs that collect field data and transmit it back to a "master station" via a communication system. The master station displays the acquired data and allows an operator to perform remote control tasks. A telemetry system is a system that is responsible for the transfers of remote measurement data to a central control station over a communication link. In the current LKBSTA Project, telemetry will be achieved through radio pathways between each of the remote stations and a central control station, which is the control panel in the Project's PS. The LKBSTA SCADA system then interfaces with the central control station and makes the measurement data available to remote user(s), as well as allows commands to be issued to equipment (control).

16.7.1 Design Criteria and Functional Requirements

Over the proving period the PS will be remotely monitored through EIP's SCADA system. The District RTU will be in the control building close to the antenna. The District-provided RTU will be a Motorola ACE unit to be compatible with the existing units already installed at other SFWMD locations, i.e., SFWMD's proprietary system consisting of an RTU and an antenna. The RTU will be capable of transmitting data to a main station via radio. Data to be transmitted is to be determined. When the PS is turned over to the District, SFWMD will have the option of remote control of the station via the SCADA system.

The EIP SCADA System has the following typical functions related to the SCADA RTU devices:

- Establishment of communications
- Operation of the communications link: Poll each RTU for data and write to RTU; log alarms and events to a hard disk and to an operator display; link inputs and outputs at different RTUs automatically
- Diagnostics

16.7.1.1 Hydraulics

Stilling well measurements are important to the function of the WCS, the PS, and overall system as SFWMD seeks to maintain canal levels between specified minima and maxima specific to each location. Headwater and tailwater stilling wells will be constructed for the WCS and PS. These stilling wells are instrumental in determining flow through WCS and determining when pumping activates. As such, still well construction is discussed herein. The stilling well tube, typically constructed of corrugated aluminum pipe, serves to dampen waves and turbulence and to reduce foaming where water level measurements will be taken. As such, it improves the accuracy and performance of the shaft encoder instrument installed in it, providing level information to the PS and the SCADA system. Stilling wells must have venting above the water level's surface to prevent air being trapped at the top of the well and enable pressure equalization. The well must be as long as the deepest measurement required with sufficient openings below that level to allow the fluid to equalize inside the tube.

16.7.1.2 PS Instrumentation and Controls

The I&C design is based on SFWMD Guidelines and SFWMD Standard
Details for monitoring and control of electric PSs, electric slide gate control structures, and electric vertical lift roller gate control structures with the focus of the LKBSTA PS based on SFWMD's Pumping Station Engineering Guidelines. The PS and control structures I&C systems are designed to be controlled locally in manual and remotely by the EIP SCADA system during the proving period then, after turnover, SFWMD Headquarters via SCADA RTU.

The microprocessor-based controller shall be an AB model or equal for machine level control applications requiring limited I/O quantities and limited communications capabilities. The PLC shall be in the control room and shall provide pump control and monitoring. A workstation with monitor will be provided for the system and be located in the PS IT room.

A general-purpose Modbus serial master/slave gateway module shall be provided for converting the network digital output to the serial communication signal that is required by the RTU (ACE).

16.7.1.3 Administrative

During preliminary and detailed design, the administrative functions associated with communications equipment will be considered. Such functions typically involve password-protected ability to configure certain aspects of the system.

16.7.1.4 Other

The transmission of information between the control panel and the RTUs and from the control panel to the SCADA system requires the use of digital hardware and software. Consideration during preliminary and detailed design will be given only to devices and equipment that are compatible with industry-standard messaging format(s) and typical protocols used in radio communications and telemetry systems (Modbus, Distributed Network Protocol, etc.).

16.8 Design Analysis (Voice, Video, and Data)

The sections below on RF transmission, reliability, pathways and spaces, and monitoring of network elements will be further developed during the preliminary and detailed design.

16.8.1 RF Transmission (Path and Coverage)

The EIP cloud-based SCADA system will employ cellular/4G/LTE radio modems for communications between the PS and the cloud-based SCADA system, and between the various WCSs and the cloud-based SCADA system. Preliminary cellular coverage maps indicate service in the LKBSTA area, indicating that omni-directional antennas, as determined by the cellular provider, will apply. If needed, directional antennas will be considered.

In terms of the SFWMD SCADA system, the LKBSTA project location is such that nearby steel microwave towers are “in sight” and can be reached. As such, a microwave communications shelter with steel tower is not likely. A 60-ft concrete pole with primary and secondary RF path antennas is more likely. It is anticipated the pole location will be at the PS only. As is standard with SFWMD SCADA SDI, latitude and longitude location coordinates of the various WCS sites will be forwarded to SDI for modelling and evaluation to determine valid communications between the site WCSs and PS and nearby existing microwave towers. Preliminary observations indicate existing microwave towers with base stations are ones referred to as the “Highlands” tower and the “OKEE-FS” or Okeechobee tower that may be the candidates within reach for District SCADA communications.

16.8.2 Reliability (Hazard Classification)

The LKBSTA does not contain a reservoir; therefore, monitoring of reservoir embankments and levees with specific instrumentation is not required. In addition, the Project is not intended for flood control. As previously mentioned, preliminary observations suggest the Project’s location is near existing steel microwave towers such that communication to them to relay data back and forth between LKBSTA and SFWMD Headquarters is possible. Therefore, a microwave communications shelter with steel tower, and the reliability it brings, is not needed.

It is for these reasons that a high degree of reliability and the equipment associated with it is not necessary. As such, there is no requirement for a microwave tower at the site.

16.8.3 Network Element Status Monitoring

16.8.4 Intrusion

During detailed design, the provision of an intrusion alarm for electrical building and/or control building access doors will be determined. This alarm involves the door contact as well as a motion sensor. The communication buildings have a separate monitoring system that includes a door-open signal. General Requirements: Comply with TIA-569-C.

16.8.4.1 HVAC

The IT room will have an air conditioner. All other rooms are ventilated only.

16.8.4.2 Power Supply

The availability of RTU AC power is sent to SCADA.

16.8.4.3 Backup Power

The absence of RTU AC power is sent to SCADA and understood as backup power. Commercial power connected, commercial power available, and generator available from the ATS will be sent to SCADA, as will the generator run status signal.

16.9 Supporting Cabling and Systems Infrastructure

Cable supports will bear OSHA's Nationally Recognized Testing Laboratory label for support of Category 6 cabling, designed to prevent degradation of cable performance and pinch points that could damage cable. Supporting hardware may include conduit, J-hooks, straps, D-rings, and other support devices.

16.10 Data Processing Systems

The performance and efficiency of a SCADA package is important. The system in which it is installed should be relatively easily upgradeable to handle future requirements. The system must have a scalable architecture, i.e., it must feature easily modifiable components as the requirements change and expand as the inputs increase. The scope of this Project requires the capability to read data from existing WCS sites, which may grow in number, decrease, or otherwise change. The scope also requires complete turn-over to District after the EIP operations period. Therefore, the processing power, available mass storage, and features than enable relatively easy change will be considered.

16.10.1 Design Criteria

The requirements for the data processing systems include:

- Any additional hardware that will be added will be of the same modular form and fit in the same space as that provided by this Project and will not impact this Project's hardware.
- The installation of SCADA hardware/control cabinets/operator displays will not negatively impact the existing District system.
- The SCADA operating system will be able to support additional requirements without major modifications.
- The application software should require little to no modification when changes for turn-over are required.

16.10.1.1 Servers

As mentioned, the SCADA system will consist of a VTSCADA application hosted by a cloud-based server service. The service will be used during the proving period, then be completely disconnected and decommissioned during the turnover to District since the District system is an existing SCADA system whose servers already exists.

16.10.1.2 Work Stations

It is anticipated there will be one PC workstation in the IT room of the PS control building to enable operations monitoring and control of the WCSs, the PS, and the PES.

16.10.1.3 Software Applications

As mentioned, the SCADA system will consist of a VTSCADA application by Trihedral. The workstation will also include a Windows-based office application.

16.10.1.4 Reliability (Hazard Classification)

Redundancy at the server level is not anticipated.

16.10.1.4.1 Operating Systems

It is anticipated the EIP enterprise system will have a Windows-operating system. The SCADA application will run on Windows as well.

