DEPARTMENT OF THE ARMY PERMIT APPLICATION

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COLLIER COUNTY
COMPREHENSIVE WATERSHED IMPROVEMENT PLAN
COLLIER COUNTY, FLORIDA

SUPPLEMENTAL INFORMATION
ATTACHMENT 5

HYDROLOGIC AND HYDRAULIC MODELING NARRATIVE

TAYLOR ENGINEERING, INC.

Collier County Watershed Improvement Project

Attachment 5 - Hydrologic and Hydraulic Modeling Narrative

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Collier County Watershed Improvement Project

Complete Hydrologic and Hydraulic Modeling Report

<u>Sub-sections:</u>

1. Golden Gate Canal Water Availability Analysis
2. Long Term Model for Vegetation Analysis
3. Design Storm Model and Impacts on Water Levels

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Introduction

Collier County Comprehensive Watershed Improvement Project (CWIP) aims to partly enhance the hydrology of Belle Meade Forest that was historically part of a much larger Rookery Bay Watershed draining from the north. Urban development and construction of I-75 cut off the northern third of the watershed, resulting in reduced freshwater flows to Rookery Bay and increased freshwater flows to Naples Bay. The CWIP aims to partly restore the historic hydrology by diverting water from Golden Gate Canal (GGC) that drains into Naples Bay and discharging the diverted water into Belle Meade Forest that drains into Rookery Bay. An integrated surface- and groundwater model, known as MIKE SHE/MIKE 11, was developed to simulate the impacts of the project on water availability in the GGC, forest, on- and off-site developments and the drainage system within the project area. For hydrologic modeling purposes, the project objectives were categorized into the following three interrelated tasks:

- To determine the availability of water in Golden Gate Canal and its operational impacts on water availability to downstream water users.
- To determine how the additional water will impact overall wetland hydrology in the Belle Meade Forest area.
- To determine the impacts of the project on existing infrastructure and determine the adequacy of proposed infrastructure during design storms.

To meet these objectives, both regional- and local-scale hydrologic modeling was performed. The regional hydrologic models used in this project evolved from the existing Big Cypress Basin (BCB) model and the local-scale model evolved from the existing Henderson Creek model. This document provides pertinent details on model development and summarizes model simulated outputs. Table below summarizes all the models used in this study.

Sub-sect. #	Brief Description	Purpose for Model Development	Sim. / Calib. Period	Model Scale
Section 1: Water Availability in Golden Gate Canal				
1.3	BCB Current Conditions: GGC re-calibration	To improve GGC calibration in the current conditions model	2012-2017	Regional BCB
1.4	BCB Water Availability: GG3 discharges	To simulate discharges through GG-3 prior to 2012	2008-2017	Regional BCB
1.5	BCB Water Availability: CWIP withdrawal impacts	To simulate impacts of the project withdrawals on GG3 downstream water users	2008 -2017	Regional BCB
Section 2: Models for Vegetation Hydrology Effects Analysis				
2.2	BCB Current Conditions: Updated	To obtain BC's for local scale models	2008 -2007	Regional BCB
2.3.1	Collier County Model: Current Conditions	To simulate and calibrate the model for current conditions in the project area.	2008- 2017	Local Scale
2.3.2	Collier County Model: CWIP Conditions	To simulate the proposed conditions.	2008- 2017	Local Scale
2.4.1	Collier County Model: PSRP* Conditions	To simulate the PSRP conditions. This does not include CWIP conditions	2008-2017	Local Scale
2.4.2	Collier County Model: PSRP & CWIP Conditions	To simulate both PSRP and CWIP conditions	2008-2017	Local Scale
Section 3: Design Storm Models for Flood Impact Analysis				
3.1	Collier County Model: Current Conditions; 100-yr design storm	To simulate current conditions under 100-year design storm conditions	3-day	Local Scale
3.1	Collier County Model: CWIP Conditions; 100-yr design storm	To simulate the CWIP conditions under 100-year design storm conditions	3-day	Local Scale
3.1	Collier County Model: Current Conditions; 100-yr design storm	To simulate current conditions under 25-year design storm conditions	3-day	Local Scale
3.1	Collier County Model: CWIP Conditions; 100-yr design storm	To simulate the CWIP conditions under 25-year design storm conditions	3-day	Local Scale

*PSRP - Picayune Strand Restoration Project

1 Golden Gate Canal Water Availability Analysis

It has been established in previous works that there is enough water in Golden Gate Canal to divert and rehydrate Belle Meade Forest. For the details, reader is referred to Atkins, 2016. In the report, a preliminary discharge of 100 cfs is recommended to be diverted from the Canal. The recommendation is based on a flow availability analysis completed for the GGC in terms of diverting freshwater flows during the wet season. This analysis and results were completed in coordination with the SFWMD to assure flow diversion would not affect groundwater stages for local water users. The results of the analysis determined that flows could only be diverted when the water level at GG-3 weir structure is lowered to elevation 6.5-feet NAVD88. Based on this elevation and the available data from the structure gage (from 2009-2014), water could be diverted, on average, 40 days per year at 100 cfs. This diversion protocol is considered conservative and appropriate at that time considering the project was still in the early conceptual phase.

1.1 GG-3 Flow Duration Analysis

Golden Gate Canal flows were re-evaluated as part of this effort for a possibility of diversions for additional days per year. For this purpose, flow duration analysis was performed at GG-3 structure for the period from 2012 to March 2019. Since no significant statistical correlation was found between GG-3 gate levels alone and discharges in the canal, as recommended in Atkins (2016) report, it was deemed more appropriate to relate diverted amount with canal flows, which are a function of headwater, tailwater, and gate levels, rather than gate levels alone. The above recommended 100 cfs has a percent exceedance of 11%, as reported in Atkins (2016). This corresponds to a discharge of 450 cfs and above through GG-3, as shown in the flow duration curve (Figure 1.1). In other words, 100 cfs can be safely diverted from the canal whenever flow at GG-3 structure exceeds 450 cfs. Furthermore, it was estimated, based on permit information available from SFWMD online database, that only ~ 63 cfs is required for all the downstream water users based on their maximum permitted amount. To be on the safe side, it was assumed that water can be diverted on additional number of days whenever the flow at GG-3 is greater than 200 cfs and less than 450 cfs. As explained earlier, it has been established that 100 cfs does not cause any reduction in the groundwater that can affect water users in the area, hence a lesser discharge, i.e. 50 cfs, can be diverted safely for additional days.

To recommend a pumping protocol for CCWIP based on GG-3 structure discharges, it was necessary to have a continuous time series of discharges for the entire simulation period, 2008 to 2017. GG3, located approximately a half mile east of CR951, became operational in 2012, replacing the old structure GOLDW3, located ~ 3 miles to the north near 17th Ave SW. The old structure was a fixed crest weir with two bottom opening sluice gates and was entirely modified to a system of operable control gates, GG-3. Since, the two structures have different operational criteria and designed discharges, it is not possible to combine time series of discharges from the two structures into one continuous time series satisfying the study simulation period. It is therefore deemed more appropriate to develop a model simulating GG-3 prior to 2012, back to 2008, and recommend pumping protocol based on model simulated discharges instead. Furthermore, it is also equally important to assess the impacts of recommended CCWIP withdrawals on downstream water users to ensure that the permitted water users are not being impacted.

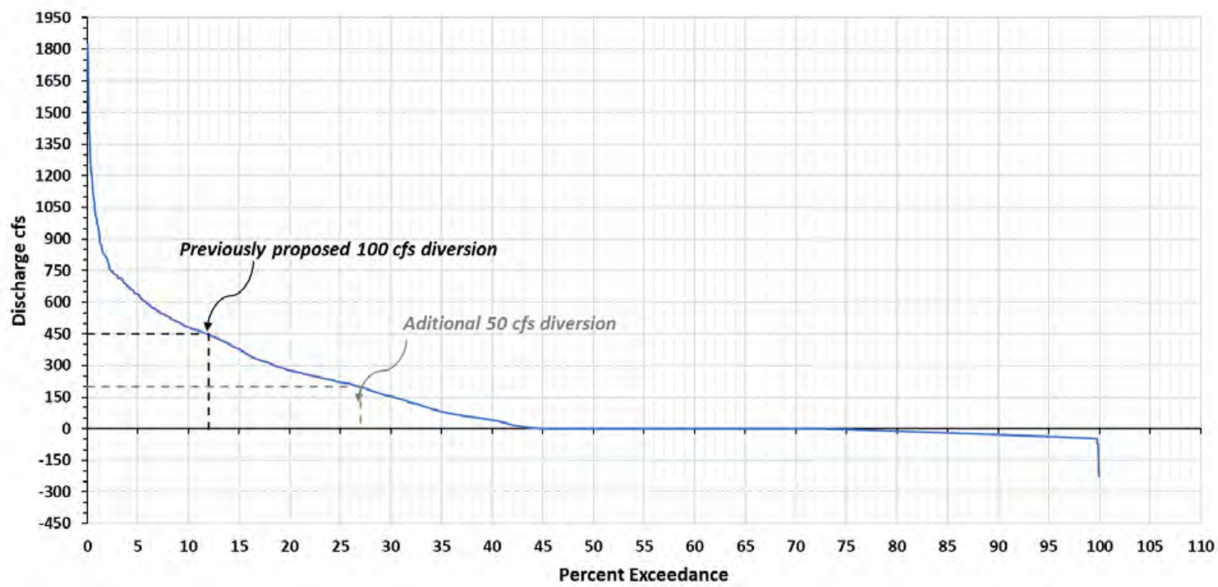


Figure 1.1 - Flow Duration Plot at GG-3 Structure based on observed flow data from DBHYDRO for the period from Dec 2011 to May 2019

1.2 Objectives of Water Availably Analysis

Based on the discussion above, water availability analysis was performed with the following objectives:

a) To refine the representation of water users in regional BCB model for the areas of interests i.e. Golden Gate Canal especially downstream of GG-3 structure, and to improve the canal calibration.
b) Use the above model to simulate GG-3 structure backwards in time, starting in 2008.
c) To assess the impacts of recommended CCWIP withdrawal on water users downstream of GG-3.

To achieve above objectives, regional BCB model, was adopted and modified. The details of each of the models and modifications are as follows in coming sections.

1.3 Improving Model Calibration for GGC

The BCB updated model, detailed in section 2 of this report, was adopted and modified with the objective of improving model calibration for Golden Gate Canal. To assess the availability of water at GG-3, it was important to include water users located in vicinity of the structure, especially downstream of GG-3, that directly or indirectly rely on GGC flows and/or levels for surface water withdrawals. Considerable time and efforts were invested in identifying all the water users that withdraw water from the GGC canal or its tributaries. The information about water users is available from SFWMD e-permit database. Furthermore, it was also important to accurately represent irrigation priorities for the water users as permitted by SFWMD. In the previous model, irrigation command areas were lumped by irrigation sources and priorities were neglected. While it was appropriate given the objectives of the model, irrigation source priorities were revised for this objective. This allowed accurate representation of the amount being consumed from each irrigation source. The following section describes water users identified and included in the model.

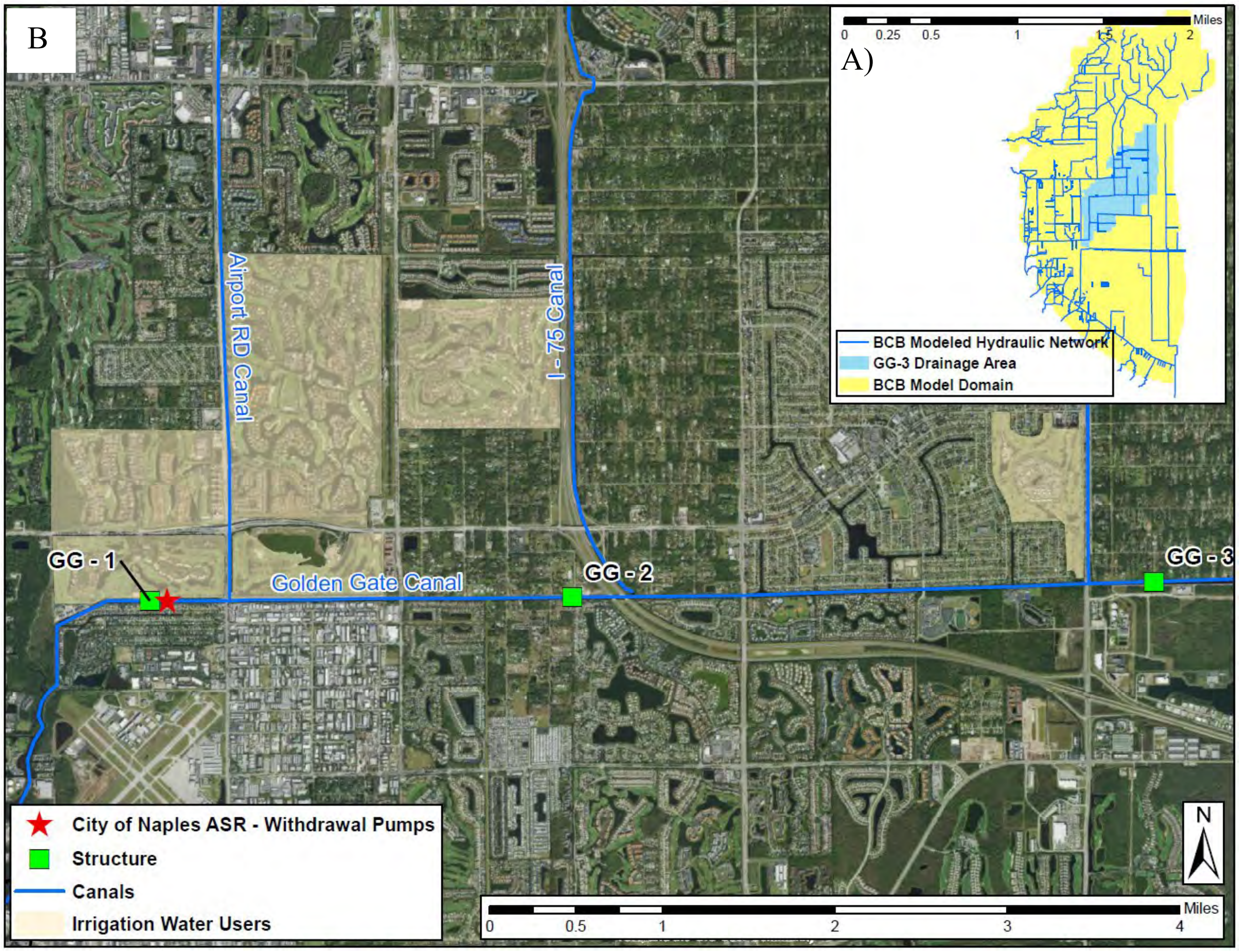


Figure 1.2 - A) BCB regional model domain and Golden Gate Canal drainage area. B) Golden Gate Canal and updated water users.

1.3.1 Irrigation Water Users

Five irrigation water users were identified and included in the model with their defined irrigation source priorities (Figure 1.2). These water users either directly pull water from Golden Gate Canal itself, or a major canal that feeds into the Golden Gate Canal system, or effect aquifer baseflows to the GGC by pumping water from Water Table and/or Tamiami Aquifers.

All the water users are residential and golf course communities. Golf course is the predominant vegetation type, followed by urban low and medium density residential units, that are permitted for irrigation under these permits. The water users were updated from the previous model from source-based ICA (irrigation command area) to priority-based ICA, as described earlier. Priority-based ICA implies that each higher priority source should be fully consumed to the permitted threshold, before the next can be exploited. The irrigation sources, along with their priorities and permitted amounts were simulated in the model based on permit information, as reproduced below (Table 1.1 to 1.5). Reclaimed water was specified as an external water source in the model as the water does not come from within the modeled domain. The treated wastewater is supplied by the city from wastewater treatment plants and hence is not part of the modeled water budget. The irrigation demand for golf course, urban low and medium density vegetation types were defined under Land Use - Vegetation section of the model.

I. Grey Oaks & Grey Oaks West (SFWMD Permit # 11-00803-W)

Total Annual Allocation : 758 MG (Million Gallons)
Total Max Monthly Allocation: 108.4 MG
Total Serviced Acreage: 629.3 acres

Table 1.1: Permit details for SFWMD Permit # 11-00803-W

Modeled Priority	Source	MIKE-SHE Type	Application Method	Calibrated Max. rate (ft^3/sec)	Permit Conditions
1	Reclaimed Water	External Source	sprinkler	0.25	As available
2	WT Aquifer	Shallow well	sprinkler	0.00583	Not more than 23.4 MGM (million gallons/month)
3	Airport Rd	River	sprinkler	0.25	Head U/S of Airport RD Weir 2 > 7.0 NGVD
4	Airport Rd	River	sprinkler	0.25	Head D/S of Airport RD Weir 2 > 2.65 NGVD
5	Lower Tamiami Aquifer	Single well	sprinkler	0.25	Water Depth > 20' above the bottom of the LT strata

II. Wyndemere Country Club (SFWMD Permit # 11-00167-W)

Total Annual Allocation: 166.5 MG
Total Max Monthly Allocation: 20.4 MG
Total Serviced Acreage: 193.9 acres

Table 1.2: Permit details for SFWMD Permit # 11-00167-W

Modeled Priority	Source	MIKE-SHE Type	Application Method	Calibrated Max. rate (ft^3/sec)	Permit Condition
1	GGC	River	sprinkler	0.25	Head U/S > 3.45 NGVD
2	Lower Tamiami Aquifer	Single well	sprinkler	0.25	Head Depth > 20' above the bottom of the strata

III. Naples Grand Golf Club (SFWMD Permit # 11-00806-W)

Total Annual Allocation: 411.0 MG
Total Max Monthly Allocation: 52.9 MG
Total Serviced Acreage: 118.2 acres

Table 1.3: Permit details for SFWMD Permit # 11-00806-W

Modeled Priority	Source	MIKE-SHE Type	Application Method	Calibrated Max rate (ft^3/sec)	Permit Condition
1	I-75 Canal	River	sprinkler	0.25	Head U/S > 7 NGVD
2	Lower Tamiami Aquifer	Single well	sprinkler	0.25	Head Depth > 20' above the bottom of the strata

IV. Bear's Paw Country Club (Permit # 11-00130-W)

Total Annual Allocation: 114 MG
Total Max Monthly Allocation: 18.3 MG
Total Serviced Acreage: 144 acres

Table 1.4: Permit details for SFWMD Permit # 11-00130-W

Modeled Priority	Source	MIKE-SHE Type	Application Method	Calibrated Max. rate (ft^3/sec)	Permit Condition
1	GGC	River	sprinkler	0.25	18.3 MGM

V. Golden Gate Country Club (Permit # 11-00138-W)

Total Annual Allocation: 119 MG
Total Max Monthly Allocation: 15.4 MG
Total Serviced Acreage: 90 acres

Table 1.5: Permit details for SFWMD Permit # 11-00138-W

Modeled Priority	Source	MIKE-SHE Type	Application Method	Calibrated Max rate (ft^3/sec)	Permit Condition
1	GGC	River	sprinkler	0.25	15.4 MGM

1.3.2 City of Naples ASR Withdrawals

City of Naples pumps water from Golden Gate canal upstream of GG-1 structure. The withdrawals are used for injection into the city's aquifer storage and recovery (ASR) system since Nov 2011. Permitted annual allocation from GG Canal is 2432 Million Gallons. On average, 10 MGD (million gallons per day) from the GG canal are injected into ASR system during wet season (between Jun to Feb). Actual amount of water withdrawals from the GGC are reported on a monthly-basis and are available from SFWMD e-permitting database under SFWMD Permit # 11-03205-W. The withdrawals were incorporated in the previous models based on permitted monthly amounts, however, in this model the amounts were revised based on real withdrawal amounts as reported in the e-permitting database. In the model, ASR withdrawals were simulated by a sink boundary condition. Figure 2.2 shows the time series of monthly withdrawals.

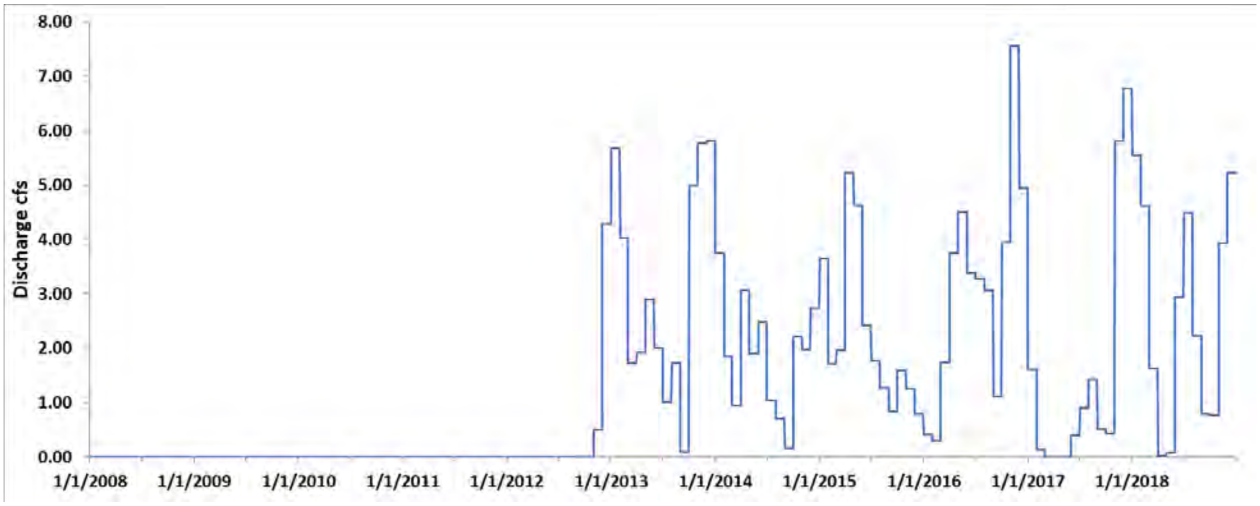


Figure 1.3 - City of Naples ASR withdrawals upstream of GG-1 Structure.

1.3.3 Results and Discussion

1.3.3.1 Calibration for Irrigation Rates

In MIKE SHE, maximum application rates for each source type must be defined. Since, it is difficult and impractical to estimate the parameter in the field, it was rather calibrated based on the amount of water required for irrigation per unit time by a specific vegetation type. The estimates of annual irrigation demand for various vegetation types is standard information and local estimates are available. Several simulations were run to calibrate source discharge rates of the ICA's until reasonable simulated irrigation rates were obtained for Golf Course, Urban Medium- and Low- Density vegetation across the entire modeled domain. The resulting maximum source discharge rates are shown in Table 2.1 to 2.5 for each source type and ICA. The irrigation rates for vegetation types and the resulting irrigation rates for each of the added ICA's are shown in Table 1.6 and 1.7.

Table 1.6: Simulated annual irrigation rates for the vegetation types. The values are based on calibrated maximum source discharge rates for five ICA's included in the updated model. The rates shown are modeled-wide.

LU Types	Irrigation rates (in/yr)
Golf Course	10.932
Urban Low Density	3.753
Urban Medium Density	3.819

Table 1.7: Annual irrigation depths for updated irrigation water users, ICA. ICA irrigation rates are based on calibrated irrigation rates for vegetation types as shown in Table 1.6.

Irrigation Water User - ICA	LU Type	Irrigation rates (in/yr)

Grey Oaks & Grey Oaks West	Golf Course, Urban Low Density	6.325
Wyndemere Country Club	Golf Course, Urban Medium Density	3.045
Naples Grand Golf Club	Golf Course	16.135
Bear's Paw Country Club	Golf Course	4.948
Golden Gate Country Club	Golf Course	10.597

1.3.3.2 Calibration for Golden Gate Canal

Water level and discharges in GGC were calibrated to groundwater leakage coefficients. Flux exchange from aquifer to river and vice versa, is determined in MIKE SHE/MIKE 11 coupling by leakage coefficient, which is representative of the conductance offered by riverbed and/or aquifer. The parameter is difficult to field estimate for each canal reach; hence it was calibrated based on annual average flux exchange. Estimates of annual average flux exchange for GGC are available from previous studies and experiences. Several simulations were run, and leakage coefficients were adjusted until satisfactory estimates of average annual baseflows for the canal were obtained. Figure 1.4 and 1.5 shows observed versus calibrated water levels and discharges, whereas, Figure 1.6 shows cumulative discharges at each of the structures.

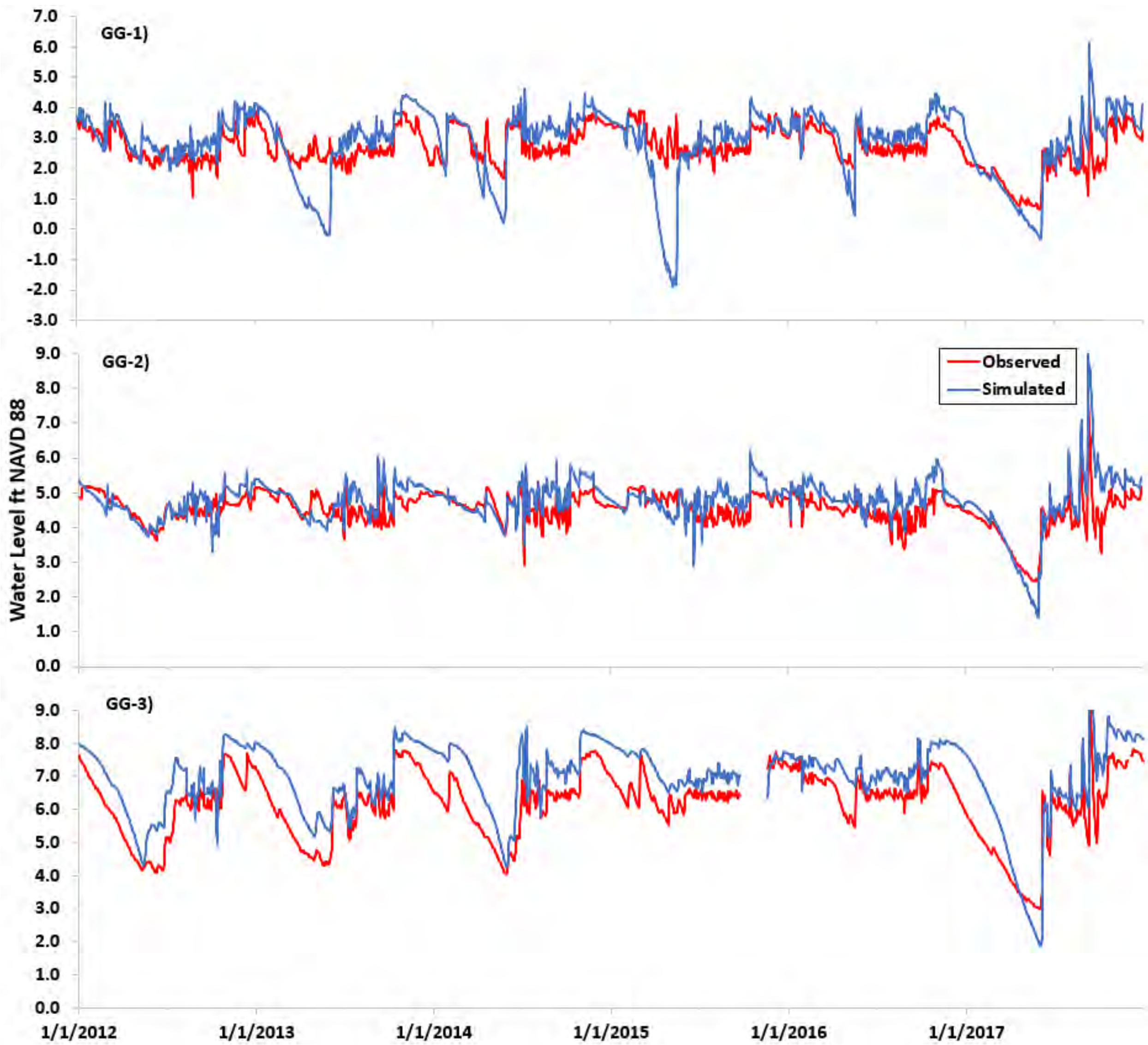


Figure 1.4 - Observed vs. simulated upstream water levels in Golden Gate Canal from the calibrated model (2012 to 2017). Top: water levels upstream of GG-1 Structure. Middle: water levels upstream of GG-2 Structure. Bottom: water levels upstream of GG-3 Structure.

As can be seen from plots in figure 2.3, the model well captures stage fluctuations in the canal throughout the simulation period, especially after late-2015. For GG-1, the model underpredicts water levels during the dry seasons up until late-2015, however, post late-2015, there is a good match between the simulated and observed data for the dry season lows. Similar pattern is observed in discharge time series (Figure 1.5). The model over predicts the discharges before late-2015, whereas, after late-2015, the simulated data is close to the observed data. It is important to mention here that there was a brief break in observed data from 9/25/2015 to 11/19/2015 for the GG-3 structure. It is believed that during the period, discharge coefficients were revised for the gates which provides better match with between the simulated and observed data, especially for GG-3. The goodness of fit for the simulation is shown in Table 2.8 and 2.9.

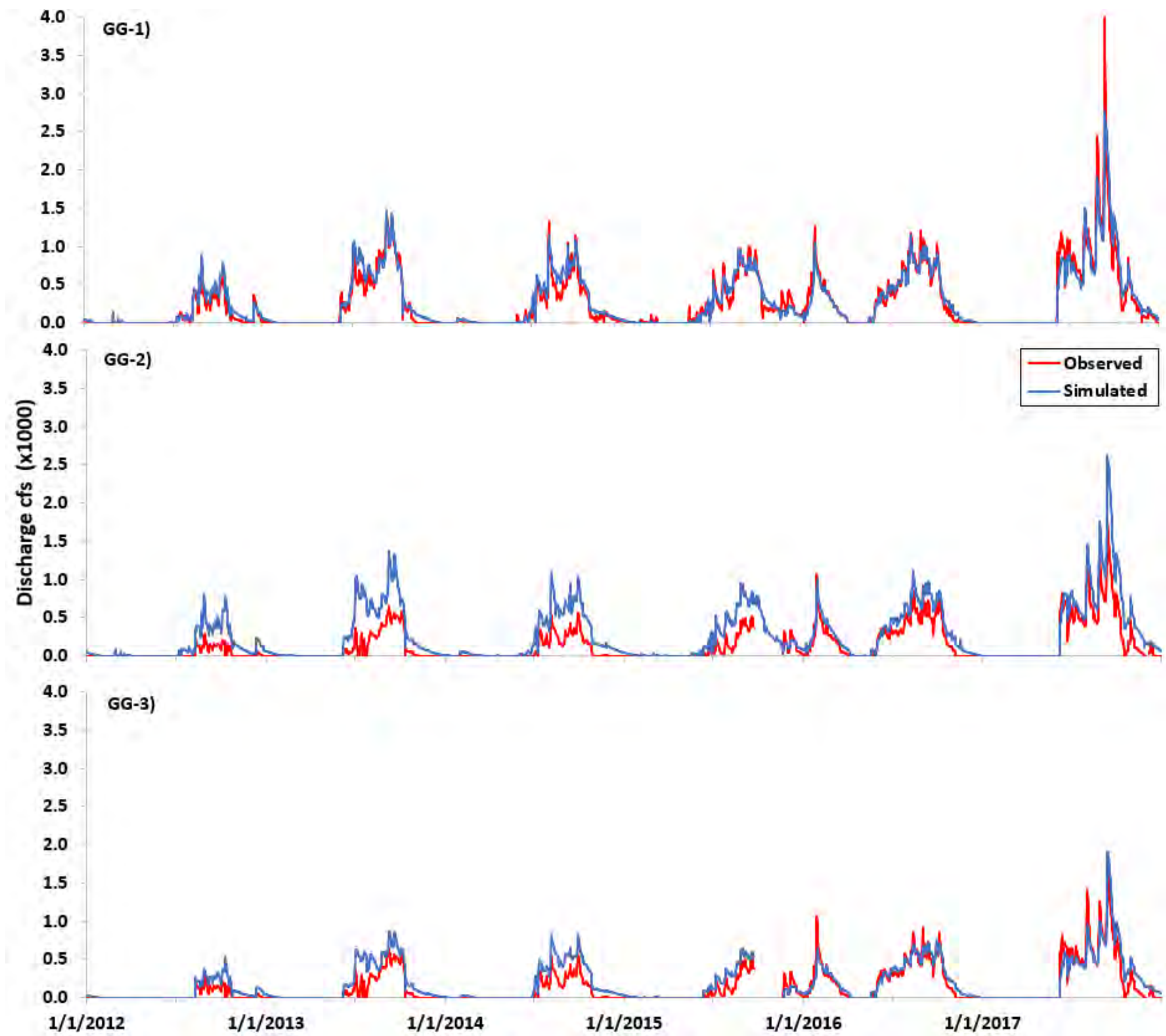


Figure 1.5 - Observed vs. simulated discharges in Golden Gate Canal from the calibrated model (2012 to 2017). Top: discharges through GG-1 Structure. Middle: discharges GG-2 Structure. Bottom: discharges through GG-3 Structure.

Table 1.8: Statistical measures for cumulative observed and simulated discharges in Golden Gate Canal. For GG-1 and GG-2, the comparison is for entire simulation period, whereas, for GG-3, the period is divided into pre and post late-2015.

Statistics	**Discharges**			
	GG-1	**GG-2**	**GG-3**	
			Before Late-2015	**After Late-2015**
%Difference between simulated and observed cumulative discharge values	6 %	15 %	29 %	7 %

Table 1.9: Statistical measures for observed vs simulated water levels in Golden Gate Canal (2012 - 2017).

Statistics	Water Level		
	GG-1	GG-2	GG-3
ME	0.04	0.07	0.23
MAE	0.18	0.11	0.25
RMSE	0.27	0.15	0.28
STDres	0.26	0.13	0.15
R(Correlation)	0.65	0.77	0.90

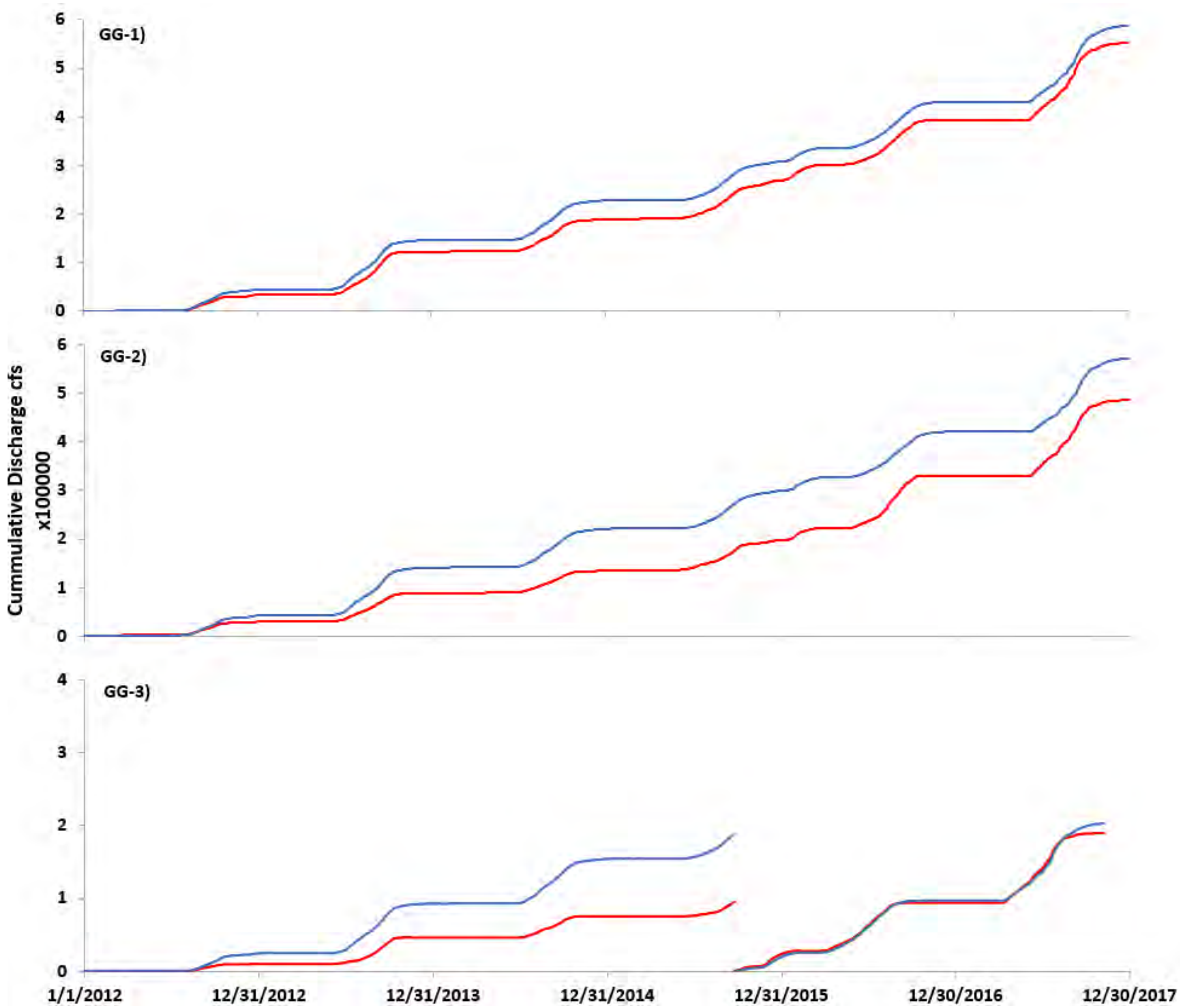


Figure 1.6 - Observed vs. simulated cumulative discharges in Golden Gate Canal from the calibrated model (2012 to 2017). Top: discharges through GG-1 Structure. Middle: discharges GG-2 Structure. Bottom: discharges through GG-3 Structure.

1.4 Extension of New GG-3 Backwards in Time

For the second objective, the above updated and calibrated model was adopted to simulate extension of GG-3 backwards in time, before 2012 back to January 2008. The old GOLDW3 structure was removed from the Golden Gate Canal and the new GG-3 was simulated as if it existed throughout the entire simulation period. Another modification is to adopt rule-based operations for gates throughout the simulation period rather than observed data for the partial period. The operating control elevations are available from SFWMD Atlas (SFWMD, 2017). The wet period (typical wet season), dry periods (typical dry season) and atypical conditions were set as follows:

> Wet Periods: When the upstream water level stages at, or above, 8.0 ft NGVD, start lowering weir, when the upstream water level lowers past 7.0 ft NGVD, raise weir.
> Dry Periods: When the upstream water level stages at, or above, 9.20 ft NGVD, start opening weir, when the upstream water level lowers past 8.75 ft NGVD, raise weir.
> Special Operations: Under unusual conditions, as may be brought about by large storm events, or under emergency conditions, following EOC directions, the structure operation will be modified, and weirs may be operated manually to optimize canal system capacity. Special operations may also include weir movements necessary for system drawdown, if such operations are deemed appropriate in advance of anticipated storm event.

1.4.1 Results and Discussion

The simulated flows through the GG-3 structure from the period from 2008 to 2017 based on operational rules, (Figure 1.7) were used to evaluate the CCWIP pumping protocol and establish a withdrawal time series for use in the local scale model, as described in section 1.1 (Figure 1.8).

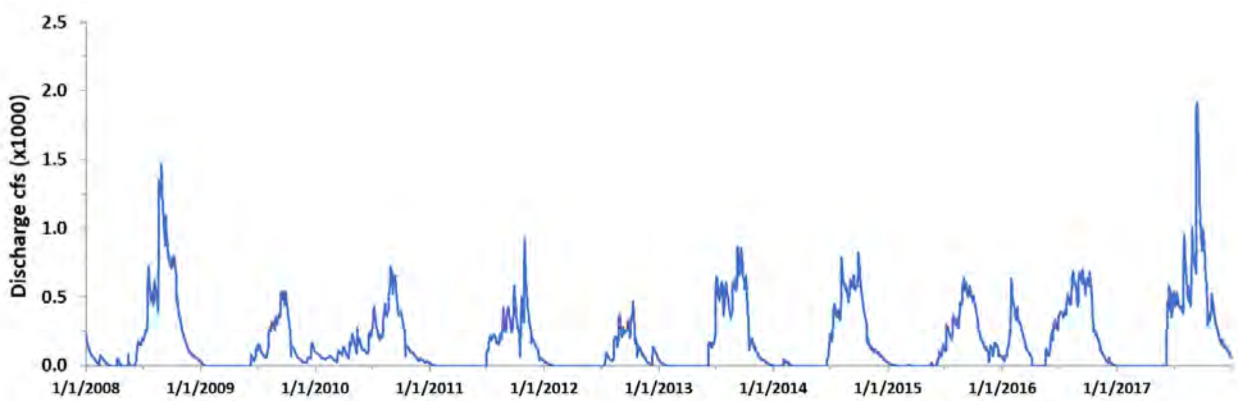


Figure 1.7 - Simulated discharges through the new GG-3 structure for the entire simulation period, 2008 to 2017.

1.4.1.1 CCWIP Pumping Protocol

Based on GG-3 flow duration analysis, described in section 1.1, it is established that 100 cfs will be diverted upstream of GG-3 whenever the flow through the structure exceeds 450 cfs, and 50 cfs will be diverted whenever the flow through the structure is between 200 cfs and 450 cfs. From the results of above simulation, continuous time series of simulated discharges through GG-3 structure, from 2008 to 2017, was compared against the above flow thresholds of 450 and 200 cfs. The resulting CCWIP pumping protocol plotted in Figure 3.8. Based on above comparison, CCWIP diverts 100 cfs for an average of 55 days/year and 50 cfs for an average of 83 days/year. 100 cfs diversion are mostly possible during wet seasons of the years whereas, additional 50 cfs has allowed for some diversions during early portions of dry seasons as well. The new pumping protocol rehydrates the Belle Meade forest for ~ 135 days per year as compared to ~ 40 days per year recommended previously. The protocol was adopted for long term vegetation analysis simulations (section 2).

