Corkscrew Watershed Initiative

Deliverable 5.2.2: Final Existing Conditions Model Report

Prepared for

South Florida Water Management District

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Prepared by

Collective Water Resources

With

J-Tech, an Alliance between Jacobs Engineering and Tetra Tech, Inc.

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Abbreviations and Acronyms

APT	aquifer performance test
Ac-ft	acre-feet
AHED	Arc Hydro Enhanced Database
BCB	Big Cypress Basin
CFD	Computational Fluid Dynamics
cfs	cubic feet per second
CREW	Corkscrew Regional Ecosystem Watershed
CSS	Corkscrew Swamp Sanctuary
CWI	Corkscrew Watershed Initiative
CWI RM	CWI Refined Model
ECM	Existing Conditions Model
DEM	digital elevation model
ERP	Environmental Resource Permit
ET	evapotranspiration
FCM	Future Conditions Model
FPLOS	Flood Protection Level of Service
ft	foot/feet
LiDAR	light detection and ranging
LWCSIM	Lower West Coast MODFLOW
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
PM	performance measure
SFWMD	South Florida Water Management District
SLCWP	South Lee County Watershed Plan
USGS	U.S. Geological Survey
WY	Water Year
1D	one-dimensional

2D	two-dimensional
3D	three-dimensional

1.0 Background/Introduction

1.1 Project Purpose

J-Tech was selected for the completion and delivery of the Corkscrew Watershed Initiative (CWI) Project. The Notice to Proceed was established for this project on March 1, 2024. The objective of the CWI Project is to develop a comprehensive strategy to achieve ecological restoration of the Corkscrew Watershed by improving wetland hydroperiods and natural flows, while reducing flood risk in nearby flood-prone areas without adversely impacting the water supply and water management needs of the Corkscrew Watershed. The objective will be accomplished through a public planning process that engages and involves key partners, stakeholders, and the public. The project will identify viable short-term and long-term strategies to achieve the CWI Project goals. The proposed restoration alternatives will be cost-effective, feasible, and resilient. The CWI Project will consider population growth, land development, and climate change, including sea level rise and future rainfall. This project aligns with the mission of the South Florida Water Management District's (SFWMD) Big Cypress Basin (BCB) 2023–2028 Strategic Plan and will contribute to the resiliency of the Big Cypress Watershed.

1.2 Project Background and Scope of Work

The CWI Project boundary encompasses southern Hendry County, northern Collier County, and southern Lee County as shown in Figure 1-1. The project area is comprised of rivers (Imperial and Estero), creeks (Halfway and Spring), and wetland sloughs/swamps (Flint Pen Strand and Upper Corkscrew Swamp) within the Estero Bay Basin Watershed in southern Lee County and the Cocohatchee Canal/River Basin and Golden Gate/Naples Bay Watershed in northern Collier County. The initial project planning area was broadened beyond what may be considered an effective area to ensure thorough consideration of vulnerable communities and ecosystems. The initial project planning area will be refined, as necessary, to support developing and evaluating projects within targeted areas.

Previous studies and monitoring documented reduced hydroperiods and increased water level recession rates in Audubon's Corkscrew Swamp Sanctuary (CSS; owned and managed by National Audubon Society), a critically important ecosystem, within the past 60 years. This three-year planning CWI Project (2024–2027) is funded by the BCB and builds upon previous efforts to determine potential causes of the shortened hydroperiod documented at the CSS. The project will develop and evaluate restoration alternatives that address the purpose and needs of the project. The Technical Working Group (TWG) is the project planning team, and is comprised of SFWMD staff, representatives of local governments in the region, and others with responsibility for the management of the Corkscrew Watershed. TWG participants are those with expertise in the Corkscrew Watershed and the ability to fund and implement projects in the CWI Project area. Collectively, representatives of the City of Bonita Springs, Collier County, CSS, Corkscrew Regional Ecosystem Watershed (CREW) Land & Water Trust, Lee County, SFWMD staff, and Village of Estero make up the TWG. The TWG will provide guidance to the SFWMD Project Manager responsible for administering the contract and acting as the liaison between the TWG, the CWI Project Consultant (J-Tech), and the SFWMD Project Development Team (PDT).

The CWI Project work plan was finalized on July 1, 2024, and includes a detailed history of work on the project that has been completed to date by the SFWMD and TWG member organizations (J-Tech 2024).

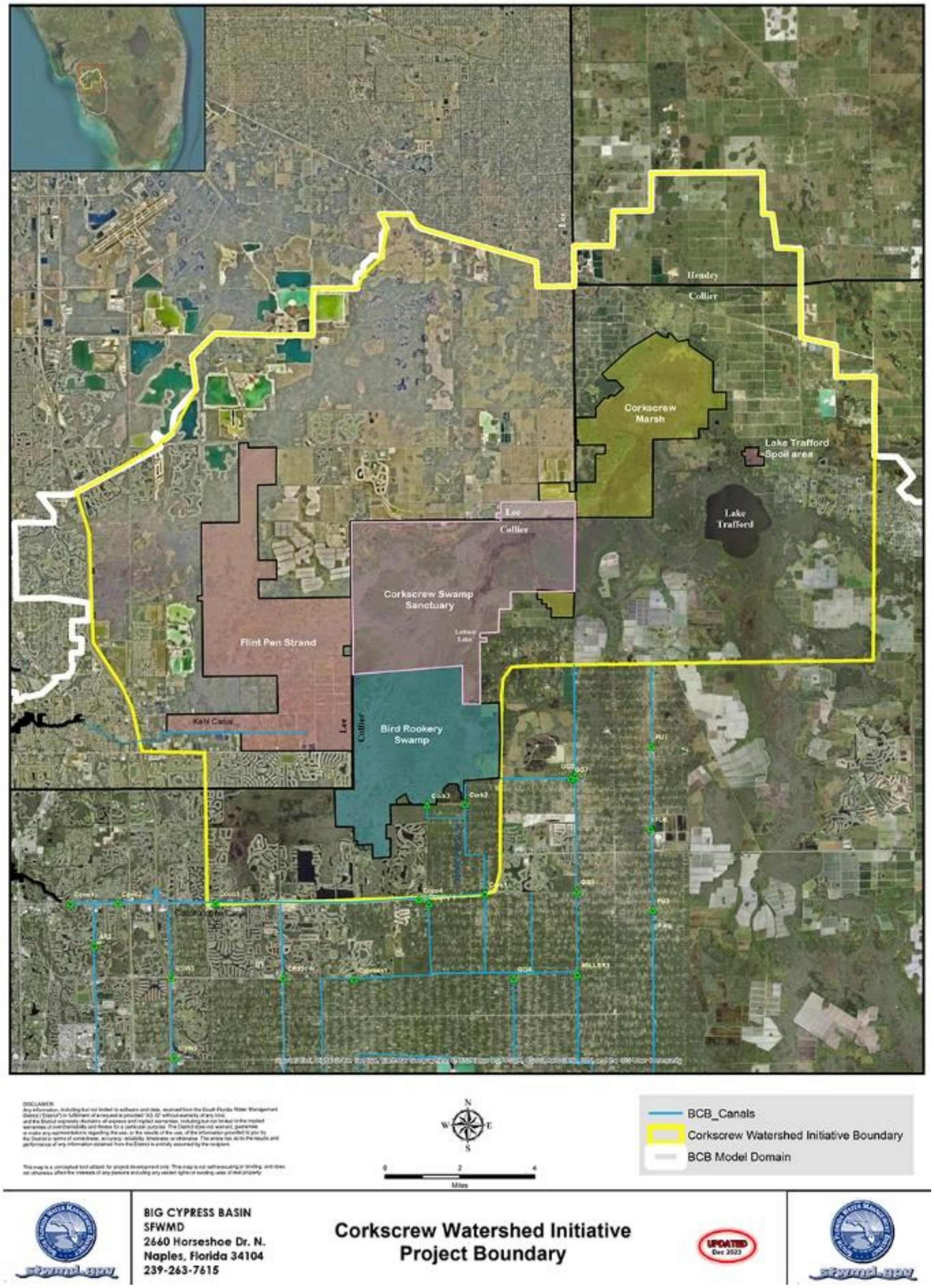

Figure 1-1. Corkscrew Watershed Initiative Project Area

The work plan also identifies the various tasks and deliverables that will be completed as part of the CWI Project. In summary, the CWI Project tasks include the following:

Task 1 – Project Coordination (on going)

Task 2 – Information Collection and Review and Site Reconnaissance (completed)

Task 3 – Develop Performance Measures and Metrics (completed)

Task 4 – Refine Project Area (completed)

Task 5 – BCB Model Refinement (completed) and Existing Conditions Simulation (in progress)

Task 6 – Develop Initial List of Potential Ecologic and Hydrologic Restoration Projects (in progress)

Task 7 – Future Baseline and Alternative Conditions Models Development/Evaluation

Task 8 – Develop Final Project Matrix Including Project Goals, Cost and Regulatory Requirements

Task 9 – CWI Public Planning Project Report, Conceptual Plan(s) and Funding Summary Memo

1.3 Document Structure

This Final Existing Conditions Model Report meets the deliverable requirements of Task 5.2.2. This document includes the following technical sections:

- 2.0 – Overview and Objective of the Existing Conditions Model (ECM)
- 3.0 – ECM Model Development
- 4.0 – Overview of Performance Measures and ECM Results
- 5.0 – Discussion and Overview of Next Phases

2.0 Overview and Objective of the Existing Conditions Model

2.1 Overview of the BCB Model Refined for CWI

The BCB Refined Model for CWI (CWI RM) is a coupled MIKE SHE/MIKE HYDRO model that includes the western watersheds of Collier County (including the Cocohatchee, Golden Gate Main, Faka Union, Henderson-Belle Meade, and Fakahatchee watersheds), the southeastern portion of Lee County that includes portions of Corkscrew, Flint Pen, and the Imperial River Basin sub-watersheds, and a small portion of Hendry County that is part of the Corkscrew East sub-watershed (Figure 2-1).

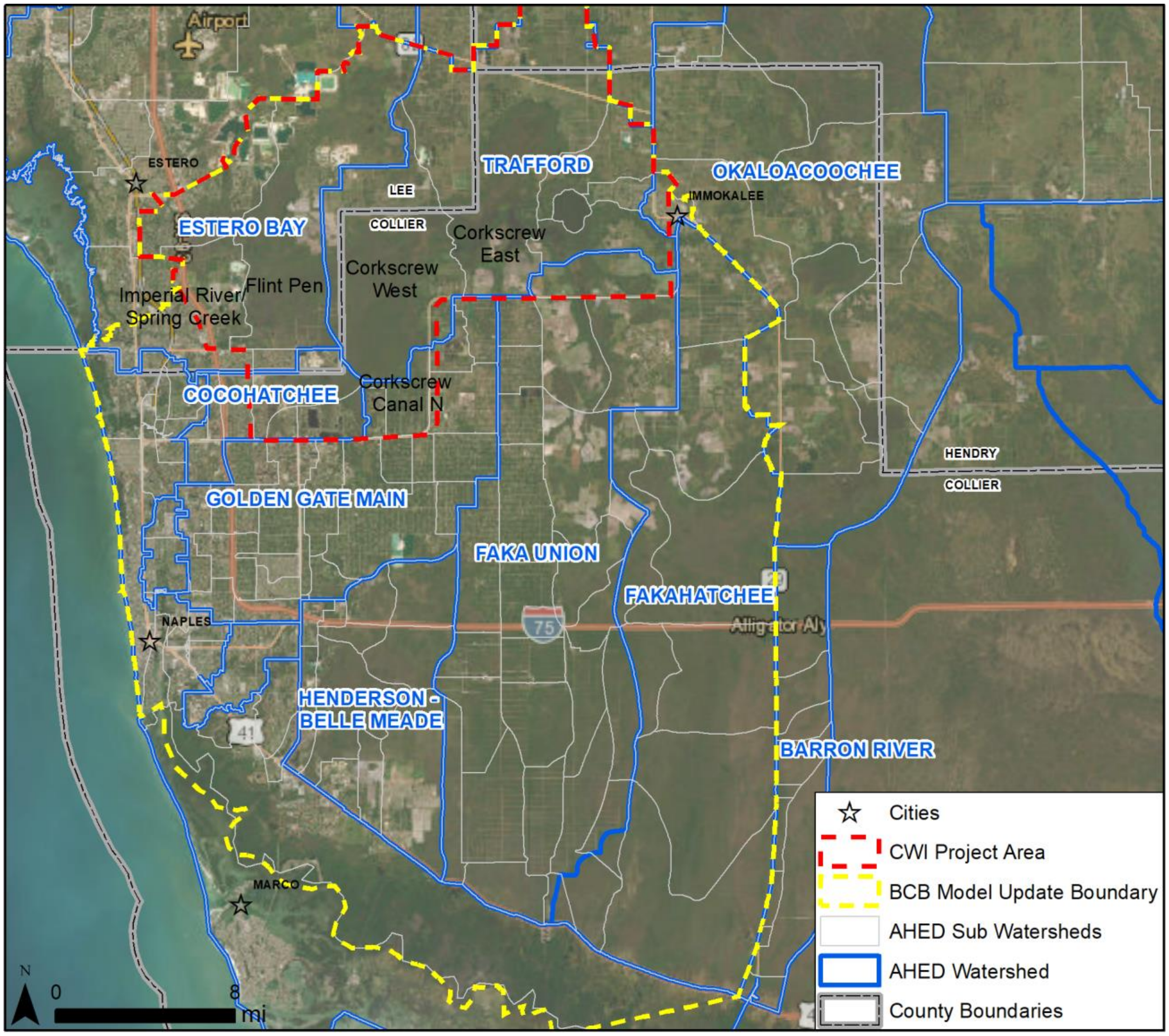

Figure 2-1. BCB Updated Model Domain, Counties, SFWMD AHED Watersheds and Sub-Watersheds

MIKE SHE is an integrated model that simulates 3D saturated groundwater flow, 2D overland flow, 1D unsaturated flow and hydrological and water use processes (i.e., rainfall, land use based evapotranspiration (ET), drainage, irrigation, pumping). MIKE HYDRO is a 1D surface water model that simulates open channel flow dynamics and hydraulic operable and non-operable structures. The CWI RM has a grid cell size of 500 feet, which is applied to all components in MIKE SHE and includes a 5-layer groundwater model that includes the Surficial Aquifer System and the upper portion of the Intermediate Aquifer System. The model layers are the hydrogeologic units shown in Table 2-1, with the three Sandstone Aquifer units represented as one layer.

Table 2-1. Hydrogeologic Units included in the BCB Model (Adapted from Zumbro et al., 2023)

<table>
<tr><th>System</th><th colspan="2">Hydrogeologic Unit</th><th colspan="2">Lithographic Unit</th></tr>
<tr><td rowspan="3">Surficial Aquifer</td><td colspan="2">Water Table Aquifer</td><td colspan="2">Undifferentiated Sediments Holocene / Pleistocene</td></tr>
<tr><td colspan="2">Tamiami Confining Unit</td><td rowspan="2">Tamiami Formation</td><td>Bonita Springs Marl / Caloosahatchee Clay Member</td></tr>
<tr><td colspan="2">Lower Tamiami Aquifer</td><td>Ochopee Limestone Member</td></tr>
<tr><td rowspan="4">Intermediate Aquifer</td><td colspan="2">Upper Hawthorn Confining Unit</td><td rowspan="4">Hawthorn Group</td><td rowspan="4">Peace River Formation</td></tr>
<tr><td rowspan="3">Sandstone Aquifer</td><td>Clastic Zone</td></tr>
<tr><td>Interconfining Unit</td></tr>
<tr><td>Carbonate Zone</td></tr>
</table>

The CWI RM was refined from the previous model version, the BCB Model Update (Hazen and Sawyer, 2025), in which the BCB Flood Protection Level of Service (FPLOS) model was extended, updated, and calibrated for the period of 2017 to 2020. The main objective of the CWI RM, conducted during the initial phases of the CWI project, was to improve the prediction of performance measures (PM) for proposed project alternatives impacting Corkscrew and adjacent basins. Model output for development of PMs includes:

- Hydroperiods and depth targets in the Corkscrew wetland areas
- Dry season groundwater levels and recession rates
- Wetland sheet flow across barriers
- Flood risk measures – stages and flows in the primary system and flood depth mapping
- Flows to estuaries, which will be tied to potential impacts in salinity
- Predicting impacts on water supply

The details that were added to the model considered these types of outputs as well as potential projects types that would be evaluated with the model. For example, to evaluate the potential benefit of removing flow barriers and uncontrolled drainages, features that represent blockages and drainage under existing conditions need to be included in the baseline model. A detailed description of the model refinements are provided in Deliverable 5.1.3: Final Refined Model Report dated July 11, 2025 (SFWMD, 2025b).

2.2 Objectives of the Existing Conditions Model

The ECM is based on the CWI RM with the main difference being that instead of the use of historical gate levels to operate the control structures, the ECM structure operations are rule-based protocols. The ECM is the initial step in building the baseline Future Conditions Model (FCM), which will be the one used to compare alternatives. The main objectives of the ECM are to identify areas of flooding, water supply, and ecological concern under long-term and design storm conditions using existing condition rule-based structure operations. The ECM model will provide a means to help evaluators understand the model and evaluation tool performance and planning use relative to their knowledge of current system behaviors, thereby providing context and a starting point to explore relative future system changes in behavior during the planning effort across common simulation periods for both long-term and storm event application.

3.0 ECM Development

3.1 Design Storm (DS) Simulations

3.1.1 DS Rainfall

The design storms simulated were the 3 day 5-, 10-, 25-, and 100-year events. The design storm rainfall data was obtained from the BCB UM model files (Hazen and Sawyer, 2025). These files were checked to confirm they matched the NOAA Atlas 14 design storm rainfall depths and the SFWMD 72-hour rainfall distribution.

3.1.2 DS Boundary Conditions

The saturated zone outer boundary conditions consisted of tidal and non-tidal for all 5 computational layers. The tidal boundary conditions originated from the model documented in the SFWMD's *Flood Protection Level of Service provided by Existing District Infrastructure for Current (2015) Sea Level Conditions and Three Future (2065) Sea Level Scenarios for Golden Gate Watershed – Final Report* (SFWMD, 2018) which were subsequently utilized in the BCB UM with timestamps adjusted for simulations conducted in the BCB UM. There are three tidal structures for which the District originally developed boundaries, COCO1, GG1, and HC1, which are nearly identical. The boundary conditions at HC1 were used for tidal boundary conditions for each design storm simulated as a part of this effort. The HC1 data time period ended at October 3, 2017 in the BCB UM, so the dates were extended through October 8, 2017 by copying earlier data for the modeling conducted herein. The non-tidal saturated zone boundary conditions were set to a constant value equal to the initial conditions.

For the 1D system, there are four non-tidal boundaries. State road 29 on the east side of the model domain was also set to a constant value equal to the saturated zone initial conditions. Alico Crossing, Estero River, and the Brooks Outfall on Halfway Creek are the other non-tidal boundaries on the northwest side of the model domain. The Estero River and Halfway Creek are stage boundaries with a constant value equal to the average wet season stage between 2015 and 2020 from the time series stage data from the South Lee County Watershed Initiative (SLCWI) Hydrological Modeling Project Model (Coastal & Heartland National Estuary Partnership, 2021).

The Alico Crossing boundary is an inflow boundary set to a constant value equal to the average discharge through culverts under Green Meadow Road for the months of August and September (for the years 2015 through 2019). The discharge was calculated from results of an adjusted version of the SLCWI Model. The model was adjusted to include information on the culvert sizes south of the boundary, at N Mallard Ln., which was obtained during a recent site visit by the J-Tech Project Team. The relatively small culvert at this location constricts the flow coming from the northern portion of the Estero watershed through the Green Meadow Road culverts to the northwestern portion of Flint Pen.

3.1.3 DS Initial Conditions and Simulation Period

The overland flow initial condition was calculated by converting the saturated zone computational layer 1, Water Table Aquifer (Holocene-Pliocene), initial potential head to depth and setting the negative values to zero. The initial conditions for each saturated zone computational layer were calculated by averaging the maximum heads during wet seasons from the CWI RM.

Initial conditions for the surface water portion of the model were hot-started using the ECM long term simulation results. The hotstart date selected was, August 22, 2017, which was just before the storm operations were triggered the conditions for basins in the project area.

The simulation period for all design storms was September 8, 2017 to October 8, 2017, but other than the surface water initial conditions, there is no historical components driving the design storm simulations.

3.2 Long Term (LT) Simulation

The purpose of the long term simulation (LTS) is to evaluate the wetland hydrologic restoration benefits of alternatives and determine any impacts in water supply under variable climatic conditions. A five-year simulation period from November 2015 to November 2020 was selected based on a rainfall analysis conducted by SFWMD BCB staff and the resulting recommendations, as shown below.

3.2.1 LT Rainfall

The period selected (November 2015 to November 2020) for the LTS alternative evaluation was recommended by SFWMD BCB staff based on a rainfall analysis from 2009 to 2021. For the analysis, data from six measured stations were averaged for three periods:

1) Wet season from May 1 to October 31,

2) Dry season from November 1 to April 30, and

3) Water year from May 1 to April 30.

The 5-year period selected includes a wet year (2017) well above the wet season average (Figure 3-1), two years (2016, 2017) well below the dry season average (Figure 3-2), and three years (2018, 2019, and 2020) of average rainfall (Figure 3-3). The figures below were adapted from the rainfall analysis provided by BCB staff.

The rainfall input in the model is based on the NEXRAD data that was processed for the BCB UM.

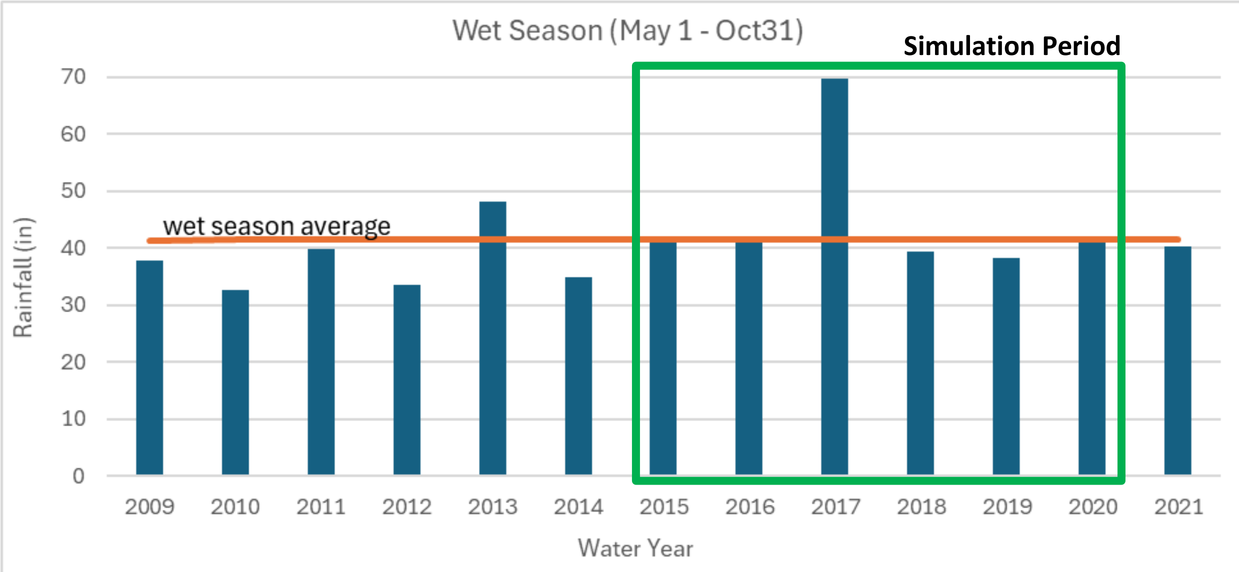

Figure 3-1. Wet season rainfall for years 2009 to 2021 with selected simulation period.

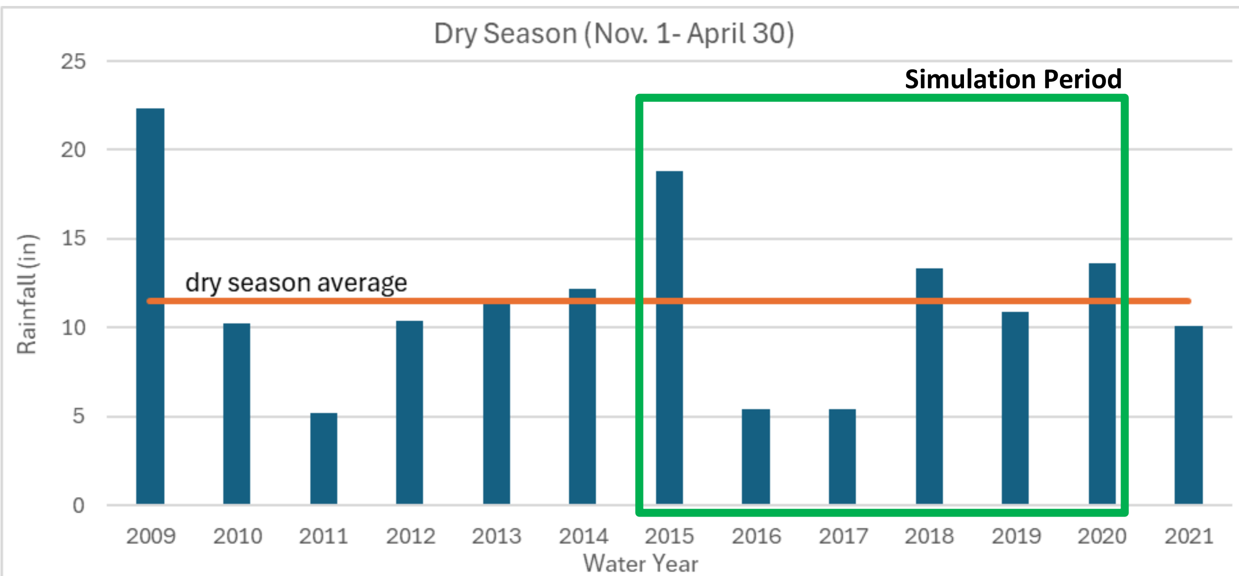

Figure 3-2. Dry season rainfall for years 2009 to 2021 with selected simulation period.

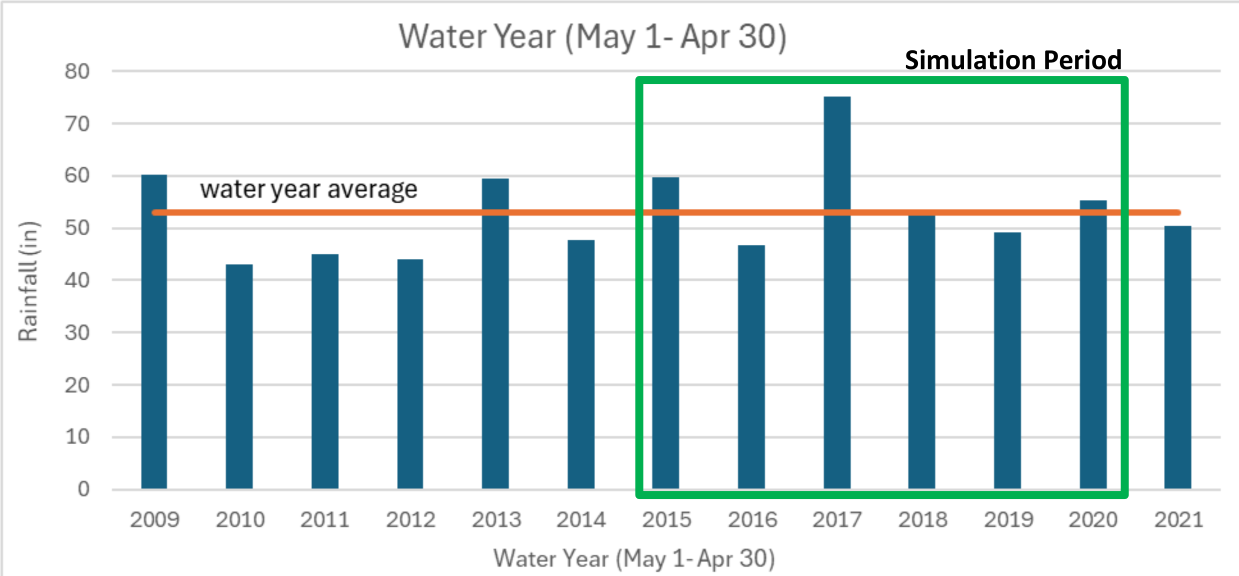

Figure 3-3. Annual water year rainfall for years 2009 to 2021 with selected simulation period.

3.2.2 LT Boundary Conditions

Similar to the CWI RM, boundary conditions for the LTS are based on the observed surface water and groundwater level data near the model boundary for the simulated period.

For the 1D system, there are three boundaries that were added that were not in the CWI RM. The Estero River and Halfway Creek are stage boundaries that were added using the measured time series stage data, which was obtained from the SLCWI Model calibration dataset. The Alico Crossing boundary is an inflow boundary set the to the simulated time varying discharge through culverts under Green Meadow road from results of the adjusted SLCWI Model, as previously described in Section 3.1.2.

3.2.3 LT Initial Conditions and Simulation Period

Initial conditions that were used for the LTS were not changed from the CWI RM initial conditions. The LTS uses a six month warmup period starting in June of 2015, which was determined sufficient to reach equilibrium before the evaluation period.

3.3 Structure Operations

Structures operations listed in this section were used for both the long term simulation and the design storm simulations. Structure locations are shown in Figure 3-4.

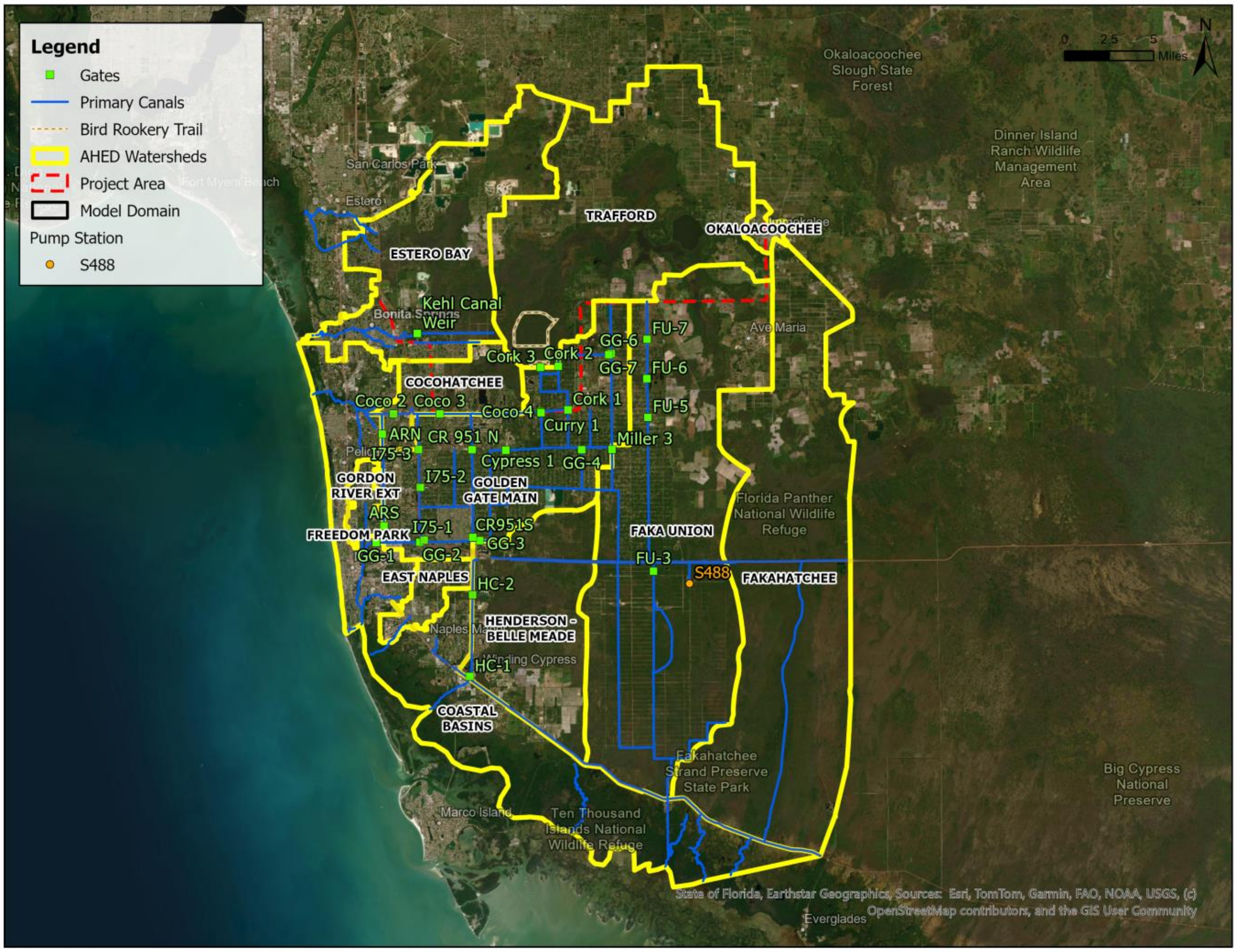

Figure 3-4. Control Structures in BCB Refined Model.

Table 3-1 presents the primary control structure operations. Structure operations at primary, District owned structures were coded in the model based on operational levels provided in the BCB Structures Master Table provided by District Staff (SFWMD, 2025c) unless otherwise noted in Table 3-1. Some structures were not available in the BCB Structures Master Table and thus operational levels were developed from information in the Water Control Operations Atlas: Big Cypress Basin System Part 2: Structure Descriptions (SFWMD, 2020). Secondary structures included rule driven operations in the CWI Refined Model used for calibration so were not updated for the CWI ECM model. Operations at automatic/remotely operated structures and manually operated structures in the model as discussed in Table 3-2, Table 3-1 includes a field to show which structures are automatically (as opposed to manually) operated.

Table 3-1. Primary Control Structure Operations

Structure	Type	Automatic Operation (Y/N)	Uses Standard Operations *	Storm Conditions	Wet Season Operation		Dry Season Operation	
				Headwater (HW) *unless noted otherwise*				
				BCB Atlas Wet Season Minimum	Optimal Maximum	Optimal Minimum	Optimal Maximum	Optimal Minimum
				(ft NAVD)				
Cocohatchee Watershed								
COCO1	Underflow	Y	Y[1]	2.7	5.4	4.3	5.5	5.2
COCO2	Underflow	Y	Y[1]	6.7	9.0	8.3	9.0	8.7
COCO3	Underflow	Y	Y[1]	8.7	10.5	9.9	10.9	10.5
COCO4	Underflow	N	N	9.7	11.2	10.2	12.2	11.7
AR2 (North)	Radial Gate	-	N	-	7.3	-	7.3	-
Golden Gate Main								
AR1 (South)	Radial Gate	-	N	-	7.3	-	7.3	-
AR1 (South) Pump[2]	Direct Discharge	-	N	-	-	-	7.23	3.73
				-	-	-	-	1.73 (TW)
GG1	Overflow	Y	Y[1]	1.7	3.2	2.9	4.2	3.9
GG2	Overflow	Y	Y[1]	3.9	5.2	4.7	5.9	5.2
GG3	Overflow	Y	Y	6.0	7.0	6.5	8.1	7.7
GG4	Underflow	Y	Y	7.7	8.7	8.1	9.4	9.0
GOLDW5 (GG5)	Underflow	N	Y[1]	9.2	10.0	9.5	10.5	10.0
GG6	Dual Leaf (Underflow and Overflow)	N	N	13.5	15.2	14.2	15.5	15.0
GG7	Dual Leaf (Underflow and Overflow)	N	N	12.1	13.5	12.5	14.5	14.0
I75W1[2]	Underflow	N	Y	4.91	*5.41*	*4.91*	*5.91*	*4.91*
I75W2	Underflow	N	Y	6.3	7.3	6.8	8.3	7.8
I75W2 Pump	Direct Discharge	-	N	-	-	-	6.72	5.22
				-	-	-	-	4.92 (TW)
I75W3	Underflow and Overflow	N	N	7.7	8.7	7.7	9.7	8.2
CYP1	Underflow	Y	Y	7.7	8.7	8.1	9.4	9.0

Structure	Type	Automatic Operation (Y/N)	Uses Standard Operations *	Storm Conditions	Wet Season Operation		Dry Season Operation	
				Headwater (HW) *unless noted otherwise*				
				BCB Atlas Wet Season Minimum	Optimal Maximum	Optimal Minimum	Optimal Maximum	Optimal Minimum
				(ft NAVD)				
CUR1	Underflow	Y	N	7.7	9.9	9.5	11.6	11.3
CR951N	Underflow	Y	N	9.2	11.2	10.7	11.7	11.3
				-	9 (TW)	10 (TW)	-	-
CR951S	Underflow	Y	Y	5.7	6.9	6.5	7.9	7.5
CR951S Pump[2]	Direct Discharge	-	N	-	-	-	7.19	5.69
				-	-	-	-	4.99 (TW)
CORK1	Underflow	Y	N	7.7	9.9	9.5	11.6	11.3
CORK2	Underflow	Y	Y	8.7	13.0	12	13.1	12.8
CORK3	Underflow	N	Y[1]	13.0	14.0	13	14.5	13.5
Faka Union Watershed								
FU3[2,3]	Overflow	N	Y	4.9	5.4	4.9	5.9	5.1
FU4S	Underflow	Y	Y	9.2	10.4	10.0	11.4	11.0
FU5	Underflow	Y	Y[1]	11.2	12.4	12.0	13.4	13.0
FU6[3]	Overflow	N	N	13.2	13.7	13.2	14.2	13.4
FU7[3]	Overflow	N	N	15.9	16.9	15.9	17.4	16.9
MILLER3	Underflow	Y	Y	7.7	8.5	8.1	9.2	8.7
Henderson Creek Watershed								
HC1	Overflow	Y	Y	2.7	4.2	3.9	4.7	4.4
HC1 Flap Gate	Underflow	N	Y	4.2	4.5	4.2	5	4.7
HC2	Underflow	Y	Y	6.7	8.7	8.0	9.7	8.7

Structure operational levels developed from BCB Structures Master Table provided by District staff (SFWMD, 2025b) unless otherwise noted in footnotes.

*Operations for structures that do not use standard operations are listed in Table 3-3 through

Table 3-11.

[1]Gates do not use additional rule to close gates when the tailwater exceeds the headwater.

[2]Operational levels were not available in the BCB Structures Master Table provided by District Staff (SFWMD, 2025b) and were developed from the Water Control Operations Atlas: Big Cypress Basin System Part 2: Structure Descriptions (SFWMD, 2020).

[3]FU6 and FU7 structures operate according to standard rules except that they open and close fully as the they are equipped with removable v-notches that are manually removed or installed.

A set of standard control operations for primary structures were developed in coordination with District staff and are shown in Table 3-2. The standard control structure operations were coded in the model to operate the gates to maintain headwater levels in the seasonal (wet/dry) optimal range (listed in Table

3-1) unless storm conditions are met, during which time the headwater level is coded such that it can drop to the BCB Atlas wet season minimum (listed in Table 3-1). This allows for pre-storm drawdown to be simulated before the peak of the storm.

Table 3-2. Standard Control Structure Operations Used in the CWI Existing Conditions Model.

Priority[1]	Condition	Model Operation
1[2]	Tailwater > Headwater	Close gates
2	Storm Conditions: Stages fall below the Atlas Wet Season minimum control elevation.	Close gates 1 ft less than the previous level. Check gate level every X hours[3].
3	Storm Conditions: Rainfall in the watershed in the next 72 hours > 7.5 inches (NOAA averages for 5-yr event)	Open gates 1 ft more than the previous level. Check gate level every X hours[3].
4	Wet season headwater stages are above wet season optimal maximum	Open gates 6 inches more than the previous level. Check gate level every X hours[3].
5	Dry season headwater stages are above dry season optimal maximum	
6	Wet season headwater stages are below wet season optimal minimum	Close gates 6 inches more than the previous level. Check gate level every X hours[3].
7	Dry season headwater stages are below dry season optimal minimum	
8	None of the conditions above, i.e., stages are within the optimal ranges.	Keep gate level unchanged.

[1] Rule for historical gate operation for calibration simulation, included as the first rule in most structure operations, are not listed in this table. Some structures include more than one historical gate operation rule. Historical gate operations are not used in the CWI ECM.
[2] Only structures that experienced negative flows during model testing include rule (or priority) #1.
[3] For automatic gates: X = 1 hour. For manual gates: X = 6 hours. This is to approximate how frequently water managers change the opening of automatic and manual gates.

As shown in Table 3-2, during normal operations (non-storm conditions), the gates are coded to open 6 inches at a time. District water managers reported 6 inches as an appropriate gate level change that could apply to the majority of gates within the model domain. Automatic gate levels are checked and operated every one hour, minimum. Manual gate levels are checked and operated every 6 hours, minimum. For example, if simulated headwater levels are above the optimal maximum level during normal operations, gates are opened 6 inches, if the simulated headwater levels are still above the optimal maximum level after 1 hour at an automatic gate, the gates are opened an additional 6 inches. Alternatively, if headwater levels drop below the optimal minimum after 1 hour at an automatic gate, gates are closed 6 inches.

As shown in Table 3-2, storm condition operations are triggered when rainfall is projected to exceed 7.5 inches in the next 72 hours. An average rainfall time series for each watershed was created from the model rainfall input. The rolling future 72 hour total rainfall total was calculated from the average rainfall time series for each watershed for the long term simulation. Each structure uses the time series for the SFWMD AHED watershed that it exists within to define projected storm conditions. For the design storm simulations, the storm conditions were set for the first 4.5 simulation days in all watersheds. During storm operations, the gates are opened 1 ft at a time. Automatic gate levels are checked and operated every 1 hour, maximum. Manual gate levels are checked and operated every 6 hours, maximum. For example, if storm operations are triggered, gate levels are opened 1 ft every hour at an automatic gate until headwater levels reach the BCB Atlas wet season minimum or the gates are fully opened.

For some primary structures within the model, the standard control operations in Table 3-2 were not appropriate due to a unique structure configuration or additional operating level criteria for that structure. These structures are marked in Table 3-1 and operations for these structures are shown separately in Table 3-3 through

Table 3-11.

Table 3-3. Control Structure Operations for COCO4, which is not modeled with standard operations.

COCO4 Structure operational rules include rules to send water west during the dry season.		
Priority[1]	**Condition**	**Model Operation**
1	Wet season and Tailwater > Headwater	Close gates
2-3	Gate operates under rules 2 -3 of standard operations representing storm operations.	
4-5	Gate operates under rules 4 and 6 of standard operations representing wet season operations	
6	Dry season COCO4 tailwater stages are above dry season optimal maximum headwater stages for CUR1 (CUR1 headwater is adjacent to the COCO4 tailwater), COCO4 Tailwater (East) is above COCO4 Headwater (West), and either the COCO3 headwater stages are below the dry season optimal minimum or the CR951S headwater is below the dry season optimal minimum. *This rule allows COCO4 to open during the dry season to move water from to the west when there is excess water in the Cocohatchee East canal to maintain the water levels in COCO3 or to push water through CR951N to maintain the water levels at CR951S*. These operations were developed based on emails from District Water Managers received on October 3, 2025.	Open gates <u>6 inches</u> more than the previous level. Check gate level every 6 hours.
7	Dry season COCO4 tailwater stages are below dry season optimal minimum headwater stages for CUR1 (CUR1 headwater is adjacent to the COCO4 tailwater).	Close gates <u>6 inches</u> more than the previous level. Check gate level every 6 hours.
8	None of the conditions above	Keep gate level unchanged.
[1] Priority does not include historical gate operations that may be included before the rules based operations listed in this table. Historical gate operations are not used in the CWI ECM.		

Table 3-4. Control Structure Operations for AR2 (north) and AR1 (south) Amil Gate Structures which are not modeled with standard operations.

AR2 (North) & AR1 (South) – Airport Road Amil Gates		
Priority[1]	**Condition**	**Model Operation**
1	• Headwater stages are above optimal maximum plus 0.2 ft and • headwater stages are above tailwater stages. • Opening 0.2 ft above the optimal maximum prevents the gates from *opening and closing too frequently.*	Open gates according to tabulated data. Tabulated data closes gates when headwater stages are below the optimal maximum and fully opens gates when headwater stages are 1 foot below the water level which will bypass the structure.
2	Headwater stages are below optimal maximum minus 0.2 ft. *Closing 0.2 ft below the optimal maximum prevents the gates from opening and closing too frequently.*	Close gates.
3	None of the conditions above	Keep gate level unchanged.
[1] Priority does not include historical gate operations that may be included before the rules based operations listed in this table. Historical gate operations are not used in the CWI ECM.		

Table 3-5. Control Structure Operations for AR1 (south) Pump, CR951S Pump, and I75W2 Pump which are not modeled with standard operations.

Pump Stations - AR1 (South) Pump, CR951S Pump, I75W2 Pump.		
Priority[1]	**Condition**	**Model Operation**
1	• It is the wet season or • headwater stages are greater dry season optimal max (HW) or • tailwater stages are less than dry season optimal min (TW).	Pumps are off
2	Headwater stages are less than dry season optimal min (HW).	Pump at pump capacity. AR1 pump = 3,000 gpm[2]. CR951S pump = 3,000 gpm[2]. I75W2 pump = 7,800 gpm[2].
3	None of the conditions above	Keep gate level unchanged.
[1]Priority does not include historical gate operations that may be included before the rules based operations listed in this table. Historical gate operations are not used in the CWI ECM. [2]Pump volumes from the BCB Atlas (SFWMD, 2020).		

Table 3-6. Control Structure Operations for CUR1, which is not modeled with standard operations.

CUR1 Structure operational rules to close CUR1 when COCO4 is sending water west during the dry season.		
Priority[1]	**Condition**	**Model Operation**
1-2	Gate operates under rules 2 -3 of standard operations representing storm operations.	
3-4	Gate operates under rules 4 and 6 of standard operations representing normal wet season operations	
5	COCO4 flow is negative (flow is going west). *This rule closes the gates when COCO4 is sending water west during the dry season*.	Close gates 6 inches more than the previous level. Check gate level every hour.
6-7	Gate operates under rules 5 and 7 of standard operations representing normal dry season operations	
8	None of the conditions above	Keep gate level unchanged.
[1] Priority does not include historical gate operations that may be included before the rules based operations listed in this table. Historical gate operations are not used in the CWI ECM.		

Table 3-7. Control Structure Operations for Cork1, which is not modeled with standard operations.

Cork1 Structure operational rules to close Cork1 when COCO4 is sending water west during the dry season.		
Priority[1]	**Condition**	**Model Operation**
1	Wet season and Tailwater > Headwater	Close gates
2-3	Gate operates under rules 2 -3 of standard operations representing storm operations.	
4-5	Gate operates under rules 4 and 6 of standard operations representing normal wet season operations	
6	COCO4 flow is negative (flow is going west). *This rule closes the gates when COCO4 is sending water west during the dry season*.	Close gates 6 inches more than the previous level. Check gate level every hour.
7-8	Gate operates under rules 5 and 7 of standard operations representing normal dry season operations	
9	None of the conditions above	Keep gate level unchanged.
[1] Priority does not include historical gate operations that may be included before the rules based operations listed in this table. Historical gate operations are not used in the CWI ECM.		

Table 3-8. Control Structure Operations for GG6 and GG7 Dual Leaf Gates which are not modeled with standard operations.

GG6 & GG7 - Dual Leaf Gates Gates are represented by one underflow gate and one overflow gate (variable weir).		
Priority[1]	**Condition**	**Model Operation**
<u>Overflow Gates</u> (variable weir) – used for normal operations.		
1	Underflow gate flow is greater than zero (underflow gate is open). *This rule ensures the overflow gate is lifted with the underflow gate when the underflow gate is opened.*	Overflow gate level is equal to underflow gate level plus the dual leaf gate height.
2 – 6	The remaining rules are the standard gate operations rules 2-6 listed in Table 3-2.	
7	Overflow gate level exceeds than the top of gate elevation fully close/overflow possible gate level (GG6 = 15.19 ft NAVD; GG7 = 13.49 ft NAVD) *This rule ensures the overflow gate does not exceed the top of gate elevation fully close/overflow possible level during normal operations. Because the rules are using the Delta Action Value type to control the gates (which allows the max gate opening to go up to the Max Level set in the gate parameters) and the Max Level in the parameters had to be set to allow the weir to lift up with the gate during storm events, the max elevation during normal operations was capped at 15.19 ft NAVD and 13.49 ft NAVD for GG6 and GG7, respectively, using this rule.*	Gate level is set to the top of gate elevation fully close/overflow possible gate level (GG6 = 15.19 ft NAVD; GG7 = 13.49 ft NAVD)
8-9	The remaining rules are the standard gate operations rules 7-8 listed in Table 3-2.	
<u>Underflow Gates</u> – used for storm operations only when overflow gates cannot maintain the wet season optimal maximum.		
1	Storm Conditions: Stages fall below the Atlas Wet Season minimum control elevation.	Close underflow gates <u>1 ft</u> less than the previous level. Check gate level every 6 hours.
2	• Storm Conditions: Rainfall in the next <u>72 hours > 7.5 inches</u> (NOAA averages for 5-yr event) and • Headwater stages are above wet season optimal maximum and • The overflow gates have been fully opened in the last hour.	Open underflow gates <u>1 ft</u> more than the previous level. Check gate level every 6 hours.

GG6 & GG7 - Dual Leaf Gates Gates are represented by one underflow gate and one overflow gate (variable weir).		
Priority[1]	**Condition**	**Model Operation**
	Underflow gate does not open unless the overflow gates cannot maintain the wet season optimal maximum stage.	
3	• Storm Conditions: Rainfall in the last 3 days > 7.5 inches (NOAA averages for 5-yr event) and • Headwater stages are above wet season optimal maximum and • The underflow gates have been fully opened in the last hour. *This rule ensures the underflow gates stay open until the headwater stages recede below the wet season optimal maximum stage.*	Open underflow gates 1 ft more than the previous level. Check gate level every 6 hours.
4 - 7	*Underflow gates are closed during normal operations* Condition 4 – 7 of standard operations representing normal optimal level operations.	Underflow gates closed.
8	None of the conditions above.	Keep gate level unchanged.
[1] Priority does not include historical gate operations that may be included before the rules based operations listed in this table. Historical gate operations are not used in the CWI ECM.		

Table 3-9. Control Structure Operations for I75W3 which has an overflow gate and an underflow gate not modeled with standard operations.

I75W3 – one underflow gate, one overflow gate.		
Priority[1]	**Condition**	**Model Operation**
Overflow Gates (variable weir) – uses standard operations. Overflow and underflow gates can both be opened at the same time in this structure.		
Underflow Gates – used for storm operations only.		
1	Tailwater > Headwater	Close gates
2-3	Gate operates under rules 2 -3 of standard operations representing storm operations.	
4 - 7	Condition 4 – 7 of standard operations representing normal optimal level operations. *Underflow gates are closed during normal operations*	Underflow gates closed.

8	None of the conditions above.	Keep gate level unchanged.
[1] Priority does not include historical gate operations that may be included before the rules based operations listed in this table. Historical gate operations are not used in the CWI ECM.		

Table 3-10. Control Structure Operations for CR951N, which is not modeled with standard operations.

CR951N Structure operational rules include additional tailwater wet season optimal stages.		
Priority[1]	**Condition**	**Model Operation**
1 - 3	Gate operates under rules 1 -3 of standard operations representing storm operations.	
4	Wet season headwater stages are above wet season optimal maximum (HW)	Open gates <u>6 inches</u> more than the previous level. Check gate level every hour.
5	Wet season headwater stages are below wet season optimal minimum (HW)	Close gates <u>6 inches</u> more than the previous level. Check gate level every hour.
6	Wet season tailwater stages are below wet season optimal minimum (TW)	Open gates <u>6 inches</u> more than the previous level. Check gate level every hour.
7	Wet season tailwater stages are above wet season optimal maximum (TW)	Close gates <u>6 inches</u> more than the previous level. Check gate level every hour.
8	Dry season CR951S headwater is below CR951S optimal minimum headwater, COCO4 flow is negative (flow is going west), and CR951N headwater is greater than CR951N tailwater. *This rule sends water south to CR951S when COCO4 is sending water west and CR951S headwater is lower than optimal.*	Open gates <u>6 inches</u> more than the previous level. Check gate level every hour.
9	Dry season headwater stages are above dry season optimal maximum	Open gates <u>6 inches</u> more than the previous level. Check gate level every hour.
10	Dry season headwater stages are below dry season optimal minimum	Close gates <u>6 inches</u> more than the previous level. Check gate level every hour.
11	None of the conditions above	Keep gate level unchanged.
[1] Priority does not include historical gate operations that may be included before the rules based operations listed in this table. Historical gate operations are not used in the CWI ECM.		

Table 3-11. Control Structure Operations for the Kehl Canal Weir and Gates which has a fixed weir and underflow gates.

Kehl Canal Gates[2] Structure includes a fixed weir and three underflow gates.		
Priority[1]	**Condition**	**Model Operation**
<u>Underflow Gates 1 and 2</u> – used for storm operations only.		
1	Storm Conditions: Stages fall below the Kehl Canal Weir sill elevation (8.65 ft NAVD).	Close gates <u>6 inches</u> more than the previous level. Check gate level every 6 hours.

Kehl Canal Gates[2] Structure includes a fixed weir and three underflow gates.		
Priority[1]	**Condition**	**Model Operation**
2	Storm Conditions: Rainfall in the next 72 hours > 7.5 inches (NOAA averages for 5-yr event)	Open gates 6 inches more than the previous level. Check gate level every 6 hours.
3	None of the conditions above	Keep gate level unchanged.
Underflow Gate 3 – used for storm operations only and only opens if Gates 1 and 2 are almost fully open.		
1	Storm Conditions: Stages fall below the Kehl Canal Weir sill elevation (8.65 ft NAVD).	Close gates 6 inches more than the previous level. Check gate level every 6 hours.
2	• Storm Conditions: Rainfall in the next 72 hours > 7.5 inches (NOAA averages for 5-yr event) and • Kehl Canal Gates 1 and 2 are 1 ft less than fully open.	Open gates 6 inches more than the previous level. Check gate level every 6 hours.
3	None of the conditions above.	Keep gate level unchanged.
[1] Priority does not include historical gate operations that may be included before the rules based operations listed in this table. Historical gate operations are not used in the CWI ECM. [2]This structure is managed by Lee County and is not a primary structure.		

Besides the structure operation changes described above, minor adjustments were included in the ECM structures. Gate speeds were adjusted so that the gate opening and frequency worked as intended give the model time step. In addition, minor corrections were made if inconsistencies were identified with the BCB flow Atlas manual, such as the maximum gate heights.

