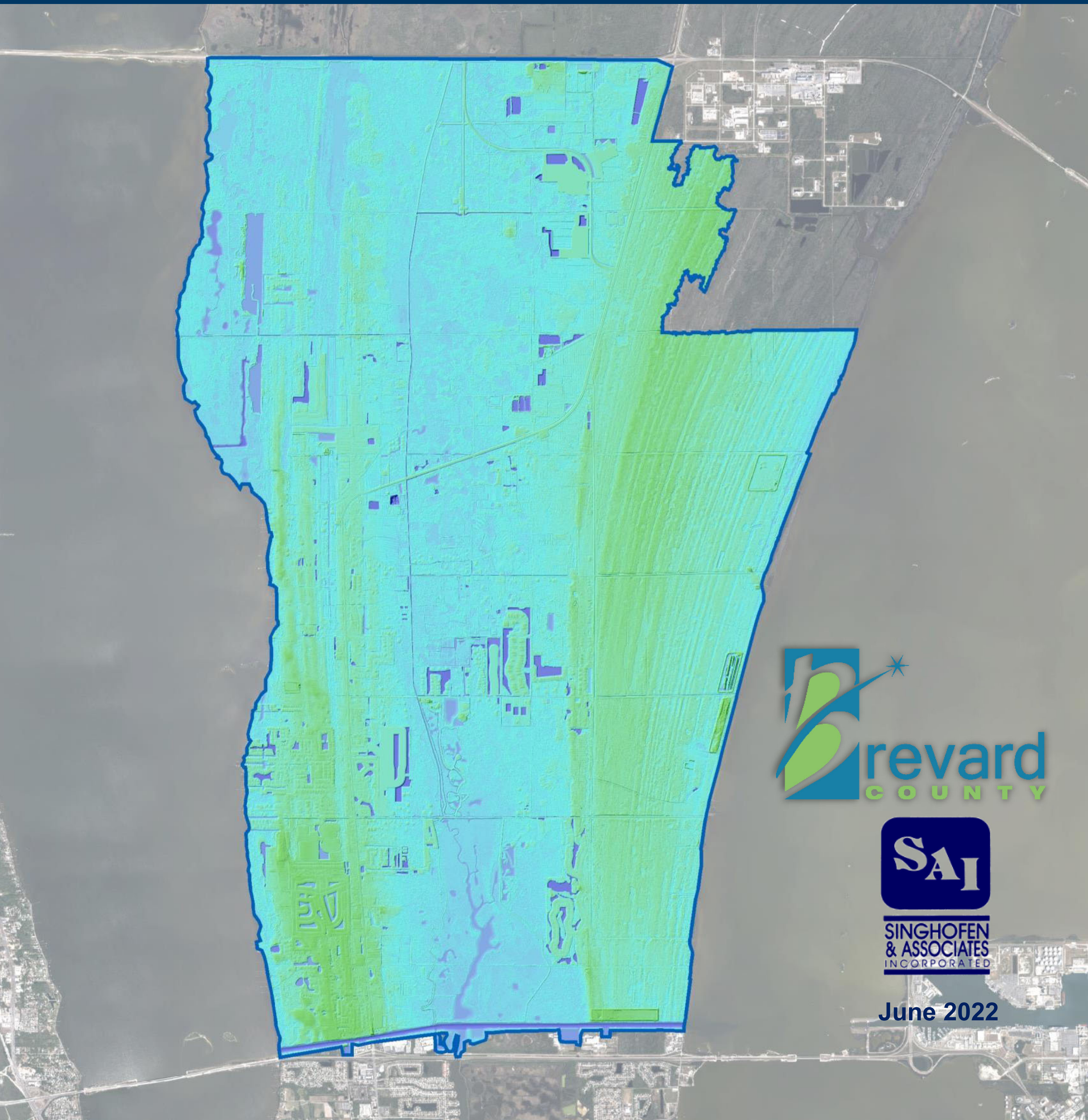
North Merritt Island
H&H Modeling Study Report



June 2022

CERTIFICATION BY A

REGISTERED PROFESSIONAL ENGINEER

PROJECT NAME: North Merritt Island H&H Modeling Study Report - FINAL

I HEREBY CERTIFY THAT THE MATERIAL AND DATA CONTAINED IN THIS DOCUMENT WAS PREPARED UNDER THE SUPERVISION AND DIRECTION OF THE UNDERSIGNED, WHOSE SEAL AS A REGISTERED PROFESSIONAL ENGINEER IN THE STATE OF FLORIDA IS AFFIXED BELOW.

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Abbreviations and Acronyms

The following is a list of acronyms and /or abbreviations used throughout this report:

1D / 2D	1 or 2 Dimensional
AEI	Applied Ecology, Inc.
CFWI	Central Florida Water Initiative
CMS	Coastal Modeling System
County	Brevard County Natural Resources Management Department
DEM	Digital Elevation Model
District	Southwest Florida Water Management District
ECTFX	East-Central Florida Transient Expanded
ERP	Environmental Resource Permit
FDEP	Florida Department of Environmental Protection
FDOT	Florida Department of Transportation
FEMA	Federal Emergency Management Agency
FIPS	Federal Information Processing Standards
ft	Feet/Foot
GDB	Geodatabase
GIS	Geographic Information System
GWIS	Geographic Watershed Information System
GW-SW	Groundwater-Surface Water
H&H	Hydrologic & Hydraulic
h:v	Horizontal:Vertical
i.e.	That is
ICPR	Interconnected Channel and Pond Routing
IRL	Indian River Lagoon
JEA	Jones Edmunds and Associates, Inc.
KM	Kilometer
LDC	Land Development Code
LiDAR	Light Detection And Ranging
LU	Lookup (Table)
M&E	Morgan & Eklund, Inc.
MAE	Mean Absolute Error
ME	Mean Error
MLLW	Mean-Lower-Low Water
NASA	National Aeronautics and Space Administration
NAD	North American Datum
NAVD88	North American Vertical Datum of 1988
NEXRAD	Next Generation Weather Radar
NGVD	National Geodetic Vertical Datum
NGVD29	National Geodetic Vertical Datum of 1929

NHD	USGS National Hydrography Dataset
NMI	North Merritt Island
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NSE	Nash-Sutcliffe Efficiency
NWL	Normal Water Level
QA	Quality Assurance
QA/QC	Quality Assurance / Quality Control
QC	Quality Control
RMSE	Root Mean Square Error
SAI	Singhofen & Associates, Inc.
SF	Square Feet/Foot
SFWMD	South Florida Water Management District
SJRWMD	St. Johns River Water Management District
Sq.	Square
SR	State Road
SW-GW	Surface Water-Groundwater
SWAMP	Stormwater Asset Management Program
SWFWMD	Southwest Florida Water Management District
Tc	Time of Concentration
USGS	United States Geological Survey
YR	Year

1.0 Introduction and Purpose

1.1 Authorization

The North Merritt Island H&H Modeling Study is being performed by Singhofen & Associates Inc. (SAI) for the Brevard County Natural Resources Management Department (County) under Agreement No. 20-4663-001-HHM.

1.2 Project Location and General Description

The North Merritt Island (NMI) Watershed is located in Brevard County and spans approximately 38 square miles, from the Barge Canal north to Nasa Parkway (see **Figure 1.1**). The watershed is bound by NASA's Kennedy Space Center to the north and by the Merritt Island National Wildlife Refuge to the east.

The NMI watershed drains into three areas: the Indian River Lagoon (IRL) to the west, the Banana River to the east, and the Canaveral Barge Canal to the south through the Sykes Creek. Much of the watershed area is being converted to suburban landscape that has changed the natural drainage patterns. Dual ridges run north-south along N. Courtney Pkwy/SR 3 to the west and the federal property to the east, creating a broad low-lying depressional area that encompasses much of the island. This depressional area is relatively flat and provides little relief, relying primarily on man-made drainage works such as sub-division ponds and open cut ditches to interconnect low lying areas to their ultimate outfalls. Compounding the lack of relief are the generally poorly drained soils that are dominant except for select areas along the higher elevation ridges, and a shallow groundwater table. Although retrofit projects have been undertaken by the County to some degree of success, flood duration and extent can be compounded by issues outside of the County's control. Due to the interconnected nature of the IRL and Banana River with the groundwater and surface water within the watershed, increases in the boundary/tailwater conditions can result in adverse impacts by removing natural soils storage, taking over natural and man-made storage, and inhibiting discharge by reducing the already limited hydraulic gradient. These issues, along with storm surge, can compound to cause extended periods of inundation for property owners that can be surrounded by flood waters that cannot be solved with portable pumps, as the water has nowhere to go.

1.3 Purpose and Objectives

This project involves the development of an integrated surface water-groundwater (SW-GW) model (ICPR4) for the NMI watershed. The completed model will yield results for analysis of current and future flooding conditions and will serve as a base model to evaluate potential flood control and natural system improvement projects and other physical changes to watershed. The model will also consider historical conditions in the IRL and Banana River as boundary conditions for the NMI watershed.

This is accomplished by completing the following objectives:

- Collect and review pertinent data from the County and/or local communities
- Acquire field data measurements
- Develop a detailed hydrologic and hydraulic (H&H) model to characterize storm events throughout the watershed, including 1D, 2D, and groundwater features
- Calibrate and verify hydrologic and hydraulic (H&H) model

1.4 General Scope of Work

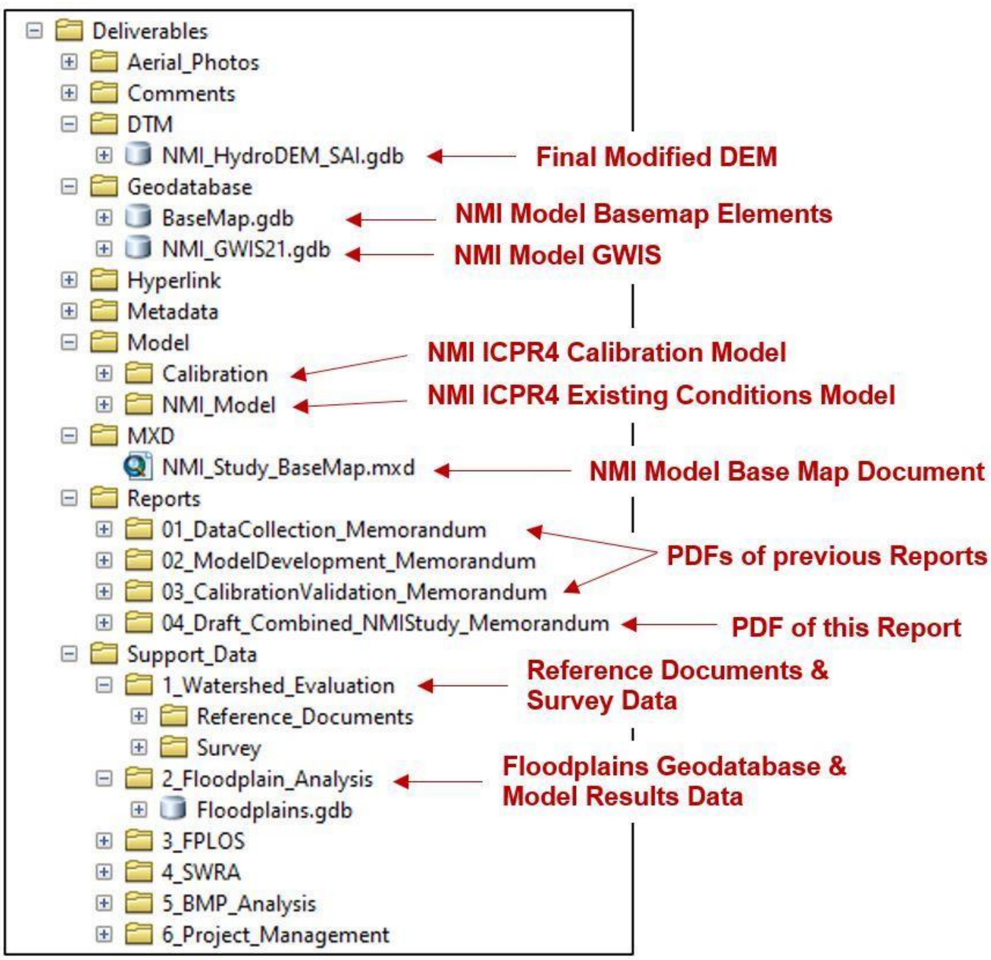
This memorandum summarizes the data collection and model development work completed for the project to date. The electronic deliverable is included on a hard drive accompanying this Model Development Memorandum. The general scope of work for these tasks is presented below:

- Task 1: Data Collection and Review – This task includes collecting and reviewing data pertinent to the model development efforts from the applicable agencies and developing a data catalog to store all data collected for the project along with spatial representation of the data where applicable.

- Task 2: Watershed Evaluation – Initial Desktop Model Element Development – This task includes the spatial development of the model network and establishing boundary conditions. Subtasks include digital elevation model (DEM) modifications for topographic voids and hydro-corrections, existing model data migration and quality control (QC), model network development of 1D, 2D, and groundwater features, and coastal boundary evaluation/trend analysis.
- Task 3: Field Data Acquisition – This task involves conducting field reconnaissance to verify drainage patterns and structure information at identified locations, identifying survey and maintenance needs as well.
- Task 4: Model Hydraulic and Hydrologic Parameterization – This task involved parameterizing the model network developed under Task 2.
- Task 5: Model Setup, Execution, Debug, and Stabilization – Only Subtask 5.1 is included in this phase of the work. Subtask 5.1 involves generating the Interconnected Channel and Pond Routing version 4 (ICPR4) model from the project Geographic Watershed Information System (GWIS) geodatabase.

1.5 Electronic Deliverables

This memorandum accompanies electronic data related to the development of the existing conditions North Merritt Island watershed model. This data includes: ICPR4 H&H model, model features in GIS (GWIS format), supporting model data in GIS (soils, landuse, etc.), the topographic digital elevation model (DEM), data collection portion of the North Merritt Island study including GIS features and reference documents (i.e., plans, reports, surveys, etc.) that are being used to develop the preliminary model elements. The project deliverables are submitted within the following directory structure:



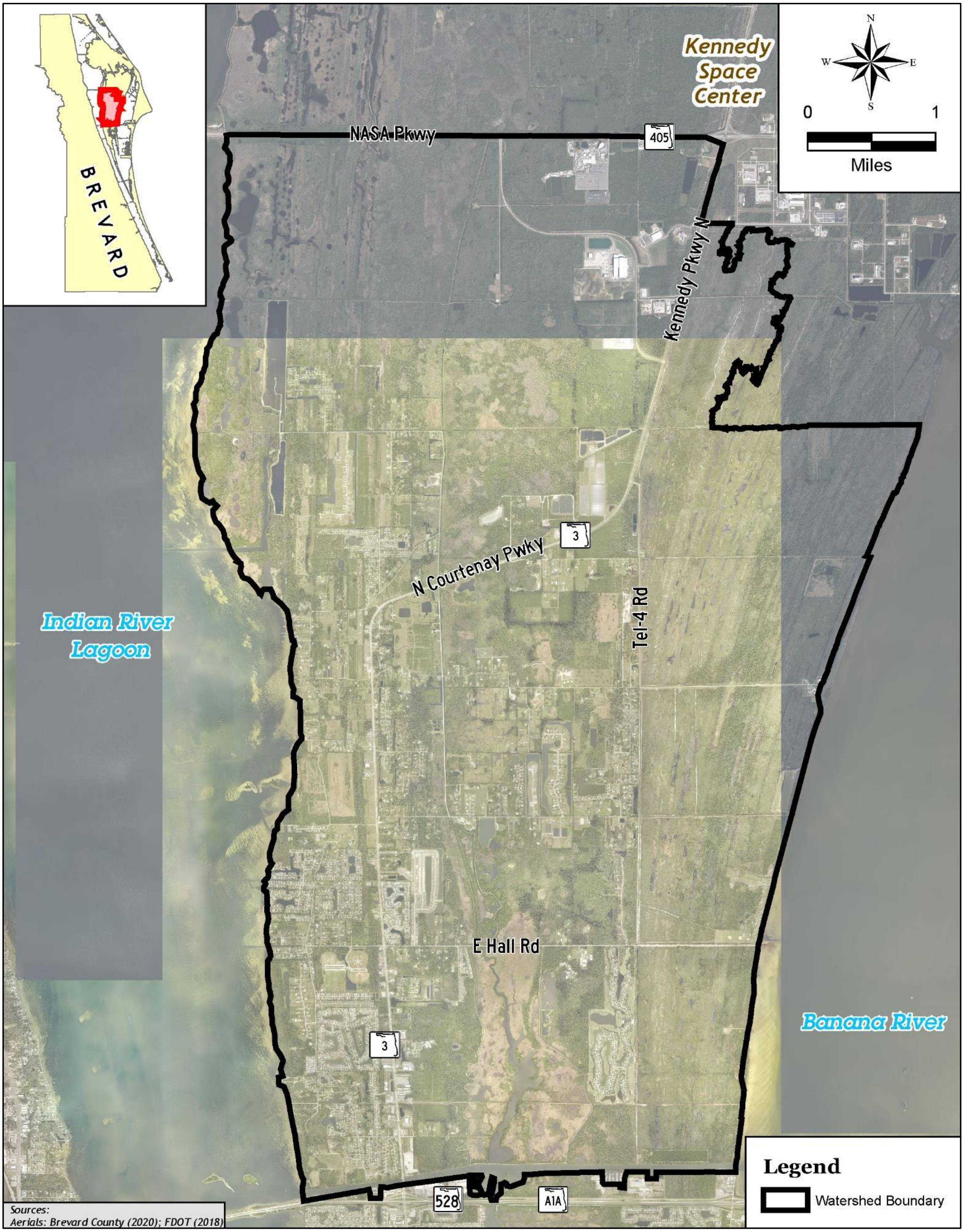


Figure 1.1: Vicinity Map of the NMI Watershed

2.0 Data Collection

2.1 Existing Model and Infrastructure Data

Model Data: Several ICPR3 models along with accompanying GIS data were provided by the County to serve as a base for the model development efforts. The following information was provided:

- ***NMI_BREVARD_w_NASA.ICP*:** This is the most current H&H model of the NMI watershed as of the start of this project, which includes updates conducted by DRMP. This model will serve as a basis for the ICPR4 model development.
- ***With_Hall&Chase&Crisafulli_Pumps.ICP*:** Design model for the Hall Road, Chase Hammock, and Crisafulli Road pump station projects. This ICPR3 model was based on the *NMI_BREVARD_w_NASA.ICP* model. It should be noted that of the three pump station designs included in this model, only the Hall Road pump station was constructed.
- ***NMI_Brevard.gdb*:** This is the model spatial geodatabase accompanying the NMI watershed model (*NMI_BREVARD_w_NASA.ICP*).
- ***DRMP_Node.shp*, *DRMP_Reach.shp*, *Sub-basins.shp*:** These shapefiles reflect the additional model features incorporated as part of the DRMP updates.

It should be noted that the GIS files provided were incomplete when compared to the model. The spatial datasets were missing over 30 basins and more than 10 nodes and links. Data included in the *NMI_BREVARD_w_NASA.ICP* model is assumed to be correct as-is, as directed by the County.

County Stormwater Infrastructure Database: The County provided the current Stormwater Asset Management Program (SWAMP) database, a geospatial database of stormwater features throughout the County. The SWAMP geodatabase (*Natural_Resources.gdb*) includes spatial location, geometry, size information, and elevation data where available. The County has indicated that all vertical information in the SWAMP database is in NAVD88 and is accurate.

In accordance with direction from the County, where data discrepancies exist between the ICPR3 model and the SWAMP database, the model information is to take precedence. Field verification will be conducted where necessary to resolve significant discrepancies based on engineering judgement.

2.2 Hydrologic & Hydraulic Data Collection

Data collected related to the hydrological characteristics of the watershed are summarized below. An electronic copy of this data is provided with the electronic deliverable accompanying this report.

2.2.1 Soil Data: Soil layers from the United States Department of Agriculture Natural Conservation Resources Service (NRCS) were obtained for this project (2019). The soils layer indicates that only 7% of the soils within the watershed are well-drained, type-A soils. Over 80% of the soils within the watershed are hydrologic soil group A/D, B/D, or C/D. These soils are well-drained to moderately drained during dry conditions and poorly drained during wet conditions. A soils map of the current NRCS soils data is included as **Figure 2.1**.

2.2.2 Land Use Characterization: Land use data (2014) was obtained from SJRWMD. The data set was updated by the SAI Team based upon a review of the 2020 high-resolution aerial imagery. The land use classifications are based on the Florida Land Use, Land Cover Classification System. The land use for the watershed is presented in **Figure 2.2**. The land use breakdown for the study area is provided in **Table 2.1**.

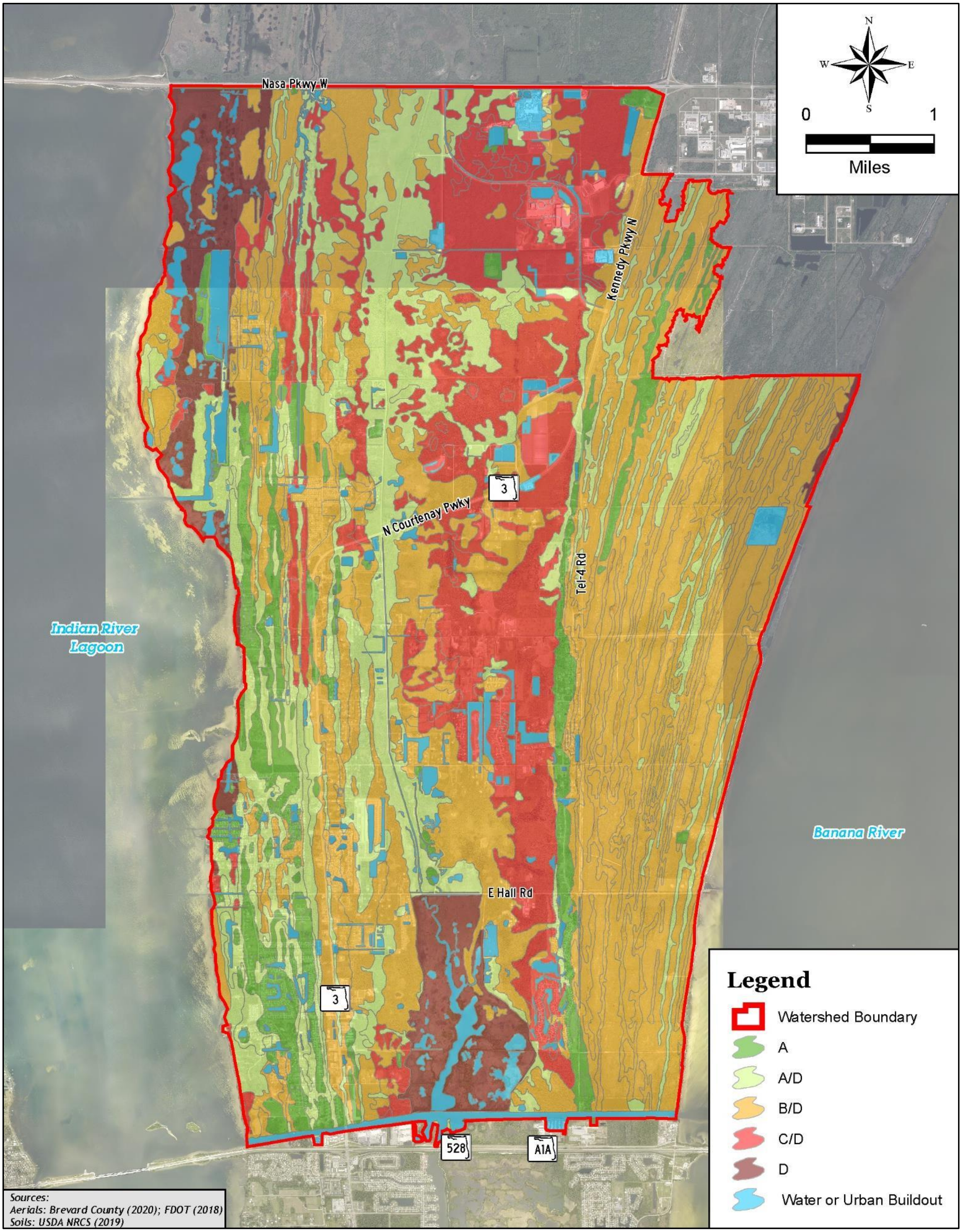


Figure 2.1: Soils Map

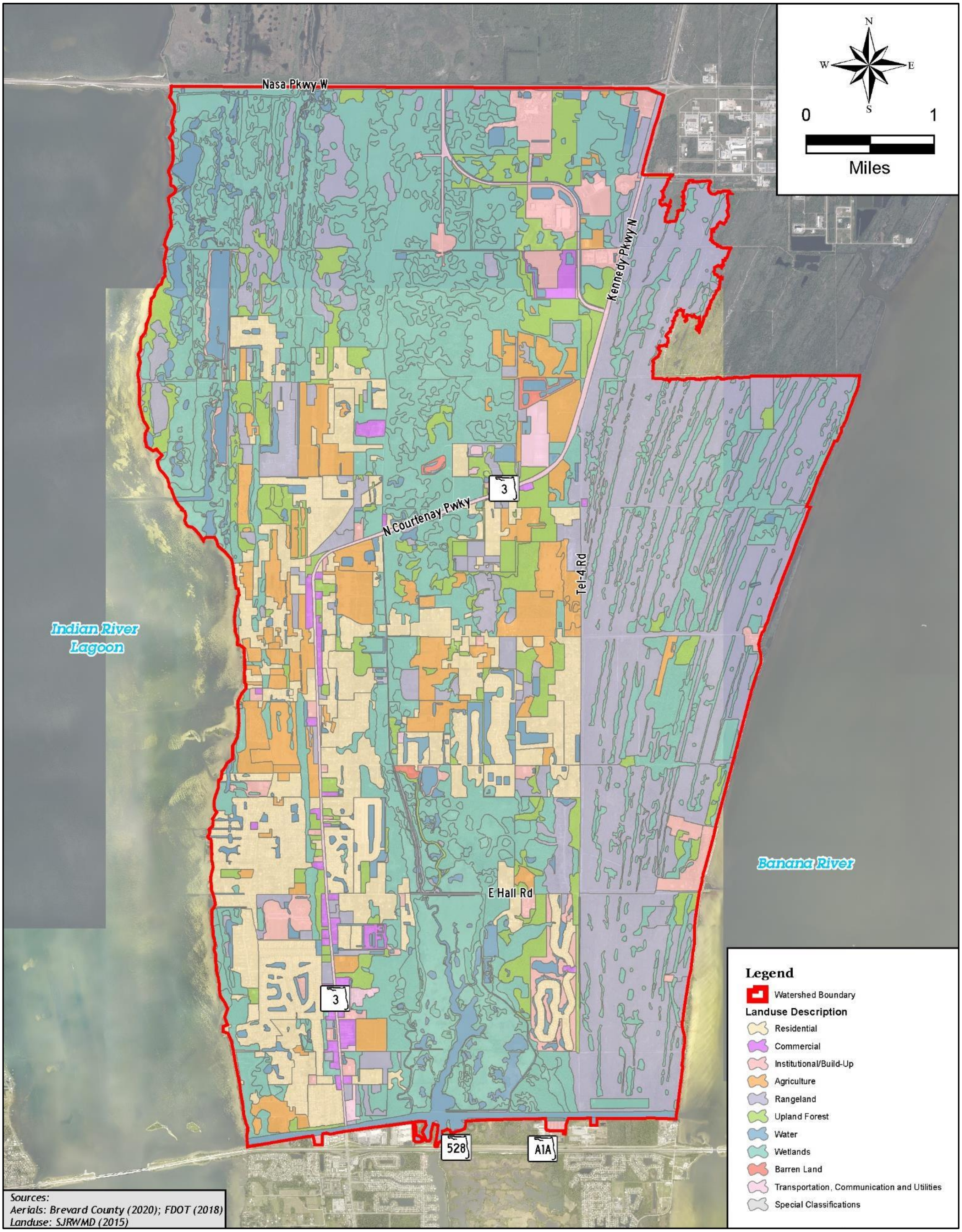


Figure 2.2: Landuse Map

Table 2.1: Land Use Composition Summary

Landuse	Total Area (acres)	% of Watershed
Residential	2,869	13.2%
Commercial and Services	216	1.0%
Institutional and Build-up	688	3.2%
Open Land	82	0.4%
Agriculture	1,391	6.4%
Rangeland	5,075	23.3%
Upland Forests	1,510	6.9%
Water	1,319	6.1%
Wetlands	8,272	38.0%
Disturbed Land	38	0.2%
Transportation, Communications, Utilities	331	1.5%

2.2.3 Rainfall Data: Both rain gage and Next Generation Weather Radar (NEXRAD) radar rainfall data were obtained to evaluate predicted rainfall within the watershed. Rain gage data was obtained in 15-minute intervals from the SJRWMD for rain gages located at Ransom Road and Kiwanis Park. NOAA rain gages located at Cape Canaveral, Rockledge, and Merritt Island were obtained in daily intervals, and rain gage data from the KSC Spaceport Weather Archive was obtained for the North Merritt Island Field Mill in hourly intervals. NEXRAD radar rainfall estimates were obtained from SJRWMD in 1-hour intervals (3.7 km^2 pixel grid), and from NOAA for the KMLB Melbourne station in 5-minute intervals (0.15 km^2 grid).

This information will be used during model calibration and forecasting efforts. Details on the processing of the above data for use in this study will be discussed in the Model Calibration Summary Memorandum.

2.2.4 Stage Gage Data: Historical stage data was collected to establish boundary conditions for the watershed model and for future calibration/validation efforts. After significant evaluation of available water level datasets and sensors, it was decided that the most scientifically valid approach was to provide two timeseries of stage level conditions bounding the study area on the east (Banana River Lagoon) and the west (Indian River Lagoon). In order to provide this information, data was collected from the following sources:

- Brevard County Stormwater Program: Staff gage data within the study area, including daily surface water stage levels.
- United States Geological Survey (USGS) National Water Information System: Haulover Canal stage levels (15-minute intervals)
- NOAA: Trident Pier Data continuous Atlantic Ocean elevation dataset
- SJRWMD: Indian River Lagoon continuous sensor stage data for Titusville, Cocoa Beach, Banana River, Indian Harbor Beach, and Melbourne

Detailed information on the processing of this data will be included in **Section 3.5** of this report.

2.3 Reference Documents

SAI obtained reference documents associated with 203 Environmental Resource Permits (ERPs) including for example, record drawings, construction plans, reports, and survey data. The ERP data was downloaded from the SJRWMD WMIS website. Each of these data sets were cataloged and saved in the reference documents folder (Support_Data\1_Watershed_Evaluation\Reference_Documents*). The reference documents were named using the following naming convention:

NMI_XXX_YY_Z

- "XXX" represents a sequential reference number assigned to the data
- "YY" represents the document type code
  - RD = record drawings/as-builts
  - CP = construction plans
  - RPT = report
  - SD = survey drawing or survey data
  - GIS = GIS files
  - MD = model data
  - PHO = aerial orthophotos
  - MI = miscellaneous information
  - MPI = miscellaneous permit information
- "Z" represents the sequence number (01, 02, etc.) for ERPs with more than one of the same document type code.

Many ERPs have several different document types. When this happens, the one with the most reliable or beneficial data is referenced; in most cases this will be a record drawing, unless there are no record drawings/as-builts available. Some ERPs may contain both record drawings and construction plans, or multiple sets of construction plans.

Each reference document is represented spatially in the reference documents geodatabase, included in the electronic deliverables accompanying this report. A polygon was drawn to show the approximate extents of each reference document. Reference document polygons are shown in **Figure 2.3**.

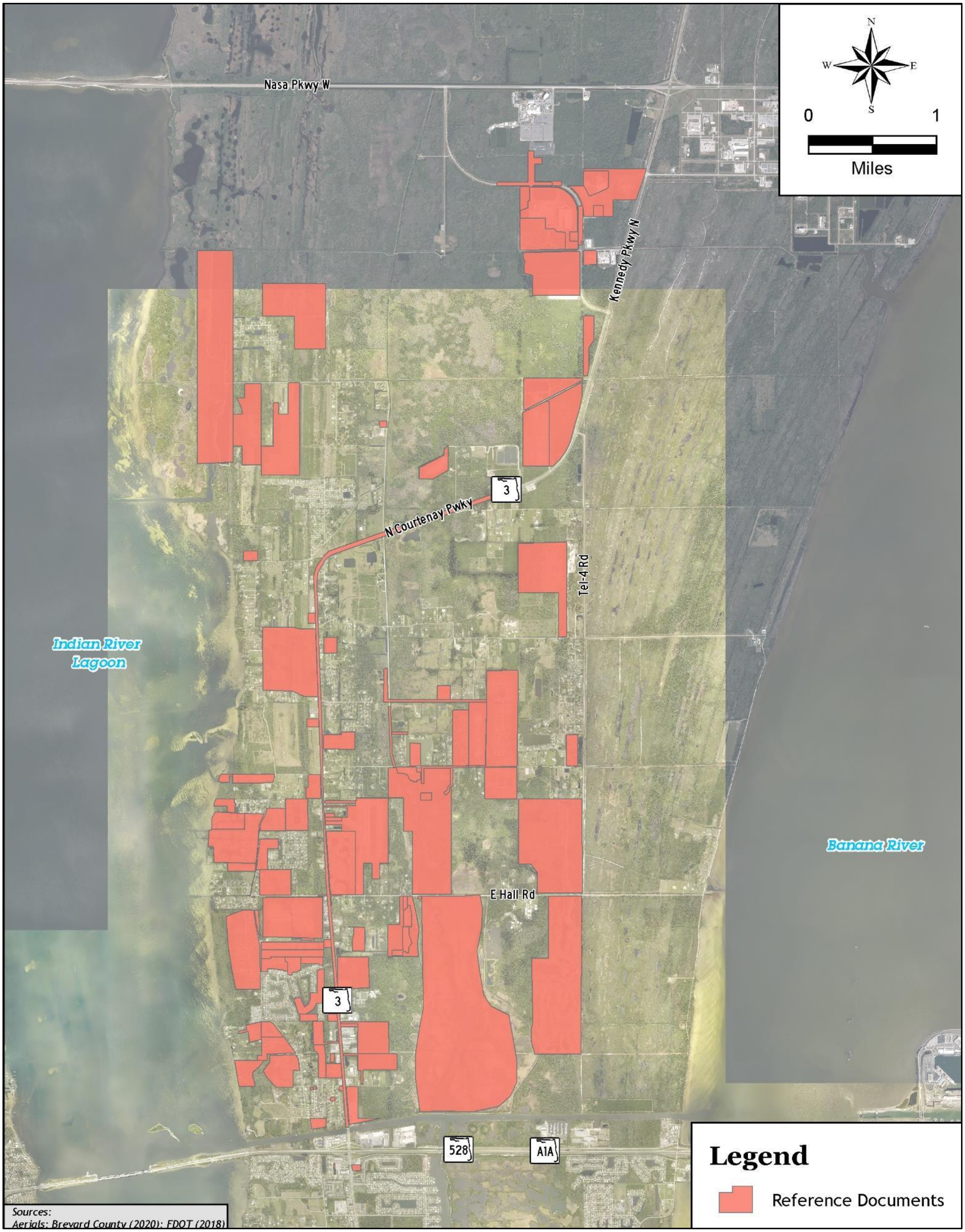


Figure 2.3: Spatial Location of Reference Documents

2.4 Groundwater Data Collection

Groundwater-Surface Water (GW-SW) interaction has been observed in the NMI watershed. As such, the model developed for this study will be an integrated GW-SW model. Data collection for the groundwater characteristics within the watershed included obtaining regional groundwater model data and gage data.

2.4.1 SJRWMD Gage Data: Gage data was obtained from the SJRWMD Upper Floridan Aquifer Well located at Kings Park in Merritt Island (ID: BR2115). Data was collected for the surficial aquifer (POR 3/12/2009-8/11/2020) and for the upper Floridan (POR 12/4/1985-8/11/2020). See **Figure 2.4**.

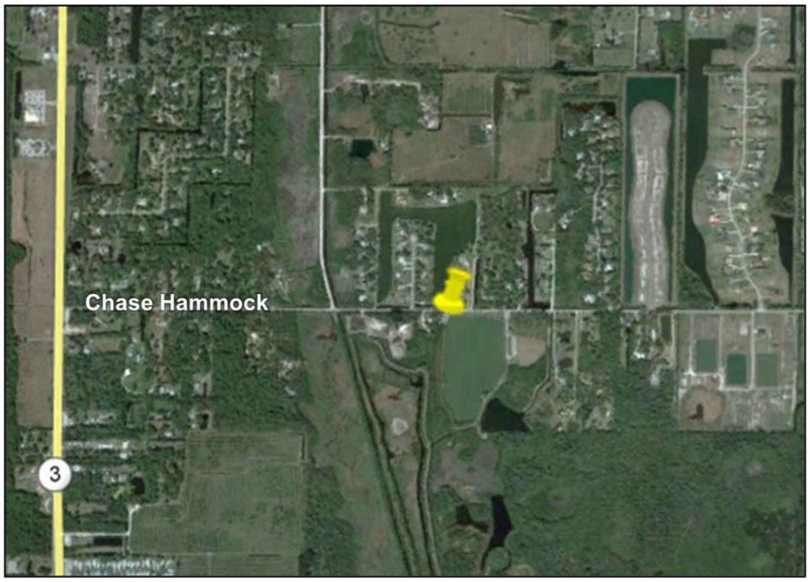


Figure 2.4: SJRWMD Well Location

2.4.2 East-Central Florida Transient Expanded Model: The East-Central Florida Transient Expanded (ECTFX) MODFLOW Model was prepared in February 2020 as part of the Central Florida Water Initiative (CFWI). The CFWI was a collaborative effort between the SJRWMD, SFWMD, SWFWMD, FDEP, and other public agencies and stakeholders. Although the CFWI includes only Central Florida counties, the model boundaries extend east into and beyond Brevard County, including the North Merritt Island watershed. **Figure 2.5** on the following page is an excerpt from the ECTFX February 2020 Report depicting the model extents.

2.5 Field Survey Data

The County requested that the SAI Team obtain survey data for several channel systems throughout the watershed that had recently been dredged and/or cleared of vegetation. This included a total of eight (8) channel cross sections throughout the watershed, including ditches along Hall Road, Judson Road, Pine Island Road, and Chase Hammock Road. This work was conducted in August 2020 by Morgan & Eklund, Inc. (M&E). Cross sections provided included both the ground surface and bottom of muck elevations at each cross section. **Figure 2.6** on the following page shows the location of the 8 cross sections. The survey data is included in the electronic deliverable accompanying this report.

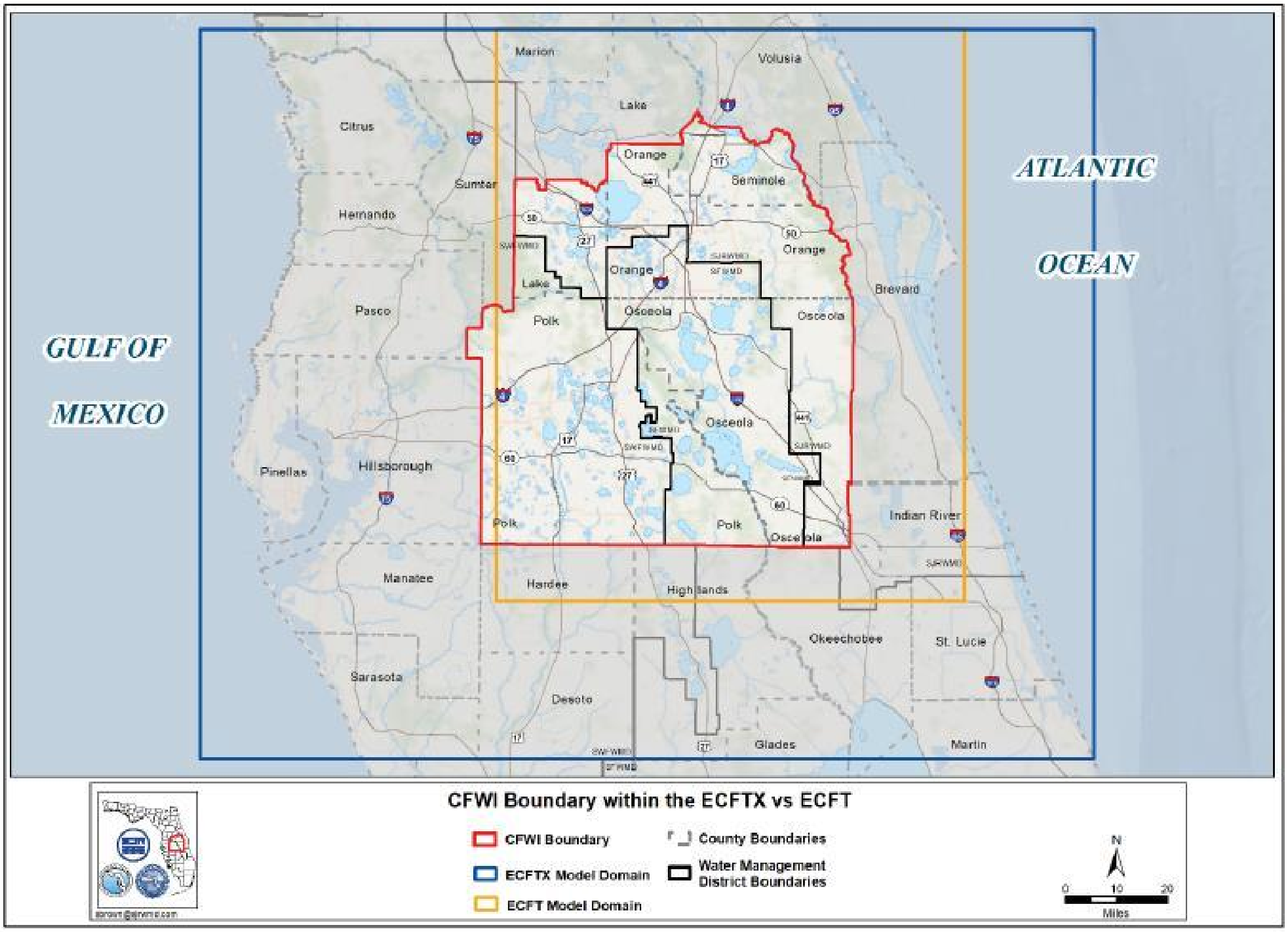


Figure 2.5: ECTFX Model Domain

Figure 2.6: Cross Section Survey Locations

2.6 Digital Terrain Model

The County provided LiDAR-derived terrain data for the NMI watershed dated 2007 (flight dates: 09/15/2007 - 09/30/2007). This Digital Elevation Model (DEM), titled *1_ftNMI.tif*, along with associated LiDAR data files and supporting features (i.e., breaklines, hydrographic features) will serve as the base terrain data for this study.

The NMI Study model will address SW-GW interactions within the region. The model will rely heavily on representative terrain data. As such, the DEM as provided will require manual modifications for several reasons:

- **Missing Data:** The original DEM surface provided by the County did not include topographic information for the Indian River Lagoon, Banana River Lagoon, Barge Canal, and several channel features directly connected to these waterbodies.
- **Hydro-Corrections:** The existing DEM provided by the County did not include bathymetric data within waterbodies (ponds and wetlands) as well as channel features.
- **Areas of New Development:** The existing DEM is based on data collected in 2007. Several areas of new development have occurred since then, resulting in new fill and stormwater facilities.

The approach to conducting the referenced DEM modifications is discussed in **Section 3** of this Study.

3.0 Watershed Inventory and Surface Water Model Development

3.1 Digital Terrain Model Development

The County provided LiDAR terrain data for the NMI watershed dated 2007 (flight dates: 09/15/2007 - 09/30/2007). This Digital Elevation Model (DEM), titled 1_ftNMI.tif, along with associated LiDAR data files and supporting features (i.e., breaklines, hydrographic features) served as the base for all DEM modifications conducted as part of the study.

3.1.1 Horizontal and Vertical Datum: Topographic data was provided in the North American Vertical Datum of 1988 (NAVD88) for vertical information (feet) and in the North American Datum (NAD) of 1983 for horizontal (feet). A conversion factor of -1.30 feet was used to convert data in NGVD29 to NAVD88. All topographic data were projected in State Plane Florida East FIPS 0901 (Feet – HARN).

3.1.2 Existing Topographic Information: The DEM provided by the County was based on LiDAR flown in September 2007 and has a grid resolution of 5-feet. The surface extends from NASA Parkway to the north to SR 528 to the south but does not include the water surface in the Indian River Lagoon, Banana River, and connected waters.

3.1.3 Hydro-Corrections: The 2007 DEM elevations within wetlands and waterbodies reflect the water surface at the time the region was flown. Hydro-corrections were made to estimate bathymetry data in these areas of the watershed to better represent key features and to promote better interaction between the surface water and groundwater model elements. Hydro-correction efforts are summarized below. For additional detail on these efforts, please refer to the *North Merritt Island HydroDEM Update Memo (Atkins, September 2020)* provided in **Appendix A**.

Boundary Waterbodies: The original DEM surface provided by the County did not include any topographic information for the Indian River Lagoon, Banana River Lagoon, Canaveral Barge Canal, and several channel features directly connected to these waterbodies. **Figure 3.1** depicts the areas missing topographic information, shown in green.

Figure 3.1: Missing Topographic Information

Data extracted from NOAA navigational charts and datum information for the Trident Pier were used to estimate topographic information for the IRL and Banana River. NOAA navigational charts provided depths, which were referenced to the Mean-Lower-Low Water (MLLW) elevation. Using the Trident Pier to establish a reference elevation for the MLLW, a datum conversion factor of -2.83 feet was used to convert the navigational depths to an elevation in NAVD88.

Channel Hydro-Corrections: Channel areas were extended below the water surface to allow groundwater interaction. Interior DEM channel updates were based on channel invert elevations and geometry, as determined from cross section survey data provided by Morgan & Eklund, the existing ICPR3 model, and the SWAMP database. An example of channel hydro-corrections is shown below in **Figure 3.2**.

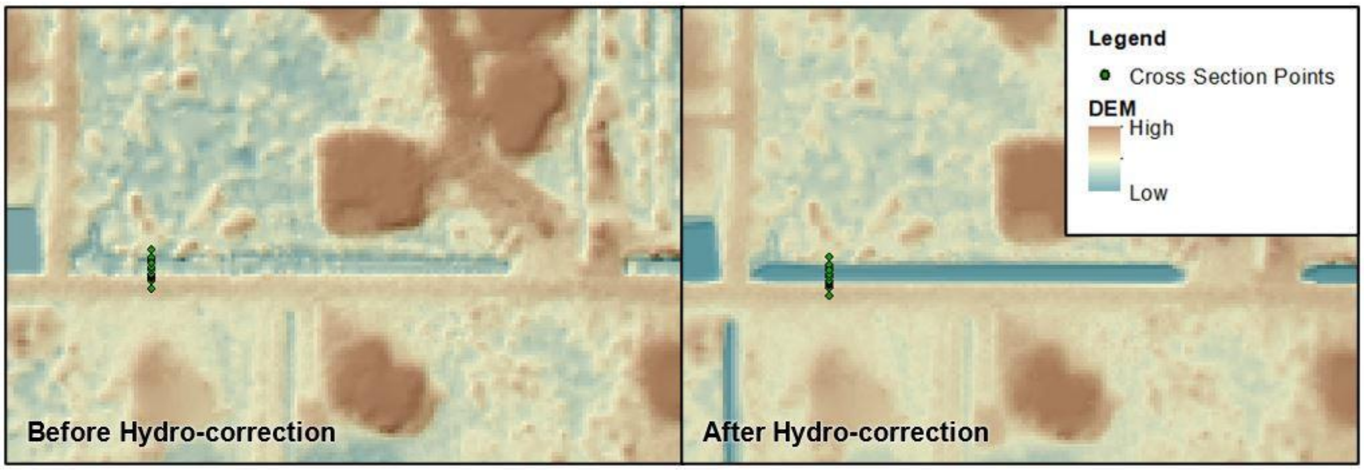


Figure 3.2: Channel Hydro-Correction Based on Surveyed Cross Section

Natural Ponding Hydro-Corrections: In areas identified in the DEM as natural ponding areas, updates were based on the extent of open water shown in the DEM and aerial imagery to estimate the extent and depth of ponded water. For the majority of these areas, a depth of 3-feet was assumed, which is consistent with nearby canals and provides sufficient connection for the groundwater model to interact with the surface water elements. The exception to this assumption is the interconnected lakes west of the Pine Island Stormwater Facility, where a depth of 5-feet was used based on connected channel inverts. The DEM was then tapered down from a 0-ft depth at the water's edge to the approximated depth at the center. An example of these hydro-corrections is shown below in **Figure 3.3**.

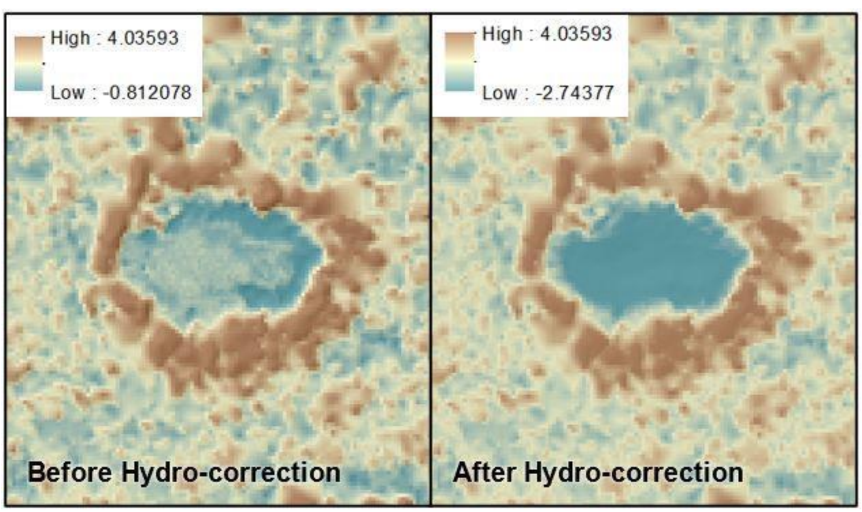


Figure 3.3: Natural Ponding Hydro-Correction

Developed Area Hydro-Corrections: As with wetlands, the DEM surface in stormwater ponds is reflective of the water surface at the time of the LiDAR flight. To encourage groundwater interaction, ponds in developed areas were extracted down from the water surface using a uniform assumption based on County and WMD development criteria. It was assumed that the water surface shown in the DEM was close to the established normal water level (NWL) for each pond. Using the standard criteria of a 5:1 (horizontal:vertical) pond slope and the water surface polygon associated with the 2007 DEM, the SAI Team extracted down the ponds in developed areas uniformly using the criteria below:

- 5:1 (h:v) slope from the water surface to 2-ft below the water surface
- 2:1 (h:v) slope from 2-ft below the water surface down for another 6-ft, resulting in a total pond depth of 8-feet below the water surface.

This approach allowed for a streamlined process of estimating bathymetry data for stormwater ponds in developed areas and is consistent with development criteria for the region. Exceptions for key locations were made, such as the Pine Island Conservation Area ponds, which were extracted down from the water surface based on construction plan and as-built survey data. **Figure 3.4** shows an example of extracted stormwater ponds in a developed area of the watershed.

