City of Cape Coral Stormwater Model
Draft Final Report

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AIM Engineering & Surveying, Inc.

A·D·A ENGINEERING

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Table of Contents

1. Project Description ... 4
2. Model Development ... 4
   2.1. Surface Water ... 5
      2.1.1. Canal Network ... 5
      2.1.2. Boundary Conditions ... 6
      2.1.3. Cross Sections ... 6
      2.1.4. Resistance Values ... 8
      2.1.5. Structures ... 9
      2.1.6. Connection to Groundwater ... 10
      2.1.7. Topography ... 10
      2.1.8. Landuse/Vegetation ... 10
      2.1.9. Rainfall and Evapotranspiration ... 11
      2.1.10. Irrigation ... 14
      2.1.11. Manning's Number ... 16
      2.1.12. Separated Flow Areas ... 17
   2.2. Subsurface Properties ... 17
      2.2.1. Soil Properties ... 17
      2.2.2. Saturated Zone Layers ... 18
3. Water Balance ... 18
   3.1. Southern Basins ... 20
4. Calibration and Validation ... 23
   4.1. Calibration Issues ... 25
5. Design Storms ... 26
   5.1. Operations ... 28
   5.2. Results ... 28
      5.2.1. Primary Flooding ... 29
      5.2.2. Secondary Flooding ... 30
6. Scenario Development ... 32
   6.1. Structure Changes Scenarios ... 33
      6.1.1. Weir 1 ... 33
      6.1.2. Weir 20 ... 34

6.2. Water Quality Storage 36
6.3. Secondary Flooding Concerns 39
6.4. Future Build Out Scenario 39
6.5. Time to Peak Stage 41
7. Conclusions 46
8. Recommendations 47
8.1. Record Keeping Protocol 47
8.2. Continued Modeling Development and Analysis 47
9. References 48
10. Appendix A: Supporting Documents 49
11. Appendix B: Calibration and Validation Results 50
11.1. Surface Water Stations in Cape Coral 51
11.2. Groundwater Stations in Cape Coral 55
11.3. Pump Stations in Cape Coral 55
11.4. Surface Waters in the North Ft. Myers Region 56
11.5. Yucca Pens 57
11.6. Babcock/Webb Wildlife Management Area 59
12. Appendix C: Calibration and Validation Statistics 60

1. Project Description

The Cape Coral Stormwater Model is a comprehensive model developed to simulate the City of Cape Coral's canal system and all contributing flows from pipe networks and offsite flows. The model was based on an existing regional study developed by A.D.A. Engineering, Inc. However, all existing model parameters were revisited and adjusted as needed during the calibration phase of model development. **Figure 1** presents the model domain.

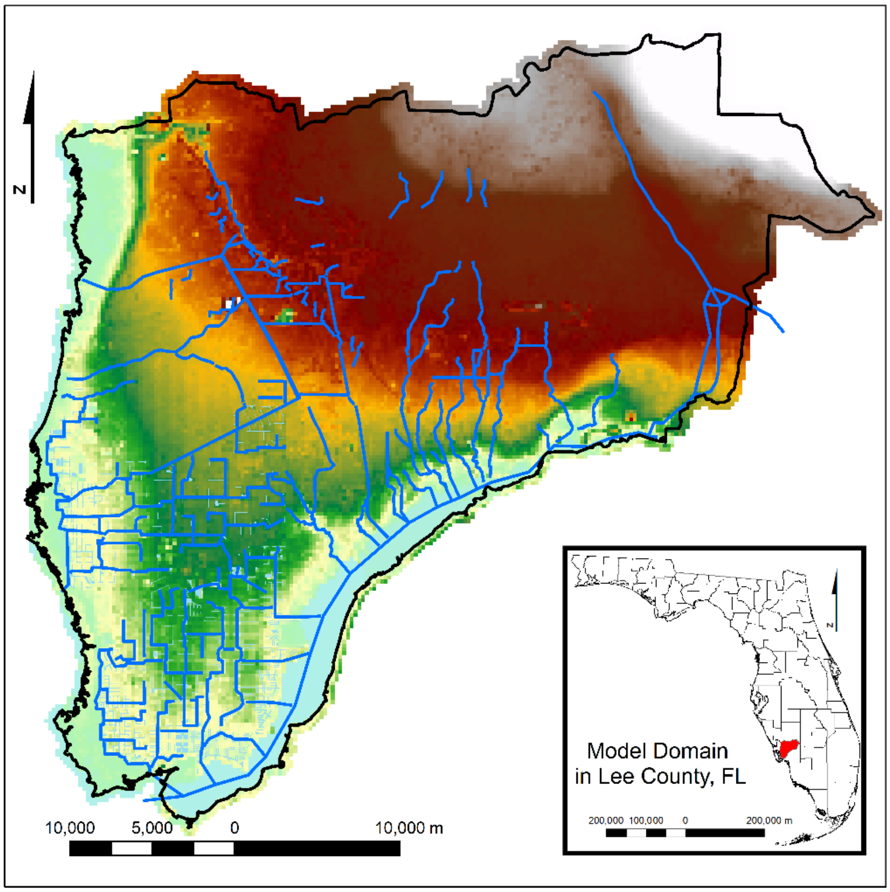

Figure 1. The model domain includes areas of Lee County and Charlotte County; including the city of Cape Coral, Yucca Pens Preserve, and the Cecil M. Webb Wildlife Management Area.

The model domain is located within Lee County and Charlotte County and includes the City of Cape Coral, Yucca Pens Preserve, and Fred C. Babcock/Cecil M. Webb Wildlife Management Area. Basins included in the domain are the Tidal Caloosahatchee River Basin, Upper Charlotte Harbor, and Lower Charlotte Harbor.

2. Model Development

The Cape Coral Stormwater Model was developed using MIKE SHE/MIKE 11, a set of commercially developed modeling tools for integrated surface water/groundwater hydrologic modeling. The model has a spatial resolution of 750 ft, and was calibrated using daily model data outputs for the 2012 simulation period. The model was then extended for validation for calendar year 2013.

2.1. Surface Water

A model of the existing surface water system was developed using the MIKE 11 modeling software, which analyzes 1-dimensional flow. The model was constructed using hydrologically-significant canals and rivers, weirs, gates, and culverts that control flows), tidal and headwater boundary conditions, canal flow resistance factors, and seepage rates between the surficial aquifer and the rivers.

2.1.1. Canal Network

The canal network was determined from the City of Cape Coral's canal shapefile and from USGS's stream delineation dataset. The networks were imported into the model from shapefiles and then modified as necessary; the final network is shown in blue in **Figure 2**. The major canals were included into the model as a part of the network, and minor canals were excluded. The volume of water for canals that were not included was accounted for using the Additional Storage tool within the cross section database, Overland Storage, or topography.

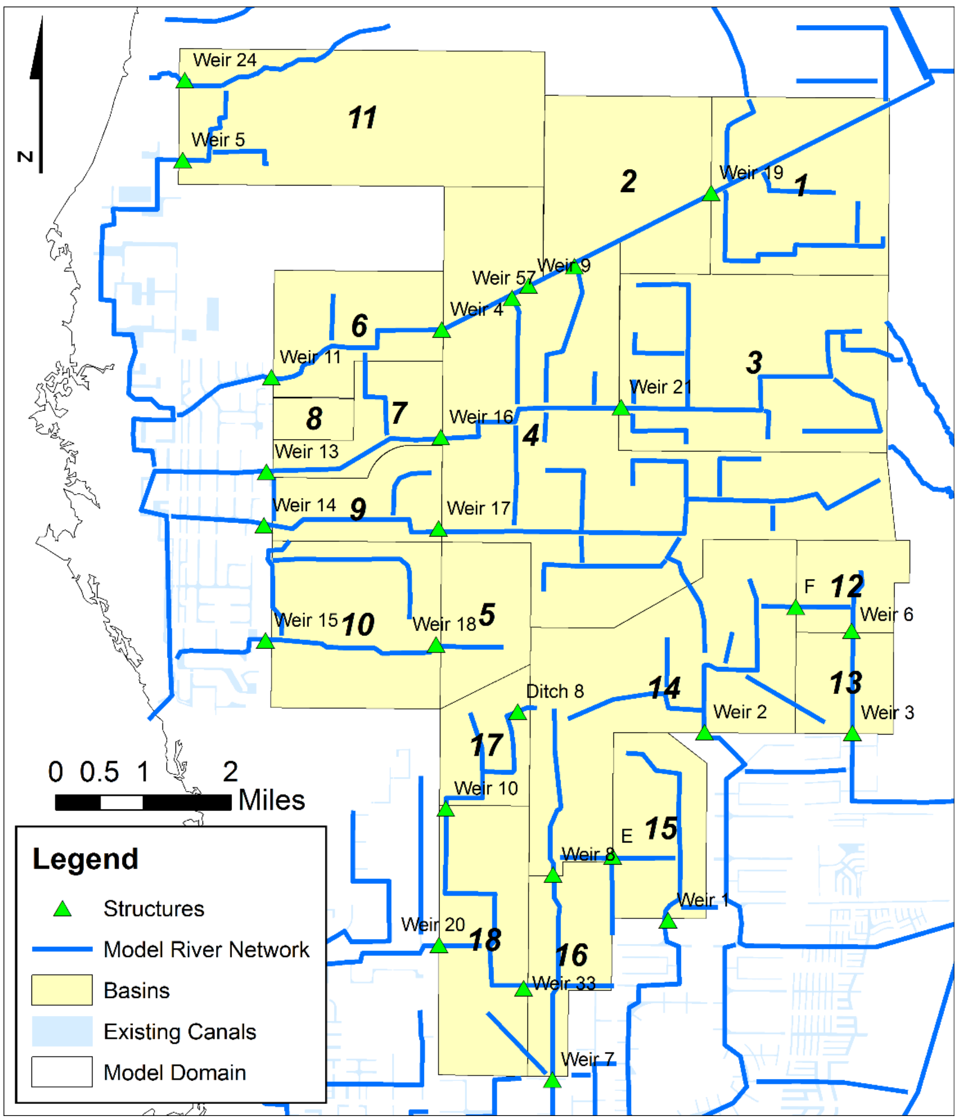

Figure 2. Canal network with locations of structures within the City of Cape Coral.

2.1.2. Boundary Conditions

Hourly tide elevations were acquired from the NOAA (National Oceanic and Atmospheric Administration) Ft. Myers station and applied as the Caloosahatchee River stage boundary condition. With the exception of short-term intense rainfall events, the tidal boundary drives most of stage elevations in the tidal/saltwater canals.

Hourly stage elevations were also applied to Durden Creek and Yucca Pens Creek as outfall boundary conditions. These monitoring stations were also influenced by the tides.

Monthly to bi-monthly stage elevations at Webb Lake were acquired from the Wildlife Management Area staff and input as a boundary condition.

2.1.3. Cross Sections

Cross section information was applied to the river network to represent canal volumes and gradients. Cross section data was obtained from surveys and soundings of the Cape Coral canal network. Gator Dredging performed a depth sounding survey of the majority of the southern Cape Coral canals. These data provided cross section estimates with varying degrees of accuracy, depending on the number of passes made by the sounding rig in any given canal. Where the rig made multiple passes through an area, the cross section information was able to show bottom depths, variation along the bottom, and smoothed bank slopes. The Gator Dredging data provided bottom elevations of the canals, and bank slopes were estimated from upstream or downstream sections. The Gator Dredging data is shown in green in **Figure 3**.

For the remaining northern canals where cross section information was missing, AIM Engineering & Surveying, Inc. surveyed the bottom of the canals at several critical locations chosen by the modelling team (shown in red in Figure 3). Survey locations were chosen to provide critical information upstream and downstream of control structures, along critical flow pathways, and in areas where too little information is known about the canals. AIM surveyed the complete cross section from bank to bank at intervals ranging from 2 ft to 20 ft, depending on the change in surface. **Figure 4** shows an example of one of the detailed cross sections completed by this survey.

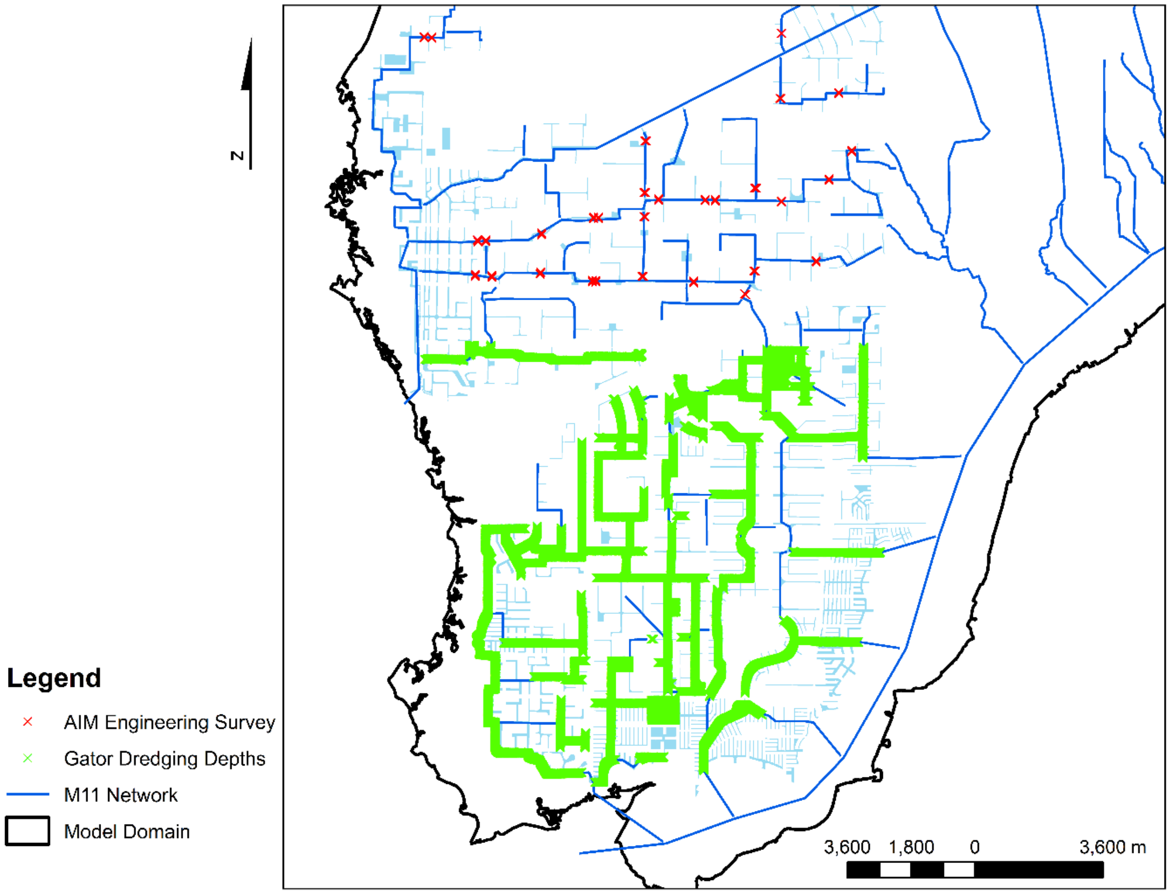

Figure 3. Data available for cross-section development.

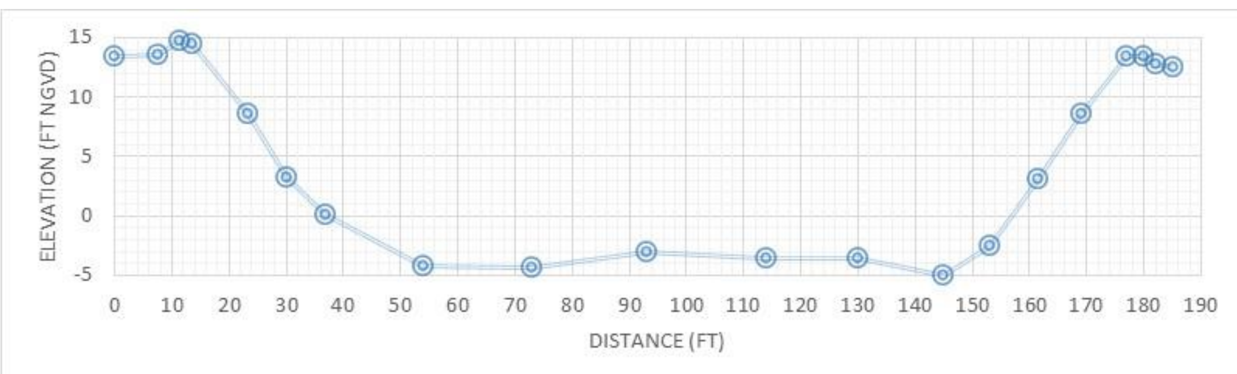

Figure 4. Example cross section data from AIM Engineering

2.1.3.1. Additional Storage

Only selected canals in Cape Coral were included in the model setup. The canals that are not represented in the MIKE 11 network do provide a significant amount of water storage which is accounted for in the following manner. The additional canal storage from these smaller canals was estimated using the following calculation procedure:

1) The total canal surface area at the top of bank was determined using the canal shapefile provided by the City of Cape Coral.
2) The top-of-bank (TOB) areas for all canals to be included in the connecting chainage for additional storage were summed. The total canal area was used as the maximum area for additional storage at TOB.
3) Processed cross-sectional information (water level cross-sectional area) was extracted from the cross section file.
4) Two new columns were created: % Change and Calculated Additional Area. The % Change is calculated by dividing the cross-sectional area at each water level with that of the water level above. This gives the percentage of area that the cross section is changing as water levels decrease. The Calculated Additional Storage for the top of bank was input and then for each decreasing water level, it is multiplied by the % Change. **Table 1** presents an example of these calculations.
5) The Calculated Additional Area was then copied into the Processed Data tab for the cross-section just downstream of the calculated additional canals.

Table 1. Example calculation table for additional storage

Water Level (ft-NGVD)	Cross Sectional Area (ft^2)	% Change	Calculated Additional Area (ft^2)
-2.34	0.00		0.00
-2.34	0.02	0.00	12.90
-1.57	38.76	0.00	23224.81
-0.80	79.08	0.49	47384.80
-0.03	121.00	0.65	72499.32
0.74	164.48	0.74	98555.48
1.52	209.55	0.78	125559.72
2.29	256.20	0.82	153512.05
3.06	304.44	0.84	182412.45
3.83	354.25	0.86	212260.94
4.60	405.64	0.87	243051.07
5.37	458.62	0.88	274795.72
6.14	513.18	0.89	307488.46
6.92	569.31	0.90	341122.83
7.69	627.03	0.91	375705.29
8.46	686.34	0.91	411242.28
9.23	747.22	0.92	447720.90
10.00	809.68	0.92	485147.60

2.1.4. Resistance Values

Resistance values were applied to the canal cross sections and were variable along certain zones. The use of variable resistance provides the modeler with the ability to include unmanaged flow zones, or natural wetlands, in the model where needed. The typical approach utilized in this model set-up was to utilize a higher bed resistance for the portions of the cross sections that are above the water line to reflect the friction caused by bank vegetation.

2.1.5. Structures

Structures which control the flow of water are considered significant and were included in the model. Structure type, dimensions, and operational schedule were obtained from 1) the City of Cape Coral WICC Program Master Plan, 2) design permits, 3) the City of Cape Coral Utilities Department records, and 4) AIM Engineering surveys. **Table 2** is a working spreadsheet provided by the Utilities Department, which was updated by ADA as new surveys were performed The table lists the important data points, such as sill elevation and length, as well as the type of structure and its data source. A map of these structures is found in **Figure 2**.

The calibrated model includes 166 weirs, 190 culverts, 8 bridges, and 28 control structures. Control structures are defined as all structures with operational schedules, such as pumps, gates, and inflatable weirs. Operational schedules were acquired from maintenance and gate reports.

Table 2. Structure information for Cape Coral weirs, gates, and culverts.

Structure #	Sill Elevation (ft NGVD)	Crest Length (ft)	Min Crest Elevation (ft NGDV)	Max Crest Elevation (ft NGVD)	Type of Structure	Data source
1	3.85	24.00	4.15	5.35	Obermeyer Weir	AIM
2	5.96	21.30	5.96	7.11	Weir with flashboards (Input as Fixed Weir)	CC
3	3.98	25.50	3.98	5.12	Weir with flashboards (Input as Fixed Weir)	AIM
4	6.42	250.50	6.42	6.42	Fixed Weir	CC
5	1.95	122.00	1.95	1.95	Fixed Weir	AIM
6	6.08	16.00	6.38	7.58	Obermeyer Weir	AIM
7	3.89	24.00	4.19	5.39	Obermeyer Weir	AIM
8	6.13	26.29	6.13	6.98	Fixed Weir	CC
8	2.59	3.00	2.59	1.20	Gated Culvert	CC
9	8.50	226.80	8.50	8.50	Fixed Weir	CC
10	5.97	24.25	5.97	5.97	Weir wall around a culvert	AIM
11	2.56	178.70	2.66	4.56	Obermeyer Weir	CC
13	2.47	101.20	2.57	3.89	Obermeyer Weir	CC
14	2.59	83.70	2.89	3.83	Obermeyer Weir	CC
15	2.35	98.02	2.45	3.55	Obermeyer Weir	AIM
15	0.00	3.00	0.00	0.00	Side culvert (Closed)	
16	6.33	112.50	6.33	7.32	Weir with flashboards	AIM
17	6.40	82.00	6.40	7.42	Weir with flashboards	AIM
18	4.99	39.00	4.99	4.99	Fixed Weir	AIM
19	10.40	30.00	10.40	12.00	Gated Weir	AIM/CC
19	6.00	8.00	4.80	6.00	Gated Culvert	AIM/CC
20	3.03	39.00	3.03	3.03	Fixed Weir	CC
20	-2.96	5.83	-2.96	2.87	4 Open Culverts	AIM
21	8.33	52.00	8.33	9.83	Obermeyer Weir	CC
24	1.97	10.00	2.00	2.00	Fixed Weir	CC
33	4.05	14.60	5.19	5.19	Weir with flashboards (Input as Fixed Weir)	AIM
57	7.20	3.92	7.20	7.20	Culvert with Fixed Weir	CC
57	7.20	3.92	7.20	7.20	Culvert with Fixed Weir	CC
57	4.31	0.40	4.31	4.31	Culvert with Flap Gate	CC
57	4.49	0.40	4.49	4.49	Culvert with Flap Gate	CC
58	7.69	10.00	7.69	8.25	Gated Weir (Input as Fixed Weir)	CC
58	10.50	50.00	10.50	10.50	Fixed Weir	CC
E	3.86	1.77	3.86	5.63	Gated Culvert	CC
F	2.84	4.00	7.99	7.99	Culvert	CC
Ditch 8	5.75	2.5	5.75	8.25	2 Open Culverts	AIM

There are eight (8) Obermeyer Weirs in Cape Coral that are maintained on regular basis (every 1 to 5 days). The bladders are refilled with pressure when the gates are to be kept open and the pressure in each bladder is released when the gates are to be lowered. An example of the Weir Maintenance reports is found in **Figure 5**. For each weir type, a specific bladder pressure relates to a specific weir height; so the reported bladder pressure was copied and converted to weir height in feet NGVD to produce a gate operation time series for each weir. This data greatly improved the calibration to observed stations.

Figure 5. Example of the Weir Maintenance reporting spreadsheet which was digitized and converted from bladder PSI to gate

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WEIR MAINTENANCE

WEIR	TANK BEFORE	TANK AFTER	BLADDER PSI				BLADDER PSI AFTER			
7	10	110	9	9	9		10	10	10	
Relief Test	Regulator									
1	85	120	10	10	10		10	10	10	
Relief Test	Regulator									
6	80	95	11	11			11	11		
Relief Test	Regulator									
21	15	110	11	11	11	11	11	11	11	11
Relief Test	Regulator									
11	80	110	12				12			
Relief Test	Regulator									
13	10	110	7				11			
Relief Test	Regulator									
14	5	115	2				11			
Relief Test	Regulator									
15	10	115	6.5				10			
Relief Test	Regulator									

elevation in feet.

2.1.6. Connection to Groundwater

All canals were connected to the groundwater system via canal bed leakance. Depending on the type of canal, either a canal bed leakance factor or the hydraulic conductivity of the contacting surficial aquifer controlled the rate of exchange between the canal and the aquifer. This was a calibration parameter.

2.1.7. Topography

Land surface elevations were acquired from a composite LiDAR topography dataset comprised of Lee/Charlotte 2007 LiDAR from FDEM, the Peace River South 2005 LiDAR from SWFWMD, and the Eastern Charlotte 2011 LiDAR from USGS. The land surface elevations were then resampled from a 10 ft resolution to the model grid size of 750 ft using the average elevations. The final topography used in the model is shown in **Figure 1**.

2.1.8. Landuse/Vegetation

As shown in **Figure 6**, landuse within the City of Cape Coral is mostly classified as either Urban-Low Density, Urban-Medium Density, or Urban-High Density. North of the city and into Yucca Pens, the classification is mostly Mesic Flatwoods or Marsh. Each landuse type has specified vegetation properties which determine how much water the vegetation will use during transpiration on a seasonal basis. These properties include Leaf Area Index (LAI), Root Depth (RD), and the crop coefficient (K_c).

