APPENDIX A
ENGINEERING APPENDIX

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A.0 ENGINEERING DESIGN APPENDIX

The Engineering Appendix of the Post Authorization Change Report (PACR) provides a comprehensive record of the technical information and analyses prepared by the District to support the conceptual design of the Tentatively Selected Plan (TSP). The Engineering Appendix is organized by technical discipline and includes, but is not limited to, the following general information: an overview of the TSP features, status of engineering design activities and analyses, general construction procedures, preliminary civil site design information; geotechnical, structural, architectural, mechanical, I&C and electrical design information and analyses; as well as hydrogeologic, hydrologic and hydraulic design information and analyses. For the summary of costs, cost considerations and assumptions, refer to **Appendix B – Cost Engineering**.

The Engineering Appendix has been prepared with the goal of providing content similar to the technical content of Appendix A of the CEPP Final PIR (prepared by SFWMD and USACE, published December 2014) and the EAA Storage Reservoir A-1 Basis of Design Report (prepared by Black & Veatch, published January 2006).

A.1 TENTATIVELY SELECTED PLAN

As described in **Section 6.1.1** of the main report, the TSP (Alternative C240A) includes a 10,500-acre above-ground storage reservoir (i.e., the A-2 Reservoir) and a 6,500-acre Stormwater Treatment Area (STA) (i.e., the A-2 STA). The A-2 Reservoir is designed to have a normal full storage depth of approximately 22.6 feet of water. With this plan, water could be conveyed to the A-2 STA located on the west side of the A-2 Reservoir or to the existing STA 3/4. The A-2 STA would be located west of the A-2 Reservoir allowing for an outfall to the Miami Canal south of the existing G-373 divide structure. The shallow A-1 Flow Equalization Basin (FEB) with an existing 60,000 acre-feet (ac-ft) of storage would remain to the east of the proposed A-2 Reservoir. The TSP will allow for flexibility in storage between the A-1 FEB and the A-2 Reservoir, with a new water control structure to be constructed between the facilities.

The TSP includes an inflow-outflow canal for the A-2 Reservoir located along the northern boundary of the project area which extends from the NNR to the Miami Canal, and is referred to as the A-2 Reservoir Inflow-Outflow Canal. Proposed Pump Station P-1 and gated spillways SW-2 and SW-3 will be jointly operated to allow for Pump Station P-1 to pump water from the A-2 Reservoir Inflow-Outflow Canal into the A-2 Reservoir with the flexibility to control flows from the NNR and Miami Canal to the A-2 Reservoir Inflow-Outflow Canal. In addition, the TSP includes improvements to conveyance between Lake Okeechobee and the proposed A-2 Reservoir by adding 1,000 cubic feet per second (cfs) of additional conveyance capacity to the Miami Canal and 200 cfs of additional conveyance capacity to the North New River Canal. Details for the proposed conveyance capacity improvements to the Miami Canal and NNR Canal are provided in the report titled "Preliminary Conveyance Capacity Assessment for Lake Okeechobee Releases through the Miami & NNR Canals," included in **Annex A-1**. A summary of the TSP project features is provided in **Table A.1-1**.

Table A.1-1. Summary of TSP Features

Feature Name/ Structure No.	Feature Description	Design Capacity	Location	Purpose of Feature
A-2 Reservoir Inflow-Outflow Canal	Canal	3,000 cfs	North boundary of project area	Allows for inflow to and outflow from the A-2 Reservoir
A-2 Reservoir	Storage Reservoir	240,000 ac-ft of storage	West side of A-1 FEB	Provide storage of 240,000 ac-ft of water
A-2 STA	Stormwater Treat. Area	6,500 ac of effective area	West side of A-2 Reservoir	Provide treatment of water from A-2 Reservoir
B-1	Bridge	2 travel lanes	Intersection of L-23 Levee w/ A-2 Reservoir Inflow-Outflow Canal (near Miami Canal)	Allows traffic along L-23 Levee road to cross over A-2 Reservoir Inflow-Outflow Canal
B-2	Bridge	2 travel lanes	Intersection of U.S. Hwy. 27 w/ A-2 Reservoir Inflow-Outflow Canal (near NNR Canal)	Allows south bound traffic along U.S. Hwy. 27 to cross over A-2 Reservoir Inflow-Outflow Canal

Table A.1-1. Summary of TSP Features (continued)

Feature Name/ Structure No.	Feature Description	Design Capacity	Location	Purpose of Feature
B-3	Bridge	2 travel lanes	Intersection of U.S. Hwy. 27 w/ A-2 Reservoir Inflow-Outflow Canal (near NNR Canal)	Allows north bound traffic along U.S. Hwy. 27 to cross over A-2 Reservoir Inflow-Outflow Canal
C-1	Gated Culvert	2,500 cfs	North side of A-2 Reservoir, near east end of A-2 Reservoir	Allows for outflow from the A-2 Reservoir to the A-2 Reservoir Inflow-Outflow Canal
C-2	Ungated Culvert	650 cfs	Southwest corner of A-2 Reservoir	Allows for A-2 STA to discharge to Miami Canal south of G-373
C-3	Gated Culvert	325 cfs	West side A-2 Reservoir	Allows for inflow to Cell 3 of the A-2 STA from the A-2 Reservoir
C-4	Gated Culvert	325 cfs	West side A-2 Reservoir	Allows for inflow to Cell 4 of the A-2 STA from the A-2 Reservoir
C-5	Gated Culvert	325 cfs	Middle of A-2 STA	Allows for inflow to Cell 1 of the A-2 STA from Cell 3 of the A-2 STA
C-6	Gated Culvert	325 cfs	Middle of A-2 STA	Allows for inflow to Cell 2 of the A-2 STA from Cell 4 of the A-2 STA
C-7	Gated Culvert	325 cfs	West side of STA	Allows for inflow to the A-2 STA Discharge Canal from Cell 1 of the A-2 STA
C-8	Gated Culvert	325 cfs	West side of STA	Allows for inflow to the A-2 STA Discharge Canal from Cell 2 of the A-2 STA
C-9	Gated Culvert	4,500 cfs	South side of A-2 Reservoir	Allows for inflow from the STA 3/4 Inflow Canal to the A-2 Reservoir or outflow from the A-2 Reservoir to the STA 3/4 Inflow Canal depending on stages in the canal & reservoir
C-10	Gated Culvert	3,000 cfs	East side of A-2 Reservoir	Allows for inflow from the A-1 FEB to the A-2 Reservoir or outflow from the A-2 Reservoir to the A-1 FEB depending on stages in the A-1 FEB and A-2 Reservoir
C-11	Ungated Culvert	180 cfs	Northeast side of A-1 FEB	Allows for the hydraulic connection between the remnant of the northern reach of the A-1 FEB Seepage Canal & the eastern reach of the A-1 FEB Seepage Canal

Table A.1-1. Summary of TSP Features (continued)

Feature Name/ Structure No.	Feature Description	Design Capacity	Location	Purpose of Feature
P-1	Pump Station	4,600 cfs	North side of A-2 Reservoir, near east end of A-2 Reservoir	Allows for water to be pumped from the A-2 Reservoir Inflow-Outflow Canal to the A-2 Reservoir
SW-1	Ungated Spillway	Restrict overflow to allowable overflow rate	North side of A-2 Reservoir	Allows for water within the A-2 Reservoir above the NFSL to overflow into the A-2 Reservoir Inflow-Outflow Canal
SW-2	Gated Spillway	3,000 cfs	Near west end of A-2 Reservoir Inflow-Outflow canal, east of B-1	Allows for the flowrate from the Miami Canal to the P-1 intake to be controlled when P-1 is pumping
SW-3	Gated Spillway	3,000 cfs	Near east end of A-2 Reservoir Inflow-Outflow canal, west of B-2	Allows for the flowrate from the NNR Canal to the P-1 intake to be controlled when P-1 is pumping
SW-4	Gated Spillway	4,000 cfs	Within STA 3/4 Inflow Canal, near south side of G-720	Allows for the west reach of the STA 3/4 Inflow Canal to be hydraulically isolated from the east reach of the STA 3/4 Inflow Canal

A site plan of the TSP project features is provided in **Figure A.1-1**. This site plan along with the earthwork typical sections referenced on the site plan are included in **Annex C-1**.

During the planning, engineering, and design phase of the project, the location and design of each TSP feature will be refined and optimized, which may include adjustments to the size and layout of the A-2 Reservoir and A-2 STA, as well as the relocation, addition, removal, and/or combination of some water control structures and conveyance features. For instance, as part of the optimization of the design of the A-2 STA, the design and number of the canals, treatment cells, and/or gated culverts within the proposed footprint of the A-2 STA may be revised, which may include the addition of a gated culvert to allow for gravity inflow from the STA 3/4 Inflow Canal to the A-2 STA.

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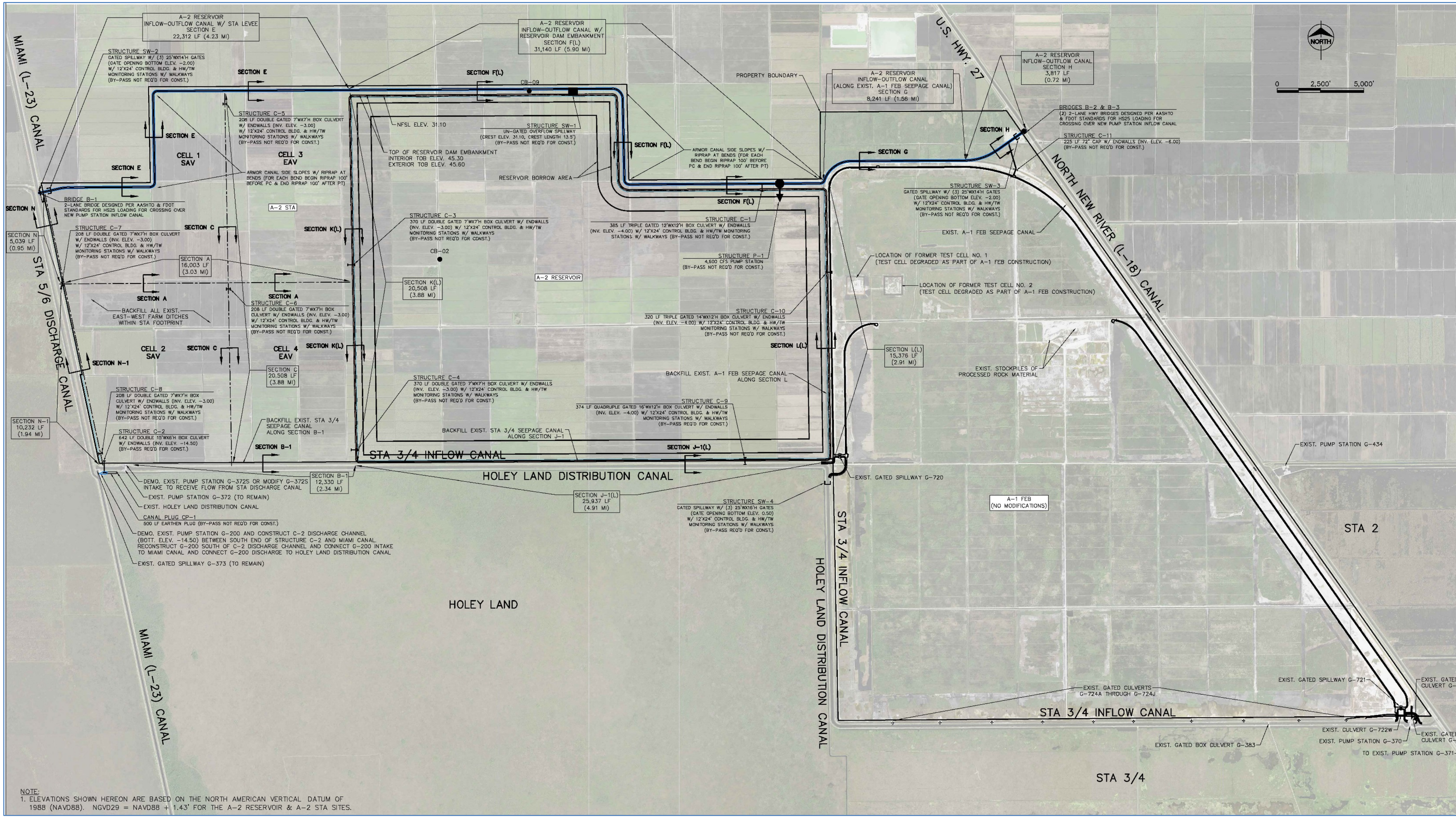

Figure A.1-1. Overall Site Plan of TSP

A.2 STATUS OF ENGINEERING DESIGN ACTIVITIES AND ANALYSES

A.2.1 LEVEL OF DESIGN EFFORTS

Design Engineering Regulation (ER) 1110-2-1150, Engineering and Design for Civil Works Projects, provides guidance for feasibility-level design to accompany decision documents. During the preparation of the CERP PACR, project risks were identified. The risks are presented in a project risk register, included in **Appendix B**. Risks to be addressed by the engineering design of the TSP from the risk register include:

- TL1 Life Cycle Cost Analysis on Pump Stations
- TL2 Internal Water Conveyance
- TL4 Porosity of Limerock is Unknown
- TL5 S-8 Flood Control Operations
- TL6 S-8 New Pump Station Design
- TL16 Sizing of New Pump
- TL20 Global Geotechnical Assumptions
- TL21 Disposal of Excess On-Site Material
- TL22 Levee Stabilization Approach
- TL23 System Not Performing As Intended
- TL24 Conveyance Improvements

These risks will be further evaluated and addressed during the preconstruction engineering and design phase (PED) of the project.

A.2.2 RECOMMENDATION FOR DESIGN COMPLETION

Features of the TSP have been designed based on available data, historic information, and preliminary engineering analyses and calculations. The design of these features (or project components) will be optimized during the PED phase for cost efficiency and performance, incorporating updated data and information as it becomes available. During PED, an economic analysis will be conducted on the pump station components to be in compliance with EM 1110-2-3102.

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A.3 GENERAL CONSTRUCTION PROCEDURES DISCUSSION

A.3.1 GENERAL CONSTRUCTION RECOMMENDATIONS

It is envisioned that the TSP 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.

A.3.2 CONSTRUCTION CONTRACTS AND SCHEDULE

During PED, decisions will be finalized regarding the specific breakdown and scheduling of construction contracts for the TSP. It is anticipated that the TSP will constructed under the following six major construction contracts.

1. Miami Canal Conveyance Improvements Construction
2. NNR Canal Conveyance Improvements Construction
3. A-2 Reservoir and A-2 STA Embankments, Canals and Control Structures (Structures C-1 through C-11 and SW-1) Construction
4. Gated Spillways Construction (Structures SW-2, SW-3, and SW-4)
5. U.S. 27 Bridges and L-23 Bridge Construction (Structures B-1, B-2, and B-3)
6. A-2 Reservoir Pump Station Construction (Structure P-1)

A preliminary construction schedule for each of these major construction contracts is included in **Appendix B**.

A.3.3 CONSTRUCTION SEQUENCING AND STAGING

A.3.3.1 General

The TSP will involve several contractors working simultaneously to complete the work within the desired schedule. The specific sequencing of the components for each construction contract will be developed by the construction contractor using constraints that will be specified in the construction documents. The major constraints during construction of the TSP are described in the following paragraphs.

A.3.3.2 Access

The TSP is located in an agricultural area and access to the A-2 Reservoir, A-2 STA, and A-2 Reservoir Inflow-Outflow Canal sites will be from U.S. Highway (Hwy.) 27 to the east, the L-23 Levee road to the west, the STA 3/4 Inflow Canal Levee road to the south, and existing unpaved farm roads located within the interior of the A-2 Reservoir and A-2 STA and located along the north boundary of the A-2 Reservoir and A-2 STA. U.S. Hwy. 27 is a major traffic route for transportation from the Fort Lauderdale area to the central Florida area, and is also a hurricane evacuation route. It is anticipated that this will be the primary access road to be used by the construction contractors during construction. It will be the responsibility of the contractors to coordinate with the Florida Department of Transportation (FDOT) regarding the maintenance of traffic along U.S. Hwy. 27 during construction. After the TSP is constructed, U.S. Hwy. 27

will provide the main access to the A-2 Reservoir and A-2 STA. Access to the Miami and NNR canals during construction will be along the existing right-of-way of these canals.

A.3.3.3 STA, Reservoir and Canal Embankments

Most of the materials to be used in the construction of the embankments for the A-2 STA, A-2 Reservoir, and A-2 Reservoir Inflow-Outflow Canal will be excavated from the canals and borrow areas that are part of the TSP. During the construction of embankments for two Test Cells in 2005 and the A-1 FEB (formerly called the A-1 Reservoir) seepage canal in 2007, it was determined that materials excavated below the Fort Thompson caprock are difficult to adequately dewater for direct placement in the embankment. Therefore, it will be necessary to excavate and stockpile these materials for two to three months in advance of embankment construction. Pre-excavation and embankment construction could be performed under separate contracts or under one contract with appropriate sequencing time between excavation and embankment fill placement. The project includes about 17.6 miles of dam embankment for the A-2 Reservoir, 6.9 miles of divider levees for the A-2 STA, and about 24.8 miles of perimeter levees for the A-2 Reservoir Inflow-Outflow Canal; thus, even if the excavation and placement are included in a single contract, there will be ample opportunity for a single contractor to excavate embankment materials well in advance of the placement.

For purposes of this report, it will be assumed that the excavation and placement is completed under a single contract. Caprock will be blasted in the canals and could be placed directly in the rockfill section of the embankment except for the rock to be used for rock processing and slope protection. Rock to be processed for construction of the roller compacted concrete (RCC) revetment for the A-2 Reservoir, can be excavated from the reservoir interior borrow area, sorted as necessary, and stockpiled for later use. The remaining material from the borrow areas will be used as random fill for construction of the embankments. This material will be stockpiled and allowed to drain for an extended period before being placed as embankment fill.

The A-2 Reservoir embankment will abut the levee along the north side of the STA 3/4 Inflow Canal and will extend across the existing STA 3/4 seepage canal. In order for the seepage canal to remain in service for as long as possible during construction, the A-2 Reservoir embankment construction along the STA 3/4 Inflow Canal will begin at the southeast corner of the A-2 Reservoir and proceed west, thereby allowing the seepage canal to remain in use for as long as possible. Likewise, the backfilling of the seepage canal within the footprint of the A-2 STA will begin at the southeast corner of the A-2 STA and proceed west. The backfilling of the seepage canal within the A-2 STA will take place after the southern embankment for the A-2 Reservoir is constructed. Maintaining the seepage canal in use during construction will assist in dewatering the portions of the seepage canal that are to be backfilled as part of the project.

Blasting will be necessary for the canal and borrow area excavation. Several on-site rock processing stations will be developed for producing filter materials for the reservoir embankment, as well ensuring that all rock processed for random fill material meets the gradation requirements for the reservoir, STA, and canal embankments. The locations for the rock processing stations will be determined during the PED phase when geotechnical investigations have been completed and areas with suitable caprock thickness and quality have been identified.

A.3.3.4 Pump Station

The A-2 Reservoir pump station (P-1) will be located along the north side of the A-2 Reservoir. It is expected that the pump station will be constructed under a separate contract from the A-2 Reservoir embankment. Coordination between the two contracts will be necessary for the portion of the embankment where the pump station will be constructed.

It will be critical to maintain continuous operations of STA 3/4 during construction. The existing G-370 and G-372 pump stations (without modification) will be used for partial filling of the A-2 Reservoir, via the proposed gated culvert C-9 and gated spillway SW-4. Disruption to STA 3/4 operations would be limited to construction of these gate structures in the STA 3/4 Inflow Canal. If the existing pump stations G-370 and G-372 are modified to pump to the NFSL of the A-2 Reservoir, which is 31.1 feet-NAVD (32.53 feet-National Geodetic Vertical Datum), modifications to these pump stations within the STA 3/4 Inflow Canal will be required. For that scenario, work will be sequenced such that only one pump bay is out of service at any time. Short periods of time will be scheduled for taking the STA 3/4 Inflow Canal out of service on one side or the other of gate structure G-383 for work on the pump stations, or for installation of additional gate structures within the STA 3/4 Inflow Canal.

A.3.3.5 Control Structures

Gated box culvert C-9 and open box culvert C-2 will require cofferdam construction to allow the structures to be constructed without taking the STA 3/4 Inflow Canal out of service. Temporary access around new structure areas will be required for SFWMD maintenance operations during construction of each structure.

A.3.3.6 Agricultural Operations

Several agricultural canals traverse the A-2 Reservoir and A-2 STA site and supply water to farming operations within and north of the A-2 Reservoir and A-2 STA footprint. It is currently anticipated that some of these canals will need to remain in service during certain periods of time during the construction of the TSP. As such, the proposed embankments that will cross these canals will be constructed near the end of the construction period and will be coordinated to minimize disruption of agricultural deliveries during the growing season. Once the canals have been dammed by the embankments, the contractor may be required to maintain temporary irrigation and drainage pumping for the remainder of the growing/harvest season for that year’s crop. At that time the contractor will also demolish the existing agricultural pump stations within the A-2 Reservoir and A-2 STA footprint and complete the embankment through those areas.

A.3.3.7 Staging

There is ample space for multiple staging areas to be constructed along the west side U.S. Hwy. 27 on the north side of the existing A-1 FEB. The number of staging areas will depend on the number of construction contracts for the TSP. Locations and size of these staging areas will be established during the PED and construction phases. Contractors may establish minor staging areas around the perimeter of the A-2 Reservoir and A-2 STA embankments to accommodate construction. Secondary staging locations will be established at the quarry processing stations.

A.3.4 DEMOLITION AND DISPOSAL

The agricultural buildings and pump stations structures within the A-2 Reservoir, A-2 STA, and A-2 Reservoir Inflow-Outflow Canal sites will be demolished by the contractor(s) for the embankment construction and the materials will be disposed of by the contractor(s). The SFWMD may determine that certain mechanical equipment should be delivered to a location of their choice, as set out in the contract documents.

Agricultural structures such as canal water control features and culverts, will have no negative impact on the completed A-2 Reservoir, and will therefore remain. These types of structures, however, could have an adverse impact on the performance of the completed A-2 STA and therefore may need to be removed from the interior of the A-2 STA. All farm ditches and canals within the A-2 STA footprint that have an east-west alignment will be backfilled so that there is no short circuiting of flow through the A-2 STA when it becomes operational.

A.3.5 OTHER PROJECTS AFFECTING CONSTRUCTION

Currently, there are no projects known that are planned to be constructed in the immediate vicinity of the TSP features that would impact their construction or operation.

A.4 GENERAL DESIGN REQUIREMENTS AND CRITERIA

A.4.1 PROJECT LIMITS AND SITE DATUM

The EAA Storage Reservoir Project limits are bounded by the Everglades Agricultural Area (EAA) FEB A-1 on the east, the Holey Land Wildlife Management Area (WMA) on the south, farmland in the EAA on the north, and the Miami Canal on the west.

The horizontal datum used in Appendix A of this report is the North American Datum of 1983 (NAD83). Unless noted otherwise, the vertical datum used in Appendix A of this report is the North American Vertical Datum of 1988 (NAVD or NAVD88). Some other reports and design documents referenced in this report or related to this project use the National Geodetic Vertical Datum of 1929 (NGVD or NGVD29) as a vertical datum. The relationship between these datums is NGVD29 = NAVD88 + 1.43 feet, for the geographic location of the proposed A-2 Reservoir and A-2 STA.

A.4.2 SERVICE LIFE

According to U.S. Army Corps of Engineers (USACE) Engineering Manuals (EM) 1110-2-3104, EM 1110-2-3102, and Major Pumping Station Engineering Guidelines, the design life for the new northeast pump station will be 50 years. With proper maintenance, this design life can be achieved by following the guidance in these documents.

The mechanical equipment will require rehabilitation or replacement over the design life. The engines and pumps will operate intermittently but will require regular maintenance. The engines may require at least one major overhaul during the design life while the pump materials will be designed to provide long service life. The architectural and structural design of the pump stations 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 described in more detail in **Sections A.10** through **A.16**.

A.4.3 UNITS

The units and system of measurement will be in the English system of measurement.

A.4.4 CODES AND STANDARDS

A.4.4.1 General

- CERP Guidance Memoranda
- Joint SFWMD, USACE, and Florida Department of Environmental Protection (FDEP) Design Criteria Memoranda (DCM)
- SFWMD Standard Design Guidelines

A.4.4.2 Site Work Design Criteria

Codes and standards: design and specification of all work shall be in accordance with latest laws and regulations of the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- American Association of State Highway and Transportation Officials (AASHTO)
- American National Standards Institute, Inc. (ANSI)
- American Society for Testing and Materials (ASTM)
- Americans with Disabilities Act Accessibility Guidelines for Buildings and Facilities (ADAAG)
- Asphalt Institute (AI)
- Federal Highway Administration (FHWA)
- FDOT
- Manual on Uniform Traffic Control Devices (MUTCD)
- SFWMD
- Uniform Federal Accessibility Standards (UFAS)
- USACE

A.4.4.3 Geotechnical Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein. Recommended and recognized standards from other organizations shall be used where required and approved to serve as guidelines for the design, fabrication, and construction when not in conflict with the standards referenced herein.

- ASTM
- Design Manual for Roller Compacted Concrete (RCC) Spillways and Overtopping Protection, Portland Cement Association, 2002
- EM 1110-2-2300, General Design and Construction Considerations for Earth and Rock-Fill Dams
  - EM 1110-2-1901, Seepage Analysis and Control For Dams
  - EM 1110-2-1902, Slope Stability
  - EM 1110-2-2006, Engineering Design – Roller Compacted Concrete
- Florida Building Code, 2017 Edition
- FDOT
- SFWMD
- USACE

A.4.4.4 Architectural Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- Florida Accessibility Code - Latest Edition
- Florida Building Code - 2017 Edition
- Occupational Safety and Health Administration - 29 CFR

A.4.4.5 Structural Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- Aluminum Design Manual "Specifications for Aluminum Structures," 2000
- American Concrete Institute (ACI)
  - ACI 318-02 "Building Code Requirements for Reinforced Concrete"
  - ACI 350-01/350R-01 "Code Requirements for Environmental Engineering Concrete Structures and Commentary"
  - ACI 350.4R-04 “Design Considerations for Environmental Engineering Concrete Structures”
  - ACI 530 “Building Code Requirements for Masonry Structures”
  - ACI 530.1 “Specification for Masonry Structures”
- American Institute of Steel Construction, Inc. (AISC): Manual of Steel Construction, Allowable Stress Design, 9th Edition
- American Society of Civil Engineers (ASCE) 7-02: Minimum Design Loads for Buildings and Structures
- American Welding Society (AWS)
  - American Welding Society, Structural Welding Code – Steel
  - American Welding Society, Structural Welding Code – Stainless Steel
  - American Welding Society, Structural Welding Code – Aluminum
- CERP Standard Design Manual, June 6, 2003
- Concrete Reinforcing Steel Institute Handbook
- Florida Building Code, 2017 Edition
- PCI Design Handbook, Precast and Prestressed Concrete
- SFWMD, Major Pumping Station Engineering Guidelines, May 9, 2008
- USACE

- EM 1110-1-2009 Architectural Concrete
- EM 1110-2-2000 Standard Practice for Concrete for Civil Works Structures, dated 1 February 1994
- EM 1110-2-2102 Waterstops and Other Preformed Joint Materials for Civil Works Structures, dated 30 September 1995
- EM 1110-2-2104 Strength Design for Reinforced Concrete Hydraulic Structures, dated 30 June 1992
- EM 1110-2-2105 Design of Hydraulic Steel Structures, dated 31 March 1993
- EM 1110-2-2502 Retaining and Flood Walls, dated 29 September 1989
- EM 1110-2-2701 Vertical Lift Gates, dated 30 November 1997
- EM 1110-2-3104 Structural and Architectural Design of Pumping Stations, dated 30 June 1989

A.4.4.6 Special Mechanical Equipment Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- AASHTO
- American Bearing Manufacturers Association (ABMA)
- American Gear Manufacturers Association (AGMA)
- American Petroleum Institute (API)
  - API Standard 620 - Design and Construction of Large Low Pressure Storage Tanks
  - API Standard 650 - Welded Steel Tanks for Oil Storage
- ASME/ANSI
  - ANSI/ASME B1.20.1 - General Purpose Pipe Threads
  - ANSI/ASME B16.1 - Cast Iron Pipe Flanges and Flanged Fittings, Class 25, 125, 250 and 800
  - ANSI/ASME B16.5 - Steel Pipe Flanges and Flanged Fittings
  - ANSI/ASME B16.11 - Forged Fittings, Socket-Welding and Threaded
  - ANSI/ASME B16.21 - Nonmetallic Flat Gaskets for Pipe Flanges
  - ANSI/ASME B16.25 - Butt-welding Ends
  - ANSI/ASME B31.10 - Pressure Piping
- ASTM
  - ASTM A36 - Structural Steel
  - ASTM A53 - Pipe, Steel, Black and Hot-Dipped, Zinc-Coated Welded and Seamless
  - ASTM A105 - Forgings, Carbon Steel for Piping Components

  - ASTM A139 - Electric Fusion Welded Steel Pipe
  - ASTM A139B - Specification for Electric-Fusion (Arc)-Welded Steel Pipe
  - ASTM A181 - Forgings, Carbon Steel for General Purpose Piping
  - ASTM A283 - Carbon Steel Plate, Shapes, or Bars
  - ASTM A307 - Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile
  - ASTM A312 - Specification for Seamless and Welded Austenitic Stainless Steel Pipe
  - ASTM A563 - Specifications for Carbon and Alloy Steel Nuts
  - ASTM A568 - Steel, Sheet, Carbon, and High Strength, Low Alloy Hot Rolled and Cold Rolled
  - ASTM A570 - Hot Rolled Carbon Steel Sheet
  - ASTM F593 – Stainless Steel Bolts, Hex Nuts, Screws, and Studs, 2000
- American Water Works Association (AWWA)
  - AWWA C200 - Steel Water Pipe 6 Inches and Larger
  - AWWA C207 - Steel Pipe Flanges for Waterworks Service, Sizes 4 Inch through 144 Inch
  - AWWA C208 - Dimensions for Fabricated Steel Water Pipe Fittings
  - AWWA M11 - Steel Water Pipe - A Guide for Design and Installation
  - AWWA C600 - Installation of Ductile-Iron Water Mains and their Appurtenances
- ANSI/ASME B36.10 - Welded and Seamless Wrought Steel Pipe
- CERP Standard Design Manual, 2003, USACE Jacksonville District and SFWMD
- EPA Regulation 40 CFR Part 280.41
- Heat Exchange Institute (HEI)
- Hydraulics Institute Standards (HI)
  - ANSI/HI Standard 9.8-1998 - Pump Intake Design
  - ANSI/HI Standard 2.1-2.6-2000 - Standards for Vertical Pumps
  - ANSI/HI Standard 9.6.1-1998 – NPSH Margin
- Manufacturers Standardization Society of Valve and Fitting Industry (MSS)
  - MSS-SP 58 (1993) Pipe Hangers and Supports + Materials, Design, and Manufacture
  - MSS-SP 69 (1996) Pipe Hangers and Supports + Selection and Application
- National Fire Protection Association (NFPA)
  - NFPA 30 - Flammable and Combustible Liquids Code
  - NFPA 30A - Automotive and Marine Station Code
  - NFPA 37 - Stationary Combustion Engines and Gas Turbines
  - NFPA 329 - System Test
- Pipe Fabrication Institute (PFI):
  - PFI-ES5 - Cleaning of Fabricated Pipe

- Steel Structures Painting Council (SSPC)
  - SSPC SP1 - Solvent Cleaning
  - SSPC SP3 - Power Tool Cleaning
  - SSPC SP5 - White Metal Blast Cleaning
  - SSPC-SP6 - Commercial Blast Cleaning
  - SSPC SP7 - Brush Off Blast Cleaning
- SFWMD, Major Pumping Station Engineering Guidelines, May 9, 2008
- Underwriters Laboratories Inc. (UL)
  - UL-142 - Steel Aboveground Tanks for Flammable and Combustible Liquids
- USACE
  - EM 1110-2-3102, General Principles of Pumping Station Design and Layout, 1995
  - EM 1110-2-3104, Structural and Architectural Design of Pumping Stations, 1989
  - EM 1110-2-3105, Mechanical and Electrical Design of Pumping Stations, 1999

A.4.4.7 HVAC, Plumbing and Fire Suppression

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. In addition to the applicable codes and standards previously identified, the system designs will also be based on but not limited to the following publications and standards:

- American Society of Heating, Refrigeration, and Air Conditioning Engineers (ASHRAE) Handbooks and Standards
- American Society of Plumbing Engineers (ASPE) Handbooks
- Florida Building Code 2017 – Mechanical
- Florida Building Code 2017 – Plumbing
- Florida Fire Protection Code
- SFWMD, Major Pumping Station Engineering Guidelines, May 9, 2008
- National Fire Protection Association Recommended Practices (NFPA) and Manuals
- Occupational Safety and Health Act (OSHA) Standards Manual
- Sheet Metal and Air Conditioning Contractor National Association (SMACNA) Handbooks

A.4.4.8 Fire Protection and Detection Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- International Building Code (International Code Council) - 2003

- International Fire Code (ICC) - 2003
- NFPA
- OSHA
- UL

A.4.4.9 Electrical Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- ANSI
  - ANSI C2, National Electrical Safety Code
  - ANSI C84.1, Electric Power Systems and Equipment - Voltage Ratings
  - ANSI A117.1, Buildings and Facilities - Providing Accessibility and Usability for Physically Handicapped People
  - ANSI/IEEE Std. 242, Recommended Practice for Protection and Coordination of Industrial and Commercial Power Systems (The Buff book)
- Institute of Electrical and Electronics Engineers (IEEE) C62.41 Surge Voltage in Low Voltage AC Power Circuits
- Illuminating Engineering Society (IES) Lighting Handbook, Reference Volume and Application Volume
- NFPA
  - NFPA 70, National Electrical Code
  - NFPA 72, National Fire Alarm Code
  - NFPA 101, Code for Safety to Life from Fire in Buildings and Structures
  - NFPA 78, Lightning Protection Code
- Uniform Federal Accessibility Standards (UFAS)
- UL 268, Smoke Detectors for Fire Protective Signaling Systems
- USACE Technical Standards, TI-800-01

A.4.4.10 Instrumentation and Controls Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- ANSI
  - ANSI C37.90 (1989) Relays and Relay Systems Associated with Electric Power Apparatus

  - ANSI C37.90.1 (1989) Surge Withstand Capability (SWC) Test for Protective Relays and Relay Systems
  - EM ANSI/EIA/TIA -232-F (2002) Interface Between Data Terminal Equipment and Data Circuit-Terminating Equipment Employing Serial Binary Data Interchange
- IEEE
  - IEEE C62.41 (1991) Recommended Practice for Surge Voltages in Low- Voltage AC Power Circuits
  - IEEE Std 100 (2000) IEEE Standard Dictionary of Electrical and Electronics Terms
  - IEEE Std 802 (1990; R 1995) Information Processing Systems, Local Area Networks: Part 4: Token Passing Bus Access Method and Physical Layer Specifications
- International Electrotechnical Commission (IEC) 61131-3 (2003) Programmable Controllers — Part 3: Programming Languages
- National Electrical Manufacturer's Association (NEMA)
  - NEMA 250 (1997) Enclosures for Electrical Equipment (1,000 Volts Maximum)
  - NEMA ICS 1 (2000) Industrial Control and Systems: General Requirements
  - NEMA ICS 2 (2000) Industrial Control and Systems: Controllers, Contactors, and Overload Relays Rated 600 volts
  - NEMA ICS 4 (2000) Industrial Control and Systems: Terminal Blocks
  - NEMA ICS 6 (1993; R 2001) Industrial Control and Systems: Enclosures
- NFPA 70 (2002) National Electrical Code
- UL
  - UL 1059 (2001) Terminal Blocks
  - UL 508 (1999; Rev thru Dec 2002) Control Equipment

A.4.4.11 Telemetry System Design Criteria

Codes and Standards: Design and specification of all work shall be in accordance with latest laws and regulations of the State of Florida and the Federal government, with applicable local codes and ordinances, and with codes and industry standards referenced herein. Following is a summary of organizations with codes and standards referenced herein.

- Electronics Industries Alliance (EIA)
  - EIA ANSI/TIA/EIA-222-F (1996) Structural Standards for Steel Antenna Towers and Antenna Supporting Structures
  - EIA ANSI/EIA/TIA-232-F (2002) Interface Between Data Terminal Equipment and Data Circuit Terminating Equipment Employing Serial Binary Data Interchange
  - EIA ANSI/EIA-310-D (1992) Racks, Panels, and Associated Equipment
- Federal Communications Commission (FCC) 47 CFR 15 Radio Frequency Devices
- SFWMD Design Standards and Guidelines

A.4.4.12 Design Criteria Memoranda

Following is a summary of the Design Criteria Memoranda and their respective effective dates.

DCM-1	Hazard Potential Classification	September 12, 2005
DCM-2	Wind and Precipitation Design Criteria for Freeboard	February 6, 2006
DCM-3	Spillway Capacity and Reservoir Drawdown Criteria	February 3, 2006
DCM-4	Minimum Dimensions of Dams and Embankments	May 9, 2008
DCM-5	Major Pump Station Engineering Guidelines	May 12, 2008
DCM-6	Geotechnical Seismic Evaluation of CERP Dam Foundations	May 16, 2005
DCM-7	Procedure for Development of Engineering Construction Costs	June 18, 2008
DCM-8	Vulnerability Protection Requirements	*N/A (Never Issued)*
DCM-9	Dam Safety Instrumentation and Monitoring	June 15, 2007
DCM-10	Construction Quality Assurance Procedures	*N/A (Never Issued)*
DCM-11	Dam Safety Program	June 18, 2007
DCM-12	Value Engineering	*N/A (Never Issued)*

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A.5 HYDROLOGY

A.5.1 HAZARD CLASSIFICATION AND EMERGENCY EVACUATION REQUIREMENTS

The A-2 Reservoir is classified as a high hazard impoundment (major impoundment), as specified in the Federal Emergency Management Agency's *Selecting and Accommodating Inflow Design Floods for Dams* (FEMA 2013) and *Design Criteria Memorandum-1: Hazard Potential Classification* (DCM-1) (Haapala et al. 2005) guidelines. U.S. Hwy. 27 and the farmland to north of the TSP site will likely be significantly impacted in the event of a breach, which may lead to life-threatening conditions for motorists along U.S. Hwy. 27 and nearby farm personnel as well as impede emergency evacuation routes for southern Florida. Since the A-2 Reservoir would be a high hazard impoundment, a dam-break analysis for the reservoir and surrounding topography will need to be completed during the PED phase of the TSP, in accordance with DCM-1. See **Section A.18** for a discussion regarding the Emergency Action Plan to be developed for the A-2 Reservoir.

A.5.2 DESIGN STORMS AND FLOODS

Four wind and precipitation design cases to be used for freeboard determination of CERP impoundments were developed and issued in DCM-2 (Haapala et al. 2006). These design conditions, which were used for the wave and overtopping analysis of the A-2 Reservoir, are described below.

A.5.2.1 Design Case 1: 100-Year Wind with Probable Maximum Precipitation

Design Case 1 represents the Probable Maximum Precipitation (PMP) event in combination with a 100-year wind.

A 72-hour PMP of 54 inches was adopted for the analysis, based on hydrologic modelling undertaken for the A-1 Reservoir (Burgi et al. 2005). This reservoir is located adjacent to the A-2 Reservoir and covers a similar area. Hence, the PMP estimates for the A-1 reservoir are considered to be appropriate for the A-2 Reservoir. The adopted PMP value aligns with estimates in other EAA reports such as the Levee High Report (CERP 2004) and DCM-2 (Haapala et al. 2006).

The procedure described in DCM-2 (Haapala et al. 2006) was followed to provide an estimate of the 100-year average recurrence interval (ARI) wind speed magnitude for the A-2 Reservoir. As specified in DCM-2, the 50-year three-second wind gust for the A-2 Reservoir site is 125 miles per hour (mph). This was converted to a 100-year, one-hour overwater wind speed of approximately 106 mph. After adjustments for duration and overwater conditions, the adopted sustained wind speed magnitude for assessment of the design wave conditions and water levels of this design case was 104.5 mph.

A.5.2.2 Design Case 2: Category Five Hurricane with 100-Year storm

Design Case 2 represents a 100-year precipitation event in combination with a Category 5 wind speed as defined by the Saffir-Simpson Hurricane Scale.

A 100-year precipitation event of 17 inches adopted for this design case, based on Figure DCM 2-3 (Haapala et al. 2006).

As recommended in DCM-2, a one-minute overwater wind speed of 156 mph was used to represent a Category 5 hurricane. After adjustments for duration to achieve fully developed wave conditions over the reservoir fetch length, the adopted wind speed for assessment of the design wave conditions and water levels was 123.3 mph.

A.5.2.3 Design Case 3: Probable Maximum Wind (200 mph)

Design Case 3 represents the Probable Maximum Wind (PMW) speed in combination with the reservoir level at the normal full storage depth (i.e. 22.6 ft for the A-2 Reservoir). As recommended in DCM-2, this particular design case was used for sensitivity testing only and not as a selected design condition (Haapala et al., 2006):

> *[The probable maximum wind…] is to be used for "sensitivity identification" and not as a design condition. Wave models are unlikely capable of yielding results within a degree of confidence for design for these extreme wind speeds, especially over relatively shallow water bodies. Even for 125-mph wind, model capabilities are most likely being "stretched" for project conditions.*

As defined in DCM-2, a one-minute averaged overwater wind speed of 200 mph was used to represent the PMW. The one-minute average wind speed was converted to an hourly averaged wind speed of 161 mph. After adjustments for duration, the adopted wind speed for assessment of the design wave conditions and water levels was 159.7 mph.

A.5.2.4 Design Case 4: Storm Specific Wind and Precipitation

Design Case 4 represents a storm-specific case of precipitation and wind conditions recorded during Hurricane Easy, which occurred in Florida in 1950.

Precipitation depths for both the 24-hour and 72-hour rainfall durations are considered in this analysis, corresponding to 38.7 inches and 45.2 inches respectively (Haapala et al. 2006).

A maximum wind speed of 125 mph (3-second gust) was recorded during Hurricane Easy (Haapala et al. 2006). After adjustments to meet DCM-2 requirements (i.e., overwater conditions, wind duration for wave development etc.) a design wind speed of 97.5 mph was used for the analysis.

A.5.2.5 Summary

Table A.5.2-1 summarizes the wind and precipitation design conditions that were used to determine the appropriate embankment height for the A-2 Reservoir.

Table A.5.2-1. Wind and Precipitation Design Conditions

Design Case	Description	Wind (mph)	Precipitation (inches)	Average Water Depth[1] (ft)
1	100 yr ARI wind + PMP	104.5	54	27.1
2	Cat 5 Hurricane + 100yr ARI Precipitation	123.3	17	24.0
3	Probable Max Wind Speed (Sensitivity Testing Only)	159.7	0	22.6
4.1	Storm Specific Wind & 24hr Precipitation (Hurricane Easy)	97.5	38.7	25.8
4.2	Storm Specific Wind & 72hr Precipitation (Hurricane Easy)	97.5	45.2	26.4

[1] Average water depth = NFSL (22.6 ft) + Precipitation

A.5.3 A-2 RESERVOIR INFLOWS AND OUTFLOWS

The A-2 Reservoir has a normal full storage level of 31.10 feet-NAVD (32.53 feet-NGVD), which corresponds to an average normal full storage depth of 22.6 feet, since the average bottom elevation of the reservoir is 8.50 feet-NAVD (9.93 feet-NGVD).

A.5.3.1 Inflow Design Storm

The inflow design storm (IDF) for the A-2 Reservoir will be the probable maximum flood (PMF) as designated by DCM-2. Because the reservoir functions as an off-line reservoir and has no contributing watershed except for its surface area, the PMF is the PMP precipitation depth of 54 inches (4.5 feet) distributed appropriately in time. To determine the total inflow, the PMP precipitation depth is multiplied by the area of the reservoir site (inflow pumps are assumed not to be operating at the time) that drains to the interior of the reservoir, which is 10,823.4 acres. The total inflow to the A-2 Reservoir from the 72-hour PMP precipitation was calculated as 48,705.3 ac-ft. Starting with a normal full storage level of 31.10 feet-NAVD (32.53 feet-NGVD) and then adding the inflow from the PMP event, the resulting storage level in the reservoir due to the PMP event would be approximately 35.60 feet-NAVD (37.03 feet-NGVD), with zero discharge from the reservoir during the PMP event.

A.5.3.2 Routing of Flood Flows

Because the A-2 Reservoir has a perimeter dam embankment and has no contributing watershed except for the surface area of the A-2 Reservoir, there are no direct gravity inflows. The A-2 Reservoir will have several gate structures capable of routing significant flood flows. See Section A.6 for discussion regarding the reservoir gate structures. During storm events, the reservoir will be capable of passing flow to the downstream areas which include the A-2 Reservoir Inflow-Outflow Canal (connects to the NNR Canal and Miami Canal), the STA-3/4 Inflow Canal, the A-1 FEB and the A-2 STA. However, as defined in DCM-2, during the PMP event which is used, in part, to determine the maximum freeboard requirements, no reservoir outflow is assumed to take place while the PMP is occurring. It is anticipated that the gates will be inoperable during the PMP making the gate routing irrelevant. In other words, the A-2 Reservoir must be designed to be capable of containing the full PMP/PMF storm because reservoir routing is assumed to not be applicable. In addition to the gate structures, the A-2 Reservoir will include an uncontrolled (un-gated) spillway (SW-1) with a crest elevation of 31.10 feet-NAVD (32.53 feet-NGVD), which will ensure that the level in the reservoir will return to its normal full storage level after the PMP event. This uncontrolled spillway will also prevent the reservoir from being accidentally overfilled by pumping operations. The uncontrolled spillway has been sized so that its outflow rate from the reservoir during the PMP event will not exceed the allowable discharge rate for the basins that the reservoir is located in (Miami Canal and NNR River Canal basins) (per SFWMD regulations), which is ¾ inch of runoff over the contributing area in 24 hours. See **Section A.6** for discussion regarding the reservoir uncontrolled spillway structure (SW-1).

A.5.3.3 Reservoir Discharges

Discharges from the A-2 Reservoir will be based on expected environmental deliveries for the A-2 Reservoir to the A-2 STA, A-1 FEB, STA 3/4, STA 2, and agricultural deliveries for the Miami Canal and NNR/Hillsboro Canal basin. Discharge structures include gates that will be sized according to the flows established based on releases from the A-2 Reservoir per the SFWMD C240 simulation in conjunction with communication with Water Managers. **Section A.6** provides details of flows considered for sizing all water control structures. Gate discharges will follow orifice flow principles and gate openings will be a function of meeting the required releases from the A-2 Reservoir. See **Section A.6** for discussion regarding the reservoir gate structures.

A.5.4 WAVE AND OVERTOPPING ANALYSIS

Wave overtopping is an important parameter in determining appropriate freeboard levels for reservoirs. The volume of water that may flow over the crest of the structure during storm events is dependent on hydrodynamic parameters (wave height and period, angle of wave attack and water depth), as well as the characteristics of the embankment (e.g. crest height, roughness and slope). Therefore, as part of the feasibility study for the A-2 Reservoir, a wave and overtopping analysis was undertaken to:

a) predict the design wave conditions that could be generated across the reservoir during extreme design wind and precipitation events; and

b) predict the wave overtopping rate of a number of embankment configurations during key design storm events in order to estimate the minimum embankment level to limit overtopping rates to acceptable levels.

The following section describes the outcomes of this analysis. Full details of the wave and overtopping assessment are provided in **Annex A-2**.

A.5.4.1 Embankment Characteristics

Figure A.5.4-1 illustrates the cross sectional design of the A-2 Reservoir embankment which was used for the overtopping analysis. A 1:3 embankment slope is proposed for the reservoir, with roller compacted concrete (RCC) material to be adopted on the surface of the water side of the embankment and grass on the landward side. A precautionary approach was adopted for the analysis, representing the water side RCC slope as a smooth and impermeable structure.

A wave wall is proposed on the landward side of the embankment crest. The overtopping analysis was used to determine the height of this wave wall required to reduce overtopping to appropriate levels. The Normal Full Storage Level (NFSL) of the A-2 Reservoir is at an elevation of 31.10 feet-NAVD (32.53 feet-NGVD).

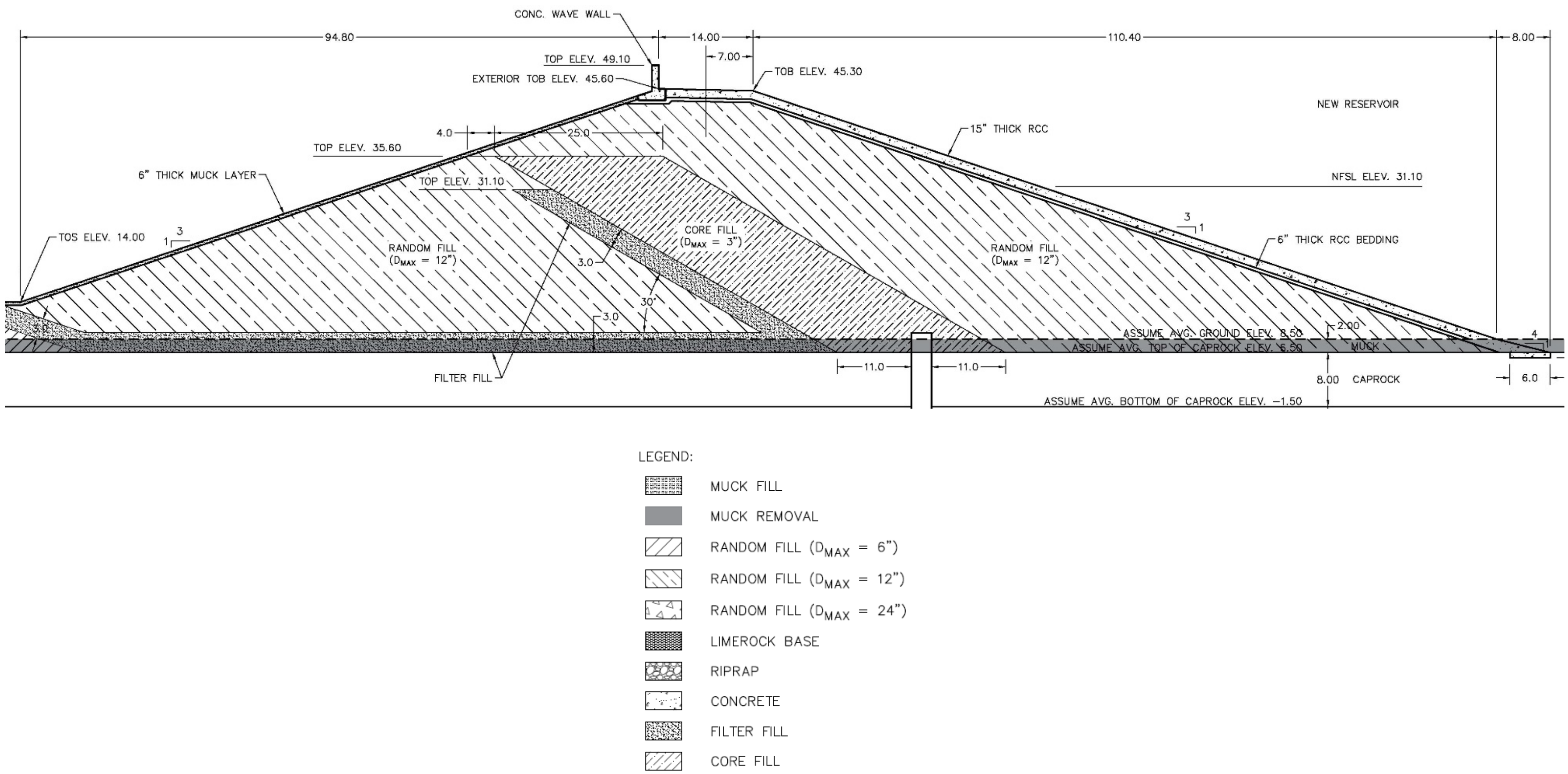

Figure A.5.4-1 Typical Cross Section for the A-2 Reservoir

A.5.4.2 Wave Characteristics

A wave analysis was undertaken to estimate wave growth within the A-2 Reservoir for the wind and precipitation conditions described in **Section A.5.2**. The wave growth section of the Automated Coastal Engineering System (ACES) model was used to identify the wave characteristics that could occur under the design winds, fetch distance, and water depths.

Shallow water restricted fetch conditions were selected within the ACES to consider the shape of the reservoir basin in the wave prediction estimates. A precautionary approach was adopted for the wave growth analysis, assuming a wind direction corresponding to the direction of the maximum fetch. Full details of the wave assessment, including validation of the ACES results, are provided in **Annex A-2**.

The wave results from the ACES analysis are summarized below in **Table A.5.4-1**. Wave heights for the design cases ranged from 8.4 feet to 10.5 feet, while peak wave periods ranged from 5.4 seconds to 6.1 seconds.

Table A.5.4-1. A-2 Reservoir Wave Prediction Results

Design Case	Wind (mph)	Effective Water Depth[1] (ft)	Effective Water Level Elevation[2] (ft-NAVD / ft-NGVD)	Significant Wave Height, Hmo (ft)	Peak Wave Period, Tp (seconds)
1	104.5	27.1	35.60 / 37.03	9.2	5.6
2	123.3	24.0	32.50 / 33.93	10.5	6.1
3 (Sensitivity Testing)	159.7	22.6	31.10 / 32.53	13.0	7.0
4.1	97.5	25.8	34.30 / 35.73	8.4	5.4
4.2	97.5	26.4	34.90 / 36.33	8.5	5.4

[1] Average water depth in Table 5.2-1= Effective Water Depth = NFSL (22.6 ft) + Precipitation

[2] Assuming average ground elevation of 8.50 ft-NAVD (9.93 ft-NGVD)

A.5.4.3 Wind Setup

Wind setup is caused by shear stress exerted on the water surface, which in turn causes a slope in the water surface resulting in wind setup at the leeward side of the reservoir. This setup level influences the water depth at the reservoir embankment, and therefore the wave run-up/overtopping discharge. Hence the calculation of wind setup is required for determination of freeboard (Haapala et al. 2006).

For reservoirs with depths equal to or greater than 16 feet, DCM-2 recommends that wind setup is calculated using the Zeider Zee equation, which calculates wind setup based on wind speed, fetch length and depth. **Table A.5.4-2** summarizes the estimated wind setup for each of the DCM-2 design cases, and the resulting maximum water depth at the leeward side of the reservoir. This maximum depth was subsequently used in the overtopping calculations.

Table A.5.4-2. Summary of Calculated Wind Setup

Design Case	Wind (mph)	Effective Water Depth[1] (ft)	Wind Setup (ft)	Maximum Water Depth[2] (ft)	Maximum Water Level Elevation[3] (ft-NAVD / ft-NGVD)	Freeboard to TOB Water Side[4] (ft)
1	104.5	27.1	1.8	28.9	37.40 / 38.83	7.9
2	123.3	24.0	2.8	26.8	35.30 / 36.73	10.0
3 (Sensitivity Testing)	159.7	22.6	4.8	27.4	35.90 / 37.33	9.4
4.1	97.5	25.8	1.6	27.4	35.90 / 37.33	9.4
4.2	97.5	26.4	1.6	28.0	36.50 / 37.93	8.8

[1] Average water depth = NFSL (22.6 ft) + Precipitation

[2] Maximum water depth = Average water depth + Wind setup

[3] Maximum water elevation based on assumed average ground level of 8.50 ft-NAVD (9.93 ft-NGVD)

[4] Freeboard to TOB water side = TOB water side elevation (45.30 ft-NAVD or 46.73 ft-NGVD) - Maximum water elevation

A.5.4.4 Overtopping Analysis

An overtopping analysis was undertaken to determine the minimum embankment level and wave wall height required to limit overtopping of the A-2 Reservoir to acceptable volumes during wave and wind-setup levels generated from the design conditions specified in **Section A.5.2**. The design wave and water level conditions adopted for the overtopping analysis are summarized below in **Table A.5.4-3**.

A range of analysis techniques, as described in the EurOtop Manual (2016), were used to estimate overtopping characteristics for the proposed 1:3 embankment slope with a vertical wave wall located on the landward side of the embankment crest. Exposure to erosion damage is highly dependent on the wave run-down characteristics on the landward side of the structure, which is a function of the slope type and resilience (e.g. grass quality, soil type, etc.). For the purposes of this feasibility study, acceptable overtopping limits were defined in terms of the mean overtopping discharge, as well as the maximum overtopping volume for a single wave as follows:

- **Mean overtopping discharge:** A mean overtopping discharge limit of 0.1 cfs per lineal foot of embankment was selected for the A-2 Reservoir, as per guidance given in DCM-2. This limit is broadly in line with recommendations in the EurOtop Manual (2016) to prevent damage to a grass sloped embankment, and relates to a “Start of Damage” condition based on guidance in the Coastal Engineering Manual (USACE, 2002). As the wind and precipitation scenarios used for the design of the A-2 Reservoir are associated with rare extreme storm scenarios, a "Start of Damage" (i.e. minor damage) condition is deemed appropriate.
- **Maximum overtopping volume for a single wave:** The limit for the maximum overtopping volume of a single wave was selected as 32.3 ft^3/ft as per recommendations in the EurOtop Manual (2016) to prevent damage to grass sloped embankments.

Table A.5.4-3. Design Water Levels and Wave Conditions Adopted for the Overtopping Analysis

Design Case	Maximum Water Depth[1] (ft)	Maximum Water Level Elevation[2] (ft- NAVD / ft-NGVD)	Significant Wave Height, H_{mo} (ft)	Peak Wave Period, T_p (seconds)
1	28.9	37.40 / 38.83	9.2	5.6
2	26.8	35.30 / 36.73	10.5	6.1
3 (Sensitivity Testing)	27.4	35.90 / 37.33	13.0	7.0
4.1	27.4	35.90 / 37.33	8.4	5.4
4.2	28.0	36.50 / 37.93	8.5	5.4

[1] Maximum water depth = NFSL (22.6 ft) + Precipitation + Wind setup

[2] Maximum water elevation = Average water depth + Wind setup

A.5.4.4.1 Mean Overtopping Discharge

The 2016 EurOtop Manual provides specific guidance for estimating the mean overtopping rate at structures similar to the design proposed for the A-2 Reservoir (a mild-sloped embankment with a vertical wave wall located on the landward side of the embankment), and therefore this methodology was adopted in the wave overtopping analysis. The equations used for the analysis were based on those specified for a “deterministic design or safety assessment” approach, which include a partial safety factor of one standard deviation (EurOtop 2016). Advice provided by Van Doorslaer et al. (2016) was used to adapt these equations for the plunging wave conditions (i.e $\xi_{m-1,0}<1.8$) generated in the A-2 Reservoir under the design wind and water levels (refer to the methodology in the C-43 Reservoir Overwash and Wave Forces Report [Hughes 2017]).

Overtopping discharges were calculated for wave wall heights ranging from 2 ft to 4 ft. The results indicate that a 3.5 ft wave wall achieves acceptable overtopping rates below 0.1 cfs/ft as shown in **Table A.5.4-4**. As per recommendations is DCM-2, Design Case 3 is used for sensitivity testing only and not as a selected design condition.

Table A.5.4-4. Results of Overtopping Analysis

Wave Wall Height (ft)	Wave Wall Top Elevation (ft-NAVD / ft-NGVD)	Design Case	Freeboard to Top of Wave Wall (ft)[1]	Overtopping Rate (cfs/ft)
2.0	47.60 / 49.03	1	10.20	0.18
		2	12.28	0.18
		3 (Sensitivity Testing)	11.70	0.98
		4.1	11.68	0.04
		4.2	11.13	0.06
2.5	48.10 / 49.53	1	10.70	0.14
		2	12.78	0.15
		3 (Sensitivity Testing)	12.20	0.84
		4.1	12.18	0.03
		4.2	11.63	0.04
3.0	48.60 / 50.03	1	11.20	0.11
		2	13.28	0.12
		3 (Sensitivity Testing)	12.70	0.72
		4.1	12.68	0.02
		4.2	12.13	0.03
3.5	49.10 / 50.53	1	11.70	0.08
		2	13.78	0.09
		3 (Sensitivity Testing)	13.20	0.61
		4.1	13.18	0.02
		4.2	12.63	0.02
4.0	49.60 / 51.03	1	12.20	0.06
		2	14.28	0.07
		3 (Sensitivity Testing)	13.70	0.52
		4.1	13.68	0.01
		4.2	13.13	0.02

[1] Freeboard to top of wave wall = Wave wall top elevation – Maximum water level elevation (including precipitation and wind setup)

A.5.4.4.2 Maximum Overtopping Volume

The maximum overtopping volume of a single wave was also estimated as per equations provided in the EurOtop Manual (2016). These equations are based on various parameters, including the mean overtopping discharge, storm duration, and the percentage of overtopping waves.

Based on the results of the mean overtopping discharge analysis (**Table A.5.4-4**), the maximum overtopping volume of a single wave was calculated for a 3.5 ft tall wave wall. **Table A.5.4-5** summarizes the results for this analysis, including the percentage of overtopping waves which is a function of the 2% run-up height (EurOtop 2016).

The results indicate that a 3.5 ft wave wall achieves acceptable overtopping for the reservoir, with the maximum overtopping volume predicted to remain below the 32.3 ft^3/ft limit for all design cases (i.e. 1, 2, and 4). The values presented in **Table A.5.4-5** are based on storm duration of approximately 3 hrs, which is considered to be precautionary based on a statistical analysis of historical hurricane events in the region (refer to **Annex A-2**).

Table A.5.4-5. Summary of Overtopping Probability and Maximum Overtopping Volume for a Single Wave (assuming 3 hr storm duration)

Design Case	Freeboard to Top of Wave Wall[1] (ft)	Probability of Overtopping	Maximum Overtopping Volume for a Single Wave (ft^3/ft)
1	11.70	27.7%	22.4
2	13.78	26.3%	29.4
3 (Sensitivity Testing)	13.20	47.1%	108.5
4.1	13.18	14.4%	8.7
4.2	12.63	17.2%	10.6

A.5.4.5 Findings and Recommendations

A wave and overtopping analysis was undertaken to support the preliminary design of the A-2 Reservoir. The wind adjustment and wave growth module of ACES was used to estimate wave conditions generated within the A-2 Reservoir for the wind and precipitation design cases specified by DCM-2. Design wave heights predicted for the reservoir ranged from 8.4 ft to 10.5 ft, while peak wave periods ranged from 5.4 seconds to 6.1 seconds. These estimates are assumed to be suitable for the purposes of the feasibility study. It is recommended that they are confirmed using a more comprehensive model (e.g., STWave) in subsequent design phases.

An overtopping analysis was undertaken to determine a suitable embankment crest configuration to limit overtopping of the A-2 Reservoir to acceptable volumes during wave and wind-setup levels generated from the DCM-2 design cases. A range of analysis techniques, as described in the EurOtop Manual (2016), were used to estimate overtopping characteristics for the proposed 1:3 embankment slope with a vertical wave wall located on the landward side of the embankment crest. The results from the analysis indicate that an embankment with a water side crest elevation at 45.30 ft-NAVD (46.73 ft-NGVD), landward crest elevation of 45.6 ft-NAVD (47.03 ft-NGVD), and a 3.5 ft tall vertical wave wall is likely to achieve acceptable overtopping rates both in terms of the mean overtopping discharge (i.e., < 0.1 cfs/ft) and the maximum overtopping volume for a single wave (i.e., < 32.3 ft^3/ft).

The proposed cross-sectional design could potentially be refined to better manage wave overtopping at the reservoir. Potential design refinements to be investigated include the following:

- Alternative wave wall design (Increased the height, inclusion of a parapet or recurve wall)
- Inclusion of an intermediate berm
- Increasing the roughness of the slope and/or crest by (e.g. quarry stones, concrete blocks) to reduce wave run-up
- Armoring of the outer (landward side) slope of the embankment to provide increased protection against overtopping
- The inclusion of intermediate dikes within the dam to limit the wave height, and hence reduce the dam elevation.

In addition, it is recommended that the spatial variability in the wave overtopping along the embankment is investigated and the design refined accordingly.

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A.6 HYDRAULIC DESIGN

A.6.1 INTRODUCTION

The TSP includes the A-2 Reservoir with a storage capacity of 240,347 acre-feet at a normal pool depth of 22.6 feet, the A-2 STA with 6,500 acres of effective treatment area that discharges to the Miami Canal, and no changes to the existing A-1 FEB infrastructure. A new Inflow-Outflow Canal would connect the Miami Canal and the North New River (NNR) Canal to the northern portion of the A-2 Reservoir. On the northern boundary of the A-2 Reservoir there is an inflow pump station (P-1), an outflow gated structure (C-1), as well as an overflow weir (SW-1) that discharges to the A-2 Reservoir Inflow-Outflow Canal (Inflow-Outflow Canal). Flows from the Miami Canal and NNR canal to P-1 will be controlled by gated spillways SW-2 and SW-3, respectively. On the eastern boundary of the A-2 Reservoir, there is an inflow/outflow gated structure (C-10) connecting the reservoir to the existing A-1 FEB. On the southern boundary of the A-2 Reservoir, there is an inflow/outflow gated structure (C-9) connecting the reservoir to the STA 3/4 Inflow Canal. On the western boundary of the A-2 Reservoir, there are two outflow gated structures (C-3 and C-4) connecting the reservoir to the A-2 STA. The upstream EAV cells (cells 3 and 4) of the STA are connected to the downstream SAV cells (cells 1 and 2) via gated structures C-5 and C-6. The SAV cells are connected to the STA Discharge Canal via outflow gated structures C-7 and C-8. On the southwest corner of the STA the outflow structure C-2 connects the STA Discharge Canal to the Miami Canal and allows for the STA to discharge to the Miami Canal south of Spillway G-373. **Figure A.6.1-1** shows a schematic site plan of the proposed EAA Storage Reservoir project.

Figure A.6.1-1. EAA Reservoir Schematic

The TSP also includes improvements to conveyance between Lake Okeechobee and the proposed A-2 Reservoir by adding 1,000 cfs of additional conveyance capacity to the Miami Canal and 200 cfs of additional conveyance capacity to the NNR Canal. Details for the proposed conveyance capacity improvements to the Miami Canal and NNR Canal are provided in **Annex A-1**.

A.6.2 GENERAL RESERVOIR DESIGN GUIDELINES

The design criteria defined for the new A-2 Reservoir and A-2 STA water control structures was determined in consultation with SFWMD staff and is based on results obtained from SFWMD's C240AE model and design reference documents (DCM-2, DCM-3, BOR, etc.). The SFMWD C240AE model simulates in a regional setting the inflows to, outflows from, and operations of the A-2 Reservoir for a 41-year period of climatological inputs (rainfall and evapotranspiration). Releases from the A-2 Reservoir will meet environmental needs for the Everglades and supplemental irrigation needs for the Miami and North New River basins of the EAA (see SFWMD modeling report for project in **Annex A-2**). The DCM-3 Guidelines for spillway capacity and reservoir drawdown criteria states that a spillway is required for high hazard potential impoundments and project works must be designed to either withstand overtopping for the loading condition that would occur during a flood or to the point where a failure would no longer cause an unacceptable additional downstream threat up to the probable maximum flood (PMF) resulting from the probable maximum precipitation (PMP) event. The total rainfall depth during the PMP event predicted for the A-1 Reservoir is 54 inches (4.5 feet) as discussed in **Section A.5**.

The reservoir design assumes that the inflow pumps will be off when the reservoir level reaches the normal full storage level (NFSL) at elevation 31.10 feet-NAVD (32.53 feet-NGVD), which is the maximum elevation of storage where drawdown outflow from the reservoir would begin. Average ground elevation is around 8.50 feet-NAVD (9.93 feet-NGVD). DCM-3 reservoir drawdown requirements are based on USACE ER 1110-2-50, Low Level Discharge Facilities for Drawdown of Impoundments. At a minimum, low level discharge facilities will be sized to be capable of reducing the normal full storage to a pool level which will result in an amount of storage in the reservoir that is 10 percent of the NFSL, within a period of four months. The beginning pool level for drawdown will be the NFSL. As outlined in the BOR, for the EAA, the SFWMD criteria allow a discharge of 20 cfs per square mile (equal to ¾ inch of runoff per 24 hours) with a five-year design frequency.

A.6.3 A-2 Reservoir and A-2 STA Water Control Structures

A.6.3.1 Overflow Weir SW-1

SW-1 is an (uncontrolled) overflow weir with a fixed crest to relieve high flood conditions within the A-2 Reservoir as required for impoundments with high hazard classification. The proposed location of the weir is at the northern boundary of the reservoir, approximately 2.3 miles east from the western boundary of the reservoir. Per the DCM and BOR criteria summarized above, the followings were the established design criteria for the emergency overflow weir:

1. Crest elevation is at the NFSL of 31.10 feet-NAVD (32.53 feet-NGVD)
2. The peak discharge rate shall not exceed ¾ inches in 24 hours for the contributing area of the reservoir when the stage in reservoir is at the PMP zero discharge stage.

Given the A-2 Reservoir contributing area of 10,823 acres (approximately 17 square miles), the allowed peak discharge would be 340 cfs. The typical weir coefficients for a broad crested weir range from 2.6 to 3.1 (HEC-RAS reference manual [Brunner 2016]). Solving the standard weir equation given a height of 4.5

and a flow rate of 340 cfs results in weir lengths of 13.7 feet and 11.5 feet for weir coefficients of 2.6 and 3.1, respectively. The weir coefficient was assumed to be in the lower range given that the lower coefficients are associated with larger weir widths (above 15 feet). The 72-hour, PMP storm was simulated for various weir sizes, starting with a weir length of 13.5 feet, which results in a peak flow of 303.5 cfs and a peak height over the weir of 4.2 feet. Thus, the approach is conservative but to match a peak flow of 340 cfs during the 72-hour PMP storm, a larger weir length is required. By iterative simulations, a weir length of 15.2 feet resulted in a peak flow of 340.6 cfs during the 72-hour, PMP storm. **Figure A.6.3-1** shows the flow hydrograph during the 72-hour PMP storm for weir lengths 13.5 and 15.2 feet. The headwater stages for the first simulation are also shown and are very similar in the second case.

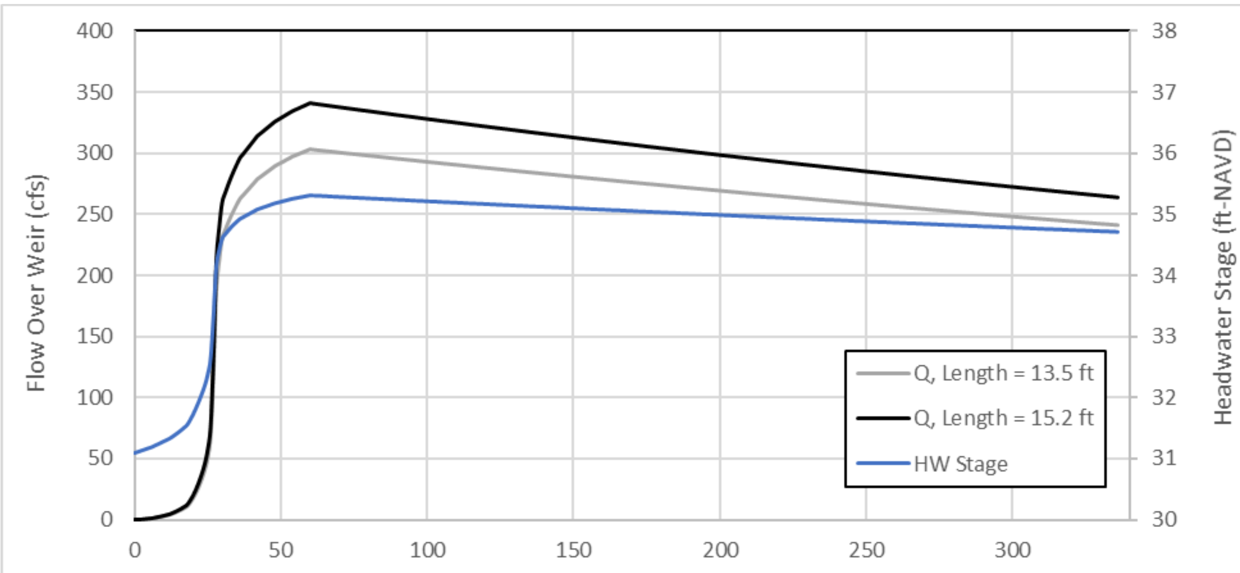

Figure A.6.3-1. Simulated Flow and Headwater Stage at S-1 for Weir Lengths 13.5 and 15.2 Feet

A.6.3.2 Gated Culvert C-1

Structure C-1 is a three-barreled gated box culvert (385 feet long) which will allow for controlled flow from the A-2 Reservoir to the Inflow-Outflow Canal. The proposed location is approximately 900 feet west of pump station P-1.

The design criteria established for structure C-1 are the following:

1. A minimum flow capacity equal to the maximum water supply flow from A-2 Reservoir to the canal system from the SFWMD C240AE model.
2. Reservoir drawdown requirements per DCM-3.
3. A maximum flow velocity of 6 feet/second at the structure. Limiting velocity values will be evaluated again during the next design stage, together with requirements for energy dissipation and development of Maximum Allowable Gate Opening (MAGO) curves.

The maximum simulated flow in the SFWMD C240AE model from the A-2 Reservoir to the canal system was approximately 2,500 cfs. The drawdown requirement, according to the DCM-3, would be to reduce the normal full storage to a pool level with an amount of storage in the reservoir equivalent to 10 percent of the NFSL within a period of four months.

For Culvert C-1 the headwater location is on the A-2 Reservoir side and the tailwater location is in the Inflow-Outflow Canal.

The reservoir storage at NFSL at a depth of 22.6 feet is approximately 240,347 acre-feet. It would take approximately 44 days to drain the reservoir at a rate of 2,500 cfs to 10% of the NFSL. Thus, the drawdown requirement would be met if the structure is sized to the maximum flow rate of 2,500 cfs. A simulation was conducted to determine the number of days it would take to drawdown the reservoir to 10% of the NSFL when the stage is initially at the NFSL. If the gates are completely open during the drawdown operation and using the final revised geometry described below, the simulation showed that it would take 17.75 days to reach the 10% of the NFSL of 10.76 feet-NAVD (12.19 feet-NGVD).

The original proposed size of the structure consisted of four 12 feet x 12 feet box culverts with gates. Using SFWMD (2015) standard discharge equations for culverts, a flow rate of about 15,000 cfs would result with the initial size and a headwater of 31.10 feet-NAVD (32.53 feet-NGVD) and a tailwater of 10.90 feet-NAVD (12.33 feet-NGVD), which correspond to the maximum stage in the reservoir and the historical maximum stages observed at the Miami and North New River canals at headwaters of G-372 and G-370, respectively. This would indicate that initial size of the structure can be reduced substantially and still meet the design criteria. However, a maximum flow velocity criterion of 6 feet/second at the structure was also considered. To meet this criterion at the design flow rate of 2,500 cfs, a total flow area of 417 feet^2 would be required. Thus, the size of the gate was reduced to three 12 feet x 12 feet box culverts with gates. The revised structure geometry is shown in **Table A.6.3-1**.

The results for several types of simulated flow and stage conditions are shown in **Table A.6.3-2**. In each flow conditions scenario, three out of four parameters (headwater, tailwater, gate opening and flow) are specified and the fourth parameter is obtained using the SFWMD standard discharge equations for culverts. For C-1, the maximum headwater is set at the A-2 Reservoir NFSL of 31.10 feet-NAVD (32.53 feet-NGVD) and the minimum tailwater is the minimum operational stage for the Miami Canal and NNR Canal plus 0.5 feet of estimated head losses, resulting in 7.50 feet-NAVD (8.93 feet-NGVD). The high tailwater stage condition was based on the maximum observed stages in the Miami and NNR Canals. The observed stages were obtained from observed time series at the headwater of pump stations G-370 and G-372 during the period of 2006 to 2018.

Other conditions included in **Table A.6.3-2** for culvert C-1 include the stage corresponding to the minimum depth (0.5 feet) in the reservoir of 9.07 feet-NAVD (10.50 feet-NGVD) and the stage corresponding to the suggested minimum depth (8.20 feet) for irrigation water supply deliveries of 16.77 feet-NAVD (10.82 feet-NGVD).

A.6.3.3 Gated Culvert C-10

Structure C-10 is a three-barreled gated box culvert (320 feet long) that would allow for controlled flow between the A-2 Reservoir and the A-1 FEB. Flow through this structure is expected to occur in both directions according to the stage conditions in both impoundments. The proposed location of the structure is approximately 4,550 feet south of the northeast corner of the A-2 Reservoir.

The design criteria established for structure C-10 are the following:

1. A minimum flow capacity of 1,000 cfs from A-2 Reservoir to A-1 FEB with a 5 feet head differential.

2. A minimum flow capacity equal to the maximum simulated flow from A-2 Reservoir to A-1 FEB from the C240AE model.
3. A maximum flow velocity of 6 feet/second at the structure. Limiting velocity values will be evaluated again during the next design stage, together with requirements for energy dissipation and development of Maximum Allowable Gate Opening (MAGO) curves.

For culvert C-10, the headwater location is on the A-2 Reservoir side and the tailwater location is in the A-1 FEB side. Flow is reversed when the tailwater is higher than the headwater and these flows are reported as negative.

The maximum simulated flow in the SFWMD C240AE model from A-2 Reservoir to A-1 FEB is approximately 3,000 cfs. The original proposed size of the structure consisted of four 12 feet x 12 feet box culverts with gates. To optimize the size of the structure, discharges were computed using SFWMD standard flow equations for a fully open culvert under various headwater and tailwater stages conditions. The NFSL for the A-1 FEB is 12.5 feet-NAVD (13.93 feet-NGVD). A flow rate of over 9,000 cfs would result with the initial size and a headwater of 17.50 feet-NAVD (18.93 feet-NGVD) and a tailwater of 12.50 feet-NAVD (13.93 feet-NGVD) (5 feet head differential). This would indicate that initial size of the structure can be reduced and still meet with the design criteria. However, to meet the flow velocity criterion of 6 feet/second at the design flow rate of 3,000 cfs, a total flow area of 500 feet^2 would be required. Thus, the size of the structure was reduced to three 14 feet x 12 feet box culverts with gates. The revised structure geometry is shown in **Table A.6.3-1**.

The results for flow and stage for different conditions for culvert C-10 are shown in **Table A.6.3-2**. In any flow conditions scenario, three out of four parameters (headwater, tailwater, gate opening and flow) are specified and the fourth parameter is obtained using the SFWMD standard discharge equations for culverts. C-10 is expected to operate as both an inflow and outflow structure. For the outflow (from A-2 to A-1) and high reservoir condition, the maximum headwater was set at the A-2 Reservoir NFSL of 31.10 feet-NAVD (32.53 feet-NGVD) and the tailwater is the minimum assumed stage for A-1 under dry conditions, 6.50 feet-NAVD (7.93 feet-NGVD), which is 2 feet below the average ground elevation. For the inflow (from A-1 to A-2) condition, the maximum headwater was set at the A-1 FEB NFSL of 12.50 feet-NAVD (13.93 feet-NGVD) and the tailwater is the minimum assumed stage for A-2 under dry conditions, which is 2 feet below the average ground elevation of 8.50 feet-NAVD (9.93 feet-NGVD). **Table A.6.3-2** also provide the headwater necessary at the A-2 FEB to pass the design flow (3,000 cfs) under the high TW condition in the A-1 FEB.

A.6.3.4 Gated Culvert C-9

Structure C-9 is a four-barreled gated box culvert (374 feet long) that would allow for controlled flow between the A-2 Reservoir and the STA 3/4 Inflow Canal. Flow is expected to occur in both directions according to stage conditions. The proposed location is approximately 6,000 feet west of the A-2 Reservoir eastern boundary.

The design criteria established for structure C-9 are the following:

1. A minimum flow capacity of 1,000 cfs from A-2 Reservoir to the STA-3/4 Inflow Canal with a 5 feet head differential.
2. A minimum flow capacity to allow the partial filling of the A-2 Reservoir at a rate of 4,500 cfs.

3. A maximum flow velocity of 6 feet/second at the structure. Limiting velocity values will be evaluated again during the next design stage, together with requirements for energy dissipation and development of Maximum Allowable Gate Opening (MAGO) curves.

For culvert C-9, the headwater location is on the A-2 Reservoir side and the tailwater location is in the STA 3/4 Inflow Canal side. Flow is reversed when the tailwater is higher than the headwater and these flows are reported as negative.

The original proposed structure consisted of four 12 feet x 12 feet gates. To meet the velocity criterion of 6 feet/second at a flow rate of 4,500 cfs, a total flow area of 750 feet^2 would be required. The original size would meet minimum flow requirements from A-2 Reservoir to the STA-3/4 Inflow Canal, resulting in a flow well above 1,000 cfs for a head differential of 5 feet. However, to meet the velocity criterion with flow from the STA 3/4 Inflow Canal to the A-2 Reservoir, the original structure would be too small. Thus, the original size of the gate was increased to four 16 feet x 12 feet gates. The revised structure geometry is shown in **Table A.6.3-1**.

The results for the flow and stage conditions for C-9 are shown in **Table A.6.3-2**. In any flow conditions scenario, three out of four parameters (headwater, tailwater, gate opening and flow) are specified and the fourth parameter is obtained using the SFWMD standard discharge equations for culverts. Since C-9 is expected to operate as both, an inflow and outflow structure, four scenarios were examined. For the outflow (from A-2 Reservoir to STA 3/4 Inflow Canal), the maximum headwater was set at the A-2 Reservoir NFSL and the tailwater is given as the minimum observed stage in the STA 3/4 Inflow Canal (G-720 headwater stage). Also, for the structure providing outflow with the high tailwater condition as the maximum observed stage in the STA 3/4 Inflow Canal (G-720 headwater), the table shows the headwater stage (in the A-2 Reservoir) necessary to pass the design flow. For the inflow (from STA 3/4 Inflow Canal to A-2), with the maximum headwater as the maximum observed stage in the STA 3/4 Inflow Canal (G-720 headwater), the table provides the highest stage in the A-2 Reservoir under which the design flow of 4,500 cfs can be obtained. In the last condition for C-9, the headwater is the maximum stage observed in the STA 3/4 inflow canal at the G-720 headwater, the tailwater is the minimum assumed stage for A-2 under dry conditions, which is 2 feet below the average ground elevation of 8.50 feet-NAVD (9.93 feet-NGVD), and the design inflow (4,500 cfs) cannot be achieved due to velocity limitations. The observed stages in the STA 3/4 Inflow Canal were obtained from the time series of breakpoint stage at the headwater of the G-720 inflow spillway to the A-1 FEB.

Table A.6.3-1. A-2 Reservoir Gated Box Culverts Structure Geometry

Structure Name	No. of Gates	Culvert Size (feet)	Gate Type	Invert Elev. (feet-NAVD / feet-NGVD)	Max. Required Flow Capacity (cfs)	Max. Allowable Flow Velocity (feet/second)
C-1	3	12Wx12H	Roller	-4.0 / -2.57	2,500	6
C-9	4	16Wx12H	Roller	-4.0 / -2.57	4,500	6
C-10	3	14Wx12H	Roller	-4.0 / -2.57	3,000	6

Table A.6.3-2. A-2 Reservoir Gated Box Culverts Simulated Flow and Stage Conditions

Structure Name	Flow Direction and Condition	HW Elev. (feet-NAVD / feet-NGVD)	TW Elev. (feet-NAVD/ feet-NGVD	Required Gate Opening (feet)	Calculated Flow (cfs)	Comments
C-1	A-2 Reservoir to Inflow Outflow Canal. Drawdown	31.10 / 32.53	10.90 / 12.33	2.2	2,500	High HW and High TW. TW water is the maximum historical elevation in the Miami and North New River Canals.
C-1	A-2 Reservoir to Inflow Outflow Canal. Drawdown	9.07 / 10.50	7.50 / 8.93	6.1	2,500	Low HW and Low TW. HW corresponds to minimum reservoir depth (0.5 feet). TW water is the control elevation in the Miami and North New River Canals plus 0.5 feet.
C-1	A-2 Reservoir to Inflow Outflow Canal. Water Supply	16.77 / 18.20	9.07 / 10.5	3.4	2,500	Low HW under water supply conditions (depth > 8.2 feet). TW water is the water supply elevation in the Miami and North New River Canals.
C-1	A-2 Reservoir to Inflow Outflow Canal. High TW	11.9 / 13.33	10.90 / 12.33	12.0	2,500	High TW corresponds to the maximum historical elevation in the Miami and North New River canals. HW necessary to pass design flow, gates fully open.
C-9	A-2 Reservoir to STA 3/4 Inflow Canal	31.10 / 32.53	10.57 / 12.00	0.50	1,000	High HW and Low TW. TW value is approximately the lowest observed HW at G-720 spillway inflow into A-1 FEB.
C-9	A-2 Reservoir to STA 3/4 Inflow Canal	19.57 / 21.00	14.57 / 16.00	4.2	1,000	TW is approximately the highest observed HW at G-720 spillway inflow into A-1 FEB
C-9	STA 3/4 Inflow Canal to A-2 Reservoir	13.55 / 14.98	14.57 / 16.00	12.0	-4,500	TW is approximately the highest observed HW at G-720 spillway inflow into A-1 FEB.
C-9	STA 3/4 Inflow Canal to A-2 Reservoir	6.50[1]/ 7.93	14.57/ 16.00	3.0	-4,000	Flow restricted by exit velocity limitations (6.0 fps).
C-10	A-2 Reservoir to A-1 FEB	31.10 / 32.53	6.50 / 7.93	2.0	3,000	High HW is maximum stage in A-2 Reservoir and Low TW is 2 feet below ground elevation in the A-1 FEB.
C-10	A-2 Reservoir to A-1 FEB	13.54 / 14.97	12.50 / 13.93	12.0	3,000	High TW at A-1 FEB. HW necessary to pass design flow, gates fully open.
C-10	A-1 FEB to A-2 Reservoir	6.50[1]/ 7.93	12.50 / 13.93	3.5	-2,700[1]	High TW at A-1 FEB. Low HW is 2 feet below ground elevation in the A-2 Reservoir. Flow restricted by exit velocity limitations (6.0 fps).

[1] Values shown assume that the headwater (HW) location is at the A-2 Reservoir. Negative flow values indicate that the flow direction is from the tailwater (TW) to the headwater of the structure.

A.6.3.5 Gated Culverts C-3 and C-4

Structure C-3 and C-4 are two-barreled gated box culverts (370 feet long each) that would allow for controlled flow from the A-2 Reservoir to the A-2 STA. C-3 connects the reservoir to EAV Cell 3 and C-4 connects the reservoir to EAV Cell 4. The proposed locations of C-3 and C-4 are along the north-south levee that separates the A-2 Reservoir from the STA, at approximately 1.7 and 3.7 miles, respectively, from the northern boundary of the A-2 STA.

The design criteria established for sizing all the STA structures are the following:

1. A minimum flow capacity equal to the maximum simulated flow from the A-2 Reservoir to the STA from the C240AE model.
2. A maximum flow velocity of 6 feet/second at the structure.
3. A maximum of 6 inches of head loss across the structure.
4. A maximum flow velocity of 2.5 feet/second in the STA distribution and collection canals.
5. Limiting velocity values will be evaluated again during the next design stage, together with requirements for energy dissipation and development of Maximum Allowable Gate Opening (MAGO) curves on the A-2 STA side of the structures.

For Culverts C-3 and C-4, the headwater location is on the A-2 Reservoir side and the tailwater location is in the A-2 STA side.

The maximum simulated flow in the SFWMD C240AE model from A-2 Reservoir to the A-2 STA is approximately 650 cfs. This flow would be equally distributed along the northern and southern cells. Thus, each structure was sized to convey 325 cfs. The structures were simulated as fully open culverts under various headwater and tailwater stages and flow conditions. The original proposed structure consisted of two 9 feet x 9 feet box culverts with gates. To meet the flow velocity criterion of 6 feet/second at a flow rate of 325 cfs, a total flow area of 54 feet^2 would be required. This criterion could be met with a structure as small as two 5 feet x 5 feet gates. However, based on model output, a minimum size of two 7 feet x 7 feet box culverts with gates would be needed to meet the maximum head loss criterion of 6 inches across the structure. The revised structure geometry is shown in **Table A.6.3-3**.

The results for the simulated flow and stage conditions, as described in **Section A.6.3.2**, for C-3 and C-4 are shown in **Table A.6.3-4**. For the high flow condition, the maximum headwater was set at the A-2 Reservoir NFSL and the tailwater is the minimum assumed stage in the A-2 STA under dry conditions, which is 2 feet below the average ground elevation of 8.50 feet-NAVD (9.93 feet-NGVD). For the design flow condition, the high tailwater stage was set at the expected maximum stage for the A-2 STA of 12.50 feet-NAVD (13.93 feet-NGVD). Higher possible flow at the expected maximum stage for the A-2 STA was also presented.

Table A.6.3-3. STA Gated Box Culverts C-3 and C-4 Structure Geometry

Structure Name	No. of Gates	Culvert Size (feet)	Gate Type	Invert Elev. (feet-NAVD / feet-NGVD)	Max. Required Flow Capacity (cfs)	Max. Allowable Flow Velocity (feet/second)
C-3	2	7Wx7H	Slide	-3.0 / -1.57	325	6
C-4	2	7Wx7H	Slide	-3.0 / -1.57	325	6

Table A.6.3-4. STA Gated Box Culverts C-3 and C-4 Simulated Flow and Stage Conditions

Structure Name	Flow Direction and Condition	HW Elev. (feet-NAVD / feet-NGVD)	TW Elev. (feet-NAVD/ feet-NGVD	Required Gate Opening (feet)	Calculated Flow (cfs)	Comments
C-3/C-4	A-2 Reservoir to STA cell 3 and 4	31.10 / 32.53	6.50 / 7.93	0.74	325	High HW and Low TW.
C-3/C-4	A-2 Reservoir to STA cell 3 and 4	14.0 / 15.43	12.50 / 13.93	7.0	600	Flow restricted by exit velocity limitations (6.0 fps).
C-3/C-4	A-2 Reservoir to STA cell 3 and 4	12.94/ 14.37	12.50 / 13.93	7.0	325	High TW expected at A-2 STA.

A.6.3.6 Gated Culverts C-5 and C-6

Structure C-5 and C-6 are gated box culverts (208 feet long each) that would allow for controlled flow from A-2 STA EAV cells to the A-2 STA SAV cells. C-5 connects EAV Cell 3 to SAV Cell 1 and C-6 connects EAV Cell 4 to SAV Cell 2. The proposed locations of C-5 and C-6 are along the north-south levee that separates the EAV cells from the SAV cells, at approximately 650 feet and 2 miles, respectively, from the northern boundary of the A-2 STA.

Design criteria for C-5 and C-6 are the same as for structures C-3 and C-4, with no need to investigate MAGO curves. Thus, the same sizes as C-3 and C-4 were used and compliance with the described above design criteria was checked. The revised structure geometry is shown in **Table A.6.3-5**.

For Culverts C-5 and C-6, the headwater locations are on cells 3 and 4 the tailwater location are on cells 1 and 2 of the A-2 STA.

The results for the simulated flow and stage conditions, as described in **Section A.6.3.2**, for C-5 and C-6 are shown in **Table A.6.3-6**. For the high flow condition, the maximum headwater was set at the expected maximum stage for the STA of 12.50 feet-NAVD (13.93 feet-NGVD) and the tailwater is the minimum assumed stage in the STA under dry conditions, which is 2 feet below the average ground elevation of 8.50 feet-NAVD (9.93 feet-NGVD). For the design flow condition, the high tailwater stage was set at the expected maximum stage for the A-2 STA of 12.50 feet-NAVD (13.93 feet-NGVD).

Table A.6.3-5. STA Gated Box Culverts C-5 and C-6 Structure Geometry

Structure Name	No. of Gates	Culvert Size (feet)	Gate Type	Invert Elev. (feet-NAVD / feet-NGVD)	Max. Required Flow Capacity (cfs)	Max. Allowable Flow Velocity (feet/second)
C-5	2	7Wx7H	Slide	-3.0 / -1.57	325	6
C-6	2	7Wx7H	Slide	-3.0 / -1.57	325	6

Table A.6.3-6 STA Gated Box Culverts C-5 and C-6 Simulated Flow and Stage Conditions

Structure Name	Flow Direction and Condition	HW Elev. (feet-NAVD / feet-NGVD)	TW Elev. (feet-NAVD/ feet-NGVD	Required Gate Opening (feet)	Calculated Flow (cfs)	Comments
C-5/C-6	STA cell 3 and 4 to STA cell 1 and 2	12.50 / 13.93	6.50 / 7.93	1.44	325	High HW and Low TW.
C-5/C-6	STA cell 3 and 4 to STA cell 1 and 2	12.88/ 14.31	12.50 / 13.93	7.0	325	High TW expected at A-2 STA.

A.6.3.7 Gated Culverts C-7 and C-8

Structure C-7 and C-8 are two-barreled gated box culverts (208 feet long each) that would allow for controlled flow from the A-2 STA SAV cells to the A-2 STA Discharge Canal. C-7 connects SAV Cell 1 to the STA Discharge Canal and C-8 connects SAV Cell 2 to the STA Discharge Canal. The proposed locations of C-7 and C-8 are along the north-south levee that separates the A-2 STA from the A-2 STA Discharge Canal, at approximately 0.9 and 2.8 miles, respectively, from the northern end of the STA Discharge Canal.

For Culverts C-7 and C-8 the headwater location is on cells 1 and 2 of the A-2 STA the tailwater location is on the Collection Canal side. Design criteria for C-7 and C-8 are the same as for structures C-3 and C-4. Thus, the same sizes as C-3 and C-4 were used and compliance with the described above design criteria was checked. The revised structure geometry is shown in **Table A.6.3-7**.

The results for the simulated flow and stage conditions, as described in **Section A.6.3.2**, for C-7 and C-8 are shown in **Table A.6.3-8**. For the high flow condition, the maximum headwater was set at the expected maximum stage for the A-2 STA of 12.50 feet-NAVD (13.93 feet-NGVD) and the tailwater is approximately the lowest observed TW at G-373 spillway. For the design flow condition, the high tailwater stage was set at approximately the highest observed TW at G-373 spillway.

Table A.6.3-7. STA Gated Box Culverts C-7 and C-8 Structure Geometry

Structure Name	No. of Gates	Culvert Size (feet)	Gate Type	Invert Elev. (feet-NAVD/ feet-NGVD)	Max. Required Flow Capacity (cfs)	Max. Allowable Flow Velocity (feet/second)
C-7	2	7Wx7H	Slide	-3.0 / -1.57	325	6
C-8	2	7Wx7H	Slide	-3.0 / -1.57	325	6

Table A.6.3-8. STA Gated Box Culverts C-7 and C-8 Simulated Flow and Stage Conditions

Structure Name	Flow Direction and Condition	HW Elev. (feet-NAVD / feet-NGVD)	TW Elev. (feet-NAVD/ feet-NGVD	Required Gate Opening (feet)	Calculated Flow (cfs)	Comments
C-7/C-8	STA cell 1 and 2 to A-2 STA discharge Canal	12.50 / 13.93	7.57 / 9.00	1.57	325	Low TW is approximately the lowest observed TW at G-373 spillway
C-7/C-8	STA cell 1 and 2 to A-2 STA discharge Canal	10.95 / 12.38	10.57 / 12.00	7.0	325	High TW is approximately the highest observed TW at G-373 spillway

A.6.3.8 Un-Gated Culvert C-2

Structure C-2 is an un-gated box culvert (642 feet long) that connects the A-2 STA Discharge Canal (headwater) to the Miami Canal just downstream of existing structure G-373 (tailwater). C-2 will be installed deep enough so that it crosses under the existing STA 3/4 Inflow Canal, without creating any obstruction of flow within the STA 3/4 Inflow Canal. Since the bottom elevation of the STA 3/4 Inflow Canal is at -7.00 feet-NAVD (-5.97 feet-NGVD), the invert elevation of C-2 was set at -14.50 feet-NAVD (-13.07 feet-NGVD) with the internal height of C-2 restricted to 6 feet. Design criteria for C-2 is that it should convey the total design flow for the A-2 STA of 650 cfs. The structure geometry is shown in **Table A.6.3-9**.

The results for the simulated flow and stage conditions, as described in **Section A.6.3.2**, for C-2 are shown in **Table A.6.3-10**. For the high flow condition, the maximum headwater was set at the expected maximum stage for the A-2 STA of 12.50 feet-NAVD (13.93 feet-NGVD) and the low tailwater is the expected minimum operational stage at the Miami Canal downstream of G-373. For the design flow condition, the high tailwater stage was set at the maximum observed stage in the Miami Canal at the tailwater of the G-373 spillway.

Table A.6.3-9. STA Un-Gated Box Culverts C-2 Structure Geometry

Structure Name	No. of Gates	Culvert Size (feet)	Gate Type	Invert Elev. (feet-NAVD/ feet-NGVD)	Max. Required Flow Capacity (cfs)	Max. Allowable Flow Velocity (feet/second)
C-2	2	15Wx6H	N/A	-14.5 / -13.07	650	6

Table A.6.3-10. STA Un-Gated Box Culverts C-2 Simulated Flow and Stage Conditions

		High Headwater, Low Tailwater			Design Flow, High Tailwater		
Structure Name	Flow Direction	HW Elev. (feet-NAVD/ feet-NGVD)	TW Elev. (feet-NAVD/ feet-NGVD)	Calculated Flow (cfs)	Calculated HW Elev. (feet-NAVD/ feet-NGVD)	TW Elev. (feet-NAVD/ feet-NGVD)	Design Flow (cfs)
C-2	STA Discharge Canal to Miami Canal	12.50 / 13.93	6.50 / 7.93	1,608	11.08 / 12.51	10.60 / 12.03	650

A.6.3.9 STA Design Storm Simulation

In addition to the structure simulations, a 100-year, 24-hour design storm MIKE 11 simulation of the A-2 STA system was conducted to estimate how much time it would take for the A-2 STA to return to its original pre-storm maximum storage depth of 4.0 feet (or a stage of 12.5 feet-NAVD (13.93 feet-NGVD)). The 100-year, 24-hour design storm for the EAA has a total rainfall depth of 9 inches (**Section A.5**). The tailwater boundary condition at the Miami Canal was held at the constant stage of 10.6 feet-NAVD, which is the maximum observed stage of the tailwater at the G-373 spillway. It is assumed that the A-2 STA will receive no inflow from the A-2 Reservoir during the storm and all the A-2 STA outflow gates are fully open throughout the duration of the storm and after the storm. The Manning's n roughness coefficients used are based on calibrated values used for previous SFWMD STA design projects for the EAV and SAV treatment cells.

Figure A.6.3-2 shows the stages during the storm at one of the upstream EAV cells and the 24-hour rainfall distribution. The peak of the storm occurs at hour 12 of the simulation and the stages reach the original stage of 12.5 feet-NAVD (13.93 feet-NGVD) at hour 62. Thus, it takes approximately 2.1 days for stages to recede back to the initial depth of 4 feet during these conditions.

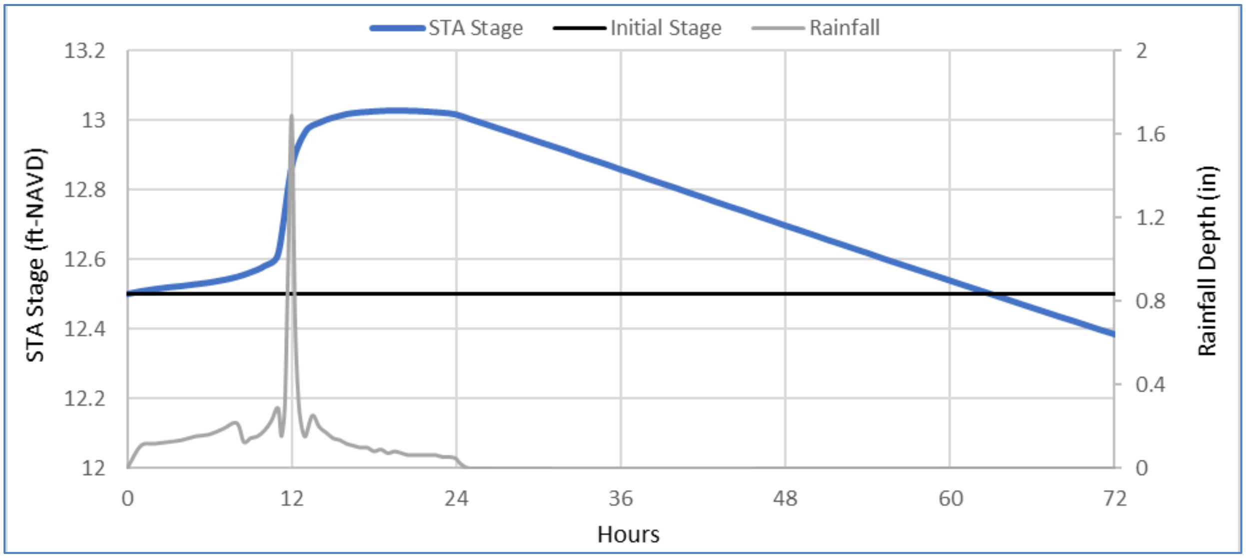

Figure A.6.3-2. STA Simulated Stages During 100-Year Design Storm

A.6.4 A-2 RESERVOIR INFLOW OUTFLOW CANAL & STA 3/4 INFLOW CANAL STRUCTURES

A.6.4.1 Gated Spillway SW-2 and SW-3

Structures SW-2 and SW-3 are bi-directional gated spillways that connect the A-2 Reservoir Inflow-Outflow Canal to the Miami Canal and NNR Canal. SW-2 is located on the west side of the Inflow-Outflow Canal connecting to the Miami Canal and SW-3 is located on the eastern side of the Inflow-Outflow Canal connecting to the NNR Canal.

The design criteria established for the SW-2 and SW-3 structures are the following:

1. A minimum flow design flow of 3,000 cfs.
2. A maximum of 4.8 inches (0.4 feet) of head loss across the structure at the design flow.
3. A maximum flow velocity of 6 feet/second at the structure.
4. Limiting velocity values will be evaluated again during the next design stage, together with requirements for energy dissipation and development of Maximum Allowable Gate Opening (MAGO) curves.

To size the structure, Case 5 spillway equations from SFWMD (2015) for uncontrolled submerged conditions with flow parameters for ogee spillways from Ansar and Chen (2009) were used under various flows and stages. The velocity criterion was also considered. **Table A.6.4-1** shows the revised structure geometry and **Table A.6.4-2** shows the calculated head loss under two design flow conditions (high and low tailwater stages) and the calculated flow under high headwater and low tailwater conditions.

For SW-2, the headwater is located on the Miami Canal side. For SW-3, the headwater is located on the NNR Canal side. For both cases, the tailwater is located on the Inflow-Outflow Canal.

Table A.6.4-1. Gated Spillways SW-2 and SW-3 Structure Geometry

Structure Name	No. of Gates	Spillway Gate Opening Size (feet)	Gate Type	Gate Bottom Elev. In Closed Position (feet-NAVD / feet-NGVD)	Gate Top Elev. In Closed Position (feet-NAVD/ feet-NGVD)	Max. Required Flow Capacity (cfs)	Max. Allowable Flow Velocity (feet/second)
SW-2	3	25Wx14H	Roller	-2.0 /-0.57	12.0 / 13.43	3,000	6
SW-3	3	25Wx14H	Roller	-2.0 / -0.57	12.0 / 13.43	3,000	6

Table A.6.4-2. Gated Spillways SW-2 and SW-3 Simulated Flow and Stage Conditions

Flow/Stage Condition	Calculated HW Elev. (feet-NAVD/ feet-NGVD)	HW Elev. (feet-NAVD/ feet-NGVD)	TW Elev. (feet-NAVD/ feet-NGVD)	Design Flow (cfs)	Calculated Flow (cfs)	Calculated Flow Velocity (feet/second)	Maximum Allowable Gate Opening (feet)
Design Flow, High TW[1]	10.91 / 12.34	-	10.80 / 12.23	3,000	-	3.1	-
Design Flow, Low TW[2]	6.77 / 8.20	-	6.50 / 7.93	3,000	-	4.7	-
High Flow (High HW, Low TW)	-	10.80 / 12.23	6.50 / 7.93	3,000	-	4.7	3.21

[1] Maximum observed stage in the Miami and North New River Canal.

[2] Minimum stage expected in the A-2 Inflow-Outflow Canal.

A.6.4.2 Gated Spillway SW-4

Structure SW-4 is a gated spillway that will serve as a divide structure in the STA 3/4 Inflow Canal, east of the C-9 structure and south of existing Spillway G-720. SW-4 would allow for the control of separate stages in the STA 3/4 Inflow Canal south of the A-2 Reservoir and south of the A-1 FEB. The spillway would be located approximately 1,500 feet south of the existing G-720 spillway that serves as one of the inflow structures for the A-1 FEB.

The design criteria established for the SW-4 structure are the following:

1. A minimum flow design flow of 4,000 cfs.
2. A maximum of 4.8 inches of head loss across the structure at the design flow.
3. A maximum flow velocity of 6 feet/second at the structure.
4. Limiting velocity values will be evaluated again during the next design stage, together with requirements for energy dissipation and development of MAGO curves.

To size the structure, Case 5 spillway equations from SFWMD, (2015) for uncontrolled submerged conditions with flow parameters for ogee spillways from Ansar and Chen (2009) were used under various flows and stages. The velocity criterion was also considered. **Table A.6.4-3** shows the revised structure geometry and **Table A.6.4-4** shows the calculated head loss under two design flow conditions (high and low tailwater stages) and the calculated flow under high headwater and low tailwater conditions.

For SW-4, the locations of headwater (north) and tailwater (south) are in the STA3/4 Inflow Canal.

Table A.6.4-3. Gated Spillway SW-4 Structure Geometry

Structure Name	No. of Gates	Spillway Gate Opening Size (feet)	Gate Type	Gate Bottom Elev. In Closed Position (feet-NAVD/feet-NGVD)	Gate Top Elev. In Closed Position (feet-NAVD/feet-NGVD)	Max. Required Flow Capacity (cfs)	Max. Allowable Flow Velocity (feet/second)
SW-4	3	25Wx16H	Roller	0.5 / 1.93	16.5 / 17.93	4,000	6

Table A.6.4-4. Gated Spillway SW-4 Simulated Flow and Stage Conditions

Flow/Stage Condition	Calculated HW Elev. (feet-NAVD /feet-NGVD)	HW Elev. (feet-NAVD / feet-NGVD)	TW Elev. (feet-NAVD / feet-NGVD)	Design Flow (cfs)	Calculated Flow (cfs)	Calculated Flow Velocity (feet/second)	Maximum Allowable Gate Opening (feet)
Design Flow, High TW[1]	15.06 / 16.49	-	14.90 / 16.33	4,000	-	3.7	
Design Flow, Low TW[2]	9.54 / 11.00	-	9.10 / 10.53	3,840	-	6.0	
High Flow (High HW, Low TW)	-	14.90 / 16.33	9.10 / 10.53	3,899	-	6.0	3.62

[1] *Maximum observed stage in the STA 3/4 Inflow Canal at G372_T*
[2] *Minimum observed stages in the STA 3/4 Inflow Canal at G372_T*

A.6.5 PUMP STATION P-1

Pump Station P-1 will serve as the main inflow structure for filling the A-2 Reservoir. The pump station is located along the north embankment of the A-2 Reservoir about 2.8 miles west of the NNR Canal. Inflows to the pump are provided by the A-2 Reservoir Inflow-Outflow Canal. The Inflow-Outflow Canal extends from the Miami Canal to the NNR Canal. Flow from the Miami Canal to the Inflow Canal is controlled by the SW-2 Gated Spillway. Gated Spillway SW-3 plays a similar role on the NNR Canal side.

The design criteria and methodology for the hydraulic design of Pump Station P-1 is provided in **Section A.12**.

A.6.6 A-2 RESERVOIR INFLOW-OUTFLOW CANAL AND A-2 STA CANALS

The canal cross sections for the A-2 Reservoir Inflow-Outflow Canal and the A-2 STA collection, distribution and discharge canals were designed to provide the necessary capacity for anticipated maximum flow rates. Flow rates and velocities were estimated using the Manning's equation with assumed normal flow depths and channel geometry as shown in the typical sections in **Annex C-1**.

A.6.6.1 A-2 Reservoir Inflow Outflow Canal

For the A-2 Reservoir Inflow-Outflow Canal a normal depth of 14.5 feet was assumed, which would be a stage elevation of 6.5 feet-NAVD. The A-2 Reservoir Pump Station P-1 required maximum flow capacity is 4,500 cfs, which is based on a maximum inflow of 3,000 cfs from the Miami Canal and a maximum inflow of 1,500 cfs from the NNR Canal. Since the Inflow-Outflow Canal was designed to have the same flow capacity throughout its entire length, 3,000 cfs was chosen as the minimum flow capacity for the Inflow-Outflow Canal. **Table A.6.6-1** shows the canal design parameters.

Table A.6.6-1. A-2 Reservoir Inflow Outflow-Canal Calculated Flow Capacity

Normal depth, d_n (feet)	Side Slope (H:V)	Bottom Elevation (feet-NAVD / feet-NGVD)	Bottom Width (feet)	Top Width (feet)	Manning's n	Calculated Flow Velocity at d_n (feet/second)	Calculated Flow Capacity at d_n (cfs)	Minimum Req. Flow Capacity (cfs)
14.5	2.5:1	-8.0 / -6.57	55	127.5	0.03	2.3	3,030	3,000

The drawdown in the A-2 Reservoir Inflow-Outflow Canal caused by discharges at Pump Station P-1 was simulated assuming that P-1 is pumping at the maximum discharge capacity of 4,600 cfs and the stages in the Miami Canal and the NNR Canal are fixed at 6.50 feet-NAVD (7.93 feet-NGVD), with no reduction of flow or headlosses across proposed Spillways SW-2 and SW-3. **Figure A.6.6-1** shows the resulting stages in the A-2 Reservoir Inflow-Outflow Canal for three different pump station locations: at mile 4.6, 6.2, and 9.6 from the western end of the canal. The minimum stage simulated was 4.60 feet-NAVD (6.03 feet-NGVD), when P-1 is located at mile 6.2. **Figures A.6.6-2** to **A.6.6-4** show the simulated east and west inflow distribution for the three P-1 locations evaluated.

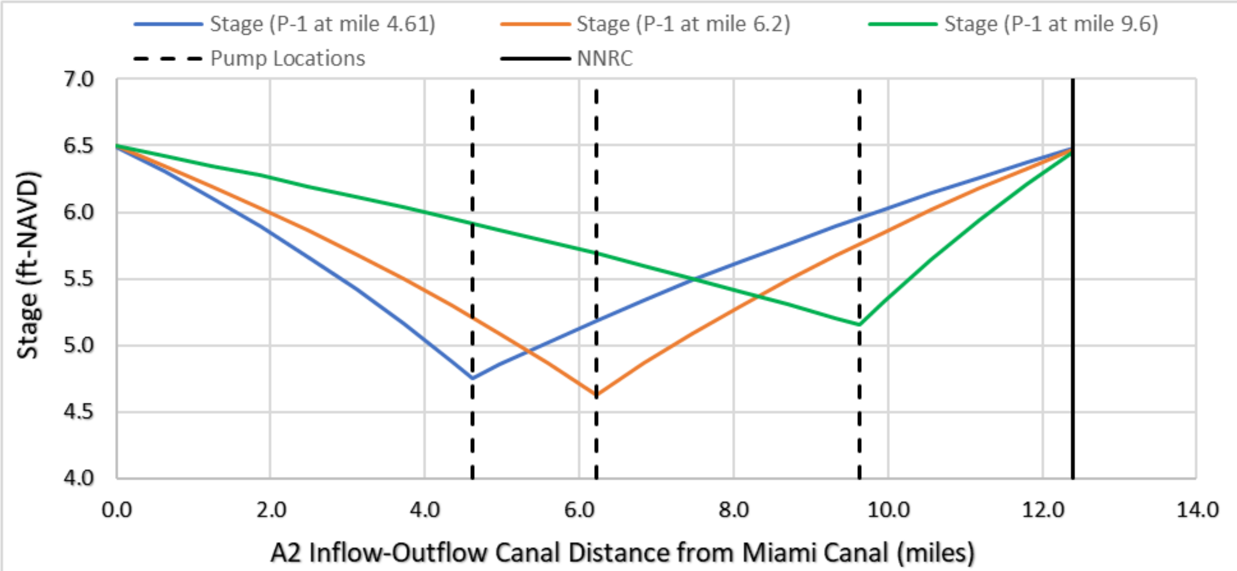

Figure A.6.6-1. Simulated Stages in the A-2 Inflow-Outflow Canal During Maximum Pumping at P-1

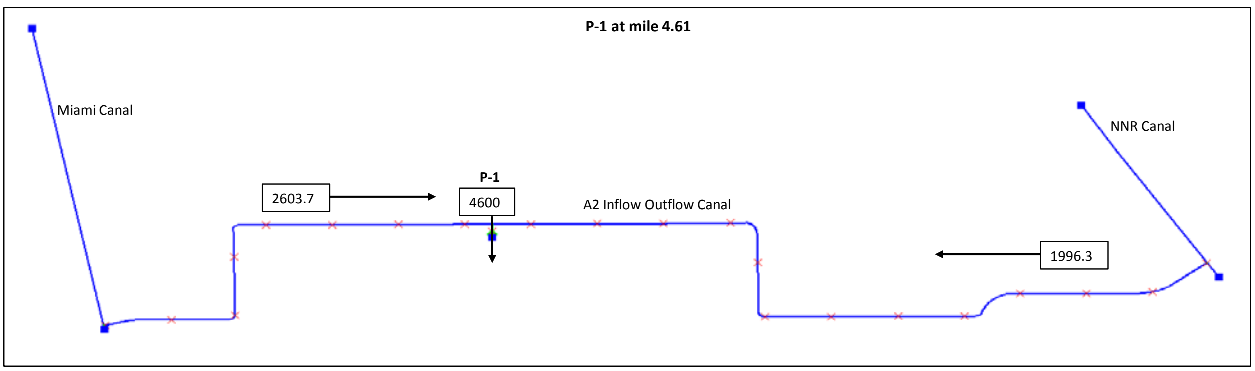

Figure A.6.6-2. Simulated Flows (cfs) in the A-2 Inflow-Outflow Canal for P-1 Location at mile 4.6

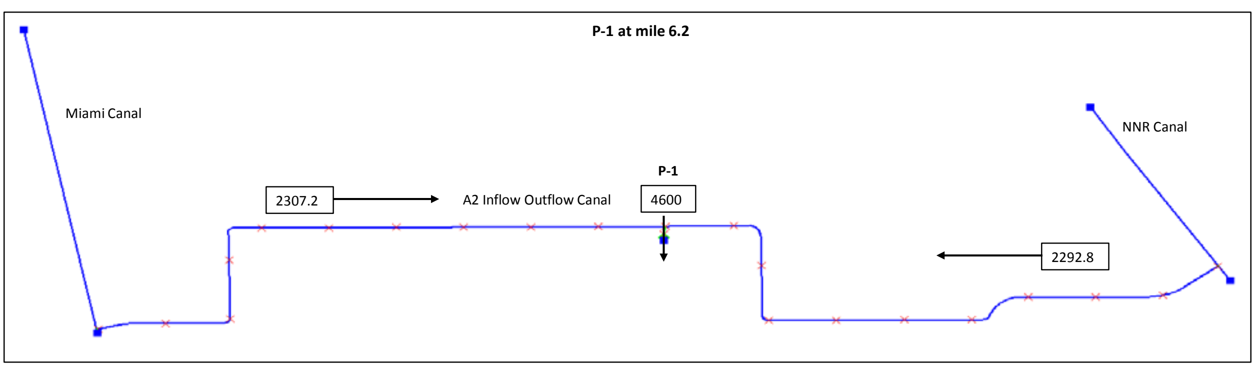

Figure A.6.6-3. Simulated Flows (cfs) in the A-2 Inflow-Outflow Canal for P-1 Location at mile 6.2

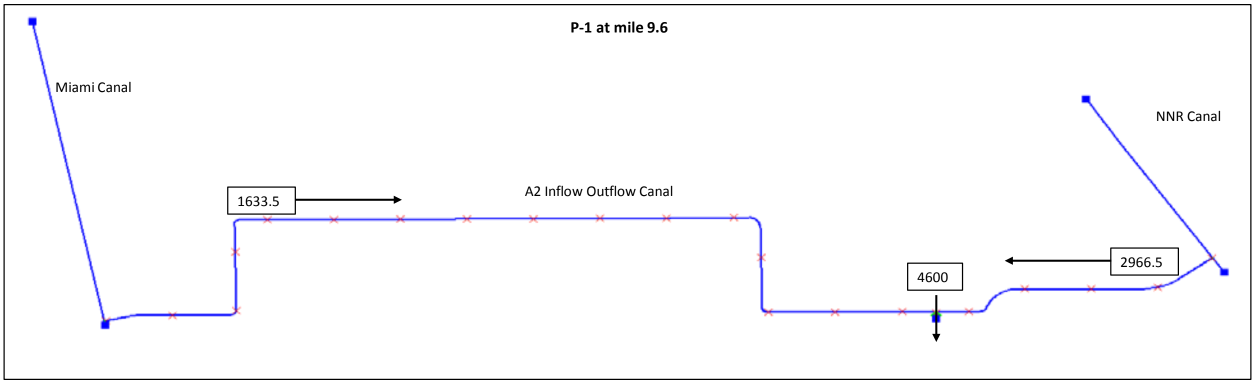

Figure A.6.6-4. Simulated Flows (cfs) in the A-2 Inflow-Outflow Canal for P-1 Location at mile 9.6

A.6.6.2 A-2 STA Distribution and Collection Canals

For the A-2 STA distribution and collection canals a normal depth of 11.5 feet was assumed. **Table A.6.6-2** shows the canal design parameters.

Table A.6.6-2. A-2 STA Distribution and Collection Canals Calculated Flow Capacity

Normal depth, d_n (feet)	Side Slope (H:V)	Bottom Elevation (feet-NAVD / feet-NGVD)	Bottom Width (feet)	Top Width (feet)	Manning's n	Calculated Flow Velocity at d_n (feet/second)	Calculated Flow Capacity at d_n (cfs)	Minimum Req. Flow Capacity (cfs)
11.5	2.5:1	-4.0 / -2.57	22	79.5	0.03	1.8	1,053	325

A.6.6.3 A-2 STA Discharge Canal

For the A-2 STA discharge canal a normal depth of 11.5 feet was assumed. **Table A.6.6-3** shows the canal design parameters.

Table A.6.6-3. A-2 STA Discharge Canal Calculated Flow Capacity

Normal depth, d_n (feet)	Side Slope (H:V)	Bottom Elevation (feet-NAVD / feet-NGVD)	Bottom Width (feet)	Top Width (feet)	Manning's n	Calculated Flow Velocity at d_n (feet/second)	Calculated Flow Capacity at d_n (cfs)	Minimum Req. Flow Capacity (cfs)
11.5	2.5:1	-4.0 / -2.57	22	79.5	0.03	1.8	1,053	650

A.6.7 REFERENCES

Ansar, M., and Z. Chen. 2009. Generalized flow rating equations at prototype gated spillways. Technical Paper, ASCE J. Hydraul. Eng. 135(7):602-608.

Brunner, G.W. 2016. HEC-RAS, River Analysis System Hydraulic Reference Manual. US Army Corps of Engineers Hydrologic Engineering Center.

South Florida Water Management District. 2015. Atlas of Flow Computations at Hydraulic Structures in the South Florida Water Management District. Technical Publication: HHB Report # 2015-001. The Hydraulic Design Unit, Applied Hydraulics Section, Hydrology and Hydraulics Bureau, Operations Engineering & Construction Division, South Florida Water Management District.

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A.7 SUBSURFACE CONSIDERATIONS FOR CONSTRUCTION

A.7.1 Geotechnical Exploration

Several geotechnical site assessments and associated laboratory materials testing programs were conducted for the adjacent A-1 site to determine the nature and engineering properties of the natural ground soils as part of the preparation of the A-1 reservoir Basis Of Design Report (BODR), which was submitted to the SFWMD on January 2006 and the Central Everglades Planning Project Final Integrated Project Implementation Report (CEPP PIR), prepared by the U.S. Army Corps of Engineers (USACE) and the SFWMD on July 2014. Among the geotechnical field exploration programs reviewed, as part of this preliminary evaluation of the A-2 site, there is a conceptual report of geotechnical exploration performed by Ardaman & Associates, Inc., a Tetra Tech Company (Ardaman) in 2003 for which 14 core borings were drilled, two of them within the A-2 site. The borehole advancement and Standard Penetration Test (SPT) sampling was conducted using rotary drilling equipment. The boreholes were drilled to depths between 60 and 180 feet. **Figure A.7-1.1** shows the approximate location of the test borings performed within the A-1 and A-2 sites from the different field exploration programs that were reviewed. The boring logs and laboratory test results of the existing geotechnical reports that were reviewed to define engineering properties of the existing soils within the project area to be used in the seepage and stability analyses of the conceptual embankment cross section are included in **Annex G-1**. These test results are from the following geotechnical reports:

- Conceptual Report of Geotechnical Exploration, Comprehensive Everglades Restoration Plan, Everglades Agricultural Area Reservoirs by Ardaman & Associates, Inc., dated May 2003.
- EAA Reservoir A-1 Geotechnical Exploration by Black & Veatch, dated June 2004
- Test Cell Program Technical Memorandum by Black & Veatch, dated January 2006.
- Everglades Agricultural Area Reservoir A-1 Geotechnical Data Report by Black & Veatch, dated March 2006.
- EAA Supplemental Geotechnical Services Geotechnical Data Report – Supplement 2 by Black & Veatch, dated March 2007.
- Field Exploration Results SFWMD EAA A-2 Storage Reservoir Project, Palm Beach County, Florida by Ardaman & Associates, Inc., dated March 13, 2018.

Additional detailed field geotechnical exploration will be required during the basic engineering design phase of the A-2 Reservoir.

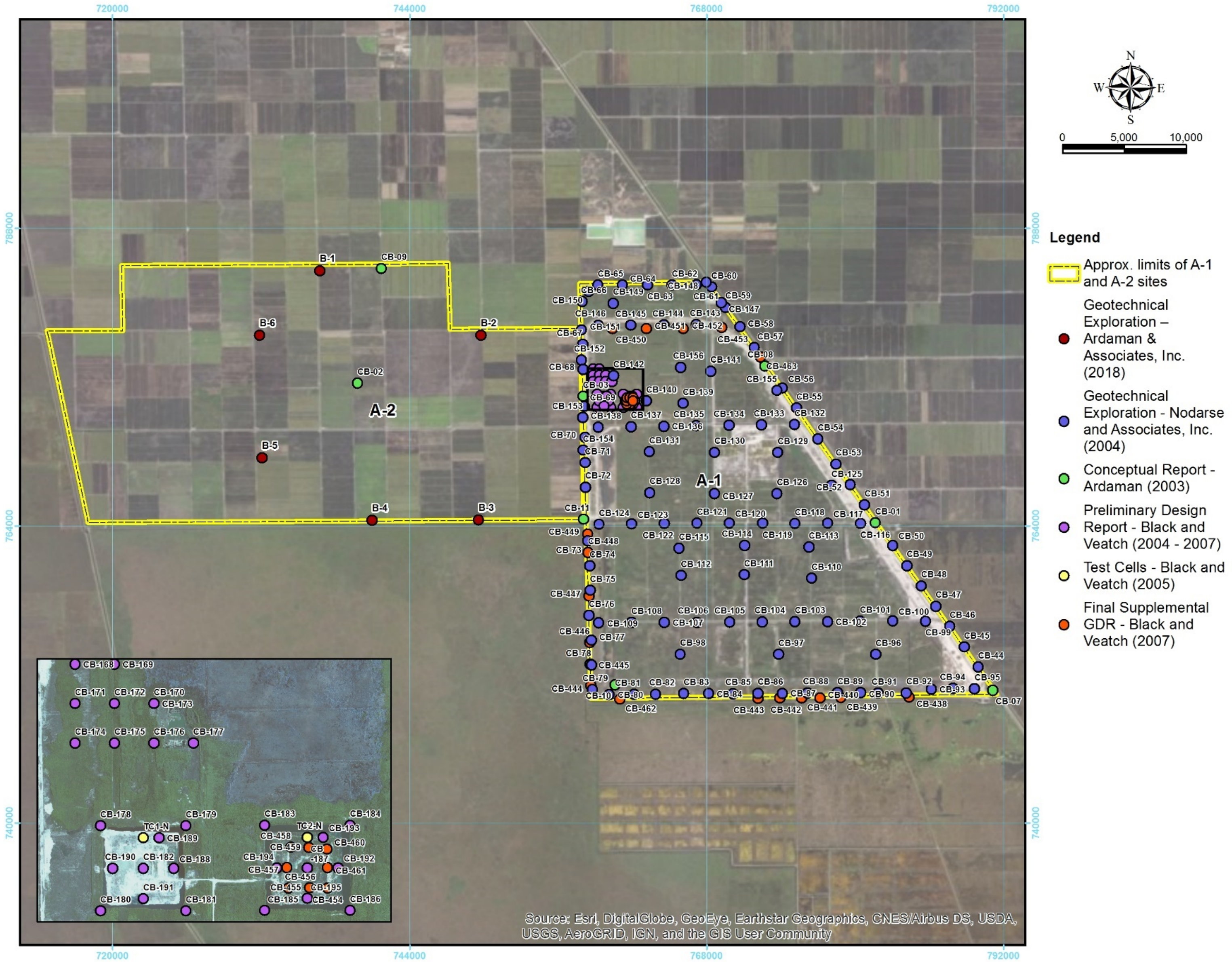

Figure A.7.1-1. Boring Locations

A.7.2 Stratigraphy

The *in situ* materials at the A-2 Reservoir site appear to be similar to those investigated beneath the adjacent A-1 site. The generalized subsurface profile for the A-2 Reservoir and STA site was generated using selected boreholes from the existing geotechnical exploration program due to their proximity to the A-2 site. The generalized subsurface profile used in the conceptual embankment design is as follows:

- Surficial peat and marl: The peat (also referred to as "muck") is a black, highly organic, fine grained soil with a variable thickness of one to two feet. In isolated areas, the muck is underlain by several inches to two feet of calcareous clay (locally called "marl").
- Caprock/upper limestone (Late Pleistocene Fort Thompson Formation): Hard, slightly weathered to un-weathered limestone layer with trace shells (generally referred to as "caprock") varying in thickness from zero to about 10 feet within the A-1 and A-2 sites. Standard Penetration Resistance, N, was in the range of 12 to 50/3".
- Silty carbonate sand with limestone layers (Fort Thompson Formation): Silty carbonate sand containing shell fragments, extending to about 35 to 54 feet deep across the sites; average calcium carbonate content is 84 percent; average percent passing the No. 200 sieve is about 22 percent. Standard Penetration Resistance, N, was in the range of 3 to 50/6".
- Sand with sparse limestone layers and intervals of hard drilling (Early Pleistocene Caloosahatchee Formation): Shelly, fine-grained, sub-rounded, quartz sand mixed with shelly carbonate sand extending to about 60 feet. Proportions of calcium carbonate to quartz vary greatly; average calcium carbonate content is 40 percent and average percent passing the No. 200 sieve is about 12 percent. Standard Penetration Resistance, N, was in the range of 5 to 50/5".
- Sand and limestone layers (Pliocene Tamiami Formation): fine to coarse grained sand interbedded with sandy clay to clayey sand, very fine to medium grained, calcareous, poorly consolidated limestone and moderately to well hardened, sandy, fossiliferous limestone extending to about 220 feet deep.

A.7.3 Laboratory Test Results

Laboratory test results performed on samples from the geotechnical site explorations conducted for the adjacent A-1 site were reviewed to define the engineering properties to be used in the preliminary seepage and stability analyses for the conceptual design of the A-2 Reservoir dam embankments. The laboratory test results that were reviewed include:

- Unconfined compressive strength (ASTM D2938) performed on twenty samples of the limestone cores from the December 2004 borings program performed at the Test Cell (within the A-1 site).
- Rock quality test, including LA Abrasion (ASTM C535) and soundness testing (ASTM D5240) performed on samples of filter drain and riprap bedding produced during construction of the Test Cells at the A-1 site.
- Gradation (ASTM D422), moisture content (ASTM D2216), carbonate content (Florida Test Method Designation FM 5-514), percent passing the No. 200 sieve (ASTM D1140), consolidated undrained triaxial tests, unconsolidated undrained triaxial tests, flexible wall permeameter tests

(ASTM D5084) and corrosivity tests (FDOT) performed on selected samples of the soils from the A-1 and A-2 site as part of the evaluations performed for the Comprehensive Everglades Restoration Plan (CERP) Agricultural Area Reservoirs Conceptual Report of Geotechnical Exploration (testing performed by Ardaman), the Test Cell Program Technical Memorandum (testing performed by Nodarse & Associates, Inc.), the Everglades Agricultural Area Reservoir A-1 Geotechnical Data Report (testing performed by Nodarse & Associates, Inc.), and the EAA Supplemental Geotechnical Services Geotechnical Data Report – Supplement 2 (testing performed by Nodarse & Associates, Inc.).

Additional laboratory testing will be required during the basic engineering design phase of the A-2 Reservoir.

A.7.4 Hydrostratigraphy

Based on the field explorations performed within the A-1 site, the surficial aquifer system consists of surficial peat/muck and organic soils underlain by the Fort Thompson Formation, the Caloosahatchee Formation, and the upper portions of the Pliocene Tamiami Formation. The confining unit at the base of the surficial aquifer system consists of the lower portions of the Tamiami Formation and the upper portions of the Miocene Hawthorn Group. The water table is close to the current ground surface.

The limestone layers in the Fort Thompson Formation were reported to be the primary source of groundwater seepage into the site excavations made during construction of the A-1 Reservoir Test Cells. Water was reportedly seen streaming from the bottom of each of the three limestone layers encountered in the dewatered excavations. The limestone layers contain fractures. The caprock contains interconnecting solution channels especially near the top, and single channels up to several inches in diameter that penetrate the full thickness of the layer. The solution channels in the caprock locally contain soil including the peat and marl. Furthermore, the unconformity between the caprock and silty carbonate sand near the top of the Fort Thompson Formation appears to act as a conduit for increased horizontal groundwater flow.

A.7.5 Seismicity

The Uniform Building Code Seismic Zone Map (Gravity Dam Design Engineer Manual 1110-2-2200 by USACE, dated June 1995), shows that the entire state of Florida is in seismic Zone 0. No capable faults or recent earthquake epicenters are known to exist near the project site.

SFWMD's requirements for seismic evaluation of CERP high hazard potential dam projects, such as A-2 Reservoir and STA A-2, are described in DCM-6. Although Southern Florida is a low seismicity region, the possibility exists for earthquake imposed seismic loads on project structures. The potential earthquake loading is low enough that compacted embankments should not be damaged, but the natural sand foundations of the embankments could potentially be affected.

Loose, saturated sandy soils are susceptible to liquefaction (loss of strength from shaking). This loss of strength could lead to sliding or settlement, possibly resulting in embankment failure. DCM-6 presents the design criteria developed jointly by the SFWMD and the U.S. Army Corps of Engineers (USACE) for evaluating liquefaction potential of CERP impoundments.

A.7.6 Borrow

The borrow material for the A-2 Reservoir and A-2 STA embankments will be derived from the caprock and silty sands of the Fort Thompson Formation. The main source of borrow will be from the excavation of the reservoir and STA canals with additional material being provided by borrow areas within the reservoir. The caprock will provide the source for large and small aggregate for roller compacted concrete (RCC), drainage aggregates, and gravel surfacing. Additional field exploration within the A-2 site will be required to further define the borrow materials.

In addition to obtaining borrow material from the excavation required for the Project, there are existing stockpiles of processed (i.e. crushed) caprock and Fort Thompson material located within the A-1 FEB, that are available for use as borrow material for the Project. These stockpiles were produced from the processing of the material obtained from the excavation of the A-1 FEB seepage canal. The location of these stockpiles is shown on **Figure A.1-1**.

A.7.7 Excavations

Peat encountered within the A-2 site will be stripped from the caprock surface during preparation of the A-2 Reservoir and A-2 STA embankments. The peat stripping procedure adopted during the Test Cells construction for the A-1 Reservoir was performed with agricultural scrapers and tractors. Test Cells stripping started with disking of the areas to promote drying. The surficial peat and marl was stripped from the entire footprint of each Test Cell including the seepage collection canals and the bench between the embankment and seepage collection canals. The areas with deeper or wet materials were completed with dozers pushing the soil into piles that were loaded to dump trucks with excavators. The stripped materials were transported to the perimeter of the active construction area and placed in berms.

Based on the subsoil conditions encountered on the A-1 site during the construction of the Test Cells, blasting is expected to be required for breaking up the caprock for excavations in the A-2 Reservoir and A-2 STA area. As previously discussed in the BODR for the A-1 Reservoir, the seepage collection canals at the Test Cell sites were 20 feet deep, 20 feet wide at the bottom and had 2H:1V side slopes. The canals were drilled and shot, generally with a pattern of three 10-foot deep blast holes across the canal width.

The procedure recommended for excavation of the canals and borrow areas at the A-1 site may be adopted for use on the A-2 site. Excavation of the canals and borrow areas at the A-2 site should be performed as follows: first the caprock should be removed and transported to the future embankment location, the rock processing plant, or other stockpiles. After the caprock is removed, the underlying silty sand can be excavated using hydraulic excavators and stockpiled directly alongside the canals or borrow pits to promote drainage of excess moisture from the material.

A.7.8 Crushing/Material Processing

Recommendations developed for crushing and material processing for the EAA A-1 Reservoir site can be adopted for the A-2 site. Creation of aggregates from the caprock will require crushing, screening, and washing. The caprock contains solution cavities and fractures filled with peat and marl. Because of the high groundwater table, the peat and marl remains moist and will adhere to the caprock when excavated. It is not possible to completely remove the muck/peat from the caprock surface during the stripping operation. A roller grizzly is typically used to effectively clean processed rock in Florida.

The wet silty sands excavated and stockpiled during Test Cell construction for the A-1 Reservoir did not drain well and the moisture content in the stockpiled material was well above optimum for compaction. The soil on the surface of the stockpile dried and formed a hard crust that sealed in the moisture. If the wet silty sand is used for the construction of the A-2 embankment, it will require disking, harrowing, or turning with motor graders to reduce the moisture content to the optimum required for compaction. Scarifying the surface of each lift prior to placement of additional lifts will also be required.

A.7.9 Design Parameters

Design parameters for the A-2 Reservoir construction are included in **Section A.8**.

A.7.10 Additional Geotechnical Investigations

As directed by SFWMD, an additional field exploration was performed from February 26, 2018 through March 1, 2018 around the perimeter of the A-2 Reservoir site, by Ardaman & Associates, consisting of six (6) boreholes with depths ranging from 40 ft to 50 ft and two borehole permeability tests, to support the preliminary design of the A-2 Reservoir. A summary of the information obtained from these borings and tests is included in **Annex G-1.**

A.8 EMBANKMENT/DAM DESIGN

A.8.1 General

This section summarizes the evaluation of the preliminary embankment cross sections proposed for development of the A-2 Reservoir. The embankment design is based on industry standard design criteria as well as various draft Design Criteria Memoranda (DCM) issued jointly by SFWMD, USACE, and FDEP as listed below in **Section A.8.3**.

This study utilized information obtained from the several geotechnical site explorations and associated laboratory materials testing programs conducted on the adjacent A-1 site, as part of the preparation of the A-1 reservoir BODR and the CEPP PIR, and data obtained from other previous soil boring programs. Detailed field exploration must be performed for the A-2 Reservoir site to understand the behavior of the *in situ* materials when excavated, placed and compacted and to assess suitability of available borrow resources.

Stability, seepage control, and erosion protection were considered, as well as potential foundation and embankment settlement. The selected embankment cross sections were developed based on the preferred conceptual cross section for the EAA A-1 Reservoir. However, the dimensions of the selected cross section were modified to accommodate a normal full storage depth of 22.6 feet corresponding to a normal full storage level (NFSL) of 31.1 feet (NAVD).