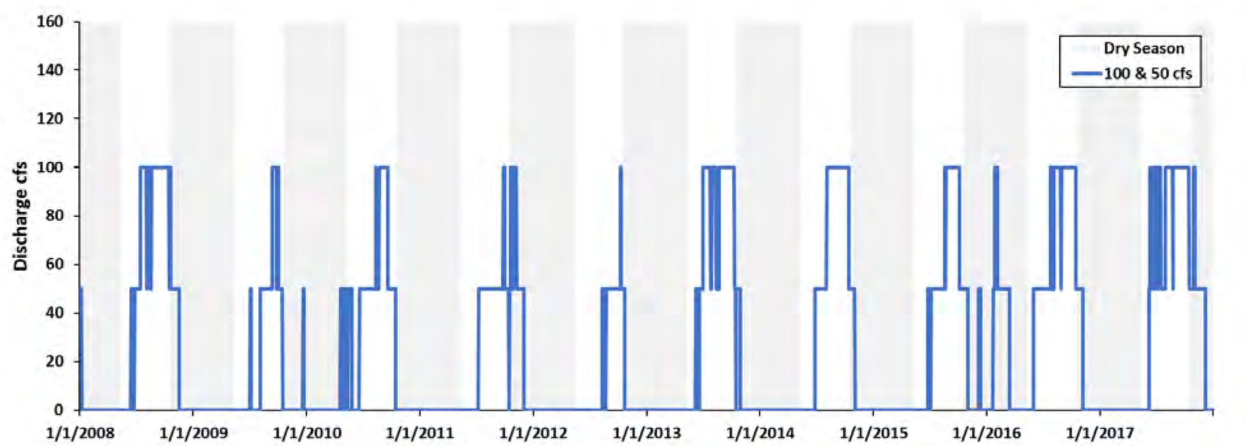


Figure 1.8 - Recommended CCWIP pumping protocol, as adopted in local scale models (section 2). Grey patches show dry season.

1.5 Impact Assessment of CWIP Withdrawals on Water Users

Finally, the recommended CCWP pumping protocol was tested against any impacts on the permitted water users located downstream of GG-3 structure. The pumping protocol (Figure 1.8) was incorporated in the model as a single point sink, i.e. the recommended discharge is simply extracted from the GGC at the defined time and location. The impact was determined by comparing pre and post CCWIP withdrawal scenarios for the period from 2008 to 2017, and against any permit limits described by the district.

1.5.1 Results and Discussion

1.5.1.1 Irrigation Water Users

Three residential and golf course communities withdraw water directly from Golden Gate canal downstream of GG-3 structure and were evaluated for CCWIP impacts. The irrigation demands for pre and post CWIP withdrawals is tabulated below (Table 1.10). As can be seen, there is no significant impact of CWIP withdrawals on yearly irrigation demands/withdrawals for each of the ICA.

Table 1.10: Impact of CCWIP withdrawals on Golden Gate Canal water users

Irrigation Water User	Pre-CCWIP Withdrawals	Post-CCWIP Withdrawals
	in/yr	in/yr
Naples Grand Golf Club	16.7	16.8
Bear’s Paw Country Club	5.0	5.1
Golden Gate Country Club	10.8	10.9

1.5.1.2 City of Naples ASR Withdrawals

City of Naples pumps water upstream of GG-1, which is located downstream of GG-3. The SFWMD permit specifies yearly and maximum monthly volumes available for diversion at GG-1, however, it further limits the permitted volumes based on water level upstream of GG-1. The permit mandates that the permitted volume can only be diverted as long as the water level upstream of GG-1 is greater than or equal to 2.23 ft NAVD 88. The pre and post CCWIP withdrawal water levels were compared against the permitted water level and are shown below (Figure 1.9).

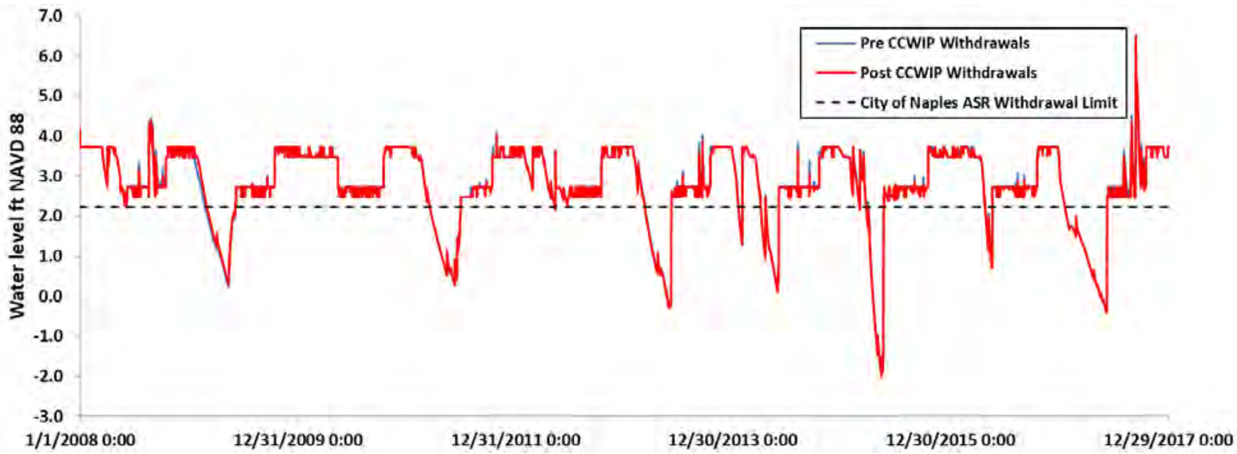


Figure 1.9 - Pre and Post CWIP withdrawals water levels in Golden gate Canal upstream of GG1 weir.

As shown in the plot, no significant impact can be seen on the prescribed water levels, which ensures that permitted amount can still be withdrawn post CCWIP withdrawals. There are small discrete decreases in water levels, mostly during wet seasons, but is still well above the permitted threshold.

From the above analysis it is established that the recommended CCWIP pumping protocol can be safely adopted without significantly impacting any water user.

2 Long Term Models for Vegetation Analysis

2.1 Introduction

A fully integrated surface-and groundwater model, MIKE SHE/MIKE 11 was developed to evaluate impacts of the project on vegetative communities in Belle Meade forest. MIKE SHE is a physically based, fully distributed modeling package which can simulate all the processes of hydrologic cycle including; Evapotranspiration, Overland flow, Unsaturated Zone Flow and Groundwater Flow, whereas, open channel flow is simulated in MIKE 11. Mike 11 is fully coupled with MIKE SHE where exchanges with overland flow, unsaturated and saturated zones are accounted for within the specified hydraulic network.

The model domain stretches over an area of 171,287 acres and extends ~ 3.33 miles east of Everglades Blvd. on the eastern boundary, ~ 8.2 miles north of Alligator Alley on the northern boundary, ~ 4.26 miles west of Collier Blvd. on the western boundary and paralleling coastline from southwest of Airport Rd. S. to about ~ 7.84 miles southeast of CR 92 on the southern side (Figure 2.1). To effectively represent the areas of interest with a reasonable simulation run-time, a small-domain, local-scale model (LSM) with a grid size of 375ft X 375ft is used. A simulation period of 10 years, from 1/1/2008 to 12/31/2017, was selected. It contains dry, normal and wet years including the 2017 Hurricane Irma event. In MIKE SHE, the simulation time step varies for each process independently and is controlled automatically during each iteration depending upon the requirements. The model tries to use the largest possible time step during each iteration without compromising the user defined accuracy and numerical stability thresholds. Hence the maximum time step is set for each component such that the changes in each process are being captured effectively.

2.2 Boundary Conditions for LSM

Physically based, distributed hydraulic and hydrologic models, such as MIKE 11 and MIKE SHE, require boundary and initial conditions covering entire model domain. For this integrated model, boundary conditions (BC's) are required for both MIKE 11 (surface-water) and MIKE SHE (groundwater) boundaries. BC's represent the actual state of the system at the boundaries and are a very important part of the simulation. Hence, they should be selected such that they represent real state of the system at the location. However, since they are usually limited by the quality of available data, it is also important that the boundaries are located far enough from the areas of interest such that their impact can be minimized as much as possible. The domain for LSM is selected such that the areas of interest for CCWIP can be represented at a refined scale while keeping the simulation run-time and output file sizes reasonable. On the southwest side, along the coast, the model is bounded by the sea and real-time observed sea level data is used as boundary conditions for both surface water and groundwater models. On the east side, parallel to Everglades Blvd., observed head data is used to confine model. However, on the northwest side of the model, no physical feature or observed data is available to specify appropriate boundary conditions. For this purpose, simulated data from a regional model that covers the entire extent of LSM are used to develop appropriate BC's. The details of the regional model are described in coming sections.

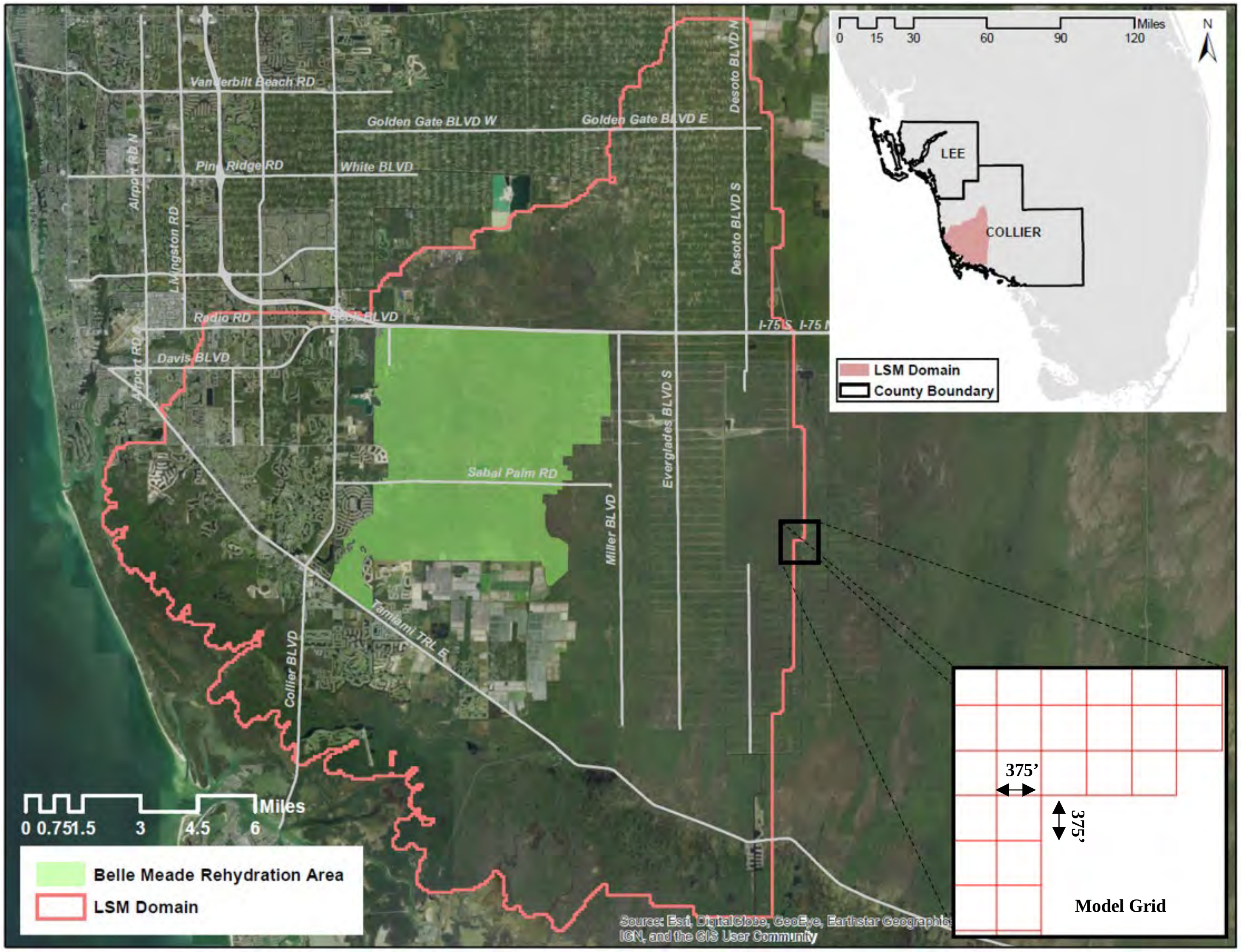


Figure 2.1 - Local Scale Model domain and boundaries along with Belle Meade Forest area targeted for rehydration

2.2.1 BCB Model Description

The latest version of the Big Cypress Basin (BCB) model, is available from a previous study conducted by Atkins (Atkins, 2016). The model covers the entire extent of western Collier County including the LSM domain (Figure 2.2) at 500ft X 500 ft grid resolution and 10-year simulation period from 1/1/2002 to 12/31/2012. The model has been approved by the district for accuracy of input data, rationality of parameters, calibration and results. The above version of BCB model was updated for simulating boundary conditions for LSM. The model was updated to incorporate major changes in the basin since its development and the representation of areas of interests was further improved as describe in upcoming sections. The simulated water levels and heads from the updated BCB model were used as boundary conditions in LSM.

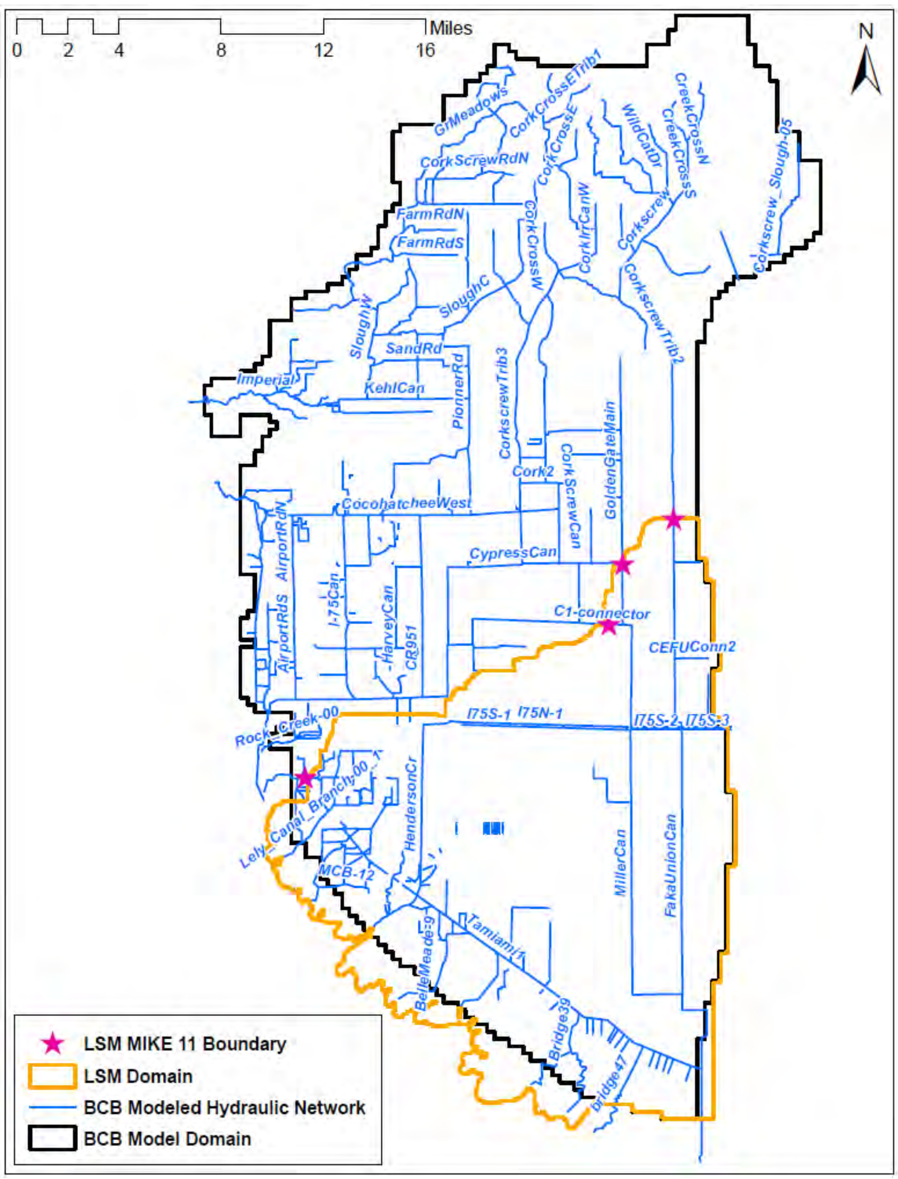


Figure 2.2 - BCB regional and LSM domain along with BCB hydraulic network. LSM boundaries are shown for hydraulic network only.

2.2.2 Updates in the BCB Model

Temporal Extension

The BCB simulation period was extended beyond 2012, up to 2017, to match the simulation period of LSM i.e. 2008 to 2017. The BCB model was previously calibrated for years 2002 to 2012 (Atkins, 2016). The calibration was deemed sufficient for the objective at hand, and with the exception of the limited re-calibration in the vicinity of the Golden Gate Canal described in **Section 1.3**, no further efforts were made to re-calibrate the model for the extended period. The following time-series were extended throughout the model domain:

- Rainfall
- Evapotranspiration
- Observed gate levels and operational rules for operable structures,
- Vegetation property file,
- Pumping wells

Rainfall, Evapotranspiration and Observed Gate Levels for BCB structures are available from SFWMD DBHYRO online database. Vegetation data was extended using the same annual crop development patterns and irrigation requirements, as used before in the model. Similar to the surface water model, groundwater model boundaries were also extended. BCB model has five layers of aquifers. Time varying heads from observed well data were used as outer boundary conditions for the water table aquifer and the bottom most Sandstone Aquifer. Time series of observed groundwater data ("DBHYDRO", n.d.) for all the available wells were also extended through 2017 and interpolated on a 500 ft by 500 ft grid (Figure 2.3). For the rest of the layers, a close or no-flux boundary was applied.

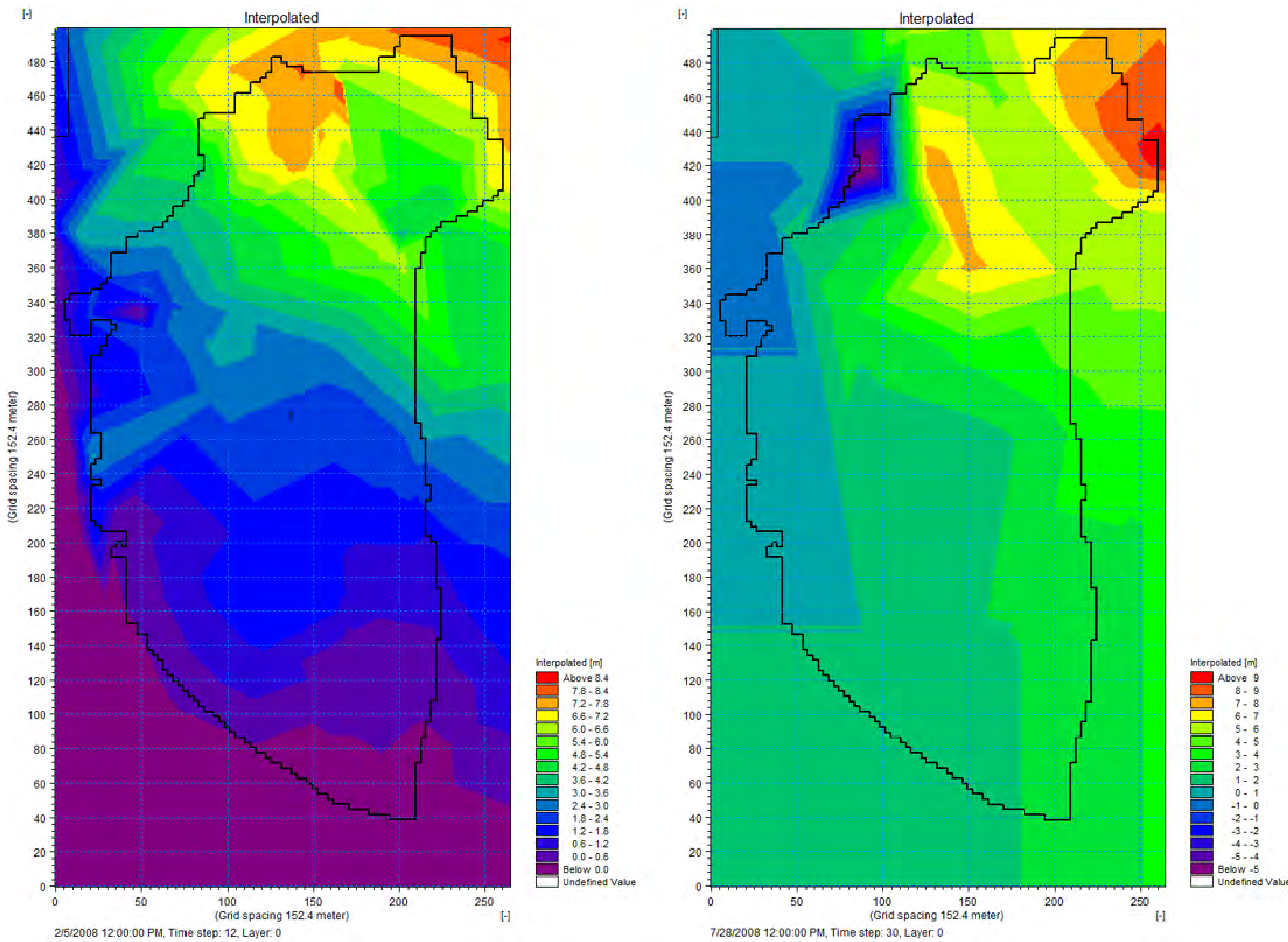


Figure 2.3 - Interpolated observed groundwater levels used as boundary conditions in BCB groundwater model. A) Water Table Aquifer. B) Lower Tamiami Aquifer.

Henderson Creek

Henderson creek lies on the west side of the Belle Meade Forest area. It is connected with I-75 North canal at the upstream end and extends down to tidal waters of Rookery Bay. The Creek cross sections and structures were previously updated by Interflow Engineering, Inc. and Taylor Engineering Inc. for the Henderson Creek watershed study (Interflow, 2014). The following structures were included in the BCB model based on HC model (Table 2.1) to improve the timings and volumes of creek discharge to Rookery Bay.

Table 2.1: Structures added in Henderson Creek in BCB model based on HC model.

Chainage	Structure/Culvert
15403	Control Structure HC2 (weir)
18094	Marino Circle
19297	A Better Way
20341	Entrance to Lee's Place Tavern
23507	The Lords Way
25013	Stub-out/dirt road
25013	Stub-out/dirt road
26198	Rattlesnake Hammock
27231	Hospital Entrance
28215	Lely Cultural Parkway
28497	Trailer Park
28839	Johns Road
29448	Amity Road
30807	Unnamed dirt drive
31447	Sabal Palm Road
34253	Verona Walk
38480	Winding Cypress

Miller 3 structure

Miller Canal Weir No. 3 (Miller-3) is located approximately 450 feet north of the westerly terminus of 8th Avenue NE near the confluence of Golden Gate and Miller Canals in Golden Gate Estates The structure was built in 1960's with significant modifications in 1983. However, in 2012, it was observed that in addition to poor structural condition, the elevation of the fixed crest was not providing the required level of flood protection. The district proposed to replace the entire structure with an operable weir by 2014. While the activities for planning, design and permitting of replacing the structure continued, an in-house interim modification of the structure was made between 2012 to 2014 to provide flood protection. In this BCB model, Miller 3 structure was updated to represent the old structure from 2008 to 2012, the interim structure, from 2012 to 2014, and the new structure from 2014 to 2017. The new structure has replaced the old weir by three operable weirs with top and bottom gates. Structural geometry and wet and dry season operational rules for the old and interim structures have been documented by the district in the modification report ("SFWMD", n.d.), whereas, for the new structure, the information is available from the districts structural atlas (SFWMD, 2016).

<u>Control Structures</u>

SFWMD updated control structures information for all major canals in 2011 (SFWMD, 2016). All the structures on canals, such as Faka Union, Miller, Golden Gate, Henderson, I-75 South and North etc., included in this model, were revised based on the atlas information. Furthermore, for some manually operated structures, the data was not available from DBHYDRO and was obtained from the district in undigitized pdf format. The updates included revising invert levels, structure geometry and operational rules etc. The structures updated are listed in Table 2.2.

Table 2.2: Control structures modified in BCB model.

Canal	DBHYDRO Structure Name
Corkscrew	CORK 2
	CORK 3
CR951	CR951-1
	CR951-2
Cypress	CYP-1
Faka Union	FU3
Henderson	HC-2
I - 75	I-75-1
	I-75-2
	I-75-3 (Top and bottom gate)
Miller	MIL3
	Mil2
Golden Gate	GG1
	GG3
	GG4
	GG5
	GG6
	GG7
Cocohatchee	COCO-1

2.2.3 Results and Discussion

2.2.3.1 Mike 11 Boundary Conditions for LSM

Simulated discharges from the updated BCB model were extracted and used as discharge boundary conditions in LSM at four locations (Table 2.3). Figure 2.2 shows the location of boundary conditions.

Table 2.3: Simulated discharges extracted from BCB to use as boundary conditions in LSM

Canal	Discharge through structure
Faka Union Canal	FU 5
Miller Canal	MIL3
Haldeman Creek	HCB-00-S0200
C1 Connector Canal	MGG-03-S0100

2.2.3.2 MIK SHE Boundary Conditions for LSM

Simulated groundwater heads from the updated BCB were extracted for the water table aquifer (Figure 2.4) and were used as boundary conditions in the local scale model along the northwestern boundary.

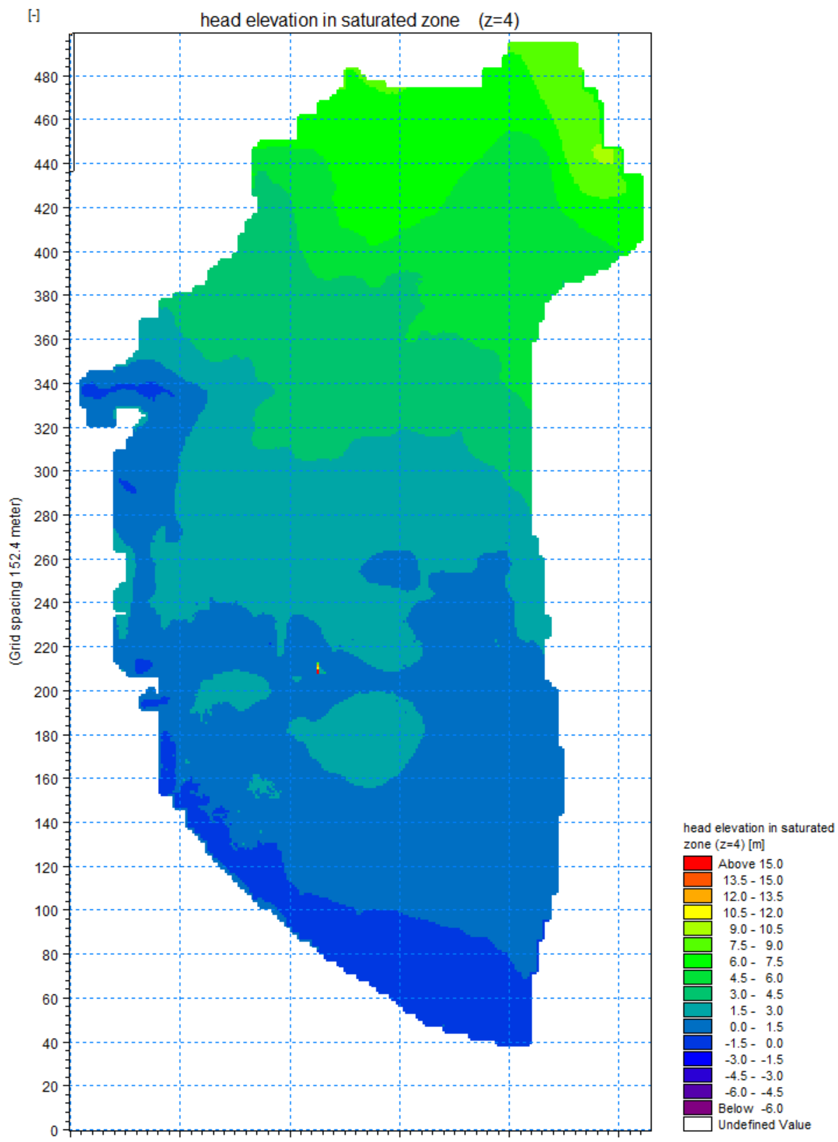


Figure 2.4 - Simulated Water Table Aquifer heads from BCB model. The grid is used to specify boundary condition in LSM Water Table Aquifer along the northwest side.

2.3 Local Scale Model Scenarios

Local scale model (LSM) scenarios were classified into current and future/with-project conditions. Current conditions model represents existing watershed conditions, whereas, the future conditions model represents post CCWIP conditions and other anticipated developments in the watershed. East of the CCWIP area, another similar rehydration project, known as the Picayune State Strand Restoration Project (PSRP), is being developed. Since the degree of overlapping in the permitting and construction schedule for the two projects cannot be anticipated at this time, two separate current and future conditions models were developed to incorporate the PSRP. The current conditions model, in this case, represents existing conditions of the watershed with PSRP in operation, whereas the future conditions model represents conditions of the watershed with both PSRP and CCWIP fully operational. PSRP modeling scenarios were included to evaluate the combined impact of the two projects on vegetation communities inside the CCWIP rehydration area.

Two previously developed MIKE-SHE/MIKE-11 models, known as the Henderson Creek Model (HC) and Big Cypress Basin (BCB), served as the source models for all LSM's. The HC model is available from a previous project on the Henderson Creek Watershed Evaluation completed for the Rookery Bay NERR (Interflow Engineering, LLC & Taylor Engineering, 2014). The latest version of BCB model is available from a recent flood protection level of service study completed for SFWMD (Taylor Engineering and Parsons, 2018). The MIKE-SHE/MIKE11 parameters for individual processes have been adopted from the above-mentioned models. Both the models have been prior approved by the SFWMD for the accuracy of input data, rationality of parameters and results. The development details for each of the models are described in the following sections.

2.3.1 Current Conditions LSM

The Current Conditions LSM serves as the baseline model scenario and was developed to effectively represent hydrologic and hydraulic processes in the areas of interest for this project. The model was developed by updating and extending the HC model as needed and was re-calibrated for surface- and groundwater levels and cumulative discharges for the simulation period i.e. 2008 to 2017. Observed rainfall and evapotranspiration data is used as the model forcing, whereas, the model explicitly simulates the following components of the hydrologic cycle; channel flow/ river hydraulics along with structures and their operational rules, 2D overland flow, 1D unsaturated zone flow and 3D saturated flow. The description of each of the processes is detailed below.

2.3.1.1 Meteorological data

<u>Rainfall</u>

Hourly NEXRAD rainfall data was obtained from SFWMD NEXRAD database for 218 pixels covering the extent of LSM domain (Figure 2.5). The rainfall was applied as forcing in both MIKESHE and MIKE 11 models.

<u>Evapotranspiration</u>

Hourly NEXRAD Reference Evapotranspiration (RET) data was extracted at 2 Km by 2 Km gird resolution for the simulation period. RET is a climatic parameter independent of crop type, crop development and management practices. In MIKESHE, the actual evapotranspiration (ET) is calculated based on RET and vegetation properties. The evapotranspiration module in linked with Vegetation module and the actual ET is calculated based on user specified Leaf Area Index (LAI), Root Depth (RD) and Crop Coefficient for each crop type. A temporal distribution of both LAI, RD and Crop Coefficient is required for each vegetation type and is defined in the Vegetative property file. The ET module is also linked with the Unsaturated Zone module and calculates plant uptake and transpiration based on soil moisture conditions in the unsaturated root zone. The ET parameters and coefficients were inherited from HC and BCB models (Interflow, 2014 and Taylor Engineering, 2018).

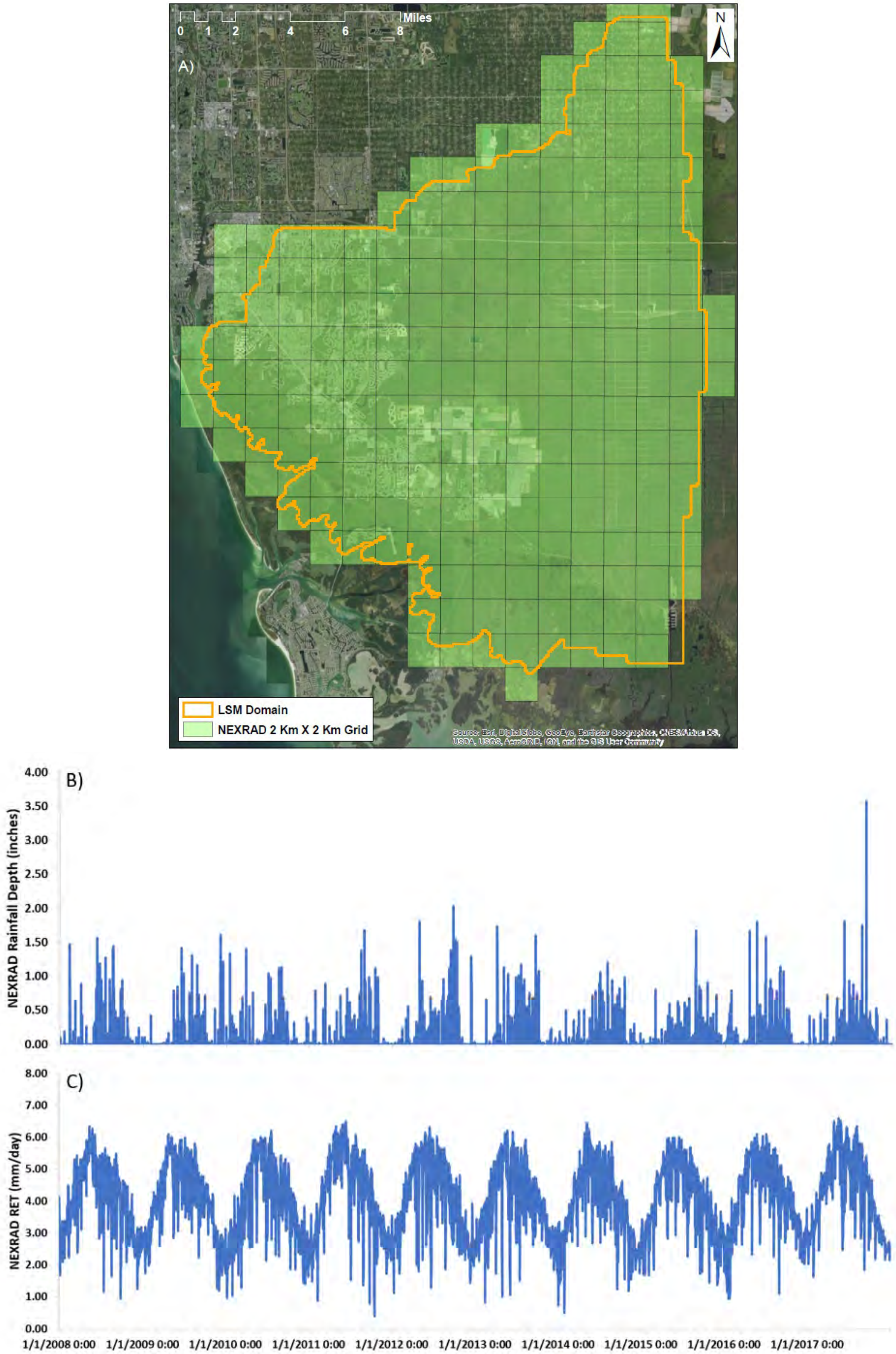


Figure 2.5 - A) NEXRAD Grid for LSM domain. B) & C) Time series of NEXRAD rainfall depths and referenced evapotranspiration (RET) rates (from 1/1/2008 to 12/31/2017), shown for one pixel (SFWMD Pixel ID # 100450) as an example.

2.3.1.2 Open Channel Flow/Canal Network

Rivers, man-made and natural canals, flow paths etc. that carry discharge as open channel flow are simulated in Mike 11. Since LSM covers a large area, it includes many critical flow path elements such as canals, channels, streams, flow structures (such as weirs, culverts, and bridges), road obstructions, and low water crossings. Accurate representation of such flow paths and elements is very important to effectively simulate open channel flow and interaction thereof with other components of the hydrologic cycle. All such important features in the project area were identified and included in the modeled hydraulic network (Figure 2.6). In the model, channel geometry is represented by cross-sections along the channel length at sufficient distances such that the changes in depth and/or width of the channel are well-captured. Whereas, the resistance to flow is represented using roughness coefficients, i.e. Manning's Coefficient (n), defined along the perimeter of the cross-section (Figure 2.7). For this model, most of the canal cross-sections and roughness coefficients were inherited from HC and BCB model. Where deemed necessary, revisions were made either in the cross-sections or coefficients to reflect updated and/or accurate field conditions. The hydraulic network in the project area can be divided into following categories.

<u>Major Canals</u>
Major canals are the canals that carry discharge from small flow-ways, ditches, residential agricultural and other project areas. In the CCWIP drainage system, major canals include, Faka-Union, Miller, I-75 South and North, Henderson Creek and Tamiami (Figure 2.6). These canals have several control structures, such as, weirs, culverts, control gates and bridges that regulates discharge throughout their drainage basins. All the structures, along with their operational rules, if any, were explicitly simulated in the model. Geometric and operational data for these structures were inherited from BCB and HC models. The structures were updated based on SFWMD revised structure database (SFWMD, 2016).

<u>Internal Drainage Systems</u>
Storm water from residential and agricultural developments is routed into major canals through drainage systems consisting of relatively small canals/ditches, detention storages, ponds and pumps. The drainage systems for on-site and off-site developments were included in all the scenarios to evaluate flooding impacts caused by the project. SixL's is an agricultural farm area that lies on the south of Belle Meade forest. SixL's hydraulic network comprises of farm canals that carry irrigation water from within a farm and discharge into designated detention storage areas through pumps. The discharge from detention basin flows through water quality structures into perimeter canals which ultimately discharges into Tamiami Canal at several outfall locations. The discharges from each farm are capped at the rates used by the USACE for the PSRP. The discharge rate from each farm were communicated by the USACE during the stakeholder meetings. Urban developments incorporated in the LSM include, Winding Cypress, Verona Walk and Naples Reserve on the west of the SixL's Area and north of Tamiami Canal, and the west side of Fiddler's Creek subdivision located south of Tamiami Canal. The hydraulic network within the urban development consists of stormwater ponds controlled by structures, such as, weirs and orifices. The discharge from each development is discharged into the forest or receiving waters as overland

flow at various outfall locations. The hydraulic network within each defined farm area or development is conceptually modeled in Mike 11, along with the pump operations and control structures, such that the operation of the farms/development is effectively and correctly represented. The canal geometric data, detention storages, pump capacities and control structure information are extracted from Environmental Resource Permit (ERP) ("Permits", n.d.) for the individual farms/developments where available. Missing information is reasonably assumed from the neighboring farms/developments. Urban developments on the west of Henderson Creek (Figure 2.6) along the bay were also included in the model using the same concept.

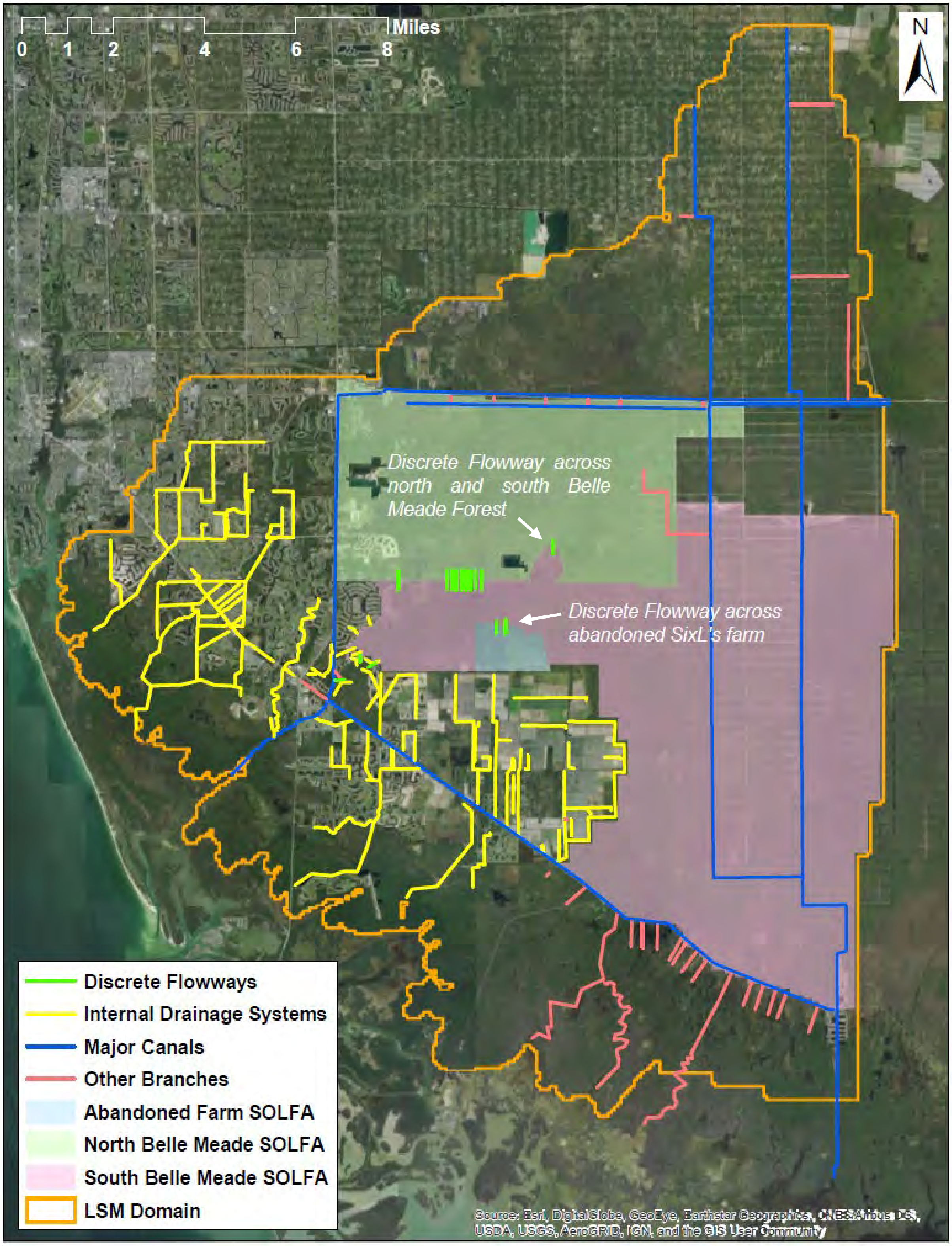


Figure 2.6 - MIKE 11 hydraulic network including; major canals, internal drainage systems, discrete flow-paths and other canals. Figure also shows conceptualization of breaches and low-lying areas allowing for inter-basin flow as MIKE 11 branches across SOLFAs.