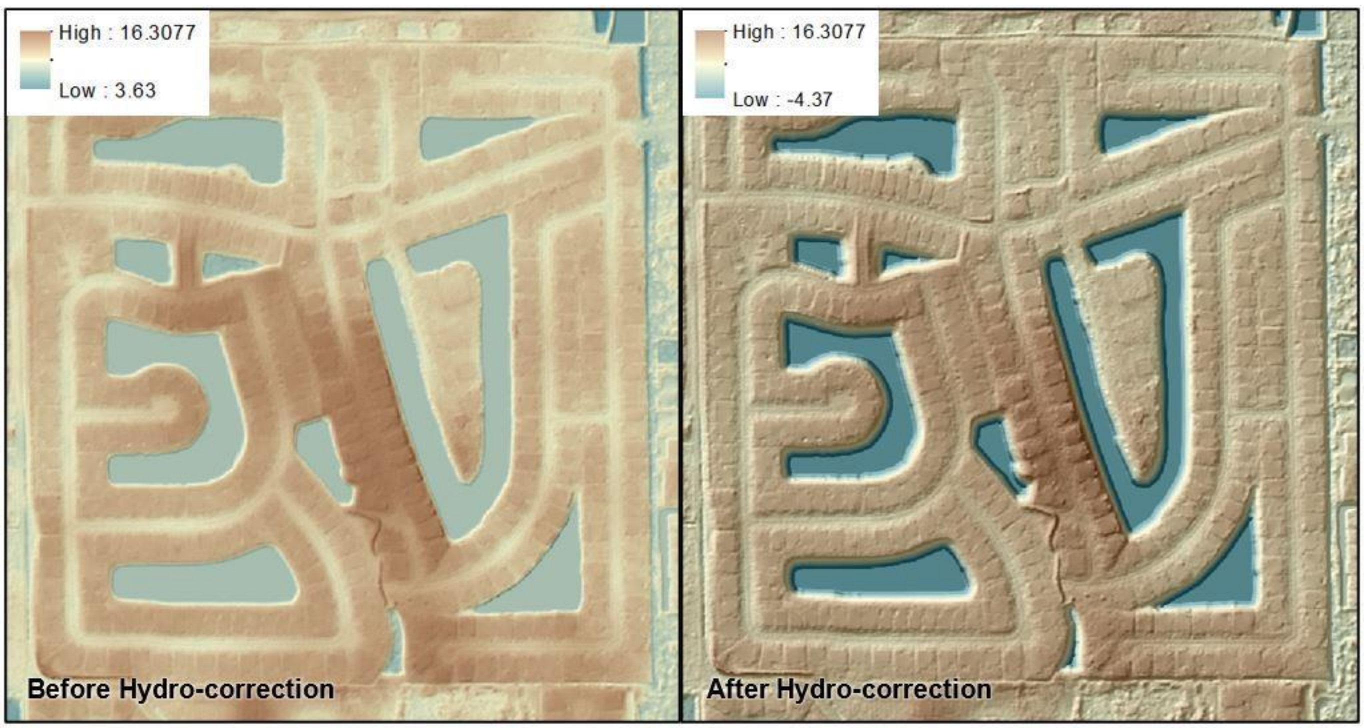


Figure 3.4: Hydro-Corrections in Existing Stormwater Ponds

3.1.4 Topographic Voids / Areas of New Development: A 2D water surface model relies heavily on accurate terrain data. Given the age of the DEM available for the watershed, identifying areas of new development and topographic voids was critical. For the purposes of this evaluation, topographic voids are defined as those areas where available digital topographic information (2007 DEM) does not accurately describe the terrain as it exists today. Topographic voids result from such things as land alterations, new development, and missing data. Identified topographic voids and areas of new development within the watershed are depicted in **Figure 3.5**. An example of a topographic void is shown in **Figure 3.6.**

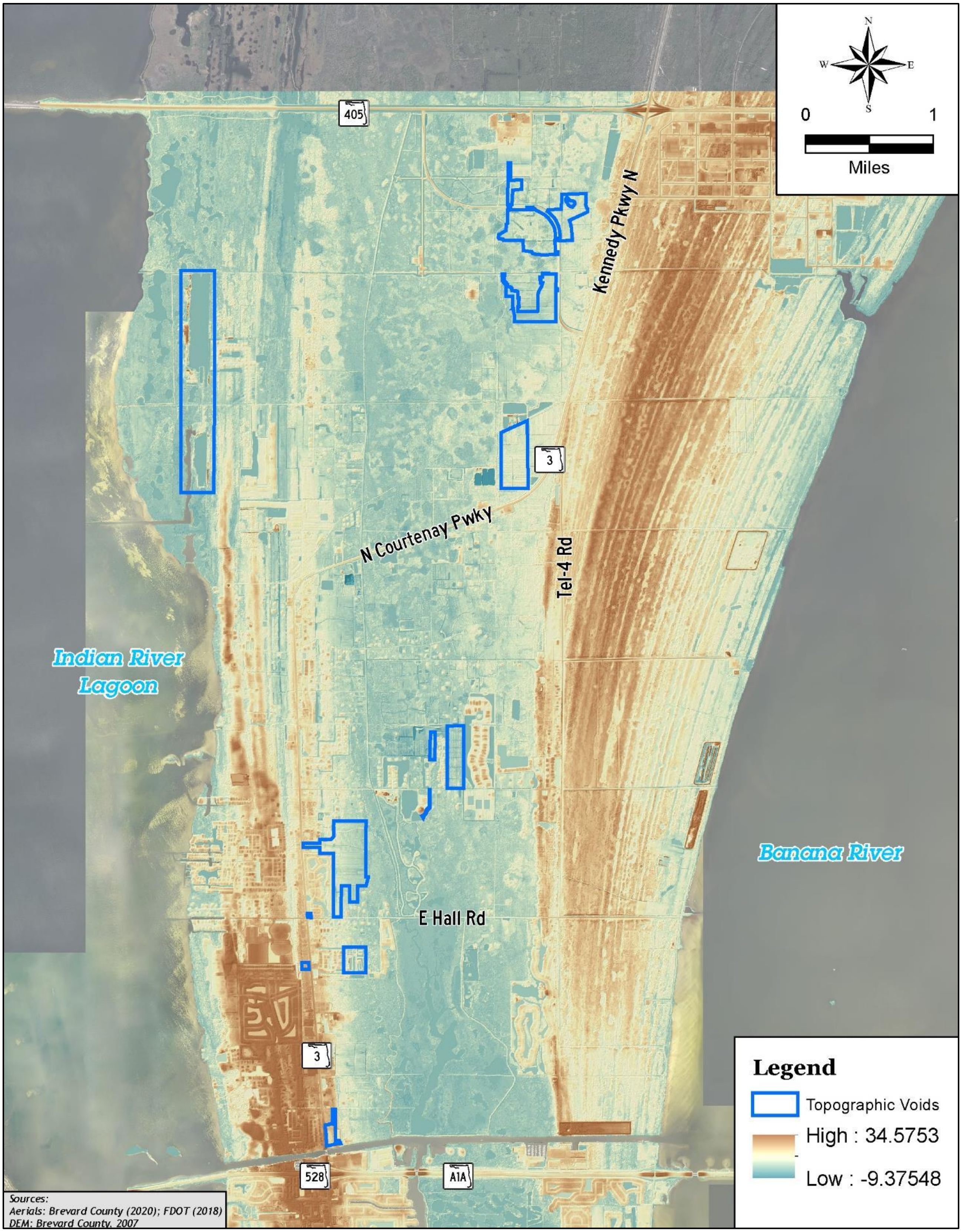


Figure 3.5: Topographic Voids on Original 2007 Digital Terrain

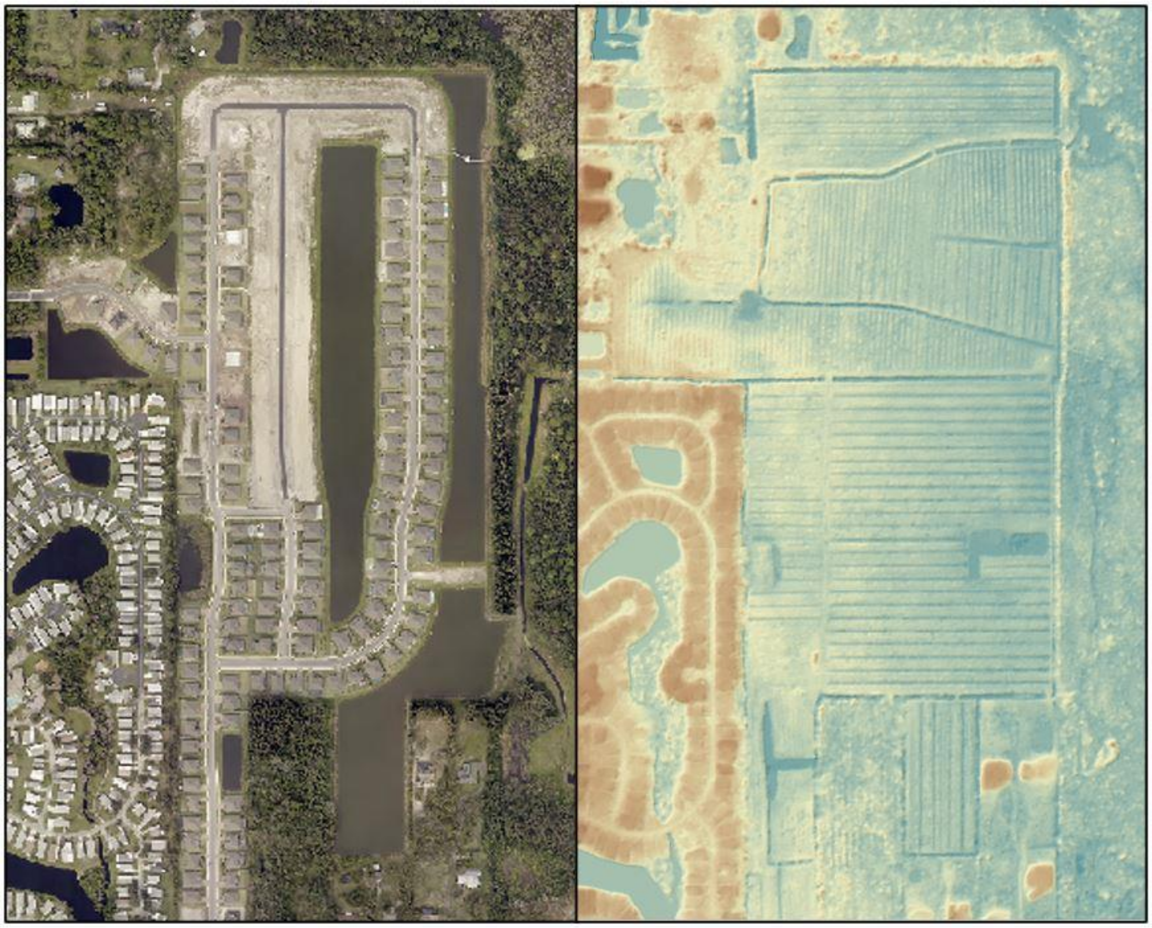


Figure 3.6: Example of Topographic Void

Evaluation of ground cover changes was conducted by comparing the DEM to aerial imagery in a systematic fashion by visually panning through the watershed while displaying both the DEM and aerials for a side-by-side comparison. Using 2020 aerial imagery to match the date-certain established in the scope of work, SAI identified 13 total topographic voids. Of these, eight were considered areas of new development, four were considered areas of land excavation, such as stormwater pond expansion or construction, and one was caused by tree cover obscuring an existing pond. The identified topographic voids within the watershed are provided in the *Topographic_Voids.gdb* geodatabase.

Topographic voids were manually corrected for this study. For each topographic void, construction plans or record drawings (if available) were georeferenced and storage areas were delineated. Delineated contours were used as breaklines to “burn-in” the storage ponds into the existing terrain data. To account for fill placed in new development areas, a minimum fill elevation – typically the top of bank elevation for stormwater ponds – was established and applied to the rest of the development. Examples of corrected topographic voids are shown in **Figures 3.7 and 3.8.**

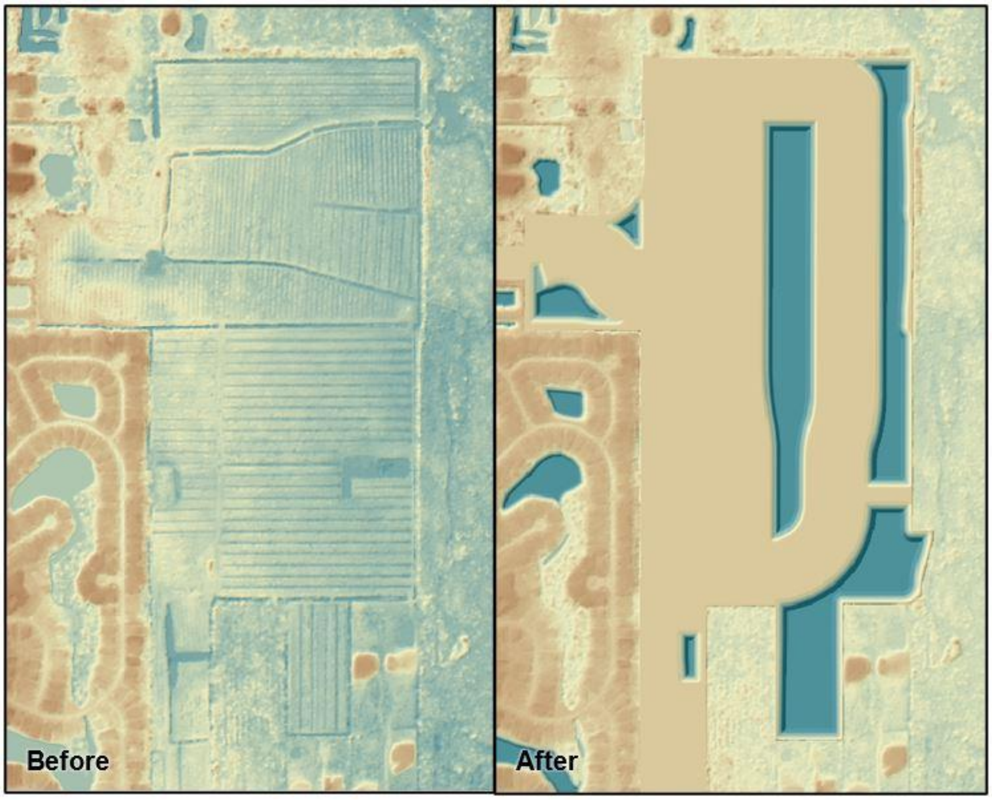


Figure 3.7: Corrected Topographic Void of New Development

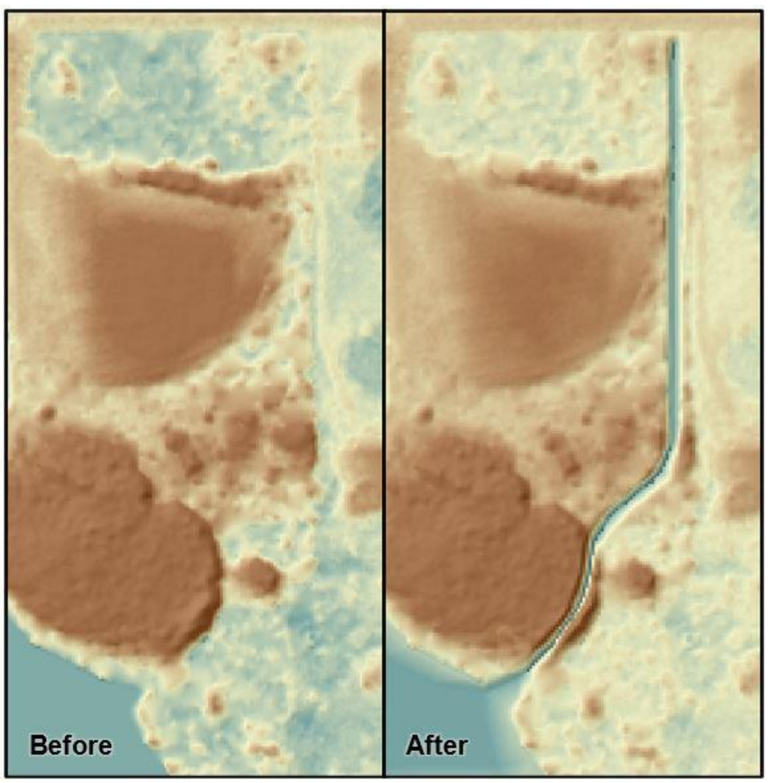


Figure 3.8: Corrected Topographic Void of New Channel Excavation

3.1.5 QA/QC Process Description: Several QC checks were performed on the revised DEM at various points throughout the revision process. These checks included systematic visual inspections of the watershed and geospatial tools used to compare two DEM data sets. At each DEM modification, the "Minus" tool was utilized in GIS to compare the previous DEM grid with the updated grid to quickly identify differences between the two and help focus visual inspections. Comments related to the DEM modifications were stored in an internal Comments GDB to ensure they were adequately addressed.

3.2 Hydrologic Features

Hydrologic Features of North Merritt Island include features that contribute to the conversion of rainfall into runoff and subsequent infiltration into the groundwater domain. In the North Merritt Island ICPR4 model, rainfall enters the model domain globally as a design storm simulation or on grid for calibration events. Runoff in Mapped Basin areas utilizes a unit hydrograph and time of concentration to load stormwater to modeled nodes. In areas outside of Mapped Basins, including 2D overland flow regions and pond or channel control volumes, runoff is loaded directly onto those features. The following subsections cover the model delineation process, soil / land use characterization, and hydrologic parametrization and simulations.

3.2.1 Subbasin Delineation Process: The effective North Merritt Island model domain included 473 basins. These basins were delineated based upon an evaluation of the terrain, simulating ridge line, and separating out areas of storage. As part of this model update process, the model domain was expanded to the edge of the Indian River Lagoon and Banana River, integrating 1D and 2D areas, and new ERP data. In total, over 750 unique areas were delineated in the updated model and the model domain was expanded from 20.5-sq. miles to 34-sq. miles. The Expanded model domain is seen in **Figure 3.9** along with the Effective Model domain and the ERP update Areas.

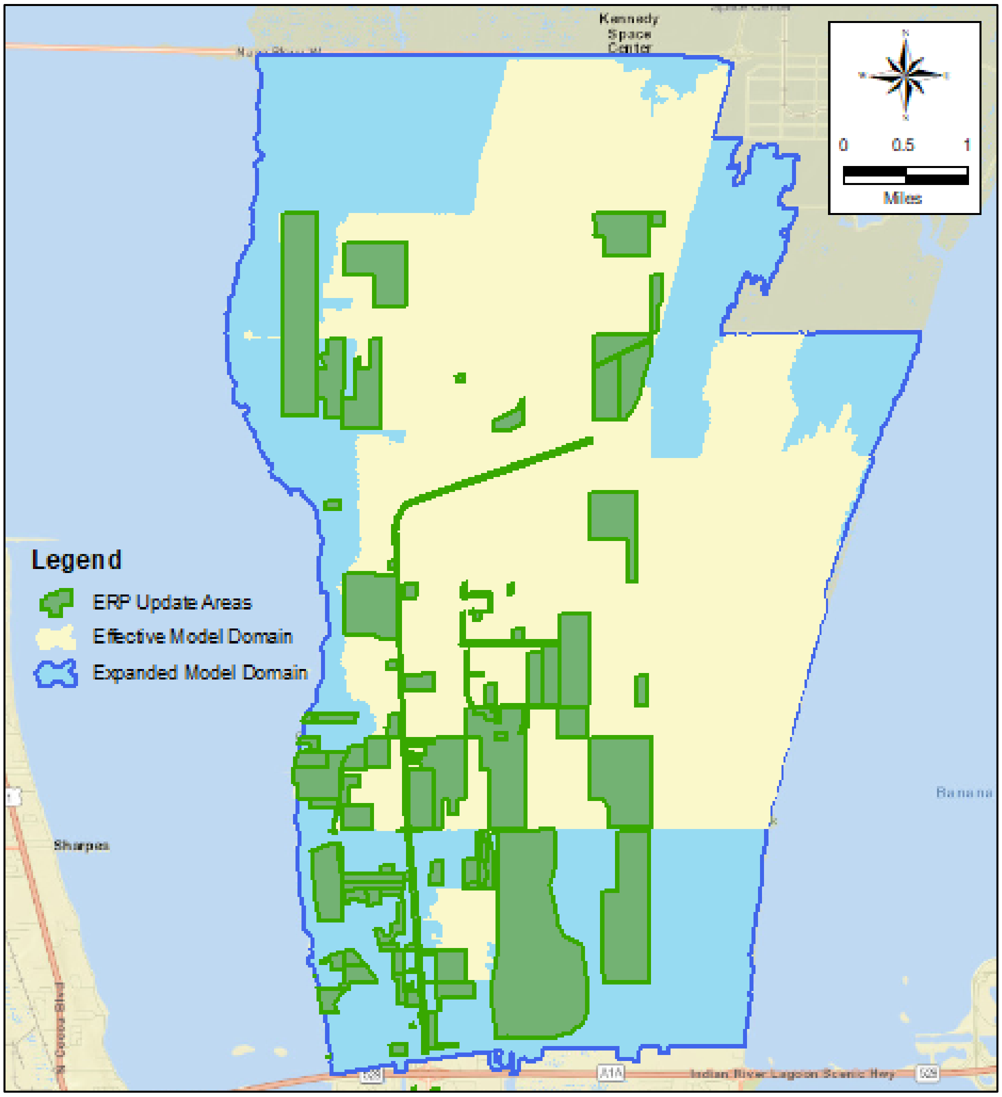


Figure 3.9: Effective vs. Expanded Model Domain and ERP Update Areas

3.2.2 Land Use Characterization: Land Use data characterization is presented in **Section 2.2** and includes land use updates based upon visual observation from 2020 aerial imagery consistent with ERP model update areas. For the ICPR4 model development, the pervious and impervious areas were separated. Pervious areas infiltrate into the soil column as soil storage allows, while impervious areas directly runoff after an initial abstraction is filled. The ICPR input table by land use code and description along with the percentage impervious, and initial abstraction used over each land use is included in the Impervious Data Set in the ICPR4 model and summarized in **Table B.1 (Appendix B)**. It should be noted that the water and wetland land use types area characterized as 0% impervious. This is because ICPR4 will only apply evaporation to "pervious" surfaces. Further, the rainfall excess for water and wetland land use types will remain high due to the underlying soil feature, which typically corresponds to high groundwater table.

3.2.3 Soil Characterization: To take advantage of stormwater infiltration into the ground and the potential re-emergence of water into wetlands, watershed soils are characterized by their infiltration capacity/soil storage, and hydraulic conductivity. The soils presented in **Section 2.2** in their general categories are presented in **Table B.2 (Appendix B)** as categorized by MUKEY. This characterization is used to uniquely parameterize the soils applying the Green-Ampt Rainfall Excess method.

3.2.4. Hydrologic Parameterization:

Time of Concentration: Times of concentration (Tc) were calculated for each 1D basin in accordance with TR-55 methodology (NRCS, 1986). Sheet flow was limited to 150-feet and a minimum Tc value of 10-minutes was implemented.

Rainfall Excess Method: The NRCS unit hydrograph method was used to convert precipitation excess into a runoff hydrograph within 1D basins. A synthetic unit hydrograph with a shape factor of 256 for all modeled basins was used, consistent with the effective model and considered appropriate for areas with mild slopes and relatively flat terrain, such as those in this watershed.

The original study utilized the NRCS curve number method to determine rainfall excess. Although the curve number method can be used in ICPR4 for surface modeling, it cannot be used for integrated surface water – groundwater modeling or continuous simulations because it does not track soil moisture and evapotranspiration. The Green-Ampt methodology will be used for hydrology initially. During the calibration period, the Vertical Layers approach may also be used.

To determine infiltration and rainfall excess, the soil data was used to first develop the initial/un-calibrated Green-Ampt and Vertical Layer soil parameters for the vadose zone. The Green-Ampt data were processed using the NRCS Soil Data Viewer ArcGIS plugin. All data were based on weighted averages using the dominant component. The Vertical Layers data were processed using the most recent NRCS SSURGO data for Brevard County published on June 8, 2020. The dominant soil component was also used for the Vertical Layers data development. Data used from the NRCS soils data for the model development are included in **Table 3.1** below.

Table 3.1: NRCS Soil Data Parameters

Green-Ampt	Vertical Layers
Percent Clay	Percent Clay
Percent Sand	Percent Sand
Percent Organic Matter	Percent Organic Matter
Bulk Density (1/3 Bar)	Bulk Density (Oven Dried)
Saturated Hydraulic Conductivity	Saturated Hydraulic Conductivity
Moisture Content (1/3 Bar & 15 Bars)	Moisture Content (1/3 Bar & 15 Bars)
Depth to Water Table	Depth to Water Table

The data in **Table 3.1** were used to develop the following Green-Ampt and Vertical Layers soil parameters.

- *Saturated Vertical Hydraulic Conductivity (ft/day):* Based on NRCS Soil Data
- *Saturated Moisture Content (L^3/L^3):* Eq. 2.15 ICPR4 Technical Reference (June 2018)
- Residual Moisture Content (L^3/L^3): Eq. 2.17 ICPR4 Technical Reference (June 2018)
- *Initial Moisture Content (L^3/L^3):* Set initially equal to field capacity
- *Field Capacity (L^3/L^3):* Moisture Content at 1/3 Bar
- *Wilting Point (L^3/L^3):* Moisture Content at 15 Bar
- *Pore Size Index:* Eq. 2.18 ICPR4 Technical Reference (June 2018)
- Bubble Pressure (in): Eq. 2.19 ICPR4 Technical Reference (June 2018)
- *Depth to Water Table (ft):* Based on NRCS soil data. This data is not used for the simulation when groundwater is being modeled.

The raw and processed soil data is provided in reference document *NMI_220*. For more details on the soil data development, refer to the *ICPR 4 User's Manual* under Base Data → Lookup Tables → Rainfall Excess Sets.

3.3 Hydraulic Features

Hydraulic routing in the NMI Watershed is performed using ICPR4 integrated surface water and ground water model including 1D and 2D model features. This section highlights the 1D hydraulic elements, model development process and hydraulic parameterization. The 1D model portion includes nodes simulating storage areas and links which represent the conveyance system between storage elements. Refer to **Exhibit 1** for a depiction of the 1D elements of the NMI watershed model network.

3.3.1 Preliminary Model Network Development Process: Using the SWAMP database to identify significant structures connecting elements, the effective ICPR3 model network was modified to incorporate structural elements that were not previously included. Then, based upon ERP data and as-built plans, the model network was updated and expanded to incorporate key structural elements and storage areas within each development. **Table 3.2** summarized the structural elements included in this model, including channels, culverts, drop structures, pumps, and structural weirs. It was noted that in the effective model, many of the depressional areas were modeled using only natural overflow weirs. Where appropriate, these areas were updated to include a structural overflow such as a drop structure or structural weir based on data in SWAMP or ERP documents. It is also of note that the reduction in natural weirs from the effective model to the updated model is due to inclusion of the 2D region which replaced many of these elements.

Table 3.2: Summary of Structural Model Elements by Type

Link Type	Effective ICPR3 Model	ICPR4 Model Update
Pipe	160	363
Drop Structure	11	86
Structural Weir	4	21
Natural Weir	809	752
Channel	165	243
Pump Station	3	7

3.3.2 Hydraulic Parameterization: Hydraulic parameterization utilized available information from ERPs and digital topography to supplement the County's SWAMP features database and field reconnaissance. Highlights of hydraulic parameterization are presented below.

Storage Representation: Lakes, ponds and wetland areas were represented by stage/area relationships assigned to model nodes. These stage/area relationships were developed utilizing the updated digital terrain data discussed in **Section 3.1**. Storage was calculated using GIS at 0.25-foot vertical increments. Channel storage was excluded from basin storage calculations based on the approximate channel cross-section extents and channel alignment.

Node Initial Water Conditions: Groundwater node, overland flow node, and 1D node initial stages are based on a "hot start" simulation for 2017. The hot start simulation starts on 01/01/2017 0.00 hours and ends on 12/31/2017 0.00 hrs. The hot start simulation results on 12/31/2017 0.00 hours were extracted from the results and used to specify the initial stages. For 1D nodes, the initial stages were extracted from the tabular node time series results. Initial stages for the groundwater and overland flow nodes were established using exported surfaces from the hot start simulation results/animations on 12/31/2017 0.00 hours.

Channel Cross Sections: Channels in the effective model were mainly simulated as trapezoidal sections with limited areas represented by irregular cross-sections, mainly in the undeveloped area near the Banana River. All effective model cross-sections were incorporated into the terrain as bathymetry points along with the channel survey data described in **Section 2.5**. The updated model then used the updated terrain to cut irregular cross-sections for every channel reach.

Manning's roughness coefficients were then applied to each section. The initial assumption based upon field visits and aerial imagery was that channels can be simulated with a Manning's n value of 0.045, corresponding to a lightly vegetated channel section. During the calibration process this assumption will be revisited with roughness adjusted as appropriate using the range of values in **Table 3.3** below.

Table 3.3: Manning's *n* Lookup Table for Channels

Channel Description	Manning's Value
Very Clean	0.025 - 0.03
Light Vegetation	0.03-0.07
Medium/Heavy Vegetation	0.06-0.15

Drainage Structure Parameterization: Structural conveyance elements including pipes, drop structures and structural weirs are modeled in ICPR4 based upon element size, invert, and roughness factors. Data for the structural elements originated from either the effective model, Brevard County's SWAMP database, ERP as-built data, portions of the NASA model developed by JEA, survey, or field observations. The highest priority of data was assumed to be Brevard County's SWAMP database, followed by field survey, ERP as-builts, NASA model, then field observations.

The Manning's roughness values applied to these elements are based upon the pipe's material using the values referenced in **Table 3.4**. It should be noted that the bridges in the original model were converted in the effective ICPR3 model to pipe elements of appropriate opening and assigned a Manning's value of 0.056, to reflect that these elements have a vegetated or bare earth bottom rather than a traditional concrete pipe element. These bridge structures were left as culvert model elements in the ICPR4 updated model.

Entrance and exit loss coefficients were set to a default of 0.5 and 1.0 respectively unless site conditions warranted alternate values such as multiple pipes in series or smooth entrance conditions would serve to limit the losses.

Weir structures, either as part of a pond control structure or structural overflow from a pond, were also included. The weir coefficients used for structural weirs are presented in **Table 3.5** and vary depending upon whether the element was sharp crested or broad crested.

Table 3.4: Manning's *n* Lookup Table for Pipes

Pipe Material	Manning's Value
PVC	0.0
RCP	0.012
CMP	0.024
Bridge Approx.	0.056

Table 3.5: Structural Weir Coefficients

Weir Type	Crest Type	Weir Coefficient
Structural Weir (Drop Structure)	Sharp Crested	3.2
Structural Weir	Broad Crested	3.0

Overland Flow Weirs: To connect model nodes that are not otherwise connected via structural elements, or in conditions when the capacity of the structural element is exceeded, an overland weir is used to simulate the conveyance and represent natural overland flow or road overtopping conditions between sub-basins. These irregular weir features are characterized by cross sectional data and a weir coefficient. The cross-sectional data used to characterize the overland weir connection was extracted from the terrain using the ICPR4 internal cross section processor. Weir coefficients were determined from literature based on flow type and ground cover as shown in **Table 3.6**; whereby the weir coefficient and resultant flow over a natural section is lower than its roadway counterpart, due to the increased roughness along the flow path.

Table 3.6: Weir Coefficients

Weir Type	Ground Cover	Weir Coefficient
Natural Overland	Grass or Light Woods	2.0
Roadway Overtopping	Gravel or Paved Surface	2.6

3.3.3 QA/QC Process Description: Model elements were compared against source data of the element for accuracy. Where deviations or model inconsistencies were found, the internal reviewer would place a spatial comment point at the location of the element being commented upon. Adjustments to the model element and/or source documentation were then made and subsequently back-checked, and adjusted if required, by the internal reviewer. In addition, a peer-review approach was implemented that allowed other members of the SAI Team to review the spatial layout of the 1D model elements. An internal "Comments" geodatabase was employed to track comments from other Team members and their response/resolution to ensure all internal QC comments were addressed prior to finalizing the 1D model network.

3.4 Overland Flow Model Features

This section of the report details the development of 2D overland flow features used in the NMI ICPR4 model. **Exhibit 2** shows the extent of the 2D region and some of the 2D features used in the model.

3.4.1 Overland Flow Region Development: Overland flow model elements were used in undeveloped areas characterized largely by overland flow, as well as in areas where significant interaction with the groundwater was anticipated. The overland flow region boundary encompasses the area to be modeled as overland flow. This region boundary is somewhat coarse to simplify the mesh generation and avoid small triangle lengths in the mesh.

3.4.2 Breaklines & Interpolated Breaklines: The breakline feature class is comprised of polylines that are utilized in the overland flow mesh generation. Breaklines force the creation of flow paths (i.e., triangle edges) in the mesh along the breakline. Breakline placement generally defines the following types of topographic features:

- *Roadways* – Breaklines were placed at along the centerline of roadways and along the adjacent swales (See **Figure 3.10**). In some cases, control volumes were drawn along the centerline and breaklines were limited to the roadway swales.
- *Ditches/Channels* – Breaklines were placed along the centerline of minor swales and ditches that were not modeled with a 1D channel link to provide conveyance within the 2D overland portion of the watershed (See **Figure 3.11**).
- *Within Ponds and Wetlands* – Breaklines were drawn within stormwater ponds and large wetland areas to ensure the groundwater mesh was aligned with the surface water mesh for groundwater-surface water interaction within the waterbodies (See **Figure 3.12**).

- <u>Significant ridges or troughs</u> – The vertices and edges of the generated mesh should include ridges and troughs to ensure stormwater is not artificially trapped in depressions or allowed to flow unimpeded through high spots. These features add definition to the mesh to ensure appropriate flow paths and overflow elevations are included in the routing (See **Figure 3.13**).

Interpolated breaklines are a special case of breaklines that can be used to simplify obstructions in the DEM such as small culvert crossings within swales and ditches. The “interpolated” option directs the model to ignore DEM elevations between the endpoints of the breakline during mesh construction. With this option in effect, elevations at the mesh vertices created along the breakline are based on interpolated values calculated based on the elevations at the start and end points of the breakline rather than actual DEM elevations between the endpoints. See **Figure 3.14** for visualization of interpolated breaklines.

Figure 3.10: Roadway Breaklines

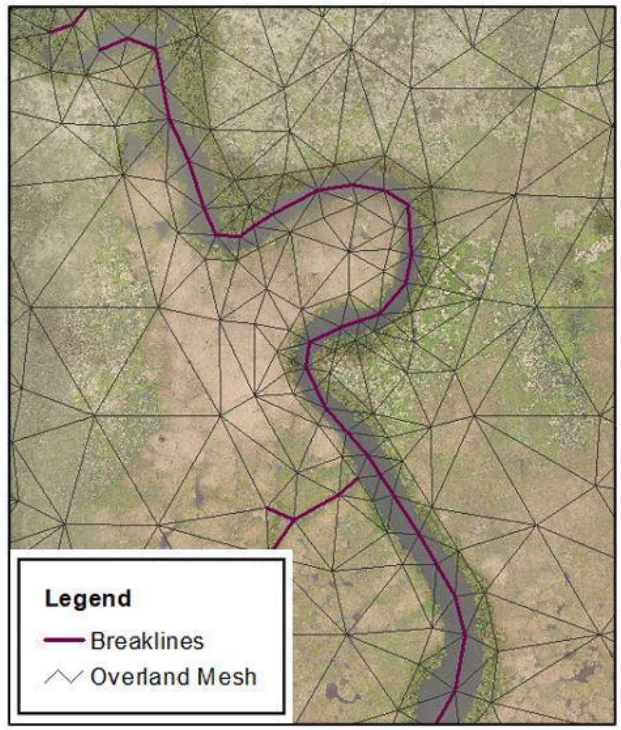


Figure 3.11: Channel Breaklines

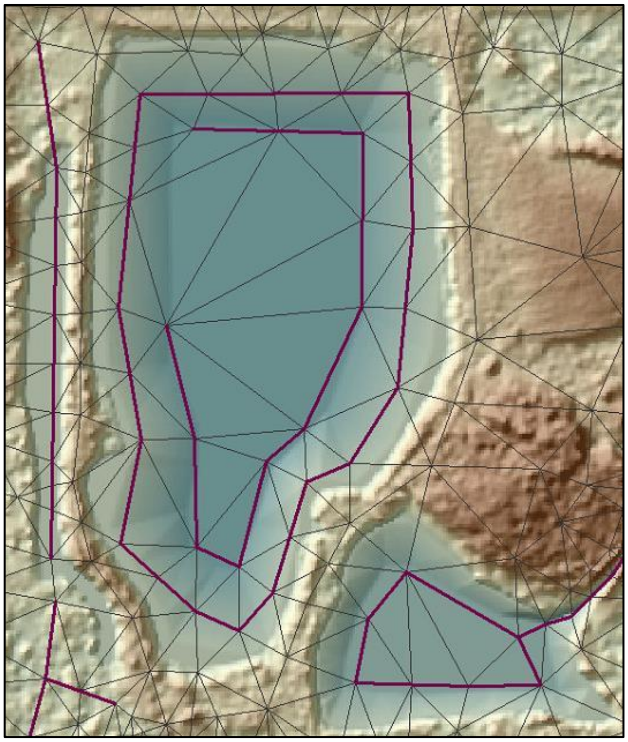


Figure 3.12: Pond Breaklines

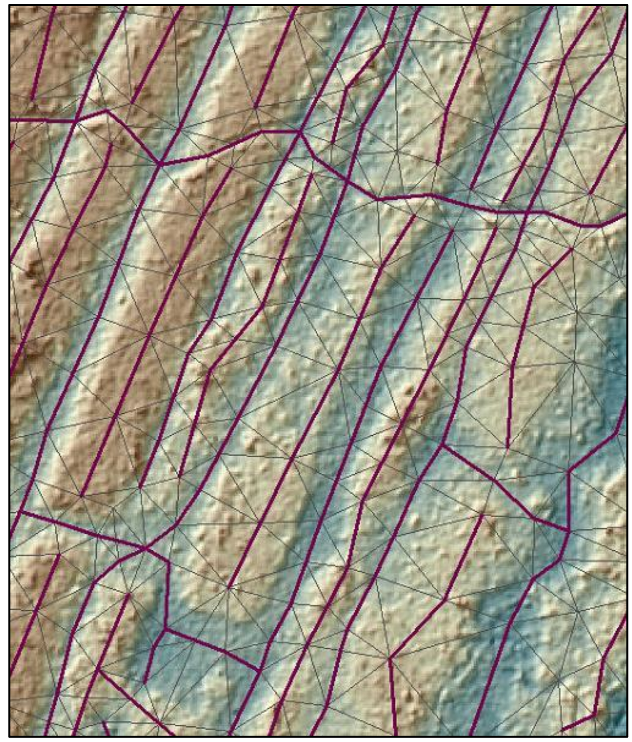


Figure 3.13: Breaklines in Ridges/Troughs

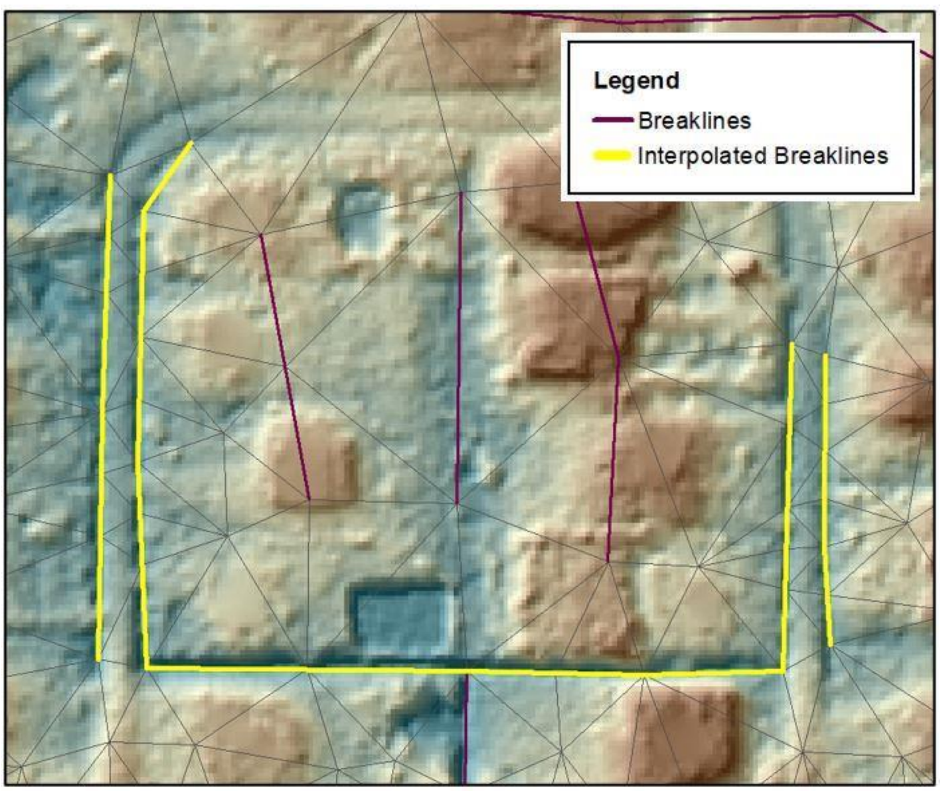


Figure 3.14: Interpolated Breaklines

3.4.3 Channel Features: This feature class represents channels or streams in the 2D model area (See **Figure 3.15**). Channel features are generally drawn along the channel flowline and are associated with channel control volumes (see **Section 3.4.4**). Channel features are used by the model to calculate flow at entry points from adjacent, individual 2D mesh links. The entry point locations are used to interpolate water surface elevations along the 1D channel based on water surface elevations at the ends of the 1D link. The interpolated elevations then serve as “local” boundary conditions for 2D link flow calculations between the overland low region and the channel itself. Channel features are included in the project’s geodatabase in the “OF_Channel” feature class.

3.4.4 Channel Control Volumes: This feature class represents control volumes in the 2D region which are assigned to 1D nodes along a 1D channel. Channel control volumes are developed based on terrain and survey data and generally extend halfway upstream and downstream along the channel links. They represent the spatial extent of 1D channels and span the extent of the cross-sectional data, often from top of bank to top of bank (See **Figure 3.16**). Each vertex along the channel control volume becomes an entry point where water can move between the 2D overland flow area and the 1D system, as explained above in **Section 3.4.3**, however, overland flow links are not included along the edges of the polygon. Channel control volumes are also incorporated within the 1D mapped basins where surface-water groundwater interaction is anticipated.

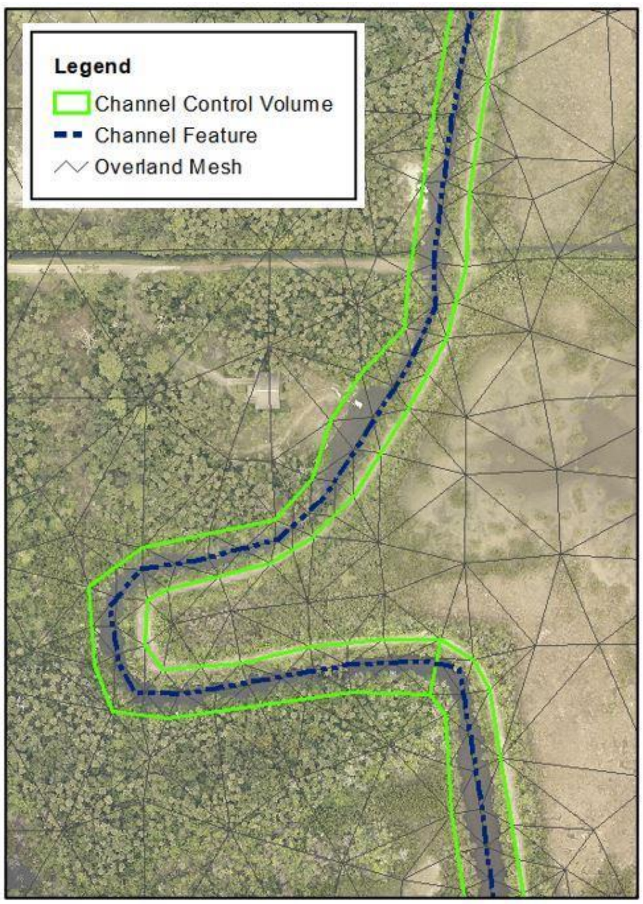


Figure 3.15: Channel Feature

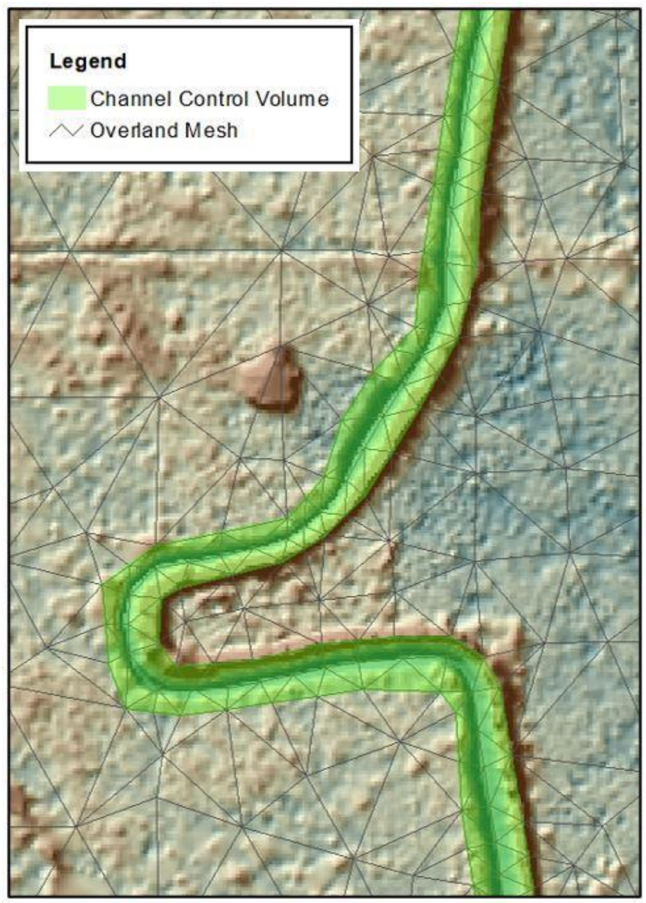


Figure 3.16: Channel Control Volume on DEM

3.4.5 Pond Control Volumes: This feature class represents control volumes that are assigned to 1D nodes used to model storage areas in the 2D region assuming "level pool" conditions (See **Figures 3.17 and 3.18**). In most cases, a pond control volume represents stormwater ponds, natural ponds, lakes, or "non-flowing" wetland areas where ponding is expected. Pond control volumes are also incorporated in low-lying areas within the 1D mapped basins where surface-water groundwater interaction is anticipated. As with channel control volumes, each vertex along the pond control volume becomes an entry point where water can move between the 2D overland flow area and the 1D system. Storage in pond control volumes is provided using stage-area relationships derived from terrain data.

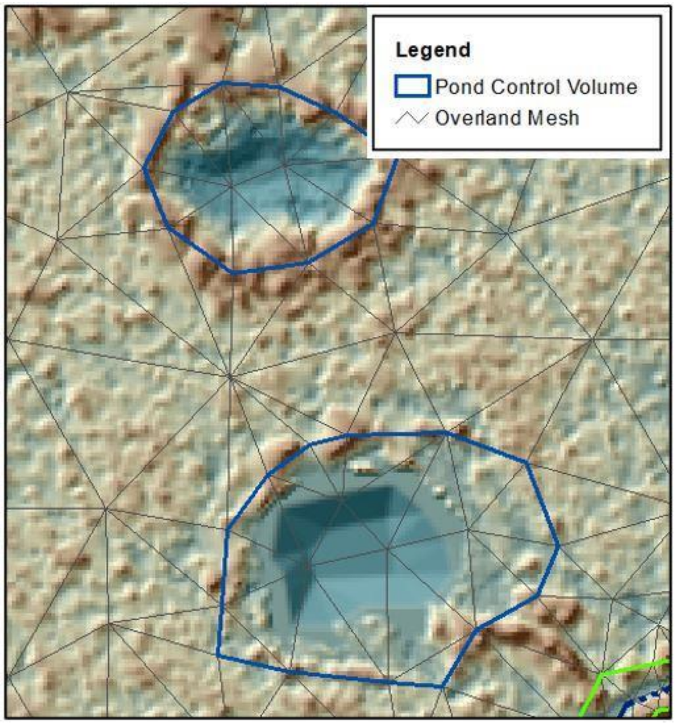


Figure 3.17: Pond Control Volumes 2D Region

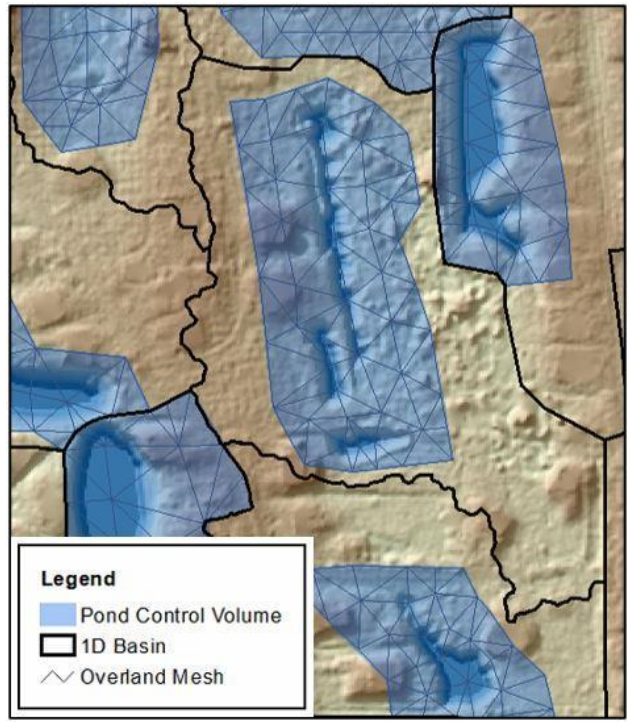


Figure 3.18: Pond Control Volumes 1D Basin

3.4.6 Coves: This feature class is used to identify coves along channels (See **Figure 3.19**). Coves represent lateral, offline level-pool storage areas along a channel. They are associated with channel control volume features. The model uses these features to set elevations in the cove for calculations of flow between the overland flow region and the channel/cove system in much the same way as channel control volumes. The water surface elevation in the cove is based on an interpolated elevation along the associated channel feature. An interpolation point is included with the cove features to identify the location along the channel that is used for this interpolation.

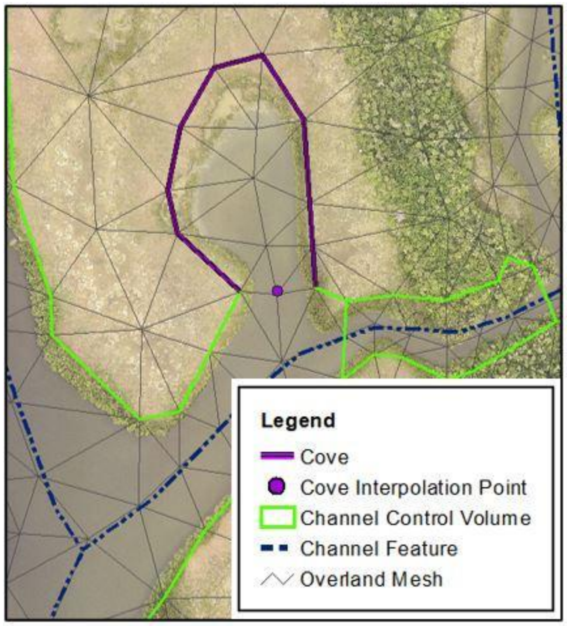


Figure 3.19: Cove and Cove Point

3.4.7 2D Weirs: 2D weirs can be used to model overflow across roadways, berms, or walls inside of the 2D overflow region. The vertices along the 2D weir lines communicate directly with the mesh to allow flow from one side of the weir to the other. 2D weir inverts for the NMI model are based on the DEM elevations along the weir line, however inverts for 2D weirs can also be manually set by the user. The use of a 2D weir simplifies the mesh in areas where multiple breaklines would otherwise be required.

3.4.8 Interface Nodes: This feature class represents locations that require an interface between the 2D overland flow areas and 1D hydraulic elements (weirs, pipes, control structures, etc.). The interface nodes are defined where pipes discharge into the overland flow mesh or at locations where other hydraulic features connect to 1D storage areas (i.e., control volumes) to the overland flow mesh. These nodes are included in the project geodatabase in the “OF_Node” feature class.

3.4.9 Roughness in 2D Areas: Manning’s roughness is assigned within the 2D overflow region based on landcover. The model uses this data to determine lag time and generate hydrographs for the 2D surface, and to route flow through the 2D links (i.e., triangle sides within the mesh). This is through the use of a Lookup Table (enter LU table name here if it exists) in the model. **Table B.3 (Appendix B)** presents the Manning’s roughness coefficients for the Roughness Data Set within the model for varying landcover types.

3.4.10 QA/QC Process Description: Internal QA/QC was performed throughout the model network development process. The SAI Team held weekly progress meetings which were used to discuss any issues, questions, or to obtain input from other Team members on the best approach for a particular area. This allowed for real-time collaboration on modeling approach and level of detail. As with QA/QC efforts detailed earlier in **Section 3.3.3**, A “Comments” geodatabase was employed to track comments and their response/resolution to ensure all internal QC comments were addressed prior to finalizing the model network.

3.5 Boundary Conditions

Boundary conditions were established to represent boundary nodes at each major outfall into the IRL and Banana River. In total, there are 12 boundary nodes: seven located in the IRL and five located in the Banana River. In addition, there are 13 boundary stage lines for the overland flow region. Boundary stage lines allow for interpolation between two boundary node points and serve as boundary conditions for the overland 2D mesh. Boundary elements are shown in **Figure 3.20**.

Boundary information for the continuous simulation is based on historical water elevations from Hurricane Irma in 2017 within the IRL and Banana River. The boundary data were established using the Coastal Modeling System (CMS-Flow) model to simulate water levels throughout 2017, including Hurricane Irma. The CMS model has been previously calibrated and was also compared to 2017 values from the USGS monitoring station at Haulover Canal. Please refer to the *Development of Input Rainfall and Stage Conditions Data for North Merritt Island* (Applied Ecology, Inc., 2021) included as **Appendix C** for more information on the development of boundary stage data for the NMI model. This report also discusses rainfall data for Hurricane Irma and anticipated rainfall under future conditions.

Boundary information for discrete storm events will be established during future phases of the NMI watershed evaluation.

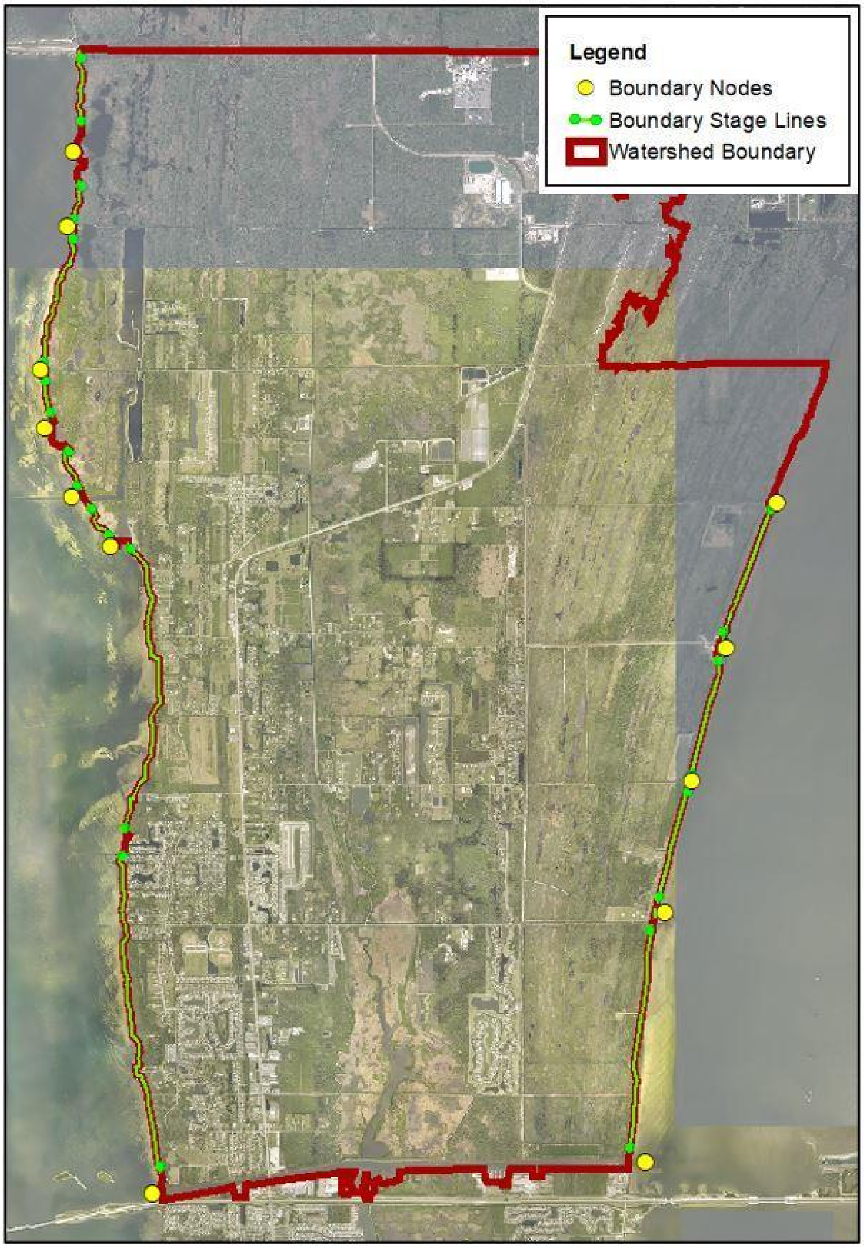


Figure 3.20: Boundary Node and Line Locations

4.0 Groundwater Model Development

4.1 Groundwater Region Development

A total of seven (7) groundwater regions were created as part of the ICPR4 model development. In general, groundwater region boundaries were split along channel features that were mostly and continuously inundated. When the water pierces the ground surface while the surface is inundated, a known head condition is placed at the corresponding groundwater nodes. The known head condition is derived from water surface elevations in the surface model component. Therefore, when two groundwater regions share a common edge along a water feature, both regions use the same known head condition. The reason for using multiple groundwater regions is to speed up the computations. Multiple regions are processed in parallel.

