Hydrologic and Hydraulic Modeling Study of Rolling Meadows Wetland Restoration Project Polk County, Florida

By

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DEFINITIONS

Acronyms

CN	Curve Number
CMP	Corrugated Metal Pipe
DEM	Digital Elevation Model
D/S	Downstream
EPA	U.S. Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FLUCCS	Florida Land Use /Cover Classification System
GIS	Geographic Information System
HSG	Hydrologic Soil Group
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NFF	National Flood Frequency
SCS	Soil Conservation Service
SFWMD	South Florida Water Management District
SSURGO	Soil Survey Geographic
SWMM	Storm Water Management Model
USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
U/S	Upstream
ZFI	ZFI Engineering & Construction, Inc.

EXECUTIVE SUMMARY

This report summarizes the hydrologic and hydraulic modeling study for Rolling Meadows Wetland Restoration Project in Polk County, Florida. The study consists of the followings:

- model construction using the U.S. EPA SWMM software;
- model hydrology validation using the USGS regional regression equation and model hydraulics validation using limited measured flow data obtained from the stream-gauging activities;
- model application to evaluate the post-improvement surface water management system flows and stages based on the 25-year and 100-year design storm events.

In comparison to the pre-improvement system, the post-improvement provides the following benefits:

- significantly alleviates flooding impacts in the upstream channels between Site 1 and Site 2;
- increases system conveyance capacity;
- significantly reduces the maximum water surface elevations in Parcel B, and hence alleviates flooding risk to the area surrounding Parcel B.

In contrast to the previous hydrologic and hydraulic study of the project (ZFI, 2011), the present study demonstrates that it does not appear to be necessary to increase the Parcel B perimeter berm elevation to prevent the "Southeast Developed Area" from flooding. The flood risk in this area is not due to high water stages in Parcel B, but rather due to high water stages in the C37 Canal as well as in the Kissimmee River.

1.0 INTRODUCTION

1.1 Project Location

The Rolling Meadows project area is located in eastern Polk County, Florida, south of Lake Hatchineha and west of the Kissimmee River, in Sections 2, 3, 4, 5, 6, 8, and 9, Township 29 South, and Range 29 East, as depicted in **Figure 1**. The area is part of the Lake Kissimmee watershed. The Rolling Meadows project area consists of three parts: The Catfish Creek watershed with a drainage area of approximately 3,736 acres; Parcel B with a drainage area of approximately 1,777 acres; and the Parcel B Surrounding Area consisting of approximately 1,497 acres that drains to Parcel B by gravity. The boundary of the three parts was defined using the latest GIS data that includes one foot contour lines and DEM layer with 5 ft resolution derived from Polk County LiDAR data. **Figure 1** shows these three parts, in which the Parcel B Surrounding Area is shown as a shaded area.

Figure 1. Location Map of Rolling Meadows Project Area

1.2 Project Background

The total drainage area of the Rolling Meadows Restoration Project is approximately 7,010 acres, through which Catfish Creek drains from Lake Pierce to Lake Hatchineha. Parcel B encompasses approximately 1,777 acres, of which approximately 1,600 acres of former sod fields are proposed to be restored back to wetlands as a result of this restoration project. The proposed restoration plan is basically as follows:

The first step of the wetland restoration, which has already occurred, is to route flow from Catfish Creek into Parcel B through the existing breach in the perimeter berm at Site 6. The breach was created as a result of the culvert failure at Site 7, which blocked flows from Catfish Creek to Lake Hatchineha. Flow from Catfish Creek currently goes into Parcel B by gravity through the existing breach. Parcel B exists as a low tract of previously farmed land (sod farm) that has minor contour changes except for the existing farmer irrigation ditches and the existing perimeter berm. Waters from Catfish Creek naturally pools in Parcel B.

The next step of the restoration plan is to improve the connection between Parcel B and Lake Hatchineha. Previously, the perimeter berm was constructed to separate Parcel B from the lake so that the parcel could be farmed. Prior to the construction of the perimeter berm, Parcel B was part of the littoral zone of Lake Hatchineha. The plan is to re-develop Parcel B as a wetland system and to provide for connectivity between the lake and the parcel. This will be accomplished by replacing the existing 48” diameter culvert with 4-72” diameter culverts with 8’ wide x 9’ high, manually operated, double-leaf slide gates at Site 11. This will provide connectivity between Parcel B and Lake Hatchineha, and a means to control the water level in Parcel B, based on the stage of Lake Hatchineha.

The existing hydraulic control structures at the other sites along Catfish Creek will also be replaced to improve the system conveyance capacity and flood control since these existing structures have deteriorated and have lost their design conveyance capacity and flood control function.

1.3 Modeling Objectives

This hydrologic and hydraulic modeling study is a basis to assist in the design of Rolling Meadows Wetland Restoration Project. The model results will be used to size hydraulic structures for the project, and to evaluate the pre- and post-improvement effects of both the 25- and 100-year 3-day design storm events. The model results are also as a basis to draft an operation strategy for the system.

2.0 HYDROLOGIC AND HYDRAULIC MODEL PRINCIPLE

We chose to use the SWMM (version 5.0), developed by U.S. EPA (see website: http://www.epa.gov/nrmrl/wswrd/wq/models/swmm), to represent the hydrologic and hydraulic processes of the project area. The SWMM is an integrated model that couples hydrologic processes over a watershed and hydraulic routing processes through a flow collection/transmission system. The SWMM model calculates runoff volume first and then routes the runoff over subcatchment areas to generate runoff hydrographs. It then applies the runoff hydrographs to the flow collection/transmission system for hydraulic routing, as diagrammed in **Figure 2**.

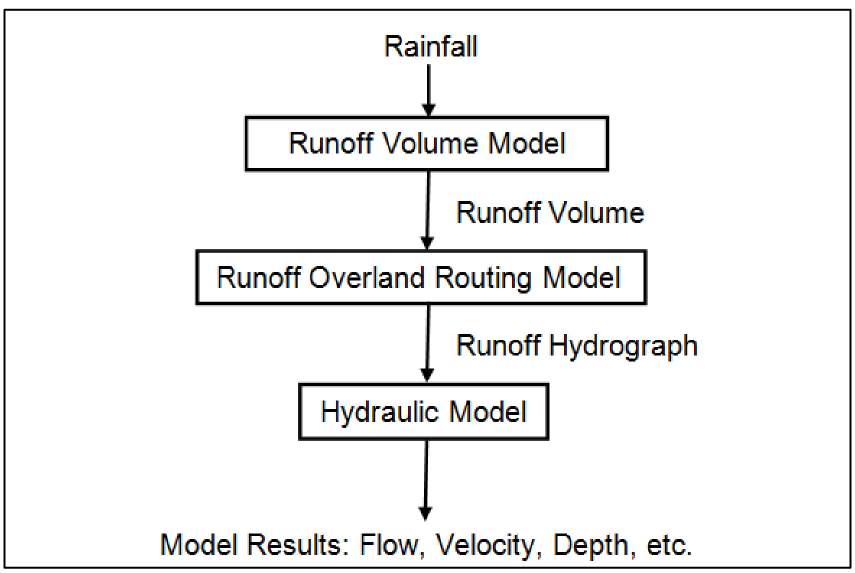

Figure 2. Schematic of the SWMM Hydrological and Hydraulic Model

2.1 Runoff Volume Model

The runoff volume model determines how much of rainfall runs off a catchment area into its drainage system after accounting for any initial losses. The SWMM provides three infiltration methods to account for initial losses: Horton, Green-Ampt and SCS Curve Number (CN) methods. We used the SCS CN method to estimate the runoff volume. During significant storm events such as the 25-year and 100-year 3-day storm events, it is reasonable to assume that evaporation/evapotranspiration losses are negligible and can be ignored. The following equation gives the depth of rainfall excess or direct runoff from a given storm:

$$P_e = \frac{(P-I_a)^2}{P-I_a+S} \tag{1a}$$

$$S = \frac{1000}{CN} - 10 \tag{1b}$$

Where

P_e: Rainfall excess or direct runoff (inches);

P: Total rainfall depth (inches);

I_e: Initial abstraction or initial loss before ponding (inches);

S: Potential maximum retention storage (inches);

CN: Runoff Curve Number.

The SCS CN is determined based on land use type and hydrologic soil group. Equation (1b) indicates that subcatchment area retention is directly related to the CN, which in turn affects the rainfall excess in Equation (1a).

2.2 Surface Runoff Routing Model

The surface runoff routing model determines how quickly the excess rainfall (P_e) enters the drainage system from a subcatchment area. **Figure 3** illustrates the conceptual view of the SWMM surface runoff routing model.

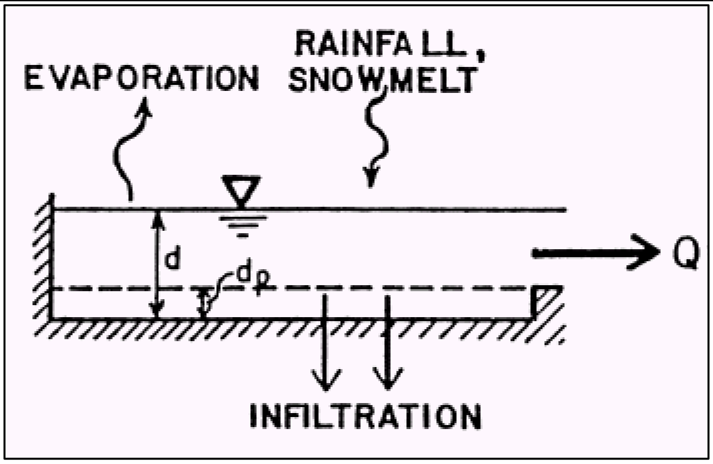

Figure 3. Schematic of the SWMM Surface Runoff Routing Model

In the model, each subcatchment surface is treated as a nonlinear reservoir, in which its inflow comes directly from precipitation. The capacity of a "reservoir" is the maximum subcatchment depression storage, which is the maximum surface storage volume provided by ponding, surface wetting, and interception. Surface runoff from a subcatchment area occurs only when the volume in the "reservoir" exceeds the maximum depression storage. The surface runoff over the subcatchment surface is given by Manning's equation:

$$Q = \frac{1.49}{n} W (d - d_{\max})^{5/3} S_b^{\ 0.5} \qquad (2)$$

where

Q: Surface runoff (cfs);

W: Subcatchment characteristic width (feet);

S_b: Subcatchment slope (%);

d: Water depth in the “reservoir” (feet);

d_{max}: Depression storage depth of the “reservoir” (feet).

n: Manning’s roughness coefficient.

Equation (2) indicates that the subcatchment area’s width and slope, and Manning’s roughness of ground surface are major factors affecting the surface runoff hydrograph from a subcatchment area.

2.3 Hydraulic Routing Model

For flow routing through the surface water collection system, the SWMM provides three flow routing model options: stable flow, kinematic wave, and dynamic wave routing. This provides a flexibility to select a suitable flow routing model according to the complexity of the system. The dynamic wave routing model can account for channel storage, backwater effects, entrance and exit losses, flow reversal, and pressurized flow. Since dynamic wave routing couples the solution for both water levels at nodes and flow in conduits, it can be applied to any general network layout, even those containing multiple downstream diversions and loops. It is the method of choice for systems subject to significant backwater effects due to downstream flow restrictions and with flow regulation through weirs and orifices. Therefore, we selected the dynamic wave flow routing model for this analysis.

2.3.1 Conveyance System

The SWMM's flow routing/transmission system contains a network of conveyance elements (channels, pipes, pumps, and regulators) and storage/treatment units that convey runoff to outfalls or to treatment facilities. Inflows to the conveyance system can come from surface runoff, groundwater interflow, dry weather flow, or from user-defined hydrographs. The components of the SWMM conveyance system are modeled with node and link objects. Nodes are points of a conveyance system that connect conveyance links together. The following are several types of nodes that can be employed:

- **Junctions** are drainage system nodes where links join together. Physically they can represent the confluence of natural surface channels, manholes in a sewer system, or pipe connections. External inflows can enter the system at junctions.
- **Outfalls** are terminal nodes of the drainage system. They are used to define final downstream boundaries under the dynamic wave flow routing. For other types of flow routing they behave as a junction.
- **Flow Dividers** are drainage system nodes that divert inflows to a specific conduit in a prescribed manner. A flow divider can have no more than two conduit links

on its discharge side. Flow dividers are only active under the kinematic wave routing and are treated as simple junctions under dynamic wave routing.

- **Storage Units** are drainage system nodes that provide storage volume. Physically they could represent storage facilities as small as a stormwater catch basin or as large as a lake. The volumetric properties of a storage unit are described by a function or table of surface area versus storage height.

Links are the conveyance components of a drainage system and always lie between a pair of nodes. Types of links include:

- **Conduits** are pipes or channels that move water from one node to another in the conveyance system. Their cross-sectional shapes can be selected from a variety of standard open and closed geometries. Irregular natural cross-section shapes are also supported, as are user-defined closed shapes.
- **Pumps** are links used to lift water to a higher elevation. A pump curve describes the relation between a pump's flow rate and conditions at its inlet and outlet nodes.
- **Flow Regulators** are structures or devices such as orifices, weirs, and outlets, used to control and divert flows within a conveyance system. They are typically used to:
    - Control releases from storage facilities
    - Prevent unacceptable surcharging
    - Divert flow to treatment facilities and interceptors

2.3.2 Hydraulic Routing Principle

We employed the dynamic wave routing method to route flow through the system. As previously noted, the dynamic wave routing can account for channel storage, entrance and exit losses, flow reversal, and pressurized flow, and is suitable for representing a system subject to significant backwater effects due to downstream flow restrictions and with flow regulation via weirs and orifices.

The dynamic wave routing solves the complete one-dimensional Saint Venant flow equations and can generally produce the most theoretically accurate results. These equations consist of the continuity and momentum equations for conduits and a volume continuity equation at nodes. The Saint-Venant continuity and momentum equations are given below:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{3a}$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\frac{\partial y}{\partial x} - gA\left(S_0 - S_f\right) = 0 \tag{3b}$$

Where

- Q: Discharge rate (cfs);
- A: Cross-section area (square feet);
- g: Acceleration due to gravity (feet/sec^2);
- t: Time (second);
- x: Distance along a channel (feet);
- y: water depth (feet);
- S_o: Bed slope (feet/feet);
- S_f: Friction slope (feet/feet).

The method employs the Manning's equation to relate flow rate to flow depth and bed (or friction) slope. The one exception is for circular conduits under pressurized flow, where either the Hazen-Williams or Darcy-Weisbach equation is used instead.

3.0 MODEL CONSTRUCTION AND DEVELOPMENT

As previously mentioned, the SWMM is an integrated hydrologic and hydraulic model that couples the hydrologic process over subcatchment areas and hydraulic routing through a collection system network. The model development consists of constructing a hydrologic model to represent hydrologic processes over defined subcatchment areas and constructing a collection system network to represent channels and hydraulic structures such as culverts, bridges, and weirs, through which flow routes to the system outlets.

3.1 Hydrologic Model

The first step of a hydrologic model construction is to delineate the project area into a number of subcatchments based on their unique geographic and hydrologic characteristics in terms of land use and soil type, and topography. The next step is to identify the basin characteristics (drainage area, basin width, and average basin slope) and hydrologic parameters (CN and Manning's n value) for each of the delineated subcatchment areas. The representativeness of the basin characteristics and hydrologic parameters in the model relies heavily on the accuracy of GIS data that are used.

3.1.1 GIS Data

Topography

Figures 4a and **4b** show the Polk County DEM with 5-ft resolution and 1-ft contour lines from the District's GIS database, which were derived from the latest LiDAR survey. From Lake Pierce to Lake Hatchineha the elevation along Catfish Creek drops about 30.0 feet from elevation approximately 77.0 ft NAVD88 to elevation 47.0 ft NAVD88. A significant portion of Parcel B is below elevation 50.0 ft NAVD88, which forms a ponding area that receives water from Catfish Creek. It also receives runoff from the area surrounding Parcel B during rainfall events.

Figure 4a. Polk County 5-ft DEM

Figure 4b. Polk County 5-ft DEM Overlaid with 1-ft Contour Lines

Land Use/Cover

The District has 2008 land use/land GIS coverage of the project area. The GIS coverage was developed using the Florida Land Use/Cover Classification System (FLUCCS) to define land use/land cover in one of the pre-defined categories. For this project, each polygon in the coverage area was assigned a FLUCCS code corresponding to the existing land use for that area. There are total thirty-four (34) land use/cover types. For the purpose of hydrologic parameter calculation, we further simplified the classification of the original land use/land cover into eight (8) types based on their similarity, as shown in **Figure 5**. It indicates that the existing land use/cover in the project area was dominated by sod farms and pasture. The major land use type in the Parcel B area used to be sod farms.