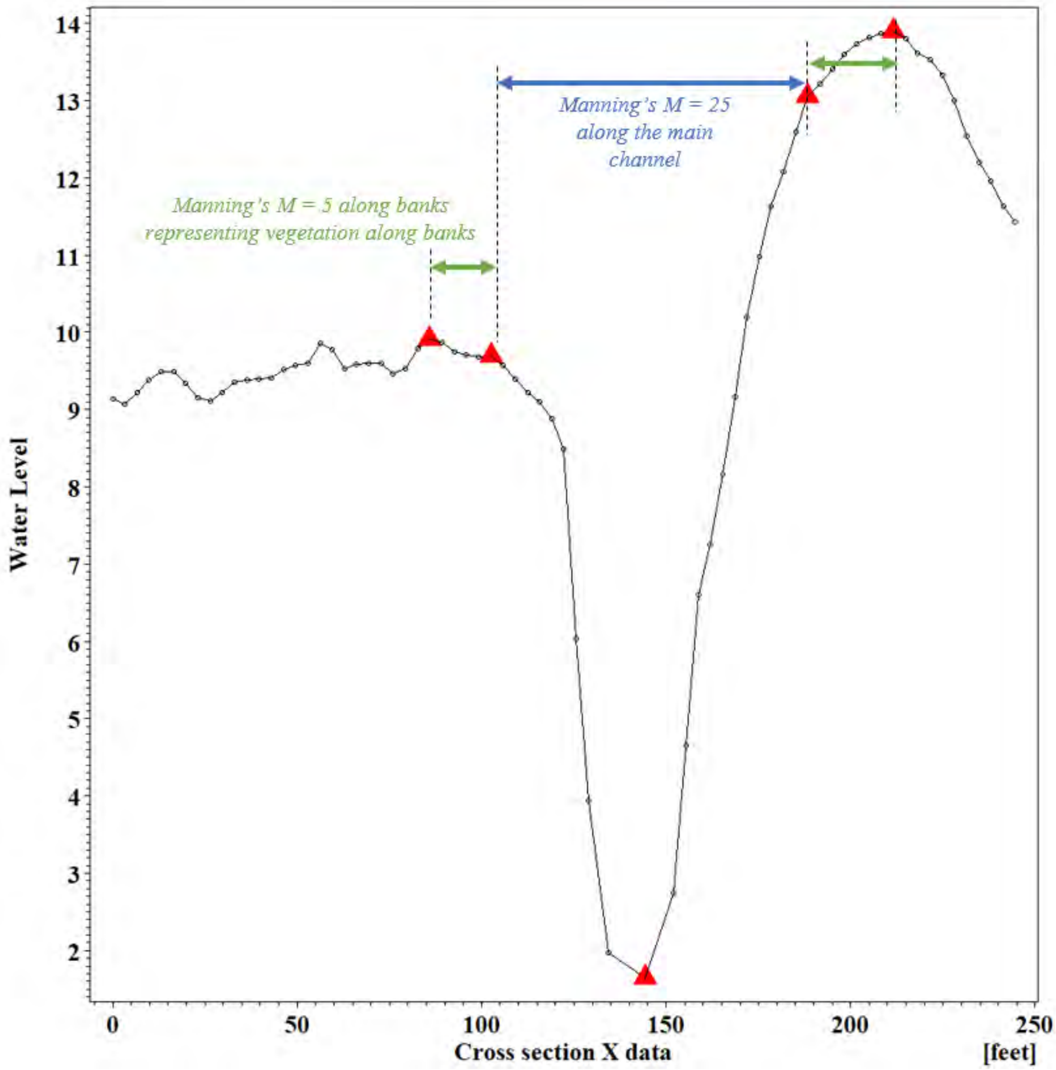


Figure 2.7 - A typical cross-section in MIKE 11. Channel conveyance is calculated based on marker positions, which are used to define the extent of channel banks and main channel.

Discrete Flow Paths and Inter-Basin Flows

There are several discrete, natural or man-made, flow-ways/flow-paths in the project area that transport water . These include shallow and narrow ditches, culverts, breaches, natural low-lying stretches etc. that allows discharge across berms or basin boundaries. Such branches are important in determining the overall flow pattern within and across areas of interest and were carefully evaluated and incorporated in the model where necessary. Winding Cypress Reserve is a new residential development located just south-east of the project flow-way. To facilitate historic flow under the subdivision access road constructed across the flow-way as part of the development, culverts and wildlife crossings have been built. Another flow-path is identified between south and north Belle Meade Forest, across Sabal Palm road. It was observed during field survey that the dirt road crossing the forest was low enough at one location (Figure 2.6) to allow for inter-basin flow especially during wet season of the year. This section of the road is an extension of Sabal Palm

Road known as Triple-G Loop. Similarly, the abandoned farm field north of Deseret farm is connected with the forest through two wide breaches located along the berm on the northwest side. The breaches allow overland flow from the forest to enter and rehydrate the abandoned field. In MIKE SHE, overland basins or areas defined by high points are represented by Separated Overland Flow Area (SOLFA) codes. The model does not allow 2-D overland flow across SOLFA boundaries; however, to simulate inter-basin flow, abovementioned flow-paths were modeled as short-length discrete MIKE 11 branches stretching across the two SOLFA's. The branches were coupled with MIKE SHE overland flow and allow for inter-basin flow transfer whenever water levels are high enough. The information about breach/channel and structure geometry is either extracted from ERP's where available, or was field surveyed, or reasonably assumed based on the digital elevation model (DEM) of the area.

MIKE 11 Initial and Boundary Conditions

In hydrodynamic simulation models, boundary conditions are required at the most upstream and downstream cross-sections of each branch. It is necessary to use real time and accurate boundary conditions representing the actual state of water level or discharge at the location. In this model, branch lengths were selected such that appropriate and effective boundary conditions can be defined. For all the major canals, branches were extended to the bay such that available observed tidal water levels ("NOAA", n.d.) can be used at the most downstream cross-sections. Whereas, for the most upstream cross-sections, simulated discharges from BCB model (see section 2.2) were used as boundary conditions. Furthermore, the locations of upstream cross-sections were selected such that they are considerably far from the areas of interest and hence the impact of boundary conditions is minimized.

For smaller hydraulic networks and discrete channels, a closed boundary condition was used for both the upstream and downstream cross-sections where warranted. One advantage of MIKE SHE/MIKE 11 integrated model is that for smaller discrete branches that lie completely within the project area, a closed boundary condition can be defined. A closed boundary represents a zero-flux condition across the boundary, however, the flow simulated by MIKE SHE---either, as overland, or variably saturated zone flow---in the vicinity of the boundary can be routed to the MIKE 11 cross-section by appropriately coupling it with MIKE SHE. This allows for real representation of the system and minimizes data needs.

Hydrodynamic simulations also require initial conditions that represent the state of system in the beginning of the simulation. Initial conditions are required at each calculation point and are linearly interpolated between the two cross-sections. It is important to use reasonable initial conditions that represent the real state of the system in the beginning as it impacts results especially in the beginning of the simulation. However, for longer simulation time periods, such as for this model, their impact diminishes soon into the simulation period. For this model, initial conditions, where available, were inherited from BCB and HC models. For new branches, reasonable initial conditions were assumed. Furthermore, for analysis of the simulated water levels and discharges, a reasonable spin-up period, e.g. 6 months to 1 year, as deemed appropriate, can be used to eliminate the impacts of initial conditions.

2.3.1.3 Topography

The topography of the project area is represented in the model using LiDAR topographic data available in the form of a 10ft by 10ft Digital Elevation Model (DEM) (Figure 2.8). The DEM dates to 2007 and 2008 and was the most recent DEM at the time of model development and has been approved for modeling purposes by the district. MIKESHE requires that the DEM has the same grid size as that of the model grid. The DEM was processed to match the model grid size, 375 ft by 375 ft. During pre-processing, median values from 10 ft DEM were used for resampling in GIS. This ensures that the topography along the channels is effectively represented in the model grid. Using median statistics, low points lying along channels are eliminated from the MIKE SHE grids, and the resulting grid represents land surface elevations in the vicinity of the channel rather than low points. The detailed topography of the channels is instead represented in MIKE 11 separately in terms of cross-sections. The vertical datum for the SFWMD LiDAR and MIKE SHE model is referenced to NAVD-88.

Within the project area, the land slopes naturally from the northeast to the southwest. While the 10-ft grid-cell size captures much more detail, the topography processed for the-LSM MIKE SHE models captures the natural slope and has an appropriate resolution to allow for accurate representation of the topography over the model domain. The preprocessed DEM was manually updated in the areas of recent development, such as, Winding Cypress, Verona Walk (south) and Naples Reserves based on the ERP’s.

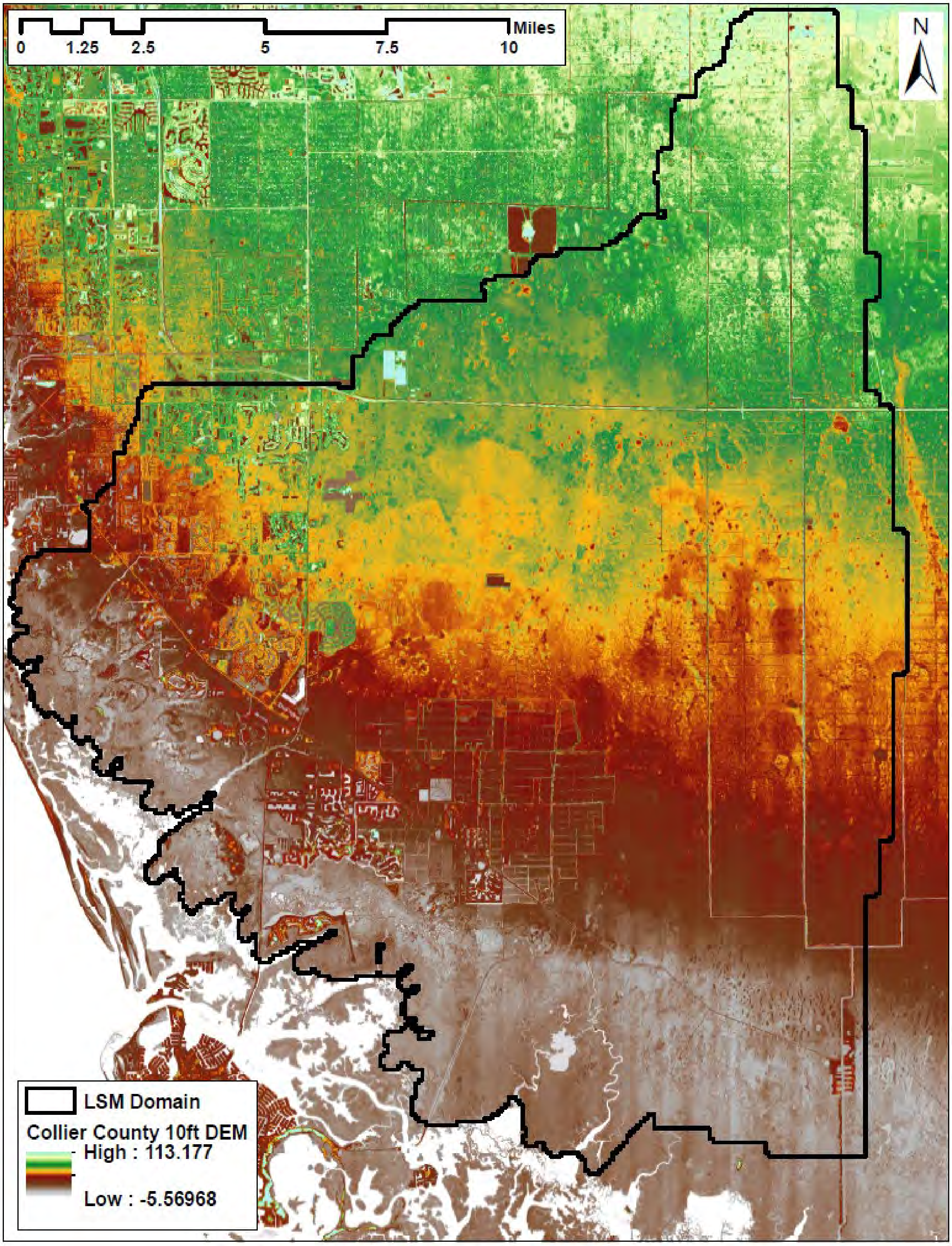


Figure 2.8 - SFWMD Digital Elevation Model for Collier County. The raster resolution is 10ft.

2.3.1.4 Land Use

Land use data for the project area was inherited from the HC and BCB models which were originally obtained from the SFWMD land use database, developed based on 2008 and 2009 aerial imagery. The data was pre-processed to effectively represent the land use distribution in the model. Originally, based on FLUCCS codes, the project area had 86 different land use types. These land uses were grouped into 23 categories based on similarity in hydrological characteristics. The land use data was updated in 2011 to reflect the latest conditions. This was incorporated in the model land use grid by taking a representative FLUCCS code value for each land use type over a 375-ft grid-cell resolution. Furthermore, to reflect recent developments in the project area, modeled land use grid was modified based on 2018 aerial imagery. This includes, Winding Cypress, Verona Walk (south) and Naples Reserves residential developments. As can be seen from Figure 2.9, the project area is dominated by wetland and forested land use categories while urban land use and water make up 13.6% and 3.5%, respectively. These land use categories are expected due to the Belle Meade forest, the extensive Mangrove and Swamp Forests along the coastline, and the number of retention ponds and canals throughout the model domain.

Vegetation Types

Land use within the model domain predominantly consists of 23 distinct vegetation types (Figure 2.9). Although not a vegetation type, it is customary to categorize water and urban developments as vegetations in MIKE SHE land-use module. Vegetation data is linked with ET and Irrigation module of the model and hence actual ET and irrigation demand is calculated based on crop type, development stage and irrigation requirements specified by the user. Yearly crop development pattern and irrigation requirement was defined in the vegetation property file for each crop. Since the crop development pattern and irrigation demand is assumed to remain the same year to year, yearly patterns were repeated over the entire simulation period. In this model, all the vegetation properties were adopted from HC and BCB models.

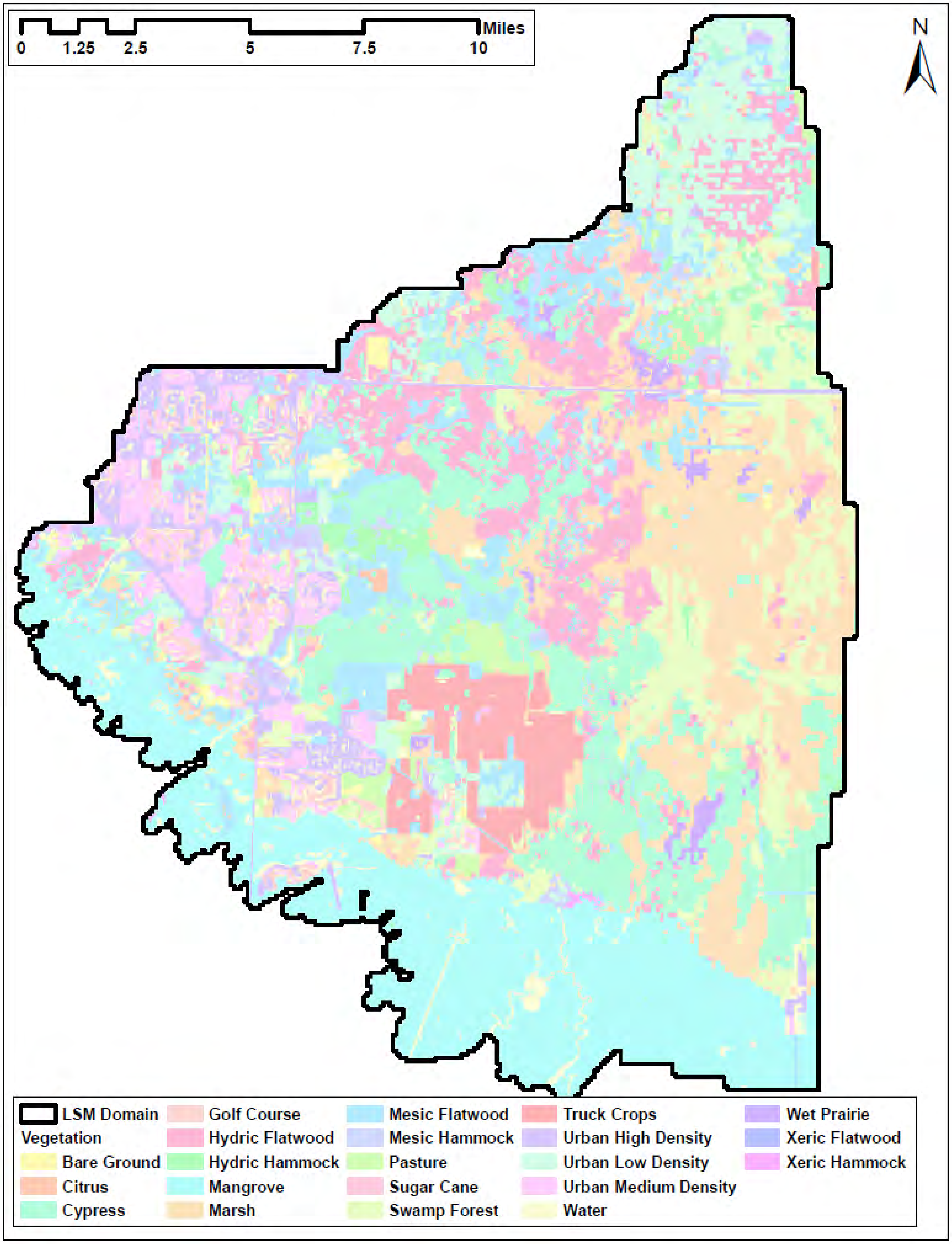


Figure 2.9 - Land use map for LSM. Different vegetation communities shown are described in the legend.

2.3.1.5 Overland Flow

Overland flow is the movement of water over the land, downslope toward a surface water body. When the rainfall intensity exceeds the soil infiltration capacity in an area, then water accumulates on the soil and after satisfying evapotranspiration and infiltration losses, it starts moving downslope, due to gravity, towards the hydraulic network. Since one of the main objectives of this project is to restore historic overland flow through the Belle Meade forest area, special emphasis was placed on the explicit simulation of overland flow in required areas. A rigorous overland flow calculation was performed over the entire modeled domain using the diffusive wave approximation of the Saint Venant equations (DHI, 2019). Following are the important factors which affect overland flow calculations and are detailed below.

Manning's M

Flow velocities over the land surface are influenced by the resistance offered by land cover and land use type. For instances, a densely vegetated forest offers higher resistance to flow than an urban area with impervious surfaces. In the model, this roughness, which is a characteristic of land use type, is represented by a roughness coefficient, Manning's M. Manning's M is the inverse of Manning's "n" and is inversely related to the resistance offered by the land use type. Consequently, a higher M value represents a land use that is less resistant, e.g. concrete surface in a residential development, and vice versa. Manning's M values are based on the vegetation types as defined in land use module and were adopted from HC and BCB models. For new residential developments added in the LSM, such as, Naples Reserves, Winding Cypress (south) and Fiddler's Creek (West), Manning's M values were updated to reflect current conditions. (Table 2.4).

Table 2.4: Manning's M values for different vegetation types.

Hydrologic Land Use	OL Manning's M
Citrus	5.88
Pasture	7.14
Sugar Cane/Sod	5.88
Truck Crops	5.88
Golf Course	7.14
Bare Ground	11.36
Mesic Flatwood	5
Mesic Hammock	3.33
Xeric Hammock	5
Hydric Flatwood	4
Hydric Hammock	2.5
Wet Prairie	3.33
Marsh	2.33
Cypress	3.33
Swamp Forest	2.5
Mangrove	5
Water	16.67
Urban Low Density	7.14

Urban Medium Density	8.33
Urban High Density	9.01

Detention Storage

Part of the overland water gets trapped in small depressions in the ground surface before it can flow as sheet flow to the adjacent cell. However, these depressions may be too small to be captured effectively in the topographic grid used in the model. To handle such situations, equivalent detention storages can be defined on each cell. The depth of ponded water must exceed the detention storage before water will flow as sheet flow to the adjacent cell. For example, in this model, the detention storage for the Belle Meade forest was set equal to 0.4 inch, which means that the depth of water on the forest surface must exceed 0.4 inch in each cell before it will be able to flow as overland flow to the adjacent cell. This detention storage value represents small sub-grid scale depressions that exist in the forest area but are not effectively captured in the model DEM. This is equivalent to the trapping of surface water in small sub-grid ponds or depressions within the project area. Detention storage values depends on the vegetation types (Table 2.5) as defined in the land use module and were adopted from HC and BCB models. The values for urban land use categories were obtained from the EPA SWMM-5 manual and other sources.

Table 2.5: Detention Storage values for different land use types.

Hydrologic Land Use	Detention Storage (inch)
Citrus	0.3
Pasture	0.25
Sugar Cane/Sod	0.25
Truck Crops	0.25
Golf Course	0.3
Bare Ground	0.15
Mesic Flatwood	0.4
Mesic Hammock	0.4
Xeric Hammock	0.4
Hydric Flatwood	0.4
Hydric Hammock	0.4
Wet Prairie	0.4
Marsh	0.4
Cypress	0.4
Swamp Forest	0.4
Mangrove	0.4
Water	0.00
Urban Low Density	0.1
Urban Medium Density	0.1
Urban High Density	0.1

Separated Overland Flow Areas

Linear man-made obstructions, such as, roadway embankments, canal berms, berms around agricultural/residential developments etc., may exist within the watershed that prohibit overland water to flow across those highpoint boundaries. However, these highpoint boundaries may be too narrow to be captured effectively in the topographic grid used in the model. Separated Overland Flow Areas (SOLFA) are used in MIKE SHE to represent those sub-grid boundaries for overland flow. It is important to account for such boundaries as they determine the amount of overland flow within and across areas of interest. The LSM was divided into SOLFA's based on topographic/aerial information, ERP's and field surveys. For example, the Belle Meade Forest was divided into north and south SOLFA's separated by Sabal Palm Rd. This prohibits the model from letting the overland water to move from north to south Belle Meade Forest. The SOLFA's were inherited from BCB and HC models. New SOLFA's were added to further refine the overland flow boundaries in the areas of interest. These include individual farms in SixL's Agricultural Area, sub-basins in Winding Cypress, Verona Walk, Fiddler's Creek and the recently constructed tie-back levee on Faka-Union and Miller canals at the USACE pump stations (Figure 2.10).

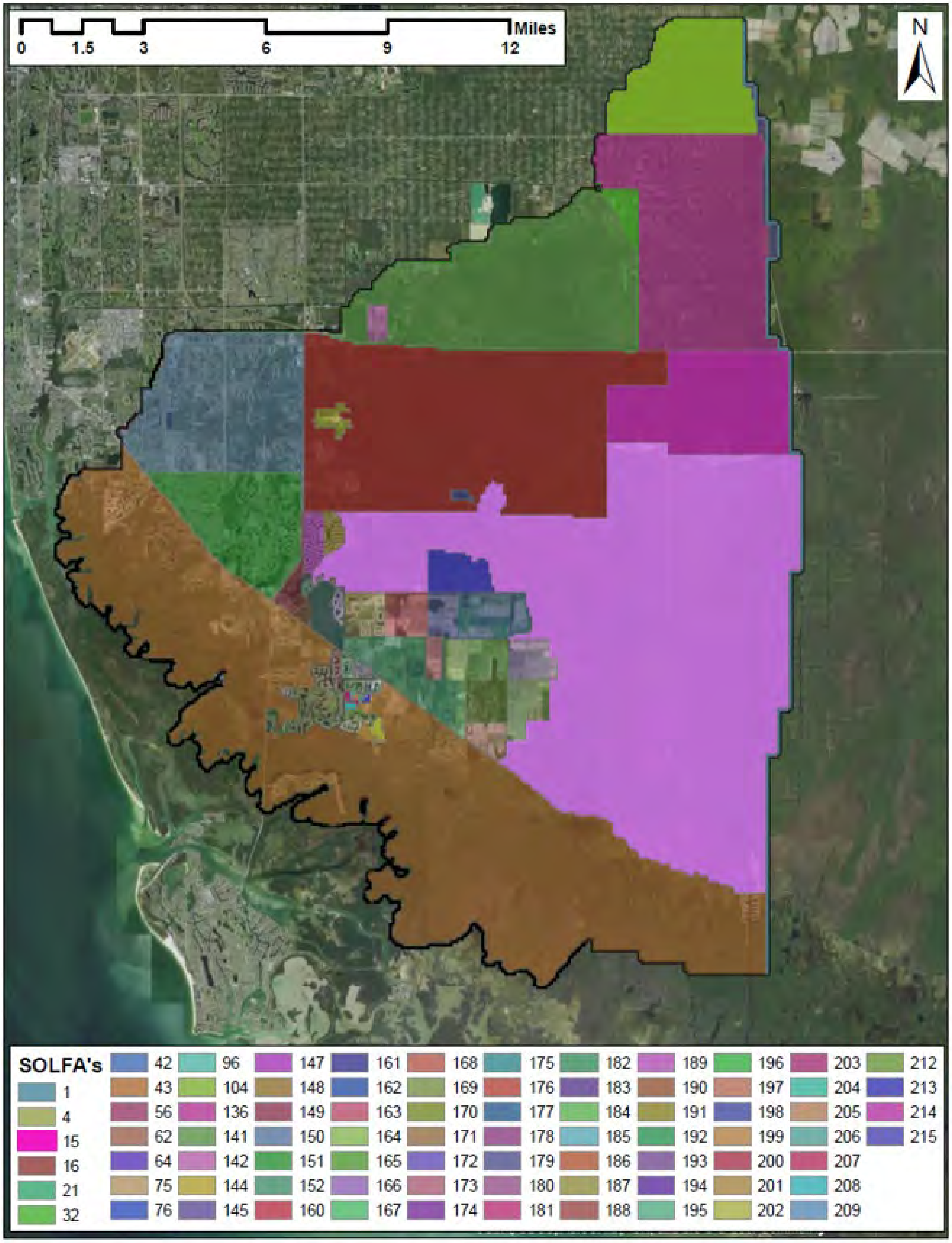


Figure 2.10 - Separated Overland Flow Areas (SOLFA's) representing overland flow boundaries within LSM domain.

Ponded Drainage

A portion of water accumulated on the urban land surface, instead of flowing as sheet flow, may quickly reach a nearby natural or man-made detention or any nearby canal by means of man-made drainage systems. For example, urban stormwater drains and ditches/creeks along a roadway discharging water away from road to a designated stormwater pond. These relatively fine-scale drainage systems are difficult and impractical to model explicitly in large-scale models. However, a considerable portion of the overland flow may be routed through such drain especially in highly urbanized areas. This is achieved conceptually in MIKE SHE by using Ponded Drainage option (Figure 2.11). In this option, a specified portion of overland flow in each catchment is routed to a nearby river network, a natural depression or outside the model area whatever is the case. The runoff coefficient is used to define the fraction of ponded water that drains to storm sewers and other surface drainage features in paved areas. For example, a value of 0.45 corresponds to 45% of the ponded water on the cell to be added directly to the drain, whereas, the remaining 55% to be routed as sheet flow using overland flow calculations. Table 2.6 lists runoff coefficients used in the model for different urban developments. A value of 0 was used for Belle Meade forest indicating no urban drainage system exist in the area and water flows as sheet flow.

Since overland flow that discharges to drains is also limited by basin boundaries, it is important to establish a source-sink network that reflects the actual conditions in the watershed. This is achieved in the model by specifying a unique drain code to each catchment area which then drains the entire catchment with the same code to one source. Furthermore, options are available to control drainage to a specified canal rather than allow the model to determine locations. This option is used for the SixL's Area, where runoff generated within each farm only drains to the canals within the farm. Another important factor to be considered in this regard is the time it takes ponded water to reach and show up at the designated sink. This is controlled in the model by Inflow and Outflow Time Constants (Table 2.7). New developments added in the LSM include, SixL's Area, Winding Cypress, Verona Walk, Naples Reserves and Fiddler's Creek.

Table 2.6: Paved runoff coefficients used in LSM for different densities of urban developments.

Hydrologic Land Use	Paved Runoff Coefficient
Urban Low Density	0.05
Urban Medium Density	0.15
Urban High Density	0.45

Table 2.7: Inflow and Outflow Time Constants used for urban developments.

Land Use	Inflow Time Constant (1/sec)	Outflow Time Constant (1/sec)
Urban High Density	0.001	0.001
Urban Medium Density	0.001	0.001
Urban Low Density	0.0005	0.0005

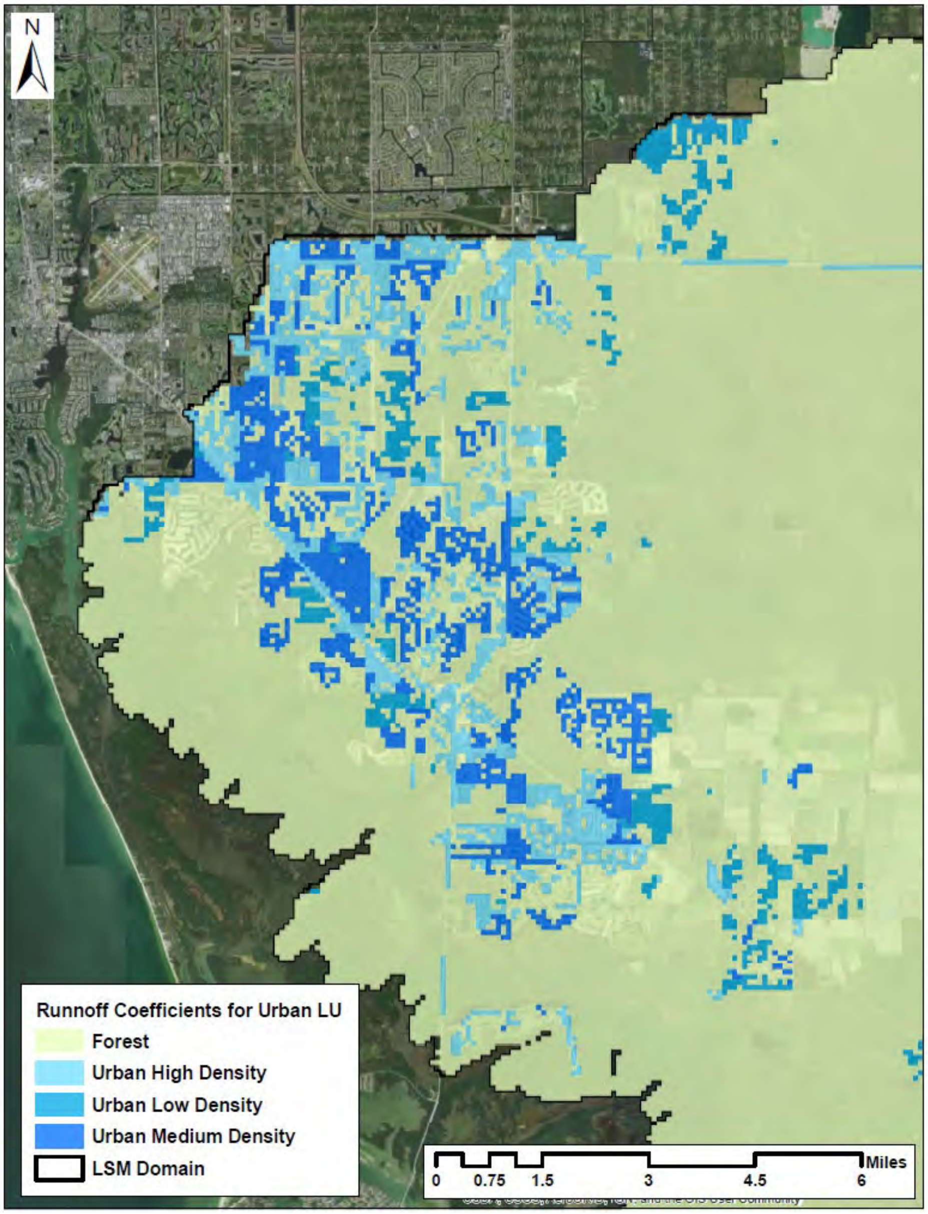


Figure 2.11 - Conceptualization of urban storm water drainage systems in residential developments as ponded drainage routine in MIKE SHE.

2.3.1.6 Unsaturated Zone Flow

The unsaturated zone is the portion of the subsurface above the groundwater table. It contains, at least some of the time, air as well as water in the pores. It is usually heterogeneous and characterized by cyclic fluctuations in the soil moisture as water is replenished by rainfall and removed by evapotranspiration and recharge to the groundwater table. Soil type distribution and hydraulic characteristics for each soil type is required to model unsaturated zone flow. In the project area, the soil data, obtained from SFWMD soil database, "sorunt", contained 39 different soil classifications. To effectively represent the soil distribution with reasonable simulation run-time, the soils were re-classified into five distinct types based on hydrologic soil drainage classification (Figure 2.12). It is customary to categorize water as a soil type in MIKE SHE because the unsaturated flow module is linked with Evapotranspiration module and the actual uptake of water by the plant root is estimated by model based on soil moisture conditions in the unsaturated root zone.

The drainage classification adopted in this model is related to the position of the water table, where the soil allows infiltration until the wetting front meets the water table at variable depths, depending on season, soil type, and other land use or water control practices. Once the wetting front reaches the water table, infiltration no longer occurs, and the soil is considered saturated. The hydraulic characteristics for the most dominant soil type (area wise) within each hydrologic soil class were adopted for the entire class.

As explained before, a fully distributed 1D solution based on Richard's equation was used in this model, which requires hydraulic parameters such as Saturated Hydraulic Conductivity and Retention Curve for each soil type. The soil properties were adopted from HC and BCB models and are defined in the Soil Property file. The relationship between hydraulic conductivity and moisture content can be estimated in MIKE SHE using several empirical relationships. For this model, Averjanov Method was used (DHI, 2019).

<u>Soil Profiles & Vertical Discretization</u>
The soil profile is defined by vertical discretization of soil type into various soil horizons each with varying hydraulic properties. The unsaturated hydraulic parameters for each soil horizon were adopted from HC and BCB models and are reproduced below (Table 2.8).

Table 2.8: Vertical discretization of soil profiles associated with each soil type shown in Figure 2.12. Unsaturated hydraulic parameters are also shown for each soil type.

Soil Type	Drainage Class	From Depth	To Depth	Horizon	Hydraulic Parameters				Retention Parameters			
					K_{sat} (ft/day)	θ_{sat}	θ_r	Averjanov n	θ_{eff}	θ_{fc}	θ_{wp}	Green & Ampt Suction at WF (ft)
Paola Fine Sand	Excessively Drained	0.000	0.492	A	49.170	0.440	0.010	5.400	0.440	1.810	4.180	-3.280
		0.492	1.247	E	55.390	0.357	0.010	5.400	0.357	1.810	4.180	-3.280
		1.247	2.165	BW	51.770	0.351	0.010	5.400	0.351	1.810	4.180	-3.280
		2.165	3.740	BA	55.349	0.301	0.010	5.400	0.301	1.810	4.180	-3.280
		3.740	5.315	BW/E//Bh	41.930	0.327	0.010	5.400	0.327	1.810	4.180	-3.280
		5.315	98.425	C	41.880	0.327	0.010	5.400	0.327	1.810	4.180	-3.280
Pineda Sand	Poorly Drained	0.000	0.262	A	11.424	0.510	0.020	8.168	0.510	1.810	4.180	-3.280
		0.262	0.656	Btg/E	0.867	0.361	0.020	14.154	0.361	1.810	4.180	-3.280
		0.656	1.247	BW1	22.592	0.449	0.020	7.815	0.449	1.810	4.180	-3.280
		1.247	2.756	BW2	18.111	0.422	0.010	7.160	0.422	1.810	4.180	-3.280
		2.756	4.331	BW3	10.743	0.389	0.012	7.929	0.389	1.810	4.180	-3.280
		4.331	5.249	CG	3.007	0.389	0.100	21.896	0.389	1.810	4.180	-3.280
		5.249	6.180	E	22.961	0.464	0.020	7.675	0.464	1.810	4.180	-3.280
		6.180	98.425	E1	15.157	0.408	0.020	6.529	0.408	1.810	4.180	-3.280
Satellite Fine Sand	Somewhat Poorly Drained	0.000	0.330	A	33.889	0.484	0.020	5.400	0.484	1.810	4.180	-3.280
		0.330	3.350	C1/C2	20.980	0.374	0.009	5.400	0.374	1.810	4.180	-3.280
		3.350	98.425	C3	21.730	0.399	0.005	5.400	0.399	1.810	4.180	-3.280
Pomello Fine Sand	Moderately Well Drained	0.000	0.492	A	17.638	0.435	0.020	5.400	0.435	1.810	4.180	-3.280
		0.492	3.838	E1\E2	21.988	0.382	0.008	5.400	0.382	1.810	4.180	-3.280
		3.838	4.003	Bh1	12.440	0.371	0.010	5.400	0.371	1.810	4.180	-3.280
		4.003	4.331	Bh2	9.330	0.416	0.030	5.400	0.416	1.810	4.180	-3.280
		4.331	4.921	BC1	19.170	0.379	0.010	5.400	0.379	1.810	4.180	-3.280
		4.921	5.512	BC2	17.630	0.010	0.010	5.400	0.010	1.810	4.180	-3.280
		5.512	98.425	C	19.685	0.323	0.004	5.400	0.323	1.810	4.180	-3.280
Plantation Muck	Very Poorly Drained	0.000	0.755	OAp	80.353	0.770	0.150	13.000	0.770	2.000	4.180	-3.280
		0.755	1.575	A/E	23.779	0.491	0.022	9.512	0.491	2.010	4.180	-3.280
		1.575	98.425	BW	32.598	0.392	0.002	6.101	0.392	2.010	4.180	-3.280

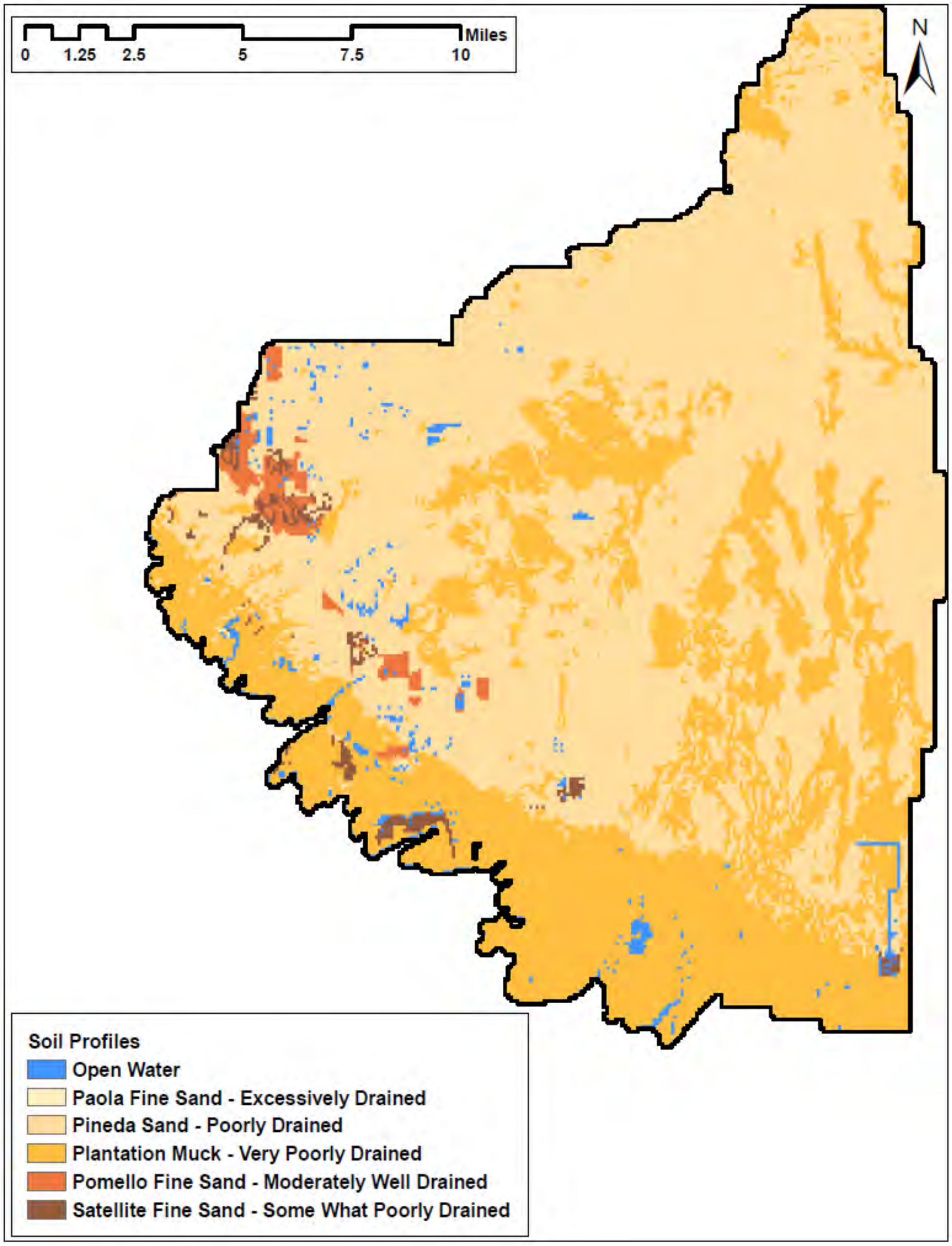


Figure 2.12 - Soil types classification with in LSM domain.

2.3.1.7 Saturated Zone Flow

The saturated zone encompasses the area below ground in which all interconnected openings within the geologic medium are completely filled with water. The saturated zone is conceptually viewed as multiple layers of aquifers and aquitards. In this model, a four-layer saturated zone is deemed sufficient to effectively achieve the modeling objectives. These layers include: Water Table Aquifer, Tamiami Confining Unit, Lower Tamiami Aquifer and Upper Hawthorn Confining Unit. In the MIKE SHE/MIKE 11 integrated model, exchange between canals and aquifers is explicitly simulated. Within most of the project area, the Water Table Aquifer is deep enough such that major exchange between canal and groundwater water occurs in this aquifer. Some small localized exchanges with a lower formation, the Lower Tamiami Aquifer, are also possible, however, the impact of further lower formations on surface water can be safely neglected. The hydrogeologic information for each layer was available from HC model. Similarly, saturated hydraulic characteristics, such as, horizontal and vertical conductivities, specific yield and specific storage, for each geological layer were inherited from HC model without any modifications. Each layer has been divided into zones of same hydraulic conductivities. For example, the Water Table Aquifer was divided into five zones of same conductivity values for each of the vertical and horizontal conductivities. The values have been previously calibrated in the HC model against observed water table data.