16.11 Communications Grounding and Bonding Systems

16.11.1 Design Criteria

Grounding and bonding of communications systems to divert current surges (lightning) coming down the antenna, antenna pole, antenna coaxial cable, etc., shall provide the lowest inductance/ resistance path to divert surges away from the RTU internals to a ground rod. This grounding method goes beyond the “surety” bond provided at the mechanical ground connector on the bottom of an RTU enclosure. This method is also a better practice compared to the backplane bond to ground via a conductor that goes to the ground buss in the RTU.

16.11.2 Design Analysis

16.11.2.1 Communications Site (Exterior)

The extent of the grounding and bonding of the building external is as described in the Design Criteria.

16.11.2.2 Electronics Spaces (Interior)

The extent of the grounding and bonding of the RTU internal is as described in the Design Criteria.

16.11.2.3 Electronic Equipment Bonding

The components of the grounding electrode system will be manufactured in accordance with ANSI/UL 467 - Standard for Safety Grounding and Bonding Equipment and will conform to the applicable requirements of NEC Article 250 and local codes.

16.11.2.4 TVSS Devices (Power and Communications Cabling)

Coaxial surge protection (PolyPhaser) surge protection device will be supplied and installed as required for protection of RTU and internal electronics.

16.11.2.5 Transmission Lines

Transmission line for EIP SCADA system-specific requirements will be determined by the cellular carrier. Coaxial transmission line between RTU and antenna will be LMR-400 by Times Microwave or equal, per SFWMD standard.

16.11.2.6 Antenna Systems

Antenna systems under consideration fall under two major categories: omni-directional and unidirectional. Cellular Antenna and cable for the EIP SCADA system shall be determined by the cellular carrier. Omi-directional whip types and cellular types for the SFWMD system will follow SFWMD standards. Yagi antennas (directional antennas) will be provisioned for, per SFWMD standards, when distances between the remote site and the base station/tower are such that require them. Early coordination will be made with other disciplines to locate the antenna pole as near as possible to the RTU to minimize resistive losses due to distance in coaxial cable run, as well as any bends the route may present to the conduit.

If concrete poles are considered, coordination with disciplines to locate as close to the control building as practical will be made such that coaxial point-to-point distance (between the RTU location and the base of the pole) is at or under 150 ft, including the vertical rising/dropping up and down walls and miscellaneous bending where required. The location of the pole will be such that it makes the antenna maintainable. Where possible, the access will be maintained by restricting an area immediately in front and the sides of the pole, leaving it clear to maneuver a bucket truck/man lift. Due to the distance involved between WCS-9 and the PS, separate concrete antenna poles—one pole for the PS, one for WCS-9, and one for WCS-13—will be considered. WHO will coordinate with WHO regarding coaxial cable installation before turn-over.

16.11.2.7 Cable Trays

There are no instances where cable tray for communications are anticipated.

16.11.2.8 Communications Towers and Antenna Support Structures

Antenna support structures will be limited to the 60-ft concrete pole or the 15-ft metal pole antenna mast. SFWMD standards will be followed for either or both where used.

16.12 PES

The main consideration in operating the PES is to monitor its cells for their dosing status and volumes. On and off status of each cell inflow gate will be determined by timers controlled by the SCADA system. The duration of the timing will be set by operations staff and vary over time, depending on infiltration rate of the media. If infiltration rates eventually slow down over time, the dose duration will be adjusted accordingly. The PES is designed to fill considerably within the 1-hour maximum duration projected to allow for this flexibility.

A level sensor monitoring array will be employed to track water elevations on the upstream and downstream sides of each cell's inflow slide gate. Given the rating curve for flow through the gate, the difference in elevation allows for calculating instantaneous flow rate. Multiplied by time, this gives the dose volume in real time. This allows for fine tuning the dose duration needed for the 18-in inflow depth.

In addition to the inflow level sensors, two additional level sensors would be deployed in the underdrain stone to monitor how the media hydraulic grade line (HGL) varies over the entire dosing cycle. A third level sensor will be deployed at the surface to monitor the duration of weir flows. Differences in HGL between these sensors and the inflow sensors allow for determination of system, stone, and media head losses. This allows for determination of media saturated hydraulic conductivity, which will be observed over time.

In this manner, the overall PES dosing and drainage dynamics will be constantly monitored in real time. The EIP team will monitor the site remotely to observe cell HGL, hydraulic loading rates, and hydraulic retention time. A remote wireless camera will be used to observe cell filling and drainage.

In addition to flow and level monitoring, auto samplers will be employed to achieve comprehensive total phosphorus monitoring.

16.13 Proposed Future Activities

During preliminary and detailed design the PES will be developed with devices and equipment that conform to SFWMD standards.

16.13.1 Preliminary Design

The following future activities are proposed for preliminary design:

- EIP will develop draft preliminary plans and specifications. Related to I&C, the draft POM, DDR, and regulatory plan will be updated, and a construction schedule and stipulated price proposal will be provided.
- Develop phosphorus treatment confirmation design approach (i.e., level of automation, sample point location and type of hardware to be employed, etc.).
- PES monitoring and control design:
  - Inlet slide gate control and monitoring
  - Outlet slide gate control and monitoring

16.13.2 Final Design

The following activities are proposed for final design:

- EIP will advance plans and specifications.
- The PES will be fully integrated into the SCADA/control system to allow full control and monitoring for EIP and full monitoring-only for the District during the EIP operations period. After turn-over, full control and monitoring of the PES will be available to the District.

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ARCHITECTURAL

Architectural design for the LKBSTA Project includes the design of multiple precast concrete buildings; one (1) PS canopy, one (1) electrical building, three (3) larger control buildings, and various small WCS control buildings. A general description of the architectural design criteria and basis of design for these buildings is presented in the paragraphs below.

17.1 Architectural Design Criteria and Codes

The buildings are pre-engineered precast concrete structures that house electrical components for the inflow and outflow control structures, spillway structures and pump stations. The building materials have been selected for long-term durability and low maintenance and are highly vandal resistant. The strong, simple geometric building form will contextually blend into the local environment.

- PS Canopy - The canopy is approximately 2400 sf in area with an overall footprint that is 60 ft long by 40 ft wide with a clear ceiling height of 10 ft. Access doors will be provided in the roof structure to allow for the removal of the pumps.
- PS Electrical Building - The building is approximately 588 sf in area with an overall footprint that is 42 ft long by 14 ft wide with a clear ceiling height of 10 ft. The building will house an electrical room that includes one double door and one single door egressing to the exterior. Access to the roof will be by means of a portable ladder and a portable ladder securing system.
- PS Control Building - The building is 476 sf in area with an overall footprint that is 34 ft long by 14 ft wide with a clear ceiling height of 10 ft. The building will house a generator room, control room, IT room, and restroom. Each room includes a single door egressing to the exterior. Access to the roof will be by means of a portable ladder and a portable ladder securing system.
- WCS-9 Control Building - This building is 140 sf in area with dimensions of 14 ft by 10 ft and a clear ceiling height of 10 ft. The building will house a control room that includes one single door egressing to the exterior. Access to the roof will be by means of a portable ladder and a portable ladder securing system.
- WCS-13 Control Building - The building is 476 sf in area with an overall footprint that is 22 ft long by 14 ft wide with a clear ceiling height of 10 ft. The building will house a generator room and control room. Each room includes a single door egressing to the exterior. Access to the roof will be by means of a portable ladder and a portable ladder securing system.
- STA WCS standard controls buildings - There are twelve (12) control buildings. These buildings are all 100 sf in area with dimensions of 10 ft by 10 ft and a clear ceiling height of 10 ft. The buildings will house a control room that includes one single door egressing to the exterior. Access to the roof will be by means of a portable ladder and a portable ladder securing system.

The building code is intended to provide minimum requirements to safeguard the public health, safety, and general welfare of the occupants of new and existing buildings and structures. The code also addresses structural strength, means of egress, sanitation, adequate ventilation and lighting, energy conservation, and life safety as they relate to new and existing buildings, facilities, and systems.

In general, the provisions of the building code apply to the construction, alteration, movement, addition, replacement, repair, equipment, use and occupancy, location, maintenance, removal and demolition of every building or structure or any appurtenances connected or attached to such buildings or structures.