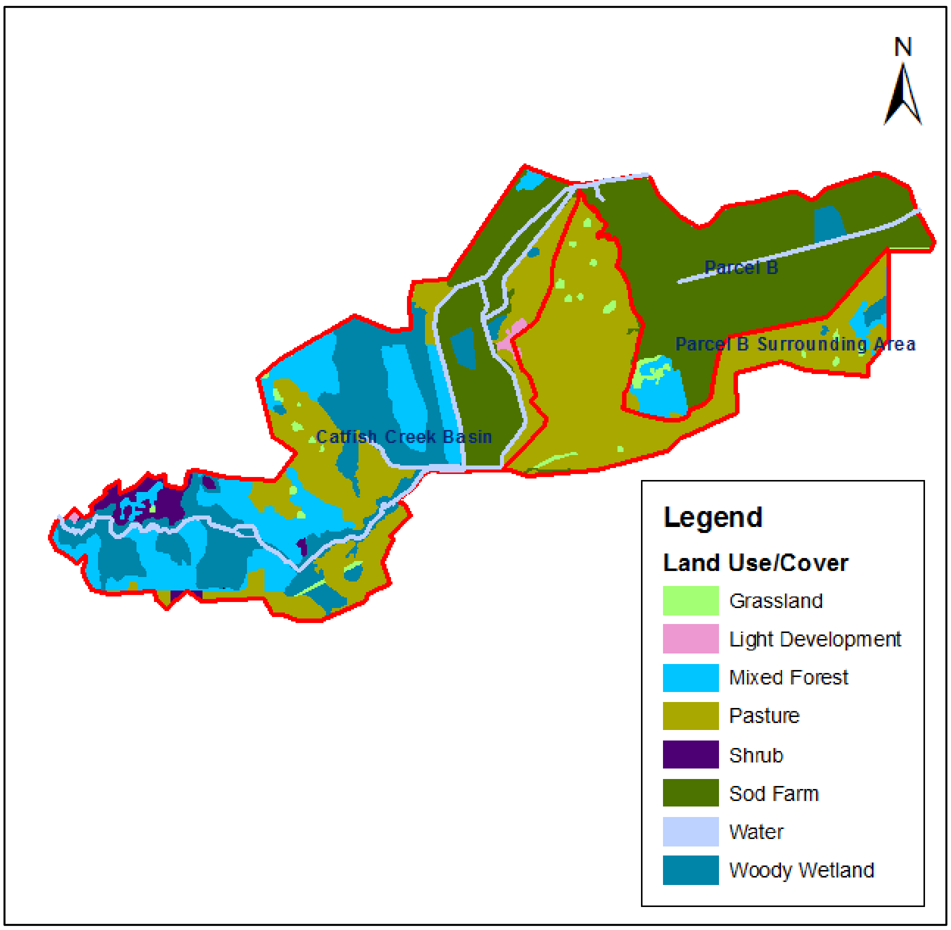

Figure 5. Land Use/Cover in the Project Area

Soil Data

Soil data was developed based on Soil Survey Geographic (SSURGO) soils data layer in the District's GIS database. Soils are classified by their hydrologic characteristics. Hydrologic Soil Groups (HSG) designation for soils are used to estimate infiltration rates, moisture storage capacity and runoff potential from precipitation. Hydrologic soil groups in the study area consist of the following designations as shown in **Figure 6.** The characteristics of these soils are described as below (USDA 1986):

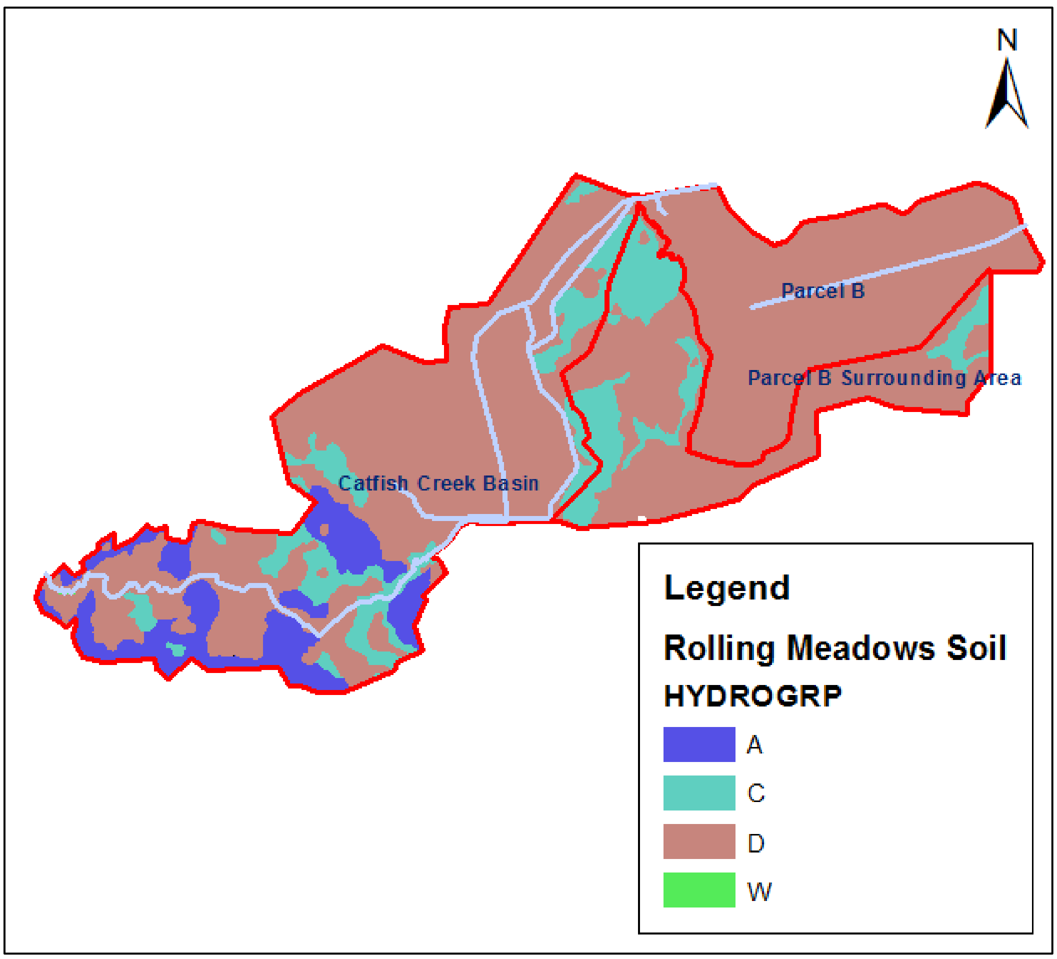

Figure 6. Soil Type by Hydrologic Soil Group

- Group A - low runoff potential: soils have high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission (greater than 0.30 in/hr).

- Group C - moderately high runoff potential: soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05 ~ 0.15 in/hr).
- Group D - high runoff potential: soils have very slow infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential. They have very low rate of water transmission (0 ~ 0.05 in/hr).

Group D soil is the predominant HSG in the study area, which is especially true within Parcel B, as depicted in **Figure 6**.

Aerial Photo

The 2008 Polk County 1-Foot Natural Color Aerial Photography shown in **Figure 1** along with the 2011 aerial photo from Google Earth as illustrated in **Figure 7** were used for this study.

Figure 7. The 2011Aerial Photo from Google Earth

3.1.2 Subcatchment Delineation

Based on the aforementioned topographical data and the aerial photos, we delineated the study area into a number of subcatchments using ArcMap (Version 10.1). The Catfish Creek watershed was divided into 23 subcatchment areas, and Parcel B and its surrounding area into 11 subcatchments, as illustrated in **Figures 8a and 8b**. We then calculated the basin characteristics: drainage area, surface slope, and flow path length of each of the subcatchments by using ArcMap. Basin width was computed by dividing drainage area by flow path length. **Table 1** presents these subcatchment characteristic values.

Figure 8a. Subcatchment Areas of the Catfish Creek Watershed

Figure 8b. Subcatchment Areas of Parcel B and Its Surrounding Area

Table 1. Subcatchment Area Characteristics and Hydrologic Parameters

Watershed	Basin ID	Drainage Area (Acres)	Slope (%)	Flow Path Length (ft)	Width (ft)	CN	Manning's n
Catfish Creek	B01-1	69.4	2.219	2512.0	1203.6	66.0	0.274
	B01-2	192.9	2.361	2736.6	3070.4	59.4	0.291
	B01-3	244.0	1.679	2588.8	4105.7	60.6	0.289
	B01-4	456.0	1.209	4557.2	4358.4	67.9	0.279
	B01-5	377.4	1.700	4389.0	3745.6	66.1	0.257
	B01-6	82.4	2.273	2086.9	1720.7	76.7	0.212
	B01-7	218.1	1.596	4585.8	2071.9	68.8	0.239
	B01W	303.3	1.155	3901.2	3386.8	68.8	0.239
	B03	29.3	0.226	3740.7	341.4	83.4	0.130
	B03E-1	41.8	0.036	2271.4	801.9	68.8	0.239
	B03E-2	14.8	0.000	588.2	1095.7	68.8	0.239
	B04	10.2	0.207	2331.8	191.1	83.4	0.130
	B04E	36.9	0.012	927.1	1734.1	68.8	0.239
	B05E	340.8	0.067	8697.7	1706.6	83.4	0.150
	B05S	214.4	0.001	3796.3	2459.9	77.8	0.200
	B05W	583.0	0.483	9984.6	2543.4	77.8	0.200
	B07	6.7	0.403	2485.1	116.7	99.0	0.080
	B13M	12.3	0.067	5313.7	100.8	99.0	0.080
	B13N	231.1	0.157	8168.4	1232.5	82.6	0.130
	B13S	96.1	0.082	6955.4	601.9	82.6	0.130
	B13SE-1	71.5	0.101	1138.7	2733.9	77.8	0.200
	B13SE-2	85.8	0.155	1705.2	2191.1	68.8	0.239
	B13SE-3	27.4	0.063	1742.0	684.9	68.8	0.239
Parcel B & Its Surrounding Area	PB01	297.5	0.150	3415.2	3794.5	77.8	0.200
	PB02-1	290.4	0.067	3204.9	3946.4	77.8	0.200
	PB02-2	64.7	0.316	1752.4	1607.2	77.8	0.200
	PB02-3	441.2	0.310	5493.0	3499.1	77.8	0.200
	PB03	76.0	0.672	1447.3	2286.5	77.8	0.200
	PB04	43.1	0.644	979.2	1916.5	77.8	0.200
	PB05	41.9	0.502	864.5	2110.4	77.8	0.200
	PB06	102.2	0.368	1662.8	2678.6	77.8	0.200
	PB07	107.5	0.299	2243.0	2087.7	77.8	0.200
	PB08	152.5	0.158	3121.8	2127.8	77.8	0.200
	PB09	1646.9	0.114	12909.5	5557.1	99.0	0.010

3.1.3 Curve Number Calculation

We first developed the curve number (CN) reference table (See **Table 2**) based on land use/cover and soil types, as shown in **Figures 5 & 6**, and then calculated an area-weighted CN for each of the subcatchment areas. The calculated CN values were further adjusted based on aerial photos, the results of hydrologic model runs, and by employing reasonable engineering judgment. The final calculated area-weighted CN value for each subcatchment is presented in **Table 1**

Table 2. Curve Number Reference Table

Land Use/Land Cover	Hydrologic Group		
	A	C	D
Grassland	49	79	84
Light Development	51	79	84
Mixed Forest	36	73	79
Pasture	49	79	84
Shrub	35	70	77
Sod Forms	49	79	84
Water	99	99	99
Woody Wetland	35	70	77

3.1.4 Manning's n Value

Similar to the procedure used in the CN calculation, we first developed the Manning's n reference table based on land use/cover, and then calculated an area-weighted Manning's n value for each of the subcatchment areas. **Table 3** lists the Manning's n value corresponding to each land use/cover. **Table 1** provides the final calculated area-weighted Manning's n value for each subcatchment.

Table 3. Manning's n Reference Table

Land Use/Land Cover	Manning's n
Grassland	0.035
Light Development	0.130
Mixed Forest	0.300
Pasture	0.200
Shrub	0.300
Sod Farms	0.200
Water	0.035
Woody Wetland	0.300

3.2 Hydraulic Routing Model

In order to develop a hydraulic model that represents a physical conveyance system, one needs to have a good understanding of the conveyance system and its various components. The components of a conveyance system generally include the followings:

- Channel/conduit length and cross section
- Channel/conduit slope
- Flow resistance – Manning's n through a main channel and overbank
- Flow resistance through a conduit
- Hydraulic structure type and dimension
- Entry and exit loss coefficients of conduits
- Stage – storage relationship for storage units representing a ponding area
- Boundary conditions

3.2.1 Existing Conveyance System

Only major channels or canals were included in the development of the model's conveyance system. **Figure 9** depicts the existing conveyance system for the project area, which includes:

- a number of major ditches
- hydraulic structures such as culverts either with or without risers at Sites 1, 2, 3, 4, 5, and 13 that interconnect the channels
- a berm breach at Site 6 that connects Catfish Creek and Parcel B
- Parcel B as a ponding area
- Outlets at Sites 7 and 11, which serve as connections between the project area and Lake Hatchineha. The culvert at Site 7 is currently not functioning

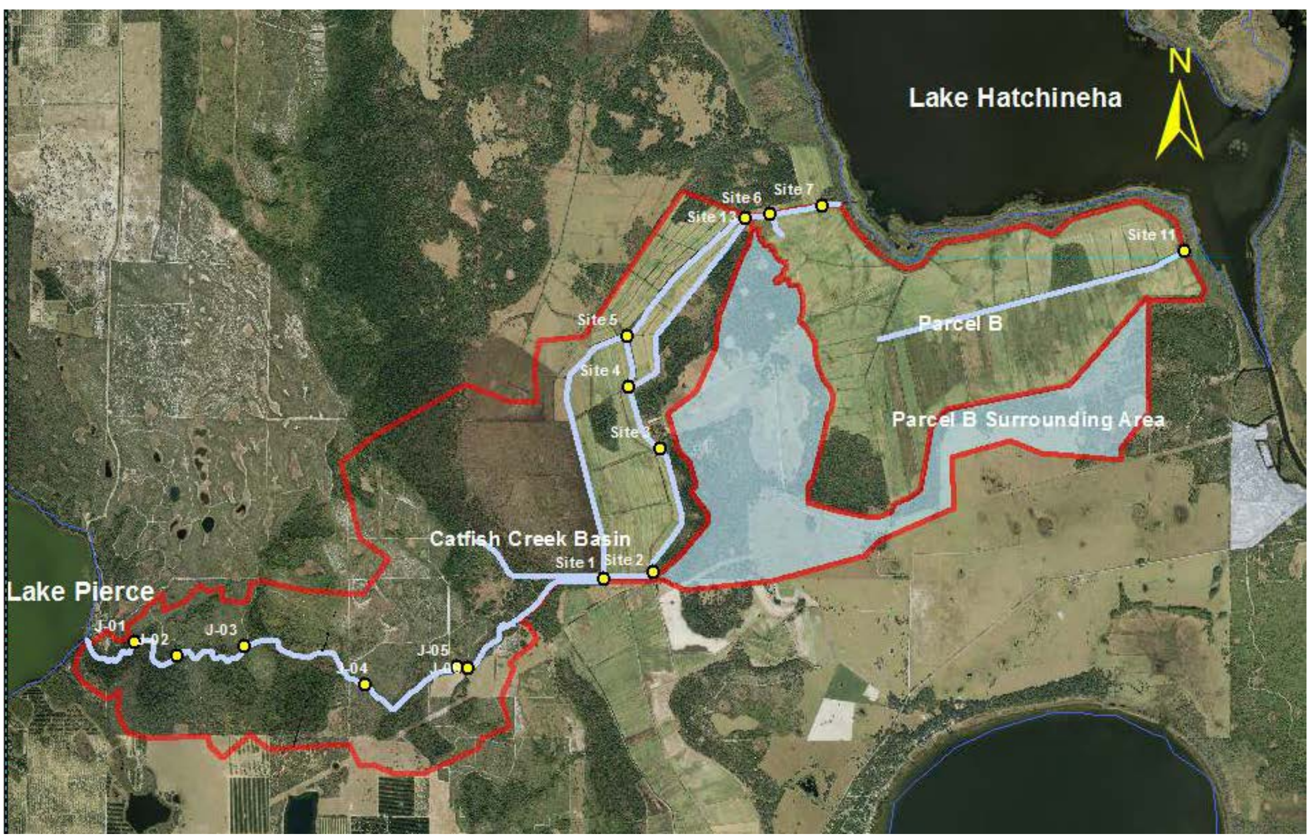

Figure 9. Conveyance System of the Project Area

Flow from the Catfish Creek upper basin routes to Site 1. From Site 1, the flow routing path splits into two major routes: northwest route to Site 5 and eventually to Site 13; and northeast route to Site 2 then through Sites 3 and 4, and eventually to Site 13. At Site 1, flow could also go west when downstream water levels are high and then eventually return to the system at Site 5 via sheet flow. Culverts at Site 7 are currently not functioning and no water flows out from Site 7 to Lake Hatchineha. Flow from the Catfish Creek watershed currently routes to Parcel B through the existing berm breach at Site 6. Flow from the Parcel B Surrounding Area routes into Parcel B as sheet flow. Flow from Parcel B currently discharges through the existing culvert at Site 11 to Lake Hatchineha when the water level inside Parcel B is higher than the invert elevation of the culverts and the water level in Lake Hatchineha.

3.2.2 Open Channels

Cross-Section: Open channels form a major portion of the Catfish Creek conveyance system. We divided the creek into a number of segments or reaches, and the channel geometry within each reach was assumed not to vary significantly. For the upper portion of Catfish Creek, upstream of Site 1, we created the cross-sections for each reach by using the Polk County DEM with 5-feet resolution since there was no survey data available. For the lower portion of Catfish Creek, we generated channel cross-sections based on the topographic survey by Morgan & Eklund in 2012. Channel length was determined by using ArcMap.

Channel Slope: The slope of each reach was calculated based on the difference between entrance and exit invert elevations divided by reach length.

Manning's *n* Value: For natural open channels, we used Manning's *n* value of 0.035 for main channels inside the channel banks, 0.15 for over bank flow, and 0.024 for the corrugated metal pipe (CMP) conduits.

Entry and Exit Lose Coefficients: The default values suggested in the SWMM model were used for entry and exit loss coefficients, i.e., entry loss coefficient = 0.5 and exit loss coefficient = 1.0.

3.2.3 Hydraulic Structures

Most of the hydraulic structures in the study area are corrugated metal pipes with flash-board risers to control flow and water level. **Figure 10a** illustrates a typical corrugated metal pipe and a flash-board riser with the flash-board fully open, **Figure 10b** a riser with the flash-board fully closed to the riser top, and **Figure 10c** a riser with the flash-board partially closed. Conduit length and dimension, and invert elevations at entry and exit for the existing structures were based on the field survey by Morgan & Eklund in 2012. **Table 4** presents the existing hydraulic structure dimensions at various sites in the existing system.