Since the model was cut off at Upper Hawthorn Confining Unit (CU), the lower lying Sandstone Aquifer was not explicitly modeled. However, it is anticipated that small localized zones of relatively high conductivity might exist in the confining unit which may allow for flux exchange between Tamiami and Sandstone Aquifer and hence impact head in the upper formations. This is achieved in the model by using simulated heads in the Sandstone Aquifer as the lower boundary condition for Upper Hawthorn Confining Unit. This allows for any possible flux exchange across the Upper Hawthorn Confining Unit. The head distribution was extracted for the period of record from the BCB model (see section 2.2 for details) and was used as a Head-controlled Flux boundary (DHI, 2017). This boundary condition was used to conceptually simulate specific fluxes, instead of heads, coming from the Sandstone Aquifer. Using vertical hydraulic conductivity estimates of Sandstone Aquifer as Leakage Coefficients, a flow resistance was incorporated. This resistance conceptually represents the Sandstone Aquifer hydrogeology.

Pumping Wells

Several pumping wells exist in the project area. Majority of the wells lie on the east of Henderson Creek and South of I-75 Canals, and along west of Faka Union Canal (Figure 2.13). Observed pumping rates were available for each well for some days in the simulation period from SFWMD online database. Since, most the wells pump water from Water Table Aquifer, it was deemed important to include those in the simulation as it can locally impact groundwater table over short periods of time.

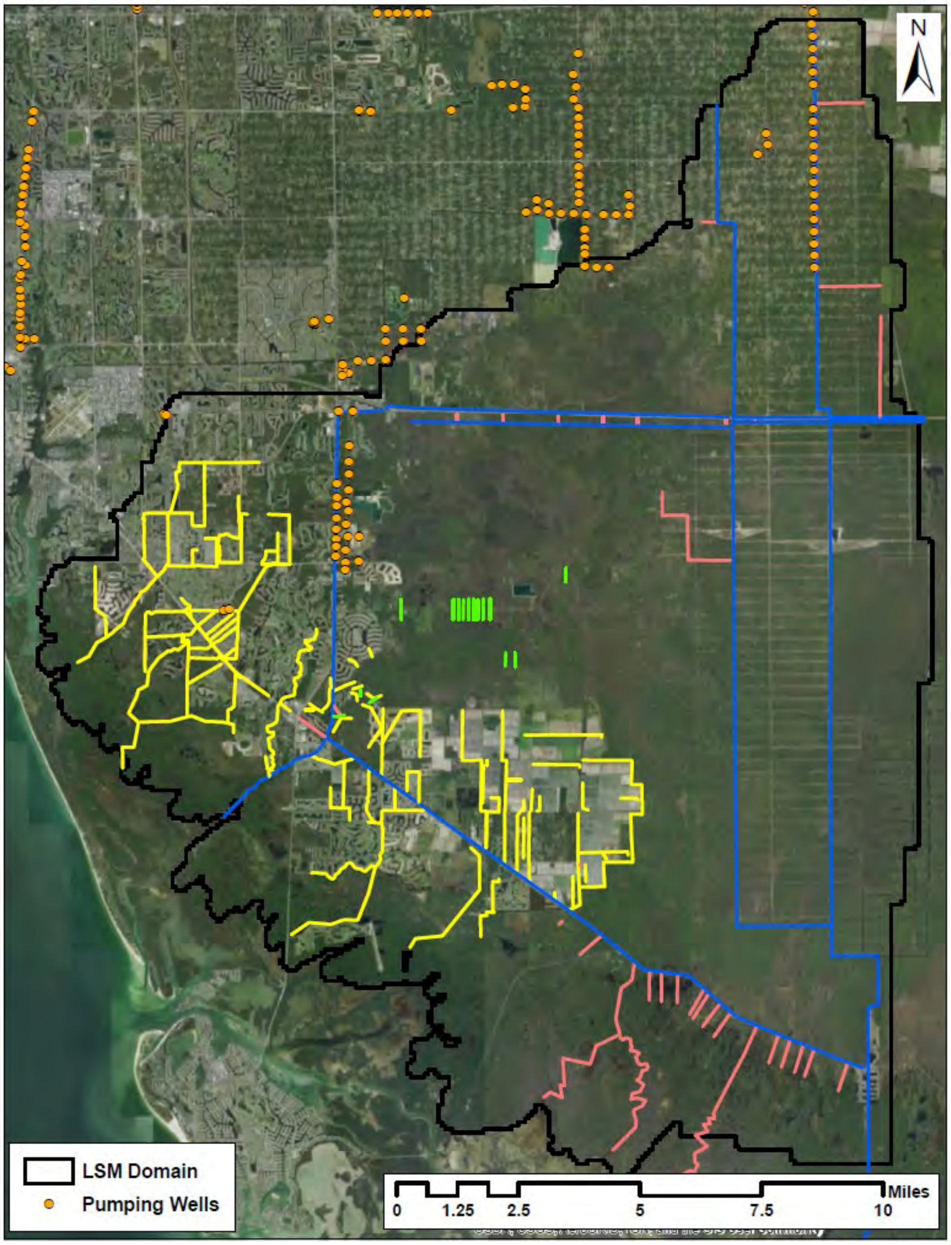


Figure 2.13 - Location of pumping wells in the area. Only those located within the LSM domain are included in the model.

Initial and Boundary Conditions

Saturated zone calculations require boundary and initial conditions along the perimeter of each layer at the outer boundary cells and over the entire model domain, respectively. Whereas, the actual heads are calculated on all internal grid cells during the simulation, the exchange with the boundaries is calculated depending upon the head difference between internal and boundary cells. It is important to use appropriate boundary conditions especially for longer simulation, as is the case with this model. Hence, a regional-scale model, BCB was used for this purpose (see section 2.2). Simulated heads from each aquifer layer, extracted from the BCB model results, were used as boundary conditions at their respective layers in the LSM.

For the water table aquifer, the boundary was divided into segments along east side, parallel to the coast and north side. The east side boundary was represented by a time varying specified head boundary condition, based on observed water table data. The observed water table, available at several well locations in the project area especially along the eastern boundary, was interpolated on a 375 ft X 375 ft grid to develop a head distribution over the entire model domain. Along the coast, a constant head of -0.51 feet, representing average sea level above the aquifer bottom was specified. Along the northern boundary, simulated heads from BCB model were used as time varying specified head boundary condition. For both the confining units, a no-flux boundary was used over the entire boundary perimeter. This represents that no lateral flow is assumed to occur through the confining units. For Lower Tamiami Aquifer, simulated heads from BCB model were used as time varying specified head boundary condition over the entire boundary perimeter.

Initial potential heads for all the four layers were inherited from the BCB model for the entire model domain. Although initial conditions impact the simulated results, for longer simulation periods, as is the case with this model, their impact diminishes soon into the simulation period.

Saturated Zone Drainage

Small drainage features, such as, ditches alongside roads, swales, underdrains etc., exist in the watershed that can drain significant amount of groundwater from the aquifers, especially in shallow aquifers during high water table conditions. It is impractical to explicitly model such features in MIKE 11, however, over large areas, groundwater drainage may form a significant portion of the overall water budget. This is achieved conceptually in MIKE SHE by using the Saturated Zone Drainage routine. In this option, groundwater in the aquifer layers is routed to a nearby river network, a natural depression or outside the model area whatever is the case. The controlling factor is depth of the water table below the ground surface. The drainage depth specification allows the model to calculate drainage from the aquifer based upon the groundwater head and drain level, as well as determining from which layer (aquifer) groundwater will drain. For example, if depth is specified to be -2 feet, whenever the water table rises during simulation and reaches 2 feet below the ground, water is removed from the aquifer and is added to the specified location. Drainage depths are only associated with agriculture and urban land use categories. All other natural land uses such as forest, wetland, or marsh have a 0-drainage depth in the model (Table 2.9).

Table 2.9: Drainage depths for different vegetation/land use types used in the saturated zone drainage routine.

Hydrologic Land Use	Drainage Depth (feet)
Citrus	4
Pasture	1
Sugar Cane/Sod	1
Truck Crops	2
Golf Course	1
Bare Ground	0
Mesic Flatwood	0
Mesic Hammock	0
Xeric Hammock	0
Hydric Flatwood	0
Hydric Hammock	0
Wet Prairie	0
Marsh	0
Cypress	0
Swamp Forest	0
Mangrove	0
Water	0
Urban Low Density	2
Urban Medium Density	2.5
Urban High Density	3

Since groundwater flow that discharges to drains is also limited by basin boundaries, it is important to establish a source-sink network that reflects the actual conditions in the watershed. Similar to the ponded drainage routine, this is achieved in the model by specifying a unique drain code to each catchment area which then drains the entire catchment with the same code to one sink (Figure 2.14). Furthermore, options are available to control drain to a specified canal rather than model determine locations. This option was used for SixL's Area in this model, where groundwater from within each farm only drains to the canals within the farm. Another important factor to be considered in this regard is the time it takes for the groundwater to reach and show up at the designated sink. This is controlled in the model by specifying Time Constants (Table 2.10).

Table 2.10: Time constants for different soil types used in LSM.

Soil Drainage Class	SZ Drain Time Constant (1/day)
Excessively Well Drained	0.01
Moderately Well Drained	0.001125
Somewhat Poorly Drained	0.00075
Poorly Drained	0.0005
Very Poorly Drained	0.00025
Water	0

New developments added in the saturated zone drainage include, SixL's Area, Winding Cypress, Verona Walk, Naples Reserves and Fiddler's Creek.

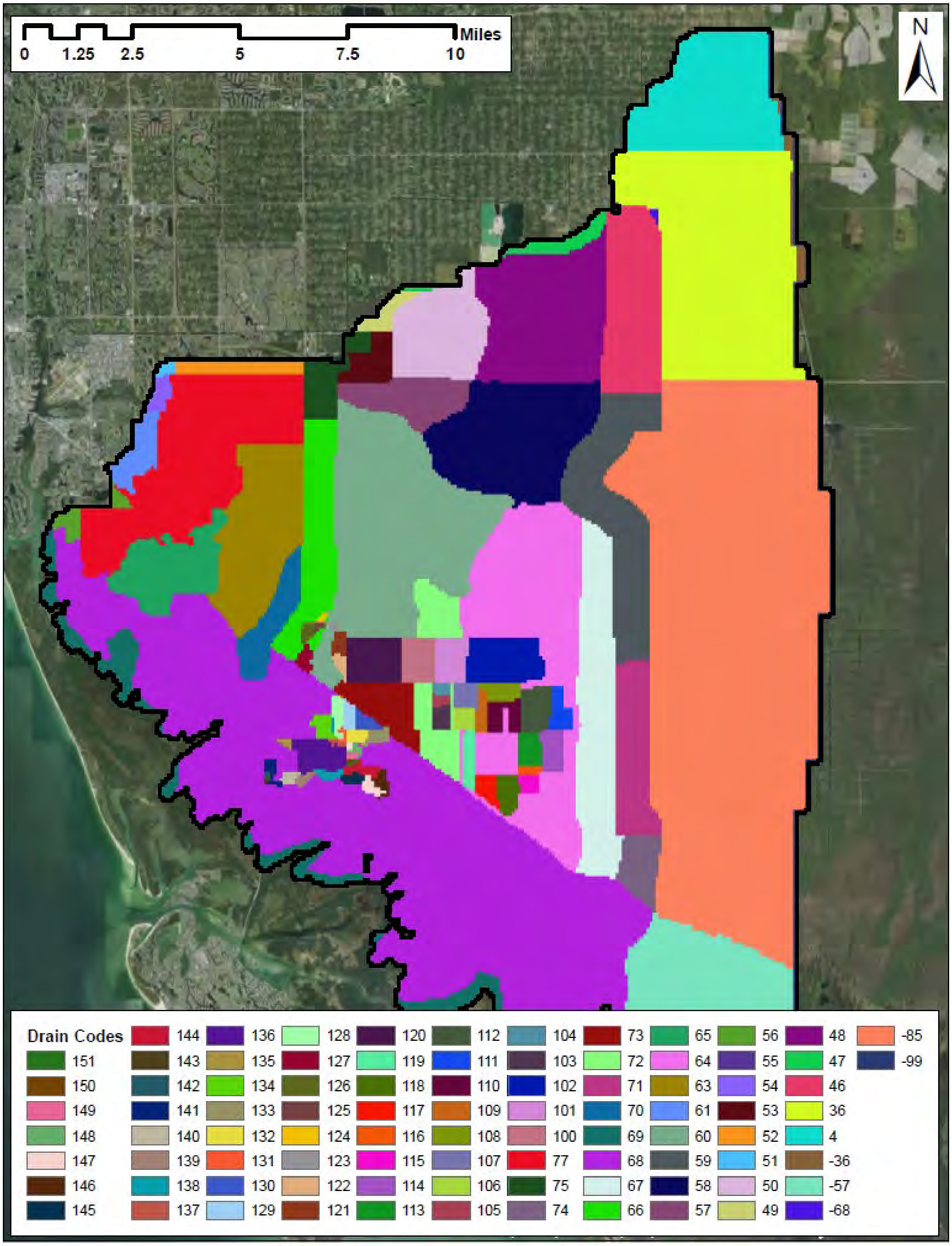


Figure 2.14 - Drainage boundaries for water table aquifer used to model saturated zone drainage. Within residential and agricultural developments, drain codes are used to represent groundwater drainage to internal canals and ponds.

2.3.1.8 Irrigation

The project area contains several agricultural areas, golf courses and urban land uses that are irrigated with either water table aquifer using shallow wells or treated water from waste treatment plants. Several irrigation areas are identified and are explicitly simulated in this model. In the MIKE SHE irrigation routine, irrigation areas are divided based on the source of irrigation water and are referred to as Irrigation Command Areas (ICA's). Given the objectives of this model, the focus of irrigation simulation is geared towards estimating the quantity of water being used for irrigation and its impacts on water level in the source where it is coming from. Considering that, the following main sources of irrigation water were identified in the project area: Water Table Aquifer, Lower Tamiami Aquifer and Treated Wastewater. The irrigation areas were grouped into nine ICA's based on the source of water (Figure 2.15).

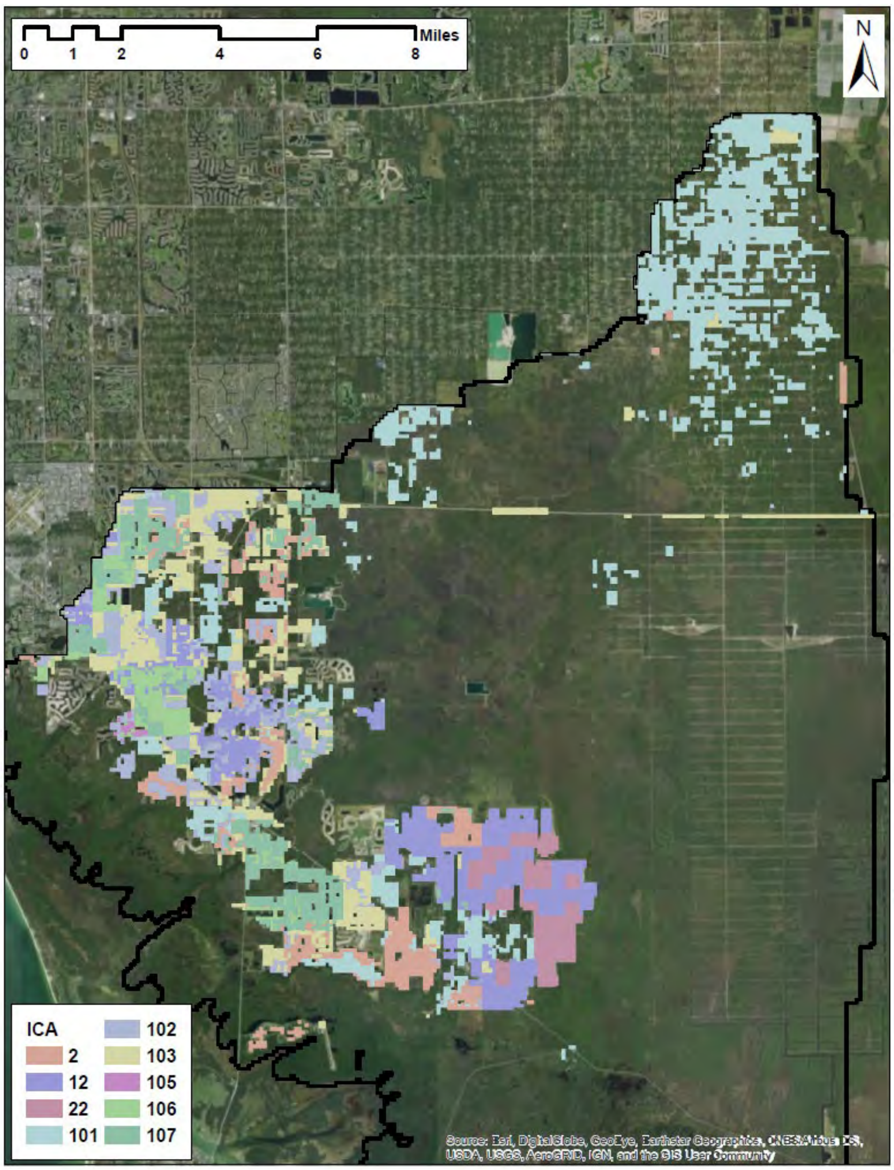


Figure 2.15 - Residential, golf course and agricultural areas included in the irrigation routine.

The quantity of water required for irrigation, knows as irrigation demand, for each crop type can be calculated in different ways in MIKE SHE. For this model, it was deemed appropriate to calculate the demand based on soil moisture deficit in the crop root zone. This method is called "Maximum Allowed Deficit". The model calculates the soil moisture characteristics and based upon user specified soil moisture deficit, applies irrigation water until a specified deficit threshold is met. The Maximum Allowed Deficit for irrigation is variable for each crop type and is based on the field capacity of the soil. This is defined in the Vegetation module of MIKE SHE. Irrigation is started in the model when the deficit exceeds the moisture deficit start value and stops at the moisture deficit end value. For instance, if user defines a start value of 0.6 and stop value of 0, this means that when the 60 % of the maximum water available in the root zone is consumed, model will replenish the root zone until field capacity of the soil is reached again.

Table 2.11: Cypress development for a year, shown as an example of vegetation properties defined in the model. The annual pattern is repeated over the entire simulation period.

End Day	Leaf Area Index	Root Depth	Crop Coefficient Kc
0	2	60	0.8
31	2	60	0.8
59	2	60	0.8
90	3	60	0.8
120	3.5	60	0.8
151	4	60	0.8
181	4	60	0.8
212	4	60	0.8
243	4	60	0.8
273	3.5	60	0.8
304	3	60	0.8
334	2.5	60	0.8
365	2	60	0.8

Table 2.12: Cypress irrigation demand for a year, shown as an example of vegetation properties defined in the model. The annual demand is repeated over the entire simulation period.

End Day	Moisture Deficit Start	Moisture Deficit Stop	Reference
0	0.25	0.2	
31	0.25	0.2	
59	0.25	0.2	
90	0.25	0.2	
120	0.25	0.2	Field Capacity
151	0.25	0.2	
181	0.25	0.2	
212	0.25	0.2	
243	0.25	0.2	

273	0.25	0.2
304	0.25	0.2
334	0.25	0.2
365	0.25	0.2

Many ICA's, that pull water from aquifers have several small wells located within the farm. It is not feasible to model all wells individually, instead, in MIKE SHE, they can be modelled collectively for each ICA as shallow wells which pull water from the aquifer within the ICA code at user specified depths. For ICA's that use treated wastewater for irrigation, an external water source is used in the model as the water does not come from within the modeled domain. The treated wastewater is supplied by the city from wastewater treatment plants and hence is not part of the modeled water budget.

2.3.1.9 Results and Discussion

Calibration for Surface- and Groundwater levels

The MIKE SHE model was calibrated to a total of 35 wells and MIKE 11 was calibrated to a total of 5 gauges for 10-year simulation period (Figure 2.16). The number of observed data points for the simulation period varies. The calibration statistics are shown below (Table 2.13 and 2.14) whereas, the plots for individual calibration point are shown in Appendix A-1 and A-2. Overall, the model closely follows dry and wet season fluctuation trends in groundwater and surface water elevations over the entire modeled domain. For groundwater model, the correlation (r) varies between a high of 0.989 to a low of 0.739, averaging around 0.892. Nash-Sutcliffe averages around 0.641 with a minimum of 0.0428 and maximum of 0.877. It is important to note here that two wells at transect 5 i.e., SGT5W2 and SGT5W2SW show poor calibration statistics and match between observed and simulated water levels. Since the calibration point is considerably far from the project area and are close to the boundary, it does not impact the simulation quality in the areas of interest, and hence not much efforts were devoted towards improvement. For surface water elevations, correlation (r) varies between a high of 0.898 to a low of 0.624, averaging around 0.812. Nash-Sutcliffe averages around 0.5 with a minimum of 0.126 and maximum of 0.789. In general, the model does a very good job in predicting local trends in groundwater and canal stages.

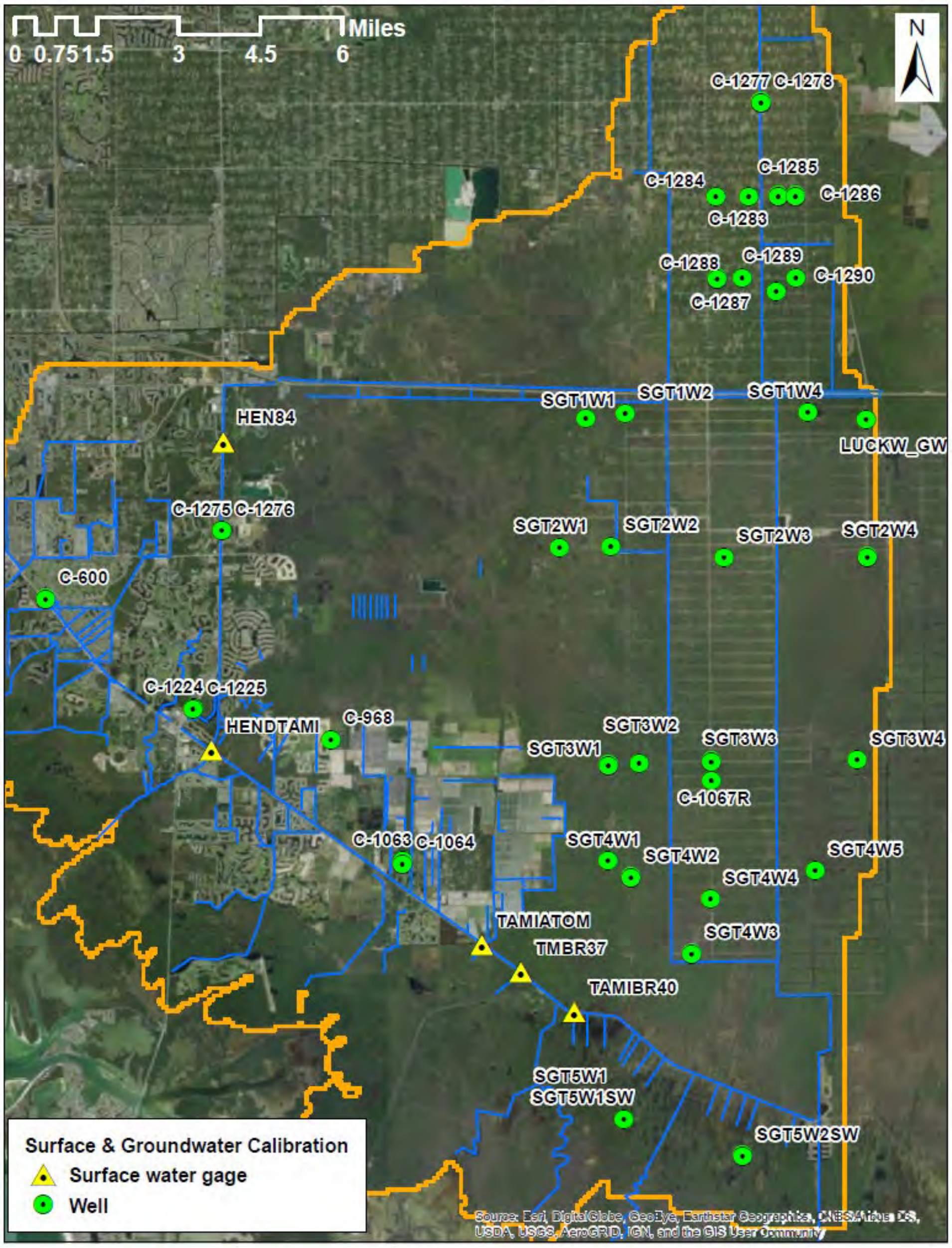


Figure 2.16 - Location of calibration points for surface- and ground water models.

Table 2.13: Calibration statistics for groundwater well.

Calibration Point	Mean Error	Mean Absolute Error	Root Mean Square Error	Standard Deviation	R(Correlation)	(Nash Sutcliffe)
SGT1W1	-0.974	1.056	1.184	0.672	0.926	0.522
SGT1W2	-0.990	1.054	1.192	0.663	0.922	0.510
SGT1W4	0.316	0.764	0.979	0.927	0.944	0.795
LUCKW_GW	-0.870	0.887	0.939	0.352	0.990	0.810
SGT2W1	0.622	0.820	1.098	0.905	0.904	0.632
SGT2W2	0.106	0.540	0.781	0.773	0.901	0.806
SGT2W3	-0.749	0.982	1.124	0.838	0.871	0.537

SGT2W4	-0.848	1.247	1.433	1.156	0.871	0.580
SGT3W1	-1.391	1.410	1.500	0.561	0.948	0.235
SGT3W2	-0.549	0.702	0.808	0.593	0.944	0.794
SGT3W3	-0.378	0.655	0.786	0.690	0.918	0.796
SGT3W4	-0.653	1.312	1.484	1.332	0.835	0.535
C-1067R	-0.598	0.707	0.856	0.613	0.937	0.759
SGT4W1	0.007	0.435	0.572	0.572	0.938	0.848
SGT4W2	0.157	0.398	0.534	0.511	0.944	0.877
SGT4W3	0.445	0.593	0.788	0.650	0.888	0.636
SGT4W4	1.122	1.130	1.263	0.581	0.916	0.235
SGT4W5	-0.523	0.926	1.064	0.927	0.868	0.640
C-1063	0.188	0.561	0.678	0.651	0.891	0.774
C-1283	-0.418	0.882	1.158	1.079	0.853	0.678
C-1284	0.087	0.833	1.127	1.124	0.865	0.747
C-1285	-0.162	0.812	1.104	1.093	0.890	0.777
C-1286	-0.483	0.831	1.078	0.964	0.920	0.786
C-1287	-0.464	1.258	1.570	1.500	0.739	0.491
C-1288	-1.118	1.303	1.580	1.117	0.787	0.043
C-1289	-0.866	1.084	1.337	1.018	0.809	0.339
C-1290	-0.347	0.816	1.056	0.998	0.860	0.708
C-600	0.064	0.343	0.423	0.418	0.908	0.759
C-1275	0.038	0.524	0.619	0.617	0.938	0.823
C-1276	-0.217	0.660	0.780	0.750	0.908	0.672
C-1277	-0.748	1.146	1.407	1.191	0.876	0.668
C-1278	-0.352	0.800	1.048	0.987	0.867	0.708

Table 2.14: Calibration statistics for surface water gages.

Calibration Point	Mean Error	Mean Absolute Error	Root Mean Square Error	Standard Deviation	R(Correlation)	R2(Nash Sutcliffe)
HEN84	-0.195	0.586	0.713	0.686	0.898	0.789
HENDTAMI-H	0.661	0.804	1.019	0.775	0.870	0.295
TAMIATOM	-0.100	0.588	0.732	0.725	0.837	0.674
TMBR37	-0.515	0.664	0.801	0.614	0.833	0.443
TAMIBR40	-0.334	0.725	0.881	0.815	0.624	0.126

Local Water Budget:

The water budget predicted by the model for each component (Figure 2.17) are in agreement with the local scale HC and regional scale BCB models, and generally accepted values for the region.

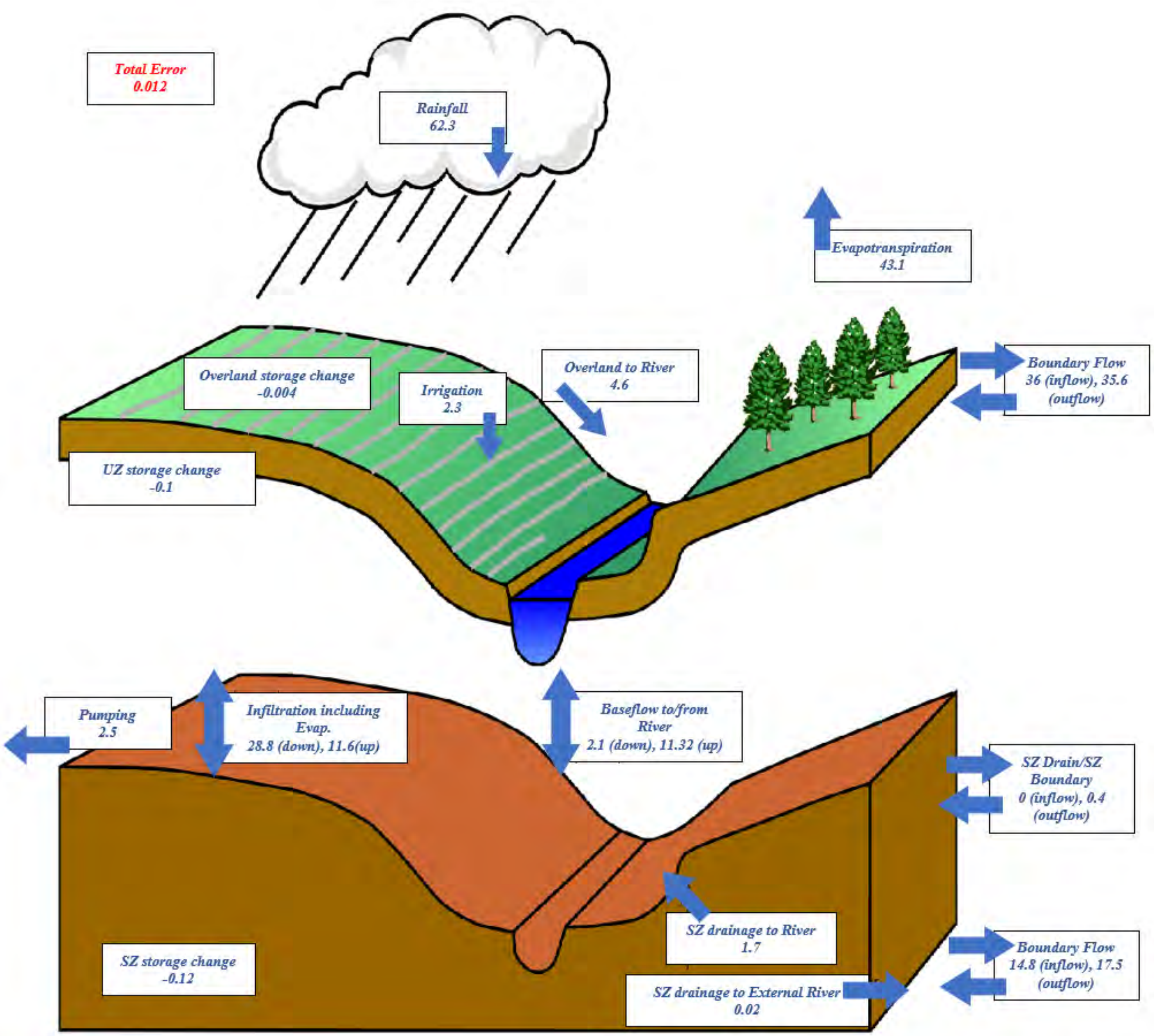


Figure 2.17 - Accumulated water balance for the period from 1/1/2008 to 12/31/2017. Values are in inches/year.

Overland Flow Results, Current Conditions

The current conditions simulation shows an average depth of 2 to 4 inches, contained primarily within the core rehydration area and small patches of low-lying areas within forested flow way (Figure 2.18). The hydroperiod varies between 2 to 4 months/year inside the core rehydration area, going up to 4 to 6 months/year in small patches. In the forested flow way and secondary flow way the hydroperiod varies between 2 to 4 months/year. These overland maps were used to calculate average water depths and hydroperiods for vegetation communities (see Supplemental Information Attachment 6: Vegetation Hydrology Effects Analysis).

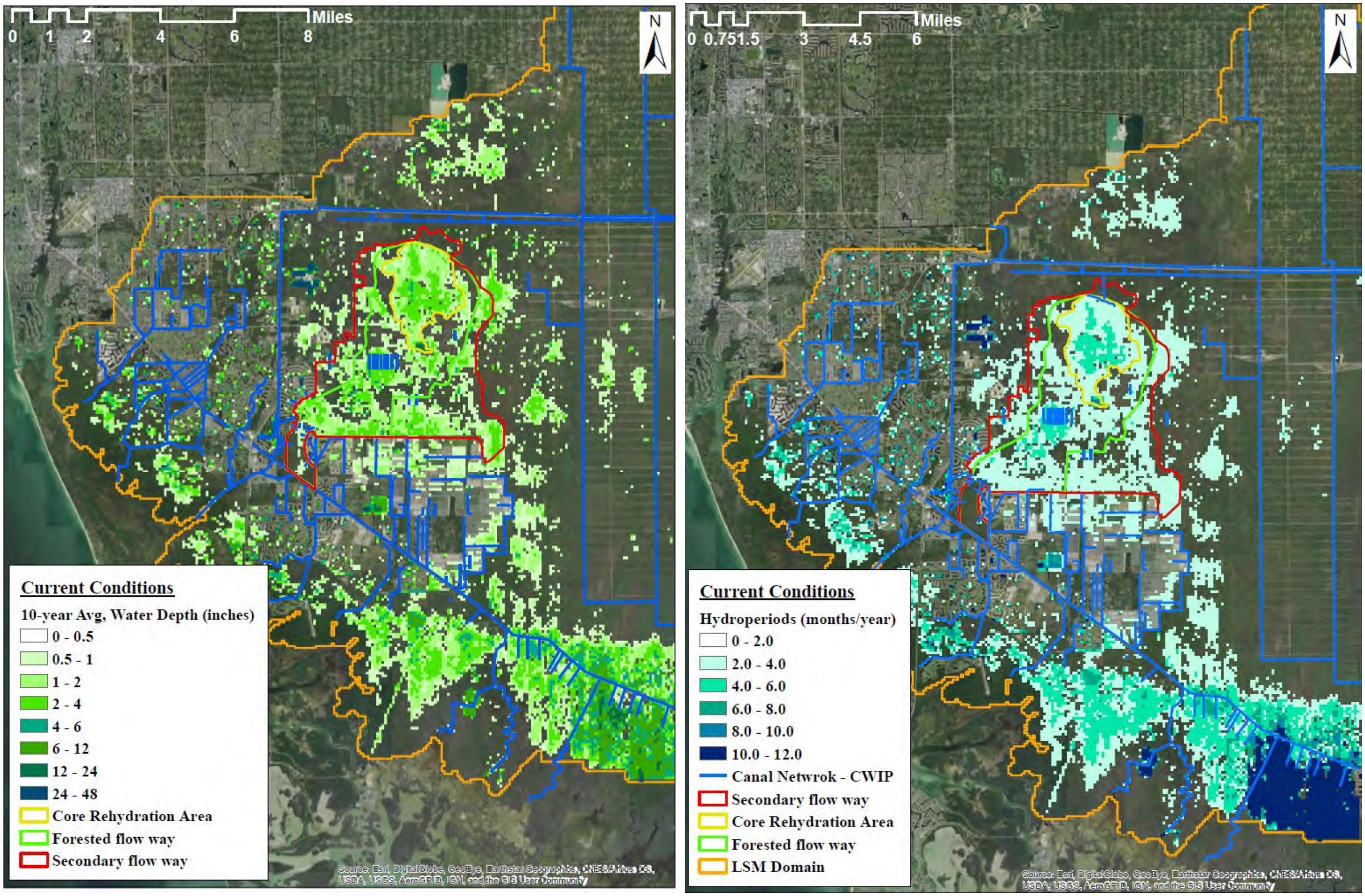


Figure 2.18 - Simulated 10-year average overland water depths and hydroperiods in the current condition. Figures also show core rehydration area, forested and secondary flow-way boundaries.

2.3.2 With only CCWI Project LSM

To simulate impacts of the proposed project, the calibrated current conditions model, described in section 2.3.1, was modified to incorporate the CCWIP. The schematics and details of the project drainage system and components are presented in the Permit Drawings. The proposed CCWIP drainage system is explained as follows (Figure 2.19):

- North Belle Meade Flow-way and Pump Station: Diversions from Golden Gate Canal will be made through a pump station proposed on the canal just upstream of the GG-3 weir. The pump station will be built on a County-owned property known as the "305 Property". The diverted water will flow southwards via a proposed flow-way into the I-75 North canal. A 4' high x 6' wide box culvert will be installed under White Lake Blvd. The water will continue to flow south into I-75 South Canal through three existing 10' x 5' box culverts under Alligator Alley. No modification is proposed under Alligator Alley.

- I-75-North Gate Structure: An operable underflow/sluice gate will be built just west of the junction between North Belle Meade Flow-way and I-75 North Canal. The purpose of the gate is to keep CWIP water from flowing westwards into the Henderson Creek Canal. The operable gate will be designed to close partially on days when CCWIP discharge is being pumped into I-75 North Canal and will ensure uninterrupted and unobstructed flows for the rest of the days when pumps are off. To further facilitate eastwards movement of water, excess vegetation will be removed along the stretch of the I-75 north and south canals between the North Belle Meade Flow-way and South Belle Meade Flow-way.

- South Belle Meade Flow-way and Pump Station: Water will be pumped out of I-75 South Canal through a proposed pump station located south of the I-75 South Canal. The pump station will be located outside the FDOT right of way of Alligator Alley. The pumped water will flow southwards into the South Belle Meade Flow-way, which will be designed to also provide settlement of solids and improve water quality of the runoff from Alligator Alley. The discharge from I-75 South Canal will be pumped into the flow-way where particles will settle at the bottom and water will continue to move south into the spreader swale. A bridge or low water crossing will be constructed to accommodate the existing horse trail north of the spreader swale.

- Spreader swale: After water quality treatment, gravity flow of water will continue into the spreader swale. Weirs will be built along the southern bank of swale to control water elevation in the entire flow-way system as well as release water into the forest as overland sheet flow.

- Flow-way Around Naples Reserve: Once released into the forest, flow of water will be driven by forest topography which slopes gradually from northeast to southwest. After infiltration and evapotranspiration losses through the forest, which amounts to ~ 56 % of the total water added (section 2.3.2.2), the remaining water, will make its way southwest. A system of flow way will be built to convey the discharge around the development. The eastern segment of the flow way will be built inside the existing 60 feet wide easements along eastern boundary of the development. However, northern segment of the flow-way, ~1600 feet long, is located on privately-owned property northeast of Naples Reserve. The county is proposing to acquire ~82 feet wide and 1700 feet long easement to accommodate the collection channel and to relocate the existing trail in this vicinity. The water will continue to flow southwards via a proposed channel along the eastern edge of Naples Reserve. The wet season water levels in the forest and the flow of water into the channel will be controlled by a proposed weir. The weir invert will be set at forest grade to avoid any drying out in the collector ditch catchment. On the western edge of Naples Reserve, an existing channel will be modified to convey water to U.S.41. The channel is disconnected with the forest at the north end as of now but is believed to have been connected historically. This channel will be reconnected with the forest and will collect some of the westward moving waters from the forest. A weir will control the wet season water levels and flow of water into the ditch. The western flow way is located in the existing 100 feet roadway easement along the western boundary of Naples Reserve. After the southern edge of the development, the channel will be widened to accommodate additional flows from the eastern flow way. This segment of the channel will be located primarily on DEP-owned property. The remainder of the forest water will continue to flow southwest under the Winding Cypress Dr. as gravity sheet flow.

- <u>Flow through U.S. 41 and South:</u> Once discharged into U.S. 41, the water will continue to flow south under Tamiami Trail E through two existing culverts (5' high x 12' wide and 4' wide x 11' high). No new culverts are proposed under the roadway. Majority of the water is expected to flow into two existing canals, Belle Meade 10 and Belle Meade 11, both of which discharges into Fiddler's Creek Lake. The lake shallows as it moves southwest along the southern boundary of Fiddler's Residential development. The water will then flow out of southern bank of the lake into Rookery Bay as sheet flow. The sheet flow will continue to flow south and southwest towards Rookery Bay. A small fraction of the flow will make its way west under existing S.R. 951 culverts. The existing five culverts are considered sufficient and hence no new culverts are proposed under S.R. 951.