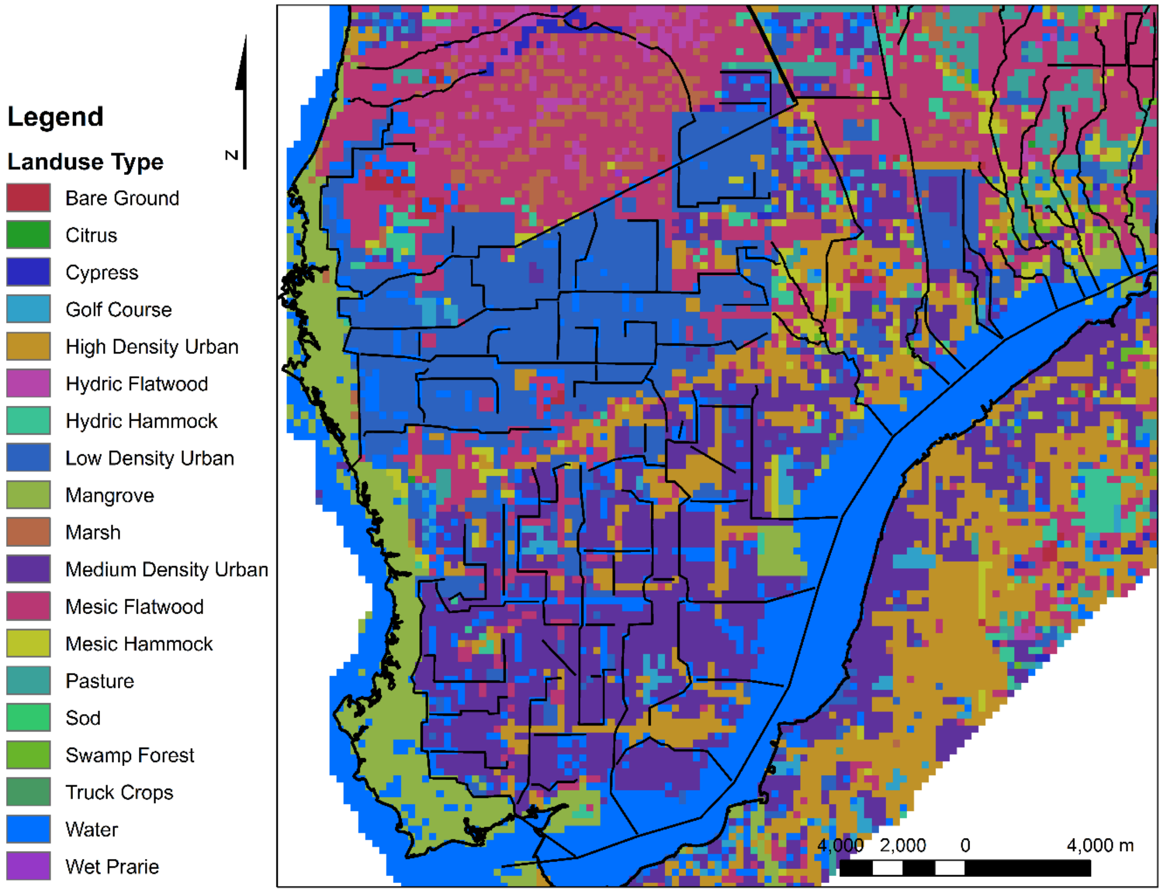


Figure 6. Landuse classifications for the City of Cape Coral.

2.1.9. Rainfall and Evapotranspiration

SFWMD provides 15-minute, hourly, or daily rainfall data for each 2km x 2km grid cell defined by the District. This rainfall dataset comes from NEXRAD data that is converted to rainfall intensities by the National Weather Service and then calibrated or adjusted locally using rainfall gauge data algorithms. This dataset is valuable as it is a balance between the more precise point data provided by rain gauges and the spatial distribution of the storm front provided by the NEXRAD sensors (Pathak and Pandey, 2008).

The final product was acquired by the modeling team from the District (SFWMD contact: Ruben Arteaga rarteaga@sfwmd.gov) for the calibration and validation period. The District provided their gridding system (shapefile) and hourly rainfall time series values for each grid number. This information was then combined to provide a temporal-spatial rainfall dataset for the model domain for the calibration and validation period.

Daily evapotranspiration (ET) data was downloaded from the Florida Water Science Center website: http://fl.water.usgs.gov/et. This dataset uses solar radiation from the Geostationary Operational Environmental Satellite (GOES) to determine potential ET (PET) and reference ET (RET) over a 2km x 2km grid (same as the rainfall data).

Leaf Area Index (LAI), Root Depth (RD), and Crop Coefficient (K_c) are vegetation properties that are assigned to a specific landuse type, as determined from the USGS land cover database. These properties modify the amount of water that is pulled from the root zone by the vegetation for evapotranspiration.

2.1.9.1. Rainfall Modification

Using several Lee County rain gage stations within the model domain, the reliability of the NEXRAD dataset was examined. Through a comparison of cumulative rainfalls for the NEXRAD and Lee County stations, rainfall in the area near Yellow Fever Creek was being over-predicted for the NEXRAD dataset as compared with measured values from the Lee County gages. For the 2-year simulation, the total rainfall was higher in the NEXRAD dataset by over 32 inches for the Yellow Fever Creek station.

Figure 7 shows the spatial distribution of the NEXRAD rainfall with contours and the location of Lee County gages with the measured cumulative rainfall in parenthesis.

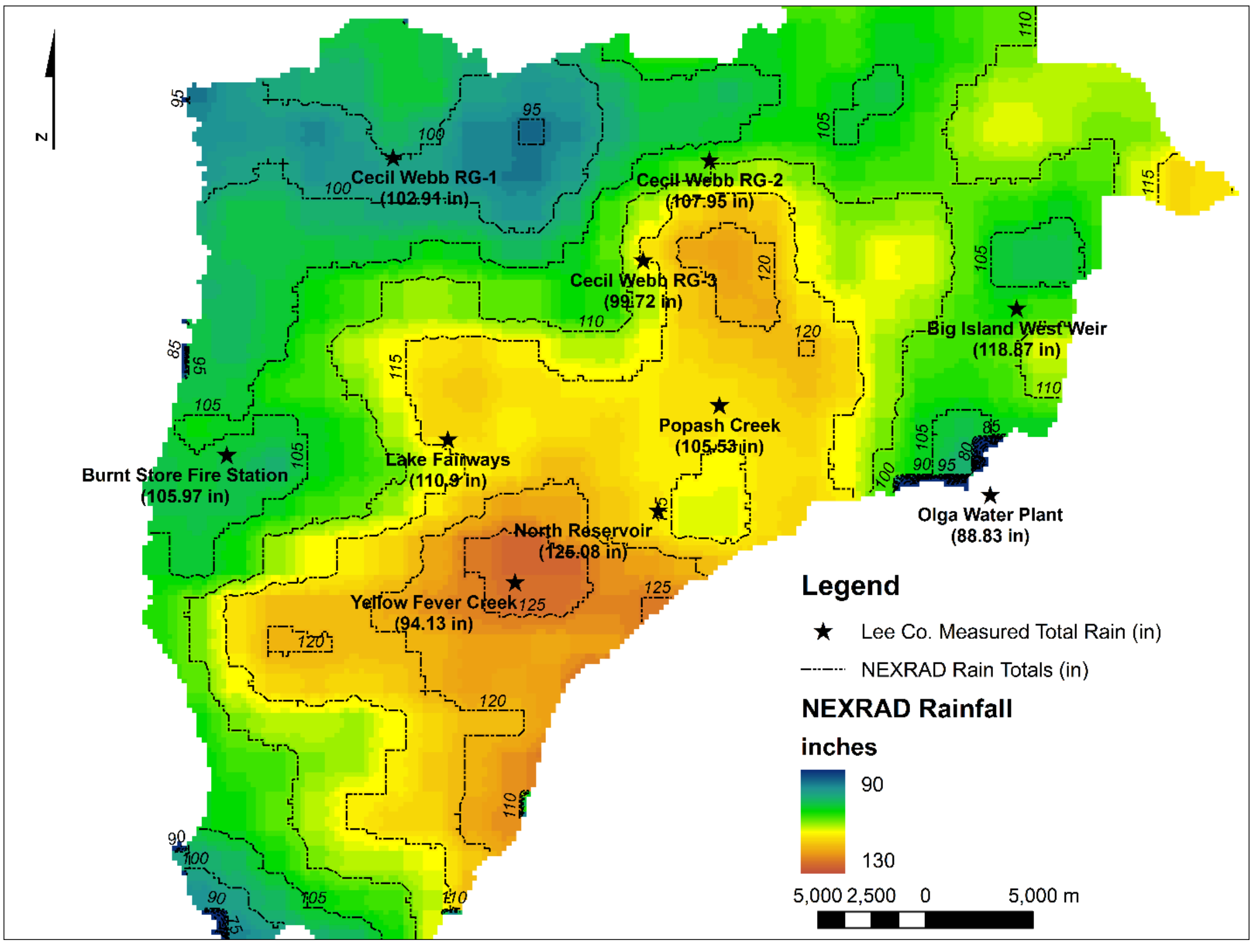


Figure 7. Spatial distribution of the cumulative NEXRAD and locations of the Lee County gages with cumulative measured rainfall in parenthesis.

By multiplying the NEXRAD daily rainfall around the Yellow Fever Creek area by 0.75, the NEXRAD and Lee County data matches up well as shown in **Figure 8**.

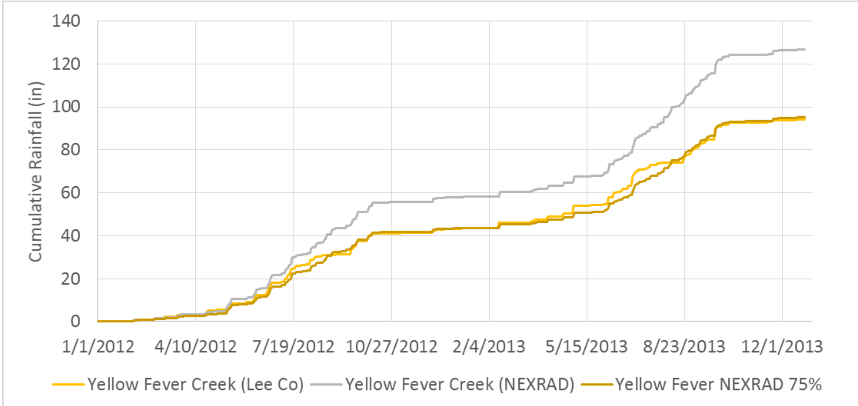


Figure 8. Cumulative rainfall estimates for Yellow Fever Creek using NEXRAD (grey), Lee County gages (yellow), and a %75 modified NEXRAD (dark yellow).

2.1.9.2. ET Modification Test

A test was performed during model calibration in an attempt to increase water in the Canal Basins south of Pine Island Road and the North-South Transfer Station. Vegetation parameters were modified to decrease ET in the urban areas. Leaf Area Index (LAI) and the crop coefficient, K_c, were modified as shown in **Table 3** for three urban vegetation types. The modifications were made to represent what is physically occurring in urban areas, which is a reduction of transpiration due to less plant coverage.

Table 3. Modified LAI and K_c values for urban vegetation types

	Original LAI	**Altered LAI**	Original K_c	**Altered K_c**
Urban-Low Density	2.5	**1**	1	**0.9**
Urban-Medium Density	2	**0.5**	1	**0.75**
Urban-High Density	2	**0.1**	1	**0.5**

LAI is (Area of Leaves)/(Area of the ground) and can vary between 0 and 7 depending on the vegetation type. A K_c value of 1 means that the maximum evapotranspiration rate will equal the reference evapotranspiration rate. It is appropriate to lower this value for areas where the field coverage is less than the FAO-defined standard of 3 - 6 in high grass. The changes made to LAI caused a 27% reduction in ET for the Southern Basins, as shown in **Table 4**. By reducing both LAI and the crop coefficient, ET in the Southern Basins was reduced by 34%. With the ET reduced, additional rainfall runoff can infiltrate and contribute to lateral inflow from the saturated zone to canals.

Table 4. Results from altering the vegetation parameters which affect ET.

	Original	**Altered LAI Scenario**		**Altered Kc Scenario**	
	ET (in)	ET (in)	% Reduction in ET	ET (in)	% Reduction in ET
Wet	24	17	29%	15	38%
Dry	17	13	24%	12	29%
Year Round	41	30	27%	27	34%

2.1.10. Irrigation

When irrigation is specified in MIKE SHE, the model uses vegetation properties such as root depth and soil properties such as soil moisture content to determine how much water the vegetation requires in order to survive. The amount of water that is required by vegetation is limited by the water table height and rainfall, the typical sources for plant uptake. However, if an area is known to receive irrigation, an additional source of water can be specified from a MIKE11 river, a distributed shallow well supply, a single well, or an external source (typically a groundwater supply below the modeled surficial aquifer). The irrigation application type can be specified as either a sprinkler, drip, or sheet flow. The model uses the demand of the vegetation to determine irrigation; therefore, it will first use rainfall and any soil moisture to feed plant uptake, and then irrigation will be used when the region is dry.

Irrigation Command Areas are defined in the model to account for regions which are receiving irrigation in addition to natural rainfall events. These were determined from water use permitting reports and city records. Irrigation in the low density suburban areas north of Pine Island Rd (ICA number 108 in **Figure 9**) was estimated based on City irrigation limitations imposed on individual homeowners. The grid cells of ICA 108 in Figure 9 were set at approximately 25% of the north Cape Coral area based on an estimate that the north Cape Coral area was approximately 25% developed in the 2011-2013 period. The irrigation control area south of Pine Island Rd (ICA number 112 in **Figure 9**) was determined from the reported Existing Service Area for the City of Cape Coral Utilities.

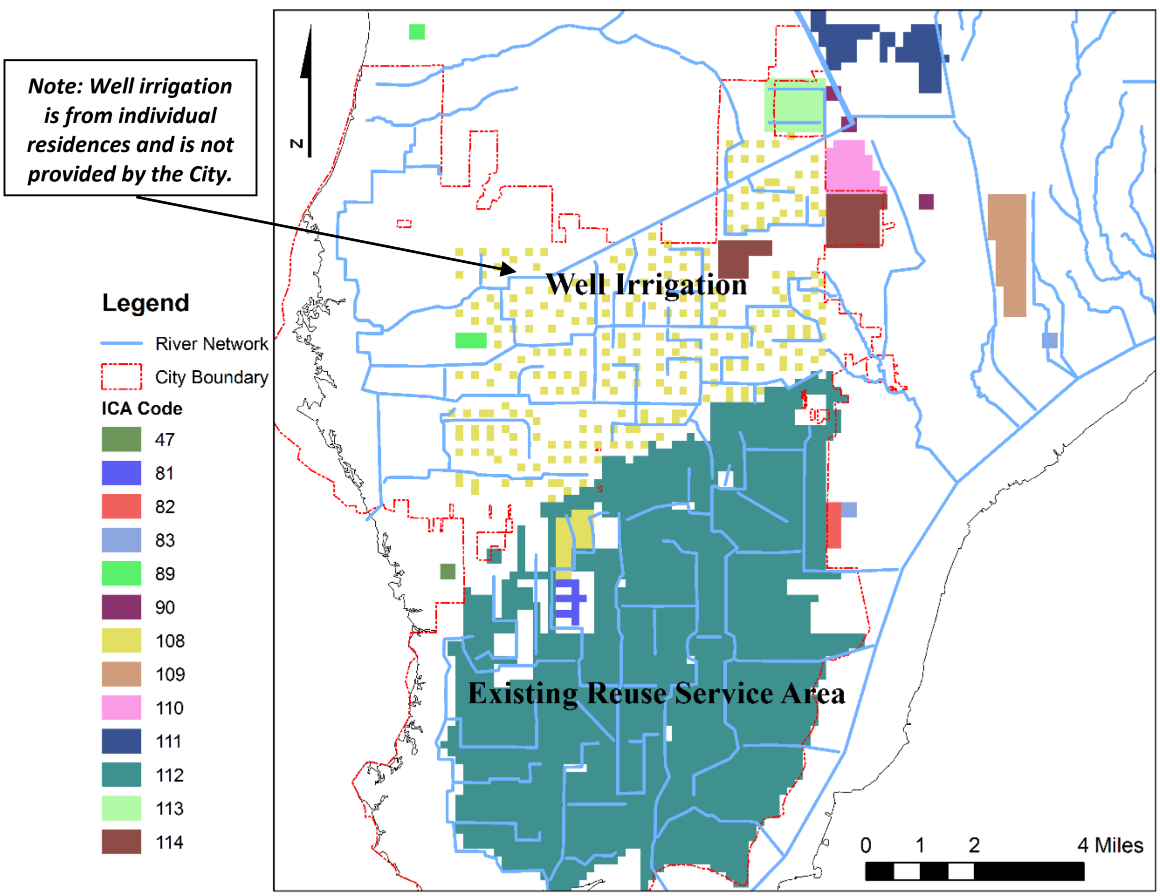


Figure 9. Irrigation Command Areas

Table 5. Irrigation Command Areas found near the City of Cape Coral and their source of irrigation.

Grid Code	Description	Source of Irrigation
47	Royal Tee Golf Club	External source and shallow well
81	Palmetto Pines Country Club	Aries Canal upstream of Weir 10 (in Basin 17)
82	Paradise Park	External source
83	Paradise Marina	External source
89	Coral Oaks Golf Course	External source
90	Raintree RV Resort and Sabal Springs Golf & Raquett Club	External source
108	North Cape Coral houses	Shallow well (10 to 32 ft bls) for irrigation
109	Powell Creek	Shallow well (10 to 32 ft bls) for irrigation
110	Del Tura Golf Course and Fountainview Park	Shallow well (10 to 32 ft bls) for irrigation
111	Heron's Glen and Magnolia Landing	Shallow well (10 to 32 ft bls) for irrigation
112	Cape Coral Reuse Service Area	Specified pumping from CPS 2, 3, 4, 5, and 8
113	Pine and Fairway Lakes	On-site lakes that are recharged with shallow groundwater
114	Coral Lakes and Entrada Estates	Deep well (32 to 66 ft bls) for irrigation

The City of Cape Coral uses five Canal Pump Stations within the City canals (CPS2, 3, 4, 5, and 8) that pull water from the canals and distribute it throughout the city for irrigation purposes. Because the irrigation water influences the water table which affects overland runoff, the canal pump stations were represented in the model. To accomplish this objective, the following procedure was followed:

1) Monthly pumpage reports for each pump station were provided by the Utilities Department. This data includes daily reporting of water volume pumped from the canals by CPS2, CP3, CPS4, CPS5, and CPS8 from 1/1/2012 to 12/31/2013. An example of the monthly pumpage reports from Cape Coral is provided in Appendix A.
2) All pumps were set-up as discharge control structures which remove water using the provided time series. Each pump has a minimum operating level or shut off level shown in **Table 6**.

Table 6 CPS shut off levels

Basin#	Controlling Structure #	CPS#	Shut off level (ft-NGVD)
16	7	2	2.2
16	7	3	2.2
15	1	4	2.5
14	2	5	4.0
14	2	8	4.0

3) Hypothetical tanks were setup in the M11 network file to receive the water pumped from the canal. This step ensures that the actual canals will not be pumped dry when linked with irrigation. The tanks (labeled TANK_CPS2, TANK_CPS3, etc.) were set-up using a sufficiently wide cross-section (3,000 ft), long chainage (greater than 2,000 ft for all tanks), and high top-of-bank (-20 ft

to 65 ft). The tanks were coupled with MIKE SHE using a leakage coefficient of zero to ensure no exchange with the groundwater.

4) The existing service area for the canal irrigation in the City of Cape Coral was provided by Cape Coral. This area was input into the irrigation grid as a new irrigation command area (as shown in **Figure 9**) with the tanks supplying the area with the required irrigation water. Irrigation is applied to ICA 112 according to internal model code that determines soil moisture within a given cell and then applies water to that cell to increase the soil moisture levels. The model input files limit daily irrigation from the tanks to the maximum value of the total daily pumping from all CPS.

2.1.11. Manning's Number

The amount and type of vegetation on the land surface determines the roughness of the land surface. This roughness or friction is a parameter that affects the flow of water over the land surface. The Manning's M (the inverse of Manning's n) for the model domain was simplified from the landuse map and shows a range of values from 0 to 9 $m^{(1/3)}/s$. In this range a value of 0 represents open water, a value of 3 represents densely vegetated or natural lands, and a value of 9 represents highly developed regions where the land surface is smoothed due to reduced vegetation and paved surfaces. An area with smoother surfaces (higher Manning's M) will exhibit quicker runoff of overland flow. **Figure 10** shows the distribution of Manning's M values used for the model.

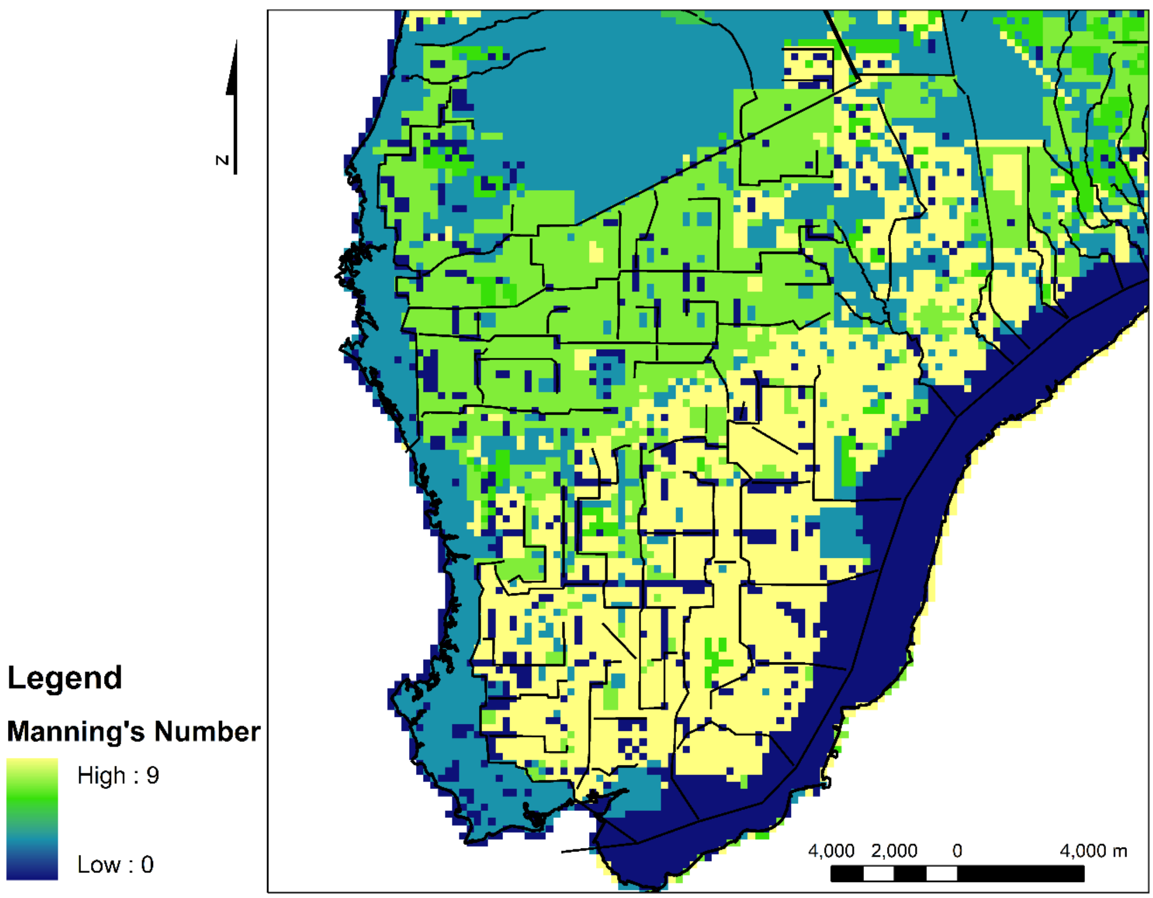


Figure 10. Manning's number used for the City of Cape Coral.

2.1.12. Separated Flow Areas

Separated Flow Areas determine sub-basins within the domain that do not necessarily share overland flows. This means that within each Flow Area, overland water will route to the nearest canal that is within the Separated Flow Area.

The City of Cape Coral's water management group has delineated a set of 18 Canal Basins which are controlled by major structures. The Canal Basins and structures are shown in **Figure 2**. For example, in Canal Basin number 3, all water that falls on the surface within that basin will route to the Horseshoe Canal upstream of Weir 21.

2.2.Subsurface Properties

The subsurface layers in this model were determined from data provided by the Southwest Florida Feasibility Study (SWFFS), conducted by SFWMD and the U.S. Army Corps of Engineers (USACE) (DHI, 2002; CDM/DHI, 2006). The following sub-sections describe how the subsurface properties have been represented.

2.2.1. Soil Properties

In the Unsaturated Zone, the majority of the model domain is covered with the Immokalee soil type (about 83% over the entire domain), as shown below in **Figure 11**. Sanibel soil type is found mostly in river flood plain regions, Plantation soils are found mostly around the coastal mangrove regions. Both soil types have higher water retention than the remaining inland soil types.

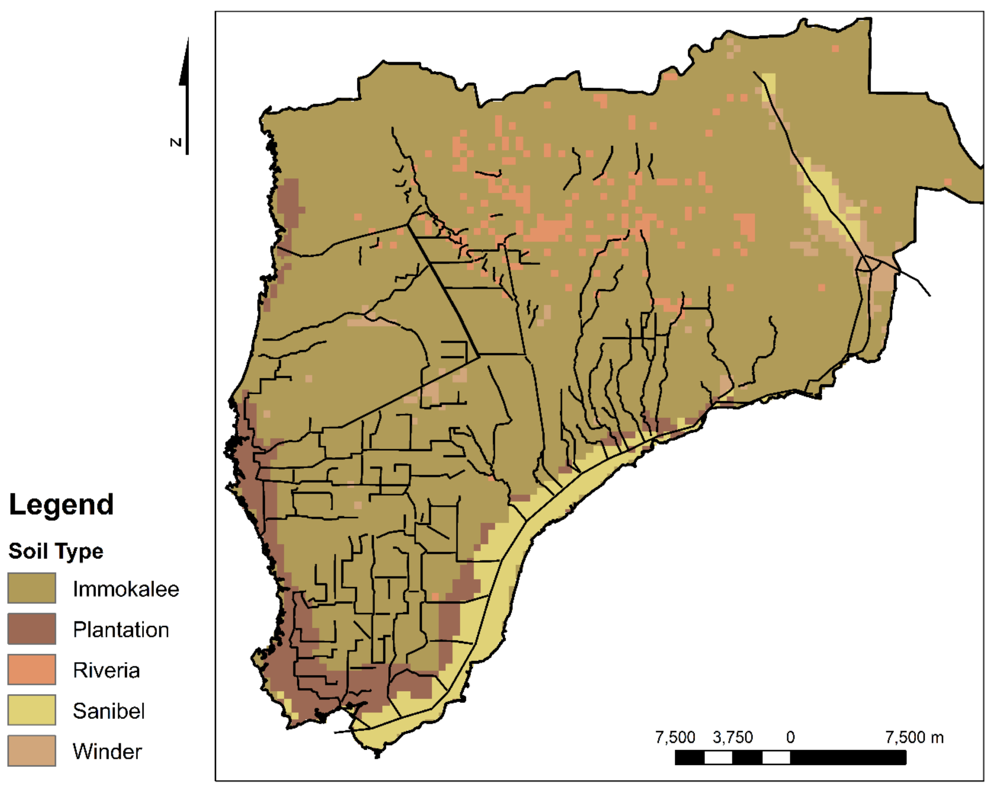


Figure 11. Spatial distribution of specific soil types used in the model.