3.4 Picayune Strand Restoration Project Addition

The fully implemented Picayune Strand Restoration Project (USACE, 2024) will be added to the FCM. As part of the ECM a portion of the project was added to reflect the changes that were in place by the end of 2020 (Figure 3-5). These changes include:

1. Merrit pump, S-488,
2. The Merritt canal removal downstream of the pump,
3. The addition of the tieback levee east and west of the pump, and
4. Connection of the pump discharge to the downstream wetland system.

The Merrit pump has a total capacity of 810 cfs and starts pumping if the headwater level exceeds 9.0 ft-NAVD and stops if the level falls to 7.7 ft-NAVD. More detailed operational information that includes target seasonal levels was recently obtained from BCB staff and will be implemented for the final deliverable. The tieback levees with the northern and western swales (or channels) were added as branches in the 1D (MIKE HYDRO) model. The height of the levee is included in the southern or eastern bank of the cross section and thus, it acts like a flow barrier since no overland flow lower than the levee height can flow from the southern or eastern side of the cross section. These levees are designed to prevent flood damage reduction in areas north of I-75 and other adjacent private properties. Finally, the discharge from the pump is connected to the southern wetlands via flood codes.

Figure 3-5. Picayune Strand Restoration Project Features (from USACE, 2024)

4.0 Performance Measures and ECM Results

4.1 Overview of Performance Measures for Alternative Evaluation

The document *Corkscrew Watershed Initiative Deliverable 3.2: Final Project Performance Measures and Metrics Technical Memorandum* (SFWMD, 2025a) provides a summary of the measures that will be used to evaluate alternatives. Some measures related to habitat restoration and project implementation will not be based on model output. The measures discussed in this section are those that are based on the model output, which are those related to hydrologic restoration of wetlands, flood protection, and water supply.

For the flood protection metrics, three out of the six performance measures that are used in the SFWMD's FPLOS program were applied. These are: FPLOS PM1 – maximum canal stage profiles, PM 5 – peak flood depths, PM 6 – flood duration. The other FPLOS PMs are not used because they have to do with assessing structure capacities and system drainage which are not relevant to the alternative evaluation for CWI. Nevertheless, flow hydrographs and headwater and tailwater stages were also plotted to provide a qualitative baseline of existing conditions at the primary structures. To calculate PM 1, PM 5 and PM 6, the model output from the four design storm simulations is used.

For the wetland hydrologic restoration and water supply metrics, six performance measures were quantified using output from the LTS model:

1. Saltwater intrusion – changes in flow volumes at the tidal structures/outlets closest to the project area (Kehl Canal, Imperial River, COCO1, and GG1) as well as changes to dry season groundwater levels.
2. Sheet flow – changes in overland flow across man-made barriers.
3. Average wet season water depth (July 1 to September 30).
4. Minimum annual dry season water levels.
5. Hydroperiods – months above ground for the water year (June 1 to May 31).
6. Water supply – changes in groundwater levels and flows in the vicinity of public supply wellfields.

Section 4.2 summarizes the flood protection results and measures, Section 4.3 summarizes the wetland hydrologic restoration measures and LTS results, and Section 4.4 shows the water supply output from the LTS that will be used to perform relative comparisons with the FCM baseline and alternatives.

In addition to the performance metrics listed above, additional output from the long term simulation was included in Appendix A: Long Term Simulation Stage and Flow Hydrographs. This output includes:

1. The headwater and tailwater stages, gate opening, and flow at key structures: Kehl Canal Weir, Cork3, Cork2, Cork1, Curry1, CR951N, GG1, and Miller3.
2. The stage and flow at the Bird Rookery north trail bridge and at the I-75 bridges crossing Miller Canal, FakaUnion Canal, and Merrit Canal.
3. Surface water discharges and stages to the Cocohatchee Canal from the north between COCO4 to COCO3. This includes flow from the developments that are north of the canal in the southern part of Corkscrew and from the southern portions of Corkscrew, southwest of Cork1 and Cork2.

4.2 Flood Protection Performance Measures and Design Storm Results

4.2.1 Maximum Canal Stage Profiles

The maximum canal stage profiles can be used to assess where peak stages exceed the canal banks. Four canal reaches within the project area were assessed under this performance metric (Figure 4-1). Maximum simulated canal stages were plotted with the canal bottom elevation and canal left and right top of bank extracted from the model canal cross sections. Modeled structures and canal junctions are also included on the profiles for reference.

The sections of the Corkscrew Canal between Cork 2 and Cypress Canal and the south of Cork 3 flow through rural residential and single family residential land use areas. The Cocohatchee canal flows through highly urbanized areas including residential and commercial land uses. The Kehl Canal upstream of the Kehl Canal Weir is non-urban land use (mostly wetlands) but downstream of the Kehl Canal Weir, there are some residential land uses adjacent to the canal system.

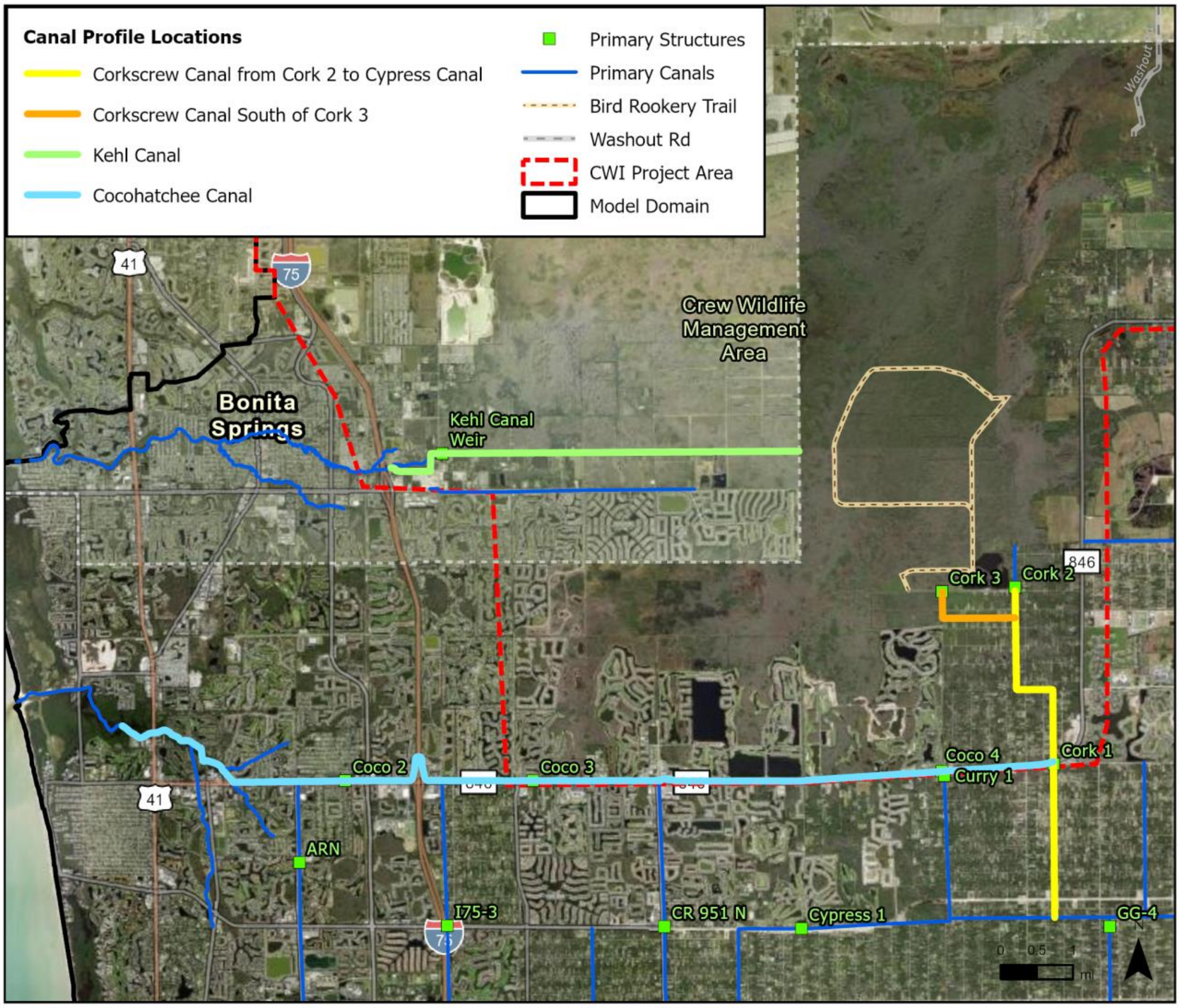

Figure 4-1. Locations of Maximum Canal Stage Profiles.

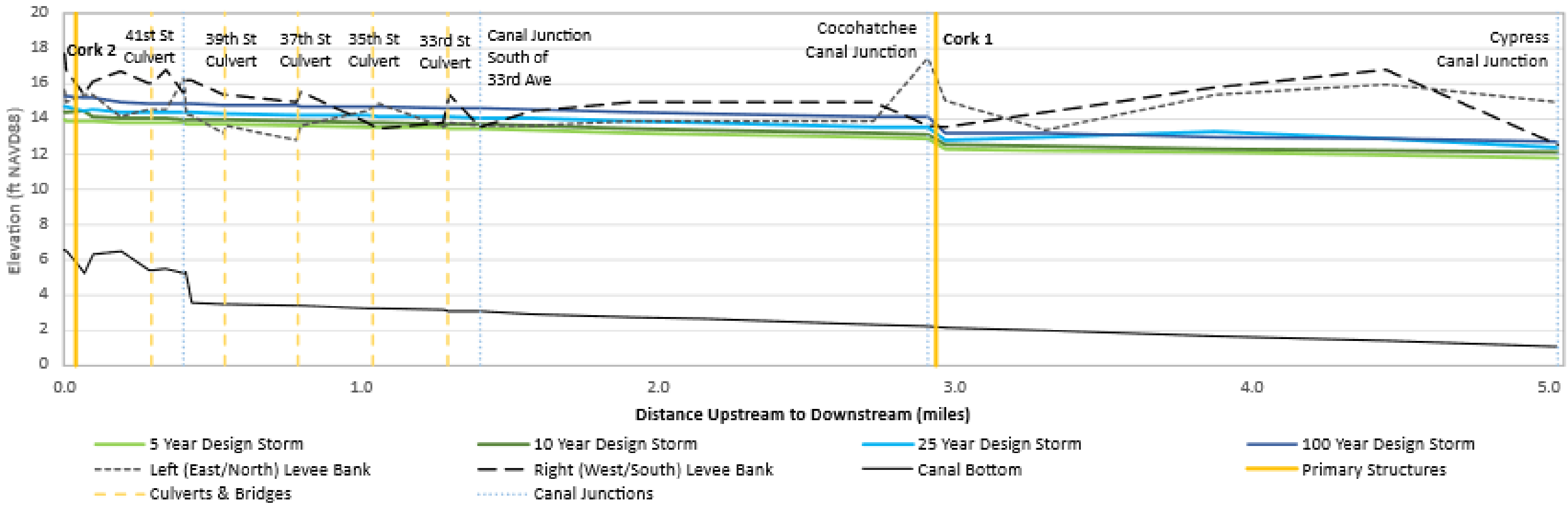

Figure 4-2. Simulated Maximum Water Surface Elevation for the Section of Corkscrew Canal Between Cork 2 and the Cypress Canal..

The section of the Corkscrew Canal between Cork 2 and the Cypress Canal structures (Figure 4-2Figure 4-2) flows through rural residential and mixed non-urban land uses. Simulated maximum water surface elevations from all storm events (100-yr, 25-yr, 10-yr, and 5-yr design events) exceed portions of the banks in this section of canal. Simulated flows from the Bird Rookery area upstream of Cork 2 and additional simulated flows from the Canal Junction between 41st and 39th along with the simulated head losses at Cork 1 and multiple culverts within the reach contribute to the high simulated maximum water surface profiles in the canal.

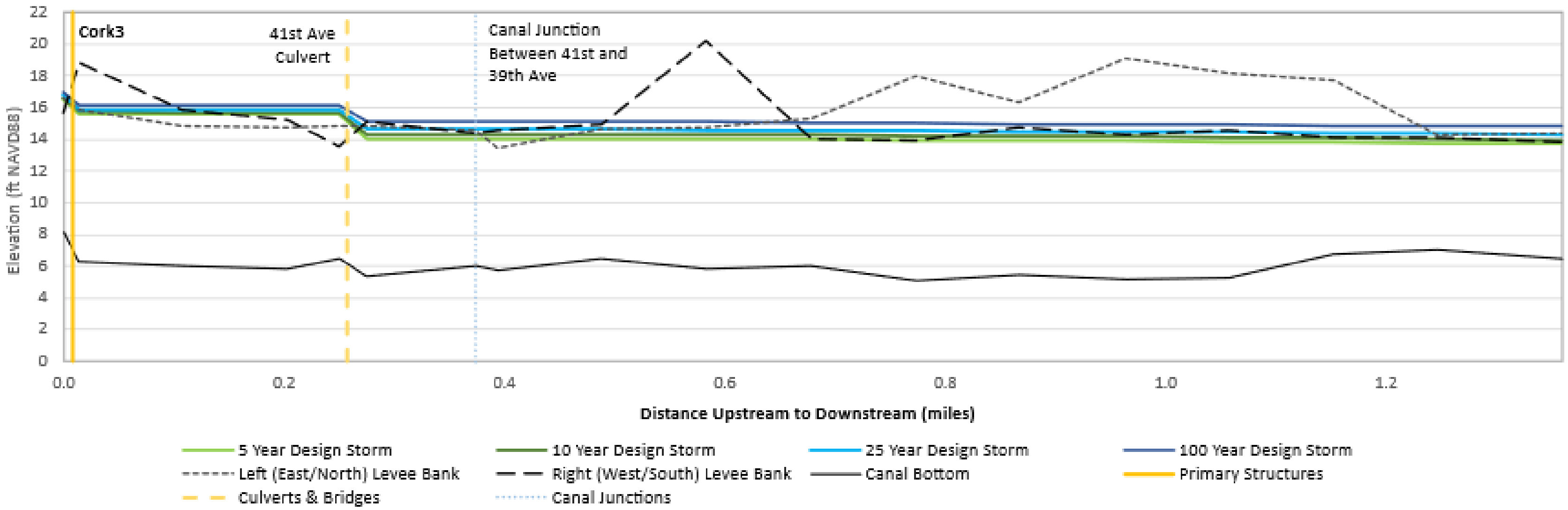

Figure 4-3. Simulated Maximum Water Surface Elevation for the Section of the Corkscrew Canal South of Cork 3.

The section of the Corkscrew Canal south of Cork 3 (Figure 4-3) flows through rural residential, single family residential, and mixed non-urban land uses. Simulated maximum water surface elevations from all storm events (100-yr, 25-yr, 10-yr, and 5-yr design events) exceed portions of the banks in the upstream area of this canal section. Simulated head loss through the 41st Ave culvert contributes to higher simulated maximum stages in this area. Land use to the west of the canal (right bank) upstream of 41st Ave culvert is wetland, while land use to the east of the canal (left bank) is rural residential. Downstream of the 41st Ave culvert, all design storm events simulated maximum stages exceed the right bank and some portions of the left bank in an area of rural residential land use.

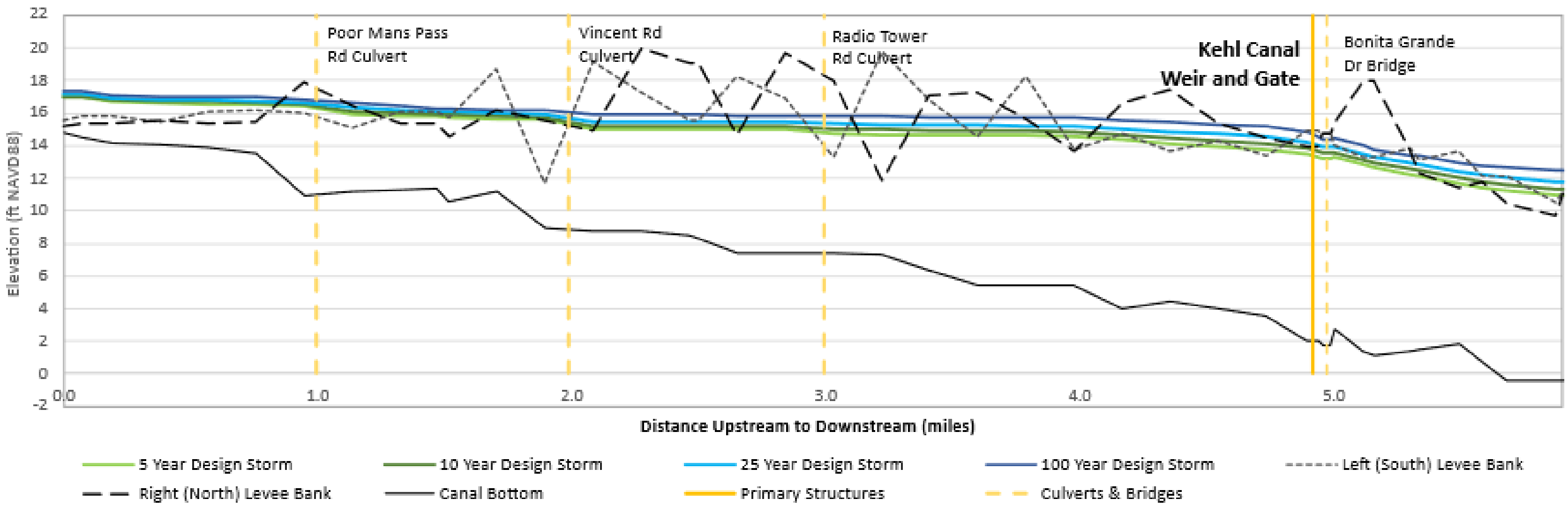

Figure 4-4. Simulated Maximum Water Surface Elevation for the Kehl Canal.

The Kehl Canal (Figure 4-4) flows through mostly wetland and other non-urban land uses. Downstream of the Kehl Canal Weir and Gate, there is some residential land use adjacent to the canal system. Simulated maximum water surface elevations from all storm events (100-yr, 25-yr, 10-yr, and 5-yr design events) exceed the banks in multiple locations along the Kehl canal. The Kehl canal collects flows from a large area within the Corkscrew Wetlands area, which contributes to high simulated stages in the canal during simulated design storm events. Simulated head loss through the culverts and Kehl Canal Weir and Gate also contribute to higher simulated stages in the upstream portion of the canal.

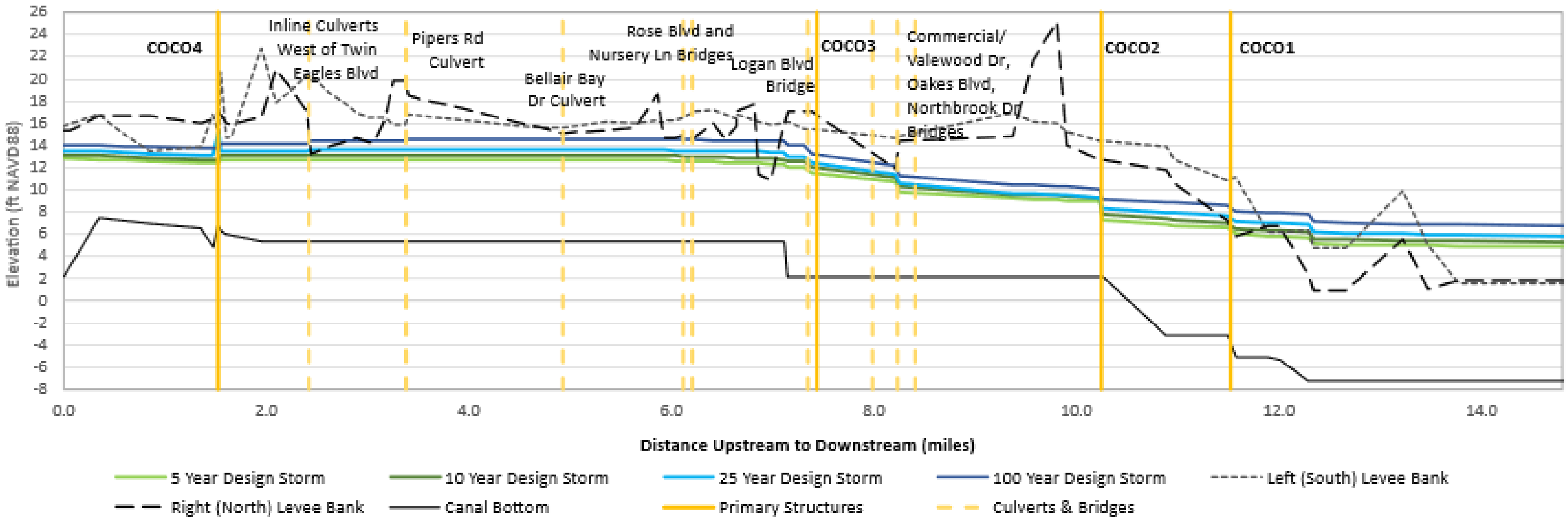

Figure 4-5. Simulated Maximum Water Surface Elevation for the Cocohatchee Canal.

The Cocohatchee canal (Figure 4-5) flows through highly urbanized areas including residential and commercial land uses. Simulated maximum water surface elevations from all storm events (100-yr, 25-yr, 10-yr, and 5-yr design events) exceed portions of the right bank between mile 6 and 8, upstream of COCO3, which is a wetland area that discharges to the Cocohatchee Canal. Other simulated maximum water surface elevations from the 25-yr and 100-yr design events that exceed the right bank near the COCO4 structure are in rural residential and single family residential areas. Downstream of COCO1, simulated maximum water surface elevations from all storm events (100-yr, 25-yr, 10-yr, and 5-yr design events) exceed the banks in the mangrove swamp land use areas.

4.2.2 Flow and Stage Hydrographs

The simulated flow and stage hydrographs resulting from the ECM design storm simulations are presented below. The results for each design storm are presented by structure. The following structures are presented in this section: Cork 3, Cork 2, Cork 1, Curry 1, COCO4, COCO3, COCO2, COCO1, the Kehl Canal Weir and Gate, CR951N, GG1, Miller Canal Bridge at I-75, and Bird Rookery North Bridge.

4.2.2.1 Cork 3

The results for the Cork 3 structure simulated flow and stage hydrographs with simulated gate levels for each design storm are presented in Figure 4-6 through Figure 4-9. The Cork3 structure simulated gate levels are open for the entire design storm simulation for all simulated design storms because the Cork 3 simulated headwater initial condition starts above the Cork 3 wet season optimal control elevation and does not recede below that level before the simulation ends. The design storm initial conditions are discussed in Section 3.1.3. Note that the discharges shown in the plots are the total discharge through the structure, including flow over the concrete weir, through which the largest portion of the total flow through the structure occurs.

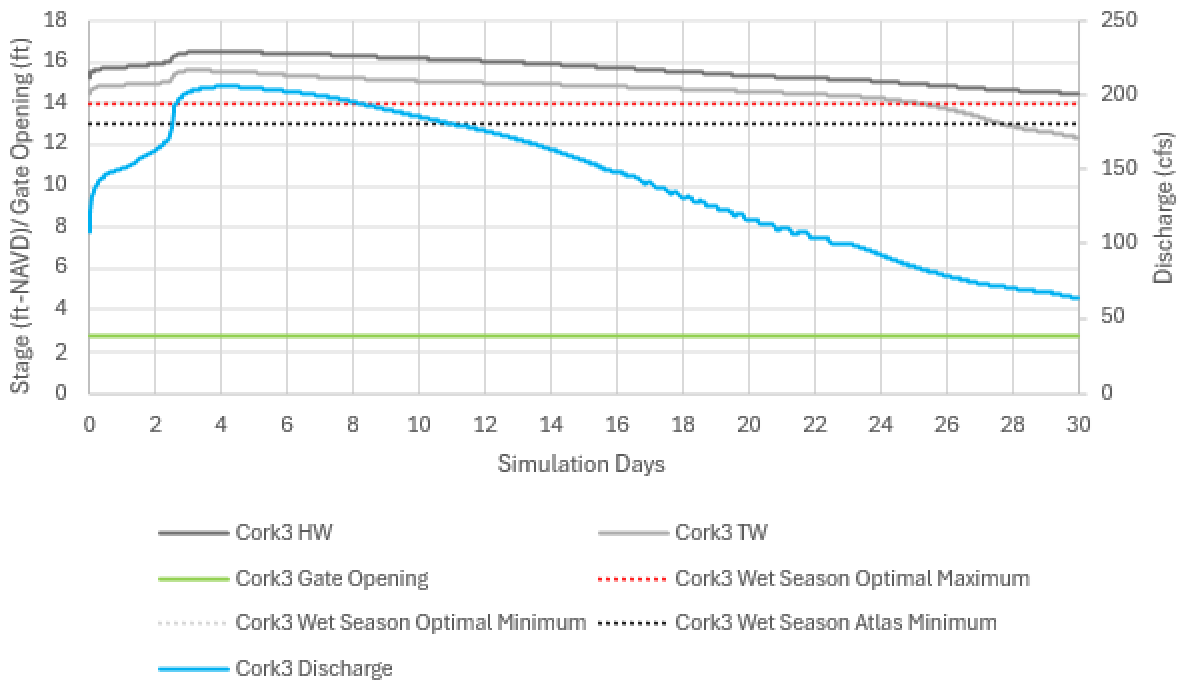

Figure 4-6. 5-year Design Storm Simulated Flow and Stage Hydrographs for Cork 3

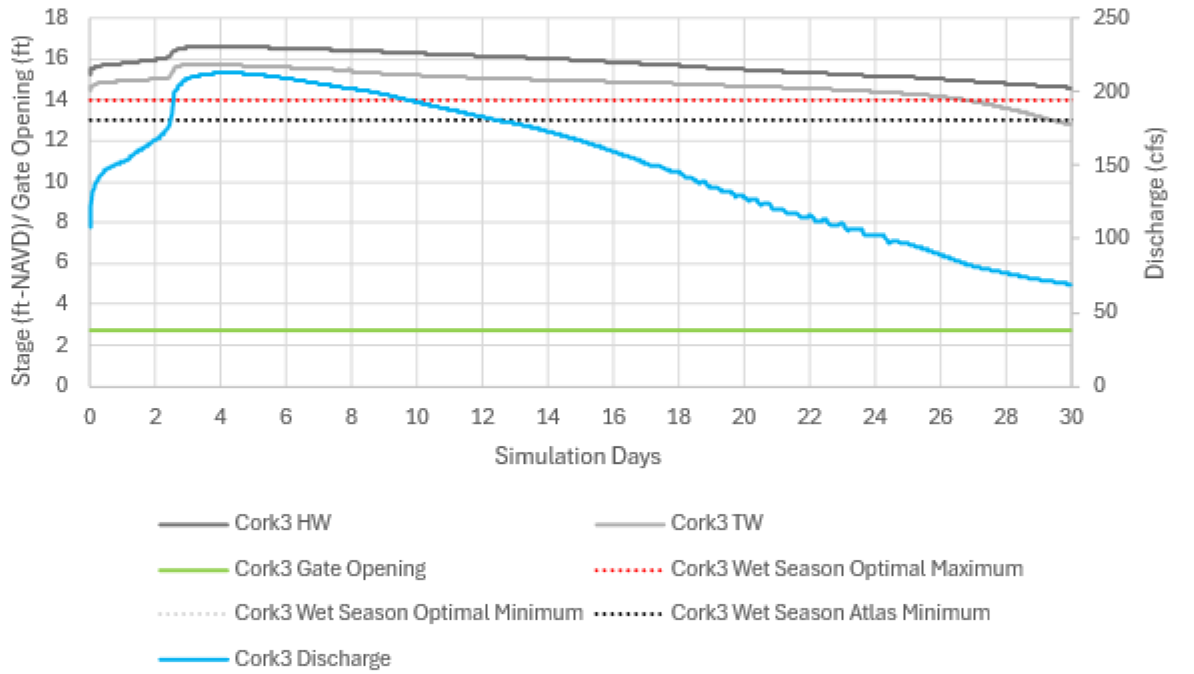

Figure 4-7. 10-year Design Storm Simulated Flow and Stage Hydrographs for Cork 3

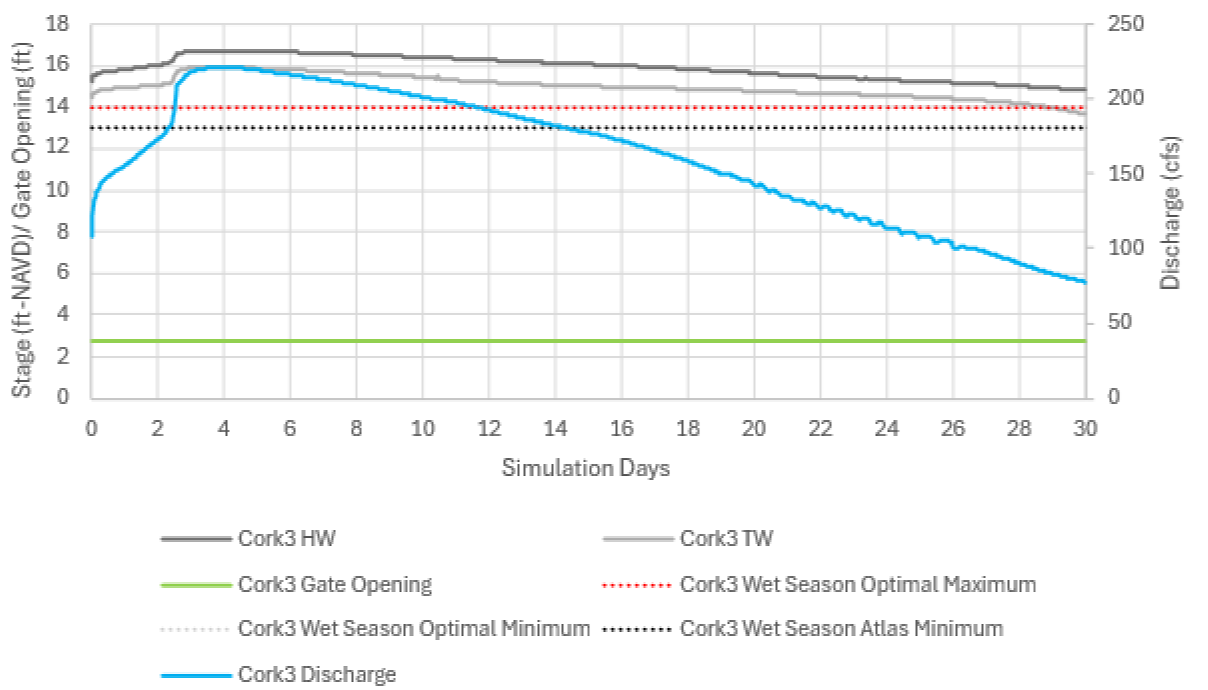

Figure 4-8. 25-year Design Storm Simulated Flow and Stage Hydrographs for Cork 3

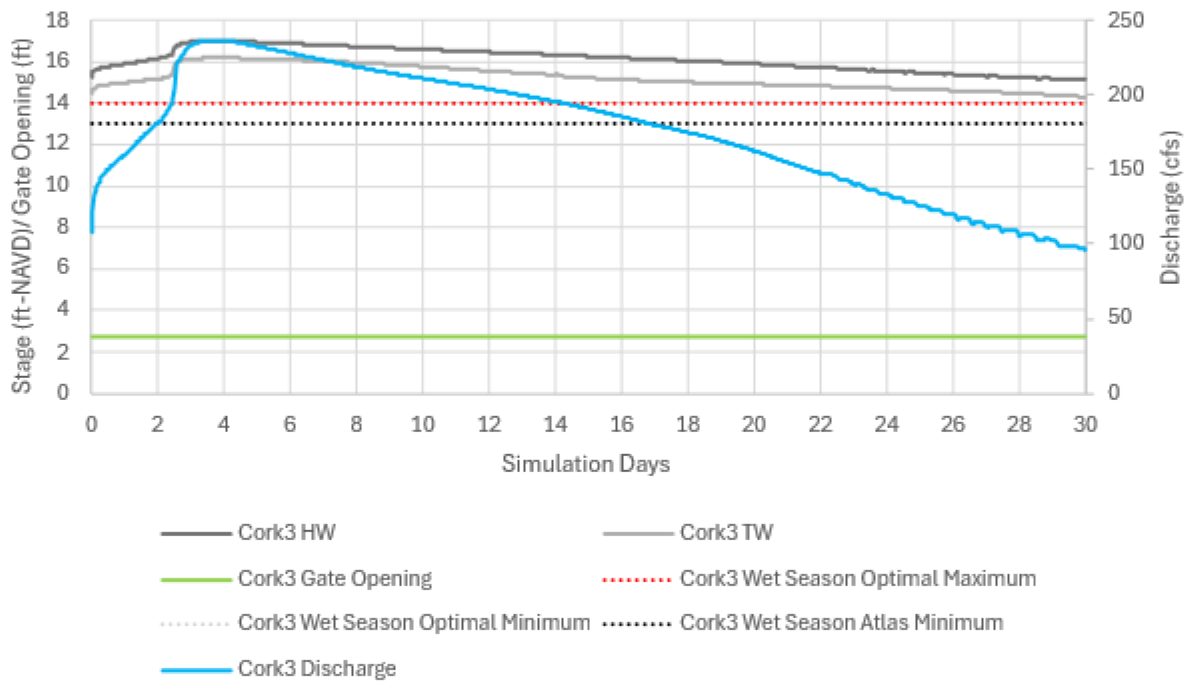

Figure 4-9. 100-year Design Storm Simulated Flow and Stage Hydrographs for Cork 3

4.2.2.2 Cork 2

The results for the Cork 2 structure simulated flow and stage hydrographs with simulated gate levels for each design storm are presented in Figure 4-10 through Figure 4-13. The Cork 2 structure simulated gate levels open fully at the beginning of the simulation and remain fully open until the simulated headwater levels reach the wet season optimal control elevation, at which time gates begin to close to maintain the headwater within the Cork 2 wet season optimal range. Note that the high tailwater levels seem to constrict the flow through the structure in all storms.

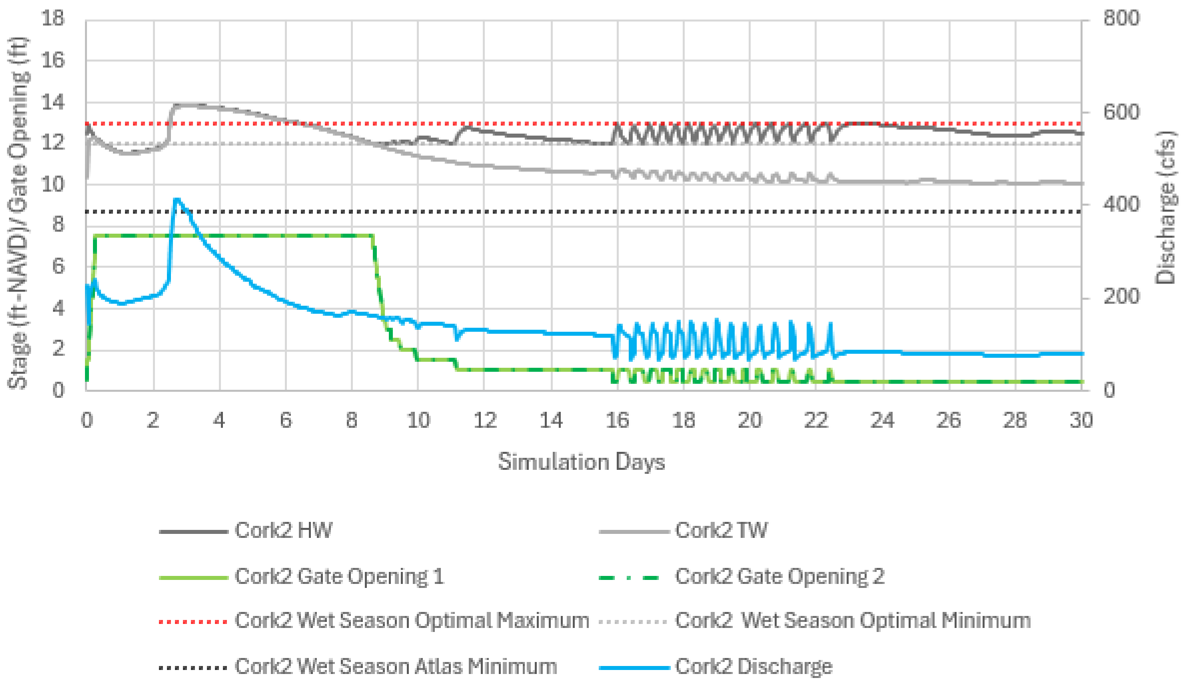

Figure 4-10. 5-year Design Storm Simulated Flow and Stage Hydrographs for Cork 2

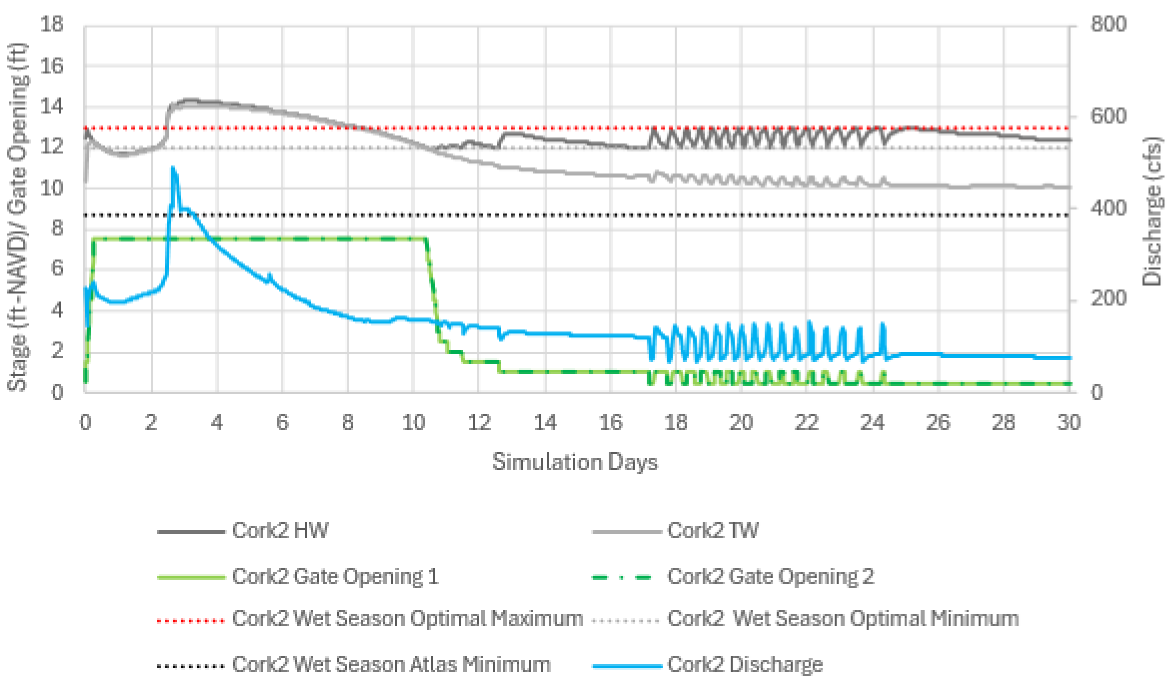

Figure 4-11. 10-year Design Storm Simulated Flow and Stage Hydrographs for Cork 2

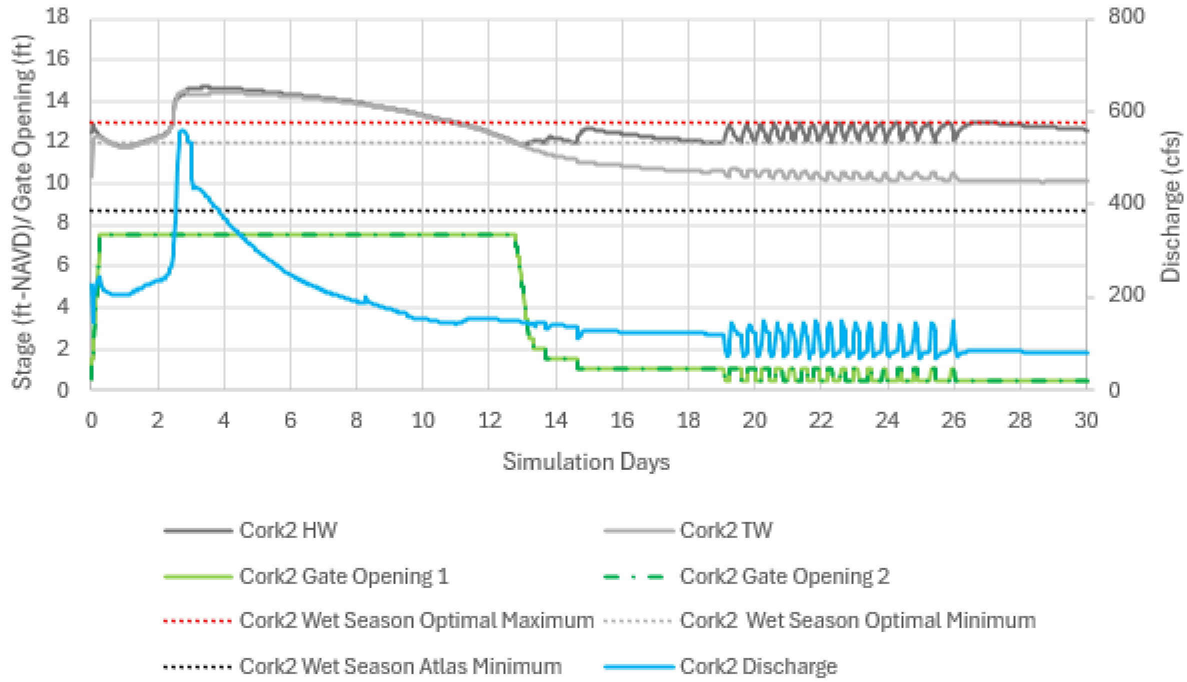

Figure 4-12. 25-year Design Storm Simulated Flow and Stage Hydrographs for Cork 2

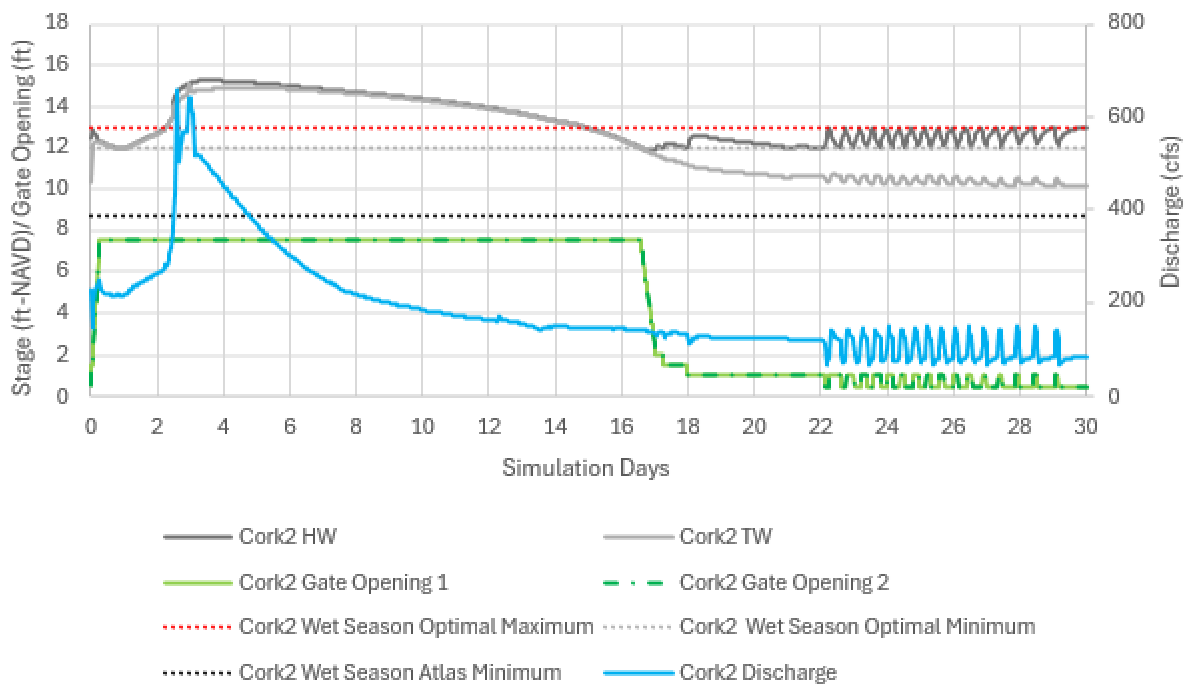

Figure 4-13. 100-year Design Storm Simulated Flow and Stage Hydrographs for Cork 2

4.2.2.3 Cork 1

The results for the Cork 1 structure simulated flow and stage hydrographs with simulated gate levels for each design storm are presented in Figure 4-14 through Figure 4-17. The Cork 1 structure simulated gate levels open fully at the beginning of the simulation and remain fully open until the simulated headwater levels reach the wet season optimal control elevation, at which time gates begin to close to maintain the headwater within the Cork 1 wet season optimal range. The decrease in simulated discharge through the Cork 1 structure within the first 5 days of the simulation is correlated with the time at which the simulated gate levels at COCO4 structure, which discharges towards the Cork 1 structure, close due to the high tailwater stages.

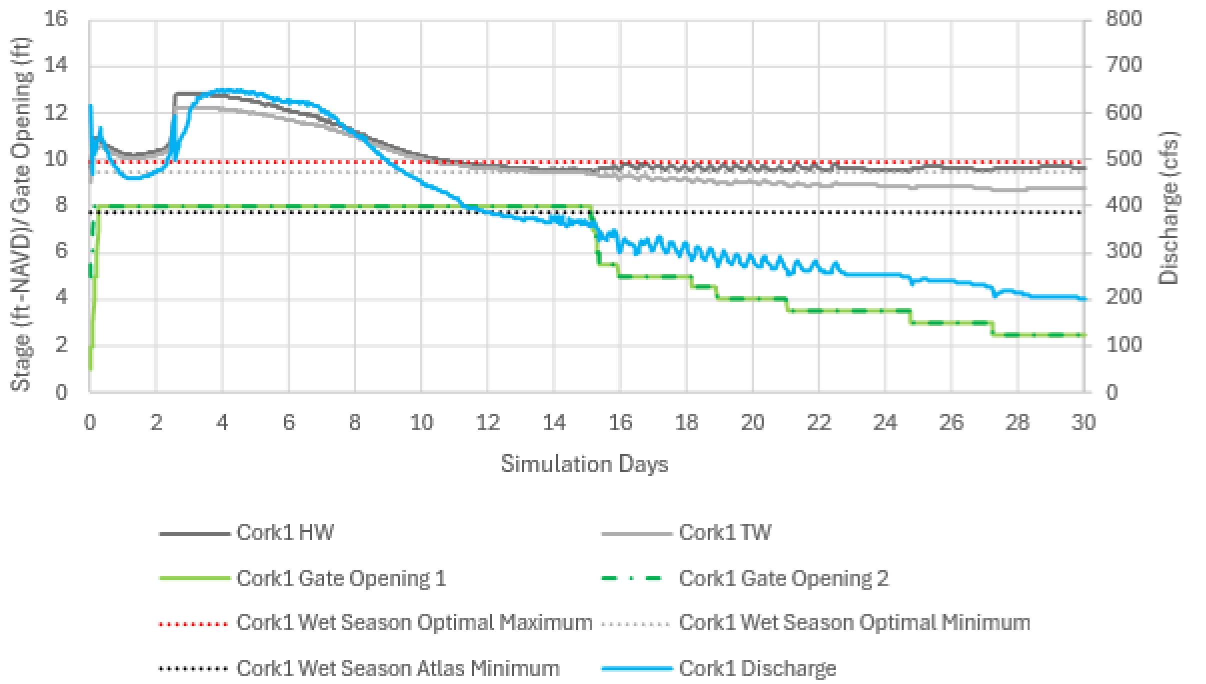

Figure 4-14. 5-year Design Storm Simulated Flow and Stage Hydrographs for Cork 1

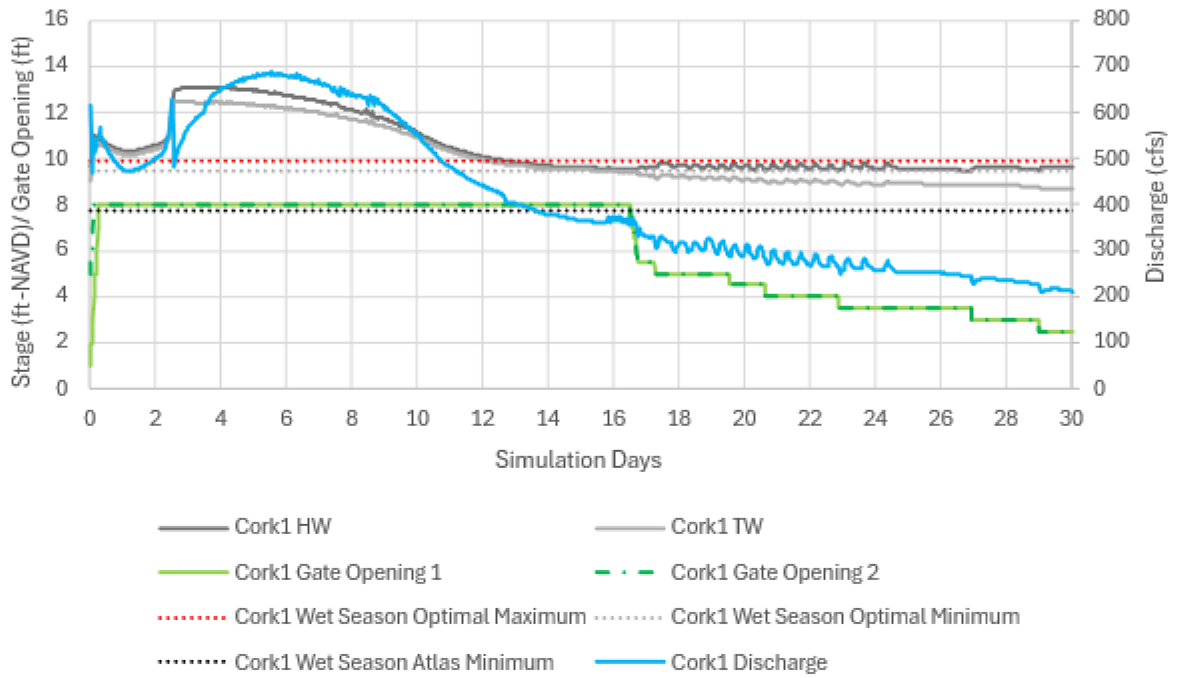

Figure 4-15. 10-year Design Storm Simulated Flow and Stage Hydrographs for Cork 1

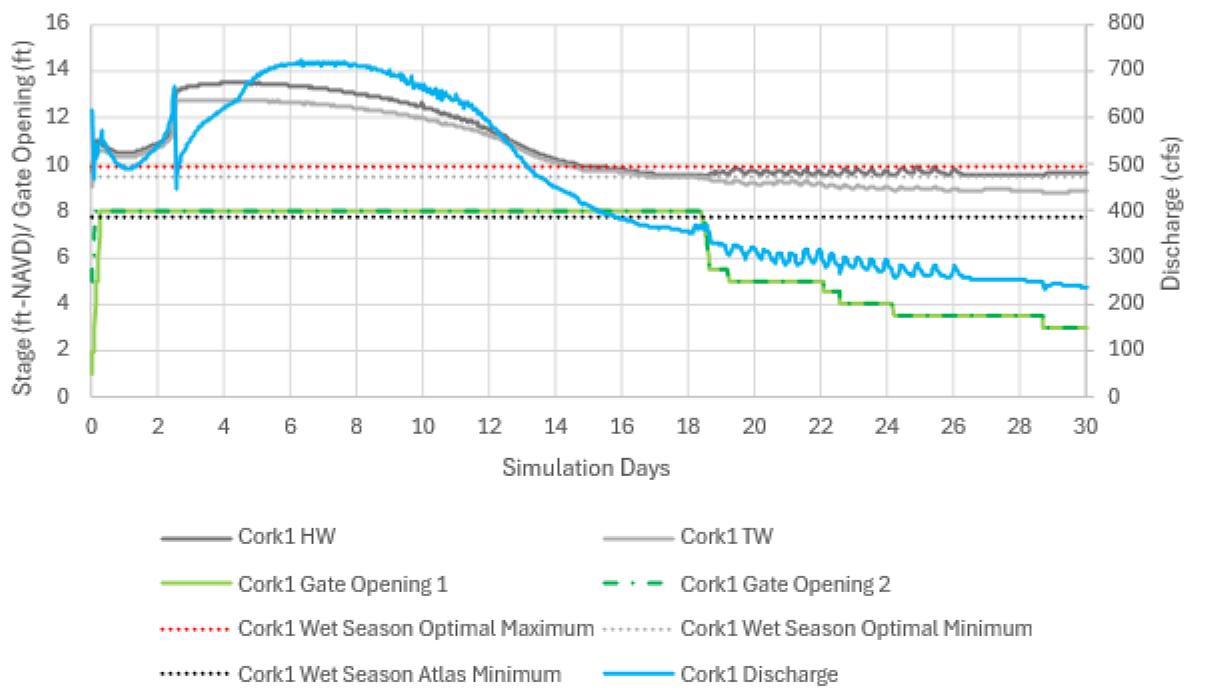

Figure 4-16. 25-year Design Storm Simulated Flow and Stage Hydrographs for Cork 1

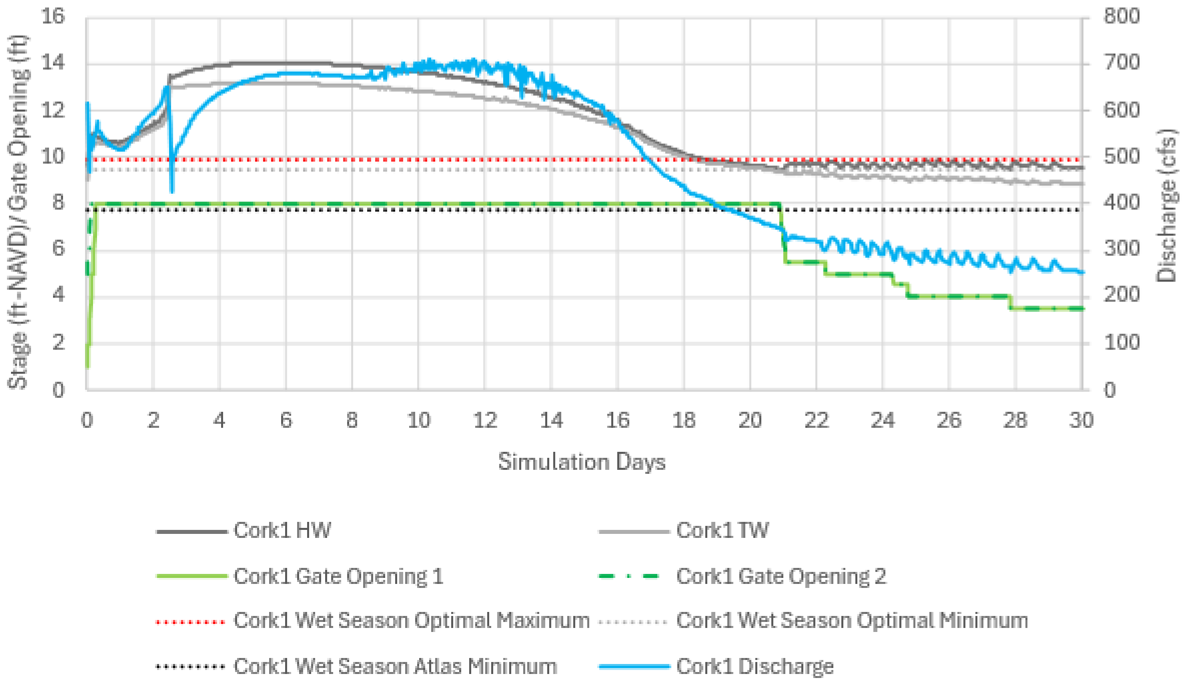

Figure 4-17. 100-year Design Storm Simulated Flow and Stage Hydrographs for Cork 1

4.2.2.4 Curry 1

The results for the Curry 1 structure simulated flow and stage hydrographs with simulated gate levels for each design storm are presented in Figure 4-18 through Figure 4-21. The Curry 1 structure simulated gate levels open fully at the beginning of the simulation and remain fully open until the simulated headwater levels reach the wet season optimal control elevation, at which time simulated gate levels vary to maintain the headwater within the Curry 1 wet season optimal range. Note that the discharges through Curry 1 seem to be constricted due to the high tailwater stages.