A.8.2 Conceptual Dam Embankment

A.8.2.1 Embankment Description

An embankment design has been developed to use materials from the required canal excavations and available on-site borrow resources, and to minimize sorting and processing of the excavated materials for embankment construction. A filter (inclined chimney drain) is provided for internal piping control and drainage, and to control the phreatic surface in the downstream shell. The filter gradation and its width will be designed during the basic engineering design phase of the project.

A horizontal blanket filter will be placed over the caprock to relieve seepage pressures and control loss of infilled fine-grained material from the caprock and upper silty sand foundation.

The downstream 3H:1V slope of the embankment will be covered with a layer of organic material, from the site stripping. The organic soil layer will be seeded to allow for vegetation growth and will be maintained in accordance with the SFWMD Standard Design criteria.

The geometry of the typical conceptual design cross section (named cross section K(L)) selected for use in the seepage and stability analyses for this preliminary study of the proposed A-2 Reservoir is presented on **Figure A.8.2-2**. Cross section K(L) is the typical section of the west embankment of the A-2 Reservoir. Three additional typical sections F(L), J-1(L), and L(L), which represent the A-2 Reservoir's north, south and east embankments, respectively, which have minor variations from typical section K(L), were also evaluated in the seepage and stability analyses. A map with the location of the typical cross sections that were evaluated in the seepage and stability analyses is presented on **Figure A.8.2-1**. The seepage and stability analyses discussed herein are for the NFSL Elevation of 31.10 feet (NAVD). A surcharge height of 4.5 feet above the NFSL was also analyzed. A rapid drawdown condition was also analyzed during which

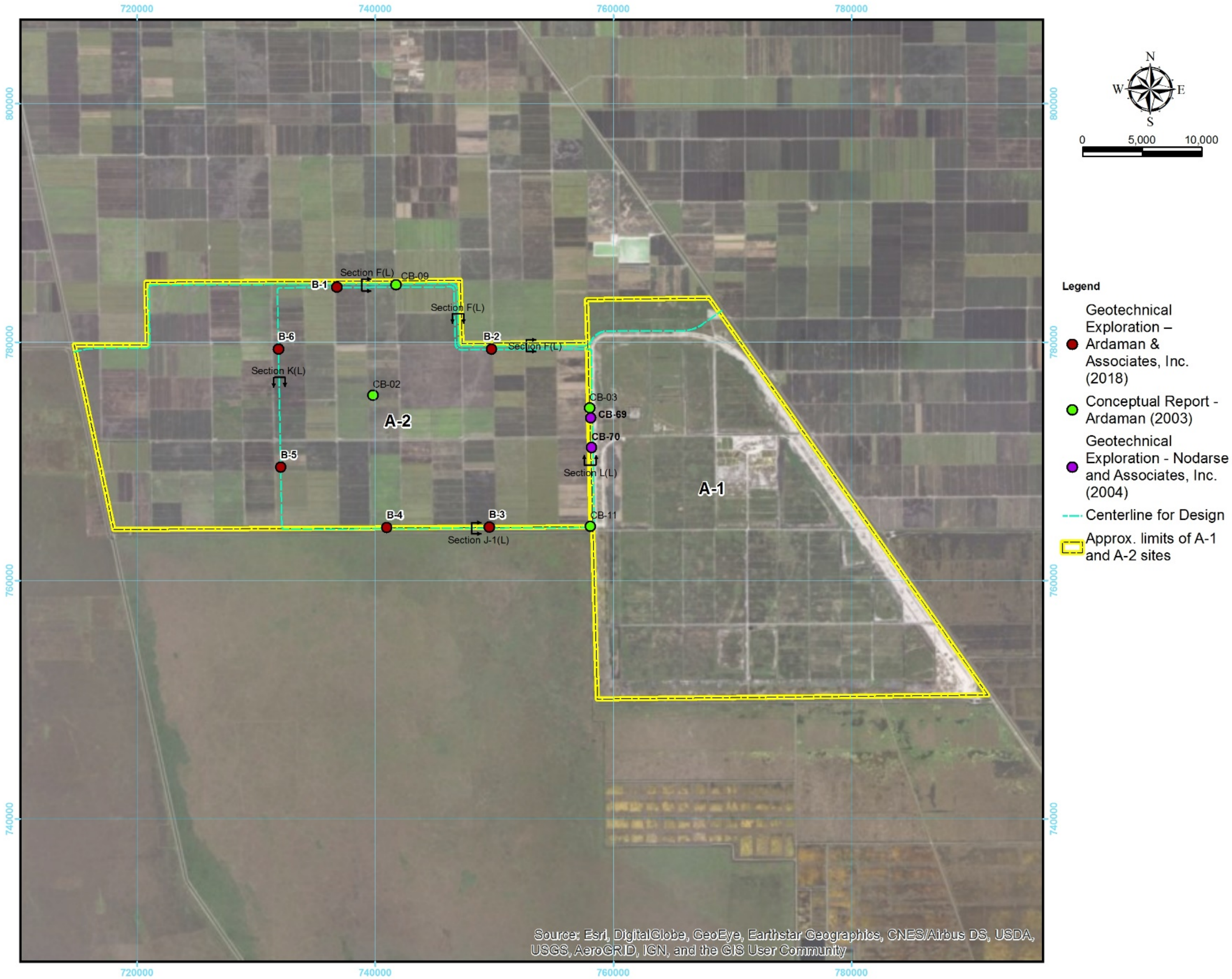

Figure A.8.2-1. Location of Typical Cross Sections

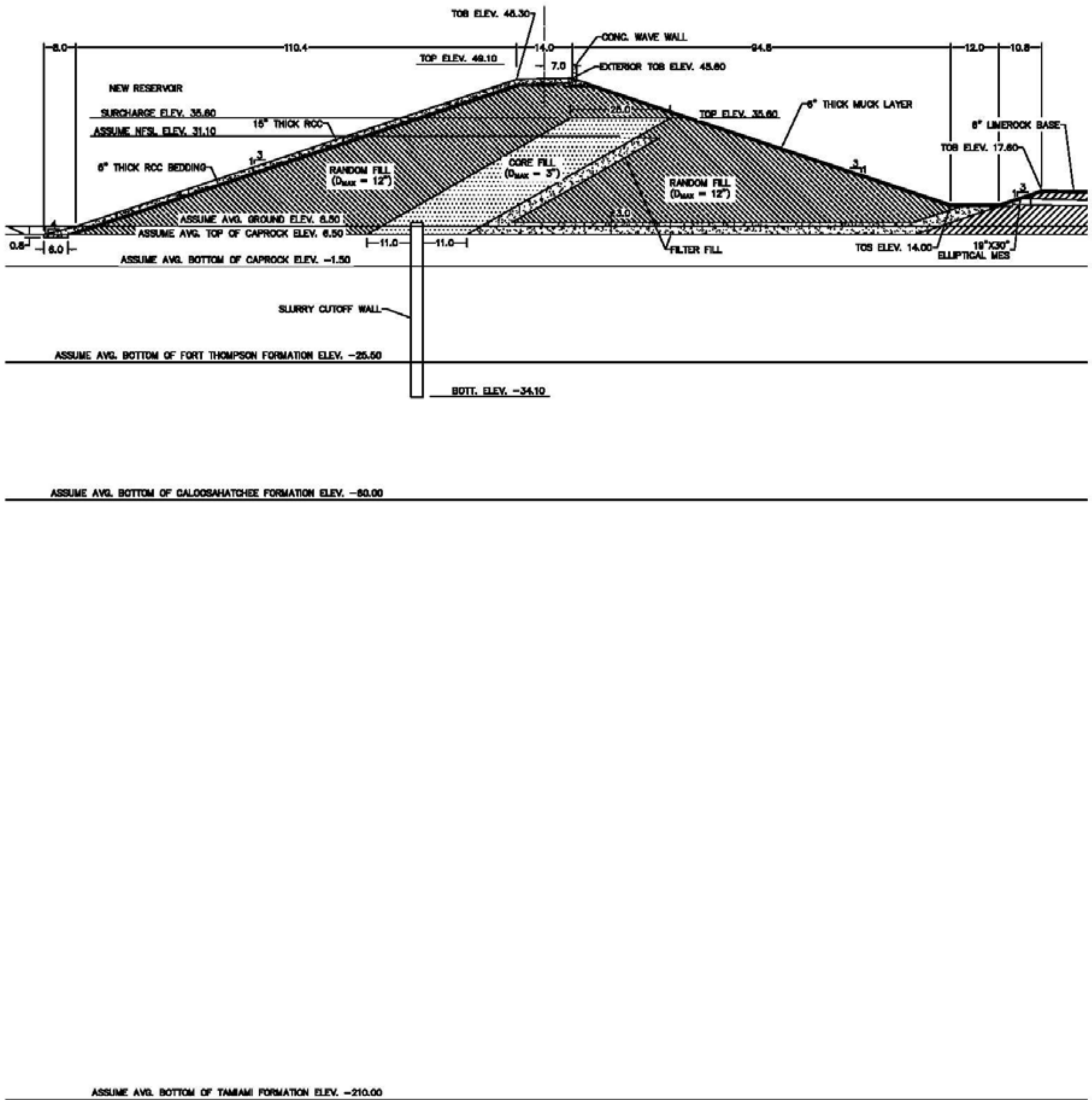

Figure A.8.2-2. Typical Dam Embankment Section - Cross Section K(L)

the water level in the reservoir is lowered to the ground surface elevation of 8.5 feet (NAVD) at a rate of 1 foot per day, i.e., the reservoir is drawn down over a period of 22.6 days and 27.1 days for the cases of an NFSL Elevation of 31.10 feet (NAVD) and surcharge pond level elevation of 35.6 feet, respectively. The seepage and stability analyses for rapid drawdown conditions were performed 10 times during the drawdown period using a uniform time increment (2.26 days and 2.71 days for the NFSL Elevation of 31.10 feet (NAVD) and surcharge pond level elevation of 35.6 feet cases, respectively. The results of seepage and stability analyses with the most critical factor safety are presented herein. The project site plan and typical cross sections K(L), F(L), J-1(L), and L(L) are presented in Annex C-1.

A.8.3 Design Criteria

A.8.3.1 Sources

United States Army Corps of Engineers (USACE) Design Manuals:

- Engineering Manual, EM 1110-2-1902, Engineering and Design: Slope Stability, 31 October 2003
- Engineering Manual, EM 1110-2-2006, Roller-Compacted Concrete, 15 January 2000
- Engineering Manual, EM 1110-2-2300, Earth and Rock-Fill Dams, General Design and Construction Considerations, 30 July 2004

Acceler8 Design Criteria Team, Design Criteria Memoranda:

- 'Hazard Potential Classification,' DCM-1, 19 August 2005
- 'Minimum Dimensions of Dams and Embankments,' DCM-4, 9 August 2005
- 'Geotechnical Seismic Evaluation of CERP Dam Foundations,' DCM-6, 16 May 2005

A.8.3.2 Embankment Slope Stability Factors of Safety

The minimum required factors of safety for each embankment design case are as follows:

Design case	Factor of safety
End of Construction	1.3
Steady Seepage at Normal Pool Level	1.5
Steady Seepage with Surcharge Pool	1.3
Steady Seepage with Earthquake Loading	1.1
Rapid Drawdown from Normal Pool	1.3
Rapid Drawdown from Surcharge Pool	1.1

A.8.3.3 Water Levels

The Maximum Hazard classification of this embankment requires that the A-2 Reservoir be sized to store the PMP as described in **Section A.5.2**. A PMP of about 4.5 feet was used as the basis for the work presented here. The total embankment height will depend on the normal water level plus the freeboard requirements. Freeboard allowance is determined from the effects of wind and rainfall and other considerations as described in **Section A.5.4**.

A.8.3.4 Seismic Loading

Pseudo-static analyses that simulate earthquake activity were performed using a gravity horizontal acceleration coefficient of 0.05 and a gravity vertical acceleration coefficient of 0.025.

Design Criteria Memorandum 6 (DCM-6) requires an evaluation of the liquefaction potential of the embankment foundations. The method of evaluation is based on assessment of continuous Standard Penetration Tests (SPT) in boreholes and comparison with standard design charts. This evaluation will be made when SPT boring data is available for the A-2 Reservoir Embankment centerline.

A.8.4 Embankment/Dam Materials

A.8.4.1 General

The economic feasibility of the EAA A-2 Reservoir depends on effective utilization of the available on-site materials during construction to the greatest extent possible. The development of the Test Cells allowed an evaluation of the suitability of on-site materials for embankment construction and erosion protection.

Materials to be used during construction will be obtained from perimeter canals and borrow areas excavated within the A-2 Reservoir interior.

Additional field exploration will be required to further define the construction materials within the A-2 Reservoir site. **Table A.8.4-1** indicates the types of construction materials that were available on the A-1 site and are expected to be available on the A-2 site.

Table A.8.4-1. Availability of Construction Materials

Embankment element	Material	Availability
Watertight barrier	In-situ soils (Fort Thompson)	On-site
	Bentonite (for a cutoff wall)	Imported
Shoulder support	In-situ soils (Caprock and Fort Thompson)	On-site
Internal drain	Caprock (crushed)	On-site
Foundation drain	Caprock (crushed)	On-site
Road stone	Caprock (crushed)	On-site
Slope Protection	Caprock	On-site
	Cement (for RCC)	Imported

A.8.4.2 Subsurface Profile

The *in situ* materials at the A-2 Reservoir site appear to be similar to those investigated beneath the adjacent EAA Reservoir A-1 site. The generalized subsurface profile is provided in **Section A.7.2**.

Core borings number CP02-EAARS-CB-0002, CP02-EAARS-CB-0003, CP02-EAARS-CB-0009, CP02-EAARS-CB-0011, CP04-EAARS-CB-00069, and CP04-EAARS-CB-0070 were used to develop a soil profile across the A-1 and A-2 sites. The location of these borings are shown in **Figure A.8.4-1**. The core boring logs used to develop the soil profile are presented in **Annex G-1**. Detailed field exploration must be performed within the A-2 site during the basic engineering design phase.

A.8.4.3 Embankment Materials

Based on the field explorations performed within the adjacent A-1 Reservoir site, the borrow material for the A-2 Reservoir and the A-2 STA will be derived from the caprock and silty sands of the Fort Thompson

Formation. The main source of borrow will be the perimeter canals with additional material being provided by borrow areas within the reservoir. The caprock will provide the source for large and small aggregate for RCC, drainage aggregates, and gravel surfacing. The silty sands of the Fort Thompson Formation would be the source of random fill for construction of the earthen embankment.

Laboratory testing for the A-1 Reservoir as well as the seepage tests performed in the Test Cells for Reservoir A-1 indicate that compaction of silty sand (-200 = 20%) from the Fort Thompson Formation should result in a horizontal hydraulic conductivity of approximately 1×10^{-4} cm/sec.

The upstream and downstream shells (shoulders) of the embankment will be constructed of silty sand fill from the Fort Thompson Formation. Rock fragments will be limited to no more than 12 inches in maximum dimension prior to compaction. Careful placement and compaction of the silty sand fill in lifts not exceeding 12 inches thick should result in a relatively dense, low permeability zone on either side of the central core.

The central core of the embankment will consist of silty sand from the Fort Thompson Formation that is processed (raked or screened) to eliminate rock fragments greater than 3 inches in maximum dimension.

A graded filter constructed of processed caprock meeting the filter requirements for the silty sand of the Fort Thompson Formation will be placed between the central core and the downstream shell and between the downstream shell and the caprock foundation. A graded filter will also be installed below the RCC slope protection to allow seepage from the upstream shell during drawdown of the reservoir to drain through weep holes in the RCC.

Additional field exploration within the A-2 site will be required to further define the borrow materials.

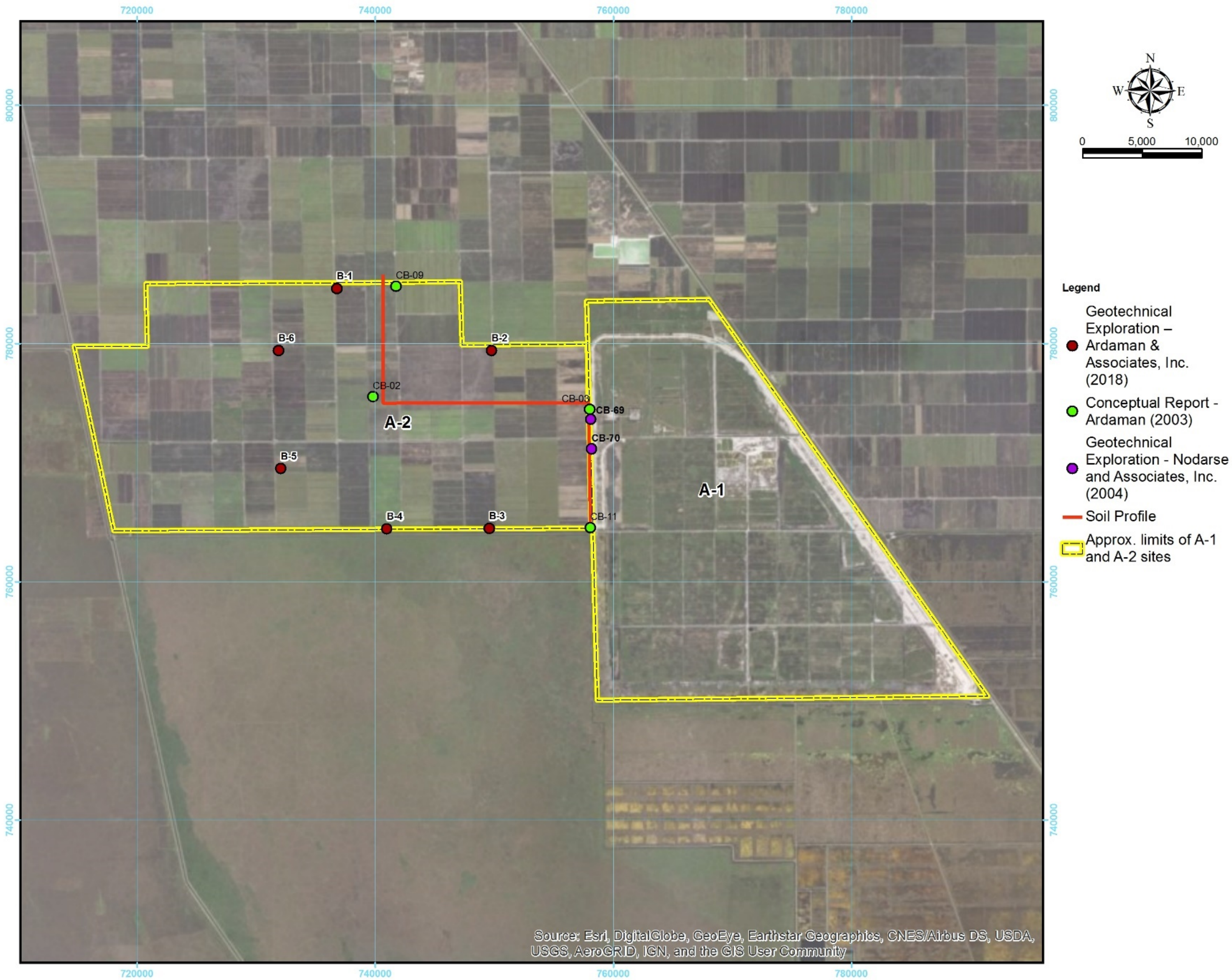

Figure A.8.4-1. Location of Soil Profile across A-1 and A-2 Sites

A.8.5 Seepage Control

A.8.5.1 General

Seepage control has two principal design functions:

- The first function is embankment and foundation stability: pore pressures and hydraulic gradients must be controlled to protect the embankment and foundation from internal erosion (piping) and to ensure stability
- The second function is to mitigate off-site impacts due to increased seepage

This section describes the minimum measures required to ensure stability. Seepage computer modeling has been performed to evaluate seepage control.

A.8.5.2 Seepage Model Parameters

The seepage analyses were performed using the engineering properties presented in **Table A.8.5-1**. These engineering properties were selected for the conceptual design cross sections of the A-2 Reservoir based on experience with similar soils on prior projects, evaluation of the test borings performed at the EAA A-1 Reservoir site, evaluation of the two boreholes performed at the A-2 Reservoir site, and a review of the parameters used for the design and analysis of the adjacent EAA A-1 Reservoir embankment. The engineering properties for use in the final design cross sections of the A-2 Reservoir will be selected after the extensive field and laboratory testing program described above is completed.

Table A.8.5-1. Seepage and Stability Analysis Parameters

Material Type	γ_{sat} (pcf[1])	ϕ (degrees)	k_v (cm/s)	k_h (cm/s)	Anisotropy
Muck (c=20psf)	70	0	1.0×10^{-4}	1.0×10^{-4}	1
Random Fill (D_{max} < 6 inch)	130	35	5.0×10^{-5}	2.0×10^{-4}	4
Random Fill (D_{max} < 12 inch)	130	35	5.0×10^{-5}	2.0×10^{-4}	4
Filter Fill	130	35	$>5 \times 10^{-3}$	$>1.0 \times 10^{-2}$	2
Core Fill (D_{max} < 3 inch)	125	35	2.5×10^{-5}	1.0×10^{-4}	4
Cutoff Wall	125	25	1.0×10^{-6}	1.0×10^{-6}	1
Roller Compacted Concrete	150	35	NA	NA	NA
Caprock	140	40	3.5×10^{-4}	3.5×10^{-2}	100
Ft. Thompson	125	33	3.5×10^{-3}	1.8×10^{-1}	50
Caloosahatchee	125	35	2.8×10^{-3}	1.4×10^{-1}	50
Tamiami	130	35	6.0×10^{-3}	1.2×10^{-2}	2

[1] Pounds per cubic foot

A.8.5.3 Embankment Seepage

Seepage through the embankment and foundation under steady-state condition was modeled using the computer program SEEP/W. SEEP/W, developed by Geo-Slope International Ltd. of Calgary, Alberta, Canada, is a two-dimensional finite element seepage modeling program that generates the phreatic surface, hydraulic head distribution, and flow quantities within a seepage domain.

Seepage analyses were performed for a NFSL water depth of 22.6 feet, which corresponds to a with a maximum design water level of 31.10 feet (NAVD). The water level inside the reservoir was represented with a fixed head boundary of 31.10 feet applied at the ground surface, inside slope face and vertical east, south, north, and west ends of cross sections K(L), F(L), J-1(L), and L(L), respectively. The approximate water level maintained in the farmland ditches was represented with a fixed head boundary of 6.1 feet on the vertical north and south ends of cross sections F(L) and J-1(L), respectively. All the cross sections were extended 1,000 feet on the reservoir side and 2,000 feet outside the reservoir. The water levels in the existing canals were also represented with fixed head boundaries. Results of the seepage analyses were obtained in the form of total head and velocity distributions within the embankment and foundation soils, and flow rates through the embankment and foundation. Results of the seepage analyses are presented in **Figures A.8.5-1** through **A.8.5-12**. The exit gradients as well as the computed factors of safety against soil heave and piping, for each case are presented in **Table A.8.5-2**. The computed factors of safety for Cross Sections K(L), F(L) and J-1(L) meet or exceed the minimum required factor of safety of 3.0. Additional seepage management measures will be evaluated for Cross Section L(L) during the Planning Engineering Design (PED) phase to help reduce the exit gradients.

The exit hydraulic gradient into the canal along the outside of the A-2 Reservoir dam embankment based on the seepage analyses is presented in **Figures A.8.5-1, A.8.5-2, A.8.5-4, A.8.5-5, A.8.5-7, and A.8.5-8**. The seepage rate from the A-2 Reservoir into the perimeter canals for cross sections K(L), F(L) and J-1(L) as well as the seepage rate from the A-2 Reservoir into the A-1 FEB for cross-section L (L) computed from the SEEP/W models is presented in **Table A.8.5-3**.

A cut-off wall was considered in the conceptual design cross sections of the A-2 embankment to force the seepage to pass vertically downward through the caprock and the Fort Thompson formation into the Caloosahatchee formation due to piping potential within the caprock layer. The cut-off wall will be located beneath the center of the core fill extending three feet above the top of the caprock layer with a bottom elevation of -34.10 feet (NAVD), as shown on **Figure A.8.2-1**. The foundation cut-off can be installed below the groundwater level by using the slurry method of trench excavation, during which the trench can remain opened by using a mixture of water and bentonite. The backfill of the cut-off wall will consist of a mixture of the excavated trench soils and processed commercial bentonite. Other types of backfill options include soil-cement-bentonite, cement-bentonite and plastic concrete.

Table A.8.5-2. Factors of Safety against Soil Heave/Piping

Case	Steady Seepage with Normal Pool				Steady Seepage with Surcharge Pool			
Cross Section	**K(L)**	**F(L)**	**J-1(L)**	**L(L)**	**K(L)**	**F(L)**	**J-1(L)**	**L(L)**
Exit Gradient	0.25	0.19	0.18	1.31	0.29	0.21	0.20	1.56
Critical Gradient, γ_w/γ'	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Factor of Safety	4.00	5.26	5.56	0.76	3.45	4.76	5.00	0.64

Note: $\gamma' = \gamma_{sat} - \gamma_w$

Table A.8.5-3. Computed Seepage from the Reservoir

Case / Cross Section	Upstream Elevation (NAVD)	Downstream Elevation (NAVD)	Seepage (cfs/mile)
K(L)	31.1	6.5	33.1
F(L)	31.1	4.5	40.0
J-1(L)	31.1	6.5	38.0
L(L)	31.1	6.5	19.7

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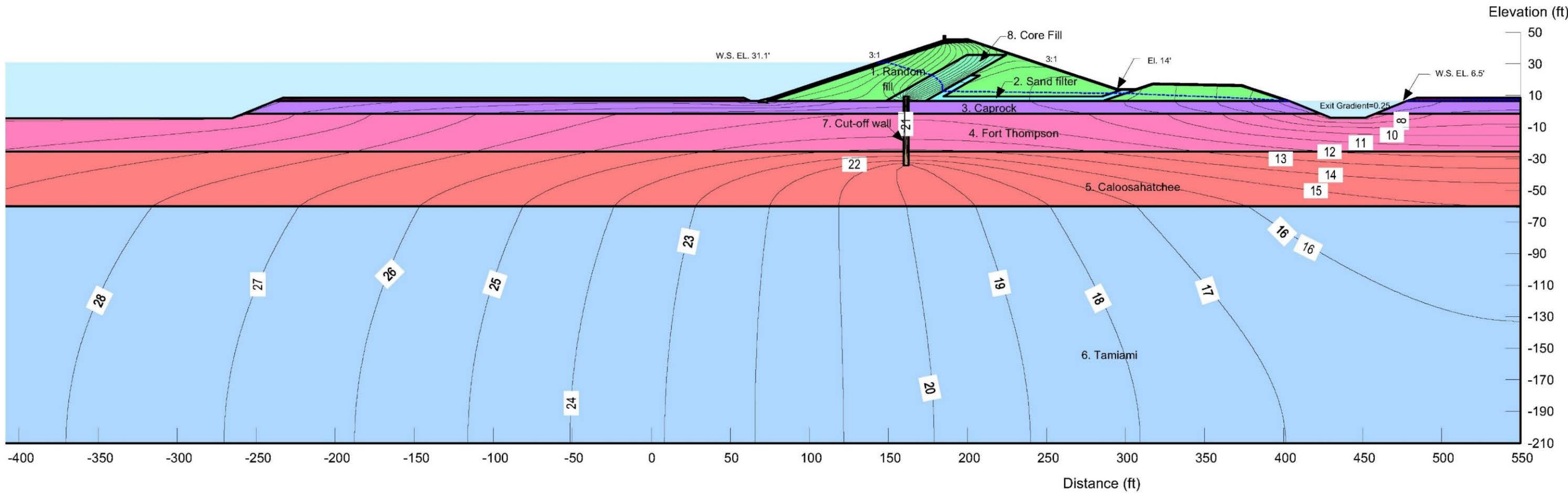

Figure A.8.5-1. Steady State Seepage with Normal Pool - Cross Section K(L)

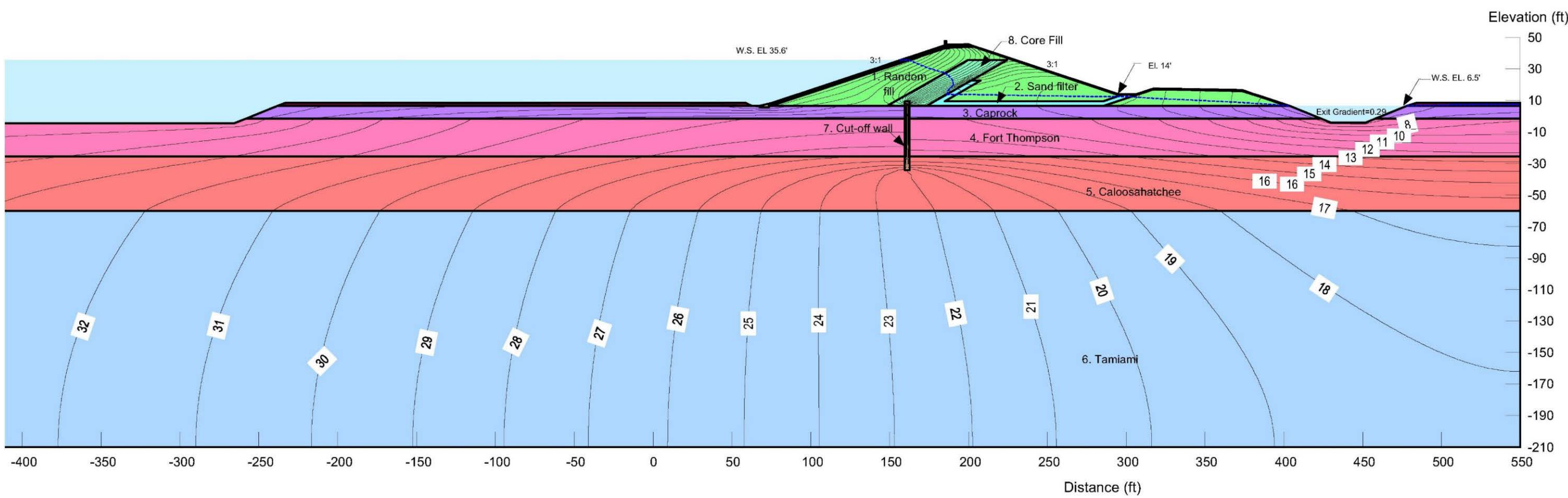

Figure A.8.5-2. Steady Seepage with Water Level at Surcharge Pool - Cross Section K(L)

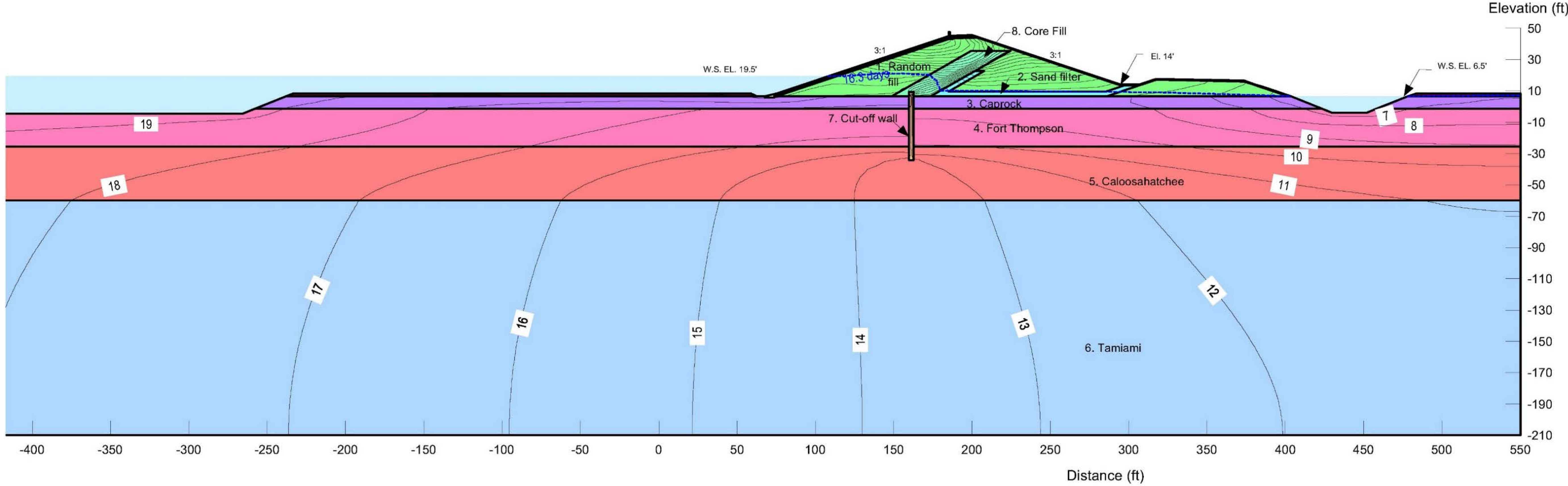

Figure A.8.5-3. Critical Section for Rapid Drawdown - Cross Section K(L)

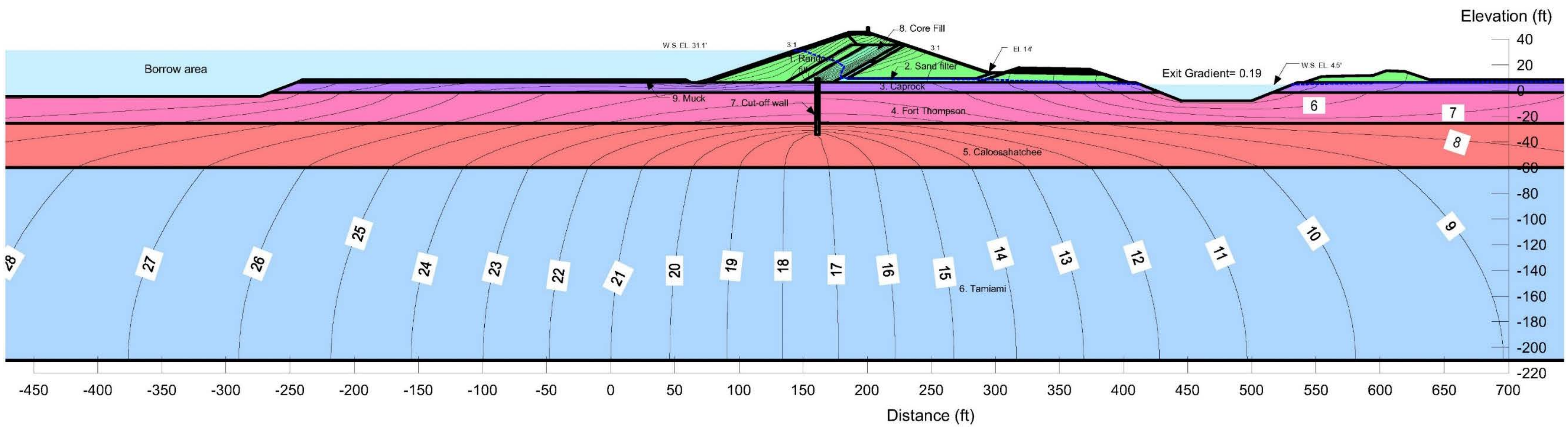

Figure A.8.5-4. Steady State Seepage with Normal Pool - Cross Section F(L)

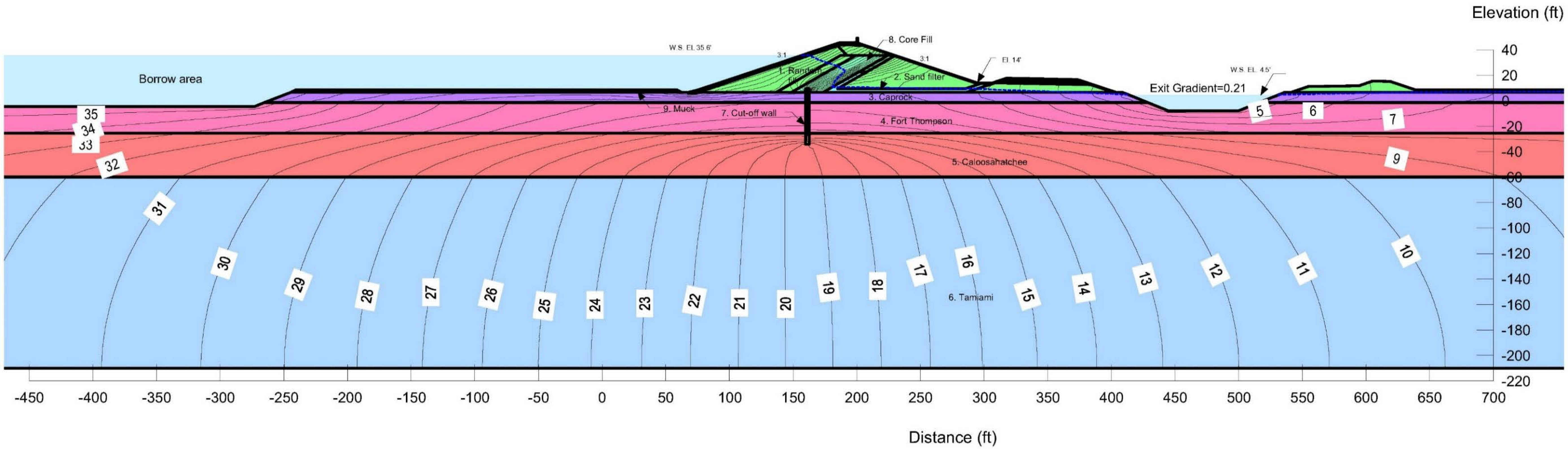

Figure A.8.5-5. Steady Seepage with Water Level at Surcharge Pool - Cross Section F(L)

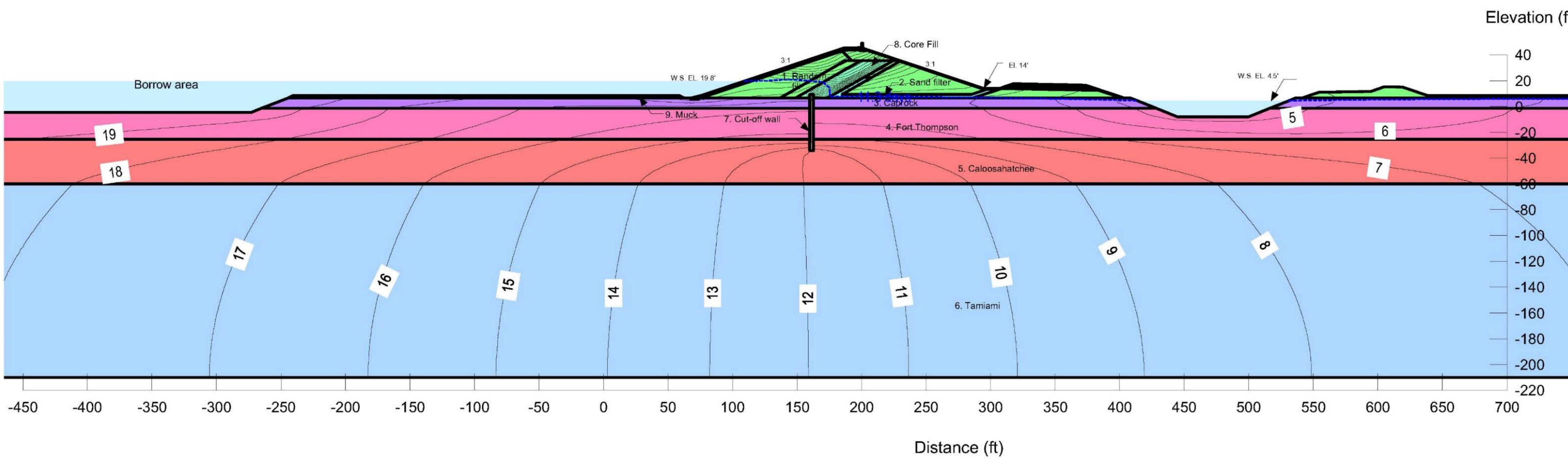

Figure A.8.5-6. Critical Section for Rapid Drawdown - Cross Section F(L)

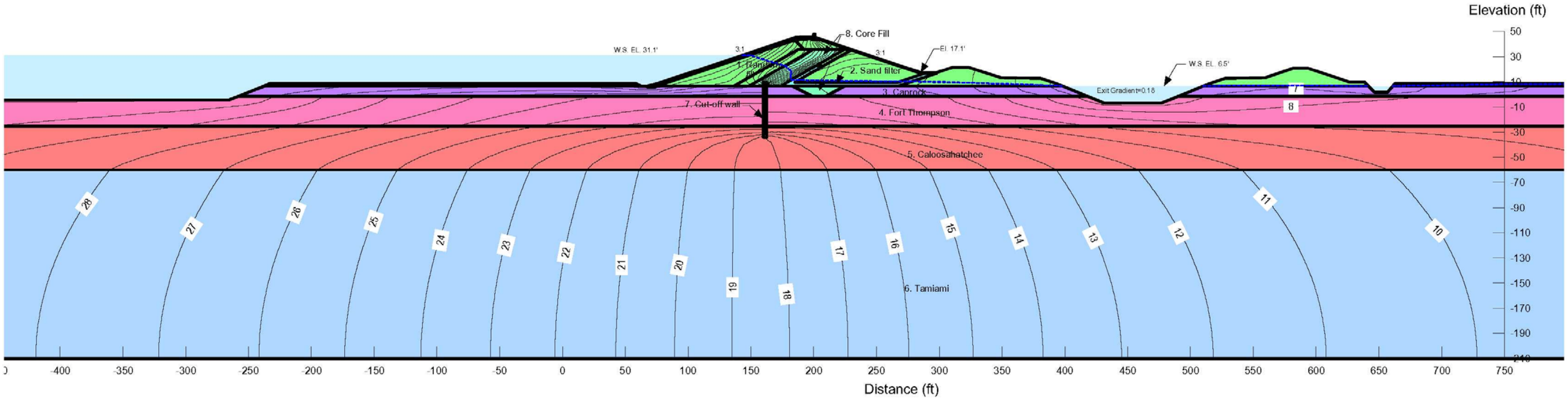

Figure A.8.5-7. Steady State Seepage with Normal Pool - Cross Section J-1(L)

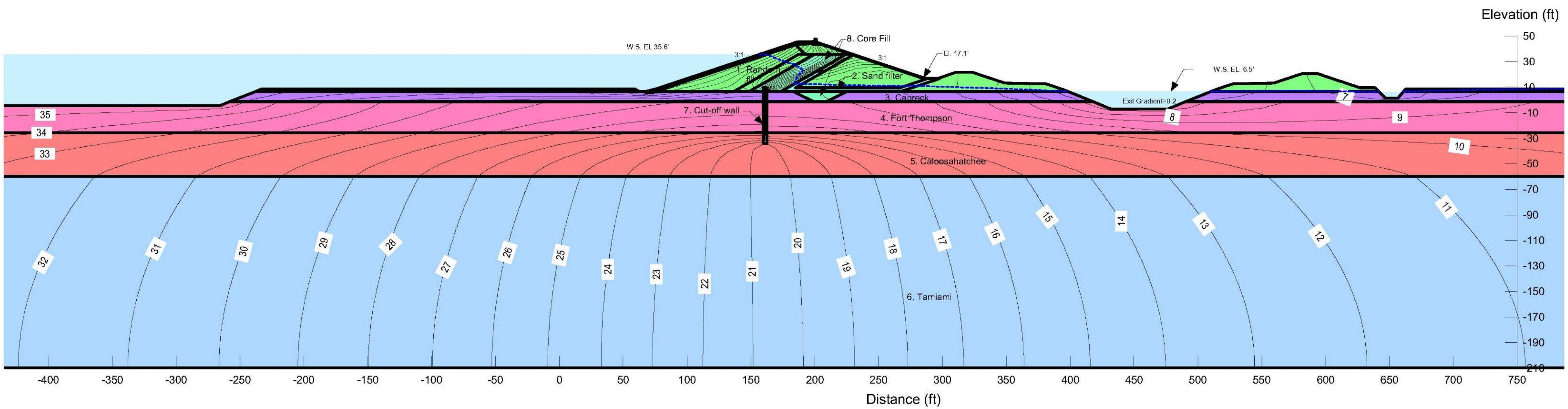

Figure A.8.5-8. Steady Seepage with Water Level at Surcharge Pool - Cross Section J-1(L)

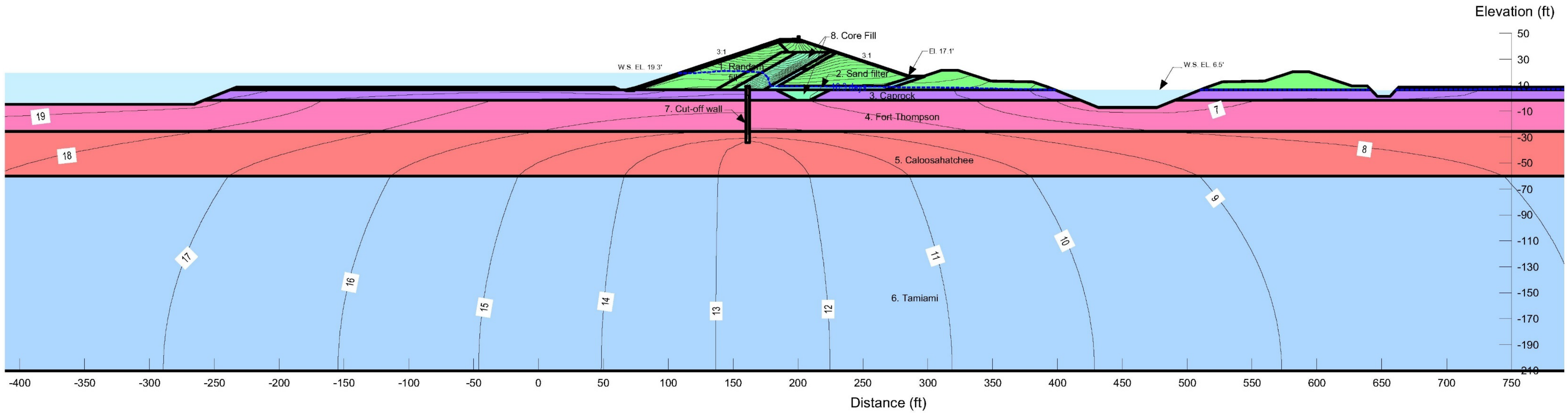

Figure A.8.5-9. Critical Section for Rapid Drawdown - Cross Section J-1(L)

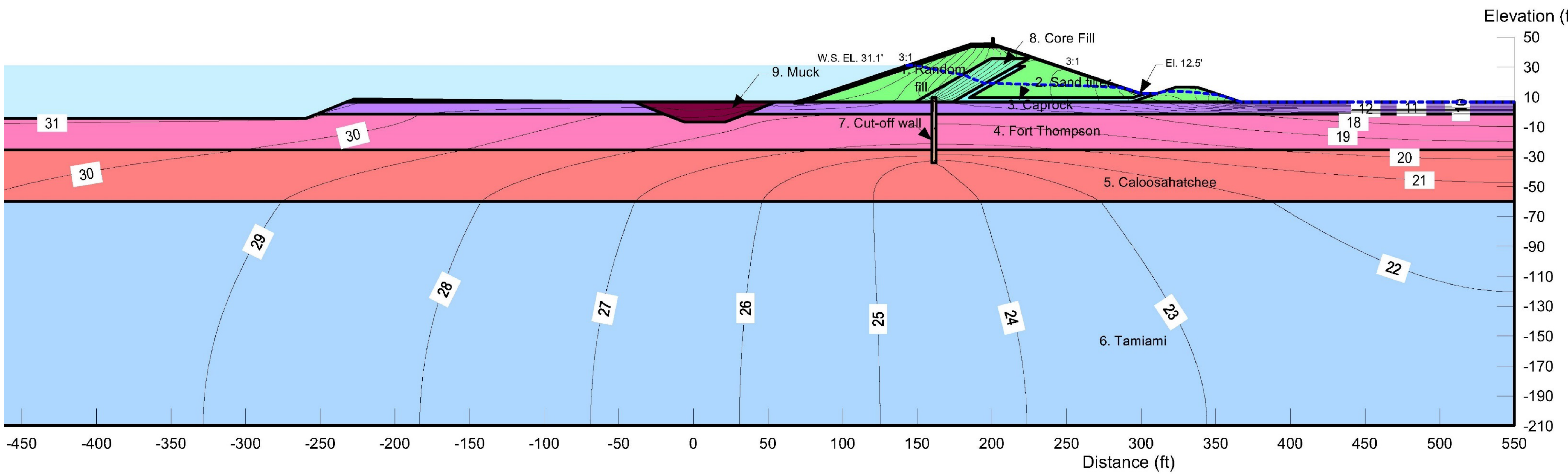

Figure A.8.5-10. Steady State Seepage with Normal Pool - Cross Section L(L)

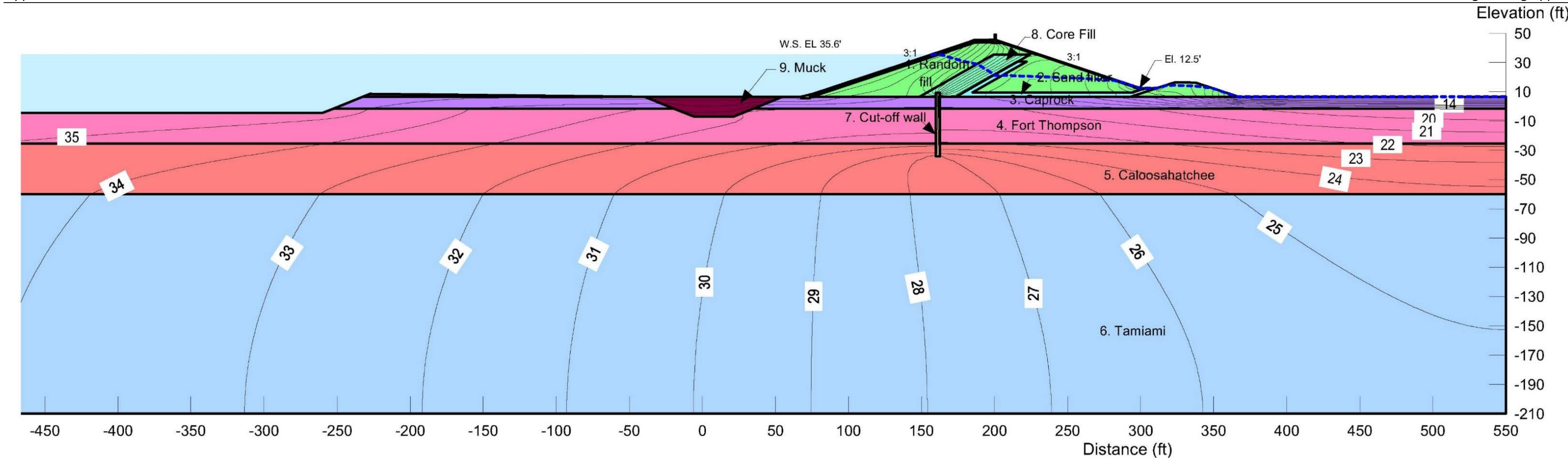

Figure A.8.5-11. Steady Seepage with Water Level at Surcharge Pool - Cross Section L(L)

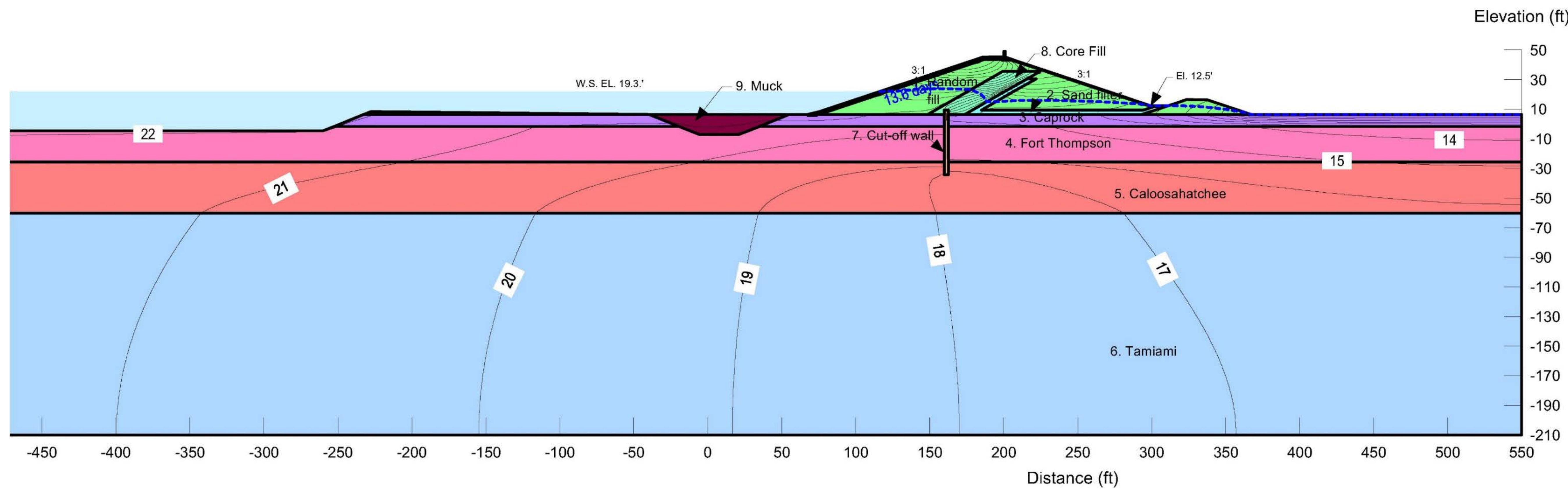

Figure A.8.5-12. Critical Section for Rapid Drawdown - Cross Section L(L)

A.8.6 Stability

A.8.6.1 General

Stability of the proposed A-2 Reservoir embankment was evaluated for an embankment height of 37.1 feet above the average elevation of the existing ground surface. The stability analyses were performed using the pore pressure distributions determined from the results of the seepage analyses presented in **Figures A.8.5-1** through **A.8.5-12**.

A.8.6.2 Material Parameters

The stability analyses were performed using the pore pressure distributions obtained from the respective seepage analyses, along with the shear strength and unit weight parameters presented in **Table A.8.5-1**. These engineering properties were selected for the conceptual design cross sections of the A-2 reservoir based on experience with similar soils on prior projects, evaluation of the test borings performed at the A-1 Reservoir site, evaluation of the two boreholes performed at the A-2 Reservoir site, and a review of the parameters used for the design and analysis of the adjacent EAA A-1 Reservoir embankment. The engineering properties for use in the final design cross sections of the A-2 Reservoir will be selected after the extensive field and laboratory testing program described above is completed.

A.8.6.3 Embankment Slope Stability

The stability analyses for the proposed A-2 Reservoir were performed using the computer model SLOPE/W. SLOPE/W, developed by Geo-Slope International Ltd. Of Calgary, Alberta, Canada, is a fully integrated slope stability analysis program. The computer program determines the critical failure surface for each failure mode by converging on the failure surface through an iterative procedure. Stability analyses on the most critical failure surfaces identified in the search routine were completed using Spencer's method, which satisfies total force and moment equilibrium, considering static and pseudo-static conditions that simulate seismic accelerations of the region.

Pseudo-static analyses that simulate earthquake activity were performed using a gravity horizontal acceleration coefficient of 0.05 and a gravity vertical acceleration coefficient of 0.025.

The results of the stability analyses and the parameters used in the analyses are presented in **Figures A.8.6-1** through **A.8.6-12**. The required factor of safety and the computed factors of safety, for static conditions and seismic conditions, for each case are presented in **Tables A.8.6-1** through **A.8.6-4**. As noted, the computed factors of safety, in all cases, meet or exceed the minimum required factors of safety.

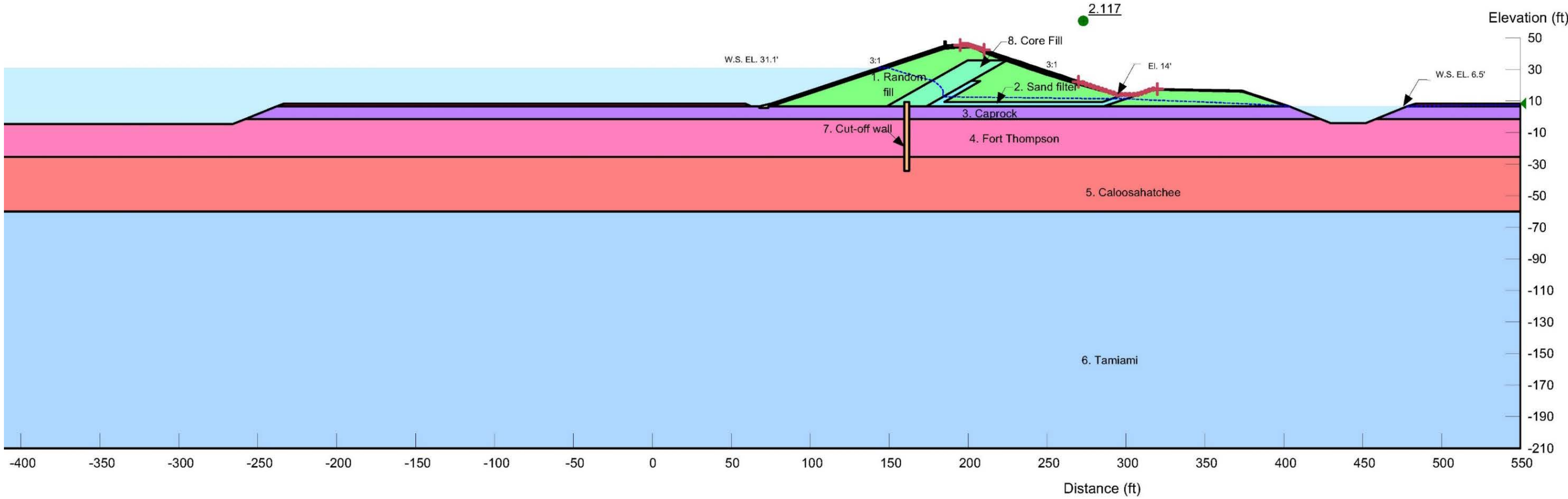

Figure A.8.6-1. Results of Stability Analysis for Steady Seepage - Cross Section K(L)

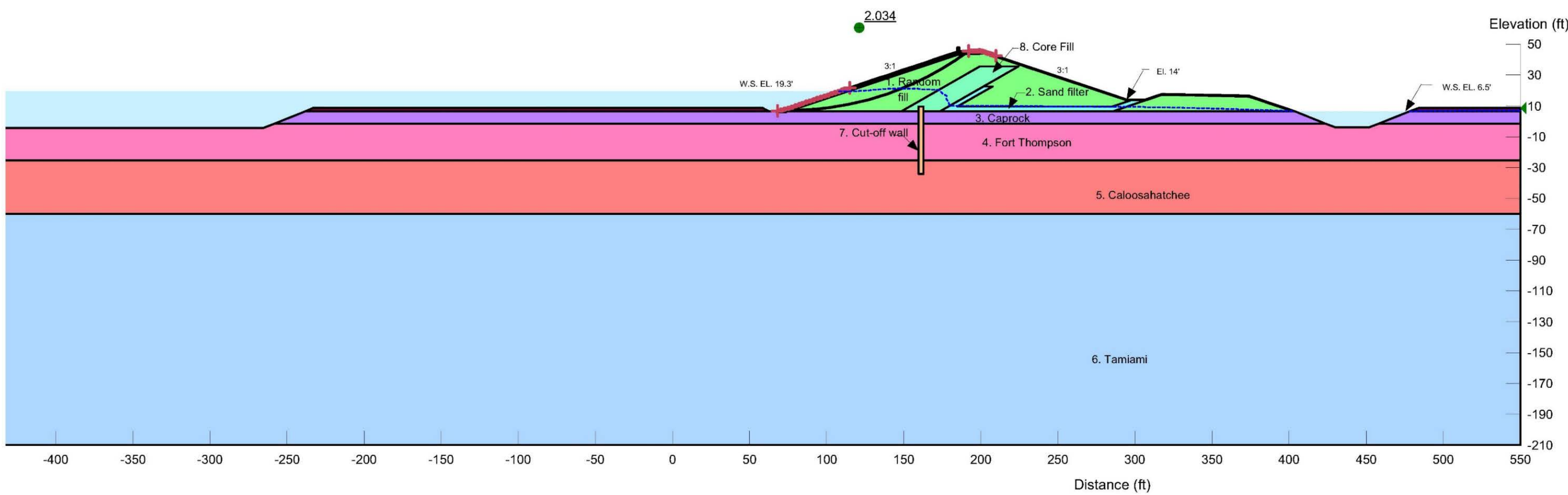

Figure A.8.6-2. Results of Stability Analysis for Rapid Drawdown - Cross Section K(L)

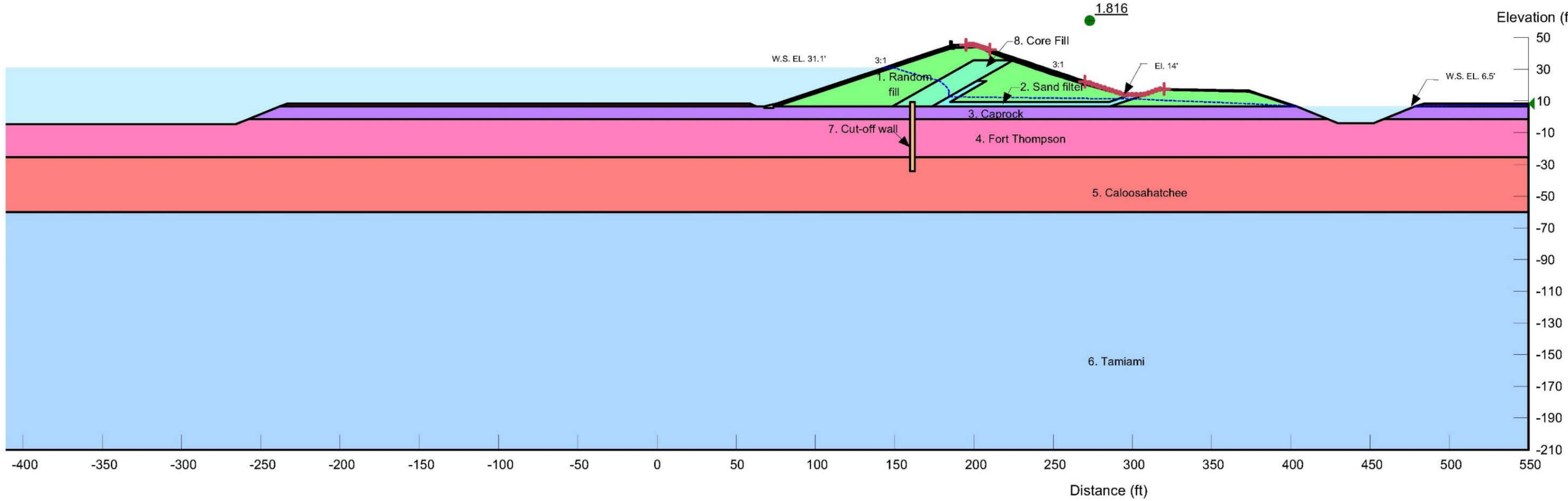

Figure A.8.6-3. Results of Stability Analysis for Earthquake Loading - Cross Section K(L)

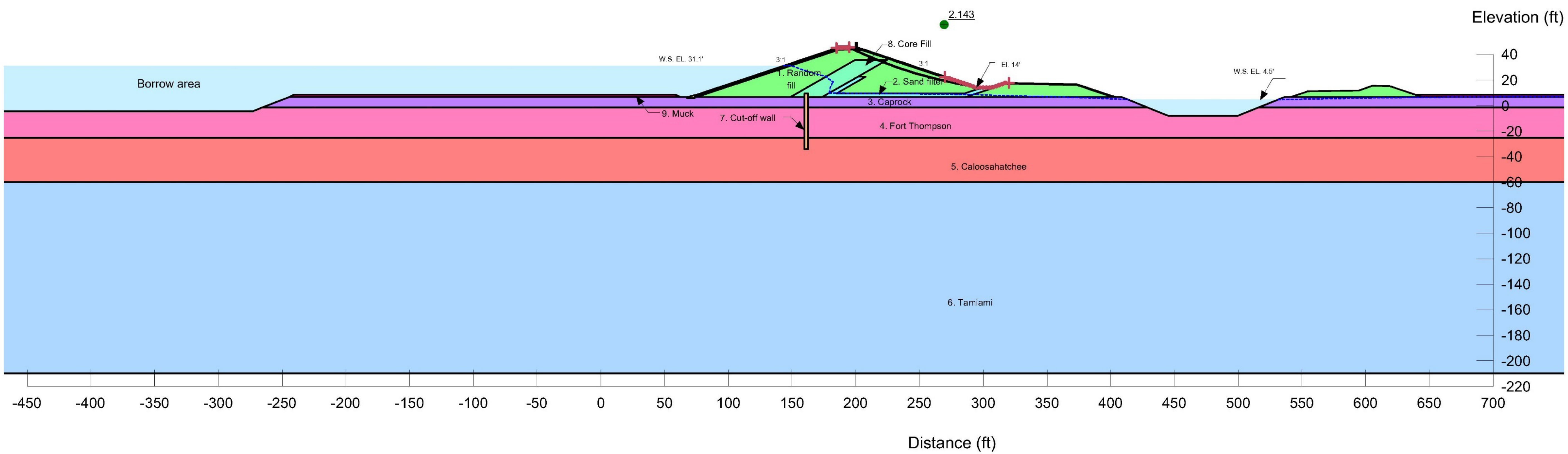

Figure A.8.6-4. Results of Stability Analysis for Steady Seepage - Cross Section F(L)

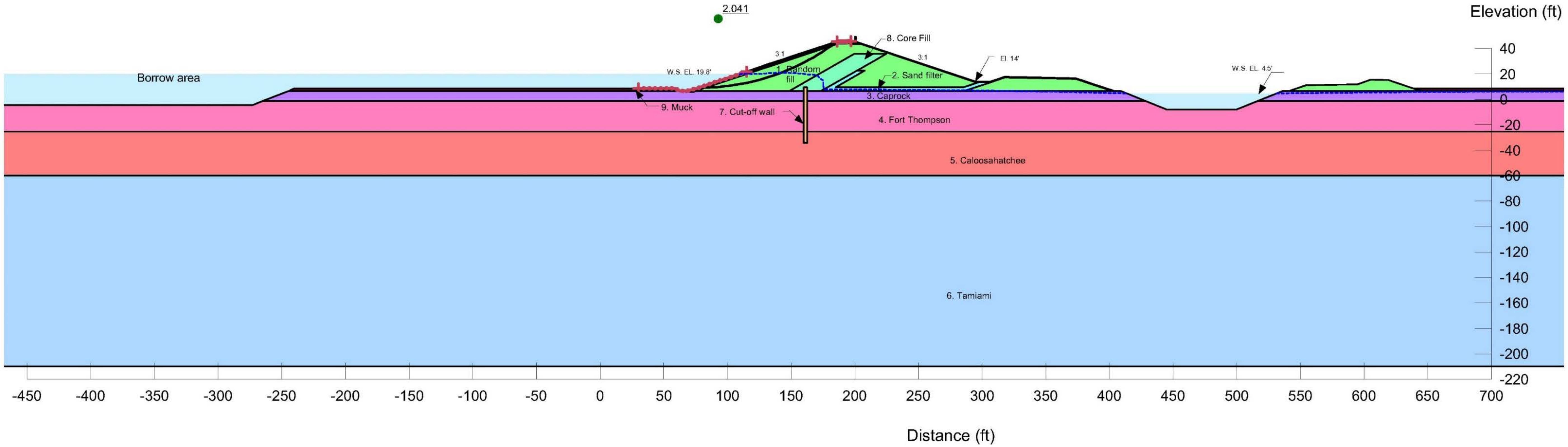

Figure A.8.6-5. Results of Stability Analysis for Rapid Drawdown - Cross Section F(L)

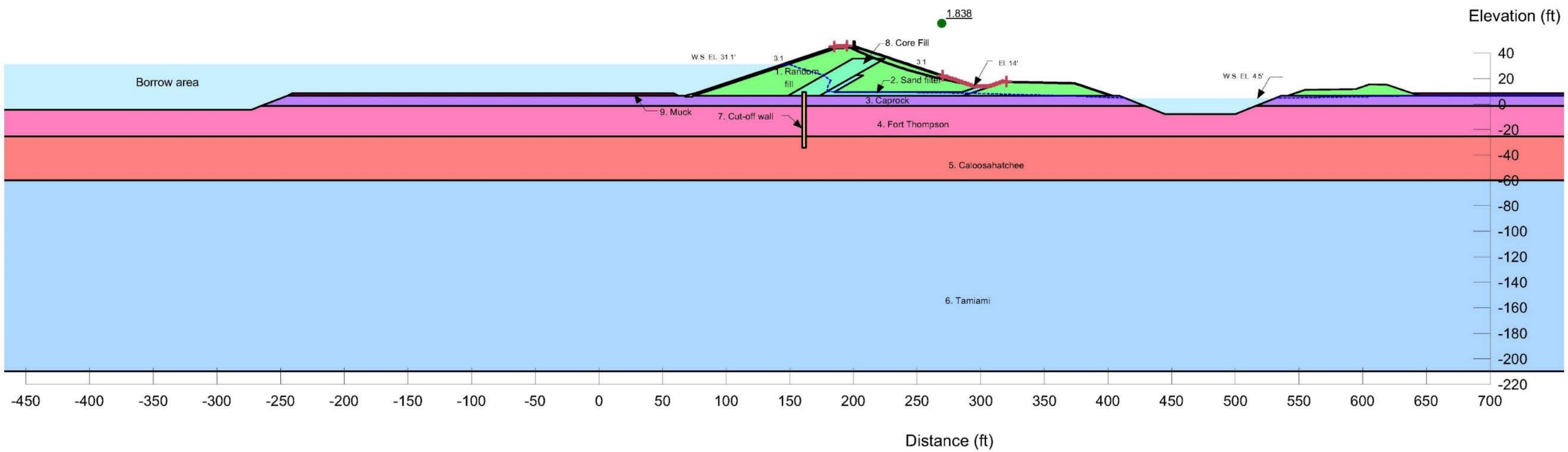

Figure A.8.6-6. Results of Stability Analysis for Earthquake Loading - Cross Section F(L)

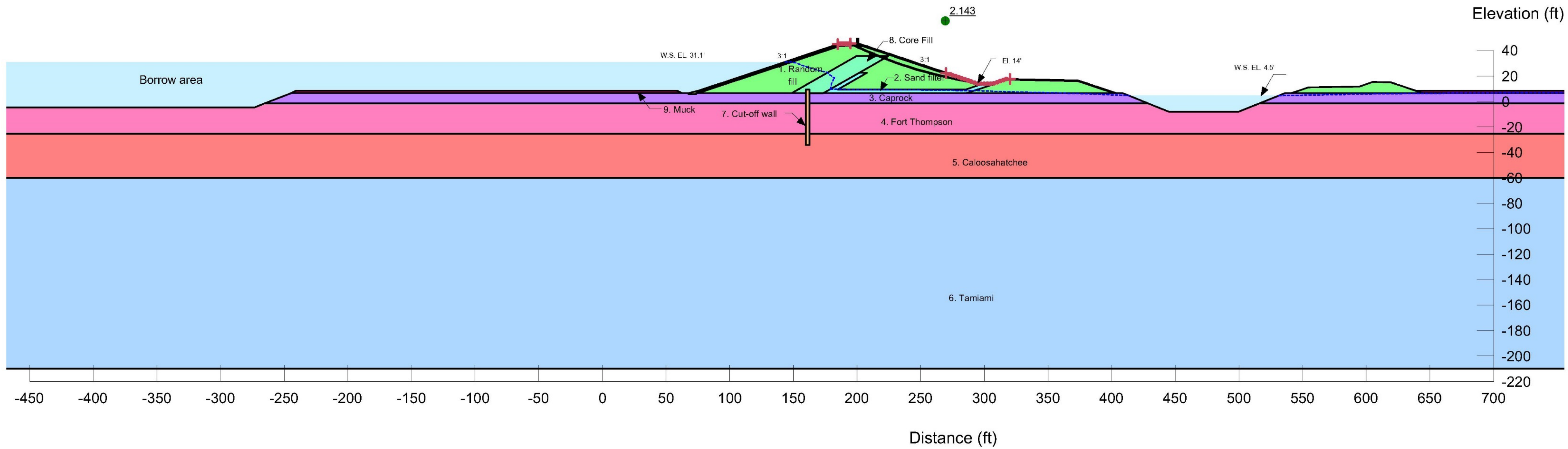

Figure A.8.6-7. Results of Stability Analysis for Steady Seepage - Cross Section J-1(L)

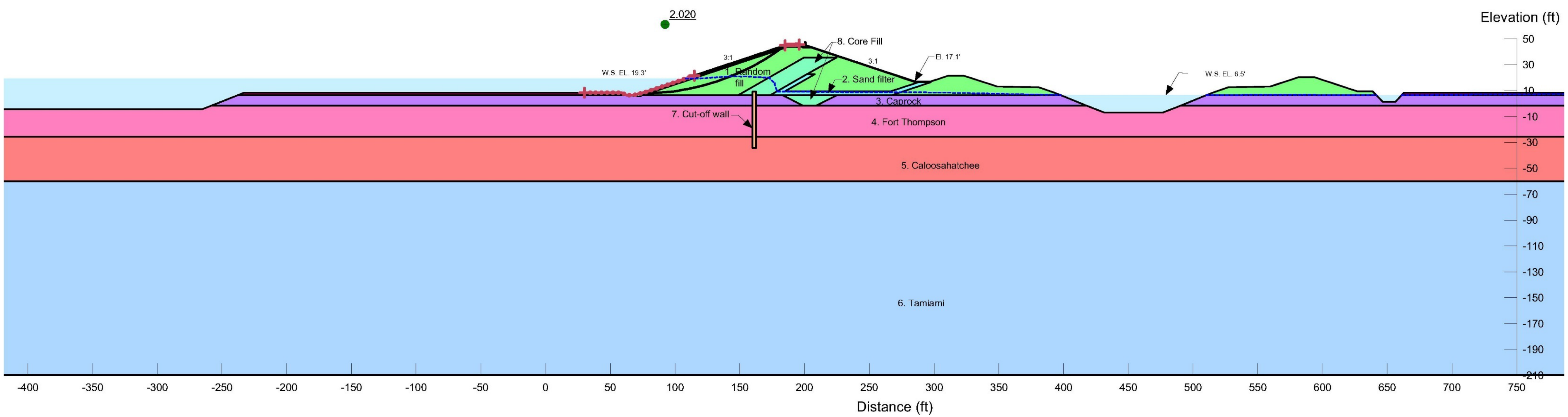

Figure A.8.6-8. Results of Stability Analysis for Rapid Drawdown - Cross Section J-1(L)

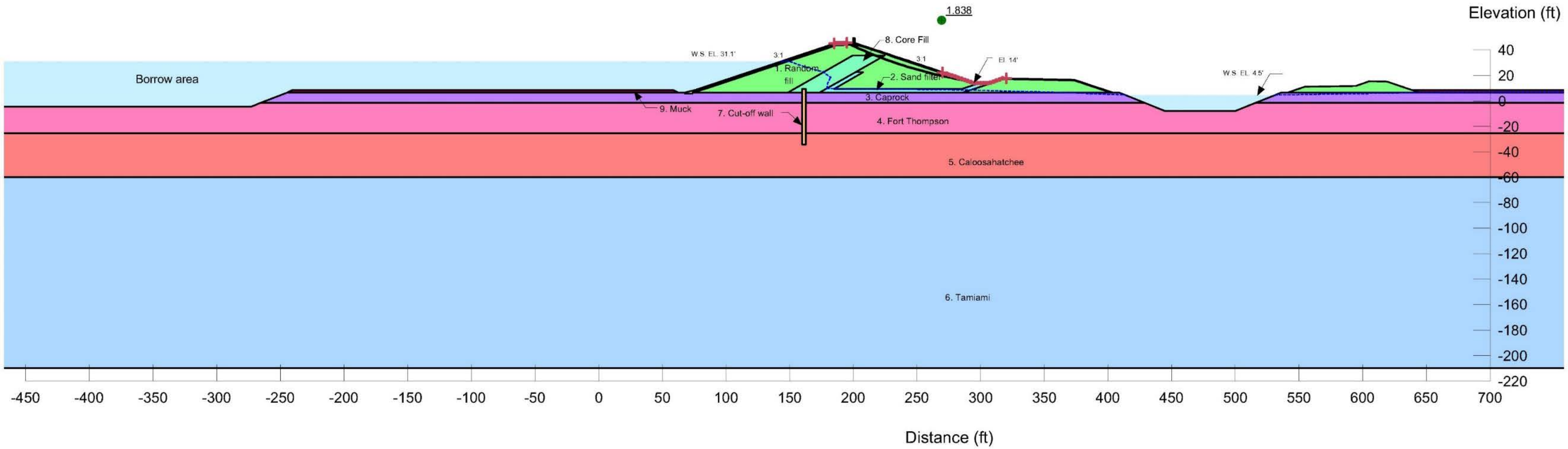

Figure A.8.6-9. Results of Stability Analysis for Earthquake Loading - Cross Section J-1(L)

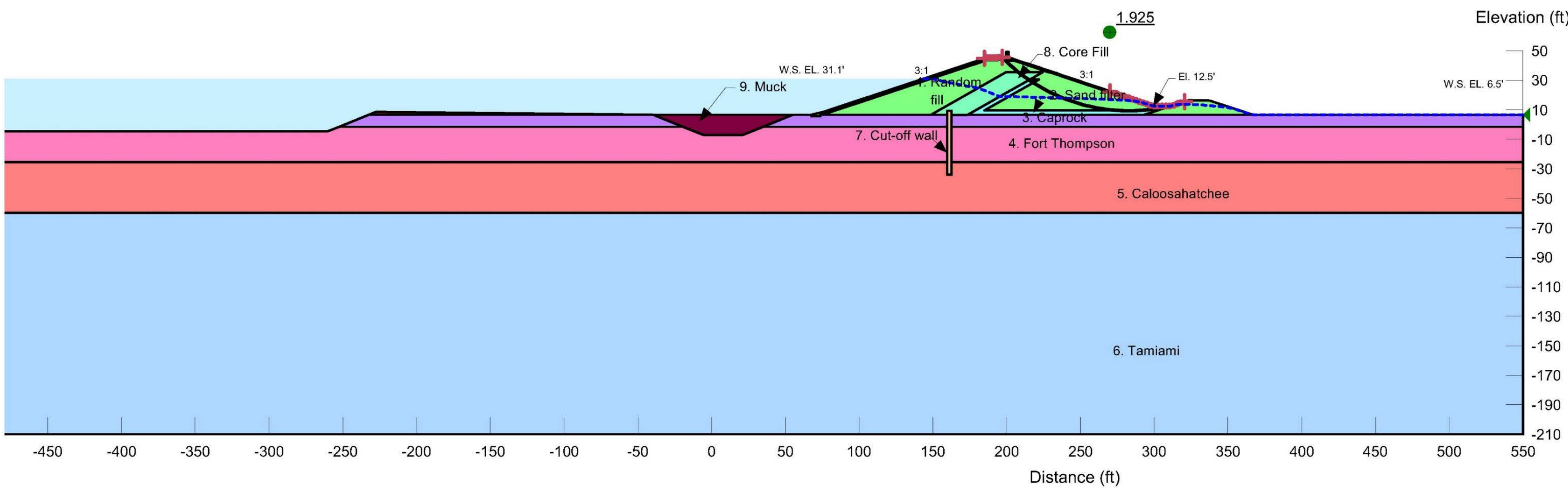

Figure A.8.6-10. Results of Stability Analysis for Steady Seepage - Cross Section L(L)

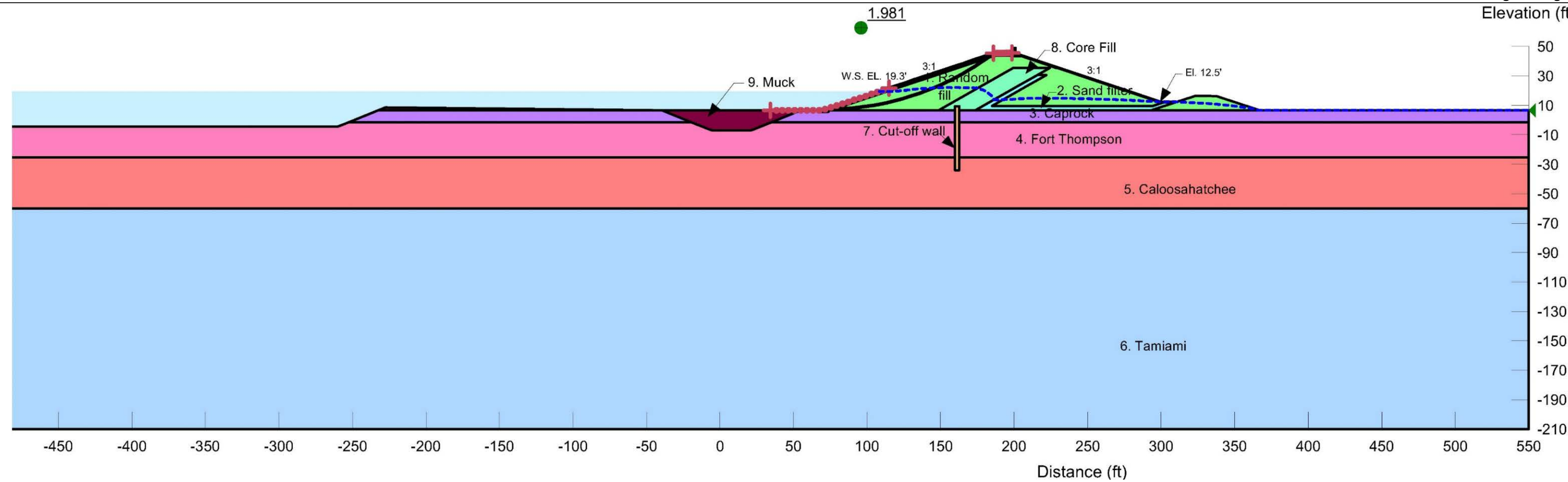

Figure A.8.6-11. Results of Stability Analysis for Rapid Drawdown - Cross Section L(L)

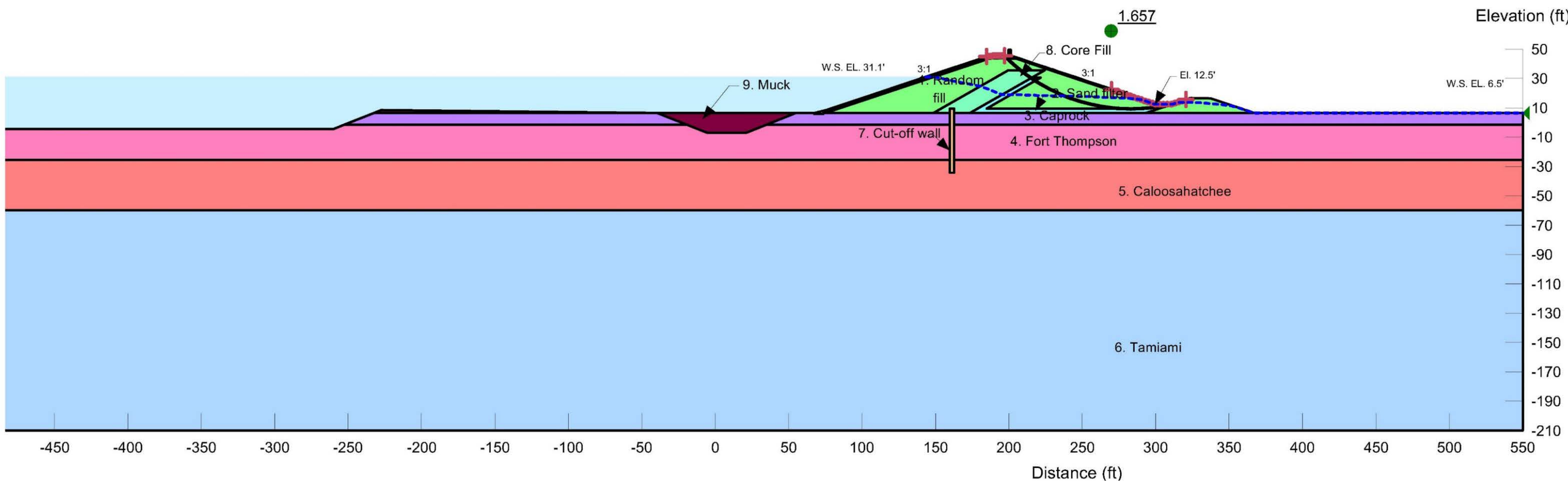

Figure A.8.6-12. Results of Stability Analysis for Earthquake Loading - Cross Section L(L)

Table A.8.6-1. Results of Stability Analysis for Cross Section K(L)

Case	Strength Parameters	Minimum Required Factors of Safety	Calculated Factor of Safety	
			Upstream Slope	Downstream Slope
End of Construction	Total	1.3	2.16	2.12
Steady Seepage with Normal Pool	Effective	1.5	2.16	2.12
Steady Seepage with Surcharge Pool	Effective	1.3	2.21	2.12
Rapid Drawdown from Normal Pool	Effective	1.3	2.06	NA
Rapid Drawdown from Surcharge Pool	Effective	1.1	2.03	NA
Steady Seepage with Earthquake Loading	Effective	1.1	1.78	1.82

Table A.8.6-2. Results of Stability Analysis for Cross Section F(L)

Case	Strength Parameters	Minimum Required Factors of Safety	Calculated Factor of Safety	
			Upstream Slope	Downstream Slope
End of Construction	Total	1.3	2.10	2.13
Steady Seepage with Normal Pool	Effective	1.5	2.06	2.14
Steady Seepage with Surcharge Pool	Effective	1.3	2.07	2.16
Rapid Drawdown from Normal Pool	Effective	1.3	2.04	NA
Rapid Drawdown from Surcharge Pool	Effective	1.1	2.07	NA
Steady Seepage with Earthquake Loading	Effective	1.1	1.71	1.84

Table A.8.6-3. Results of Stability Analysis for Cross Section J-1(L)

Case	Strength Parameters	Minimum Required Factors of Safety	Calculated Factor of Safety	
			Upstream Slope	Downstream Slope
End of Construction	Total	1.3	2.10	2.14
Steady Seepage with Normal Pool	Effective	1.5	2.06	2.14
Steady Seepage with Surcharge Pool	Effective	1.3	2.07	2.14
Rapid Drawdown from Normal Pool	Effective	1.3	2.02	NA
Rapid Drawdown from Surcharge Pool	Effective	1.1	2.02	NA
Steady Seepage with Earthquake Loading	Effective	1.1	1.68	1.84

Table A.8.6-4. Results of Stability Analysis for Cross Section L(L)

Case	Strength Parameters	Minimum Required Factors of Safety	Calculated Factor of Safety	
			Upstream Slope	Downstream Slope
End of Construction	Total	1.3	2.11	2.14
Steady Seepage with Normal Pool	Effective	1.5	2.07	1.93
Steady Seepage with Surcharge Pool	Effective	1.3	2.07	1.80
Rapid Drawdown from Normal Pool	Effective	1.3	1.97	NA
Rapid Drawdown from Surcharge Pool	Effective	1.1	1.98	NA
Steady Seepage with Earthquake Loading	Effective	1.1	1.71	1.66

A.8.7 Erosion Protection

A.8.7.1 General

A variety of alternative wave protection systems are used in reservoir and coastal engineering schemes including: riprap, concrete slabs, concrete blocks, RCC flat plate, RCC stair step, bitumen systems, and various shapes of precast concrete blocks. Typically, the lowest cost protection is provided by using on-site materials if they are suitable. The conceptual design cross sections selected for use in the seepage and stability analyses for this preliminary study of the proposed A-2 Reservoir incorporate RCC as a wave protection system.

A.8.7.2 Roller Compacted Concrete (RCC)

RCC is considered to be an appropriate means of erosion protection for the A-2 Reservoir embankments.

As previously recommended in the BODR for the A-1 Reservoir, the RCC would be installed on a 3H:1V slope at a thickness of 15 inches. A control joint designed to accommodate shrinkage and control of irregular crack development (probably some type of lap joint configuration) should be provided at the top of the slope placement. A drainage layer should be provided beneath the RCC to remove water from behind the RCC during drawdown of the reservoir level.

A.8.8 Foundations

When the embankment crosses local features, such as the existing canals, special cleaning and backfill will be required to avoid differential movement. Foundation bearing capacity is not a significant consideration for the conceptual embankment cross section at this site.

A.8.9 Settlement

A.8.9.1 Foundation Settlement

The most compressible material in the existing ground is the organic peat surface layer. This layer will be removed from the foundation prior to the A-2 Reservoir dam embankment construction. Materials beneath the peat are expected to deform elastically with minimal long-term residual movement under the stress of an embankment. It is not considered necessary to make allowance in the embankment height for settlement of the foundation.

A.8.9.2 Embankment Dam

The materials comprising the embankment will consist of random excavation and “raked” random fill materials from the Fort Thompson Formation. These materials consist of rock pieces (up to 15 percent) and gravel and shells mixed with predominantly sandy silts and silty sands. At the water contents and densities anticipated after construction, it is not considered necessary to make significant allowance for settlement of embankment materials.

A.8.10 Borrow

A.8.10.1 General

Material resources to support construction of the earthen embankment and RCC revetment (excluding cement and additives) are expected to be available on site, based on the field geotechnical explorations performed within the A-1 Reservoir site and the two boreholes performed within the A-2 Reservoir site. However, detailed field exploration must be performed within the A-2 Reservoir site during basic engineering design to further define the borrow materials.

A.8.10.2 Embankment

A.8.10.2.1 Random Fill (D_{max} < 12 inch)

Material for the random fill (D_{max} < 12 inch) can be obtained from the layer of caprock/upper limestone existing immediately below the surface soils. Blasting is required to adequately break up this layer for fill material use. The blasting pattern should be selected such that the random fill (D_{max} < 12 inch) is produced at the optimum gradation for direct use without processing.

The random fill will be hauled to the embankment location and stockpiled either on the interior bench between the embankment and the internal borrow area, or in the location of its final placement in the embankment.

A.8.10.2.2 Random Fill (D_{max} < 6 inch)

Material excavated from the Fort Thompson Formation immediately below the caprock/upper limestone will serve as the source for random fill (D_{max} < 6 inch). In the central zone of the embankment, rock fragments larger than three inches will need to be removed to develop the low permeability core (water barrier) of the embankment. This sorting will occur on the embankment after initial spreading and before compaction using a “rock rake.” This material is expected to be readily available beneath the caprock/upper limestone in all site excavations.

There are typically two layers of limestone within the upper 15 feet of the Fort Thompson Formation. These limestone layers are of low strength and can be removed with an excavator. Additional handling or raking will be required to remove the larger limestone pieces from the central random fill material zone of the embankment.

A.8.10.2.3 Filter Fill

Filter fill will be obtained by crushing, screening, and washing the excavated caprock/upper limestone to the specified gradation. Since the preparation of the filter fill require the use of a crusher, the source of materials is expected to be the interior borrow areas.

A.8.10.2.4 Roller Compacted Concrete (RCC)

RCC will be obtained from a central batching plant and properly stored for use. Aggregates can be obtained by processing on-site rock materials.

A.8.10.2.5 Topsoil

In accordance with SFWMD Design Standards, a layer of topsoil is to be added to the exterior face of an embankment prior to seeding. Area practice is that this topsoil material is obtained from the local peat (muck), and is expected to be available from the material removed from the embankment construction area. The peat can be stockpiled adjacent to the location of the exterior toe of embankment to reduce handling and cost.

A.8.11 Embankment Sections Evaluation

The evaluation of the conceptual embankment sections is discussed below.

A.8.11.1 Typical Dam Embankment Sections

An embankment with a central core of compacted silty sand and inclined chimney drain supported by an inside and outside shoulder (shell) comprised of compacted silty sand with rock fragments was evaluated for the A-2 Reservoir embankment. A typical conceptual cross section (Cross Section K(L)) was analyzed and was presented in **Figure A.8.2-1**. Cross Sections F(L), J-1(L), and L(L) were also evaluated in the seepage and stability analyses. These embankment alternatives were developed to utilize materials expected to be obtained from the borrow excavations with minimum material sorting and processing. The upstream random fill (D_{max} < 12 inches) section will consist of blasted material from the caprock/upper limestone of the Fort Thompson Formation. The random fill (D_{max} < 6 inches) consists of smaller unsorted rock pieces (less than 6 inches maximum size) and silty sand placed without sorting or processing. The processed random fill zone (watertight barrier) between the inside shoulder and inclined chimney drain is to be processed on the fill by raking to eliminate all rock pieces larger than three inches prior to compaction. The inclined chimney is provided for internal drainage, to protect against internal erosion of fines within the random fill, and to control the phreatic line in the downstream random fill zone.

A horizontal blanket filter extends over the caprock to relieve seepage pressures and control loss of infilled fine-grained material from the caprock and upper silty sand foundation. The horizontal drain discharges into a seepage collection ditch at the downstream toe of the embankment.

Top soil (using muck or peat stripped from the embankment foundation) and seeding is provided on the downstream slope. Upstream slope protection is provided by RCC using flat plate construction on the 3H:1V slope extending to the top of the embankment. Muck will also be used to backfill the existing A-1 FEB seepage canal along the east embankment of the A-2 Reservoir.

Foundation preparation for these conceptual design cross sections include blading the caprock surface to remove muck and clay remaining after stripping and brushing the caprock surface using a power broom. The soil-bentonite cutoff wall will be located generally beneath the center of the embankment sections and extended a minimum of three feet above the caprock surface into the central core. The cutoff trench will be widened through the caprock to allow placement of a lean concrete seal on each side of the cutoff.

A.9 RESERVOIR SEEPAGE

A.9.1 INTRODUCTION

This section describes the methods for quantifying and managing the anticipated seepage losses from the A-2 Reservoir and A-2 STA. As with other surface water features such as STAs and canals, seepage will occur from the A-2 Reservoir and A-2 STA because the soil within approximately 200 feet below the surface of the site is highly permeable. Changes to existing groundwater flow patterns beneath the A-2 Reservoir and A-2 STA will be caused by seepage from the A-2 Reservoir and A-2 STA and how the seepage controls are built and operated.

Both three-dimensional (3D) MIKE SHE groundwater modeling and two-dimensional (2D) SEEP/W groundwater modeling was performed to analyze seepage from the A-2 Reservoir. The aquifer parameters used in the A-2 Reservoir groundwater model were determined from previous calibration of the groundwater models prepared for the EAA Reservoir A-1 BODR and the EAA A-1 FEB Final DDR. The parameters for the caprock and Fort Thompson formations were verified through geotechnical investigations performed by Ardaman & Associates, Inc. on the A-2 site during February and March of 2018, as described **Section A.7.10.** The SEEP/W methodology and results are presented in **Section A.8**. The groundwater models were used to evaluate the following seepage impacts:

- The effect of seepage on the A-2 Reservoir embankment stability
- The amount of water the A-2 Reservoir and A-2 STA loses to seepage
- The amount of seepage that is collected and returned to the A-2 Reservoir
- The effectiveness of various seepage control alternatives
- The amount of unrecoverable seepage, if any, that migrates to surrounding areas for the various seepage control alternatives
- The effect of any unrecoverable seepage on groundwater levels in the surrounding areas

The surrounding areas include: 1) farmland to the north of the A-2 Reservoir, west of the Miami Canal and the east of the NNR Canal, 2) U.S. Hwy. 27 immediately east of the A-1 FEB, 3) STA-3/4 to the south of the A-1 FEB, and 4) the Holey Land to the south of the A-2 Reservoir. Goals for managing seepage to each of these areas are as follows:

- <u>Farmland</u>: Control seepage to prevent impacts to surrounding groundwater levels or impacts to existing farming operations.
- <u>U.S. Hwy. 27</u>: Control seepage to prevent groundwater levels from rising into the base of the highway or into the adjacent drainage ditches along the east and west side of U.S. Hwy. 27 (as required by the Florida Department of Transportation).
- <u>STA-3/4</u>: Control seepage to STA 3/4 and to the STA 3/4 Inflow Canal (which conveys water to STA 3/4) to an acceptable percentage of the capacity of STA 3/4. If an acceptable percentage cannot be defined, there may be the need to eliminate seepage to STA 3/4 at an added cost to the Project.
- <u>Holey Land</u>: Control seepage impacts to the Holey Land to maintain regulation schedule and avoid any undesirable impacts. Assess potential for adjustments to water deliveries to offset additional seepage water entering Holey Land.

For the purposes of this analysis, impacts to the surrounding areas caused by seepage are defined as any change in groundwater levels greater than the predictive accuracy of the three-dimensional groundwater model. For the purposes of this analysis, impacts are defined as greater than 0.3 feet of increase in the groundwater level.

A.9.2 GROUNDWATER / SURFACE WATER MODEL DEVELOPMENT

A coupled surface water and groundwater MIKE SHE / MIKE 11 model was used to evaluate the potential seepage impact of the A-2 Reservoir and A-2 STA in surrounding lands. The model area includes the A-2 Reservoir, the A-2 STA, the A-1 FEB and surrounding areas. A horizontal computational grid size of 150 feet x 150 feet was specified.

A.9.2.1 Groundwater Model Boundary

The model boundary was delineated at a distance far enough away from the A-2 Reservoir so that the boundary effects would be minimal and not affect the seepage estimates. Approximately half of the model boundary, along the north, northwest and northeast sides, was delineated along farm ditches. The rest of the boundary, in the south, southwest, and southeast sides falls along larger canals adjacent to natural wetlands or a treatment wetland area. The canals along the southern boundary from west to east are: the STA 5 Discharge Canal, STA 6 Discharge Canal, the L-3 Canal, the L-4 Canal, the STA 3/4 Discharge Canal, the L-5 Canal, L-6 Canal, and STA 2 Discharge Canal. The yellow polyline shown in **Figure A.9.2-1** represents the model boundary.

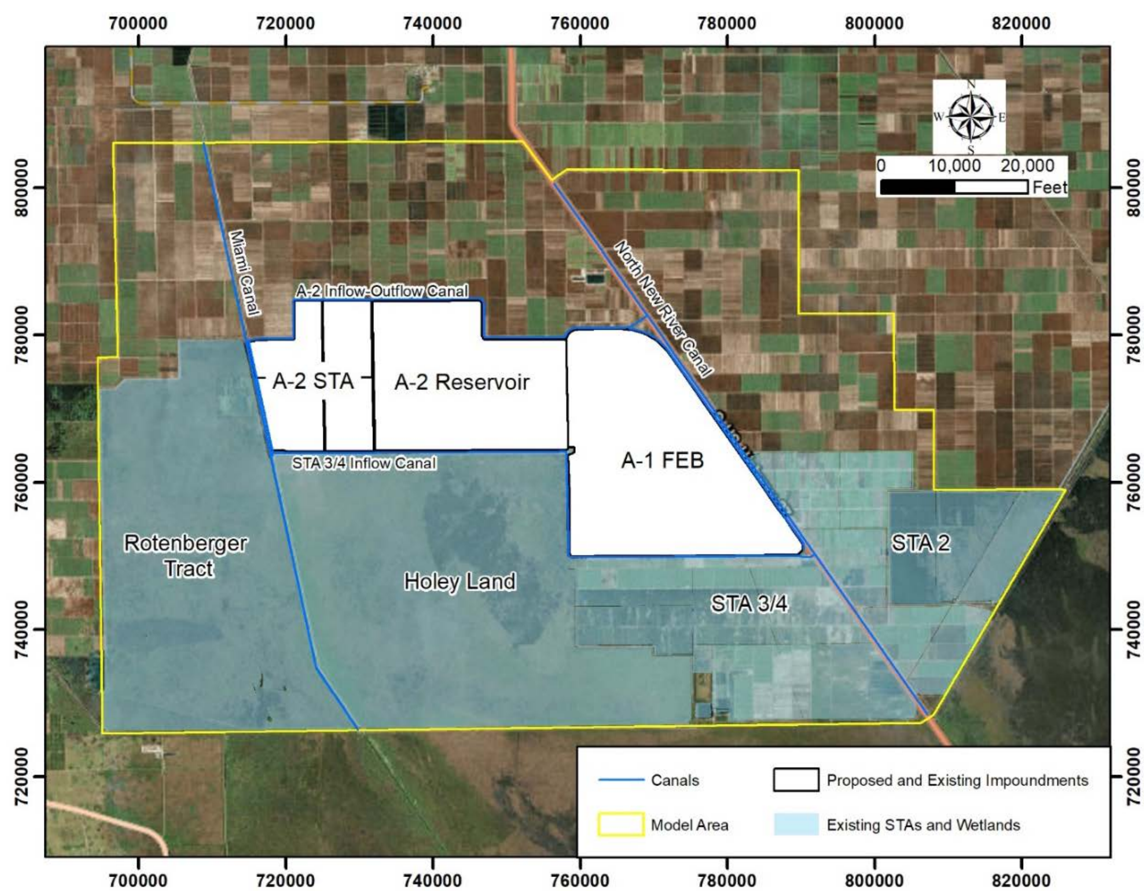

Figure A.9.2-1. Groundwater Model Area

A.9.2.2 Groundwater Model Boundary Conditions

A.9.2.2.1 External Boundary Conditions

A fixed groundwater elevation boundary condition of 6.1 ft-NAVD was specified where the model boundary is located within farm fields, which represents approximate groundwater level in the farm fields controlled by the network of farm canals.

Since the canals adjacent to the wetland areas are maintained at a higher elevation than the farm canals, a fixed groundwater elevation boundary condition of 9.0 ft-NAVD was specified where the model boundary is located within wetland areas (i.e. Rotenberger Tract, Holey Land, STA 3/4 and STA 2). To determine this value, the measured data at various locations along the boundary canals for the most recent years and available period of record were averaged.

A.9.2.2.2 Internal Boundary Conditions

Internal boundary conditions are specified in the top (muck) layer of the model to maintain fixed stages within the A-2 Reservoir, A-2 STA, A-1 FEB, STA-2 and STA 3/4 throughout the simulation. For the existing A-1 FEB and the STAs the stages were obtained from the average measured data available for these areas. The fixed stages specified for A-1 FEB, STA 3/4, and STA 2, were 10.4, 10.0, and 11.0 ft-NAVD, respectively. The fixed stages specified for the A-2 Reservoir and A-2 STA were 31.1 and 12.5 ft-NAVD, respectively, which are the NFSLs for the two proposed impoundments.

No internal boundary conditions were used for the agricultural areas and agricultural ditches are not simulated. Thus, the farm canals within the model boundary are not operated and the groundwater levels at farm land is allowed to increase above its normal operating levels. Although this assumption is not an actual representation of existing conditions, it provides a measure of how much additional pumping would have to occur to mitigate the effect of seepage from the A-2 Reservoir and A-2 STA without any other seepage measures.

A.9.2.3 Groundwater Model Parameters

The groundwater model developed was discretized into 5 hydrogeological layers with aquifer parameters based on the seepage model parameters presented in **Section A.8.5.2**. The parameters for the caprock and Fort Thompson formations were verified through geotechnical investigations performed by Ardaman & Associates, Inc. on the A-2 site during February and March of 2018, as described **Section A.7.10**. The seepage cutoff wall surrounding the A-2 Reservoir was represented in the model by modeling it as a sheet pile wall according to the geometry of the conceptual design cross-sections (provided in **Annex C-1**). **Table A.9.2-1** shows the geological layers and properties included in the model.

Table A.9.2-1. Groundwater Model Parameters

Hydrogeologic Unit	Model Layer	Bottom Elevation (ft-NAVD)	Horizontal Conductivity (ft/day)	Vertical Conductivity (ft/day)
Muck/Embankment Fill	1	6.5	100.0/0.3	100/0.07
Fort Thomson - Caprock	2	-1.5	99.2	1.0
Fort Thompson - Sand	3	-25.5	510.2	10.0
Caloosahatchee	4	-60.0	396.9	8.0
Tamiami	5	-210.0	34.0	17.0
Seepage Cutoff Wall*	Sheet Pile	-34.1	0.003	0.003

*Specified depth was simulated according to the scenarios described in **Section A.9.3.**

The 5-foot resolution LIDAR dataset for the EAA (2007-08_HHDEAA_5-ft_DEM2C_v1), obtained from the SFWMD GIS database, was used to define the model topography and top elevation of the groundwater model. The DEM was averaged to the 150-foot model resolution. The embankment heights for the A-2 Reservoir, the A-2 STA, and the A-1 FEB were added to the model topography and the conductivity values specified in **Table A.9.2-1** were used. The proposed borrow area within the A-2 Reservoir as shown in the conceptual design cross-sections (provided in **Annex C-1**) was also included in the model.

A.9.2.4 Surface Water Model Parameters

The canals surrounding the A-2 Reservoir, the A-2 STA, and the A-1 FEB were included in the model to set stage boundary conditions and receive seepage from the project features. The cross-section for each canal was obtained from the conceptual design cross-sections (provided in **Annex C-1**), HEC-RAS models developed for the project, and the A-1 FEB record (as-built) drawings. **Table A.9.2-2**, summarizes the canals and their fixed stages specified in the model.

Table A.9.2-2. Groundwater Model Parameters

Canal Name	Fixed Stage (ft-NAVD)	Basis for Stage
A-2 Reservoir Inflow-Outflow Canal*	8.9	Historical Stage Data for NNR & Miami Canals / Seepage Recovery
Miami Canal	8.9	Average Historical Stage Data
NNR Canal	8.9	Average Historical Stage Data
STA 3/4 Inflow Canal	9.1	Average Historical Stage Data
A-1 FEB Seepage Canal	8.0	Average Historical Stage Data
A-2 STA Discharge Canal	8.9	Average Historical Stage Data

* Specified stage was simulated according to the scenarios described **Section A.9.3**.

Each simulation was run for three years, which was sufficient time for the model to reach steady-state seepage rates.

A.9.2.5 Comparison of the 3D Groundwater Model with the 2D Groundwater Models

A comparison of the results from the 3D MIKE SHE groundwater model for the project and the 2D SEEP/W groundwater models of the A-2 Reservoir (described in **Section A.8**) was performed. The model comparison was done for the north and south embankment cross-sections of the A-2 Reservoir, which are referred to as sections F(L) and J-1(L), respectively. The 2D and 3D model results are affected by the boundary conditions specified in the models. Specifically, the large head gradients that occur along the edge of the A-2 Reservoir can vary according to the internal heads specified in the various layers of the model, producing different seepage rates. The internal boundary conditions in the 3D model, described in **Section A.9.2.2** and subsequently used for in the simulations described in **Section A.9.3**, were modified to more closely match the 2D model boundaries for the purpose of the model comparison. For example, on the reservoir side of each embankment cross-section, the 2D models use a fixed head boundary at a distance of 1,000 feet from the internal toe of the slope of the reservoir embankment in all the layers of each 2D model. To recreate a similar condition in the 3D model, an internal boundary was added in the 3D model at approximately this distance from the internal toe of the slope of the reservoir embankment in the lower layers of the model. However, the relative distances along all the features in the cross-section in the coarser resolution 3D model are not as precise as the 2D model due to the finer mesh size in the 2D model. **Table A.9.2-3** shows the seepage rates estimated by both models across the northern A-2 Reservoir embankment (cross-section F(L)) and the southern A-2 Reservoir embankment (cross-section J-1(L)).

Table A.9.2-3. Comparison of Seepage Rates between 2D and 3D Models

A-2 Reservoir Embankment Cross-Section	Stages (ft-NAVD)		Seepage Rate (cfs/mile)		% Difference
	Upstream	Downstream	3D Model	2D Model	
Cross-section F(L)	31.1	4.5	35.6	40.0	11.7
Cross-section J-1(L)	31.1	6.5	32.5	38.0	15.6

Figures A.9.2-2 and **A.9.2-3** show the head elevation contour profiles for cross-sections F(L) and J-1(L) simulated in the 3D model. These figures were prepared for the purpose of comparing results between the 3D and 2D seepage models. These figures can be compared to the 2D seepage model cross-sections shown in **Figures A.8.5-4** and **A.8.5-7**, presented in **Section A.8**.

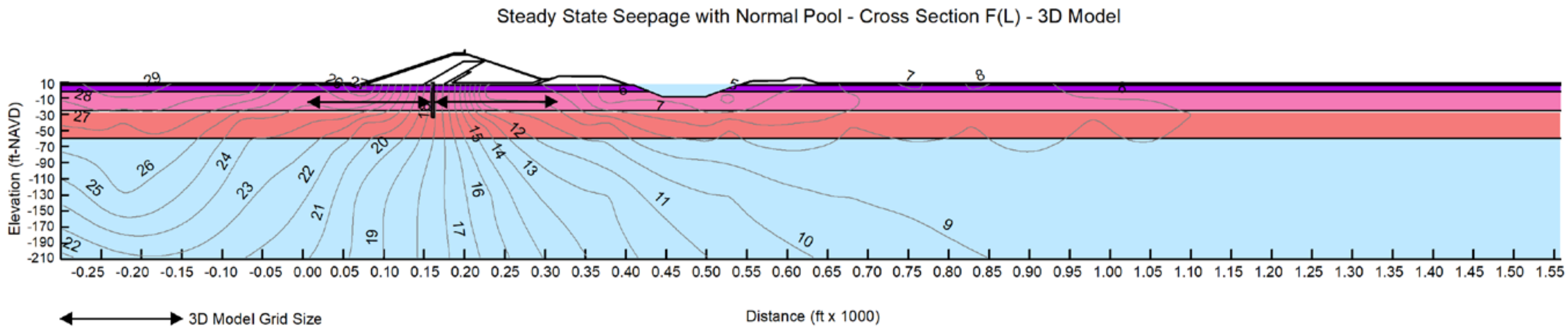

Figure A.9.2-2. Head Elevation Contours, F(L) Cross-section

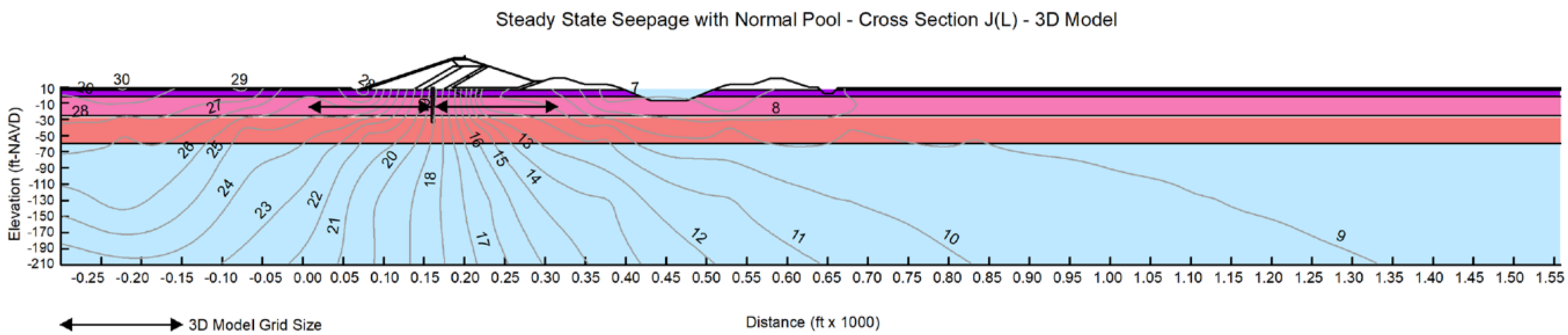

Figure A.9.2-3. Head Elevation Contours, J-1(L) Cross-section

A.9.3 Groundwater Model Simulations and Seepage Management

A.9.3.1 Existing Conditions Model

A model representing the existing conditions was developed to establish a baseline for comparing the potential seepage impact of the A-2 Reservoir and A-2 STA. The model uses the same surface water and groundwater model features and parameters described in **Section A.9.2**, but it excludes all the features that represent the A-2 Inflow-Outflow Canal, the A-2 Reservoir, and the A-2 STA.

A.9.3.2 Proposed Project with Passive Seepage Management

The proposed passive management model includes the A-2 Inflow-Outflow Canal, the A-2 Reservoir, and the A-2 STA, as described in **Section A.9.2**. The passive seepage management simulation provides a scenario where operations (and the resulting stage) along the A-2 Reservoir Inflow-Outflow Canal are controlled by the upstream stage conditions in the Miami Canal and North New River Canal. The resulting seepage impacts provides the justification for additional seepage management measures such as controlling the stages of the A-2 Reservoir Inflow-Outflow Canal to mitigate seepage impacts. The A-2 Reservoir seepage cutoff wall and the A-2 Reservoir Inflow-Outflow Canal cross sections were simulated based on the conceptual design cross sections (provided in **Annex C-1**).

To measure the potential impact of the model boundary in the seepage estimates, fluxes out of the external boundary of the model are compared **(Table A.9.3-1)** in the existing conditions model and proposed conditions (project) model (with passive seepage management). The extent of the north and south boundaries is defined within the Miami and NNR Canals and the ends of east and west boundary extents were defined from the north and south portions just west and east of the Miami and NNR Canals, respectively. Since the stages are fixed in the existing STAs, the fluxes out of the STA 3/4 and STA2 were excluded from the south and east boundaries, respectively. The largest change in the simulated fluxes between the proposed passively managed project and existing conditions simulations was 0.18 cfs/mi in the northern boundary. This value is a small percentage of the seepage estimates (**Table A.9.3-2**), 0.8%. Thus, the effect of the boundary is considered negligible.

Table A.9.3-1. Simulated Fluxes out of the External Boundary in Existing and Proposed Conditions

Boundary Side*	Flux (cfs/mile)	
	Existing Conditions	Proposed Project
South	0.04	0.06
West	0.11	0.12
North	0.38	0.56
East	0.56	0.58

*Refer to the description above of the boundary extents.

Figure A.9.3-1 shows the difference in head elevation in the caprock layer between the existing conditions simulation and the proposed project simulation.

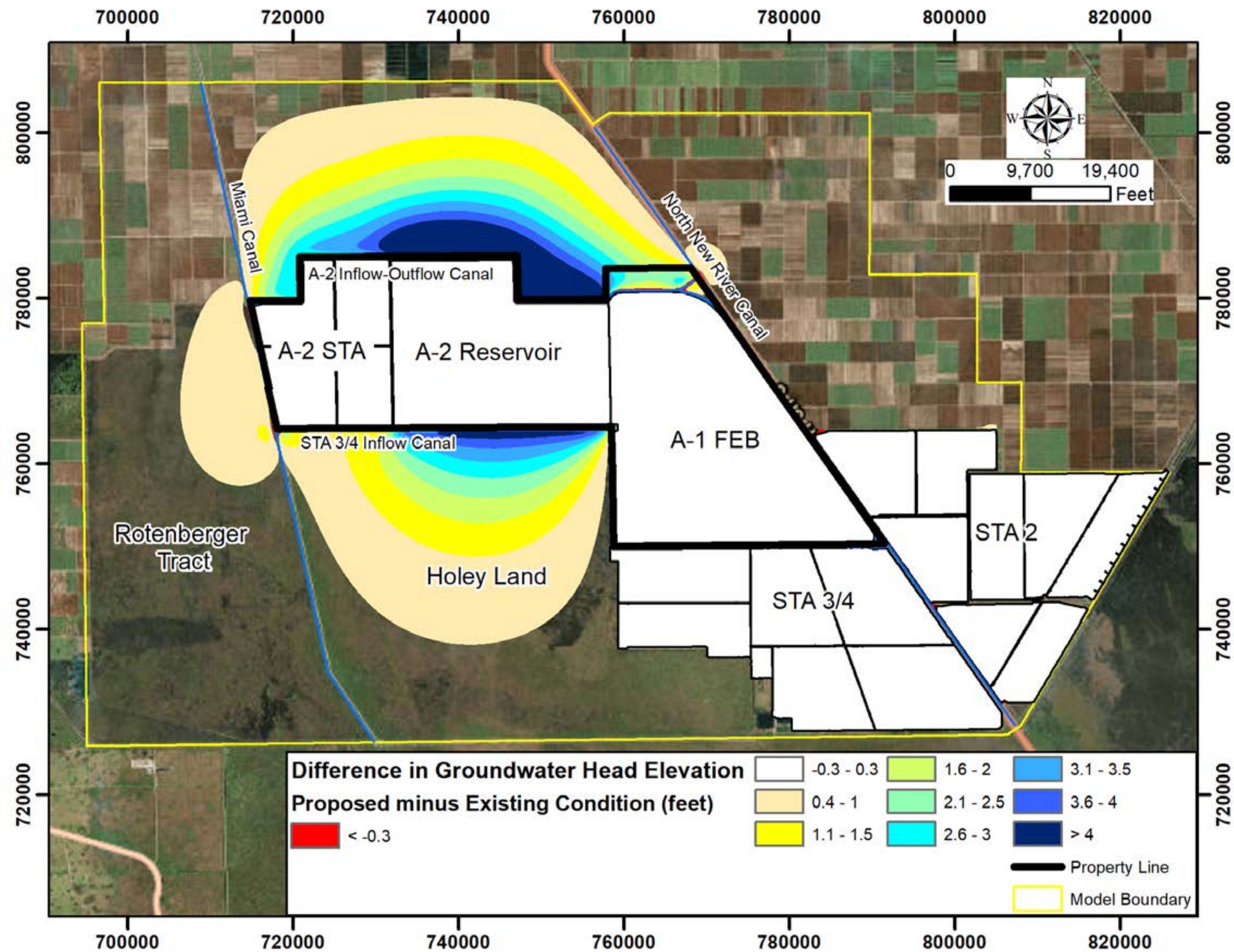

Figure A.9.3-1. Simulated Difference in Groundwater Head Elevation between Passively-Managed Proposed Project Model and Existing Conditions Model

Table A.9.3-2 shows the simulated seepage rates of seepage flows out of the A-2 Reservoir and A-2 STA, beneath and through the embankments of these impoundments.

Table A.9.3-2. Simulated Seepage Flow Rates for Seepage Flow Out of Impoundments for Proposed Project with Passive Seepage Management

Levee/Dam that Seepage is Flowing Beneath & Through	Seepage Rate (cfs/mi)
A-2 Reservoir North	24.9
A-2 Reservoir South	26.5
A-2 Reservoir East	24.0
A-2 Reservoir West	20.4
A-2 STA North	2.2
A-2 STA South	5.3
A-2 STA West	5.9

A.9.3.3 Proposed Project Alternatives with Active Seepage Management

Four alternatives to the proposed design were simulated to mitigate any off-site seepage impacts on the lands north of the A-2 Reservoir and reduce seepage recovery pumping costs while maintaining dam embankment stability. The alternatives consist of modifying the proposed seepage cutoff wall depth in the north side of the A-2 Reservoir and the A-2 Reservoir Inflow-Outflow Canal depth along the portion of the canal adjacent to the north boundary of the A-2 Reservoir. In addition, the alternatives conceptualize an active seepage management system which consists of simulating seepage pumps controlling the stage in the A-2 Reservoir Inflow-Outflow Canal within a specified range when the A-2 Reservoir Inflow-Outflow Canal is not conveying flows to or from the A-2 Reservoir and Spillways SW-2 and SW-3 are closed. The seepage pumps will be the electric motor driven pumps at Pump Station P-1 and they will pump seepage water from the A-2 Reservoir Inflow-Outflow Canal into the A-2 Reservoir. **Section C.16** of the Draft Project Operating Manual included in **Annex C** further describes the proposed seepage management.

For each the alternative, the A-2 Reservoir Inflow-Outflow Canal was initially set to the minimum design stage of 4.5 ft-NAVD (**Section A.6**) for Pump Station P-1. If the alterative configurations showed negligible seepage impact on the lands north of the A-2 Reservoir, then the stage in the canal would be increased in 0.5-foot increments until a significant impact (i.e., groundwater head increase > 0.3 feet) is shown. If the alternative configuration showed a significant impact north of the A-2 Reservoir, then the canal stages would be decreased in 0.5-foot increments until no impact is shown. Further simulations were conducted for the first 3 alternatives that show that decreasing the stages in the A-2 Reservoir Inflow-Outflow Canal is effective in substantially reducing the seepage impacts to the north.

Table A.9.3-3 shows the cut-off wall depths and canal bottom elevations used for each alternative and the final stages in the A-2 Reservoir Inflow-Outflow Canal required for no impacts north of the A-2 Reservoir.

Table A.9.3-3. Proposed Project Alternatives for Active Seepage Management

Alternative	Seepage Cutoff Wall Bottom Elev. (ft-NAVD)	A-2 Reservoir Inflow-Outflow Canal Bottom Elev. (ft-NAVD)	A-2 Reservoir Inflow-Outflow Canal Stage (ft-NAVD)
1	-34.1	-8.0	0.5
2	-34.1	-16.0 (along A-2 Reservoir boundary) -8.0 (along all other portions of canal)	0.5
3	-65.0	-8.0	3.5
4	-65.0	-16.0 (along A-2 Reservoir boundary) -8.0 (along all other portions of canal)	4.5

Table A.9.3-4 shows, for each alternative evaluated, the total simulated seepage flows out of the A-2 Reservoir along its northern embankment and the total groundwater flow that reaches the A-2 Reservoir Inflow-Outflow Canal from the reservoir side, i.e., the southern side of the canal. The results show that the deeper cutoff wall (Alternatives 3 and 4) reduces the seepage flow rate out of the A-2 Reservoir to approximately half the seepage flow rate produced with the shallower cutoff wall. The results also show that increasing the depth of the A-2 Reservoir Inflow-Outflow Canal or reducing the stage in the A-2 Reservoir Inflow-Outflow Canal, both increase the amount the seepage flow from the A-2 Reservoir that is intercepted by the A-2 Reservoir Inflow-Outflow Canal.

Table A.9.3-4. Simulated Seepage Flows in Proposed Project Alternatives Evaluated

Alternative	A-2 Reservoir Inflow-Outflow Canal Stage (ft-NAVD)	Total Groundwater Flow Out of A-2 Reservoir Beneath & Through A-2 Reservoir Northern Embankment (cfs)	Groundwater Flow from the A-2 Reservoir Directly Intercepted by the A-2 Reservoir Inflow-Outflow Canal (cfs)
1	4.5	186	96
1	0.5	215	116
2	4.5	186	98
2	0.5	216	117
3	4.5	88	44
3	3.5	92	47
4	4.5	91	52

Figure A.9.3-2, Figure A.9.3-3, Figure A.9.3-4, and **Figure A.9.3-5** show the groundwater head difference maps for alternatives 1, 2, 3, and 4, respectively, showing little or no impact north of the A-2 Reservoir.

The results of the seepage modeling indicate that seepage impacts to off-site areas can be mitigated with active management of stage levels in the A-2 Reservoir Inflow-Outflow Canal with or without deepening the proposed A-2 Reservoir seepage cutoff walls or A-2 Reservoir Inflow-Outflow Canal. While all the alternatives essentially eliminated the seepage impacts to the land north of the A-2 Reservoir, shown in **Figure A.9.3-1**, only Alternative 4 achieved this when the canal stage is at the minimum design stage for Pump Station P-1 of 4.5 ft-NAVD. The simulated seepage rates provided in **Table A.9.3-4**, can be used to estimate the seepage pumping rate required to maintain the water level in the A-2 Reservoir Inflow-

Outflow Canal at the stage shown for each alternative, to mitigate seepage impacts to the land north of the A-2 Reservoir.

The results also indicate that the existing seepage management system of the A-1 FEB, working in unison with the A-2 seepage management system is effective at mitigating impacts to the east. There may be minor seepage flows to the Rotenberger property which could be mitigated during PED phase of the project or may be deemed as beneficial given the natural condition of these lands. The model results show more significant flows are observed to the south in the Holey Land which may require additional seepage mitigation during PED phase of the project. Some beneficial impacts to Holey Land may be realized in reducing the current inflow pumping required to minimize undesirable dry conditions.

Additional analysis and optimization of the seepage management system of the project should be performed during PED phase of the project when more detailed geotechnical analysis and modeling could be performed.

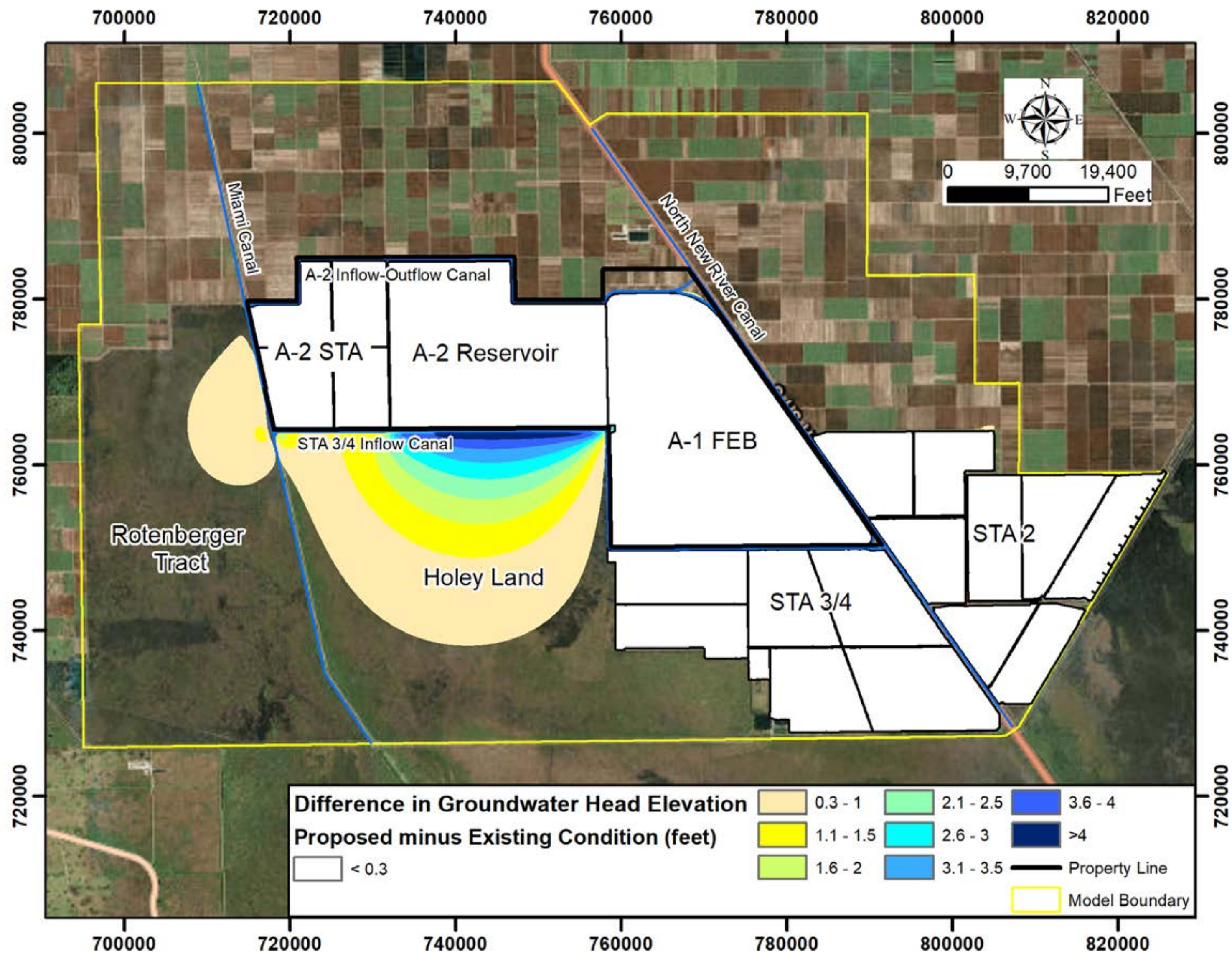

Figure A.9.3-2. Simulated Difference in Groundwater Head Elevation Proposed Project minus Existing Conditions – Alternative 1

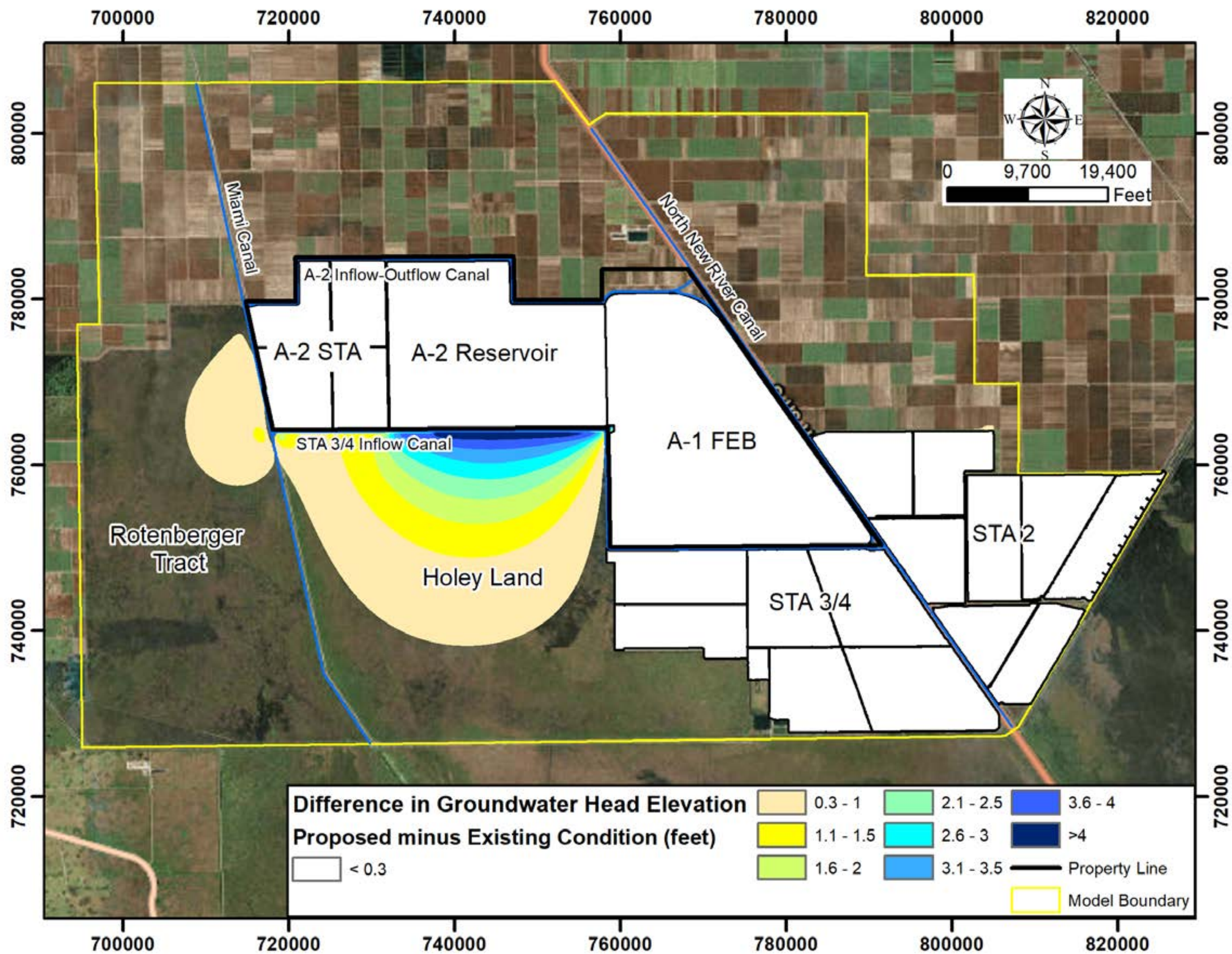

Figure A.9.3-3. Simulated Difference in Groundwater Head Elevation Proposed Project minus Existing Conditions – Alternative 2

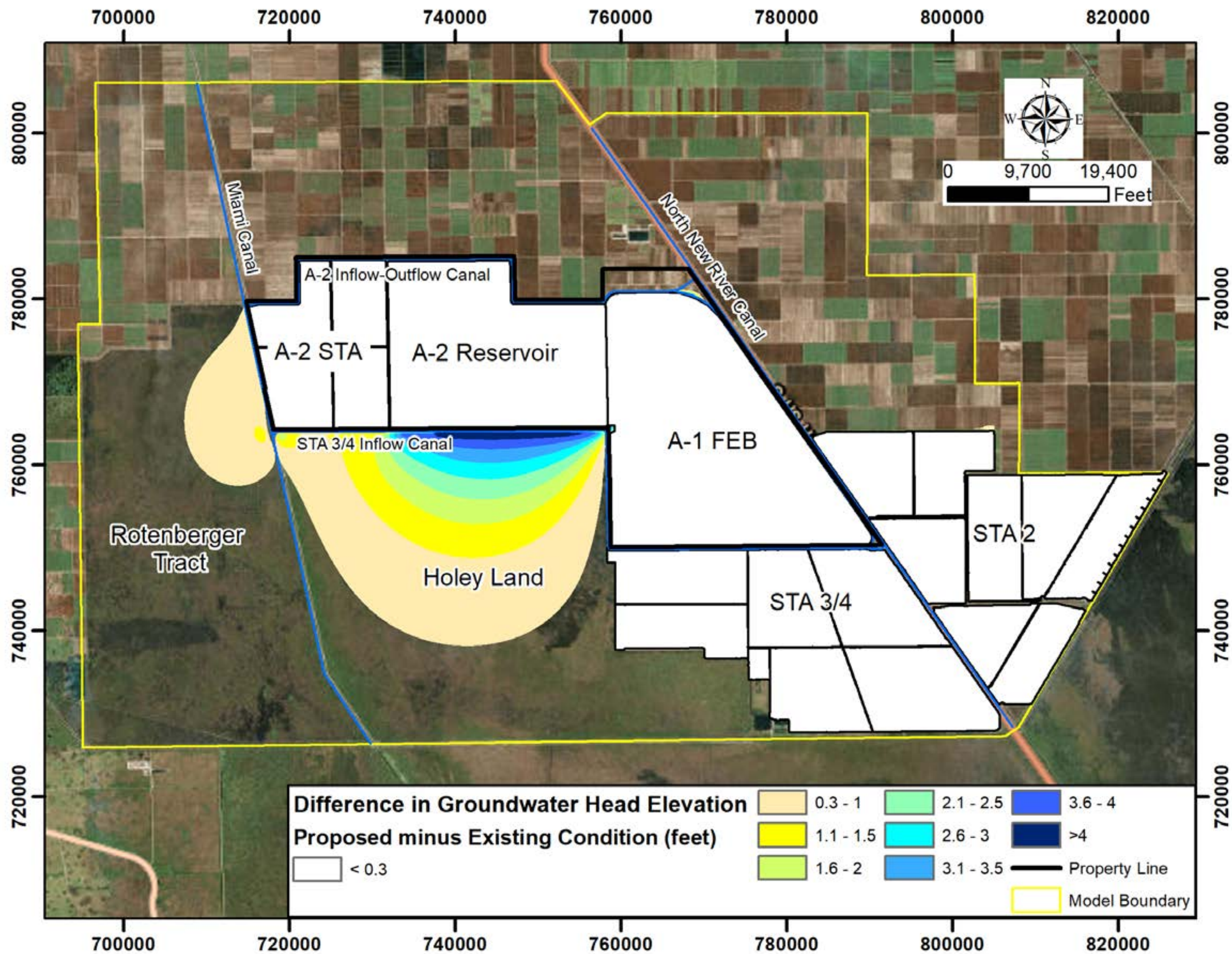

Figure A.9.3-4. Simulated Difference in Groundwater Head Elevation Proposed Project minus Existing Conditions – Alternative 3

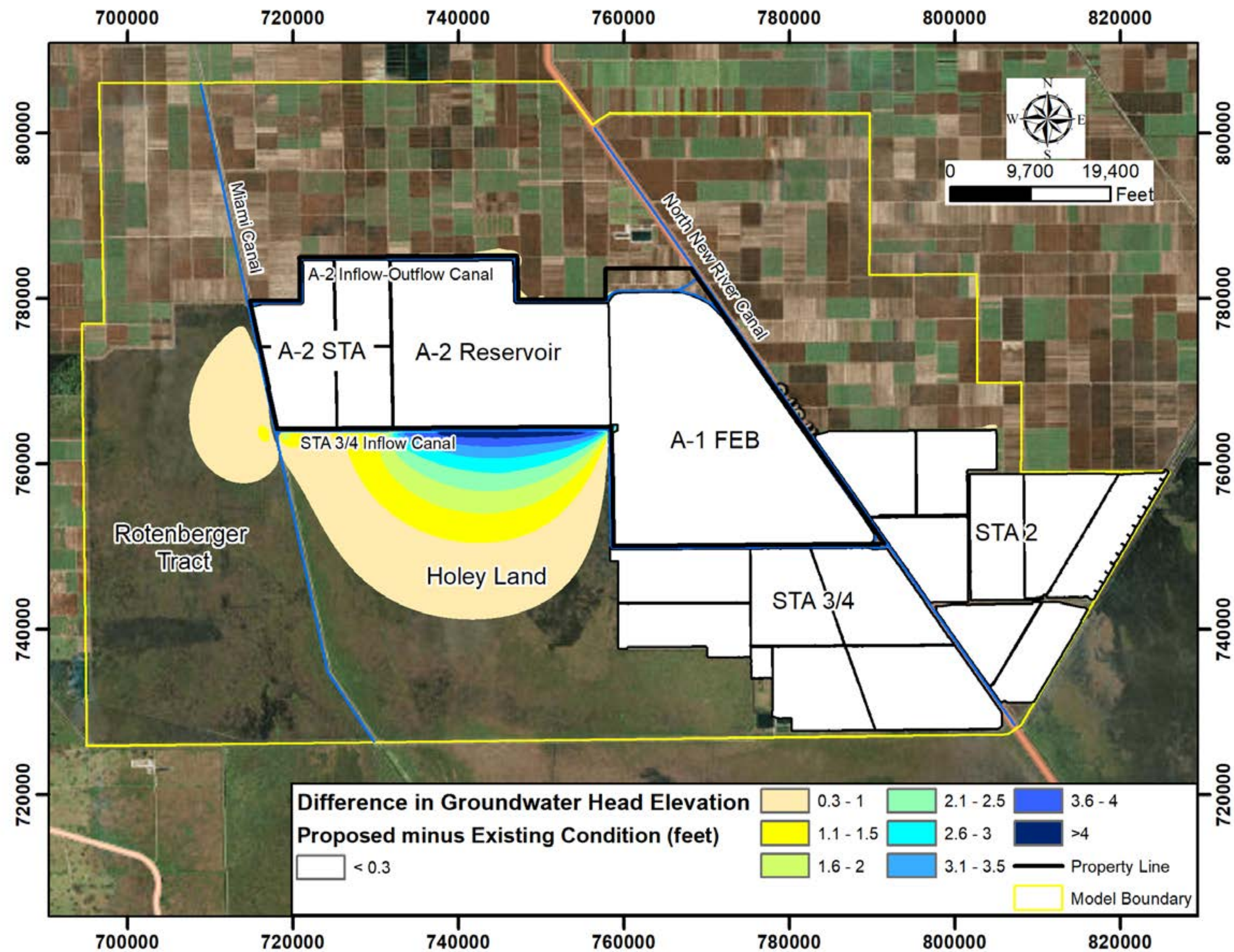

Figure A.9.3-5. Simulated Difference in Groundwater Head Elevation Proposed Project minus Existing Conditions – Alternative 4

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A.10 STRUCTURAL DESIGN CRITERIA

This section describes the basis of structural design for new or modified facilities.