Figure 10a. Corrugated Metal Pipe with a Flash-Board Riser (Flash Board Fully Open)

Figure 10b. Corrugated Metal Pipe with a Flash-Board Riser (Flash-Board Fully closed to the Top of the Riser)

Figure 10c. Corrugated Metal Pipe with a Flash-Board Riser (Flash-Board Partially Closed)

Table 4. Existing Hydraulic Structures

Site Name	Pipe	Pipe/ Riser	# of Barrels	Diameter (in)	Length (ft)	Entry Invert (ft, NAVD)	Exit Invert (ft, NAVD)	Riser Top/ Weir Crest El (ft, NAVD)	Note
Site 1	Pipe 1W	Pipe	1	48	30	53.40	53.20		
		Riser		72				61.41	
	Pipe 1N	Pipe1	1	48	25	55.30	55.00		
		Riser1		72				60.90	
		Pipe2	1	48	30	53.20	53.60		
		Riser2		72				61.32	
Site 2	Pipe 2	Pipe	1	60	30	52.50	52.10		
		Riser		72				62.02	
Site 3	Pipe 3	Pipe	1	60	20	51.20	50.00		
		Riser		96				59.57	
Site 4	Pipe 4S	Pipe	1	60	40	50.50	50.40		
		Riser		72				60.34	
	Pipe 4W	Pipe	1	54	20	51.20	51.20		
		Riser		72				58.95	
	Pipe 4N	Pipe	1	18	95				
		Riser		18				60.21	
Site 5	Pipe 5S	Pipe	1	24	25	52.80	52.00		
		Riser	N/A						
	Pipe 5N	Pipe	1	96	30	50.90	51.20		Riser on both pipe ends
		N. Riser		96				62.68	
		S. Riser		96				62.94	
Site 13	Pipe	Pipe	1	30	25	49.90	50.80		
		Riser		30				56.97	
	Pipe from Field to Channel	Pipe	1	72	25	48.80	48.60		Riser on both pipe ends
		W. Riser		72				57.44	
		E. Riser		72				57.37	
Site 6	Levee Breach to Parcel B	Weir	1	240				51.00	
Site 7	Pipe 7	Pipe1	1	36	50	N/A	47.20		Plugged and no flow to the Lake
		Pipe 2	1	36	50	N/A	46.20		
		Pipe 3	1	36	50	N/A	46.00		
Site 11	Pipe 11	Pipe	1	48	40	48.40	48.40		

3.2.4 Modeling Flash-Board Riser in SWMM

We present a flash-board riser in the SWMM Model by using two weirs: a lower weir with its length equal to the riser diameter and its crest elevation equal to the designated flash-board elevation to calculate flow over the flash-board; and an upper weir with its length equal to half of the riser's circumference with its crest elevation equal to riser-top elevation to compute flow over the riser when the water surface rises above the top of the riser. Total flow through the riser is equal to the flow over the flash-board plus the flow over the top of the riser, as given below:

$$Q_T = Q_{Flash} + Q_{Riser} \tag{4a}$$

$$Q_{Flash} = CDH_{Flash}^{3/2} \tag{4b}$$

$$Q_{Riser} = {C\pi DH_{Riser}^{3/2}}\Big/{2} \tag{4c}$$

Where

Q_T:	Total flow through the riser (cfs);
Q_{Flash}:	Flow over the flash-board (cfs);
Q_{Riser} :	Flow over the top of the riser (cfs);
C:	Runoff coefficient = 2.85 ~ 3.30;
D:	Diameter of the riser (feet);
H_{Flash}:	Water depth above the set flash-board elevation (feet);
H_{Riser}:	Water depth above the top of the riser (feet).

For the final flow rates delivered over the riser-controlled structures such as culverts, the model takes the lesser of the total flow through the riser and the maximum flow through the culvert.

3.2.5 Parcel B Stage-Storage Relationship

Waters from the Catfish Creek watershed to Parcel B naturally store in Parcel B. Parcel B was represented as a storage unit in the model. Parcel B's stage-area relationship was developed based on the Polk County 1-ft contour lines using ArcMap. **Figure 11** depicts the stage-area relationship of Parcel B.

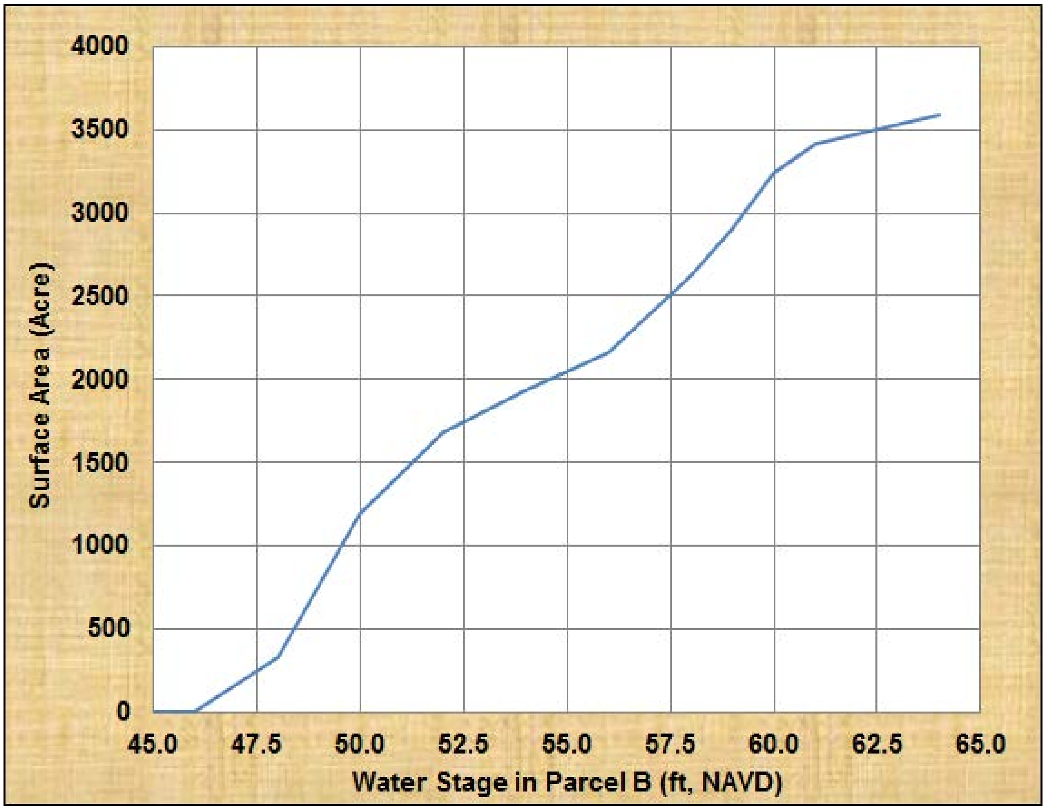

Figure 11. Parcel B Stage – Area Relationship

3.2.6 Boundary Conditions

There are two outfall points for the system: one at Site 7 and one at Site 11. We assigned the outfall boundary conditions, i.e., outfall stage at these outfalls to be the same stage as Lake Hatchineha. SFWMD (1991) provided water stages at Lake Hatchineha corresponding to given design storm frequencies (5-, 10-, 50-, and 100-year), as presented in **Table 5**. This information represents projected lake levels as a result of implementing the Lake Kissimmee Headwaters Revitalization Project. The water stage information provided was referenced to the NGVD29 datum, which was converted to the NAVD88 datum by reducing the stages by 1.18 feet. **Figure 12** illustrates the water stages in Lake Hatchineha corresponding to the design storm frequencies. In this study, we utilized the 25-year and 100-year 3-day design storms to conduct the hydrologic analyses. The outfall stage corresponding to the 25-year design storm was estimated based on a straight line interpolation between the 10-year and 50-year water stages in Lake Hatchineha. **Table 6** lists the boundary water stages in Lake Hatchineha corresponding to 25-year and 100-year, 3-day design storm events.

Table 5. Water Stage at Lake Hatchineha

Return Period (Year)	Water Stage (ft, NGVD)	Water Stage (ft, NAVD)
5	53.81	52.63
10	54.05	52.87
50	54.81	53.63
100	55.15	53.97

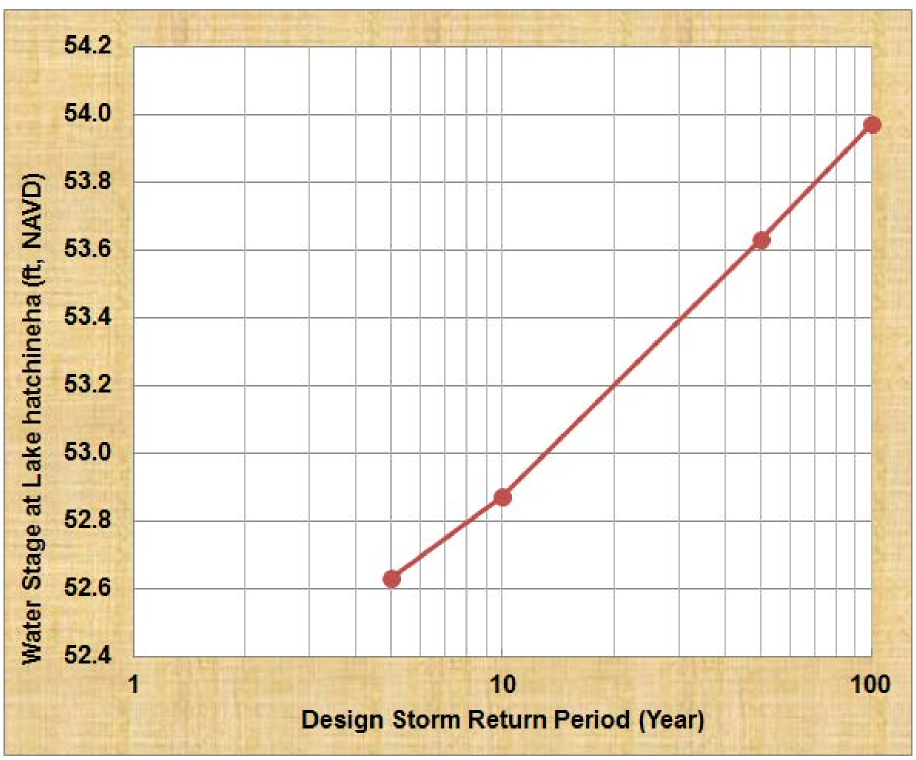

Figure 12 Stages at Lake Hatchineha Corresponding to Design Storm Frequencies

Table 6. Outfall Water Stage at Sites 7 and 11

Design Storm	Water Stage (ft, NAVD)
25 year, 3 day	53.30
100 year, 3 day	53.97

4.0 HYDROLOGIC MODEL INPUT - DESIGN STORM

The depth of rainfall for a specific occurring frequency and duration, and the storm temporal distribution are the most important model input parameters necessary to estimate design flows of a stormwater management system. In this study, we evaluated two design storm events: the 25-year, 3-day design storm for designing hydraulic structures and the conveyance system, and the 100-year, 3-day design storm for peak water stage evaluation.

4.1 Design Rainfall Depth

We derived the 25-year and 100-year, 3-day rainfall depths by using a straight line interpolation based on the rainfall isohyetal maps of South Florida (SFWMD, 2009), as shown in **Figures 13a and 13b**. **Table 7** presents the 25-year and 100-year, 3-day rainfall depths.

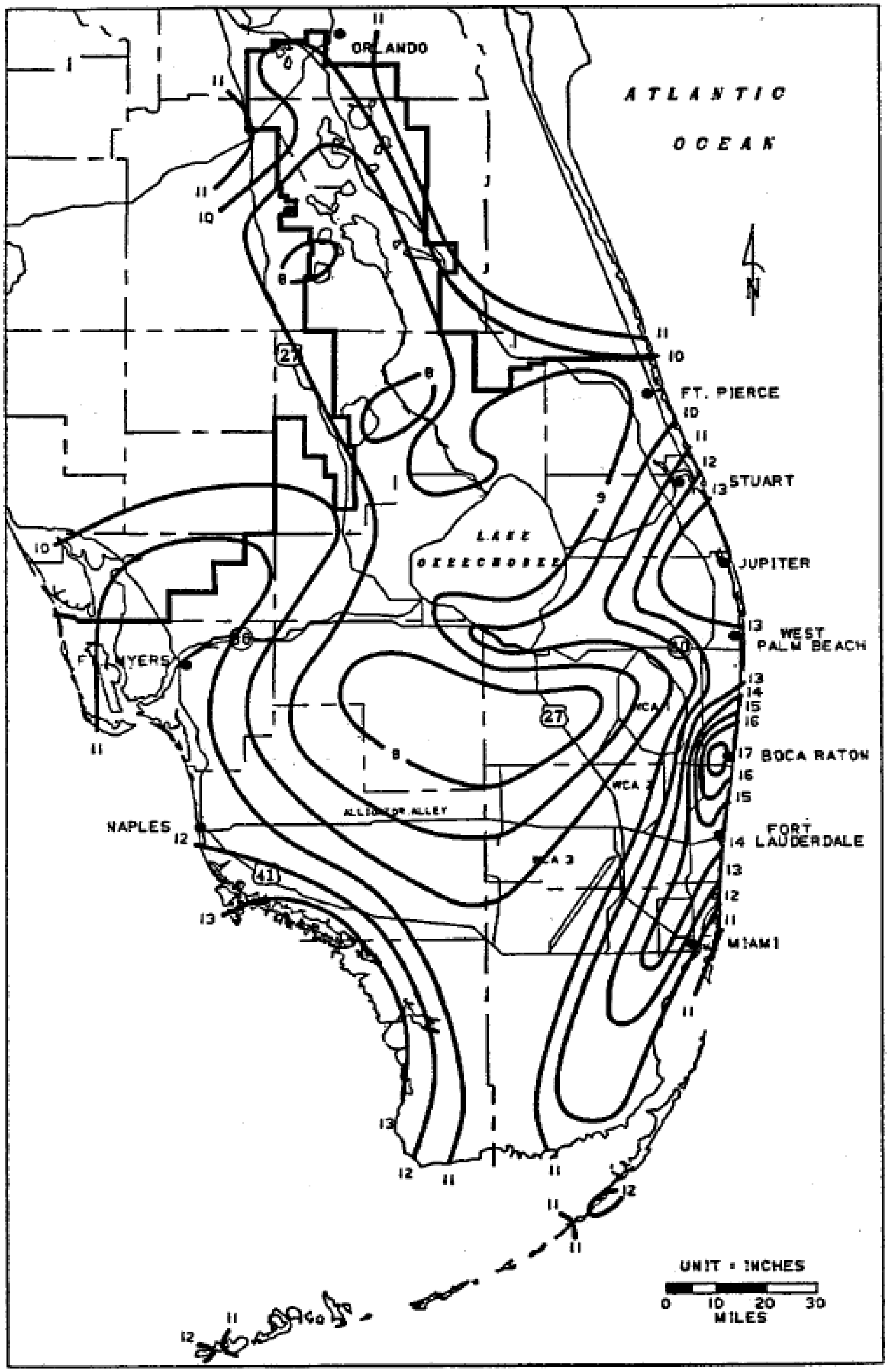

Figure 13a. Rainfall Isohyetal Map for 25-Year, 3-Day Storm

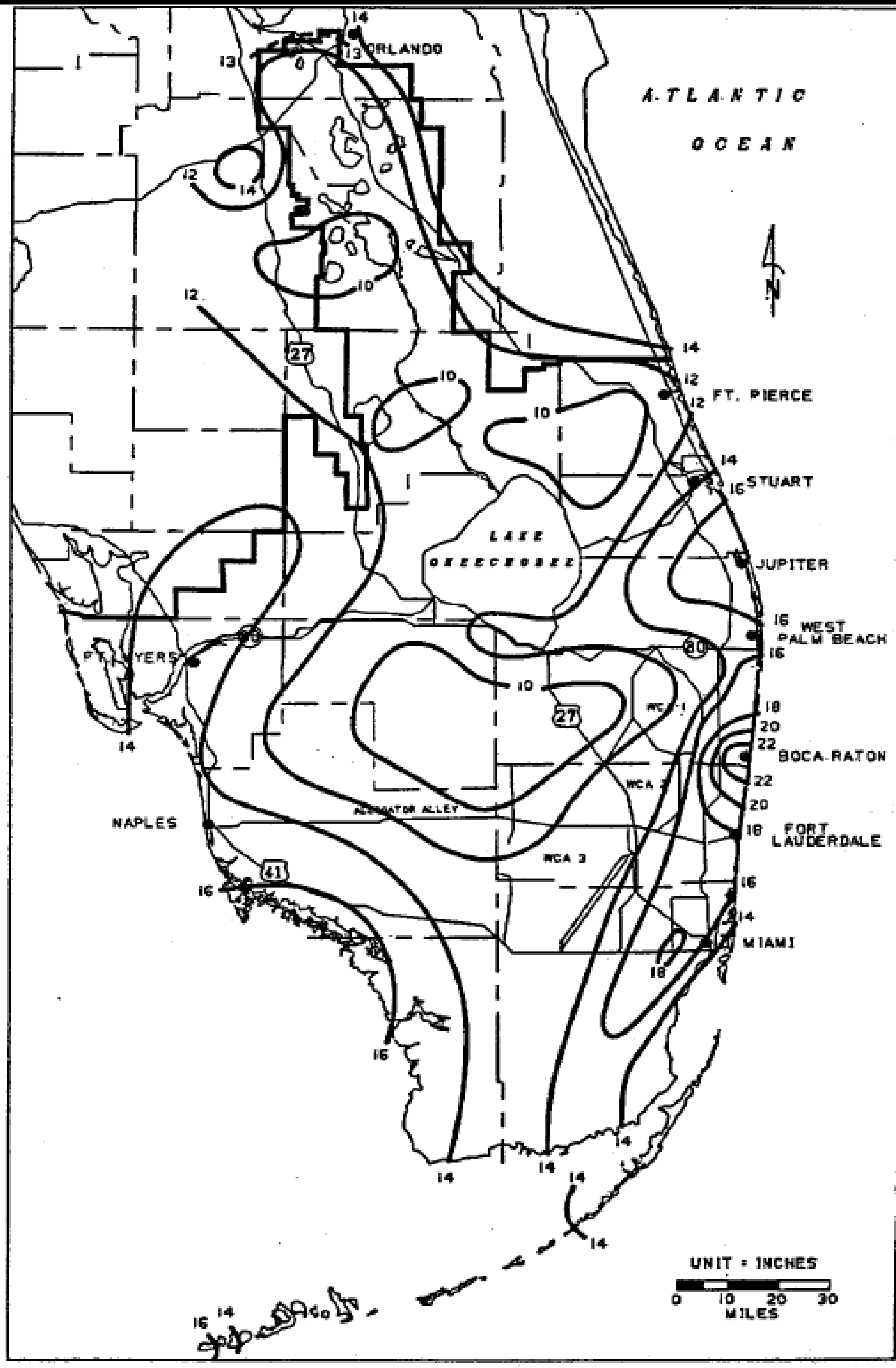

Figure 13b. Rainfall Isohyetal Map for 100-Year, 3-Day Storm

Table 7. Design Rainfall Depths

Return Frequency (year)	Duration (day)	Depth (in)
25	3	8.3
100	3	10.0

4.2 Rainfall Temporal Distribution

In addition to the rainfall depth, the rainfall temporal distribution also affects the peak stage and discharge rate. The 3-day rainfall temporal distribution, developed by the District (SFWMD 2009), was used for the analyses. **Figure 14** illustrates the 3-day rainfall distribution for both the 25-year and 100-year design storms in terms of percentage of total rainfall.