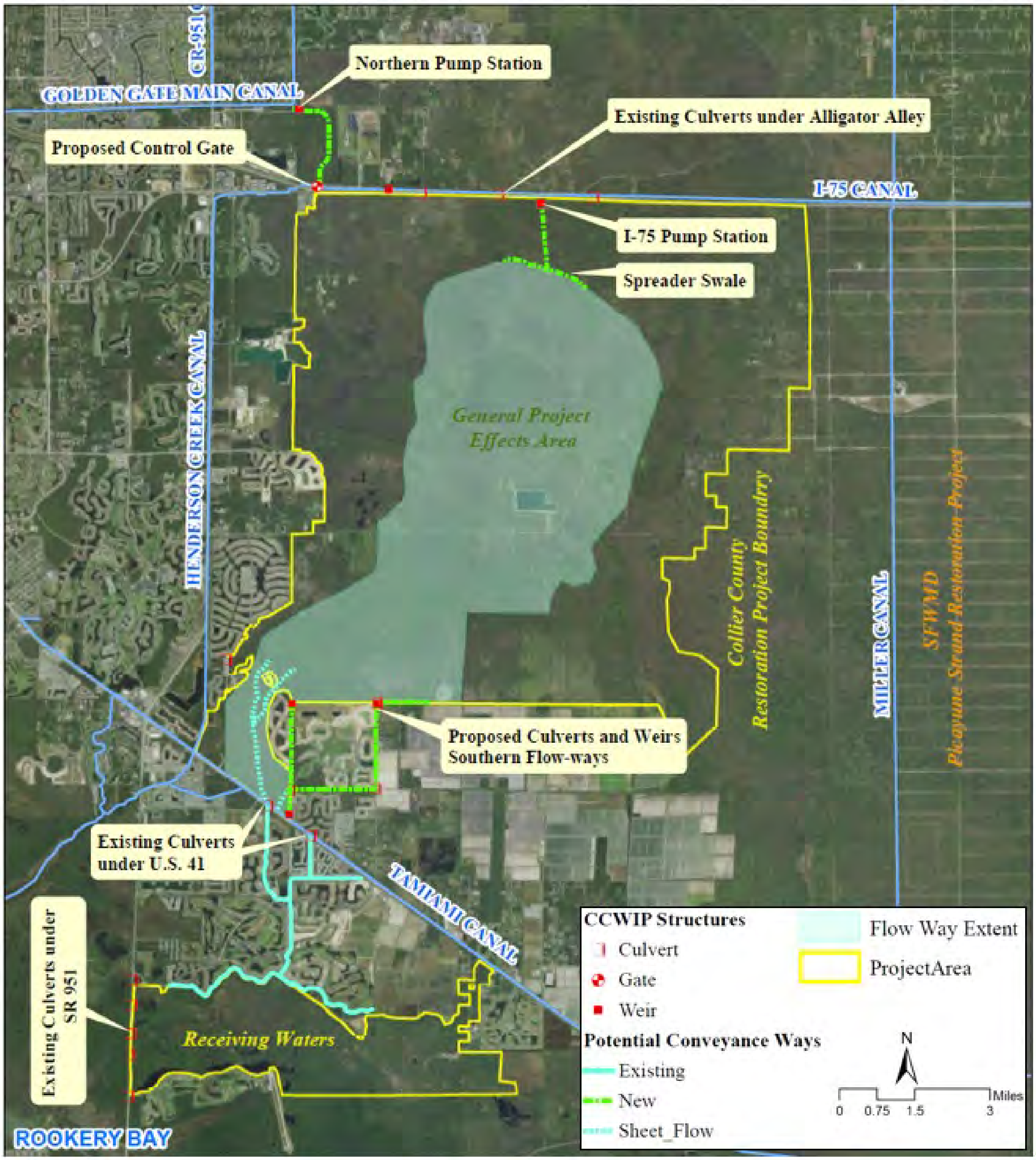


Figure 2.19 - CCWI project drainage system and infrastructural components.

2.3.2.1 Modifications in Current Conditions LSM

Calibrated current conditions LSM was updated to incorporate CCWIP components as described above. The components of the project were included in MIKE 11 and were coupled with MIKE SHE to simulate exchange with overland and variably saturated zone flow where warranted. The

CCWIP model does not require any modifications in the overland, unsaturated and saturated flow components of the current conditions LSM.

The project components are designed not only for annual and seasonal hydrologic conditions, but also for the 100-year, 72-hour design storm. The drainage system is capable of conveying design storm discharges without causing any adverse flooding impacts. The design incorporated in the CCWIP LSM is based in part on the design storm analysis, details of which are described latter in section 3 of this report. It is important to note that this document only outlines the infrastructure details sufficient for modeling purposes and suffices the needs of this conceptual ERP permit application but are not necessarily sufficiently detailed for construction applications.

<u>North Belle Meade Flow-way and Pump Station</u>
The Northern pump station was represented by varying capacity pumps, capable of discharging as per the operations plan (see Supplemental Information Attachment 1: Project Overview).The proposed lay out of the northern flow way has been coordinated with development of the 305 property. In the model, the flow-way was represented by a trapezoidal channel, 6 feet wide at the bottom, 2:1 side slope. The depth of the channel varies with the topography between 4 to 3 feet. A 6 ft wide x 4 ft deep box culvert is proposed added under White Lake Blvd. The invert level was set at 6.0 feet NGVD 88, which ensures positive outfall drainage to I-75 North canal and adheres to FDOT culvert design requirements. The lay out details and a typical cross section are shown on sheets 3 through 6 of the Permit Drawings.

<u>I-75 North Gate Structure and Canal Improvements</u>
An operable underflow/sluice gate is proposed to keep the pumped water from flowing westwards into the Henderson Creek Canal and is located west of the junction between northern flow-way and I-75 North Canal. The gate will be operated to close partially on days when CCWIP discharge is being pumped into I-75 North Canal and ensures unobstructed flows for the rest of the days. The invert was kept at the same level as that of culvert under Alligator Alley, i.e. 3.84 NGVD 88, to ensure that the proposed structure will not cause any increase in the upstream water levels during days of no pumping. The gate will be 11.5 ft wide, whereas, the gate opening is a model calibrated gate height that does not cause a significant increase in discharges at the HC-1 structure because of CCWIP pumping. The optimum gate opening was determined to be 0.59' feet above the gate invert. To further refine the gate operations and ensure least impacts on normal water levels in the canals, a lag time of 0.5 hour is added between the time pumping starts into the Northern Flow-way and the gate is raised. This lag-time is simulated. The water moves southwards towards I-75 South primarily by means of three existing box culverts under Alligator Alley. The removal of vegetation was represented by a lower manning's n value in the CCWIP model. Also, the weir representing an existing earthen dich block located on I-75 North canal was lowered by 1 ft, which will require minor re-grading. The details are shown on sheet 7 of the Permit Drawings.

<u>South Belle Meade Flow-way and Pump Station</u>
The Northern pump station was represented by varying capacity pumps, capable of discharging as per the operations plan (see Supplemental Information Attachment 1: Project Overview). The

southern flow-way system consists of a deep channel (essentially a linear pond) and a spreader swale. The flow-way was represented by a deep trapezoidal channel, 128 ft wide, ~8.5 ft deep and 3300 ft long. The spreader swale was represented by a trapezoidal channel, 75 ft wide and 3.3 ft deep, with rectangular weirs, located along the length at a control elevation of 9.5 ft NGVD 88. The layout details and a typical cross section of the flow way are shown on sheets 8 and 9 of the Permit Drawings.

Flow-way Around Naples Reserve:
The eastern flow-way was represented by a trapezoidal channel, 8 feet wide at the bottom, 2:1 side slope. The depth of the channel varies with the topography between 2.5 to 4.6 feet. A weir was added upstream of the flow-way, at an elevation of 6 ft NGVD 88, Three sets of two (2) circular culverts, 4ft diameter each, were added under an existing trail crossing, Green way Rd. and Naples Reserve Blvd. The western flow way was represented by a trapezoidal channel, 4 feet wide at the bottom, 2:1 side slope and ~ 4 feet deep. The lay out details, typical cross sections and structure details are shown on sheets 11 through 15 of the Permit Drawings.

2.3.2.2 Results and Discussion

Overland Flow Results – 10-Year Simulation

The simulation shows that the project will increase the average depth to 5 inches within most of the core rehydration area going up to 8 inches in small low-lying patches (Figure 2.20). The hydroperiod will be increased to between 4 to 5.5 months/year inside most of the core rehydration area, up to 6 months/year in small patches. In the forested flow way and secondary flow way the hydroperiod will vary between 2 to 4 months/year. These overland maps were used to calculate average water depths and hydroperiods for vegetation communities (see Supplemental Information Attachment 6: Vegetation Hydrology Effects Analysis).

To evaluate the rehydration impacts of the project, current average depths and hydroperiods (Figure 2.18), were subtracted from with-project average depths and hydroperiod (Figure 2.20). The results (Figure 2.21) shows that the project will cause an increase of ~3 inches, in terms of average depth, and ~1.5 months/year, in terms of hydroperiod, inside the core rehydration area; and ~1.5 inches, in terms of depth, and ~0.5 months/year, in terms of hydroperiod inside the forested flow way and most of the secondary flow-way. The impacts of the increment on vegetation communities in the area were evaluated and detailed in Supplemental Information Attachment 6: Vegetation Hydrology Effects Analysis.

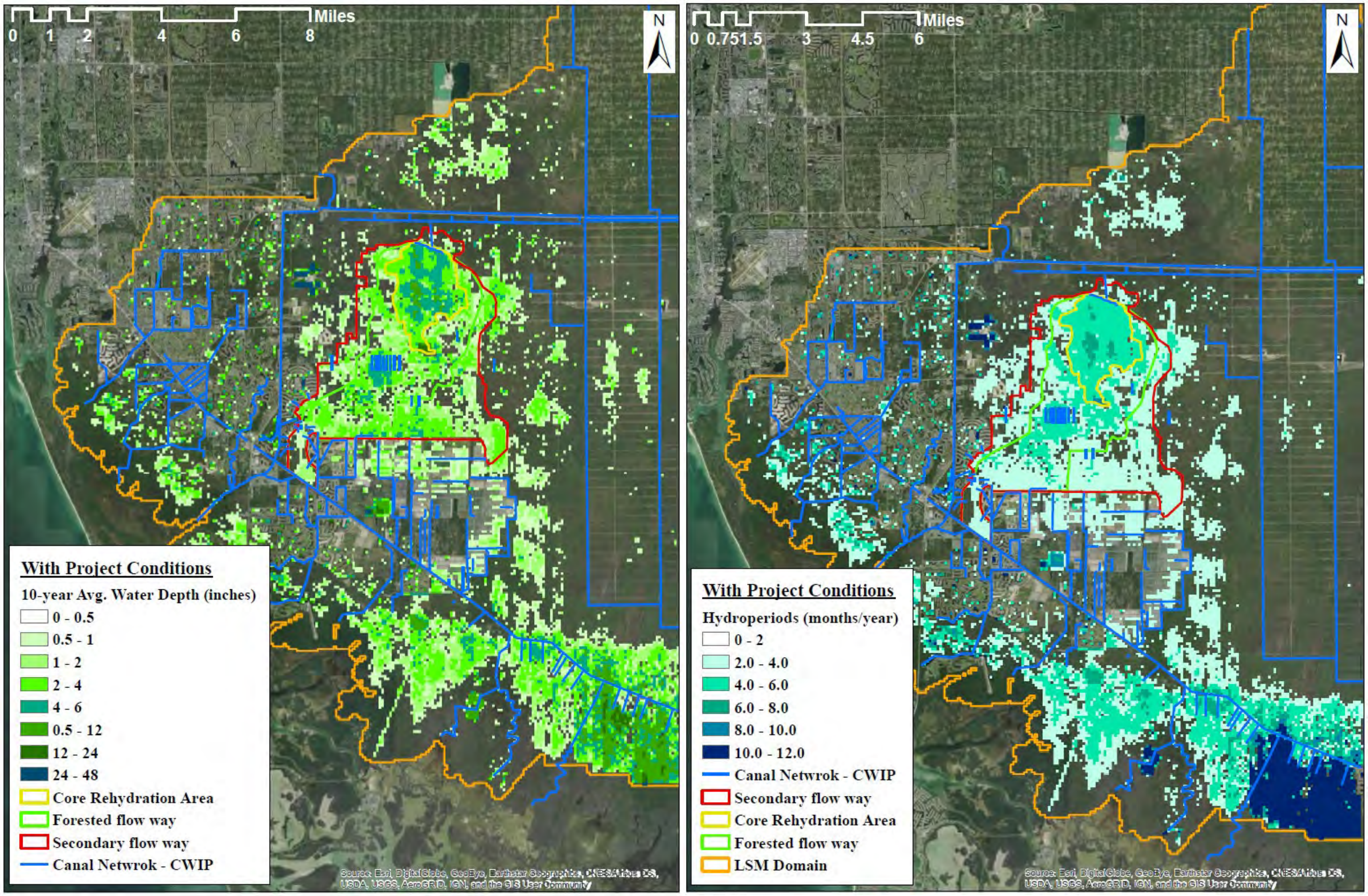
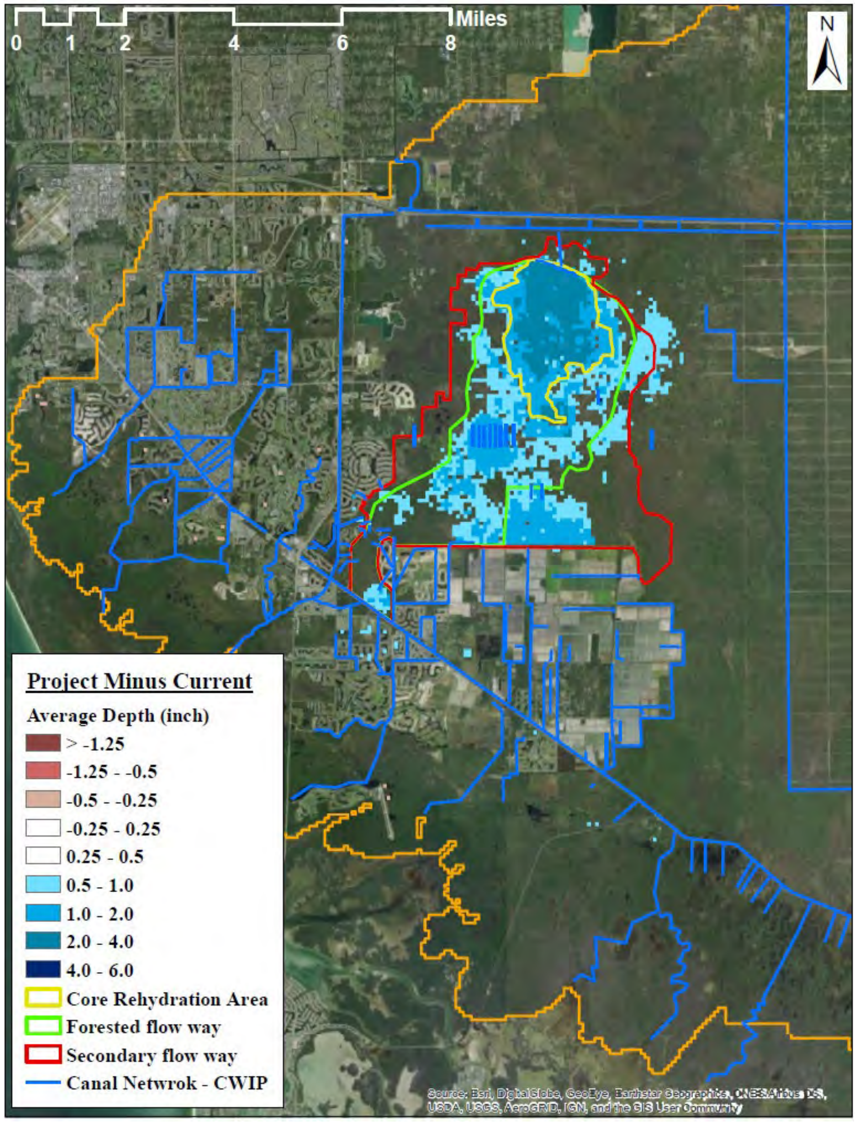


Figure 2.20 - Simulated 10-year average overland water depths and hydroperiods in the with-CWIP conditions. Figures also show location of core rehydration area, forested and secondary flow-way boundaries.



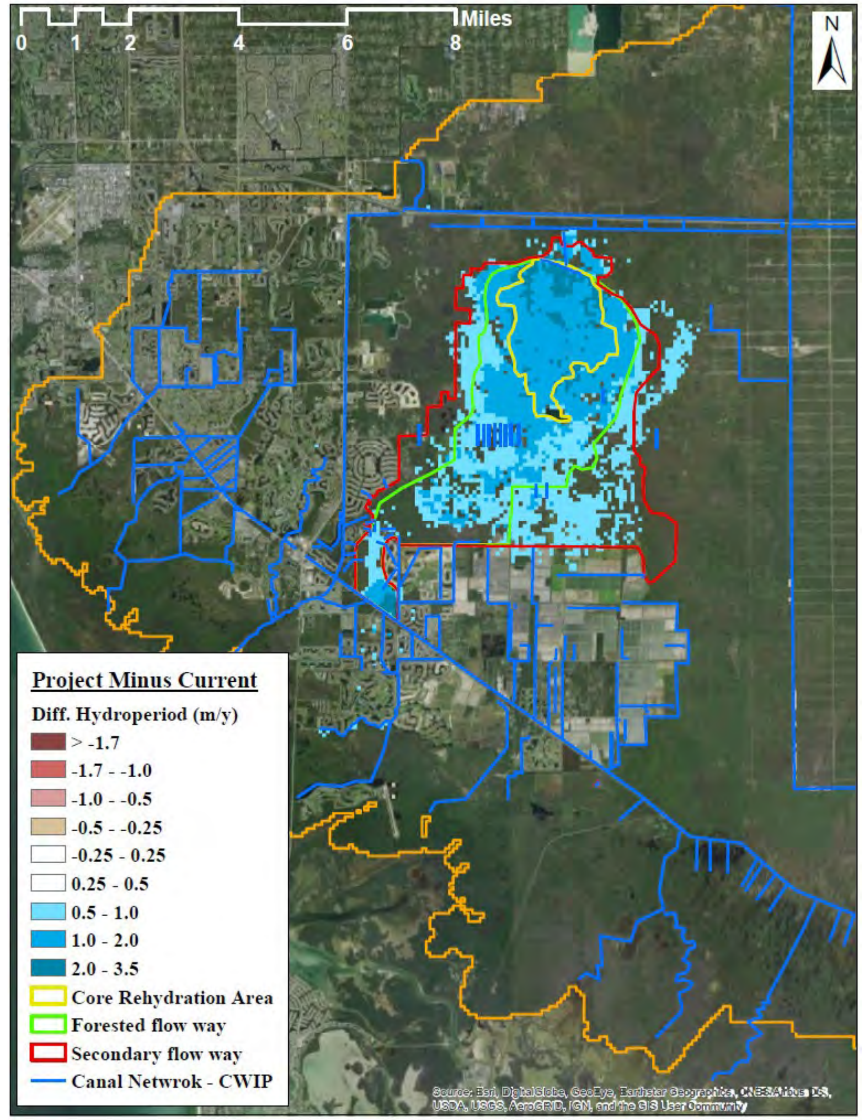


Figure 2.21 - Difference in average overland water depth and hydroperiod between CWIP and current conditions.

Average discharge increments

The 10-year average net increments in discharges caused by the project were calculated at various locations along the drainage system. Net discharges were calculated by subtracting with-project discharges from current conditions discharges only during the days when the pumps are on (Figure 2.22). The results show that most of the water from the forest will be collected by the collector ditch located east of Naples Reserve. On average, a total of ~ 31 cfs (45%) will make its way to the southwest end, whereas, the remaining ~39 cfs (55%) of the added water will be lost to infiltration and evapotranspiration. The results show that the majority of the remaining diverted water will flow through the existing US 41 culverts and flow-ways through the Fiddler's Creek development.

Although, because of the proposed control gate at I-75, pumped water will be prevented from flowing directly into the Henderson Creek canal, a small amount of base flow and overland flow from the rehydrated area will make its way south-west towards the canal through existing flow-ways and culverts under Winding Cypress Drive, and via the subsurface. This will cause discharges to increase slightly (approximately 5.6 cfs) in the with-project conditions.

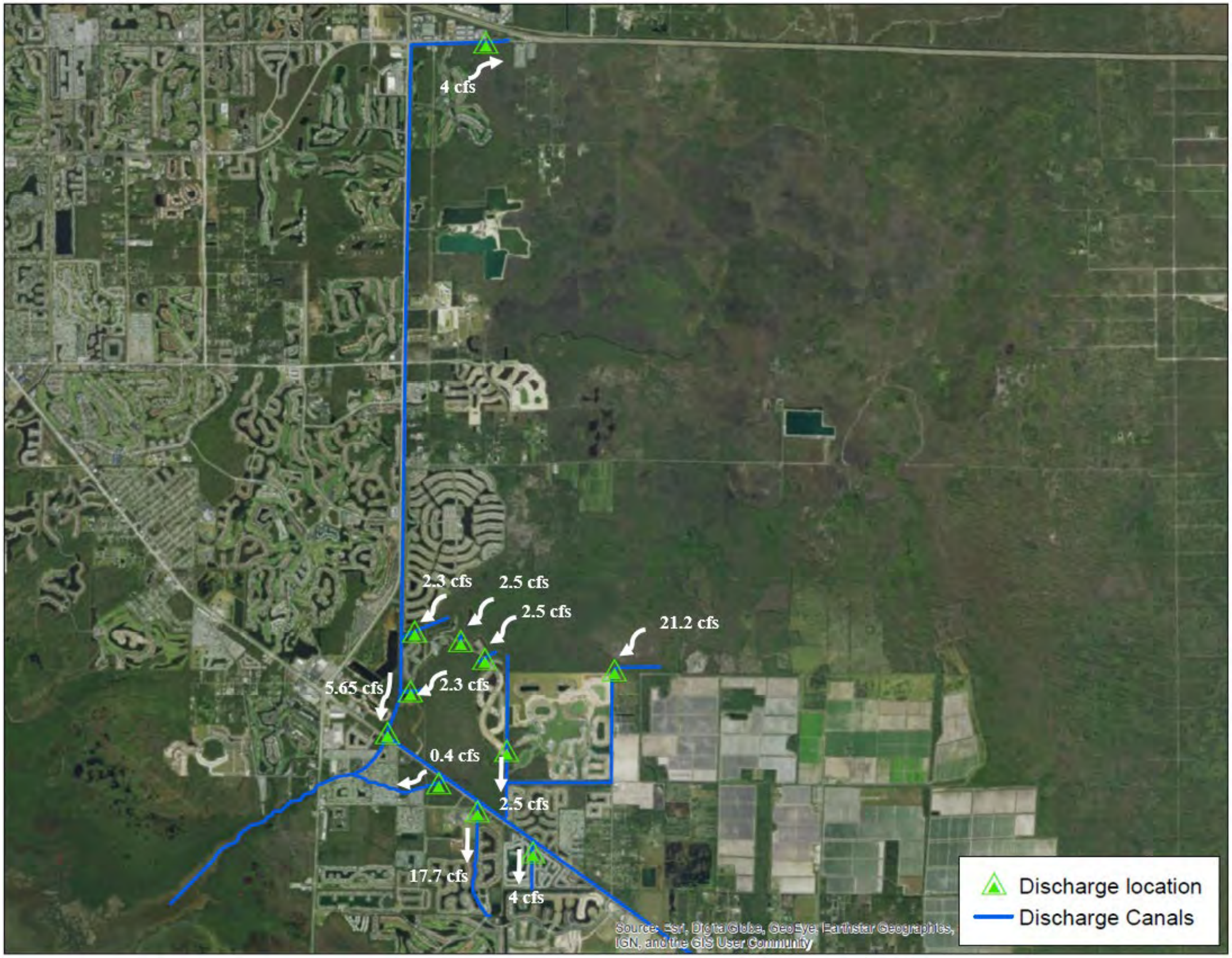


Figure 2.22 - 10-year net average discharge reaching southwest corner of the rehydration area. These discharges were calculated only during the pumping days and represent net discharge increment caused by the project.

2.4 Interactions with Picayune State Strand Forest Project

On the east of the Belle Meade Forest area lies Picayune Strand Restoration Project (PSRP) . Like the Belle Meade Forest, this area has also suffered dehydration due to urbanization in the watershed. The USACE and SFWMD, along with other federal agencies, are working on a similar, larger-scale rehydration project in the area. The project involves restoring historic sheet flow through the forest and improving the ecology and environment of the forest. The details of the project were shared by the USACE through a series of meetings attended by the CCWIP project team. Figure 2.23 shows the extent of PSRP along with CCWIP. In the northern portion, the two projects are separated by a natural ridge, however, in the south, close to the Six L's Agricultural Area, there is a possibility of potential overlap of sheet flows between the two projects. Moreover, since the two projects share the same aquifer basin, their impact on groundwater levels can also be overlapping. Therefore, the combined impact of the two projects was assessed.

Although not operational, some of the PSRP components have already been built, such as the levee and pump stations on Faka Union and Miller canals. Given the uncertainty involved in the permitting, design and construction processes for both the projects, it cannot be anticipated at this time as to which project will be operational first. Hence, it is deemed important to model scenarios where the PSRP project is already operational and the forest to the east of the CWIP project area is already being rehydrated. Therefore, two additional LSM scenarios were developed based on the assumption that PSRP is operational in the current conditions and CCWIP and PSRP will both be operational in the future scenario. These model scenarios will allow assessment of combined impact of the two projects as well as any potential interactions thereof.

2.4.1 Current Conditions with PSR Project

To simulate impacts of the PSRP, the calibrated current conditions model, described in section 2.3.1, was modified to incorporate project components. Information about PSRP infrastructure was provided by the USACE Interagency Modeling Center (IMC) staff through regular participation by the CWIP team members in the PSRP team coordination meetings and through individual meetings and information exchanges between key members of the two project teams. As of today, the PSRP can be divided into the following infrastructural components (Figure 2.23):

- <u>Pump stations and spreader swales on Miller and Faka Union canals:</u> Two pump stations are built on Miller and Faka Union canals, located ~ 2.23 miles south of the junction with I–75 canals. When operational, water will be pumped out into the spreader swales located just downstream of the pump stations. The spreader swales have low weirs which will spread water into the forest as sheet flow.

- <u>Levee across Miller and Faka Union canals:</u> A tie-back levee is constructed just upstream of the spreader swales on both the canals. The levee stretches between Miller and Merritt Blvd. perpendicular to the canals. Downstream of the levee, the existing canals will be backfilled.

- Southwest Protection Feature (Berm and canal on the eastern side of SixL's Area): A levee berm will be constructed parallel to the eastern boundary of SixL's area. It will start at the northern edge of Deseret Farms and will go south to U.S. 41. The berm will prevent water from the rehydrated forest to mix with the farm discharges. To collect seepage and provide more conveyance for farm discharges during flood conditions, a new canal will be constructed parallel to the existing perimeter canal on the west of the berm. Additional capacity to convey flows under US 41 is included as a component of this feature.

2.4.1.1 Modifications in Current Conditions LSM

The calibrated current conditions LSM was updated to incorporate PSRP components as described above. The components of project drainage system were included in MIKE 11 and were coupled with MIKE SHE to simulate exchange with overland and variably saturated zone flow where warranted. In the overland module of the model, SOLFA's were modified to represent berm as described in the following sections. The PSRP model did not require any modifications in the unsaturated and saturated flow setup components of the current conditions LSM. The project information incorporated in the model were consulted with the USACE and were up to date when the model was being developed. Furthermore, the PSRP details being presented in this document or included in the RSPR model are intended to simulate big picture impacts of the PSRP flows on the CCWIP area and hence does not claim to represent the exact or final PSRP project in certain details of the PSRP still under design, such as the disposition of the additional flows under US 41 east of Tomato Road and through Collier Seminole State Park. This component will have no impact to the CWIP due to the geographical separation of several miles between the two project's ultimate outfalls.

Pump stations and spreader swales on Miller and Faka Union canals

The Miller and Faka Union canals will be blocked approximately 2.23 miles south of I-75 canals, where two pump stations are located. Each pump station is located on one canal and when operational will pump water out of the canal into their respective spreader swales downstream. The Miller canal pump station has one pump with a capacity of 75 cfs and five pumps each with a capacity of 235 cfs. The Faka Union pump station has one pump with a capacity of 300 cfs and five pumps each with a capacity of 470 cfs. The pumps operation will be designed to allow for flexibility in the pumping rates depending upon the season and downstream conditions. All the pumps will be auto-operated based on head water levels in the canals i.e. whenever the water level in the canal exceeds a defined threshold, pumps will be turned on. For simplicity in the model, both the spreader swales were incorporated using channels and flood codes downstream of the levee. The channel allows the water to flow across the SOLFA boundary, whereas flood codes, linked with the canals, allow the canal water to come out into the forest and flow as overland sheet flow.

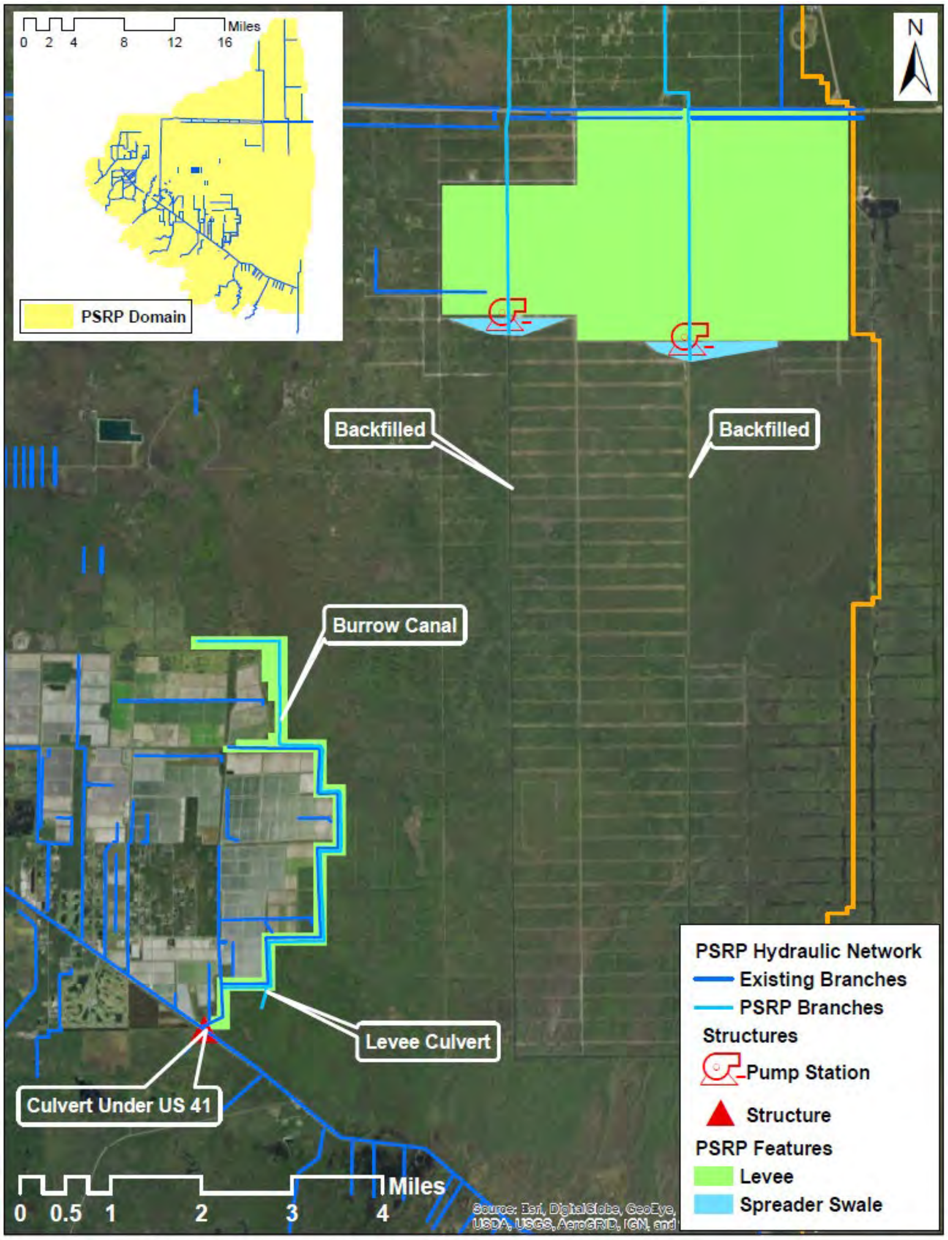


Figure 2.23 - PSR project components simulated in the local scale model.

<u>Levee across Miller and Faka Union canals</u>

Downstream of where the canals are backfilled, the levee will keep the pumped water from flowing back to the north, maintaining flood protection for I-75. This was incorporated in MIKE SHE as a SOLFA code, where overland flow is prevented from flowing across the SOLFA boundaries.

<u>Berm and canal on the eastern side of SixL's Area</u>

Once released into the forest, sheet flow will be driven by forest topography. Most of the PSRP water will flow towards the south, however, some water is anticipated to flow southwest towards

SixL's Agricultural Farms. In the existing conditions, an existing perimeter canal, running parallel to the farms on the eastern side, conveys farm discharges down to Tamiami canal. The perimeter canal does not have a berm on the eastern side, hence some of the water historically flows out in the forest. The new berm will restrict the canal water from flowing into the forest. and a new canal will be built parallel to the perimeter canal which will allow for additional conveyance. Furthermore, a culvert will be constructed through the new berm to release some of the water into the forest as sheet flow (Figure 2.23).

The proposed levee berm was represented in the model by a SOLFA code which prevents water from the farms to enter the forest and vice versa. The new canal is represented in MIKE SHE by a trapezoidal channel, 50 ft wide at the top and 20 ft wide at the bottom, running parallel to the existing perimeter canal. On average the new canal is 4 ft deep. Two culverts 5 ft diameter allows the water to flow across the berm into the forest area south of the canal (Figure 2.23).

2.4.1.2 Results and Discussion

Overland Flow Results

The results of the PSRP model were only evaluated to determine impacts on current condition average depth and hydroperiods in the CWIP area. The simulation shows that inside the core rehydration, forested flow way and secondary flow way area, the increments will be similar to that of the current conditions model without the PSRP (section 2.2.1.1, Figure 2.18). This shows that the PSRP project will not have any significant impact on the CWIP project area in the current conditions.

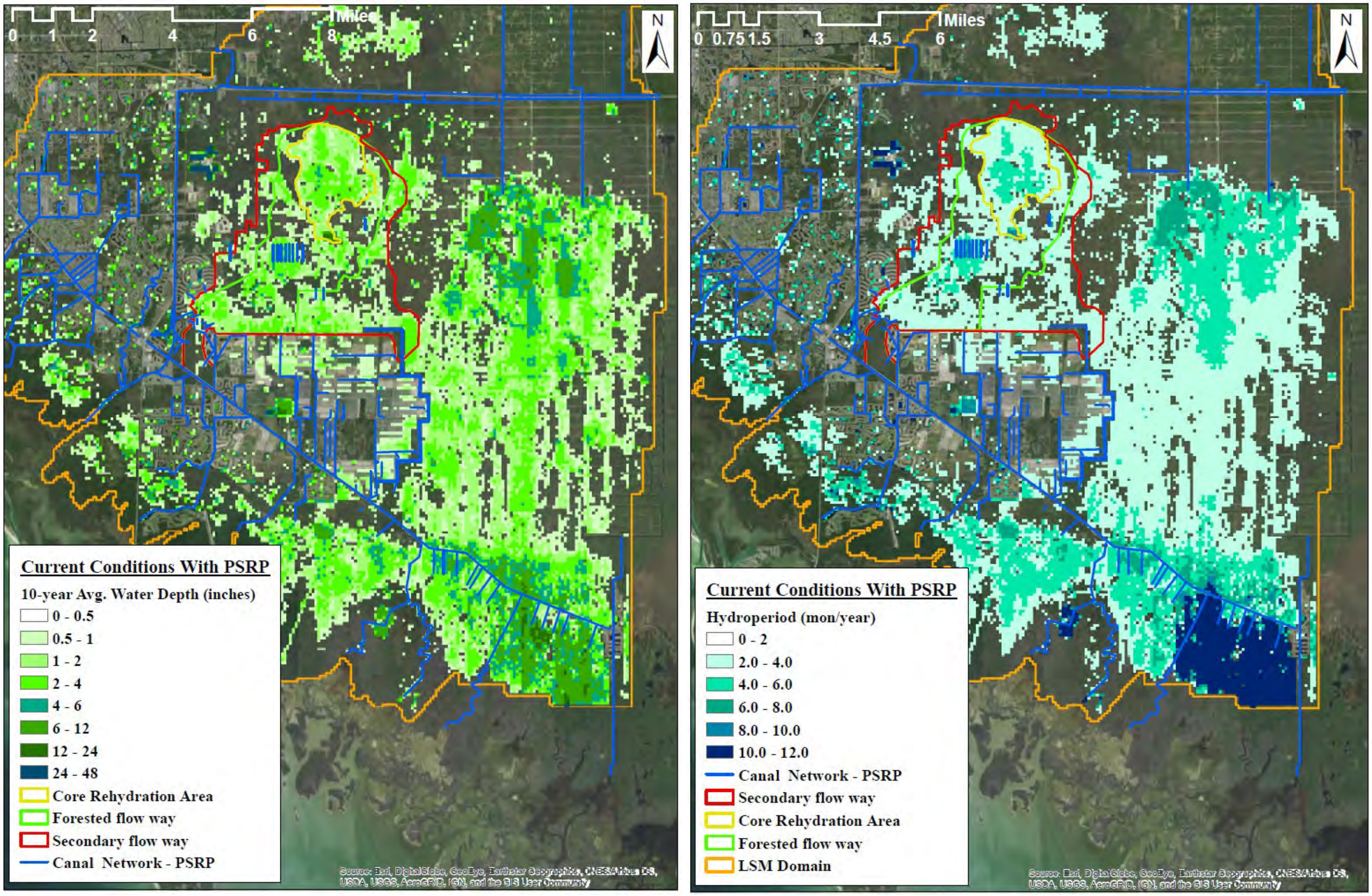


Figure 2.24 - Simulated 10-year average overland water depths and hydroperiods in the PSRP conditions. Figures also show core rehydration area, forested and secondary flow-way boundaries.

2.4.2 With both PSRP and CCWIP LSM

This scenario contains both PSRP and CCWIP. Consequently, this model combines the infrastructural elements of each of the projects as explained in detail in sections 2.3.2 and 2.4.1.

2.4.2.1 Results and Discussion

<u>Overland Flow Results</u>

The results of PSRP and CCWIP models were only evaluated to determine impacts on future conditions average depths and hydroperiods in the CWIP area. The simulation shows that inside the core rehydration, forested flow way and secondary flow way area, the increments will be similar to that of the CCWIP conditions model without the PSRP (section 2.3.2, Figure 2.20). This shows that the PSRP will not have any significant impact on the CWIP area in the future conditions. The difference maps, shown in figure 2.26, are similar to that of the without-PSRP conditions in the core rehydration, forested flow-way and secondary flow extents.

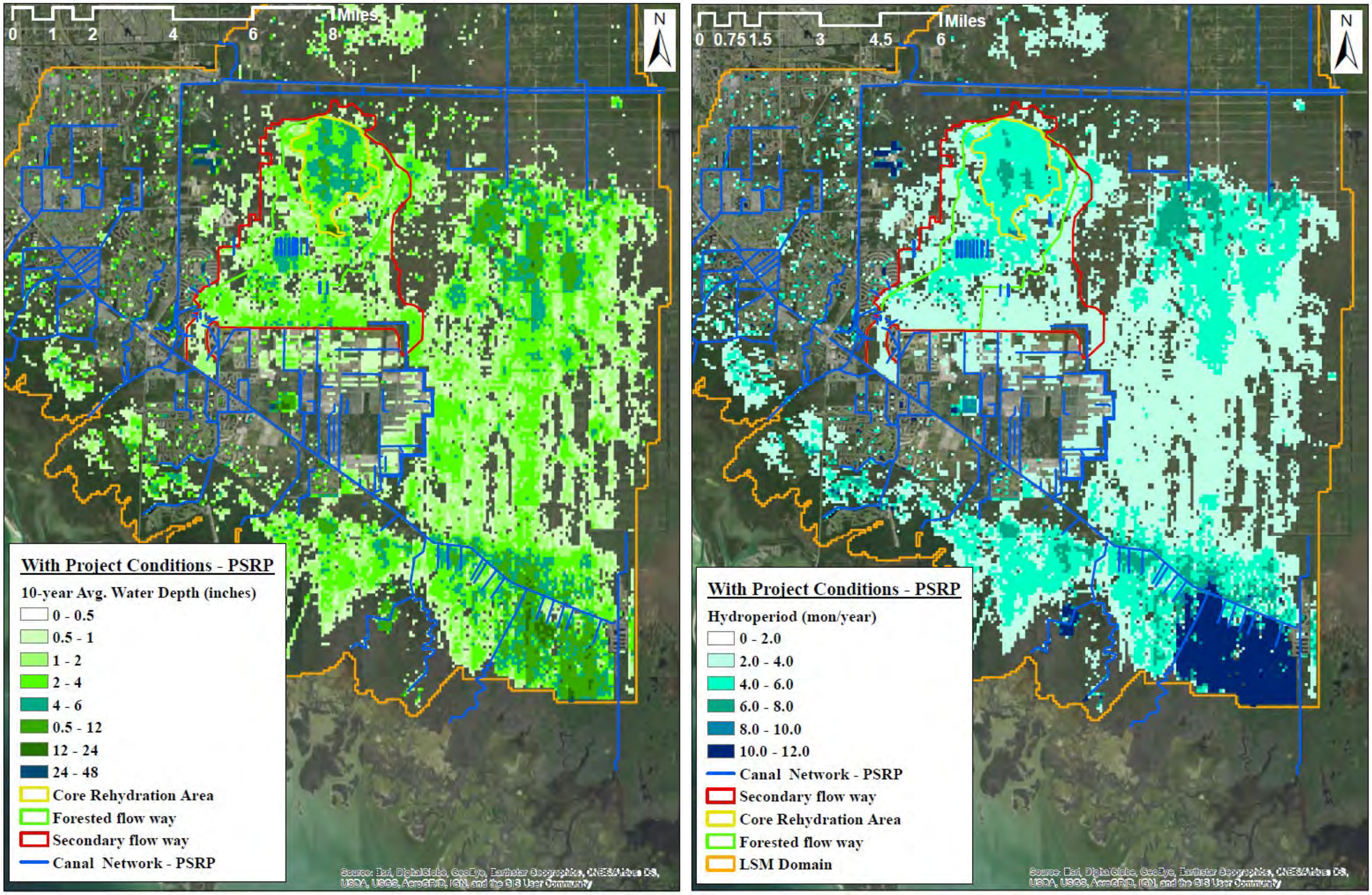


Figure 2.25 - Simulated 10-year average overland water depths and hydroperiods in the PSRP and CWIP conditions. Figures also show core rehydration area, forested and secondary flow-way boundaries.

