South Florida Water Management District

C2, C3W, C4, C5 & C6 WATERSHEDS
FPLOS ASSESSMENT FOR CURRENT AND FUTURE SEA LEVEL RISE CONDITIONS

Final Comprehensive Assessment Report

Prepared By:

chen moore and associates

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500 Austrailian Avenue South, Suite 850
West Palm Beach, FL 33401
State of Florida Engineering Business
Licence No. 4593

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DELIVERABLE 6.2
FINAL COMPREHENSIVE ASSESSMENT REPORT

chen moore and associates

Prepared for:

South Florida Water Management District

Prepared by:

Brent Whitfield, PE

Stephanie Long-Marquez, PhD, PE

Laura Vogel, PhD, PE

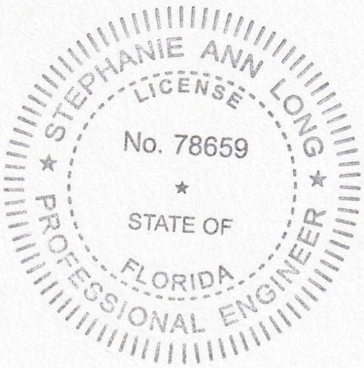
Justin Tagle, PE

Chen Moore and Associates

West Palm Beach Office:

500 S Australian Ave, Suite 850

West Palm Beach, FL 33401



Stephanie Long-Marquez, PE, PhD
Sr. Project Engineer

Chen Moore and Associates
T: 561-277-2367
E: slong@chenmoore.com

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EXECUTIVE SUMMARY

INTRODUCTION

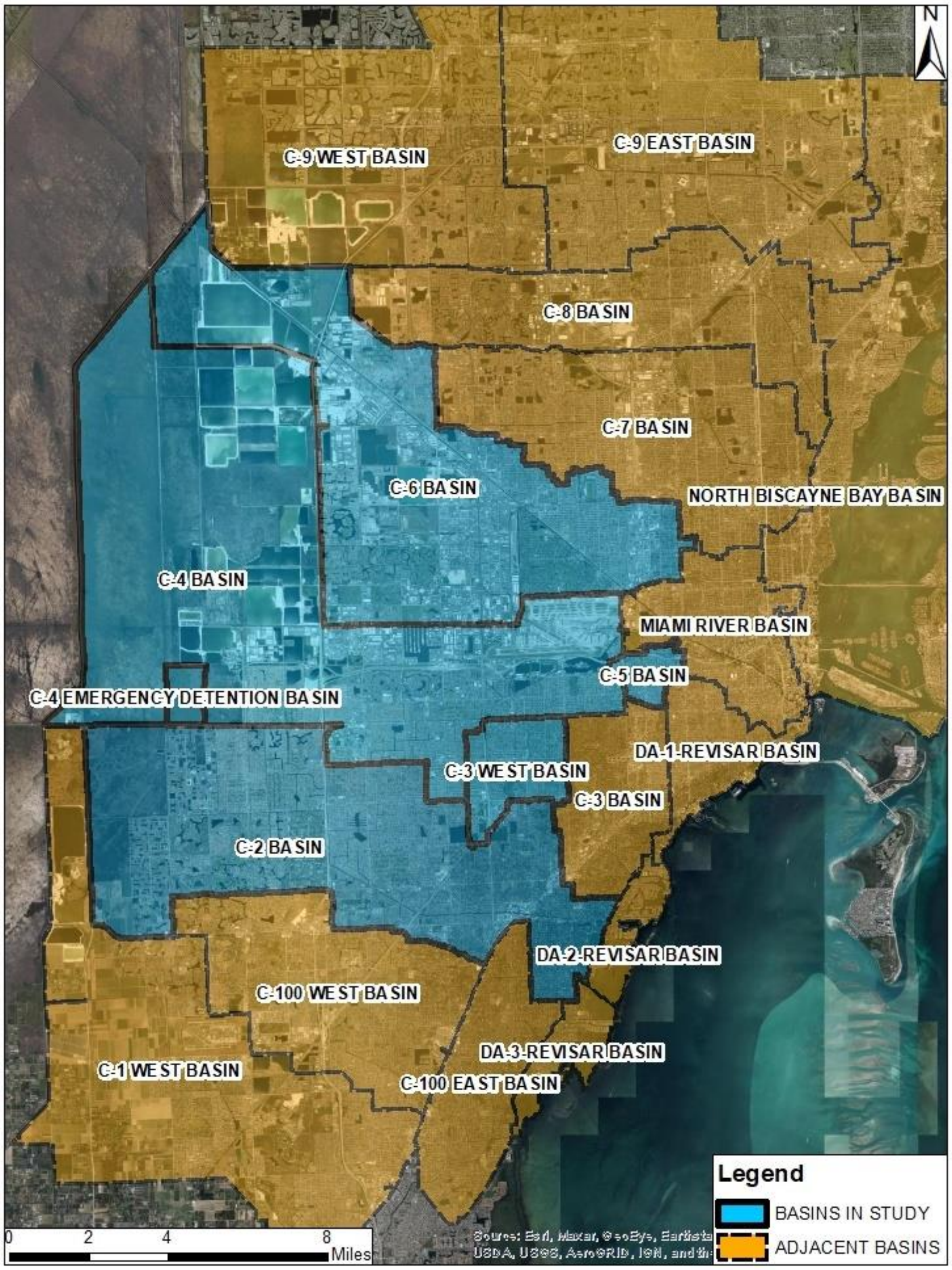
The South Florida Water Management District (District) is conducting a system-wide review of the regional water management infrastructure to determine the flood protection level of service (FPLOS) being provided by existing infrastructure under current and future conditions. The FPLOS describes the amount of protection provided by the water management facilities within a watershed considering sea level rise (SLR), future development, and known water management issues in each watershed. Notably, the SLR scenarios will also consider associated changes in groundwater levels and land use changes.

Chen Moore and Associates (CMA) was tasked with preparing the FPLOS analysis for the C2, C3W, C4, C5 and C6 watersheds in central Miami Dade County. This effort involved developing a calibrated and validated hydrologic and hydraulic (H&H) model of the subject watersheds, as shown in **Figure 1**. This region includes a significant extent of flood protection infrastructure including an extensive primary canal network with District owned and operated control structures throughout the highly managed system. Although the District canals and structures represent the primary infrastructure for providing flood protection in the area, the secondary drainage system is a significant component. In particular there are large canals, culverts and pumps in the project area that are owned and operated by Miami Dade County and the municipalities of Sweetwater, City of Miami, and West Miami.

After researching a variety of options for modeling tools it was determined that MIKE SHE was the best software available to evaluate FPLOS for the subject area in consideration of the low relief topography, high-water table, unique hydrogeology, and complex control structure operations. The FPLOS analysis involved developing a calibrated and validated hydrologic and hydraulic (H&H) model in MIKE SHE/MIKE 1D of the subject watersheds, while considering the interconnectivity of the canal network with adjacent watersheds. Overall, the FPLOS analysis included five (5) separate tasks. Task 1 consisted of selecting a model and collecting data. Task 2 consisted of developing an integrated groundwater-surface water modeling tool. Task 3 consisted of calibrating and validating the model for the roughly 200 square mile model domain. Task 4 consisted of simulating the design storm events for existing conditions and determining the FPLOS for current conditions. Task 5 consisted of simulating future sea level rise (SLR) conditions and determining the FPLOS. The future conditions simulations incorporated +1 foot, +2 feet, and +3 feet of SLR, projected land use, and projected groundwater levels.

Once the FPLOS assessments were completed for a suite of performance metrics under existing and future SLR conditions, a narrative describing preliminary recommendations for potential flood mitigation projects was provided for the C2, C3W, C4, C5 and C6 watersheds as part of Task 6.

Figure 1 - Map of C2, C3W, C4, C5, and C6 FPLOS Project Domain and Related Watersheds



MODEL CALIBRATION

The calibration study used the May 2020 storm event while the validation study used 2017's Hurricane Irma. After initial model simulations the following parameters were revised to improve model results.

- The interaction between the groundwater and surface water was analyzed to determine that the most representative scenario involved aquifer transmissivity alone, versus aquifer transmissivity plus a specified bed resistance.
- The values for Manning's "n" were updated to a minimum roughness coefficient of 0.033, throughout the model domain.
- A time-varying distributed boundary for the Saturated Zone was created using measured well-data to represent groundwater conditions more accurately at the limits of the model domain.
- Rock mining facilities and public water supply groundwater withdrawals were added to the model to represent groundwater conditions more accurately.
- Updated flow rating coefficients were obtained and was input into each control structure.

A variety of sensitivity tests were performed to test individual variables within the model. The vertical and horizontal conductivities for groundwater layers 1 and 2 as well as specific yield were tested to improve groundwater results. The comparison of model results demonstrated that the simulated stages were not sensitive to the modifications of K_H, K_V, and specific yield that were made. As such, the final calibration simulation was determined to utilize the same conductivity and specific yield values defined in the 2016 USGS MODFLOW model developed for Miami Dade County. The specific yield for all layers was also changed to test sensitivity of this parameter in the Saturated Zone model, but no significant benefits were identified in modifying the reference values. The Manning's roughness for primary and secondary canals were changed to 0.04 to test sensitivity in the hydraulic model. In consideration that the effects of increasing the roughness value on simulation performance was not an overall improvement and considering the typical operations and maintenance of the primary canals, the roughness values for the primary and secondary canals were recommended to remain 0.033 in the final calibration. In the overland flow mode, detention storage, overland Manning's roughness, and drainage level were tested by altering the values ±50 %. Considering that the results of these sensitivity modifications either did not appreciably improve or show a negative impact to the simulated results, no change to the parameterization of detention storage, overland Manning's roughness, or drainage level was recommended.

After a series of additional evaluations of input data and model methodology were implemented a final refinement of the calibration was performed to adjust pump speeds, allow overbank spilling, reduce computational timesteps, adjust overland thresholds, and add separated flow areas and drain codes. These final refinements led to the final calibration of the MIKE model. A review of the results of the final simulation of the May 2020 calibration period demonstrated significant alignment of the simulated stages with respect to peak magnitude and timing. The target error tolerance of less than 0.5 feet was met in nearly all locations for nearly all statistical evaluations. The simulated flow was also relatively representative with a visual comparison showing the magnitude and timing of peak flow matched well with measured values in most locations.

The validation results demonstrate that the calibrated parameterization for the May 2020 rainfall event did not reflect the natural system response within the model as accurately for the September 2017 rainfall event. For the validation simulation, the mean absolute error for the simulated headwater stages at the key tidal spillways (G93, S22, S25, S25B, and S26) was within the 0.5 feet tolerance for the one-week period associated with rainfall event. However, at upstream stage monitoring locations and most groundwater monitoring locations the error exceeded the 0.5 feet tolerance. Improvements in the validation model could be realized by modifying parameters in the calibration configuration or locally within the model, however the intent of the validation simulation is to demonstrate the model's ability to recreate the natural system not for additional calibration efforts. Additionally, it should be noted that the validation simulation represents a period in the dry season that may not be typical flooding conditions for South Florida. Based on these factors, no changes were recommended to model parameters based on validation results.

The calibrated model demonstrated accuracy in representing peak canal stages, which is a critical component to evaluating the capability of the model to properly represent the flood protection provided by existing infrastructure under various rainfall and tailwater conditions. Based on the results shown in this report, the calibrated and validated model can be used to evaluate the FPLOS, especially in consideration of the performance of the simulation for canal stages.

LEVEL OF SERVICE ANALYSIS FOR CURRENT AND FUTURE CONDITIONS

CMA prepared the FPLOS analysis for the C2, C3W, C4, C5, and C6 watersheds in central Miami Dade County. To perform this evaluation, the calibrated and validated model was modified for design storm analysis. Modifications to the model for analysis of current conditions included developing inland and coastal boundary conditions for groundwater and surface water, determining groundwater initial conditions from the USGS seasonal high-water table, determining the surface water initial conditions from the end of the validation simulation, developing spatially distributed rainfall files at a 15-minute timestep using NOAA Atlas 14 data for the magnitudes of the rainfall, coding the documented operational strategies for the gate operations, and determining the pumping rates for the public water supply wells from the average October rate from Miami Dade Water and Sewer Department.

For the Future Conditions model setup, the Current Conditions model was used as a base and modified to reflect projected changes that would occur over a 50-year period. Topography and land use were modified in selected parcels within the Urban Development Boundary that were considered vacant or were built prior to 1970, which represents structures over 100 years old. Modifications to topography consisted of setting minimum ground elevations on these selected parcels based on Miami Dade County's flood criteria. Overland parameters such as percent impervious (or runoff coefficient, in model terms) and Manning’s roughness were modified using the future conditions land use data.

Groundwater boundaries and groundwater initial conditions were established from the mean high-water table (MHWT) map, modified for SLR scenarios. In addition, groundwater pumping withdrawals for water supply were modified from the September average to account for the 44 % increase in projected usage at the wellfields in the model domain.

Additional branches were added from Miami Dade County to account for future planned canals. Surface water boundaries and surface water initial conditions in the canals were modified for each SLR condition. However, gate operations remained the same as current conditions for these scenarios.

No changes to rainfall were considered for future conditions analysis for this evaluation.

FPLOS was determined for each watershed using design storm simulations for both current and future conditions with SLR. The watersheds were then evaluated based on the highest design storm event during which the watershed could provide flood protection. The design storm events under evaluation for this study were the 5-year 3-day, 10-year 3-day, 25-year 3-day, and 100-year 3-day design storms. A suite of six (6) performance metrics (PMs) were also evaluated for each watershed and each design storm simulation. The details of each PM are defined as follows:

- **PM #1:** Maximum stage in primary canals.
- **PM #2:** Maximum daily discharge capacity through the primary canals.
- **PM #3:** Tidal structure flow performance – effects of sea level rise.
- **PM #4:** Peak storm runoff - maximum conveyance capacity of the watershed.
- **PM #5:** Frequency of flooding – stage-based LOS for sub-watersheds.
- **PM #6:** Duration of flooding – effects of sea level rise.

Based on the H&H modeling and a detailed evaluation of each PM, the current conditions and future sea level rise conditions FPLOS was determined for each of the five (5) subject watersheds. **Table 1** summarizes the findings of the analysis for all conditions and each watershed. The highest LOS that was considered passing for the simulation was given to each watershed. If the watershed does not provide LOS for any of design storm events for the sea level conditions, it was given a ranking of "<5-year".

Table 1 – Summary of Current LOS Provided by Each Watershed

CONDITION	PM	C2	C3W	C4	C5	C6
Current Conditions	PM #1	10-year	25-year	25-year	25-year	25-year
	PM #5	25-year	25-year	10-year	10-year	5-year
	PM #6	25-year	25-year	10-year	25-year	<5-year
	Overall LOS	**10-year**	**25-year**	**10-year**	**10-year**	**5-year**
Future Conditions SLR +1 foot	PM #1	5-year	10-year	10-year	10-year	10-year
	PM #5	25-year	25-year	10-year	5-year	5-year
	PM #6	25-year	10-year	10-year	10-year	<5-year
	Overall LOS	**5-year**	**10-year**	**10-year**	**5-year**	**5-year**
Future Conditions SLR +2 feet	PM #1	<5-year	5-year	5-year	<5-year	<5-year
	PM #5	10-year	10-year	5-year	5-year	<5-year
	PM #6	10-year	10-year	5-year	5-year	<5-year
	Overall LOS	**<5-year**	**5-Year**	**5-year**	**<5-year**	**<5-year**
Future Conditions SLR +3 feet	PM #1	<5-year	<5-year	<5-year	<5-year	<5-year
	PM #5	10-year	10-year	<5-year	<5-year	<5-year
	PM #6	10-year	5-year	<5-year	<5-year	<5-year
	Overall LOS	**<5-year**	**<5-year**	**<5-year**	**<5-year**	**<5-year**

To focus the overall findings of each watershed, the design storms identified as the LOS for all PMs were aggregated with additional weight given to PM #1, #5, and #6 due to their direct relationship to flooding. Specifically, PM #1 focused on canal stages and overtopping of canal banks and structures and had more weight when determining the overall level of service for the watershed, in general this determined the minimum LOS if it was the lowest. PM #5 and PM #6, which focused on the inundation depth and duration, are also critical for understanding watershed flooding impacts and could lower the minimum LOS provided by PM #1.

PM #2 through #4 focus more on structure and watershed discharge capacity, which are helpful for understanding how the flooding is occurring and where there are flood protection deficiencies in the watershed, or how the watershed is impacted by SLR. While these are critical PMs and useful for comparing how SLR is impacting discharge capacity, their LOS ranking is not applicable for the overall understanding of the watershed flooding and are therefore not included in the summary **Table 1**.

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APPENDICES

1. INTRODUCTION

The South Florida Water Management District (District) is conducting a system-wide review of regional water management infrastructure to determine the Flood Protection Level of Service (FPLOS) being provided by existing facilities under current and future conditions. Final FPLOS reporting will describe the amount of protection provided by the water management facilities within a watershed considering future development, known water management issues in each watershed, and sea level rise (SLR). SLR scenarios will also consider associated changes in groundwater levels and land use changes.

Chen Moore and Associates (CMA) was tasked with preparing the FPLOS analysis for the C2, C3W, C4, C5 and C6 watersheds in Central Miami Dade County. This effort involved developing a calibrated and validated hydrologic and hydraulic (H&H) model of the subject watersheds, as shown in **Figure 1-1**. This region includes a significant extent of flood protection infrastructure including an extensive primary canal network with District owned and operated control structures throughout the highly managed system. **Figure 1-2** illustrates the extent of the District canals and structures within the subject watersheds. Although the District canals and structures represent the primary infrastructure for providing flood protection in the area, the secondary drainage system is a significant component, especially the facilities that are owned and operated by the County and the municipalities.

The model elevations and report results were measured using the vertical datum of feet NAVD88 (ft-NAVD). However, some structure information provided by the District is referenced in feet NGVD29 (ft-NGVD) and will therefore be reported using both vertical datums.

Figure 1-1: Project Basins

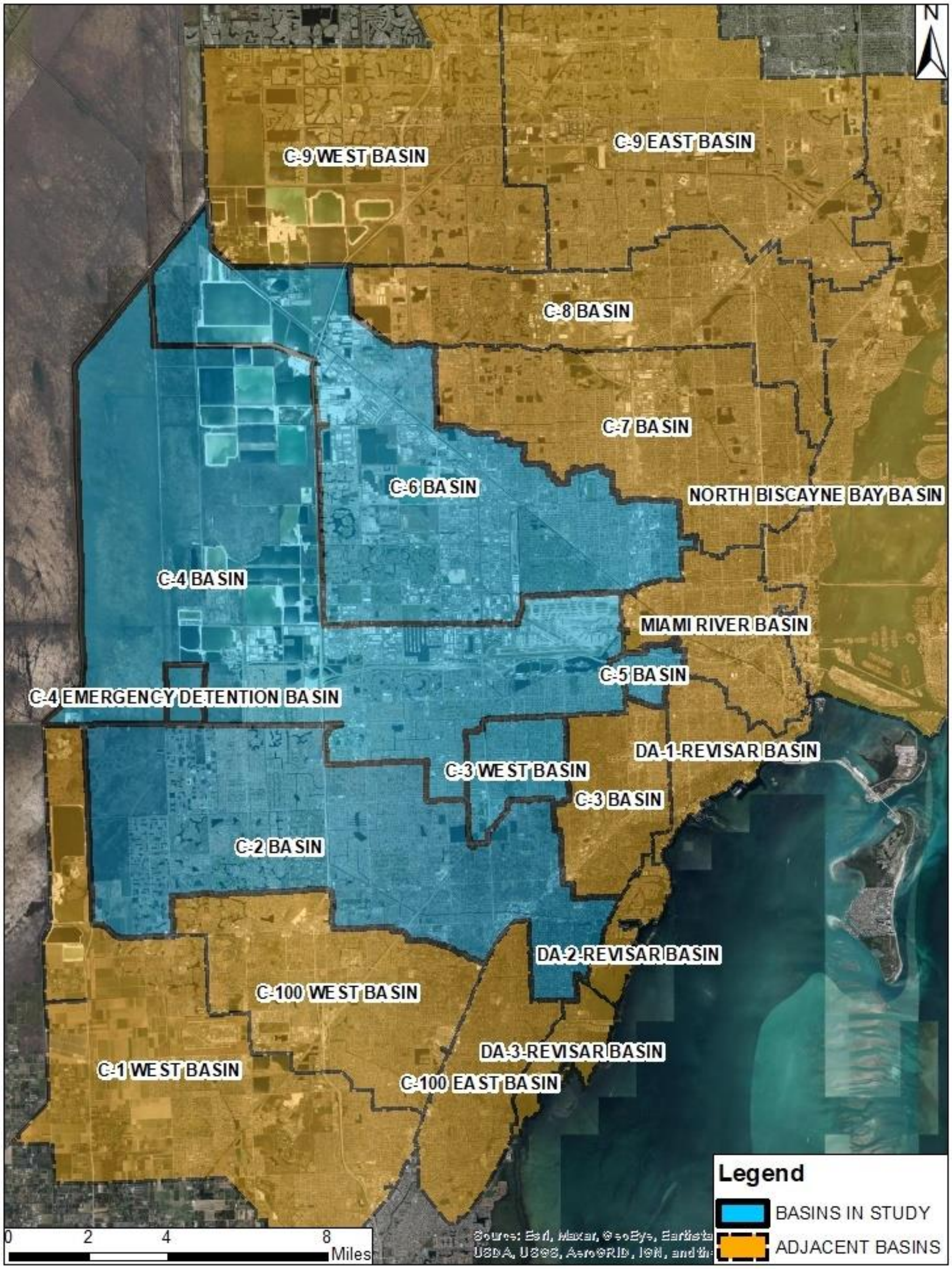
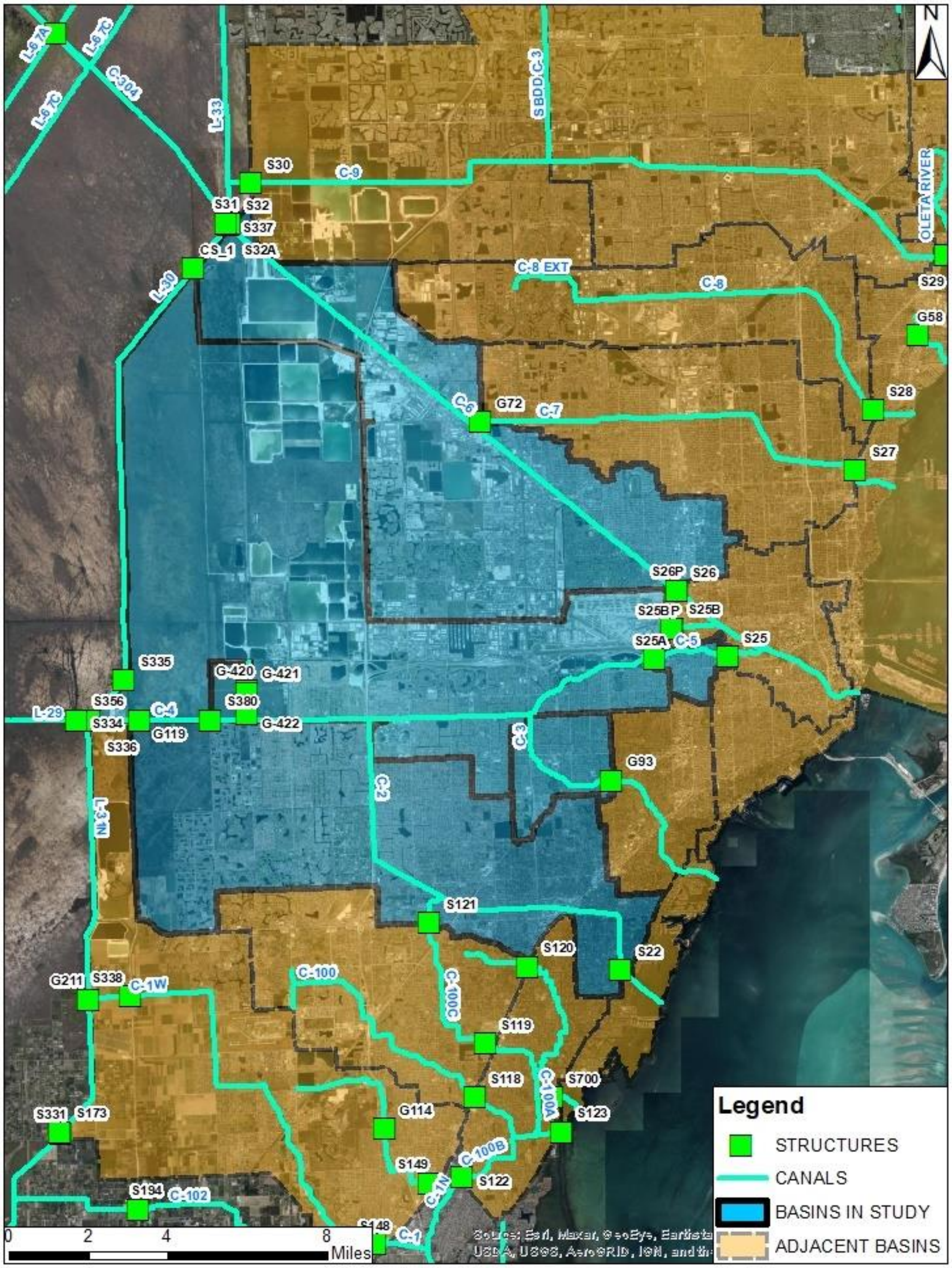


Figure 1-2: Primary Canal Network and District Water Control Structures



2. MODEL AND TOOL SELECTION AND WORK PLAN

The C2, C3W, C4, C5 and C6 Basins are located in Miami Dade County where the limestone geology is such that groundwater and surface water have a direct interface throughout the canal conveyance network. Considering this condition, the modeling tools used for the FPLOS analysis need to have not only hydrologic and hydraulic capabilities, but also hydrogeologic capabilities. As an additional consideration, Miami Dade County has very little topographic relief, which means overland sheet flow and tidal flooding at the coastal boundary are common occurrences. In a one-dimensional hydrologic and hydraulic model, these processes are not easily represented with accuracy. Therefore, a two-dimensional representation of the land surface is the preferred approach. The low-relief topography of the area also lends itself to hydraulic conditions where the flow in various canals can be reversed if the conditions dictate an east-to-west hydraulic gradient. An example of this condition would be during flooding events when the C4 Impoundment is utilized and the flow in the C4 Canal is reversed to provide additional detention storage of floodwaters on a temporary basis. Not all models easily accommodate reversals in flow direction during a continuous simulation, so this characteristic is an important capability to consider during model selection. For the subsurface, it is important that the selected model can simulate multiple layers to represent the varied hydrostratigraphy within Miami Dade County and that can seamlessly interface with the surface water model.

There are several other hydrologic processes that should be represented by the selected model including how runoff generation is related to the available field capacity for infiltration in the unsaturated zone and the effect of evapotranspiration on increasing the capacity for infiltration in the soil. Because these processes can be a critical component of rainfall-runoff generation, models that use physically based parameterization of the soils and vegetation are preferrable.

2.1. PAST MODELING STUDIES IN THE PROJECT AREA

Due to the combination of a high risk for flooding and significant urban development in the project area, there have been several modeling studies performed that can be used as a reference for this effort. In specific, The District has developed several models to evaluate operating rules in the C4 Watershed (SFWMD, 2011) and to evaluate FPLOS in the C4 Watershed (SFWMD, 2015). The Miami Dade County Department of Regulatory and Economic Resources has been maintaining a series of models for each watershed in the County under the Watershed Management Division since 2003. These models are routinely updated including for the watersheds being evaluated in this project (MD-RER, 2005; MD-RER, 2006; MD-RER, 2007; MD-RER, 2018). There have also been several efforts that have simulated groundwater conditions in the project area including the District with the Lower East Coast Subregional MODFLOW Model (SFWMD, 2006) and the USGS in partnership with Miami Dade Water and Sewer Department for the Urban Miami Dade Model (USGS, 2016).

2.2. HYDROLOGIC & HYDRAULIC MODELING TOOLS

In Miami Dade County, there are several modeling tools that are commonly used for hydrologic and hydraulic modeling by the public and private sectors. These tools vary in capabilities and complexity and are often selected based on the requirements of the situation. The sub-sections below describe options for this project based on commonly used software.

2.2.1. HEC-HMS / HEC-RAS / HEC-RAS2D

The US Army Corps of Engineers (CoE) has developed the HMS and RAS suites to simulate hydrology and hydraulics, respectively. Within HEC-HMS the hydrologic processes of a watershed are typically represented by developing sub-basins that are assigned characteristics to represent available storage and infiltration which recreate the expected rainfall-runoff relationship. These hydrographs can then be exported to HEC-RAS and routed through a conveyance network that simulates the canals, culverts, bridges, and structures of the physical environment. The newest release of HEC-RAS2D provides a more robust approach for HEC-RAS to represent rainfall-runoff processes and can provide a more thorough representation of the land surface.

This software is frequently used around the world to represent the expected flow within natural watersheds. However, in a low-relief, urban environment dominated by stormwater management infrastructure and in a location where the canal network significantly interfaces with regional groundwater due to the geology, these tools require simplification of the physical environment in a manner that does not always accurately reflect reality. Prior to this study a series of models of the C4 Basin were developed within HEC-RAS that represented the hydraulic conditions of the primary conveyance network accurately but did not thoroughly represent the secondary and tertiary systems. In addition, these efforts utilized a parallel simulation of MODFLOW to represent the groundwater condition. This prior modeling effort began as a tool to refine flood control operations in the region (SFWMD, 2011) and later evolved into a pilot project for the FPLOS program (SFWMD, 2015). For this FPLOS effort, which includes multiple basins and has a broader focus on representing the integration of the surface and ground water processes, a tool with direct coupling of surface water and groundwater processes was preferred.

2.2.2. ICPR

From its initial development, Interconnected Pond Routing (ICPR) has been well suited for low-relief, urban stormwater systems similar to Miami Dade (Streamline Technologies, 2014). This suitability is demonstrated by ICPR's widespread use in regulatory permitting. In South Florida, the majority of design simulations submitted by land developers and public entities utilize ICPR to verify compliance with regulatory requirements. The newest version of ICPR (Version 4) incorporates a two-dimensional representation of the land surface and includes the option to simulate a single layer groundwater system as a lower boundary. These advancements in the latest version make ICPR4 a sensible candidate for use in this study. Unfortunately, the Miami Dade groundwater system is more complex than a single-layer system can accommodate. The most commonly used representation of groundwater flow in Miami Dade is the Lower East Coast Sub-Regional (LECsR) model which has three-layers, while newer representations of groundwater will likely represent the hydrogeology in five layers.

In addition to the groundwater considerations, one of the elements of the Miami Dade County hydraulics that would not be well represented in ICPR is the operation of control structures. At the upstream and tidal boundaries of each of the primary canals in the subject basins there is a spillway structure and/or pump station that operates based on headwater and tailwater conditions. The flow at each structure is not easily defined within ICPR where the most common simulation approach is to represent the structure as a fixed weir or a rating curve. Additionally, the most common use of ICPR is for event-based scenarios such as design storms and not continuous scenarios such as extended calibration or validation periods.

2.2.3. MIKE SHE / MIKE HYDRO RIVER

The MIKE SHE suite of products provides the capability for two-dimensional representation of the land-surface and fully integrated, groundwater-surface water representation with no simplifications of the model configuration. The groundwater component can be used to represent many layers that vary in thickness and conductance. The hydrologic representation of the rainfall-runoff characteristics can be developed using physically based processes such as the Green-Ampt infiltration method in MIKE SHE, while the hydraulic conveyance within the primary, secondary, and tertiary canal network can utilize detailed cross-section and culvert data within MIKE HYDRO River. In addition to these complex tools, MIKE SHE allows for the physical configuration of a spillway to be included within the model and the gates can be operated based on known gate opening data for calibration and validation purposes or based on pre-defined control rules for flood control. The configuration of the gate structures and the representation of the unsaturated and saturated layers of the subsurface allows for both continuous simulations and event-based simulations that would apply to calibration scenarios and design storm scenarios. The integrated surface water groundwater configuration also allows for the effects of SLR to be represented as changes to both the downstream hydraulic boundary and the initial groundwater elevations throughout the project area.

2.2.4. SWMM (EPA SWMM5 / INFOSWMM / PCSWMM / XP-SWMM)

For over two decades, Miami Dade County has utilized a variant of SWMM called XP-SWMM to simulate each basin within the County limits. These models have been used very effectively to identify flooding issues and to provide a basis for capital improvement plan development. XP-SWMM is a proprietary version of the public use software EPA SWMM that focuses on an easier to use interface. Like HEC-HMS, the hydrologic approach for SWMM relies on subdividing the watershed into sub-basins and assigning each sub-basin rainfall-runoff characteristics. SWMM does have a mechanism to represent control structures and pump stations that can utilize measured gate opening data during calibration events or control logic during design storms. XP-SWMM utilizes a Real Time Control (RTC) module that allows the user to include editable parameterization to represent complex control rules that could be used to reflect the District's control logic, however in practice the RTC capabilities cannot capture the full extent of District operations at the most complex structures. XP-SWMM also does not utilize a physically based soil moisture accounting approach to determine the capacity of the soil for infiltration. These capabilities are important to represent both continuous simulations and event-based simulations. In addition, the version of XP-SWMM utilized by Miami Dade County does not incorporate a two-dimensional representation of the land surface.

PCSWMM and InfoSWMM are tools that can provide two-dimensional capabilities and could be an alternative that would utilize a similar platform as the County models. As an example of the use of these alternative tools, the ongoing USACE Back Bay Study is incorporating modeling in PCSWMM. One significant concern for the FPLOS study in using a SWMM oriented platform was that it cannot be easily coupled with a groundwater model to properly recreate the integrated surface water groundwater conditions common to Miami Dade County. Because the XP-SWMM platform utilizes an empirical approach to groundwater flow representation and not a physically based approach, there is no available methodology to provide an integrated groundwater/surface water model.

2.3. RECOMMENDED MODEL

Based on the review of models above, it was recommended that MIKE SHE/MIKE 1D be utilized for the FPLOS efforts in the C2, C3W, C4, C5 and C6 watersheds. With respect to hydrology, this tool allows for a two-dimensional representation of the land surface with physical processes that define the rainfall-runoff relationship. With respect to hydraulics, this tool also allows for the most accurate approach to representing known and predicted water control structure operations. With respect to surface water groundwater interaction, this tool allows for the most seamless connectivity between the surface and sub-surface conditions, while accommodating hydrostratigraphy with multiple layers.

2.4. FPLOS PROCESSING TOOLS

Once the calibration was completed and production runs were simulated with design events for existing condition and future conditions, the results of the modeling were evaluated with respect to six Performance Measures (PM). These are the same PMs being applied to all other previous and ongoing FPLOS studies throughout Miami Dade County. A detailed description of each PM is described below:

- **PM #1:** Maximum stage in primary canals.
- **PM #2:** Maximum daily discharge capacity through the primary canals.
- **PM #3:** Tidal structure flow performance – effects of sea level rise.
- **PM #4:** Peak storm runoff – maximum conveyance capacity of the watershed.
- **PM #5:** Frequency of flooding – stage-based LOS for sub-watersheds.
- **PM #6:** Duration of flooding – effects of sea level rise.

The majority of the PMs were evaluated using the post-processing tools of MIKE SHE. For PM #1, the hydraulic water surface profile for each canal was generated for each scenario. These results were exported from MIKE SHE for comparison and analysis in Microsoft Excel. For PM #2, similarly the canals were evaluated within the MIKE 1D component of the software to determine conveyance capacity available prior to overbank flow. For PMs #3 and #4, the MIKE 1D software simulated the discharge capacity under existing conditions, future conditions, and evaluated the maximum capacity. For PM #5, the results generated for peak flood conditions by MIKE SHE were exported to a raster format that was compared against property and infrastructure data within ArcGIS to determine inundation areas for different flooding depths for each SLR condition. For PM #6, the MIKE SHE platform simulated the recession of flood waters and duration of flooding for the conveyance network and was be utilized to compare the difference between existing conditions and SLR conditions.

2.5. WORK PLAN FORMULATION

Based on the selection of the MIKE SHE/MIKE 1D River platform as the hydrologic, hydraulic and hydrogeologic model for use in this FPLOS effort, an overview of key elements of the modeling approach are described below.

2.5.1. DETERMINING MODEL DOMAIN

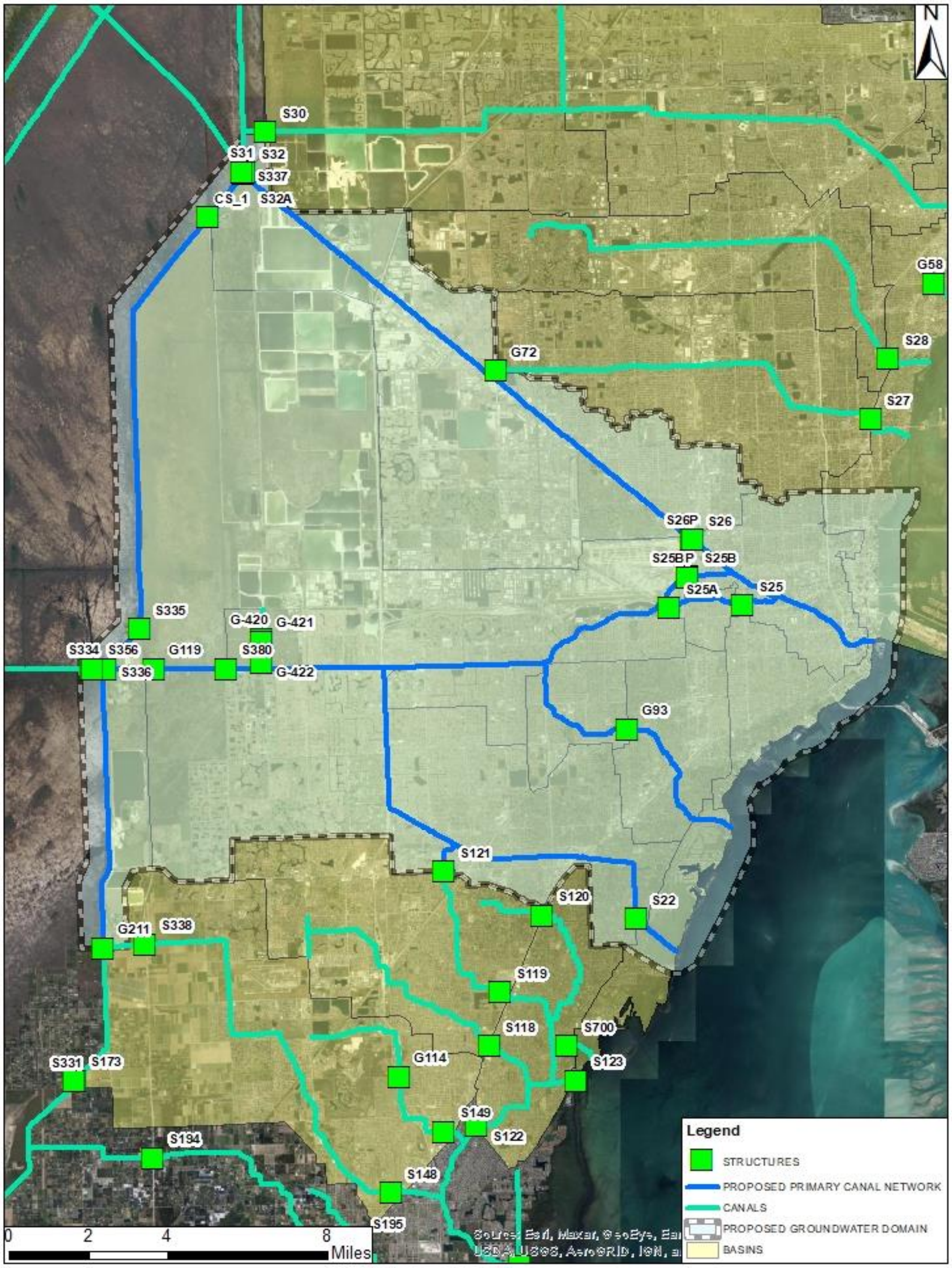
As noted in previous sections, the primary areas of interest for this FPLOS are the C2, C3W, C4, C5 and C6 watersheds. Considering the significant interconnectivity of these watersheds and the capability of MIKE SHE as a regional modeling platform, the proposed approach was to simulate all these watersheds combined in a single model. An additional concern due to the interconnected nature of the hydrology and hydrogeology of the region was the potential for inaccurate simulations when using the watershed limits as the model domain. By extending the model domain beyond the watershed limits, the possibility of the boundary conditions suppressing hydraulic responsiveness within the area of interest was reduced for the five watersheds.

Figure 2-1 illustrates the proposed hydraulic and hydrogeologic model domain with the primary canal network to be included. The proposed area is roughly 260 square miles which is about 25% larger than the combined area of the C2, C3, C3W, C4, C5 and C6 watersheds. Because the District maintains such a thorough surface water monitoring network at each control structure, these locations were used to define boundary conditions and calibration reference points. The sub-sections that follow describe the assumptions used to develop the proposed limits of the model domain.

2.5.1.1. HYDRAULIC DOMAIN LIMITS

The hydraulic network of the model consists of primary and secondary canals within each simulated watershed. For a typical hydraulic model of a canal, the upstream boundary is represented by the measured or calculated flow from a District control structure on the primary canal, such as the S336 culvert at the western limit of the C4 Canal. For the downstream boundary in a typical hydraulic model of a canal the downstream boundary is represented by the tailwater condition at the tidal structure, such as the S25B spillway on the C4 Canal.

Figure 2-1 – Proposed Primary Canal Network, Hydraulic Boundaries and Calibration Points



For the eastern boundary, there is no downstream structure east of the tidal structures within each watershed, however for the purposes of calibration and validation of the model, the hydraulic network was extended to the mouth of each primary canal to Biscayne Bay. This approach allowed for the tidal structures to serve as additional calibration reference points and for the model to represent the easternmost communities in each watershed. By virtue of being unmanaged with respect to the SFWMD primary network, an FPLOS assessment for these areas was not included as part of this project.

Notably, although measured data is available for the calibration and validation scenarios in the immediate vicinity of the proposed boundaries, for the design storm and event-based simulations the hydraulic boundary conditions were modified to force tidal conditions just downstream of the tidal structures.

2.5.1.2. HYDROLOGIC DOMAIN LIMITS

To provide sufficient boundary conditions for the representation of canal stages within the primary network, the model domain for the groundwater model must include a sufficient representation of the surrounding area well beyond the primary canal network. To the west of C4 and C6 watersheds, this approach meant including an eastern segment of Water Conservation Area 3B (WCA3B). To the west of the C2 watershed, this approach meant including a portion of the C1W Basin and Everglades National Park. To the east, the domain extended to Biscayne Bay.

2.5.1.3. MODEL GRID SET-UP

The District has ongoing FPLOS investigations underway to the north (C7, C8 and C9) and to the south (C1, C100, C102, C103). The majority of this modeling is being done utilizing the same software, MIKE SHE. In order to facilitate the potential for interconnecting all of these models to simulate the entirety of Miami Dade County in a single model, the grid for this model was setup in a manner that is similar to the adjacent model grids.

2.5.2. RAINFALL EVENT SELECTION

Based on a review of available precipitation data, eight different storm events from the past 25 years were considered for use in the calibration and validation simulations. **Table 2-1** describes the period evaluated, the total rainfall amount during that period, and the maximum one-day rainfall as measured at the rain gauge located at the S26 structure.

Table 2-1 – Rainfall Events Considered for Calibration and Validation

Number	Period	S26 R	
		Total Rainfall [in]	Maximum One Day Rainfall [in]
1	October 14 – 31, 1999	12.53	5.31
2	October 2 – 4, 2000	17.26	13.05
3	June 18 – 20, 2008	6.14	3.02
4	October 29 – 31, 2011	5.30	2.79
5	August 1 – September 30, 2012	18.3	2.78
6	December 4 – 6, 2015	6.51	4.09
7	September 9 – 10, 2017	7.53	5.27
8	May 20 – 28, 2020	12.90	5.45

Reviewing the recorded rainfall totals, the top four events for single day rainfall were October 2000, May 2020, October 1999, and September 2017. At S26, October 2000 had a maximum one-day rainfall total of 13.05 inches which is slightly greater than the 12.8 inches associated with the 50-year 1-day storm defined by NOAA Atlas 14 for the area. The next three largest events had a maximum one-day rainfall between 5 and 6 inches which is analogous to a 3-year return period. To determine which two events were selected for use in the calibration and validation simulations, an assessment of the spatial distribution and the hydraulic conditions for those periods was prepared.

2.5.2.1. SPATIAL VARIABILITY OF RAINFALL

Because the study area is roughly 260 square miles and rainfall in South Florida can have significant spatial variability, it was also important to evaluate the spatial distribution of rainfall for the events being considered. To provide more insight to the preferred selection for representative rainfall events, data was compiled for seven (7) locations that provide coverage for the project area. This review of various gauge data was used for the purposes of identifying the preferred storm events for the calibration and validation simulations, NEXRAD data was the recommended input for rainfall. **Figure 2-2** illustrates the location and name of all rain gauges in the region and highlights the seven (7) selected locations. **Figure 2-3** illustrates the Theissen Polygons associated with the selected rain gages and allows for the rainfall values to be weighted by area for evaluation purposes.

Table 2-2 describes statistics for the selected rain gauges to demonstrate the spatial variability for each storm event throughout the region. Average rainfall amounts were weighted based on the relative percentage of area associated with each of the seven stations. It was assumed that a lower standard deviation for the selected stations was equal to a more evenly distributed rainfall throughout the project area. To provide further verification of the spatial distribution, a review of NEXRAD data from the District is provided in subsequent sections.

Table 2-2 – Statistics for Preferred Rainfall Events for Selected Rain Gauges

Statistics for Selected Gauges	Oct 14-31, 1999	Oct 2-4, 2000	Sept 9-10, 2017	May 20-28, 2020
Areal Average Total Rainfall for Period [in]	10.84	12.29	7.62	11.37
Areal Average for One-Day Maximum [in]	6.57	9.66	5.24	4.79
Standard Deviation of Total Rainfall for Period [in]	2.70	4.24	0.46	1.44
Standard Deviation of One-Day Maximum [in]	1.99	3.17	0.57	1.15

Based on the cross-comparison of the seven (7) regional rainfall gauges, the September 2017 and the May 2020 events have the most even distribution of rainfall across the project area. Based on this criterion these two storms were prioritized for calibration and validation scenarios.

Figure 2-2 – Rainfall Gauges in the Project Area

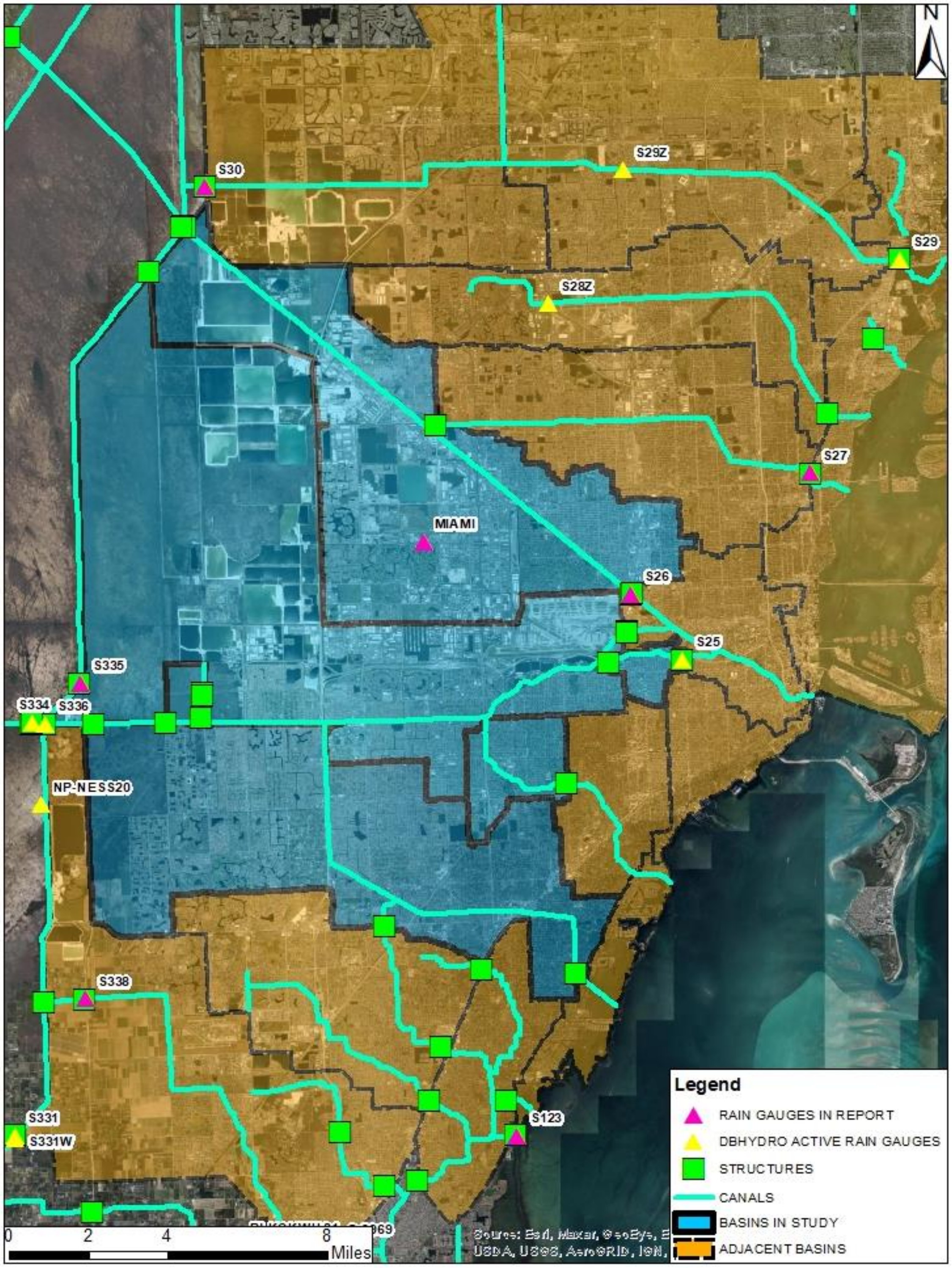
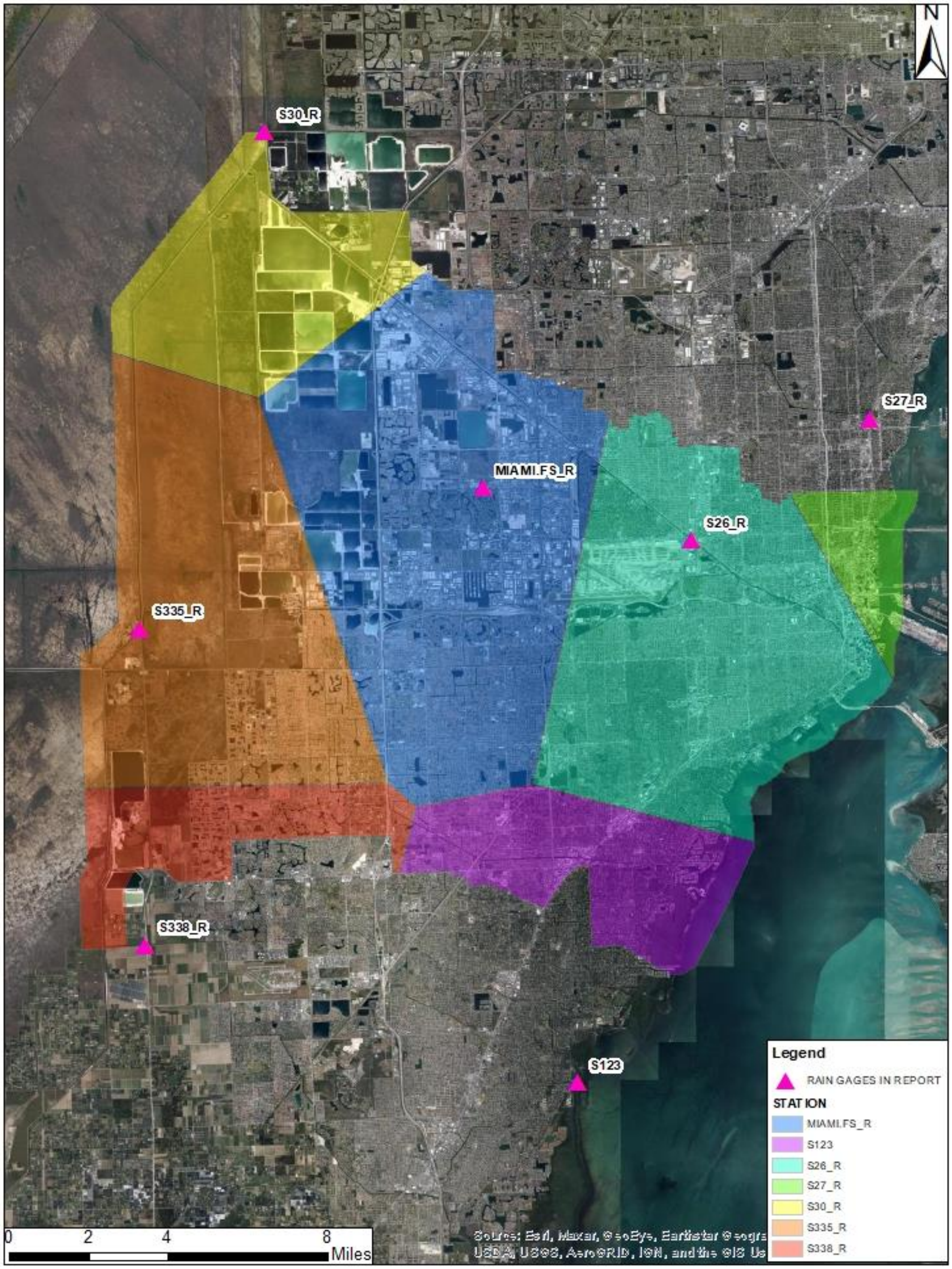


Figure 2-3 – Theissen Polygons for Selected Rainfall Gauges



2.5.3. TIDAL CONDITIONS DURING STORM EVENT

Prior to selecting the storm events to be used for the calibration and validation events, it was important to evaluate the downstream hydraulic conditions. It was anticipated that the SLR scenarios will have adverse tailwater conditions that limit discharges at the easternmost control structures. Considering this condition, it was valuable to have a range of downstream conditions for the calibration and validation simulations. To assess the potential for this situation, the headwater and tailwater stages at four of the tidal structures (S22, S25, S26, and G93) were reviewed. As noted in **Figure 2-4** through **Figure 2-19**, three out of the four storm events had a headwater that was higher than the tailwater during the storm event, while the September 2017 event had an adverse tailwater condition for some portion of the high discharge period at all four tidal structures. For design storm events, the District provided tailwater stage boundary conditions at the downstream side of tidal structures for current and future SLR event runs.

Figure 2-4 – Headwater and Tailwater Conditions for S22 (1999)

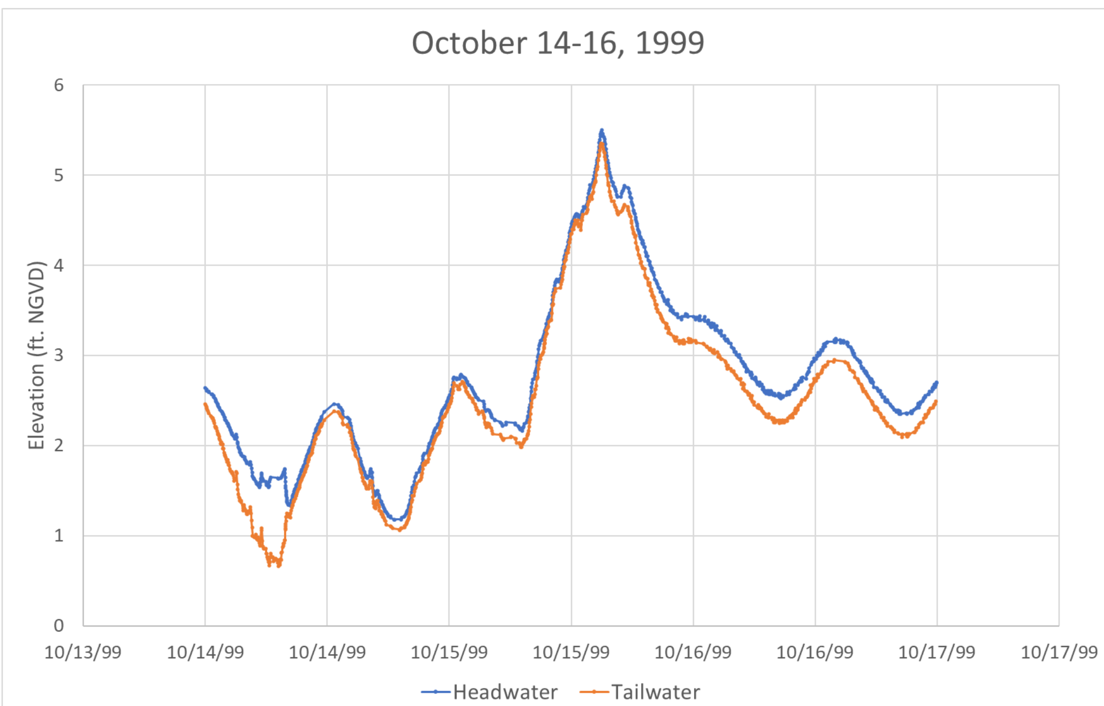


Figure 2-5 – Headwater and Tailwater Conditions for S22 (2000)

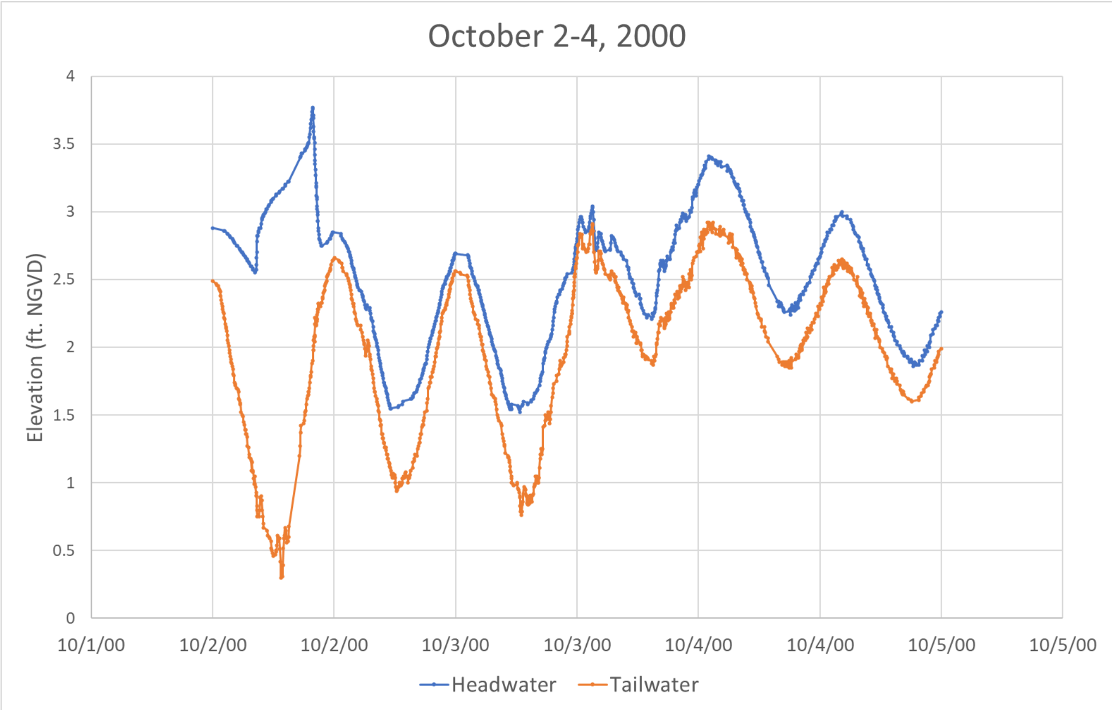


Figure 2-6 – Headwater and Tailwater Conditions for S22 (2017)

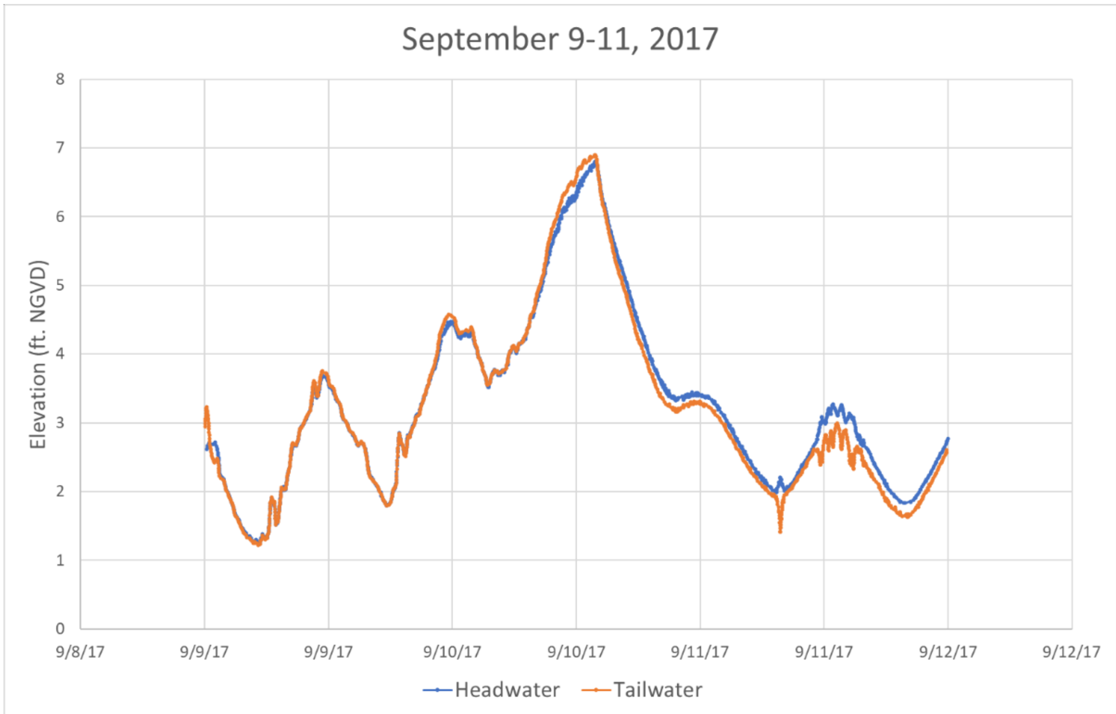


Figure 2-7 – Headwater and Tailwater Conditions for S22 (2020)

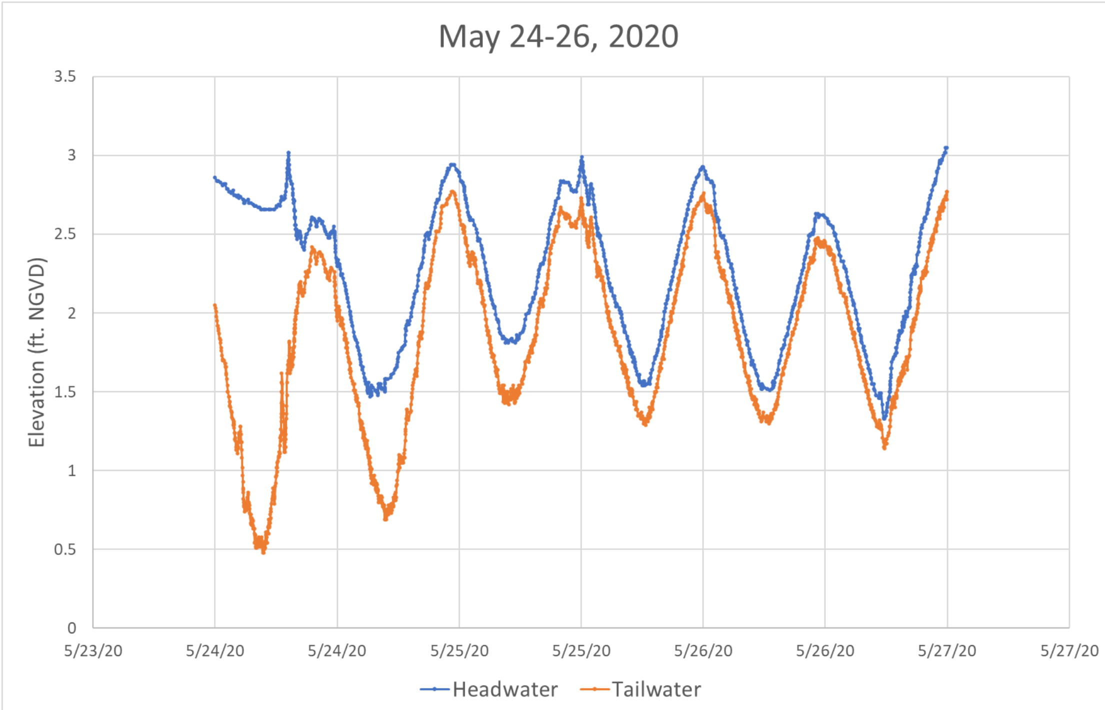


Figure 2-8 – Headwater and Tailwater Conditions for S25 (1999)

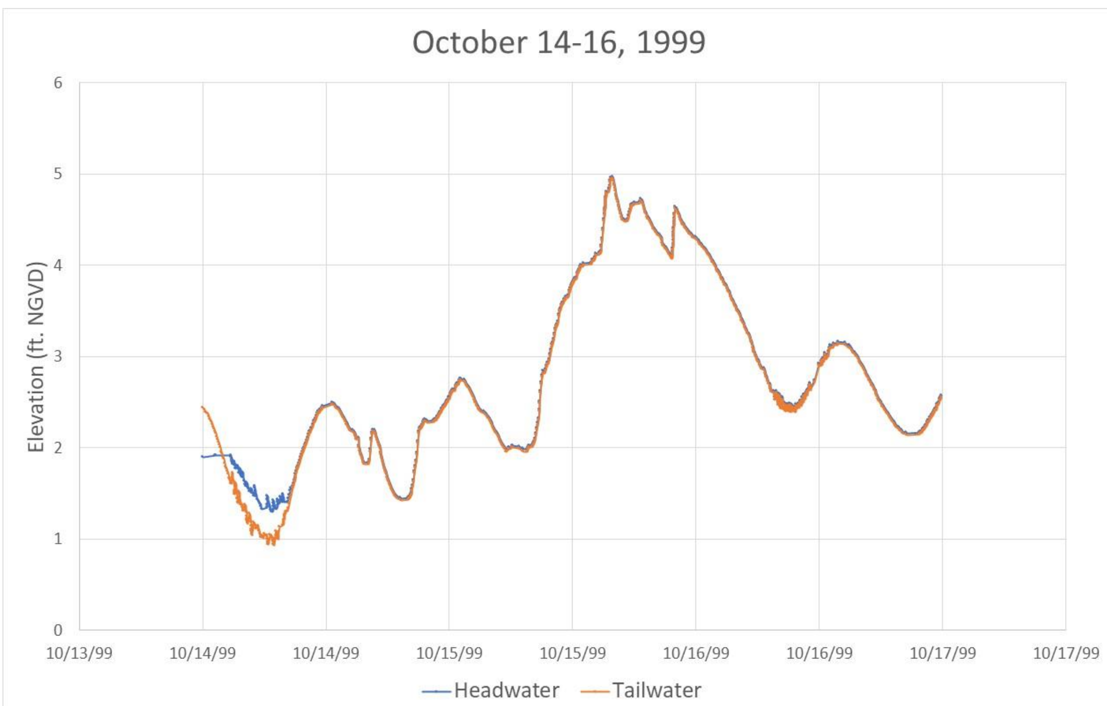


Figure 2-9 – Headwater and Tailwater Conditions for S25 (2000)

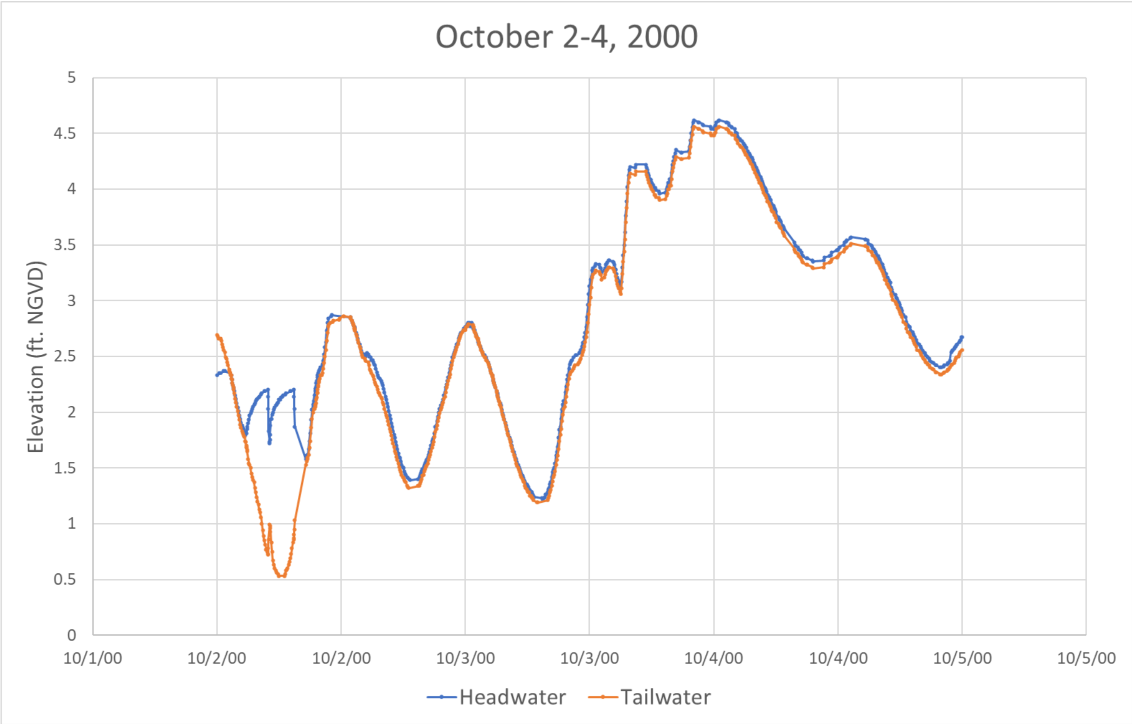


Figure 2-10 – Headwater and Tailwater Conditions for S25 (2017)

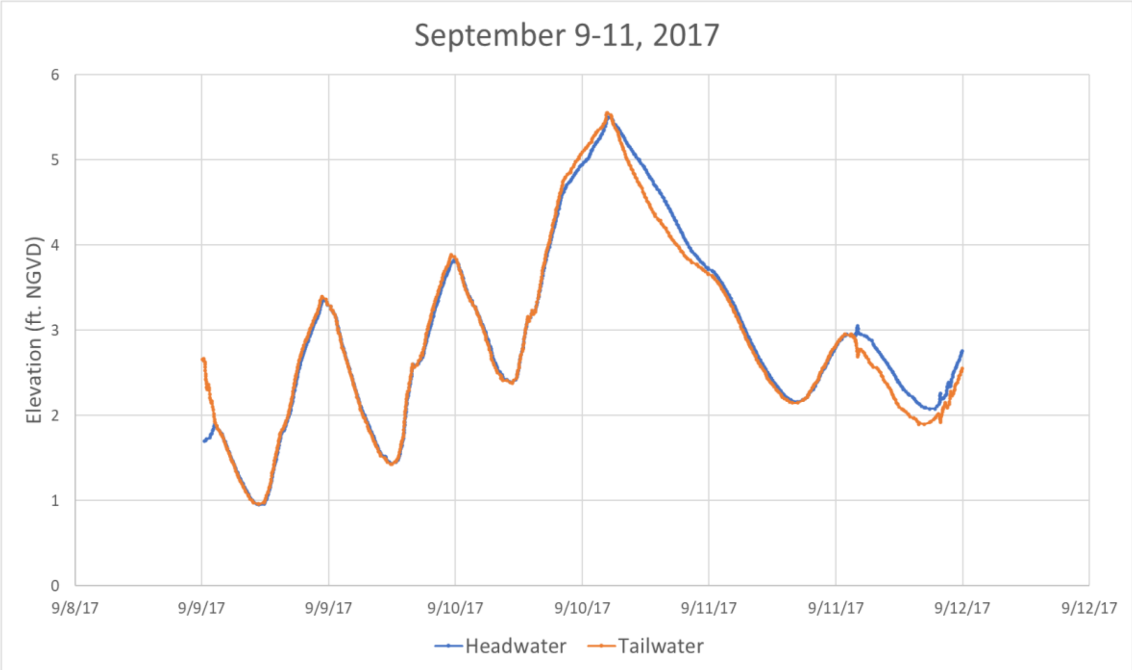


Figure 2-11 – Headwater and Tailwater Conditions for S25 (2020)

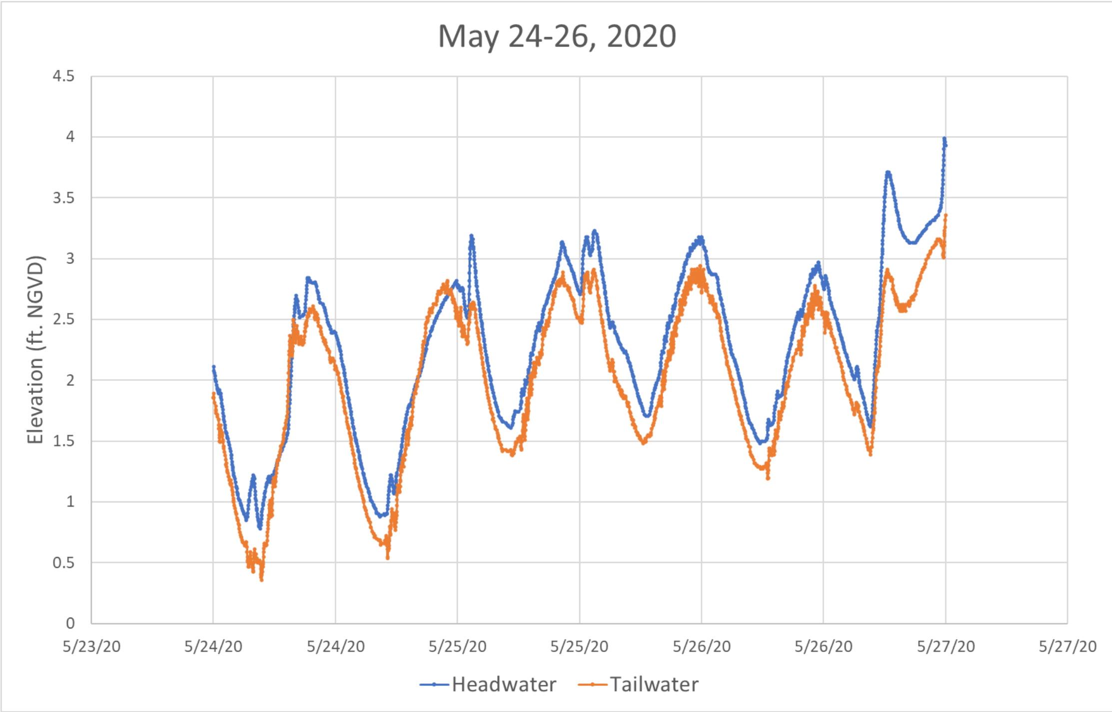


Figure 2-12 – Headwater and Tailwater Conditions for S26 (1999)

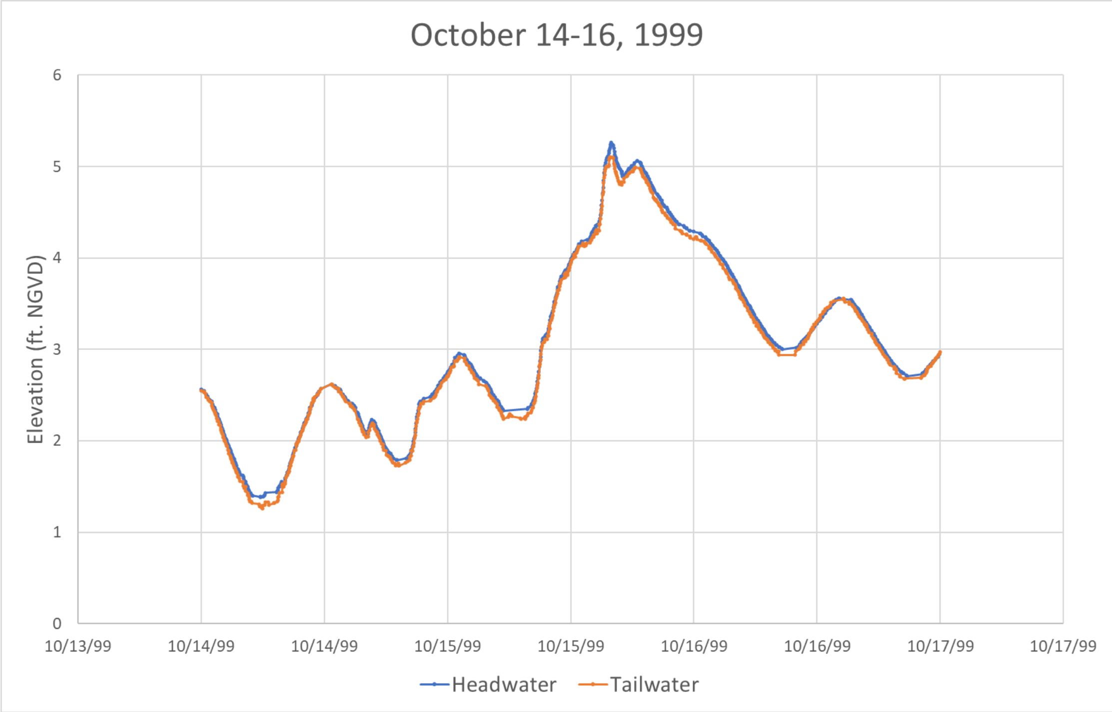


Figure 2-13 – Headwater and Tailwater Conditions for S26 (2000)

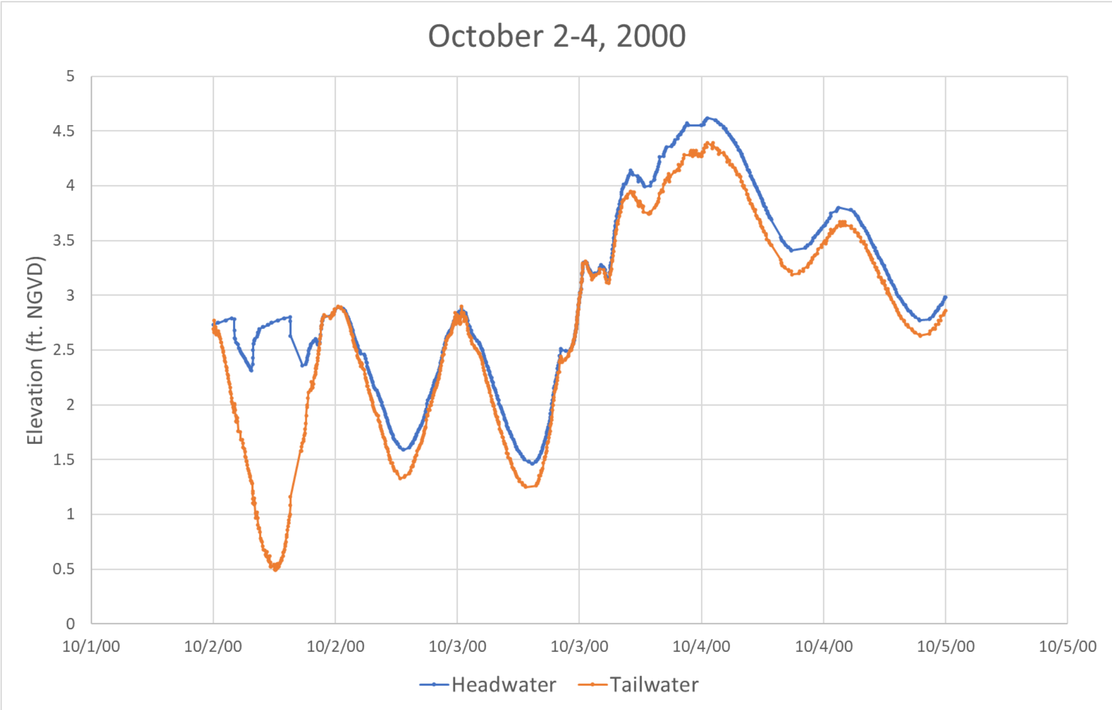


Figure 2-14 – Headwater and Tailwater Conditions for S26 (2017)

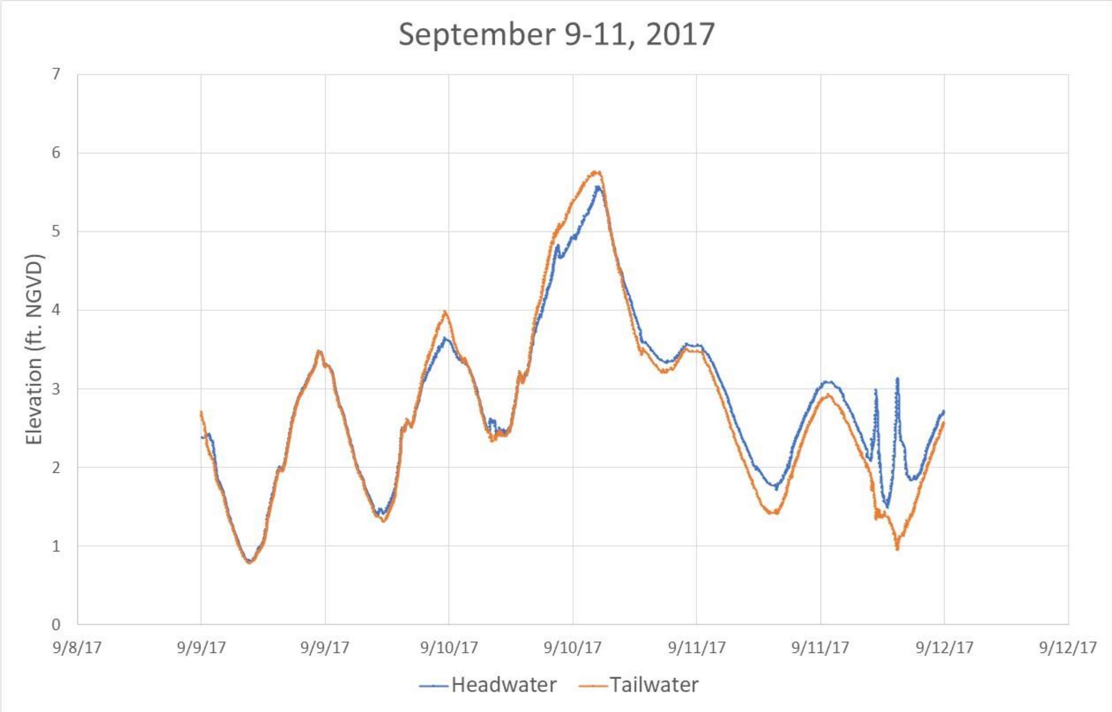


Figure 2-15 – Headwater and Tailwater Conditions for S26 (2020)

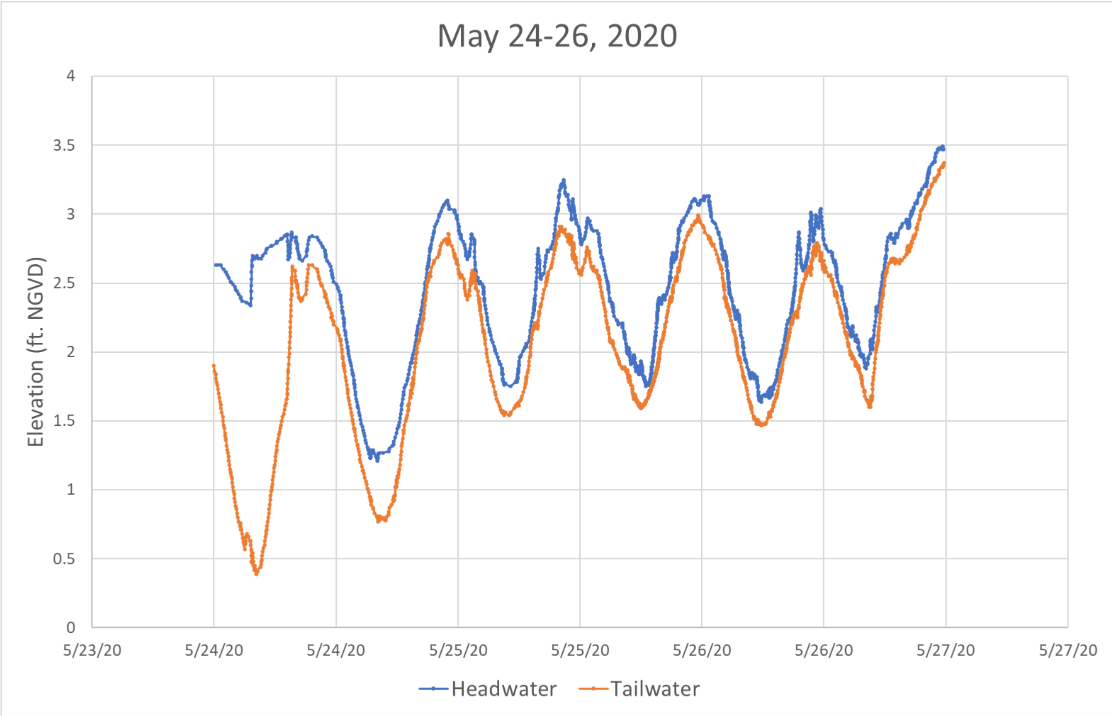


Figure 2-16 – Headwater and Tailwater Conditions for G93 (1999)

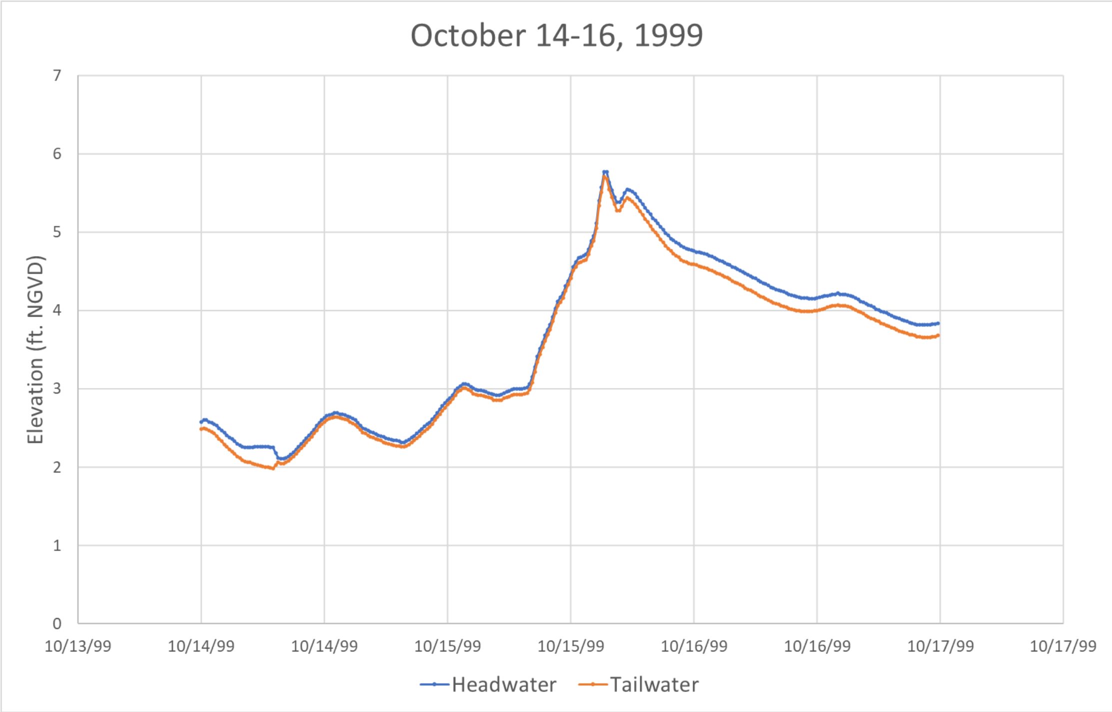


Figure 2-17 – Headwater and Tailwater Conditions for G93 (2000)

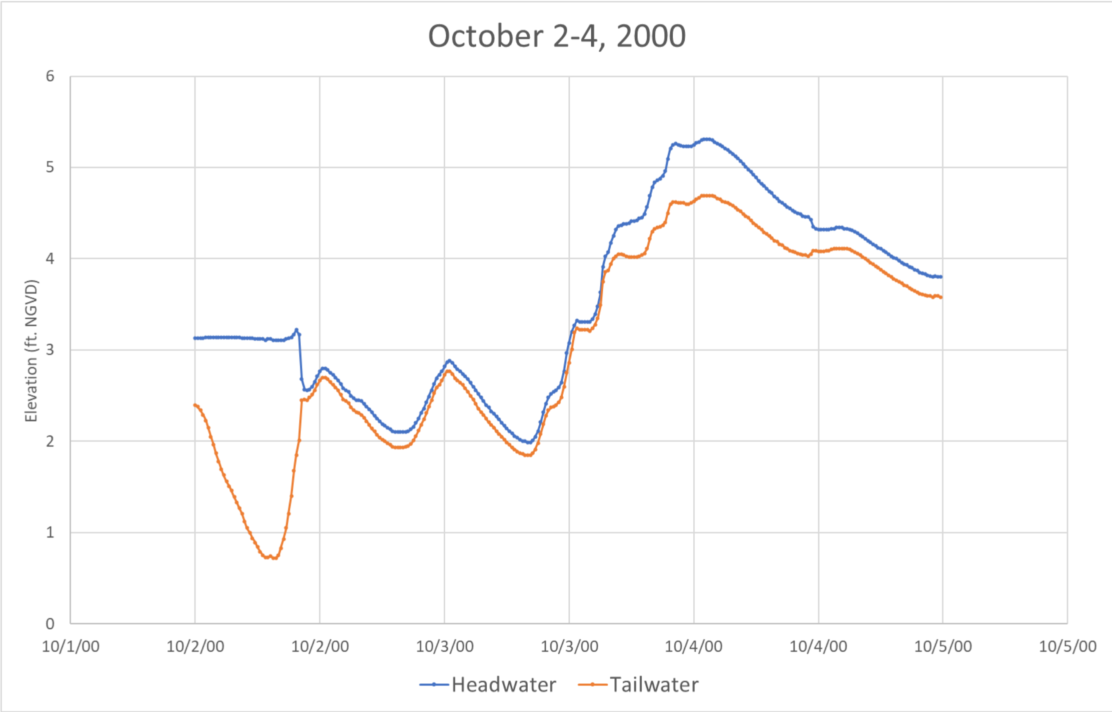


Figure 2-18 – Headwater and Tailwater Conditions for G93 (2017)

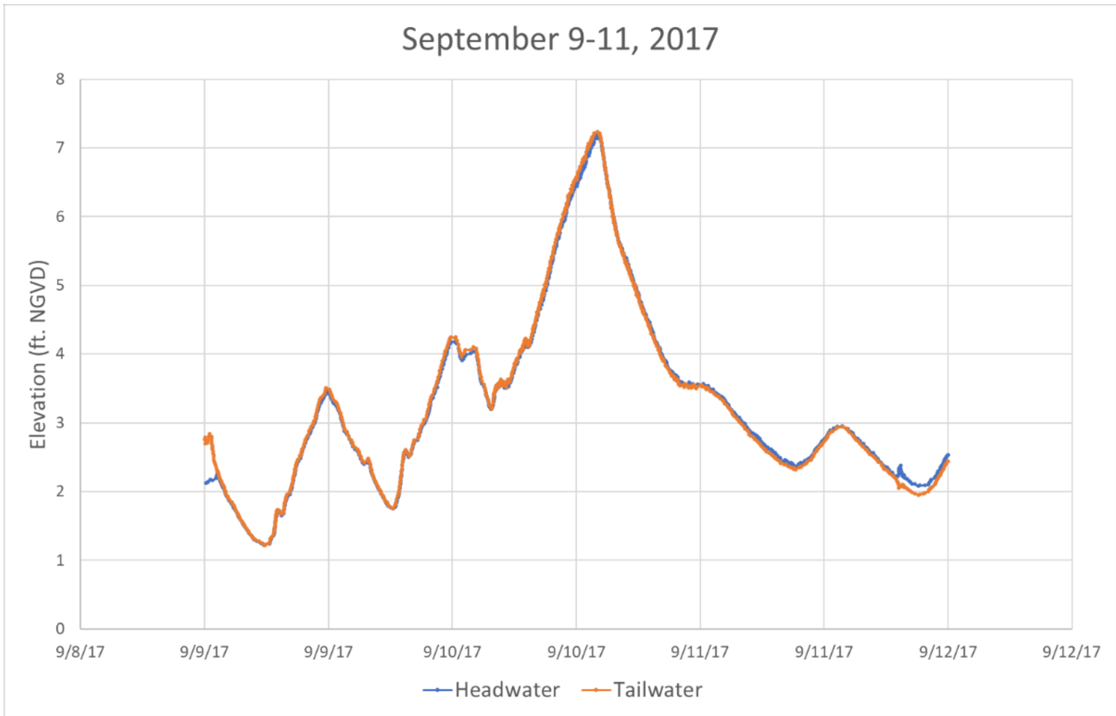
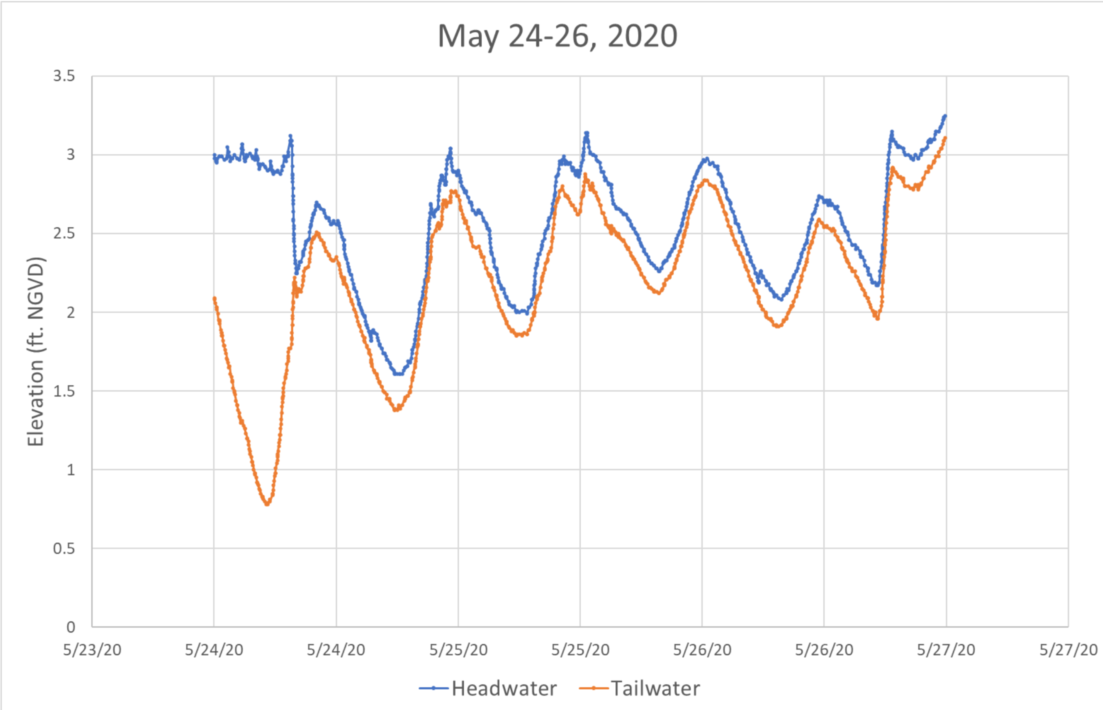


Figure 2-19 – Headwater and Tailwater Conditions for G93 (2020)



2.5.4. RECOMMENDED CALIBRATION AND VALIDATION PERIODS

Based on a review of rainfall events from the past 25 years, the recommended storm event for calibration was May 2020 while the recommended validation event was September 2017. These events were selected based on being large rainfall events with the most even spatial distribution and a variety of downstream hydraulic conditions. In addition, these two events were the most recent and therefore were more likely to represent the existing land use and infrastructure. In particular, the drainage infrastructure for the C4 and C6 watersheds changed significantly after 2004.