The following are building codes referenced for the Basis of Design:

Governing codes and standards:

- – FBC: Florida Building Code, 2020
- – FBC: Accessibility, 2020
- – FBC: Energy Conservation, 2020

- Florida Fire Prevention Code, 7th Edition: Effective date: December 31, 2020 (based on NFPA 1, Fire Code, 2015 Edition and NFPA 101, Life Safety Code, 2018 Edition)

17.1.1 Interior

Construction materials and finishes include the following:

- Precast concrete floor slabs
- Precast concrete walls with a painted finish system – color white
- Precast concrete ceiling with a painted finish system – color white

17.1.2 Exterior

Construction materials and finishes include the following:

- Precast concrete walls with a flat smooth finish with a neutral beige coating system
- Precast concrete panels impregnated with a chemical moisture barrier to prevent seepage
- Hollow metal doors and frames with a painted finish system and vandal resistant hardware
- Doors with a Florida Product Approval or Miami-Dade Notice of Acceptance (NOA), including missile impact rating door shall be approved for use in High Velocity Hurricane Zone (HVHZ) where applicable
- Aluminum louvers with a clear anodized finish
- Portable ladder securing system

17.2 Life Safety

The buildings will be designed to meet the minimum construction and life safety requirements as required by the applicable codes and criteria. As described in Section 17.4, appropriate type, size, and quantity of fire extinguishers will be provided in compliance with all applicable fire and life safety codes.

17.3 Command and Control Room Requirements

17.3.1 Furnishings and Finishes

One (1) workstation with required accessories and all software (including SCADA software) loaded and configured. Design-Builder will provide desk and chair for workstation. Design-Builder will coordinate with the Owner regarding workstation requirements.

17.4 Material and Life Cycle

The buildings and canopy shall be designed to minimize life cycle cost, energy consumption, and maintenance through proper selection of mass, form, materials, and construction standards. The design life of the buildings and canopy shall be a minimum of 50 years. Refer to Section 12 for seismic and wind loading design criteria. The service life span will be the same as the building service life, except for the following: protective elements, wall, primary weather-barrier elements, joint sealers, surfaces exposed to view, and roof covering weather barriers. These will have varying service lives, as shown in Table 17-1.

Table 17-1. Varying Service Life Cycles

Material	Life Cycle
Protective Elements	Minimum 20 Years
Wall Primary Weather-Barrier Elements	Minimum 50 Years Functional and Aesthetic Service Life, Excluding Joint Sealers
Joint Sealers (Fuel Resistant)	Minimum 20 Years Before Replacement
Surfaces Exposed to View	Minimum 20 Years Aesthetic Service Life - No Color Fading, Crazing and Delamination of Applied Coatings
Roof Covering Weather-Barriers	Minimum 20 Years, Fully Functional

17.5 Proposed Future Activities

17.5.1 Preliminary Design

The following future activities are proposed for Preliminary Design:

- In the Preliminary Design phase, EIP will develop draft preliminary plans and specifications. In addition to these technical specifications, related to the architectural design, the draft POM, DDR, and regulatory plan will be updated, and a construction schedule and stipulated price proposal will be provided.
- Develop strategy for accessing pumps through the pump station canopy structure (access hatches, removable roof panels, or other).
- Determine the amount and type of furniture to be provided for the IT Room.

17.5.2 Final Design

- No additional items are anticipated during final design.

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HVAC AND PLUMBING

Building Mechanical design includes the design for the Heating, Ventilation and Air Conditioning (HVAC) in the PSS, precast concrete electrical buildings, generator buildings, and gate control buildings. A general description of the design criteria of these structures is in this section.

HVAC systems shall be designed to provide adequate working environments, to preserve equipment life (electrical and instrumentation panels), or to minimize or eliminate confined spaces. Design solutions will consider industry standards and best practices.

18.1 Heating Ventilation and Air Conditioning Overview

The LKBSTA project is designed to SFWMD standards and guidelines, as well as state and local building codes, and national standards applicable to the project. Standards used:

- 2020 FBC, 7th Edition
- 2020 Florida Plumbing Code, 7th Edition
- 2020 Florida Mechanical Code, 7th Edition
- 2020 Florida Energy Conservation Code, 7th Edition
- Sheet Metal and Air Conditioning Contractors' National Association (SMACNA) Design Guidelines
- American Society of Heating Refrigerating and Air-Conditioning Engineers (ASHRAE) HVAC Design Guidelines

18.1.1 HVAC Outdoor Design Conditions

Florida is designated as Climate Zone 2A by the IECC for the purposes of building envelope design criteria and minimum energy efficiency standards of equipment. Table 18-1 gives the ASHRAE Design Conditions for Orlando.

Table 18-1. ASHRAE Design Conditions for Orlando, FL

Heating DB (°F)		Cooling DB (°F)		20-year Record Temp (°F)	
99.6%	99%	0.4%	1%	Low	High
38.3	42.3	93.8	92.4	24.7	99.2

All areas shall be ventilated at a minimum rate per code to ensure air movement. Where heat-producing equipment is present, ventilation rate shall be set based on the heat production of the equipment, outdoor conditions, and temperature constraints of the equipment.

18.1.2 Design Criteria for HVAC Systems

The following is a summary of the assumptions used for the HVAC design:

- Generator rooms shall be ventilated as required by the generator. Ventilation fan will be provided for ventilation when generator is in standby.
- Electrical rooms will be ventilated as necessary to control room temperature due to heat gains from equipment operation and building envelope heat gain.
- Control buildings shall be provided with ventilation air per ASHRAE 62.1 standards, and mechanically cooled based on equipment loads inside the space.

- Ventilated rooms will be 10°F or less above outside temperature – if the outside is 90°F then interior will be 100°F or less. Ventilation cooling is for equipment, not comfort – buildings not occupied.
- HVAC equipment will be controlled locally. There is no site or building designated Building Management Systems (BMS). Any trouble alarms or status signals will be communicated generically through the site SCADA system.
- Fresh air openings shall be provided with intake/exhaust hood per district standard. Openings too large for a hood shall be provided with security louvers, Miami-Dade hurricane rated.
- Control rooms with temperature sensitive IT equipment will be cooled with mechanical cooling.
- No equipment in the project is sensitive to humidity, so dehumidification is not considered.

Individual requirements for each space are outlined below.

18.2 STA Inflow and Outflow Structures Control Buildings

Control Buildings at the STA inflow and outflow structures are small, single room, precast concrete structures with minimal equipment.

18.2.1 Design Criteria

The control rooms are unoccupied spaces. Ventilation will be provided to meet ASHRAE 62.1 Standards. Equipment load is between 200 and 300 watts maximum heat load. Ventilation air provided is sufficient to cover this minimal heat load.

18.2.2 Design Approach

An intake louver will be provided by the door. An exhaust fan will be placed across the room from the intake louver, to facilitate airflow across the room. This will ensure even and consistent ventilation and prevent heat build-up. Best practice is to avoid placing exhausts near doorways, where possible, to avoid short circuiting.

18.3 WCS-9 Control Building

This control building is 140 sf precast concrete structure.

18.3.1 Design Criteria

The control rooms are unoccupied spaces. Minimal ventilation will be provided to meet ASHRAE 62.1 Standards. Equipment load is between 200 and 300 watts maximum heat load. Ventilation air provided is sufficient to cover this minimal heat load.

18.3.2 Design Approach

An intake louver will be provided by the door. An exhaust fan will be placed across the room from the intake louver, to facilitate airflow across the room. This will ensure even and consistent ventilation and prevent heat build-up.

18.4 WCS-13 Control Building

This precast concrete structure houses a control room and a generator combined in a single building. Entrance to each room will be by means of exterior door, with no connection between the two spaces internally.

18.4.1Design Criteria

In the unoccupied control room, equipment load is estimated between 200 and 300 watts heat load. Ventilation air provided that will be provided to this room will be sufficient to cover this heat load.

In the generator room, the generator requires large amounts of intake air for radiator cooling and combustion. In addition, an alternator and additional electrical equipment in the room requires additional ventilation airflow for cooling. These loads shall be obtained from the generator manufacturer, and inlets, outlets, and fans shall be sized such that the generator will have the air required for operation.

18.4.2 Design Approach

In the control room an intake louver will be provided by the control room door. An exhaust fan will be placed across the room from the intake louver, to facilitate airflow across the room.