Figure 4.1: Groundwater Regions

4.2 Breaklines & Breakpoints

Groundwater breaklines and breakpoints were placed to refine the groundwater mesh. Groundwater breakpoints used a point spacing pattern that was two times the spacing of the overland flow breakpoints. The groundwater breakpoints were also placed so they aligned with the overland flow breakpoints. Groundwater breaklines are consistent with the overland flow breaklines that were placed in recharge areas where groundwater/surface water interaction is anticipated. Generally, these are along the bottom of channels and ponds.

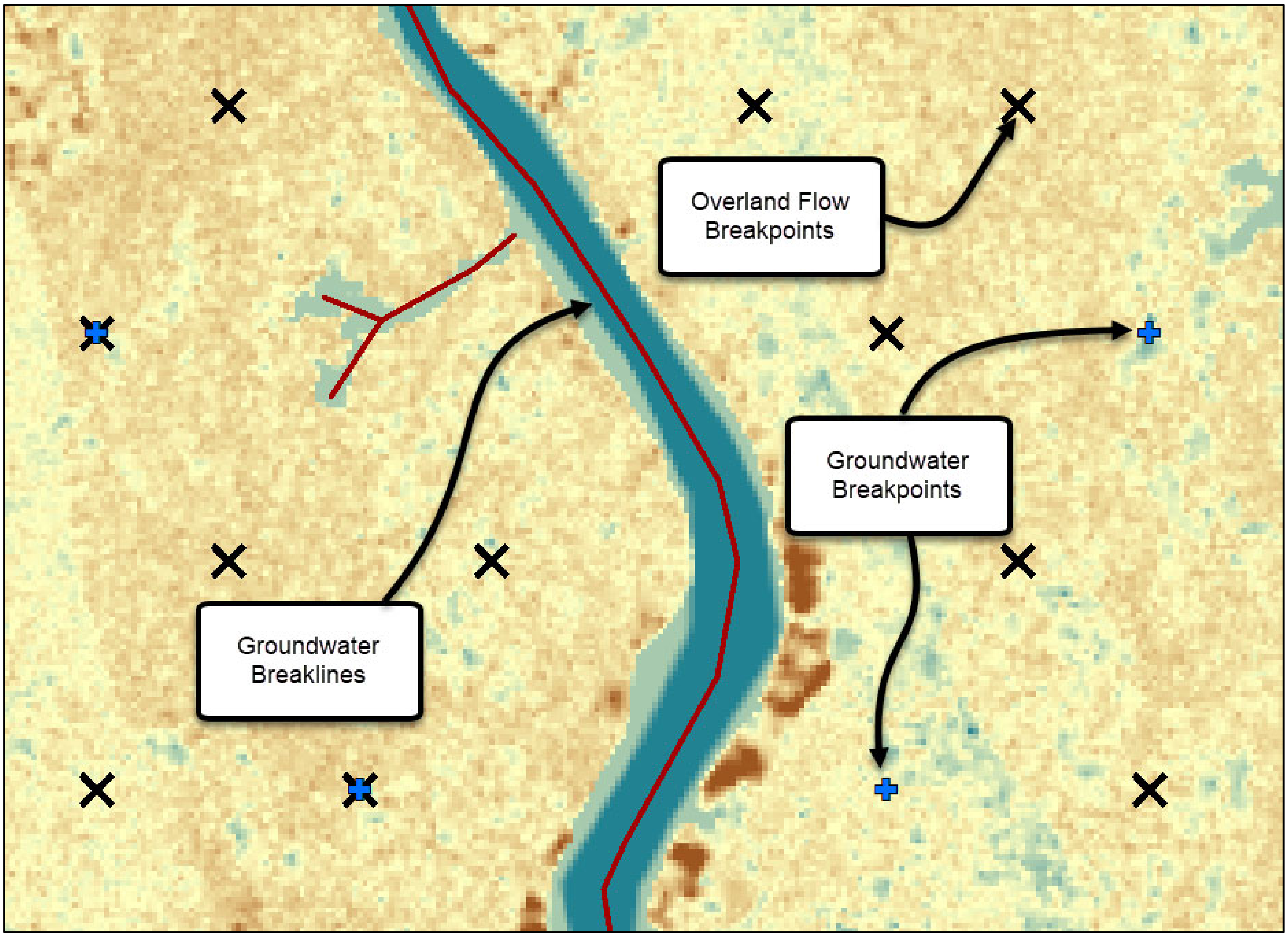


Figure 4.2: Groundwater Breakpoint & Breakline Placement

4.3 Groundwater Parameterization

Groundwater parameters were based on data from the East-Central Florida Transient Expanded (ECFTX) Model. Data extracted from the ECFTX model includes:

- Surficial Aquifer Base
- Surficial Aquifer Saturated Hydraulic Conductivity
- Surficial Aquifer Porosity
- Confining Layer Bottom Elevation
- Confining Layer Saturated Hydraulic Conductivity

A 1,250-ft X 1,250-ft gridded map layer was also created from the ECFTX model data to parameterize the groundwater (**Figure 4.3**). Corresponding porosity and saturated hydraulic lookup tables were developed based on each map layer zone. The model specific information can be found in the provided ICPR4 model and the ECFTX model (Reference Document NMI_216).

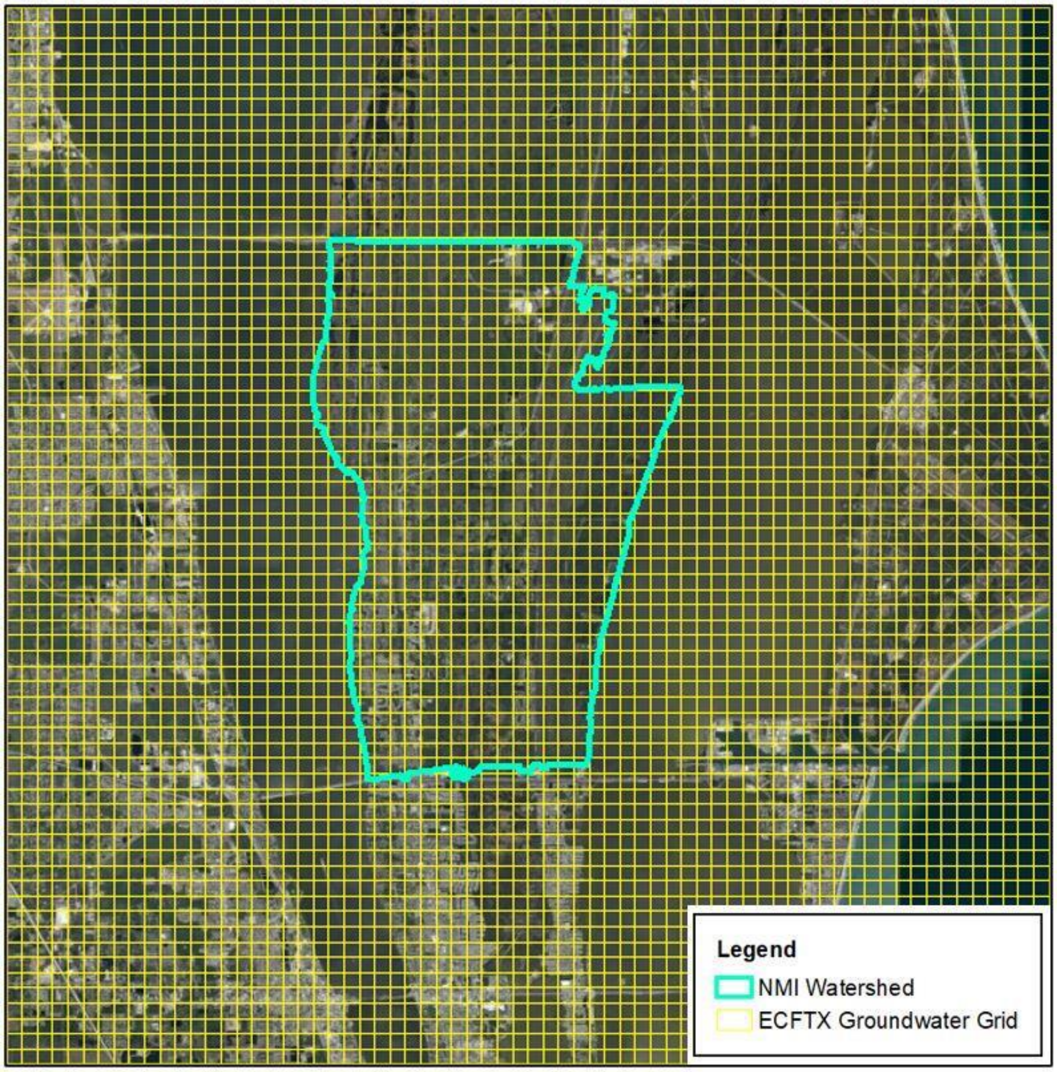


Figure 4.3: ECFTX Groundwater Data Grid

4.4 Boundary Conditions

The boundary conditions for the groundwater model are consistent with those for the overland flow region. The boundary conditions were incorporated into the model using 14 groundwater boundary stage lines as shown in **Figure 4.4**.

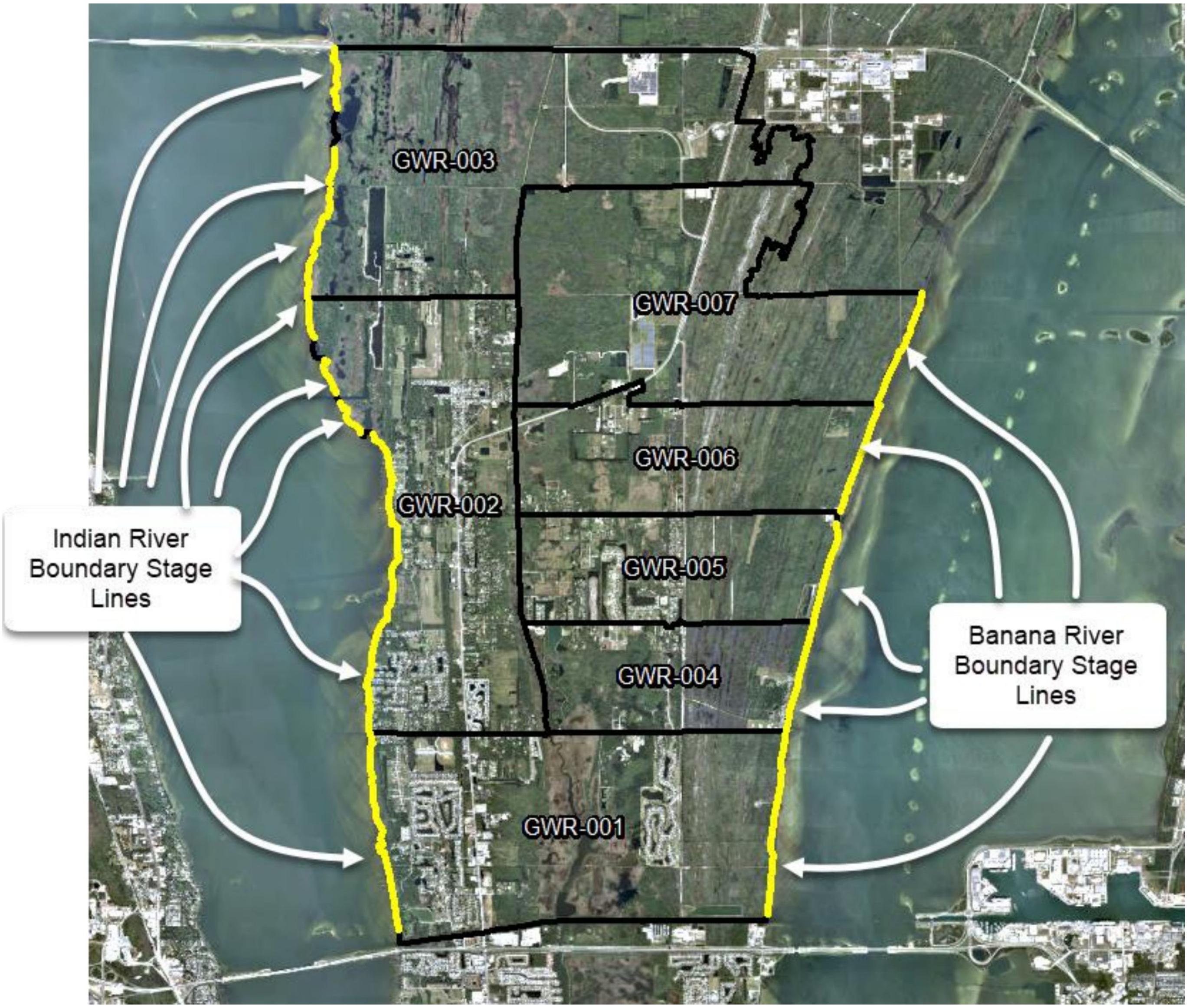


Figure 4.4: Groundwater Boundary Stage Line Locations

5.0 Field Data Acquisition Summary

5.1 Field Verification Efforts

Field data collection in the North Merritt Island Watershed occurred at sites where data evaluated from SWAMP database or ERP documentation was inconsistent or absent. Field crews photographed and documented each hydraulic feature visited noting condition, material, and dimensions for the 85 sites identified. As appropriate crews also verified drainage patterns where available digital data proved inconclusive or did not provide enough information to determine the drainage pattern. Depending upon the field observations, recommendations were made to provide immediate maintenance and/or provide a survey of the observed structures. **Figure 5.1** provides a spatial view of structures visited, highlighting those with additional survey needs. See **Appendix D** for the complete Field Data Collection Memorandum, which includes field observations and representative photos of each site visited.

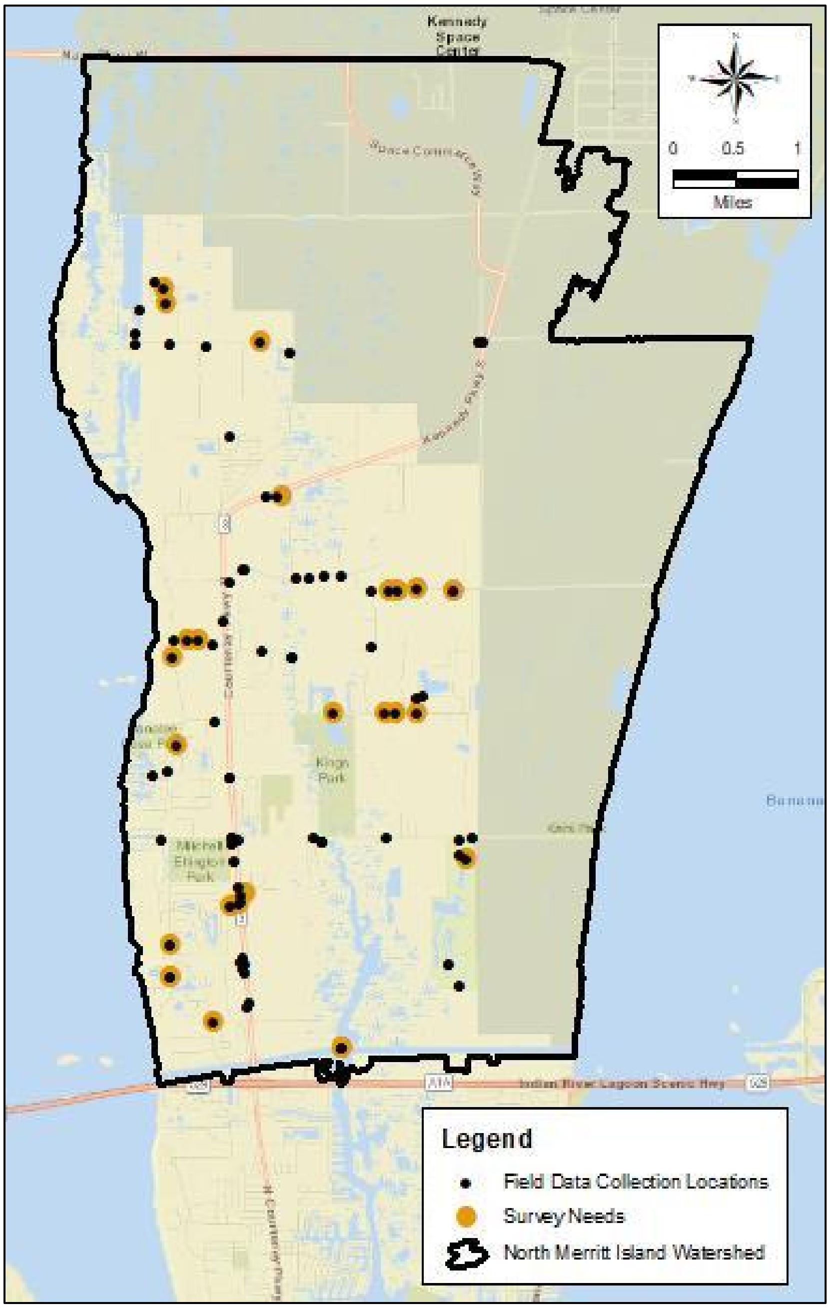


Figure 5.1: North Merritt Island Field Data Collection Sites

5.2 Survey Needs Assessment

Based upon the field review of structural elements, 28 sites were identified as needing survey to characterize the structure beyond the data that was available from field reconnaissance, and 32 structures where maintenance is recommended. The structures where survey data and/or maintenance activities is recommended are listed in **Table 5.1**. A complete listing of all sites visited along with the date of the field visit, survey needs, and maintenance needs, are provided in the Field Data Collection Memorandum included in **Appendix D**.

Table 5.1: Summary of Structures Requiring Survey and/or Maintenance

Link Name	Link Type	SWAMP ID	Date Visited	Survey Required	Maintenance Required
DSykesS_1	Drop Structure	BCE611CS012		Y	
DSykesS_2	Drop Structure	BCE611CS010		Y	
DSykesS_3	Drop Structure	BCE611CS008		Y	
PB2020_1	Pipe	P016D634021022	03-Nov-20		Y
PB4040_1	Pipe	233634CU47AB	03-Nov-20	Y	Y
WB3060_1	Weir	Not in SWAMP	03-Nov-20		Y
PC1080_1	Pipe	NO FEATURE CODE	03-Nov-20		Y
PC1092_1	Pipe	Not in SWAMP	03-Nov-20	Y	Y
PC1130_1	Pipe	Not in SWAMP	03-Nov-20	Y	Y
PC1160_1	Pipe	In SWAMP wo feature code	03-Nov-20	Y	Y
PD1070_1	Pipe	0000CU0000	03-Nov-20		Y
PDD1002_1	Pipe	BC233624CU009	03-Nov-20	Y	Y
PDD1010_1	Pipe	BC233624CU008	03-Nov-20	Y	
PDD3315_1	Pipe	P015D623029030	03-Nov-20		Y
PDD3335_1	Pipe	P021D623042041	03-Nov-20		Y
PDD3345_1	Pipe	P012D623024023	03-Nov-20		Y
PDD3405_1	Pipe	P007D623013014	03-Nov-20		Y
PEE1060_1	Pipe	P094D634135133	03-Nov-20		Y
PEE3020_1	Pipe	In SWAMP wo feature code	03-Nov-20	Y	
PEE3040_1	Pipe	In SWAMP wo feature code	03-Nov-20	Y	Y
PEE3280_1	Pipe	Not in SWAMP	03-Nov-20	Y	Y
PEE4160_1	Pipe	P161E603217218	03-Nov-20	Y	Y
PEE5060_1	Pipe	P023E610033034	03-Nov-20	Y	Y
PEE5060_2	Pipe	P025E610039040	03-Nov-20	Y	Y
PF2110_1	Pipe	NO FEATURE CODE	03-Nov-20		Y
DFF1230_1	Drop Structure	BCE611CS096	05-Nov-20		Y
PFF1060_1	Pipe	P088E602133134	05-Nov-20	Y	
PFF1180_1	Pipe	P042E611074075	05-Nov-20		Y
PFF1210_1	Pipe	P045E611079080	05-Nov-20		Y
PGG1010_1	Pipe	P001E601011010	05-Nov-20	Y	
PGG1060_2	Pipe	Not in SWAMP	05-Nov-20		Y
PGG1150_1	Pipe	P014E612021022	05-Nov-20		Y
DL1790_1	Drop Structure	BCD625CS099	05-Nov-20	Y	
PL1345_1	Pipe	BC233625CU010	03-Nov-20	Y	Y

PM2970_1	Pipe	P006D624008007	03-Nov-20		Y
PM2980_1	Pipe	0000CU0000	03-Nov-20	Y	Y
PM3000_1	Pipe	0000CU0000	03-Nov-20	Y	Y
PO2960_1	Pipe	Not in SWAMP	03-Nov-20	Y	
PO3030_2	Pipe	BC233636CU003	05-Nov-20	Y	
PPI2010_1	Pipe	0000CU0000	03-Nov-20	Y	Y
PPI2010_2	Pipe	0000CU0000	03-Nov-20	Y	Y
PR3200_1	Pipe	0000CU0000	03-Nov-20	Y	
DS1140_1	Drop Structure	0000SC0000	03-Nov-20	Y	
PU4220_1	Pipe	0000CU0000	03-Nov-20	Y	Y

6.0 Model Calibration and Verification

The North Merritt Island ICPR4 calibration/verification analysis was comprised of a continuous simulation for the year 2017. The model was calibrated to a single historical event (Hurricane Irma: 08/31/2017-09/29/2017) and then verified using two subsequent storm events that occurred between 10/01/2017-11/03/2017. The model results were compared to the recorded stage measurements at the 16 active Brevard County gages (**Figure 6.1**). Gage readings were only available for the calibration/validation period (08/31/2017 thru 11/03/2017). Additionally, all gage readings were recorded manually by Brevard County staff. The gage records are provided in reference document NMI_222.

6.1 Statistical Metrics

Comparisons between the measured and model data include the following statistical metrics:

- Coefficient of Determination (R^2)
- Nash-Sutcliffe Model Efficiency Coefficient (NSE)
- Mean Error (ME)
- Mean Absolute Error (MAE)
- Root Mean Square Error (RMSE)
- Ratio "RMSE/Standard Deviation (Observed)" (RSR)
- ME, MAE and RMSE within ½ Standard Deviation (Observed)

Calibration targets were established for the modeling effort to assess the accuracy of the model data versus measured data. These targets are consistent with the calibration targets set in the *ICPR4 Hydrologic Modeling Support for Johns and Avalon Lakes* report (03/26/2021) by Streamline Technologies, Inc. (SLT) for the St. Johns River Water Management District (SJRWMD). Moriasi, et al. (2007, 2015) provides guidance on performance ratings with categories of "Very Good", "Good", "Satisfactory" and "Not Satisfactory". **Table 6.1** is derived from those two references and from SLT's modeling experience in Florida.

Table 6.1: Statistical Metrics

Metric	Very Good	Good	Satisfactory	Not Satisfactory
Coefficient of Determination (R^2)	$R^2 > 0.85$	$0.75 < R^2 <= 0.85$	$0.60 <= R^2 < 0.75$	$R^2 < 0.60$
Nash-Sutcliffe Efficiency (NSE)	NSE > 0.80	0.70 < NSE <= 0.80	0.50 < NSE <= 0.70	NSE <= 0.50
Mean Error (ME) ft	|ME| <= 0.25'	0.25' < |ME| <= 0.5'	0.50' < |ME| <= 1.0'	|ME| > 1.0'
Mean Absolute Error (MAE) ft	MAE <= 0.50'	0.50' < MAE <= 0.75'	0.75' < MAE <= 1.5'	0.75' < MAE <= 1.5'
Root Mean Square Error (RMSE) ft	RMSE <= 0.75'	0.75' < RMSE <= 1.00'	1.00' < RMSE <= 2.00'	RMSE > 2.00'
Ratio "RMSE/SD-Observed" (RSR)	RSR <=0.50	0.50 < RSR <=0.60	0.60 < RSR <= 0.70	RSR > 0.71
½ Standard Deviation (Observed) ft	3 |ME|, MAE, RMSE	2 |ME|, MAE, RMSE	1 |ME|, MAE, RMSE	0 |ME|, MAE, RMSE

The following notes were taken from Moriasi, et al. (2007, 2015).

R^2 – The coefficient of determination, R^2, is widely used in hydrologic modeling studies and describes the degree of collinearity between simulated and observed data. It is oversensitive to high extreme values and insensitive to additive and proportional differences between model predictions and measured data. The slope and y-intercept of the best-fit regression line can indicate how well simulated data match measured data. The slope indicates the relative relationship between simulated and measured values. The y-intercept indicates the presence of a lag or lead between model predictions and measured data, or that data sets are not perfectly aligned. The intercept should be close to zero and the gradient close to 1.0 for good agreement.

NSE – The Nash-Sutcliffe efficiency, NSE, is a normalized magnitude of the residual variance ("noise") compared to the measured data variance ("information"). NSE indicates how well the plot of observed versus simulated data fits the 1:1 line. It is widely used and is good for continuous simulations. NSE cannot identify model bias and cannot be used to identify differences in timing and magnitudes of peaks and shape of recession curves. NSE is sensitive to extreme values due to the squared differences.

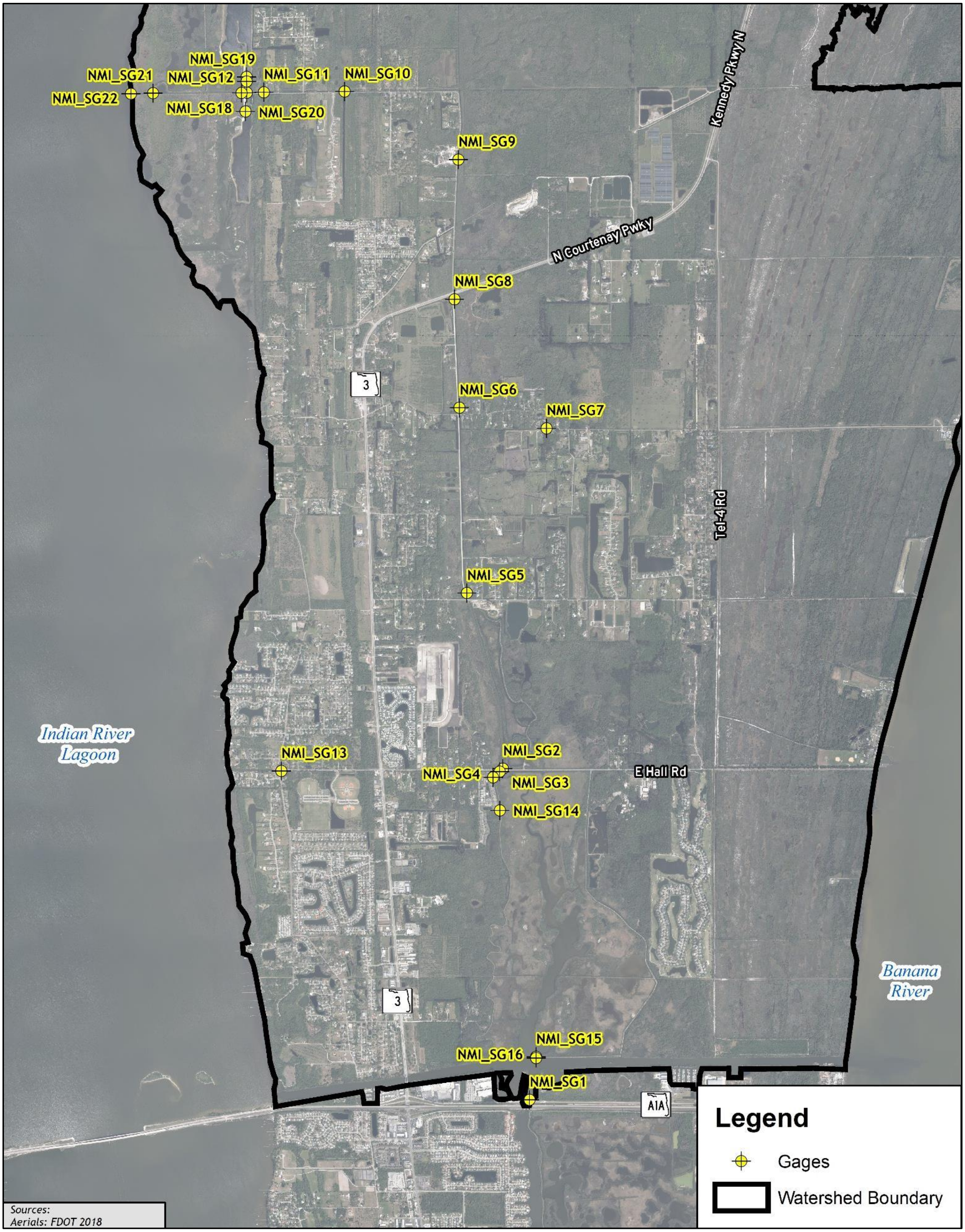


Figure 6.1: Brevard County Gage Locations

ME, MAE & RMSE – These parameters work well for long-term continuous simulations and are commonly used in model performance evaluation. They are reported in the same units as the model output and easy to interpret. Singh et al (2004) stated that RMSE and MAE values less than half the standard deviation of the observed data may be considered low and that either is appropriate for model evaluation.

RSR – RSR includes a scaling/normalization factor and consequently removes some of the arbitrariness of setting a target value for RMSE. One disadvantage is that it gives more weight to high values when compared to low values. It has not been widely used in hydrologic modeling literature since it is a relatively new statistical performance measure.

6.2 Model Calibration

The calibration period of record for this study is from 08/31/2017 – 09/29/2017 (Hurricane Irma). The calibration analysis includes comparisons between measured and modeled results for each of the 16 staff gages with adjustments to the model input as needed. The rainfall data and boundary condition information for the study area was provided by Applied Ecology, Inc (AEI). For details on these data, refer to Sections 2 and 3 of this report. The subsequent section discusses the parameter adjustments performed as part of the calibration analysis as well as the final results for the calibration simulations.

6.2.1 Parameter Adjustments

Initial Stages: The initial stages for the surface and groundwater nodes were established by running a preliminary simulation with estimated initial stages at the onset (01/01/2017 0.00 hrs). The results from the preliminary simulation at 12/31/2017 23.00 hrs were then used to specify the initial stages for the remaining calibration simulations at 01/01/2017 0.00 hrs. The initial conditions were not modified for the final calibration simulation mainly because it was determined that the initial conditions had no noticeable effect on the calibration-verification analysis and the calibration-verification storm events occurred much later in 2017 (September-October). Essentially, the "spin up" period prior to these calibration-verification storm events was sufficient for the modeled stages to represent actual field conditions during the calibration-verification period.

Green-Ampt Parameter Adjustments: During the initial calibration simulation for Hurricane Irma, infiltration and recharge to the groundwater appeared extremely high for pervious areas based on comparisons with observed staff gage data. This resulted in modeled stages well below the observed stages at all gage locations except SG1 which is located along the Barge Canal and is tidally influenced.

It was determined that the low runoff volumes were caused by the relatively high vertical saturated hydraulic conductivities that were based on the NRCS soils parameters. Consequently, several iterations with lower vertical saturated hydraulic conductivity values were evaluated. Based on the analysis, it appears the NRCS data drastically overestimate vertical saturated hydraulic conductivity for the study area. Therefore, the Green Ampt vertical conductivities were reduced approximately two orders of magnitude for the modeled and measured stages to be comparable. The calibrated Green-Ampt parameters are provided in the calibrated model included with this submittal.

Groundwater Parameter Adjustments: The calibration effort indicated that the initial values used for the horizontal saturated hydraulic conductivity (Ksat) for the groundwater were too high. As a result, the Ksat value for each groundwater conductivity zone was lowered to a uniform value of 5 ft/day.

Boundary Conditions: While the boundary data are the best available information, the time-stage data for the boundary conditions were based on the hydrodynamic model results provided by AEI. The calibration results showed that the stage-hydrographs recovered much faster than what was observed in the field. The model network was reviewed along with the input parameters for any warranted changes to the input data. While the model network was revised and changes made to the model input/parameters (i.e., increasing Manning's n for channels), the subsequent calibration simulation results showed little to no impact on the stage hydrograph recovery at the gage locations. As a result, the boundary conditions were then evaluated.

Several calibration simulations were conducted with varying increases in the boundary stages. Based on that calibration analysis, it was determined a uniform adjustment of +6.0-inches to all boundary conditions yielded the best accuracy between the model results and measure data at the gage locations. The change to the boundary conditions was discussed with staff at AEI and it was verified that this was an acceptable parameter adjustment for the calibration-verification analysis.

Pump Rating Curve Links Conditions: The pump rating curves at Pine Island and the Mosquito Impoundment were adjusted to match the pump operation, as best as possible, during the calibration and verification storm events. At Pine Island, the pump rating curve links (RPI1030_2 & RPI1030_1) were adjusted to account for the initial drawdown observed in the gage data prior to Hurricane Irma. A temporary pump (Link PICA_Temp) was also added to the model based on discussions with County staff.

At SG3 East Hall Road Pump House, the pumping rates for the pump rating curve links (RO6A & RO6) were reduced by 50% based on model calibration results. Additionally, a temporary pump (Link Hall_Temp_Pump) was incorporated into the calibration-verification model. This temporary pump was in place during the calibration-verification period per the County staff.

It is important to note that, per discussions with County staff, there were several manual adjustments to pump operation at both locations that are not reflected on the pump logs. Consequently, an additional calibration-verification simulation was developed by placing time-stage nodes in the vicinity of gages SG1, SG4 and SG17 to mimic the actual pump operation and its effects on water levels upstream of the pumps. The locations that were converted to time-stage nodes are listed in **Table 6.2**. Essentially, this simulation was developed to confirm that if the pump operation data were available, the calibration-verification comparisons would be more accurate.

Table 6.2: Calibration Simulation #2 Additional Boundary Conditions

Gage	Nodes Convert to Time/Stage
SG1	NHH1010 NHH1020 NHH1030 NHH1040 NHH1050
SG4	NFF2020
SG17	NPI1030

6.3 Calibration Analysis

As previously mentioned, there are a total of 16 active gages within the study area. The calibration period of record is from 08/31/2017 – 09/29/2017 (Hurricane Irma). The calibration analysis included comparisons between measured and modeled results for each of these staff gages. The results of the final calibration analysis results are provided in the subsequent sections. Note that there are two calibration analyses for each gage unless otherwise stated. Calibration #1 is the simulation that includes the final parameter adjustments and the adjusted boundary conditions provided by AEI. Calibration #2 is identical to Calibration #1 except the internal boundary conditions at the locations specified in **Table 6.2** were incorporated into the model. Each analysis includes comparisons between the gage, denoted by the "SG" prefix, and the node in the model used for the comparisons.

6.3.1 Gage SG1 Sykes Creek at Sea Ray Dr. – Calibration Results

Gage SG1 Sykes Creek is located along the Barge Canal at the far south end of the study area. The stage hydrograph comparisons at this location are provided in **Figure 6.2**. As shown in the figure, the modeled and simulated results compare very well with stages generally within 4.1-inches of one another. The peak stages for the measured (1.72-ft, NAVD88) and simulated (1.48-ft, NAVD88) differ by just 2.9 inches. No comparisons for the Calibration #2 simulation are necessary at this location because it is one of the locations that was converted to a time-stage node for that analysis.

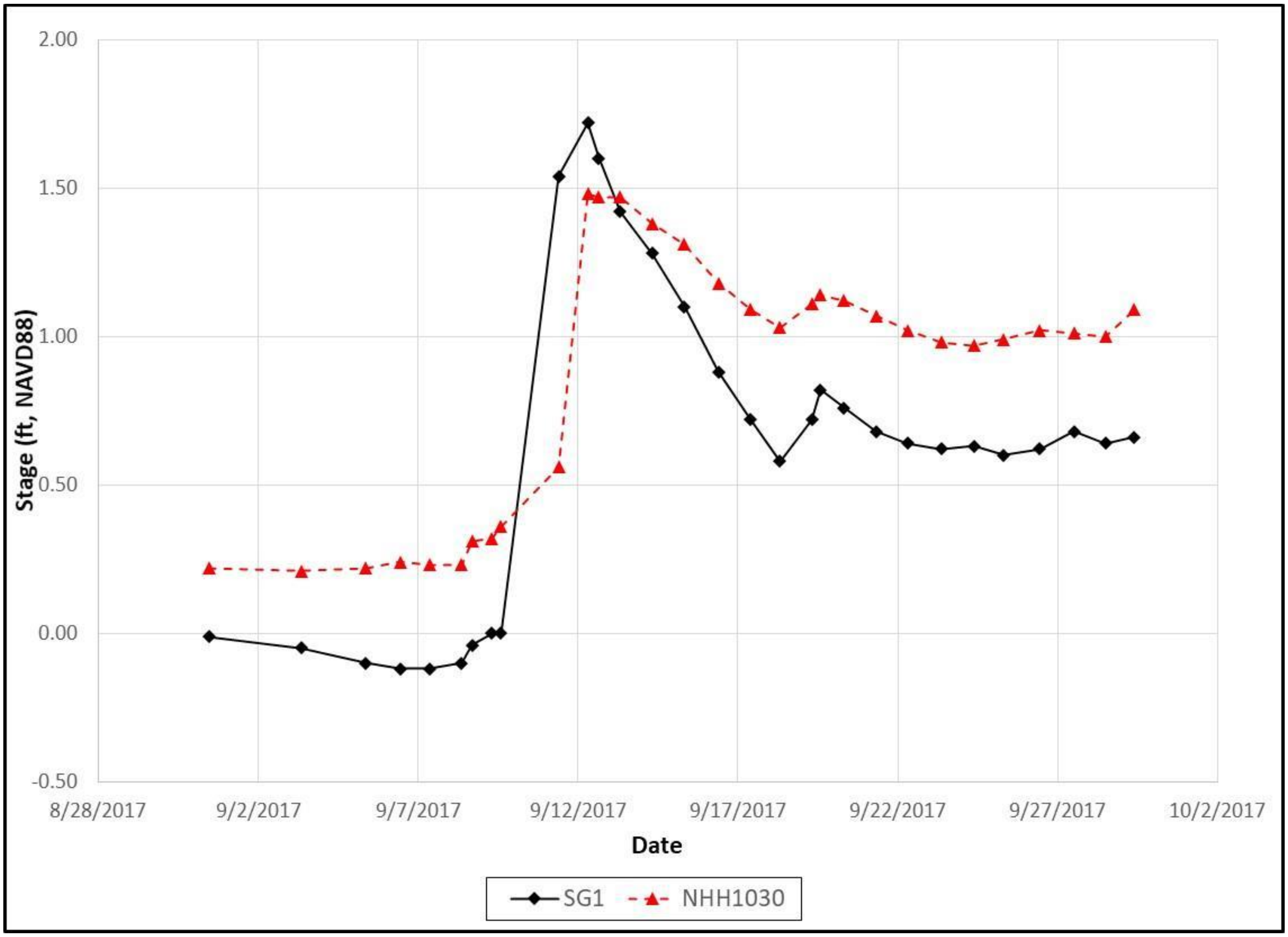


Figure 6.2: SG1 Sykes Creek at Sea Ray Dr. Calibration#1 Comparisons

The statistical comparisons between measured and modeled data are provided in **Table 6.3** below. As shown, all metrics are classified as either "Very Good" or "Satisfactory" which indicates the model is representative at this gage location. Keep in mind that this area is sensitive to the water levels in both the Indian River and Banana River. Consequently, if gage data were available in the Banana River and Indian River at the Barge Canal, it is anticipated that the model results would be improved even further.

Table 6.3: Calibration Statistical Metrics SG1

Metric Parameter	Calibration Simulation #1	Quality #1
R^2	0.742	Satisfactory
NSE	0.524	Satisfactory
ME	-0.249	Very Good
MAE	0.339	Very Good
RMSE	0.371	Very Good
RSR	0.678	Satisfactory
1/2 Standard Deviation Obs.	1	Satisfactory

Note: Number of pair data (observed and simulated) = 30

6.3.2 Gage SG2 East Hall Rd. North – Calibration Results

Gage SG2 is located just upstream of the East Hall Road Pump House. The stage hydrograph comparisons for the Calibration #1 and Calibration #2 are provided in **Figure 6.3** and **Figure 6.4**, respectively. Both calibration simulations compare very well for the maximum measured stage. Calibration #1 modeled peak stage (2.64-ft, NAVD88) is only 3.2-inches above the measured peak stage (2.37-ft, NAVD88). The model tends to recover slightly faster than the measured data. But overall, the model compares very well to the measured data. Calibration #2 modeled peak stage (2.53-ft, NAVD88) is only 1.9-inches above the measured peak stage. Additionally, the staging hydrographs are virtually identical between the measured and modeled stages as would be expected.

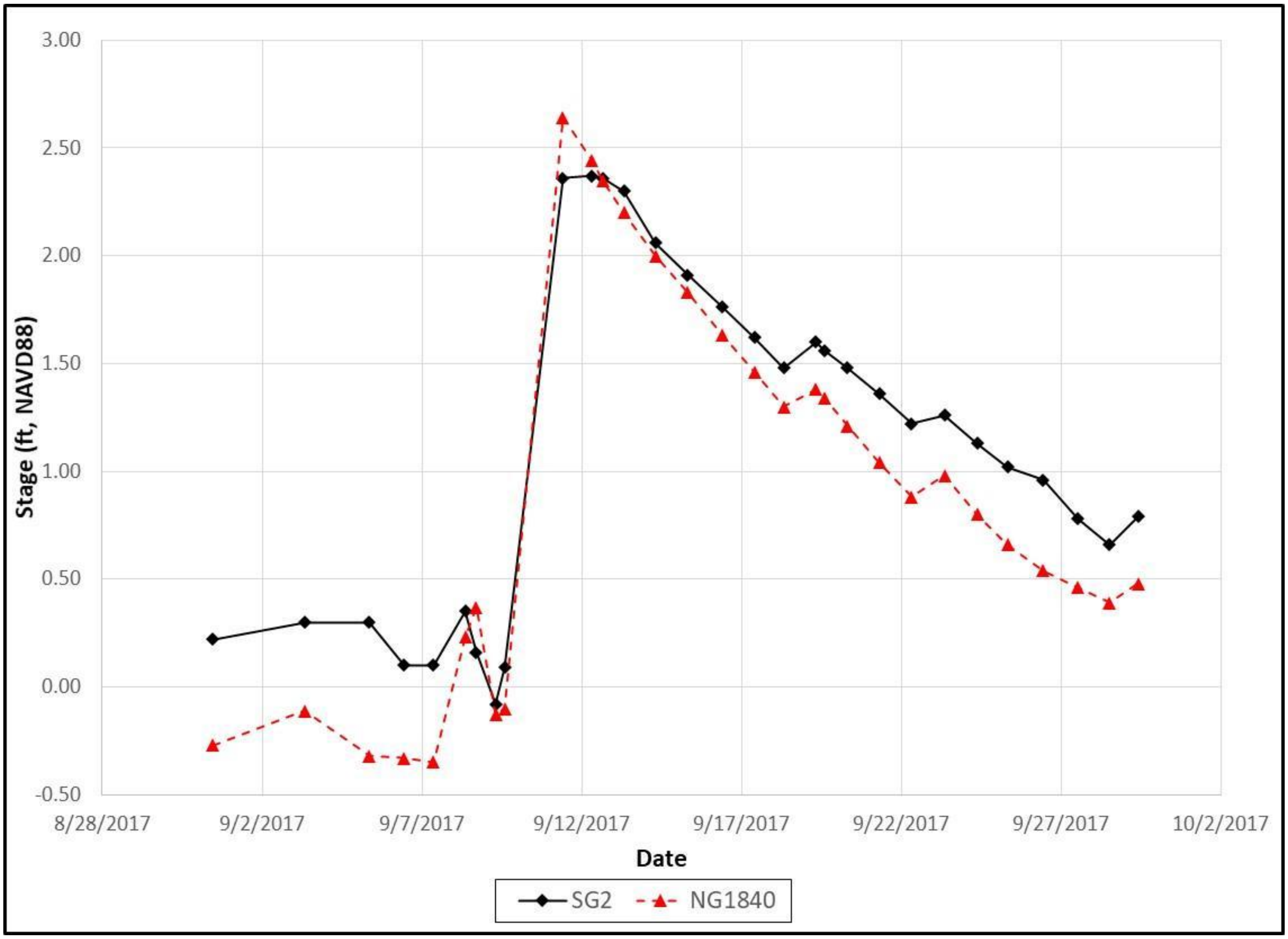


Figure 6.3: SG2 East Hall Rd. North Calibration#1 Comparisons

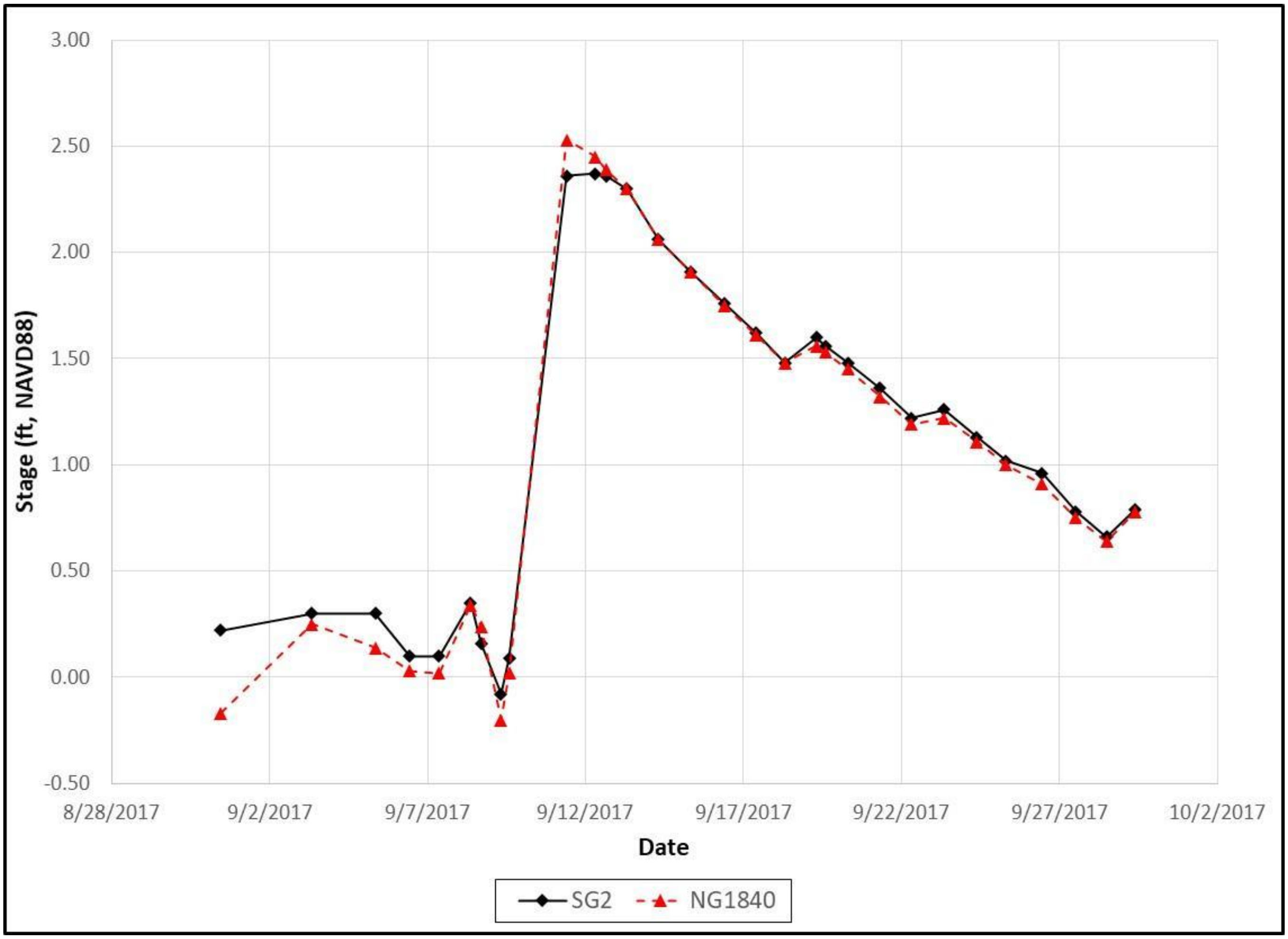


Figure 6.4: SG2 East Hall Rd. North Calibration#2 Comparisons

The statistical metrics for both calibration simulations are provided in **Table 6.4** below. Both sets of simulation results compare well to the measured data with all metrics classified as very good. However, Calibration #2 compares better for every statistical metric.

Table 6.4: Calibration Statistical Metrics SG2

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.963	Very Good	0.992	Very Good
NSE	0.853	Very Good	0.985	Very Good
ME	0.219	Very Good	0.032	Very Good
MAE	0.257	Very Good	0.056	Very Good
RMSE	0.295	Very Good	0.094	Very Good
RSR	0.377	Very Good	0.121	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.3 Gage SG3 East Hall Rd. Pump House – Calibration Results

Gage SG3 is located just west of the East Hall Rd. Pump House. The Calibration #1 stage hydrograph comparisons are provided in **Figure 6.5**. Based on the hydrographs, there is good correlation between the measured and modeled stages. The modeled stages are generally within ~3-inches of the measured stages. Additionally, there is only a minor difference (1.2-inches) between model peak stage (2.46-ft, NAVD88) and measured peak stage (2.36-ft, NAVD88). No comparisons were conducted for at this gage for Calibration #2 since node NFF2020 was converted to a time-stage node for that analysis.

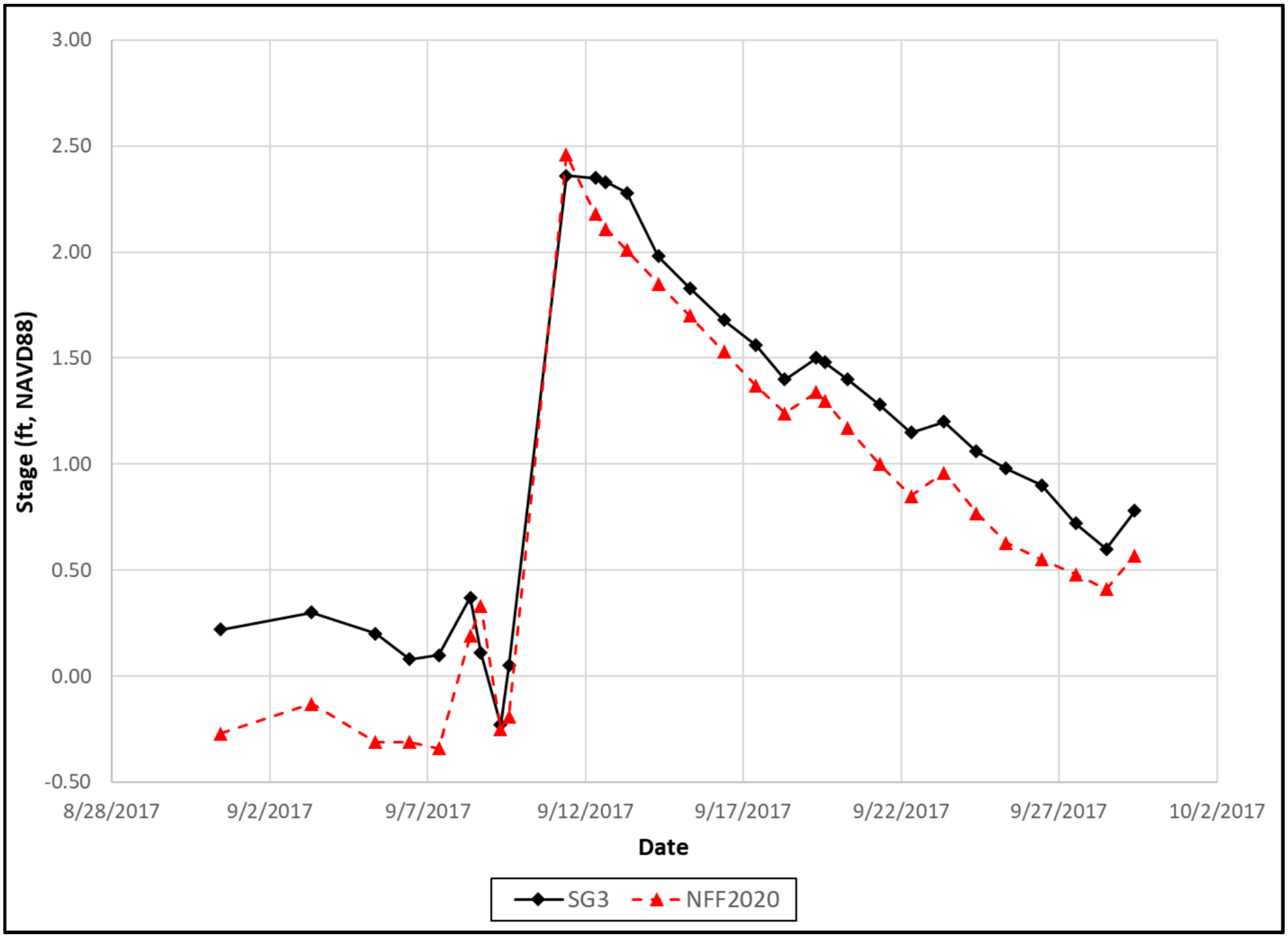


Figure 6.5: SG3 East Hall Rd. North Calibration#1 Comparisons

The statistical metrics for this gage provided in **Table. 6.5** show very good correlation between the measured and modeled stage data.