2.5.5. EXISTING GROUNDWATER MODEL DATA

The groundwater model developed for MIKE SHE was based on data from previous or ongoing efforts using MODFLOW based models to represent the behavior of groundwater within the project area. There are three models that have been developed since 2006 that provide coverage across the entire proposed model domain and one model currently under development. **Table 2-3** describes each model, the responsible agency, and the year it was published.

Table 2-3 – MODFLOW Models Covering the Project Area

DESCRIPTION	RESPONSIBLE AGENCY	YEAR PUBLISHED
Lower East Coast subRegional (LECsR)	SFWMD	2006
C-4 Central Dade Coastal (C4CDC)	SFWMD	2010
Urban Miami Dade County Model	USGS	2016
East Coast Surficial Model (ECSM)	SFWMD	Underway

The technical documentation and parameter files associated with each of these models were reviewed and evaluated to support the development of the hydrostratigraphy and the characterization of each layer within the MIKE SHE model throughout the domain.

2.5.6. PRELIMINARY REVIEW OF AVAILABLE DATA

The development of an integrated surface water-groundwater model requires a significant amount of spatial and temporal data to properly characterize the existing environmental conditions. Governmental agencies such as the District, USGS, USACE, NOAA, USDA, and Miami Dade County provide a significant amount of data that can be compiled and leveraged to calibrate the FPLOS model. **Table** 2-4 provides a preliminary overview of the data that was needed and the potential agencies and resources for the data.

2.6. MODELING TOOLS AND INPUT DATA RECOMMENDATIONS

The FPLOS program requires a thorough analysis of a wide array of environmental factors to properly evaluate risks and mitigation measures. In central Miami Dade County the low relief topography, high-water table and unique hydrogeology require robust tools to perform the FPLOS analysis. Based on a thorough review of available tools, it was recommended that the MIKE SHE/1D software be used to perform the H&H modeling computations needed for the assessment of the FPLOS for the C2, C3W, C4, C5 and C6 watersheds.

Considering the interconnectivity of the subject watersheds, a single model was recommended with a hydraulic and hydrogeologic domain that extends beyond the watershed boundary in several places. Based on a review of rainfall events from the past 25 years, the recommended rainfall events to be used for the calibration and validation simulations were from September 2017 and May 2020, respectively. These events were selected based on the magnitude of rainfall, the spatial distribution, and their relative recency.

Table 2-4 – Preliminary Overview of Data Needed for MIKE SHE Model

INPUT DATA	DATA TYPE	DATA SOURCE
NEXRAD Rainfall Data	2D Grid at 15-minute timestep	SFWMD
Tidal Data	Measured Timeseries	SFWMD Control Structure TW; NOAA Virginia Key
Evapotranspiration	Single timeseries applied uniformly across the model domain	SFWMD
Measured Hydraulic Data (Flows, Stages, Gate Opening)	Recorded timeseries measured at structures	SFWMD
Hydraulic Geometry: Primary & Secondary (Canal Cross-Sections, Culvert Dimensions, Bridge and Structure Geometry)	Physical Dimensions	Past SFWMD Models, MD County, As-Builts, Additional Field Survey
Tertiary Stormwater Management (Detention Storage, Exfiltration Trenches, Drainage Wells)	Shapefiles	SFWMD Permitting Records, MD County Permitting Records, As-Builts
Hydrogeologic Data (Aquifer Thickness, Conductance, Well Locations, Well Withdrawals)	2D Grids, Point Shapefiles, Timeseries	USGS, Past Models, SFWMD Permitting Records
Topography and Bathymetry	2D Grids	SFWMD LiDAR, MD County DEM, NOAA
Unsaturated Layer (Soils)	2D Grid	NRCS-USDA (SSURGO)
Land Use	2D Grid	SFWMD, MD County

3. DATA COLLECTION AND ASSIMILATION

3.1. SITE VISIT / FIELD INVESTIGATION

On February 18, 2021, CMA staff attended a site visit with members of the District staff and the FPLOS Program Management Team. Considering that the project area is roughly 200 square miles, it was not possible to investigate each canal feature and control structure. The focus of the site visits was the C4, C5 and C6 watersheds with an emphasis on control structures, key canal elements and the emergency detention basin in the western portion of the C4 watershed. **Table 3-1** provides an overview of the sites visited during the field investigation. **Figure 3-1** through **Figure 3-11** below illustrate the site conditions at each location during the site visit.

Table 3-1 – Sites Visit Locations

LOCATION	BASIN	JURISDICTION	DESCRIPTION
S-25B	C4	SFWMD	Underflow Spillway / Tidal Structure with Forward Pump Station
S-26	C6	SFWMD	Underflow Spillway / Tidal Structure with Forward Pump Station
C4 Flood Wall (C4.CORAL)	C4	SFWMD	Rehabilitated Canal Bank with Maintenance Access and Consistent Top of Bank Elevation
Belen Pumps	C4	MD County	County Owned and Operated Pumps that discharge to the C4 Canal from the City of Belen
C4 Flood Wall (T5W)	C4	SFWMD	Rehabilitated Canal Bank with Buried Sheet Pile and Concrete Cap
G-119	C4	SFWMD	Two culvert Structure with Sluice Gates
S-380	C4	SFWMD	Five Culvert Structure with Sluice Gates
G-420	C4	SFWMD	Seepage Canal Pump Station for C4 Emergency Detention Basin
G-421	C4	SFWMD	Underflow Spillway / Outlet Structure for C4 Emergency Detention Basin
G-422	C4	SFWMD	Inflow Pump Station for C4 Emergency Detention Basin
S-336	C4	SFWMD	Three Culvert Structure with Sluice Gates

Some notable observations on the site visit were that stage at the western most point visited was 4.8 ft-NAVD while the stage at the eastern most point was 1.2 ft-NAVD. This head difference of 3.6 feet over a 14-mile distance demonstrated how low the hydraulic gradients can be in the subject area. Also of note were the water stains at the tidal structures (S25B and S26), which illustrated that during high tide the water line is near an elevation that would overtop the existing underflow gates when the gates are closed.

Figure 3-1 – Site Conditions at the S-25B Structure

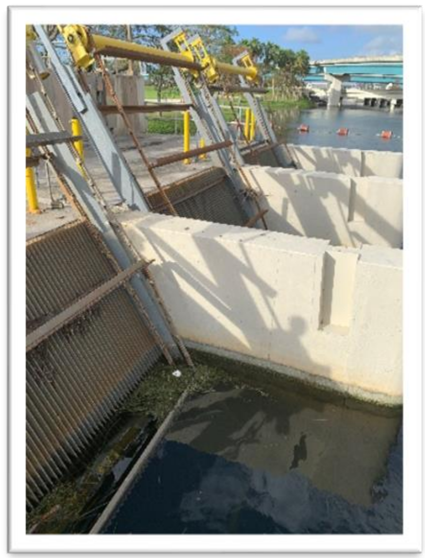


Figure 3-2 – Site Conditions at the S-26 Structure

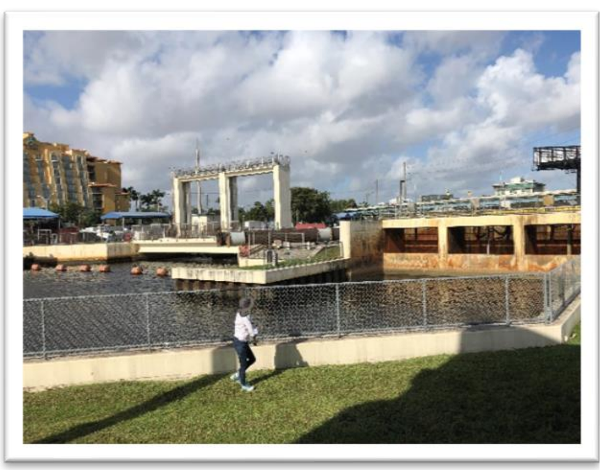


Figure 3-3 – Site Conditions at the C4 Flood Wall (C4.CORAL)

Figure 3-4 – Site Conditions at the Belen Pumps

Figure 3-5 – Site Conditions at the C4 Flood Wall (T5W)

Figure 3-6 – Site Conditions at the G-119 Structure

Figure 3-7 – Site Conditions at the S-380 Structure

Figure 3-8 – Site Conditions at the G-420 Structure

Figure 3-9 – Site Conditions at the G-421 Structure

Figure 3-10 – Site Conditions at the G-422 Structure

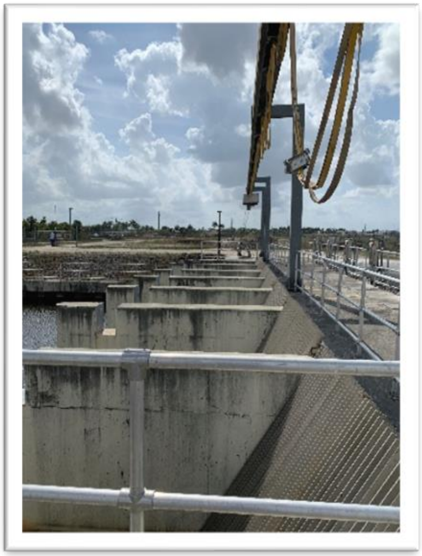


Figure 3-11 – Site Conditions at the S-336 Structure

3.2. SPATIAL DATA

There were various types of spatial data required to represent the existing conditions of the regional hydrology and hydrogeology of the model domain. The spatial data required included the following:

- Digital Elevation Model (DEM) of the topography,
- Land use that can be used to infer hydraulic parameters of the land surface,
- Extent of Environmental Resource Permits,
- Soils and soil parameters to determine the infiltration capacity of the land surface, and
- Thickness and conductance of subsurface layers in the saturated zone.

The sub-sections below describe the source, characteristics, and assumptions for each of these datasets.

3.2.1. TOPOGRAPHY

In Miami Dade County, the best available resource for topography on a regional basis is from the Light Detection And Ranging (LiDAR) dataset made available by the SFWMD. The Digital Elevation Model (DEM) used as the source data for model development utilizes a vertical datum of NAVD and has a horizontal resolution of 5 feet. The horizontal resolution of the DEM is based on the resolution of the available data sources. **Figure 3-12** Illustrates the 5 data sets that were mosaicked by District GIS staff to create the DEM, from the USGS, Miami Dade County, and SFWMD over a range of dates from 2007 through 2020 (\\gisdata1\layers\Elevation\Basic_SouthFlorida).

This dataset provides detail sufficient for representing hydrology at a local scale comparative to the tertiary network of an urban stormwater management system, however for regional modeling the 5-foot resolution is computationally inefficient. A coarser resolution DEM that maintains the necessary resolution to be representative of the primary and secondary network is more appropriate for use in the FPLOS simulations. However, any detailed data extracted for use in the model, such as berm elevations, canal cross-sections or local depressions, utilizes the finer resolution 5 feet DEM as the source.

In order to develop a more computationally efficient DEM for use in the FPLOS model, a portion of the data that represents the model domain was excerpted and then re-sampled at a grid-cell resolution that matches all other datasets being used in the MIKE SHE simulation. The re-sampled grid was developed using a raster averaging technique to upscale the 5 feet horizontal resolution to match the 125 feet computational grid cell resolution being utilized in the MIKE SHE framework. **Figure 3-13** illustrates the DEM at the original 5-foot resolution, while **Figure 3-14** illustrates the DEM that was used in MIKE SHE based on re-sampling to a resolution of 125 feet. The potential for the averaging technique to skew the data in grids where the LIDAR is picking up localized high and low elevations was evaluated using a slope calculation on the dataset. As is evident in **Figure 3-15**, the slope calculation demonstrates that elevation variations between adjacent grid cells are limited to less than 5% for all locations except select features such as highways, landfills, and mining pits. Considering this analysis, the grid averaging technique was determined to be appropriate. A visual comparison demonstrates that the effects of re-sampling is not significant at a regional scale, however as noted in a comparison of **Figure 3-16** and **Figure 3-17** which illustrate the Miami International Airport, the impact of upscaling is seen only at the local scale.

Figure 3-12 – Data Sources for Topographic Data

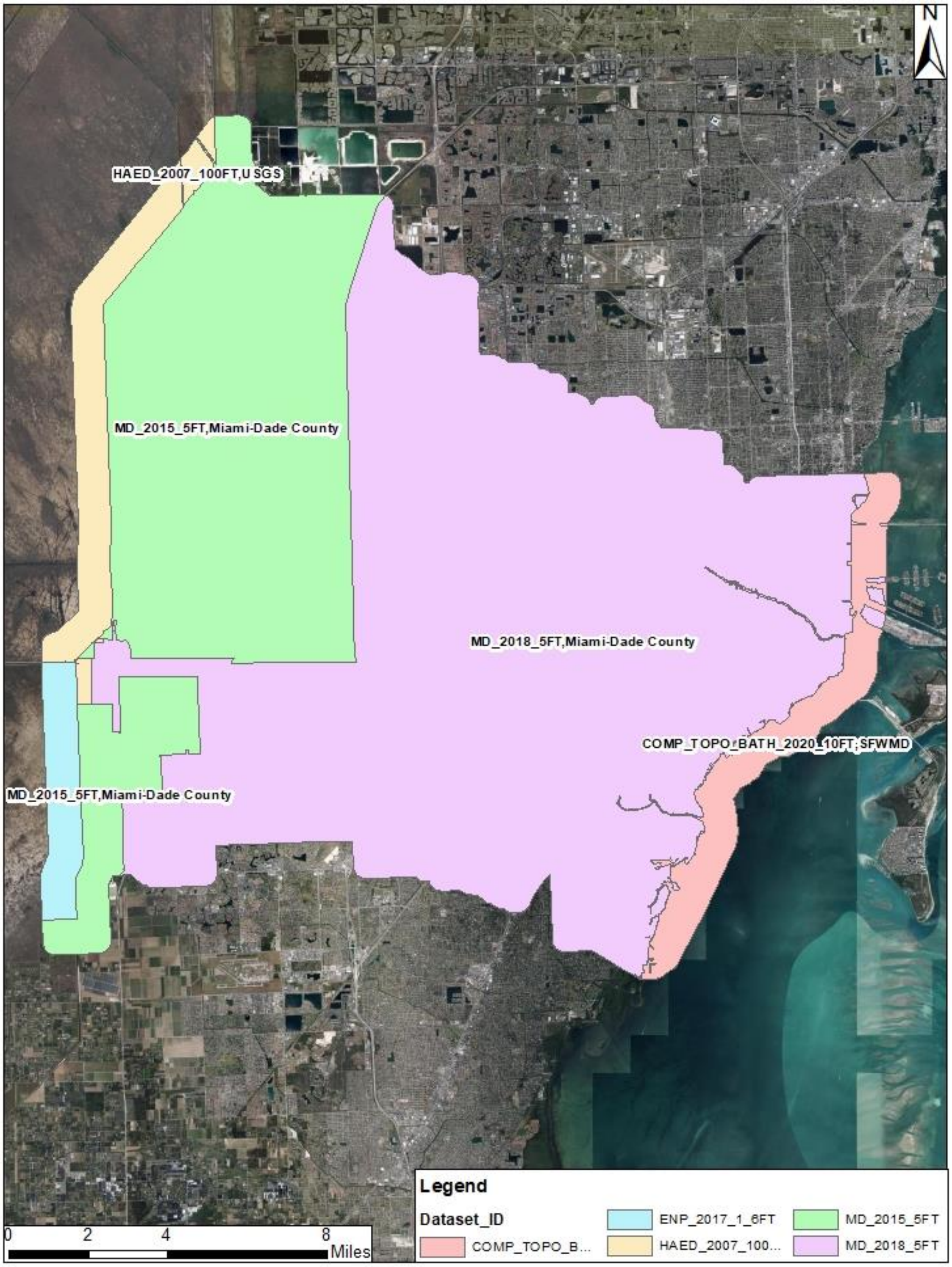


Figure 3-13 – Regional Scale DEM of the Model Domain at 5-ft Horizontal Resolution

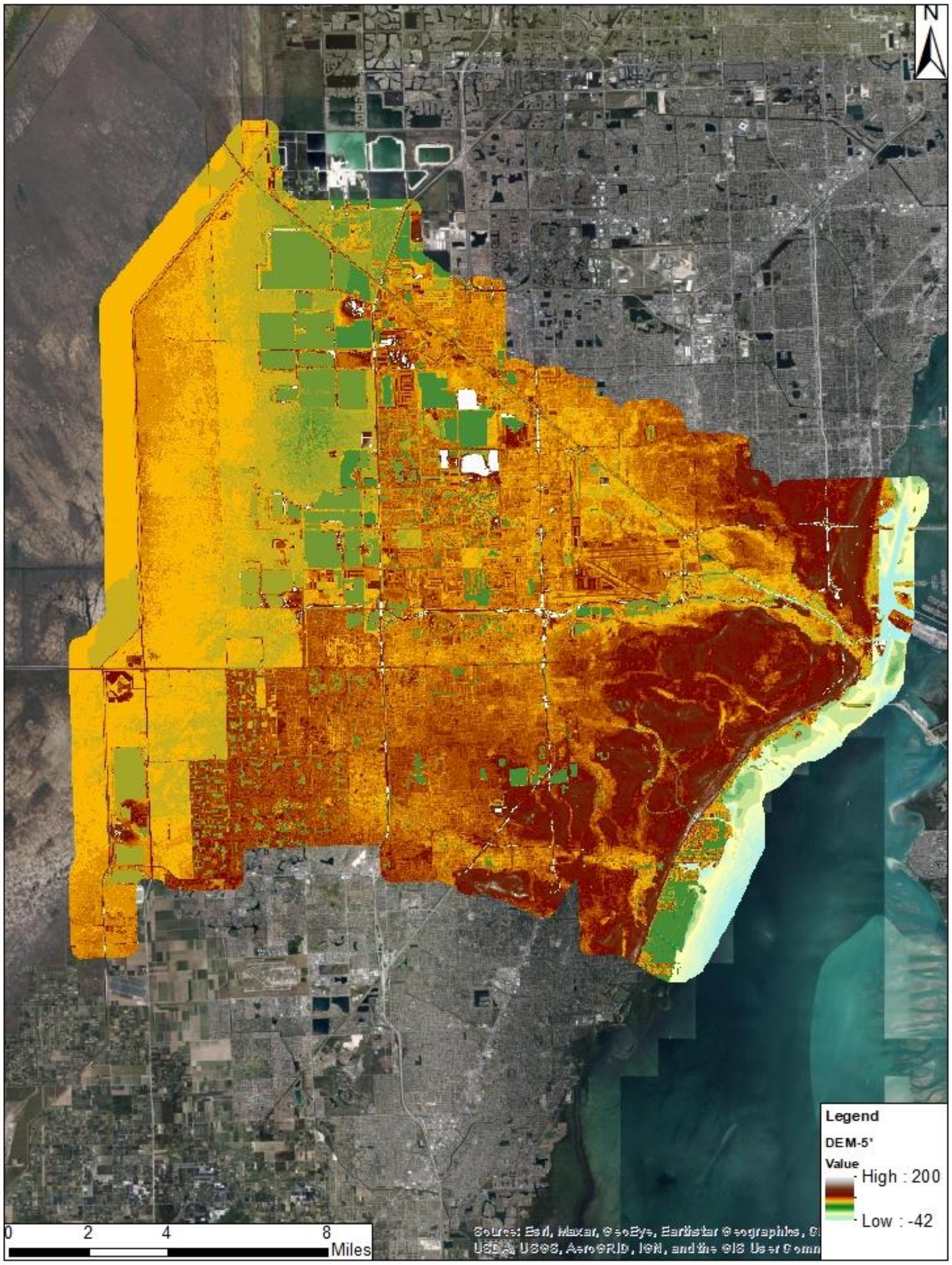


Figure 3-14 – Regional Scale DEM of the Model Domain at 125-ft Horizontal Resolution

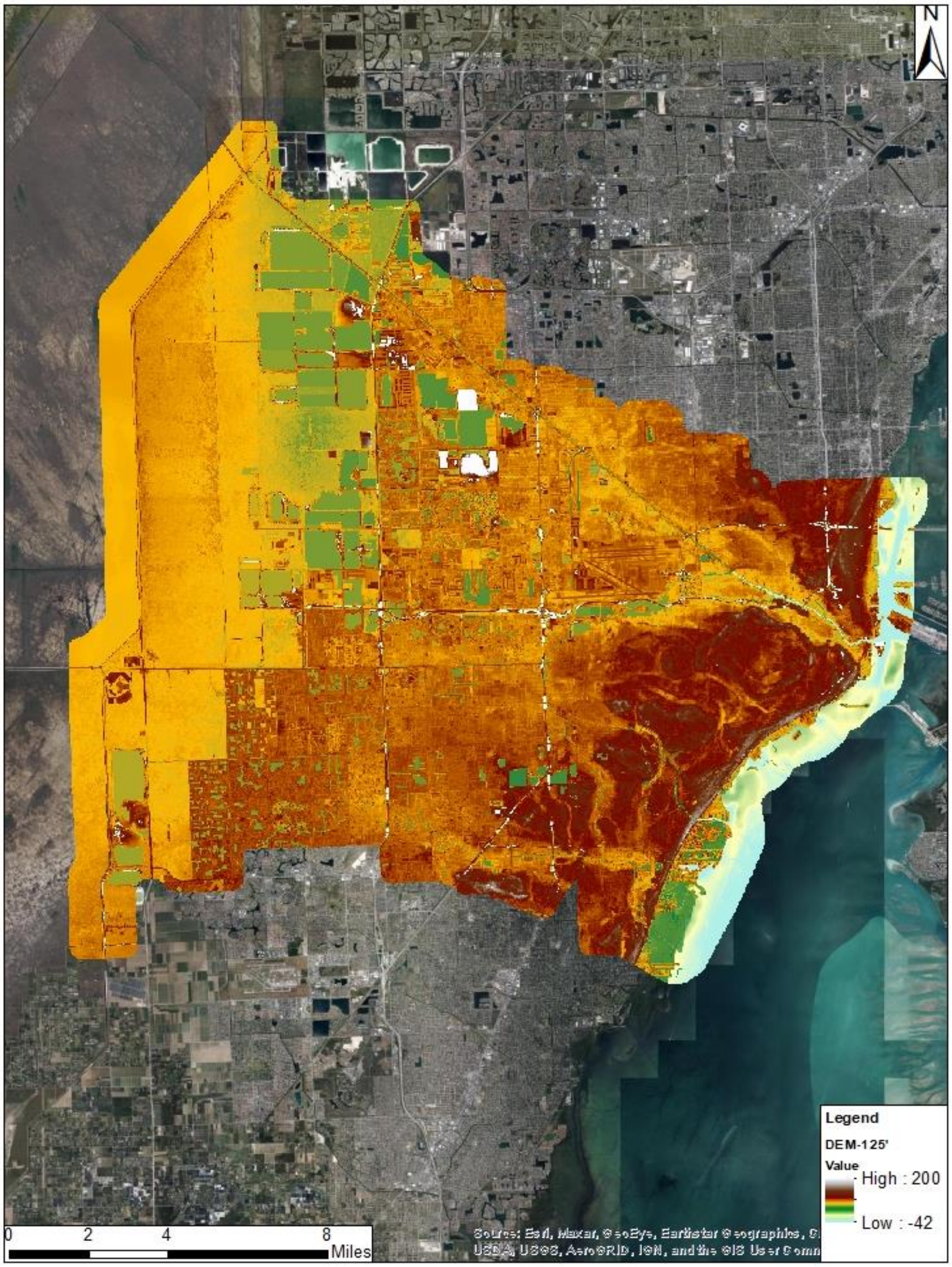


Figure 3-15 – Percentage of Slope for 5-ft Resolution DEM

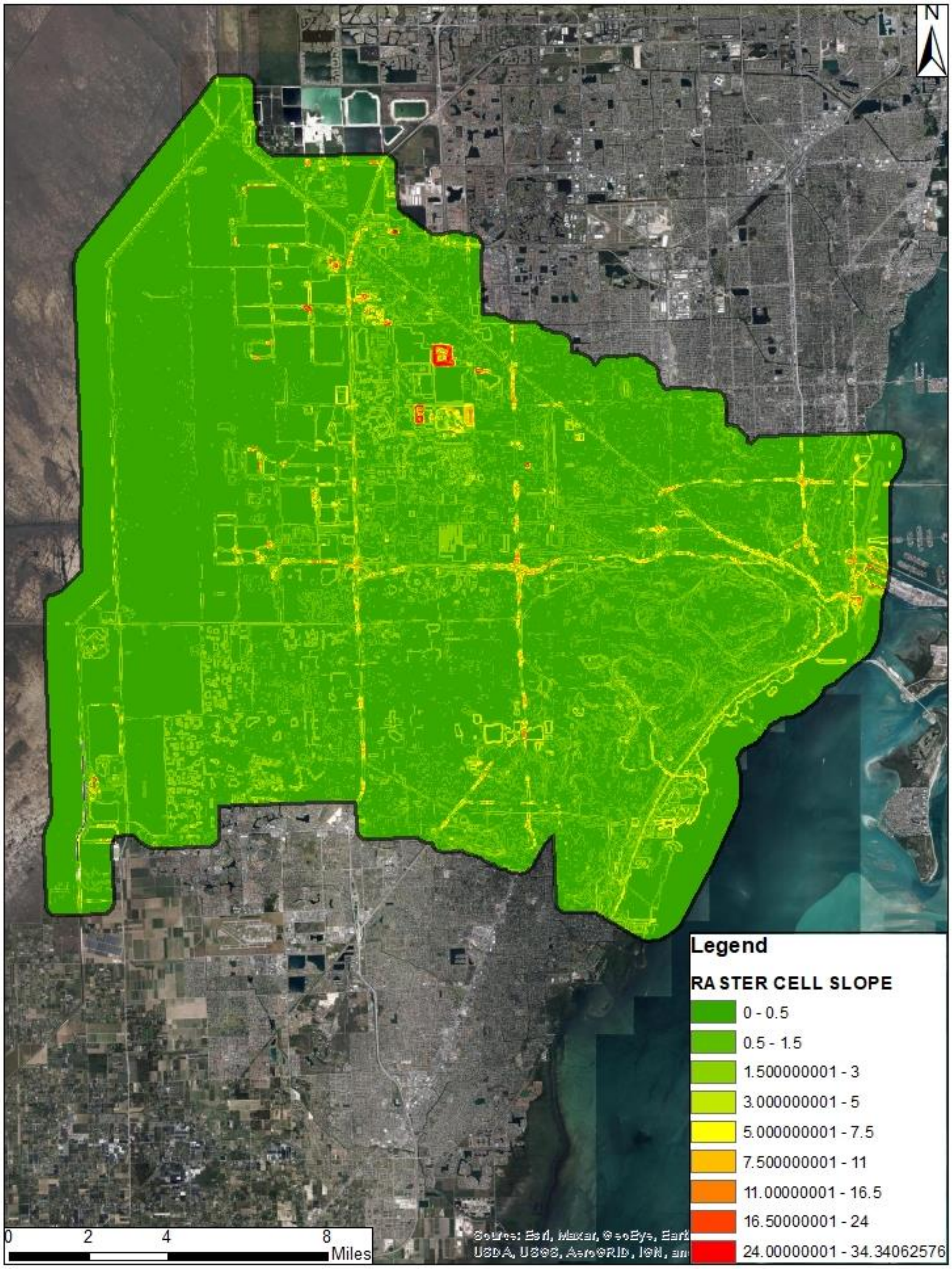


Figure 3-16 – Local Scale DEM for Miami International Airport at 5-ft Horizontal Resolution

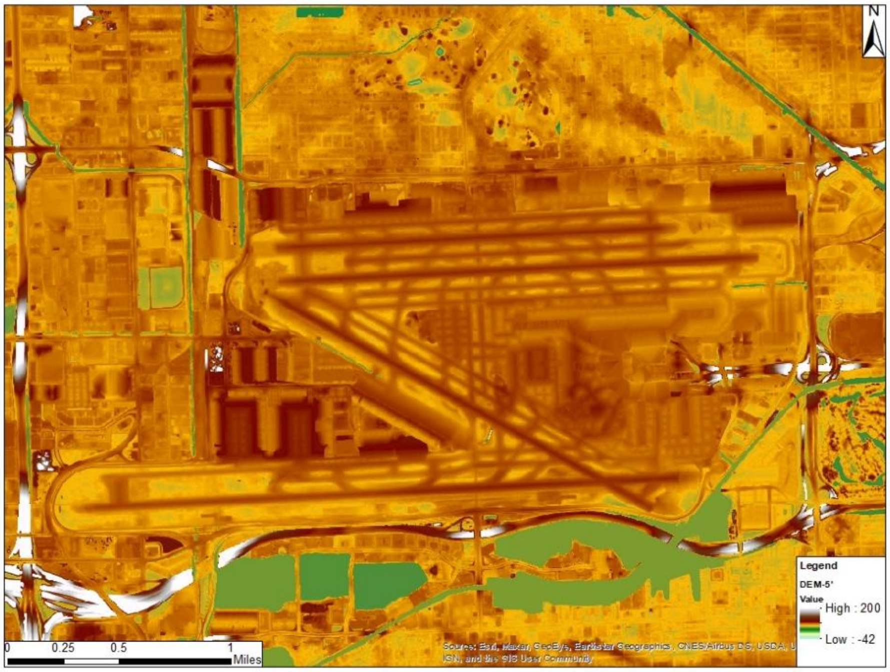
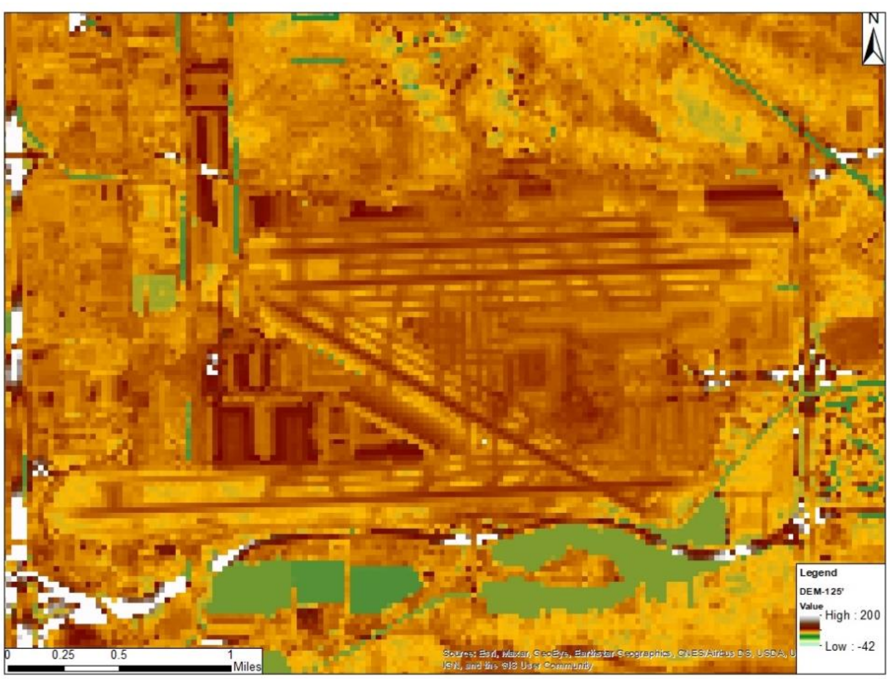


Figure 3-17 – Local Scale DEM for Miami International Airport at 125-ft Horizontal Resolution



The comparison of the resolution at the Miami International Airport illustrated that the re-sampling of the topographic data did decrease the level of detail with which the edges of individual features were represented. However, even at the lower resolution, the topographic features that affect hydrology were still prominent as evidenced by the grading differences between the impervious runways and taxiways at the airport and the pervious infield areas between them. Considering that the objective of this project was to represent the primary and secondary stormwater management systems which are not sensitive to small variations in topography, the re-sampled DEM provided the level of detail necessary while being more computationally efficient than the high-resolution DEM.

3.2.2. LAND USE - ROUGHNESS AND DETENTION STORAGE

In addition to topographic data, representation of overland flow in MIKE SHE also requires parameterization of the roughness coefficient and the available detention storage for the land surface. A map of land use based on the Florida Land Use Cover Classification System (FLUCCS) categorization codes was available for the model domain through the Florida Department of Environmental Protection (FDEP) for a range of years between 2017 and 2019. This dataset provided an analog to the potential resistance for overland flow and availability of detention storage. **Figure 3-18** illustrates the variability in land use within the model domain as representative, while **Table 3-2** provides a description of each FLUCCS code and the corresponding Manning’s M value, Manning’s n value and detention storage in inches.

Manning’s M is the roughness coefficient framework utilized by the MIKE SHE platform and is computed as the inverse of Manning’s n. The definition of Manning’s M for each land use was developed based on prior models in the region and professional judgment. Detention storage is defined within the MIKE SHE platform as the inches of ponding required before overland flow occurs. Rainfall held in detention storage was available for infiltration into the unsaturated zone and recharge to the saturated zone. Similar to Manning’s M, the parameterization of detention storage for each land use type was developed based on a review of prior models in the region and professional judgement with values ranging from 0.0 to 0.4 inches.

Figure 3-19 illustrates the extent of ERP coverage within the model domain. This coverage was reviewed and included in the model for the overland detention storage. For locations where the surface water management system documented in the ERP has a detention or retention system with an outfall structure, these facilities were represented by drain codes with a representative time attenuation constant.

Figure 3-18 – Land use Map for Model Domain

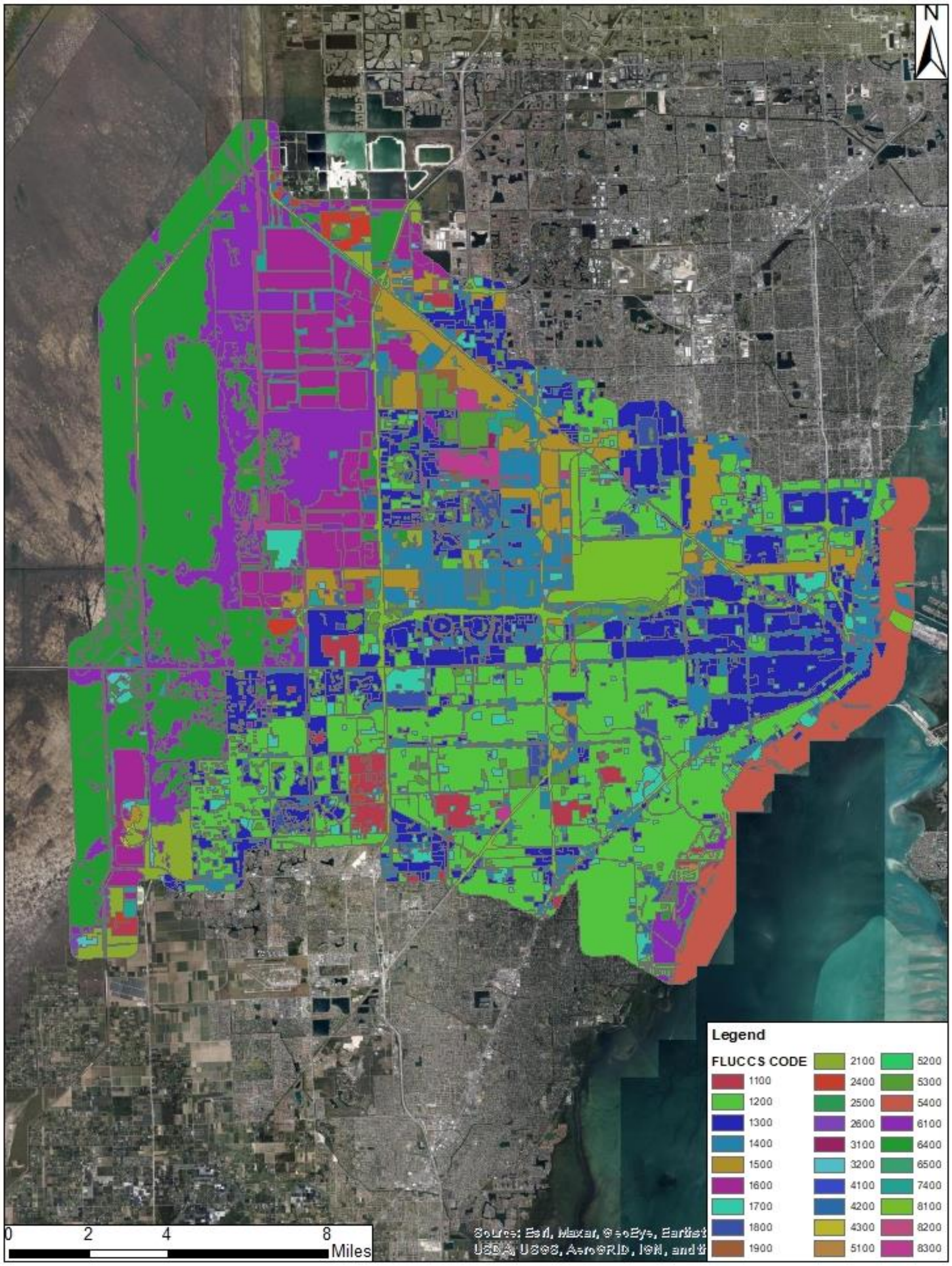
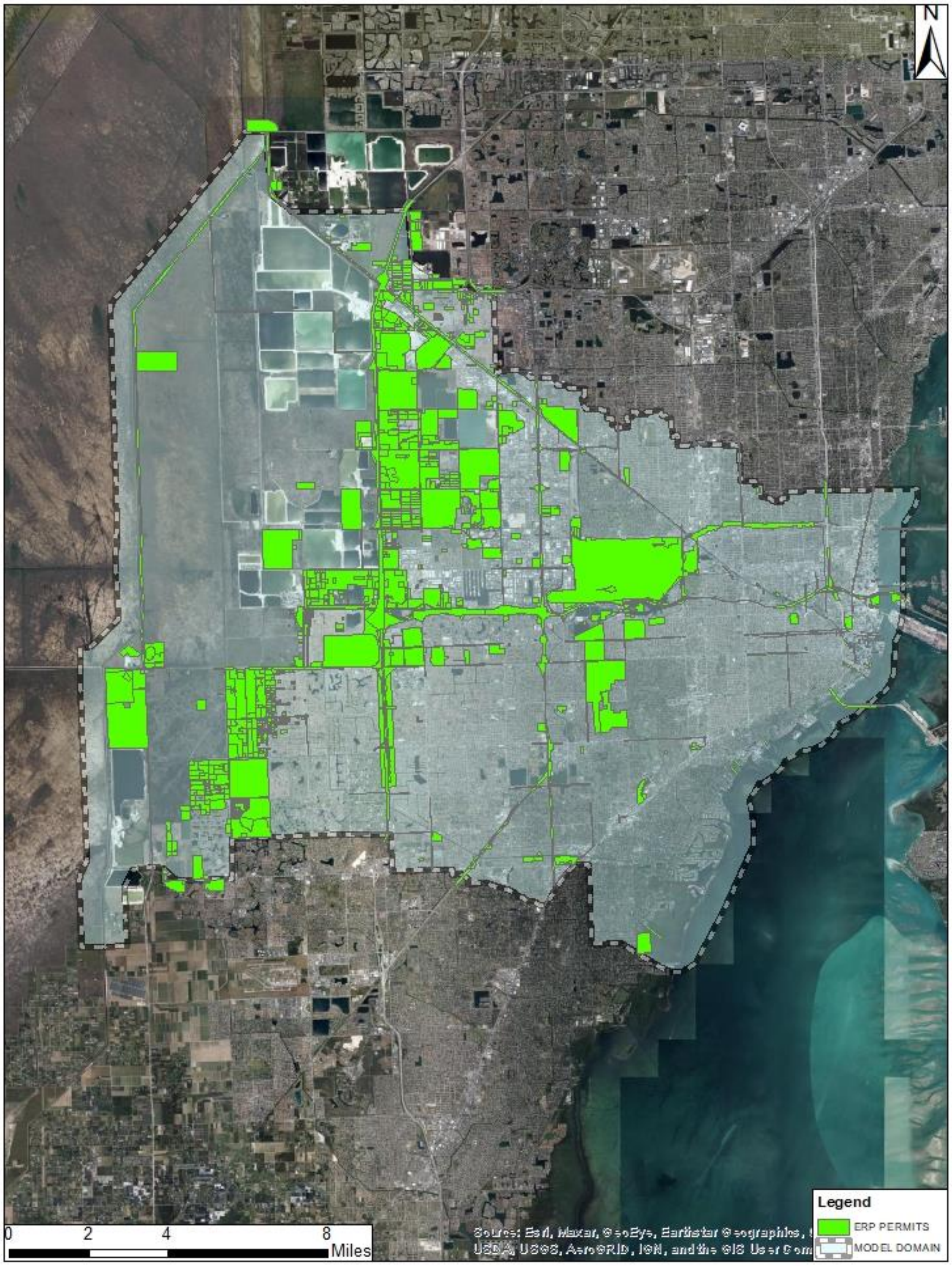


Table 3-2 – Manning's M for Land Uses in the Model Domain

FLUCCS Code	Land Use	Manning's M	Manning's N	Detention Storage [in]
1100	Residential, Low Density	7.14	0.14	0.10
1200	Residential, Medium Density	8.33	0.12	0.10
1300	Residential, High Density	9.09	0.11	0.10
1400	Commercial and Services	14.29	0.07	0.10
1500	Industrial	14.29	0.07	0.10
1600	Extractive	16.67	0.06	0.10
1700	Institutional	7.69	0.13	0.10
1800	Recreational	7.69	0.13	0.30
1900	Open Land	7.14	0.14	0.15
2100	Cropland and Pastureland	5.88	0.17	0.15
2200	Tree Crops	5.88	0.17	0.25
2300	Feeding Operations	5.88	0.17	0.25
2400	Nurseries and Vineyards	5.88	0.17	0.25
2500	Specialty Farms	5.88	0.17	0.25
2600	Other Open Lands - Rural	7.14	0.14	0.15
3100	Herbaceous (Dry Prairie)	7.69	0.13	0.15
3200	Upland Shrub and Bushland	3.33	0.30	0.15
3300	Mixed Rangeland	3.33	0.30	0.15
4100	Upland Coniferous Forest	2.22	0.45	0.40
4200	Upland Hardwood Forest	2.22	0.45	0.40
4300	Upland Mixed Forest	2.22	0.45	0.40
4400	Tree Plantations	2.22	0.45	0.40
5100	Streams and Waterways	16.67	0.06	0.00
5200	Lakes	16.67	0.06	0.00
5300	Reservoirs	16.67	0.06	0.00
5400	Bays and Estuaries	16.67	0.06	0.00
5700	Ocean and Gulf	16.67	0.06	0.00
6100	Wetland Hardwood Forest	2.22	0.45	0.40
6400	Vegetated Non-Forested Wetlands	3.33	0.30	0.40
6500	Non-Vegetated Wetlands	6.67	0.15	0.00
7200	Sand other than Beaches	16.67	0.06	0.10
7400	Disturbed Land	7.14	0.14	0.10
8100	Transportation	9.09	0.11	0.10
8200	Communications	7.14	0.14	0.10
8300	Utilities	7.14	0.14	0.10

Figure 3-19: Limits of Environmental Resource Permits within the Model Domain



3.2.3. UNSATURATED ZONE

Another characteristic that requires parameterization to properly represent the rainfall-runoff process is the infiltration capacity of the soil near the surface in the unsaturated zone. Using the 2-Layer Water Balance Method, the MIKE SHE framework allows for infiltration capacity to be characterized for each soil type within the model domain based on three (3) soil parameters: porosity field capacity, and wilting point. To provide a representative parameterization of the soil characteristics across the model domain, a regional soils dataset was required.

The Natural Resources Conservation Service (NRCS) maintains a spatial database of soil types and characteristics throughout the country. This database provided a map of the spatial extent as well as a variety of detailed information about the soil characteristics as collected by the National Cooperative Soil Survey. This data was available in Miami Dade County throughout the extent of the model domain in the format of the Soil Survey Geographic Database (SSURGO) and the 2018 version of the dataset was available for download from the Florida Geographic Data Library. **Figure 3-20** illustrates the variability in soil type within the model domain, while **Table 3-3** provides a description of each soil type and the corresponding infiltration capacity characteristics. The soil characteristics provided describe the water content under various conditions as well as the saturated hydraulic conductivity. The reference source used to define the quantitative values of these characteristics was the soil data characteristics documented within the NRCS Web Soil Survey database.

3.2.4. SATURATED ZONE (HYDROGEOLOGY)

The saturated zone modeled in MIKE SHE is using 3D finite difference computations. As such various data grids were needed to satisfy the computational inputs. This model's saturated zone was parameterized based on the layering from the Urban Miami Dade MODFLOW model created by USGS (Hughes and White, 2016). In this model setup, three model layers represent the Biscayne aquifer from the land surface to the top of the Tamiami Formation, as illustrated in **Figure 3-21**. No leakance was assumed beneath the lowest layer.

The bottom elevation, hydraulic conductivity, specific storage, and specific yield of each layer was rasterized from the MODFLOW datasets to create appropriate input data for the MIKE model. The Urban Miami Dade MODFLOW model used a 1640.42-foot by 1640.42-foot (500-meter by 500-meter) grid to compute the groundwater data. To best incorporate this data into the model, the grid was resampled using the "nearest neighbor" technique to fit the 125-foot by 125-foot grid created for this model.

Table 3-3 – Infiltration Capacity Characteristics of Soils within the Model Domain

MUKEY	Soil Name	Water Content at Saturation [%]	Water Content at Field Capacity [%]	Water Content at Wilting Point [%]	Saturated Hydraulic Conductivity [ft/day]
631567	Biscayne gravelly marl, drained	15.00	15.00	7.50	12.09
631577	Biscayne marl	17.00	15.00	8.00	9.46
631580	Biscayne marl, drained	17.00	17.00	10.00	9.77
631588	Biscayne marl-rock outcrop complex	18.00	17.00	10.00	14.31
631586	Chekika very gravelly loam	10.00	14.00	7.50	11.06
631578 1389873	Dania muck, frequently ponded, 0 to 1 percent slopes	36.00	49.80	11.30	24.58
631590	Demory sandy clay loam-rock outcrop complex	12.00	8.60	3.30	7.64
631596	Hallandale fine sand, 0 to 2 percent slopes	8.00	8.90	2.70	19.82
631572	Krome very gravelly loam	10.00	14.00	11.00	9.54
631568 1389883	Lauderhill muck, frequently ponded, 0 to 1 percent slopes	40.00	55.00	12.00	25.12
631587	Matecumbe muck	22.00	90.00	45.00	23.87
631585	Opalocka sand-rock outcrop complex	4.00	5.20	1.50	44.56
631592	Pahokee muck, depressional	23.00	90.00	45.00	25.77
631593	Pennsuco marl, tidal	17.00	44.30	13.90	3.69
631576	Perrine marl	31.00	56.20	4.80	7.87
631595	Plantation muck	13.00	46.90	22.00	25.62
631599	Rock outcrop-vizcaya-biscayne complex	4.00	0.00	0.00	22.11
631582	Tamiami muck, depressional	22.00	71.50	36.10	21.24
631594	Terra ceia muck, tidal	35.00	90.00	45.00	26.08
631603	Udorthents, limestone substratum, 0 to 5 percent slopes	4.00	4.00	1.10	25.62
631574	Udorthents, limestone substratum-urban land complex	4.00	0.00	0.00	25.81
631575	Udorthents, marl substratum-urban land complex	23.00	15.20	8.00	10.85
631573	Udorthents-water complex	4.00	0.00	0.00	18.43
631579	Urban land, 0 to 2 percent slopes	30.00	20.00	8.00	25.80

Figure 3-20 – Soils Map for Model Domain

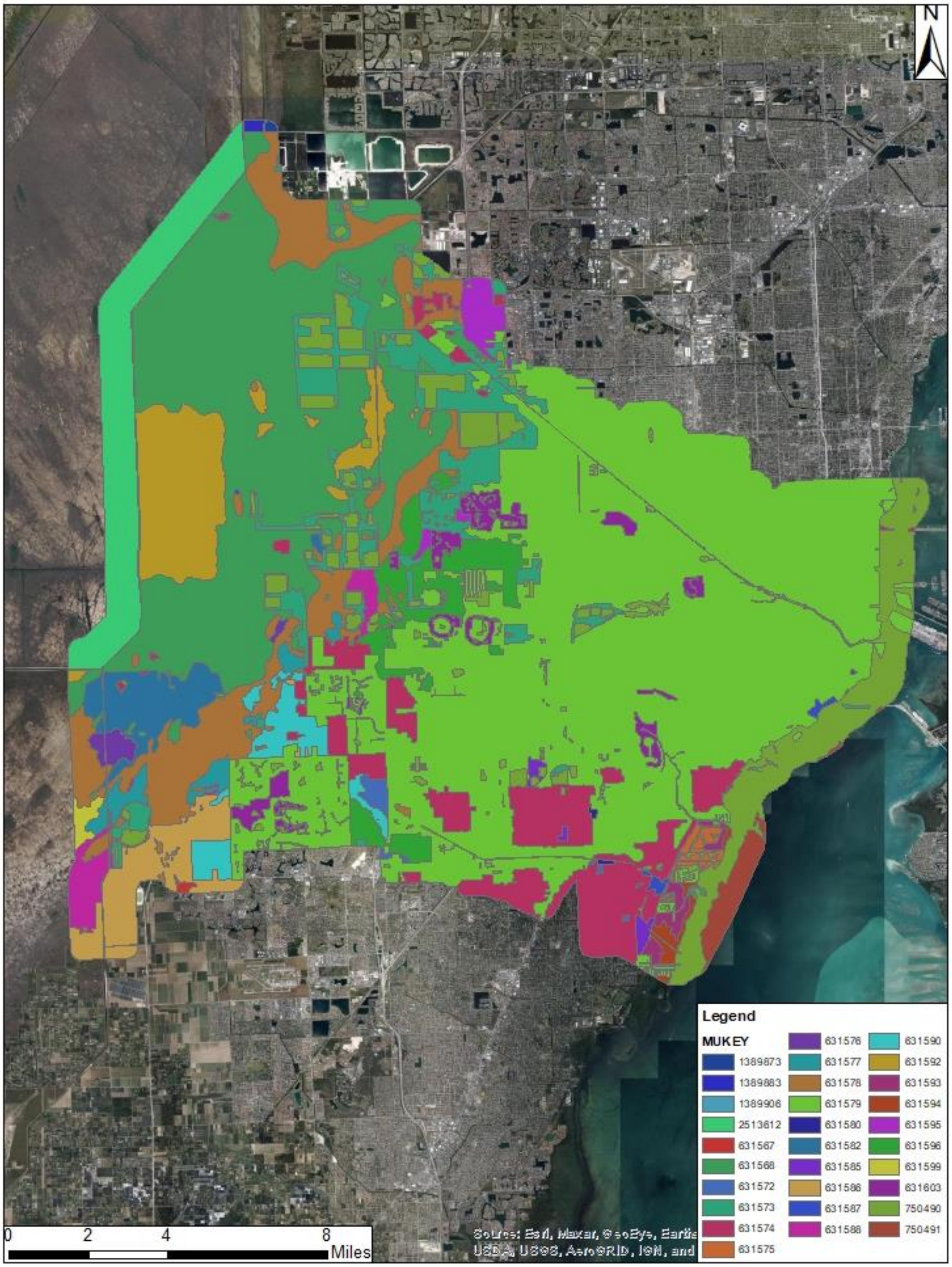
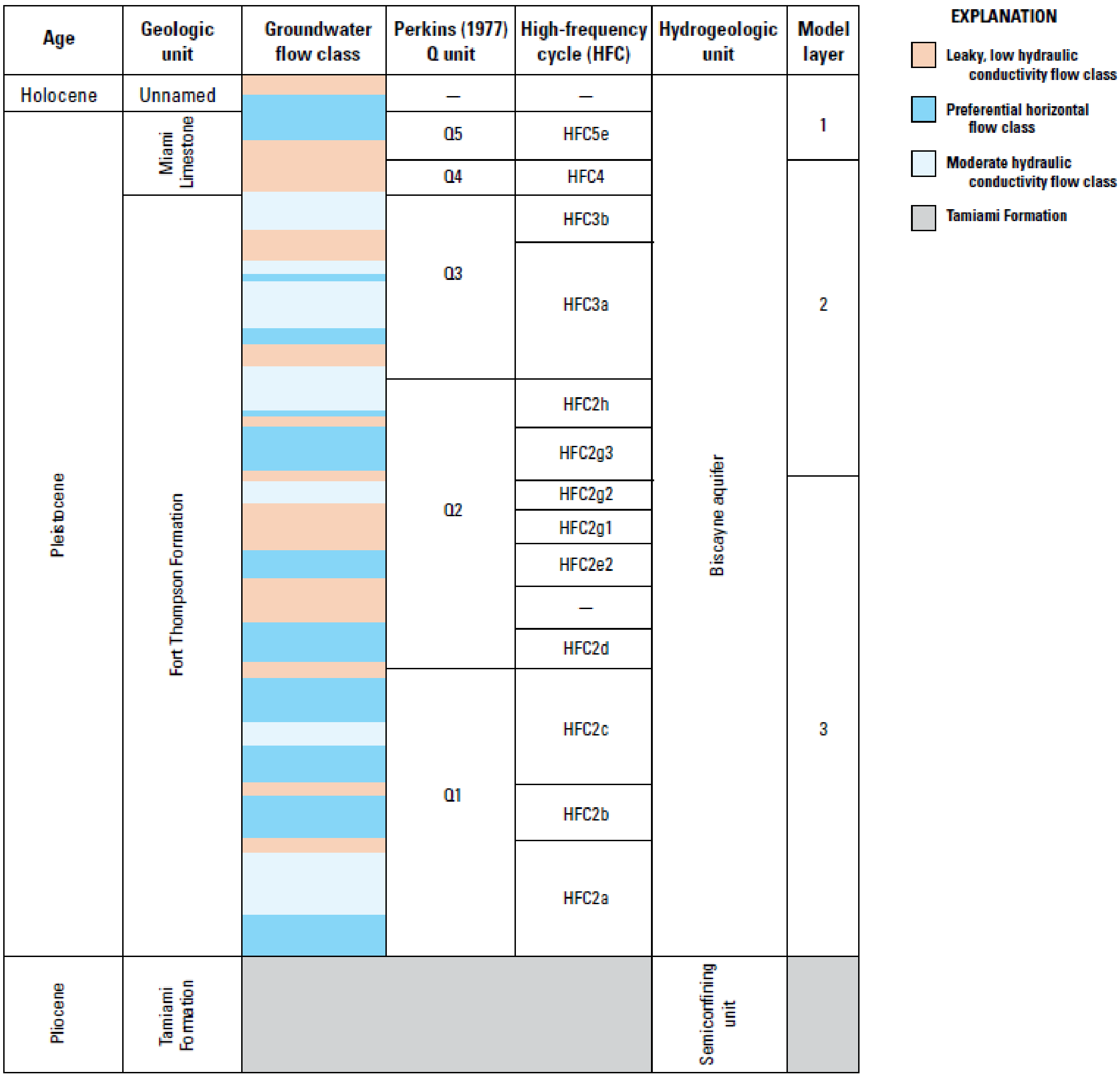


Figure 3-21 – Representation of Model Layers for Biscayne Aquifer (Hughes and White, 2016)



3.2.5. LAYER THICKNESS

Figure 3-22 through **Figure 3-24** show the bottom elevation of the three saturated zone layers in the MIKE software.

3.2.6. CONDUCTANCE

The Urban Miami Dade USGS model assumed that the differences between the vertical and horizontal hydraulic conductivity for the groundwater layers were small, therefore the data in the MODFLOW model showed equal values for horizontal and vertical hydraulic conductivity. A more typical modeling assumption is a ratio of horizontal to vertical conductivity of 10 to 1. This relationship was found in the LECsR and ECSM parameterization. This ratio was later identified as a calibration parameter and tested as part of Model Calibration. The range of hydraulic conductivity values for each model layer as defined by the USGS model within our domain are provided in **Table 3-4**, while **Figure 3-25** through **Figure 3-27** illustrate the spatial variability.

Table 3-4 – Range of Hydraulic Conductivity and Specific Yield from Urban Miami Dade Model

MODEL LAYER	HYDRAULIC CONDUCTIVITY (FT/DAY)	SPECIFIC YIELD
L1	317 to 10000	0.079 to 0.8
L2	0.614 to 1000	0.079 to 0.8
L3	387 to 10000	0.079 to 0.8

3.2.7. SPECIFIC STORAGE AND YIELD

In unconfined aquifers, the specific yield is equated to the storage coefficient. In the USGS model there is horizontal variability shown for the specific yield values, but the values are the same for each of the three vertical layers. In the USGS model the values for specific yield vary from 0.1 to 1.0 across the model domain. The high values are associated with locations where mining activities have created deep lakes that intercept each of the vertical layers. **Figure 3-28** demonstrates the spatial variability of specific storage and yield documented in the USGS model. A sensitivity analysis was performed on this parameter to determine the effects on the model and is further discussed in **Section 5**.

Figure 3-22 – Lower-Level Depth of Layer 1 for the Saturated Zone Model

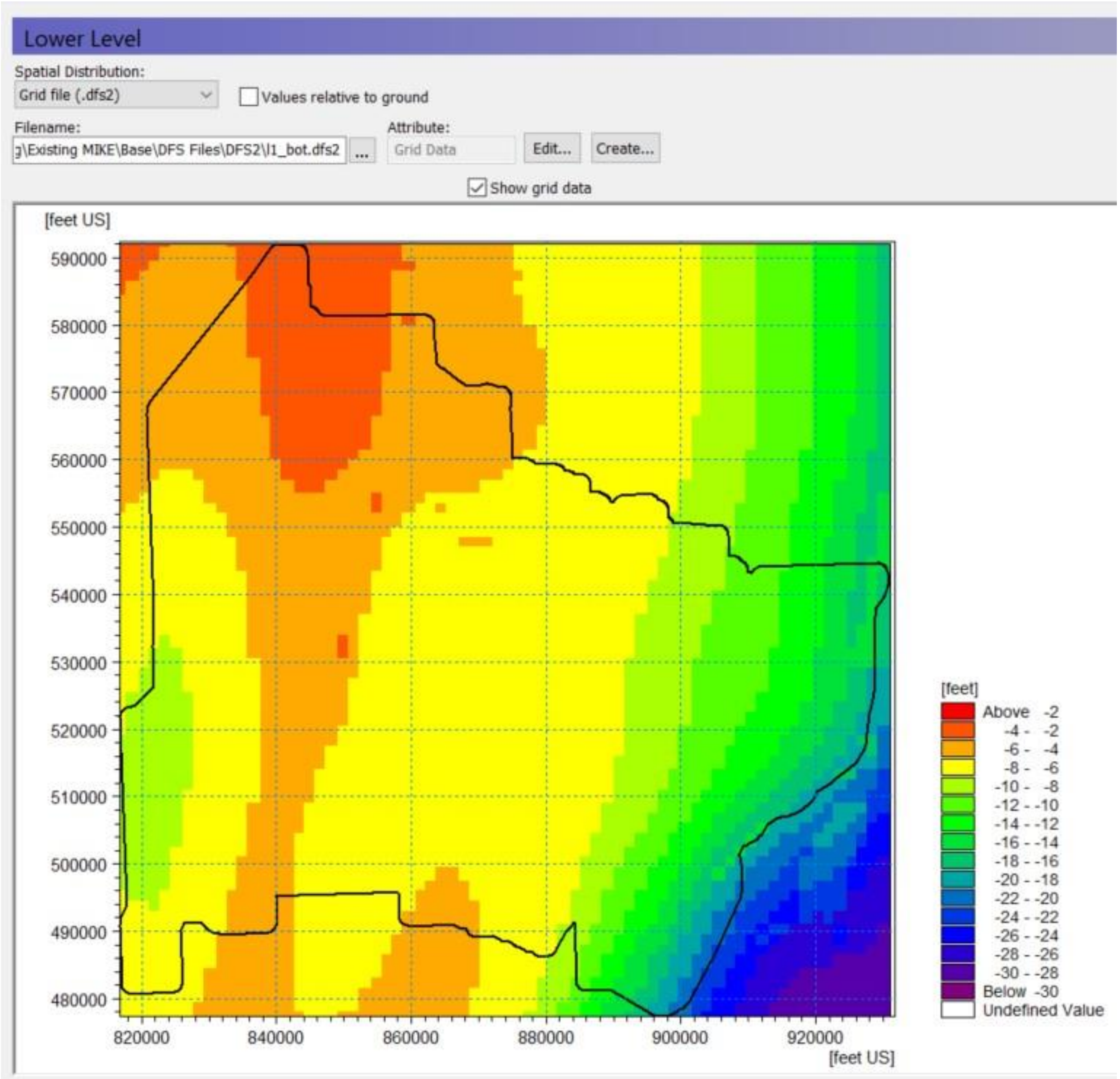


Figure 3-23 –Lower-Levell Depth of Layer 2 for the Saturated Zone Model

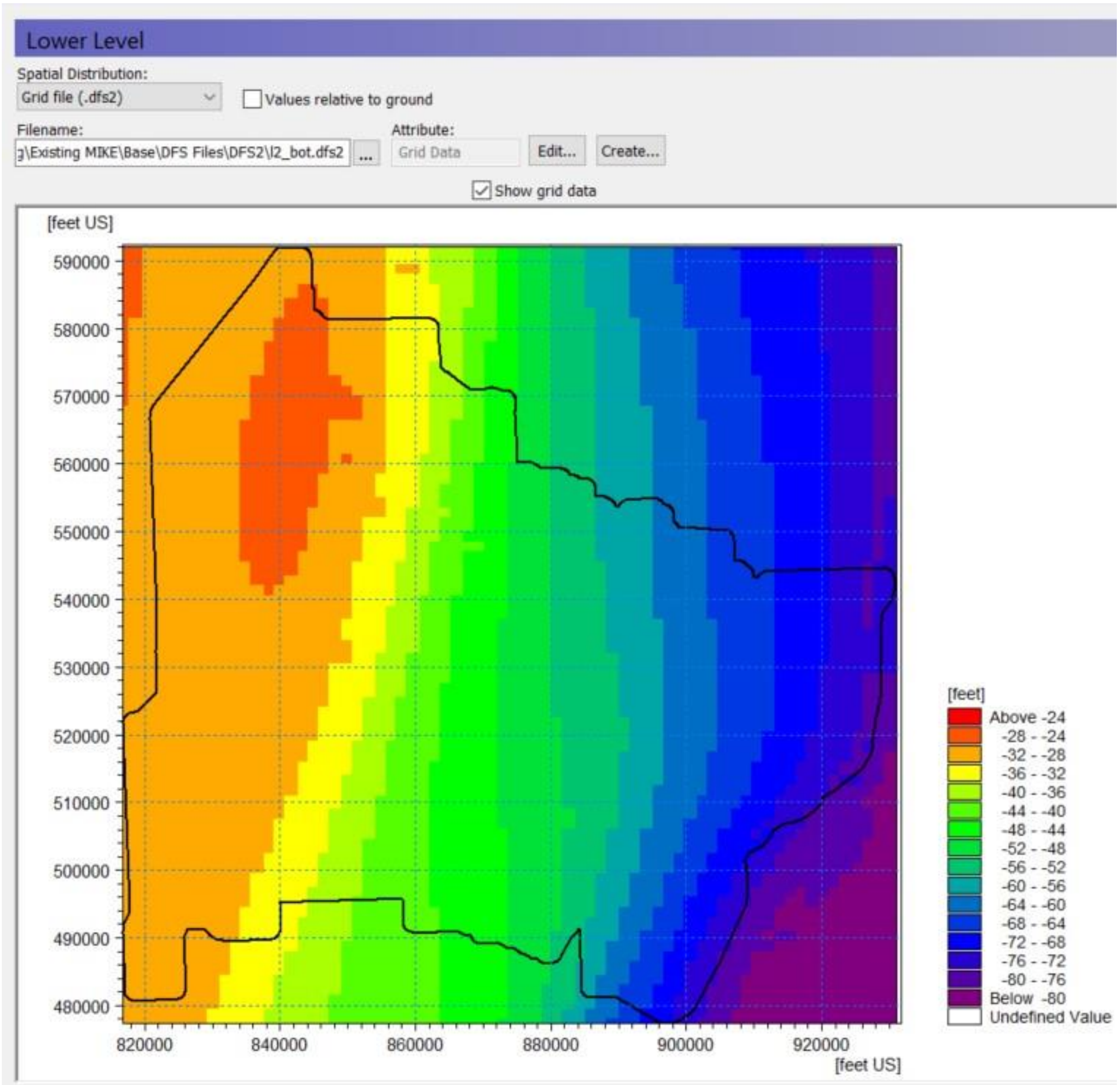


Figure 3-24 – Lower-Level Depth of Layer 3 for the Saturated Zone Model

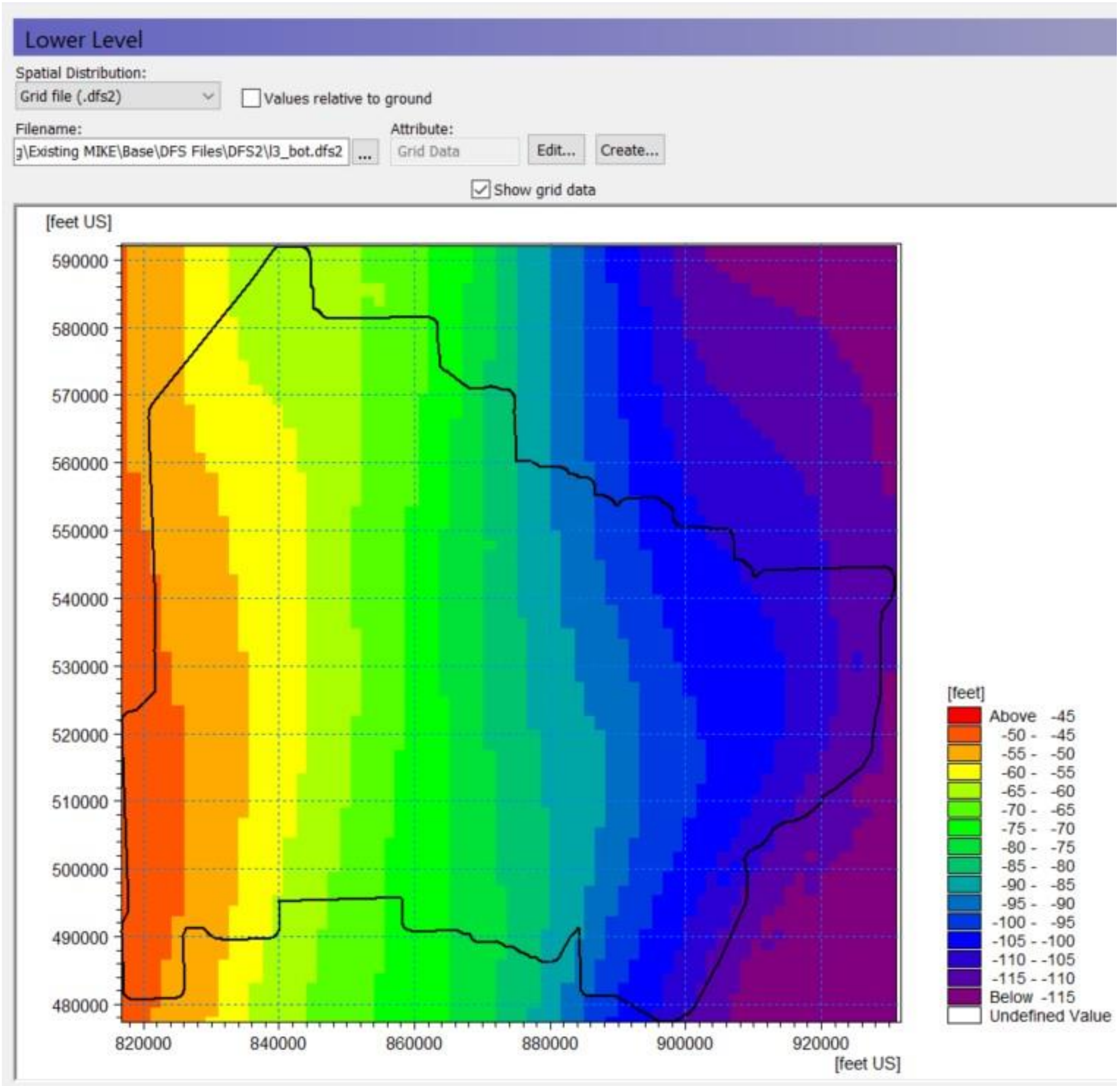


Figure 3-25 – Horizontal Conductivity of Layer 1 for the Saturated Zone Model

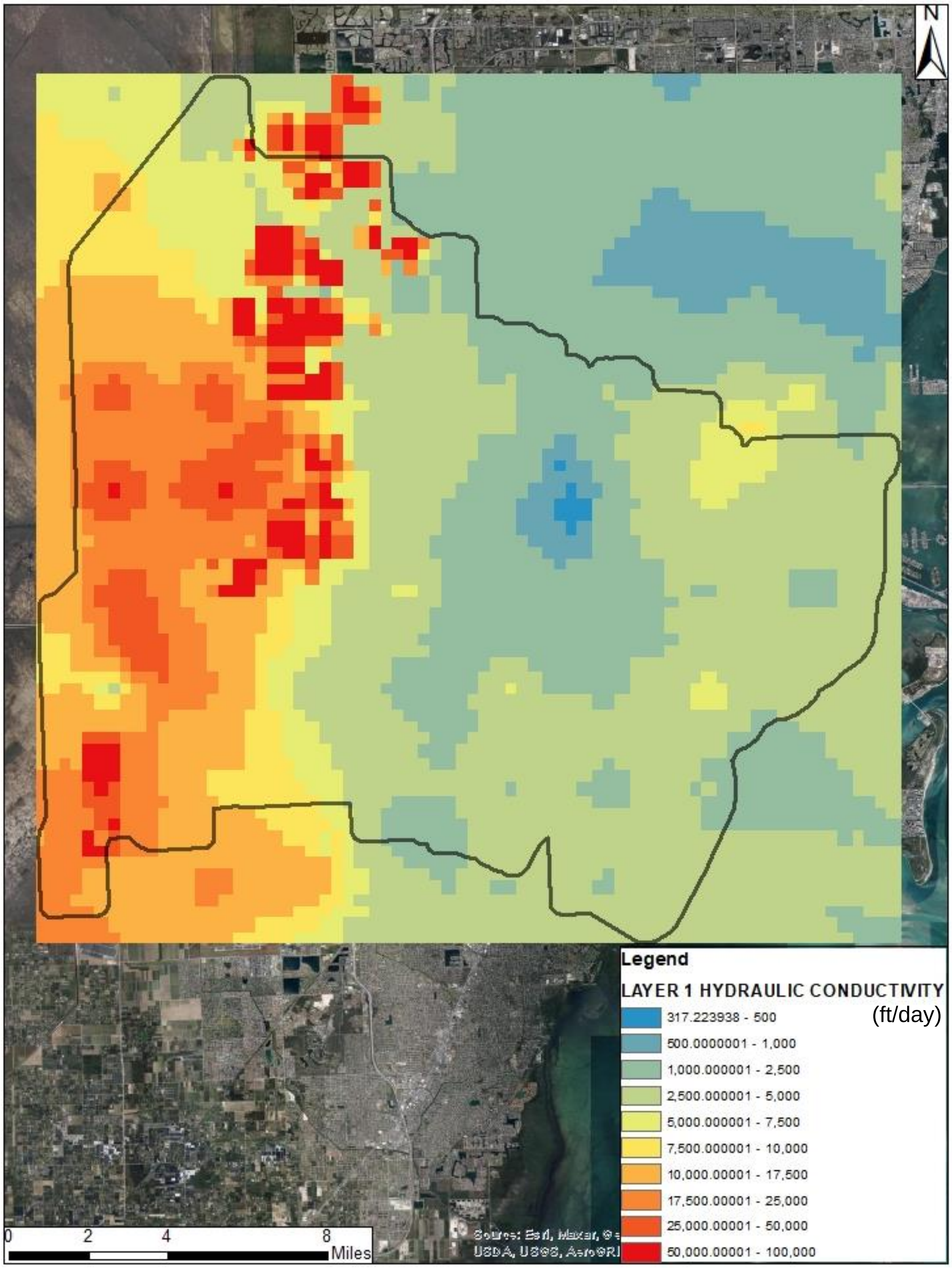


Figure 3-26 – Horizontal Conductivity of Layer 2 for the Saturated Zone Model

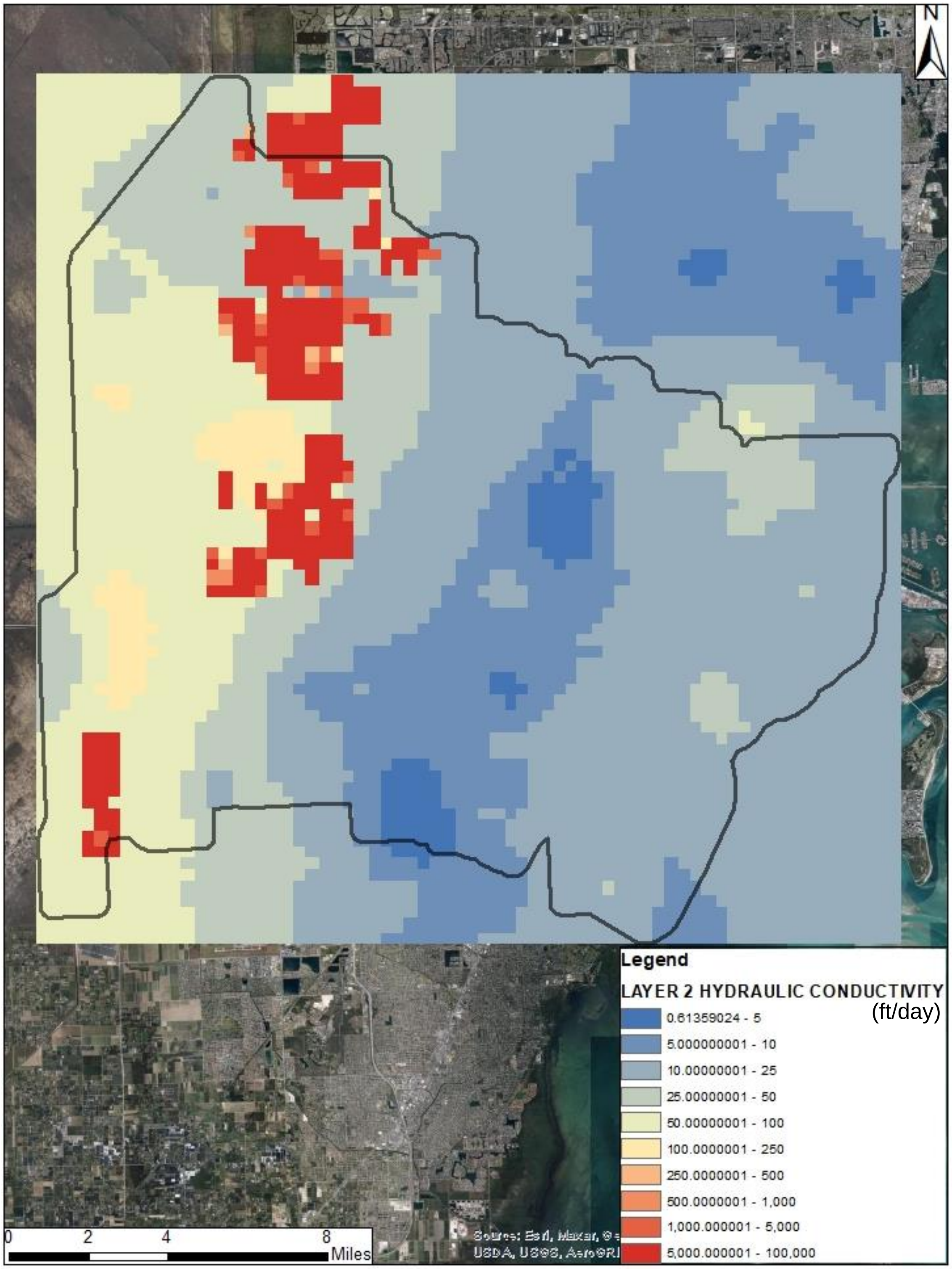


Figure 3-27 – Horizontal Conductivity of Layer 3 for the Saturated Zone Model

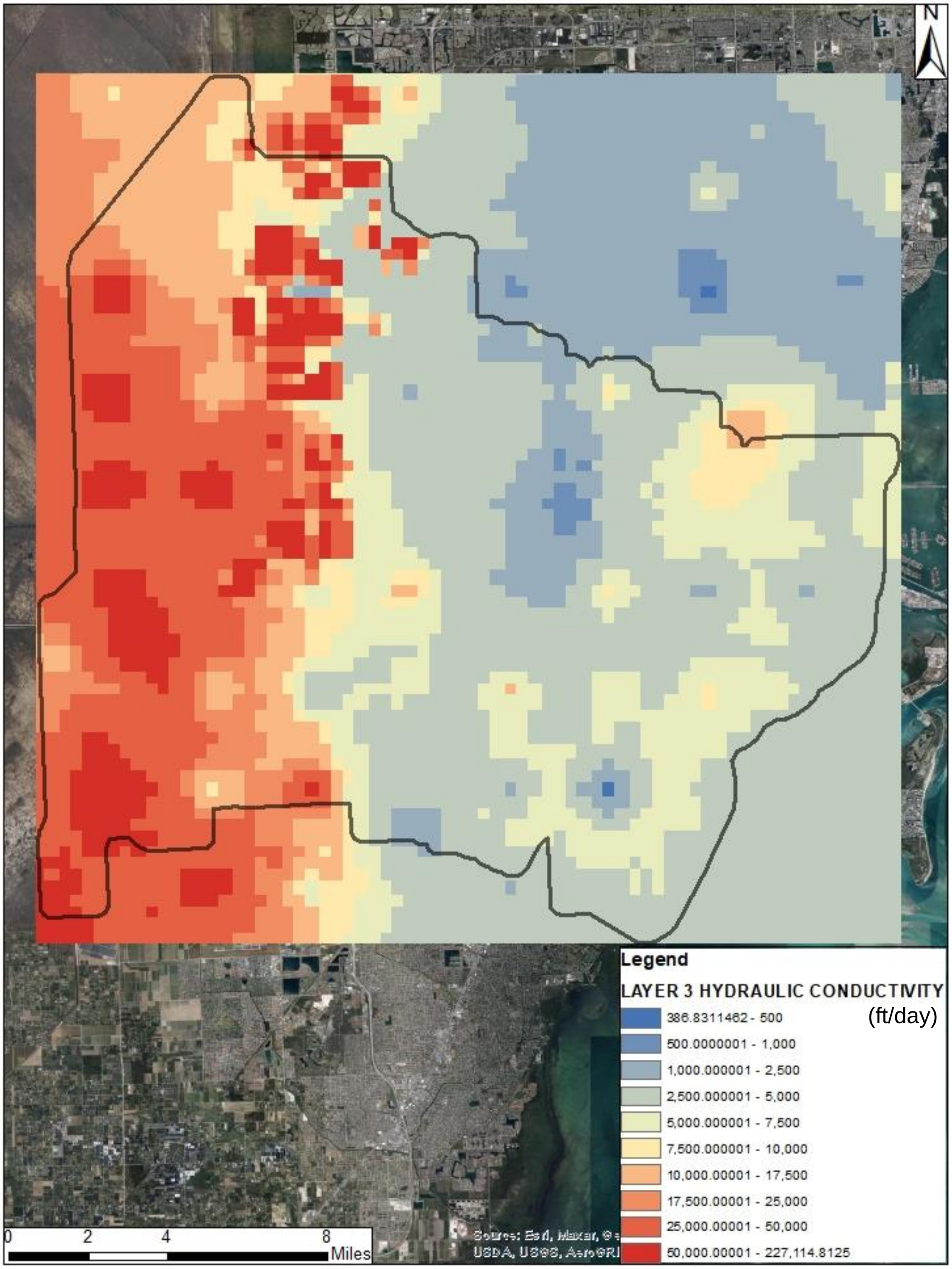
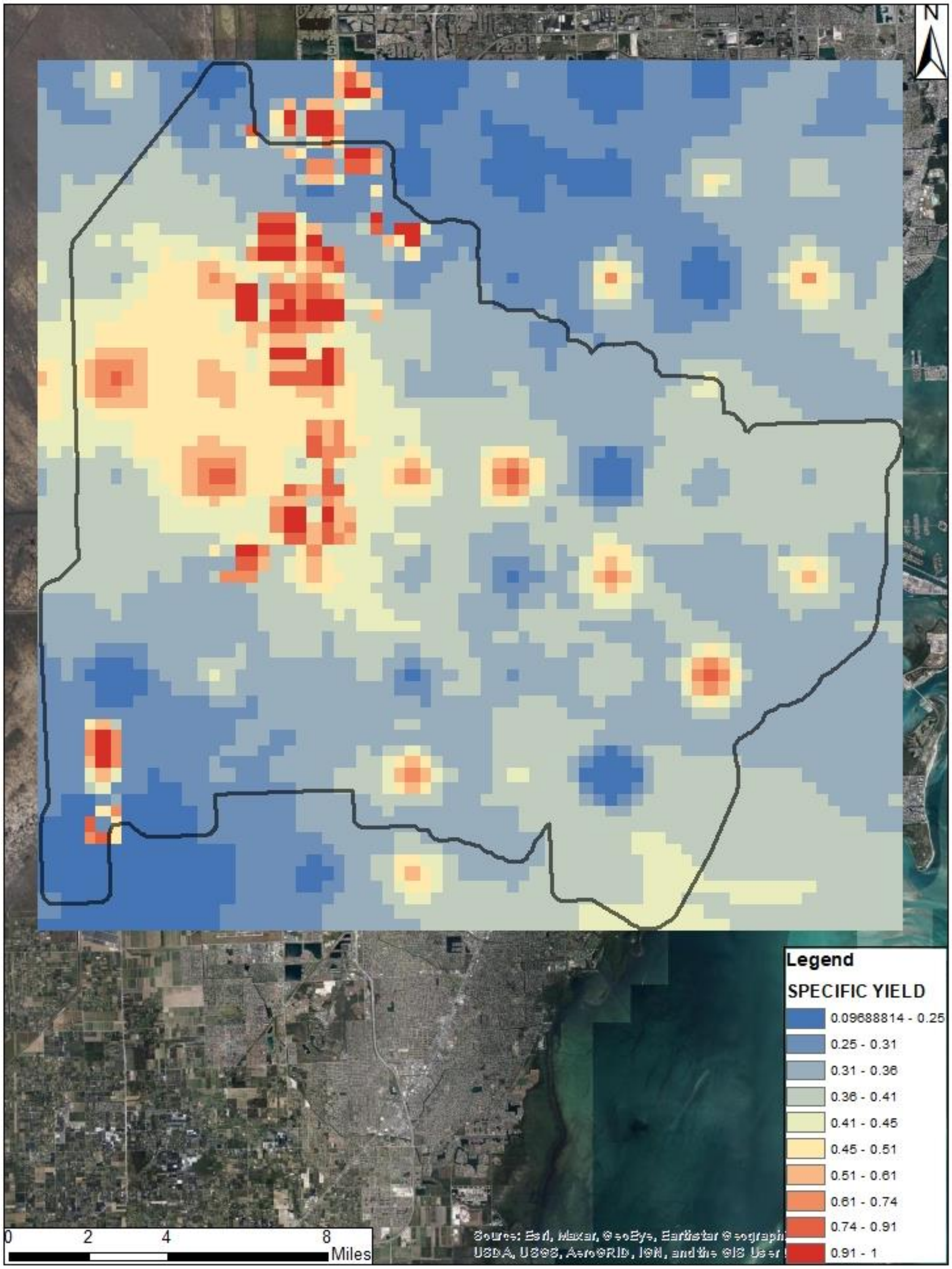


Figure 3-28 – Specific Storage for the Saturated Zone Model



3.3. HYDRAULIC GEOMETRY DATA

In addition to a representation of the land surface and subsurface, the hydraulics of the subject watersheds require data defining the location and geometry of conveyance features including canals, culverts, and water control structures. The subsections below describe the data source, characteristics, and assumptions for the hydraulic geometry data, as well as how the data is configured for MIKE 1D.