2.2.2. Saturated Zone Layers

As defined from the SWFFS hydrogeological report, the Saturated Zone was developed into three layers:

- **Aquifer Layer 1** – Holocene to Pleistocene sands and Late Pliocene (Pinecrest) limestone, where present.
- **Aquifer Layer 2** – Early Pliocene (Ochopee Limestone). This unit conforms to the historical definition of Lower Tamiami aquifer where confined and to the lower part of the Water-Table aquifer where unconfined. The vertical extent of the unit is defined from the top of Ochopee Limestone to the top of the Peace River Formation.
- **Aquifer Layer 3** – Sandstone aquifer. This-unit is defined from the top of the sandstone unit (Lehigh Acres Sandstone) in the Peace River Formation to the top of the basal clay in the Peace River formation.

In addition, two (2) geologic lenses were defined at the base of Aquifer Layer 1 and the base of Aquifer Layer 2. The lens at the base of Aquifer Layer 1 is called the Bonita Springs Marl. This lens is only present in sparse areas of the domain (absent throughout most of the City of Cape Coral) and is therefore considered a semi-confining unit. The Bonita Springs Marl has a hydraulic conductivity that is over 100 times less than that of Aquifer Layer 1.

The Upper Peace River Confining Unit is at the base of Aquifer Layer 3 and is present throughout the model domain. This unit has a hydraulic conductivity that is significantly less than that of Aquifer Layer 2 or 3, making it difficult for water to travel between the layers. This lense acts as a confining unit.

3. Water Balance

Water balance analysis was performed for the separate drainage basins. Instabilities and errors between the Canal Basins water balance and the MIKE SHE water balance were examined during calibration. The MIKE 11 water balance setup is shown in **Figure 12**.

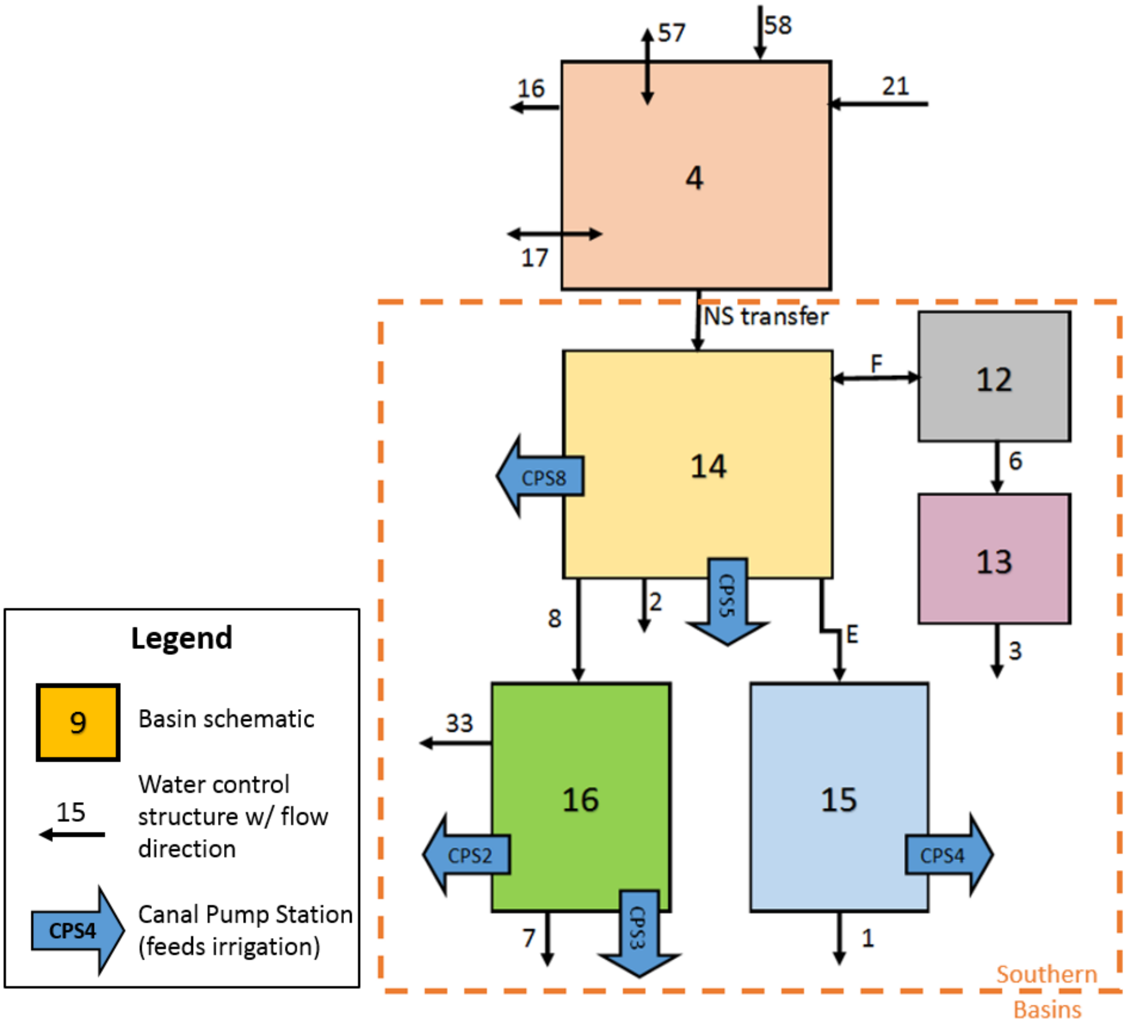


Figure 12 Canal Basins water balance schematic.

Basin 4 receives water primarily from Gator Slough through structure 58 which is a broad-crested weir with a crest at elevation 8.25 ft NGVD. This structure is located in Syracuse Canal which then discharges to Horseshoe, Zanzibar, and Hermosa Canals. Weir 58 controls much of the discharge entering Basin 4, which is important for supplying flows to the North-South (NS) Transfer Station pumps that feed the southern basins. Weirs 16 and 17 and the Structure 17 pump station also provide water level control for Basin 4. Pump station 17 transfers water from Hermosa Canal west of Weir 17 (Chiquita Blvd) upstream into Basin 4. Pump station 21 is a forward pump in Horseshoe Canal at Santa Barbara Blvd that can move water west from Basin 3 into Basin 4 during periods when water levels in Horseshoe Canal at Weir 21 are below the crest elevation. Record-keeping of pump outages and supplementary pumps to augment flows during pump outages was not sufficient at the NS Transfer Station to completely understand flow deliveries to Basin 14.

Basin 14 receives discharges primarily from the NS transfer and from the F Interconnect between Basin 12 and 14. The NS Transfer must provide enough water to fulfill the demands of the Canal Pump Stations which remove water from the canals and store it in hypothetical Tanks. The water stored in these Tanks is then used for irrigation in the specified areas of Cape Coral that are served by the Irrigation Supply force mains.

Basin 16 receives water primarily through structure 8 which includes a fixed weir with a crest elevation at 5.78 ft NGVD and an underflow by-pass structure set at 2.59 ft NGVD with a sluice gate that opens to 4.68 ft NGVD. This structure controls water levels in Basin 16 and maintains minimum operating levels for CPS2 and CPS3. Prior to 2013, records were not maintained for gate operations at Structure 8. Since water levels in Basin 16 are regulated by the releases through the gate at Structure 8, this lack of data complicated calibration.

Basin 15 only receives canal inflow from the E Interconnect on Damao Canal. Approximately one-third of the total water in the canals comes from base flow and drainage from the saturated zone (Lateral Inflow). Again, gate operations for the E Interconnect were not available for the calibration period, and this lack of data complicated the calibration. Furthermore, construction along Santa Barbara Blvd above this Interconnect affected flows in 2012, and the actual dates of this disruption of flow were not available.

3.1.Southern Basins

The Southern Basins are comprised of Basins 12, 13, 14, 15, and 16 as shown in **Figure 14**. They receive canal water from only one source, the NS Transfer Station. Outfalls of this group are Weirs 1, 2, 3, 7, and 33. In addition, all of the Canal Pump Stations (CPS 2, 3, 4, 5, and 8) remove water from these basins.

Figure 13 illustrates a complete wet season water balance that was used to calibrate parameters within the basins. The rainfall, evapotranspiration, irrigation, and lateral inflow from groundwater was calculated over the entire region and summed for the wet season and dry season. Discharge at the inflow and outflow locations in the canals was summed to determine the total canal inflow and outflow, as well as the total pumping from the Canal Pump Stations. **Figure 14** presents a detailed view of the basins, canals, weirs and canal pump stations in the Southern Basins.

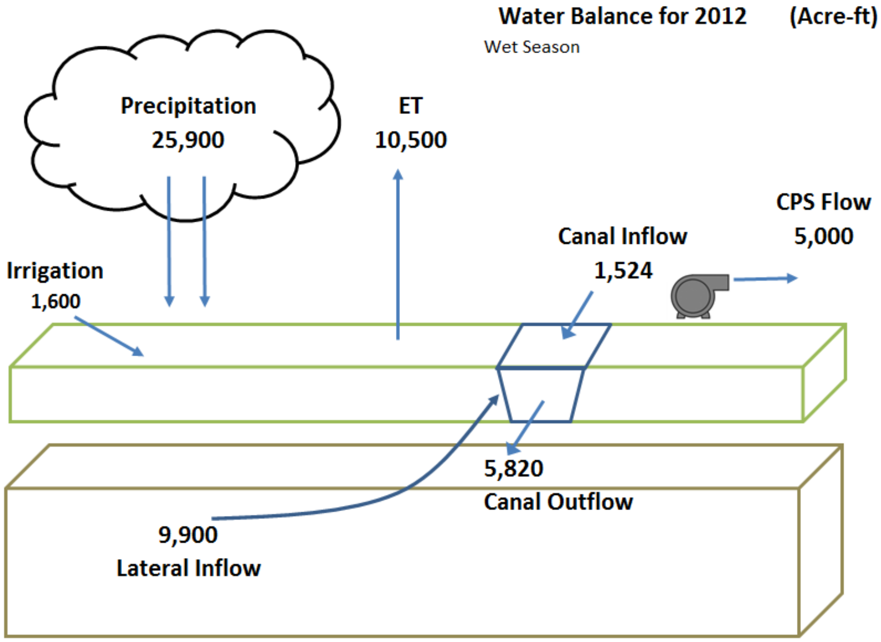


Figure 13 Water Balance schematic for the Southern Basins in acre-ft.

For the Southern Basins, all of the Canal Inflow comes from the North-South Transfer pump station which moves water south of Pine Island Road. The remaining water demand to maintain canal water levels and CPS flow is met through net rainfall (rainfall – evapotranspiration) and irrigation. Net rainfall and irrigation can flow into canals via three possible pathways: surface runoff, subsurface runoff (or drain flow) or groundwater baseflow. These three types of flows are referred to in the model as lateral inflows. Thus, the total inflow to the Southern Basin canals is sum of the three types of lateral inflows plus the flow from the North-South Transfer pump station. The total outflow is the sum of the flow through the CPS pumps and the outflow structures (Weirs 1, 2, 3, and 7). The difference between the inflow and the outflow is equal to the change in storage (i.e., the water volume in the canal system) for the period of time evaluated. The cumulative wet season values for the Southern Basins shown in **Figure 13** are summarized below:

Canal Outflow = Canal Inflow + Lateral Inflow – CPS Flow +/- Error

5,820 Ac-ft = 1,524 Ac-ft + 9,900 Ac-ft - 5,000 Ac-ft – 604 Ac-ft

Error is 604/11,424 or 5.3%

The model was sensitive to parameters such as drainage depth, leakance, and vegetation coverage, which were modified during calibration. The model results indicate that the model is able to provide a reasonable overall representation of the major processes that affect Cape Coral canal flows and water levels.

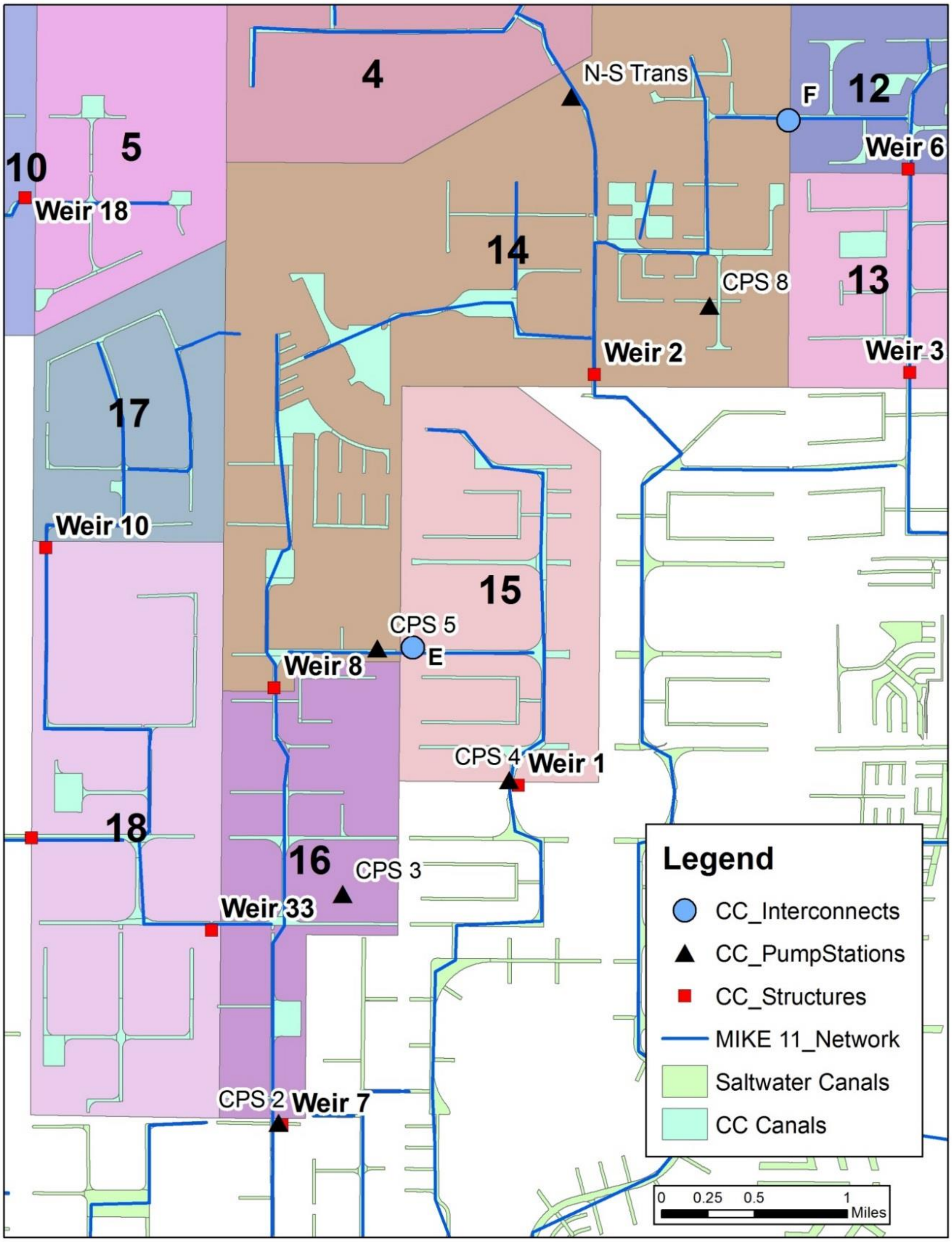


Figure 14. Basins and structures which are critical for maintaining a balanced water budget for the City of Cape Coral south of Pine Island Rd. Basin 12, 13, 14, 15, and 16 all receive flows from the N-S transfer station and are part of a larger water budget termed the Southern Basins.

4. Calibration and Validation

The model calibration focused on improving water levels in the City Canal Basins for the 2012 calibration period (1/1/2012 through 12/31/2012). The calibration period of 2012 was chosen because it demonstrates the full range of seasonal hydrological phenomena typical of south Florida; from wet in the summer to dry in the winter.

After implementing the appropriate controlling features for surface water and subsurface parameters, as described in the Model Development section above, parameters such as hydraulic conductivities, surface water/ groundwater interactions, flow control factors, etc. were modified on a global and local scale in order to fine-tune the model and achieve desirable calibration.

Modeled results were compared with measured data from 10 surface water monitoring stations and 2 groundwater monitoring wells within the City. Data from these stations was available on the SFWMD DBHydro and the USGS website databases. At each surface water monitoring location, USGS recorded water levels upstream of the weir, and flows were calculated based on a pre-determined stage-discharge relationship associated with each weir.

Model validation was performed for the period of 1/1/2013 through 12/24/2013. Model validation tests whether the assumptions and calibrated parameters determined for the model reasonably represent the real-world system. Calibrated model parameters were kept the same for the 2013 validation period.

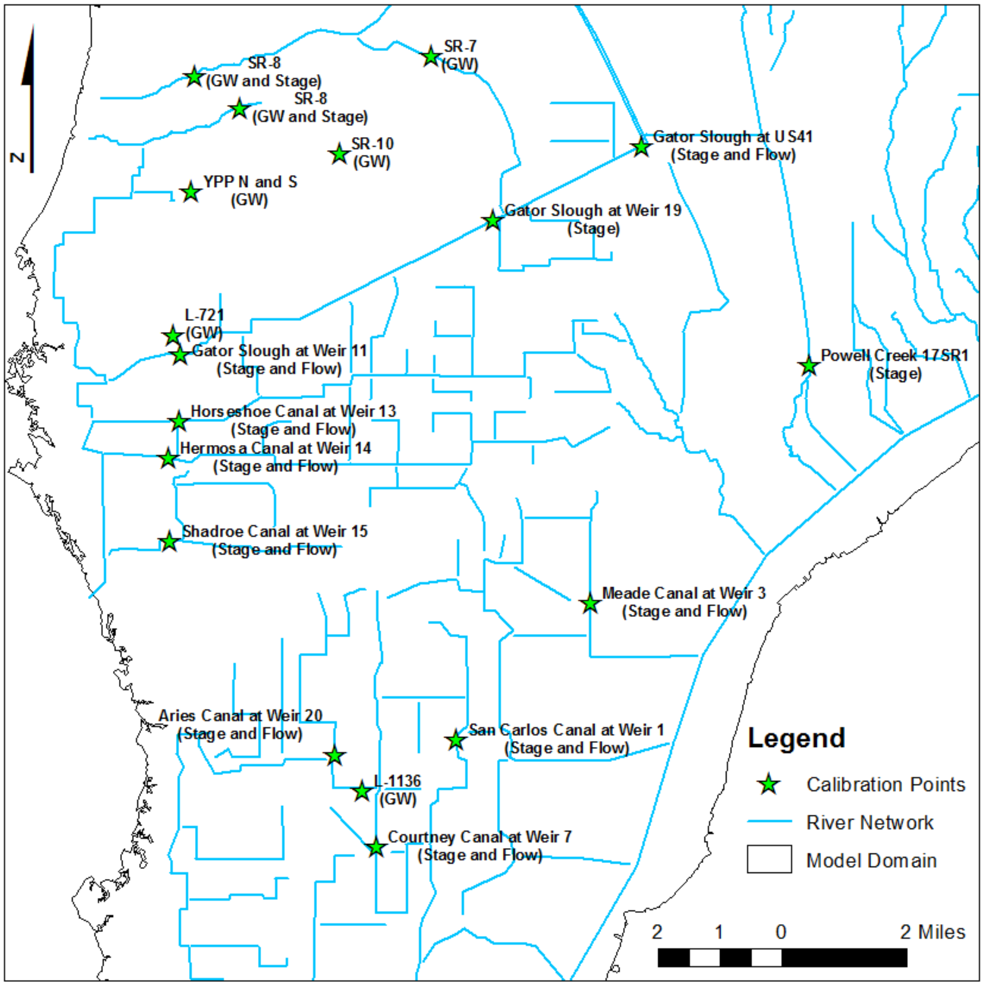


Figure 15. Observed data points used for model calibration and validation.

Graphs of observed versus modeled results are found in **Appendix B: Calibration and Validation Results**. Within the City of Cape Coral, the model results compared well with observed values. Excluding the pumping stations, the modeled matched observed with correlation coefficient of 79% for the 2012 calibration period. During the validation period, the correlation between modeled and observed was around 61%. The drop in the correlation between measured and simulated values for 2013 is due to lack of observed data points as well as a number of other factors, as explained below.

Table 7 provides the correlation coefficients for the full simulation period (1/1/2012 through 12/24/2013), the calibration period (1/1/2012 through 12/31/2012), and the validation period (1/1/2013 through 12/24/2013). Only observed values within the City of Cape Coral were compared with simulated. ND indicates that there is little to no observed data available for comparison. The correlation coefficient is used to measure the strength and direction of the linear relationship between the measured data and the results at that location in the model. A value of 0 indicates no correlation, a value of 1 indicates an exact correlation, and a negative value indicates an inverse relationship (i.e. the measured data is high when the modeled data is low and vice versa).

Table 7. Correlation coefficients for observed water levels and flows within the Cape Coral area for the full simulation period, the 2012 calibration period, and the 2013 validation period.

Cape Coral Data	2012-2013	2012 Calibration	2013 Validation
Gator Slough at US41 Stage	0.90	0.93	0.94
Gator Slough at US41 Flow	0.84	0.88	0.89
Gator Slough at Weir 19 HW	0.93	0.97	0.58
Gator Slough at Weir 11 HW	0.81	0.92	0.54
Gator Slough at Weir 11 Flow	0.86	0.87	0.85
Horseshoe at Weir 13 HW	0.82	0.91	-0.22
Horseshoe at Weir 13 Flow	0.78	0.78	ND
Hermosa at Weir 14 HW	0.81	0.90	0.41
Hermosa at Weir 14 Flow	0.57	0.76	0.39
Shadroe at Weir 15 HW	0.79	0.92	0.56
Shadroe at Weir 15 Flow	0.65	0.65	0.32
Meade at Weir 6 HW	0.65	0.66	0.64
Meade at Weir 6 Flow	0.62	0.60	0.74
San Carlos at Weir 1 HW	0.82	0.89	0.45
San Carlos at Weir 1 Flow	0.59	0.58	ND
Courtney at Weir 7 HW	0.39	0.48	0.52
Courtney at Weir 7 Flow	0.65	0.63	0.71
Aries at Weir 20 HW	0.76	0.82	0.90
Aries at Weir 20 Flow	0.62	0.62	ND
L-721 GW	0.87	0.97	0.74
L-1136 GW	0.88	0.87	0.99
Average	**0.74**	**0.79**	**0.61**

In **Table 7**, the upper basins north of Pine Island Road (Basin 1, 6, 7, 9, and 10) are highlighted with green, the lower basins (Basins 12, 15, 16, and 18) are highlighted with yellow, and groundwater stations are highlighted with orange.

4.1. Calibration Issues

During the model calibration process, model parameters were adjusted such that the following water budgeting priorities could be met:

- **Basin 4 water levels:** Gator Slough must be receiving enough flow and must be draining down into the City from Weir 58. In addition, proper weir operations must be established at Weir 16 and Weir 17 and proper pumping rates must be established at Pump 17 and Pump 21. There are limited water monitoring stations within Basin 4; however, pumping at the North-South Transfer station will cease if the minimum operating levels at the pump station headwater basin (2.5 ft-NGVD) are reached.

- **Water levels in the Southern Basins:** Correct pumping operations at the North-South Transfer station must be determined in order to provide the City Canal Basins south of Pine Island Road with enough water to operate the Canal Pump Stations above their minimum operating levels. Because this area is highly managed by structures, records of gate operations must be kept at regular intervals in order to properly predict water levels. Lack of information, for example relating to the effective gate heights at Weir 8 and its bypass structure, was a limitation during calibration. Irrigation schemes, overland runoff parameters, vegetation parameters, and leakance to groundwater were calibrated to provide the canals with enough lateral inflow to support the pump station demands.

Water within the City of Cape Coral is largely controlled by the extensive network of canals, pumps, gates, and fixed weirs. However, it is impossible to perfectly match modeled results with measured data due to several factors:

- **Undocumented water control operations:** Because the City of Cape Coral is a highly managed system of water control structures and pumps, it is necessary to centrally manage and document the gate and pump operations as part of an overall water control strategy for the City. It was determined that the City installed temporary auxiliary pump operations at the N-S Transfer Station during a portion of 2012, however there are no records available to determine the flows associated with these auxiliary pumps. Records of gate and pump operations for the 2012 to 2013 period were incomplete for some areas. Missing data for the City included records of the placement and removal of risers, as well as the condition of the risers (i.e. leaking or broken). Smaller structures were missing operations records, such as the Weir 8 by-pass or cross-connect structure E, which was a crucial structure in the control of water levels at Weir 7.
- **Debris in the flow way:** Debris can accumulate in canals, culverts, weirs, grates, inlets and other structures located along the path of the water flow at any time during the year. This debris can create flooding issues during the storm season. In the calibration results, debris issues can be seen in the measured data with uncharacteristically high spikes in water levels, usually only lasting a day. One example of this type of event occurred on 9/23/2012 in Meade Canal at Weir 3 (Viscaya Pkwy and SE 21st Ave). The water level in the canal spike to almost 7 ft NGVD that day. The model is unable to simulate such an event, despite having very good calibration for the rest of the year. One possible explanation for the observed high spike is that a blockage occurred at the Weir 3 carp grate.

5. Design Storms

After final calibration and validation of the model, a series of design storm simulations were developed, implemented and analyzed to determine the effect of hydrological stressors on City operated weirs. A series of designed storms (shown in Table 8) were developed into the existing NEXRAD gridded rainfall data. The total rainfall for each event was derived for the Cape Coral area from Pathak, 2001. The rainfall hydrographs for the storm events are shown in Figure 16.

Table 8. Design Storms implemented into the model.