Table 6.5: Calibration Statistical Metrics SG3

Metric Parameter	Calibration Simulation #1	Quality #1
R^2	0.968	Very Good
NSE	0.872	Very Good
ME	0.227	Very Good
MAE	0.249	Very Good
RMSE	0.274	Very Good
RSR	0.352	Very Good
1/2 Standard Deviation Obs.	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.4 Gage SG4 East Hall Rd. Barge Canal Ditch – Calibration Results

Gage SG4 is located just southwest of the East Hall Rd. Pump House. The Calibration #1 stage hydrograph comparisons are provided in **Figure 6.6**. The difference between the simulated peak stage (2.46-ft, NAVD88) and measured peak stage (2.34-ft, NAVD88) is approximately 1.4-inches. No comparisons were conducted for this particular gage for Calibration #2 since this location was converted to a time-stage node for that analysis.

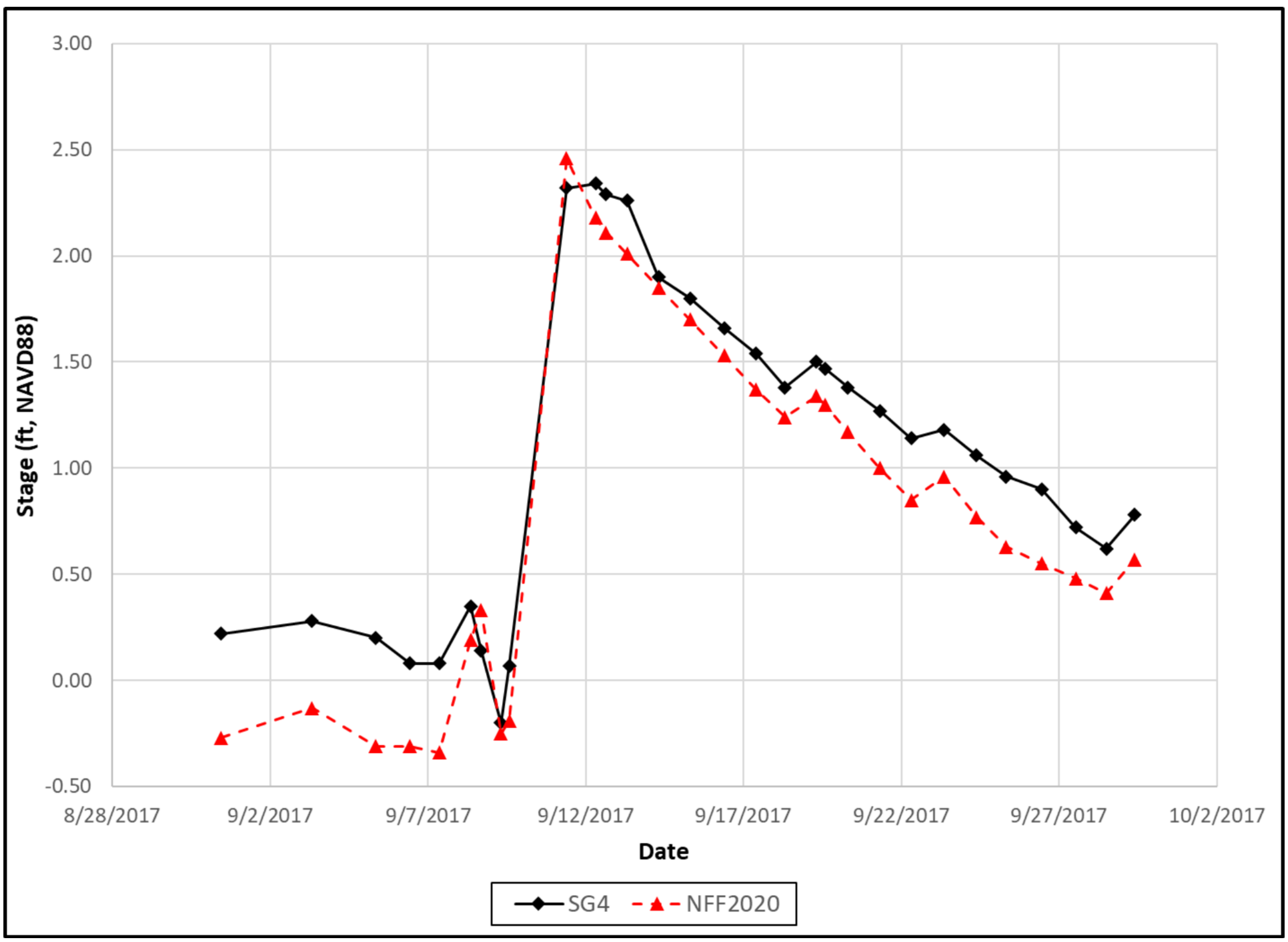


Figure 6.6: SG4 East Hall Rd. Barge Canal Ditch Calibration#1 Comparisons

Additionally, the statistical metrics provided in **Table 6.6** show very good correlation between the measured and modeled stage data.

Table 6.6: Calibration Statistical Metrics SG4

Metric Parameter	Calibration Simulation #1	Quality #1
R^2	0.970	Very Good
NSE	0.876	Very Good
ME	0.216	Very Good
MAE	0.238	Very Good
RMSE	0.265	Very Good
RSR	0.347	Very Good
1/2 Standard Deviation Obs.	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.5 Gage SG5 Chase Hammock at Judson Rd. – Calibration Results

The SG5 gage is located just north of the Judson Rd. and Chase Hammock Rd. intersection along the north-south drainage ditch. Stage hydrograph comparisons for Calibration #1 and Calibration #2 are shown in **Figure 6.7** and **Figure 6.8**, respectively. Both calibration simulations compare very well for the maximum measured stage. Calibration #1 modeled peak stage is 2.78-ft (NAVD88), which is approximately 3.4-inches above the measured peak stage (2.50-ft, NAVD88). Calibration #2 modeled peak stage is 2.69-ft, which is 2.3-inches above the measured peak stage.

The statistical metrics for this gage provided in **Table 6.7** range from good to very good for Calibration #1. **Table 6.7** shows very good correlation between the measured and modeled stage data for Calibration #2. The improvement from Calibration #1 to Calibration #2 is mostly due to the recovery leg of the hydrograph. It should be noted that stages recover faster in the Calibration #1 results compared to the measured data. The Calibration #2 stage results, however, are much more consistent with the measured data. This indicates the pump operation at gages SG17 and SG4 influences the recovery at the gage location.

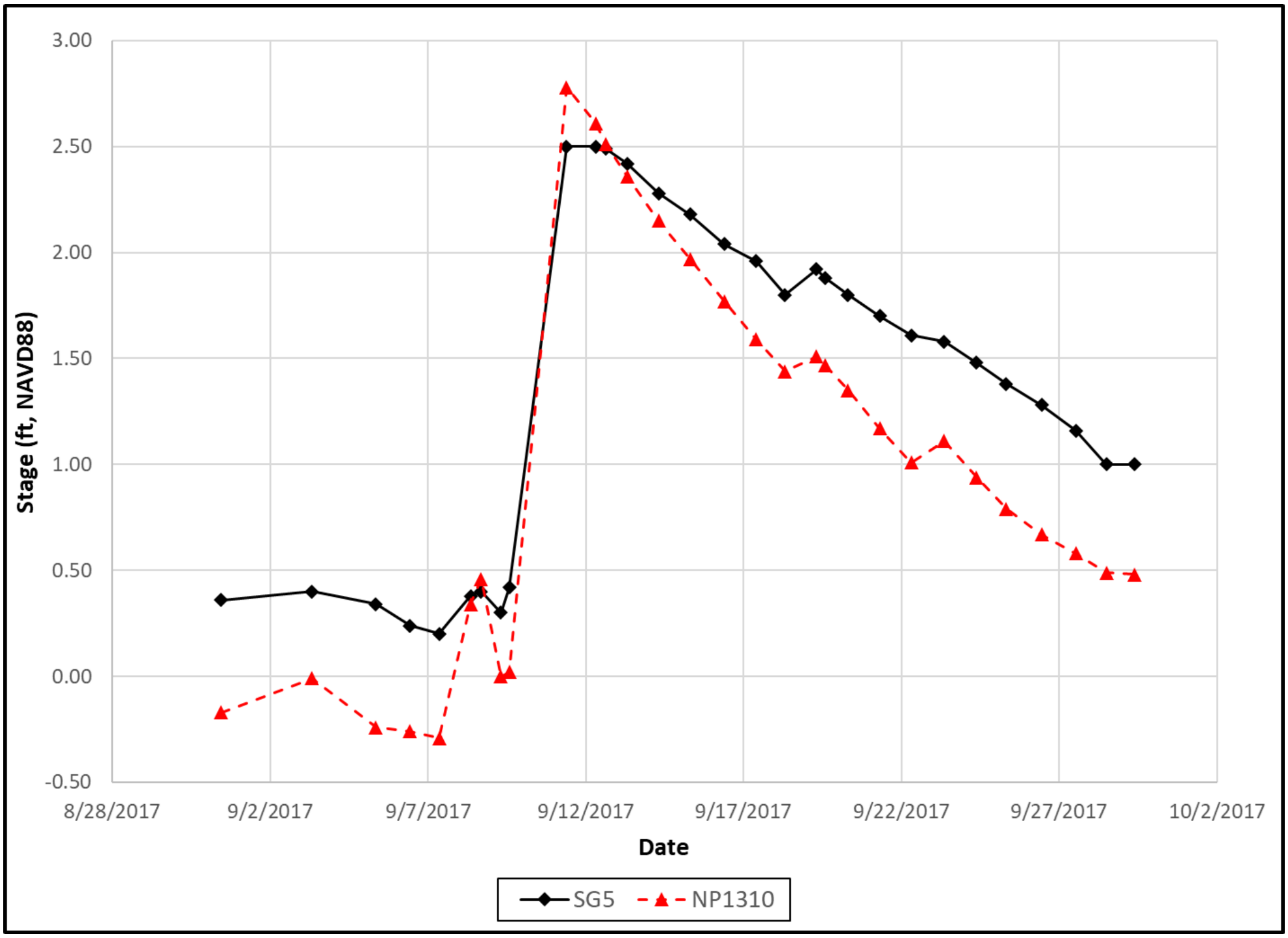


Figure 6.7: Gage SG5 Chase Hammock at Judson Rd. Calibration#1 Comparisons

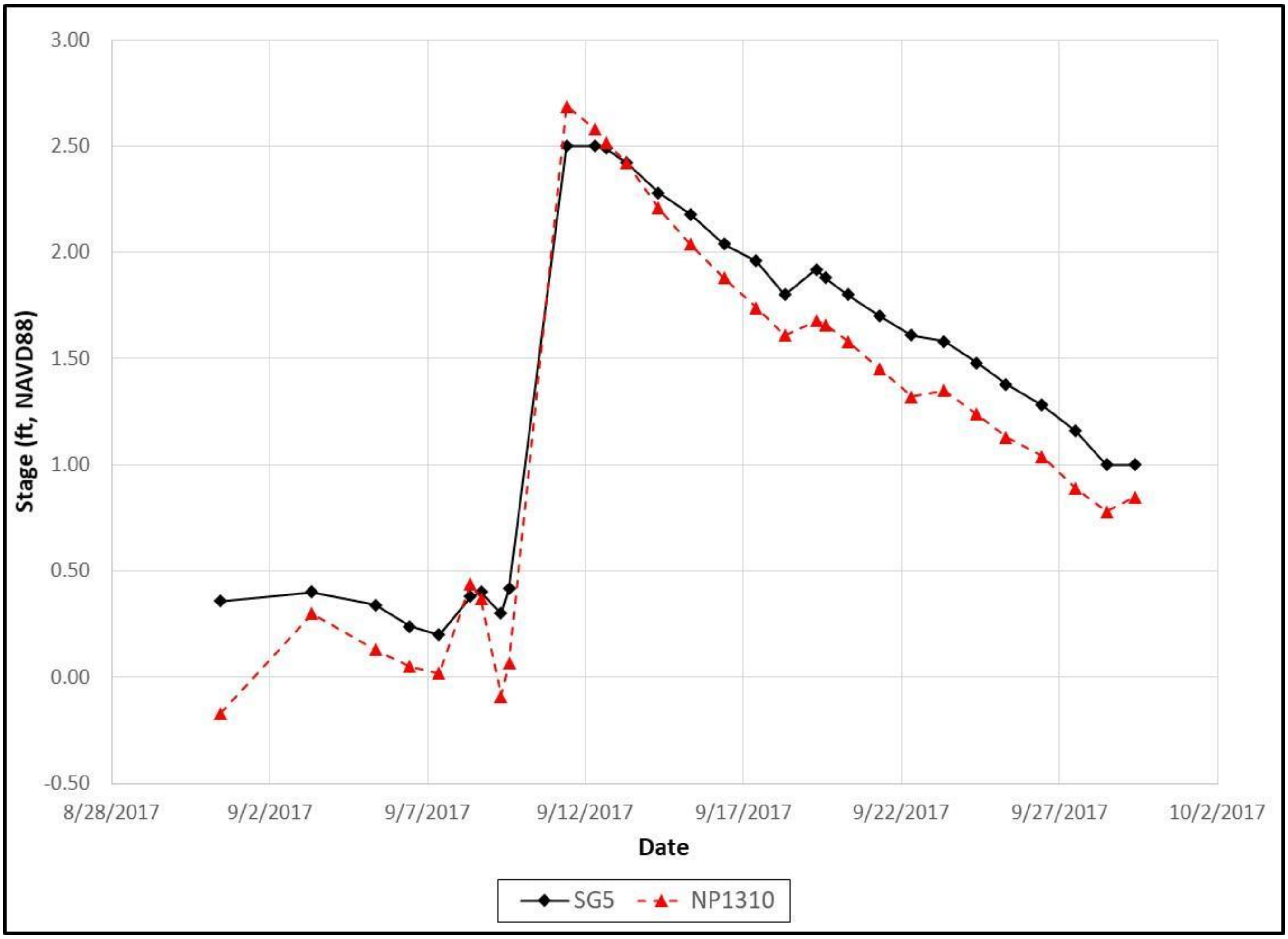


Figure 6.8: Gage SG5 Chase Hammock at Judson Rd. Calibration#2 Comparisons

Table 6.7: Calibration Statistical Metrics SG5

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.943	Very Good	0.977	Very Good
NSE	0.710	Good	0.916	Very Good
ME	0.347	Good	0.174	Very Good
MAE	0.378	Very Good	0.198	Very Good
RMSE	0.421	Very Good	0.226	Very Good
RSR	0.529	Good	0.285	Very Good
1/2 Standard Deviation Obs.	2	Good	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.6 Gage SG6 Crisafulli at Judson Rd. – Calibration Results

The SG6 gage is located just north of the Judson Rd. and E. Crisafulli Rd. intersection along the north-south drainage ditch. The stage hydrograph comparisons for Calibration #1 and Calibration #2 are provided in **Figure 6.9** and **Figure 6.10**, respectively. The Calibration #1 modeled peak stage is 2.88-ft (NAVD88), which is approximately 3.1-inches above the measured peak stage (2.62-ft, NAVD88). The Calibration #2 modeled peak stage is 2.83-ft, which is 2.5-inches above the measured peak stage.

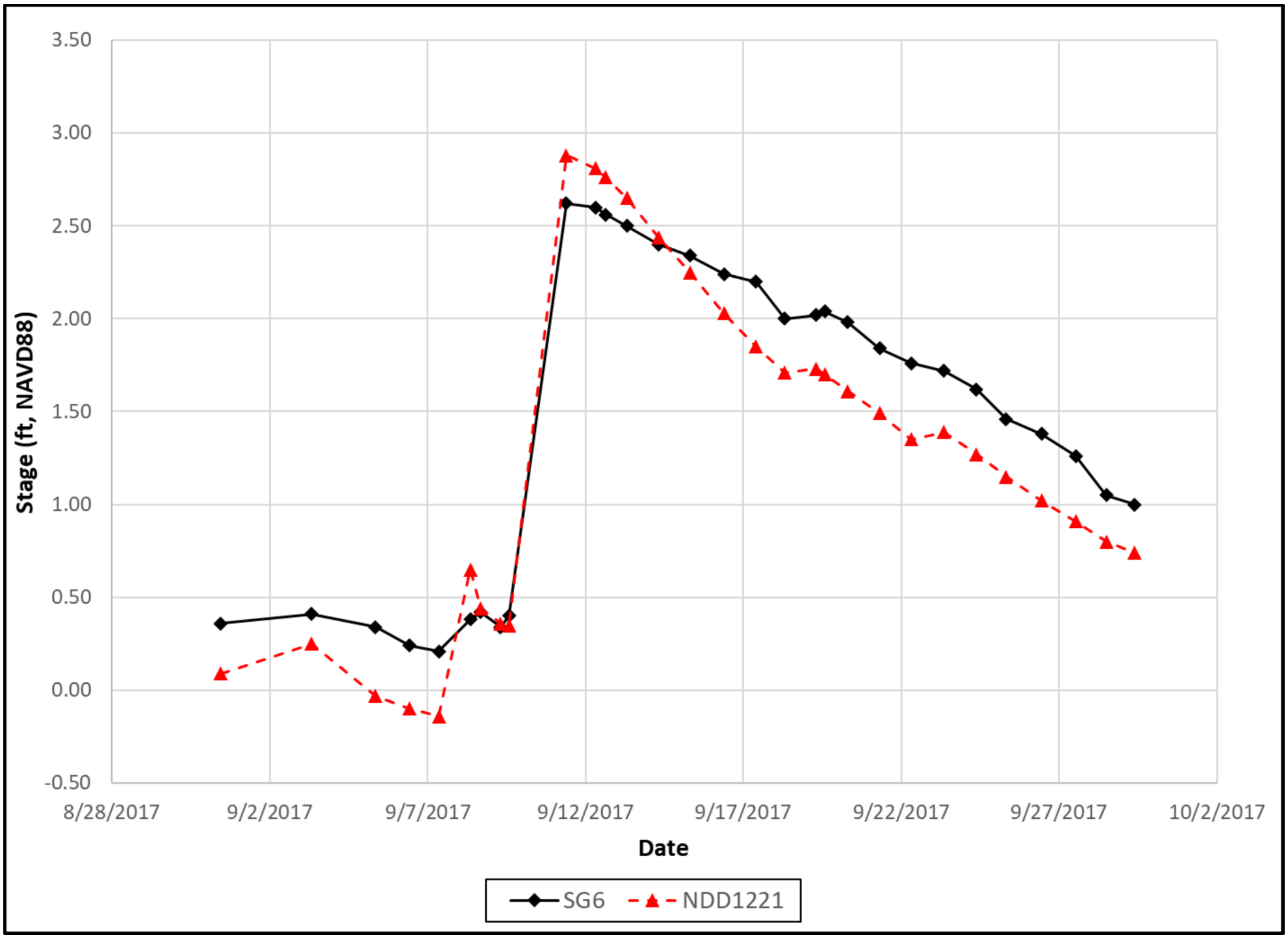


Figure 6.9: Gage SG6 Crisafulli at Judson Rd. Calibration#1 Comparisons

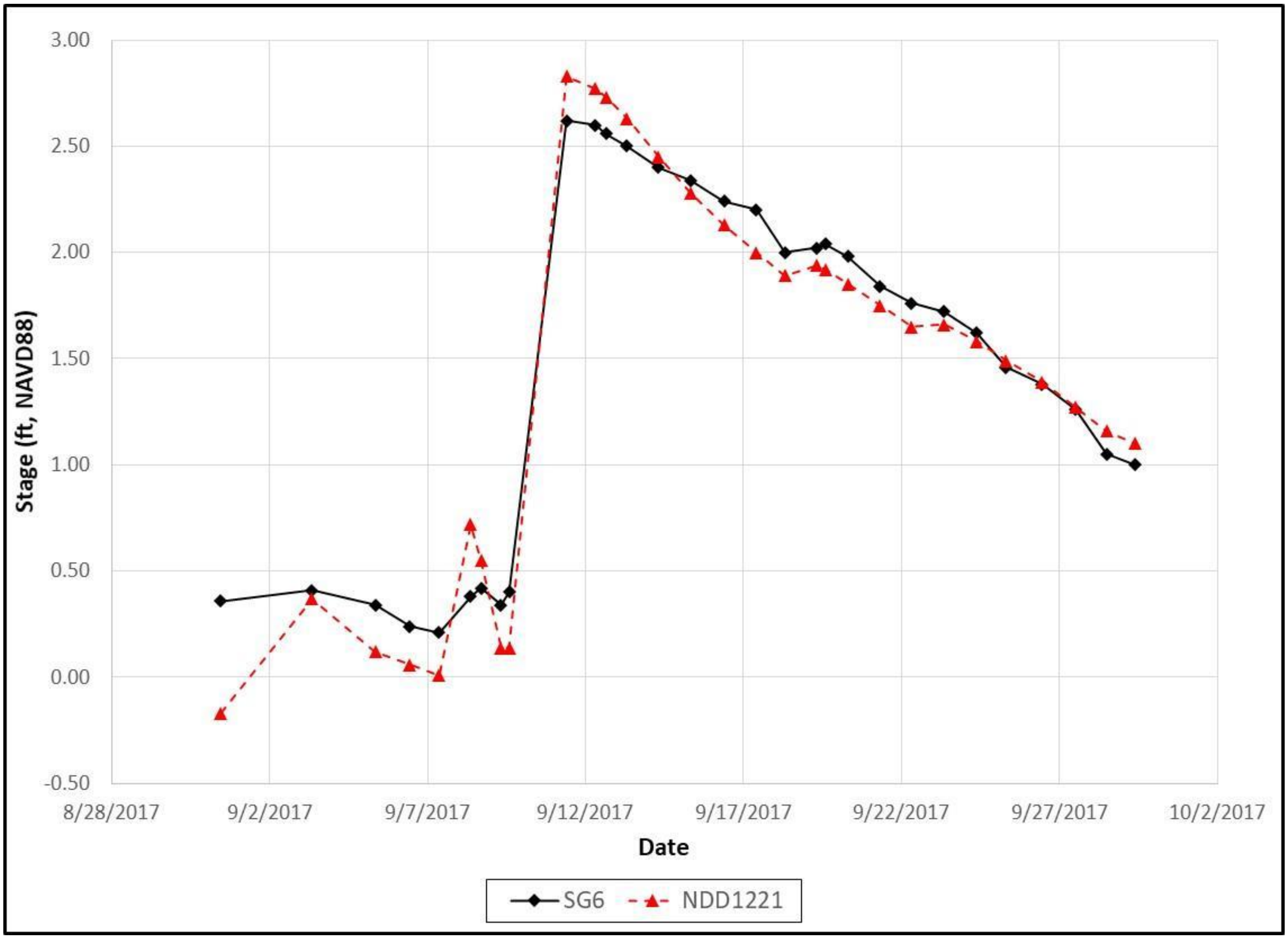


Figure 6.10: Gage SG6 Crisafulli at Judson Rd. Calibration#2 Comparisons

While the statistical metrics for this gage provided in **Table. 6.8** are all classified as very good, the table shows that the SG6 Calibration #2 simulation tends to compare better to the measured stages. Similar to SG5 comparisons, the improvement from Calibration #1 to Calibration #2 is also related to the recovery leg of the stage hydrograph. As shown, the Calibration #2 stage results are much more consistent to the measured data for the recovery leg of the hydrograph after the peak of the storm event. This also indicates the pump operation at gages SG17 and SG4 influences the recovery at this the gage location.

Table 6.8: Calibration Statistical Metrics SG6

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.946	Very Good	0.969	Very Good
NSE	0.889	Very Good	0.956	Very Good
ME	0.176	Very Good	0.043	Very Good
MAE	0.254	Very Good	0.140	Very Good
RMSE	0.278	Very Good	0.175	Very Good
RSR	0.327	Very Good	0.206	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.7 Gage SG7 East Crisafulli at Joseph Ct. – Calibration Results

The SG7 gage is located west of the Joseph Ct. and E. Crisafulli Rd. intersection. The stage hydrograph comparisons for Calibration #1 and Calibration #2 are provided in **Figure 6.11** and **Figure 6.12**, respectively. The Calibration #1 modeled peak stage is 2.94-ft (NAVD88), which is approximately 3.4-inches above the measured peak stage (2.66-ft, NAVD88). The Calibration #2 modeled peak stage is 2.89-ft, which is 2.8-inches above the measured peak stage.

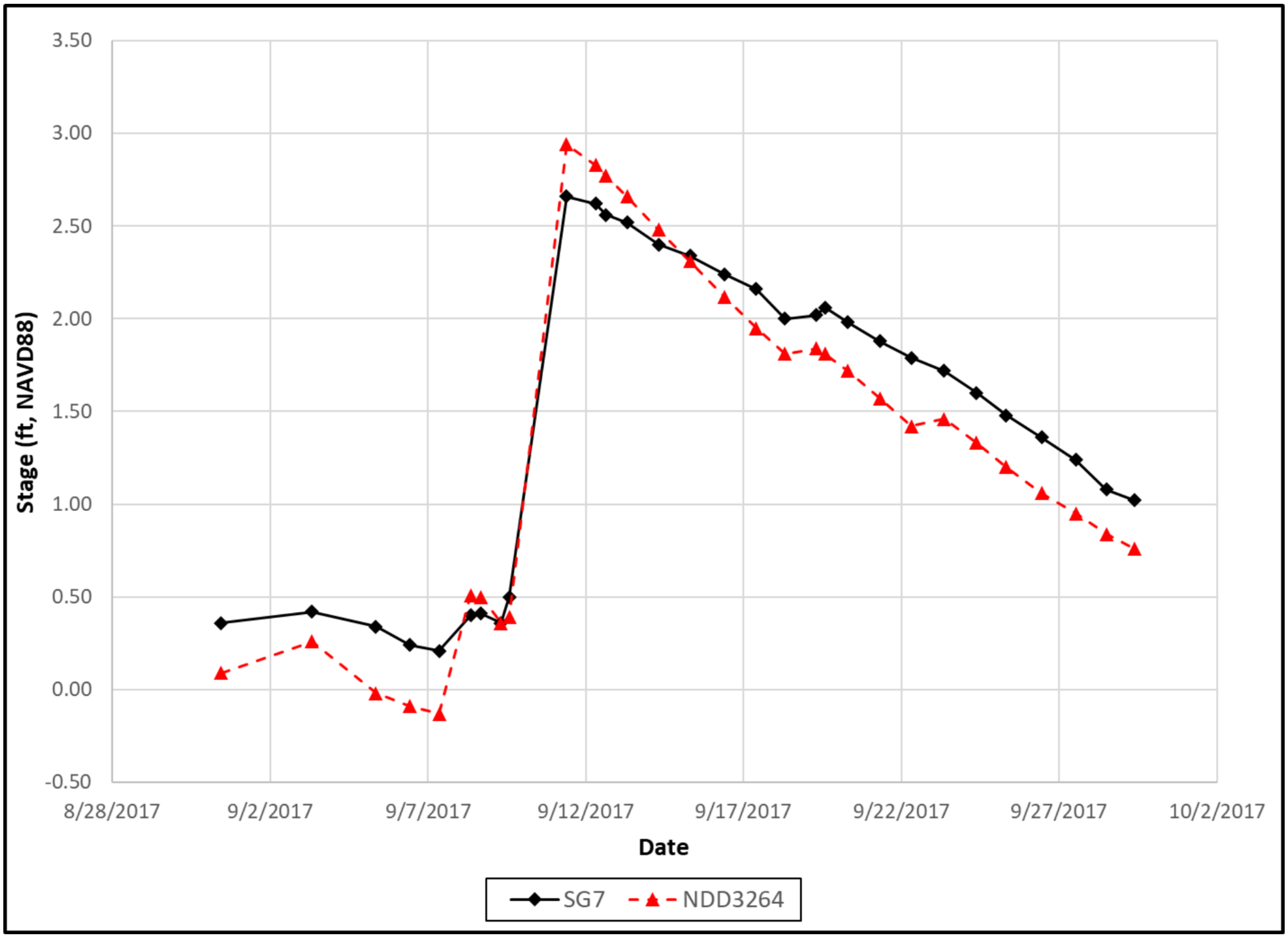


Figure 6.11: Gage SG7 East Crisafulli at Joseph Ct. Calibration#1 Comparisons

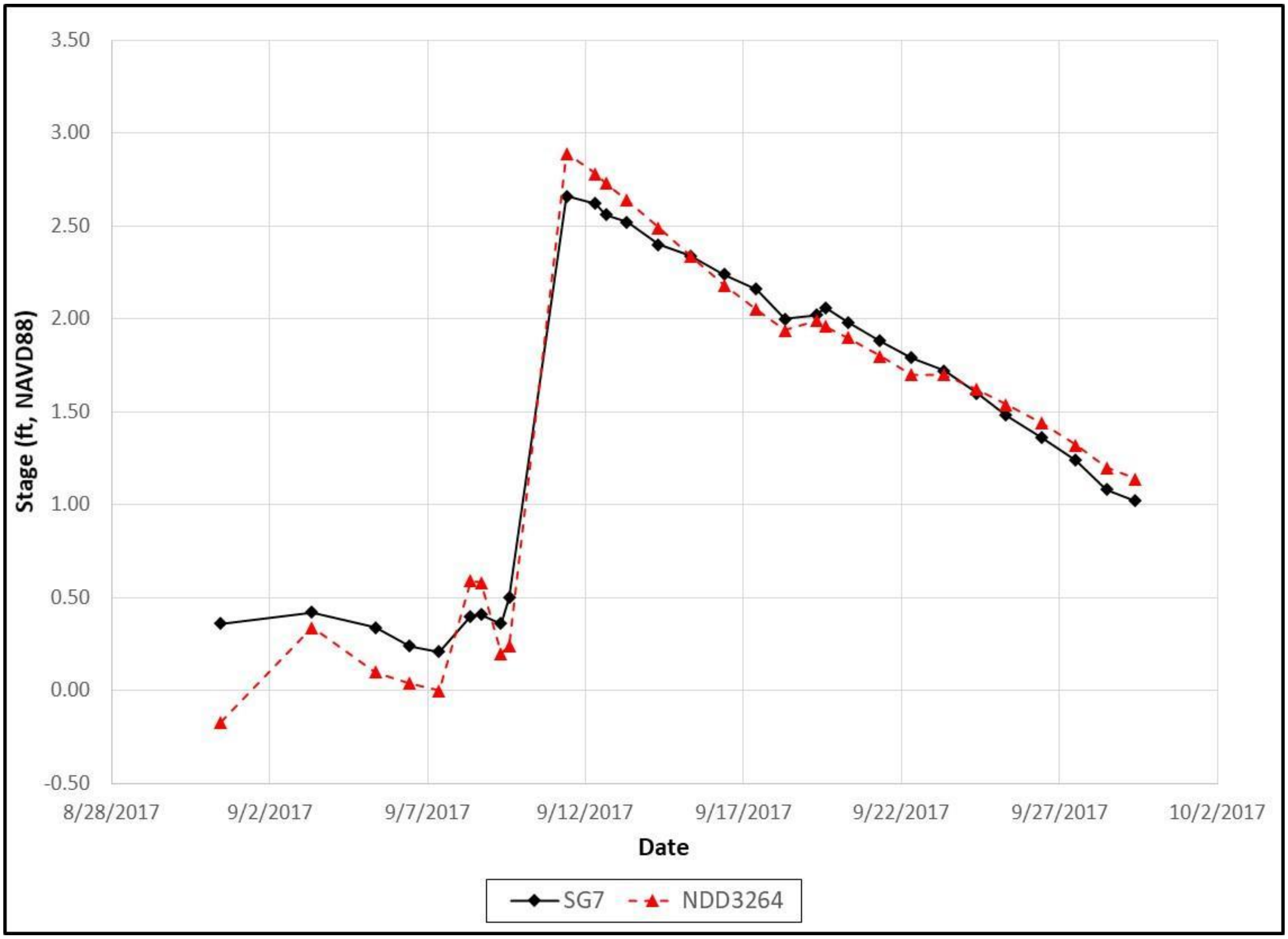


Figure 6.12: Gage SG7 East Crisafulli at Joseph Ct. Calibration#2 Comparisons

The statistical metrics are provided in **Table 6.9**. The statistical comparisons for both simulations show very good correlation between the measured and modeled data. There is only a slight difference between the two simulations with Calibration #2 being marginally better compared to Calibration #1.

Table 6.9: Calibration Statistical Metrics SG7

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.964	Very Good	0.975	Very Good
NSE	0.919	Very Good	0.961	Very Good
ME	0.142	Very Good	0.023	Very Good
MAE	0.217	Very Good	0.131	Very Good
RMSE	0.237	Very Good	0.165	Very Good
RSR	0.280	Very Good	0.194	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.8 Gage SG8 N Courtenay at Pine Island – Calibration Results

Gage SG8 is located south of the Judson Rd. and N. Courtney Parkway intersection. The stage hydrograph comparisons for Calibration #1 and Calibration #2 are provided below in **Figure 6.13** and **Figure 6.14**, respectively. The Calibration #1 modeled peak stage is 2.85-ft (NAVD88), which is approximately 3.0-inches above the measured peak stage (2.60-ft, NAVD88). The Calibration #2 modeled peak stage is 2.80-ft, which is 2.4-inches above the measured peak stage.

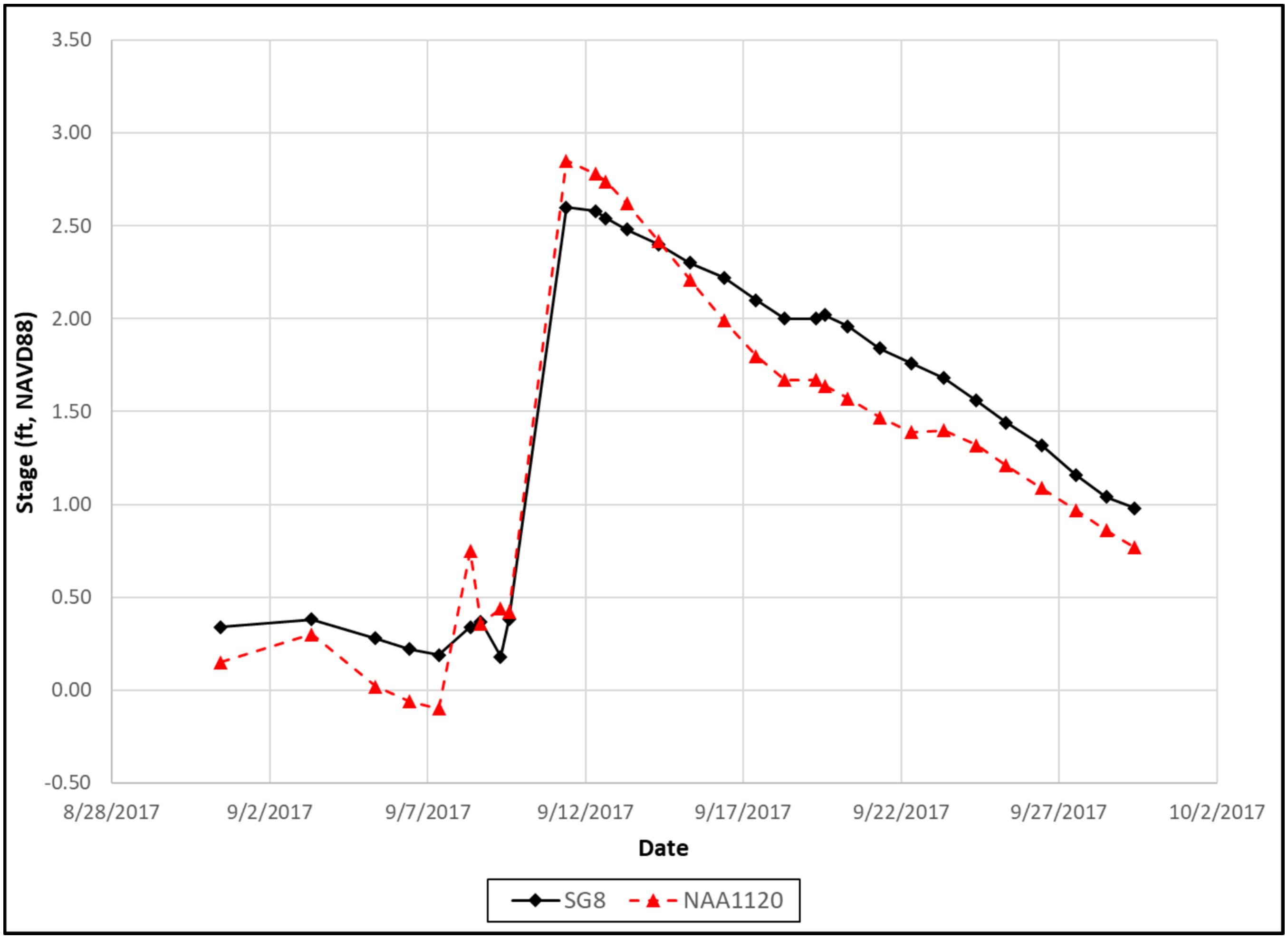


Figure 6.13: Gage SG8 N Courtenay at Pine Island Calibration#1 Comparisons

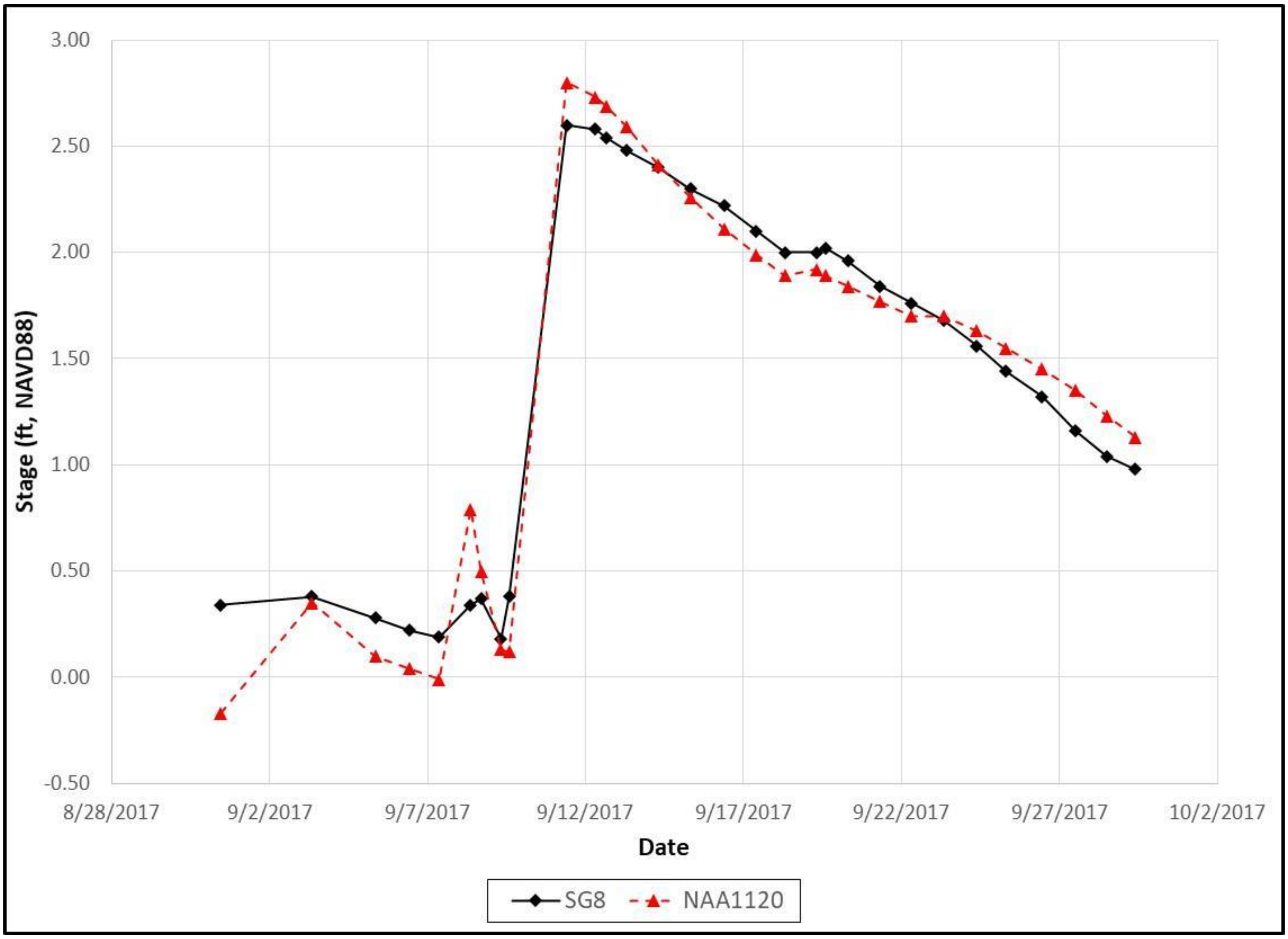


Figure 6.14: Gage SG8 N Courtenay at Pine Island Calibration#2 Comparisons

The statistical metrics are provided in **Table 6.10**. The statistical comparisons for both simulations show very good correlation between the measured and modeled data. There is only a slight difference between the two simulations with Calibration #2 being marginally better compared to Calibration #1.

Table 6.10: Calibration Statistical Metrics SG8

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.937	Very Good	0.962	Very Good
NSE	0.908	Very Good	0.955	Very Good
ME	0.131	Very Good	0.006	Very Good
MAE	0.233	Very Good	0.143	Very Good
RMSE	0.256	Very Good	0.179	Very Good
RSR	0.298	Very Good	0.209	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.9 Gage SG9 Pine Island 1 Mile North of North Courtenay – Calibration Results

Gage SG9 is located along Pine Island Rd. about 1 mile north of North Courtney Parkway. The stage hydrograph comparisons for Calibration #1 and Calibration #2 are provided below in **Figure 6.15** and **Figure 6.16**, respectively. The Calibration #1 modeled peak stage is 2.73-ft (NAVD88), which is 3-inches above the measured peak stage (2.48-ft, NAVD88). The peak stage for Calibration #2 is 2.66-ft (NAVD88) which is 2.2-inches above the measured peak stage.

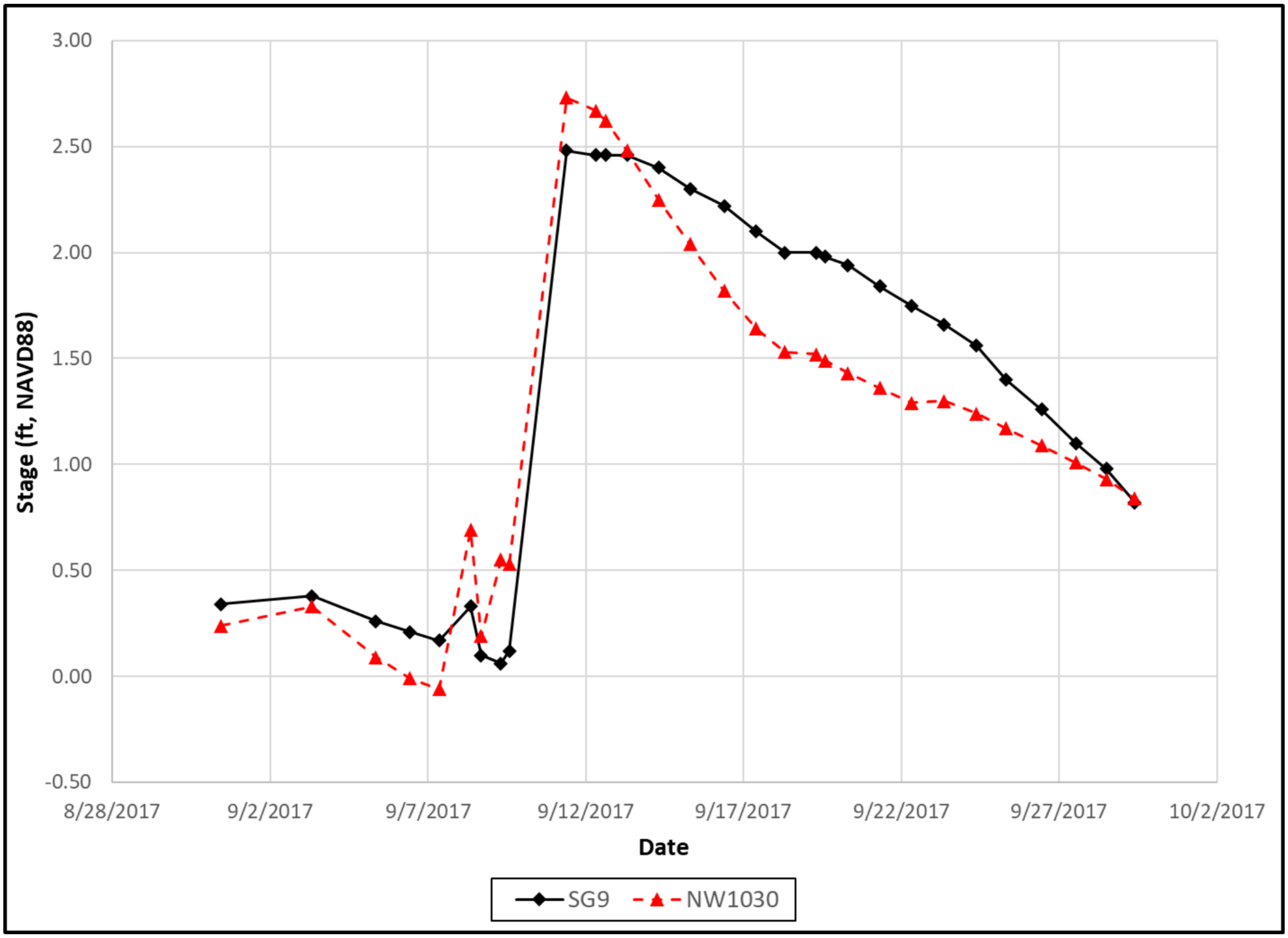


Figure 6.15: Gage SG9 Pine Island 1 Mile North of North Courtenay Calibration#1 Comparisons

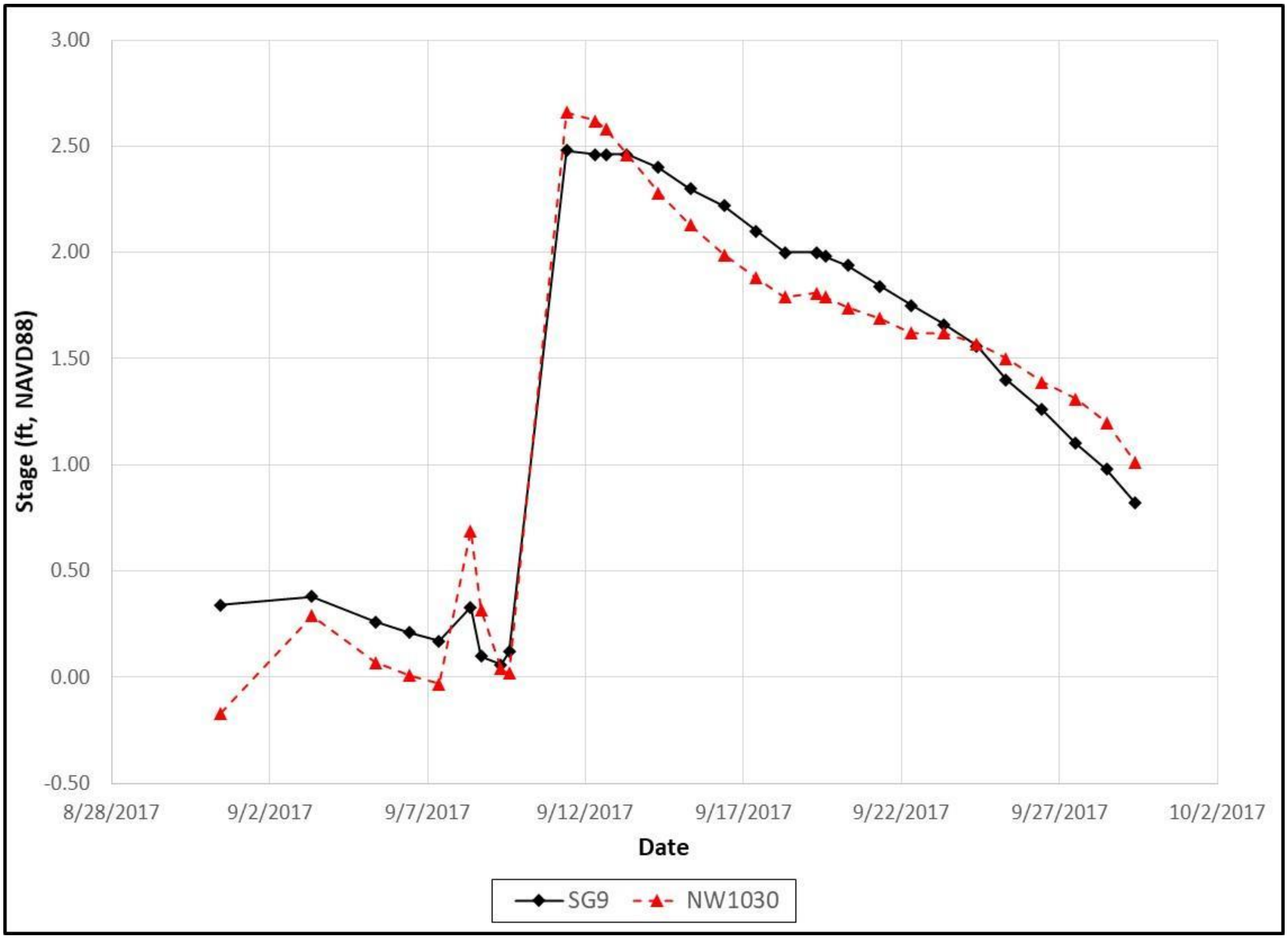


Figure 6.16: Gage SG9 Pine Island 1 Mile North of North Courtenay Calibration#2 Comparisons

The statistical metrics are provided in **Table 6.11**. The statistical comparisons for both simulations show very good correlation between the measured and modeled data. Calibration #2 tends to compare better because the recovery leg of the stage hydrograph is more consistent with measured data for that simulation.

Table 6.11: Calibration Statistical Metrics SG9

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.893	Very Good	0.953	Very Good
NSE	0.867	Very Good	0.949	Very Good
ME	0.138	Very Good	0.042	Very Good
MAE	0.272	Very Good	0.169	Very Good
RMSE	0.316	Very Good	0.195	Very Good
RSR	0.359	Very Good	0.221	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.10 Gage SG10 Pine Island Harvey Grove Pump – Calibration Results

Gage SG10 is located about 3,000-ft east of the Pine Island Grove Pumps along Pine Island Rd. The stage hydrograph comparisons for Calibration #1 and Calibration #2 are provided below in **Figure 6.17** and **Figure 6.18**, respectively. The Calibration #1 modeled peak stage is 2.44-ft (NAVD88), which is approximately 3.6-inches above the measured peak stage (2.14-ft, NAVD88). The peak stage for Calibration #2 is 2.29-ft (NAVD88), which is 1.8-inches above the measured peak stage. While the modeled peak stage estimates are comparable for both calibration simulations, the recovery legs of the stage hydrograph are quite different.