3.3.1. CANAL GEOMETRY

The primary canal network for the project area consists of the C2, C3W, C4, C5 and C6 Canals. These canals have been simulated extensively as part of various projects from the SFWMD and Miami Dade County in the past (SFWMD, 2011; SFWMD, 2015; Miami Dade County, 2005; Miami Dade County, 2006; Miami Dade County, 2007; Miami Dade County 2018). The secondary canal network for the project area consists of a series of county and municipal canals that interconnect local drainage networks with the primary conveyance system. There are 83.4 miles of primary and secondary canals within the model domain that are simulated within the MIKE 1D framework.

The proposed model utilized the newest available cross-sections from each model to define the canal network for this simulation. **Figure 3-29** illustrates the canal configuration for the FPLOS simulation and describes the data source used to define canal geometry characteristics such as cross-sections and roughness. The data source for the primary canal network and a portion of the secondary network was the HEC-RAS model developed to evaluate the interim operating rules for the C4 Canal (SFWMD, 2011). An additional data source for the primary canal network included the bathymetric surveys performed as part of the Canal Conveyance Capacity Program (SFWMD, 2020). The data source for the secondary canal network was the XPSWMM models developed by Miami Dade County as part of their watershed management program. An additional data source for the secondary canal network was the digitized point data from Miami Dade County that translated paper as-builts to a GIS point shapefile (Miami Dade County, 2021) as well as cross-sections from the South Miami Dade (SMD) model for the FPLOS program. Although not all the cross-sections had been imported into the model, the review of data sources demonstrated that there was a sufficient density of cross-sections for each canal reach being simulated.

The canals and cross-sections from the HEC-RAS model were imported into the MIKE 1D format as illustrated in **Figure 3-30**. For the remaining canal segments, the data source for the XPSWMM models was confirmed based on a review of node-link diagrams and tabular data from the reports and documentation archived with Miami Dade County.

Figure 3-29 – Data Sources for Canal Network

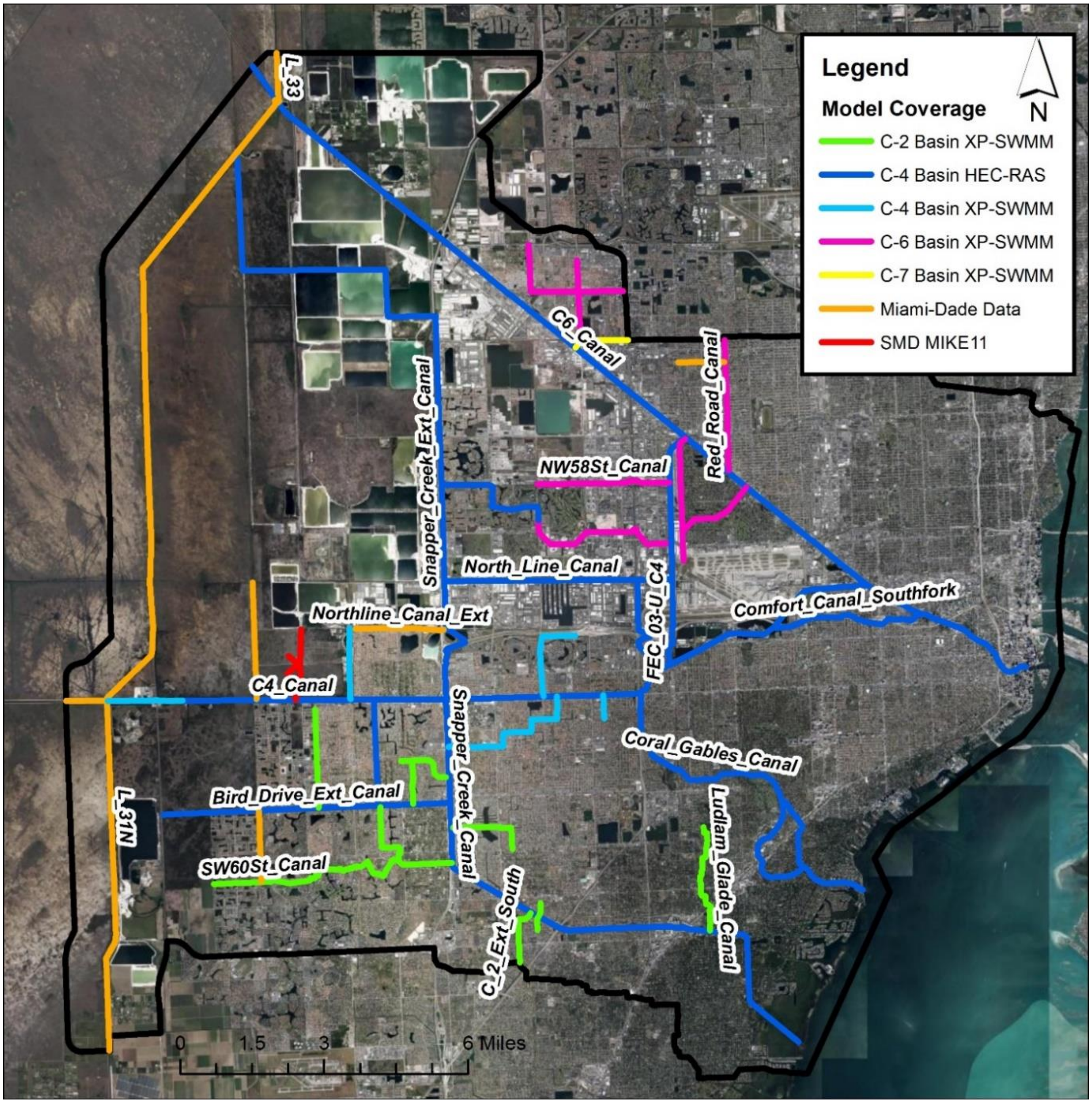
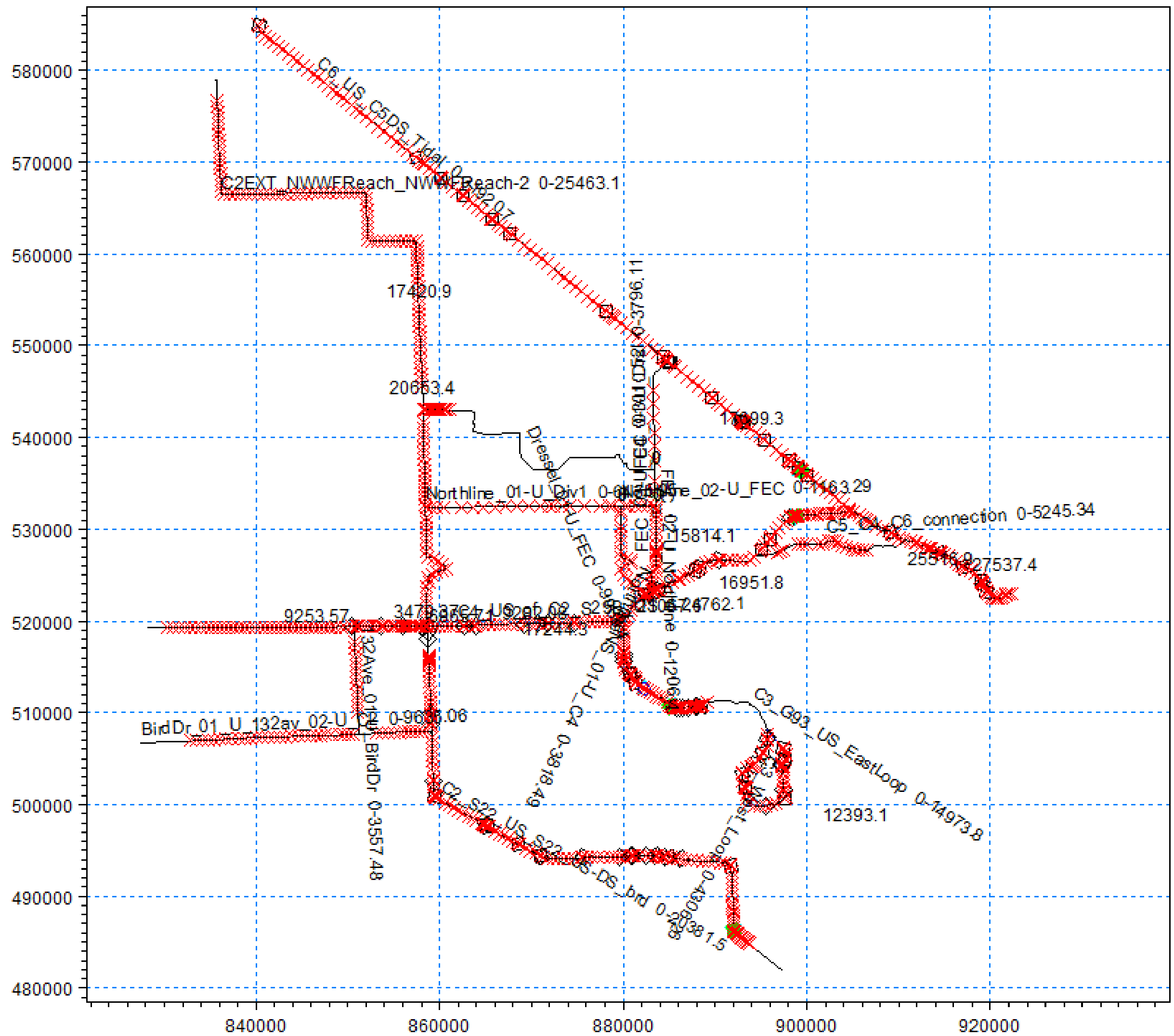


Figure 3-30 – Primary Canal Network Configuration within MIKE Platform



3.3.2. CULVERT AND BRIDGES

The HEC-RAS model contained thirty (30) structures within the model network that represent bridges or culverts. This was significantly less than the number of culverts and bridges that exist within the conveyance network. Of the nine (9) bridges included in the HEC-RAS model on the C6 Canal, there were none east of the confluence with the FEC Canal (west of Miami International Airport). This lack of data was a concern, as there are several bridges along the C6 Canal and Miami River east of the FEC Canal that have a relatively low bridge deck and low chord elevation, such as the Curtiss Parkway Bridge or the CSX Bridge.

The Miami Dade County models in XPSWMM provided a resource for the geometric data needed to represent the missing structures. As an example of how many more structures are included in the XPSWMM models, a comparison of the model documentation shows there are 26 bridges simulated on the C4 Canal west of S25B and there are 119 culverts located on the canals simulated in the C6 watershed. As part of the model development, each of the culverts and bridges represented within the XPSWMM models were exported and incorporated within the MIKE 1D framework.

3.3.3. PRIMARY NETWORK WATER CONTROL STRUCTURES

The District owns, operates, and maintains 24 water control structures either within or adjacent to the model domain. These structures have a variety of configurations and serve various purposes, including underflow spillways and forward pump stations at the eastern tidal interface and gated culverts that manage transfers of water from WCA 3A and Everglades National Park (ENP) east into the urban service area of Miami Dade County. To simulate each of these structures a series of characteristics are required, including the geometry of the structure and the operational considerations for opening and closing the structure. The District's Hydrology and Hydraulics Bureau authored an Atlas documenting the configuration of each of the primary structures in North and Central Miami Dade County (SFWMD, 2016). The Atlas was utilized to develop the simulated representation of each structure including the operations by utilizing the "Logical Operands" parameterization scheme within the MIKE framework.

Table 3-5 and **Table 3-6** provides a list of the primary structures within the model domain, including culverts and spillways. Culvert details were pulled from the Atlas of Flow Equations and sluice formula parameters were been acquired from the Atlas and Rakib and Zeng, 2017 for specific structure parameters. Structures with generalized parameters use the Generalized Flow Ratings Equation parameters developed by Ansar and Chen in 2009. In addition, pumps S25BP, S26P, and S356 are included in the model.

Table 3-5 – Primary Culverts within the Model Domain

CULVERT	CANAL	CONTROL	# OF CULVS.	INVERT ELEV. (FT-NGVD)	INVERT ELEV. (FT-NAVD)	CULV. DIAM. OR HEIGHT AND WIDTH	MANNING'S n
G72	C-7	FLASHBOARD	4	-2.3	-3.87	6 ft	0.024
CS_1	L-30	GATED					
G119	C-4	GATED	2	-2.5	-4.07	6 ft	0.024
G211	L-31N	GATED	6	-2.5	-4.07	6 ft	0.024
S120	C-100A	GATED	1	-3	-4.57	9ft x 9ft	0.012
S121	C-100C	GATED	1	-4.5	-6.07	8ft x 8ft	0.012
S25	C-5	GATED	1	-4	-5.57	8 ft	0.024
S25A	C-5	GATED	1	-1.7	-3.27	4.5 ft	0.024
S30	C-9 EXT	GATED	3	-5	-6.57	7 ft	0.012
S30 (4th culvert)	C-9 EXT	GATED	1	-2.5	-4.07	6 ft	0.024
S31	MIAMI CANAL	GATED	3	-3	-4.57	7 ft	0.024
S32	L-33 BORROW PIT	GATED	2	-2	-3.57	5 ft	0.024
S32A	L-30	GATED	1	-3	-4.57	7 ft	0.024
S336	C-4	GATED	3	-1.8	-3.37	4.5ft	0.024
S337	C-6	GATED	6	-3	-4.57	7 ft	0.024
S338	C-1W	GATED	2	-5.98	-7.55	7 ft	0.024
S380	C-4	GATED	5	-3	-4.57	6 ft	0.024

Table 3-6 – Primary Spillways within the Model Domain

SLUICE GATES	# OF GATES	HEIGHT (FT)	WIDTH (FT)	SILL ELEV. (FT-NGVD)	CSFC		USFC		CFFC		UFFC
					a	b	a	b	a	b	a
S22_S	2	15	17	-11	1.0073	0.28	1.19	0.3	0.9	0.3	0.71
S335_S	1	12.2	20	-4.2	0.9487	0.2734	1.19	0.3	0.9	0.33	0.71
S26_S	2	14.1	26	-10.1	1.05	0.3	1.19	0.3	0.9	0.33	0.71
S334_S	1	34.2	29	-6.9	1.05	0.3	1.19	0.3	0.9	0.33	0.71
S25B_S	2	11.9	22	-7.9	1.05	0.3	1.19	0.3	0.9	0.33	0.71
G93_S	2	5	10	-1.8	1.05	0.3	1.19	0.3	0.9	0.33	0.71
S27_S	2	15	27.7	-11	1.05	0.3	1.19	0.3	0.9	0.33	0.71
G421_S	1	4	20	-4	1.05	0.3	1.19	0.3	0.9	0.33	0.71
Atlas of Flow Computations, 2015				Rakib and Zeng, 2017				Ansar and Chen, 2009			

3.3.4. SECONDARY NETWORK WATER CONTROL STRUCTURES

There are several municipal structures that provide flood control to large sections of the model area at a sub-regional scale. **Figure 3-31** illustrates the locations of all secondary structure data that was compiled from various culverts, pumps, and spillways. As a specific example of the types of facilities included, between the cities of Miami, West Miami, Sweetwater, and unincorporated Miami Dade County there are 13 pump stations ranging in capacity from 15 CFS to 100 CFS with a total capacity of 679 CFS. In most cases, these stations are at a collection point of the urban drainage network that makes up the tertiary stormwater management system. **Table 3-7** describes the characteristics of each of the 13 pump stations.

3.3.5. TERTIARY STORMWATER MANAGEMENT SYSTEMS

Within the model domain there are areas of residential, commercial, and industrial development with a significant density of urban stormwater management systems. These systems typically consist of a series of curb inlets along roadway gutters that are interconnected to ponds or exfiltration trenches by underground pipes. The ponds or exfiltration trenches act as a feature that provides water quality treatment and attenuation of peak flows prior to discharging downstream through a control structure.

Although the MIKE modeling framework has tools that can represent these features individually, for a regional model such as this, it is not an efficient approach. The alternative within MIKE SHE is the use of the Drain Codes feature. Drain Codes allow the model to define an area where there is an existing system that facilitates drainage from the surface of MIKE SHE to the nearest branch in MIKE 1D (or to a specified branch). The parameterization of drain codes is based on two characteristics: depth below the surface where drainage is applied and time constant to attenuate the delivery of the drainage volume similar to the manner in which the time of concentration reflects the delay between the beginning of runoff and peak flow.

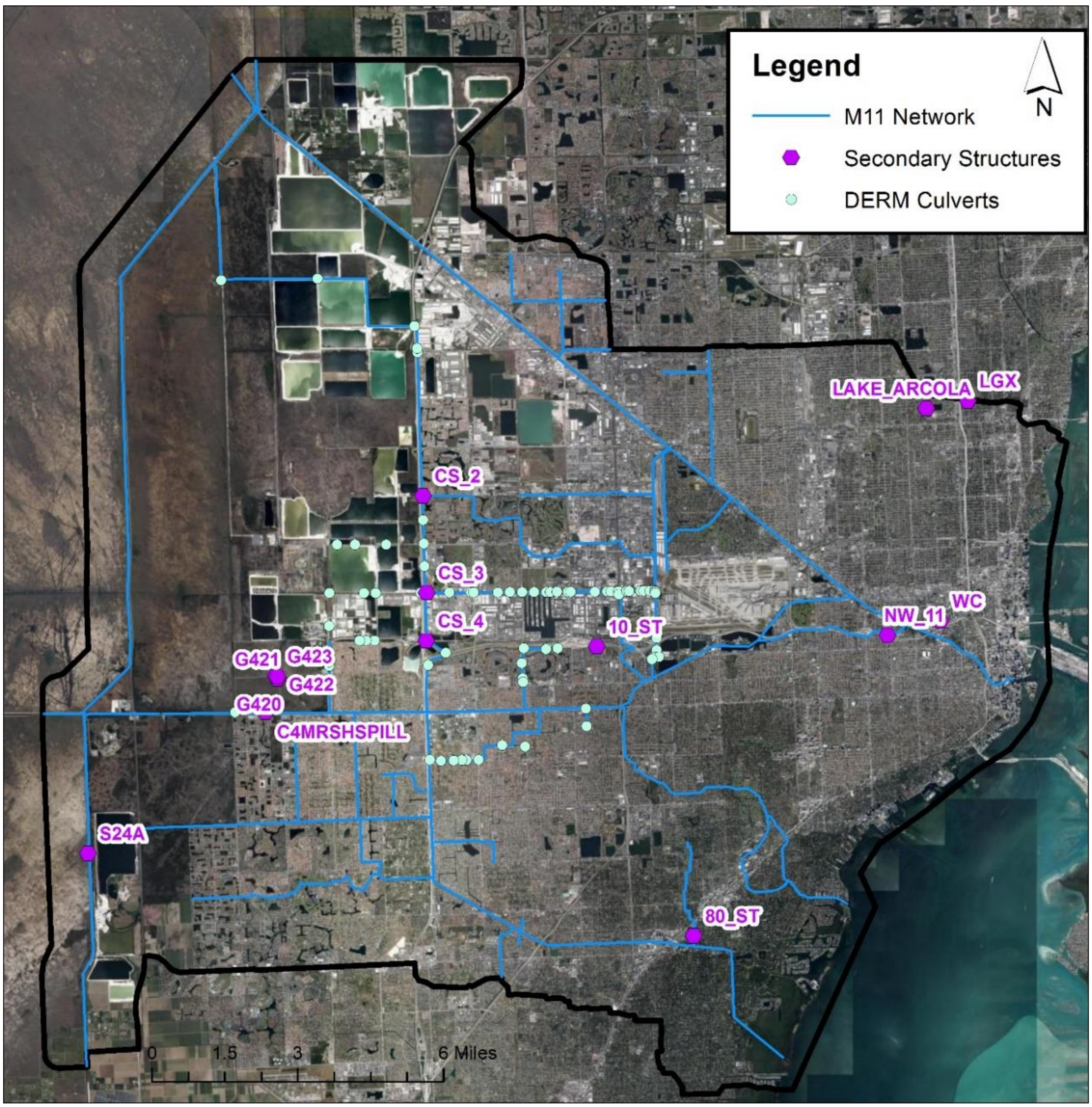
For other MIKE SHE models in the Miami Dade area, the drainage time constant has been set to a range of values from $1E^{-6}$/second in South Miami Dade to $5E^{-7}$/second in the C8 and C9 Watersheds. It is recommended that an initial value of $5E^{-7}$/second is used for areas where tertiary drainage features exist. However, if a higher density of drainage features is found, the number may be increased to represent greater drainage conveyance. In undeveloped areas or in water land use types, the time constant is set to zero, which turns off this drainage module in those cells.

Table 3-7 – Municipal Pump Locations and Operations

City	Pump Station	Pump Location	Pump Capacity [CFS]	T5W Off Elev. [FT-NAVD]	Wet Well Off Elev. [FT-NAVD]	Wet Well On Elev. [FT-NAVD]
Sweetwater	B1	SW 102nd Pl & 2nd St	20	3.43	-6.57	1.43
	B2	SW 103rd Ave between SW 5th/6th St	20	3.43	-6.57	1.43
	B15	SW 112th Ave between NW 5th Terr/St	25.4	3.43	-5.57	-1.57
	B16	SW 112th Ave between NW 3rd Terr/St	25.4	3.43	-5.57	-1.57
Belen	PS1	SW 7th St & SW 127th Ave	100	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS
	PS2	SW 6th St & SW 120th Ave	100	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS
West Miami	#1	25 45'33.77" N 80 18'06.75" W SW 65th Ave & SW 12th St	90	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS
	#2	25 45'17.74" N 80 17'50.63" W SW 63rd Ave & SW 16th Terr	100	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS
City of Miami	#1	25 45'54.85" N 80 17'55.62" W SW 63rd Ct & SW 6th St	30	3.43	-7.32	2.95ft: 25CFS 3.00ft: 100CFS
	#2	25 46'08.04" N 80 17'56.03" W SW 63rd Ct & SW 2nd St	60	3.43	-7.32	2.95ft: 25CFS 3.00ft: 100CFS
	#3	25 46'27.15" N 80 18'05.44" W NW 64th Ct at C4 canal	40	3.43	-9.57	1.43
	#4	25 46'33.55" N 80 17'51.05" W NW 62nd Ct at C4 canal	54	3.43	-9.57	1.43
	#5	25 46'42.82" N 80 16'45.69" W NW 52nd Ave & NW 7th St	15	3.43	-8.57	1.43

In locations where the development patterns precede the regulatory requirements of surface water permitting, the land use and local infrastructure were reviewed. If tertiary drainage infrastructure exists in these locations, drain codes were applied and the SHWT was used as the reference depth. The extent of the area was considered for calculating the time attenuation constant and the limits were based on either development patterns in the land use or sub-basins as defined in the XPSWMM models from Miami Dade County.

Figure 3-31 – Locations of Secondary Structures



3.4. TEMPORAL DATA

The representation of hydrology, hydrogeology, and hydraulics within the MIKE SHE model were utilized to simulate various scenarios for calibration, validation, and design conditions. There are various types of temporal data necessary to facilitate those scenarios with respect to meteorologic forcings, hydraulic boundary conditions and calibration reference data. The sub-sections below describe the data source, characteristics, and assumptions for each of these temporal datasets, as well as how the data is configured for the MIKE SHE modeling platform.

3.4.1. NEXRAD RAINFALL

The nature of the climate in Florida generates rainfall distributions that are highly variable spatially from individual storm events. To account for the significant degree of rainfall variability NEXRAD data was utilized to define precipitation for the calibration and validation simulations. The District maintains a record of rainfall depths in inches for each NEXRAD grid-cell at a time-step interval of 15 minutes. The data for the following time periods was extracted from the District’s dataset for model development:

- May 10, 2020 through June 1, 2020
- August 20, 2017 through September 22, 2017

These two events represent the calibration and validation storms. The extent of the time periods selected includes periods before and after the flooding events in order to capture the antecedent conditions and recession of floodwaters. **Figure 3-32** illustrates the NEXRAD grid cells utilized in the MIKE SHE model development. The data provided by SFWMD for all NEXRAD cells in the region was reconfigured into a format that can be read by MIKE SHE to apply the 15-minute rainfall at each grid cell in the model domain throughout the rainfall period.

3.4.2. EVAPOTRANSPIRATION

3.4.2.1. POTENTIAL EVAPOTRANSPIRATION

Evapotranspiration data was input into the model as potential evapotranspiration (PET) downloaded from the S331W Station (DBKey TA787), located approximately 3 miles south of the proposed model domain as shown in **Figure 3-33**. PET data was input into the model as time varying daily values with uniform spatial distribution. **Figure 3-34** and **Figure 3-35** below provide the daily PET data at Station S331W for September 2017 and May 2020, respectively. Of note, the recorded PET does not accurately represent actual ET data in the field during the wet season.

Figure 3-32 – NEXRAD Grid Cells Utilized for Existing Conditions Simulations

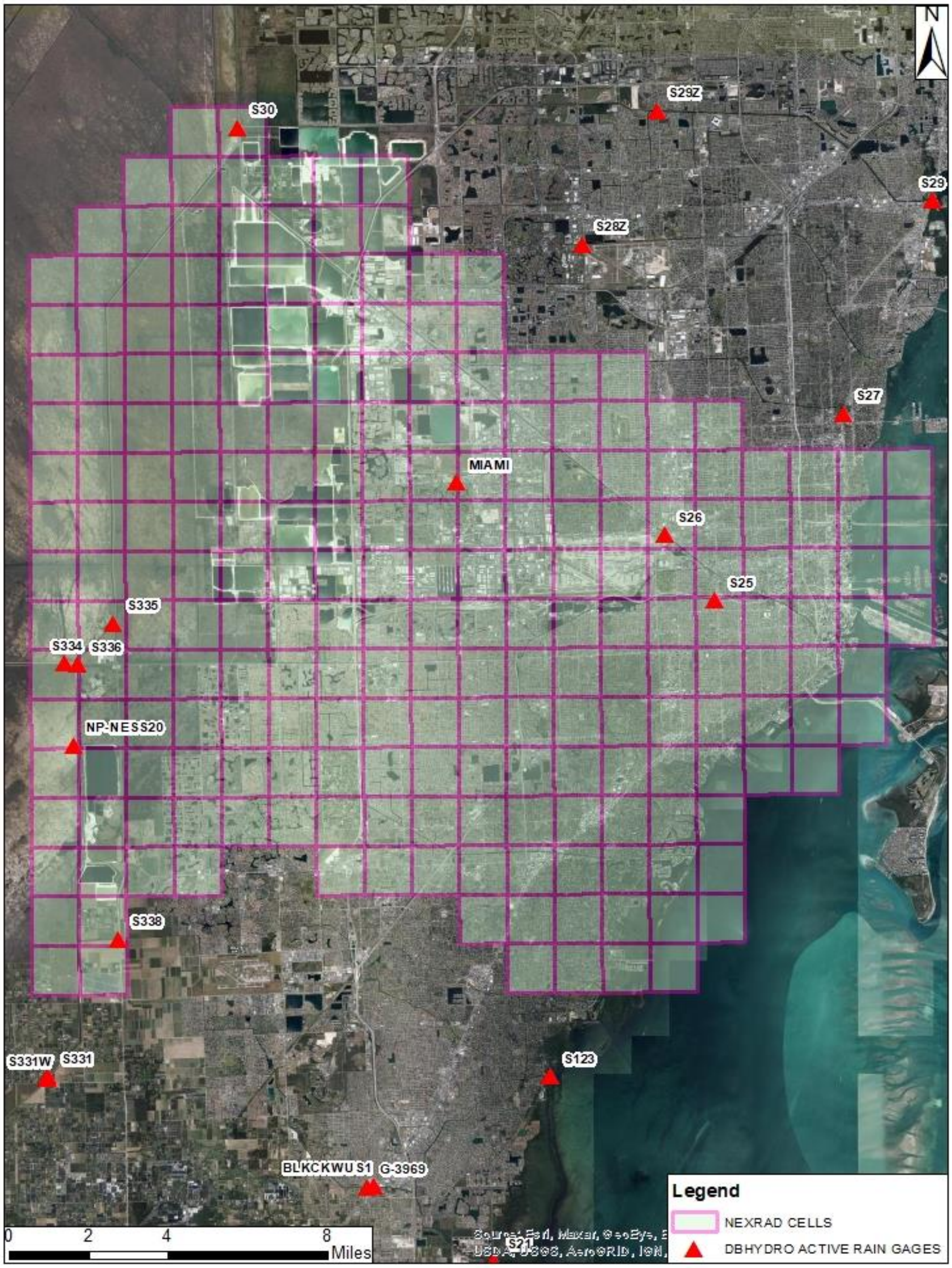


Figure 3-33 – Location of S331W Meteorological Station for PET

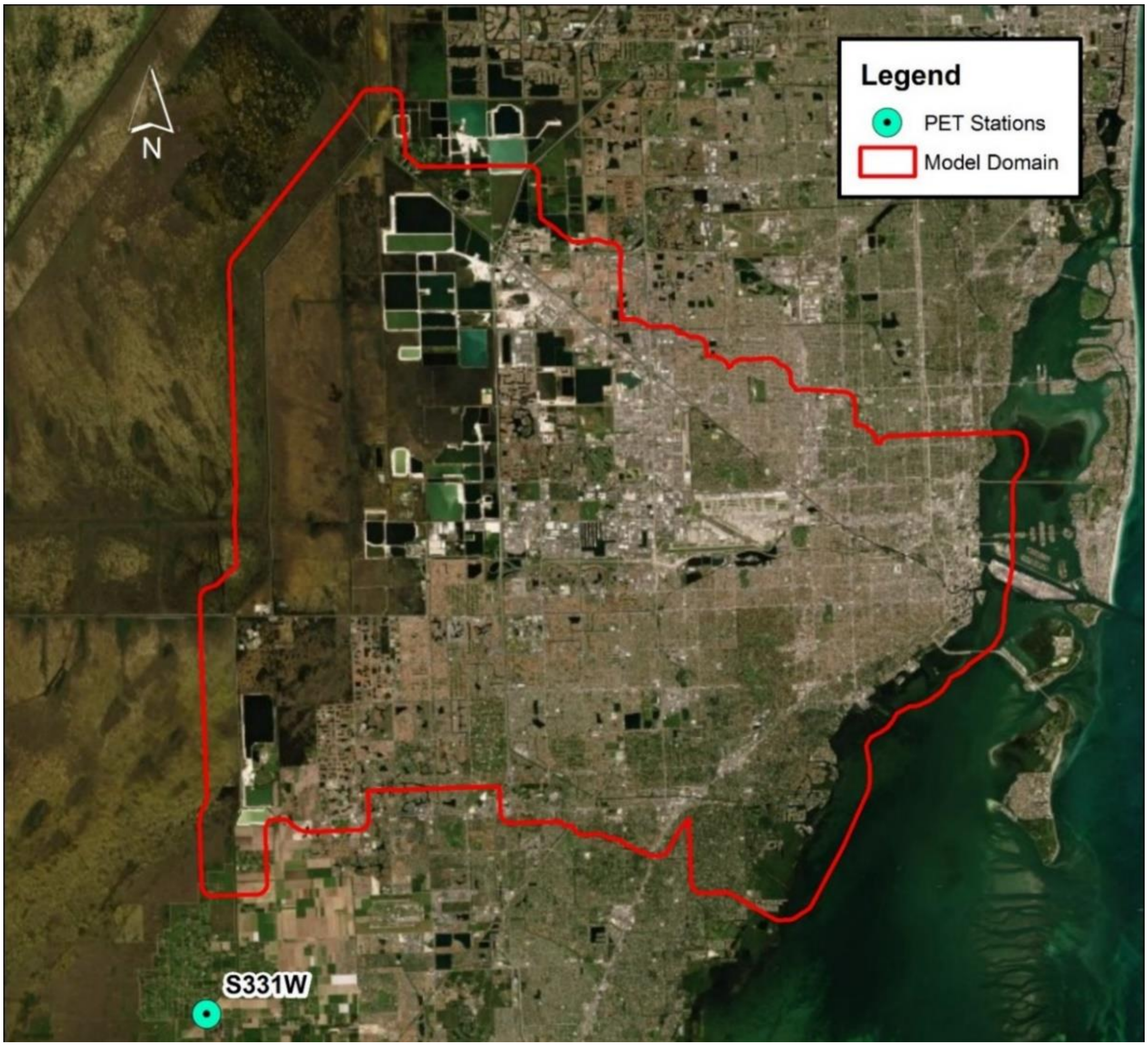


Figure 3-34 – PET Data for September 2017 at S331W

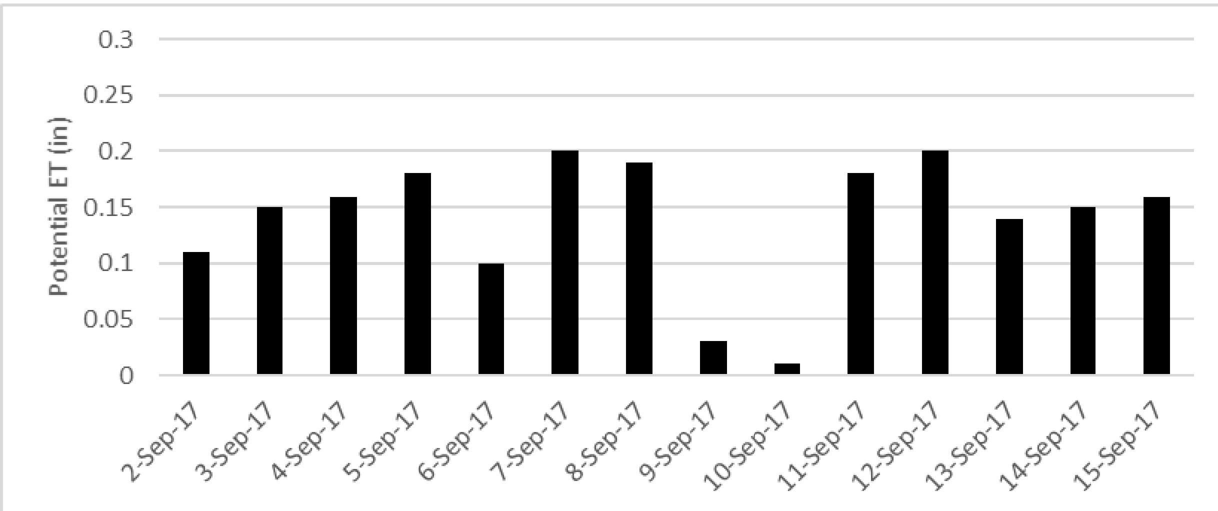
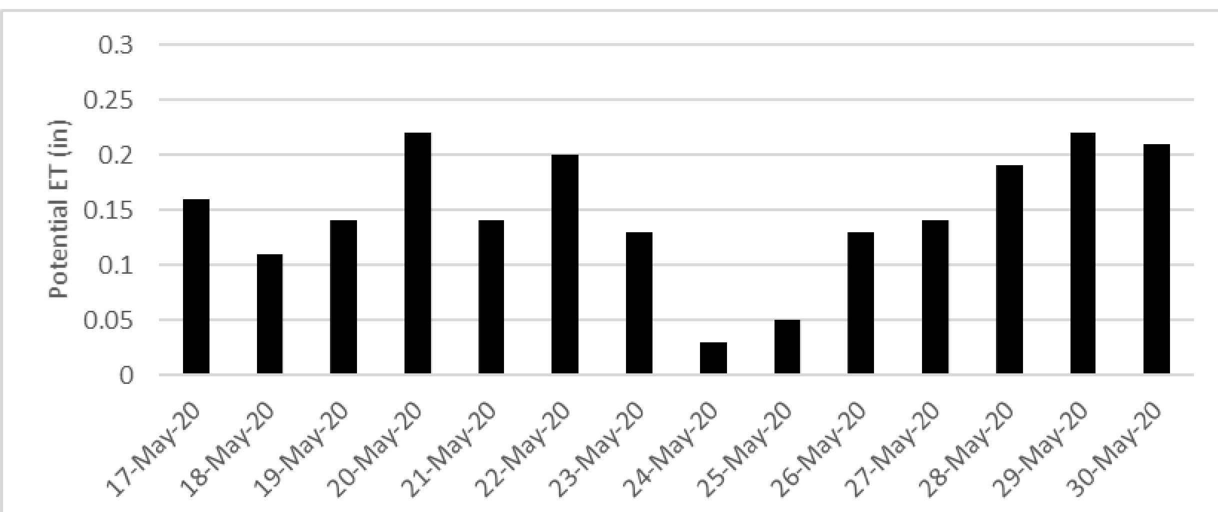


Figure 3-35 – PET Data for May 2020 at S331W



3.4.2.2. VEGETATION PARAMETERS

The model uses land use types to specify vegetation parameters such as Leaf Area Index (LAI), Root Depth (RD), and Crop Coefficients (Kc) to calculate Actual ET based on the PET, the density of the vegetation, the soil moisture content in the root zone, and the root density. The crop coefficient can be specified to adjust the potential ET to the appropriate crop, since the reference for the PET calculation is for a short grass plane with ample water supply.

Since the calibration and validation periods selected represent two different times in crop cycles (i.e., May and September), time varying vegetation parameters were based on the calibration results from the Kissimmee Basin Modeling and Operations Study (Earth Tech, 2007) and generalized using the selected FLUCCS for May and September.

Table 3-8 provides the LAI, RD, and Kc values for each FLUCCS Code for both May and September. For both LAI and RD, the values stay the same for most land use types, except for crops (FLUCCS 2100). Crop Coefficients also show only minor variation between the two-time frames, with the exception of Low Density Residential.

Table 3-8 – Vegetation Parameters for May and September

LUCCS Code	Land Use	LAI [-]		RD [in]		Kc [-]	
		May	Sept	May	Sept	May	Sept
1100	Residential, Low Density	1.6	1.6	7.87	7.87	0.58	0.71
1200	Residential, Medium Density	1.45	1.45	7.87	7.87	0.49	0.59
1300	Residential, High Density	1.25	1.25	7.87	7.87	0.41	0.47
1400	Commercial and Services	1.05	1.05	7.87	7.87	0.32	0.35
1500	Industrial	1.05	1.05	7.87	7.87	0.32	0.35
1600	Extractive	0.3	0.3	3.94	3.94	0.85	0.85
1700	Institutional	1.25	1.25	7.87	7.87	0.41	0.47
1800	Recreational	1.6	1.6	7.87	7.87	0.58	0.71
1900	Open Land	1.6	1.6	7.87	7.87	0.58	0.71
2100	Cropland and Pastureland	4.5	3	29.53	5.98	0.96	0.97
2200	Tree Crops	4.5	4.5	49.21	49.21	0.91	1.03
2300	Feeding Operations	4.5	4.5	29.53	29.53	1.00	0.92
2400	Nurseries and Vineyards	1.6	1.6	7.87	7.87	0.58	0.71
2500	Specialty Farms	1.6	1.6	7.87	7.87	0.58	0.71
2600	Other Open Lands - Rural	1.6	1.6	7.87	7.87	0.58	0.71
3100	Herbacous (Dry Prairie)	3	3	47.99	47.99	0.73	0.75
3200	Upland Shrub and Bushland	3	3	47.99	47.99	0.73	0.75
3300	Mixed Rangeland	3	3	47.99	47.99	0.73	0.75
4100	Upland Coniferous Forest	3	3	47.99	47.99	0.73	0.75
4200	Upland Hardwood Forest	4	4	24.02	24.02	0.73	0.75
4300	Upland Mixed Forest	4	4	24.02	24.02	0.73	0.75
4400	Tree Plantations	3	3	47.99	47.99	0.73	0.75
5000	Water	0	0	0	0	0.98	0.98
6100	Wetland Hardwood Forest	4	4	24.02	24.02	0.73	0.75
6400	Vegetated Non-Forested Wetlands	4	4	5.98	5.98	0.81	0.81
6500	Non-Vegetated Wetlands	0	0	0.00	0.00	0.98	0.98
7200	Sand other than Beaches	0.3	0.3	3.94	3.94	0.85	0.85
7400	Disturbed Land	0.3	0.3	3.94	3.94	0.85	0.85
8100	Transportation	1.25	1.25	7.87	7.87	0.41	0.47
8200	Communications	1.6	1.6	7.87	7.87	0.58	0.71
8300	Utilities	1.6	1.6	7.87	7.87	0.58	0.71

3.4.3. HYDRAULIC INITIAL CONDITIONS AND BOUNDARY CONDITIONS

The District's DBHYDRO database provides a detailed archive of meteorologic, surface water and groundwater data that can be searched and queried to extract model input data. This DBHYDRO data was extracted to define the measured stage data that was used to define the tidal and upstream boundary conditions for the hydraulic model during calibration and validation.

3.4.3.1. INITIAL CONDITIONS

Although the flooding events in 2017 and 2020 that were selected for the calibration and validation simulations are only 6 and 7 days respectively, the proposed approach for Model Development was to prepare up to one-month simulations that were be used to define the antecedent conditions within the model domain and generate a "hot start" data set for the key periods of interest.

3.4.3.2. TIDAL BOUNDARY CONDITIONS

For the calibration and validation simulations, tidal stage data from the MRMS4 monitoring location was extracted for use as a downstream boundary for Biscayne Bay at the mouth of the C2 and C6 canals. **Figure 3-36** and **Figure 3-37** illustrate the tidal stage data that were utilized in the MIKE-SHE model for the downstream boundary.

Figure 3-36 – MRMS4 Stage Data for May 18-30, 2020

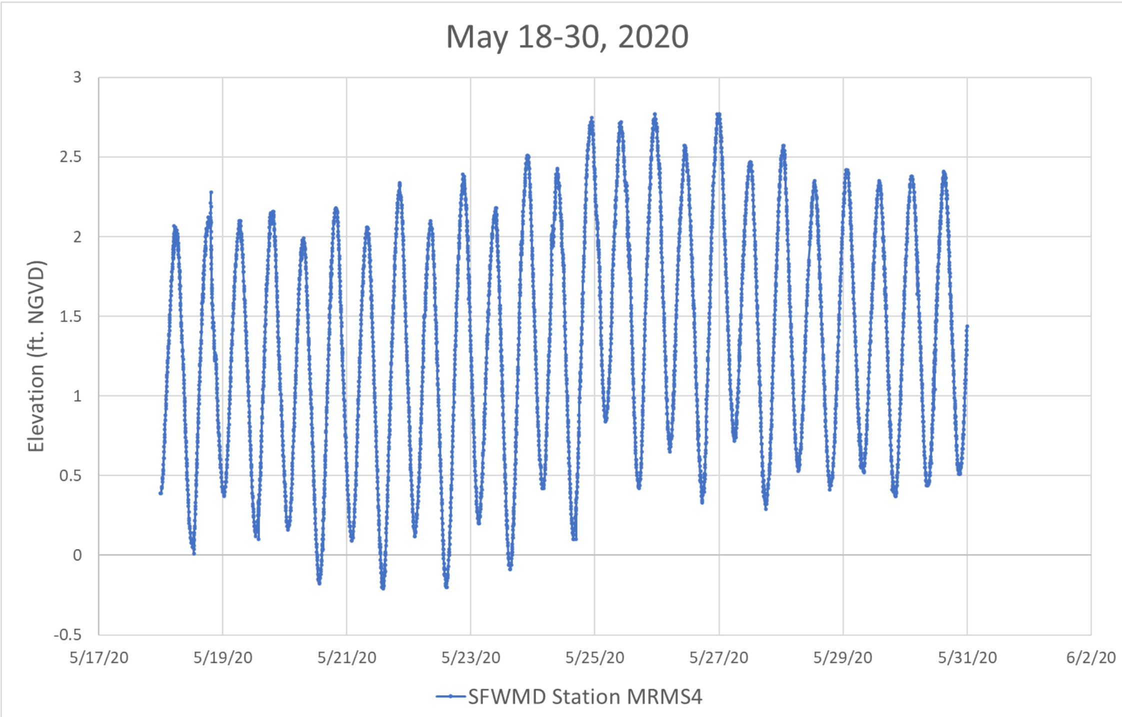
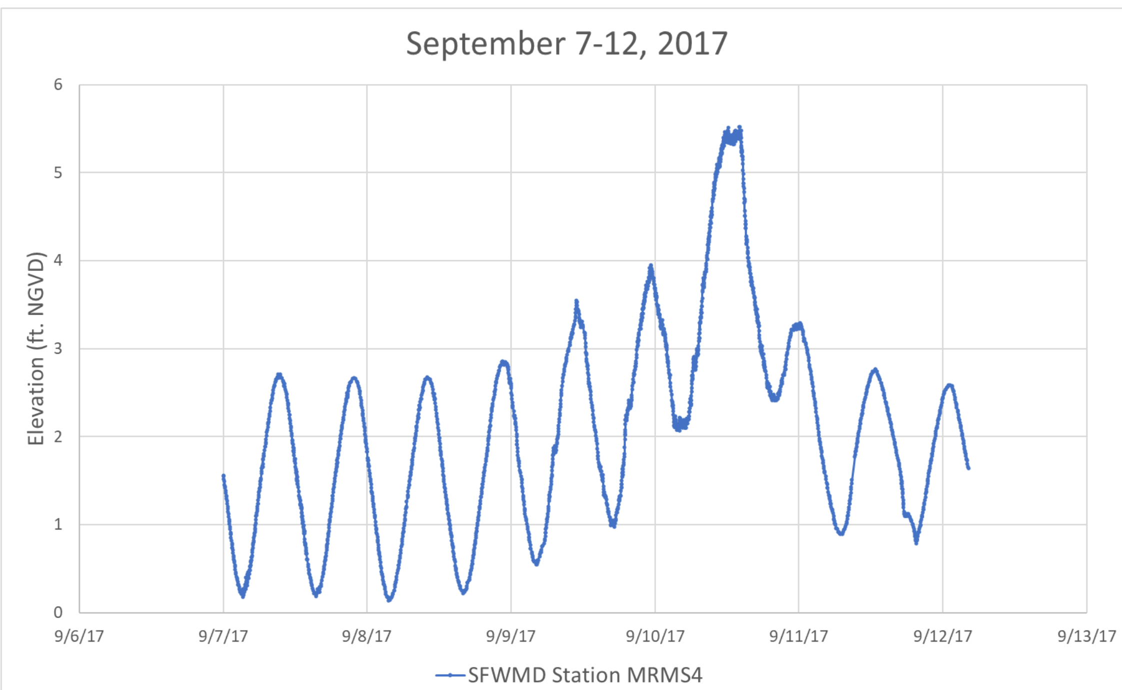


Figure 3-37 – MRMS4 Stage Data for September 7-12, 2017



3.4.3.3. UPSTREAM BOUNDARY CONDITIONS

DBHYDRO data was extracted for each of the structures along the upstream limits of the model domain. However, these stages can only be applied effectively as upstream boundary conditions if the downstream structures are open during the simulation period providing hydraulic connectivity. The gate opening data was acquired from DBHYDRO for each of these western structures to verify that the boundary conditions were properly applied.

Based on this review, **Table 3-9** describes that during the selected storm events on the C4 Canal the G119 structure is closed while S380 is either open or partially opened at various points, while on the C6 Canal the S31, S32 and S32A structures are closed for most of each event. Based on this review and the configuration of the canals, a fixed head boundary was used at the following locations upstream of the hydraulic network: S31 headwater, S32 headwater, S334 tailwater, S338 headwater, and G72 headwater. **Table 3-10** describes the external and internal boundary conditions that were applied to each of the primary canal reaches. **Figure 3-38** and **Figure 3-39** illustrate the measured stage at each of the structures selected as hydraulic boundaries. **Figure 3-40** illustrates the locations of the boundary condition monitoring stations.

Table 3-9 – Gate Operations at Upstream Water Control Structures

Structure	September 2017						May 2020						
	7	8	9	10	11	12	21	22	23	24	25	26	27
S-336	C	C	C	C	C	C	C	C	C	C	C	C	C
S-335	P	P	C	C	C	C	C	C	C	C	C	C	C
S-334	P	C	C	C	C	C	P	P	P	P	C	C	C
G-119	C	C	C	C	C	C	C	C	C	C	C	C	C
S-380	O	C	C	C	P	C	C	C	C	P	P	C	C
S-25	Tidally Dependent Opening						Tidally Dependent Opening						
S-25B	Tidally Dependent Opening						Tidally Dependent Opening						
S-26	Tidally Dependent Opening						Tidally Dependent Opening						
S-30	P	C	C	C	C	C	C	C	C	C	C	C	C
S-31	P	C	C	C	C	C	C	C	C	C	C	C	C
S-32	C	C	C	C	C	C	C	C	C	C	C	C	C
S-32A	C	C	C	C	C	C	C	C	C	C	C	C	C
S-337	C	C	C	C	C	C	P	P	P	P	P	P	C
G-211	P	P	P	P	P	P	P	P	P	P	C	C	C
S-22	Tidally Dependent Opening						Tidally Dependent Opening						
S-27	Tidally Dependent Opening						Tidally Dependent Opening						
O – Fully Open			P – Partially Open					C – Closed					

Table 3-10 – Boundary Conditions for Primary Canal Reaches

Canal Reach	Upstream Boundary	Downstream Boundary
C2 Canal	C4 Canal (Internal Model)	Virginia Key (Tide)
C3 Canal	C4 Canal (Internal Model)	Virginia Key (Tide)
C4 Canal	S334 Tailwater	C6 Canal / Miami River (Internal Model)
C5 Canal	C4 Canal (Internal Model)	C6 Canal / Miami River (Internal Model)
C6 Canal	S31 Headwater	MRMS4
L30 Canal	S31 Headwater	C4 Canal (Internal Model)
L31N Canal	C4 Canal (Internal Model)	S338 Headwater

3.4.4. HYDRAULIC CALIBRATION REFERENCE DATA AND FORCING

DBHYDRO data was also extracted to define the measured stage and flow data that was used as a reference to evaluate the calibrated model's effectiveness of representing the known conditions. In particular at stage monitoring locations such as C4.CORAL and at locations where the flow is calculated such as at S-26, the DBHYDRO measured data was compared with the simulated data to determine how closely the model was able to re-create the measured stages and flows. The comparison utilizes the following statistical tools:

- Mean Error (ME)
- Root Mean Square Error (RMSE)
- Correlation (R)
- Nash-Sutcliffe Efficiency (E)

In addition to stage and flow data for calibration purposes, gate opening, and pump operation data was extracted from DBHYDRO to provide forcings to the MIKE 1D River simulation. **Table 3-11** describes the data available at each calibration reference point and operational structure within the model domain.

Table 3-11 – DBHYDRO Data for Calibration Reference Points and Operable Structures

Structure	Headwater	Tailwater	Stage	Flow	Gate Opening	Pump
C2GSW1			X			
C2SW1			X			
C2SW2			X			
C4.CORAL			X	X		
G72	X	X		X		
G93	X	X		X	X	
MRMS1			X			
MRMS4			X			
S25	X	X		X	X	
S25A				X		
S25B	X	X		X	X	X
S25B_P	X	X				
S26	X	X		X	X	
S26_P						X
S30	X	X		X	X	
S31	X	X		X	X	
S32	X	X		X	X	
S32A				X	X	
S121	X	X		X	X	
S337	X	X		X	X	
S380					X	
T5W			X			

Figure 3-38 – Primary Canal Boundary Condition Stage Data for May 18-30, 2020

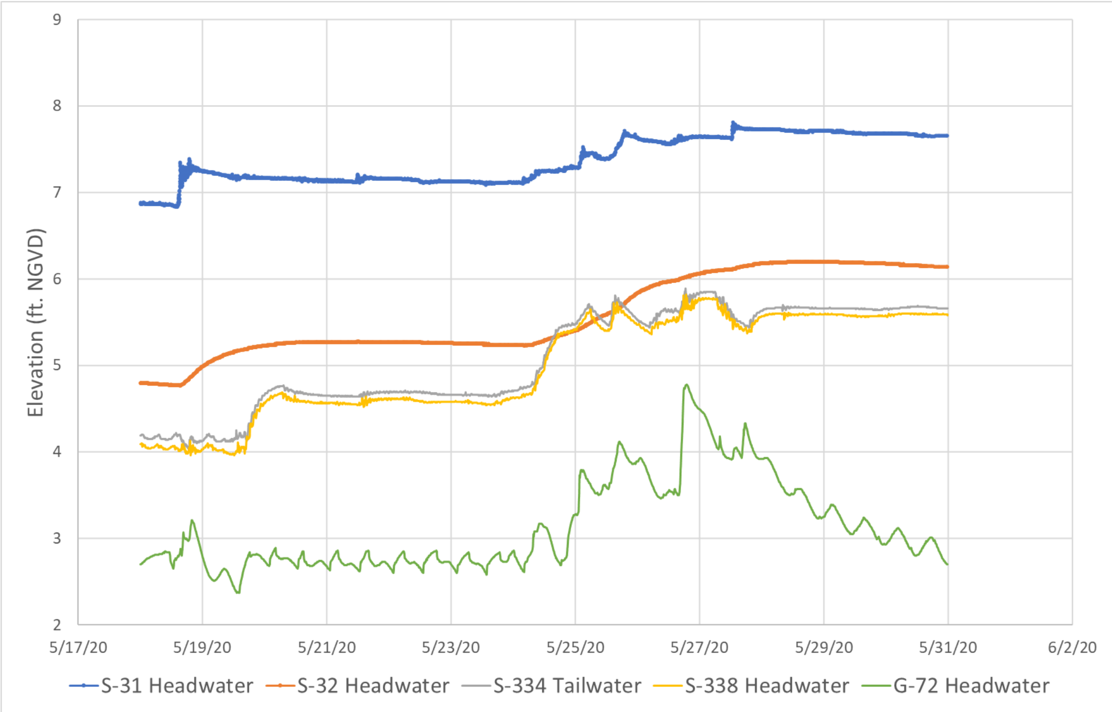


Figure 3-39 – Primary Canal Boundary Condition Stage Data for September 7-12, 2017

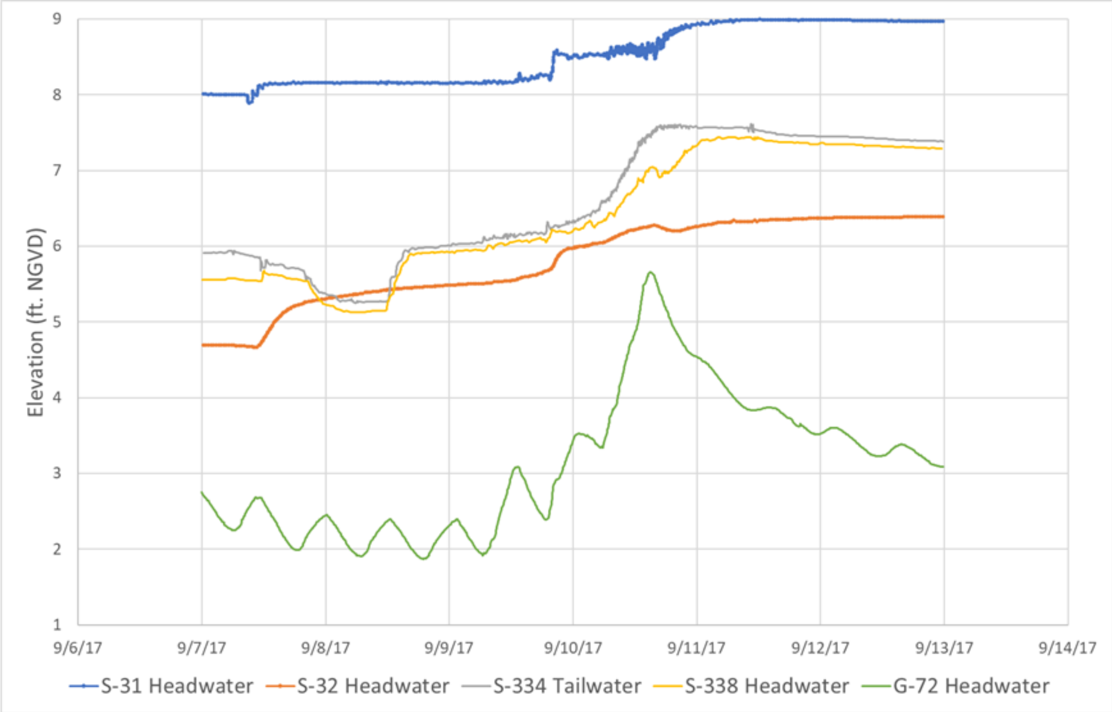
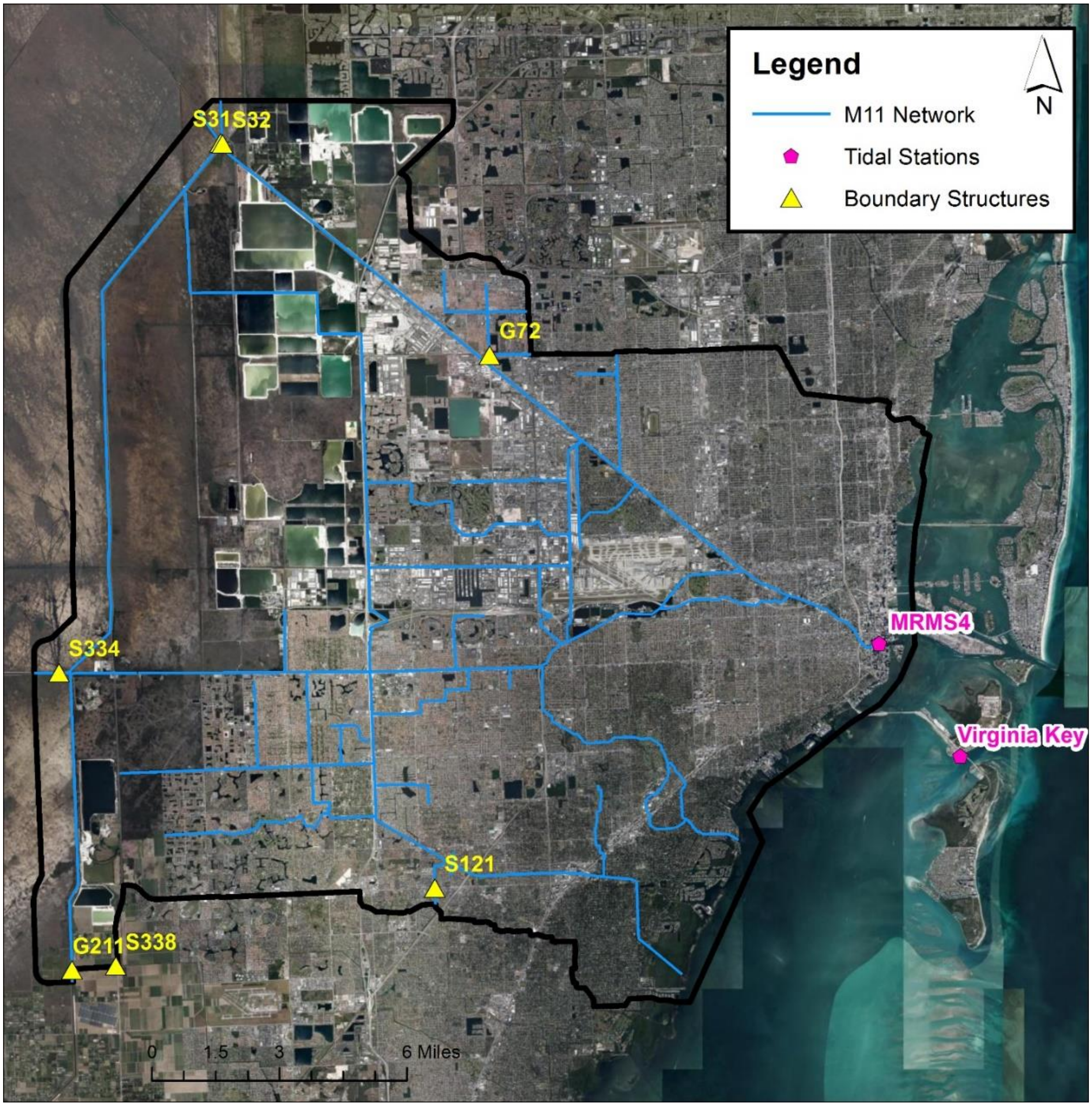


Figure 3-40 – Boundary Condition Locations



3.4.5. GROUNDWATER MONITORING DATA AND BOUNDARY CONDITIONS

Similar to stage and flow monitoring stations for the hydraulic model, groundwater data was needed from monitoring wells at locations throughout the project area to provide boundary conditions and calibration reference points for the saturated zone portion of the MIKE SHE model. DBHYDRO contains timeseries data for 94 monitoring wells either within the model domain or in the adjacent vicinity. **Figure 3-41** illustrates the locations of all monitoring wells in the region and highlights the locations where data was used for the saturated zone model.

3.4.5.1. MONITORING WELL DATA

Monitoring well locations that are within the model domain were used as calibration reference points, whereas well data from outside the perimeter of the model domain were used to define the fixed head boundaries needed for the saturated zone model. The temporal frequency varies at each location with some data available on a breakpoint basis and other data reported at a less frequent timescale such as daily maximum. Of the 94 monitoring wells, there are 26 wells with breakpoint data that provide elevation data at a temporal interval between 1 minute and 30 minutes, while 68 of the wells provide daily maximum elevations only. The depths of the monitoring wells vary from as shallow as 12 feet to as deep as 130 feet.

3.4.5.2. SATURATED ZONE BOUNDARY CONDITIONS

Utilizing a combination of the monitoring well data and surface water the boundary conditions are specified within MIKE SHE in the Computational Layers of the Saturated Zone module. The approach used was to specify boundary conditions using groundwater monitoring stations in the inland areas and a tidal gage for the coast. For locations where groundwater data was lacking, the model boundary was aligned with surface water features and in these areas surface water monitoring station data was used.

Figure 3-42 provides the Saturated Zone boundary conditions for all three surficial aquifer layers. Daily groundwater and surface water monitoring data was used for all inland boundaries, while the coastal boundary uses breakpoint data from the Virginia Key tidal gage. The western boundary is comprised of measured data from groundwater stations G-968, SBS1-PZ2, G-3576, and G-3578. Notably, in areas such as WCA3B, the surface water stage and groundwater elevation are virtually indistinguishable. A comparison of the surface water monitoring station EDEN7 within the WCA3B with the G-968 groundwater well demonstrated the measurements were nearly identical.

At some locations along the northern and southern boundaries, measured surface water data was used. Along the C7 Canal, station data at G72_H and S27_H were used to interpolate stages along the canal between the two stations. The length of boundary that is labeled “C7_Interp” is the result of a small-scale model interpolating between two stations adjacent stations and using the resulting time-varying spatial data. All boundaries shown are proposed to be developed as fixed head boundaries.

Figure 3-41 – Locations of Groundwater Monitoring Wells Available for Calibration Reference

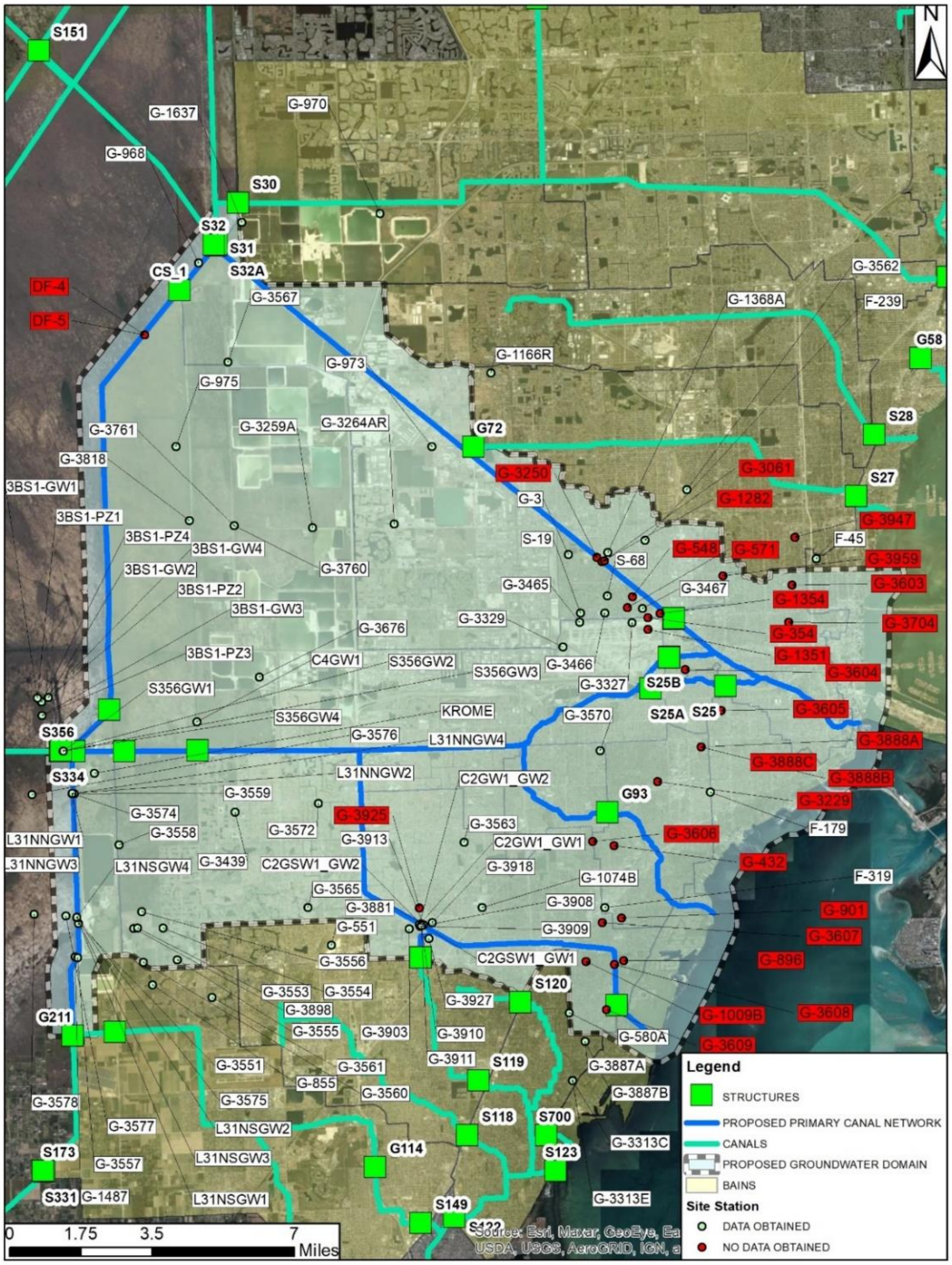
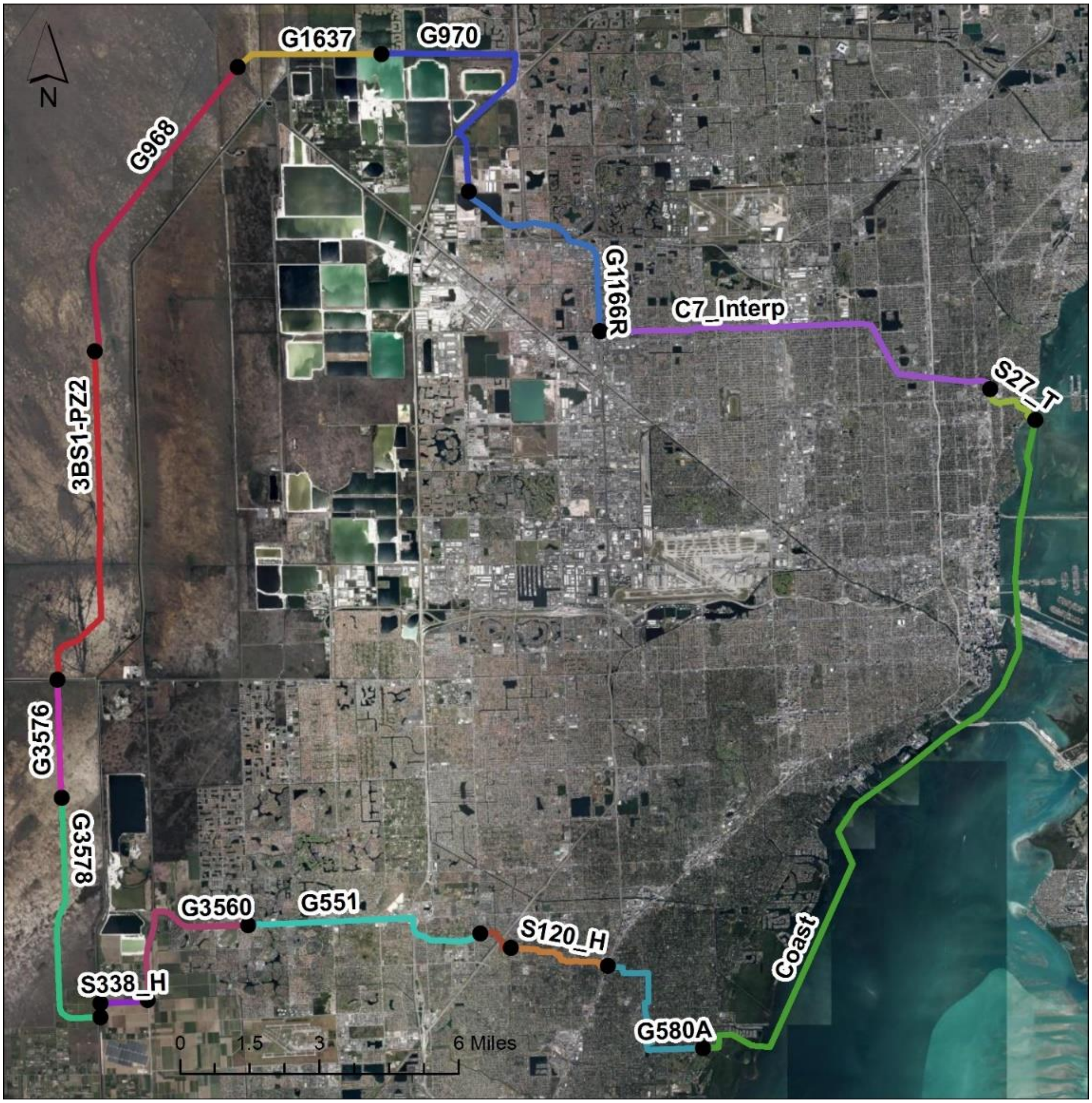


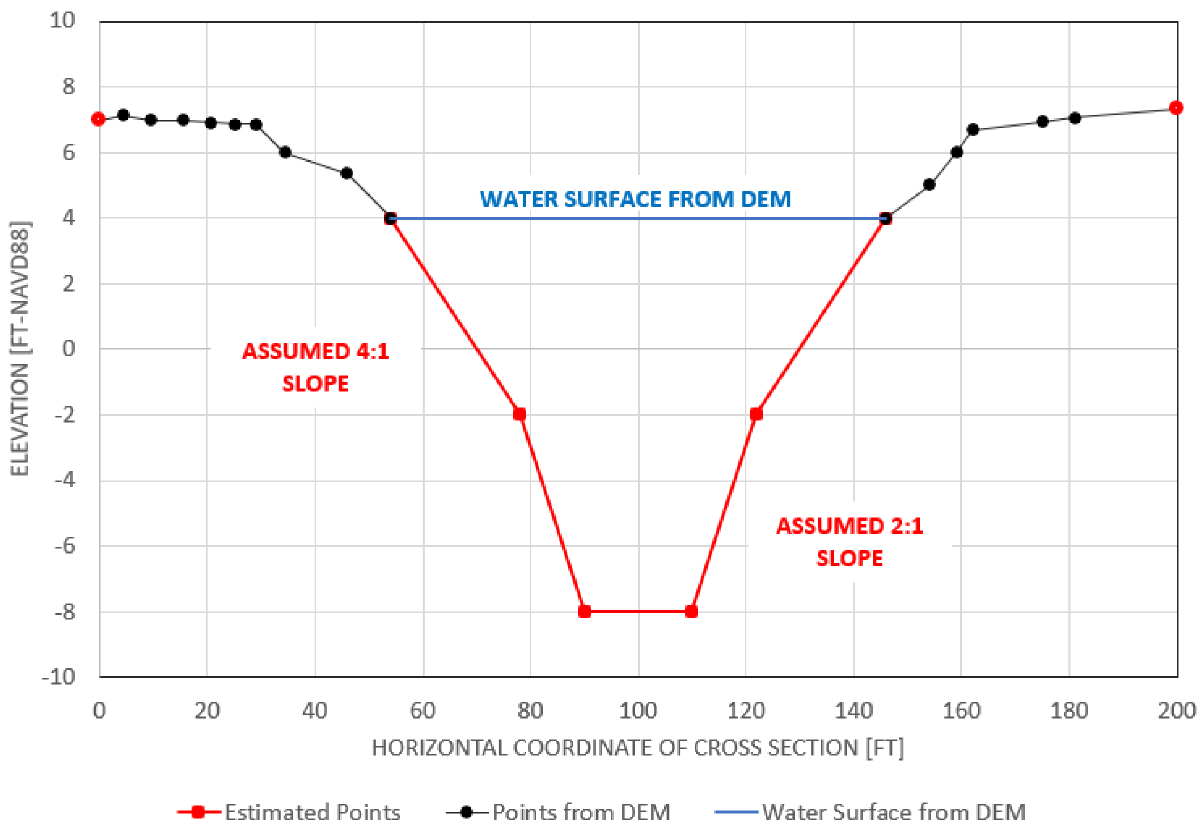
Figure 3-42 – Boundary Condition Reference Data for the Saturated Zone Model



3.5. DATA GAP FILLING

For canal sections where data was unavailable, the geometry was estimated based on the top width of the canal as measured from aerial imagery and the assumed depth of the canal based on the closest available structure invert. The side slopes of the canal were assumed based on typical canals in the region. **Figure 3-43** illustrates a template for developing a cross-section where no data is available, the elevation data above the water line was extracted from the 5 feet resolution DEM, while the subaqueous elevations were based on assumed slopes of 4:1 and 2:1. For missing sections of available monitoring data timeseries, statistical techniques were utilized based on any available data at that location from different times or adjacent locations at the same time.

Figure 3-43 – Template for Developing Canal Cross-Sections without Survey



4. MODEL DEVELOPMENT

4.1. SPATIAL DATA

The MIKE SHE model required a series of spatial datasets that provided a parameterization of the land surface and subsurface necessary to represent the hydrology and hydrogeology of the model domain. During the model development process there were several changes in assumptions that resulted in revisions to the spatial datasets prior to incorporating them within the MIKE SHE model. These modifications are described in the following subsections.

4.2. MODEL DOMAIN

At various points along the northern and southern extent of the proposed model domain in the Data Collection Task, there were revisions in the boundary conditions that necessitated minor changes in the limits. The most pronounced difference was the addition of the C7 Canal as a fixed head boundary for the saturated zone model. Because the model domain described in the Data Collection Task had a northern boundary of the C6 and C7 watershed divide, the updated domain had to be expanded to the north to include a larger area. At the southern limit, the model domain was refined to reflect a series of topographic divides and surface water features outside of the subject watershed limits. **Figure 4-1** Illustrates the updated model domain in comparison to the subject basins.

4.3. TOPOGRAPHY

In consideration of the updated model domain, additional topographic data was collected from the District. This data utilized the same sources documented in the Data Collection Task and consists of a Digital Elevation Model (DEM) in the NAVD datum at a 5-foot spatial resolution. The dataset was compiled based on 5 datasets that were mosaicked from sources including the USGS, Miami Dade County at the District over a range of dates from 2007 through 2020 (\\gisdata1\layers\Elevation\Basic_SouthFlorida).

As noted in the Data Collection Section, the DEM was re-sampled, for use in the MIKE SHE model, using a raster averaging technique to develop a revised version with a 125-foot resolution to match the computational grid scale used by MIKE SHE. The re-sampling technique was evaluated to demonstrate that the raster averaging approach did not impact the ability of the topographic data to provide the level of detail needed to properly represent the hydrology within MIKE SHE. While the larger resolution topographic data was used as a model input, the original 5-foot resolution DEM was utilized for any calculations of cross-section data or other computational efforts. As shown in **Figure 4-2** a larger area for the topography was included than was needed for the limits of the expanded model domain. This approach was intended to accommodate any potential changes in the future to the model domain limits.

4.4. NEXRAD DATA

The change in model domain also required additional NEXRAD data for the expanded limits, as shown in **Figure 4-3** which also includes an illustration of the location of rain gages in the region. To incorporate this data into the MIKE SHE framework, rainfall depth at a 15-minute interval was applied evenly to all grid cells within each respective NEXRAD cell. The format used is a time-varying dfs2 file that incorporates a separate map of spatially varying rainfall depth for each time-step during the simulation duration. An example of the NEXRAD data from the time varying dfs2 file for September 10, 2017, at 7:30 AM is shown in **Figure 4-4**, demonstrating how the spatial variability of the storm event is applied to each grid cell within the model domain at each timestep. This graphic demonstrates a pattern reflective of the “rain bands” that Hurricane Irma presented during landfall. By utilizing the NEXRAD dataset for the model forcings, the spatial variability of rainfall is properly represented within each watershed.

An analysis of the NEXRAD data for both the May 2020 period and September 2017 period demonstrated the variation in peak rainfall within the model domain for both the 1-day maximum and 3-day maximum. **Figure 4-5** through **Figure 4-8** illustrate the results of the analysis in the format of contour maps. These maps demonstrate that for the 2020 event there were some locations in the model domain that had over 7 inches of rain in 24 hours and over 15 inches in 72 hours, while other locations in the model domain had as little as 1 inch and 6 inches respectively for the same periods. For the 2017 event, there were some locations in the model domain that had over 6 inches of rain in 24 hours and over 8 inches in 72 hours, while other locations in the model domain had as little as 3 inches and 5 inches respectively for the same periods. This analysis demonstrates that the 2020 calibration event has locations with more intense rainfall, but is less spatially uniform, while the 2017 validation event has a more even spatial distribution but less localized intensity.