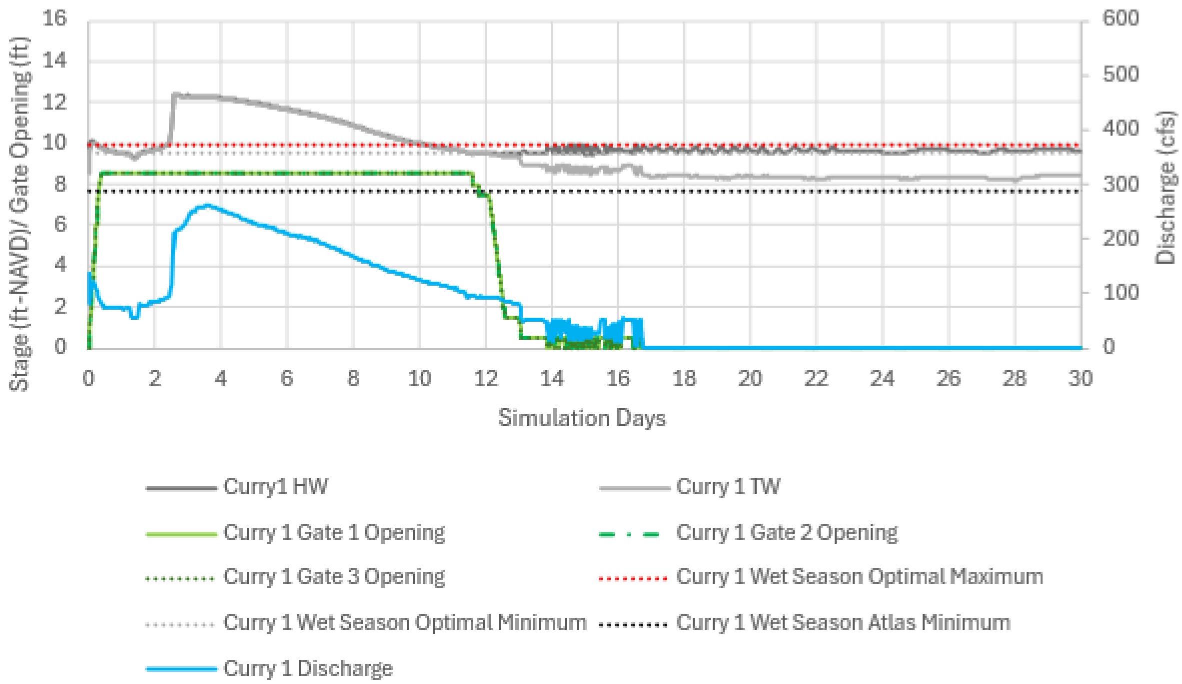

Figure 4-18. 5-year Design Storm Simulated Flow and Stage Hydrographs for Curry 1

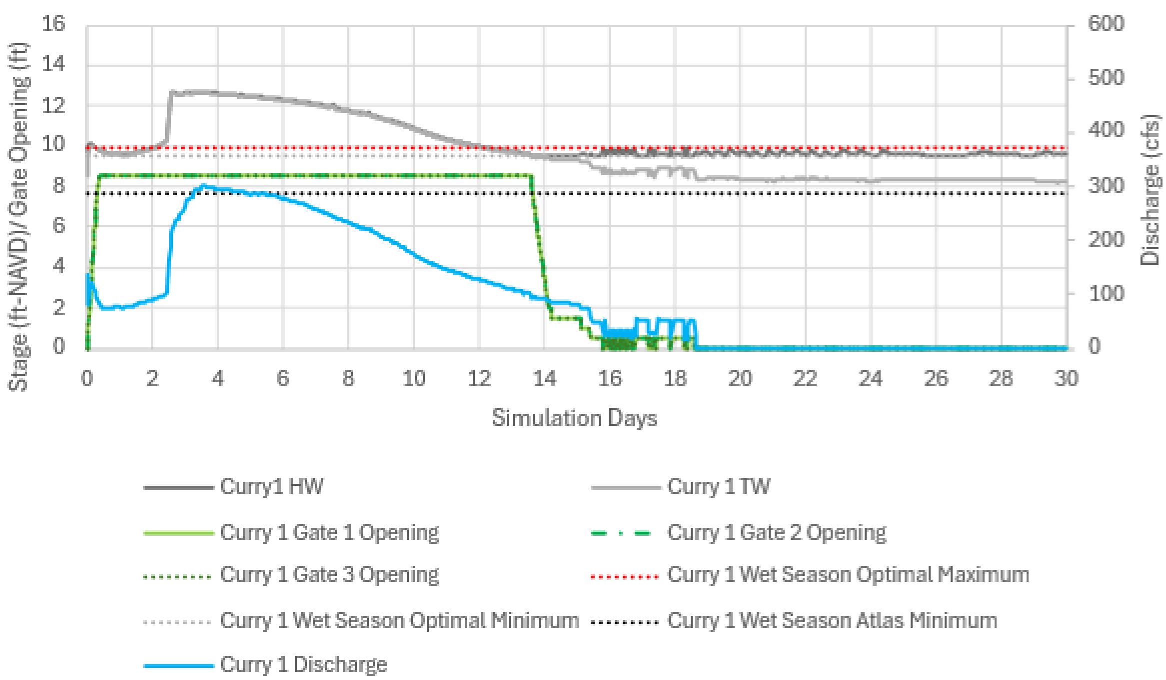

Figure 4-19. 10-year Design Storm Simulated Flow and Stage Hydrographs for Curry 1

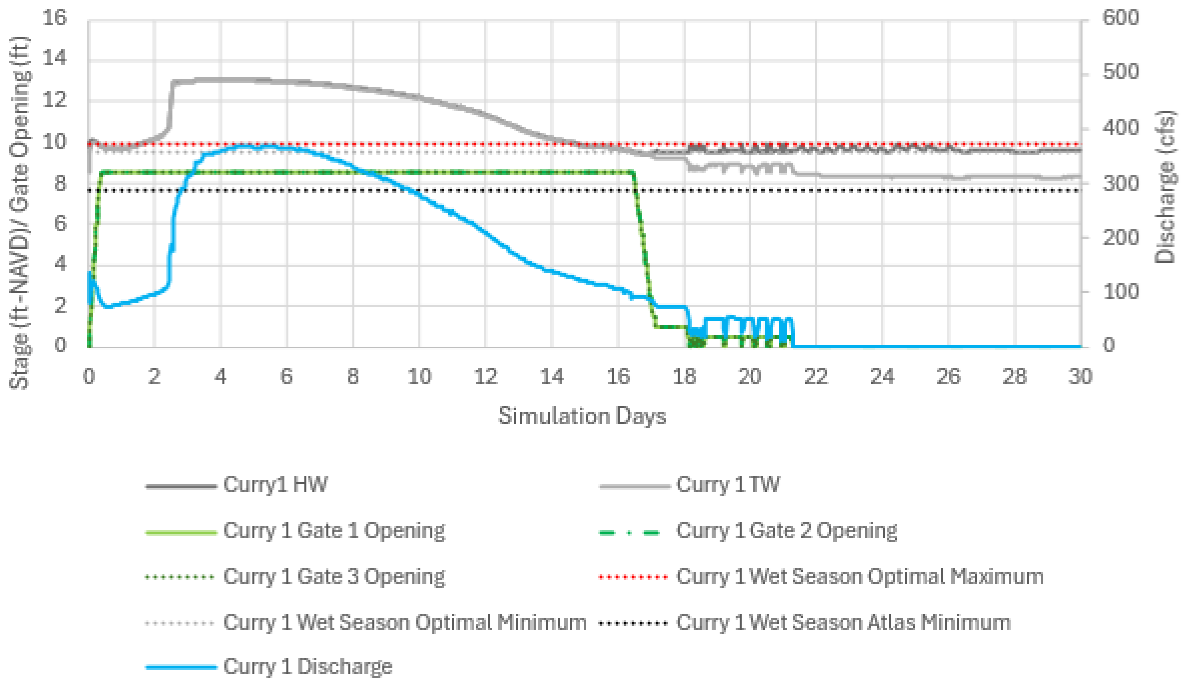

Figure 4-20. 25-year Design Storm Simulated Flow and Stage Hydrographs for Curry 1

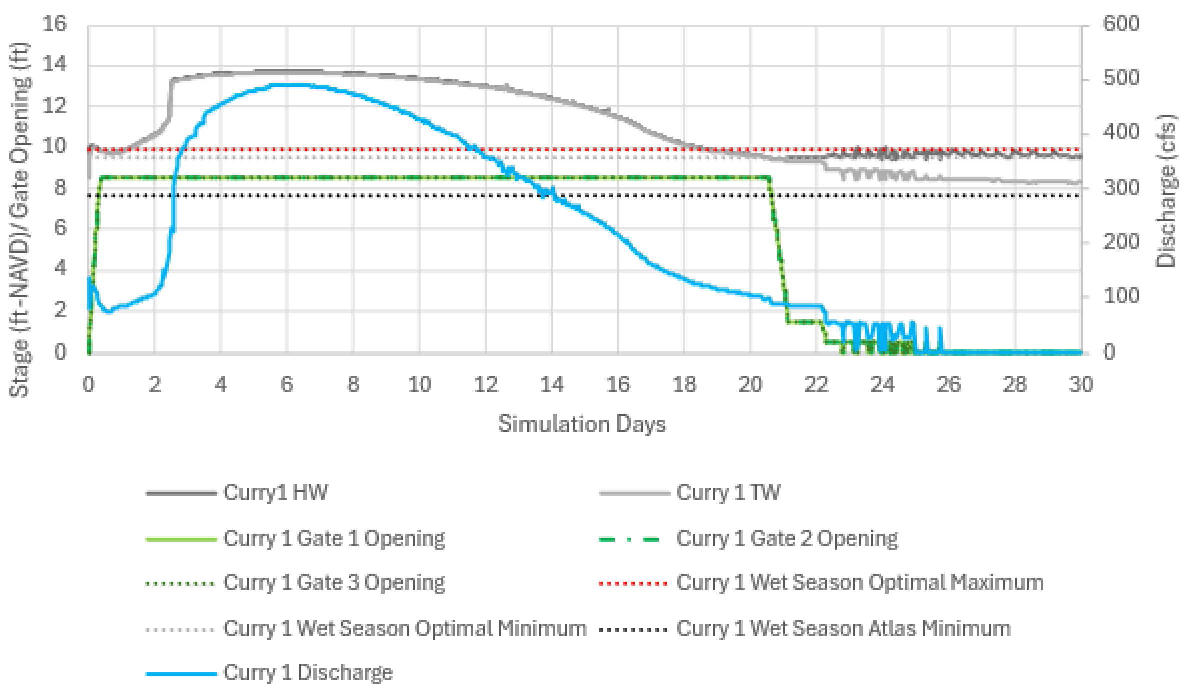

Figure 4-21. 100-year Design Storm Simulated Flow and Stage Hydrographs for Curry 1

4.2.2.5 COCO4

The results for the COCO4 structure simulated flow and stage hydrographs with simulated gate levels for each design storm are presented in Figure 4-22 through Figure 4-25. The COCO4 structure simulated gate levels open at the beginning of the simulation but close when simulated headwater levels drop below the wet season optimal minimum control elevation. The simulated gate levels close again before the peak to prevent reverse flow when the COCO4 simulated tailwater is higher than the COCO4 simulated headwater. Because the COCO4 structure is manually operated, the simulated levels are coded to only change every 6 hours. Therefore, the simulated gate levels take approximately 24 hours to fully open from closed level when opening 1 ft every 6 hours in storm conditions and take 48 hours to fully open from closed level when opening 0.5 ft every 6 hours outside of storm conditions.

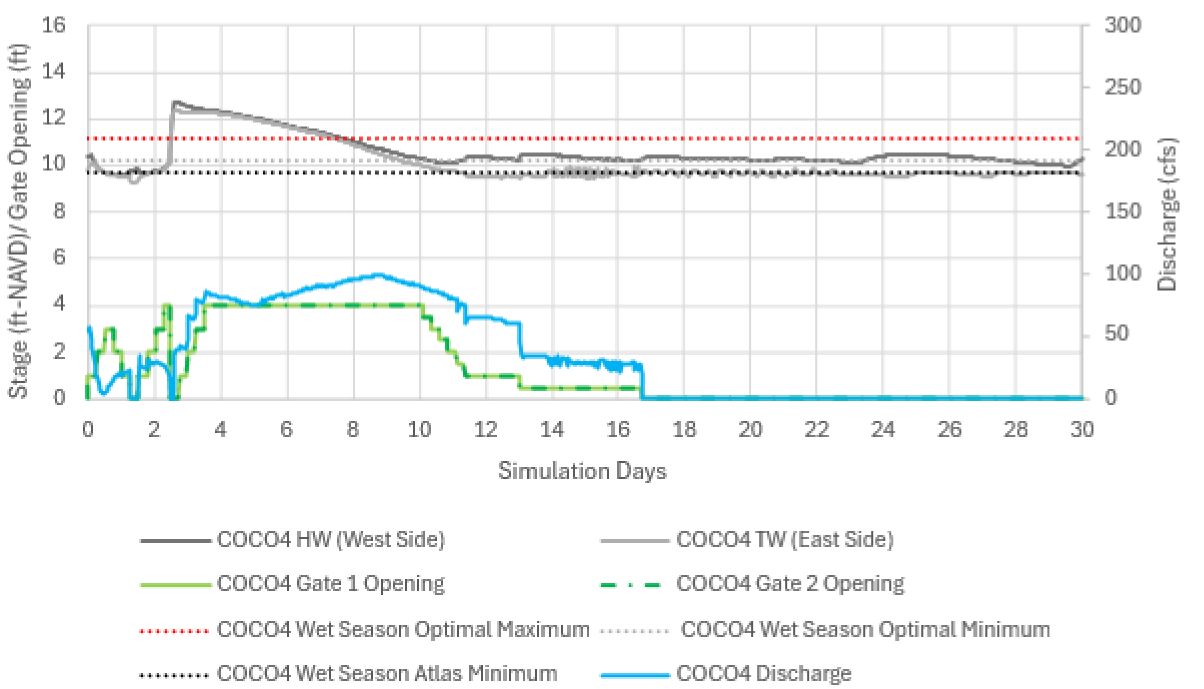

Figure 4-22. 5-year Design Storm Simulated Flow and Stage Hydrographs for COCO4

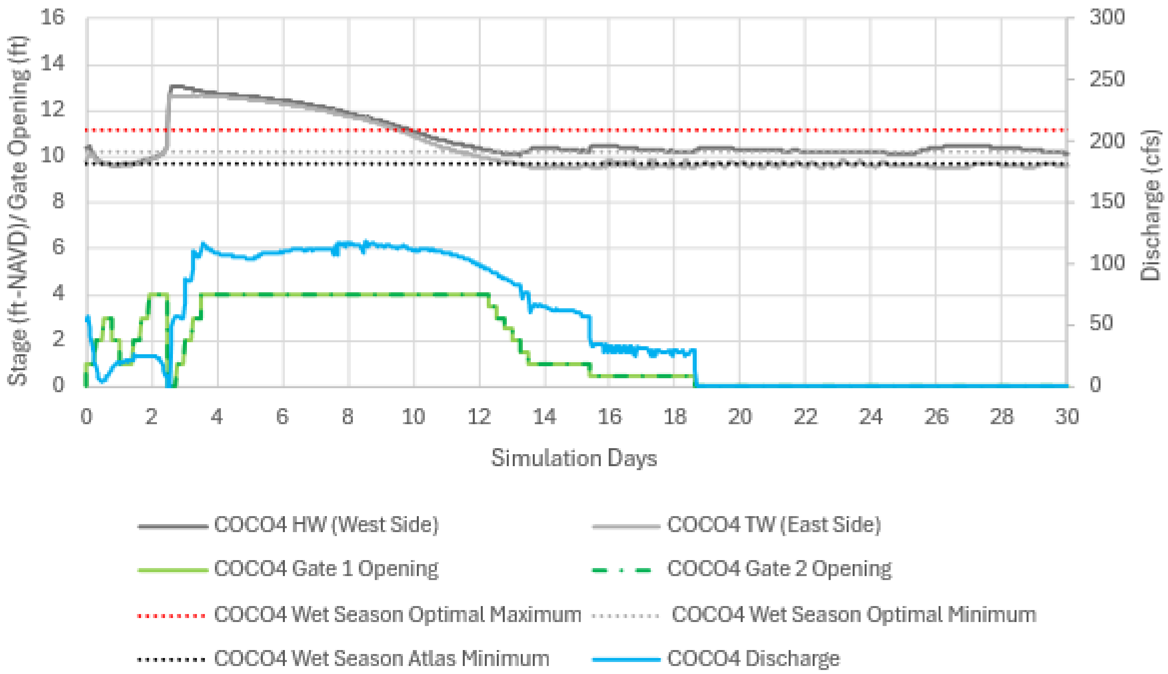

Figure 4-23. 10-year Design Storm Simulated Flow and Stage Hydrographs for COCO4

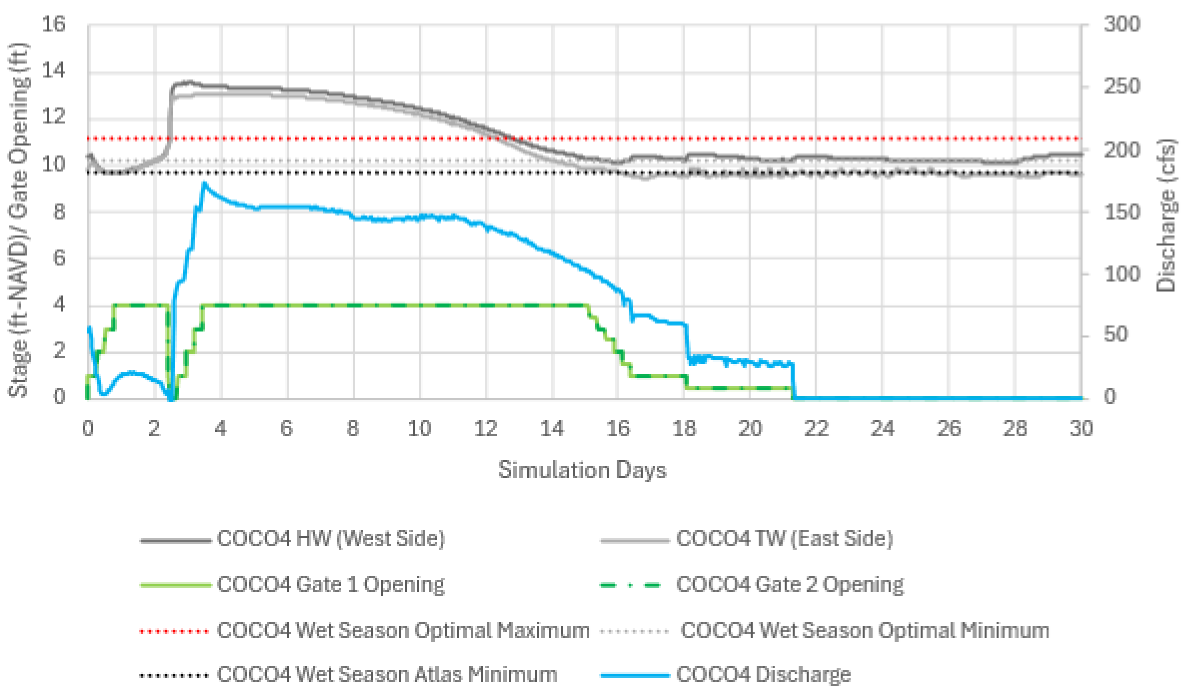

Figure 4-24. 25-year Design Storm Simulated Flow and Stage Hydrographs for COCO4

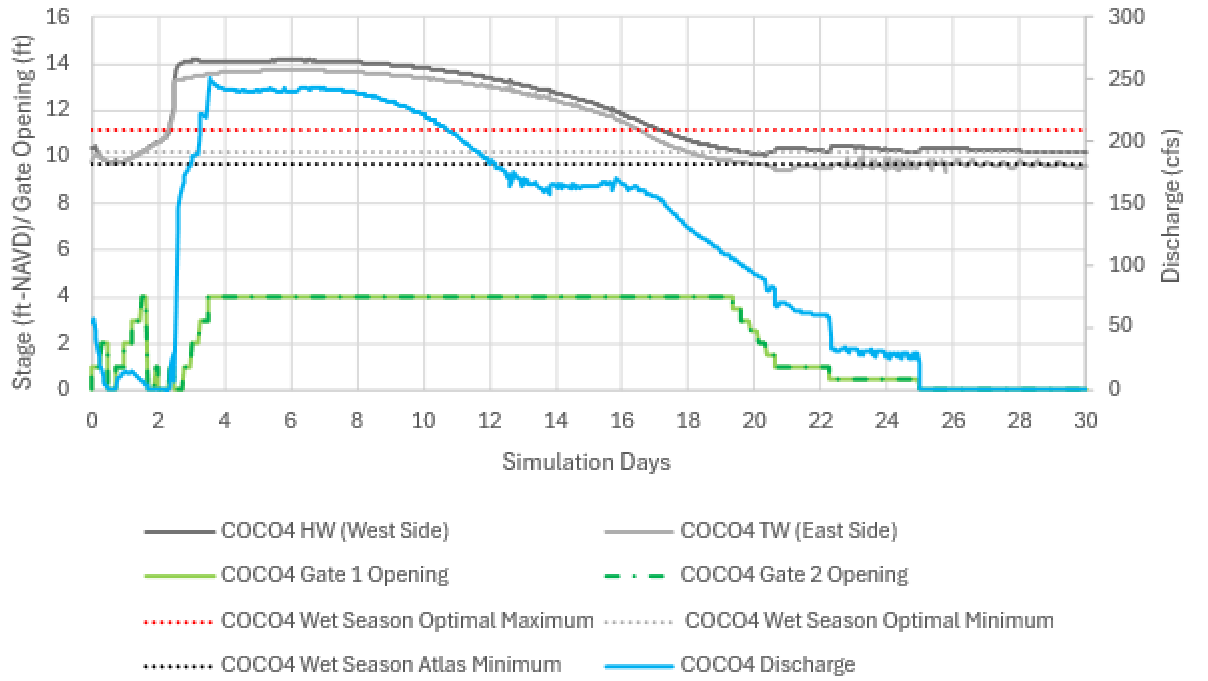

Figure 4-25. 100-year Design Storm Simulated Flow and Stage Hydrographs for COCO4

4.2.2.6 COCO3

The results for the COCO3 simulated flow and stage hydrographs with simulated gate levels structure for each design storm are presented in Figure 4-26 through Figure 4-29. The COCO3 structure simulated gate levels open fully at the beginning of the simulation and remain fully open until the simulated headwater levels reach the wet season optimal control elevation, at which time gates begin to close to maintain the headwater within the COCO3 wet season optimal range.

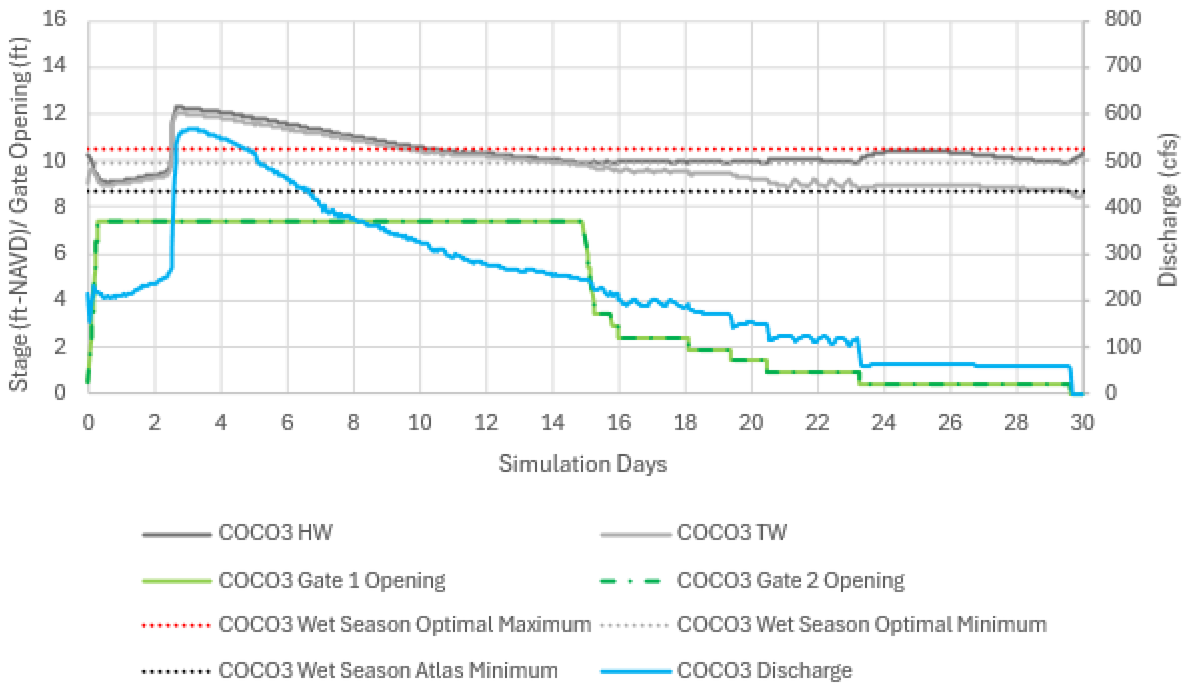

Figure 4-26. 5-year Design Storm Simulated Flow and Stage Hydrographs for COCO3

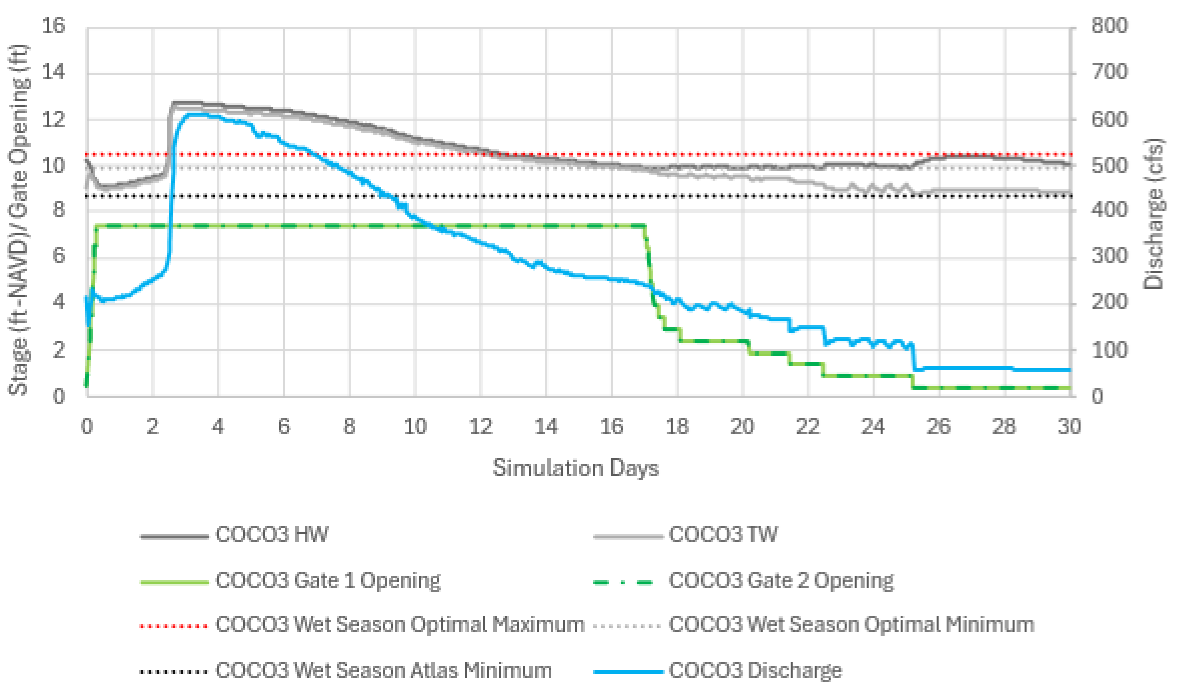

Figure 4-27. 10-year Design Storm Simulated Flow and Stage Hydrographs for COCO3

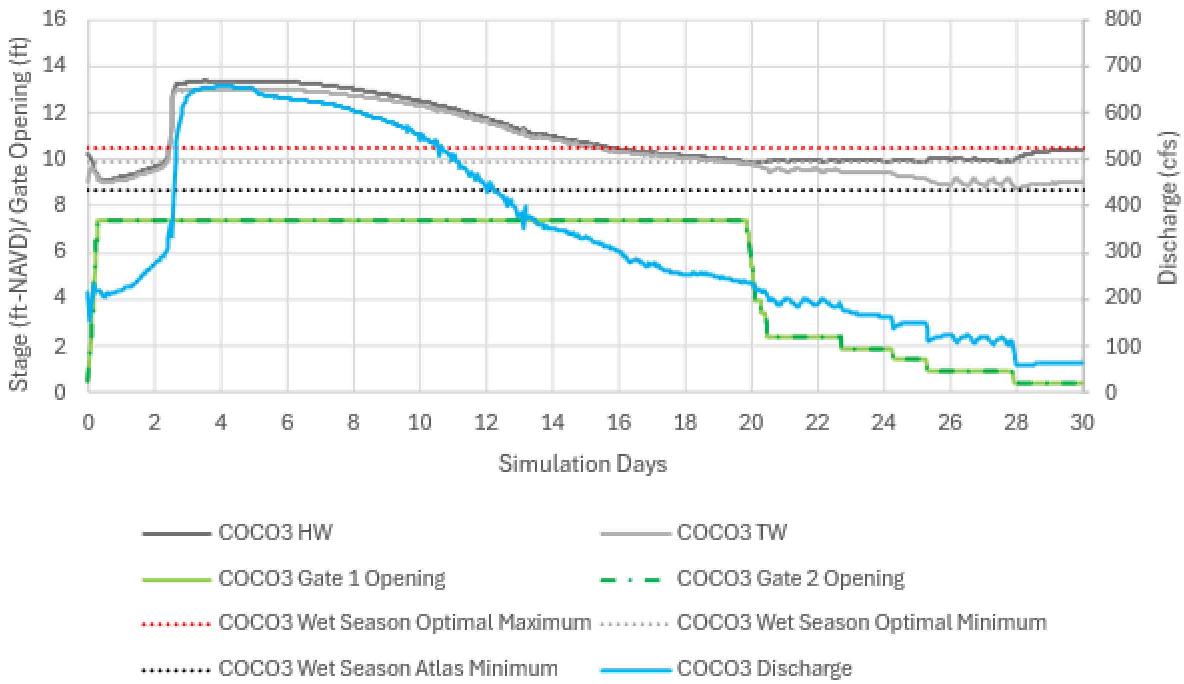

Figure 4-28. 25-year Design Storm Simulated Flow and Stage Hydrographs for COCO3

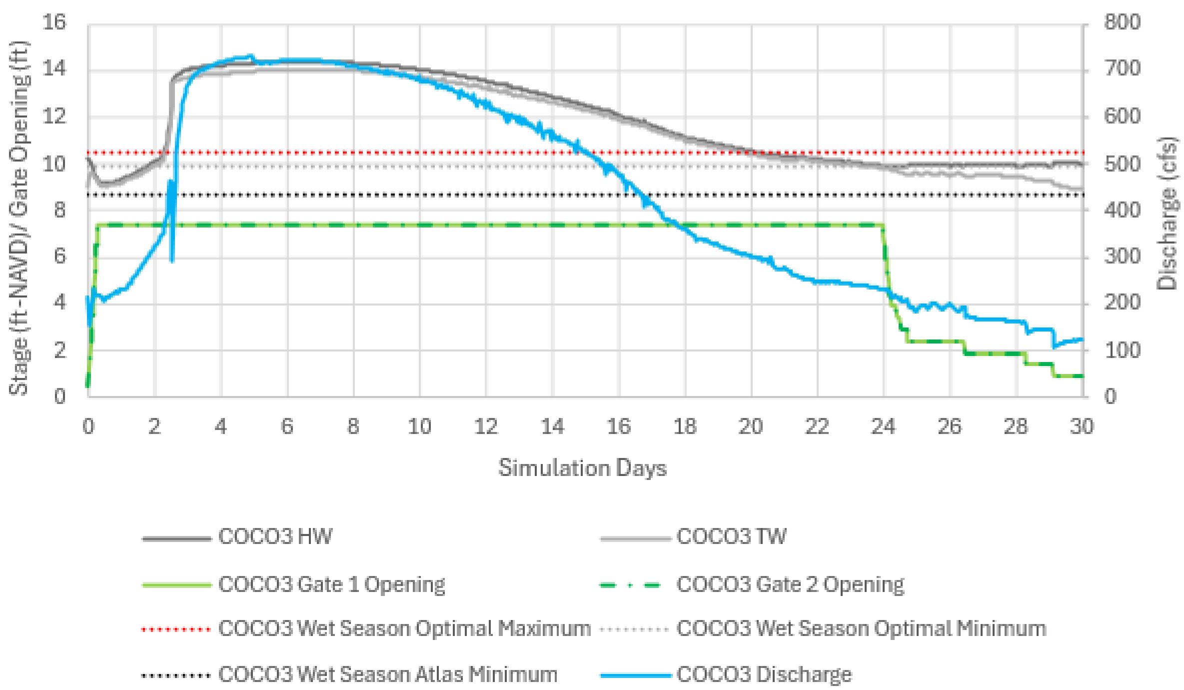

Figure 4-29. 100-year Design Storm Simulated Flow and Stage Hydrographs for COCO3

4.2.2.7 COCO2

The results for the COCO2 simulated flow and stage hydrographs with simulated gate levels structure for each design storm are presented in Figure 4-30 through Figure 4-33. The COCO2 structure simulated gate levels lower multiple times at the beginning of the simulations because the simulated headwater levels drop below the wet season optimal minimum control elevation at the COCO2 structure. A 1-ft gate opening at this structure results in a larger fluctuation in headwater stage and flow than upstream structures, COCO3 and COCO4, due to a larger difference in headwater and tailwater at this structure. The COCO2 structure is at a location where the modeled canal slope and water surface elevation slope increases (see Cocohatchee Canal profile in Figure 4-5).

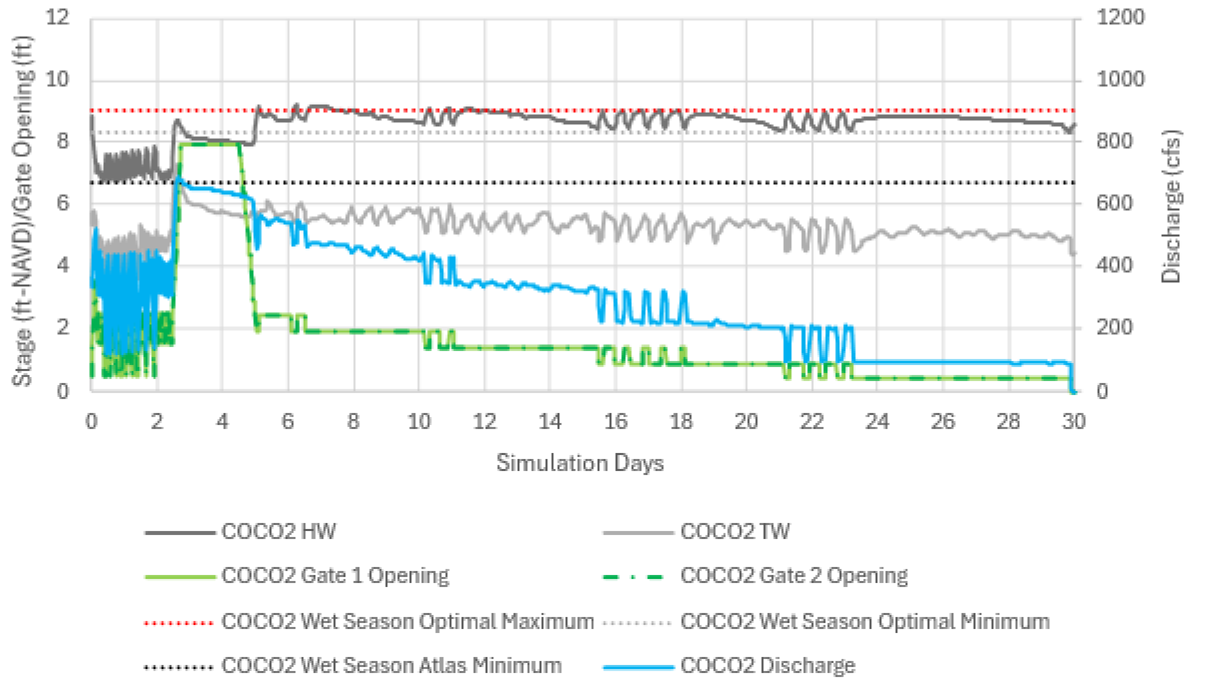

Figure 4-30. 5-year Design Storm Simulated Flow and Stage Hydrographs for COCO2

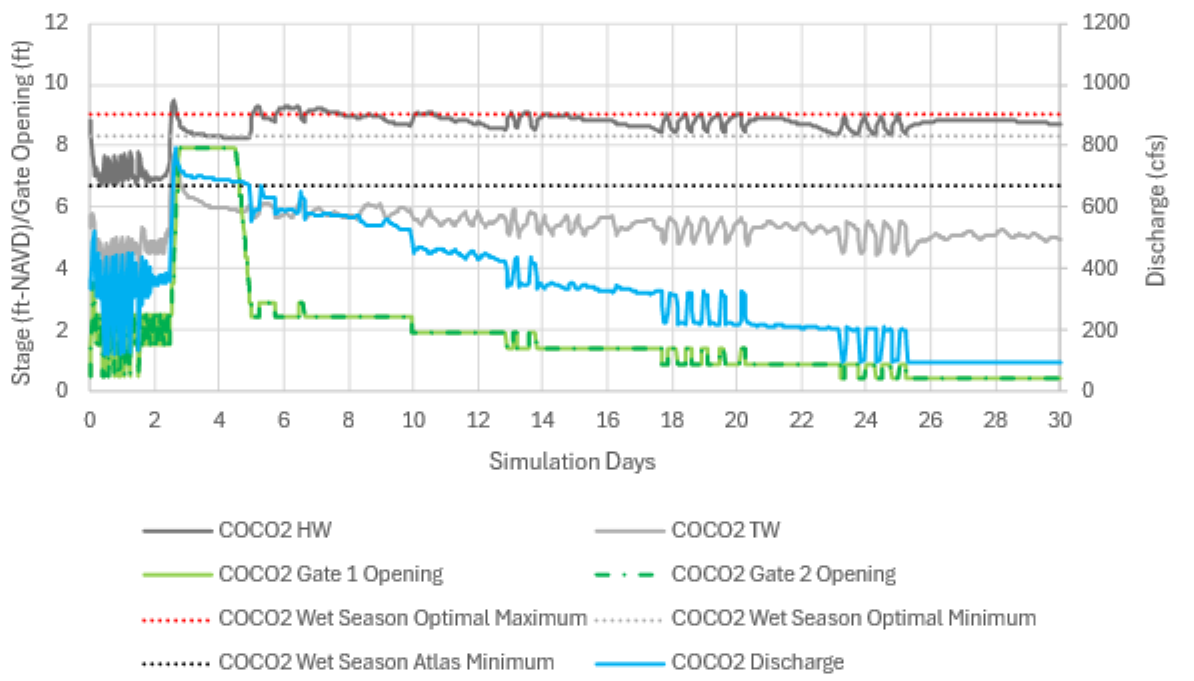

Figure 4-31. 10-year Design Storm Simulated Flow and Stage Hydrographs for COCO2

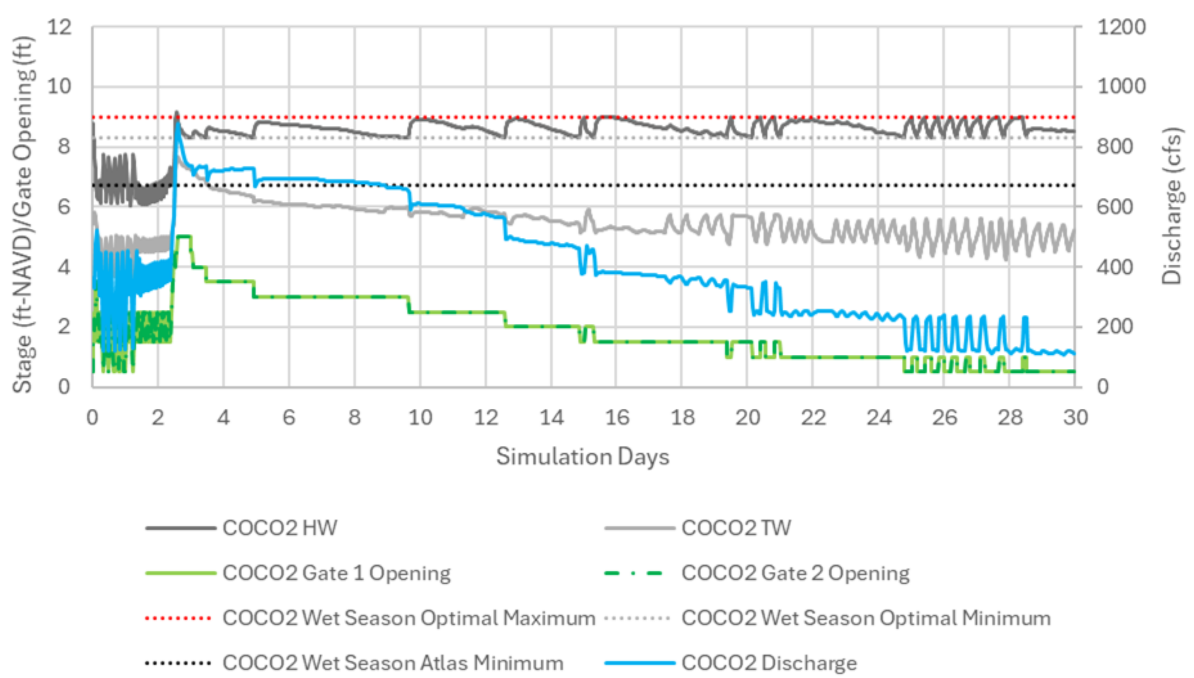

Figure 4-32. 25-year Design Storm Simulated Flow and Stage Hydrographs for COCO2

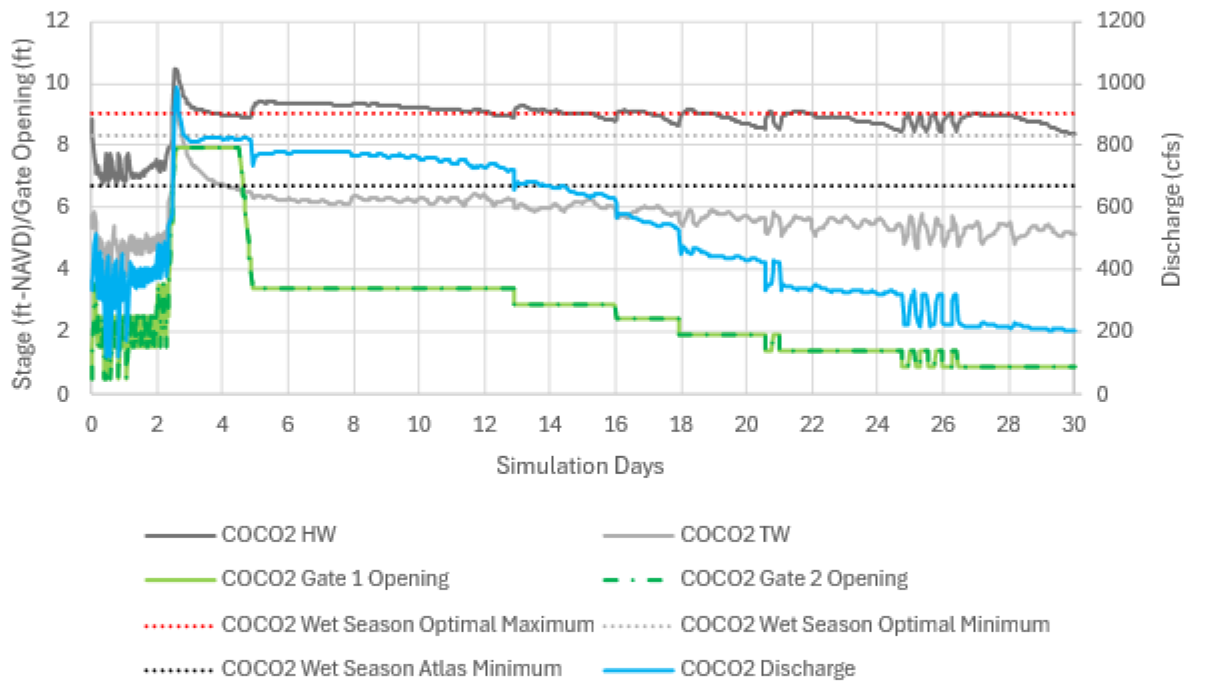

Figure 4-33. 100-year Design Storm Simulated Flow and Stage Hydrographs for COCO2

4.2.2.8 COCO1

The results for the COCO1 structure simulated flow and stage hydrographs with simulated gate levels for each design storm are presented in Figure 4-34 through Figure 4-37. The COCO1 structure simulated gate levels lower multiple times at the beginning of the simulations because the simulated headwater levels drop below the wet season optimal minimum control elevation at the COCO1 structure. A 1-ft gate opening at this structure results in a larger fluctuation in headwater stage and flow than upstream structures, COCO3 and COCO4, due to a larger difference in headwater and tailwater at this structure. The COCO1 tailwater is in the tidal portion of the Cocohatchee Canal.

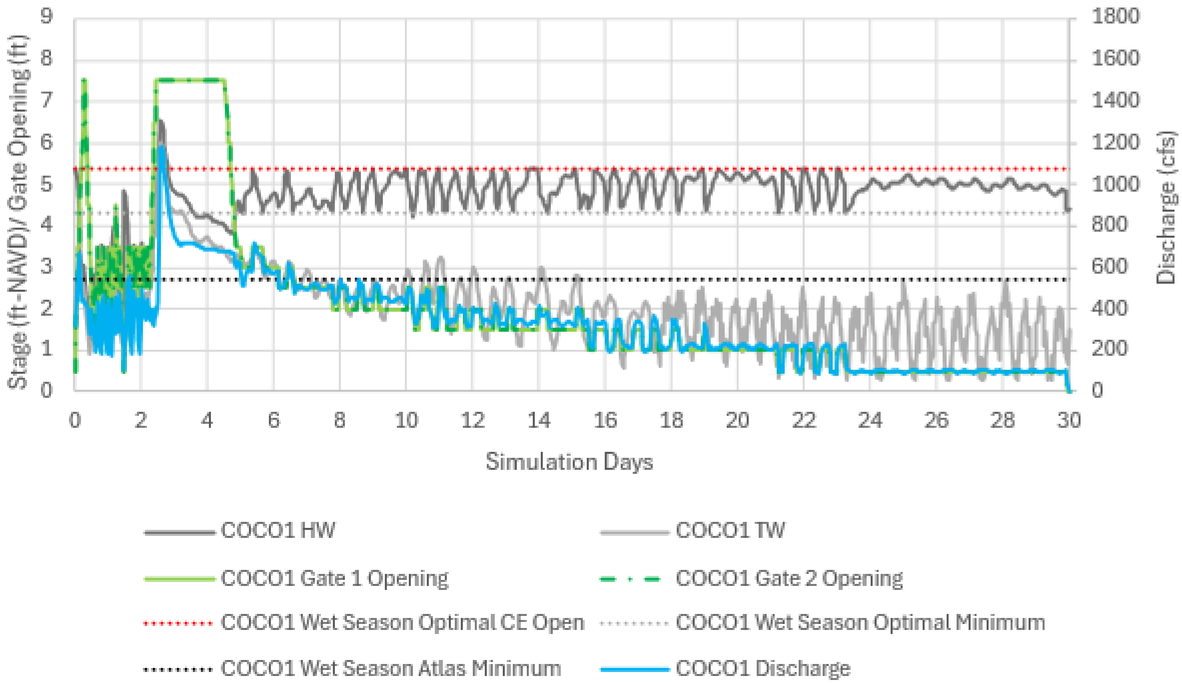

Figure 4-34. 5-year Design Storm Simulated Flow and Stage Hydrographs for COCO1

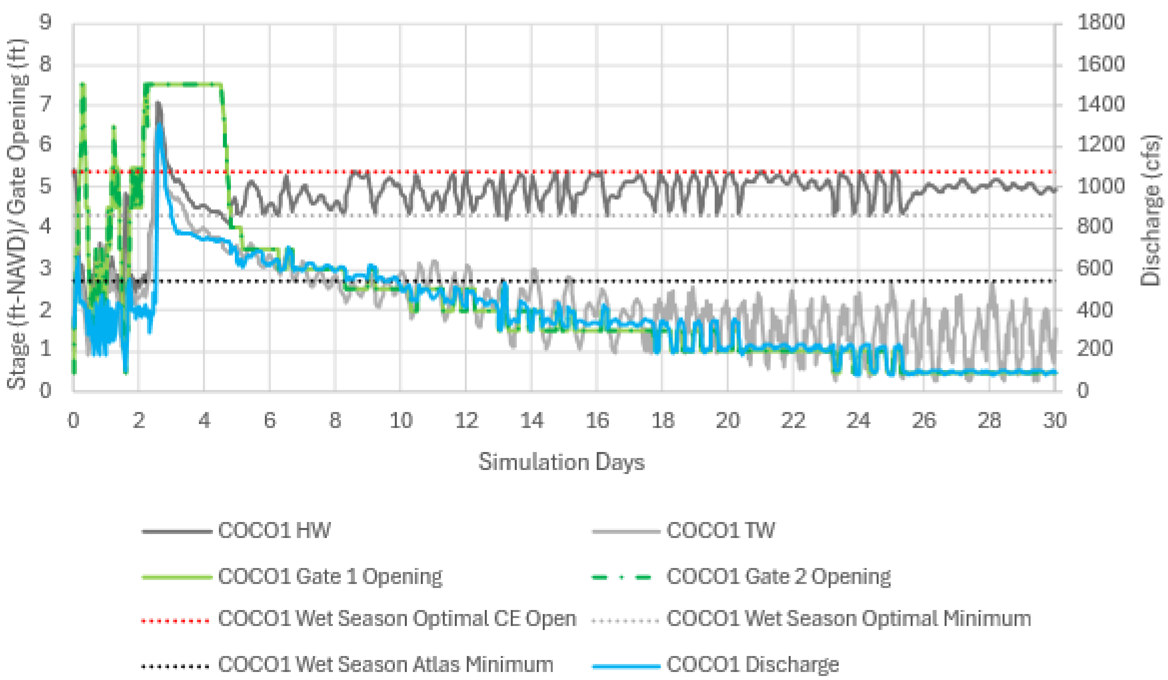

Figure 4-35. 10-year Design Storm Simulated Flow and Stage Hydrographs for COCO1

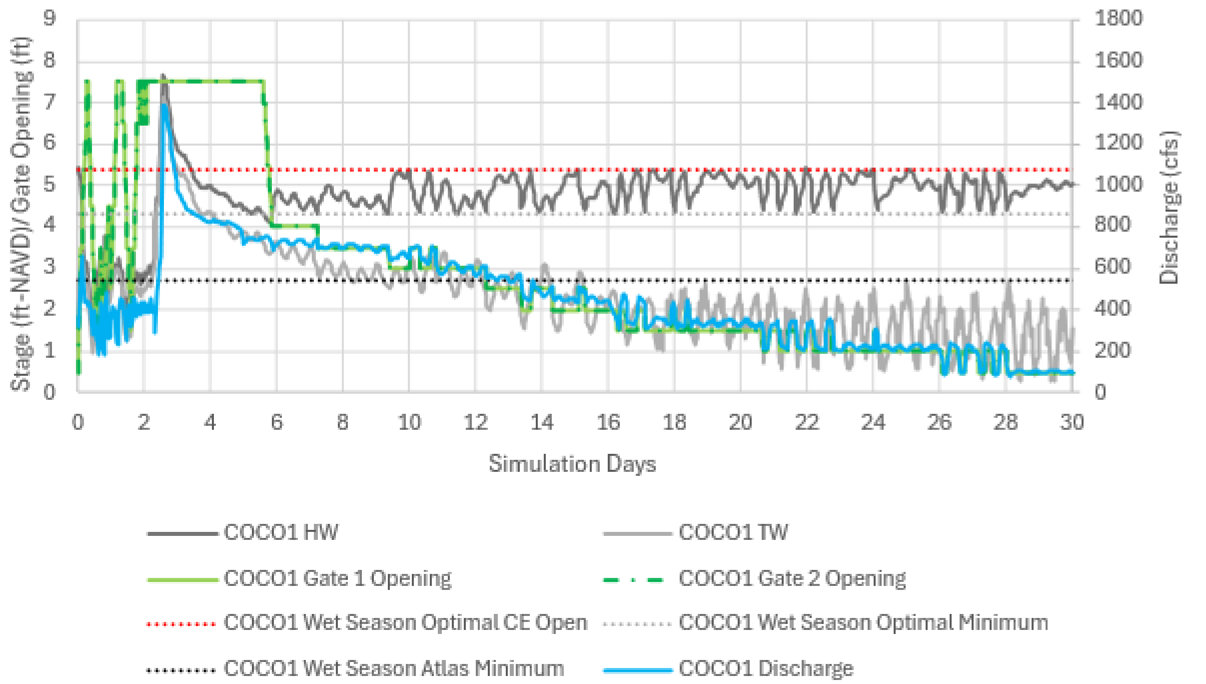

Figure 4-36. 25-year Design Storm Simulated Flow and Stage Hydrographs for COCO1

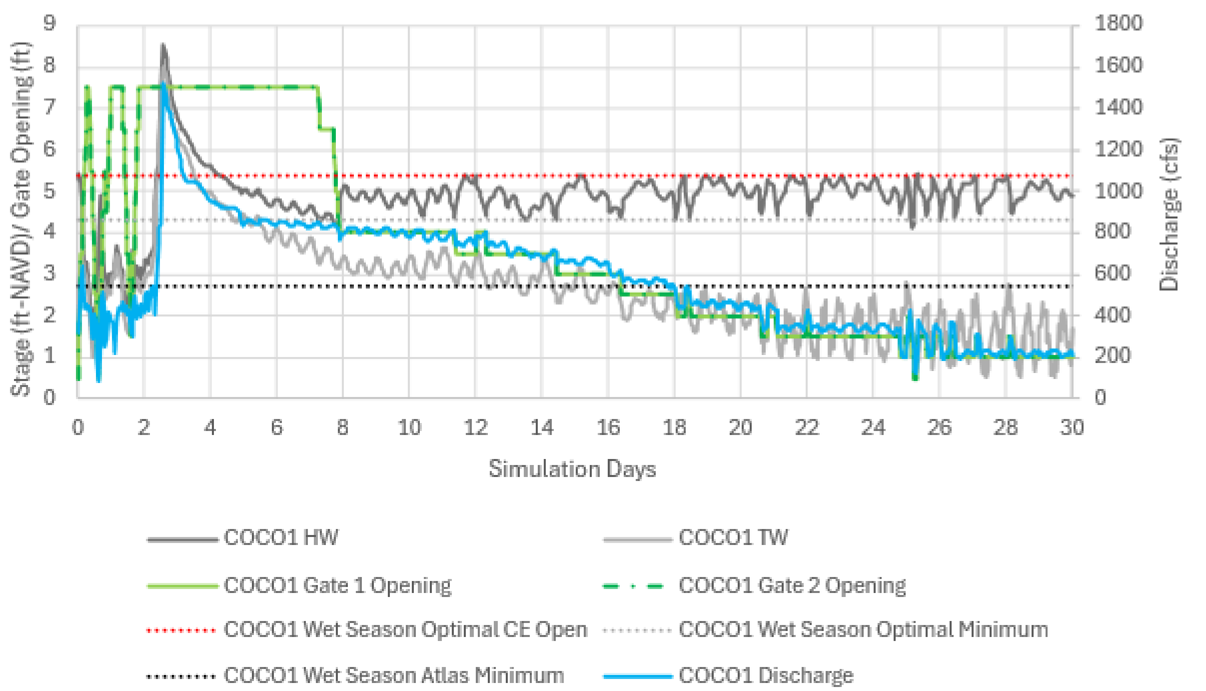

Figure 4-37. 100-year Design Storm Simulated Flow and Stage Hydrographs for COCO1

4.2.2.9 Kehl Canal Weir and Gates

The Kehl Canal Fixed Weir and Gated structure simulated flow and stage hydrographs and gate levels for each design storm are presented in Figure 4-38 through Figure 4-41. The Kehl Canal structure simulated gate levels open fully at the beginning of the simulation and remain fully open until the simulated headwater levels reach the fixed weir elevation of 8.65 ft NAVD, at which time simulated gate levels begin to close. The Kehl Canal Gate 3 does not start to open until Gate 1 and 2 are 1 ft less than fully open, as intended in the model coded rules.