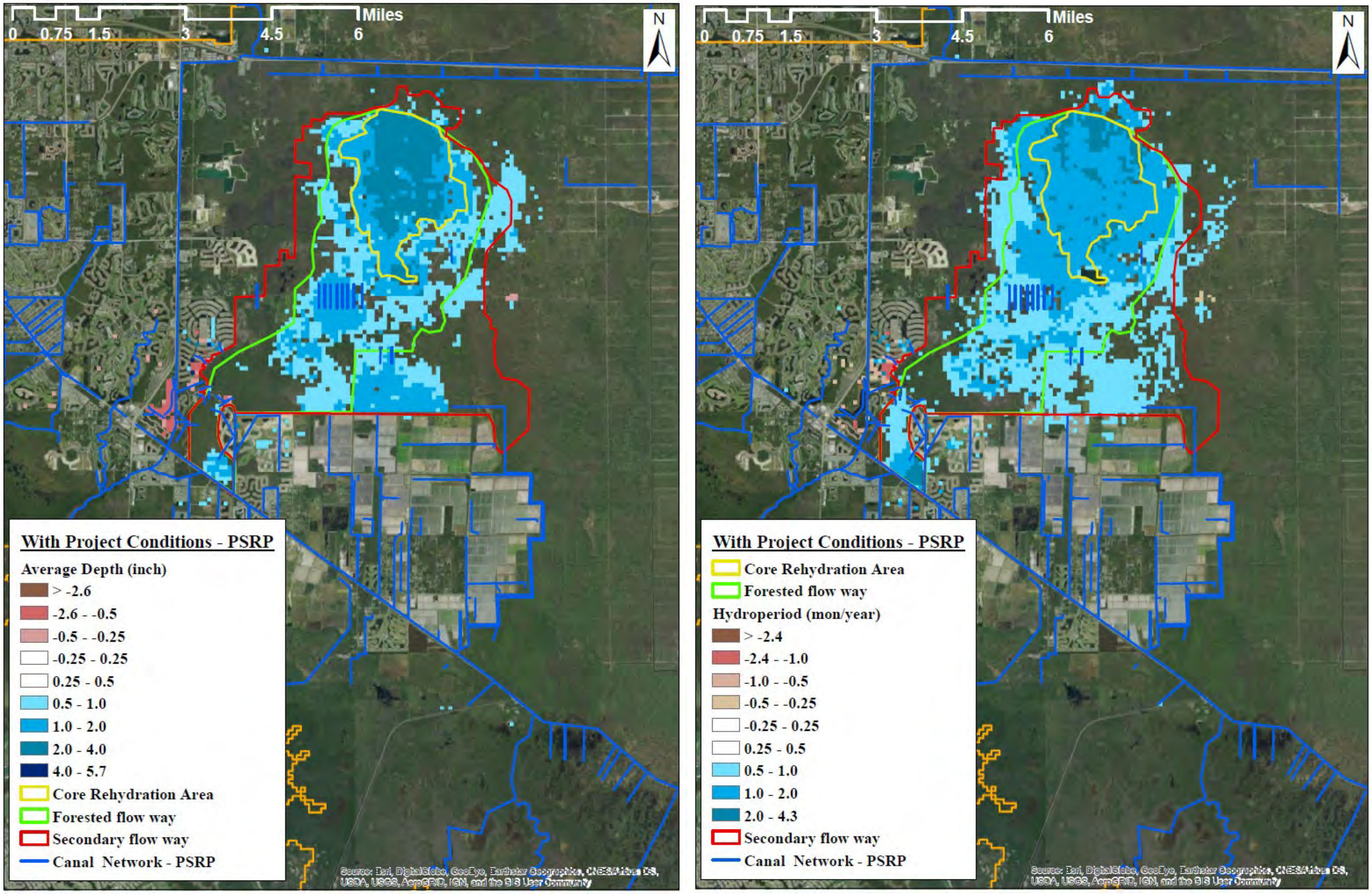


Figure 2.26 - Difference in average overland water depth and hydroperiod between PSRP & CWIP and PSRP conditions.

3 Design Storm Models and Impact on Water Levels

Infrastructural components of the CWIP, including new flow-ways, improvements and modifications in the existing systems, pump stations etc. are explained in detail along with their proposed layout in section 2 of this report. To determine the capacity and level of service of existing and proposed infrastructure, design storm models were developed for both current and with-project conditions. The impacts of project on on-site and off-site developments and major canals were evaluated for 25-year and 100-year design storm events. Both the current and with-project local-scale 10-year models were modified to develop design storm models. Detailed development of local-scale models 10-year simulations is available in section 2 of this report. This section details the modifications made in each of the local-scale models to represent design storm conditions.

3.1 Design Storm Modifications in LSM

3.1.1 Design Storm Rainfall

A Thiessen-polygon approach was used for spatial distribution of design storm rainfall over the modeled domain. The centroid of each polygon corresponds to a rainfall gage location in the watershed. Rainfall 3-day distribution and totals for 25-year and 100-year return period are based on the SFWMD Environmental Resource Permit Information Manual Volume IV, Water Resource Regulation Department (July 2010 version). Rainfall totals (Table 3.1) varied from polygon to polygon (Figure 3.1) based on the position of each centroid relative to the isohyets published in the Manual.

Table 3.1: Total rainfall depths for each polygon within the modeled domain for 100-year and 25-year design storm.

Rainfall Gage	100-year Rainfall Total (in)	25-year Rainfall Total (in)
COLGOV	14.75	11.81
COLSEM	15.91	12.51
DANHP	15.86	12.23
GOLDF2	13.68	10.94
MARCO	16.1	12.85
ROOK	15.37	12.08
SGGEWX	14.28	11.05
GG#3	14.27	11.33
FU#5	12.3	10.02
Area-Weighted 3-Day Rainfall Depth	14.89	11.67

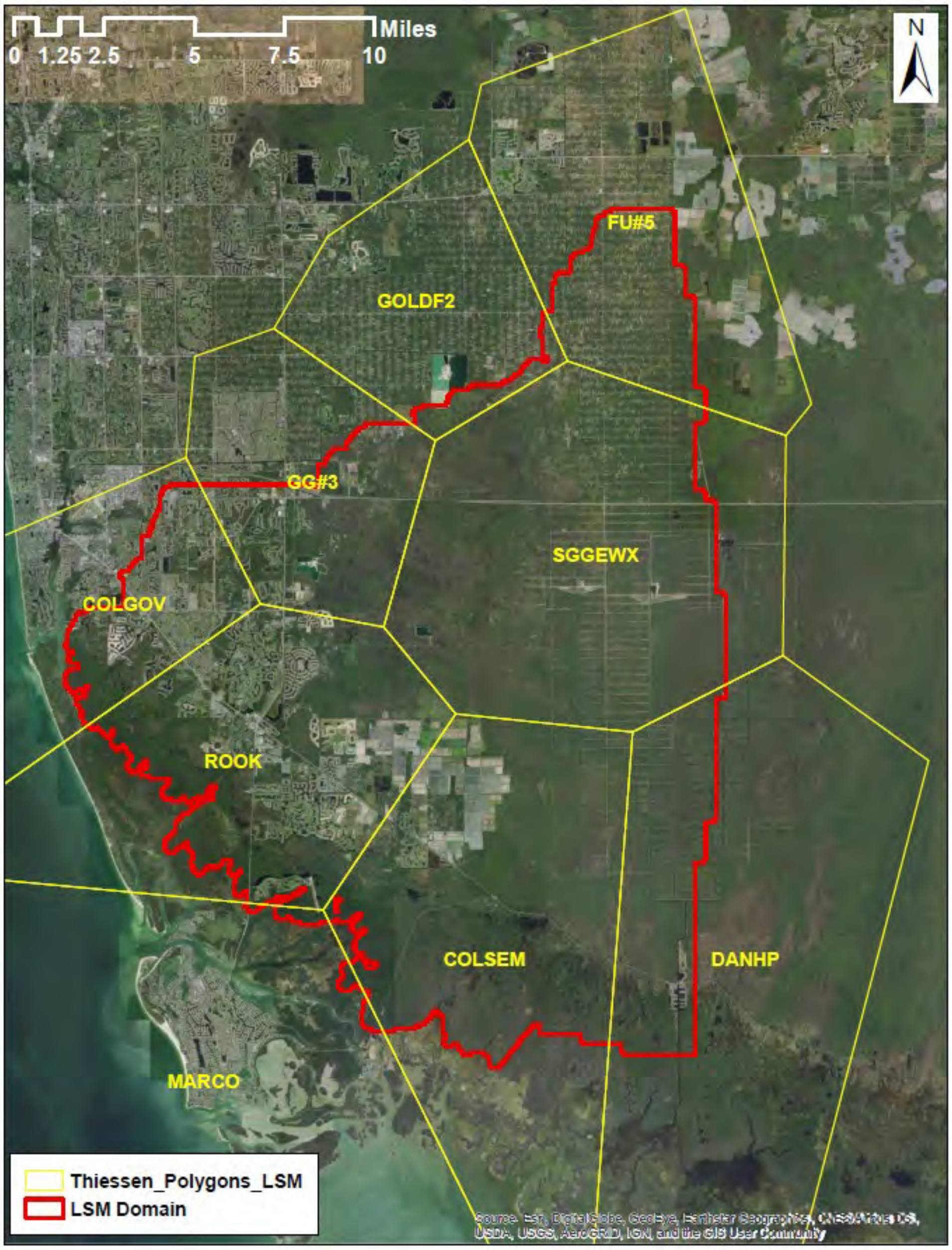


Figure 3.1 - Theisen Polygons for design storm rainfall distribution over the modeled domain.

3.1.2 Initial conditions

To represent wet antecedent watershed conditions, initial conditions were modified in the LSM current conditions, including canal water levels, overland depths and groundwater elevations. It is important to note here that although CWIP pumps will be shut off during major storm events, the antecedent wet conditions in design storm model represents water added by the project prior to storm. Other than initial conditions no modifications were made in current and with project model scenarios.

3.1.2.1 MIKE 11

For all major canals in the MIKE 11 hydraulic network, initial water levels were set at control elevations determined by control structures in the segment. This represents that the canal storages will be fully consumed prior to the storm event and any water added by the storm will initiate discharge through the structures. For example, for the segment of I-75 North Canal west of Miller Canal, the HC-2 structure, located on Henderson Creek, controls the canal elevation at 7.3 ft NAVD 88. The initial water levels for the entire segment between Miller Canal and HC-2 are thus set at 7.3 ft NAVD 88. The initial water levels were modified for I-75 South, Henderson Creek, Miller, Faka Union and Tamiami canals. MIKE 11 initial conditions are same in both the current and with project scenarios.

3.1.2.2 MIKE SHE

To represent appropriate wet antecedent conditions for overland depths and groundwater elevations, simulated 80th percentile values from the 10-year model runs were used to set initial conditions over the entire modeled domain. This approach is adopted from the recently completed Flood Protection Level of Service Study of the Big Cypress Basin commissioned by SFWMD (Parsons & Taylor Engineering, 2018). The 80th percentile values for each design storm scenario were extracted from their respective 10-year local scale model runs (Figure 3.2). This represents that overland and groundwater storages will be consumed by the project prior to the storm event and a limited storage will be available during the storm. Initial surface water and groundwater levels within the Belle Meade flow-way are generally predicted to be higher in the with-project scenario, due to the proposed re-hydration of this area. These higher initial levels will result in increased runoff volumes from the forest, in the with-project scenario.

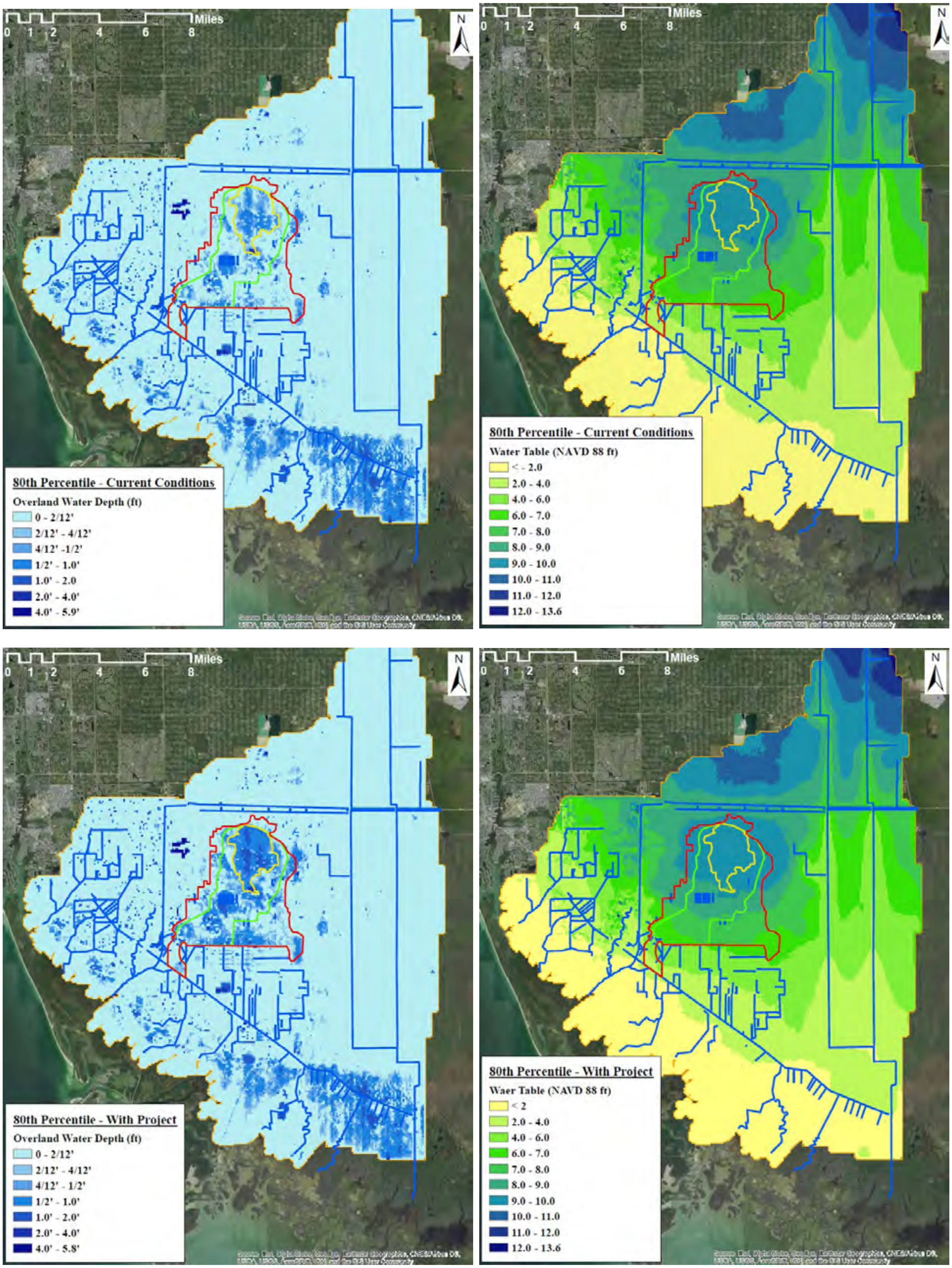


Figure 3.2 - 80th percentile groundwater elevations and overland depths used as initial conditions in the design storm models.

3.1.3 Results and Discussion

3.1.3.1 100-year Maximum Water Level Analysis

A comparison of simulated 100-year maximum water levels between current and with-project scenarios was performed and any changes in peak canal stages caused by the CWIP project were determined along the entire drainage system. The details are as follows.

<u>Impacts on Water Levels in Major Canals</u>

For the I-75-N and I-75-S canals (Figure 3.3), the simulation shows a minimal change, on the order of 0.01’ to 0.02’ (Table 3.8 and 3.9), which is de minimis and well within the margin of error for this modeling analysis.

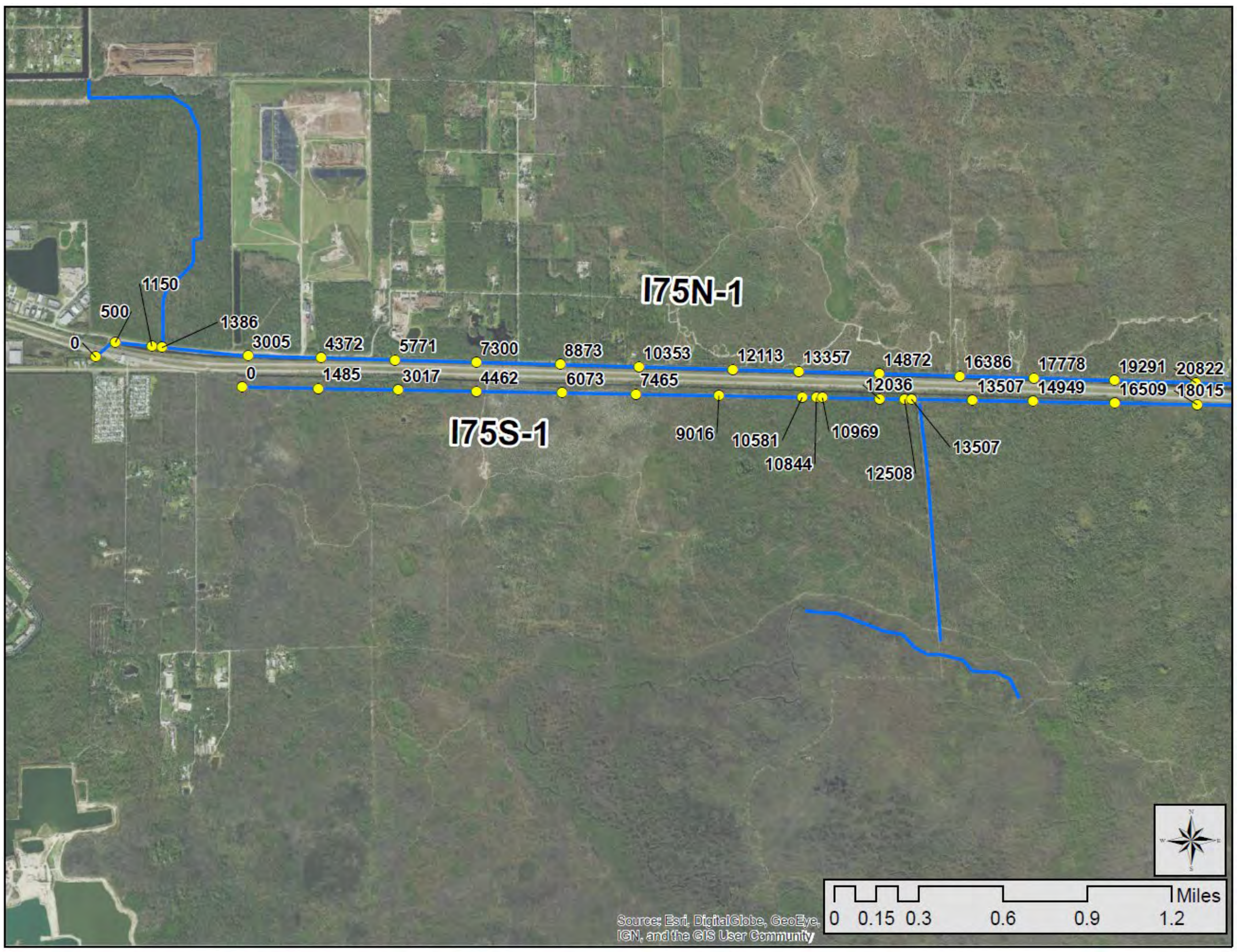


Figure 3.3 - Location of model calculation points along I-75-North and South canals at which 100-year maximum water levels are compared.

The southern portion of the drainage system includes stretch of U.S. 41 canal between Henderson Creek and Sandpiper Dr. (Figure 3.4). The area is located close to Rookery Bay and is influenced by tidal waters. Consequently, regulatory flood elevations in the area are controlled by storm surge rather than riverine surge. The FEMA flood map (FEMA 2014) shows a storm surge elevation of 7.0 ft NAVD 88 in the area, which is substantially higher than the CWIP 100-year maximum riverine water levels, ranging between 3.55 to 5.20 ft NAVD88 (Table 10). Furthermore, the lowest edge of pavement on Tamiami Trail E varies between 7.0 ft NAVD88, near Henderson Creek, to 8.0 ft NAVD 88, near Sandpiper Dr., which is 2.8 ft to 3.42 ft above the CWIP simulated maximum levels. The simulation shows a maximum increment of 0.19 ft, at the confluence with Naples Reserve flow-way, which decreases northwest to 0.08 ft at the confluence with Henderson Creek. From the above discussion it follows that incremental water level changes resulting from the CWIP in canal stages along U.S. 41 are well below the elevation of the low edge of pavement and the FEMA flood elevations and therefore will not constitute an adverse impact on FDOT-maintained infrastructure. The proposed peak stage information has been reviewed and approved by FDOT (Appendix C).

Figure 3.4 - Location of model calculation points along U.S. 41 Canal at which 100-year maximum water levels are compared.

Flows added by the project at I-75 North canal are prevented from flowing directly into Henderson Creek by means of a control structure (see section 2 for details), however, due to high groundwater levels and overland water elevations in the forest, a small fraction of water is expected to flow into Henderson Creek as baseflow and overland flow. Throughout most of the canal length, the simulation shows a minimal change in water levels, on the order of 0.01' to 0.02' (Table 3.7), which again is de minimis and well within the margin of error for this modeling analysis. The southern section of the canal, between chainages 42333 ft and 50113 ft, shows an increase between 0.03’ to 0.08’. The overland water from the rehydrated area flows south-west towards Henderson Creek and enters the canal at chainage 42333 ft through existing flow-ways and culverts under Winding Cypress Dr. (Figure 3.4) and causes water levels to increase slightly. As explained earlier, this area is controlled by storm surge rather than riverine surge. The CWIP 100-year maximum riverine water levels, ranging between 5.63ft to 2.39 ft NAVD88 (Table 3.7), are well below the FEMA storm surge elevation for the area, 7.0 ft NAVD 88.

Sabal Palm Rd. Culverts

The water flows southwest in the forested flow way under the Sabal Palm Rd. through existing culverts (Figure 3.10). The water levels in the forest are higher than the roadway elevation in the vicinity of the culverts. On average, the water levels are 0.85 ft higher than the roadway elevation. The increments caused by the project averages around ~0.15’ on the east side culverts and 0.06’ on the west side culverts (Table 3.13). If required, the County may propose to increase pavement thickness of the roads where needed to mitigate any potential increase of flood levels along the roadway. The results are shown only for a few select culverts for the sake of explanation.

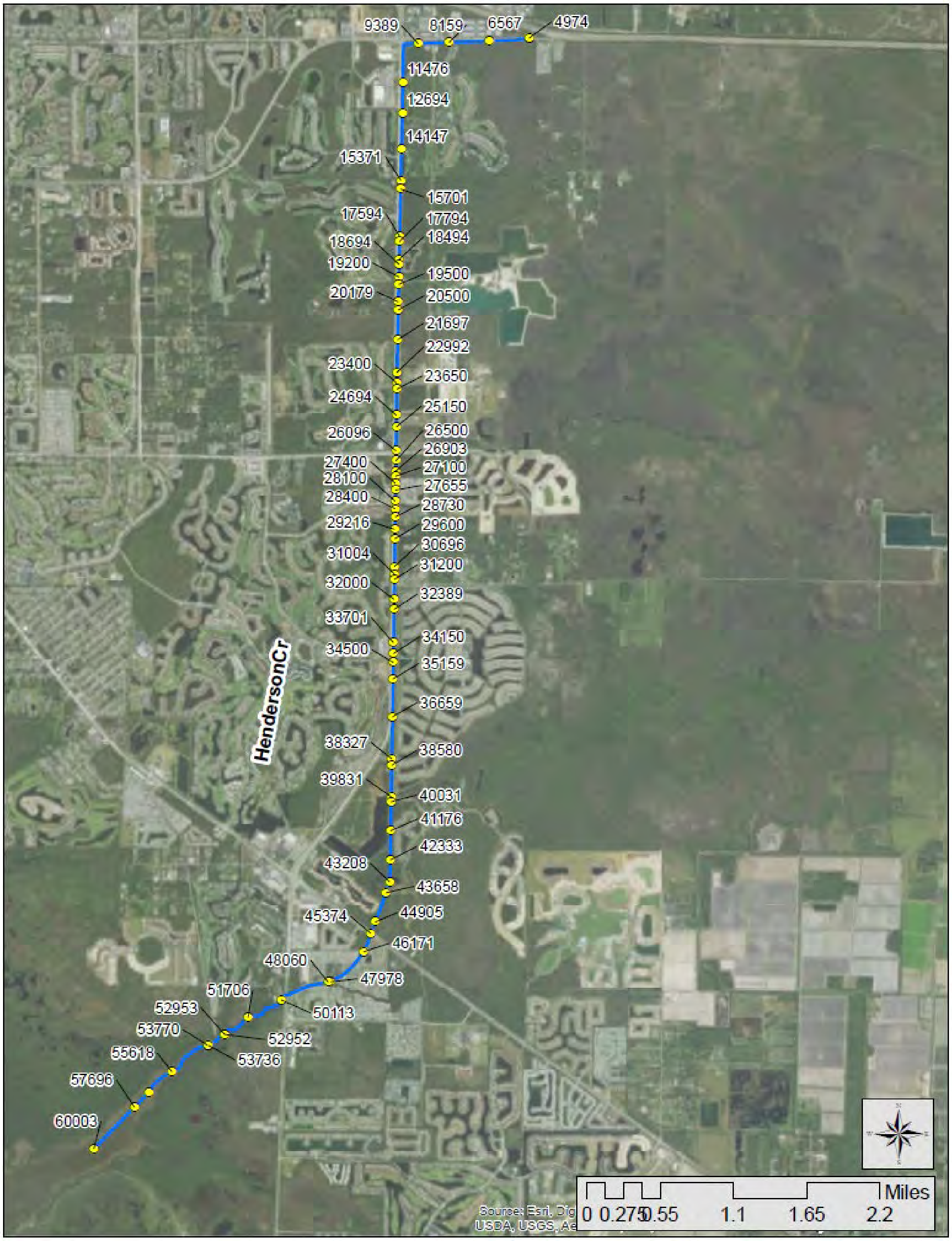


Figure 3.5 - Location of model calculation points in Henderson Creek canal at which 100-year maximum water levels are compared.

<u>Impacts on Water Levels in Residential Developments</u>

The CCWIP rehydration areas also include residential developments located southwest of Belle Meade Forest and south and north of U.S. 41. These include Winding Cypress, Naples Reserve, Fiddlers Creek and adjoining developments. Due to the proposed rehydration of the forest, higher overland water elevations and groundwater levels are expected in the with-project scenario in areas surrounding the residential developments. The 100-year overland water elevations are compared with either the design/as-built elevations, where available, or the most recent LiDAR-based DEM (USGS 2018) elevations to determine potential inundation. The design/as-built elevations for proposed and existing developments are available from SWFMD e-Permitting database, while for some existing developments, the information is extracted from latest LiDAR data (USGS, 2018).

The rehydration of the forest during the wet season increases the groundwater table along the north side of the Naples Reserve development, which in turn impacts the lake levels slightly. Internal lakes in Naples Reserve show an increase of 0.10' and 0.15' above the existing conditions along southern and northern boundary lines respectively (Figure 3.6).The maximum with-project water level within the development is 6.37 ft NAVD 88 which is 1.13' below the 100-year design elevation for the development, 7.5 ft NAVD 88 (Table 3.4). In the development southeast of Naples Reserve, the simulation shows a minimal change, on the order of 0.01' to 0.02' (Table 3.11), which is de minimis and well within the margin of error for this modeling analysis.

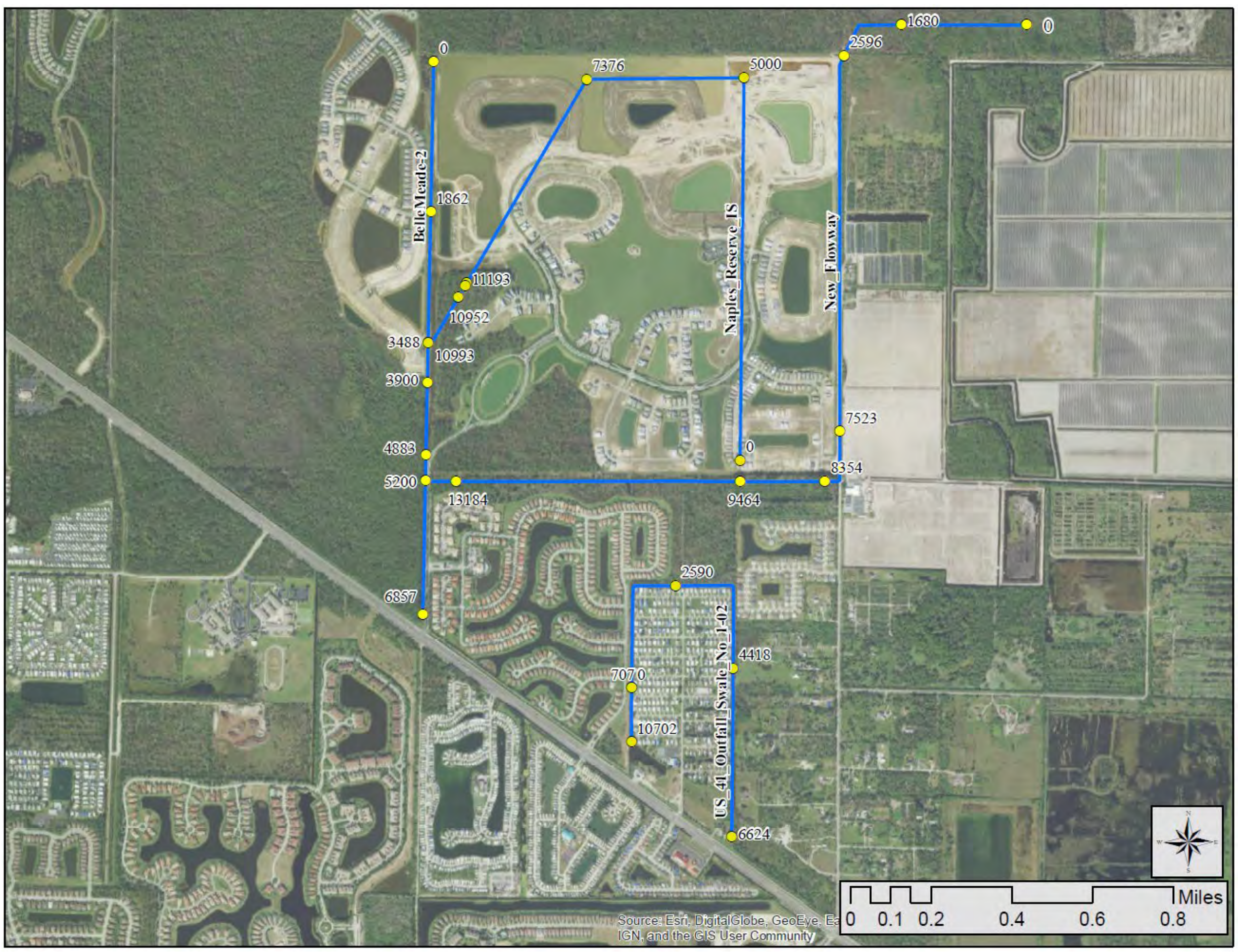


Figure 3.6 - Location of model calculation points in residential developments north of U.S. 41 at which 100-year maximum water levels are compared.

Two flow-ways are proposed along the west and east boundary of Naples Reserve to convey project water around the developments (Figure 6). BelleMeade-2 is an existing channel with some proposed design modifications, whereas, "New_Flowway" is a proposed channel along the eastern boundary of the development. Both the channels will be located within existing easements or right-of-way reservations. In Bellemeade-2, design storm water levels are considerably lower than the berm elevations of Winding Cypress sub-division on the east side and Naples Reserve berm on the west side (Table 3.2). In New_Flowway, the water levels are considerably lower than the berm elevations of the adjacent residential developments. The water level may be close to or slightly higher than the existing ground elevations within the easement at some locations along the east-west stretch, however, these are ungraded areas reserved for access roads and are proposed to be graded and/or raised during construction.

Winding Cypress development is spread north and south of Winding Cypress Dr., which cuts through the project flow-way at its southwest end. For modeling purposes, the development is divided into seven subunits with five located south of Winding Cypress Dr. and two on the north (Figure 3.7). The 100-year water levels in the developments south of Winding Cypress Dr., are in the range of 1.57 ft to 2.79 ft below the lowest edge of the pavements, except for Winding Cypress IS2, where the water level is 0.90 ft below the lowest edge of pavement (Table 3.5). The 100-year FEMA flood elevations in the area surrounding the developments varies between 7.0 and 8.0 ft NAVD 88 which is higher than the predicted maximum 100-year water elevation of 6.93 ft NAVD 88. From the above discussion it follows that incremental water level changes resulting from the CWIP are well below the elevation of the low edge of pavements and FEMA flood elevations and therefore will not constitute an adverse impact on the developments.

On the northern side of Winding Cypress Dr., Winding Cypress IS3, and IS4 show a maximum increase of 0.10 ft and 0.26 ft above the existing conditions respectively, however, simulated 100-year water levels in the forest slightly exceed the lowest edge of pavement in both the sub-units under existing and with-project conditions. Under existing conditions, the water levels are 0.12 ft above the lowest edge of the pavement which increased to 0.22 ft in the with-project conditions. In Winding Cypress IS4, the low edge of pavement varies between 7.50 and 8.5 ft NAVD 88 (Figure 3.8) and only small stretches along the road are below the 100-year water elevations. Whereas, in Winding Cypress IS3 only one point along the roadway is at 7.6 ft NAVD 88 and the rest is higher than flood elevations. All roadways in the subdivision will remain safely passable during a 100-year flood, and no homes, businesses, or any other infrastructure will be threatened. Therefore, the minor increases expected here can be considered *de minimums* impacts. However, if required, the County may propose to increase pavement thickness of the roads where needed to mitigate any potential increase of flood levels within the developments.

South of U.S. 41 two main canal systems, BelleMeade 10 and BelleMeade 11 (Figure 3.9), convey the majority of water through the Fiddler's Creek residential developments and into the Rookery bay. The maximum water level increment caused by the project south of U.S. 41 is 0.18' above the existing conditions (Table 3.2, 3.3 and 3.12). The 100-year maximum water level in all the canals is below the approximate minimum pavement elevation in residential units and/or the canal bank elevations and hence does not cause any adverse impacts on the adjacent residential developments. Again, 100-year base flood elevations in this area are governed by the FEMA storm surge elevations.

3.1.3.2 25-year Maximum Water Level Analysis

A comparison of simulated 25-year maximum water levels between current and with-project scenarios was performed and the increases in canal stages caused by the CWIP project were determined along the entire drainage system similar to the methodology adopted in 100-year design storm. The 25-year water levels are generally considerably lower than the 100-year levels and do not cause any adverse impact on the infrastructure or residential developments described earlier. In Winding Cypress, subdivisions IS3 and IS4, 25-year maximum water levels are 0.16’ and 0.22’ lower than the lowest edge of pavement and does not cause any inundation.

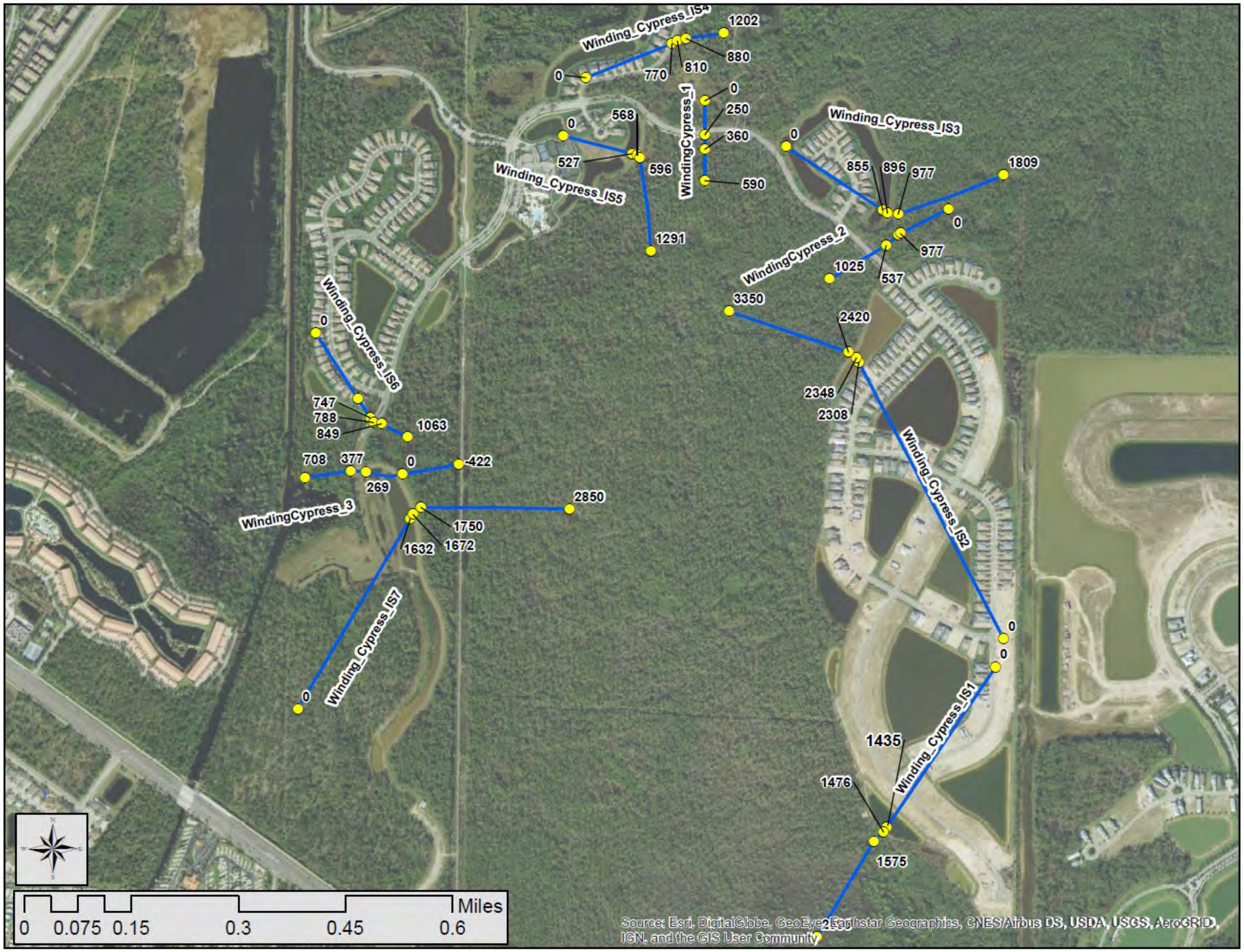
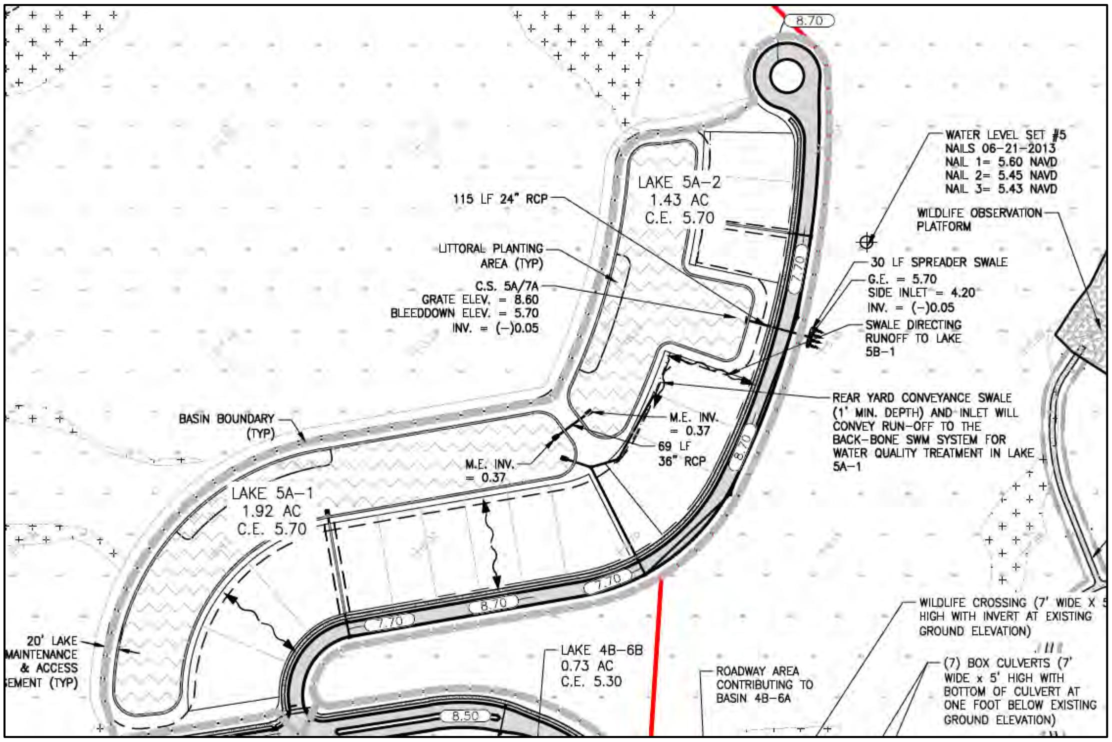


Figure 3.7 - Location of model calculation points in the subunits of Winding Cypress development at which 100-year maximum water levels are compared.