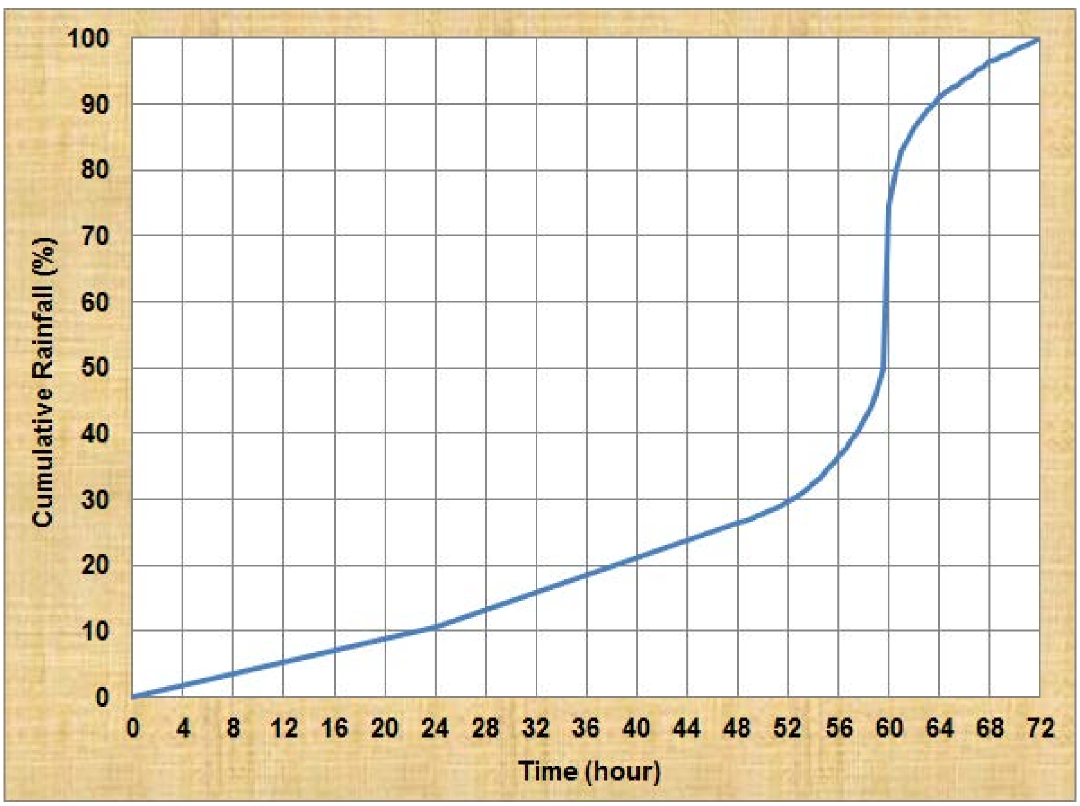

Figure 14. Temporal Distribution for 25-Year and 100-Year, 3-Day Design Rainfall

5.0 MODEL CALIBRATION/VALIDATION

We constructed the model by entering the previously developed hydrologic and hydraulic variables and parameters into the SWMM. Model calibration is an essential step in the hydrologic and hydraulic modeling construction process. A computer model of a watershed is basically a mathematical representation of actual physical runoff generating and routing processes of a watershed. Model construction is basically providing data that describe the physical characteristics of the system as well as input data and boundary conditions into a computer program that simulates the behavior of the real system. Model simulation results are typically utilized to assist in decision-making for the purpose of planning, construction, restoration, etc. Therefore, it is imperative to ensure the constructed model reasonably represents the real system. Otherwise, the results provided by the model will be of limited value or misleading in the worst case. Model calibration is the process of adjusting model parameters until the model performance is in reasonable agreement with observed system performance.

5.1 Data Collection

In the project area, there was one archived USGS gage station (02267000) located close to the upstream end of Catfish Creek. Almost all of the project area lies downstream of this gage station. Therefore the data from this station could not be used for calibrating the constructed model.

On October 6, 2010, ZFI conducted stream-gauging at three locations as shown in **Figure 15** (ZFI, 2011). **Table 8** presents water stages and flow rates observed at these locations. ZFI stated that the stage at the culvert west of the maintenance area appeared to be inaccurate, and therefore this stage was not used in the model calibration.

In addition, there is a rain gauge station, SNIVELY_R at 27^{o}58’18.068” latitude and -81^{o}25’3.242” longitude, which is located near the southern boundary of the project area, as shown in **Figure 16**. We reviewed the historic rainfall records at SNIVELY_R, and found that there was 1.23 inches of rainfall on September 28, 2010 at this rain gauge station. This rainfall event was related to Tropical Storm Nicole, as mentioned by ZFI (2011). However, between September 29 and October 6, 2010, there was only 0.01 inch rainfall recorded, and that was on October 1, 2010. This indicates that there had been almost no rain for seven (7) days before ZFI conducted their stream-gauging on October 6, 2010. ZFI also reported that the flow measurements were conducted under sunny-day conditions (ZFI, 2011). These facts indicate that the measured flow may represent the base flow condition of the watershed after the wet season.

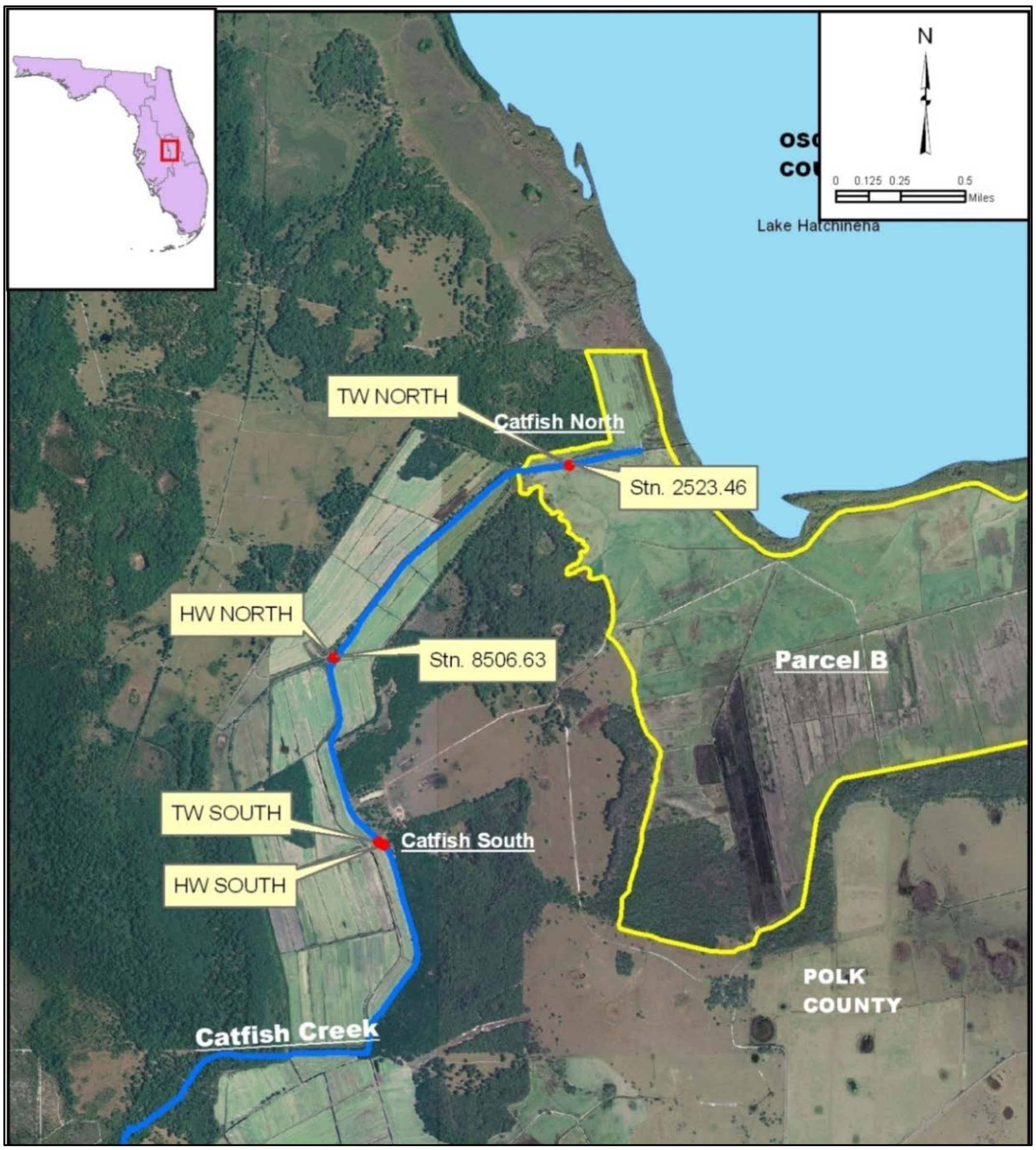

Figure 15. Data Collection Locations (Extracted from ZFI, 2011)

Table 8 Water Stage and Flow Rate at ZFI Stream-Gauging Locations

Station	Location	Latitude	Longitude	Elevation (ft, NAVD 88)	Flow Rate (cfs)	Site No Corresponding to Present Project
HW South	At Culvert West of Maintenance Area	27° 58' 50.4"	81° 26' 25.02"	56.417		U/S of Site 3
TW South	At Culvert West of Maintenance Area	27° 58' 51.1"	81° 26' 26.52"	56.836		D/S of Site 3
HW North	East of Old Bridge	27° 59' 27.8"	81° 26' 37.68"	53.750	36.73	D/S of Site 5
TW North	On Canal (East of Old Bridge) near berm breach	28° 00' 07.3"	81° 25' 47.16"	53.490	48.73	D/S of Site 13 and U/S of Site 6

Figure 16. Location of Snively_R Rain Gauge

5.2 Model Validation Based on Measured Flow Data

To evaluate flow distribution along the model network, we conducted a model simulation by using a base flow of 48.73 cfs as an inflow at the upstream end of Catfish Creek, and by setting the water stage of Sites 7 and 11 at 53.49 ft NAVD as the model boundary condition. **Table 9** presents the comparison between the measured and modeled flows. It indicates that the absolute relative errors between the measured and the modeled flows are less than 5%. The measured flow of 36.73 cfs downstream of Site 5 is 75% of total measured flow of 48.73 cfs downstream of Site 13. The SWMM model calculated a flow of 35.00 cfs at Site 5, which is 71% of the total flow of 49.50 cfs calculated by the SWMM model. These results demonstrate that the constructed model network appears to reasonably represent the flow split/distribution along the collection network.

Table 9. Comparison between Measured and Computed Flow Rate

Site No	ZFI Streamgauging (cfs)	SWMM (cfs)	Difference	
			cfs	%
D/S of Site 5	36.73	35.00	-1.73	-4.71
D/S of Site 13	48.73	49.50	0.77	1.58

5.3 Model Validation by the USGS Regional Regression Equation

The USGS developed a computer program titled “National Flood Frequency” or “NFF” that estimates the flood frequency and magnitude for ungaged sites through the application of the appropriate regional regression equations (USGS 1994). The USGS relates flood characteristics to watershed and climatic characteristics at gaging stations by conducting regression analyses. Flood characteristics are defined as flood-peak discharges for selected T-year recurrence intervals (such as 100-year flood). Because these flood characteristics may vary substantially between regions due to differences in climate, topography, and geology, the Nation was divided into 210 hydrologic regions, each of them having relatively homogenous flood characteristics. The USGS has defined regional regression equations corresponding to T-year recurrence intervals for each of the regions. The State of Florida has four hydrologic regions, as shown in **Figure 17**.

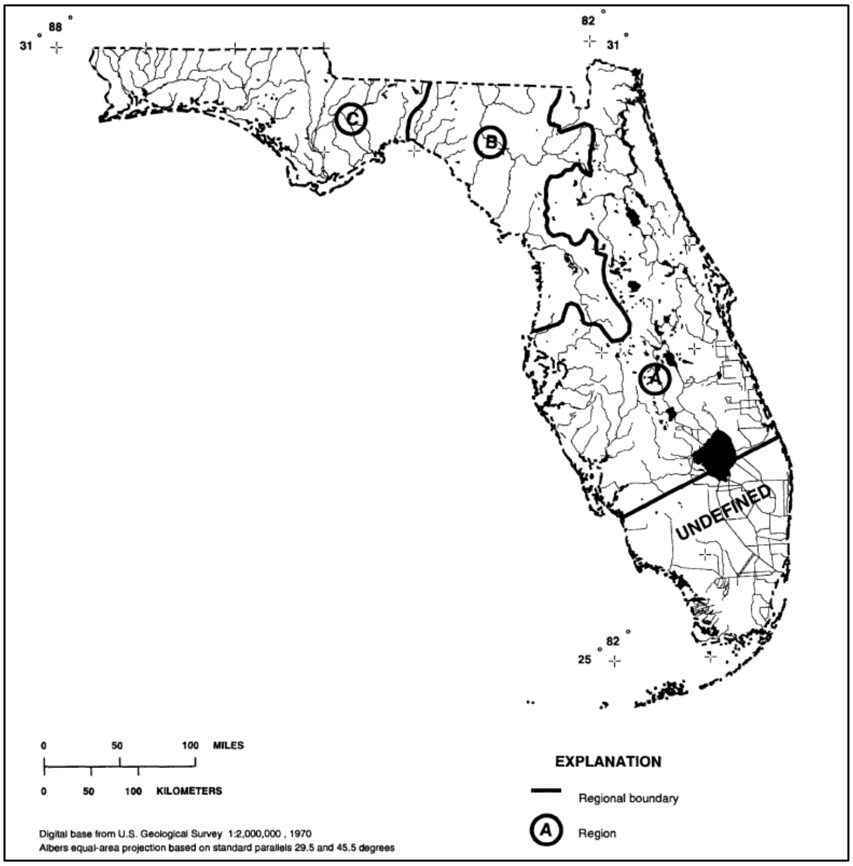

Figure 17. Flood-Frequency Region Map for Florida (Extracted from USGS, 1994)

The Rolling Meadows Project area is located within Region A. The regression equation for the 100-year flood peak flow is given below:

$$Q_{100} = 609DA^{0.685}SL^{0.227}(LK+3)^{-0.695} \tag{4}$$

Where

Q_{100}: Flood peak flow with 100-year recurrence (cfs);

DA: Drainage area in square miles;

SL: Channel slope in feet per mile;

LK: The area of lakes and ponds as a percentage of the drainage area.

The NFF program also provides some accuracy measurement for the regression equation. The most frequently used measure of accuracy is the standard error of estimate, usually reported in percentage. This standard error is a measure of the variation between the regression estimates and the station data for those stations used in deriving the regression equations. For Florida Region A, the standard error is 53% for the estimate of 100-year flood peak flows.

The USGS regional regression equation provides a general guideline on flood characteristics, i.e., peak flow across a specific hydrologic region, which also provides another way to validate the model hydrology of an ungaged watershed. In the FEMA flood insurance study program, FEMA considers it a reasonable estimate if the peak flow for a given return period generated by a hydrologic model falls within the range of the estimate by the USGS regional regression equation ± one standard error of the estimate, i.e., within 90% confidence interval.

We compared the modeled peak flows with those estimated using the USGS regression equations at Junction J-01, J-02, J-03, J-04, J-05, J-06, Site 1, and Site 13, as shown in **Figure 9**. Flows at Site 2, Site 3, Site 4, and Site 5 were not compared to those by the USGS regression equation since the network among the sites is interconnected and it is almost impossible to define contributing area of each site separately. **Table 10** shows the comparison of peak flows calculated by the SWMM model with the peak flows estimated using the USGS regression equations for the 100-year, 3-day design storm event. This comparison indicates that the computed peak flows by the SWMM model are well within the range of the 90% confidence interval of the estimates by the USGS regression equation.

Note that, the peak flow, 826.65 cfs at Site 13 calculated by the SWMM model is approximately 16% less than the peak flow, 985.50 cfs at Junction J-06, even though the contribution area to Site 13 is more than 2 times that of Junction J-06. This is due mainly to the fact that when Catfish Creek enters its downstream segments starting at Site 1, the terrain becomes flat and the conveyance system capacity and its storage play a major role in attenuating flood peaks. This physical process cannot be represented by the USGS regression equation.

Based on the aforementioned model validation by using the measured flows from the stream-gauging activities and by utilizing the USGS regression equation, we concluded that the constructed hydrologic and hydraulic model reasonably represents the physical processes of runoff generating and routing through the project area.