A.10.1 APPLICABLE CODES AND STANDARDS

Design of structural elements will comply with the design codes and standards included in the Codes and Standards portion of **Section A.4**.

A.10.2 DESIGN STRESSES

A.10.2.1 Minimum Concrete Compressive Strength (Unconfined)

- Mass concrete, concrete deduction factor (f'c) 3,000 pounds per square inch (psi) at 28 days
- Structural concrete, f'c 4,000 psi at 28 days

A.10.2.2 Reinforcing Steel

- ASTM A615, steel yield strength (fy) 60,000 psi

A.10.2.3 Structural Steel

- Wide flange shapes, ASTM A572, Grade 50, fy 50,000 psi
- Angles, channels and plates, ASTM A36, fy 36,000 psi
- Pipe sections, ASTM A53, Type E, fy 35,000 psi
- Tube sections, ASTM A500, Type B or C, fy 46,000 psi

A.10.2.4 Masonry

- Concrete masonry units (CMU), Grade N-1, compressive strength 1,900 psi
- Compressive strength of mortar, Type S 1,800 psi
- Compressive strength of grout 2,000 psi
- Masonry unit assembly, compression strength (f'm) 1,500 psi

A.10.3 LOADING CRITERIA

A.10.3.1 Dead Loads

- Equipment Actual
- Phantom load 1 kip[1] at secondary beams & 2 kips at primary beams
- Bridge crane or monorail Actual crane beam + rail only
- Roof, superimposed Actual, 15 pounds per square foot (psf) minimum

[1] A unit of weight equal to 1,000 pounds or 455 kilograms

A.10.3.2 Live Loads, per Major Pumping Station Engineering Guidelines

• Roof (minimum, unreduced)	50 psf
• Operating floors	250 psf, or the heaviest piece of machinery anticipated to be placed therein, whichever is larger
• Control rooms	100 psf
• Restrooms	100 psf
• Equipment and storage rooms	200 psf
• Maintenance work areas	300 psf
• Stairways	100 psf
• Elevator lift and handicap ramp	200 psf
• Deck grating	250 psf
• Service bridge	HS-25 or SFWMD 40T truck crane loading (P&H 440TC), whichever is larger
• Guardrails (at top rail)	50 pounds per linear foot (plf) + 200 pound concentrated load, acting in any direction
• Bridge crane or monorail, vertical load	Rated capacity (full wheel load) + 25 percent impact
• Bridge crane or monorail, lateral and longitudinal loads	Lateral load = 20 percent of the sum of the weights of the lifted load + the crane trolley. Longitudinal load = 10 percent of the maximum wheel load.

For large equipment areas the combined weight of equipment and base plus an additional live load of 50 psf over the base area will be used as the live load.

A.10.3.3 Lateral Loads

• Active earth pressure	Conducted at 30 percent design
• At-rest earth pressure	Conducted at 30 percent design
• Passive earth pressure	Conducted at 30 percent design
• Lateral surcharge load from compaction (decreases linearly)	400 psf at the ground surface, 0 psf at the depth equal to 400 psf divided by the earth pressure
• Hydrostatic	63 pounds per cubic foot (pcf)
• Vertical surcharge, at locations subject to truck or equipment loads	Surcharge shall be calculated based on the equipment listed in **Section 10.3.2**, subject to a 500 psf minimum

The active pressure values will only be used for site retaining walls that are free to rotate.

A.10.3.4 Snow Loads - Not Applicable

A.10.3.5 Seismic Loads

Earthquake loads will not be considered, in accordance with the 2017 Florida Building Code.

A.10.3.6 Wind Loads – Pump Station

- Design 3-second gust wind speed 155 mph
- Height and exposure coefficient Exposure C
- Structure importance factor 1.51
- Building type Partially enclosed

A.10.3.7 Wind Loads – Flood Control Elements

- Design 3-second gust - wind speed 130 mph
- Height and exposure coefficient Exposure C
- Structure importance factor 1.30
- Building type Partially enclosed

A.10.3.8 Flood Load (Hydrostatic + Wave)

- Dynamic Pressure Coefficient (ASCE 7-02, Table 5-2) 3.5

A.10.4 HYDRAULIC STRUCTURES AND PUMPING STATION SUBSTRUCTURE

A.10.4.1 Materials of Construction

Hydraulic structures and the new A-2 Reservoir pump station substructure will be constructed of reinforced concrete. Because water in the Everglades is aggressive to concrete, Type II Cement will be specified. Any platforming associated with these items will be constructed of aluminum shapes, aluminum grating and aluminum guardrail. Connection bolts will be either stainless steel or aluminum. Reinforced concrete platforming will be used in locations where the use of grating is not appropriate.

A.10.4.2 Design Procedures and Assumptions

Hydraulic structures and the new A-2 Reservoir pump station substructure will be designed based upon the loads, load combinations and allowable stresses contained in EM 1110-2-2104, subject to meeting the requirements of the SFWMD's latest Design Standards and ACI 318-02. Temperature and shrinkage reinforcement and cracking limits will be in accordance with ACI 350.

- For reinforcement in shear, the required strength is 1.3 times the excess applied shear (Vu) less shear carried by the concrete (NVc). Thus NVs >1.3 (Vu-NVc), where NVs is the design capacity of shear reinforcement.
- Rectangular walls may be analyzed as two-way rectangular plates when the aspect ratio of length to height is 2H:1V or less. The boundary conditions will be chosen to give reasonably conservative results. If the aspect ratio exceeds 2H:1V, the wall will be designed as a one-way rectangular plate and the corners will be investigated assuming a 2H:1V ratio.

- The design of water containment walls will consider both flexure and tension in the walls. The horizontal reinforcement on the water side will be apportioned for 100% flexure steel plus 100% tension steel.
- Direct tension in the foundation and top slabs due to internal water pressure will be accounted for in the design of the slab's horizontal reinforcing. The foundation's top reinforcement will be assumed to resist 100% of the tension at the foundation. The tension in the top slab may be resisted equally between the top and bottom reinforcement for reasonably thin slabs.
- A minimum reinforcement for shrinkage and temperature will be provided in accordance with ACI 350. As indicated in ACI 350, a minimum reinforcement ratio of 0.5% will be provided in basin walls and base slab with a basin dimension of 50 feet or more in any direction. Reinforcement ratios in the direction where the structure dimension is less than 50 feet will be in accordance with ACI 350. Minimum size of shrinkage and temperature reinforcement will be #4 and will be divided equally between the two surfaces of the concrete section. Concrete sections greater than 24 inches thick may have minimum reinforcing based on a 24-inch thickness. The shrinkage and temperature reinforcement in the bottom of slabs reinforced top and bottom, in contact with the subgrade, can be reduced to one-half the values calculated.
- Hydrostatic groundwater pressure for structures adjacent to the A-2 Reservoir will be based on the water level of the A-2 Reservoir. In accordance with USACE EM 1110-2-2104, the uplift pressure distribution along the base of foundations will be assumed to be linear between the upstream and downstream edges of the foundation. The pressure distribution will be modified to take into account any foundation drains or groundwater cutoff devices. Uplift reduction at drains may not exceed 50 percent of the difference between the full uplift head at the pump station intake and the drain.

A.10.4.3 Design Load Cases

Listed below is a summary of the loading assumptions and load factors for design, where:

DL = Dead load

LL = Live load

Hw = Lateral hydrostatic pressure

Hweq = Lateral hydrodynamic pressure

Hs = Lateral Static Soil Load (including at-rest soil plus groundwater hydrostatic pressure, surcharge, and compaction pressures)

Fa = Flood load

W = Wind load

U = Required strength to resist factored loads

A.10.4.4 Service Water Condition

For a maximum service water level, ignore the soil backfill loads unless soil loads are additive to the overall loading on a structural element, and consider internal tensile forces in the wall with a hydraulic factor of 1.65, and load combinations as follows:

Flexure: U = 1.3[1.4(DL) + 1.7(LL) + 1.7(Hw)]

Shear: U = [1.4(DL) + 1.7(LL) + 1.7(Hw)]

Flexure and Shear: U = 0.9D + 1.6W + 1.6Hw

A.10.4.5 Flood/Overflow Water Condition

For maximum water level at the flood/overflow elevation (highest water elevation that could occur hydraulically, which is not necessarily at the top of wall), where the cracking limit is not applicable, ignore the soil backfill loads unless the soil loads are additive to the overall loadings on a structural element. Then consider internal tensile forces in the wall, and load combinations are as follows:

Flexure and Shear: U = [1.4(DL) + 1.7(LL) + 1.7(Hw)]

U = 1.2DL + 0.8W + 1.0Fa + LL + 1.7Hw

U = 0.9DL + 0.8W + 1.0Fa + 1.6Hw

A.10.4.6 Seismic Water Condition

Earthquake loads will not be considered, in accordance with the 2017 Florida Building Code.

A.10.4.7 Service Soil Condition

For maximum soil backfill height with at rest pressure, with and without internal liquid loads, and the groundwater table is at its normal elevation, include soil compaction or soil surcharge whichever controls the load. Load combinations are as follows:

Flexure: U = 1.3[1.4(DL) + 1.7(LL) + 1.7(Hs)]

Shear: U = [1.4(DL) + 1.7(LL) + 1.7(Hs)]

Flexure and Shear: U = 1.2DL + 0.8W + 1.0Fa + LL + 1.7Hw

U = 0.9DL + 0.8W + 1.0Fa + 1.6Hw

A.10.4.8 Flood Soil Condition

For maximum soil backfill height with at-rest pressure plus hydrostatic pressure of groundwater at 100-year flood level and A-2 Reservoir at maximum full storage level, with internal liquid loads, including soil compaction or soil surcharge whichever controls the load, load combinations are as follows:

Flexure and Shear: U = 0.75[1.4(DL) + 1.7(LL) + 1.7(Hs)]

U = 1.2DL + 0.8W + 1.0Fa + LL + 1.7Hw

U = 0.9DL + 0.8W + 1.0Fa + 1.6Hw

Note: that the one-third allowable stress increase is included in the above equations.

A.10.4.9 Steel Hydraulic Structures

Steel hydraulic structures will be designed in accordance with the Allowable Stress Design Method listed in EM-1110-2-2105 and the AISC Manual of Steel Construction. Allowable stresses will be reduced by 0.83 in accordance with Type B modifications listed in section 4-4 of EM-1110-2-2105. No corrosion allowance will be added to steel cross-sections.

A.10.4.10 Overturning, Sliding and Flotation

Overturning stability, sliding safety factor and the flotation safety factor shall be in accordance with the following values, based on service level loads, and neglecting live loads. (See **Table A.10.4-1**)

Table A.10.4-1. Overturning, Sliding, and Flotation Factors

Aspect	Usual	Unusual	Extreme
Percent of Base in Compression	100	75	Resultant must be within the base
Sliding Safety Factor	2	2	1.33
Flotation Safety Factor	1.5	1.3	1.1

A.10.5 BUILDING STRUCTURES

Building structures, excluding structural concrete, will be designed based upon the loads, load combinations and allowable stresses contained in the 2017 Florida Building Code. Structural concrete design will be based on strength design in accordance with the SFWMD's latest Design Standard and ACI 318-02. The additional concrete design requirements of ACI 350-01 and EM1110-2-2104 will not be considered applicable for building structures unless exposed to water, wastewater or aggressive chemicals such as saltwater. Additionally, building structures and their components that are subject to equipment impact and vibration will be designed in accordance with the applicable recommendations of ACI 350.4R subject to engineering judgment.

Lateral wind loads will be transferred to the foundation from their origin in a rational manner. The horizontal distribution of wind loads will be based upon the assumption that the roof/floor diaphragms are both rigid and flexible for steel deck diaphragms, and rigid for cast in place or precast concrete diaphragms. Where the diaphragm is assumed to behave in a flexible manner, the wind lateral load distribution will be based upon the tributary area to the resisting elements. Where the diaphragm is assumed to behave as a rigid panel, the wind lateral load distribution is based on the relative rigidities of the resisting elements.

A.10.6 INSPECTION REQUIREMENTS

Inspection will be required per the 2017 Florida Building Code, Chapters 1 and 17.

A.10.7 BRIDGES

Two new bridges (Structures B-2 and B-3) will be constructed to carry traffic on U.S. 27 over the new A-2 Reservoir Inflow-Outflow Canal. U.S. Hwy. 27 is a divided highway at the planned location of the new A-2 Reservoir Inflow-Outflow Canal; therefore, dual bridges will be constructed, consisting of a reinforced concrete slab superstructure, supported on two end bents and intermediate bents. Each of the end and intermediate bents will consist of square prestressed concrete piles with reinforced concrete cap beams.

A third bridge (Structure B-1) will be constructed to carry traffic on the L-23 Levee Road (along the east side of the Miami Canal) over the new A-2 Reservoir Inflow-Outflow Canal. This bridge will be designed in a similar manner to the two new bridges for U.S. Hwy. 27 (Structures B-1 and B-2).

The bridge configuration under any conditions should maintain a minimum of two feet of freeboard above the design high water level of the connector canal.

The Bridge Analysis Report and Location Hydraulic Report for the bridge development process have not been completed. Completion of these reports will be made upon approval of the A-2 Reservoir Inflow-Outflow Canal location, size and design. The three proposed bridges will be designed in accordance with AASHTO, Standard Specifications for Highway Bridges, 17th Edition. The bridge will be designed for an HS25 loading.

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A.11 SITE CIVIL DESIGN

A.11.1 PROJECT LAYOUT

The configuration of the A-2 Reservoir and A-2 STA embankments directly affect the total amount of storage available for the A-2 Reservoir. In order to achieve a storage volume of 240,000 acre-feet in the A-2 Reservoir at the lowest possible NFSL using the land available between the Miami Canal and the west side of the existing A-1 FEB, the site plan developed for the project was designed to incorporate existing levees along the east, south and west boundaries of the project site while adhering to the geometrical requirements for levee and dam embankments in DCM-4 (see drawings provided in **Annex C-1**). The site plan includes the A-2 STA with 6,500 acres of effective treatment area and the A-2 Reservoir with slightly more than 240,000 acre-feet of storage at the NFSL of 31.10 ft-NAVD (32.53 ft-NGVD) (which corresponds to an average above ground storage depth of 22.6 feet) and a reservoir footprint area of approximately 10,500 acres. **Table A.11-1** shows the calculated storage volume at the NFSL.

Table A.11-1. A-2 Reservoir Stage-Storage

Reservoir Stage (ft-NAVD / ft-NGVD)	Stage Description	Surface Area (acres)	Cumulative Storage (acre-feet)
45.30 / 46.73	Interior Crest Elevation of Reservoir Dam Embankment	10,804	392,982
31.10 / 32.53	NFSL	10,699	240,348
8.50 / 9.93	Avg. Bottom Elevation of Reservoir (Avg. existing ground surface elevation)	10,571	0

A.11.2 SITE ACCESS, ROADWAYS AND BRIDGES

General access to the A-2 Reservoir, A-2 STA and associated structures will be limited to SFWMD staff and their guests. Public access to the A-2 Reservoir and A-2 STA will only be allowed through designated public access points, as described in **Appendix F**. Public access locations will be designed to support nature based recreation in accordance with SFWMD standards.

Section A.17 provides a description of the permanent access features and roadways to be constructed as part of the project. In general, all of the A-2 Reservoir and A-2 STA embankments will be designed to have a minimum crest width of 14 feet 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. **Section A.10.7** provides a description of three bridges proposed for this project (B-1, B-2 and B-3) and their design requirements and considerations. **Section A.3.3.2** provides a description of the site access to be used during construction.

A.11.3 STORMWATER CONTROL/SITE DRAINAGE

A.11.3.1 During Construction

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 contract documents for the Project’s construction. The objective of the SWPPP will be to prevent erosion where construction activities are occurring, prevent pollutants from mixing with stormwater, and prevent pollutants from being discharged by containing them on-site, before they can affect the receiving waters. The contractors will be required to prepare and submit a comprehensive SWPPP that will be tailored to their sequence of construction. The contractors will be provided conceptual plans, guidelines, and criteria so that detailed drainage plans for all phases and sequences of construction can be prepared. Maintenance of existing agricultural drainage and water supply facilities during construction is discussed in **Section A.3.3.6**.

A.11.3.2 Permanent Construction

The site grading around the A-2 Reservoir pump station (P-1) as well as the three gated spillways (SW-2, SW-3, and SW-4) will include provisions for capturing and treating, where necessary, the stormwater runoff. As shown in typical Sections K(L), J-1(L), L(L) and F(L) provided in **Annex C-1**, the exterior of the A-2 Reservoir dam embankment will include a grassed perimeter swale with 3H:1V side slopes and a bottom width of 12 ft, that will collect runoff from the exterior side slope of the A-2 Reservoir dam embankment. Along Section K(L) the swale will drain to culverts spaced every 1,000 ft that will discharge to the A-2 STA Distribution Canal. Along Section J-1(L) the swale will drain to culverts spaced every 1,000 feet that will discharge to the STA 3/4 Inflow Canal. Along Section L(L) the swale will drain to culverts spaced every 1,000 feet that will discharge to the A-1 FEB. Along Section F(L) the swale will drain to culverts spaced every 1,000 feet that will discharge to the A-2 Reservoir Inflow-Outflow Canal. Stormwater calculations and facilities will be designed to comply with local and State guidelines and regulations.

A.11.4 UTILITIES

A.11.4.1 Electric Power

A description of the existing electric power utilities at the Project site is provided in **Section A.13.1.1**.

The existing electric transmissions lines located along the U.S. Hwy. 27 highway easement, can be used to provide electric power for the proposed A-2 Reservoir Pump Station.

The proposed gated culverts and spillways that are part of the Project, will each require a primary power line. Sources for the power line to each structure will be investigated during PED and at this stage include:

- The transmission line in the U.S. Hwy. 27 easement
- The existing primary line supplying power to the G-720 spillway
- The existing primary line supplying power to the G-372 pump station

A.12 MECHANICAL DESIGN

A.12.1 INTRODUCTION

This section describes the preliminary design of the proposed pump station (P-1) which will serve as the inflow pump station for the A-2 Reservoir as part of the TSP. The pump station facility will be located adjacent to and south of the A-2 Reservoir Inflow-Outflow Canal, and north of the A-2 Reservoir. The A-2 Reservoir Inflow-Outflow Canal is situated between the NNR Canal (L-18) and the Miami Canal (L-23). The pump station will serve as a lifting facility, raising water from the canal to the A-2 Reservoir. This section covers the pump station, approach channel and discharge force mains to the reservoir. When needed, water from the A-2 Reservoir will be returned to the A-2 Reservoir Inflow-Outflow Canal by gravity via gated culvert C-1. The pump station will provide a maximum design flow of 4,600 cfs.

A.12.2 PUMP STATION

A.12.2.1 Design Criteria

The A-2 Reservoir Pump Station will be located near the northeast corner of the proposed A-2 Reservoir. The pump station will provide for screening the flow from the canal to protect the pumps from damage. The pump station shall be equipped with ventilation, air conditioning in electrical spaces and personnel areas. A potable water and sanitary waste system will be provided. A unisex restroom with shower facilities will be provided. A breakroom will also be provided.

Figure A.12.2-1 shows the A-2 Reservoir Pump Station site plan, and **Figure A.12.2-2** shows a section of the A-2 Reservoir Pump Station, through the easternmost intake bay and discharge pipe.

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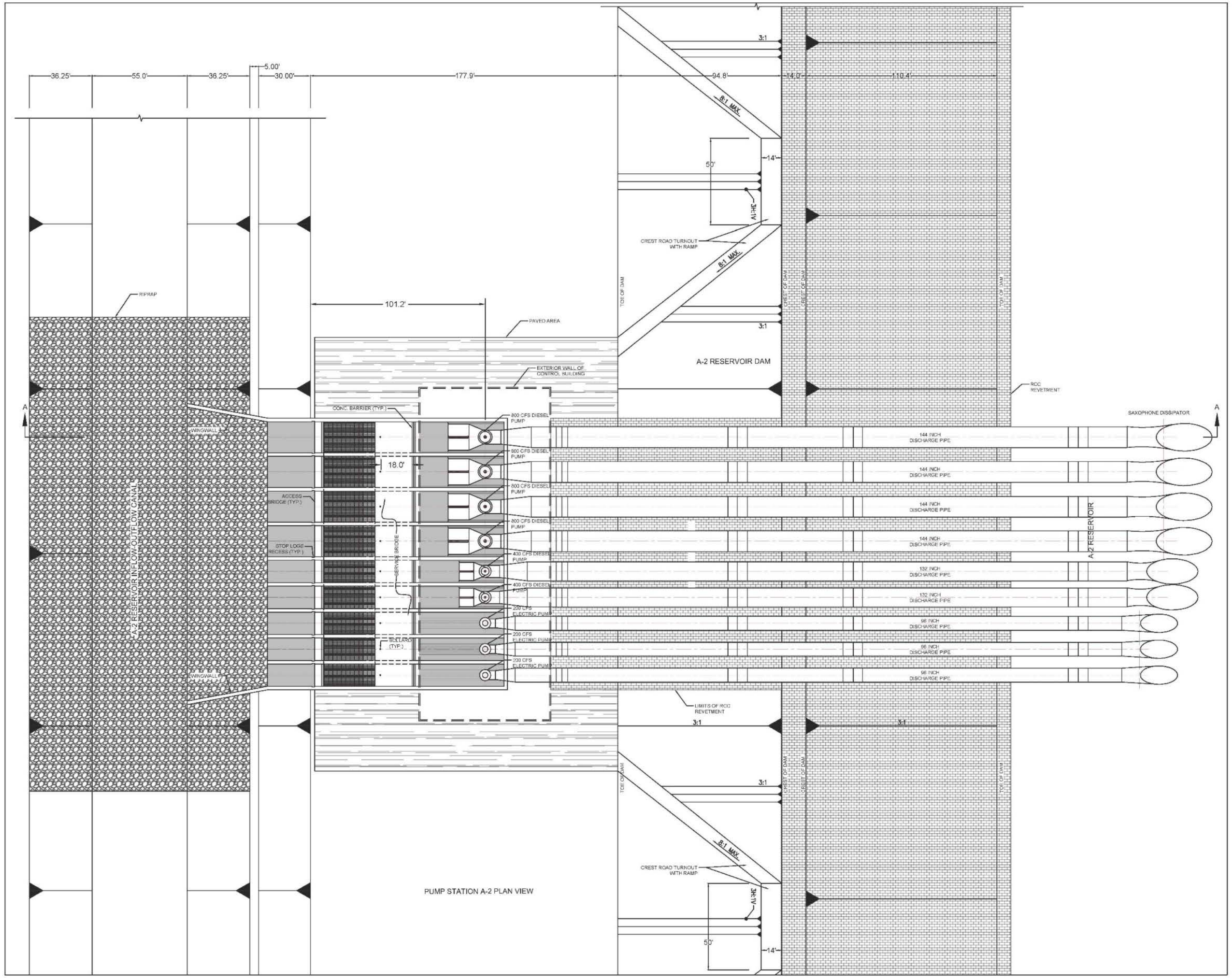

Figure A.12.2-1. A-2 Reservoir Site Plan

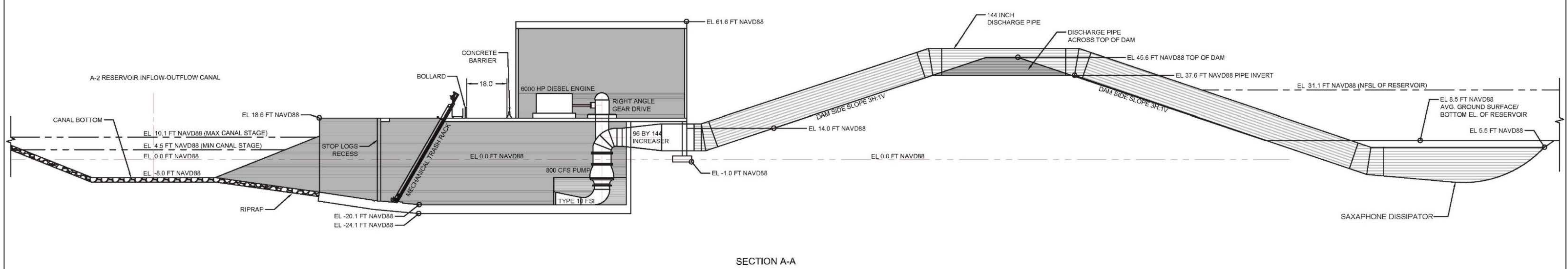

Figure A.12.2-2. A-2 Reservoir Section (A-A)

A.12.2.2 Equipment

- Pump Station Capacity: 4,600 cfs
- Number of Pumps/Bays: 9
- Pump Capacity: Four (4) Units - 800 cfs
  Two (2) Units - 400 cfs
  Three (3) Units - 200 cfs
- Design Static Head, Min/Max: -3.57 feet / 24.5 feet
- Discharge Pipe Invert at Dam "Hump": 37.60 ft-NAVD (39.03 ft-NGVD)
- Pump Configuration: Vertical, Wet Pit, Mixed Flow
- Pump Intake: Suction Bell or FSI
- Pump Driver (800 and 400 cfs): Diesel Engine with Right-Angle Drive
- Pump Driver (200 cfs): Electric Motor
- Engine Fuel Type: Diesel
- Discharge Configuration: Steel Pipe over reservoir dam to submerged outlet
- Trash Racks: Mechanically Cleaned Bar Screens
- Rack Bar Spacing: 3 inch
- Vacuum Priming: Provided on all Discharge Lines
- Vacuum Pump Capacity: TBD
- Air Compressor System: TBD

A.12.2.3 Protection Elevation

The operating floor elevation at the A-2 Reservoir Pump Station should limit the possibility of flood damage to the pump, electrical and ancillary mechanical equipment. As the pump station is situated along the south side of the A-2 Reservoir Inflow-Outflow Canal, and the operating deck is set by the maximum canal elevations plus 4 ft. Maximum water surface in the A-2 Reservoir Inflow-Outflow Canal is set at 10.10 ft-NAVD (11.53 ft-NGVD), and the minimum pump deck is to be 14.50 ft-NAVD (15.93 ft-NGVD).

There are two fundamental methods for a discharge configuration into the A-2 Reservoir, over-the-embankment or through-the-embankment discharge. Over-the-embankment discharge configuration has been selected for this facility, and the criteria for minimum pipe invert is 2 feet above the maximum reservoir design elevation plus 2 feet. Due to design storm rise of 4.5 feet and wave run-up, the top of the proposed A-2 Reservoir embankment is set at elevation 45.6 ft-NAVD (47.03 ft-NGVD), which is 37.1 feet above the average existing grade elevation of 8.50 ft-NAVD (9.93 ft-NGVD) at the A-2 Reservoir site and 16.5 feet above the normal maximum pool stage of the A-2 Reservoir of 31.1 ft-NAVD (32.53 ft-NGVD). Therefore, for the over-the-embankment discharge configuration, the pipe invert minimum elevation is 35.6 ft-NAVD (37.03 ft-NGVD). Clearances necessary for below the base plate pump discharge and its coupling may be the critical dimension and must be coordinated with the pump manufacturer.

A.12.2.4 Connector Canal Considerations

The connector canal to the northeast pump station shall intersect the A-2 Reservoir Inflow-Outflow Canal at 90 degrees and proceed a short distance south, crossing a buffer berm and to the northeast corner of the A-2 Reservoir. The ideal hydraulic condition is for the connector canal to be in line with the intake centerline. The flow approaching the pump intake should ideally be steady and uniformly distributed both laterally and vertically. Due to the perpendicular configuration of the connector channel to the RIOC, there will be intermittent situations where a transverse velocity in the A-2 Reservoir Inflow-Outflow Canal will create non-uniform flow profiles in the connector channel. In practice it is not possible to completely eliminate non-uniform or unsteady flow conditions. The A-2 Reservoir pump station wingwalls should be at an angle of no more than 10 degrees from the centerline. It should also be noted, a surface drop can occur across a partially blocked trash rack, or whenever the pumps have lowered the water level in the sump to the point at which all pumps are about to be switched off. Therefore, the path between the sump entrance and the pump inlets must be sufficiently long for the air bubbles to rise to the surface and escape before reaching the pumps.

A.12.2.5 Mechanical Arrangement Considerations

The mechanical design of the pump station includes pumps, drivers and appurtenances necessary to provide a functional and reliable system. The conceptual design is intended to provide satisfactory hydraulic configuration, good working access around equipment, crane access to systems and to vehicles. The design is intended to follow DCM-5. The flow to the station is taken from the A-2 Reservoir Inflow-Outflow Canal through a wide, multi-channel, screened approach to the individual pumps.

A.12.2.6 System Analysis of Pump Station

A.12.2.6.1 System Design Requirements

This facility considers both USACE EM 1110-2-3105 and SFWMD Design Criteria Memorandum: DCM-5 in the design of systems and selection of appropriate components. The pump design must take into account the range of static head generated by the elevation in the supply canal and the reservoir from empty to full. For this facility, it represents a wide range compared to most facilities in the South Florida region. There is also the capability to vary capacity on the engine-driven pumps. The result of the combination of friction and static head variations represent a challenge for vertical wet pit axial or mixed flow pumps. The design of this facility also includes a high point in the discharge piping that will preclude backflow to the supply canal based on the maximum reservoir design elevation plus a design year storm rise. This presents a starting condition that the pumps and engines must be able to overcome before a siphon is established to reduce the operating head. This determines the pump design condition and the horsepower required of the engine for successful operation. In addition, it is desired that the pumps operate within an “Acceptable Operating Range” as previously defined by the Hydraulic Institute in Section 9.6.

A.12.2.6.2 System Analysis

The pump static head design conditions are a function of the operating levels in the A-2 Reservoir Inflow-Outflow Canal and the operating levels in the A-2 Reservoir. The following table summarizes the canal and reservoir range of elevation conditions.

Table A.12.2-1. Pump Station Canal and Reservoir Conditions

Maximum Canal Stage (ft-NAVD / ft-NGVD)	Minimum Canal Stage (ft-NAVD / ft-NGVD)	Maximum Reservoir Stage (ft-NAVD / ft-NGVD)	Minimum Reservoir Stage (Empty) (ft-NAVD / ft-NGVD)	Minimum Reservoir Stage (Drought) (ft-NAVD / ft-NGVD)	Pipe Invert at "Hump" (ft-NAVD / ft-NGVD)
10.10 / 11.53	4.50 / 5.93	31.10 / 32.53	8.50 / 9.93	6.50 / 7.93	37.60 / 39.03

The pipe invert at the top of the dam (hump) is based on the maximum reservoir elevation of 31.10 ft-NAVD (32.53 ft-NGVD) at pump shut-off, plus a design rain event resulting in a rise of 4.5 feet, and a 2-foot buffer. Wave action is not considered in this calculation.

Table A.12.2-2. Pump Static Head

Maximum Static Head with Siphon (ft)	Minimum Static Head with Siphon (ft)	Maximum Static Head Over "Hump" (ft)	Minimum Reservoir Stage (ft-NAVD / ft-NGVD)	Pipe Invert at "Hump" (ft-NAVD / ft-NGVD)
26.60	-3.57	39.10 (for 12' Dia. pipe) 38.60 (for 11' Dia. pipe) 37.10 (for 8' Dia. pipe)	8.50 / 9.93 (Empty) 6.50 / 7.93 (Drought)	37.60 / 39.03

The maximum static head is based on the minimum canal elevation of 4.5 ft and a reservoir shut-off elevation of 31.1 ft-NAVD (32.53 ft-NGVD) and a down-leg siphon. Maximum static over the hump for each pipe size is based on water elevation in the pipe when half full. Minimum static head is surface-to-surface between the canal and the reservoir in drought conditions and with a siphon established. Conversations with pump manufacturers suggest that in that condition, not establishing the siphon for a period of time may be preferable. The siphon condition currently assumes that the downleg pipes are flowing approximately 2/3 full when vacuum is applied. This can be adjusted when final piping geometry is known.

Table A.12.2-3. Fitting Friction Factors (K)

FSI	90° Elbows	45° Elbows	Increaser	Outlet Loss
K = 0.15	K = 0.30	K = 0.23	K = 0.52-0.63 Variable	K = 1.0

Friction losses are calculated based on a fitting-specific factor (K) multiplied by the velocity head in that fitting. Velocity head is computed by the formula:

$$V_h=V^2/2*g$$

Where: V_h is the velocity head for that pipe/fitting diameter

V is the velocity based on flow and diameter
g is the acceleration of gravity (32.2 ft/s^2)

For initial calculations, the outlet loss has been assumed as a square outlet, which is a worst case. This will be revised to a "Saxophone Outlet" when pump information is received and reviewed. The K factor for the increaser is calculated based on specific geometry and the ratio of diameters. Pipe flow losses are computed assuming full pipe flow, using the Williams and Hazen formula. This formula, solved for head loss per thousand feet, is:

$$s = \left[\frac{v}{c * r^{0.63} * 0.001^{-.04}}\right]^{1.85}$$

Where: s is head loss
v is velocity
c is the roughness coefficient
r is the hydraulic radius (D/4 for round conduits flowing full)

Table A.12.2-4. Friction Losses at Maximum Capacity

Capacity (cfs)	96" FSI Entrance (ft) [1 ea]	96" 90° Elbow (Ft) [1 ea]	96"X144" Increaser [1 ea]	144" 45° Elbow (ft) [5 ea]	144" Outlet Loss (ft) [1 ea]	144" Pipe Loss (ft)	Total (ft)
800	0.59	1.38	0.41	1.36	0.78	0.2	4.72
	84" FSI Entrance (ft) [1 ea]	**84" 90° Elbow (ft) [1 ea]**	**84"X132" Increaser [1 ea]**	**132" 45° Elbow (ft) [5 ea]**	**132" Outlet Loss (ft) [1 ea]**	**132" Pipe Loss (ft)**	**Total (ft)**
400	0.25	0.59	0.16	0.48	0.28	0.1	1.85
	60" Flare Inlet (ft) [1 ea]	**60" 90° Elbow (ft) [1 ea]**	**60"X96" Increaser [1 ea]**	**96" 45° Elbow (ft) [5 ea]**	**96" Outlet Loss (ft) [1 ea]**	**96" Pipe Loss (ft)**	**Total (ft)**
200	0.06	0.56	0.15	0.43	0.25	0.1	1.58

Based on the information above, the friction losses are small in comparison to the maximum static head, and particularly the dry starting head. In addition, the minimum head when the reservoir is empty presents an extreme range for the pumps to handle. These calculations are from the pump suction flange to discharge nozzle, internal losses are to be included in the manufacturer's performance curves. It is noted that the velocity in the 144 inch pipe on the 800 cfs pumps is higher than normally desirable, but larger pipe is not available, and splitting the flow into two pipes on the large pumps makes the footprint of the station and piping over the dam excessive.

A.12.2.6.3 Pump Performance Requirements

Table A.12.2-5. Pump Design Capacity at Rated Conditions (Running)

Pumps	Capacity (cfs/GPM)	Velocity Head (ft)	Static Head (ft)	Friction Head (ft)	Pump Losses (ft)	Total Head (ft)	Brake HP
Large	800/360,000	0.8	26.6	4.7	2.5	34.6	3,996
Medium	400/180,000	0.3	26.6	1.9	2.5	31.3	1,824
Small	200/90,000	0.2	26.6	1.6	2.5	30.9	860

Above values based on operation with siphon in effect.

Table A.12.2-6. Pump Design Capacity at Rated Conditions (Starting)

Pumps	Capacity (cfs/gpm)	Velocity Head (ft)	Static Head (ft)	Friction Head (ft)	Pump Losses (ft)	Total Head (ft)	Brake HP
Large	800/360,000	0.8	39.1	3.3	2.5	45.7	5,091
Medium	400/180,000	0.3	38.6	1.3	2.5	42.7	2,466
Small	200/90,000	0.2	37.1	1.1	2.5	40.9	1,181

Above values based on starting without siphon in effect, full pipe to hump invert.

A.12.2.6.4 Model Studies

Based on the size and capacity of this pump station, modelling of the intakes and connector canal is recommended. The HI standard ANSI/HI 9.8 - 2009 recommends intakes of pump stations with an individual pump capacity exceeding 40,000 gpm, or non-uniform flow to the pump sump be modelled. However, the designer must decide the necessity of a model study on a case by case basis.

Experience has shown that modeling of pump intakes can predict issues and use of the model to simulate physical solutions to issues can result in prevention of problems in the full size facility. Modelling relies on dimensional analysis and the laws of similitude or similarity. These laws permit the application of certain relations by which the test data can be applied to other cases. The laws of similitude make it possible to predict the performance of the prototype from tests made with a model.

Geometric similarity means the model and the prototype are identical in shape but differ only in size. The scale factor or the ratio of the linear dimensions of the prototype to the corresponding dimensions of the model is an important consideration to ensure an accurate model.

If two systems are dynamically similar, corresponding forces must be in the same ratio. Dynamic similitude is achieved when two flow systems which are geometrically similar satisfy the dimensionless equation of motion. Any deviation is termed a scale effect. The dimensionless terms that must have the same value in both flow systems include:

- Relative submergence = h8 / ro
- Circulation number = Gn = Gro / Q
- Froude number Fn = (Q / ro h8) / (g h8)0.5
- Reynolds number = Rn = Q/v h8

The objective of a model study is to ensure the intake design generates favorable flow conditions in the inlet to the pump. Intake models are operated using Froude similarity since the flow process is controlled by gravity and inertial forces. In modeling an intake it is important to select a reasonably large geometric scale to minimize viscous and surface tension scale effects and reproduce the flow pattern in the vicinity of the pump. The model must be large enough to allow visual observations of the flow patterns, accurate measurements of swirl and velocity distribution and sufficient dimensional control.

Comparison of model to prototype in regard to vortex formation indicates negligible scale effects for Froude scaled models with weak vortices and surface dimples. Some scale effects were detected for models in which air core vortices occurred. Compensation for these scale effects is possible by some increase in model flow above the Froude scaled value. It is important the Reynolds and Weber numbers be sufficiently high to avoid the potential of scale effects. Models at higher scale ratios yield higher Reynolds and Weber numbers at the same Froude number.

A.12.2.7 Station Mechanical – Major Equipment and Auxiliary Systems

A.12.2.7.1 Axial Flow Pumps

A.12.2.7.1.12 General Design Requirements

The pump equipment should be designed for intermittent service which is a normally idle piece of equipment that is capable of immediate automatic or manual start-up and continuous operation. The pump equipment including auxiliaries shall 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 1,500 hours, with the majority of this operating time requiring continuous operation for several days. The characteristic of flow to the pumps includes storm water that may contain sand, silt, and floating or transported debris capable of passing the trash rack. Water temperature range should in the range of 80 to 90 degrees F.

The pump should be designed to facilitate routine and heavy maintenance. ANSI/HI 2.4-2000 provides guidance for the installation, operation, and maintenance of vertical pumps. Major parts, such as the bowl components, should be designed and manufactured to ensure accurate alignment on reassembly. For vertical pumps with bell intakes, the propeller should be removable from bottom of pump bowl without dismantling pump. The larger pumps in this facility will operate through formed suction inlets, requiring maintenance to be performed from the pump deck.

A.12.2.7.1.13 Dynamic Analysis

The pump manufacturer will be required to provide the following analysis to ensure the critical speed of the pump does not coincide with the rated operating speed.

A.12.2.7.1.14 Lateral Critical Speed

The manufacturer shall determine the lateral (dry) critical speed of the pump rotor using static deflection calculations as described in ANSI/HI 9.6.4.2.1 - 2000. A critical speed shall not occur within 25 percent above or below the rated operating speed of the pump.

A.12.2.7.1.15 Torsional Critical Speed

The manufacturer shall determine the torsional (dry) critical speed of the pump rotor using manual calculation methods as described in ANSI/HI 9.6.4.2.3 - 2000. A critical speed shall not occur within 25 percent above or below the rated operating speed of the pump.

A.12.2.7.1.16 Lateral Dynamic Analysis

A lateral dynamic analysis shall be performed for each pump on this project. Prior to manufacture of any equipment, the pump manufacturer and the engine manufacturer in accordance with the ANSI/HI 9.6.4.2.2 - 2000 shall determine the critical speeds of the equipment in the lateral directions. A natural frequency that occurs within 25 percent above or below the rated operating speed of the pump will not be accepted. The dynamic analysis model shall be constructed using a commercially available program that uses finite element analysis methods. The system shall be analyzed at the run (wet) condition considering the effect of water mass in the column and the damping effect of the highest and lowest sump water levels. The model shall incorporate the critical frequency and mass elastic diagram information provided by the gear manufacturer. The completed dynamic analysis report shall be submitted to the Engineer prior to start of fabrication.

A.12.2.7.1.17 Torsional Dynamic Analysis

A torsional analysis shall be performed for each of the A-2 Reservoir Pump Station pumps. Prior to manufacture of any equipment in accordance with ANSI/HI 9.6.4.2.4-2000, the pump manufacturer shall determine the torsional critical speed characteristics of the equipment, including the pump and driver rotational inertias, pump and driver shaft rigidities and inertias and the rigidities of all other rotating equipment in the drive train between the pump and the driver. The analysis shall be performed using a finite element analysis method commercially available with the mass elastic information provided by the pump and gear drive manufacturers. A torsional critical speed that occurs within 25 percent above or below the rated operating speed of the pump and the driver will not be accepted. The completed dynamic analysis report shall be submitted to the Engineer prior to start of fabrication.

Pump Components

The following components are part of the design used in this pump station. Some details of construction will vary with the manufacturer, and specific details will be required from the manufacturer in the form of cross-sectional drawings identifying internal configuration. These drawings shall identify materials of construction with federal spec references (ASTM, ANSI, ASME, etc.)

Component Design Criteria

Base Plate

Vertical pumps of this size and capacity are provided with a steel base plate, designed to transmit the static weight of the pump and angle gear reducer and dynamic forces generated by the mechanical components to the underlying structure. Base plate includes provisions for bolting of the mechanical components and for anchoring to the concrete structure underneath. The plate shall have a concentric opening sufficient to remove the assembled pump, less the FSI. Proper design of the base plate is required to preclude harmonic vibrations due to resonance induced by the mechanical system. Base plates of this size typically have reinforcing gussets beneath the plate for enhanced strength and stiffness. It is also important that the base plate achieve uniform bearing on the underlying structure.

Drive Pedestal

A drive pedestal is generally provided between the motor or gearbox and the pump, mounted on the base plate to provide access to the shaft seal and shaft coupling, if so equipped. The pedestal is fabricated steel and designed to support the weight and dynamic loads of the motor or gear box. The floor of the pedestal serves as a sump to catch and contain seal leakage. Two or more openings are provided in the wall of the pedestal to provide access to the coupling/seal, and equipped with safety guards. The upper and lower plates of the pedestal are machined flat and parallel, with a register fit to assure concentricity of the vertical components. The pedestal is provided with the pump, and the manufacturer is responsible for assuring accurate fit between pump and driver. The pedestal has fastener holes that align with mating holes in the base plate.

A.12.2.7.1.18 Discharge Column and Nozzle

The pump head assembly is connected to the baseplate by a fabricated steel column, designed to direct flow from the pump head to the discharge nozzle and to provide support for the shaft. The column diameter is sized to limit the liquid velocity and associated friction losses. The column also serves to support and align the bearing spiders that hold the bearings that guide and stabilize the shaft. The ends of the column are typically attached with flanges with a register fit to the bowl assembly and the base plate to assure alignment. The column section(s) shall have lifting lugs, properly gusseted to support the sections without deformation. The upper end of the column is a solid section with a penetration for the shaft, and associated seal or stuffing box.

The design of this station will utilize a “Below Deck” discharge configuration, consisting of a plain end nozzle with thrust restraint lugs. The nozzle will have a connection for an air release valve to vent the column during startup or if air is aspirated during operation. Nozzle will be a mitered design with at least 5 segments to smooth the flow around the turn.

An inner column tube shall enclose the column shaft, providing for lubrication flow and supporting shaft sleeve bearings. The inner column shall be mechanically connected to the upper end of the pump head and to the underside of the drive pedestal. A tension nut shall be located at the upper end of the inner column to assure alignment in the column and around the shaft. The lower end of the inner column shall have a throttle bushing to limit excessive relief of the lubrication water in the column.

A.12.2.7.1.19 Pump Head

The pump head consists of three components, the inlet, the propeller bowl, and the diffuser. The components are bolted together with register fits for alignment and support the shaft over its length. The inlet usually includes a bell-type suction fitting unless an FSI is used, in which case it is flanged to the FSI. The inlet also includes the foot bearing, supported by a spider, which supports the outboard end of the propeller shaft. The propeller bowl contains the impeller and is shaped to provide an efficient flow pattern and close running tolerance with the propeller periphery. The diffuser is fitted with multiple vanes that straighten the flow exiting the propeller, creating an axial flow pattern in the column. The vane count should not be an even multiple of the number of propeller vanes to reduce hydraulic resonance in the pump head due to vane passing frequencies. The upper pump head bearing is mounted on a spider in the diffuser, maintaining the running clearances in the pump head. The propeller is supported in the pump head by the headshaft that extends from the foot bearing to above the diffuser bearing, where it couples to the column shaft.

The pump propeller shall be of axial or mixed flow design, depending on the required head on the pump. Vanes shall be smooth in order to provide maximum efficiency and fabricated unit dynamically balanced. Propeller shall be retained on the shaft by both axial and radial keys. Propeller shall be polished, with smooth flow surfaces.

A.12.2.7.1.20 Shafting

Shafting shall be machined from Type 316L stainless steel, and shall be sized to handle the full rated horsepower of the driver as well as total dead and thrust load of the rotating assembly, with a conservative safety factor. Shafting shall be machined and polished over the full length.

Shafting shall be manufactured in accordance with ASME B106.1M – Design of Transmission Shafting, for a safety factor of 5.0 based on ultimate tensile strength of the shaft material and the rated horsepower of the engine; also, 75 percent of the yield strength of the shaft material and the maximum horsepower of the engine. The shaft stiffness shall limit deflections under the most severe dynamic conditions over the allowable operating range of the pump in accordance with the performance requirements of the shaft seals and bearings. The running clearances shall be sufficient to ensure dependability of operation and freedom of seizure under all specified operating conditions. All shafts shall be designed to operate within the allowable vibration tolerances in the preferred operating region and ensure the lateral and torsional first critical speeds occurs 25 percent above or below the rated pump speed.

A.12.2.7.1.21 Shaft Sleeves

Shaft sleeves shall be provided at the seal/stuffing box, and at each sleeve bearing to provide a renewable surface without replacing the shaft. Sleeves shall be pressed on the shaft and locked with pins or threaded dowels. Other locking means may be submitted and evaluated. The surface finish of the sleeve shall be at least 16 micro-inch RMS for seals and 32 micro-inch RMS at bearings. Surface finish requirements may be increased if required by the seal and bearing suppliers. Surface finish requirements shall be the responsibility of the pump manufacturer.

A.12.2.7.1.22 Shaft Seals

Shaft seals shall be lip-type with stainless steel lip, and shall be sealed to the stationary component.

A.12.2.7.1.23 Shaft Couplings

Shaft couplings shall be rigid and keyed to the shaft ends and of the same material as the shafting. Couplings shall have a torque transmission capacity at least equal to that of the shaft. Coupling machining shall assure concentricity of adjacent shaft ends, and the finished couplings shall be factory balanced. Coupling bore and exterior surface shall be polished. Coupling spacing shall be as determined by the pump manufacturer, based on the shaft design. Coupling design shall be subject to review and approval.

A.12.2.7.1.24 Bearings

Shaft column bearings shall be water lubricated (hydrodynamic) design of a non-metallic synthetic polymer alloy Thordon SXL or approved alternate. The bearing design, and running tolerance between bearing and shaft sleeve shall be the responsibility of the pump manufacturer. Bearing spacing and tolerance shall prevent lateral harmonic vibration and excessive runout. Bearings shall be field replaceable. The bowl suction bearing shall be a sand cap to exclude sand and grit from entering.

A.12.2.7.1.25 Stuffing Box

The pump shaft shall be sealed at the drive pedestal floor with a water-lubricated or grease-lubricated stuffing box. Access to the stuffing box shall be through openings in the drive pedestal, which shall be sized to allow ample maintenance clearance. Seal shall be provided by multiple rings of braided packing which are compressed by a follower gland at the upper box opening. A split lantern ring shall be located near the midpoint of the packing rings, with an external lubrication and relief fitting.

A.12.2.7.1.26 Materials of Construction

Materials for the pump must be resistant to abrasion as well as corrosive/brackish waters that come into contact with the operating components. Materials must be compatible with fabrication techniques used in the manufacture of the pumps. All materials used should be subject to applicable federal specifications similar to those indicated in **Table A.12.2-7**. Minimum material requirements are listed in the table.

Table A.12.2-7. Pump Material Specifications

Component	Material Specification
Base plate	Carbon Steel—ASTM A36
Discharge column and elbow	Carbon Steel—ASTM A283 Grade C or A516 Grade 70
Drive pedestal	Carbon Steel—ASTM A36
Pump head components	Cast Iron—ASTM A48 Class 30
Suction bell or inlet casting	Cast Iron—ASTM A48 Class 30
Shafting	Stainless Steel—ASTM A276 Type 316L
Shaft couplings	Stainless Steel—ASTM A276 Type 316L
Inner column	Stainless Steel—ASTM A276 Type 316L
Shaft sleeves	Stainless Steel—ASTM A276 Type 316L
Propeller	Cast Copper Alloy—ASTM B584-C87500
Packing Gland	Stainless Steel—ASTM A743 Type 316L
Nuts, bolts, dowels, keys, fasteners	Stainless Steel—ASTM A193 Type 316L

A.12.2.8 Diesel Engine Drivers

A.12.2.8.1 General Description and Design Requirements:

Based on the site of the location of the A-2 Reservoir Pump Station, there is not sufficient available electrical capacity to use electric pumps on the larger pumps. Based on this, the 400 and 800 cfs pumps will be operated on diesel engines. The 200 cfs pumps will be powered with electric motors. When used for driving vertical, axial/mixed flow wet pit pumps, the diesel engine couples to a right-angle gearbox (drive) through a short horizontal drive shaft with universal joints on each end. A power take-off and clutch assembly is typically provided to disengage the engine for service and to allow starting of the engine without the couple load. The engine can be remotely monitored and operated via telemetry, typically radio-based on more remote locations. Local manual starting and monitoring is provided. The engine's electronic control module typically outputs engine parameters and functions via a data path to a programmable logic controller.

The engines will be compression-ignition (diesel) type, four cycle for stationary applications. In this size range, the engines are typically selected from marine offerings. The engine is usually a horizontal, in-line design with six or eight cylinder, or a "V" configuration with eight to sixteen cylinders. The engine will be of cast iron construction and either naturally aspirated or turbocharged and aftercooled. Engines will be current models of a type in regular production with all devices specified or normally furnished with the engine. The engines selected will be sized for the pumping system requirements, including the additional intermittent power required to initiate flow over the discharge line high point at the top of the dam. The engine models proposed shall be current commercial or marine service unit with a satisfactory service record of not less than 36 months operating at least 1,200 hours/year under similar or more severe duty conditions. The engine manufacturer/supplier shall provide a fully-outfitted package including, but not limited to: intake air filtration; starting system; fuel system; cooling system; monitoring and control components; and engine exhaust system. The engines will be started with compressed air and cooled via a liquid-to-liquid heat exchanger or a factory provided radiator and engine-driven fan.

The output power to be delivered by the engine will be based on the input power required by the pump and transmission throughout the pump curve from shut off head to the maximum operating flow range as determined by the pump manufacturer. The engine shall not be over loaded through pump's allowable operating region. The engine's output power shall be determined by the engine manufacturer in coordination with the pump manufacturer. It should be noted the engine power ratings are based on the total power output capability at the flywheel. The required engine output shall include the horsepower requirements of the engine auxiliaries.

The engine will generally be started with the pump engaged. The engine manufacturer in coordination with the pump manufacturer, shall ensure the engine proposed has adequate accelerating torque under full load start-up conditions (additional torque required above normal operating torque) for the pump to attain the rated speed in a reasonable amount of time. The engine manufacturer shall also ensure there are no damaging overload conditions during the engine's warm-up period.

A.12.2.8.1.12 Engine Rating

The engine service shall be "Continuous Duty" intended for continuous use for load application requiring uninterrupted service at full power. The standard reference conditions, methods of declaring the power, fuel consumption, lubricating oil consumption, and test methods for diesel engines is in accordance with applicable sections of ISO 3046 for the conditions listed below. The basis for gross engine power rating, methods for correcting observed power to reference conditions and the method for determining gross full load engine power with a dynamometer is SAE J1995.

A.12.2.8.1.13 Project Site Conditions

- Maximum air temperature: 105 degrees F
- Minimum air temperature: 35 degrees F
- Maximum raw water temperature: 90 degrees F
- Minimum raw water temperature: 60 degrees F
- Elevation: sea level
- Relative humidity: 80 percent

A.12.2.8.1.14 Engine Speed at the Rated Condition

The engine speed should be selected by review of the available engine models for the required horsepower/torque range of operation. For small to medium sized engines, (< 600 Hp) the rated speed is typical 1,800 to 2,100 rpm. The engine speeds for the larger 800 cfs pump will probably be in the range of 1,000 rpm.

A.12.2.8.1.15 Fuel Requirements

Again the designer is limited to the standard engine models which operate on 2 or 2-D, (regular) diesel fuel oil, 40 cetane, (minimum), ASTM D396 and ASTM D975.

A.12.2.8.1.16 Fuel Consumption

The standard reference conditions and methods of declaring fuel consumption shall be in accordance with applicable sections of ISO 3046. Typically the fuel consumption rate shall not exceed 0.45 pounds per bhp-hour between 75 percent and 100 percent of rated full load for the following conditions:

- Fuel heat value: 19,350 BTU

- Unit elevation: sea level unless otherwise noted.
- Intake air temperature: 90 degrees F
- Barometric pressure: greater than 28.25 inches mercury

A.12.2.8.1.17 Emissions Requirements

The Environmental Protection Agency (EPA) emissions regulations for stationary diesel engine applications are being drafted for the finished installation to comply with EPA Tier 2, Stage II emissions requirements. Engine manufacturer's are well aware of these requirements and have had to deal with them for their on- road engines. This technology has on many models been adapted to the modern stationary production models. The designer should specify the EPA regulation requirement to ensure the Contractor satisfies the most current EPA requirements. The engines used in this analysis are Tier I compliant.

A.12.2.8.1.18 Engine Electronics

The engine’s electronic control module provides monitoring of vital engine parameters and control of engine operation. The system regulates emissions and optimizes fuel economy and provides condition monitoring to prevent engine damage. The electronic control system has programmable speed control. The electronic package also provides standard data-link to a logic controller for manual and remote monitoring and operation via telemetry facilities. A standard factory supplied engine control and monitoring panel can be specified for manual operation.

A.12.2.8.1.19 Rotation

The rotation of the engine should be the SAE standard rotation with the speed reducer and pump to match this rotation. This direction, looking towards the front of the engine, is anti-clockwise.

It is intended that the engine deliver power in one direction only and an anti-reverse rotation device shall be provided by the reduction gear to prevent reverse rotation by the backflow of water through the pump at shut down.

A.12.2.8.1.20 Engine Mounting

Complete equipment foundation plates, sole plates, mounting straps, brackets or structural bases with suitable anchor bolts, nuts, sleeves, washers and shims or wedge plates and vibration dampers or isolation blocks should be furnished as required. Resilient mounts should also be provided and should be capable of fully restraining the engine and limiting its motion under acceleration induced forces and torque reactions. The engine mounts shall be capable of alignment and leveling. It is important the Contractor coordinate the foundation requirements with the various trades for the anchorage and foundation details to be provided.

A.12.2.8.1.21 Exhaust System

A complete and separate exhaust system should be provided for each engine. The engine exhaust system piping should be provided and laid out with the shortest and straightest runs possible consistent with the location of the exhaust silencers in relation to the engines. Sharp bends shall be avoided by the use of long sweep fittings wherever practical. Horizontal sections of the piping shall be sloped downward away from the engine to a condensate trap and drain valve.

A.12.2.8.1.22 Piping

All piping should be 304 stainless steel in accordance with ASTM A240/A240M. All pipe sections should be flanged where practical. Piping smaller than two inches in diameter should be Schedule 80. Piping with a diameter of two inches or larger should be Schedule 40. The vertical exhaust piping shall be provided with a hinged, gravity-operated, stainless steel, self-closing cap. Thermal expansion and/or vibration shall be addressed by a short length or lengths of an approved multi-ply stainless steel bellows type flexible sections at each engine. Suitable stainless steel sleeves with retainer rings should be provided together with suitable packing for wall penetrations to allow free movement of the pipe in accordance with NFPA 37.

A.12.2.8.1.23 Supports

Pipe supports for the exhaust lines and braces for the exhaust silencer and tailpipe shall be provided as necessary. Pipe hangers shall be in accordance with MSS SP-58 and MSS-69. The designer may want to provide details of the supports and hangers in the construction contract drawings to ensure a quality installation.

A.12.2.8.1.24 Exhaust Silencer

The designer needs to review the local noise ordnances to ensure compliance of the proposed installation. Unless required otherwise, the noise level taken three feet from the silencer shall not exceed 86 dBA. The exhaust silencer should be at a minimum a critical grade chamber type exhaust muffler mounted on the exterior of the pump station building. The exhaust silencer, support, and miscellaneous fasteners should be ASTM A276 type 304 stainless steel. The designer should provide the silencer support details. The engine manufacturer needs to provide input to the Contractor for the proper selection of the silencer that will provide the most effective system considering; noise levels generated, pressure drop and physical size of the silencer.

A.12.2.8.1.25 Exhaust Line Insulation

All exhaust lines for the engines inside the building need to be insulated with not less than three inch thickness of ASTM C 533 calcium silicate insulation. The insulation shall be secured with stainless steel bands and covered with an aluminum jacket. The aluminum jacket should overlap not less than three inches longitudinal and circumferential joints and should be secured by bands at not more than 12-inch centers. Longitudinal joints shall be overlapped down. Circumferential joints should be sealed with a coating that is recommended by the insulation manufacturer. Aluminum should be smooth sheet 0.016-inch nominal thickness and have a factory applied polyethylene and kraft paper moisture barrier. At pipe flanges and expansion joints, the insulation at each side of the flanged connection should be tapered for a short section to permit removal of bolts without disturbing the insulation.

A.12.2.8.1.26 Air Intake System

A complete and separate air intake system shall be provided for each engine. The contractor shall be responsible for the design and installation of the air intake system in accordance with the engine manufacturer’s requirements and the project site conditions specified above.

A.12.2.8.1.27 Air Intake Filter

The air intake filter for each engine shall consist of high-efficiency, washable paper elements packaged in a low restriction waterproof housing. The filter shall be provided in a location convenient for servicing.

A.12.2.8.1.28 Inline Silencer

For turbo-charged engines, an inline silencer shall be provided on the air intake. The silencer shall be of the high frequency filter type. A combined filter silencer unit meeting the requirements for the separate filter and silencer items may be provided.

A.12.2.8.1.29 Engine Cooling System

The system can either be a closed or flow through system depending on the engine size and the cost of the systems. The decision of the cooling system type shall be determined at a later design phase. For any system specified, the contractor shall be responsible for the details of the design and installation of the system in accordance with the engine manufacturer's requirements and the project site conditions specified. For this analysis it was assumed the cooling water system is a flow through system with the cooling water provided by the station's service water system which includes turbine pumps and a filtration system.

The cooling water system for each engine should operate automatically while the engine is running. The closed cooling water system typically have an engine driven jacket water pump, a submerged pipe heat exchanger (Keel Cooler or equal), expansion tank, and an automatic temperature regulating valve. The cooling system shall be designed for the maximum raw water temperature and the maximum ambient temperature. The system circulates jacket coolant through the engine at the temperature and flow rate recommended by the engine manufacturer. The coolant is typically an ethylene-glycol water mixture. The engine driven jacket water pump forces water through the engine cooling passages, the heat exchanger, expansion tank, and back to the pump. The pump is typically the manufacturer's standard centrifugal type pump properly sized for the intended purpose.

For the closed system, each engine cooling system shall include pipe or coil submerged type heat exchanger, (Keel Cooler or equal) located on the wall of the intake. The heat exchanger shall be of ample capacity to match the engine with maximum water temperature in the intake. The jacket water shall flow from the engine to the cooling coils and then to the expansion tank before returning to the jacket water pump inlet. The temperature rise of the coolant across the engine shall not exceed the recommendations of the engine manufacturer.

Each engine cooling system shall include one thermostatically controlled proportioning valve of appropriate size and temperature rating installed at the after cooler and bypass line. The valve shall be complete with automatic control element. A bypass with an automatic temperature regulator shall be installed around the heat exchanger so that the temperature of the jacket water may be regulated.

Each engine shall be equipped with a coolant temperature sensor and coolant level sensor. The temperature sensors shall provide signals for coolant temperature indication and high coolant temperature alarms.

A jacket water expansion tank shall be furnished for each engine. The tank shall be of welded steel construction and shall be hot dipped galvanized inside and out after fabrication. The tank shall have a capacity of not less than 10 gallons and shall be suitable for an operating temperature of 250 degrees F and a working pressure of 125 pounds per square inch gauge (psig). The tank shall be tested and stamped in accordance with ASME BPV VIII Div 1 and registered with the National Board of Boiler and Pressure Vessel Inspectors. The tank shall be properly fitted for vent, overflow, expansion, and make-up lines and mounted so the bottom of the tank is above the top of the engine. A brass water gage with valves shall be provided on the tank. The Contractor shall submit the details of the tanks support for approval.

A.12.2.8.1.30 Oil Lubrication System

The engine lubricating oil system shall be of the manufacturer's standard design for the model engine proposed. The lubricating system shall be monitored and controlled by the engine’s electronic control system to insure proper lubrication for the application proposed. The system shall be readily accessible for service such as draining and refilling. Each system shall permit addition of oil and have oil-level indication. All items of equipment shall be furnished and installed as complete units ready for operation.

A.12.2.8.1.31 Lube Oil Sensors

Each engine shall be equipped with lube-oil temperature and pressure sensors. The temperature sensors shall provide signals for high lube-oil temperature indication and alarm. In addition, low lube-oil pressure indication and alarm sensors shall be provided.

A.12.2.8.1.32 Lubricating Oil Filter

Each engine lubricating oil system shall include a suitable lubricating oil full-flow, duplex (80) micron filter of the throw away cartridge type. The filter medium shall be absorbent type as recommended for use with the type of oil used in the engine. The filter shall be readily accessible and capable of being changed without disconnecting the piping or disturbing other components. The filter shall have the inlet and outlet connections plainly marked.

A.12.2.8.2 Starting System

For this analysis it was assumed the engines would be started by a compressed air system via an air motor supplied by the engine manufacturer. However, it is suggested at the next preliminary design stage investigate the economic advantages of an electric starting system consisting of a 24 VDC battery starting system, manually (or remotely started), from the engine control panel. The engine direct current starting system would separate from the engine control panel. The starting system shall be designed to have sufficient capacity to start the engine with the pump engaged. Starting motors are in accordance with SAE ARP 892.

For an electric starting system, a starting battery system is provided, one system for the station, which includes batteries, battery charger with over-current protection, battery rack, inter-cell connectors, spacers, metering, and relays. The simpler option is to follow the design of the auto industry with a separate starting battery for each engine and an alternator to recharge the battery while the engine is running. The lead acid type battery shall meet or exceed the requirements of SAE J537. A standard requirement of the battery sufficient capacity to provide the minimum cranking cycle consisting of no fewer than three cranking periods of up to eight seconds per period with eight second intervals between crank periods or shall be sized in accordance with the engine manufacturer’s requirements.

The battery charger shall have a current limiting 10 ampere battery charger, conforming to UL 1236, and shall be provided to automatically recharge the battery bank. The charger shall be capable of providing both automatic float charging and equalizing charging of the battery installation. The battery charger shall be capable of providing a floating charge rate for maintaining the batteries in a fully charged condition. An ammeter and voltmeter shall be provided on the charger to indicate charging rate and voltage. The charger shall have alarm functions providing indications of low battery voltage, high battery voltage, and battery charger malfunction.

A.12.2.8.3 Drive Shaft and Coupling Assembly

For small horsepower applications the speed reducer is typically connected to the driver by a Carden shaft and double, heavy duty, needle bearing type universal joints. The bearings should have minimum rating of a B10 life of not less than 16,000 hours (including applicable service factor for driver utilized) and shall have a service factor of two based on the maximum rated load. In addition, at maximum overload conditions, the stresses shall not exceed 80 percent of yield strength. Universal joints shall have forged steel yokes and spiders and shall have sealed needle roller bearings. Universal joints shall be installed in pairs. The angle between each shaft and the intermediate shaft shall be equal and not exceed the manufacturer's recommendations. The driving pins on the yokes attached to the intermediate shaft shall be set parallel to each other. The universal joints shall be dynamically balanced to AGMA balance classification seven or better and shall be grease lubricated unless self-lubricated.

To address torsional vibration, rubber torsional coupling between the engine output shaft and the Carden shaft is recommended.

A.12.2.9 Speed Reducers

In order to couple a large diesel engine with horizontal output shaft to a vertical wet pit pump, a right angle gear assembly is required. To reduce the output shaft rotary speed of the engine to the input shaft speed of the pump requires a speed reducer. These functions are combined in one unit - a right angle speed reducer. Right angle speed reducers perform the following functions:

- Transmit the power from the diesel engine driver to the vertical wet pit pump
- Redirect the power from the horizontal shaft of the driver to rotate the vertical shaft of the pump
- Reduce the speed of the shaft rotation of the engine driver to the required rpm of the pump shaft
- Prevent rotation of the pump shaft from backflow of water after shutdown of the driver
- Provide a thrust bearing(s) to address the up-thrust and down-thrust hydraulic loads of the pump

Speed reducers are standard products of manufacturers and conform to American Gear Manufacturer's Association (AGMA) standards. The furnished unit should display the AGMA insignia as evidence of conformance to the requirements of AGMA 6010-F97 or AGMA 6025-D98. Standard practices shall be as defined and set forth by the AGMA. The procedure outlined in AGMA 2005-C96 and AGMA 6010-F97 shall be followed.

The more detailed description of the typical speed reducer used in the vertical pump application is a single reduction right angle spiral-bevel gear. The reducer’s low speed output shaft is of a hollow shaft design. This arrangement permits the pump head shaft to pass concentrically through the reducer shaft for vertical adjustment of the pump propeller. The reducer’s high speed input shaft is connected to the driver by two universal joints and an intermediate shaft (Carden Shaft).

Bevel gears are used to connect shafts whose axes intersect. Spiral bevel gears have obliquely curved teeth with a spiral angle such that the face advance is greater than the circular pitch. This results in a continuous pitch line contact in the plane of axes of the gears. The contact between teeth begins at one end of the tooth and progresses obliquely across the face of the tooth.

A.12.2.9.1 Right Angle Gear Performance Requirements

The performance requirements of the reducer are determined by coordination with the pump and diesel engine requirements. The unit's primary function is to transmit the necessary torque from the approved driver to the pump shaft for the entire operating range of the system. A service factor of 2.0 shall be applied to the manufacturer's published rating. The reducer’s shaft output speed is designed to equal the pump rotative speed at the rated condition. The overall reduction ratio shall properly match the driver speed with the pump rpm at the pump’s rated condition. The rotation of the input shaft of the speed reducer should match the typical rotation of the driver.

The reducer should have a continuous mechanical horsepower rating of not less than 150 percent of the horsepower rating of the engine driver. The pump input power (Pp) at the rated condition is defined by the requirements of the vertical pump. The reducer shall be designed with sufficient capacity to stall the driver without injury to the reducer. The reducer includes a thrust bearing(s) to address the up-thrust and down-thrust hydraulic loads of the pump. Speed reducers have efficiencies in the range of 96 percent. Before assembly, each gear and shaft assembly shall be dynamically balanced in accordance with ANSI/AGMA 2005-C96.

A.12.2.9.2 Operating Conditions

The pump manufacturer should obtain or develop the following operating conditions for the design of the speed reducer:

- Maximum input power
- Driver speed at rated condition
- Speed reducer ratio
- Maximum pump reverse over-speed
- Low speed shaft downward thrust including weight
- Low speed shaft upward thrust during start-up or shut-down (if applicable)
- High speed shaft direction of rotation
- Low speed shaft speed direction of rotation
- Overhung load
- Maximum engine overload torque transmitted through the clutch
- Reverse torque load on backstop

A.12.2.9.3 Component Specifications

The following list of components is a generalized list and should not be considered a complete and comprehensive description of all the component pieces of a finished speed reducer. It should also be recognized, reducer designs vary from manufacturer to manufacturer and the component descriptions may not be representative of a particular design.

A.12.2.9.3.12 Gears

As discussed, the gearing of the reducer is the single reduction right angle spiral-bevel design. The gear teeth are precision ground or precision cut and lapped. The spiral bevel gears are gas nitrided or carburized, hardened, and lapped in pairs after heat treatment. In addition to rating the gears according to ANSI/AGMA 6010-F97 and ANSI/AGMA 2005-C96, gear stresses are specified to not exceed 80 percent of yield strength for any overload, or engine overload condition.

A.12.2.9.3.13 Backstop Device

A self-actuated backstop device to prevent reverse rotation of the pump due to loss of power, or drive failure, is installed as an integral part of the transmission unit. Its action is instantaneous and without backlash. The design is typically of the cam clutch type or drop-pin type and is of a capacity adequate to prevent reverse rotation with backflow through the pump due to the maximum differential head from the discharge line at the dam top to the wetwell minimum elevation. Lubrication is provided by the transmission lube oil system. The backstop is installed on the low speed output shaft. The torque is transmitted directly to the gear housing. The backstop shall operate at a temperature of less than 160 degrees F under all operating conditions.

A.12.2.9.3.14 Shafts

Each shaft shall be heat treated stainless steel. Welded shafts are not acceptable. Input shaft size and configuration shall be compatible with the driver. The pump head shaft shall accommodate the hollow shaft design of the reducer’s output shaft to permit vertical adjustment. Sufficient thread length shall be provided to the top of the pump shaft to permit 1-inch adjustment, either up or down of the pump shaft. The adjusting nut shall be designed to support the total axial load and thrust of the pump and be locked in position to prevent movement.

A.12.2.9.3.15 Seals

The down output shaft shall have a drywell design seal. The input shaft shall have a lip seal to prevent leakage of the oil and exclude dirt. Lip seals shall utilize hardened steel wear sleeves to preclude shaft repair or replacement.

A.12.2.9.3.16 Lubricating System

The reducer is provided with an oil lubrication system that provides continuous lubrication to the gears, bearings, and backstop. The system consists of an oil circulating pump, heat exchanger, piping, filters, and controls. Each reducer is provided its own system. The oil circulating pump is a positive displacement type pump driven from one of the reducer shafts.

A.12.2.9.3.17 Heat Exchanger

The maximum oil sump temperature at the rated speed and load shall be 160 degrees F at an ambient temperature of 105 degrees F. The exchanger may be either an air cooled or water cooled system. In no case, however, shall the lubricating oil piping be circulated through the water in the intake bay. If a shell and tube type lubricating oil cooler is provided, the unit shall be of adequate capacity to prevent the lubricating oil from exceeding allowable temperature limits with an entering raw water temperature of 85 degrees F.

If an oil to air exchanger is to be used, the tubes and fins shall be aluminum, copper, or copper alloy. The working pressure shall not exceed the oil pump working pressure. The exchanger shall withstand a test pressure of 150 percent of the design pressure for a period of four hours during which time the exchanger will be checked for leaks. Any leakage is cause for rejection. The oil to air heat exchanger system shall include a fan, motor, and controls to maintain the required oil temperature.

Oil to water exchanger are either water cooled shell and tube type, or water cooled plate type, or an internal water cooled coils within the reducer sump. The heat exchanger tubes are 90-10 copper nickel alloy, plates are Type 316 stainless steel. The minimum wall thickness of the tubes is typically 16 gauge and designed for the pressure rating. Water shall be circulated through the tubes and plates and the design shall be such that the tubes and plates can be cleaned. The exchanger shall withstand a test pressure of 150 percent of the design pressure for a period of four hours during which time the exchanger will be checked for leaks. Any leakage is cause for rejection. The oil to water heat exchanger shall have a thermo-mechanical control valve to adjust flow rate through the exchanger to maintain a minimum oil temperature of 120 degrees F in the housing sump.

A.12.2.9.3.18 Piping and Fittings

Oil lines up to two inches outside diameter (O.D.) are seamless steel tubing with 37 degree flare or flareless fittings. Oil pipe equal to or larger than two inches O.D. are be black steel with welded fittings. Water piping is typically copper or copper alloy with brazed or 95-5 soldered joints. All piping, tubing, and fittings conform to ASME B31.1- Process Piping.

A.12.2.9.3.19 Lubricating Oil

Lubricating oil shall be mineral oil or synthetic hydrocarbon as recommended in ANSI/AGMA 6010-F97 for an ambient temperature range of 15 to 125 degrees F.

A.12.2.9.3.20 Oil and Breather Filters

The lubricating system shall have two oil filters on the pump outlet side. One filter shall be for removing particles and the other for water removal. Each filter shall incorporate an oil-filled differential pressure gauge to indicate the pressure drop across the filter. The filter assemblies shall be sized for a pressure drop for a clean filter of no greater than four psi. Filters shall have a bypass setting of 40 to 60 psi. Element collapse rating shall not be less than 150 psi.

- Oil Particle Filter: The Beta rating shall be B6>75 at 60 psi differential per ANSI/NFPA T3.10 1990 Filter Elements or an approved alternative. The filter shall be sized to avoid bypass at a start-up oil temperature of 80 degrees F
- Oil/Water Filter: The filter shall maintain the water content in the oil of no greater than 200 ppm
- Breather Filter: The breather filter shall have a Beta rating of B6>75 and a desiccant chamber to remove water.

A.12.2.9.3.21 Rolling Bearings

Rolling bearing elements are located on the shaft using shoulders, collars, or other positive locating devices and shall be retained on the shaft with an interference fit and fitted into the housing with a diametral clearance, both in accordance with the recommendations of ISO 286 (ANSI/ABMA 7 - 1995). The rolling element bearing life shall have a basic rating of L10 per ISO 281 (ANSI/ABMA 11 - 1990) of at least 100,000 hours with continuous operation at the rated condition, and at least 16,000 hours at maximum radial and axial loads and rated speed.

A.12.2.9.3.22 Thrust Bearings

The entire weight of the rotating element of the pump and hydraulic thrust, (up-thrust and down-thrust), imposed by the propeller and any radial loads created by the reduction gear shall be carried by the thrust bearing located in the reducer. The thrust bearing shall be sized for continuous operation under all specified conditions and shall provide full load capabilities if the pump's normal direction is reversed. The thrust bearing shall be a steep angle tapered bearing type. Misalignment of the outer and inner bearing rings shall be limited to 0.001 radian for cylindrical and tapered-roller bearings and 0.0087 radian for spherical ball bearings. Bearings shall be mounted directly on the shaft, bearing carriers are not acceptable.

A.12.2.9.3.23 Radial Loads

Radial load can be addressed by the thrust bearing(s) or separate rolling element bearings can be provided.

A.12.2.9.3.24 Housing

The reducer housing shall be cast or fabricated steel, stress relieved prior to machining, and reinforced to carry all applied loads and maintain gear alignment. The unit may be made in several sections, split as required, for service and assembly and heavily ribbed to insure strength and rigidity. The housing shall be so constructed as to provide stability that maintains precise alignment of the gears and shafts. All joints shall be finished machined and oil tight.

A.12.2.9.3.25 Inspection Openings

Inspection openings with cover plates shall be provided over each set of gears. All inspection, access, service and other type openings shall be provided with suitable metal covers, vented, screened and easily removable as necessary to insure continuous protection against the entrance of insects, rodents and the elements throughout the expected life of the equipment.

A.12.2.9.3.26 Lifting Lugs

The unit shall be provided with eye bolts or lifting lugs for installation and removal.

A.12.2.9.3.27 Instrumentation

The instrumentation supplied with the reducer shall be a complete working package that has been coordinated with the pump and driver supplied. The reducer shall have the following devices:

- High Oil Temperature: An oil temperature sensor shall be provided to monitor the oil temperature in the reducer sump. The alarm and shut down shall be part of the system's control and monitoring system. Lower settings may be used if recommended by reducer manufacturer. Typically the alarm is set at 180 degrees F, the shut down at 200 degrees F
- Oil Pressure: Provide a gauge after the oil pump to monitor oil pressure. The gauge shall be oil or glycerin filled and shall have an isolation valve
- Temperature Gauges: Provide thermometers in the sump, in the oil line after the heat exchanger, and the backstop
- Oil Level Sight Gauge: Provide an oil level sight gauge to monitor oil levels in the sump of the reducer
- Vibration Switch: Vibration switch with the alarm and shut down shall be provided as part of the system's control and monitoring system. The manufacturer shall be responsible for the

vibration switches proper settings to accommodate initial and running vibrations to avoid nuisance tripping of the switch. A time delay shall be incorporated into the control system if required. Set alarm at 0.5 inch per second or at baseline level recommended by the reducer manufacturer

A.12.2.10 Fuel System

The fuel system design for the diesel engine drivers must conform to the requirements of National Fire Protection Association (NFPA) 30-Flammable and Combustible Liquids Code and NFPA 37-Stationary Combustion Engines and Gas Turbines. The fuel oil supply system for each engine typically consists of a motor driven fuel oil transfer pump, day tank, and an aboveground fuel storage tank(s). The fuel oil flows through a strainer from the outside fuel oil storage tanks to the day tank. From the day tank, fuel oil flows to the engine. Overflow and drip lines from the engine return the oil to the day tank. If the main storage tank is the lowest point in the engine fuel system, a pump may need to be provided to deliver fuel from the tank to the day tank.

A.12.2.10.1 Primary Storage Tanks

The primary storage tanks should be aboveground ballistic double walled tanks. The tanks are UL 142 listed and normally store the specified petroleum product at atmospheric pressure but should also be designed to withstand a pneumatic pressure test. The secondary containment structure shall also be UL 142 listed. The number of tanks can depend on a number of criteria. All tanks should be of equal capacity. A minimum of two tanks is preferred to allow for out of service maintenance. Capital cost is typically the over-riding factor in the determination of the number of tanks. The tanks nominal capacity should provide for a minimum of ten days of fuel for continuous operation of all units operating at maximum horsepower.

The primary tank is fabricated from steel and is a welded construction throughout. The fabrication is in accordance with UL-142 with steel conforming to ASTM materials, grades and thickness. All welded seams of the tanks must have full penetration and complete fusion. All welds are subjected to a soap film test, using a vacuum device or other approved method. A factory pneumatic pressure test is typically specified. The air pressure test should be applied in the manufacturer's shop at five psig held for a period of two hours without a pressure drop after the test apparatus has been removed.

The secondary containment shall be a second steel wall. All tank openings shall be located on top of the tank. Catwalks shall be provided on top of the tanks to permit access to all tank openings and piping connections.

The tank openings and piping connections that are recommended to be furnished:

- Emergency vent
- Overfill containment
- Tanker fill
- Fuel oil supply
- Vent
- Fuel Oil Return

- Level gauge
- Remote fuel inventory.

Storage tank accessories that are recommended to be specified:

- Lockable fill cap
- Vent cover with 40-mesh screen over the outlet, and an aluminum cover to prevent rain from entering vent line
- Emergency vent to relieve internal pressures in excess of 2.5 psi. The vent shall be sized according to NFPA 30 requirements

A.12.2.10.2 Day Tank

The day tank is a unit composed of a small capacity fuel storage tank with secondary containment, fuel transfer pumps, and level controls and is located inside the pump house near the engine driver. The unit serves to transfer fuel to the engine from the outside storage tank at a controlled suction head and delivery rate. It also functions as a collection point for transfer of the return fuel to the outside tank. A day tank is furnished for each engine. The tanks are of steel construction, double walled with leak detection monitoring, and built in accordance with the applicable provisions of the NFPA 30 and UL 142. A motor driven fuel oil positive displacement type pump, with built-in relief valve and capacity as recommended by the engine manufacturer, is typically furnished with each day tank. The pump transfers fuel oil from the storage tank to the day tank. The engine's fuel pump transfers the fuel oil from the day tank to the engine. A line is provided for return of unused fuel to the day tank and motor driven fuel oil positive displacement type pump is provided for return of overflow fuel back to the storage tank. Both fuel transfer pump assemblies should be included as part of the day tank package. The capacity and performance criteria of the day tank unit specified shall be verified by the designer to ensure proper performance of the engine supplied. In addition, the designer should confirm the piping distance from the engine to the day tank is acceptable to the engine manufacturer. Typically the fuel storage capacity of the day tank is based on the fuel return rate and the volume of fuel needed to ensure proper cooling of the fuel in accordance with the requirements of the engine. The total storage capacity of all the day tanks located inside the pump station shall not exceed 1,320 gallons.

The day tank highest fuel level shall be located below the engine injector to prevent run-on. The day tank’s lowest fuel level needs to be above the engine-driven fuel pump to ensure the pump maintains its prime. The day tank should be completely factory assembled, wired, painted and tested. The following additional features are typically specified:

- Ports for supply, inflow, fill, and overflow lines
- Vent with flame arrestor connections. The vent shall exhaust to the building exterior and at an elevation of five feet above the top of the tank
- Level Indicator, side mounted, direct reading float controlled liquid level indicator
- Liquid level switch for automatic control of the fuel oil transfer pump and high and low level alarms
- Drain with shut-off valve

The tank should be all seam welded, square, atmospheric tank of heavy gauge steel with internal reinforcements and pressure tested to five psi test pressure. The tank should be provided with welded flange pipe fittings for overflow, vent, and drain lines. All fittings except drain located above normal full level. The unit should be mounted on heavy gauge steel channel feet with mounting holes. The tank should also have a removable steel top cover. The tank requires an overflow basin for containment of 150 percent of the tank capacity. The basin needs to circle the tank and include a drain. The unit should have corrosion resistant interior and exterior finishes.

A.12.2.10.3 Transfer Pumps

The fuel transfer pump system includes high vacuum, single stage, internal-gear, positive- displacement rotary type pumps of non-corrosive alloy and carbon composition with leak-proof mechanical rotary shaft seal. Each pump is driven by a 120V, single-phase open drip proof (ODP) motor with thermal overload protection. Each pump is protected by a pressure relief valve appropriately sized for the pump provided. The fuel oil transfer pump shall be sized to provide 150 percent of the combined fuel consumption rate.

Fuel Inlet Equipment

The fuel inlet equipment includes:

- Fuel strainer
- Priming tee and check valve
- Solenoid valve, 120V
- Foot valve shall be installed at supply line termination in main storage tank

A.12.2.10.4 Controls

The day tank controls and features can vary from model to model. The following list is typical of the requirements specified:

- Pump run-off-automatic operation
- Press to test pump pushbutton
- Pump start-stop automatic control. Level control switch shall be industrial quality mechanical float switch with double pole double throw (DPDT), 2-Hp contacts, welded steel balanced float, and adjustable pick-up to drop-out differential.
- Local/remote low fuel level alarm consisting of red alarm light plus dry signal contacts for remote alarm. Activated by separate float switch sensing at 30 percent of day tank fuel capacity.
- Local/remote high fuel alarm consisting of red alarm light plus dry signal contacts for remote alarm. Activated by separate float switch sensing 95 percent of day tank fuel capacity.
- High fuel level emergency pump stop switch to override main float switch and stop pump motor at 95 percent of day tank capacity.
- Mechanical float gauge
- Indicators shall be long life, bright, large display light emitting diodes (LEDs) and shall include the following indication functions:

- Fuel level
- Power available
- Switch off (flashing)
- Pump running for each pump in duplex package
- Low level alarm
- Overflow alarm/pump control backup activated

- Supply the following outputs
  - Pump start-stop
  - Low level alarm
  - High level alarm
- Low Level Float Switch: Provide low level float switch for engine shut down to prevent starving of injectors and need to re-prime the engine. Engine shut down shall be shall be activated by engine control system in accordance with normal shut down procedures.

A.12.2.10.5 Installation

To ensure the safety controls work properly, a high level alarm test shall be performed. The day tank shall be manually filled to a level above the overfill limit. The level that activates the alarm shall be recorded and the shutdown of the fuel transfer pump shall be verified. The day tank shall be drained below the overfill limit following the test. In a similar manner a low fuel alarm test shall be performed. Fuel from the day tank shall be drained to lower the fuel level below the low fuel level limit to test the audible alarm.

A.12.2.10.6 Fuel Piping and Auxiliaries

The following piping and auxiliaries are part of the fuel system:

A.12.2.10.6.12 Fuel Filters

Each engine supply line shall have a duplex filter with valve installed on the inlet side of the engine fuel pump. The filter shall be rated for filtering out particles down to 25 micron size.

A.12.2.10.6.13 Fuel Strainers

A full flow fuel strainer is be provided in the fuel oil system upstream of the engine and duplex filters. The strainer shall be a replaceable cartridge type, rated for removal down to 125 micron size.

A.12.2.10.6.14 Fuel Meter

A rotating disc type fuel meter is furnished for measuring fuel oil supplied to each day tank. The meter is designed for petroleum service and calibrated in U.S. gallons, with a five place cyclometer dial. The meter is located in the system to measure net engine fuel consumption.

A.12.2.10.6.15 Single-wall Piping

Single-wall piping is required to meet the standards set forth in ANSI/ASME B36.10. Pipe shall conform to ASTM A53 Grade B, Schedule 40, seamless or electric resistance welded. No pipe or fittings in the piping systems should be galvanized. Fittings for screwed pipe are typically specified as 3,000-pound forged steel conforming to ANSI/ASME B16.11. Flanges shall be standard weld-neck type, 150-pound forged steel, ASTM A181, and conforming to ANSI/ASME B16.5. Flange facings shall correspond to the equipment to which the piping is joined, and, unless otherwise required shall be standard 1/16-inch raised face flanges. Machine bolts are heavy hexagonal alloy steel conforming to ASTM A307, Grade B. Nuts shall be heavy hexagon alloy steel conforming to ASTM A563, Grade A. All flexible oil lines, such as connections to the engines, should be specified as reinforced nitrile hydraulic hose with stainless steel braided sheathing.

A.12.2.10.6.16 Double Containment Steel Fuel System Piping

Double wall piping consists of a steel carrier pipe within a steel containment pipe. The internal carrier piping is typically standard weight carbon steel, ASTM A53, Grade B pipe. All carrier pipe joints are butt-welded for 2.5 inches and greater, and socket-welded for 2-inches and below. Carrier pipe fittings are carbon steel butt weld or socket weld fittings. The secondary containment pipe is fabricated out of ASTM A-139B, Grade B, ASTM A-120, Grade B or ASTM A53, Grade B carbon steel, Schedule 40 for pipe diameters less than six inches, and schedule 10 for six-inch diameter and above. Joints of secondary containment pipe are butt-welded with carbon steel butt-weld fittings. The carrier pipe inside the containment casing is supported at 10-foot intervals or less. The supports are designed to allow for continuous air flow and drainage. The support spacing is dependent on the pipe diameter. Carrier pipe and containment pipe are required to be air tested. Double containment piping shall be used exterior to the station in areas where spill protection is required such as the supply and return lines to the fuel storage tanks.

A.12.2.10.6.17 Pipe Hangers and Supports

Pipe support or hanger spacing and arrangements should conform to ANSI/ASME B31.1 Code for Pressure Piping. Pipe supports or hangers are provided as required and at changes in pipe direction to limit pipe deflection under the applied load and suppress vibration. The complete hanger assemblies need to be adequately rated for the applied load and be designed for potential expansion. Pipe hanger and supports shall be of the types listed in Table 1 "Hanger and Support Selection," MSS Standard Practice SP-69 except that the following figure types given in Figure 1 are not acceptable: Types 5, 6, 11, 12, 7, 9, 10, 16, 17, 23, 20 and 25.

Install fuel piping systems in accordance with NFPA 30, NFPA 30A, local codes, manufacturer's requirements for warranty and latest EPA and state regulations for fuel storage tank systems. The following installation requirements should be followed:

- Install each run with a minimum of joints and couplings, but with adequate and accessible unions for disassembly and maintenance or replacement of valves and equipment
- Reduce sizes (where indicated) by use of reducing fittings
- Align pipe accurately at connections within 1/16-inch misalignment tolerance
- Comply with ANSI/ASME B31.1 - Code for Pressure Piping
- Locate piping runs, vertically and horizontally (pitched to drain)
- Align horizontal runs parallel with walls and column lines
- Hold piping close to walls, overhead construction, columns and other structural and permanent-enclosure elements of the building
- Thread pipe in accordance with ANSI/ASME B1.20.1

- Welding shall be accomplished by the use of the shielded metal arc process and shall be in strict accordance with ANSI/ASME B31.1
- Butt welding end preparation on all pipe shall conform to ANSI/ASME B16.25
- Provide sleeves for all openings in walls required for pipes and tubing
- Paint all exposed steel piping

A.12.2.11 Mechanically Cleaned Trash Racks

Debris entering the pump feed channels from the canal is removed by mechanical trash racks in front of each pump. Openings are set at three inches, which will remove all material that would be damaging to the pumps. Material blocked by the bars is conveyed to the top of the rack and dropped in containers on the pump station deck. Containers are emptied by trucks as needed, from an access drive on the pump deck. The amount of debris collected is a function of the flow in the RIOC Canal and recent weather conditions. Screens are custom, heavy duty units designed for service in stormwater applications.

A.12.2.11.1 Description of Equipment

The screening system consists of heavy duty bars with a 3-inch clear spacing set on a 70° angle from horizontal. The bars are cleaned by a front clean-front return collection mechanism. Debris that is retained on the bars is collected by rakes attached to two continuous heavy duty chains and deposited in containers on the pump deck. The rakes are spaced on 63-inch centers to prevent buildup on the bars when the mechanism is in operation. The chains are driven by the headshaft at the top of the sidebeams. The headshaft is driven by a gearmotor through a chain and sprocket arrangement. The chains are guided by a stationary track with a low-friction, replaceable cover to the bottom of the screen, where they return, riding on the bars and then a deadplate. The debris discharges from a chute that mounts at the top of the deadplate. The entire mechanism and support structure are fabricated of Type 316 stainless steel. Design of the screens does not require any maintenance activities be performed in the approach channel.

A.12.2.11.2 Controls

Screen controls will be enclosed in a NEMA 4X stainless steel control panel located adjacent to the individual screens. Control logic will be by a PLC, which will handle all functions of the screens and protective systems. Screens are operated through a Hand-Off-Automatic selector, such that Hand mode runs continuously, Off is inactive, and Automatic operates based on differential head on the screen. In the Automatic mode, an adjustable timer operates the screen for an adjustable, preset time regardless of differential head. The control panels will be equipped with self-contained air conditioning units to protect the electronics from high ambient heat.

A.12.2.12 Discharge Piping and Appurtenances

A.12.2.12.1 Pipe

Discharge piping for the pumps shall be spiral-welded steel conforming to AWWA C200 and AWWA M11 with the exception of post fabrication hydrostatic testing will not be required. Minimum pipe thickness shall be as follows:

Table A.12.2-8. Pipe Wall Thickness

Pipe Diameter (inches)	Thickness (Inches)
144	0.75
132	0.75
96	0.75
84	0.75
60	0.625

Materials:
Piping shall be fabricated from one of the following materials:

- Sheet or coil conforming to ASTM A570, Grade 30, 33, 36 or 40
- Plate in coil form conforming to ASTM A36, A283 Grades C or D, or ASTM A572 Grade 42
- Coil conforming to ASTM A139, Grades A or B

Pipe Joints:
All joints shall conform to AWWA C200 and AWWA C207 with a Class B pressure rating and be drilled to ANSI B16.1 Class 25.

A.12.2.12.2 Fittings and Special Connections

Elbows shall be fabricated from tested pipe to conform to AWWA C208 and shall be reinforced in accordance with applicable provisions of AWWA M11. Openings for air vent connections shall be provided with flanged outlets and shall be flanged in accordance with ANSI/ASME B16.5 standard 125 pound flange.

Harnessed Coupling
A flexible mechanical coupling, Dresser style or equal, shall be provided to connect the pump discharge elbow to the discharge piping. All components of the coupling shall be stainless steel. The connecting ends of the discharge pipe shall be fabricated in accordance with the requirements of the coupling provided. Adjustable thrust rods shall be provided to transfer thrust loads to the discharge piping or wall thimble. All bolts, rods, nuts, and associated hardware shall conform to ASTM F593 Type 316 stainless steel.

A.12.2.12.2.12 Wall Thimble
A wall thimble shall be provided for embedment in the intake back wall and connection to the pump discharge elbow and the discharge piping or flap valve. The thimble shall have a seal ring centered in wall when embedded and shall have flanged ends to mate to the discharge piping.

A.12.2.12.3 Gaskets and Bolting Materials

Gaskets for flanged joints shall conform to ANSI B16.21, 1/8-inch thick full-face synthetic rubber. Full-face gaskets for all pump and equipment connections shall be provided. Bolts for flanged joints shall conform to ASTM F593 Type 316 stainless steel. Nut and bolt heads shall be hexagonal.

A.12.2.13 Lubrication Oil System

A lube oil system suitable for unloading, storage, and transfer of supply and waste lube oil will be provided, including all necessary storage tanks, pumps, piping, valves, controls, and accessories. Lube oil and waste lube oil storage systems will have a minimum capacity of 30 days storage, based on equipment manufacturer recommended oil change capacity and intervals.

Storage tanks will be aboveground single wall type and will be designed and constructed in accordance with applicable industry codes, including API and UL. Tanks will be provided with level detection and overflow prevention devices. The system will be designed for truck unloading of lube oil and loading of lube oil waste, to include all necessary storage tanks, pumps, piping, valves, and accessories for unloading lube oil and loading lube oil waste. System design will facilitate minimal loading and unloading time. Lube oil and waste lube oil pumps will be self-priming, positive displacement type. Pumps will be motor-driven and equipped with an integral internal relief valve. Lube oil piping will be ASTM A53 or A-106 black steel piping. Minimum pipe wall thickness will be based on ASME B31.3. Lube oil piping and fittings will be butt or socket welded. Butt weld fittings will be in accordance with ASME B16.9 and socket weld fittings will be in accordance with ASME B16.11. Valve construction and class will be in accordance with ASTM B16.34. Underground piping will have secondary containment using a fiberglass reinforced plastic containment system. Containment piping will be capable of withstanding H-20 highway loading, as defined by AASHTO HB-16.

A.12.2.14 Vacuum Priming System

The vacuum system for each station will consist of two electric liquid ring vacuum pumps, one on-line and one standby, to remove air from the pump discharge pipe to establish full flow through the pump discharge. To establish vacuum in each inflow pump discharge conduit, air will be drawn out through two eight-inch to 12-inch ports from the discharge tube which will then be manifolded together and directed to a barometric tube and then to a vacuum pump. Each vacuum line will have its respective vacuum release valves, with both auto and manual operation, providing two vent areas for siphon break to minimize time to break siphon on shut down. The vacuum pumps, which will alternate, will be manually started from each engine control panel and will utilize a barometric tube separator between the vacuum pump and the main pumps to protect the vacuum pump from water slugs. The system will signal run status to the control console. A sensor will monitor vacuum level and send a high vacuum alarm to the control console. Seal water for the vacuum pumps, if required, will be supplied by the cooling water system and will have solenoid controls. Selection of pump size should be based upon an eight to 12 minute time to evacuate all air from submerged suction and discharge tubes.

A.12.2.15 Compressed Air System

Dual air compressors, one duty and one standby, will be provided to power the air starting system for the diesel engines and for utility and instrument air requirements. Compressors will be two-stage, oil-lubricated, air cooled, belt-driven and mounted on a common base with the drive motor. Compressors will be pressure lubricated with a replaceable cartridge oil filter. An air-cooled intercooler will be provided between the first and second stages. System will be rated for minimum 175 psi operation, and a minimum 50% duty cycle. Air delivery capacity will be as required by the engine starting systems. Package will include intake air filters, silencers and vibration snubbers.

Two commercial grade refrigerated air dryers will be provided on the instrument air supply, rated for the full capacity of the air compressors. Air dryers shall have ultra-fine filtration and oil mist separators.

The primary receiver will be ASME code stamped with a minimum capacity of 120 gallons. A separate ASME code stamped receiver of 60-gallon capacity will be dedicated to instrument air demands. Each receiver will be provided with an automatic condensate drain. Pressure regulators will be mounted on each receiver, as well as an ASME safety relief valve and pressure gauges before and after the regulators.

Compressed air piping will be ASTM A53 or A-106 black steel piping. Minimum pipe wall thickness will be based on ASME B31.3. Fittings will be in accordance with ASME B16.9 and B16.11. Valve construction and class will be in accordance with ASTM B16.34.

A.12.2.16 Backflow and Dewatering Gates and Operators

The hydraulic design of the pump station provides for a vacuum breaker at the peak elevation of the discharge pipe at the top of the dam. This prevents backflow from the reservoir to the RIOC Canal. In order to allow maintenance on the mechanical screens, provision for needle beams will be installed between the canal and the screens. A crane will be provided for installing and removing the beams, and a storage rack provided. No motorized gates are required.

A.12.2.17 Station Emergency Power

The pump station will require backup generators to power controls, HVAC, water system, communications, fire alarm and security. Two generators will be installed to provide redundancy during outages or storm events.

A.12.2.18 Stage Monitors

Canal conditions east and west of the pump station will be monitored with level transmitters installed in stilling wells. A level measurement will be taken in each pump approach channel with a level sensor in a stilling well to prevent screen blinding from causing operating levels from falling below manufacturer’s minimum recommendation. Level transducers will communicate via a 4-20 ma DC signal, powered by a 120 volt to 24 volt power supply. Level information will be transmitted to the pump station and available for remote monitoring.

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A.13 ELECTRICAL DESIGN

A.13.1 DESIGN CRITERIA

A.13.1.1 Utility Power

Florida Power & Light (FPL) overhead 3 phase 13.2 kilovolt (kV) power lines are existing alongside of U.S. Hwy. 27, approximately 3 miles east of the proposed A-2 Reservoir pump station (P-1). FPL overhead 3 phase 13.2 kV power lines are existing at the south west corner of A-2 STA at existing pump station G-372. FPL overhead single phase 13.2 kV power lines are existing at the south east corner of the A-2 Reservoir at structure G-720. Preliminary contact with FPL was made to inform them of the proposed pump station and gate structures and the anticipated power demands. No additional information was received from FPL at this time concerning what overhead lines would be extended to serve the new pump station and gate structures.

A.13.1.2 Pump Station Equipment Voltage

The A-2 Reservoir pump station voltage will be 4,160 volts, three phase, 60 hertz. In general, station equipment voltages will be specified to operate at the following voltages.

Motors rated 500 Horsepower (Hp) and larger	4160 volts, 3 phase
Motors rated one Horsepower (Hp) to 450 Hp	460 volts, 3 phase
Motors less than one Hp	115 volts, 1 phase
Lighting	115 volts, 1 phase
Convenience receptacles	115 volts, 1 phase

A.13.1.3 Pump Station Power Distribution

A preliminary one-line diagram (**Figure A.13.1-1**) for the A-2 Reservoir pump station is included on the following page. The distribution system will be serviced by the FPL at 13.2 kV, three phase, 60 hertz with primary metering. The FPL primary shall be connected to a District owned stepdown transformer to provide 4,160 volts, three phase, 60 hertz power to the pump station. The load side of the stepdown transformer shall be connected to the pump station Main Breaker in the medium voltage Motor Control Center (MCC) that will have the motor starters for the three (3) 200 CFS electric motor driven low flow pumps and a 4,160 volts, three phase, 60 hertz breaker to feed a stepdown transformer to provide 480 volts, three phase, 60 hertz power to the pump station. The load side of the 4,160/480 volt stepdown transformer shall be connected to a low voltage main breaker that will be connected to an automatic transfer switch. The automatic transfer switch is also connected to a diesel powered emergency generator sized to provide power for the six (6) diesel powered reservoir pumps, auxiliary support systems and house loads with the load side of the automatic transfer switch connected to the station Switchboard. The Station Switchboard shall have a breaker that is connected to a smaller Automatic Transfer Switch that will be connected to a smaller LP emergency generator. The load side of the smaller Automatic Transfer Switch will be connected to panels that provide power to pump station house loads. House loads shall include lighting, HVAC, Security and Access Control Systems, PLC SCADA and Communication Systems and potable water system.

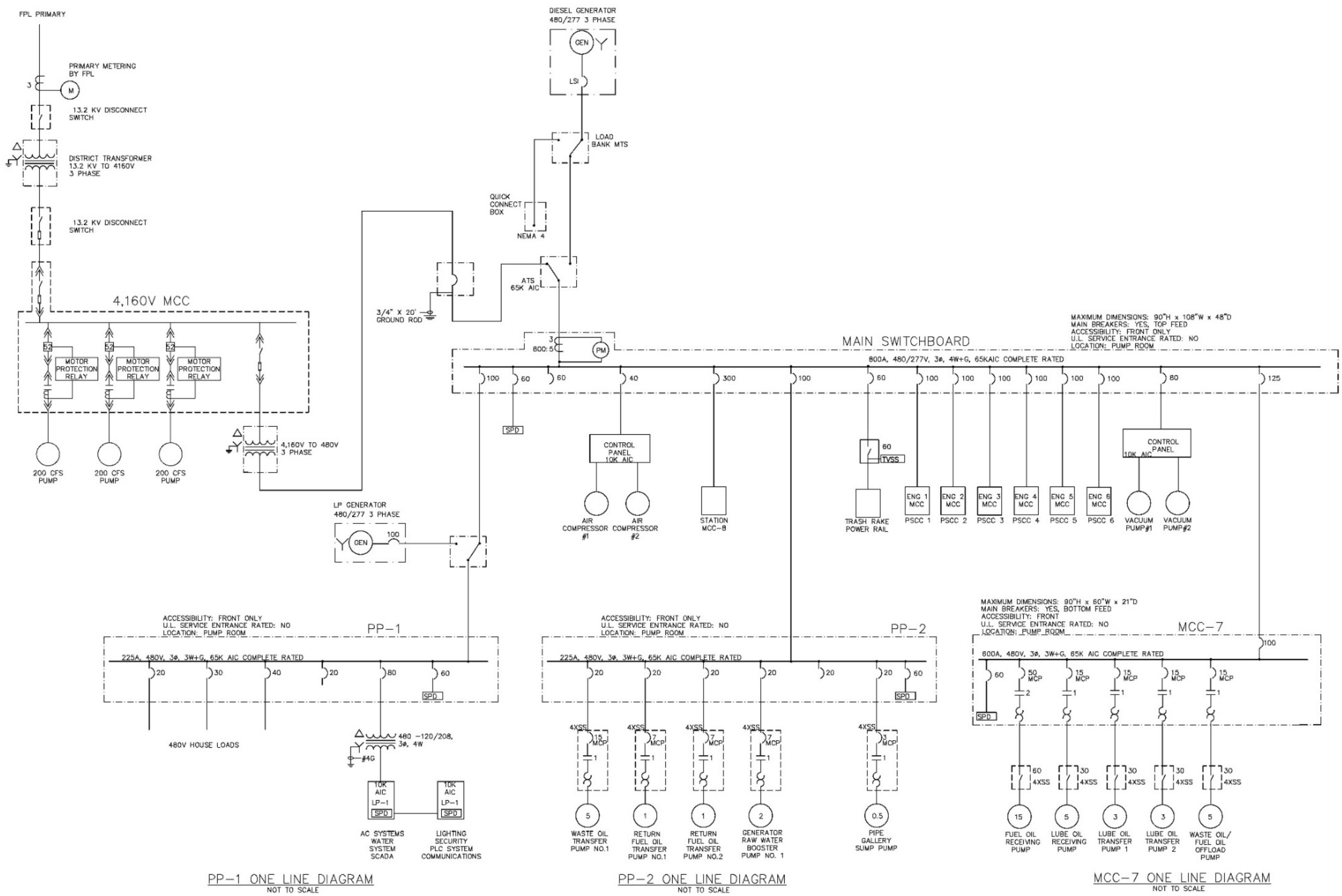

Figure A.13.1-1. Preliminary Pump Station One-Line Diagram

A.13.1.4 Pump Station Switchgear

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:

- Two vacuum system pumps
- Two air compressors
- Six motor control centers for diesel engine pump support equipment.
- Motor control centers for miscellaneous loads
- Crane and hoist

The motor control centers for miscellaneous loads will supply power to individual pumps that are not part of a vendor supplied package and other loads as indicated below. The list of equipment is tentative and subject to change during final design.

- Building supply fans
- Building exhaust fans
- Two waste fuel oil pumps
- Fuel oil receiving pump
- Two lube oil supply pumps
- Lube oil receiving pump
- Six cooling water pumps
- Two fresh water pumps
- Six water lubrication pumps
- Two potable water pumps
- Two lube oil pumps.
- Two waste lube oil pumps
- Generator block heater
- Two traveling trash rakes
- Four rotating strainers
- Two lighting panels (120/208 volt, three phase)
- 14 motor operated valves
- Water heater
- Instrument air compressor
- Two engine cooling water valves
- Hvac power panel (120/240 volt, single phase)
- Fire alarm and security system power panel (120/240 volt, single phase)
- Drainage pump bay-drawdown pump receptacle
- Cooling water pump receptacle
- Other loads as required

A.13.1.5 Standby Generator Power

In addition to normal utility power, the A-2 Reservoir pump station will have a diesel engine powered generator. Generator will be sized to operate the station six diesel engines, auxiliary support systems and house loads should the normal utility power fail. Fuel storage requirements will be based on generator operation for a minimum of seven days. In addition to the diesel engine powered generator a second smaller LP engine powered generator and automatic transfer switch to power the station house loads if the diesel generator is not required to power the diesel pumps or if the diesel generator does not start or fails during operation.

Upon failure of the utility power, a transfer switch will start the generator and automatically transfer power supply to a generator. A manual generator start will be provided to exercise the unit.

A.13.1.5.1 Motor

Motors below 150 Hp will be totally enclosed, fan cooled, and of premium efficiency. Motors 200 Hp and above shall be W1 enclosures. All outdoor motors will have integral space heaters. Indoor motors five Hp and larger will have integral space heaters.

A.13.1.5.2 Monitors

The 4,160 volt MCC and 480 volt switchboard and the motor control centers will each have a power monitor that will provide line and phase voltages, phase currents, kilowatt (kW), kilovolt-ampere reactive (kVAR), power factor, and kilovolt-ampere (KVA).

A.13.1.5.3 Lighting and Receptacles

Lighting panel boards will be rated for 120/208 volts, three phase. Bus bars will be copper. Circuit breakers will be thermal magnetic bolt-on type.

High bay areas of the pump station will be provided with metal halide light fixtures. The pipe gallery area will have LED light fixtures. Control room, break room, and offices will have LED light fixtures. Outdoor light fixtures will be wall mounted and controlled by a photoelectric switch. The diesel tank storage area lighting will be pole mounted metal halide fixtures. Lighting levels will be in accordance with the USACE EM 1110-2-3105, Chapter 21 standard.

Major paths of exit will have LED type exit signs on a dedicated circuit. Emergency lighting will also be provided.

Switches used for lighting will be rated 20 amperes, 120 volts. Duplex receptacles will be rated 20 amperes, 120 volts. Ground fault circuit interrupter (GFCI) type receptacles will be used outdoors and in the restrooms. Office receptacles will have stainless steel plates. Outdoor receptacles will have “in-use” weatherproof covers.

A.13.1.5.4 Conduits and Wiring

Conduits above grade will be Galvanized Rigid Steel. Conduits below grade will be Polyvinyl Chloride (PVC) Schedule 40 pipe. Underground conduits, in general, will be encased in concrete.

Liquid-tight flexible metal conduit will be used at all motors, transformers, instruments and any other equipment that can vibrate or move. Galvanized Rigid Steel conduits will be terminated at equipment and boxes with insulated plastic bushings. The cable tray will be reviewed for use in the pump station during final design.

Wire for 480 volt power applications will be thermoplastic high heat resistant nylon coated (THHN)/thermoplastic heat and water resistant nylon coated (THWN) insulation with stranded copper conductors. The minimum size wire will be 12 gauge.

Wire for control and alarm circuits will be multi-conductor type THHN/THWN insulation, with stranded copper conductors, and 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 will be single pair shielded instrument cable, type Thermoplastic Fixture Wire Nylon Jacketed (TFN) insulation, 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.

A.13.1.6 Pump Station Building Systems

A.13.1.6.1 Lightning Protection

The building will have air terminals on the roof interconnected with copper conductors.

A.13.1.6.2 Grounding

A ground ring will be installed around the pump station consisting of 4/0 copper cable and ground rods to establish a resistance of five 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 the National Electric Code.

A.13.1.6.3 Fire Alarm System

A zoned, supervised fire detection and alarm system will be installed. Ionization type smoke detectors will be used in the pump room and the generator room. 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.

A.13.1.6.4 Closed Circuit Television System

A closed circuit television (CCTV) system, per standards for major pump stations, will be installed in the P-1 reservoir pump station.

A.13.1.6.5 Electrical Design

In the electrical design of the pump station, the feeder breakers for the MCCs are located on the main switchboard to allow for the emergency generator to operate all pump station loads.

A.13.1.6.6 Materials of Construction

The switchboard manufacturer should have a distributor and authorized service representative within the State of Florida. The equipment should be manufactured in the United States. Acceptable manufacturers will be Square D, Siemens, Cutler Hammer, Allen-Bradley, General Electric, or approved equal.

Lighting and 480 volt distribution panel boards and lighting fixtures should be made in the U.S. Light fixtures will be industrial grade.

Generators will be Cummins, Onan, Caterpillar, Detroit Allison, or approved equal. Automatic transfer switches will be Cummins, Onan, Asco, Zenith, or approved equal.

Galvanized Rigid Steel conduit will be Allied, Triangle, or approved equal. PVC conduit will be Carlon, Certain-Teed, or approved equal. Liquid-tight conduit will be Electri-Flex, Carol Cable, Anamet, or approved equal.

Wire and cable will be Okonite, Alpha, or approved equal.

A.13.1.7 Valve Operators and Controls

The valve operators will be similar to Limitorque operators which have an integral reversing starter, limit switches, control power transformer, open, stop, and close pushbuttons, and local-remote selector switch. The operators will require 480 volt, three phase power from the MCCs. A local mounted safety disconnect switch will be provided near each operator.

A.13.1.8 Gate Operators and Controls

The gate operators will be similar to Limitorque operators with the drive motor and limit switches but without the integral reversing starter, control power transformer, open, stop, and close pushbuttons, and local-remote selector switch. The operator voltage, number of phases will be determined based on the available power at each Culvert site. A local mounted safety disconnect switch pushbutton station will be provided near each operator.

A.13.2 PUMP STATION ENGINEERING GUIDELINES

The SFWMD has in place a standard titled "Major Pumping Station Engineering Guidelines" dated May 9, 2008 (DCM-5). That document was used in preparing this basis of design section concerning the electrical design of the A-2 Reservoir pump station.

A.14 INSTRUMENTATION AND CONTROLS

A.14.1 DESIGN CRITERIA

This section defines the instrumentation and controls design criteria for the water control facilities, A-2 Reservoir pump station, and telemetry systems. All systems will be designed in accordance with SFWMD standards. All systems and facilities, as general practice, will be monitored and controlled from a local control system in the pump station. The local control system will be Programmable Logic Controller (PLC) based. Monitoring and control will be available from the Remote Terminal Unit (RTU) of the SFWMD Supervisory Control and Data Acquisition (SCADA) facilities. The existing telemetry system centralized at the SFWMD headquarters will be extended to include the new facilities. Instrumentation and control features will include the following features:

- The master control PLC will be an Allen-Bradley ControlLogix 1756 PLC system. Packaged systems in the pump station will be provided with stand-alone Allen-Bradley ControlLogix PLCs communicating over Ethernet-IP.
- Monitoring and control of remote sites, including gated spillways, gated culverts, and monitoring stations will be over RF radios. Equipment will be controlled by the site's RTU.
- Pumps will have control from the SFWMD Control Center, through the RTU. Gated structures will also have control through the Control Center
- Analog control signals will be 24 VDC, 4-20 mA. Discrete signals will be 24 Volt, direct current (VDC). Interposing relays shall be used where necessary to provide isolation and conversion to 24 VDC. Discrete output signals will interface field devices through interposing relays. Surge suppression shall be provided for all instrumentation. The SFWMD design details will be followed

A.14.2 A-2 RESERVOIR, A-2 STA AND CANALS

The level of the A-2 Reservoir, A-2 STA and all canals associated with the A-2 Reservoir Pump Station (P-1) will be monitored through a Motorola ACE RTU. The signal will be transmitted to the SCADA system for display.

The SFWMD standard for level monitoring is the Waterlog absolute encoder, located in a stilling well. For variable applications exceeding 20 feet, the Rittmeyer pressure transmitter provides increased range. Stilling wells shall be installed in accordance with the SFWMD design details. Water levels in the embankment will be monitored with piezometers. Water quality monitoring will be provided as outlined in the SFWMD design details.

A.14.3 WATER CONTROL FACILITIES

Monitoring and control of the gated spillways (SW-2, SW-3 and SW-4) will be through the RTU. Control of the gates will be either manual or from the control system be based on water levels in the Miami Canal, NNR Canal, A-2 Reservoir Inflow-Outflow Canal and/or the STA 3/4 Inflow canal.

Monitoring and control of gated culverts will be through Motorola ACE RTUs.

A.14.4 A-2 RESERVOIR PUMP STATION

Pump station 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 the remote operation of a station. Electric motor drivers have far less auxiliary systems than a diesel engine driven pump station and therefore, a much simpler control and monitoring system. Electric driven pump stations are also typically not used for flood control applications due to the possibility of power outages during a storm event. Therefore, pump system reliability of a flood control station does not apply to an electric motor driven station. SFWMD pump station auxiliary systems and the start/stop of the driver are controlled by a PLC. The receiving and sending of the control and monitoring data are via an RTU.

A.14.4.1 Operation

In the more recently constructed SFWMD flood control stations, there are typically 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 HMIs within the facility) and remote/auto (same as local/auto but based on Station/Central Key switch permissive in main station control panel - set to Central). Remote operation also consists of operating the engine via the SFWMD's telemetry system with the auxiliaries automatically controlled by the local automation. Remote or manual start-up and shutdown, as well as alarm shutdown sequences for the engine are automatically controlled by the engine's electronics and PLCs.

Emergency shutdown of the engine requires an immediate deceleration of the engine speed to zero. The control panel includes a hard-wired emergency shutdown system to shutdown the engine in the event that a failure occurs and the engine controller fails to initiate its own shutdown sequence. The emergency shutdown also functions when the "manual-auto-off" switch is in the auto mode. The circuit generates a shutdown on the actuation of the emergency stop pushbutton. The trash rake and fuel auxiliary systems are also shutdown during an emergency stop condition.

A.14.4.2 Diesel Engine Driver Control and Monitoring System

The diesel engine's electronic control module (ECM) provides monitoring of vital engine parameters and control of engine operation. The ECM output is data-linked to the engine PLC through a converter that translates automotive protocol standard SAE-J1939 serial data to the required digital input of the input/output (I/O) modules of the PLC. The ECM J1939 output data includes all monitoring data, diagnostic information, and operating history and can be displayed by the PLC's monitor. The Station PLC through a converter shall be networked to the RTU (ACE) for communication to the SFWMD's operational center. The various auxiliary functions, such as low water level shut-down, trash rake operation and alarm, and high reduction gear oil temperature or vibration, are monitored and controlled by the engine's PLC. The PLC shall have the capability to automatically shut the engine down given an alarm condition. Instrumentation signals for station monitors such as stage data, electrical service power phase monitoring shall be connected directly to the RTU (ACE) unit.

A.14.4.3 Logic Controller (PLC)

The microprocessor based controller shall be an Allen-Bradley model or equal for machine level control applications requiring limited I/O quantities and limited communications capabilities. The PLC shall be mounted in the engine control panel and shall provide control and monitoring of the engine, transmission, and the auxiliary systems. The engine ECM I/O serial data via a converter (J1939/DF1-RS232) shall be linked to the I/O module of the controller. The PLC will be in a network linked to other PLCs and from the station PLC a RS232 connection shall be stablished with the RTU (ACE) via a Modbus over serial converter. A station PLC shall be located in the control room with control and monitoring capabilities for all station systems.

A.14.4.4 Display Panel

An Allen-Bradley HMI display panel will be provided for each PLC. The display will be mounted in the engine control panel door as well as in the Pump Station control room.

A.14.4.5 Converter (J1939/DF1-RS232)

A communication device will be provided to convert the J1939 serial communication data to the DF1-RS232 digital input that is required by the PLC.

A.14.4.6 Interface Module (Modbus/RS232)

A ProSoft InRax module MVI-56-MCME module shall be provided for conversion of the network digital output to the serial communication signal that is required by the RTU (ACE).

A.14.4.7 Surge Suppressor

A surge suppressor will be provided that will protect the I/O modules of the PLC from lightning induced surges, electrical fast transients and EMI/RFI noise. The surge suppressor shall meet or exceed 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.

A.14.4.8 Vibration Switch

Each reduction gear shall be provided with a vibration control switch to protect the equipment from damaging shock or vibration. Each switch shall be a 24 VDC powered electro-mechanical device, with two Single Pole, Double Throw (SPDT) snap acting switches rated at 2A up to 30 VDC, and mounted in a National Electrical Manufacturers Association (NEMA) four enclosure. Each switch shall have a remote reset to allow reset of the tripped unit from a remote location, an adjustable time delay to override trip operation for a preset length of time (to prevent trips during transient pump cavitation events, for instance), and a fine adjustment to precisely select the degree of sensitivity.

A.14.4.9 Temperature Monitors

The temperature probes provided shall be resistance temperature detectors (RTDs) and shall comply with ANSI 34. RTDs shall be 100 ohm 3-wire platinum in a Type 304 stainless steel sheath with watertight connection head.

A.14.4.10 Liquid Level Gage

A combination liquid level gage with adjustable low limit switch will be used to provide visual indication of oil level and signal low level conditions.

A.14.4.11 Indicating Level Switches

Indicating level switches shall be provided to signal a low-level condition. Water level sensors shall be installed to signal a low-level condition. Sensors shall be floatless, pressure sensitive, diaphragm actuated switches.

A.14.4.12 Monitoring Instrumentation

Depending on a variety of causative factors, some structures will require to be monitored for water quality, this 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.

A.14.4.13 Station Emergency Power

Given a power outage, the flood control station requires emergency electric power backup. Typically, an adequately sized engine generator is provided for backup station service during the time that utility power is lost. In a large flood control station, redundant system is provided. Controls are provided in the 125 ampere Automatic Transfer Switch (ATS) to either manually or automatically start the engine-generator. A RTU output command will remotely STOP the engine generator should the unit unnecessarily start.

A.14.4.14 Stage Monitors

Upstream and downstream 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. Upon the loss of 120 VAC service, the RTU’s “PULSE ANALOG CIRCUITS” program will intermittently power the 24 VDC power supply and scan the analog water level transmitters.

A.14.5 TELEMETRY

The SCADA system RTU will be located in the control building close to the antenna. The RTU will be a Motorola ACE or equal unit to be compatible with the existing units already installed at other SFWMD locations. The pump station will be remotely monitored through the SFWMD’s SCADA system. This is the 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. SFWMD may require the remote control of the station and the SCADA system, of the station should provide for this type.

A.15 ARCHITECTURAL

A.15.1 DESIGN CRITERIA

A.15.1.1 Introduction

The A-2 Reservoir pump station building will be constructed to accommodate the pumps, motors, generators, and ancillary systems. In addition, adequate area will be provided for a control room, offices, break room, toilet, locker/shower, and mechanical equipment.

A.15.1.2 Design Requirements

A.15.1.2.1 Codes and Standards

Design and specifications of all work will be in accordance with the latest laws and regulations of the Federal government, applicable State and local codes and ordinances, and applicable industry standards. Other recommended standards will be used where required to serve as guidelines for design, fabrication, and construction when not in conflict with the above standards. The building will be designed in accordance with Florida Accessibility Code and Americans with Disabilities Act Accessibility Guidelines (ADAAG). Anti-Terrorism/Force Protection measures for the building will be addressed during the 30 percent design phase. There is no requirement to incorporate the principles of Sustainable Design and Development for the building.

A.15.1.2.2 Life Safety

The building will be designed to meet the minimum construction and life safety requirements as required by the applicable codes and criteria. As described in **Section A.16**, appropriate type, size, and quantity of fire extinguishers will be provided in compliance with all applicable fire and life safety codes, including a sprinkler system in designated areas.

A.15.1.2.3 Material and Life Cycle

The building shall be designed to minimize life cycle cost, energy consumption, and maintenance through proper selection of mass, form, materials, and construction standards. Integrally colored materials shall be used as much as possible to eliminate painting. The design life of the building shall be a minimum of 50 years. Refer to **Sections A.10.3.5** and **A.10.3.6** 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 A.15.1-1**.

Table A.15.1-1. Exceptions to the Building Service Life Span

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

A.15.2 EXTERIOR ARCHITECTURAL FEATURES

A.15.2.1 Shell

The elements forming usable enclosed space and separating that space from the external environment comprise the shell and consist of the following.

A.15.2.2 Superstructure

The superstructure includes all elements forming floors and roofs above grade, and the elements required for their support, insulation, fireproofing and fire stopping. The structural system for the superstructure shall be a steel or reinforced concrete frame with reinforced concrete walls and poured in place reinforced concrete roof and shall be designed in accordance with the applicable building codes as defined in **Section A.10.5**.

A.15.2.3 Exterior Enclosure

The exterior enclosure includes all essentially vertical elements forming the separation between exterior and interior conditioned space, including exterior skin, components supporting weather barriers, and jointing and interfacing components; not including the interior skin unless an integral part of the enclosure. The exterior enclosure will be a reinforced concrete wall with required exterior paint/coating. Thermal performance for the exterior enclosure is not applicable to main equipment rooms. Exterior enclosures will be insulated for all air-conditioned spaces.

All exterior doors will be painted, hollow metal doors with painted metal frames. Insulated doors will secure air-conditioned spaces. Overhead doors shall be roll-formed galvanized steel construction, electrically operated and shall be sized to fit the largest equipment for the building. Louvers will be designed as required for ventilation of the spaces and equipment. The building wall openings for fans and louvers will have missile barrier protection over screens constructed to withstand 155 mph wind loading and windborne debris in accordance with the wind load design criteria specified in **Section A.10.3.6**. All doors and louvers will be hurricane impact resistant.

A.15.2.4 Roofing

Roofing includes all elements forming weather and thermal barriers at horizontal roofs, decks, and roof fixtures. A single ply roofing membrane will be used over the reinforced poured concrete roof deck. The roof will be sloped to stainless steel drain scuppers formed through the parapet. The roof runoff is directed down the walls via downspouts made from hollow structural tubing to resist missile impact during hurricane events. All flashing, trim, and accessories will be of stainless steel sheet metal. Access to the roof will be provided by a roof hatch and will be controllable by authorized personnel only.

A.15.3 INTERIOR ARCHITECTURAL FEATURES

A.15.3.1 Floor

All floor slabs will be sealed reinforced poured concrete.

A.15.3.2 Partitions

Partitions provided for physical separation between spaces will be constructed to achieve fire ratings required by code; appropriate security between adjacent spaces; and visual, acoustical, olfactory, and atmospheric isolation as necessary to maintain desirable conditions in each space. Partitions will comprise the following elements: Fixed partitions of fully-grouted, reinforced, full-height CMU; and partial height partitions of fixed, solid, opaque visual barriers for toilet compartments. The control room will have glass panels to allow the operator an unobstructed view of the operation floor. The control room/break room will be designed for sound proofing with a minimum Sound Transmission Coefficient (STC) of 49.

A.15.3.3 Interior Doors and Windows

All interior doors shall be painted, hollow metal doors with painted metal frames. Interior windows will be provided between adjacent spaces. Fixed interior windows and operable interior windows, when closed, will function as partition elements and will not degrade performance of partitions below the levels specified. Sound insulated doors and windows will be provided to meet the STC of not less than 49.

A.15.3.4 Interior Finishes

Offices/Control Room/Break Room

- Wall: Painted
- Floor: Non-skid ceramic tiles
- Ceiling: Suspended acoustical ceiling tiles

Toilets/Showers

- Wall: Ceramic tiles
- Floor: Non-skid ceramic tiles
- Ceiling: Moisture resistant gypsum board

Locker Room

- Wall: Painted
- Floor: Non-skid ceramic tiles
- Ceiling: Moisture-resistant gypsum board

Equipment Room/Maintenance Shop/Janitor's Closet

- Wall: Painted
- Floor: Sealed concrete
- Ceiling: None. All exposed concrete will be painted

Fan/Filter Rooms

- Wall: Painted
- Floor: Sealed concrete
- Ceiling: None. All exposed concrete will be painted

A.15.3.5 Vertical Circulation

Stairs will be provided for access to mechanical spaces and equipment mezzanines. Also, a vertical lift that meets accessibility requirements will provide access to the control room.

A.15.4 INTERIOR FIXTURES

Interior fixtures permanently attached to interior walls, ceilings, and floors, except for equipment items, will be provided and comprise the following elements:

A.15.4.1 Identifying Devices

Informational accessories, including room numbers, signage, and directories.

A.15.4.2 Storage Fixtures

Items intended primarily for storing or securing objects, materials, and supplies, including cabinets, casework, and shelving.

A.15.4.3 Accessory Fixtures

Specialty items intended to provide service or amenity to building interiors, including toilet and bath accessories, visual display surfaces, and telecommunications fixtures.

A.15.4.4 Interior Fixtures

Other items fixed to the interior construction that enhance comfort or amenity in building spaces.

A.16 HVAC, PLUMBING AND FIRE SUPPRESSION SYSTEMS

A.16.1 DESIGN CRITERIA

The following describes the basis of mechanical design and criteria associated with the heating, ventilating, and air conditioning (HVAC); plumbing; and fire suppression systems for the A-2 Reservoir pump station. **Table A.16.1-1** details the A-2 Reservoir project site design criteria; **Table A.16.2-1** details the indoor design criteria for the A-2 Reservoir project.

Table A.16.1-1. Project Site Design Criteria

Site Elevation	
Above sea level, feet NAVD88 (feet NGVD29)	8.50 (9.93)
Site Location	
North latitude, degrees	26
West longitude, degrees	80
Ambient Design Temperatures[1]	
Winter, design dry bulb, degrees Fahrenheit (°F)	42
Summer, design dry bulb/mean coincident wet bulb, °F	93/77
Dehumidification, design dew point, °F	78
Degree Days	
Heating (Base 65°F), days	418
Cooling (Base 50°F), days	8,924
Rainfall Intensity[2]	
Actual, inches/hour	4.7
Design, inches/hour	5

[1] The winter and summer design temperatures are based on the American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE) frequency levels 99.6 percent and 1 percent, respectively.
[2] The actual rainfall intensity rate is based on a 60-minute duration and 100 year return period.

A.16.2 HVAC

The following is a description of the HVAC systems:

A.16.2.1 Heating Systems

Electric wall heaters will be provided in the men’s and women’s toilet/locker rooms for supplemental heat.

A.16.2.2 Ventilation Systems

A forced air ventilation system will be provided for the operating floor area of the pump station. The system will utilize centrifugal fans for supply and propeller fans for exhaust. The supply air system will consist of louvers for air intake, automatic roll filters for filtering, centrifugal fans for supply, and a below floor air distribution header for supplying air to the operating floor area.

The roll filters and supply fans will be located in rooms along the wall opposite from the pump’s engines. The exhaust fans will be located high above the floor on the engine side of the pump station. The ventilation system will remove the heat gains from the equipment as well as supply make up air for the engine air intakes.

The intake and exhaust louvers will be Miami-Dade County approved, and will be provided with missile barriers.

The ventilation system fans will be controlled by their individual ON-OFF-AUTO selectors switches. When the exhaust fan selector switches are in the "AUTO" position, the exhaust fans will be interlocked with the controls for the supply fans. When the supply fan selector switches are in the "AUTO" position, the quantity of supply fans operated will be automatically controlled based upon the quantity of engines operating in the engine pump room and controlled by the room thermostats in the engine pump room.

A.16.2.3 Air Conditioning Systems

The air conditioning systems will be split system type heat pumps. A heat pump will be provided for the shop, control room, break room, locker room, and restroom. The heat pump serving the break room, locker room, and restroom will also be ducted to provide a backup to the control room's air conditioning system. Each heat pump will be provided with a backup emergency electric heating coil. Each unit will be controlled by a remote wall mounted thermostat to maintain the desired space temperature. The air handling units and heat pumps will be located inside the pump station. The locker room, restroom, and janitor's closet will be exhausted by duct fans ducted to exhaust louver or wall caps.

Table A.16.2-1. Indoor Design Criteria for Project

Area	Design Temperatures (°F) (1)			Ventilation Requirements	Ventilation Notes
	Summer	Winter			
	Design	Design	Setpoint		
Engine Pump Room	100	50	50	1.5 cfm/sf (C)	Note 2
Shop	78	72	72	--	Note 4
Janitor's Closet	100	--	--	--	Note 3
Control Room	70	70	70	--	Note 4
Break Room	78	72	72	--	Note 4
Locker Room	78	72	72	--	Note 3
Restroom(s)	78	72	72	--	Note 3

AC/HR = designates air changes per hour
(C) = designates the ventilation system operates continuously
(I) = designates the ventilation system operates intermittent
(1) = Indoor conditions reflect operating temperatures for personnel comfort, code/standard recommendations, or equipment protection.

Notes:

1. The ventilation system will be sized on the more restrictive of the AC/HR (or cubic feet per minute per square foot – cfm/sf) listed, or the airflow required to maintain the indoor design temperature based on the summer outside design temperature
2. Additional intermittent ventilation will be provided as required to maintain the indoor design temperature based on the summer outside design temperature, or to meet the engine combustion air requirements
3. The exhaust rate will be based on the most stringent requirement of: 0.5 cfm/sf of floor area; 50 cfm per toilet or urinal; or 100 cfm minimum
4. The ventilation rate will be based on the exhaust requirements or as required by ASHRAE 62- 1989, whichever is more stringent

A.16.3 POTABLE WATER

Investigation of potable water usage at the existing major pumping stations (G-310 and G-370) indicates low demand and infrequent use of potable water. Potable water is supplied to a kitchen sink, restrooms and showers. Bottled water is used for drinking. It was reported that the current potable water systems are sized for more demand than the system experiences, and as a result, the treatment systems are experiencing problems due to a lack of flow.

As an alternative to the potable water supply system installed at the existing pumping stations, which is canal water processed through sand filters and reverse osmosis (RO) membranes, the use of a shallow water well will be considered. Treatment of this water could be with aeration, canister filtration, chlorination, and softening. The design will incorporate storage that will serve the typical low demand but also accommodate the infrequent periods of larger demands when the pump station houses personnel during extreme weather events. Changing the potable water source to a well would require water quality sampling and analyzing, and based on the results of the analysis, an appropriate water purification system would be compared to the current RO treatment system. Alternative systems will be considered during the engineering design phase of the project.

The potable water system selected shall supply potable water to restrooms, sinks and showers. An electric-powered domestic water heater will be provided to supply water at 120 degrees Fahrenheit to the sinks and showers.

A.16.4 FRESH WATER SUPPLY SYSTEM

A fresh water system will be provided to supply water for lubricating water, seal water, and hose bibs for area washdown. The fresh water system will be supplied by water from the adjacent canal, and treated using in-line strainers.

A.16.5 COOLING WATER SYSTEM

Cooling water for use in the pump engines and gear reducers will be provided by strained water from the adjacent canal. This will be a once-through cooling system, and the used water will be collected and discharged back into the canal.

A.16.6 SANITARY SYSTEM

All plumbing fixtures that require drainage will discharge to the sanitary system. In addition, floor drains located in the locker room and restrooms will discharge to the sanitary system. Floor drains will not be provided in the pump room so that potential oily waste will not be discharged to the sanitary system. Sanitary drainage from the building will be collected in a septic tank. Soil tests will be conducted to verify the efficiency of a septic drain (leach) field. If the soil conditions are not favorable for a drain field, or the amount of discharge is determined to be minimal, the septic tank could be used for storage of wastewater and pumped regularly for removal off site.

A.16.7 STORMWATER SYSTEM

Storm drainage will be collected from the roof drains and leaders. All storm drainage at the pump station will be routed to the forebays.

A.16.8 FIRE SUPPRESSION SYSTEM

It is expected that an automatic fire sprinkler and detection system will be required for the entire pump station 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 shall 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.

A.17 ACCESS AND SECURITY

A.17.1 ACCESS

Public access to the A-2 Reservoir and A-2 STA and associated facilities will be for recreational opportunities as discussed in **Appendix F**. Public access to the top of the A-2 Reservoir and A-2 STA embankments will be provided at various locations along the perimeter of the A-2 Reservoir as described in **Appendix F**.

The public and SFWMD staff will be able to access the A-2 Reservoir and A-2 STA from an access road that connects to the west side of U.S. Hwy. 27 near the northeast corner of the existing A-1 FEB as well as from the existing levee road on top of the L-23 Levee along the east side of the Miami Canal. From the access road that connects to U.S. Hwy. 27 or from the L-23 Levee road, the following areas will be accessible.

- Top of the A-2 Reservoir and A-2 STA embankments
- Inside the A-2 Reservoir and A-2 STA
- Outside the A-2 Reservoir and A-2 STA

Access ramps and pullout areas (for turnaround and passing maneuvers) will provided along the A-2 STA and A-2 Reservoir embankments at the required intervals per DCM-4.