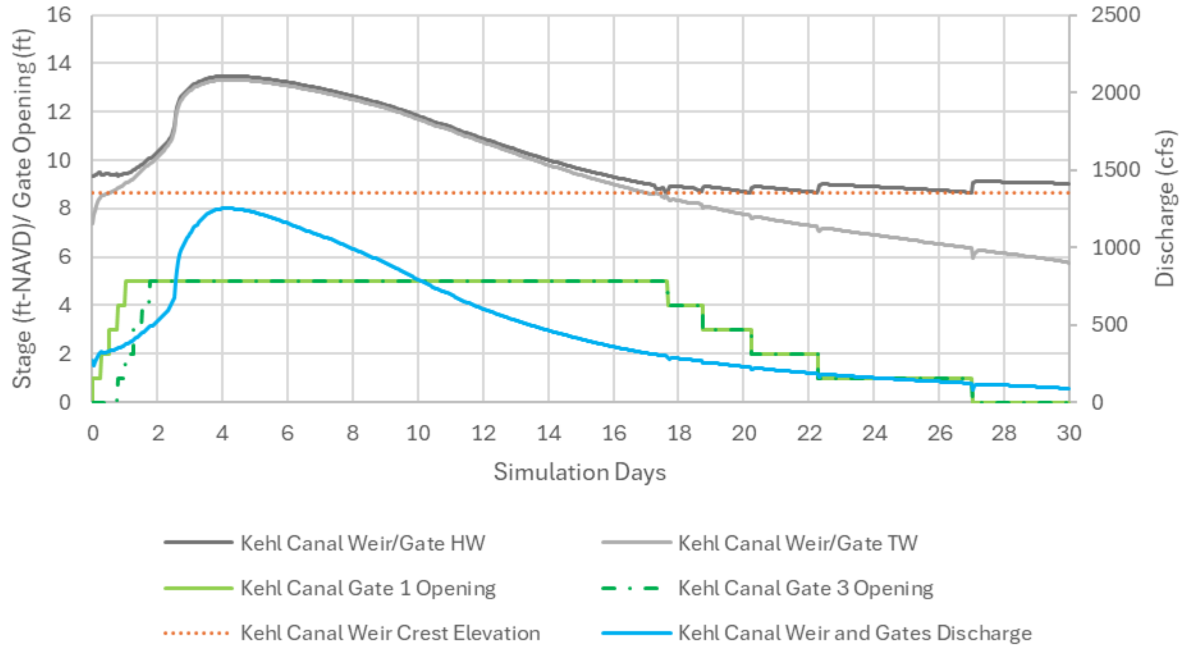

Figure 4-38. 5-year Design Storm Simulated Flow and Stage Hydrographs for Kehl Canal Weir and Gates

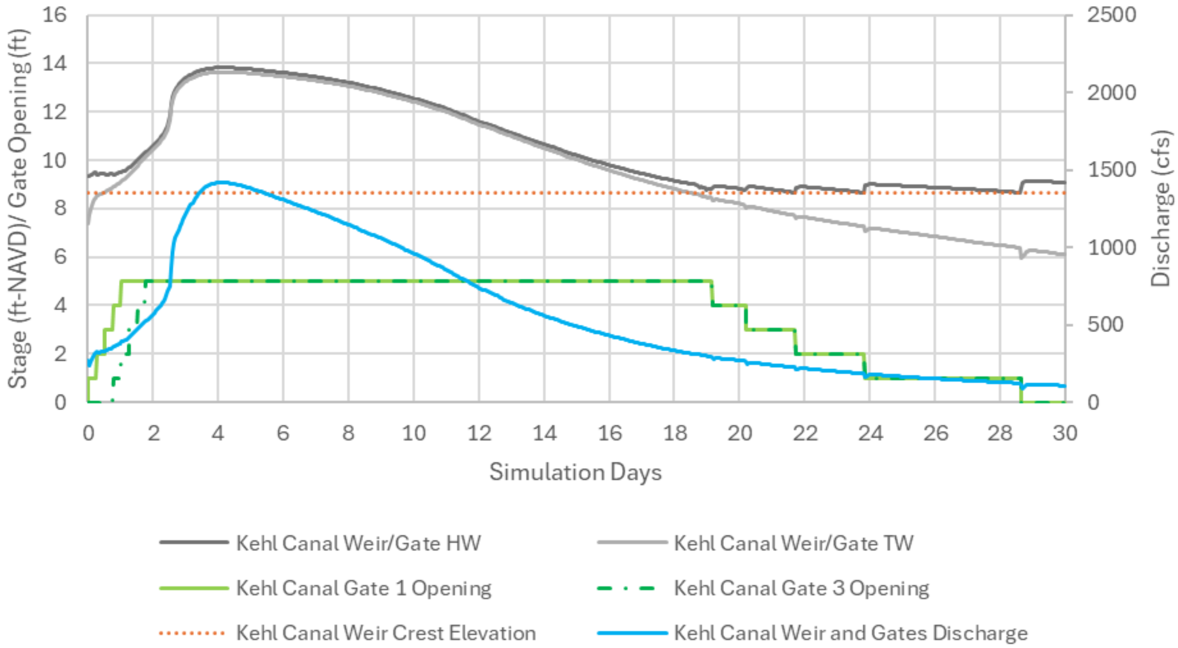

Figure 4-39. 10-year Design Storm Simulated Flow and Stage Hydrographs for Kehl Canal Weir and Gates

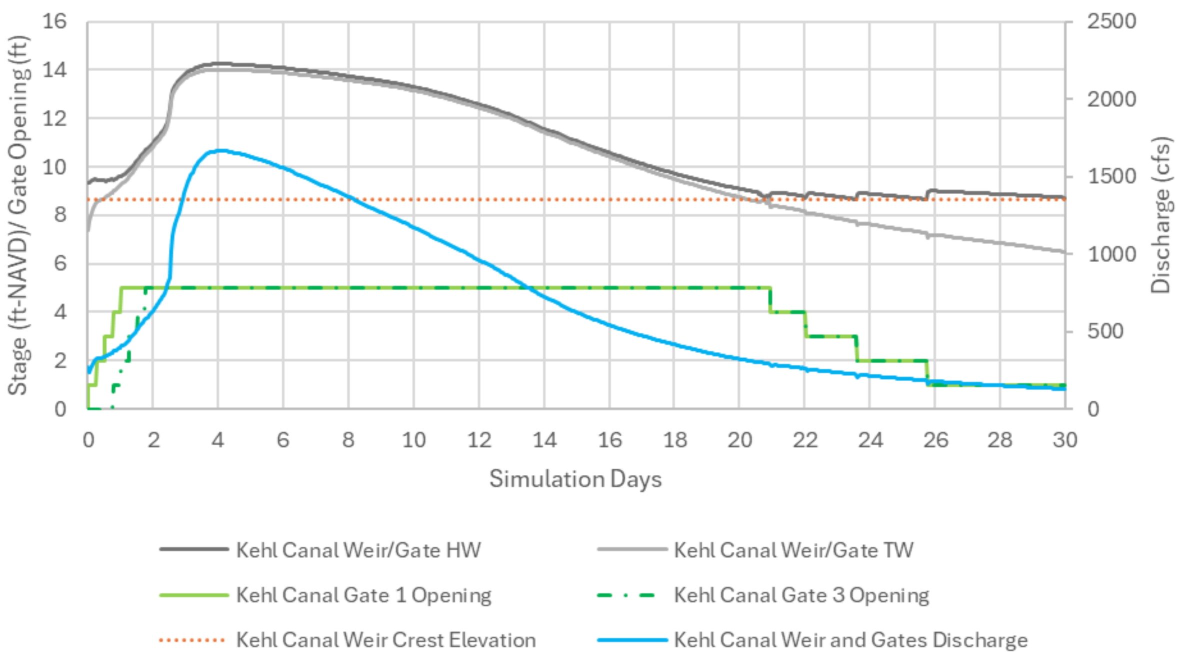

Figure 4-40. 25-year Design Storm Simulated Flow and Stage Hydrographs for Kehl Canal Weir and Gates

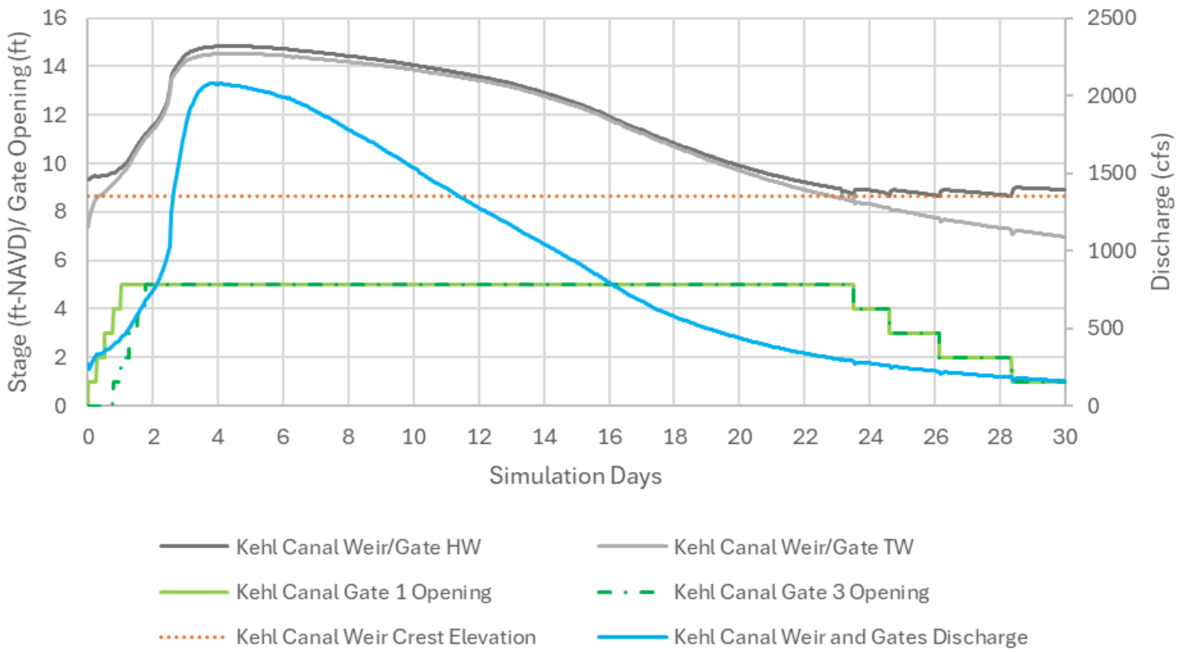

Figure 4-41. 100-year Design Storm Simulated Flow and Stage Hydrographs for Kehl Canal Weir and Gates

4.2.2.10 CR951N

The CR951N structure simulated flow and stage hydrographs and gate levels for each design storm are presented in Figure 4-42 through Figure 4-45. The CR951N structure simulated gate levels lower multiple times at the beginning of the simulations because the simulated headwater levels drop below the wet season optimal minimum control elevation at the CR951N structure. The CR951N structure has additional optimal levels for the tailwater during the wet season as shown in the CR951N control rules in Table 3-10. While the headwater optimal levels take precedence, if the headwater is within the optimal range and the tailwater is below the optimal range, the gates will open to increase the tailwater stage.

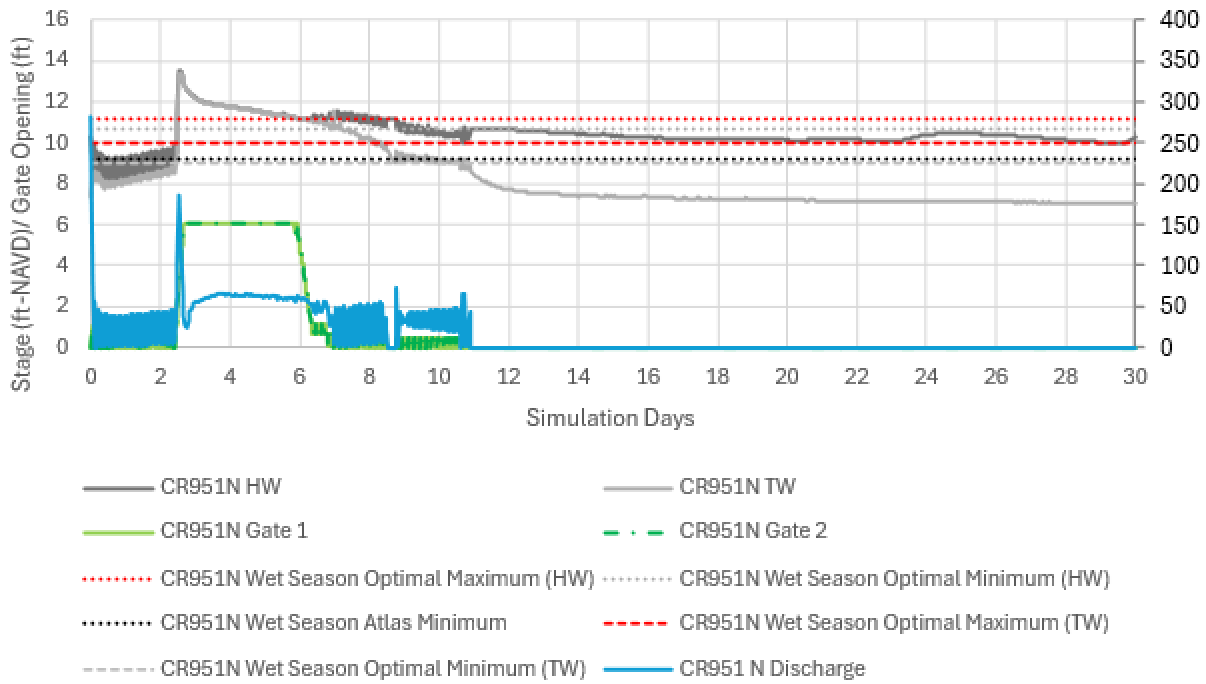

Figure 4-42. 5-year Design Storm Simulated Flow and Stage Hydrographs for CR951N

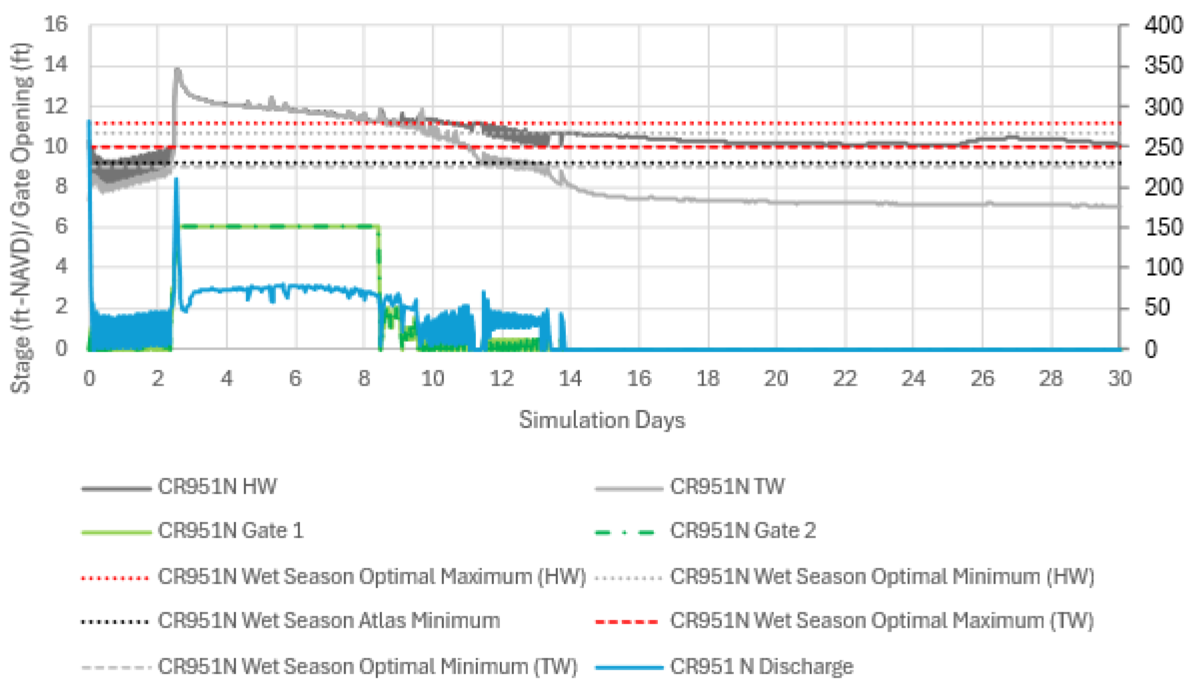

Figure 4-43. 10-year Design Storm Simulated Flow and Stage Hydrographs for CR951N

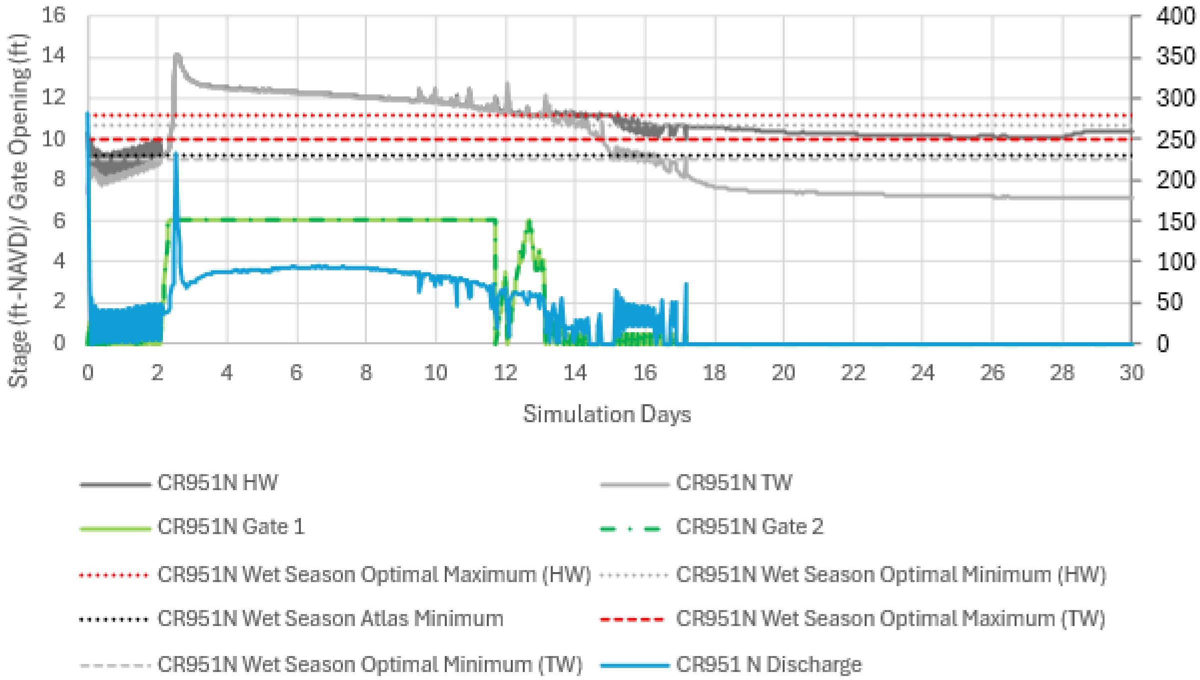

Figure 4-44. 25-year Design Storm Simulated Flow and Stage Hydrographs for CR951N

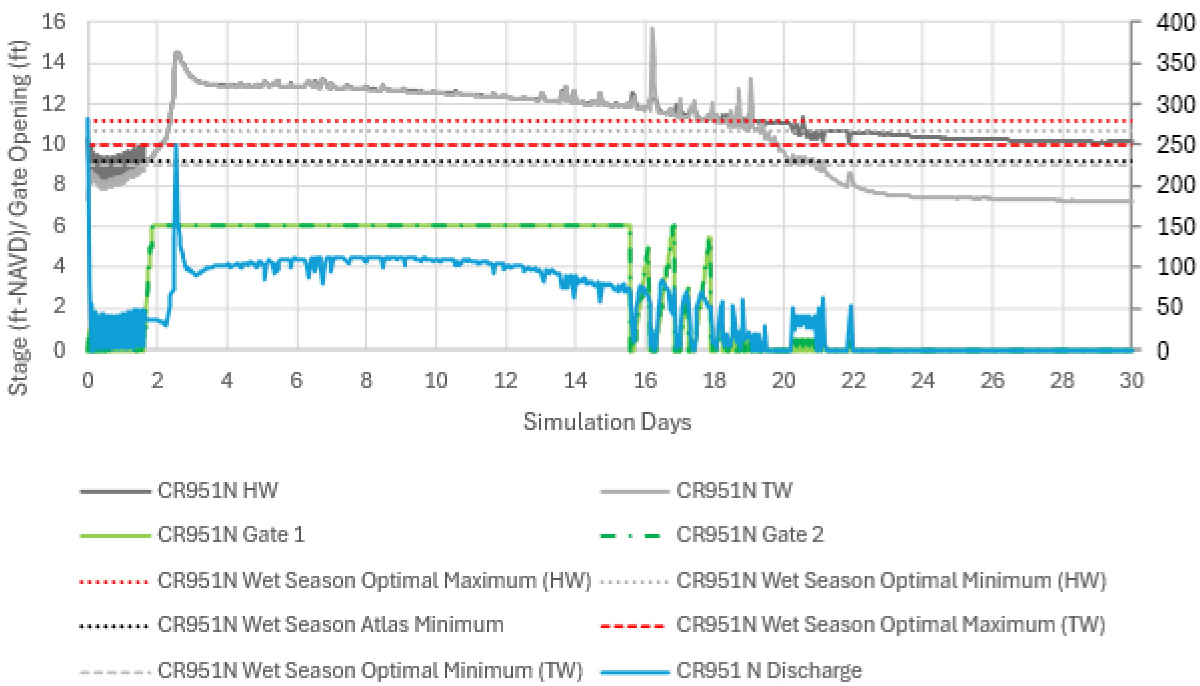

Figure 4-45. 100-year Design Storm Simulated Flow and Stage Hydrographs for CR951N

4.2.2.11 GG1

The GG1 structure simulated flow and stage hydrographs and varying weir levels for each design storm are presented in Figure 4-46 through Figure 4-49. The GG1 structure is a weir overflow structure so simulated weir levels are shown in Figure 4-46 through Figure 4-49 instead of simulated gate opening. The GG1 structure simulated weir levels fluctuate multiple times at the beginning of the simulations because the simulated headwater levels drop below the wet season optimal minimum control elevation at the GG1 structure. During the peak of the storm the weir remains fully open until the simulated headwater levels reach the wet season optimal control elevation, at which time simulated weir levels vary to maintain the headwater within the GG1 wet season optimal range.

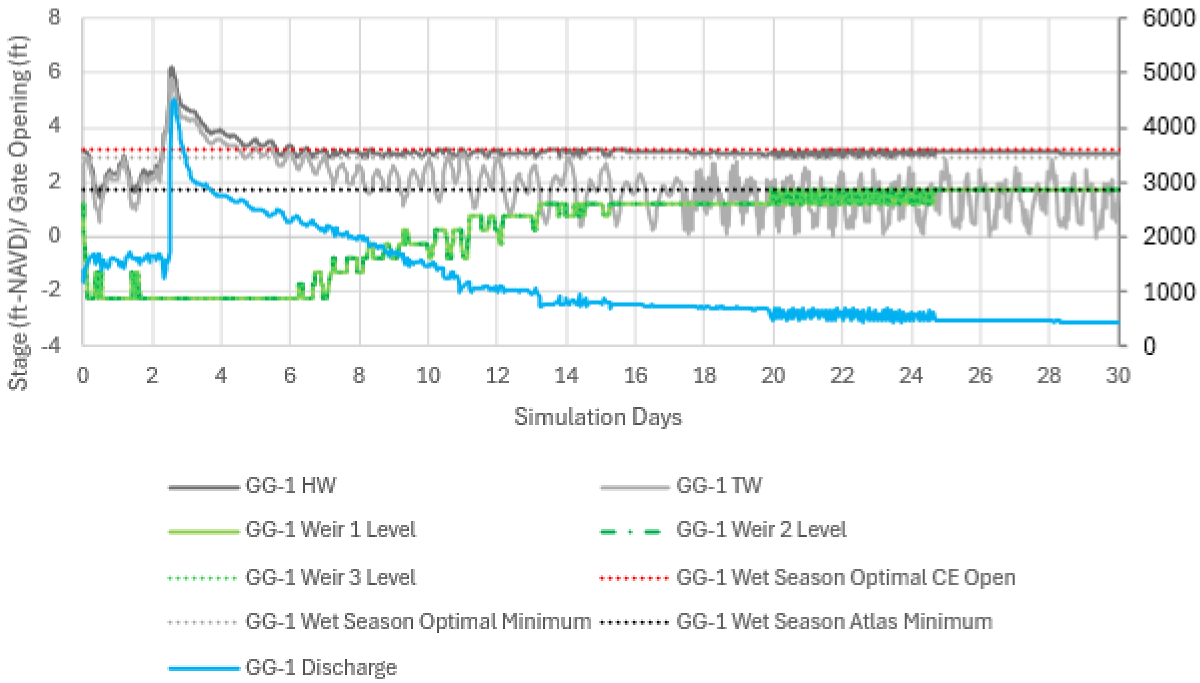

Figure 4-46. 5-year Design Storm Simulated Flow and Stage Hydrographs for GG1

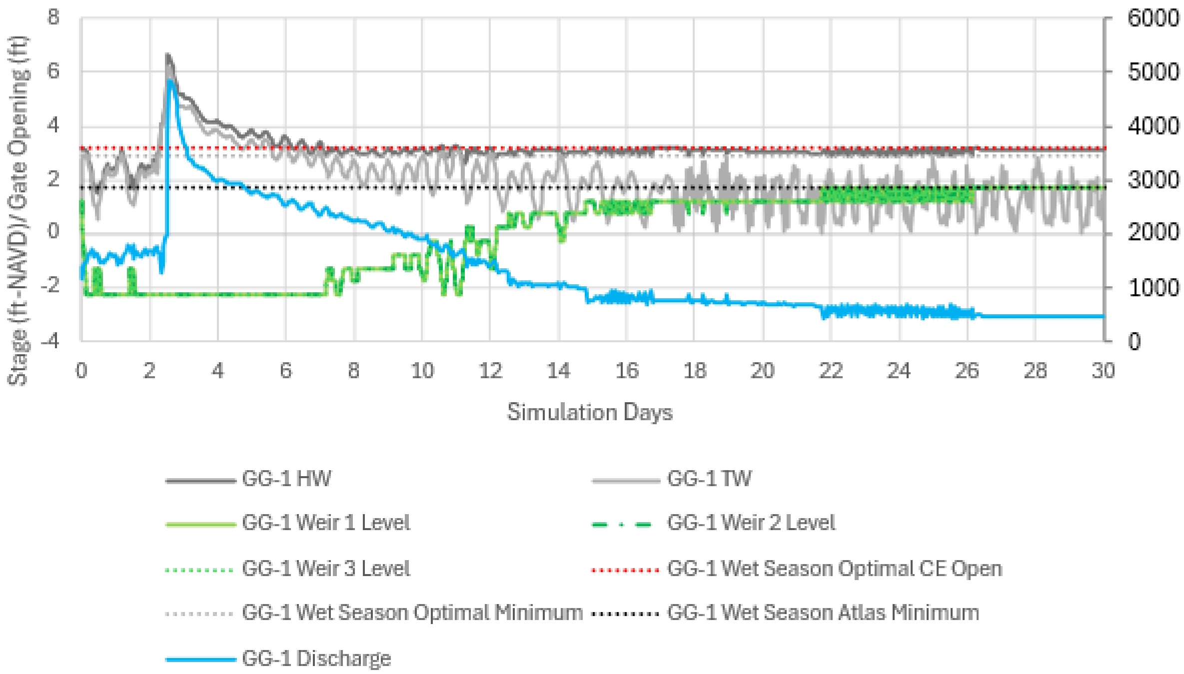

Figure 4-47. 10-year Design Storm Simulated Flow and Stage Hydrographs for GG1

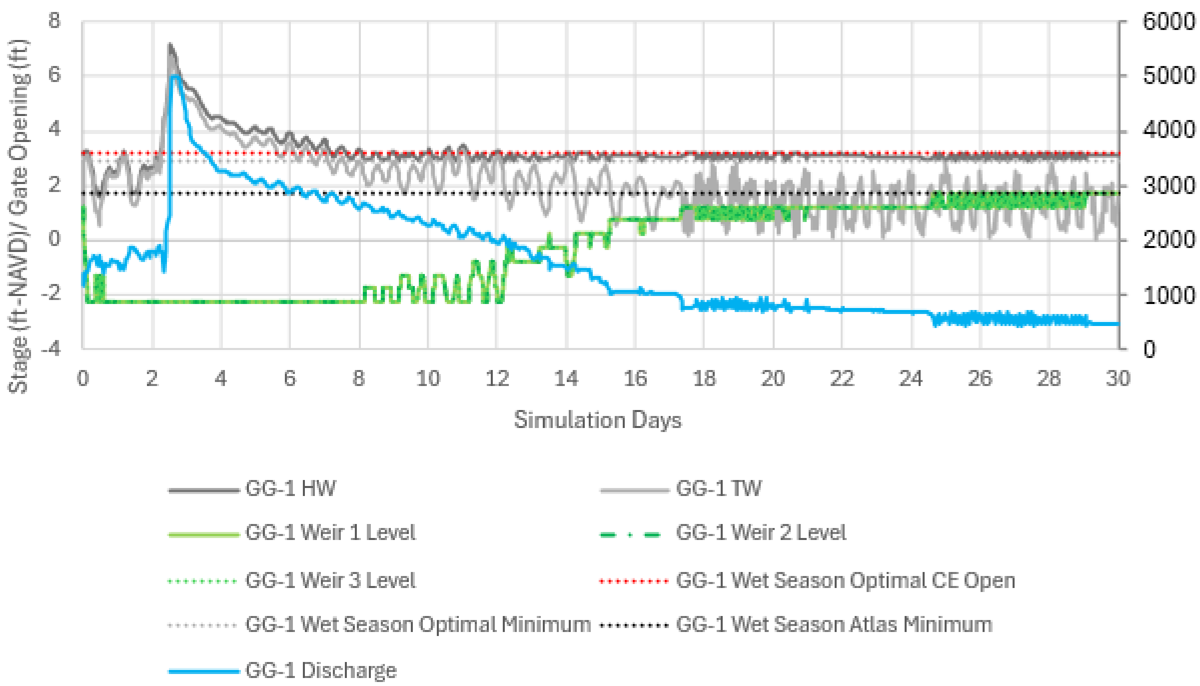

Figure 4-48. 25-year Design Storm Simulated Flow and Stage Hydrographs for GG1

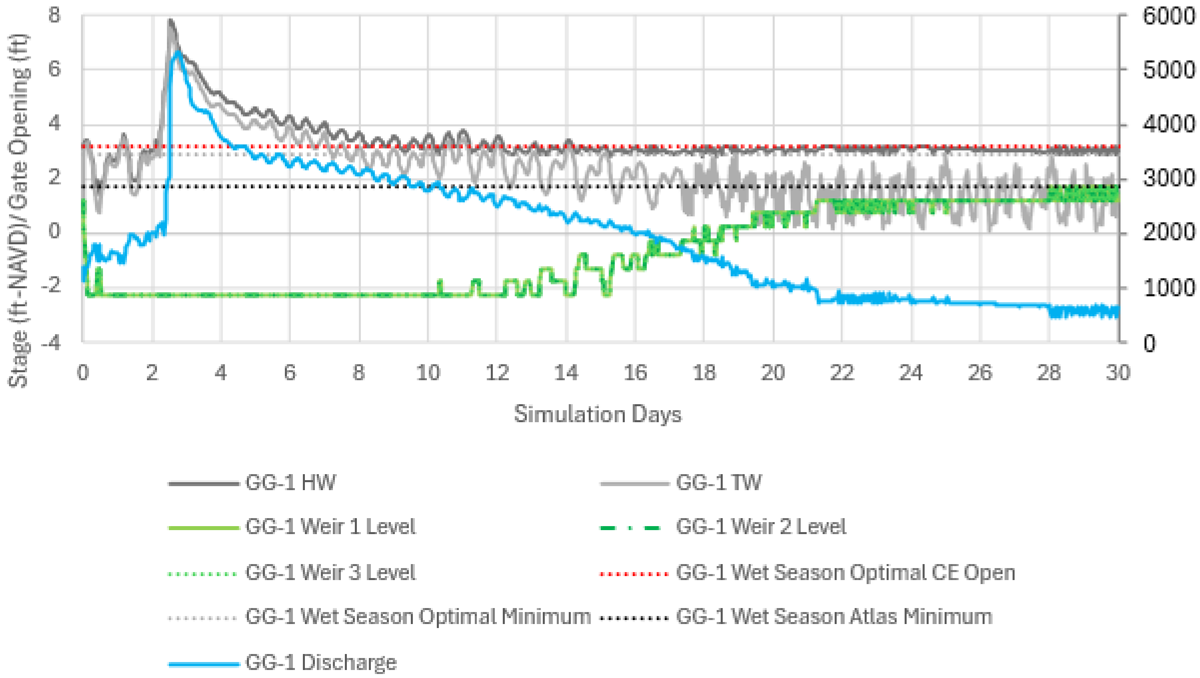

Figure 4-49. 100-year Design Storm Simulated Flow and Stage Hydrographs for GG1.

4.2.2.12 Miller Canal Bridge at I-75

The Miller Canal Bridge at I-75 simulated flow and stage hydrographs for each design storm are presented in Figure 4-50 through Figure 4-53.

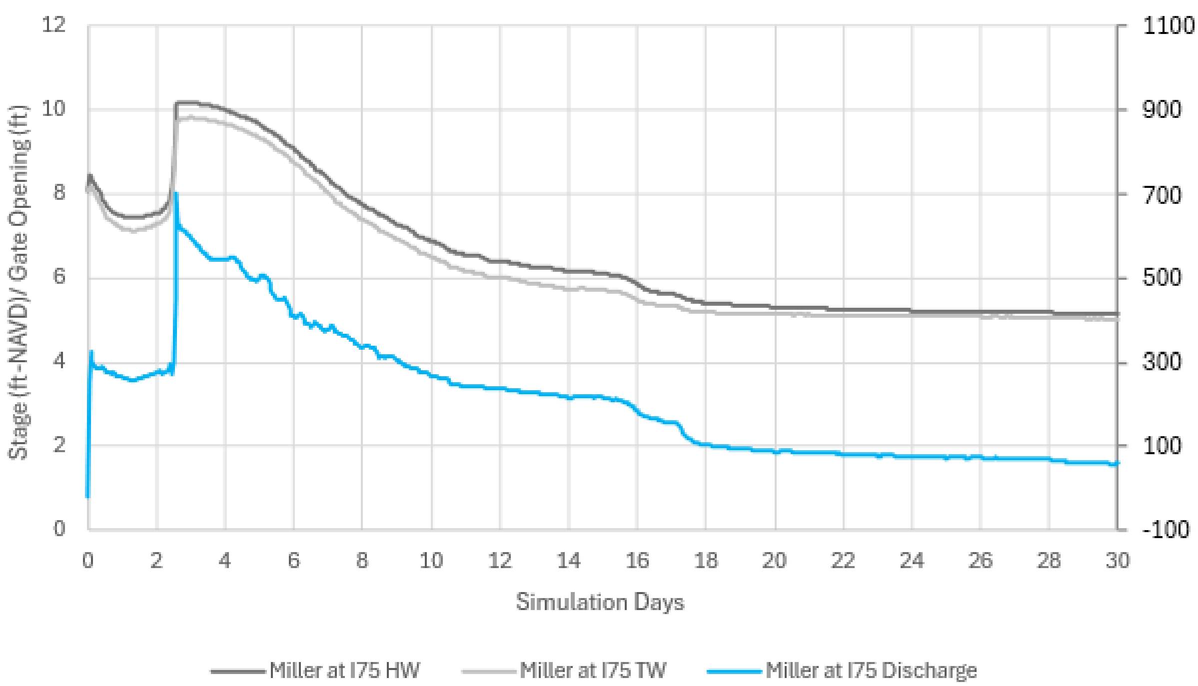

Figure 4-50. 5-year Design Storm Simulated Flow and Stage Hydrographs for the Miller Canal Bridge at I-75.

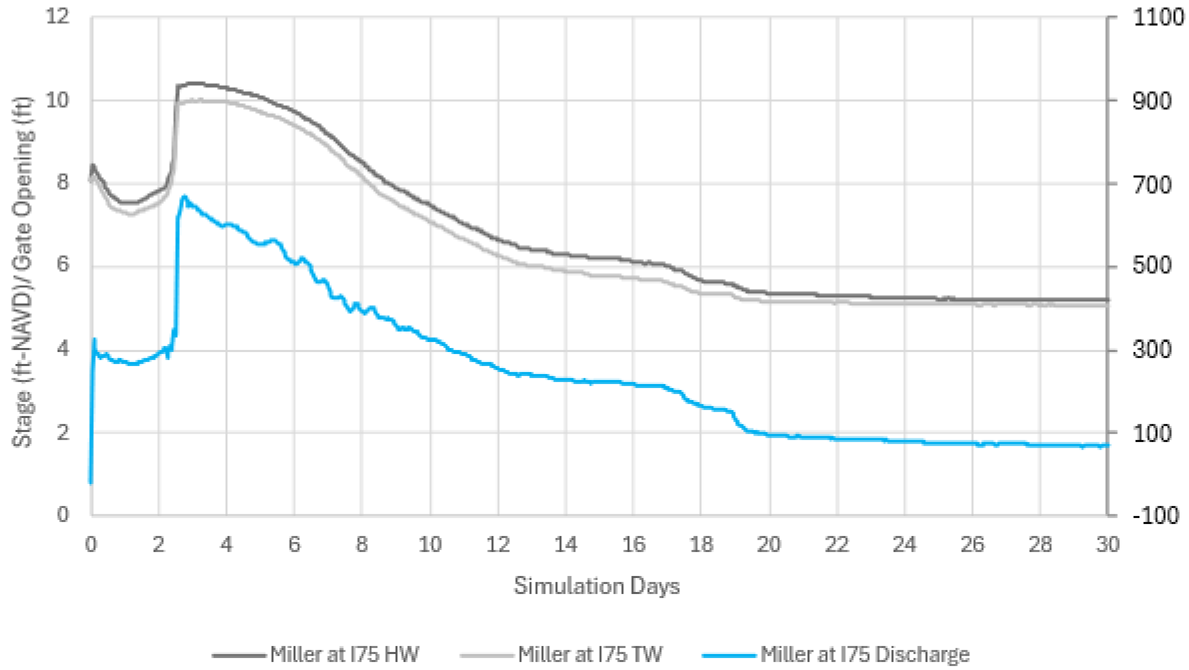

Figure 4-51. 10-year Design Storm Simulated Flow and Stage Hydrographs for the Miller Canal Bridge at I-75.

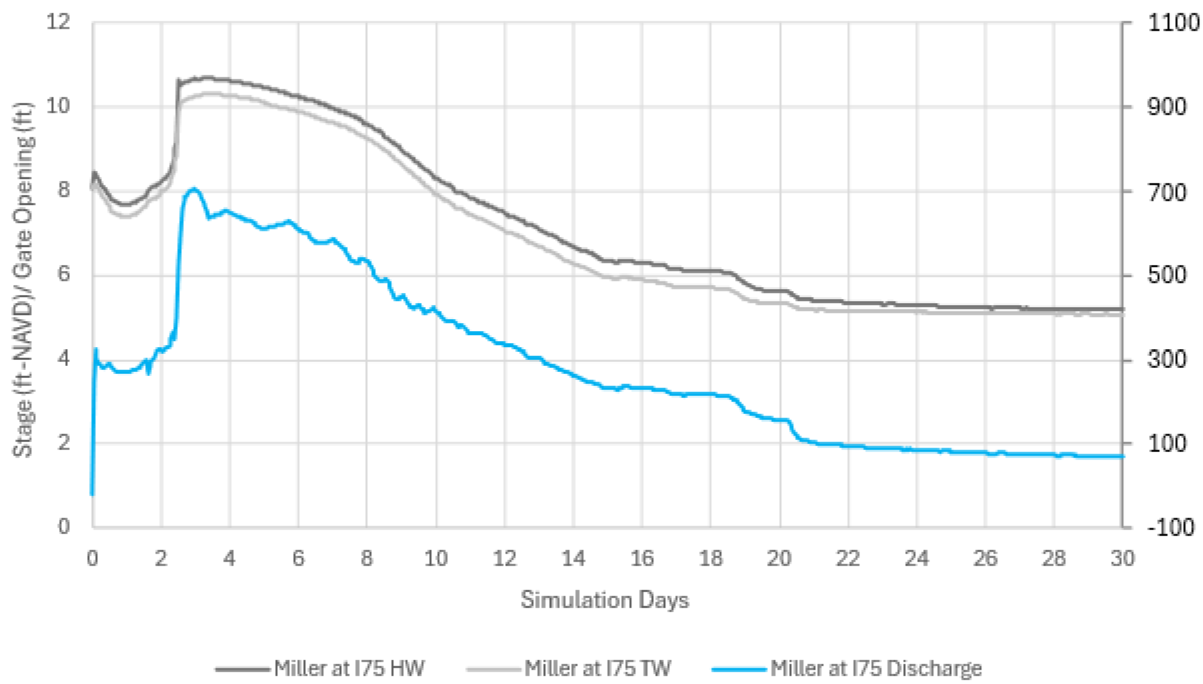

Figure 4-52. 25-year Design Storm Simulated Flow and Stage Hydrographs for the Miller Canal Bridge at I-75.

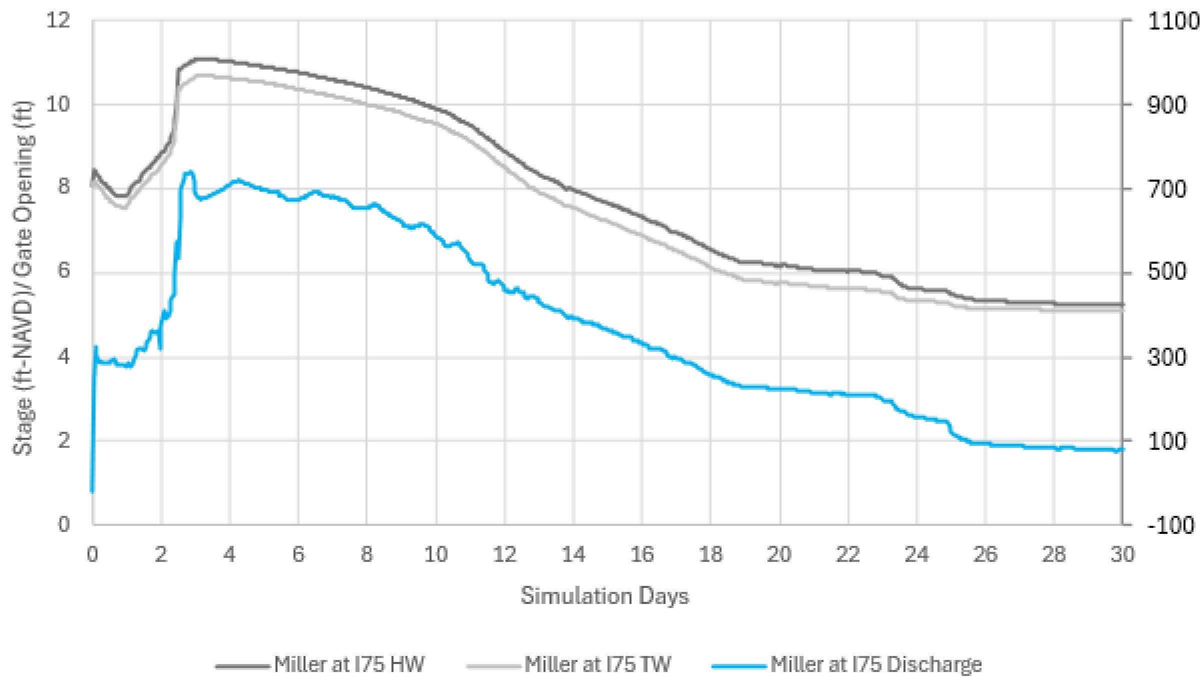

Figure 4-53. 100-year Design Storm Simulated Flow and Stage Hydrographs for the Miller Canal Bridge at I-75.

4.2.2.13 Bird Rookery North Bridge

The Bird Rookery North Bridge simulated flow and stage hydrographs for each design storm are presented in Figure 4-54 through Figure 4-57.

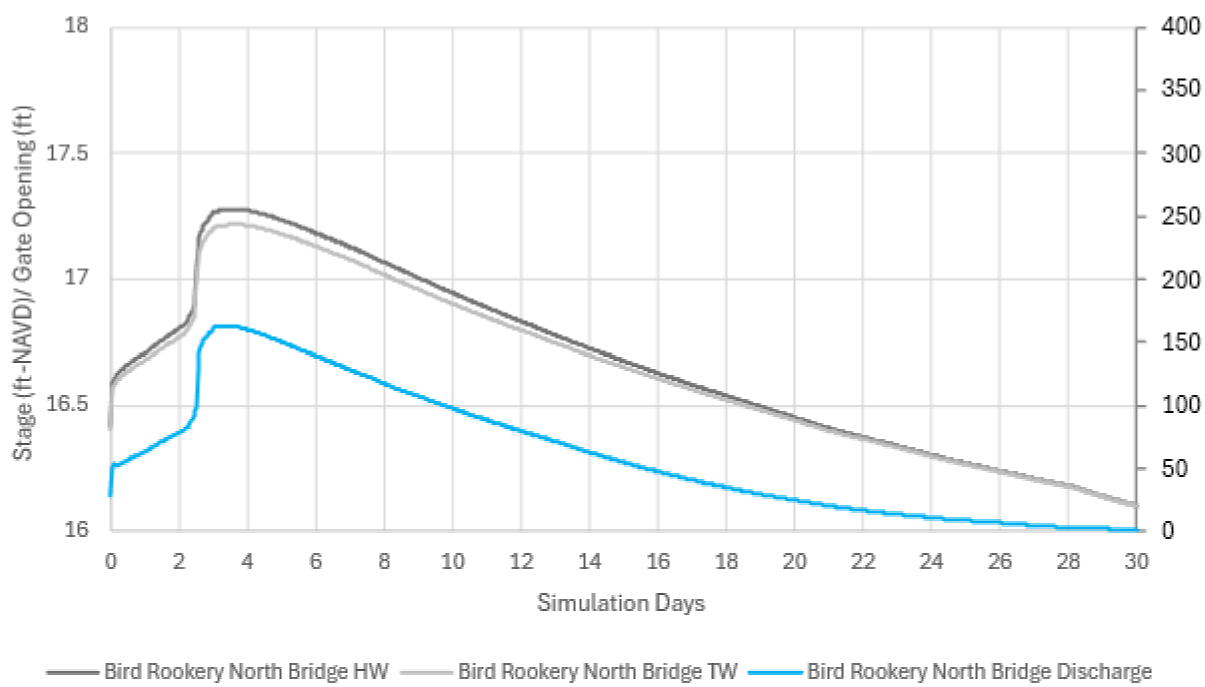

Figure 4-54. 5-year Design Storm Simulated Flow and Stage Hydrographs for the Bird Rookery North Bridge.

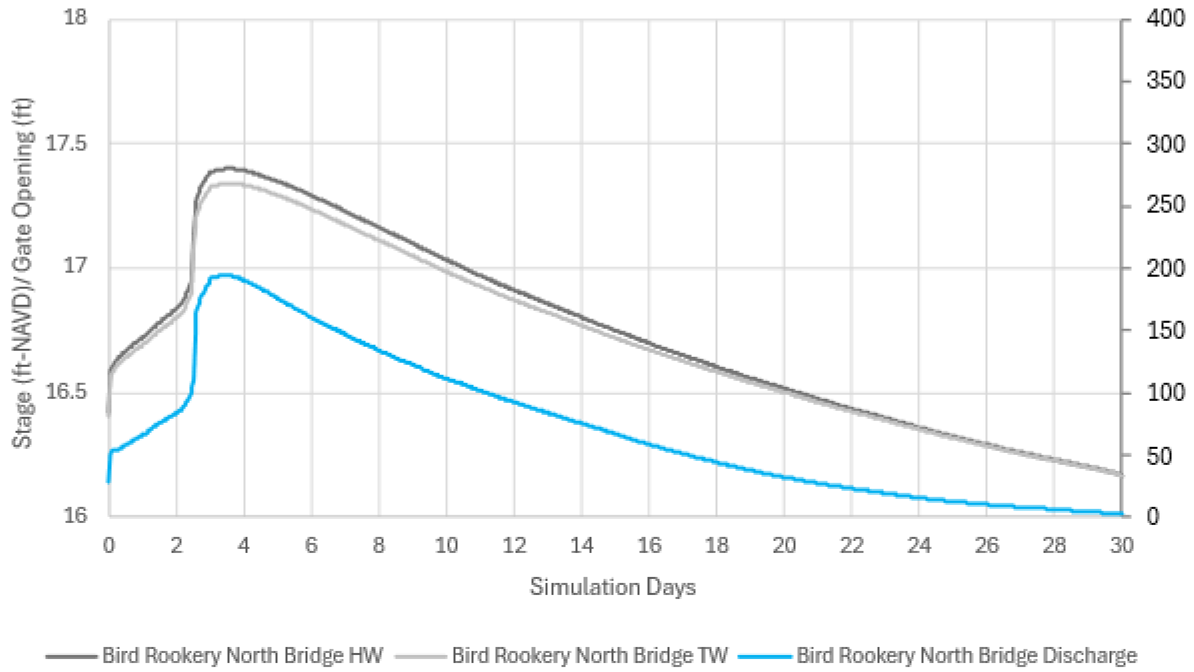

Figure 4-55. 10-year Design Storm Simulated Flow and Stage Hydrographs for the Bird Rookery North Bridge.