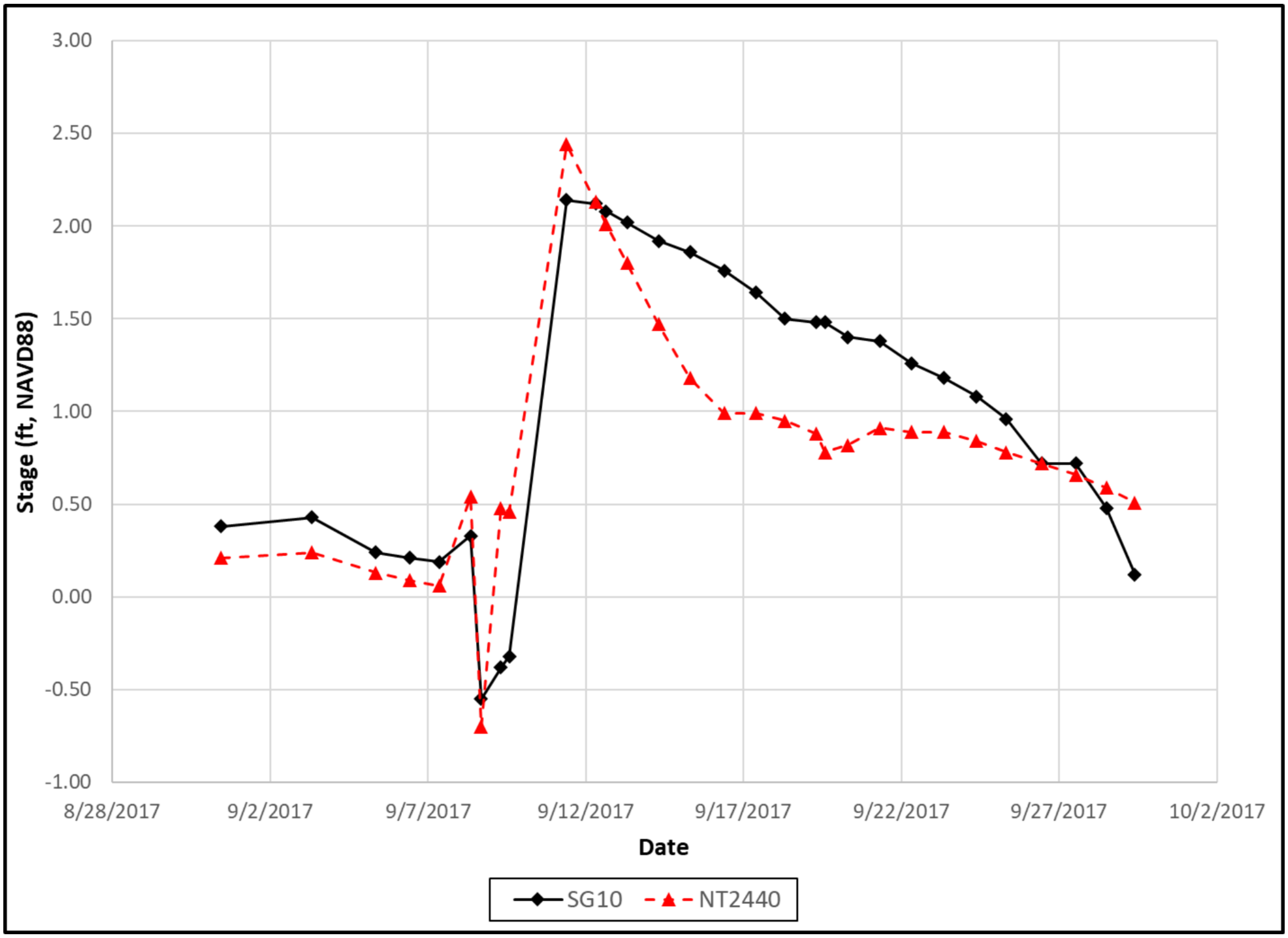


Figure 6.17: Gage SG10 Pine Island Harvey Grove Pump Calibration#1 Comparisons

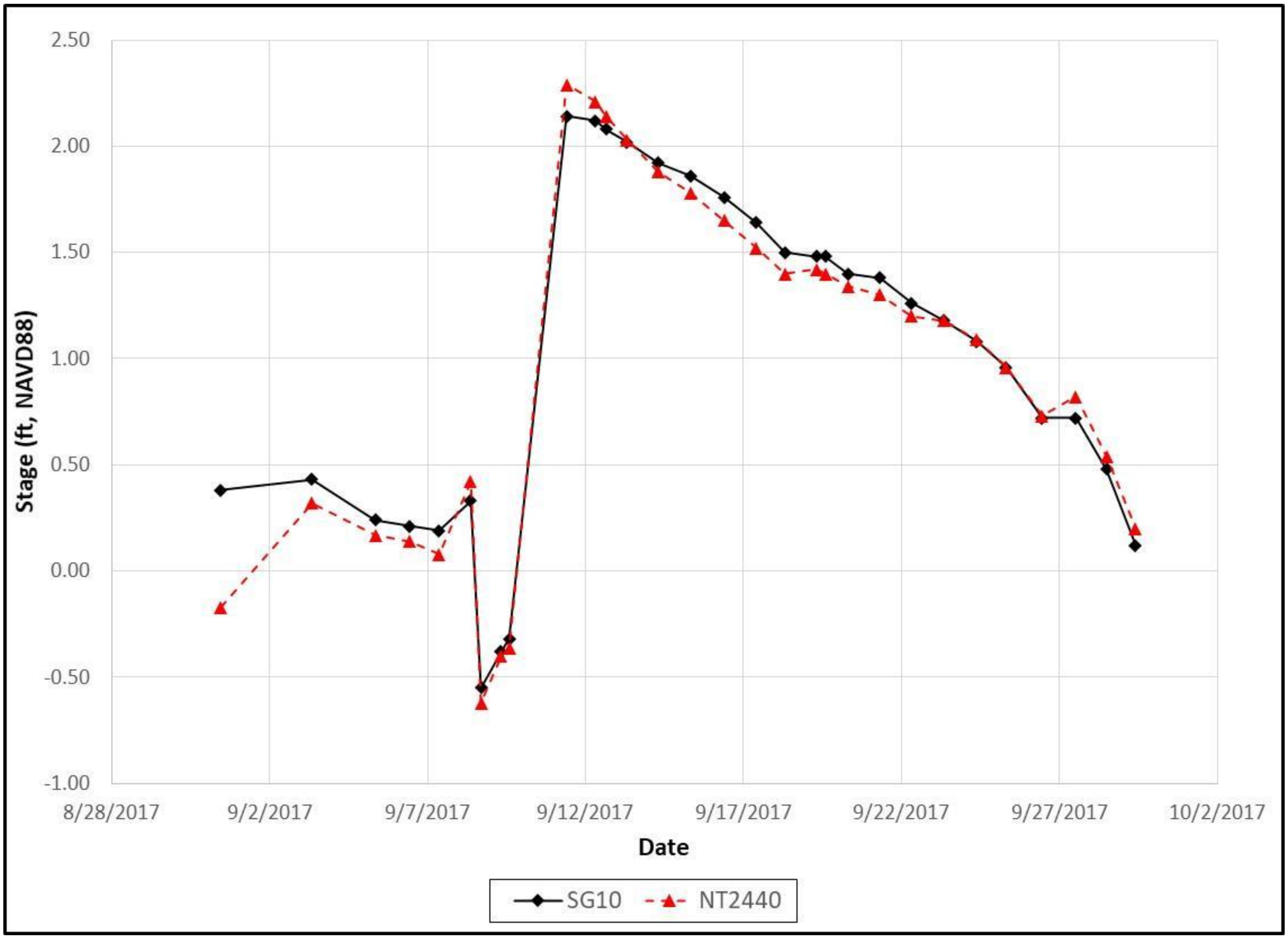


Figure 6.18: Gage SG10 Pine Island Harvey Grove Pump Calibration#2 Comparisons

The statistical metrics are provided in **Table 6.12**. While the statistical metrics for the Calibration #1 simulation are acceptable ranging from good to very good, the Calibration #2 results have very good correlation between the measured and modeled data. This indicates that the stage recovery at this gage is noticeably affected by the pump operations at Pine Island.

Table 6.12: Calibration Statistical Metrics SG10

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.753	Good	0.979	Very Good
NSE	0.704	Good	0.975	Very Good
ME	0.170	Very Good	0.039	Very Good
MAE	0.347	Very Good	0.083	Very Good
RMSE	0.430	Very Good	0.126	Very Good
RSR	0.535	Good	0.156	Very Good
1/2 Standard Deviation Obs.	2	Good	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.11 Gage SG11 Pine Island West – Calibration Results

Gage SG11 is located about 580-ft east of the Pine Island Grove Pumps along Pine Island Rd. The stage hydrograph comparisons for Calibration #1 and Calibration #2 are provided below in **Figure 6.19** and **Figure 6.20**, respectively. Like the comparisons for SG10, the modeled and measured peak stages compare quite well. The Calibration #1 modeled peak stage is 2.11-ft (NAVD88), which is approximately 0.1-inches above the measured peak stage (2.10-ft, NAVD88). The peak stage for Calibration #2 is 2.02-ft (NAVD88), which is ~1.0-inch below the measured peak stage. Additionally, the recovery legs of the stage hydrographs are quite different between the two simulations. Notice that Calibration #2 is much more consistent with the measure data. This indicates that the stage recovery at SG11 is sensitive to the pumping rates at Pine Island. Note that node NPI1030 was converted to a time-stage node for the Calibration #2 analysis. The comparisons for Calibration #2 were included in this section to show how the stages upstream of the gage are fairly consistent with stages at the pump location (SG17 at node NPI1030).

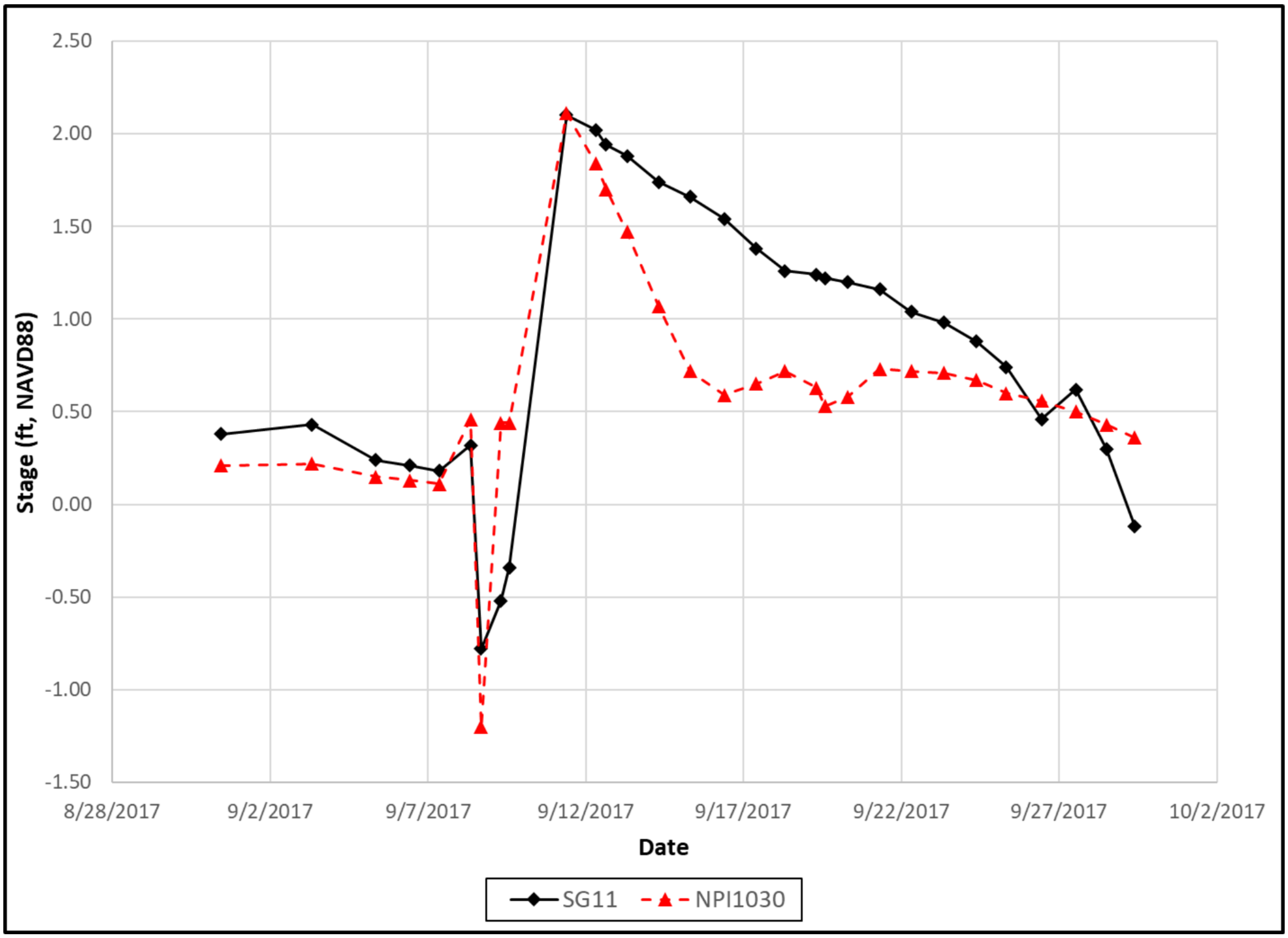


Figure 6.19: Gage SG11 Pine Island West Calibration#1 Comparisons

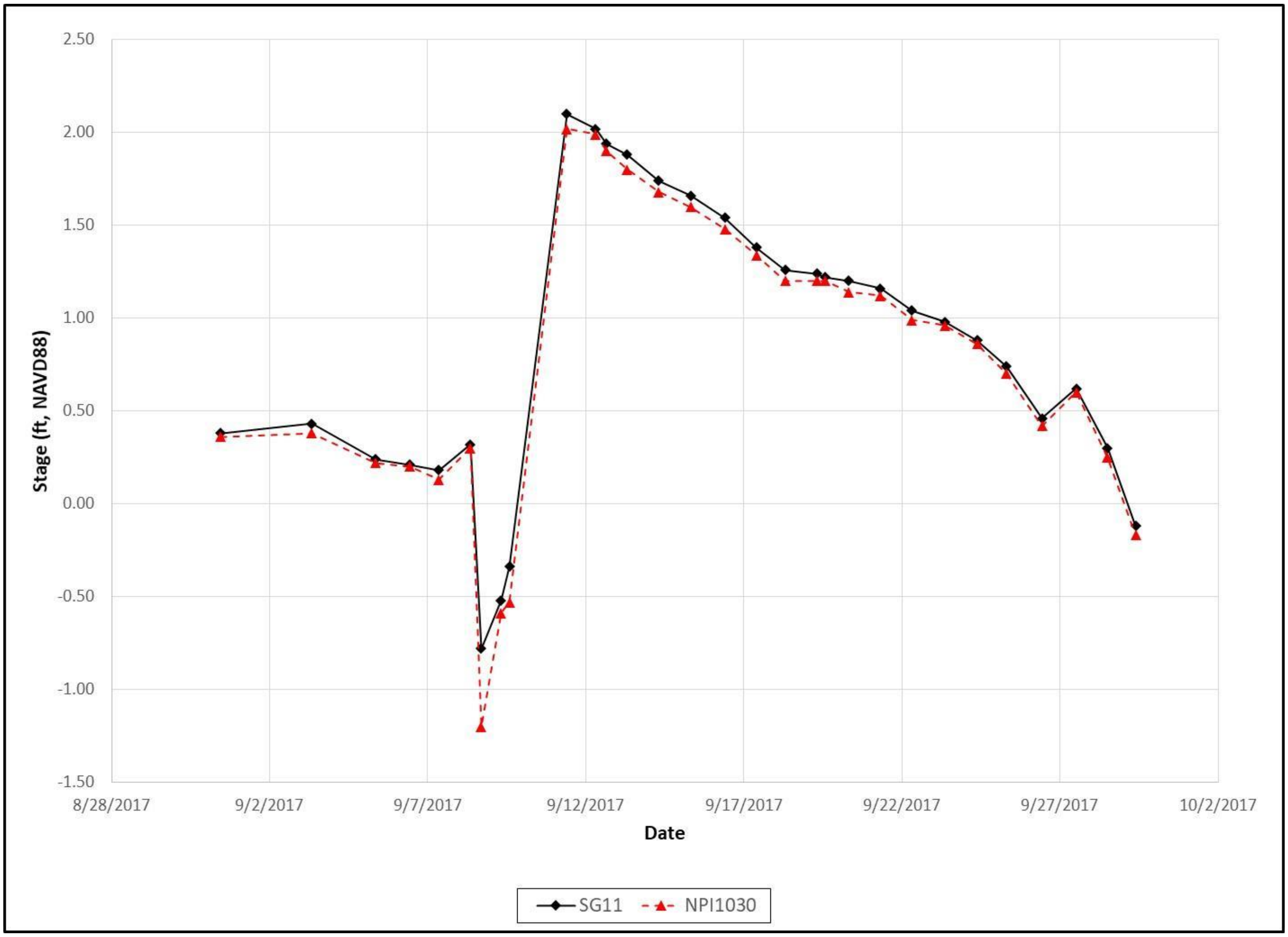


Figure 6.20: Gage SG11 Pine Island West Calibration#2 Comparisons

This is even more evident when looking at the statistical metrics in **Table 6.13.** Specifically, Calibration #2 has very good correlation between the measured and modeled stages while Calibration #1 has four parameters that are classified as satisfactory. This comparison demonstrates how sensitive the upstream stages are to the pumping operations at Pine Island.

Table 6.13: Calibration Statistical Metrics SG11

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.678	Satisfactory	0.993	Very Good
NSE	0.595	Satisfactory	0.984	Very Good
ME	0.217	Very Good	0.060	Very Good
MAE	0.390	Very Good	0.060	Very Good
RMSE	0.485	Very Good	0.096	Very Good
RSR	0.626	Satisfactory	0.123	Very Good
1/2 Standard Deviation Obs.	1	Satisfactory	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.12 Gage SG12 PICA South – Calibration Results

Gage SG12 is located at the downstream side of the Pine Island Pump House. The stage hydrograph comparison for Calibration #1 is provided below in **Figure 6.21.** Like the comparisons for SG11, the modeled and measured peak stages compare quite well. The Calibration #1 modeled peak stage is 2.11-ft (NAVD88), which is 0.6-inches above the measured peak stage (2.06-ft, NAVD88). Like SG11, the stage hydrograph recovers faster than the measured stage data after the peak. This is also attributed to the unknown pumping operations at the Pine Island Harvey Grove Pumps. No comparisons were conducted for Calibration #2 since node NPI1030 was converted to a time-stage node for that analysis.

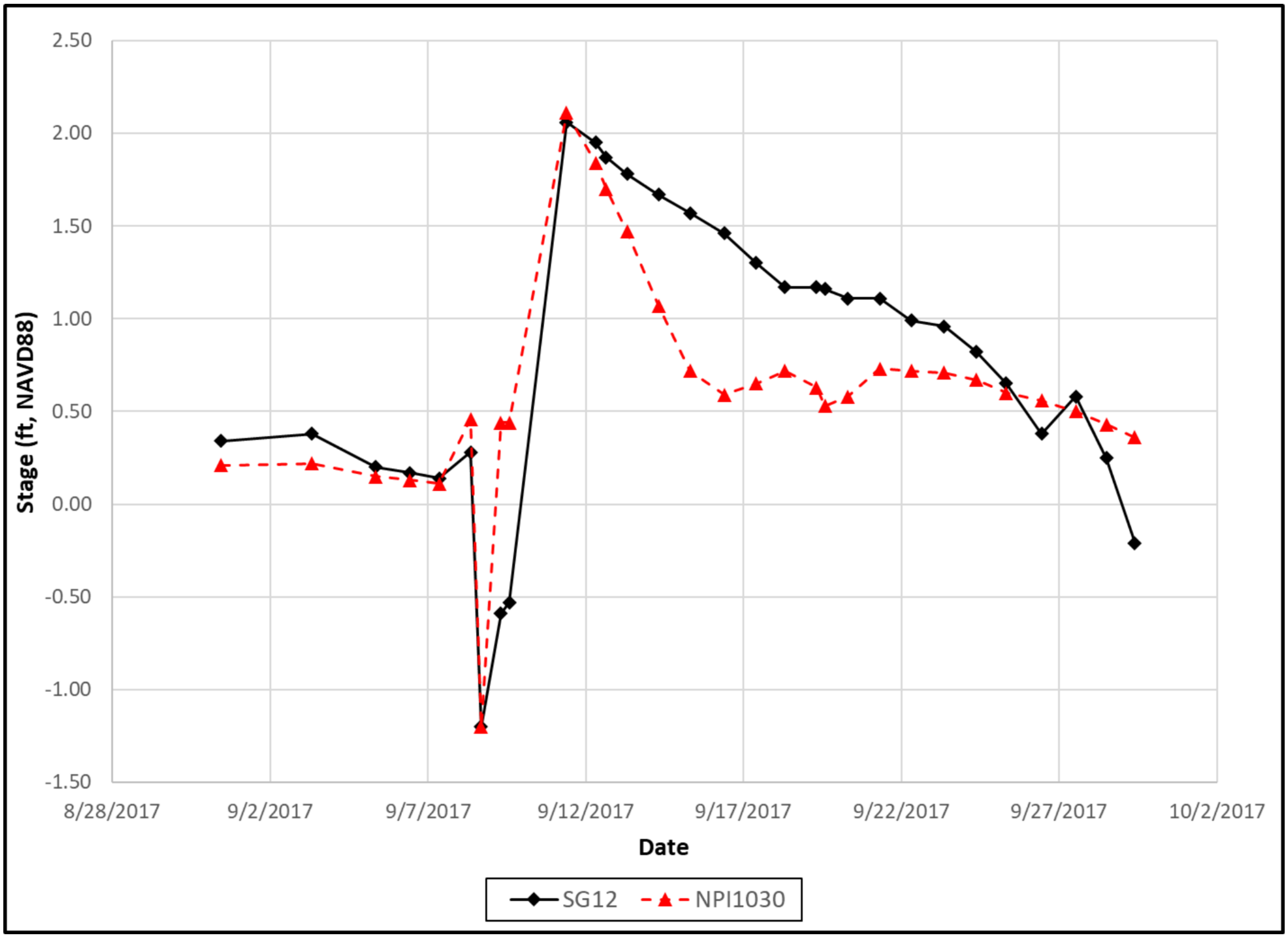


Figure 6.21: Gage SG12 PICA South Calibration #1 Comparisons

The statistical metrics shown in **Table 6.14** show that the model results are acceptable when compared to the measured data.

Table 6.14: Calibration Statistical Metrics SG12

Metric Parameter	Calibration Simulation #1	Quality #1
R^2	0.701	Satisfactory
NSE	0.662	Satisfactory
ME	0.138	Very Good
MAE	0.349	Very Good
RMSE	0.460	Very Good
RSR	0.572	Good
1/2 Standard Deviation Obs.	2	Good

Note: Number of pair data (observed and simulated) = 30

6.3.13 Gage SG13 W Hall Rd. West at N. Tropical Trail – Calibration Results

Gage SG13 is located at the southeast corner of the W. Hall Rd. and N. Tropical Trail intersection. The stage hydrograph comparisons for Calibration #1 and Calibration #2 are provided below in **Figure 6.22** and **Figure 6.23**, respectively. The statistical metrics are provided in **Table 6.15**.

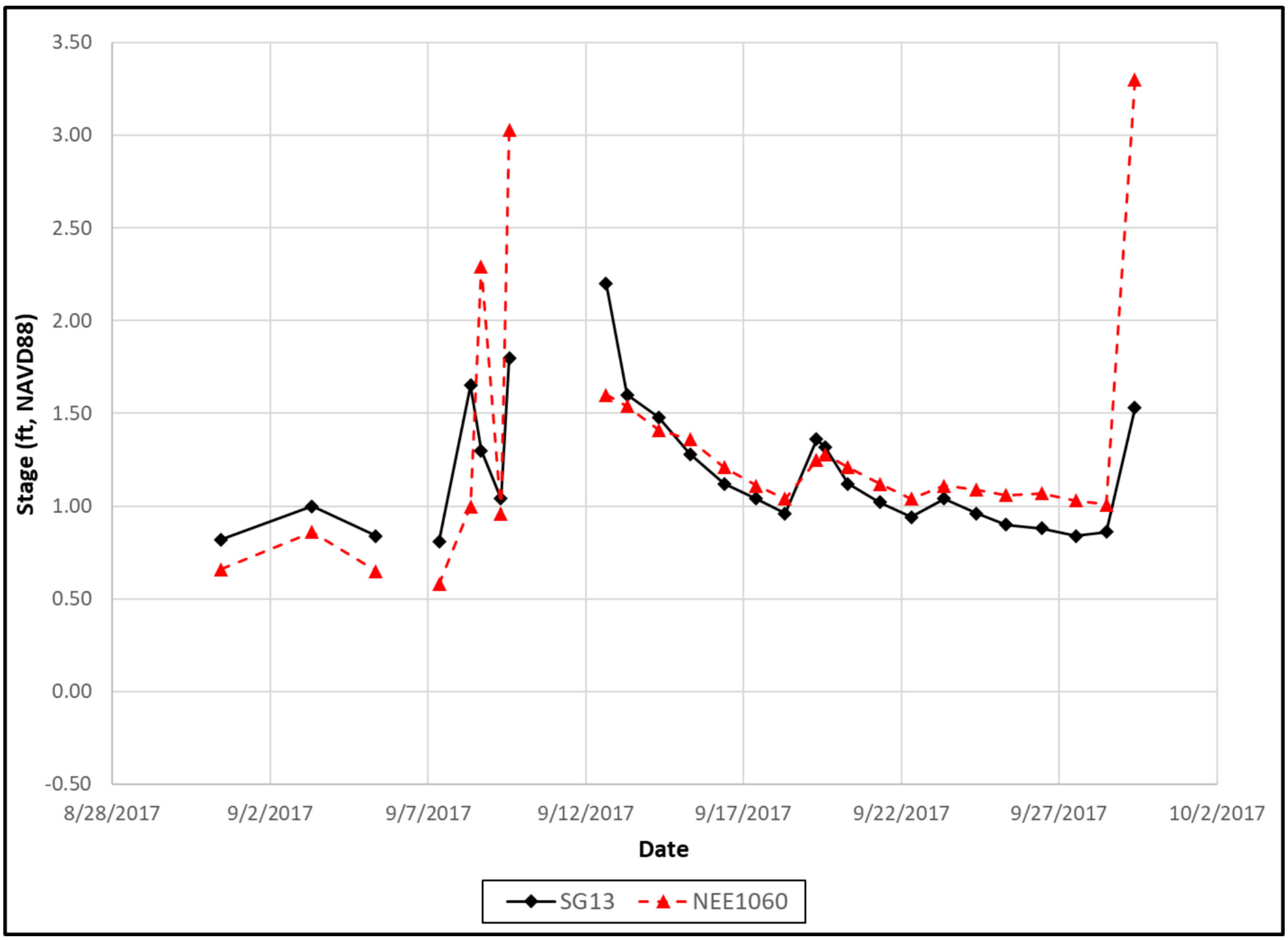


Figure 6.22: Gage SG13 W Hall Rd. West at N. Tropical Trail Calibration #1 Comparisons

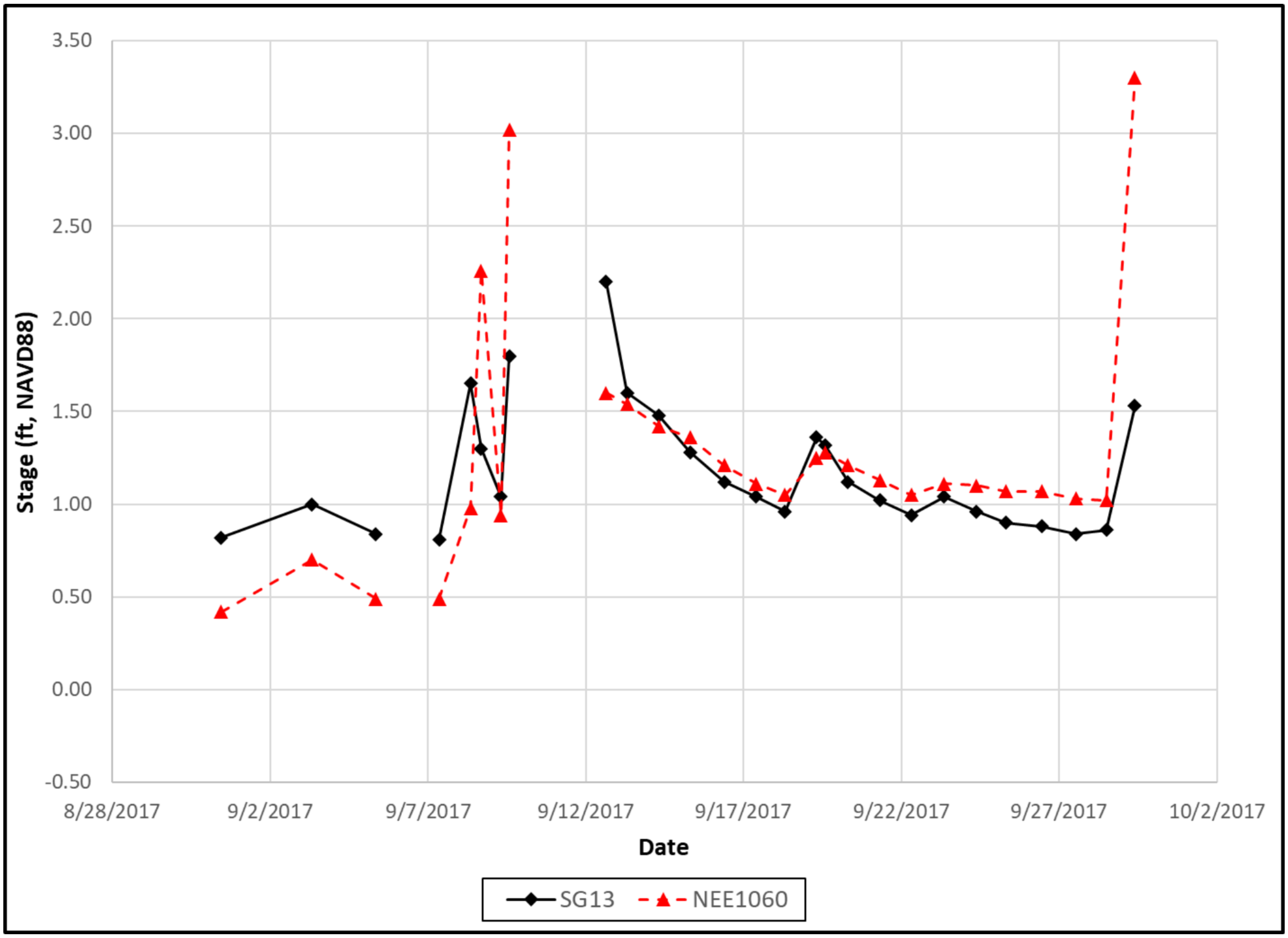


Figure 6.23: Gage SG13 W Hall Rd. West at N. Tropical Trail Calibration #2 Comparisons

As seen in the figures above, the model results for both simulations are nearly identical. Additionally, the model tends to agree well with the lower recorded stages for both simulations. Conversely, the model tends to overpredict maximum conditions. Three of the statistical metrics are not satisfactory based on the study criteria. However, four of the statistical metrics were in acceptable ranges with three being very good.

One important item to note is that County staff estimated peak stages at this location during Hurricane Irma to be approximately 6.2-ft (NAVD88) based on field observations. However, this elevation was not recorded in the gage logs. Based on the model results, the maximum overall stage simulated during Hurricane Irma occurred approximately on Sept 9, 2017, at 2:45 am was 5.93-ft which shows the model is within 3.2-inches of the estimated stage based on the field observations for Calibration #1. Though this calibration report was intended to compare with surveyed flood levels, comparison with the field observations does indicate that the model provides a fairly good estimate for the overall peak stage.

Several calibration iterations were conducted to improve the comparisons at this location, but all had little impact. Ideally, a new survey would be conducted to verify the pipe dimensions, materials, inverts, and site conditions. However, the drainage at this location was improved (RefDoc NMI_221_CP) after the 2017 calibration/verification events and prior to the start of this study. Given that the hydraulic network has been upgraded since 2017, no further changes were made to the model to calibrate to this gage.

Table 6.15: Calibration Statistical Metrics SG13

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.394	**Not Satisfactory**	0.402	**Not Satisfactory**
NSE	-1.144	**Not Satisfactory**	-1.239	**Not Satisfactory**
ME	-0.117	Very Good	-0.093	Very Good
MAE	0.290	Very Good	0.316	Very Good
RMSE	0.501	Very Good	0.512	Very Good
RSR	1.437	**Not Satisfactory**	1.468	**Not Satisfactory**
1/2 Standard Deviation Obs.	1	Satisfactory	1	Satisfactory

Note: Number of pair data (observed and simulated) = 27

6.3.14 Gage SG17 PICA Basin – Calibration Results

Gage SG17 is located on the east side of the access road to the Pine Island Harvey Grove Pumps. The stage hydrograph comparisons for Calibration #1 are provided below in **Figure 6.24**. Like the comparisons for SG11 and SG12, the modeled and measured peak stages compare quite well. The Calibration #1 modeled peak stage is 2.11-ft (NAVD88), which is approximately 1.1-inches above the measured peak stage (2.02-ft, NAVD88). However, the model's stage recovery tends to be much faster than what was measured in the field. As previously stated, the actual pump operation data during the calibration period was unavailable. Consequently, the modeled pumping rate at this location was based on the pump station's design plans. However, stage hydrograph comparisons indicate that the actual pump operation during the calibration period were quite different than the designed pumping rates. Specifically, the pumping rate appears to be much lower during the calibration storm event. This shows that the pumping operation at Pine Island influences the recovery of the drainage system upstream of the access road. Regardless of differences in the recovery leg, the statistical metrics (**Table 6.16**) range from satisfactory to very good, indicating that the model reasonably represents what physically occurred during Hurricane Irma.

No comparisons were conducted for this particular gage for Calibration #2 since this was a location in the model that was converted to a time-stage node for that analysis.

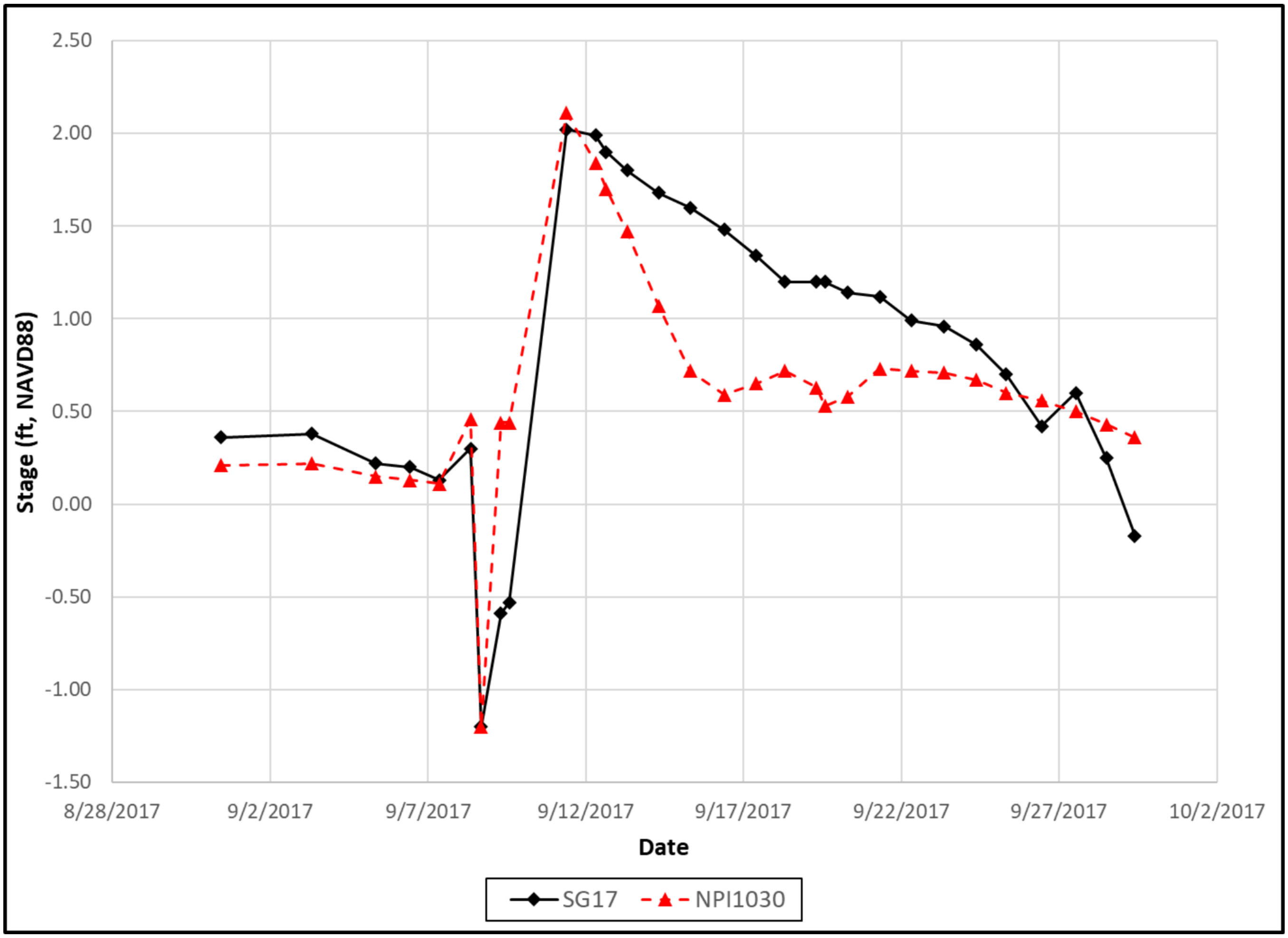


Figure 6.24: Gage SG17 PICA Basin Calibration #1 Comparisons

Table 6.16: Calibration Statistical Metrics SG17

Metric Parameter	Calibration Simulation #1	Quality #1
R^2	0.695	Satisfactory
NSE	0.648	Satisfactory
ME	0.157	Very Good
MAE	0.363	Very Good
RMSE	0.472	Very Good
RSR	0.584	Good
1/2 Standard Deviation Obs.	2	Good

Note: Number of pair data (observed and simulated) = 30

6.3.15 Gage SG18 PICA Riverside – Calibration Results

SG18 is located downstream of the Pine Island Harvey Grove Pumps access road. The stage hydrograph comparisons for Calibration #1 and Calibration #2 are provided below in **Figure 6.25** and **Figure 6.26**, respectively. The Calibration #1 modeled peak stage is 1.83-ft (NAVD88), which is approximately 0.8-inches below the measured peak stage (1.90-ft, NAVD88). The peak stage for Calibration #2 is 1.88-ft (NAVD88), which is 0.08-inches below the measured peak stage.

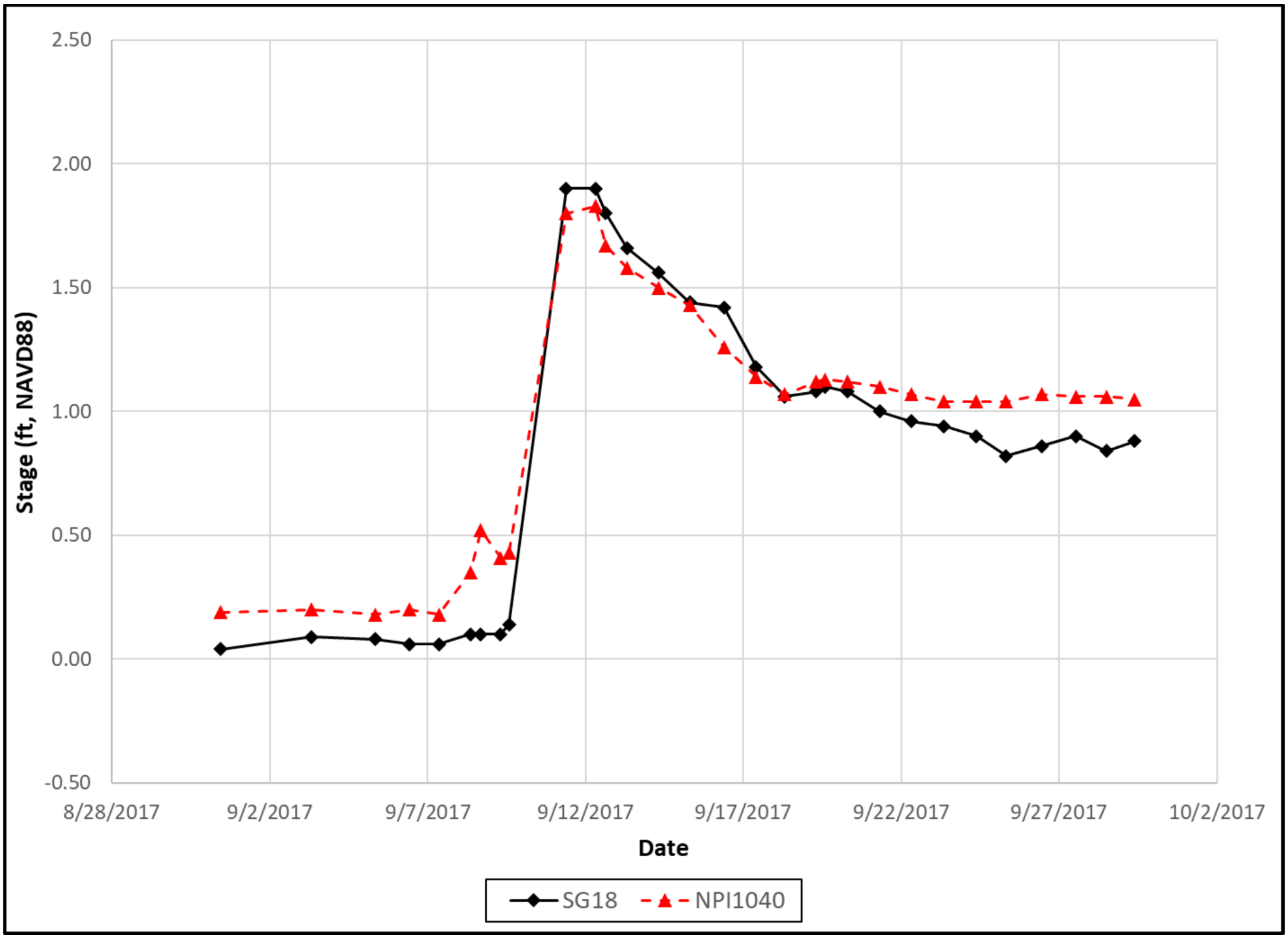


Figure 6.25: Gage SG18 PICA Riverside Calibration #1 Comparisons

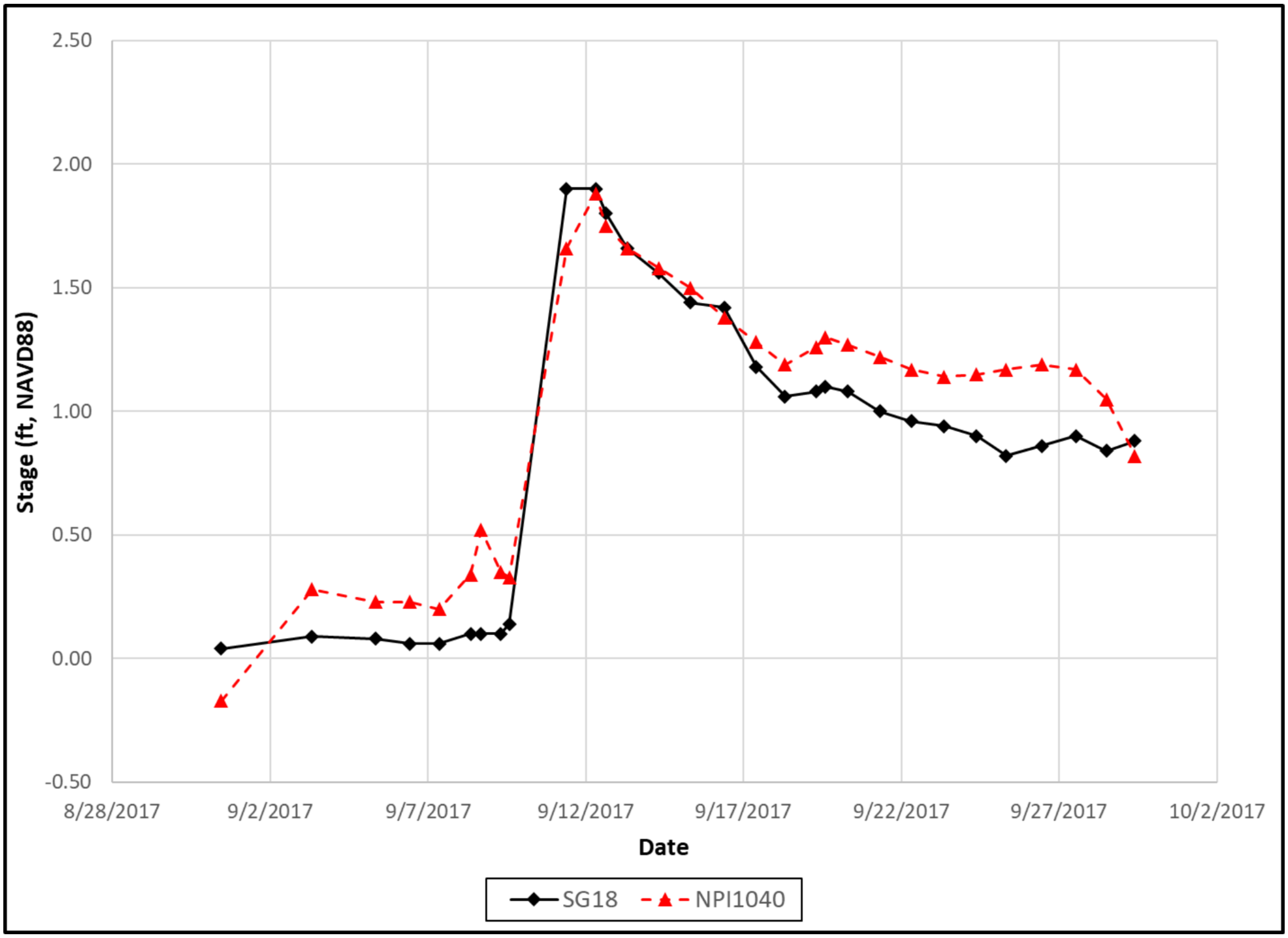


Figure 6.26: Gage SG18 PICA Riverside Calibration #2 Comparisons

The statistical metrics provided in **Table 6.17** show very good correlation between the measured and modeled stage data.

Table 6.17: Calibration Statistical Metrics SG18

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.971	Very Good	0.938	Very Good
NSE	0.923	Very Good	0.883	Very Good
ME	-0.093	Very Good	-0.135	Very Good
MAE	0.136	Very Good	0.176	Very Good
RMSE	0.165	Very Good	0.203	Very Good
RSR	0.274	Very Good	0.302	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.3.16 Gage SG19 PICA North – Calibration Results

SG19 is located within the northern Pine Island impoundment. The stage hydrograph comparisons for Calibration #1 and Calibration #2 are provided below in **Figure 6.27** and **Figure 6.28**, respectively. The Calibration #1 modeled peak stage is 2.16-ft (NAVD88), which is approximately 1.1-inches above the measured peak stage (2.07-ft, NAVD88). The peak stage for Calibration #2 is 2.10-ft (NAVD88), which is 0.4-inches above the measured peak stage.

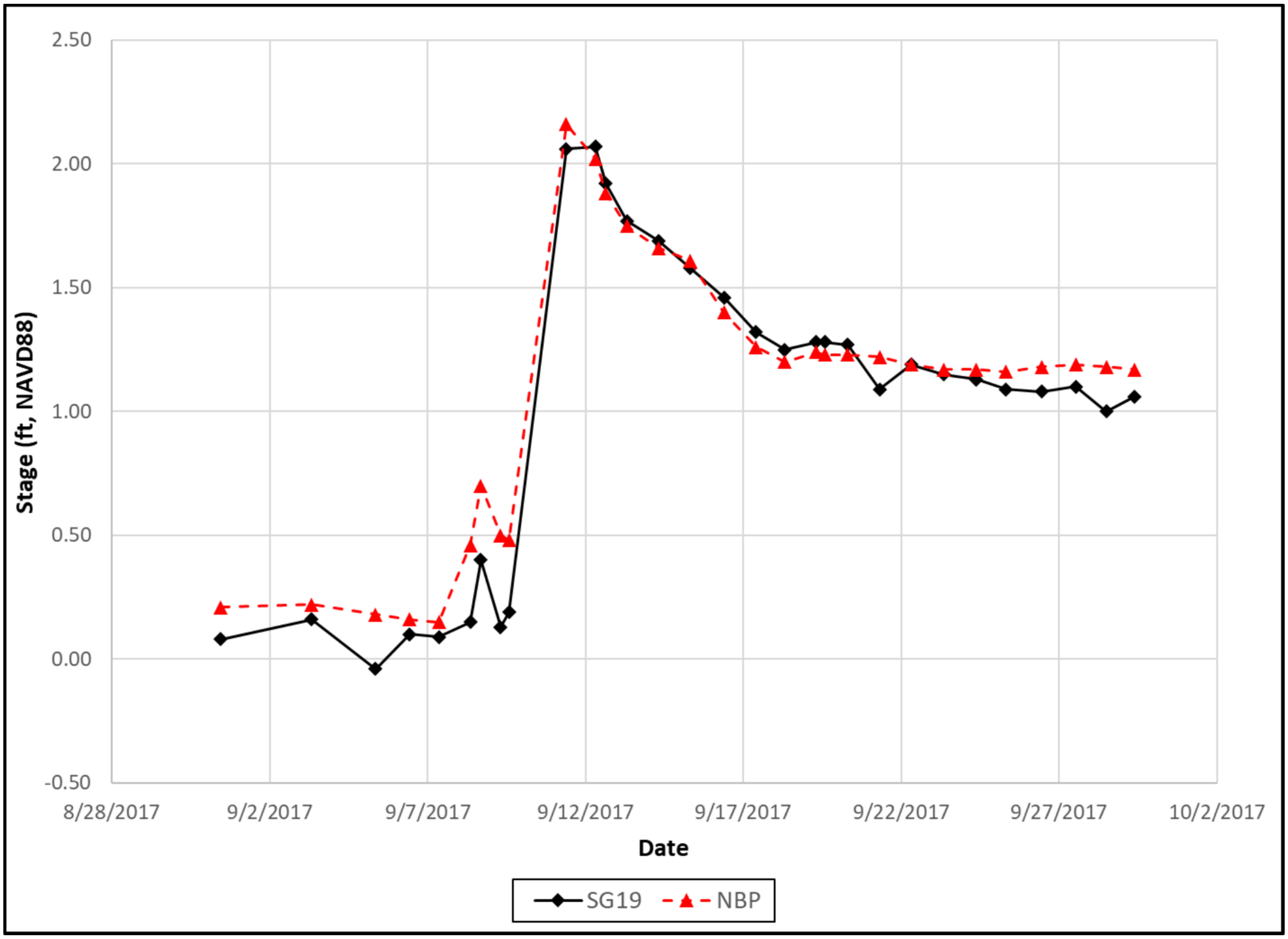


Figure 6.27: Gage SG19 PICA North Calibration #1 Comparisons

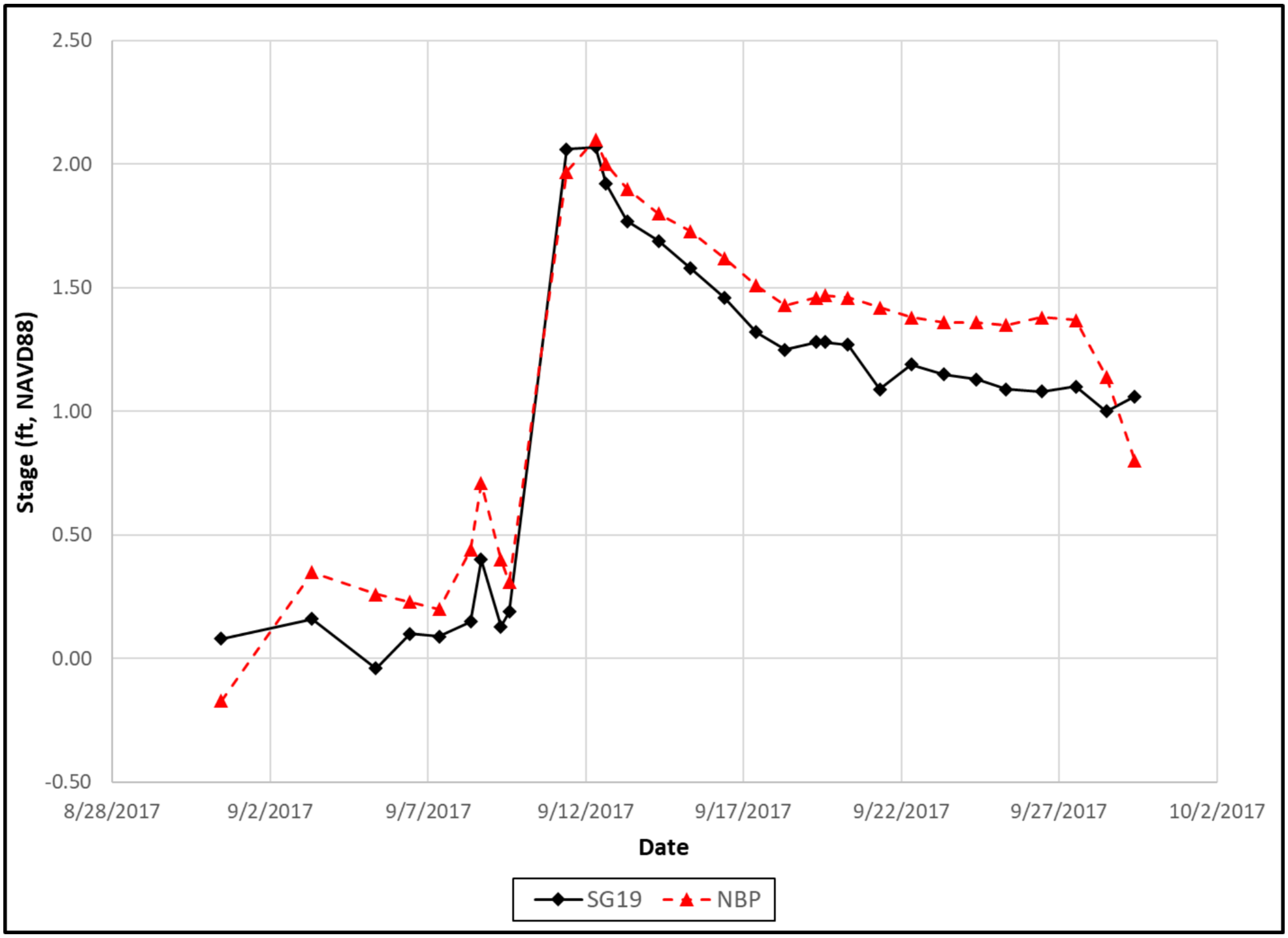


Figure 6.28: Gage SG19 PICA North Calibration #2 Comparisons

Like the SG18 comparisons, both simulations show very good correlation between the measured and modeled data as shown in **Table 6.18** below.

Table 6.18: Calibration Statistical Metrics SG19

Metric Parameter	Calibration Simulation #1	Quality #1	Calibration Simulation #2	Quality #2
R^2	0.975	Very Good	0.951	Very Good
NSE	0.950	Very Good	0.891	Very Good
ME	-0.074	Very Good	-0.155	Very Good
MAE	0.104	Very Good	0.195	Very Good
RMSE	0.142	Very Good	0.209	Very Good
RSR	0.220	Very Good	0.325	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 30

6.4 Verification Analysis

The verification period of record is from 10/01/2017 – 11/03/2017. Like the calibration, the verification analysis included comparisons between measured and modeled results for each of these staff gages. The results of the final verification analyses are provided in the subsequent sections. Like the calibration, there are two verification analyses for each gage unless otherwise stated. Verification #1 is the simulation that includes the calibration parameter adjustments and the adjusted boundary conditions provided by AEI. Verification #2 is identical to Verification #1 except internal boundary conditions were incorporated into the model at the locations specified in **Table 6.2**.

6.4.1 Gage SG1 Sykes Creek at Sea Ray Dr. – Verification Results

The verification analysis at SG1 indicates good correlation between the measured and modeled results (**Figure 6.29**), especially for the larger storm event in early October. Additionally, the modeled peak stage (1.49-ft, NAVD88) is approximately 0.72-inches above the measured peak stage (1.42-ft, NAVD88), however, the model does have a tendency to over-predict the stage for the smaller storm event in mid to late October. With that said, the average difference is fairly minor (3-inches). No comparisons were conducted for this particular gage for Verification #2 since this location in the model was converted to a time-stage node for that analysis.