Table 10. Comparison of 100-Year Peak Flows for Catfish Creek

Junction ID	Cumulative Drainage Area (Sq Mile)	Peak Flow by SWMM (cfs)	Peak Flows by USGS Regression Equation (cfs)			
			Peak Flow	Standard Error	Lower Limit (Estimate - Standard Error)	Upper Limit (Estimate + Standard Error)
J-1	0.108	79.90	74.1	39.3	34.8	113.4
J-2	0.410	244.57	186.1	98.6	87.5	284.8
J-3	0.791	443.09	305.9	162.2	143.8	468.1
J-4	1.504	683.01	444.6	235.7	209.0	680.3
J-5	2.093	919.84	614.0	325.4	288.6	939.5
J-6	2.222	985.50	646.8	342.8	304.0	989.6
Site 1	2.563	835.42	784.2	415.6	368.6	1199.8
Site13	5.852	826.65	1255.5	665.4	590.1	1920.9

6.0 Model Simulation

6.1 Post-Improvement Hydraulic Structures

The existing surface water management control structures within the project area were constructed in the 1950s and have significantly deteriorated over time. One purpose of this project is to replace these existing structures with new ones to improve the water management capabilities of the system. In addition to the model representing the existing system, we also constructed a model representing the proposed structures to evaluate the post-improvement system conveyance. **Table 11** presents the proposed new control structures.

Table 11. Post-Development Hydraulic Structures

Site Name	Pipe	Pipe/ Riser	# of Barrels	Diameter (in)	Length (ft)	Entry Invert (ft, NAVD)	Exit Invert (ft, NAVD)	Riser Top/ Weir Crest El (ft, NAVD)
Site 1	Pipe 1W	Pipe	1	48	31.0	53.5	53.5	
		Riser	1	72				59.0
	Pipe 1N	Pipe	2	48	37.0	53.5	53.5	
		Riser	2	72				59.0
Site 2	Pipe 2	Pipe	2	60	40.0	50.0	50.0	
		Riser	2	84				59.0
Site 3	Pipe 3	Pipe	1	72	47.0	50.0	50.0	
		Riser	1	N/A				
Site 4	Pipe 4S	Pipe	1	60	37.0	50.0	50.0	
		Riser		84				59.0
	Pipe 4W	Pipe	2	72	36.0	50.0	50.0	
		Riser	2	96				58.0
	Pipe 4N	Pipe	1	24	39.0	54.0	54.0	
		Riser	1	36				59.0
Site 5	Pipe 5S	Pipe	2	72	31.0	50.0	50.0	
		Riser	2	96				59.0
	Pipe 5N	Pipe	3	84	40.0	50.0	50.0	
Site 13	Pipe	Pipe	N/A					
		Riser						
	Pipe 13	Pipe	3	72	39.0	47.0	47.0	
Site 6	Pipe 6	Pipe	2	60	42.0	49.5	49.5	
		Riser	2	84				56.5
Site 7	Pipe 7	Pipe	3	72	40.0	46.0	46.0	
		Riser	3	96				54.0
Site 11	Pipe 11	Pipe	4	72	85.0	43.0	42.5	
		Riser	4	96				54.0

By comparing the post-improvement structures in **Table 11** to the pre-improvement or existing ones in **Table 4**, the changes are readily apparent, especially at Site 6, Site 7, and Site 11. At Site 6, the existing breach is replaced by 2 - 60" culverts with 84" risers at the entrance of each culvert. At Site 7, the 3 existing 36" culverts are replaced by 3 - 72" culverts with 96" diameter riser at the entrance of each culvert. At Site 11, a single 48" culvert is replaced by 4-72" culverts with 8' wide x 9' high manually operated double-leaf gates at the entrance of each culvert.

6.2 Simulation Comparison between Pre- and Post- Improvements

6.2.1 Design Condition: 25-Year Design Storm

The 25-year 3-day design storm was used to design the drainage conveyance system. Water stages in the area south of the canals between Site 1 and Site 2 (marked as blue color lines in **Figure 18**) under post-improvement should not be higher than those before replacing the existing structures, or pre-improvement. **Figure 18** also shows the conduit names in the SWMM representing these canals.

Figure 18. SWMM Conduits Representing the Canal Segments between Site 1 & Site 2

We conducted simulation runs for both pre- and post-improvement conditions using the 25-year 3-day design storm. **Appendix A** presents the summary of the simulation runs. **Figures 19a** through **19e** compare the water depths for the pre- and post-improvement scenarios in these canals. These diagrams demonstrate that the proposed improvements significantly attenuate flooding in the canal segment (C67) just upstream of Site 1 and also in the canals between Site 1 and Site 2 (C-Imp1SN, C-Imp1S-2, C-Imp1-1E, and C-Imp1-E). The ditch overtopping duration of post-improvement is much shorter than that of pre-improvement.

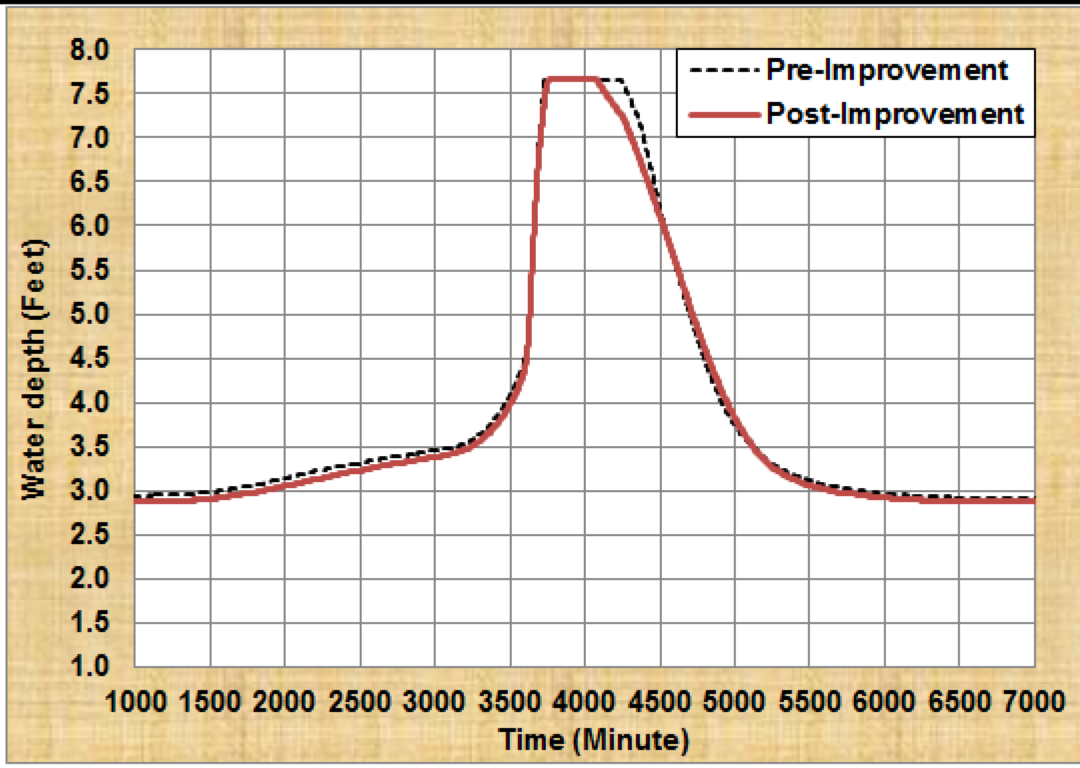

Figure 19a. Water Depth in C67 for Pre- and Post-Improvements

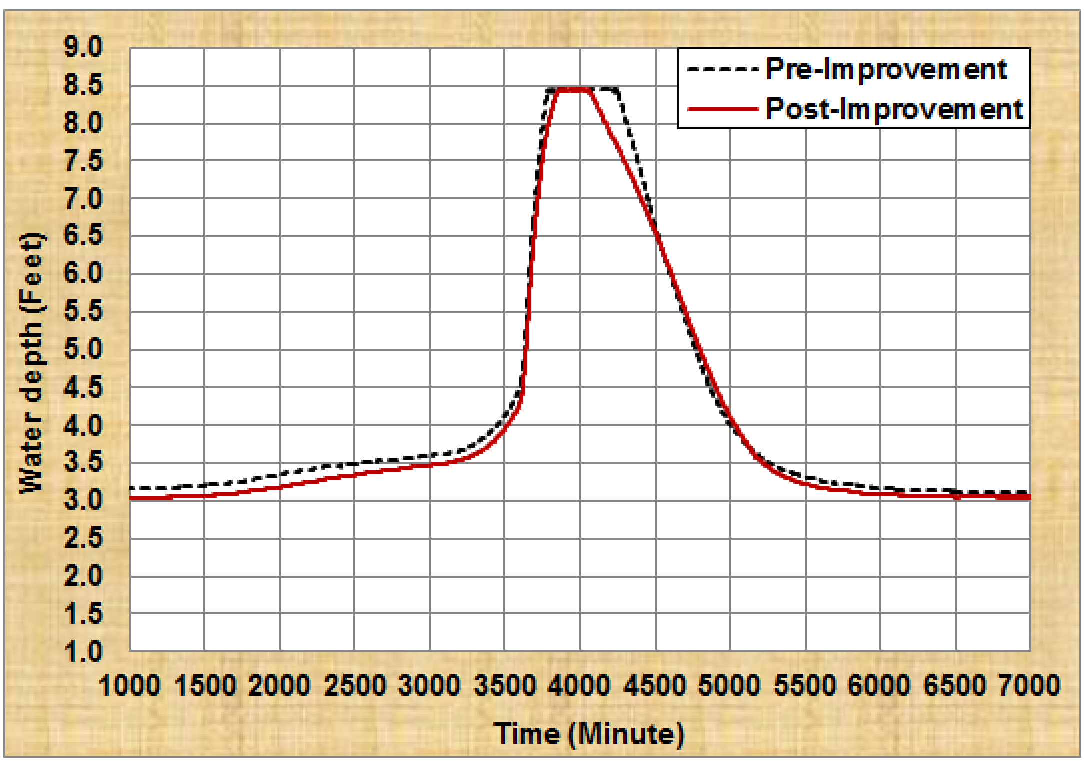

Figure 19b. Water Depth in C-Imp1SN for Pre- and Post-Improvements

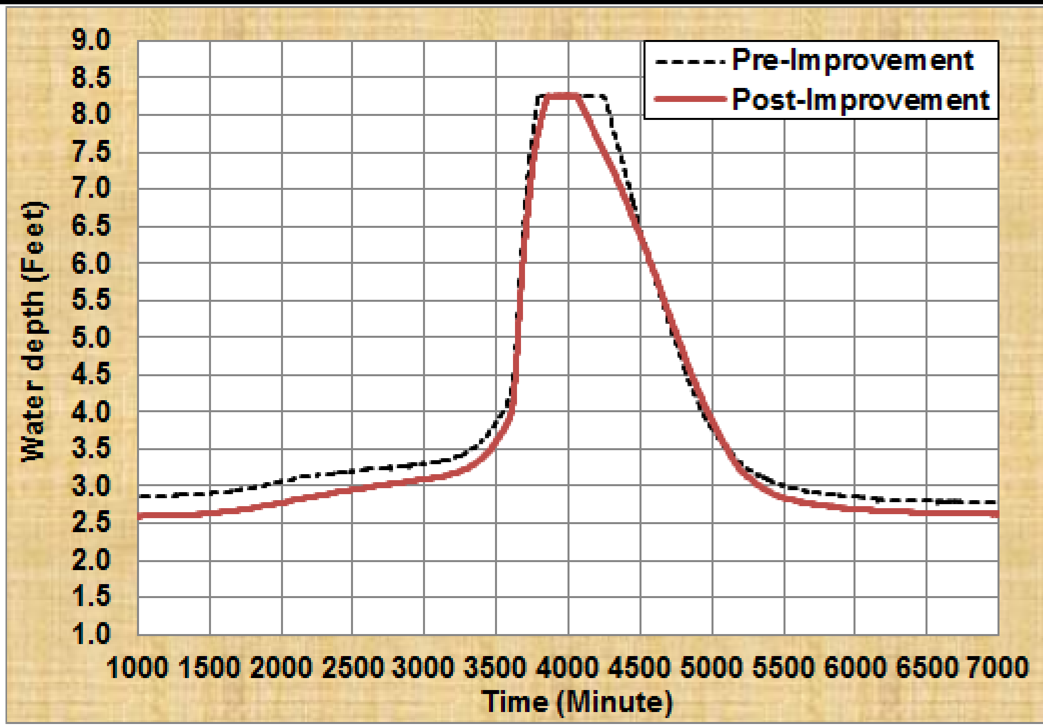

Figure 19c. Water Depth in C-Imp1S-2 for Pre- and Post-Improvements

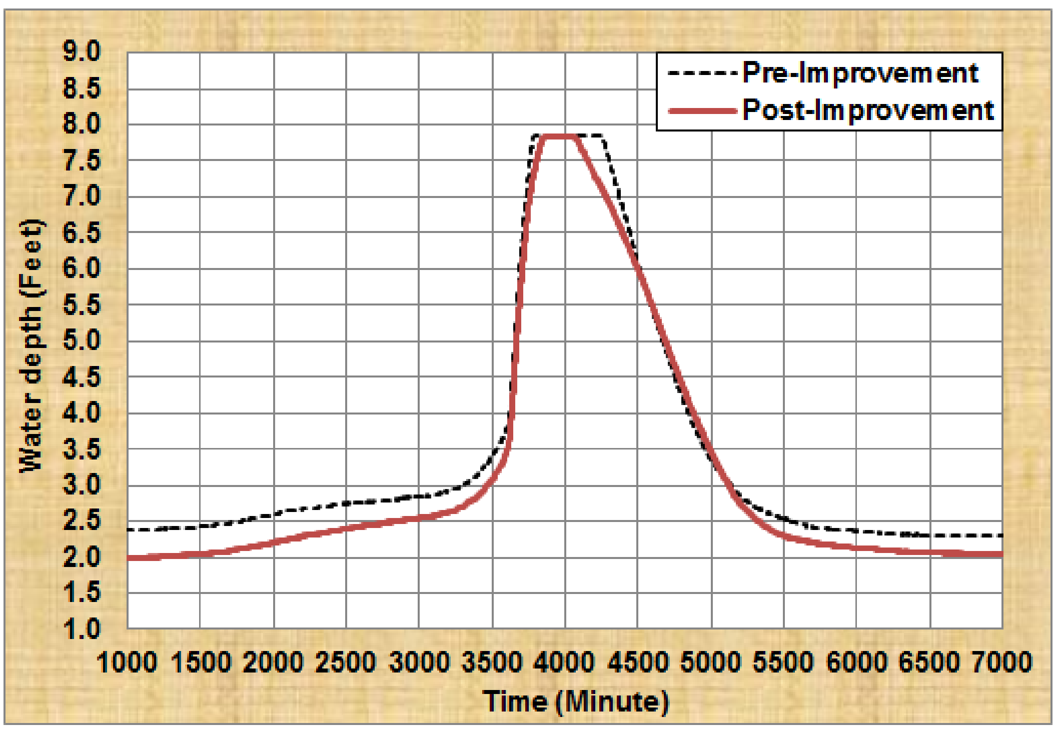

Figure 19d. Water Depth in C-Imp1-1E for Pre- and Post-Improvements

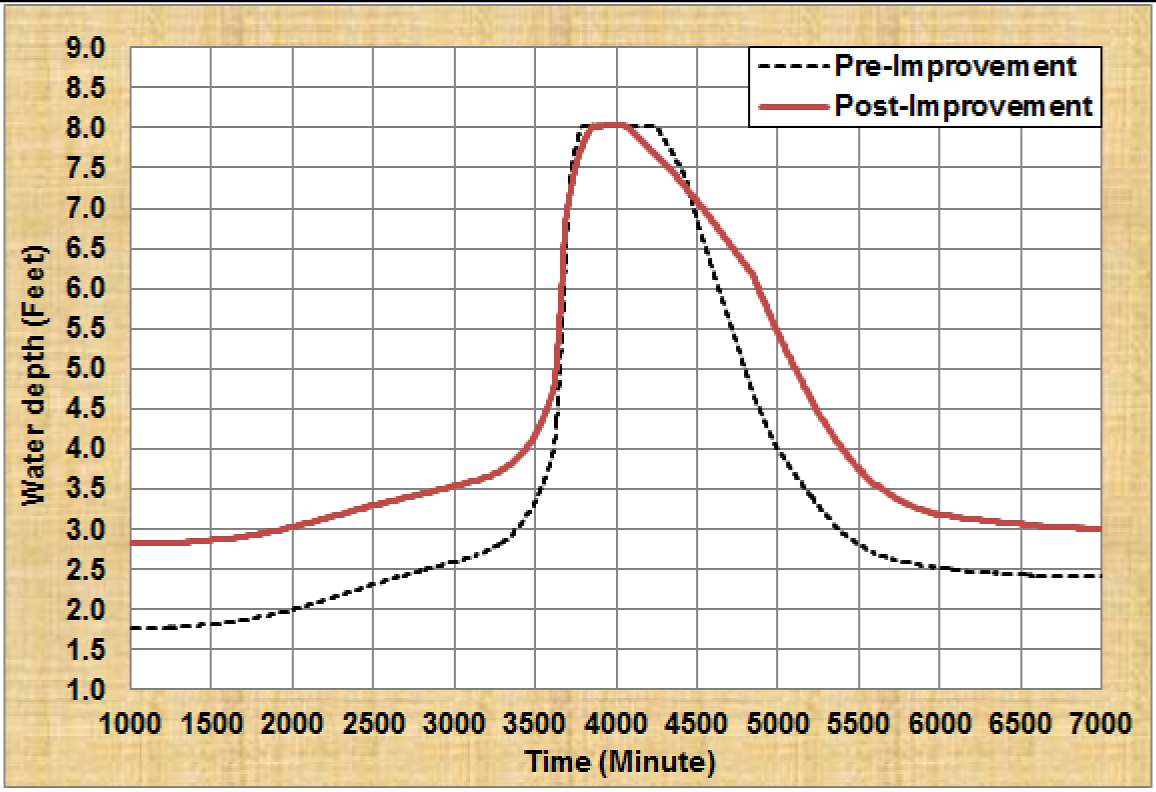

Figure 19e. Water Depth in C-Imp1-E for Pre- and Post-Improvements

Figure 20 compares water stages in Parcel B for pre- and post-improvement conditions. It indicates that based on the proposed improvements, the peak water stage will be 54.06 ft, NAVD88, while it would be 55.08 ft, NAVD88 for the pre-improvement conditions. The proposed improvements reduce the peak water surface elevation in Parcel B by approximately 1.0 ft.