A.17.2 SECURITY

The A-2 Reservoir and A-2 STA and all associated project features will follow the security guidelines of the SFWMD and the U.S. Department of Homeland Security.

A.17.2.1 Fences and Gates

The A-2 Reservoir and A-2 STA is located in a remote rural area. It is not anticipated that the entire A-2 Reservoir and A-2 STA will be surrounded by a perimeter fence. The proposed A-2 Reservoir pump station and other proposed gated water control structures will have controlled access through the use of fences and gates. Fences at the water control structures will be provided with locked gates keyed to match the SFWMD's current lock system. Electric gates and locks are not anticipated. Gates will be provided at vehicle access points located along the A-2 Reservoir and A-2 STA embankments.

A.17.2.2 Site Monitoring at A-2 Reservoir Pump Station

A closed circuit television system will be employed for security at the proposed A-2 Reservoir pump station. Cameras will be located at each entrance to the pump station control building as well as strategically located within the building. Cameras will also provide views of vehicle entrance gates.

A.17.2.3 Building Access at A-2 Reservoir Pump Station

Items that will be considered when controlling access to the building will include:

- Door position switches
- Interior motion sensors
- Keypad access with timed alarm override

All security features and elements will be coordinated with the SFWMD prior to final design.

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A.18 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 A.5.1**, the A-2 Reservoir is a high hazard impoundment. U.S. Hwy. 27 and the farmland to north of the project site will likely be significantly impacted in the event of a breach, which will lead to life threatening conditions for motorists along U.S. Hwy. 27 and nearby farm personnel as well as impede emergency evacuation routes for southern Florida. The A-2 Reservoir will need a comprehensive EAP that reflects its classification as a high hazard impoundment. The EAP would need to be developed in conjunction with impoundment breach modeling of the A-2 Reservoir that should be completed during the PED phase of the project.

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ANNEX A-1
HYDRAULIC DESIGN

- Preliminary Conveyance Assessment for Lake Okeechobee Releases through the Miami & NNR Canals Report
- Pump Station Hydraulic Calculations

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Preliminary Conveyance Assessment for Lake Okeechobee Releases through the Miami & NNR Canals

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1.0 Objective and Scope

The purpose of the canal conveyance evaluation presented here is to provide a preliminary assessment of the capacities of the Miami and North New River (NNR) canals under the range of proposed operational constraints associated with the releases of excess water from Lake Okeechobee (LO). Sustained discharges of excess lake water through spillways S-351 and S-354 at the rates of 200 cfs and 1,000 cfs, respectively, are currently under consideration. In addition, pumping facilities at structures G-370 and G-372 will be used to maintain their head water stages within the range of 8.0 – 9.0 feet NGVD under wet conditions and at 11.0 feet NGVD under dry conditions.

2.0 Method of Analysis

2.1 Software and Backwater Calculations

The capacity of each canal to convey required flows was evaluated by computing the steady water surface profiles associated with the sustained flows. The calculations were carried out with HecRas version 5.0.3. This software uses the Standard Step method to compute steady water surface profiles.

2.2 Methodology, Data and Assumptions

The capacity of each canal was evaluated with respect to its ability to convey the specified lake releases along with the maximum expected inflows of pumped agricultural storm water. All inflows of excess storm water were assumed to discharge directly into either the Miami or NNR canal at locations of tributary canal junctions or locations of outfalls provided by the Everglades Technical Support Bureau. For conservative design, it was assumed that all permitted discharges occurred simultaneously.

For both canals, their capacities were evaluated in a number of lake release scenarios that addressed a variety of design and water management conditions under which lake releases could conceivably occur. These scenarios include:

- Removal rates of both ¾ inch per day and 1 inch per day.
- Both existing and improved canal geometry.
- Controlled G-372 and G-370 head water stages of 7.5 – 9.0 feet NGVD.
- A maximum tolerable tail water stage of 12.5 feet NGVD at both S-351 and S-354.

The existing canal geometry was based on survey data acquired in 2003. Two improvements to the existing geometry were considered. First, each canal has at least one location in the vicinity of its junction with the Bolles Canal where its bottom is elevated. Historically, these features have been colloquially referred to as “humps”. The removal of each hump as a design improvement was considered. Additionally, in order to satisfy the upstream stage constraint mentioned above, widened trapezoidal cross sections that could be located within the available right-of-way for the Miami canal were evaluated. No widened cross sections were considered for the NNR canal due to right-of-way limitations along with the proximity of the U.S. 27 embankments. **Figure 1** shows the approximate limits of widening in the Miami Canal and hump removal in the NNR Canal that was modeled.

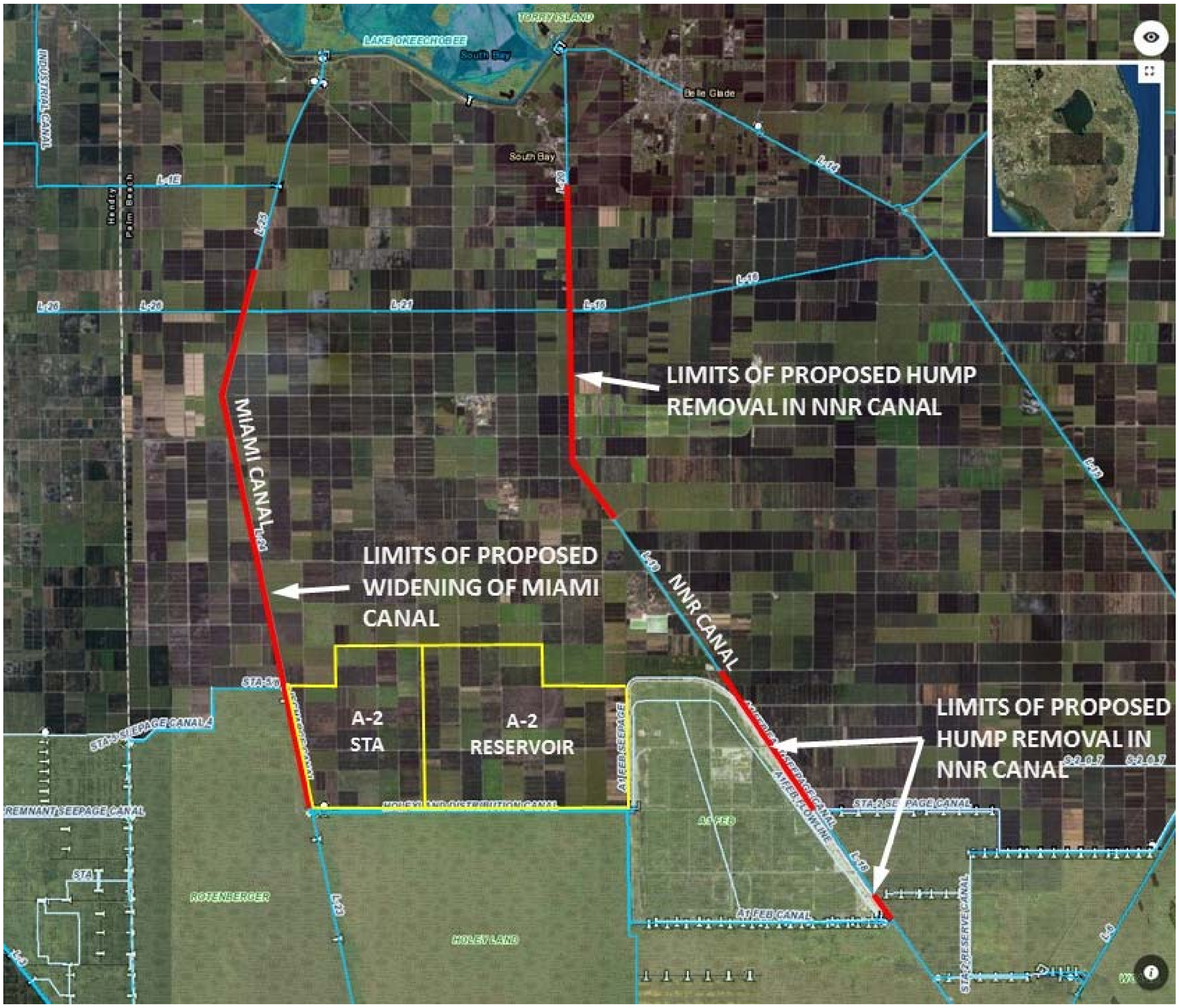

Figure 1. Limits of modeled canal conveyance improvements

The scenarios with the lake releases, geometries and storm water inflow rates indicated were modeled for each canal by performing backwater calculations between the downstream end with the specified G-370 or G-372 head water stage and the upstream end of the canal at either S-351 or S-354. **Table 1** summarizes the suite of conveyance scenarios simulated. Although scenarios with a downstream boundary stage of 7.5 feet were considered, the maintenance of a G-370 or G-372 head water stage lower than 8.0 feet is not currently feasible without mechanical improvements to each pump station.

Table 1. Lake Okeechobee release scenarios

Canal	LO Release (cfs)	Storm Water Removal Rate (in/day)	Downstream Control Stage (ft NGVD)	Canal Geometry/Design
NNR	200	3/4, 1	7.5, 8.0, 8.5, 9.0	Existing, existing with humps removed
Miami	1,000			Existing, existing with humps removed, widened trapezoidal

3.0 Results

3.1 NNR Canal

Table 2 summarizes the computed S-351 tail water stages associated with the conveyance analysis for the NNR canal. For each combination of runoff removal rate and canal geometry, the goal was to identify the downstream maximum control stage that would enable the canal to convey the lake release and storm water inflows without the upstream stage exceeding 12.5 feet NGVD. The gray boxes in Table 2 designate scenarios that were not simulated since it can be deduced from other scenario results whether the upstream stage would exceed the allowable limit. It is apparent that the NNR canal cannot convey both the designated lake release and the storm water removal rate of 1 inch/day without exceeding the upstream stage limit, even with the humps removed. Furthermore, with the humps removed the canal should be able to convey the lake release along with a storm water removal rate of ¾ in/day without exceeding 12.5 feet at S-351 as long as the G-370 head water stage does not exceed 9.0 feet. If no channel improvements are carried out, the G-370 head water stage should not exceed 8.0 feet.

Table 2. Computed S-351 tail water stages for a lake release rate of 200 cfs

Canal Geometry	Pumped Runoff Inflow Rate (in/day)	Downstream Stage (feet NGVD) at G-370			
		7.5	***8.0***	***8.5***	***9.0***
Existing	3/4		12.59		
	1	14.21			
Existing with Humps Removed	3/4		12.08	12.30	12.53
	1	13.60			

Table 3 provides the computed average cross section velocity at the downstream end of the channel where the maximum flow and minimum stage occur. All values exceed the desired limit of 2.5 ft/s. Consequently, some channel stabilization measures may be needed in the lower reaches of the NNR canal near G-370.

Table 3. Computed NNR canal velocities (ft/s) at G-370 for a lake release rate of 200 cfs

Canal Geometry	Pumped Runoff Inflow Rate (in/day)	Downstream Stage (feet NGVD) at G-370			
		7.5	***8.0***	***8.5***	***9.0***
Existing	3/4		3.75		
	1	5.15			
Existing with Humps Removed	3/4		2.96	2.85	2.75
	1	4.02			

3.2 Miami Canal

3.2.1 Existing Geometry with Minor Improvements under Storm Conditions

Table 4 summarizes the computed S-354 tail water stages associated with the conveyance analysis for the Miami canal. For each runoff removal rate, the goal was to identify the downstream maximum control stage that would enable the canal to convey the lake release and stormwater inflows without the upstream stage exceeding 12.5 ft-NGVD. It is apparent that the Miami canal cannot convey both the designated lake release and either storm water removal rate without exceeding the upstream stage limit, even with the humps removed.

Table 4: Computed S-354 tail water stages for a lake release of 1,000 cfs

Canal Geometry	Runoff Rate (in/day)	Downstream Stage (ft-NGVD) at G-372			
		7.5	**8**	**8.5**	**9**
Existing	0.75	14.14	14.27	14.42	14.57
	1.00	15.79	15.89	16	16.11
Existing, with humps removed	0.75	13.71	13.87	14.03	14.2
	1.00	15.39	15.5	15.62	15.75

3.2.2 Improved Canal Geometry with an Improved Constant Cross Section under Storm Conditions

In order to resolve the capacity limitations identified above, new design cross sections were developed for the Miami Canal to carry the designated lake release and storm water removal rates while meeting the upstream stage limit. In these model scenarios all cross sections had a uniform bottom elevation of -10 ft-NGVD and side slopes of 1V:1H. The required bottom widths determined through model simulations are presented in **Tables 5 and 6**.

Table 5: Required bottom width for the Miami canal with a lake release of 1,000 cfs + 0.75 inch/day runoff rate

Downstream Stage (ft-NGVD)	Bottom Width Required (ft)
7.5	65
8	65
8.5	70
9	75

Table 6: Required bottom width for the Miami canal with a lake release of 1,000 cfs + 1 inch/day runoff rate

Downstream Stage (ft-NGVD)	Bottom Width Required (ft)
7.5	80
8	85
8.5	90
9	90

Table 7 provides the computed average cross section velocity at the downstream end of the channel where the maximum flow and minimum stage occur. All values exceed the desired limit of 2.5 ft/s. Consequently, some channel stabilization measures may be needed in the lower reaches of the Miami canal near G-372.

Table 7: Computed Miami canal velocities (ft/s) at G-372 for a lake release rate of 1,000 cfs

Canal Geometry	Runoff Rate (in/day)	Downstream Stage (ft-NGVD)			
		7.5	8	8.5	9
Design	0.75	3.29	3.18	2.90	2.66
	1.00	3.52	3.24	2.99	2.90

The improved design cross section with the widest bottom width of 90 feet is depicted below in **Figures 2 – 5** along with existing cross section geometries and ROW widths. These drawings are for conceptualization purposes only and do not include the levees on either side of the Miami canal. The river station numbers shown are in units of feet, where the river station at the downstream end is zero.

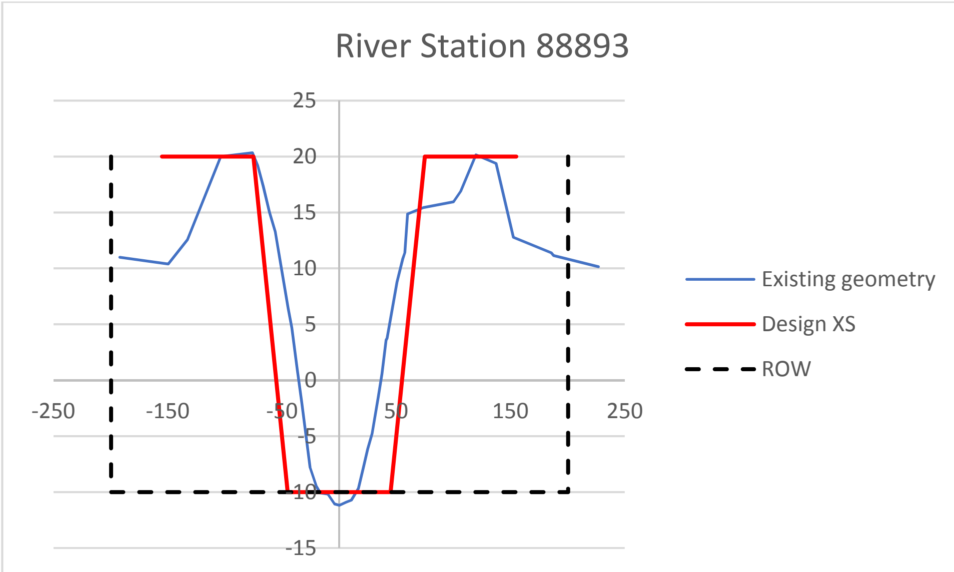

Figure 2. Existing and improved cross sections at station 88893

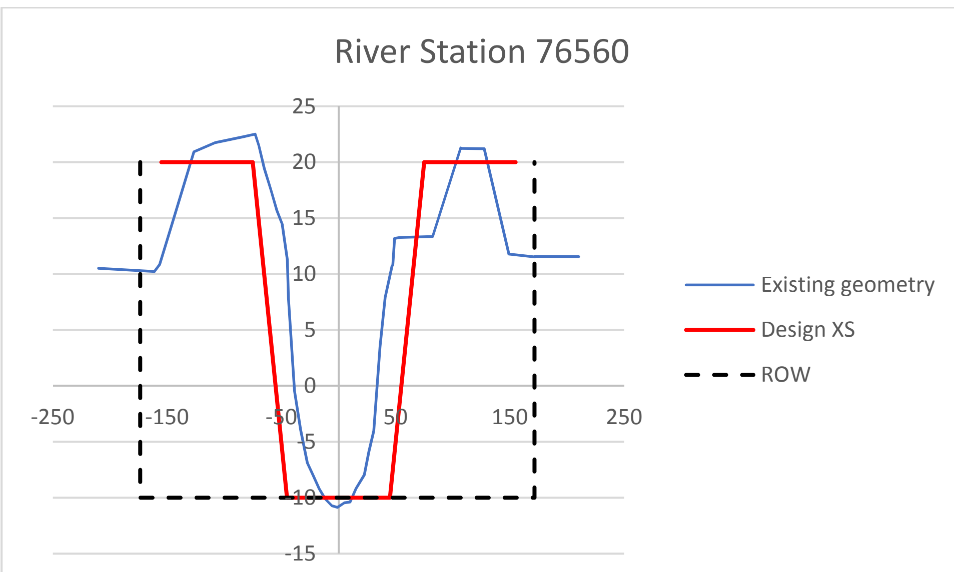

Figure 3. Existing and improved cross sections at station 76560

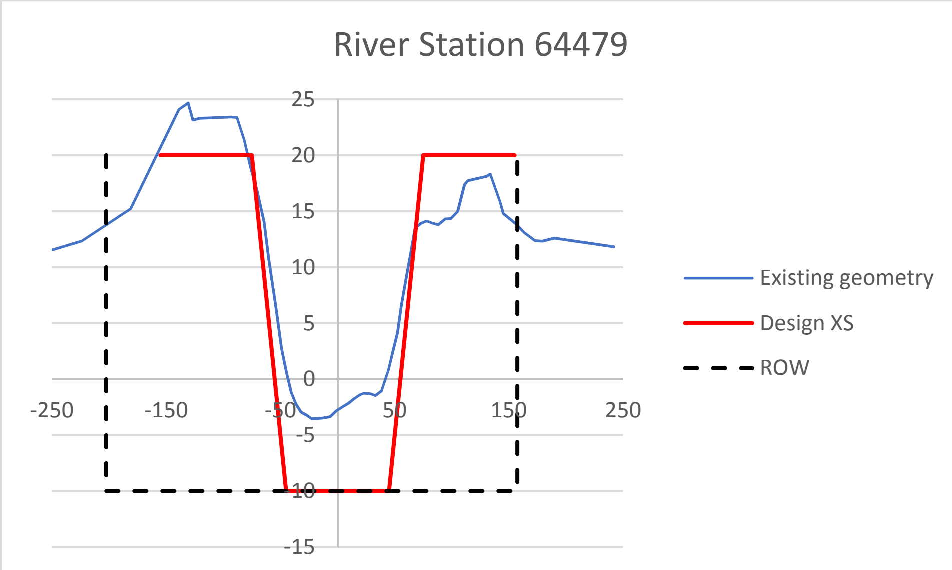

Figure 4. Existing and improved cross sections at station 64479

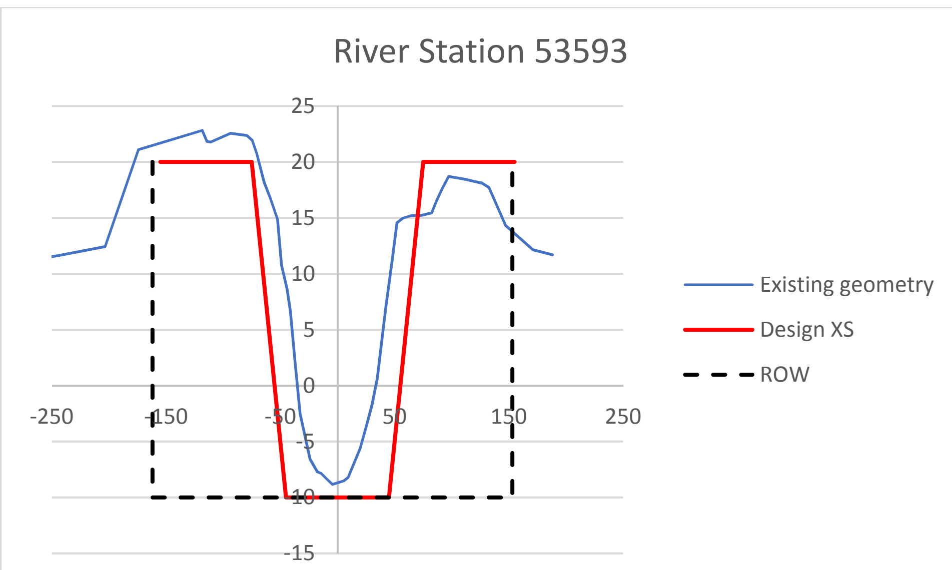

Figure 5. Existing and improved cross sections at station 53593

3.2.3 Improved Canal Geometry with an Improved Variable Cross Section under Storm Conditions

In order to reduce planned construction costs, additional scenarios were modeled in which the improved design canal geometry varied along its length. The total length of the Miami canal was divided into three reaches (**Figure 6**): the first reach spans river station 98081 through 70050, the

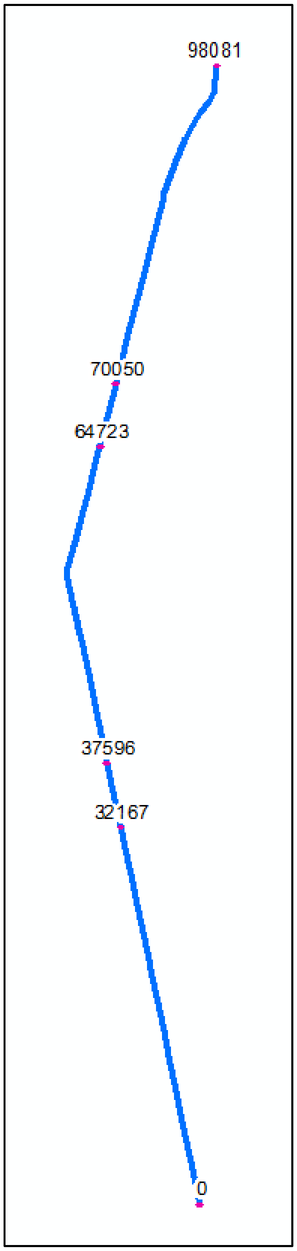

Figure 6. Locations of design geometry changes

second section is located between river stations 64723 and 37596, and the third section extends from river station 32167 through 0. These model scenarios also included the following features:

- Max allowable stage at the upstream boundary: 12.5 ft-NGVD
- Downstream boundary: 8 ft-NGVD – 9 ft-NGVD
- Lake Okeechobee discharge: 1,000 cfs
- Watershed discharges: 0.75 in/day (3,751 cfs)
- Total discharges: 4,751 cfs

Improved cross sections were designed to convey the lake release along with the accumulated storm water discharges while not exceeding the maximum allowable stage of 12.5 ft-NGVD at the upstream boundary. The design cross sections had a uniform bottom elevation of -10 ft-NGVD and side slopes of 1V:1H. The required bottom widths for the design cross sections for this scenario are presented in **Table 8**.

Table 8: Scenario 1A Design Cross Section Bottom Widths

Downstream Stage (ft-NGVD)	Upstream Stage (ft-NGVD)	Cross Section Bottom Widths (ft)		
		RS 98081 - RS 70050	RS 64723 - RS 37596	RS 32167 - RS 0
8	12.50	Existing	50	90
8.5	12.50	Existing	55	90
9	12.43	Existing	65	90

3.2.4 Existing Canal Geometry under Dry Conditions

Several water supply scenarios based on the existing canal geometry were also examined, where 1,000 – 1,500 cfs discharges were released from Lake Okeechobee while up to 500 cfs of irrigation withdrawals occurred along the canal. In these scenarios, the G-372 head water stage was maintained at 10.5 ft-NGVD and the maximum allowable stage at the upstream boundary was 11.5 ft-NGVD. Table 7 presents the computed S-354 tail water stages. It appears that the existing canal can accommodate releases up to 1,500 cfs, depending on the irrigation withdrawal rates.

Table 8: Computed S-354 tail water stages for the water supply scenarios

Lake Okeechobee Discharge (cfs)	Irrigation Withdrawals (cfs)	Upstream Stage (ft-NGVD)
1,000	0	11.18
1,000	500	10.9
1,500	500	11.54

3.2.4 S-354 Capacity

An additional analysis was also performed to check the ability of structure S-354 to pass 1,000 cfs (500 cfs per gate) given the expected head water and tail water stages as well as the maximum allowable gate openings of the structure. To represent flood control conditions, the tail water of S-354 was held at 12.5 ft-NGVD while the head water of S-354 ranged from 13 to 15.5 ft-NGVD. For the water supply conditions, the tail water of S-354 was held at 11.5 ft-NGVD while the head water of S-354 ranged from 13 to 15.5 ft-NGVD. The results are presented in **Table 9**. Under both the wet and dry conditions examined, S-354 was able to release 1,000 cfs.

Table 9: S-354 capacity check

Scenario	TW (ft-NGVD)	HW (ft-NGVD)	Max G_0 (ft)	Q_{gate} (cfs)	Q_{total} (cfs)
Flood control	12.5	13.00	8	867	1734
	12.5	15.50	3.8	961	1922
Water supply	11.5	13.00	4.7	855	1710
	11.5	15.50	2.9	838	1675

4.0 Recommendations

If storm water discharges within the NNR canal basin are limited to ¾ in/day, the results of the conveyance assessment suggest that the existing NNR canal can convey the resultant inflows along with a sustained LO release rate of 200 cfs if the head water stage at G-370 does not exceed 8.0 feet NGVD. If the existing known channel bottom humps are removed, the maintained G-370 head water stage can be increased to 9.0 feet NGVD. In either case, some channel stabiliization measures may be needed within the lower reaches of the canal near G-370 due to velocities exceeding 2.5 ft/s. Widening the channel to improve channel capacity does not appear to be an option for the NNR canal due to right-of-way limitations.

Modeling results indicate that the existing Miami canal cannot convey both the LO release rate of 1,000 cfs and the watershed discharges of ¾ in/day without exceeding the maximum S-354 tail water stage of 12.5 feet NGVD. This is the case even if the G-372 head water is maintained at 7.5 feet NGVD which is below the minimum sustainable stage of 8.0 feet. Moreover, this outcome cannot be rectified by removing the canal bottom hump. Additional model simulations carried out with improved trapezoidal cross sections indicate that a canal cross section with a bottom elevation of -10 feet, side slopes of 1H:1V and a bottom width of 65 – 75 feet would be needed. It needs to be verified, however, that such channel improvements can be contained within the existing right of way boundaries.

Under dry season conditions with no storm water inflows, the ability of the Miami canal to convey the same LO release rate with a maintained G-372 head water stage of 10.5 feet was also evaluated. The results indicate that the existing canal can convey a 1,000 cfs sustained release rate with a S-354 tail water stage of 11.18 feet, which is below the allowable stage of 11.5 feet. If 500 cfs of irrigation withdrawals occur along the canal, the computed S-354 tail water stage decreased to 10.9 feet. If, under these same conditions, the LO release rate was increased to 1,500 cfs, the S-354 tail water stage increased to 11.54 feet. These results suggest that the existing canal can convey the LO release rate of 1,000 cfs under dry season conditions and it may be possible to increase this rate, depending on the amount of irrigation withdrawals that are occurring.

SOUTH FLORIDA WATER MANAGEMENT DISTRICT
Structure P1 4600 CFS Pump Station
WILLIAMS AND HAZEN CALCULATIONS

DSN: Van Kile
12/28/2017
CHK:
Date:

Williams and Hazen Equation: $s=(v/(c*r^{0.63}*0.001^{-.04}))^{1.85}$

Where: s=pipe slope in feet per 1000 feet of pipe=head loss
v=velocity in the pipe
c=roughness coefficient,generally ranging from 80 to 140 depending on pipe material and smoothness
r=hydraulic radius or flow area divided by the wetted perimeter; diameter/4 for full pipes
$0.001^{-.04}$ is a constant

Pipe Flow Data based on William and Hazen Equation

Pipe Dia (in)	96	96	96	96	120	120	120	120	108	108	108	108
Pipe Dia (ft)	8	8	8	8	10	10	10	10	9	9	9	9
Pipe Flow (CFS)	800	700	600	400	800	700	600	400	400	325	250	200
Pipe Area (ft^2)	50.3	50.3	50.3	50.3	78.5	78.5	78.5	78.5	63.6	63.6	63.6	63.6
Velocity (FPS)	15.9	13.9	11.9	8.0	10.2	8.9	7.6	5.1	6.3	5.1	3.9	3.1
Vel. Hd. (ft)	3.93	3.01	2.21	0.98	1.61	1.23	0.91	0.40	0.61	0.41	0.24	0.15
C	140	140	140	140	140	140	140	140	140	140	140	140
r (full pipe)	2	2	2	2	2.5	2.5	2.5	2.5	2.25	2.25	2.25	2.25
'/k'	4.788	3.740	2.812	1.328	1.617	1.263	0.950	0.448	0.749	0.510	0.314	0.208

Pipe Dia (in)	132	132	132	132	132	132	144	144	144	144
Pipe Dia (ft)	11	11	11	11	11	11	12	12	12	12
Pipe Flow (CFS)	800	700	600	400	300	200	800	700	600	400
Pipe Area (ft^2)	95.0	95.0	95.0	95.0	95.0	95.0	113.1	113.1	113.1	113.1
Velocity (FPS)	8.4	7.4	6.3	4.2	3.2	2.1	7.1	6.2	5.3	3.5
Vel. Hd. (ft)	1.10	0.84	0.62	0.28	0.15	0.07	0.78	0.59	0.44	0.19
C	140	140	140	140	140	140	140	140	140	140
r (full pipe)	2.75	2.75	2.75	2.75	2.75	2.75	3	3	3	3
'/k'	1.017	0.794	0.597	0.282	0.166	0.078	0.666	0.520	0.391	0.185

Pipe Dia (in)	96	96	96	96	96	84	84	84	84	72	72	72	72
Pipe Dia (ft)	8	8	8	8	8	7	7	7	7	6	6	6	6
Pipe Flow (CFS)	250	200	150	125	100	400	325	275	200	250	200	150	125
Pipe Area (ft^2)	50.3	50.3	50.3	50.3	50.3	38.5	38.5	38.5	38.5	28.3	28.3	28.3	28.3
Velocity (FPS)	5.0	4.0	3.0	2.5	2.0	10.4	8.4	7.1	5.2	8.8	7.1	5.3	4.4
Vel. Hd. (ft)	0.38	0.25	0.14	0.10	0.06	1.68	1.11	0.79	0.42	1.21	0.78	0.44	0.30
C	140	140	140	140	140	140	140	140	140	140	140	140	140
r (full pipe)	2	2	2	2	2	1.75	1.75	1.75	1.75	1.5	1.5	1.5	1.5
'/k'	0.557	0.368	0.216	0.154	0.102	2.544	1.732	1.272	0.706	2.257	1.494	0.877	0.626

Pipe Dia (in)	78	66	66	66	66	60	60	60	60	60
Pipe Dia (ft)	6.5	5.5	5.5	5.5	5.5	5	5	5	5	5
Pipe Flow (CFS)	800	200	175	125	100	200	175	150	125	100
Pipe Area (ft^2)	33.2	23.8	23.8	23.8	23.8	19.6	19.6	19.6	19.6	19.6
Velocity (FPS)	24.1	8.4	7.4	5.3	4.2	10.2	8.9	7.6	6.4	5.1
Vel. Hd. (ft)	9.03	1.10	0.84	0.43	0.28	1.61	1.23	0.91	0.63	0.40
C		140	140	140	140	140	140	140	140	140
r (full pipe)		1.375	1.375	1.375	1.375	1.25	1.25	1.25	1.25	1.25
'/k'		2.281	1.782	0.956	0.633	3.627	2.833	2.130	1.520	1.006

SOUTH FLORIDA WATER MANAGEMENT DISTRICT
Structure P1 4600 CFS Pump Station
SYSTEM HEAD

SFWMD EAA Pump Station
Structure P1 4500 CFS Pump Station
SYSTEM HEAD
DSN: Van Kile
Date: 12/28/2017
CHK:
Date:

Pump Friction Head:

800 CFS Pumps								Fitting			Pipe	Starting**
Fitting:	Size (in):	Number:	"K" Factor:	Flow (CFS):	Area (sq ft)	Vel. (F/S):	V_H (ft):	H_L (ft)	H_L (ft/K ft):	Pipe L (ft):	H_L (ft)	
FSI:	96	1	0.15	800	50.3	15.9	3.9	0.59				
Miter Elbow 90 Deg,:	96	1	0.35	800	50.3	15.9	3.9	1.38				
Increaser (96"-144")	144	1	0.52	800	113.1	7.1	0.8	0.41				
Miter Elbow 45 Deg,:	144	5	0.35	800	113.1	7.1	0.8	1.36				
Square Outlet:	144	1	1	800	113.1	7.1	0.8	0.78				
Pipe 12' Diameter	144			800				4.51	0.666	324.2	0.2	
										Total:	4.72	3.30
FSI:	96	1	0.15	600	50.3	11.9	2.2	0.33				
Miter Elbow 90 Deg,:	96	1	0.35	600	50.3	11.9	2.2	0.77				
Increaser (96"-144")	144	1	0.52	600	113.1	5.3	0.4	0.23				
Miter Elbow 45 Deg,:	144	5	0.35	600	113.1	5.3	0.4	0.76				
Square Outlet:	144	1	1	600	113.1	5.3	0.4	0.44				
Pipe 12' Diameter	144			600				2.54	0.597	324.2	0.2	
										Total:	2.73	1.89
FSI:	96	1	0.15	400	50.3	8.0	1.0	0.15				
Miter Elbow 90 Deg,:	96	1	0.35	400	50.3	8.0	1.0	0.34				
Increaser (96"-144")	144	1	0.52	400	113.1	3.5	0.2	0.10				
Miter Elbow 45 Deg,:	144	5	0.35	400	113.1	3.5	0.2	0.34				
Square Outlet:	144	1	1	400	113.1	3.5	0.2	0.19				
Pipe 12' Diameter	144			400				1.13	0.185	324.2	0.1	
										Total:	1.19	0.83
400 CFS Pumps												
FSI:	84	1	0.15	400	38.5	10.4	1.7	0.25				
Miter Elbow 90 Deg,:	84	1	0.35	400	38.5	10.4	1.7	0.59				
Increaser (84"-132")	132	1	0.60	400	95.0	4.2	0.3	0.16				
Miter Elbow 45 Deg,:	132	5	0.35	400	95.0	4.2	0.3	0.48				
Square Outlet:	132	1	1	400	95.0	4.2	0.3	0.28				
Pipe 11' Diameter	132			400				1.76	0.282	324.2	0.1	
										Total:	1.85	1.34
FSI:	84	1	0.15	300	38.5	7.8	0.9	0.14				
Miter Elbow 90 Deg,:	84	1	0.35	300	38.5	7.8	0.9	0.33				
Increaser (84"-132")	132	1	0.60	300	95.0	3.2	0.2	0.09				
Miter Elbow 45 Deg,:	132	5	0.35	300	95.0	3.2	0.2	0.27				
Square Outlet:	132	1	1	300	95.0	3.2	0.2	0.15				
Pipe 11' Diameter	132			300				0.99	0.166	324.2	0.1	
										Total:	1.04	0.75
FSI:	84	1	0.15	200	38.5	5.2	0.4	0.06				
Miter Elbow 90 Deg,:	84	1	0.35	200	38.5	5.2	0.4	0.15				
Increaser (84"-132")	132	1	0.60	200	95.0	2.1	0.1	0.04				
Miter Elbow 45 Deg,:	132	5	0.35	200	95.0	2.1	0.1	0.12				
Square Outlet:	132	1	1	200	95.0	2.1	0.1	0.07				
Pipe 11' Diameter	132			200				0.44	0.078	324.2	0.0	
										Total:	0.47	0.34
200 CFS Pump												
Flare Inlet:	60	1	0.04	200	19.6	10.2	1.6	0.06				
Miter Elbow 90 Deg,:	60	1	0.35	200	19.6	10.2	1.6	0.56				
Increaser (60"-96")	96	1	0.63	200	50.3	4.0	0.2	0.15				
Miter Elbow 45 Deg,:	96	5	0.35	200	50.3	4.0	0.2	0.43				
Square Outlet:	96	1	1	200	50.3	4.0	0.2	0.25				
Pipe 8' Diameter	96			200				1.46	0.368	324.2	0.1	
										Total:	1.58	1.10
Flare Inlet:	60	1	0.04	150	19.6	7.6	0.9	0.04				
Miter Elbow 90 Deg,:	60	1	0.35	150	19.6	7.6	0.9	0.32				
Increaser (60"-96")	96	1	0.63	150	50.3	3.0	0.1	0.09				
Miter Elbow 45 Deg,:	96	5	0.35	150	50.3	3.0	0.1	0.24				
Square Outlet:	96	1	1	150	50.3	3.0	0.1	0.14				
Pipe 8' Diameter	96			150				0.82	0.216	324.2	0.1	
										Total:	0.89	0.62
Flare Inlet:	60	1	0.04	100	19.6	5.1	0.4	0.02				
Miter Elbow 90 Deg,:	60	1	0.35	100	19.6	5.1	0.4	0.14				
Increaser (60"-96")	96	1	0.63	100	50.3	2.0	0.1	0.04				
Miter Elbow 45 Deg,:	96	5	0.35	100	50.3	2.0	0.1	0.11				
Square Outlet:	96	1	1	100	50.3	2.0	0.1	0.06				
Pipe 8' Diameter	96			100				0.36	0.102	324.2	0.0	
										Total:	0.40	

**Starting head based on full pipe to top of dam, no outlet, 1/2 header length and two less 45 degree els

Pump:	800 CFS			400 CFS			200 CFS		
	Static:	Flow	TDH	Static:	Flow	TDH	Static:	Flow	TDH
Preliminary Pump Head:	26.6	800	31.32	26.6	400	28.45	26.6	200	28.18
with Siphon	-3.57	800	1.15	-3.57	400	-1.72	-3.57	200	-1.99
	26.6	600	29.33	26.6	300	27.64	26.6	150	27.49
	-3.57	600	-0.84	-3.57	300	-2.53	-3.57	150	-2.68
	26.6	400	27.79	26.6	200	27.07	26.6	100	27.00
	-3.57	400	-2.38	-3.57	200	-3.10	-3.57	100	-3.17

ANNEX A-2
HYDROLOGIC MODELING

- A-2 Reservoir Wave and Overtopping Analysis Report
- EAA Storage Reservoir Project Baseline Runs Model Documentation Report
- EAA Storage Reservoir Project Final Array of Alternatives Model Documentation Report
- EAA Storage Reservoir Project C240 Tentatively Selected Plan Model Documentation Report

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A-2 RESERVOIR WAVE AND OVERTOPPING ANALYSIS

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TECHNICAL MEMORANDUM

South Florida Water Management District
EAA Reservoir A-2

Project: EXXI2512
File Reference:
EXXI2512-A.P4.MA.46-CM-MEM-001
Issued: 16 January 2018

A-2 Reservoir Wave and Overtopping Analysis

To: Shawn Waldeck and Raymond Sciortino

From: Jessica Ryan
Reviewed: Joris Jorissen
Approved: Sam Watkin

1.0 INTRODUCTION

1.1 BACKGROUND

The Everglades Agricultural Area (EAA) Storage Reservoir Project is currently being undertaken as part of the Comprehensive Everglades Restoration Plan (CERP), an approved framework for restoring the south Florida ecosystem whilst providing for other water-rated needs of the region. The purpose of the EAA Storage Reservoir Project is:

- to capture basin runoff capture and regulatory releases from Lake Okeechobee,
- to improve the timing of environmental water supply deliveries to Water Conservation Areas,
- to meet supplemental agricultural deliveries,
- to reduce Lake Okeechobee regulatory releases to the estuaries, and
- to increase flood protection within the EAA.

As part of this project, the South Florida Water Management District (SFWMD) is undertaking a feasibility assessment for the design and construction of the EAA A-2 Reservoir and STA. Jacobs have been engaged to undertake a wave and overtopping analysis to support this feasibility assessment. This memo documents the procedure and outcomes of this analysis.

1.2 A-2 RESERVOIR

1.2.1 General Layout

The A-2 Reservoir and STA is located in western Palm Beach County, generally in Township 46 and Range 37. It is situated in the EAA, approximately 15.5 miles south of Lake Okeechobee (**Figure 1-1**). The A-2 Reservoir covers an extent of approximately 5 miles (east to west) by 4 miles (north to south) as shown in the hatched area in **Figure 1-2**.

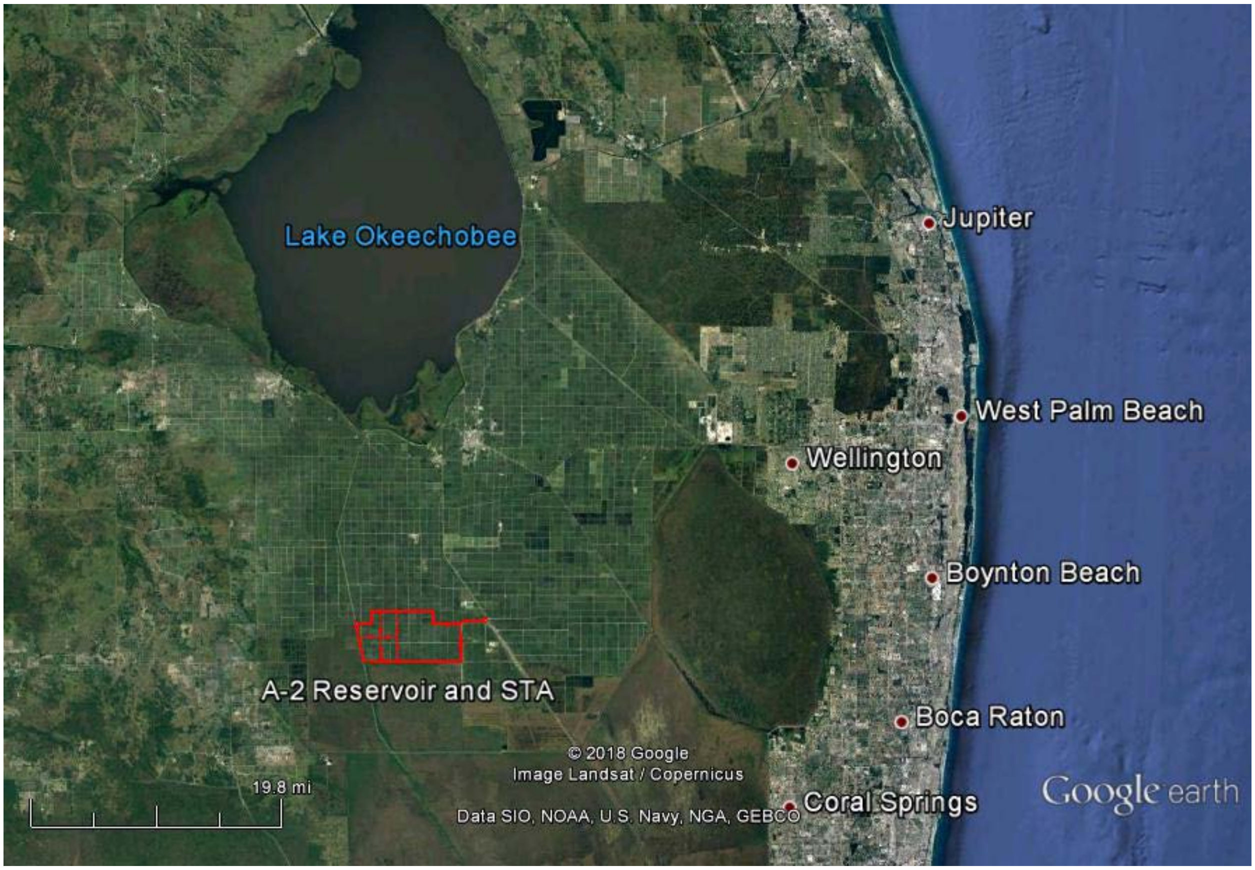

Figure 1-1. Location of A-2 Reservoir and A-2 STA

Figure 1-2. Layout of EAA A-2 Reservoir and Stormwater Treatment Area (STA)

1.2.2 Embankment Characteristics

Figure 1-3 illustrates the cross sectional design of the A-2 Reservoir embankment which was used for the overtopping analysis. A 1:3 slope is proposed for inner and outer slopes of the embankment, with Roller Compacted Concrete (RCC) material to be adopted along the inner slope and grass on the outer slope. A wave wall is proposed on the landward side of the embankment crest to reduce overtopping of the embankment. The overtopping analysis described in **Section 4.0** was used to determine the height of the wave wall to limit overtopping to appropriate levels. The Normal Full Storage Level (NFSL) of the A-2 Reservoir is at an elevation of 31.1 ft-NAVD (32.53 ft-NGVD).

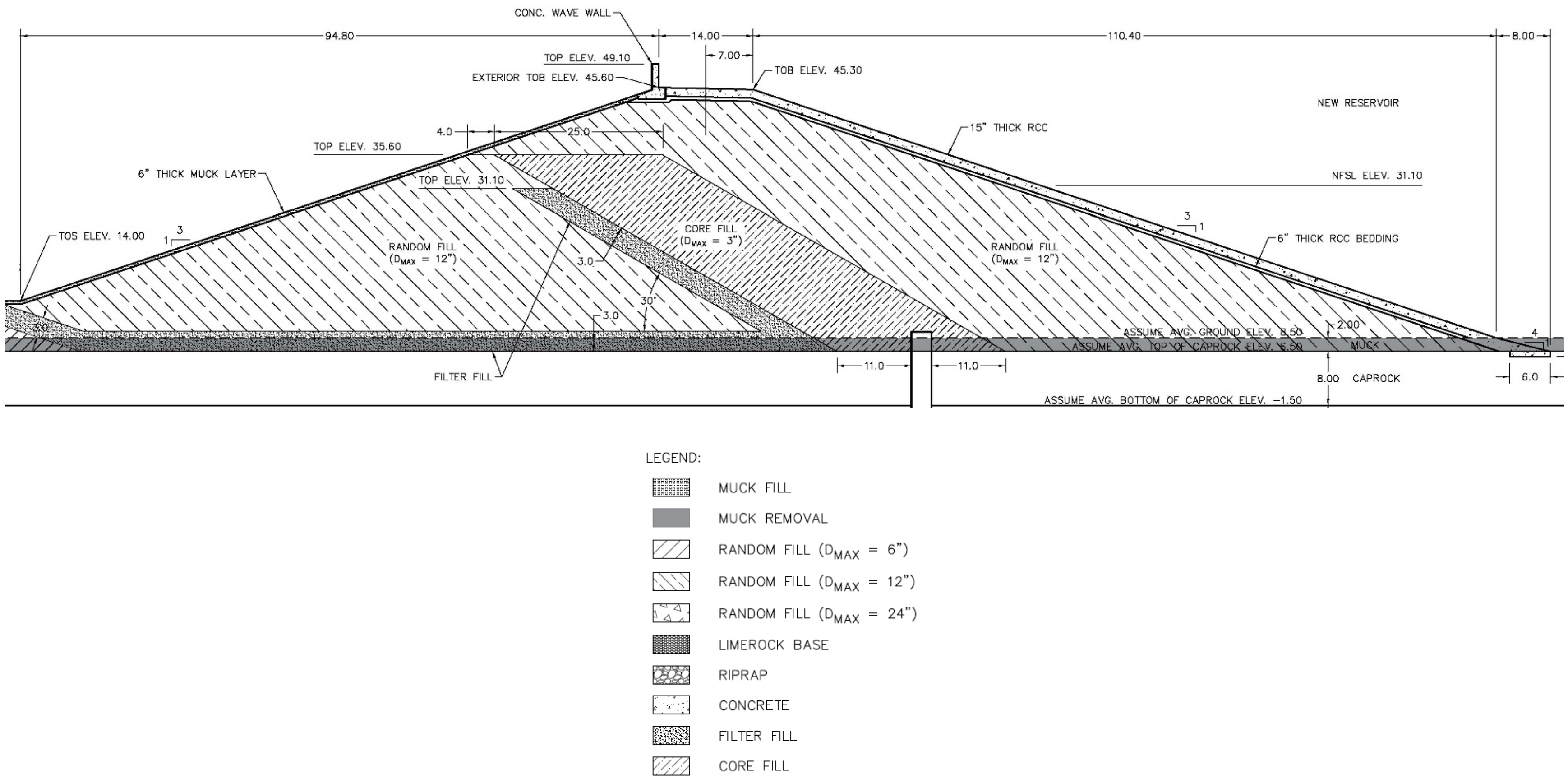

Figure 1-3. Typical Cross Section for the EAA A-2 Reservoir

1.3 OBJECTIVES

Objectives of the wave and overtopping analysis are as follows:

a) To predict the design wave conditions that could be generated across the reservoir during extreme design wind and precipitation events; and

b) To predict the wave overtopping rate of a number of embankment configurations during key design storm events in order to estimate the minimum embankment level to limit overtopping rates to acceptable levels.

2.0 DESIGN STORM EVENTS

Four combinations of extreme winds and precipitation, as described in DCM-2 (Haapala et al. 2006), were used to provide a preliminary assessment of the design wave conditions for the reservoir embankments, and evaluate the associated wave overtopping volumes. Details regarding these conditions are summarized below.

2.1 DESIGN CASE 1: PMP COMBINED WITH 100-YEAR WIND

Design Case 1 represents the Probable Maximum Precipitation (PMP) event in combination with a 100-year wind.

A 72-hour PMP of 54 inches was adopted for the analysis, based on hydrologic modelling undertaken for the EAA A-1 Reservoir (Burgi et al. 2005). This reservoir is located adjacent to the A-2 reservoir and covers a similar area. Hence the PMP estimates for the A-1 reservoir are considered to be appropriate for the A-2 Reservoir. The adopted PMP value aligns with estimates in other EAA reports such as the Levee High Report (CERP 2004) and DCM-2 (Haapala et al. 2006).

The procedure described in DCM-2 (Haapala et al. 2006) was followed to provide an estimate of the 100-year average recurrence interval (ARI) wind speed magnitude for the A-2 Reservoir. As specified in DCM-2, the 50-year three second wind gust for the A-2 Reservoir site is 125 mph. This was converted to a 100-year, one-hour overwater wind speed of approximately 106 mph. After adjustments for duration and overwater conditions, the adopted sustained wind speed magnitude for assessment of the design wave conditions and water levels of this design case was 104.5 mph.

2.2 DESIGN CASE 2: 100-YEAR PRECIPITATION COMBINED WITH CATEGORY 5 HURRICANE WINDS

Design Case 2 represents a 100-year precipitation event in combination with a Category 5 wind speed as defined by the Saffir-Simpson Hurricane Scale.

A 100-year precipitation event of 17 inches adopted for this design case, based on Figure DCM 2-3 (Haapala et al. 2006).

As recommended in DCM-2, a one-minute overwater wind speed of 156 mph was used to represent a Category 5 hurricane. After adjustments for duration to achieve fully developed wave conditions over the reservoir fetch length, the adopted wind speed for assessment of the design wave conditions and water levels was 123.3 mph.

2.3 DESIGN CASE 3: PROBABLE MAXIMUM WIND SPEED

Design Case 3 represents the Probable Maximum Wind (PMW) speed in combination with the reservoir level at the normal full storage depth (i.e. 22.6 ft for the A-2 Reservoir). As recommended in DCM-2, this particular design case was used for sensitivity testing only and not as a selected design condition (Haapala et al. 2006):

> [The probable maximum wind…] is to be used for "sensitivity identification" and not as a design condition. Wave models are unlikely capable of yielding results within a degree of confidence for design for these extreme wind speeds, especially over relatively shallow water bodies. Even for 125-mph wind, model capabilities are most likely being "stretched" for project conditions.

As defined in DCM-2, a one-minute averaged overwater wind speed of 200 mph was used to represent the PMW. The one-minute average wind speed was converted to an hourly averaged wind speed of 161 mph. After adjustments for duration, the adopted wind speed for assessment of the design wave conditions and water levels was 159.7 mph.

2.4 DESIGN CASE 4: STORM SPECIFIC WIND AND PRECIPITATION

Design Case 4 represents a storm specific case of precipitation and wind conditions recorded during Hurricane Easy which occurred in Florida in 1950.

Precipitation depths for both the 24-hour and 72-hour rainfall durations are considered in this analysis, corresponding to 38.7 inches and 45.2 inches respectively (Haapala et al. 2006).

A maximum wind speed of 125 mph (3 second gust) was recorded during Hurricane Easy (Haapala et al. 2006). After adjustments to meet DCM-2 requirements (i.e. overwater conditions, wind duration for wave development etc.) a design wind speed of 97.5 mph was used for the analysis.

2.5 SUMMARY

Table 2-1 summarizes the wind and precipitation design conditions used as a basis for the wave and overtopping assessment.

Table 2-1. Wind and Precipitation Design Conditions

Design Case	Description	Wind (mph)	Precipitation (inches)	Average Water Depth[1] (ft)
1	100 yr ARI wind + PMP	104.5	54	27.1
2	Cat 5 Hurricane + 100yr ARI Precipitation	123.3	17	24.0
3	Probable Max Wind Speed (Sensitivity Testing Only)	159.7	0	22.6
4.1	Storm Specific Wind & 24hr Precipitation (Hurricane Easy)	97.5	38.7	25.8
4.2	Storm Specific Wind & 72hr Precipitation (Hurricane Easy)	97.5	45.2	26.4

1. Average water depth = NFSL (22.6 ft) + Precipitation

3.0 WAVE ANALYSIS

3.1 INTRODUCTION

A wave analysis was undertaken to provide a preliminary assessment of the design wave conditions for the A-2 Reservoir. The USACE's Automated Coastal Engineering System (ACES) analysis software was used to model wave growth across the reservoir during the design storm events described in Section 2.0. Details regarding the ACES model setup, results, and validation are discussed below.

3.2 AUTOMATED COASTAL ENGINEERING SYSTEM (ACES)

The ACES package forms part of the Coastal Engineering Design and Analysis System (CEDAS), an interactive analysis system focused on the fields of coastal, ocean, and hydraulic engineering. The wind adjustment and wave growth module of ACES was used to estimate wave conditions within the A-2 Reservoir. This module provides estimates for wave growth over open-water and restricted fetches in both deep and shallow water based on a function of wind speed, fetch, and water depth. The methods used are primarily based on those of Vincent (1984), the SPM (1984), and Smith (1991).

3.3 ACES SETUP

Within ACES, the wave height and period can be calculated based on either shallow water equations or deep water equations. The SPM recommends adopting shallow water equations for relative depths less than 0.5 (i.e. $d/L_o < 0.5$). For the A-2 Reservoir, relative depths of approximately 0.11 – 0.21 were calculated for the various design conditions. Hence the wave growth analysis was undertaken using shallow water equations.

A rrestricted fetch methodology was adopted within the ACES setup to take into consideration the shape of the reservoir basin in the wave prediction estimates. An effective fetch of approximately 5.9 miles was calculated within ACES, based on radial fetch lengths provided at 7.5 degrees (refer to **Figure 3-1**). The wind direction was assumed to correspond to the direction of the maximum fetch length from a point of interest in the southeast corner of the reservoir.

Water depths and overwater wind speeds were specified in the ACES model as described in **Table 2-1**. It was assumed that winds were observed at a height of 33 feet.

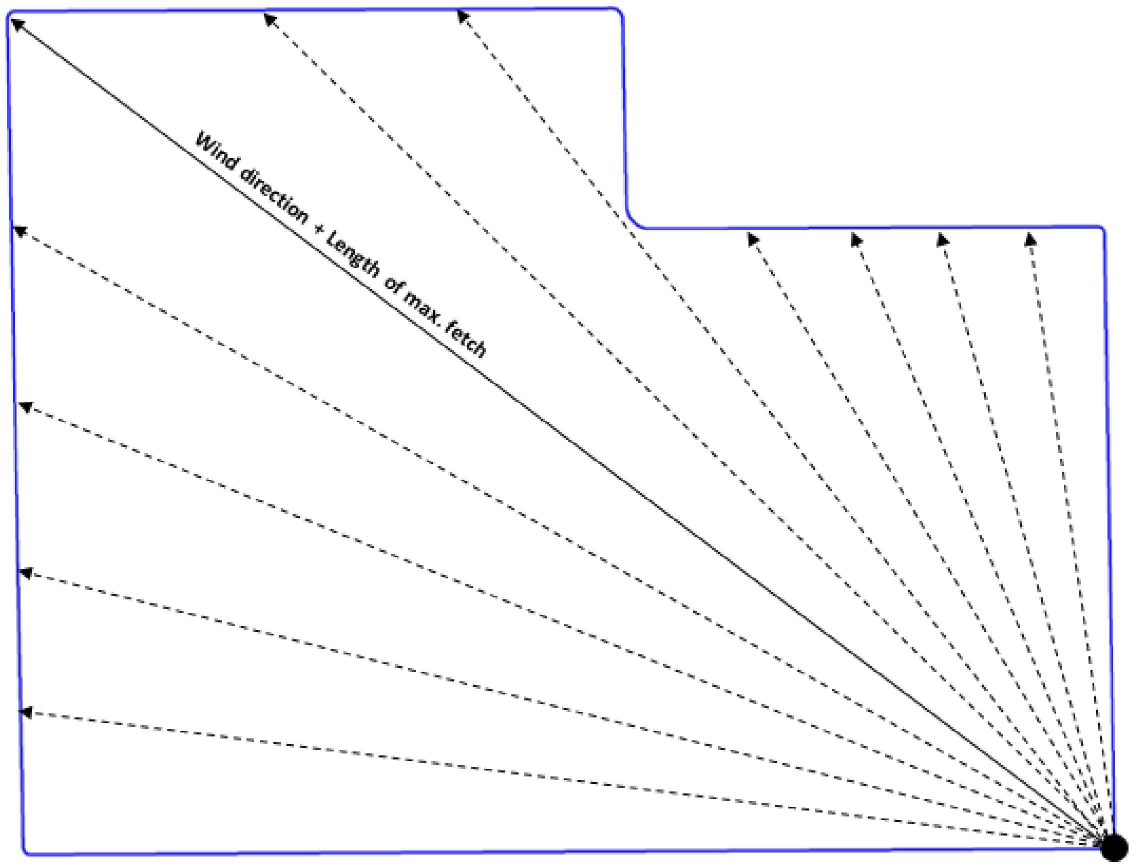

Figure 3-1. Effective fetch adopted for input into ACES

3.4 RESULTS

The wave results from the ACES analysis are summarized below in **Table 3-1**. Wave heights for the design conditions ranged from 8.4 ft to 10.5 ft, whilst peak wave periods ranged from 5.4 seconds to 6.1 seconds.

Table 3-1. A-2 Reservoir Wave Prediction Results

Design Case	Wind (mph)	Effective Water Depth[1] (ft)	Effective Water Level Elevation[2] (ft-NAVD / ft-NGVD)	Significant Wave Height, H_{mo} (ft)	Peak Wave Period, T_p (seconds)
1	104.5	27.1	35.60 / 37.03	9.2	5.6
2	123.3	24.0	32.50 / 33.93	10.5	6.1
3 (Sensitivity Testing)	159.7	22.6	31.10 / 32.53	13.0	7.0
4.1	97.5	25.8	34.30 / 35.73	8.4	5.4
4.2	97.5	26.4	34.90 / 36.33	8.5	5.4

1. *Average water depth = NFSL (22.6 ft) + Precipitation*
2. *Assuming average ground elevation of 8.50 ft-NAVD (9.93 ft-NGVD)*

3.5 VERIFICATION

Wave conditions predicted using ACES were verified against the SPM 1984 empirical methodology as recommended by DCM-2.

The SPM 1984 methodology revisits shallow water wave prediction formulae presented in SPM 1977 (SMB wave prediction curves) to include an intermediate calculation of wind stress and be consistent with the JONSWAP formulae. The calculations estimate the wind generated wave climate based on the effective fetch length, water depth, and wind stress. To align with the ACES assumptions, a precautionary effective fetch was determined from a point of interest in the southeast corner of the reservoir. The effective fetch length was calculated as per recommendations provided in the SPM, i.e. averaging the fetch lengths for nine radials extending from the point of interest at 3 degree intervals. Based on this analysis an effective fetch of approximately 5.7 miles was calculated for the A-2 Reservoir.

The results from the verification analysis indicate that the wave height and period estimates produced from the ACES model correlate well with those predicted using the SPM 1984 methodology (refer to **Figure 3-2** and **Figure 3-3**). In subsequent design phases, it is recommended that a more comprehensive spectral wave model (such as STWave) is used to verify these wave conditions and investigate the spatial variation throughout the reservoir.

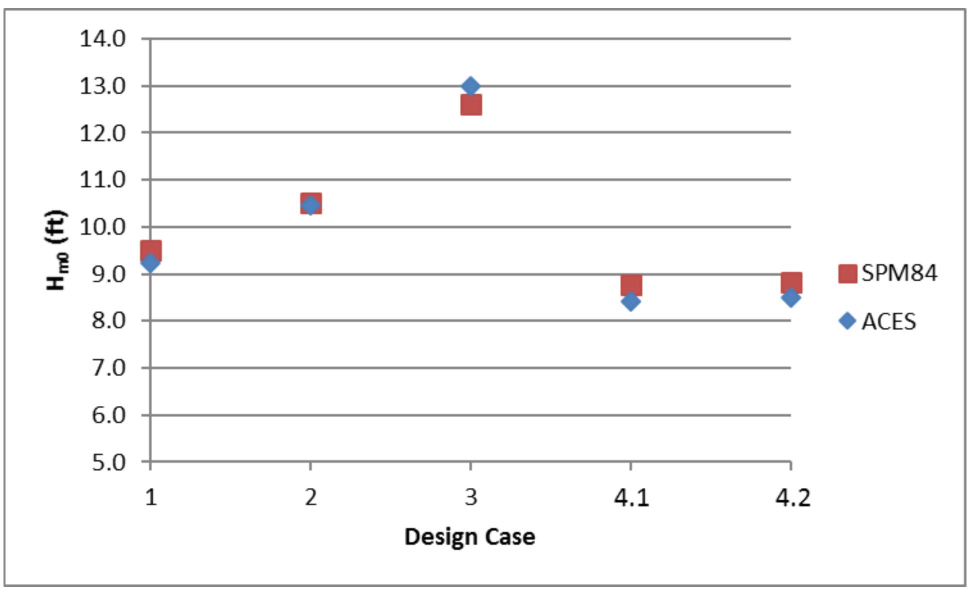

Figure 3-2. Wave Height Verification

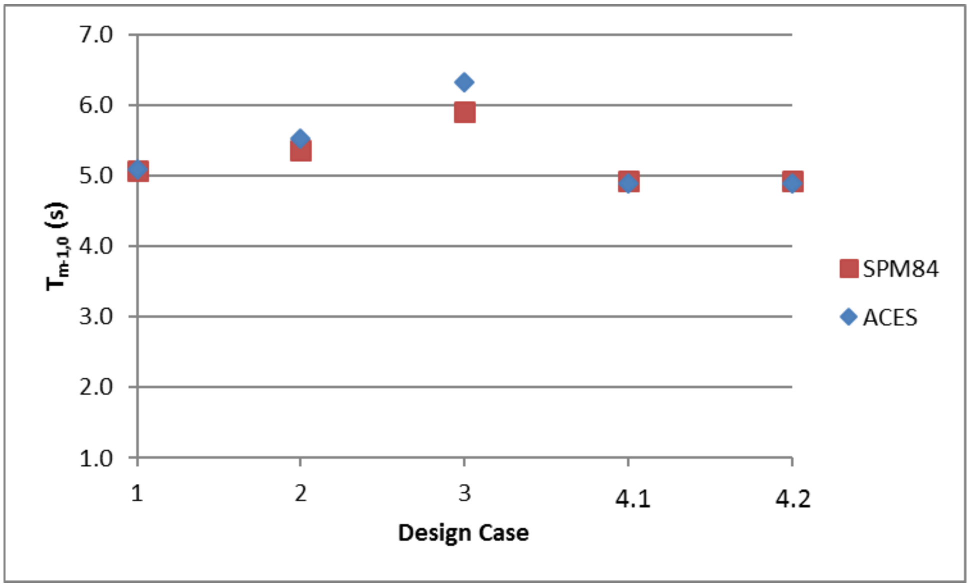

Figure 3-3. Wave Period Verification

4.0 OVERTOPPING ANALYSIS

4.1 INTRODUCTION

Wave overtopping is an important parameter in determining appropriate freeboard levels for reservoirs. The volume of water that may flow over the crest of the structure during storm events is dependent on hydrodynamic parameters (wave height and period, angle of wave attack and, water depth), as well as the characteristics of the embankment (e.g. crest height, roughness and slope).

Significant volumes of overtopping discharge can result in structural damage to the crest and leeward side of the embankment, threatening the safety of the reservoir. Hence an overtopping analysis was undertaken to estimate the minimum crest elevation of the A-2 Reservoir embankment to limit overtopping during design storm events to acceptable levels.

4.2 METHODOLOGY

DCM-2 (Haapala et al. 2006) recommends the use of ACES to calculate the wave run-up and overtopping for EAA reservoirs. The ACES software package is based on the methodologies published by Weggel (1976) and Ahrens (1977). Many advances have subsequently been made in the prediction of wave-run up and overtopping, with the latest recommendations being published in the 2016 revision of the EurOtop Manual (EurOtop 2016).

The 2016 EurOtop Manual provides specific guidance for estimating the mean overtopping rate at structures similar to the design proposed for the A-2 Reservoir (a mild-sloped embankment with a vertical wave wall located on the landward side of the embankment), and therefore this methodology was adopted in the wave overtopping analysis. The equations used for the analysis were based on those specified for a "deterministic design or safety assessment" approach, which include a partial safety factor of one standard deviation (EurOtop 2016). Advice provided by Van Doorslaer et al. (2016) was used to adapt these equations for the plunging wave conditions (i.e., $\xi_{m-1,0}<1.8$) generated in the A-2

Reservoir under the design wind and water levels (refer to the methodology in the C-43 Reservoir Overwash and Wave Forces Report, Hughes 2017).

In addition to the mean overtopping discharge rate, the maximum overtopping volume of a single wave was also estimated as per equations provided in the EurOtop Manual (2016). These equations are based on various parameters, including the mean overtopping discharge, storm duration, and the percentage of overtopping waves.

4.3 WIND SETUP

Wind setup is caused by shear stress exerted on the water surface, which in turn causes a slope in the water surface resulting in wind setup at the leeward side of the reservoir. This setup level influences the water depth at the reservoir embankment, and therefore the wave run-up/overtopping discharge. Hence the calculation of wind setup is required for determination of freeboard (Haapala et al. 2006).

For reservoirs with depths equal to or greater than 16 feet, DCM-2 recommends that wind setup is calculated using the Zeider Zee equation, which calculates wind setup based on wind speed, fetch length and depth. **Table 4-1** summarizes the estimated wind setup for each of the DCM-2 design cases, and the resulting maximum water depth at the leeward side of the reservoir. This maximum depth was subsequently used in the overtopping calculations.

Table 4-1. Summary of Calculated Wind Setup

Design Case	Wind (mph)	Effective Water Depth[1] (ft)	Wind Setup (ft)	Maximum Water Depth[2] (ft)	Maximum Water Level Elevation[3] (ft-NAVD / ft-NGVD)	Freeboard to TOB Water Side[4] (ft)
1	104.5	27.1	1.8	28.9	37.40 / 38.83	7.9
2	123.3	24.0	2.8	26.8	35.30 / 36.73	10.0
3 (Sensitivity Testing)	159.7	22.6	4.8	27.4	35.90 / 37.33	9.4
4.1	97.5	25.8	1.6	27.4	35.90 / 37.33	9.4
4.2	97.5	26.4	1.6	28.0	36.50 / 37.93	8.8

1. Average water depth = NFSL (22.6 ft) + Precipitation
2. Maximum water depth = Average water depth + Wind setup
3. Maximum water elevation based on assumed average ground level of 8.50 ft-NAVD (9.93 ft-NGVD)
4. Freeboard to TOB water side = TOB water side elevation (45.30 ft-NAVD or 46.73 ft-NGVD) - Maximum water elevation

4.4 DESIGN CRITERIA

4.4.1 Overtopping Limits

Acceptable overtopping limits were specified in terms of the mean overtopping discharge, as well as the maximum overtopping of a single wave. Exposure to erosion damage is highly dependent on the wave run-down characteristics on the landward side of the structure, which is a function of the slope type and resilience (e.g., grass quality, soil type, etc.). For the purposes of the preliminary design in this feasibility study, a mean overtopping discharge limit of 0.1 cfs per lineal foot of embankment was adopted as per guidance in DCM-2. This limit is broadly in line with recommendations in the EurOtop Manual (2016) to prevent damage to a grass sloped embankment, and relates to a “Start of Damage” condition based on guidance in the Coastal Engineering Manual (USACE 2002). As the wind and precipitation scenarios used for the design of the A-2 Reservoir are associated with rare extreme storm scenarios, a "Start of Damage" (i.e., minor damage) condition is deemed appropriate.

Wave overtopping is a dynamic and irregular process and generally a few large waves account for most of the overtopping discharge. Therefore, it is useful to consider the proportion of waves that is likely to overtop the revetment and the maximum overtopping volume from one individual wave in a storm. The limit for the maximum overtopping volume of a single wave was selected as 32.3 ft^3/ft as per recommendations for grass sloped embankments in the EurOtop Manual (2016).

4.4.2 Design Water Levels and Wave Conditions

The design water levels and wave conditions adopted for the overtopping analysis are summarized below in **Table 4-2**. The maximum water depth including wind setup was used for the analysis.

A precautionary approach was adopted for the purposes of the overtopping assessment, assuming the angle of wave attack is perpendicular to the structure. Whilst this is relevant for a localized portion of the embankment which is directly exposed to the longest fetch, the majority of the embankment will be subjected to waves approaching at a smaller angle of attack (when considering the design waves generated along the maximum fetch length). Hence a wave directionality assessment (including a comprehensive STWave model) is recommended to be undertaken during subsequent design phases to assess the opportunity to refine the embankment cross section design around the perimeter of the reservoir.

Table 4-2. Design Water Levels and Wave Conditions Adopted for the Overtopping Analysis

Design Case	Maximum Water Depth[1] (ft)	Maximum Water Level Elevation[2] (ft-NAVD / ft-NGVD)	Significant Wave Height, H_{mo} (ft)	Peak Wave Period, Tp (seconds)
1	28.9	37.40 / 38.83	9.2	5.6
2	26.8	35.30 / 36.73	10.5	6.1
3 (Sensitivity Testing)	27.4	35.90 / 37.33	13.0	7.0
4.1	27.4	35.90 / 37.33	8.4	5.4
4.2	28.0	36.50 / 37.93	8.5	5.4

1. Maximum water depth = NFSL (22.6 ft) + Precipitation + Wind setup
2. Maximum water elevation = Average water depth + Wind setup

4.4.3 Structural Parameters

The overtopping analysis was based on the cross-sectional design shown in **Figure 1-3** which consists of RCC material at a 1:3 slope, with a RCC crest and a concrete wave wall on the landward side of the embankment crest. In the wave overtopping modelling, the embankment and crest is assumed to be smooth and impermeable, a precautionary assumption.

4.4.4 Storm Duration

The maximum overtopping volume by one single wave during a storm is dependent on the length of time that peak storm conditions prevail. In order to determine an appropriate storm duration for the design storm affecting the A-2 Reservoir, an analysis of historical hurricanes within the region was undertaken, based on the best track information included in NOAA's International Best Track Archive.

Figure 4-2 provides a summary of the results of this analysis, presenting the probability of exceedance of the mean forward moving speed of hurricanes within the region during the period 1950 – 2015. The figure shows that 95% of the hurricanes move with a forward speed greater than approximately 2.1 mph. Hence, considering the maximum fetch distance of the A-2 Reservoir (approx. 6.3 miles), a storm duration of 3 hours was selected as a precautionary estimate of the peak storm duration for the assessment.

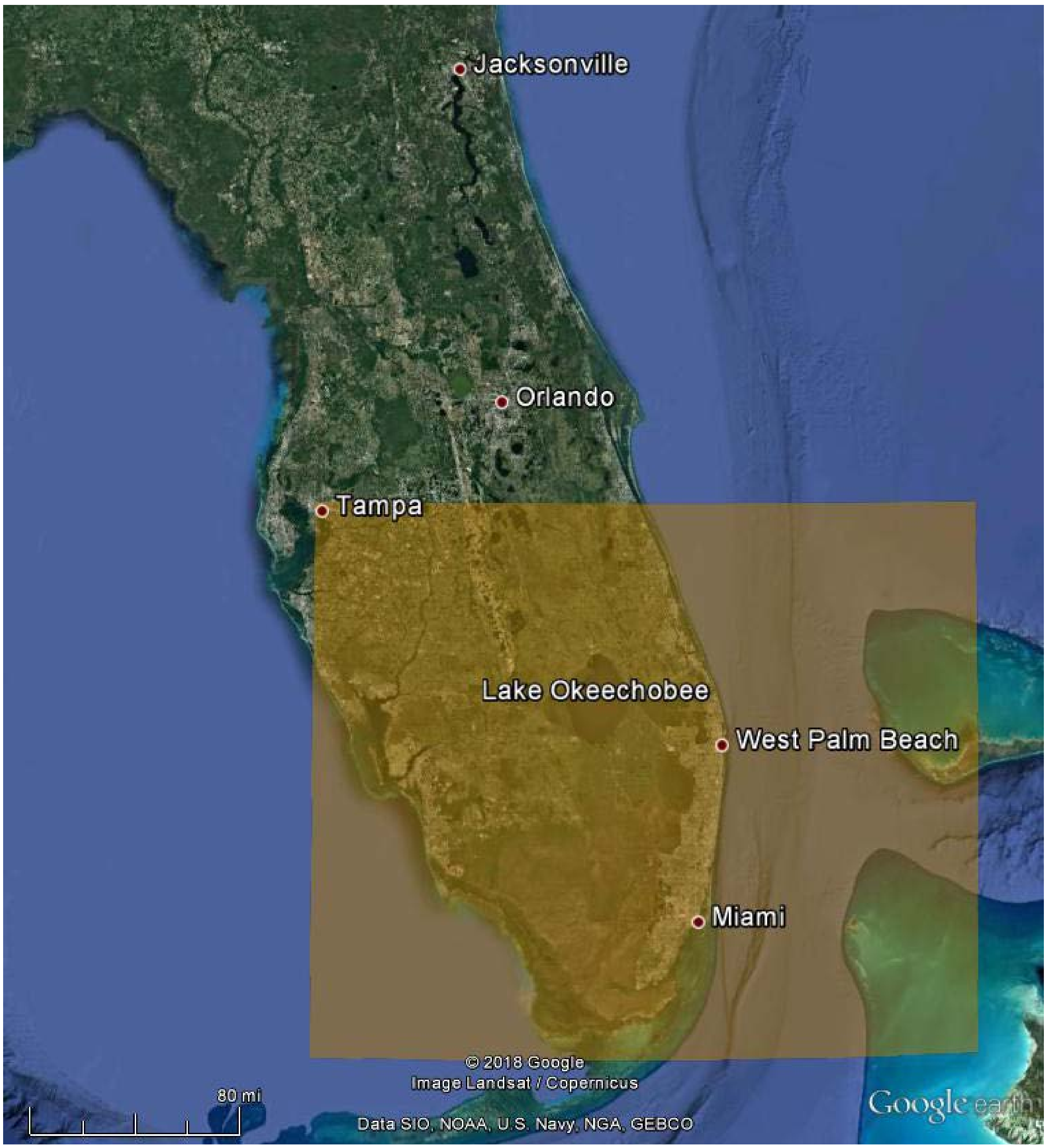

Figure 4-1. Extents of Statistical Analysis of Hurricane Speeds Between 1950 - 2015

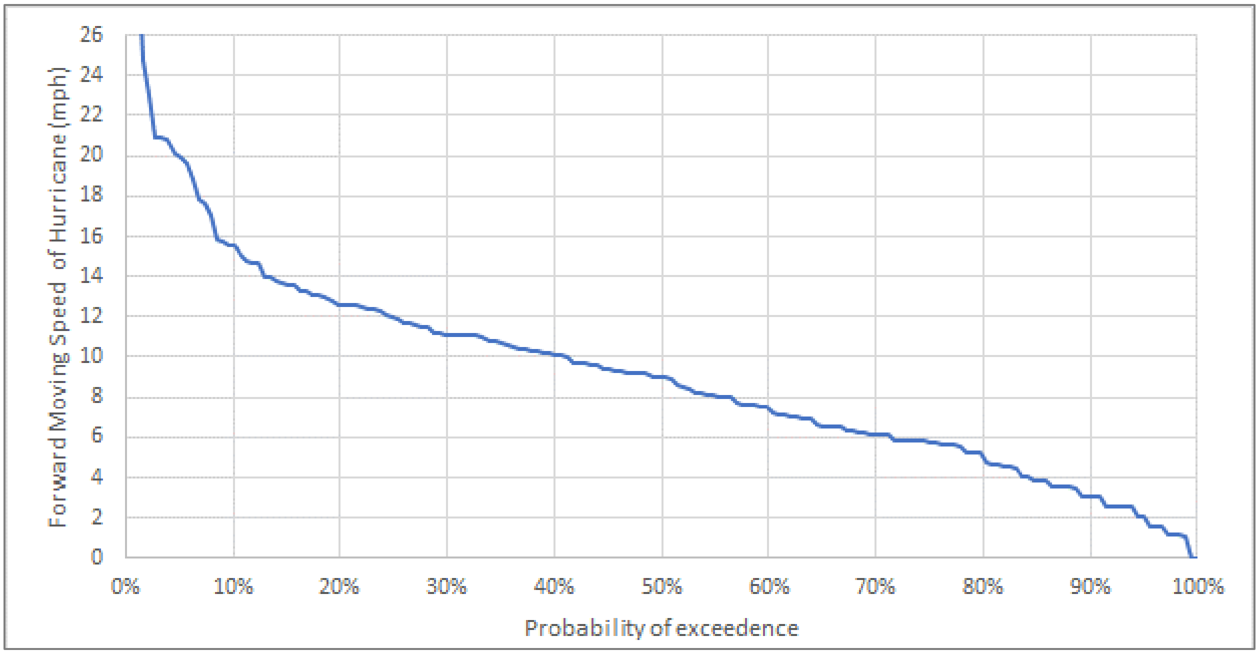

Figure 4-2. Probability of Exceedance of the Forward Moving Speed of Hurricanes in the Florida Region

4.5 RESULTS

4.5.1 Mean Overtopping Discharge

Overtopping discharges were calculated for wave wall heights ranging from 2ft – 4ft. The results indicate that a 3.5 ft wave wall achieves acceptable overtopping rates below 0.1 cfs/ft as shown in **Table 4-3**. As per recommendations is DCM-2, Design Case 3 is used for sensitivity testing only and not as a selected design condition.

Table 4-3. Summary of Calculated Mean Overtopping Discharge

Wave Wall Height (ft)	Wave Wall Top Elevation (ft-NAVD / ft-NGVD)	Design Case	Freeboard to Top of Wave Wall[1] (ft)	Overtopping Rate (cfs/ft)
2.0	47.60 / 49.03	1	10.20	0.18
		2	12.28	0.18
		3 (Sensitivity Testing)	11.70	0.98
		4.1	11.68	0.04
		4.2	11.13	0.06
2.5	48.10 / 49.53	1	10.70	0.14
		2	12.78	0.15
		3 (Sensitivity Testing)	12.20	0.84
		4.1	12.18	0.03
		4.2	11.63	0.04
3.0	48.60 / 50.03	1	11.20	0.11
		2	13.28	0.12
		3 (Sensitivity Testing)	12.70	0.72
		4.1	12.68	0.02
		4.2	12.13	0.03
3.5	49.10 / 50.53	1	11.70	0.08
		2	13.78	0.09
		3 (Sensitivity Testing)	13.20	0.61
		4.1	13.18	0.02
		4.2	12.63	0.02
4.0	49.60 / 51.03	1	12.20	0.06
		2	14.28	0.07
		3 (Sensitivity Testing)	13.70	0.52
		4.1	13.68	0.01
		4.2	13.13	0.02

1. Freeboard to top of wave wall = Top of wave wall elevation – Maximum water level elevation

4.5.2 Maximum Overtopping Volume

Based on the results of the mean overtopping discharge analysis (**Table 4-3**), the maximum overtopping volume of a single wave was calculated for a 3.5 ft tall wave wall. Table 4-4 summarizes the results for this analysis, including the percentage of overtopping waves which is a function of the 2% run-up height (EurOtop, 2016).

The results indicate that a 3.5 ft wave wall achieves acceptable overtopping for the reservoir, with the maximum overtopping volume predicted to remain below the 32.3 ft^3/ft limit for all design cases (i.e. 1, 2, and 4). The values presented in **Table 4-4** are based on a storm duration of approximately 3 hrs (refer to section 4.4.4), and hence are precautionary with shorter storm durations resulting in lower maximum overtopping volumes.

Table 4-4. Summary of Overtopping Probability & Maximum Overtopping Volume for a Single Wave (assuming 3 hr storm duration)

Design Case	Freeboard to Top of Wave Wall[1] (ft)	Probability of Overtopping	Maximum Overtopping Volume for a Single Wave (ft^3/ft)
1	11.70	27.7%	22.4
2	13.78	26.3%	29.4
3 (Sensitivity Testing)	13.20	47.1%	108.5
4.1	13.18	14.4%	8.7
4.2	12.63	17.2%	10.6

5.0 FINDINGS AND RECOMMENDATIONS

A wave and overtopping analysis was undertaken to support the preliminary design of the A-2 Reservoir. The wind adjustment and wave growth module of ACES was used to estimate wave conditions generated within the A-2 Reservoir for the wind and precipitation design cases specified by DCM-2. Design wave heights predicted for the reservoir ranged from 8.4 ft to 10.5 ft, whilst peak wave periods ranged from 5.4 seconds to 6.1 seconds. These estimates are assumed to be suitable for the purposes of the feasibility study. It is recommended that they are confirmed using a more comprehensive model (e.g. STWave) in subsequent design phases.

An overtopping analysis was undertaken to determine a suitable embankment crest configuration to limit overtopping of the A-2 Reservoir to acceptable volumes during wave and wind-setup levels generated from the DCM-2 design cases. A range of analysis techniques, as described in the EurOtop Manual (2016), were used to estimate overtopping characteristics for the proposed 1:3 embankment slope with a vertical wave wall located on the landward side of the embankment crest. The results from the analysis indicate that an embankment with a water side crest elevation at 45.30 ft-NAVD (46.73 ft-NGVD), landward crest elevation of 45.6 ft-NAVD (47.03 ft-NGVD), and a 3.5 ft tall vertical wave wall is likely to achieve acceptable overtopping rates both in terms of the mean overtopping discharge (i.e. < 0.1 cfs/ft) and the maximum overtopping volume for a single wave (i.e., < 32.3 ft^3/ft).

The proposed cross-sectional design could potentially be refined to better manage wave overtopping at the reservoir. Potential design refinements to be investigated include the following:

- Alternative wave wall design (Increased the height, inclusion of a parapet or recurve wall)
- Inclusion of an intermediate berm
- Increasing the roughness of the slope and/or crest by (e.g. quarry stones, concrete blocks) to reduce wave run-up
- Armoring of the outer (landward side) slope of the embankment to provide increased protection against overtopping.
- The inclusion of intermediate dikes within the dam to limit the wave height, and hence reduce the dam elevation.

In addition, it is recommended that the spatial variability in the wave overtopping along the embankment is investigated and the design refined accordingly.

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Modeling Section, H&H Bureau
South Florida Water Management District

Everglades Agricultural Area Storage Reservoir Project Baseline Runs Model Documentation Report

March 2018

1.0 Overview

Identification

The Everglades Agricultural Area Storage Reservoir Project (EAASR) is an expedited planning effort undertaken as a project component of the Comprehensive Everglades Restoration Plan (CERP). This project planning effort was led by the South Florida Water Management District (SFWMD) and seeks to enhance the performance of the Central Everglades Planning Project (CEPP) which has already been authorized by Congress. The project will be designed to: 1) reduce the high-volume freshwater discharges from Lake Okeechobee to the Northern Estuaries, 2) identify storage, treatment and conveyance south of Lake Okeechobee to increase flows to the Everglades system and 3) reduce ongoing ecological damage to the Northern Estuaries and Everglades system. The project worked throughout late 2017 and early 2018 and combines planning and design activities for three primary areas of interest in the south Florida system as follows: 1) Next increment of storage and necessary treatment to provide progress towards the level of restoration envisioned for the CERP, 2) Continue to improve the quantity, quality, timing and distribution of water flows to the Northern Estuaries and central Everglades and 3) Be consistent with federal program and policy requirements. Modeling support to the EAASR effort was provided by a team comprised of modelers from the Modeling Section of the Hydrology and Hydraulics Bureau of the SFWMD.

Scope and Objectives

Modeling support for EAASR focused on working with the larger project planning team and other interested parties to formulate and test project features leading to the ultimate identification and refinement of a tentatively selected plan (TSP). Modeling products were developed at the appropriate level of detail to support feature screening and detailed representation of project features and to provide information to all necessary evaluations required for plan development and documentation. The project plan formulation framework is built upon work already completed as part of the CEPP planning effort and utilizes the same tools and techniques by performing initial screening followed by detailed evaluation to identify final project planning alternatives and ultimately a TSP for the effort.

The CEPP Modeling Strategy document (**SFWMD, 2012a**) describes the modeling process and tools utilized, the associated rationale of the selection process and the means by which the tools could expediently support the project workflow. Given that the EAASR effort is being pursued as a change to an authorized CERP project, utilization of comparable modeling strategies and tools as those used in the development of the authorized CEPP plan was a guiding principle of EAASR modeling work. The primary model support tools utilized in EAASR project refinement are as follows:

Screening Tool and Water Quality Assessment:
- Dynamic Model for Stormwater Treatment Areas (DMSTA)

Detailed Planning Models:
- Regional Simulation Model Basins (RSMBN)
- Regional Simulation Model Glades-LECSA (RSMGL)

From a modeling deliverable perspective, the entirety of the EAASR modeling support can be summarized by reviewing the following three Model Documentation Reports (MDRs):
1. EAASR Baseline – Reviews the various non-EAASR model representations (e.g., current and future without project conditions) used in various aspects of the project planning (this document, **SFWMD, 2018a**).
2. EAASR Final Array of Alternatives – Reviews the model-supported feature screening efforts undertaken to size the reservoir and treatment facilities and detailed evaluation of three modeled “with EAASR” project model representations examined during plan formulation (**SFWMD, 2018b**)
3. EAASR Tentatively Selected Plan – Reviews the model representation of the optimized plan identified in the final steps of plan formulation and project assurance planning (**SFWMD, 2018c**).

This Baseline Runs MDR describes the assumptions, model implementation steps and observed outcomes associated with modeling representations of the current and future without project condition model scenarios. These model runs were predominantly used as a basis of comparison for many of the evaluations performed in support of plan formulation or project assurances assessment. This document will focus on the modeling details of these scenarios.

2.0 Basis

Project Assumptions

This MDR describes the assumptions, model implementation steps and observed outcomes associated with modeling the following RSMBN/RSMGL scenarios:

EAASR Final Plan Baselines - released 11/6/2017
- 2017 Existing Condition Baseline (EARECB)
- 2050 Future Without Project Baseline (EARFWO)

In general, the existing conditions baseline scenarios attempt to model assumed hydrologic conditions circa a defined date (e.g. 2017 at EAASR project initiation for the EARECB scenario) and include current system infrastructure assumptions and current operational practices. Consistency with the modeling used in the development of the original CEPP authorized project was identified as a guiding principle for modeling support. In general, the future projected 2050 conditions include, relative to existing conditions, additional representations of planned future project activities, including state, federal and CERP projects. Detailed project assumption tables for each scenario are provided in **Appendix A** and key elements of model implementation are described in Section 3.

Model Limitations and Intended Use of Results

The primary modeling products of EAASR were evaluated based on outputs from the Regional Simulation Model (RSM; **SFWMD, 2005a** and **2005b**). The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g, use available input-driven options to represent more complex project operations).

As described in **Figure 2.1**, the EAASR modeling workflow strives for appropriate application of modeling tools (particularly DMSTA and RSM) for their intended use. It is neither efficient nor necessary to force intermediate modeling products to reflect a higher level of detail or consistency than is needed at that time to be robust for decision making. Along the modeling workflow, there are many opportunities for refinement. Intermediate products serve the immediate need and then are enhanced, incorporating feedback and information as the process progresses.

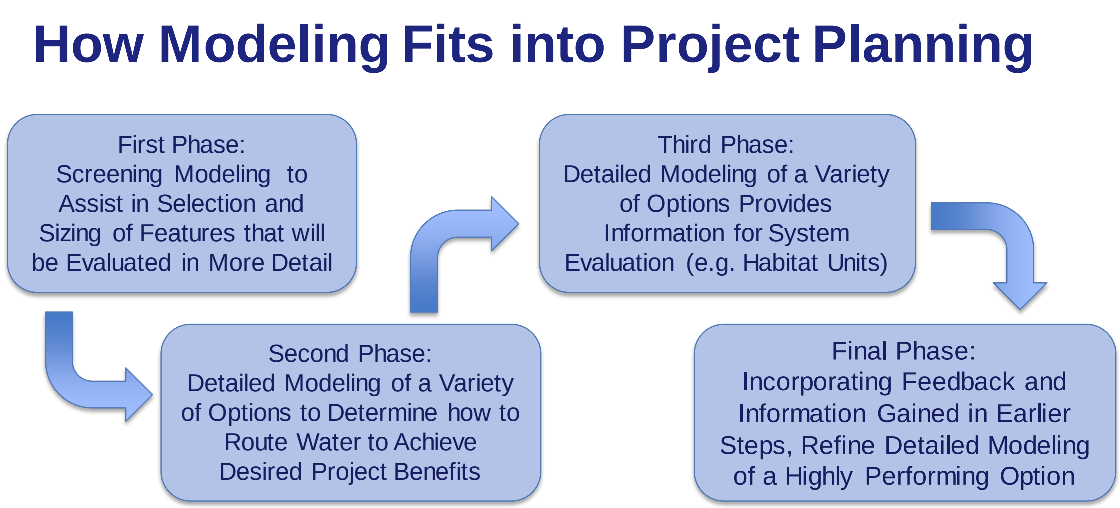

Figure 2.1. Typical EAASR Modeling Workflow

The RSMBN (**SFWMD, FDEP & FDACS, 2009a, 2009b**), RSMGL (**SFWMD, 2010** and **2011**), and DMSTA (**Walker & Kadlec, 2005**; **Wang, 2012**) models were reviewed through the USACE validation process for engineering software, as part of the CEPP project. The RSM and DMSTA models were classified as “allowed for use” for South Florida applications in August 2012 and January 2013, respectively.

3.0 Simulation

Modeling Tools Used

RSM Version 2.3.5R was used to run both the RSMBN and RSMGL models.
Release date 10/26/2017, SVN Version #5205.

This RSM version is derived from the RSM release used in support of CEPP project planning and incorporates minor updates to allow for simulation of different operational scenarios that needed to be evaluated as part of the EAASR effort (e.g., using a reservoir to meet both downstream environmental and agricultural water supply demands as envisioned in CERP).

Model Set Up

The EAASR baseline scenarios were developed using the decoupled RSMBN and RSMGL models. Collectively, these two models cover the spatial extent of the project planning area as shown in **Figure 3.1**. The RSMBN and RSMGL modeling for EAASR were built from the Final Baselines and Final Tentatively Selected Plan scenarios developed for the CEPP project (**SFWMD & IMC, 2014a and 2014c**). The period of simulation for both models utilizes a climate record from 1965 to 2005.

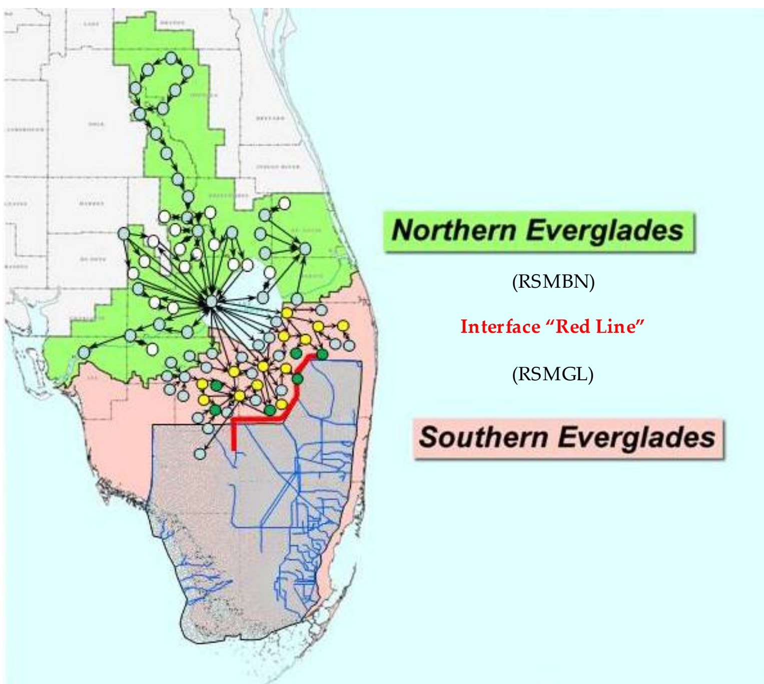

Figure 3.1. Decoupled EAASR Modeling Approach using RSMBN and RSMGL models.

In general, assumed modeled data sets (e.g. topography, water control districts, etc…) and/or system features (structure operations, etc…) are consistent with previous planning exercises, unless identified as changed in this section or in the assumptions tables in **Appendix A**. In order to maintain consistency with the CEPP modeling effort, the EAASR modeling effort utilized previous CEPP modeling to establish baseline scenarios.

The EAASR Existing Condition Baseline (EARECB) attempts to represent on-ground conditions circa 2017 and uses assumptions per the CEPP RSMBN ECB and IORBL1 simulations (depending on sub-basin) and the CEPP RSMGL 2012EC (as defined in CEPP Project Implementation Report). Relative to the CEPP RSMBN ECB scenario, the RSMBN EARECB scenario is unchanged in the modeling domain with the exception of the following:

- Inclusion of Lower Kissimmee River Restoration consistent with on-ground conditions in late 2017 and not complete at the time of the CEPP ECB modeling
- Inclusion of the A1 FEB and full Compartments B/C buildout, again consistent with on-ground conditions in late 2017 and not complete at the time of the CEPP ECB modeling

In both of these cases, modeling representation of these features was consistent with the assumptions previously used in the CEPP RSMBN IORBL1 scenario. The EARECB scenario is therefore a hybrid of the CEPP RSMBN ECB and CEPP RSMBN IORBL1 from a modeling perspective. The RSMGL EARECB scenario is the same as the RSMGL 2012EC from an assumptions perspective, but utilizes updated northern boundary conditions from the updated RSMBN model. Therefore, the simulated data sets of the EARECB scenarios will be changed relative to CEPP for both the RSMBN and RSMGL scenarios. While some operational changes have occurred in the southern portion of the system relative to the CEPP 2012EC condition, these changes are in transition as ongoing planning efforts to identify the Combined Operations Plan (COP) for the southern system are pursued. Given the transitory and uncertain nature of these ongoing refinements, it was deemed appropriate to retain 2012EC assumptions for the EAASR effort since these not only represent the authorized operations, but also to facilitate direct comparison of EAASR effects relative the original CEPP modeling efforts.

The EAASR Future Without Project Baseline (EARFWO) attempts to represent the projected future conditions circa 50 years in the future if there was no EAA Storage Reservoir Project and uses assumptions per RSMBN ALT4R2 and RSMGL ALT4R2 (CEPP Selected Plan + Other Authorized Projects). Since the CEPP modeling tools are being used in the EAASR effort and no assumption changes are made relative to CERP or non-CERP projects in the future condition (changes in existing condition only relate to implementation progress and not ending outcome of efforts in 50 years), the simulated data sets of the EARFWO are identical to the corresponding CEPP TSP scenarios.

A number of project updates are assumed in the EARFWO relative to the EARECB as shown in **Figure 3.2.** The subsequent sub-sections will explain the modeling setup for each of these areas as assumed in the EARECB and EARFWO scenarios. Details about project intent can be found in the associated project reports for each effort.

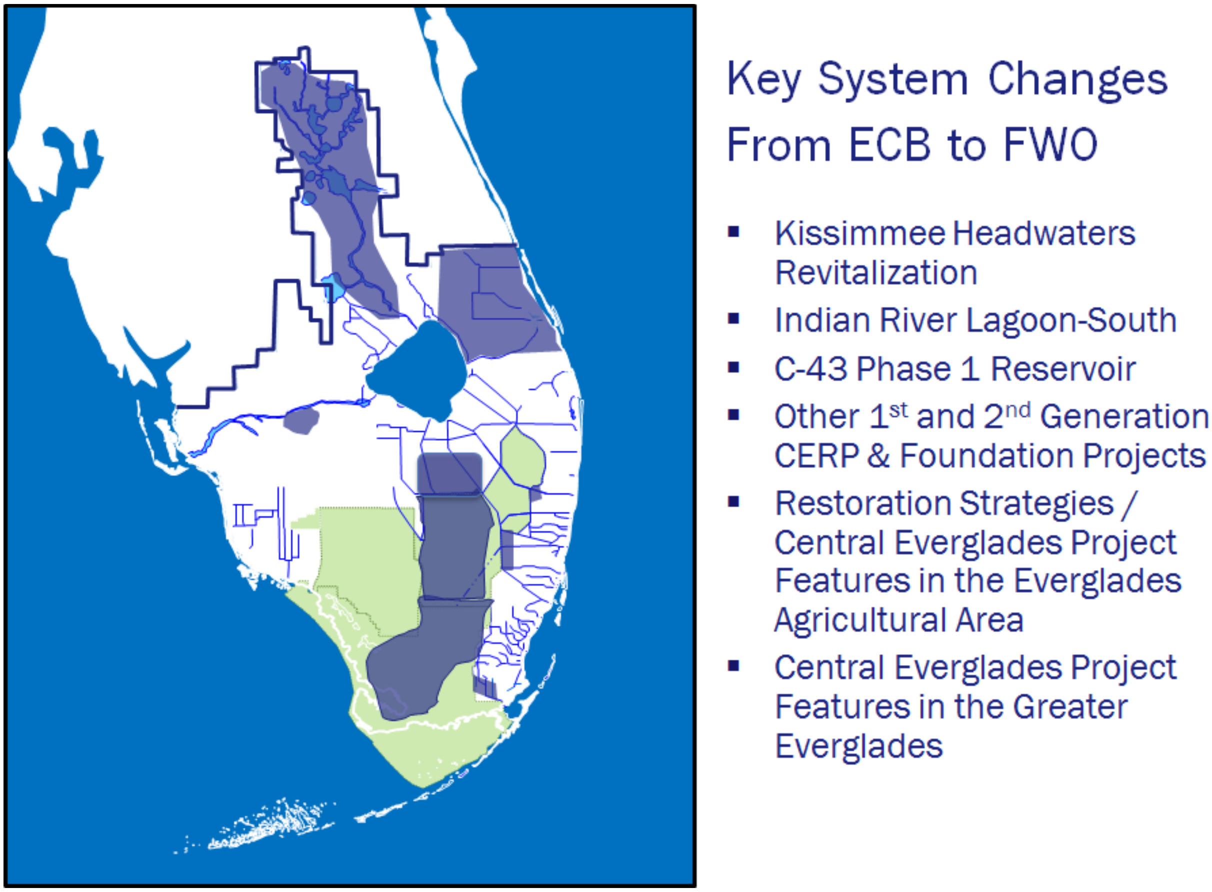

Figure 3.2. Key system updates differentiating the EARECB (existing condition baseline) and EARFWO (future condition without project baseline) model runs.

Kissimmee Headwaters Revitalization

Several projects seek to improve the water resource management and ecosystem performance for the Kissimmee River and the Upper Chain of Lakes (**SFWMD, 2007**). As considered in the RSMBN model, the following assumptions are made for operations at S65 and state of Kissimmee River restoration moving from EARECB to EARFWO:

- Modification to the Lake Kissimmee regulation schedule for releases at S65 in EARECB RSMBN is consistent with CEPP ECB (**SFWMD & IMC, 2014a**) as seen in **Figure 3.3**. **Figure 3.4** shows Lake Kissimmee regulation schedule for RSMBN EARFWO.
- Both EARECB and EARFWO include full Kissimmee River Restoration as shown in **Figure 3.5**.

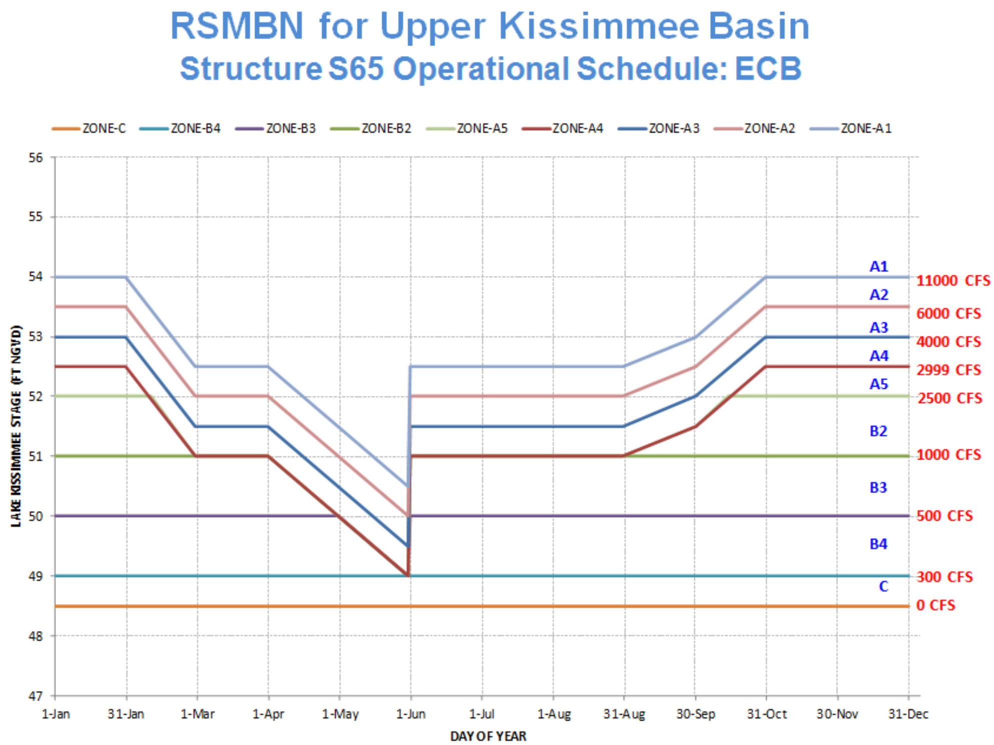

Figure 3.3. Lake Kissimmee Regulation Schedule for releases at S65 structure for the EARECB run.

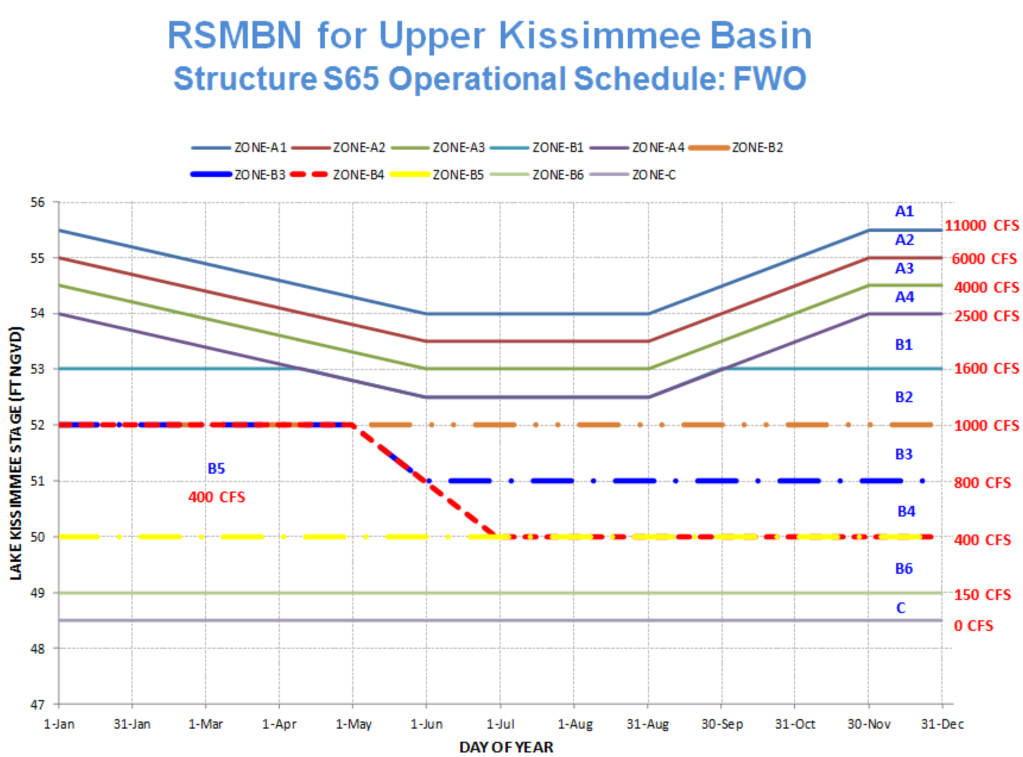

Figure 3.4. Lake Kissimmee Regulation Schedule for releases at S65 structure for the EARFWO run.

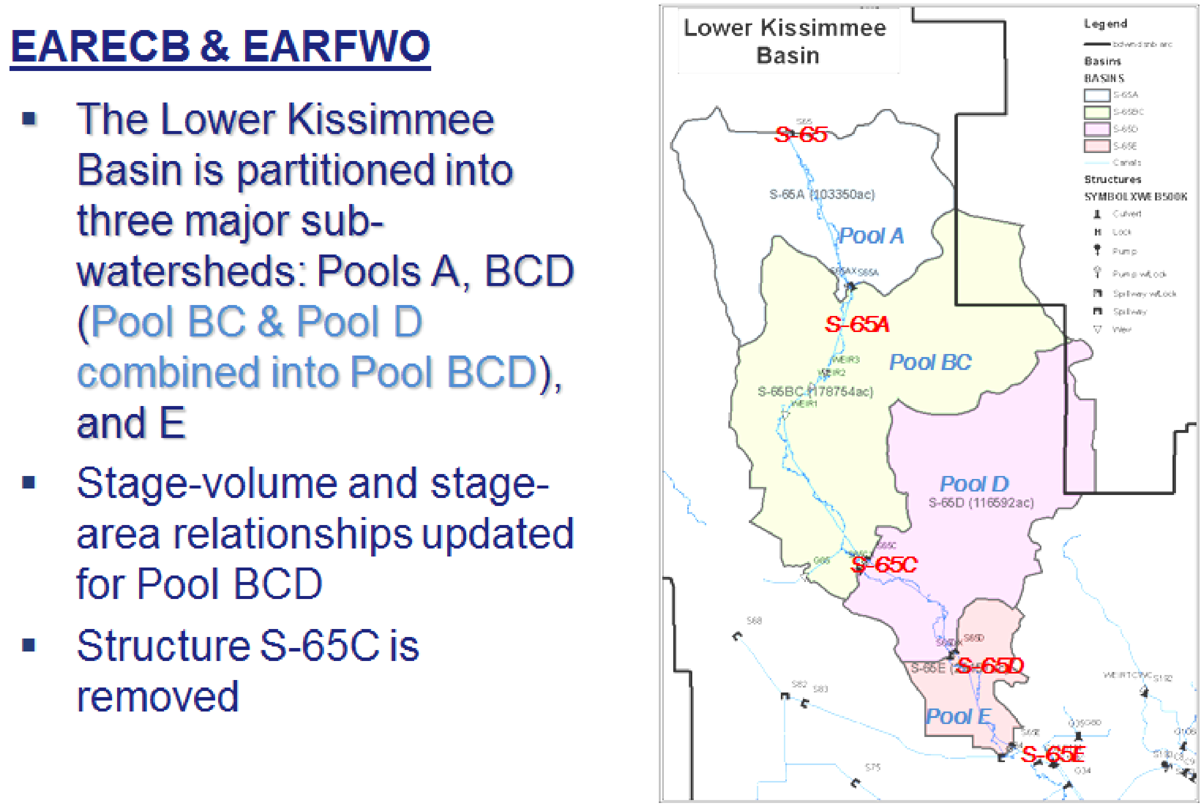

Figure 3.5. Kissimmee River Restoration as assumed in both the EARECB and EARFWO runs.

Indian River Lagoon-South

The purpose of the IRL South and Ten Mile Creek projects is to improve surface-water management in the C-23/C-24, C-25, and C-44 basins for habitat improvement in the Saint Lucie River Estuary and southern portions of the Indian River Lagoon**.** The RSMBN EARECB modification is consistent with CEPP ECB **(SFWMD & IMC, 2014a)** configuration and is shown in **Figure 3.6**. **Figures 3.6 and 3.7** show the model configuration for the RSMBN EARECB and EARFWO run, respectively:

General EARFWO Project Features

- Consistent with latest CERP Indian River Lagoon – South DDRs that update the authorized 2004 PIR.
- Includes operational intent (Opti6) per St Lucie River Watershed Protection Plan (January 2009) to meet St. Lucie estuary environmental targets.
- Water in the C44 reservoir is discharged and allowed to backflow to Lake Okeechobee when the Lake is below the baseflow zone consistent with the CEPP TSP operation. Environmental targets for the St Lucie Estuary are met from the reservoir prior to implementing this operation. C44 basin backflow is allowed when Lake stages are below 14.5 ft
- Basin demands can be met by project features.

C44 Reservoir and STA

- Storage capacity: 50,246 acre-feet

- Footprint: 12,125 acres (assumed 9700 effective acres / 80%)
- Inlet: 1060 cfs capacity, modeled as pump; source: C44 Basin
- Inlet: 250 cfs capacity, modeled as pump; source: C23 Basin
- Outlet: 550 cfs capacity, modeled as pump; destination: C44 Basin
- Cannot divert Lake Okeechobee regulatory releases into storage

C23/24 Reservoir

- Storage capacity: 92,094 acre-feet
- Footprint: 8675 acres (assumed 6940 effective acres / 80%)
- Inlet: 900 cfs capacity, modeled as pump; source: C23 Basin
- Inlet: 900 cfs capacity, modeled as pump; source: C24 Basin
- Outlet: 300 cfs capacity, modeled as pump; destination: C23 Basin
- Outlet: 300 cfs capacity, modeled as pump; destination: C24 Basin
- Outlet: 200 cfs capacity, modeled as pump; destination: C23/C24 STA

C23/C24 STA

- Storage capacity: 3852 acre-feet
- Footprint: 3323 acres (assumed 2568 effective acres / 80%)
- Inlet: 200 cfs capacity, modeled as pump; source: C23/C24 Reservoir
- Outlet: 200 cfs capacity, modeled as pump; destination: TMC Basin

Ten Mile Creek Reservoir and STA

- Storage capacity: 7078 acre-feet
- Footprint: 820 acres (assumed 656 effective acres / 80%)
- Inlet: 360 cfs capacity, modeled as pump; source: TMC Basin
- Outlet: 200 cfs capacity, modeled as pump; destination: TMC Basin

Indian River Lagoon ECB in RSMBN

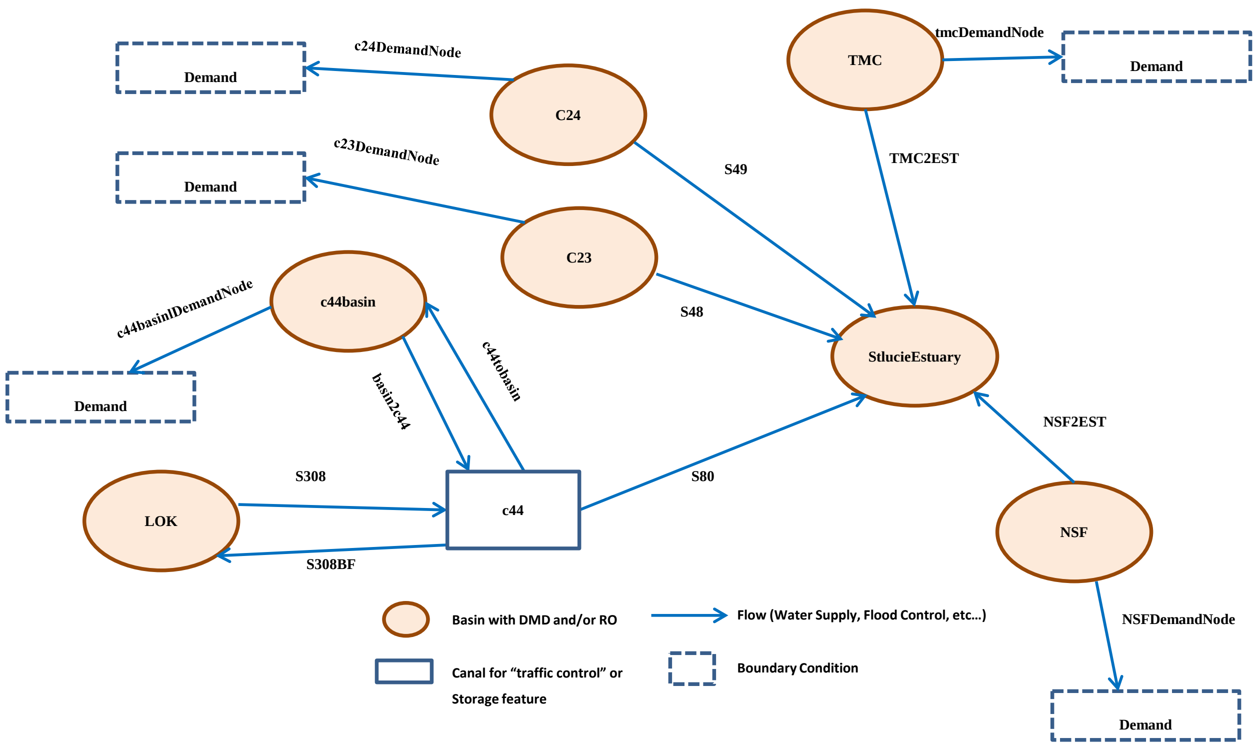

Figure 3.6. Indian River Lagoon-South routing in RSMBN for EARECB run.

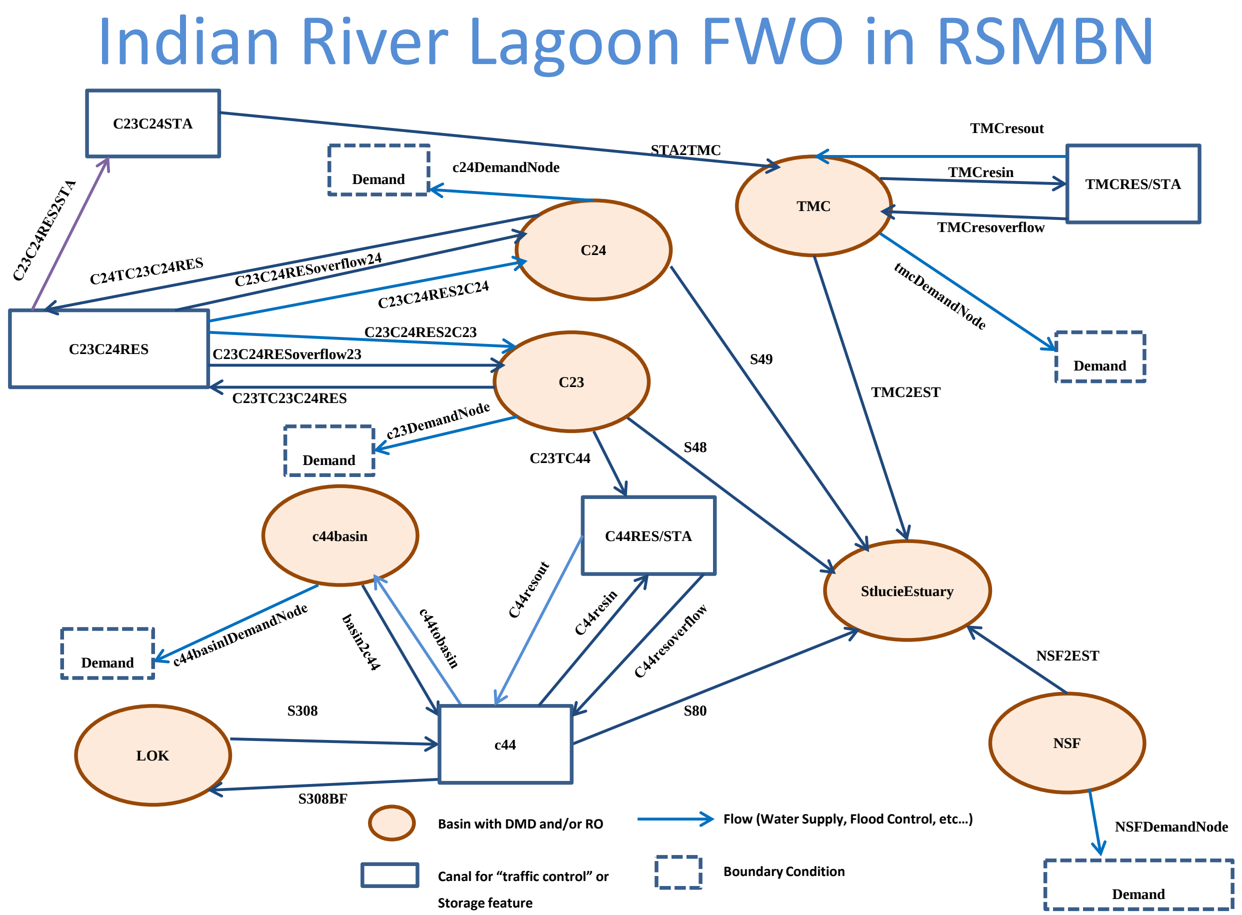

Figure 3.7. Indian River Lagoon-South routing in RSMBN for EARFWO run.

C-43 Phase 1 Reservoir

The purpose of the C-43 Basin Storage Reservoir - Part 1 project is to improve the timing, quantity, and quality of freshwater flows to the Caloosahatchee River estuary. The RSMBN EARECB modification is consistent with CEPP ECB (**SFWMD & IMC, 2014a**) configuration and is shown in **Figure 3.8**. **Figures 3.8 and 3.9** show the model configuration for the RSMBN EARECB and EARFWO run, respectively:

General FWO Project Features

- Modeled consistent with September 2007 PIR
- Storage capacity: 175,800 acre-feet
- Maximum footprint: 9,379 acres
- Inflow, capacity 1500 cfs, modeled as pump; destination: C43 Reservoir
- Outflow, capacity 1200 cfs modeled as pump; destination: C43 Canal
- Operates to meet estuary environmental target time-series (EST05)
- Can divert Lake Okeechobee regulatory releases into storage

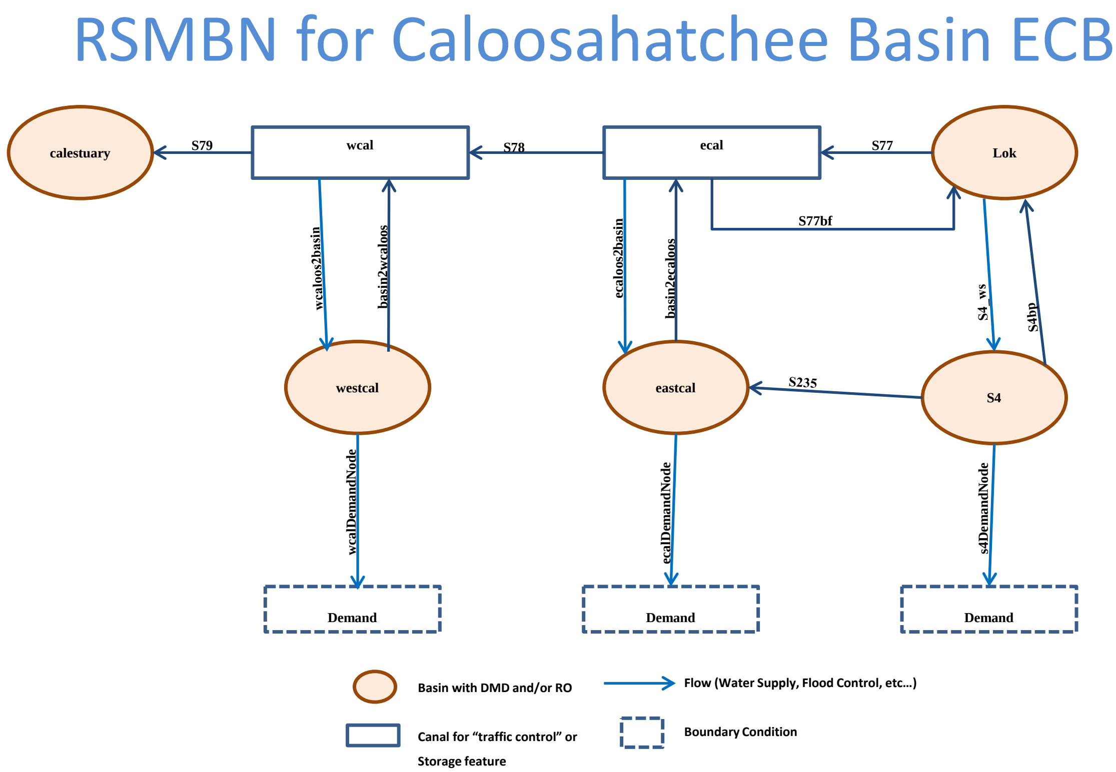

Figure 3.8. C43 Basin and Reservoir routing for ECB in RSMBN.

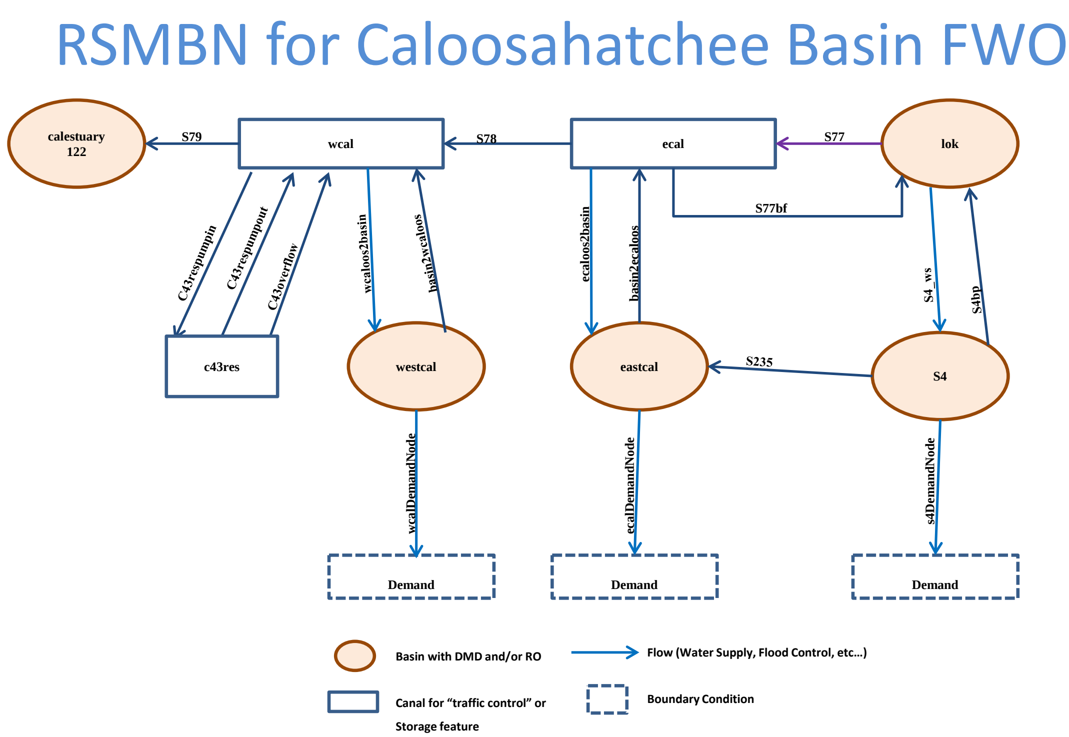

Figure 3.9. C43 Basin and Reservoir routing for FWO in RSMBN.

Other 1st and 2nd Generation CERP and Foundation Projects

A number of other projects in the South Florida area are assumed to be implemented in the period of time between the existing (EARECB) and future (EARFWO) conditions. While in some cases progress has been made on these features in the real world between 2012 and 2107, the RSMGL EARECB attempts to maintain consistency with the assumptions of the RSMGL 2012EC to facilitate comparisons back to the original CEPP authorized plan. In all cases in this section, the hydrologic effects of retaining legacy assumptions are not deemed to significantly affect the evaluation of EAASR due to their remote proximity to proposed plan features. The following inclusions or modifications are included when comparing the EARFWO to the EARECB:

Site 1 Impoundment

The Site 1 Impoundment Project (**Figure 3.10**) is designed to meet water supply demands in the LEC Service Area 1 through the collection of stormwater runoff from the Hillsboro canal that is currently discharged to tide. The project also intends to improve water levels in WCA-1/WCA-2A, provide long term storage for dry season water supply deliveries to the LWDD, and reduce demand from the regional system (Lake Okeechobee and WCA-1) to meet water supply needs.

- Footprint: Design: 1660 acres, Modeled: 1599 acres
- Inflow pump S-525A, capacity 650 cfs
- Pump on when G-56-H stage is > 7.7 ft (wet season) and 8.7 ft (dry season), and off when impoundment reaches 18.0 ft. NGVD
- Water supply outflows through S-526A culvert with discharge capacity 700 cfs
- Water supply priority for Hillsboro canal is first from the impoundment and second from S-39 regional supply

Assumed levee seepage coefficients in the southern L-40 reach were reduced by 75% to be consistent with best available information from the project field observations.

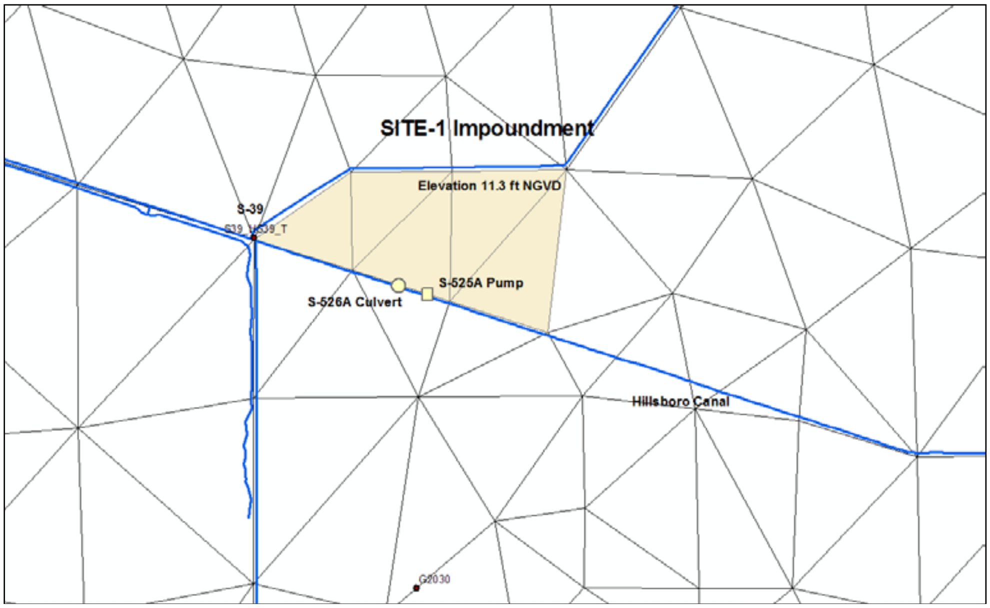

Figure 3.10. Site 1 Impoundment Project location with RSMGL mesh overlay.

Broward County Water Preserve Areas

The Broward County Water Preserve Areas Project in Broward County, is designed to reduce seepage losses from WCA-3A and WCA-3B and to capture and store excess runoff from the C-11 basin which is currently discharged untreated into WCA-3A via the S9 and S9A pumps. The project consists of two above ground impoundments, the C-11 impoundment (**Figure 3.11**) north of the C-11 canal, and the C-9 impoundment (**Figure 3.12**) north of the C-9 canal.

C-11 Impoundment

- Footprint: Design: 1078 acres, Modeled: 1355 acres
- Inflow pump S-503, capacity 1050 cfs
- Pump on when C-11 canal reaches 3.3 ft NGVD, and off when C-11 impoundment reaches 10.3 ft NGVD
- Outflow structure S-504 (1000 cfs) into C-9 impoundment when there is capacity
- Water supply outflow structure S-503A (200 cfs capacity)

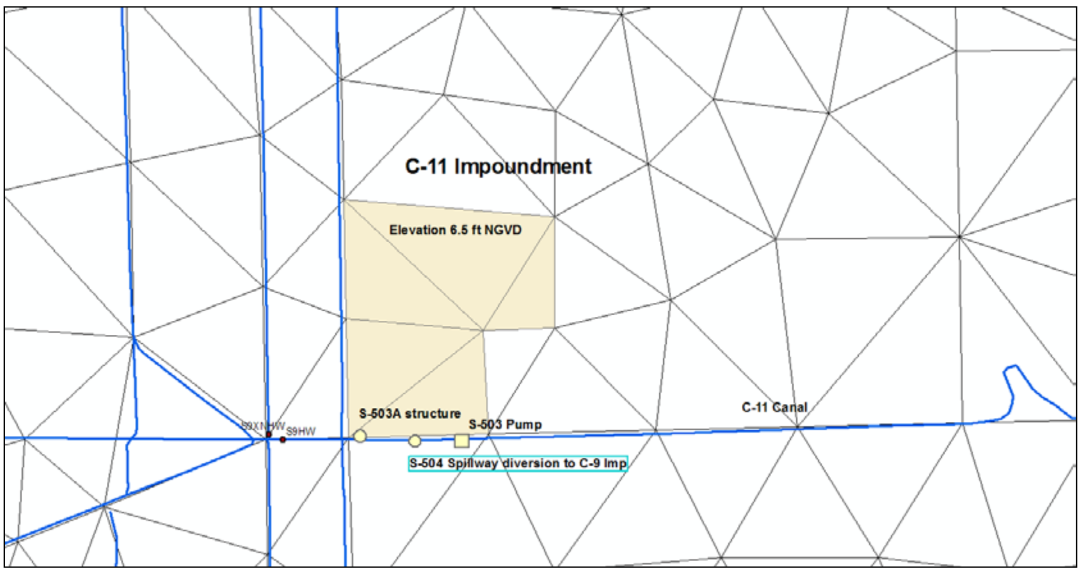

Figure 3.11. C-11 Impoundment Project location with RSMGL mesh overlay.

C-9 Impoundment

- Footprint: Design: 1641 acres, Modeled: 1970 acres
- Inflow Pump S-509, 1075 cfs capacity
- Pump on when C-9 canal stage above 2.7 ft NGVD and off when impoundment reaches 8.5 ft NGVD
- Outflow structure S-510, 500 cfs capacity
- Diversion flows into C-9 impoundment from C-11 impoundment through US27 canal and S-504 structure (capacity 1000cfs)

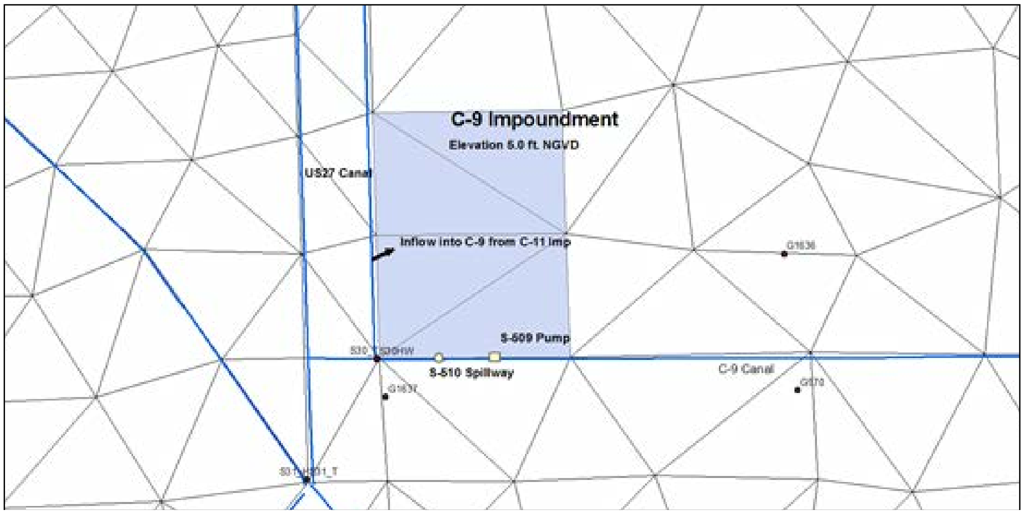

Figure 3.12. C-9 Impoundment Project location with RSMGL mesh overlay.

One-mile Tamiami Trail Bridge

Construction of the One-mile Tamiami Trail bridge per the 2008 Tamiami Trail Limited Reevaluation Report is intended to improve connectivity between ENP and upstream water bodies and allow for increased flow into northeastern ENP

- Modeled as a one-mile weir located east of the L67 extension and west of the S334 structure. Tamiami Trail culverts that fall within the one-mile location are removed.

C111 & 8.5 Square Mile Area

The C-111 project is a federally authorized project that has altered the south Florida system over the last twenty years to improve seepage management from ENP and hydrologic conditions in Taylor Slough. Consistent with a 1994 General Reevaluation Report, the project has constructed a number of shallow impoundments and associated water control structures along the south-eastern boundary of ENP (adjacent to urban development) and also altered levees and spoil mounds in the vicinity.

The 8.5 Square Mile Area Project (8.5 SMA) is a part of the Modified Water Deliveries (MWD) to ENP Project, authorized in the 2000 Water Resources Development Act and authorized specifically by the U.S. Congress in the 2003 Appropriations Act. Operations are based on the proposed water management operating criteria in this Environmental Assessment (**USACE, 2011**) which are interim and subject to change prior to completion of the ongoing long-term construction of the MWD Project and the Canal-111 South Dade Project.

EARECB:
Assumed project features are as follows and illustrated in **Figure 3.13**:

- Partial construction of C-111 project reservoirs consistent with 2009 as-built information from USACE.
- 8.5 SMA project per Alternative 6D in the General Reevaluation Report (**USACE, 2000**); operations per USACE 2011 Interim Operating Criteria.