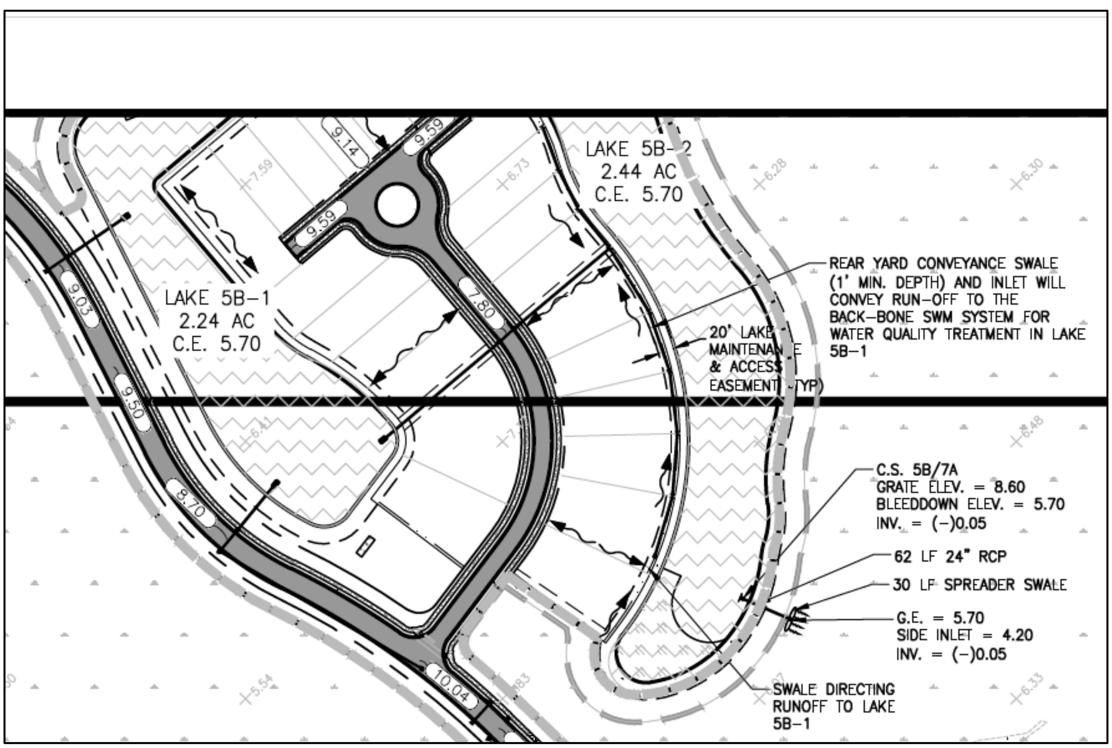


Figure 3.8 - Winding Cypress ERP drawings (SFWMD Permit # 11-02132-P). Designed roadway elevations are shown within the residential units: IS 4 (left) and IS 3 (right).

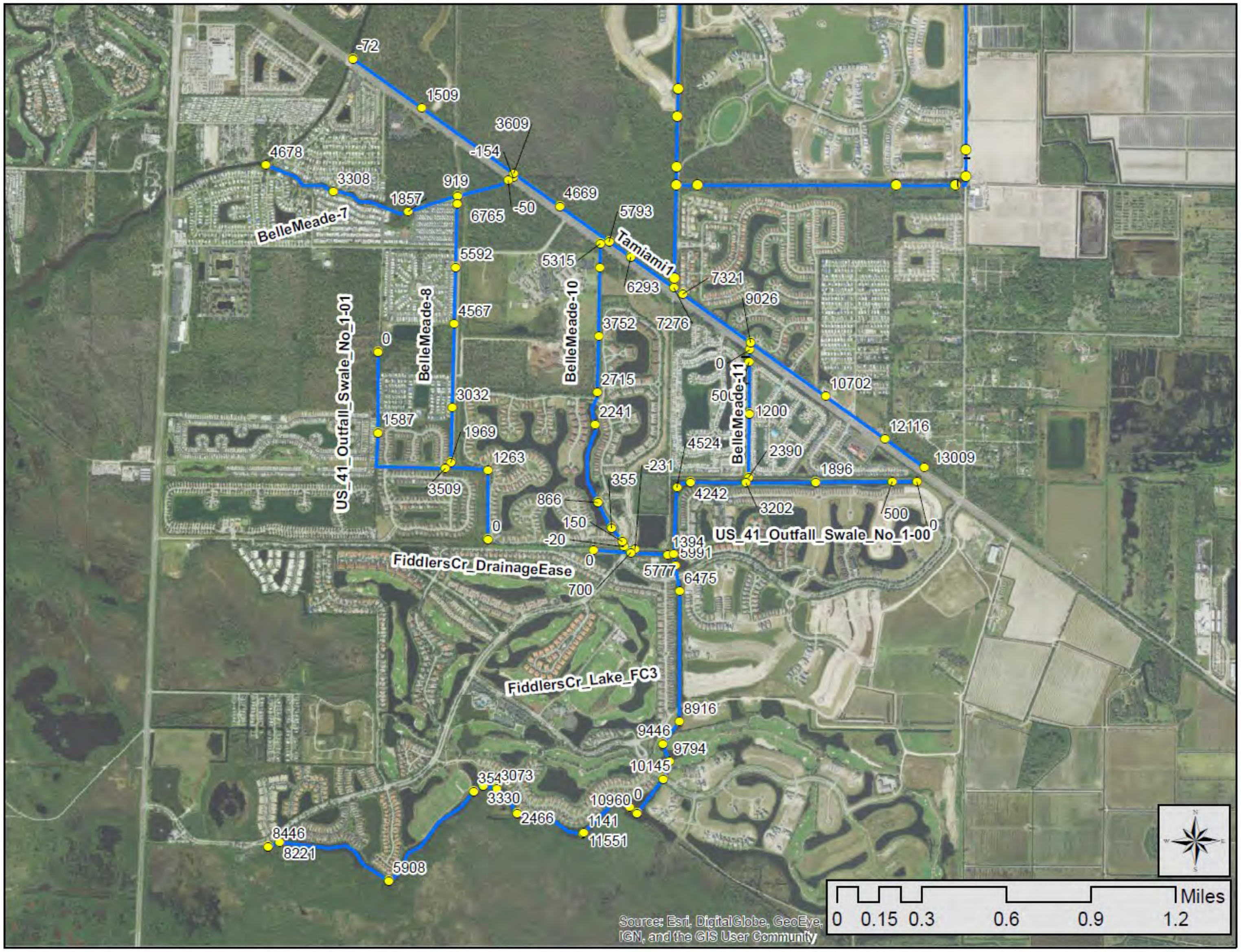


Figure 3.9 - Location of model calculation points in residential development south of U.S. 41at which 100-year maximum water levels are compare

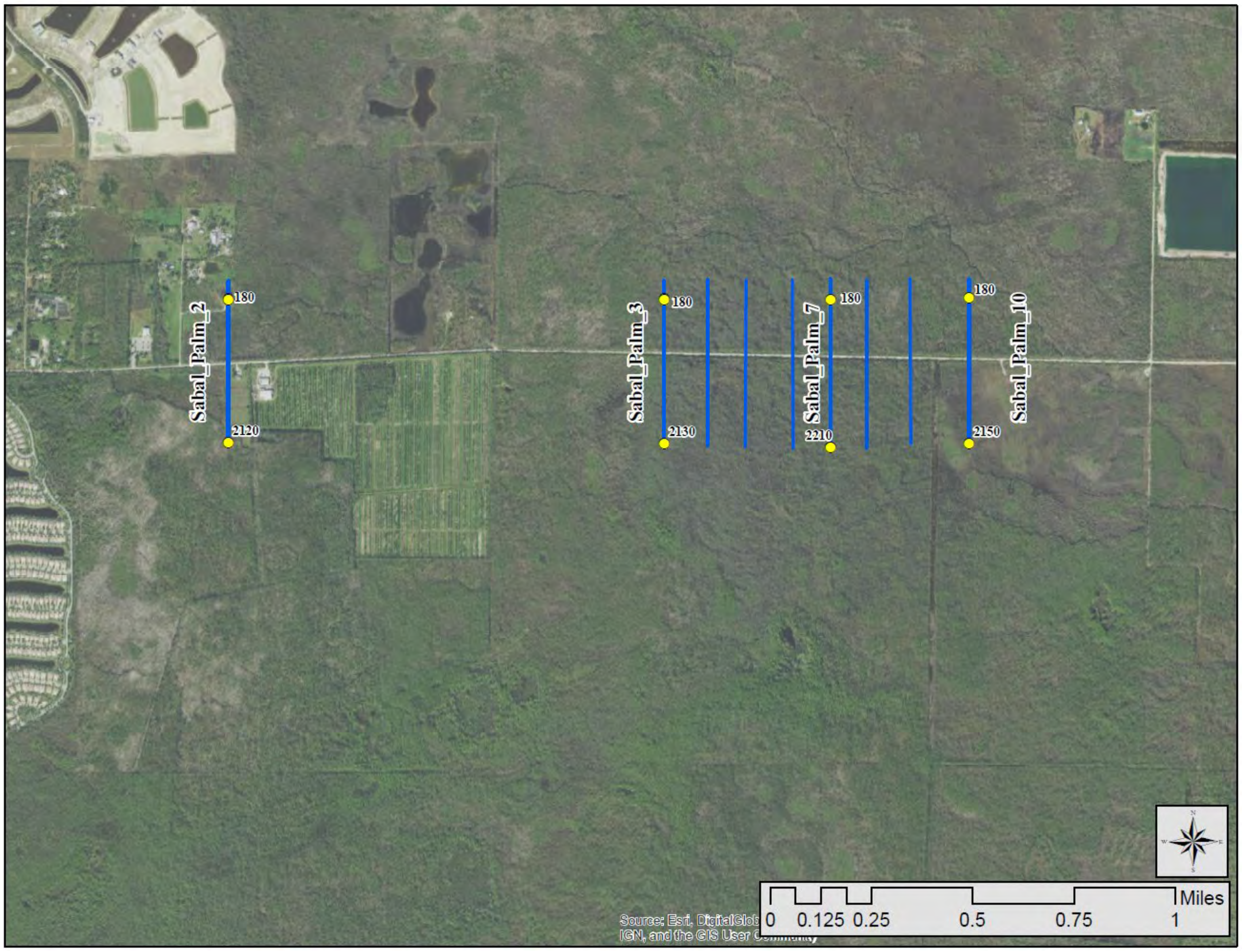


Figure 3.10 - Location of model calculation points along Sabal Palm Rd. at which 100-year maximum water levels are compared.

Table 3.2: 100-year and 25-year water levels in canals going through Fiddler's Creek residential areas south of U.S.41. All elevations are in feet NAVD 88. Approximate minimum elevation shown are for the adjacent pavements based on DEM 2018 near the h-point and only at locations where developments exists.

Canal Name	Chainage	100-Year			25 -Year			Approx. Min. Elev.
		Current Conditions	With Project	Diff	Current Conditions	With Project	Diff	
BelleMeade-10	231	3.59	3.68	0.09	3.26	3.39	0.13	5.00
	20	3.62	3.71	0.09	3.28	3.41	0.13	5.00
	150	3.74	3.85	0.11	3.37	3.52	0.15	5.00
	355	3.88	4.01	0.13	3.48	3.65	0.17	5.00
	866	3.88	4.01	0.13	3.48	3.65	0.17	5.00
	2241	3.88	4.01	0.13	3.48	3.65	0.17	5.00
	2715	3.99	4.16	0.16	3.55	3.74	0.19	5.00
	3752	4.07	4.24	0.17	3.61	3.81	0.20	5.59
	4814	4.17	4.35	0.18	3.70	3.91	0.22	-
	5315	4.20	4.40	0.19	3.73	3.95	0.22	-
BelleMeade-11	0	3.47	3.55	0.08	3.10	3.24	0.14	5.20
	500	3.39	3.48	0.10	3.06	3.17	0.12	5.50
	1200	3.16	3.30	0.14	2.84	2.98	0.14	6.00
	2390	2.65	2.67	0.02	2.43	2.46	0.02	6.50
BelleMeade-2	0	4.98	7.85	N/A	4.86	7.48	2.62	-
	1862	4.99	5.33	0.34	4.86	5.00	0.14	8.00
	3488	4.99	5.30	0.31	4.86	4.99	0.13	8.00
	3900	4.99	5.30	0.31	4.86	4.98	0.12	-
	4883	4.98	5.17	0.19	4.85	4.94	0.09	7.25
	5200	4.96	5.14	0.18	4.84	4.94	0.11	7.25
	6857	4.20	4.37	0.18	3.74	3.94	0.20	7.25

Table 3.3: 100-year and 25-year water levels in lakes south of Fiddler's Creek residential area. All elevations are in feet NAVD 88. Approximate minimum elevation is shown for canal banks based on DEM 2018 near the h-point and only at locations where developments exists.

Canal Name	Chainage	100-Year			25 -Year			Approx. Min. Elev.
		Current Conditions	With Project	Diff	Current Conditions	With Project	Diff.	
US_41_OUTFALL SWALE_NO_1-00	0	2.65	2.67	0.02	2.43	2.46	0.02	Varies between 5.0 to 7.0 ft
	240	2.65	2.67	0.02	2.43	2.46	0.02	
	1896	2.65	2.67	0.02	2.43	2.46	0.03	
	3202	2.65	2.67	0.02	2.43	2.46	0.03	
	3202	2.65	2.67	0.02	2.43	2.46	0.03	
	4242	2.64	2.67	0.02	2.43	2.45	0.02	
	4524	2.65	2.67	0.02	2.43	2.45	0.02	
	5777	2.64	2.66	0.02	2.43	2.45	0.02	
	5777	2.64	2.66	0.02	2.43	2.45	0.02	
	5991	2.65	2.67	0.02	2.43	2.45	0.02	
	6475	2.64	2.67	0.02	2.43	2.45	0.02	
	8916	2.64	2.66	0.02	2.43	2.45	0.02	
	9446	2.62	2.64	0.02	2.41	2.44	0.02	
	9793	2.63	2.65	0.02	2.42	2.44	0.02	
	10145	2.62	2.64	0.02	2.41	2.44	0.02	
	10960	2.60	2.62	0.02	2.40	2.43	0.02	
	10960	2.60	2.62	0.02	2.40	2.43	0.02	
	11551	2.60	2.62	0.02	2.40	2.42	0.02	
	13290	2.10	2.12	0.02	1.81	1.83	0.02	
	15199	1.96	1.98	0.02	1.69	1.71	0.02	
	16393	1.84	1.86	0.01	1.59	1.61	0.02	
	19329	1.55	1.55	0.00	1.54	1.55	0.01	
	22395	1.58	1.58	0.00	1.58	1.58	0.00	
	24704	1.60	1.60	0.00	1.60	1.60	0.00	
FIDDLERSCR	0	3.59	3.68	0.09	3.26	3.39	0.13	

DRAINAGEEASE	700	3.59	3.68	0.09	3.26	3.39	0.13	Varies between a min. of 5.0 to 6.0 ft
	700	3.59	3.68	0.09	3.26	3.39	0.13	
	1394	2.64	2.66	0.02	2.43	2.45	0.02	
FIDDLERSCR LAKE_FC3	0	2.62	2.60	-0.02	2.40	2.43	0.02	
	1141	2.62	2.60	-0.02	2.40	2.42	0.02	
	2466	2.61	2.59	-0.02	2.40	2.42	0.02	
	3073	2.61	2.59	-0.02	2.40	2.42	0.02	
	3330	2.61	2.59	-0.02	2.40	2.42	0.02	
	3541	2.61	2.59	-0.02	2.40	2.42	0.02	
	5908	2.61	2.59	-0.02	2.40	2.42	0.02	
	8221	2.61	2.59	-0.02	2.40	2.42	0.02	
	8446	2.61	2.59	-0.02	2.40	2.42	0.02	

Table 3.4: 100-year and 25-year water levels internal lakes in Naples Reserve. residential area. All elevations are in feet NAVD 88. Approximate minimum elevation shown are 100-year designed elevations based on ERP (SFWMD Permit # 11-00090-S-02).

Chainage	**Chainage**	**100-Year**			**25 -Year**			**Approx. Min. Elev.**
		Current Conditions	**With Project**	**Diff**	**Current Conditions**	**With Project**	**Diff**	
NAPLES_RESERVE_IS	0	6.27	6.37	0.10	6.16	6.16	-0.01	7.5
	5000.01	6.26	6.37	0.10	6.16	6.16	-0.01	
	7376.09	6.26	6.37	0.10	6.16	6.16	-0.01	
	10952	6.27	6.37	0.10	6.16	6.16	-0.01	
	10993	5.21	5.35	0.15	5.05	5.13	0.08	
	11193	5.21	5.35	0.15	5.05	5.13	0.08	
	11993	5.00	5.30	0.30	5.00	5.00	0.00	

Table 3.5: 100-year and 25-year water levels internal lakes in Winding Cypress residential area. All elevations are in feet NAVD 88. Approximate minimum elevation shown are minimum pavement elevations inside each sub-division based on ERP (SFWMD Permit # 11-02132-P).

Chainage	100-Year			25 -Year			
	Current Conditions	With Project	Diff	Current Conditions	With Project	Diff	Approx. Min. Elev.
WINDING_CYPRESS_IS1	5.43	5.59	0.16	5.17	5.31	0.15	8.10
	5.43	5.59	0.16	5.17	5.31	0.15	
	5.00	5.00	0.00	5.00	5.00	0.00	
	4.69	4.89	0.20	4.46	4.67	0.21	
	4.69	4.89	0.20	4.46	4.67	0.21	
WINDING_CYPRESS_IS2	6.80	6.88	0.08	6.42	6.49	0.08	7.60
	6.80	6.88	0.08	6.42	6.49	0.08	
	6.82	6.90	0.08	6.47	6.55	0.08	
	6.82	6.90	0.08	6.47	6.55	0.08	
	6.84	6.93	0.08	6.48	6.56	0.08	
WINDING_CYPRESS_IS3	7.71	7.81	0.10	7.34	7.42	0.09	7.60
	7.71	7.81	0.10	7.34	7.42	0.09	
	7.72	7.82	0.10	7.35	7.43	0.09	
	7.72	7.82	0.10	7.35	7.43	0.09	
	7.72	7.82	0.10	7.35	7.44	0.09	
WINDING_CYPRESS_IS4	6.92	7.18	0.26	6.54	6.65	0.11	7.50
	6.92	7.18	0.26	6.54	6.65	0.11	
	7.58	7.72	0.14	7.13	7.28	0.15	
	7.58	7.72	0.14	7.13	7.28	0.15	
	7.58	7.72	0.14	7.13	7.28	0.15	
WINDING_CYPRESS_IS5	6.71	6.84	0.13	6.35	6.43	0.08	8.00
	6.71	6.84	0.13	6.35	6.43	0.08	
	6.73	6.82	0.09	6.38	6.46	0.08	
	6.73	6.82	0.09	6.38	6.46	0.08	
	6.73	6.82	0.09	6.38	6.46	0.08	
WINDING_CYPRESS_IS6	5.70	5.79	0.09	5.09	5.21	0.12	7.50

	5.70	5.79	0.09	5.09	5.21	0.12	
	5.88	5.96	0.08	5.54	5.62	0.08	
	5.88	5.96	0.08	5.54	5.62	0.08	
	5.88	5.96	0.08	5.54	5.62	0.08	
WINDING_CYPRESS_IS7	5.13	5.23	0.11	4.70	4.70	0.00	6.90
	5.13	5.23	0.11	4.70	4.70	0.00	
	5.50	5.56	0.06	5.14	5.21	0.07	
	5.50	5.56	0.06	5.14	5.21	0.07	
	5.50	5.56	0.06	5.14	5.21	0.07	

Table 3.6: 100-year and 25-year water levels in culverts under Winding Cypress Dr. The approximate minimum elevations shown are for the Winding Cypress Drive near the culverts based on DEM. All elevations are in feet NAVD 88.

Chainage	100-Year			25 -Year			Approx. Min. Elev.
	Current Conditions	With Project	Difference	Current Conditions	With Project	Difference	
WINDINGCYPRESS_1	7.08	7.20	0.12	6.67	6.77	0.10	12.00
	7.08	7.19	0.12	6.67	6.77	0.10	
	7.07	7.18	0.11	6.66	6.76	0.10	
	7.07	7.18	0.11	6.66	6.76	0.10	
WINDINGCYPRESS_2	7.08	7.17	0.09	6.68	6.77	0.08	10.50
	7.07	7.16	0.09	6.68	6.76	0.08	
	7.06	7.15	0.09	6.67	6.75	0.08	
	7.06	7.15	0.09	6.67	6.75	0.08	
WINDINGCYPRESS_3	5.51	5.56	0.04	5.11	5.13	0.02	7.90
	5.51	5.55	0.04	5.11	5.13	0.02	
	5.49	5.53	0.04	5.11	5.12	0.02	
	5.47	5.51	0.03	5.10	5.11	0.02	
	5.47	5.51	0.03	5.10	5.11	0.02	

Table 3.7: 100-year and 25-year water levels in Henderson Creek. The approximate elevations shown are for Collier Blvd. on the west side of the canal. Th lowest edge of the pavement elevations are shown for random points along the length for an approximate roadway profile. All elevations are in feet NAVD 88.

Henderson Creek Canal Chainage	100-Year			25 -Year			Approx. Min. Elev.
	Current Conditions	With CWI Project	Difference	Current Conditions	With CWI Project	Difference	
4974	10.78	10.77	-0.02	10.46	10.44	-0.01	14.40
6567	10.71	10.70	-0.01	10.39	10.39	-0.01	
8159	10.62	10.62	0.00	10.29	10.29	0.00	
9389	10.57	10.58	0.00	10.25	10.26	0.00	
11476	10.57	10.57	0.00	10.24	10.25	0.01	13.50
12694	10.57	10.57	0.00	10.24	10.28	0.03	
14147	10.54	10.54	0.00	10.22	10.22	0.01	
15371	10.44	10.44	0.00	10.12	10.13	0.01	
15701	10.15	10.15	0.00	9.67	9.72	0.05	
17594	10.12	10.12	0.01	9.63	9.68	0.06	12.90
17794	10.11	10.12	0.01	9.62	9.68	0.06	
18494	10.09	10.09	0.01	9.60	9.65	0.05	
18694	10.08	10.09	0.01	9.59	9.65	0.05	
19200	10.06	10.07	0.01	9.57	9.62	0.05	
19500	10.03	10.04	0.01	9.54	9.59	0.05	
20179	10.02	10.02	0.01	9.52	9.57	0.05	
20500	9.31	9.31	0.01	8.71	8.73	0.02	
21697	9.25	9.25	0.01	8.61	8.64	0.02	
22992	9.18	9.19	0.01	8.54	8.56	0.03	
23400	9.14	9.15	0.01	8.50	8.52	0.03	
23650	9.11	9.12	0.01	8.47	8.49	0.03	
24694	9.06	9.07	0.01	8.41	8.44	0.03	
25150	9.02	9.03	0.01	8.38	8.41	0.03	
26096	8.96	8.97	0.01	8.32	8.35	0.03	
26500	8.93	8.94	0.01	8.30	8.32	0.03	

26903	8.92	8.93	0.01	8.27	8.30	0.03	
27100	8.89	8.90	0.01	8.25	8.27	0.03	
27400	8.86	8.87	0.01	8.22	8.24	0.03	
27655	8.86	8.87	0.01	8.22	8.24	0.03	11.7
28100	8.82	8.83	0.01	8.18	8.20	0.03	
28400	8.79	8.80	0.01	8.15	8.17	0.02	
28730	8.57	8.58	0.01	7.95	7.97	0.02	
29216	8.56	8.57	0.01	7.94	7.96	0.02	
29600	8.45	8.45	0.01	7.85	7.88	0.02	
30696	8.39	8.40	0.01	7.80	7.82	0.01	
31004	8.35	8.36	0.01	7.77	7.78	0.01	11.8
31200	8.30	8.31	0.01	7.71	7.72	0.02	
32000	8.09	8.10	0.01	7.59	7.61	0.02	
32389	7.97	7.98	0.01	7.48	7.50	0.02	
33702	7.60	7.61	0.01	7.14	7.16	0.02	
34150	7.50	7.51	0.01	7.05	7.07	0.02	
34500	7.47	7.48	0.01	7.03	7.05	0.02	
35159	7.41	7.42	0.01	6.97	6.99	0.02	
36659	7.20	7.20	0.01	6.72	6.74	0.02	11.6
38327	6.57	6.59	0.02	6.14	6.16	0.03	
38580	6.46	6.48	0.02	6.07	6.09	0.02	
39831	6.14	6.15	0.01	5.76	5.78	0.02	
40031	6.13	6.14	0.02	5.75	5.77	0.02	
41176	5.94	5.96	0.02	5.56	5.58	0.02	
42333	5.60	5.63	0.03	5.21	5.23	0.02	
43208	5.47	5.51	0.03	5.10	5.11	0.02	
43658	5.40	5.43	0.03	5.03	5.05	0.02	
44905	5.24	5.27	0.03	4.93	4.94	0.01	
45374	3.48	3.56	0.08	3.02	3.09	0.07	7.5
46171	3.30	3.37	0.07	2.84	2.91	0.07	-
47978	2.93	2.97	0.04	2.44	2.51	0.07	-

48060	2.89	2.93	0.04	2.40	2.47	0.07	-
50113	2.36	2.39	0.03	1.96	2.01	0.05	-
51706	2.02	2.04	0.02	1.69	1.73	0.04	-
52952	1.73	1.74	0.02	1.59	1.59	0.00	-
52953	1.72	1.74	0.02	1.59	1.59	0.00	-
53736	1.59	1.59	0.00	1.59	1.59	0.00	-
53770	1.59	1.59	0.00	1.59	1.59	0.00	-
55618	1.59	1.59	0.00	1.59	1.59	0.00	-
56872	1.59	1.59	0.00	1.59	1.59	0.00	-
57696	1.60	1.60	0.00	1.60	1.60	0.00	-
60003	1.60	1.60	0.00	1.60	1.60	0.00	-

Table 3.8: 100-year and 25-year water levels in I-75 North Canal. The approximate elevations shown are for Alligator Alley on the south side of the canal. Th lowest edge of the pavement elevations are shown for random points along the length for an approximate roadway profile. All elevations are in feet NAVD 88.

I-75 North Canal Chainage	100-Year			25 -Year			Approx. Min. Elev.
	Current Conditions	With CWI Project	Difference	Current Conditions	With CWI Project	Difference	
0	10.78	10.77	-0.02	10.46	10.44	-0.01	13.00
500	10.84	10.82	-0.02	10.52	10.50	-0.02	
1150	10.84	10.82	-0.02	10.52	10.50	-0.02	
1386	10.84	10.85	0.01	10.52	10.52	0.00	
3005	10.85	10.85	0.01	10.52	10.52	0.00	
4372	10.85	10.85	0.00	10.52	10.53	0.00	
5771	10.85	10.85	0.00	10.52	10.53	0.00	
7300	10.96	10.93	-0.03	10.60	10.57	-0.03	
8873	10.96	10.93	-0.03	10.60	10.57	-0.03	
10353	10.96	10.93	-0.03	10.60	10.57	-0.03	13.40
12114	10.96	10.93	-0.03	10.60	10.57	-0.03	
13357	10.96	10.93	-0.03	10.60	10.57	-0.03	
14873	10.96	10.93	-0.03	10.60	10.57	-0.03	

16386	10.96	10.93	-0.03	10.60	10.57	-0.03	
17778	10.96	10.93	-0.03	10.60	10.57	-0.03	
19291	10.96	10.93	-0.03	10.59	10.56	-0.03	
20822	10.96	10.93	-0.03	10.59	10.56	-0.03	
22352	10.96	10.93	-0.03	10.58	10.56	-0.03	
23858	10.96	10.93	-0.02	10.58	10.56	-0.02	
25339	10.96	10.93	-0.02	10.58	10.56	-0.02	14.00
26848	10.96	10.93	-0.02	10.58	10.55	-0.02	
28317	10.96	10.93	-0.02	10.57	10.55	-0.02	
29839	10.96	10.93	-0.02	10.57	10.55	-0.02	
31321	10.96	10.93	-0.02	10.57	10.55	-0.02	
32834	10.95	10.93	-0.02	10.56	10.54	-0.02	
34372	10.95	10.93	-0.02	10.55	10.53	-0.02	
35837	10.95	10.92	-0.02	10.54	10.52	-0.02	
37277	10.42	10.42	0.00	9.72	9.72	0.00	14.20

Table 3.9: 100-year and 25-year water levels in I-75 North Canal. The approximate elevations shown are for Alligator Alley on the north side of the canal. Th lowest edge of the pavement elevations are shown for random points along the length for an approximate roadway profile. All elevations are in feet NAVD 88.

Table 3.10: 100-year and 25-year water levels in U.S. 41 Canal stretch between Henderson Creek and Sandpiper Dr. All elevations are in feet NAVD 88

U.S. 41/Tamiami Canal Chainage	100-Year			25 -Year			Approx. Min. Elev.
	Current Conditions	**With CWI Project**	**Difference**	**Current Conditions**	**With CWI Project**	**Difference**	
-72	3.48	3.56	0.08	3.02	3.09	0.07	Varies between 7.0 ft and 8.0 ft
1509	4.42	4.59	0.17	4.01	4.23	0.22	
3609	4.42	4.59	0.17	4.00	4.22	0.22	
4669	4.39	4.57	0.19	3.95	4.19	0.24	
5793	4.20	4.40	0.19	3.73	3.95	0.22	
6293	4.20	4.40	0.19	3.73	3.94	0.21	

7321	4.20	4.37	0.18	3.74	3.94	0.20	
9026	3.47	3.55	0.08	3.10	3.24	0.14	
10702	4.84	4.86	0.02	4.57	4.59	0.02	
12116	5.19	5.20	0.01	4.93	4.94	0.01	
13009	5.19	5.20	0.01	4.92	4.93	0.01	

Table 3.11: 100-year and 25-year water levels in residential development southeast of Naples reserve. All elevations are in feet NAVD 88.

Chainage	100-Year			25 -Year			Approx. Min. Elev.
	Current Conditions	With CWI Project	Difference	Current Conditions	With CWI Project	Difference	
US_41_OUTFALL_SWALE_NO_1-02 0	5.24	5.26	0.01	4.97	4.99	0.02	Varies between 5 to 5.5
	5.24	5.26	0.01	4.97	4.99	0.02	
	5.24	5.25	0.01	4.97	4.99	0.02	
	5.23	5.25	0.01	4.97	4.98	0.02	
	5.19	5.20	0.01	4.93	4.94	0.01	

Table 3.12: 100-year and 25-year water levels. All elevations are in feet NAVD 88.

Canal Name	Chainage	100-Year			25 -Year			Approx. Min. Elev.
		Current Conditions	With Project	Diff	Current Conditions	With Project	Diff	
BelleMeade-8	0	4.45	4.50	0.05	4.20	4.24	0.05	4.80
	1263	4.45	4.50	0.05	4.19	4.24	0.05	5.20
	1969	4.45	4.50	0.05	4.19	4.24	0.05	5.50
	1969	4.45	4.50	0.05	4.19	4.24	0.05	5.01
	3032	4.44	4.49	0.05	4.18	4.23	0.05	5.20
	4567	4.42	4.48	0.06	4.14	4.20	0.06	5.00
	5592	4.39	4.46	0.07	4.08	4.16	0.08	5.03
	6765	4.30	4.41	0.11	3.91	4.05	0.14	3.40
US_41_OUTFALL	0	4.45	4.50	0.05	4.19	4.24	0.05	6.40

SWALE_NO_1-01	1587	4.45	4.50	0.05	4.19	4.24	0.05	6.00
	3509	4.45	4.50	0.05	4.19	4.24	0.05	5.20

Table 3.13: 100-year and 25-year water levels at Sabal Palm Rd. culverts. The approximate minimum elevations are shown for Sabal Palm Rd. in the vicinity of each culvert All elevations are in feet NAVD 88.

Branch Name	Chainage	100-Year			25 -Year			Approx. Min. Elev.
		Current Conditions	With Project	Diff.	Current Conditions	With CWI Project	Diff.	
SABAL_PALM_2	180	9.67	9.72	0.06	9.13	9.22	0.09	9.60
	2120	9.12	9.18	0.06	8.91	8.96	0.05	
SABAL_PALM_3	180	9.09	9.24	0.15	8.79	8.97	0.18	8.40
	2130	9.07	9.22	0.15	8.77	8.95	0.18	
SABAL_PALM_7	180	9.14	9.29	0.16	8.85	9.03	0.18	8.40
	2210	9.12	9.28	0.16	8.84	9.02	0.18	
SABAL_PALM_10	180	9.31	9.46	0.15	9.04	9.21	0.17	8.50
	2150	9.28	9.43	0.15	9.02	9.19	0.17	

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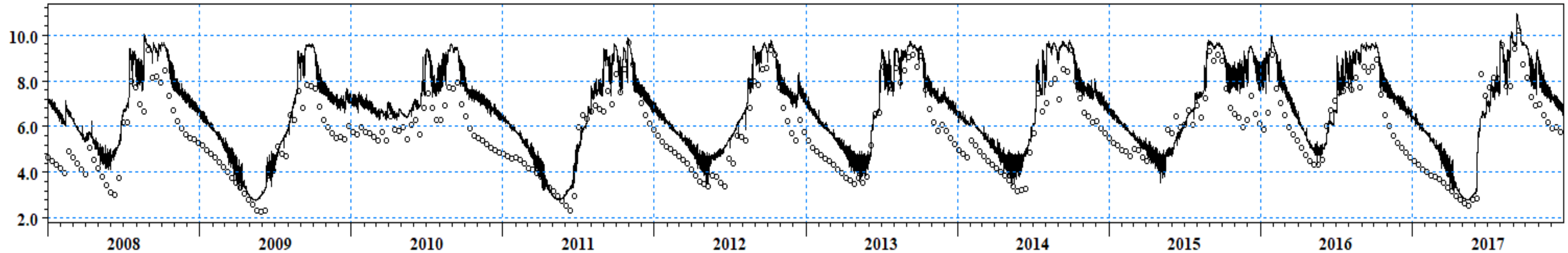
Appendix A-1

Calibration: Observed vs. Simulated Groundwater Levels

A-1.1 SGT1W1



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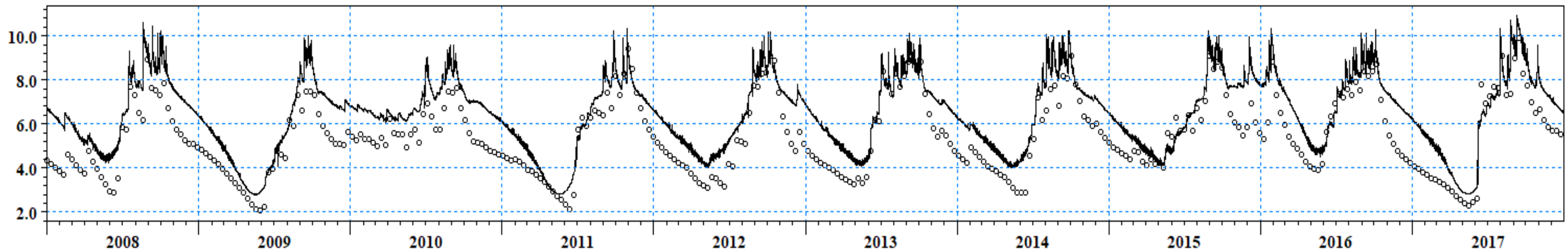

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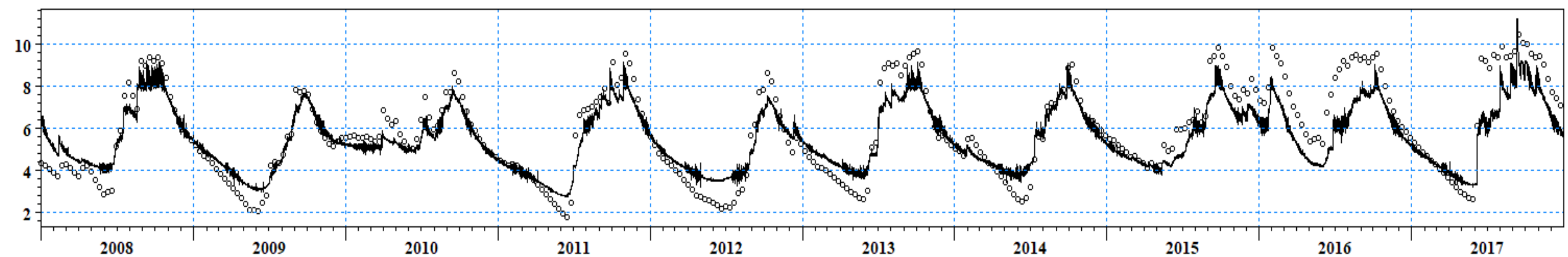

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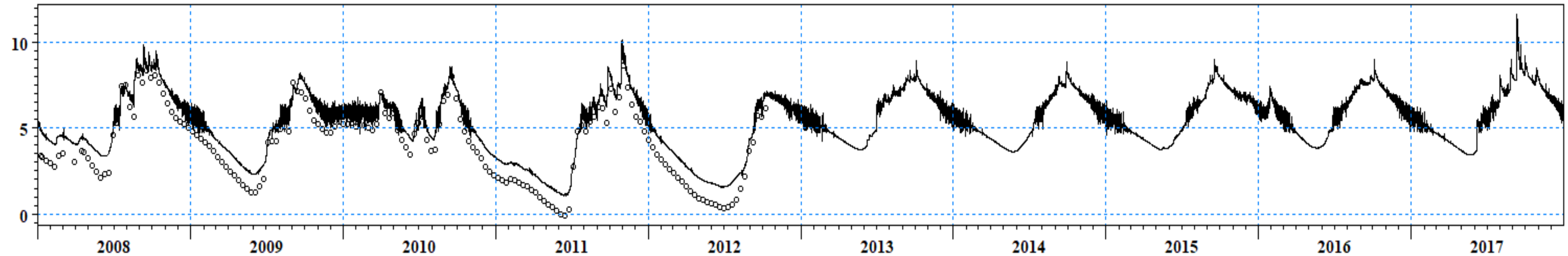

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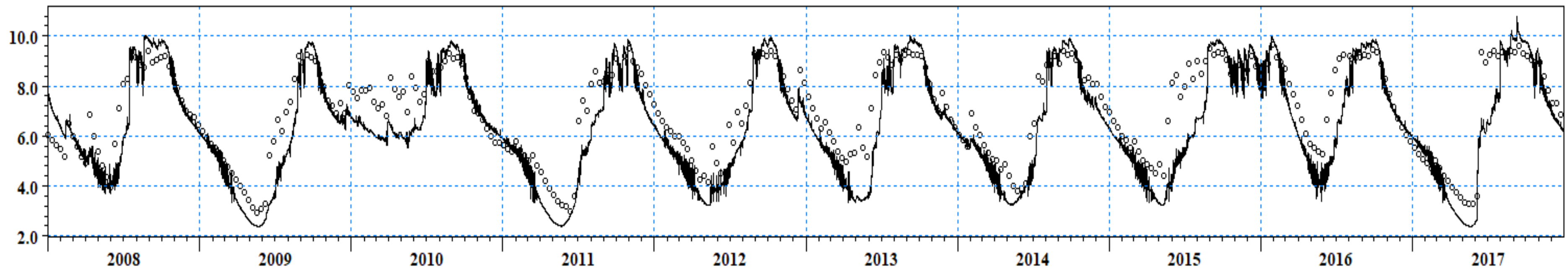

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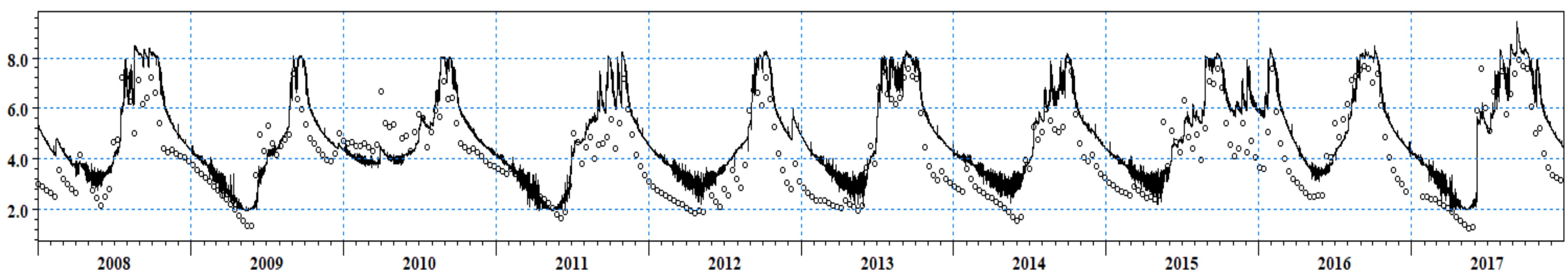

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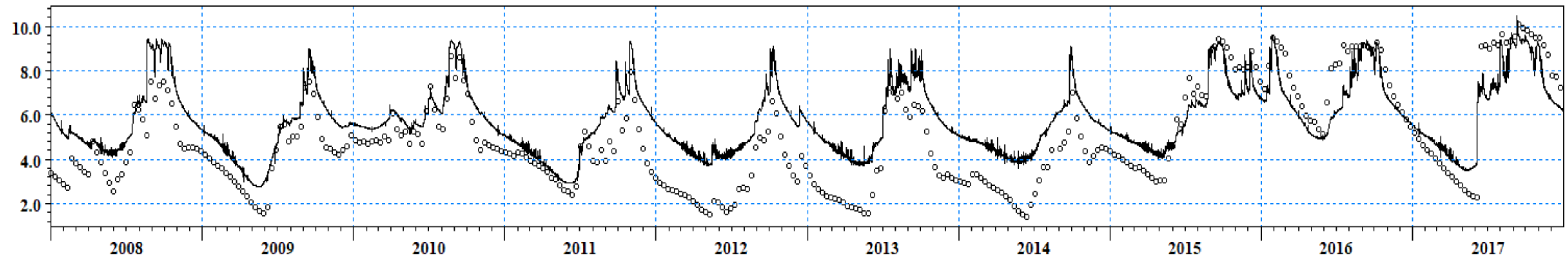

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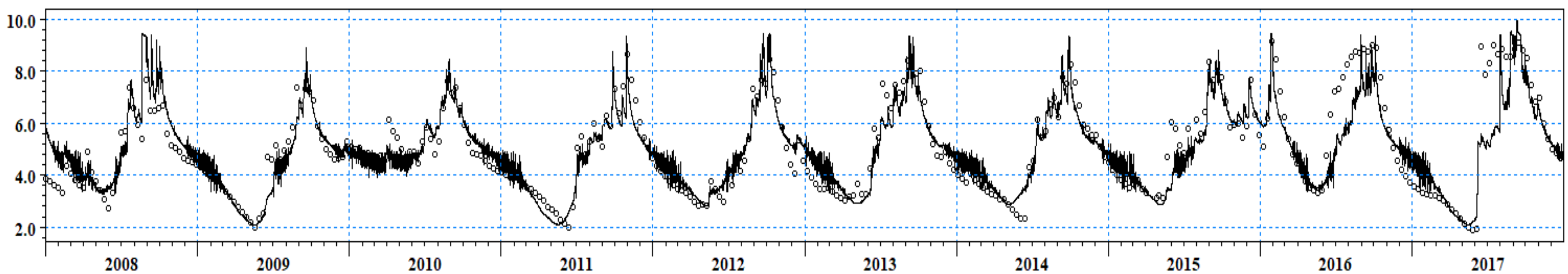