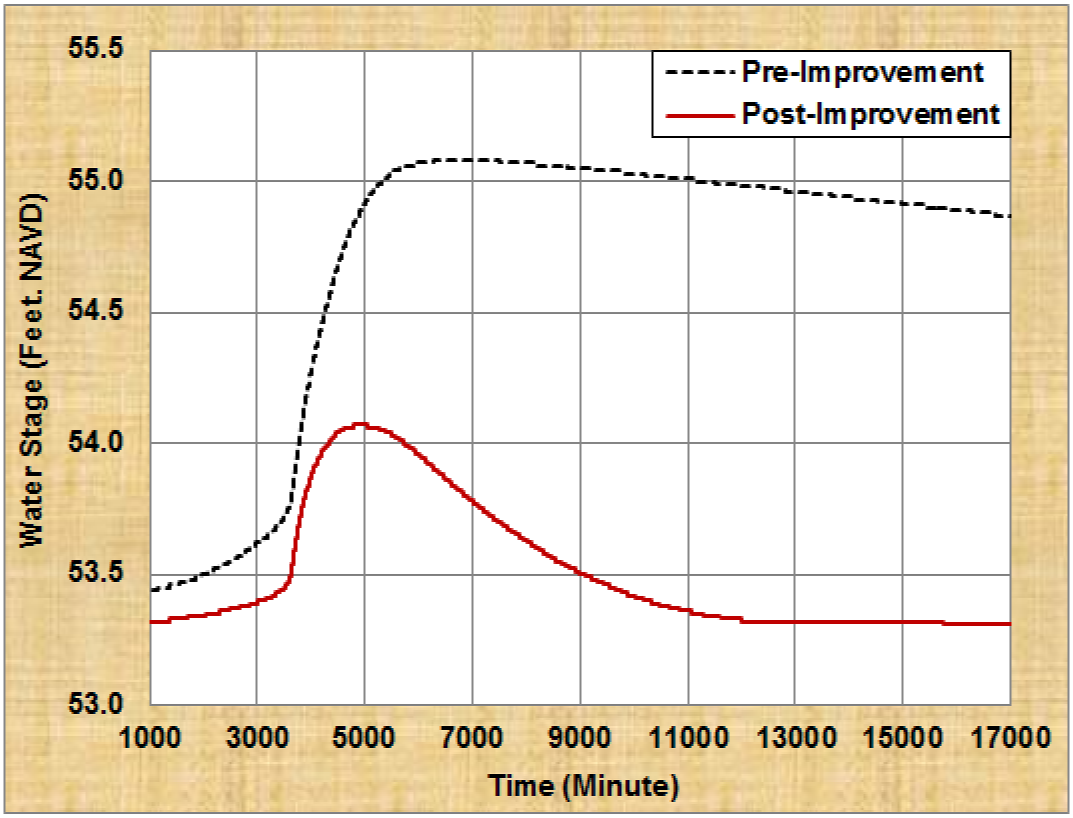

Figure 20.Water Stage in Parcel B for Pre- and Post-Improvements

In summary, for the 25-year, 3-day design storm event, in comparison to pre-improvement conditions, the proposed improvements provide the followings:

- significantly reduced peak water stages and durations in the canal just upstream of Site 1 and in the canals between Site 1 and Site 2;
- significantly reduced the peak water stages in Parcel B, and hence reduced flooding risk to the Parcel B Surrounding Area.

6.2.2 Flooding Condition: 100-Year Design Storm

Similar to the 25-year, 3-day design storm event, we compared water stages and flow hydrographs for the pre- and post-improvement conditions for the 100-year, 3-day design storm event. Results are similar to those for the 25-year design storm. **Appendix B** provides a summary of the simulation results. For the purpose of illustration, only the flow hydrograph at Site 13, the water stage at Parcel B, and water depths in Conduit C67 are presented in **Figures 21 through 23**. In comparison to the existing or pre-improvement system, the improved system will provide more conveyance capacity (see **Figure 21**); and will significantly reduce peak stages in Parcel B (see **Figure 22**), while not worsening the flooding risk in the canal segment just upstream of Site 1, and in the canal segments between Site 1 and Site 2 (See **Figure 23**).

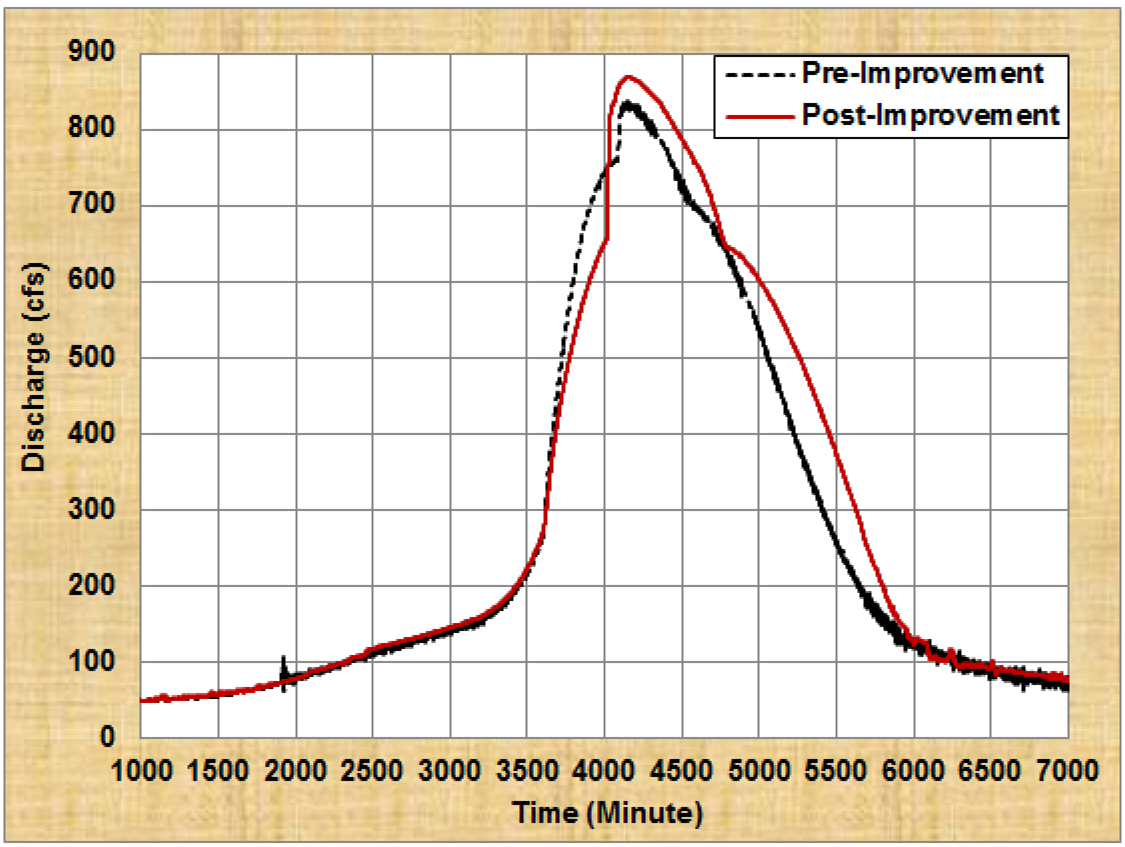

Figure 21. Flow at Site 13 for Pre- and Post-Improvements (100-Year, 3-Day Storm)

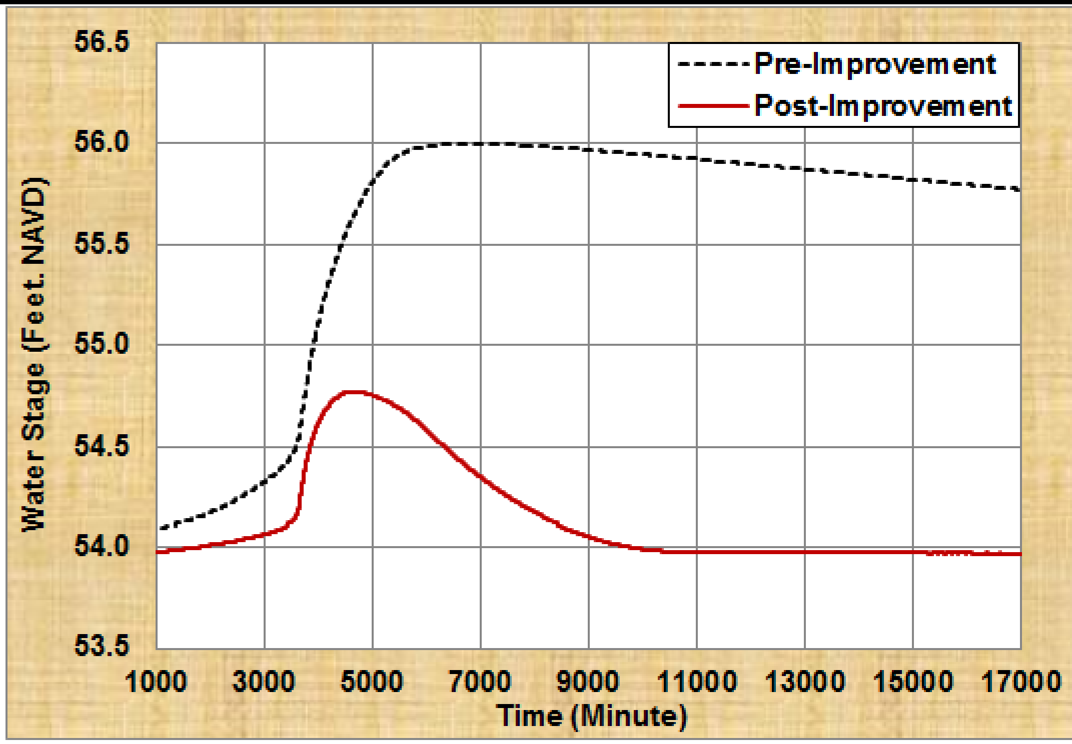

Figure 22.Water Stage at Parcel B for Pre- and Post-Improvements (100-Year 3-Day Storm)

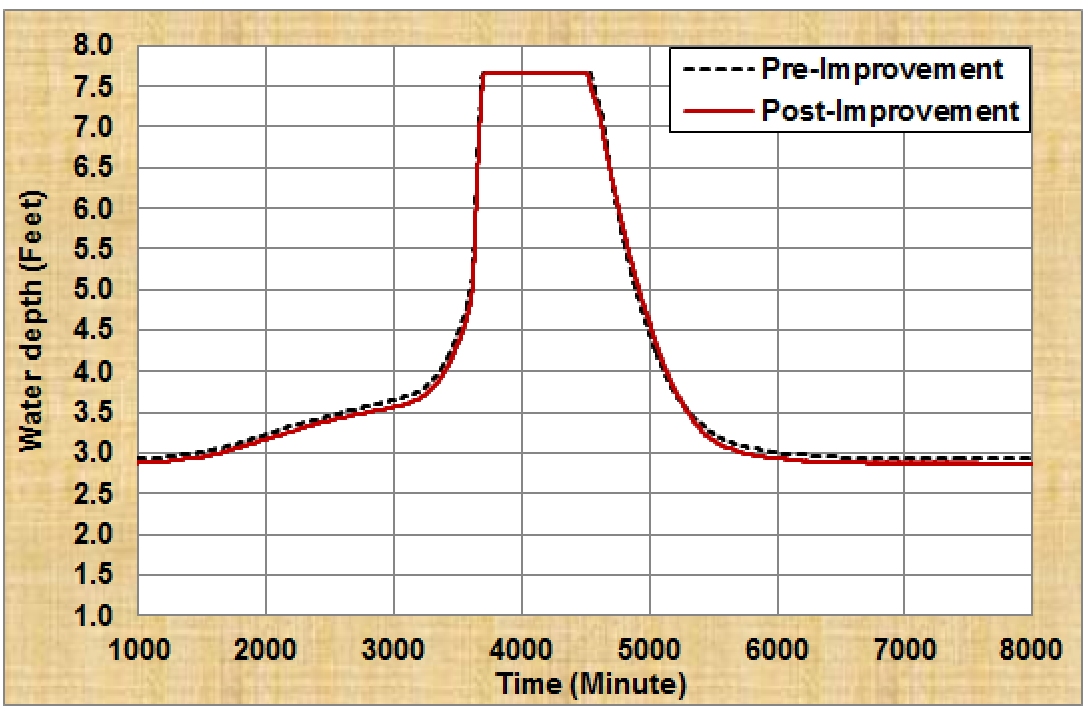

Figure 23. Water Depth in C67 for Pre- and Post-Improvements (100-Year, 3-Day Storm)

6.3 Flooding Concerns on Downstream Southeast Developed Area

Based on the previous hydrologic and hydraulic studies by ZFI (2011), there were some concerns that the proposed improvements may increase the flooding potential in the area located southeast of the project site along the Kissimmee River. The area is labeled as the “Southeast Developed Area” in **Figure 1**. The previous study (ZFI, 2011) suggested increasing the elevation of the current perimeter berm/access road surrounding Parcel B to 55.80 ft NAVD. Our present study demonstrates that the previous hydrologic and hydraulic study and its recommendation may have been overly conservative due to the followings.

- The ground surface elevation of the Parcel B Surrounding Area is approximately 57 ~ 58 ft NAVD88, as depicted in **Figure 24**. This analysis indicates that during 100-year, 3-day storm event, the peak water surface stage in Parcel B under the post-improvement conditions, may temporally get as high as elevation of 54.83 ft NAVD (see **Figure 22**), which is approximately 1.2 ft lower than that of the current pre-improvement conditions, and approximately 2.2 ft lower than its surrounding ground surface elevation of 57 ft NAVD88. The high ground surface of the Parcel B Surrounding Area forms another layer of flood protection for the Southeast Developed Area.
- The ground surface elevation around the Southeast Developed Area is approximately 59 ~ 60 ft NAVD88 (see **Figure 24**), which is significantly higher than the highest anticipated water stage (54.83 ft NAVD88) in Parcel B for the 100-year, 3-day storm event.
- The Southeast Developed Area is adjacent to the C37 Canal and its storm drainage system directly connects and discharges to the C37 Canal. Hence, this area is more prone to flooding due to high stages of the C37 Canal than due to high stages in Parcel B since high stages in the C37 Canal may prevent water from the area discharging to the canal.
- The proposed 4-72” culverts at Site 11 will allow water to discharge more quickly to Lake Hatchineha when the water stage in Parcel B is higher than that of Lake Hatchineha.

Based on the foregoing, it does not appear to be necessary to increase the current perimeter berm elevation to 55.80 ft NAVD, if the purpose of increasing elevation of the berm is to prevent the Southeast Developed Area from flooding due to high water stages in Parcel B.

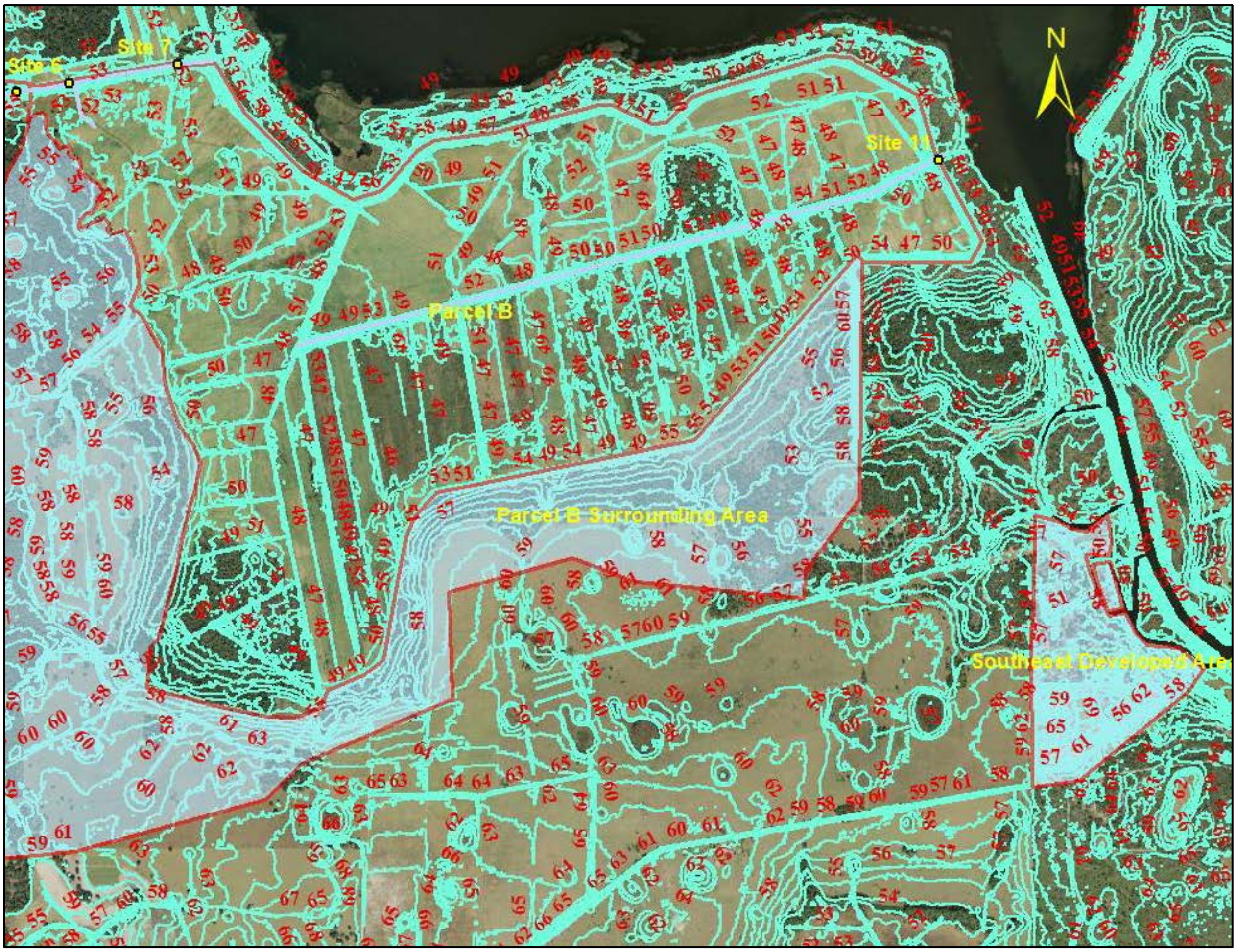

Figure 24. Ground Surface Grade of Parcel B and Its Surrounding Area

7.0 RISER FLASHBOARD ELEVATION FOR SYSTEM NORMAL OPERATION

We conducted model simulations to set riser flashboard elevations at the hydraulic structures that can allow the system to convey more water to Parcel B during normal and dry weather conditions, and during the 25- and 100-year storm events, can allow the system to back up water to the old sod-farm field between Site 2 and Site 13 (See **Figure 9**), while will not increase flooding risks in the canal segments between Site 1 and Site 2 (See **Figure 18**). **Table 12** below presents the maximum flashboard elevations at Site 1, Site 2, Site 4, Site 5, Site 6, Site 7, and Site 11 that we can set to achieve the aforementioned bi-fold objectives. During system normal operation, the bottom of the gate at Site 6 will be lifted up 3.0’, i.e., will leave 7.0’x3.0’ opening to convey water to Parcel B. **Figures 25a through 25f** compare the water depths in the canals between Site 1 and Site 2 for pre-improvements and post-improvements with the maximum flash-board elevations under the 25-year, 3-day design storm; and **Figures 26a through 26f** under 100-year, 3-day design storm. **Figures 25a through 25f** indicate that the post-improvements with the assigned flash-board elevations will still be able to alleviate the flooding risks around the canal segments between Site 1 and Site 2, and **Figures 26a through 26f** demonstrate that the post-improvements will not worsen the flooding risks around the canal segments.