Name	Event Frequency	Duration	Total Rainfall (inches)
2 yr	2-year	24 hours	3.5
5 yr	5-year	24 hours	5
10 yr	10-year	24 hours	6
25 yr	25-year	72 hours	11 - 11.5
100 yr	100-year	72 hours	13 - 14
3in1hr	NA	1 hour	3

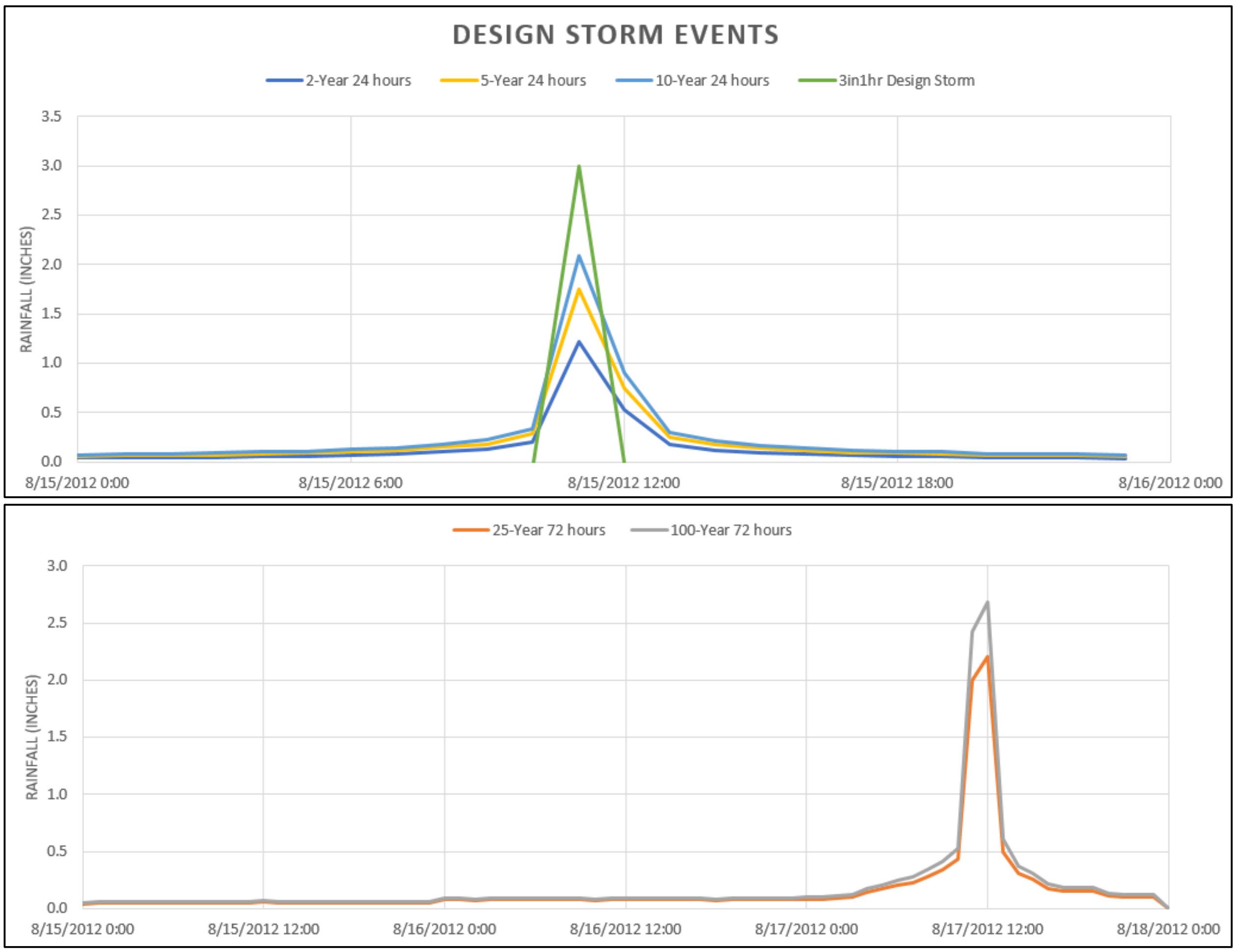


Figure 16. The rainfall intensity curves for the developed into the Design Storm simulations. The short duration events are shown on the top and the long-duration events are shown on the bottom.

The rainfall events were inserted into the existing NEXRAD dataset starting on August 15, 2012. The date chosen represents average wet-season conditions, which allows the soils to be saturated but not flooded at the start of the simulation.

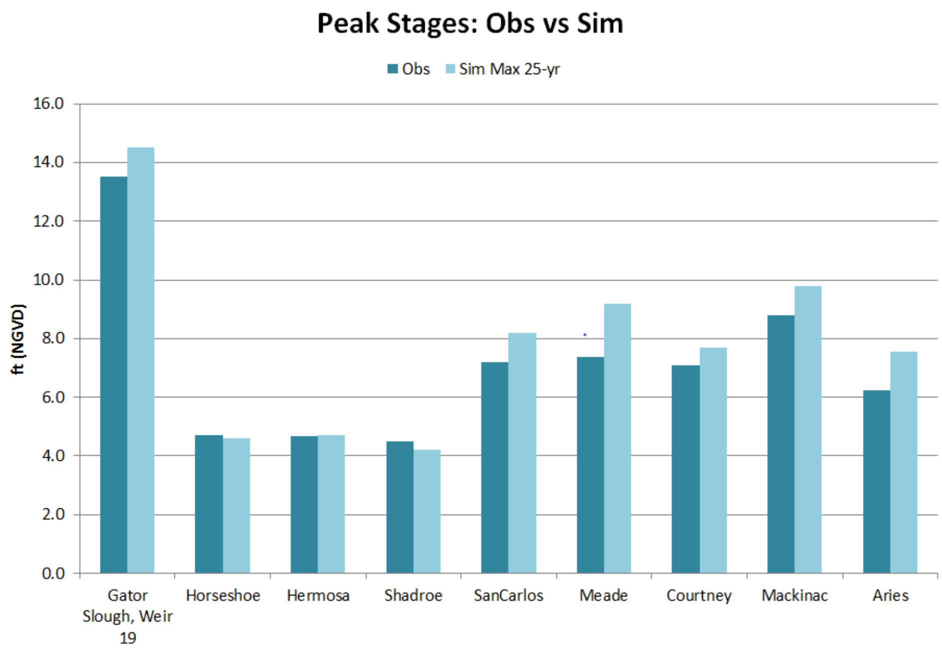


Figure 17. Comparison of 25-yr Design Storm peak stages with a 25-year period of record for observed data. Peak stages at each of the locations in Cape Coral during the Design Storm are similar to max observed values in the 25 year period of record.

To validate that the Design Storms are reasonable for the City of Cape Coral, the 20-25 year period of record for observed data in each of the canals was obtained. Maximum stages for that period were compared with modeled peak stages obtained during the 25-year Design Storm, as shown in **Figure 17**. The comparison shows that the peak design storm stages were similar to observed peak stages.

5.1.Operations

The design storm simulation input files were built using the calibrated model (existing conditions) and it was assumed that gates were operated according what was used in the calibrated model from 1/1/2012 to 8/12/2012. On 8/12/2012 the following structure operations were enforced:

- Structure 19 Overflow gates open, underflow gates open if the headwaters exceed the highest level of the concrete weir.
- All Obermeyer gates open (air bladders empty). This applies to Weirs 1, 6, 7, 11, 13, 14, 15, and 21.
- Structure 8 gate and side structure open.
- Flashboards removed for structures 2, 3, 4, 8, 16, 17, and 33

For three (3) days the structures stayed in their open position, mimicking current protocol which the City may use in preparation for forecasted storms. The design storms were then initiated on 8/15/2012, with the gates remaining down for the entire simulation of 30 days.

5.2.Results

Results for the Design Storm simulations were analyzed in two hydrological components: surface water and groundwater. Surface water flows include canals which were simulated in the river network file.

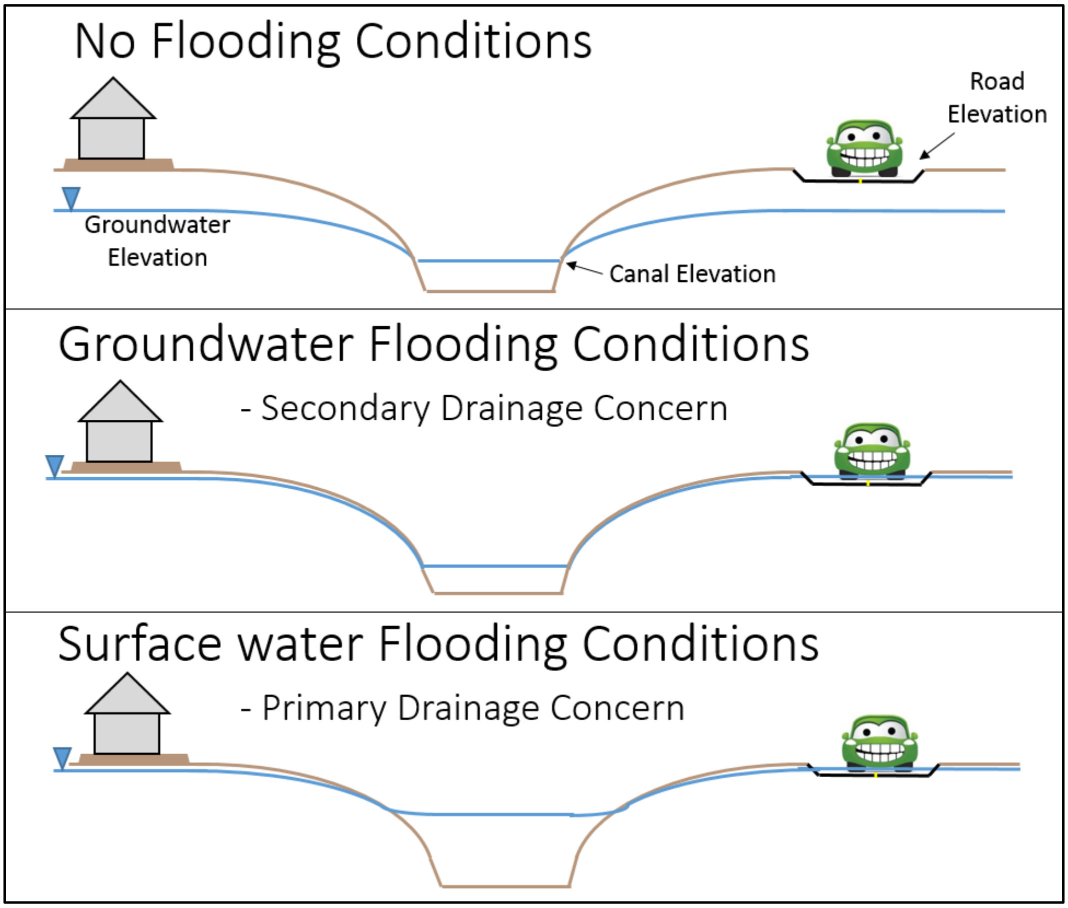


Figure 18. Flooding analysis for the Design Storm scenarios

5.2.1. Primary Flooding

To determine the flooding on the roads, road elevations were first evaluated for each basin. To do this, LiDAR land elevations were extracted every 10 ft along City of Cape Coral roadways. A percent exceedance curve was created for each basin which provides an estimation of how much of the roads in the basin are above a certain elevation (see Figure 19).

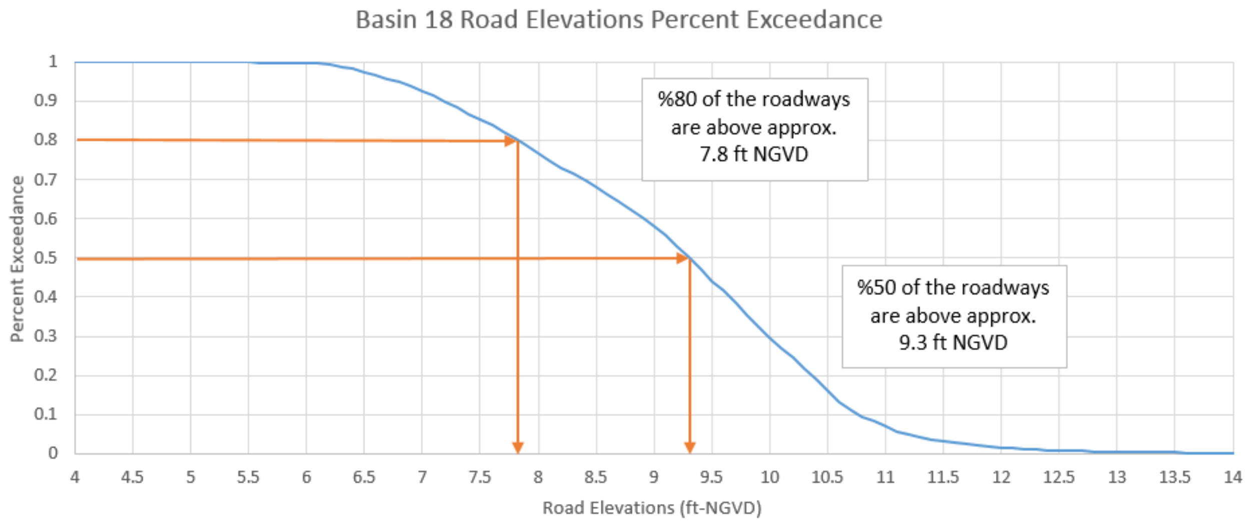


Figure 19. Basin 18 Roads Percent Exceedance Curve

The percent exceedance was then used to determine the percentage of roads that were below the peak canal stage upstream of the controlling structure for the basin.

For all subsequent analysis documented in this report the crown of the road was used to determine the impact of Design Storms, Structural Design Scenarios, Water Quality Storage Scenarios, and Future Build-Out Scenarios. It is noted that docks and seawall elevations were not used to determine the impact of peak canal stages.

5.2.2. Secondary Flooding

Secondary flooding is identified when the Saturated Zone rises above the land surface. This is caused by the mounding up of water over areas: 1) with low elevation, 2) with low horizontal hydraulic conductivity, and 3) areas that are sufficiently far away from drainage canals. The model is limited by the cell size (750ft), plus not every canal or drainage ditch could be included (the model does not allow multiple river connections per cell). This limits the drainage options for groundwater during a quick and heavy storm event. However, in reality, roadside swales, drains, ditches, and minor canals help to channel water toward the major canals, which then flow to tide.

The percent of time that the saturated zone was above the surface was evaluated. Figure 20, Figure 21, and Figure 22 illustrate only the cells that flooded over 3 inches above the surface for more than one day. Some areas may show excessive flooding due to the fact that the topography was averaged from 10-ft resolution LiDAR data, producing a low topographic area where lakes or large canals may have contributed to the elevations.

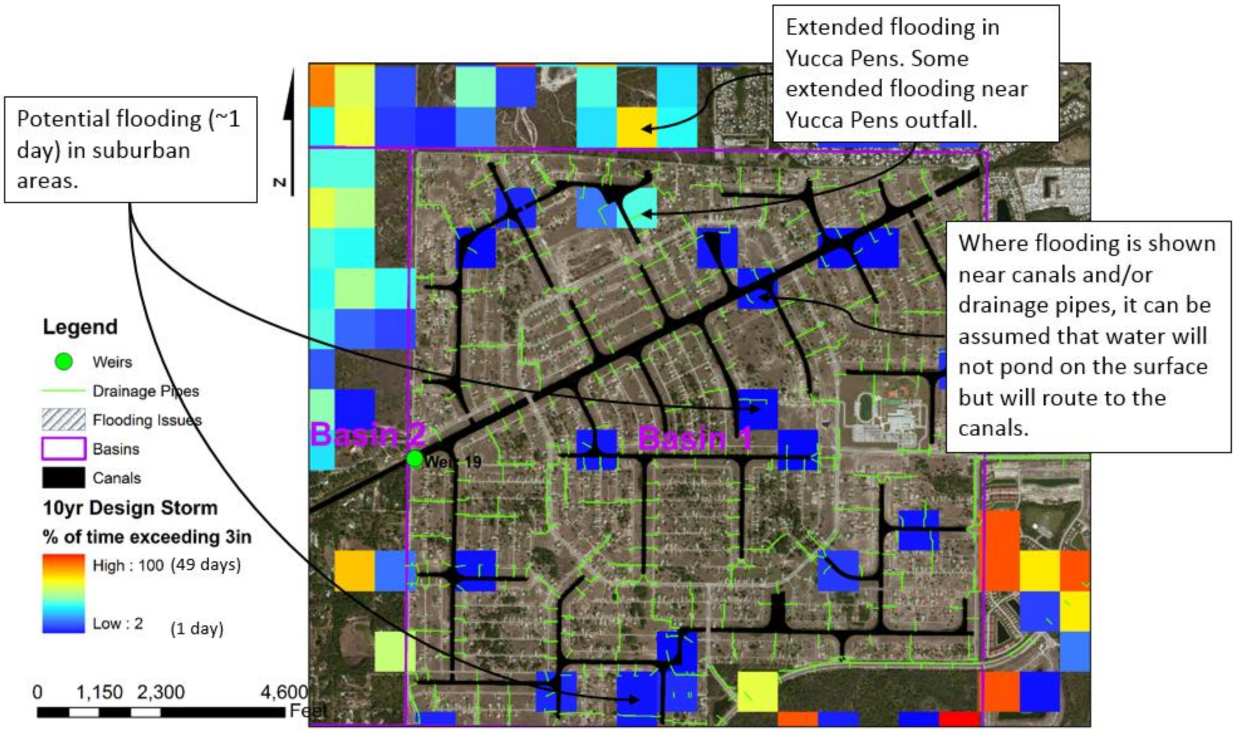


Figure 20. Secondary flooding in Basin 1. The colored squares indicate the percent of time that there is more than 3 inches of water during the 10 year Design Storm simulation.

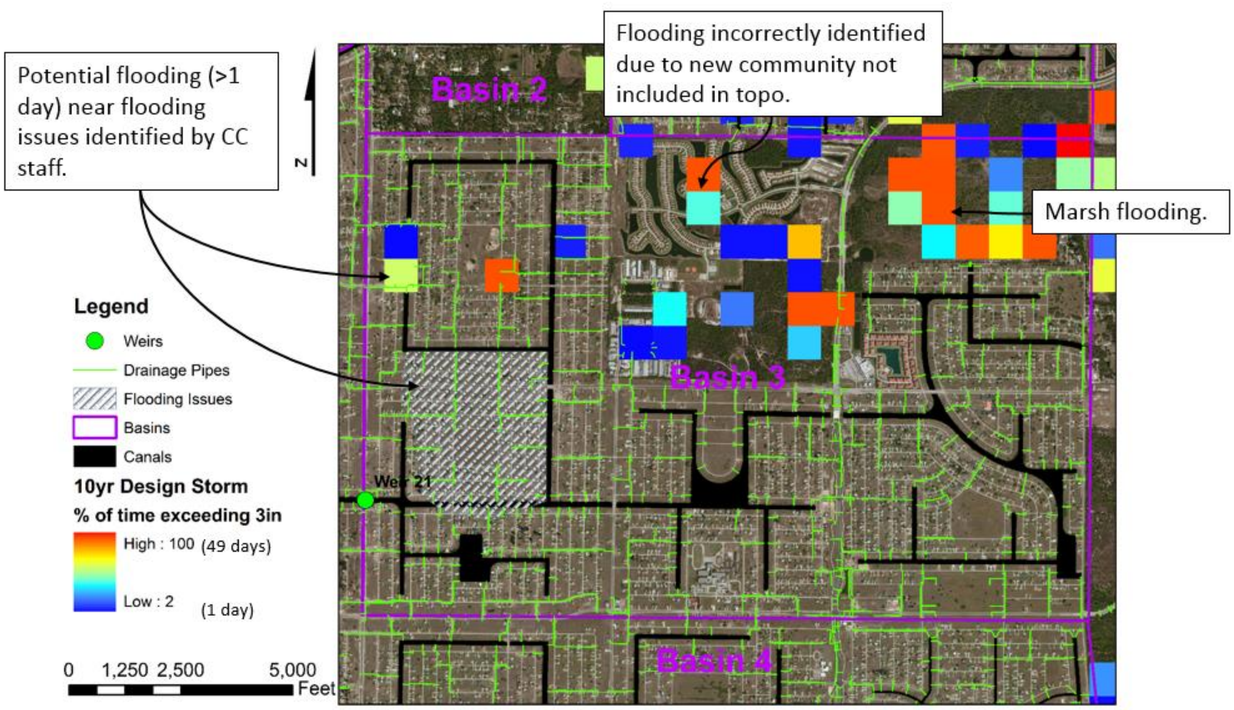


Figure 21. Secondary flooding in Basin 3. The colored squares indicate the percent of time that there is more than 3 inches of water during the 10 year Design Storm simulation.

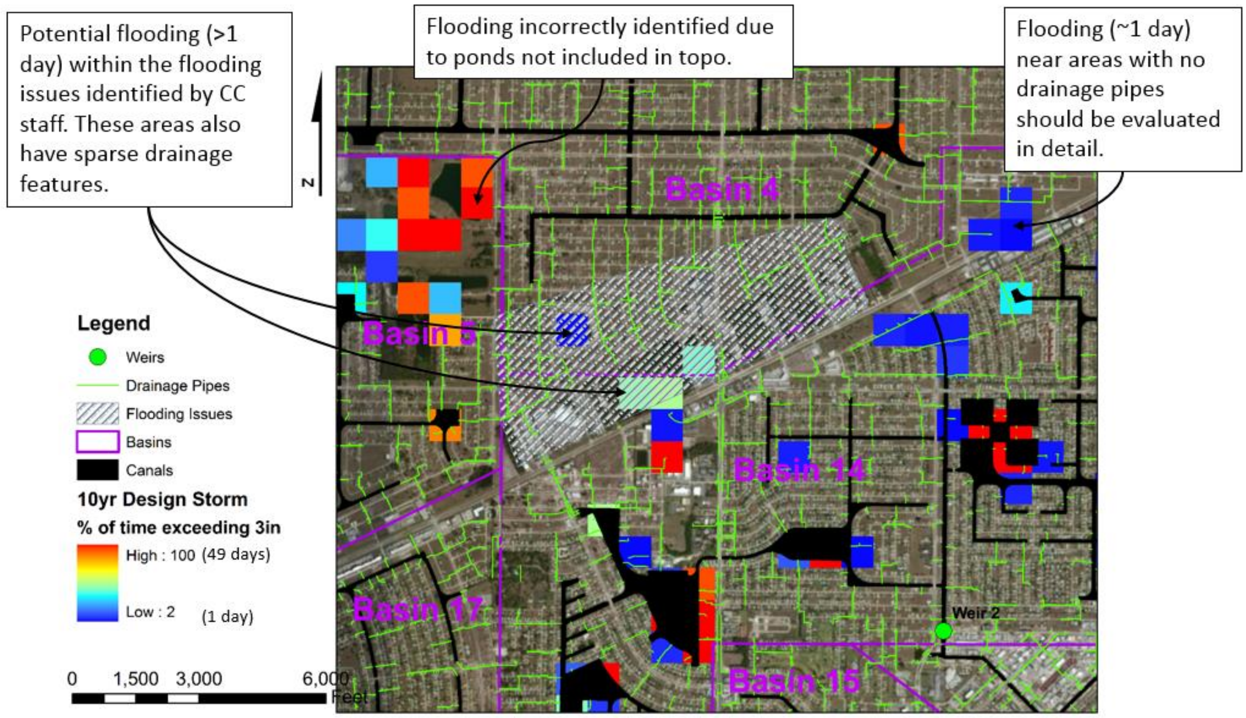


Figure 22. Secondary flooding in Basin 4 and 14. The colored squares indicate the percent of time that there is more than 3 inches of water during the 10 year Design Storm simulation.

6. Scenario Development

Two flow control structures in the City of Cape Coral have been identified as critical structures during a 25-year design storm event: Weir 1 and Weir 20 (based on the miles of flooded roads shown below in Table 9). Each of these structures have the potential to flood over 10 miles of roadway in their Basins during the extreme event while operating at their lowest gate level. To provide the City with an increased level of service, the following structure redesigns have been developed into a hydrological model for testing. The results were then analyzed for the potential to improve flooding and evaluated for cost effectiveness.

Table 9. Table showing the percentage of roads flooded and the linear miles of flooded roads per Basin for the 25-year design storm.

	Controlling Structure	% of Roads Flooded	Linear Miles of Flooded Roads
Basin 1	**Weir 19**	0.4%	0.4
Basin 2	**Weir 9**	0.0%	0.0
Basin 3	**Weir 21**	0.1%	0.1
Basin 4	**Weir 16**	0.0%	0.1
Basin 5	**Weir 18**	0.0%	0.0
Basin 6	**Weir 11**	0.0%	0.0
Basin 7	**Weir 13**	0.1%	0.0
Basin 9	**Weir 14**	0.3%	0.1
Basin 10	**Weir 15**	0.0%	0.0
Basin 12	**Weir 6**	0.6%	0.1
Basin 13	**Weir 3**	**8.9%**	2.6
Basin 14	**Weir 8**	0.0%	0.1
Basin 15	**Weir 1**	**21.5%**	10.6
Basin 16	**Weir 7**	**4.3%**	1.6
Basin 17	**Weir 10**	0.3%	0.1
Basin 18	**Weir 20**	**20.0%**	11.8

6.1. Structure Changes Scenarios

Once the critical Basins were identified using the Design Storm Scenarios, it was then possible to propose changes to the water control structures for those Basins. The changes would serve to alleviate flooding by reducing peak Design Storm stages. The focus for these Structure Change Scenarios was on Weirs 1 and 20.

6.1.1. Weir 1

Several scenarios were developed to determine how to most effectively lower peak water levels during the 25-year Design Storm event in Basin 15. Table 10 shows the various scenarios that were developed for Weir 1, such as increasing the effective weir length and adding side structures with lower sill elevations. It was determined that the best flood reduction was achieved with the addition of two underflow gates with a sill elevation of 3’ NGVD.

Table 10. Structural change scenarios developed for the analysis of flood reduction for Basin 15 (Weir 1).

		Miles of flooded roads (25 yr)	**Miles of flooded roads (100 yr)**
Exist.	*Existing Conditions*	*10.6*	*21.1*
Scenario 1	Increase Weir length to 36' (50% increase)	3.6	15.4
Scenario 2	2'x4' underflow side structure @ 3' NGVD	3.1	16.0
Scenario 3	2 - 2'x4' underflow side structure @ 3' NGVD	0.8	11.1

6.1.2. Weir 20

Several scenarios were developed to determine how to most effectively lower peak water levels during the 25-year Design Storm event in Basin 18. Structure 20 is currently a fixed weir at 42’ long with a sill elevation at 3’ NGVD. However, the culverts just downstream of the weir, under Chiquita Blvd, were shown to be a flow restriction during the extreme flood event. Increasing the cross-sectional flow area under Chiquita Blvd will reduce road flooding by approximately 45% in Basin 18. This is demonstrated with Scenario 1, as shown in Table 11.

Figure 23. High flows through Weir 20 on 7/16/2014. Culverts are almost full after a short rainfall event. High-water marks are visible well above the culvert crests.

Table 11. Structural change scenarios developed for the analysis of flood reduction for Basin 18 (Weir 20).