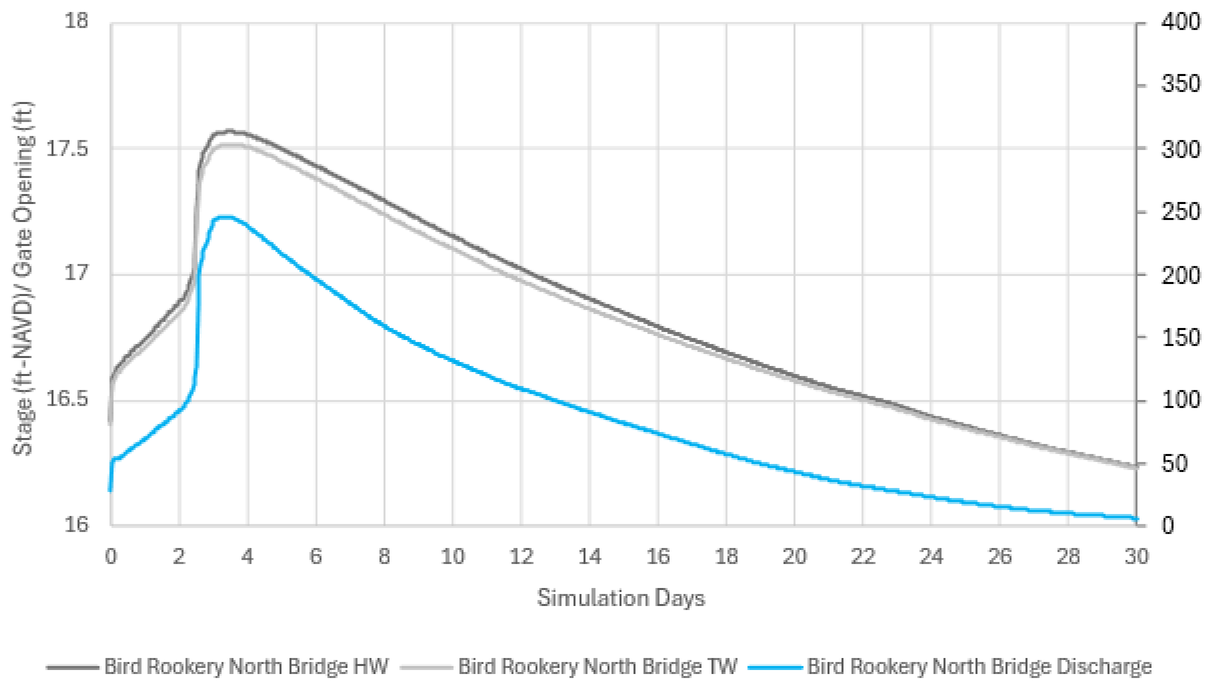

Figure 4-56. 25-year Design Storm Simulated Flow and Stage Hydrographs for the Bird Rookery North Bridge.

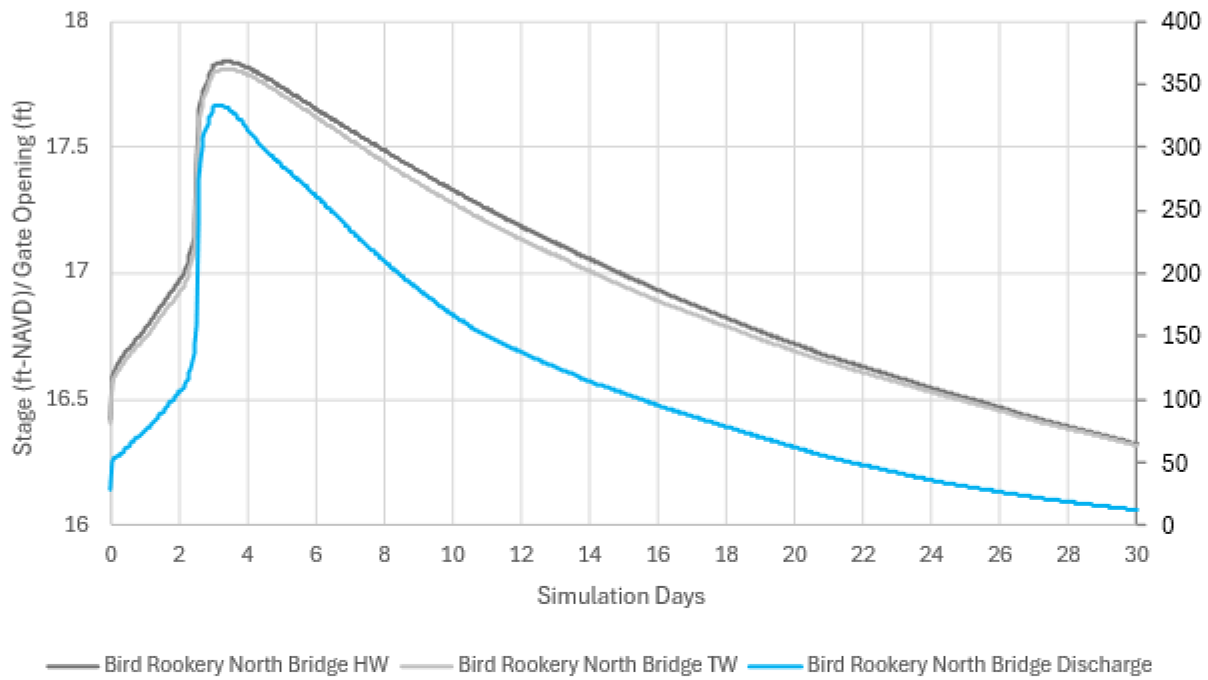

Figure 4-57. 100-year Design Storm Simulated Flow and Stage Hydrographs for the Bird Rookery North Bridge.

4.2.3 Peak Stage and Discharge

The simulated peak flow and stage hydrographs resulting from the ECM design storm simulations are presented below. The results for each design storm are presented by structure. The peak was calculated from a 1 hour results output interval, excluding the first day of the simulation to ensure the reported peak is due to the storm peak and not initial instabilities that may occur.

Table 4-1. Design Storm Simulated Peak Stage and Peak Discharge Summary

Structure	Peak Stage (ft NAVD)				Peak Discharge (cfs)			
	5-yr	10-yr	25-yr	100-yr	5-yr	10-yr	25-yr	100-yr
Cocohatchee Watershed								
COCO1	6.6	7.1	7.7	8.5	1184	1309	1387	1525
COCO2	9.2	9.5	9.7	10.5	686	792	851	988
COCO3	12.3	12.8	13.4	14.4	568	613	658	731
COCO4	12.7	13.1	13.6	14.2	100	119	174	251
AR2 (North)	10.1	10.6	11.2	11.7	274	264	245	239
Golden Gate Main								
AR1 (South)	10.0	10.3	10.7	11.2	898	932	980	1022
GG1	6.2	6.6	7.1	7.9	4532	4841	5004	5313
GG2	8.1	8.5	8.9	9.5	3290	3444	3770	3891
GG3	9.9	10.5	10.9	11.5	2182	2336	2419	2533

Structure	Peak Stage (ft NAVD)				Peak Discharge (cfs)			
	5-yr	10-yr	25-yr	100-yr	5-yr	10-yr	25-yr	100-yr
GG4	11.5	11.9	12.3	12.6	1328	1414	1312	1399
GOLDW5 (GG5)	11.9	12.5	12.9	13.6	1528	1795	2050	2426
GG6	14.9	15.6	16.2	16.8	228	273	370	523
GG7	14.2	14.5	14.8	15.3	13	248	331	440
I75W1	9.0	9.4	9.9	10.5	1805	1927	1875	1773
I75W2	10.7	11.1	11.6	12.2	843	902	968	1079
I75W3	11.4	11.8	12.2	12.6	281	294	314	364
CYP1	11.7	12.0	12.3	12.5	395	404	450	471
CUR1	12.4	12.7	13.1	13.7	262	301	369	490
CR951N	13.5	13.8	14.1	14.5	187	211	233	249
CR951S	10.2	10.7	11.1	11.4	808	842	823	782
CORK1	12.9	13.1	13.5	14.1	651	687	722	711
CORK2	13.9	14.3	14.7	15.3	414	491	561	661
CORK3	16.5	16.6	16.8	17.0	206	213	221	237
Faka Union Watershed								
FU1	4.3	4.6	5.0	5.6	2580	2766	3053	3317
FU3	9.7	10.2	10.6	11.1	1860	2006	2198	2379
FU4S	12.1	12.1	12.9	13.1	1636	1704	1778	1828
FU5	15.1	15.0	15.5	16.0	1217	1313	1395	1549
FU6	16.2	16.5	17.1	17.6	989	1222	1532	1777
FU7	18.3	19.0	19.4	20.1	469	609	853	1134
MILLER3	11.6	12.0	12.4	12.8	178	212	337	482
Henderson Creek								
HC1	5.3	5.6	5.9	6.3	614	646	696	783
HC2	9.3	9.5	9.8	10.1	296	298	303	316
Estero Bay								
Kehl Canal Weir and Gate	13.5	13.8	14.2	14.8	1253	1418	1667	2082

4.2.4 Peak Flood Depth

The peak flood depths for the design storm simulations are presented in this section. The peak flood depths were calculated based on the simulated maximum overland depth. The results are presented by design storm across the study area in tables and figures. Table 4-2 through Table 4-5 identify the simulated area of inundation in urban and agricultural land use areas for the peak flood depths for the basins, i.e., the SFWMD AHED Watershed near the project area. Figure 4-58 through Figure 4-61 provide a visual depiction of the simulated peak flood depths in urban and agricultural land use areas. Note that the figures zoom into the project area but the total flooded areas were added for the entire basins.

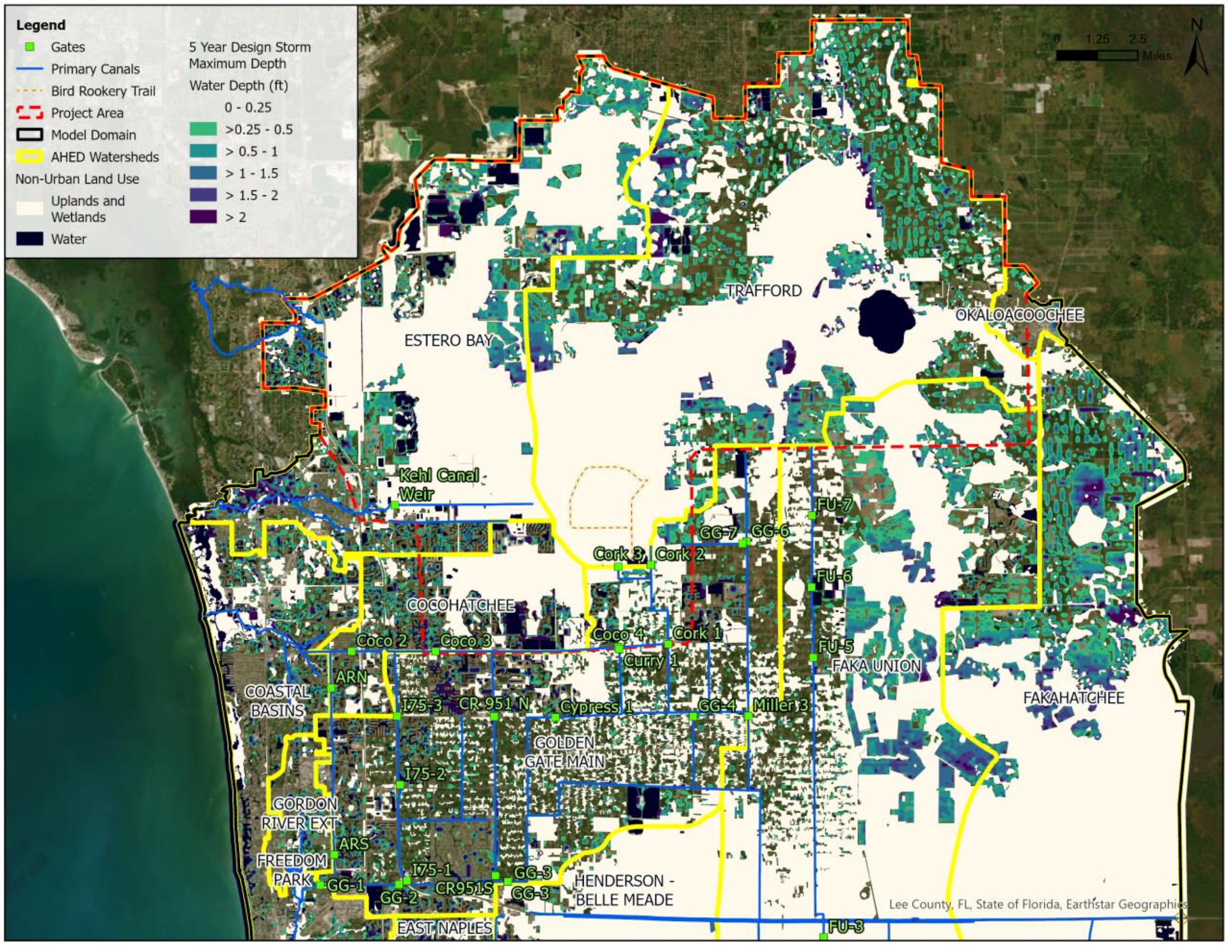

Figure 4-58. Five-Year Design Storm Peak Flood Depths for Urban and Agricultural Land Uses

Table 4-2. Five-Year Design Storm Urban and Agricultural Flooded Areas per Basin

Depth (ft)	5-yr Design Storm Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	COCOHATCHEE	FAKA UNION	GOLDEN GATE MAIN	TRAFFORD
0.25 - 0.5 ft	22	3	39	24	51
0.5 - 1 ft	37	4	39	10	47
1 - 1.5 ft	19	2	28	4	24
1.5 - 2 ft	7	2	11	4	13
≥ 2 ft	15	6	9	16	17

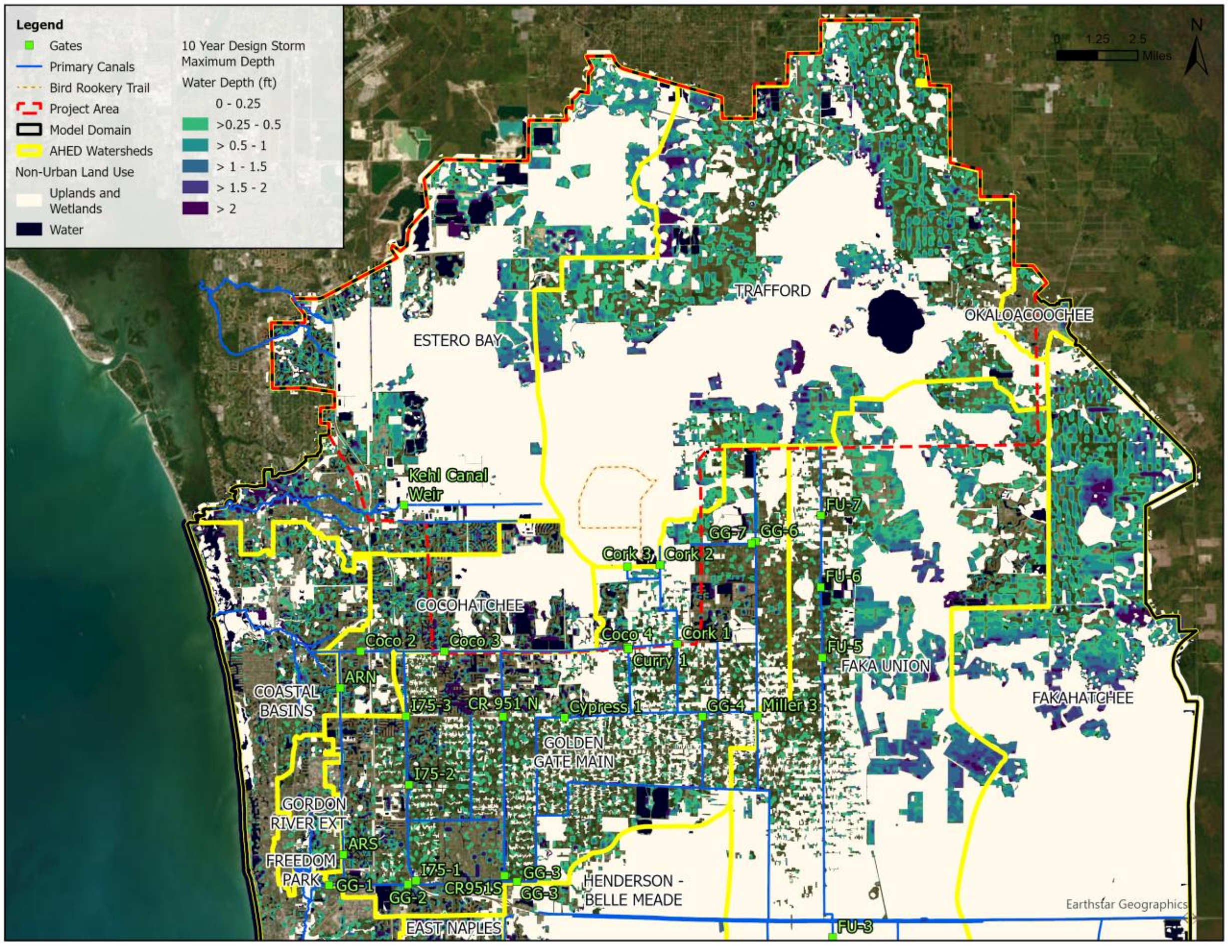

Figure 4-59. Ten-Year Design Storm Peak Flood Depths for Urban and Agricultural Land Uses

Table 4-3. Ten-Year Design Storm Urban and Agricultural Flooded Areas per Basin

Depth (ft)	10-yr Design Storm				
	Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	COCOHATCHEE	FAKA UNION	GOLDEN GATE MAIN	TRAFFORD
0.25 - 0.5 ft	25	6	49	53	67
0.5 - 1 ft	38	4	43	16	53
1 - 1.5 ft	23	3	31	4	29
1.5 - 2 ft	10	1	16	4	16
≥ 2 ft	16	7	11	17	20

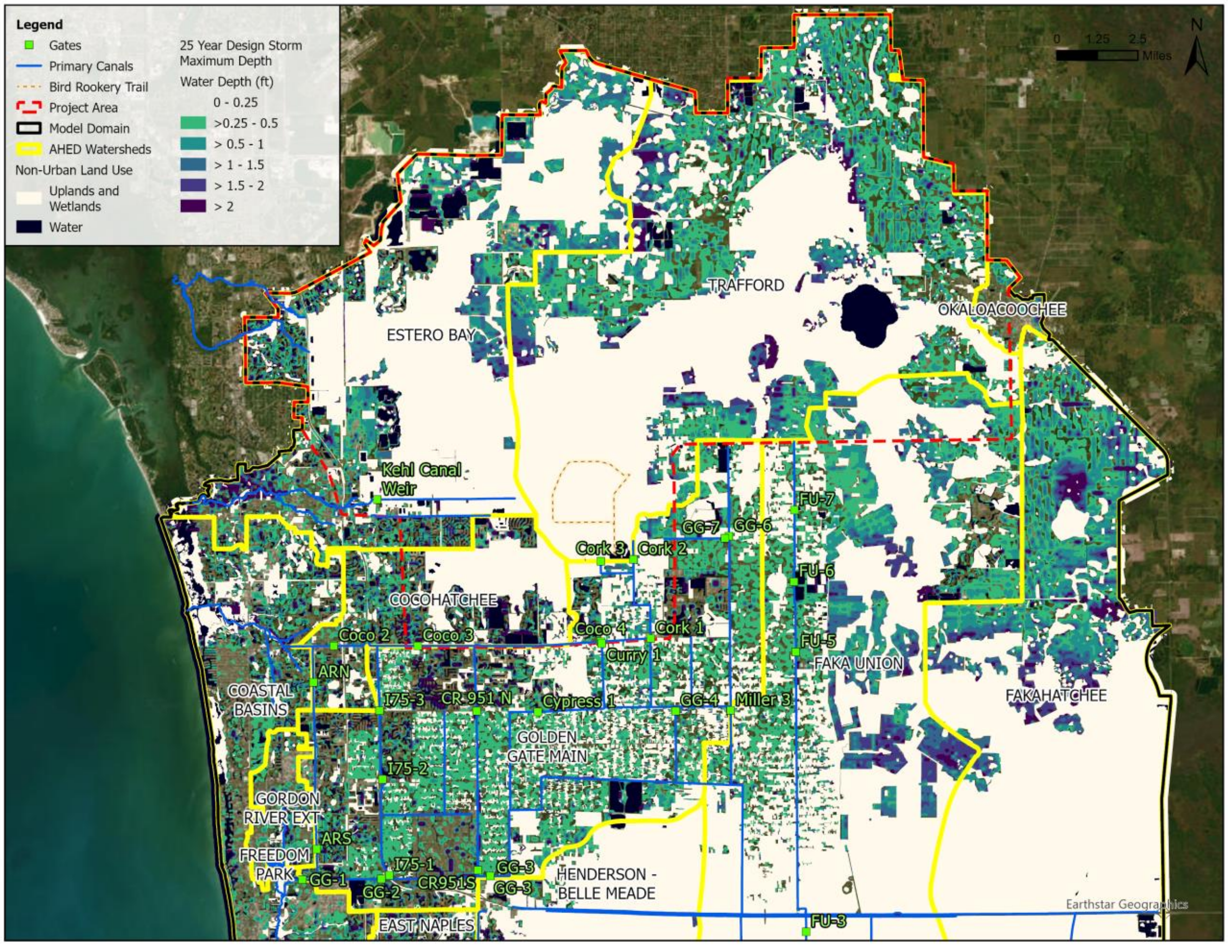

Figure 4-60. Twenty Five-Year Design Storm Peak Flood Depths for Urban and Agricultural Land Uses

Table 4-4. Twenty Five-Year Design Storm Urban and Agricultural Flooded Areas per Basin

Depth (ft)	25-yr Design Storm				
	Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	COCOHATCHEE	FAKA UNION	GOLDEN GATE MAIN	TRAFFORD
0.25 - 0.5 ft	35	13	82	146	97
0.5 - 1 ft	36	4	48	27	62
1 - 1.5 ft	30	3	32	9	35
1.5 - 2 ft	12	2	24	4	20
≥ 2 ft	19	8	14	19	26

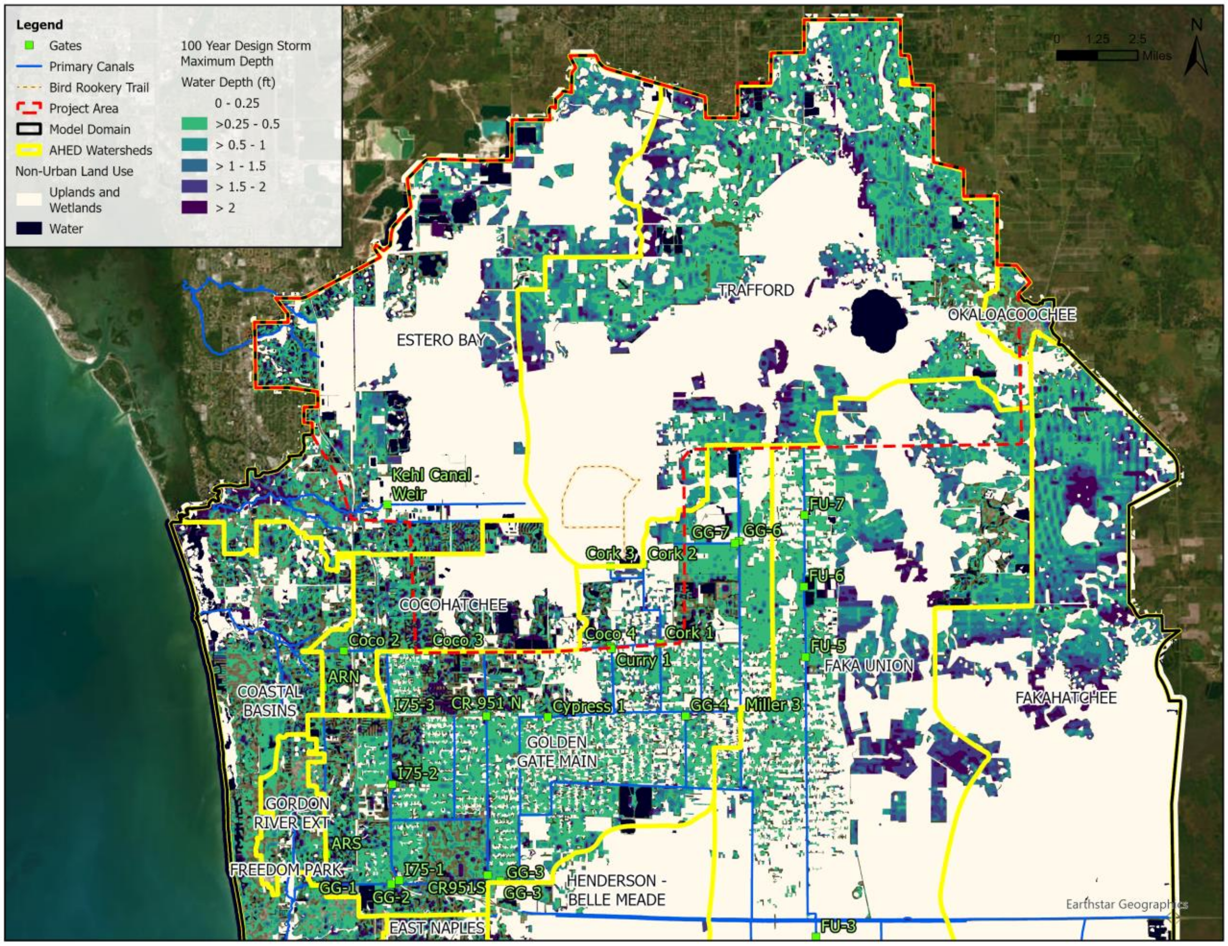

Figure 4-61. Hundred-Year Design Storm Peak Flood Depths for Urban and Agricultural Land Uses

Table 4-5. Hundred-Year Design Storm Urban and Agricultural Flooded Areas per Basin

Depth (ft)	100-yr Design Storm Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	**COCOHATCHEE**	**FAKA UNION**	**GOLDEN GATE MAIN**	**TRAFFORD**
0.25 - 0.5 ft	46	23	93	198	121
0.5 - 1 ft	33	6	60	57	75
1 - 1.5 ft	34	2	37	17	44
1.5 - 2 ft	19	3	28	8	28
≥ 2 ft	26	10	28	23	37

4.2.5 Flood Duration

The flood duration for the design storm simulations are presented in this section. The results are presented by design storm across the study area in tables and figures.

Table 4-6 through Table 4-9 identify the area of inundation in urban and agricultural land use areas based on the flood duration. Figure 4-62 through Figure 4-65 provide a visual depiction of the simulated flood duration in urban and agricultural land use areas.

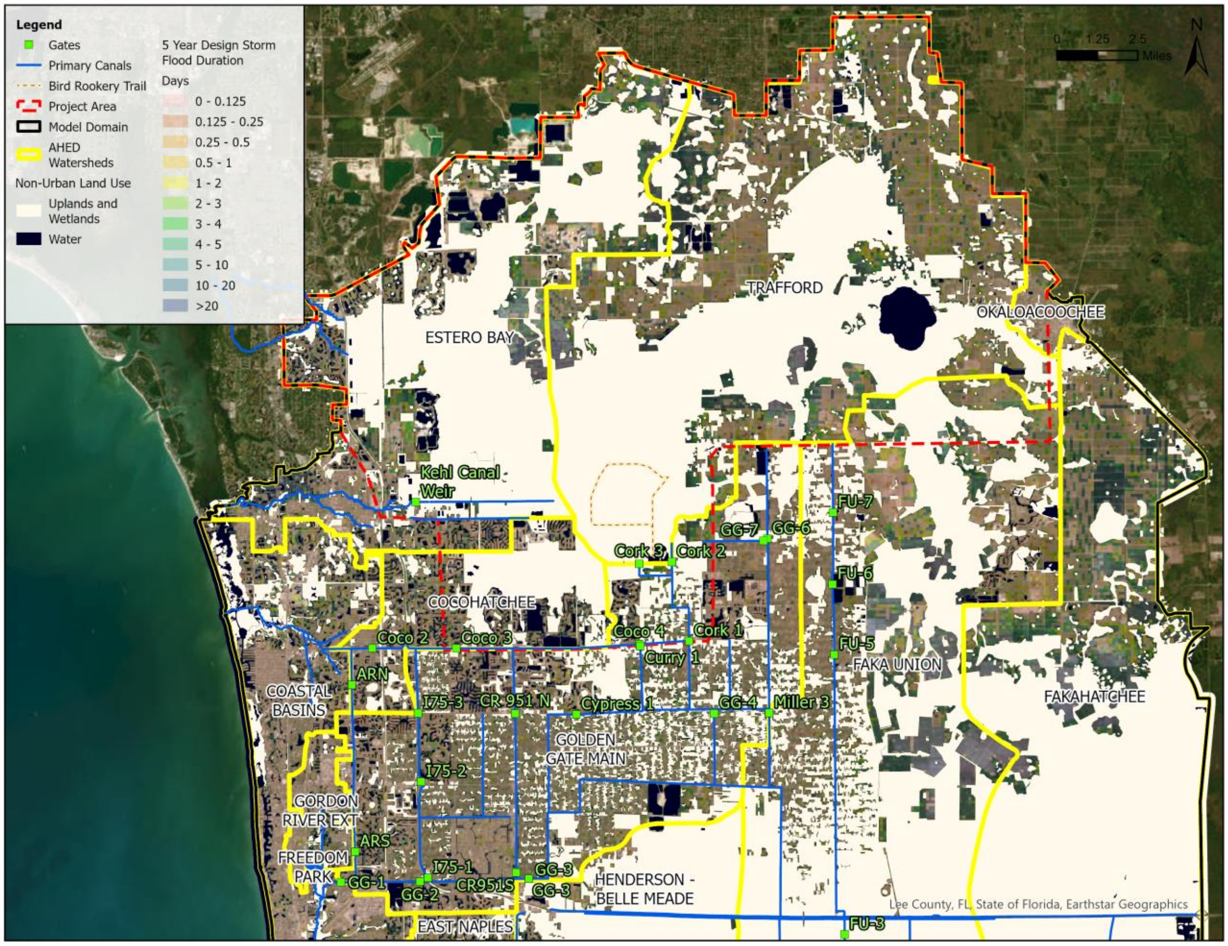

Figure 4-62. Five-Year Design Storm Flood Duration (Days) for Urban and Agricultural Land Uses

Table 4-6. Five-Year Design Storm Flood Duration Areas for Urban and Agricultural Land Uses

	Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
Duration (Days)	**ESTERO BAY**	**COCOHATCHEE**	**FAKA UNION**	**GOLDEN GATE MAIN**	**TRAFFORD**
0 - 0.125	2	0	7	8	11
0.125 - 0.25	2	0	5	4	6
0.25 - 0.5	2	1	5	3	9
0.5 - 1	5	1	8	5	12
1 - 2	4	1	8	5	13
2 - 3	4	0	5	4	8
3 - 4	4	1	3	2	6
4 - 5	4	0	4	2	5

Duration (Days)	Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	COCOHATCHEE	FAKA UNION	GOLDEN GATE MAIN	TRAFFORD
5 - 10	12	1	13	3	14
10 - 20	18	2	18	2	17
≥ 20	43	9	51	19	53

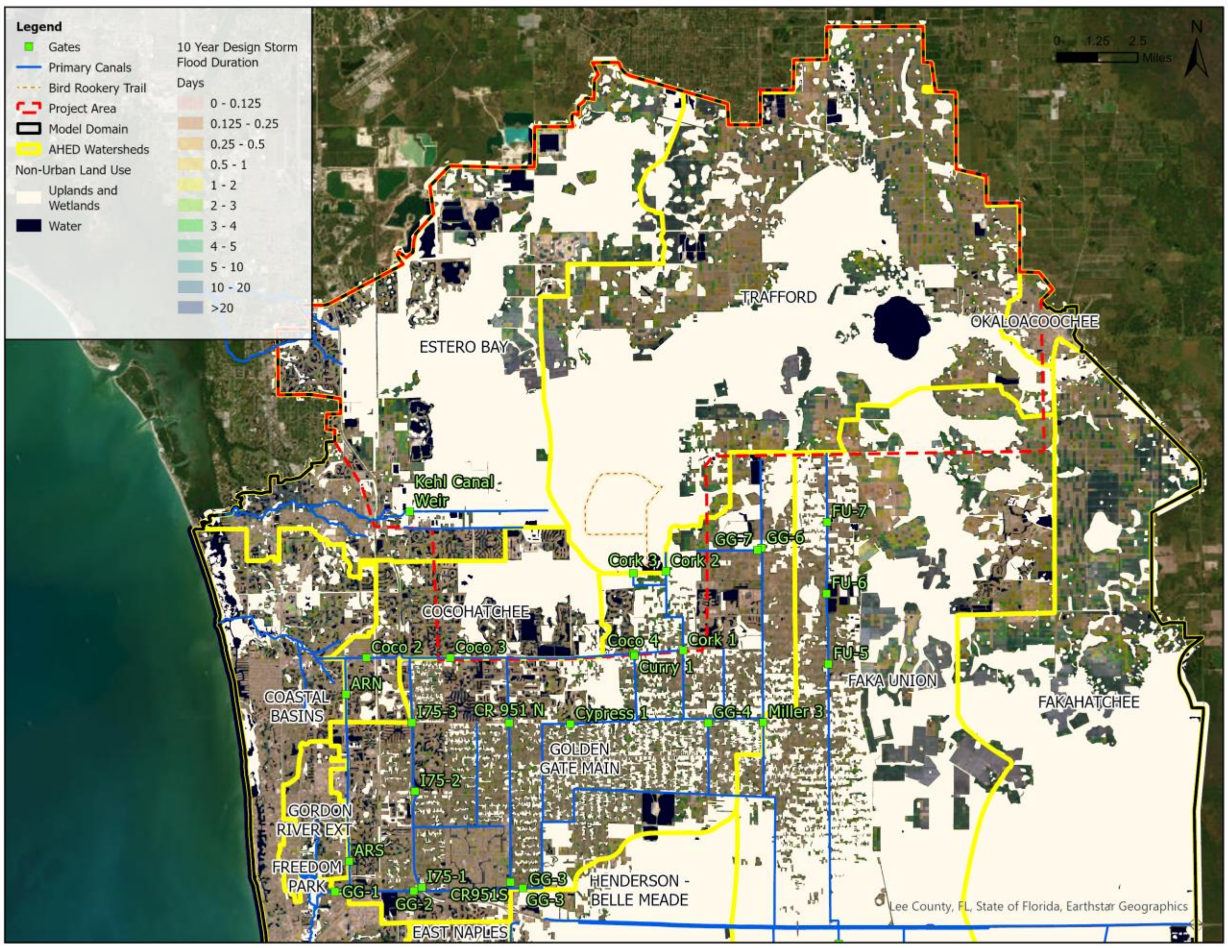

Figure 4-63. Ten-Year Design Storm Flood Duration (Days) for Urban and Agricultural Land Uses

Table 4-7. Ten-Year Design Storm Flood Duration Areas for Urban and Agricultural Land Uses

Duration (Days)	Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	COCOHATCHEE	FAKA UNION	GOLDEN GATE MAIN	TRAFFORD
0 - 0.125	6	2	13	26	20
0.125 - 0.25	2	1	8	7	9
0.25 - 0.5	2	1	8	7	10
0.5 - 1	5	1	10	6	15
1 - 2	6	2	10	8	16

Duration (Days)	Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	COCOHATCHEE	FAKA UNION	GOLDEN GATE MAIN	TRAFFORD
2 - 3	4	0	6	5	9
3 - 4	3	0	5	4	7
4 - 5	4	0	3	2	7
5 - 10	15	1	15	6	18
10 - 20	19	2	18	3	19
≥ 20	46	10	54	20	56

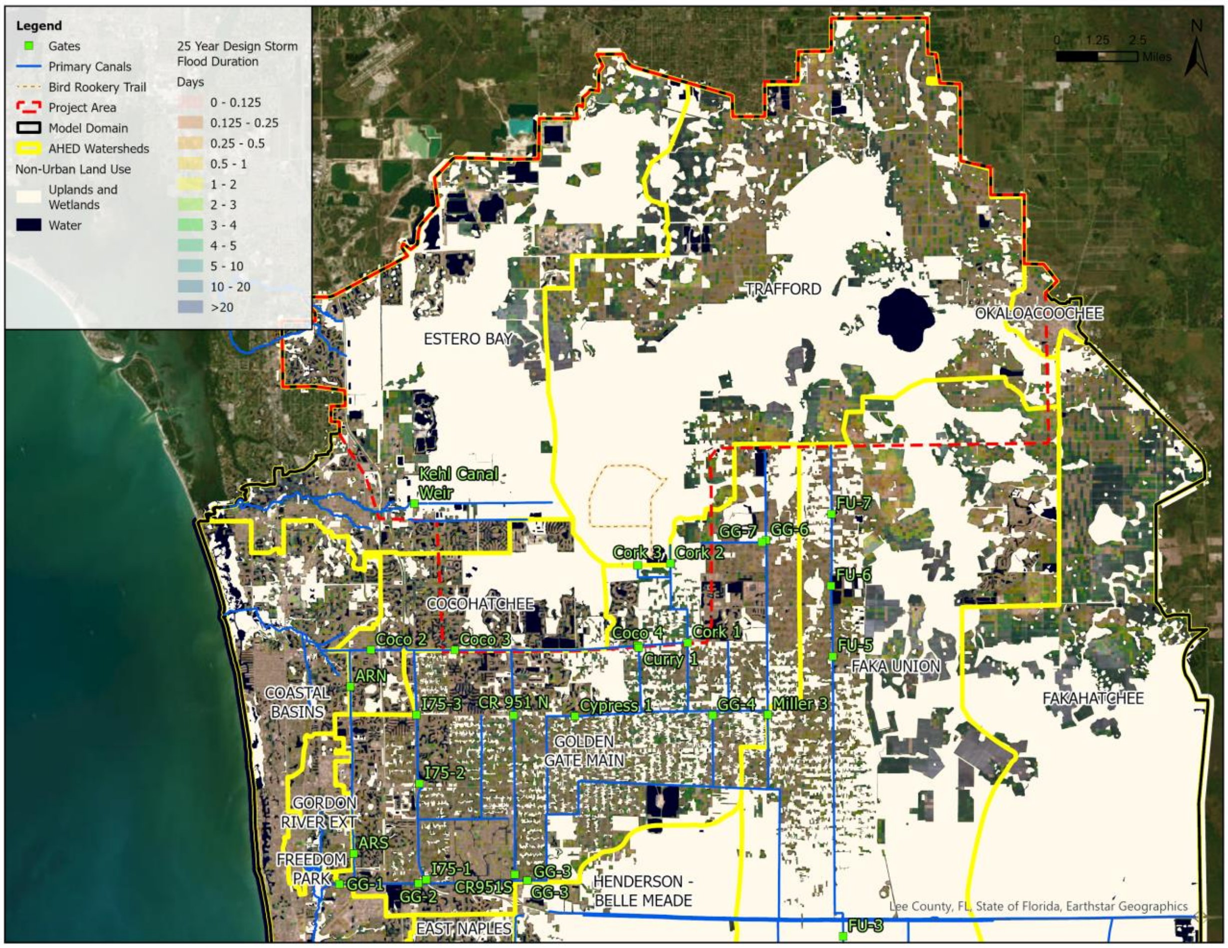

Figure 4-64. Twenty Five-Year Design Storm Flood Duration (Days) for Urban and Agricultural Land Uses

Table 4-8. Twenty Five-Year Design Storm Flood Duration Areas for Urban and Agricultural Land Uses

Duration (Days)	Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	COCOHATCHEE	FAKA UNION	GOLDEN GATE MAIN	TRAFFORD
0 - 0.125	15	8	41	104	50
0.125 - 0.25	3	1	10	15	12

Duration (Days)	Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	COCOHATCHEE	FAKA UNION	GOLDEN GATE MAIN	TRAFFORD
0.25 - 0.5	3	1	10	9	12
0.5 - 1	5	2	13	10	18
1 - 2	7	2	15	11	19
2 - 3	5	1	9	8	11
3 - 4	4	0	5	6	8
4 - 5	3	0	3	4	7
5 - 10	16	1	16	15	22
10 - 20	20	2	20	4	23
≥ 20	49	11	60	20	59

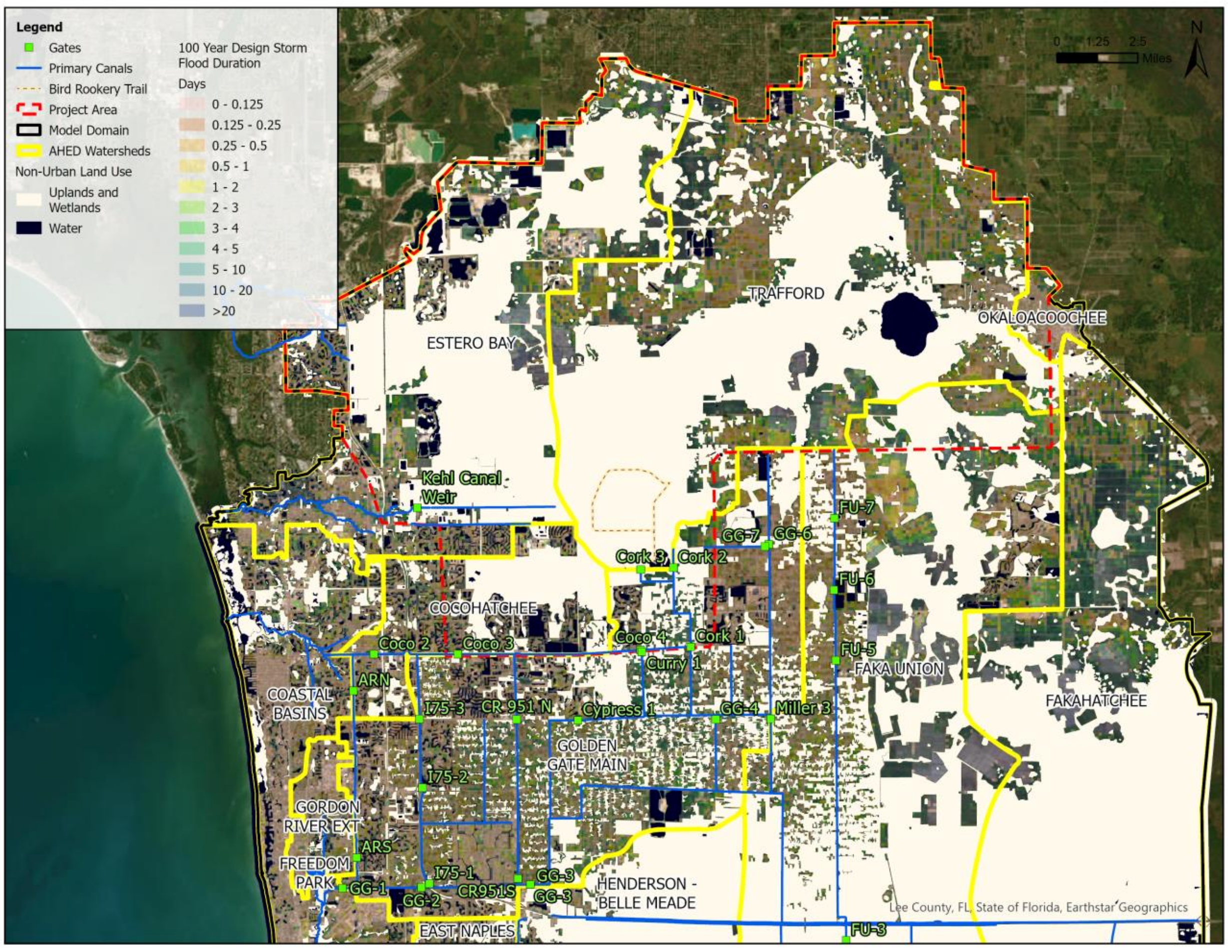

Figure 4-65. Hundred-Year Design Storm Flood Duration (Days) for Urban and Agricultural Land Uses

Table 4-9. Hundred-Year Design Storm Flood Duration Areas for Urban and Agricultural Land Uses

Duration (Days)	Area of Inundation > 0.25 ft in Urban and Agricultural Land Use Area (Acres)				
	ESTERO BAY	COCOHATCHEE	FAKA UNION	GOLDEN GATE MAIN	TRAFFORD
0 - 0.125	27	17	59	151	75
0.125 - 0.25	4	2	9	21	15
0.25 - 0.5	4	2	10	13	16
0.5 - 1	7	2	15	14	24
1 - 2	8	2	17	16	22
2 - 3	5	1	12	9	14
3 - 4	5	1	8	8	11
4 - 5	3	1	7	6	9
5 - 10	18	2	19	25	28
10 - 20	22	3	24	18	28
≥ 20	54	12	66	21	65

In general, the peak flood depths and duration are fairly low for most urban areas, less than 0.5 feet and 1 day, respectively. Some of the agricultural fields are have relatively high flood peaks and flood durations, which is due to the assumed drainage infrastructure and operations of the fields.

4.3 Wetland Hydrologic Restoration Performance Measures and LTS Results

In this section, maps and tables are presented in support of the wetland hydrologic restoration measures and metrics. Some of the PMs will be quantified and compared with the targets values described in the document *Corkscrew Watershed Initiative Deliverable 3.2: Final Project Performance Measures and Metrics Technical Memorandum* (SFWMD, 2025a), which will provide an assessment of the existing performance, while some results are shown qualitatively to be compared with the FCM baseline and future alternatives.

In addition to water depths and hydroperiod maps, values are summarized for the different types of vegetation communities based on the District's land use map and for wetland hydrology indicators. Sixty-one wetland hydrology indicator locations for the various types of plant communities in Corkscrew were developed by BCB staff in coordination with Audubon's ecological consultant, Mike Duever. Note the some of the plant community categories described in the wetland hydrology indicator dataset differ from the model vegetation categories. The vegetation category classification in the model from the SFWMD Land Use Dataset was conducted during the BCB update as described in the report by Hazen and Sawyer (Hazen, 2025). Table 4-10 shows the relationship between the wetland vegetation categories in the model and the land use FLUCC codes and Figure 4-66 shows a map of the model vegetation spatial distribution.

Table 4-10. Model Vegetation Categories and SFWMD Land Use Dataset FLUCCS Codes

MIKE SHE Code	Wetland Vegetation	FLUCCS Codes
8	Mesic Flatwood	3100, 3200, 3210, 3300, 4110, 4410
9	Mesic Hammock	4200, 4220, 4270, 4271, 4300, 4340, 4370
10	Xeric Flatwood	4130, 2240

MIKE SHE Code	Wetland Vegetation	FLUCCS Codes
11	Xeric Hammock	3220
12	Hydric Flatwood	6240, 6250, 6180
13	Hydric Hammock	4240, 4280, 6191
14	Wet Prairie	6430, 2540
16	Marsh	6400, 6410, 6440,
17	Cypress	6200, 6210, 6215, 6216
18	Swamp Forest	6170, 6172, 6190, 6300
19	Mangrove	6120, 6420
20	Water	1660, 5110, 5120, 5200, 5300, 5410, 5420, 5430, 5720, 6510, 8360

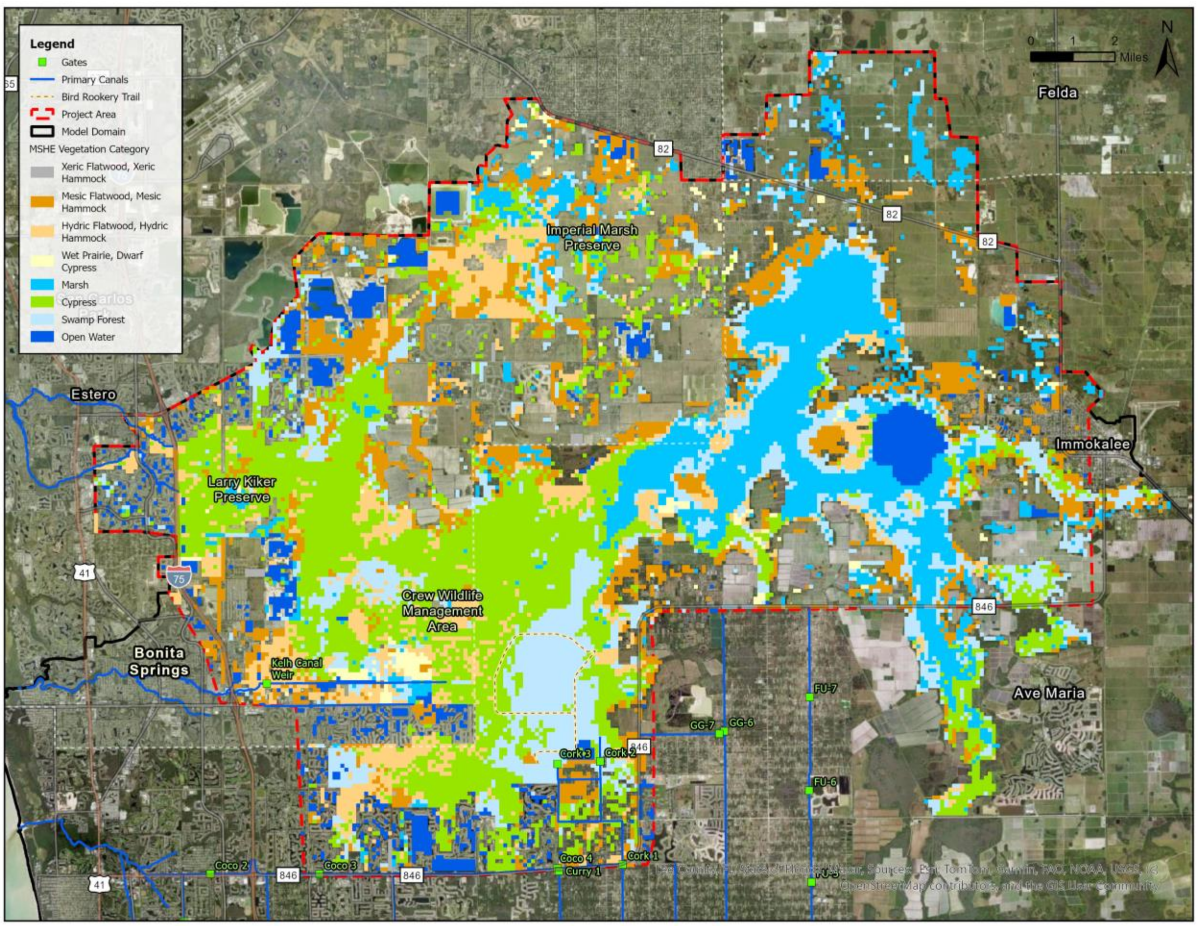

Figure 4-66. Model Wetland Vegetation Categories

Table 4-11 provides a reference for target values for the metrics for Southwest Florida plant communities provided by Mike Duever for the CWI project. Figure 4-67 shows a map of the wetland hydrology indicator locations. If the description in the indicator point differs from the model vegetation category the point in the map shows the two classifications. The averaging calculations that were grouped into plant communities were conducted according to the model categories, but values for each indicator point were also extracted and presented in the sections below.

Table 4-11. Hydrologic Regimes of Major Southwest Florida Plant Communities

Plant Community	Hydroperiod (months)	Seasonal Water Level (in)	
		Wet Season	Dry Season (1,10)*
Xeric Flatwood and Hammock	0	≤-24	-60, -90
Mesic Flatwood and Hammock	≤1	≤2	-46, -76
Hydric Flatwood and Hammock	1-2	2-6	-30, -60
Wet Prairie and Dwarf Cypress	2-6	6-12	-24, -54
Marsh	6-10	12-24	-6, -46
Cypress	6-8	12-18	-16, -46
Swamp Forest	8-10	18-24	-6, -36
Open Water	>10	>24	< 24, -6

*1 = average year low water level; 10 = 1 in 10 year drought.

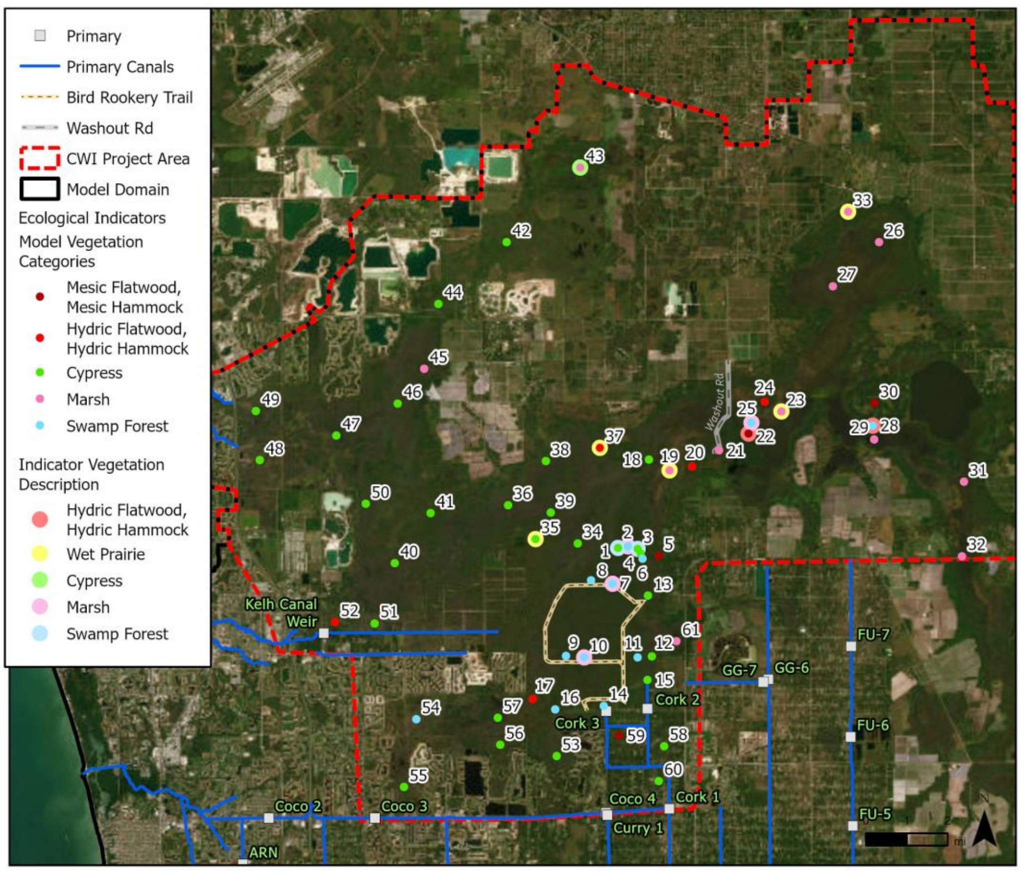

Figure 4-67. Wetland Hydrology Indicators for Various Plant Communities.

4.3.1 Average Daily Wet Season Water Depth

The purpose of the average wet season depth metric is to assess the volume of surface water for the various plant communities so that aquatic fauna can build up their populations through the wet season. Wet season depths are averaged for the period of July 1 to September 30 to capture the wettest portion of the wet season after the raising water table in June and before the decline in October (Duever, 2024).

Deliverable 3.2 specifically defines the average daily wet season PM as the median wet season water depth averaged over the five simulation years. Figure 4-68 shows the simulated median wet season depths averaged over the 5-year simulation period. To calculate the wet season water depths above ground, the median water table for each year was subtracted from the high resolution corrected LiDAR (as described in Deliverable 5.1.3 (SFWMD, 2025b)). Table 4-12 shows the average wet season depth for each of the plant communities in the project area. Figure 4-69 shows a plot of the average wet season depth for each of the wetland hydrologic indicators.