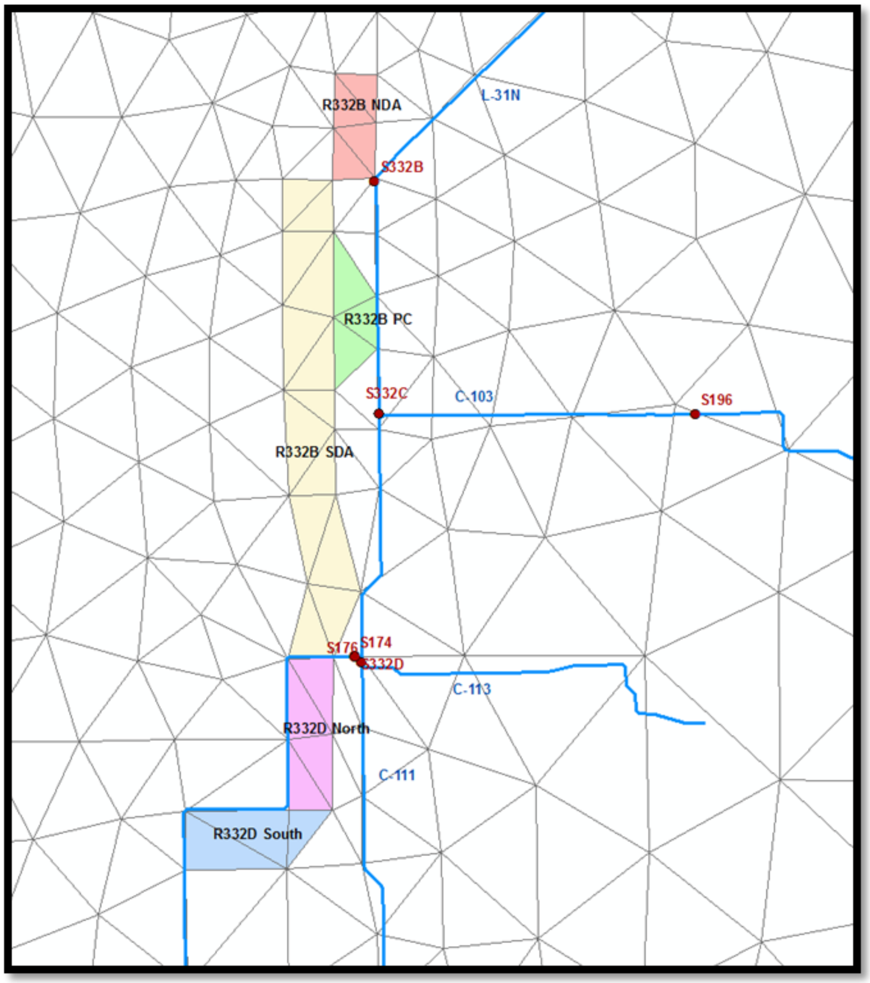

Figure 3.13: C-111 features as implemented in ECB

Five reservoirs (**Table 3.1**) are added as impoundments per 2009 as-built C-111 South Dade project:

Table 3.1: C-111 project reservoirs details

Reservoirs	Average topography (ft NGVD29)	Modeled area (acres)
R-332B North Detention Area (R-332B NDA)	6.5	257
R-332B South Detention Area (R-332B SDA)	6	1404
R-332 Partial Connector (R-332PC)	6	264
R-332 D North (R-332DN)	5.5	399
R-332 D South (R-332BN)	5.5	428

Table 3.2 shows the structures that are added for the purpose of flood control in S332 reservoir area.

Table 3.2: S332 reservoir flood control structures

	Type (as modeled)	capacity	upstream	downstream
S316D	gravity	400	R332B NDA	eastern cell
S322A	gravity	350	R332B SDA	R332PC
S322B	gravity	125	R332B SDA	R332PC
S322D	gravity	250	R332B SDA	R332PC
S322E	gravity	500	R332B SDA	R332PC
S322G	gravity	1500	R332B SDA	eastern cell
SDNWE	gravity	500	R332DN	R332DS
S329	gravity	1000	R332DS	south cell

Other information:
S-332, S-174 and S-175 are not operational. S-332A pump is not added.

EARFWO:
Assumed project features are as follows and illustrated in **Figure 3.14**:

- Full construction of C-111 project including addition of contract 8 and 9 features. Contract 9 includes degrading 500 feet of S-327, but the high head cell is not represented in the RSM due to resolution.
- 8.5 SMA project same as EARECB, plus 1259 acre R-332B North Detention Area plus outflow assumed to the C-111 North Detention Area from 8.5 SMA through S-360E and S-360W structures.

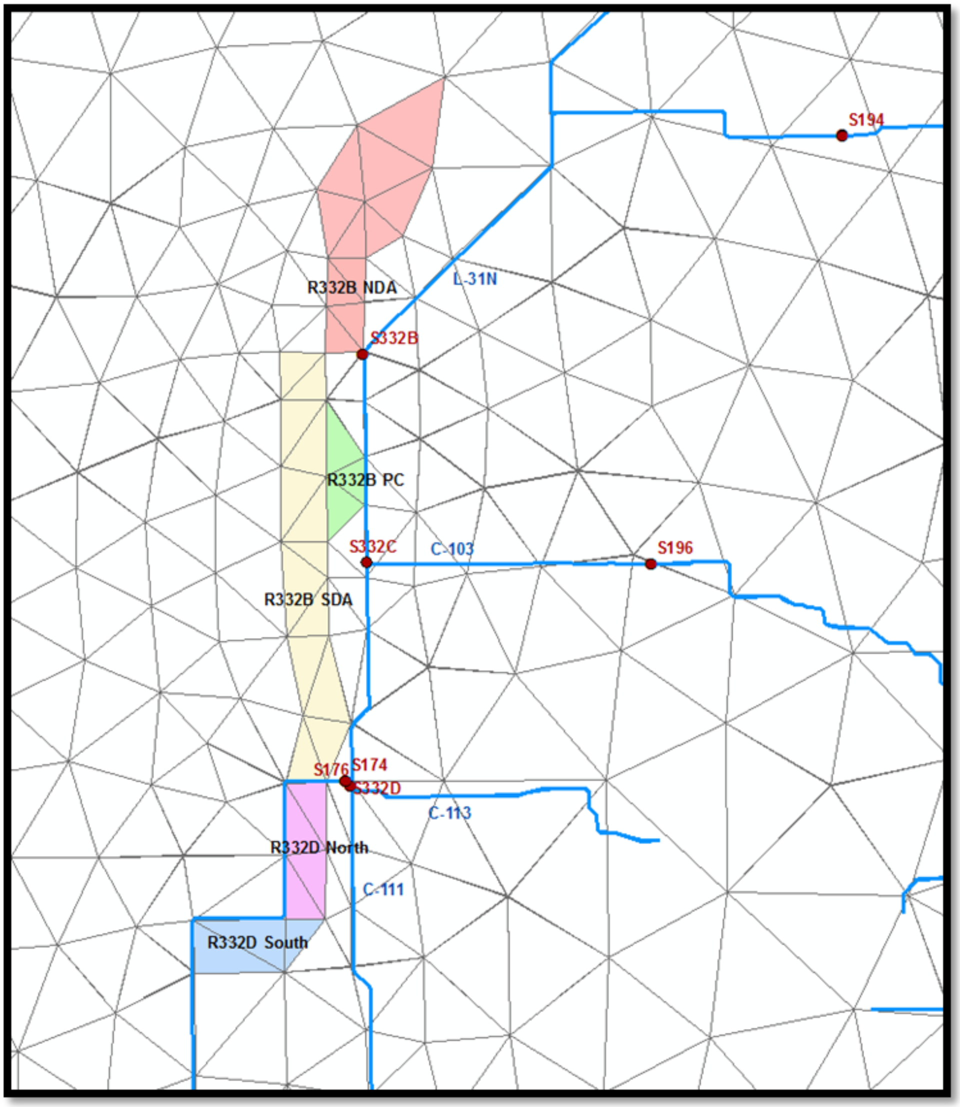

Figure 3.14: C-111 features as implemented in FWO

In addition to the flood control structures added in S-332 reservoir areas (shown in **Table 3.1**), following flood control structures are also added (**Table 3.3**):

Table 3.3: S332 reservoir flood control structures (in addition to ECB)

Structures	Type (as modeled)	capacity	upstream	downstream
S316A	gravity	500	R332B NDA	eastern cell
S316B	gravity	500	R332B NDA	eastern cell
S316C	gravity	500	R332B NDA	eastern cell
S322C	gravity	500	R332B SDA	R332PC
S322F	gravity	500	R332B SDA	eastern cell
S322H	gravity	500	R332B SDA	eastern cell

Restoration Strategies / Central Everglades Project Features in the Everglades Agricultural Area

As part of the Everglades Construction Project and Restoration Strategies program, the state of Florida plans to expand water quality treatment facilities to improve the quality of water entering the Everglades Protection Area. In the Everglades Agricultural Area, a full build-out of SFWMD Stormwater Treatment Area (STA) features contemplated in the Everglades Construction Project (including Compartment B & C) expansions are modeled in both the EARECB and EARFWO and details can be found the assumption tables in **Appendix A**. The primary differences between the two baselines are assumptions related to assumed Flow Equalization Basin (FEB) features as follows:

EARECB A-1 FEB Assumptions:

- Assumed Flowage Equalization Basin Effective Footprint = 15,853 acres
- FEB operating limits:
  - o EAA runoff accepted when FEB stage < 3.8 ft.
  - o Lake Okeechobee flood control water can be accepted into FEB
  - o Discharges discontinued when depths < 0.5 ft.
  - o No supplemental water supply provided to FEB.
  - o FEB outflows are used to help meet established inflow targets, as estimated using the Dynamic Model for Stormwater Treatment Areas (**Wang, 2012**) at STA-3/4, STA-2N, and STA-2S

EARFWO A-1/A-2 FEB (as shown in **Figure 3.15)** Assumptions:

- In RSMBN, this feature is modeled as a single element
- Assumed Flowage Equalization Basin Effective Footprint = 28,467 acres
- FEB operating limits:
  - o EAA runoff accepted when FEB stage < 3.8 ft.
  - o Lake Okeechobee water accepted when FEB stage < 2.0 ft.
  - o Discharges discontinued when depths < 0.5 ft.
  - o No supplemental water supply provided to FEB.
  - o FEB outflows are used to help meet established inflow targets, as estimated using the Dynamic Model for Stormwater Treatment Areas (**Wang, 2012**) at STA-3/4, STA-2N, and STA-2S
- The operations of the FEB are integrated with the regional objectives by including operational modifications to the Lake Okeechobee regulation schedule as follows:
  - o Lake Okeechobee regulatory releases to the south are made when the Lake is in or above the baseflow zone of the LORS08 schedule and when criteria as identified in **Figure 3.16** are satisfied.
  - o In order to promote opportunity for Lake discharges to the south, release criteria from the Northern Estuaries are also modified to result in lower overall discharges. Specific changes can be found in **Appendix B.**

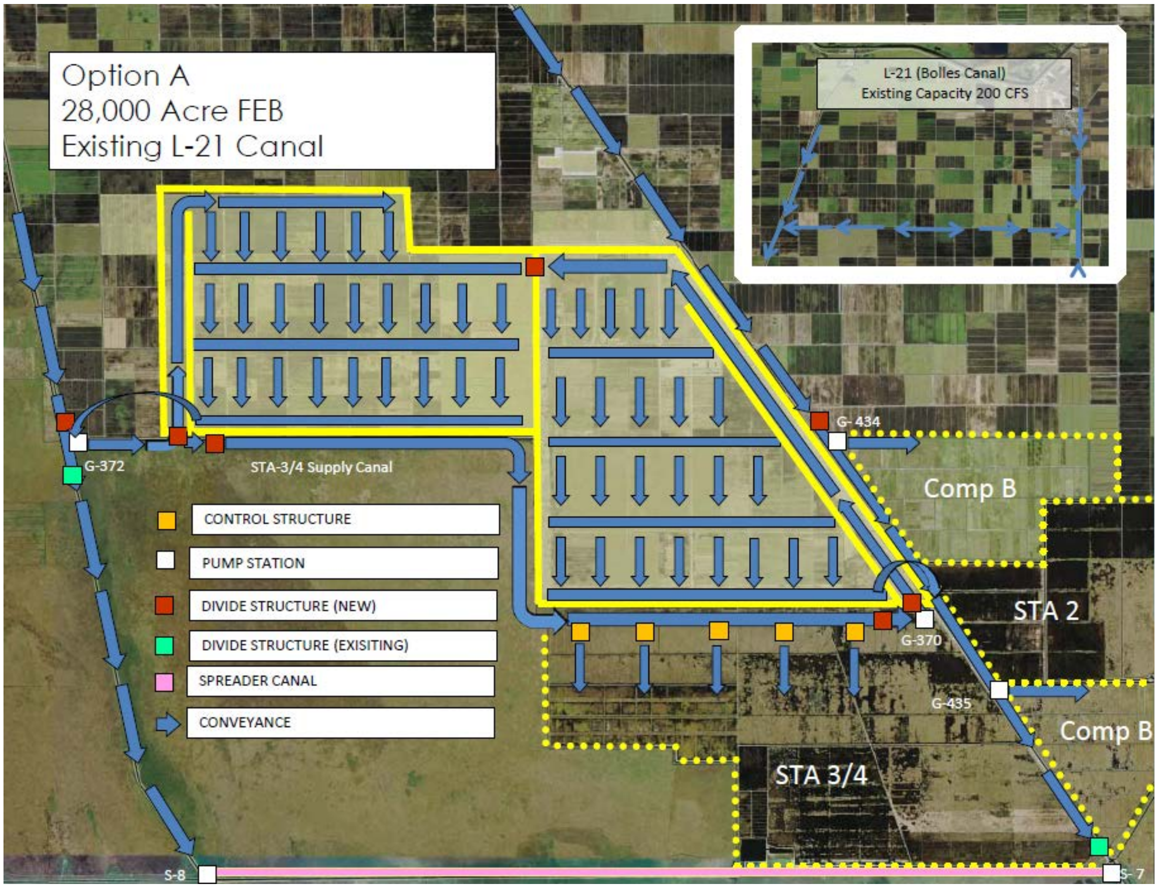

Figure 3.15. A1/A2 FEB Schematic Diagram Provided by CEPP Project Team

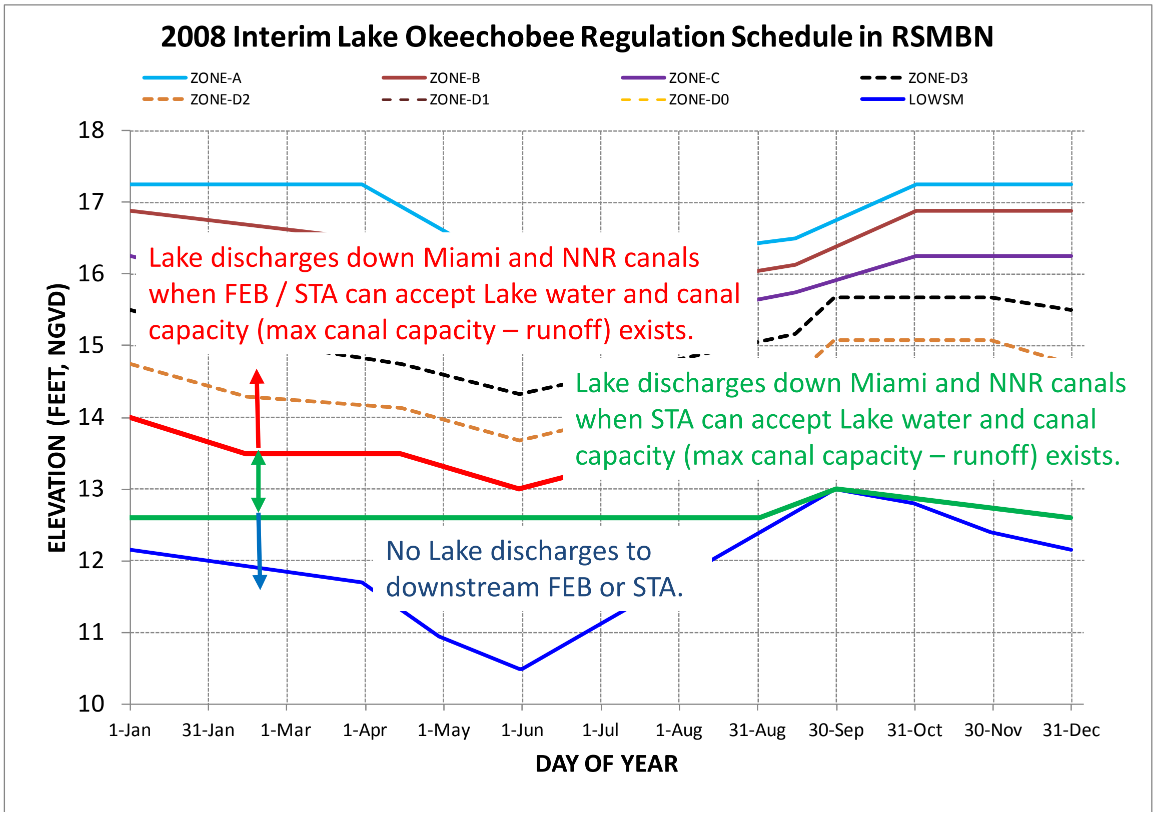

Figure 3.16. Lake Okeechobee Operational Criteria for Determining Discharges South to FEB and STA Facilities.

Central Everglades Project Features in the Greater Everglades

In the EARECB scenario, the Everglades Restoration Transition Plan (ERTP) is assumed (consistent with the CEPP RSMGL 2012EC scenario) and simulates the operations of WCA3a, inflows to ENP and protection of the Cape Sable Seaside Sparrow. As modeled in the EARECB:

- Modeled as per Run 9E1 of March 2011 EIS (**USACE, 2011**).
- Revised WCA3A Regulation Schedule with lower Zone A line (**Figure 3.17**).
- Modifications to Tamiami Trail structure operations and closures consistent with Run 9E1 (e.g. S12 closure dates).
- L-29 stage constraint for operation of S-333 assumed to be 7.5 ft, NGVD.
- G-3273 constraint for operation of S-333 assumed to be 6.8 ft, NGVD.

Figure 3.17: WCA3A ERTP Schedule, EARECB in RSMGL

In the EARFWO scenario, full implementation of the CEPP selected plan as shown in **Figure 3.18** are assumed (consistent with the CEPP RSMGL ALT4R2 scenario). The details of the model representation of these features can be found in the CEPP TSP MDR (**SFWMD&IMC, 2014b**). It is important to restate that the EAASR modeled EARFWO scenarios are identical to the corresponding CEPP ALT4R2 model scenarios.

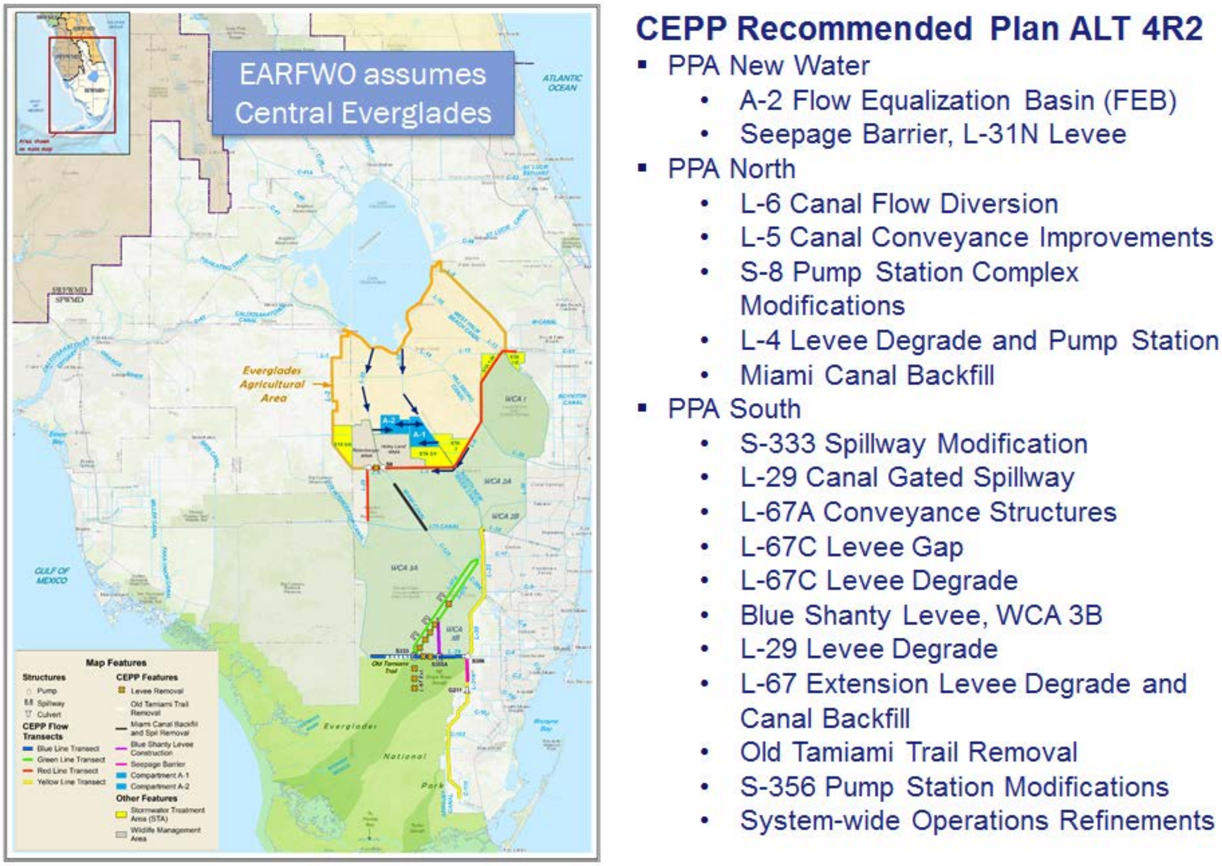

Figure 3.18. Central Everglades features assumed in EARFWO

4.0 Results

Final EAASR Baseline modeling products have been uploaded to the SFWMD FTP site, it includes model input data, select model output data, source code/executable files and documentation. It can be accessed at ftp://ftp.sfwmd.gov/pub/EAASR/
EAASR modeling products can be accessed directly at the project page:
https://www.sfwmd.gov/our-work/cerp-project-planning/eaa-reservoir.

While the modeling products have been archived in the above systems, **Table 4.1** below lists more specific information including model version, inputs used and detailed archival location. Version numbers and "svnroot" paths refer to a model version control system found on the SFWMD network that is not generally accessible, but inputs, model executables and source code have been copied into the ftp system for ease of access.

Table 4.1 Version information and model file locations

RSMBN ECB 11092017	**RSM_v5205 and xml_v12365**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmbn/baselines/EARECB Output : .../dfsroot/data/hesm_nas/projects/CEPP_EAR/FilesToFTP//PlanFormulation/Baselines/01_06Nov2017/rsmbn_model_output/EARECB	
RSMBN FWO 11092017	**RSM_v5205 and xml_v12365**
Input:…svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmbn/baselines/EARFWO Output: .../dfsroot/data/hesm_nas/projects/CEPP_EAR/FilesToFTP//PlanFormulation/Baselines/01_06Nov2017/rsmbn_model_output/EARFWO	
RSMGL ECB 11092017	**RSM_v5205 and xml_v12388**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmgl/baselines/EARECB Output: .../dvsroot/data/hesm_nas/projects/CEPP_EAR/FilesToFTP//PlanFormulation/Baselines/01_06Nov2017/rsmgl_model_output/EARECB	
RSMGL FWO 11092017	**RSM_v5205 and xml_v12388**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmgl/baselines/EARFWO Output: .../dvsroot/data/hesm_nas/projects/CEPP_EAR/FilesToFTP//PlanFormulation/Baselines/01_06Nov2017/rsmgl_model_output/EARFWO	

Review of Local and Regional-Level Results

The RSMBN and RSMGL baseline modeling scenarios were reviewed from the perspective of ensuring that localized effects of project implementations were observed as expected and that regional performance was considered reasonable. Specific checks on RSM outputs included the following:

- Regional results for Lake Okeechobee and the Northern Estuaries demonstrated expected trends (reduced high discharge events, improved baseflow to the Caloosahatchee estuary, etc resulting from IRL, C43, A1FEB and other project activities as shown in **Figures 4.1** and **4.2**.
- Regional results for the WCAs and ENP demonstrated expected trends (higher WCA3A stages, increased flows to eastern ENP and decreased flows to western ENP, etc.) resulting from ERTP, Tamiami Trail and other project activities as shown in **Figures 4.3** to **4.6.**
- LEC and ENP buffer impoundment performance was reasonable and demonstrated expected sub-regional changes in LEC groundwater as shown in **Figures 4.7** through **4.10**.

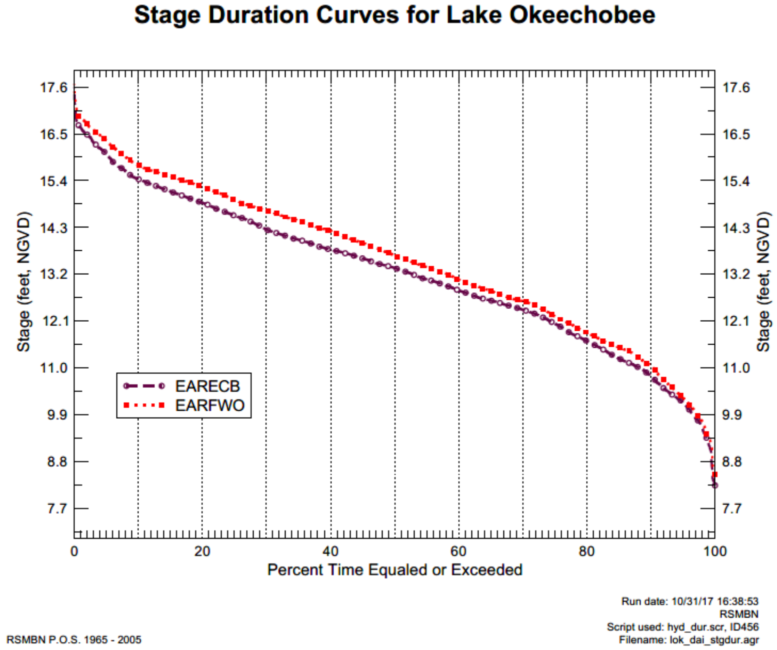

Figure 4.1. Lake Okeechobee demonstrated expected trends resulting from project activities included in the EARFWO run.

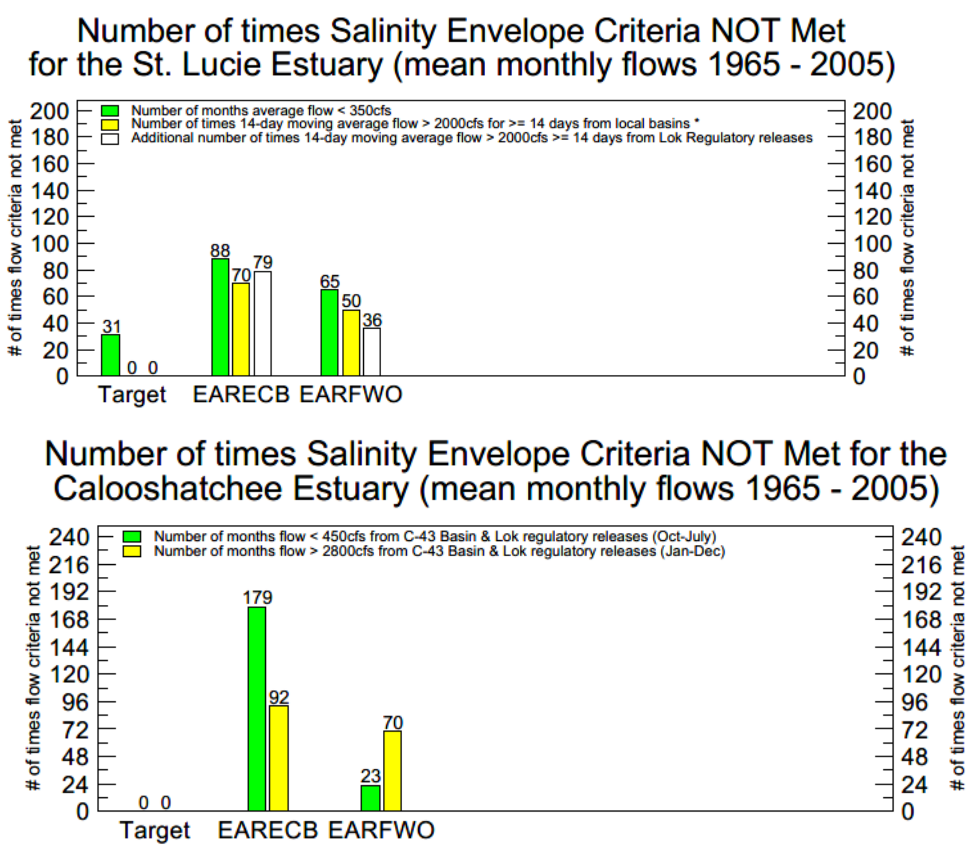

Figure 4.2. Northern Estuaries (St. Lucie and Caloosahatchee) demonstrated expected trends resulting from project activities included in EARFWO runs.

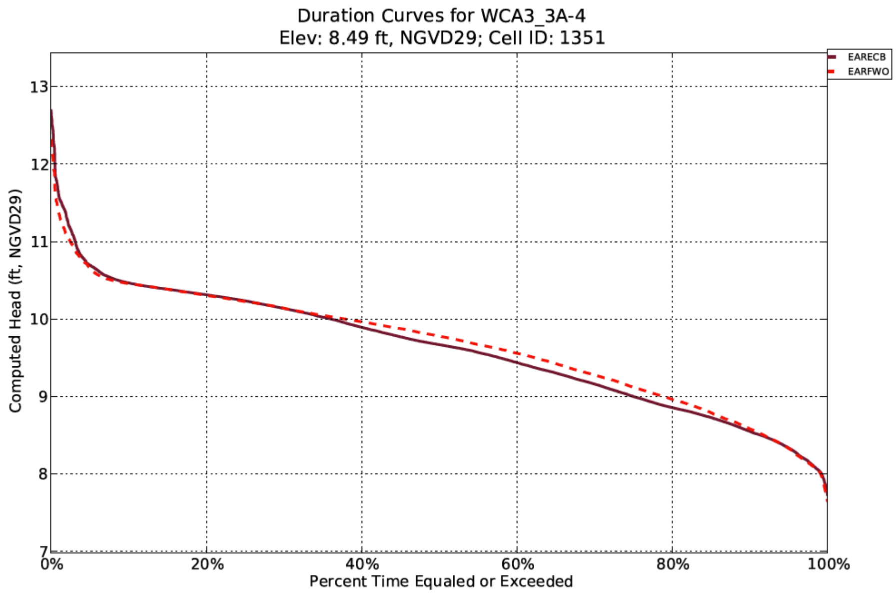

Figure 4.3. A representative gauge in WCA3A showing changes in stage duration between EARECB and EARFWO.

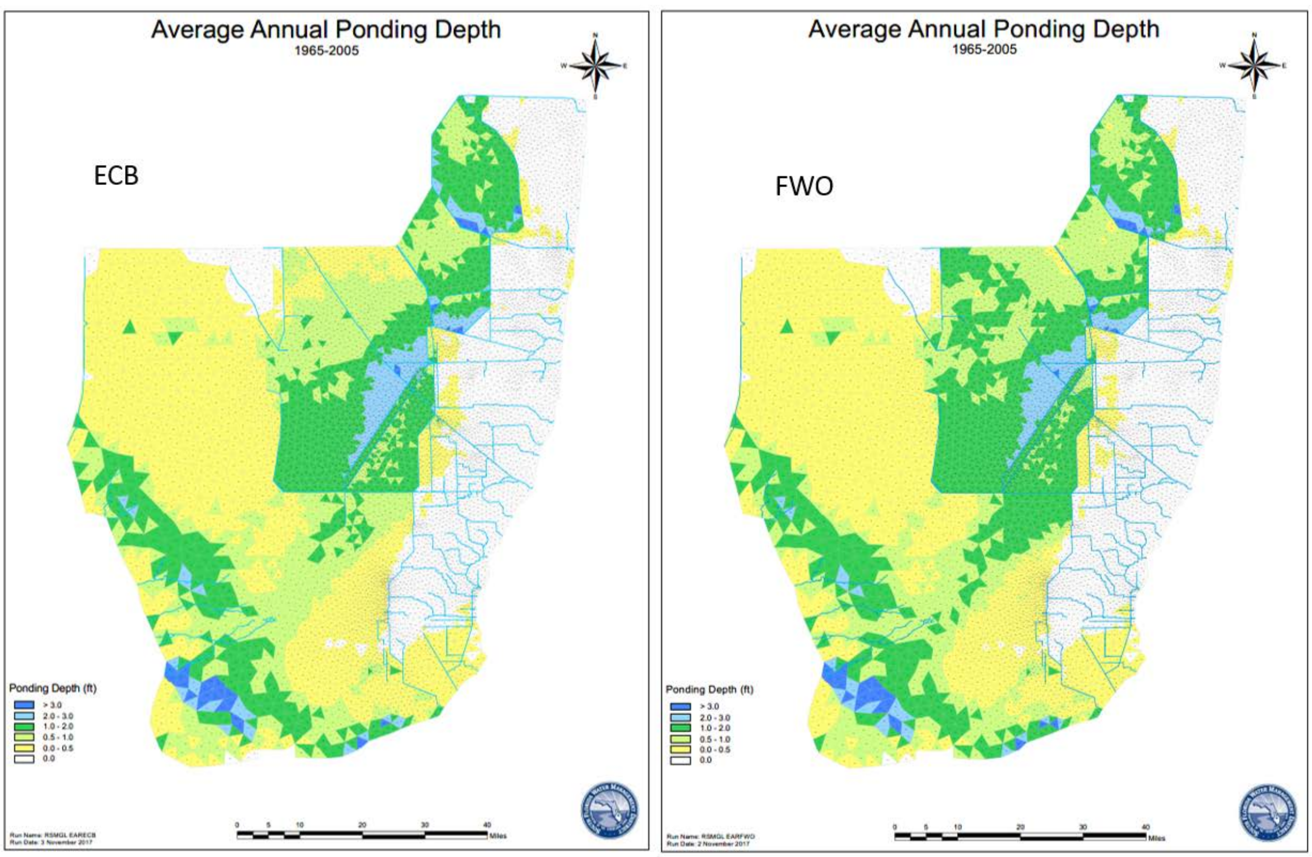

Figure 4.4. System-wide average ponding EARECB and EARFWO.

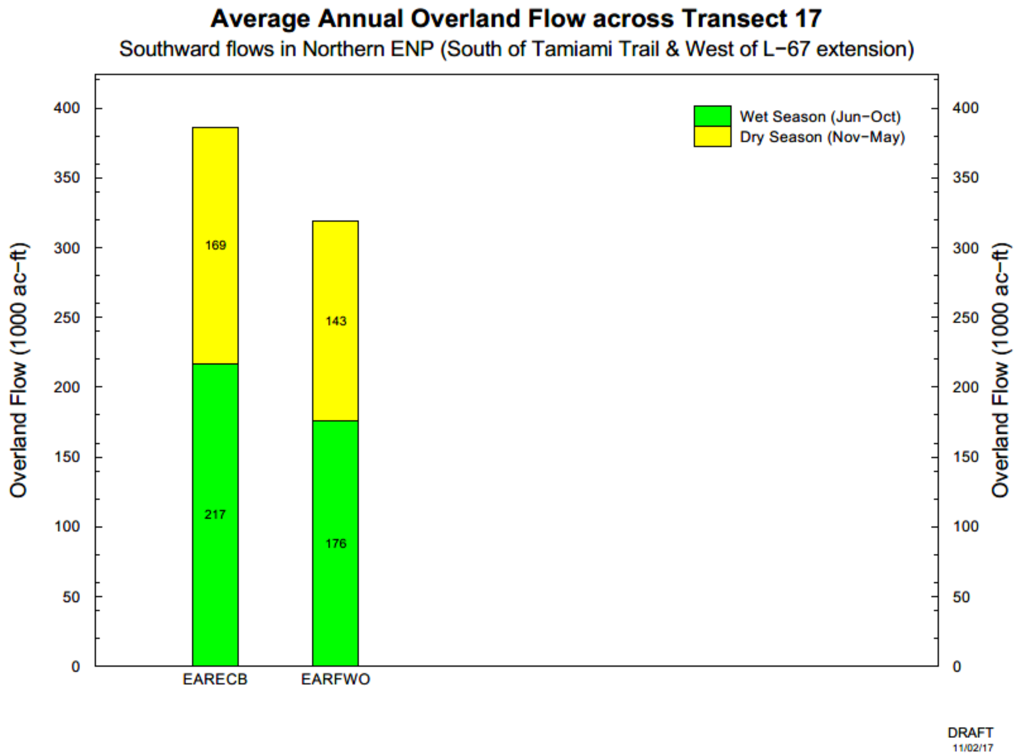

Figure 4.5. Flow differences between EARECB and EARFWO at western Tamiami Trail.

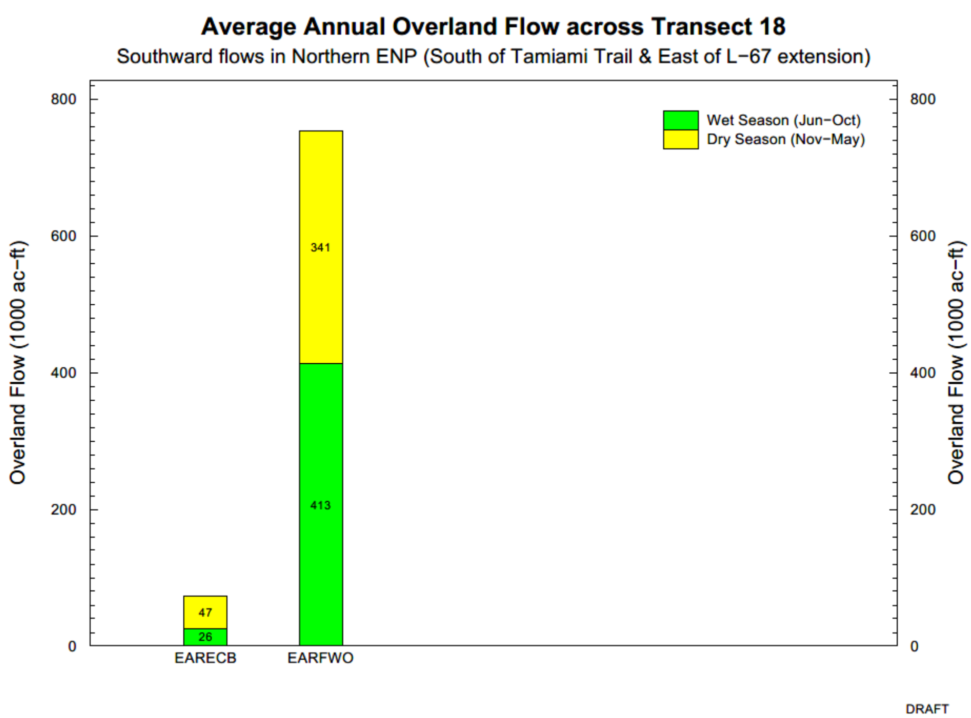

Figure 4.6. Flow differences between EARECB and EARFWO at eastern Tamiami Trail.

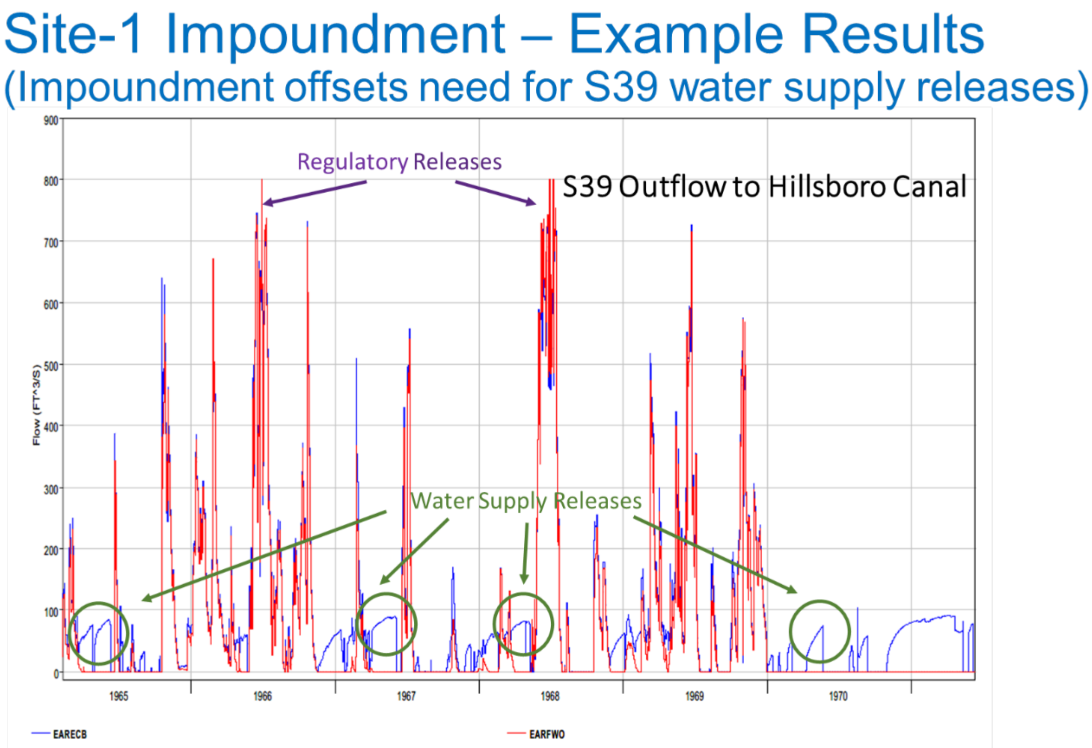

Figure 4.7. Site 1 Impoundment implementation example results.

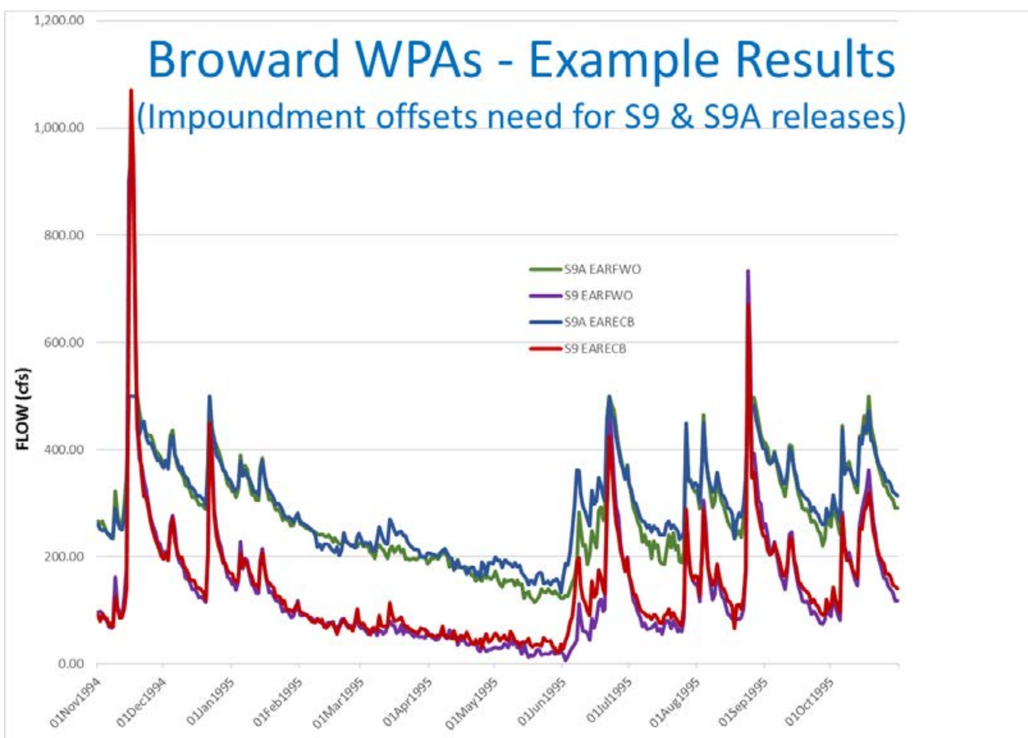

Figure 4.8. Broward County Water Preserve Areas implementation example results

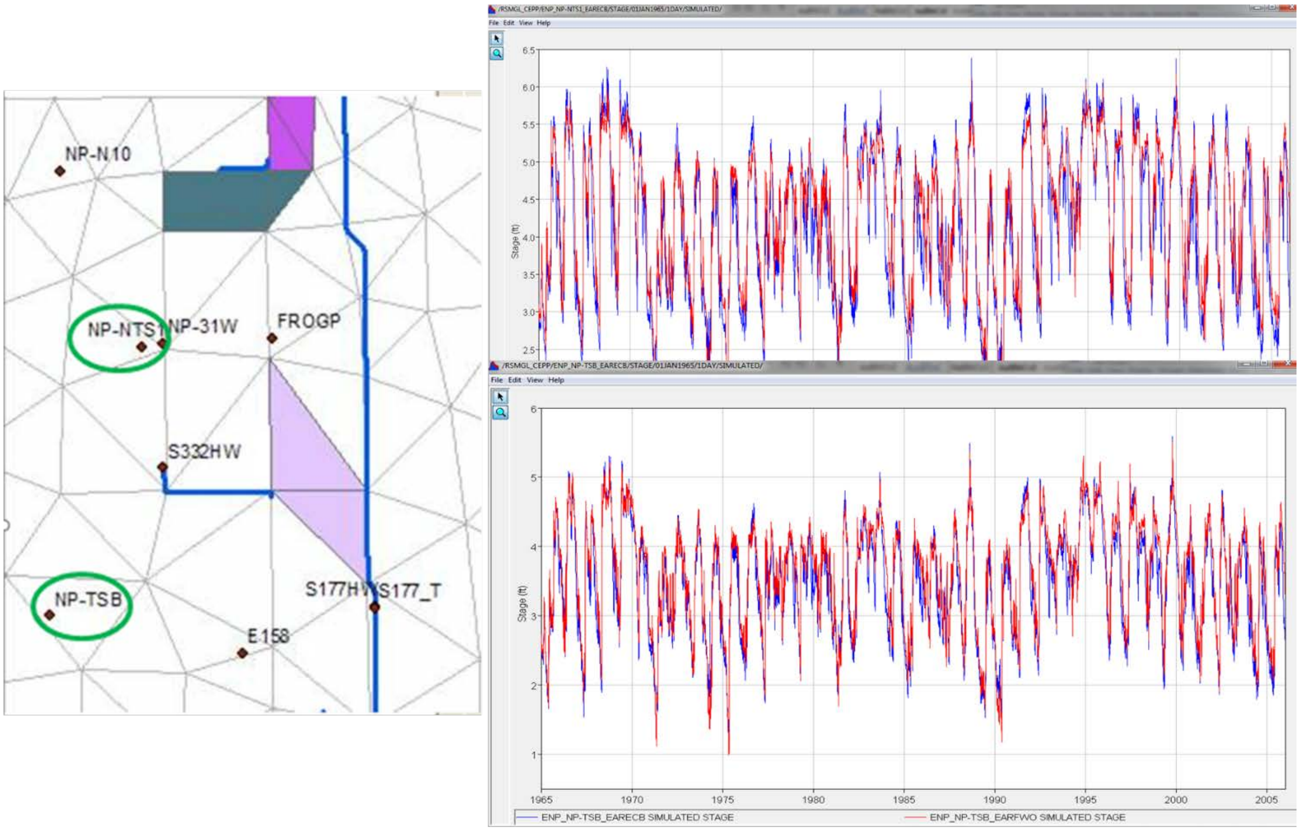

Figure 4.9. C-111 Spreader Canal Project implementation example results

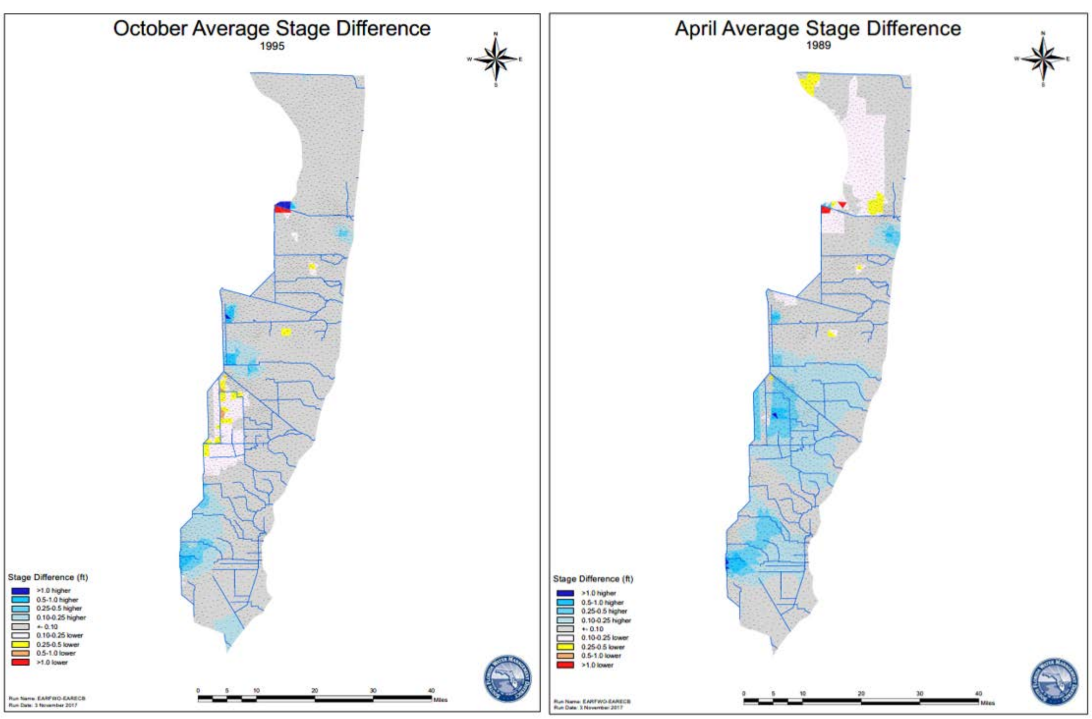

Figure 4.10. Difference between EARFWO and EARECB stages in an average water year, in October 1995 (left) and April 1989 (right).

In summary, the delivered baseline runs provided to the EAASR project team are deemed to adequately represent the intended planning conditions and when utilized in conjunction with proposed with-EAASR project alternatives, provide a reasonable basis of comparison for the necessary evaluations required to draft the PIR. Additionally, the modeling products for EAASR are consistent with those performed for the original CEPP support and can be directly compared to illustrate project changes.

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Appendix A – Tables of Assumptions

RSMBN: EARECB, EARFWO
RSMGL: EARECB, EARFWO

Modeling Section, H&H Bureau

Regional Simulation Model Basins (RSMBN)
EAA Reservoir Existing Conditions Baseline (EARECB)
Table of Assumptions

Feature	
Climate	• The climatic period of record is from 1965 to 2005 • Rainfall estimates have been revised and updated for 1965-2005 • Revised evapotranspiration methods have been used for 1965-2005
Topography	The topography dataset for RSM was updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004 • High Accuracy Elevation Data, US Geological Survey 2007 • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004 • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007 • Palm Beach County LiDAR Survey, Dewberry and Davis, 2004 • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/2012, as reflected in the LOSA Ledger produced by the Water Use Bureau. Project features simulated in the EAA (above and beyond the Everglades Construction Project) remove land from agricultural production. • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).
Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008) o Includes Lake Okeechobee regulatory releases to tide via L8/C51 canals o Lake Okeechobee regulatory releases limited to 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal based on studies performed by USACE. o A regional hydrologic surrogate for the 2010 Adaptive Protocol operations utilized. This attempts to mimic desired timing of releases without estimating salinity criteria • Lake Okeechobee Water Shortage Management (LOWSM) Plan • Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and S-3 is to be minimized) • “Temporary” forward pumps as follows: o S354 – 400 cfs o S351 – 600 cfs o S352 – 400 cfs

Feature	
	o All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages recover to greater than 11.2 ft. • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Operational intent is to treat LOK regulatory releases to the south through STA-3/4 • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Kissimmee River inflows based on interim schedule for Kissimmee Chain of Lakes using the UKISS model • Kissimmee River Restoration complete. • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012 (see land use assumptions row). • Public water supply daily intake from the river is included in the analysis.
St. Lucie Canal Basin	• St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012(see land use assumptions row). • Basin demands include the Florida Power & Light reservoir at Indiantown.
Seminole Brighton Reservation	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons per month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal rights to these quantities are preserved • LOWSM applies to this agreement
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved • LOWSM applies to this agreement

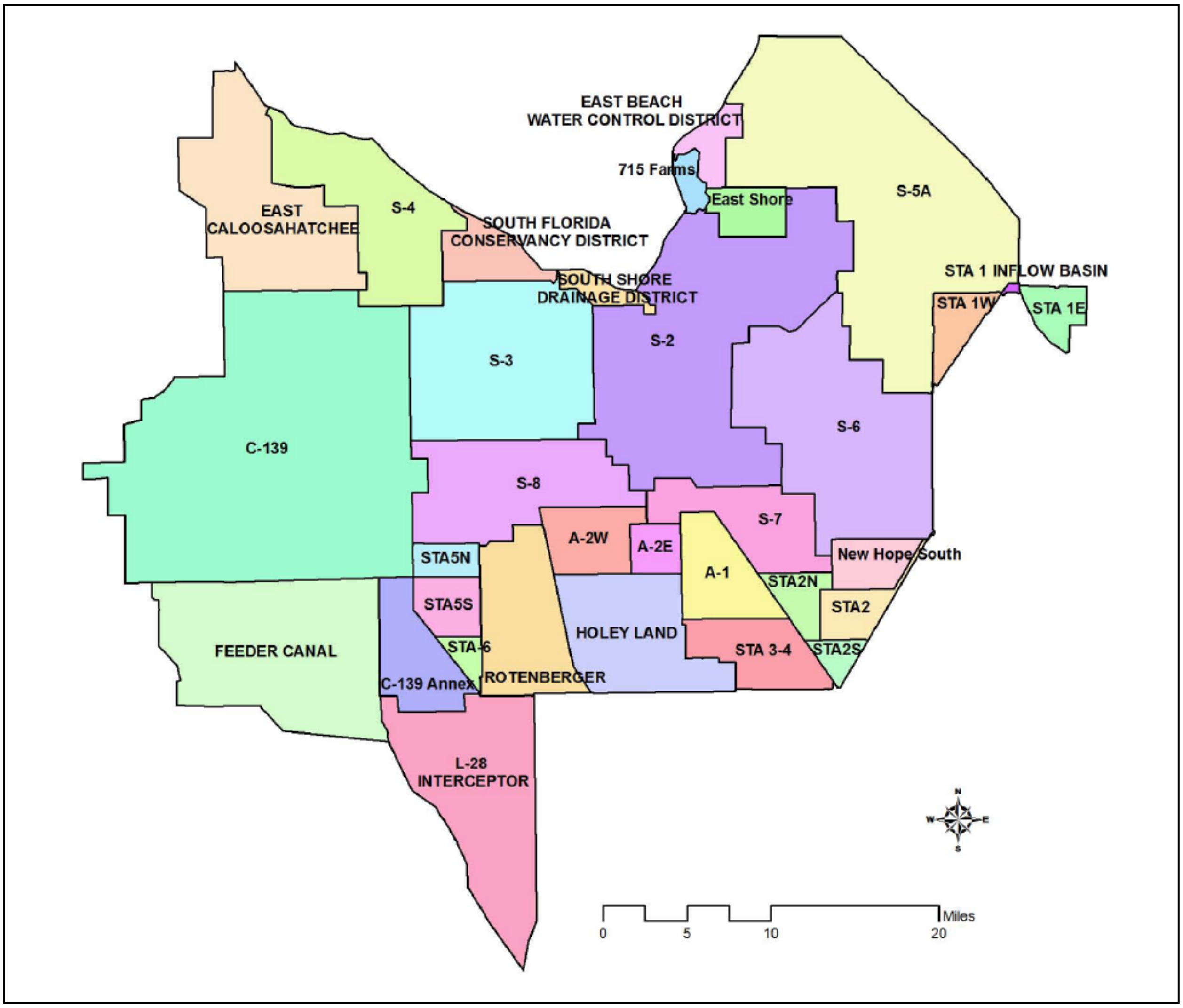
Feature	
Everglades Agricultural Area	• Model water-body components as shown in Figure 1. • Simulated runoff from the North New River – Hillsboro basin apportioned based on the relative size of contributing basins via S7 route vs. S6 route. • G-341 acts as a divide between S-5A Basin and Hillsboro Basin. • RSMBN ECB EAA runoff and irrigation demand compared to SFWMM ECB simulated runoff and demand from 1965-2005 for reasonability.

Everglades Construction Project Stormwater Treatment Areas	• STAs are simulated as single waterbodies
	• STA-1E: 6,546 acres total area
	• STA-1W: 7,488 acres total area
	• S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E
	• STA-2: cells 1,2 & 3: 7,681 acres total area
	• STA-2N: cells 4,5 & 6; refers to Comp B-North; 6,531 acres total area
	• STA-2S: cells 7 & 8; refers to Comp B-South; 3,570 acres total area
	• STA-3/4: 17,126 acres total area
	• STA-5N: includes cells 1 & 2: 5,081 acres total area
	• STA-5S: includes cells 3, 4 & 5; uses footprint of Compartment C: 8,469 acres total area
	• STA-6: expanded with phase 2: 3,054 acres total area
	• Assumed operations of STAs:
	  o 0.5 ft minimum depth below which supply from external sources is triggered
	  o 4 ft maximum depth above which inflows are discontinued
	  o Inflow targets established for STA-3/4, STA-2N and STA-2S based on DMSTA simulation; met from local basin runoff, LOK regulatory discharge and available A1FEB storage.
	  o STA-3/4 receives Lake Okeechobee regulation target releases approximately at 60,000 acre-feet annual average for the entire period of record.
	• A 15,853-acre Flow Equalization Basin (FEB) located north of STA-3/4 with assumed operations as follows:
	  o FEB inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south.
	  o FEB outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at STA-3/4, STA-2N, and STA-2S if EAA basin runoff and LOK regulatory discharge are not sufficient.
	  o 0.5 ft minimum depth below which no releases are allowed
	  o 3.8 ft maximum depth above which inflows are discontinued
	  o Assumed inlet pump from STA-3/4 supply canal with capacity equal to combined capacity of G-372 and G-370 structures.
	  o Outflow weirs, with similar discharge characteristics as STA-3/4

Feature	
	outlet structure, discharging into lower North New River canal. o Structure capacities and water quality operating rules are consistent with modeling assumptions assumed during the A-1 FEB EIS application process.
Holey Land Wildlife Management Area	• G-372HL is the only inflow structure for Holey Land used for environmental purposes only • Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FWC and the SFWMD
Rotenberger Wildlife Management Area	• Operational Schedule as defined in the Operation Plan for Rotenberger WMA (SFWMD, March 2010)
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL.
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5N; G508 flows routed to STA5S; G406 flows routed to STA6C139 basin demand is met primarily by local groundwater
Water Shortage Rules	• Reflects the existing water shortage policies as in South Florida Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the eastern and southern boundaries of the RSMBN model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Glades-LECSA Model (RSMGL). The SFWMM was the source of the eastern boundary groundwater/surface water flows, while the RSMGL was the source of the southern boundary structural flows.
- RSMBN EARECB assumptions were adopted from the Central Everglades Planning Project IORBL1 (6/2/2013) scenario with some assumptions returned to the Central Everglades Planning Project ECB (5/25/12) scenario to better reflect 2017 system conditions.



Water-Body Components:
Miami Water-Body = S3 + S8 + A-2W
NNR/HILLS Water-Body = S2 + S6 + S7 + A-2E + New Hope South
WPB Water-Body = S-5A
A1FEB = A-1

Figure A-1. RSMBN Basin Definition within the EAA

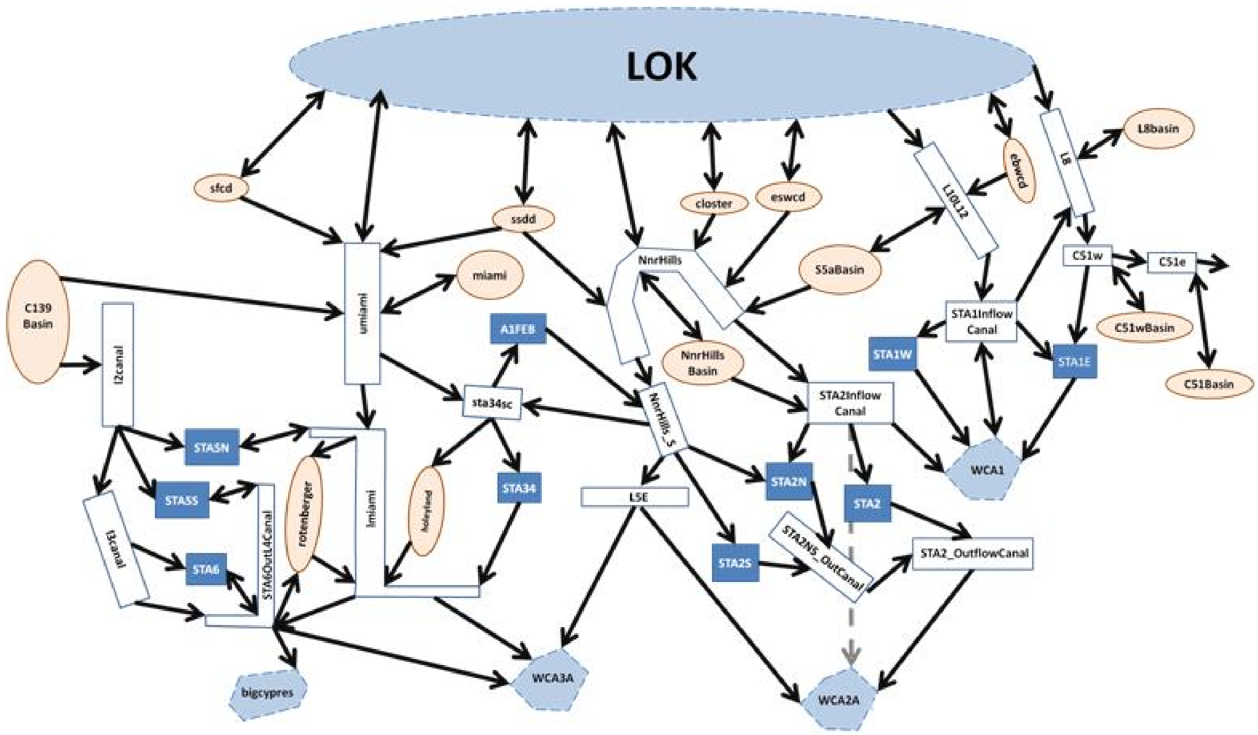


Figure A-2 RSMBN Link-Node Routing Diagram: ECB Baseline Simulation

Modeling Section, H&H Bureau

Regional Simulation Model Basins (RSMBN)
EAA Reservoir Future Without Project Baseline (EARFWO)
Table of Assumptions

Feature	
Climate	• The climatic period of record is from 1965 to 2005. • Rainfall estimates have been revised and updated for 1965-2005. • Revised evapotranspiration methods have been used for 1965-2005.
Topography	The Topography dataset for RSM was Updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004; • High Accuracy Elevation Data, US Geological Survey 2007; • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004; • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007; • Palm Beach County LiDAR Surve, Dewberry and Davis, 2004; and • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/2012, as reflected in the LOSA Ledger produced by the Water Use Bureau. • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information. • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass.
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).
Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008) o CEPP optimized release guidance in order to improve selected performance within LOK, the northern estuaries and LOSA while meeting environmental targets in the Glades. o Lake Okeechobee can send flood releases south through the Miami Canal and North New River Canal to the FEB when the LOK stage is above the bottom of Zone D and the FEB depth is below 2' (EAA basin runoff used to limit conveyance capacity: 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal). o Lake Okeechobee can send flood releases south to help meet water-quality based flow targets at STA-3/4, STA-2N, and STA-2S when the LOK stage is above the bottom of the Baseflow Zone (EAA basin runoff used to limit conveyance capacity: 1,550 cfs for Miami Canal and 1,350 cfs for North New River Canal). o Includes Lake Okeechobee regulatory releases to tide via L8 canal.

Feature	
	o Releases via S-77 can be diverted into C43 Reservoir • Lake Okeechobee Water Shortage Management (LOWSM) Plan. • Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and S-3 is to be minimized). • "Temporary" forward pumps as follows: o S354 – 400 cfs o S351 – 600 cfs o S352 – 400 cfs o All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages recover to greater than 11.2 ft • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Headwaters Revitalization schedule for Kissimmee Chain of Lakes using the UKISS model. • Kissimmee River Restoration complete. • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012. (see land use assumptions row) • Public water supply daily intake from the river is included in the analysis. • Maximum reservoir height of 41.7 ft NGVD with a 9,379-acre footprint in Western C43 basin with a 175,800 acre-feet effective storage. • Proposed reservoir meets estuary demands while C-43 basin supplemental demands for surface water irrigation are met by Lake Okeechobee.
St. Lucie Canal Basin	• St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012 (see land use assumptions row). • Excess C-44 basin runoff is allowed to backflow into the Lake if lake stage is below 14.5 ft before being pumped into the C-44 reservoir. • Basin demands include the Florida Power & Light reservoir at Indiantown. • Indian River Lagoon South Project features • Ten-mile Creek Reservoir and STA: 7,078 acre-feet storage capacity at 10.79 maximum depth on 820 acre footprint; receives excess water from North Folk Basin; • C-44 reservoir: 50,246 acre-feet storage capacity at 5.18 feet maximum depth on 12,125 acre footprint; C44 reservoir releases water back to Lake Okeechobee when Lake stages are below the bottom of the Baseflow Zone. • C-23/C-24 reservoir: 92,094 acre-feet storage capacity at 13.27 maximum depth on 8,675 acre footprint;

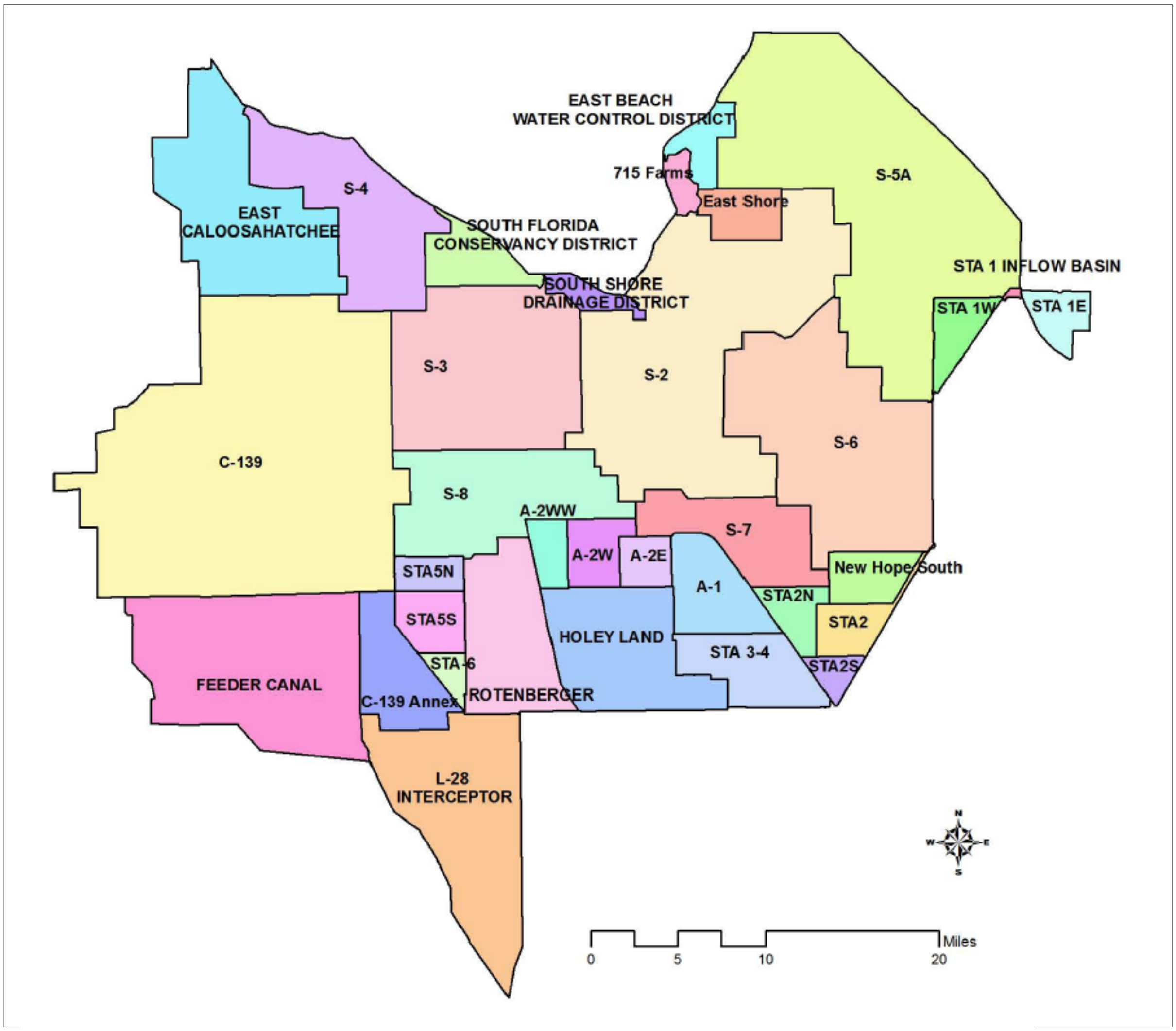
Feature	
	• C-23/C-24 STA: 3,852 acre-feet storage capacity at 1.5 maximum depth on 2,568 acre footprint; • All proposed reservoirs meet estuary demands. • IRL operations assumed are consistent with the March 2010 St. Lucie River Water Reservation Rule update. • Excess C23 basin water not needed to meet estuary demands can be diverted to the C44 reservoir if capacity exists. • C44 reservoir can discharge to C44 canal and backflow to Lake Okeechobee when the lake is below the baseflow zone.
Seminole Brighton Reservation	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons per month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal rights to these quantities are preserved. • LOWSM applies to this agreement.
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM. • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved. • LOWSM applies to this agreement.
Everglades Agricultural Area	• Model water-body components as shown in Figure 1. • Simulated runoff from the North New River – Hillsboro basin apportioned based on the relative size of contributing basins via S7 route vs. S6 route. • G-341 acts as a divide between S-5A Basin and Hillsboro Basin. • RSMBN ECB EAA runoff and irrigation demand compared to SFWMM ECB simulated runoff and demand from 1965-2005 for reasonability.
Everglades Construction Project Stormwater Treatment Areas	• STAs are simulated as single waterbodies • STA-1E: 6,546 acres total area • STA-1W: 7,488 acres total area • S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E • STA-2: cells 1,2 & 3: 7,681 acres total area • STA-2N: cells 4,5 & 6; refers to Comp B-North; 6,531 acres total

Feature	
	area • STA-2S: cells 7 & 8; refers to Comp B-South; 3,570 acres total area • STA-3/4: 17,126 acres total area • STA-5N: includes cells 1 & 2: 5,081 acres total area • STA-5S: includes cells 3, 4 & 5; uses footprint of Compartment C: 8,469 acres total area • STA-6: expanded with phase 2: 3,054 acres total area • Assumed operations of STAs: o 0.5 ft minimum depth below which supply from external sources is triggered; o 4 ft maximum depth above which inflows are discontinued; and o Inflow targets established for STA-3/4, STA-2N and STA-2S based on DMSTA simulation; met from local basin runoff, LOK flood releases and available FEB storage. • A 29,617-acre Flow Equalization Basin (FEB) is located north of STA-3/4 and Holeyland. The total footprint represents the original 15,853-acre A-1 footprint plus the additional 13,764-acre A-2 footprint operated as follows: o Assumed average topography of 9.63 ft NGVD. FEB inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south; o FEB outflows are used to help meet established inflow targets at STA-3/4, STA-2N, and STA-2S if EAA basin runoff and LOK flood releases are not sufficient; o 0.5 ft minimum depth below which no releases are allowed; o 3.8 ft maximum depth above which inflows are discontinued; o No supplemental water supply provided to FEB; o Assumed inlet pump from STA-3/4 supply canal with capacity equal to combined capacity of G-372 and G-370 structures; and o Outflow weirs, with similar discharge characteristics as STA-3/4 outlet structure, discharging into lower Miami and lower North New River canals.
Holey Land Wildlife Management Area	• G-372HL is the only inflow structure for Holey Land used for keeping the water table from going lower than half a foot below land surface elevation. • Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FL Fish and Wildlife Conservation (FWC) Commission and the SFWMD.
Rotenberger Wildlife Management Area	• Operational Schedule as defined in the Operation Plan for Rotenberger WMA. (SFWMD, March 2010)

Feature	
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL FWO.
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5N; G508 flows routed to STA5S; G406 flows routed to STA6. • C139 basin demand is met primarily by local groundwater.
Water Shortage Rules	• Reflects the existing water shortage policies as in South Florida Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan.

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the eastern and southern boundaries of the RSMBN model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Glades-LECSA Model (RSMGL). The SFWMM was the source of the eastern boundary groundwater/surface water flows, while the RSMGL was the source of the southern boundary structural flows.
- The RSMBN EARFWO assumptions were adopted from the Central Everglades Planning Project ALT4R2 (2/19/2013) scenario.



Water-Body Components:
Miami Water-Body = S3 + S8 + A-2WW
NNR/HILLS Water-Body = S2 + S6 + S7 + New Hope South
WPB Water-Body = S-5A
FEB = A-2W + A-2E + A-1

Figure A-3 RSMBN Basin Definition within the EAA: Future Without Project Baseline (EARFWO)

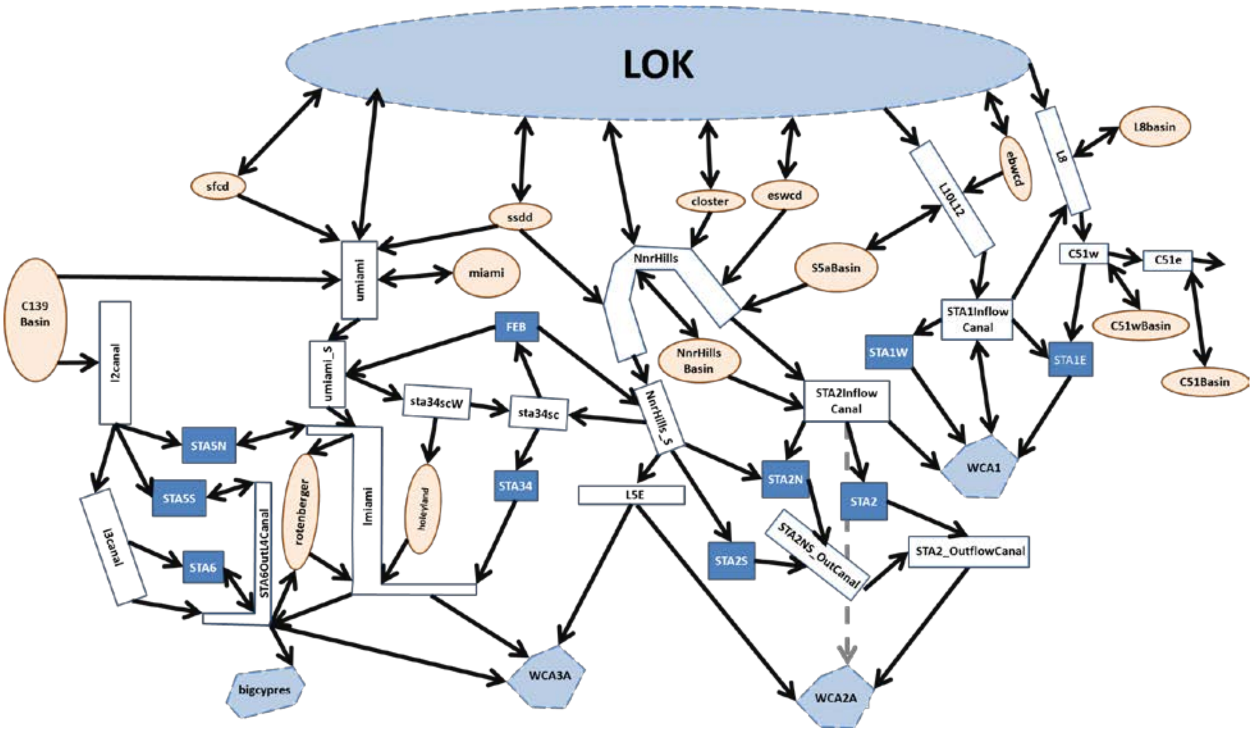


Figure A-4 RSMBN Link-Node Routing Diagram: EARFWO

Modeling Section, H&H Bureau
South Florida Water Management District

Regional Simulation Model Glades-LECSA (RSMGL)
EAA Reservoir Existing Conditions Baseline (EARECB)
Table of Assumptions

Feature	
Meteorological Data	• Rainfall file used: rain_v3.0_beta_tin_14_05.bin • Reference Evapotranspiration (RET) file used: RET_48_05_MULTIQUAD_v1.0.bin (ARCADIS, 2008)
Topography	• Same as calibration topographic data set except where reservoirs are introduced (STA1-E, C4 Impoundment and C-111 reservoirs). • United States Geological Survey (USGS) High-Accuracy Elevation Data Collection (HAEDC) for the Water Conservation Areas (1, 2A, 2B, 3A, and 3B), the Big Cypress National Preserve and Everglades National Park.
Tidal Data	• Tidal data from two primary (Naples and Virginia Key) and five secondary NOAA stations (Flamingo, Everglades, Palm Beach, Delray Beach and Hollywood Beach) were used to generate a historic record to be used as sea level boundary conditions for the entire simulation period.
Land Use and Land Cover	• Land Use and Land Cover Classification for the Lower East Coast urban areas (east of the Lower East Coast Flood Protection Levee) use 2008-2009 Land Use coverage as prepared by the SFWMD, consumptive use permits as of 2011 were used to update the land use in areas where it did not reflect the permit information. • Land Use and Land Cover Classification for the natural areas (west of the Lower East Coast Flood Protection Levee) is the same as the Calibration Land Use and Land Cover Classification for that area. • Modified at locations where reservoirs are introduced (STA1-E, C4 Impoundment, Lakebelt Lakes and C-111 Reservoirs).
Water Control Districts (WCDs)	• Water Control Districts in Palm Beach and Broward Counties and in the Western Basins assumed.
Lake Belt Lakes	• Based on 2005 Lake Belt Lake coverage obtained from USACE.
Water Conservation Area 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge)	• Current C&SF Regulation Schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 14 ft. The bottom floor of the schedule (Zone C) is the area below 14 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow. • Structure S10E connecting LNWR to the northeastern portion of WCA-2A is no longer considered part of the simulated regional System

Feature	
Water Conservation Area 2A & 2B	• Current C&SF regulation schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels in WCA-2A are less than minimum operating criteria of 10.5 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Water Conservation Area 3A & 3B	• Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 (USACE, 2012). • Includes regulatory releases to tide through LEC canals. Documented in Water Control Plan (USACE, June 2006) • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 7.5 ft in WCA-3A. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Everglades Construction Project Stormwater Treatment Areas	• STA-1E: 5,132 acres total treatment area. • A uniform bottom elevation equal to the spatial average over the extent of STA-1E is assumed.
Everglades National Park	• Water deliveries to Everglades National Park are based upon Everglades Restoration Transition Plan (ERTP), with the WCA-3A Regulation Schedule including the lowered Zone A (compared to IOP) and extended Zones D and E1. • L-29 stage constraint for operation of S-333 assumed to be 7.5 ft, NGVD. • G-3273 constraint for operation of S-333 assumed to be 6.8 ft, NGVD. • Tamiami Trail culverts east of the L67 Extension are simulated. • 5.5 miles remain of the L-67 Extension Levee. • S-355A & S-355B are operated. • S-356 is not operated. • Partial construction of C-111 project reservoirs consistent with the 2009 as-built information from USACE (does not include contract 8 or contract 9). A uniform bottom elevation equal to the spatial average over the extent of each reservoir is assumed. • S-332DX1 is not operated. • 8.5 SMA project feature as per federally authorized Alternative 6D of the MWD/8.5 SMA Project (USACE, 2000 GRR); operations per 2011 Interim Operating Criteria (USACE, June 2011) including S-331 trigger shifted from Angel's well to LPG-2.
Other Natural Areas	• Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay

Feature	
Pumpage and Irrigation	• Public Water Supply pumpage for the Lower East Coast was updated using 2010 consumptive use permit information as documented in the C-51 Reservoir Feasibility Study; permits under 0.1 MGD were not included • Residential Self Supported (RSS) pumpage are based on 2030 projections from the SFWMD Water Supply Bureau. • Industrial pumpage are based on 2030 projections from the SFWMD Water Supply Bureau. • Irrigation demands for the six irrigation land-use types are calculated internally by the model. • Seminole Hollywood Reservation demands are set forth under VI. C of the Tribal Rights Compact. Tribal sources of water supply include various bulk sale agreements with municipal service suppliers.

Canal Operations	• C&SF system and operating rules in effect in 2012
	• Includes operations to meet control elevations in the primary coastal canals for the prevention of saltwater intrusion
	• Includes existing secondary drainage/water supply system
	• C-4 Flood Mitigation Project
	• Western C-4, S-380 structure retained open
	• C-11 Water Quality Treatment Critical Project (S-381 and S-9A).
	  o S9/S9A operations modified for performance consistency with SFWMM ECB.
	• S-25B and S-26 pumps are not modeled since they are used very rarely during high tide conditions and the model uses a long-term average daily tidal boundary
	• Northwest Dade Lake Belt area assumes that the conditions caused by currently permitted mining exist and that the effects of any future mining are fully mitigated by industry
	• ACME Basin A flood control discharges are sent to C-51, west of the S-155A structure, to be pumped into STA-1E. ACME Basin B flood control discharges are sent to STA-1E through the S-319 structure
	• Releases from WCA-3A to ENP and the South Dade Conveyance System (SDCS) will follow the Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1
	  o Structures S-343A, S-343B, S-344 and S-12A are closed Nov. 1 to July 15
	  o Structure S-12B is closed Jan. 1 to July 15
	• South Dade Conveyance System operations will follow ERTP for protection of the Cape Sable seaside sparrow

Canal Configuration	• Canal configuration same as calibration except only 5.5 miles remain of the L-67 Extension Canal.

Feature	
Lower East Coast Service Area Water Shortage Management	• Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data. • Periods where the Lower East Coast is under water restriction due to low Lake Okeechobee stages were extracted from the corresponding RSMBN ECB simulation.

Notes

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.
- RSMGL EARECB assumptions were adopted from the Central Everglades Planning Project 2012EC (2/28/2013) scenario.

Modeling Section, H&H Bureau

Regional Simulation Model Glades-LECSA (RSMGL)
EAA Reservoir Future Without Project Baseline (EARFWO)
Table of Assumptions

Feature	
Meteorological Data	• Rainfall file used: rain_v3.0_beta_tin_14_05.bin • Reference Evapotranspiration (RET) file used: RET_48_05_MULTIQUAD_v1.0.bin (ARCADIS, 2008)
Topography	• Same as calibration topographic data set except where reservoirs are introduced (STA1-E, C4 Impoundment and C-111 reservoirs). • United States Geological Survey (USGS) High-Accuracy Elevation Data Collection (HAEDC) for the Water Conservation Areas (1, 2A, 2B, 3A, and 3B), the Big Cypress National Preserve and Everglades National Park.
Tidal Data	• Tidal data from two primary (Naples and Virginia Key) and five secondary NOAA stations (Flamingo, Everglades, Palm Beach, Delray Beach and Hollywood Beach) were used to generate a historic record to be used as sea level boundary conditions for the entire simulation period.
Land Use and Land Cover	• Land Use and Land Cover Classification for the Lower East Coast urban areas (east of the Lower East Coast Flood Protection Levee) use 2008-2009 Land Use coverage as prepared by the SFWMD, consumptive use permits as of 2011 were used to update the land use in areas where it did not reflect the permit information. • Land Use and Land Cover Classification for the natural areas (west of the Lower East Coast Flood Protection Levee) is the same as the Calibration Land Use and Land Cover Classification for that area. Modified at locations where reservoirs are introduced (STA1-E, Site 1 Impoundment, Broward WPAs, C4 Impoundment, Lakebelt Lakes and C-111 Reservoirs).
Water Control Districts (WCDs)	• Water Control Districts in Palm Beach and Broward Counties and in the Western Basins assumed. • 8.5 SMA seepage canal is modeled as a WCD in ENP area.
Lake Belt Lakes	• Based on the permitted 2020 Lake Belt Lakes coverage obtained from USACE.
CERP Projects	• 1st Generation CERP – Site 1 Impoundment project is modeled as an above ground reservoir of area 1600 acres, with a maximum depth of 8 ft. • 2nd Generation CERP – Broward County Water Preserve Areas (WPAs) comprised of C-11 and C-9 impoundments were modeled as above ground reservoirs with areas 1221 and 1971 acres and maximum depths 4.3 and 4.0 ft. respectively. Operations refined in RSM model to closer represent project intent and outcomes. • 2nd Generation CERP – C-111 Spreader Canal Project includes the Frog Pond Detention Area, which is modeled as an above ground impoundment with the S200 A, B and C pumps as inflow

Feature	
	structures. In addition, the Aerojet canal is modeled with the inflow pumps S199 A, B and C. The S199 and S200 pumps are turned off based on the stage at the remote monitoring location EVER4 for the protection of the CSS Critical Habitat Unit 3. • 2nd Generation CERP – Biscayne Bay Coastal Wetlands project features were not modeled since these features along the coast in Miami-Dade County were not considered significant for CEPP. • Areal corrections were applied to the impoundment storages to account for the discrepancies of the areas in the model of the impoundments not matching the design areas.
Water Conservation Area 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge)	• Current C&SF Regulation Schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 14 ft. The bottom floor of the schedule (Zone C) is the area below 14 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow. • Structure S10E connecting LNWR to the northeastern portion of WCA-2A is no longer considered part of the simulated regional System
Water Conservation Area 2A & 2B	• Current C&SF regulation schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 10.5 ft in WCA-2A, defined as when WCA2-U1 marsh gauge falls below 10.5 ft or L38 canal stage falls below 10.0 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Water Conservation Area 3A & 3B	• Diversion of L-6 flows with additional 500 cfs structure and improvements to the L-5 canal • STA-3/4 outflows routed based on Rainfall Driven Operations (RDO) – a maximum of 2500 cfs is routed to S8 and G404, with the remainder being sent to S7 • Western L-4 levee degrade with 1.5 miles retained west of S8 (west of S-8 = 3,000 cfs capacity) • Miami Canal backfilled and spoil mound removed 1.5 miles south of S-8 to I-75 • Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 (USACE, 2012) • One 500 cfs gated structure in L-67A north of Blue Shanty levee (S345D) and associated gap in L-67C levee • Two 500 cfs gated structures in L-67A (S345F & S345G) discharging into Blue Shanty Flowway • Environmental target deliveries through the S345s are determined through RDO and is spatially distributed as 40% to 345D, 35% to 345F and 25% to 345G • Blue Shanty Flowway assumed as follows:

Feature	
	o Construction of ~8.5 mile levee in WCA 3B, connecting L-67A to L-29 o Removal of L-67C levee in Blue Shanty Flowway (no canal back fill) o Removal of L-29 levee in Blue Shanty Flowway. • Includes regulatory releases to tide through LEC canals. Documented in Water Control Plan (USACE, June 2002) • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 7.5 ft in WCA-3A, defined as when 3-69W marsh gauge falls below 7.5 ft or CA3 canal stage falls below 7.0 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Everglades Construction Project Stormwater Treatment Areas	• STA-1E: 5,132 acres total treatment area. • A uniform bottom elevation equal to the spatial average over the extent of STA-1E is assumed.
Everglades National Park	• Water deliveries to Everglades National Park are based upon Everglades Restoration Transition Plan (ERTP), with the WCA-3A Regulation Schedule including the lowered Zone A (compared to IOP) and extended Zones D and E1. The environmental component of the schedule is defined by RDO. If hydraulic capacity exists at the 345s, then flood control discharges are made into 3B instead of at the S12s. • S-333 capacity increased to 2,500 cfs • L29 Divide structure assumed and is operated to send water from L29W to L29E to equilibrate canals when L29E falls below 7 ft. • L29 canal can receive inflow up to 9.7 ft (applies to both E and W segments / i.e. S333 & S356 as well as S345F & S345G structure on Blue Shanty Flowway) • G-3273 constraint for operation of S-333 assumed to be 9.5 ft, NGVD. • The one mile Tamiami Trail Bridge as per the 2008 Tamiami Trail Limited Reevaluation Report is modeled as a one mile weir. Located east of the L67 extension and west of the S334 structure. • Western 2.6 mile Tamiami Trail Bridge, modeled as a 2.6 mile long weir, and is located east of Osceola Camp and west of Frog City. • Tamiami Trail culverts east of the L67 Extension are simulated where the bridge is not located. • Removal of the entire 5.5 miles L-67 Extension levee, with backfill of L-67 Extension canal • S-355A & S-355B are operated. • Capacity of S-356 pump increased to 1000 cfs. S-356 is operated to manage seepage. • Full construction of C-111 project reservoirs consistent with the as-built information from USACE plus addition of contract 8 and

Feature	
	contract 9 features. A uniform bottom elevation equal to the spatial average over the extent of each reservoir is assumed. • 8.5 SMA project feature as per federally authorized Alternative 6D of the MWD/8.5 SMA Project (USACE, 2000 GRR); operations per 2011 Interim Operating Criteria (USACE, June 2011) including S-331 trigger shifted from Angel's well to LPG-2. Outflow assumed from 8.5 SMA detention cell to the C-111 North Detention Area. o An additional length of seepage canal is assumed in the model to allow water to be collected for S357 operation. • Partial depth, approximately 4 mile long seepage barrier south of Tamiami Trail (along L-31N)
Other Natural Areas	• Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay
Pumpage and Irrigation	• Public Water Supply pumpage for the Lower East Coast was updated using 2010 consumptive use permit information as documented in the C-51 Reservoir Feasibility Study; permits under 0.1 MGD were not included • Modeling of the TSP assumes an additional public water supply withdrawal of 12 MGD in Service Area 2 and 5 MGD in Service Area 3. • Residential Self Supported (RSS) pumpage are based on 2030 projections of residential population from the SFWMD Water Supply Bureau. • Industrial pumpage is also based on 2030 projections of industrial use from the Water Supply Bureau. • Irrigation demands for the six irrigation land-use types are calculated internally by the model. • Seminole Hollywood Reservation demands are set forth under VI. C of the Tribal Rights Compact. Tribal sources of water supply include various bulk sale agreements with municipal service suppliers.
Canal Operations	• C&SF system and operating rules in effect in 2012 • Includes operations to meet control elevations in the primary coastal canals for the prevention of saltwater intrusion • Includes existing secondary drainage/water supply system • C-4 Flood Mitigation Project • Western C-4, S-380 structure retained open • C-11 Water Quality Treatment Critical Project (S-381 and S-9A) • S-25B and S-26 backflow pumps are not modeled since they are used very rarely during high tide conditions and the model uses a long-term average daily tidal boundary • Northwest Dade Lake Belt area assumes that the conditions caused by currently permitted mining exist and that the effects of any future mining are fully mitigated by industry • ACME Basin A flood control discharges are sent to C-51, west of the S-155A structure, to be pumped into STA-1E. ACME Basin B flood control discharges are sent to STA-1E through the S-319 structure • Releases from WCA-3A to ENP and the South Dade Conveyance System (SDCS) will follow the Everglades Restoration Transition

Feature	
	Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 o Structures S-343A, S-343B, S-344 and S-12A are closed Nov. 1 to July 15 o Structure S-12B is closed Jan. 1 to July 15 • Water supply deliveries from regional system (from WCA3A: S-151/S-337) are used to maintain the L30 canal with a minimum seasonal level varying from 6.25 ft in the dry season to 5.2 ft. at the beginning of the wet season • G-211 / S338 operational refinements; use coastal canals to convey seepage toward Biscayne Bay during drier times.
Canal Configuration	• Canal configuration same as calibration except no L-67 Extension Canal and CERP & CEPP project modifications.
Lower East Coast Service Area Water Shortage Management	• Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data. • Periods where the Lower East Coast is under water restriction due to low Lake Okeechobee stages were extracted from the corresponding RSMBN FWO simulation.

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.
- RSMGL EARFWO assumptions were adopted from the Central Everglades Planning Project ALT4R2 (6/25/2013) scenario.

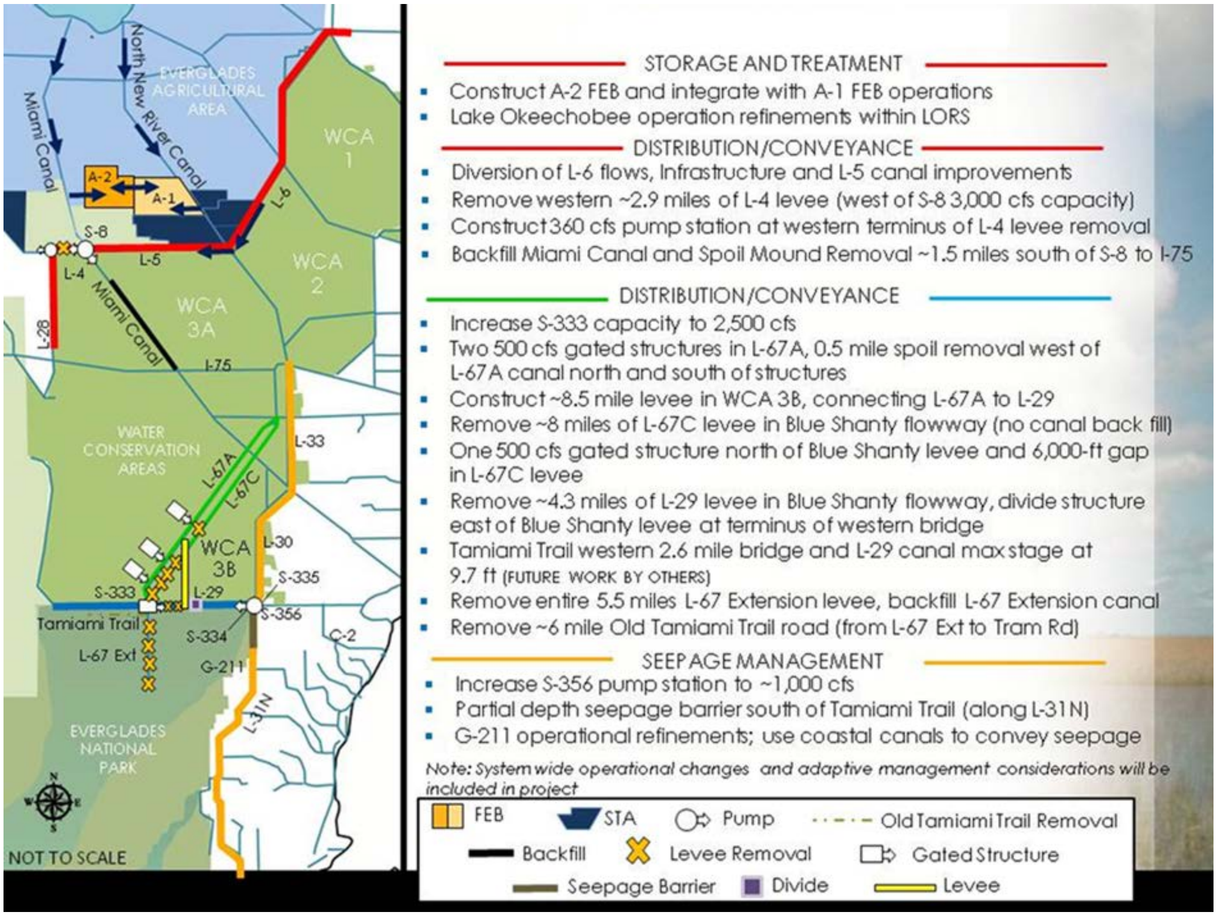


Figure A-5. EARFWO Features in RSMGL

Appendix B – LORS08 Operations Schedule

The LORS08 schedule is used for operation of Lake Okeechobee in the ECB baseline. **Figure B-1** shows the LORS08 schedule. The LORS08 schedule used for operation of Lake Okeechobee in the ECB baseline was modified as shown in Figure **B-1.1** for use in the FWO baseline. **Figures B-2 and B-3** show the pulse releases from Lake Okeechobee into the Caloosahatchee and St. Lucie Estuary, respectively.

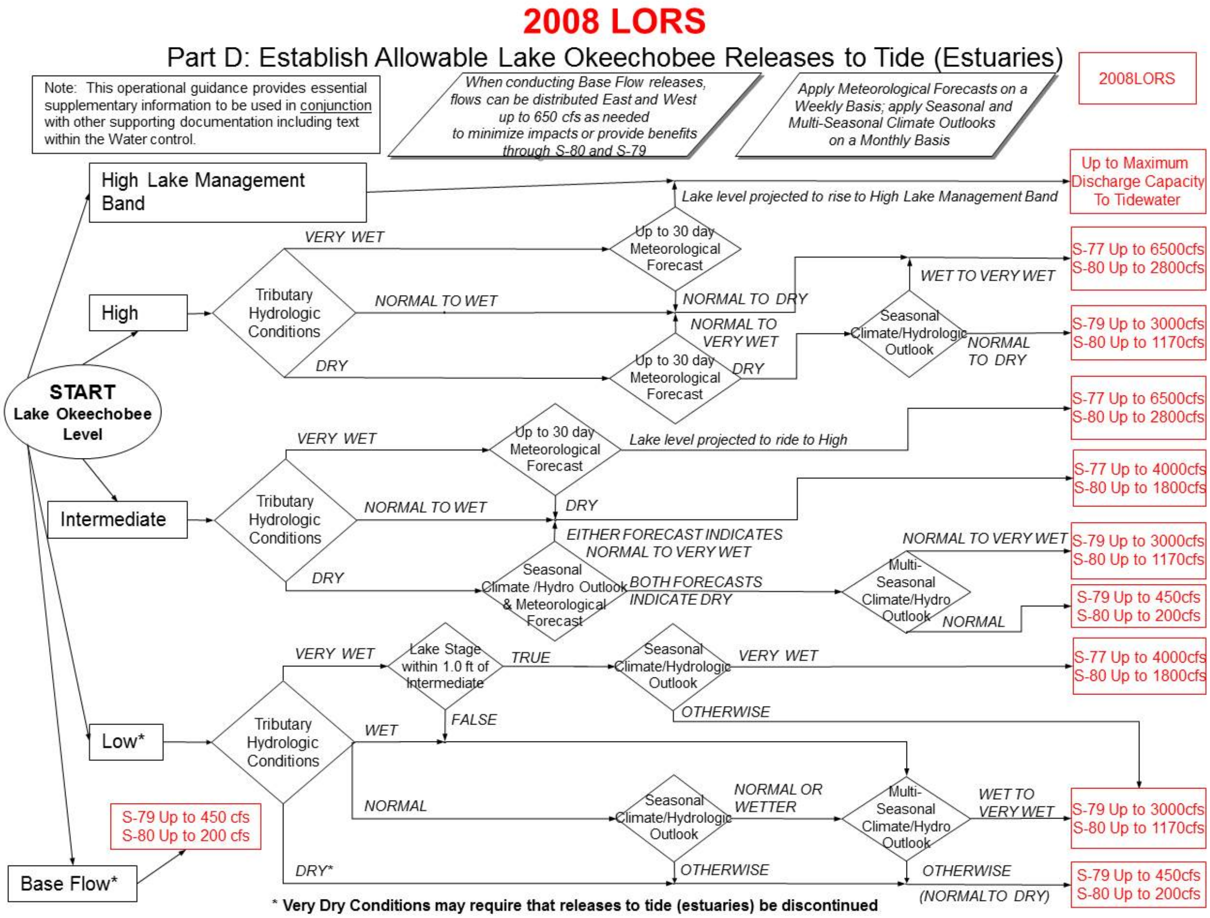


Figure B-1. LORS08 operations schedule.

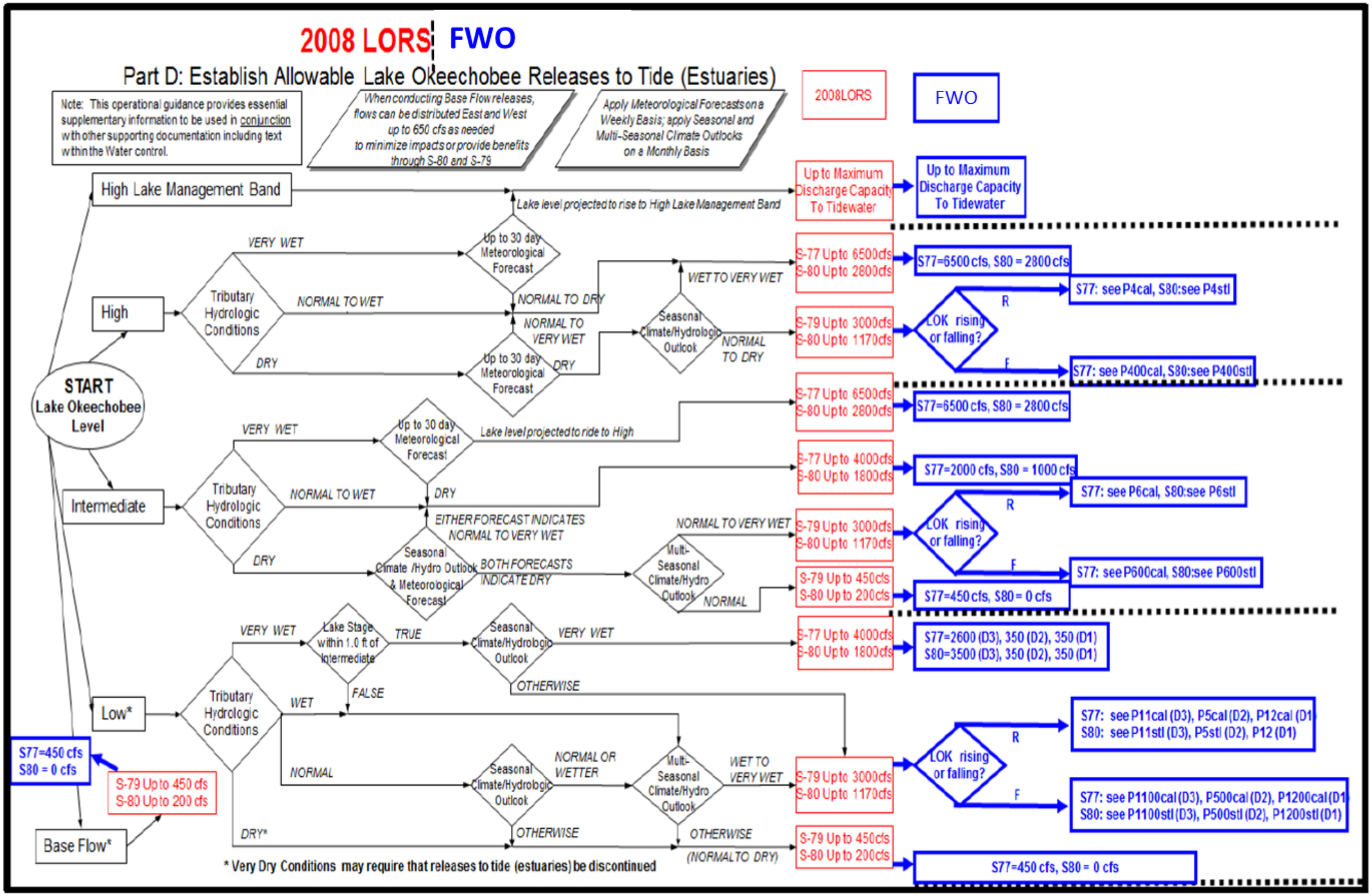


Figure B.1.1. LORS08 operations schedule, with FWO modifications as modeled shown in blue.

	High		Intermediate		Low					
zone	A	A	B	B	D3	D3	D2	D2	D1	D1
	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling
Day of Pulse	P4cal	P400cal	P6cal	P600cal	P11cal	P1100cal	P5cal	P500cal	P12cal	P1200cal
1	500	125	840	504	125	125	850	850	504	504
2	1,375	343.75	1,920	1,152	344	344	2,200	2,200	1,152	1,152
3	1,625	406.25	2,220	1,332	406	406	2,600	2,600	1,332	1,332
4	1,250	312.50	1,680	1,008	313.50	313.50	2,000	2,000	1,008	1,008
5	1,000	250	1,440	864	250	250	1,600	1,600	864	864
6	750	187.50	1,140	684	188.50	188.50	1,200	1,200	684	684
7	500	125	960	576	125	125	850	850	576	576
8	250	62.50	720	432	62.50	62.50	500	500	432	432
9	125	31.25	540	324	31.25	31.25	350	350	324	324
10	125	31.25	540	324	31.25	31.25	350	350	324	324
Average Flow (cfs)	**750**	**187.5**	**1200**	**720**	**187.7**	**187.7**	**1250**	**1250**	**720**	**720**
Volume (ac-ft)	**14,873**	**3,718**	**23,796**	**14,278**	**3,722**	**3,722**	**24,788**	**24,788**	**14,278**	**14,278**
†equivalent depth (ft)	**0.03**	**0.01**	**0.05**	**0.03**	**0.01**	**0.01**	**0.05**	**0.05**	**0.03**	**0.03**
† : Volume to depth Conversion based on average Lake surface area of 467,000 acres.										

Figure B-2. Baseline pulse releases (as a function of lake level) from Lake Okeechobee into Caloosahatchee Estuary in cubic feet per second (cfs).

	High		Intermediate		Low					
zone	A	A	B	B	D3	D3	D2	D2	D1	D1
	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling
Day of Pulse	P4cal	P400cal	P6cal	P600cal	P11cal	P1100cal	P5cal	P500cal	P12cal	P1200cal
1	500	125	840	504	125	125	850	850	504	504
2	1,375	343.75	1,920	1,152	344	344	2,200	2,200	1,152	1,152
3	1,625	406.25	2,220	1,332	406	406	2,600	2,600	1,332	1,332
4	1,250	312.50	1,680	1,008	313.50	313.50	2,000	2,000	1,008	1,008
5	1,000	250	1,440	864	250	250	1,600	1,600	864	864
6	750	187.50	1,140	684	188.50	188.50	1,200	1,200	684	684
7	500	125	960	576	125	125	850	850	576	576
8	250	62.50	720	432	62.50	62.50	500	500	432	432
9	125	31.25	540	324	31.25	31.25	350	350	324	324
10	125	31.25	540	324	31.25	31.25	350	350	324	324
Average Flow (cfs)	**750**	**187.5**	**1200**	**720**	**187.7**	**187.7**	**1250**	**1250**	**720**	**720**
Volume (ac-ft)	**14,873**	**3,718**	**23,796**	**14,278**	**3,722**	**3,722**	**24,788**	**24,788**	**14,278**	**14,278**
†equivalent depth (ft)	**0.03**	**0.01**	**0.05**	**0.03**	**0.01**	**0.01**	**0.05**	**0.05**	**0.03**	**0.03**
† : Volume to depth Conversion based on average Lake surface area of 467,000 acres.										

Figure B-3. Baseline pulse releases (as a function of lake level) from Lake Okeechobee into St. Lucie Estuary in cubic feet per second (cfs).

Details of the model implementation of the schedule can be found in **Figures B-4, B-5 and B-6.** **Figure B-4** lists the range of values used to classify the tributary hydrologic conditions. **Figure B-5** lists the range of values used to classify the net inflow seasonal outlook. **Figure B-6** lists the range of values used to classify the net inflow multi-seasonal outlook.

LORS2008

Classification of Lake Okeechobee Tributary Hydrologic Conditions

Palmer Index Class Limits	2-wk Mean LO Inflow Class Limit	Tributary Hydrologic Classification*
> 3.0	>= 6000 cfs	Very Wet
1.5 to 2.99	2500 - 5999 cfs	Wet
-1.49 to 1.49	500 - 2499 cfs	Near Normal
-2.99 to -1.5	-5000 - 500 cfs	Dry
-3.0 or less	< -5000 cfs	Very Dry

*use the wettest of the two indicators

FWO-RSMBN

Classification of Lake Okeechobee Tributary Hydrologic Conditions

Palmer Index Class Limits	2-wk Mean LO Inflow Class Limit	Tributary Hydrologic Classification*
> 3.3	>= 8700 cfs	Very Wet
0.01 to 3.29	1000 - 8699 cfs	Wet
-0.49 to 0.0	500 - 999 cfs	Near Normal
-2.99 to -0.50	-5000 - 500 cfs	Dry
-3.0 or less	< -5000 cfs	Very Dry

*use the wettest of the two indicators

Figure B-4. LORS08 and FWO Tributary Hydrologic Conditions Classifications

LORS2008

Classification of Lake Okeechobee Net Inflow Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does not include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Seasonal Outlook
> 0.93	> 2.0	Very Wet
0.71 to 0.93	1.51 to 2.0	Wet
0.35 to 0.70	0.75 to 1.5	Normal
< 0.35	< 0.75	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

FWO-RSMBN

Classification of Lake Okeechobee Net Inflow Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Seasonal Outlook
> 1.43	> 3.06	Very Wet
0.91 to 1.43	1.93 to 3.06	Wet
0.46 to 0.90	0.98 to 1.92	Normal
< 0.46	< 0.98	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

Figure B-5. LORS08 and FWO Seasonal Outlook Classifications

LORS2008		
Classification of Lake Okeechobee Net Inflow Multi-Seasonal Outlook**		
Lake Net Inflow Prediction (million acre-feet) (Does not include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Multi-Seasonal Outlook
> 2.0	> 4.3	Very Wet
1.18 to 2.0	2.51 to 4.3	Wet
0.5 to 1.17	1.1 to 2.5	Normal
< 0.5	< 1.1	Dry
**volume-depth conversion based on average lake surface area of 467,000 acres.		

FWO-RSMBN		
Classification of Lake Okeechobee Net Inflow Multi-Seasonal Outlook**		
Lake Net Inflow Prediction (million acre-feet) (Does include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Multi-Seasonal Outlook
> 2.0	> 4.3	Very Wet
1.18 to 2.0	2.51 to 4.3	Wet
0.6 to 1.17	1.3 to 2.5	Normal
< 0.6	< 1.3	Dry
**volume-depth conversion based on average lake surface area of 467,000 acres.		

Figure B-6. LORS08 and FWO Multi-Seasonal Outlook Classifications

Appendix C – Structure Operations in South Miami-Dade County for EAASR FWO Baseline, Final Array Runs, and TSP

In **Table C.1**, the list of structures is color-coded in three groupings:

green	Structures on the L-67, L-28, and L-29 canals. Structures included are S-345, S-349, S-344, S-343A-B, S-12A-D, S-333, S-334, S-355, and S-356
blue	Structures on the L-30 and part of the L-31 canals Structures included are S-337, S-151, S-335, S-338, G-211, S-173 & S-331P (COMBQ), S-176 and S-174
yellow	Structures on part of the L-31 canal and L-31W and C-111 canals Structures included are S-332A-D, S-357, S-332, S-175, S-200, S-199, S-177, S-18C, S-197 and S-332E

Table C.1 includes the Future Without Baseline (FWO), and the final array alternatives and TSP (same operations for alternatives ALT R240, ALT R360, and ALT C360 and for TSP ALT C240).

Table C.1. Existing Condition (EARECB) and Future Condition (EARFWO, ALT R240, ALT R360, and ALT C360 and TSP ALT C240)

Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations
L-67	S-345*	Non-existent	S345D, S345F & S345G 3 gated spillway at L-67A Design Q= 500 cfs each flood control only
	S-349*	Non-existent	Non-existent
L-28	S-344*	Special code Design Q=250 cfs 1) Closed Nov 1- Jul 15 2)flood control only	Special code Design Q=250 cfs 1) Closed Nov 1- Jul 15 2)flood control only
	S-343A-B*	Special code 1) Closed Nov 1- Jul15 2) flood control only 3)S343A&B- Design Q= 200 cfs each	Special code 1) Closed Nov 1- Jul15 2) flood control only 3)S343A&B- Design Q= 200 cfs each
L-29	S-12A-D*	per ERTP S12A closed Nov 1 to Jul 31; S12B closed Jan 1 to Jul 31; S12C no closure dates. S12D no closure dates. Special code 1) S12s = 8000 cfs per structure. 2) Each structure modeled individually. 3) Each Structure is a spillway 4) Flood Control only	per ERTP S12A closed Nov 1 to Jul 31; S12B closed Jan 1 to Jul 31; S12C no closure dates. S12D no closure dates. Special code 1) S12s = 8000 cfs per structure. 2) Each structure modeled individually. 3) Each Structure is a spillway 4) Flood Control only
	S-333*	Special code 1) L-29 stage constraint of 7.5 Wet/Dry 2) Design Q=1350 cfs 3) G-3273 stage constraint of 6.8 4) Flood Control only	Special code (S333 has higher priority over S12s) 1) L-29 canal max stage of 9.7 Wet/Dry 2) Design Q=2500 cfs 3) G-3273 stage constraint of 9.5 4) Flood Control only
	S-334	Non-existent Special code 1) Flood Control 2) No open/close ops, structure flow is based on L31N stage 4) Design Q=1230 cfs	Non-existent No IOP wraparound operations. So, no flow through S334
	S-355*	Special code 1) S355 A and B Modeled Design Q = 1000 cfs each, 2) L-29 Max stage of 7.5' 3) Flood control only 4)G-3273 stage constraint of 6.8 5)L-29 stage constraint of 7.5	Special code 1) S355 A and B Modeled Design Q = 1000 cfs each, 2) L-29 Max stage of 9.7 3) Flood control only 4)G-3273 stage constraint of 9.5
	S-356*	Not operational	**6.0 / 5.5** open/closed wet season **6.0 / 5.8** open/closed dry season 1) Design Q = 1000 cfs 2) Flood control only

<table>
<tr><th rowspan="2">Canal</th><th rowspan="2">Structure</th><th>EARECB (RSMGL)</th><th>EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)</th></tr>
<tr><th>Open/Close (ft NGVD)
(Optimum stage ft NGVD)
Wet /Dry Season Normal FC Operations</th><th>Open/Close (ft NGVD)
(Optimum stage ft NGVD)
Wet /Dry Season Normal FC Operations</th></tr>
<tr><td rowspan="4">L-30</td><td rowspan="2">S337</td><td>1) Water Supply / Flood Control
2) Design Q=1100 cfs (discharge coef = 1053 cfs in msestruc*.xml)</td><td>1) Water Supply / Flood Control
2) Design Q=1100 cfs (discharge coef = 1053 cfs in msestruc*.xml)</td></tr>
<tr><td>mse_unit inlet L30; L30 localLevel=7.0
mse_unit outlet C-304; C304 localLevel=99.0

S337_HWi & S337TWi (input variable for WCA3A_WCA3B_regulatory in Special_Assessors_2050FWO.so)

S337_FracGO & S337_FracGo_high & S337_FracGo_low variable output maxfracS12s xml</td><td>mse_unit inlet L30; L30 maintLevel from rc id=993110
maintLevel(01 Nov-31 May=6.45. 01 Jun-31 Oct =5.4)
L30 resLevel from rc id = 993111
resLevel(01 Nov-31 May=6.25. 01 Jun-31 Oct=5.2)
mse_unit outlet C-304; C304 localLevel=99.0

S337_HWi & S337TWi (input variable for WCA3A_WCA3B_regulatory in Special_Assessors_ALT4R.so)

S337_FracGO & S337_FracGo_high & S337_FracGo_low variable output maxfracS12s xml</td></tr>
<tr><td>S-151*</td><td>Flow target based on WCA-3A regulation schedule

1) Water Supply / Flood Control
2) Design Q=1800 cfs (discharge coef. = 1154.48 in msestruc*.xml)

mse_unit outlet for WCA-3A; "WCA3A local" localLevel = 7.5
mse_unit inlet C-304; C304 localLevel=99.0

Special Code for S151:
S151_reg_max_zoneA=1000 (input variable Special_Assessors_2050FWO.so)
S151_reg_max_zoneBC=500 (input variable Special_Assessors_2050FWO.so)
S151_TWi & S151_HWi (input variable Special_Assessors_2050FWO.so)
S151_FracGO & S151_FracGo_high & S151_FracGo_low variable output maxfracS12s xml</td><td>Flow target based on WCA-3A regulation schedule</td></tr>
<tr><td>S-335</td><td>7.5 / 7.2 open/closed wet & dry season
1) Water Supply / Flood Control
2) Design Q=1170 (discharge coef. = 1468 cfs in msestruc*.xml)
3) twHeadLimit name "S335 twHeadLimit" = 6.0
mse_unit outlet for L30; L30 localLevel=7.0
mse_unit inlet for L31NC; localLevel=99.0</td><td>7.6 / 7.4 open/closed wet &dry season
1) Water Supply / Flood Control
2) Design Q=1170 (discharge coef. = 1468 cfs in msestruc*.xml)
3) twHeadLimit name "S335 twHeadLimit" = 6.0
mse_unit outlet for L30; L30 maintLevel from rc ID 993110
maintLevel(01 Nov-31 May=6.45. 01 Jun-31 Oct=5.4)
L30 resLevel from rc ID=993111
resLevel(01 Nov-31 May=6.25. 01 Jun-31 Oct=5.2)
mse_unit inlet for L31NC; localLevel=99.0</td></tr>
</table>

Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations
L-31N	S-338	**5.8 / 5.5** open/closed wet & dry season 1) Water Supply / Flood Control 2) Design Q=305 (discharge coef. = 393 cfs in msestruc*xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for C1; C1 maintLevel=3.0 mse_unit inlet for L31NC; localLevel=99.0	**5.8 / 5.5** open/closed wet season **5.7 / 5.5** open/closed dry season 1) Water Supply / Flood Control 2) Design Q=305 (discharge coef. = 393 cfs in msestruc*xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for C1; C1 maintLevel=3.0
	G-211	**6.0 / 5.5** open/closed wet & dry season twHeadLimit name "G211 twHeadLimit" = 5.3 1) Water Supply / Flood Control 2) Design Q=1100 (discharge coef. = 943 cfs in msestruc*.xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for L31N; L31N localLevel=99.0 no rulecurve	**6.0 / 5.7** open/closed wet season **5.8 / 5.5** open/closed dry season twHeadLimit name "G211 twHeadLimit" = 5.3 1) Water Supply / Flood Control 2) Design Q=1100 (discharge coef. = 943 cfs in msestruc*.xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for L31N; L31N localLevel=99.0
	S-173 & S-331P (COMBQ) *	1) Water Supply / Flood Control 2) Design Q=1161 (special code Special_Assessors_ECB_2010-11.so) mse_unit outlet for L31N; localLevel = 99.0 mse_unit inlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit inlet for L31S; "L31S res" resLevel = 3.5 mse_unit outlet for L31N; mse_unit "L31N local" localLevel = 99.0 S331_TW_lim = 6.0 Operations defined in S331_ECB_2010-11.cc: S331_HW_levels = 4.0 4.5 5.0 5.5 (LPG2 stage criteria) S331 OPERATING CRITERIA: “Discharges through S-331 can be made if the S-331 tailwater stage is below 6.0 feet and the S-176 headwater stage is below 5.5 feet. If either of those water levels of S-331 and S-176 were exceeded, discharges at S-331 would be terminated until the S-176 headwater recedes to 5.0 feet." S176_Cond is dependent on S331_TW and S176_HW -> true if either stage prohibits S331 releases.	1) Water Supply / Flood Control 2) Design Q=1161 (special code Special_Assessors_ECB_2010-11.so) mse_unit outlet for L31N; localLevel = 99.0 mse_unit inlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit inlet for L31S; "L31S res" resLevel = 3.5 mse_unit outlet for L31N; mse_unit "L31N local" localLevel = 99.0 S331_TW_lim = 6.0 Operations defined in S331_ECB_2010-11.cc: S331_HW_levels = 4.0 4.5 5.0 5.5 (LPG2 stage criteria) S331 OPERATING CRITERIA: “Discharges through S-331 can be made if the S-331 tailwater stage is below 6.0 feet and the S-176 headwater stage is below 5.5 feet. If either of those water levels of S-331 and S-176 were exceeded, discharges at S-331 would be terminated until the S-176 headwater recedes to 5.0 feet." S176_Cond is dependent on S331_TW and S176_HW -> true if either stage prohibits S331 releases.

Canal	Structure	EARECB (RSMGL)	EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet/Dry Season Normal FC Operations
L-31N (cont)	S-173 & S-331P (COMBQ) * (cont)	S331 High Range: If the water level at LPG2 well is < 5.5 ft, S331 HW will have no limit. S331 Intermediate Range: If the level at LPG2 well is > or = 5.5 and < 6.0 ft, the average daily water level upstream of the S-331 will be maintained between 5.0 ft., and 4.5 ft if permitted by d/s conditions. S331 Low Range: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. S331 Low Range Adjustment: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are not limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. Use previous day LPG2 stage, S331_TW and S176_HW, C-357 WL for current day operations	S331 High Range: If the water level at LPG2 well is < 5.5 ft, S331 HW will have no limit. S331 Intermediate Range: If the level at LPG2 well is > or = 5.5 and < 6.0 ft, the average daily water level upstream of the S-331 will be maintained between 5.0 ft., and 4.5 ft if permitted by d/s conditions. S331 Low Range: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. S331 Low Range Adjustment: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are not limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. Use previous day LPG2 stage, S331_TW and S176_HW, C-357 WL for current day operations
	S-176	**5.0 / 4.75** open/closed wet & dry season 1) Water Supply / Flood Control 2) desigh Q = 1100 cfs (discharge coef. = 1135 cfs in msestruc*.xml) mse_unit outlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit outlet for L31S; "L31S res" resLevel = 3.5 mse_unit inlet for C111; maintenance level and reserve level determined in high_rf_events assessor S176_HW_levels 5.0 5.5 (input variable for S331 ops in Special_Assessors_ECB_2010-11.so)	**5.0 / 4.75** open/closed wet season **5.1 / 4.8** open/closed dry season 1) Water Supply / Flood Control 2) design Q = 1100 cfs (discharge coef. = 1135 cfs in msestruc*.xml) mse_unit outlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit outlet for L31S; "L31S res" resLevel = 3.5 mse_unit inlet for C111; maintenance level and reserve level determined in high_rf_events assessor S176_HW_levels 5.0 5.5 (input variable for S331 ops in Special_Assessors_ECB_2010-11.so)
	S-174*	S174 not in model; canal is blocked near structure	S174 not in model; canal is blocked near structure

<table>
<tr><td rowspan="2">Canal</td><td rowspan="2">Structure</td><td colspan="2">EARECB (RSMGL)</td><td colspan="2">EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)</td></tr>
<tr><td colspan="2">Open/Close (ft NGVD)
(Optimum stage ft NGVD)
Wet Season/Dry Season Normal FC Operations</td><td colspan="2">Open/Close (ft NGVD)
(Optimum stage ft NGVD)
Wet Season/Dry Season Normal FC Operations</td></tr>
<tr><td rowspan="3">L-31N
(cont)</td><td rowspan="2">S-332A, B,C,D
(pumps)</td><td>S332A 5.0/4.7
S332B 5.0/4.7
S332BN 5.0/4.7
S332C 5.0/4.7
S332D 4.85/4.65</td><td>S332A Non-Existent
S332B1 4.7/4.5
S332B2 5.0/4.7
S332BN1 4.7/4.5
S332B2 5.0/4.7
S332C1 4.7/4.5
S332C2 5.0/4.7
S332D1 4.65/4.50
S332D2 4.85/4.65</td><td>S332A Non-Existent
S332B1 4.7/4.5
S332B2 5.0/4.7
S332BN1 4.7/4.5
S332B2 5.0/4.7
S332C1 4.7/4.5
S332C2 5.0/4.7
S332D1 4.65/4.50
S332D2 4.85/4.65</td><td>S332A non-existent
S332B 5.0/4.7
S332BN 5.0/4.7
S332C 5.0/4.7
S332D 4.85/4.65</td></tr>
<tr><td colspan="2">S332A =300 cfs
S332B =325 cfs
S332BN =250cfs
S332C = 575 cfs
S332D = 500 cfs Jul16-Nov30, 325cfs Dec 1-Jan31, 165cfs Feb 1-Jul15
All Flood Control</td><td colspan="2">S332B1=125 cfs
S332B2=125 cfs
S332BN1 =125cfs
S332BN2=125 cfs
S332C1 = 250 cfs
S332C2=250 cfs
S332D1 = 250 cfs
S332D2 = 250 cfs Jul15-Nov30, 75cfs Dec 1-Jan31, 0cfs Feb 1-Jul14
All Flood Control</td></tr>
<tr><td>S-357 (pump)</td><td colspan="2">6.2 / 5.7 open/closed wet & dry season
1) Flood Control
Pump Q = 126 cfs</td><td colspan="2">S357A (5.7' Nov 1-May31, 5.2' Jun1-Oct31)/(5.4' Nov 1-May31, 4.9' Jun1-Oct31)
S357B (6.0' Nov 1-May31, 5.5' Jun1-Oct31)/(5.7' Nov 1-May31, 5.2' Jun1-Oct31)

S357A = 250 cfs
S357B = 250 cfs</td></tr>
<tr><td rowspan="2">L-31W</td><td>S-332
(pumps)*</td><td colspan="2">Non-existent</td><td colspan="2">Non-existent</td></tr>
<tr><td>S-175</td><td colspan="2">Non-existent</td><td colspan="2">Non-existent</td></tr>
<tr><td rowspan="7">C-111</td><td>S-200</td><td colspan="2">S-200A=75cfs; 3.8/3.6
S-200B=75cfs; 3.9/3.6
S-200C=75cfs; 4.0/3.6</td><td colspan="2">S-200A=75cfs; 3.6/3.4
S-200B=75cfs; 3.7/3.4
S-200C=75cfs; 3.8/3.4</td></tr>
<tr><td>S-199</td><td colspan="2">S-200A=75cfs; 3.8/3.6
S-200B=75cfs; 3.9/3.6
S-200C=75cfs; 4.0/3.6</td><td colspan="2">S-200A=75cfs; 3.8/3.6
S-200B=75cfs; 3.9/3.6
S-200C=75cfs; 4.0/3.6</td></tr>
<tr><td>S-177</td><td colspan="2">4.2/3.6 (*Open/Close determined in high rainfall event Special Code.)
1) Water Supply & Flood Control
Spillway w/1 gate Design Q = 2900 cfs.</td><td colspan="2">4.2/3.6 (*Open/Close determined in high rainfall event Special Code.)
1) Water Supply & Flood Control
Spillway w/1 gate Design Q = 2900 cfs.</td></tr>
<tr><td>S-18C</td><td colspan="2">2.6 / 2.3 open/closed wet & dry season
1) Water Supply & Flood control
Spillway w/2 gates Design Q=3200 cfs.</td><td colspan="2">2.6 / 2.3 open/closed wet & dry season
1) Water Supply & Flood control
Spillway w/2 gates Design Q=3200 cfs.</td></tr>
<tr><td>S-197*</td><td colspan="2">****Same as IOP,See Note
1) S197 ops see below ****
2) Flood control only
13 Culverts w/gates Design Q=6000 cfs.</td><td colspan="2">****Same as IOP,See Note
1) S197 ops see below ****
2) Flood control only
13 Culverts w/gates Design Q=6000 cfs.</td></tr>
<tr><td>S-332E (pump)</td><td colspan="2">Non-existent</td><td colspan="2">Non-existent</td></tr>
</table>

* SFWMM uses special code. Open/Close may or may not be used in operations.

**** S-197 Ops: Open 3 gates if S-177 full open & S-177>4.1 ft or S-18C> 2.8 ft

Open 7 gates if S-177 > 4.2 ft for 24 hrs or S-18C > 3.1 ft

Open 13 gates if S-177 > 4.3 ft or S-18C > 3.3 ft

Close when all the conditions below are met

1) S-176 < 5.2 ft and S-177 < 4.2 ft
2) Storm moved away from basin
3) After 1 and 2 are met, keep the number of S-197 culverts open necessary only to match residual flow through S-176

Modeling Section, H&H Bureau
South Florida Water Management District

EAA Storage Reservoir Project (EAASR)
Final Array of Alternatives Model Documentation Report

March 2018

1.0 Overview

Identification

The Everglades Agricultural Area Storage Reservoir Project (EAASR) is an expedited planning effort undertaken as a project component of the Comprehensive Everglades Restoration Plan (CERP). This project planning effort was led by the South Florida Water Management District (SFWMD) and seeks to enhance the performance of the Central Everglades Planning Project (CEPP) which has already been authorized by Congress. The project will be designed to: 1) reduce the high-volume freshwater discharges from Lake Okeechobee to the Northern Estuaries, 2) identify storage, treatment and conveyance south of Lake Okeechobee to increase flows to the Everglades system and 3) reduce ongoing ecological damage to the Northern Estuaries and Everglades system. The project worked throughout late 2017 and early 2018 and combines planning and design activities for three primary areas of interest in the south Florida system as follows: 1) Next increment of storage and necessary treatment to provide progress towards the level of restoration envisioned for the CERP, 2) Continue to improve the quantity, quality, timing and distribution of water flows to the Northern Estuaries and central Everglades and 3) Be consistent with federal program and policy requirements. Modeling support to the EAASR effort was provided by a team comprised of modelers from the Modeling Section of the Hydrology and Hydraulics Bureau of the SFWMD.

Scope and Objectives

Modeling support for EAASR focused on working with the larger project planning team and other interested parties to formulate and test project features leading to the ultimate identification and refinement of a tentatively selected plan (TSP). Modeling products were developed at the appropriate level of detail to support feature screening and detailed representation of project features and to provide information to all necessary evaluations required for plan development and documentation. The project plan formulation framework is built upon work already completed as part of the CEPP planning effort and utilizes the same tools and techniques by performing initial screening followed by detailed evaluation to identify final project planning alternatives and ultimately a TSP for the effort.

The CEPP Modeling Strategy document (**SFWMD, 2012a**) describes the modeling process and tools utilized, the associated rationale of the selection process and the means by which the tools could expediently support the project workflow. Given that the EAASR effort is being pursued as a change to an authorized CERP project, utilization of comparable modeling strategies and tools as those used in the development of the authorized CEPP plan was a guiding principle of EAASR modeling work. The primary model support tools utilized in EAASR project refinement are as follows:

Screening Tool and Water Quality Assessment:

- Dynamic Model for Stormwater Treatment Areas (DMSTA)

Detailed Planning Models:

- Regional Simulation Model Basins (RSMBN)
- Regional Simulation Model Glades-LECSA (RSMGL)

From a modeling deliverable perspective, the entirety of the EAASR modeling support can be summarized by reviewing the following three Model Documentation Reports (MDRs):

1. EAASR Baseline – Reviews the various non-EAASR model representations (e.g., current and future without project conditions) used in various aspects of the project planning (**SFWMD, 2018a**).
2. EAASR Final Array of Alternatives – Reviews the model-supported feature screening efforts undertaken to size the reservoir and treatment facilities and detailed evaluation of three modeled “with EAASR” project model representations examined during plan formulation (this document, **SFWMD, 2018b**)
3. EAASR Tentatively Selected Plan – Reviews the model representation of the optimized plan identified in the final steps of plan formulation and project assurance planning (**SFWMD, 2018c**).

This Final Array of Alternatives MDR describes the assumptions, model implementation steps and observed outcomes associated with modeling representations of various future with-EAASR project features model scenarios. These model runs were predominantly used to identify the EAASR project features carried forward into the EAASR TSP in support of plan formulation. This document will focus on the modeling details of these scenarios; information on the use and rationale for the definition of these conditions is contained in the CEPP PACR (**SFWMD, 2018d**). Additionally, this document will describe the feature sizing and screening work performed using DMSTA informed by baseline hydrology quantified in RSMBN.

2.0 Basis

Project Assumptions

This Final Array of Alternatives MDR describes the assumptions, model implementation steps and observed outcomes associated with modeling the following scenarios:

EAASR Final Array of Alternatives - released 12/21/2017

- Alternatives R240, R360 and C360

Per Florida statute Chapter 2017-10, two storage options on parcels within the EAA were identified for further modeling evaluation as follows: 1) 240,000 acre-feet of storage and necessary treatment on A-2 Parcel plus conveyance improvements and 2) 360,000 acre-feet of storage and necessary treatment on A-1 and A-2 Parcels plus conveyance improvements. To determine feature sizing that would efficiently achieve project objectives within the bounds identified in state law, screening analysis was undertaken using the DMSTA model. DMSTA routing was updated (**Figure 2.1**) to incorporate EAASR potential project features and a range of scenarios were evaluated to help identify potential sizing characteristics of project features capable of increasing restoration flows to the Everglades while meeting established water quality standards for phosphorous. Initially, screening efforts in plan formulation utilized the CERP goal of sending flow south into the Everglades as narrated in the Central Everglades plan. Central Everglades was a first incremental step in Everglades restoration and did not formally quantify the performance of the CERP program, but the CEPP report stated that the CEPP project performance of increasing flow south by ~210 kac-ft on average annually achieved approximately two-thirds of the CERP performance. This implied an initial target CERP flow increase of approximately 300 kac-ft on average annually for use in EAASR screening work for identification of Final Array features. Once again, key elements of model implementation are described in Section 3.

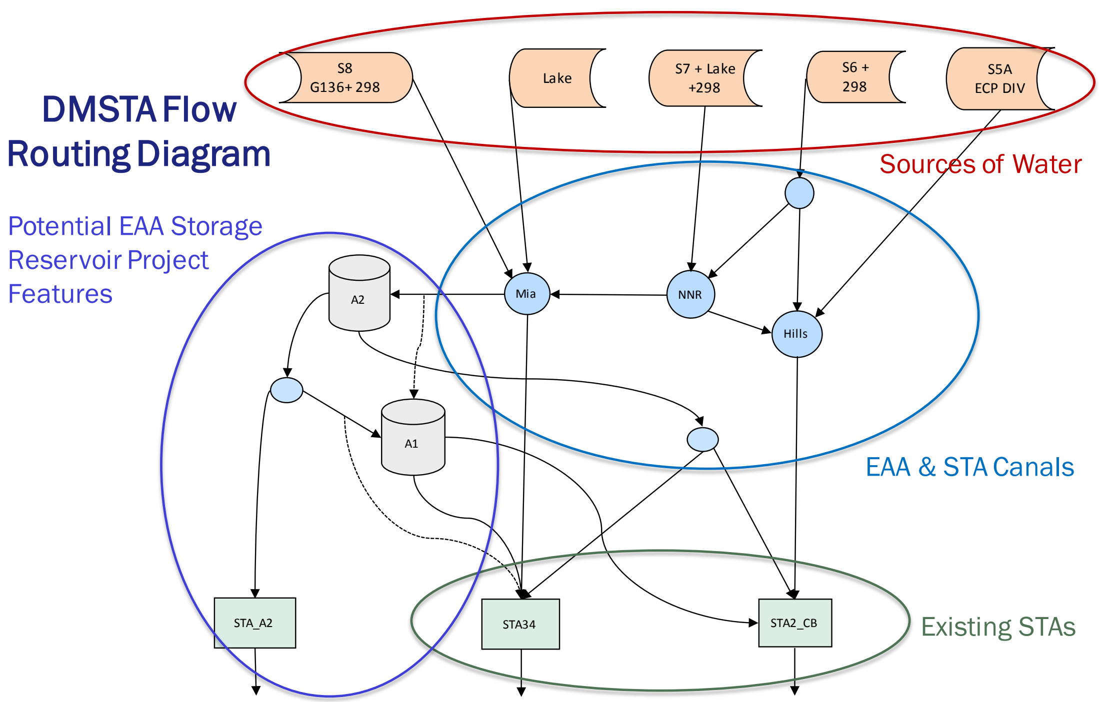


Figure 2.1 DMSTA Flow Routing Diagram Incorporating Potential EAASR Project Features

Utilizing screening results from DMSTA, the EAASR project team identified a series of infrastructure changes for each RSMBN alternative as shown in **Figure 2.2**. Reservoir and STA effective footprints were selected from DMSTA screening outcomes and canal conveyance assumptions were iteratively reduced from CERP-assumed levels to identify a more cost-effective set of canal expansion assumptions that would maintain improved estuary performance. More detailed figures describing each alternative are included in Section 3 and illustrated in **Appendix A.**

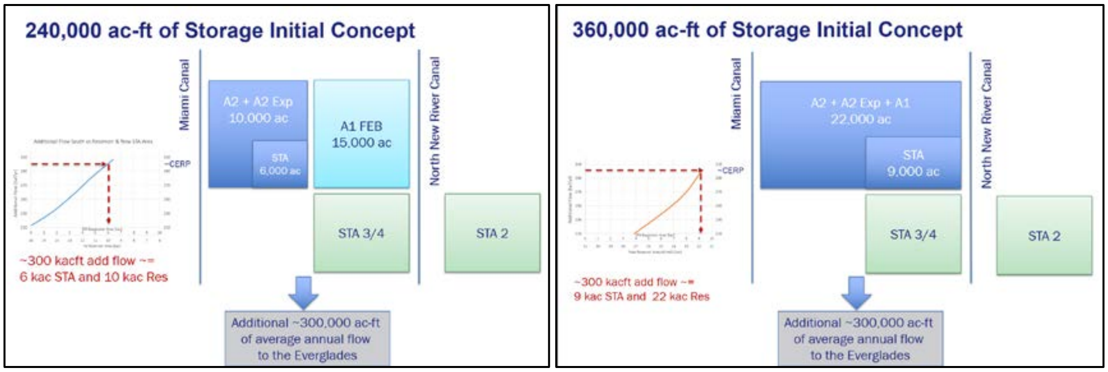


Figure 2.2. Generalized view of infrastructure changes modeled for alternatives R240, R360, and C360.