The generator room will be designed with both the needs of the generator while operating and the needs of the room when the generator is not operating in mind. A large hurricane-proof intake (hood or louver based on airflow) will be provided, sized to provide the cooling air and combustion air the generator requires. An exhaust hood/louver for the generator cooling will be provided based on the generator’s cooling airflow requirements. A separate exhaust fan inside the room will provide ventilation airflow when the generator is not in operation.

18.4.3PS Electrical Building

This is a precast concrete building that houses an electrical room and multiple pieces of electrical equipment.

18.4.4 Design Criteria

The ventilation equipment will be sized for not more than a ten degree temperature rise above ambient within the room from heat let off from the electrical equipment. Thermostatic control shall be provided to ensure the electrical building does not rise above the design temperature. The electrical equipment is limited to a maximum operating 154°F operating temperature. Filtration will be provided to ensure dust, dirt, and debris are not drawn into the room along with the ventilation air.

18.4.5 Design Approach

Electrical equipment loads will be obtained during design and building details and equipment shall be run through load calculation software to determine the ventilation requirements for the building. Air shall be provided by fans in regular intervals, designed to spread fresh outdoor air over the heat-producing equipment, and air shall be exhausted by fans opposite the supply fans for continuous flow of ventilation air. All incoming air will be filtered. The internal temperature will be continuously monitored by thermostat.

18.5 Pump Control Station with Generator

This precast concrete building consists of a generator room, control room, and IT room.

18.5.1 Design Criteria

When in operation, the generator requires large amounts of intake air for radiator cooling and combustion. In addition, an alternator and additional electrical equipment in the room requires additional ventilation airflow for cooling. These loads shall be obtained from the generator manufacturer, and inlets, outlets, and fans shall be sized such that the generator will have the air required for operation.

The IT room within the building shall be provided with mechanical cooling for sensitive equipment related to the functioning of the Control Station. This mechanical cooling shall be provided by split systems with the condensing unit located inside the generator room for security. Mechanical cooling shall maintain the space at 85°F or below year round.

In the unoccupied control room, equipment load is between 200- and 300-watts maximum heat load. Ventilation air provided is sufficient to cover this heat load.

18.5.2 Design Approach

The IT room shall be cooled by a split system and a thermostat. Split system shall be entirely indoors, and condensate shall be piped through the wall to a splash pad outside the building. The condensing unit for the IT room will be placed within the generator room, to protect from the elements and vandalism.

For the generator room, the alternator and equipment cooling will be handled by a sidewall fan, while the Generator Radiator is cooled with its own fan. Exhaust louvers will be provided for the generator and a single intake louver will provide air for the building. The intake louvers and exhaust fan will be sized to account for the heat added to the space by the condensing unit, and to remove that heat from the space.

In the control room an intake louver will be provided by the control room door. An exhaust fan will be placed across the room from the intake louver, to facilitate airflow across the room.

18.5.3 Plumbing Design

The Control Station shall be provided with a restroom consisting of a composting toilet and hand sanitizing station. The hand sanitizing station shall be designed to work without potable water, since none is available on site. The composting toilet does not require water.

18.6 Proposed Future Activities

18.6.1 Preliminary Design

The following future activities are proposed for Preliminary Design:

- In the Preliminary Design phase, EIP will develop draft preliminary plans and specifications. In addition to these technical specifications, related to the HVAC and plumbing, the draft POM, DDR, and regulatory plan will be updated, and a construction schedule and stipulated price proposal will be provided.
- Details on the requirements of composting toilets for installation and maintenance will need to be addressed.

18.6.2 Final Design

No additional items are anticipated during final design.

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SPECIAL CONSIDERATIONS

Special considerations for the Project include site-specific and project-specific design modifications. These considerations include site-specific environmental considerations related to wetlands and endangered species. These considerations also include project-specific considerations related to recreation, access, and security.

19.1 Environmental

19.1.1 Wetland Impact and Benefits

The wetlands onsite consist predominantly of freshwater marsh and wet prairie heavily impacted by agricultural activities. There are approximately 265 ac of wetlands, all of which have been significantly impacted by many miles of ditches that drain the water out as quickly as possible in order to keep the area suitable for cattle grazing. Nearly all of the native vegetation has been eliminated and replaced with Bahia grass and infested with exotic and nuisance species. Because of the extremely low quality of the wetlands and the minimal ecological function, wetland avoidance during the design is not necessary. Furthermore, the wetland ecological function of the freshwater marshes created in the STA cells will be significantly higher than the existing wetland ecological function, providing a net benefit to fish and wildlife, not only onsite but in the surrounding area and downstream.

19.1.2 Threatened and Endangered Species

Several threatened and/or endangered species have been identified as occurring or with high potential for occurrence onsite. These species necessitate consideration during design, permitting, construction, and operation. These special considerations for the species include the following:

- Eastern indigo snake – although this species has not been directly observed onsite, there is a potential for occurrence onsite. Training will be conducted prior to construction for all contractors to ensure that this species is not harmed incidental to construction activities. Also, during gopher tortoise burrow excavation, if this species is encountered the proper procedures will be followed for relocation. There will be a qualified biologist onsite during construction to monitor this species and ensure there are no adverse effects.
- Bald eagle – there is one verified bald eagle nest onsite, and one potential nest near the edge of the Project. The trees with nests will be avoided as part of the Project design, and construction will not occur within the nest buffer that is determined to be appropriate during the permitting process. There will be a qualified biologist onsite during construction to monitor this species and ensure there are no adverse effects.
- Audubon's crested caracara – there are three nests in the vicinity of the Project, one onsite and two offsite but close by. The Project design has already been modified to avoid the onsite nest tree cluster. Surveys will be conducted to determine the territorial utilization of the species onsite. These birds will be monitored during construction, and during the nesting season. Construction will not occur within the nesting territory determined in coordination with USFWS. After construction, the birds will continue to be monitored to determine the effects, if any, of the Project on reproductive viability. There will be a qualified biologist onsite during construction to monitor this species and ensure there are no adverse effects.

- Gopher tortoise – there are many gopher tortoise burrows located within the Project boundary and on immediately adjacent lands. The onsite gopher tortoises will be relocated prior to construction, and barriers will be placed at appropriate locations to keep offsite gopher tortoises from coming onto the site. Additionally, after construction, barriers will be placed at appropriate locations to prohibit gopher tortoises from immigrating onsite. The nature and location of these barriers will be developed with FFWCC during the permitting process. There will be an Authorized Agent onsite during construction to monitor this species and ensure there are no adverse effects.
- West Indian manatee – this species is known to occur in the C-38 canal. Therefore, protection devices will be included in the design of the structures that contact the L-62 canal and C-38 canal to ensure the safety of this species. During construction, there will be a qualified biologist onsite to monitor this species and ensure there are no adverse effects.
- Migratory birds – an avian protection plan or equivalent will be developed during the permitting process to address migratory bird utilization of the site during construction and during operation. During construction, particular attention will be focused on wading birds and shorebirds utilizing areas for nesting and avoiding the nests if this occurs. Post-construction, the STA cells may attract migratory birds, including the Everglades snail kite and a variety of wading birds. If this occurs, a management plan will be implemented. There will be a qualified biologist onsite during construction to monitor this species and ensure there are no adverse effects.

19.1.3 Recreation

Recreation and public access planning at the Project will meet the District policies and follow typical designs of recent projects. Public access locations will be designed to support nature based recreation in accordance with SFWMD standards. Space for the earthworks to provide public access to the Project and pedestrian circumnavigation of the Project will be incorporated during the design, but no public access will be allowed during EIP ownership.

Due to some uncertainty, common to projects, it is not necessary to narrowly define the expected activities. After the Project is operated for some time, the actual activities allowed will be determined and managed using the initial access facilities provided. With demonstrated need, additional “bolt on” facilities can be built later, on the existing earthworks. The public use activities are controlled by District Public Use Rule 40E-7.

19.2 Access

Pedestrians would have access to most of the site with potential fencing limited to areas around the PS. All buildings would be locked. Vehicular public access would be limited by locked gates at all roadways up to external perimeter embankments.

The primary private secured access would be constructed at the northwest corner off State Road 70. The secondary private secured access location is at the end of SW 21st Parkway off Granada Avenue. This secondary location has enough space for potential public access in the future. The primary and secondary gates are used for both temporary access during construction and permanent access after the Project is complete.