Figure 4-1 – Updated Model Domain

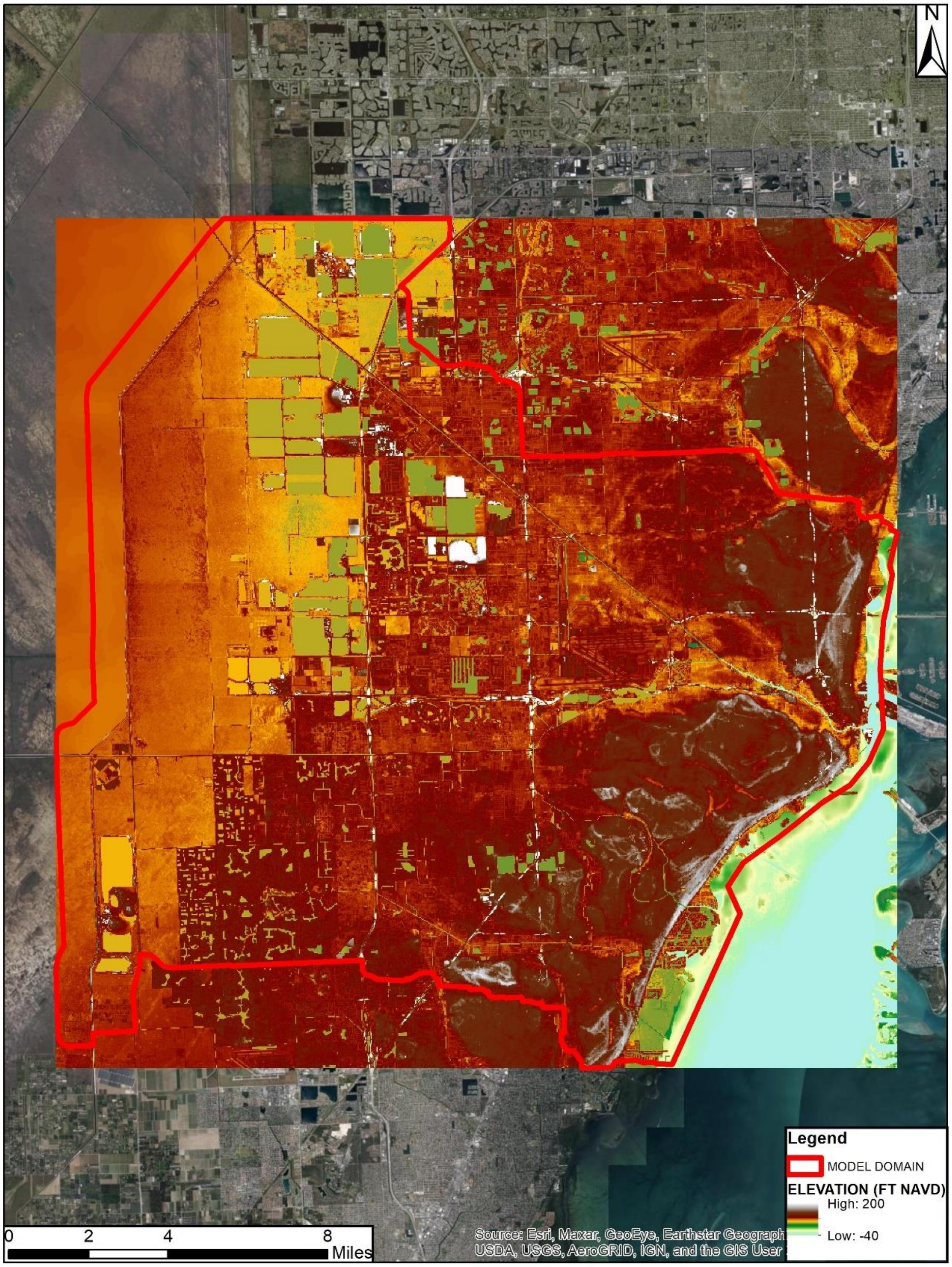


Figure 4-2 – Updated DEM at the 125-ft Computational Grid Cell Size Resolution

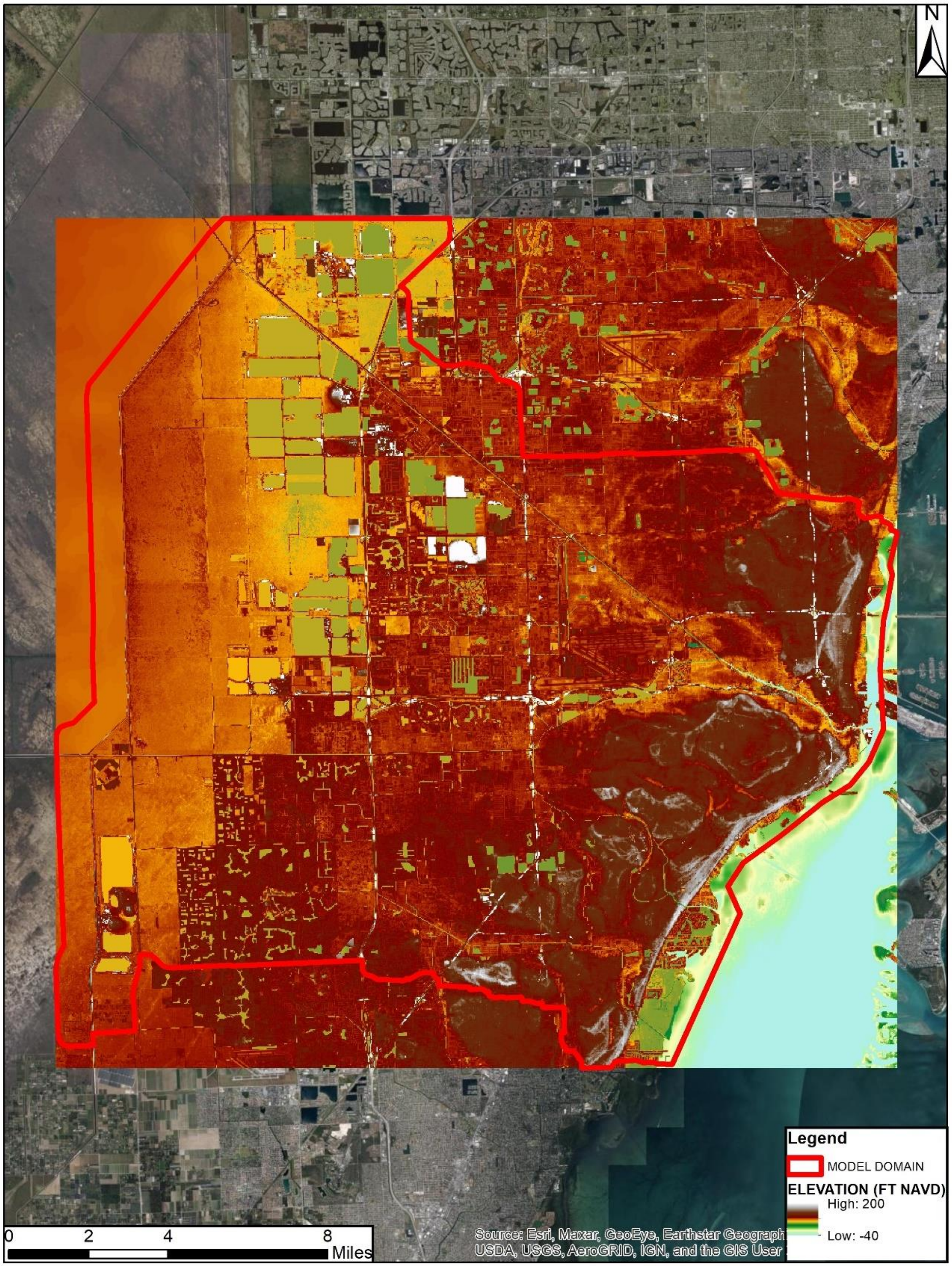


Figure 4-3 – NEXRAD Grid Cells for Model Input

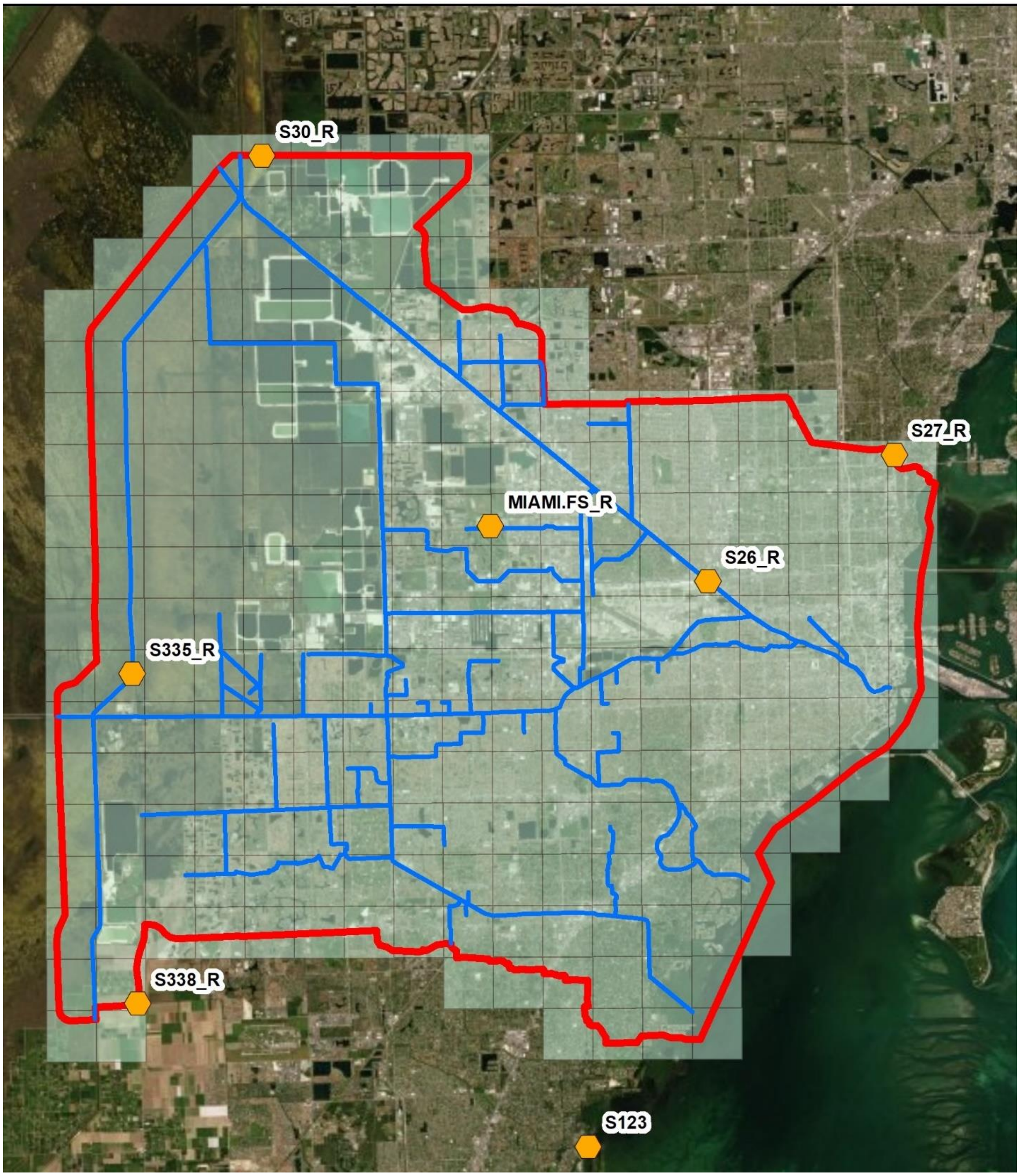


Figure 4-4 – NEXRAD Data for 2017 Storm Event

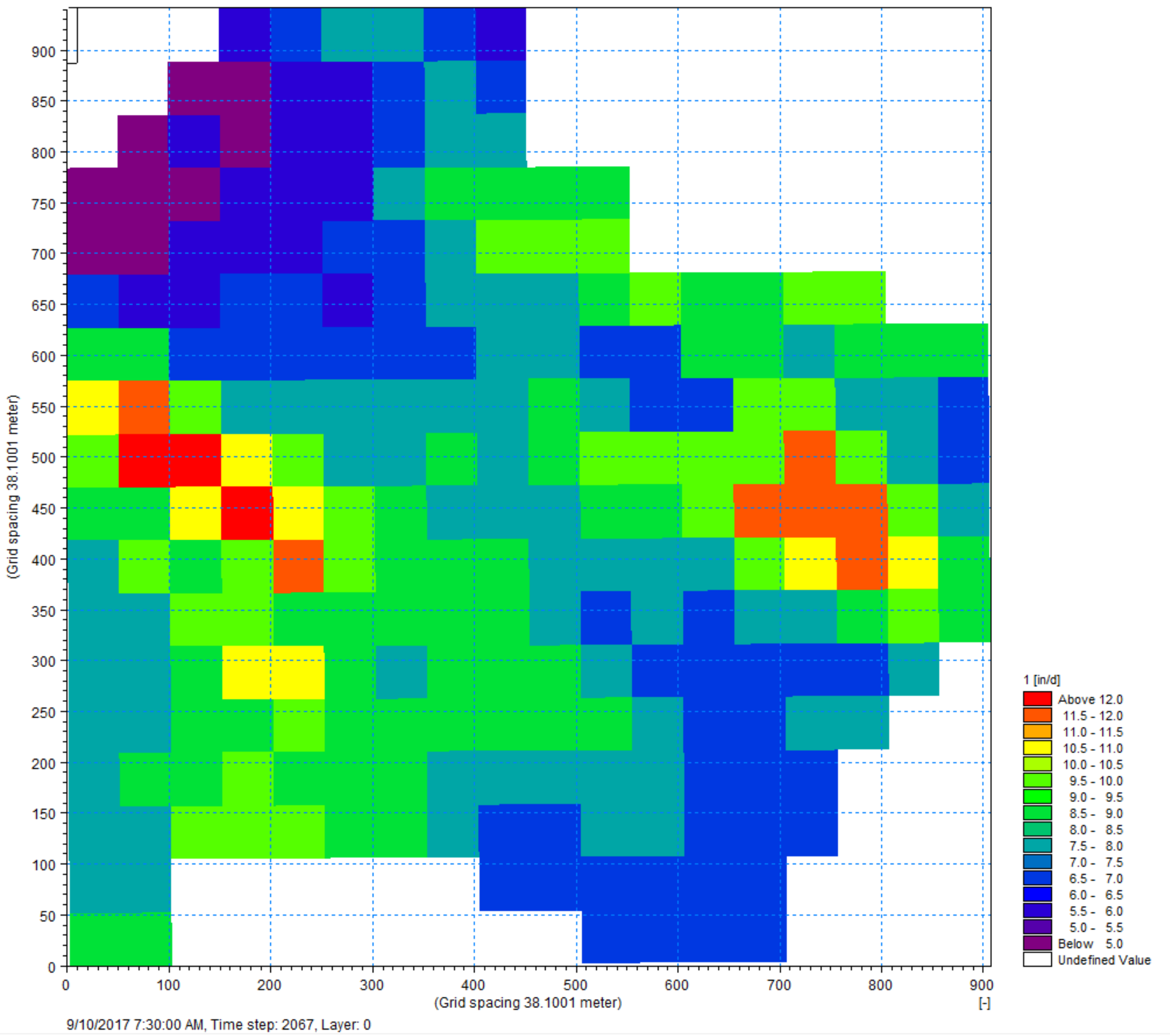


Figure 4-5 – Spatial Variability of Maximum 1-Day Rainfall for May 25, 2020

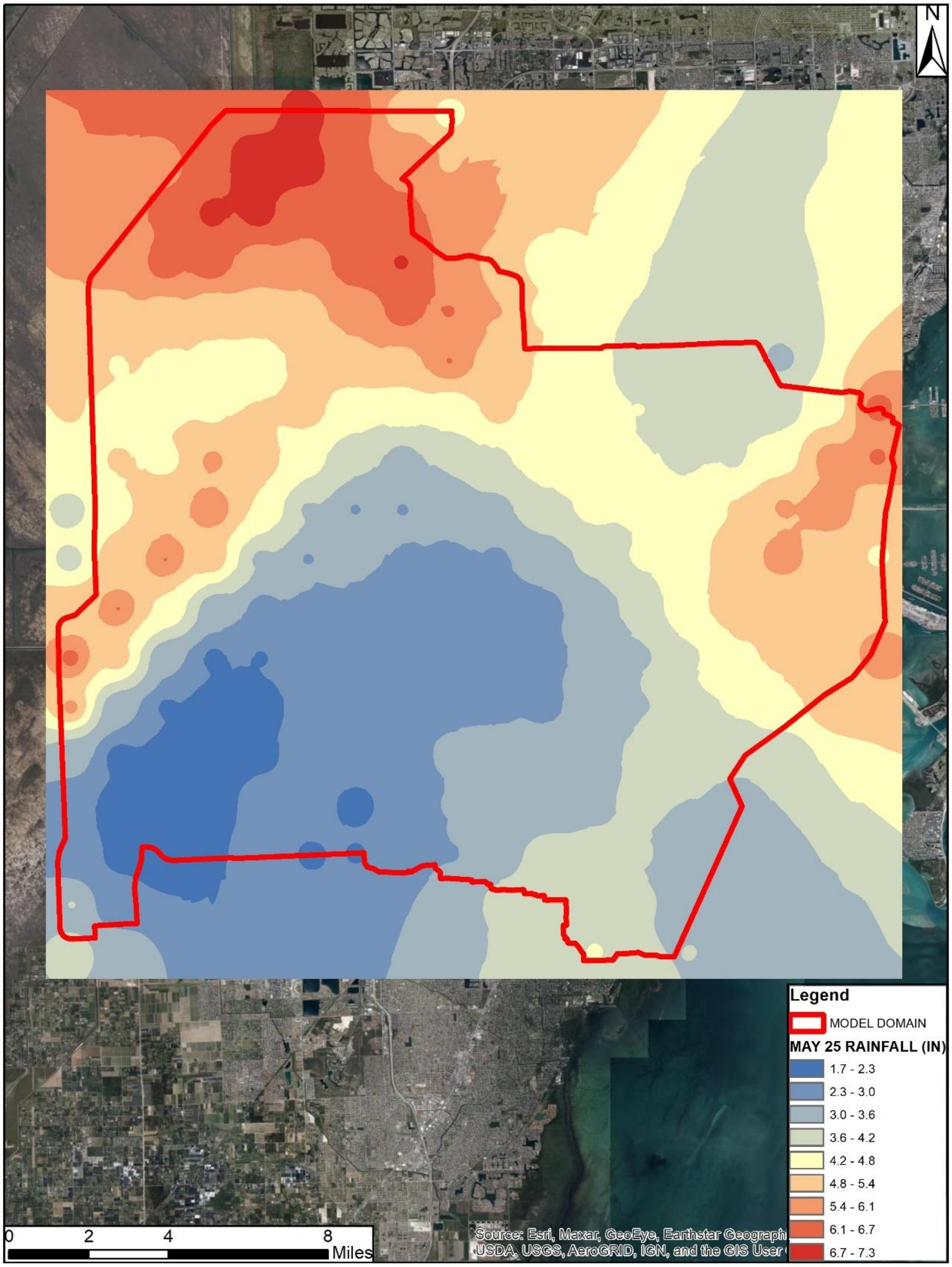


Figure 4-6 – Spatial Variability of Maximum 1-Day Rainfall for September 10, 2017

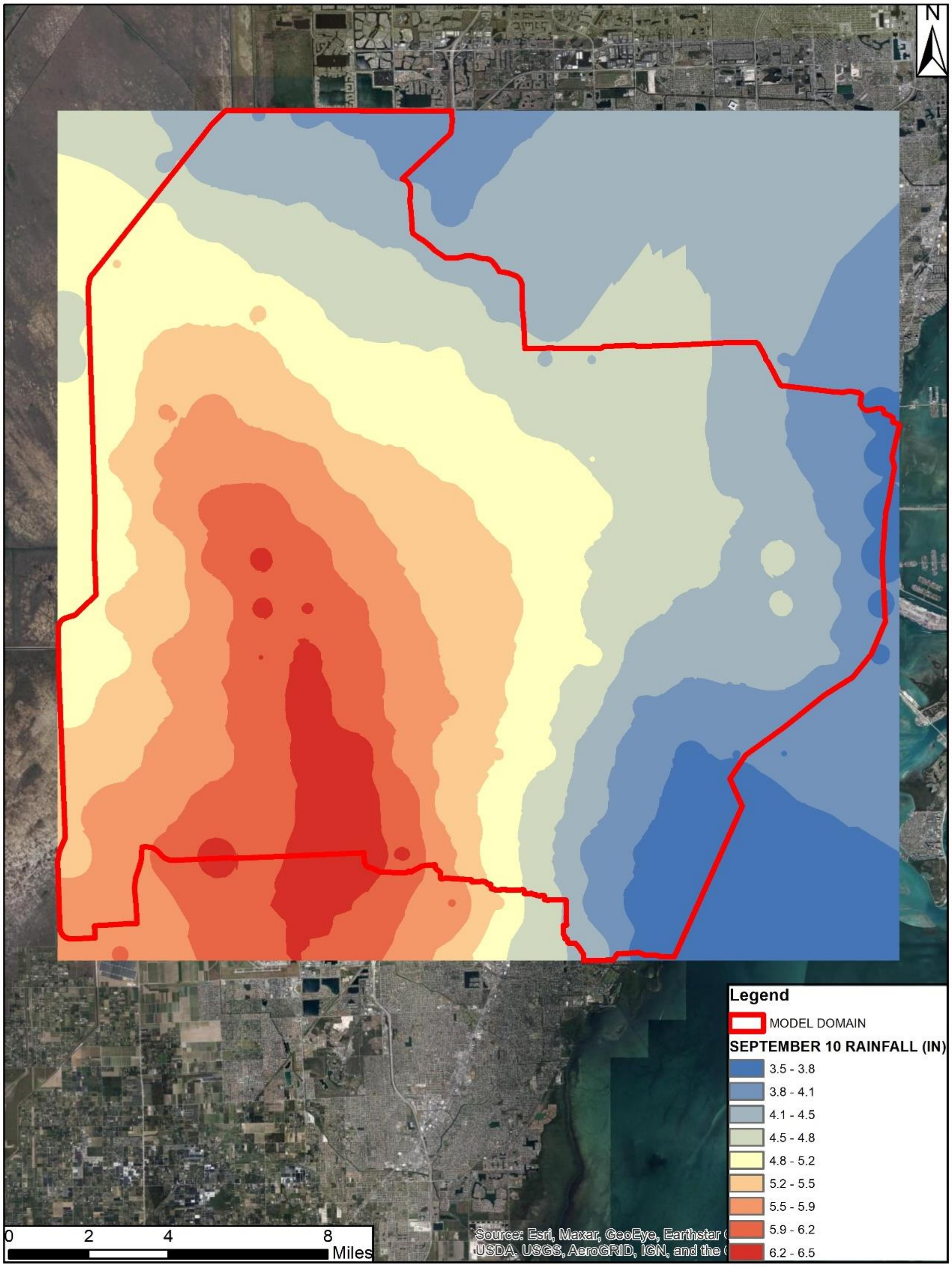


Figure 4-7 – Spatial Variability of Maximum 3-Day Rainfall for May 24-26, 2020

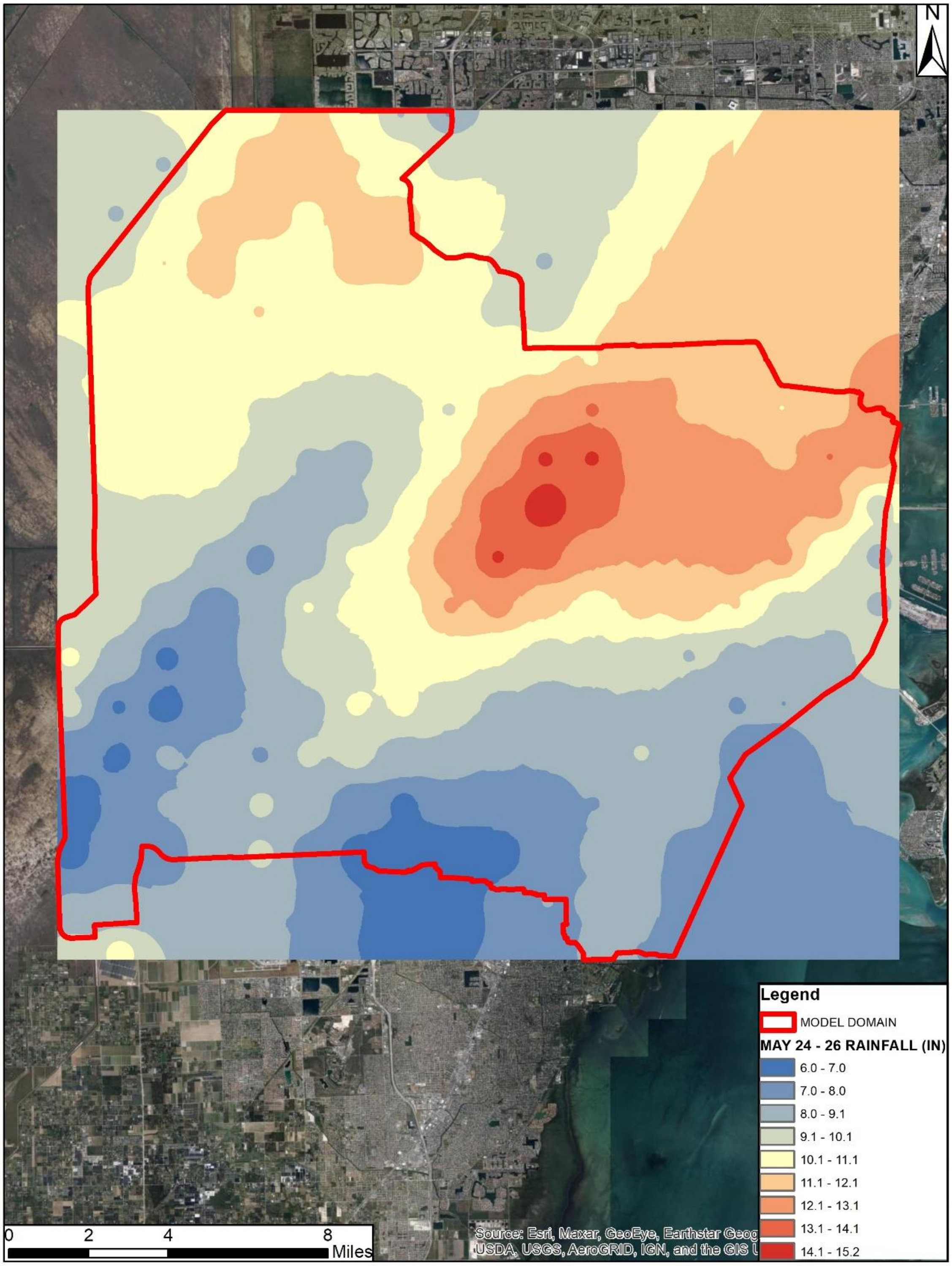
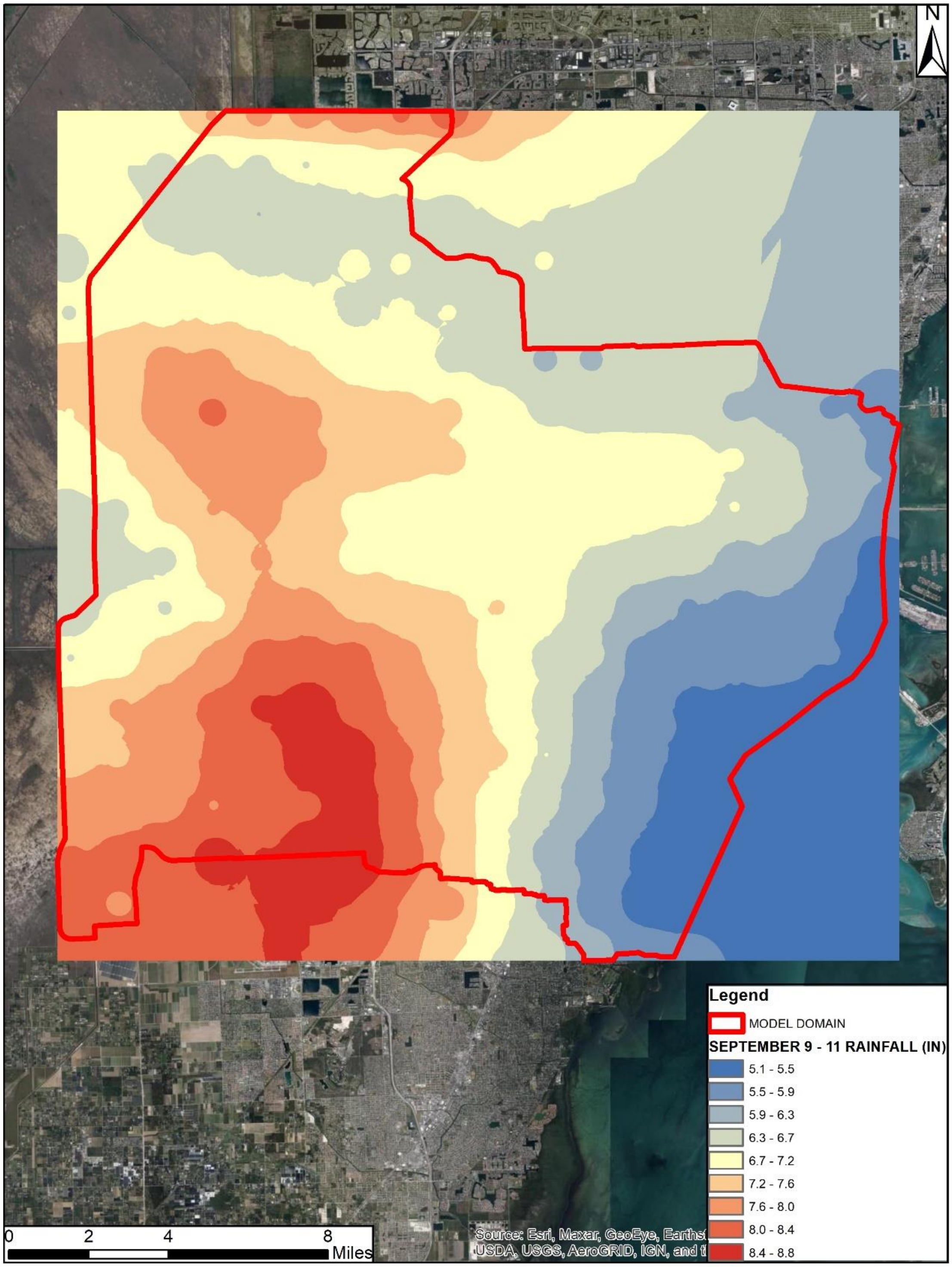


Figure 4-8 – Spatial Variability of Maximum 3-Day Rainfall for September 9-11, 2017



4.5. LAND USE - ROUGHNESS AND DETENTION STORAGE

As documented in earlier sections, the parameterization of overland flow within MIKE SHE utilizes an assumption for hydraulic roughness and detention storage that is based on land use. A map of land use based on the Florida Land Use Cover Classification System (FLUCCS) categorization codes was available for the model domain through the Florida Department of Environmental Protection (FDEP) for a range of years between 2017 and 2019. This dataset provided an analog to the potential resistance for overland flow and availability of detention storage. The variability in land use within the expanded model domain is shown in **Figure 4-9**. The relationship between land use code, Manning's M, detention storage, drainage time constant and runoff coefficient is documented in **Table 4-1**.

4.6. UNSATURATED ZONE

Water within the unsaturated zone is stored in the pore space of the soil matrix and is available for either evapotranspiration through root uptake or recharge to the saturated zone through downward infiltration. As the pore space in the unsaturated zone fills up with water, rainfall is diverted from infiltration to overland flow. Within the MIKE SHE framework the unsaturated zone can be represented using one of the following three schemes to approximate the movement of water within the soil matrix:

- **Richards Equation** - the most computationally intensive, but also the most accurate when the unsaturated flow is dynamic.
- **Gravity Flow** - assumes a uniform vertical gradient and ignores capillary forces to provide a solution for time varying recharge to the groundwater table based on actual precipitation and evapotranspiration and not the dynamics in the unsaturated zone.
- **Two Layer Water Balance** - divides the unsaturated zone into two zones: the root zone and the zone between the roots and the water table.

The Richards Equation is the most robust approach for a long-term, continuous simulation of hydrology. However, for a wet season simulation, where the soil column should remain saturated for the entirety of the simulation period, the dynamics of the unsaturated zone are not of critical importance; and, therefore, the increased complexity of model computation with the Richards Equation increases the simulation run time without offering a significant improvement in accuracy. By comparison, for a single storm event where drying and re-wetting of the soil matrix are not a primary concern, the Gravity Flow method is a very representative approach that is more computationally efficient. Therefore, for the model of the C2, C3W, C4, C5 and C6 watersheds the Gravity Flow method was selected to represent the dynamics within the unsaturated zone.

The datasets used to populate the parameters required to represent the unsaturated zone within the Gravity Flow schema from the Soil Survey Geographic Database (SSURGO) were exported from the 2018 dataset available from the Florida Geographic Data Library (FGDL). This dataset provided quantified values for the following soil parameters:

- Water Content at Saturation,
- Water Content at Field Capacity,
- Water Content at Wilting Point, and
- Saturated Hydraulic Conductivity.

During the model development and debugging process it was determined that for a few of the soil types, the values for water content at saturation provided by the 2018 FGDL data showed water content at field capacity exceeding water content at saturation, which did not reflect the physics of the MIKE SHE model. A 2014 dataset made available by the USGS was reviewed for each soil type, with the values published for porosity used as the replacement values for water content at saturation (Wieczzorek, 2014). An updated map of the soils for the revised model domain is included in **Figure 4-10** and the revised parametrization for each soil type is included in **Table 4-2**.

Table 4-1 – Model Parameters for Land uses within the Model Domain

FLUCCS	Land Use	Manning's M	Manning's n	Detention [in]	Drainage Time Constant [s^{-1}]	Runoff Coefficient [-]
1100	Residential, Low Density	7.14	0.14	0.10	5.79e-06	0.075
1200	Residential, Medium Density	8.33	0.12	0.10	5.79e-06	0.22
1300	Residential, High Density	9.09	0.11	0.10	5.79e-06	0.45
1400	Commercial and Services	14.29	0.07	0.10	5.79e-06	0.75
1500	Industrial	14.29	0.07	0.10	5.79e-06	0.40
1600	Extractive	16.67	0.06	0.10	0	0.40
1700	Institutional	7.69	0.13	0.10	5.79e-06	0.30
1800	Recreational	7.69	0.13	0.30	5.79e-06	0.075
1900	Open Land	7.14	0.14	0.15	5.79e-06	0.075
2100	Cropland and Pastureland	5.88	0.17	0.15	5.79e-06	0.10
2200	Tree Crops	5.88	0.17	0.25	5.79e-06	0.10
2300	Feeding Operations	5.88	0.17	0.25	5.79e-06	0.10
2400	Nurseries and Vineyards	5.88	0.17	0.25	5.79e-06	0.10
2500	Specialty Farms	5.88	0.17	0.25	5.79e-06	0.10
2600	Other Open Lands - Rural	7.14	0.14	0.15	5.79e-06	0.075
3100	Herbaceous (Dry Prairie)	7.69	0.13	0.15	5.79e-06	0.075
3200	Upland Shrub and Bushland	3.33	0.30	0.15	5.79e-06	0.075
3300	Mixed Rangeland	3.33	0.30	0.15	5.79e-06	0.075
4100	Upland Coniferous Forest	2.22	0.45	0.40	5.79e-06	0.075
4200	Upland Hardwood Forest	2.22	0.45	0.40	5.79e-06	0.075
4300	Upland Mixed Forest	2.22	0.45	0.40	0	0.075
4400	Tree Plantations	2.22	0.45	0.40	0	0.075
5100	Streams and Waterways	16.67	0.06	0.00	0	0.00
5200	Lakes	16.67	0.06	0.00	0	0.00
5300	Reservoirs	16.67	0.06	0.00	0	0.00
5400	Bays and Estuaries	16.67	0.06	0.00	5.79e-06	0.00
5700	Ocean and Gulf	16.67	0.06	0.00	5.79e-06	0.00
6100	Wetland Hardwood Forest	2.22	0.45	0.40	5.79e-06	0.00
6400	Vegetated Non-Forested Wetlands	3.33	0.30	0.40	5.79e-06	0.00
6500	Non-Vegetated Wetlands	6.67	0.15	0.00	5.79e-06	0.56
7200	Sand other than Beaches	16.67	0.06	0.10	5.79e-06	0.075
7400	Disturbed Land	7.14	0.14	0.10	5.79e-06	0.15
8100	Transportation	9.09	0.11	0.10	5.79e-06	0.075
8200	Communications	7.14	0.14	0.10	5.79e-06	0.22
8300	Utilities	7.14	0.14	0.10	5.79e-06	0.45

Table 4-2 – Infiltration Capacity Characteristics of Soils within the Model Domain

MUKEY	Soil Name	Water Content at Saturation [%]	Water Content at Field Capacity [%]	Water Content at Wilting Point [%]	Saturated Hydraulic Conductivity [ft/day]	Amount of Model Domain by Area [%}
631567	Biscayne Gravelly Marl, Drained	57.89	15.00	7.50	12.09	0.02
631577	Biscayne Marl	62.25	15.00	8.00	9.46	0.75
631580	Biscayne Marl, Drained	61.93	17.00	10.00	9.77	0.03
631588	Biscayne Marl-Rock Outcrop Complex	58.65	17.00	10.00	14.31	0.87
631586	Chekika Very Gravelly Loam	44.99	14.00	7.50	11.06	1.99
631578	Dania Muck, Frequently Ponded, 0 To 1 Percent Slopes	87.66	49.80	11.30	24.58	7.59
631590	Demory Sandy Clay Loam-Rock Outcrop Complex	52.52	8.60	3.30	7.64	1.29
631596	Hallandale Fine Sand, 0 To 2% Slopes	44.50	8.90	2.70	19.82	2.52
631572	Krome Very Gravelly Loam	51.25	14.00	11.00	9.54	0.31
631568	Lauderhill Muck, Frequently Ponded, 0 To 1 Percent Slopes	89.68	55.00	12.00	25.12	22.09
631587	Matecumbe Muck	86.51	90.00	45.00	23.87	0.12
631585	Opalocka Sand-Rock Outcrop Complex	42.54	5.20	1.50	44.56	0.18
631592	Pahokee Muck, Depressional	77.49	90.00	45.00	25.77	3.85
631593	Pennsuco Marl, Tidal	64.23	44.30	13.90	3.69	0.33
631576	Perrine Marl	63.03	56.20	4.80	7.87	0.30
631595	Plantation Muck	64.56	46.90	22.00	25.62	0.78
631599	Rock Outcrop-Vizcaya-Biscayne Complex	66.19	0.00	0.00	22.11	0.18
631582	Tamiami Muck, Depressional	80.32	71.50	36.10	21.24	1.69
631594	Terra Ceia Muck, Tidal	88.01	90.00	45.00	26.08	0.24
631603	Udorthents, Limestone Substratum, 0 To 5 Percent Slopes	47.17	4.00	1.10	25.62	1.35
631574	Udorthents, Limestone Substratum-Urban Land Complex	47.17	0.00	0.00	25.81	6.16
631575	Udorthents, Marl Substratum-Urban Land Complex	59.34	15.20	8.00	10.85	0.31
631573	Udorthents-Water Complex	47.17	0.00	0.00	18.43	5.08
631579	Urban Land, 0 To 2 Percent Slopes	47.17	0.20	0.08	25.80	41.61

4.7. SATURATED ZONE (HYDROGEOLOGY)

During initial model development the model hydrogeology was represented by three saturated zone layers in the MIKE SHE model with the same thickness, horizontal conductivity, vertical conductivity, specific storage, and yield as the Urban Miami Dade MODFLOW model created by USGS (Hughes and White, 2016). However, through extensive literature review, a series of sensitivity analyses, and the general model calibration process documented in **Section 5**, these parameters and layering were refined to perform better throughout the model domain.

The following subsection describes the initial modeling approach for drain level.

4.7.1. DRAINAGE LEVEL

Within MIKE SHE the drainage level function allows for representation of tertiary drainage features that function to collect runoff and manage surficial groundwater levels but are too small in scale to be represented within the hydraulic model. Examples of these types of tertiary stormwater management features include shallow swales, underdrains, and exfiltration trenches. The drainage level function allows for areas within the model domain to be designated as managing stormwater to direct water to runoff that is in the Saturated Zone above a specified depth.

Based on the patterns of development in Miami Dade County and typical infrastructure types, the model assumes a drainage level equal to 1.5 feet of depth below the surface for all developed areas. For undeveloped areas there is no specified drainage level (zero feet of depth), and for a handful of managed agricultural areas it is assumed that the water table management operations are such that the drainage level is at 2.5 feet of depth below the surface. Wetlands are treated as undeveloped areas with no specified drainage level, or 0.5 feet of depth below the surface. Despite referring to values below the ground surface, the sign convention for drainage level is positive as that is consistent with the MIKE SHE sign convention.

These values are consistent with assumptions from other FPLOS modeling efforts in the region, specifically the models for the C8, C9 and South Miami Dade Basins. **Figure 4-11** illustrates the drainage levels designated within the model domain. Of note, most of the model domain is urban and suburban development which has a drainage level of 1.5 feet. The undeveloped areas mostly consist of the Pennsuco Wetlands in the western portions of the C4 and C2 watersheds. There are minimal areas that are classified as agricultural uses within the model domain. A mix of tree farms, row crops and grasslands make up the handful of parcels defined as having a drainage level of 2.5 feet.

Figure 4-9 – Updated Land Use Map for Model Domain

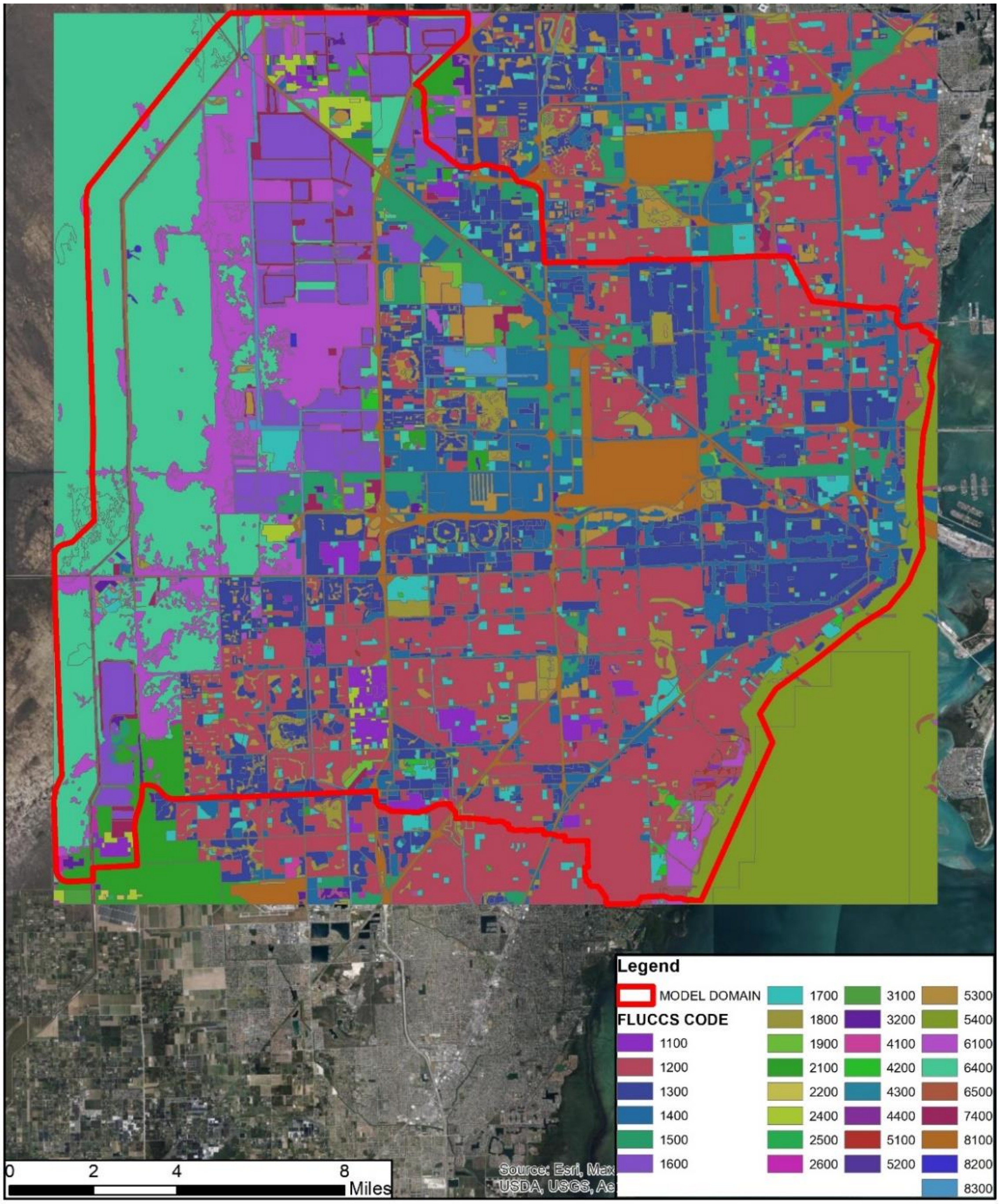


Figure 4-10 – Soils Map for Model Domain

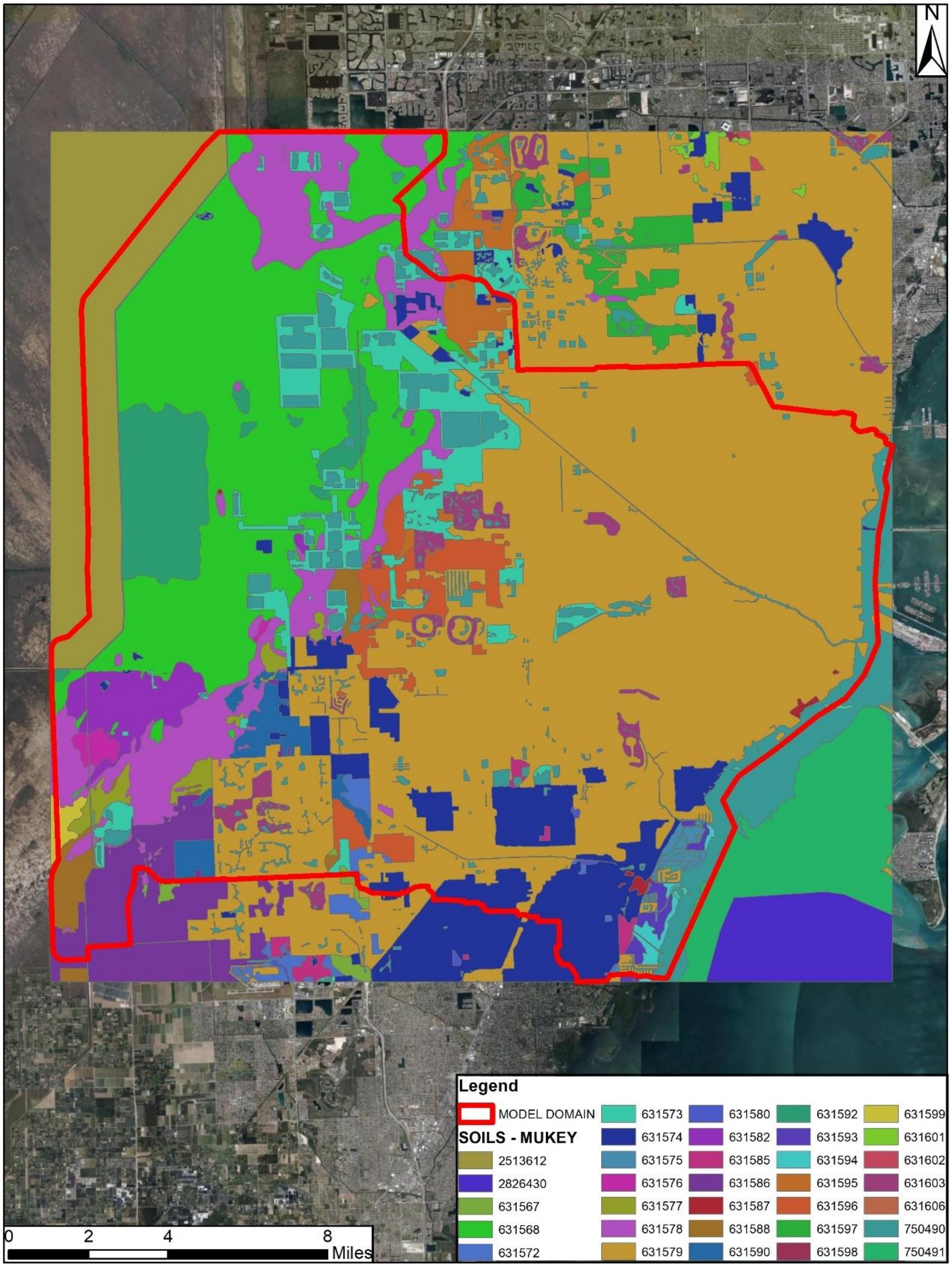
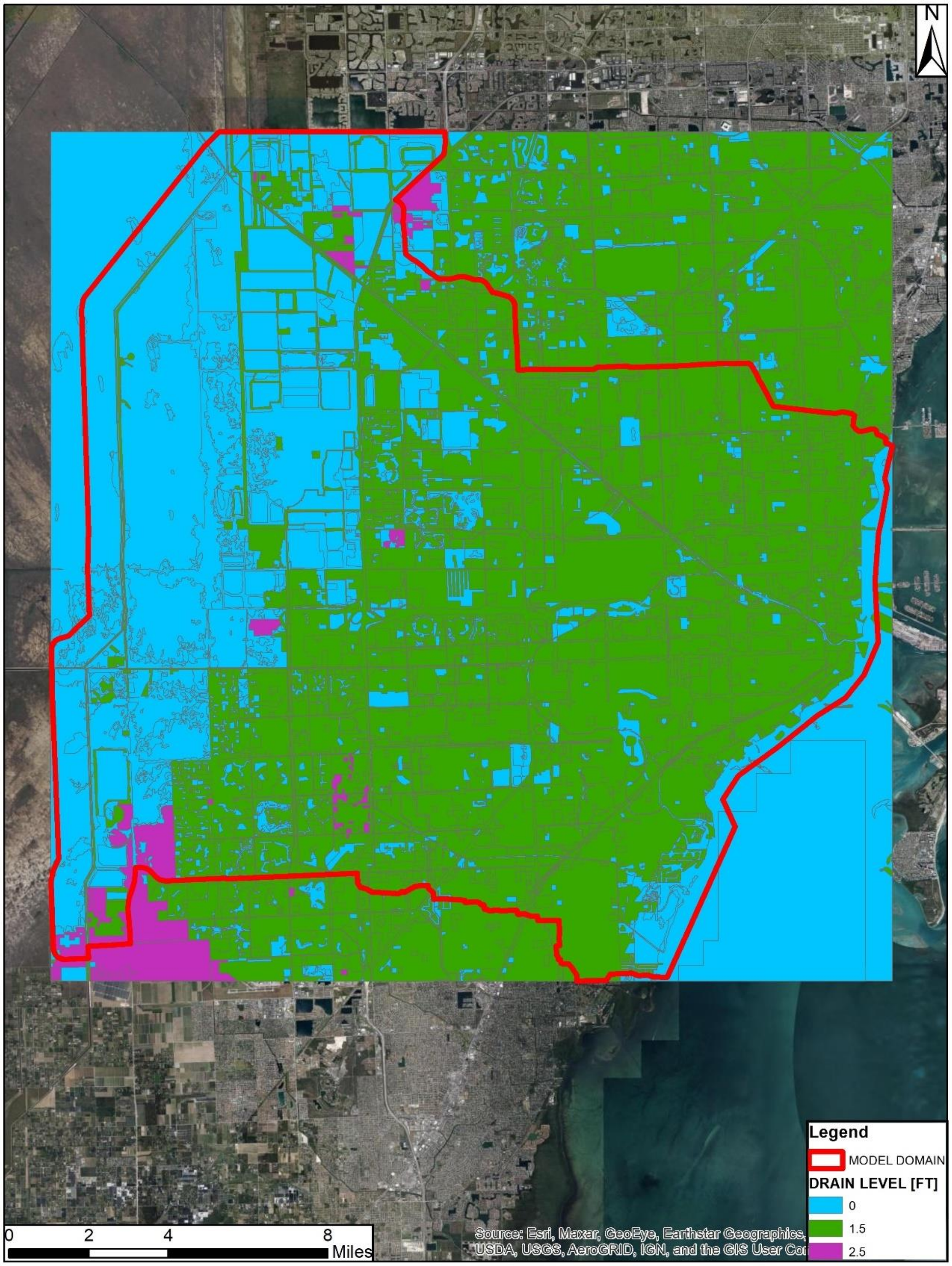


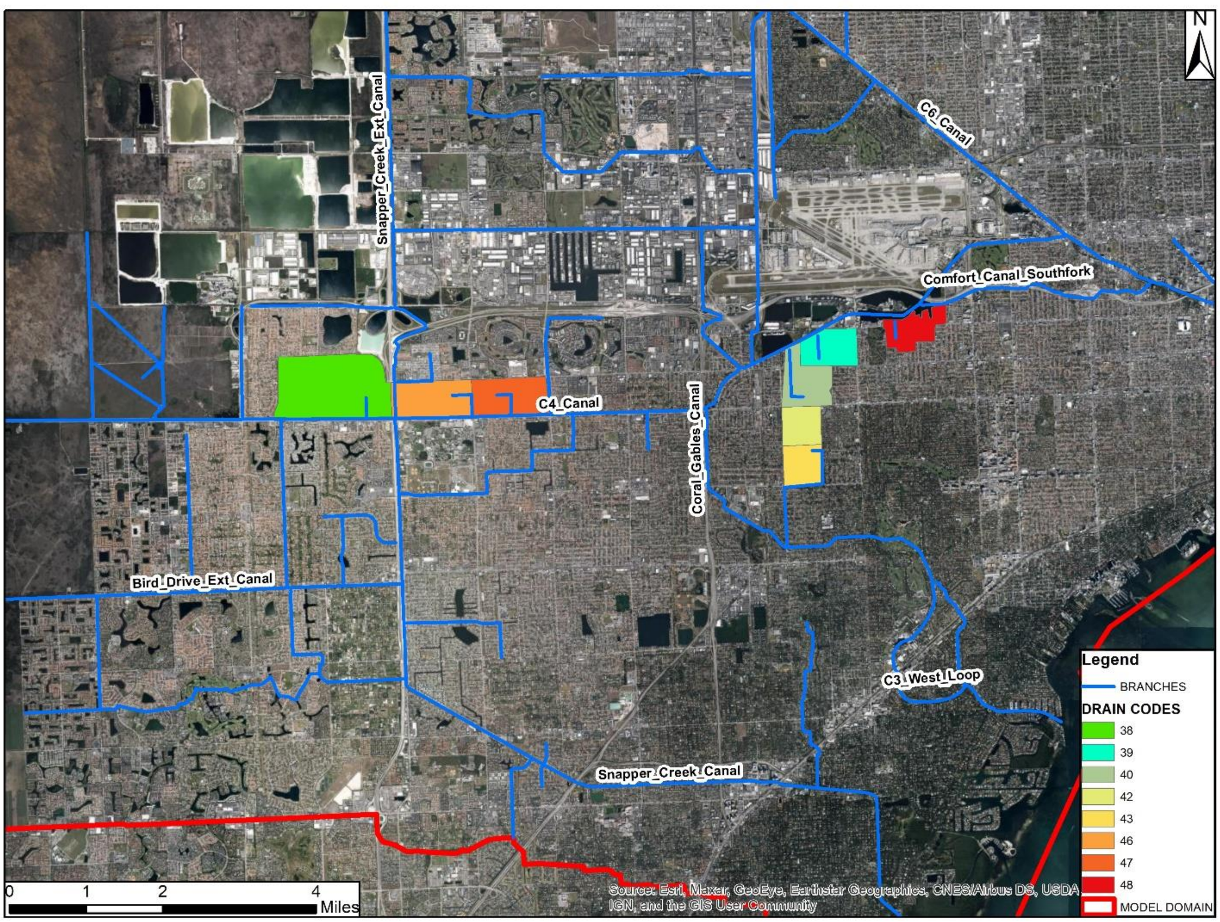
Figure 4-11 – Drainage Levels Specified for the Saturated Zone



4.8. SPECIFIED DRAINAGE - DRAIN CODES

One element that interconnects the MIKE SHE and MIKE 1D models is the parameterization of specified drainage. Once the two-dimensional and one-dimensional components of the model are coupled, the default setting within MIKE SHE is for overland flow runoff to be routed to the nearest segment of MIKE 1D reach and applied as lateral inflow to the channel. However, there are locations where the model reach that is the closest spatially is not the correct receiving water. The primary instance of this is for the locations where municipal stormwater pumps collect the runoff from an urbanized area and discharge to a specific location. This occurs within the C3W and C4 watersheds as part of the municipal drainage facilities of Sweetwater, Belen, Miami, and West Miami. **Figure 4-12** illustrates the revised areas used to define the drain codes for the specified drainage parameterization.

Figure 4-12 – Drain Codes for the C3W and C4 Watersheds



4.9. INITIAL CONDITIONS

In preparing the first model simulations for each of the selected time periods, establishing representative initial conditions is considered best practice. For the hydraulic model, the initial conditions were based on measured stage data at the model boundaries, which consist of tidal data and structure data. For the saturated zone, the initial conditions were based on measured data in monitoring wells.

Figure 4-13 illustrates the initial groundwater conditions for the model domain at the beginning of the 2017 simulation period, while **Figure 4-14** illustrates the initial groundwater condition for the 2020 simulation period. These datasets were developed by creating a linearly interpolated surface of the water table using measured well data. Of note there is drawdown visible in the initial conditions for 2017 in the south at the Snapper Creek and Alexander Orr wellfields as well as in the north at the Miami Gardens wellfields. However, there is limited drawdown seen at the Northwest wellfield and this is due to utility operations at the time of the 2017 model start when there were lower withdrawal rates from the Northwest wells.

The same approach of interpolating values from monitoring wells to determine saturated zone elevations was performed on a daily basis to generate a time-varying fixed-head boundary condition along the model domain. This approach limits boundary effects for the MIKE SHE model. For the overland flow, there is a no-flow boundary at the model domain. This no-flow boundary is based on the selection of the domain limits at regional high points like major roadways or hydrologic boundaries such as canals. This approach for the overland flow boundary is consistent with the FPLOS modeling for the C8 and C9 Watersheds.

Figure 4-13 – Initial Conditions for the Saturated Zone for the 2017 Simulation

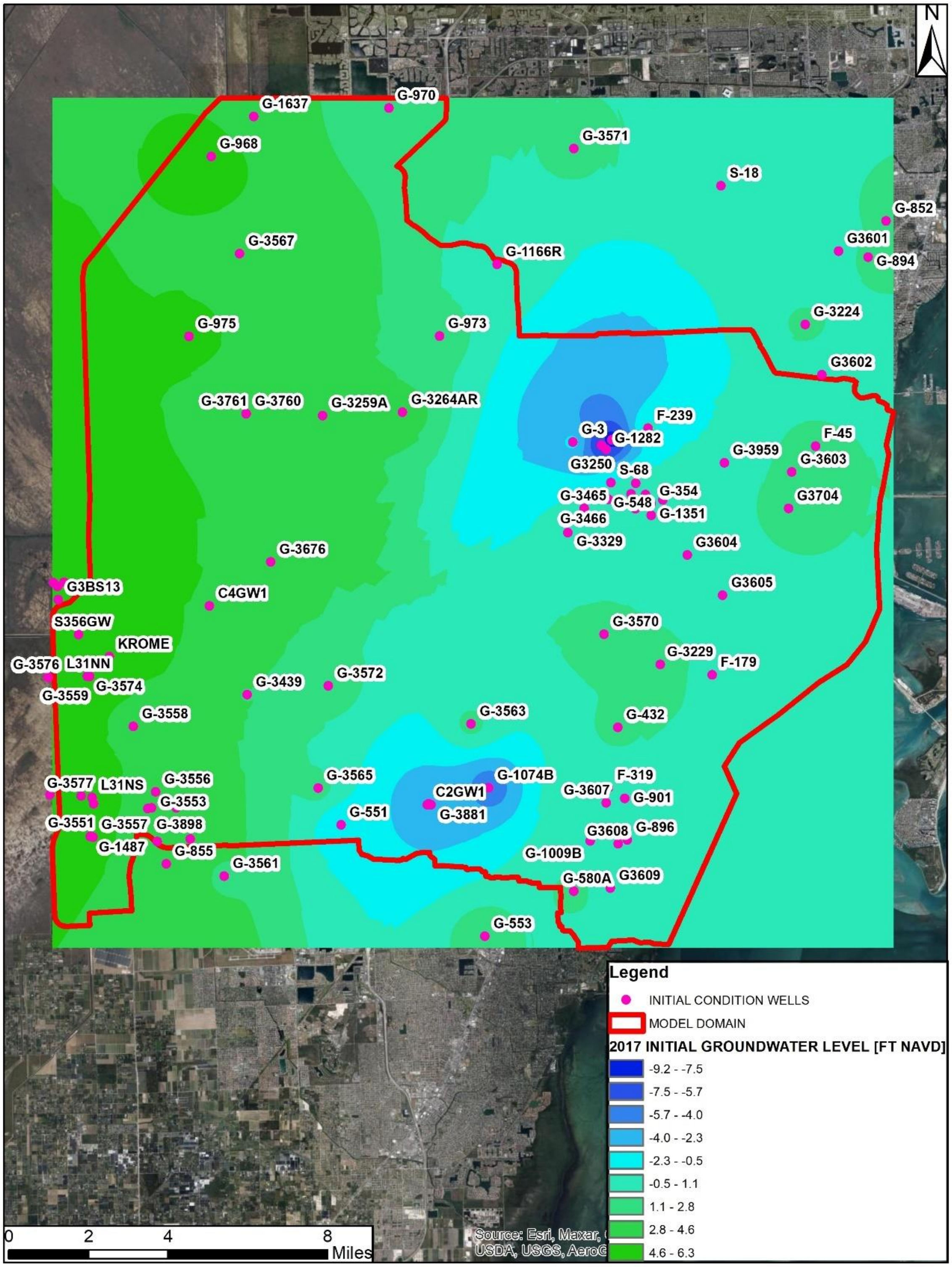
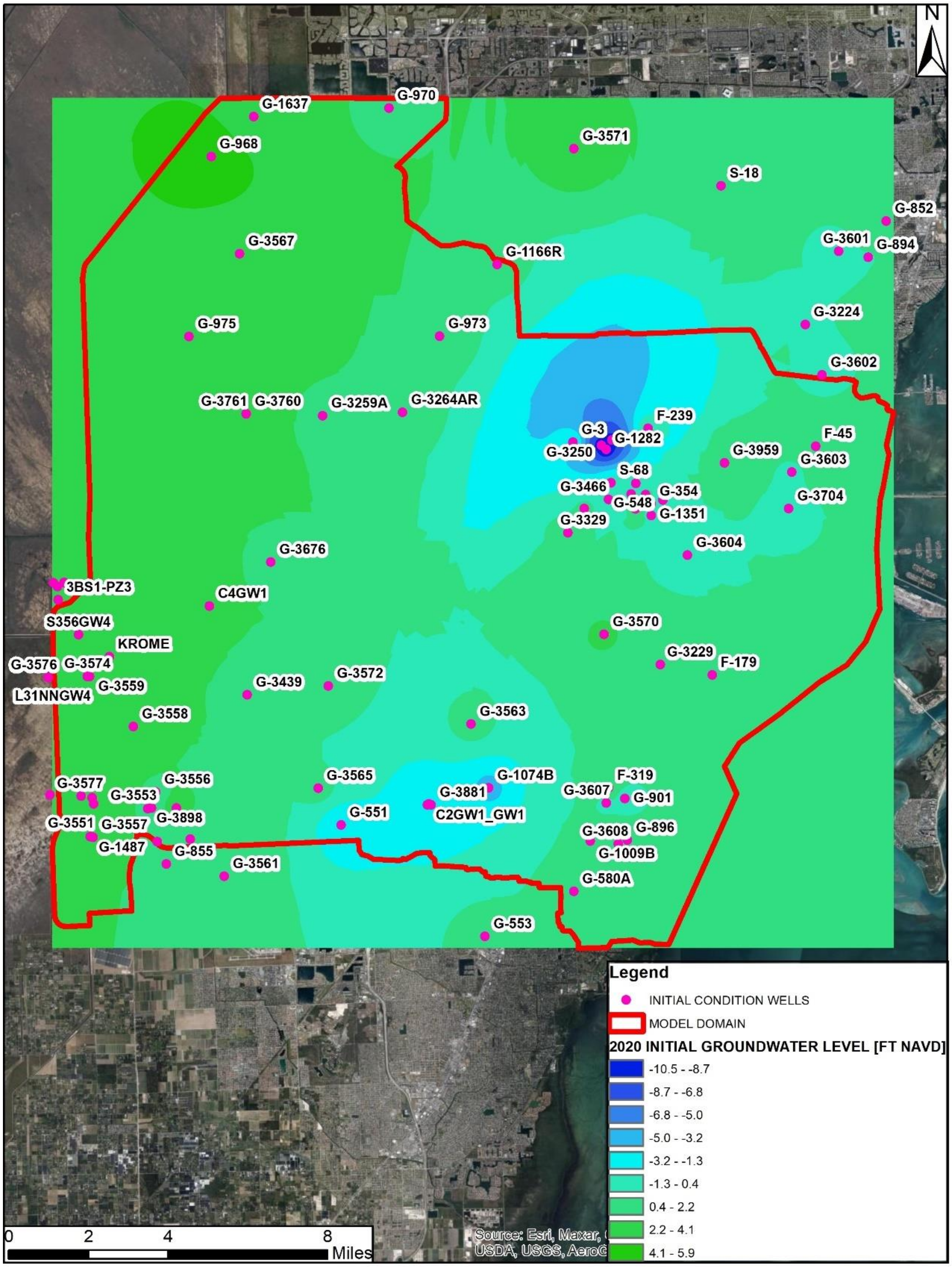


Figure 4-14 – Initial Conditions for the Saturated Zone for the 2020 Simulation



4.10. CANAL GEOMETRY ADJUSTMENTS DURING DEBUGGING

Figure 4-15 illustrates the canal network color coded by the source model for canal geometry data, including the Miami Dade County SWMM models in green and District HEC-RAS models in blue.

For many of the subject canals, the abundance of cross-sections available from the combined Miami Dade County SWMM models and District HEC-RAS models cause the MIKE 1D hydraulic model to be overly complex. Within MIKE 1D the model discretizes the canal reaches into computational segments that include “h-points” where stage is calculated and “q-points” where flow is calculated. When the MIKE SHE and MIKE 1D models are coupled, there can be issues when h-points or q-points are too close together and fall within the same computational grid cell for MIKE SHE. As noted in **Section 4.3**, the computational grid cell size for the model is 125 feet. In consideration of this issue, several cross-sections were selectively removed from the hydraulic network to simplify the simulation and reduce errors. In other locations where there is a significant density of cross-sections but minimal changes in the depth, width, and cross-sectional area some cross-sections were removed to reduce unnecessary complexity in the hydraulic model. **Figure 4-16** illustrates the full model network within MIKE 1D and the distribution of cross-section data within each reach.

4.11. CANAL-AQUIFER INTERACTION

The initial model development was developed by coupling all of the canal reaches to MIKE SHE using the “Aquifer + Bed” parameterization. This assumes that the flow of water across the interface of the canal bed and the aquifer is based on a combination of the following:

- The riverbed conductance specified by the user as a leakance value ($1x10^{-6}$ sec^{-1}), and
- The horizontal hydraulic conductivity specified in Layer 1 of the Saturated Zone.

Based on initial simulations, it appears that this methodology is overly limiting to the conductance of water across the canal-aquifer interface. An alternative approach is the “Aquifer Only” parameterization which utilizes only the horizontal hydraulic conductivity specified in Layer 1 of the Saturated Zone. As is always the case with overland flow, the overland water can interact with the canals at each H-point in the canal network, if water overtops the canal embankment elevation.

During the calibration process, a combination of Aquifer Only and Aquifer + Bed were used, depending on the branch and the connection with groundwater.

Figure 4-15 - Data Sources for Canal Cross-Sections

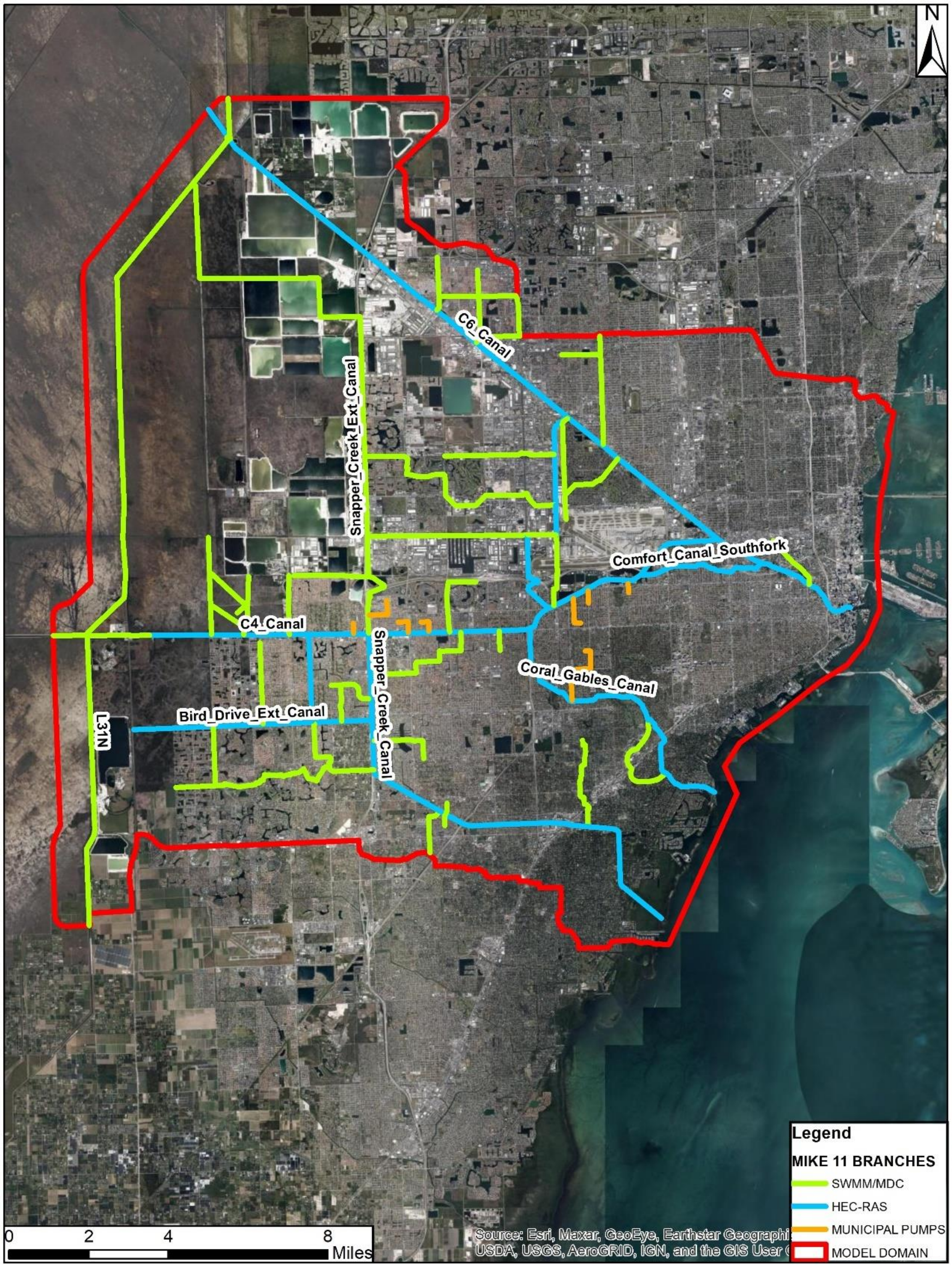
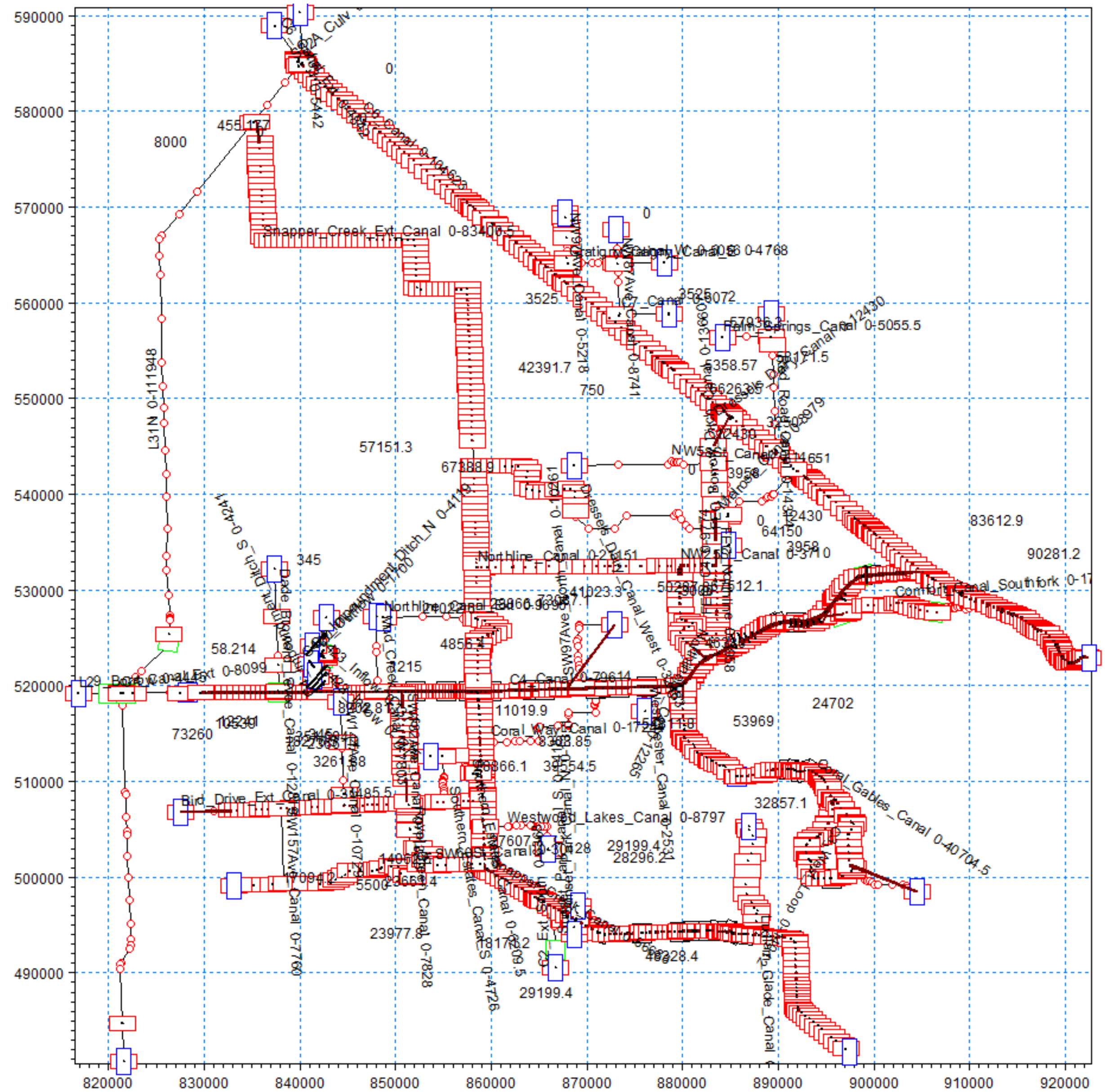


Figure 4-16 – MIKE 1D Hydraulic Network and Initial Cross-Section Locations



4.12. CULVERTS AND BRIDGES

The model originally included 131 culverts and 95 bridges as imported from the HEC-RAS and XPSWMM models. During the debugging process the quality control review included adding or refining the channel markers that define the top of bank, toe of slope, and abutment or pier location. **Figure 4-17** and **Figure 4-18** demonstrates the configuration of an example bridge geometry for the Le Jeune Road crossing of the C6 Canal within MIKE 1D and how the channel markers define key features of the upstream and downstream cross-sections. **Table 4-3** provides a table detailing the location and geometry of each bridge in the model.

During the debugging processes for the simulation of culverts and bridges, some modifications were necessary. In the case where two culverts were inserted onto a single reach without a known cross-section between them, a new cross-section was created based on the closest adjacent data and inserted between the culverts. This approach was used to resolve the requirement to have a cross-section above and below each culvert. In addition, for locations where the culvert invert is below the thalweg of the upstream or downstream cross-section, a small "artificial cut" was created within the upstream and downstream cross-sections to allow the simulation to run.

As a quality control check, a comparison was performed between the top of pipe for inserted culverts and the elevation from the LiDAR at the culvert location. It was assumed that there should be at least 1 foot of pipe cover for construction purposes. The SWMM model data from Miami Dade County included culverts with a wide range of sizes. Each of the SWMM culverts that were equal to or less than 1 foot in diameter were reviewed on a case-by-case basis. In all cases those culverts were determined to be too small to be considered for conveyance in a major flooding event and were likely a modeling artifact that was included to represent "leakage" or "seepage". Based on this finding the small culverts were removed from the model, and no impact on drainage continuity with the existing stormwater facilities were found.

Table 4-3: Modeled Bridge Location and Geometry

Canal	Chainage (ft)	Name	Bridge Bottom (ft-NAVD)	Bridge Top (ft-NAVD)	Length (ft)
C3 West Loop	933.1	Bridge at Blue Road (west)	6.998	12.048	44.00
C3 West Loop	2528.7	Bridge at Pisano Drive	4.668	6.368	50.00
C3 West Loop	4346	Bridge47	2.918	5.768	120.00
C3 West Loop	5879.1	Bridge at Dickinson St	5.248	10.448	15.00
C3 West Loop	7467.7	Bridge B-14 at Ponce de Leon	8.698	11.198	75.00
C3 West Loop	7665.9	Bridge B-15 at Dixie Highway	7.148	11.968	98.00
C3 West Loop	11276.9	Bridge at Maynada	15.048	18.248	33.00
C3 West Loop	13579.9	Bridge B-22 at Granada Blvd.	11.278	17.978	70.00
C4 Canal	22212.15	Bridge at SW 132nd Ave	8.33	10.33	56.00
C4 Canal	25436.2	Bridge at SW 127th Ave	8.42	11.33	100.00
C4 Canal	27789.8	Bridge at 122nd Ave	8.00	11.45	95.00
C4 Canal	29830.8	Bridge at Turnpike SW ramp	8.13	10.12	72.00
C4 Canal	34495.2	Bridge at SW 109th Ave	7.54	8.85	62.00
C4 Canal	35812.7	Bridge at SW 107th Ave	7.47	9.96	80.00
C4 Canal	41085.8	Bridge07	8.34	10.33	75.00
C4 Canal	42439.4	Bridge at SW 94th Ave	7.16	8.45	56.00
C4 Canal	43587.8	Bridge at SW 92nd Ave	8.38	9.45	68.00
C4 Canal	46436.9	Bridge at 87th (Galloway) Ave	7.46	9.45	67.00
C4 Canal	51555.2	Bridge at SR 826	25.46	28.45	145.00
C4 Canal	55408.7	Bridge at Flagler St	7.70	12.95	98.00
C4 Canal	55796.5	Bridge at Milam Dairy Road	8.12	10.15	82.00
C4 Canal	56698.4	Bridge at FEC RR west	6.96	8.45	20.00
C4 Canal	57247.2	Bridge @ FEC Railroad	6.96	8.45	20.00
C4 Canal	61873	Bridge at NW 7th Street	8.98	12.27	80.00
C4 Canal	62322.7	Bridge at Pan American Hospital	7.26	9.20	38.00
C4 Canal	64516	Bridge59	7.46	9.65	100.00
C4 Canal	69687	Bridge at SR 836 - Dolphin Expressway	12.96	14.95	130.00
C4 Canal	71217.6	Bridge at Airport parking lot	11.46	13.45	26.00
C4 Canal	74545.7	Bridge at LeJeune Road	16.32	19.70	25.00
C6 Canal	311.2	Bridge at Krome Ave (177th Ave)	8.46	11.45	40.00
C6 Canal	22678.4	Bridge at 117th Ave	7.37	9.45	50.00
C6 Canal	26197.4	Bridge at 138th Street (from XPSWMM)	9.17	11.45	52.00
C6 Canal	29249.4	Bridge at 127th Street	8.76	10.75	80.00
C6 Canal	33387.2	Bridge at Smith Crossing (from XPSWMM)	5.36	7.35	82.00

Canal	Chainage (ft)	Name	Bridge Bottom (ft-NAVD)	Bridge Top (ft-NAVD)	Length (ft)
C6 Canal	35830.9	Bridge @ 116th Way (from XPSWMM)	8.86	11.85	112.00
C6 Canal	49219.3	Bridge at NW 79th Ave (form XPSWMM)	8.17	11.16	96.00
C6 Canal	57245.4	Bridge 28 @ 12th Ave	7.46	12.45	79.00
C6 Canal	57878.3	Bridge 2A and 2B - Hialeah Express Way	26.76	29.73	106.00
C6 Canal	58182.2	Bridge 3 FEC Railroad	6.43	11.55	19.50
C6 Canal	58290	Bridge 4 Miami Metrorail	35.06	38.45	37.00
C6 Canal	64157.5	Bridge 5 - concrete pedestrian bridge	23.76	27.35	7.00
C6 Canal	68220.3	Swing Bridge 6 at Hook Square	5.83	8.22	25.00
C6 Canal	68374.7	Bridge 7 Hook Square pedestrian bridge	6.96	11.75	17.00
C6 Canal	68594.3	Bridge 8 @ Hook Square (vertical lift)	5.96	7.25	32.00
C6 Canal	71415.6	Bridge 9 @ SE 4th Ave	7.26	13.55	82.00
C6 Canal	75056.4	Bridge at LeJeune Road	7.36	12.25	240.00
C6 Canal	76223.7	Bridge 12 @ NW 36th Ave	8.36	14.45	52.00
C6 Canal	77561.6	Bridge 13 @ SAC Railroad	6.36	10.55	20.00
C6 Canal	88519.9	Bridge 15 @ NW 22nd Ave	26.26	31.25	66.00
C6 Canal	94507.1	Bridge 18 @ NW 12th Ave	15.26	22.05	60.00
C6 Canal	97591.6	Bridge 19 at NW 7th Ave	10.76	14.45	60.00
C6 Canal	100042	Bridge 20 at Flagler St	32.16	43.45	65.00
C6 Canal	100505.6	Bridge 21 SW 1st St	17.60	22.45	60.00
C6 Canal	101520	Bridge 22 at I-95	24.66	32.65	230.00
Coral Gables Canal	5508.2	Bridge at SW 77th Ave (Coral Way)	7.26	9.75	112.00
Coral Gables Canal	6083	Bridge at SR 826 ramp	11.86	14.85	50.00
Coral Gables Canal	6714.6	Bridge at 75th Ave	7.18	8.86	40.00
Coral Gables Canal	9350	Bridge at RR Trestle just west of SW 72nd Ave	8.32	11.91	17.00
Coral Gables Canal	9455.2	Bridge at SW 72nd Ave	9.03	11.75	70.00
Coral Gables Canal	11134.4	Railroad bridge at 69th Ave	13.86	15.54	40.00
Coral Gables Canal	18366.3	B-8 bridge at SW 57th Ave just DS of G93	7.59	11.48	43.00

Canal	Chainage (ft)	Name	Bridge Bottom (ft-NAVD)	Bridge Top (ft-NAVD)	Length (ft)
Coral Gables Canal	19056.2	Bridge at Alhambra Circle	7.44	8.93	30.00
Coral Gables Canal	19628.1	Bridge at Biltmore Golf course Site BB	8.57	11.47	18.00
Coral Gables Canal	20064.5	Bridge at Biltmore Golf course Site BE	4.56	6.05	8.00
Coral Gables Canal	20972.6	Bridge at Biltmore Golf Course wooden bridge at B-10	4.76	5.95	8.00
Coral Gables Canal	22075.6	Bridge at Biltmore Golf Course wooden golf cart bridge at BC	5.69	6.95	8.00
Coral Gables Canal	22167.6	Bridge at Biltmore Golf course Site BB	8.39	11.25	18.00
Coral Gables Canal	22411.6	Golf cart bridge at Biltmore golf course Site BA	4.96	6.14	8.00
Coral Gables Canal	23150.3	Bridge at Bird Road	7.51	11.54	48.00
Coral Gables Canal	25124	Bridge B-22 at Granada Blvd.	8.48	11.45	32.00
Coral Gables Canal	27149.5	Bridge at Blue Road (east)	7.64	11.43	40.00
Coral Gables Canal	28011.8	Bridge B-14 at Ponce de Leon	8.71	11.20	75.00
Coral Gables Canal	28221.2	Bridge B-15 at Dixie Highway	7.16	11.97	98.00
Coral Gables Canal	32257	Bridge B-23 at Hardee Rd	11.46	17.15	40.00
Snapper Creek Canal	693.5	Turnpike entrance from SW 8th Street	12.16	16.18	75.00
Snapper Creek Canal	1358	Bridge at Turnpike north bound exit to SW 9th Street	7.26	10.76	40.00
Snapper Creek Canal	9581.3	Bridge at north bound entrance to Turnpike	9.57	13.93	66.00
Snapper Creek Canal	10666.4	Bridge at SW 40th Ave (Bird Drive)	5.57	7.46	107.00
Snapper Creek Canal	16889.5	Bridge B-22 at SW 56th Ave (Miller Drive)	8.37	12.06	106.00
Snapper Creek Canal	18595.2	Bridge aB-28A at SW 117th Ave	8.96	12.86	97.00
Snapper Creek Canal	24805.6	Bridge B-31 at SW 107th Ave	8.06	12.26	90.00
Snapper Creek Canal	25265.6	Bridge at SW 72nd (Sunset) Ave	7.08	10.13	104.00

Canal	Chainage (ft)	Name	Bridge Bottom (ft-NAVD)	Bridge Top (ft-NAVD)	Length (ft)
Snapper Creek Canal	29170.9	Bridge B-33 at SW 99th Ave	8.06	10.21	37.00
Snapper Creek Canal	31817.6	Bridge B-36A R/R west of SR 874 Express Way	6.43	9.83	13.00
Snapper Creek Canal	31977.1	Bridge at SR 874	10.90	13.90	200.00
Snapper Creek Canal	36484.6	Bridge B-41 @ SW87 Ave	6.96	8.68	82.00
Snapper Creek Canal	40504.5	Bridge B-42 at SW 79th (Kings Creek) Ave	6.68	8.66	40.00
Snapper Creek Canal	41800	Bridge at SW 77th Ave	6.07	10.56	36.00
Snapper Creek Canal	42000	Bridge at Palmetto Express Way + Ramp (combined)	7.10	13.76	200.00
Snapper Creek Canal	43369.5	Foot bridge B-43B behind Dadeland Mall	7.46	8.11	7.00
Snapper Creek Canal	44556.4	Bridge at SW 72nd Ave	7.51	12.76	75.00
Snapper Creek Canal	45403.2	Bridge at SW 70th Ave	8.29	13.60	100.00
Snapper Creek Canal	45938.1	Bridge at US1 + Metro Rail (combined)	9.11	11.96	196.00
Snapper Creek Canal	47257.6	Bridge at SW 67th (Ludlam) Ave	8.78	13.08	54.00
Snapper Creek Canal	52730.2	Bridge at SW 88th Street and Old Cutler Rd	4.16	8.89	70.00

Figure 4-17 – Example of Bridge Geometry for the Le Jeune Road Bridge

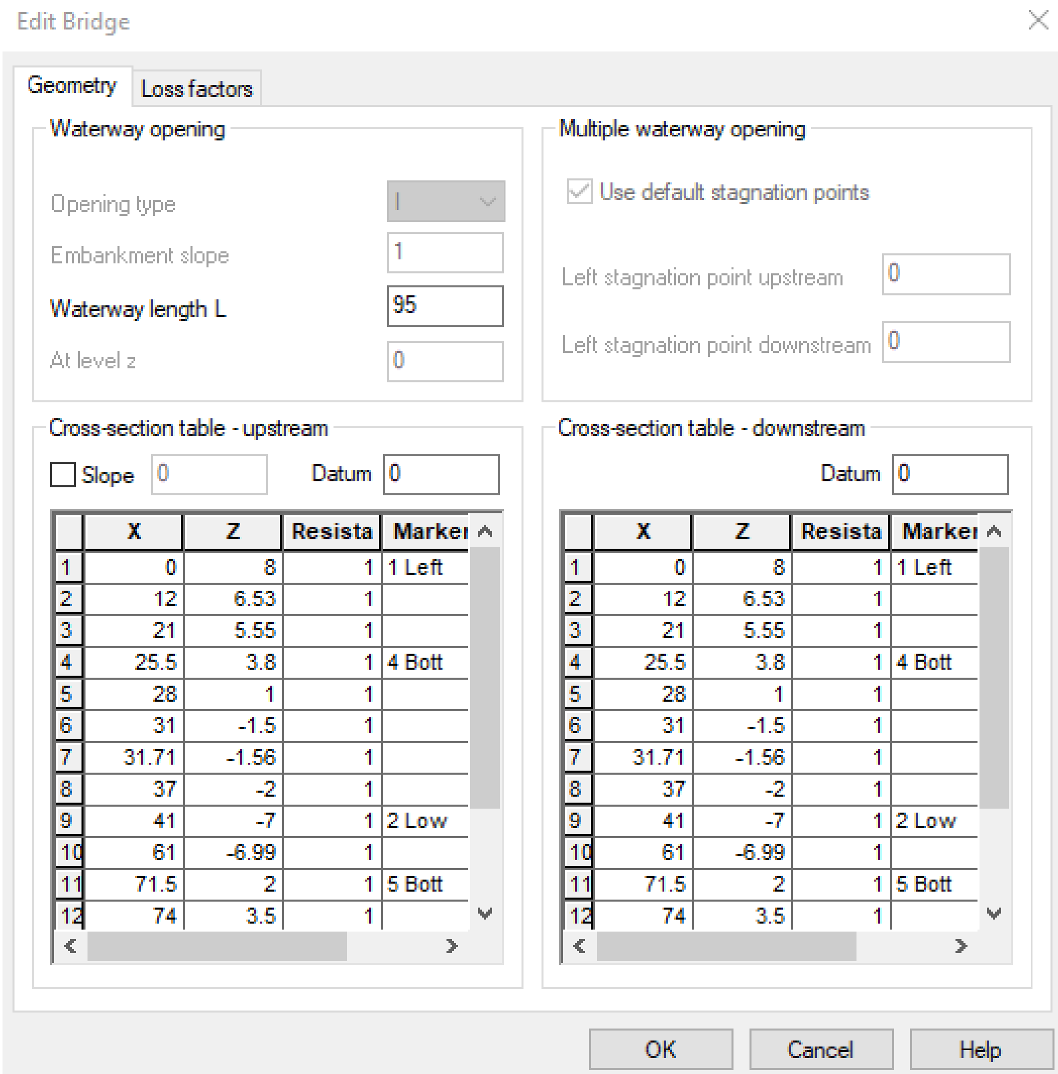
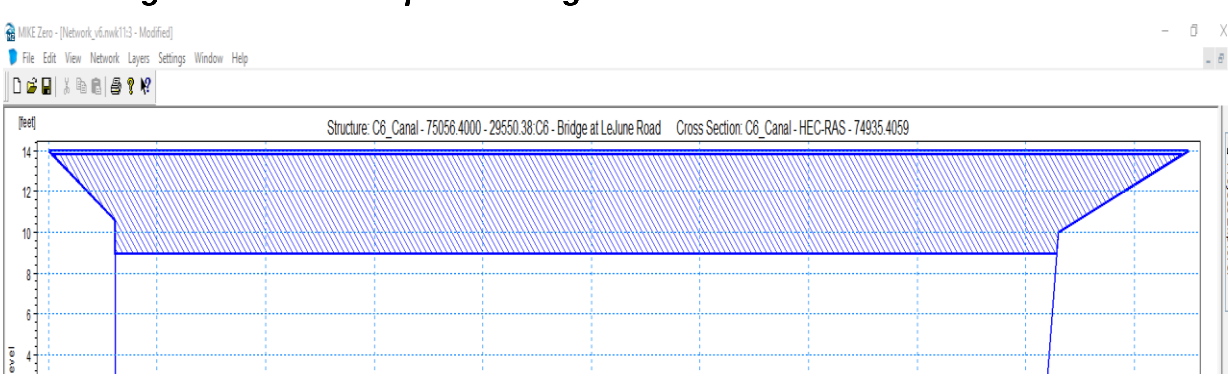


Figure 4-18 – Example of Bridge Cross Section for Le Jeune Road



4.13. PRIMARY NETWORK WATER CONTROL STRUCTURES

All District owned and operated water control structures were included in the model utilizing the geometry defined in the Atlas documenting the configuration of each of the primary structures in North and Central Miami Dade County (SFWMD, 2016).

Within MIKE 1D, there is a separate parameterization for each structure gate to accommodate for situations where individual gates are operated independently at the same structure. For gated culverts the model parameterization is set as an “Underflow” structure type that utilizes fixed head loss factors of 0.5 for inflow and 1.0 for outflow. For spillways, the model parameterization is set as a “Sluice Gate” structure type which utilizes different coefficients based on flow regimes as described in the Atlas as well as Rakib and Zeng (2017).

4.13.1. CALIBRATION AND VALIDATION OPERATIONS

For the calibration and validation simulations, these structures utilize measured data to define the operations. For gated spillways and gated culverts, the measured data is a time-series defining the measurement of the gate opening at an irregular time-interval based on when the gates move. For pump stations, the measured data is a time-series of discharges based on total static head and rating curves derived by the District (Imru and Wang, 2014).

4.13.2. PRODUCTION SIMULATION OPERATIONS

For the production runs, when design storms were utilized instead of actual events, the District operational rules as defined in the Atlas and the Miami Dade County Flood Mitigation Program, C-4 Basin Operating Plan (SFWMD, 2010) were utilized to develop a simulated representation utilizing the “Logical Operands” parameterization scheme within the MIKE framework. This approach allows for gate opening of simulated culverts and spillways and discharge amounts at pump stations to vary based on head conditions at specified locations within the canal network.

In particular, the Operating Plan identifies condition specific operations of the S-25, S-25A, S-25B, G-93 and S-22 structures based on measured data at the T5, C4.CORAL, S-25B, G-420, G-422 and MRMS1 stage monitoring locations. As an example of how these operations are applied, when the stage at T5 is measured above an elevation of 2.43 ft-NAVD, or 4.0 ft-NGVD, and discharges at S-25B are limited to below 600 CFS due to tidal conditions, the S-25B forward pumps will be engaged. Whereas to prevent adverse conditions downstream, the forward pumping at S-25B and S-26 will stop when the stages at MRMS1 exceed 3.18 ft-NAVD, or 4.75 ft-NGVD. The Operating Plan also includes specific operational protocol for the C-4 Emergency Detention Basin. Each of these operating rules were added to the MIKE framework’s “Logical Operands” for the production runs so that the design storm and sea level rise scenarios accurately reflect the anticipated operations of the flood protection system.