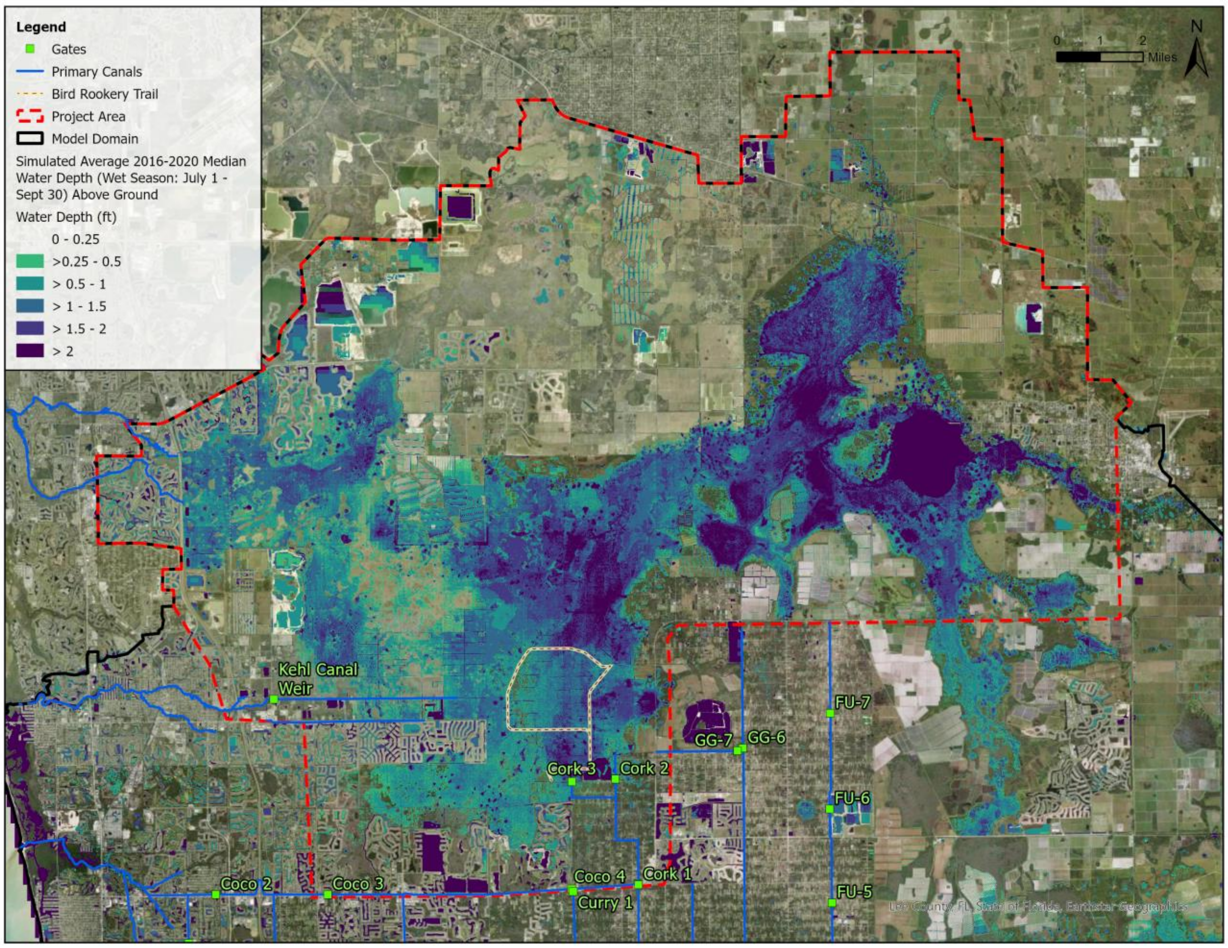

Figure 4-68. Simulated Average Wet Season Depths for the Years 2016 to 2020.

Table 4-12. Simulated Average Wet Season Depths for Plant Communities (inches)

Plant Community	2016	2017	2018	2019	2020	Avg.	Reference Value[1] (in)
Xeric Flatwood, Xeric Hammock	1.0	1.5	0.2	0.9	0.1	0.4	≤-24
Mesic Flatwood, Mesic Hammock	4.8	5.7	3.1	3.6	2.1	3.5	≤2
Hydric Flatwood, Hydric Hammock	6.6	8.5	4.5	5.8	3.4	5.4	2-6
Wet Prairie, Dwarf Cypress	5.2	8.0	3.3	4.5	1.7	3.9	6-12
Marsh	16.6	17.7	13.6	14.1	11.5	14.5	12-24
Cypress	13.1	14.6	9.6	12.0	8.6	11.3	12-18
Swamp Forest	13.1	14.5	9.8	11.0	7.9	11.0	18-24
Open Water	25.4	26.5	20.4	24.6	18.5	22.9	>24

[1] Reference values for wet season depths (Table 4-11)

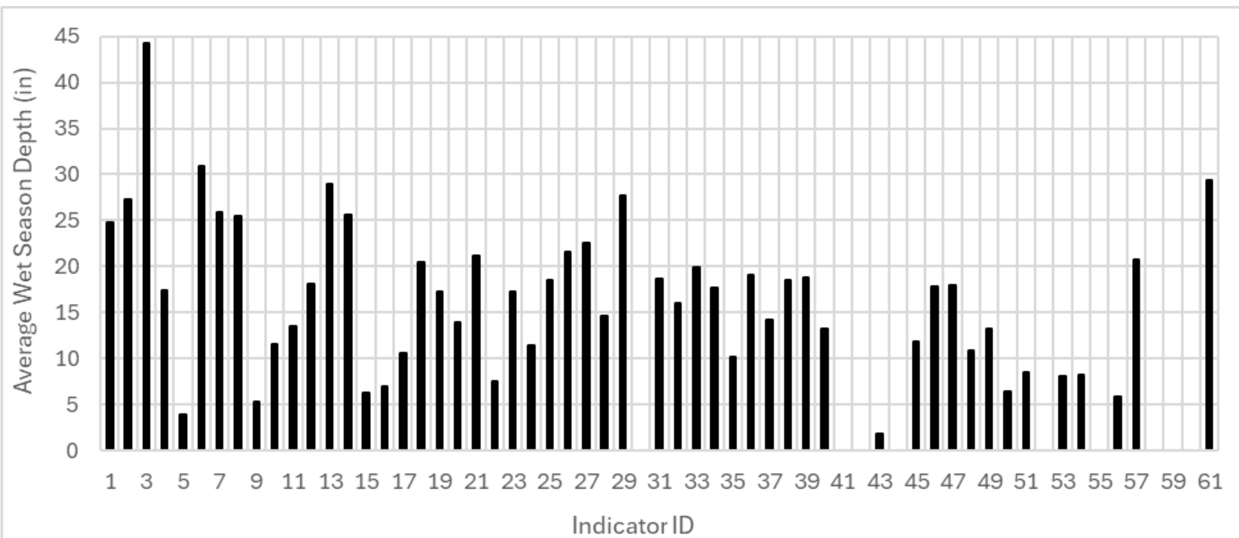

Figure 4-69. Wet Season Average Depth for the Wetland Hydrology Indicators (Figure 4-67)

4.3.2 Minimum Dry Season Water Depth

This metric provides an assessment of dry conditions necessary for: “1) germination and survival of some important wetland plant species; 2) provide wetland foraging sites for numerous species of predators during their breeding season; 3) annual aeration of wetland substrates that can be critical to the survival of wetland plants; and 4) create appropriate fire conditions that maintain the diversity of natural southwest Florida plant communities” (Duever, 2024). This metric is currently being reevaluated by the TWG from the one defined in Deliverable 3.2, which is the median water level during the dry season months (November 1 to May 31). The ongoing evaluation of this metric was recommended by TWG members after a recent analysis of the Picayune Strand Restoration Project data, as well as further analysis of historical data in the context of the project goals. Thus, no results for this metric are shown in this document, but an amendment to this report will be provided once the TWG reaches consensus on this metric.

4.3.3 Hydroperiod

Hydroperiod in the context of this study is defined as the duration of time that water depths are above ground during a water year, which is defined from June 1 to May 31. This measure provides an assessment of the ability of the wetland system to "minimize colonization by upland vegetation, so that aquatic organisms have sufficient time to mature and build up their populations through the wet season. It is also significant for supporting a natural fire regime and minimizing wildfires" (Duever, 2024).

Figure 4-70 shows the simulated average annual hydroperiod for the four water years from June of 2016 to June of 2020. The first water year in the simulation was excluded because the initial months of the simulation that would be needed for a full water year are considered part of the warmup period. Table 4-13 shows the simulated average hydroperiod averaged over the various plant communities in the project area for each water year (WY) and the average over the four years. Figure 4-71 shows the simulated average hydroperiod for each wetland hydrologic indicator.

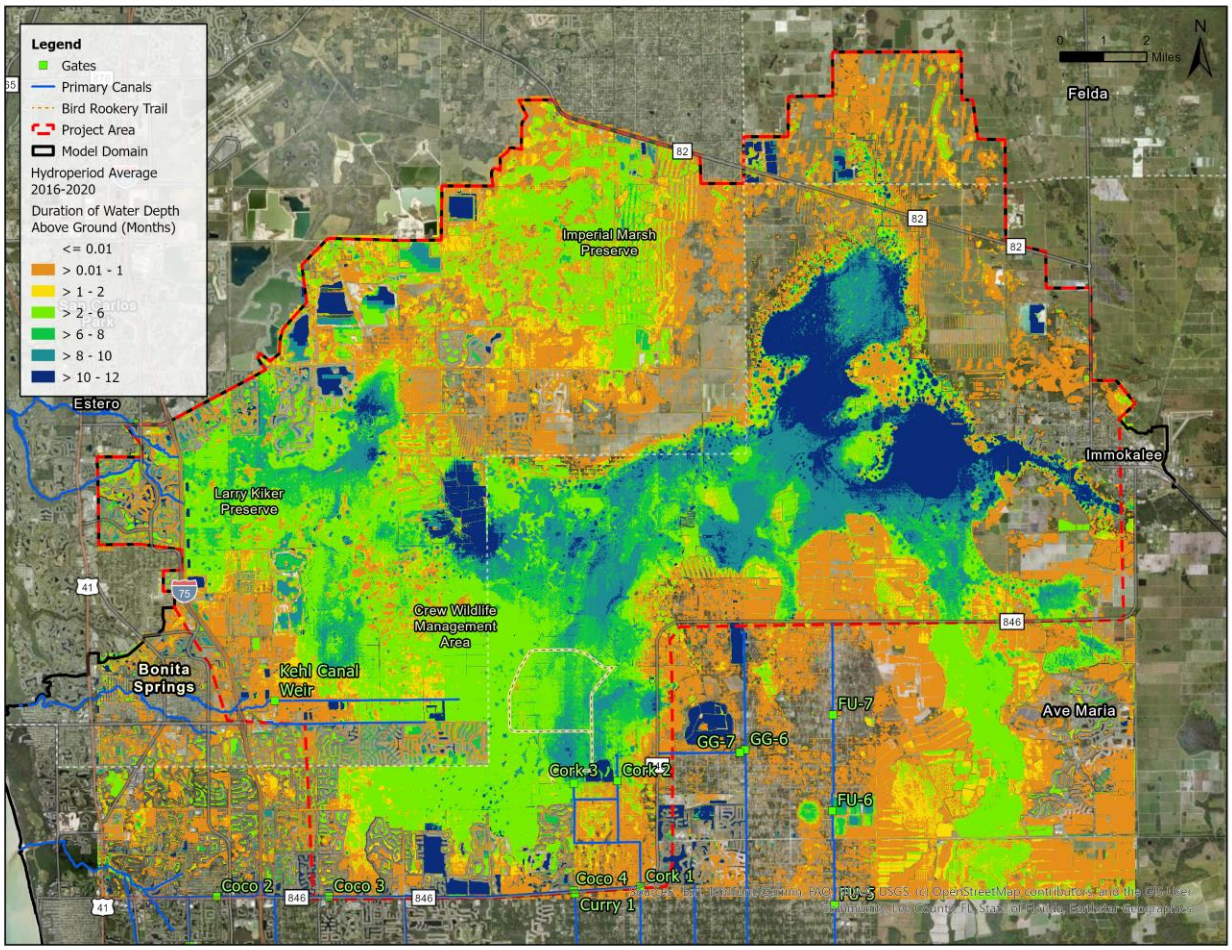

Figure 4-70. Simulated Average Hydroperiod.

Table 4-13. Simulated Average Hydroperiod (months) for Plant Communities

Plant Community	WY 2016-2017	WY 2017-2018	WY 2018-2019	WY 2019-2020	Avg.	Ref.[1]
Xeric Flatwood, Xeric Hammock	0.6	1.2	0.3	0.5	0.6	0
Mesic Flatwood, Mesic Hammock	2.4	3.1	2.2	1.9	2.4	≤1
Hydric Flatwood, Hydric Hammock	3.0	3.9	2.8	2.5	3.1	1-2
Wet Prairie, Dwarf Cypress	2.4	3.4	2.3	2.1	2.6	2-6
Marsh	6.1	6.9	7.2	6.1	6.6	6-10
Cypress	4.7	5.7	5.4	4.7	5.1	6-8
Swamp Forest	4.7	5.7	5.4	4.6	5.1	8-10
Open Water	6.7	7.4	7.0	6.8	7.0	>10

[1] Reference values for hydroperiod (Table 4-11)

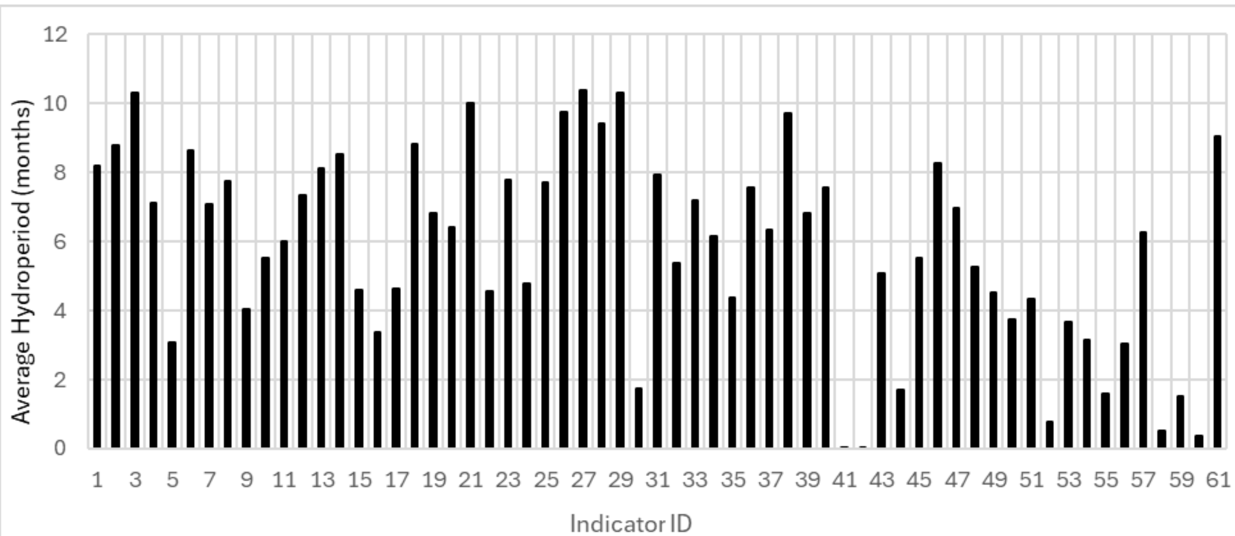

Figure 4-71. Average Hydroperiod for the Wetland Hydrology Indicators (Figure 4-67)

4.3.4 Recession Rates

Restoration of the historical recession rates in Corkscrew is one of the critical goals of the CWI project. Groundwater level plots at selected indicators (Figure 4-72) serve to provide a visual and qualitative representation to compare changes in recession rates with the FCM baseline and alternatives. The 15 indicators from the 61 points shown in Figure 4-67 were selected based on critical areas that have been identified for restoration targets as discussed during the project TWG meetings. Figure 4-73 to Figure 4-87 show plots of the daily groundwater levels, the two-week averaged groundwater levels, and the two-week recession rates. The two-week recession rates were calculated by subtracting the two-week (or 15-day) rolling average groundwater level at the end of a two-week (or 15-day) cycle from the value at the beginning of the cycle. Table 4-14 shows the average monthly recession rates for each location. The figures and tables not only serve as a reference for the FCM baseline and alternatives, but also capture the spatial and temporal patterns of recession rates through these areas of Corkscrew.

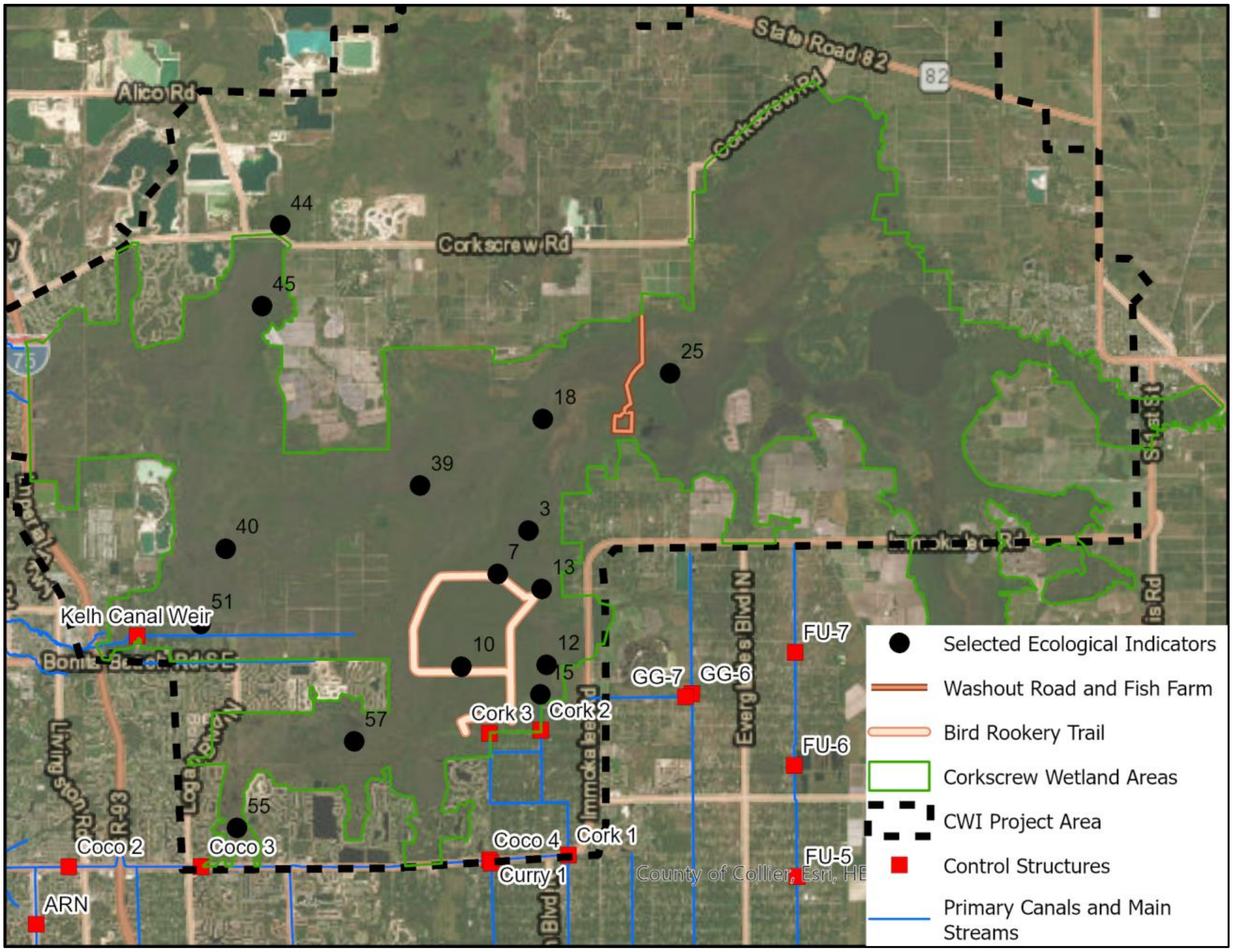

Figure 4-72. Selected Indicators for Groundwater Level and Recharge Plots

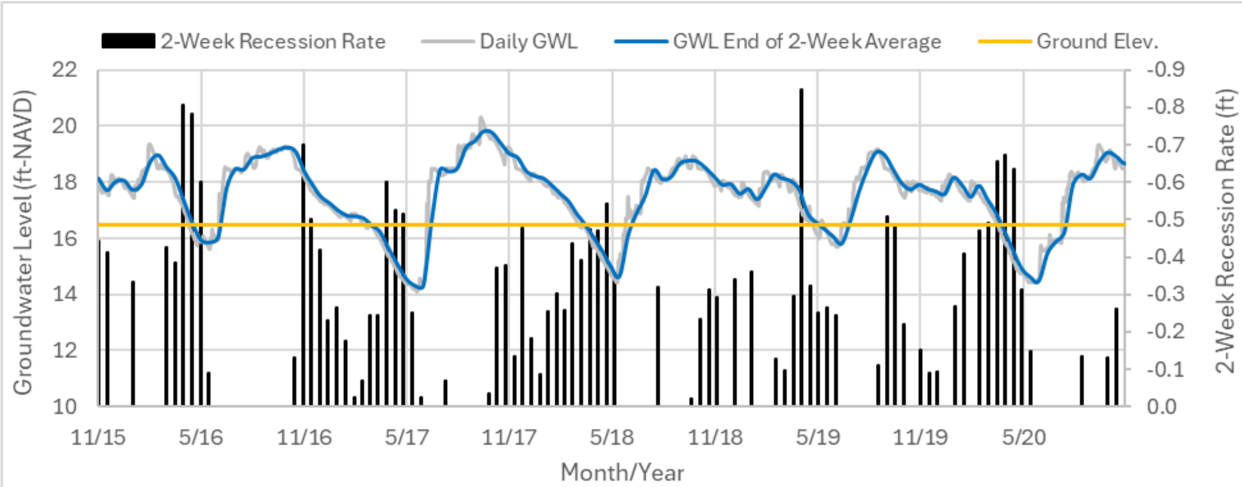

Figure 4-73. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID25

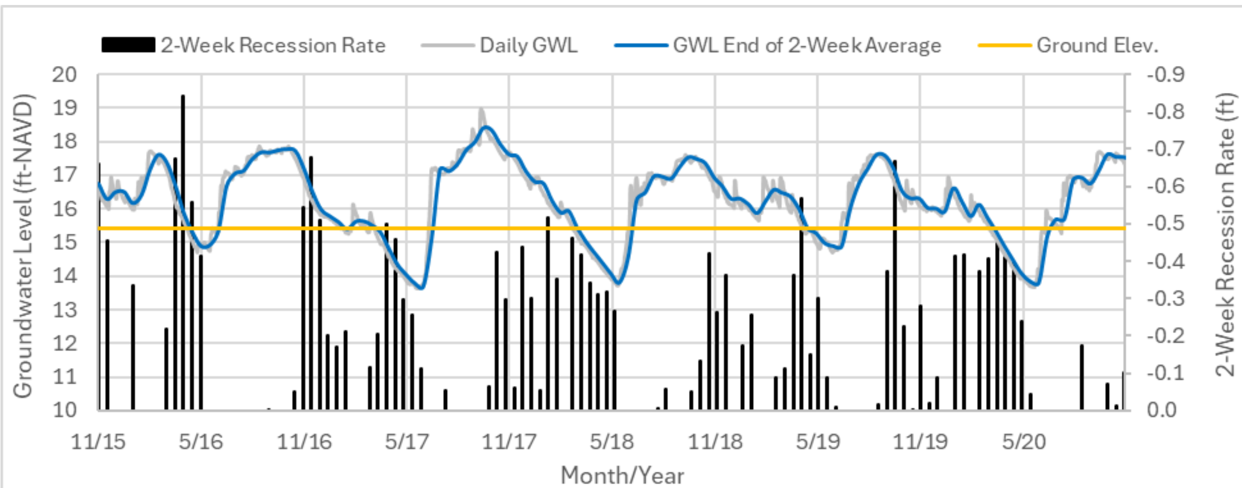

Figure 4-74. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID18

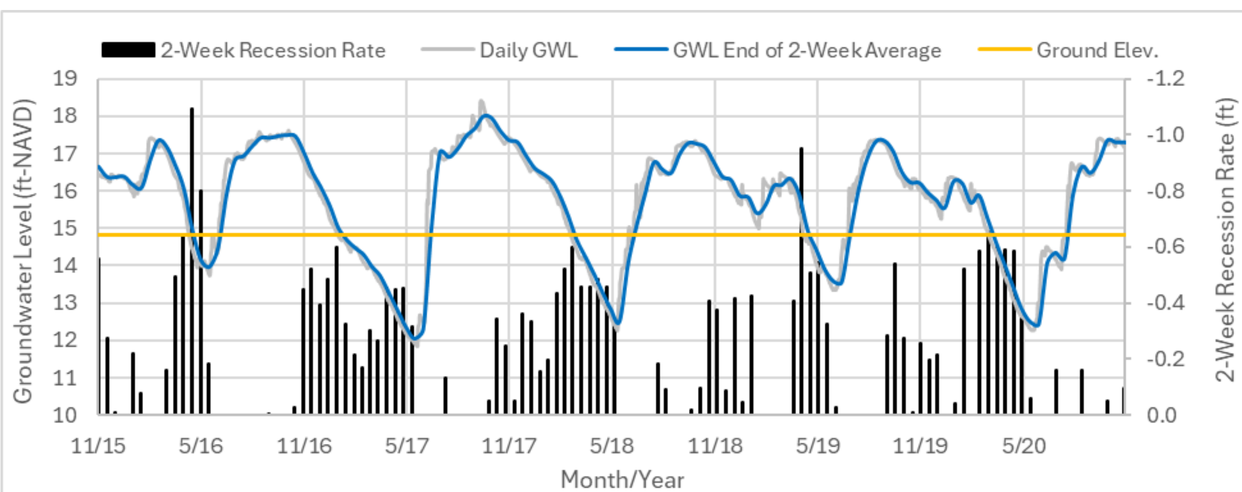

Figure 4-75. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID3

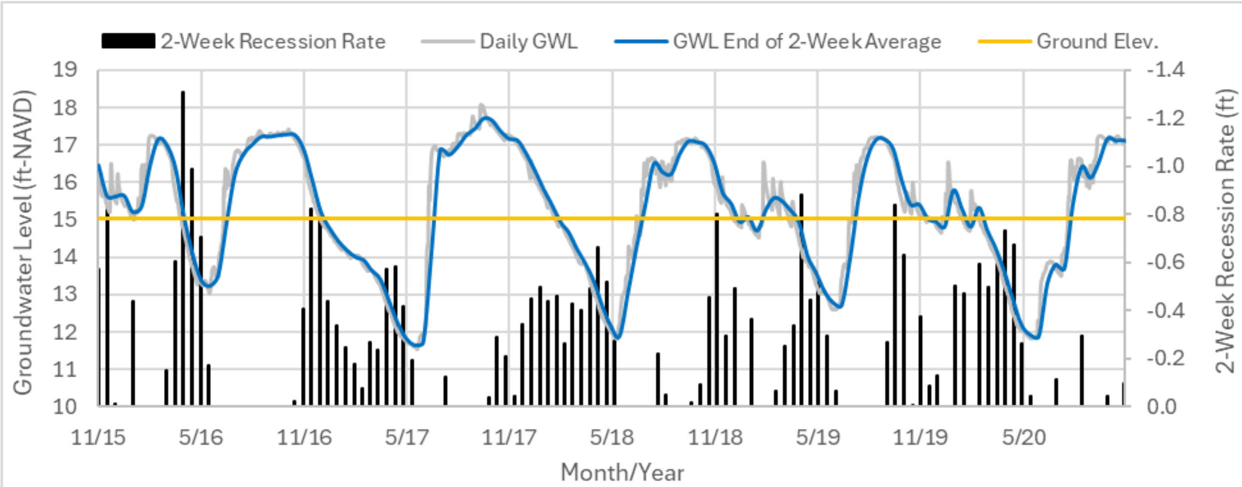

Figure 4-76. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID7

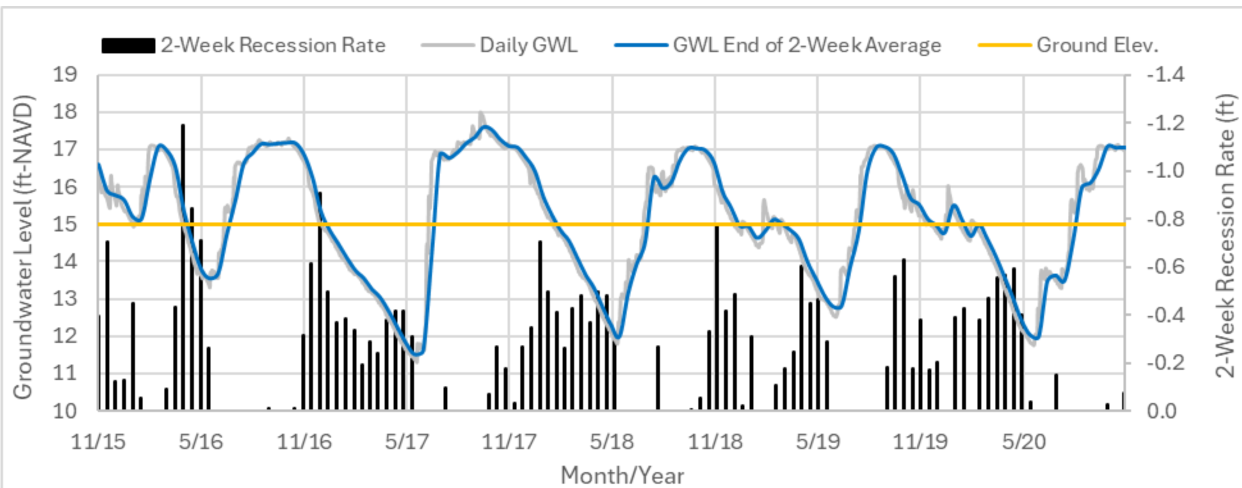

Figure 4-77. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID13

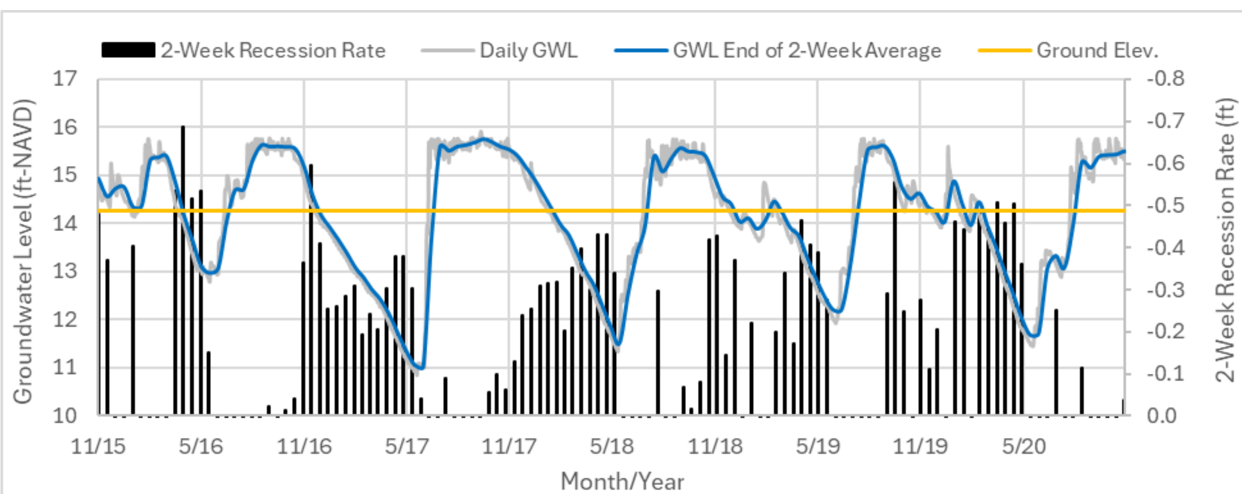

Figure 4-78. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID12

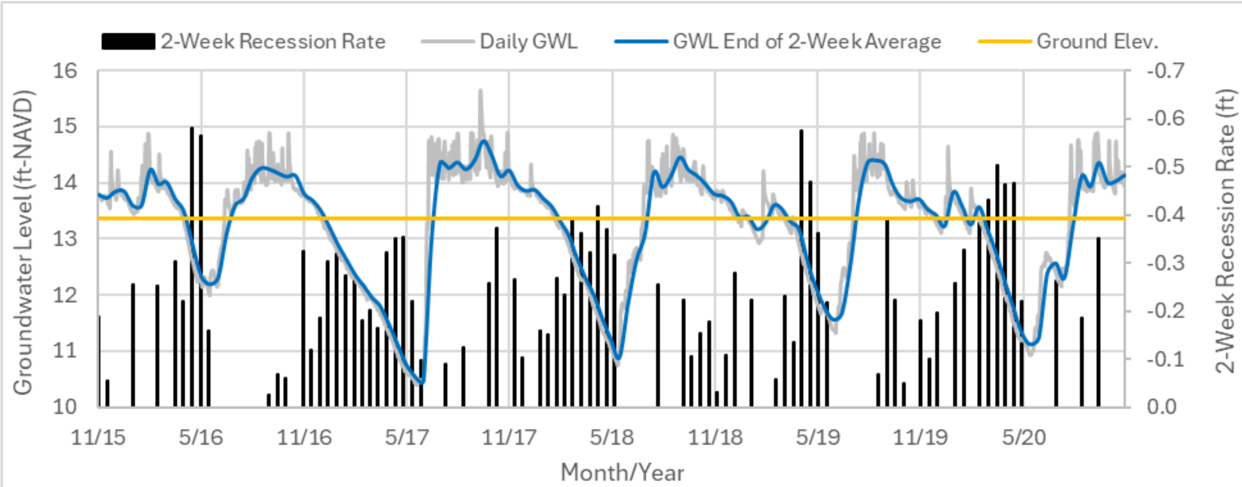

Figure 4-79. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID15

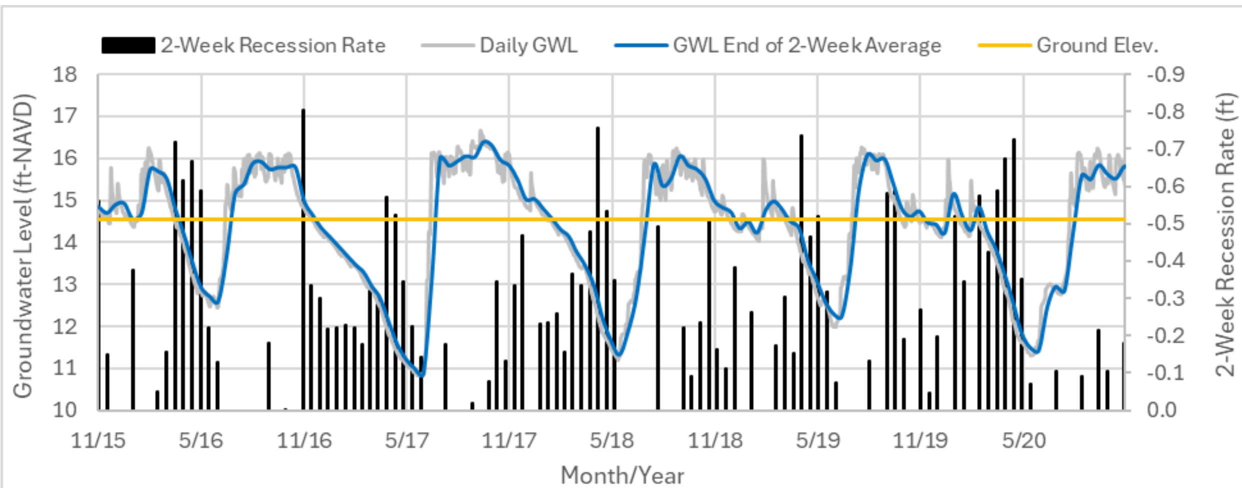

Figure 4-80. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID10

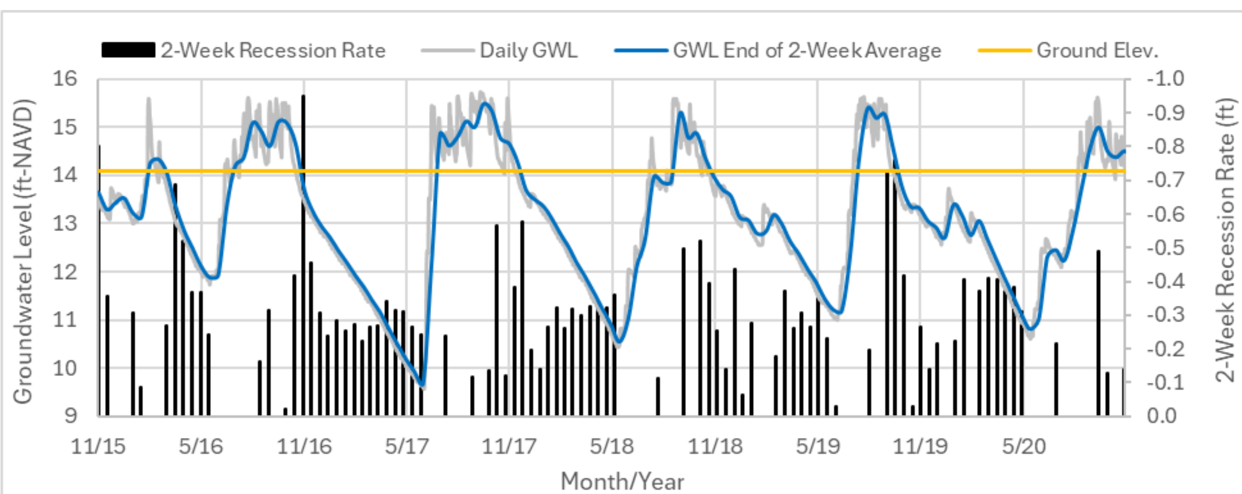

Figure 4-81. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID57

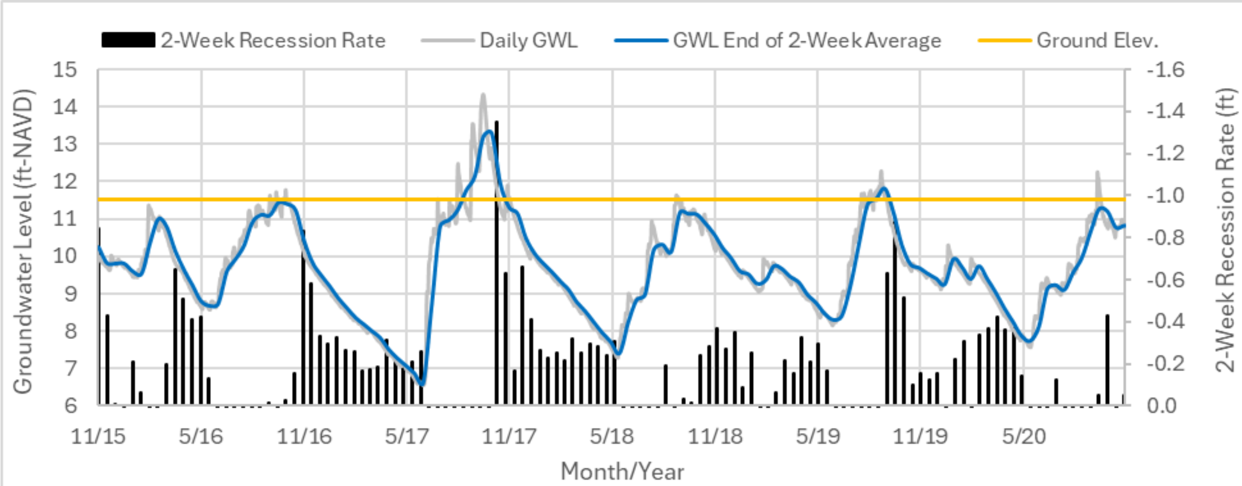

Figure 4-82. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID55

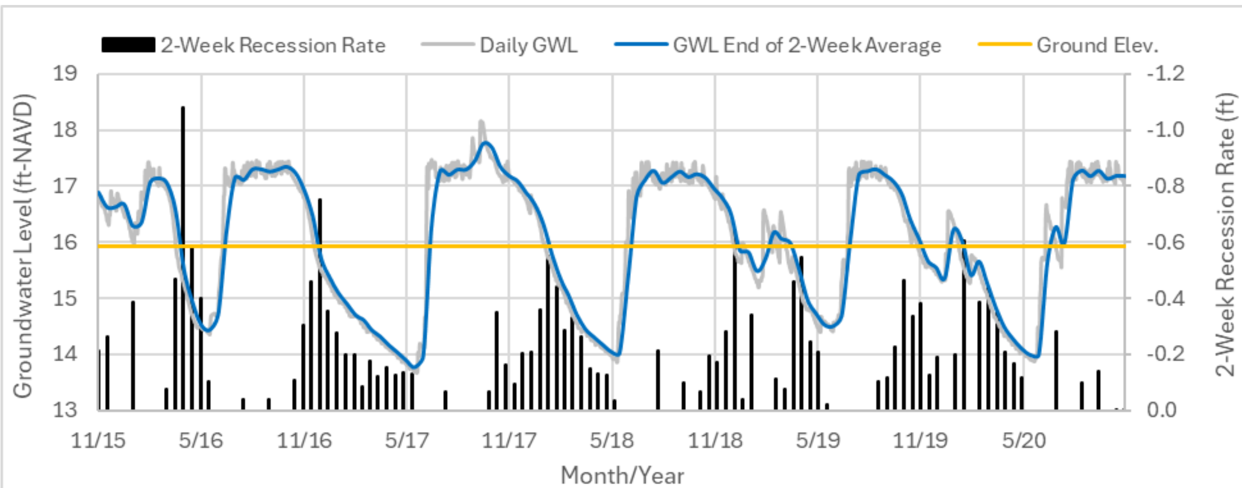

Figure 4-83. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID39

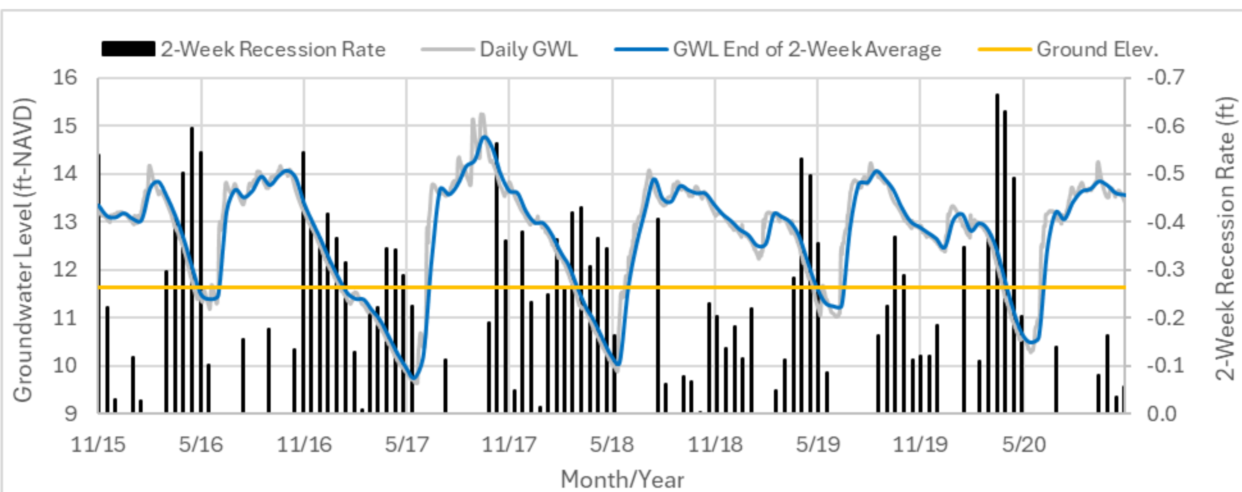

Figure 4-84. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID40

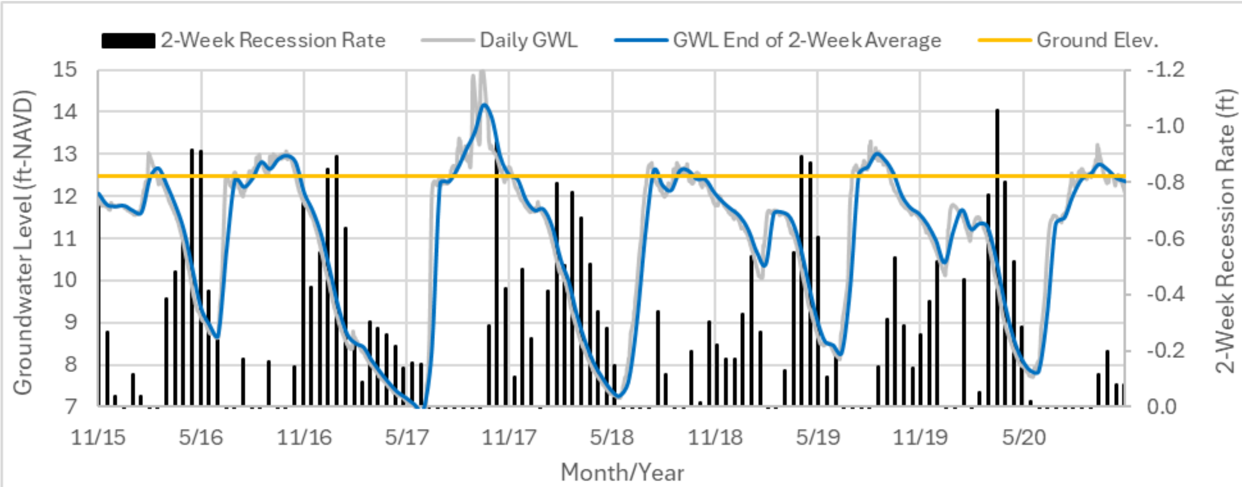

Figure 4-85. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID51

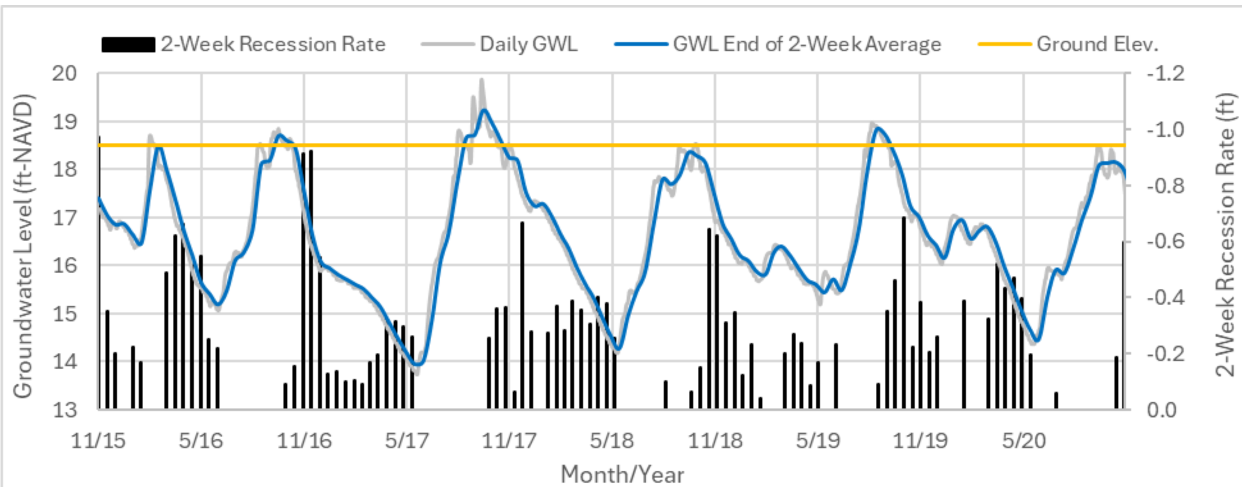

Figure 4-86. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID44

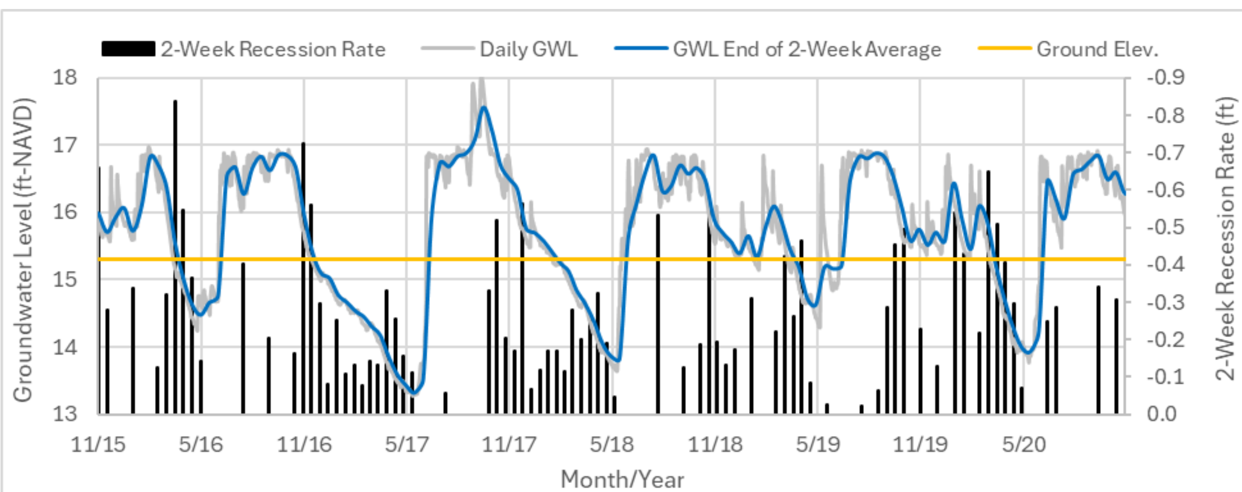

Figure 4-87. Simulated Groundwater Levels and Two-Week Average Recession Rates for Indicator ID45

Table 4-14. Simulated Average Monthly Recession Rates (inches)[1]

ID	Site Description	Nov	Dec	Jan	Feb	Mar	Apr	May
25	Marsh east of Washout Road	-6.5	-3.5	-1.9	3.0	-8.0	-14.7	-9.3
18	Cypress west of Washout Road	-7.6	-4.1	-2.9	4.2	-7.4	-11.5	-6.3
3	Swamp forest south of Lettuce Lake	-6.5	-6.0	-3.3	0.8	-8.0	-14.7	-10.9
7	Marsh north of BR trail	-9.2	-7.3	-3.8	4.2	-8.4	-17.2	-10.0
13	Eastern Flow way (north of berm east of BR trail)	-7.9	-8.5	-5.9	2.6	-7.4	-13.6	-10.6
12	Eastern Flow way (Cypress shell bed south of berm and north of Cork 2)	-5.9	-4.3	-3.2	1.4	-7.6	-10.1	-9.1
15	Just north of Cork 2	-2.2	-2.7	-2.4	-0.4	-6.0	-9.3	-8.4
10	BR north of the southern trail	-6.0	-3.6	-2.1	2.7	-8.8	-13.8	-10.4
57	Southwest Corkscrew Flow way #1	-8.1	-4.8	-2.9	1.0	-8.2	-8.5	-7.6
55	Southwest Corkscrew Flow way #2	-8.8	-6.3	-3.0	2.6	-6.7	-7.9	-6.0
39	Cypress flow diversion to Lee County	-5.2	-6.5	-3.4	0.2	-5.9	-9.1	-4.0
40	Cypress Flint Pen	-5.7	-4.7	-1.8	1.2	-6.3	-11.0	-7.7
51	Cypress near Kehl Canal	-7.5	-8.1	-4.1	0.1	-9.1	-14.7	-10.4
44	Cypress north of Corkscrew Rd	-12.5	-7.0	-1.0	3.2	-6.0	-9.6	-7.5
45	Marsh south of Corkscrew Rd	-7.4	-2.2	-0.9	2.6	-8.2	-8.9	-0.9

[1]Positive values water table increase; negative values water table decrease.

In general, out of the seven dry season months, the average simulated recession rates for the 5-year period were lowest during January and February and highest during April and May. However, the relative rates at the various locations should not be interpreted without context because it depends on the total rise and fall of the water table. For example, the highest and lowest April recession rates occurred at indicator locations 7 (north of Bird Rookery) and 55 (in southwest Corkscrew just north of COCO3), respectively. Plotting those two locations side by side (Figure 4-88), show that the water table in location 55 is in general less variable than in location 7. Moreover, water levels relative to the topography are higher in location ID7 (Figure 4-76) than in location ID55 (Figure 4-82) Thus, it is difficult to quantify recession rates as a performance measure to assess the restoration objectives without accounting for both the variability of the water table and the depth in relation to the ground surface.

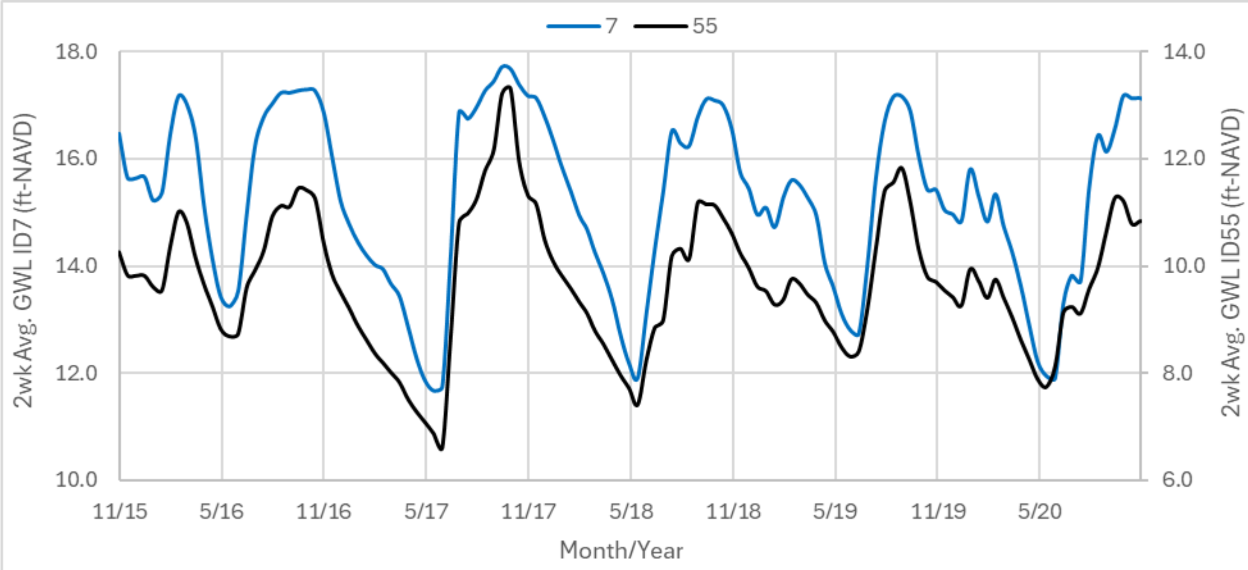

Figure 4-88. Comparison of the Highest and Lowest April Recession Rates Locations.