19.2.1 Fire Protection

It is possible that an automatic fire sprinkler and detection system will be required for the entire PS facility. Further code investigation will confirm this requirement during detailed design. If a sprinkler system is required, a pre-action system will be provided. The sprinkler system and portable fire extinguishers will be installed in accordance with NFPA 13 standards.

The SFWMD will review all design assumptions, criteria, and calculations. Verification with the SFWMD and the SFWMD's insurance underwriter shall be done for the fire protection systems.

Fire sprinklers are not anticipated in the control buildings since they are considered unmanned structures and house operational equipment only. Dry type portable fire extinguishers will be provided as fire protection in the control buildings for telemetry equipment and WCSs.

With regards to controlled burns of vegetation on the site, accommodations in design of both PES and STA infrastructure and burn methodology will be investigated for fire protection.

19.3 Security

Physically securing the Project's facilities from un-authorized persons, as well as safety monitoring, is accomplished by using electronic security systems. The criterion is discussed here as its components impact the PS. Some aspects overlap with the Access section above. The security systems at each PS will be evaluated and integrated into the system-wide SCADA and communications system.

There will be an access control/security system for all buildings on the Project. The system will be compatible with the existing SFWMD-wide security system and coordinated with the SFWMD for the latest hardware and software requirements. With vehicular access limited to structures but pedestrian access unimpeded, building materials will be selected based on qualities of having long term durability, being low maintenance, and being highly vandal resistant.

There are three major areas of concern regarding site security from the public: neighboring residents, access from State Road 70, and access from the existing HHD.

With regard to site security from neighboring residents, it is assumed that fencing and gates, are needed for features directly adjacent to neighboring residents (i.e., substation). It is assumed due to the remoteness of the site's internal features (inflow/outflow structures, PS, PES) along with the seepage canal running the site perimeter that fencing and gates will be limited around these features.

Installation of fencing along the eastern portion of the site to prevent access by cattle from neighboring ranch lands into areas outside the Project grading limits will be investigated. Any existing ranch leases within the Project boundary will not be renewed and lessee-owned equipment will be removed.

Access along most of the south side of State Road 70 will be limited by the seepage canal running along the northern perimeter. The primary site access from State Road 70 off the northwest corner of the site will have a locked gate.

For securing the existing HHD along the west side of the Project site, an existing District gate currently used for maintaining structure S-65E will remain in place for site security. District access to the HHD will remain the same with additional security measures as required along the proposed extended portions of the HHD.

19.4 Decommissioning of Wells

Existing wells within the Project limits will be plugged and abandoned following District specifications or transferred to EIP/District ownership as necessary.

19.4.1 Revising or Closing Out Use Permits

Any existing consumptive use and environmental resource permits within the Project boundary will be transferred, revised, or closed out as necessary.

19.5 Proposed Future Activities

19.5.1 Preliminary and Final Design

Pre-application meetings have been initiated with the agencies responsible for permitting of the threatened and endangered species, including USFWS and FFWCC. As the Project progresses and permit applications are developed and submitted, discussions with these agencies will continue through Preliminary and Final Design.

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PUBLIC ENGAGEMENT

No adverse public impacts have been identified at this time. The Project site is relatively remote and primarily surrounded by agricultural lands. There are a few residential communities located west and north of the Project site and State Road 70 is located north of the Project site. The Project is being designed to maintain the existing level of flood protection provided by the L-62 canal and to avoid negative impacts to adjacent lands and all associated stormwater management systems, drinking water wells and onsite sewage treatment and disposal systems (OSTDS) or septic tanks.

20.1 Stakeholders

Key Project stakeholders that have been identified to date are provided in Table 20-1. The list of stakeholders will continue to be modified as the Project advances.

Table 20-1. Key Project Stakeholders

Stakeholder Type	Stakeholder/Organization
Federal Government	U.S. Army Corps of Engineers (USACE)
Federal Government	U.S. Fish and Wildlife Service (USFWS)
Federal Government	U.S. Department of Interior (USDOI)
Federal Government	U.S. Natural Resources Conservation Service (NRCS)
State Government	Florida Department of Environmental Protection (FDEP)
State Government	Florida Fish and Wildlife Conservation Commission (FFWCC)
State Government	Florida Department of Agriculture and Consumer Services (FDACS)
State Government	Florida Department of Transportation (FDOT)
State Government	Florida Division of Historical Resources / State Historic Preservation Office (SHPO)
State Government	South Florida Water Management District (SFWMD)
Local Government	Okeechobee County
Local Government	Highlands County
Local Government	Glades County
Local Government	Hendry County
Local Government	St. Lucie County
Local Government	Martin County
Local Government	Palm Beach County
Local Government	Lee County
Local Government	Charlotte County
Local Government	City of Okeechobee
Tribal	Seminole Tribe of Florida (STOF)
Tribal	Miccosukee Tribe of Indians of Florida (MTIF)
Business	Florida Power & Light Company (FPL)

20.2 Public Access and Recreation Facilities

Design accommodations will be made to provide space for future public access and recreational facilities such as informational kiosks, picnic shelters, signage, etc. The design, construction, and maintenance of specific infrastructure associated with public access to the Project and future recreational facilities is proposed to be implemented by the SFWMD after EIP transfers the land or turns over the Project to the SFWMD.

20.3 Proposed Future Activities

20.3.1 Preliminary Design

EIP intends to coordinate closely with the SFWMD to facilitate engagement with stakeholders due to the fact that water quality improvement projects within the Lake Okeechobee watershed are of critical importance to numerous communities, state and federal agencies and tribal nations.

20.3.2 Final Design

At this time, there are no future public engagement activities proposed during Final Design.

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PROJECT OPERATIONS PLAN

The Project Operations Plan, also referred to in this document as the Project Operations Manual (POM), provides a framework of recommended operational guidelines ranging from day-to-day to special operations and is provided in Appendix 5. The POM includes information related to operational objectives, start-up operations, normal operations, non-normal operations, structures, seepage management facilities and the PES.

21.1 Proposed Future Activities

21.1.1 Preliminary Design

The EIP team plans to update the POM as the design progresses through Preliminary Design.

21.1.2 Final Design

The EIP team plans to update the POM as the design progresses through Final Design.

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50-YEAR LIFE CYCLE COST ANALYSIS FOR OPERATIONS AND MAINTENANCE

Historically, the District has utilized traditional design-bid-build contracting practices to construct STAs on state-owned land. However, due to the innovative nature of this Project's delivery model, risks associated with Project implementation are shifted from the District to EIP, as contract payments are tied to EIP successfully meeting Project milestones, including designing, constructing, and operating the Project.

At the conclusion of Phase One of the Project, EIP will submit a proposal for Phase Two (Stipulated Payments and Deliverables Proposal), which is expected to include a detailed fee proposal and schedule for final design activities, permitting, land transfer, construction, operations and turnover to SFWMD. This approach allows for the establishment of a fixed fee to complete the Project, as opposed to the typical design-bid-build practice which relies on cost estimates. As such, an opinion of probable construction costs was not included in this report. This section provides preliminary estimated quantities for the Project and a 50-year lifecycle cost analysis for operations and maintenance.