4.14. SECONDARY NETWORK WATER CONTROL STRUCTURES

Within the C3 and C4 watersheds, there are seventeen (17) municipal pump stations that have been constructed to contribute to address local flooding issues. **Figure 4-19** illustrates the location of each station.

Within the MIKE SHE / MIKE 1D model, the effects of the pump stations were modeled using a distinct approach. First, the drain codes within MIKE 1D were modified to ensure the spatial limits of the stormwater pipe network upstream of each station are defined as the contributing area. In some cases, pump stations that are proximate to each other are combined for modeling purposes with a single contributing area. As an example, Belen PS1 and Belen PS2 are modeled as separate locations with the community of Belen divided into two contributing areas, while Sweetwater B15 and B16 are combined to serve the area north of West Flagler Street and west of NW 107th Avenue. A similar approach was utilized for each location to determine the appropriate contributing area for each pump station, with record drawings from ERP exhibits utilized to define the contributing areas. **Figure 4-20** illustrates the initial drain code setup in MIKE SHE, with the pump station service areas being delineated as separate basins.

Once the contributing area is defined for each pump station, the hydraulics of the facility have to be represented in the model. Each pump station is constructed in an urbanized area where the upstream control volume that the pump draws on consists of a wet well and a network of stormwater gravity pipes that are flowing full. This type of infrastructure is very small scale in comparison to the FPLOS model and is not appropriate to include in the MIKE 1D network. Therefore a “proxy” reach was created to act as a stilling basin for each pump station to draw water from before discharging. All overland flow from the upstream contributing area is directed to the proxy reach where it can be discharged by the pump into the appropriate receiving canal. The initial geometry of each proxy reach is estimated based on best professional judgement, however, during the calibration process the size of the proxy reaches were adjusted to correctly reflect the service area of the pump station(s). The discharge location for each station was applied to the respective canal at the point noted in as-built drawings and municipal records. **Table 4-4** describes the pump capacities and operational protocols for each pump station as well as the approximate location of the outfall. Noticeably some pump stations share an outfall location as the downstream force mains can be combined in a manifold to reduce construction and maintenance costs.

Table 4-4 – Municipal Pump Locations and Operations

City	Pump Station Name	Pump Capacity [CFS]	T5W Off Elev. [FT-NAVD]	Wet Well Off Elev. [FT-NAVD]	Wet Well On Elev. [FT-NAVD]	Pump Outfall Location
Sweetwater	B1	20	3.43	-6.57	1.43	SW 102nd Ave at C4
	B2	20	3.43	-6.57	1.43	
	IIA #3	27	3.43	-5.57	-1.57	SW 108th Ave at C4
	IIA #4	27	3.43	-5.57	-1.57	
	IIB #1	27	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS	SW 110th Ave at C4
	IIB #2	27	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS	
	B15	25.4	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS	NW 4th Terr at C2
	B16	25.4	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS	
Belen	PS1	100	3.43	-7.32	2.95ft: 25CFS 3.00ft: 100CFS	SW 127th Ave at C4
	PS2	100	3.43	-7.32	2.95ft: 25CFS 3.00ft: 100CFS	SW 120th Ave at C4
West Miami	#1	90	3.43	-9.57	1.43	SW 64th Ave at C4
	#2	100	3.43	-9.57	1.43	SW 64th Ave at C3
City of Miami	#1	30	3.43	-8.57	1.43	SW 67th Ave at C4
	#2	60	3.43	-8.57	1.44	NW 65th Ave at C4
	#3	40	3.43	-8.57	1.44	SW 64th Ave at C4
	#4	54	3.43	-8.57	1.44	NW 63rd Ave at C4
	#5	15	3.43	-8.56	1.45	NW 7th St near NW 52nd Ave at C4

Figure 4-19 – Locations of Municipal Pump Stations

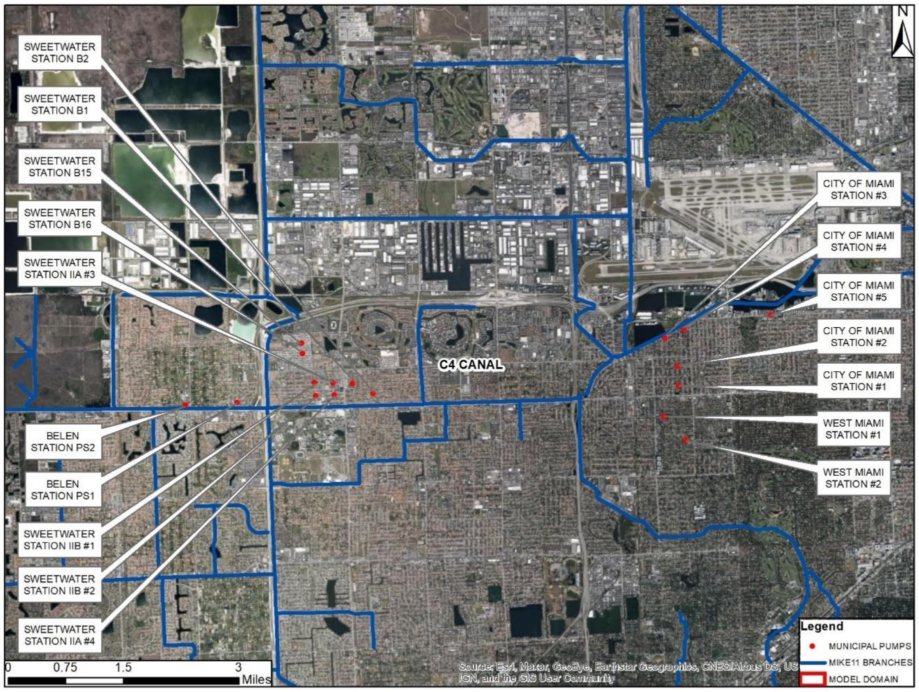
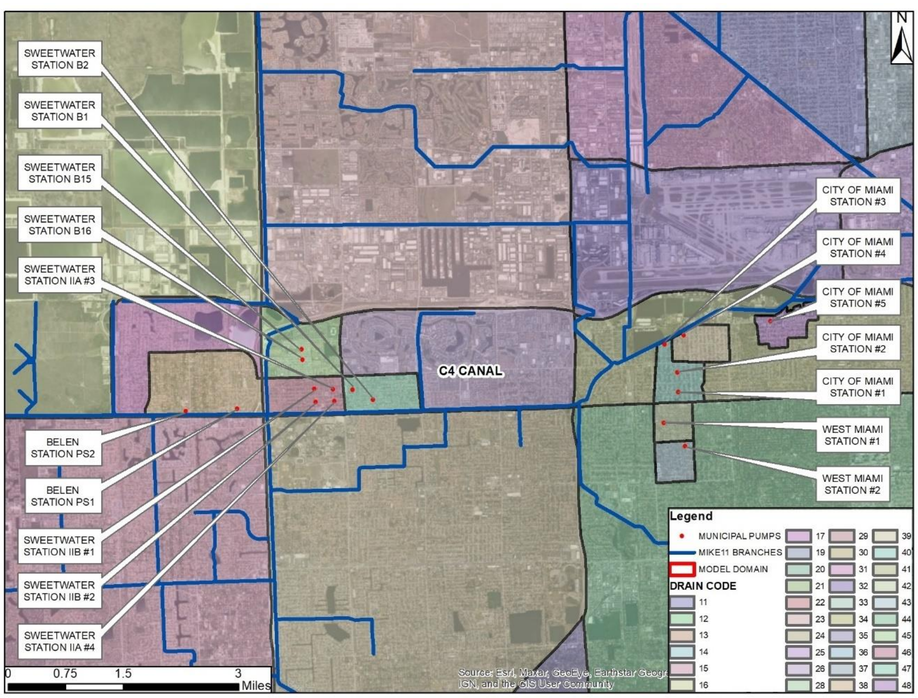


Figure 4-20 – Drain Code Delineation of Contributing Areas for Municipal Pump Stations



5. MODEL CALIBRATION AND VALIDATION

5.1. INITIAL CALIBRATION

After the completion of the model development phase, a calibration simulation was developed for the model domain for the period from May 10, 2020, through June 1, 2020. The calibration simulation results were compared to measured data at multiple monitoring locations where surface water and groundwater measurements were performed. **Figure 5-1** through **Figure 5-5** illustrate the calibration points that will be discussed in the following sections. For clarity purposed the calibration points were separated by watershed.

Over the three-week simulation period the measured total rainfall at gage locations was 19.9 inches, with a 72-hour maximum of 15 inches and a 24-hour maximum of 7 inches. The initial results of the un-calibrated model were compared to measured data and presented to District staff. There were several discrepancies between the simulated and measured stages and flows noted that were discussed. Some of the key concerns included:

- Simulated stages that were lower than measured at all surface water monitoring locations,
- Simulated flows at coastal structures that did not align with expected flows based on simulated stages,
- Unevenly distributed saturated zone (SZ) flow across the domain boundary,
- Inability to physically represent depressional storage in western mining pits, and
- Simulated groundwater elevations that were higher than measured at locations adjacent to public water supply wells.

Although there are modeling results at every monitoring point, only a subset of the results from representative locations are shown in this section. **Figure 5-6** through **Figure 5-15** demonstrate the issues described above as they are reflected in the headwater stage and the flow at S22, stage at T5W and groundwater elevations at G-3. All simulated and measured stages are referenced to the North American Vertical Datum of 1988 (NAVD).

Figure 5-1 – Calibration Stations – C2 Watershed

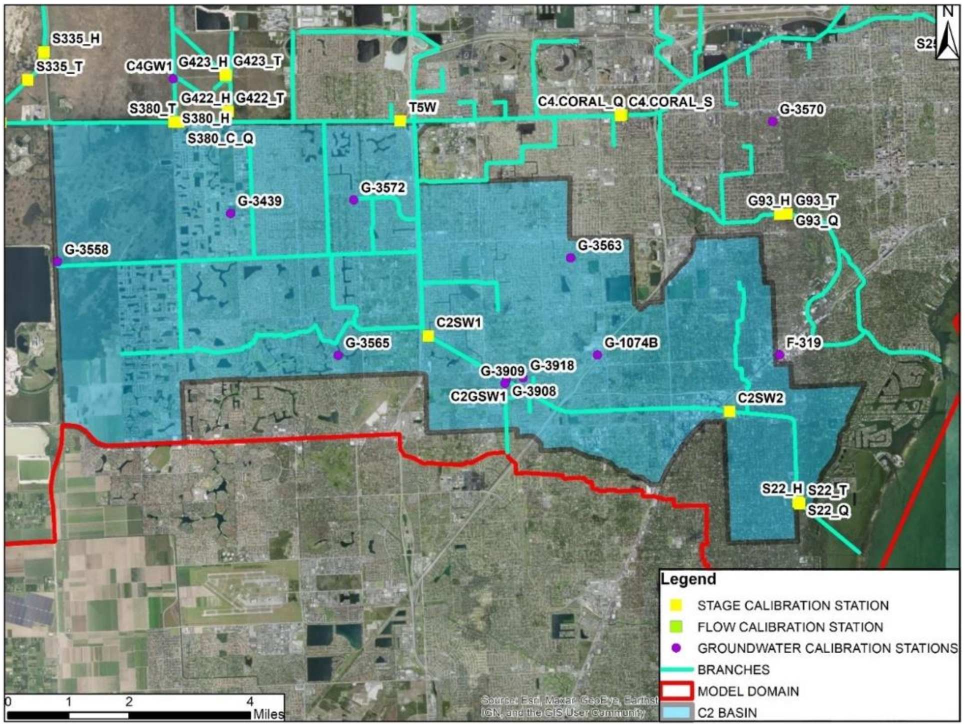


Figure 5-2 – Calibration Stations – C3W Watershed

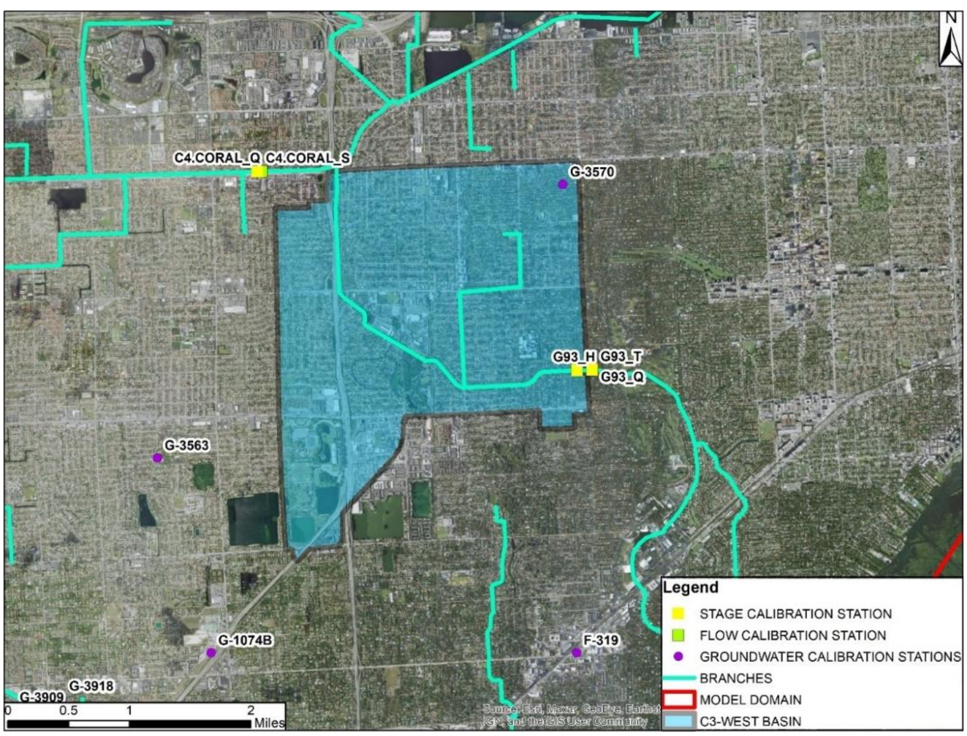


Figure 5-3 – Calibration Stations – C4 Watershed

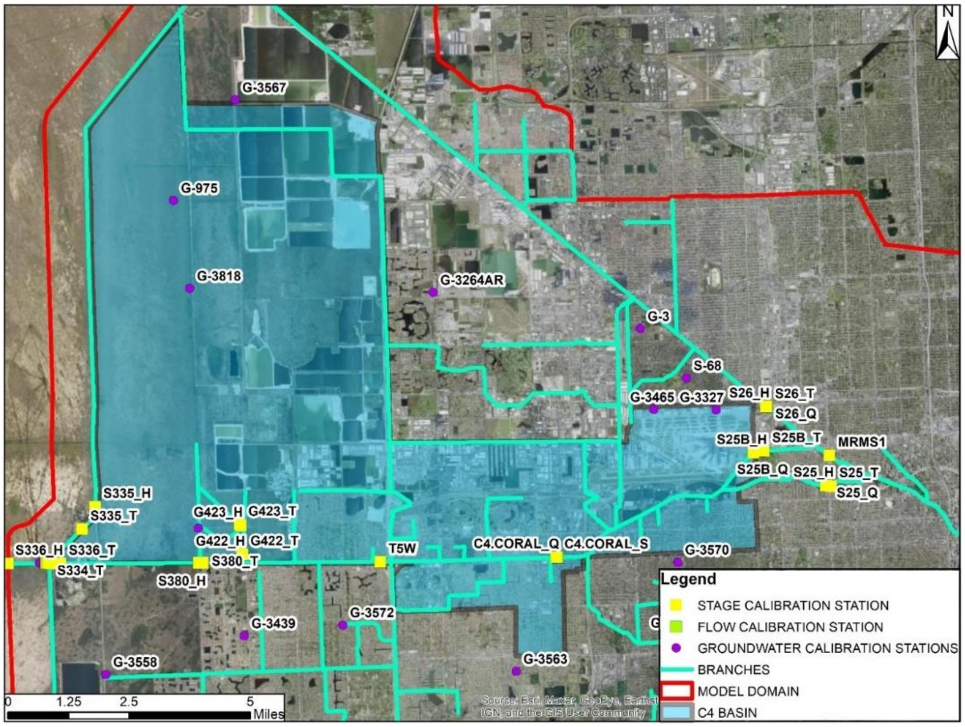


Figure 5-4 – Calibration Stations – C5 Watershed

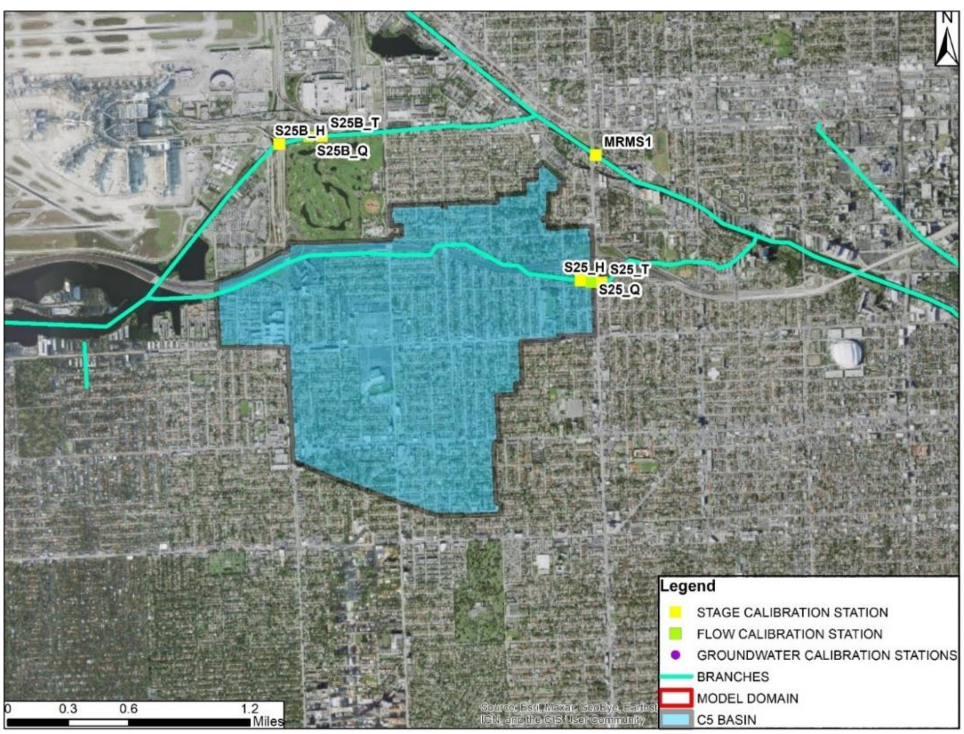


Figure 5-5 – Calibration Stations – C6 Watershed

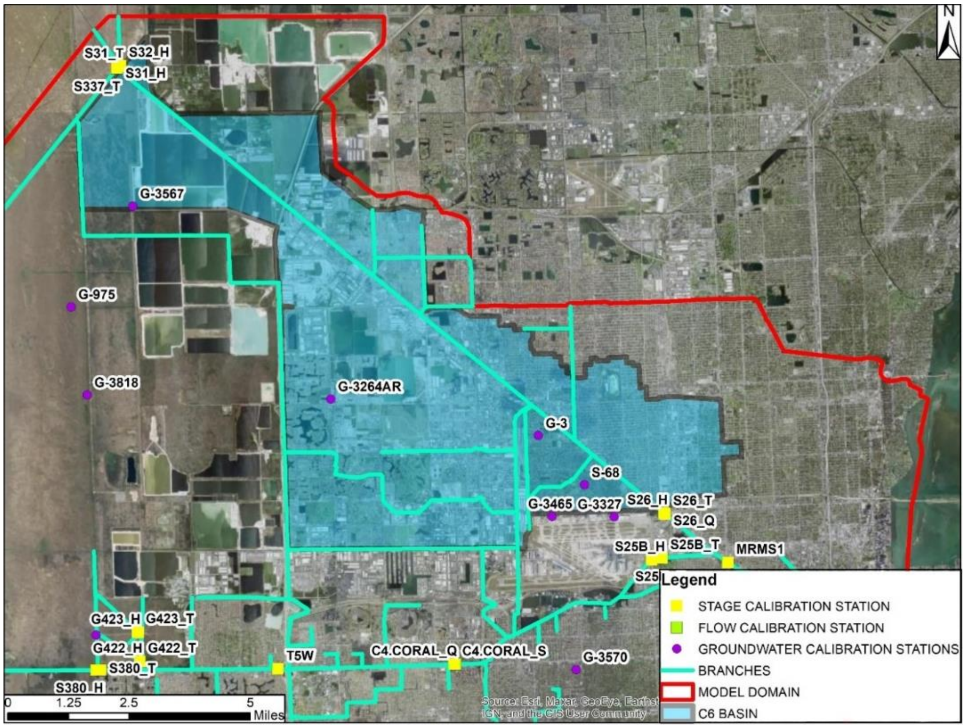


Figure 5-6 – Measured and Initial Simulated Stage at the S-22 Tidal Spillway Structure

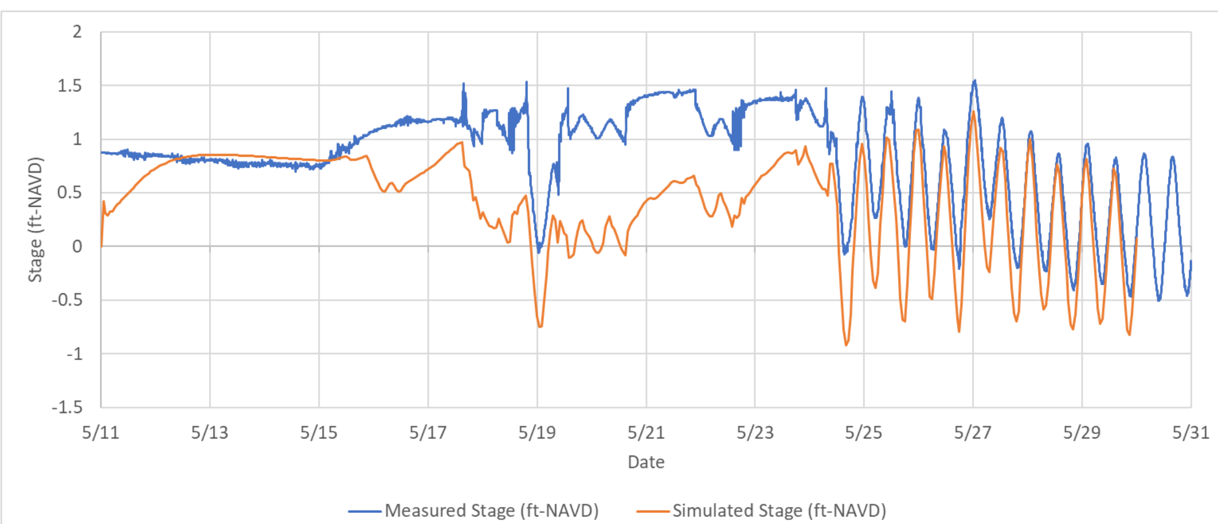


Figure 5-7 – Measured and Initial Simulated Headwater Flow at the S-22 Tidal Spillway Structure

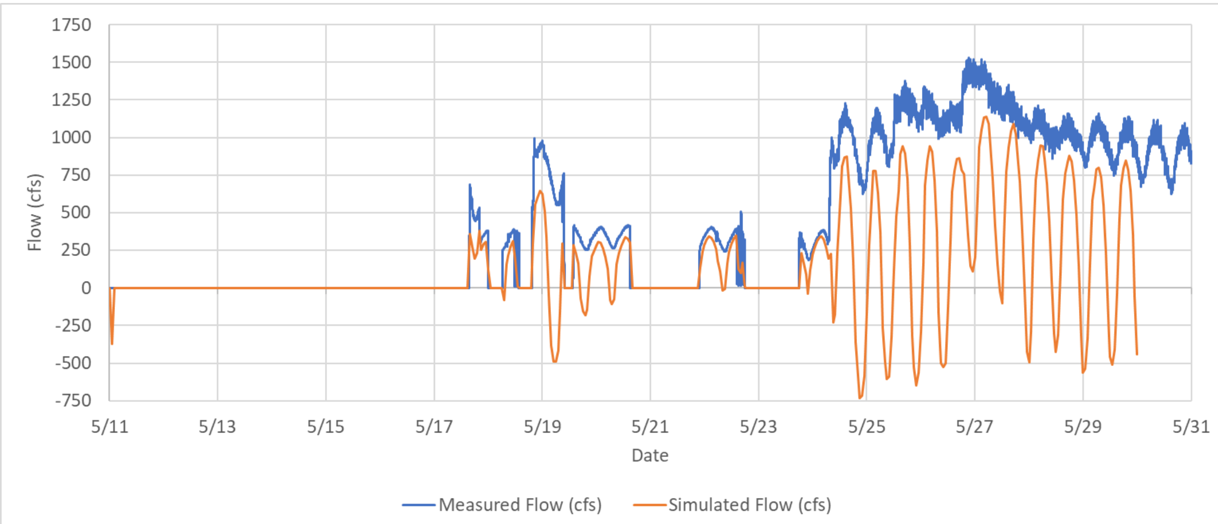


Figure 5-8 – Measured and Initial Simulated Stage at T5W (West of Turnpike on C4 Canal)

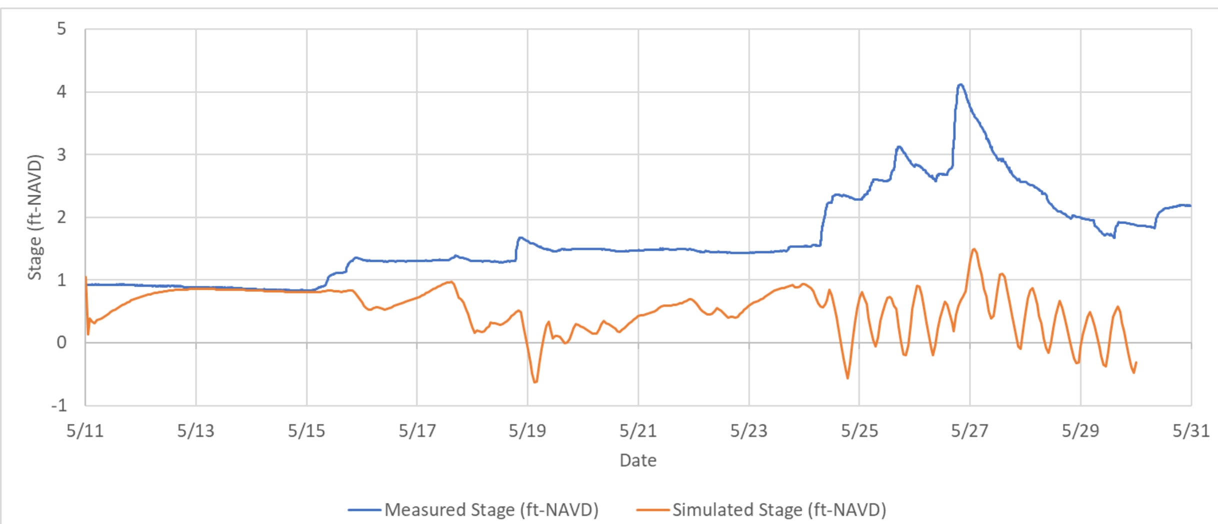


Figure 5-9 – Measured and Initial Simulated Groundwater Elevation at G-3

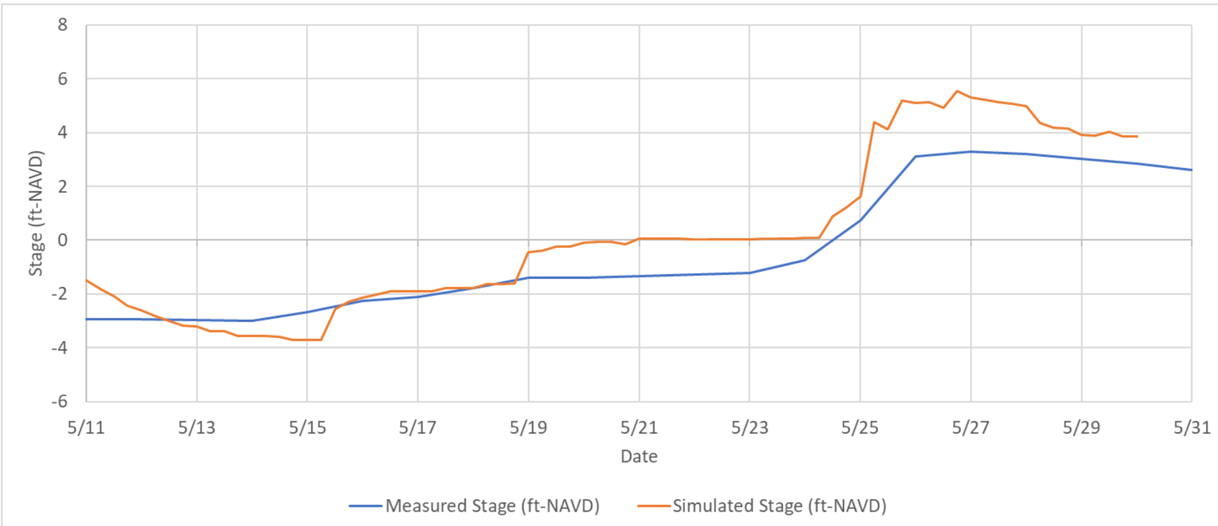
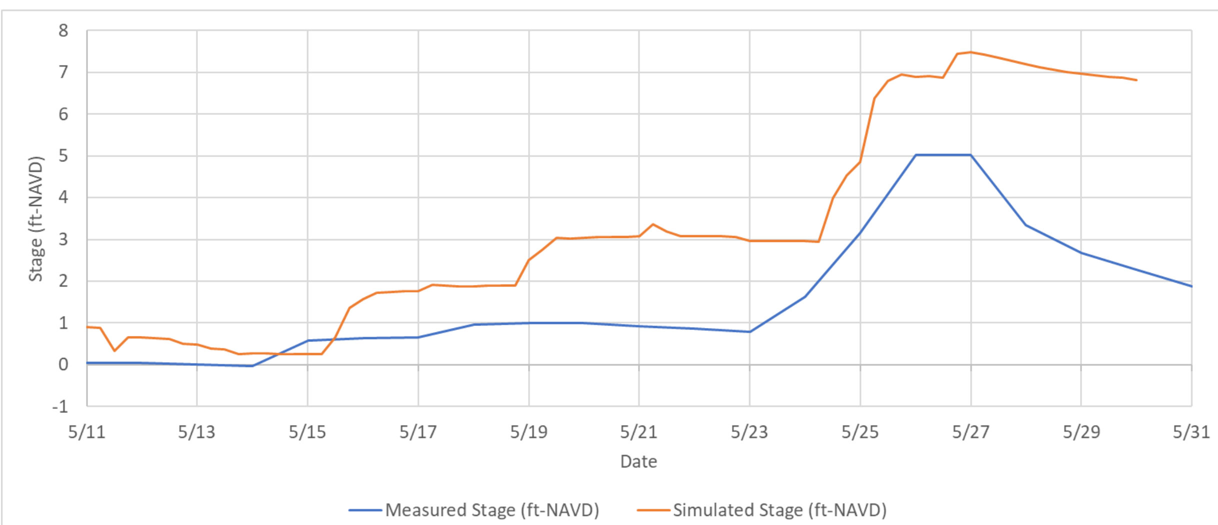


Figure 5-10 – Measured and Initial Simulated Groundwater Elevation at G-3465



The statistical performance of the initial calibration model over the 3-week simulation period is shown in **Table 5-1**, with the following statistical parameters: Mean Error (ME), Mean Absolute Error (MAE), Root Mean Squared Error (RMSE), Correlation (R), and Nash-Sutcliffe Efficiency (NSE). The ME is calculated as the average of the differences between the simulated value and the measured value. The MAE is calculated as the average of the absolute value of the difference between the simulated value and the measured value. The RMSE is calculated as the square-root of the average of the difference of the simulated and the measured values squared, The R value uses the Pearson correlation equation to determine correlation between the simulated and measured values. The NSE determines the accuracy of the simulated values versus the mean of the measured values. Negative NSE values demonstrate that the mean of the measured value is a better predictor of the measured values as compared with the simulated values. For all statistical performance metrics, the value used for comparison is the simulated value that is closest in time to the measured value. For stage values, the calibration target is 0.5-feet for ME, MAE, and RMSE. For flow values, a calibration target of ±20% was chosen for ME, MAE, and RMSE. Values that are not within the calibration target range are highlighted in light orange below.

Table 5-1 – Statistical Performance of Initial Calibration

STRUCTURE	ME	MAE	RMSE	R	NSE
S-22-H	0.49	0.50	0.59	0.33	0.74
S-25-H	0.16	0.41	0.52	0.50	0.60
S-25B-H	1.00	1.01	1.22	0.70	0.33
S-26-H	0.49	0.49	0.55	0.26	0.89
S-31-T	-3.90	3.90	3.91	0.35	0.94
S-336-T	0.46	0.46	0.48	0.14	0.99
S-380-T	2.36	2.37	3.09	1.99	-0.64
G93-H	0.50	0.51	0.63	0.38	0.74
C2SW1	0.97	0.99	1.24	-0.14	-2.84
C2SW2	0.66	0.68	0.77	0.44	-4.60
C4.CORAL	1.11	1.11	1.26	0.44	-4.60
T5W	1.19	1.19	1.53	0.97	-0.22

5.1.1. INITIAL MODEL REVISIONS

In response to these concerns, a series of initial modifications were made to the model set-up and parameterization of the calibration simulation. The subsections below describe these modifications and illustrate the results of the changes to the model.

5.1.1.1. GROUNDWATER – SURFACE WATER INTERACTION

To address the discrepancy between the simulated and measured stages in the canals, the first modification was made to the parameterization of the interface between groundwater and surface water to ensure that sufficient seepage from the groundwater was reaching each canal. The initial parameterization within the calibration model assumed that the flow from the SZ into each canal reach was dependent on the combined conductivity of the aquifer and the soil matrix that lines the bank and bed of the channel. Since the groundwater elevations were initially too high and the canal stages were too low, it was assumed that there should be more flow from the SZ into the canals. In addition, the site investigations throughout the model area demonstrate that there is minimal soil matrix along the bank or bed of conveyance canals, as the canal banks are largely exposed Oolitic Limestone. Based on these observations the model was changed such that the only driver of conductivity between the SZ and a canal reach is the aquifer and there is no assumed resistance from the channel bed. The effect of utilizing "aquifer only" to define the conductance between the SZ and canal reaches varied from significant in some portions of the model domain (**Figure 5-11** and **Figure 5-13**), moderate in other locations (**Figure 5-12** and **Figure 5-14**), and negligible in some locations (**Figure 5-15**).

Figure 5-11 – Measured and Initial Simulated Headwater Stage at the S-22 for "Aquifer Only"

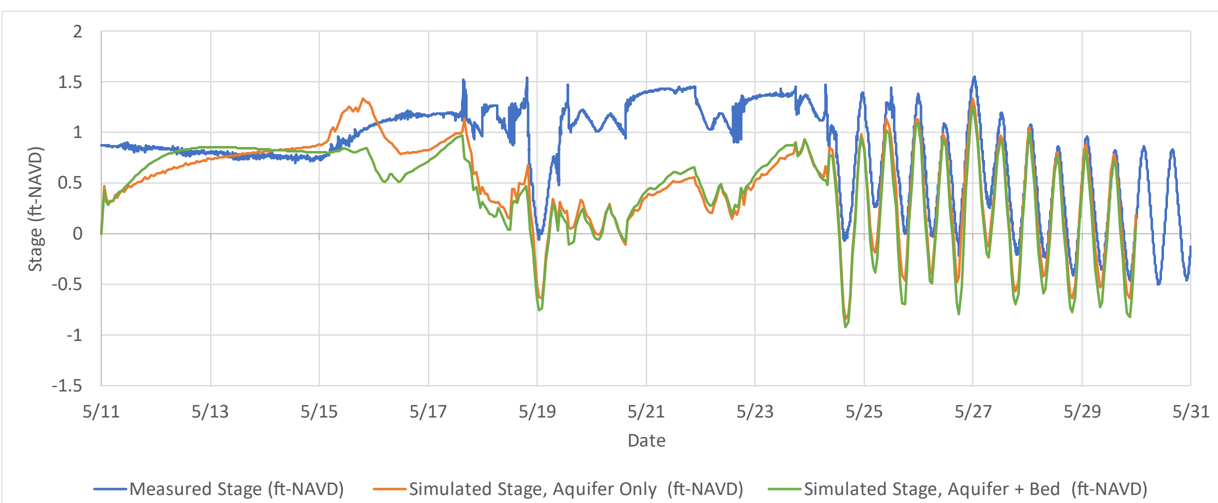


Figure 5-12 – Measured and Initial Simulated Flow at the S-22 for "Aquifer Only"

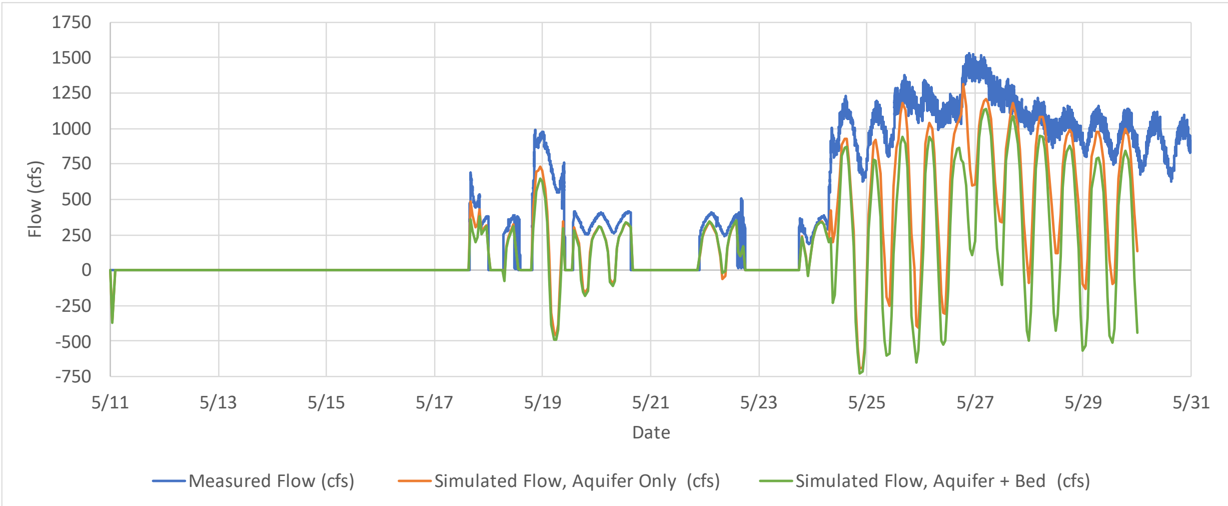


Figure 5-13 – Measured and Initial Simulated Stage at T5W for "Aquifer Only"

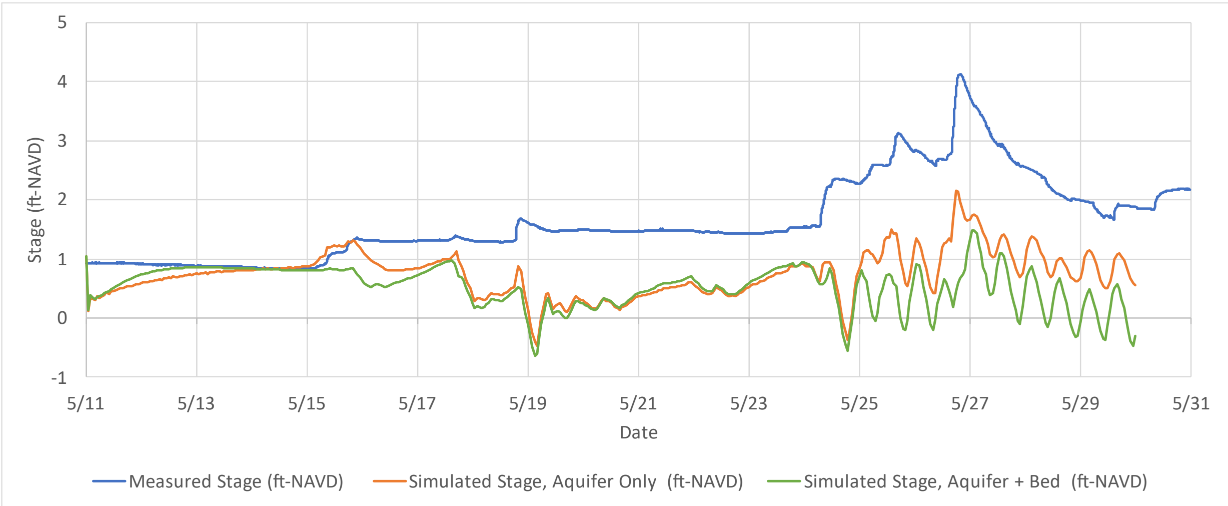


Figure 5-14 – Measured and Initial Simulated Groundwater Elevation at G-3 for "Aquifer Only"

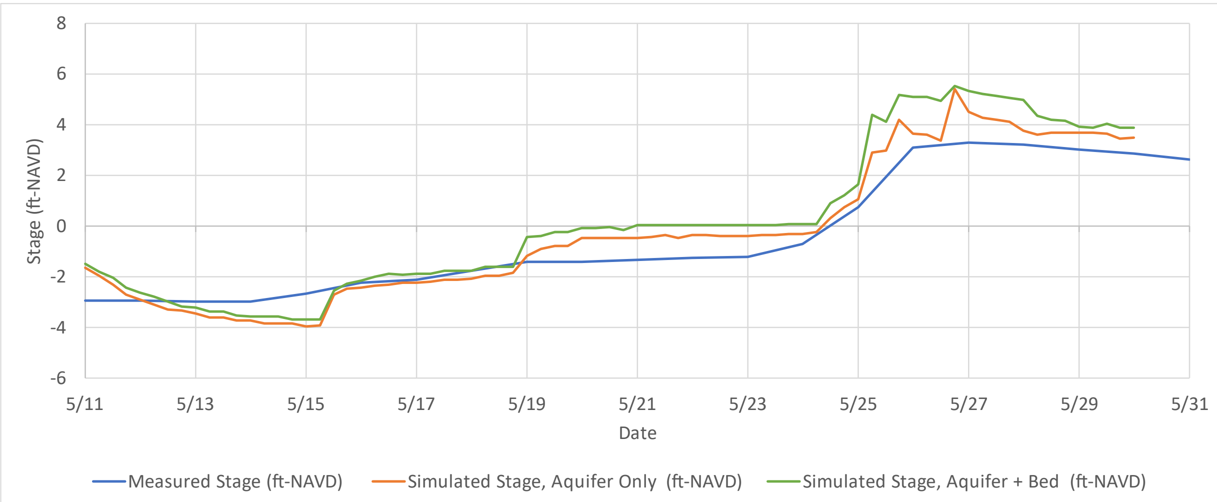
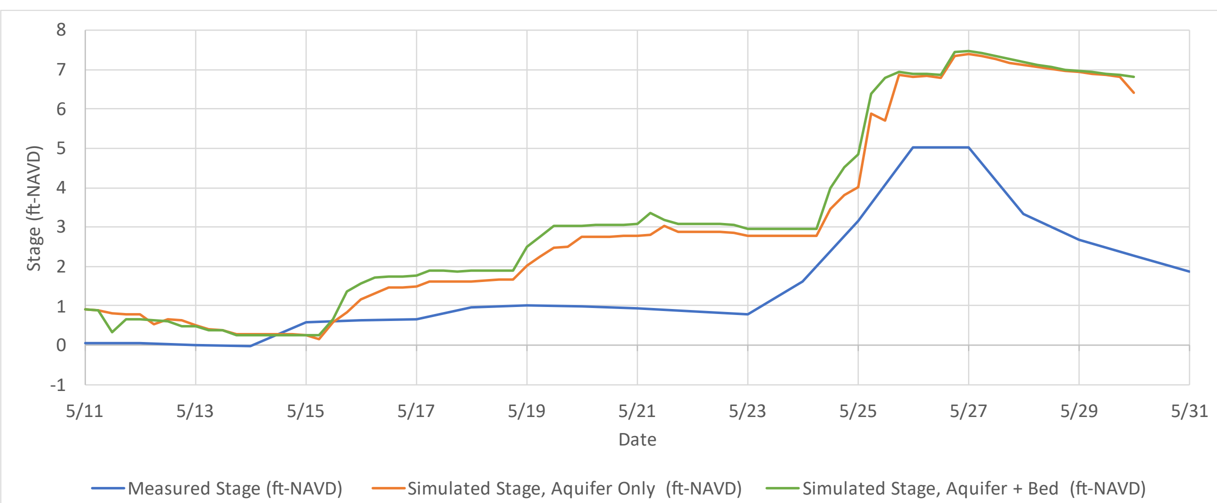


Figure 5-15 – Measured and Initial Simulated Groundwater Elevation at G-3456 for "Aquifer Only"



5.1.1.2. MINIMUM MANNING'S "N" FOR CONVEYANCE CANALS

Another modification made to address the discrepancy between measured and simulated stages was to provide a uniform value for Manning's n of 0.033 as opposed to the original parameterization which included some Manning's n values as low as 0.020 based on other model calibration efforts (SFWMD, 2010). For unlined canals, such as the ones included in the model domain, a Manning's n value of 0.020 is too low and does not accurately represent the interaction between the canal bottom and the flow of water. There were minimal impacts on simulated stages and flows, so in consideration of standard conventions the revised parameterization for conveyance features was a constant value of 0.033 throughout the canal network.

5.1.1.3. SATURATED ZONE BOUNDARY CONDITION

For the initial model configuration, the SZ boundary at the model domain was defined based on the closest available groundwater monitoring well. The domain perimeter was divided into a series of "reaches" that were closest to a monitoring location and all grid cells along that reach were assigned the measured elevation from that point. After reviewing the initial calibration simulation, the flow across the domain boundary for the SZ had significant discontinuities at locations where the boundary reference switched from one monitoring location to another. To resolve these discontinuities, a new input dataset was generated providing a linear interpolation of the boundary values along the SZ domain perimeter. **Figure 5-16** illustrates the updated SZ boundary conditions that utilize interpolated values.

5.1.1.4. REPRESENTING MINING FACILITIES

The initial configuration of the SZ model for the calibration simulation was based on the USGS MODFLOW model of Miami Dade County (Hughes and White, 2016). Because this model did not have a surface water component, the mining pits in the western portion of the domain were represented with areas of high horizontal and vertical conductivity values and no surficial storage. The transition to a fully integrated surface water-groundwater model necessitated a change in these assumptions. During the initial calibration effort, the high conductivity values found within the limits of each mining pit were removed and replaced with interpolated values from adjacent cells. After revising the conductivity values in the SZ model, reaches were developed for each mining pit that were disconnected from the canal network. Cross-sections were defined for the new reaches that extended to the depth of typical mining operations (-24 feet) and the initial water level was set equal to the interpolated initial SZ level at each location. This technique allows for the storage that is available within each mining pit to be represented within the model. The conductance between the mining pit reach and SZ is defined by the conductivity of the aquifer only. **Figure 5-17** illustrates initial parameterization of typical mining pits in the western portion of the model domain, while **Figure 5-18** illustrates the revised approach for typical mining pits in the model. Of note, there are areas of higher conductivity illustrated that were verified in the USGS documentation and likely based on findings from hydrogeologic field investigations.

5.1.1.5. PUBLIC WATER SUPPLY GROUNDWATER WITHDRAWALS

A key model input that was not available during the initial model development was public water supply withdrawal data. During the initial calibration effort, the Miami Dade Water and Sewer Department (MD WASD) provided daily pumping rates in millions of gallons during the calibration and validation periods for 91 individual wells within the following wellfields:

1. Northwest Wellfield
2. Miami Springs Upper Wellfield
3. Miami Springs Lower Wellfield
4. Preston Wellfield
5. Hialeah Wellfield
6. West Wellfield
7. Southwest Wellfield
8. Snapper Creek Wellfield
9. Alexander Orr Wellfield

These public water supply wells were added to the model using the location and depth data available from consumptive use permit documentation for MD WASD. The pumping rates were then added to the model to represent withdrawals from Layer 1 and Layer 2 of the SZ (depending on well depth). The effects of this model revision were notable in the drawdown observed in **Figure 5-19**. A detailed evaluation of the pumping data provided by MD WASD is included in **Section 5.3.3** of this report.

Figure 5-16 – Revised Saturated Zone Boundary Condition – Peak Storm

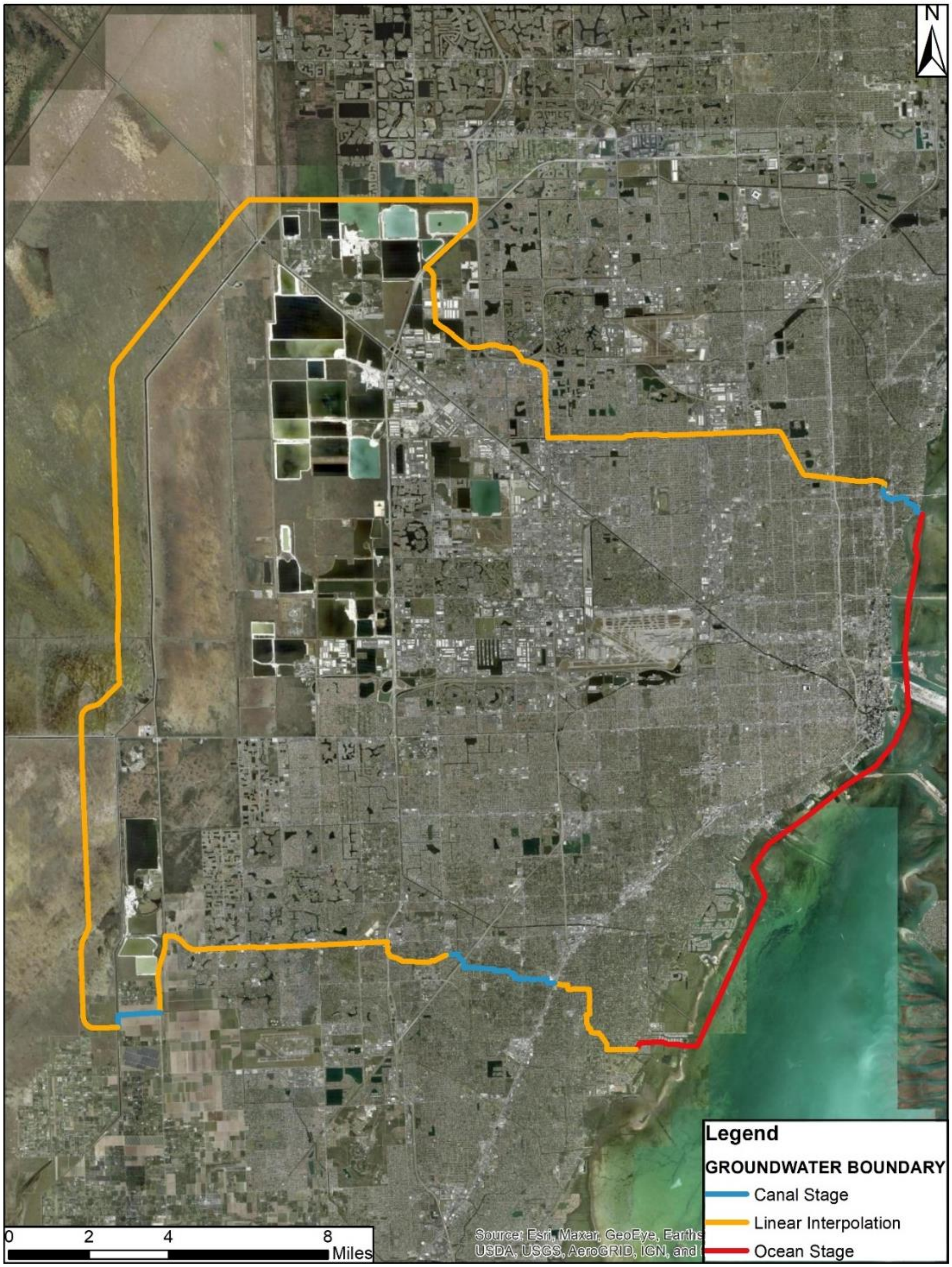


Figure 5-17 – Typical Initial Mining Pit Parameterization – Layer 1 Conductivity

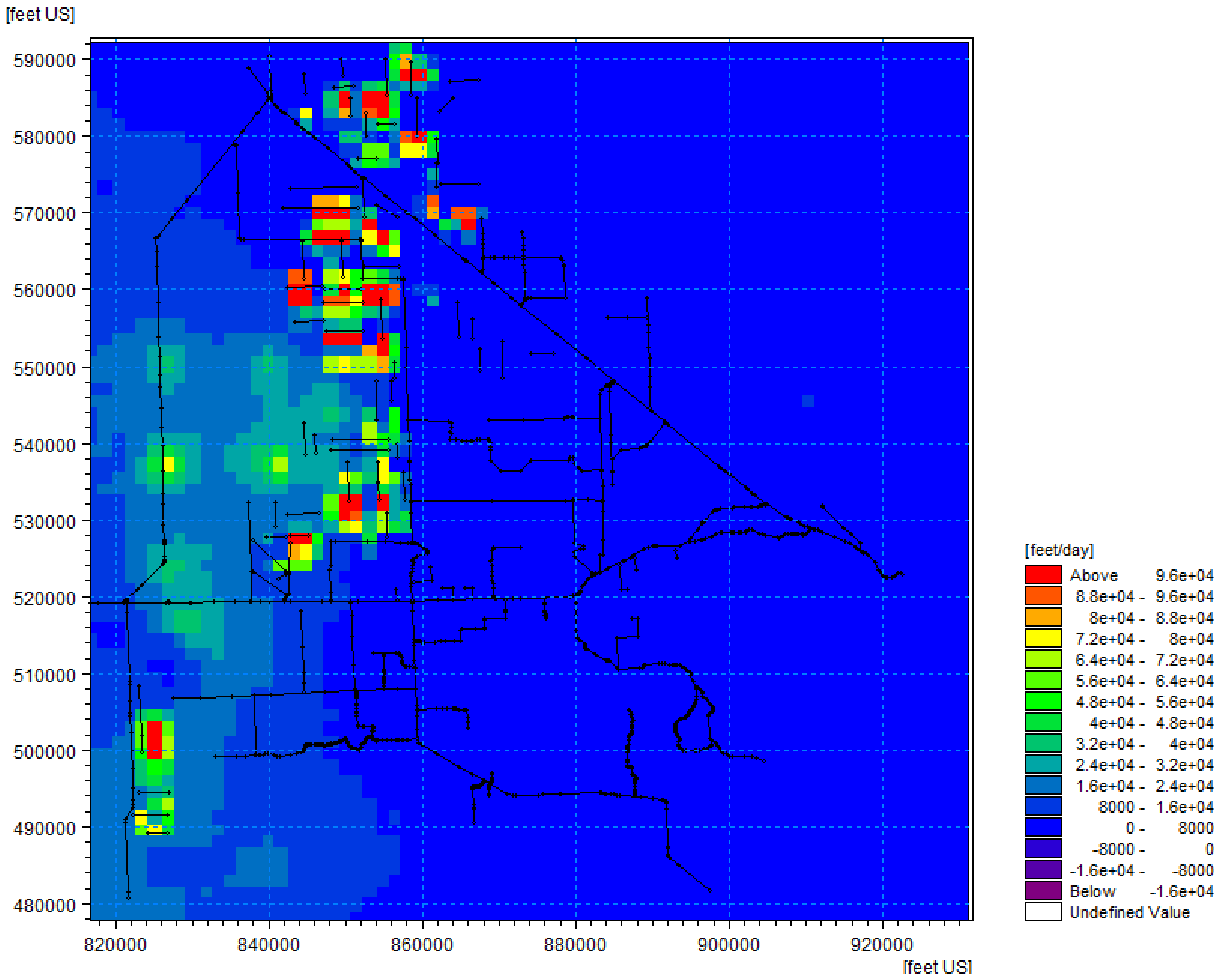


Figure 5-18 – Typical Revised Mining Pit Parameterization – Layer 1 Conductivity

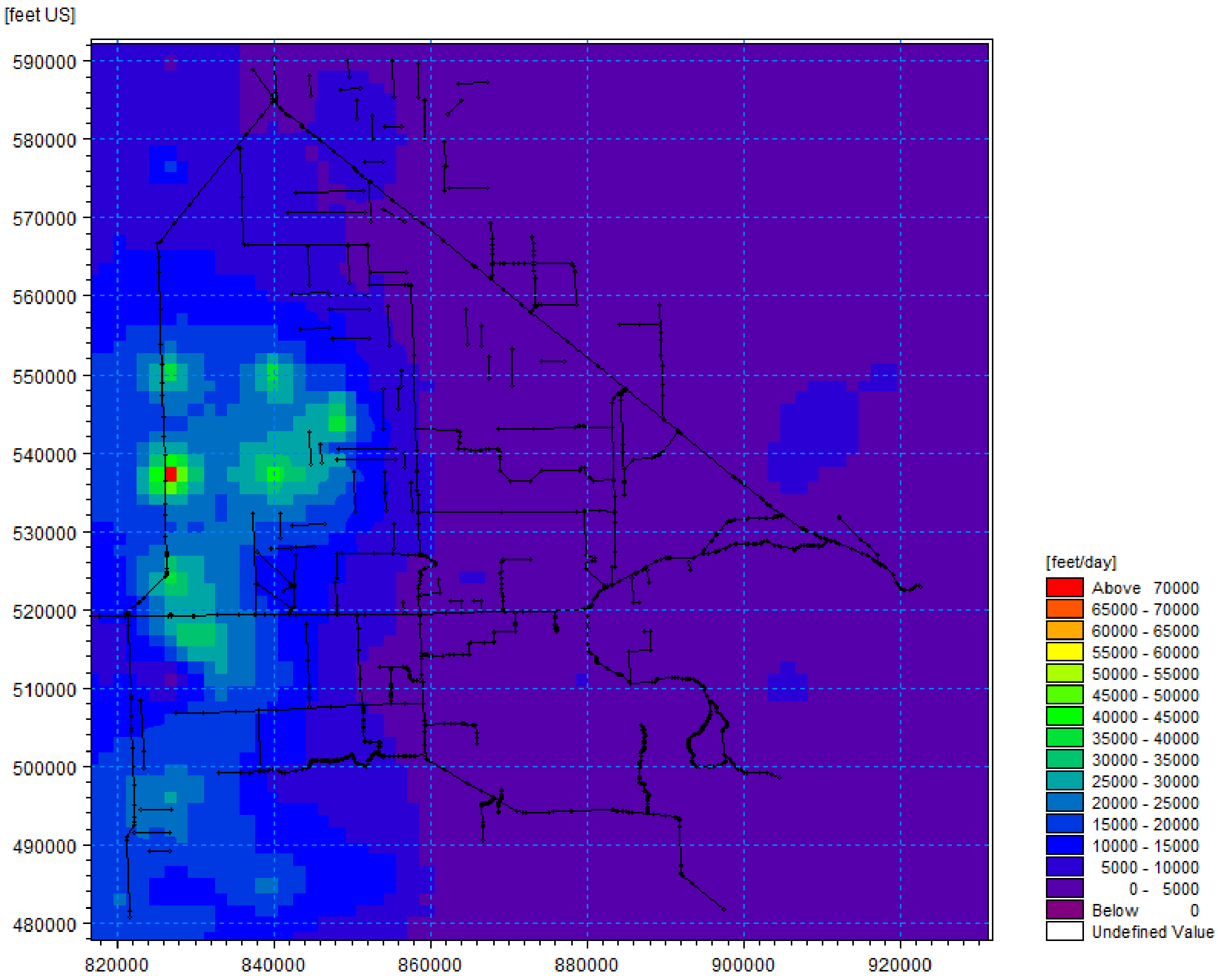
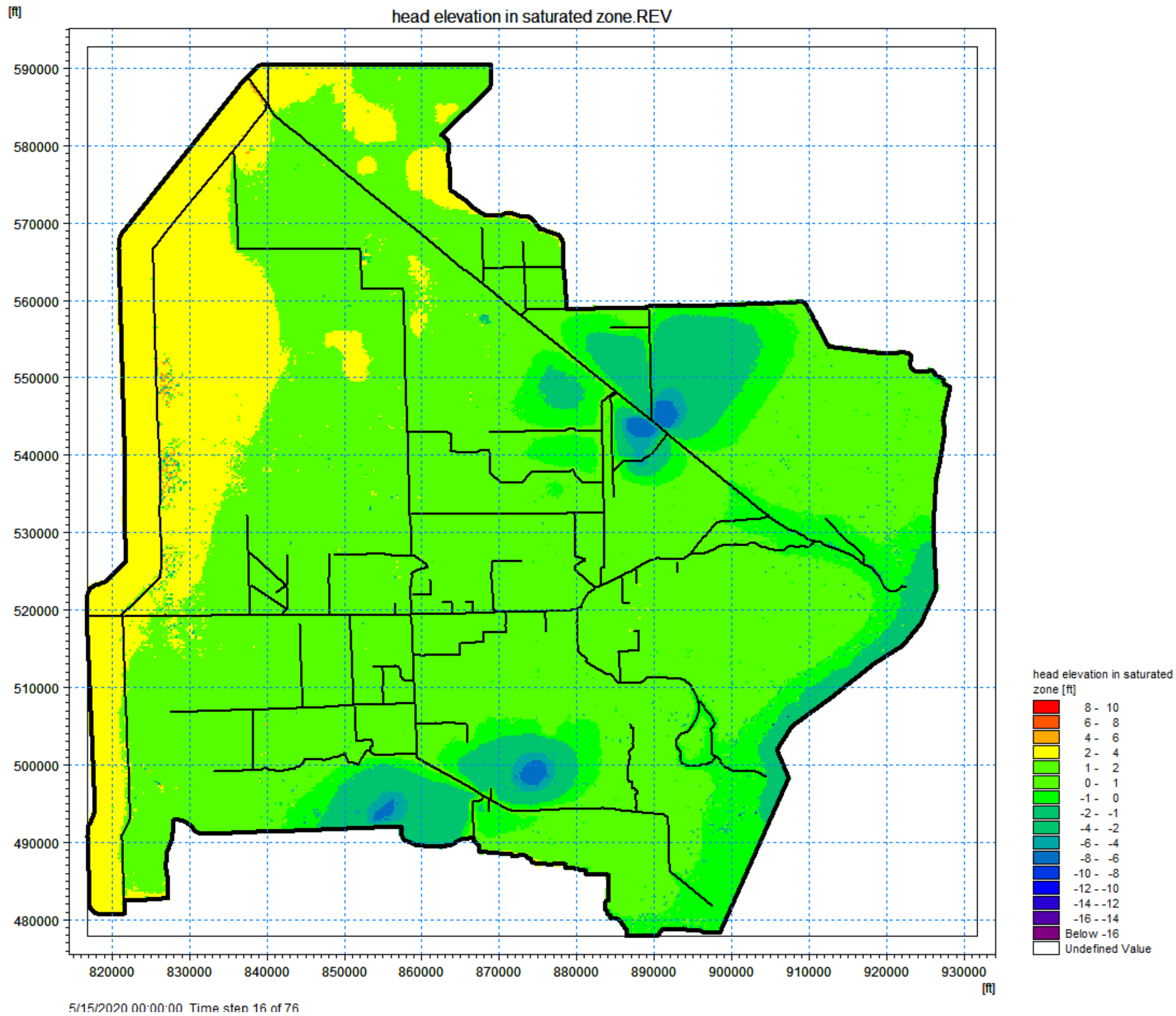


Figure 5-19 – Saturated Zone Drawdown at Wellfields within the Model Domain



5.1.1.6. FLOW RATING COEFFICIENTS FOR SPILLWAY STRUCTURES

The final revision made during the initial calibration effort was to update the default flow parameters for spillways to match documented reference values from the District. During the initial model development, the "a" and "b" values used to define the Controlled Submerged Flow Coefficients (CSFC) were 1.05 and 0.3, respectively. These values were based on the District's reference document, Ansar and Chen (2009). During the initial calibration effort, the values were updated based on new structure flow rating data that evaluated each location individually. The updated values are shown in **Table 5-2** below.

Table 5-2 – Updated Flow Coefficients for Spillways within the Model Domain

STRUCTURE	PRIMARY CANAL	CSFC "a"	CSFC "b"
S-22	C2 CANAL (Snapper Creek)	1.0073	0.2824
S-25B	C4 CANAL (Tamiami Canal)	1.0335	0.2582
S-26	C6 CANAL (Miami River)	1.1483	0.3849
S-335	L30 CANAL	0.9487	0.2734
S-334	C4 CANAL (West Of Krome Ave)	0.9623	0.481
G-421	C4 CANAL (Emergency Detention Basin)	1.05	0.3
G-93	C3 CANAL (Coral Gables Canal)	0.9276	0.2652

5.1.2. INITIAL CALIBRATION RESULTS

After the initial calibration revisions, the model performance was significantly improved. **Figure 5-20** through **Figure 5-24** demonstrate the issues described above as they are reflected in the headwater stage and the flow at S22, simulated and measured stage at T5W and simulated and measured groundwater stages at G-3 and G-3456. Notably, the groundwater monitoring data is represented by daily values instead of the continuous timeseries with a 15-minute output interval that is generated by the model.

Figure 5-20 – Measured and Revised Simulated Headwater Stage at the S-22 Tidal Spillway Structure

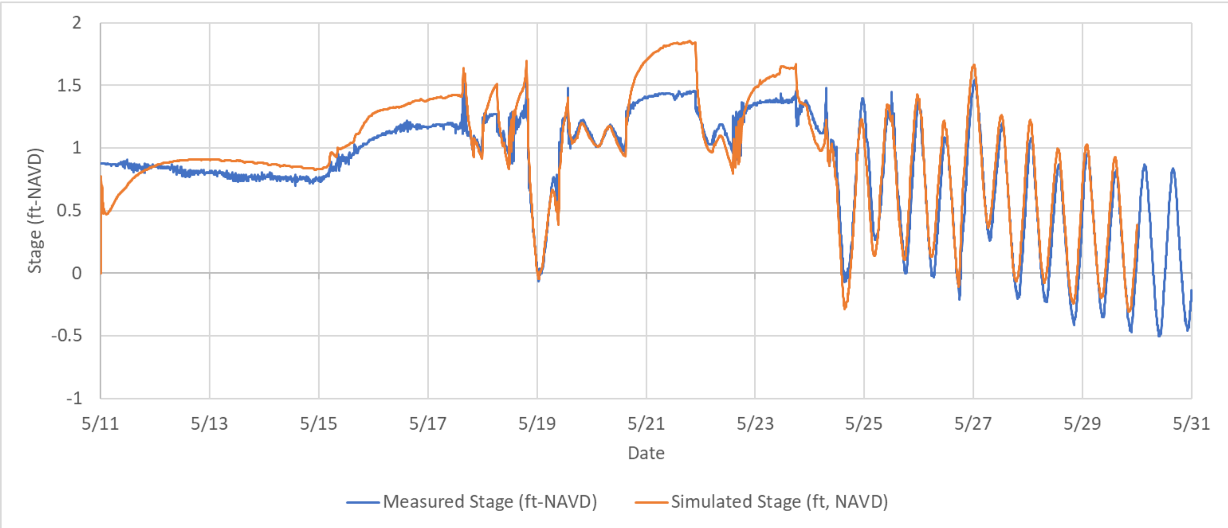


Figure 5-21 – Measured and Revised Simulated Flow at the S-22 Tidal Spillway Structure

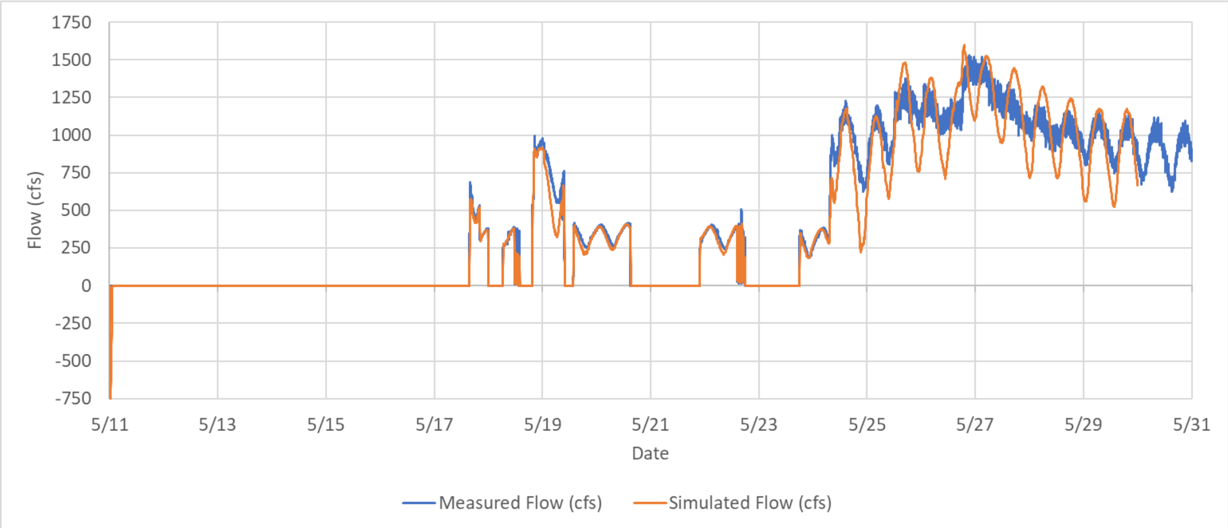


Figure 5-22 – Measured and Revised Simulated Stage at T5W

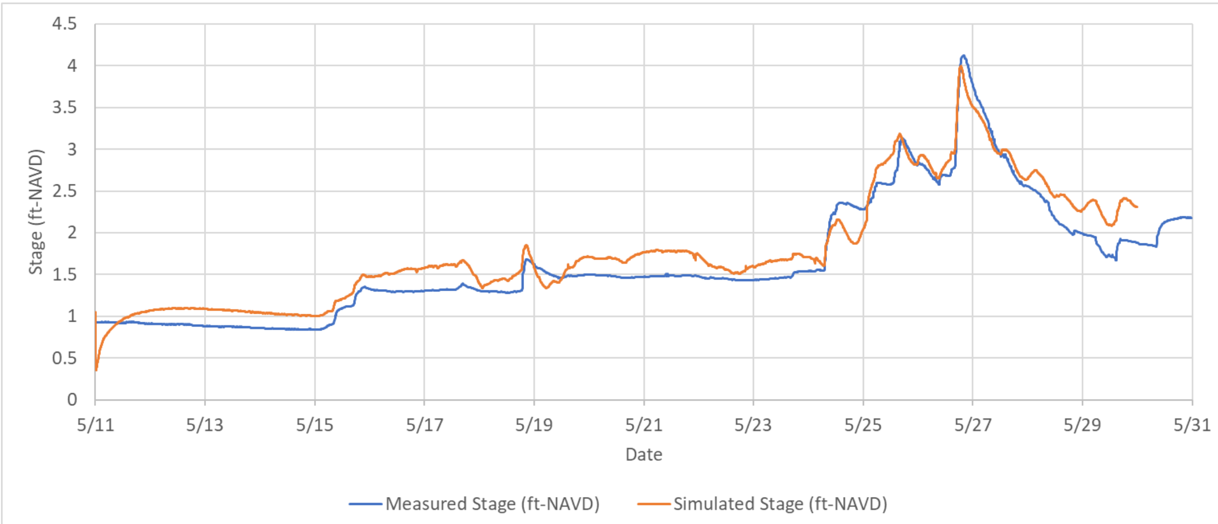


Figure 5-23 – Measured and Revised Simulated Groundwater Elevation at G-3

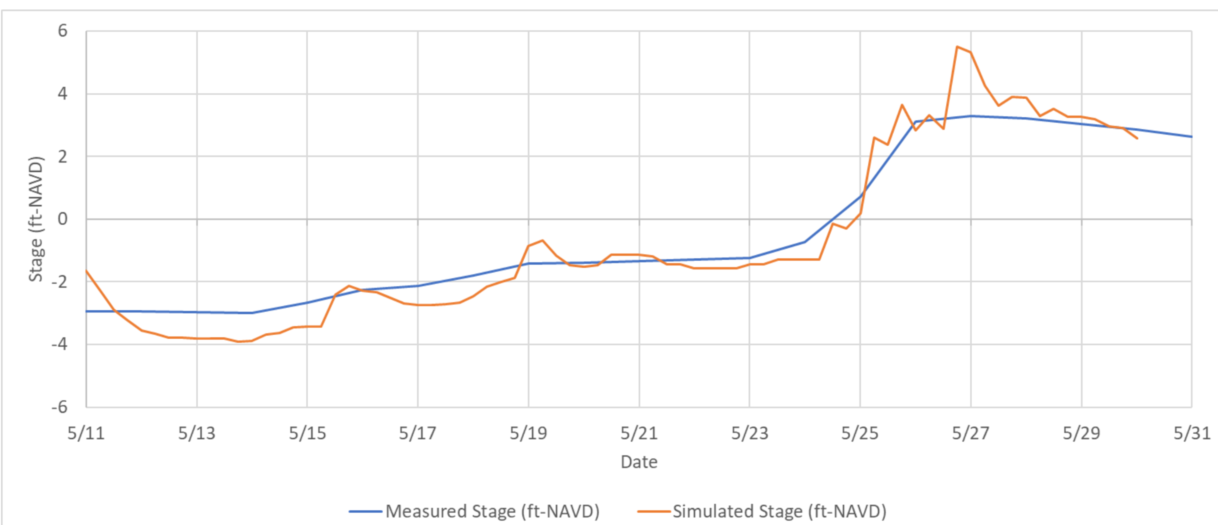
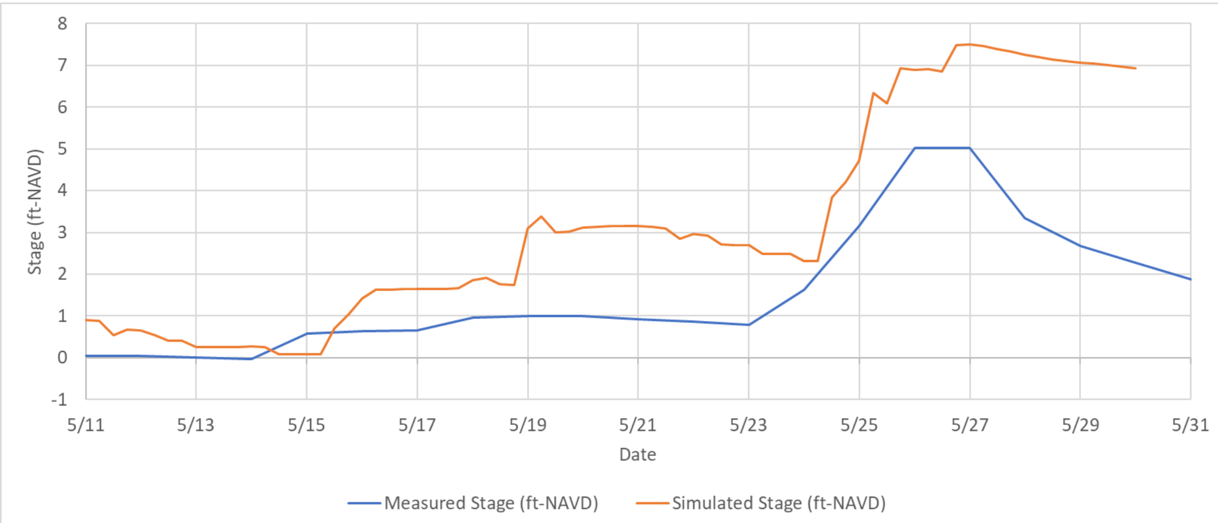


Figure 5-24 – Measured and Initial Simulated Groundwater Elevation at G-3456 for "Aquifer Only"



The statistical performance of the initial calibration model over the 3-week simulation period is shown in **Table 5-3**, with the following statistical parameters: Mean Error (ME), Mean Absolute Error (MAE), Root Mean Squared Error (RMSE), Correlation®, and Nash-Sutcliffe Efficiency (NSE). Locations where the model performance exceeded the calibration target of ±0.5 feet were limited to the headwater at G-93, and those exceedances were not significantly outside the target ranges.

Table 5-3 – Statistical Performance of Revised Calibration

STRUCTURE	ME	MAE	RMSE	R	NSE
S-22-H	-0.07	0.14	0.17	0.16	0.94
S-25-H	-0.21	0.32	0.39	0.33	0.86
S-25B-H	0.14	0.18	0.26	0.22	0.89
S-26-H	0.04	0.14	0.19	0.19	0.95
S-31-T	0.14	0.18	0.25	0.21	0.98
S-336-T	-0.05	0.29	0.38	0.38	0.96
S-380-T	-0.29	0.30	0.34	0.18	0.97
G93-H	-0.52	0.55	0.67	0.42	0.62
C2SW1	-0.14	0.20	0.23	0.17	0.97
C2SW2	-0.03	0.15	0.18	0.18	0.88
C4.CORAL	0.09	0.17	0.23	0.22	0.92
T5W	-0.12	0.19	0.21	0.17	0.98

5.2. SENSITIVITY TESTING

Once the revised calibration model set-up was established a series of modifications were made to the model parameters to determine the sensitivity of the results to each parameter. The following sensitivity analyses were performed:

- Tested effects of modifying K_H and K_V in Saturated Zone (SZ) model
- Tested effects of spatially uniform specific yield in SZ model
  - Uniform value of 0.25
  - Uniform value of 0.40
- Tested effects of Manning's roughness for primary canals from 0.033 to 0.04
- Tested effects of increasing the Manning's roughness for secondary canals from 0.033 to 0.04
- Tested effects of detention storage
  - Increasing by 0.25 feet
  - Decreasing by 0.25 feet
- Tested effects of modifying overland flow Manning's roughness
  - Increasing by 10%
  - Decreasing by 10%
- Tested effects of modifying drainage level in Saturated Zone
  - Increasing by 0.5 feet
  - Decreasing by 0.5 feet
- Tested the effect of turning off secondary system municipal pump stations.

5.2.1. SENSITIVITY OF K_H AND K_V IN LAYERS 1 AND 2 OF THE SATURATED ZONE

One of the notable discrepancies between measured and simulated results from the revised calibration simulation was the discrepancy at several groundwater monitoring locations where the simulated peak elevation exceeded the peak measured elevation, and the post-storm recession did not match measured. To evaluate the potential of horizontal and vertical and hydraulic conductivity to reduce the simulated discrepancy, a series of eight scenarios were considered.

Prior to evaluating sensitivity scenarios of hydraulic conductivity, a review of the assumed conditions was warranted. The base calibration model utilized a three-layer saturated zone where horizontal and vertical conductivity were equal as defined by the USGS MODFLOW model (Hughes and White, 2016). **Table 5-4** describes the characteristics of each SZ layer with respect to thickness and conductivity. As noted in this description, Layer 2 has a much lower conductivity relative to Layers 1 and 3.

Table 5-4 – Saturated Zone Layer Characteristics

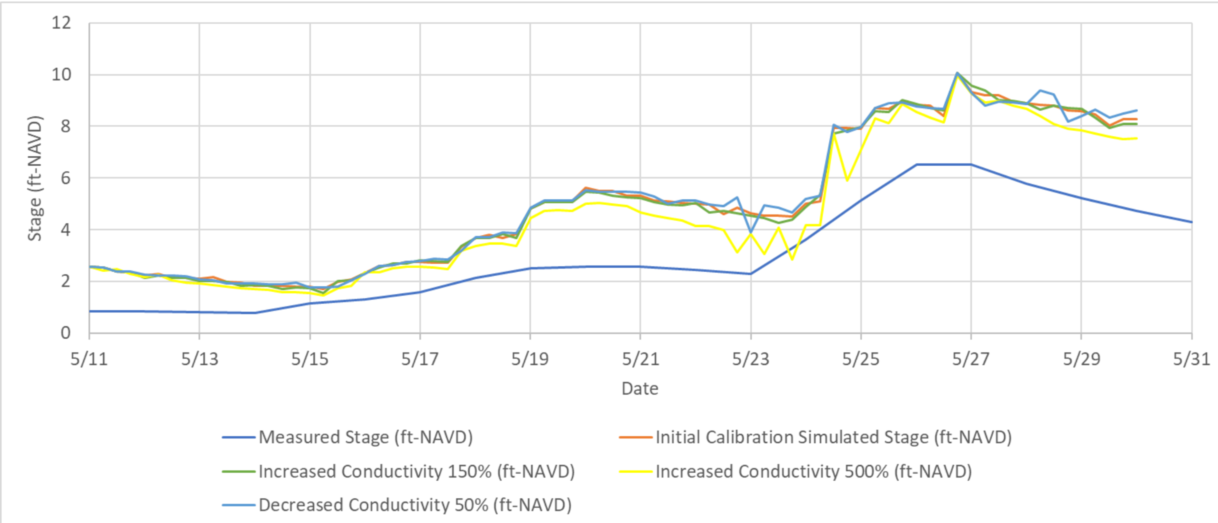
Layer	Range of Depth to Bottom [ft]	Range of Thickness [ft]	Range of K_H and K_V [ft/d]
1	(-)2 to (-)30	2 to 30	317 to 73775
2	(-)24 to (-)80	20 to 50	0.614 to 255.696
3	(-)45 to (-)115	20 to 35	387 to 152938

To test the effect of modifications to hydraulic conductivity in Layer 1, the following four scenarios were simulated:

1. Increasing spatially distributed values by 150%
2. Increasing spatially distributed values by 500%
3. Decreasing spatially distributed values by 50%

A comparison of the measured groundwater elevations with the results of the base calibration simulation and each sensitivity scenario for the G-3570 monitoring location demonstrates a minimal effect of the Layer 1 parameter modifications. G-3570 was selected as a representative monitoring well based on location and performance that is similar to other calibration locations within the model. The final calibration results for all monitoring well locations are included in **Section 5.4**. The hydraulic conductivity analysis for Layer 1 is demonstrated in **Figure 5-25**.

Figure 5-25 – Comparison of Sensitivity Scenarios for Layer 1 Hydraulic Conductivity

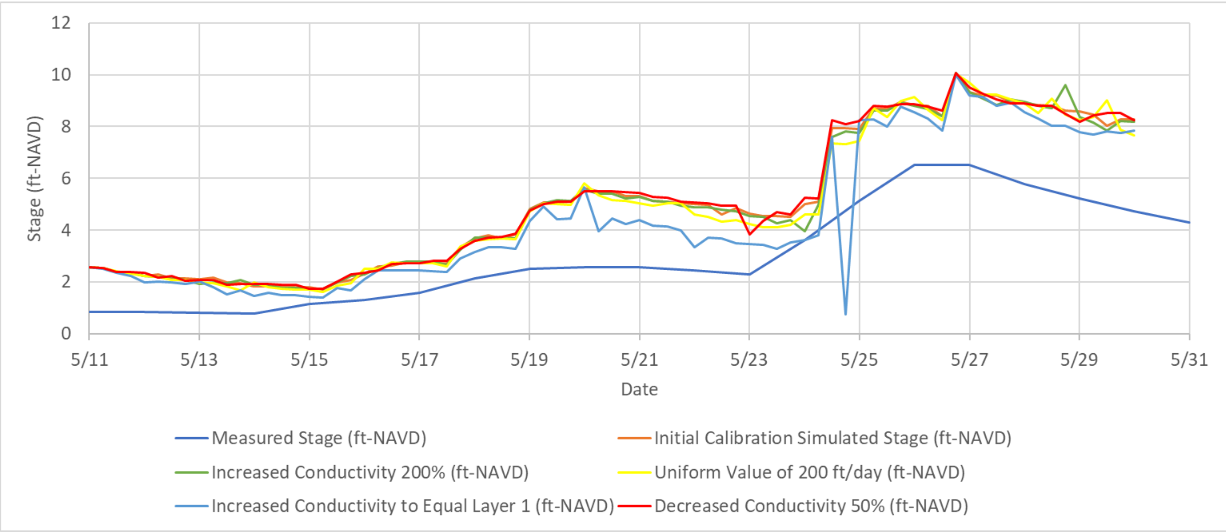


Since the modifications to Layer 1 had a minimal effect on groundwater results, an evaluation of the sensitivity of hydraulic conductivity in Layer 2 was performed. To test the effect of modifications to Layer 2, the following four scenarios were simulated:

1. Increasing spatially distributed values by 200%
2. Replacing spatially distributed values with a uniform value of 200 feet/day
3. Increasing spatially distributed values to equal Layer 1
4. Decreasing spatially distributed values by 50%

A comparison of the measured groundwater elevations with the results of the base calibration simulation and each sensitivity scenario for the G-3570 monitoring location demonstrates the minimal effect of the Layer 2 parameter modifications. The hydraulic conductivity analysis for Layer 2 is demonstrated in **Figure 5-26**.

Figure 5-26 – Comparison of Sensitivity Scenarios for Layer 2 Hydraulic Conductivity



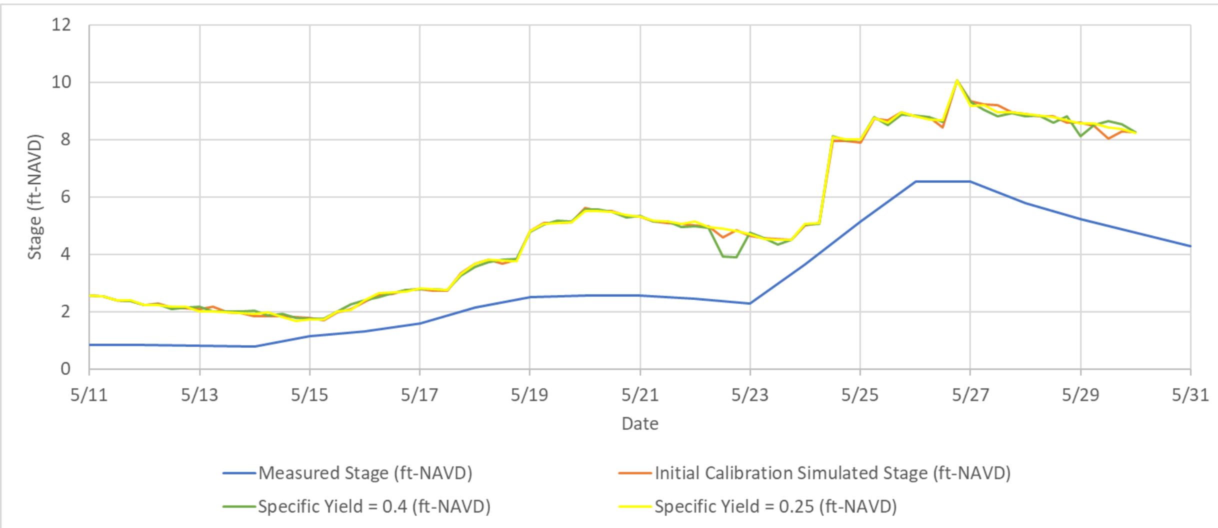
With the exception of the downward spike on May 24th for the scenario where Layer 2 conductivity was set equal to Layer 2, the comparison of model results demonstrates that the simulated stages were not particularly sensitive to the modifications of K_H and K_V that were made. The downward spike appears to be a temporary response in the Saturated Zone due to the higher conductivity and a dip in stages within the C4 Canal. This result was not determined to be representative of a larger pattern of responsiveness with no other similar results during the simulation duration. Based on this comparison, the calibration simulation utilized the same conductivity values defined in the USGS MODFLOW model. However, further review of the hydrogeological layering and parameterization is provided in **Section 5.5**.

5.2.2. SENSITIVITY OF SPECIFIC YIELD FOR SATURATED ZONE

Another parameter of the SZ model that was available for modification was specific yield. Similar to hydraulic conductivity, the value for specific yield used within the calibrated model was also developed based on the USGS MODFLOW model (Hughes and White, 2016). The values for specific yield exported from the USGS model is a spatially variable dataset that varies from 0.09 to 0.50. For the sensitivity analysis, two values were being tested were uniformly applied across the model domain: 0.25 and 0.40. A comparison of the measured groundwater elevations with the results of the base calibration simulation and each sensitivity scenario for the G-3570 monitoring location demonstrates the minimal effect of modifications to specific yield as demonstrated in **Figure 5-27**.

The comparison of model results demonstrated that the simulated stages are not sensitive to the modifications of specific yield that were made. As such, the final calibration simulation utilized the same spatially variable values for specific yield that were defined in the USGS MODFLOW model.