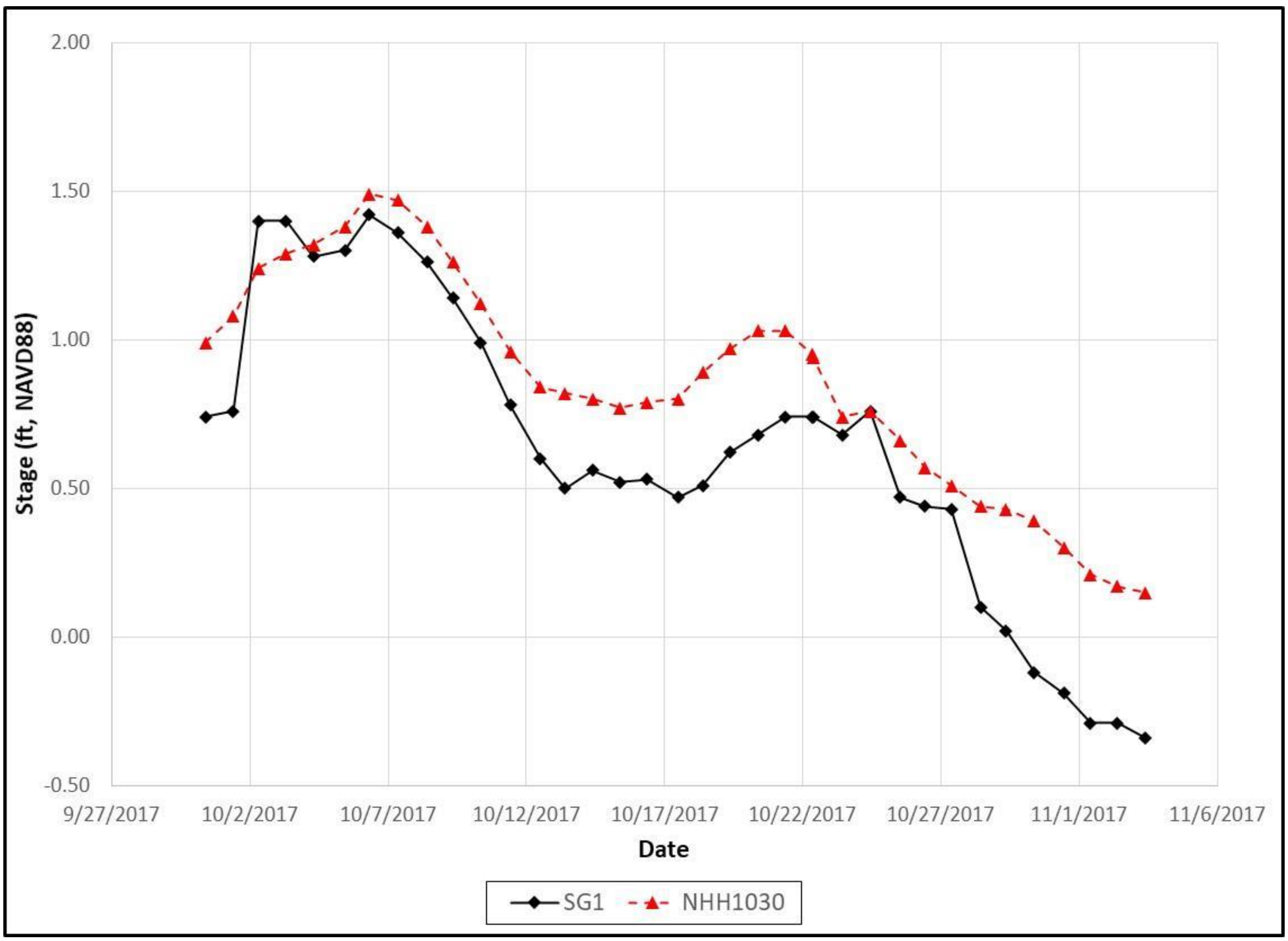


Figure 6.29: Gage SG1 Sykes Creek at Sea Ray Dr. Verification #1 Comparisons

The statistical metrics listed in **Table 6.19** also show acceptable correlation between the measured and modeled stages. Most parameters are good to very good. The only exception is the satisfactory NSE.

Table 6.19: Verification Statistical Metrics SG1

Metric Parameter	Verification Simulation #1	Quality #1
R^2	0.941	Very Good
NSE	0.675	Satisfactory
ME	-0.229	Very Good
MAE	0.244	Very Good
RMSE	0.282	Very Good
RSR	0.562	Good
1/2 Standard Deviation Obs.	2	Good

Note: Number of pair data (observed and simulated) = 37

6.4.2 Gage SG2 East Hall Rd. North – Verification Results

The verification analysis for SG2 shows excellent correlation between measured and model data for both verification simulations (**Figure 6.30**). The stages are within 4.2-inches for the modeled peak stage (2.27-ft, NAVD88) compared to the measured peak stage (1.92-ft, NAVD88) for the Verification #1 simulations.

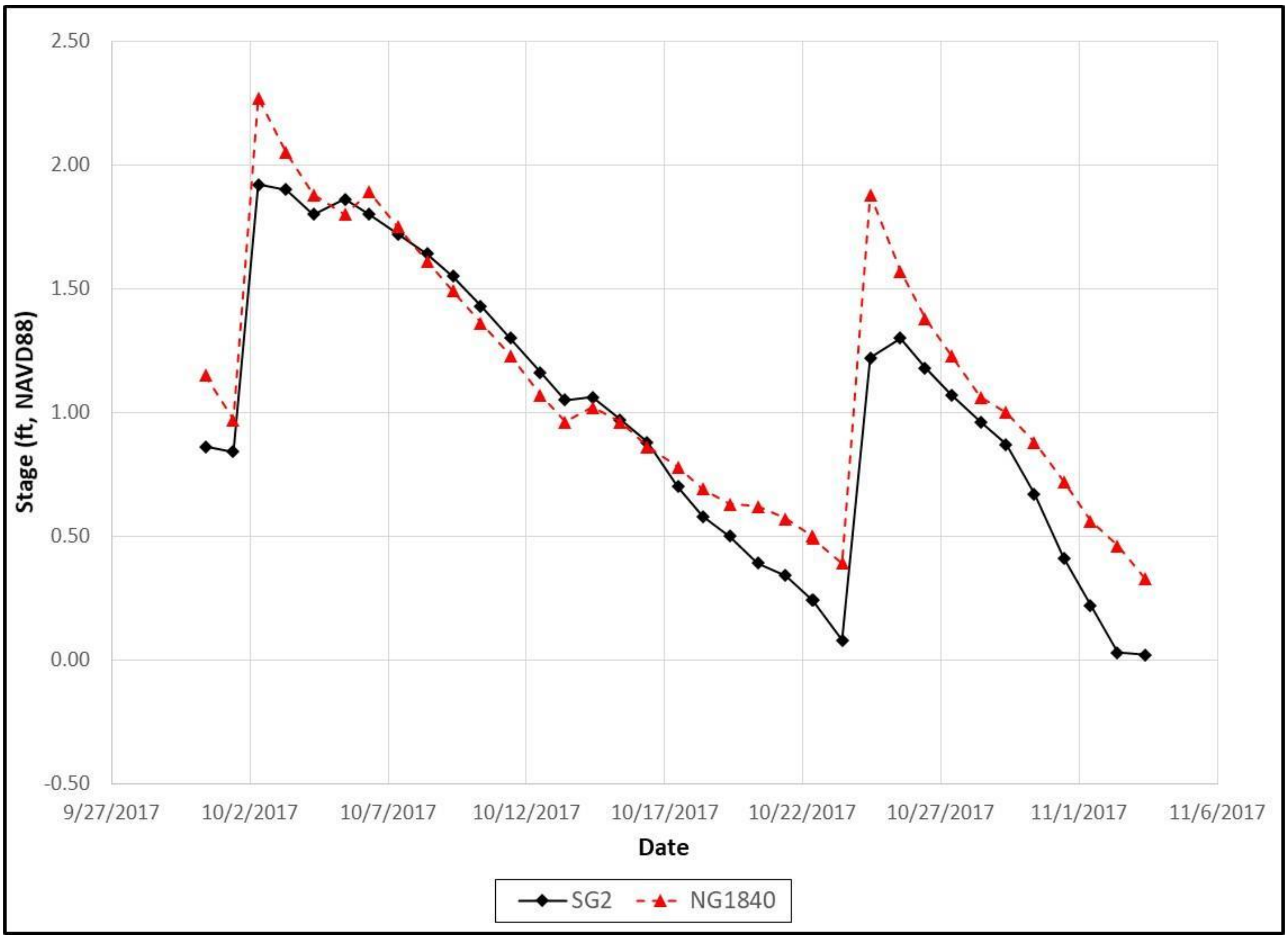


Figure 6.30: Gage SG2 East Hall Rd. North Verification #1 Comparisons

There is a much better correlation between the modeled and measured stages for the Verification #2 simulation. Like the calibration analyses, this gage is sensitive to the operation of the East Hall Road Pumps. This is evident looking at the Verification #2 results in **Figure 6.31.** In fact, the modeled peak stage (2.07-ft, NAVD88) is only 1.8-inches above the measured peak stage.

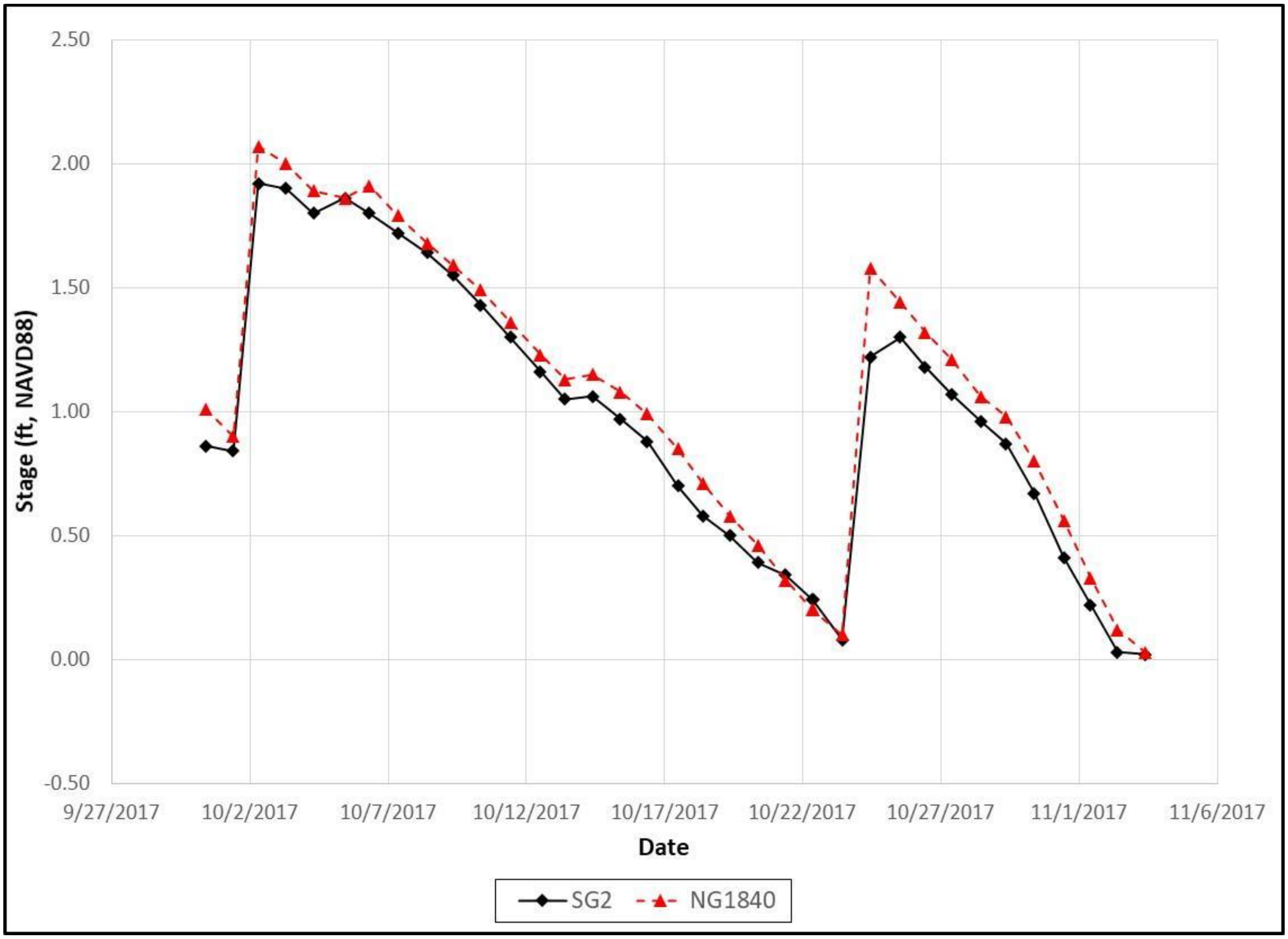


Figure 6.31: Gage SG2 East Hall Rd. North Verification #2 Comparisons

The statistical results in **Table 6.20** show very good correlation between the model results and the measured stages for both simulations at this gage location.

Table 6.20: Verification Statistical Metrics SG2

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.915	Very Good	0.987	Very Good
NSE	0.846	Very Good	0.961	Very Good
ME	-0.147	Very Good	-0.089	Very Good
MAE	0.177	Very Good	0.095	Very Good
RMSE	0.224	Very Good	0.113	Very Good
RSR	0.387	Very Good	0.196	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 37

6.4.3 Gage SG3 East Hall Rd. Pump House – Verification Results

The verification analysis for SG3 shows good correlation between measured and model data (**Figure 6.32).** The peak for the Verification #1 simulation is 2.32-ft (NAVD88) compared to the measured peak stage of 1.90-ft (NAVD88). This is a difference in peak stage of approximately 5.0-inches. The subsequent smaller storm event in late October also shows simulated stages higher than the measured stage. However, this gage is in close proximity of the East Hall Road Pumps. Therefore, it is sensitive to the pump operation. Keep in mind that the pumps at this location were modeled with best available information, however, the operation information specified in the model does not account for all pump operation changes that occurred during this verification time period as they were not documented. No comparisons were conducted for this gage for Verification #2 because node NFF2020 was converted to a time-stage node for the Verification #2 analysis.

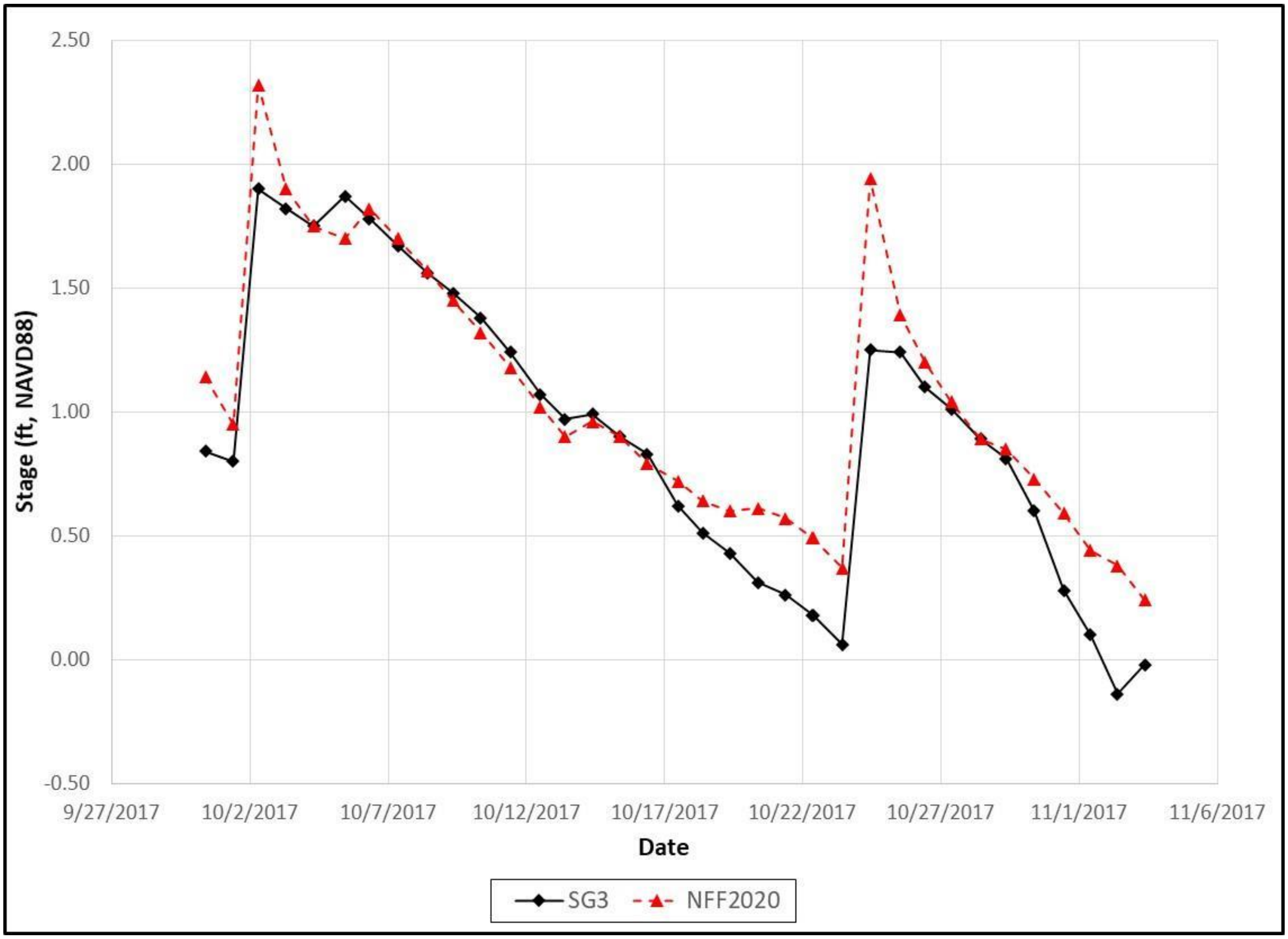


Figure 6.32: Gage SG3 East Hall Rd. Pump House Verification #1 Comparisons

Regardless of the pump operation sensitivity, the results in **Table 6.21** show very good correlation between the measured and model stages at this location.

Table 6.21: Verification Statistical Metrics SG3

Metric Parameter	Verification Simulation #1	Quality #1
R^2	0.903	Very Good
NSE	0.842	Very Good
ME	-0.140	Very Good
MAE	0.168	Very Good
RMSE	0.233	Very Good
RSR	0.392	Very Good
1/2 Standard Deviation Obs.	3	Very Good

Note: Number of pair data (observed and simulated) = 37

6.4.4 Gage SG4 East Hall Rd. Barge Canal Ditch – Verification Results

Similar to the SG3 comparisons, the verification analysis for SG4 shows good correlation between measured and model data (**Figure 6.33**). The peak stage for Verification #1 is 2.32-ft (NAVD88) compared to the measured peak stage of 1.89-ft (NAVD88). This is a difference in peak stage of approximately 5.2-inches. The subsequent smaller storm event in late October also shows simulated stages higher than the measured stage like what occurred at SG3 for the Verification #1 simulation. However, this gage is in very close proximity of the Hall Road Pumps. Therefore, it is also sensitive to the pump operation. No comparisons were conducted for this gage for Verification #2 since this location was converted to a time-stage node for the Verification #2 analysis.

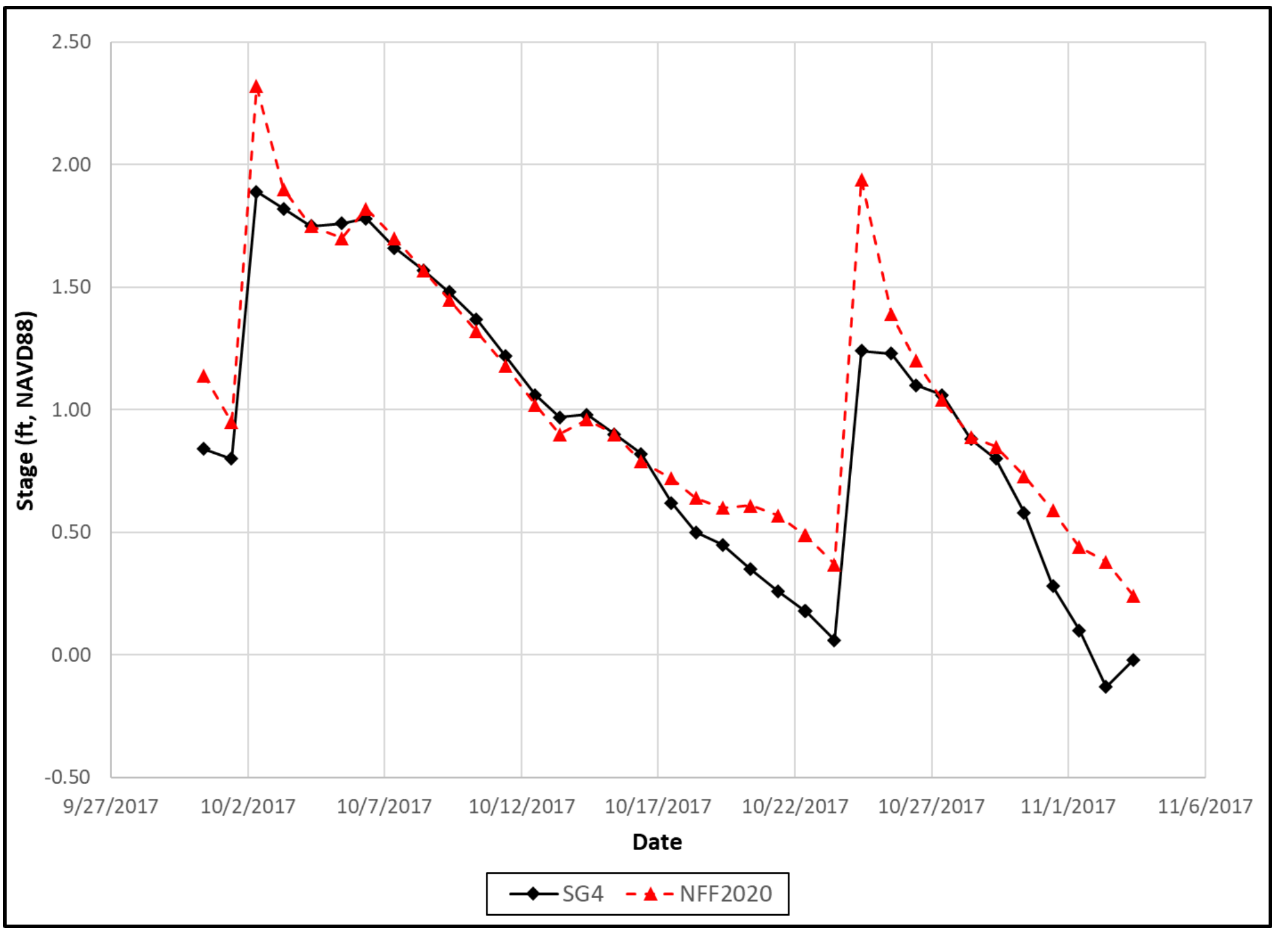


Figure 6.33: Gage SG4 East Hall Rd. Barge Canal Ditch Verification #1 Comparisons

The statistical results in **Table 6.22** show very good correlation between the measured and model stages.

Table 6.22: Verification Statistical Metrics SG4

Metric Parameter	Verification Simulation #1	Quality #1
R^2	0.905	Very Good
NSE	0.841	Very Good
ME	-0.143	Very Good
MAE	0.163	Very Good
RMSE	0.231	Very Good
RSR	0.394	Very Good
1/2 Standard Deviation Obs.	3	Very Good

Note: Number of pair data (observed and simulated) = 37

6.4.5 Gage SG5 Chase Hammock at Judson Rd. – Verification Results

The verification analysis for SG5 shows good correlation between measured and model data (**Figure 6.34** and **Figure 6.35**). The difference between the modeled (2.27-ft, NAVD88) and measured peak stage (2.10-ft, NAVD88) is only 2.0-inches for the Verification #1 simulation. The subsequent smaller storm event in late October has a modeled peak stage within 5-inches of the measured value for Verification #1 as well.

The Verification #2 simulation shows even better correlation between the modeled and measured data. For example, the modeled peak stage is approximately 2.20-ft (NAVD88) which is within 1.2-inches of the measured peak stage. The statistical metrics shown in **Table 6.23** also show very good correlation between the modeled and measured stages for both simulations.

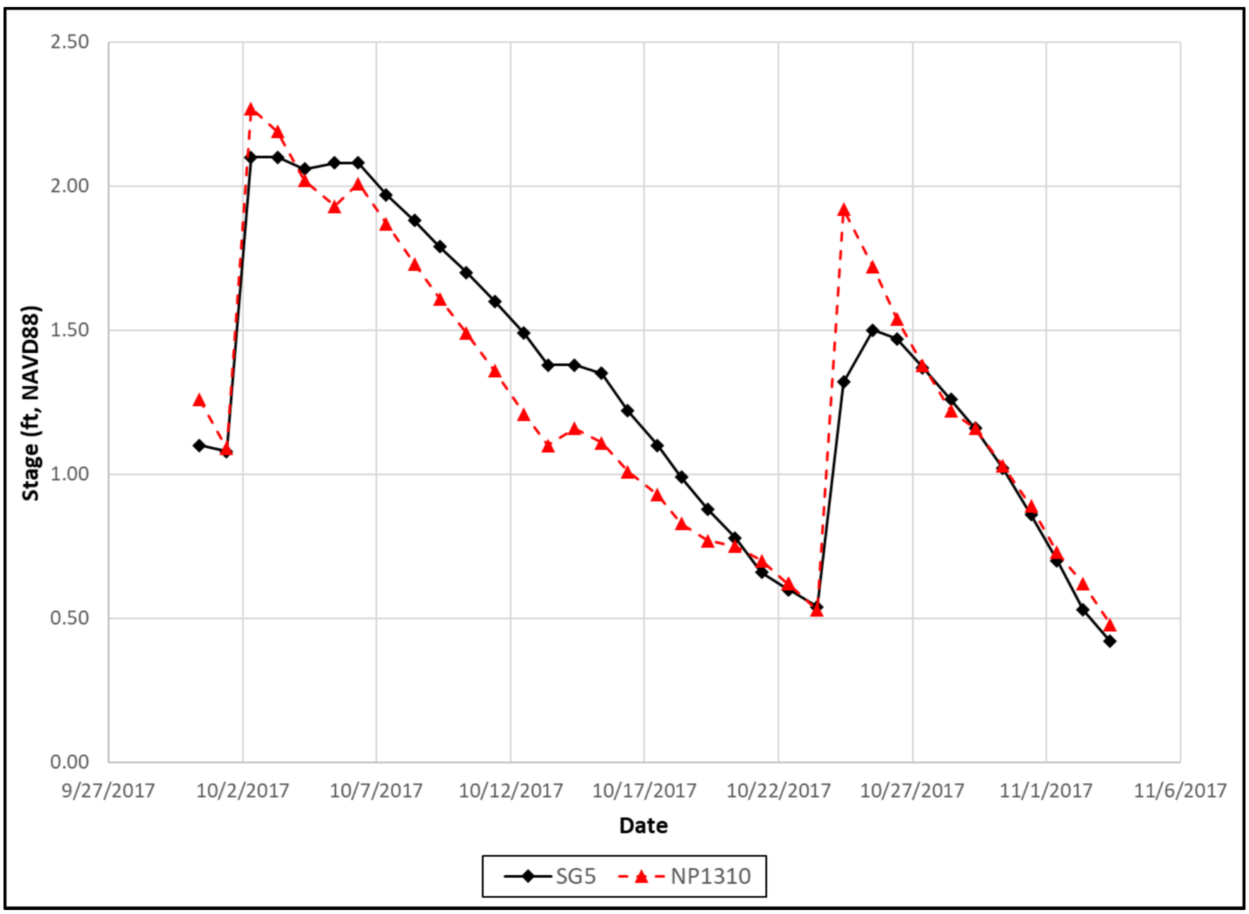


Figure 6.34: Gage SG5 Chase Hammock at Judson Rd. Verification #1 Comparisons

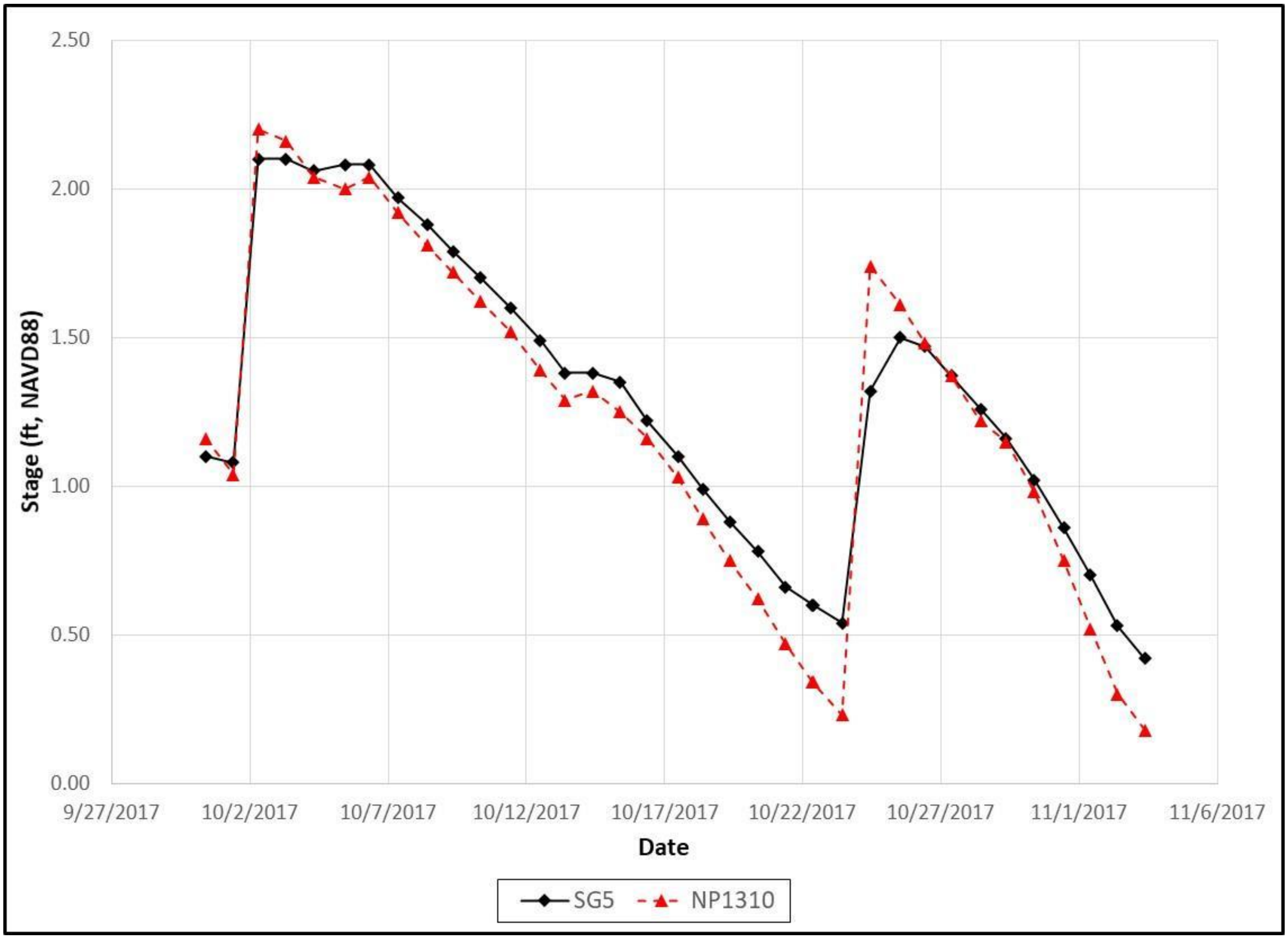


Figure 6.35: Gage SG5 Chase Hammock at Judson Rd. Verification #2 Comparisons

Table 6.23: Verification Statistical Metrics SG5

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.891	Very Good	0.967	Very Good
NSE	0.883	Very Good	0.917	Very Good
ME	0.035	Very Good	0.070	Very Good
MAE	0.126	Very Good	0.112	Very Good
RMSE	0.172	Very Good	0.145	Very Good
RSR	0.337	Very Good	0.284	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 37

6.4.6 Gage SG6 Crisafulli at Judson Rd. – Verification Results

For both verification simulations, the model results agree quite well with the measured stage readings at SG6 (**Figure 6.36** and **Figure 6.37**). The modeled peak stages for Verification #1 and Verification #2 are 2.44-ft (NAVD88) and 2.49-ft (NAVD88), respectively. Both are within approximately 2.3-inches of the measured maximum stage (2.30-ft, NAVD88). Additionally, the peak stages during the second storm event in October are within 6-inches of the measured peak stage for both verification simulations.

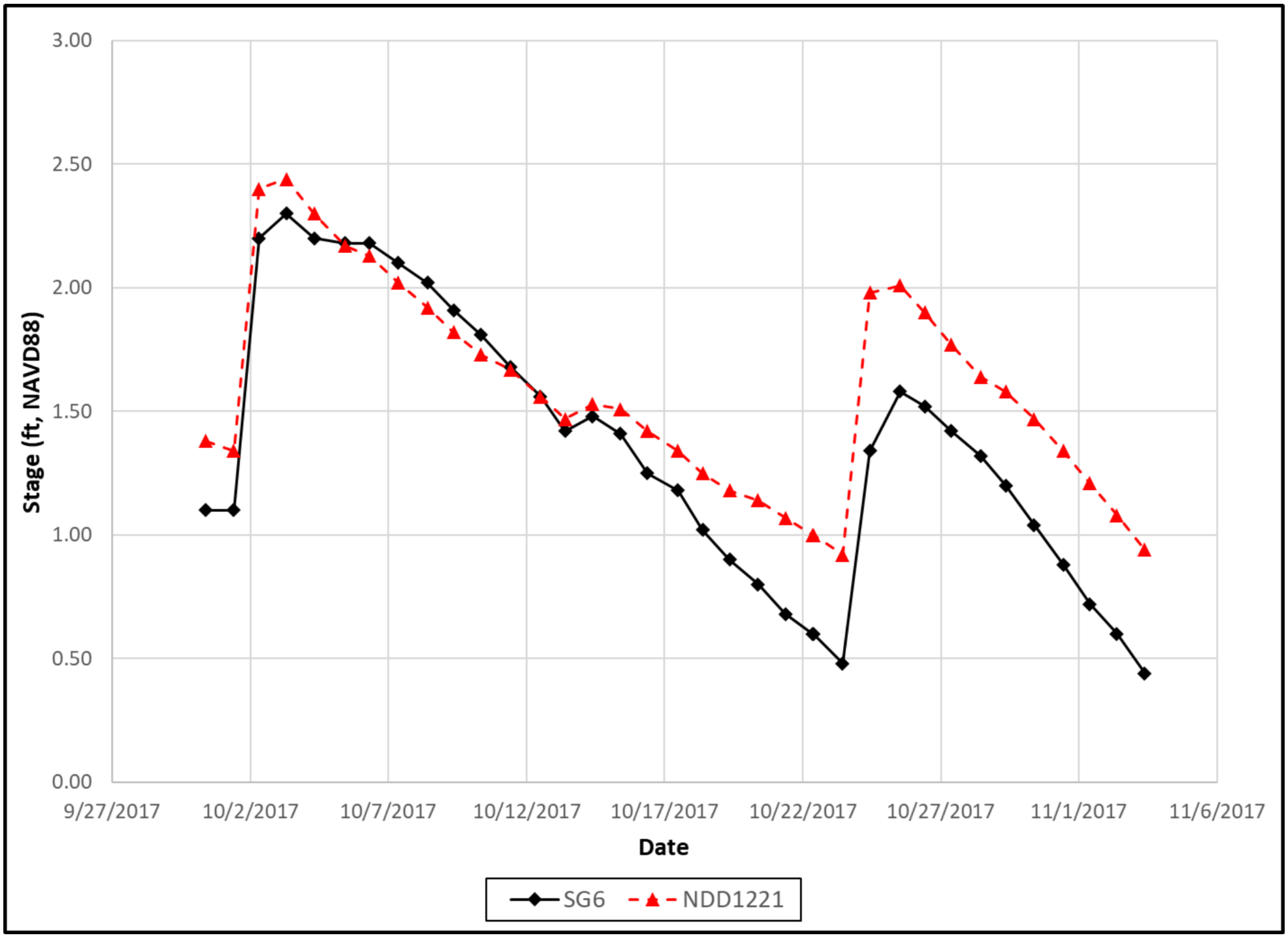


Figure 6.36: Gage SG6 Crisafulli at Judson Rd. Verification #1 Comparisons

The statistical metrics shown in **Table 6.24**, show both verification simulations agree well with the measured data. Although, the results for Verification #2 agree slightly better based on the statistical metric criteria.

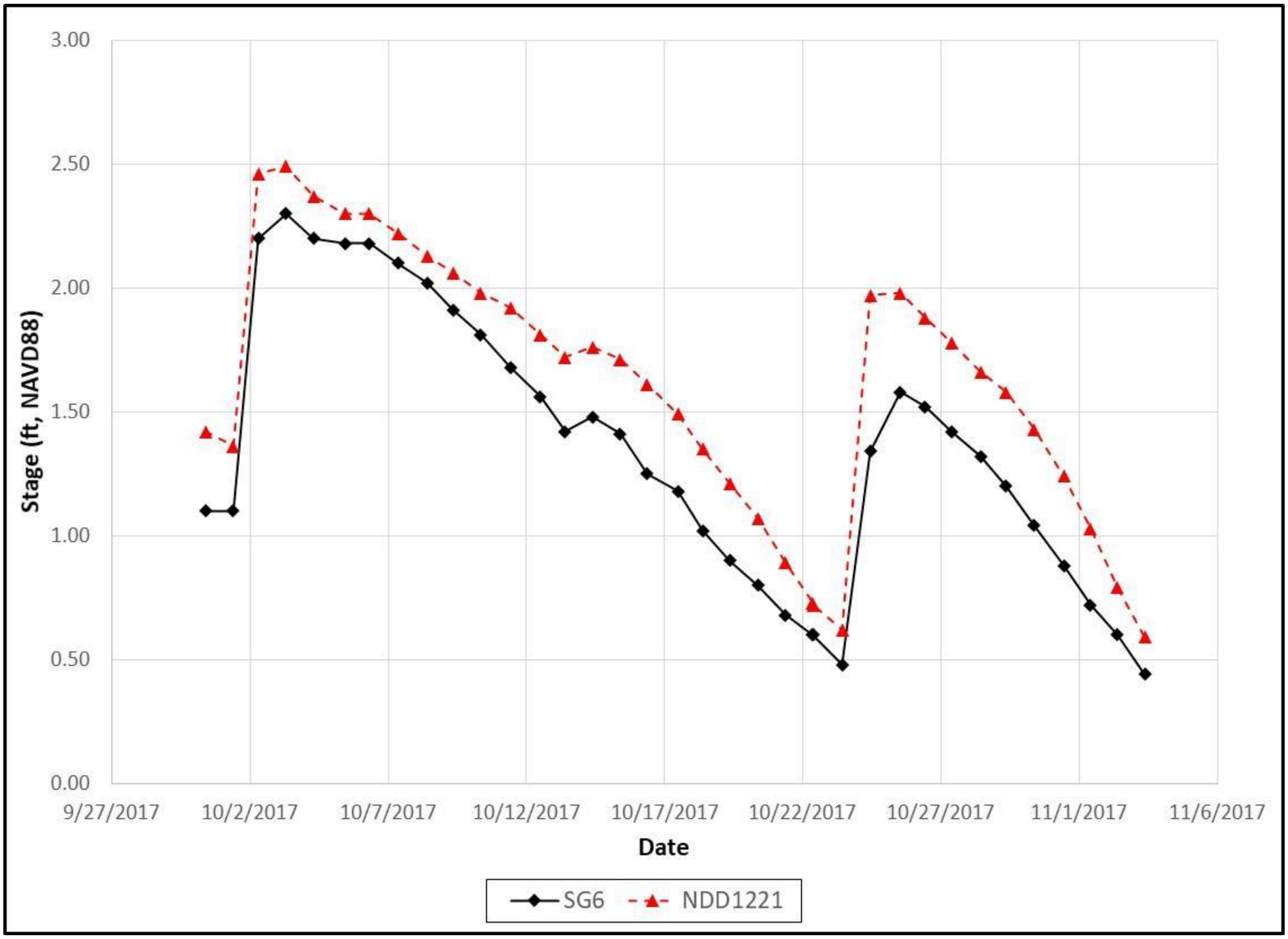


Figure 6.37: Gage SG6 Crisafulli at Judson Rd. Verification #2 Comparisons

The statistical metrics shown in **Table 6.24** are mostly classified as very good, this shows that both verification simulations agree well with the measured data. Although, the results for Verification #2 agree slightly better based on the statistical metric criteria.

Table 6.24: Verification Statistical Metrics SG6

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.893	Very Good	0.959	Very Good
NSE	0.680	Satisfactory	0.731	Good
ME	-0.234	Very Good	-0.261	Good
MAE	0.257	Very Good	0.261	Very Good
RMSE	0.309	Very Good	0.284	Very Good
RSR	0.558	Good	0.512	Good
1/2 Standard Deviation Obs.	2	Good	2	Good

Note: Number of pair data (observed and simulated) = 37

6.4.7 Gage SG7 East Crisafulli at Joseph Ct. – Verification Results

Like SG6, the model results at SG7 agree quite well with the measured stage readings (**Figure 6.38** and **Figure 6.39**). The modeled peak stages for Verification #1 and Verification #2 are 2.48-ft (NAVD88) and 2.53-ft (NAVD88), respectively. Both are within approximately 1.6-inches of the measured maximum stage (2.40-ft, NAVD88). Additionally, the peak stages during the second storm event in October are also within 6-inches of the measured peak stage for both verification simulations.

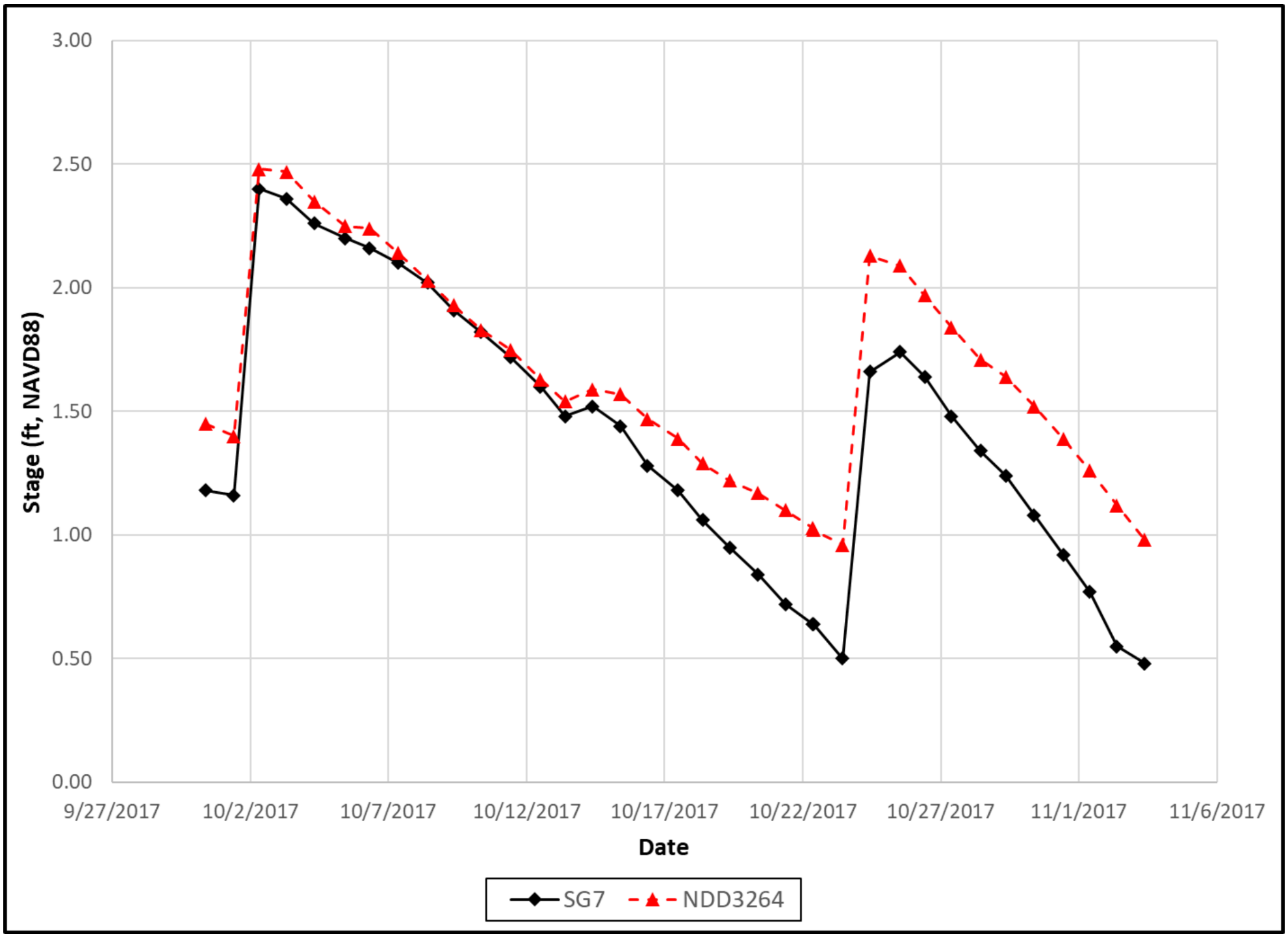


Figure 6.38: Gage SG7 East Crisafulli at Joseph Ct. Verification #1 Comparisons

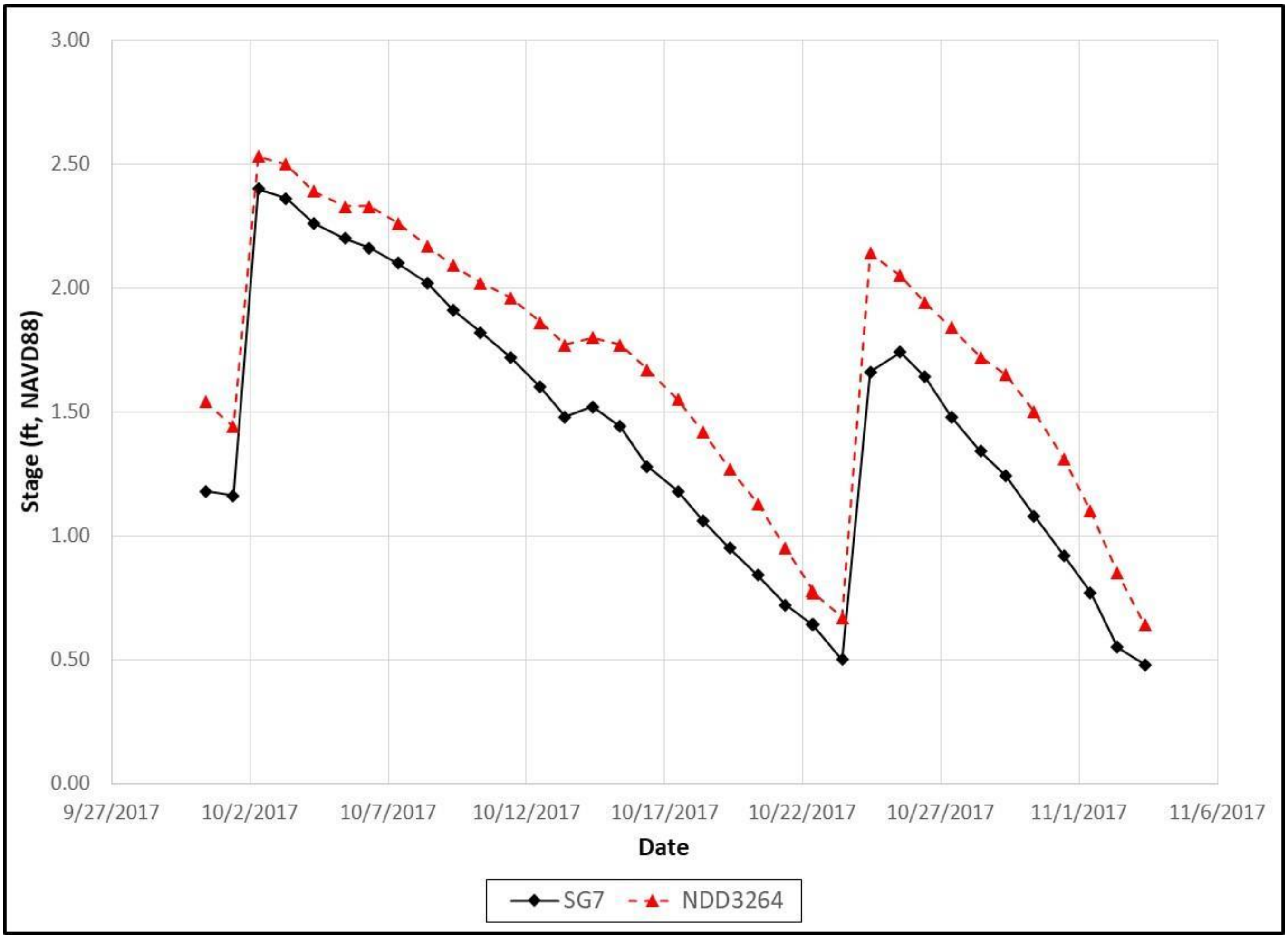


Figure 6.39: Gage SG7 East Crisafulli at Joseph Ct. Verification #2 Comparisons

The statistical metrics shown in **Table 6.25** are mostly classified as very good, this shows that both verification simulations agree well with the measured data. However, the results for Verification #1 agree slightly better based on the statistical metrics.

Table 6.25: Verification Statistical Metrics SG7

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.937	Very Good	0.968	Very Good
NSE	0.707	Good	0.735	Good
ME	-0.248	Very Good	-0.269	Good
MAE	0.248	Very Good	0.269	Very Good
RMSE	0.301	Very Good	0.287	Very Good
RSR	0.534	Good	0.508	Good
1/2 Standard Deviation Obs.	2	Good	2	Good

Note: Number of pair data (observed and simulated) = 37

6.4.8 Gage SG8 N Courtenay at Pine Island – Verification Results

The measured vs. modeled stage hydrograph comparisons for both verification simulations are provided in **Figure 6.40** and **Figure 6.41**. Both simulations compare extremely well based on visual inspection of the stage hydrographs. The maximum stages for Verification #1 and Verification #2 are 2.41-ft (NAVD88) and 2.49-ft (NAVD88), respectively. Both are within approximately 2.5-inches of the measured maximum stage (2.28-ft, NAVD88). Additionally, the measured peak stage for the second storm event in October is approximately within 6-inches of the modeled peak stage for both verification simulations.