Given the conceptual diagrams provided by the project team, RSMBN modeled representation of the project assumptions could be simplified into two modeling scenarios: R240 and R360 that could represent several different engineering realizations of the local reservoir / STA features. Another operational scenario was also identified: C360 in which the reservoir feature was not dedicated to environmental deliveries (as assumed in the “R” model runs), but rather assumed a multi-purpose reservoir facility as envisioned in the original CERP plan that could meet both environmental and consumptive use water supply demands. All project features affect inputs to the RSMBN model and the resulting flows simulated from RSMBN provide updated boundary conditions to the southern RSMGL model. Other than refined Everglades inflows, no other changes are assumed for the RSMGL model. In addition to the northern infrastructure changes identified in **Figure 2.2**, the project team also identified a series of performance objectives to help guide operational decision-refinement as follows:

For all scenarios:

- Reduce the high-volume freshwater discharges from Lake Okeechobee to the Northern Estuaries
- Reduce ongoing ecological damage to the Northern Estuaries and Everglades system
- Maintain a hydrologic regime in the FEB and STAs consistent with water quality objectives
- Strive to achieve approximately 300 kac-ft average annual additional water delivery to the Everglades system, promoting significant additional dry season deliveries
- Improve wetland hydroperiod inundation duration in the Ridge and Slough landscape of the Greater Everglades
- Maintain the weighted average of Lake Okeechobee Standard Score performance measures near baseline levels.

For the R240 and R360 scenarios,

- Do not impact the frequency, duration and severity of Lake Okeechobee triggered water shortage events with emphasis on the “8 worst years” of drought conditions.

For the C360 scenario,

- Reduce the frequency, duration and severity of Lake Okeechobee triggered water shortage events with emphasis on the “8 worst years” of drought conditions.

Model Limitations and Intended Use of Results

The primary modeling products of EAASR were evaluated based on outputs from the Regional Simulation Model (RSM; **SFWMD, 2005a** and **2005b**). The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g, use available input-driven options to represent more complex project operations).

As described in **Figure 2.3**, the EAASR modeling workflow strives for appropriate application of modeling tools (particularly DMSTA and RSM) for their intended use. It is neither efficient nor necessary to force intermediate modeling products to reflect a higher level of detail or consistency than is needed at that time to be robust for decision making. Along the modeling workflow, there are many opportunities for refinement. Intermediate products serve the immediate need and then are enhanced, incorporating feedback and information as the process progresses.

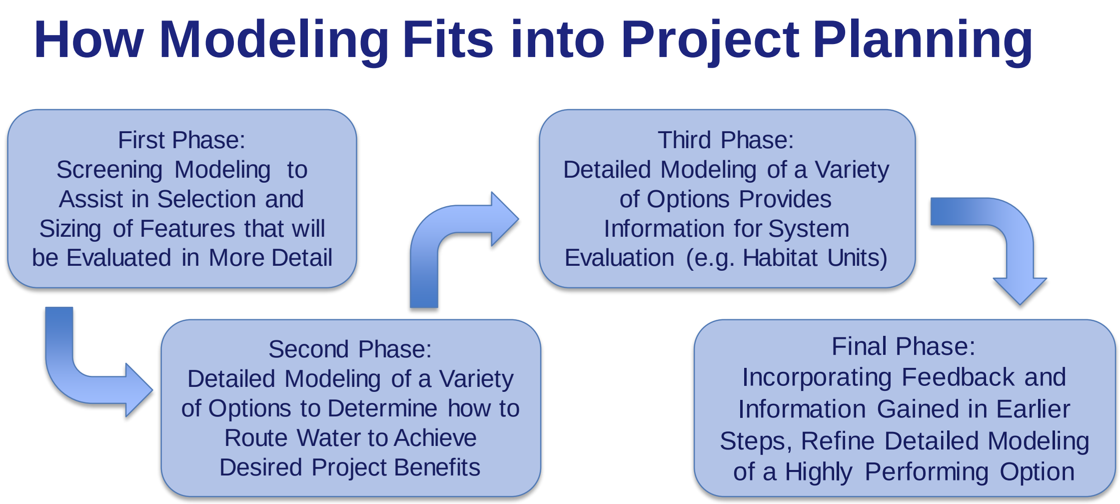


Figure 2.3. Typical EAASR Modeling Workflow

The RSMBN (**SFWMD, FDEP & FDACS, 2009a, 2009b**), RSMGL (**SFWMD, 2010** and **2011**), and DMSTA (**Walker & Kadlec, 2005**; **Wang, 2012**) models were reviewed through the USACE validation process for engineering software, as part of the CEPP project. The RSM and DMSTA models were classified as “allowed for use” for South Florida applications in August 2012 and January 2013, respectively.

3.0 Simulation

Modeling Tools Used

RSM version 2.3.5R was used to run the RSMBN and RSMGL models.
Release date 11/10/2017, SVN Version #5207.
DMSTA v2c2b

Feature Screening & Sizing Analysis

As an initial effort to identify feasible sizing of potential EAASR reservoir and STA sizing, DMSTA was utilized to develop performance curves for use by the project team. DMSTA had been previously applied in Central Everglades planning to confirm that the CEPP ALT4R2 selected plan complied with planning targets for water quality treatment. The same version of DMSTA utilized in CEPP (and consistent with the work performed in the SFWMD Restoration Strategies program support) and the associated water quality assumptions were utilized in the EAASR effort. In this screening analysis, combinations of reservoir and STA footprints were assumed and then "restoration flows" were simulated as boundary flows from Lake Okeechobee into DMSTA (and its associated regional design tool). For each assumed combination of storage and treatment, Lake boundary flows were scaled up from the CEPP simulated flows toward the identified goal of 300 kac-ft above current condition until water quality targets could not be met. This effort produced **Figures 3.1 and 3.2** to help inform sizing selection for EAASR alternatives.

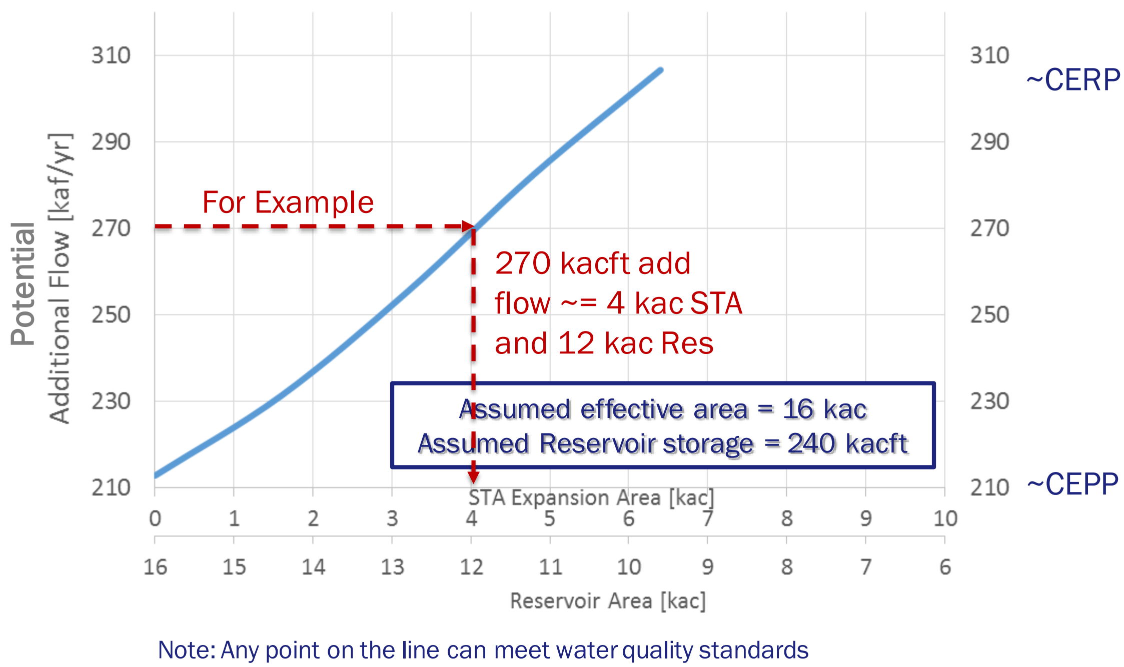


Figure 3.1. Potential Additional Flow South versus 240kaf Reservoir & STA Acreage

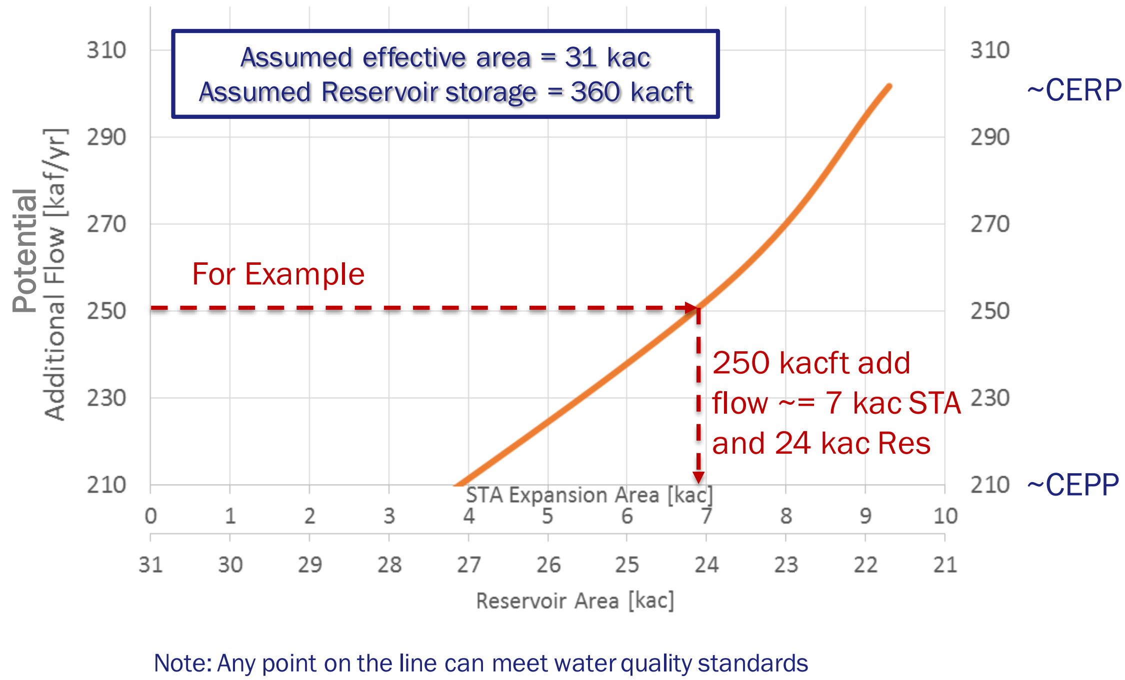


Figure 3.2. Potential Additional Flow South versus 360kaf Reservoir & STA Acreage

It is important to note that DMSTA modeled outcomes in this round of work (as presented) show potential flow increase relative to the DMSTA baseline used in the original CEPP support which was the SFWMD's Restoration Strategies plan (assuming underlying hydrology from the South Florida Water Management Model). The total flows determined to be treated in DMSTA to achieve water quality standards were transferred into RSM and as a result, relative flow changes will honor desired trends, but may not illustrate the same absolute magnitude.

Canal Conveyance Screening Analysis

CERP identified conveyance improvements in the Miami and North New River canals were relatively large compared to the capacity of contemplated storage in the EAA as shown in **Figure 3.3**. A desktop analysis of northern estuary discharge events still occurring with the CEPP authorized plan in place illustrated that a canal capacity of 4500 cfs was capable of capturing all remaining CEPP discharges if downstream storage was available (see **Figure 3.4**). Using 4500 cfs as a starting point, intermediate RSMBN simulation of EAASR configurations evaluated smaller conveyance assumptions with the goal of identifying a more cost-effective set of canal expansion assumptions that would maintain improved estuary performance. Results of this analysis will be discussed in Section 3 and a more detailed description of this screening analysis about project rationale for identifying assumed canal improvements can be found in the CEPP PACR Appendix E.

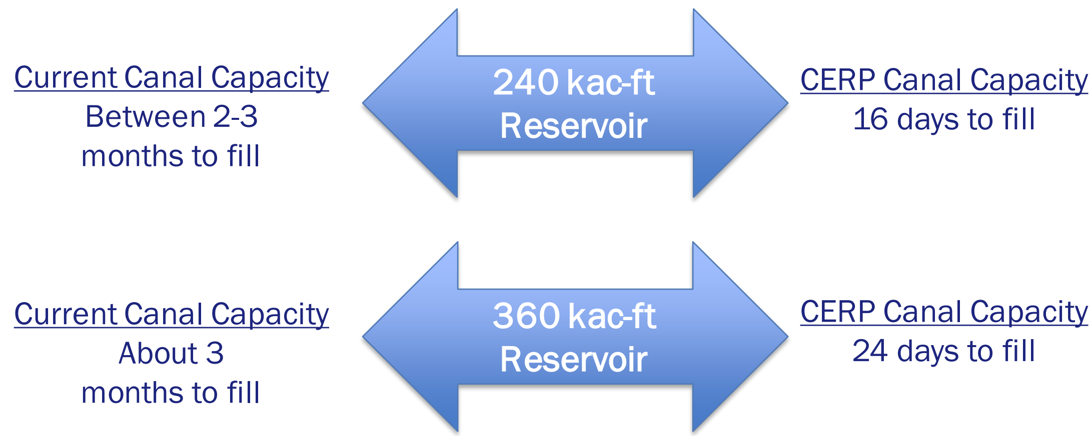


Figure 3.3. Range of potential filling times for EAASR storage volumes

Above-Ground Storage Reservoir Volume (acre-feet)	Additional Conveyance Capacity in Miami Canal (cfs)	Additional Conveyance Capacity in North New River Canal (cfs)	Number of Times the 14-day Moving Average Flow to St. Lucie Estuary Exceeds 2,000 cfs for 14 or More Days due to Lake Okeechobee Releases	Number of Months Flow to Caloosahatchee Estuary Exceeds 2,800 cfs due to the C-43 Basin Runoff and Lake Okeechobee Releases	Total Lake Okeechobee Regulatory Releases to St. Lucie and Caloosahatchee Estuaries (acre-feet)
240,000	4,000	3,500	25	63	327,000
	3,000	1,500	25	63	327,000
	1,000	200	26	63	325,000
	0	0	35	64	345,000
360,000	4,000	3,500	26	62	329,000
	3,000	1,500	26	62	329,000
	1,000	200	26	63	328,000
	0	0	31	63	346,000

Figure 3.4. Magnitude of excess discharge events to be captured by EAASR project

RSM Model Set Up

This section will focus on the project plan formulation alternatives and assumed model implementation changes relative to the EAASR FWO project baseline (**SFWMD, 2018a**), which is the same as the CEPP ALT4R2 selected plan (**SFWMD&IMC, 2014c**). The EAASR final array of alternatives was developed using the RSMBN and RSMGL models. Collectively these two models cover the spatial extent of the project planning area as shown in **Figure 3.5**. The RSMBN modeling for the final array utilizes corresponding DMSTA scenarios to inform operational strategies that maintain water quality performance. Three RSMBN scenarios were defined as Alternatives (ALT R240, ALT

R360, and ALT C360). The period of simulation utilizes a climate record from 1965 to 2005. As previously stated, all project features affect inputs to the RSMBN model and the resulting flows simulated from RSMBN provide updated boundary conditions to the southern RSMGL model. Other than refined Everglades inflows, no other changes are assumed for the RSMGL model. Details about project rationale for defining these scenarios can be found in the CEPP PACR (**SFWMD, 2018d**).

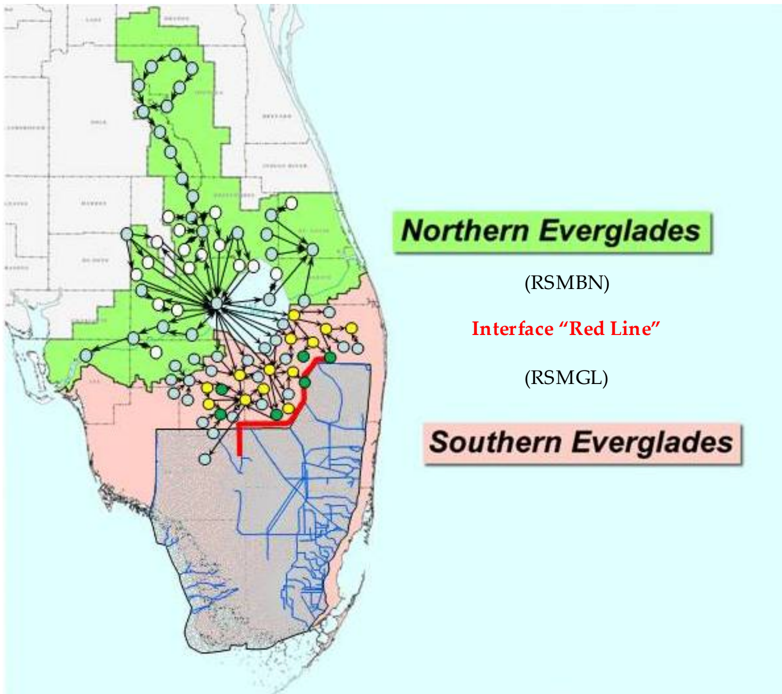


Figure 3.5. Area of interest within EAASR Modeling Approach

For modeling purposes, three alternatives were considered north of the redline, using RSMBN (see **Figures 3.6 and 3.7** for approximate component locations). It is important to note that given the regional scale of the RSM, these alternatives could represent several different engineering realizations of the local reservoir / STA features. The alternative scenarios assume the following:

R240 scenario

A 240 kac-ft storage reservoir located on 10,100 acre effective footprint (A2 RES) located north of Holeyland and assumed operations as follows:

- A2 RES inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south (up to ~ 4ft buffer depth from full level to allow attenuation of peak EAA runoff events).
- A2 RES outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at A1FEB, STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient.

- 0.5 ft minimum depth below which no releases are allowed
- 23.5 ft maximum depth above which inflows are discontinued
- Inflows at the reservoir inflow pump station are assumed to convey up to 3000 cfs from the Miami canal and 1500 cfs from the NNR canal (combined basin runoff and Lake O water); inflow to the EAA reservoir can also be made from the existing G370 and G372 pump stations up to a 6 ft depth.

A 15,853-acre Flow Equalization Basin (A1 FEB, consistent with EARFWO) located north of STA-3/4 with assumed operations as follows:

- FEB inflows are from the A2 RES and are consistent with established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas). FEB inflows are limited to 500 cfs when depths are above 2.5 ft.
- FEB outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient.
- 0.5 ft minimum depth below which no releases are allowed
- 3.8 ft maximum depth above which inflows are discontinued
- Assumed inlet structure of 1500 cfs capacity from A2 RES for modeling purposes.

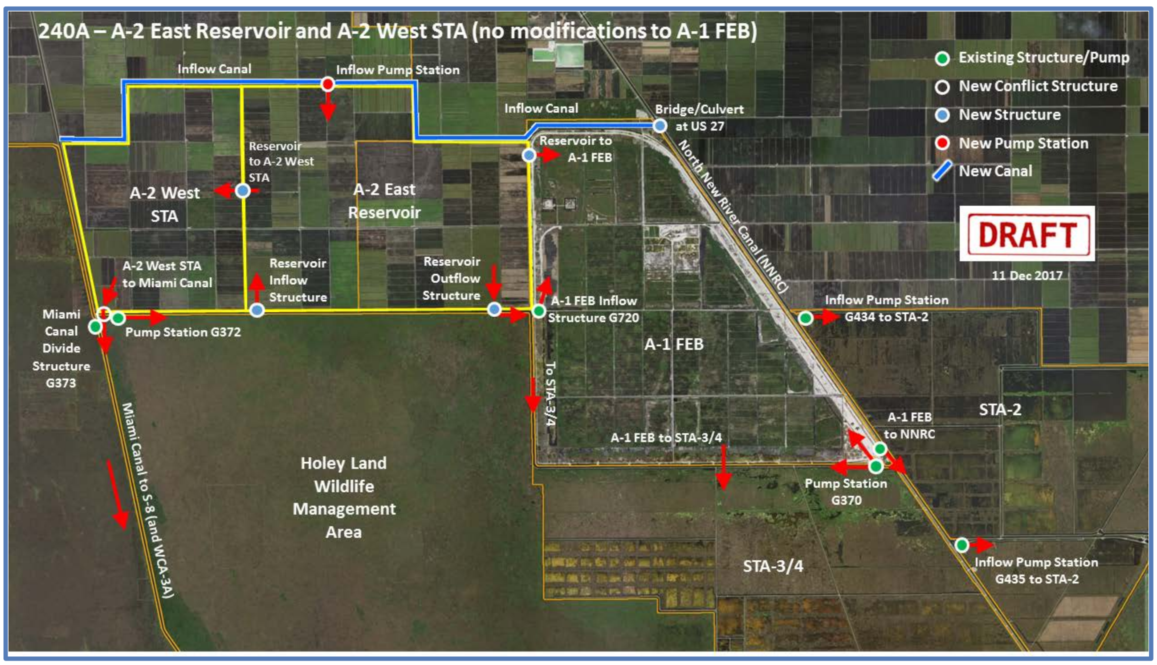


Figure 3.6. Example R240 Schematic Diagram Provided by EAASR Project Team

R360 and C360 scenarios

A 360 kac-ft storage reservoir located on 19,700-acre effective footprint (A1/A2 RES) located north of STA3/4 & Holeyland and assumed operations as follows:

- A1/A2 RES inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south (only until ~ 2ft buffer depth from full level; buffer retained to allow attenuation of peak EAA runoff events).

- A1/A2 RES outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient.
- 0.5 ft minimum depth below which no releases are allowed
- 18.2 ft maximum depth above which inflows are discontinued
- Inflows at the reservoir inflow pump station are assumed to convey up to 3000 cfs from the Miami canal and 1500 cfs from the NNR canal (combined basin runoff and Lake O water); inflow to the EAA reservoir can also be made from the existing G370 and G372 pump stations up to a 6 ft depth.
- For C360 only, supplemental irrigation demands in the Miami and NNR/Hillsboro basins can be met from the reservoir when reservoir depths exceed 6.3 feet.

Figure 3.7. Example R360 Schematic Diagram Provided by EAASR Project Team

The operations of the assumed reservoir and FEB features in each alternative are integrated with the regional objectives by including operational modifications to the Lake Okeechobee regulation schedule as follows:

- Lake Okeechobee regulatory releases to the south are made when the Lake is in or above the baseflow zone of the LORS08 schedule and when criteria as identified in **Figure 3.8** are satisfied.
- In order to promote opportunity for Lake discharges to the south, release criteria from the Northern Estuaries are also modified to result in lower overall discharges. These operations were identified using Latin Hypercube sub-sampling optimization in a manner similar to that employed in Lake Okeechobee Watershed Restoration

Project support. Given the complexity of these operations and that the final array results will be further refined in the development of the TSP, specific documentation of each alternative operational scheme is not provided in this report. Outcomes of the subsequent TSP will be identified for use in development of the Draft Project Operating Manual.

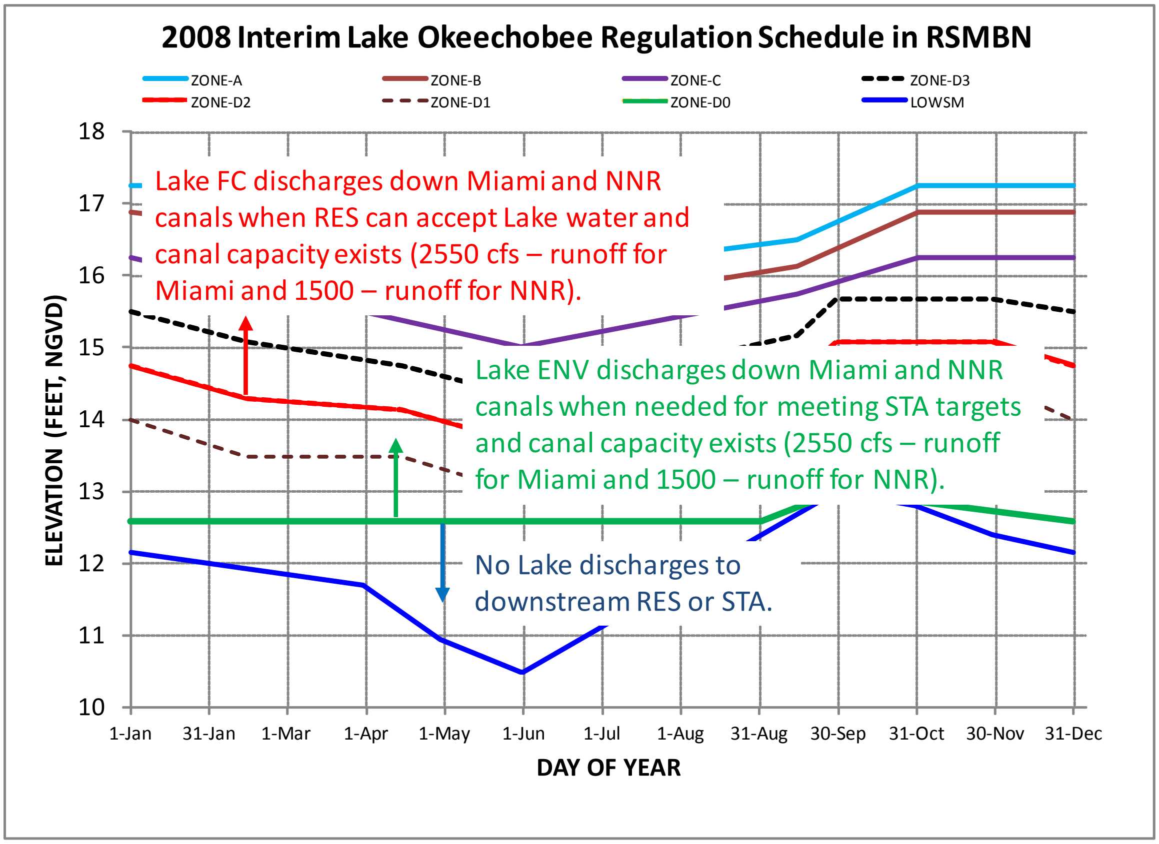


Figure 3.8. Lake Okeechobee Operational Criteria for Determining Discharges South to FEB and STA Facilities.

The EAASR project alternatives all share a common configuration south of the Redline which is based on CEPP ALT4R2, as shown in **Figure 3.9,** operated to convey the additional EAASR water provided to the Everglades.

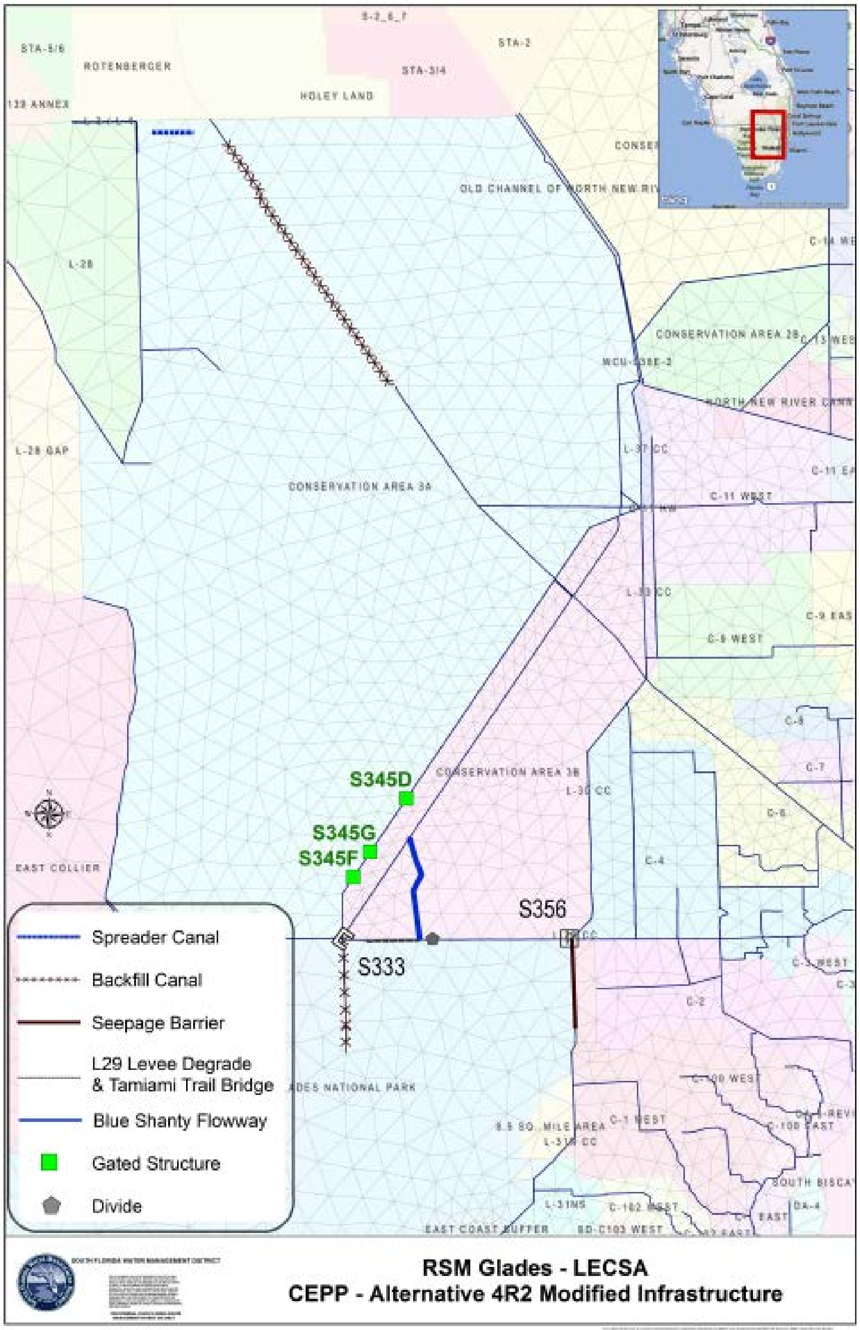


Figure 3.9. Configuration south of the redline common to all final array alternatives.

4.0 Results

Final EAASR modeling products will be uploaded to the Statewide Model Management System (SMMS), a geographic information system (GIS) based application that includes model input data, select model output data, source code/executable files and documentation. This system can be accessed at http://apps.sfwmd.gov/smmsviewer/ . EAASR Project modeling products in SMMS can be accessed directly at the project page:

http://apps.sfwmd.gov/smmsviewer/ProjectReport.aspx?projectID=TBD

While the modeling products have been archived in the above systems, the table below lists more specific information including model version, inputs used and detailed output archival location. Version numbers and “svnroot” paths refer to a model version control system found on the SFWMD network that is not generally accessible, but inputs, model executables and source code have been copied into the SMMS system for ease of access.

Version information and model file locations

RSMBN ALT R240 12212017	**RSM_REL_2.3.5R and xml_v12646**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmbn/alternatives/R240/input Output (NAS): ...projects/CEPP_EAR/FilesToFTP/PlanFormulation/Alternatives/02_21Dec2017/rsmbn_model_output/R240	
RSMBN ALT R360 12212017	**RSM_REL_2.3.5R and xml_v12647**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmbn/alternatives/R360/input Output (NAS): ...projects/CEPP_EAR/FilesToFTP/PlanFormulation/Alternatives/02_21Dec2017/rsmbn_model_output/R360	
RSMBN ALT C360 12212017	**RSM_REL_2.3.5R and xml_v12639**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmbn/alternatives/C360/input Output (NAS): ...projects/CEPP_EAR/FilesToFTP/PlanFormulation/Alternatives/02_21Dec2017/rsmbn_model_output/C360	
RSMGL ALT R240 12212017	**RSM_REL_2.3.5R and xml_v12660**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmgl/alternatives/R240/input Output (NAS): ...projects/CEPP_EAR/FilesToFTP/PlanFormulation/Alternatives/02_21Dec2017/rsmgl_model_output/R240	
RSMGL ALT R360 12212017	**RSM_REL_2.3.5R and xml_v12657**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmgl/alternatives/R360/input Output (NAS): ...projects/CEPP_EAR/FilesToFTP/PlanFormulation/Alternatives/02_21Dec2017/rsmgl_model_output/R360	
RSMGL ALT C360 12212017	**RSM_REL_2.3.5R and xml_v12658**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmgl/alternatives/C360/input Output (NAS): ...projects/CEPP_EAR/FilesToFTP/PlanFormulation/Alternatives/02_21Dec2017/rsmgl_model_output/C360	

Canal Conveyance Analysis

As described in Section 3, an intermediate step in developing the RSM final array alternatives involved performing sensitivity analysis to determine a more cost-efficient canal capacity to assume that would not impact the ability of the project to deliver desired benefits (in particular for the northern estuaries where peak flood discharge are routed in the EARECB and EARFWO baselines). An initial run was made with full assumed conveyance of 4500 cfs and then iterative modeling was performed to reduce initial assumed conveyance until observed counts of estuary events and total flow were impacted. As shown in **Table 4.1**, 1200 cfs maintains these desired outcomes within model tolerance and as such, for all alternatives, the canal capacity is assumed to be increased by 1000 cfs in the Miami Canal and 200 cfs in the NNR canal above existing capacity to help convey water from Lake Okeechobee to the EAA Storage Reservoir. Additional analysis was performed on individual events within the simulation period to ensure that the newly assumed conveyance was not constraining performance and as shown in **Figure 4.1**, performance of the 4500cfs and 1200 cfs sensitivity runs are comparable.

Table 4.1 Similar northern estuary performance between initial (4500 cfs) and refined (1200 cfs) canal capacity sensitivity runs

	Additional number of times 14-day moving average flow > 2000cfs >= 14 days from Lok Regulatory releases to St. Lucie Estuary	Number of months flow > 2800cfs from C-43 Basin & Lok regulatory releases (Jan-Dec) to Caloosahatchee Estuary	Total Lake Regulatory discharge toward the Caloosahatchee and St. Lucie Estuaries (kac-ft)
R240_1200	26	63	325
R240_4500	25	63	327
R360_1200	26	63	328
R360_4500	26	62	329

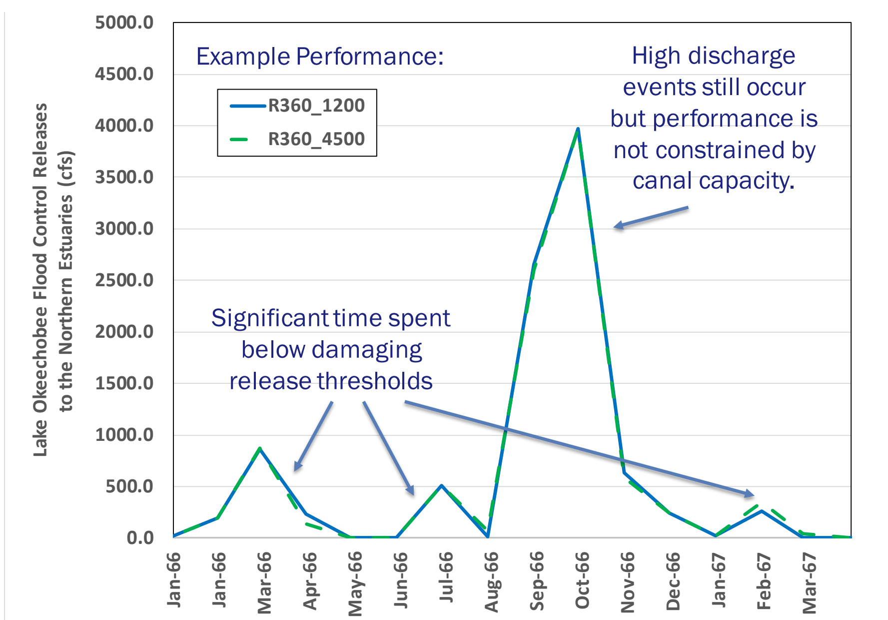


Figure 4.1. Comparison of northern estuary event performance for 4500 cfs and 1200 cfs canal capacity sensitivity runs.

Review of Local and Regional-Level Results

The RSMBN and RSMGL alternative modeling scenarios were reviewed from the perspective of ensuring that localized effects of project implementations were observed as expected and that regional performance was considered reasonable. Specific checks on RSM outputs included the following:

- The RSMBN alternative scenarios generally maintain Lake Okeechobee performance relative to EARFWO as shown in **Figure 4.2 and Table 4.2**.
- EAASR ALTs R240, R360, and C360 reduce the number of high discharge events to northern estuaries relative to the baselines as shown in **Figure 4.3 (a) and (b)**. It can also be observed that low flow event frequency is increased in the Caloosahatchee and St. Lucie Estuaries. It is expected that additional time would allow further operational refinement and avoidance of these low event outcomes.
- EAASR ALTs R240 and R360 scenarios generally maintain LOSA and Tribal water supply performance relative to EARFWO as shown in **Figures 4.4 to 4.6.** C360 shows improved performance relative to the EARFWO.
- Compared to EARECB, EAASR R240, R360, and C360 provide over 300 kac-ft additional flow to Everglades as shown in **Table 4.3**.
- Performance of the R240 A1FEB is generally too deep relative to previous depth regimes observed the EARECB and EARFWO scenarios. It is desired that depth regimes previously identified as consistent with the “design” criteria on an emergent vegetation flowway be maintained and this will be accomplished in the TSP refinement step. Flow regimes for all central flow path STAs were checked in DMSTA (**Wang 2012**) to verify compliance with applicable water quality planning standards.
- RSMGL EAASR R240, R360, and C360 scenarios show expected trends in hydrologic performance in the Everglades. Each alternative has a customized distribution of northern WCA3A inflows consistent with the delivered flows and CEPP downstream rainfall driven operational schemes. An example of these differing outcomes is illustrated in **Figure 4.7**.
- In general, more flow is moving through WCA3A and ENP systems in all alternatives relative to EARFWO (**Figures 4.8 & 4.9**). While stage performance is more similar to the EARFWO baseline, a general wetting trend is observed in most gages (**Figures 4.10 & 4.11**).
- Generally, the C360 scenario tends to show beneficial timing improvements relative to the “R” scenarios that do not include the multi-purpose operation envisioned in the original CERP efforts. An example of this can be seen in **Figure 4.12** where dry year hydroperiod improvements are illustrated for C360.

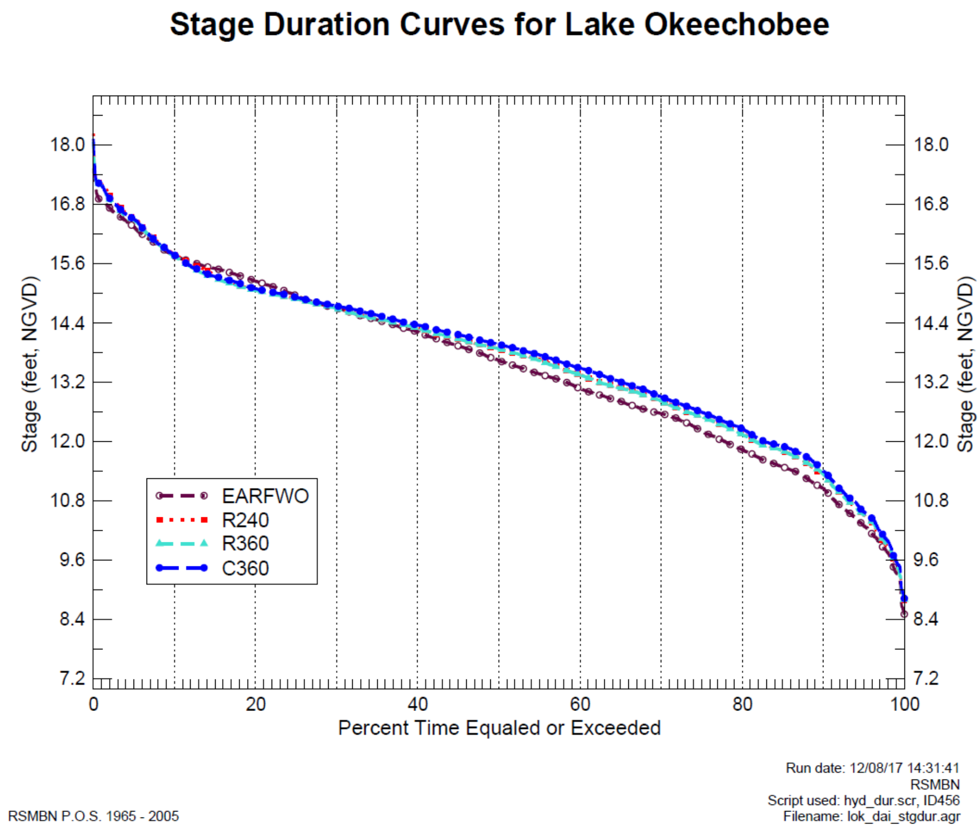


Figure 4.2. Lake Okeechobee performance for ALT C240 relative to the baselines.

Table 4.2. Lake Okeechobee Standard Score Performance Measure

	EARFWO	ALT R240	ALT R360	ALT C360
Low Lake (LO1)	88.62	91.22	91.38	92.85
High Lake (LO2	97.78	90.91	92.68	92.24
Score Below Env (LO3)	47.95	59.08	59.38	63.06
Score Above Env (LO3)	71.76	70.53	71.49	69.88
Weighted Average:	86.6	84.7	85.8	86.1

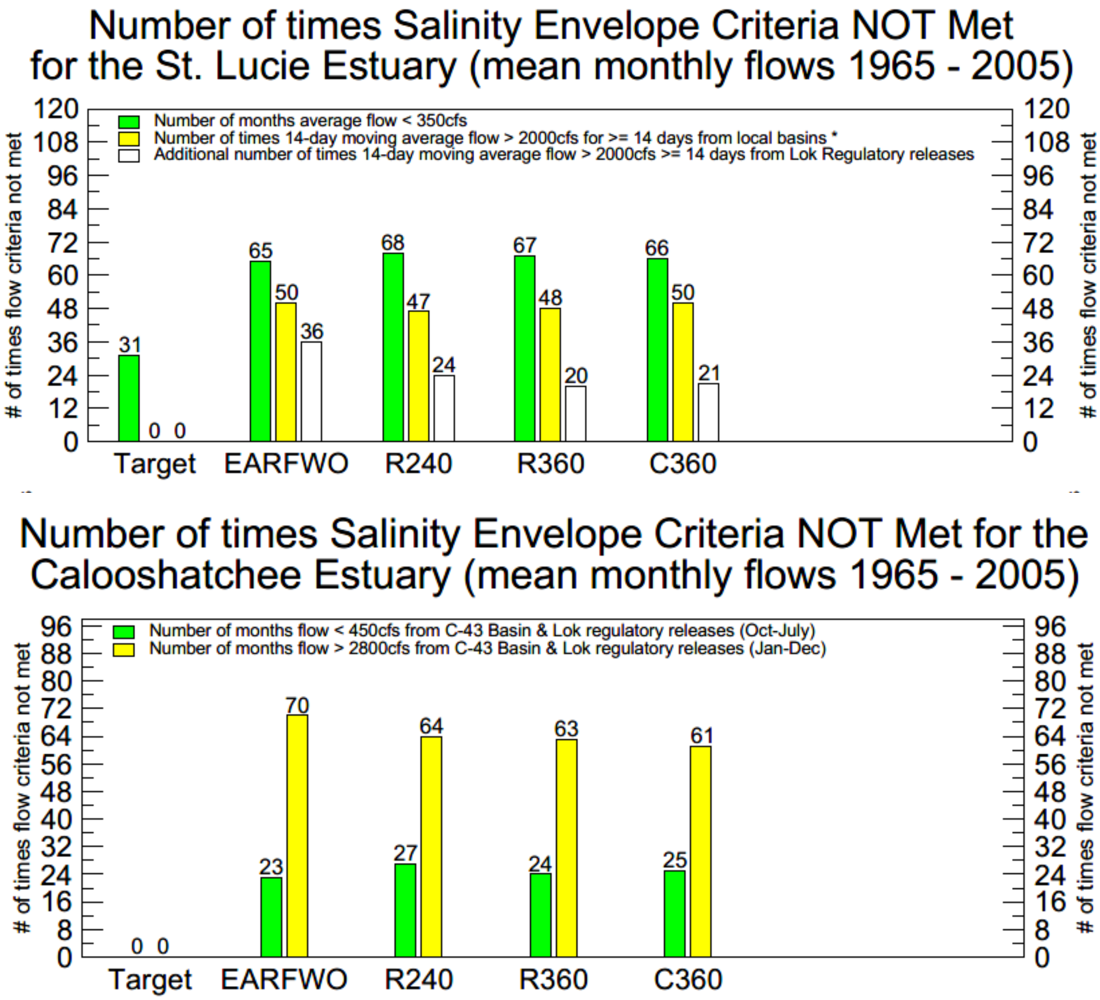


Figure 4.3. High discharge events to northern estuaries (a) St. Lucie Estuary and (b) Caloosahatchee Estuary relative to the baselines for the ALT R240, ALT R360, and ALT C360 alternatives

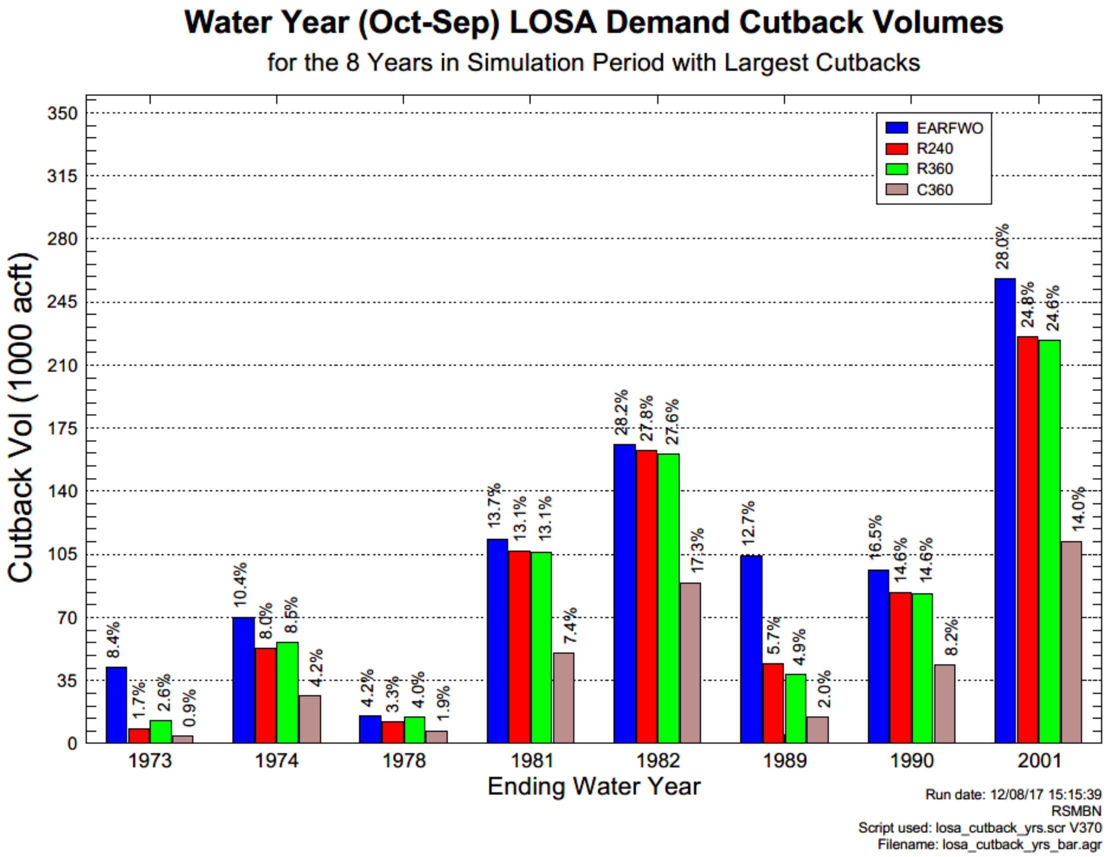


Figure 4.4. Water shortage cutbacks for water years with large cutback

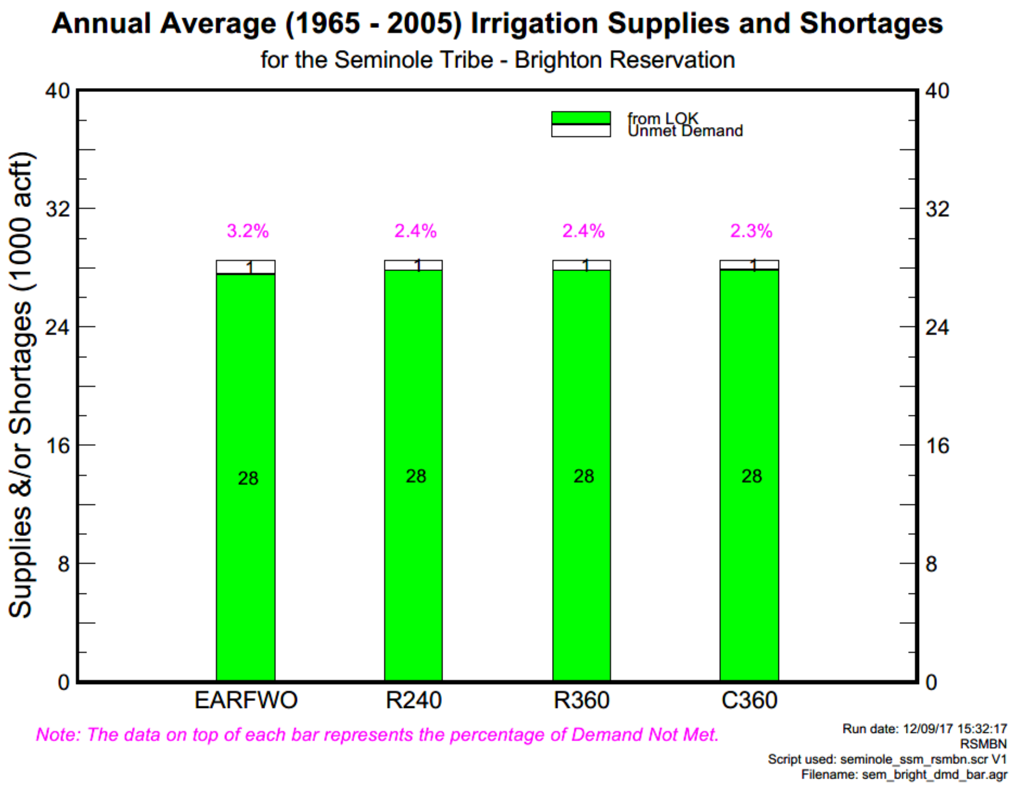


Figure 4.5. Average annual water shortage cutbacks for the Seminole Brighton Reservation

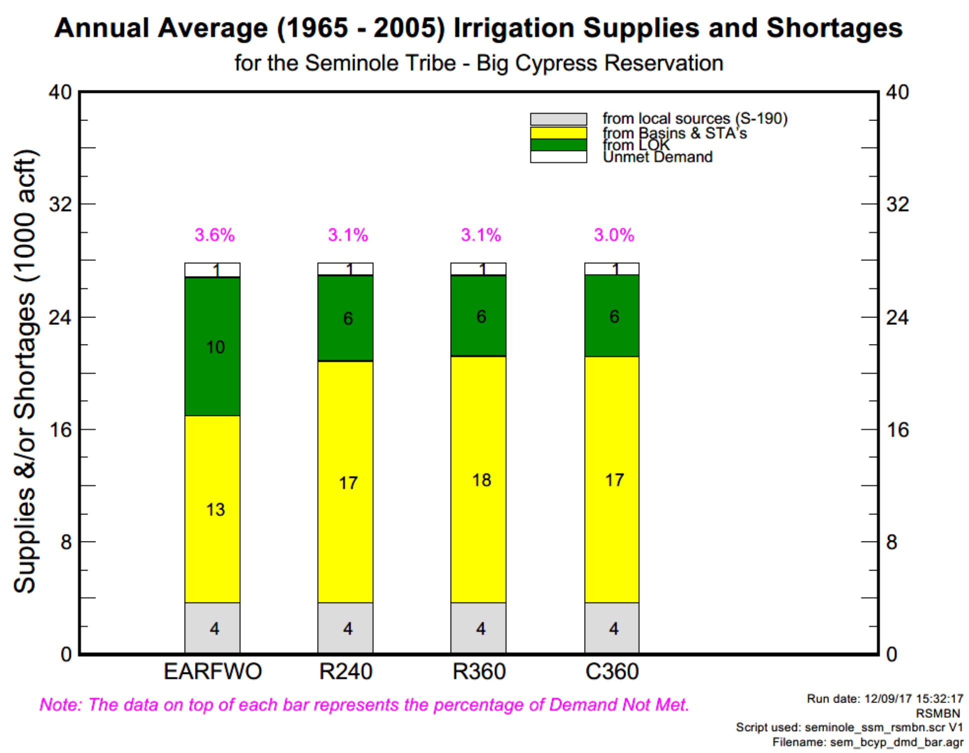


Figure 4.6. Average annual water shortage cutbacks for the Seminole Big Cypress Reservation

Table 4.3. Average annual Discharges (kac-ft) from STA2 and STA34 toward the Greater Everglades

Average Annual Flow (kac-ft, 1965-2005)

	EARECB	**EARFWO**	**R240**	**R360**	**C360**
STA34OUT	383.2	596.2	459.3	376.4	382.8
STA2TOWCA2A	377.1	383.7	486.3	453.2	453.6
ERSTA_TO_LMIAMI	N/A	N/A	144.1	270.7	268.0
	760.3	979.9	1089.7	1100.4	1104.4
Increase over EARECB		**219.7**	**329.4**	**340.1**	**344.1**

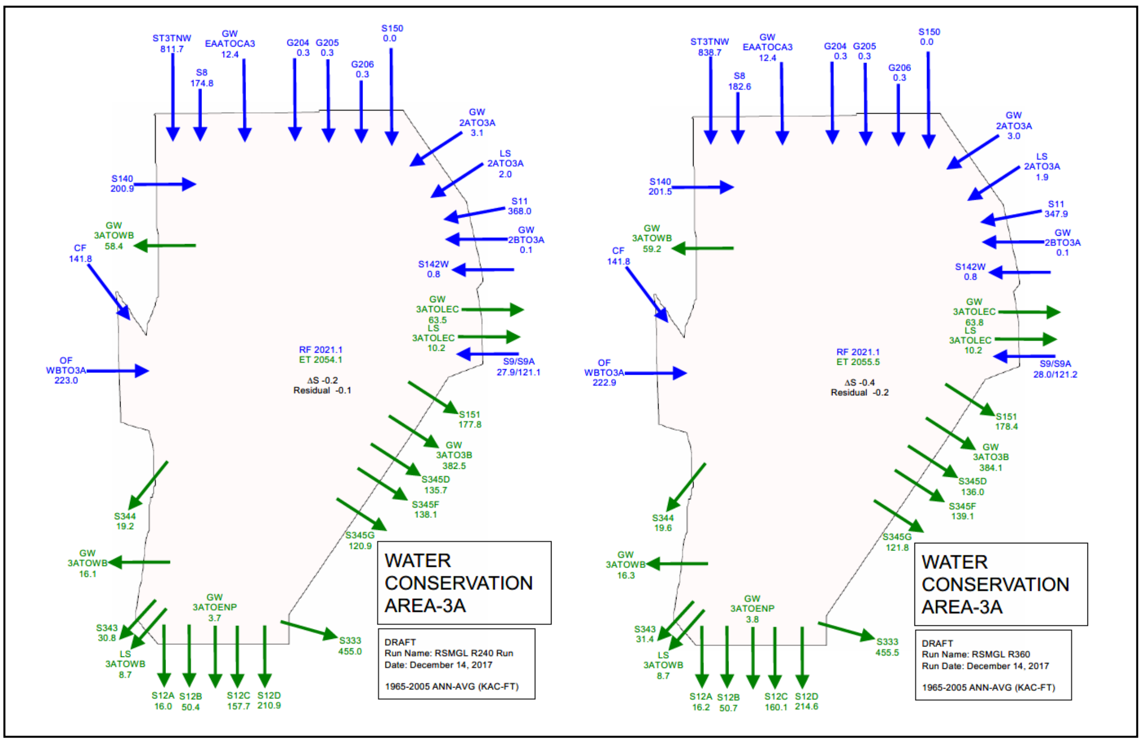


Figure 4.7. Average annual water budget flows (kac-ft) for WCA3A between ALT R240 (left) and ALT R360 (right)

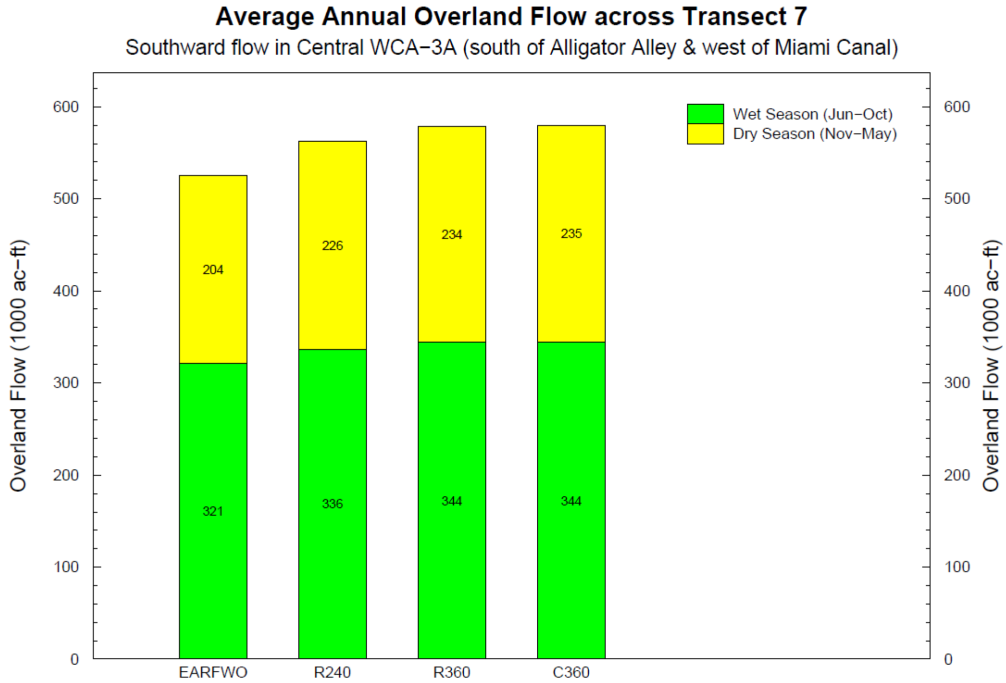


Figure 4.8. Flow differences between EARFWO and Alternatives in central WCA3A

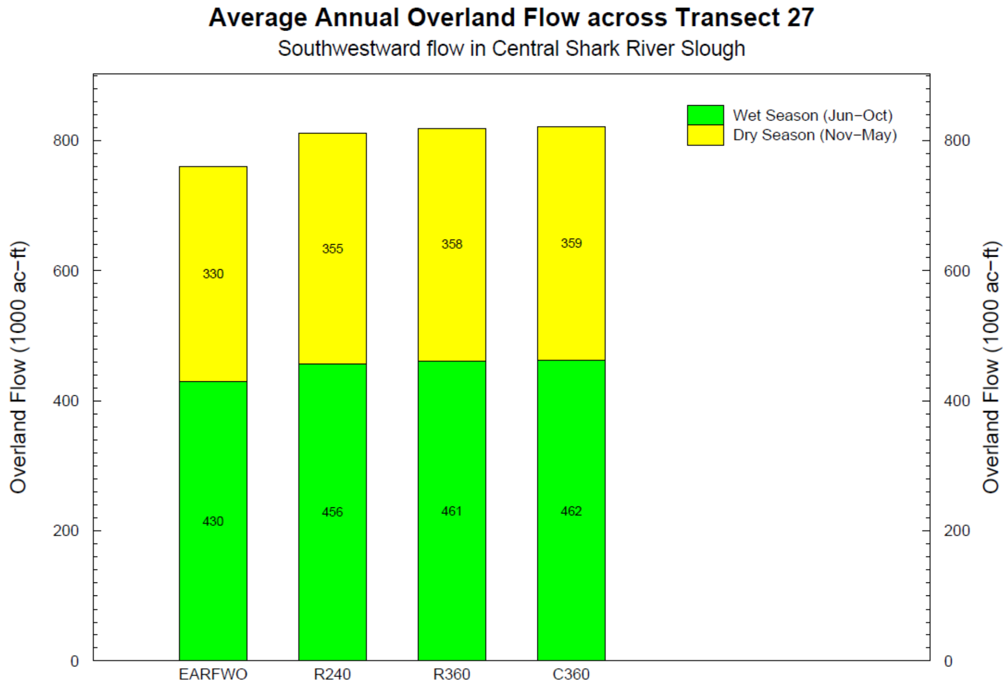


Figure 4.9. Flow differences between EARFWO and Alternatives in Central Shark River Slough in ENP

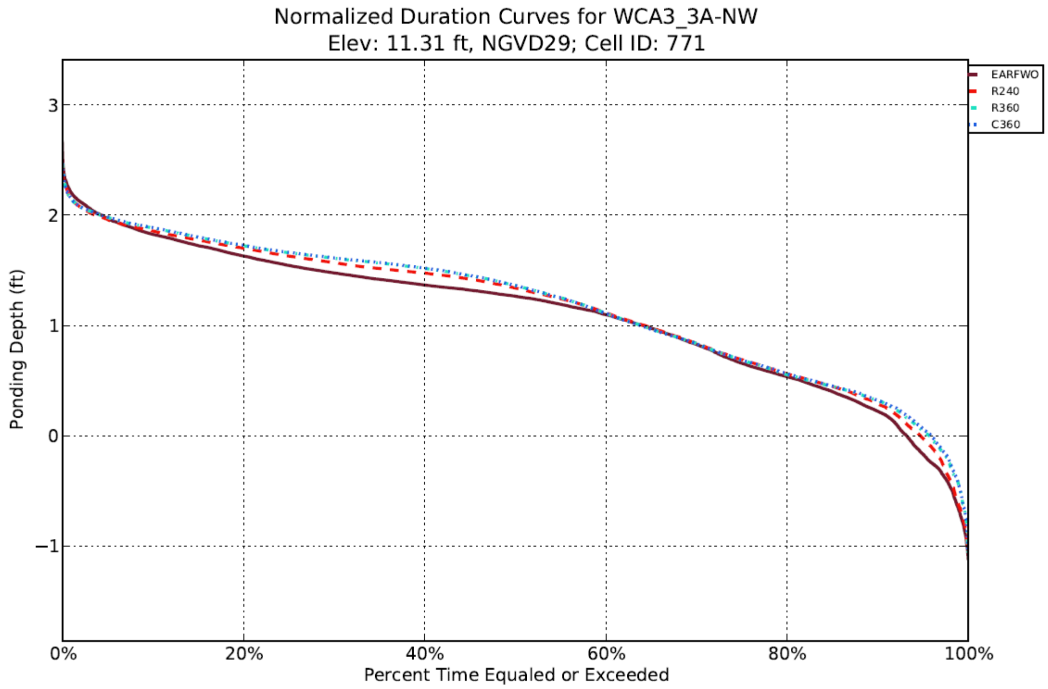


Figure 4.10. Depth differences between EARFWO and Alternatives in Northwest WCA3A

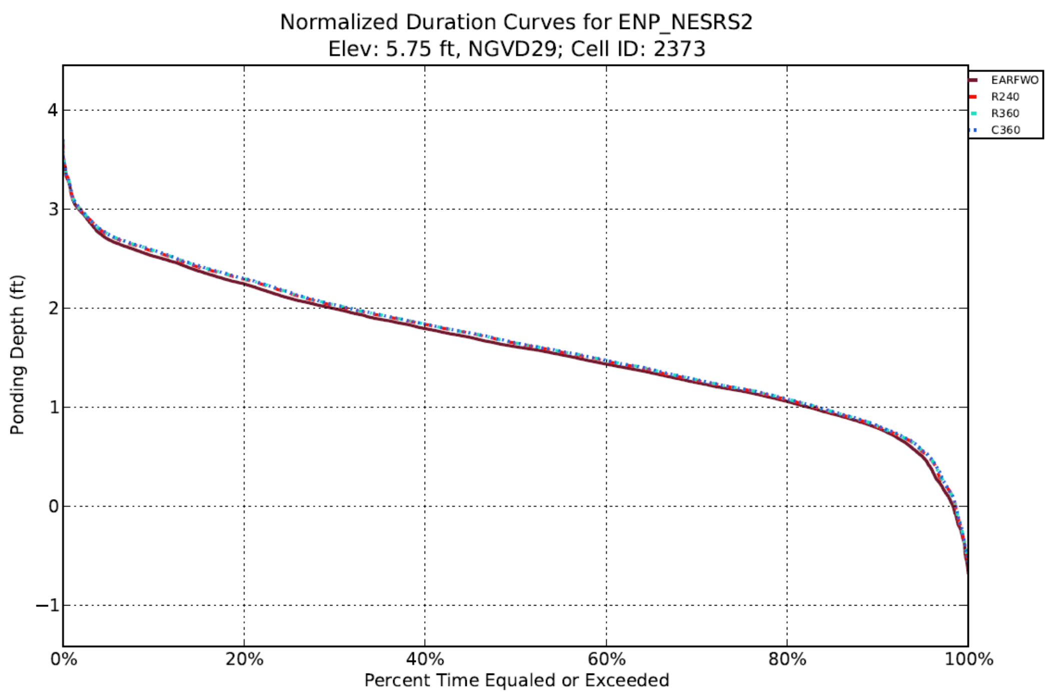


Figure 4.11. Depth differences between EARFWO and Alternatives in Northeast ENP

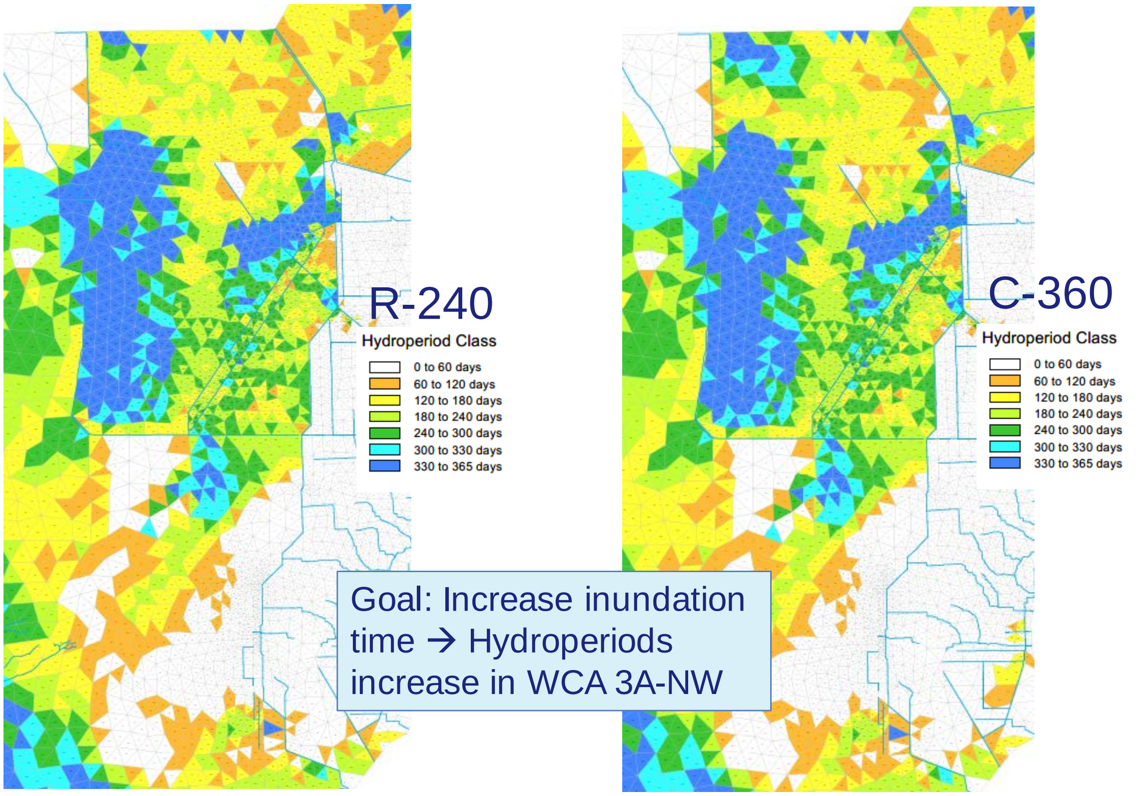


Figure 4.12. Example timing improvements in C360 hydroperiods in a dry year (1989)

In summary, the three delivered alternative runs provided to the EAASR project team are deemed to adequately represent the intended planning conditions and when utilized in conjunction with proposed EAASR project baselines, provide a reasonable basis of comparison for the necessary evaluations required to draft the PACR.

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Appendix A – Details of Alternatives

Figure A.1 shows the differences between the final array alternatives of the Everglades Agricultural Area Storage Reservoir Project. Details of Alternatives ALT R240A, ALT R240B, ALT R360C, ALT R360D, and ALT C360 are shown in **Figures A.2, A.3, A.4, A.5 and A.6** respectively.

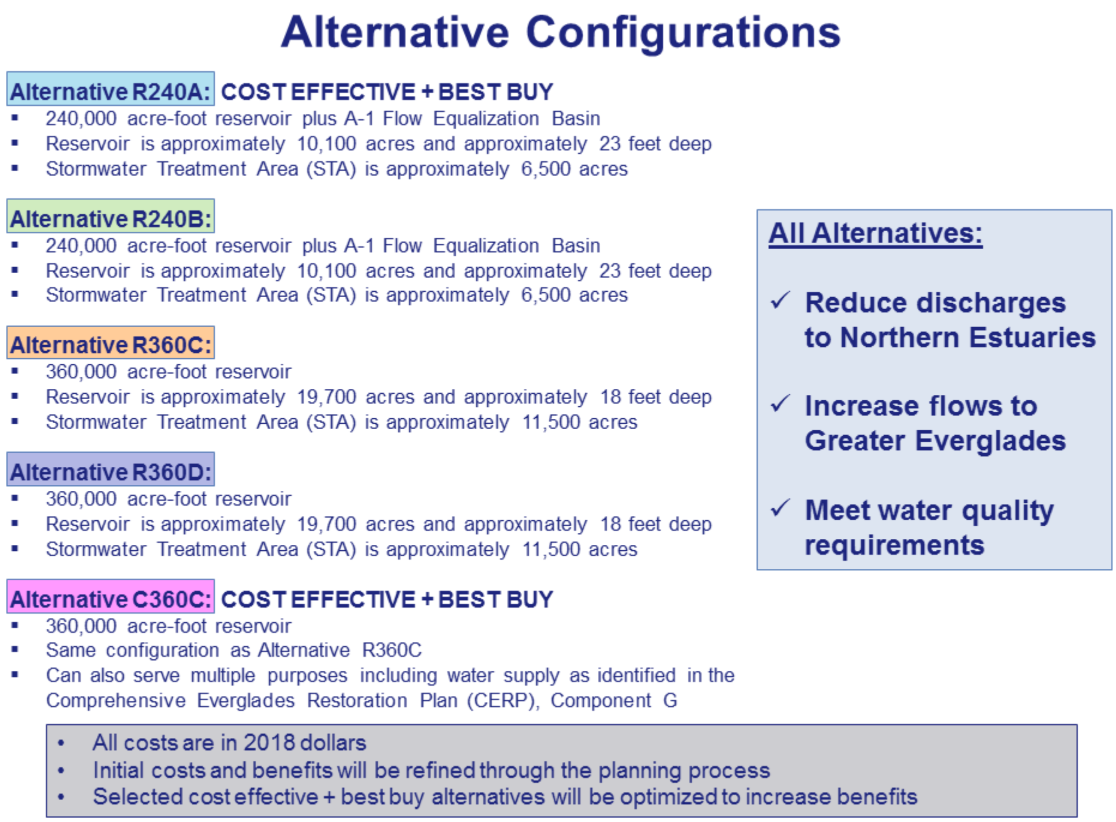


Figure A.1. Differences between the final array alternatives of the Everglades Agricultural Area Storage Reservoir Project.

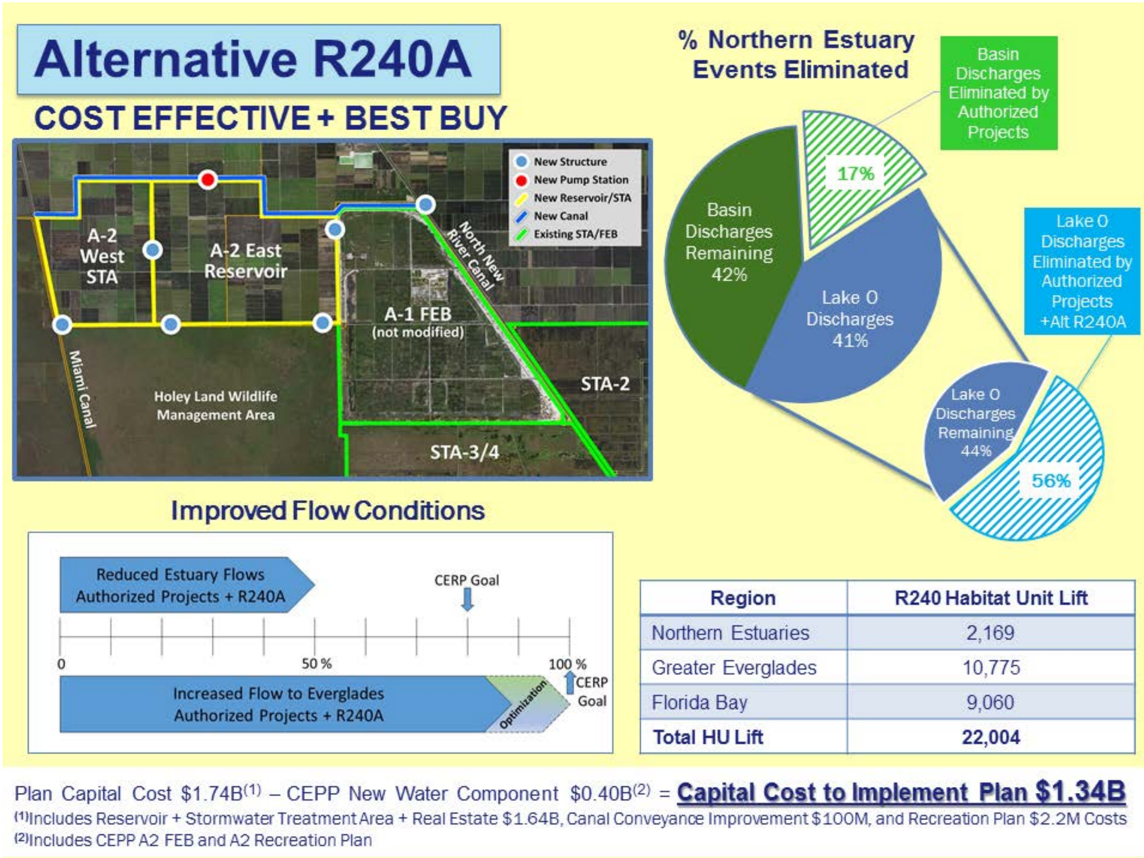
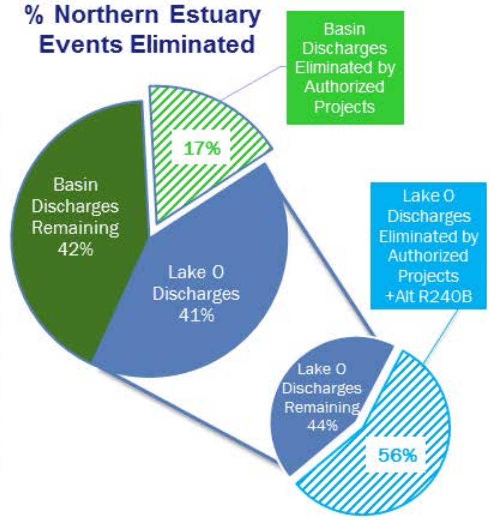
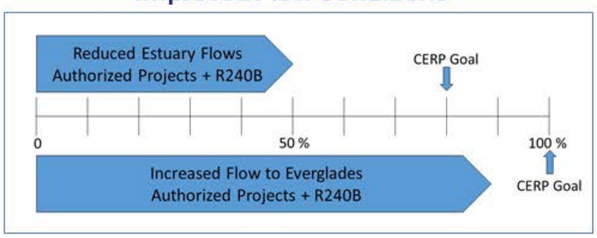


Figure A.2. Proposed alternative ALT R240A

Alternative R240B



Improved Flow Conditions



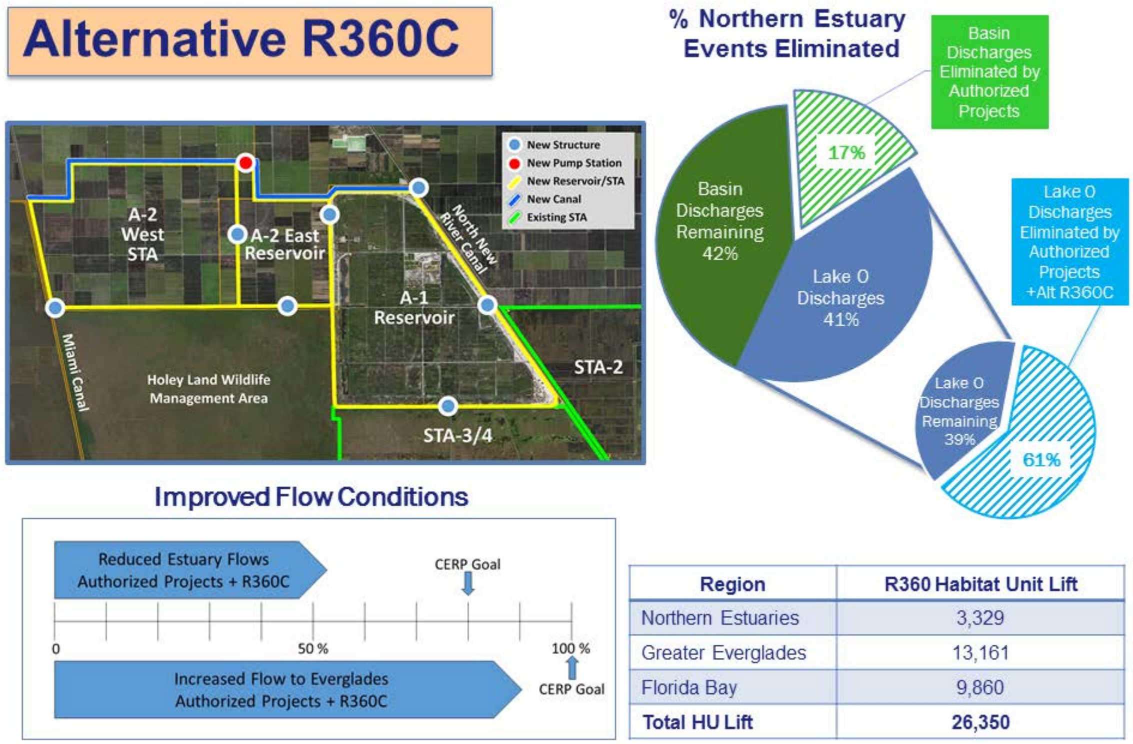
Region	R240 Habitat Unit Lift
Northern Estuaries	2,169
Greater Everglades	10,775
Florida Bay	9,060
Total HU Lift	**22,004**

Plan Capital Cost $1.76B[(1)] – CEPP New Water Component $0.40B[(2)] = **Capital Cost to Implement Plan $1.36B**

[(1)]Includes Reservoir + Stormwater Treatment Area + Real Estate $1.66B, Canal Conveyance Improvement $100M, and Recreation Plan $2.2M Costs

[(2)]Includes CEPP A2 FEB and A2 Recreation Plan

Figure A.3. Proposed alternative ALT R240B



Region	R360 Habitat Unit Lift
Northern Estuaries	3,329
Greater Everglades	13,161
Florida Bay	9,860
Total HU Lift	**26,350**

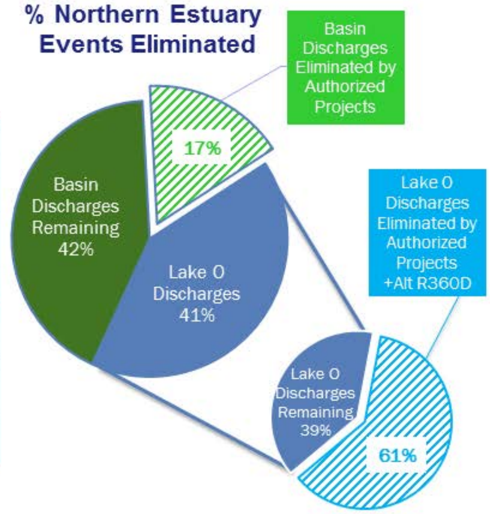
Plan Capital Cost \$2.11B[1] – CEPP New Water Component \$0.40B[2] = **Capital Cost to Implement Plan \$1.71B**

[1]Includes Reservoir + Stormwater Treatment Area + Real Estate \$2.01B, Canal Conveyance Improvement \$100M, and Recreation Plan \$2.2M Costs

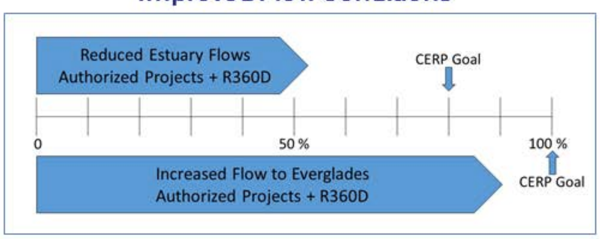
[2]Includes CEPP A2 FEB and A2 Recreation Plan

Figure A.4. Proposed alternative ALT R360C

Alternative R360D



Improved Flow Conditions



Region	R360 Habitat Unit Lift
Northern Estuaries	3,329
Greater Everglades	13,161
Florida Bay	9,860
Total HU Lift	**26,350**

Plan Capital Cost $2.11B[(1)] – CEPP New Water Component $0.40B[(2)] = **Capital Cost to Implement Plan $1.71B**

[(1)]Includes Reservoir + Stormwater Treatment Area + Real Estate $2.01B, Canal Conveyance Improvement $100M, and Recreation Plan $2.2M Costs

[(2)]Includes CEPP A2 FEB and A2 Recreation Plan

Figure A.5. Proposed alternative ALT R360D

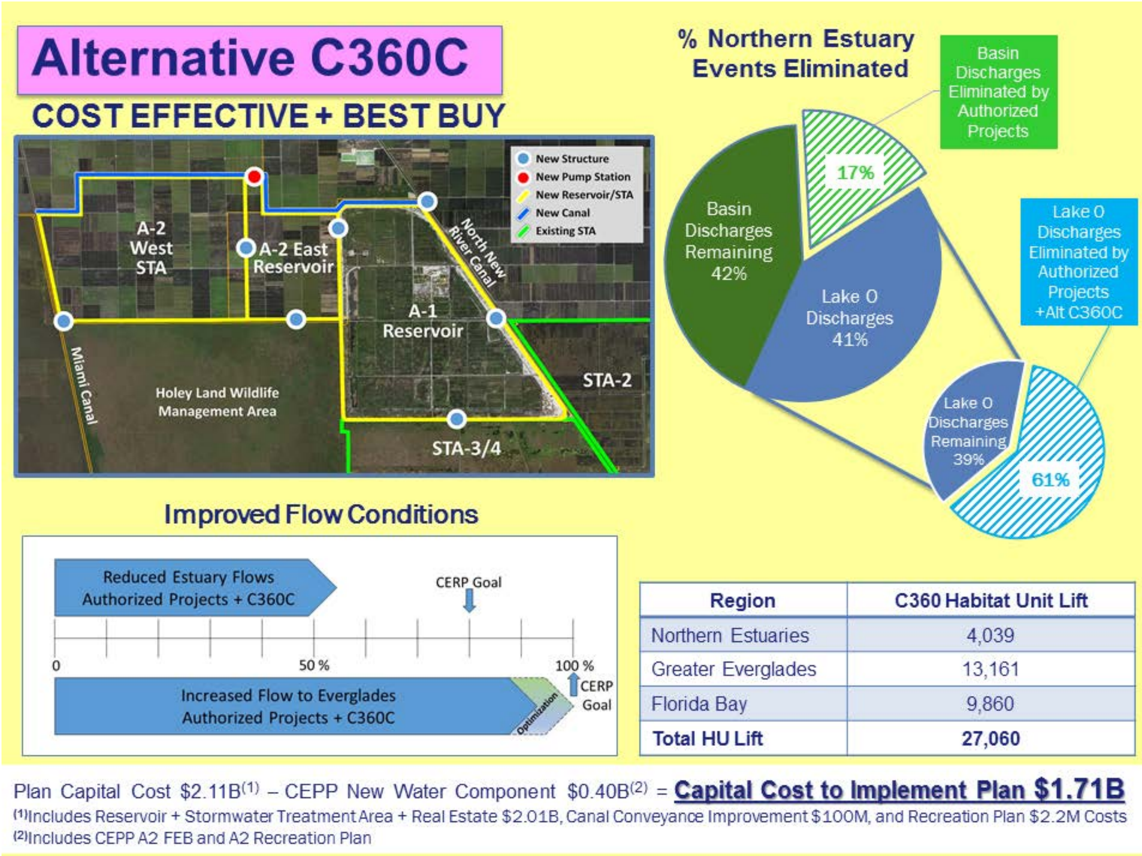


Figure A.6. Proposed alternative ALT C360C

Appendix B – Tables of Assumptions

RSMBN: R240, R360, C360
RSMGL: R240, R360, C360

Modeling Section, Hydrology & Hydraulics Bureau
South Florida Water Management District

Regional Simulation Model Basins (RSMBN)
EAA Reservoir Final Array Modeling (R240, R360 & C360)
Table of Assumptions

Feature	
Climate	• The climatic period of record is from 1965 to 2005. • Rainfall estimates have been revised and updated for 1965-2005. • Revised evapotranspiration methods have been used for 1965-2005.
Topography	The Topography dataset for RSM was Updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004; • High Accuracy Elevation Data, US Geological Survey 2007; • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004; • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007; • Palm Beach County LiDAR Survey, Dewberry and Davis, 2004; and • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/2012, as reflected in the LOSA Ledger produced by the Water Use Bureau. Project features simulated in the EAA (above and beyond the Everglades Construction Project) remove land from agricultural production. • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information. • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass.
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).

Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008)
	  o EAASR optimized release guidance in order to improve selected performance within LOK, the northern estuaries and LOSA while meeting environmental targets in the Glades.
	  o Lake Okeechobee can send flood releases south through the Miami Canal and North New River Canal to the EAA Reservoir when the LOK stage is above the bottom of Zone D1 (EAA basin runoff used to limit conveyance capacity: 2,550 cfs for Miami Canal and 1,550 cfs for North New River Canal).
	  o Lake Okeechobee can send flood releases south to help meet water-quality based flow targets at STA-3/4, STA-2N, and STA-2S when the LOK stage is above the bottom of the Baseflow Zone (EAA basin runoff used to limit conveyance capacity: 2,550 cfs for Miami Canal and 1,550 cfs for North New River Canal).

Feature	
	o Includes Lake Okeechobee regulatory releases to tide via L8 canal. o Releases via S-77 can be diverted into C43 Reservoir • Lake Okeechobee Water Shortage Management (LOWSM) Plan. • Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and S-3 is to be minimized). • "Temporary" forward pumps as follows: o S354 – 400 cfs o S351 – 600 cfs o S352 – 400 cfs o All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages recover to greater than 11.2 ft • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Headwaters Revitalization schedule for Kissimmee Chain of Lakes using the UKISS model. • Kissimmee River Restoration complete. • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012. (see land use assumptions row) • Public water supply daily intake from the river is included in the analysis. • Maximum reservoir height of 41.7 ft NGVD with a 9,379-acre footprint in Western C43 basin with a 175,800 acre-feet effective storage. • Proposed reservoir meets estuary demands while C-43 basin supplemental demands for surface water irrigation are met by Lake Okeechobee.
St. Lucie Canal Basin	• St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012 (see land use assumptions row). • Excess C-44 basin runoff is allowed to backflow into the Lake if lake stage is below 14.5 ft before being pumped into the C-44 reservoir. • Basin demands include the Florida Power & Light reservoir at Indiantown. • Indian River Lagoon South Project features • Ten-mile Creek Reservoir and STA: 7,078 acre-feet storage capacity at 10.79 maximum depth on 820 acre footprint; receives excess water from North Folk Basin; • C-44 reservoir: 50,246 acre-feet storage capacity at 5.18 feet maximum depth on 12,125 acre footprint; C44 reservoir releases water back to Lake Okeechobee when Lake stages are below the bottom of the Baseflow Zone. • C-23/C-24 reservoir: 92,094 acre-feet storage capacity at 13.27 maximum depth on 8,675 acre footprint;

Feature	
	• C-23/C-24 STA: 3,852 acre-feet storage capacity at 1.5 maximum depth on 2,568 acre footprint; • All proposed reservoirs meet estuary demands. • IRL operations assumed are consistent with the March 2010 St. Lucie River Water Reservation Rule update. • Excess C23 basin water not needed to meet estuary demands can be diverted to the C44 reservoir if capacity exists. • C44 reservoir can discharge to C44 canal and backflow to Lake Okeechobee when the lake is below the baseflow zone.
Seminole Brighton Reservation	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons per month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal rights to these quantities are preserved. • LOWSM applies to this agreement.
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM. • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved. • LOWSM applies to this agreement.
Everglades Agricultural Area	• Model water-body components as shown in Figure 1. • Simulated runoff from the North New River – Hillsboro basin apportioned based on the relative size of contributing basins via S7 route vs. S6 route. • G-341 acts as a divide between S-5A Basin and Hillsboro Basin. • RSMBN ECB EAA runoff and irrigation demand compared to SFWMM ECB simulated runoff and demand from 1965-2005 for reasonability. • For R240 scenario only: a 240 kac-ft storage reservoir located on 10,100 acre effective footprint (A2 RES) located north of Holeyland and assumed operations as follows: o A2 RES inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south (up to ~ 4ft buffer depth from full level to allow attenuation of peak EAA runoff events). o A2 RES outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater

<table>
<tr><th>Feature</th><th></th></tr>
<tr><td></td><td>
Treatment Areas) at A1FEB, STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient.

0.5 ft minimum depth below which no releases are allowed
23.5 ft maximum depth above which inflows are discontinued

For R360 and C360 scenarios only: a 360 kac-ft storage reservoir located on 19,700 acre effective footprint (A1/A2 RES) located north of STA3/4 & Holeyland and assumed operations as follows:

A1/A2 RES inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south (only until ~ 2ft buffer depth from full level; buffer retained to allow attenuation of peak EAA runoff events).
A1/A2 RES outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient.
0.5 ft minimum depth below which no releases are allowed
18.2 ft maximum depth above which inflows are discontinued
For C360 only, supplemental irrigation demands in the Miami and NNR/Hillsboro basins can be met from the reservoir when reservoir depths exceed 6.3 feet.

Inflows at the EAA reservoir inflow pump station are assumed to convey up to 3000 cfs from the Miami canal and 1500 cfs from the NNR canal (combined basin runoff and Lake O water); inflow to the EAA reservoir can also be made from the existing G370 and G372 pump stations up to a 6 ft depth.
Canal capacity is assumed to be increased by 1000 cfs in the Miami Canal and 200 cfs in the NNR canal above existing capacity to help convey water from Lake Okeechobee to the EAA Storage Reservoir.

</td></tr>
<tr><td>Everglades Construction Project Stormwater Treatment Areas</td><td>

STAs are simulated as single waterbodies
STA-1E: 6,546 acres total area
STA-1W: 7,488 acres total area
S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E
STA-2: cells 1,2 & 3: 7,681 acres total area
STA-2N: cells 4,5 & 6; refers to Comp B-North; 6,531 acres total area
STA-2S: cells 7 & 8; refers to Comp B-South; 3,570 acres total area
STA-3/4: 17,126 acres total area
STA-5N: includes cells 1 & 2: 5,081 acres total area
STA-5S: includes cells 3, 4 & 5; uses footprint of Compartment C: 8,469 acres total area
STA-6: expanded with phase 2: 3,054 acres total area

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ERSTA: Proposed STA receiving outflow from EAA deep storage reservoir and discharging to lower Miami Canal.

R240, effective area = 6,500 acres
R360 and C360, effective area = 11,500 acres

Assumed operations of STAs:

0.5 ft minimum depth below which supply from external sources is triggered;
4 ft maximum depth above which inflows are discontinued
Inflow targets established for STA-3/4, STA-2N and STA-2S based on DMSTA simulation; met from local basin runoff, LOK flood releases and available RES/FEB storage. ERSTA inflow targets based on DMSTA simulation and met by EAA reservoir storage.

For R240 scenario only: a 15,853-acre Flow Equalization Basin (A1 FEB, consistent with EARFWO) located north of STA-3/4 with assumed operations as follows:

FEB inflows are from the A2 RES and are consistent with established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas). FEB inflows are limited to 500 cfs when depths are above 2.5 ft.
FEB outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient.
0.5 ft minimum depth below which no releases are allowed
3.8 ft maximum depth above which inflows are discontinued
Assumed inlet structure of 1500 cfs capacity from A2 RES for modeling purposes.
Outflow weirs, with similar discharge characteristics as STA-3/4 outlet structure, discharging into lower North New River canal.
Structure capacities and water quality operating rules are consistent with modeling assumptions assumed during the A-1 FEB EIS application process.

</td></tr>
<tr><td>Holeyland Wildlife Management Area</td><td>

G-372HL is the only inflow structure for Holeyland used for keeping the water table from going lower than half a foot below land surface elevation.
Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FL Fish and Wildlife Conservation (FWC) Commission and the SFWMD.

</td></tr>
<tr><td>Rotenberger Wildlife Management Area</td><td>

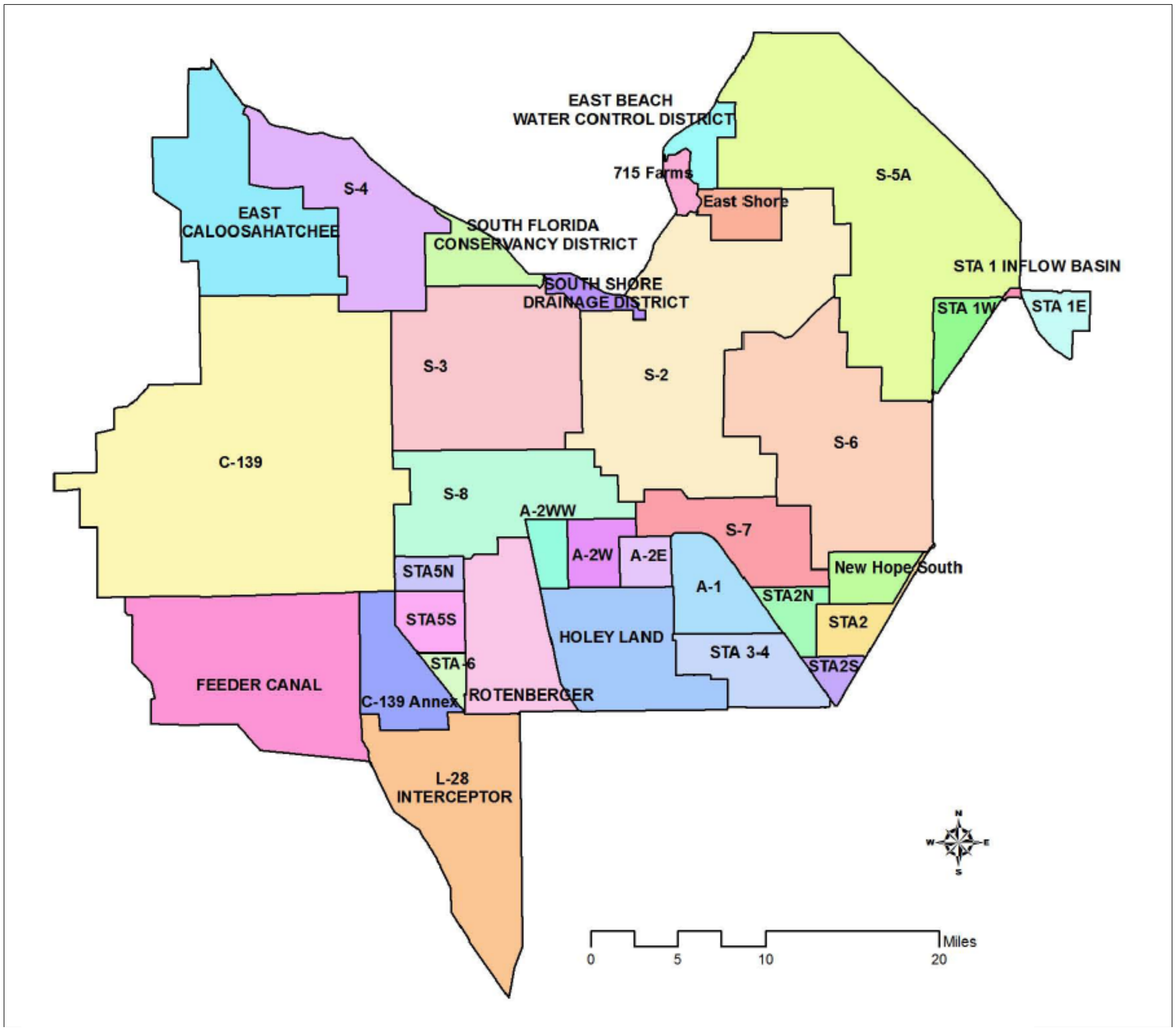
Operational Schedule as defined in the Operation Plan for Rotenberger WMA. (SFWMD, March 2010)

</td></tr>
</table>

Feature	
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL FWO.
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5N; G508 flows routed to STA5S; G406 flows routed to STA6. • C139 basin demand is met primarily by local groundwater.
Water Shortage Rules	• Reflects the existing water shortage policies as in South Florida Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan.

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the eastern and southern boundaries of the RSMBN model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Glades-LECSA Model (RSMGL). The SFWMM was the source of the eastern boundary groundwater/surface water flows, while the RSMGL was the source of the southern boundary structural flows.
- The RSMBN R240, R360 and C360 assumptions were built upon the RSMBN EARFWO scenario (11/6/17).



Water-Body Components:
Miami Water-Body = S3 + S8 + A-2WW
NNR/HILLS Water-Body = S2 + S6 + S7 + New Hope South
WPB Water-Body = S-5A
A1FEB (R240) = A-1
A2RES (R240) = Portion of A-2E, A-2W and A-2WW
A1/A2RES (R360 and C360) = A-1 + Portion of A-2E, A-2W and A-2WW
ERSTA = Portion of A-2E, A-2W and A-2WW

Figure B-1. RSMBN Basin Definition within the EAA

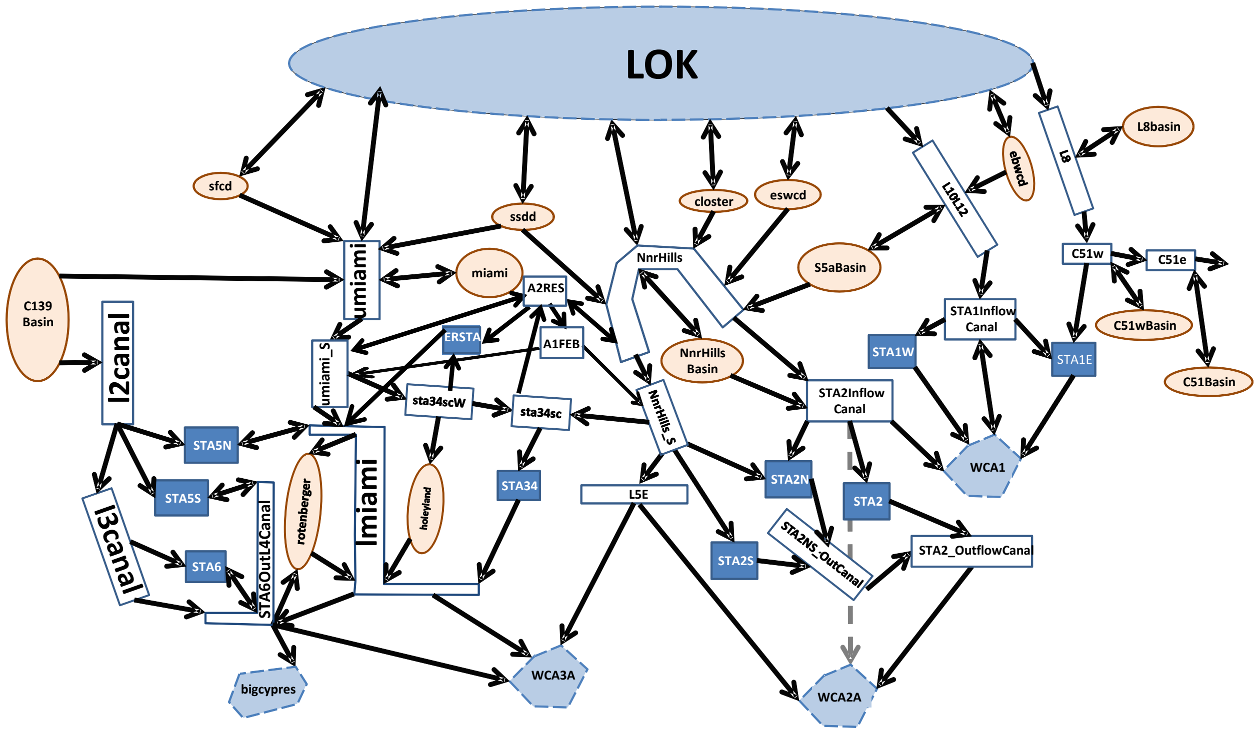


Figure B-2a. RSMBN Link-Node Routing Diagram, R240

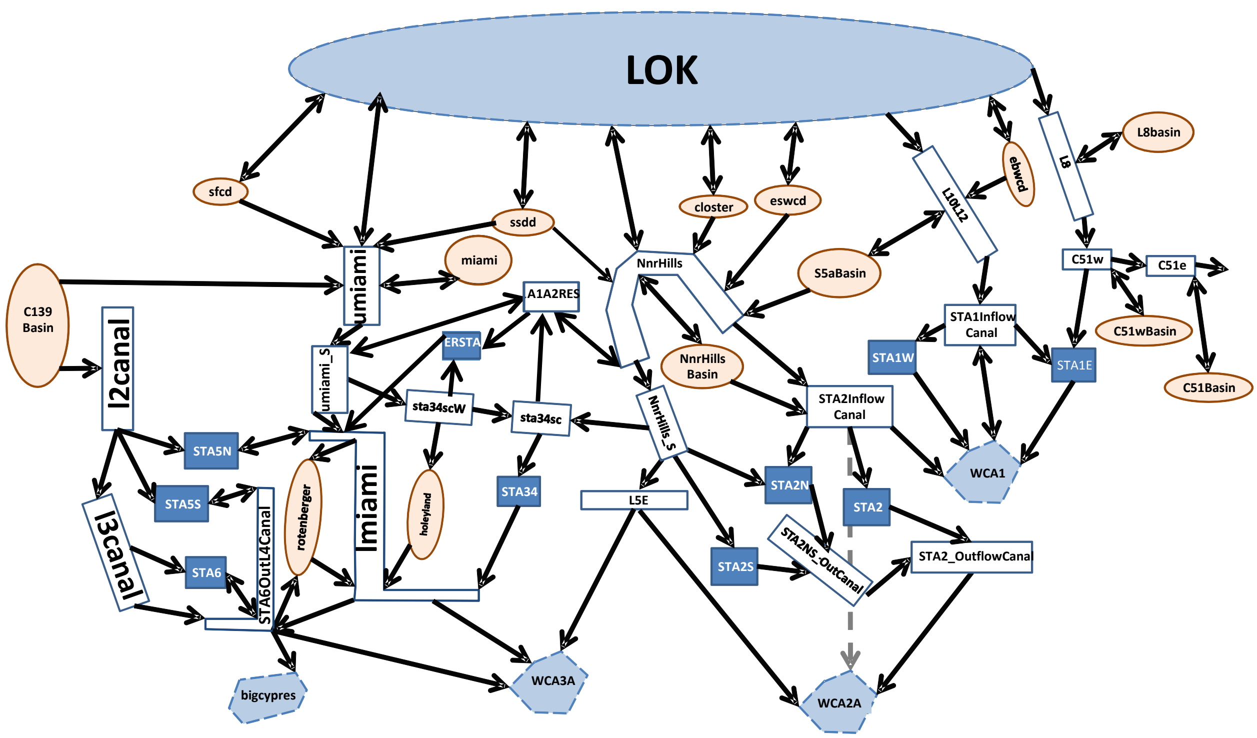


Figure B-2b. RSMBN Link-Node Routing Diagram, R360

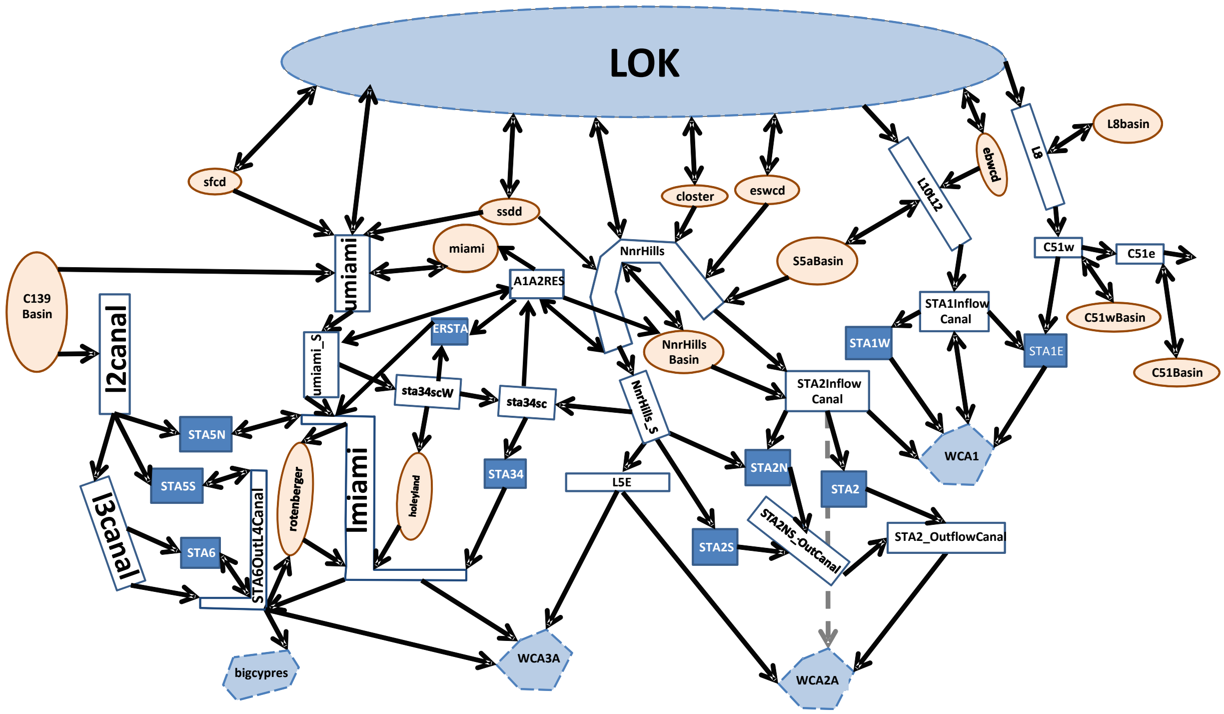


Figure B-2c. RSMBN Link-Node Routing Diagram, C360

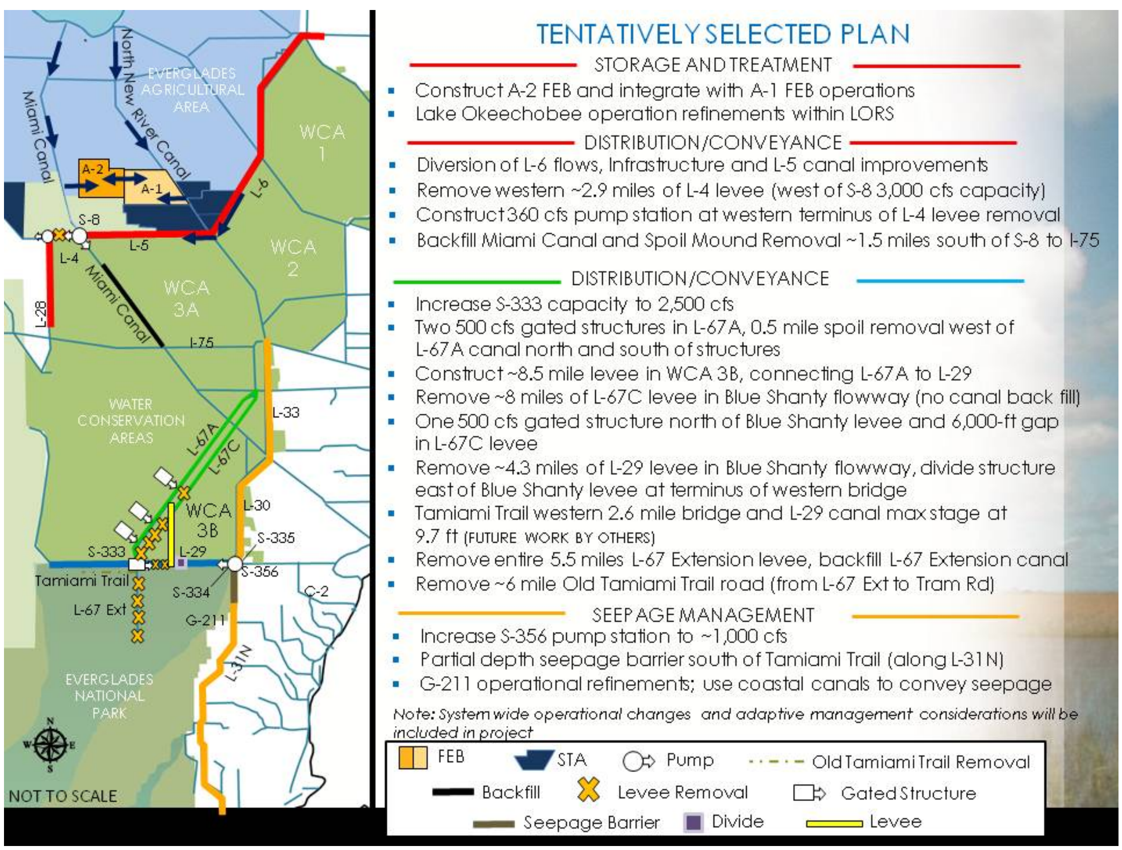


Figure B-3. CEPP ALT4R2 Features as defined by CEPP project team

Modeling Section, Hydrology & Hydraulics Bureau
South Florida Water Management District

Regional Simulation Model Glades-LECSA (RSMGL)
EAA Reservoir Final Array Modeling (R240, R360 & C360)
Table of Assumptions

Feature	
Meteorological Data	• Rainfall file used: rain_v3.0_beta_tin_14_05.bin • Reference Evapotranspiration (RET) file used: RET_48_05_MULTIQUAD_v1.0.bin (ARCADIS, 2008)
Topography	• Same as calibration topographic data set except where reservoirs are introduced (STA1-E, C4 Impoundment and C-111 reservoirs). • United States Geological Survey (USGS) High-Accuracy Elevation Data Collection (HAEDC) for the Water Conservation Areas (1, 2A, 2B, 3A, and 3B), the Big Cypress National Preserve and Everglades National Park.
Tidal Data	• Tidal data from two primary (Naples and Virginia Key) and five secondary NOAA stations (Flamingo, Everglades, Palm Beach, Delray Beach and Hollywood Beach) were used to generate a historic record to be used as sea level boundary conditions for the entire simulation period.
Land Use and Land Cover	• Land Use and Land Cover Classification for the Lower East Coast urban areas (east of the Lower East Coast Flood Protection Levee) use 2008-2009 Land Use coverage as prepared by the SFWMD, consumptive use permits as of 2011 were used to update the land use in areas where it did not reflect the permit information. • Land Use and Land Cover Classification for the natural areas (west of the Lower East Coast Flood Protection Levee) is the same as the Calibration Land Use and Land Cover Classification for that area. Modified at locations where reservoirs are introduced (STA1-E, Site 1 Impoundment, Broward WPAs, C4 Impoundment, Lakebelt Lakes and C-111 Reservoirs).
Water Control Districts (WCDs)	• Water Control Districts in Palm Beach and Broward Counties and in the Western Basins assumed. • 8.5 SMA seepage canal is modeled as a WCD in ENP area.
Lake Belt Lakes	• Based on the permitted 2020 Lake Belt Lakes coverage obtained from USACE.
CERP Projects	• 1st Generation CERP – Site 1 Impoundment project is modeled as an above ground reservoir of area 1600 acres, with a maximum depth of 8 ft. • 2nd Generation CERP – Broward County Water Preserve Areas (WPAs) comprised of C-11 and C-9 impoundments were modeled as above ground reservoirs with areas 1221 and 1971 acres and maximum depths 4.3 and 4.0 ft. respectively. Operations refined in RSM model to closer represent project intent and outcomes. • 2nd Generation CERP – C-111 Spreader Canal Project includes the Frog Pond Detention Area, which is modeled as an above ground impoundment with the S200 A, B and C pumps as inflow

Feature	
	structures. In addition, the Aerojet canal is modeled with the inflow pumps S199 A, B and C. The S199 and S200 pumps are turned off based on the stage at the remote monitoring location EVER4 for the protection of the CSS Critical Habitat Unit 3. • 2nd Generation CERP – Biscayne Bay Coastal Wetlands project features were not modeled since these features along the coast in Miami-Dade County were not considered significant for CEPP. • Areal corrections were applied to the impoundment storages to account for the discrepancies of the areas in the model of the impoundments not matching the design areas.
Water Conservation Area 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge)	• Current C&SF Regulation Schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 14 ft. The bottom floor of the schedule (Zone C) is the area below 14 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow. • Structure S10E connecting LNWR to the northeastern portion of WCA-2A is no longer considered part of the simulated regional System
Water Conservation Area 2A & 2B	• Current C&SF regulation schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 10.5 ft in WCA-2A, defined as when WCA2-U1 marsh gauge falls below 10.5 ft or L38 canal stage falls below 10.0 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Water Conservation Area 3A & 3B	• Diversion of L-6 flows with additional 500 cfs structure and improvements to the L-5 canal • STA-3/4 outflows routed based on Rainfall Driven Operations (RDO) – a maximum of 2500 cfs is routed to S8 and G404, with the remainder being sent to S7 • Western L-4 levee degrade with 1.5 miles retained west of S8 (west of S-8 = 3,000 cfs capacity) • Miami Canal backfilled and spoil mound removed 1.5 miles south of S-8 to I-75 • Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 (USACE, 2012) • One 500 cfs gated structure in L-67A north of Blue Shanty levee (S345D) and associated gap in L-67C levee • Two 500 cfs gated structures in L-67A (S345F & S345G) discharging into Blue Shanty Flowway • Environmental target deliveries through the S345s are determined through RDO and is spatially distributed as 40% to 345D, 35% to 345F and 25% to 345G

<table>
<tr><th>Feature</th><th></th></tr>
<tr><td></td><td>Blue Shanty Flowway assumed as follows:
Construction of ~8.5 mile levee in WCA 3B, connecting L-67A to L-29
Removal of L-67C levee in Blue Shanty Flowway (no canal back fill)
Removal of L-29 levee in Blue Shanty Flowway.
Includes regulatory releases to tide through LEC canals. Documented in Water Control Plan (USACE, June 2002)
No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 7.5 ft in WCA-3A, defined as when 3-69W marsh gauge falls below 7.5 ft or CA3 canal stage falls below 7.0 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.</td></tr>
<tr><td>Everglades Construction Project Stormwater Treatment Areas</td><td>STA-1E: 5,132 acres total treatment area.
A uniform bottom elevation equal to the spatial average over the extent of STA-1E is assumed.</td></tr>
<tr><td>Everglades National Park</td><td>Water deliveries to Everglades National Park are based upon Everglades Restoration Transition Plan (ERTP), with the WCA-3A Regulation Schedule including the lowered Zone A (compared to IOP) and extended Zones D and E1. The environmental component of the schedule is defined by RDO. If hydraulic capacity exists at the 345s, then flood control discharges are made into 3B instead of at the S12s.
S-333 capacity increased to 2,500 cfs
L29 Divide structure assumed and is operated to send water from L29W to L29E to equilibrate canals when L29E falls below 7 ft.
L29 canal can receive inflow up to 9.7 ft (applies to both E and W segments / i.e. S333 & S356 as well as S345F & S345G structure on Blue Shanty Flowway)
G-3273 constraint for operation of S-333 assumed to be 9.5 ft, NGVD.
The one mile Tamiami Trail Bridge as per the 2008 Tamiami Trail Limited Reevaluation Report is modeled as a one mile weir. Located east of the L67 extension and west of the S334 structure.
Western 2.6 mile Tamiami Trail Bridge, modeled as a 2.6 mile long weir, and is located east of Osceola Camp and west of Frog City.
Tamiami Trail culverts east of the L67 Extension are simulated where the bridge is not located.
Removal of the entire 5.5 miles L-67 Extension levee, with backfill of L-67 Extension canal
S-355A & S-355B are operated.
Capacity of S-356 pump increased to 1000 cfs. S-356 is operated to manage seepage.</td></tr>
</table>

Feature	
	• Full construction of C-111 project reservoirs consistent with the as-built information from USACE plus addition of contract 8 and contract 9 features. A uniform bottom elevation equal to the spatial average over the extent of each reservoir is assumed. • 8.5 SMA project feature as per federally authorized Alternative 6D of the MWD/8.5 SMA Project (USACE, 2000 GRR); operations per 2011 Interim Operating Criteria (USACE, June 2011) including S-331 trigger shifted from Angel's well to LPG-2. Outflow assumed from 8.5 SMA detention cell to the C-111 North Detention Area. o An additional length of seepage canal is assumed in the model to allow water to be collected for S357 operation. • Partial depth, approximately 4 mile long seepage barrier south of Tamiami Trail (along L-31N)
Other Natural Areas	• Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay
Pumpage and Irrigation	• Public Water Supply pumpage for the Lower East Coast was updated using 2010 consumptive use permit information as documented in the C-51 Reservoir Feasibility Study; permits under 0.1 MGD were not included • Modeling of the TSP assumes an additional public water supply withdrawal of 12 MGD in Service Area 2 and 5 MGD in Service Area 3. • Residential Self Supported (RSS) pumpage are based on 2030 projections of residential population from the SFWMD Water Supply Bureau. • Industrial pumpage is also based on 2030 projections of industrial use from the Water Supply Bureau. • Irrigation demands for the six irrigation land-use types are calculated internally by the model. • Seminole Hollywood Reservation demands are set forth under VI. C of the Tribal Rights Compact. Tribal sources of water supply include various bulk sale agreements with municipal service suppliers.
Canal Operations	• C&SF system and operating rules in effect in 2012 • Includes operations to meet control elevations in the primary coastal canals for the prevention of saltwater intrusion • Includes existing secondary drainage/water supply system • C-4 Flood Mitigation Project • Western C-4, S-380 structure retained open • C-11 Water Quality Treatment Critical Project (S-381 and S-9A) • S-25B and S-26 backflow pumps are not modeled since they are used very rarely during high tide conditions and the model uses a long-term average daily tidal boundary • Northwest Dade Lake Belt area assumes that the conditions caused by currently permitted mining exist and that the effects of any future mining are fully mitigated by industry • ACME Basin A flood control discharges are sent to C-51, west of the S-155A structure, to be pumped into STA-1E. ACME Basin B flood control discharges are sent to STA-1E through the S-319 structure

Feature	
	• Releases from WCA-3A to ENP and the South Dade Conveyance System (SDCS) will follow the Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 o Structures S-343A, S-343B, S-344 and S-12A are closed Nov. 1 to July 15 o Structure S-12B is closed Jan. 1 to July 15 • Water supply deliveries from regional system (from WCA3A: S-151/S-337) are used to maintain the L30 canal with a minimum seasonal level varying from 6.25 ft in the dry season to 5.2 ft. at the beginning of the wet season • G-211 / S338 operational refinements; use coastal canals to convey seepage toward Biscayne Bay during drier times.
Canal Configuration	• Canal configuration same as calibration except no L-67 Extension Canal and CERP & CEPP project modifications.
Lower East Coast Service Area Water Shortage Management	• Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data. • Periods where the Lower East Coast is under water restriction due to low Lake Okeechobee stages were extracted from the corresponding RSMBN simulation.

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.
- The RSMGL R240, R360 and C360 assumptions were built upon the RSMGL EARFWO scenario (11/6/17), with the only changes being updated northern boundary inflows from the corresponding RSMBN scenarios.

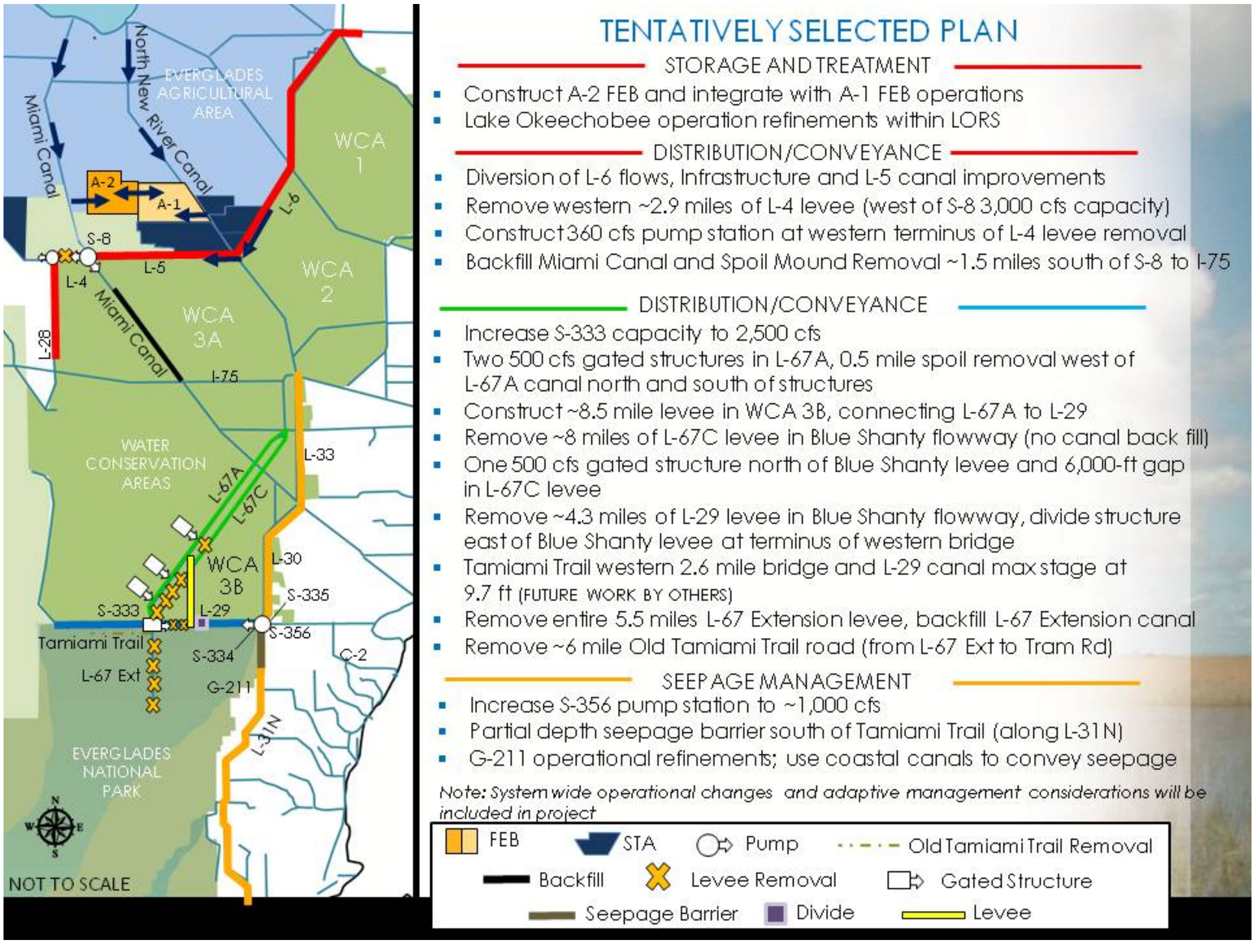


Figure B-4. CEPP ALT4R2 Features as defined by CEPP project team

Appendix C – Structure Operations in South Miami-Dade County for EAASR FWO Baseline, Final Array Runs, and TSP

In **Table C.1**, the list of structures is color-coded in three groupings:

green	Structures on the L-67, L-28, and L-29 canals. Structures included are S-345, S-349, S-344, S-343A-B, S-12A-D, S-333, S-334, S-355, and S-356
blue	Structures on the L-30 and part of the L-31 canals Structures included are S-337, S-151, S-335, S-338, G-211, S-173 & S-331P (COMBQ), S-176 and S-174
yellow	Structures on part of the L-31 canal and L-31W and C-111 canals Structures included are S-332A-D, S-357, S-332, S-175, S-200, S-199, S-177, S-18C, S-197 and S-332E

Table C.1 includes the Future Without Baseline (FWO), and the final array alternatives and TSP (same operations for alternatives ALT R240, ALT R360, and ALT C360 and for TSP ALT C240).

Table C.1. Existing Condition (EARECB) and Future Condition (EARFWO, ALT R240, ALT R360, and ALT C360 and TSP ALT C240)

Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations
L-67	S-345*	Non-existent	S345D, S345F & S345G 3 gated spillway at L-67A Design Q= 500 cfs each flood control only
	S-349*	Non-existent	Non-existent
L-28	S-344*	Special code Design Q=250 cfs 1) Closed Nov 1- Jul 15 2) flood control only	Special code Design Q=250 cfs 1) Closed Nov 1- Jul 15 2) flood control only
	S-343A-B*	Special code 1) Closed Nov 1- Jul15 2) flood control only 3) S343A&B- Design Q= 200 cfs each	Special code 1) Closed Nov 1- Jul15 2) flood control only 3) S343A&B- Design Q= 200 cfs each
L-29	S-12A-D*	per ERTP S12A closed Nov 1 to Jul 31; S12B closed Jan 1 to Jul 31; S12C no closure dates. S12D no closure dates. Special code 1) S12s = 8000 cfs per structure. 2) Each structure modeled individually. 3) Each Structure is a spillway 4) Flood Control only	per ERTP S12A closed Nov 1 to Jul 31; S12B closed Jan 1 to Jul 31; S12C no closure dates. S12D no closure dates. Special code 1) S12s = 8000 cfs per structure. 2) Each structure modeled individually. 3) Each Structure is a spillway 4) Flood Control only
	S-333*	Special code 1) L-29 stage constraint of 7.5 Wet/Dry 2) Design Q=1350 cfs 3) G-3273 stage constraint of 6.8 4) Flood Control only	Special code (S333 has higher priority over S12s) 1) L-29 canal max stage of 9.7 Wet/Dry 2) Design Q=2500 cfs 3) G-3273 stage constraint of 9.5 4) Flood Control only
	S-334	Non-existent Special code 1) Flood Control 2) No open/close ops, structure flow is based on L31N stage 4) Design Q=1230 cfs	Non-existent No IOP wraparound operations. So, no flow through S334
	S-355*	Special code 1) S355 A and B Modeled Design Q = 1000 cfs each, 2) L-29 Max stage of 7.5' 3) Flood control only 4)G-3273 stage constraint of 6.8 5)L-29 stage constraint of 7.5	Special code 1) S355 A and B Modeled Design Q = 1000 cfs each, 2) L-29 Max stage of 9.7 3) Flood control only 4)G-3273 stage constraint of 9.5
	S-356*	Not operational	**6.0/5.5** open/closed wet season **6.0/5.8** open/closed dry season 1) Design Q = 1000 cfs 2) Flood control only

<table>
<tr><td rowspan="2">Canal</td><td rowspan="2">Structure</td><td>EARECB (RSMGL)</td><td>EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)</td></tr>
<tr><td>Open/Close (ft NGVD)
(Optimum stage ft NGVD)
Wet /Dry Season Normal FC Operations</td><td>Open/Close (ft NGVD)
(Optimum stage ft NGVD)
Wet /Dry Season Normal FC Operations</td></tr>
<tr><td rowspan="4">L-30</td><td rowspan="2">S337</td><td>1) Water Supply / Flood Control
2) Design Q=1100 cfs (discharge coef = 1053 cfs in msestruc*.xml)</td><td>1) Water Supply / Flood Control
2) Design Q=1100 cfs (discharge coef = 1053 cfs in msestruc*.xml)</td></tr>
<tr><td>mse_unit inlet L30; L30 localLevel=7.0
mse_unit outlet C-304; C304 localLevel=99.0

S337_HWi & S337TWi (input variable for WCA3A_WCA3B_regulatory in Special_Assessors_2050FWO.so)

S337_FracGO & S337_FracGo_high & S337_FracGo_low variable output maxfracS12s xml</td><td>mse_unit inlet L30; L30 maintLevel from rc id=993110
maintLevel(01 Nov-31 May=6.45. 01 Jun-31 Oct =5.4)
L30 resLevel from rc id = 993111
resLevel(01 Nov-31 May=6.25. 01 Jun-31 Oct=5.2)
mse_unit outlet C-304; C304 localLevel=99.0

S337_HWi & S337TWi (input variable for WCA3A_WCA3B_regulatory in Special_Assessors_ALT4R.so)

S337_FracGO & S337_FracGo_high & S337_FracGo_low variable output maxfracS12s xml</td></tr>
<tr><td>S-151*</td><td>Flow target based on WCA-3A regulation schedule

1) Water Supply / Flood Control
2) Design Q=1800 cfs (discharge coef. = 1154.48 in msestruc*.xml)

mse_unit outlet for WCA-3A; "WCA3A local" localLevel = 7.5
mse_unit inlet C-304; C304 localLevel=99.0

Special Code for S151:
S151_reg_max_zoneA=1000 (input variable Special_Assessors_2050FWO.so)
S151_reg_max_zoneBC=500 (input variable Special_Assessors_2050FWO.so)
S151_TWi & S151_HWi (input variable Special_Assessors_2050FWO.so)
S151_FracGO & S151_FracGo_high & S151_FracGo_low variable output maxfracS12s xml</td><td>Flow target based on WCA-3A regulation schedule</td></tr>
<tr><td>S-335</td><td>7.5 / 7.2 open/closed wet & dry season
1) Water Supply / Flood Control
2) Design Q=1170 (discharge coef. = 1468 cfs in msestruc*.xml)
3) twHeadLimit name "S335 twHeadLimit" = 6.0
mse_unit outlet for L30; L30 localLevel=7.0
mse_unit inlet for L31NC; localLevel=99.0</td><td>7.6 / 7.4 open/closed wet &dry season
1) Water Supply / Flood Control
2) Design Q=1170 (discharge coef. = 1468 cfs in msestruc*.xml)
3) twHeadLimit name "S335 twHeadLimit" = 6.0
mse_unit outlet for L30; L30 maintLevel from rc ID 993110
maintLevel(01 Nov-31 May=6.45. 01 Jun-31 Oct=5.4)
L30 resLevel from rc ID=993111
resLevel(01 Nov-31 May=6.25. 01 Jun-31 Oct=5.2)
mse_unit inlet for L31NC; localLevel=99.0</td></tr>
</table>

Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations
Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations
L-31N	S-338	**5.8 / 5.5** open/closed wet & dry season 1) Water Supply / Flood Control 2) Design Q=305 (discharge coef. = 393 cfs in msestruc*xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for C1; C1 maintLevel=3.0 mse_unit inlet for L31NC; localLevel=99.0	**5.8 / 5.5** open/closed wet season **5.7 / 5.5** open/closed dry season 1) Water Supply / Flood Control 2) Design Q=305 (discharge coef. = 393 cfs in msestruc*xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for C1; C1 maintLevel=3.0
	G-211	**6.0 / 5.5** open/closed wet & dry season twHeadLimit name "G211 twHeadLimit" = 5.3 1) Water Supply / Flood Control 2) Design Q=1100 (discharge coef. = 943 cfs in msestruc*.xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for L31N; L31N localLevel=99.0 no rulecurve	**6.0 / 5.7** open/closed wet season **5.8 / 5.5** open/closed dry season twHeadLimit name "G211 twHeadLimit" = 5.3 1) Water Supply / Flood Control 2) Design Q=1100 (discharge coef. = 943 cfs in msestruc*.xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for L31N; L31N localLevel=99.0
	S-173 & S-331P (COMBQ) *	1) Water Supply / Flood Control 2) Design Q=1161 (special code Special_Assessors_ECB_2010-11.so) mse_unit outlet for L31N; localLevel = 99.0 mse_unit inlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit inlet for L31S; "L31S res" resLevel = 3.5 mse_unit outlet for L31N; mse_unit "L31N local" localLevel = 99.0 S331_TW_lim = 6.0 Operations defined in S331_ECB_2010-11.cc: S331_HW_levels = 4.0 4.5 5.0 5.5 (LPG2 stage criteria) S331 OPERATING CRITERIA: "Discharges through S-331 can be made if the S-331 tailwater stage is below 6.0 feet and the S-176 headwater stage is below 5.5 feet. If either of those water levels of S-331 and S-176 were exceeded, discharges at S-331 would be terminated until the S-176 headwater recedes to 5.0 feet." S176_Cond is dependent on S331_TW and S176_HW -> true if either stage prohibits S331 releases.	1) Water Supply / Flood Control 2) Design Q=1161 (special code Special_Assessors_ECB_2010-11.so) mse_unit outlet for L31N; localLevel = 99.0 mse_unit inlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit inlet for L31S; "L31S res" resLevel = 3.5 mse_unit outlet for L31N; mse_unit "L31N local" localLevel = 99.0 S331_TW_lim = 6.0 Operations defined in S331_ECB_2010-11.cc: S331_HW_levels = 4.0 4.5 5.0 5.5 (LPG2 stage criteria) S331 OPERATING CRITERIA: "Discharges through S-331 can be made if the S-331 tailwater stage is below 6.0 feet and the S-176 headwater stage is below 5.5 feet. If either of those water levels of S-331 and S-176 were exceeded, discharges at S-331 would be terminated until the S-176 headwater recedes to 5.0 feet." S176_Cond is dependent on S331_TW and S176_HW -> true if either stage prohibits S331 releases.