Figure 5-27 – Comparison of Sensitivity Scenarios for Specific Yield



5.2.3. SENSITIVITY OF MANNING'S ROUGHNESS FOR PRIMARY CANALS

As noted in **Section 5.1.1.2**, the values for Manning's roughness in each canal reach were set equal to an "n" value of 0.033 for the segment of the cross-section that is between the left top of bank and right top of bank. Roughness values outside of the top of bank were set to 0.08 because out of bank flow is expected to include various surface features that will act as impediments to flow. For the purposes of sensitivity testing, the effect of increasing the channel roughness for the primary canals to 0.04 was evaluated. For the purposes of this scenario, the primary canals were defined as the C2, C3, C4, C5 and C6 canals. As shown in **Figure 5-28** and **Figure 5-29**, the effect of increasing channel roughness varied throughout the model domain. At the western limit of the C6 Canal, the S-31 tailwater stages were improved by the increase in roughness, while in the western portion of the C4 Canal, the T5W stages were made worse by the same change.

Figure 5-28 – Comparison of Sensitivity at S-31 TW for Roughness of Primary Canals

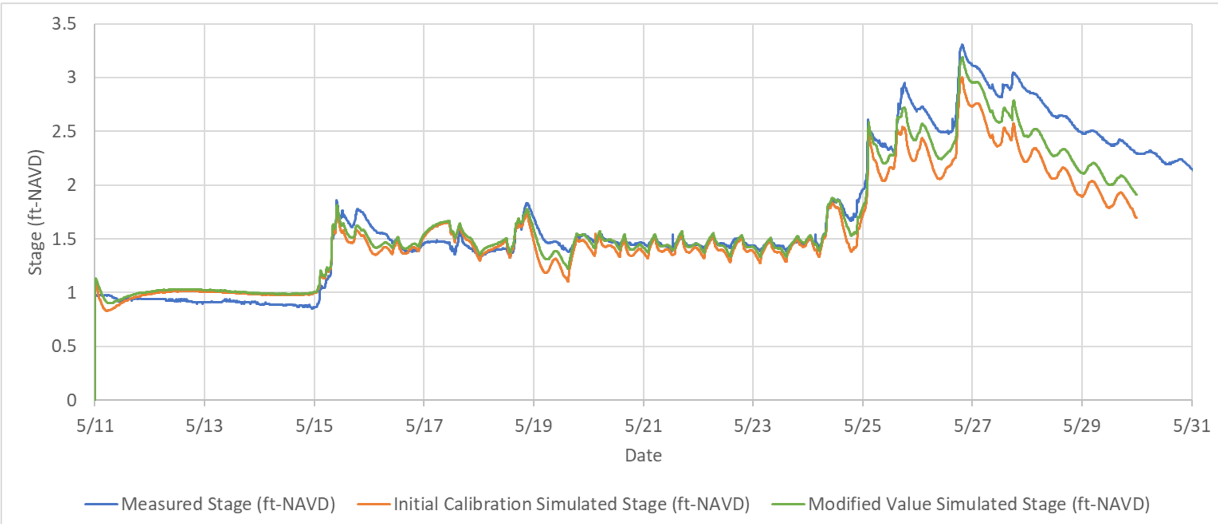
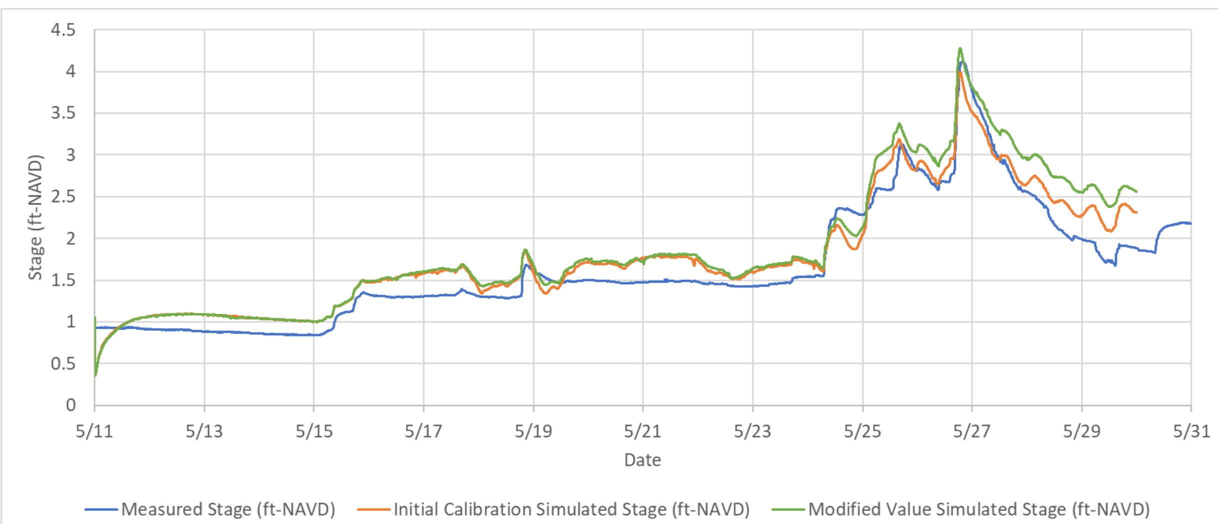


Figure 5-29 – Comparison of Sensitivity at T5W for Roughness of Primary Canals

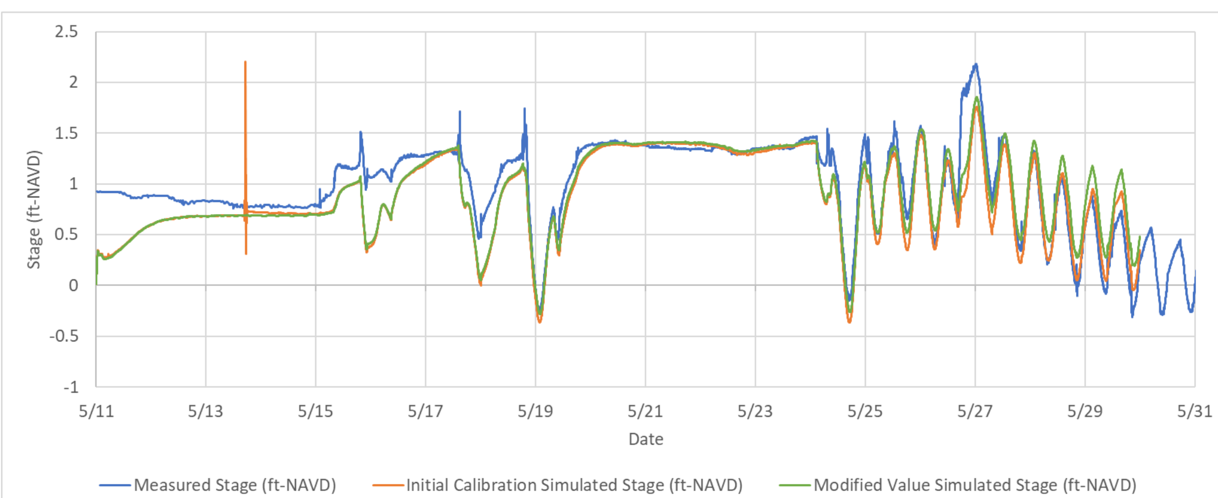


The initial calibration value of 0.033 is consistent with standard roughness values for District canals that are well maintained. As an example of the District's routine maintenance of canals within the model domain, records show vegetation removal from the C6 Canal in August 2017, immediately prior to the validation run. In consideration that the effects of increasing the roughness value on simulation performance is a mix of positive and negative, and considering the typical operations and maintenance of the primary canals, the roughness values for the primary canals was recommended to remain 0.033 in the final calibration.

5.2.4. SENSITIVITY OF MANNING'S ROUGHNESS FOR SECONDARY CANALS

Although vegetation removal is routinely performed on primary canals by District Field Station personnel, the secondary canals are not as routinely maintained. Considering this fact, a sensitivity scenario was performed where values of channel roughness in the secondary canals was increased to 0.04. The potential effect of this change would be an attenuation of the peak stage at watershed outfalls as the delivery of runoff is delayed. However, **Figure 5-30** shows the change in the secondary canals had a minimal effect on simulation performance at calibration monitoring locations for the majority of the simulation period. The only exceptions are a slight shift upward in simulated stage after the storm event and what appears to be an instability on May 13th, prior to the rainfall event, that caused a temporary spike in simulated stages. Considering the differences in simulated stages were not significant and there was no additional data available to support a change in the roughness values, no change in channel roughness for secondary canals was recommended for the final calibration.

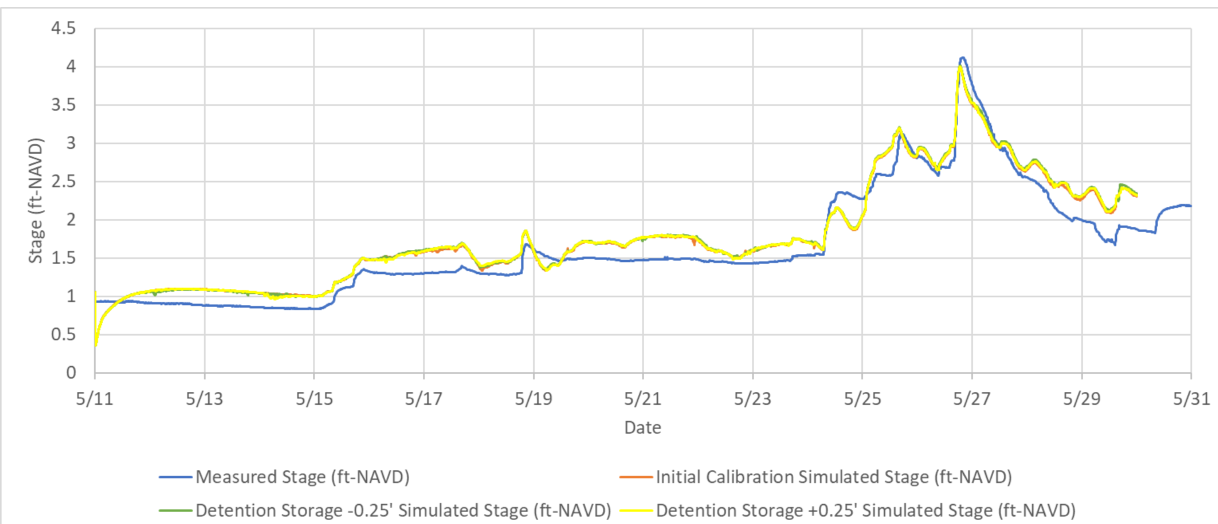
Figure 5-30 – Comparison of a Sensitivity at S25B for Roughness of Secondary Canals



5.2.5. SENSITIVITY OF DETENTION STORAGE PARAMETER FOR OVERLAND FLOW

A value for detention storage was assigned to each land use based on the type of development and land management operations. Values for detention storage in the initial model parameterization vary from 0.0 feet to 0.4 feet depending on land use. These values are consistent with other FPLOS modeling efforts in the region (Kimley-Horn, 2020 and Taylor, 2020). For the sensitivity scenarios, the values for detention storage at each location were increased by 0.25 feet and decreased by 0.25 feet. In the case where the assumed detention storage was less than 0.25 feet the sensitivity scenario assumed no detention storage. **Figure 5-31** illustrates that at the T5W location, the effect on increasing detention storage was a minor improvement while decreasing detention storage caused the results to be slightly worse. Considering that the results of these sensitivity modifications did not appreciably improve the simulated results, no change to the parameterization of detention storage was recommended for the final calibration.

Figure 5-31 – Comparison of a Sensitivity Scenarios at T5W for Detention Storage



5.2.6. SENSITIVITY OF MANNING'S ROUGHNESS FOR OVERLAND FLOW

Another parameter defined by land use in the model was Manning's M roughness for overland flow. Values for overland flow roughness vary in the initial model parameterization from 2.22 for Upland Forest and Wetland land use to 16.67 for Water land use. These values are consistent with other FPLOS modeling efforts in the region (Kimley-Horn, 2020 and Taylor, 2020). For the sensitivity scenarios, the values for overland flow roughness were increased and decreased by 10%, 25%, and 50% throughout the model. The results of these scenarios are shown for C2SW1 in **Figure 5-32** through **Figure 5-34**.

Considering that the results of these sensitivity modifications do not appreciably improve the simulated results, no change to the parameterization of overland flow roughness was recommended for the final calibration.

Figure 5-32 – Comparison of a Sensitivity Scenarios at C2SW1 for Overland Flow Roughness - ±10%

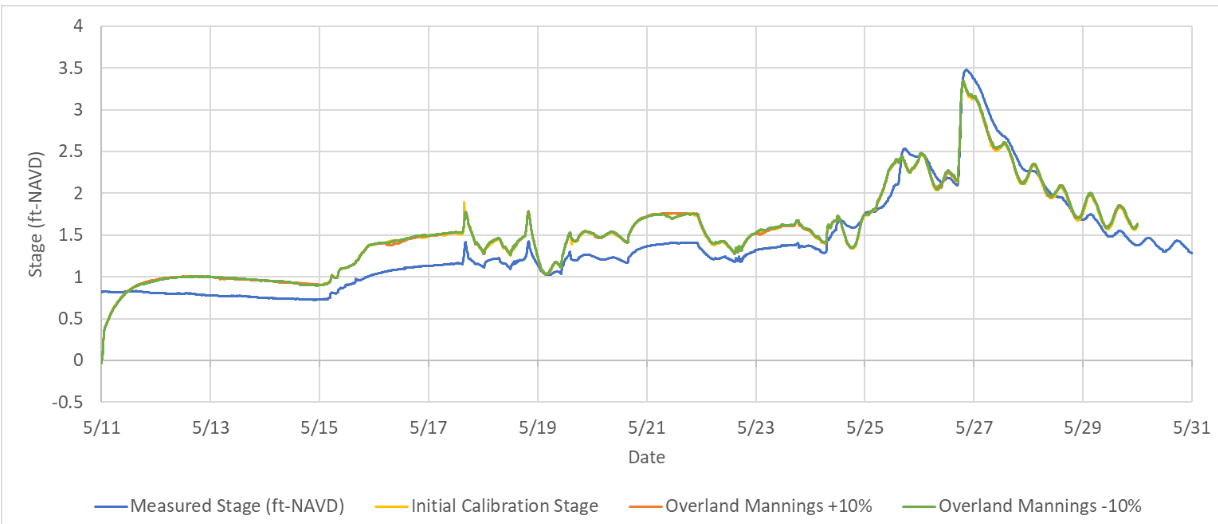


Figure 5-33 – Comparison of a Sensitivity Scenarios at C2SW1 for Overland Flow Roughness - ±25%

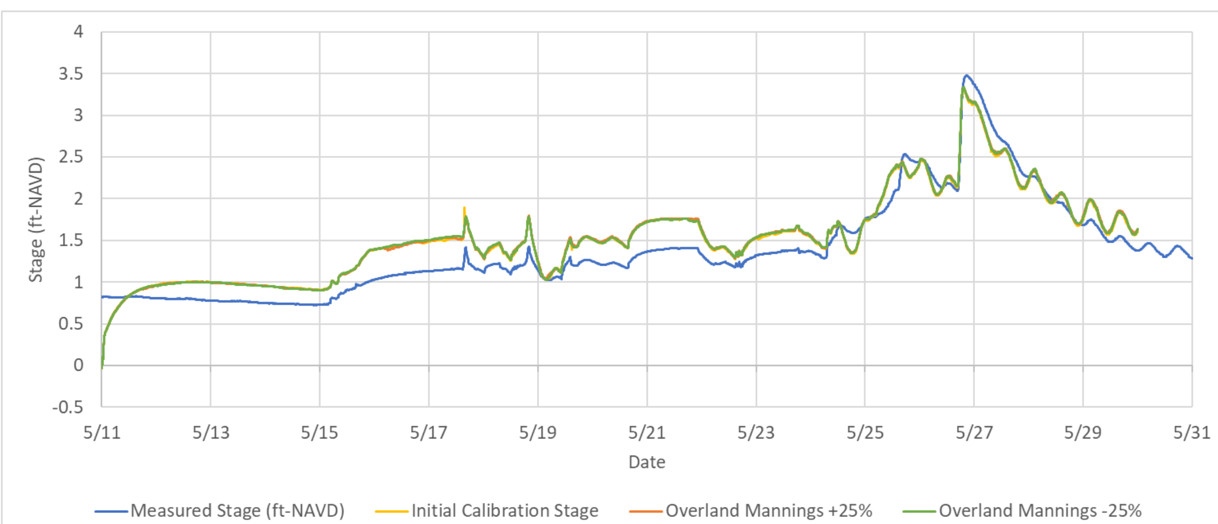
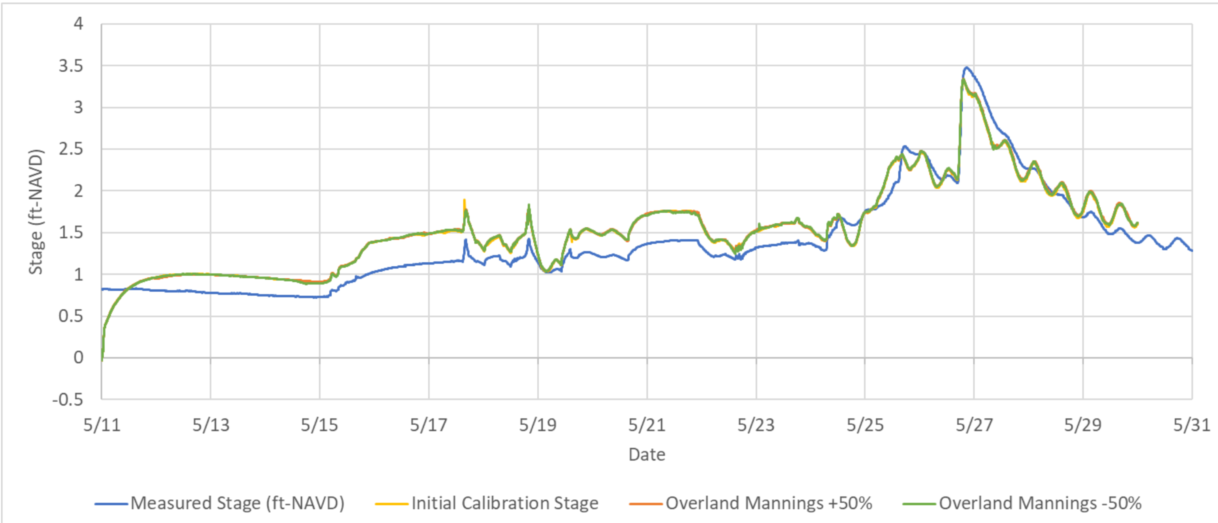


Figure 5-34 – Comparison of a Sensitivity Scenarios at C2SW1 for Overland Flow Roughness - ±50%



5.2.7. SENSITIVITY OF DRAINAGE LEVEL IN THE SATURATED ZONE

An additional parameter defined by land use in the model is drainage level within the SZ. The drainage level parameter represents the effect of stormwater management systems that manage the water table to increase infiltration availability. Values for drainage level vary in the initial model parameterization from 2.5 feet for agricultural land to 0.0 feet for undeveloped land. These values are consistent with other FPLOS modeling efforts in the region (Kimley-Horn, 2020 and Taylor, 2020). For the sensitivity scenarios, the values for drainage level were increased by 0.5 feet and decreased by 0.5 feet where applied. The results of these scenarios are shown for S-26 in **Figure 5-35** and for S-22 in **Figure 5-36**. Considering that the results of these sensitivity modifications showed a negative impact of increasing drainage level and did not appreciably improve the simulated results when the drainage level was decreased, no change to the parameterization of drainage level was recommended for the final calibration.

Of note, the MIKE model conceptualization of drainage level in the SZ is such that it is duplicative to also use the Ponded Drainage option from the Overland Flow package. Essentially, when the SZ is fully saturated and is above the land surface, the same calculation will occur to route drainage to the specified area based on the drain codes used in the SZ Drainage module.

Figure 5-35 – Comparison of a Sensitivity Scenarios at S-26 for Drainage Level

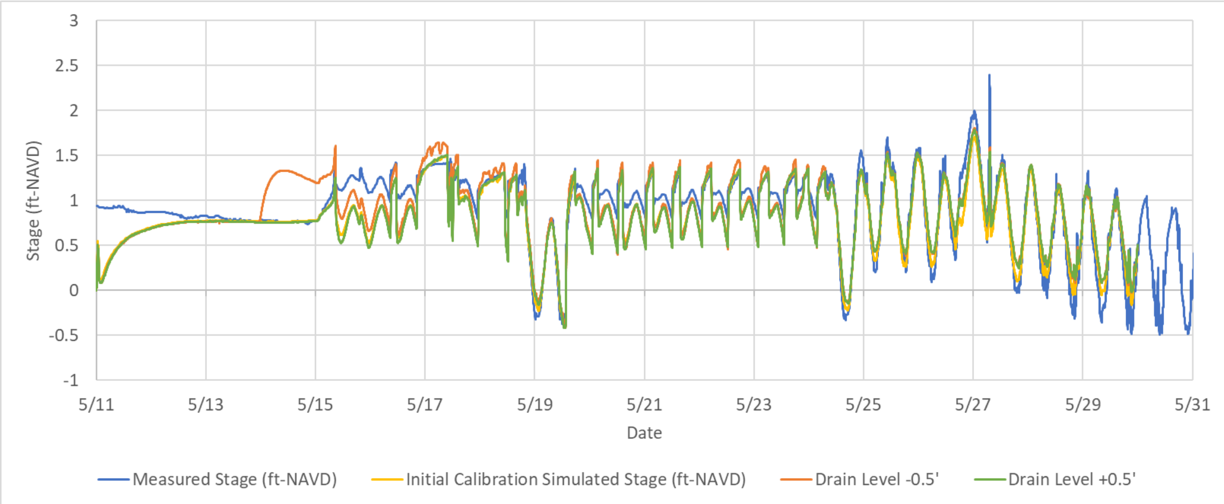
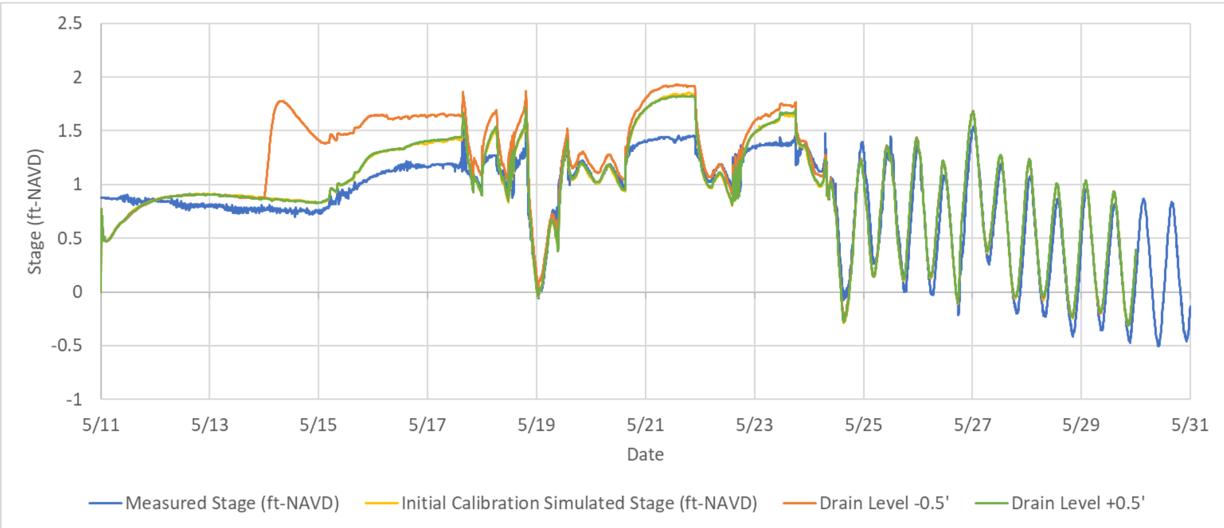


Figure 5-36 – Comparison of a Sensitivity Scenarios at S-22 for Drainage Level



5.2.8. SENSITIVITY OF MUNICIPAL PUMP STATION OPERATIONS

The only operational variable considered during the sensitivity analysis was with regard to the municipal pump operations. The local government agencies that operate the pump stations could not provide records of when secondary pump stations were engaged. For the initial calibration simulation, the pumps were assumed to turn on at 01:30 on May 25th and ran at full capacity until 01:30 on May 31st. For the sensitivity evaluation, simulations were performed where none of the pumps turned on as well as a case where only West Miami pumps were turned off. An alternate scenario was performed where the pumps were operated based on operational criteria specific to the C4 Canal, assuming all municipal pumps are turned on when the simulated stages reach 3.23 feet NAVD, or 4.8 feet-NGVD, at the T5 station and are turned off when the Detention Basin is full or when the stages at T5 are below 2.43 feet NAVD, or 4.0 feet-NGVD, whichever condition occurs first. The simulation results showed varying levels of improvement for these scenarios as demonstrated in **Figure 5-37** through **Figure 5-40**.

Figure 5-37 – Comparison of a Sensitivity Scenarios at S-22 for Municipal Pump Operations

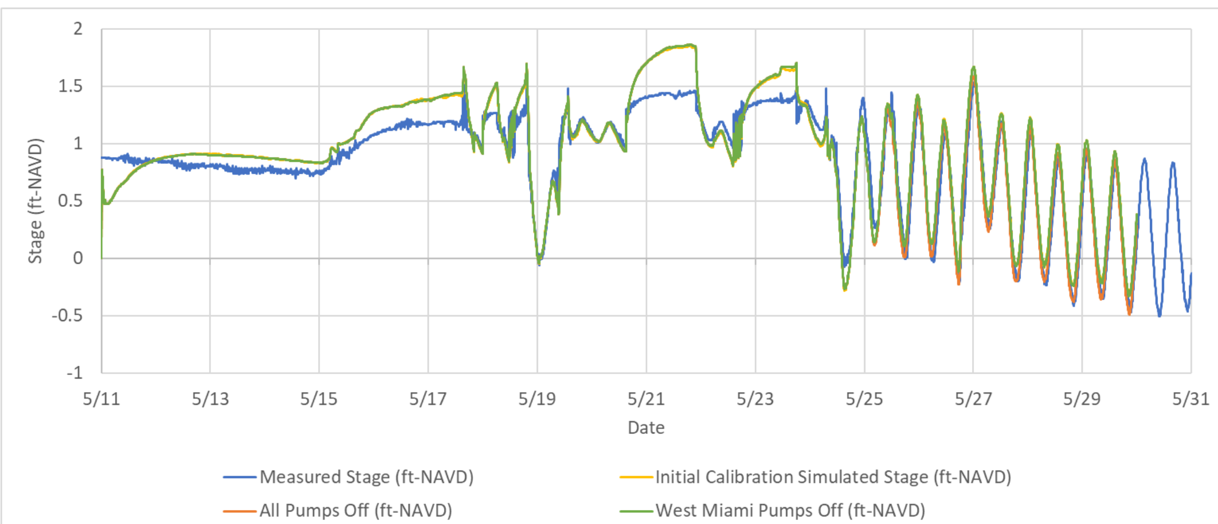


Figure 5-38 – Comparison of a Sensitivity Scenarios at S-25B for Municipal Pump Operations

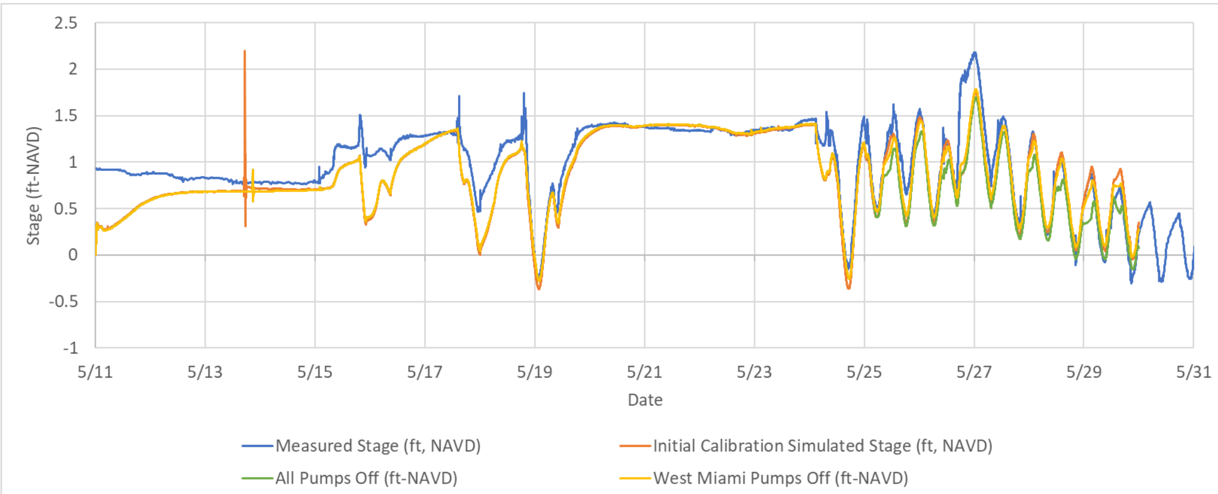


Figure 5-39 – Comparison of a Sensitivity Scenarios at S-26 for Municipal Pump Operations

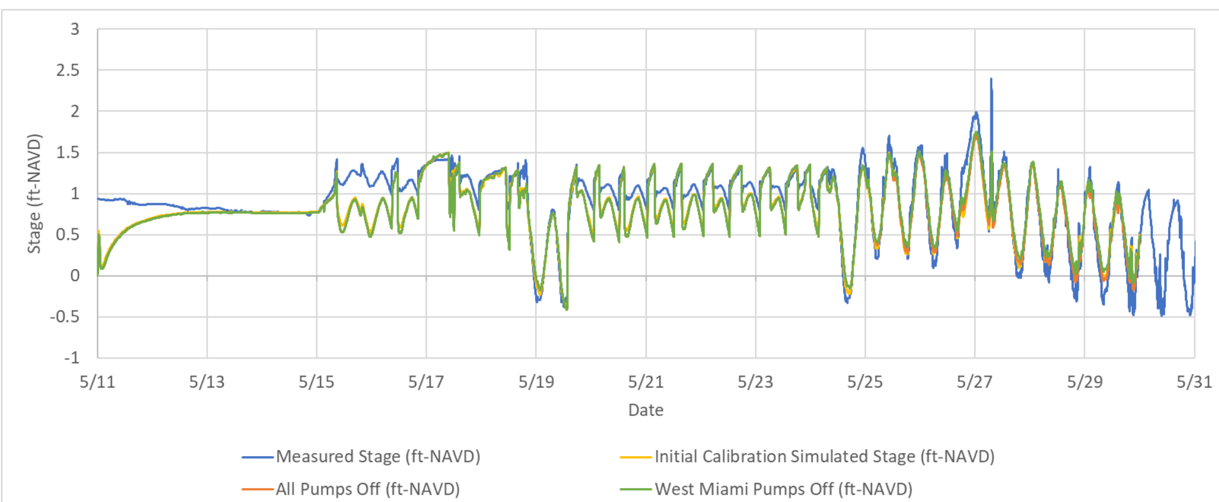
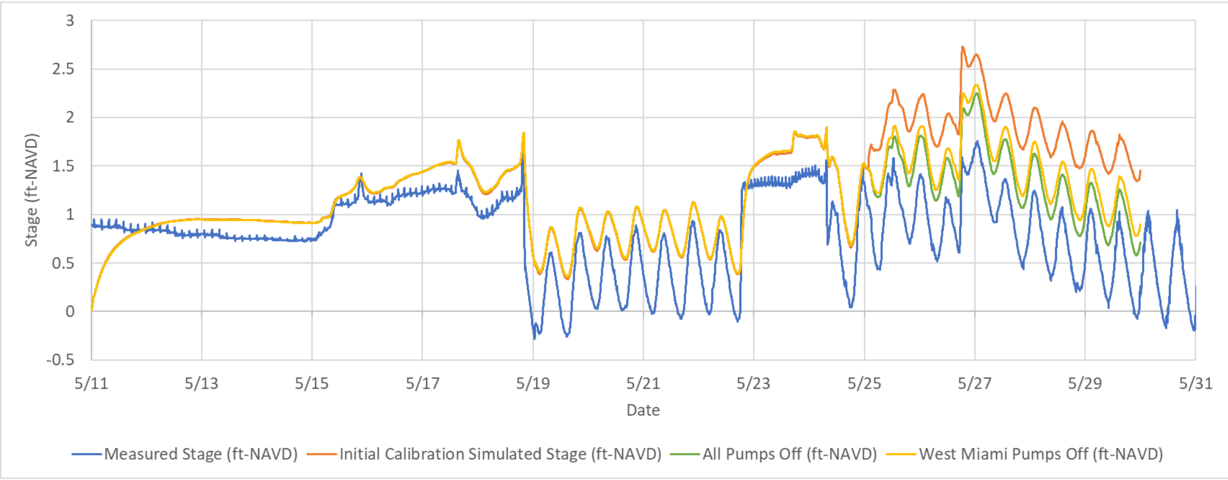


Figure 5-40 – Comparison of a Sensitivity Scenarios at G-93 for Municipal Pump Operations



As noted in the results above, for the C2, C4, and C6 canals the effect of the municipal pump station operations were minimal. For the C3 Canal at the G-93 tidal spillway structure there was a noticeable improvement when the municipal pumps were turned off. There were no municipal operations records available, but archived notes from District operations staff at 8:30 PM on May 26th note that it was not clear if the Sweetwater municipal pumps were turned on, it was believed that the Belen and Miami pumps were on during the event, and there was no information available for the West Miami pumps. Based on these records and the model results, it was assumed that all pumps were on for the calibration simulation, except for the West Miami pump stations.

5.3. ADDITIONAL ANALYSIS

In response to discussions with District staff, additional analysis was performed to test the accuracy of the NEXRAD data compared to rain gage data, to evaluate the pumping rates for the Miami Dade wellfields, and to test the interaction between the groundwater and surface water within the model.

5.3.1. NEXRAD EVALUATION

Analysis was performed to test the accuracy of the NEXRAD data throughout the model domain. Six rain gages were identified within the NEXRAD rain grid cells and chosen as verification points. The selected rain gages can be seen below in **Figure 5-41** and each is located directly within a NEXRAD pixel.

Both the calibration and validation storm events were evaluated during this analysis. Initially, the entire simulation period was used for analysis, however it became apparent that the periods before and after heavy rainfall could cause the statistics to show less error. To remedy this issue the 1-day peak and 3-day peak sum of the rain events were also used in this analysis. The 1-day peak for the 2017 storm event was found to be on September 10th, 2017, and the 1-day peak for the 2020 storm event was found to be on May 5th, 2020. The 3-day peak for the 2017 storm event occurred between September 9th, 2017, and September 11th, 2017, and the 3-day peak for the 2020 storm event occurred between May 24th, 2020, and May 26th,2020.

This evaluation showed varied results. When evaluating the 1-day storm maximums, the 2017 and 2020 storms exhibited almost opposite results. In 2017, gages S335-R, S26-R, S27-R, and MIAMI.FS-R showed less rainfall in the NEXRAD pixels than in the rain gages. In 2020, however, only S26-R and S338-R showed less rainfall in the NEXRAD pixels compared to the rain gages. This abnormality continued when evaluating the 3-day storm maximums. The results for the analysis can be seen in **Table 5-5** through **Table 5-7** which also includes the percent difference between the gage and NEXRAD totals as well as the average percent difference for all locations. The percent differences were calculated in reference to NEXRAD rainfall values. Considering that the difference for the peak periods and the entire simulation is not large or strongly biased, it was decided that the NEXRAD data was sufficient for use for both calibration and validation storm scenarios.

Table 5-5 – NEXRAD Evaluation – Full Simulation Duration

GAUGE	2017 SUM			2020 SUM		
	NEXRAD [IN]	GAGE [IN]	DIFF [%]	NEXRAD [IN]	GAGE [IN]	DIFF [%]
S30-R	12.47	12.63	-1.28%	18.86	18.69	0.90%
S338-R	16.74	16.88	-0.84%	13.39	14.63	-9.26%
S335-R	12.39	13.95	-12.59%	14.32	14.11	1.47%
S26-R	15.35	17.75	-15.64%	19.64	19.87	-1.17%
S27-R	11.95	13.45	-12.55%	19.62	19.91	-1.48%
MIAMI.FS-R	19.91	18.54	6.88%	17.24	17.4	-0.93%
AVERAGE			-6.00%			-1.75%

Table 5-6 – NEXRAD Evaluation – One Day Peak

GAUGE	VALIDATION 1-DAY PEAK (3-YEAR, 1-DAY = 5-IN TO 6-IN)			CALIBRATION 1-DAY PEAK (5-YEAR, 1-DAY = 6-IN TO 7-IN)		
	9/10/2017			5/25/2020		
	NEXRAD [IN]	GAGE [IN]	DIFF [%]	NEXRAD [IN]	GAGE [IN]	DIFF [%]
S30-R	4.18	4.06	2.87%	6.54	6.46	1.22%
S338-R	5.57	4.57	17.95%	2.5	2.71	-8.40%
S335-R	5.29	5.91	-11.72%	6.12	5.36	12.42%
S26-R	4.6	5.07	-10.22%	4.35	4.58	-5.29%
S27-R	3.76	4.65	-23.67%	6.39	5.3	17.06%
MIAMI.FS-R	5.14	5.57	-8.37%	3.64	3.5	3.85%
AVERAGE			-5.53%			3.48%

Table 5-7 – NEXRAD Evaluation – Three Day Peak

GAUGE	VALIDATION 3-DAY PEAK (10-YEAR, 3-DAY = 10-IN TO 11-IN)			CALIBRATION 3-DAY PEAK (25-YEAR, 3-DAY = 11-IN TO 14-IN)		
	9/9/2017 – 9/11/2017			5/24/2020 – 5/26/2020		
	NEXRAD [IN]	GAGE [IN]	DIFF [%]	NEXRAD [IN]	GAGE [IN]	DIFF [%]
S30-R	7.79	7.76	0.39%	10.71	10.6	1.03%
S338-R	7.97	6.65	16.56%	8.51	9.53	-11.99%
S335-R	6.86	7.68	-11.95%	9.96	9.57	3.92%
S26-R	6.94	7.62	-9.80%	12.37	12.75	-3.07%
S27-R	5.71	6.96	-21.89%	13.05	12.62	3.30%
MIAMI.FS-R	7.29	7.98	-9.47%	10.96	11.18	-2.01%
AVERAGE			-6.03%			-1.47%

An additional analysis was performed on the calibration and validation events to illustrate the spatial variability of the NEXRAD rainfall dataset.

Figure 5-42 through **Figure 5-45** illustrate the depth at each grid cell in the model domain for the 1-day and 3-day peak rainfall periods. In reviewing this analysis, the one-day rainfall total for the calibration event (May 2020) is roughly consistent with a 5-year return period event, while the three-day rainfall total for the calibration event (May 2020) is roughly consistent with a 25-year return period event. Similarly, the 1-day rainfall total for the validation event (Sept. 2017) is consistent with a 3-year return period event, while the 3-day rainfall total for the validation event (Sept. 2017) is just less than a 10-year return period event.

Figure 5-41 – Location of Rain Gauges within NEXRAD Cells

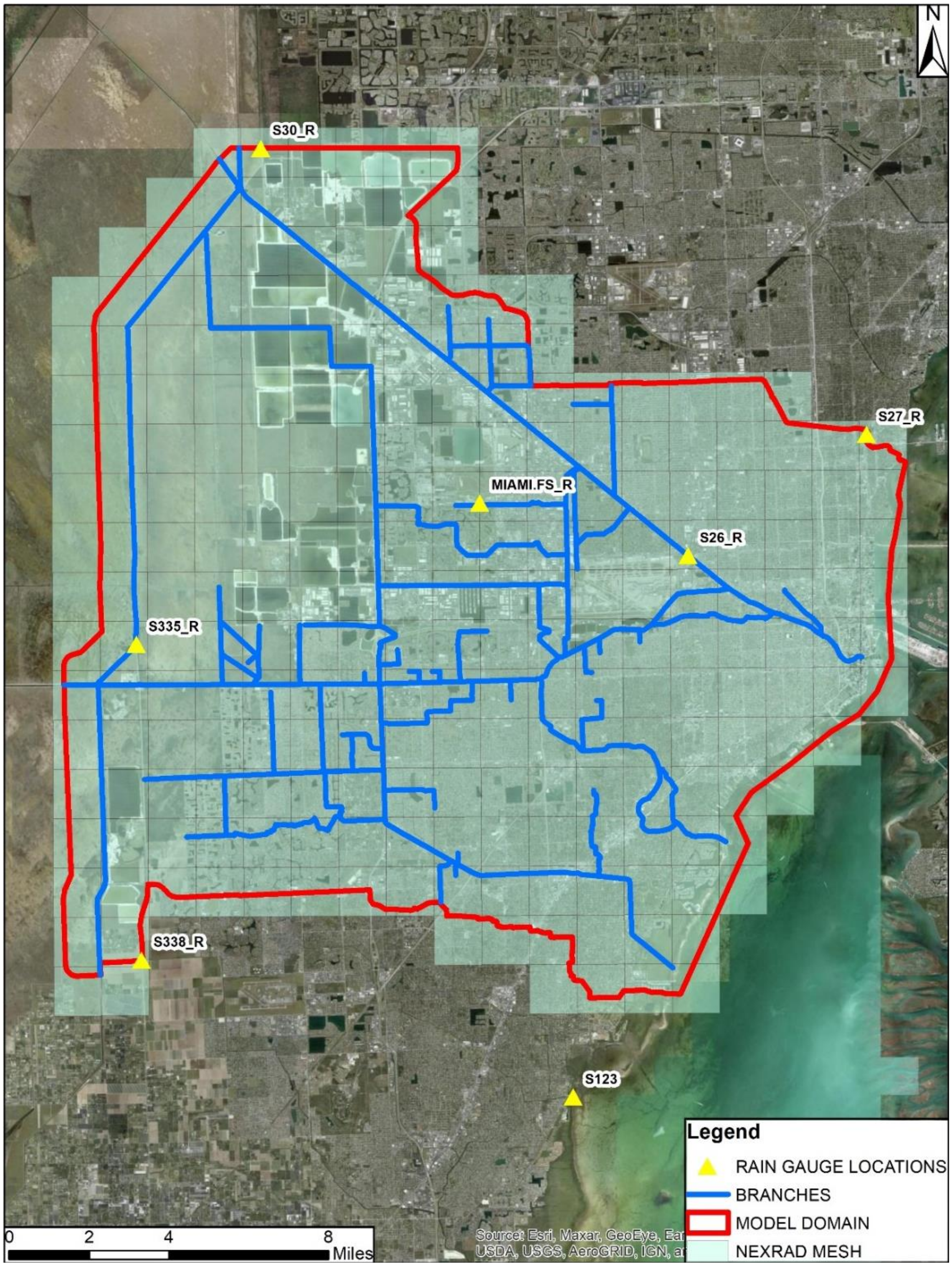


Figure 5-42 – Rainfall Depth Grid for 2017 One-Day Peak

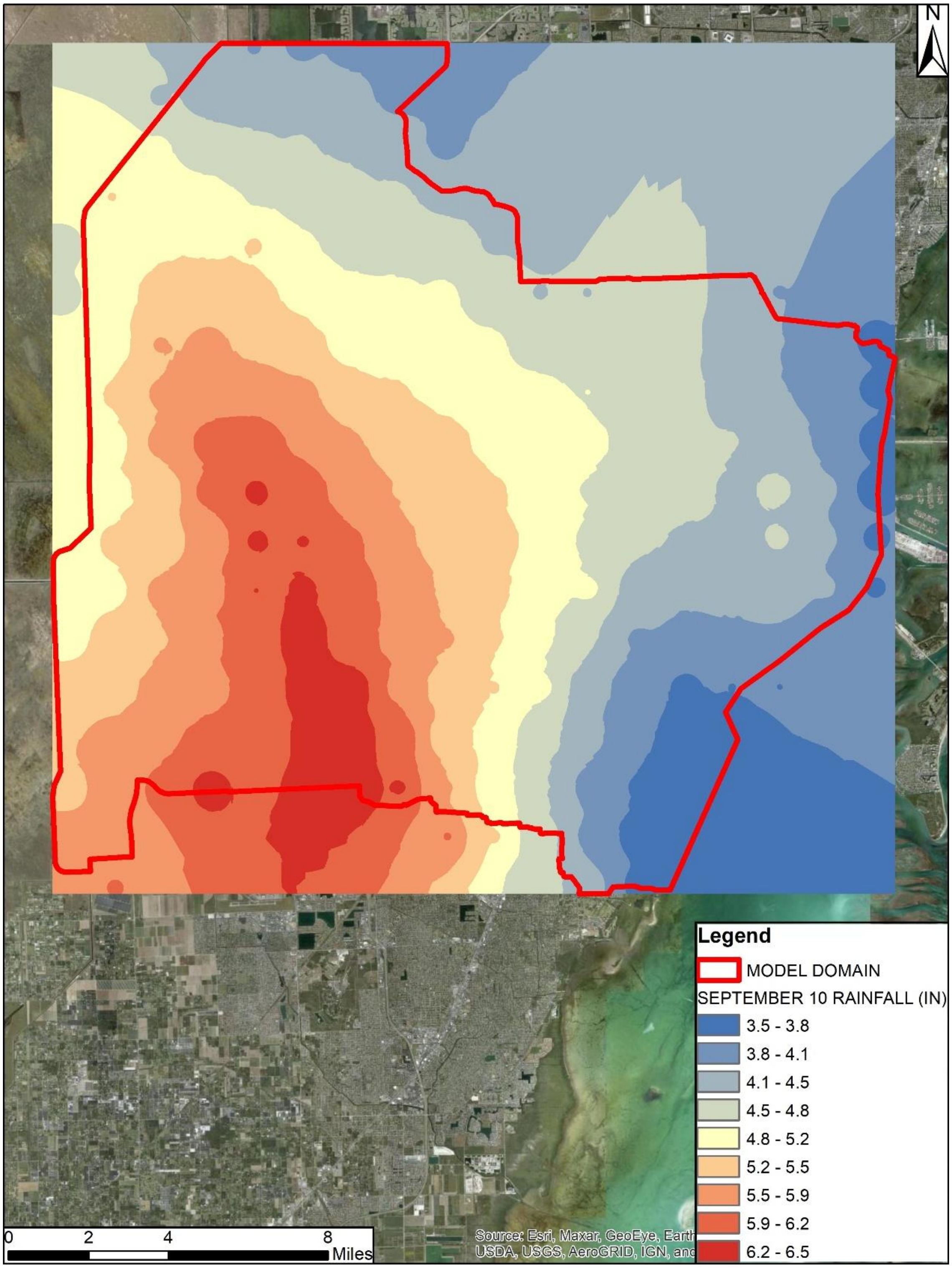


Figure 5-43 – Rainfall Depth Grid for 2020 One-Day Peak

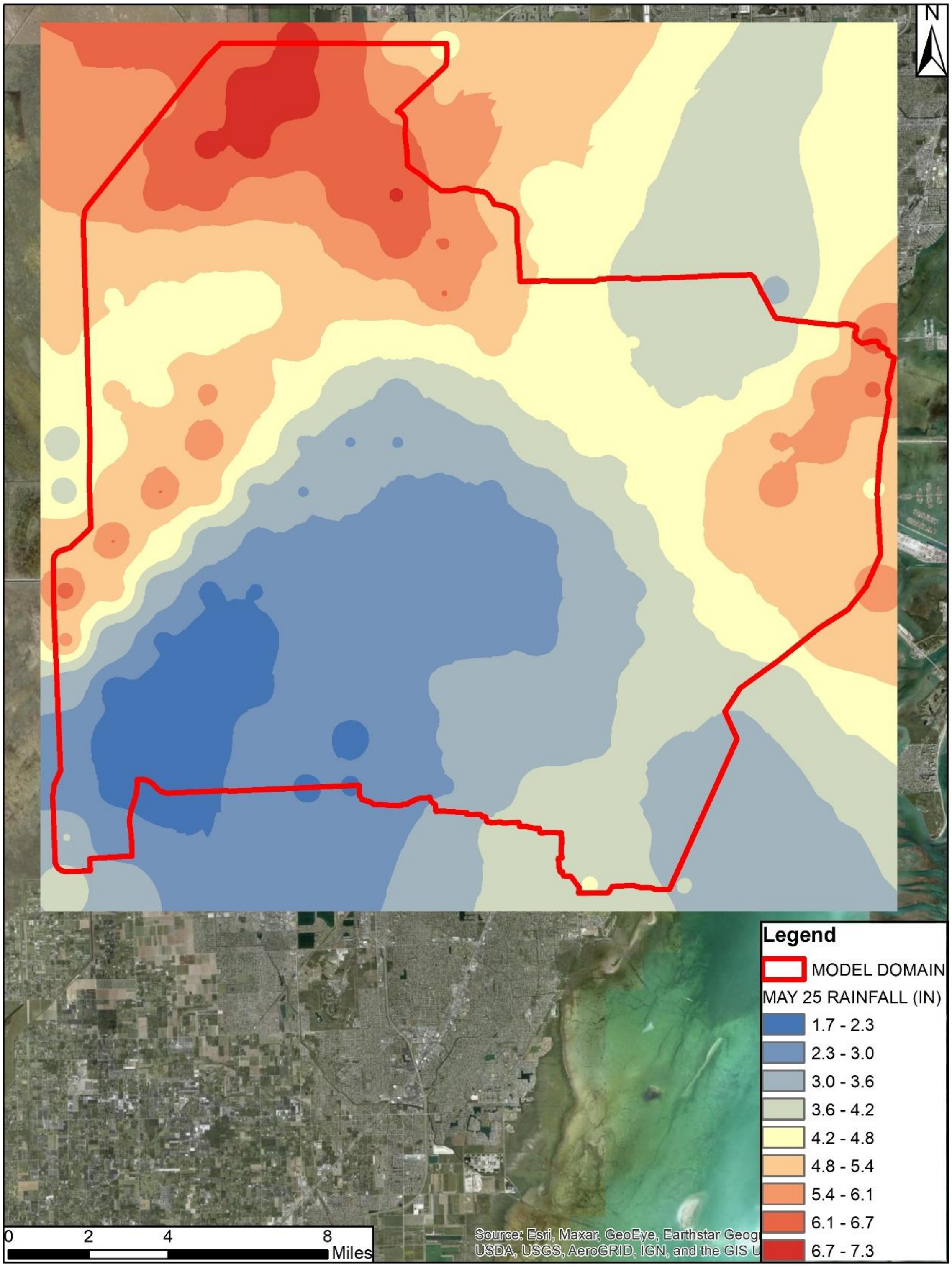


Figure 5-44 – Rainfall Depth Grid for 2017 Three-Day Peak

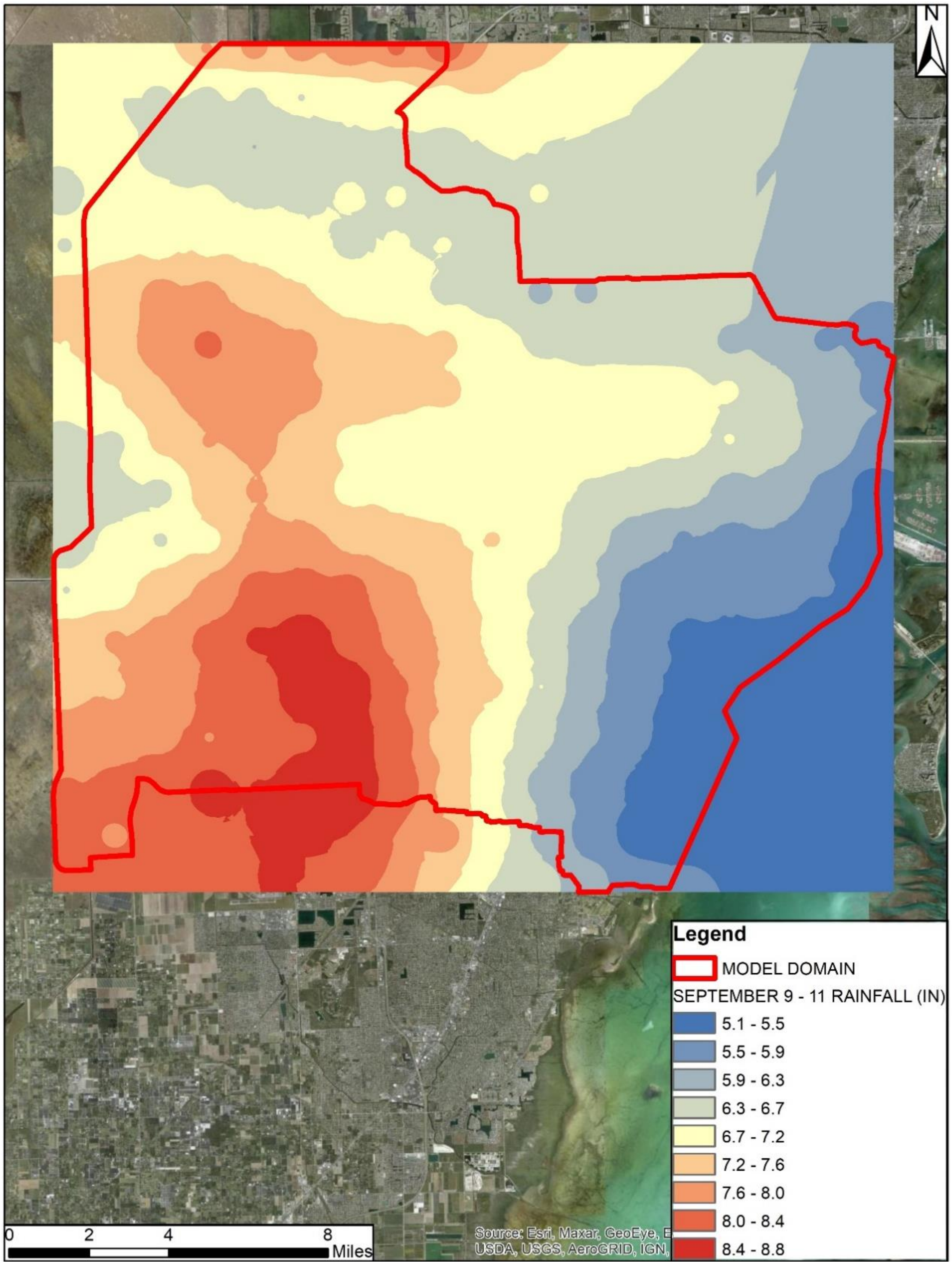
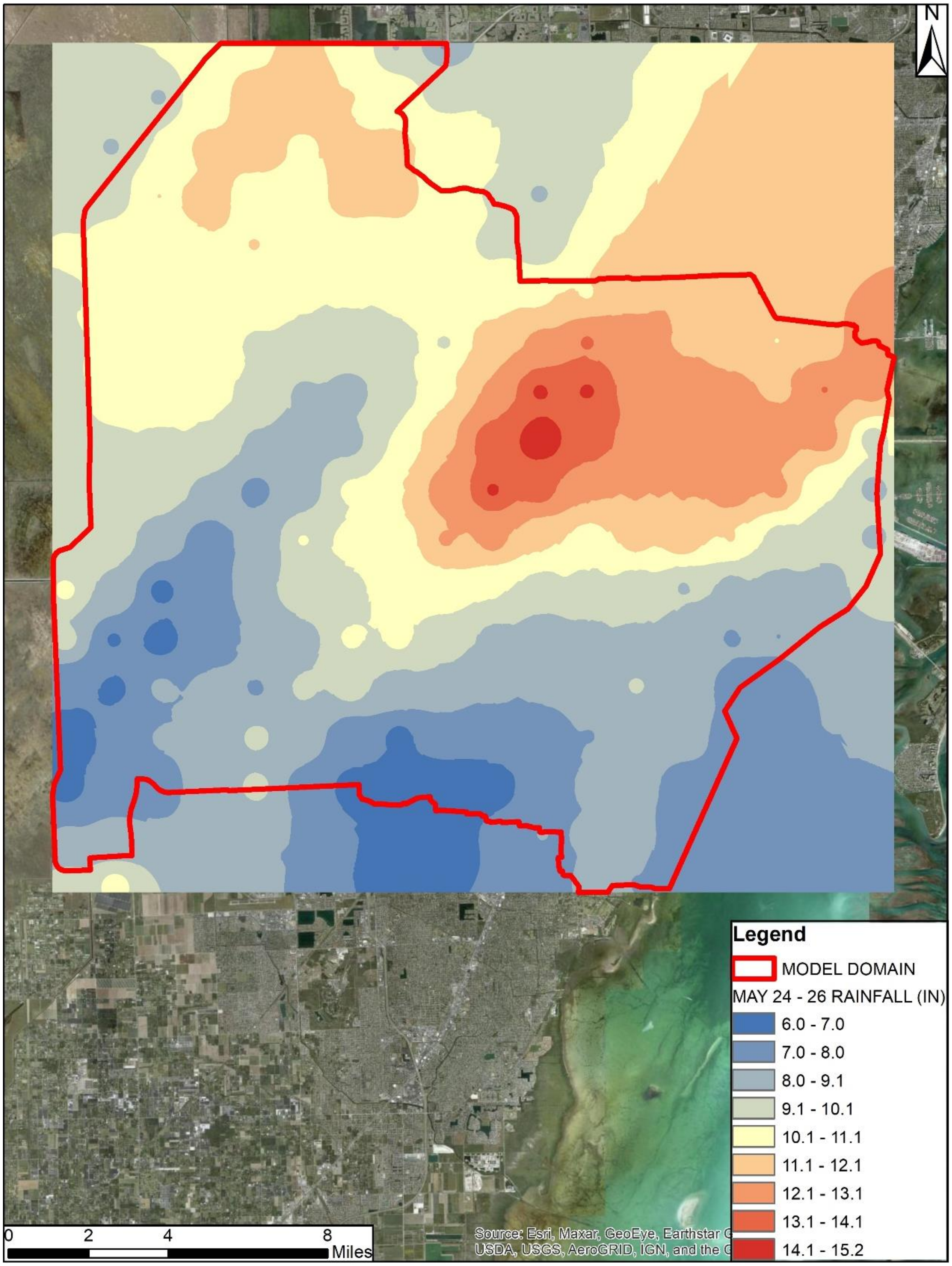


Figure 5-45 – Rainfall Depth Grid for 2020 Three-Day Peak



5.3.2. RAIN GAUGE EVALUATION

As an additional evaluation of using NEXRAD data as a model input, a scenario of the model was developed using the seven rain gauges shown in **Figure 5-41** and applied to the model domain based on Theissen polygons. The simulation was performed for the calibration period and the results of the gauge rainfall model were compared with the calibration results for the NEXRAD rainfall. **Table 5-8** below illustrates a comparison of the results from each scenario and demonstrates that there is little difference between the two simulations for the selected period. The rain gauge value at S380H was abnormally high compared to adjacent rain gauge information, therefore this value was determined to be an outlier and not included in the analysis. Based on this result and the analysis of **Section 5.3.1**, the NEXRAD data was confirmed to be the preferred approach for the calibration effort.

Table 5-8 – NEXRAD versus Rain Gauge Simulation Comparison

Basin	Station	Simulation Period (5/11/2020 - 5/30/2020)	
		Peak Stage (ft NAVD) using NEXRAD Rainfall Data	Peak Stage ft NAVD) using Rain Gauge Data
C2	C2SW1	4.334	4.17
	C2SW2	4.018	3.991
	S22H	3.96	3.928
C3W	G93H	4.015	4.097
C4	C4.Coral	4.137	4.135
	S25BH	3.686	3.712
	S336T	6.011	6.957
	S380H	6.573	12.449*
	S380T	4.148	4.391
	T5W	4.326	4.223
C5	S25H	3.94	3.974
C6	S26H	3.566	3.63
	S31T	3.422	3.541

5.3.3. WELLFIELD DATA REVIEW

The total pumping within each wellfield was compared to ensure proper drawdown was being modeled. SFWMD raised concerns about the accuracy of the pump data as the Northwest wellfield was not showing much groundwater response. Using pump data collected from Miami Dade County, groundwater pumping wells were grouped based on wellfield location as shown in **Figure 5-46** below. **Figure 5-47** illustrates the wellfield locations and nomenclature as an exhibit with demonstrating maximum drawdown potential. The Preston, Hialeah, Miami Springs Upper, and Miami Springs Lower wellfields were combined to represent the Northeast wellfields (NE), and the Alexander Orr and Snapper Creek wellfields were combined to represent the Southeast wellfields (SE). The remaining wellfields were abbreviated such that the Southwest wellfield is represented as SW, the West wellfield is represented as W, and the Northwest wellfield is represented as NW. Notably, in reviewing this evaluation the NW and W wellfields have much lower pumping rates than the other wellfields within the model domain. The effect of the public water supply withdrawals on the simulation results are shown in **Figure 5-48** and **Figure 5-49**. These plots demonstrate the drawdown during a wet and dry period which is most prominent in the Southwest, Northeast, and Southeast wellfields.

Figure 5-46 – Evaluation of Pumping Rates for Calibration Period by Wellfield

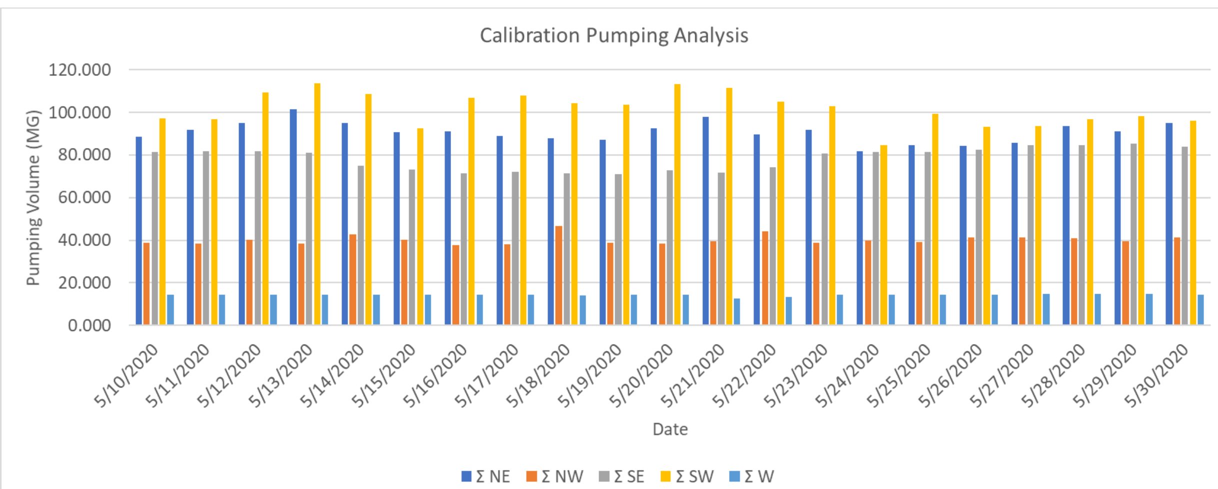


Figure 5-47 – Groundwater Pumping Well Configuration and Approximate Drawdown Area

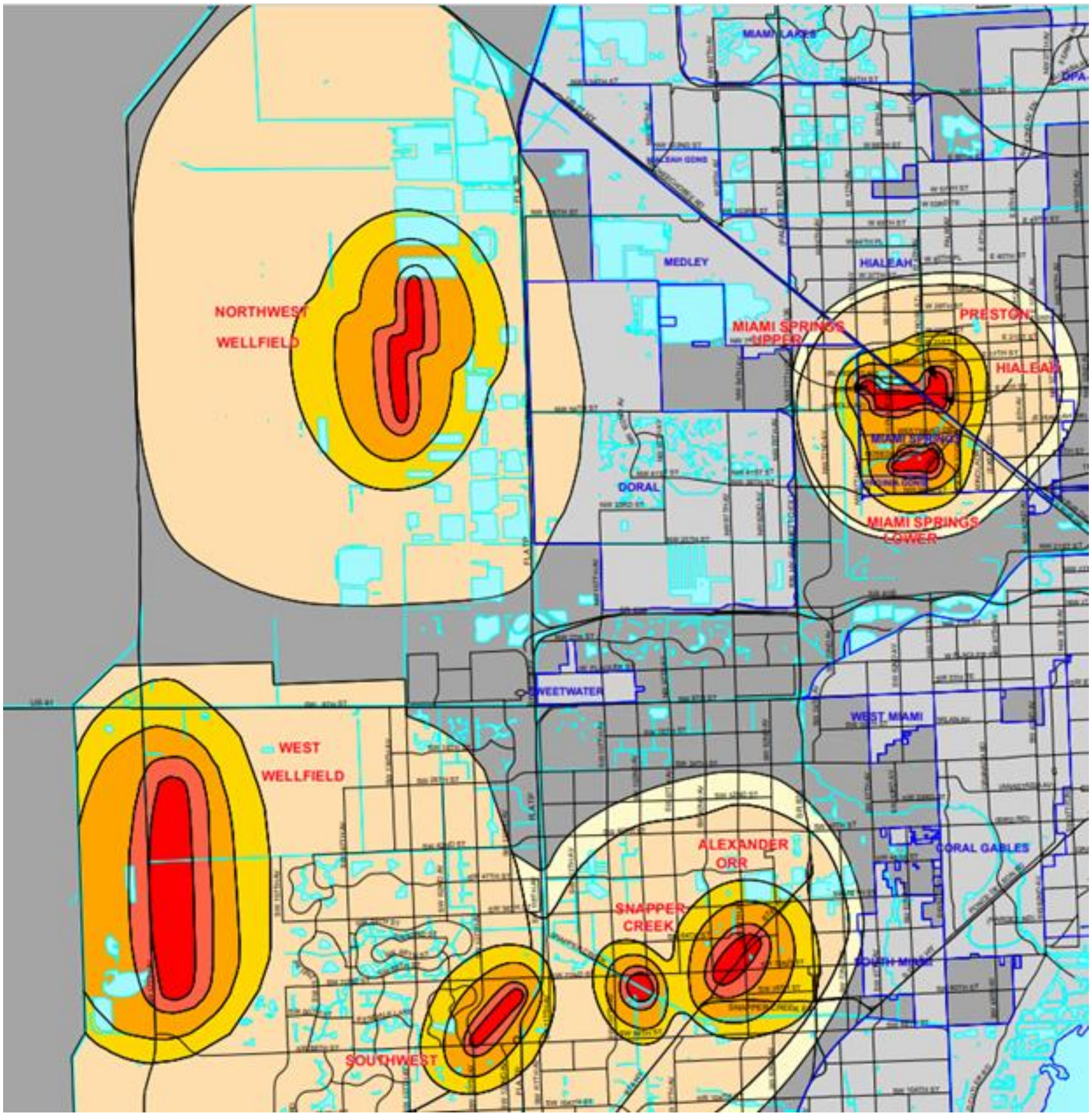


Figure 5-48 – Computed Head Elevation in Saturated Zone, Dry Period

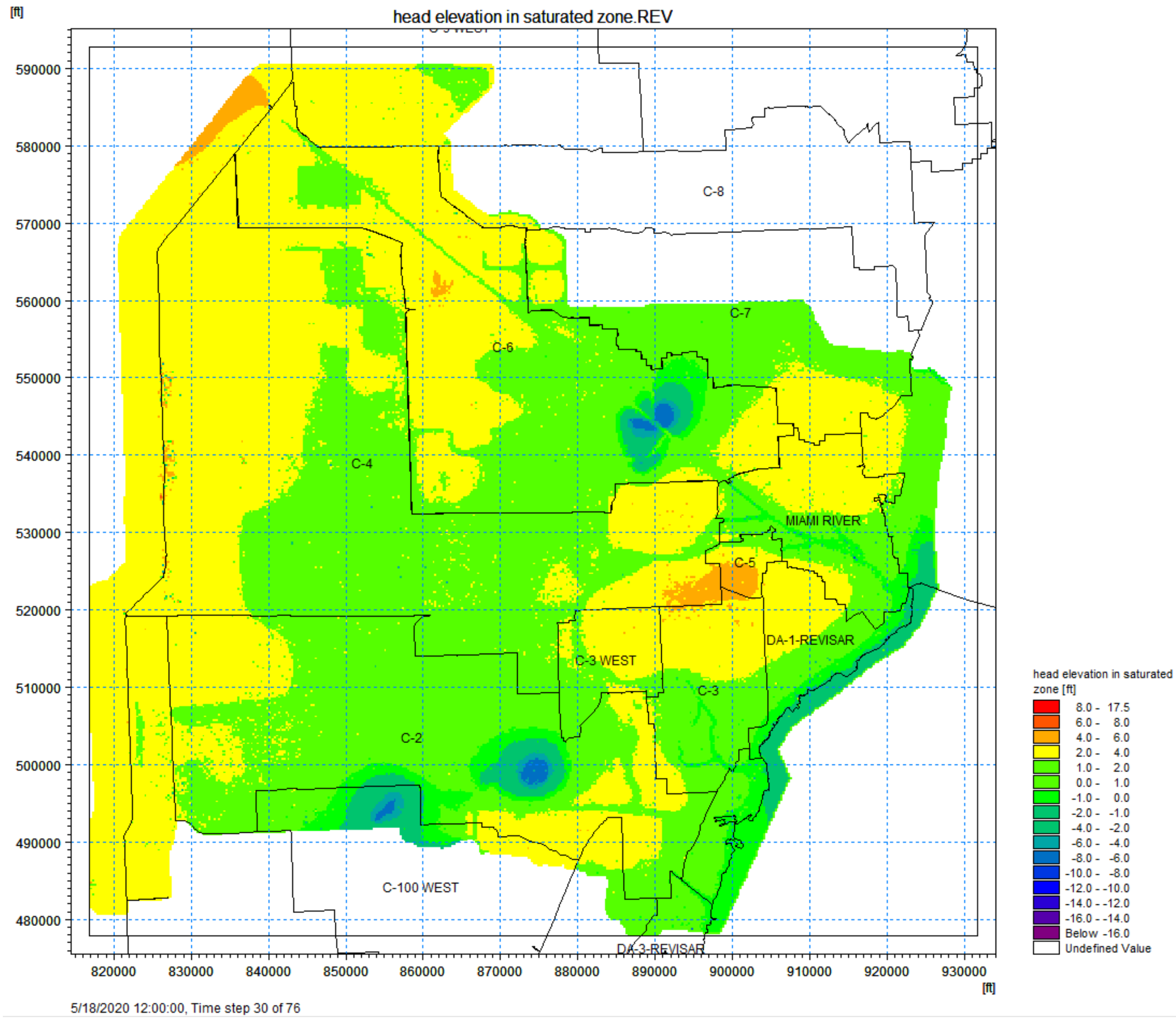
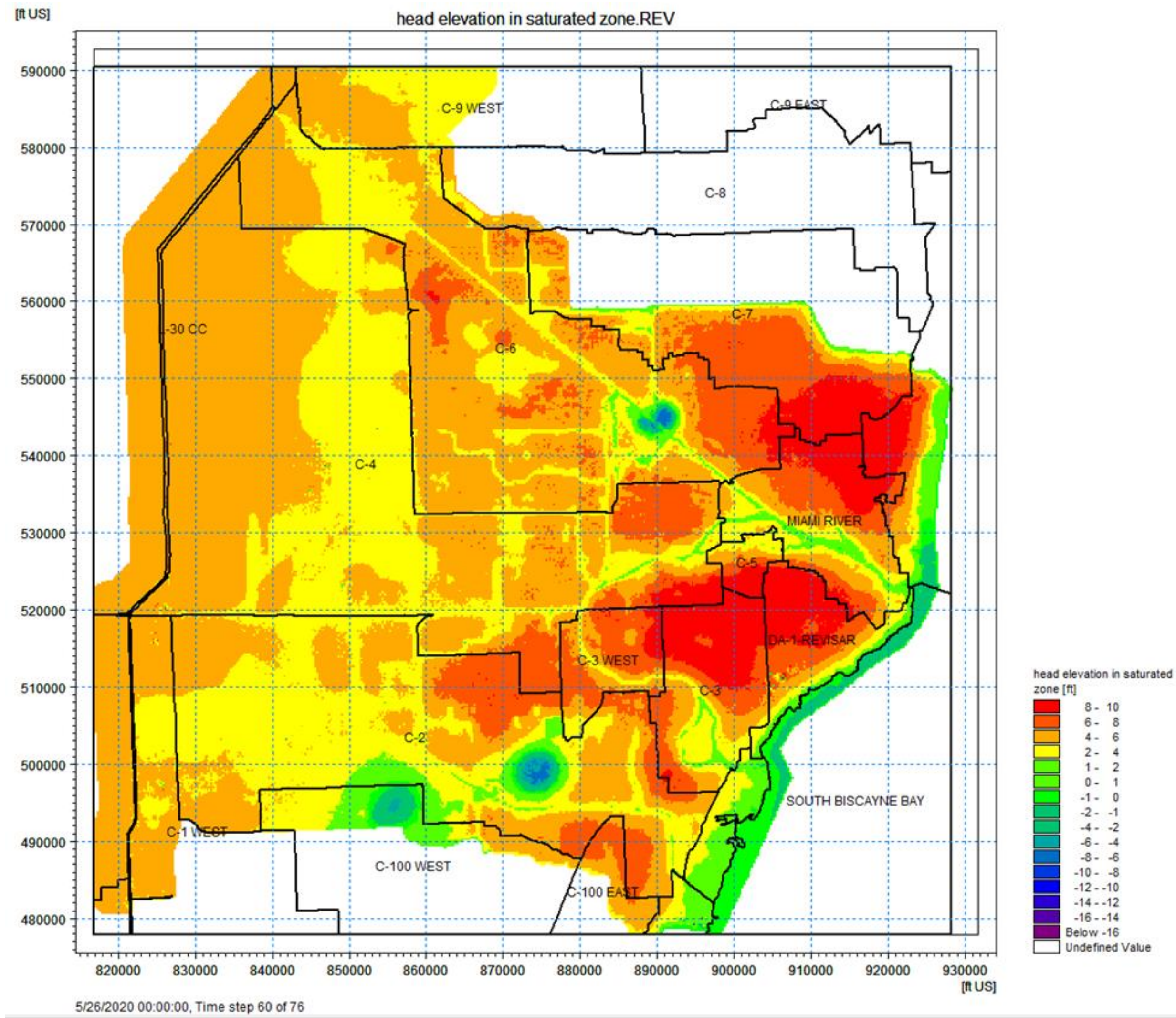


Figure 5-49 – Computed Head Elevation in Saturated Zone, Wet Period



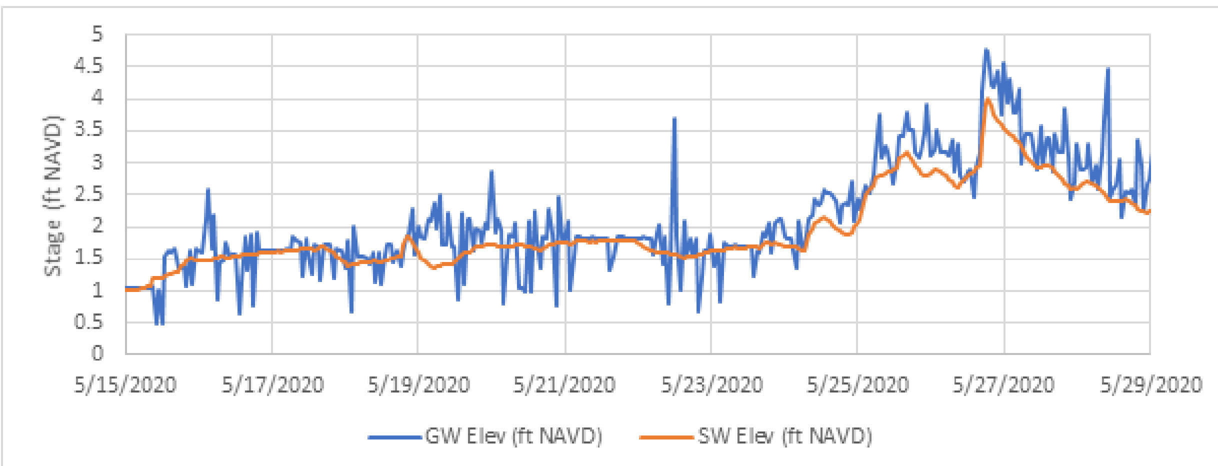
5.3.4. GROUNDWATER / SURFACE WATER INTERACTION TESTING

One of the key elements of hydrologic-hydraulic modeling in Miami Dade County is the interaction of canals (surface water) with the aquifer (groundwater). As demonstrated in **Section 5.1.1.1**, there are two parameterization schemes to calculate the flow of water from the aquifer into the channel: "Aquifer Only" and "Aquifer+Bed". To examine the model performance for each scheme, a series of calculations were performed at discrete locations within the model to compare the water levels in a single SZ cell with stages at a single "h-point" in a canal branch. While this was performed informally at several locations, the following sections show results extracted for the T5W station and its associated groundwater location.

5.3.4.1. AQUIFER ONLY EXAMINATION

Using the Aquifer Only parameterization, the simulated groundwater levels in the grid cell located at the T5W stage monitoring location were compared with the simulated stages at the corresponding canal "h-point", which is the canal reach "C4_Canal" at chainage 8414.1 m. The comparison of water levels to stages demonstrates that the simulated values in the grid cell and canal reach track very closely in magnitude and pattern as shown in **Figure 5-50**.

Figure 5-50 – Aquifer-Only Simulation of Water Level in SZ Cell versus Stage at H Point in Canal



In order to estimate the flow from the grid cell to the canal reach for the "Aquifer Only" mechanism, the calculation is based on the following equation:

$$Q = C \times \Delta h$$

Where ∆h is the head difference across the canal-aquifer interface and C is the conductance as defined by the following equation:

$$C = \frac{K \times da \times dx}{ds}$$

The components of the conductance equation include the parameters defined below and illustrated in **Figure 5-51**.

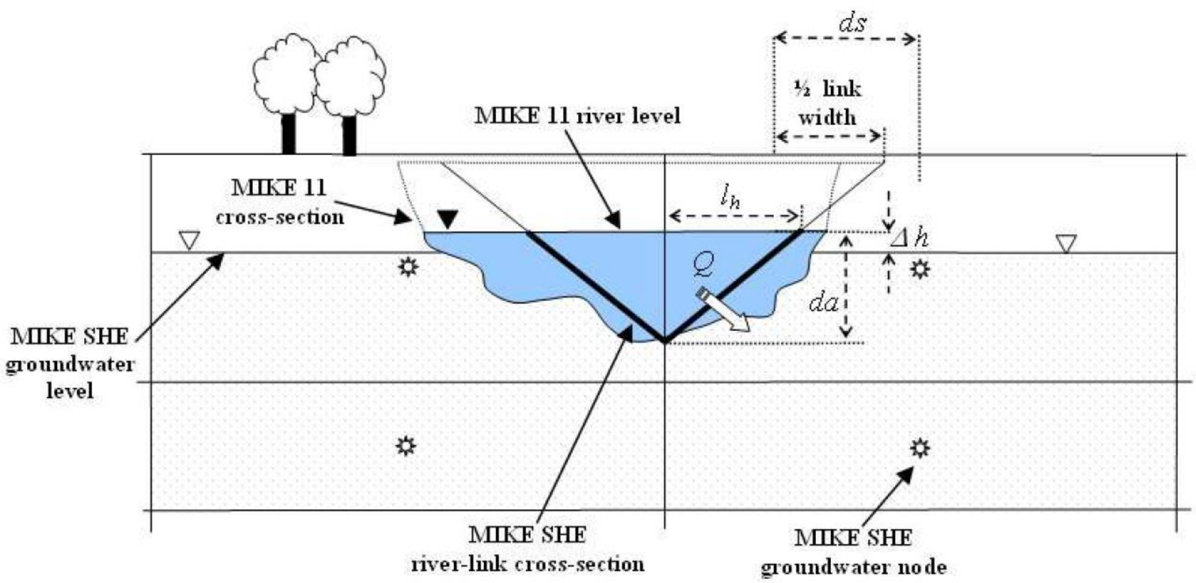
K = Horizontal Conductivity in the SZ cell

da = vertical surface available for exchange flow

dx = grid cell size

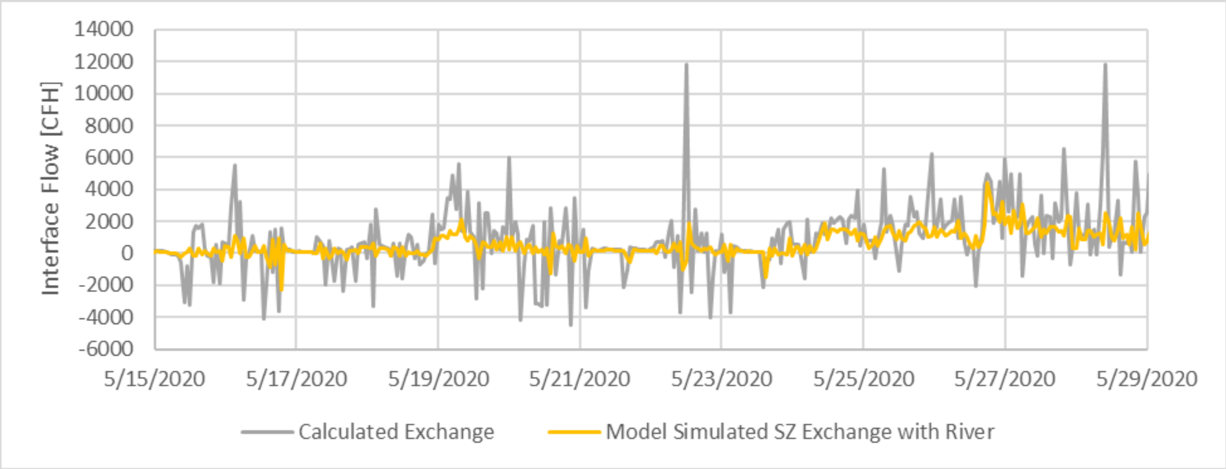
ds = average flow length or distance from the grid node to the middle of the riverbank

Figure 5-51 – Configuration of Typical Canal Geometry for Calculating Interface Flow



To calculate the interface flow, the values for “da” and “ds” have to be estimated at each timestep since they vary with the stage in the canal. Using an estimating approach for “da” and “ds” based on the canal cross-section, the interface flow was calculated separately from the model on an hourly basis in units of cubic feet per hour. The interface flow estimate was compared with the interface flow as defined by the model simulation in **Figure 5-52**. Since the calculated flow was based on estimated parameters, the comparison was not a one-to-one match, but the general trends were very similar.

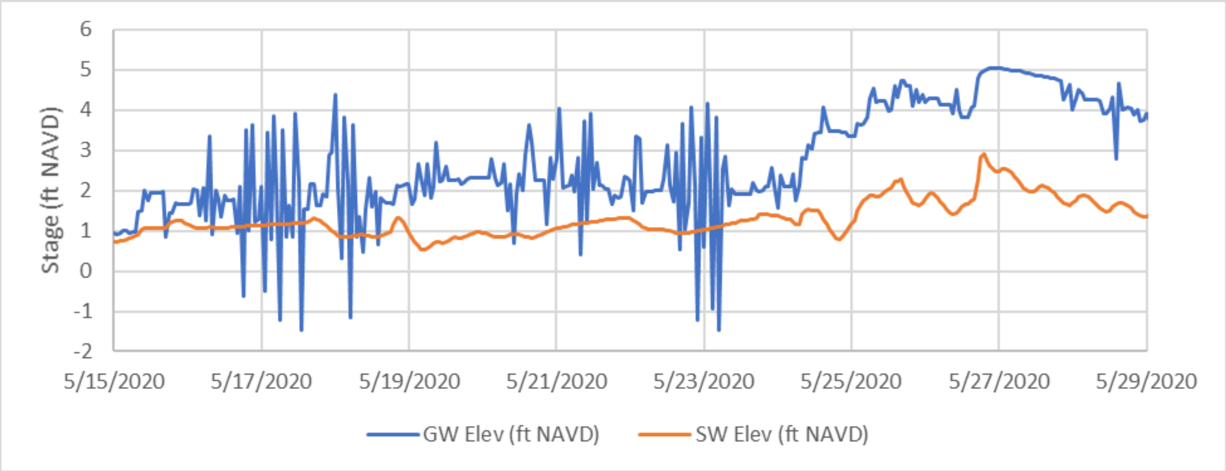
Figure 5-52 – Aquifer-Only Simulated Lateral Flow at Interface of SZ Cell and Canal Reach



5.3.4.2. AQUIFER+BED EXAMINATION

As a point of contrast, a similar comparison of groundwater levels in the grid cell with surface water stages in the canal was performed using the Aquifer+Bed parameterization at the same location. **Figure 5-53** illustrates that the water levels in the grid cell are very different than the stages in the canal if flow across the interface between the aquifer and the canal is limited by bed conductance.

Figure 5-53 – Aquifer + Bed Simulation of Water Level in SZ Cell versus Stage at H Point in Canal



To estimate the flow from the grid cell to the canal reach for the "Aquifer+Bed" mechanism, the calculation is based on the same equation, except that the conductance (C) value is calculated using the following equation:

$$C = \frac{1}{\frac{ds}{K \times da \times dx} + \frac{1}{Lc \times w \times dx}}$$

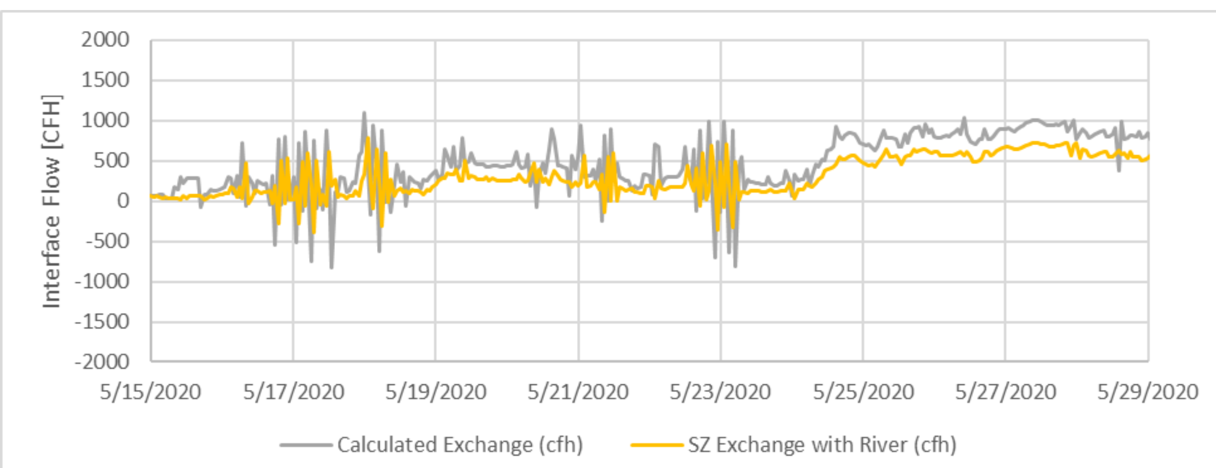
Where the parameter definitions match the "Aquifer Only" approach and the additional parameters are defined as follows:

Lc = Leakage coefficient of the bed material

W = wetted perimeter of the canal cross-section

Similar to the Aquifer-Only calculation, an estimating approach was used for "da", "ds" and "w" based on the canal cross-section and simulated stage on an hourly basis. Using these values an estimate for interface flow was calculated on an hourly basis in terms of cubic-feet and compared with the interface flow as defined by the model simulation. **Figure 5-54** illustrates the comparison of simulated interface flow and calculated flow based on estimated parameters. Although the comparison was not a one-to-one match, the general trends were very similar and noticeably less than the flow calculated or simulated using the "Aquifer Only" approach.

Figure 5-54 – Aquifer + Bed Simulated Lateral Flow at Interface of SZ Cell and Canal Reach



The additional investigation into the model's groundwater-surface water interaction processes demonstrates that the "Aquifer Only" parameterization provides significantly larger flows between the SZ grid cells and canal reaches. The simulated flows across the interface appear to utilize the physics stated in model documentation with the discrepancies between estimated and simulated values attributable discrepancies in geometric parameters calculated at higher resolution time steps. For the purposes of this FPLOS model, the "Aquifer Only" approach was the best fit to ensure that flows from the groundwater into the canals were maximized in a manner that matched field observation and calibration simulations. The final calibration and validation runs used Aquifer Only for the majority of the canals with the exception of the Snapper Creek Ext, Bird Drive Extension, L31N and L33, which used Aquifer + Bed.