		Miles of flooded roads (25 yr)	**Miles of flooded roads (100 yr)**
Exist.	*Existing Conditions*	*11.8*	*22.2*
Scenario 1	Convert the culverts under Chiquita to 3 - 6'x10' box culverts	6.9	16.8
Scenario 2	2 - 10' gates @ 2' NGVD (plus Scenario 1)	4.3	14.9
Scenario 3	Increase Weir length to 80' (plus Scenario 1)	6.0	14.9

Plans for the improvement (widening) of Chiquita Blvd are on file at the City of Cape, as shown below in Figure 24. At Weir 20, the plans indicate that the four existing CMP's will be removed and replaced with new, longer CMP's of the same 6' diameter. In addition, the plans indicate that Weir 20 will be removed and rebuilt with the same dimensions slightly farther east. It is recommended that these plans be addressed and modifications are made in accordance with the findings of this report.

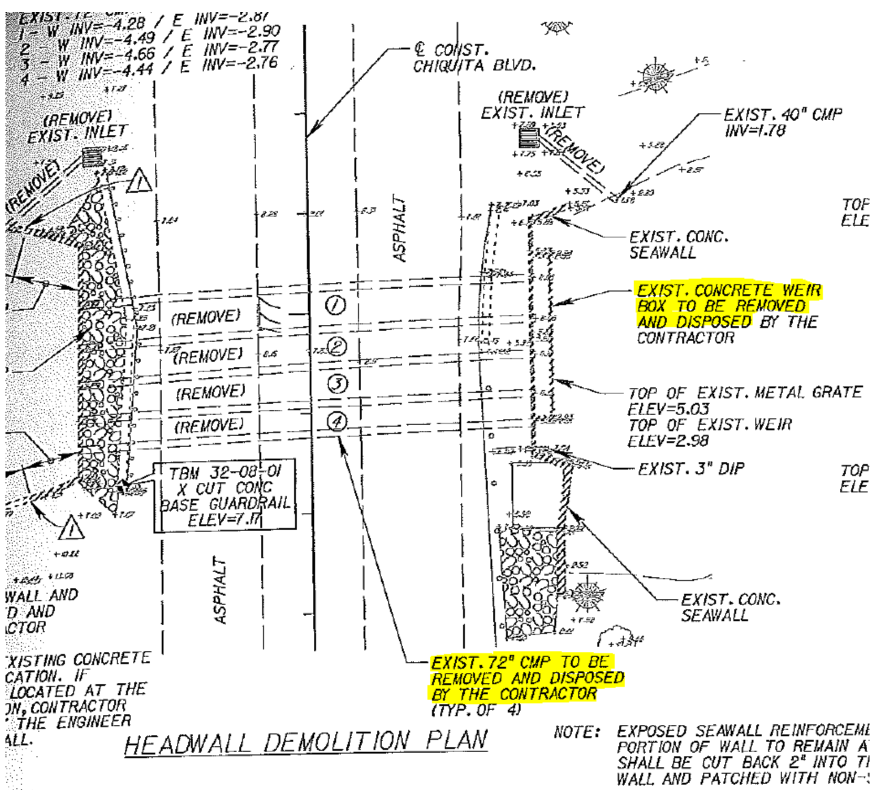


Figure 24. Chiquita Blvd Improvement plans indicate that the existing culverts and Weir 20 will be removed and disposed of and then replaced with structures of the same or similar dimensions and elevations.

6.2. Water Quality Storage

A simulation was developed to determine the Basins within the City of Cape Coral that may allow for improved water quality storage during flood events.

Design storm procedures initially set all of the control structures in the City to open (gates at lowest possible sill elevation). The Water Quality Storage simulation was run with all of the gate wet season control elevations raised by 0.2 ft. In addition, a “fully up” scenario was also evaluated with all of the gates in the closed position (dry season elevation).

LiDAR topography was compared with the peak canal stage to determine the percent of roads flooded in each basin. Weirs 1 and 20 showed a high degree of sensitivity to the modified sill elevations and should not be considered as possible basin storage areas. Many of the Basins (with the exception of Basin 9) could maintain even higher gate levels, as indicated by the low flooding impacts with the “fully up” scenario.

Table 12. Miles of flooded roads for the 25-year Design Storm with the proposed conditions model, the 0.2ft Up model, and the Fully Up model.

	Structure	Miles of Flooded Roads with All Gates Down (with proposed mods)	Miles of Flooded Roads with Gates at 0.2ft Up	Miles of Flooded Roads with All Gates Up
Basin 1	Weir 19	0.4	0.4	0.9
Basin 2	Weir 9	0.0	0.0	0.0
Basin 3	Weir 21	0.1	0.1	0.1
Basin 4	Weir 16	0.1	0.1	0.2
Basin 5	Weir 18	0.0	0.0	0.0
Basin 6	Weir 11	0.1	0.1	0.4
Basin 7	Weir 13	0.0	0.1	0.1
Basin 9	Weir 14	0.2	0.3	4.1
Basin 10	Weir 15	0.0	0.0	0.6
Basin 12	Weir 6	0.1	0.1	3.3
Basin 13	Weir 3	1.7	2.5	6.8
Basin 14	Weir 8	0.0	0.1	6.2
Basin 15	Weir 1	0.4	11.4	13.2
Basin 16	Weir 7	0.9	1.3	2.8
Basin 17	Weir 10	0.0	0.1	0.1
Basin 18	Weir 20	6.0	9.5	13.7

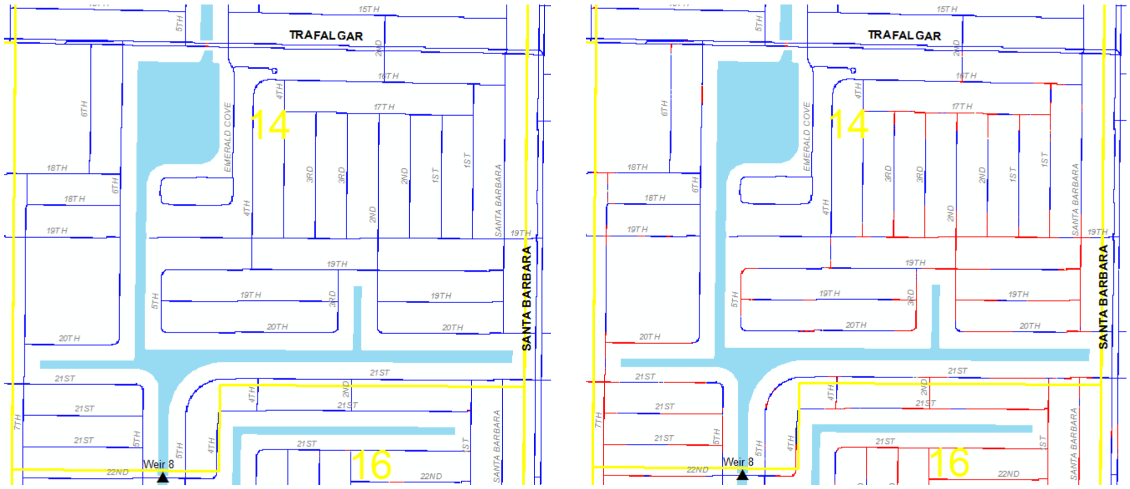


Figure 25. Basin 14 road flooding (red lines) for the "All Gates Down" scenario in the left image and the "Fully Up" scenario in the right image.

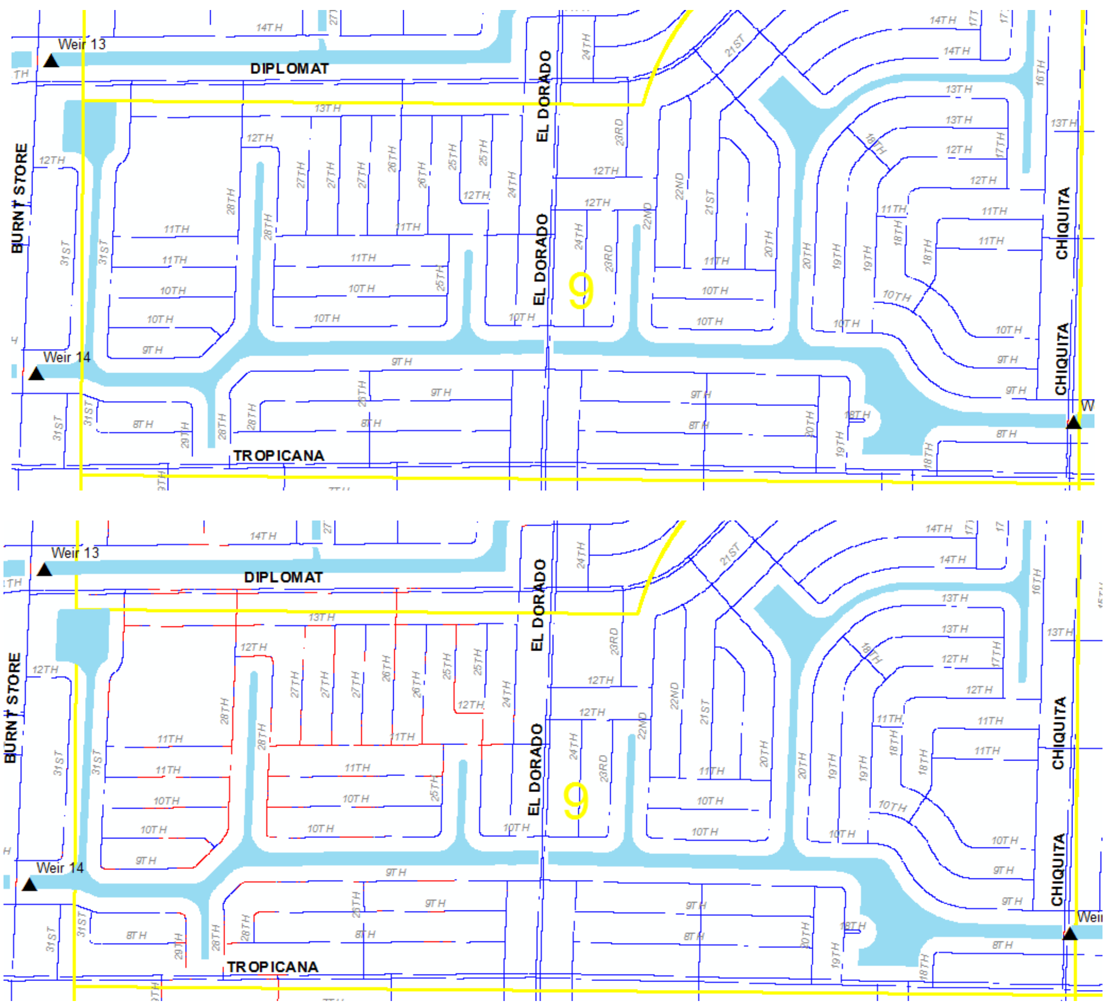


Figure 26. Basin 9 road flooding (red lines) for the "All Gates Open" scenario in the top image and the "Fully Up" scenario on the bottom.

Figure 25 and **Figure 26** show the effect of raising the gates to their fully up (closed) position. Red lines show road flooding along residential streets. This illustrates that even in areas where road flooding is not the most critical, the level of service provided to City residents could be decreased if proper storm event protocols are not followed and structures are left closed.

Table 13. Comparison of flow with the weirs 0.2 ft above the top of the sill or in the closed position during the 5-year Design Storm.

		EXISTING	0.2ft UP		FULLY UP	
Basin	**Weir**	Flow (ac-ft)	Flow (ac-ft)	% Change in Flow	Flow (ac-ft)	% Change in Flow
16	**7**	2,697	2,474	-8%	781	-71%
6	**11**	21,500	21,152	-2%	20,495	-5%
9	**14**	5,789	5,072	-12%	4,478	-23%
7	**13**	12,030	10,905	-9%	11,681	-3%
14	**2**	1,004	981	-2%	2,170	116%
13	**3**	1,321	1,248	-6%	884	-33%
10	**15**	6,763	6,544	-3%	5,928	-12%
Total		**51,105**	**48,375**	**-5%**	**46,416**	**-9%**

Weirs that release flows to tide were compared to determine how much flow reduction can be achieved during a storm event. **Table 13** shows the flows for the existing conditions, the 0.2-ft Up and the Fully Up scenarios (Weirs 1 and 20 were not included as increasing their sill elevations increases flooding as shown in **Table 12**). There was an overall decrease in flows to tide of 5%, or about 2,700 ac-ft, when the weirs were raised 0.2-ft. With the Fully Up scenario the flows to tide decreased by 9%, or about 4,700 ac-ft. In the Fully Up scenario, Weir 2 shows a large increase in flows due to the piling up of water in Basin 4 which can then only release through Weir 2.

Table 14. Suggested gate operations for the City of Cape Coral. Weirs with the greatest potential for water quality storage are listed as fully up (in blue).

Structure	Suggested Operations	Structure	Suggested Operations
Weir 1	Proposed Changes	Weir 14	0.2ft Up
Weir 2	0.2ft Up	Weir 15	0.2ft to Fully Up
Weir 3	0.2ft Up or Less	Weir 16	Fully Up
Weir 4	Static	Weir 17	Fully Up
Weir 6	0.2ft Up	Weir 18	Fully Up
Weir 7	0.2ft Up or Less	Weir 19	0.2-1.0ft Up
Weir 8	0.2ft Up	Weir 20	Proposed Changes
Weir 9	Static	Weir 21	Fully Up
Weir 10	Static	Weir 33	0.2ft Up or Less
Weir 11	Fully Up	Weir 57	Static
Weir 13	0.2ft to Fully Up	Weir 58	Static

6.3. Secondary Flooding Concerns

Figure 20, presented above in Section 5.2.2, indicated secondary flooding in the vicinity of Durdin Parkway east of Andalusia Blvd. There is an inlet north of Durdin Parkway at NE 11th Place that has overflowed during numerous wet seasons. This inlet leads to a number of small culverts that convey flows south to a tributary to Gator Slough. Additional culverts are recommended for high flow periods. Because the conveyance is only needed during periods of high stages in the wetlands north of Durdin Parkway, it is suggest that a secondary inlet structure be constructed that would be limited to operation during the critical high stage conditions.

AIM Engineering, in conjunction with the City, also analyzed selected secondary flooding issues using the Interconnected Channel and Pond Routing (ICPR) hydraulic model software. Three sub-basin areas were selected through review of the findings presented above in Section 5.2.2, as well as collaboration with City staff who have documented known problem areas. The following sub-basin areas were modeled with recommendations made to improve secondary flooding issues:

- Basin 4 - Along NW 2nd Ave. northeast of the Roger Dean Chevrolet dealership.
- Basin 10 – North of Tropicana Pkwy, between NW 26th Ct & NW 24th Place
- Basin 14 – Areas along Hancock Bridge Pkwy, south of the Kia Dealership, west to Cultural Park Blvd.

By inspection of the modeling results, proposed modifications to the existing stormwater system for each sub-basin study area are shown to achieve improved flow and reduced peak stages during large storm events (10-year, 3-day event was utilized in the ICPR analysis). Due to budget constraints, it is anticipated that all modifications cannot be constructed at one time. However, it was shown in the hydraulic analysis that by simply increasing the pipe diameters just upstream of the outfall of each sub-basin, there was a reduction of roadway flooding to a certain extent. The methodology and recommendations of the secondary flooding analysis can be found in the separate Hydraulic Analysis Summary Report.

6.4. Future Build Out Scenario

Future build-out (FBO) conditions of the City of Cape Coral were considered and developed into the model. The scenario was evaluated for flooding potential using the 25 year Design Storm criteria.

The planned future build-out for the City of Cape Coral was developed from the City's Future Land Use Plan, shown in **Figure 27**. Under existing conditions, landuse north of Pine Island Road was classified as low density residential. For the future build-out scenario, the area north of Pine Island Road was assumed to be medium density residential. The Paved Area Coefficient, which is the percentage of impervious surfaces in each cell, for medium density were changed from 0.50 to 0.65, and for high density were changed from 0.75 to 0.80. Manning's number, a measure of the surface roughness, was changed for areas north of Pine Island Road to reflect the increasing area of smoother impervious surfaces throughout Cape Coral.

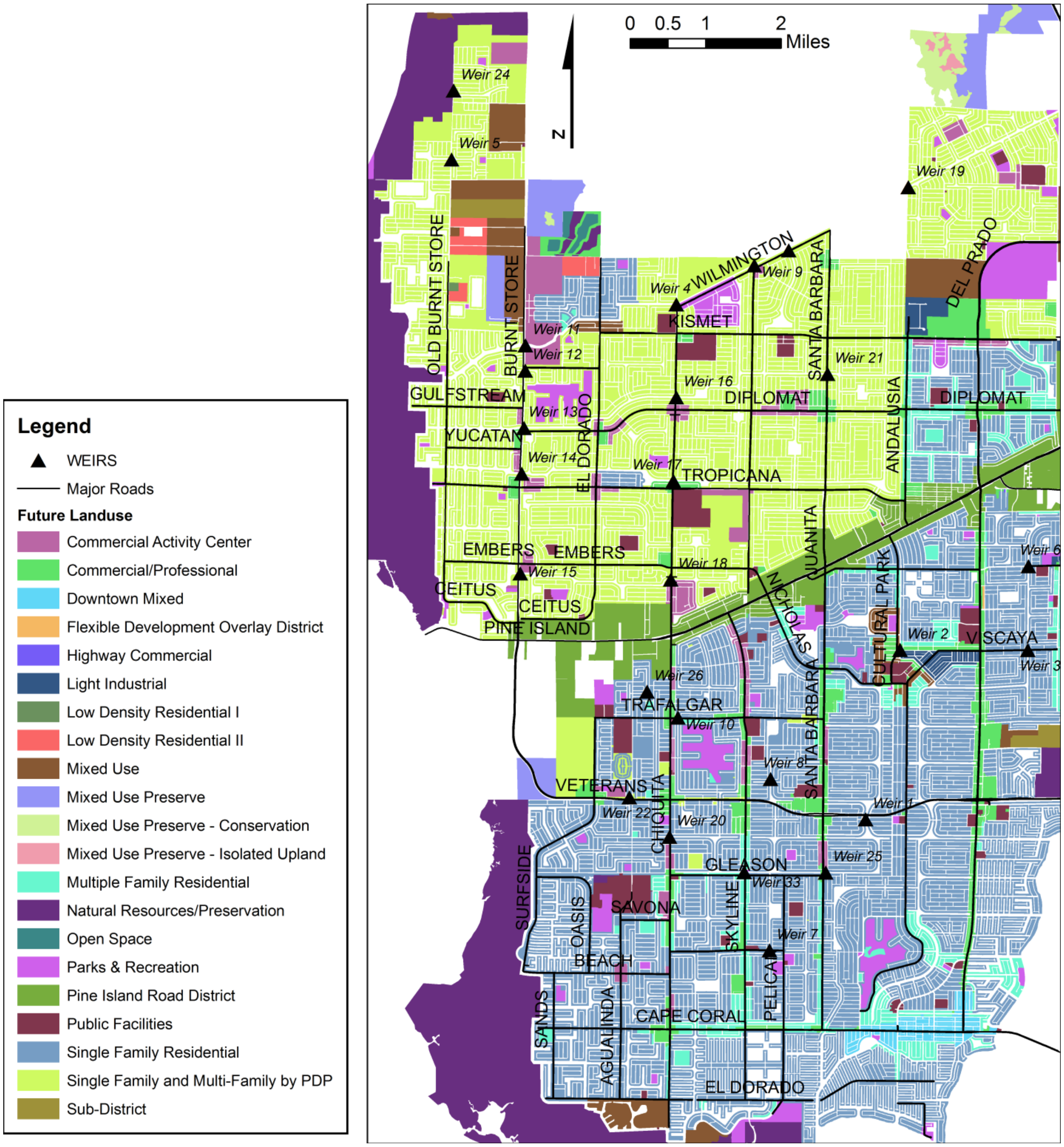


Figure 27. Future Land Use plan for the City of Cape Coral (data obtained from www.capecoral.net)

In addition, the proposed structural and operational changes described in the Midsummer Canal (Weir 29) Structural Improvements Assessment were implemented such that Weir 29 remains at a sill elevation of 9.33ft NGVD and Weir 21 remains at sill elevation of 9.83 during the design storm simulation. All other structures in the City are fully opened (at minimum sill elevation).

Table 15 compares the miles of roads flooded in each Basin for the 25-year Design Storm scenarios of existing buildings with proposed structures, and future build-out with proposed structures.

Table 15. Increase in linear miles of flooded roads for each basin with the Future Build Out 25-year Design Storm simulation.

	Controlling Structure	Increase in Peak Stages (ft)	Increase in Road Flooding (miles)
Basin 1	Weir 19	0.00	0.00
Basin 2	Weir 9	0.00	0.00
Basin 3	Weir 21	0.09	0.00
Basin 4	Weir 16	0.13	0.01
Basin 5	Weir 18	0.81	0.00
Basin 6	Weir 11	0.00	0.00
Basin 7	Weir 13	0.10	0.01
Basin 9	Weir 14	0.24	0.25
Basin 10	Weir 15	0.40	0.00
Basin 12	Weir 6	0.20	0.60
Basin 13	Weir 3	0.17	0.39
Basin 14	Weir 8	0.17	0.07
Basin 15	Weir 1	0.19	0.52
Basin 16	Weir 7	0.30	0.33
Basin 17	Weir 10	0.39	0.16
Basin 18	Weir 20	0.11	0.79

The results of the Full Build-Out scenario, shown in **Table 15**, illustrate that the FBO scenario with the addition of the proposed structures will increase the amount of flooded roads due to the increased imperviousness and smoothness applied over the City. The increase in peak stages and the increase in the linear miles of roadway flooding are considered minor for a future full build-out design. Key basins that show sensitivity to increased imperviousness are Basins 9, 12, 13, 15, 16, and 18. In some Basins, such as Basin 5, the road elevations may be low enough to support additional peak stages before additional road flooding occurs.

6.5.Time to Peak Stage

In the City of Cape Coral, basin size, drainage distribution, and weir flow capacity are key factors that impact the timing of peak runoff during a design storm event. To explore how the water elevations peak in the City, hydrographs were extracted upstream of the major Weirs.

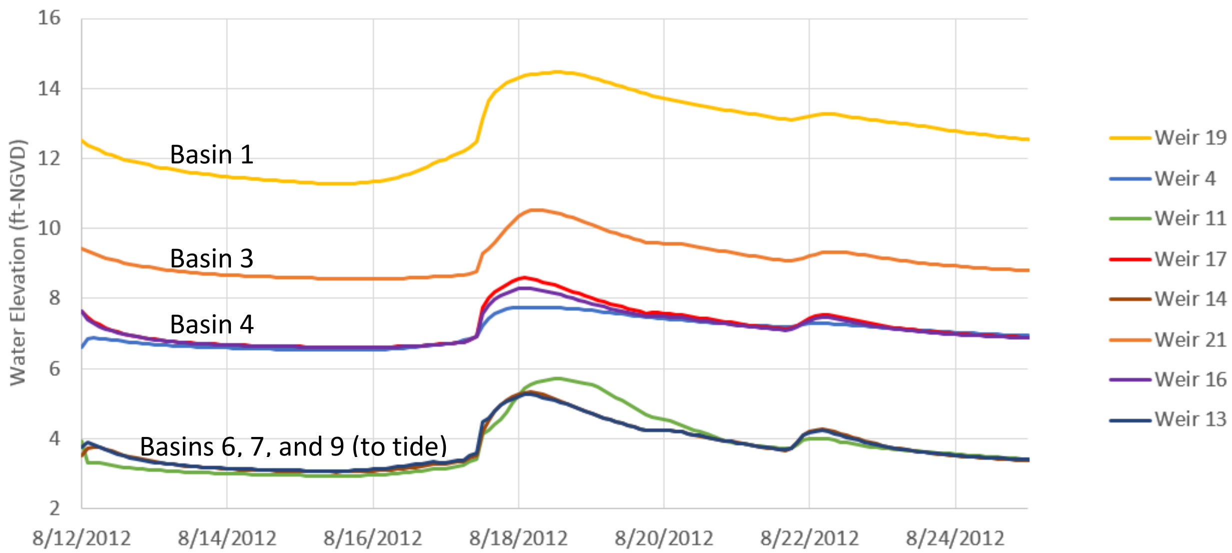


Figure 28.Hydrographs for the Upper Basins of Cape Coral for the first 13 days of the 25yr Design Storm simulation with Existing Conditions.

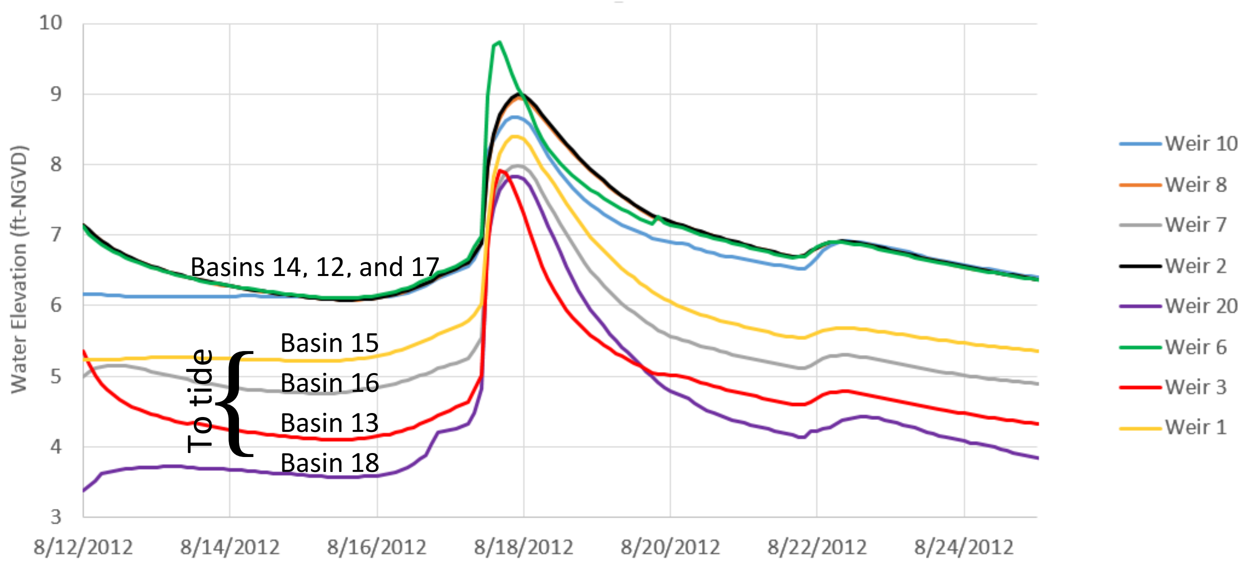


Figure 29. Hydrographs for the Lower Basins of Cape Coral for the first 13 days of the 25yr Design Storm simulation with Existing Conditions.