4.3.5 Sheet Flow

Cumulative overland flow volumes were calculated at locations where obstructions to the natural wetland sheet flow occur due to man-made features, such as roads and berms. Overland flow is completely blocked in the model along these obstructions by way of the Separated Flow Area parameter, but flow can be simulated across them via structures (culverts and weirs) in the 1D surface water model and via the groundwater flow component.

Overland flow was extracted upstream and downstream of the flow blockages. The total flow across the four lines shown in Figure 4-89 are shown in Figure 4-90 to Figure 4-93. The plots show how overland flow is substantially reduced from the upstream to the downstream sides of these blockages. As flow goes across the blockages, it narrows through the 1D structures and travels through the preferential pathways represented by the 1D flow ways.

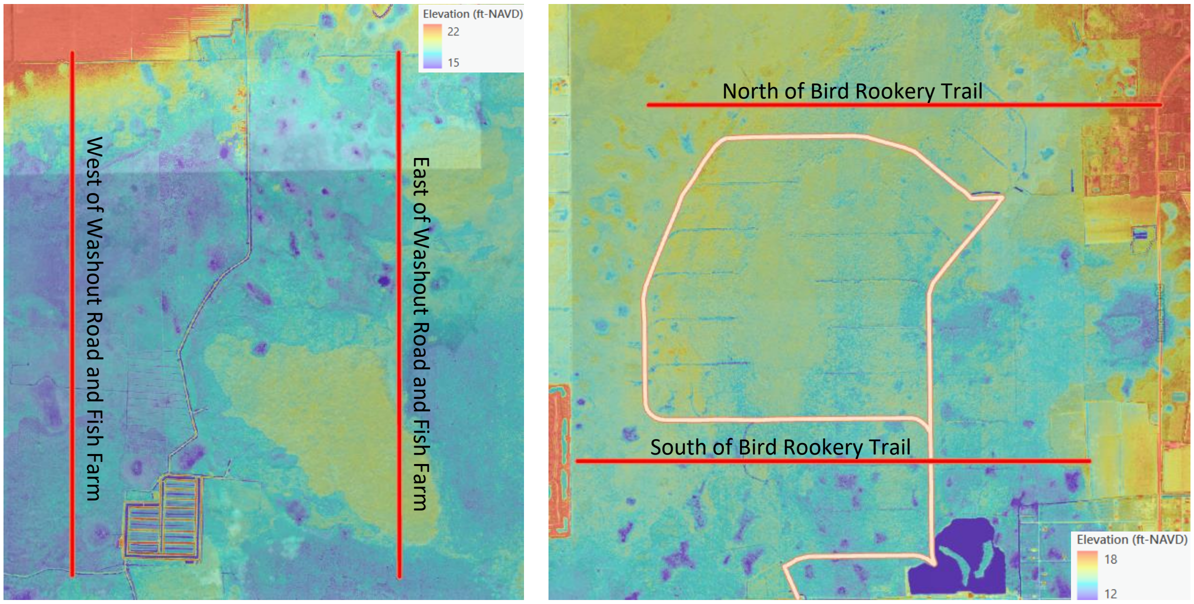

Figure 4-89. Locations where overland flow was extracted

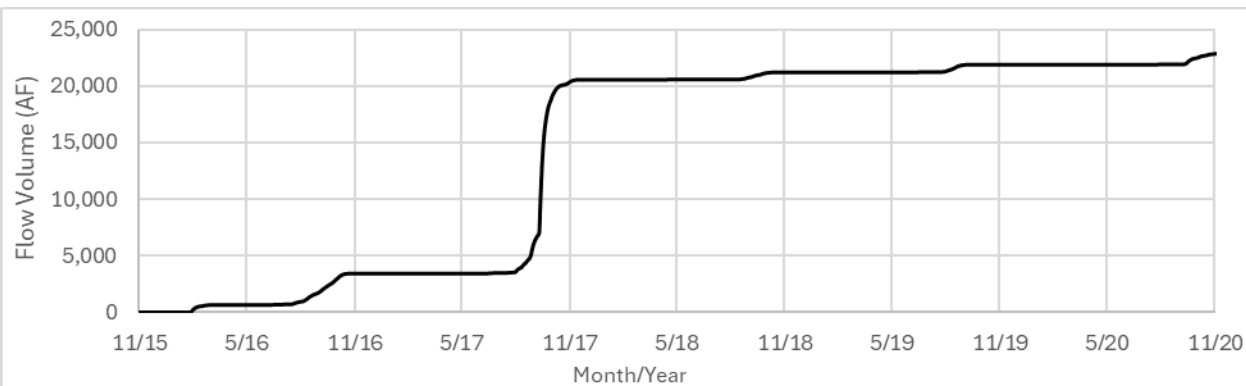

Figure 4-90. Simulated Cumulative Overland Flow Volume East of Washout Road and Fish Farm (acre-feet, AF)

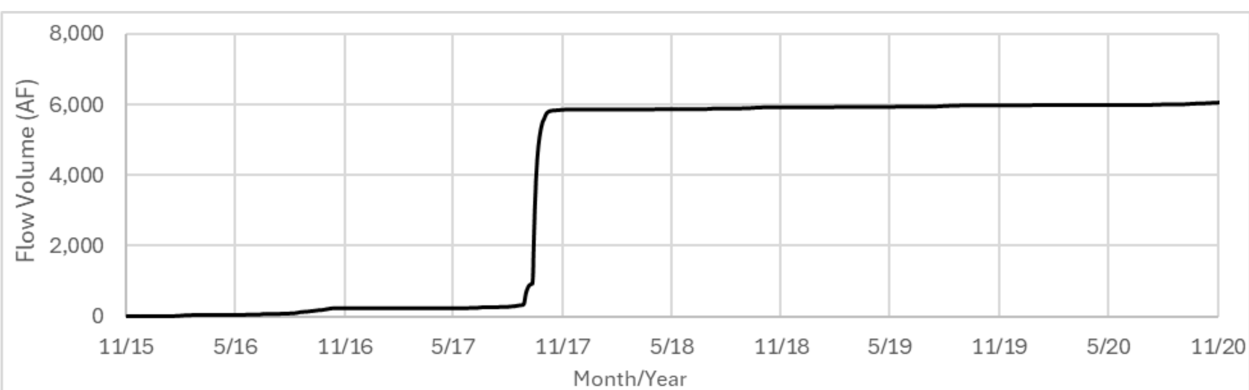

Figure 4-91. Simulated Cumulative Overland Flow Volume West of Washout Road and Fish Farm (acre-feet, AF)

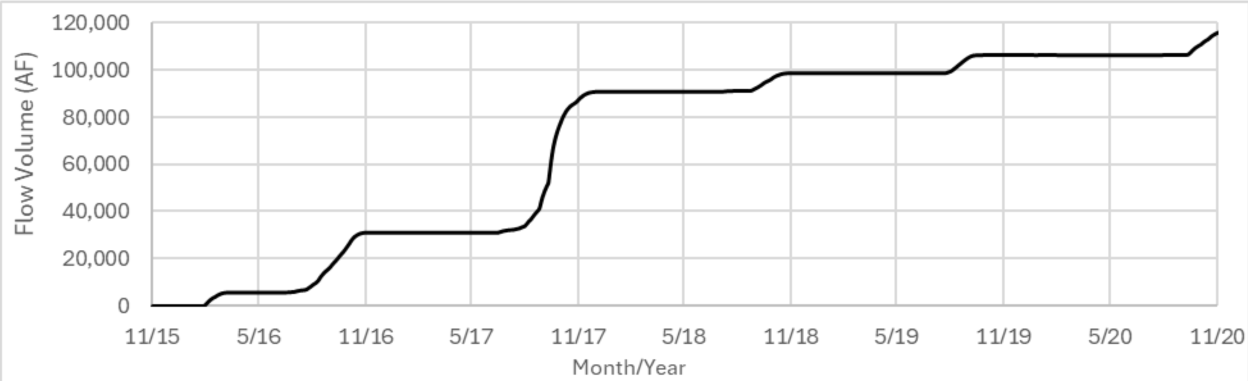

Figure 4-92. Simulated Cumulative Overland Flow Volume North of the Bird Rookery Trail (acre-feet, AF)

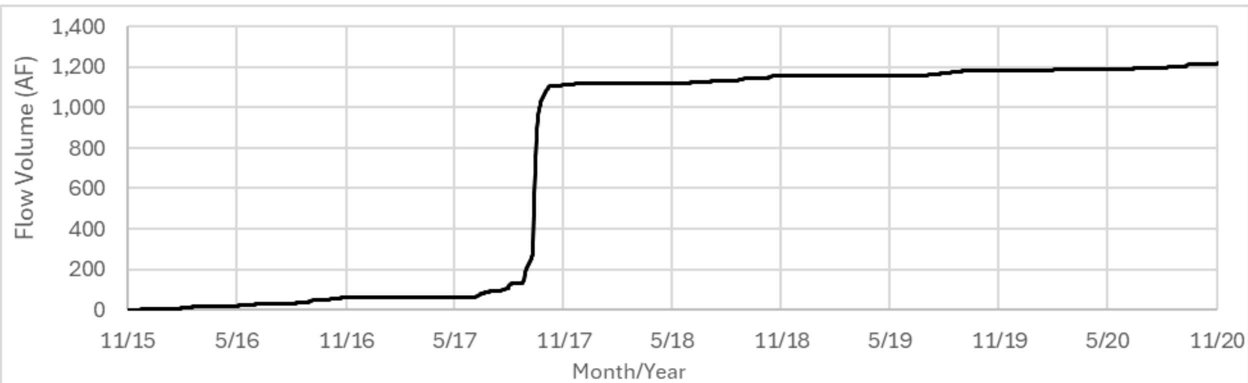

Figure 4-93. Simulated Cumulative Overland Flow Volume South of the Bird Rookery Trail (acre-feet, AF)

4.3.6 Flow Volumes at Tidal Structures

Changes in flow volumes at the tidal structure or drainage outlets serve as a measure of potential of changes in susceptibility to saltwater intrusion. A reduction in flow from the baseline is considered an increased risk of saltwater intrusion. Figure 4-94 to Figure 4-97 show the cumulative flow volumes at the main outlets to tide near the project area for existing conditions.

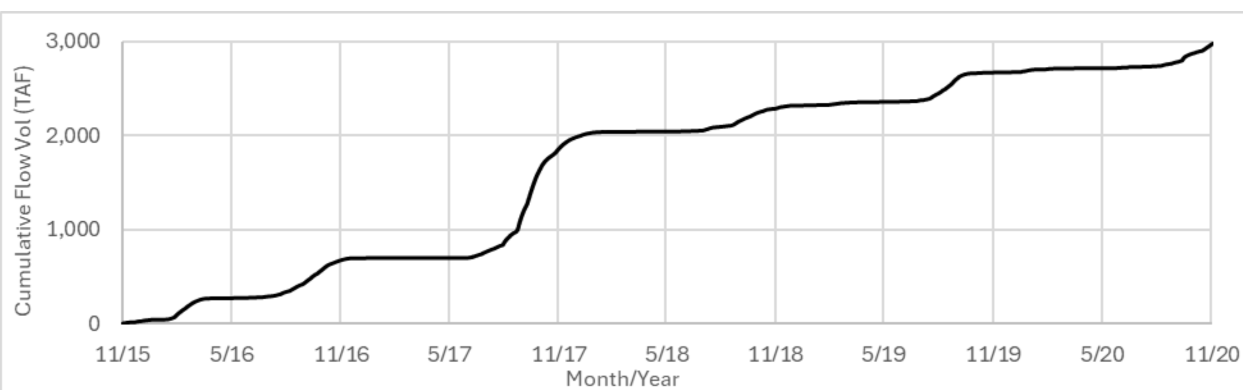

Figure 4-94. Simulated Cumulative Flow Volume at COCO1 (thousand acre-feet, TAF)

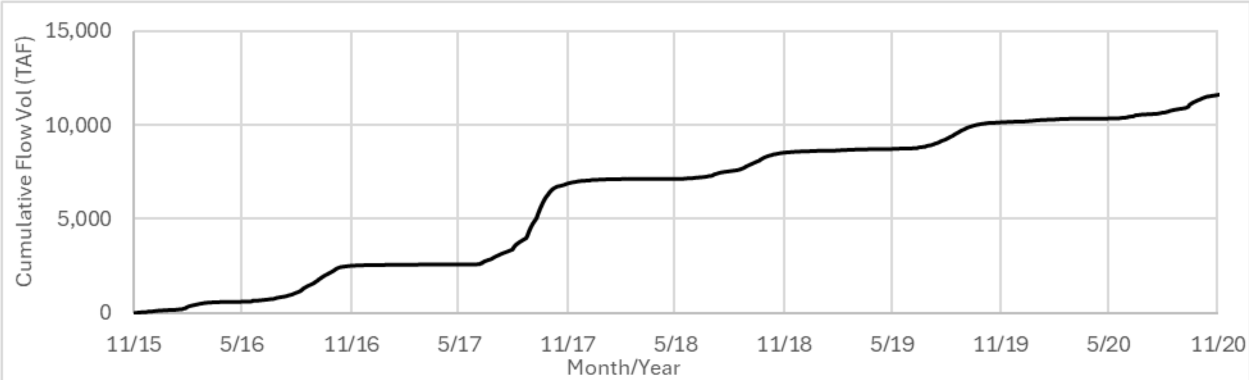

Figure 4-95. Simulated Cumulative Flow Volume at GG-1 (thousand acre-feet, TAF)

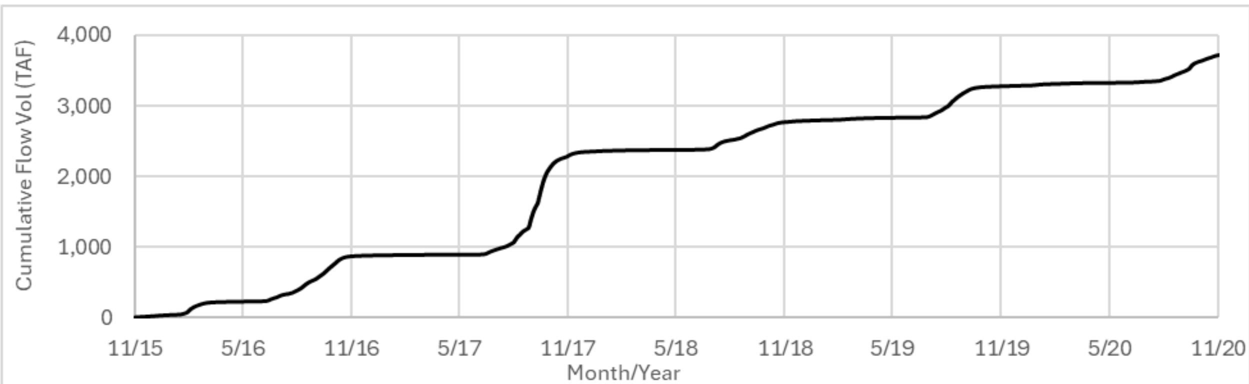

Figure 4-96. Simulated Cumulative Flow Volume at the Kehl Canal Weir (thousand acre-feet, TAF)

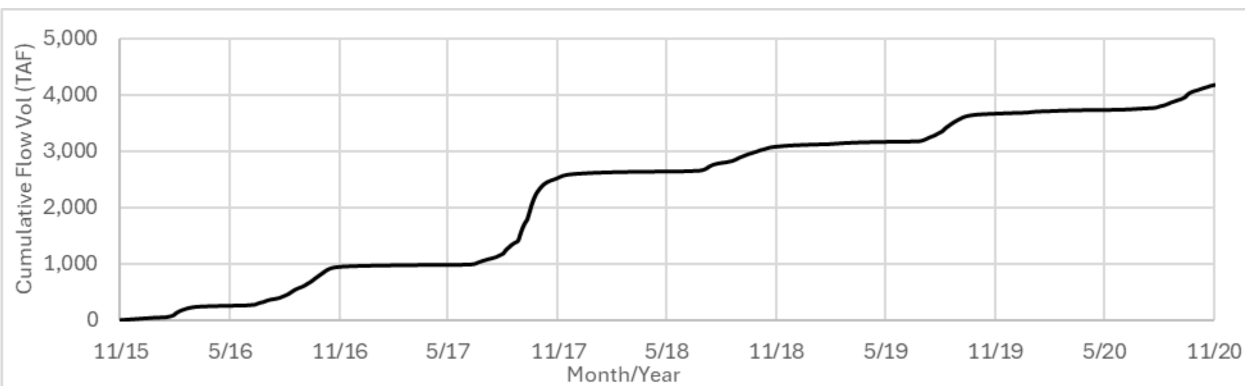

Figure 4-97. Simulated Cumulative Flow Volume at the Imperial River just east of I-75 (thousand acre-feet, TAF)

In addition to the flows, changes to the minimum groundwater levels presented in the water supply section below can also be quantified for increased susceptibility to saltwater intrusion.

4.4 Water Supply Performance Measures

One of the goals of the CWI project is ensure that projects are not going to impact water supply. Groundwater levels and flows at selected locations near the larger public water supply wells were extracted as to establish a comparison measure for the FCM baseline and alternatives. Sources for these well fields vary and include wells in the Surficial, Intermediate and Floridan Aquifers, most have wells that pump from either the water table, the Lower Tamiami, and/or the Sandstone aquifer. Thus, results for these three layers in the model were extracted.

Figure 4-98 shows selected locations that serve as indicators to changes in water supply nearby public water supply wells. The location of the selected flow extraction lines were based on the groundwater flow vector field of the Lower Tamiami aquifer where more pumping activity was evident from the higher flow directions towards the pumping wells. Table 4-15 shows the average and minimum water levels for the three aquifer layers in the model during the months of January to May, which are the month with the lower groundwater levels.

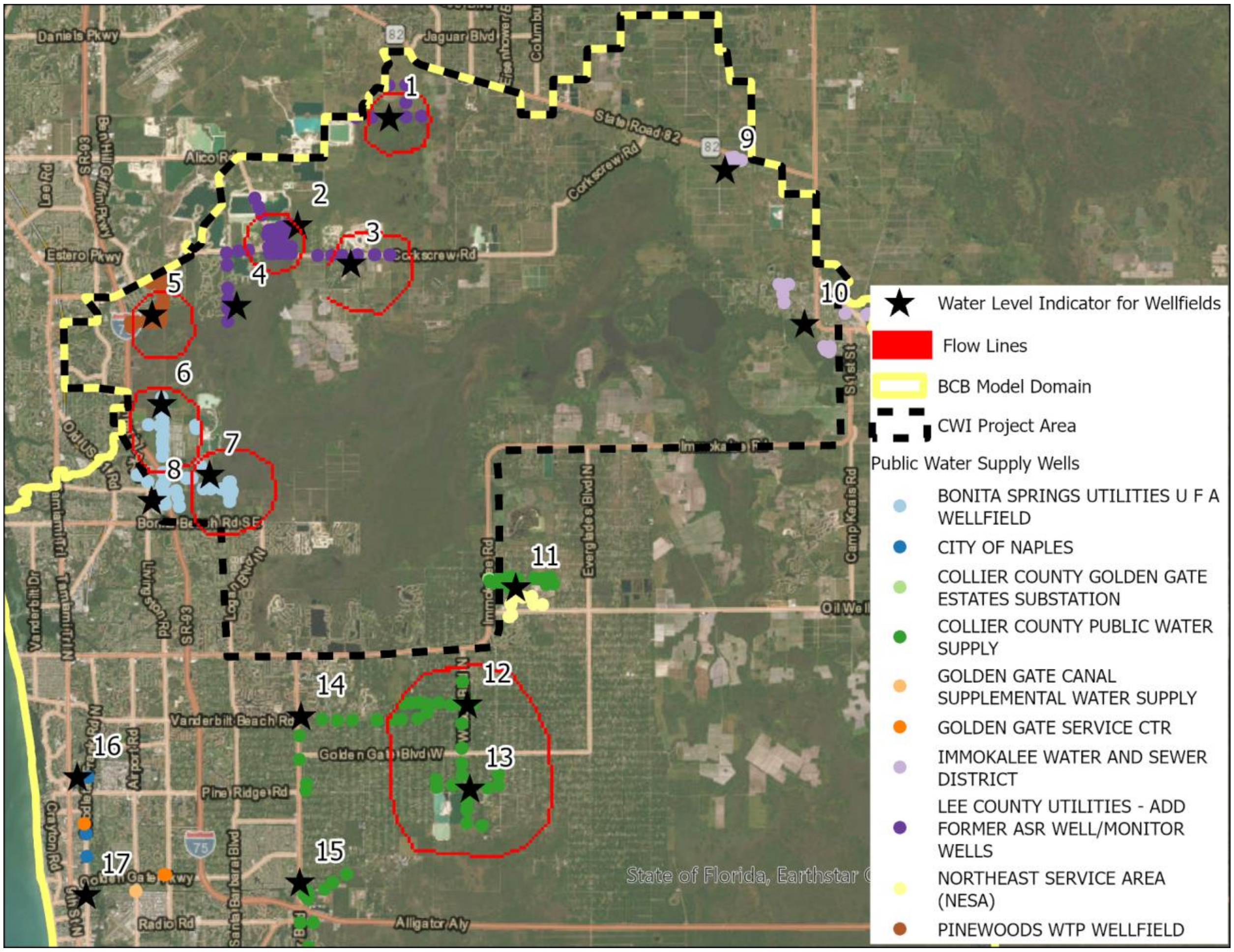

Figure 4-98. Water Level and Flow Indicator Locations for Changes to Water Supply

Table 4-15. Simulated Average During the First Five Months of the Year and Minimum Water Levels (ft-NAVD)

Point ID	Water Table Aquifer		Lower Tamiami Aquifer		Sandstone Aquifer	
	Average	Minimum	Average	Minimum	Average	Minimum
1	24.2	21.3	24.0	20.8	-2.5	-18.7
2	18.1	16.1	18.0	15.8	-11.5	-25.0
3	18.6	12.8	18.4	13.0	3.1	-7.5

Point ID	Water Table Aquifer		Lower Tamiami Aquifer		Sandstone Aquifer	
	Average	Minimum	Average	Minimum	Average	Minimum
4	14.4	12.3	14.5	12.5	-3.3	-12.5
5	12.7	9.9	12.2	9.4	2.0	-2.9
6	11.0	8.9	9.6	7.6	8.2	5.6
7	9.4	6.9	7.4	5.5	7.5	5.6
8	5.7	3.9	5.6	3.8	5.8	3.7
9	28.2	26.7	27.9	26.2	3.2	-4.8
10	19.7	17.5	19.6	17.4	8.7	1.2
11	9.8	8.4	10.6	7.8	8.3	-0.8
12	7.0	4.0	5.8	2.9	6.0	3.2
13	6.4	3.9	4.0	1.3	4.4	1.8
14	7.4	5.1	6.8	4.5	6.9	4.7
15	5.5	4.2	5.4	3.7	5.5	3.7
16	3.2	-0.9	2.6	-0.2	0.8	-1.9
17	-0.7	-3.9	-0.6	-3.7	-2.7	-6.1

Table 4-16. Total Groundwater Flow Volume (Ac-ft) Across the Selected Flow Lines (Figure 4-98)

Month	Water Table Aquifer	Lower Tamiami Aquifer	Sandstone Aquifer
January	799	8,070	748
February	1,491	7,516	564
March	1,209	9,070	963
April	1,564	8,591	819
May	2,058	8,789	815
Total	**7,121**	**42,036**	**3,909**

5.0 Discussion and Overview of Next Phases

The ECM results presented above provide a baseline overview of flood protection, wetland hydrology, and water supply conditions, for the CWI project area prior to 2020.

The surface water results show that the simulated structure operation protocols are functioning properly as intended, which are based on guidance from the BCB water management staff. This was accomplished via several iterative adjustments of the control logic rules, the timing of the gate openings and gate speeds in relation to the model time steps.

The design storm results show locations of canal overtopping simulated along the primary canals that drain the project area. Some of the areas along the Corkscrew Canal that show flooding downstream of Cork3 and Cork2 are near 33rd Avenue and upstream of 41st Avenue. In the Cocohatchee Canal banks are exceeded mostly in areas where the canal levees are lower, close to ground topography, near natural or low density/rural residential areas. In the Kehl Canal, the levee is discontinuous along most of its length and the lower portions of the canal banks are exceeded during all storms. Most of these areas are undeveloped, except for the western urban areas near the Kehl Canal weir. The stages and flow plots show constrictions to flow in the areas north of COCO4 and Cork1. High tailwater levels seem to constraint the flows in Cork1, Cork2, and Cork3. In the Cocohatchee Canal the differences between the headwater and tailwater stages between COCO3, COCO2, and COCO1, as well as discharges, increase towards the west.

Flood maps show areas of flood peak and duration for all four design storms in the Estero Bay, Cocohatchee, FakaUnion, Golden Gate Main, and Trafford watersheds. Within the project area, some of the areas with higher flooding peaks and duration are the 6L Farms, other farm areas north of Corkscrew Road, and in central Corkscrew and some residential areas near the southern portions of Corkscrew. Some of these areas adjacent to Corkscrew experience long duration of flooding, even during the 5-year event. Some of these farm areas will be converted into residential areas for Future Conditions, with a different representation of the storage and drainage based on permit information.

Water depths and hydroperiods maps show the simulated spatial patterns of wet and dry areas in Corkscrew. Factors that contribute to these spatial patterns include: relative changes in topography, land use and water management, wetland flow barriers, ET and water use impacts, and downstream drainage. Overland flow volumes before and after flow barriers provide an insight into these spatial patterns. Results show how overland flow is substantially reduced as it travels between barriers. The barriers force sheet flow movement to be concentrated across the culvert road crossings and then short circuited in preferential pathways further downstream. Spreader canals upstream and downstream of the culvert road crossings should be included in the alternatives that target the improvement of wetland flow across these barriers and adequately represented in the model to see how they impact the sheet flow.

In the next deliverable, the baseline FCM will be developed to include the changes in land use and infrastructure that have occurred since the period of model development (2016 to 2020) and those projected to occur within the project planning period. A sea level rise condition will also be incorporated into the FCM. The planning period horizon and details will be further discussed with the project team and TWG during the FCM development. Since the baseline FCM will be the one used for alternative assessment as part of the FCM deliverable, in the FCM document a qualitative comparison of the ECM results presented here with the FCM baseline can serve to identify the impact of the planning horizon.

6.0 References

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USACE, 2024. Comprehensive Everglades Restoration Plan, Picayune Strand Restoration Project. Final Operating Manual. Jacksonville District U.S. Army Corps of Engineers. July 2024.

Appendix A: Long Term Simulation Stage and Flow Hydrographs

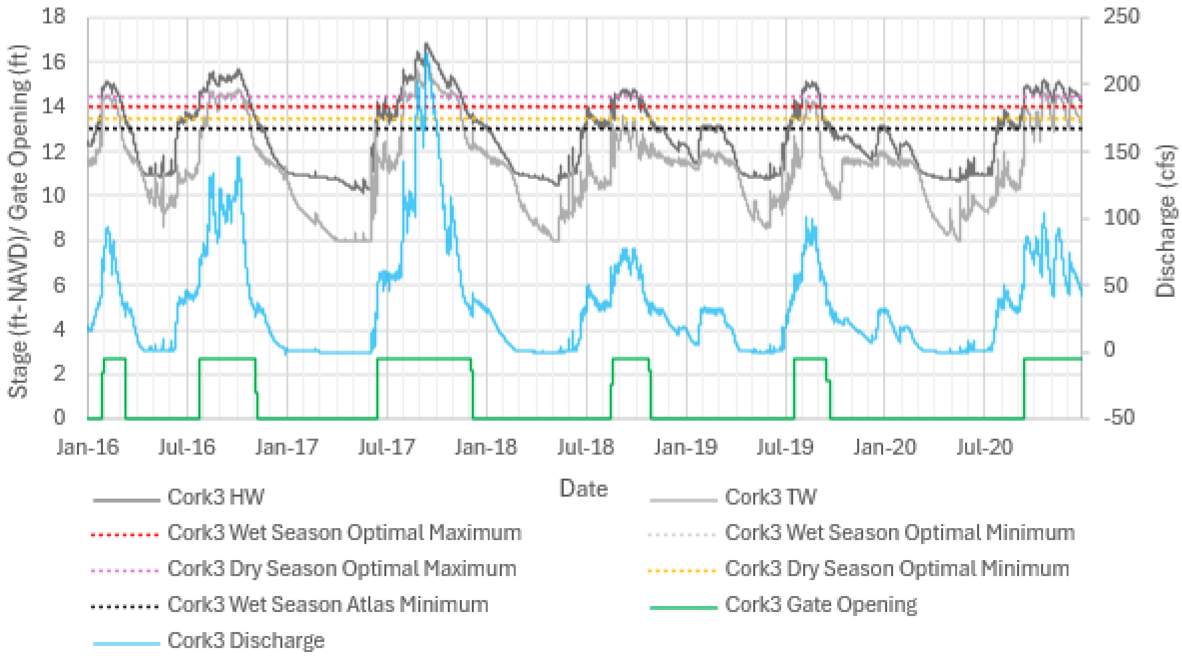

Figure A-1. Long Term Simulation Flow and Stage Hydrographs for Cork 3

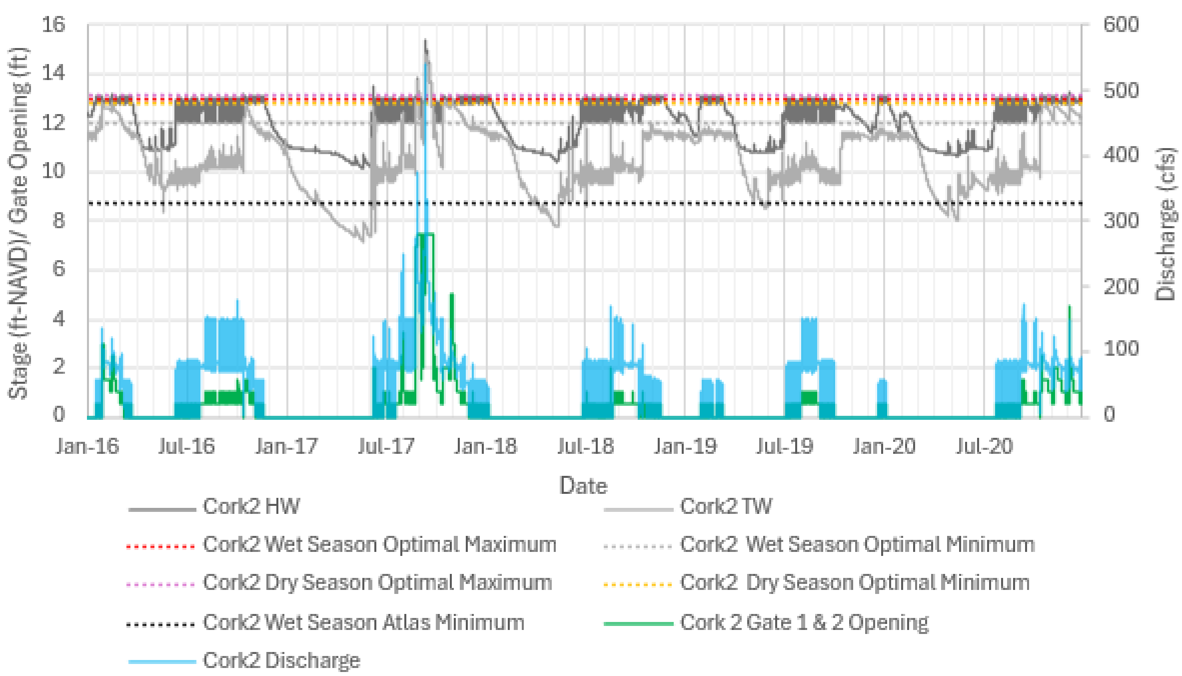

Figure A-2. Long Term Simulation Flow and Stage Hydrographs for Cork 2

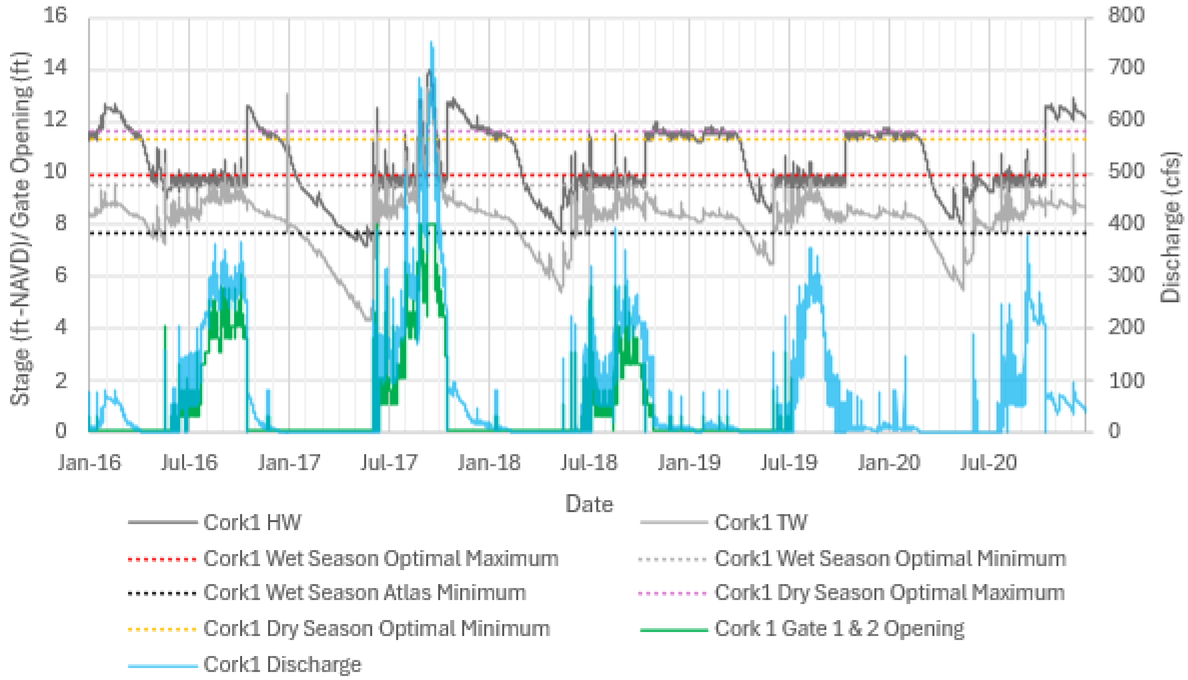

Figure A-3. Long Term Simulation Flow and Stage Hydrographs for Cork 1.

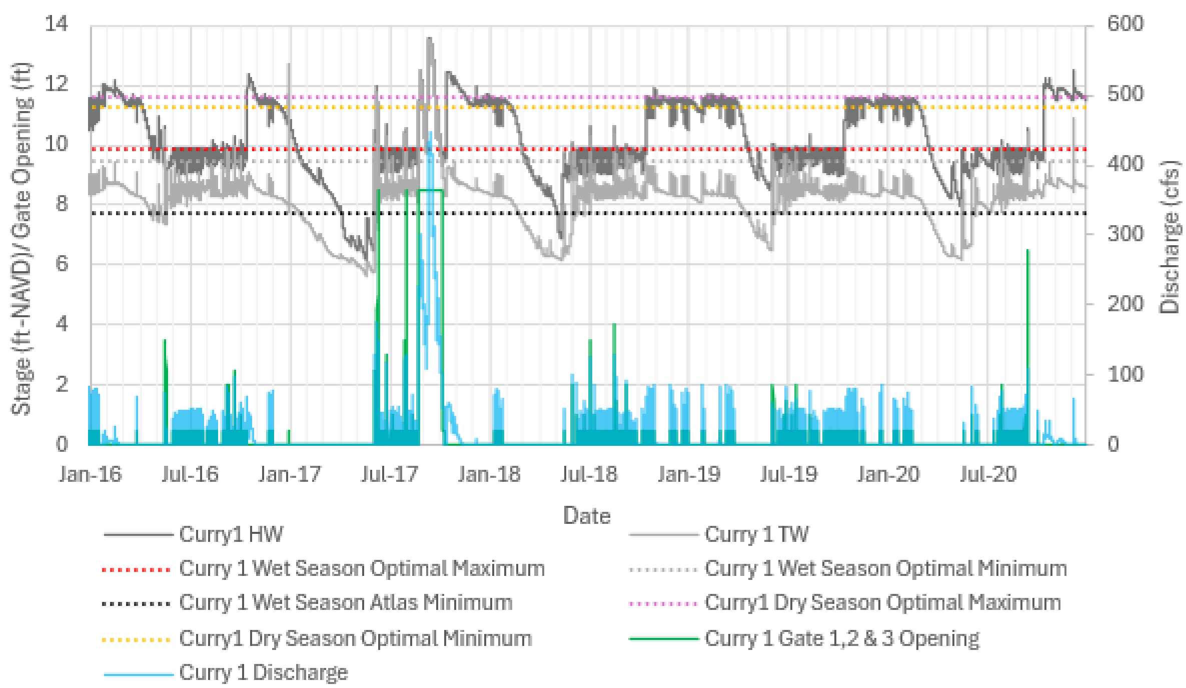

Figure A-4. Long Term Simulation Flow and Stage Hydrographs for Curry 1.

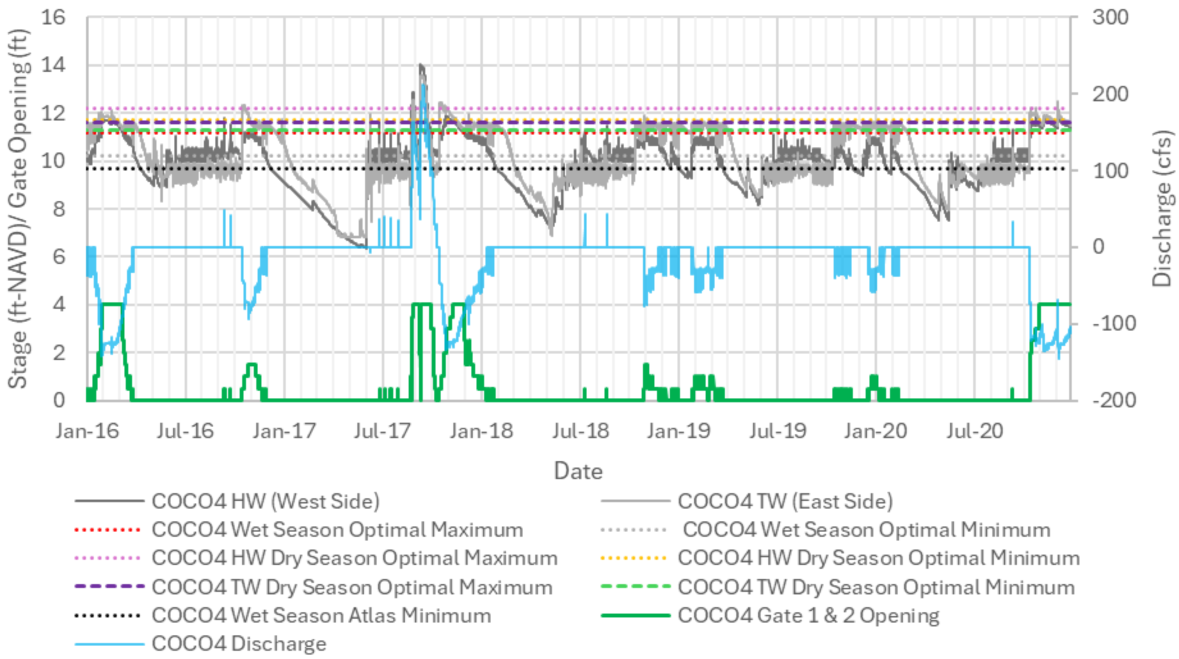

Figure A-5. Long Term Simulation Flow and Stage Hydrographs for COCO4.

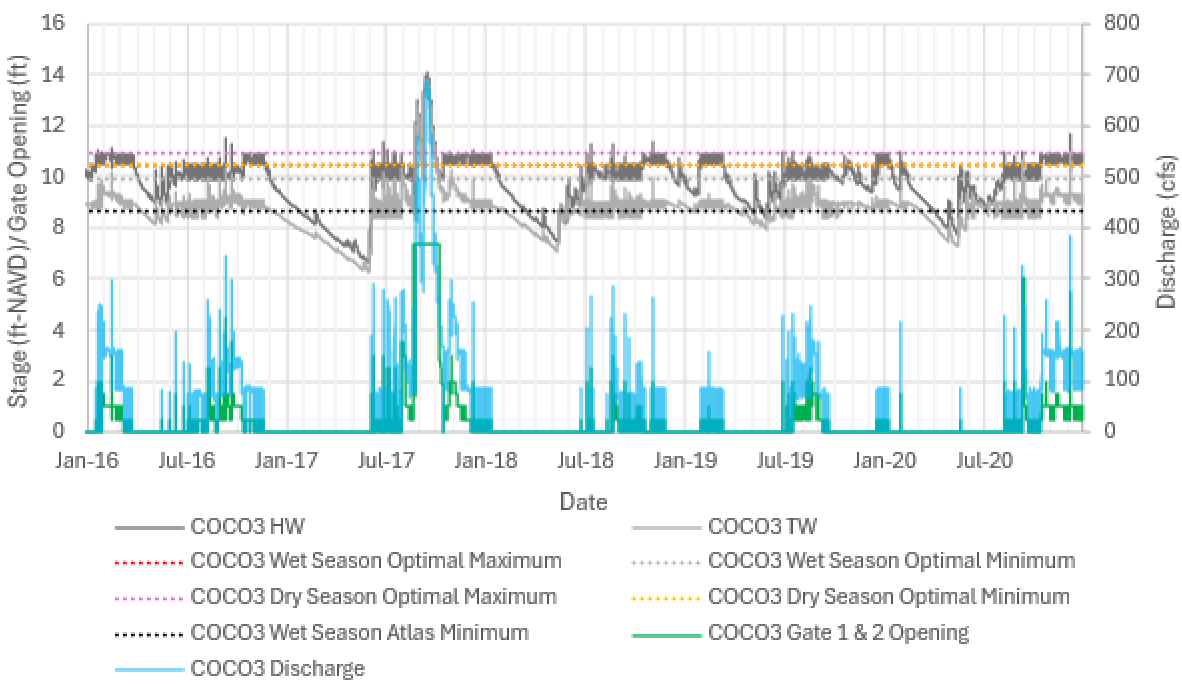

Figure A-6. Long Term Simulation Flow and Stage Hydrographs for COCO3.

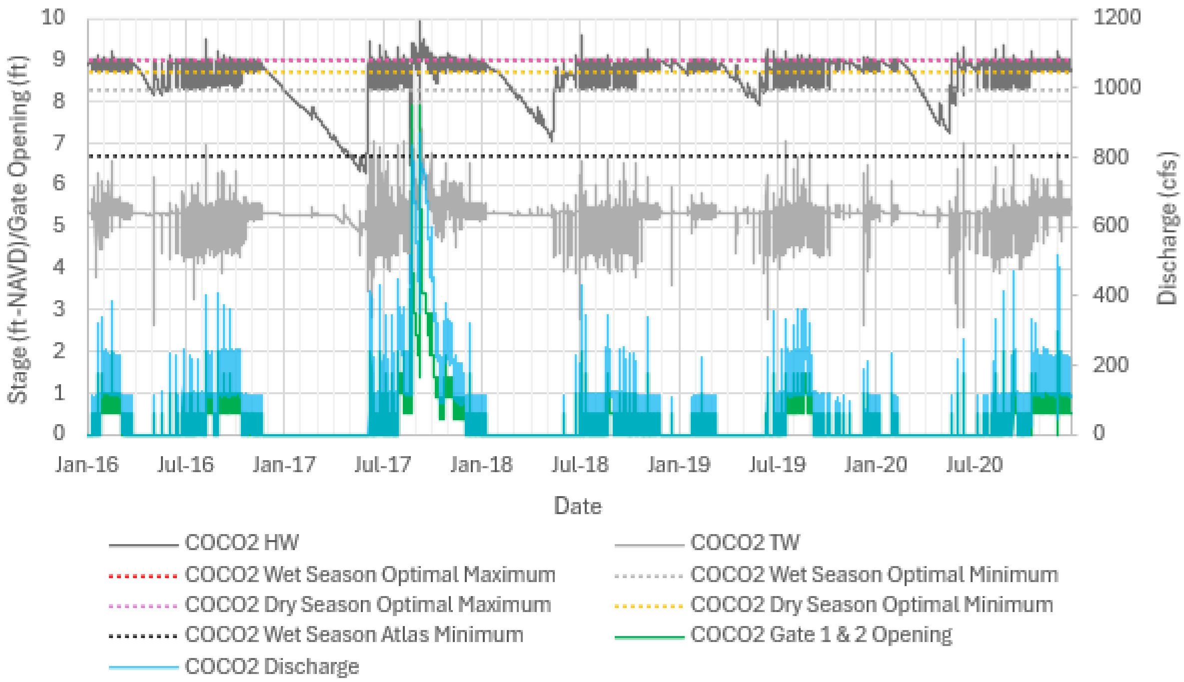

Figure A-7. Long Term Simulation Flow and Stage Hydrographs for COCO2.

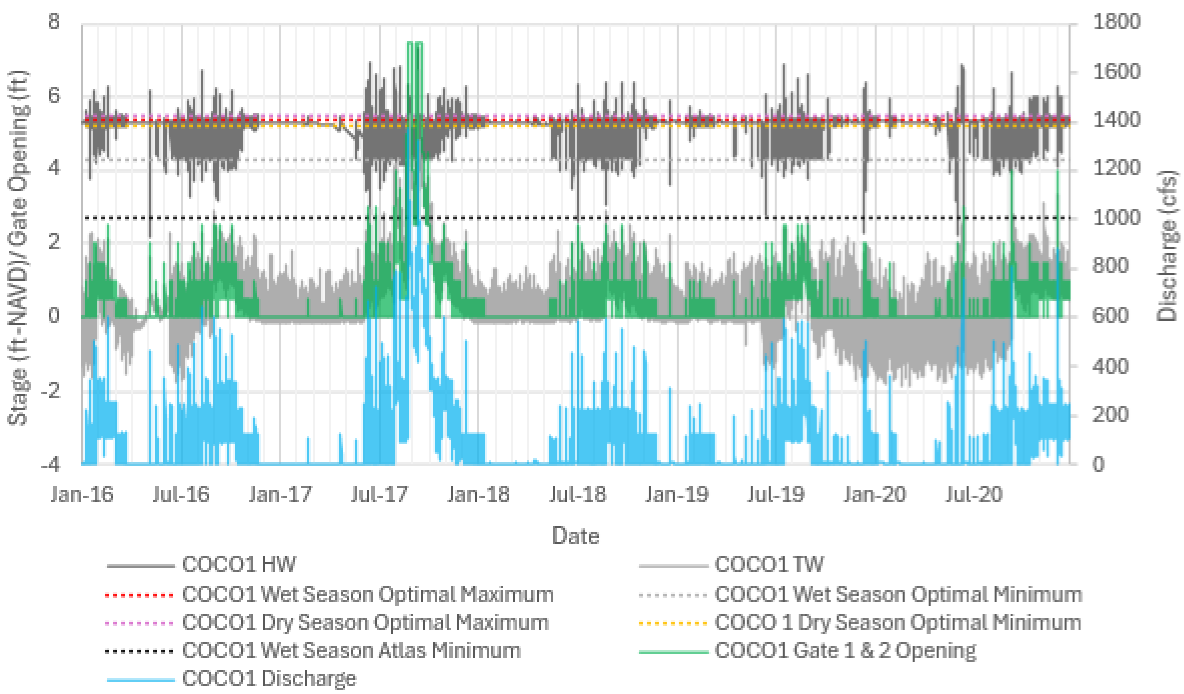

Figure A-8. Long Term Simulation Flow and Stage Hydrographs for COCO1.

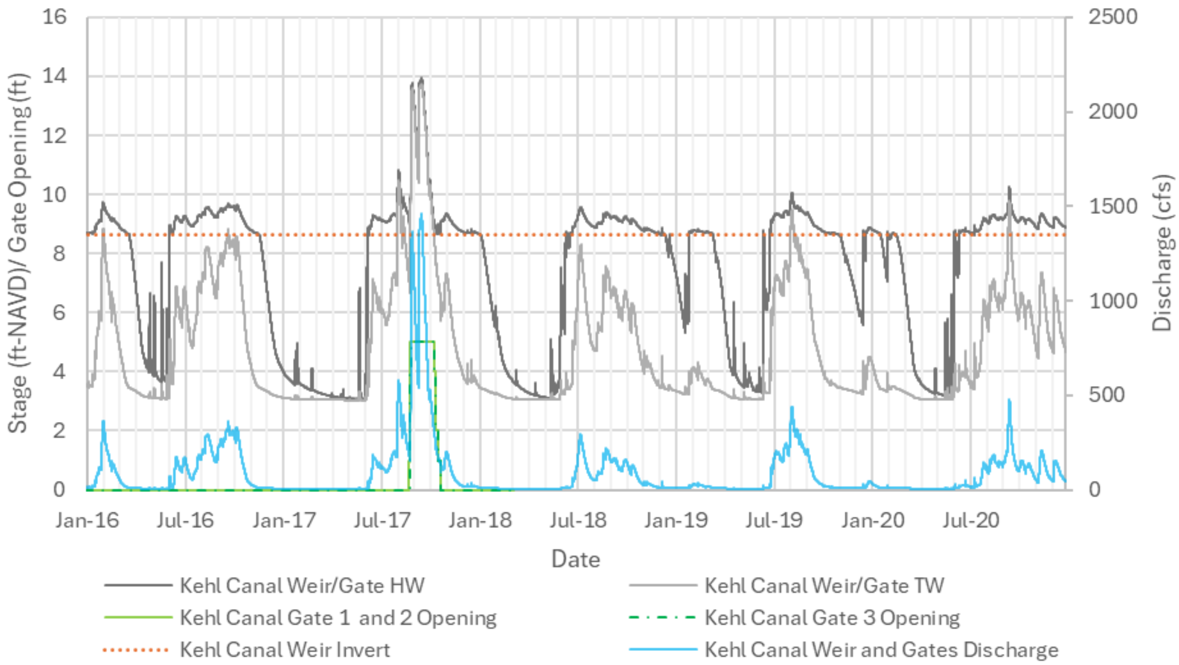

Figure A-9. Long Term Simulation Flow and Stage Hydrographs for Kehl Canal Weir/Gate.