Canal	Structure	EARECB (RSMGL)	EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet/Dry Season Normal FC Operations
L-31N (cont)	S-173 & S-331P (COMBQ) * (cont)	S331 High Range: If the water level at LPG2 well is < 5.5 ft, S331 HW will have no limit. S331 Intermediate Range: If the level at LPG2 well is > or = 5.5 and < 6.0 ft, the average daily water level upstream of the S-331 will be maintained between 5.0 ft., and 4.5 ft if permitted by d/s conditions. S331 Low Range: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. S331 Low Range Adjustment: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are not limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. Use previous day LPG2 stage, S331_TW and S176_HW, C-357 WL for current day operations	S331 High Range: If the water level at LPG2 well is < 5.5 ft, S331 HW will have no limit. S331 Intermediate Range: If the level at LPG2 well is > or = 5.5 and < 6.0 ft, the average daily water level upstream of the S-331 will be maintained between 5.0 ft., and 4.5 ft if permitted by d/s conditions. S331 Low Range: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. S331 Low Range Adjustment: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are not limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. Use previous day LPG2 stage, S331_TW and S176_HW, C-357 WL for current day operations
	S-176	**5.0 / 4.75** open/closed wet & dry season 1) Water Supply / Flood Control 2) desigh Q = 1100 cfs (discharge coef. = 1135 cfs in msestruc*.xml) mse_unit outlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit outlet for L31S; "L31S res" resLevel = 3.5 mse_unit inlet for C111; maintenance level and reserve level determined in high_rf_events assessor S176_HW_levels 5.0 5.5 (input variable for S331 ops in Special_Assessors_ECB_2010-11.so)	**5.0 / 4.75** open/closed wet season **5.1 / 4.8** open/closed dry season 1) Water Supply / Flood Control 2) design Q = 1100 cfs (discharge coef. = 1135 cfs in msestruc*.xml) mse_unit outlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit outlet for L31S; "L31S res" resLevel = 3.5 mse_unit inlet for C111; maintenance level and reserve level determined in high_rf_events assessor S176_HW_levels 5.0 5.5 (input variable for S331 ops in Special_Assessors_ECB_2010-11.so)
	S-174*	S174 not in model; canal is blocked near structure	S174 not in model; canal is blocked near structure

Canal	Structure	EARECB (RSMGL)		EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)	
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations	
L-31N (cont)	S-332A, B,C,D (pumps)	S332A 5.0/4.7 S332B 5.0/4.7 S332BN 5.0/4.7 S332C 5.0/4.7 S332D 4.85/4.65	S332A Non-Existent S332B1 4.7/4.5 S332B2 5.0/4.7 S332BN1 4.7/4.5 S332B2 5.0/4.7 S332C1 4.7/4.5 S332C2 5.0/4.7 S332D1 4.65/4.50 S332D2 4.85/4.65	S332A Non-Existent S332B1 4.7/4.5 S332B2 5.0/4.7 S332BN1 4.7/4.5 S332B2 5.0/4.7 S332C1 4.7/4.5 S332C2 5.0/4.7 S332D1 4.65/4.50 S332D2 4.85/4.65	S332A non-existent S332B 5.0/4.7 S332BN 5.0/4.7 S332C 5.0/4.7 S332D 4.85/4.65
		S332A =300 cfs S332B =325 cfs S332BN =250cfs S332C = 575 cfs S332D = 500 cfs Jul16-Nov30, 325cfs Dec 1-Jan31, 165cfs Feb 1-Jul15 All Flood Control		S332B1=125 cfs S332B2=125 cfs S332BN1 =125cfs S332BN2=125 cfs S332C1 = 250 cfs S332C2=250 cfs S332D1 = 250 cfs S332D2 = 250 cfs Jul15-Nov30, 75cfs Dec 1-Jan31, 0cfs Feb 1-Jul14 All Flood Control	
	S-357 (pump)	**6.2 / 5.7** open/closed wet & dry season 1) Flood Control Pump Q = 126 cfs		S357A (5.7' Nov 1-May31, 5.2' Jun1-Oct31)/(5.4' Nov 1-May31, 4.9' Jun1-Oct31) S357B (6.0' Nov 1-May31, 5.5' Jun1-Oct31)/(5.7' Nov 1-May31, 5.2' Jun1-Oct31) S357A = 250 cfs S357B = 250 cfs	
L-31W	S-332 (pumps)*	Non-existent		Non-existent	
	S-175	Non-existent		Non-existent	
C-111	S-200	S-200A=75cfs; **3.8/3.6** S-200B=75cfs; **3.9/3.6** S-200C=75cfs; **4.0/3.6**		S-200A=75cfs; **3.6/3.4** S-200B=75cfs; **3.7/3.4** S-200C=75cfs; **3.8/3.4**	
	S-199	S-200A=75cfs; **3.8/3.6** S-200B=75cfs; **3.9/3.6** S-200C=75cfs; **4.0/3.6**		S-200A=75cfs; **3.8/3.6** S-200B=75cfs; **3.9/3.6** S-200C=75cfs; **4.0/3.6**	
	S-177	**4.2/3.6** (*Open/Close determined in high rainfall event Special Code.) 1) Water Supply & Flood Control Spillway w/1 gate Design Q = 2900 cfs.		**4.2/3.6** (*Open/Close determined in high rainfall event Special Code.) 1) Water Supply & Flood Control Spillway w/1 gate Design Q = 2900 cfs.	
	S-18C	**2.6 / 2.3** open/closed wet & dry season 1) Water Supply & Flood control Spillway w/2 gates Design Q=3200 cfs.		**2.6 / 2.3** open/closed wet & dry season 1) Water Supply & Flood control Spillway w/2 gates Design Q=3200 cfs.	
	S-197*	****Same as IOP,See Note 1) S197 ops see below **** 2) Flood control only 13 Culverts w/gates Design Q=6000 cfs.		****Same as IOP,See Note 1) S197 ops see below **** 2) Flood control only 13 Culverts w/gates Design Q=6000 cfs.	
	S-332E (pump)	Non-existent		Non-existent	

* SFWMM uses special code. Open/Close may or may not be used in operations.

**** S-197 Ops: Open 3 gates if S-177 full open & S-177>4.1 ft or S-18C> 2.8 ft
Open 7 gates if S-177 > 4.2 ft for 24 hrs or S-18C > 3.1 ft
Open 13 gates if S-177 > 4.3 ft or S-18C > 3.3 ft
Close when all the conditions below are met

1) S-176 < 5.2 ft and S-177 < 4.2 ft
2) Storm moved away from basin
3) After 1 and 2 are met, keep the number of S-197 culverts open necessary only to match residual flow through S-176

Modeling Section, H&H Bureau
South Florida Water Management District

EAA Storage Reservoir Project (EAASR) C240 Tentatively Selected Plan Model Documentation Report

March 2018

1.0 Overview

Identification

The Everglades Agricultural Area Storage Reservoir Project (EAASR) is an expedited planning effort undertaken as a project component of the Comprehensive Everglades Restoration Plan (CERP). This project planning effort was led by the South Florida Water Management District (SFWMD) and seeks to enhance the performance of the Central Everglades Planning Project (CEPP) which has already been authorized by Congress. The project will be designed to: 1) reduce the high-volume freshwater discharges from Lake Okeechobee to the Northern Estuaries, 2) identify storage, treatment and conveyance south of Lake Okeechobee to increase flows to the Everglades system and 3) reduce ongoing ecological damage to the Northern Estuaries and Everglades system. The project worked throughout late 2017 and early 2018 and combines planning and design activities for three primary areas of interest in the south Florida system as follows: 1) Next increment of storage and necessary treatment to provide progress towards the level of restoration envisioned for the CERP, 2) Continue to improve the quantity, quality, timing and distribution of water flows to the Northern Estuaries and central Everglades and 3) Be consistent with federal program and policy requirements. Modeling support to the EAASR effort was provided by a team comprised of modelers from the Modeling Section of the Hydrology and Hydraulics Bureau of the SFWMD.

Scope and Objectives

Modeling support for EAASR focused on working with the larger project planning team and other interested parties to formulate and test project features leading to the ultimate identification and refinement of a tentatively selected plan (TSP). Modeling products were developed at the appropriate level of detail to support feature screening and detailed representation of project features and to provide information to all necessary evaluations required for plan development and documentation. The project plan formulation framework is built upon work already completed as part of the CEPP planning effort and utilizes the same tools and techniques by performing initial screening followed by detailed evaluation to identify final project planning alternatives and ultimately a TSP for the effort.

The CEPP Modeling Strategy document (**SFWMD, 2012a**) describes the modeling process and tools utilized, the associated rationale of the selection process and the means by which the tools could expediently support the project workflow. Given that the EAASR effort is being pursued as a change to an authorized CERP project, utilization of comparable modeling strategies and tools as those used in the development of the authorized CEPP plan was a guiding principle of EAASR modeling work. The primary model support tools utilized in EAASR project refinement are as follows:

Screening Tool and Water Quality Assessment:
- Dynamic Model for Stormwater Treatment Areas (DMSTA)

Detailed Planning Models:
- Regional Simulation Model Basins (RSMBN)
- Regional Simulation Model Glades-LECSA (RSMGL)

From a modeling deliverable perspective, the entirety of the EAASR modeling support can be summarized by reviewing the following three Model Documentation Reports (MDRs):
1. EAASR Baseline – Reviews the various non-EAASR model representations (e.g., current and future without project conditions) used in various aspects of the project planning (**SFWMD, 2018a**).
2. EAASR Final Array of Alternatives – Reviews the model-supported feature screening efforts undertaken to size the reservoir and treatment facilities and detailed evaluation of three modeled “with EAASR” project model representations examined during plan formulation (**SFWMD, 2018b**)
3. EAASR Tentatively Selected Plan – Reviews the model representation of the optimized plan identified in the final steps of plan formulation and project assurance planning (this document, **SFWMD, 2018c**).

This Tentatively Selected Plan MDR describes the assumptions, model implementation steps and observed outcomes associated with the initial representation and subsequent refinement of the EAASR TSP. These model runs were predominantly used by the EAASR project team as the with-project plan representation compared back to various project baselines for various purposes. This document will focus on the modeling details of these scenarios; information on the use and rationale for the definition of these conditions is contained in the CEPP PACR (**SFWMD, 2018d**).

2.0 Basis

Project Assumptions

This Tentatively Selected Plan MDR describes the assumptions, model implementation steps and observed outcomes associated with modeling the following scenario:

EAASR Final Tentatively Selected Plan - released 1/30/2018
- 2050 Future With Project Alternative C240 (ALT C240)

Upon review of the Final Array modeling results, the EAASR project team identified the R240A and C360C alternatives (represented by the R240 and C360 model runs) as cost effective best-buy plans and an optimization effort was initiated to identify the EAASR TSP combining elements from the two best buy plans. This effort culminated in the C240 scenario and was produced with the following high level intent:

1. Largely retain engineering and footprint details from the R240 final array scenario. Review of updated engineering documents available at the time of C240 modeling indicated that an effective footprint of 6550 acres could be assumed for C240 (up from 6500 ac in R240) within the same project levee boundary.
2. Generally, adopt the operational strategy of the C360 scenario in which the EAA reservoir component is operated as a multi-purpose reservoir facility as envisioned in the original CERP plan that could meet both environmental and consumptive use water supply demands. In the final array, it was demonstrated that the C360 outperformed the R360 scenario for environmental benefit, so this operational strategy was adopted as a starting point for further optimization.
3. Increase total flow to the Everglades given updated DMSTA evaluation. In the final array work, an initial target CERP flow increase of approximately 300 kac-ft on average annually was used based on the prior CEPP narrative. Since final array outcomes identified a high potential for the EAASR project to approach CERP goals (beyond just providing an additional incremental step toward restoration), a more rigorous evaluation of CERP programmatic performance was performed to solidify the desired target flow. This analysis compared the Pre-CERP Baseline per RECOVER (**2005a**) with the CERPA scenario from the RECOVER 2005 Initial CERP Update effort (**RECOVER, 2005b**) and identified a target flow increase of 323 kac-ft on average annually based on the 36 year modeled simulation period (1965-2000) available from the RECOVER efforts. This refined number became the updated target for optimization plan formulation work toward the development of the C240 scenario.
4. Utilize improved DMSTA target time-series. In the time between final array modeling and TSP modeling, the operational schemes in DMSTA were enhanced to allow for simulation of the multi-purpose reservoir option. Additionally, the underlying DMSTA assumed hydrology was updated to use RSMBN estimates in place of previously used SFWMM outcomes. These tool improvements helped to increase the correspondence between RSM and DMSTA further justifying the assertion that modeled flow used in project benefit calculations are consistent with planning for water quality standards. In no other previous project (including CEPP) has there been this level of consistency between DMSTA and the corresponding hydrologic model (RSM or SFWMM). **(Wang, 2018)**

The modeling team in implementing the RSM scenarios relied heavily on the outcomes of the screening and optimization steps summarized in the EAASR Baselines and Final Array MDRs (**SFWMD, 2018a, 2018b**). Completion of these scenarios within the expedited project schedule would not have been possible without extensive use of DMSTA to inform operational guidance prior to detailed modeling. All project features affect inputs to the RSMBN model and the resulting flows simulated from RSMBN provide updated boundary conditions to the southern RSMGL model. Other than refined Everglades inflows, no other changes are assumed for the RSMGL model.

A detailed project assumption table for the TSP scenario is provided in **Appendix A** and key elements of model implementation are described in Section 3.

Model Limitations and Intended Use of Results

The primary modeling products of EAASR were evaluated based on outputs from the Regional Simulation Model (RSM; **SFWMD, 2005a** and **2005b**). The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g, use available input-driven options to represent more complex project operations).

As described in **Figure 2.3**, the EAASR modeling workflow strives for appropriate application of modeling tools (particularly DMSTA and RSM) for their intended use. It is neither efficient nor necessary to force intermediate modeling products to reflect a higher level of detail or consistency than is needed at that time to be robust for decision making. Along the modeling workflow, there are many opportunities for refinement. Intermediate products serve the immediate need and then are enhanced, incorporating feedback and information as the process progresses.

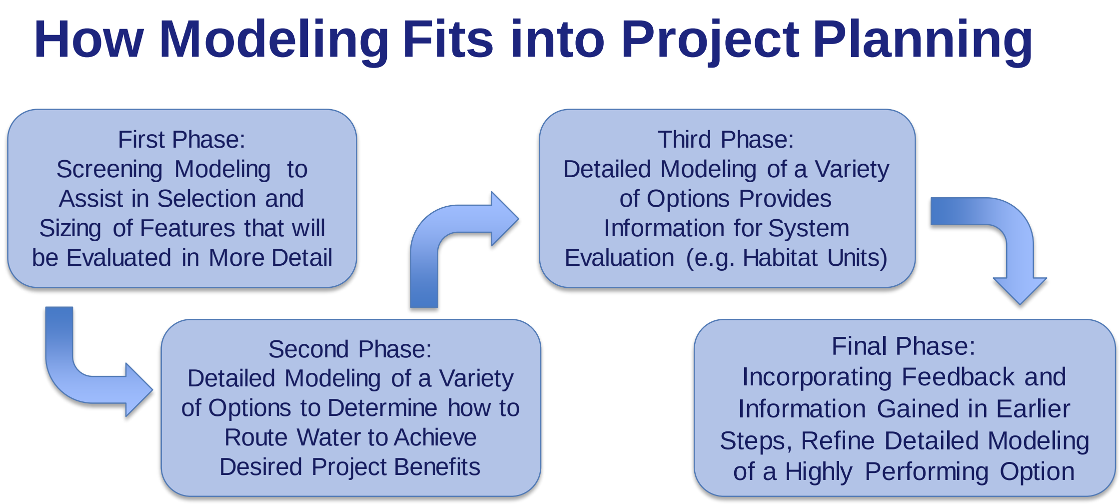


Figure 2.3. Typical EAASR Modeling Workflow

The RSMBN (**SFWMD, FDEP & FDACS, 2009a, 2009b**), RSMGL (**SFWMD, 2010** and **2011**), and DMSTA (**Walker & Kadlec, 2005**; **Wang, 2012**) models were reviewed through the USACE validation process for engineering software, as part of the CEPP project. The RSM and DMSTA models were classified as "allowed for use" for South Florida applications in August 2012 and January 2013, respectively.

3.0 Simulation

Modeling Tools Used

RSM version 2.3.5R was used to run both the RSMBN and RSMGL models.
Release date 11/10/2017, SVN Version #5207.
DMSTA v2c2b

Model Set Up

The EAASR TSP scenario was developed using the RSMBN and RSMGL models. Collectively, these two models cover the spatial extent of the project planning area as shown in **Figure 3.1**. The RSMBN modeling for the TSP scenario was built upon the EAASR ALT R240 final array representation. The RSMGL modeling for the final array was built on the EAASR ALT R240 final array representation. The RSMBN modeling for the TSP utilizes corresponding DMSTA scenarios to inform operational strategies that maintain water quality performance. The period of simulation utilizes a climate record from 1965 to 2005. As previously stated, all project features affect inputs to the RSMBN model and the resulting flows simulated from RSMBN provide updated boundary conditions to the southern RSMGL model. Other than refined Everglades inflows, no other changes are assumed for the RSMGL model. Details about project rationale for defining these scenarios can be found in the CEPP PACR (**SFWMD, 2018d**).

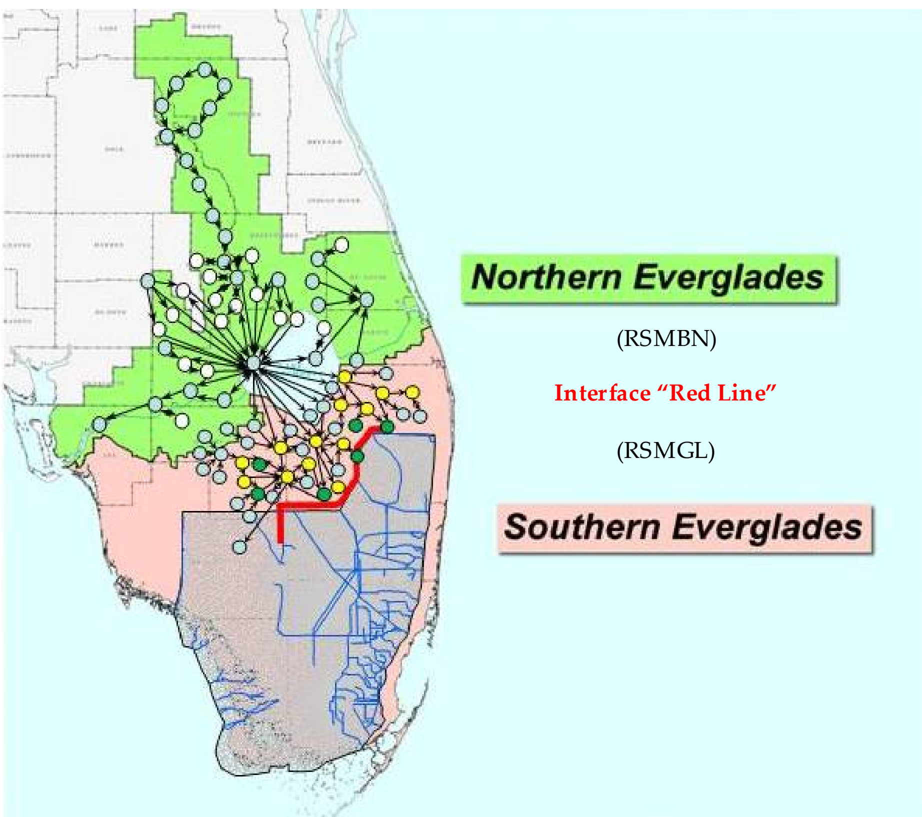


Figure 3.1. Decoupled CEPP Modeling Approach

The C240 TSP scenario assumes the following (see **Figure 3.2** for approximate component locations):

A 240 kac-ft storage reservoir located on 10,100 acre effective footprint (A2 RES) located north of Holeyland and assumed operations as follows:

- A2 RES inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south (up to ~ 4ft buffer depth from full level to allow attenuation of peak EAA runoff events).
- A2 RES outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at A1FEB, STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient.
- 0.5 ft minimum depth below which no releases are allowed
- 23.5 ft maximum depth above which inflows are discontinued
- Inflows at the reservoir inflow pump station are assumed to convey up to 3000 cfs from the Miami canal and 1500 cfs from the NNR canal (combined basin runoff and Lake O water); inflow to the EAA reservoir can also be made from the existing G370 and G372 pump stations up to a 6 ft depth.
- Supplemental irrigation demands in the Miami and NNR/Hillsboro basins can be met from the reservoir when reservoir depths exceed 8.2 feet.

A 15,853-acre Flow Equalization Basin (A1 FEB, consistent with EARFWO) located north of STA-3/4 with assumed operations as follows:

- FEB inflows are from the A2 RES and are consistent with established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas). FEB inflows are limited to 500 cfs when depths are above 2.5 ft.
- FEB outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient.
- 0.5 ft minimum depth below which no releases are allowed
- 3.8 ft maximum depth above which inflows are discontinued
- Assumed inlet structure of 1500 cfs capacity from A2 RES for modeling purposes.

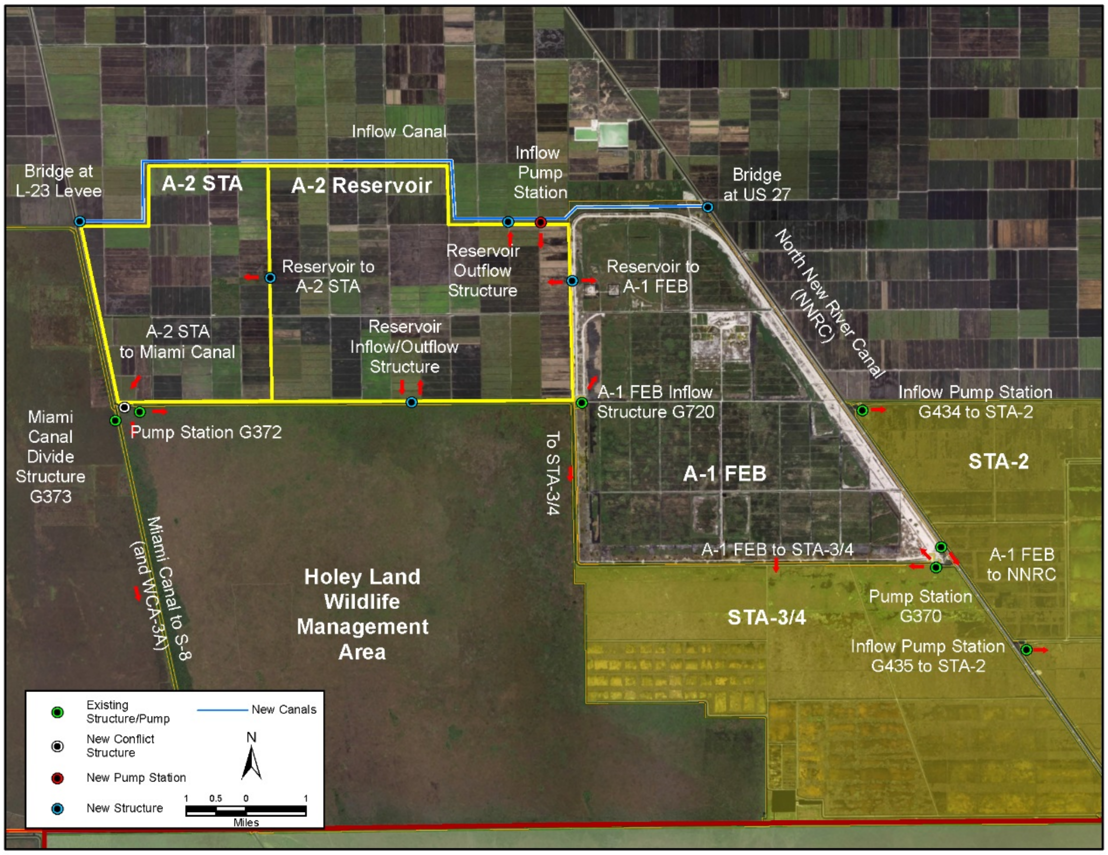


Figure 3.2. C240A Schematic Diagram Provided by EAASR Project Team

The operations of the assumed reservoir and FEB features in the C240 alternative are integrated with the regional objectives by including operational modifications to the Lake Okeechobee regulation schedule as follows:

- Lake Okeechobee regulatory releases to the south are made when the Lake is in or above the baseflow zone of the LORS08 schedule and when criteria as identified in **Figure 3.3** are satisfied.
- In order to promote opportunity for Lake discharges to the south, release criteria from the Northern Estuaries are also modified to result in lower overall discharges. These operations were identified using Latin Hypercube sub-sampling optimization in a manner similar to that employed in Lake Okeechobee Watershed Restoration Project support. Documentation of the C240 Lake Okeechobee operational scheme is provided in this report in **Appendix C** and is available for use in development of the Draft Project Operating Manual.

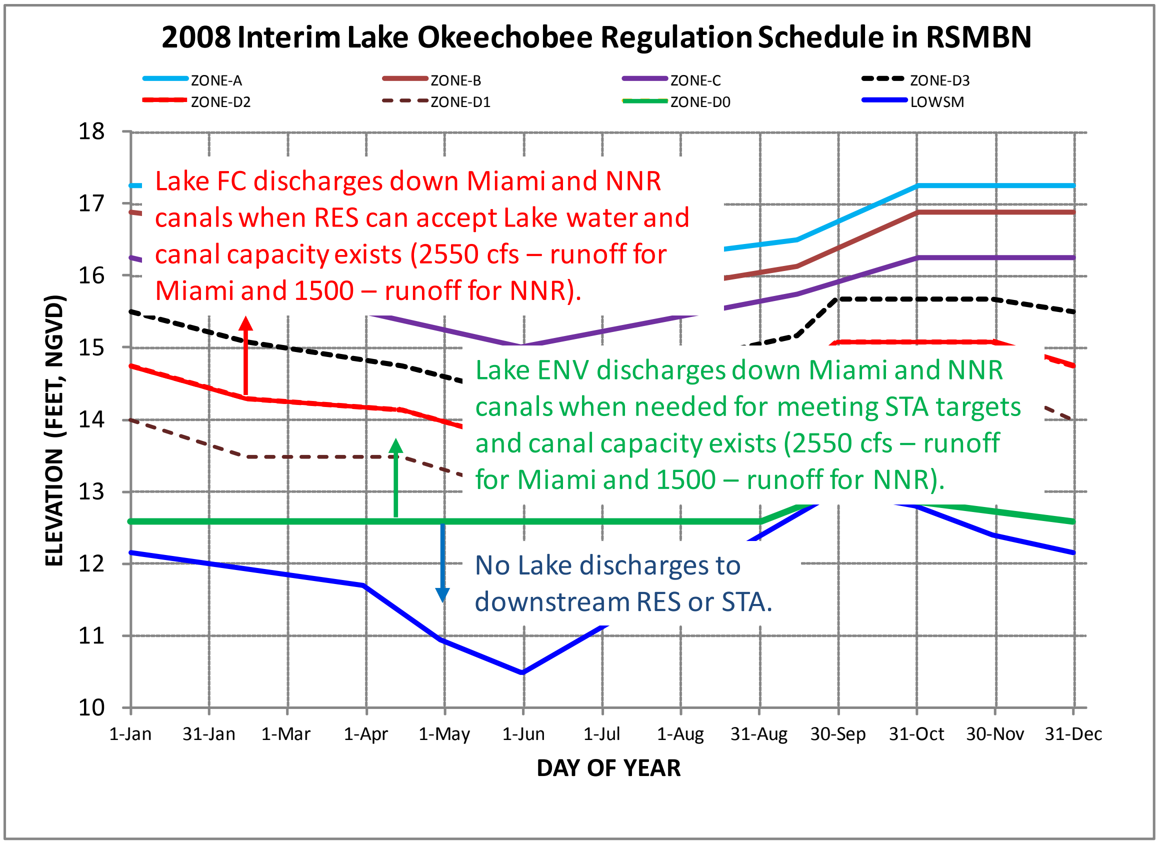


Figure 3.3. Lake Okeechobee Operational Criteria for Determining Discharges South to FEB and STA Facilities.

The EAASR C240 TSP shares a common configuration south of the Redline with all final array simulations (based on CEPP ALT4R2), as shown in **Figure 3.4,** operated to convey the additional EAASR water provided to the Everglades.

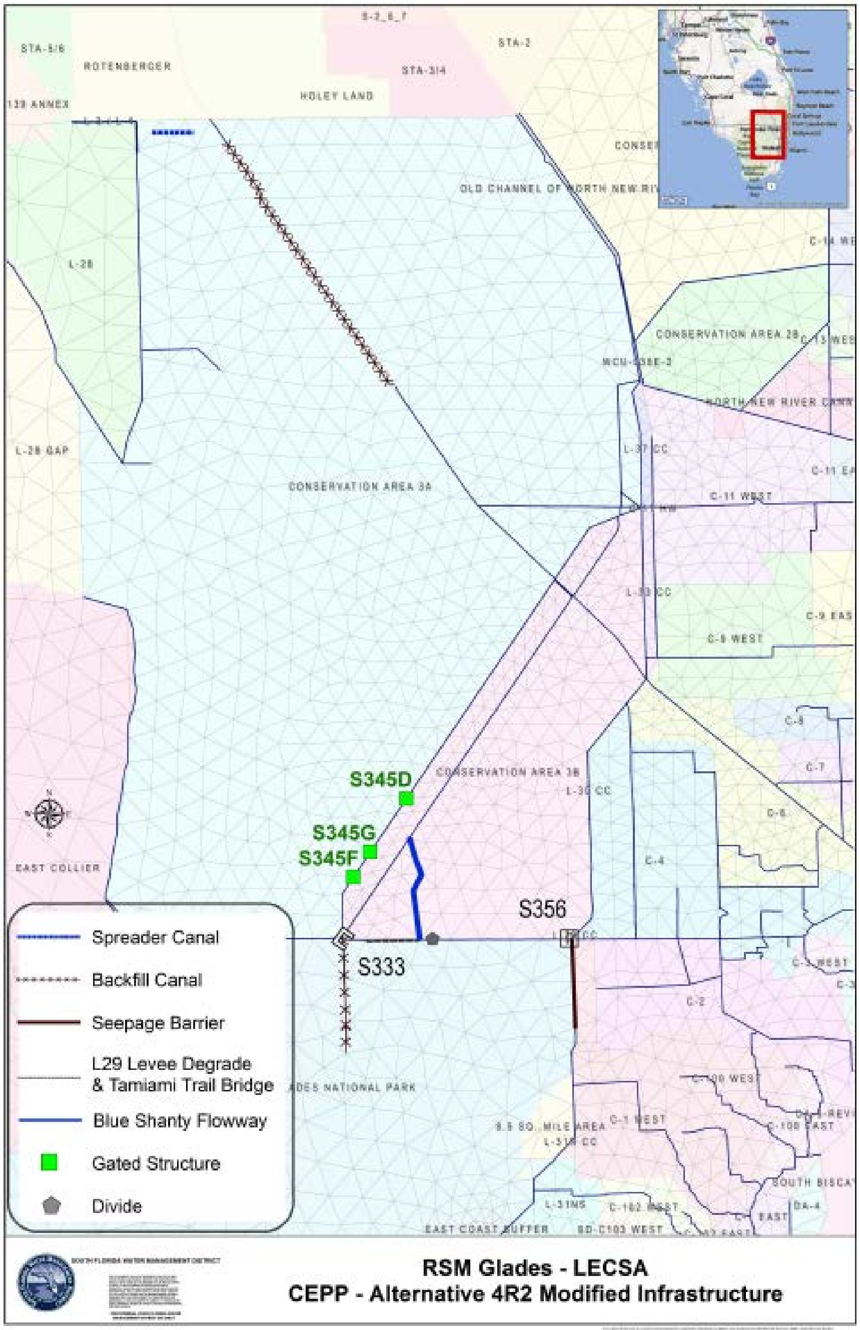


Figure 3.4. Configuration south of the redline in C240 TSP

4.0 Results

Final EAASR modeling products will be uploaded to the Statewide Model Management System (SMMS), a geographic information system (GIS) based application that includes model input data, select model output data, source code/executable files and documentation. This system can be accessed at http://apps.sfwmd.gov/smmsviewer/ . EAASR Project modeling products in SMMS can be accessed directly at the project page:

http://apps.sfwmd.gov/smmsviewer/ProjectReport.aspx?projectID=TBD

While the modeling products have been archived in the above systems, the table below lists more specific information including model version, inputs used and detailed output archival location. Version numbers and "svnroot" paths refer to a model version control system found on the SFWMD network that is not generally accessible, but inputs, model executables and source code have been copied into the SMMS system for ease of access.

Version information and model file locations

RSMBN ALT C240 011718	**RSM_REL_2.3.5R and xml_v12774**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmbn/alternatives/C240/input Output: projects/CEPP_EAR/FilesToFTP/PlanFormulation/Alternatives/05_17Jan2018/rsmbn_model_output/C240	
RSMGL ALT C240 011718	**RSM_REL_2.3.5 and xml_v12773**
Input: …svnroot/trunk/rsm_imp/CEPP_EAR/Models/rsmgl/alternatives/C240/input Output: projects/CEPP_EAR/FilesToFTP/PlanFormulation/Alternatives/05_17Jan2018/rsmgl_model_output/C240	

Review of Local and Regional-Level Results

The RSMBN and RSMGL alternative modeling scenarios were reviewed from the perspective of ensuring that localized effects of project implementations were observed as expected and that regional performance was considered reasonable. Specific checks on RSM outputs included the following:

- The RSMBN C240 TSP scenario generally maintains Lake Okeechobee performance relative to EARFWO as shown in **Figure 4.1 and Table 4.1**.
- EAASR C240 TSP scenario reduces the number of high discharge events to northern estuaries relative to the baselines as shown in **Figure 4.2 (a) and (b)**. It can also be observed that low flow event frequency is increased in the Caloosahatchee and St. Lucie Estuaries. It is expected that additional time would allow further operational refinement and avoidance of these low event outcomes.
- EAASR C240 TSP scenario generally improves LOSA and Tribal water supply performance relative to EARFWO as shown in **Figures 4.3 to 4.5.**
- Compared to EARECB, C240 provide ~370 kac-ft additional flow to Everglades as shown in **Table 4.2**. For reference, other relative increases are included in **Table 4.3** to help navigate the "soundbites" used in previous or early steps of the planning process.
- Performance of the C240 A1FEB is generally comparable to previous depth regimes observed the EARECB and EARFWO scenarios as shown in **Figure 4.6.** It is desired that depth regimes previously identified as consistent with the "design" criteria on an emergent vegetation flowway be maintained and this will be accomplished in the TSP refinement step. Flow regimes for all central flow

path STAs were checked in DMSTA (**Wang 2012**) to verify compliance with applicable water quality planning standards.

- RSMGL EAASR C240 scenario shows expected trends in hydrologic performance in the Everglades. Distribution of northern WCA3A inflows consistent with the delivered flows and CEPP downstream rainfall driven operational schemes.
- In general, more flow is moving through WCA3A and ENP systems in C240 relative to EARFWO (**Figures 4.7 & 4.8**). While stage performance is more similar to the EARFWO baseline, a general wetting trend is observed in most gages (**Figures 4.9, 4.10 & 4.11**).
- From a project assurances perspective, C240 shows little to no difference in the Lower East Coast compared to EARFWO (CEPP) and shown in **Figure 4.12** and generally maintains or improves flow toward Biscayne Bay relative to EARECB and EARFWO as shown in **Table 4.4**.

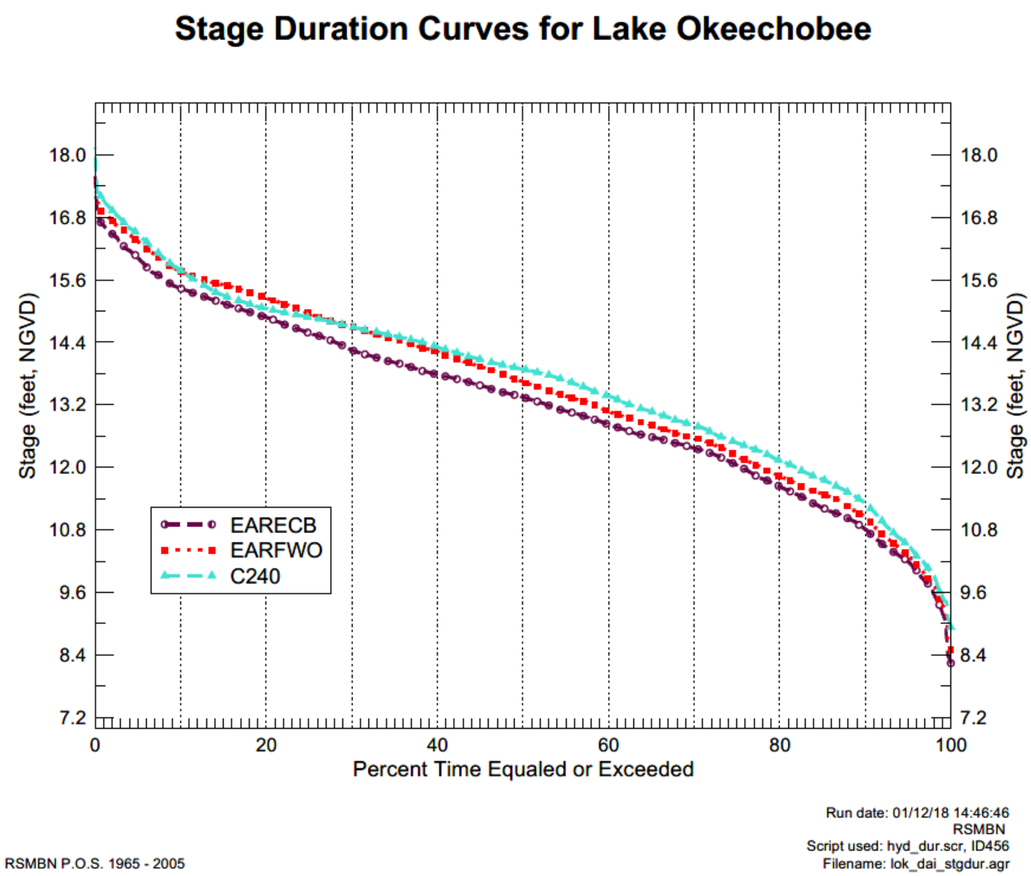
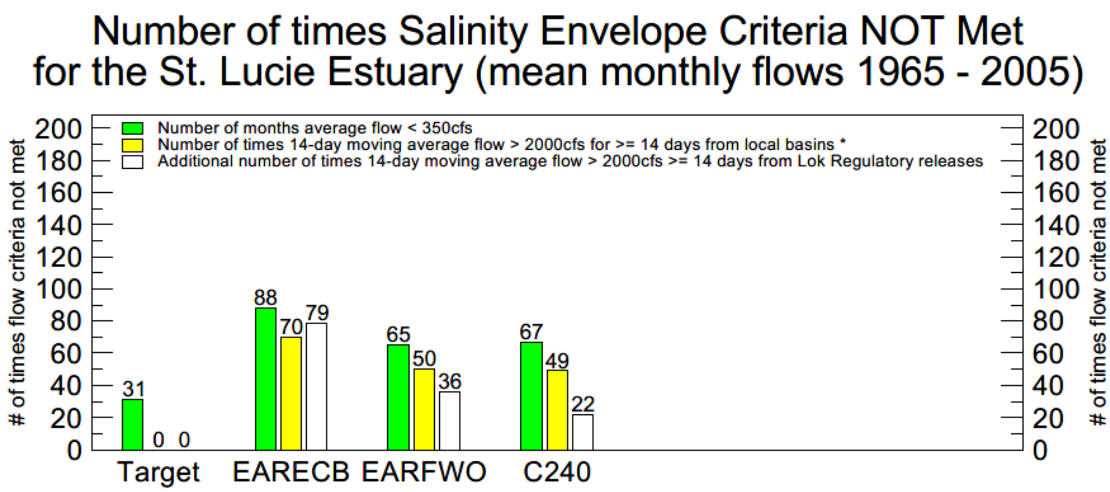


Figure 4.1. Lake Okeechobee performance for TSP ALT C240 relative to the baselines.

Table 4.1. Lake Okeechobee Standard Score Performance Measure

	EARFWO	ALT C240
Low Lake (LO1)	88.62	91.38
High Lake (LO2	97.78	92.24
Score Below Env (LO3)	47.95	58.62
Score Above Env (LO3)	71.76	71.02
Weighted Average:	86.6	85.5



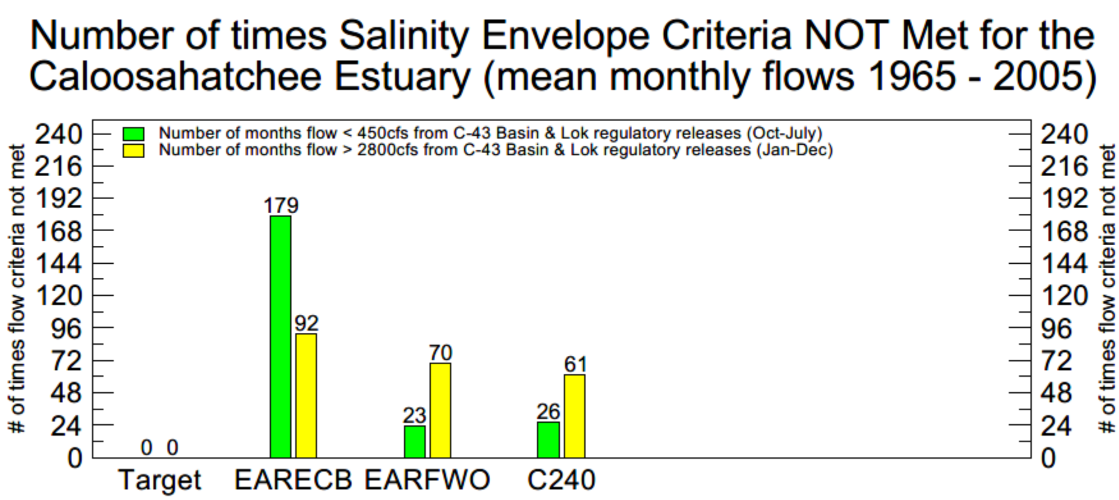


Figure 4.2. High & low discharge events to northern estuaries (a) St. Lucie Estuary and (b) Caloosahatchee Estuary relative to the plan formulation baselines for TSP scenarios.

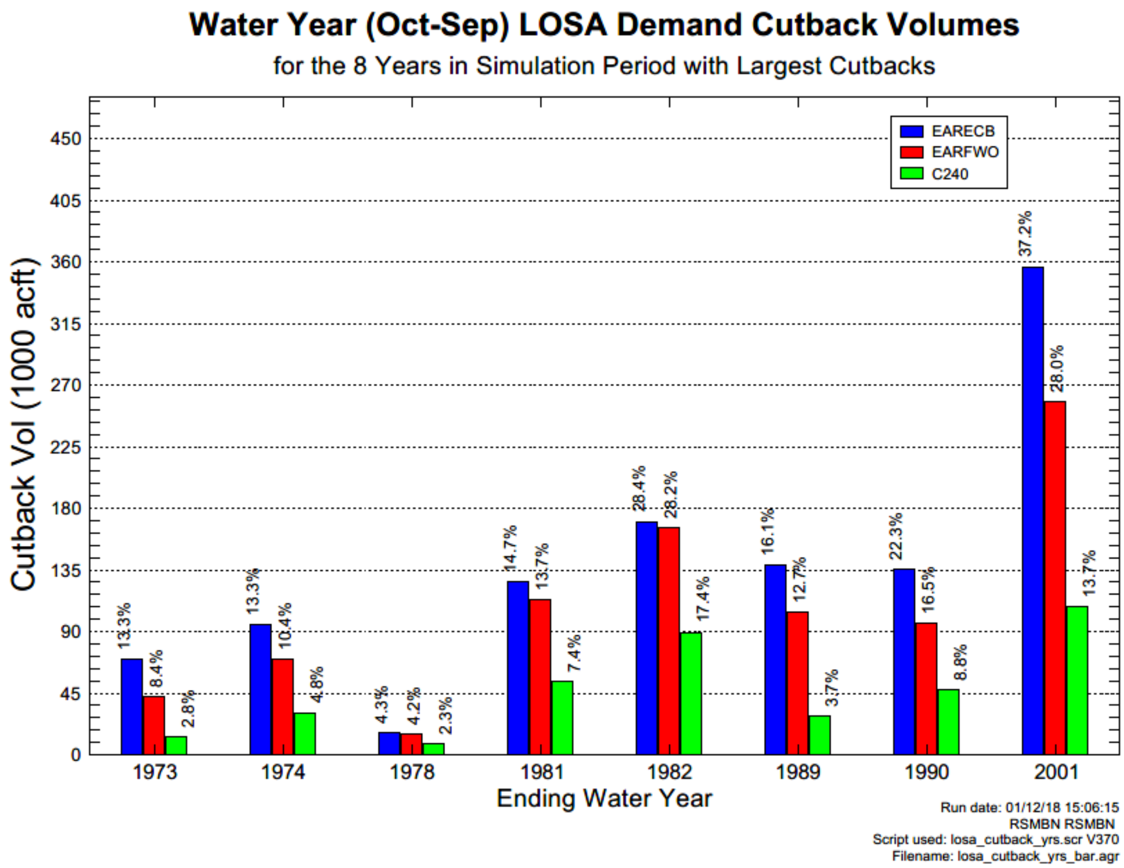


Figure 4.3. Water shortage cutbacks for water years with large cutback volumes

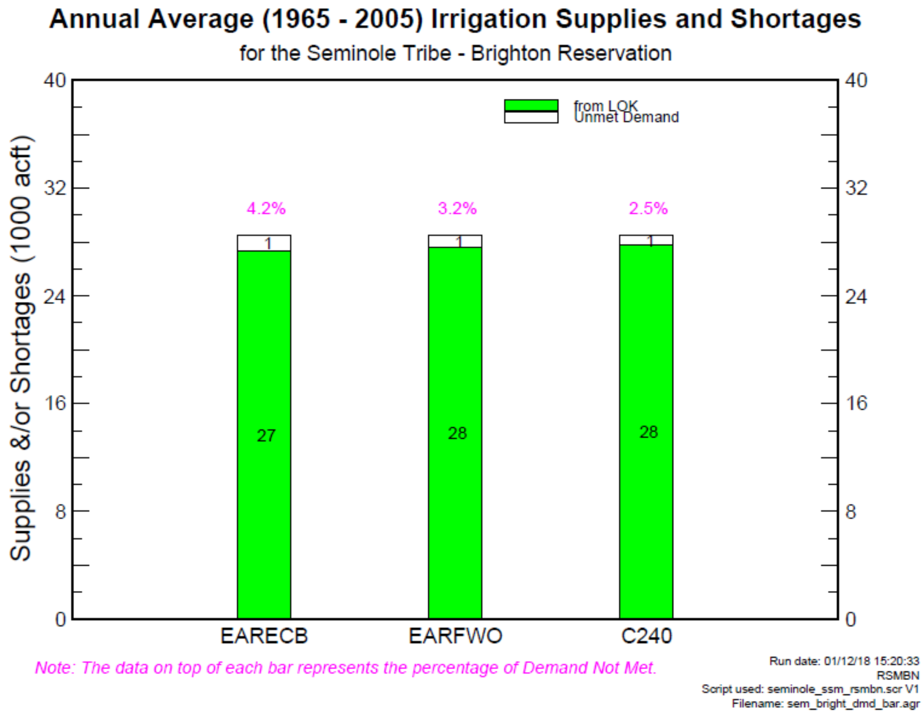


Figure 4.4. Average annual water shortage cutbacks for the Seminole Brighton Reservation

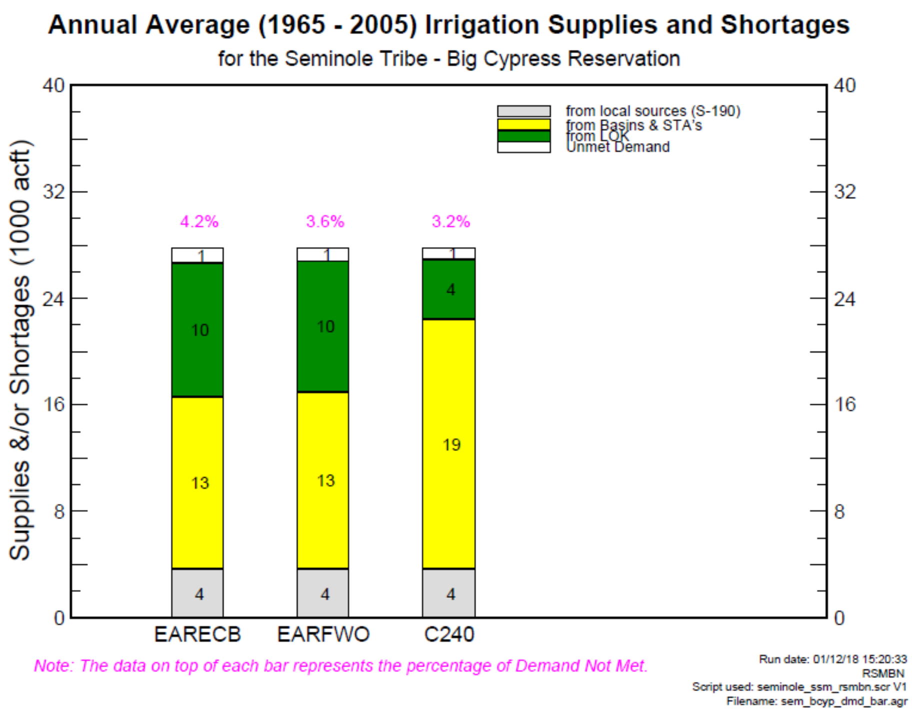


Figure 4.5. Average annual water shortage cutbacks for the Seminole Big Cypress Reservation

Table 4.2. Average annual Discharges (kac-ft) from STA2, STA34 and ERSTA toward the Greater Everglades

Average Annual Flow (kac-ft, 1965-2005)

	EARECB	**EARFWO**	**C240**
STA34OUT	383.2	596.2	453.6
STA2TOWCA2A	377.1	383.7	514.3
ERSTA_TO_LMIAMI	N/A	N/A	162.1
	760.3	979.9	1130.0
Increase over EARECB		**219.7**	**369.7**

Table 4.3. Average annual flow increase referenced to other planning efforts and periods of simulation (P.O.S.)

	STA outflow delta in CEPP (41 year relative to CEPP 2012EC)	**Updated STA delta for CEPP EARFWO(41 year relative to EARECB)**	**"Redline" Increase 36yr (1965-2000 to compare to CERP)**	**Progress toward CERP (36 yr)**
CEPP	**212**	**220**	**193**	**60%**
C240		**370**	**314**	**97%**
CERP			**323**	

Note: CERP defined as RECOVER (CERPA – PCB), only available for 1965-2000 P.O.S.

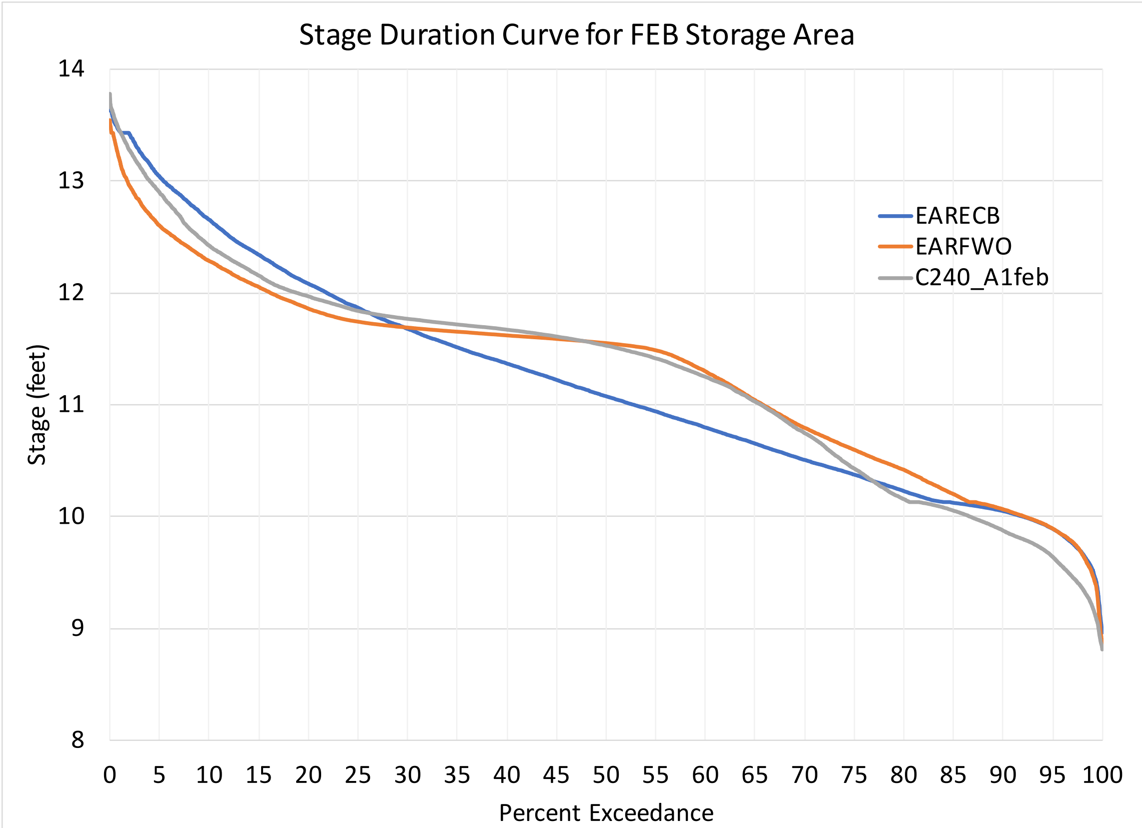


Figure 4.6. Stage duration curve for A1 FEB illustrating depth regime consistent with EMG plant assumptions.
Note: C240_A1feb shown is a sensitivity scenario with comparable regional performance that corrects localized issues with A1FEB performance as released on 1/30/18.

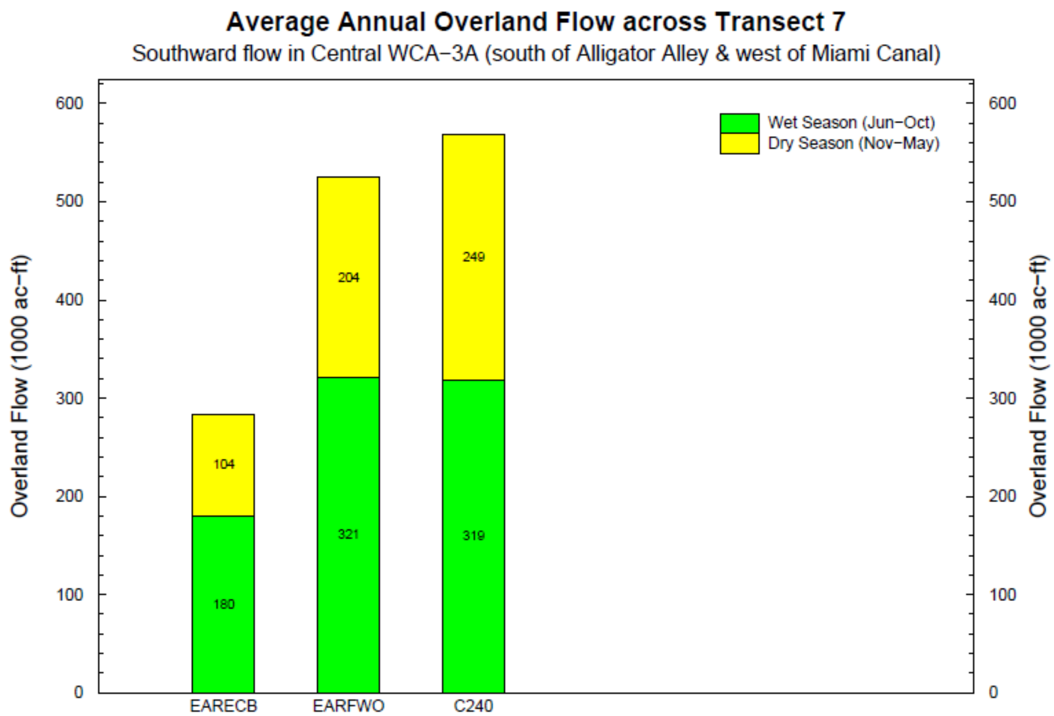


Figure 4.7. Flow differences between EARFWO and Alternatives in central WCA3A

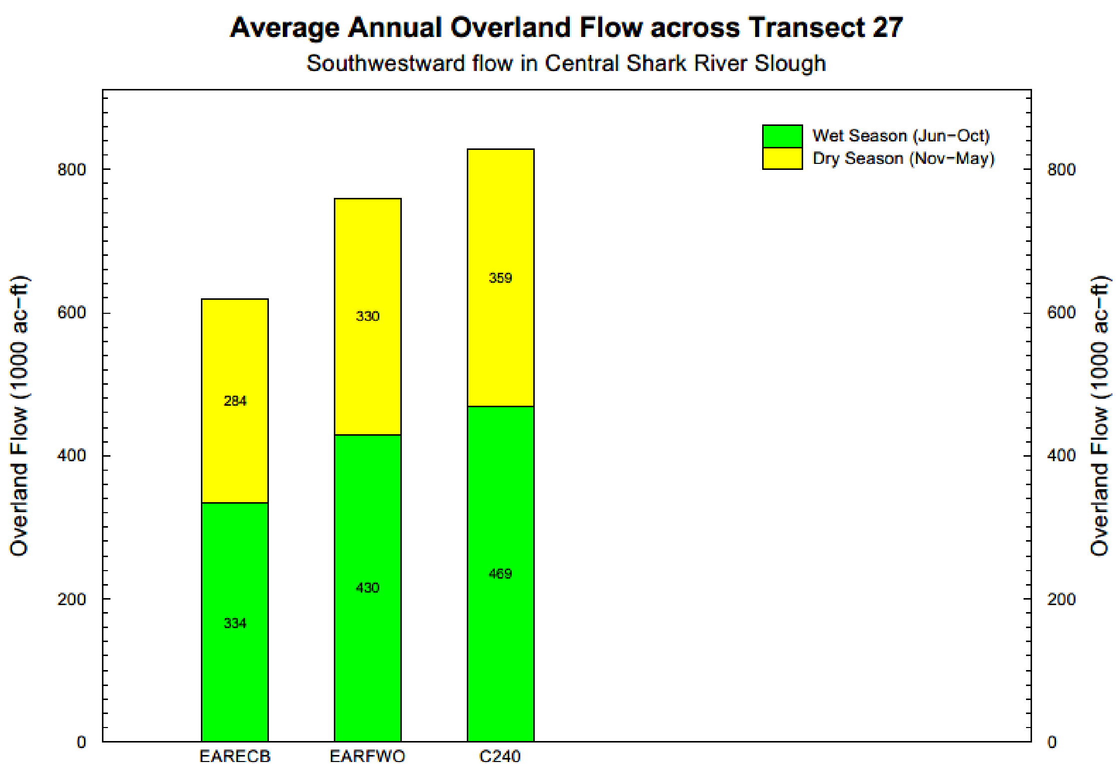


Figure 4.8. Flow differences between ECB, FWO and ALT C240 in central Shark River Slough

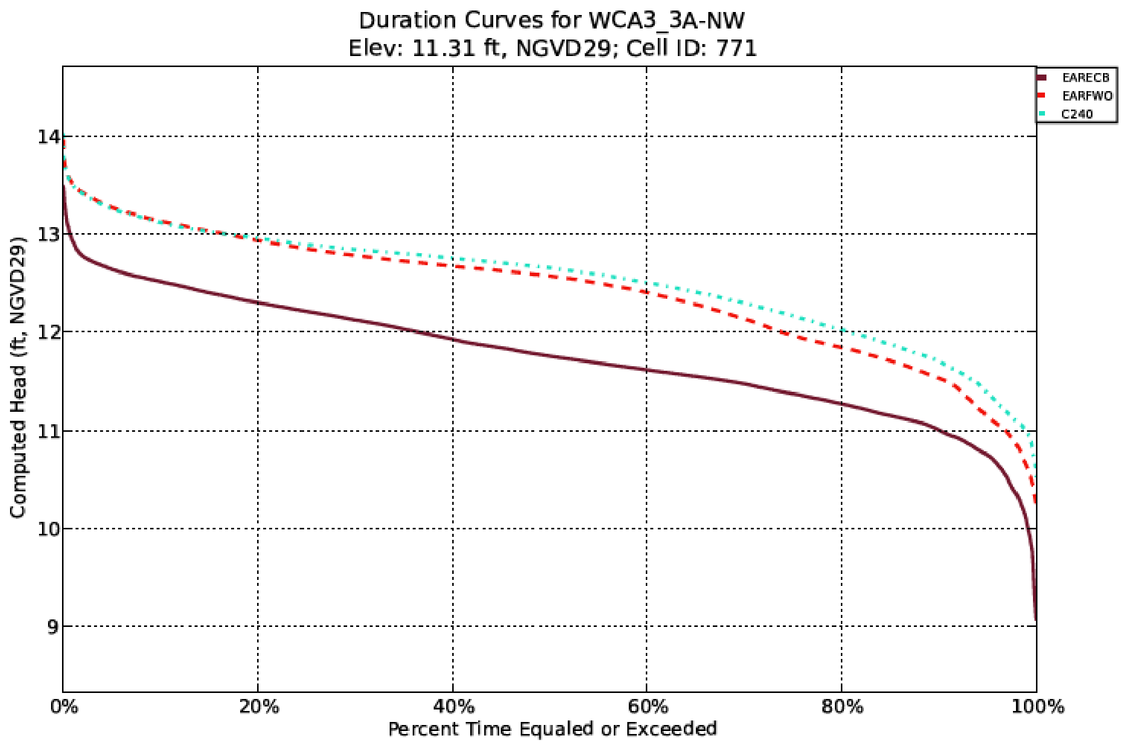


Figure 4.9. Stage duration curve for Gauge 3A-NW in Northern Water Conservation Area 3A

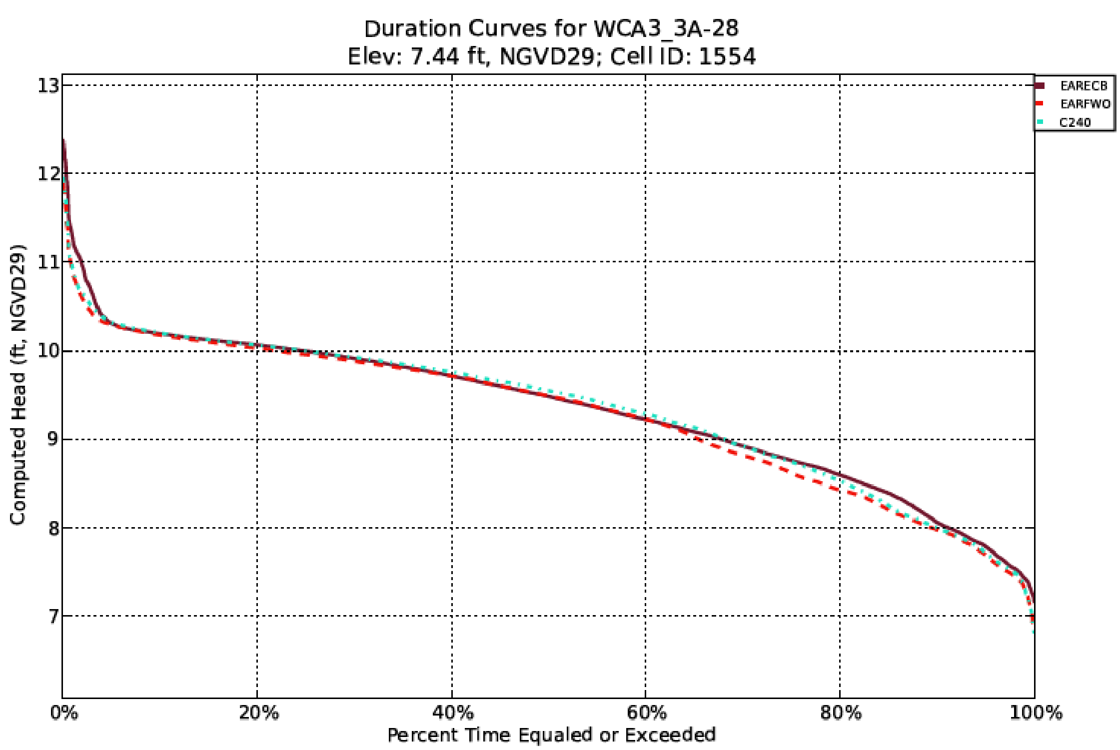


Figure 4.10. Stage duration curve for Gauge 3A-28 in Water Conservation Area 3A

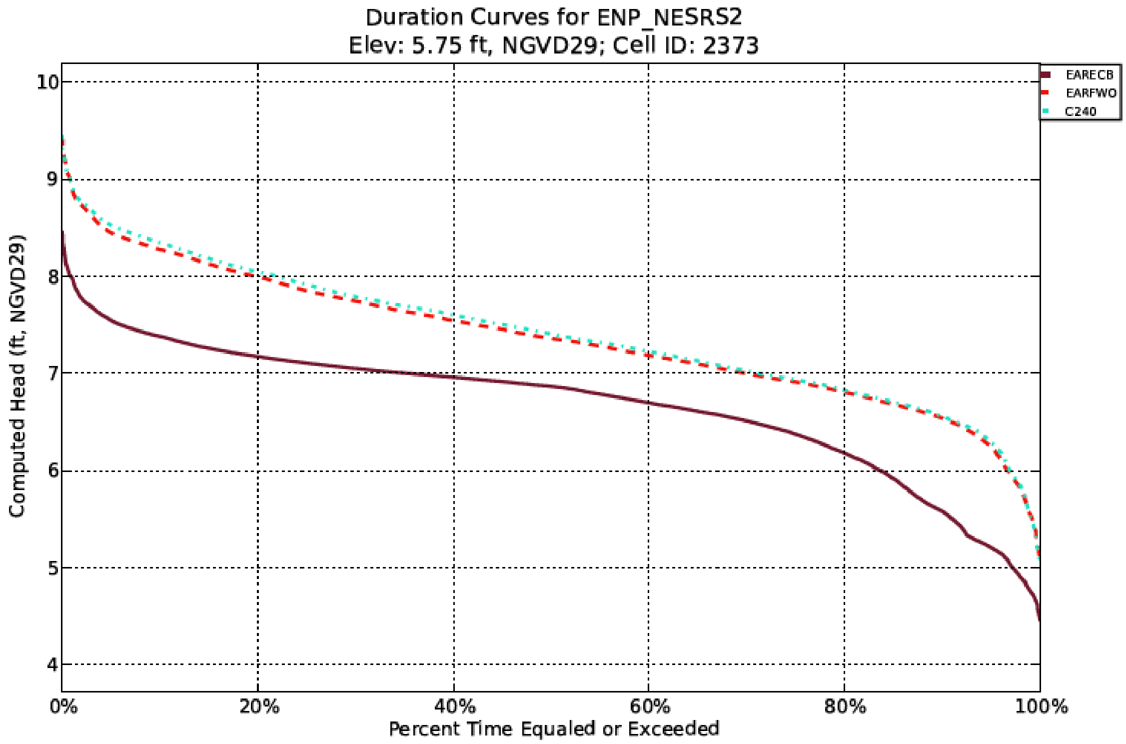


Figure 4.11. Stage duration curve for Gauge NESRS2 in Everglades National Park

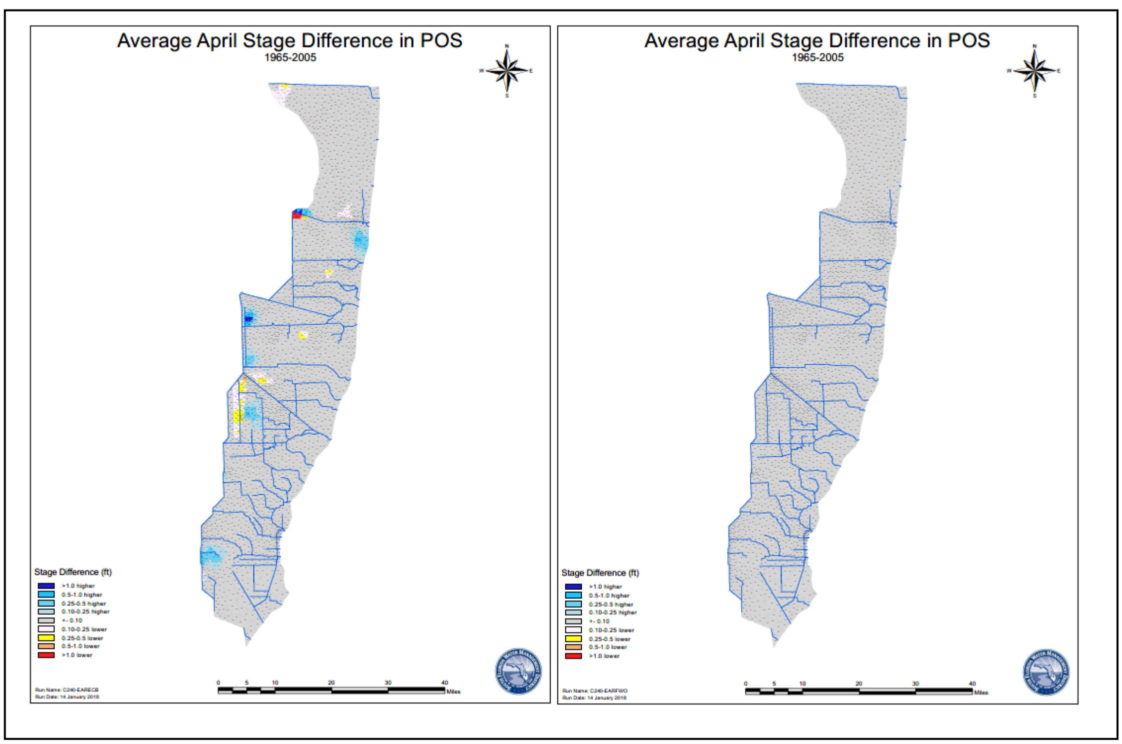


Figure 4.12. Difference between C240 and ECB (left) and C240 and FWO (right) stages for an average April condition in the Lower East Coast

Table 4.4. Average annual (kac-ft) surface water discharges at coastal structures toward Biscayne and Florida Bay (comparison with plan formulation baselines).

	ECB	ECB	FWO			C240		
Structure	Mean	% within PM	Mean	% dif ECB mean	% within PM	Mean	% dif ECB mean	% within PM
S29	280.7	66%	310.8	11%	73%	315.7	12%	73%
S28	90.9	--	90.8	0%	--	91.1	0%	--
S27	115.1	--	115.1	0%	--	116.0	1%	--
S26	130.7	--	124.9	-4%	--	133.4	2%	--
S25B	109.1	--	105.6	-3%	--	107.1	-2%	--
S25	9.7	--	9.7	0%	--	9.7	0%	--
G93	28.5	--	27.8	-2%	--	28.4	0%	--
S22	121.2	12%	117.7	-3%	12%	119.5	-1%	12%
S123	17.3	22%	17.7	2%	22%	17.8	3%	23%
S21	99.9	66%	115.3	15%	65%	118.1	18%	66%
S21A	59.0	47%	62.8	6%	44%	64.2	9%	44%
S20G	0.3	--	0.4	1%	--	0.4	2%	--
S20F	146.0	43%	154.9	6%	43%	156.8	7%	42%
S20	6.5	--	6.6	1%	--	6.6	1%	--
S197	22.2	3%	11.3	-49%	2%	12.2	-45%	2%
S26+S25B+S25	249.5	34%	240.2	-4%	34%	250.2	0%	33%

In summary, the delivered C240 TSP runs provided to the EAASR project team are deemed to adequately represent the intended planning conditions and when utilized in conjunction with the EAASR Baseline scenarios, provide a reasonable basis of comparison for the necessary evaluations required to draft the PACR.

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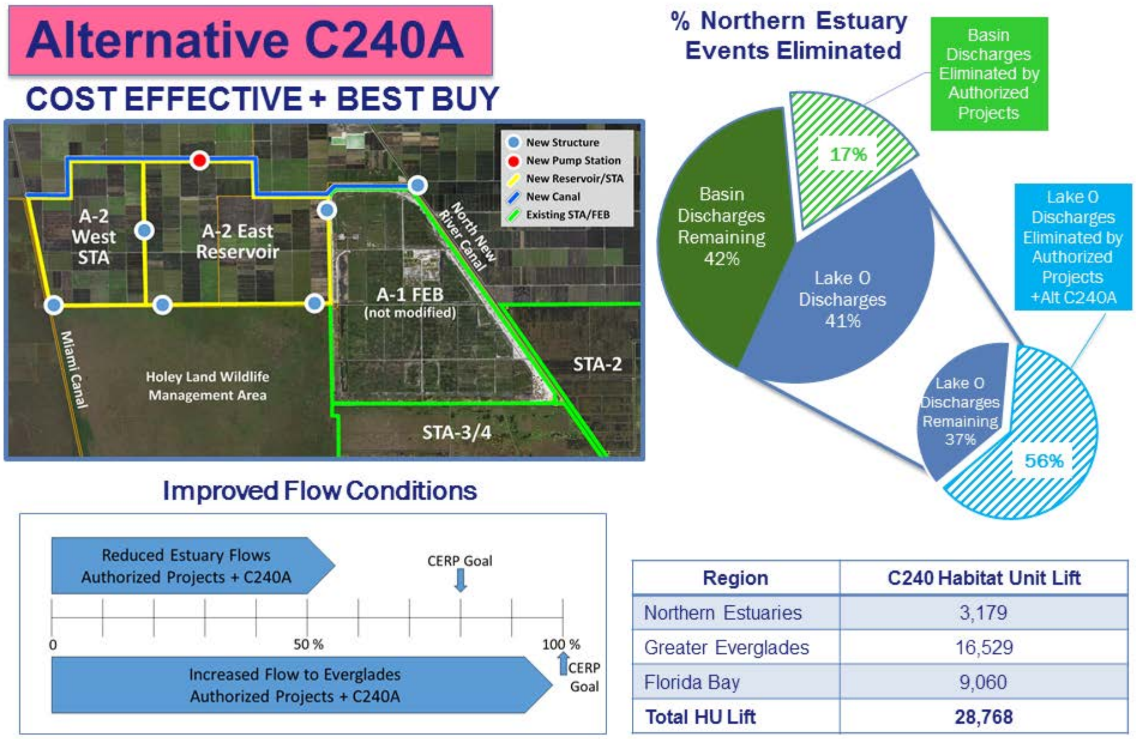
Appendix A – Details of Alternatives

Figure A.1 shows the tentatively selected plan (C240) of the Everglades Agricultural Area Storage Reservoir Project.



Region	C240 Habitat Unit Lift
Northern Estuaries	3,179
Greater Everglades	16,529
Florida Bay	9,060
Total HU Lift	**28,768**

Plan Capital Cost $1.74B(1) – CEPP New Water Component $0.40B(2) = **Capital Cost to Implement Plan $1.34B**

(1)Includes Reservoir + Stormwater Treatment Area + Real Estate $1.64B, Canal Conveyance Improvement $100M, and Recreation Plan $2.2M Costs

(2)Includes CEPP A2 FEB and A2 Recreation Plan

Figure A.1. Generalized view of infrastructure changes modeled for alternative C240

Appendix B – Tables of Assumptions

RSMBN:

- **C240**

RSMGL:

- **C240**

Modeling Section, Hydrology & Hydraulics Bureau
South Florida Water Management District

Regional Simulation Model Basins (RSMBN)
EAA Reservoir C240 Tentatively Selected Plan
Table of Assumptions

Feature	
Climate	• The climatic period of record is from 1965 to 2005. • Rainfall estimates have been revised and updated for 1965-2005. • Revised evapotranspiration methods have been used for 1965-2005.
Topography	The Topography dataset for RSM was Updated in 2009 using the following datasets: • South Florida Digital Elevation Model, USACE, 2004; • High Accuracy Elevation Data, US Geological Survey 2007; • Loxahatchee River LiDAR Study, Dewberry and Davis, 2004; • St. Lucie North Fork LiDAR, Dewberry and Davis, 2007; • Palm Beach County LiDAR Surve, Dewberry and Davis, 2004; and • Stormwater Treatment Area stage-storage-area relationships based on G. Goforth spreadsheets.
Land Use	• Lake Okeechobee Service Area (LOSA) Basins were updated using consumptive use permit information as of 2/21/2012, as reflected in the LOSA Ledger produced by the Water Use Bureau. Project features simulated in the EAA (above and beyond the Everglades Construction Project) remove land from agricultural production. • C-43 Groundwater irrigated basins – Permitted as of 2010, the dataset was updated using land use, aerial imagery and 2010 consumptive use permit information. • Dominant land use in EAA is sugar cane other land uses consist of shrub land, wet land, ridge and slough, and sawgrass.
LOSA Basins	• Lower Istokpoga, North Lake Shore and Northeast Lake Shore demands and runoff estimated using the AFSIRS model and assumed permitted land use (see land use assumptions row).
Lake Okeechobee	• Lake Okeechobee Regulation Schedule 2008 (LORS 2008) o EAASR optimized release guidance in order to improve selected performance within LOK, the northern estuaries and LOSA while meeting environmental targets in the Glades. o Lake Okeechobee can send flood releases south through the Miami Canal and North New River Canal to the EAA Reservoir when the LOK stage is above the bottom of Zone D1 (EAA basin runoff used to limit conveyance capacity: 2,550 cfs for Miami Canal and 1,550 cfs for North New River Canal). o Lake Okeechobee can send flood releases south to help meet water-quality based flow targets at STA-3/4, STA-2N, and STA-2S when the LOK stage is above the bottom of the Baseflow Zone (EAA basin runoff used to limit conveyance capacity: 2,550 cfs for Miami Canal and 1,550 cfs for North New River Canal).

Feature	
	- o Includes Lake Okeechobee regulatory releases to tide via L8 canal. - o Releases via S-77 can be diverted into C43 Reservoir • Lake Okeechobee Water Shortage Management (LOWSM) Plan. • Interim Action Plan (IAP) for Lake Okeechobee (under which backpumping to the lake at S-2 and S-3 is to be minimized). • "Temporary" forward pumps as follows: - o S354 – 400 cfs - o S351 – 600 cfs - o S352 – 400 cfs - o All pumps reduce to the above capacities when Lake Okeechobee stage falls below 10.2 ft and turn off when stages recover to greater than 11.2 ft • No reduction in EAA runoff associated with the implementation of Best Management Practices (BMPs); No BMP makeup water deliveries to the WCAs • Backpumping of 298 Districts and 715 Farms into lake minimized
Northern Lake Okeechobee Watershed Inflows	• Headwaters Revitalization schedule for Kissimmee Chain of Lakes using the UKISS model. • Kissimmee River Restoration complete. • Fisheating Creek, Istokpoga & Taylor Creek / Nubbin Slough Basin Inflows calculated from historical runoff estimates.
Caloosahatchee River Basin	• Caloosahatchee River Basin irrigation demands and runoff estimated using the AFSIRS model and assumed permitted land use as of February 2012. (see land use assumptions row) • Public water supply daily intake from the river is included in the analysis. • Maximum reservoir height of 41.7 ft NGVD with a 9,379-acre footprint in Western C43 basin with a 175,800 acre-feet effective storage. • Proposed reservoir meets estuary demands while C-43 basin supplemental demands for surface water irrigation are met by Lake Okeechobee.
St. Lucie Canal Basin	• St. Lucie Canal Basin demands estimated using the AFSIRS model and assumed permitted land use as of February 2012 (see land use assumptions row). • Excess C-44 basin runoff is allowed to backflow into the Lake if lake stage is below 14.5 ft before being pumped into the C-44 reservoir. • Basin demands include the Florida Power & Light reservoir at Indiantown. • Indian River Lagoon South Project features • Ten-mile Creek Reservoir and STA: 7,078 acre-feet storage capacity at 10.79 maximum depth on 820 acre footprint; receives excess water from North Folk Basin; • C-44 reservoir: 50,246 acre-feet storage capacity at 5.18 feet maximum depth on 12,125 acre footprint; C44 reservoir releases water back to Lake Okeechobee when Lake stages are below the bottom of the Baseflow Zone. • C-23/C-24 reservoir: 92,094 acre-feet storage capacity at 13.27 maximum depth on 8,675 acre footprint;

Feature	
	• C-23/C-24 STA: 3,852 acre-feet storage capacity at 1.5 maximum depth on 2,568 acre footprint; • All proposed reservoirs meet estuary demands. • IRL operations assumed are consistent with the March 2010 St. Lucie River Water Reservation Rule update. • Excess C23 basin water not needed to meet estuary demands can be diverted to the C44 reservoir if capacity exists. • C44 reservoir can discharge to C44 canal and backflow to Lake Okeechobee when the lake is below the baseflow zone.
Seminole Brighton Reservation	• Brighton reservation demands were estimated using AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work plan equals 2,262 MGM (million gallons per month). AFSIRS modeled 2-in-10 demands equaled 2,383 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per Table 7, Agreement 41-21 (Nov. 1992), tribal rights to these quantities are preserved. • LOWSM applies to this agreement.
Seminole Big Cypress Reservation	• Big Cypress Reservation irrigation demands and runoff were estimated using the AFSIRS method based on existing planted acreage. • The 2-in-10 demand set forth in the Seminole Compact Work Plan equals 2,606 MGM. • AFSIRS modeled 2-in-10 demands equaled 2,659 MGM. • While estimated demands, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities as per the District's Final Order and Tribe's Resolution establishing the Big Cypress Reservation entitlement, tribal rights to these quantities are preserved. • LOWSM applies to this agreement.

Everglades Agricultural Area	• Model water-body components as shown in Figure 1.
	• Simulated runoff from the North New River – Hillsboro basin apportioned based on the relative size of contributing basins via S7 route vs. S6 route.
	• G-341 acts as a divide between S-5A Basin and Hillsboro Basin.
	• RSMBN ECB EAA runoff and irrigation demand compared to SFWMM ECB simulated runoff and demand from 1965-2005 for reasonability.
	• A 240 kac-ft storage reservoir located on 10,100 acre effective footprint (A2 RES) located north of Holeyland and assumed operations as follows:
	  o A2 RES inflows are from excess EAA basin runoff above the established inflow targets at STA-3/4, STA-2N, and STA-2S, and from LOK flood releases south (up to ~ 4ft buffer depth from full level to allow attenuation of peak EAA runoff events).
	  o A2 RES outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater

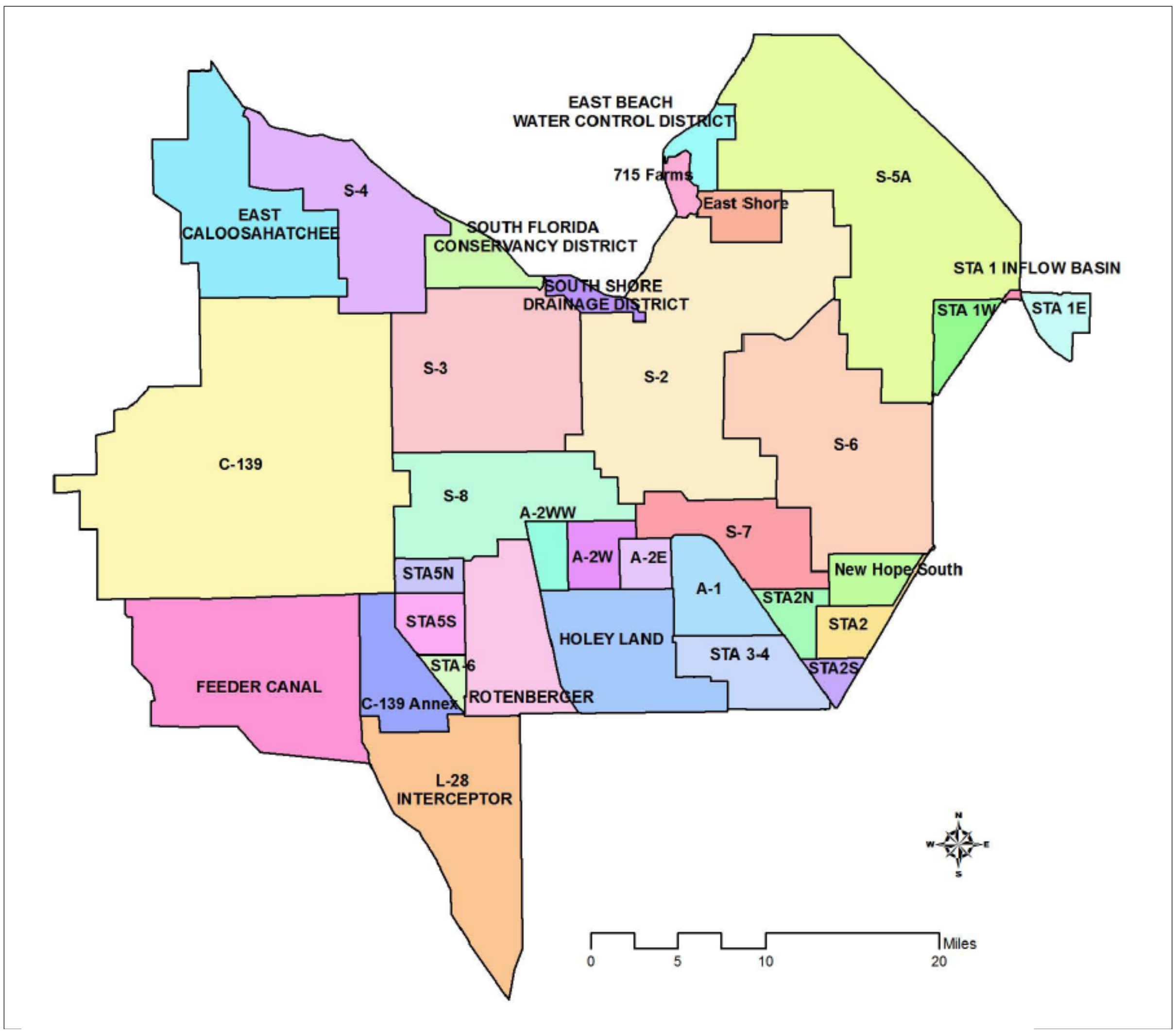
Feature	
	Treatment Areas) at A1FEB, STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient. o 0.5 ft minimum depth below which no releases are allowed o 23.5 ft maximum depth above which inflows are discontinued o Supplemental irrigation demands in the Miami and NNR/Hillsboro basins can be met from the reservoir when reservoir depths exceed 8.2 feet. • Inflows at the EAA reservoir inflow pump station are assumed to convey up to 3000 cfs from the Miami canal and 1500 cfs from the NNR canal (combined basin runoff and Lake O water); inflow to the EAA reservoir can also be made from the existing G370 and G372 pump stations up to a 6 ft depth. • Canal capacity is assumed to be increased by 1000 cfs in the Miami Canal and 200 cfs in the NNR canal above existing capacity to help convey water from Lake Okeechobee to the EAA Storage Reservoir.
Everglades Construction Project Stormwater Treatment Areas	• STAs are simulated as single waterbodies • STA-1E: 6,546 acres total area • STA-1W: 7,488 acres total area • S-5A Basin runoff is to be treated in STA-1W first and when conveyance capacities are exceeded, rerouted to STA-1E • STA-2: cells 1,2 & 3: 7,681 acres total area • STA-2N: cells 4,5 & 6; refers to Comp B-North; 6,531 acres total area • STA-2S: cells 7 & 8; refers to Comp B-South; 3,570 acres total area • STA-3/4: 17,126 acres total area • STA-5N: includes cells 1 & 2: 5,081 acres total area • STA-5S: includes cells 3, 4 & 5; uses footprint of Compartment C: 8,469 acres total area • STA-6: expanded with phase 2: 3,054 acres total area • ERSTA: Proposed STA receiving outflow from EAA deep storage reservoir and discharging to lower Miami Canal. o Effective area = 6,550 acres • Assumed operations of STAs: o 0.5 ft minimum depth below which supply from external sources is triggered; o 4 ft maximum depth above which inflows are discontinued o Inflow targets established for STA-3/4, STA-2N and STA-2S based on DMSTA simulation; met from local basin runoff, LOK flood releases and available RES/FEB storage. ERSTA inflow targets based on DMSTA simulation and met by EAA reservoir storage. • A 15,853-acre Flow Equalization Basin (A1 FEB, consistent with EARFWO) located north of STA-3/4 with assumed operations as follows:

Feature	
	o FEB inflows are from the A2 RES and are consistent with established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas). FEB inflows are limited to 500 cfs when depths are above 2.5 ft. o FEB outflows are used to help meet established inflow targets (as estimated using the Dynamic Model for Stormwater Treatment Areas) at STA-3/4, STA-2N, STA-2S and ERSTA if EAA basin runoff and LOK regulatory discharge are not sufficient. o 0.5 ft minimum depth below which no releases are allowed o 3.8 ft maximum depth above which inflows are discontinued o Assumed inlet structure of 1500 cfs capacity from A2 RES for modeling purposes. o Outflow weirs, with similar discharge characteristics as STA-3/4 outlet structure, discharging into lower North New River canal. Structure capacities and water quality operating rules are consistent with modeling assumptions assumed during the A-1 FEB EIS application process.
Holeyland Wildlife Management Area	• G-372HL is the only inflow structure for Holeyland used for keeping the water table from going lower than half a foot below land surface elevation. • Operations are similar to the existing condition as in the 1995 base simulation for the Lower East Coast Regional Water Supply Plan (LECRWSP, May 2000), as per the memorandum of agreement between the FL Fish and Wildlife Conservation (FWC) Commission and the SFWMD.
Rotenberger Wildlife Management Area	• Operational Schedule as defined in the Operation Plan for Rotenberger WMA. (SFWMD, March 2010)
Public Water Supply and Irrigation	• Regional water supply demands to maintain Lower East Coast canals as simulated from RSMGL FWO.
Western Basins	• C139 RSM basin is being modeled. Period is 1965-2005. • C139 basin runoff is modeled as follows: G136 flows is routed to Miami Canal; G342A-D flows routed to STA5N; G508 flows routed to STA5S; G406 flows routed to STA6. • C139 basin demand is met primarily by local groundwater.
Water Shortage Rules	• Reflects the existing water shortage policies as in South Florida Water Management District Chapters 40E-21 and 40E-22, FAC, including Lake Okeechobee Water Shortage Management (LOWSM) Plan.

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).

- The boundary conditions along the eastern and southern boundaries of the RSMBN model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Glades-LECSA Model (RSMGL). The SFWMM was the source of the eastern boundary groundwater/surface water flows, while the RSMGL was the source of the southern boundary structural flows.
- The RSMBN C240 assumptions were built upon the RSMBN EARFWO scenario (11/6/17) and incorporate components of the R240 and C360 scenarios (12/21/17).



Water-Body Components:
Miami Water-Body = S3 + S8 + A-2WW
NNR/HILLS Water-Body = S2 + S6 + S7 + New Hope South
WPB Water-Body = S-5A
A1FEB = A-1
A2RES = Portion of A-2E, A-2W and A-2WW
ERSTA = Portion of A-2E, A-2W and A-2WW

Figure B-1. RSMBN Basin Definition within the EAA

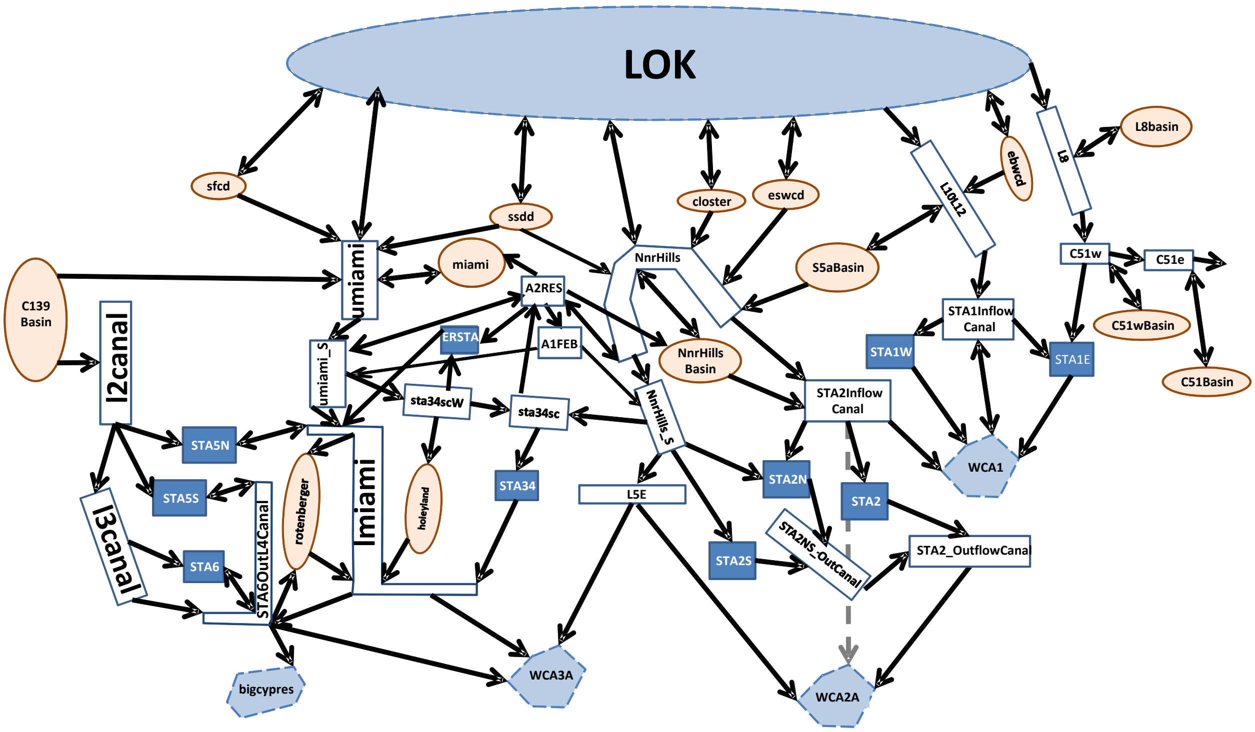


Figure B-2. RSMBN Link-Node Routing Diagram, C240

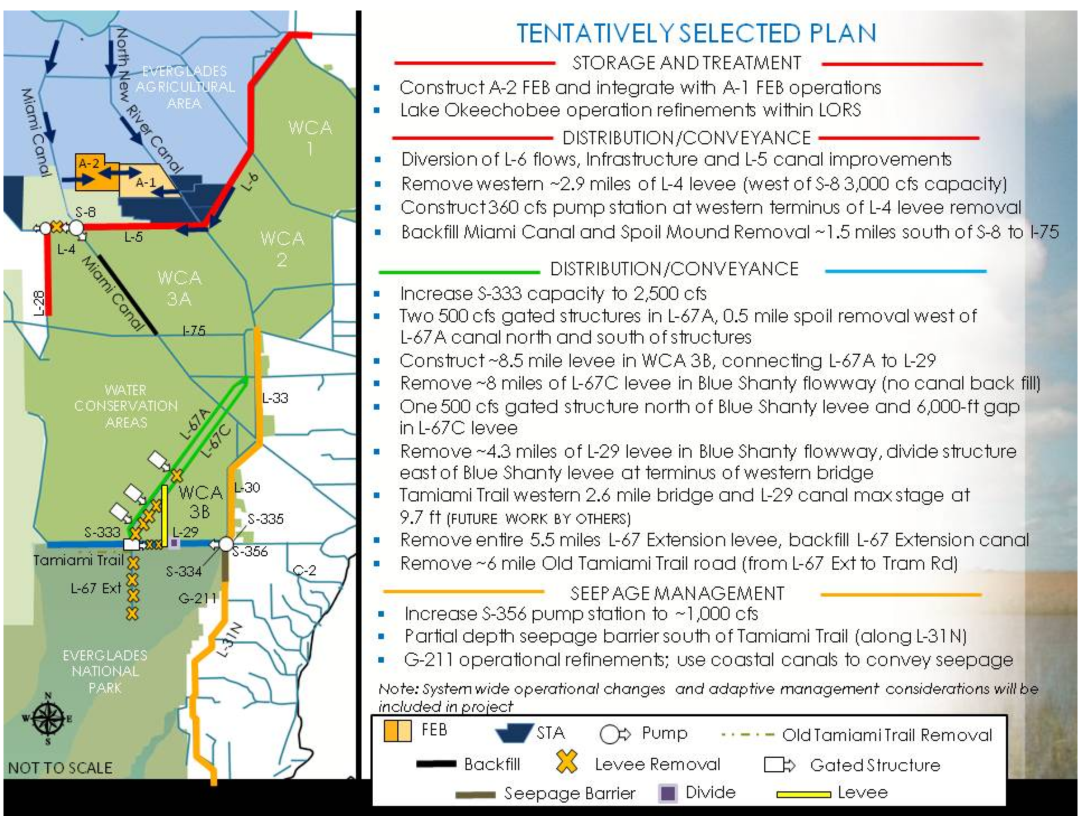


Figure B-3. CEPP ALT4R2 Features as defined by CEPP project team

Modeling Section, Hydrology & Hydraulics Bureau
South Florida Water Management District

Regional Simulation Model Glades-LECSA (RSMGL)
EAA Reservoir C240 Tentatively Selected Plan
Table of Assumptions

Feature	
Meteorological Data	• Rainfall file used: rain_v3.0_beta_tin_14_05.bin • Reference Evapotranspiration (RET) file used: RET_48_05_MULTIQUAD_v1.0.bin (ARCADIS, 2008)
Topography	• Same as calibration topographic data set except where reservoirs are introduced (STA1-E, C4 Impoundment and C-111 reservoirs). • United States Geological Survey (USGS) High-Accuracy Elevation Data Collection (HAEDC) for the Water Conservation Areas (1, 2A, 2B, 3A, and 3B), the Big Cypress National Preserve and Everglades National Park.
Tidal Data	• Tidal data from two primary (Naples and Virginia Key) and five secondary NOAA stations (Flamingo, Everglades, Palm Beach, Delray Beach and Hollywood Beach) were used to generate a historic record to be used as sea level boundary conditions for the entire simulation period.
Land Use and Land Cover	• Land Use and Land Cover Classification for the Lower East Coast urban areas (east of the Lower East Coast Flood Protection Levee) use 2008-2009 Land Use coverage as prepared by the SFWMD, consumptive use permits as of 2011 were used to update the land use in areas where it did not reflect the permit information. • Land Use and Land Cover Classification for the natural areas (west of the Lower East Coast Flood Protection Levee) is the same as the Calibration Land Use and Land Cover Classification for that area. Modified at locations where reservoirs are introduced (STA1-E, Site 1 Impoundment, Broward WPAs, C4 Impoundment, Lakebelt Lakes and C-111 Reservoirs).
Water Control Districts (WCDs)	• Water Control Districts in Palm Beach and Broward Counties and in the Western Basins assumed. • 8.5 SMA seepage canal is modeled as a WCD in ENP area.
Lake Belt Lakes	• Based on the permitted 2020 Lake Belt Lakes coverage obtained from USACE.
CERP Projects	• 1st Generation CERP – Site 1 Impoundment project is modeled as an above ground reservoir of area 1600 acres, with a maximum depth of 8 ft. • 2nd Generation CERP – Broward County Water Preserve Areas (WPAs) comprised of C-11 and C-9 impoundments were modeled as above ground reservoirs with areas 1221 and 1971 acres and maximum depths 4.3 and 4.0 ft. respectively. Operations refined in RSM model to closer represent project intent and outcomes. • 2nd Generation CERP – C-111 Spreader Canal Project includes the Frog Pond Detention Area, which is modeled as an above ground

Feature	
	impoundment with the S200 A, B and C pumps as inflow structures. In addition, the Aerojet canal is modeled with the inflow pumps S199 A, B and C. The S199 and S200 pumps are turned off based on the stage at the remote monitoring location EVER4 for the protection of the CSS Critical Habitat Unit 3. • 2nd Generation CERP – Biscayne Bay Coastal Wetlands project features were not modeled since these features along the coast in Miami-Dade County were not considered significant for CEPP. • Areal corrections were applied to the impoundment storages to account for the discrepancies of the areas in the model of the impoundments not matching the design areas.
Water Conservation Area 1 (Arthur R. Marshall Loxahatchee National Wildlife Refuge)	• Current C&SF Regulation Schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 14 ft. The bottom floor of the schedule (Zone C) is the area below 14 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow. • Structure S10E connecting LNWR to the northeastern portion of WCA-2A is no longer considered part of the simulated regional System
Water Conservation Area 2A & 2B	• Current C&SF regulation schedule. Includes regulatory releases to tide through LEC canals • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 10.5 ft in WCA-2A, defined as when WCA2-U1 marsh gauge falls below 10.5 ft or L38 canal stage falls below 10.0 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Water Conservation Area 3A & 3B	• Diversion of L-6 flows with additional 500 cfs structure and improvements to the L-5 canal • STA-3/4 outflows routed based on Rainfall Driven Operations (RDO) – a maximum of 2500 cfs is routed to S8 and G404, with the remainder being sent to S7 • Western L-4 levee degrade with 1.5 miles retained west of S8 (west of S-8 = 3,000 cfs capacity) • Miami Canal backfilled and spoil mound removed 1.5 miles south of S-8 to I-75 • Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 (USACE, 2012) • One 500 cfs gated structure in L-67A north of Blue Shanty levee (S345D) and associated gap in L-67C levee • Two 500 cfs gated structures in L-67A (S345F & S345G) discharging into Blue Shanty Flowway • Environmental target deliveries through the S345s are determined through RDO and is spatially distributed as 40% to 345D, 35% to 345F and 25% to 345G

Feature	
	• Blue Shanty Flowway assumed as follows: o Construction of ~8.5 mile levee in WCA 3B, connecting L-67A to L-29 o Removal of L-67C levee in Blue Shanty Flowway (no canal back fill) o Removal of L-29 levee in Blue Shanty Flowway. • Includes regulatory releases to tide through LEC canals. Documented in Water Control Plan (USACE, June 2002) • No net outflow to maintain minimum stages in the LEC Service Area canals (salinity control), if water levels are less than minimum operating criteria of 7.5 ft in WCA-3A, defined as when 3-69W marsh gauge falls below 7.5 ft or CA3 canal stage falls below 7.0 ft. Any water supply releases below the floor will be matched by an equivalent volume of inflow.
Everglades Construction Project Stormwater Treatment Areas	• STA-1E: 5,132 acres total treatment area. • A uniform bottom elevation equal to the spatial average over the extent of STA-1E is assumed.
Everglades National Park	• Water deliveries to Everglades National Park are based upon Everglades Restoration Transition Plan (ERTP), with the WCA-3A Regulation Schedule including the lowered Zone A (compared to IOP) and extended Zones D and E1. The environmental component of the schedule is defined by RDO. If hydraulic capacity exists at the 345s, then flood control discharges are made into 3B instead of at the S12s. • S-333 capacity increased to 2,500 cfs • L29 Divide structure assumed and is operated to send water from L29W to L29E to equilibrate canals when L29E falls below 7 ft. • L29 canal can receive inflow up to 9.7 ft (applies to both E and W segments / i.e. S333 & S356 as well as S345F & S345G structure on Blue Shanty Flowway) • G-3273 constraint for operation of S-333 assumed to be 9.5 ft, NGVD. • The one mile Tamiami Trail Bridge as per the 2008 Tamiami Trail Limited Reevaluation Report is modeled as a one mile weir. Located east of the L67 extension and west of the S334 structure. • Western 2.6 mile Tamiami Trail Bridge, modeled as a 2.6 mile long weir, and is located east of Osceola Camp and west of Frog City. • Tamiami Trail culverts east of the L67 Extension are simulated where the bridge is not located. • Removal of the entire 5.5 miles L-67 Extension levee, with backfill of L-67 Extension canal • S-355A & S-355B are operated. • Capacity of S-356 pump increased to 1000 cfs. S-356 is operated to manage seepage.

<table>
<tr><th>Feature</th><th></th></tr>
<tr><td></td><td>Full construction of C-111 project reservoirs consistent with the as-built information from USACE plus addition of contract 8 and contract 9 features. A uniform bottom elevation equal to the spatial average over the extent of each reservoir is assumed.
8.5 SMA project feature as per federally authorized Alternative 6D of the MWD/8.5 SMA Project (USACE, 2000 GRR); operations per 2011 Interim Operating Criteria (USACE, June 2011) including S-331 trigger shifted from Angel's well to LPG-2. Outflow assumed from 8.5 SMA detention cell to the C-111 North Detention Area.
An additional length of seepage canal is assumed in the model to allow water to be collected for S357 operation.
Partial depth, approximately 4 mile long seepage barrier south of Tamiami Trail (along L-31N)</td></tr>
<tr><td>Other Natural Areas</td><td>Flows to Biscayne Bay are simulated through Snake Creek, North Bay, the Miami River, Central Bay and South Bay</td></tr>
<tr><td>Pumpage and Irrigation</td><td>Public Water Supply pumpage for the Lower East Coast was updated using 2010 consumptive use permit information as documented in the C-51 Reservoir Feasibility Study; permits under 0.1 MGD were not included
Modeling of the TSP assumes an additional public water supply withdrawal of 12 MGD in Service Area 2 and 5 MGD in Service Area 3.
Residential Self Supported (RSS) pumpage are based on 2030 projections of residential population from the SFWMD Water Supply Bureau.
Industrial pumpage is also based on 2030 projections of industrial use from the Water Supply Bureau.
Irrigation demands for the six irrigation land-use types are calculated internally by the model.
Seminole Hollywood Reservation demands are set forth under VI. C of the Tribal Rights Compact. Tribal sources of water supply include various bulk sale agreements with municipal service suppliers.</td></tr>
<tr><td>Canal Operations</td><td>C&SF system and operating rules in effect in 2012
Includes operations to meet control elevations in the primary coastal canals for the prevention of saltwater intrusion
Includes existing secondary drainage/water supply system
C-4 Flood Mitigation Project
Western C-4, S-380 structure retained open
C-11 Water Quality Treatment Critical Project (S-381 and S-9A)
S-25B and S-26 backflow pumps are not modeled since they are used very rarely during high tide conditions and the model uses a long-term average daily tidal boundary
Northwest Dade Lake Belt area assumes that the conditions caused by currently permitted mining exist and that the effects of any future mining are fully mitigated by industry
ACME Basin A flood control discharges are sent to C-51, west of the S-155A structure, to be pumped into STA-1E. ACME Basin B flood control discharges are sent to STA-1E through the S-319 structure</td></tr>
</table>

Feature	
	• Releases from WCA-3A to ENP and the South Dade Conveyance System (SDCS) will follow the Everglades Restoration Transition Plan (ERTP) regulation schedule for WCA-3A, as per SFWMM modeled alternative 9E1 o Structures S-343A, S-343B, S-344 and S-12A are closed Nov. 1 to July 15 o Structure S-12B is closed Jan. 1 to July 15 • Water supply deliveries from regional system (from WCA3A: S-151/S-337) are used to maintain the L30 canal with a minimum seasonal level varying from 6.25 ft in the dry season to 5.2 ft. at the beginning of the wet season • G-211 / S338 operational refinements; use coastal canals to convey seepage toward Biscayne Bay during drier times.
Canal Configuration	• Canal configuration same as calibration except no L-67 Extension Canal and CERP & CEPP project modifications.
Lower East Coast Service Area Water Shortage Management	• Lower east coast water restriction zones and trigger cell locations are equivalent to SFWMM ECB implementation. An attempt was made to tie trigger cells with associated groundwater level gages to the extent possible. The Lower East Coast Subregional (LECsR) model is the source of this data. • Periods where the Lower East Coast is under water restriction due to low Lake Okeechobee stages were extracted from the corresponding RSMBN simulation.

Notes:

- The RSM is a robust and complex regional scale model. Due to the scale of the model, it is frequently necessary to implement abstractions of system infrastructure and operations that will, in general, mimic the intent and result of the desired project features while not matching the exact mechanism by which these results would be obtained in the real world. Additionally, it is sometimes necessary to work within established paradigms and foundations within the model code (e.g. use available input-driven options to represent more complex project operations).
- The boundary conditions along the northern boundary of the RSMGL model were provided from either the South Florida Water Management Model (SFWMM) or the RSM Basins Model (RSMBN). The SFWMM was the source of the northern boundary groundwater/surface water flows, while the RSMBN was the source of the northern boundary structural flows.
- The RSMGL C240 assumptions were built upon the RSMGL EARFWO scenario (11/6/17), with the only changes being updated northern boundary inflows from the corresponding RSMBN scenario.

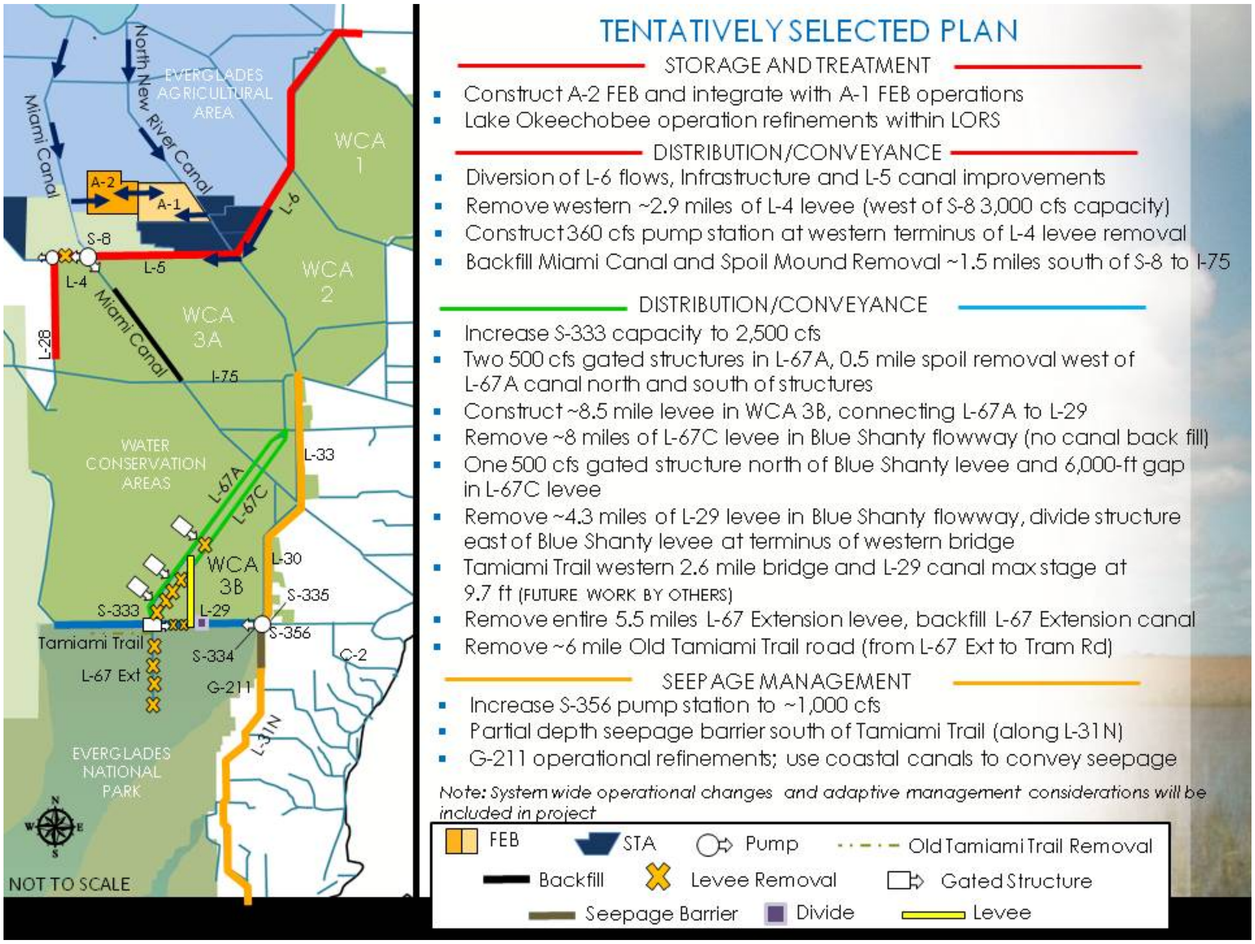


Figure. B-4 CEPP ALT4R2 Features as defined by CEPP project team

Appendix C – LORS08 Operations Schedule

The LORS08 schedule used for operation of Lake Okeechobee in the CEPP baselines was modified as shown in **Figure C.1** for use in the TSP. **Figures C.2 and C.3** show the pulse releases from Lake Okeechobee into the Caloosahatchee and St. Lucie Estuary, respectively.

Details of the TSP model implementation of the schedule can be found in **Figures C.4, C.5 and C.6.** **Figure C.4** lists the range of values used to classify the tributary hydrologic conditions. **Figure C.5** lists the range of values used to classify the net inflow seasonal outlook. **Figure C.6** lists the range of values used to classify the net inflow multi-seasonal outlook.

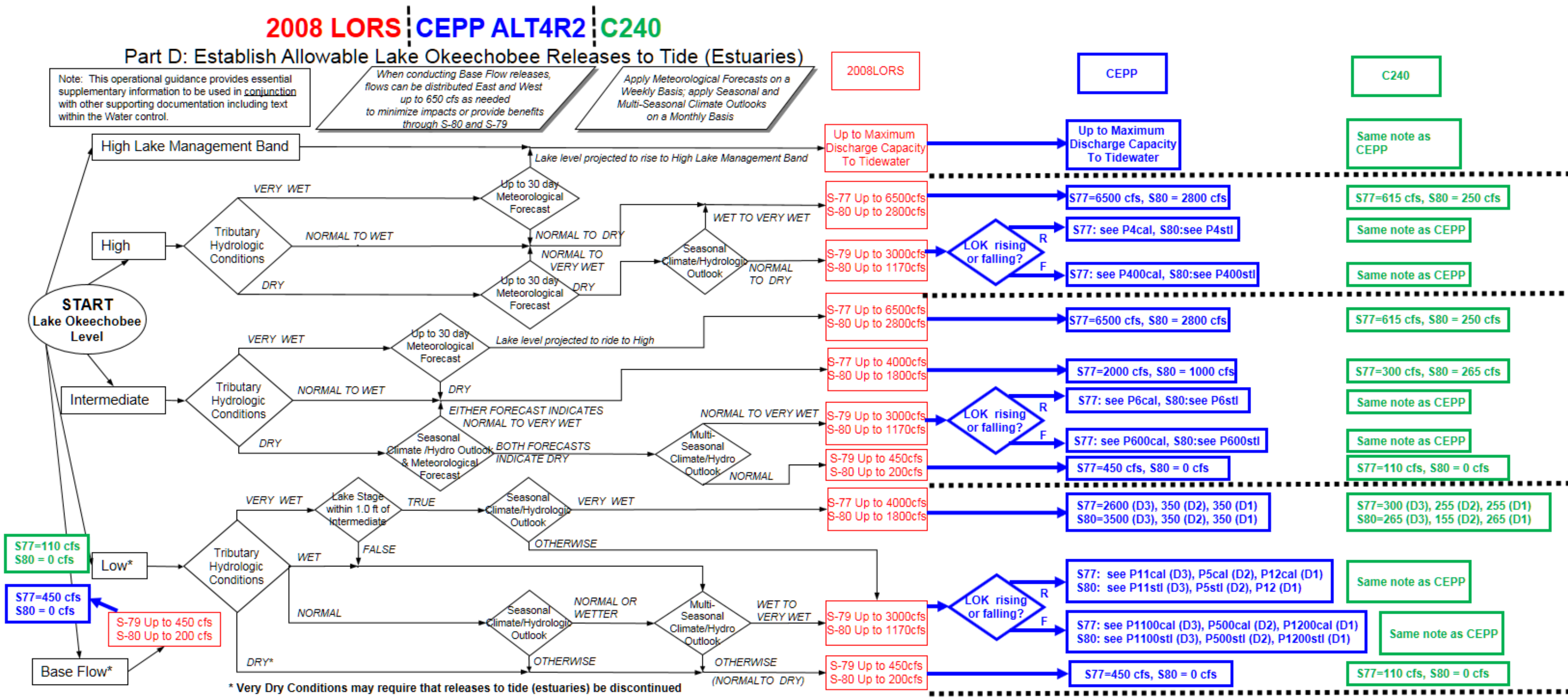


Figure C-1. LORS08 operations schedule, with FWO modifications as modeled shown in blue, and TSP modifications as modeled shown in green.

	High		Intermediate		Low					
zone	A	A	B	B	D3	D3	D2	D2	D1	D1
	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling
Day of Pulse	P4cal	P400cal	P6cal	P600cal	P11cal	P1100cal	P5cal	P500cal	P12cal	P1200cal
1	600	150	310	75	120	40	815	285	60	20
2	1,655	415.00	705	170	380	115	2,110	735	135	45
3	1,955	495.00	815	200	390	135	2,490	870	155	55
4	1,505	380.00	615	150	300.00	105.00	1,915	670	120	40
5	1,205	305	530	130	240	85	1,535	535	100	35
6	900	225.00	420	100	180.00	65.00	1,150	400	80	30
7	600	150	350	85	120	40	815	285	65	25
8	300	75.00	265	65	60.00	20.00	480	165	50	15
9	150	40.00	200	50	30.00	10.00	335	115	40	15
10	150	40.00	200	50	30.00	10.00	335	115	40	15
Average Flow (cfs)	902	227.5	441	107.5	180	62.5	1198	417.5	84.5	29.5
Volume (ac-ft)	17,887	4,511	8,745	2,132	3,569	1,239	23,756	8,279	1,676	585
†equivalent depth (ft)	0.04	0.01	0.02	0.00	0.01	0.00	0.05	0.02	0.00	0.00
† : Volume to depth Conversion based on average Lake surface area of 467,000 acres.										

Figure C.2. TSP pulse releases (as a function of lake level) from Lake Okeechobee into Caloosahatchee Estuary in cubic feet per second (cfs).

	High		Intermediate		Low					
zone	A	A	B	B	D3	D3	D2	D2	D1	D1
	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling	R=Rising	F=Falling
Day of Pulse	P4stl	P400stl	P6stl	P600stl	P11stl	P1100stl	P5stl	P500stl	P12stl	P1200stl
1	540	135.00	265	65	110.00	40.00	720	250	50	15
2	720	180	350	85	145	50	960	335	65	25
3	630	160.00	310	75	125.00	45.00	865	300	60	20
4	450	115.00	220	55	90.00	30.00	575	200	40	15
5	300	75.00	155	40	60.00	20.00	430	150	30	10
6	270	70.00	130	30	55.00	20.00	335	115	25	10
7	180	45.00	90	20	35.00	10.00	240	85	15	5
8	180	45.00	90	20	35.00	10.00	240	85	15	5
9	120	30	0	0	25	10	190	65	0	0
10	120	30	0	0	25	10	0	0	0	0
Average Flow (cfs)	351.0	88.5	161.0	39.0	70.5	24.5	455.5	158.5	30.0	10.5
Volume (ac-ft)	6,960	1,755	3,193	773	1,398	486	9,033	3,143	595	208
†equivalent depth (ft)	0.01	0.00	0.01	0.00	0.00	0.00	0.02	0.01	0.00	0.00
†: Volume to depth Conversion based on average Lake surface area of 467,000 acres.										

Figure C.3. TSP pulse releases (as a function of lake level) from Lake Okeechobee into St. Lucie Estuary in cubic feet per second (cfs).

LORS2008

Classification of Lake Okeechobee Tributary Hydrologic Conditions

Palmer Index Class Limits	2-wk Mean LO Inflow Class Limit	Tributary Hydrologic Classification*
> 3.0	>= 6000 cfs	Very Wet
1.5 to 2.99	2500 - 5999 cfs	Wet
-1.49 to 1.49	500 - 2499 cfs	Near Normal
-2.99 to -1.5	-5000 - 500 cfs	Dry
-3.0 or less	< -5000 cfs	Very Dry

*use the wettest of the two indicators

EARTSP-C240

Classification of Lake Okeechobee Tributary Hydrologic Conditions

Palmer Index Class Limits	2-wk Mean LO Inflow Class Limit	Tributary Hydrologic Classification*
> 1.5	> 4620 cfs	Very Wet
0.00 to 1.50	-960 - 4620cfs	Wet
-1.50 to 0.0	-2380 - -960 cfs	Near Normal
<= -1.50	< -2380	Dry
NOT USED	**NOT USED**	Very Dry

*use the wettest of the two indicators

Note: EAASR model results released on 1/30/18 do not reflect the Inflow class llimits listed in the table

Figure C.4. LORS08 and EAASR TSP modified Tributary Hydrologic Conditions Classifications.

LORS2008

Classification of Lake Okeechobee Net Inflow Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does not include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Seasonal Outlook
> 0.93	> 2.0	Very Wet
0.71 to 0.93	1.51 to 2.0	Wet
0.35 to 0.70	0.75 to 1.5	Normal
< 0.35	< 0.75	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

EARTSP-C240

Classification of Lake Okeechobee Net Inflow Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Seasonal Outlook
> 2.3	> 4.92	Very Wet
1.7 to 2.3	3.64 to 4.92	Wet
1.4 to 1.7	2.99 to 3.64	Normal
< 1.4	< 2.99	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

Figure C.5. LORS08 and EAASR TSP modified Seasonal Outlook Classifications.

LORS2008

Classification of Lake Okeechobee Net Inflow Multi-Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does not include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Multi-Seasonal Outlook
> 2.0	> 4.3	Very Wet
1.18 to 2.0	2.51 to 4.3	Wet
0.5 to 1.17	1.1 to 2.5	Normal
< 0.5	< 1.1	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

EARTSP-C240

Classification of Lake Okeechobee Net Inflow Multi-Seasonal Outlook**

Lake Net Inflow Prediction (million acre-feet) (Does include ET)	Equivalent Depth (feet)	Lake Okeechobee Net Inflow Multi-Seasonal Outlook
NOT USED	**NOT USED**	Very Wet
>1.3	>2.78	Wet
0.8 to 1.30	1.71 to 2.78	Normal
< 0.8	< 1.71	Dry

**volume-depth conversion based on average lake surface area of 467,000 acres.

Figure C.6. LORS08 and EAASR TSP modified Multi-Seasonal Outlook Classifications.

Appendix D – Structure Operations in South Miami-Dade County for EAASR FWO Baseline, Final Array Runs, and TSP

In **Table D.1**, the list of structures is color-coded in three groupings:

green	Structures on the L-67, L-28, and L-29 canals. Structures included are S-345, S-349, S-344, S-343A-B, S-12A-D, S-333, S-334, S-355, and S-356
blue	Structures on the L-30 and part of the L-31 canals Structures included are S-337, S-151, S-335, S-338, G-211, S-173 & S-331P (COMBQ), S-176 and S-174
yellow	Structures on part of the L-31 canal and L-31W and C-111 canals Structures included are S-332A-D, S-357, S-332, S-175, S-200, S-199, S-177, S-18C, S-197 and S-332E

Table D.1 includes the Future Without Baseline (FWO), and the final array alternatives and TSP (same operations for alternatives ALT R240, ALT R360, and ALT C360 and for TSP ALT C240).

Table D.1. Existing Condition (EARECB) and Future Condition (EARFWO, ALT R240, ALT R360, and ALT C360 and TSP ALT C240)

Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations
L-67	S-345*	Non-existent	S345D, S345F & S345G 3 gated spillway at L-67A Design Q= 500 cfs each flood control only
	S-349*	Non-existent	Non-existent
L-28	S-344*	Special code Design Q=250 cfs 1) Closed Nov 1- Jul 15 2) flood control only	Special code Design Q=250 cfs 1) Closed Nov 1- Jul 15 2) flood control only
	S-343A-B*	Special code 1) Closed Nov 1- Jul15 2) flood control only 3) S343A&B- Design Q= 200 cfs each	Special code 1) Closed Nov 1- Jul15 2) flood control only 3) S343A&B- Design Q= 200 cfs each
L-29	S-12A-D*	per ERTP S12A closed Nov 1 to Jul 31; S12B closed Jan 1 to Jul 31; S12C no closure dates. S12D no closure dates. Special code 1) S12s = 8000 cfs per structure. 2) Each structure modeled individually. 3) Each Structure is a spillway 4) Flood Control only	per ERTP S12A closed Nov 1 to Jul 31; S12B closed Jan 1 to Jul 31; S12C no closure dates. S12D no closure dates. Special code 1) S12s = 8000 cfs per structure. 2) Each structure modeled individually. 3) Each Structure is a spillway 4) Flood Control only
	S-333*	Special code 1) L-29 stage constraint of 7.5 Wet/Dry 2) Design Q=1350 cfs 3) G-3273 stage constraint of 6.8 4) Flood Control only	Special code (S333 has higher priority over S12s) 1) L-29 canal max stage of 9.7 Wet/Dry 2) Design Q=2500 cfs 3) G-3273 stage constraint of 9.5 4) Flood Control only
	S-334	Non-existent Special code 1) Flood Control 2) No open/close ops, structure flow is based on L31N stage 4) Design Q=1230 cfs	Non-existent No IOP wraparound operations. So, no flow through S334
	S-355*	Special code 1) S355 A and B Modeled Design Q = 1000 cfs each, 2) L-29 Max stage of 7.5' 3) Flood control only 4)G-3273 stage constraint of 6.8 5)L-29 stage constraint of 7.5	Special code 1) S355 A and B Modeled Design Q = 1000 cfs each, 2) L-29 Max stage of 9.7 3) Flood control only 4)G-3273 stage constraint of 9.5
	S-356*	Not operational	**6.0 / 5.5** open/closed wet season **6.0 / 5.8** open/closed dry season 1) Design Q = 1000 cfs 2) Flood control only

Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations
L-30	S337	1) Water Supply / Flood Control 2) Design Q=1100 cfs (discharge coef = 1053 cfs in msestruc*.xml)	1) Water Supply / Flood Control 2) Design Q=1100 cfs (discharge coef = 1053 cfs in msestruc*.xml)
		mse_unit inlet L30; L30 localLevel=7.0 mse_unit outlet C-304; C304 localLevel=99.0 S337_HWi & S337TWi (input variable for WCA3A_WCA3B_regulatory in Special_Assessors_2050FWO.so) S337_FracGO & S337_FracGo_high & S337_FracGo_low variable output maxfracS12s xml	mse_unit inlet L30; L30 maintLevel from rc id=993110 maintLevel(01 Nov-31 May=6.45. 01 Jun-31 Oct =5.4) L30 resLevel from rc id = 993111 resLevel(01 Nov-31 May=6.25. 01 Jun-31 Oct=5.2) mse_unit outlet C-304; C304 localLevel=99.0 S337_HWi & S337TWi (input variable for WCA3A_WCA3B_regulatory in Special_Assessors_ALT4R.so) S337_FracGO & S337_FracGo_high & S337_FracGo_low variable output maxfracS12s xml
	S-151*	Flow target based on WCA-3A regulation schedule 1) Water Supply / Flood Control 2) Design Q=1800 cfs (discharge coef. = 1154.48 in msestruc*.xml) mse_unit outlet for WCA-3A; "WCA3A local" localLevel = 7.5 mse_unit inlet C-304; C304 localLevel=99.0 Special Code for S151: S151_reg_max_zoneA=1000 (input variable Special_Assessors_2050FWO.so) S151_reg_max_zoneBC=500 (input variable Special_Assessors_2050FWO.so) S151_TWi & S151_HWi (input variable Special_Assessors_2050FWO.so) S151_FracGO & S151_FracGo_high & S151_FracGo_low variable output maxfracS12s xml	Flow target based on WCA-3A regulation schedule
	S-335	**7.5 / 7.2** open/closed wet & dry season 1) Water Supply / Flood Control 2) Design Q=1170 (discharge coef. = 1468 cfs in msestruc*.xml) 3) twHeadLimit name "S335 twHeadLimit" = 6.0 mse_unit outlet for L30; L30 localLevel=7.0 mse_unit inlet for L31NC; localLevel=99.0	**7.6 / 7.4** open/closed wet &dry season 1) Water Supply / Flood Control 2) Design Q=1170 (discharge coef. = 1468 cfs in msestruc*.xml) 3) twHeadLimit name "S335 twHeadLimit" = 6.0 mse_unit outlet for L30; L30 maintLevel from rc ID 993110 maintLevel(01 Nov-31 May=6.45. 01 Jun-31 Oct=5.4) L30 resLevel from rc ID=993111 resLevel(01 Nov-31 May=6.25. 01 Jun-31 Oct=5.2) mse_unit inlet for L31NC; localLevel=99.0

Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations
Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet /Dry Season Normal FC Operations
L-31N	S-338	**5.8 / 5.5** open/closed wet & dry season 1) Water Supply / Flood Control 2) Design Q=305 (discharge coef. = 393 cfs in msestruc*xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for C1; C1 maintLevel=3.0 mse_unit inlet for L31NC; localLevel=99.0	**5.8 / 5.5** open/closed wet season **5.7 / 5.5** open/closed dry season 1) Water Supply / Flood Control 2) Design Q=305 (discharge coef. = 393 cfs in msestruc*xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for C1; C1 maintLevel=3.0
	G-211	**6.0 / 5.5** open/closed wet & dry season twHeadLimit name "G211 twHeadLimit" = 5.3 1) Water Supply / Flood Control 2) Design Q=1100 (discharge coef. = 943 cfs in msestruc*.xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for L31N; L31N localLevel=99.0 no rulecurve	**6.0 / 5.7** open/closed wet season **5.8 / 5.5** open/closed dry season twHeadLimit name "G211 twHeadLimit" = 5.3 1) Water Supply / Flood Control 2) Design Q=1100 (discharge coef. = 943 cfs in msestruc*.xml) mse_unit outlet for L31NC; L31NC localLevel=99.0 mse_unit inlet for L31N; L31N localLevel=99.0
	S-173 & S-331P (COMBQ) *	1) Water Supply / Flood Control 2) Design Q=1161 (special code Special_Assessors_ECB_2010-11.so) mse_unit outlet for L31N; localLevel = 99.0 mse_unit inlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit inlet for L31S; "L31S res" resLevel = 3.5 mse_unit outlet for L31N; mse_unit "L31N local" localLevel = 99.0 S331_TW_lim = 6.0 Operations defined in S331_ECB_2010-11.cc: S331_HW_levels = 4.0 4.5 5.0 5.5 (LPG2 stage criteria) S331 OPERATING CRITERIA: “Discharges through S-331 can be made if the S-331 tailwater stage is below 6.0 feet and the S-176 headwater stage is below 5.5 feet. If either of those water levels of S-331 and S-176 were exceeded, discharges at S-331 would be terminated until the S-176 headwater recedes to 5.0 feet." S176_Cond is dependent on S331_TW and S176_HW -> true if either stage prohibits S331 releases.	1) Water Supply / Flood Control 2) Design Q=1161 (special code Special_Assessors_ECB_2010-11.so) mse_unit outlet for L31N; localLevel = 99.0 mse_unit inlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit inlet for L31S; "L31S res" resLevel = 3.5 mse_unit outlet for L31N; mse_unit "L31N local" localLevel = 99.0 S331_TW_lim = 6.0 Operations defined in S331_ECB_2010-11.cc: S331_HW_levels = 4.0 4.5 5.0 5.5 (LPG2 stage criteria) S331 OPERATING CRITERIA: “Discharges through S-331 can be made if the S-331 tailwater stage is below 6.0 feet and the S-176 headwater stage is below 5.5 feet. If either of those water levels of S-331 and S-176 were exceeded, discharges at S-331 would be terminated until the S-176 headwater recedes to 5.0 feet." S176_Cond is dependent on S331_TW and S176_HW -> true if either stage prohibits S331 releases.