5.4. REFINED CALIBRATION AND VALIDATION

After the additional evaluation and based on input from District Staff, a final refinement was performed of the parameters of the calibrated model. Although there were a few minor changes made to specific parameters such as increasing the pump speed at forward pumping stations to account for maximum discharge rates, there were six types of refinements that were made to the entire model domain. A description of each of those modifications is described below.

5.4.1. ALLOW OVERBANK SPILLING

For each canal reach in the MIKE 1D model, there is an option to allow "Overbank Spilling" which provides a mechanism to route water from the reach onto the overland surface. For the calibration and validation simulations, there were no instances where the simulated water surface elevation exceeded the top of bank so there was no effect of this parameter setting. However, for extreme design events and future case scenarios, it is anticipated that there will be instances of bank overtopping that should be represented. For verification purposes, a sensitivity test was performed with Overbank Spilling turned on and no major changes were found other than a minor increase in the computational run time.

5.4.2. COMPUTATIONAL TIMESTEPS

To improve on instabilities in groundwater results in locations closer to public water supply withdrawals and in canal stage results in locations where gate openings or head differences across a structure are small, the computational timestep was decreased. The timestep was reduced to six minutes for the Model Initial, Overland, Unsaturated Zone, and Saturated Zone modules. This improved instabilities in many areas and provided some improvements to calibration. The only concern was the additional computation increased model run times by almost 5 hours.

5.4.3. REDUCING OVERLAND STABILITY PARAMETERS

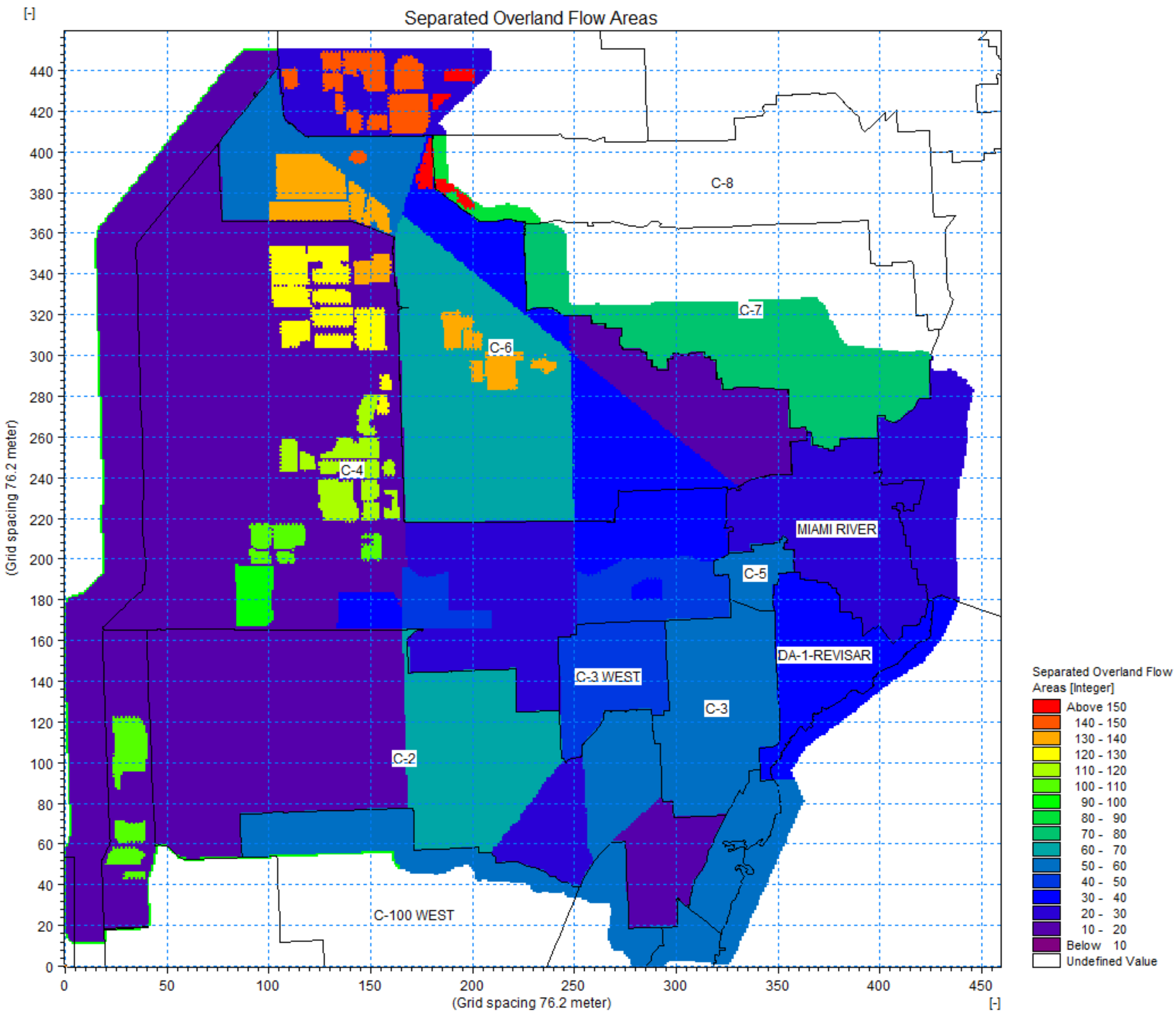
For the overland flow module, there are computational settings for the threshold water depth and threshold gradient that are especially applicable to low gradient regions such as Miami Dade County. The threshold depth is the minimum depth of water on the ground surface before overland flow is calculated, and often shallow depth settings will lead to numerical instabilities. Of note, the threshold depth is not related to detention storage which separately accounts for local storage such as would be found in a detention or retention facility associated with a surface water management system. The threshold gradient is used to prevent instabilities in flat areas between overland cells at the same elevation by using a damping function to essentially increase the resistance to flow between cells.

For the refined model calibration, the threshold depth was reduced to 0.0328 feet (0.01 meter) to promote runoff consistent with wet season conditions and flooding events. The threshold gradient was reduced to 0.005. A sensitivity test was performed of a lower threshold gradient of 0.001, but the result was a significant increase in model run times that was unrealistic for further simulations.

5.4.4. SEPARATED FLOW AREAS

Separated flow areas prevent overland flow from being routed to the closest reach based on distance and gradient. Instead, the boundaries of the separated flow area force runoff within a specified area to be routed only to reaches within that area similar to a subbasin or catchment delineation. Within the model domain, there are locations where surface water management systems or major roadways create boundaries that overland flow should not cross. During model development, an early approach utilized the Miami Dade County XPSWMM model subbasins to define hundreds of separated flow areas. This approach created unnecessary computational complexity and limited the model’s two-dimensional capabilities to recreate natural runoff processes. The initial and revised calibration assumed minimal separated flow areas which created situations where overland runoff was being misattributed to various reaches across watershed boundaries. The final recommendation was a hybrid version of separated flow areas that included watershed boundaries as well as some sub-basin boundaries, as illustrated in **Figure 5-55**.

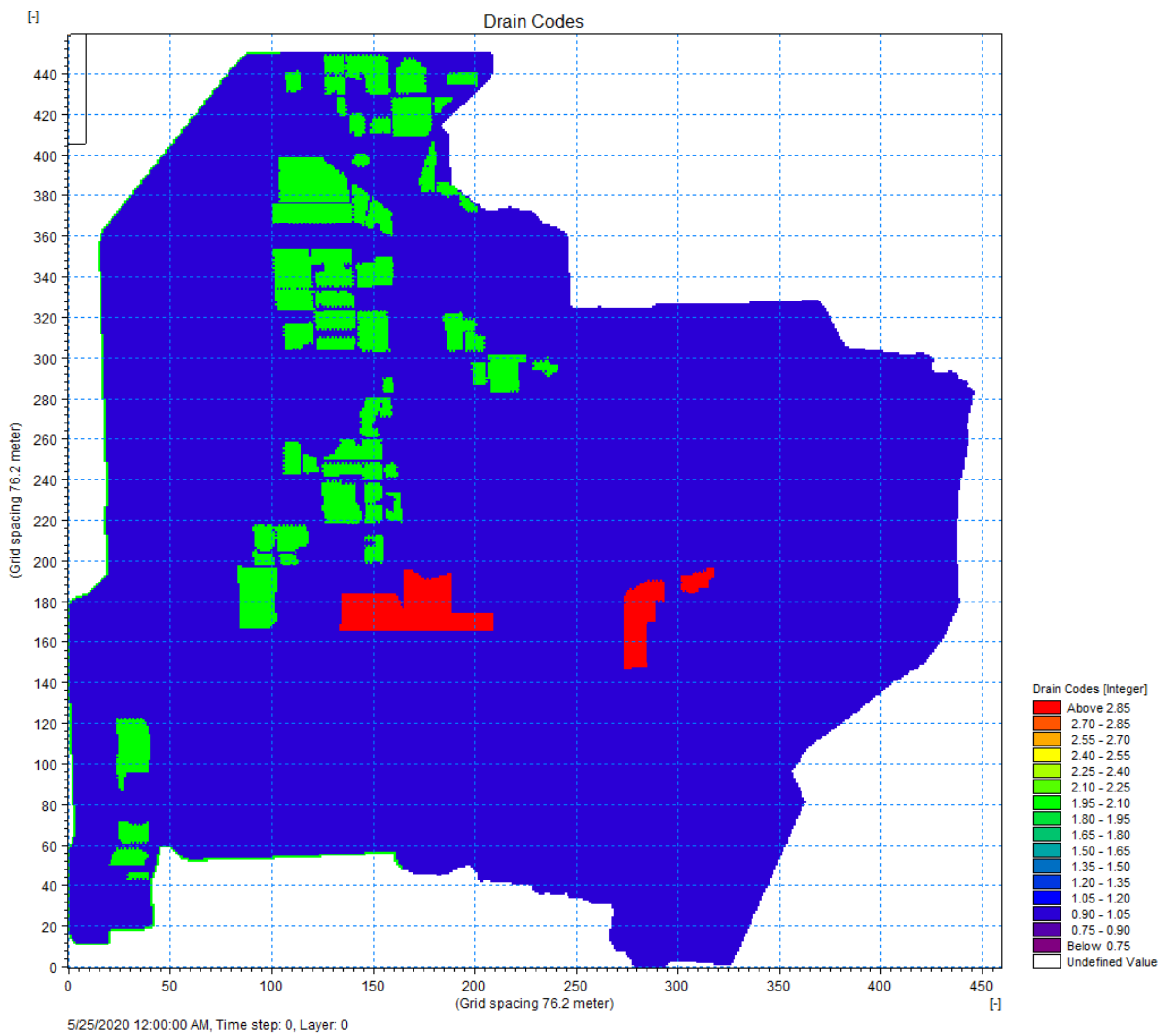
Figure 5-55 – Recommended Separated Flow Areas for Refined Calibration



5.4.5. DRAIN CODES

Similar to the function of separated flow areas, drain codes define the contributing area for a given reach to route rainfall from the land-surface to the canal. Unlike separated flow areas, drain codes are applied to the saturated zone and not to overland flow, meaning that water eligible to be routed by drain codes has infiltrated the soil column, but exceeds the expected drain level of the local surface water management system that manages the surficial water table. Through sensitivity testing, drain codes are more likely to affect simulated groundwater elevation than flooding and can also impact canal stages based on the volume of water delivered to the downstream reach. The initial and revised calibration assumed drain codes only for the locations where municipal pump stations were used for flood mitigation. Based on a series of sensitivity testing scenarios to determine the most effective balance of routing water from the SZ to adjacent reaches, the final recommendation is illustrated in **Figure 5-56**.

Figure 5-56 – Recommended Drain Codes for Refined Calibration



5.4.6. DOWNSTREAM BOUNDARY CONDITIONS

For the initial and revised calibration simulations, the hydraulic boundary condition for all primary canals was based on the MRMS4 timeseries of measured stages near the mouth of the Miami River. It was assumed this was the best approximation available of Biscayne Bay stages and could be applied at the mouth of each primary canal. However, further examination of the measured data at the tailwater of each salinity structure demonstrated that the expected stages in Biscayne Bay differ in the northern portion of the model domain than in the southern portion of the model domain.

The approach used to define the tidal boundaries at Biscayne Bay for each watershed was to analyze projected data from structures that are closest to Biscayne Bay at either end of the coastline of the model domain and prepare an interpolated hydrograph for each outlet based on the distance between the north and south data points. S-22 is the southernmost structure, located near Biscayne Bay approximately 1.43 miles upstream on the C-2 Canal or Snapper Creek. S-27 is the northernmost structure, located near Biscayne Bay approximately 1.2 miles upstream on the C-7 Canal or Little River. Using distance weighted interpolation, the tidal boundary at the mouth of the Miami River (which includes the S-25, S-25B and S-26 structures) as well as the Coral Gables Canal (which includes the G-93 structure) was developed for the calibration and validation simulations.

5.4.7. CALIBRATION SIMULATION

Based on the results of the sensitivity analysis, there were no changes made to the initial calibration parameterization except for the municipal pump operations where the West Miami pump station was assumed to be off. The values used for the following parameters were consistent with other regional models as described **Section 5.1** of this document.

- K_H and K_V in Saturated Zone (SZ) model
- Manning’s roughness for primary and secondary canals
- Detention storage for Overland (OL) model
- Manning’s roughness for Overland (OL) model
- Drainage level in Saturated Zone (SZ) model

The subsections below illustrate the results of the calibration simulation at various points within the model from a graphical and statistical perspective to demonstrate the capability of the developed model to represent the natural system.

5.4.7.1. GRAPHICAL COMPARISON BY WATERSHED

Figure 5-57 through **Figure 5-92** illustrate the performance of the model with respect to simulated stage, flow, and groundwater elevation within the model domain. For each figure, the simulated stage is represented by an orange line, while the measured stage is represented by a blue line. A visual review of the results illustrated that there was good general agreement between the simulated and measured data with respect to the patterns and the magnitude of the runoff response at most calibration locations. A statistical comparison of the performance of the model is included in **Section 5.4.7.2**.

For FPLOS modeling, the representation of peak stages is a critical measure of the model's utility. As shown in the results that follow, the comparisons of simulated and measured stage demonstrate good agreement especially during the peak rainfall period. When comparing flow results, the calculation of discharge by the model utilizes the latest flow rating data to define the empirical coefficients that relate headwater and tailwater stages to flow through a gate opening of a known size, as noted in **Section 5.1.1.6**. Considering that the easternmost spillways have tailwater conditions that are largely defined by the tidal boundary and considering the gate opening data at any moment is known based on District records, the main driver of flow estimation in the model is the simulation of headwater stage. As such, where there is good agreement between the simulated and measured headwater stage, there is expected to be good agreement for simulated flow. The visual comparison of the results for the calibration demonstrates that the model represents the runoff response and discharges with reasonable accuracy.

The results also demonstrate the performance of the Saturated Zone model with respect to simulating groundwater elevation at various locations where monitoring wells are located. For the graphical comparisons shown below, a third line was added to the plots in green to illustrate the daily maximum elevation simulated by the model. The daily max is included to provide a comparison with DBHYDRO data that is typically recorded in the daily maximum format. As demonstrated in the visual comparison of results, there is some discrepancy at various locations, such as G-3570 and G-3563, with respect to the magnitude of the increase in groundwater level due to rainfall response. In these locations the measured drawdown after the storm was also not well simulated within the model. There were other locations within the model where the groundwater response was a good representation of what has been measured in the natural system, such as G-3570 and G-3563. Based on the sensitivity analyses and additional research described earlier in this report, there was no clear indication of a system-wide parameter change that would resolve the localized discrepancies without impacting the locations where the simulation matched well. To maintain model-wide consistency and not incorporate localized parameter modifications, no additional adjustments were made to achieve a better fit of the simulated to measured data.

The graphical results presented below are grouped by watershed in sequential order from C2 to C6. For reference of the location of calibration points, see **Figure 5-1** through **Figure 5-5**.

5.4.7.1.1. C2 WATERSHED – CANAL STAGE

The figures below demonstrate good agreement between the timing and magnitude at all three locations where canal stages are measured within the C2 watershed. At each location the performance of the model during the peak rainfall response is more accurate than in the initialization and warmup period. This may reflect that the parameterization of the model is more representative of the hydrology and hydrogeology of the domain during wet periods than during dry periods with minimal rainfall.

Figure 5-57 – Calibration Model Canal Stage – C2SW1

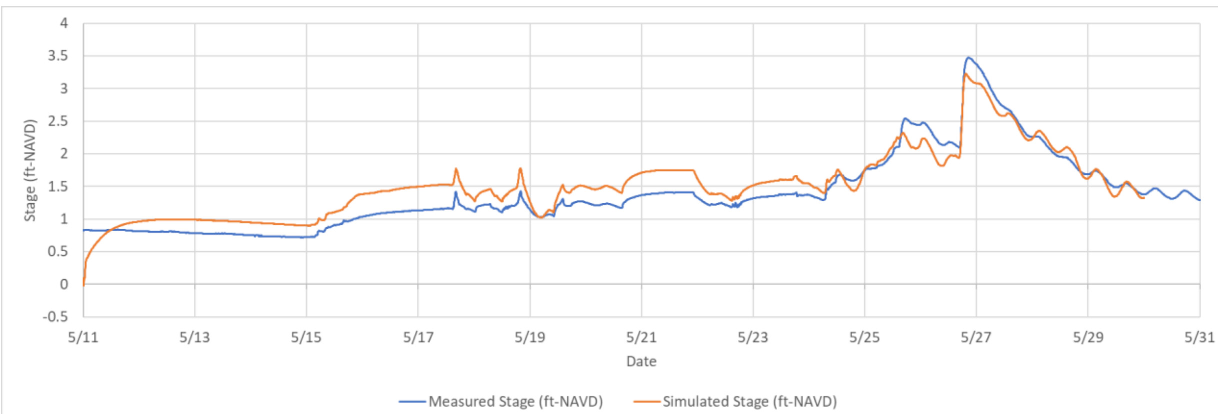


Figure 5-58 – Calibration Model Canal Stage – C2SW2

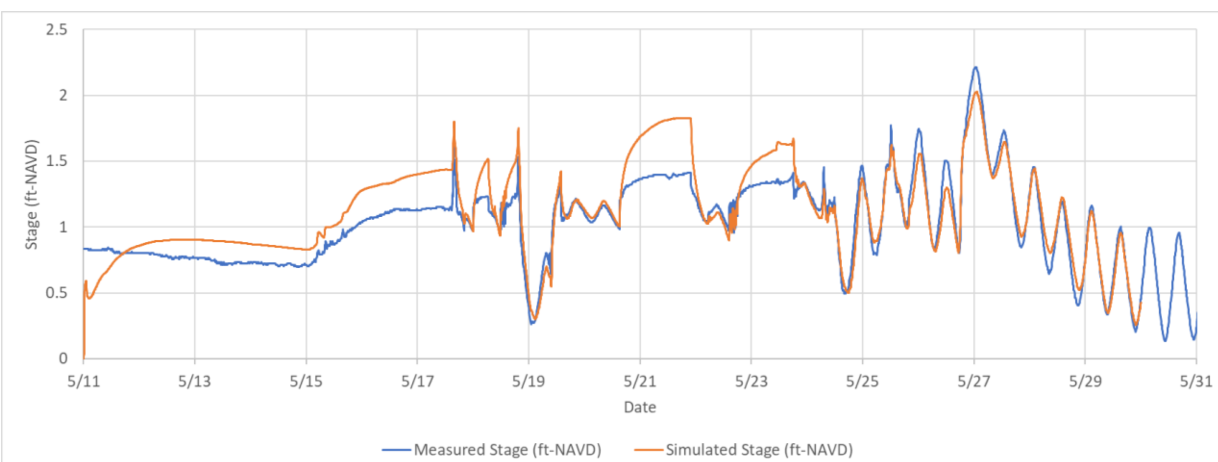
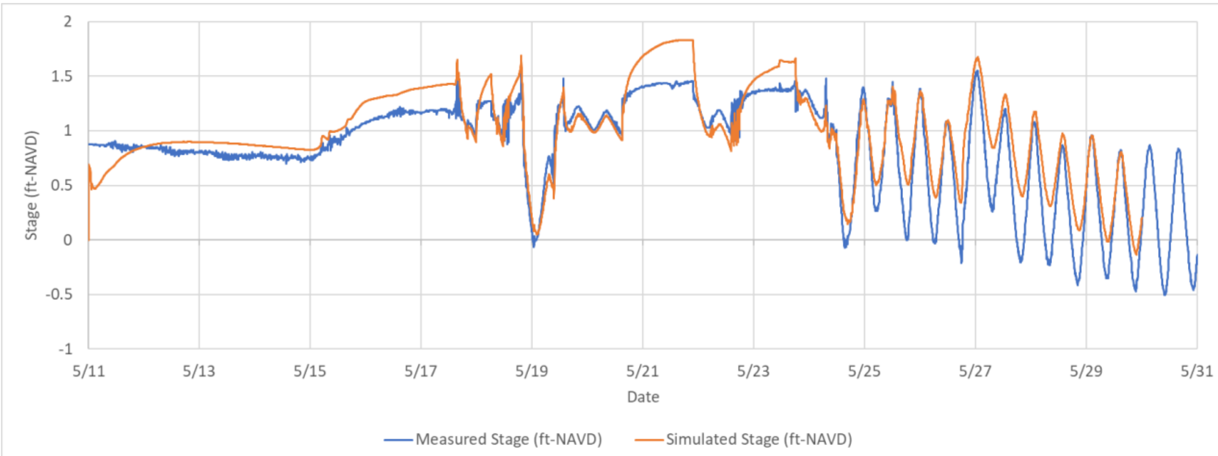


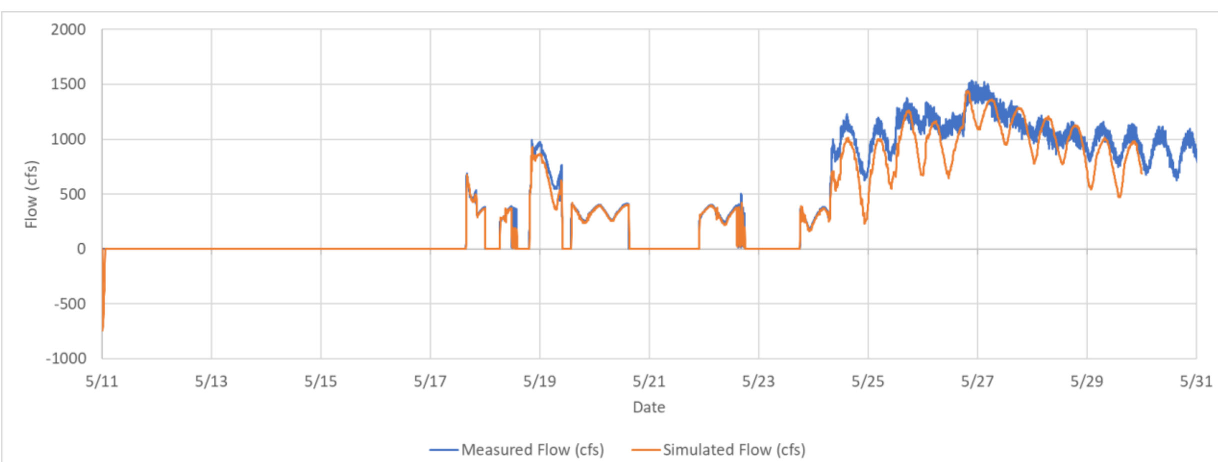
Figure 5-59 – Calibration Model Canal Stage – S22-H



5.4.7.1.2. C2 WATERSHED – CANAL FLOW

The simulation results compare favorably at the S22 spillway. Considering the proximity to a measured boundary condition, the accuracy of the simulated discharge was largely determined by the model's accuracy in predicting headwater stage since the gate geometry and operations are known parameters during the calibration period.

Figure 5-60 – Calibration Model Canal Flow – S22-Q



5.4.7.1.3. C2 WATERSHED – GROUNDWATER STAGE

The groundwater results for G-3558 and G-3565 showed very good agreement with respect to the pattern and magnitude of the simulation response, whereas G-3563 and G3572 were less accurate with an overestimation of the increase in groundwater elevation in response to the rainfall infiltrating to the Saturated Zone. In the case of G-3563, the simulation had an insufficient recession after the simulated peak. With respect to location, G-3563 was the only monitoring well east of the C2 Canal and may not have had sufficient recession due to the distance from boundary conditions or the primary canal network which could provide additional drainage.

Figure 5-61 – Calibration Model Groundwater Stage – G-3558

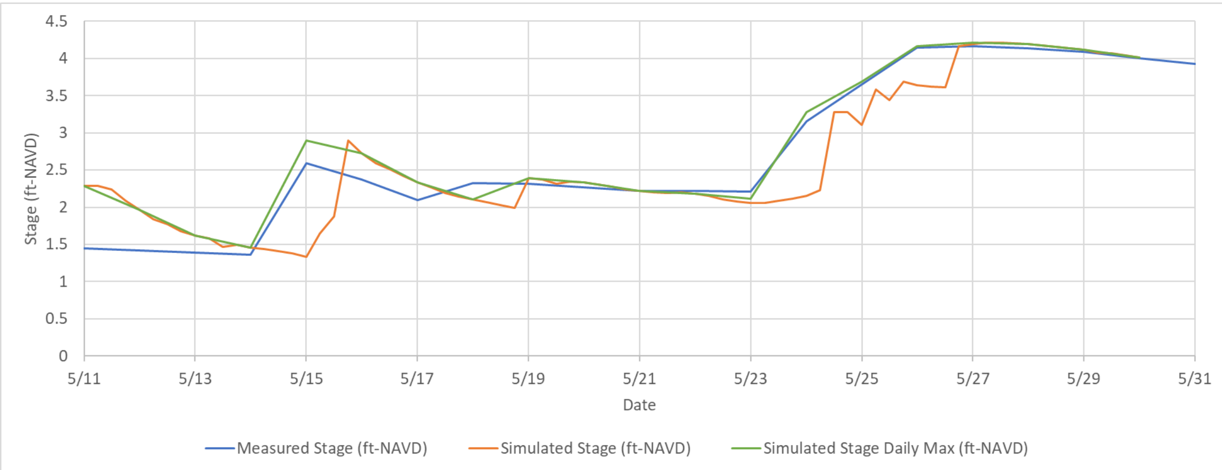


Figure 5-62 – Calibration Model Groundwater Stage – G-3563

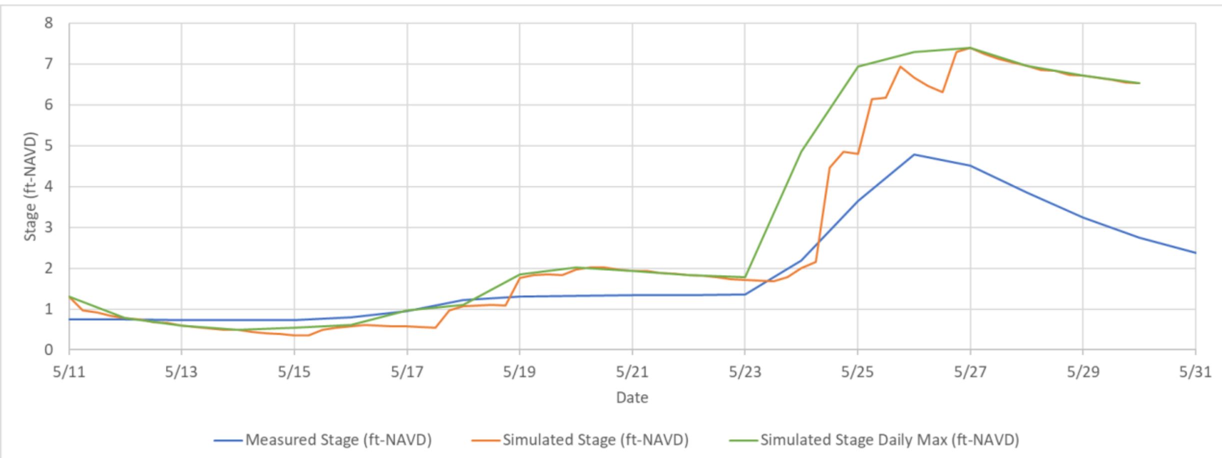


Figure 5-63 – Calibration Model Groundwater Stage – G-3572

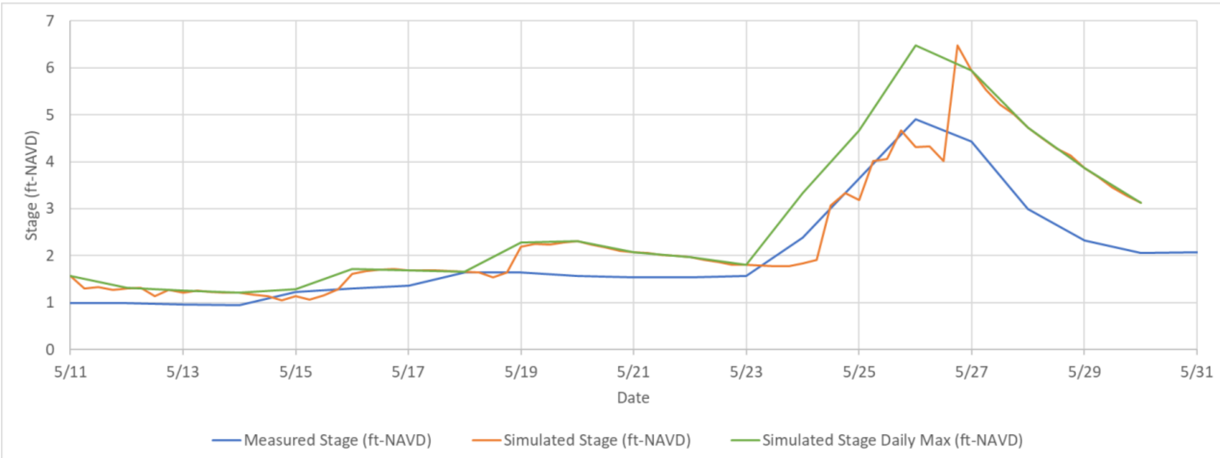
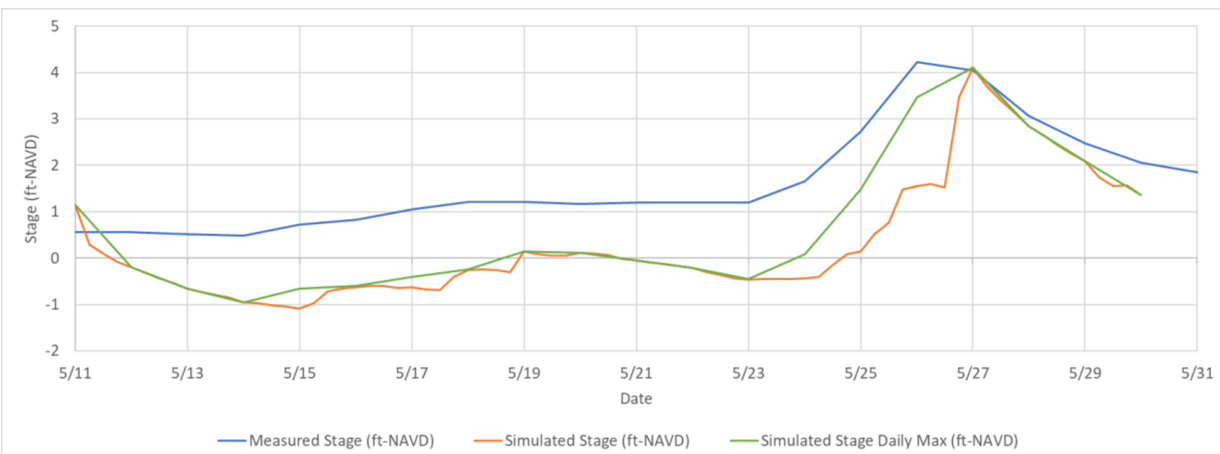


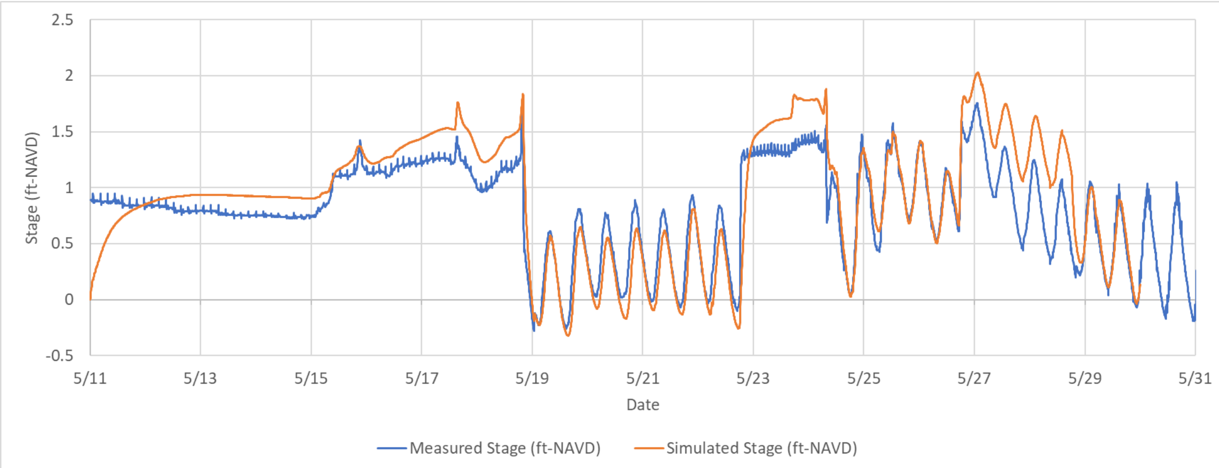
Figure 5-64 – Calibration Model Groundwater Stage – G-3565



5.4.7.1.4. C3W WATERSHED – CANAL STAGE

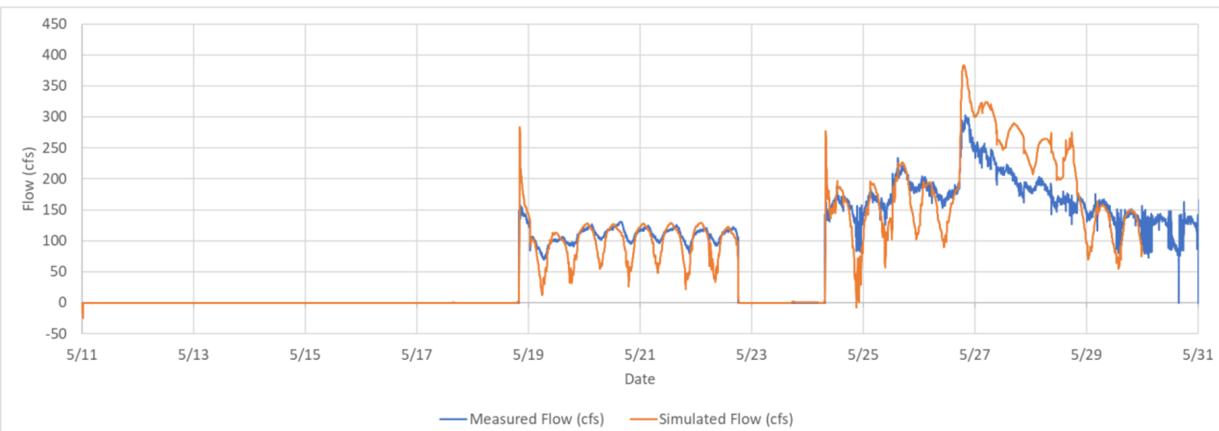
The only location within the C3W watershed where canal stage or flow is measured is at the G93 structure. Although the magnitude and pattern of the stage and flow response was within the target tolerance, the simulated results were not as accurate as S22. This difference in performance was to be expected considering the G93 structure is roughly 3.5 miles up the Coral Gables Canal from Biscayne Bay, and the boundary condition at the Coral Gables Canal was estimated while the boundary condition at the Snapper Creek Canal was measured.

Figure 5-65 – Calibration Model Canal Stage – G93-H



5.4.7.1.5. C3W WATERSHED– CANAL FLOW

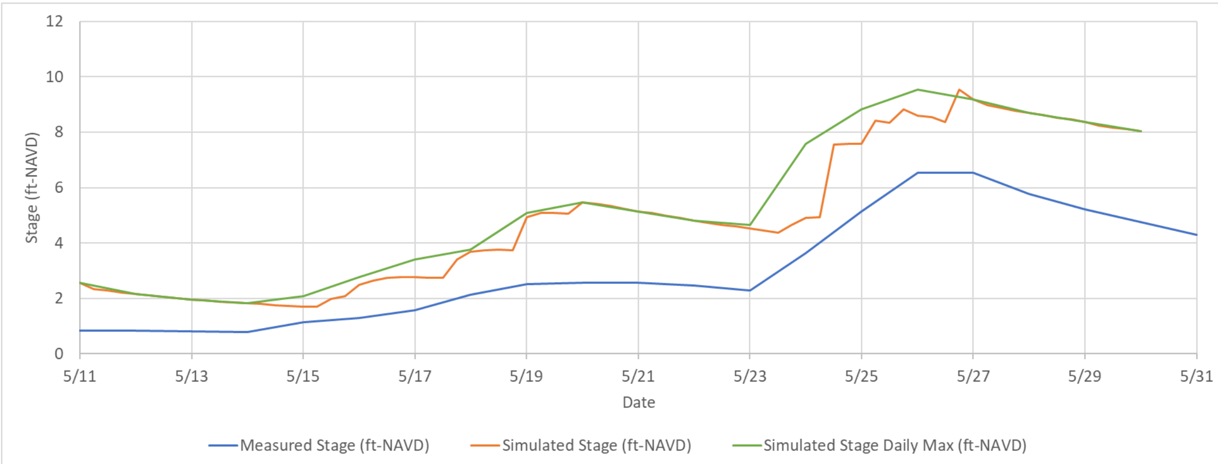
Figure 5-66 – Calibration Model Canal Flow – G93-Q



5.4.7.1.6. C3W WATERSHED – GROUNDWATER STAGE

The only groundwater monitoring location in the C3W watershed is G3570. The results below demonstrate good matching of the measured elevations with respect to the pattern of response, but not with respect to the magnitude of increase in groundwater elevation. Despite using published aquifer characteristics to define the parameterization of the Saturated Zone the model overestimates the increase in groundwater elevation in response to rainfall during the simulation period.

Figure 5-67 – Calibration Model Groundwater Stage – G-3570



5.4.7.1.7. C4 WATERSHED– CANAL STAGE

There are seven (7) locations where canal stages are measured in the C4 watershed. At all locations the model showed relatively good agreement with respect to timing and magnitude during the rainfall response period. The results were less accurate during the warmup period indicating that the model parameterization was better suited to wet periods than dry periods, which is preferable for FPLOS modeling. For the C4.Coral location there was monitoring data available at a 15 minute timestep from USGS, while it was only reported at a daily timestep from DBHYDRO. The graphical comparison in **Figure 5-74** illustrates that the simulated peak does not show good agreement with the 15-minute recorded data with an under-simulation of the peak stage by nearly 1 foot.

Figure 5-68 – Calibration Model Canal Stage – S25B-H

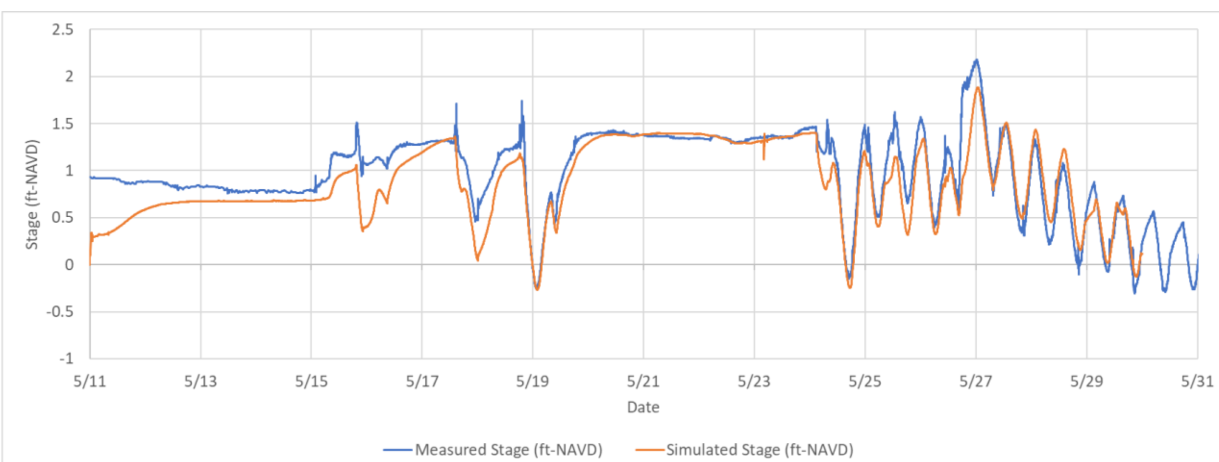


Figure 5-69 – Calibration Model Canal Stage – S336-T

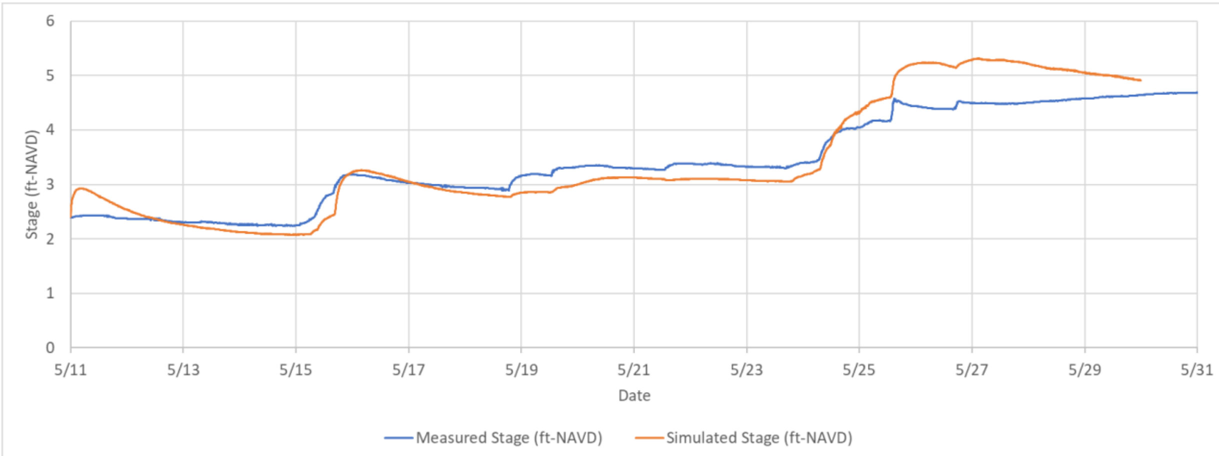


Figure 5-70 – Calibration Model Canal Stage – S380-H

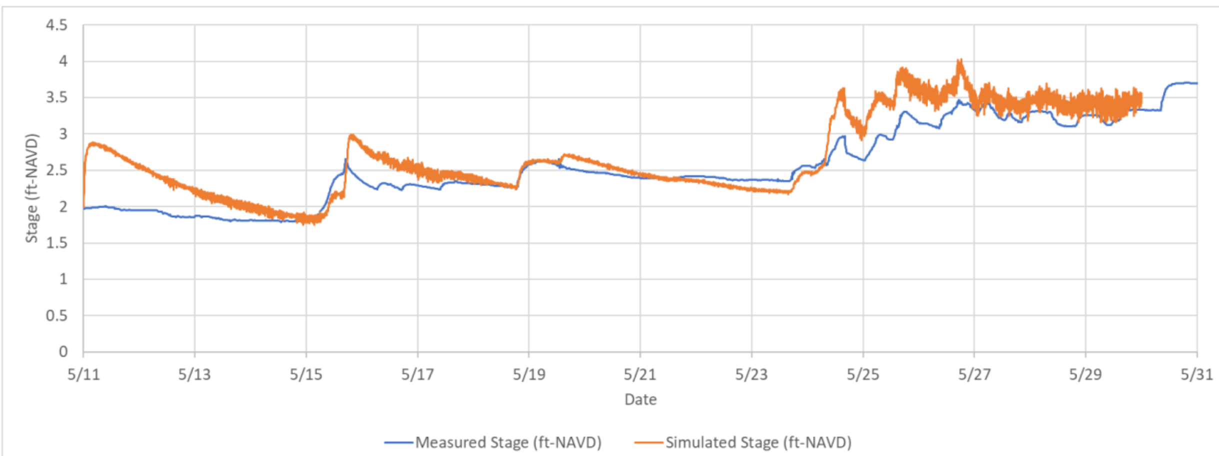


Figure 5-71 – Calibration Model Canal Stage – S380-T

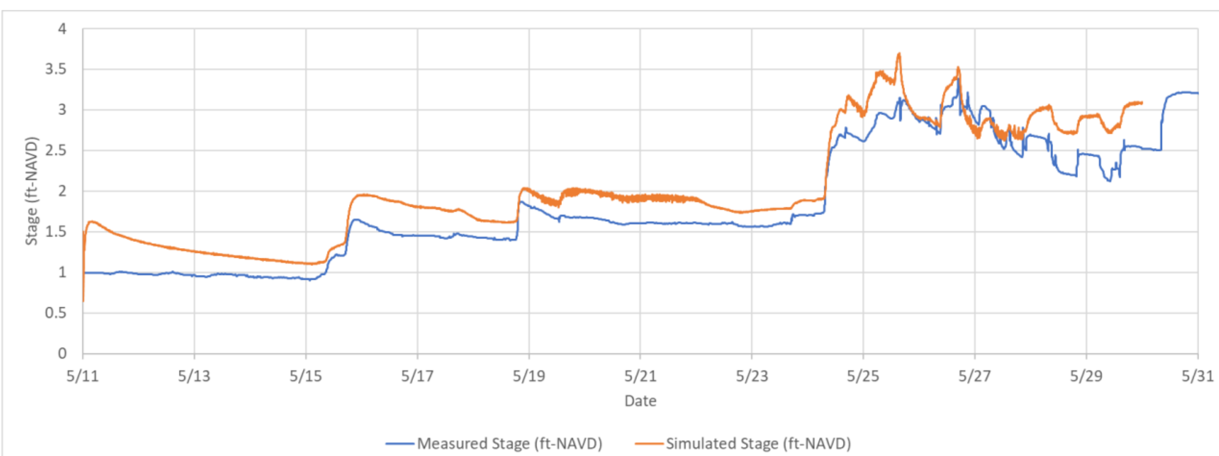


Figure 5-72 – Calibration Model Canal Stage – T5W

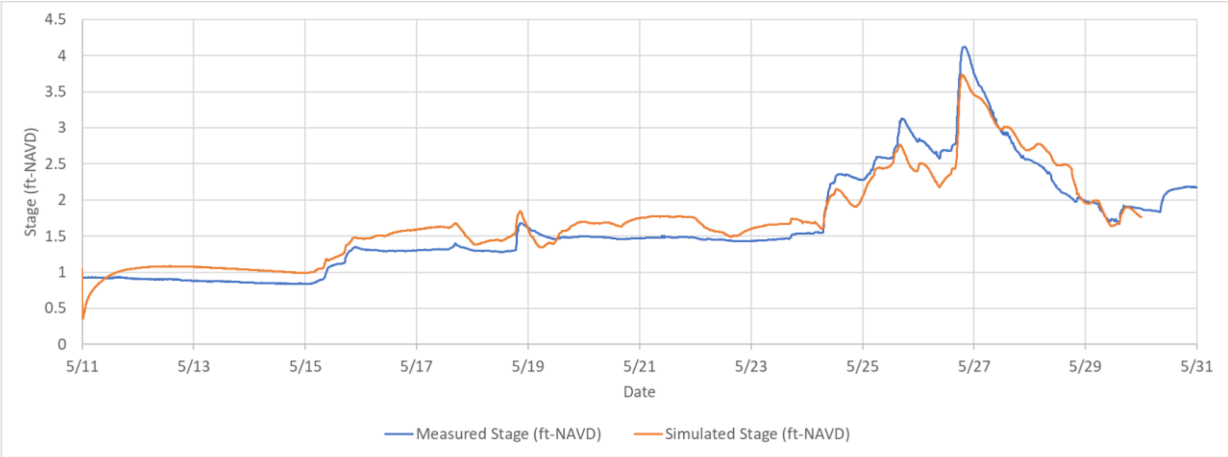


Figure 5-73 – Calibration Model Canal Stage – USGS 02287497 (NW Wellfield)

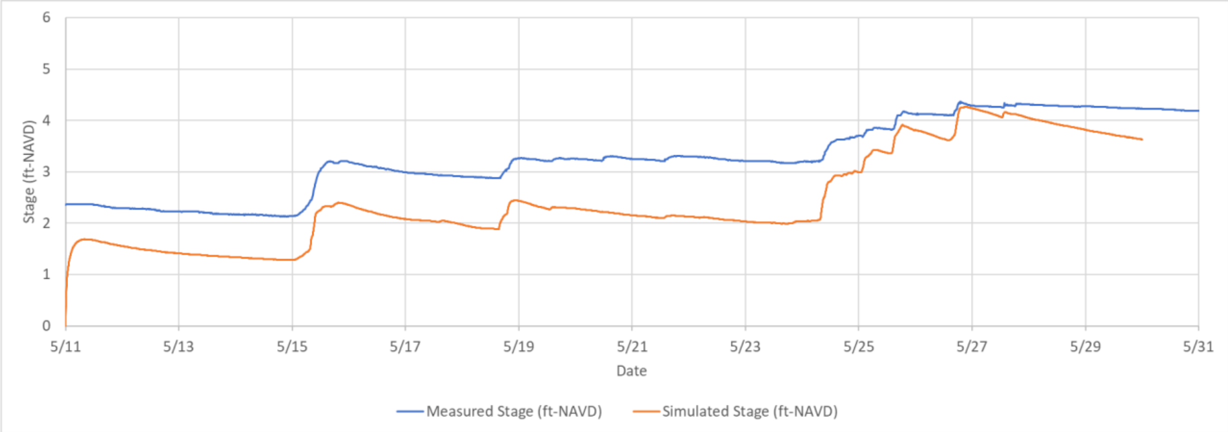
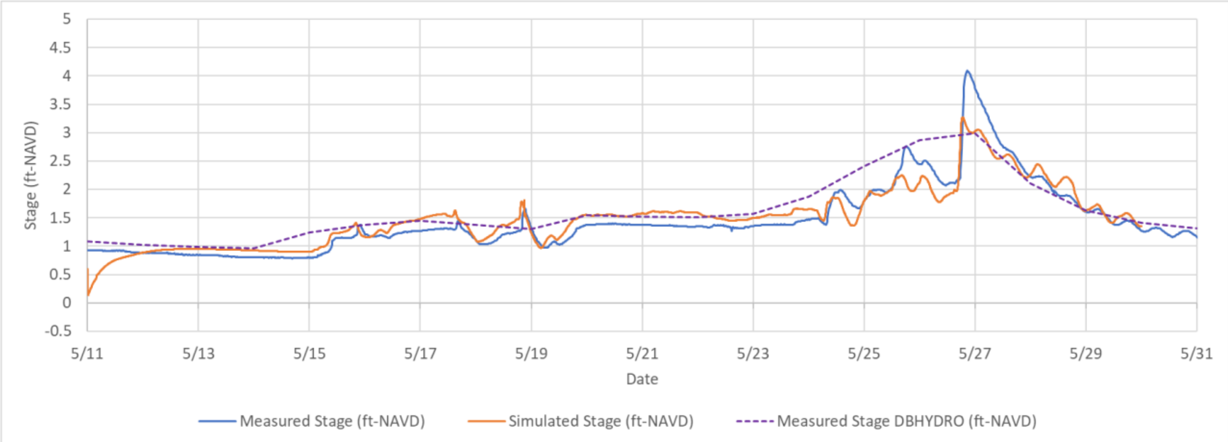


Figure 5-74 – Calibration Model Canal Stage – USGS 02289500 (C4.Coral)



5.4.7.1.8. C4 WATERSHED – CANAL FLOW

There are four (4) locations within the C4 watershed where flow is measured including at the S-25B spillway structure, which has two discharge mechanisms both a combined gravity spillway and a forward pumping station. The gravity spillway allows for large discharges during low tide and the forward pump station allows for continued discharge even at high tide. The model underestimated flow in the warmup periods during smaller rainfall events when the gate was opened between May 15th and 20th but matched relatively well with the combined measured flow from the spillway (S25B_S) and the pump station (S25B_P). The comparison of simulated and measured flow at the C4.Coral location showed an underestimation of flow during the peak runoff period of approximately 200 CFS within the western portion of the basin, but this discrepancy was not reflected in the S25B discharges which indicated that the differences in the western basins were attenuated as the hydraulic routing moves downstream.

The simulation results showed virtually no flow at S336 other than simulated instabilities whereas at the S380 gated culvert, the operation of this structure was unusual in consideration that it is only partially opened during pumping operations at the Emergency Detention Basin to provide additional water to the C4 Canal and prevent excessive drawdown. For this reason, flow only occurs when the Detention Basin inflow is engaged at which time the simulated results show some instabilities due to the comparatively small gate opening.

Figure 5-75 – Calibration Model Canal Flow – S25B-Q

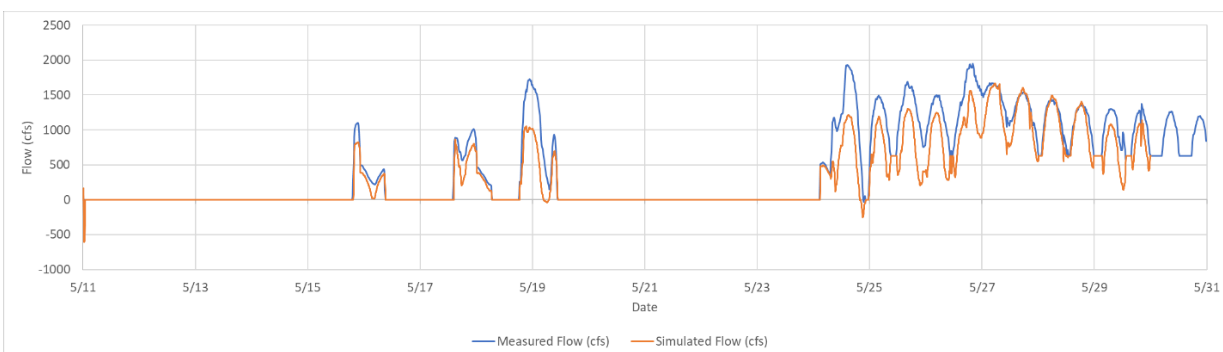


Figure 5-76 – Calibration Model Canal Flow – USGS 02289500 (C4.Coral)

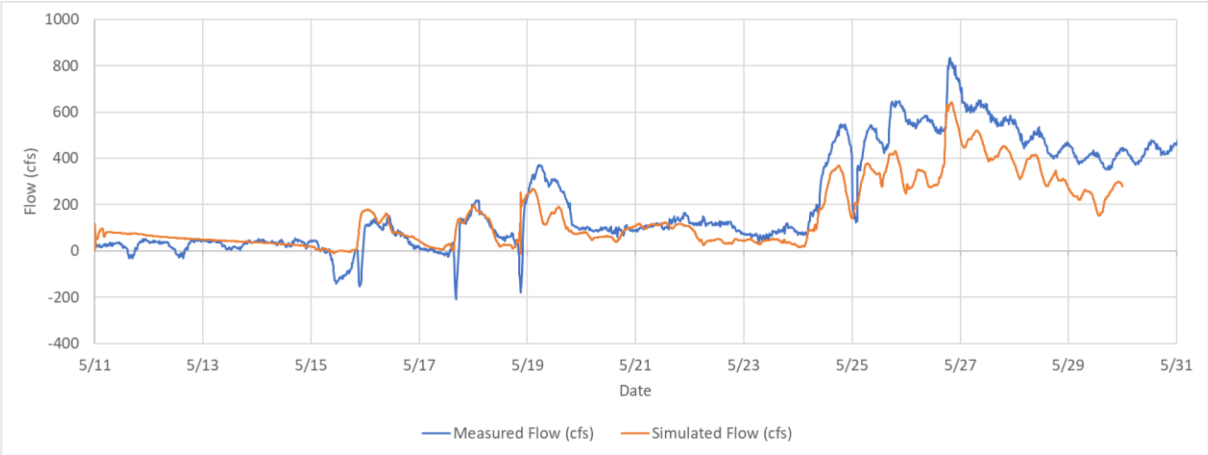


Figure 5-77 – Calibration Model Canal Flow – USGS 02287497 (NW Wellfield)

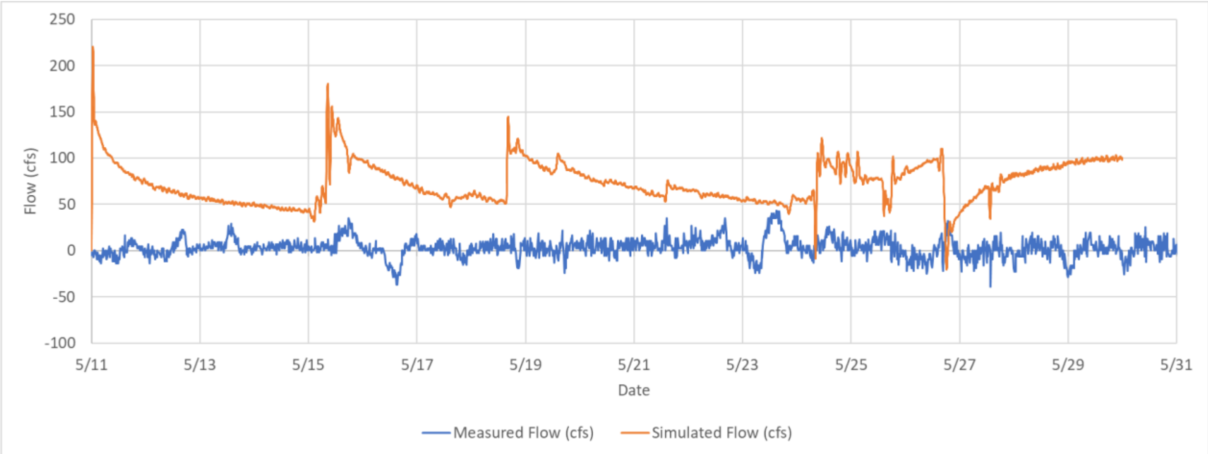


Figure 5-78 – Calibration Model Canal Flow – S380-Q

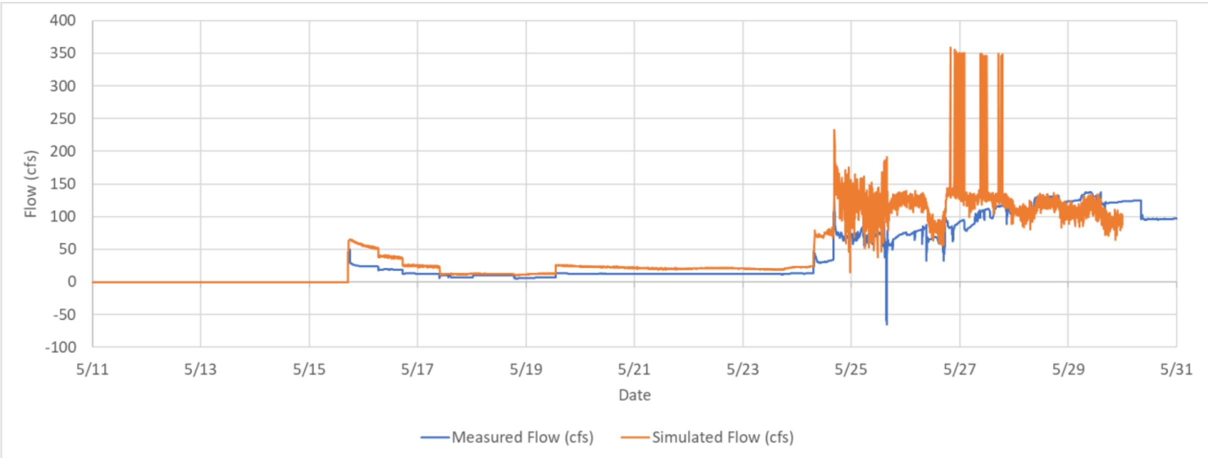
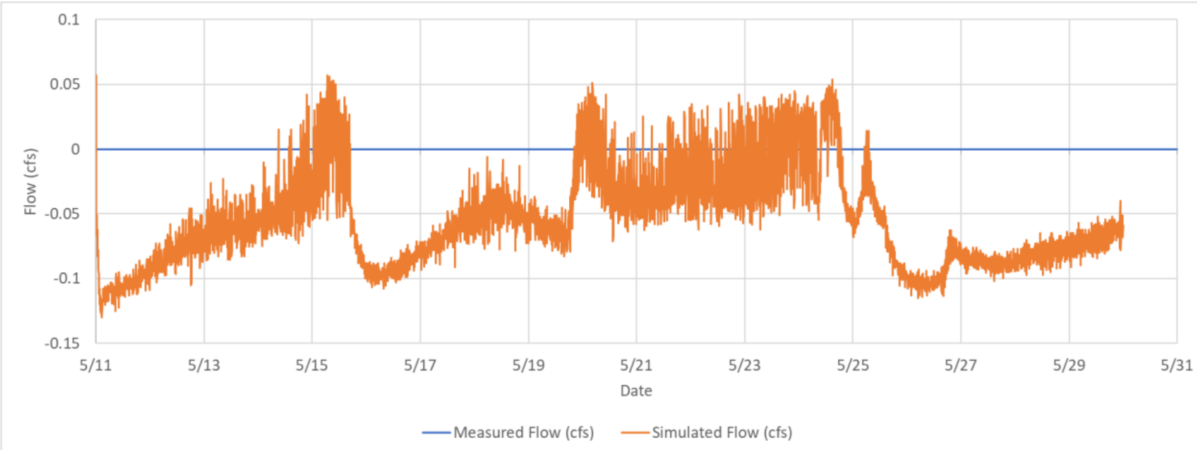


Figure 5-79 – Calibration Model Canal Flow – S336-Q



5.4.7.1.9. C4 WATERSHED– GROUNDWATER STAGE

A comparison of the groundwater results at C4GW1, G-975, and S356GW1 demonstrate that the Saturated Zone model was providing a good representation of the groundwater's response to the rainfall event in the western portions of the model domain. At the G-3465 monitoring well, the model results show that the Saturated Zone overestimated the response to the rainfall event and did not have sufficient recession of the groundwater after the event occurs. Similar to G-3563 in the C2 watershed, the G-3465 location was in the eastern portion of the model domain and may not have had sufficient recession due to the distance from boundary conditions or the primary canal network which could provide additional drainage.

Figure 5-80 – Calibration Model Groundwater Stage – C4GW1

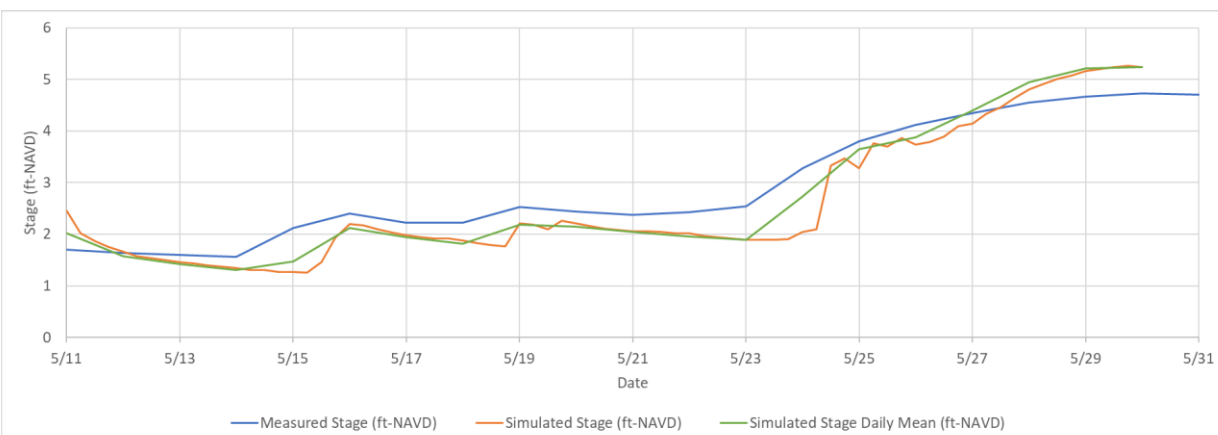


Figure 5-81 – Calibration Model Groundwater Stage – G-3465

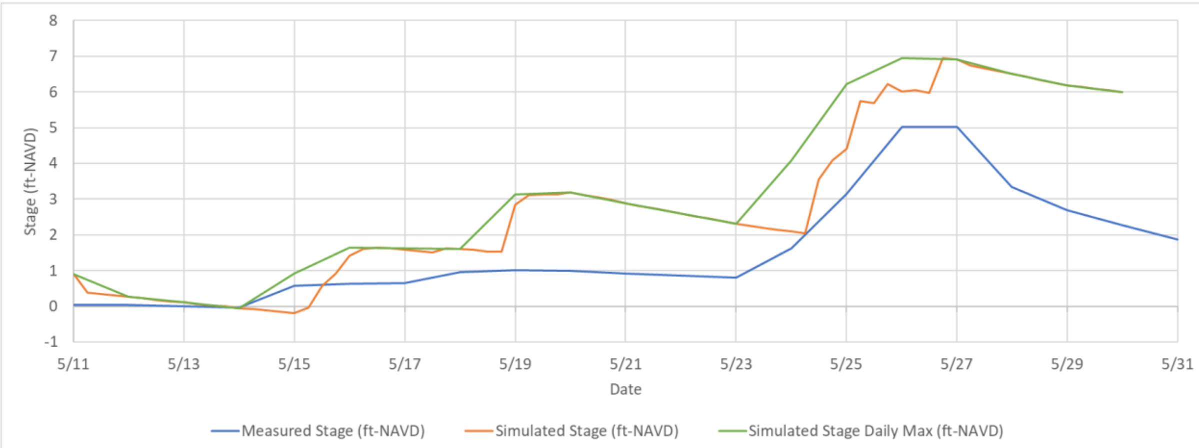


Figure 5-82 – Calibration Model Groundwater Stage – G-975

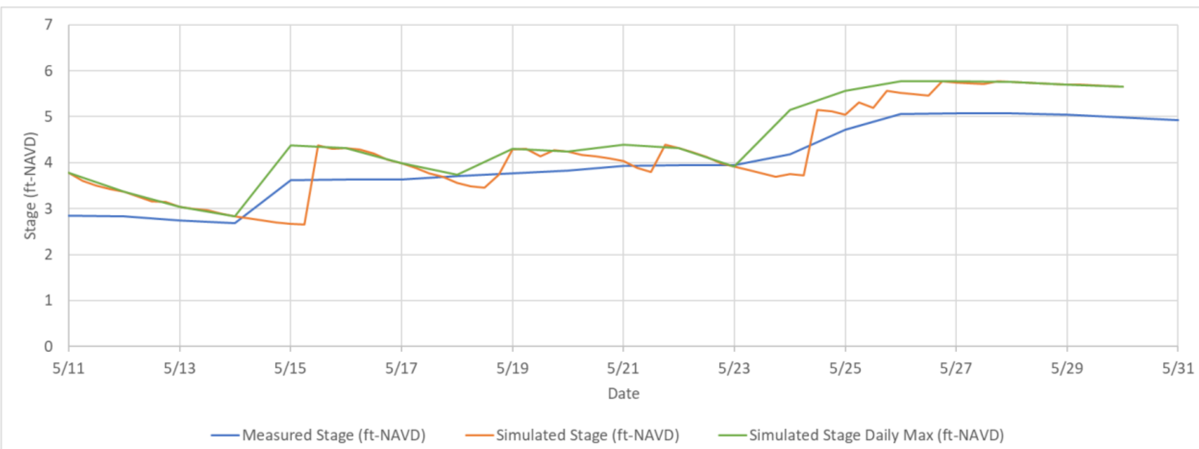
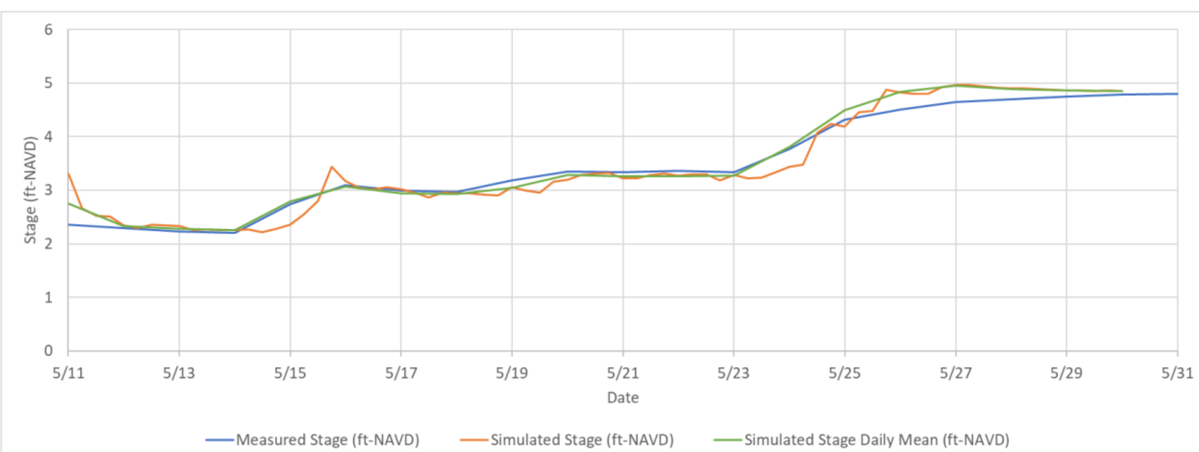


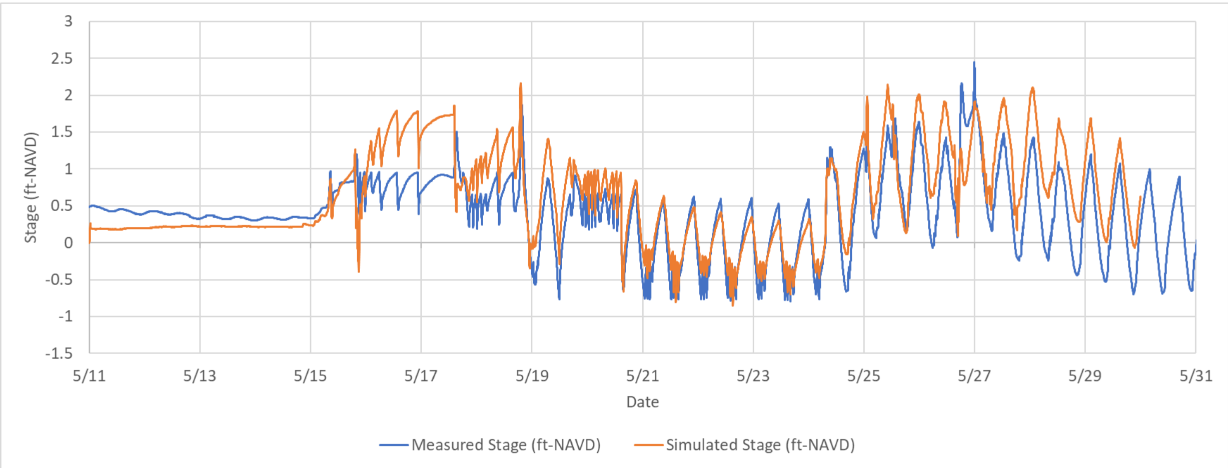
Figure 5-83 – Calibration Model Groundwater Stage – S356GW1



5.4.7.1.10. C5 WATERSHED – CANAL STAGE

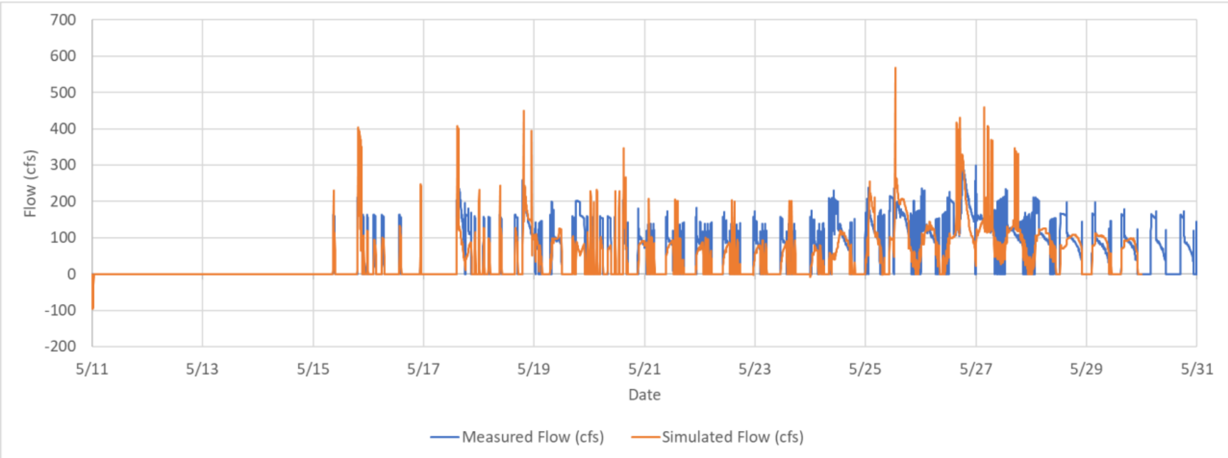
As the C5 is the smallest watershed in the model domain, it also has the least number of calibration points. As shown below the simulated stages and flows showed reasonably good agreement with the measured data during the warmup period and peak rainfall period.

Figure 5-84 – Calibration Model Canal Stage – S25-H



5.4.7.1.11. C5 WATERSHED – CANAL FLOW

Figure 5-85 – Calibration Model Canal Flow – S25-Q



5.4.7.1.12. C6 WATERSHED – CANAL STAGE

There are two stage monitoring locations in the C6 watershed, one at each end of the C6 Canal. At S26, which is the discharge structure for the Miami River, the simulated stage at the headwater side matched the measured stage almost identically during the peak rainfall period, as did the simulated tailwater stage at the westernmost structure S31. Based on this result, the model accurately represented stages in the C6 Canal during the calibration period.

Figure 5-86 – Calibration Model Canal Stage – S26-H

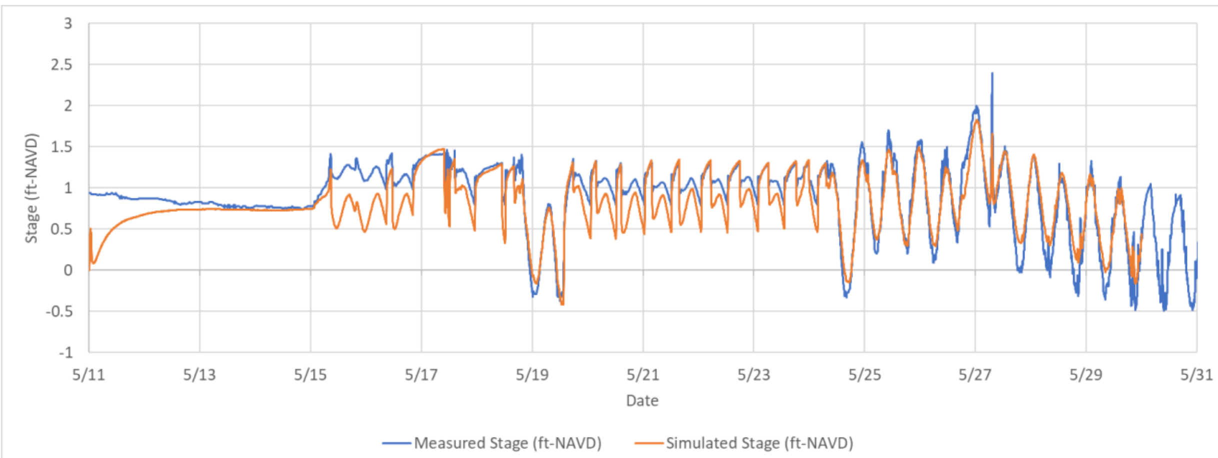
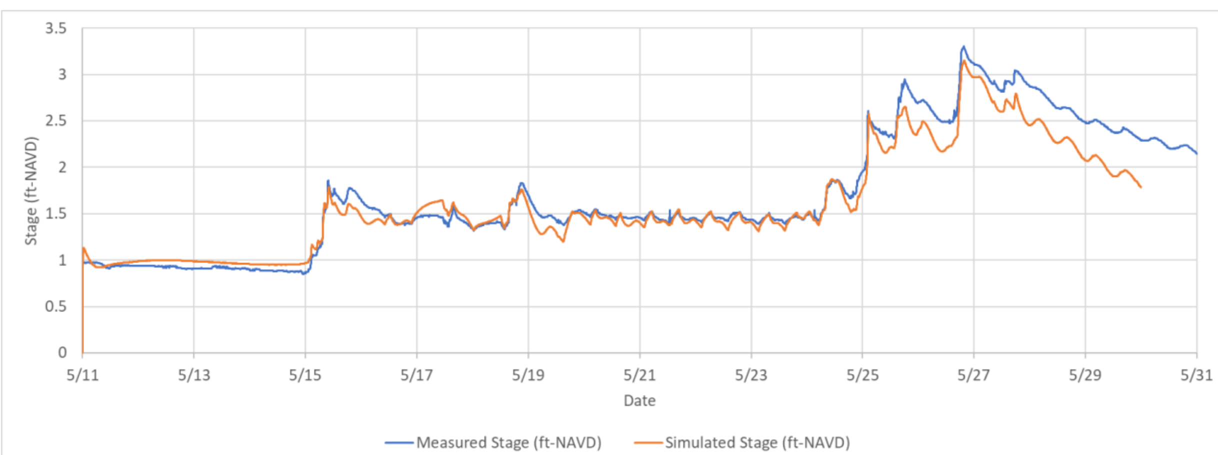


Figure 5-87 – Calibration Model Canal Stage – S31-T



5.4.7.1.13. C6 WATERSHED – CANAL FLOW

During the calibration period, there was no inflow from WCA3A to the Miami River as exemplified by no flow at the S31 structure. For the S26 structure, the simulated flows matched the measured flows well in consideration that the upstream stages were accurately simulated, and the gate openings were an input parameter. Similar to S25B, the S26 structure has both a spillway and a forward pump station. For this period both the spillway and pump station were engaged after the rainfall event as is evidenced by the simulated and measured flow remaining above 500 CFS during the peak period even during high or adverse tide conditions. The model appears to underestimate spillway flow during the rainfall event considering that pump discharge rates are included as a model input.

Figure 5-88 – Calibration Model Canal Flow – S26-Q

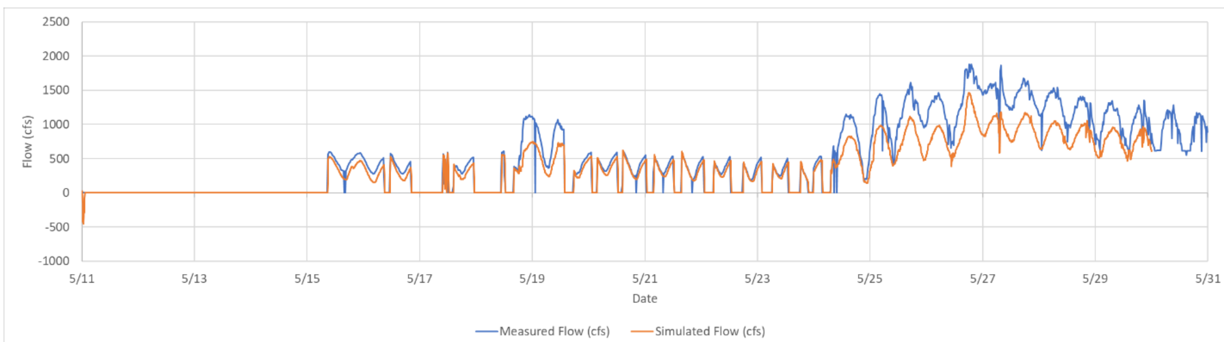
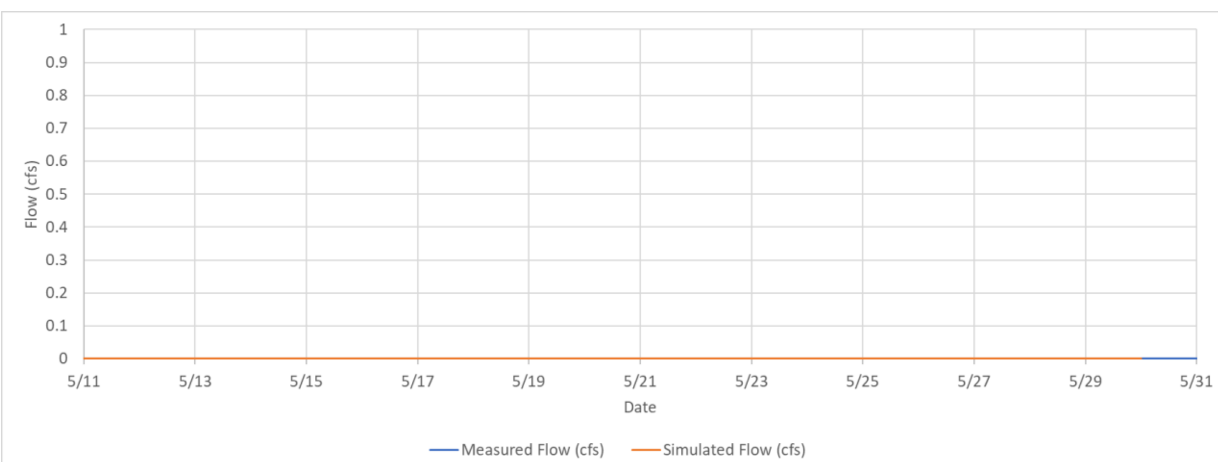


Figure 5-89 – Calibration Model Canal Flow – S31-Q



5.4.7.1.14. C6 WATERSHED – GROUNDWATER STAGE

The simulated groundwater elevations to the east (G-3 and S-68) and west (G-3567) showed a good representation of the pattern of response after the rainfall event as well as the recession of groundwater after the rainfall event. However, the accuracy of the magnitude of increase was not as consistent with the eastern stations being overestimated by the model while the western station was underestimated by the model. This is consistent with other results in the C2 and C4 watersheds where the increase in the simulated Saturated Zone elevations was overestimated at eastern monitoring locations as compared with western sites.

Figure 5-90 – Calibration Model Groundwater Stage – G-3

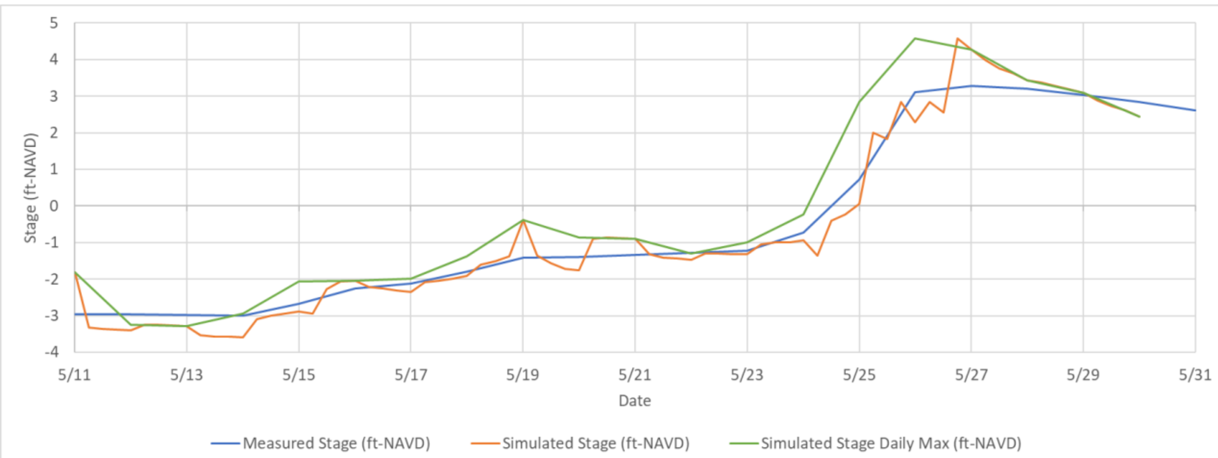


Figure 5-91 – Calibration Model Groundwater Stage – G-3567

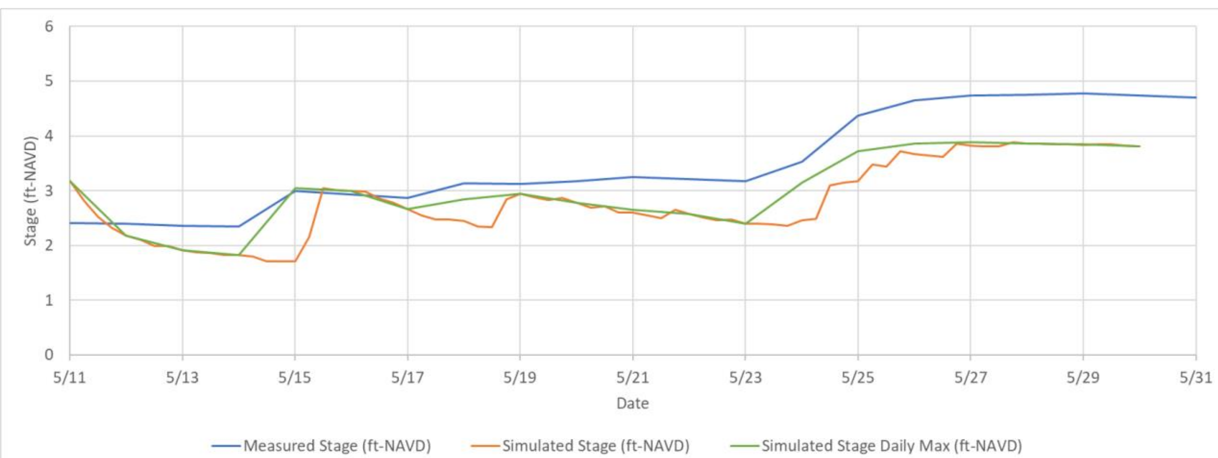
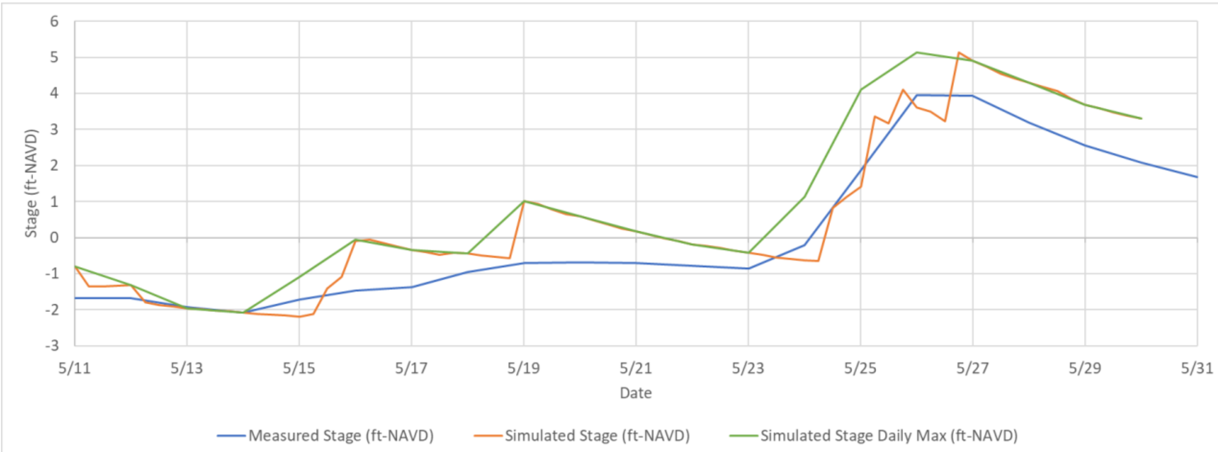


Figure 5-92 – Calibration Model Groundwater Stage – S-68



5.4.7.2. STATISTICAL COMPARISON

In addition to the graphical comparison provided above, a statistical comparison of the simulated versus measured data sets was performed. The following statistical metrics were utilized to demonstrate the performance of the model:

- Mean Error (ME)
- Mean Absolute Error (MAE)
- Root Mean Squared Error (RMSE)
- Correlation (R)
- Nash Sutcliffe Efficiency (NS)

The calculation of these statistics was based on standard practices comparing the simulated dataset to the measured data set at each time step where measured data is available. The values for ME, MAE, and RMSE represent different approaches for determining the average difference between the simulated and measured data, where ME allows for the negative and positive errors to cancel each other, while the MAE utilizes absolute values and the RMSE uses squared values to evaluate the error. For MAE the absolute method weighs all errors equally, while for RMSE the squared method gives more weight to errors that are greater due to simulated values that are less accurate. The R value provides a description of the correlation between simulated and measured values with a value of 1.0 showing perfect correlation and a value of 0.0 showing no correlation. The NS values provide a comparison of how valuable the simulated results are in comparison to the average of measured values. This coefficient is calculated by subtracting the variance of the datasets from 1.0, such that 1.0 implies no variance between the simulated and measured values. The equations below describe the methodology for calculating each statistical parameter:

$$ME = \frac{\sum_{i=1}^{n}(Measured_i - Simulated_i)}{n}$$

$$MAE = \frac{\sum_{i=1}^{n}|Measured_i - Simulated_i|}{n}$$

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n}(Measured_i - Simulated_i)^2}{n}}$$

$$R = \frac{\sum_{i=1}^{n}(Measured_i - Avg\ Measured)(Simulated_i - Avg\ Simulated)}{\sqrt{\sum_{i=1}^{n}(Measured_i - Avg\ Measured)^2}\sqrt{\sum_{i=1}^{n}(Measured_i - Avg\ Simulated)^2}}$$

$$NS = 1 - \frac{\sum_{i=1}^{n}(Measured_i - Simulated_i)^2}{\sum_{i=1}^{n}(Measured_i - Avg\ Measured)^2}$$

Table 5-9 describes the statistical performance of each monitored stage location with a full dataset in the District's DBHYDRO records. All stage errors were calculated in terms of feet. The identified target for stage calibration was an error (ME, MAE, RMSE) less than 0.5 feet. The calibration target was selected to conform with the approach used in other FPLOS efforts in the region. As noted below for the statistical analysis of the entire simulation period, none of the locations had an error exceeding the target. Since it was more difficult to accurately simulate the runoff response during a short period with a large rainfall event an additional statistical analysis was provided to evaluate the effectiveness of the model over a seven-day period during the rainfall event. **Table 5-10** demonstrates that during the shorter period that represents the peak event, the calibration target of no errors exceeding 0.5 feet was still met. Notably, data from the G119 station was not included considering that automated monitoring appears to have ended in 2014 with staff gauge readings supplementing the time-series since that time.