22.1 Preliminary Estimated Quantities

The preliminary estimated quantities for the Project are provided in Table 22-1 to assist in documenting the scope and scale of construction activities anticipated for the Project, which include the following:

- One (1) inflow PS
- Six (6) STA cell inflow WCS (WCS; #1a-6a)
- Six (6) STA cell outflow WCS (#1b-6b)
- Three (3) PES related inflow/outflow WCS (#7a, 7b, 8)
- One (1) elevated conduit to enable pumped STA inflows to be conveyed across the L-62 canal to Cells 5 and 6 and the PES (#10)
- One (1) elevated conduit to enable STA outflows to be conveyed across the re-routed L-62 canal (#12)
- One (1) WCS within the re-routed L-62 canal located just upstream of the inflow PS (#9)
- One (1) WCS within the re-routed L-62 canal located at/near the confluence of the re-routed L-62 canal and the C-38 canal (coincident with the existing HHD alignment) (#13)
- One (1) WCS to manage seepage from Cells 5 and 6 and the eastern inflow canal (#11)
- Grading/land leveling of six (6) STA cells
- Excavation of STA inflow/outflow canals and construction of associated embankments
- Backfilling of a portion of the existing L-62 canal
- Excavation of the new (re-routed) L-62 canal and construction of associated embankments
- Excavation of spreader/distribution and collection canals internal to the six (6) STA cells

- Excavation of seepage collection/offsite stormwater management canals and construction of associated embankments
- Construction of external and internal embankments

Table 22-1. Preliminary Estimated Quantities

WCS and PSs	
Number of Cell WCS	12
Number of PSs	1
Number of Elevated Conduits	2
Number of WCS in L-62 Canal	2
Number of PES WCS	3
Number of Seepage Discharge WCS	1
Total Number of Structures	21
Length of Embankment	
Seepage Collection (linear ft)	35,600
External (linear ft)	35,120
Internal (linear ft)	69,100
Inflow/Outflow Canal (linear ft)	45,400
Spreader/Distribution Canal (linear ft)	28,900
Collection Canal (linear ft)	26,600
L-62 Canal Reroute (linear ft)	6,900
Total Length of Embankment (linear ft, (mi))	247,620
Embankment Area	
Embankment Sloped Area (ac)	101.3
Embankment Flat Area (ac)	113.1
Total Embankment Area (ac)	214.4
Excavation (cy)	1,407,000
Grading (cy)	3,801,000
Sod (sy)	1,053,000

22.2 50-Year Lifecycle Cost Analysis for Operations and Maintenance

22.2.1 Stormwater Treatment Area

EIP prepared an updated cost analysis incorporating operation and maintenance (O&M) costs provided in SFWMD's Engineering Submittal Requirements document (dated March 22, 2016) and information specific to the alternative identified by the EIP Team in the Reconnaissance Study to be advanced to the DDR and Preliminary Design phases (e.g., PS operation and maintenance, levee maintenance, control structure maintenance, mowing, STA vegetation management, electrical costs, operations and permit compliance monitoring, staff time, etc.). This cost analysis resulted in an O&M cost estimate of $721 per acre per year, which is slightly higher than the $636 per acre per year O&M cost provided in the Reconnaissance Study Final Report.

SFWMD data indicates that in Fiscal Year 2019, SFWMD expended $22 million for the O&M of 57,000 acres of STA effective treatment area. This figure includes all O&M salaries, fuel, electricity, structure maintenance, canal and levee maintenance, mowing, seeding/sodding, vegetation management, site management, telemetry, and compliance monitoring of water levels, flows and water quality. This equates to an average unit O&M cost rate of $386 per acre per year. When escalated from 2019 to 2022 dollars, the average unit O&M cost rate is $416 per acre per year.

A range of annual O&M costs was calculated: 1) a "low" annual O&M cost estimate was calculated using the $416 per acre per year cost derived from cost information provided by SFWMD for the 57,000 acres of STA described above; and 2) a "high" annual O&M cost estimate was calculated using the EIP-prepared costs developed specifically for the advanced alternative ($721 per acre per year).

To estimate the total projected O&M cost for a 50-year life cycle, a discount rate of 2.5% was used, which is equal to the average annual Consumer Price Index (CPI) increase during 2000-2022 (source: U.S. Bureau of Labor Statistics https://www.bls.gov). Table 22-2 provides the estimated annual and 50-year O&M costs for the alternative identified by the EIP Team to be advanced to the DDR and Preliminary Design phases. Detailed information is provided in Appendix 6.

Table 22-2. Estimated Annual and 50-Year O&M Costs (STA)

Estimated Effective Treatment Area (acre)	2,500
Estimated Annual O&M Cost per Acre – low ($) [1]	$416
Estimated Annual O&M Cost per Acre – high ($) [1]	$721
Estimated Annual O&M Cost – low ($) [1]	$1,040,000
Estimated Annual O&M Cost – high ($) [1]	$1,803,000
Estimated 50-year O&M Cost – low ($) [2]	$29,500,000
Estimated 50-year O&M Cost – high ($) [2]	$51,100,000

Notes:

1 – 2022 dollars

2 – Present Value

In addition, the EIP team obtained annual O&M cost projections from SFWMD for two fiscal years for Lakeside Ranch and Taylor Creek STAs. The annual cost projections provided are for field station activities only and do not include other STA management costs (e.g., vegetation management, water quality sampling, etc.). Lakeside Ranch STA annual O&M costs range from $507,238 to $724,196 and average $616,000 per year. Taylor Creek STA annual O&M costs range from $75,854 to 92,755 and average $84,000 per year. With effective treatment areas of 1,707 acres (Lakeside Ranch STA) and 118 acres (Taylor Creek STA), O&M costs for Lakeside Ranch STA and Taylor Creek STA are $361 and $714 per acre per year, respectively.

The higher per acre O&M costs for the Project, as compared to Lakeside Ranch STA for example, are not unexpected due to the higher electricity costs from the anticipated near year-round use of the inflow PS and due to higher maintenance costs for the other Project structures.

22.2.2 PES

The EIP team prepared a cost analysis to estimate the O&M costs associated with the PES, an innovative water quality treatment technology proposed for the Project consisting of a vertical engineered media filtration system.

The estimated annual and 50-year O&M costs were prepared for a 6-acre PES module (see Table 22-3), also using a discount rate of 2.5%. Annual PES O&M activities include weekly inspections, sampling and monitoring, and trench cleaning, plant burning, and parts replacement, as needed. For planning purposes, this cost analysis also assumed PES media replacement twice in the 50-year life of the Project (in years 20 and 40). PES media replacement includes costs to excavate, load, haul and dispose of the spent media (including tipping fees at the Okeechobee landfill), deliver, prepare, and spread new media in the PES facility and plant emergent aquatic vegetation in the new PES media. Detailed information is provided in Appendix 6.

Table 22-3. Estimated Annual and 50-year O&M Costs for a 6-acre PES

	Annual Cost ($) [1]	Total Cost ($) [1]	50-year O&M Cost ($) [2]
Estimated PES O&M Cost	$116,000	$5,800,000	$3,290,000
Estimated PES Media Replacement Cost (Year 20)	NA	$9,330,000	$5,694,000
Estimated PES Media Replacement Cost (Year 40)	NA	$9,330,000	$3,475,000
Total		$24,460,000	$12,459,000

Notes:

1 – 2022 dollars

2 – Present Value

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CONSTRUCTION COORDINATION

It is envisioned that the LKBSTA Project will be constructed using conventional means and methods. The features are designed to capitalize on the use of on-site material, reduce multiple handling scenarios, utilize existing infrastructure where appropriate, and maintain flood control operations and level of service provided by existing features.

23.1 Permits

As a District project, Okeechobee and Highlands Counties may waive the building permit requirements, however, at this time, it is expected that building permit(s) will be required. FPL may also require a building permit to obtain a structure address. See Section 4 (Regulatory Considerations) for additional information on permits.

23.2 Schedule

The LKBSTA construction is expected to start as early as 12 months after execution of the Phase Two contract. Construction is expected to last 18-24 months, with substantial completion expected within 18-20 months and final completion expected within 20-24 months. Construction activities will include efforts to advance the establishment of vegetation in specific constructed STA cells prior to implementing flow-through operations for the entire Project. Construction of infrastructure that requires Section 408 authorization by the USACE will commence once approval is granted, however, construction in areas that do not require this approval are expected to begin earlier.

The submittal and work plan review process can begin to provide a pathway for early works such as Stormwater Pollution Prevention Plan (SWPPP) initiation, temporary utilities, access and security control establishment, as well as drainage work that must take place to maintain the safety of adjacent properties. This initial phase is expected to require approximately 60 days. As this initial phase is completed, heavy earthwork, structure dewatering and excavation, shoring and heavy equipment access activities will commence. Major milestones will be identified through the critical path method (CPM) schedule that will include stationed canal and embankment phases associated with STA cell construction including WCS. Definable features of work may include those associated with cast in place completion milestones, mechanical infrastructure installation, energizing, STA cell water level conditions and vegetation establishment, and commissioning. The adopted milestones will be agreed upon by all relevant parties and a CPM schedule will be developed.

23.3 Other Projects Affecting Construction

At this time, there are no known projects, either in design or construction, that are anticipated to affect the LKBSTA Project.