Table 12. Riser Flashboard Elevation for System Normal Operation

Site Name	Pipe	Pipe/ Riser	# of Barrels	Diameter (in)	Length (ft)	Entry Invert (ft, NAVD)	Exit Invert (ft, NAVD)	Riser Top/Weir Crest El (ft, NAVD)	Flash Board Bottom El (ft, NAVD)	Maximum Flash Boord El (ft, NAVD)
Site 1	Pipe 1W	Pipe	1	48	31.0	53.5	53.5			
		Riser	1	72				59.0	53.5	55.5
	Pipe 1N	Pipe	2	48	37.0	53.5	53.5			
		Riser	2	72				59.0	53.5	55.0
Site 2	Pipe 2	Pipe	2	60	40.0	50.0	50.0			
		Riser	2	84				59.0	50.0	52.0
Site 3	Pipe 3	Pipe	1	72	47.0	50.0	50.0			
		Riser	1	N/A						
Site 4	Pipe 4S	Pipe	1	60	37.0	50.0	50.0		50.0	55.0
		Riser		84				59.0		
	Pipe 4W	Pipe	2	72	36.0	50.0	50.0		50.0	53.0
		Riser	2	96				58.0		
	Pipe 4N	Pipe	1	24	34.0	54.0	54.0			
		Riser	1	36				59.0		
Site 5	Pipe 5S	Pipe	2	72	31.0	50.0	50.0		50.0	55.0
		Riser	2	96				60.0		
	Pipe 5N	Pipe	3	84	40.0	50.0	50.0			
Site 13	Pipe	Pipe	N/A							
		Riser								
	Pipe 13	Pipe	3	72	39.0	47.0	47.0			
Site 6	Pipe 6	Pipe	2	60	42.0	49.5	49.5			
		Riser	2	84				56.0	52.5	56.0
Site 7	Pipe 7	Pipe	3	72	40.0	46.0	46.0			
		Riser	3	96				54.0	46.0	53.0
Site 11	Pipe 11	Pipe	4	72	85.0	43.0	42.5			
		Riser	4	96				54.0	50.5	

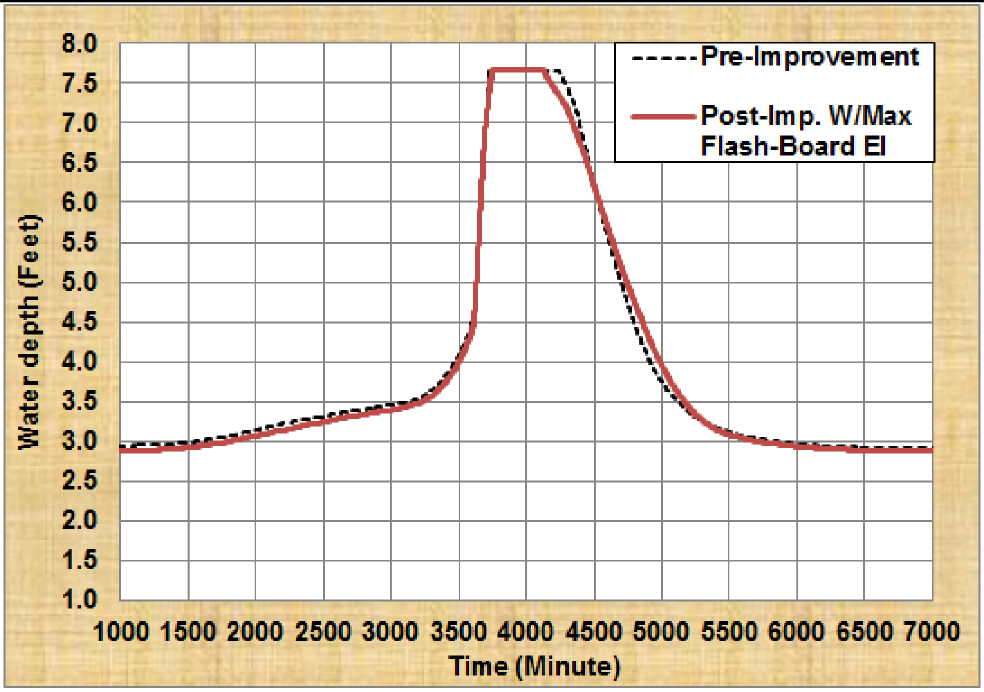

Figure 25a. Water Depth in C67 for Pre-Improvement and Post-Improvement with Maximum Flashboard Elevation for the 25-year, 3-Day Design Storm

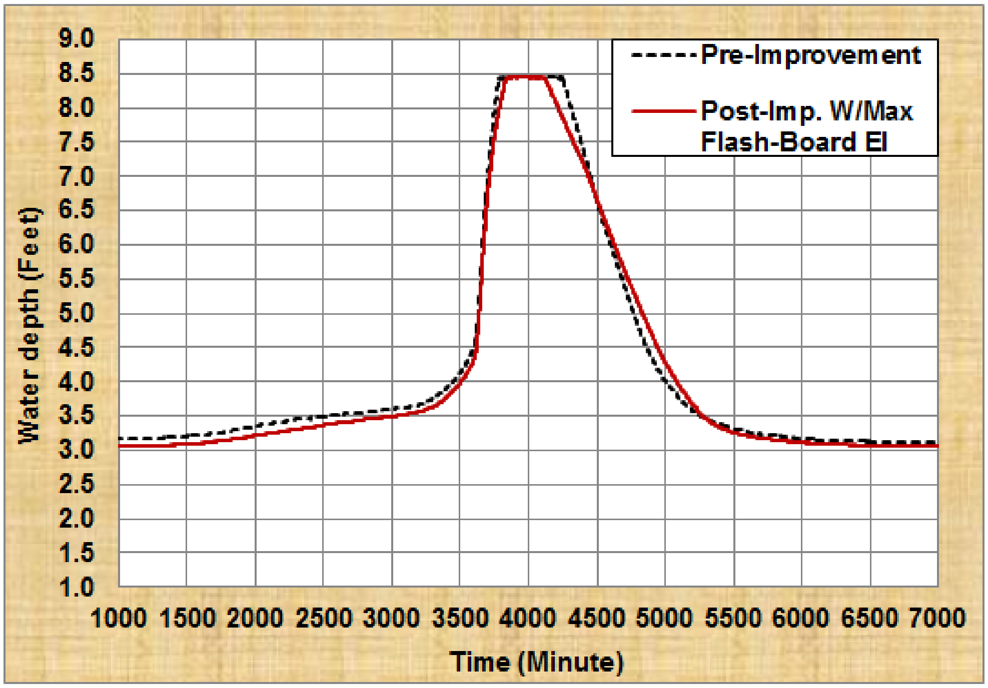

Figure 25b. Water Depth in C-Imp1SN for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 25-year, 3-Day Design Storm

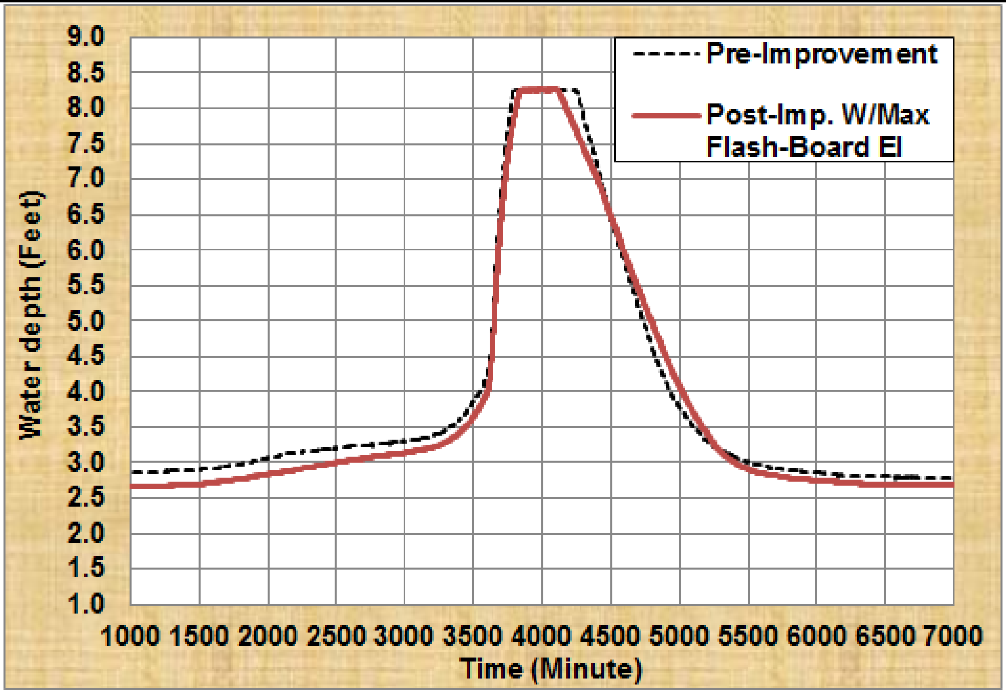

Figure 25c. Water Depth in C-Imp1S-2 for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 25-year, 3-Day Design Storm

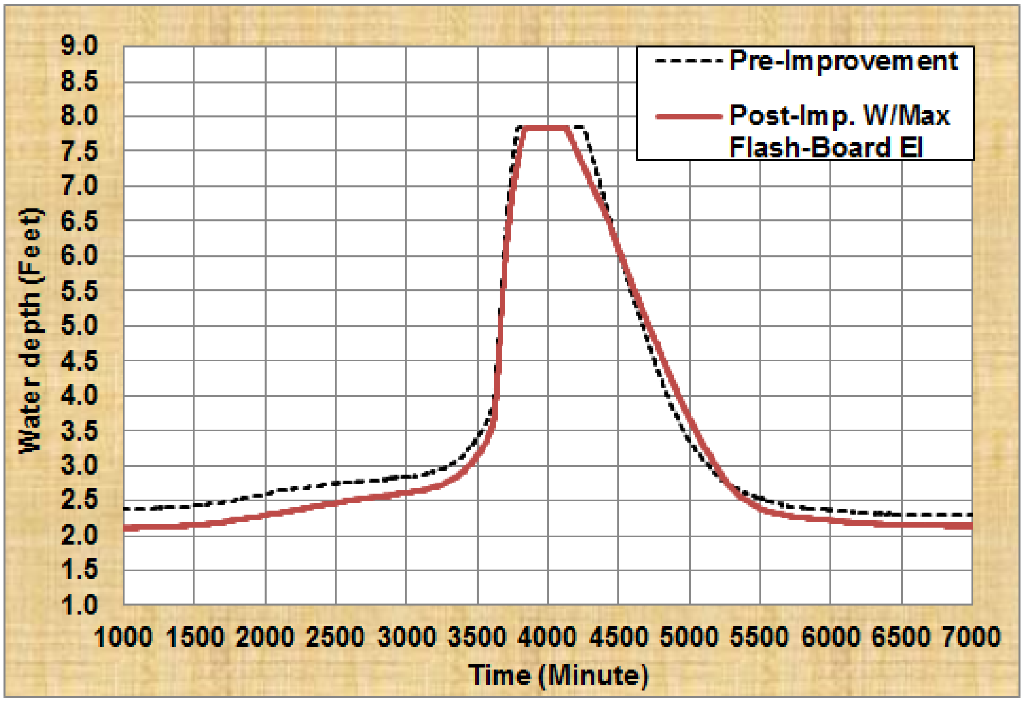

Figure 25d. Water Depth in C-Imp1-1E for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 25-year, 3-Day Design Storm

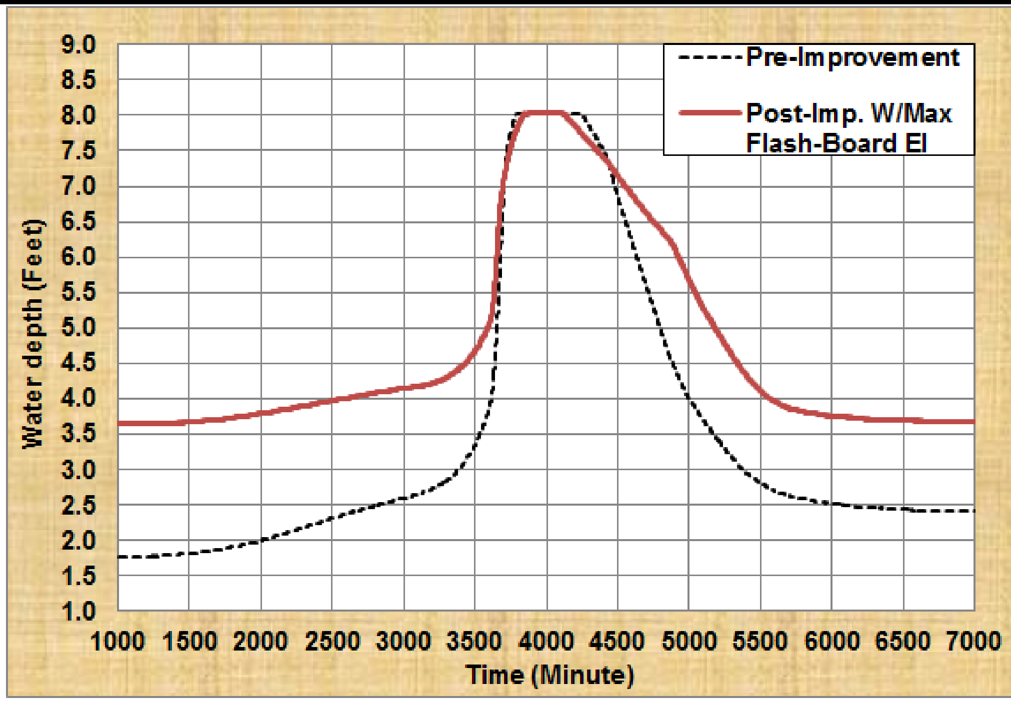

Figure 25e. Water Depth in C-Imp1-E for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 25-year, 3-Day Design Storm

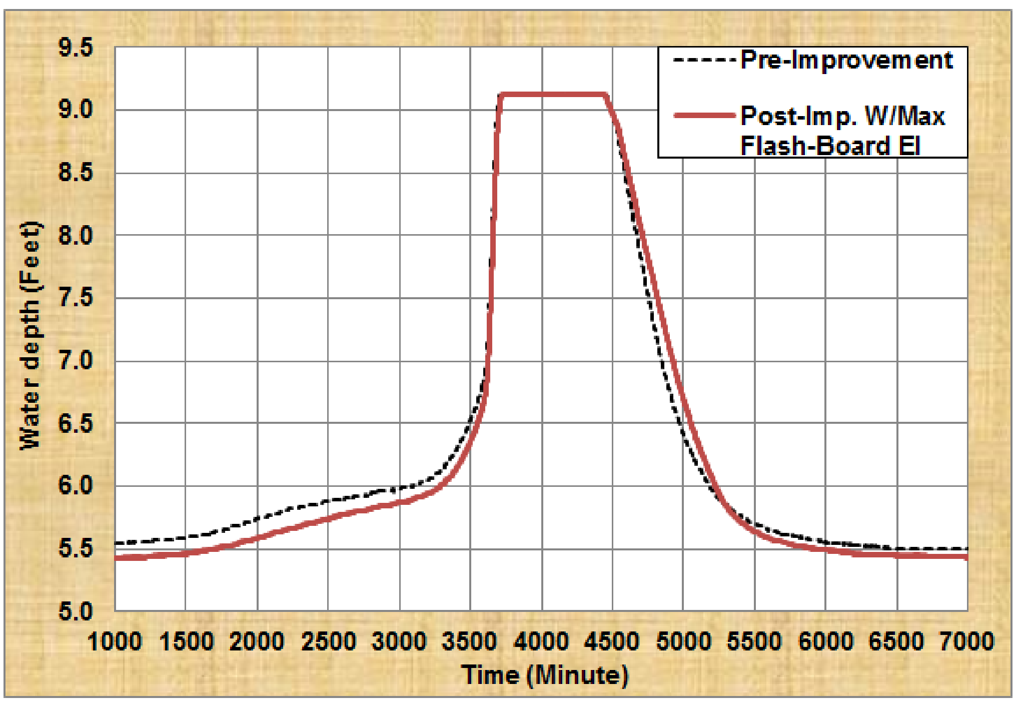

Figure 25f. Water Depth in C-Imp1S-Store for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 25-year, 3-Day Design Storm