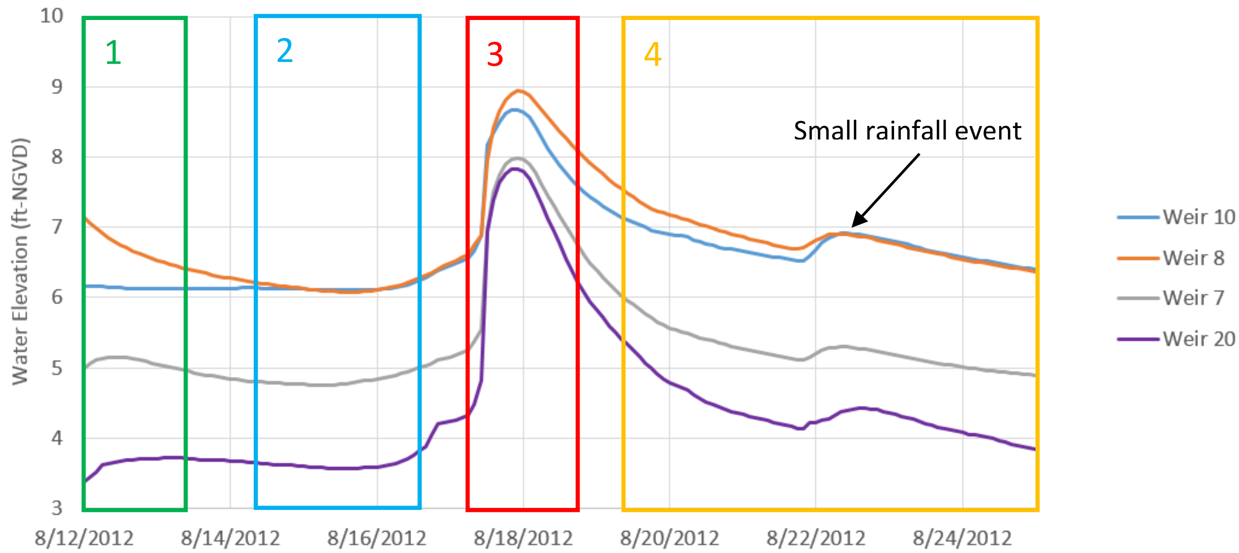


Figure 30. Depiction of the different phases of Basin drainage during a 25 yr Design Storm

There are several phases of the Design Storm drainage that are visible in the hydrograph. **Figure 30** shows the different phases of the Design Storm for some of the lower Basins. Weir 8 represents water levels in Basin 14, Weir 10 represents Basin 17, Weir 7 represents Basin 16, and Weir 20 represents water levels in Basin 18.

1. All gates in the city are opened up in the first hour of 8/12/2012, at which point water begins to drain from Basin 14. Water levels at Weir 7 (Basin 16) and at Weir 20 (Basin 18) show a temporary increase as Basin 14 drains into Basin 16 and then to Basin 18. This temporary increase or "bump" is only seen in receiving basins that are downstream of a large basin in which the sill elevations are lowered all at once.
2. Water levels in each Basin continue to decrease to the lowered sill elevations, with the exception of Weir 10, which is a fixed weir. At all Basins in the City, the lowest sill elevation is reached *after* the initiation of the Design Storm event (8/15/2012). In addition, the Basin does not always have enough time to lower water elevations to the lowest sill elevation, specifically the lowest water elevation for Weirs 20 and 19 were more than 6 inches above the sill elevation before the waters began to increase from the Design Storm. At Weir 20, this indicates that either the capacity of the Weir is too small and cannot remove water from the Basin and from contributing Basins quickly enough. In the case of Weir 19, the drainage area includes Basin 3 and expansive catchment area for Gator Slough (which receives drainage from the U.S.41 ditches, communities and agricultural areas between U.S.41 and I-75, and Cecil Webb Wildlife Preserve).
3. Water levels in each Basin continue to rise until they peak at around 60 to 86 hours after the initiation of the 25yr Design Storm event on 8/15/2012. In **Figure 30**, all of the Basin levels shown peak 70 hours after the initiation. The Design Storm event lasts 72 hours, until the first hour of 8/18/2012 when the rainfall decreases to zero.
4. Finally, water levels begin to taper off in the basins. The shape of the tail of the hydrograph after the peak shows how long drainage takes after a storm event, and may indicate weir flow capacities and drainage lag-time from surface water runoff and upstream basins. The real gridded

rainfall data begins again after 8/18/2012, so the small increase in water levels on 8/22/2012 is due to a small rainfall event.

Table 16. The hours it takes for critical weirs to drain to the lowest stage before an event and raise to the peak stages during the 25 yr Design Storm Event for the Existing and Proposed Conditions.

		Existing Conditions			Proposed Conditions		
Basin	Weir	Hours to Drain	Hours to Peak	Peak minus Trough (ft)	Hours to Drain	Hours to Peak	Peak minus Trough (ft)
1	19	84	86	3.2	84	86	3.2
3	21	94	78	2.0	94	78	1.9
4	4	86	74	1.2	86	74	1.2
4	17	90	74	2.0	90	74	2.0
4	16	90	74	1.7	90	74	1.7
5	18	74	60	1.6	76	60	1.6
6	11	82	84	2.8	82	84	2.8
7	13	82	74	2.2	82	74	2.2
9	14	84	76	2.3	84	76	2.3
10	15	82	64	1.8	82	64	1.8
12	6	82	64	3.6	82	64	3.6
13	3	82	64	3.8	82	64	3.8
14	8	84	70	2.9	84	70	2.9
14	2	86	70	2.9	86	70	2.9
15	1	78	70	3.2	88	70	3.5
16	7	78	70	3.2	78	68	2.8
17	10	88	70	2.6	88	68	2.5
18	20	84	70	4.3	88	66	4.1

The time to reach the lowest stage before a design storm event and the time to reach the peak stage during a design storm event was evaluated for the existing conditions model and the proposed conditions model, in which Weir 1 and Weir 20 have significant modifications (see **Table 16**). The 25-year Design Storm event was used for this analysis. Additionally, the highest stage during the peak of the event was compared with the lowest stage during the initial drawdown. For the majority of the weirs, the general rule is that the longer it takes to reach the event peak, the larger the difference between the peak and the trough; this is seen in Basins 1 and 6 especially. Alternately, basins with a quick time to peak have a smaller peak size, as in Basins 5 and 10. However, Weirs 6 and 3 (Basins 12 and 13, respectively) receive stormwater from very small, highly urbanized, highly drained areas, which creates a large peak in stormwater run-off very quickly. The timing of this high intensity peak closely mimics that of the Design Storm peak shown in **Figure 16**, and then tapers off quickly, lasting only about 6 hours.

When comparing existing conditions with proposed conditions for Weir 20, the lowest stage in Basin 18 was about 0.62 ft lower and the peak elevation was about 0.74 ft lower with the proposed modifications to the weir. For Weir 1, even though the peak minus trough condition increases with the proposed modifications, the lowest stage in Basin 15 was about 1.38 ft lower and the peak elevation was about 1 ft lower with the proposed modifications to the weir. This means that Basin 15 will be able to drain to a much lower elevation with the proposed side structures than without. This gives the Basin ample vertical storage for flood waters before reaching critical road elevations.

Figure 31 illustrates the time to peak stage for each of the critical basins in the City. There is a general difference between how the basins north and south of Pine Island Road drain. With the exception of Basins 5 and 10, which do not receive direct waters from Gator Slough, the basins north of Pine Island road take a longer time to reach the peak stage during a 25 year Design Storm than the basins south of Pine Island Road. This may be attributed to the fact that they receive drainage from large areas, increasing drainage time. Flows in the northern basins are also substantially higher than in the lower basins.

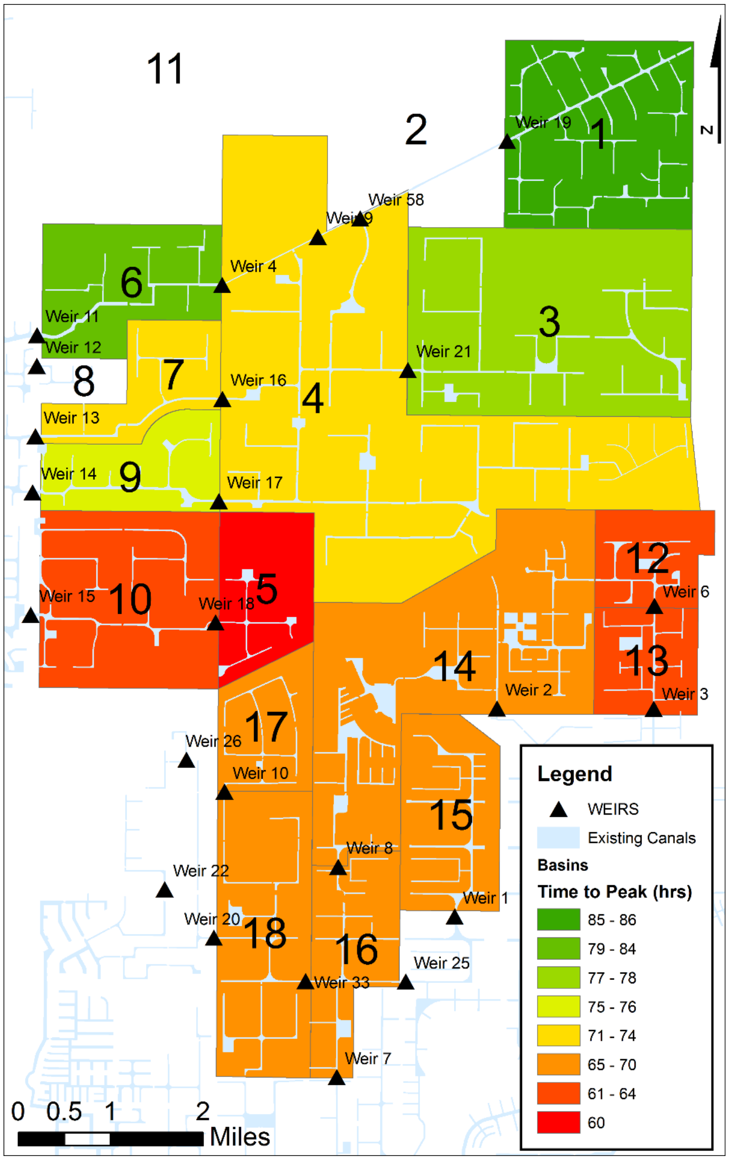


Figure 31. Map of the time to peak water elevation for critical Basins in the City of Cape Coral with existing structures and operations.

With all of the gates in the City lowered to the lowest sill elevation three (3) days prior to the storm event, there is minimal flooding during the 25-year storm event due to the fact that there is adequate basin storage for most of the City. Flooding issues in Basin 18 may be alleviated by giving Basin 14 adequate time to drain out through Basins 16, 18, 12, and 13. Currently flow through Weir 20 may be restricted by undersized culverts under Chiquita Blvd, as discussed in Section 6.1.2. Therefore, flows which go through Weir 8 into Basin 16 and then through Weir 33 into Basin 18 may need more time to drain out before a storm event. It is recommended that Weir 8 is lowered with adequate time before a major storm event for both Basins 15 and 18 to drain to the lowest sill elevation. Further analysis should be performed to determine the exact time that is needed for drainage; however, the initial estimation is between 4 to 6 days of lowered sill elevations.

7. Conclusions

The hydrology of the City of Cape Coral was modeled using MIKE SHE/MIKE11 software. All components of the City of Cape Coral's water usage and cycling was considered in the model. The model was calibrated for the 1/1/2012-12/31/2012 period to around 80% accuracy, and validated for the 1/1/2013-12/24/2013 period with a 60% accuracy. Model accuracy during the 2013 period suffered due to a lack of available measured data describing gate and canal pump station operations.

Design storm analysis shows that the existing City of Cape Coral canal structures are highly effective at channeling stormwater out of the City and reducing flooding for residents. The 2-year, 5-year, 10-year, and 3-inch-1hr design storms produced little to no flooding on roadways throughout the City. For the 25-year design storm, critical road flooding was identified in Basins 15 and 18.

Several scenarios were then developed with modifications to Weirs 1 and 20 to help alleviate flooding in these basins. The final modification suggestions were:

- *i)* to provide a Weir 1 side structure(s) at a lower invert than the weir that can be opened only during storm events,
- *ii)* to increase conveyance in the culverts downstream of Weir 20 by working with the City Planners during future Chiquita Boulevard Improvements, and
- *iii)* to modify the Weir 20 design by adding a gate which can be lowered one foot below existing sill only during storm events.

Secondary flooding improvements are recommended for Durdin Parkway at NE 11th Place (east of Andalusia Blvd.). Also, the separate Hydraulic Analysis Summary Report proposes modifications to the existing stormwater system for the three sub-basin study areas that were evaluated. Improved flow and reduced peak stages were modeled and achieved using ICPR during a 10-year, 3-day storm event. The methodology and recommendations of the secondary flooding analysis can be found in the separate Hydraulic Analysis Summary Report.

Water Quality Storage analysis tested the limits of how high the water control structures can be kept during severe rainfall events. It was determined that Weirs 11, 16, 17, and 21 can be kept at their highest sill elevation during a 25-yr Design Storm Event to decrease flows to tide without greatly increasing flooding in the City.

Scenarios were developed to test the impacts of future increases to City imperviousness due to future build-out. Results suggest that only minor increases in peak stages of less than a foot will occur during the

25 year Design Storms with the implementation of the future percent imperviousness, Manning's roughness, and landuse types. The most sensitive Basin to future development is Basin 18.

8. Recommendations

8.1. Record Keeping Protocol

During model development and calibration, hydraulic data for multiple water control structures and pumps in the City was either complete, intermittent, incomplete, or missing. The most complete dataset was for the Canal Pump Station, which provided daily flow records for each pump station since 2011. Water levels for all non-fixed weirs in the City was provided as intermittent data, at best, and many pump stations had missing data altogether. In some instances, only anecdotal accounts of pumping were available to explain phenomena in the measured data. To help facility any future modeling studies in the City, It is recommended that:

1) gate levels at all of the inflatable Obermeyer weirs (Weirs 1, 6, 7, 11, 13, 14, and 21) are recorded in an electronic database that is updated monthly to bi-monthly,
2) all manual gate changes and flashboard placement and removal is accurately recorded with the date and height of gate/board,
3) accurate "pump on" and "pump off" times and pumping rates are recorded for all pump stations (including pumps 17 and 21, as well as each pump at the North-South transfer station), and
4) all temporary or emergency pumping or water redirection is accurately recorded in terms of flow rates and times.

In addition, any blockages or maintenance interruptions of flow should be recorded, as they occur, with the precise date and description of the event.

8.2. Continued Modeling Development and Analysis

Recent projects have utilized the regional model and continue to improve upon it's calibration as new data is obtained. The following advancements to the model are currently underway:

- Increasing understanding of the structures and tributary canals of Yellow Fever Creek have led to a better characterization of the Yellow Fever Creek basin.
- Improved understanding of the Cecil Webb Wildlife Area and Yucca Pens Preserve has been underway with geological exploration and intensive model calibration for this region.

New water level monitoring stations and record-keeping protocols for gate operations and pump stations will lead to a data-rich 2014-2015 water year. The dataset that continues to develop for the City of Cape Coral could provide additional model inputs and water calibration targets that will contribute to the advancement of the Stormwater Model calibration, and to the understanding of how water management decisions may lead to an improvement of the level-of-service for the City.

In addition, future planning efforts related to water storage and supply should be included in model build-out and future planning scenarios as the planning and design process advances. As Cape Coral continues to develop, it is recommended that the City re-evaluate the changes to land and water usage within each Basin to provide accurate flooding analysis and stormwater design.

9. References

AECOM (2010), North Fort Myers Surface Water Management Plan, Lee County, FL October 2010.

CDM/DHI, Inc. (2006), Southwest Florida Feasibility Study, Tidal Caloosahatchee Basin Modification of Hydrologic Model, Final Report, submitted to US Army Corps of Engineers, July 2006.

DHI, Inc. (2002), Tidal Caloosahatchee Basin Model, Model Calibration and Validation, Submitted by DHI Water and Environment to SFWMD, July 2002.

Pathak, Chandra and Madhav Pandey, 2008. South Florida Environmental Report, Appendix 2-1: Hydrological Monitoring Network of the South Florida Water Management District.

10. Appendix A: Supporting Documents

Pumpage Report

This report must be completed and submitted to the South Florida Water Management District as required by your Permit.

PLEASE COMPLETE ITEMS 1 THRU 7.

1 Permit Number: **36-00998-W**

2 Issued to: **Cape Coral WICC Program (Canal Pump Station #2)**

7 Address: **3920 S.W. 5th Place**

City, State, Zip: **Cape Coral, Florida 33915-0027**

Phone Number: **(239) 574-0782**

3 Recording Period: **As Required by Your Permit**

4 Report Due: **As Required by Your Permit**

5 Month: **November-12**

Day			Day		
1	7,103,783	Gallons	16	2,359,370	Gallons
2	3,107,119	Gallons	17	7,462,597	Gallons
3	5,738,482	Gallons	18	4,355,185	Gallons
4	7,170,043	Gallons	19	2,375,464	Gallons
5	2,468,056	Gallons	20	469,329	Gallons
6	550,087	Gallons	21	4,628,956	Gallons
7	2,610,995	Gallons	22	4,685,111	Gallons
8	4,110,437	Gallons	23	2,737,300	Gallons
9	2,303,625	Gallons	24	4,519,003	Gallons
10	5,040,510	Gallons	25	4,213,788	Gallons
11	5,909,060	Gallons	26	2,141,379	Gallons
12	2,326,366	Gallons	27	443,009	Gallons
13	2,964,444	Gallons	28	4,277,600	Gallons
14	6,134,449	Gallons	29	4,793,456	Gallons
15	5,779,260	Gallons	30	2,714,772	Gallons

TOTAL MONTHLY PUMPAGE: **115,493,036** Gallons

6 Name of Person Completing Form: **Jeffrey Walter**

7 Signature: *Jeffrey Walter* Date: **December 11, 2012**

RETURN TO: South Florida Water Management District
ATTN: Regulation Department / Water Use
P.O. Box 24680
West Palm Beach, Florida 33416-4680

Figure 32. Example of the monthly Pumpage Report for one of the Canal Pump Stations.

11. Appendix B: Calibration and Validation Results

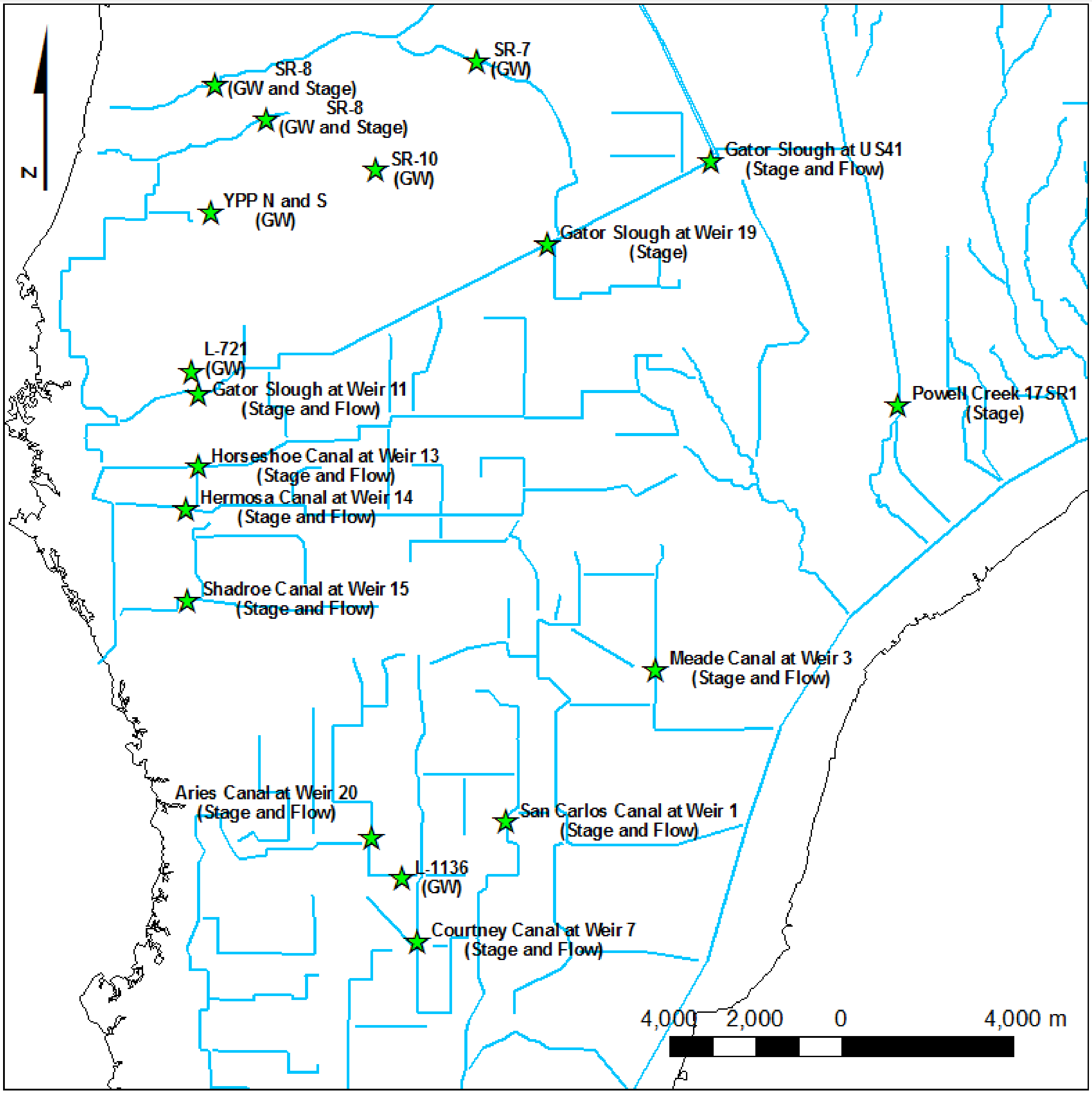
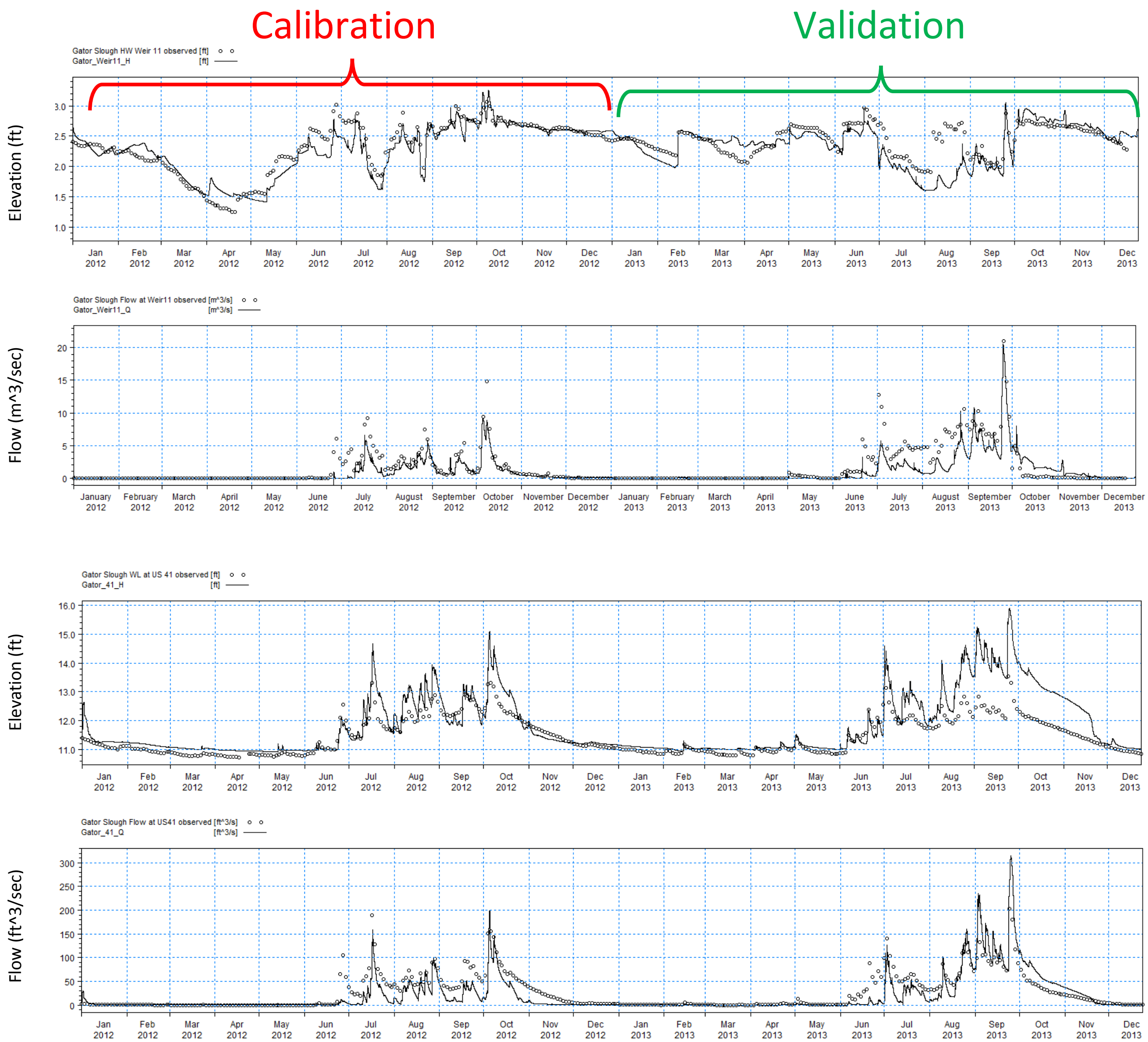
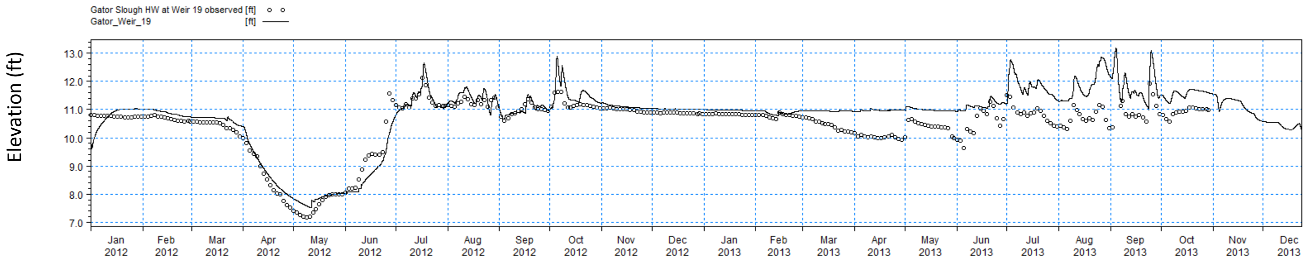
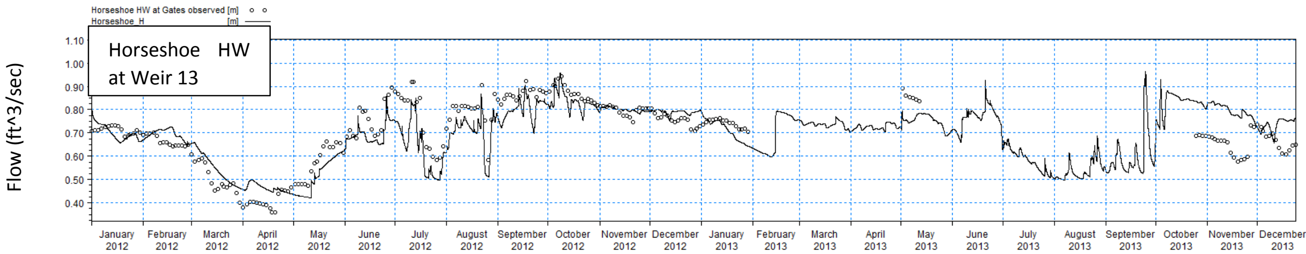


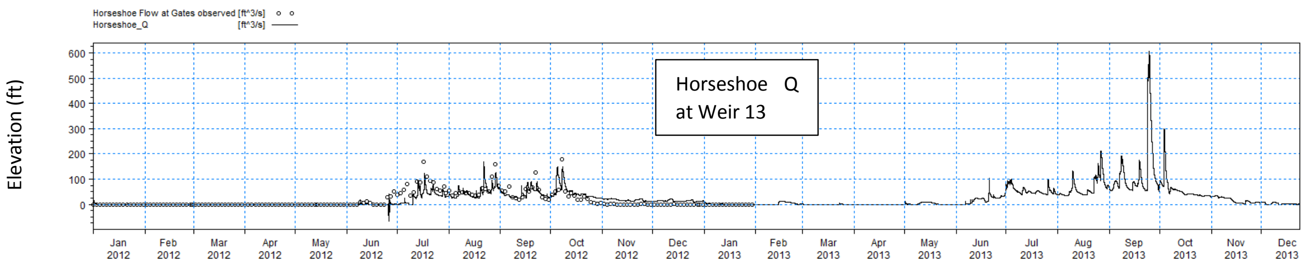
Figure 33. Critical Calibration Points within the City of Cape Coral.