23.4 Construction Sequencing

Initial perimeter best management practices (BMP) will be installed prior to clearing and grubbing activities advancing. Initial construction activities will include early works throughout STA Cell 1 (Figure 23-1) to allow for early drainage management through existing culverts that deliver stormwater from the north side of State Road 70. Managing off-site water entering the construction area will be a top priority before moving into on-site construction activities. This ensures existing drainage of offsite areas continues as required and minimizes the potential impacts of offsite water on construction activities. As earthwork advances, seepage canals and perimeter berms will be constructed prior to interior cell grading, facilitating on-site stormwater management.

During the LKBSTA Preliminary Design, decisions will be finalized regarding the specific breakdown and scheduling of construction definable features of work. It is anticipated that the LKBSTA will be constructed under the following major construction definable features of work.

- Off-Site drainage conveyance and perimeter embankment construction
- HHD WCS construction
- LKBSTA headworks construction
- STA cell construction
- PES construction

23.5 Staging Areas

Proposed staging areas associated with the LKBSTA construction are shown in Figure 23-1.

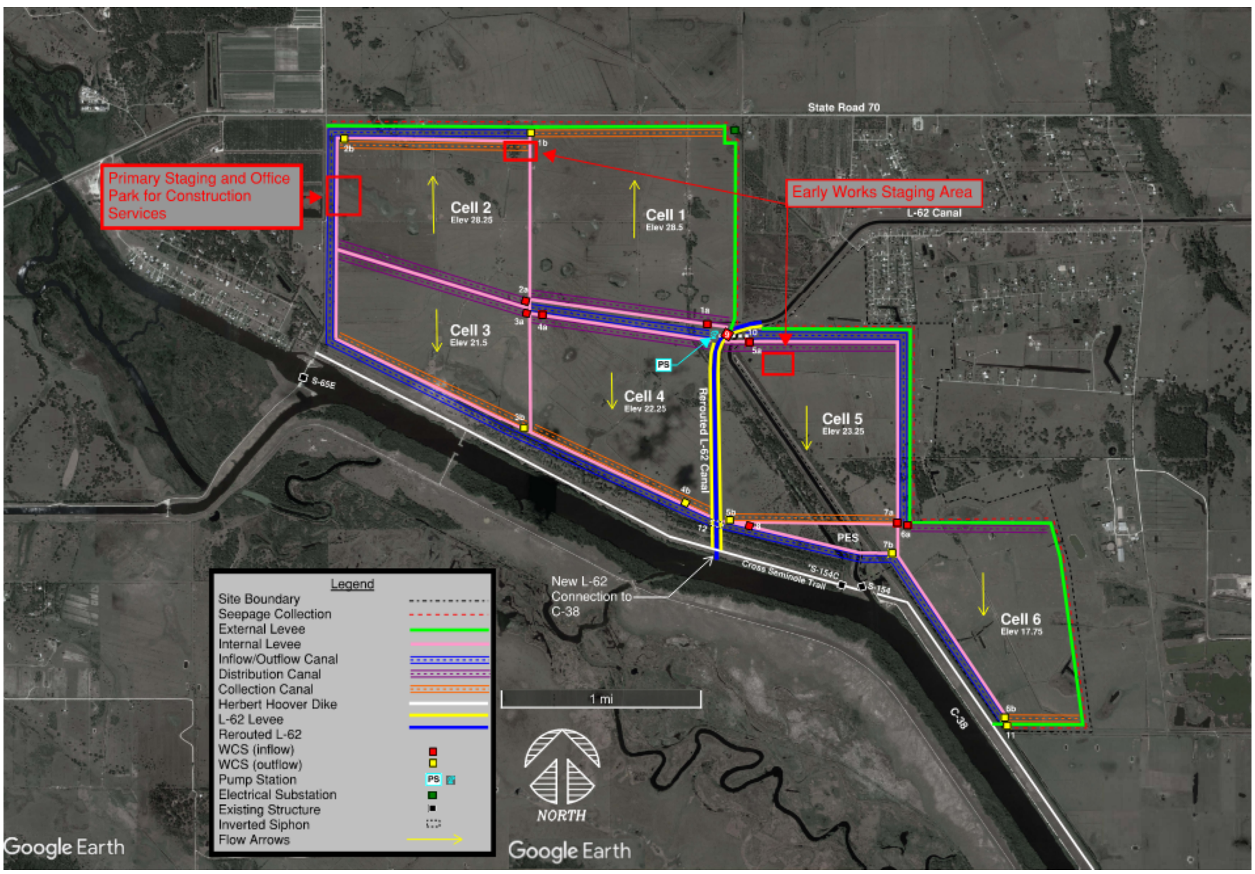

Figure 23-1. Staging Areas

23.6 Construction Access and Traffic Routing

Site access will initially be provided via the existing entrances along State Road 70, SW 128th Avenue, and SW 87th Terrace. Permanent construction entrances will be constructed from SW 128th Avenue (Cell 2), and SW 87th Terrace (Cell 5). Site security will be maintained throughout construction and will include necessary security fencing, gates, and video monitoring. If found necessary, additional security including full-time staff and/or local officials will be called upon for 24-hour duty. Access through non-construction areas from south of these three access points will not be permitted without prior authorization. Escorts will be mandatory for non-Project related personnel.

23.7 Demolition and Disposal

The existing agricultural and residential buildings, drainage culverts, and fencing within the LKBSTA Project will be demolished when appropriate and materials will be disposed of in accordance with all local and state requirements. Certain mechanical equipment may be transported to an offsite location in accordance with relevant contract documents. Results from the ESA Phase 2 will help determine treatment and disposal mitigation measures and will be included in future deliverables. Unless the on-site groundwater wells are re-purposed for use by the Project, they will be properly plugged and abandoned in accordance with relevant state requirements.

23.8 Agency Review

Prior to construction initiation, construction kick-off meetings will be conducted with all agencies that have issued permits. Agency consultation will continue throughout construction, in particular with regard to wildlife monitoring as it is likely that there will be areas within and timeframes that construction cannot occur. For example, during the nesting season for the bald eagle and Audubon's crested caracara, certain areas must be avoided. All such permit restrictions will be complied with and continuous close coordination with relevant agencies is anticipated. There will be on site job specific training, associated with environmental management and threatened and endangered species, with staff prior to construction.

23.9 Quality Assurance/Quality Control Requirements

The EIP Team proposes to follow, in strict compliance, a Construction Quality Management/Control Plan similar to and crafted around the USACE methodology. All team members will be selected based on similar project experience. All key leadership resumes will be provided to EIP, the District, and the Engineer of Record prior to starting construction activities. Utilization of a three-step acceptance process along with third party participation is proposed and ensures complete transparency throughout the construction process.

Throughout the construction phases of work, the Contractor Project Manager (CPM) will be responsible for overall Quality Control (QC) of the Project. The Contractor Quality Control Manager (CQCM) will be responsible for overall site implementation of the Construction Quality Management/Control Plan, in close coordination with District Staff, sub-contractors, and Contractors Management on site. The CQCM will report to Contractor's corporate office and safety staff, not the on-site CPM.

Through third party inspection as well as internal tracking, the EIP team proposes to utilize the following five-phase inspection method and work process for each definable feature of work: 1) preparatory phase with approved work plan; 2) initial phase; 3) follow-up phase; 4) pre-final inspection; and 5) final inspection.

As construction work progresses, a deficiency tracking log will be utilized for tracking construction deficiencies from identification through corrective action.

Prequalified independent sampling and testing labs, based upon standards adopted by the Engineer of Record, will be utilized on site and report directly to EIP. On site laboratories may be utilized and certified in accordance with USACE and/or other jurisdictional standards. Preparation of the

Construction Quality Management/Control Plan is ongoing and will be delivered with the LKBSTA Preliminary Design.

23.10 Communications and Data Processing Systems

23.10.1 Testing and Commissioning

The purpose of testing and commissioning plan is to ensure that all systems fully satisfy design intent, and the operating staff are trained to operate and maintain the facility.

Our approach involves distinct steps critical to ensuring that the new systems operate as intended. The first step is startup and testing, followed by commissioning. Startup and testing activities are intended to ensure that equipment can be energized without the risk of damage or injury. Commissioning activities prove that the systems deliver the expected level of functionality and reliability.