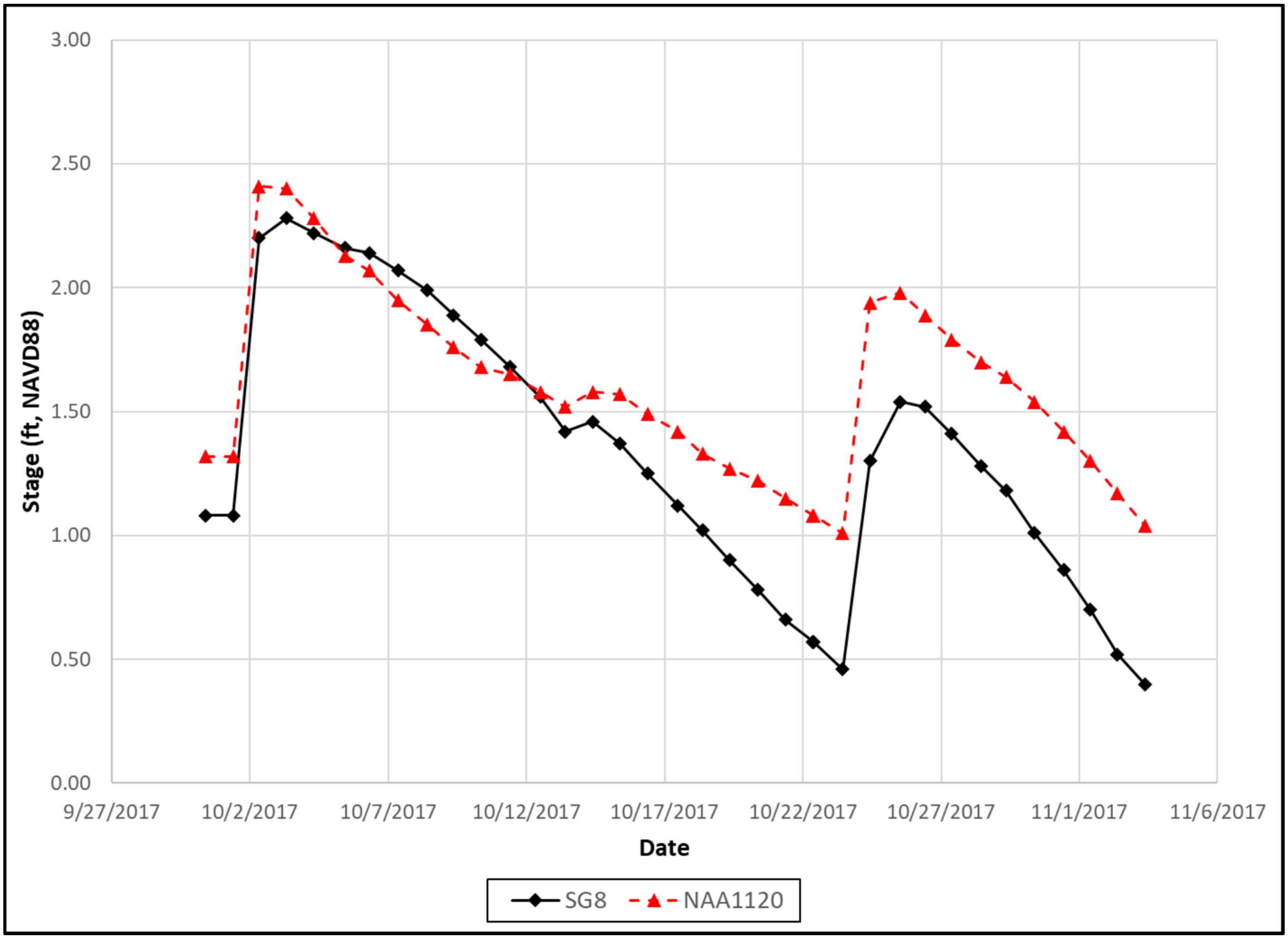


Figure 6.40: Gage SG8 N Courtenay at Pine Island Verification #1 Comparisons

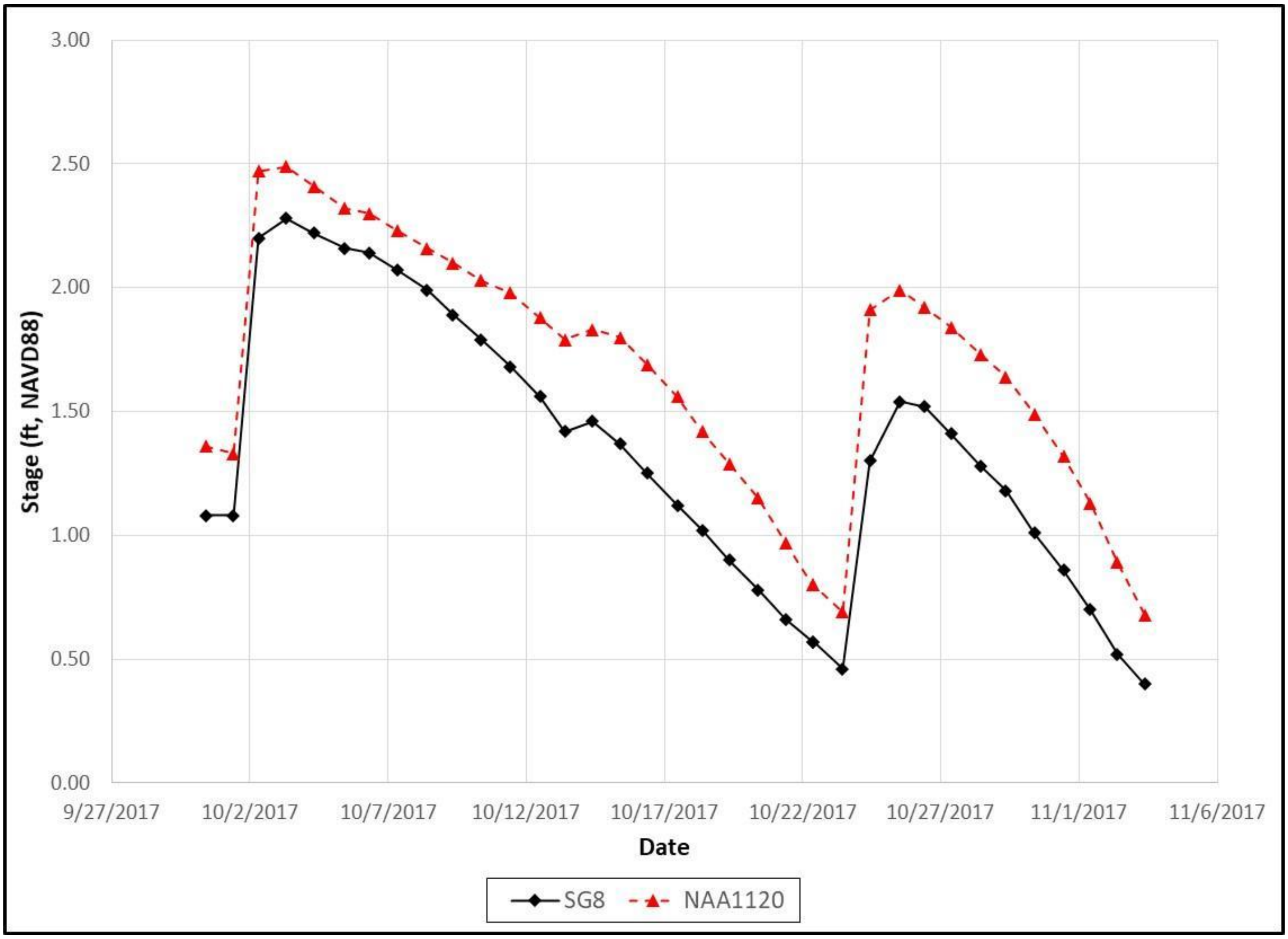


Figure 6.41: Gage SG8 N Courtenay at Pine Island Verification #2 Comparisons

The statistical metrics for this gage provided in **Table 6.26** range from satisfactory to very good. Both verification simulations agree well with the measured data, however, results for Verification #2 agree slightly better based on the statistical metrics.

Table 6.26: Verification Statistical Metrics SG8

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.861	Very Good	0.960	Very Good
NSE	0.544	Satisfactory	0.597	Satisfactory
ME	-0.280	Good	-0.332	Good
MAE	0.315	Very Good	0.332	Very Good
RMSE	0.373	Very Good	0.350	Very Good
RSR	0.666	Satisfactory	0.626	Satisfactory
1/2 Standard Deviation Obs.	1	Satisfactory	1	Satisfactory

Note: Number of pair data (observed and simulated) = 37

6.4.9 Gage SG9 Pine Island 1 Mile North of North Courtenay – Verification Results

The model results for both verification simulations at SG9 agree well with the measured data as shown in **Figure 6.42** and **Figure 6.43**. There is one outlier in the measured data noted on 10/14/2017. The reading on this date seems inconsistent with the other readings suggesting either a misread when recording the stage or that blockage somewhere in the drainage system may have occurred which resulted in increased stages. Regardless, the peak stages for Verification #1 (2.29-ft, NAVD88) and Verification #2 (2.40-ft, NAVD88) are within 2.9-inches of the measured peak stage (2.16-ft, NAVD88). Additionally, the modeled peak stages for the second storm event are approximately within 6-inches of the measured peak stage for both verification simulations.

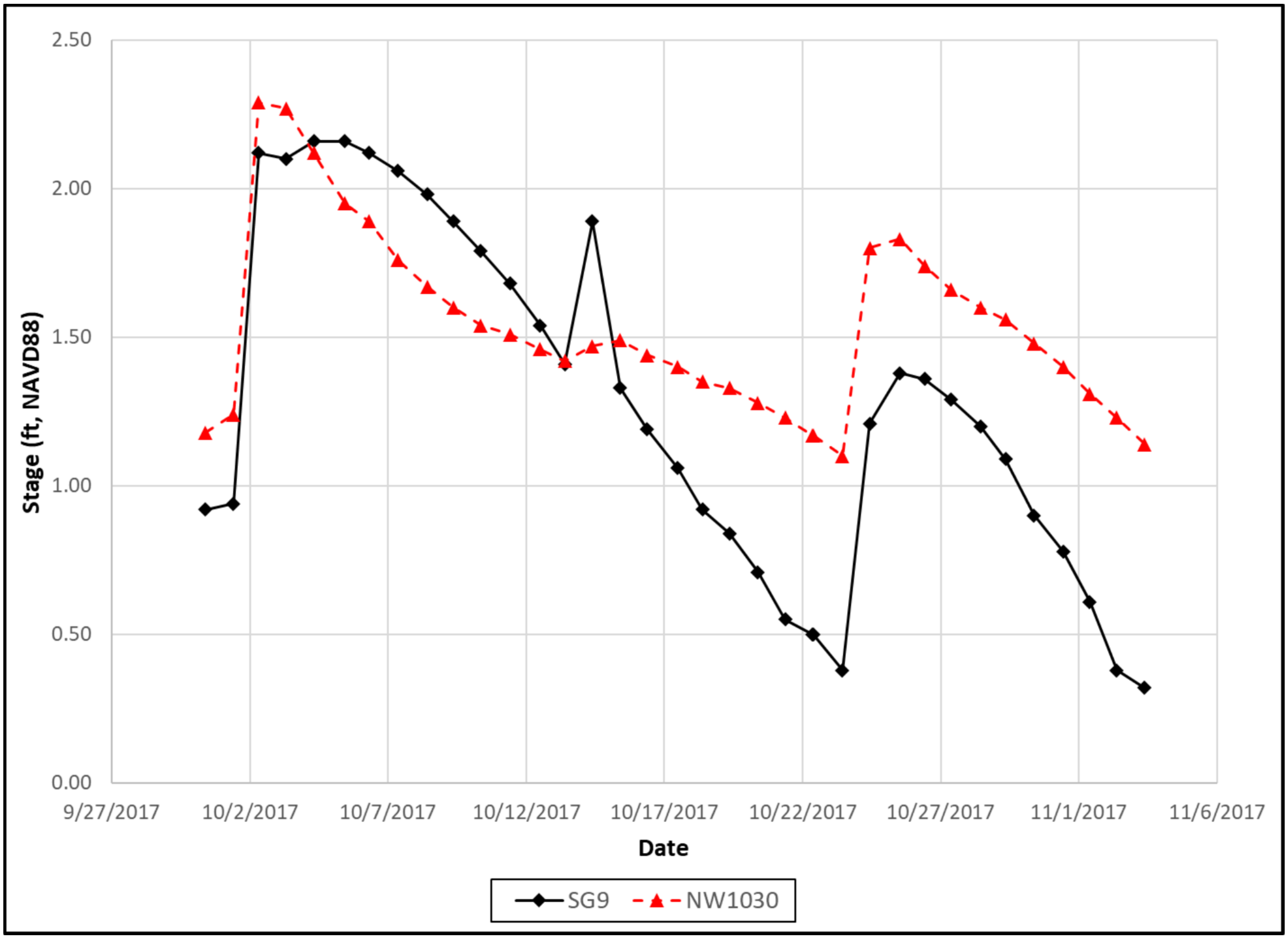


Figure 6.42: Gage SG9 Pine Island 1 Mile North of North Courtenay Verification #1 Comparisons

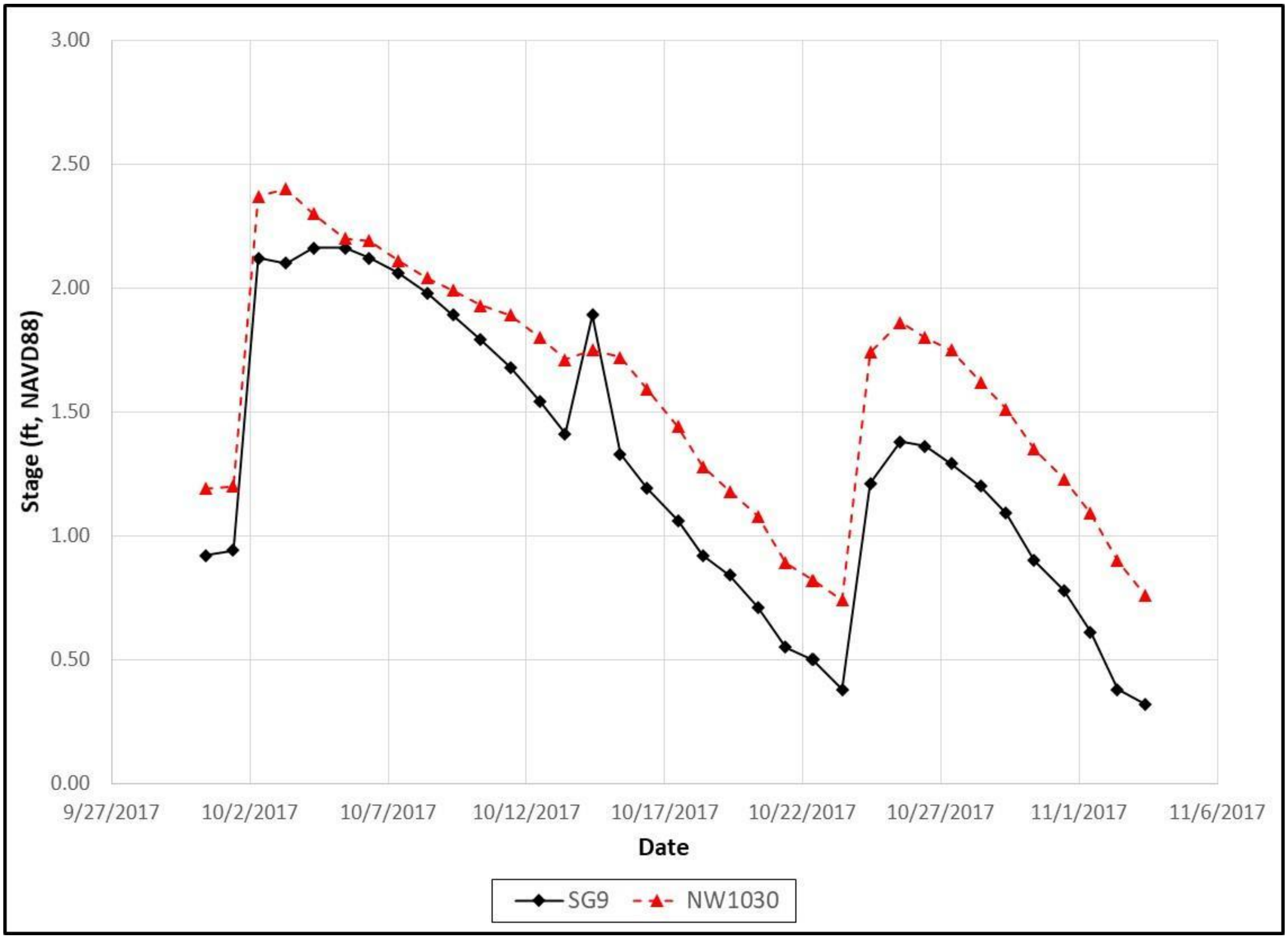


Figure 6.43: Gage SG9 Pine Island 1 Mile North of North Courtenay Verification #2 Comparisons

The statistical metrics for this gage provided in **Table 6.27** generally range from satisfactory to very good, however two metrics are classified as not satisfactory for Verification #1. Results for Verification #2 agree much better based on the statistical analysis indicating this area is also sensitive to the pump operation at Pine Island.

Table 6.27: Verification Statistical Metrics SG9

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.699	Satisfactory	0.946	Very Good
NSE	0.380	**Not Satisfactory**	0.650	Satisfactory
ME	-0.273	Good	-0.305	Good
MAE	0.401	Very Good	0.313	Very Good
RMSE	0.457	Very Good	0.343	Very Good
RSR	0.776	**Not Satisfactory**	0.583	Good
1/2 Standard Deviation Obs.	1	Satisfactory	1	Satisfactory

Note: Number of pair data (observed and simulated) = 37

6.4.10 Gage SG10 Pine Island Harvey Grove Pump – Verification Results

As shown in **Figure 6.44**, the modeled peak stage for Verification #1 (1.99-ft, NAVD88) matches the measured value (1.86-ft, NAVD88) quite well. The Verification #1 peak stage for the second October storm event (1.02-ft, NAVD88) is identical to the measured maximum stage (1.02-ft). The recovery legs for both storm events are, however, inaccurate for this simulation. This is primarily attributed to the pump operation.

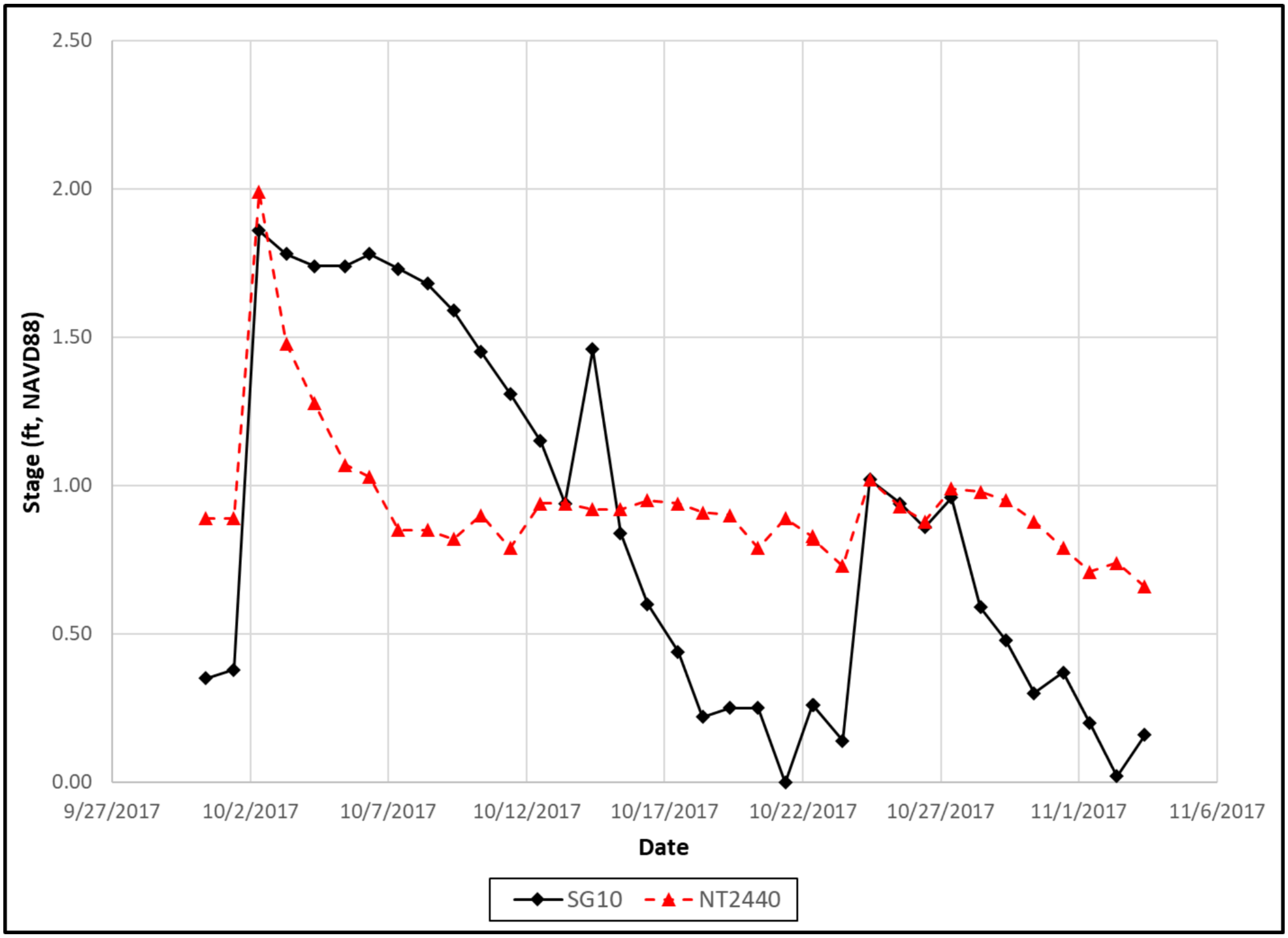


Figure 6.44: Gage SG10 Pine Island Harvey Grove Pump Verification #1 Comparisons

The impact of the pump operations is evident based on the stage hydrograph comparisons shown in **Figure 6.45**. The results for Verification #2 compare much better with the measure stages. The modeled peak stage for Verification #2 (1.94-ft, NAVD88) is 1-inch above the measured value (1.86-ft, NAVD88).

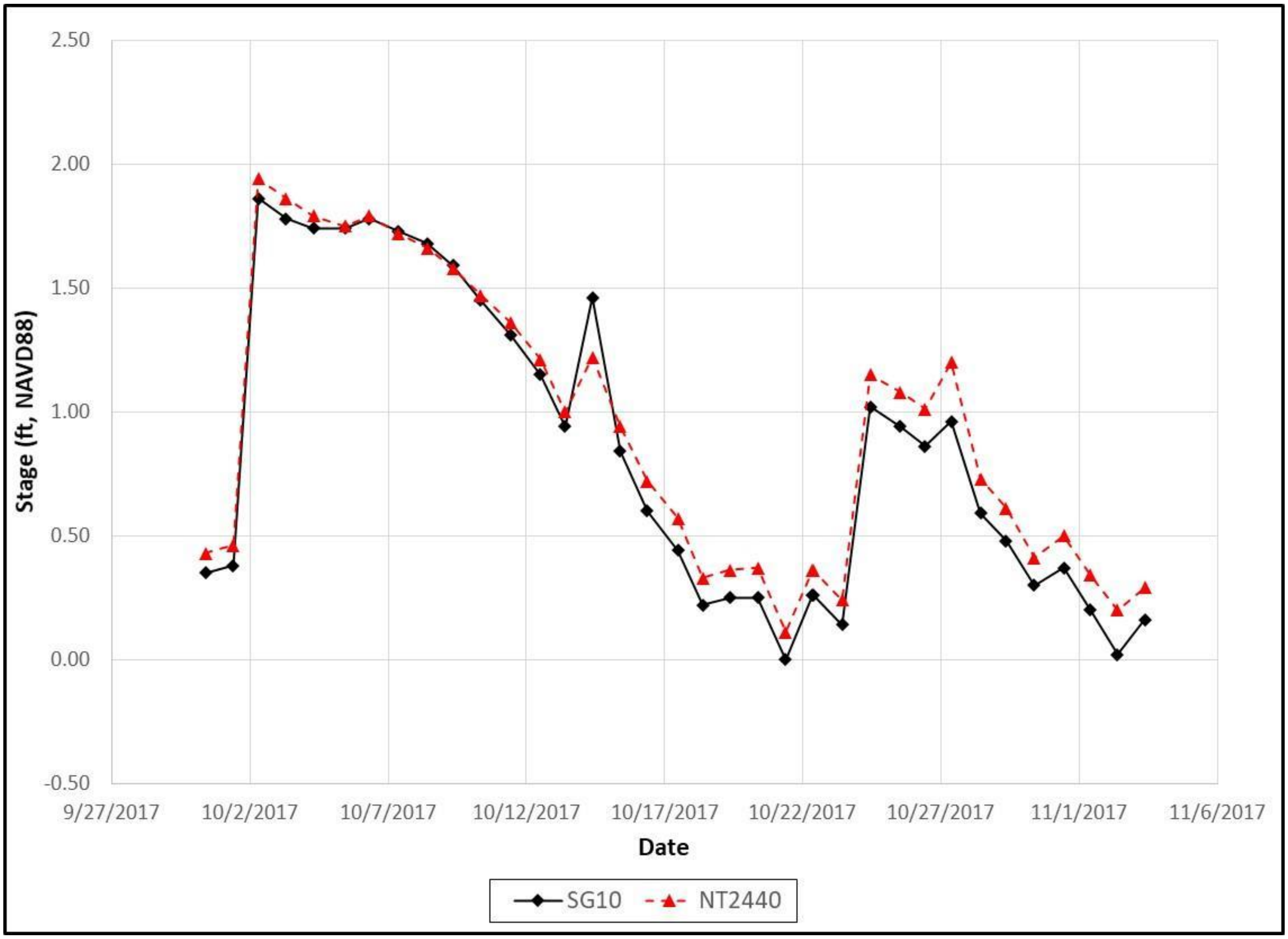


Figure 6.45: Gage SG10 Pine Island Harvey Grove Pump Verification #2 Comparisons

Additionally, the statistical analysis (**Table 6.28)** shows that Verification #1 has three parameters that are not satisfactory while Verification #2 has very good correlation between the measured and modeled stages.

Table 6.28: Verification Statistical Metrics SG10

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.314	**Not Satisfactory**	0.988	Very Good
NSE	0.251	**Not Satisfactory**	0.965	Very Good
ME	-0.104	Very Good	-0.084	Very Good
MAE	0.465	Very Good	0.099	Very Good
RMSE	0.530	Very Good	0.114	Very Good
RSR	0.853	**Not Satisfactory**	0.184	Very Good
1/2 Standard Deviation Obs.	1	Satisfactory	3	Very Good

Note: Number of pair data (observed and simulated) = 37

6.4.11 Gage SG11 Pine Island West – Verification Results

The comparisons at SG11 are similar to those discussed for SG10. The modeled maximum stage matches the measured maximum stage (**Figure 6.46**), but the modeled recovery is much faster for Verification #1. Again, this is attributed to the unknowns related to the pump operation at Pine Island. When measured stages are compared to the modeled stages in Verification #2 (**Figure 6.47**), the modeled results compare much better. This is because of the internal boundary condition that was included at SG17 where water levels were set identical to those measured in the field. This approach forces stages downstream of SG11 to be consistent with the actual pump operation during the validation period. The modeled peak stage for Verification #2 (1.67-ft, NAVD88) is also within 0.03-ft of the measured peak stage.

Like the calibration analysis, node NPI1030 was converted to a time-stage node for the Verification #2 analysis. The comparisons for Verification #2 were included in this section to show how the stages upstream of the gage are fairly consistent with stages at the pump location (SG17 at node NPI1030). Additionally, this comparison demonstrates how sensitive the upstream stages are to the pumping operations at Pine Island.

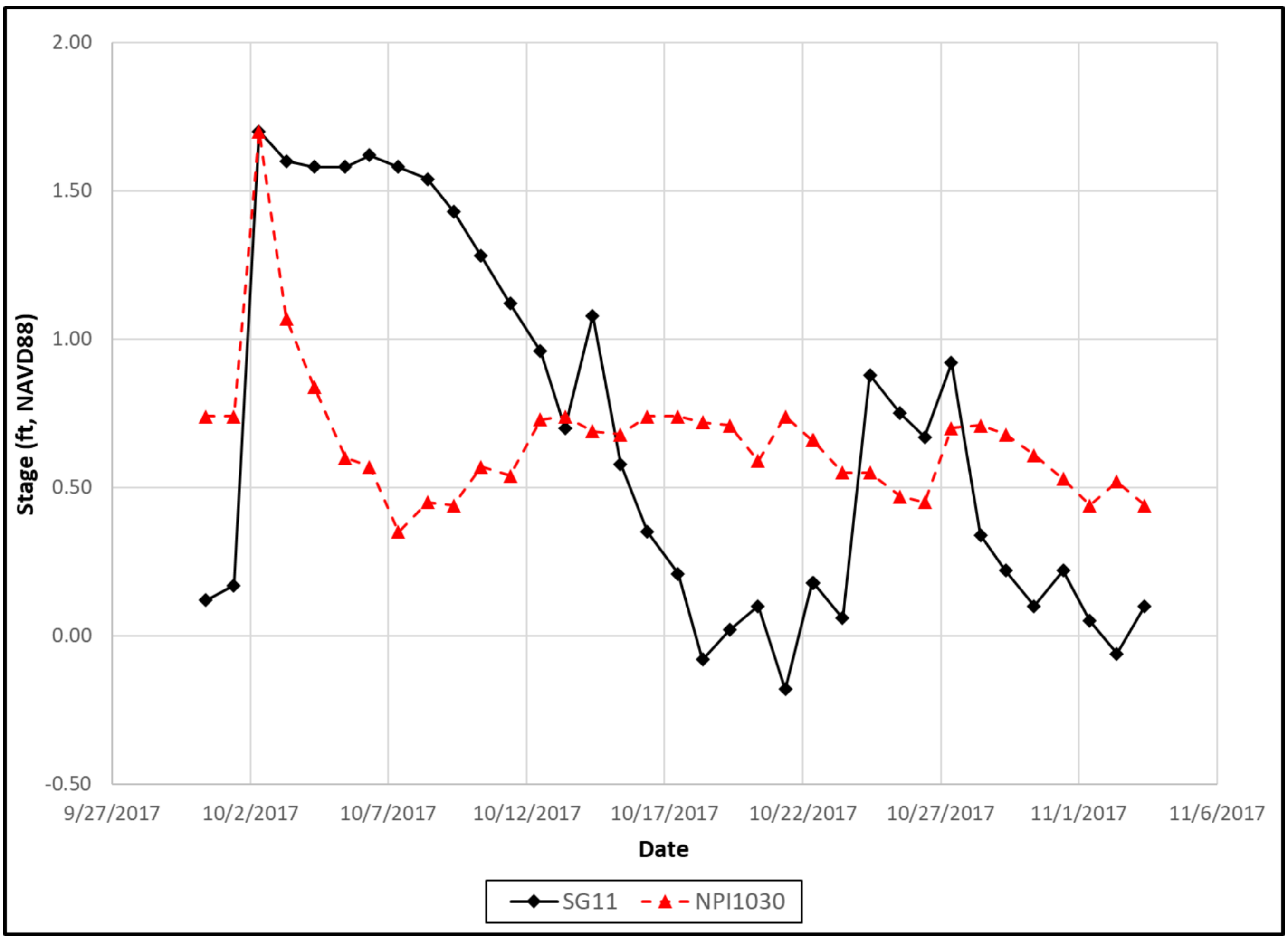


Figure 6.46: Gage SG11 Pine Island West Verification #1 Comparisons

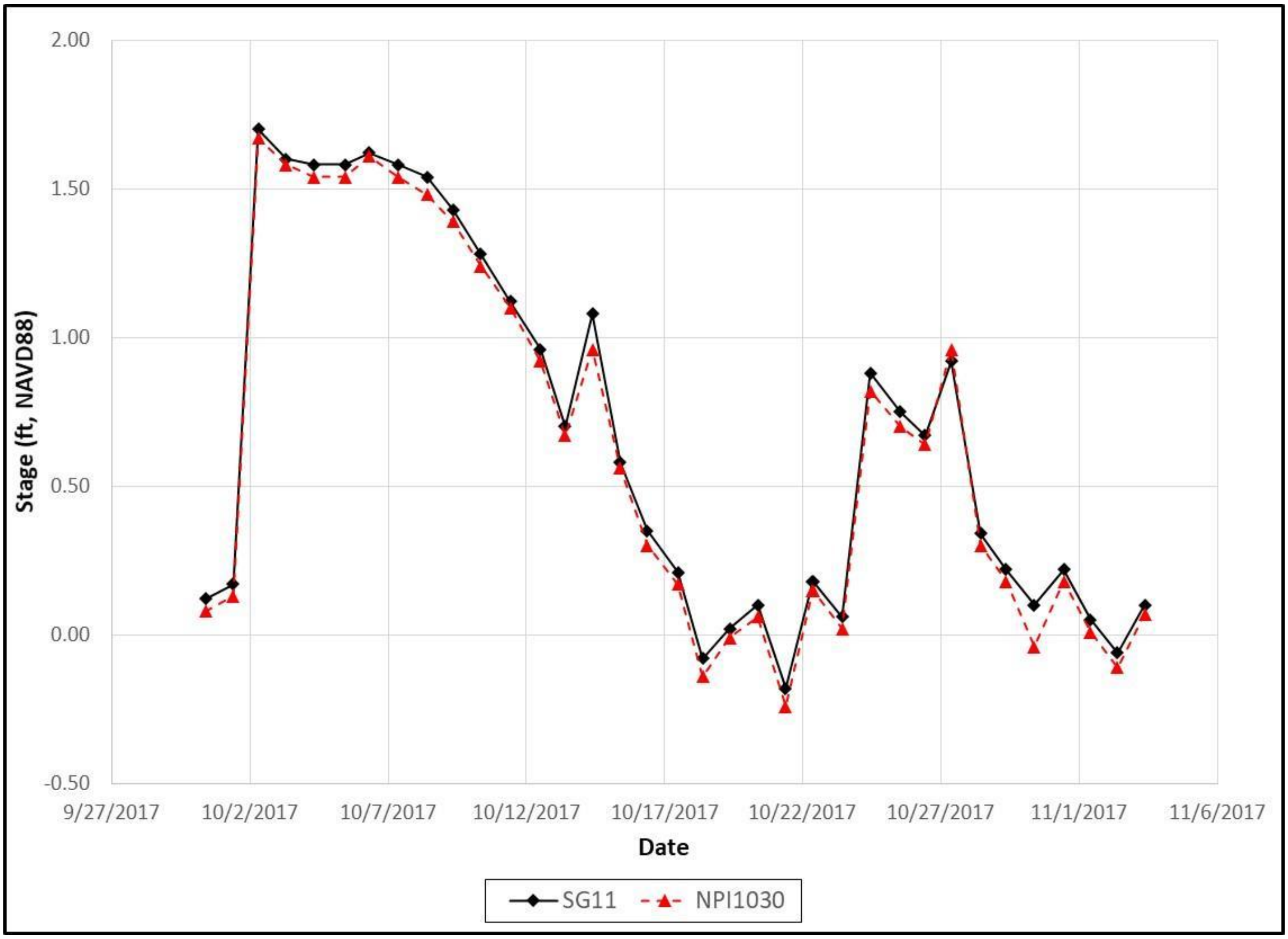


Figure 6.47: Gage SG11 Pine Island West Verification #2 Comparisons

Additionally, the statistical analysis (**Table 6.29**) shows that Verification #1 has three parameters that are not satisfactory. However, Verification #2 has very good correlation between the measured and modeled stages with the boundary condition at SG17 which is expected.

Table 6.29: Verification Statistical Metrics SG11

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.040	Not Satisfactory	0.998	Very Good
NSE	0.011	Not Satisfactory	0.993	Very Good
ME	0.000	Very Good	0.041	Very Good
MAE	0.531	Good	0.044	Very Good
RMSE	0.607	Very Good	0.050	Very Good
RSR	0.980	Not Satisfactory	0.080	Very Good
1/2 Standard Deviation Obs.	1	Satisfactory	3	Very Good

Note: Number of pair data (observed and simulated) = 37

6.4.12 Gage SG12 PICA South – Verification Results

The model results at SG12 are similar to those at SG11 for Verification #1. The modeled peak stages tend to agree extremely well with the measured peak stages (**Figure 6.48**), but the modeled recovery is much faster for Verification #1. Again, this is attributed to unknowns in the pump operation at Pine Island. At this gage location, the modeled peak stage (1.70-ft, NAVD88) is within an inch of the measured peak stage (1.64-ft, NAVD88). Additionally, the measured peak stage (0.90-ft, NAVD88) and modeled peak stage (0.70-ft, NAVD88) for the second storm event in October agree very well. Like the previous location, however, the pump operation at Pine Island reflected in the model data does not appear to be consistent with actual operation conducted during the validation period. This is also evident in the statistical comparisons shown in **Table 6.30**. Consequently, this location (Node: NPI1030) is one where internal boundary conditions were defined for Verification #2. As such, no comparisons are included for this location.

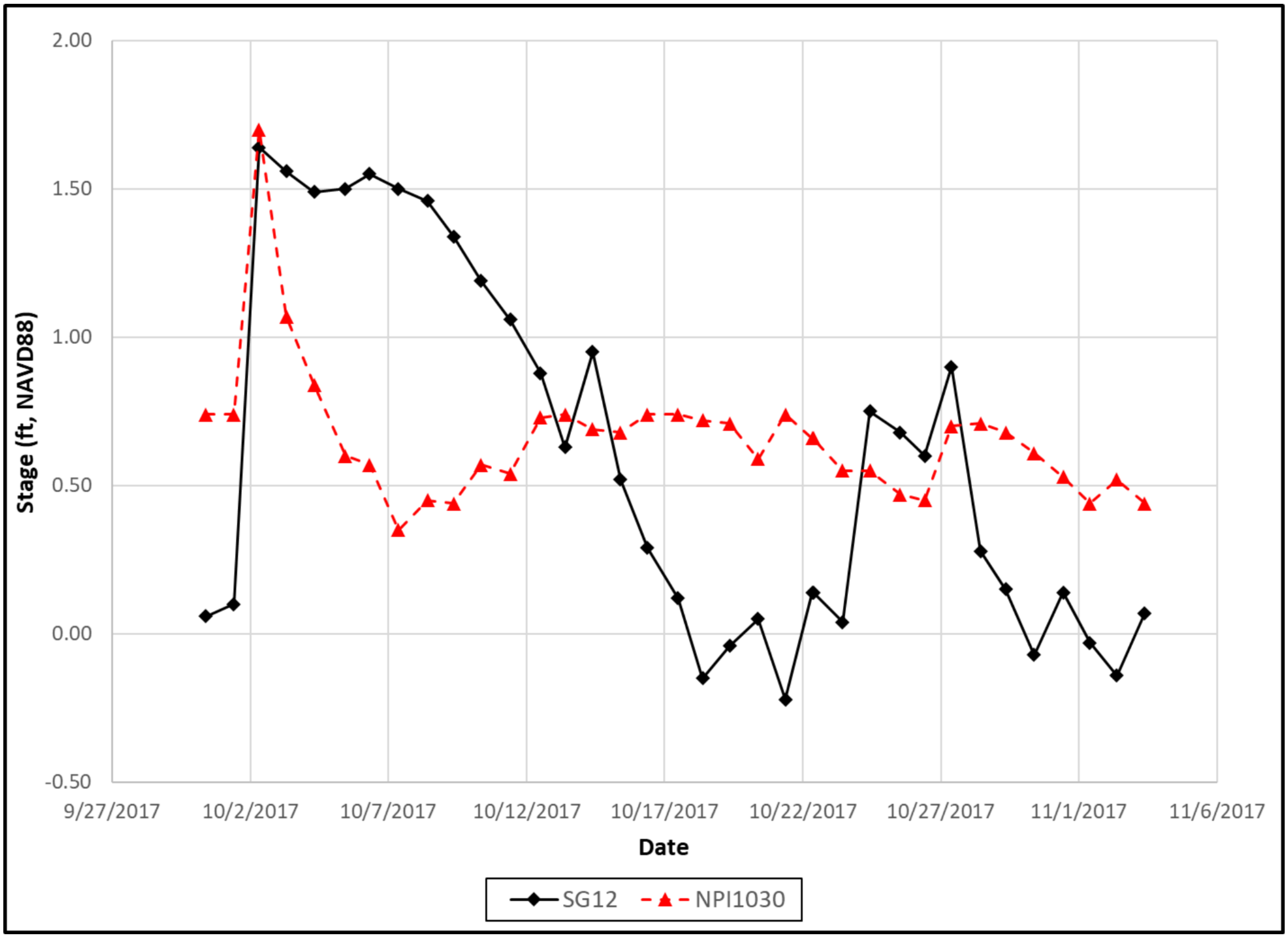


Figure 6.48: Gage SG12 PICA South Verification #1 Comparisons

Table 6.30: Verification Statistical Metrics SG12

Metric Parameter	Verification Simulation #1	Quality #1
R^2	0.043	**Not Satisfactory**
NSE	0.003	**Not Satisfactory**
ME	-0.070	Very Good
MAE	0.536	Good
RMSE	0.605	Very Good
RSR	0.984	**Not Satisfactory**
1/2 Standard Deviation Obs.	1	Satisfactory

Note: Number of pair data (observed and simulated) = 37

6.4.13 Gage SG13 W Hall Rd. West at N. Tropical Trail – Verification Results

Similar to the calibration results at this location, Verification #1 and Verification #2 results are nearly identical (**Figure 6.49** and **Figure 6.50**). Also, the modeled stages for both simulations tend to agree well with the measured stage data except during peak conditions. Statistically, however, the model adequately represents the measured data during the verification period for both verification simulations (**Table 6.31**).

It is recommended that model calibration in this area be revisited to incorporate current conditions at this location. As stated in the calibration analysis, the drainage system at this location was recently upgraded prior to the start of this study. Therefore, additional calibration/verification efforts are recommended to further improve the model accuracy at this gage location.

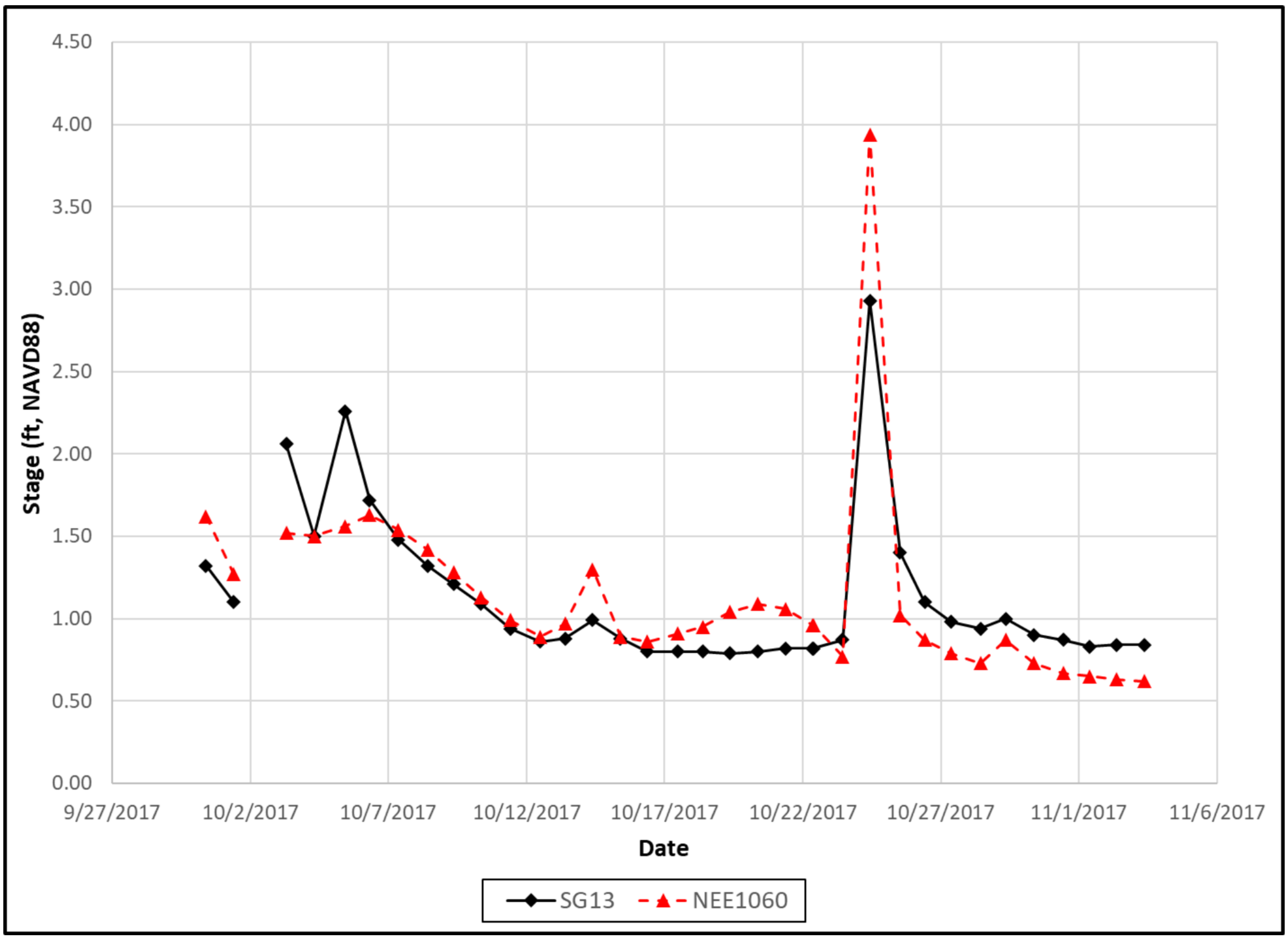


Figure 6.49: Gage SG13 W Hall Rd. West at N. Tropical Trail Verification #1 Comparisons

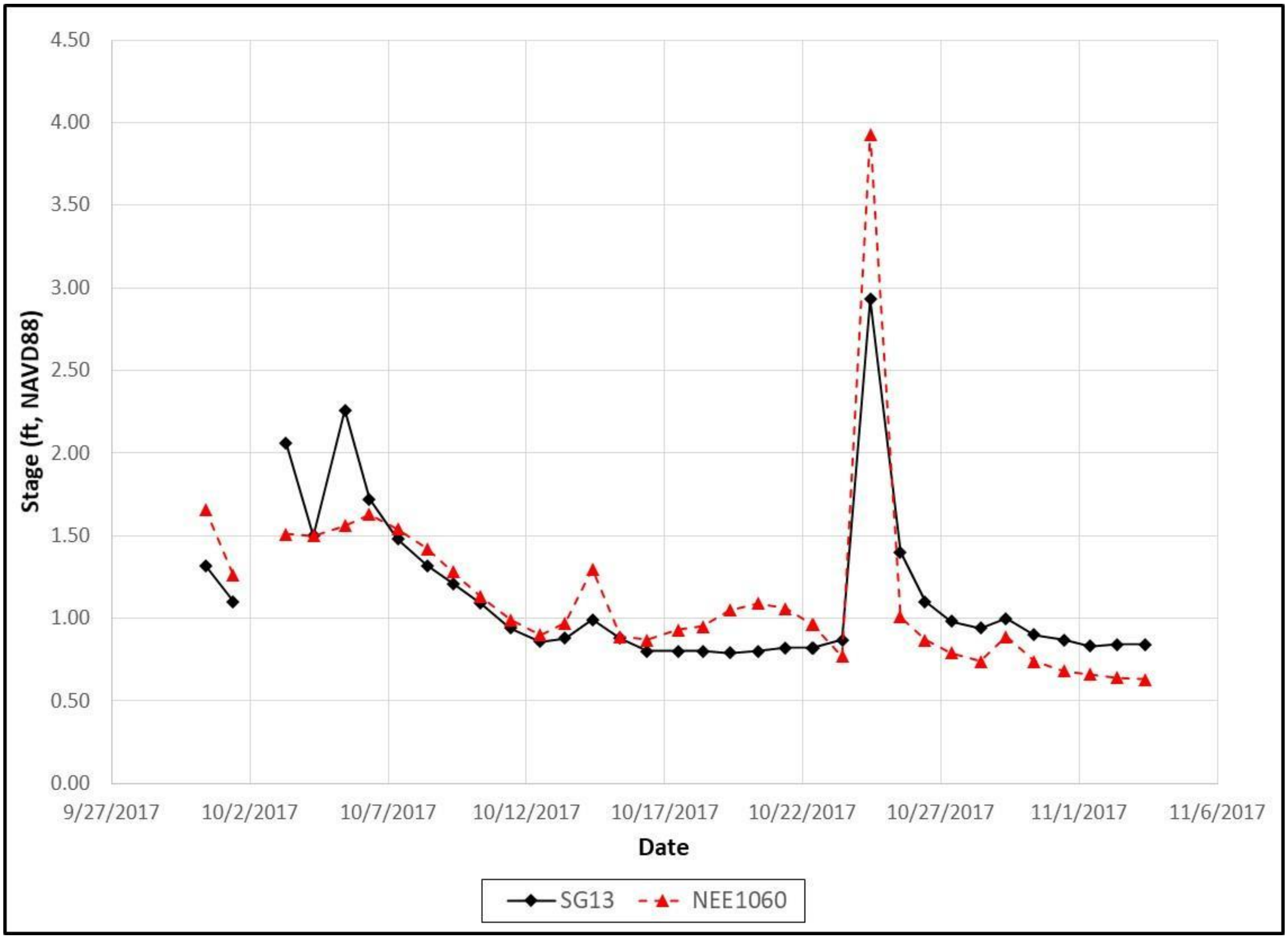


Figure 6.50: Gage SG13 W Hall Rd. West at N. Tropical Trail Verification #2 Comparisons

Table 6.31: Verification Statistical Metrics SG13

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.749	Satisfactory	0.746	Satisfactory
NSE	0.634	Satisfactory	0.634	Satisfactory
ME	-0.002	Very Good	-0.006	Very Good
MAE	0.205	Very Good	0.205	Very Good
RMSE	0.285	Very Good	0.285	Very Good
RSR	0.596	Good	0.596	Good
1/2 Standard Deviation Obs.	2	Good	2	Good

Note: Number of pair data (observed and simulated) = 36

6.4.14 Gage SG17 PICA Basin – Verification Results

Gage SG17 is in the immediate vicinity of SG12. Consequently, the comparisons between the measured and model data display similar behavior. The modeled peak stage for Verification #1 (1.70-ft, NAVD88) corresponds extremely well to the measured peak stage (1.67-ft, NAVD88). However, the modeled stage recovery tends to occur much more quickly than what was measured (**Figure 6.51**). As previously stated, the pump operation is based on best available information, but it appears to be inconsistent with actual field conditions which affects the recovery in the model. This is apparent in the statistical analysis shown in **Table 6.32** as well.

As previously discussed, this model location was converted to a time-stage node for use as an internal boundary for the Verification #2 analysis. The time-stage data were based on measured information at SG17. The result is flood staging that accounts for the actual pump operation.

This was done to determine if comparisons between the modeled and measured data at the gage locations affected by the Pine Island pump operation would improve. Note that no comparisons are included for this gage location for Verification #2 since this model location was converted to a time-stage node for the Verification #2 analysis.

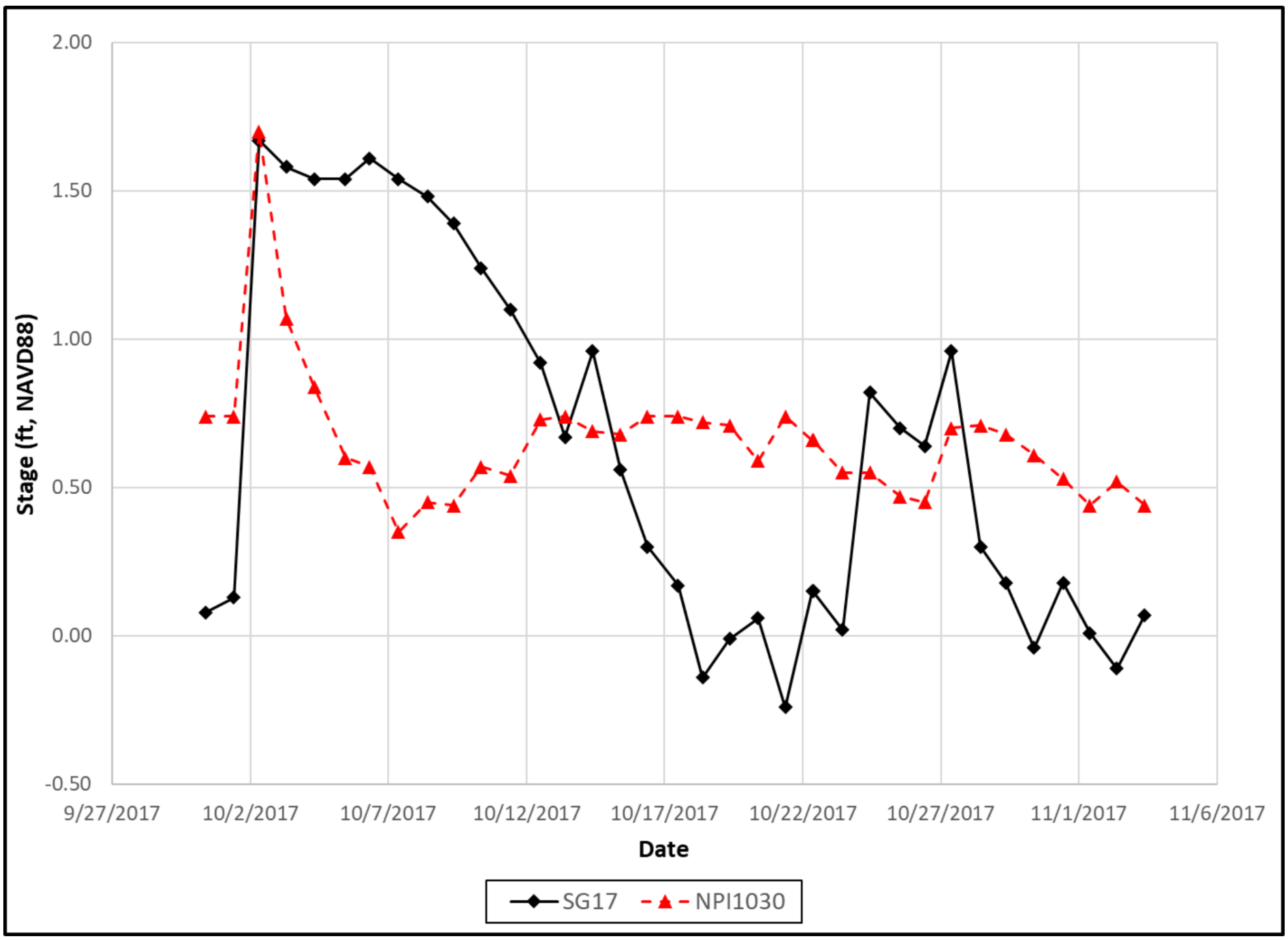


Figure 6.51: Gage SG17 PICA Basin Verification #1 Comparisons

Table 6.32: Verification Statistical Metrics SG17

Metric Parameter	Verification Simulation #1	Quality #1
R^2	0.041	**Not Satisfactory**
NSE	0.010	**Not Satisfactory**
ME	-0.041	Very Good
MAE	0.541	Good
RMSE	0.613	Very Good
RSR	0.981	**Not Satisfactory**
1/2 Standard Deviation Obs.	1	Satisfactory

Note: Number of pair data (observed and simulated) = 37

6.4.15 Gage SG18 PICA Riverside – Verification Results

Modeled stages for both Verification #1 and Verification #2 compare extremely well to the measured stages at gage SG18 as shown in **Figure 6.52** and **Figure 6.53**. The peak stages for Verification #1 (1.59-ft, NAVD88) and Verification #2 (1.62-ft, NAVD88) are nearly identical to the measured peak stage (1.60-ft, NAVD88) for both simulations.