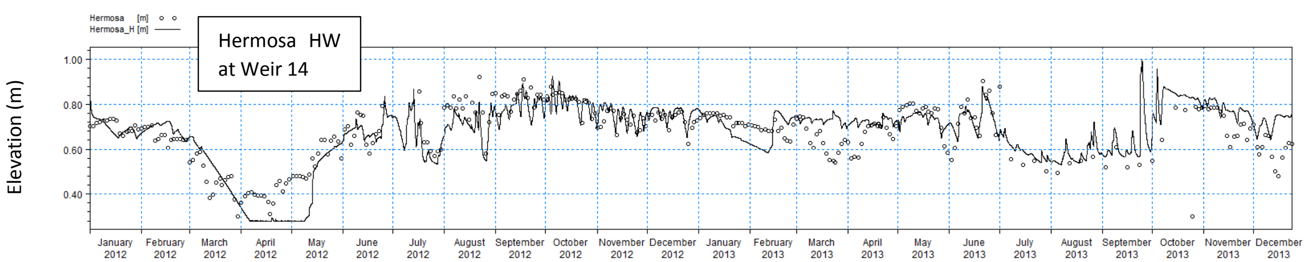
11.1. Surface Water Stations in Cape Coral

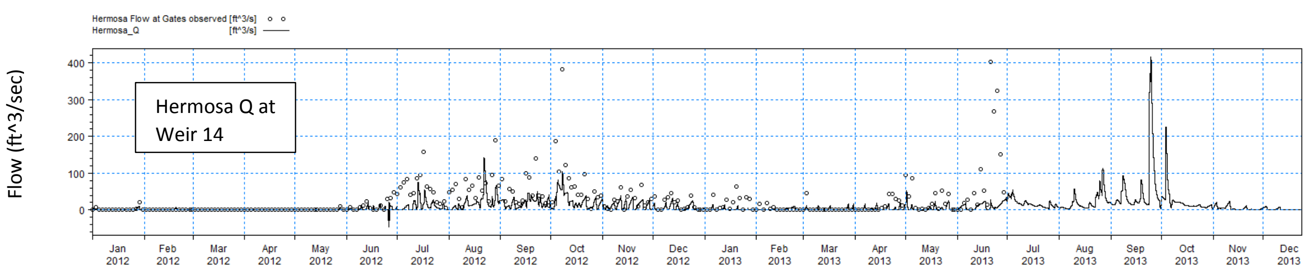










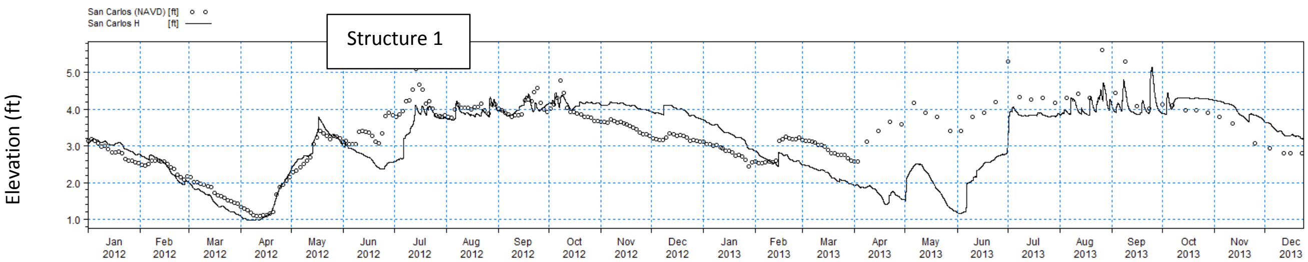


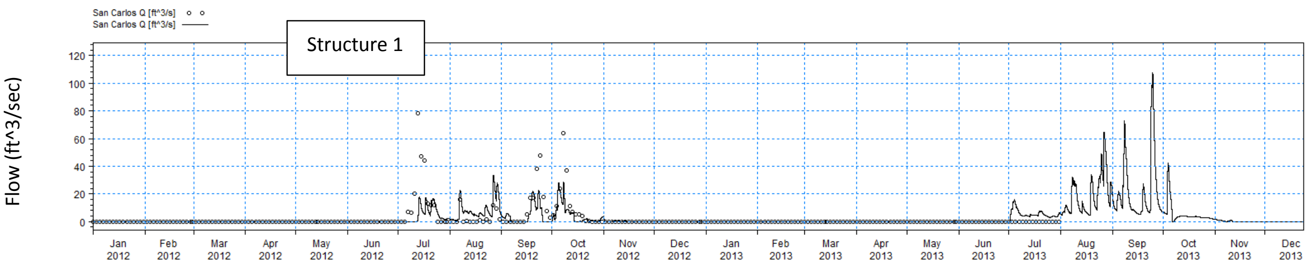
Legend:

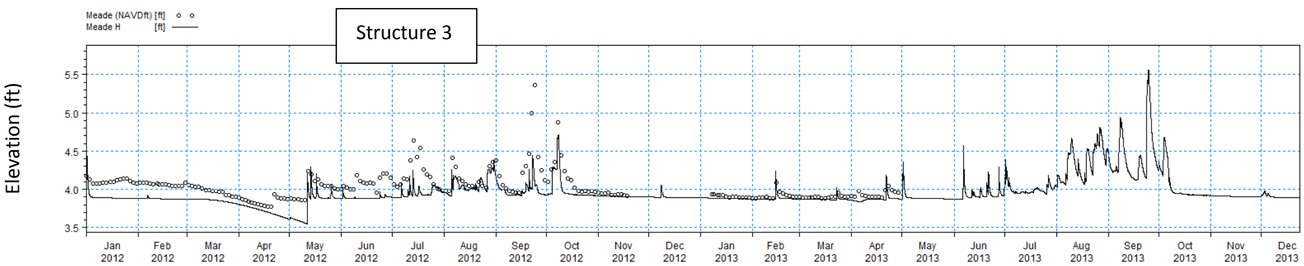
Q = Flow Rate

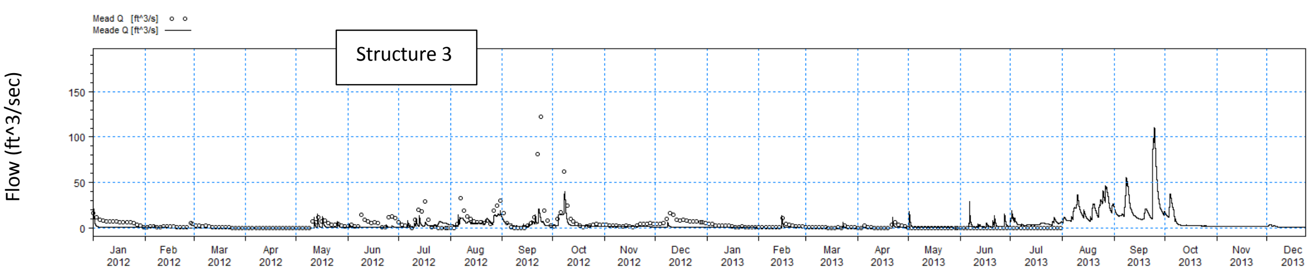
H = Elevation

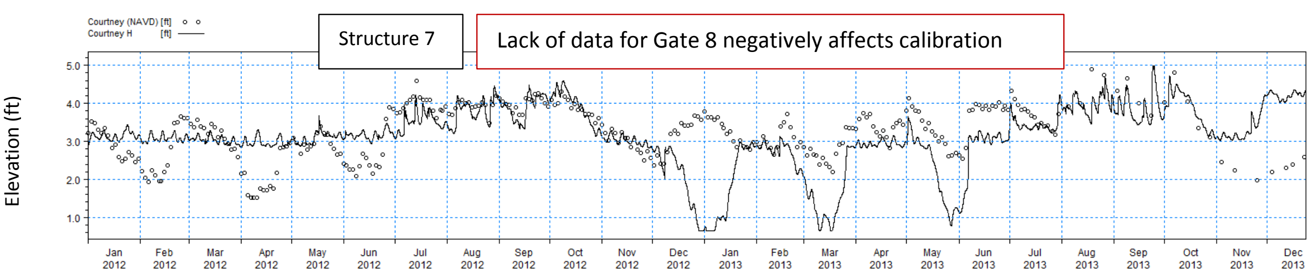
HW = Head Water Elevation







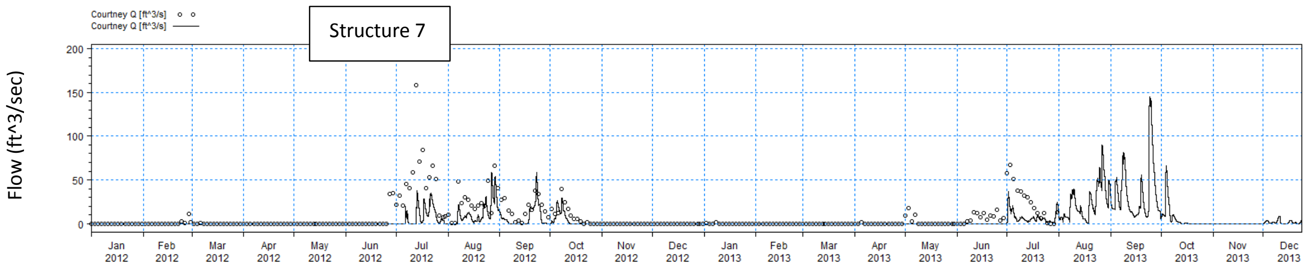


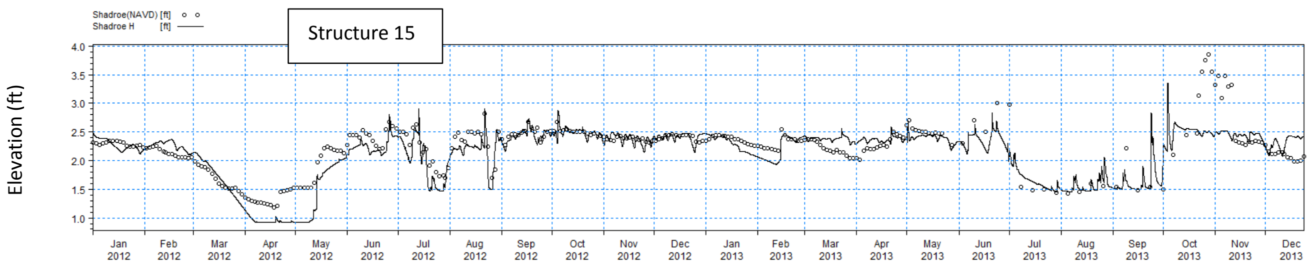


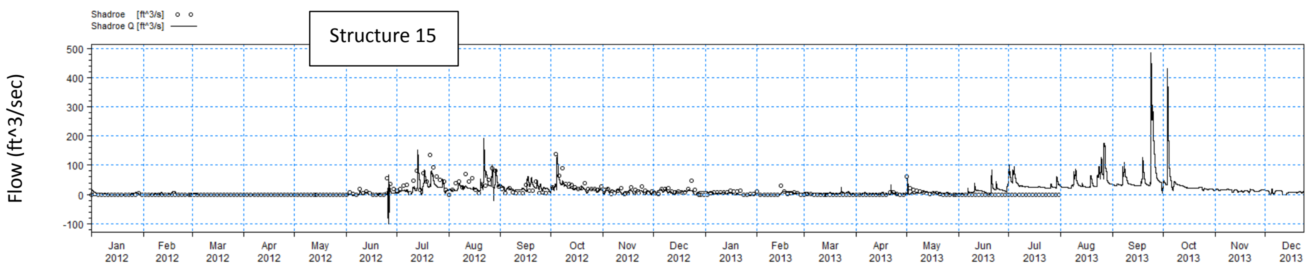
Legend:

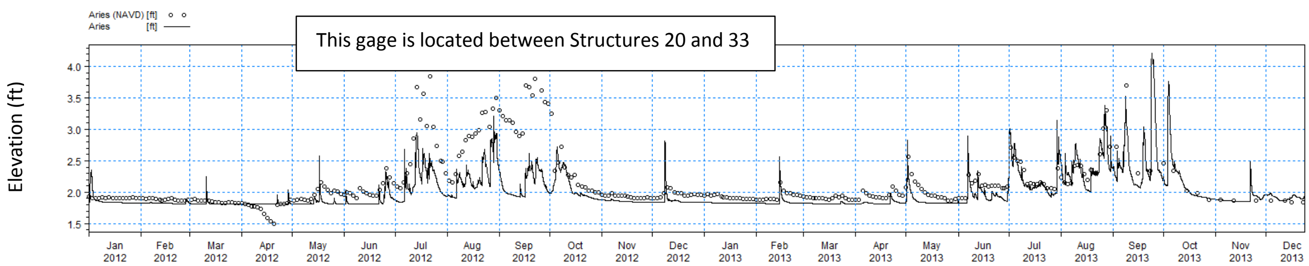
Q = Flow Rate

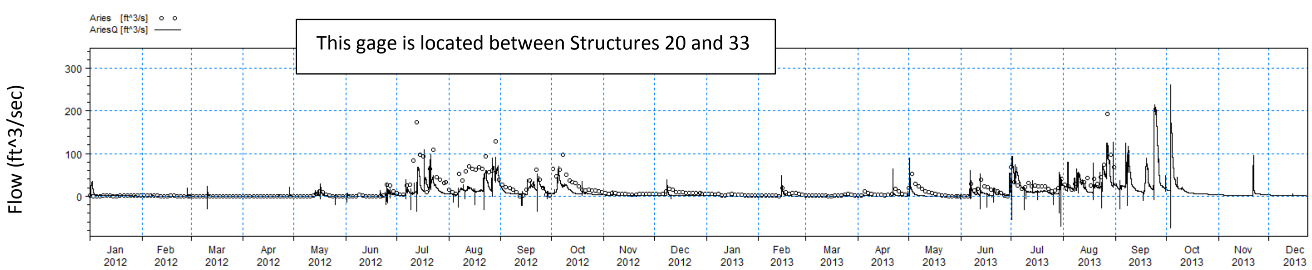
H = Elevation











Legend:

Q = Flow Rate

H = Elevation

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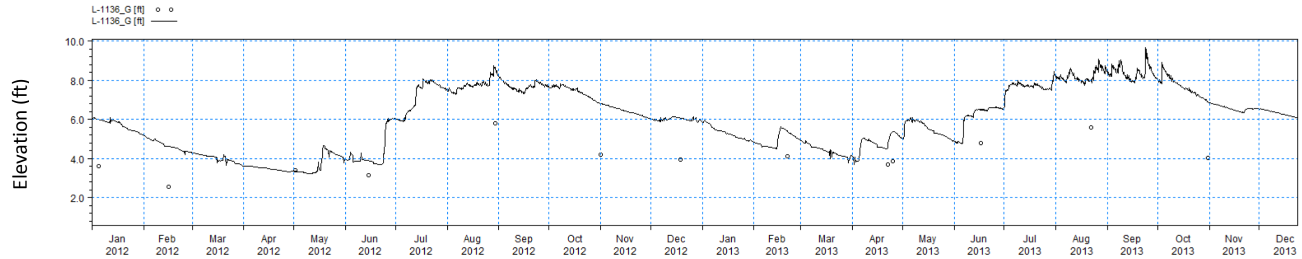
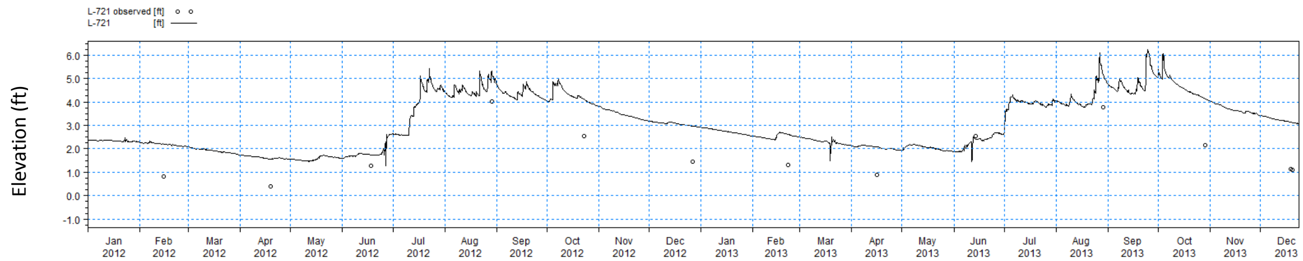
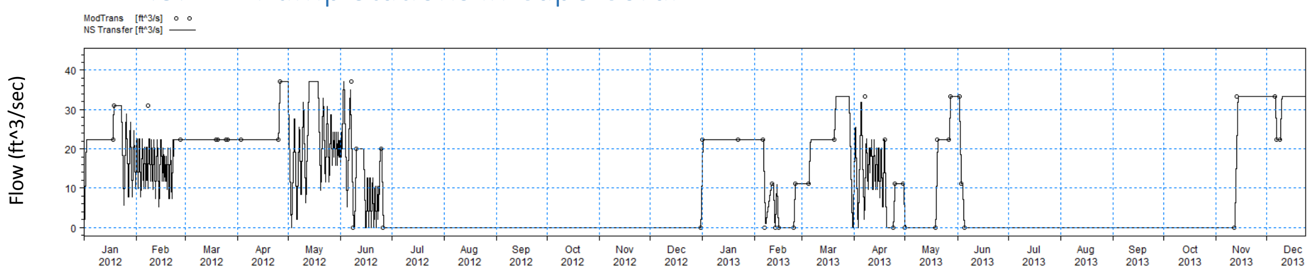
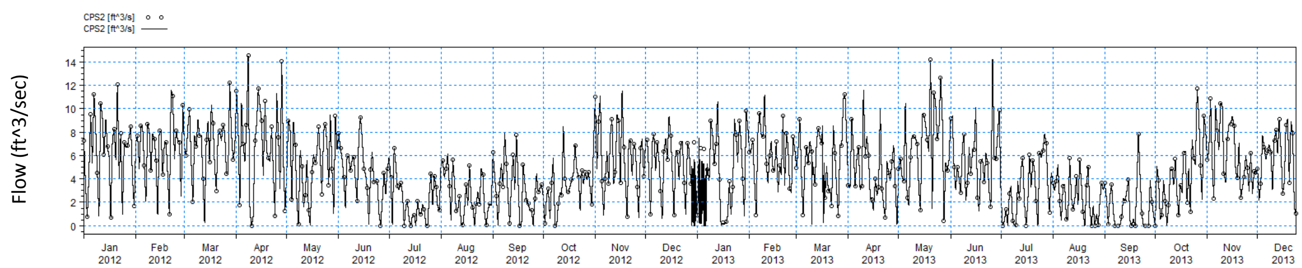
11.2. Groundwater Stations in Cape Coral

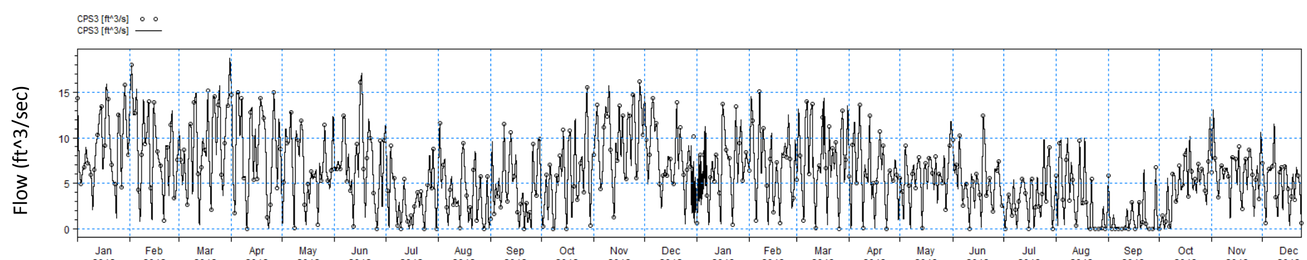


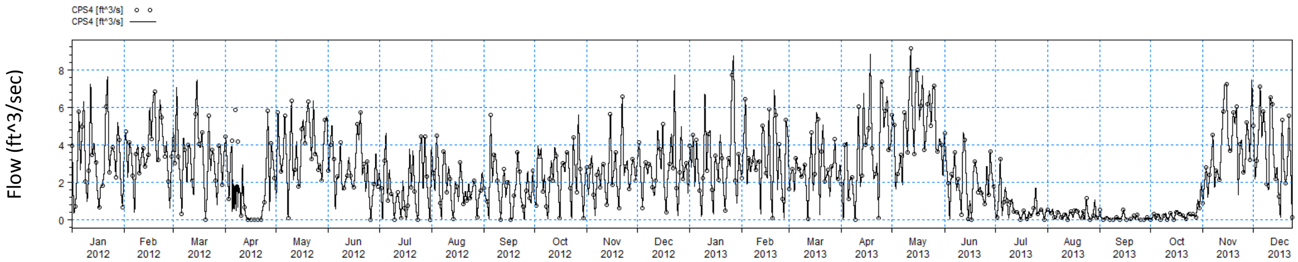


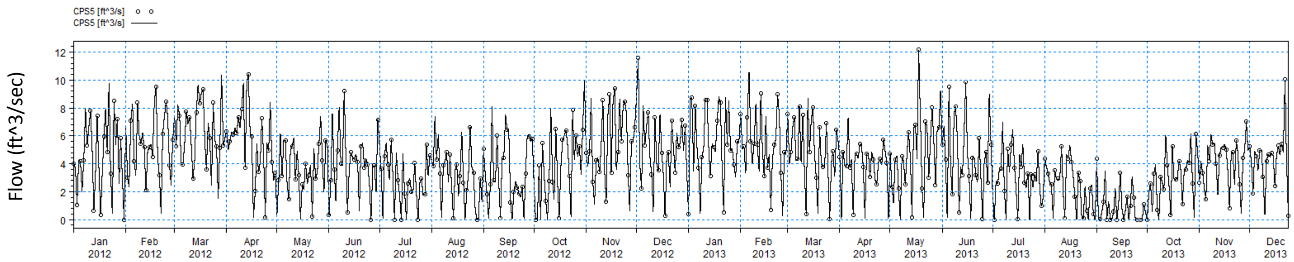
11.3. Pump Stations in Cape Coral

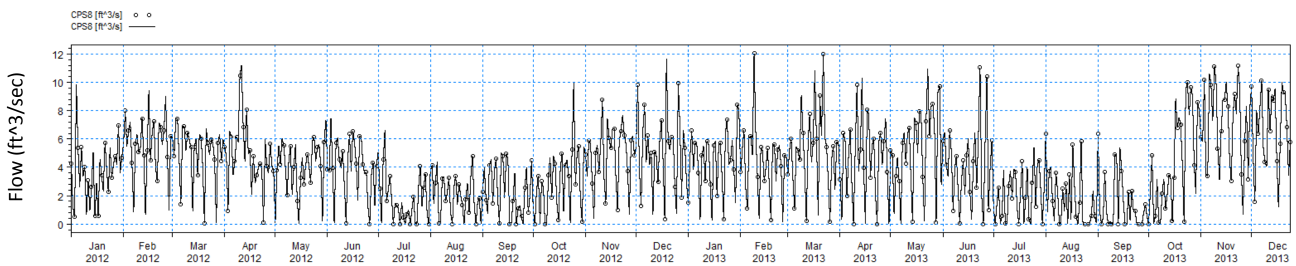




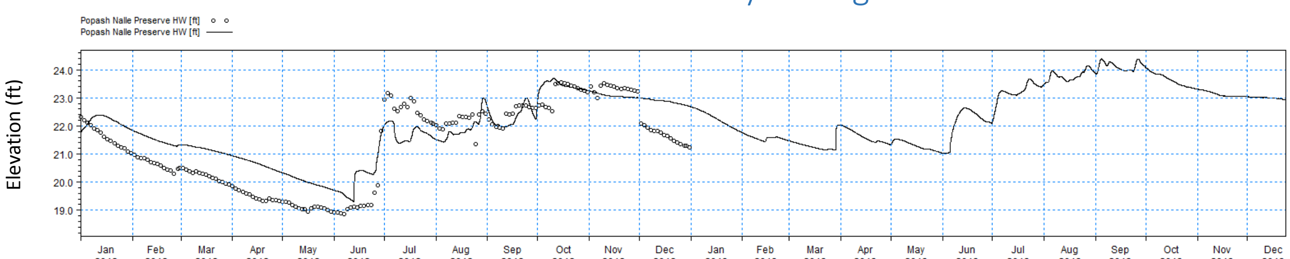


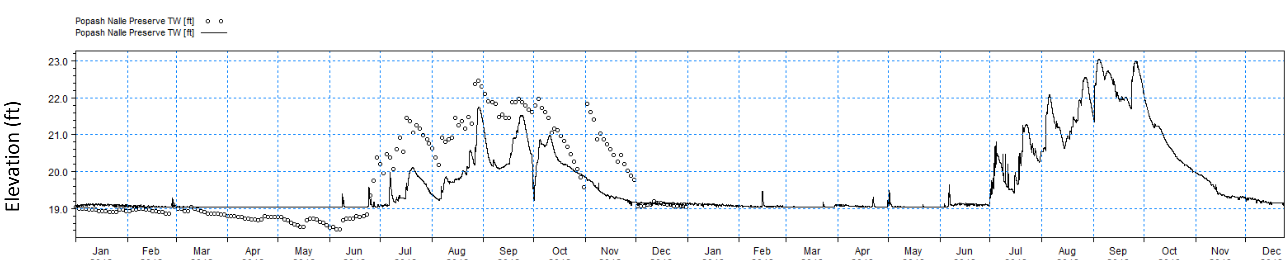






11.4. Surface Waters in the North Ft. Myers Region

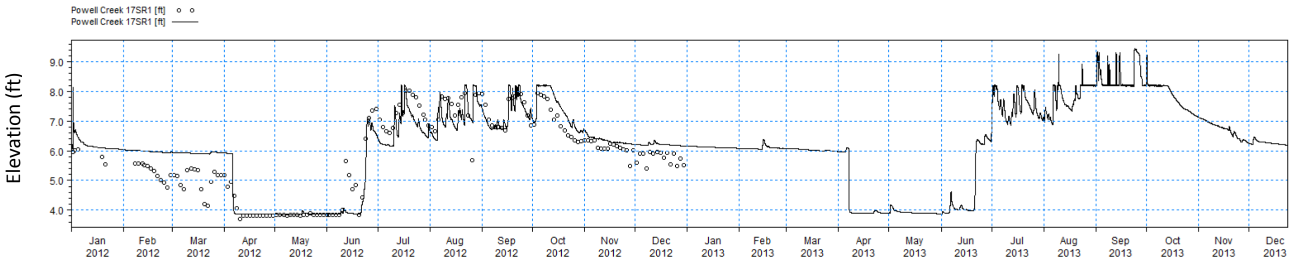


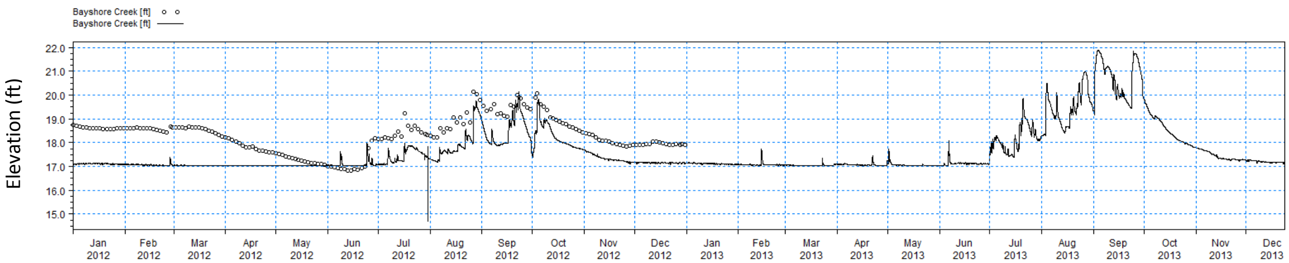


Legend:

TW = Tail Water

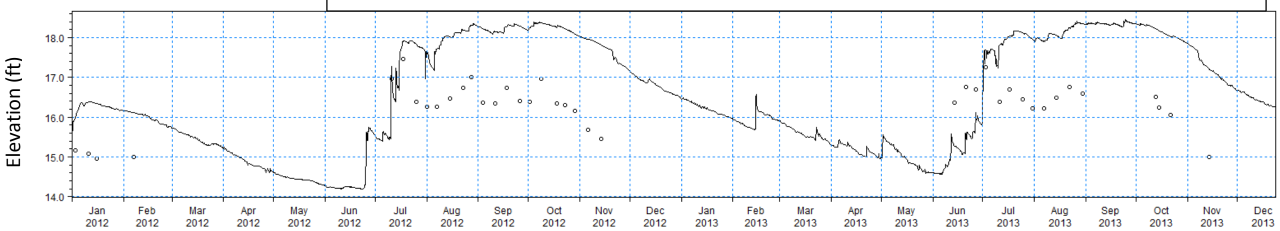
HW = Head Water Elevation

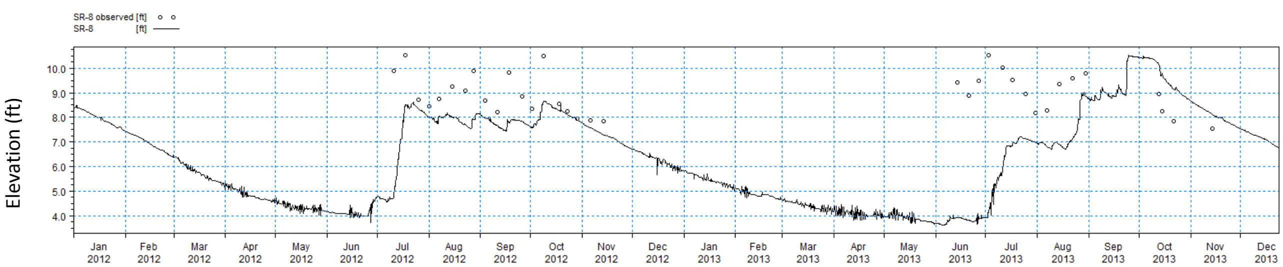


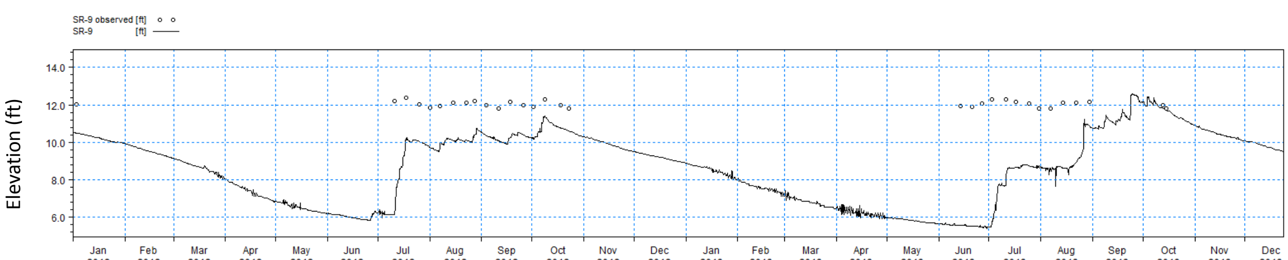


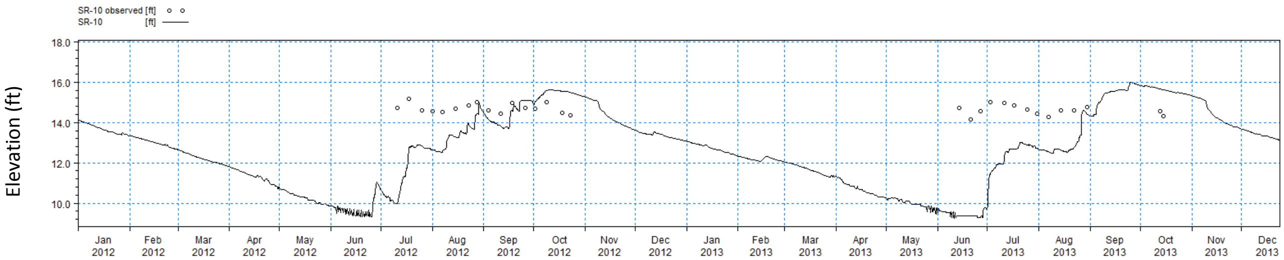
11.5. Yucca Pens

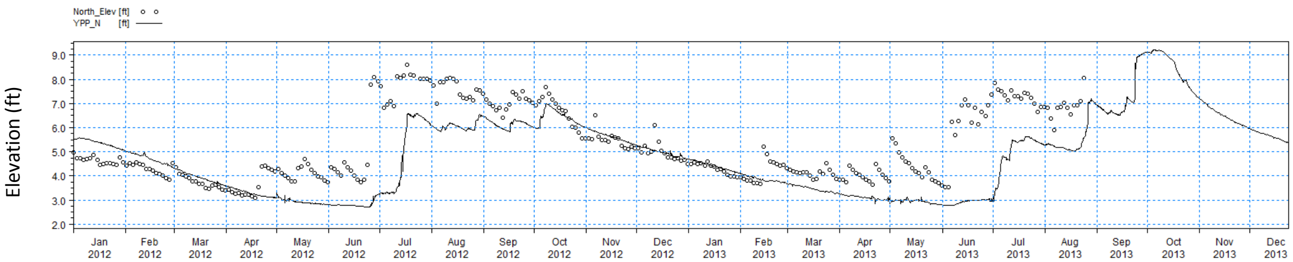
A lack of information on Yucca Pens hydrogeology limits the ability to calibrate water levels

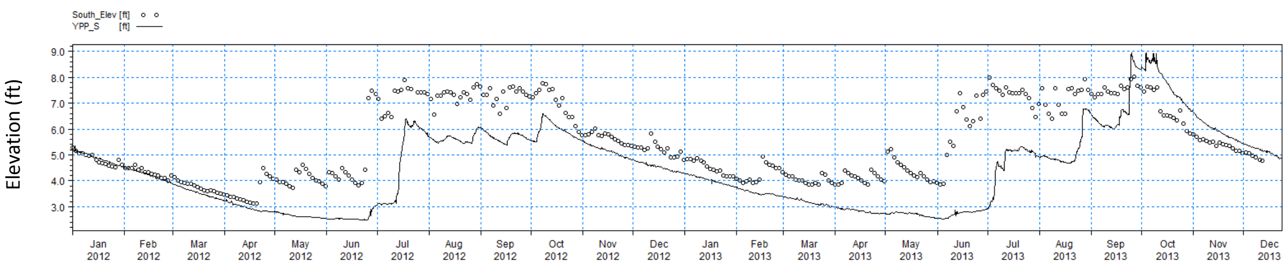


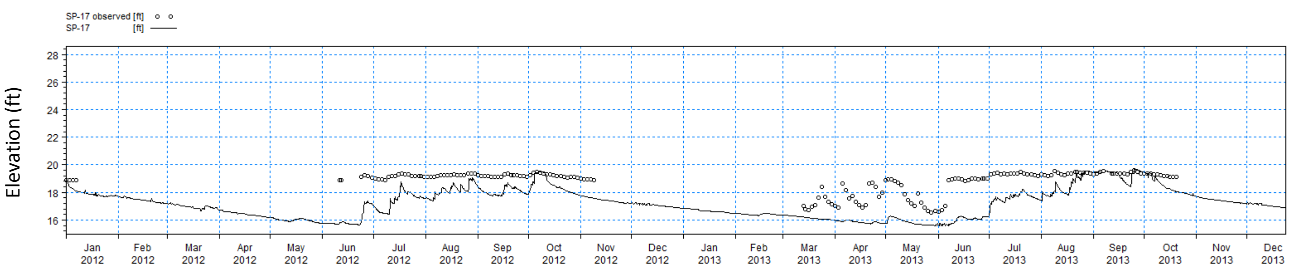


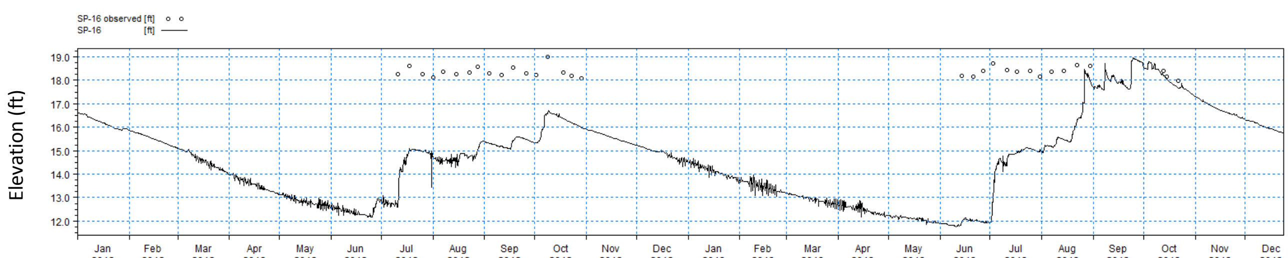


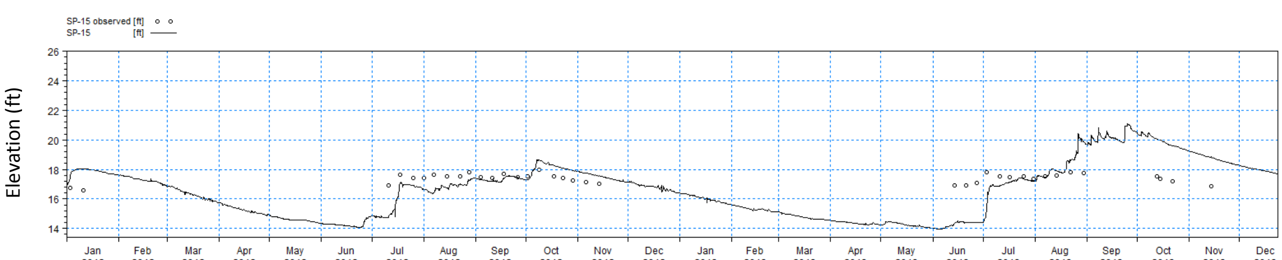

SR-10 observed [ft]
SR-10 [ft]
Elevation (ft)
18.0
16.0
14.0
12.0
10.0
Jan 2012 Feb 2012 Mar 2012 Apr 2012 May 2012 Jun 2012 Jul 2012 Aug 2012 Sep 2012 Oct 2012 Nov 2012 Dec 2012 Jan 2013 Feb 2013 Mar 2013 Apr 2013 May 2013 Jun 2013 Jul 2013 Aug 2013 Sep 2013 Oct 2013 Nov 2013 Dec 2013


North_Elev [ft]
YPP_N [ft]
Elevation (ft)
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
Jan 2012 Feb 2012 Mar 2012 Apr 2012 May 2012 Jun 2012 Jul 2012 Aug 2012 Sep 2012 Oct 2012 Nov 2012 Dec 2012 Jan 2013 Feb 2013 Mar 2013 Apr 2013 May 2013 Jun 2013 Jul 2013 Aug 2013 Sep 2013 Oct 2013 Nov 2013 Dec 2013

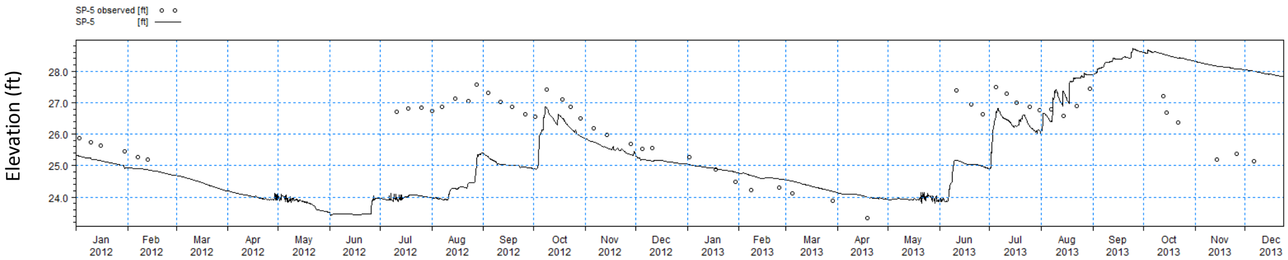

South_Elev [ft]
YPP_S [ft]
Elevation (ft)
9.0
8.0
7.0
6.0
5.0
4.0
3.0
Jan 2012 Feb 2012 Mar 2012 Apr 2012 May 2012 Jun 2012 Jul 2012 Aug 2012 Sep 2012 Oct 2012 Nov 2012 Dec 2012 Jan 2013 Feb 2013 Mar 2013 Apr 2013 May 2013 Jun 2013 Jul 2013 Aug 2013 Sep 2013 Oct 2013 Nov 2013 Dec 2013

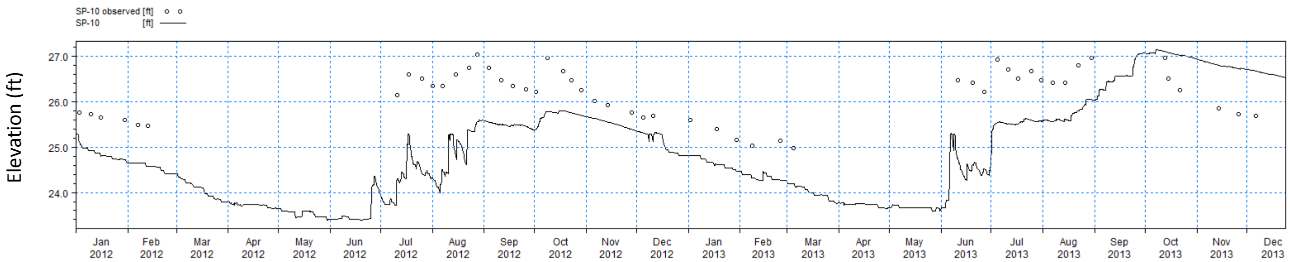

SP-17 observed [ft]
SP-17 [ft]
Elevation (ft)
28
26
24
22
20
18
16
Jan 2012 Feb 2012 Mar 2012 Apr 2012 May 2012 Jun 2012 Jul 2012 Aug 2012 Sep 2012 Oct 2012 Nov 2012 Dec 2012 Jan 2013 Feb 2013 Mar 2013 Apr 2013 May 2013 Jun 2013 Jul 2013 Aug 2013 Sep 2013 Oct 2013 Nov 2013 Dec 2013

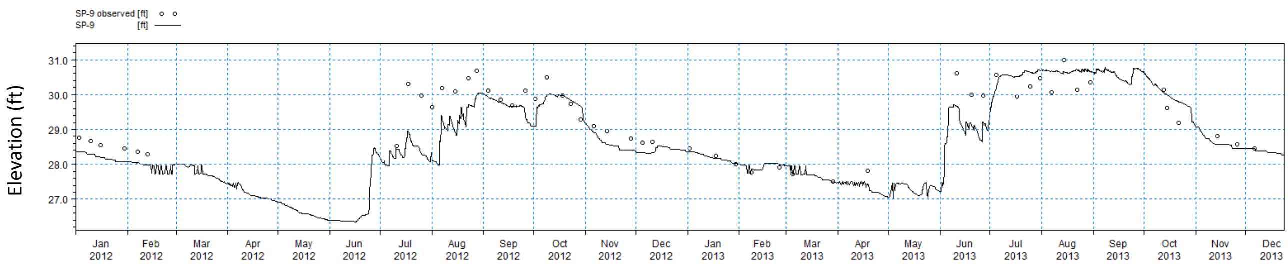

SP-16 observed [ft]
SP-16 [ft]
Elevation (ft)
19.0
18.0
17.0
16.0
15.0
14.0
13.0
12.0
Jan 2012 Feb 2012 Mar 2012 Apr 2012 May 2012 Jun 2012 Jul 2012 Aug 2012 Sep 2012 Oct 2012 Nov 2012 Dec 2012 Jan 2013 Feb 2013 Mar 2013 Apr 2013 May 2013 Jun 2013 Jul 2013 Aug 2013 Sep 2013 Oct 2013 Nov 2013 Dec 2013

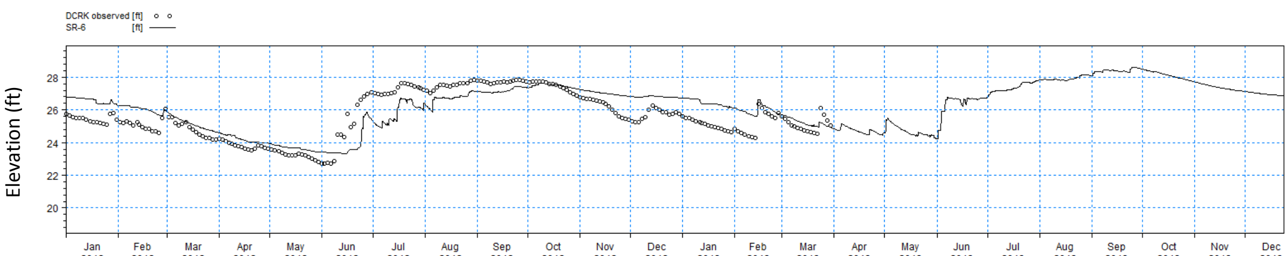

SP-15 observed [ft]
SP-15 [ft]
Elevation (ft)
26
24
22
20
18
16
14
Jan 2012 Feb 2012 Mar 2012 Apr 2012 May 2012 Jun 2012 Jul 2012 Aug 2012 Sep 2012 Oct 2012 Nov 2012 Dec 2012 Jan 2013 Feb 2013 Mar 2013 Apr 2013 May 2013 Jun 2013 Jul 2013 Aug 2013 Sep 2013 Oct 2013 Nov 2013 Dec 2013

11.6. Babcock/Webb Wildlife Management Area









12. Appendix C: Calibration and Validation Statistics

Tables C-1 and C-2 below show the statistics for the calibration period (1/1/2012-12/31/2012) and validation period (1/1/2013-12/31/2013), respectively. ME = mean error; MAE = mean absolute error; RMSE = root mean square error; Std Dev = standard deviation; and R = correlation coefficient. ME, MAE, RMSE, and Std Dev are in feet if the data type is stage and in cfs if the data type is flow.Table C - 1. Calibration statistics for 2012.

Name	Mean Error	Mean Absolute Error	RMSE	STDres	R Correlation
Courtney Canal at Weir 7 HW	0.03	0.46	0.68	0.68	0.48
SR2	0.00	0.12	0.22	0.22	0.98
SP-8	-1.89	1.89	2.28	2.28	0.24
SP-9	-0.93	0.93	1.10	1.10	0.83
SP-13	-2.34	2.34	2.68	2.68	0.69
SR8	-1.32	2.13	2.63	2.63	0.29
SR9	0.73	2.56	4.36	4.36	0.02
Gator_Weir11_H	-0.05	0.13	0.17	0.17	0.93
Gator_41_H	0.37	0.45	1.45	1.45	0.48
Gator_Weir_19	0.10	0.23	0.31	0.31	0.97
Hermosa_H	0.07	0.32	0.62	0.62	0.50
Horseshoe_H	-0.07	0.18	0.22	0.22	0.91
San Carlos H	0.00	0.33	0.45	0.45	0.89
Meade H	0.30	0.60	1.34	1.34	0.12
Shadroe H	-0.11	0.17	0.23	0.23	0.92
Popash Pritchett 23SR1	0.10	0.41	0.52	0.52	0.63
Popash Nalle Preserve HW	0.44	0.73	0.83	0.83	0.89
Powell Creek 17SR1	0.78	1.01	2.20	2.20	0.49
Telegraph Creek	0.42	0.56	0.80	0.80	0.47
Bayshore Creek	-0.79	1.00	1.65	1.65	0.33
Aries	-0.27	0.29	0.48	0.48	0.82
Gator_Weir11_Q	21.31	21.79	49.29	49.29	0.87
Gator_41_Q	-12.49	13.26	21.97	21.97	0.88
Hermosa_Q	-17.64	18.74	35.68	35.68	0.76
Horseshoe_Q	-1.67	11.07	20.09	20.09	0.78
San Carlos Q	-1.23	2.80	8.53	8.53	0.58
Meade Q	-4.28	4.78	12.85	12.85	0.60
Courtney Q	-6.19	6.56	16.04	16.04	0.63
Shadroe Q	-3.32	9.11	18.03	18.03	0.65
Aries Q	-9.48	11.27	20.22	20.22	0.62

Table C - 2. Validation statistics for 2013.

Name	Mean Error	Mean Absolute Error	RMSE	STDres	R Correlation
Courtney Canal at Weir 7 HW	-0.71	0.81	1.07	1.07	0.52
SR2	ND	ND	ND	ND	ND
SP-8	-1.29	1.41	1.75	1.75	0.58
SP-9	-0.61	0.74	0.99	0.99	0.78
SP-13	-0.29	0.51	0.54	0.54	0.96
SR8	-1.97	1.97	2.13	2.13	0.15
SR9	ND	ND	ND	ND	ND
Gator_Weir11_H	-0.06	0.21	0.29	0.29	0.52
Gator_41_H	0.51	0.53	0.80	0.80	0.94
Gator_Weir_19	0.77	0.77	1.46	1.46	-0.05
Hermosa_H	0.11	0.23	0.29	0.29	0.40
Horseshoe_H	1.20	1.27	1.65	1.65	0.31
San Carlos H	-0.29	0.63	0.78	0.78	0.45
Meade H	0.19	0.26	0.94	0.94	-0.06
Shadroe H	-0.06	0.23	0.37	0.37	0.56
Popash Pritchett 23SR1	ND	ND	ND	ND	ND
Popash Nalle Preserve HW	ND	ND	ND	ND	ND
Powell Creek 17SR1	ND	ND	ND	ND	ND
Telegraph Creek	ND	ND	ND	ND	ND
Bayshore Creek	ND	ND	ND	ND	ND
Aries	-0.10	0.13	0.15	0.15	0.90
Gator_Weir11_Q	42.47	42.52	92.24	92.24	0.85
Gator_41_Q	-0.84	13.23	22.92	22.92	0.89
Hermosa_Q	-23.23	25.06	65.00	65.00	0.39
Horseshoe_Q	ND	ND	ND	ND	ND
San Carlos Q	ND	ND	ND	ND	ND
Meade Q	-1.14	1.16	1.83	1.83	0.74
Courtney Q	-3.98	4.17	10.57	10.57	0.71
Shadroe Q	-1.01	4.83	9.33	9.33	0.32
Aries Q	ND	ND	ND	ND	ND