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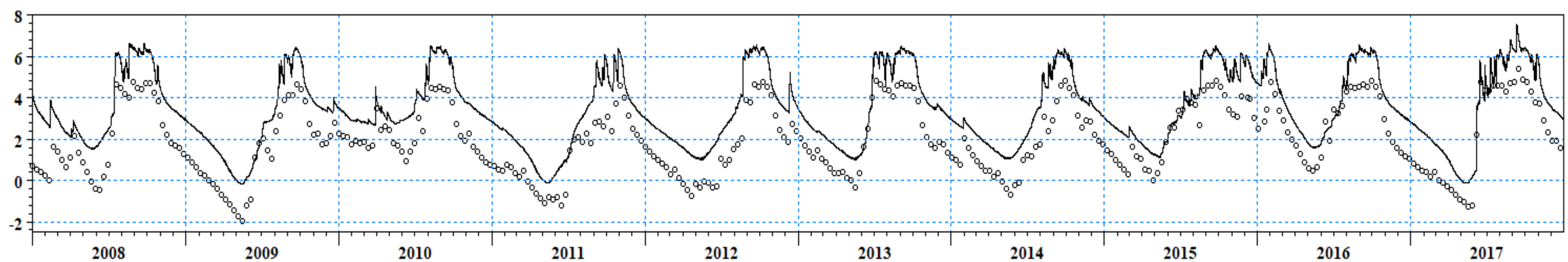

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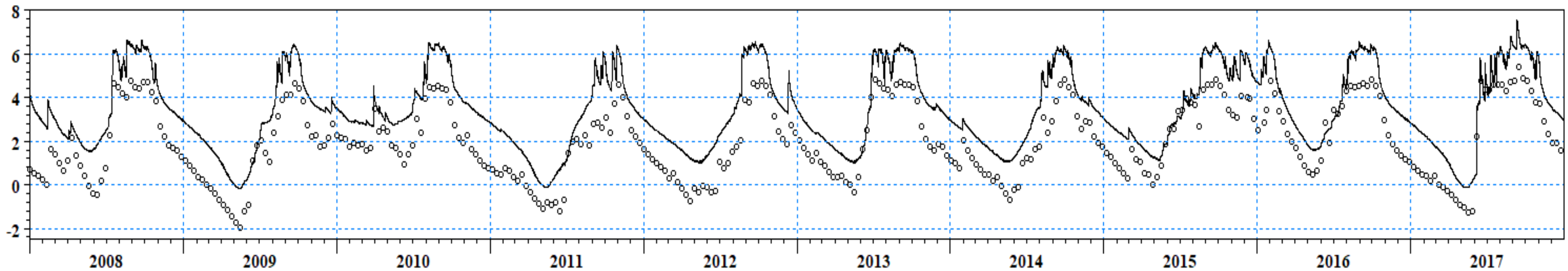

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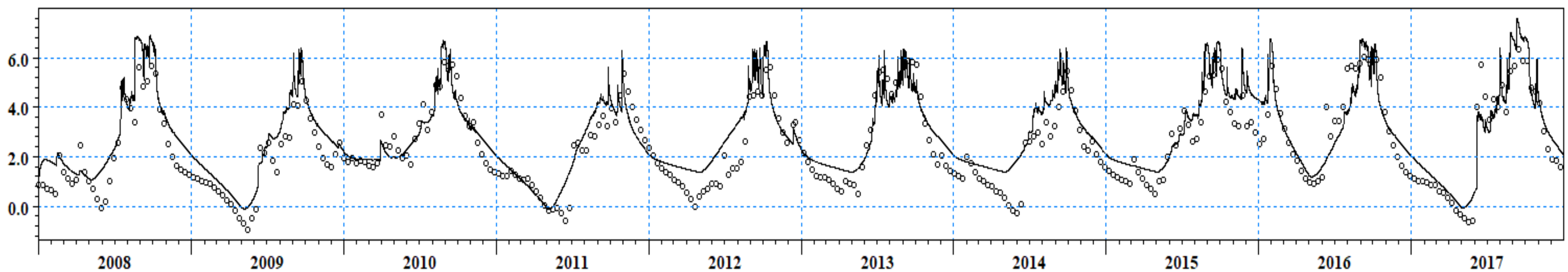

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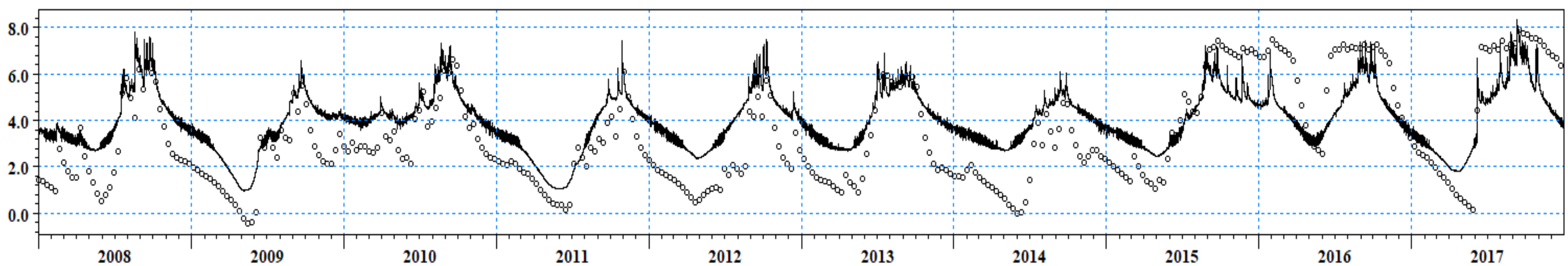

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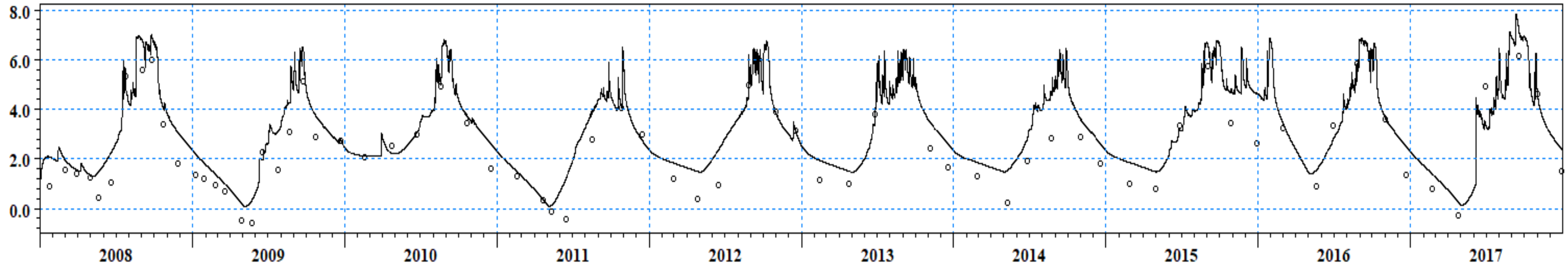

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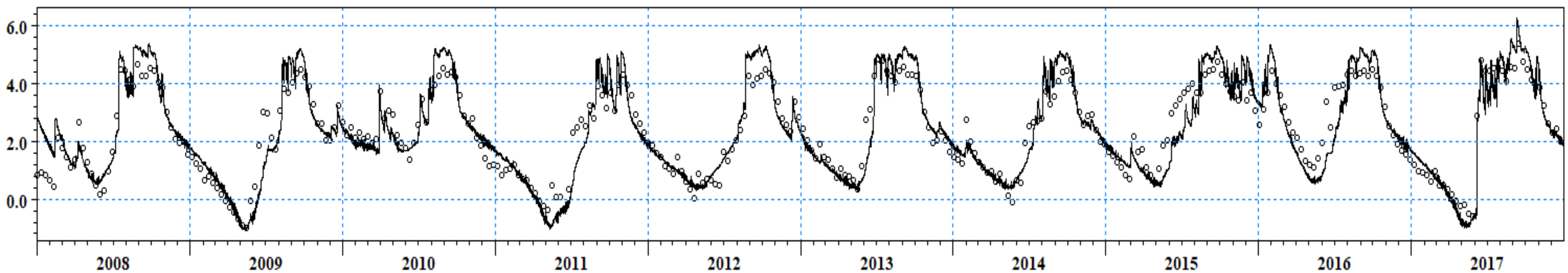

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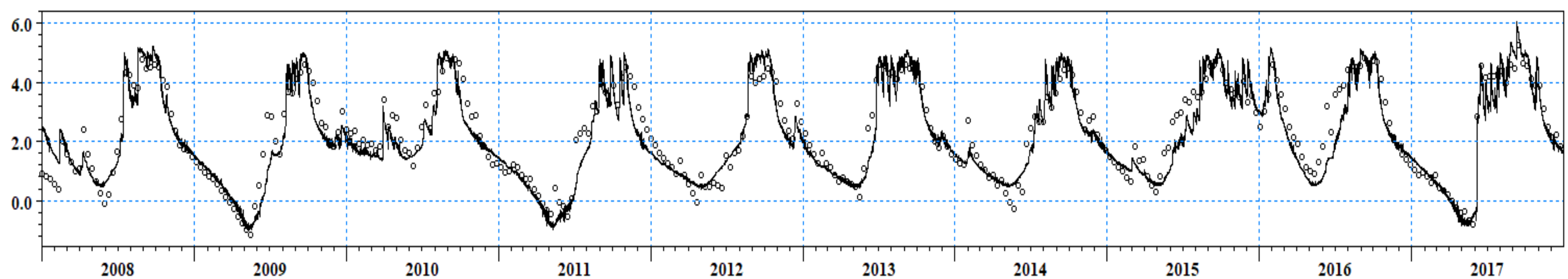

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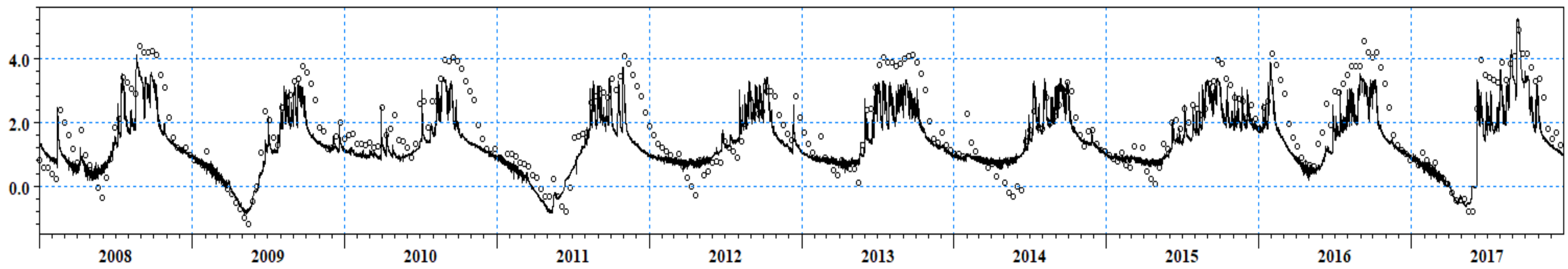

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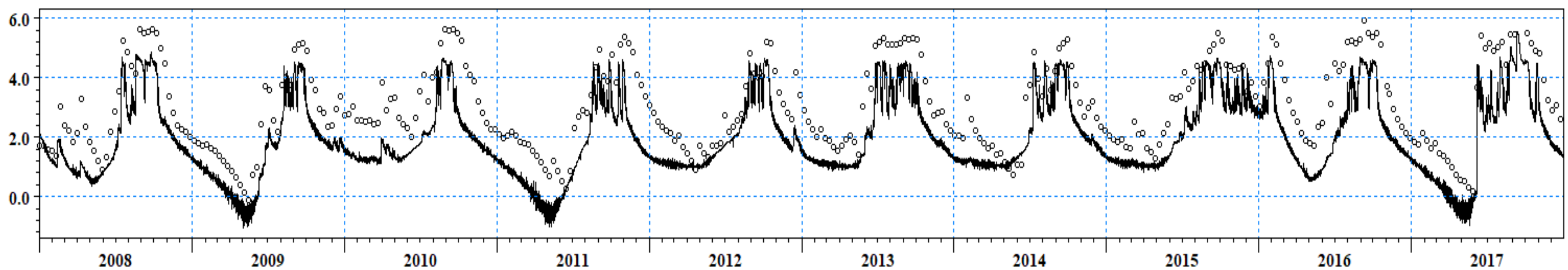

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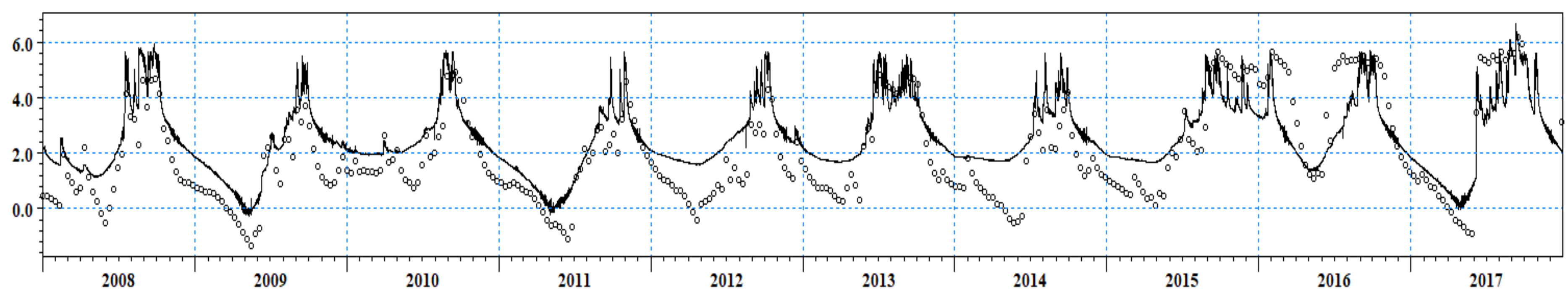

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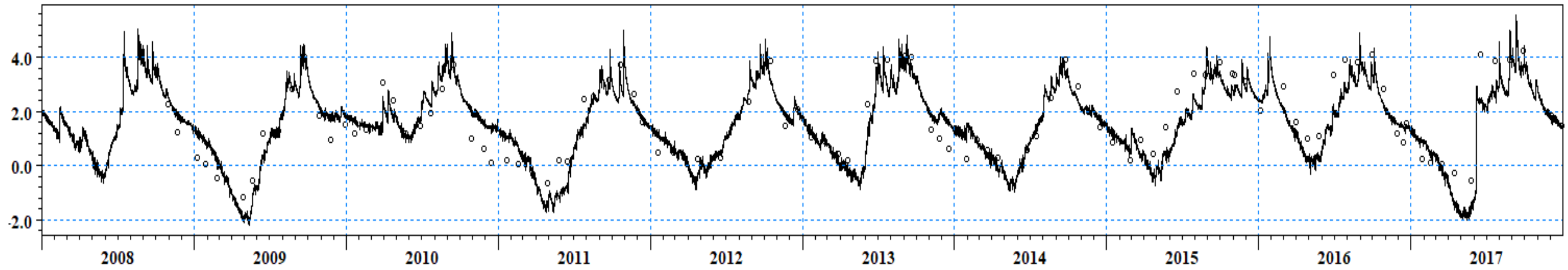

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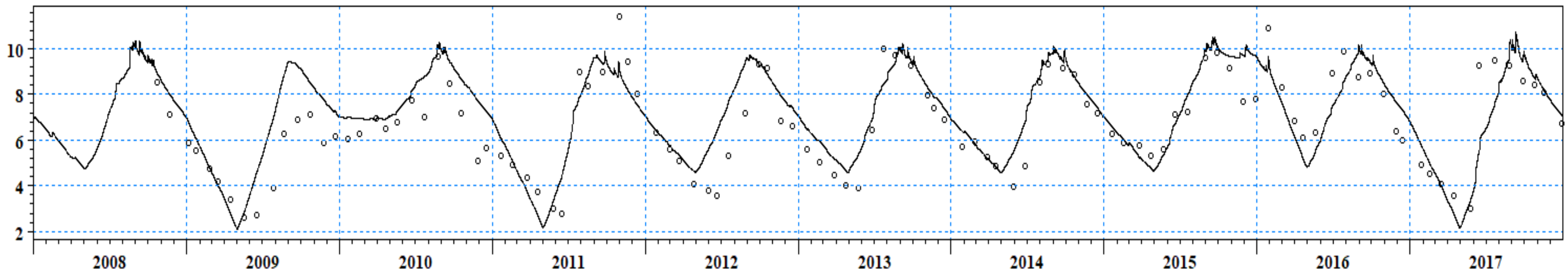

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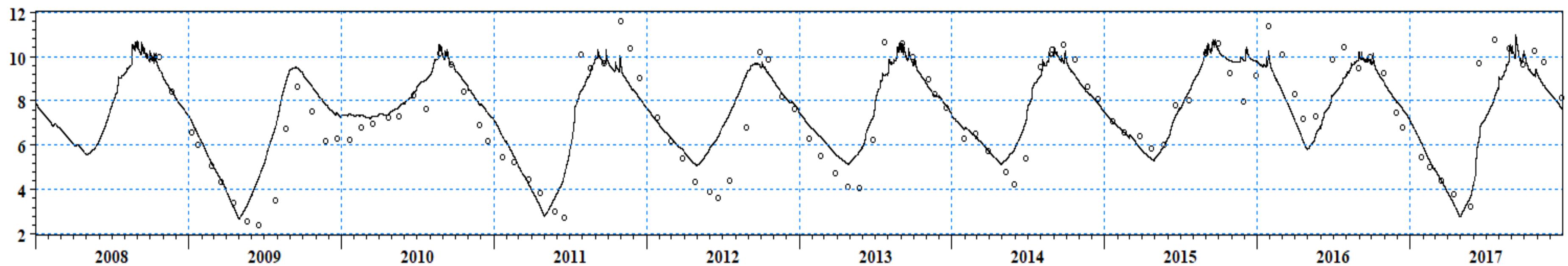

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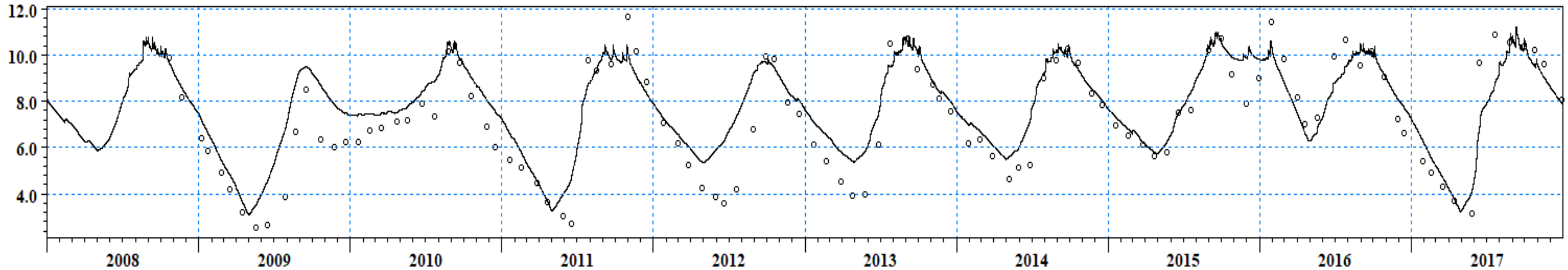

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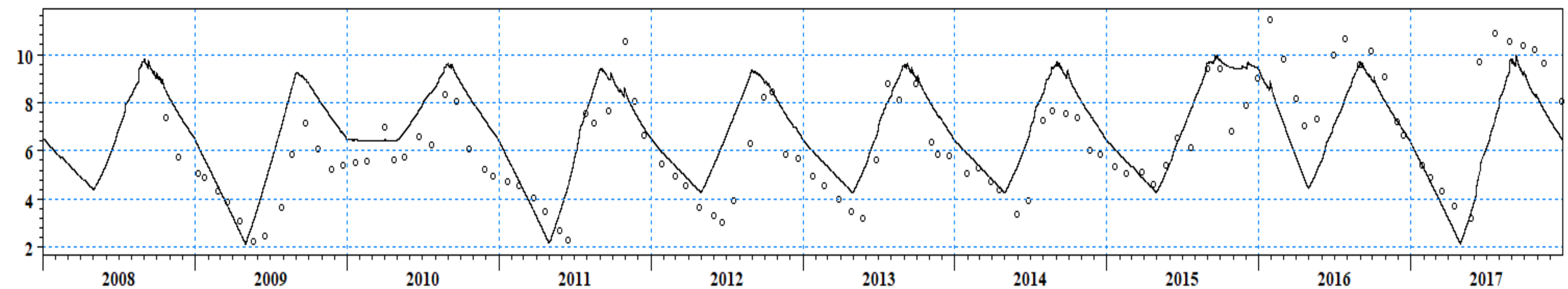

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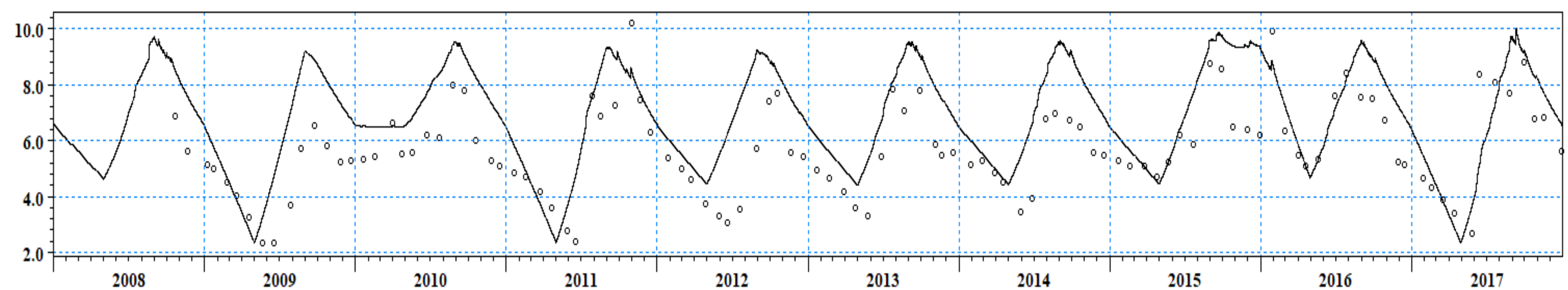

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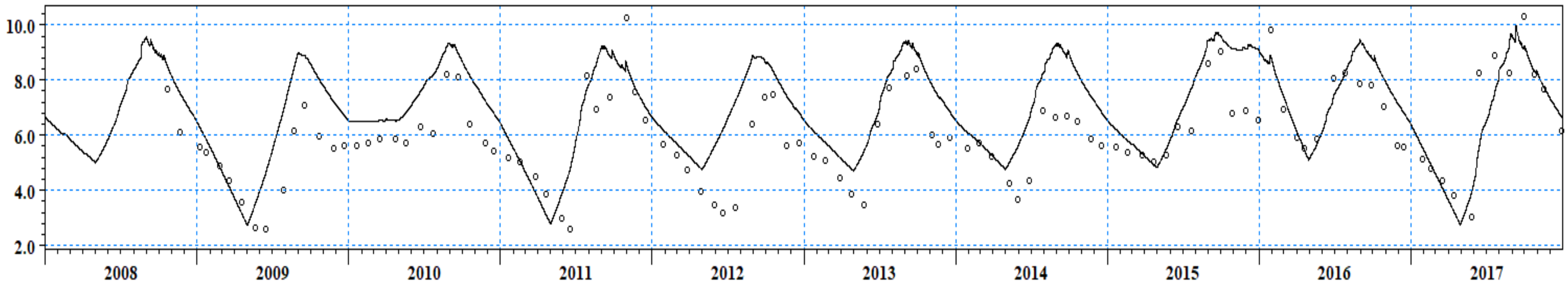

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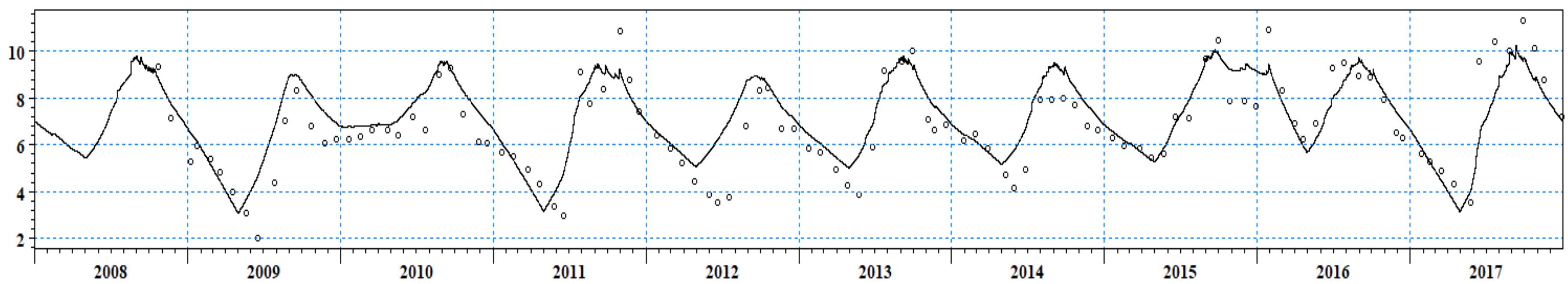

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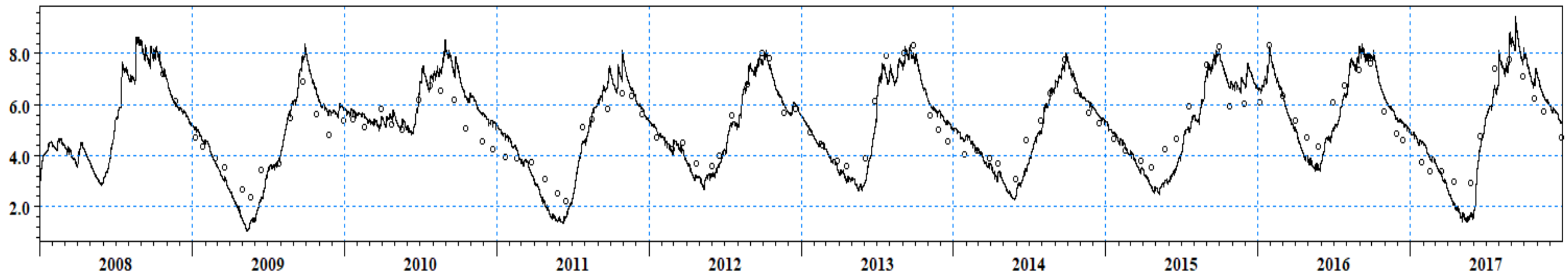

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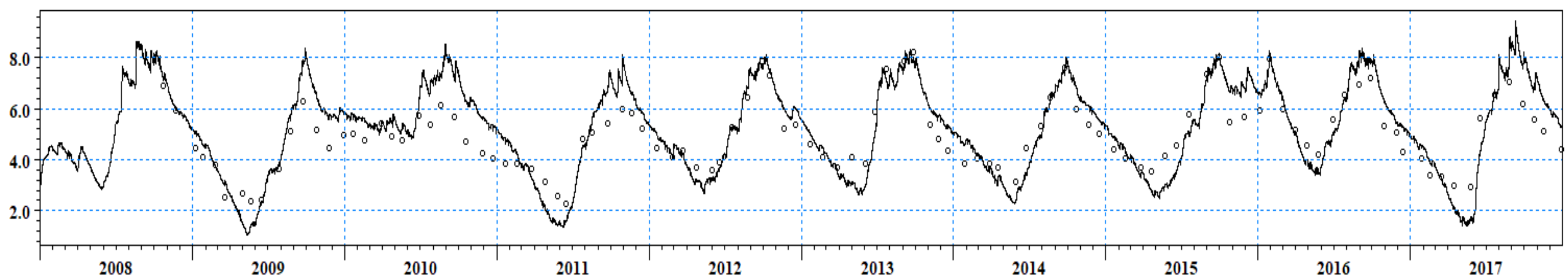

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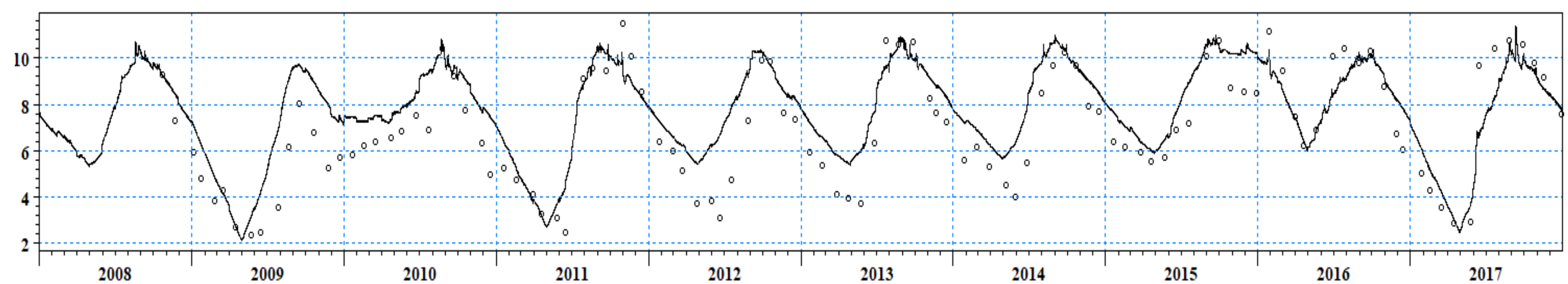

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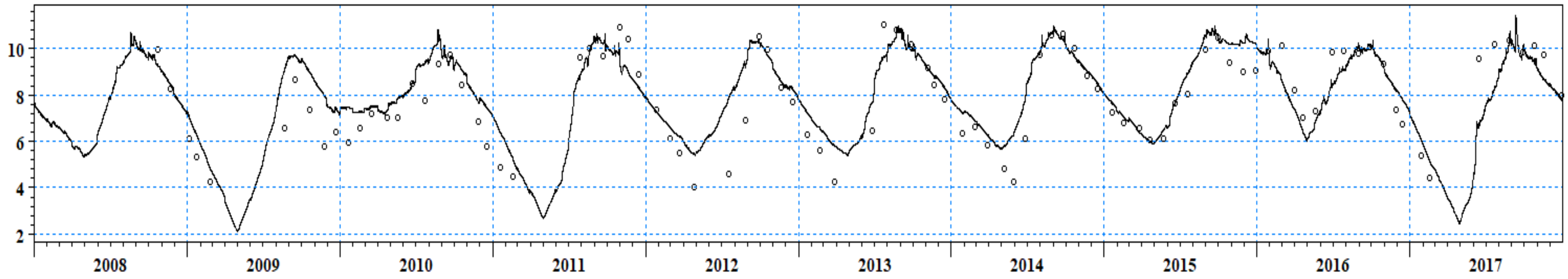
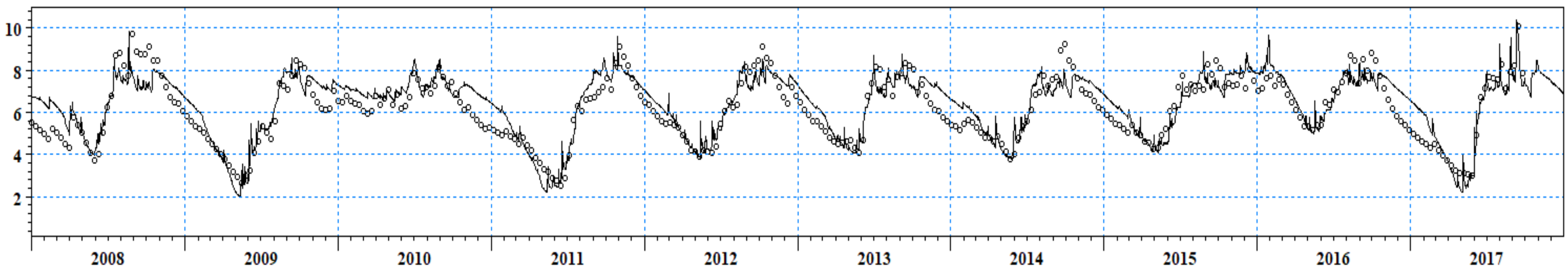


Appendix A-2

Calibration: Observed vs. Simulated Canal Water Levels

A-2.1 HEN84



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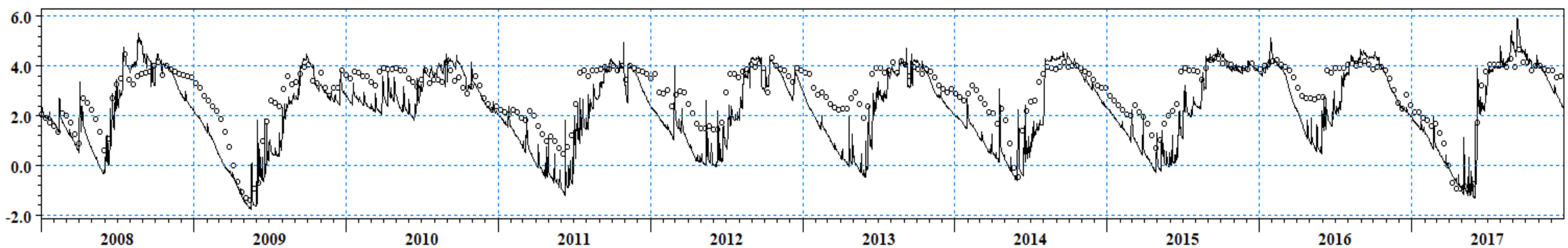
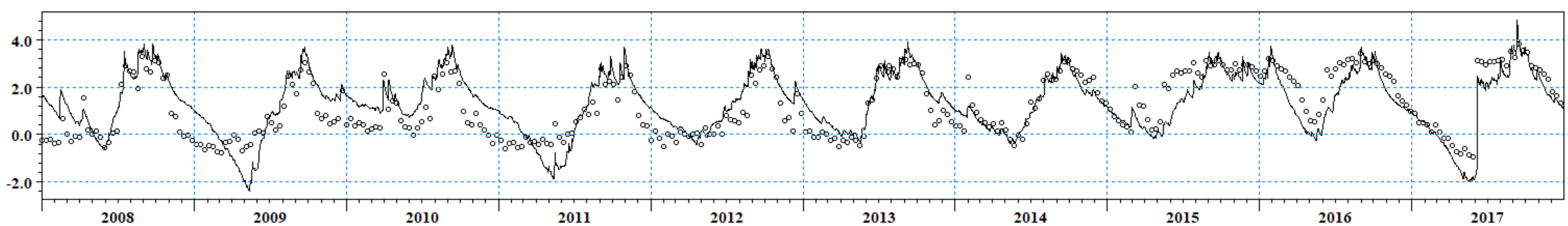
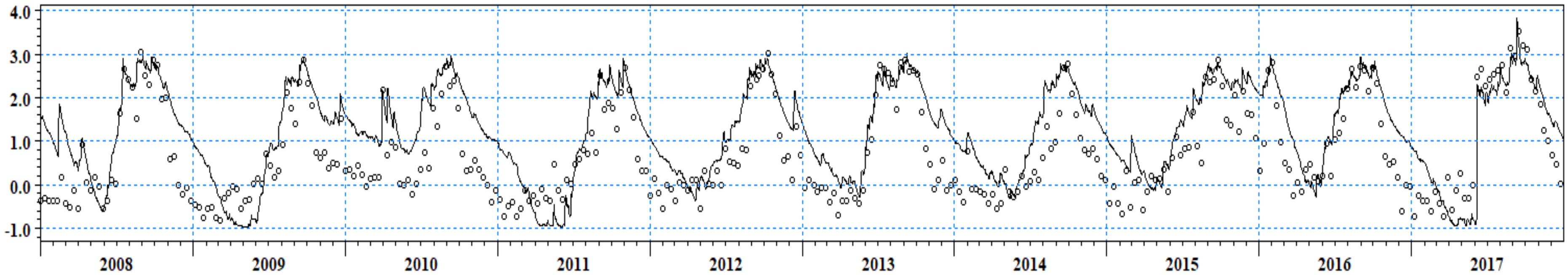


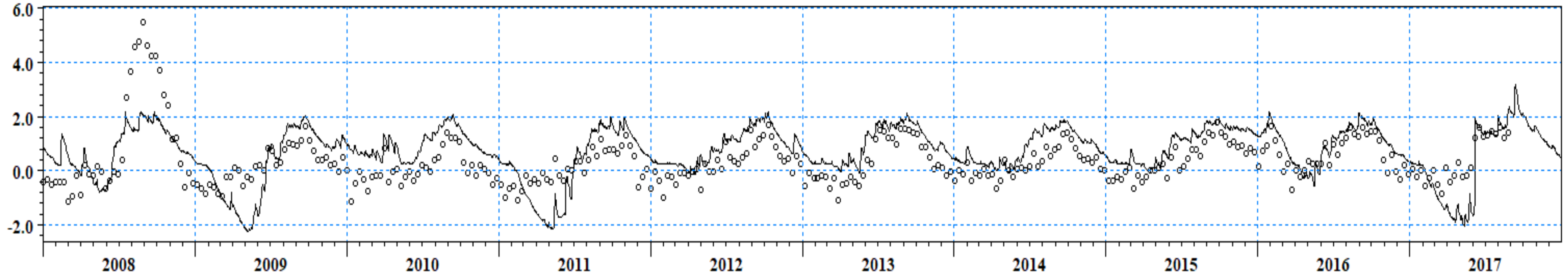
A-2.3 TAMIATOM



A-2.4 TMBR37



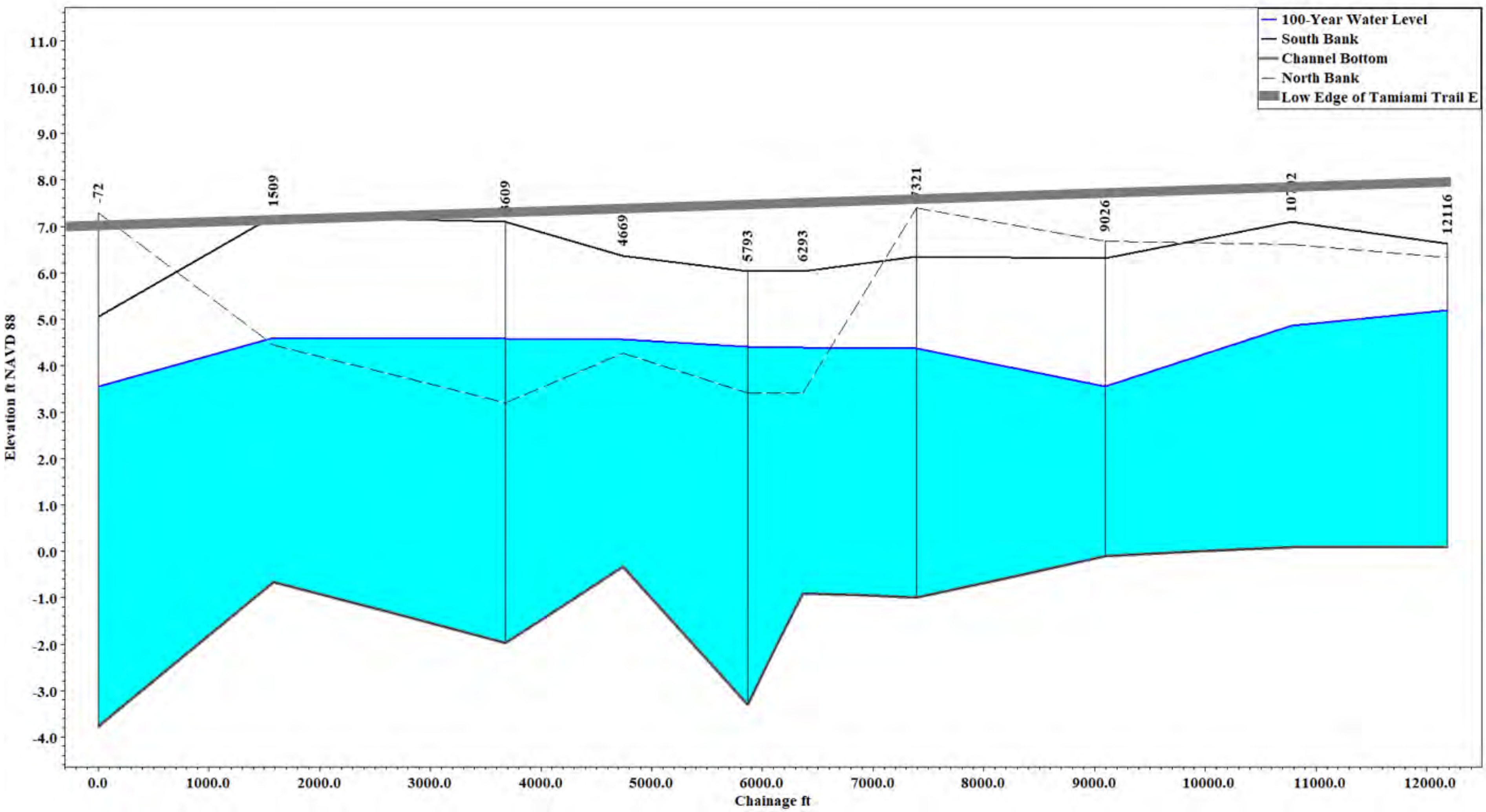
A-2.5 TMBR40



Appendix B

100-Year Maximum Water Level Profiles in Major Canals

B-1 U.S. 41



Appendix C

Letters of Concurrence

FDOT

Florida Department of Transportation

RON DESANTIS
GOVERNOR

801 North Broadway Avenue
Bartow, FL 33830-1249

KEVIN J. THIBAULT, P.E.
SECRETARY

January 31, 2020

Gary McAlpin
Collier County Coastal Zone Management
2685 North Horseshoe Drive, Unit 103
Naples, Florida 34101

Re: Collier County Watershed Improvement Project

Dear Mr. McAlpin:

I have reviewed the conceptual plan for the Collier County Watershed Improvement Project prepared by Taylor Engineering, Inc., December 2019. The Florida Department of Transportation (FDOT) does not believe the proposed conceptual plan will create an adverse impact upon FDOT's systems. The project will be required to submit for review via a Drainage Connection Permit to obtain final approval.

If you have any questions regarding this correspondence, please feel free to contact me at:

Brent Setchell, P.E.
District Drainage Design Engineer
Florida Department of Transportation
801 N. Broadway Avenue
Bartow, Florida 33830
863-519-2557
Brent.Setchell@dot.state.fl.us

Sincerely,

Brent Setchell Digitally signed by: Brent Setchell Date: 2020.01.31 15:10:40 -05'00'

Brent Setchell, P.E.
District Drainage Design Engineer

www.fdot.gov