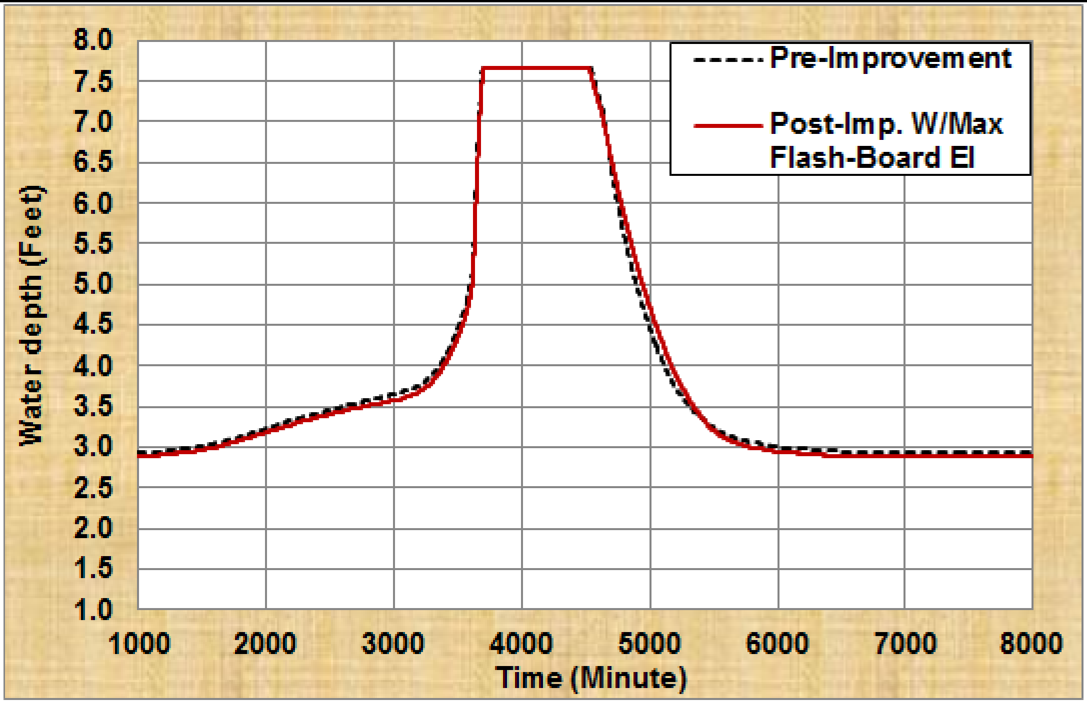

Figure 26a. Water Depth in C67 for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 100-year, 3-Day Design Storm

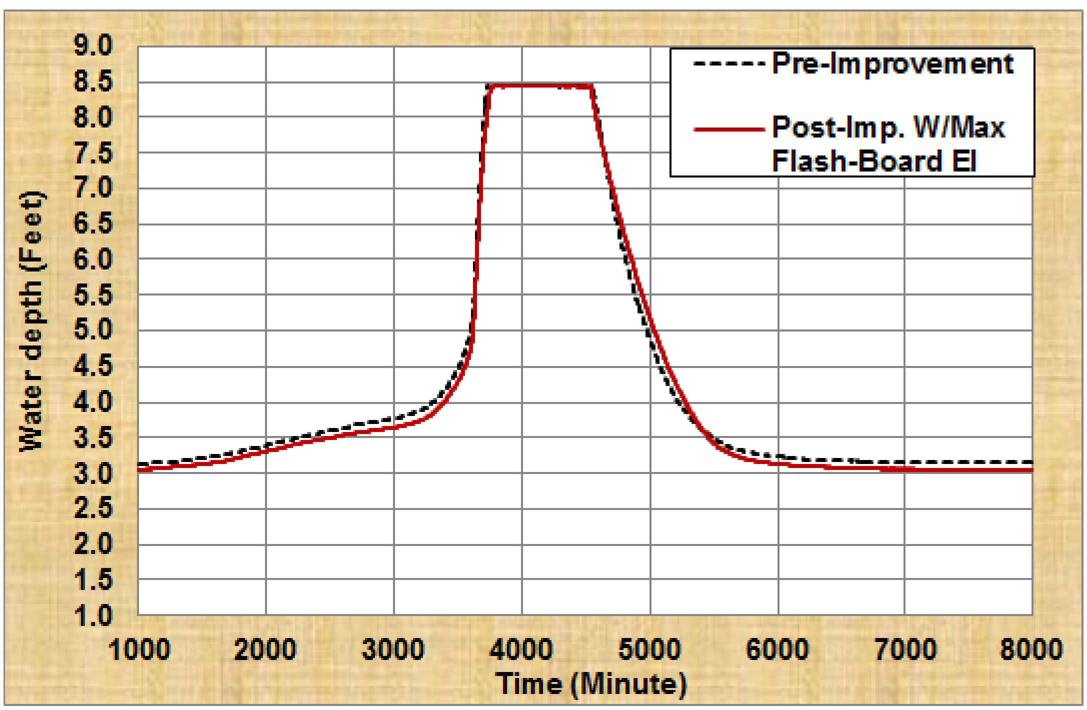

Figure 26b. Water Depth in C-Imp1SN for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 100-year, 3-Day Design Storm

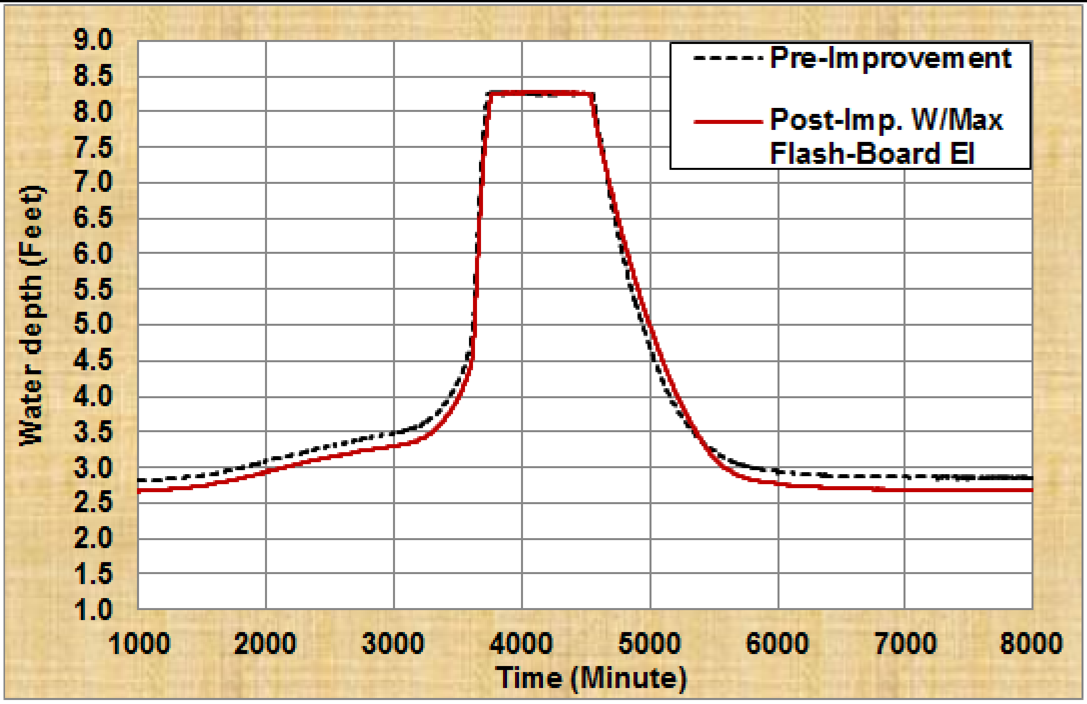

Figure 26c. Water Depth in C-Imp1S-2 for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 100-year, 3-Day Design Storm

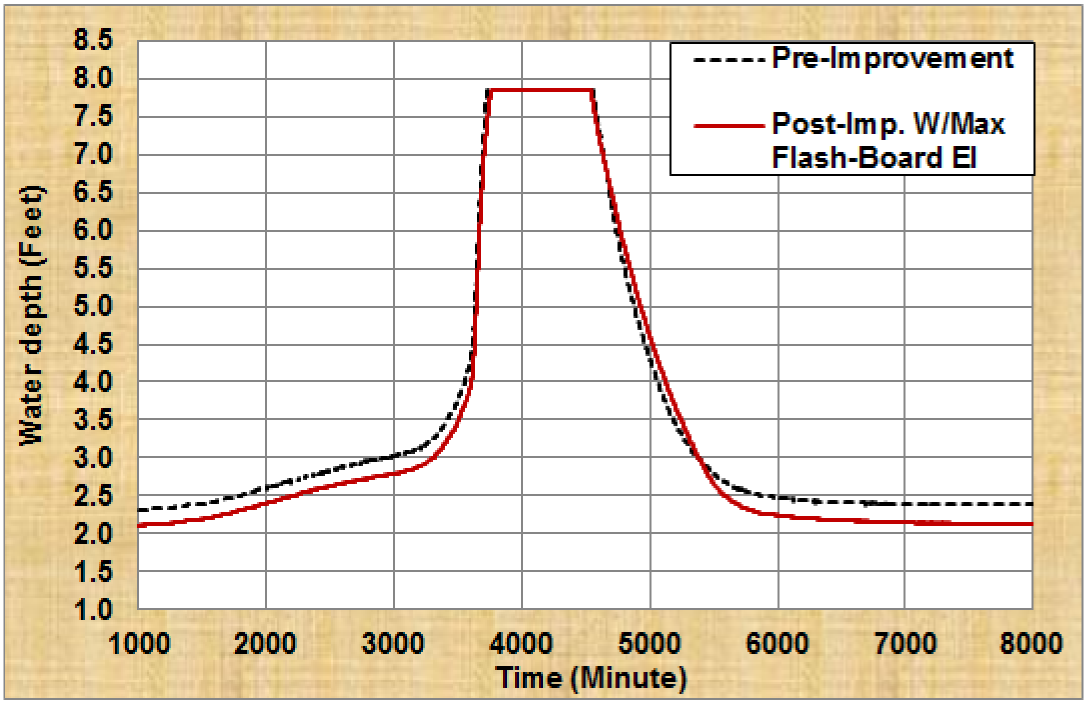

Figure 26d. Water Depth in C-Imp1-1E for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 100-year, 3-Day Design Storm

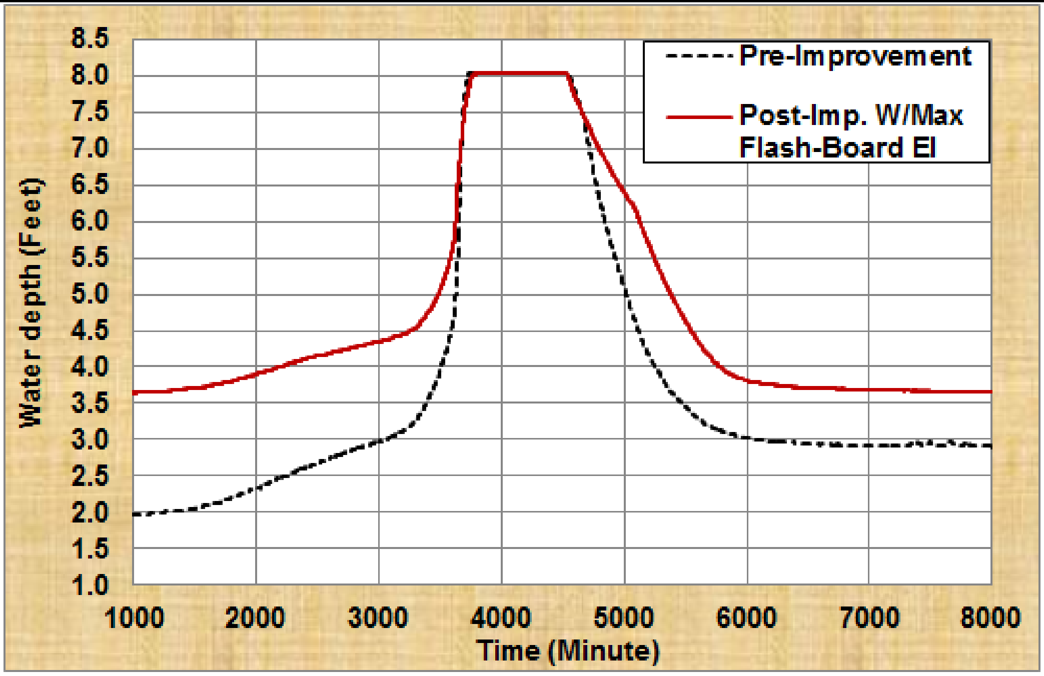

Figure 26e. Water Depth in C-Imp1-E for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 100-year, 3-Day Design Storm

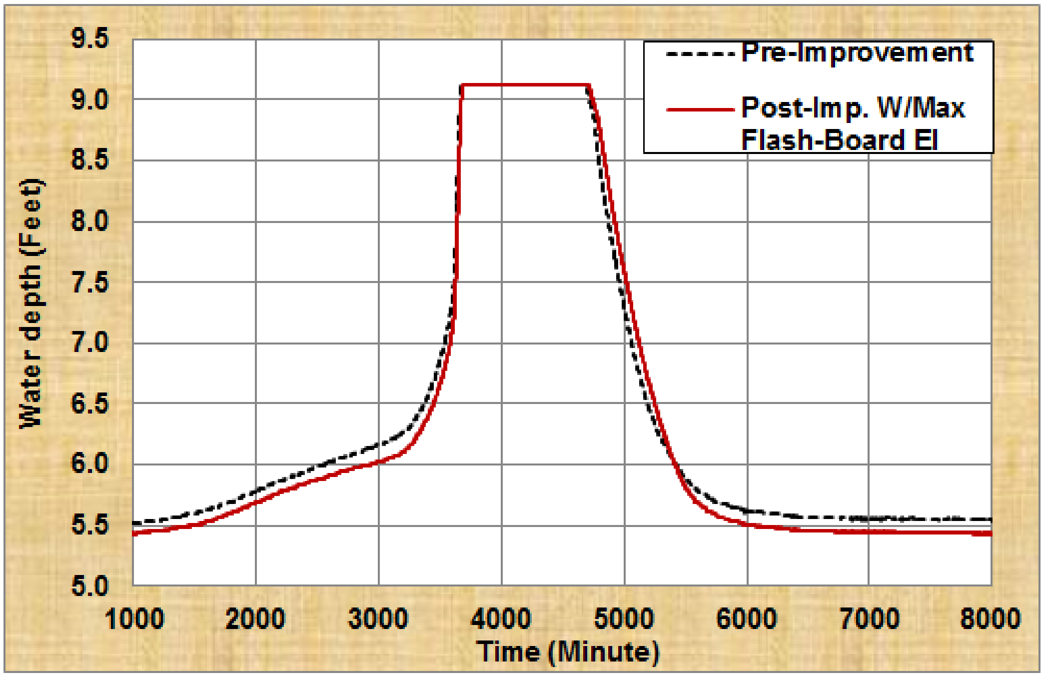

Figure 26f. Water Depth in C-Imp1S-Store for Pre-Improvement and Post-Improvement with Maximum Flash-board Elevation for the 100-year, 3-Day Design Storm

8.0 CONCLUDING REMARKS

We developed the integrated hydrologic and hydraulic SWMM model for the project area based on the most up-to-date, and readily available geographic information data - terrain data, land use/land cover information, soil data, and structure and canal survey data. We validated the model hydrology by using USGS regression equation, and hydraulic routing network based on the limited existing stream-gauging data. The model was utilized to evaluate the capacity of the proposed hydraulic structures and the anticipated water stages during the 25-year 3-day design storm event. We also reviewed the calculated water stages during the 100-year, 3-day design storm event for pre- and post-improvement scenarios. Based on the model simulation results, in comparison to the pre-improvement conditions, the proposed improvements provide the following benefits:

- significantly attenuate flooding potential in the canal just upstream of Site 1 and the canal segments between Site 1 and Site 2;
- increase system conveyance capacity;
- significantly reduce the peak water surface elevation in Parcel B;
- in contrast to the previous study by ZFI (2011), it does not appear to be necessary to increase perimeter berm elevation around Parcel B to reduce the flooding potential in the Southeast Developed Area due to high water stage in Parcel B.

For the system normal operation, based on the model simulations, we set up the maximum flashboard elevations of the risers at the hydraulic structures that will allow the system to convey more water to Parcel B during normal and dry weather conditions, and allow the system to back up water to the old sod-farm field between Site 2 and Site 13 during the 25- and 100-year storm events, while will not increase flooding risks in the canal segments between Site 1 and Site 2.

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ZFI Engineering and Construction, Inc., 2011. Rolling Meadows Wetland Restoration Phase I Hydrologic Modeling: Final Model Summary Report. Contract No. 4600000958-WO02R1. South Florida Water Management District, West Palm Beach, Florida.

Appendix A: Pre- and Post-Improvement Simulation Results for 25-Year, 3-Day Design Storm (in electronic format and will be provided)

Appendix B: Pre- and Post-Improvement Simulation Results for 100-Year, 3-Day Design Storm (in electronic format and will be provided)