A commissioning team will be involved in the Project from design through Project closeout. During the preconstruction phase, the team will develop preliminary commissioning and startup procedures to review the design and ensure it meets the operability objectives. During construction, startup and testing, activities will be coordinated with all relevant Project stakeholders and performed by construction personnel, equipment manufacturer representatives, and SFWMD, as required. The results will be verified by a standardized checklist that will be used to guide these activities. This checklist will be used to provide written notification to EIP and SFWMD when a portion of the Project is ready for commissioning.

Operator training will include preparation, implementation, and performance of comprehensive staff training programs specifically geared to the upgraded systems. Both classroom and field training will be conducted by the Project team and manufacturers' representatives.

23.10.2 Outage (Downtime)

The following procedures will be followed in anticipation of and during downtimes:

- Collaborate with the design engineer and operations staff to develop detailed plans and contingencies for each of the required shutdowns, bypasses, and/or tie-ins.
- Develop detailed step-by-step procedures and utilize BIM and models for the construction process (MOPO).
- Maintain extra materials on hand.
- Close coordination with operations staff with LKBSTA contingency plans.
- Conduct work during low flow periods to minimize downtime to critical systems.

23.10.3 Removal/Disposal

Proper removal and disposal procedures will be established during the preconstruction phase. Hazardous materials will be identified and properly disposed of per state and federal guidelines with chain of custody documentation provided upon request.

23.10.4 Decommissioning

When systems are no longer required for operations, systems will be decommissioned. The team will develop detailed step-by-step procedures for decommissioning with reviews conducted by all relevant Project stakeholders. All plans will include detailed instructions for lock out/tag out for proper isolation of decommissioned equipment. The Project team will work with SFWMD to identify equipment for salvage and demolition.

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EMERGENCY ACTION PLAN

An Emergency Action Plan (EAP) is commonly defined as a plan developed by a property owner that establishes procedures for notification to State and Federal agencies, public off-site authorities, and other agencies of emergency actions to be taken in an impending or actual failure of a high hazard impoundment. Agencies with EAP guidance include the Federal Energy Regulatory Commission (FERC), United States Bureau of Reclamation (USBR), State Dam Safety Offices, as well as local community/county representatives. Impoundments designated as high hazard typically require the most stringent and detailed emergency action plans. As discussed in Section 8, LKBSTA is anticipated to be a low hazard impoundment at this time. No direct loss of life is anticipated in the case of a breach of the Project embankments.

Further analysis to determine if the Project instead falls into a significant or high hazard classification will be completed in the Preliminary Design phase, including an impoundment breach analysis (see Section 8 for further discussion). Neighboring developments, including a tree farm and residential subdivisions near the Project site could potentially be impacted in the event of a breach. If the Project classification is determined to be a significant or high hazard impoundment, the EAP would need to reflect as such and need to be developed in conjunction with impoundment breach modeling.

24.1 EAP (External Impacts)

The following sections describe the EAP regarding impacts to areas external to the Project.

24.1.1 North Perimeter (State Road 70)

24.1.1.1 Land Use Description

The area north of the Project parallel to State Road 70 along Cells 1 and 2.

24.1.1.2 Direction of Flow

A breach of the north perimeter embankment would initially result in flow to the north directly into State Road 70 due to Project head and then due to elevation difference the flow would go back south towards the Project.

24.1.1.3 Damage Potential

A breach along this section could result in State Road 70 being overtopped or roadway pavement being damaged. Additionally, the palm tree farm northeast of the Project site could be impacted. As mentioned in Section 8, a Dam Break Analysis will be completed during preliminary design to assess the extent of potential damage in the event of an embankment breach.

24.1.2 Northeast Perimeter (Adjacent to Residential Neighborhood and Palm Tree Farm)

24.1.2.1 Land Use Description

The northeastern corner of the Project site east of Cell 1 and north of Cell 5 and adjacent to residential neighborhood and palm tree farm.

24.1.2.2 Direction of Flow

A breach of the northeastern perimeter embankment would initially result in flow to the northeast directly into the existing palm tree farm and potentially into the adjacent residential neighborhood.

24.1.2.3 Damage Potential

A breach along this section could result flooding of the palm tree farm and residential neighborhood northeast of the Project site. As mentioned in Section 8, a Dam Break Analysis will be completed during preliminary design to assess the extent of potential damage in the event of an embankment breach.

24.1.3 East Perimeter (Along Cell 5 and Cell 6)

24.1.3.1 Land Use Description

The area east of the Project along Cells 5 and 6.

24.1.3.2 Direction of Flow

A breach of the eastern perimeter embankment would initially result in flow to the east directly into the existing cattle ranch.

24.1.3.3 Damage Potential

A breach along this section could result flooding of the neighboring ranch east of the Project site. It is likely to have minimal damage. As mentioned in Section 8, a Dam Break Analysis will be completed during preliminary design to assess the extent of potential damage in the event of an embankment breach.

24.1.4 West Perimeter (SW 128th Avenue)

24.1.4.1 Land Use Description

The area west of the Project along Cells 2 and 3 parallel to SW 128th Avenue.

24.1.4.2 Direction of Flow

A breach of the northwestern perimeter embankment would initially result in flow to the west directly and would flow back into Cells 2 and 3.

24.1.4.3 Damage Potential

A breach along this section could result flooding of the neighboring ranch west of the Project site. It is likely to have minimal damage. As mentioned in Section 8, a Dam Break Analysis will be completed during preliminary design to assess the extent of potential damage in the event of an embankment breach.

24.1.5 Southwest Perimeter (HHD)

24.1.5.1 Land Use Description

The area south of the Project along Cells 3, 4, 5 and 6 parallel to HHD.

24.1.5.2 Direction of Flow

A breach of the southern Project perimeter embankment between Cells 3 and 4 would result in water stacking up in the outflow canal at WCS-12. A breach of the southern Project perimeter embankment parallel to Cell 5 would result in flows to the south into the PES and discharge out of the existing S-154 structure. A breach of the southern Project perimeter embankment located along the west side of Cell 6 would result two potential flow directions: (1) The flows could stack up at WCS-11 and (2) the flows could move south of the Project directly into the existing ranch.

24.1.5.3 Damage Potential

A breach along this section could result either the flooding of the Project cells or the flooding of the existing property south of the Project site. It is likely to have minimal damage. As mentioned in Section 8, a Dam Break Analysis will be completed during preliminary design to assess the extent of potential damage in the event of an embankment breach.

24.2 EAP (Internal Project Impacts)

The following sections describe the EAP regarding impacts to areas internal to the Project.

24.2.1 HHD Realignment (WCS-13)

WCS-9 will be designed to have bidirectional flow to be able to withstand the water from the L-62 in the event of WCS-13 failing. To avoid impacts to the Project site if WCS-13 failed WCS-9 would remain open to allow flow to L-62 in order to avoid flooding of the Project inflow PS.

24.3 Dam Break Analysis, including Flood Inundation Mapping

It is anticipated that a dam break analysis will be completed during the Preliminary Design phase of work (see Section 8 for additional information).

24.4 Proposed Future Activities

This section includes the proposed future activities that will be provided at a later deliverable. The EAP will be refined as the design progresses.

24.4.1 Preliminary Design

As part of the stipulated price proposal and permitting application requirements, certain sections of the EAP will be more refined than others during preliminary design. It is anticipated that these sections include the dam break analysis, additional detail added to sections 24.1 and 24.2, and the addition of a communications tree in the event of an emergency.

24.4.2 Final Design

As part of final design, the EAP will be refined based on District and permitting agency comments. It is anticipated that the final EAP will identify details of emergencies beyond flooding and will refine all sections from previous drafts.

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SIGNATURE AND SEAL

The Design Documentation Report for the LKBSTA project, SFWMD Contract No. 4600004527, has been prepared under the direction of the following design professional, licensed in the State of Florida.

Jeff Kivett
Brown and Caldwell
Florida Professional Engineer, License No. 59597

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Submitted To:

South Florida Water Management District
3301 Gun Club Road
West Palm Beach, FL, 33406
Contract: No. 4600004527 (PO 9500009421)

Submitted By:

EIP Florida Water Quality, LLC
5550 Newbury Street, Suite B
Baltimore, MD 21209

Contact:
Jeremy McBryan
jeremy@ecosystempartners.com

Ecosystem Investment Partners

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