Canal	Structure	**EARECB (RSMGL)**	**EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)**
		Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet Season/Dry Season Normal FC Operations	Open/Close (ft NGVD) (Optimum stage ft NGVD) Wet/Dry Season Normal FC Operations
L-31N (cont)	S-173 & S-331P (COMBQ) * (cont)	S331 High Range: If the water level at LPG2 well is < 5.5 ft, S331 HW will have no limit. S331 Intermediate Range: If the level at LPG2 well is > or = 5.5 and < 6.0 ft, the average daily water level upstream of the S-331 will be maintained between 5.0 ft., and 4.5 ft if permitted by d/s conditions. S331 Low Range: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. S331 Low Range Adjustment: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are not limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. Use previous day LPG2 stage, S331_TW and S176_HW, C-357 WL for current day operations	S331 High Range: If the water level at LPG2 well is < 5.5 ft, S331 HW will have no limit. S331 Intermediate Range: If the level at LPG2 well is > or = 5.5 and < 6.0 ft, the average daily water level upstream of the S-331 will be maintained between 5.0 ft., and 4.5 ft if permitted by d/s conditions. S331 Low Range: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. S331 Low Range Adjustment: If the level at LPG2 well is > or = 6.0 ft and S-357 constraints are not limiting the ability of maintaining C-357 avg daily WL below 6.2 ft, the average daily water level upstream of S-331 will be maintained between 4.5 ft. and 4.0 ft if permitted by d/s conditions. Use previous day LPG2 stage, S331_TW and S176_HW, C-357 WL for current day operations
	S-176	**5.0 / 4.75** open/closed wet & dry season 1) Water Supply / Flood Control 2) desigh Q = 1100 cfs (discharge coef. = 1135 cfs in msestruc*.xml) mse_unit outlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit outlet for L31S; "L31S res" resLevel = 3.5 mse_unit inlet for C111; maintenance level and reserve level determined in high_rf_events assessor S176_HW_levels 5.0 5.5 (input variable for S331 ops in Special_Assessors_ECB_2010-11.so)	**5.0 / 4.75** open/closed wet season **5.1 / 4.8** open/closed dry season 1) Water Supply / Flood Control 2) design Q = 1100 cfs (discharge coef. = 1135 cfs in msestruc*.xml) mse_unit outlet for L31S; "L31S maint" maintLevel = 4.0 mse_unit outlet for L31S; "L31S res" resLevel = 3.5 mse_unit inlet for C111; maintenance level and reserve level determined in high_rf_events assessor S176_HW_levels 5.0 5.5 (input variable for S331 ops in Special_Assessors_ECB_2010-11.so)
	S-174*	S174 not in model; canal is blocked near structure	S174 not in model; canal is blocked near structure

<table>
<tr><td rowspan="2">Canal</td><td rowspan="2">Structure</td><td colspan="2">EARECB (RSMGL)</td><td colspan="2">EARFWO (CEPP), ALT R240, R360, C360, and TSP C240 (RSMGL)</td></tr>
<tr><td colspan="2">Open/Close (ft NGVD)
(Optimum stage ft NGVD)
Wet Season/Dry Season Normal FC Operations</td><td colspan="2">Open/Close (ft NGVD)
(Optimum stage ft NGVD)
Wet Season/Dry Season Normal FC Operations</td></tr>
<tr><td rowspan="3">L-31N (cont)</td><td rowspan="2">S-332A, B,C,D (pumps)</td><td>S332A 5.0/4.7
S332B 5.0/4.7
S332BN 5.0/4.7
S332C 5.0/4.7
S332D 4.85/4.65</td><td>S332A Non-Existent
S332B1 4.7/4.5
S332B2 5.0/4.7
S332BN1 4.7/4.5
S332B2 5.0/4.7
S332C1 4.7/4.5
S332C2 5.0/4.7
S332D1 4.65/4.50
S332D2 4.85/4.65</td><td>S332A Non-Existent
S332B1 4.7/4.5
S332B2 5.0/4.7
S332BN1 4.7/4.5
S332B2 5.0/4.7
S332C1 4.7/4.5
S332C2 5.0/4.7
S332D1 4.65/4.50
S332D2 4.85/4.65</td><td>S332A non-existent
S332B 5.0/4.7
S332BN 5.0/4.7
S332C 5.0/4.7
S332D 4.85/4.65</td></tr>
<tr><td colspan="2">S332A =300 cfs
S332B =325 cfs
S332BN =250cfs
S332C = 575 cfs
S332D = 500 cfs Jul16-Nov30, 325cfs Dec 1-Jan31, 165cfs Feb 1-Jul15
All Flood Control</td><td colspan="2">S332B1=125 cfs
S332B2=125 cfs
S332BN1 =125cfs
S332BN2=125 cfs
S332C1 = 250 cfs
S332C2=250 cfs
S332D1 = 250 cfs
S332D2 = 250 cfs Jul15-Nov30, 75cfs Dec 1-Jan31, 0cfs Feb 1-Jul14
All Flood Control</td></tr>
<tr><td>S-357 (pump)</td><td colspan="2">6.2 / 5.7 open/closed wet & dry season
1) Flood Control
Pump Q = 126 cfs</td><td colspan="2">S357A (5.7' Nov 1-May31, 5.2' Jun1-Oct31)/(5.4' Nov 1-May31, 4.9' Jun1-Oct31)
S357B (6.0' Nov 1-May31, 5.5' Jun1-Oct31)/(5.7' Nov 1-May31, 5.2' Jun1-Oct31)

S357A = 250 cfs
S357B = 250 cfs</td></tr>
<tr><td rowspan="2">L-31W</td><td>S-332 (pumps)*</td><td colspan="2">Non-existent</td><td colspan="2">Non-existent</td></tr>
<tr><td>S-175</td><td colspan="2">Non-existent</td><td colspan="2">Non-existent</td></tr>
<tr><td rowspan="6">C-111</td><td>S-200</td><td colspan="2">S-200A=75cfs; 3.8/3.6
S-200B=75cfs; 3.9/3.6
S-200C=75cfs; 4.0/3.6</td><td colspan="2">S-200A=75cfs; 3.6/3.4
S-200B=75cfs; 3.7/3.4
S-200C=75cfs; 3.8/3.4</td></tr>
<tr><td>S-199</td><td colspan="2">S-200A=75cfs; 3.8/3.6
S-200B=75cfs; 3.9/3.6
S-200C=75cfs; 4.0/3.6</td><td colspan="2">S-200A=75cfs; 3.8/3.6
S-200B=75cfs; 3.9/3.6
S-200C=75cfs; 4.0/3.6</td></tr>
<tr><td>S-177</td><td colspan="2">4.2/3.6 (*Open/Close determined in high rainfall event Special Code.)
1) Water Supply & Flood Control
Spillway w/1 gate Design Q = 2900 cfs.</td><td colspan="2">4.2/3.6 (*Open/Close determined in high rainfall event Special Code.)
1) Water Supply & Flood Control
Spillway w/1 gate Design Q = 2900 cfs.</td></tr>
<tr><td>S-18C</td><td colspan="2">2.6 / 2.3 open/closed wet & dry season
1) Water Supply & Flood control
Spillway w/2 gates Design Q=3200 cfs.</td><td colspan="2">2.6 / 2.3 open/closed wet & dry season
1) Water Supply & Flood control
Spillway w/2 gates Design Q=3200 cfs.</td></tr>
<tr><td>S-197*</td><td colspan="2">****Same as IOP,See Note
1) S197 ops see below ****
2) Flood control only
13 Culverts w/gates Design Q=6000 cfs.</td><td colspan="2">****Same as IOP,See Note
1) S197 ops see below ****
2) Flood control only
13 Culverts w/gates Design Q=6000 cfs.</td></tr>
<tr><td>S-332E (pump)</td><td colspan="2">Non-existent</td><td colspan="2">Non-existent</td></tr>
<tr><td colspan="6">* SFWMM uses special code. Open/Close may or may not be used in operations.
**** S-197 Ops: Open 3 gates if S-177 full open & S-177>4.1 ft or S-18C> 2.8 ft
Open 7 gates if S-177 > 4.2 ft for 24 hrs or S-18C > 3.1 ft
Open 13 gates if S-177 > 4.3 ft or S-18C > 3.3 ft
Close when all the conditions below are met
1) S-176 < 5.2 ft and S-177 < 4.2 ft
2) Storm moved away from basin
3) After 1 and 2 are met, keep the number of S-197 culverts open necessary only to match residual flow through S-176</td></tr>
</table>

ANNEX C-1
CIVIL PLATES

- Overall Site Plan for TSP
- Earthwork Typical Sections for TSP

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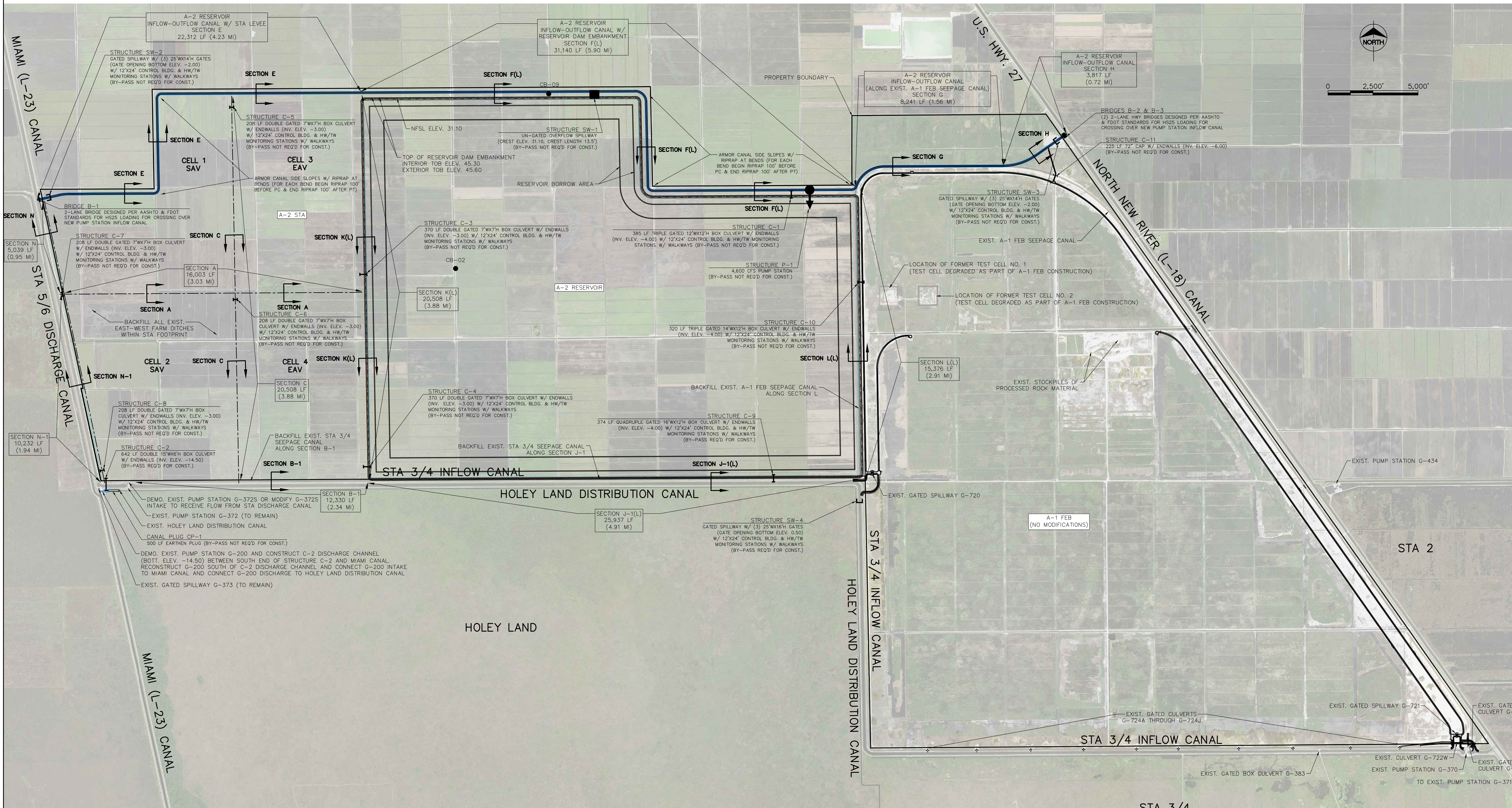
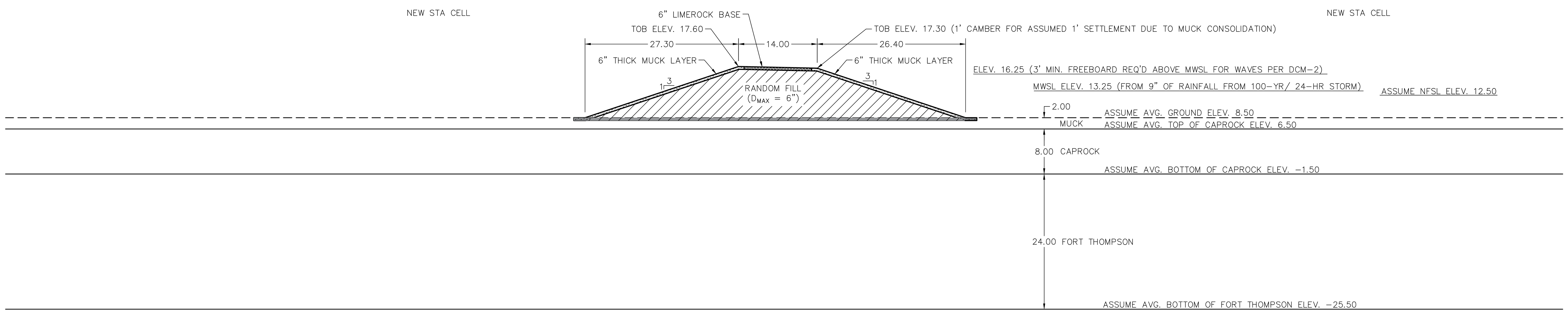
EAA STORAGE RESERVOIR
CONCEPT 240–A

3/9/2018



NOTE:
1. ELEVATIONS SHOWN HEREON ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88). NGVD29 = NAVD88 + 1.43' FOR THE A–2 RESERVOIR & A–2 STA SITES.



TYPICAL SECTION A
NOT TO SCALE

TYPICAL SECTION B-1
NOT TO SCALE

LEGEND:
- MUCK FILL
- MUCK REMOVAL
- RANDOM FILL (D_{MAX} = 6")
- RANDOM FILL (D_{MAX} = 12")
- RANDOM FILL (D_{MAX} = 24")
- LIMEROCK BASE
- RIPRAP
- CONCRETE
- FILTER FILL
- CORE FILL
- CAPROCK EXCAVATION
- FORT THOMPSON EXCAVATION

NOTE:
1. ELEVATIONS SHOWN HEREON ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88). NGVD29 = NAVD88 + 1.43' FOR THE A-2 RESERVOIR & A-2 STA SITES.

TYPICAL SECTION C
NOT TO SCALE

TYPICAL SECTION E
NOT TO SCALE

LEGEND:

- MUCK FILL
- MUCK REMOVAL
- RANDOM FILL (D_{MAX} = 6")
- RANDOM FILL (D_{MAX} = 12")
- RANDOM FILL (D_{MAX} = 24")
- LIMEROCK BASE
- RIPRAP
- CONCRETE
- FILTER FILL
- CORE FILL
- CAPROCK EXCAVATION
- FORT THOMPSON EXCAVATION

NOTE:
1. ELEVATIONS SHOWN HEREON ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88). NGVD29 = NAVD88 + 1.43' FOR THE A-2 RESERVOIR & A-2 STA SITES.

LEGEND:

- MUCK FILL
- MUCK REMOVAL
- RANDOM FILL (D_{MAX} = 6")
- RANDOM FILL (D_{MAX} = 12")
- RANDOM FILL (D_{MAX} = 24")
- LIMEROCK BASE
- RIPRAP
- CONCRETE
- FILTER FILL
- CORE FILL
- CAPROCK EXCAVATION
- FORT THOMPSON EXCAVATION

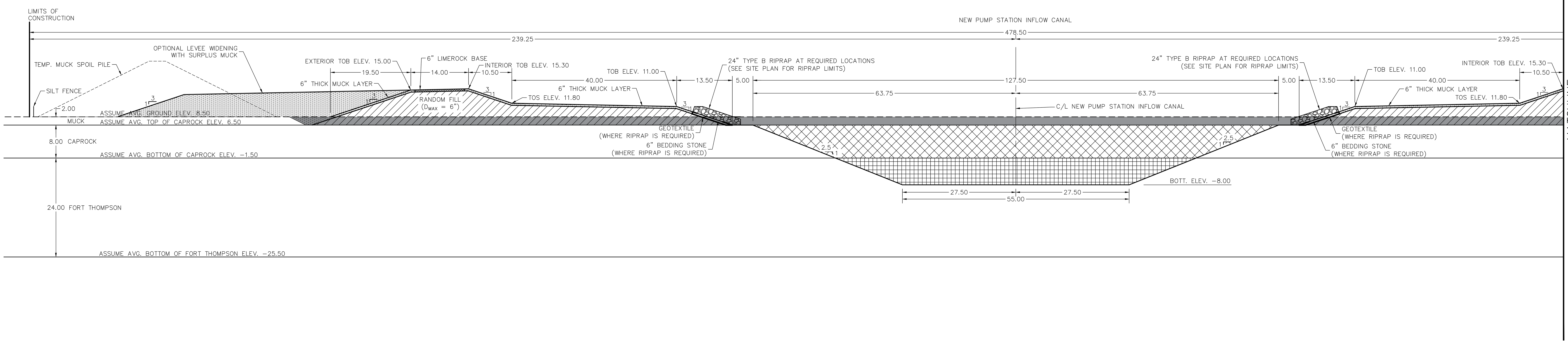
TYPICAL SECTION F(L)
NOT TO SCALE

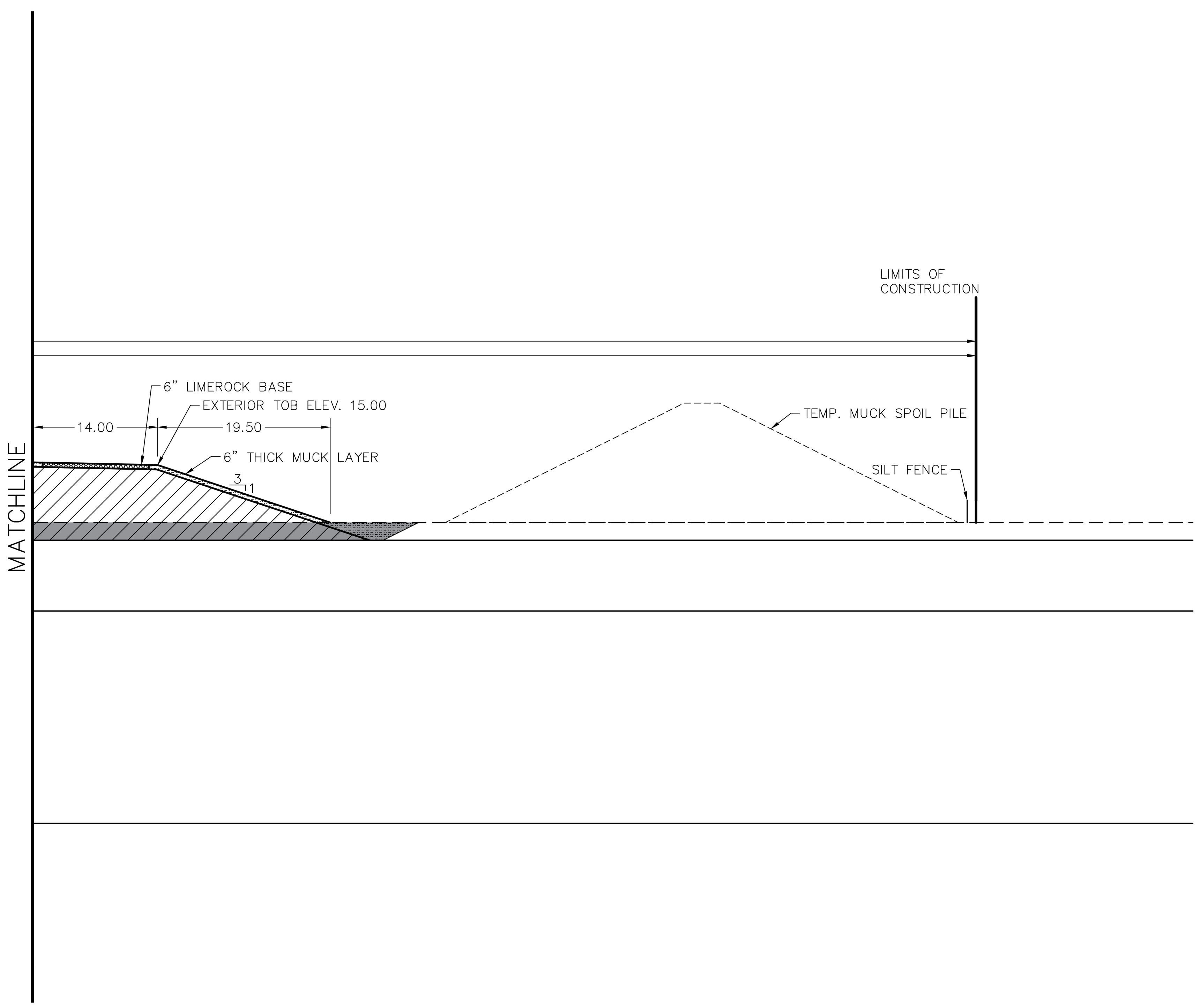
NOTE:
1. ELEVATIONS SHOWN HEREON ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88). NGVD29 = NAVD88 + 1.43' FOR THE A-2 RESERVOIR & A-2 STA SITES.

TYPICAL SECTION G

NOT TO SCALE

NOTE:
1. ELEVATIONS SHOWN HEREON ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88). NGVD29 = NAVD88 + 1.43' FOR THE A-2 RESERVOIR & A-2 STA SITES.





LEGEND:

- MUCK FILL
- MUCK REMOVAL
- RANDOM FILL (D_{MAX} = 6")
- RANDOM FILL (D_{MAX} = 12")
- RANDOM FILL (D_{MAX} = 24")
- LIMEROCK BASE
- RIPRAP
- CONCRETE
- FILTER FILL
- CORE FILL
- CAPROCK EXCAVATION
- FORT THOMPSON EXCAVATION

TYPICAL SECTION H

NOT TO SCALE

NOTE:
1. ELEVATIONS SHOWN HEREON ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88). NGVD29 = NAVD88 + 1.43' FOR THE A-2 RESERVOIR & A-2 STA SITES.

LEGEND:

- MUCK FILL
- MUCK REMOVAL
- RANDOM FILL (D_{MAX} = 6")
- RANDOM FILL (D_{MAX} = 12")
- RANDOM FILL (D_{MAX} = 24")
- LIMEROCK BASE
- RIPRAP
- CONCRETE
- FILTER FILL
- CORE FILL
- CAPROCK EXCAVATION
- FORT THOMPSON EXCAVATION

TYPICAL SECTION J-1(L)

NOT TO SCALE

NOTE:
1. ELEVATIONS SHOWN HEREON ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88). NGVD29 = NAVD88 + 1.43' FOR THE A-2 RESERVOIR & A-2 STA SITES.

TYPICAL SECTION K(L)

NOT TO SCALE

NOTE:
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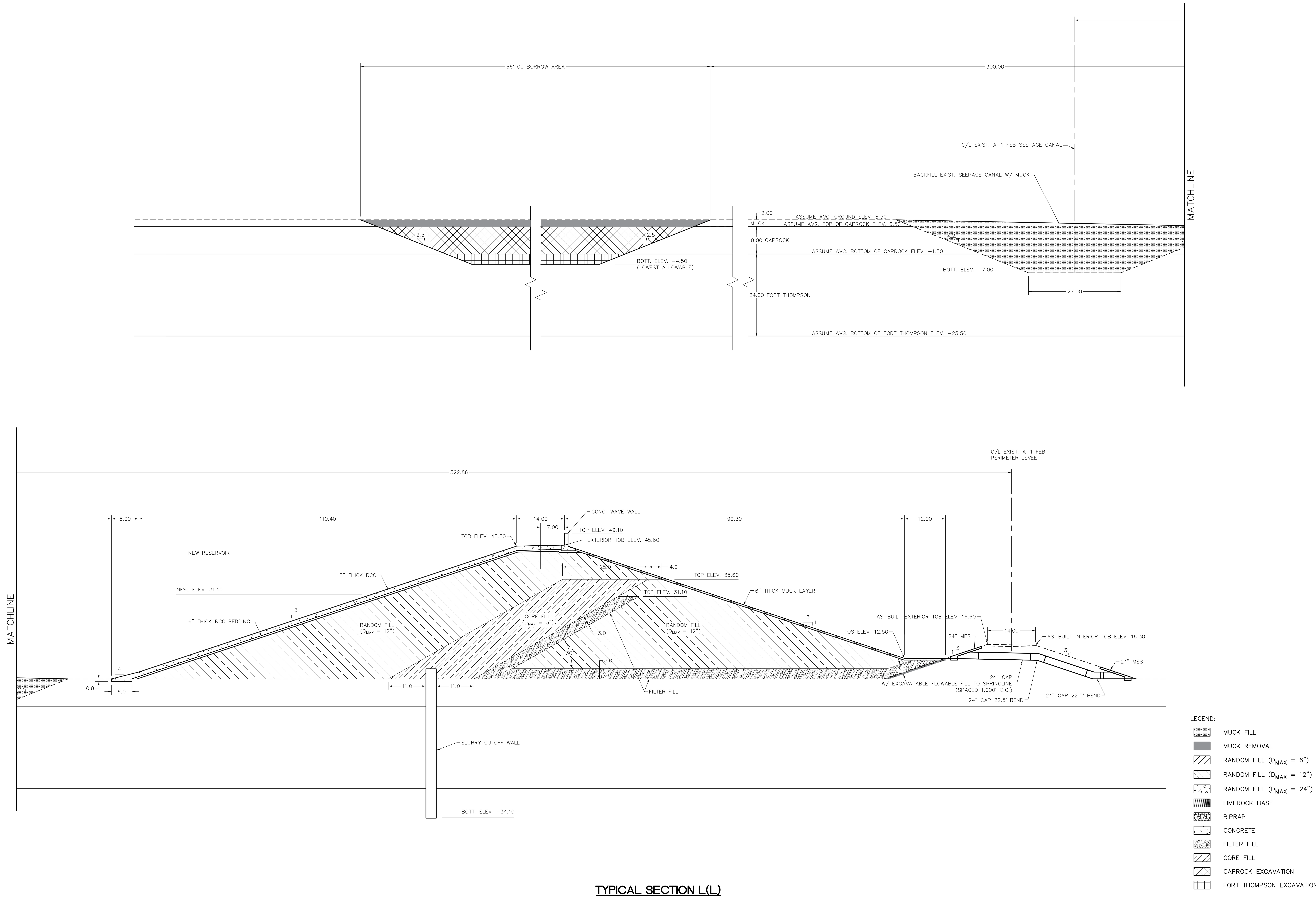

TYPICAL SECTION L(L)
NOT TO SCALE

NOTE:
1. ELEVATIONS SHOWN HEREON ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88). NGVD29 = NAVD88 + 1.43' FOR THE A-2 RESERVOIR & A-2 STA SITES.

TYPICAL SECTION N
NOT TO SCALE

NOTE:
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LEGEND:

- MUCK FILL
- MUCK REMOVAL
- RANDOM FILL (D_{MAX} = 6")
- RANDOM FILL (D_{MAX} = 12")
- RANDOM FILL (D_{MAX} = 24")
- LIMEROCK BASE
- RIPRAP
- CONCRETE
- FILTER FILL
- CORE FILL
- CAPROCK EXCAVATION
- FORT THOMPSON EXCAVATION

TYPICAL SECTION N-1
NOT TO SCALE

NOTE:
1. ELEVATIONS SHOWN HEREON ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88). NGVD29 = NAVD88 + 1.43' FOR THE A-2 RESERVOIR & A-2 STA SITES.

ANNEX D-1
MECHANICAL PLATES (SELECTED SFWMD GUIDELINE DRAWINGS)

- Drawing G-G1 (1 of 4) – Cover Sheet
- Drawing G-G1 (2 of 4) – Sheet Index
- Drawing G-G1 (3 of 4) – Standard Abbreviations
- Drawing G-G1 (4 of 4) – Design Requirements
- Drawing G-S1 (1 of 2) – Water Control Struct. w/ Vert. Lift Roller Gate Section & Elevation
- Drawing G-S1 (2 of 2) – Water Control Struct. w/ Vert. Lift Roller Gate Plan
- Drawing G-S7 (1 of 2) – Control Building w/ Generator Plan
- Drawing G-S7 (2 of 2) – Control Building w/ Generator Elevations
- Drawing G-S13 (2 of 2) – Gated Box Culvert Double Barrel Arrangement

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ENGINEERING DESIGN STANDARDS FOR WATER RESOURCE FACILITIES

DESIGN GUIDELINES

JULY 2008

The consultant, contractor or other parties associated with District projects shall comply with Florida Statutes 119.07 (6) (ee). These Standards and Guidelines are the property of the District and must be secured and maintained in a confidential manner. Review by any unauthorized individual or outside/third party not performing work necessary for District projects is prohibited.

Users of these standards and guidelines are advised that the users are responsible for the function and safety of the intended facilities. Any changes must be approved by the District. District approval does not relieve the user of their responsibility.

STANDARDS FOR CONSTRUCTION OF WATER RESOURCE FACILITIES
DESIGN GUIDELINES

INDEX TO DRAWINGS

DWG #	SHEET #	TITLE	LATEST UPDATE	UPDATE INFORMATION
		GENERAL		
G-G1	1 OF 4	COVER SHEET	Aug-07	general revision list update
G-G1	2 OF 4	SHEET INDEX	Aug-07	Added Update Information
G-G1	3 OF 4	Standard Abbreviations	May-06	-
G-G1	4 OF 4	Design Requirements	Aug-07	Added Security Design Requirements
G-I1	1 OF 1	SCADA System - Type 1 Pump Station	Feb-07	New guideline drawing
G-I2	1 OF 1	SCADA System - Type 2 Pump Station	Feb-07	New guideline drawing
G-I3	1 OF 1	SCADA System - Type 3 Pump Station	Feb-07	New guideline drawing
G-S1	1 OF 2	Water Control Structure w/ Vert. Lift Roller Gate Sec. and Elev.	Oct-06	Changed remarks for dimension j and key note 3; added cast-in-place option to gen note 1.
G-S1	2 OF 2	Water Control Structure w/ Vert. Lift Roller Gate Plan	Aug-07	Added Security Design Requirements
G-S2	1 OF 3	Culvert - Muti Metal Barrel w/ Slide Gate	Aug-07	Added Design Note no. 6
G-S2	2 OF 3	Culvert - Muti Metal Barrel Elevations, Sections and Details	Jan-06	Added cut marks to plan; added seep shield (20) to Sec A; extended arrow for #19, Sec E; lowered culverts in Elev. View; added L to note in Detail 1
G-S2	3 OF 3	Culvert - Muti Metal Barrel Key Notes	Jul-06	added spec no's, changed mark 2,4,6,9,12,20,b
G-S3	1 OF 4	Pump Station Type 1 Diesel Driver - Section	Aug-07	Added Security Design Requirements
G-S3	2 OF 4	Pump Station Type 1 Diesel Driver - Elevations	Aug-07	Added note #9
G-S3	3 OF 4	Pump Station Type 1 Diesel Driver - General Arrangement Plan	Oct-06	added Z dims to rt. side of plan view
G-S3	4 OF 4	Pump Station Type 1 Diesel Driver - Key Notes	Aug-07	Added Key Notes 52, 55, & 56
G-S4	1 OF 4	Pump Station Type 2 Elec Motor Driver - Section	Aug-07	Added Security Design Requirements
G-S4	2 OF 4	Pump Station Type 2 Elec Motor Driver - Elevations	Aug-07	Added notes 8 & 9
G-S4	3 OF 4	Pump Station Type 2 Elec Motor Driver - General Arrangement Plan	Oct-06	added Z dims to rt side of plan view
G-S4	4 OF 4	Pump Station Type 2 Elec Motor Driver - Key Notes	Aug-07	Added Key Notes 55, 58 & 59
G-S5	1 OF 2	Pump Station Vertical Type 3 - Electric Submersible - Plan	Aug-07	Added Security Design Requirements
G-S5	2 OF 2	Pump Station Vertical Type 3 - Electric Submersible - Elev & Section	Jul-06	ADDED 21,22
G-S6	1 OF 6	Single Lane Bridge - Site Plan and Sections	Jul-06	added notes and signs and remarks
G-S6	2 OF 6	Single Lane Bridge - Plan, Sections & Elev.	Jul-06	changed key note 3,5,6,9. changed 20' to 30', 9'-6" to 9'-6 1/2"
G-S6	3 OF 6	Bridge Plan & Details of End Bents 1 and 6	Jul-06	changed mark numbers
G-S6	4 OF 6	Bridge Plan & Details of Intermediate Bents	May-06	added bent 2-5 in bridge elev. Table
G-S6	5 OF 6	Bridge Superstructure - Plan and Details	Jun-05	-
G-S6	6 OF 6	Single Lane Bridge Superstructure - Unit A and B Details	Jun-05	-
G-S7	1 OF 2	Control Building w/ Generator - Plan	Aug-07	Revised Notes 2A2, C6, F, G
G-S7	2 OF 2	Control Building w/ Generator - Elevations	Aug-07	Added provisions for future card reader, lighting, and security camera
G-S8	1 OF 2	Control Building w/o Generator - Plan	Aug-07	Revised Notes 2A2, C6, F, G
G-S8	2 OF 2	Control Building w/o Generator - Elevations	Aug-07	Added provisions for future card reader, lighting, and security camera
G-S9	1 OF 1	Gauge Monitoring Layout	Aug-07	Rev Title; Rev Notes 5 & 8
G-S10	1 OF 1	Control Structure Layout	Aug-07	Revised notes 8 & 10
G-S11	1 OF 3	Primary Microwave Communications Site Plan	Aug-07	Added General Notes 7 & 8 and Key Note 32; Revised Key Note 21
G-S11	2 OF 3	Primary Microwave Communications Site - Equipment Shelter	Aug-07	Revised Key Note 25 and added 54 & 55
G-S11	3 OF 3	Primary Microwave Communications Site - Generator Building	Aug-07	Revised Key Note 1 and added notes 24 & 25
G-S12	1 OF 1	Generator Building	Aug-07	Revised Notes 2A2, C5, E, F
G-S13	1 OF 2	Gated Box Culvert - Single Barrel Arrangement	Aug-07	New Guideline Drawing Created from Detail Drawing C6
G-S13	2 OF 2	Gated Box Culvert - Double Barrel Arrangement	Aug-07	New Guideline Drawing Created from Detail Drawing C6

PLOT SCALE: scale
FILE NAME: G-G1.02

APPROVED approved
REVISION NO. rev #
EFFECTIVE DATE Aug 2007

SOUTH FLORIDA
WATER MANAGEMENT DISTRICT

STANDARD GENERAL
GUIDELINES
SHEET INDEX

G-G1

ABBREVIATIONS AND ACRONYMS – STANDARD

AASHTO	AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS
ACI	AMERICAN CONCRETE INSTITUTE
ADD'L	ADDITIONAL
AL	ALUMINUM
ALUM	ALUMINUM
APPROX	APPROXIMATELY
ASCE	AMERICAN SOCIETY OF CIVIL ENGINEERS
ASTM	AMERICAN SOCIETY FOR TESTING AND MATERIALS
AWG	AMERICAN WIRE GAUGE
BLDG	BUILDING
BOTT	BOTTOM
CFM	CUBIC FEET PER MINUTE
CFS	CUBIC FEET PER SECONDS
CL	CLEAR
CLR	CLEARANCE
CMP	CORRUGATED METAL PIPE
D	DIAMETER
DIA	DIAMETER
DIM	DIMENSION
DOT	FLORIDA DEPARTMENT OF TRANSPORTATION
DW	DESIGN WATER
DWG	DRAWING
E	EXPANSION END
ELEV	ELEVATION
ELEC	ELECTRIC OF ELECTRICAL
ETC	ET CETERA
EXT	EXTERIOR
F	FIXED END
FDOT	FLORIDA DEPARTMENT OF TRANSPORTATION
FF	FINISHED FLOOR
FPL	FLORIDA POWER AND LIGHT
FPS	FEET PER SECOND
FT	FEET OR FOOT
GFI	GROUND FAULT INTERRUPTER
GRD	GROUND
HW	HEAD WATER
LB	POUND
MAX	MAXIMUM
MFGR	MANUFACTURER
MIN	MINIMUM
MPH	MILES PER HOUR
NEMA	NATIONAL ELECTRICAL MANUFACTURERS ASSOCIATION
NO.	NUMBER
OC	ON CENTER
PL	POLYMER CONCRETE
POS	POSITION
PROJ	PROJECTION
PSF	POUNDS PER SQUARE FOOT
PSI	POUNDS PER SQUARE INCH
Q	FLOW RATE
R	RADIUS
REF	REFERENCE
REIF	REINFORCEMENT
REQD	REQUIRED
RTU	REMOTE TELEMETRY UNIT
R/W	RIGHT OF WAY
S.F.W.M.D.	SOUTH FLORIDA WATER MANAGEMENT DISTRICT
SQ	SQUARE
SS	STAINLESS STEEL
SPECS	SPECIFICATIONS
SST	STAINLESS STEEL TYPE 304
TJB	TELEMETRY JUNCTION BOX
TW	TAILWATER
TYP	TYPICAL
UL	UNDERWRITERS LABORATORIES INC.
UPS	UNINTERRUPTED POWER SUPPLY

SYMBOLOGY

'	FEET
"	INCHES
#	NUMBER
%	PERCENT
&	AND
@	AT
°	DEGREES
W/	WITH
X	BY
℄	CENTERLINE
&	AND
ø	DIAMETER

STRUCTURAL SHAPES

C	CHANNEL
L	ELBOW
PL	PLATE
W	W SHAPE

PLOT SCALE: 1:1
FILE NAME: G3_01.DWG

The consultant, contractor or other parties associated with District projects shall comply with Florida Statutes 119. These Standards and Guidelines are the property of the District and must be secured and maintained in a confidential manner. Review by any unauthorized individual or outside/third party not performing work necessary for District projects is prohibited.

APPROVED
REVISION NO.
EFFECTIVE DATE May 2006

SOUTH FLORIDA
WATER MANAGEMENT DISTRICT

STANDARD GENERAL DETAILS

G-G1

3 SHEET OF 3

PUMP DESIGN REQUIREMENTS:

FLOW DATA ARE NEEDED FOR WATER SUPPLY, ENVIRONMENTAL PROTECTION, FLOOD CONTROL, AND ECOSYSTEM RESTORATION. PUMP FLOW DISCHARGE ESTIMATION ACCURACY DEPENDS ON THE ACCURACY OF STAGE (UPSTREAM (U/S), DOWNSTREAM (D/S)) AND ENGINE SPEED DATA. FOR FLOW DATA MONITORING PURPOSE, THE FOLLOWING REQUIREMENTS SHOULD BE CONSIDERED IN THE DESIGN OF PUMP DESIGN.

1. PUMP SIZES AND NUMBER OF PUMPS MUST MEET HIGH AND LOW FLOW NEEDS. THE TOTAL CAPACITY MUST ACCOMMODATE MAXIMUM FLOW EXPECTED WHILE AT LEAST ONE LOW-CAPACITY UNIT SHOULD BE PROVIDED TO HANDLE VERY LOW FLOWS EXPECTED AT THE LOCATION.

2. FOR MULTIPLE PUMPS, DESIGN FOR SINGLE AND PARALLEL OPERATION CONDITIONS. IF MULTIPLE PUMPS NEED TO BE OPERATED SIMULTANEOUSLY, THE DESIGN SHOULD PROVIDE FOR SIMILAR CAPACITY PUMPS OPERATING ANY ONE TIME FOR BETTER FLOW MONITORING ACCURACY.

3. PUMP STATION DESIGNS MUST FACILITATE ACCURATE FLOW ESTIMATION WITH SUBMERGED PUMP INLET AND OUTLET CONDITION AT ALL TIMES. THE APPROACH AND DISCHARGE CANALS SHALL BE WELL DEFINED WITHOUT ABRUPT CHANGE OF DIRECTION.

4. IN ADDITION, THEY SHALL BE LONG ENOUGH SO THAT THE WATER STAGES CAN BE MEASURED WITHOUT SIGNIFICANT EFFECT OF THE DRAWDOWN CURVE EXPRESSED IN TERMS OF THE VELOCITY HEAD NOR EDDIES THAT MAY RESULT FROM ASYMMETRIC FLOWS INDUCED BY UNEVEN OPERATION OF THE PUMPS.

5. ACCURATE U/S AND D/S STAGE MEASUREMENT FACILITIES (STILLING WELLS) MUST BE PROVIDED WITH ORIENTATIONS AND LOCATIONS FREE FROM SURGE AND DRAWDOWN EFFECTS OF THE PUMP UNITS.

6. SELECTION OF PUMP UNIT CAPACITIES SHOULD FACILITATE OPTIMUM (HIGH EFFICIENCY) OPERATION CONDITIONS FOR ALL UNITS AS MUCH AS POSSIBLE.

7. ACCESS TO BOTH U/S AND D/S SIDE OF THE PUMP STATION SHOULD BE INCLUDED TO FACILITATE FIELD FLOW MEASUREMENTS NEEDED FOR RATING CALIBRATION AND VERIFICATION. BOAT RAMPS SHOULD BE PROVIDED FOR STREAMGAUGING.

8. THE DESIGN REPOST SHALL INCLUDE PUMP PERFORMANCE CURVES AND FLOW RATING EQUATIONS APPLICABLE TO THE PROPOSED DESIGN AND OPERATION. THE RATING EQUATIONS SHALL COVER THE EXPECTED OPERATING RANGES OF HEADWATER, TAILWATER, AND ENGINE/PUMP SPEED.

9. THE DESIGN SHOULD CONSIDER MINIMIZING HEAD LOSS IN THE SYSTEM.

CULVERT DESIGN REQUIREMENTS:

1. THE CULVERT SHOULD BE DESIGNED IN SUCH A WAY THAT IT IS OPERATED UNDER FULL PIPE FLOW CONDITION AS MUCH AS POSSIBLE.
2. FLASH BOARD AT THE CULVERT SHOULD BE AVOIDED FOR ACCURATE FLOW MONITORING. AN APPROACH CHANNEL (AT LEAST 25 FT LONG, PREFERABLY 50 FT OR LONGER, IF POSSIBLE) IN FRONT OF THE CULVERT SHOULD BE PROVIDED FOR STREAMGAUGING. STREAMGAUGING IS NECESSARY FOR ACCURATE FLOW COMPUTATION AT THE CULVERT.
3. BOAT RAMP FOR STREAMGAUGING SHOULD BE PROVIDED.
4. SENSORS SHOULD BE CAREFULLY SELECTED SO THAT THE DEVICE CAN BE ACCURATELY CALIBRATED BEFORE THE CULVERT OPERATION STARTS.
5. GATE OPENING INDICATOR WITH HIGH ACCURACY (AT ABOUT 0.15 FT) THAT CAN BE EASILY READ AT THE SITE IS DESIRABLE.
6. WHEN REQUIRED, HEADWALLS SHALL NOT BE CONSTRUCTED OF SAND CEMENT RIP-RAP.

RERERENCES:

U.S. BUREAU OF RECLAMATION (1987). DESIGN OF SMALL DAMS, 3RD ED., U.S. DEPARTMENT OF INTERIOR, DENVER, COLORADO.

U.S. BUREAU OF RECLAMATION (2001). WATER MEASUREMENT MANUAL, U.S. DEPARTMENT OF INTERIOR, DENVER, COLORADO. [HTTP://USBR.GOV/PMTS/HYDRAULICS_LAB/PUS/EMM/]

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BOS, M.G., REPLOGLE, J.A., AND CLEMENS, B. (1984) FLOW MEASURING FLUMES FOR OPEN CHANNEL SYSTEMS, JOHN WELY & SONS, INC., AND AMERICAN SOCIETY OF AGRICULTURAL ENGINEERS, NEW YORK, 1991.

SOUTH FLORIDA WATER MANAGEMENT DISTRICT (2004). QUALITY ASSURANCE PRACTICES FOR HYDROMETEOROLOGICAL AND HYDRAULIC MONITORING. DRAFT REPORT. [FTP://FTP.SFWMD.GOV/PUB/JGONZAL/QASR/QAPHHM-DRAFT.PDF]

SPILLWAY DESIGN REQUIREMENTS:

1. SPILLWAY DESIGN, INCLUDING CREST TYPE, GEOMETRY AND DIMENSIONS OF THE APPROACH CANAL AND ENERGY DISSIPATING STRUCTURE SHALL FOLLOW THE STANDARD DESIGN GUIDELINES DEVELOPED BY THE USSR AND USACE AS DESCRIBED IN PUBLICATIONS SUCH AS THE DESIGN OF SMALL DAMS AND HYDRAULIC DESIGN OF SPILLWAYS MANUALS. TYPICALLY, THE WEIR CREST GEOMETRY SHALL CONFIRM TO THE FOLLOWING CRITERIA:
   a. UPSTREAM FACE SHALL BE VERTICAL.
   b. THE DOWNSTREAM PORTION OF THE OGEE CREST SHALL HAVE THE SHAPE SHOWN IN FIGURE 9-21(A) IN THE DESIGN OF SMALL DAMS MANUAL. THIS PORTION IS GIVEN BY Y/HO--K (X/HO)N, WHERE BOTH K AND N ARE FUNCTIONS OF HA/HO AS SHOWN IN FIGURE 9-21(B) AND (C) IN THE DESIGN OF SMALL DAMS MANUAL.
   c. THE UPSTREAM CREST IS A COMPOUND CURVE WITH THE MINOR RADIUS AND MAJOR RADIUS GIVEN AS A FUNCTION OF THE RATIO OF THE VELOCITY HEAD AND DESIGN HEAD HA/HO AS SHOWN IN FIGURE 9-21(F) IN THE DESIGN OF SMALL DAMS MANUAL.
   d. CONNECTION TO THE STILLING BASIN APRON SHALL BE A 10-FT RADIUS CURVE MAX.
2. THE STRUCTURAL STABILITY OF THE SPILLWAY AND RELATED WORKS SHOULD BE ASSESSED BASED ON EXTREME FLOWS ACCORDING TO CURRENT ENGINEERING PRACTICE GUIDELINES.
3. THE BANKS AND BED OF THE APPROACH CHANNEL AND DISCHARGE CANALS AND RESPECTIVE REVETMENTS SHOULD BE ABLE TO SUSTAIN THE TRACTIVE FORCES RESULTING FROM MEAN AND EXTREME FLOWS UNDER THE DESIGN OPERATING CONDITIONS. NON-ERODING CONDITIONS SHOULD BE INSURED AT ALL TRANSITIONS FROM THE STRUCTURALLY - PROPOSED REVETMENT TO NATURAL STREAMBED TO INSURE STRUCTURAL STABILITY.
4. THE APPROACHING AND DISCHARGE CANALS SHALL BE WELL DEFINED WITHOUT ABRUPT CHANGE OF DIRECTION. IN ADDITION, THEY SHALL BE LONG ENOUGH SO THAT THE WATER STAGES CAN BE MEASURED WITHOUT SIGNIFICANT EFFECT OF THE DRAWDOWN CURVE EXPRESSED IN TERMS OF THE VELOCITY HEAD NOR EDDIES THAT MAY RESULT FROM ASYMMETRIC FLOWS INDUCED BY UNEVEN OPERATION OF THE GATES. SOME PRACTICAL GUIDELINES ON APPROACH FLOW CONDITIONS NEAR A MONITORING SITE CAN BE FOUND IN CHAPTER 2 OF THE USSR WATER MEASUREMENT MANUAL (2001).
5. FINAL DESIGN SHALL ABIDE TO THE OPERATIONAL PLAN IN TERMS OF STAGES AND STRUCTURE OPERATION AS CLOSE AS POSSIBLE SO THAT THE FREQUENCY OF LEAST-ACCURATE, DIFFICULT-TO-MONITOR TYPE OF FLOW SUCH US UNCONTROLLED SUBMERGED FLOW IS MINIMIZED FROM THE STAND POINT OF TOTAL VOLUME. NOT ABIDING TO THIS CONSTRAINT MAY RESULT IN INACCURATE MONITORING FOR ENVIRONMENTAL COMPLIANCE.
6. GENERAL GUIDELINES ON INSTRUMENTATION FOR STAGE MONITORING ARE GIVEN IN THE QUALITY ASSURANCE SYSTEM REQUIREMENT REPORT (SFWMD, 2004). LOCATION OF STILLING WELLS FOR HEADWATER MONITORING SHOULD FOLLOW THE RECOMMENDATION OF BOS ET AL. (1981), I.E., IT SHOULD BE LOCATED AT A DISTANCE NOT LESS THAN THREE TIMES HO FROM THE SPILLWAY'S SILL. HOWEVER, IT MAY NEED TO BE LOCATED FURTHER UPSTREAM TO INSURE A MINIMAL DRAWDOWN EFFECT. THE WELLS SHOULD COMPLY WITH THE BASIC CONSIDERATIONS TO DAMPEN THE EFFECT OF SURGES AND WIND WAVES OF THE OUTSIDE PROPOSED IN CHAPTER 6 OF THE USBR WATER MEASUREMENT MANUAL (2001).
7. ACCESS TO BOTH THE UPSTREAM AND DOWNSTREAM SIDE OF THE STRUCTURE SHOULD BE INCLUDED WHENEVER POSSIBLE FOR THE PURPOSE OF FIELD FLOW MEASUREMENTS NEEDED FOR RATING CALIBRATION AND VERIFICATION, AS WELL AS FOR MONITORING OF THE BEDSTREAM STABILITY DURING EXTREME EVENT OR HARSH OPERATIONAL CONDITIONS.
8. DESIGNS DEPARTING FROM THE USBR AND USACE GUIDELINES SHALL BE TECHNICALLY JUSTIFIED AND SUPPORTED BY DATA GATHERED AT FULL OR REDUCED SCALE STRUCTURES OR OBTAINED FROM COMPUTATIONAL FLUID DYNAMICS SIMULATIONS OF THE FLOW THROUGH STRUCTURES OF SIMILAR GEOMETRY, LAYOUT AND HYDRAULIC DESIGN CONDITIONS.
9. DESIGN DRAWINGS SHOULD BE SUBMITTED WITH A DESIGN ANALYSIS REPORT
10. THE DESIGN REPORT SHOULD SUMMARIZE ANY PERTINENT HYDROLOGIC AND HYDRAULIC ANALYSIS FOR: "SELECTING DESIGN PARAMETERS, DIMENSIONING OF THE STRUCTURE FOR INSURING STRUCTURAL STABILITY, AND INSURING THE STABILITY OF BOTH NATURAL STREAMBED AND CHANNEL BANKS".
11. THE REPORT SHALL ALSO INCLUDE RATING CURVES AND RATING EQUATIONS APPLICABLE TO THE PROPOSED DESIGN AND OPERATION. THE RATINGS CURVES AND EQUATIONS SHALL COVER ALL THE RANGE OF HEADWATER, TAILWATER, GATE OPENINGS AND OPERATING CONDITIONS AND FLOW TYPES EXPECTED AT THE STRUCTURE. SEPARATE RATING CHARTS AND EQUATIONS MUST BE PROVIDED FOR EACH OF THE FOLLOWING FLOW TYPES: CONTROLLED FREE, UNCONTROLLED FREE, CONTROLLED SUBMERGED AND UNCONTROLLED SUBMERGED FLOW.

SAFETY DESIGN REQUIREMENTS:

1. DESIGN OF PUMP STATIONS, WATER CONTROL STRUCTURES, GATED CULVERTS, NAVIGATIONAL LOCKS, BUILDINGS, ETC. SHALL INCORPORATE ADEQUATE MEASURES AND PERMANENT FEATURES THAT WILL PROVIDE PROTECTION AGAINST FALLS GREATER THAN 4 FEET BY FACILITATING THE INSTALLATION OF ENGINEERED FALL PROTECTION, PREVENTION, RESTRAINT AND ARREST SYSTEMS SUCH AS STANDARD RAILINGS ALONG THE EDGES OF LEVEL CHANGES, ENGINEERING ANCHOR POINTS FOR UTILIZATION IN AREAS INCAPABLE OF HAVING STANDARD RAILINGS INSTALLED, PROPERLY PROTECTED STAIRS FOR FREQUENTLY USED LEVEL CHANGES GREATER THAN 4 FEET, FIXED VERTICAL LADDERS EQUIPPED WITH LADDER CLIMBING DEVICES, EXTENDED HANDRAILS AT TRANSITION POINTS AND SELF-CLOSING ACCESS GATES AND ENGINEERED HORIZONTAL LIFELINES (DESIGNED FOR MIN. 2 WORKERS). FALL PREVENTION DEVICES, HORIZONTAL LIFELINE, VERTICAL LIFELINE AND PERSONAL FALL ARREST SYSTEMS SHALL BE PROVIDED AS COMPLETE, PRE-MANUFACTURED SYSTEMS INCLUDING ANCHOR COMPONENTS AND INSTALLED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.
2. OPEN SIDED FLOORS, PLATFORMS, WALKWAYS, LANDINGS, AND RUNWAYS REQUIRE STANDARD RAILINGS WHEN THE CHANGE IN ELEVATION BETWEEN THE WALKING SURFACE AND THE NEXT LOWER LEVEL IS 4 FEET OR GREATER. STANDARD RAILINGS SHALL RESIST A MIN. APPLIED FORCE OF 200 IBS. SEE DESIGN DETAIL DRAWING S5.
3. ELEVATED FLOORS, PLATFORMS, WALKWAYS, LANDINGS, RUNWAYS, AND OPENINGS IN FLOORS WHERE THE POTENTIAL OF WORKERS BELOW EXIST SHALL BE EQUIPPED WITH AN APPROVED TOE BOARD TO PREVENT FALL HAZARDS ASSOCIATED WITH TOOLS, PARTS, EQUIPMENT, ETC. SEE DESIGN DETAIL DRAWING S5.
4. GAPS OR OPENINGS BETWEEN HANDRAILS AND WALLS, FENCES, EQUIPMENT OR OTHER FIXED OBJECTS SHALL NOT EXCEED 4 INCHES. ALL ACCESS OPENINGS IN HANDRAILS SHALL HAVE AN APPROVED, SELF-CLOSING SWING GATE. CHAINS, ROPES, ETC. SHALL NOT BE USED TO PROTECT OPENINGS IN HANDRAIL SYSTEMS.
5. STANDARD RAILINGS SHALL BE PLACED ALONG THE OUTSIDE EDGE OF ALL ROOF STRUCTURES THAT ARE FREQUENTLY ACCESSED FOR ROUTINE OPERATIONS AND MAINTENANCE OR A PARAPET WALL SHALL BE PROVIDED WITH A MINIMUM HEIGHT OF 42". A STAIR AND WALKWAY SYSTEM SHALL BE UTILIZED FOR CROSSING INTERIOR PARAPETS OVER 19 INCHES HIGH.
6. EVERY FLIGHT OF STAIRS HAVING FOUR OR MORE RISERS SHALL BE EQUIPPED WITH STANDARD STAIR RAILINGS OR STANDARD HANDRAILS AS SPECIFIED IN OSHA PARAGRAPHS 29CFR1910.23(D)(1)(I) THROUGH (V) OF THIS SECTION, THE WIDTH OF THE STAIR TO BE MEASURED CLEAR OF ALL OBSTRUCTIONS EXCEPT HANDRAILS. ALL RISE AND RUN DIMENSIONS SHALL BE IDENTICAL.
7. DESIGNS FOR GATED STRUCTURES, CULVERTS, PUMP STATIONS, NAVIGATION LOCKS, AND BUILDINGS SHALL CONSIDER THE NEED FOR LADDERS TO ACCESS AREAS FOR ROUTINE OPERATION AND MAINTENANCE. WHERE THE USE OF PORTABLE LADDERS IS REQUIRED, DESIGNS SHALL INCLUDE THE LOCATION AND DETAILING OF PERMANENT ENGINEERED METHODS TO SECURE THE PORTABLE LADDERS IN PLACE. SEE DESIGN DETAIL DRAWINGS S1 AND S6.
8. FIXED LADDERS WITH FALL DISTANCES GREATER THAN OR EQUAL TO 20 FEET SHALL HAVE A CABLE LADDER CLIMBING SYSTEM FOR FALL PROTECTION. SEE DESIGN DETAIL DRAWING S6.
9. PROVIDE SAFETY WARNING SIGNS FOR BOATERS ON ALL DISTRICT FACILITIES THAT ARE ACCESSIBLE BY THE GENERAL PUBLIC (SEE DESIGN DETAIL C5). PROVIDE ILLUMINATION LIGHTING AND/OR REFLECTORS OR REFLECTIVE MARKING TAPE ON ALL STRUCTURES LOCATED WITHIN DISTRICT CANALS OR WATER BODIES THAT ARE ACCESSIBLE BY THE GENERAL PUBLIC DURING NON-DAYLIGHT HOURS.

WIND LOAD DESIGN REQUIREMENTS:

1. WIND LOADS ON ALL NEW FACILITIES SHALL BE DETERMINED IN ACCORDANCE WITH THE FLORIDA BUILDING CODE FOR BUILDINGS (FBCB)(LATEST EDITION), INCLUDING SECTION 1620, USING ASCE 7 (LATEST EDITION), MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES, EXCEPT AS NOTED HEREIN. THE VELOCITY PRESSURE (q) SHALL BE CALCULATED USING ASCE 7 (LATEST EDITION) AND MODIFIED AS NOTED BELOW.
2. FOR NONCRITICAL FACILITIES, WHICH HAVE NO IMPACT ON WATER SUPPLY, ENVIRONMENTAL PROTECTION OR FLOOD PROTECTION, THE ULTIMATE DESIGN WIND SPEED (Vult) SHALL BE DETERMINED AS SPECIFIED IN FBCB SECTION 1609.3 FOR RISK CATEGORY II STRUCTURES, CORRESPONDING TO A NOMINAL MEAN RECURRENCE INTERVAL (MRI) OF 50 YEARS.
3. FOR WATER CONTROL STRUCTURES, UNMANNED PUMP STATIONS AND OTHER UNMANNED FACILITIES THAT DO NOT PROVIDE FLOOD PROTECTION, THE ULTIMATE DESIGN WIND SPEED (Vult) SHALL BE DETERMINED AS SPECIFIED IN FBCB SECTION 1609.3 FOR RISK CATEGORY III AND IV STRUCTURES, CORRESPONDING TO A NOMINAL MEAN RECURRENCE INTERVAL (MRI) OF 100 YEARS.
4. FOR WATER CONTROL STRUCTURES AND PUMP STATIONS THAT PROVIDE FLOOD PROTECTION, AS WELL AS OTHER FACILITIES THAT MAY BE MANNED DURING A HURRICANE EVENT AND FOR COMMUNICATION SHELTERS, THE ULTIMATE DESIGN WIND SPEED (Vult) SHALL BE DETERMINED AS SPECIFIED IN FBCB SECTION 1609.3 FOR RISK CATEGORY III AND IV STRUCTURES. THE CALCULATED VELOCITY PRESSURE (q) SHALL BE MULTIPLIED BY 1.14 TO INCREASE THE NOMINAL MEAN RECURRENCE INTERVAL (MRI) TO 200 YEARS.
5. FOR "SAFE ROOMS" (THE PORTION OF A FACILITY PROVIDED FOR PERSONNEL PROTECTION, TO BE OCCUPIED DURING EXTREME CONDITIONS), THE ULTIMATE DESIGN WIND SPEED (Vult) SHALL BE DETERMINED AS SPECIFIED IN FBCB SECTION 1609.3 FOR RISK CATEGORY III AND IV STRUCTURES. THE CALCULATED VELOCITY PRESSURE (q) SHALL BE MULTIPLIED BY 1.32 TO INCREASE THE NOMINAL MEAN RECURRENCE INTERVAL (MRI) TO 500 YEARS.
6. DESIGN WIND PRESSURES (PSF) FOR COMPONENTS AND CLADDING SHALL BE SHOWN ON THE CONTRACT DOCUMENTS FOR BOTH ULTIMATE DESIGN WIND SPEED, (Vult) AND NOMINAL DESIGN WIND SPEED (Vasd).

SECURITY DESIGN REQUIREMENTS:

DESIGNER SHALL INCORPORATE THE FOLLOWING HIGH SECURITY FEATURES FOR FACILITY LOCATIONS AS DESIGNATED BY THE DISTRICT.
GENERAL FACILITY SITE PLANNING:

1. ANY BUILDINGS OR STRUCTURES ON THE SITE SHALL BE SET BACK A MINIMUM OF 75 FEET FROM ANY PERIMETER SECURITY FENCE.
2. ANY BUILDINGS OR STRUCTURES ON THE SITE SHALL BE SET BACK A MINIMUM OF 75 FEET FROM ANY PUBLIC PARKING AREA.
3. PUBLIC ACCESS DRIVES AND PARKING AREAS SHALL BE SEPARATE FROM OTHER PARKING AREAS AND BE PROVIDED WITH 14 INCH HIGH CURBS.
4. PROVIDE SECURITY BOLLARDS ACROSS DRIVES AND WALKWAYS AT CRITICAL LOCATIONS.
5. SECURITY FENCING SHALL BE CONTINUOUS AND FORM A COMPLETE SECURE PERIMETER OF THE FACILITY.
6. PROVIDE A MINIMUM 10 FOOT "CLEAR ZONE" ON THE NON-SECURE SIDE OF A SECURE PERIMETER FENCE. INDICATE THIS AREA MUST BE KEPT FREE OF SHRUBS AND TALL GRASS TO AID IN FENCE SURVEILLANCE.

FUEL SYSTEMS:

1. DUE TO SITE CONSTRAINTS, SHOULD AN ABOVE GROUND FUEL TANK BE LOCATED WITH LESS THAN THE RECOMMENDED SEPARATION DISTANCE FROM OTHER ASSETS, THE DESIGNER SHALL PROVIDE A BLAST WALL BETWEEN THE FUEL TANK AND ANY OTHER ABOVE GROUND ASSETS. BLAST WALL SHALL BE APPROPRIATELY DESIGNED TO PROTECT ADJACENT ASSETS FROM THE ANTICIPATED WORST CASE CATASTROPHIC FUEL TANK EXPLOSION.
2. ALL FUEL LINES SHALL BE INSTALLED UNDERGROUND TO THE GREATEST POSSIBLE EXTENT. ANY PORTIONS OF EXPOSED FUEL PIPING (ABOVE GROUND) SHALL BE SHIELDED FROM BALLISTIC DAMAGE AS DIRECTED BY THE DISTRICT.

PUMP STATIONS:

1. SECURITY FENCING SHALL BE CONTINUOUS AND FORM A COMPLETE SECURE PERIMETER OF THE FACILITY.
2. INCORPORATE BUILDING AND FACILITY "HARDENING" MEASURES AT LOCATIONS DESIGNATED BY THE DISTRICT AS HIGH SECURITY.

GATED SPILLWAYS:

1. SECURITY FENCING SHALL BE CONTINUOUS AND FORM A COMPLETE SECURE PERIMETER OF THE FACILITY.
2. INCORPORATE BUILDING AND FACILITY "HARDENING" MEASURES AT LOCATIONS DESIGNATED BY THE DISTRICT AS HIGH SECURITY.
3. PROVIDE ARMORING OF UPPER AND LOWER BLOCK AND TACKLE ASSEMBLIES AT LOCATIONS DESIGNATED BY THE DISTRICT AS HIGH SECURITY.

MICROWAVE TELEMETRY TOWERS:

1. SECURITY FENCING SHALL BE CONTINUOUS AND FORM A COMPLETE SECURE PERIMETER OF THE FACILITY. FENCING SHALL BE EXTENDED ALONG WINGWALLS AND DEWATERING BEAMS AS REQUIRED.
2. INCORPORATE BUILDING AND FACILITY "HARDENING" MEASURES AT LOCATIONS DESIGNATED BY THE DISTRICT AS HIGH SECURITY.
3. PROVIDE BULLET RESISTANT FENCE ENCLOSURE FOR WALL MOUNTED AIR CONDITIONING UNITS AT LOCATIONS DESIGNATED BY THE DISTRICT AS HIGH SECURITY.

FACILITY HARDENING MEASURES:

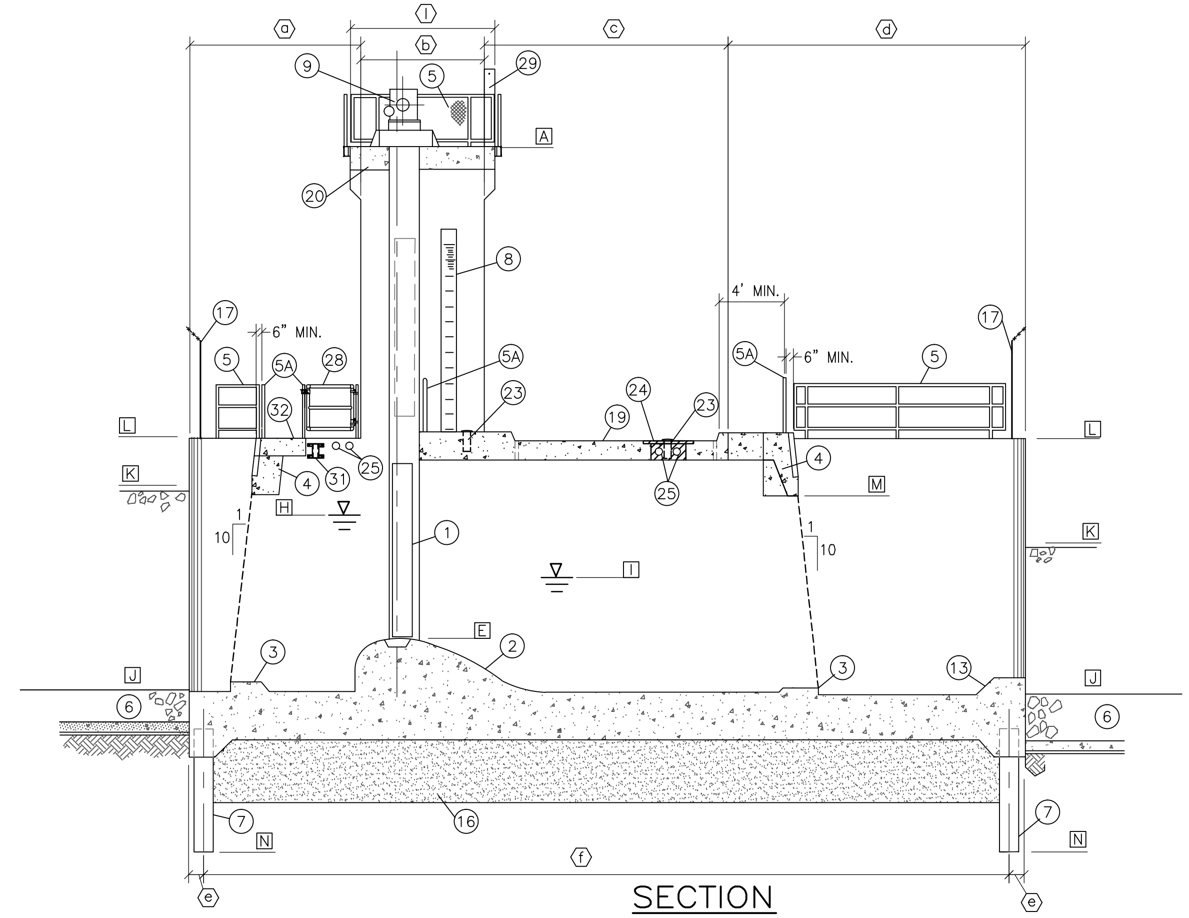
1. ARMORING OF UPPER AND LOWER BLOCK AND TACKLE ASSEMBLIES FOR GATED SPILLWAYS.
2. ARMORING OF ELECTRIC SERVICE/METER FOR BULLET RESISTANCE.
3. TAMPER RESISTANT HARDWARE AND CONCEALED HINGES FOR ELECTRICAL BOXES.
4. 6" THICK PRECAST CONCRETE WALLS FOR STRUCTURES.
5. BULLET RESISTANT LOUVERS AND/OR BULLET RESISTANT LOUVER HOODS.
6. TAMPER RESISTANT FASTENERS.

APPROVED ______
REVISION NO. V1.3
EFFECTIVE DATE MAR 2016
CHANGE NO. EDCR0592

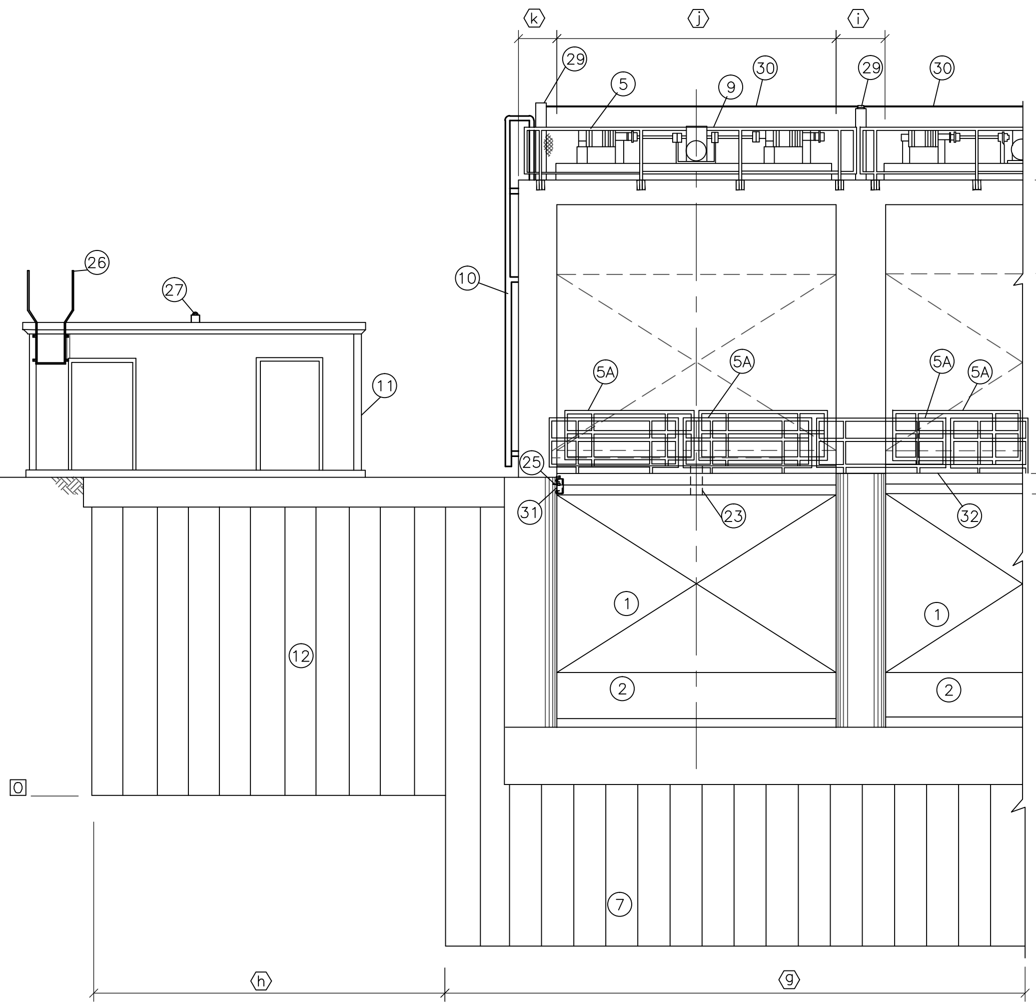
SOUTH FLORIDA
WATER MANAGEMENT DISTRICT

DESIGN GUIDELINES

G-G1



SECTION



ELEVATION

SCHEDULE — DIMENSION

MARK	DESCRIPTION	REQUIRED DIMENSION
a	FOREBAY	10'-0" (MIN)
b	TOWER WIDTH	8'-0" (MIN)
c	SERVICE BRIDGE	16'-0"
d	STILLING BASIN	16'-0" (MIN)
e	CUTOFF WALL SETBACK	1'-0" (MIN)
f	DISTANCE BETWEEN DRIVE LINES	48'-0" (MIN)
g	CUTOFF WALL	DEPENDENT ON BAY WIDTH AND NUMBER
h	WINGWALL	DEPENDENT ON CANAL SIDE SLOPE
i	PIER WIDTH	3'-0", TYP.
j	BAY WIDTH	AS PER HYDRAULIC DESIGN (SPECIFY IN WHOLE FOOT INCREMENTS)
k	ABUTMENT/COLUMN WIDTH	2'-6", MIN.
l	OPERATING PLATFORM	9'-0" (MIN) (MAINTAIN 28" MIN. CLEAR FOR EQUIP. ACCESS)

SCHEDULE — ELEVATIONS

MARK	DESCRIPTION	REMARKS
A	OPERATING PLATFORM	MINIMUM 6 FT. ABOVE TOP OF GATE IN OPEN POSITION
B	TOP OF GATE OPEN POSITION	BOTTOM OF GATE SHALL HAVE A MIN. 2 FT. CLEARANCE OVER DESIGN HIGH WATER STAGE IN OPEN POSITION
C	TOP OF SERVICE BRIDGE	SET ABOVE BYPASS ELEVATION OF STRUCTURE (MIN. 2 FT ABOVE DESIGN HIGHWATER STAGE FOR INTAKE)
D	TOP OF GATE CLOSED	GATE HEIGHT TO BE 2 FT. ABOVE MAXIMUM OPERATING POOL ELEVATION
E	WEIR CREST	AS PER HYDRAULIC DESIGN
F	SPILLWAY TOP OF SLAB	FLOOR TO BE SET AT CANAL BOTTOM ELEVATION
G	GRADE AT CONTROL BLDG.	SET AT OR ABOVE BYPASS ELEVATION OF STRUCTURE
H	DESIGN HEADWATER	AS PER HYDRAULIC DESIGN
I	DESIGN TAILWATER	AS PER HYDRAULIC DESIGN
J	PROPOSED CANAL ELEVATION	AS PER HYDRAULIC DESIGN
K	HEIGHT OF STONE PROTECTION	TOP OF STONE PROTECTION TO BE SET AT DESIGN HIGH WATER ELEVATION
L	TOP OF ABUTMENT OR PIER	SET AT SERVICE BRIDGE ELEVATION
M	BOTTOM OF NEEDLE BEAM	SET ABOVE DESIGN HIGH WATER WITH MAX. HEIGHT ABOVE FLOOR NOT GREATER THAN 15 FT. (22' IS STANDARD NEEDLE LENGTH)
N	BOTTOM OF SHEET PILE CUTOFF WALL	SET 10 FT. BELOW BOTTOM OF SLAB (MIN.)
O	BOTTOM OF SHEET PILE WINGWALL	SET AT BOTTOM OF SLAB MIN.

KEY NOTES:

MARK	DESCRIPTION	REMARKS
1	VERTICAL LIFT ROLLER GATE	SEE M5
2	WEIR CREST	SEE NOTE 6
3	DEWATERING NEEDLE SILL	SEE S3 & S4
4	CAST IN PLACE CONC. DEWATERING BEAM	SEE NOTE 8
5	STRUCTURE RAILING	SEE S5
5A	STRUCTURE RAILING -REMOVABLE	SEE S5
6	STONE PROTECTION	
7	STEEL SHEET PILE CUTOFF WALL	SEE NOTE 5
8	GATE POSITION INDICATOR	SEE M1
9	DRUM AND CABLE HOIST	SEE M7
10	LADDER WITH CABLE CLIMBING SYSTEM	SEE S6
11	CONTROL BUILDING	SEE G-S7
12	WINGWALL	SEE S8
13	END SILL	
14	APPROACH SLAB	10' x 14'
15	CONCRETE LANDING	6' x 8'
16	TREMIE	IF REQUIRED
17	SECURITY FENCE	SEE C4
18	SECURITY DOUBLE SWING GATE	SEE C4
19	SERVICE BRIDGE	
20	OPERATING PLATFORM	
21	TELEMETRY TOWER	SEE I2
22	FUEL TANK	SEE M4
23	CAST-IN-PLACE MAST-DAVIT SLEEVE	SEE GENERAL NOTE 9
24	SERVICE HATCH	4'x4' TRAFFIC BEARING
25	CONCRETE ANCHOR FOR PORTABLE LADDER	SEE GENERAL NOTE 12
26	PORTABLE LADDER SECURING SYSTEM	SEE GENERAL NOTE 12
27	ROOF TOP ANCHOR	SEE GENERAL NOTE 12
28	SELF CLOSING GATE	SEE S5
29	COLUMN FOR HORIZONTAL LIFELINE	SEE GENERAL NOTE 12
30	HORIZONTAL LIFELINE SYSTEM	SEE GENERAL NOTE 13
31	WALL MOUNT MAST DAVIT SLEEVE	SEE GENERAL NOTE 9
32	SERVICE WALKWAY BRIDGE (3-FT WIDE)	3-FT WIDE

APPLICATION:

CONTROL OF CANAL WATER STAGE UNDER LOW HEAD CONDITIONS.

GENERAL NOTES:

1. THE STRUCTURE CONSISTS OF CAST-IN-PLACE CONCRETE BASE SLAB, ABUTMENTS, COLUMNS, AND PIERS WITH A PRECAST OR REINFORCED CAST-IN-PLACE CONCRETE SERVICE BRIDGE AND OPERATING PLATFORM. ALL DESIGN SHALL BE IN ACCORDANCE WITH THE LATEST EDITION OF ACI-318.
2. THE REINFORCED CONCRETE DESIGN FOR THE ABUTMENTS SHALL ASSUME THE LATERAL EARTH PRESSURE DEVELOPED UNDER THE CONDITION OF A DEWATERED ADJACENT BAY WITH A SATURATED BACKFILL TO FINISHED GRADE AND AN ADDITIONAL 500psf OF SURCHARGE.
3. DESIGN LOADING FOR PIERS SHALL BE TAKEN AS HYDROSTATIC WITH THE ADJACENT BAY DEWATERED.
4. THE SERVICE BRIDGE LOADING SHALL BE AASHTO HL-93.
5. STEEL SHEET PILE CUTOFF WALL TO BE CONTINUOUS WITH WINGWALLS AND SHALL BE COMPOSED OF THE SAME SHEET TYPE.
6. PROVIDE FIRST ORDER BENCH MARK ON ABUTMENT WALL IN ACCORDANCE WITH DISTRICT STANDARDS.
7. STEEL BEAM AND RECESS IN CONCRETE MAY BE USED IN LIEU OF CAST IN PLACE BEAM WITH DISTRICT APPROVAL. SEE STANDARD DETAIL SHEET S3.
8. ALL DESIGNS SHALL CONSIDER THE FOLLOWING REFERENCES FOR DESIGN:

   ARMY CORPS OF ENGINEER'S DESIGN MANUALS
   - EM 1110-2-2000 STANDARD PRACTICE FOR CONCRETE FOR CIVIL WORKS STRUCTURES.
   - EM 1110-2-2104 STRENGTH DESIGN FOR REINFORCED CONCRETE HYDRAULIC STRUCTURES.
   - EM 1110-2-2505 DESIGN OF SHEET PILE WALLS.
   - ETL 1110-1-256 SLIDING STABILITY FOR CONCRETE STRUCTURES.
   - ETL 1110-1-307 FLOTATION STABILITY CRITERIA FOR CONCRETE HYDRAULIC STRUCTURES.
   - EC 1110-2-6058 STABILITY ANALYSIS OF CONCRETE STRUCTURES.

   FLORIDA BUILDING CODE
   - STEEL CONSTRUCTION MANUAL – AISC
   - AMERICAN WELDING SOCIETY – AWS
   - BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE – ACI 318.
9. MAST DAVIT SLEEVES SHALL BE TYPE 304 STAINLESS STEEL. ALL SLEEVES SHALL INCLUDE A TYPE 304 STAINLESS STEEL CAP WITH LETTERING TO READ: "USE MAST DAVIT MFG/MODEL # XXXXXX ONLY."
10. PROVIDE FLOATING SAFETY BARRIERS AND BOATER SAFETY WARNING SIGNS ON FACILITIES THAT ARE ACCESSIBLE BY THE GENERAL PUBLIC. SEE DESIGN DETAILS C3 AND C5.
11. PROVIDE ILLUMINATION LIGHTING FOR NIGHTTIME OPERATION OF FACILITY. SEE DESIGN DETAIL E15
12. FALL PROTECTION DESIGN DETAILS SHALL BE IN ACCORDANCE WITH MANUFACTURER RECOMMENDATIONS AND REQUIREMENTS FOR EACH COMPONENT. COORDINATE MANUFACTURER SELECTION AND DESIGN WITH DISTRICT STAFF.
13. HORIZONTAL LIFELINE SYSTEMS AND COLUMNS SHALL BE DESIGNED FOR (MIN.) 2 WORKERS.

PLOT SCALE: 1:1
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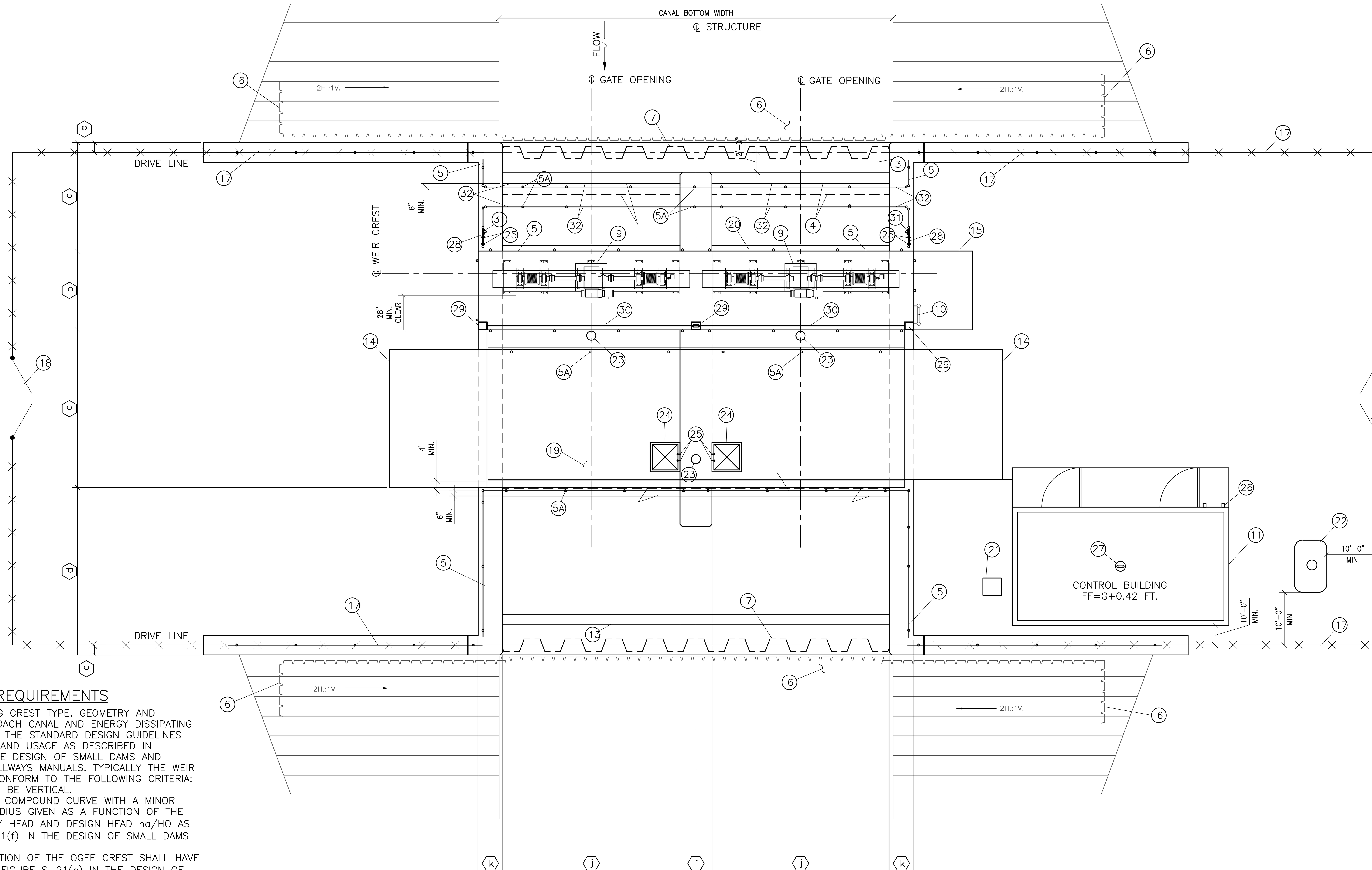
REVISION NO. V1.2
EFFECTIVE DATE MAY 2012
CHANGE NO. EGCR0054, EGCR0080, EGCR0246

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

STANDARD GUIDELINES
WATER CONTROL STRUCTURE WITH VERTICAL LIFT ROLLER GATE
SECTION AND ELEVATION

G-S1



PLAN

SPILLWAY DESIGN REQUIREMENTS

1. SPILLWAY DESIGN, INCLUDING CREST TYPE, GEOMETRY AND DIMENSIONS OF THE APPROACH CANAL AND ENERGY DISSIPATING STRUCTURE SHALL FOLLOW THE STANDARD DESIGN GUIDELINES DEVELOPED BY THE USBR AND USACE AS DESCRIBED IN PUBLICATIONS SUCH AS THE DESIGN OF SMALL DAMS AND HYDRAULIC DESIGN OF SPILLWAYS MANUALS. TYPICALLY THE WEIR CREST GEOMETRY SHALL CONFORM TO THE FOLLOWING CRITERIA:
   a. UPSTREAM FACE SHALL BE VERTICAL.
   b. UPSTREAM CREST IS A COMPOUND CURVE WITH A MINOR RADIUS AND MAJOR RADIUS GIVEN AS A FUNCTION OF THE RATIO OF THE VELOCITY HEAD AND DESIGN HEAD ha/HO AS SHOWN IN FIGURE 9-21(f) IN THE DESIGN OF SMALL DAMS MANUAL.
   c. THE DOWNSTREAM PORTION OF THE OGEE CREST SHALL HAVE THE SLOPE SHOWN IN FIGURE S-21(c) IN THE DESIGN OF SMALL DAMS MANUAL. THIS PORTION IS GIVEN BY Y/Ho=K(X/Ho) WHERE BOTH K AND n ARE FUNCTIONS OF ha/Ho AS SHOWN IN FIGURE 9-21(b) AND (c) IN THE DESIGN OF SMALL DAMS MANUAL.
   d. CONNECTION TO THE STILLING BASIN APRON SHALL BE 10ft. RADIUS CURVE MAX.
2. DESIGNS DEPARTING FROM THOSE GUIDELINES SHALL BE TECHNICALLY JUSTIFIED AND SUPPORTED BY DATA GATHERED AT FULL OR REDUCED SCALE STRUCTURE OR OBTAINED FROM COMPUTATIONAL FLUID DYNAMICS SIMULATIONS OF THE FLOW THROUGH STRUCTURES OF SIMILAR GEOMETRY, LAYOUT AND HYDRAULIC DESIGN CONDITIONS.
3. SUBMIT A DESIGN ANALYSIS REPORT SUMMARIZING THE DESIGN AND DIMENSIONING OF THE STRUCTURE. THE REPORT SHALL INCLUDE RATING CURVES AND EQUATIONS APPLICABLE TO THE PROPOSED SPILLWAY DESIGN FOR THE RANGE OF HEADWATER, TAILWATER AND GATE OPENINGS EXPECTED AT THE STRUCTURE.

SECURITY DESIGN REQUIREMENTS:

1. SECURITY FENCING SHALL BE CONTINUOUS AND FORM A COMPLETE SECURE PERIMETER OF THE FACILITY.
2. INCORPORATE BUILDING AND FACILITY "HARDENING" MEASURES AT LOCATIONS DESIGNATED BY THE DISTRICT AS HIGH SECURITY.
3. PROVIDE ARMORING OF UPPER AND LOWER BLOCK AND TACKLE ASSEMBLIES AT LOCATIONS DESIGNATED BY THE DISTRICT AS HIGH SECURITY.

PLOT SCALE: 1:1
FILE NAME: G-S1 02.DWG

The consultant, contractor or other parties associated with District projects shall comply with Florida Statutes 119. These Standards and Guidelines are the property of the District and must be secured and maintained in a confidential manner. Review by any unauthorized individual or outside/third party not performing work necessary for District projects is prohibited.

APPROVED
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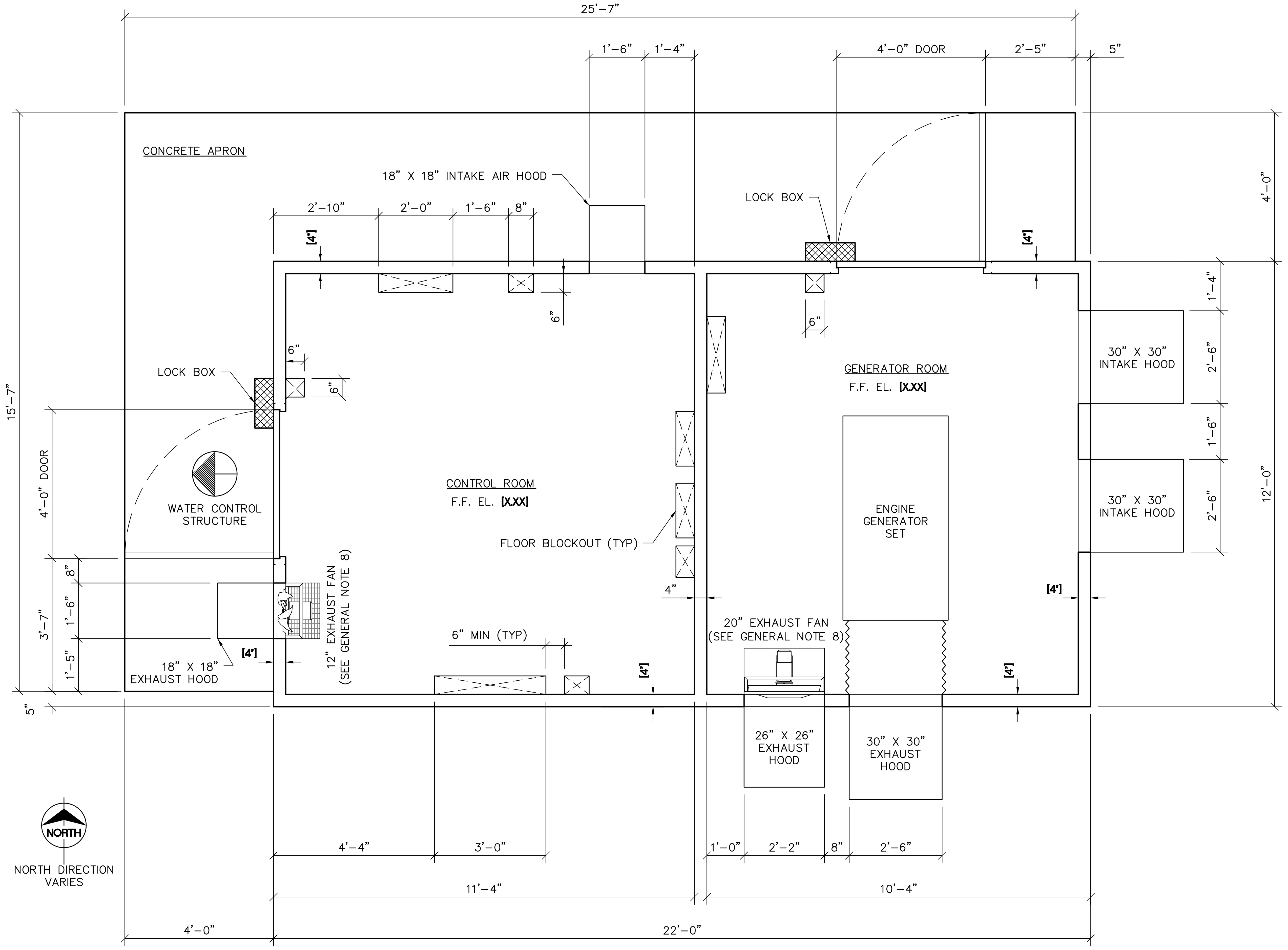
CHANGE NO. EGCR0172

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

STANDARD GUIDELINES
WATER CONTROL STRUCTURE WITH VERTICAL LIFT ROLLER GATE
PLAN

G-S1

2 SHEET OF 2



CONTROL BUILDING FLOOR PLAN

SCALE: 1/2" = 1'-0"

GENERAL NOTES

1. DESIGN REQUIREMENTS:
   A. CONTRACTOR SHALL PROVIDE A PRECAST CONCRETE BUILDING IN ACCORDANCE WITH SPECIFICATION SECTION 13120. STRUCTURE SHALL BE DESIGNED TO MEET THE REQUIREMENTS OF THE **[2010]** FLORIDA BUILDING CODE (FBC).
   B. SEE STRUCTURAL DESIGN CRITERIA ON THIS SHEET FOR MINIMUM DESIGN LOADS.
   C. WIND LOADS SHALL BE CALCULATED IN ACCORDANCE WITH ASCE 7 (LATEST EDITION).
   D. BUILDING SHALL BE DESIGNED FOR MAINTENANCE-FREE USE.
   E. BUILDING SHALL BE DESIGNED TO BE TRANSPORTED AND PLACED USING STANDARD LIFT POINTS IN THE OUTSIDE WALLS.

2. COMPONENTS:
   A. WALLS, ROOF AND FLOOR:
      a. WALL, ROOF AND FLOOR PANELS SHALL BE MANUFACTURED AS SINGLE-COMPONENT MONOLITHIC PANELS. NO WALL, ROOF OR FLOOR JOINTS ARE PERMITTED, EXCEPT AT CORNERS.
      b. WALLS, INCLUDING PARTITIONS, SHALL BE FOUR INCHES (4") MINIMUM THICKNESS. WALLS SHALL BE SIX INCHES (6") MINIMUM THICKNESS AT LOCATIONS DESIGNATED BY DISTRICT AS HIGH SECURITY AND AS NEEDED TO MEET PERFORMANCE REQUIREMENTS. ADJUST OVERALL BUILDING DIMENSIONS AS REQUIRED TO ACCOMMODATE INCREASED WALL THICKNESS.
      c. FLOOR SHALL BE SIX INCHES (6") MINIMUM THICKNESS.
      d. ROOF SYSTEM SHALL BE DESIGNED TO PROTECT EQUIPMENT IN BUILDING FROM FALLING DEBRIS.
      e. ROOF SHALL BE DESIGNED CLEAR SPAN WITHOUT THE USE OF INTERIOR SUPPORTS OF ANY TYPE.
      f. ROOF SHALL BE FIVE INCHES (5") MINIMUM THICKNESS.
      g. ROOF SHALL BE PITCHED 1/4 INCH PER FOOT.
      h. ROOF SHALL HAVE AN OVERHANG OF ½" ON ALL SIDES.
   B. BUILDING INTERIOR:
      a. CEILING FINISHED HEIGHT SHALL BE TEN FEET (10') MINIMUM.
      b. INTERIOR FINAL COATING COLOR SHALL BE WHITE.
      c. INTERIOR COATINGS SHALL CONFORM TO COATING SYSTEM C-1 IN SPECIFICATION SECTION 09900.
   C. BUILDING EXTERIOR:
      a. EXTERIOR FINISH SHALL BE SMOOTH AND FREE OF VOIDS AND CRACKS.
      b. EXTERIOR FINAL COATING COLOR SHALL BE BEIGE.
      c. EXTERIOR COATINGS SHALL CONFORM TO COATING SYSTEM C-1 IN SPECIFICATION SECTION 09900.
      d. BUILDING SHALL BE DESIGNED TO BE HIGHLY VANDAL RESISTANT. BUILDING ENVELOPE SHALL BE DESIGNED AND CONSTRUCTED TO BE BULLET RESISTANT TO UL 752, LEVEL 4.
      e. ANY EXPOSED FASTENERS SHALL BE TAMPER RESISTANT TYPE.
   D. DOORS:
      a. DOORS AND DOOR FRAMES SHALL BE PROVIDED IN ACCORDANCE WITH SPECIFICATION SECTION 08110. DOORS SHALL BE 4'-0" X 7'-0".
      b. DOORS SHALL HAVE A FLORIDA PRODUCT APPROVAL (FPA) OR MIAMI-DADE NOTICE OF ACCEPTANCE (NOA), INCLUDING MISSILE IMPACT RATING.
      c. IN ALL CASES, DOORS, FRAMES AND CONNECTIONS SHALL HAVE A PRESSURE RATING THAT MEETS OR EXCEEDS SPECIFIED DESIGN PRESSURES.
      d. DOOR HARDWARE: EACH DOOR SHALL BE EQUIPPED WITH A MINIMUM OF 1 AND 1/2 PAIR OF HEAVY DUTY SECURITY HINGES, DOOR CLOSER, THRESHOLD, BOTTOM SEAL AND A LOCK-SET WITH A LOCK BOX. DOOR HARDWARE SHALL CONFORM TO SPECIFICATION SECTION 08700.
      e. PROVIDE CAST-IN JUNCTION BOXES AND CAST-IN CONDUIT FOR FUTURE CARD READER ACCESS CONTROL SYSTEM AND CCTV.

3. THE BUILDING SHALL BE MANUFACTURED WITH CONCRETE FLOOR BLOCKOUTS FOR CONDUIT PENETRATIONS. UNLESS OTHERWISE SHOWN ON THE DRAWINGS, LOCATIONS, SPACING AND DIMENSIONS OF FLOOR BLOCKOUTS SHALL BE DETERMINED BY THE CONTRACTOR IN CONSULTATION WITH THE SUBCONTRACTOR AND COORDINATED WITH THE PRECAST BUILDING MANUFACTURER. FLOOR BLOCKOUTS SHALL BE FILLED ON SITE WITH NON-SHRINK GROUT AFTER COMPLETION OF CONDUIT INSTALLATION.

4. ALL METAL EQUIPMENT SHALL HAVE A #4 STRANDED COPPER EQUIPMENT GROUND WITH APPROVED LUG AND TIE-BACK TO THE STRUCTURE GROUNDING GRID. SEE ELECTRICAL DRAWINGS FOR GROUNDING LOOP AND DETAILS.

5. THE MANUFACTURER SHALL COORDINATE DESIGN OF FAN OPENINGS WITH EXHAUST FAN SUPPLIER. FANS SHALL BE ACTIVATED BY THERMOSTATS (WIRED PER MANUFACTURER).

6. PROVIDE TWO (2) PORTABLE FIRE EXTINGUISHER MOUNT ON BUILDING WALL WITH MANUFACTURER'S STANDARD BRACKETS.
   A. GENERATOR ROOM: ONE (1) MULTIPURPOSE, DRY CHEMICAL, TYP: UL RATE 4-A: 60-B: A8.
   B. CONTROL ROOM: ONE (1) CARBON DIOXIDE OR CLEAN AGENT, 10-LB NOMINAL CAPACITY.

7. HOODS:
   E. HOODS SHALL BE PROVIDED AT EACH AIR INTAKE AND EXHAUST FAN OPENING AND AT THE GENERATOR EXHAUST OPENING.
   F. HOOD MATERIAL SHALL BE ⅜" THICK, TYPE 304 STAINLESS STEEL.
   G. HOODS SHALL BE PROVIDED WITH REMOVABLE INSECT SCREENS (INTAKE AIR OPENINGS) OR REMOVABLE EXPANDED METAL SCREENS (EXHAUST FAN OPENINGS).
   H. SEE MECHANICAL DRAWINGS FOR HOOD DETAILS.

8. FANS:
   A. 20" EXHAUST FAN (GENERATOR ROOM) SHALL BE COOLAIR CBA20L, 1 HP, 4702 CFM @ 3/8" SP, OR APPROVED EQUAL. PROVIDE WITH BASKET TYPE, PVC COATED STEEL WIRE, SAFETY GUARD, COOLAIR TYPE SG OR APPROVED EQUAL.
   B. 12" EXHAUST FAN (CONTROL ROOM) SHALL BE COOLAIR CDU12F17, 1/6 HP, 1210 CFM @ 1/4" SP, OR APPROVED EQUAL.
   C. PROVIDE EACH FAN WITH A BASKET TYPE, PVC COATED STEEL WIRE, SAFETY GUARD, COOLAIR TYPE SG OR APPROVED EQUAL.

STRUCTURAL DESIGN CRITERIA		
ITEM	VALUE	REMARKS
ULTIMATE DESIGN WIND SPEED (Vult)		FBC 2010 AND ASCE 7-10 **[ALSO SEE NOTE TO DESIGNER 2]**
NOMINAL DESIGN WIND SPEED (Vasd)		**[ALSO SEE NOTE TO DESIGNER 2]**
RISK CATEGORY	IV	ADJUSTED FOR 200-YR MRI
WIND EXPOSURE CATEGORY	**[C OR D]**	
ENCLOSURE CLASSIFICATION	PARTIALLY ENCLOSED	
INTERNAL PRESSURE COEFFICIENT (Gcpi)	±0.55	
ROOF LIVE LOAD	60 PSF	
FLOOR LIVE LOAD	150 PSF	
FLOOR DEAD LOAD	75 PSF	BASED ON MINIMUM 6" THICK FLOOR SLAB
ALLOWABLE SOIL BEARING PRESSURE	2500 PSF	

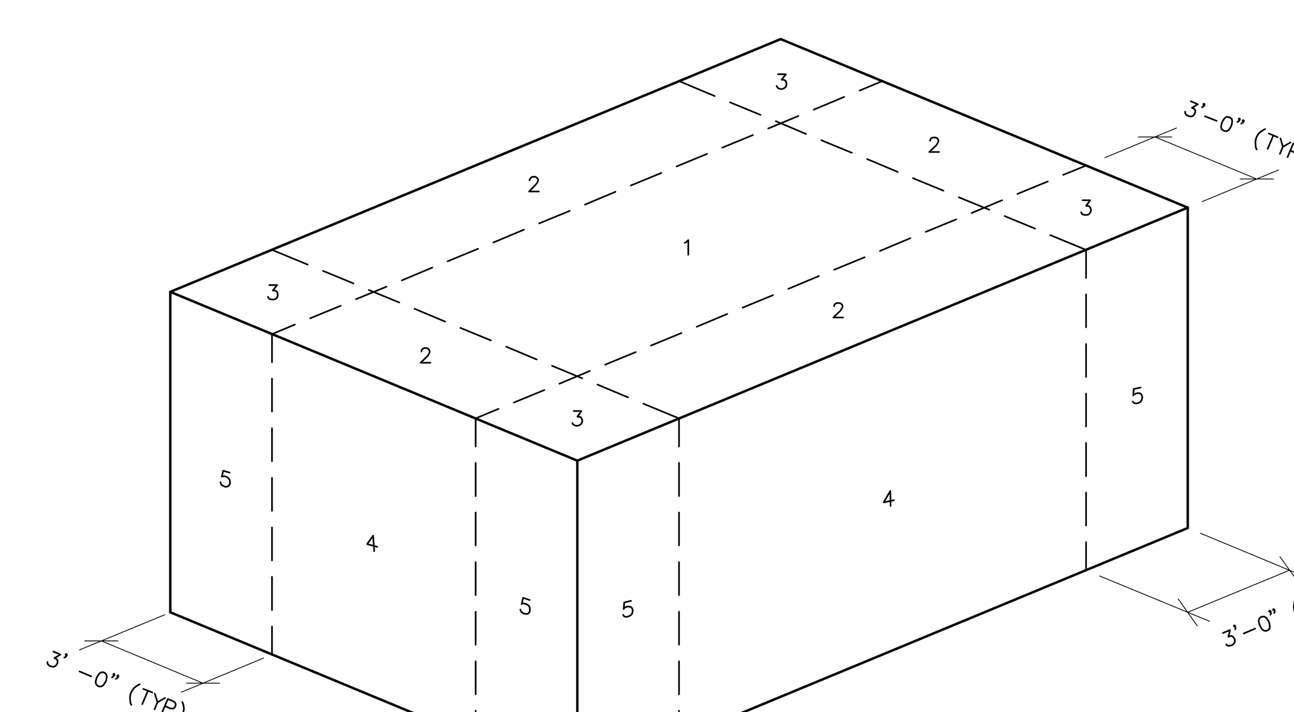
CONTROL BUILDING WITH GENERATOR WIND PRESSURES - COMPONENTS AND CLADDING												
	NOMINAL DESIGN WIND PRESSURES (PSF)						ULTIMATE DESIGN WIND PRESSURES (PSF)					
	TRIBUTARY AREA						TRIBUTARY AREA					
	10 SF		50 SF		100 SF		10 SF		50 SF		100 SF	
ROOF ZONE												
1												
2												
3												
OVERHANG ZONE												
1 & 2												
3												
WALL ZONE												
4												
5												

WIND PRESSURE TABLE NOTES:

1. DESIGN WIND PRESSURES ABOVE REPRESENT THE NET PRESSURE (SUM OF INTERNAL AND EXTERNAL PRESSURE) APPLIED NORMAL TO ALL SURFACES.
2. LINEAR INTERPOLATION BETWEEN VALUES OF TRIBUTARY AREA IS PERMISSIBLE.
3. (+) POSITIVE VALUES ARE WIND TOWARDS THE SURFACE; (-) NEGATIVE VALUES ARE WIND AWAY FROM SURFACE.

NOTES TO DESIGNER:

1. **THIS DISTRICT GUIDELINE DETAIL (G-S7) IS FOR A CONTROL BUILDING WITH GENERATOR ROOM INTENDED FOR USE AT GATED WATER CONTROL STRUCTURES REQUIRING A STANDBY GENERATOR.**
   a. **CONTROL BUILDING DIMENSIONS, EQUIPMENT CLEARANCES, VENTILATION FAN SIZES, OPENING SIZES, ETC., ARE DESIGNED FOR A STANDBY GENERATOR SIZED FOR A CAPACITY OF 25 KW. THE DESIGNER SHALL PERFORM A GENERATOR LOAD SIZING CALCULATION TO DETERMINE REQUIRED STANDBY GENERATOR CAPACITY. IF THE REQUIRED CAPACITY EXCEEDS 25 KW, THE DESIGNER SHALL EVALUATE AND MODIFY AS NECESSARY THE BUILDING DIMENSIONS, EQUIPMENT CLEARANCES, VENTILATION FAN SIZES, OPENING SIZES, ETC., TO ACCOMMODATE THE LARGER GENERATOR. CALCULATIONS AND FINDINGS SHALL BE INCLUDED IN THE DESIGN REPORT.**
   b. **MINIMUM STANDBY GENERATOR CAPACITY SHALL BE 20 KW.**
2. **DESIGNER TO CALCULATE DESIGN WIND SPEED BASED ON THE FLOOD CONTROL FACILITY CRITERION IN ACCORDANCE WITH DISTRICT GUIDELINE DETAIL G-G1-04.**
3. **THE FLOOR PLAN SHOWN IS THE PREFERRED OPTION, HOWEVER THERE ARE ALTERNATIVE FLOOR PLANS AND CORRESPONDING ELEVATION VIEWS AVAILABLE TO THE DESIGNER TO ACCOMMODATE VARYING SITE CONDITIONS.**
4. **HOODS: THE HOODS SHOWN ARE FOR A STANDARD INSTALLATION. THE DESIGNER SHALL SPECIFY NUMBER AND LOCATION OF HOODS, IF DIFFERENT FROM STANDARD. ALSO, SPECIFY THE MINIMUM VERTICAL DISTANCE FROM THE GROUND LEVEL TO THE BOTTOM OF THE INTAKE HOODS THAT WILL PROVIDE THE MINIMUM DESIGN INTAKE FLOW. SEE STANDARD DETAIL A3, SHEET 2 OF 2 FOR HOOD DETAIL.**
5. **THE BUILDING SHALL BE ORIENTED ON SITE TO ALLOW PERSONNEL INSIDE THE BUILDING TO OBSERVE THE CONTROL STRUCTURE THROUGH THE CONTROL ROOM DOOR OPENING. (SEE SITE PLAN FOR BUILDING LOCATION)**
6. **PROVIDE CAST-IN JUNCTION BOXES AND CONDUIT FOR A FUTURE CARD READER ACCESS CONTROL SYSTEM.**
7. **SPECIFY 4" OR 6" MINIMUM THICK REINFORCED CONCRETE WALLS AS REQUIRED BY SPECIFICATION SECTION 13120. INCREASE WALL THICKNESSES AS REQUIRED TO MEET THE PERFORMANCE REQUIREMENTS. INCREASE THE OVERALL BUILDING OUTSIDE DIMENSIONS TO ACCOUNT FOR ANY INCREASES IN WALL THICKNESS.**
8. **THE DOOR SHALL HAVE A BULLET RESISTANT RATING IF REQUIRED AS DESCRIBED IN SPECIFICATION SECTION 08110. DOOR SHALL HAVE A FLORIDA PRODUCT APPROVAL OR MIAMI-DADE NOTICE OF ACCEPTANCE INCLUDING MISSILE IMPACT RATING OR THE DOORWAY SHALL BE ASSUMED TO BE AN OPENING FOR ENCLOSURE CLASSIFICATION CALCULATIONS AS REQUIRED BY THE FLORIDA BUILDING CODE. IN ALL CASES, THE DOOR, FRAME AND CONNECTIONS SHALL HAVE A PRESSURE RATING THAT MEETS OR EXCEEDS THE SPECIFIED DESIGN PRESSURE.**
9. **APPLICABLE DISTRICT DESIGN DETAILS OR GUIDELINES: DISTRICT DESIGN DETAILS A3 (HOODS), A6 (LOCK BOX), E22 (SOLAR PANEL DETAIL), AND M4 (EXHAUST CROSS SECTION) AND GUIDELINE DETAIL G-G1.**



COMPONENT AND CLADDING PRESSURE ZONES

NOT TO SCALE

PLOT SCALE: 1:1
FILE NAME: G-S7_01.DWG

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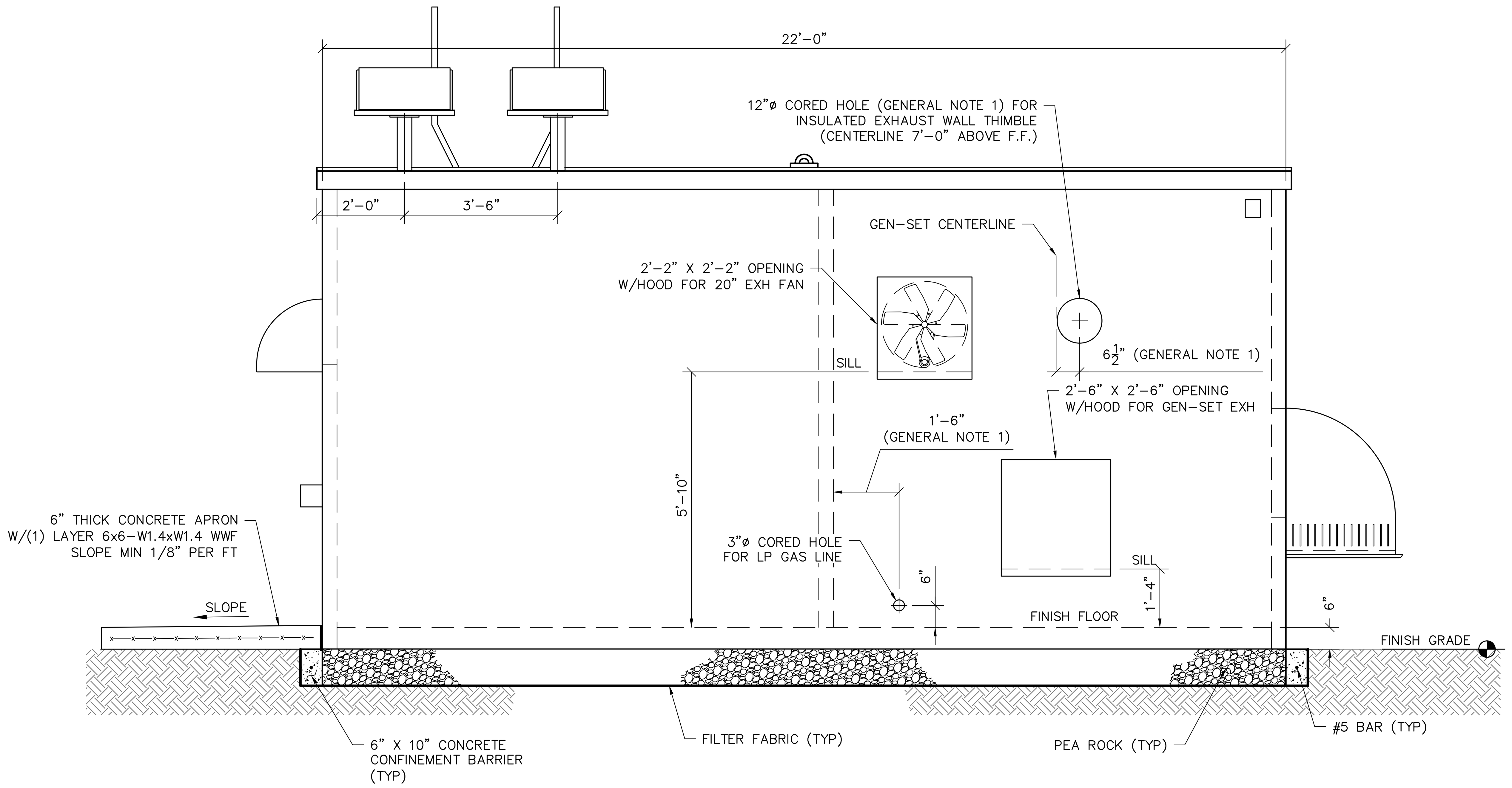
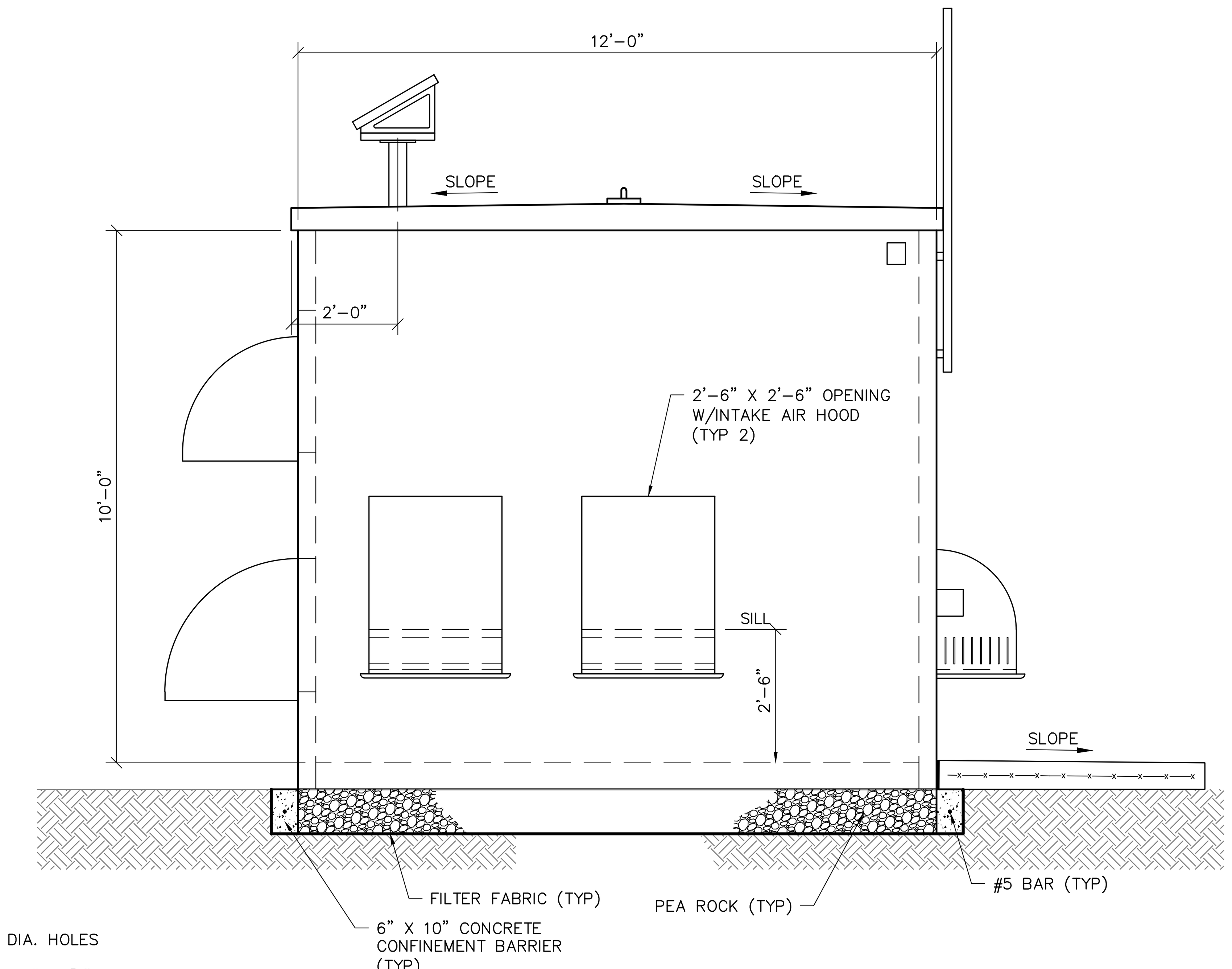
SOUTH FLORIDA WATER MANAGEMENT DISTRICT

STANDARD STRUCTURAL DETAILS
CONTROL BUILDING WITH GENERATOR - PLAN

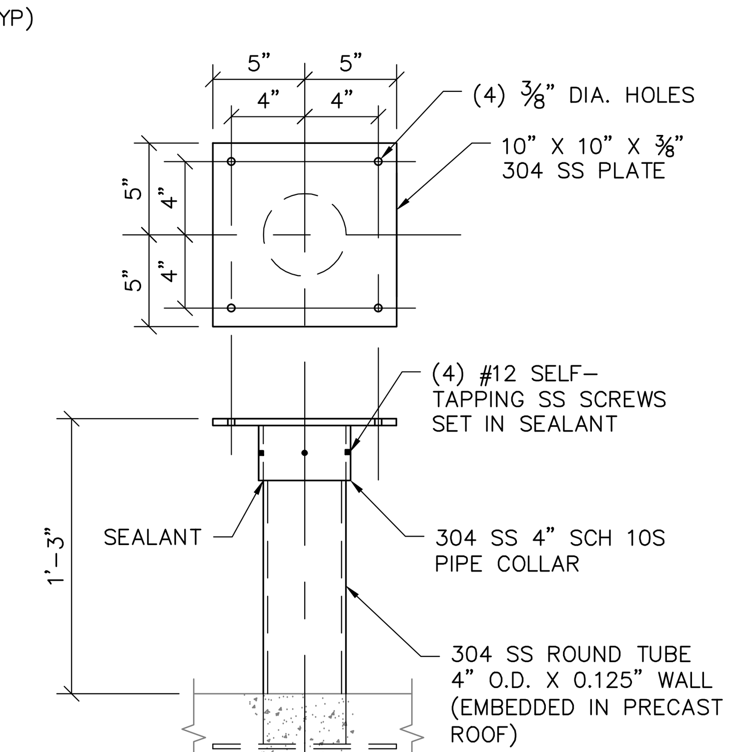
G-S7



SOUTH ELEVATION
SCALE: 1/2" = 1'-0"



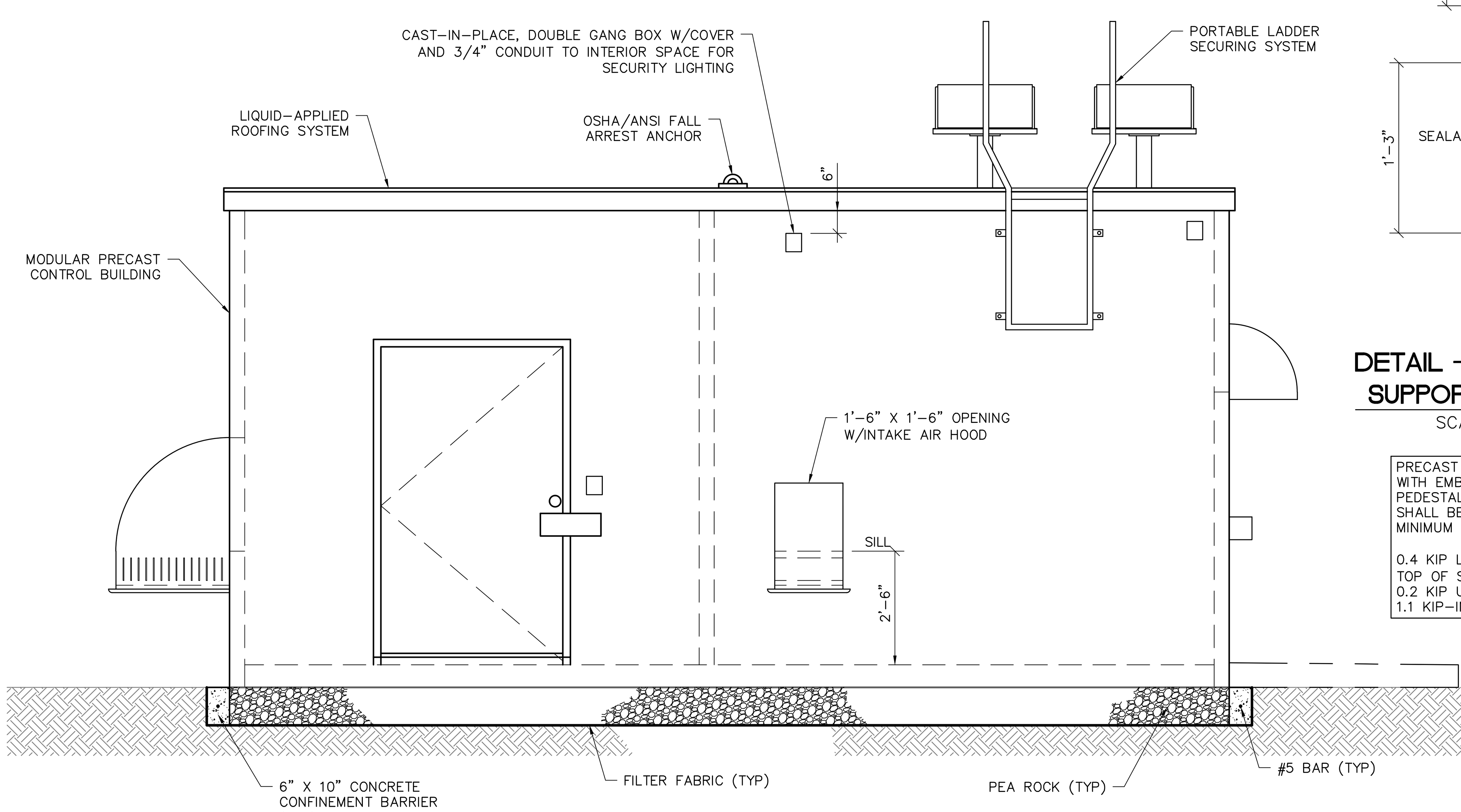
WEST ELEVATION
SCALE: 1/2" = 1'-0"



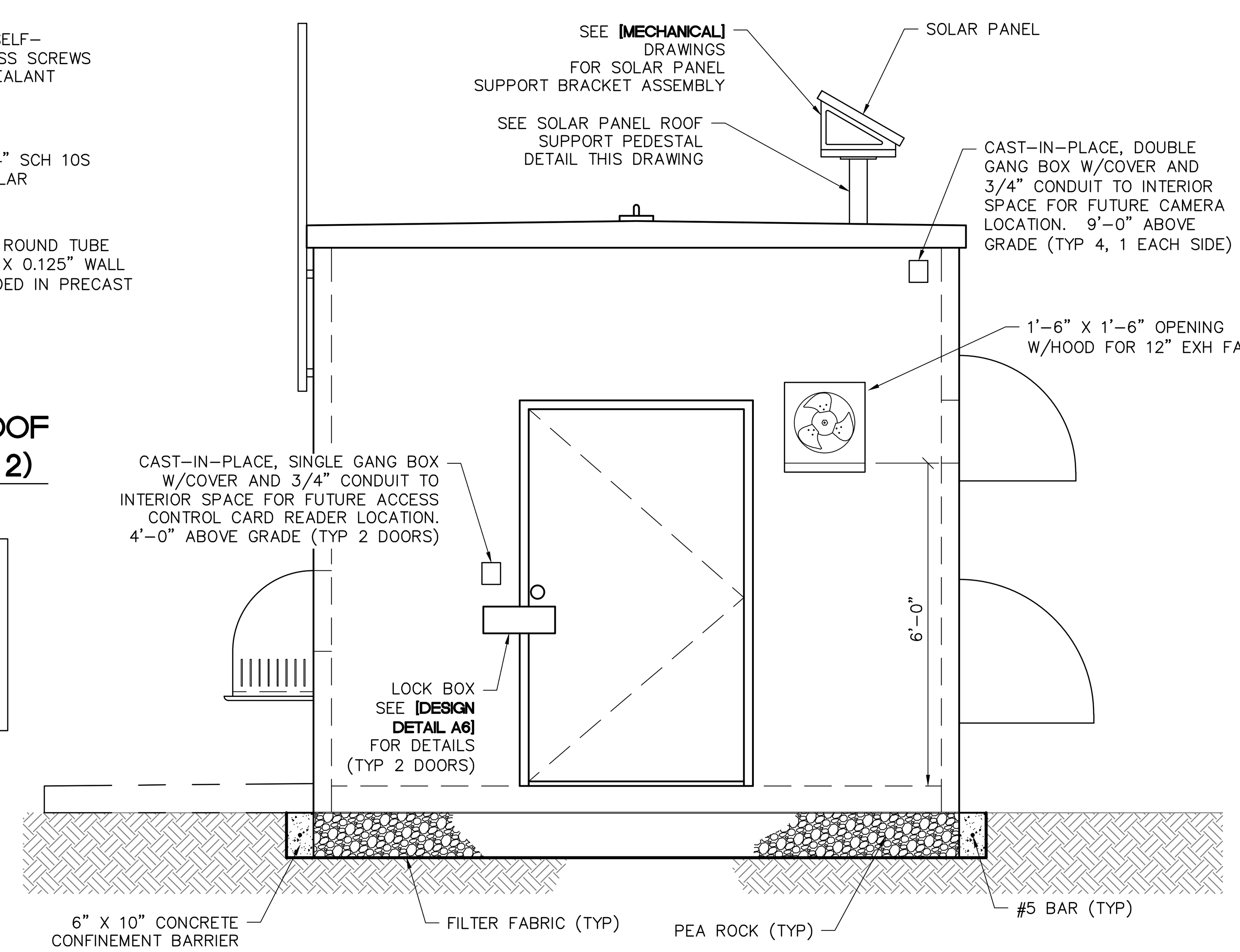
DETAIL - SOLAR PANEL ROOF SUPPORT PEDESTAL (TYP 2)
SCALE: 1-1/2" = 1'-0"

PRECAST BUILDING SHALL BE SUPPLIED WITH EMBEDDED SOLAR PANEL SUPPORT PEDESTALS AS SHOWN. SUPPORTS SHALL BE DESIGNED FOR THE FOLLOWING MINIMUM NOMINAL LOADS:

0.4 KIP LATERAL FORCE (APPLIED AT TOP OF SUPPORT)
0.2 KIP UPLIFT FORCE
1.1 KIP-IN TORQUE



NORTH ELEVATION
SCALE: 1/2" = 1'-0"



EAST ELEVATION
SCALE: 1/2" = 1'-0"

GENERAL NOTES:

1. DIMENSION SHOWN IS APPROXIMATE. CONTRACTOR SHALL COORDINATE DIMENSIONS AS REQUIRED TO ACCOMMODATE ENGINE GENERATOR SET FURNISHED.
2. FOR ROOF MOUNTED SOLAR PANEL DETAILS SEE ELECTRICAL DRAWINGS.
3. PROVIDE SEALANT OR OTHER WATERSTOP METHOD.

PLOT SCALE: 1:1
FILE NAME: G-S7_02.DWG

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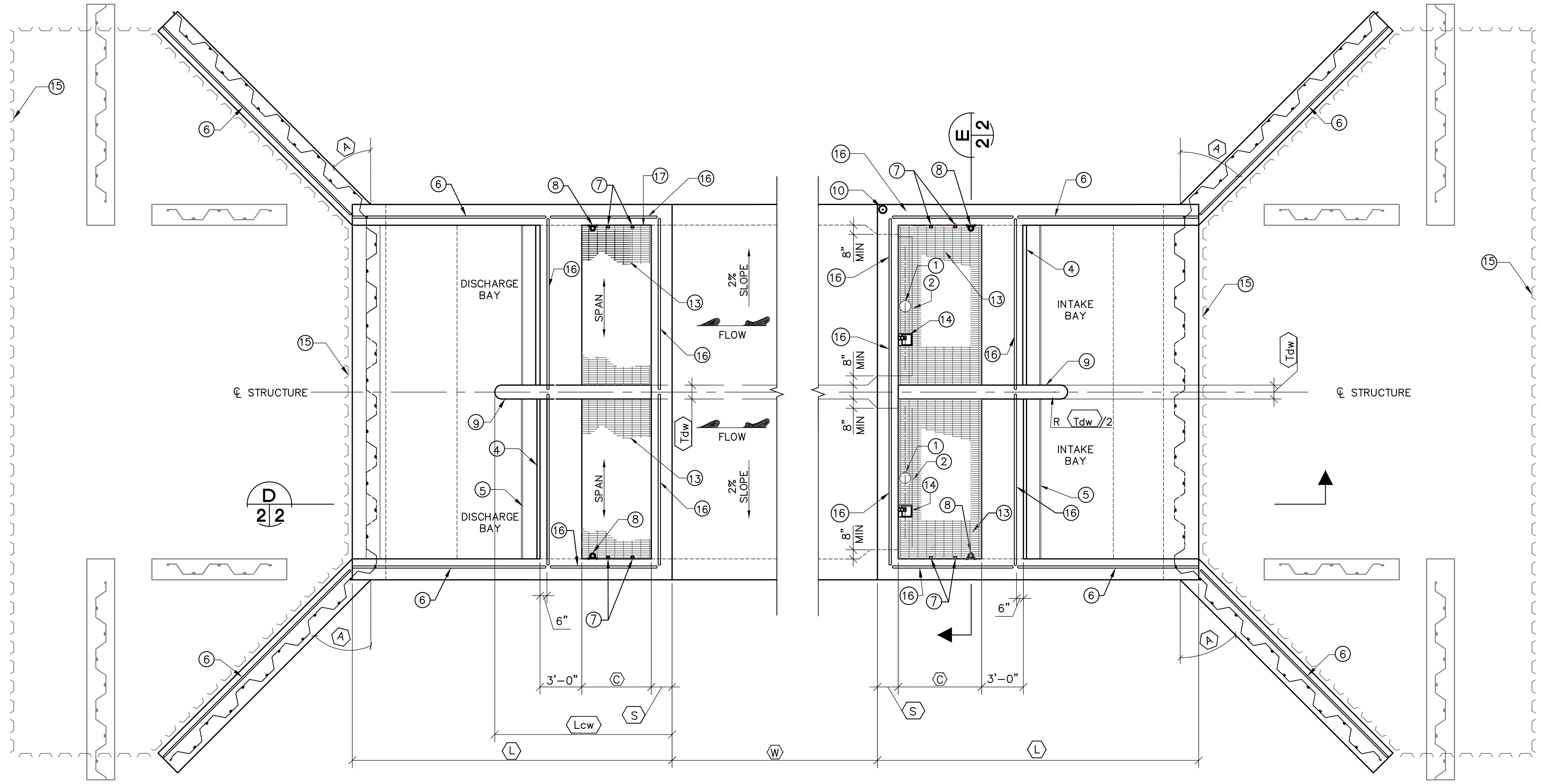
EGCR0327

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

STANDARD STRUCTURAL DETAILS
CONTROL BUILDING WITH GENERATOR - ELEVATIONS

G-S7



PLAN (DOUBLE BARREL)

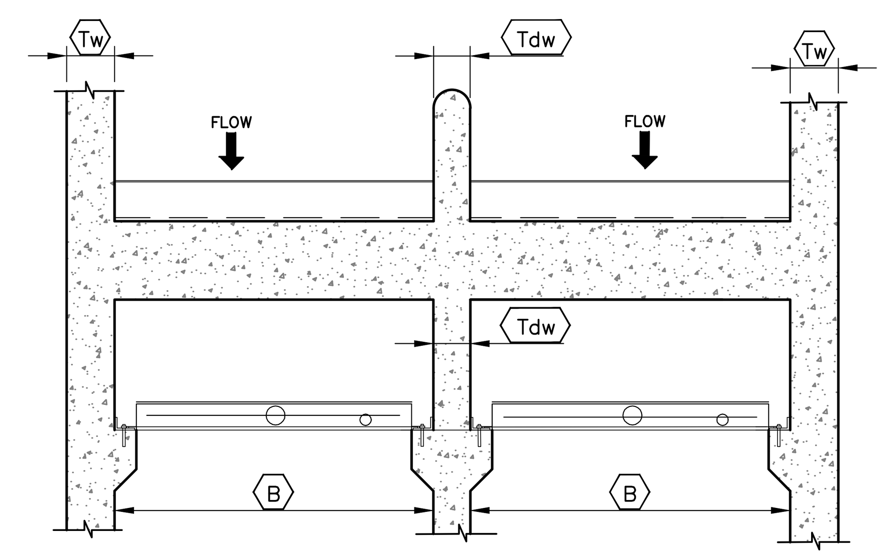
NOT TO SCALE

NOTE TO DESIGNER:

1. **SEE GENERAL NOTES, NOTES TO DESIGNER, ELEVATION AND DIMENSION SCHEDULES ON SHEET 1 OF 2.**

KEY NOTES

MARK	DESCRIPTION	REMARKS
1	GATE OPERATOR	SEE **[DESIGN DETAIL SHEET C7]** & SECTION 11291
2	CHANNEL FACE MOUNTED SLIDE GATE & FRAME	SEE **[DESIGN DETAIL SHEET C7]** & SECTION 11290
3	BOX CULVERT	
4	NEEDLE BEAM INCLUDING SUPPORT GRATING	SEE **[SEE DESIGN DETAIL SHEET S4]** **[PROVIDE SUFFICIENT BEARING FOR DEWATERING NEEDLES]**
5	NEEDLE SILL PLATE	SEE **[DESIGN DETAIL SHEET S4]**
6	STANDARD RAILING	SEE **[DESIGN DETAIL SHEET S5]**
7	D-RING ANCHORS FOR PORTABLE LADDER TIE OFF	SEE **[DESIGN DETAIL SHEET S19]**
8	WALL MOUNTED MAST-DAVIT SLEEVE	SEE **[DESIGN DETAIL SHEET S19]**
9	CENTERWALL	ON TOP OF HEADWALL
10	BENCH MARK FIRST ORDER	
11	SELECT FILL	SEE SECTION 02200
12	SHEET PILE CUT OFF WALL	
13	REMOVABLE GRATING	
14	GATE POSITION INDICATOR HOUSING	SEE **[DESIGN DETAIL SHEET M2 OR M9]**
15	RIP RAP	SEE SECTION 02370
16	STANDARD RAILING (REMOVABLE W/KICK PLATE)	SEE **[DESIGN DETAIL SHEET S5]** KICK PLATE TO THE CENTER OF 4" MAX CLEARANCE ±1/4" BETWEEN RAILING SECTIONS



SECTION F 2|2

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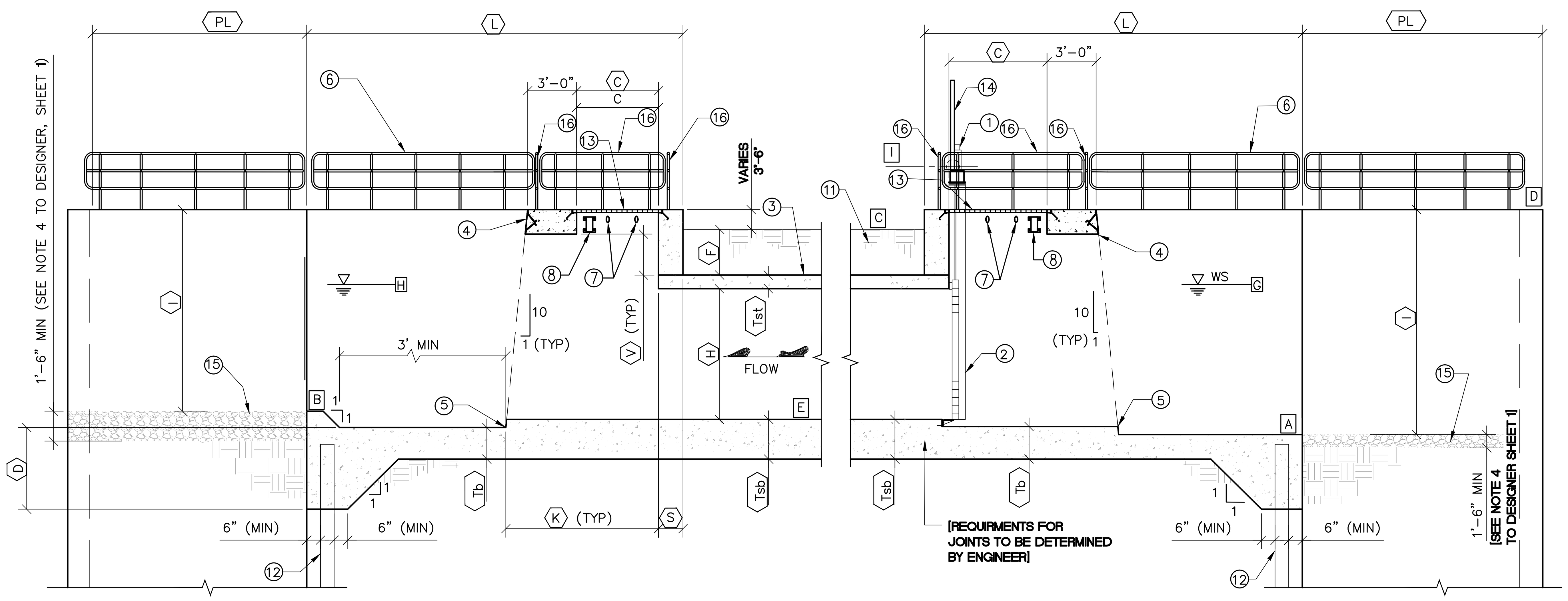
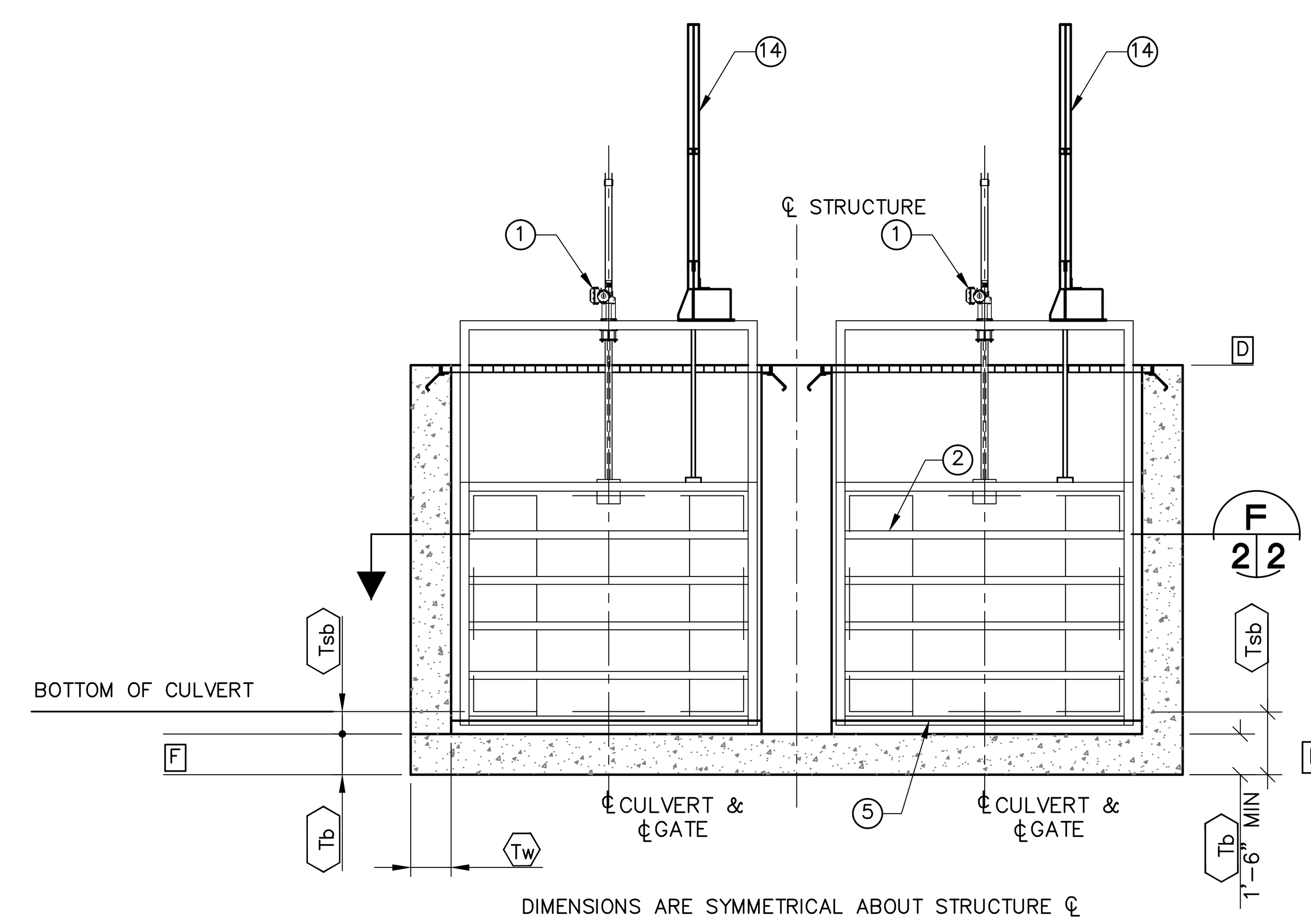


SECTION (DOUBLE BARREL) D 2|2

NOT TO SCALE



SECTION E 2|2

NOT TO SCALE

PLOT SCALE: 1:1

FILE NAME: G-S13_3.DWG

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CHANGE NO. EGCR0151 EGCR0355

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

STANDARD GUIDELINES
GATED BOX CULVERTS
DOUBLE BARREL ARRANGEMENT

G-S13