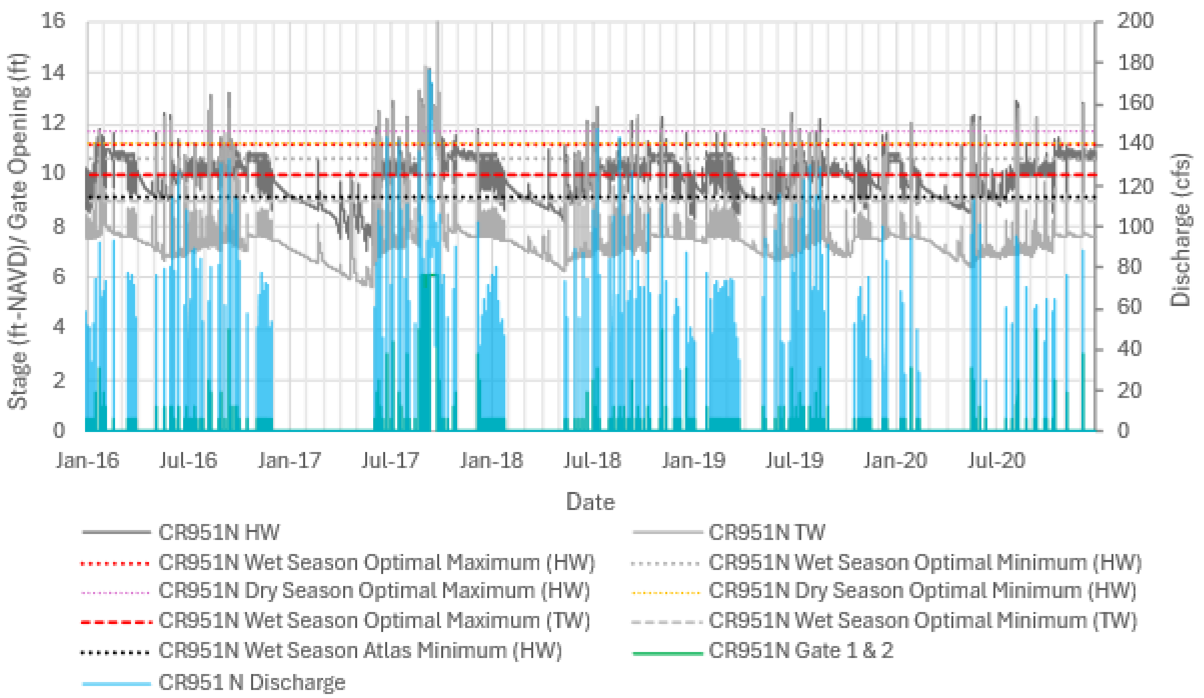

Figure A-10. Long Term Simulation Flow and Stage Hydrographs for CR951N

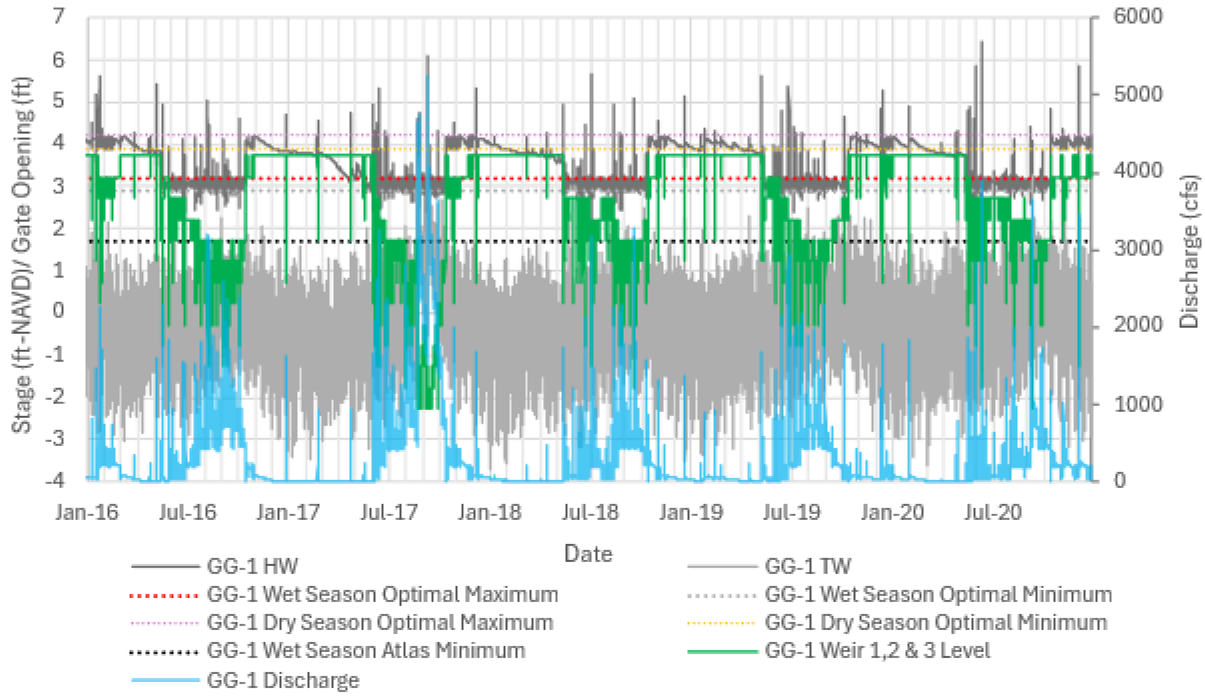

Figure A-11. Long Term Simulation Flow and Stage Hydrographs for GG1

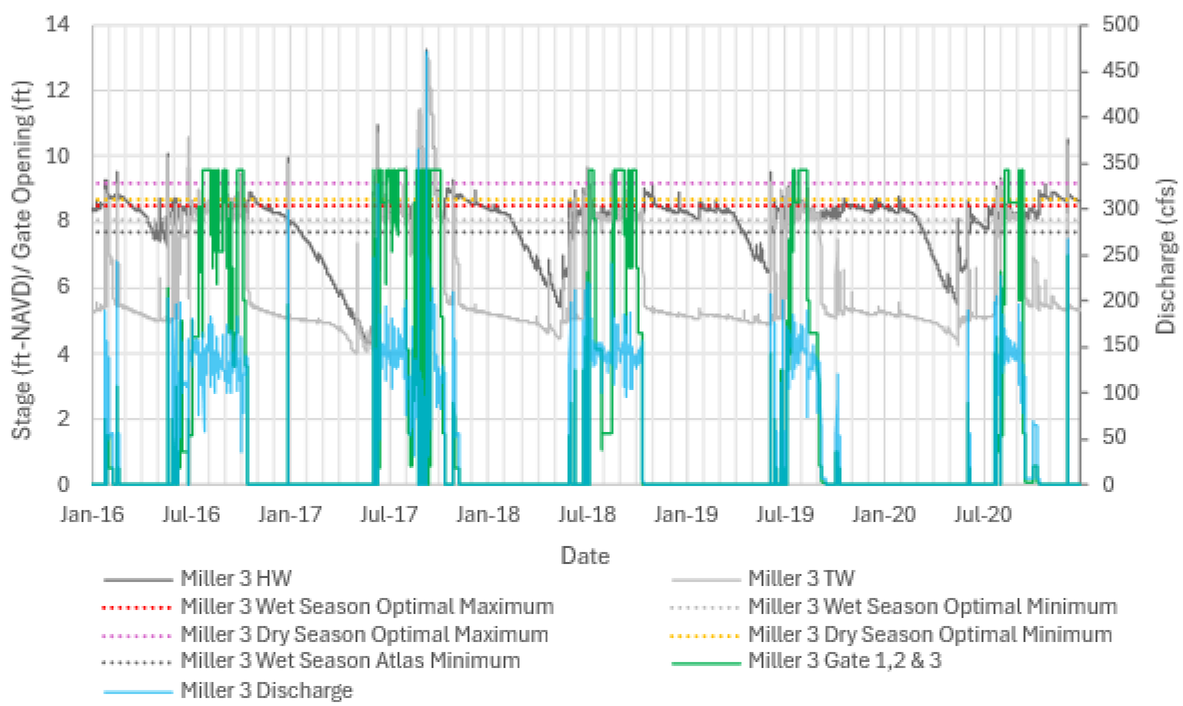

Figure A-12. Long Term Simulation Flow and Stage Hydrographs for Miller 3

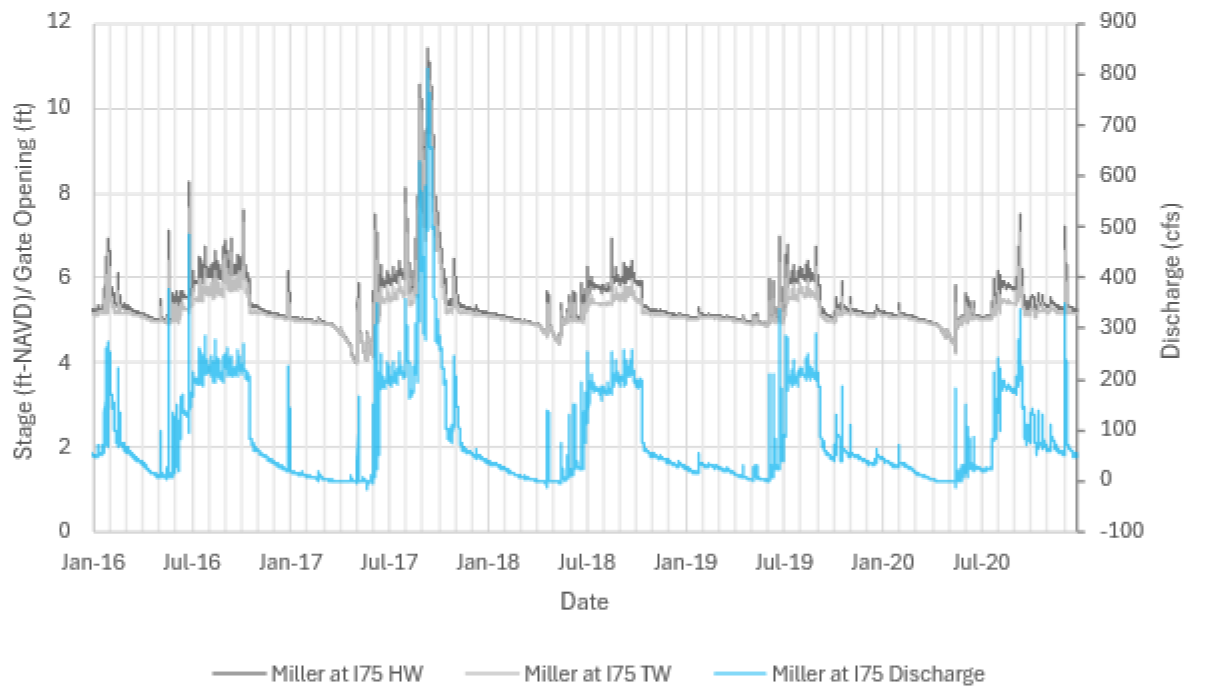

Figure A-13. Long Term Simulation Flow and Stage Hydrographs for Miller Canal Bridge at I-75

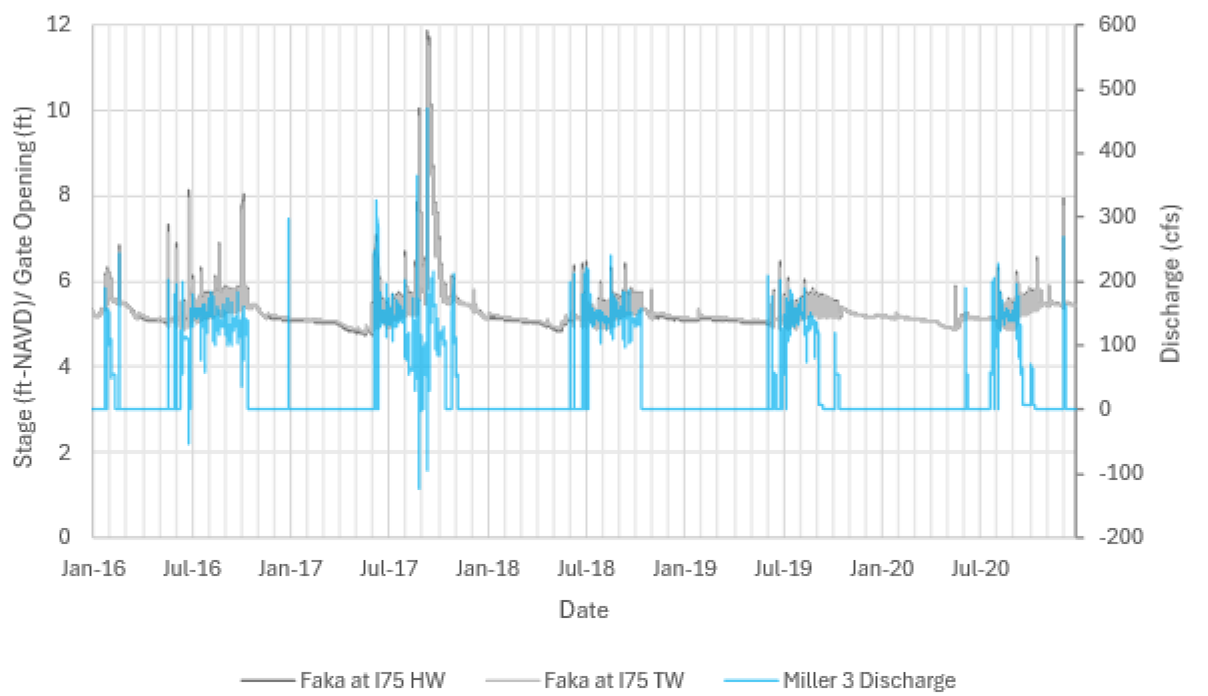

Figure A-14. Long Term Simulation Flow and Stage Hydrographs for Faka Union Canal Bridge at I-75

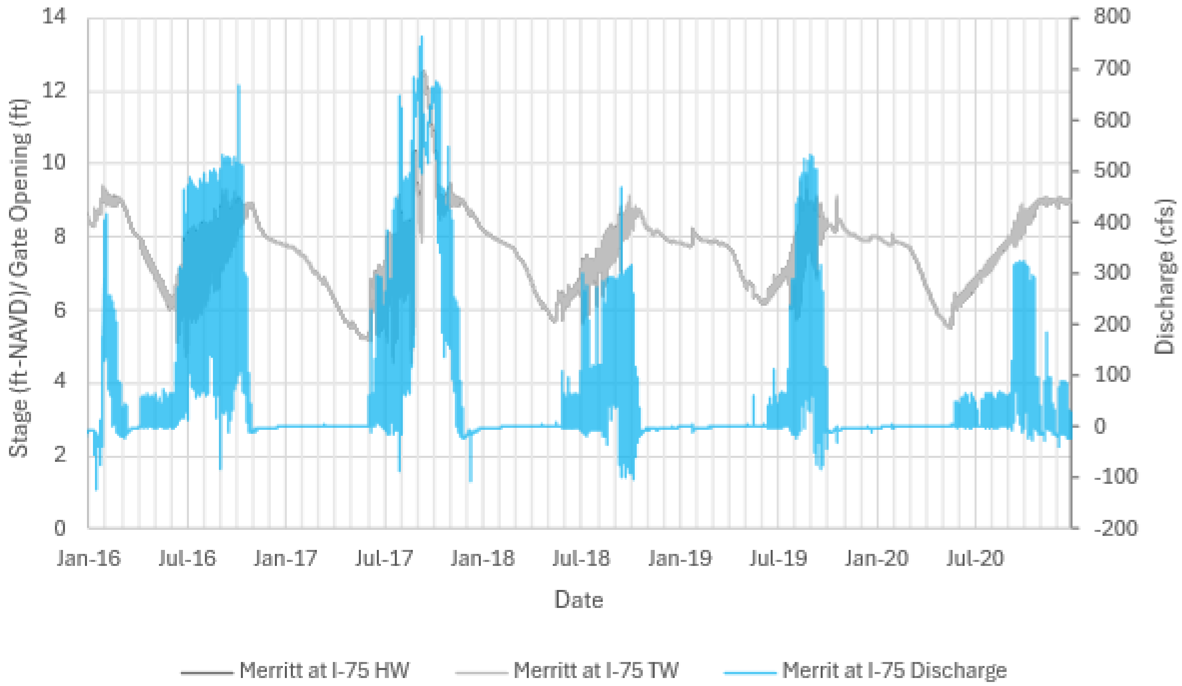

Figure A-15. Long Term Simulation Flow and Stage Hydrographs for Merrit Canal Bridge at I-75

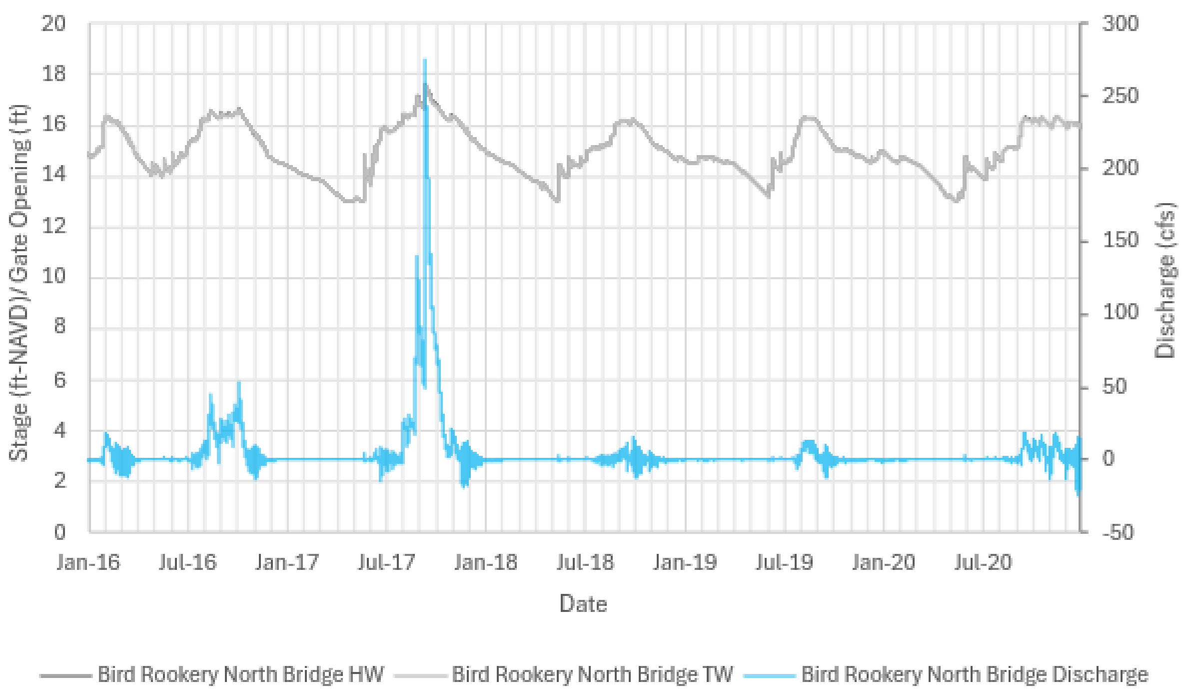

Figure A-16. Long Term Simulation Flow and Stage Hydrographs for Bird Rookery North Bridge

Stages and flow from the connections south of Shady Hollow Boulevard to the Corkscrew Canal downstream of Cork3 (Figure A-17) are shown in the figures Figure A-18 and Figure A-19.

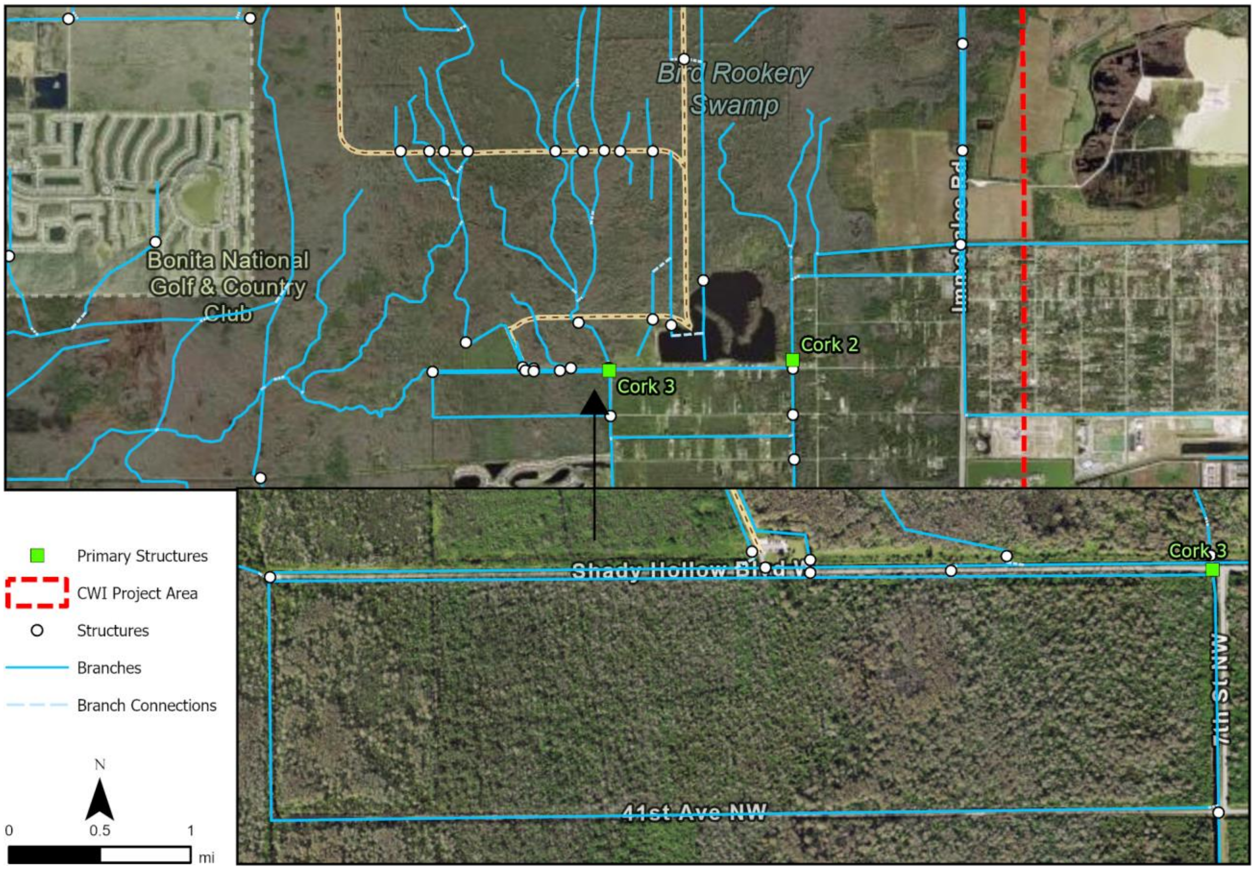

Figure A-17. Surface Water Discharge Locations to Corkscrew Canal, South of Cork3

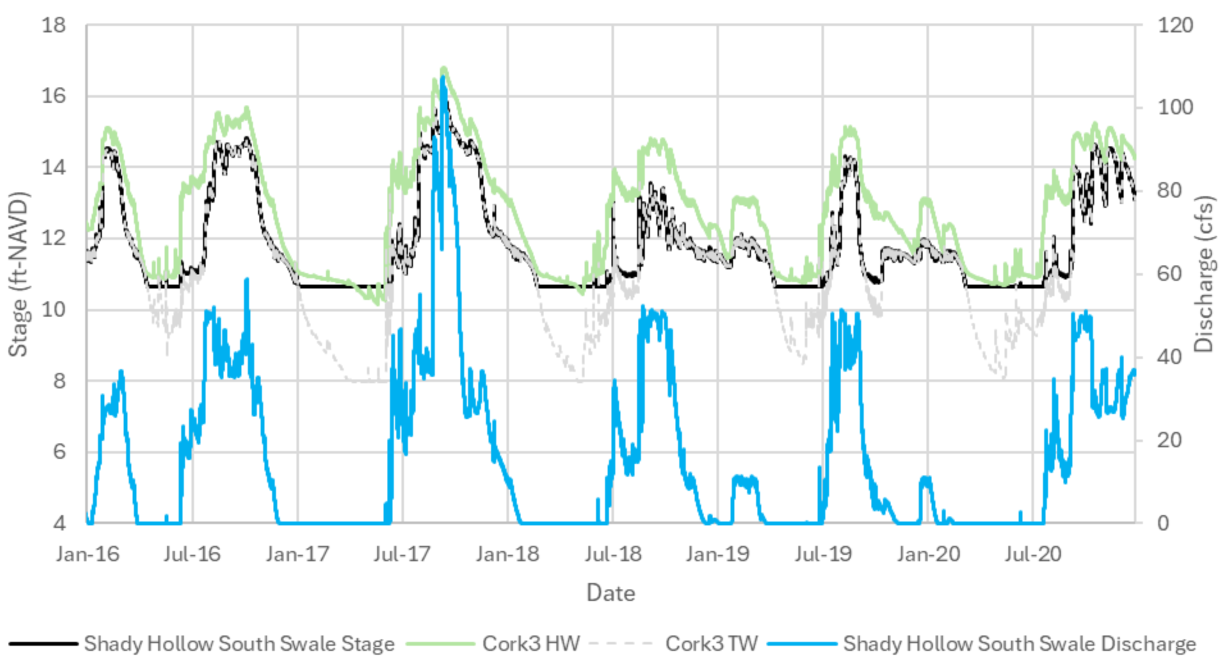

Figure A-18. Long Term Simulation Flow and Stage Hydrographs for Shady Hollow Boulevard South Swale

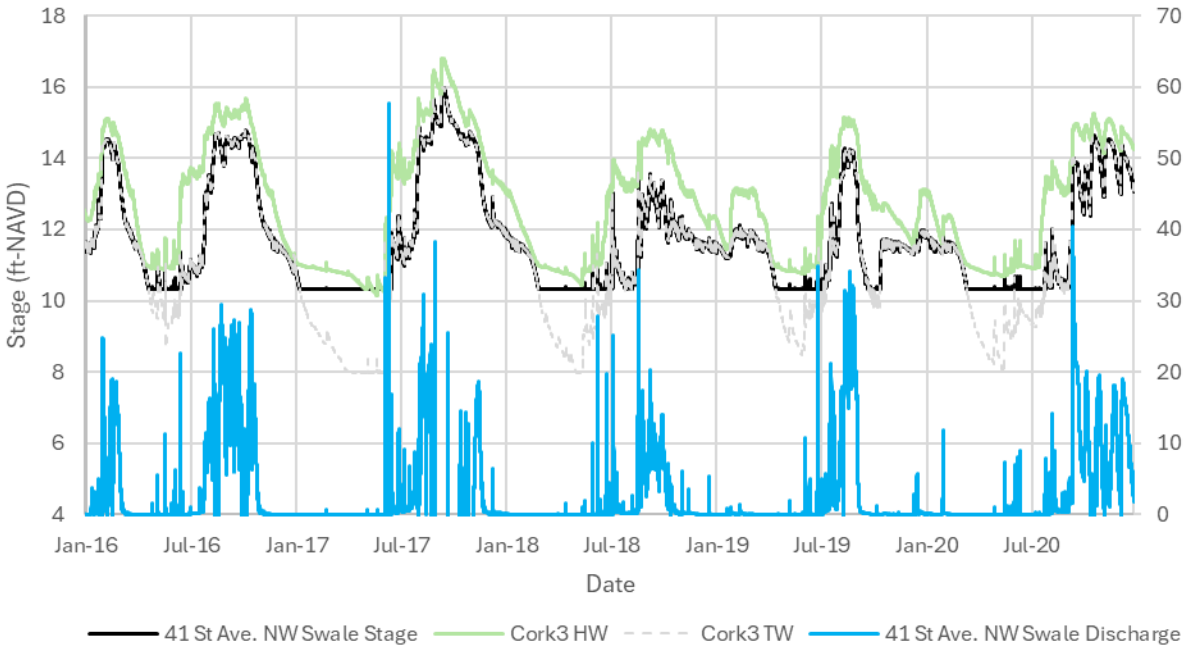

Figure A-19. Long Term Simulation Flow and Stage Hydrographs for the 41st Ave. NW North Swale

Stages and flow to the Cocohatchee Canal from the north are shown in the plots below. The locations are show in Figure A-20.

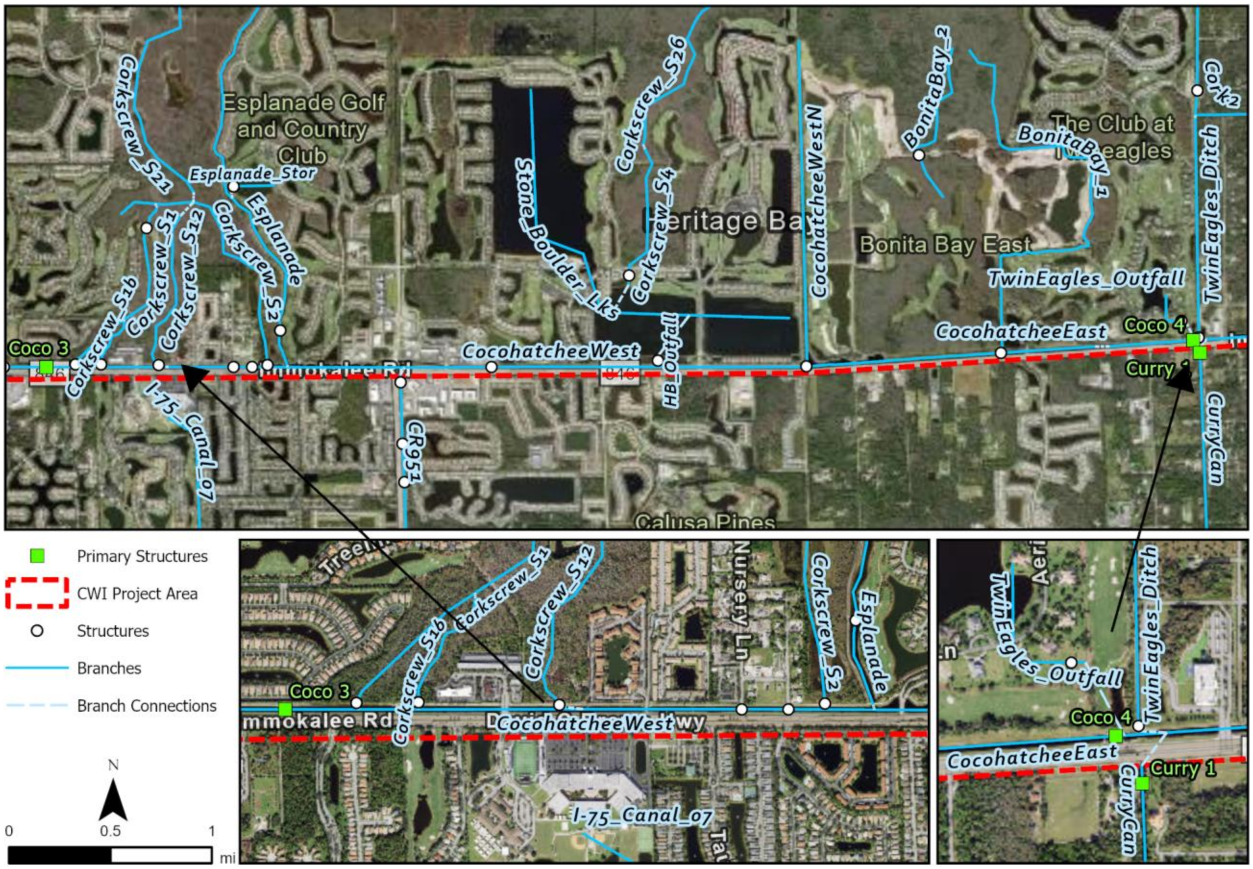

Figure A-20. Surface Water Discharge Locations to Cocohatchee Canal from COCO4 to COCO3

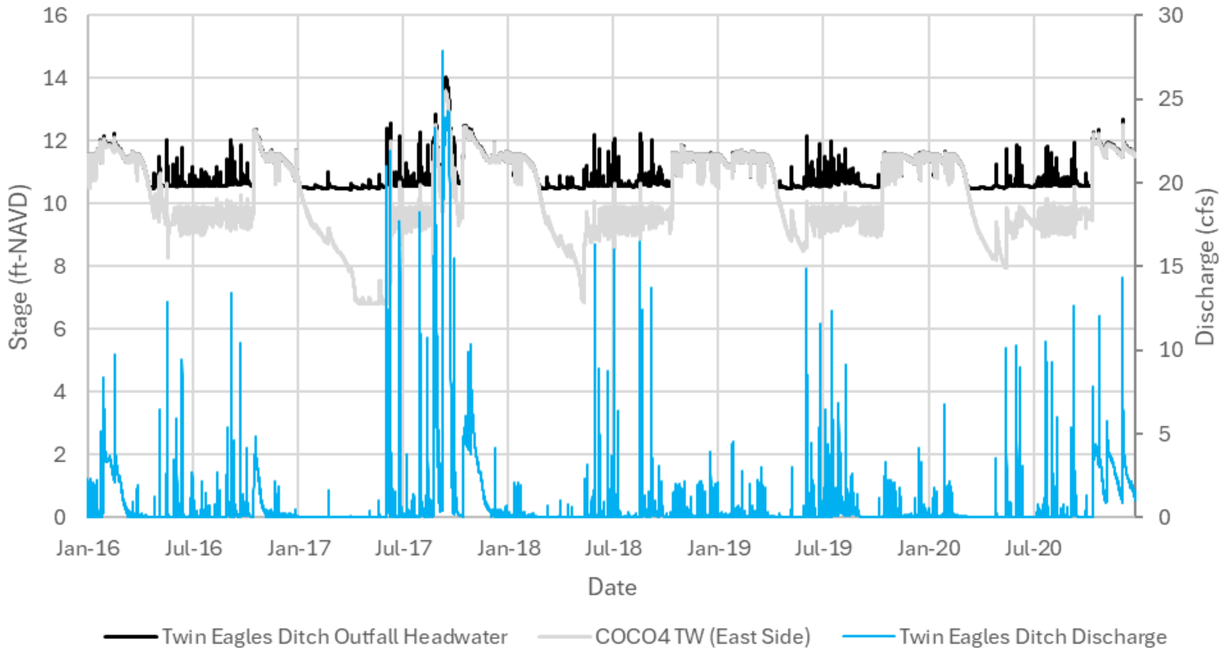

Figure A-21. Long Term Simulation Flow and Stage Hydrographs for Twin Eagles Ditch, East of COCO 4

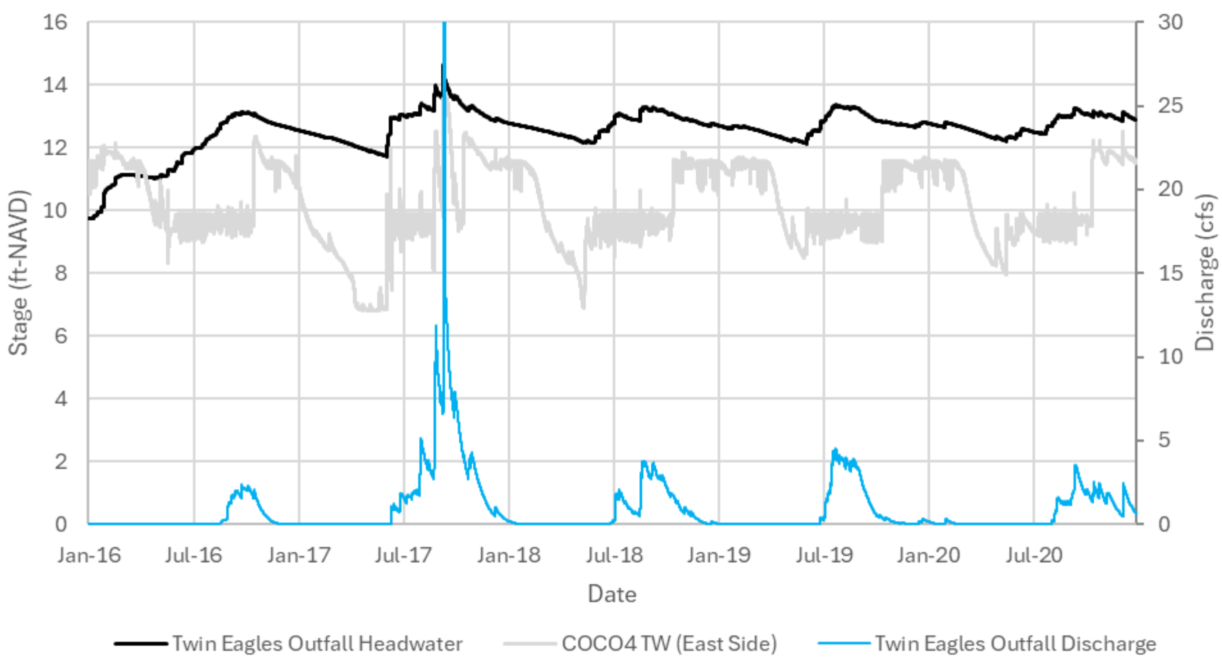

Figure A-22. Long Term Simulation Flow and Stage Hydrographs for Twin Eagles Outfall, East of COCO 4.

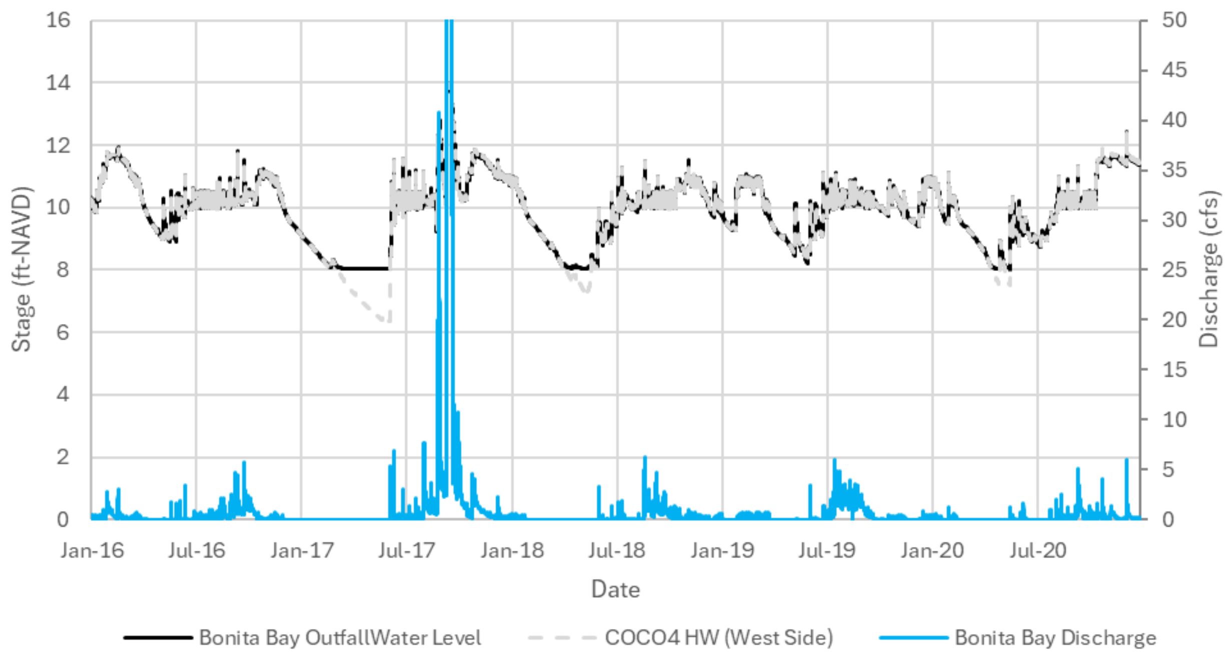

Figure A-23. Long Term Simulation Flow and Stage Hydrographs for Bonita Bay, West of COCO4

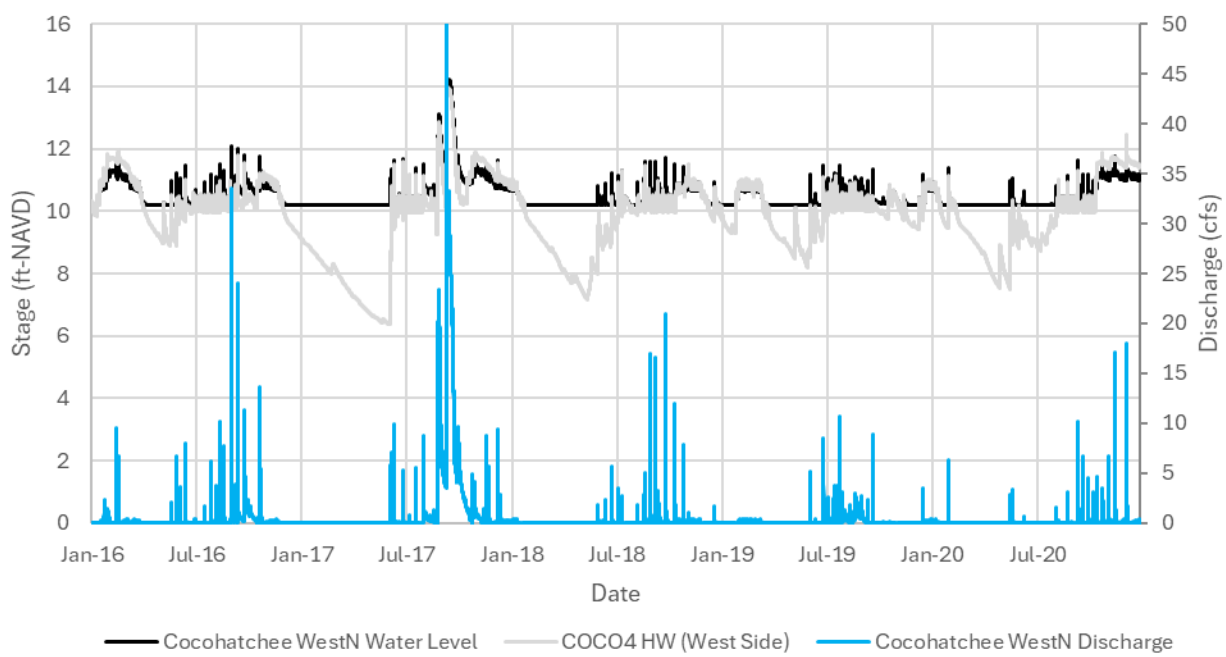

Figure A-24. Long Term Simulation Flow and Stage Hydrographs for Cocohatchee West N, West of COCO4

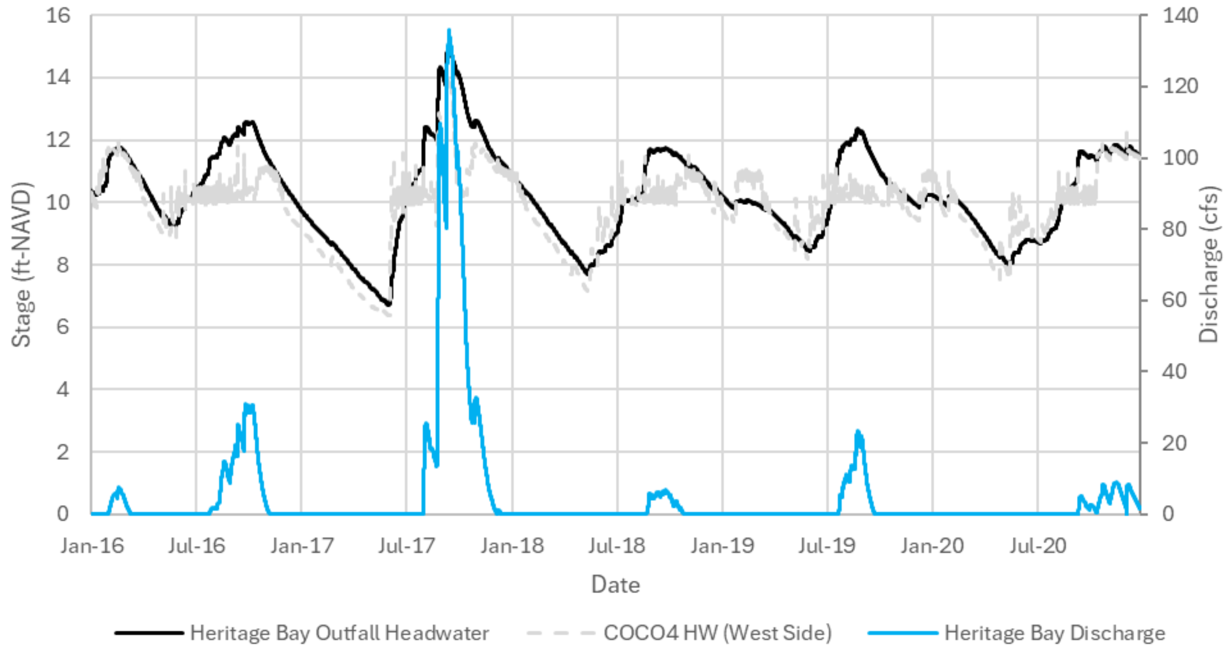

Figure A-25. Long Term Simulation Flow and Stage Hydrographs for Heritage Bay, West of COCO4

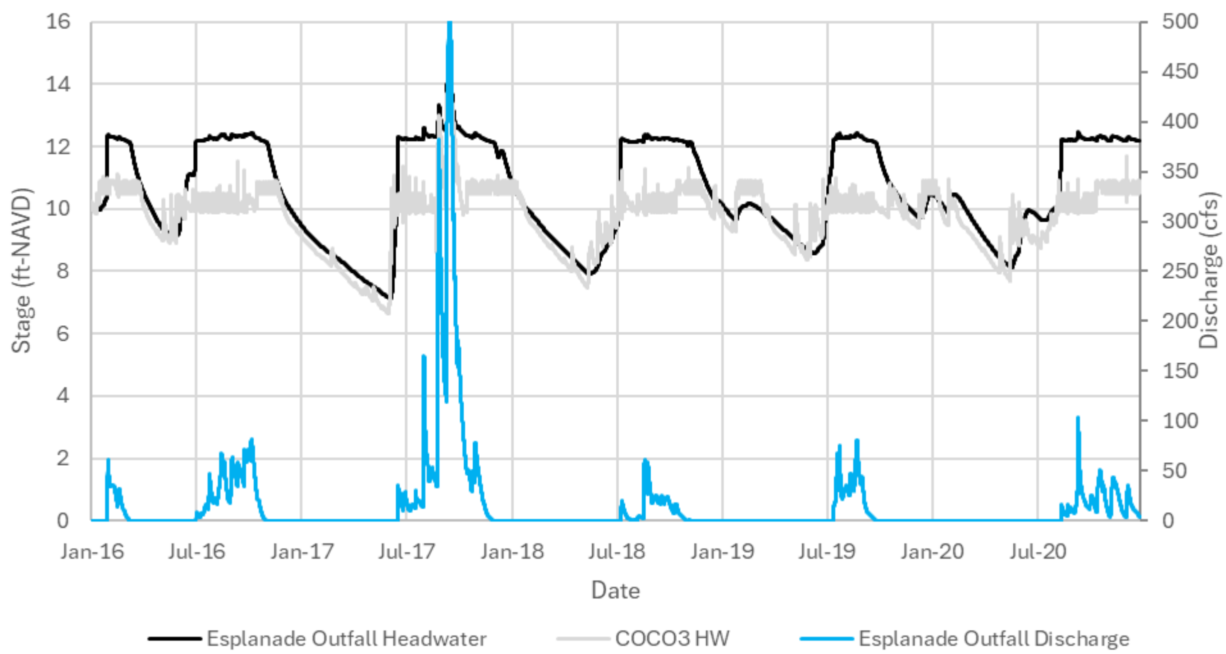

Figure A-26. Long Term Simulation Flow and Stage Hydrographs for Esplanade, East of COCO3.

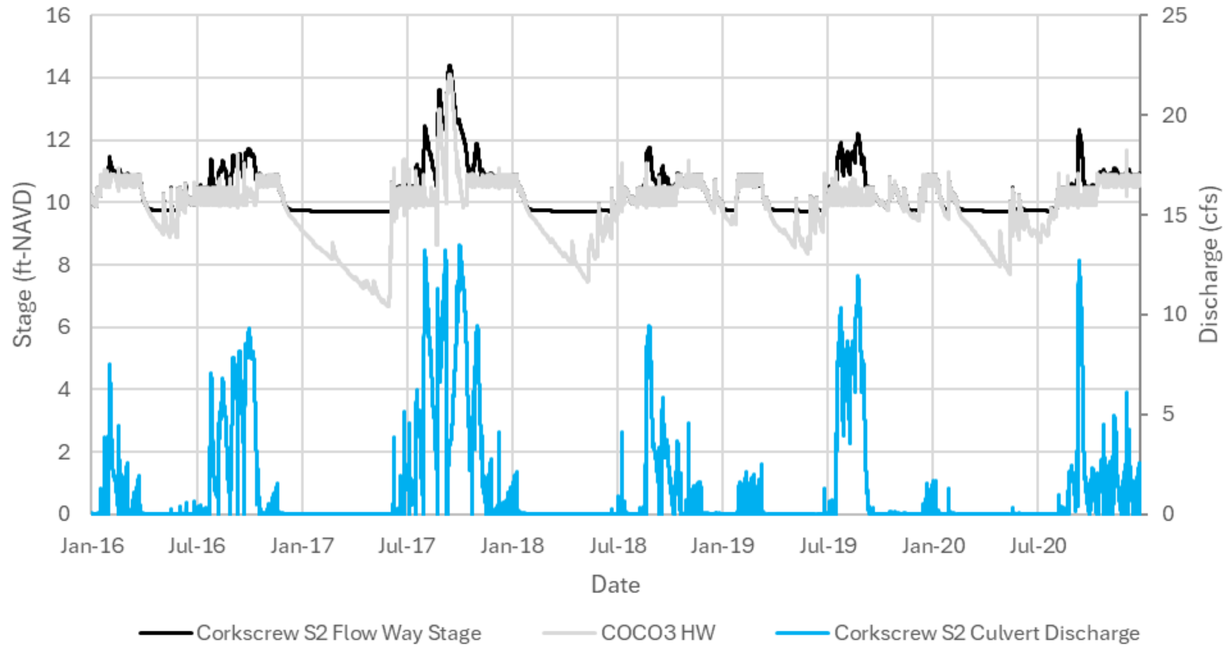

Figure A-27. Long Term Simulation Flow and Stage Hydrographs for Corkscrew S2 Flow Way, East of COCO3.

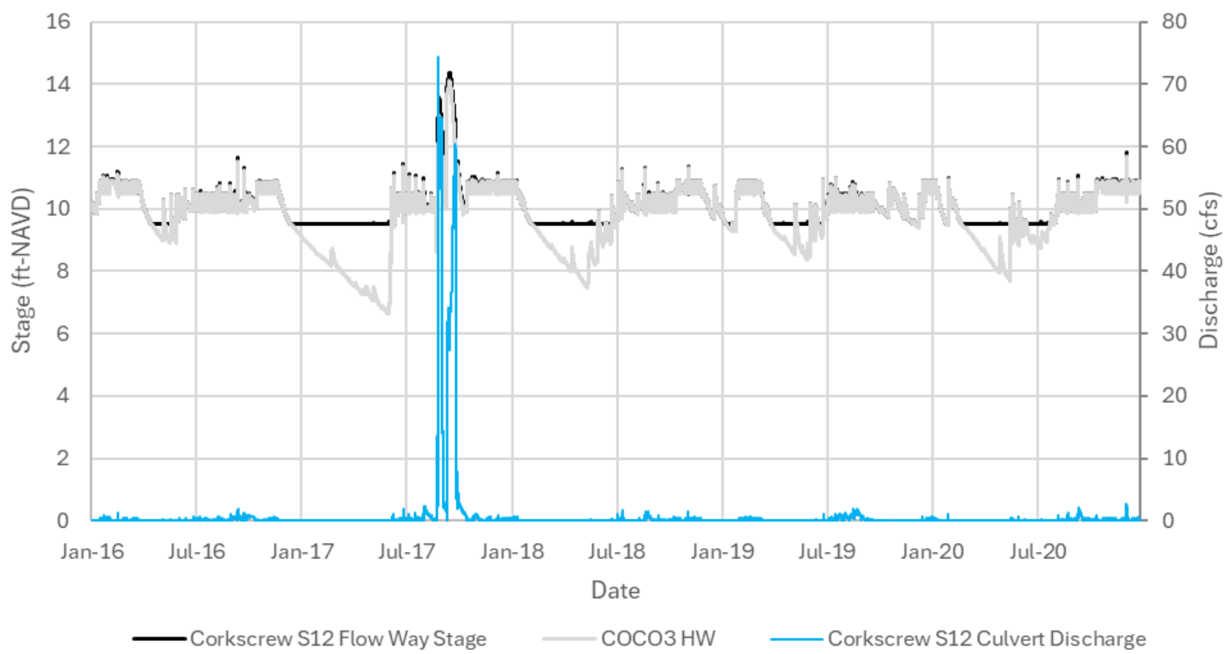

Figure A-28. Long Term Simulation Flow and Stage Hydrographs for Corkscrew S12 Flow Way, East of COCO3.

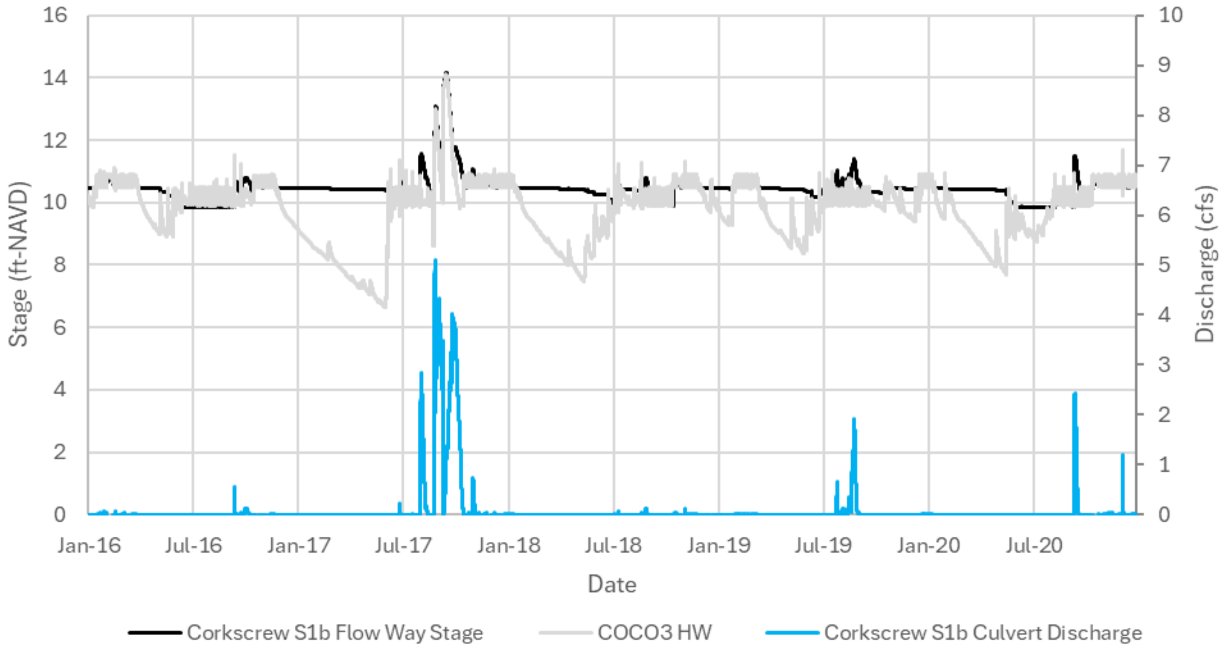

Figure A-29. Long Term Simulation Flow and Stage Hydrographs for Corkscrew S1b Flow Way, East of COCO3.

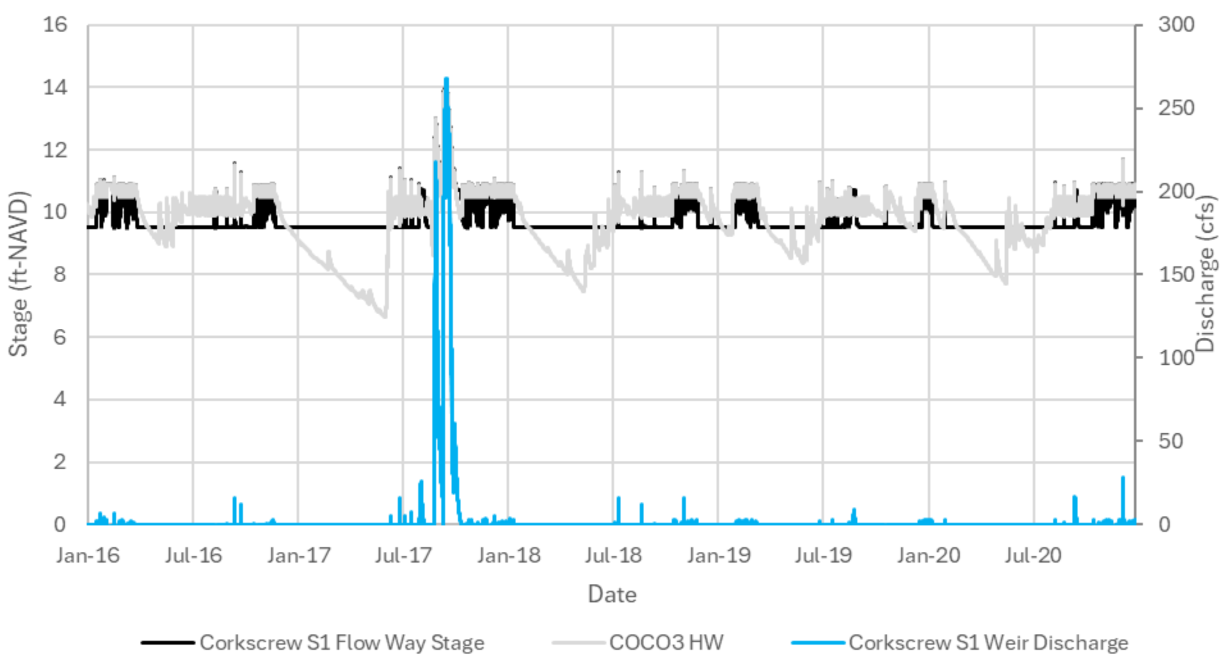

Figure A-30. Long Term Simulation Flow and Stage Hydrographs for Corkscrew S1 Flow Way, East of COCO3.