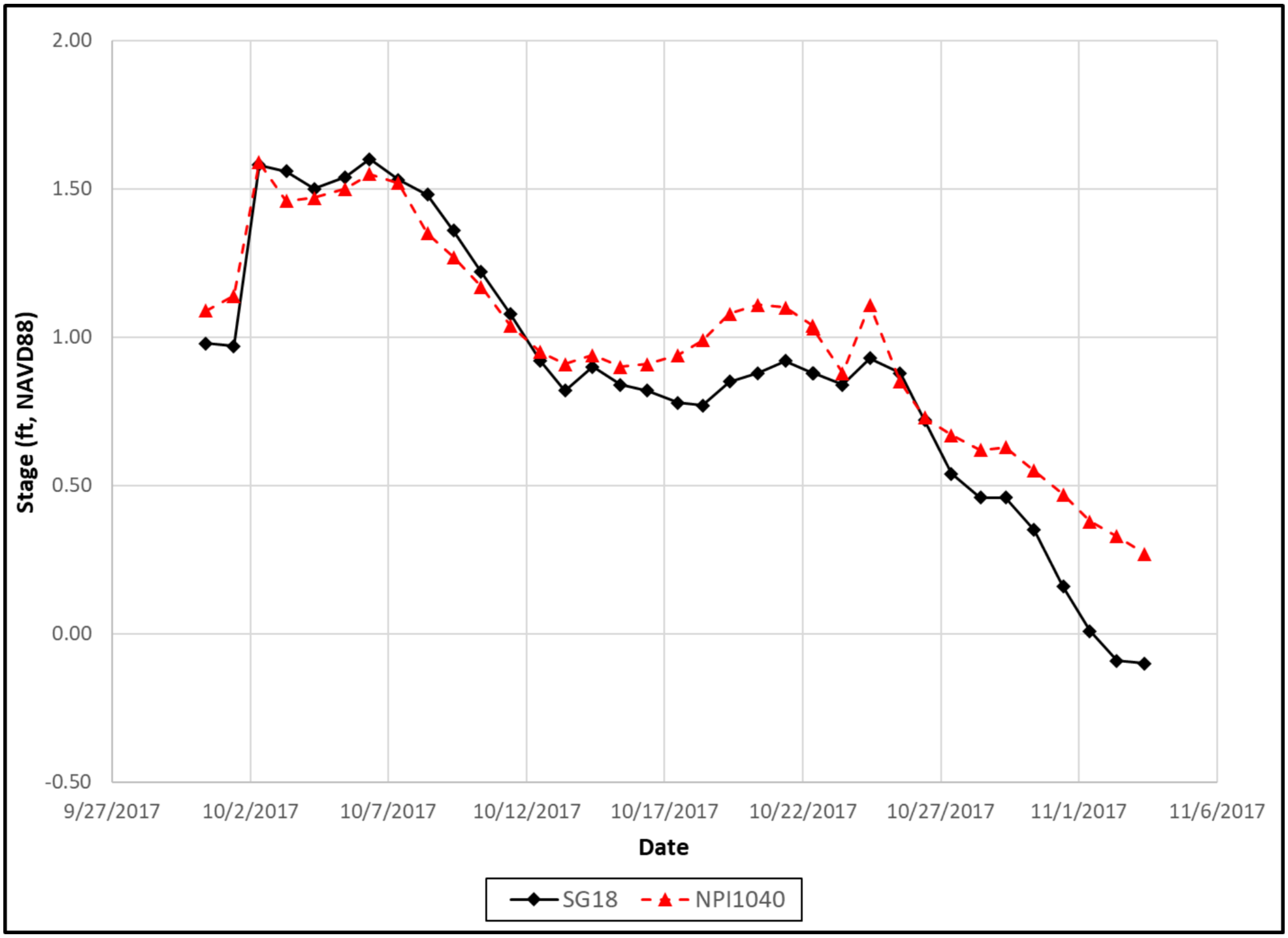


Figure 6.52: Gage SG18 PICA Riverside Verification #1 Comparisons

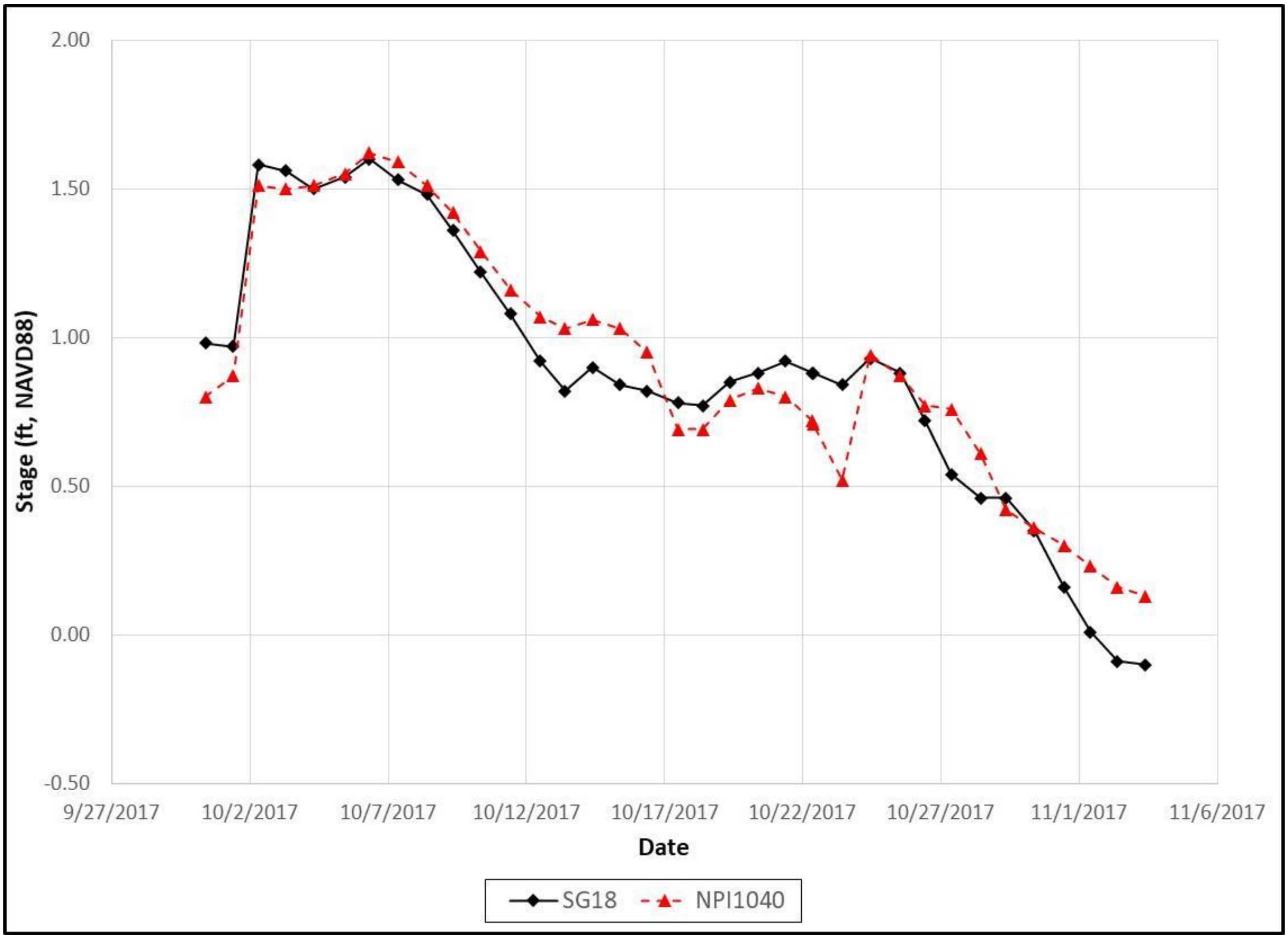


Figure 6.53: Gage SG18 PICA Riverside Verification #2 Comparisons

Additionally, the statistical results in **Table 6.33** show that both verification simulations compare well with the measured data. All the statistical parameters are classified as very good based on the metrics used for the study.

Table 6.33: Verification Statistical Metrics SG18

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.959	Very Good	0.915	Very Good
NSE	0.858	Very Good	0.911	Very Good
ME	-0.103	Very Good	-0.026	Very Good
MAE	0.135	Very Good	0.110	Very Good
RMSE	0.172	Very Good	0.136	Very Good
RSR	0.372	Very Good	0.295	Very Good
1/2 Standard Deviation Obs.	3	Very Good	3	Very Good

Note: Number of pair data (observed and simulated) = 37

6.4.16 Gage SG19 PICA North – Verification Results

Model results for Verification #1 (**Figure 6.54**) and Verification #2 (**Figure 6.55**) compare extremely well to the measured data at gage SG19. Peak stages for Verification #1 (1.87-ft, NAVD88) and Verification #2 (1.79-ft, NAVD88) are both within 1.9-inches or less of the measured peak stage (1.71-ft, NAVD88). Verification #1, however, tends to compare much better to the measured data based on the very good correlation shown in **Table 6.34**. This may be partly related to using the estimated Pine Island pump operations while also specifying the time-stage data at the upstream end of the pump for the Verification #2 simulation. Regardless, both simulations are representative of the actual field conditions during the validation period based on the statistical metrics (**Table 6.34**).

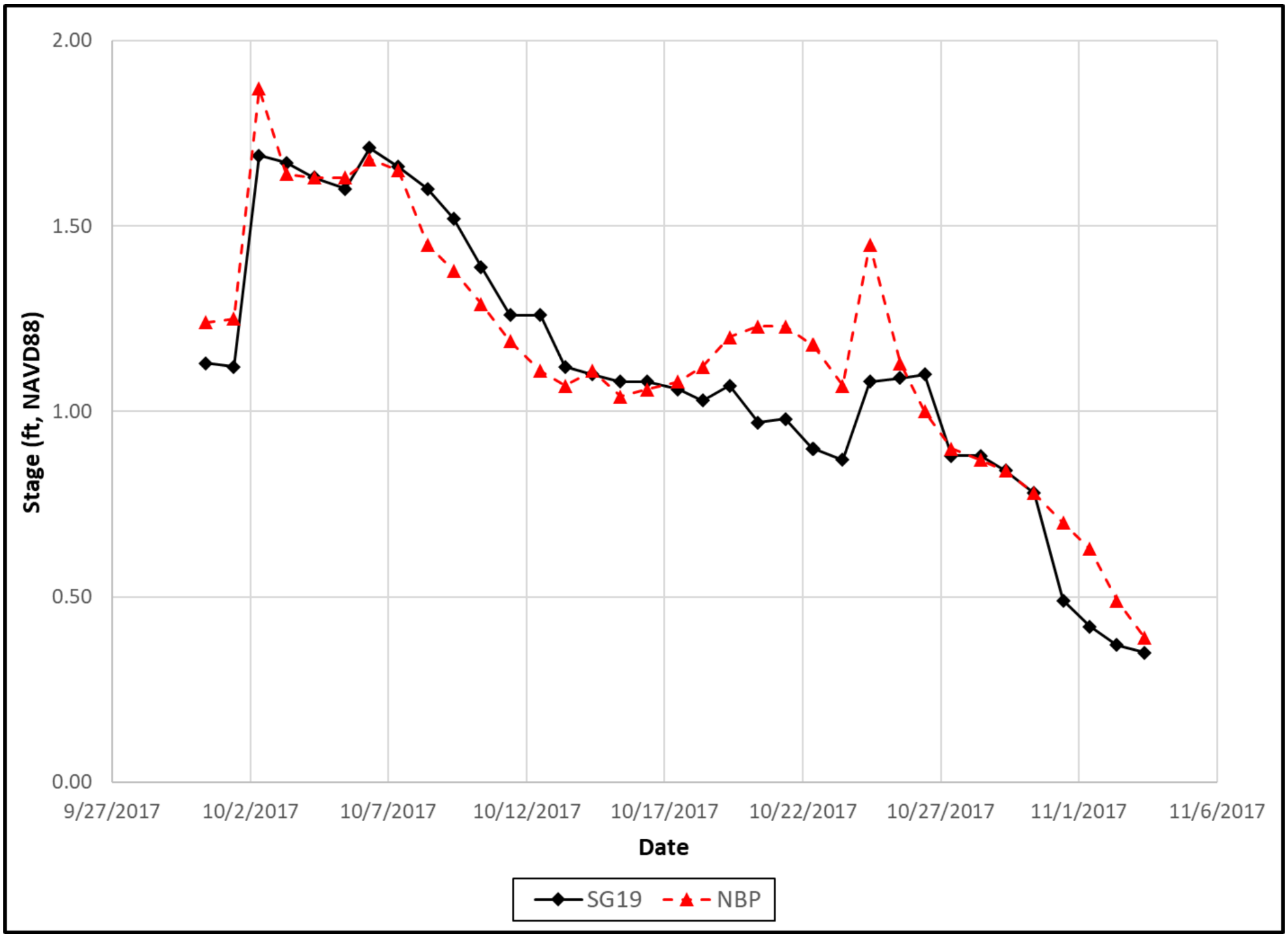


Figure 6.54: Gage SG19 PICA North Verification #1 Comparisons

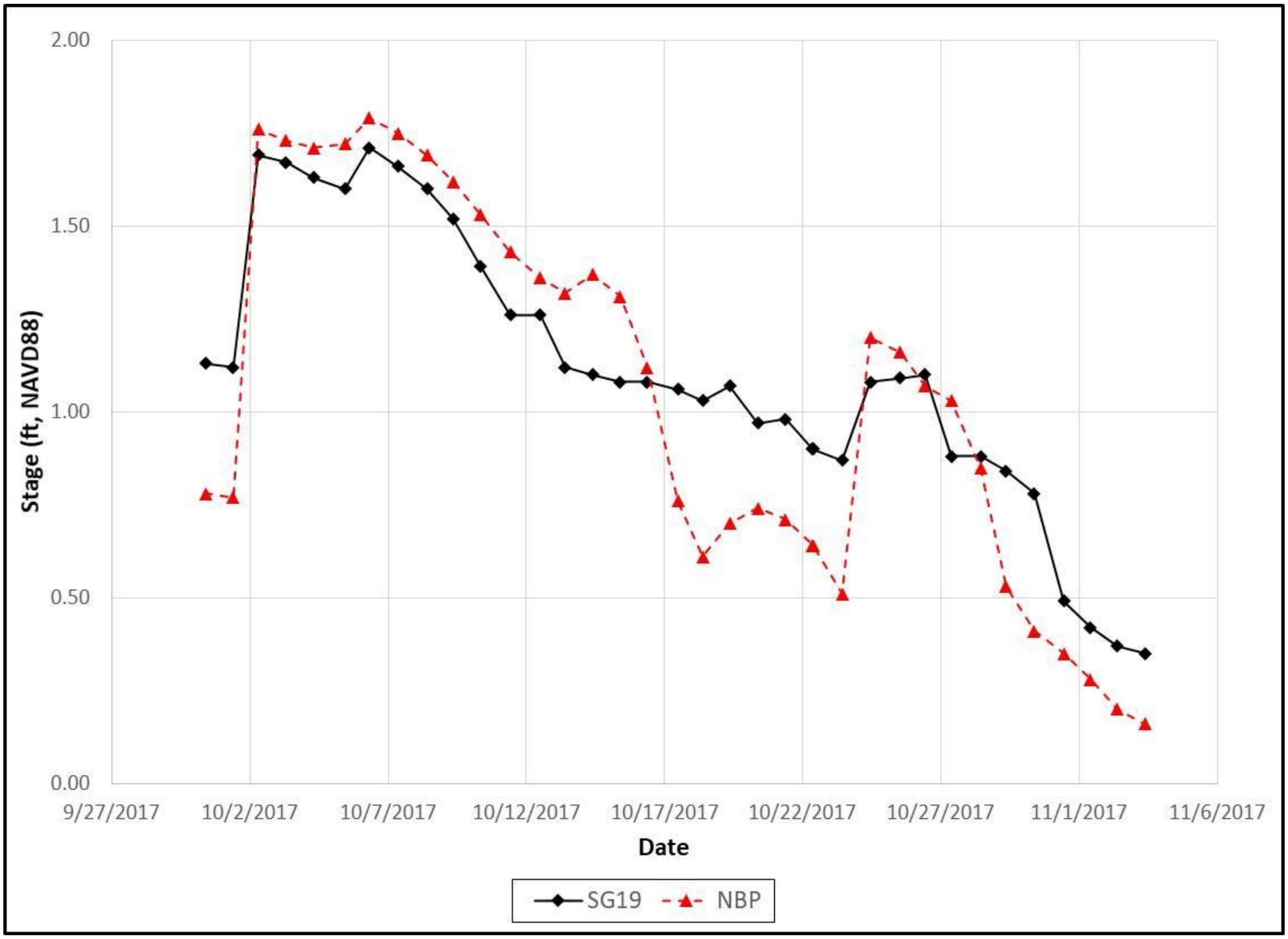


Figure 6.55: Gage SG19 PICA North Verification #2 Comparisons

Table 6.34: Verification Statistical Metrics SG19

Metric Parameter	Verification Simulation #1	Quality #1	Verification Simulation #2	Quality #2
R^2	0.869	Very Good	0.871	Very Good
NSE	0.844	Very Good	0.646	Satisfactory
ME	-0.058	Very Good	0.066	Very Good
MAE	0.108	Very Good	0.187	Very Good
RMSE	0.145	Very Good	0.218	Very Good
RSR	0.390	Very Good	0.587	Good
1/2 Standard Deviation Obs.	3	Very Good	1	Satisfactory

Note: Number of pair data (observed and simulated) = 37

6.5 Calibration / Verification Conclusions

In general, the model results compare very well to the measured data at most of the gage locations for the calibration and verification simulations. The model peak stages, in particular, agree extremely well with measured peak stages at every gage location except SG13. There are some inconsistencies with the recovery behavior of the stage hydrographs at some gage locations for the Calibration #1 and Verification #1 simulations. However, Calibration #2 and Verification #2 model results generally compare much better with the measured data in most cases. The improvements shown in the Calibration #2 and Verification #2 results suggests unknown operations at East Hall Rd. Pumps and Pine Island Harvey Grove Pumps are the cause of the discrepancies.

Lastly, it is recommended that further calibration and verification analyses be conducted at SG13 using current site conditions and storm events subsequent to the stormwater improvements to improve the model comparisons. Also, it is recommended that further calibration be conducted during the dry season to more accurately represent the study area during dry conditions, if possible.

7.0 Existing Conditions Analysis

This section of the report details the work performed as part of the existing conditions analysis of the NMI Watershed, including existing conditions model updates, critical duration analysis, design storm simulation, and floodplain development. This section also presents a discussion on the results of the H&H analysis.

7.1 Existing Conditions Model Updates

Post-2017 Improvements: The model used for calibration/verification was based on conditions at the time of Hurricane Irma in September 2017. To conduct the existing conditions analysis, model updates were performed to incorporate post-2017 improvements. A summary of these improvements and the affected model features is included in **Table 7.1** below.

Table 7.1: Summary of Drainage Structure Updates for Post-2017 Improvements

Description of Update	Model Feature Changes
Pine Island temporary pump in place for Hurricane Irma removed from the model	Link Flow set to NONE: PICA_Temp
Pine Island Pumps Operating Tables simulating manual drawdown pre-Irma	Operating Tables NB.p1_Drawdown and NB.p2_Drawdown were deleted from the model
Hall Road temporary pump in place for Hurricane Irma removed from the model	Link Flow set to NONE: Hall_Temp_Pump
Hall Road Pump Station Improvements (Reference Document NMI_020)	Updated Pump Links: RO6 and RO6A and their associated operating tables Added new Hall Rd. Cross Culvert Link: PG1840_3
W. Hall Road Outfall Improvements (Reference Document NMI_074)	New/Updated Links: PEE1060 and PEE2061
Bottom Clip Table simulating 2017 operations schedule of flashboards for control structures discharging to the Barge Canal was removed	Bottom Clip Tables Removed for Links: DSykesS_1, DSykesS_2, DSykes_3, L-3470DS, L-3480DS, L-3520DS, L-3530DS, L-3240DS, L-3550DS, L-3560DS, L-3610DS, L-3620DS

Boundary Conditions: In addition to incorporating post-2017 improvements, the boundary stage data were also updated for the design storm simulations. Boundary data in the calibration/verification model simulations were based on analysis of tailwater conditions throughout 2017, as discussed in previous sections of this report. For existing conditions and design storm simulations, a constant tailwater condition was used based on the Mean Higher High Water (MHHW) elevation at the Trident Pier NOAA gage (Gage 8721604) located in Port Canaveral, Florida (See **Figure 7.1**). The Mean Higher-High Water elevation (el. 1.10 NAVD) was chosen as the tailwater elevation, as it most closely correlated to historical stages observed in the IRL. This value is also in-line with data from the 2017 tailwater analysis, which show an estimated peak stage in the Banana River of 1.06-feet during Hurricane Irma. Refer to **Appendix C** for more information on the tailwater analysis.

Node Initial Water Conditions: Initial water elevations were revised based on the updated model network and new boundary conditions. Initial stages for groundwater nodes, overland flow nodes, and 1D nodes are based on a “hot start” simulation. Preliminary initial water elevations were set at 1D Nodes based on the tailwater elevation of 1.10-ft. Preliminary initial water elevations for groundwater and overland flow nodes were based on the model results from the calibration model for the date August 1, 2017. Using these elevations, the hot start simulation was then run. The hot start simulation for the existing conditions model starts at time = 0 and ends at time = 40 hours. The Hall Road and Pine Island pumps are operational during the hot start simulation, as the County has stated that drawdown at these pump stations 24-hours prior to a forecasted storm event is part of their standard emergency operating protocol. The results of the hot start simulation at 24 hours were extracted and used to specify the final initial stages for the existing conditions model.

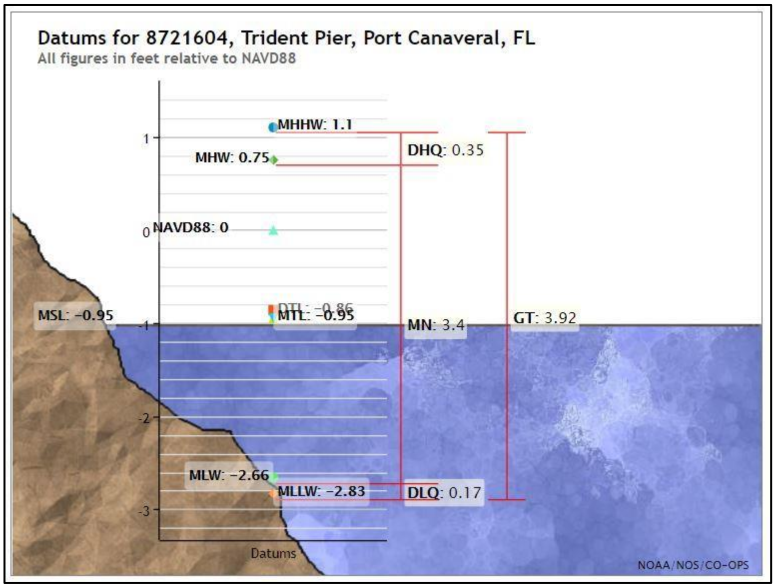


Figure 7.1: NOAA Trident Pier Gage Datum Information

Overland Flow Weirs: Upon completion of the model updates discussed above, the model was simulated using the 100-year storm event and 8 critical durations, as discussed in **Section 7.2** below. The peak stages at each node were used to generate floodplain polygons which were reviewed in detail for any "glass walls" within 1D areas or between control volumes. 1D overflow weir links were added at each identified glass wall to allow flow between the 1D basins and/or control volumes.

7.2 Critical Duration Analysis

A critical duration storm analysis was performed for the NMI watershed through evaluation of its responses to storms of varying duration and return frequencies. The critical duration storm is defined for this study as the duration that produces the highest flood stages throughout the study area. A total of 48 storm events were simulated with durations of 1, 2, 4, 8, 24, 72, 96, 168, and 240 hours for each return frequency of Mean Annual, 5, 10, 25, 50, and 100-year, consistent with the FDOT critical duration approach. Rainfall volumes for the critical duration storm events were obtained from NOAA Atlas-14 Precipitation Frequency Estimates at the approximate center of the watershed. The rainfall amounts used for each simulation are presented in **Table 7.2** below.

Table 7.2: Critical Duration Storm Rainfall Amounts (inches)

Duration	Mean	5-Year	10-Year	25-Year	50-Year	100-Year
1 Hour	2.20	2.65	3.01	3.50	3.88	4.25
2 Hour	2.74	3.29	3.74	4.35	4.81	5.28
4 Hour	3.20	3.89	4.47	5.29	5.95	6.62
8 Hour	3.71	4.65	5.47	6.69	7.70	8.77
24 Hour	4.68	6.10	7.44	9.51	11.3	13.2
72 Hour	5.98	7.59	9.13	11.5	13.6	15.9
168 Hour (7 day)	7.62	9.22	10.8	13.1	15.2	17.5
240 Hour (10 day)	8.67	10.3	11.9	14.3	16.4	18.6

Each return frequency was evaluated and the storm duration producing the highest peak stage at each 1D and 1D Interface Node was identified. In general, the 24-hour storm event was found to produce the highest peak stages for the majority of the watershed, with the exception of smaller storm events (5-year and Mean Annual), where the critical duration was found to be 8-hours. The results of the critical duration storm analysis are summarized in **Table 7.3** below, which presents the quantity of nodes experiencing peak stage for each critical duration event. For example, for the 100-year storm event, the peak stage produced during the 24-hour duration exceeded the peak stage produced during the other durations for 705 nodes. As a result, the 24-hour storm is considered the critical duration for those nodes.

Table 7.3: Critical Duration Storm Analysis Summary

Quantity of Nodes with Highest Stages for Each Critical Duration Event

Duration	Mean	5-Year	10-Year	25-Year	50-Year	100-Year
1 Hour	212	199	186	160	140	129
2 Hour	54	51	34	26	19	18
4 Hour	365	186	85	29	14	13
8 Hour	391	479	495	439	389	380
24 Hour	244	418	553	671	740	705
72 Hour	14	27	32	30	28	97
168 Hour (7 day)	12	21	13	12	12	12
240 Hour (10 day)	191	96	33	17	13	12

Lowest value *Highest value*

Notes: 1. "Nodes" includes both 1D Nodes and 1D Interface Nodes.
2. Some nodes have more than 1 critical duration identified and are counted under each identified duration. For example, some nodes predicted equal peak stages for the Mean Annual 4-hr and Mean Annual 8-hr, so those nodes are counted twice under the Mean Annual storm, once for the 4-hr and once for the 8-hr duration. As such, node totals may not be equal for each return frequency.

The critical duration was also found to vary spatially across the watershed, appearing to be primarily related to landuse and ground elevation. For each return frequency, the 24-hour duration produced the highest stages for nodes located in undeveloped, wetland, low-lying, and rural areas of the watershed, while the majority of nodes located in urbanized and high elevation areas experienced an 8-hour critical duration. **Figures 7.2, 7.3 and 7.4** depict the spatial distribution of critical durations for the 10-year, 25-year, and 100-year return frequencies, respectively.

Peak stages for all critical duration storm simulations (1D and 1D Interface nodes) are included in the electronic deliverables accompanying this report, under "Support Data".

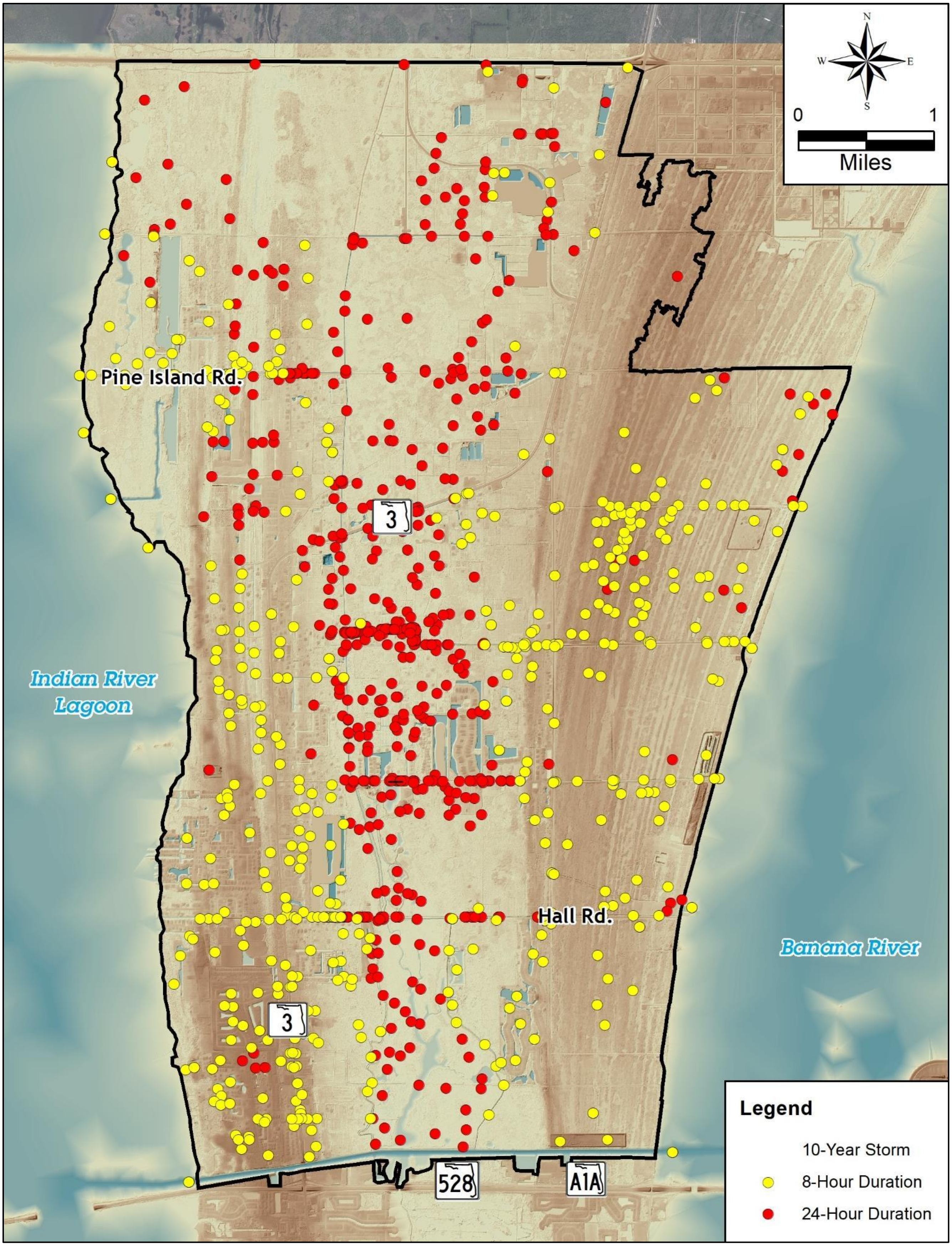


Figure 7.2: Critical Duration Analysis (10-year event)

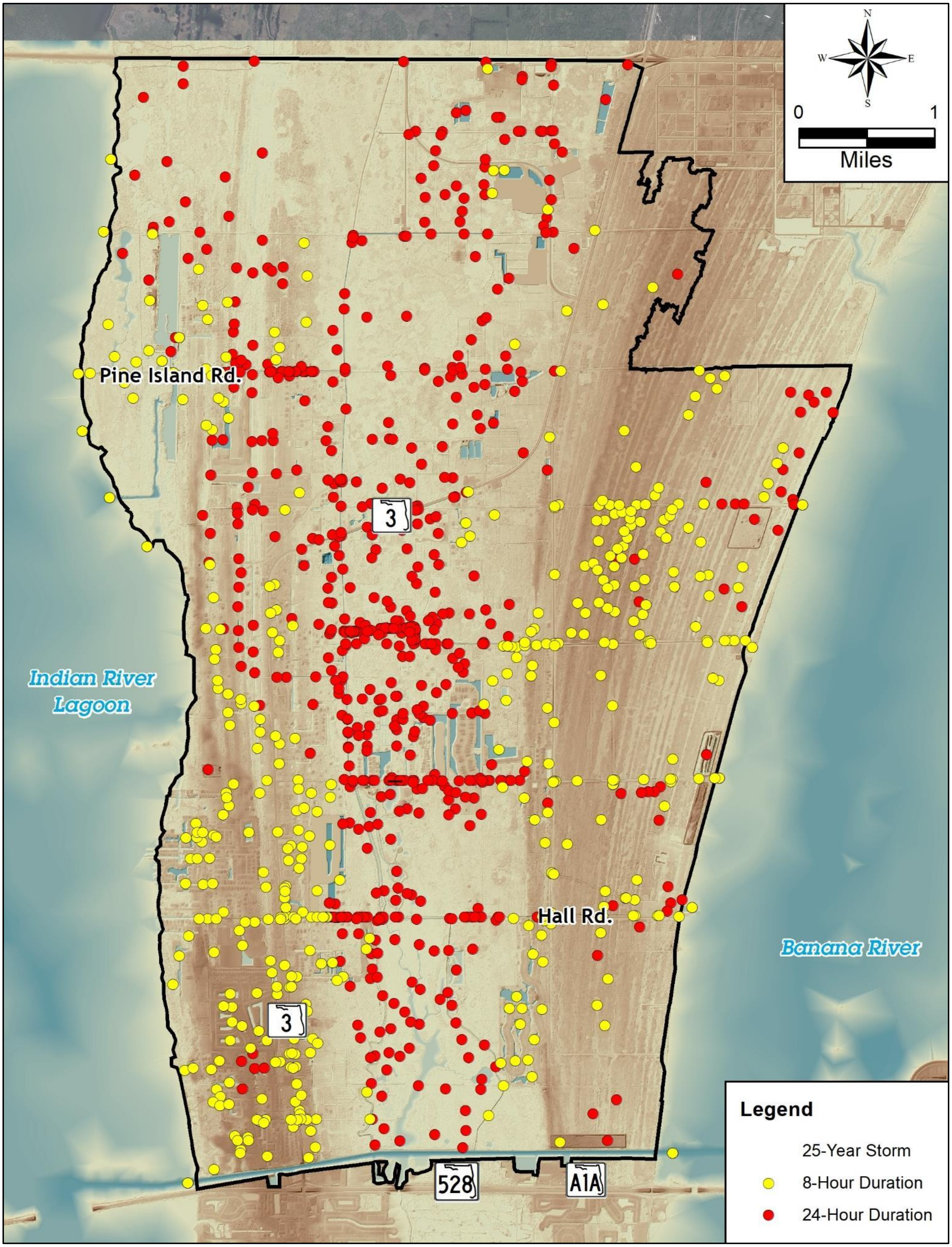


Figure 7.3: Critical Duration Analysis (25-year event)

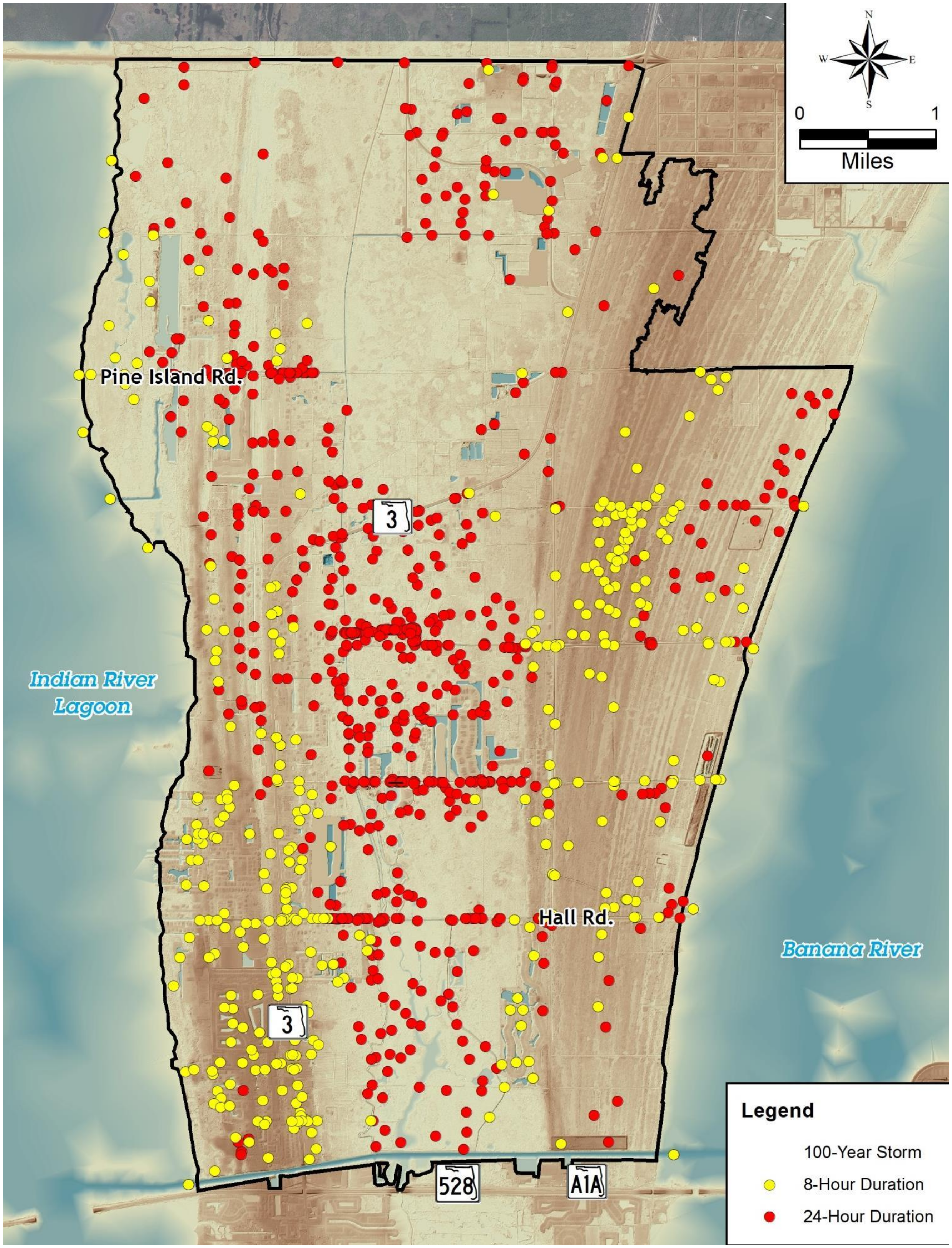


Figure 7.4: Critical Duration Analysis (100-year event)

7.3 Existing Conditions Analysis and Floodplain Generation

7.3.1 Design Storm Simulations: Nine design storms were simulated to evaluate the existing conditions throughout the NMI watershed. The 24-hour duration event was simulated for return frequencies of Mean Annual, 10, 25, 50, and 100-years. For larger storm events (25-year and 100-year), the 96-hour storm was also simulated based on SJRWMD permit requirements for landlocked systems, along with the 72-hour storm event. Rainfall amounts were obtained from SJRWMD, NOAA Atlas-14, and Brevard County's Land Development Code (LDC) and reviewed to determine the highest rainfall amount for each simulated storm event based on present-day engineering literature. NOAA Atlast-14 rainfall amounts were based on the approximate center of the watershed. Rainfall amounts and distributions for each simulated storm event are shown in **Table 7.4** below.

Table 7.4: Design Storm Rainfall Amounts (inches)

Return Frequency	Storm Duration (hr)	Rainfall Distribution	Rainfall NOAA Atlas-14	Rainfall SJRWMD	Rainfall Brevard Co. LDC	Rainfall Amount Used	Source Used
Mean Annual	24	SCS Florida Modified	4.68	5.0	-	**5.0**	**SJRWMD**
10-Year	24	SCS Florida Modified	7.44	7.75	7.9	**7.9**	**Brevard LDC**
25-Year	24	SCS Florida Modified	9.5	9.5	9.0	**9.5**	**SJRWMD / NOAA Atlas-14**
50-Year	24	SCS Florida Modified	11.3	-	-	**11.3**	**NOAA Atlas-14**
100-Year	24	SCS Florida Modified	13.2	13.0	11.0	**13.2**	**NOAA Atlas-14**
25-Year	72	SFWMD-72	11.5	-	-	**11.5**	**NOAA Atlas-14**
100-Year	72	SFWMD-72	15.9	-	-	**15.9**	**NOAA Atlas-14**
25-Year	96	SJRWMD-96	12.1	12.5	12.5	**12.5**	**SJRWMD**
100-Year	96	SJRWMD-96	16.5	17.0	-	**17.0**	**SJRWMD**

7.3.2 Floodplain Mapping: Floodplains were developed for all nine design storm simulations. These floodplains were developed using two methods: ❶ Within the 1D areas (1D basins, pond control volumes, and channel control volumes), level-pool floodplains were mapped based on the maximum stage for the node assigned to that feature, and ❷ Within the 2D areas, floodplains were mapped based on the *Two-Dimensional Overland Flow Floodplain Development* document prepared by Streamline Technologies, Inc. which uses surfaces generated in ICPR4 based on a 0.25-ft (3-inch) flood depth threshold. This process uses the maximum elevation animation to generate a surface in ICPR4 and compares that surface to the project DEM and a "ground" DEM based on the ICPR4 triangular mesh. The resulting floodplains were processed to remove "spackle" areas (floodplain polygons less than 2,500-ft^2 in size). Note that a feature class named "New_Development_Areas" was included in the "BaseMap" geodatabase deliverable. This feature class identifies locations where new developments have been constructed since the original DEM's LiDAR collection date. Modifications have been made to the DEM in these locations to account for storage and groundwater interaction based on the best available data (as discussed in **Section 3**). However, the DEM is not completely representative of the current ground surface in these areas. Therefore, floodplains within these areas have been excluded. Floodplain graphics for the 10, 25, and 100-year 24-hour storm events, along with the 25 and 100-year 96-hour storm events are shown in attached **Exhibits 3, through 7** that accompany this report. Floodplains for all nine simulated design storm events are provided with the electronic deliverables that accompany this report.

8.0 Discussion

8.1 Floodplain Discussion

Design Storm Simulations: The floodplains for the NMI watershed depict significant inundation for most simulated storm events, particularly in the low-lying central wetland areas of the watershed. This includes the Sykes Cree/ mosquito impoundment area north towards East Crisafulli Road. Roadway flooding is shown along several collector roads during the 10-year storm event, including East Crisafulli Road (**Figure 8.1**) and West Crisafulli Road (**Figure 8.2**).

Figure 8.1: 10-Year, 24-hr Floodplain along E. Crisafulli Rd.

Figure 8.2: 10-Year, 24-hr Floodplain along W. Crisafulli Rd.

Both locations above, along with other areas such as East Hall Road below (**Figure 8.3**), are expected to experience significant inundation of low-lying yards and driveways during the 10-year event.

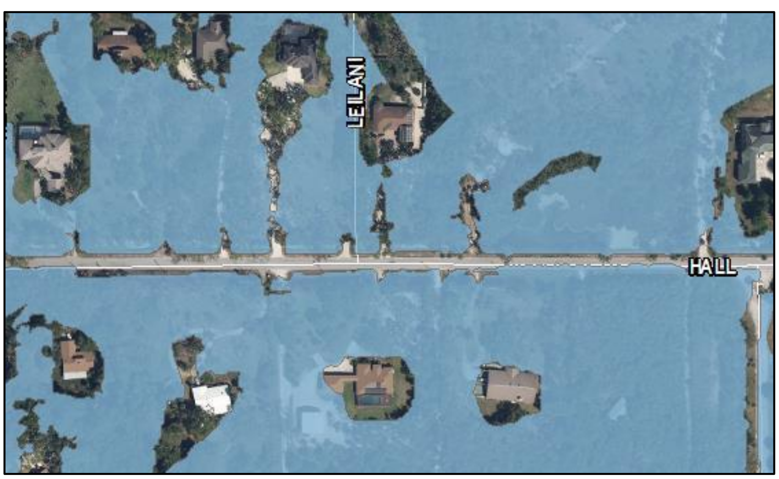


Figure 8.3: 10-Year, 24-hr Floodplain along E. Hall Rd.

It should be noted that, while the above instances highlight a few observed areas of concern, a complete level of service evaluation of the watershed was not conducted as part of this study.

FEMA Comparison: The 100-year, 24-hour floodplains were compared to the effective Federal Emergency Management Agency (FEMA) floodplains for the watershed (Effective 01/29/2021). The floodplains developed for this study are largely in agreement with the FEMA floodplains where mapped. The current study's floodplains also include areas not previously mapped by FEMA, including much of the area south of West Hall Road and west of State Road 3, and the relic dunes along the east side of the watershed that border the Banana River. As discussed in **Section 7** of this report, areas of new development were excluded from the floodplains. In total, the 100-year, 24-hour floodplains developed for this study removed about 850-acres of floodplain area compared to the effective FEMA floodplains and added over 4600-acres of floodplain area. **Figure 8.4** below presents a visual depiction of the developed NMI floodplains compared to the Effective FEMA floodplains.

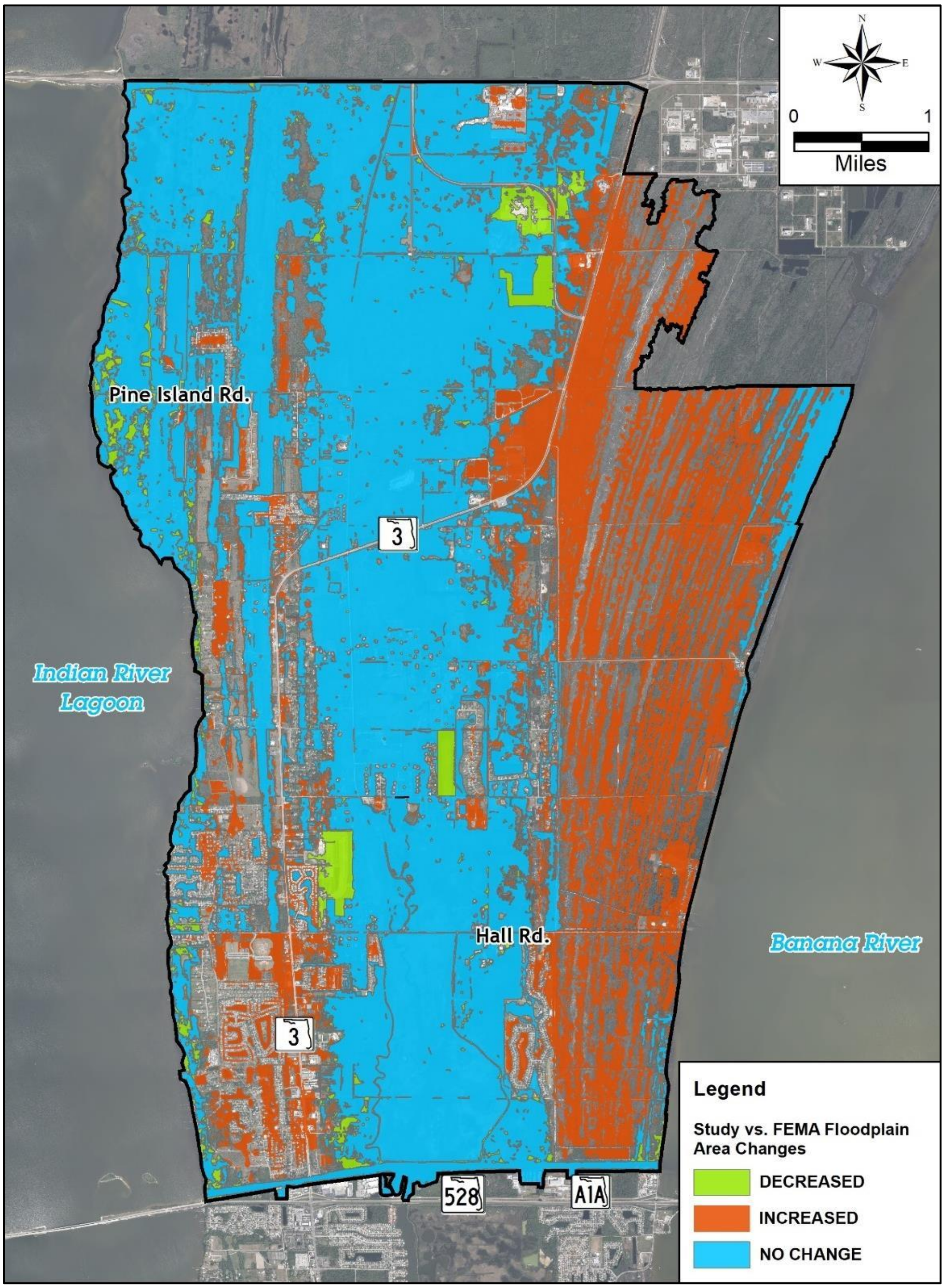


Figure 8.4: FEMA vs. 100-year, 24-hr Floodplain Comparison

8.2 Groundwater Discussion

Groundwater in the NMI watershed is affected by both rainfall and backwater effects from the IRL and Banana River. During large storm events, soils in low-lying areas become saturated and groundwater levels rise to the ground surface becoming ponded surface water. In these instances, recovery of the groundwater is achieved through evapotranspiration and lateral seepage into adjacent outfall canals. This scenario is reflected in the calibration model results for Hurricane Irma.

In the plan and profile shown below (**Figure 8.5**), the modeled groundwater within the area south of Hall Road can be seen rising above the ground surface during Hurricane Irma. The predicted high groundwater levels in this area are consistent with observations made by both residents and County staff for this area.

Figure 8.5: Plan and Profile of Groundwater Model Results at Hall Road

Figure 8.6 below presents the time-series data for several groundwater nodes located north of Hall Road, each located in low-lying areas where the groundwater levels exceeded or nearly exceeded the ground surface during Hurricane Irma and the October rain events that followed. In this graph, groundwater nodes 15366 and 15379 are both located within 75-feet of a roadside canal which connects to the IRL, and groundwater node 13444 is located over 2,000-feet away from the canal bank. The time series data presented shows the recovery of these groundwater nodes after Hurricane Irma and the October rain events. Groundwater recovery at the two nodes located adjacent to the outfall canal (15366 and 15379) is notably quicker than the recovery at groundwater node 13444. This can be attributed to increased lateral seepage from the adjacent nodes into the outfall canal given their close proximity to the ditch. Groundwater recovery at node 13444 is more limited and results in prolonged periods of high groundwater and saturated soils. High groundwater conditions are compounded with subsequent storm events, such as those seen in October 2017 following Hurricane Irma and can result in recurrent and more severe flooding than what is predicted under normal soil moisture conditions.

Conversely, dry season groundwater levels at Nodes 15366 and 15379 do not fall as low or as quickly as the node located further away. Groundwater levels in these areas are affected by surface water fluctuations in the adjacent outfall canal and IRL, whereas Node 13444 is further away and is buffered from the fluctuations thus exhibiting reduced levels of impact.

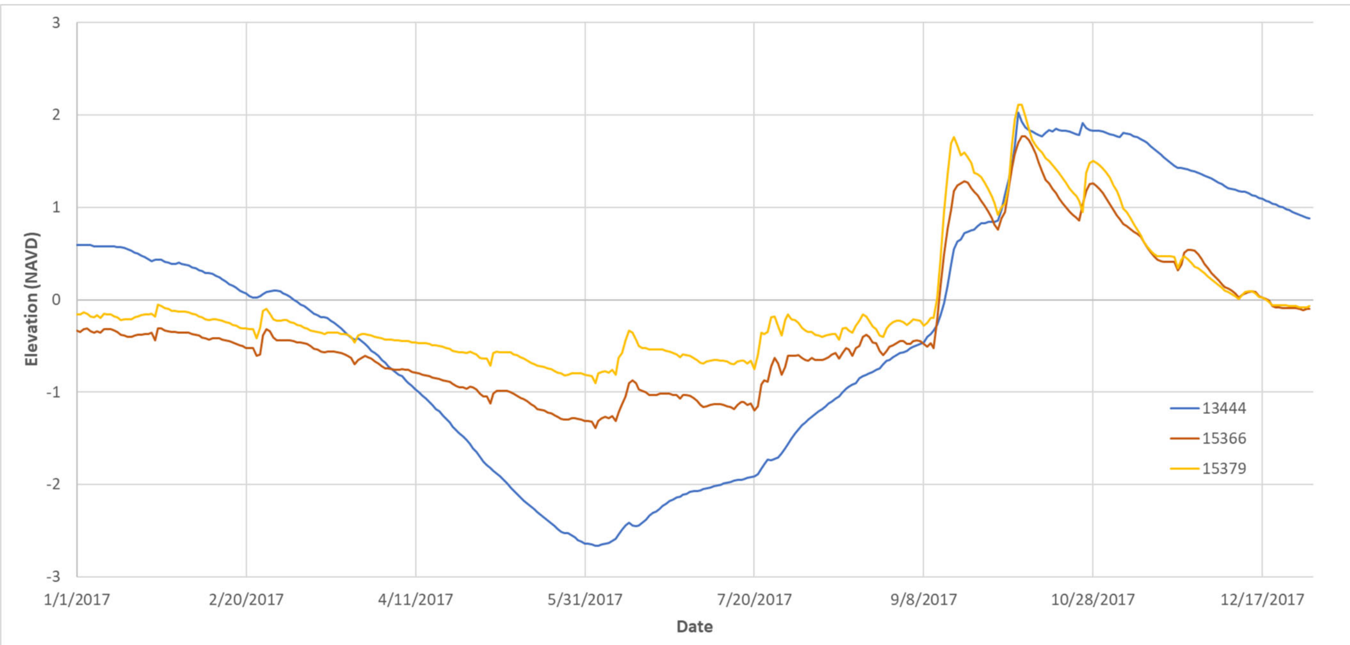


Figure 8.6: Groundwater Time-Series Graph for Nodes 13444, 15366, and 15379 from 2017 Calibration / Verification Model results

References

Collective Water Resources, LLC, 2019. Minimum Flows and Levels Program Hydrologic Modeling Services for Lakes Johns and Avalon in Orange County, St. Johns River Water Management District Contract 32922, Work Order 1 July 2019.

Downer, Charles W. and Ogden, F.L. 2006. Gridded Surface Subsurface Hydrologic Analysis (GSSHA) User's Manual. Army Corps of Engineers. ERDC/CHL SR-06-1,

Interflow Engineering, LLC, 2008. Myakka River Watershed Initiative, Water Budget Model Development and Calibration - Final Report. Tampa, Florida

Moriasi, D., Arnold, J., Van Liew, M., Bingner, R., Harmel, R. & Veith, T., 2007. Model evaluation guidelines for systemic quantification of accuracy in watershed simulations. Trans. ASABE, 50(3), 885-900.

Moriasi, D., Gitau, M., Pai, N., & Daggupati, P., 2015. Hydrologic and Water Quality Models: Performance Measures and Evaluation Criteria. Trans. ASABE, 58(6), 1763-1785.

NRCS, 1986. Technical Release 55 Urban Hydrology for Small Watersheds. United States Department of Agriculture. Washington, D.C.

Ree, W.O. and Palmer, V.J., 1949. Flow of Water in Channels Protected by Vegetative Linings. United States Department of Agriculture. Washington, D.C.

Singh, J., Knapp, H., & Demissie, M., 2004. Hydrologic modeling of the Iroquois River watershed using HSPF and SWAT. ISWS CR 2004-08. Champaign, Ill.: Illinois State Water Survey.

Smajstrla, A.G., 1990. Agricultural Field Scale Irrigation Requirements Simulation (AFSIRS) Model Technical Manual Version 5.5. Agricultural Engineering Department, University of Florida, Gainesville, FL.

Streamline Technologies, Inc., 2020. Southern Lee County Flood Mitigation Plan, ICPR4 Modeling of East of I-75 Overland Collection Drainageway Concept Project and Crew-Flint Pen Hydrologic Restoration Concept – Final Report. Winter Springs, Florida.

Streamline Technologies, Inc., 2021. ICPR4 Hydrologic Modeling Support For Johns and Avalon Lakes. Winter Springs, Florida.