Table 5-9 – Calibration Model Canal Stage Statistics – Full Model Run

WATERSHED	STATION	19 - DAY SIMULATION (5/11/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	C2SW1	-0.12	0.19	0.22	0.96	0.88
	C2SW2	-0.06	0.12	0.16	0.91	0.77
	S22H	-0.13	0.18	0.23	0.90	0.72
C3W	G93H	-0.10	0.21	0.28	0.89	0.57
C4	C4.Coral	0.13	0.17	0.26	0.92	0.79
	S25BH	0.13	0.18	0.25	0.89	0.72
	S336T	-0.07	0.29	0.37	0.97	0.78
	S380H	-0.21	0.25	0.32	0.91	0.58
	S380T	-0.25	0.27	0.31	0.97	0.82
	T5W	-0.03	0.19	0.22	0.97	0.93
C5	S25H	-0.24	0.35	0.42	0.85	0.51
C6	S26H	0.01	0.16	0.21	0.93	0.84
	S31T	0.09	0.13	0.18	0.99	0.93

Table 5-10 – Calibration Model Canal Stage Statistics – Peak Rain Period

WATERSHED	STATION	7 - DAY SIMULATION (5/23/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	C2SW1	0.03	0.14	0.16	0.97	0.92
	C2SW2	0.01	0.09	0.11	0.96	0.92
	S22H	-0.21	0.24	0.29	0.92	0.67
C3W	G93H	-0.21	0.24	0.32	0.88	0.43
C4	C4.Coral	0.19	0.24	0.35	0.85	0.60
	S25BH	0.10	0.18	0.25	0.90	0.77
	S336T	-0.37	0.48	0.53	0.97	-0.35
	S380H	-0.27	0.31	0.36	0.89	-0.30
	S380T	-0.24	0.27	0.33	0.87	0.41
	T5W	0.10	0.22	0.25	0.94	0.84
C5	S25H	-0.32	0.40	0.46	0.88	0.55
C6	S26H	-0.04	0.15	0.19	0.96	0.88
	S31T	0.22	0.22	0.26	0.97	0.77

Table 5-11 describes the statistical performance of each estimated flow location based on the data available within the District's DBHYDRO records. All flow errors are calculated in terms of cubic feet per second (CFS). For flow values, a calibration target of ±20% of the measured peak flow was chosen for the error (ME, MAE, and RMSE). The calibration target was selected to conform with the approach used in other FPLOS efforts in the region. Similar to stage, a statistical analysis was provided to evaluate the effectiveness of the model with respect to flow over a seven-day period during the peak rainfall event **Table 5-12** describes the statistical performance of the model with respect to flow during the shorter period that represents the peak event. Notably, S336 and S31Q have minimal to no flow during the simulation period as reflected in the statistical calculations. The negative NS values indicate that the mean of the measured data is a better predictor of measured data than the simulated results. Since there is negligible flow, minor variations from zero cause the simulated data to be a worse predictor than zero, which is the mean of the measured. For S380Q, the section of the C4 Canal contributing to the S380 structure is fed only by groundwater interactions (because S336 is closed), and the small head differences can generate instabilities and oscillations in the simulated values. Considering the effect on the overall model is negligible, no additional effort was applied to improve the NS values at these locations.

Table 5-11 – Calibration Model Flow Statistics – Full Model Run

Watershed	Station	19 - Day Simulation (5/11/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	S22Q	58.07	75.01	126.34	0.98	0.93
C3W	G93Q	-0.34	18.73	33.44	0.94	0.83
C4	S25BQ	-26.59	206.05	312.03	0.77	0.52
	S380Q	-23.09	26.47	44.56	0.74	0.05
	S336Q	0.05	0.06	0.06	-1	-1
C5	S25Q	11.08	33.89	53.38	0.69	0.42
C6	S26Q	-71.68	160.46	216.81	0.84	0.65
	S31Q	0.00	0.00	0.00	-1	-1

Table 5-12 – Calibration Model Flow Statistics – Peak Rain Period

WATERSHED	STATION	7 - DAY SIMULATION (5/23/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	S22Q	119.56	138.65	176.85	0.94	0.79
C3W	G93Q	-5.78	35.39	48.09	0.89	0.59
C4	S25BQ	-124.53	309.77	386.34	0.66	0.18
	S380Q	-37.96	44.31	60.56	0.36	-0.89
	S336Q	0.05	0.06	0.07	-1	-1
C5	S25Q	16.45	44.88	62.20	0.60	0.21
C6	S26Q	-137.10	197.98	244.59	0.78	0.43
	S31Q	0.00	0.00	0.00	-1	-1

Table 5-13 describes the statistical performance of each groundwater monitoring location based on the data available within the District's DBHYDRO records. All stage errors were calculated in terms of feet. For the entire simulation period, the Mean Error exceeded a tolerance of 0.5 feet at six (6) of the twelve (12) locations.

Table 5-14 describes the statistical performance of the model with respect to groundwater elevation during the shorter period that represents the peak event. For this period the Mean Error was within a 0.5 feet tolerance for only four (4) of the twelve (12) locations.

Table 5-13 – Calibration Model Groundwater Stage Statistics – Full Model Run

WATERSHED	STATION	19 - DAY SIMULATION (5/11/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	G-3558	0.08	0.31	0.47	0.88	0.77
	G-3563	-0.93	1.10	1.67	0.95	-0.57
	G-3572	-0.45	0.60	0.80	0.87	0.49
	G-3565	1.20	1.28	1.46	0.80	-0.74
C3W	G-3570	-2.11	2.11	2.27	0.98	-0.38
C4	C4GW1	0.38	0.46	0.53	0.94	0.75
	G-3465	-1.77	1.81	2.15	0.89	-1.00
	G-975	-0.34	0.49	0.55	0.90	0.52
	S356GW1	-0.10	0.19	0.28	0.96	0.90
C6	G-3	0.08	0.61	0.77	0.97	0.89
	G-3567	0.58	0.68	0.76	0.83	0.23
	S-68	-0.59	0.81	0.96	0.95	0.77

Table 5-14 – Calibration Model Groundwater Stage Statistics – Peak Rain Period

Watershed	Station	7 - Day Simulation (5/23/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	G-3558	0.28	0.30	0.46	0.91	0.50
	G-3563	-2.16	2.21	2.59	0.83	-4.71
	G-3572	-0.65	0.99	1.17	0.71	-0.10
	G-3565	1.23	1.30	1.65	0.73	-1.68
C3W	G-3570	-2.73	2.73	2.81	0.94	-3.33
C4	C4GW1	0.60	0.60	0.66	0.97	0.16
	G-3465	-2.55	2.55	2.80	0.76	-2.98
	G-975	-0.41	0.51	0.56	0.97	-0.76
	S356GW1	-0.11	0.19	0.23	0.97	0.78
C6	G-3	-0.21	0.45	0.67	0.97	0.86
	G-3567	0.94	0.94	0.95	0.99	-1.59
	S-68	-0.63	0.96	1.12	0.93	0.55

Based on these statistical analyses, a graphic was developed illustrating the locations where the MAE for the calibration simulation was within the targeted tolerance as shown in **Figure 5-93**.

As an additional metric of model performance, the simulated and measured peak stage were compared at each monitoring location. **Table 5-15** provides a comparison of the peak stages with respect to the absolute difference and percent difference. This analysis demonstrates that at only one location (S25) was the difference in the peak stage larger than the calibration target of 0.5 feet, and the average discrepancy across all locations was 10%. **Table 5-16** illustrates the difference in simulated and measured peak flow, with a calibration target of ±20% of the peak. The values at S25B and S26 each include the simulated discharge for both the spillway and the forward pump station.

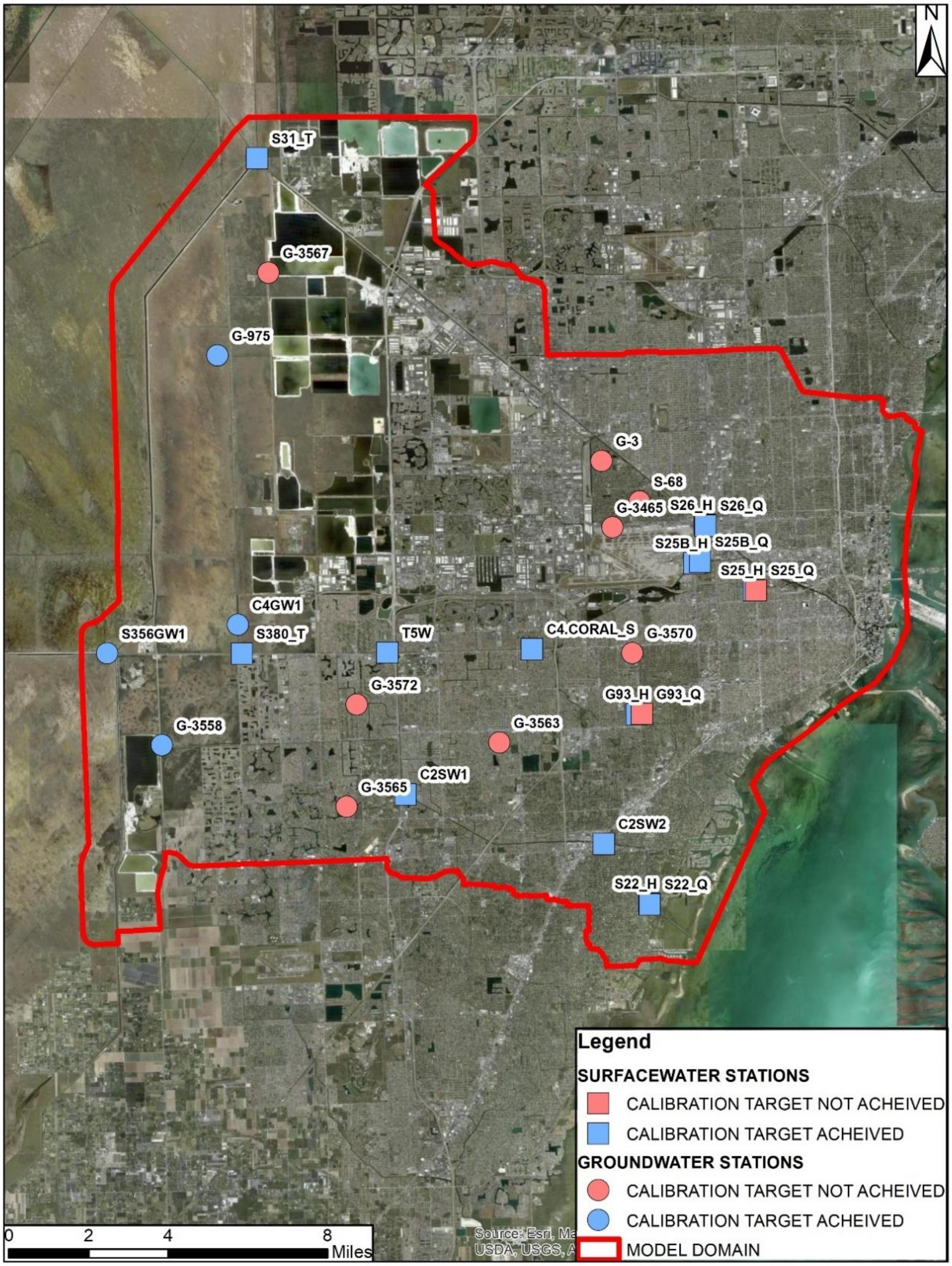
Table 5-15 – Calibration Model Peak Stage Comparison

Calibration Point	Peak Stage [ft-NAVD]			
	Measured	Simulated	Absolute Difference	Percent Difference
C2SW1	3.478	3.227	0.25	7%
C2SW2	2.215	2.030	0.19	8%
C4.Canal	2.988	3.273	0.29	10%
G93H	1.758	2.031	0.27	16%
S22H	1.548	1.832	0.28	18%
S25BH	2.181	1.885	0.30	14%
S25H	2.445	2.165	0.28	11%
S26H	2.395	1.828	0.57	24%
S31T	3.308	3.153	0.16	5%
S380T	3.385	3.698	0.31	9%
T5W	4.118	3.737	0.38	9%

Table 5-16 – Calibration Model Peak Flow Comparison

Calibration Point	Peak Flow [CFS]			
	Measured	Simulated	Absolute Difference	Percent Difference
S25	299.7	568.4	268.7	90%
S25B	2007.1	1662.1	345.0	17%
S26	1426.6	1461.7	35.1	2%
G93	303.3	383.8	80.5	27%
S22	1533.1	1442.9	90.2	6%

Figure 5-93 – Location of Calibration Points with MAE within Target Tolerance



5.4.7.3. WATER BUDGET EVALUATION BY WATERSHED

Table 5-17 through **Table 5-31** provide a summary of the water balance calculated for each basin within the model domain. For each basin there is an accounting of the water balance for MIKE SHE, which represents the hydrology and groundwater simulation. There are also tables for each basin representing the water balance for MIKE 1D, which represents the hydraulic simulation. For the hydraulic water balance, there are separate tables representing the inflow and outflow from the overland and groundwater contributions as well as a breakdown of direct discharges into and out of the hydraulic network at the confluence of canals or the boundaries of the network. All water balance calculations are performed in terms of inches in consideration of the significant differences in the contributing areas of each watershed. The water balance terms include various components including the following:

- **Rainfall:** Water from the atmosphere to the land surface.
- **Evapotranspiration (ET):** Water from the land surface to the atmosphere.
- **Saturated Zone (SZ) to River:** Water that moves from the groundwater into the canal network.
- **River to Saturated Zone (SZ):** Water that moves from the canal network into the groundwater.
- **Overland (OL) to River:** Hydrologic runoff from the land surface to the river.
- **Groundwater (GW) Boundary:** Flows from or to the groundwater boundary at the domain limits.
- **Overland (OL) Boundary:** Flows from or to the overland boundary at the domain limits.
- **Groundwater (GW) Pumping:** Flows from groundwater for water supply (outside the domain).
- **Change in Channel Storage:** Difference of inflows and outflows from the model reaches.

The results below demonstrate that the model does not lose or gain water from an unquantifiable or unknown source and that the magnitude of movement within the budget is within reasonable values.

One notable result shown in the water balance calculation is the large negative flow from Overland to River in the C4 and C6 watersheds. An analysis of this computation illustrated that the driver of this value is associated with the rock mine pits in the western portion of the model domain. Because these pits are modeled as river reaches, any overtopping of the mine pit is allocated as negative flow in the Overland to River category.

Table 5-17 – C2 Watershed (33,511 ac) – MIKE SHE Water Balance

MIKE SHE WATER BALANCE [IN]	IN	OUT
Rainfall	12.93	
ET		1.24
SZ -> River		4.05
River -> SZ	0.83	
OL -> River		0.44
GW Boundary Inflows	0.83	
GW Boundary Outflows		2.70
OL Boundary Inflows	0.01	
OL Boundary Outflows		0.00
GW Pumping		1.85
TOTAL	**14.60**	**10.28**
OL Change in Storage	1.31	
GW Change in Storage	2.97	
Residual	-0.03	

Table 5-18 – C2 Watershed (33,511 ac) – MIKE 1D Water Balance

CHANNEL WATER BALANCE [IN]	IN	OUT
MIKE 1D Inflows	1.28	
MIKE 1D Outflows		4.80
OL -> River	0.44	
SZ -> River	4.05	
River -> SZ		0.83
TOTAL	**5.78**	**5.63**
Channel Δ Storage	0.15	

Table 5-19 – C2 Watershed (33,511 ac) – MIKE 1D Inflow and Outflow

INFLOW LOCATION	TOTAL FLOW [IN]
Snapper Creek Canal	1.19
SW 132nd Ave Canal	0.05
Coral Way Canal	0.04

OUTFLOW LOCATION	TOTAL FLOW [IN]
S121	0.00
S22	4.80

Table 5-20 – C3W Watershed (3,580 ac) – MIKE SHE Water Balance

MIKE SHE WATER BALANCE [IN]	IN	OUT
Rainfall	14.06	
ET		1.07
SZ -> River		6.46
River -> SZ	0.07	
OL -> River		0.42
GW Boundary Inflows	1.71	
GW Boundary Outflows		2.24
OL Boundary Inflows	0.02	
OL Boundary Outflows		0.04
GW Pumping		0.00
TOTAL	**15.86**	**10.23**
OL Change in Storage	1.84	
GW Change in Storage	3.73	
Residual	-0.04	

Table 5-21 – C3W Watershed (3,580 ac) – MIKE 1D Water Balance

CHANNEL WATER BALANCE [IN]	IN	OUT
MIKE 1D Inflows	2.71	
MIKE 1D Outflows		9.79
OL -> River	0.42	
SZ -> River	6.46	
River -> SZ		0.07
TOTAL	**9.60**	**9.86**
Channel ∆ Storage	-0.27	

Table 5-22 – C3W Watershed (3,580 ac) – MIKE 1D Inflow and Outflow

INFLOW LOCATION	TOTAL FLOW [IN]
Coral Gables Canal	2.71

OUTFLOW LOCATION	TOTAL FLOW [IN]
G93	9.79

Table 5-23 – C4 Watershed (53,904 ac) – MIKE SHE Water Balance

MIKE SHE WATER BALANCE [IN]	IN	OUT
Rainfall	15.48	
ET		1.62
SZ -> River		15.62
River -> SZ	0.22	
OL -> River		-7.48
GW Boundary Inflows	1.50	
GW Boundary Outflows		1.10
OL Boundary Inflows	0.04	
OL Boundary Outflows		0.04
GW Pumping		0.51
TOTAL	**17.24**	**11.41**
OL Change in Storage	3.80	
GW Change in Storage	1.83	
Residual	-0.19	

Table 5-24 – C4 Watershed (53,904 ac) – MIKE 1D Water Balance

CHANNEL WATER BALANCE [IN]	IN	OUT
MIKE 1D Inflows	-0.09	
MIKE 1D Outflows		3.98
OL -> River	-7.48	
SZ -> River	15.62	
River -> SZ		0.22
TOTAL	**8.05**	**4.21**
Channel Δ Storage	3.85	

Table 5-25 – C4 Watershed (53,904 ac) – MIKE 1D Inflow and Outflow

LOCATION	TOTAL INFLOW [IN]	TOTAL OUTFLOW [IN]
S-336	0.00	
FEC C4 Canal	-0.09	
S25B		2.61
Snapper Creek Canal		0.74
SW132 Ave Canal		0.03
Coral Gables Canal		0.03
Dressels Dairy Canal		0.58

Table 5-26 – C5 Watershed (1,215 ac) – MIKE SHE Water Balance

MIKE SHE Water Balance [in]	In	Out
Rainfall	20.47	
ET		0.93
SZ -> River		13.07
River -> SZ	0.10	
OL -> River		2.23
GW Boundary Inflows	5.54	
GW Boundary Outflows		4.22
OL Boundary Inflows	0.07	
OL Boundary Outflows		0.00
GW Pumping		0.00
Total	**26.18**	**20.45**
OL Change in Storage	0.93	
GW Change in Storage	4.45	
Residual	0.05	

Table 5-27 – C5 Watershed (1,215 ac) – MIKE 1D Water Balance

Channel Water Balance [in]	In	Out
MIKE 1D Inflows	0.00	
MIKE 1D Outflows		15.79
OL -> River	2.23	
SZ -> River	13.07	
River -> SZ		0.10
Total	**15.30**	**15.89**
Channel Δ Storage	-0.59	

Table 5-28 – C5 Watershed (1,215 ac) – MIKE 1D Inflow and Outflow

Inflow Location	Total Flow [in]
S25A	0.00

Outflow Location	Total Flow [in]
S25	15.79

Table 5-29 – C6 Watershed (33,919 ac) – MIKE SHE Water Balance

MIKE SHE Water Balance [in]	In	Out
Rainfall	18.74	
ET		1.26
SZ -> River		12.46
River -> SZ	1.20	
OL -> River		-4.27
GW Boundary Inflows	1.13	
GW Boundary Outflows		1.60
OL Boundary Inflows	0.08	
OL Boundary Outflows		0.20
GW Pumping		1.80
Total	**21.15**	**13.05**
OL Change in Storage	4.12	
GW Change in Storage	3.93	
Residual	-0.10	

Table 5-30 – C6 Watershed (33,919 ac) – MIKE 1D Water Balance

Channel Water Balance [in]	In	Out
MIKE 1D Inflows	-0.64	
MIKE 1D Outflows		4.59
OL -> River	-4.27	
SZ -> River	12.46	
River -> SZ		1.20
Total	**7.55**	**5.78**
Channel Δ Storage	1.76	

Table 5-31 – C6 Watershed (33,919 ac) – MIKE 1D Inflow and Outflow

Inflow Location	Total Flow [in]
G72	0.01
S31	0.00
S32	0.00
S32A	0.00
Red Road Canal	-0.33
NW 97 Ave Canal	-0.31

Outflow Location	Total Flow [in]
S26	4.74
FEC C4 Canal	-0.15

5.4.8. VALIDATION SIMULATION

Using the same parameterization as the final calibration for the 2020 event, a validation simulation was performed for the Hurricane Irma storm event in 2017. The simulation period for the validation event was September 1 through September 30, 2017, with the peak rainfall occurring on September 9 and 10, 2017. The validation simulation utilized gate opening data from District records to define operations and assumed all municipal pump stations were engaged.

5.4.8.1. GRAPHICAL COMPARISON BY WATERSHED

Similar to the review of the calibration results, a graphical comparison of simulated versus measured stage, flow, and groundwater elevation was prepared and is presented in **Figure 5-94** through **Figure 5-126**. As with the calibration period, the results demonstrate generally good agreement at most monitoring locations for the event peak, with some discrepancies before and after the storm. Of note, for the validation period the Saturated Zone model did not simulate groundwater elevations that match as well with measured data as shown in the calibration period. Another notable element of the validation results was the discrepancy at several locations for the initial conditions. To evaluate if the results for the overall simulation could be improved by differing the initial conditions, a test scenario was simulated with higher canal stages using extracted values from September 30, 2020, as the initial conditions. This scenario represented a high wet season value for the canal network. Although the simulated stages started closer to measured stages in the test scenario, it did not improve the validation results at the runoff event peak, and as such was not used.

The graphical results presented below are grouped by watershed in sequential order from C2 to C6. For reference of the location of calibration points, see **Figure 5-1** through **Figure 5-5**.

5.4.8.1.1. C2 WATERSHED – CANAL STAGE

At each of the three stage monitoring locations within the C2 watershed, the simulated stage for the validation period was underestimated prior to the rainfall event but matches the peak and recession period well with respect to the magnitude and pattern of the response. Similar to the calibration simulation, with respect to the C2 Canal stages, the model provided a better representation of wet conditions than dry conditions.

Figure 5-94 – Validation Model Canal Stage – C2SW1

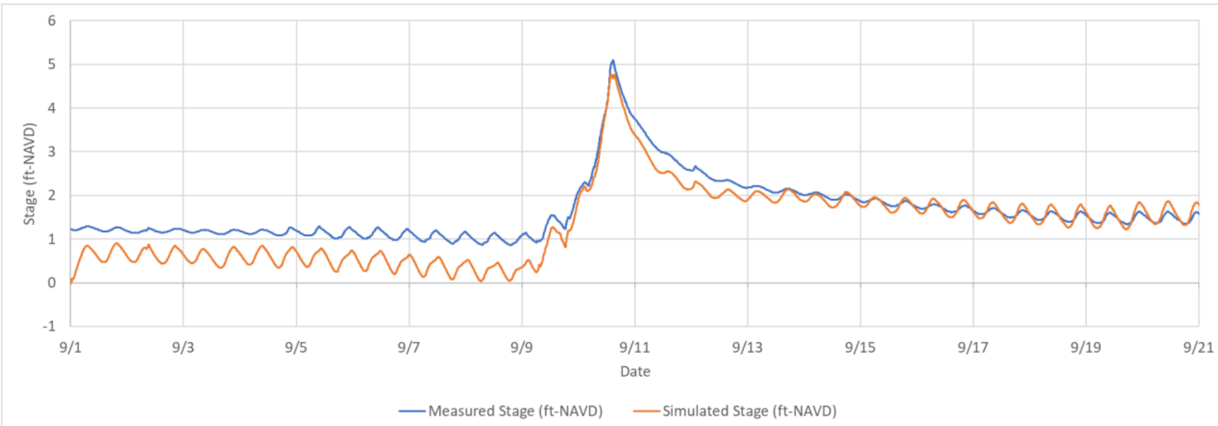


Figure 5-95 – Validation Model Canal Stage – C2SW2

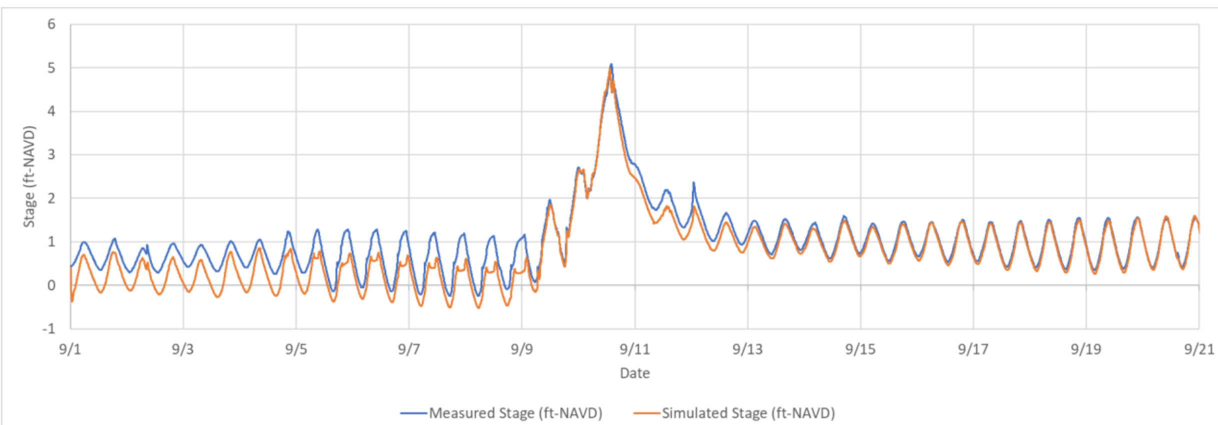
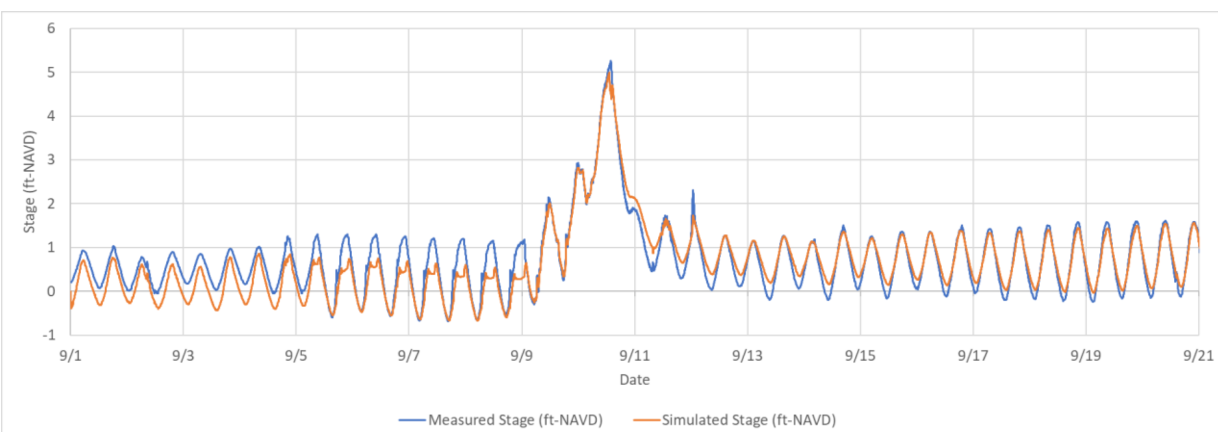


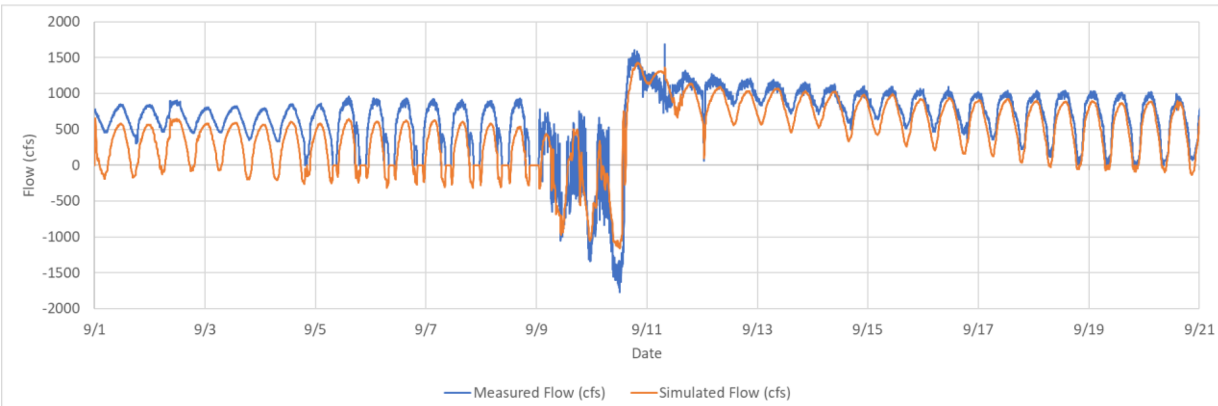
Figure 5-96 – Validation Model Canal Stage – S22-H



5.4.8.1.2. C2 WATERSHED – CANAL FLOW

Similar to the calibration simulation, because the gate geometry and opening data is an input and the tailwater condition was defined by a known boundary condition, where the headwater stage was accurately represented, the simulated flow was accurately represented as in the wet condition. For the warmup or dry period, the headwater stage was underestimated and as such that simulated flow was underestimated as well.

Figure 5-97 – Validation Model Canal Flow – S22-Q



5.4.8.1.3. C2 WATERSHED – GROUNDWATER STAGE

For the validation period the simulated groundwater stage receded from the initial condition to the rainfall response such that there was a discrepancy between the simulated and measured of between 0.5 and 2 feet. During the recharge caused by the infiltration of rainfall the Saturated Zone responded differently at each location, with the increase at G-3558 being insufficient to reach the measured elevation, while at G-3563 the increase significantly overestimated the groundwater elevation at the time of the rainfall peak. For G-3572 and G-3565 the rainfall response brought the simulated groundwater elevation up to a level that was in relative agreement with the measured values. Similar to the calibration simulation, the location where the groundwater elevation was most overestimated was in the eastern portion of the model domain.

Figure 5-98 – Validation Model Groundwater Stage – G-3558

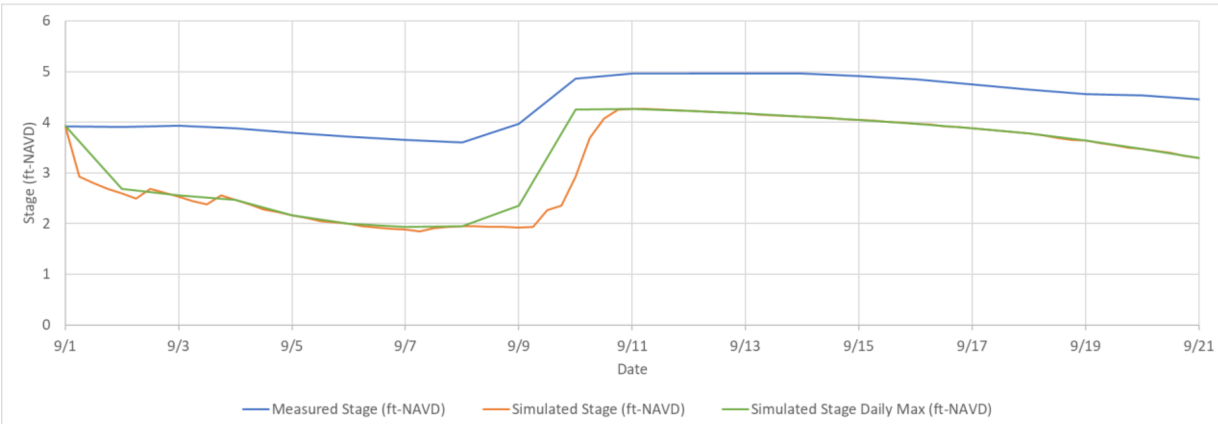


Figure 5-99 – Validation Model Groundwater Stage – G-3563

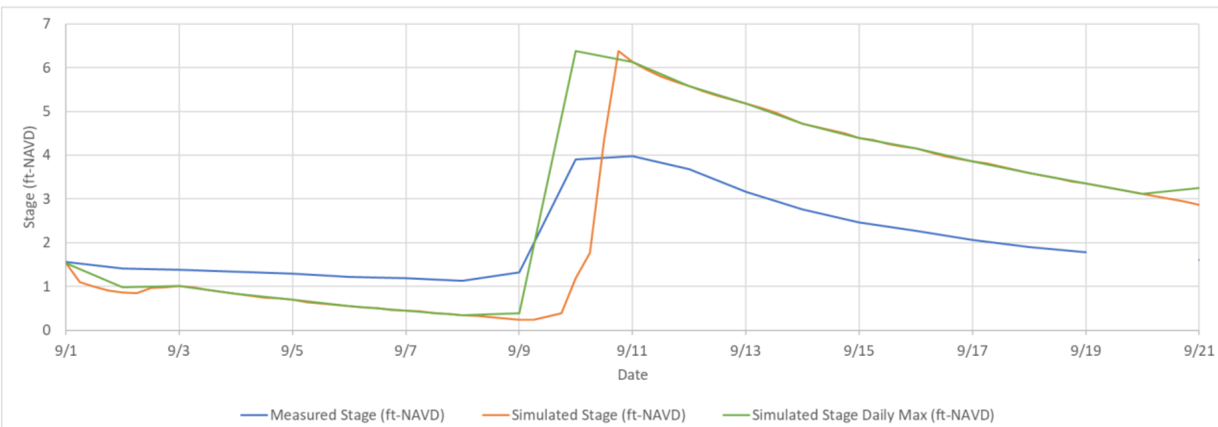


Figure 5-100 – Validation Model Groundwater Stage – G-3572

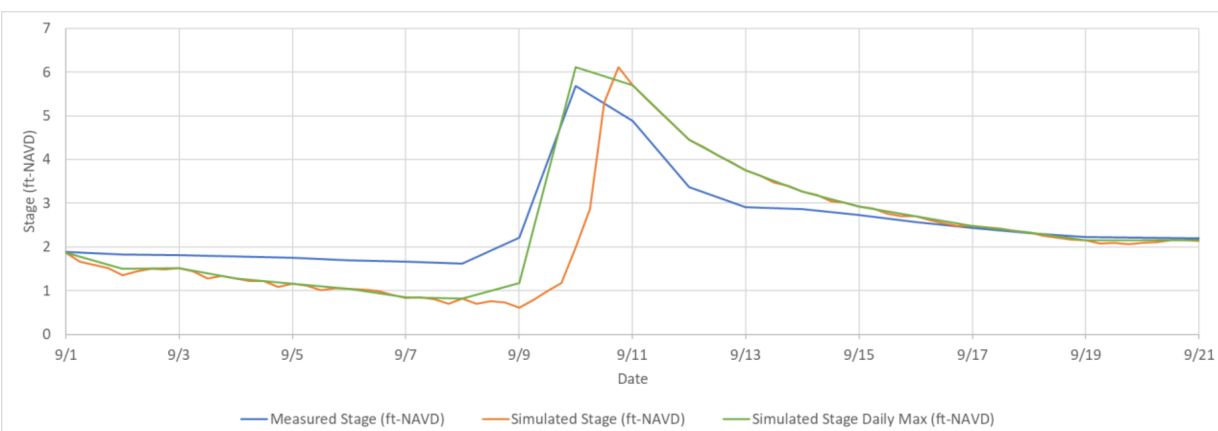
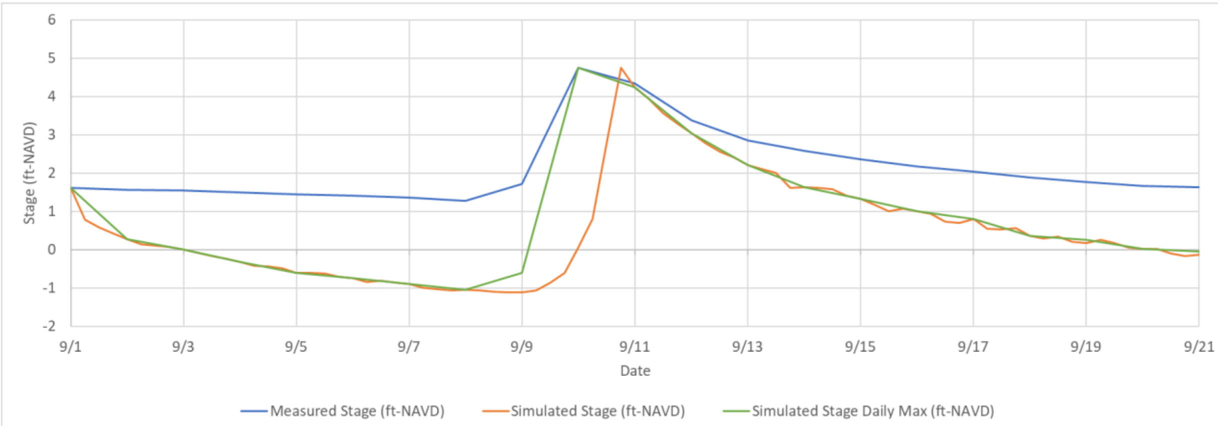


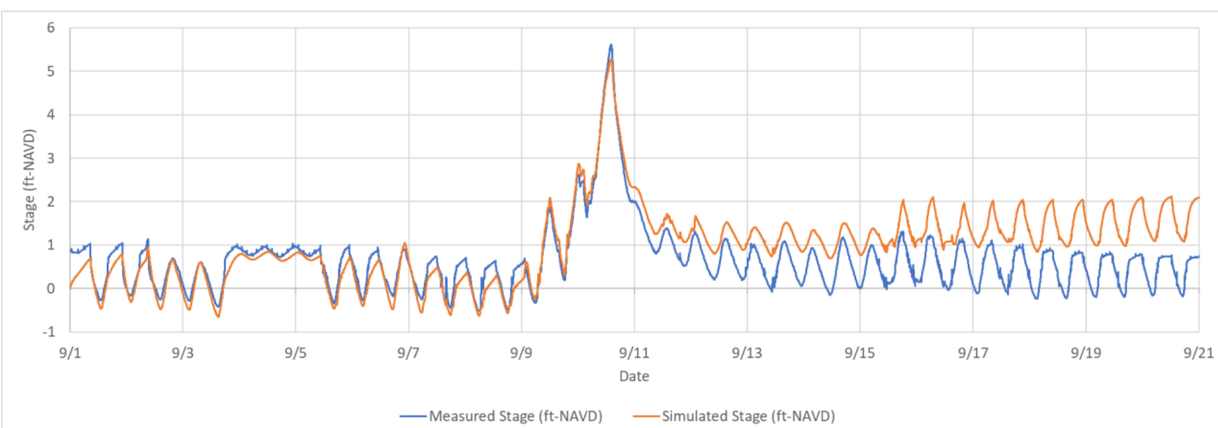
Figure 5-101 – Validation Model Groundwater Stage – G-3565



5.4.8.1.4. C3W WATERSHED – CANAL STAGE

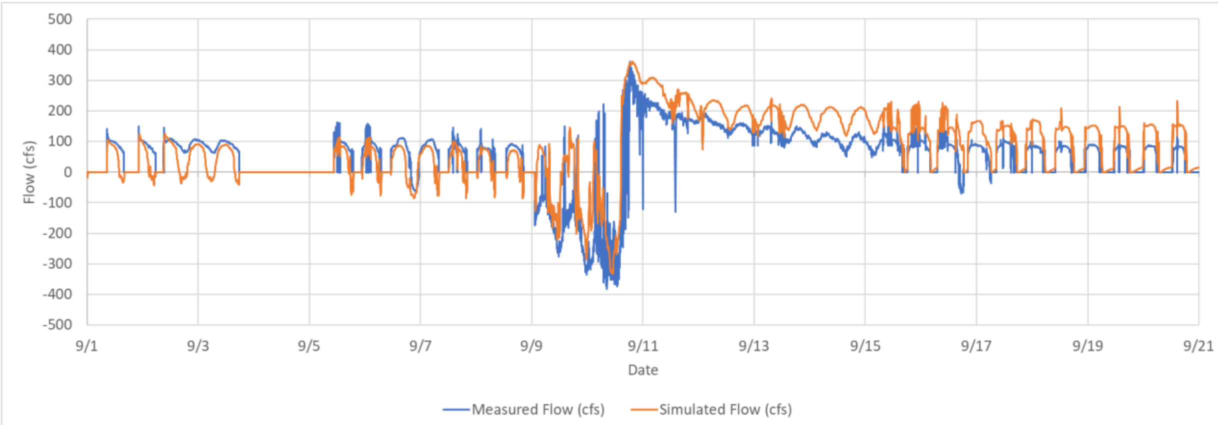
For the validation simulation, the simulated stages and flows matched the measured values closely during the warmup period and peak rainfall event. After the receding limb of the peak in the stage hydrograph there was a notable discrepancy where the stages were overestimated as compared with the measured data. This discrepancy caused an overestimation of flow at the G93 structure during periods when the gate was open. Although this discrepancy is an approximate shift of 1-ft in magnitude, the timing was after the flooding had subsided and therefore would have a minimal impact on FPLOS calculations.

Figure 5-102 – Validation Model Canal Stage – G93-H



5.4.8.1.5. C3W WATERSHED – CANAL FLOW

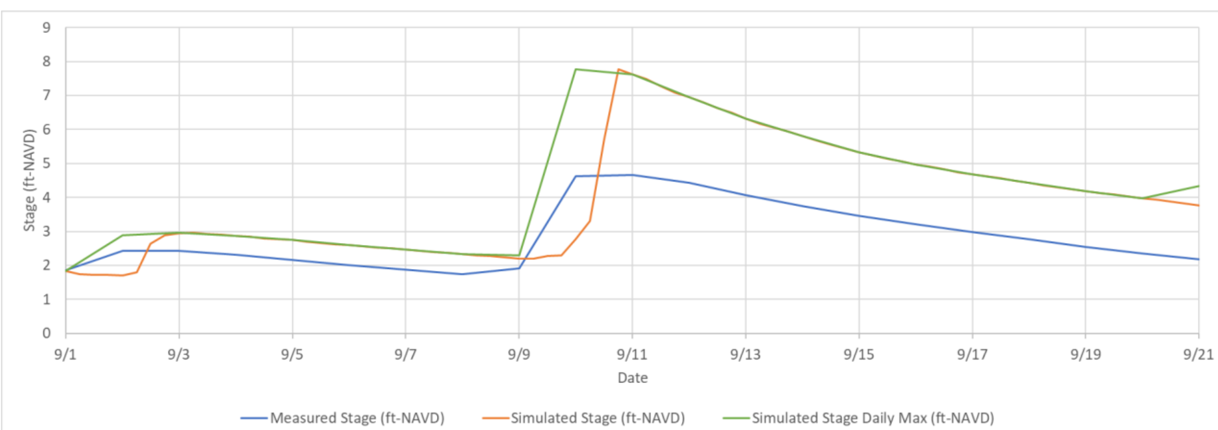
Figure 5-103: Validation Model Canal Flow – G93-Q



5.4.8.1.6. C3W WATERSHED – GROUNDWATER STAGE

Similar to G-3563 in the C2 watershed, the groundwater elevations simulated for G-3570 overestimated the response of the rainfall event significantly. Although the slope of the simulated and measured recession lines after the peak matched, the large discrepancy in the rise in groundwater elevation in response to the rainfall recharge caused the simulated values to be consistently overestimated for the remainder of the period.

Figure 5-104: Validation Model Groundwater Stage – G-3570



5.4.8.1.7. C4 WATERSHED – CANAL STAGE

The simulated stages in the C4 Canal at the tidal discharge structure (S25B) and the monitoring points on either side of the confluence with Snapper Creek (T5W and C4.Coral) demonstrated an accurate representation of the magnitude and pattern of the response to the peak rainfall event. The period before and after the rainfall event showed general agreement with measured values at these locations. In the westernmost reaches of the model, there were significant discrepancies between the simulated values and measured values at S336 and S380. The differences appear most stark in the periods when there was no rainfall, at which time the simulated stages appear to recede while the measured stages hold steady. Considering there is minimal hydraulic movement currently, it was assumed this is reflective of an exaggeration of seepage out of the canal reach and into the Saturated Zone of the model. Although this would be a concern for long-term simulation, for the FPLOS analysis this parameterization is not a critical factor.

Figure 5-105: Validation Model Canal Stage – S25B-H

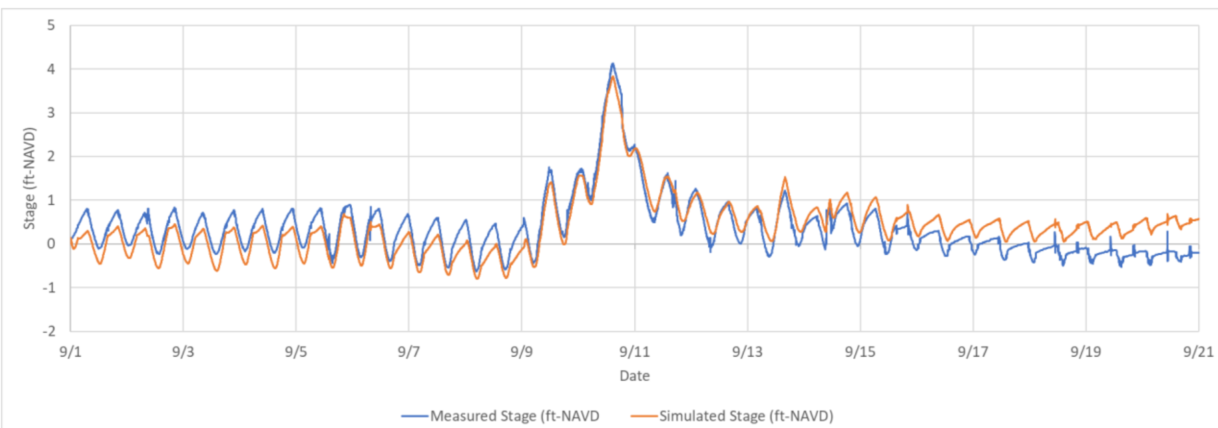


Figure 5-106: Validation Model Canal Stage – S336-T

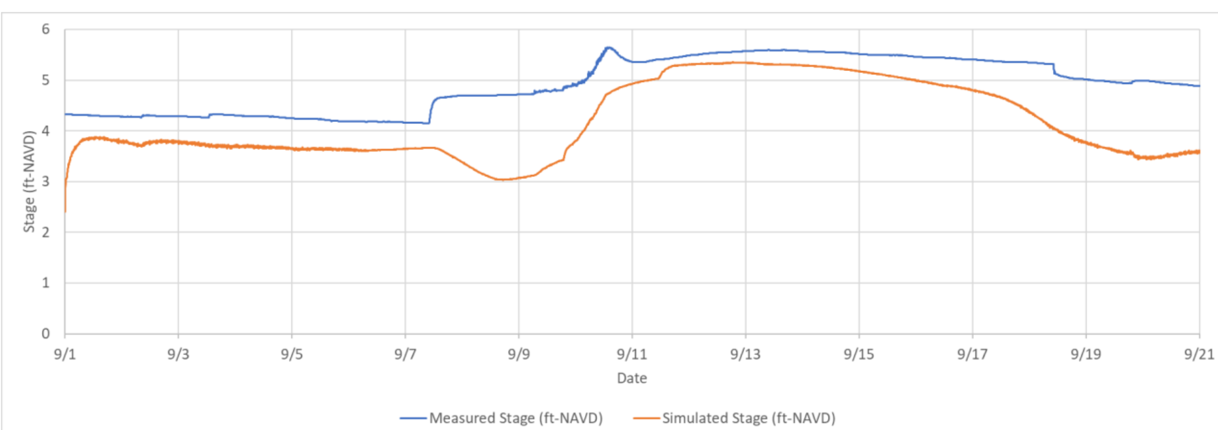


Figure 5-107: Validation Model Canal Stage – S380-H

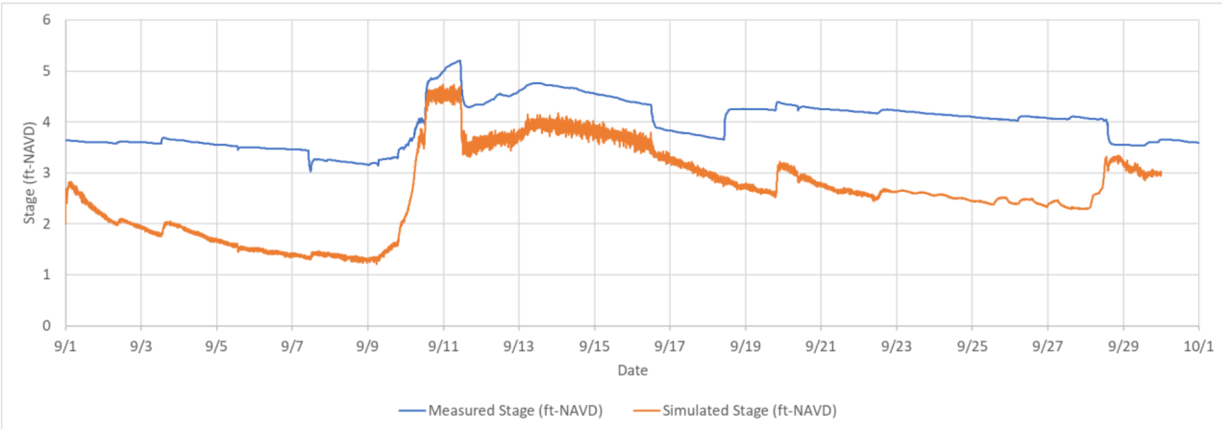


Figure 5-108: Validation Model Canal Stage – S380-T

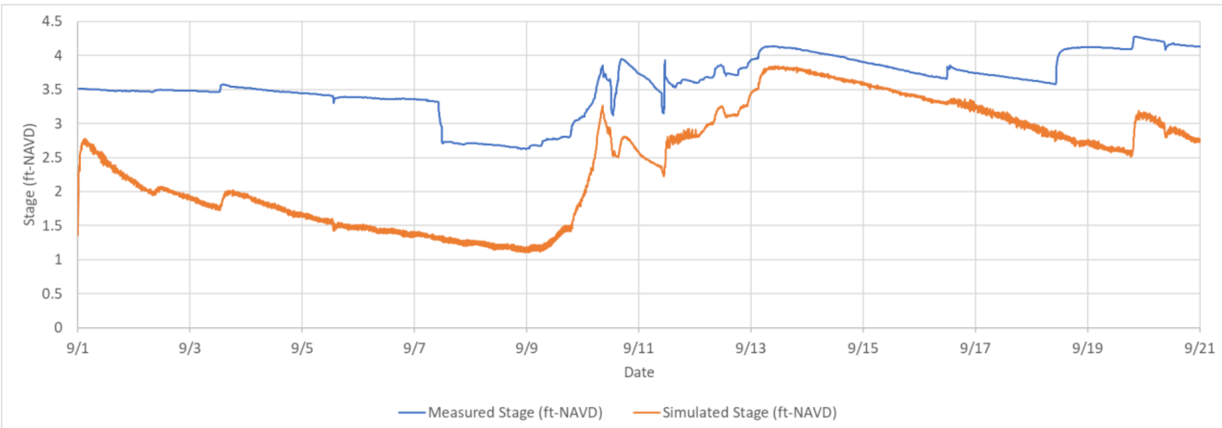


Figure 5-109: Validation Model Canal Stage – T5W

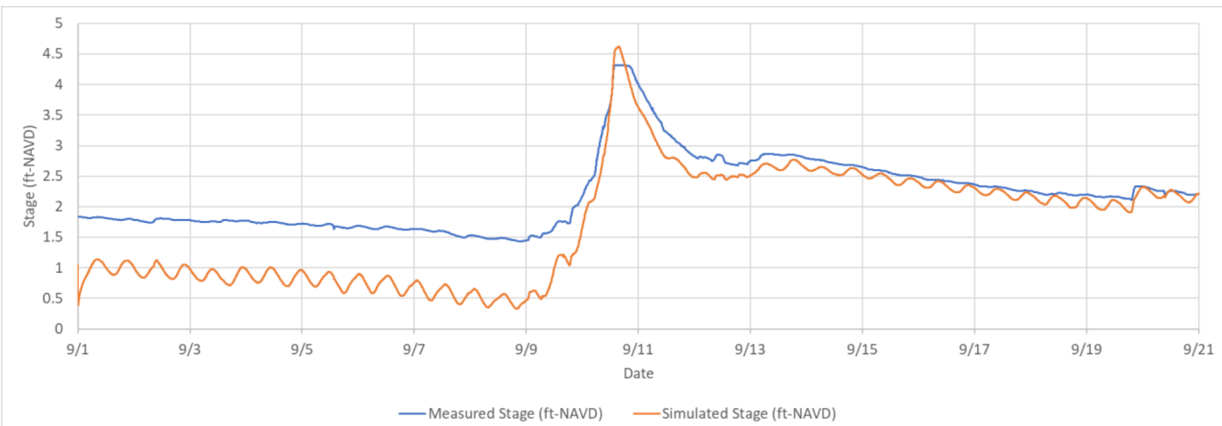
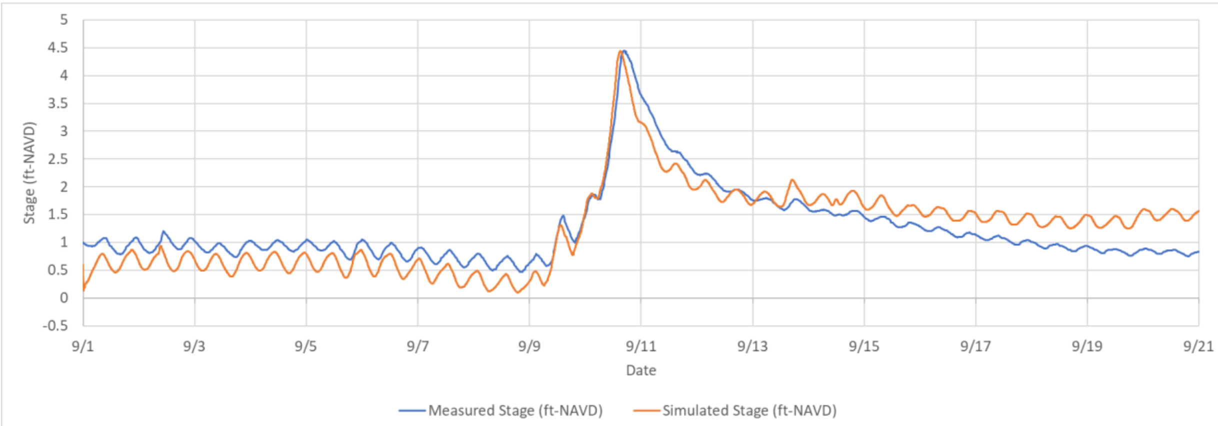


Figure 5-110: Validation Model Canal Stage – USGS 02289500 (C4.Coral)



5.4.8.1.8. C4 WATERSHED – CANAL FLOW

The simulated flow at the outlet for the C4 watershed underestimated the magnitude for the warmup period but was a very accurate representation during the peak rainfall event and after. This reflects the combined flow of the spillway and forward pump station. Similar to the calibration simulation, there were significant instabilities at the S380 structure likely due to the operations that are associated with the Emergency Detention Basin, while the S336 discharge was zero during the entire simulation period.

Figure 5-111: Validation Model Canal Flow – S25B-Q

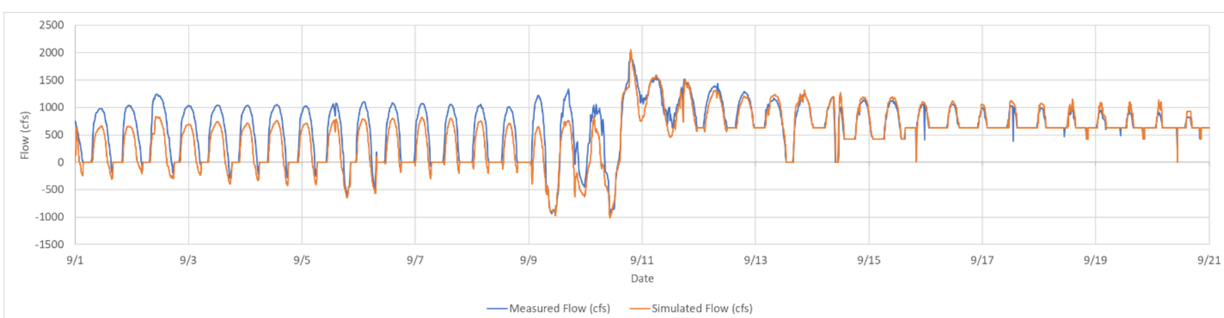


Figure 5-112: Validation Model Canal Flow – S380-Q

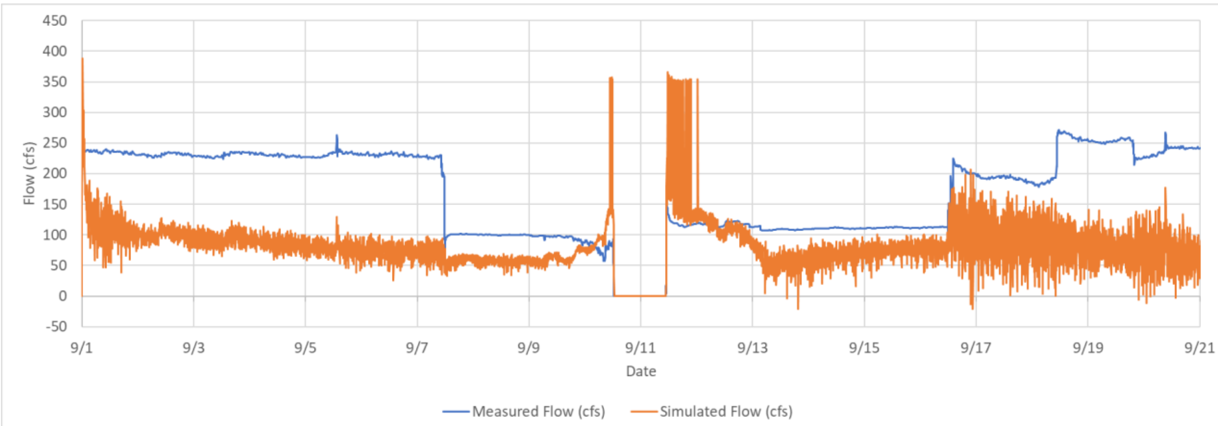
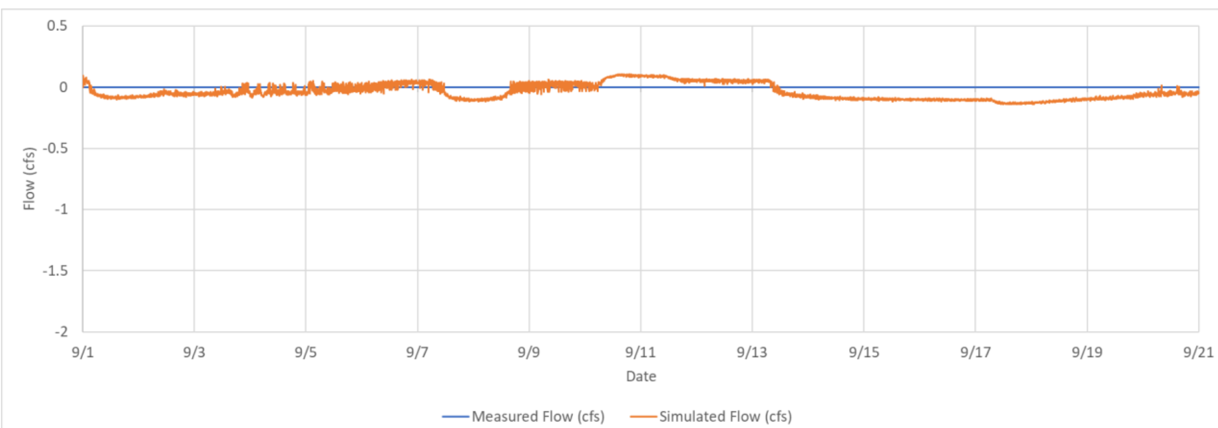


Figure 5-113: Validation Model Canal Flow – S336-Q



5.4.8.1.9. C4 WATERSHED – GROUNDWATER STAGE

Similar to other watersheds and the calibration simulation, in the C4 watershed the eastern groundwater elevations appeared to overestimate the response to the rainfall event while the western locations showed an underestimate. Of note there were a few locations where the daily maximum elevation was not recorded leaving a gap in the dataset.

Figure 5-114: Validation Model Groundwater Stage – C4GW1

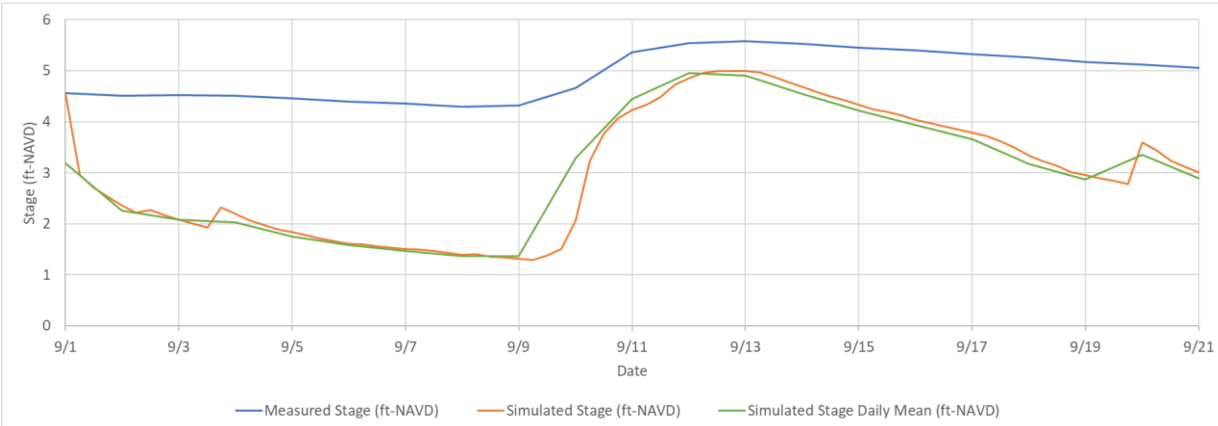


Figure 5-115: Validation Model Groundwater Stage – G-3465

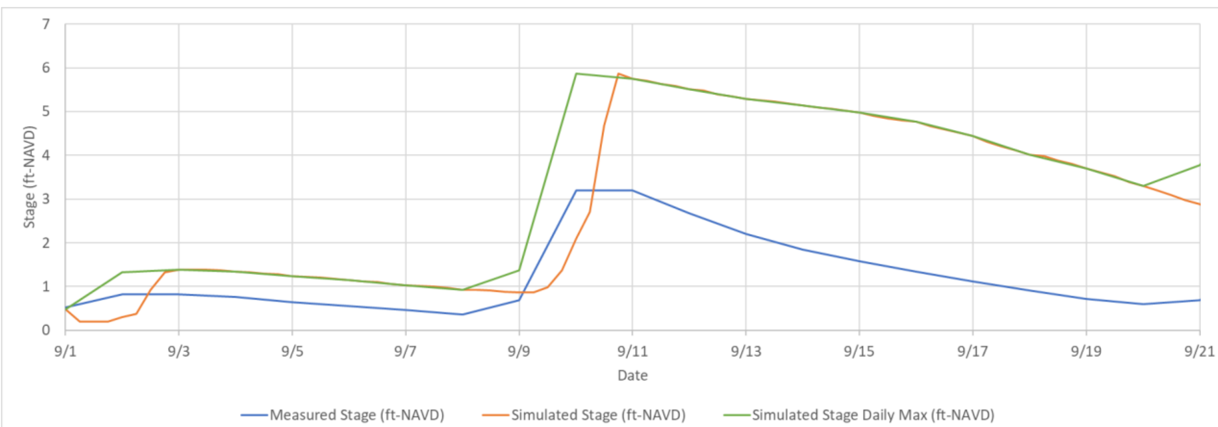


Figure 5-116: Validation Model Groundwater Stage – G-975

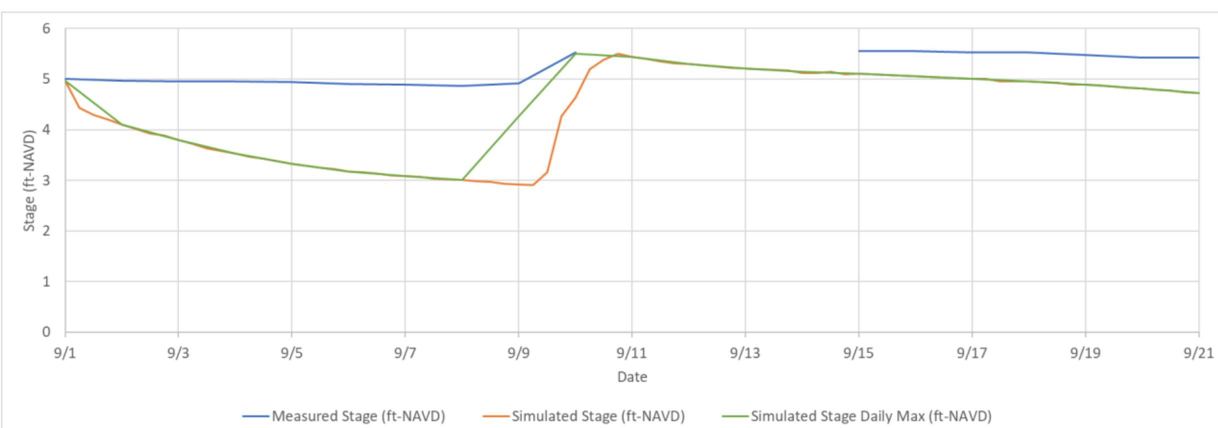
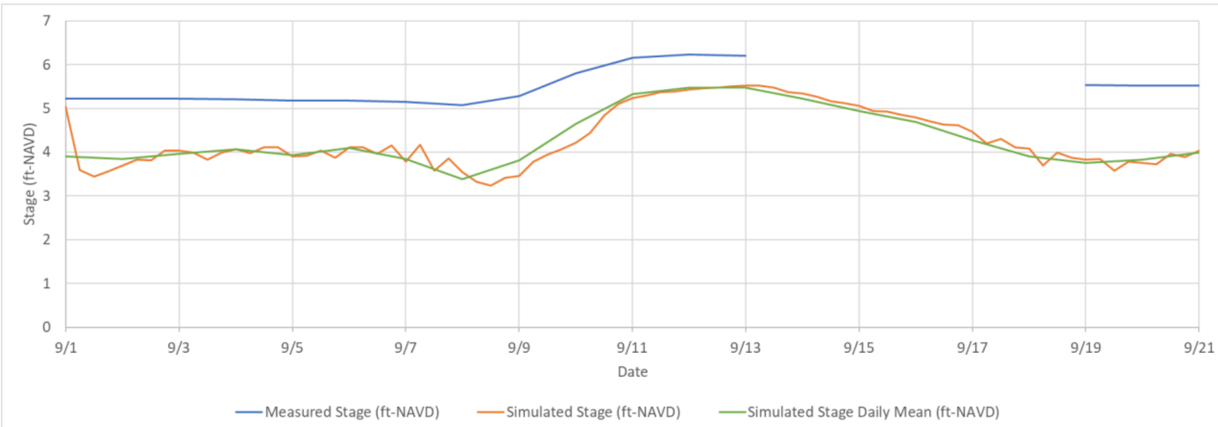


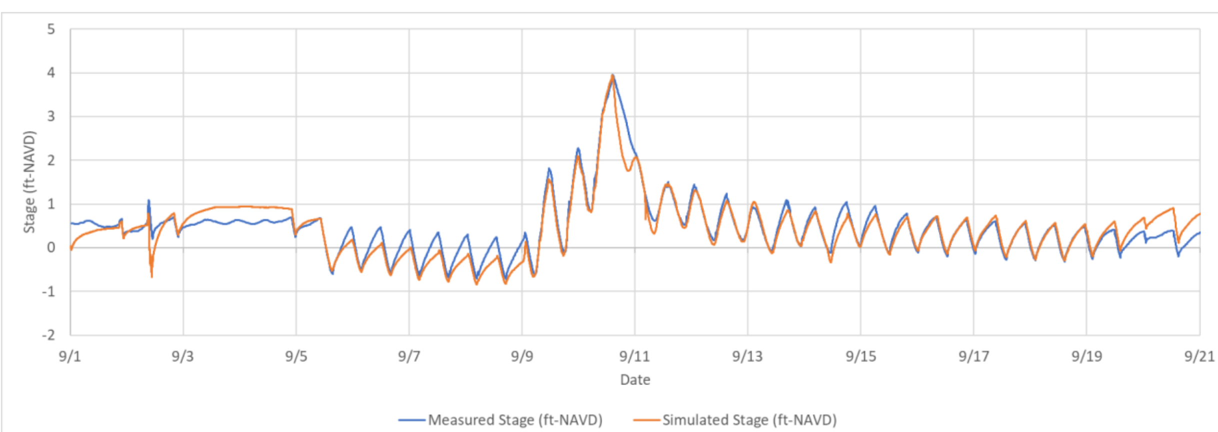
Figure 5-117: Validation Model Groundwater Stage – S356GW1



5.4.8.1.10. C5 WATERSHED – CANAL STAGE

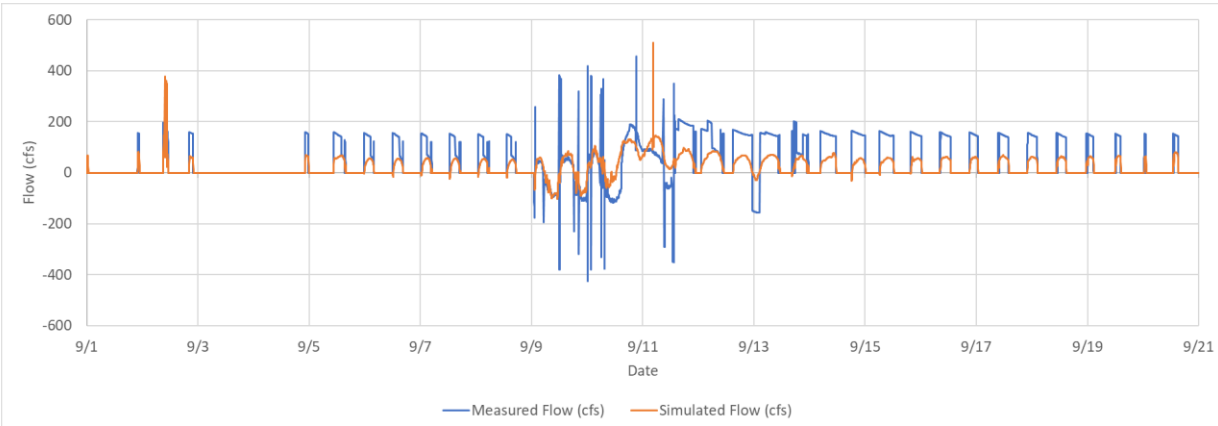
Although the upstream stage at S25 was very well represented in the validation model, the flow did not seem to be as accurate. Considering that that gate geometry and operations were model inputs, one possible reason for the discrepancy between simulated and measured flow was that the downstream stages in the Comfort Canal (C5) were higher in the simulation than reality, due to the boundary condition at the mouth of the Miami River or the simulated head losses upstream.

Figure 5-118: Validation Model Canal Stage – S25-H



5.4.8.1.11. C5 WATERSHED – CANAL FLOW

Figure 5-119: Validation Model Canal Flow – S25-Q



5.4.8.1.12. C6 WATERSHED – CANAL STAGE

The simulated headwater stage at S26 matched very closely with the measured values throughout the validation period with the exception of a single instability on September 8th during the warmup period. For the upstream structure at S31, the simulated values followed the pattern and timing of the measured values but underestimated the magnitude by roughly 0.5 to 1.0 feet. One parameter modification that would address this condition is the roughness of the C6 Canal, however inquiries with Field Station Staff into canal conditions indicated that vegetation maintenance had recently occurred, and the canal was relatively clean during this period.

Figure 5-120: Validation Model Canal Stage – S26-H

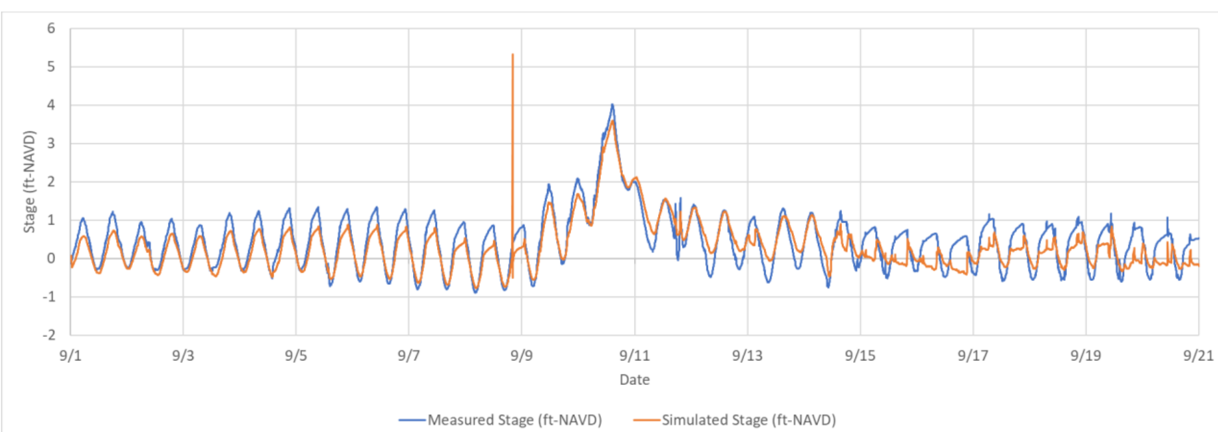
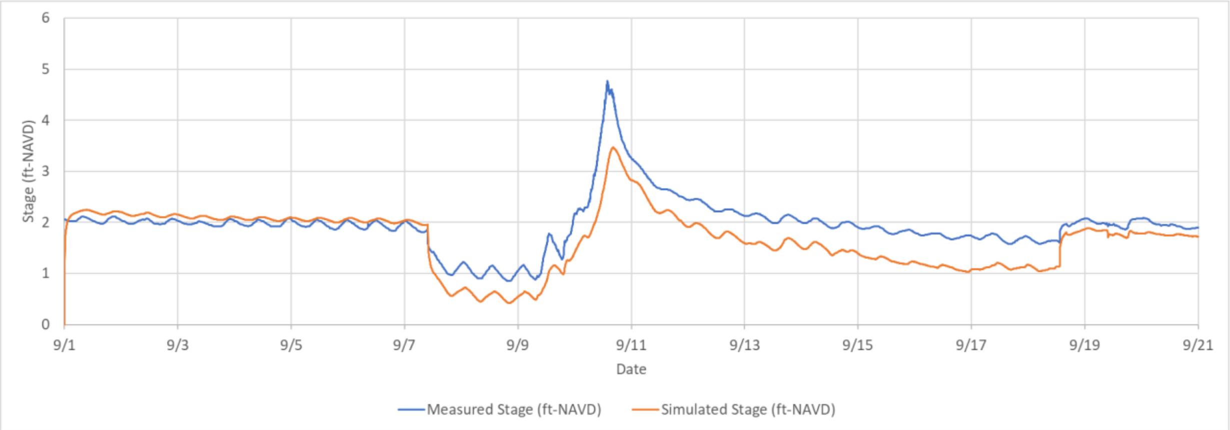


Figure 5-121: Validation Model Canal Stage – S31-T



5.4.8.1.13. C6 WATERSHED – CANAL FLOW

Similar to the simulated flow at S25, the flow at S26 was underestimated in comparison to the measured flow. Considering that the headwater stage was accurately represented, and the gate geometry, operations and pump station discharges are all model inputs, the most likely cause of this discrepancy is the simulated tailwater of the structure either due to an inaccurate boundary condition at the mouth of the Miami River or an inaccurate estimate of head losses upstream. The flow at S31 was accurately represented in the validation model as the gate is notably closed during the peak of the storm.

Figure 5-122: Validation Model Canal Flow – S26-Q

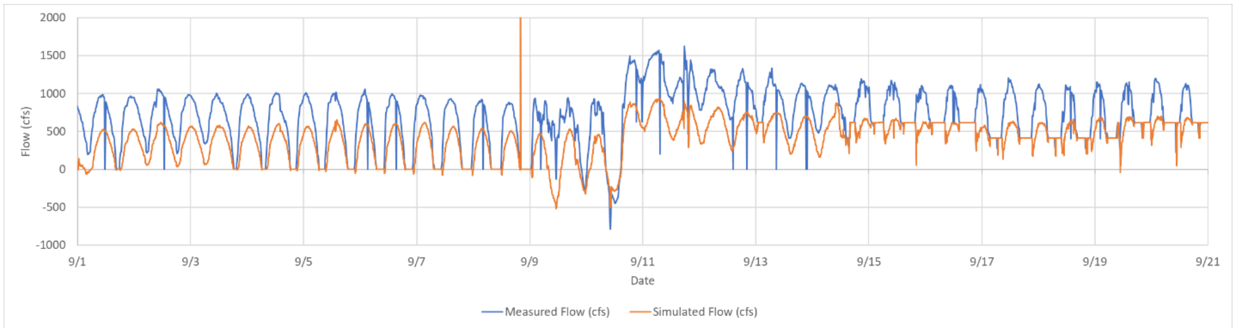
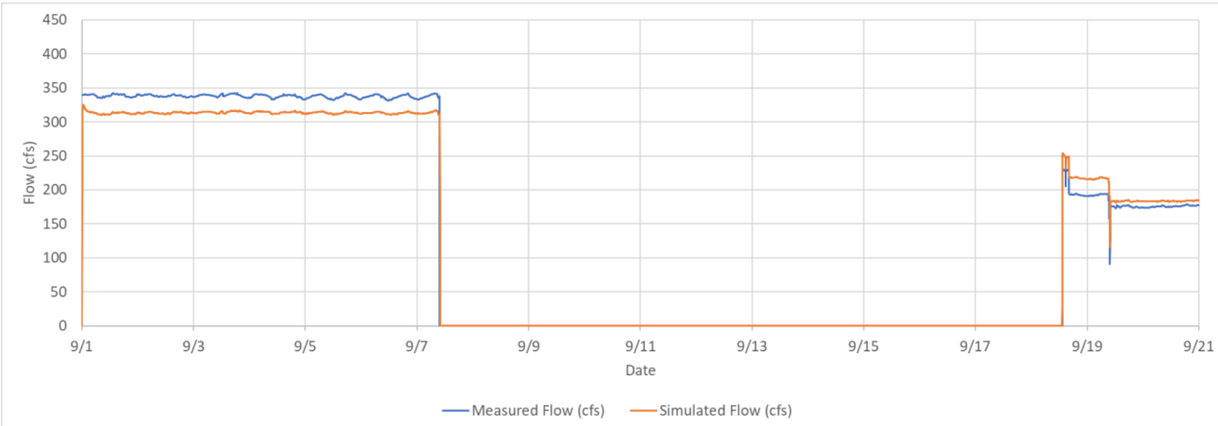


Figure 5-123: Validation Model Canal Flow – S31-Q



5.4.8.1.14. C6 WATERSHED – GROUNDWATER STAGE

Unlike other watersheds, the differentiation between overestimating and underestimating the peak groundwater elevation was not based on the eastern or western location of the monitoring well. The G-3 location is in the eastern portion of the model domain and underestimated the measured stage similarly to the G-3567 which is in the far northwest portion of the model domain. The S-68 location overestimated the peak groundwater elevation and is located slightly southeast of the G-3 station, both of which are in the Miami Gardens area just north of the Miami Airport.

Figure 5-124: Validation Model Groundwater Stage – G-3

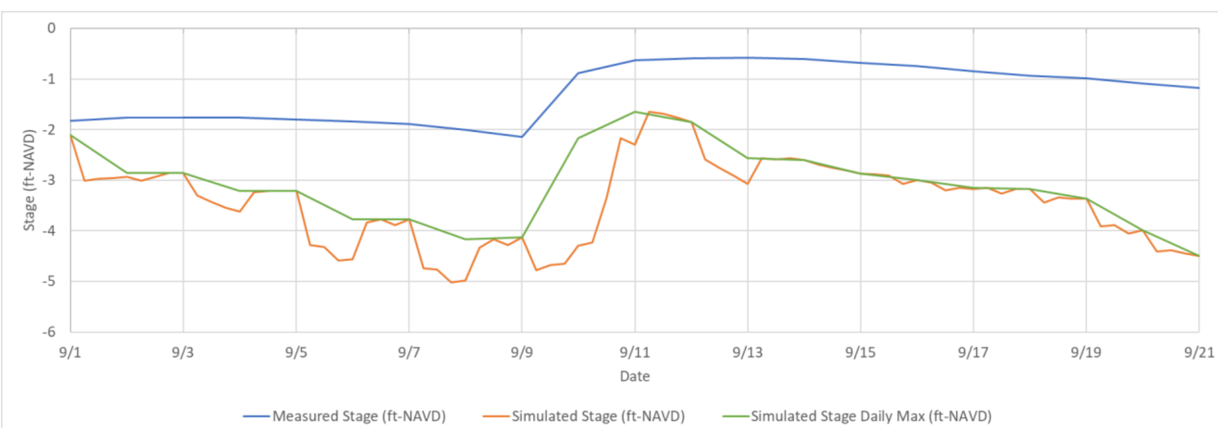


Figure 5-125: Validation Model Groundwater Stage – G-3567

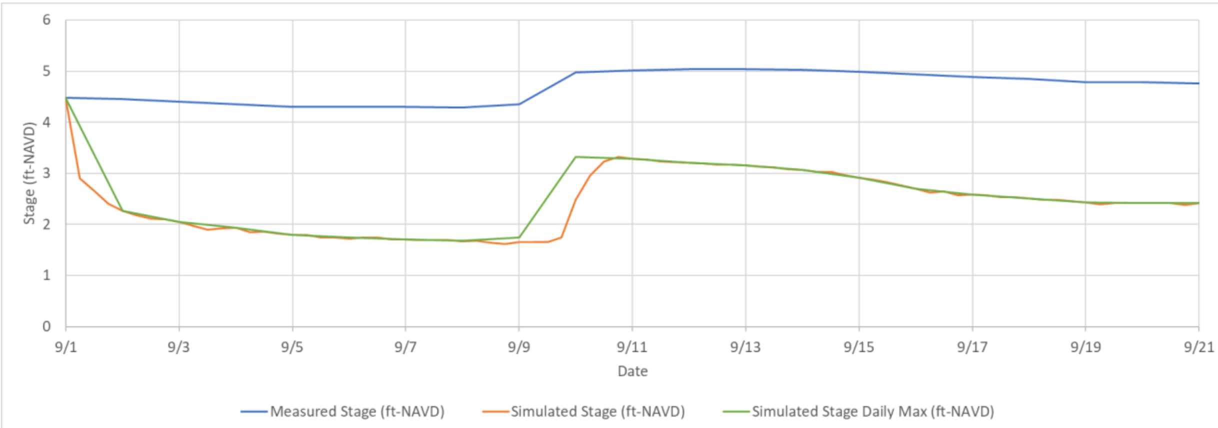
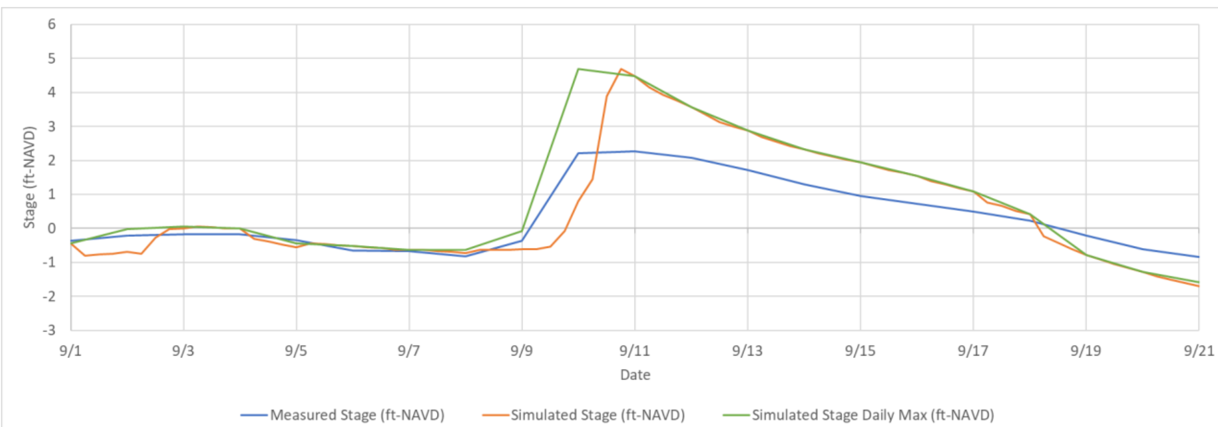


Figure 5-126: Validation Model Groundwater Stage – S-68



5.4.8.2. STATISTICAL COMPARISON

To supplement the visual comparison of the measured and simulated values for the validation run, a statistical analysis was performed to quantify the performance of the model. The statistical metrics utilized were the same as the statistical analysis for the calibration. As described in **Table 5-32** through **Table 5-37** statistics were performed for the canal stages, structure flows and groundwater elevations.

The statistical analysis was performed for the full model run and for the 7-day period that captures the rainfall event (September 8 through 14, 2017). A review of simulated canal stage data for the full simulation period shows nine (9) of the thirteen (13) locations had a mean absolute error less than the calibration target of 0.5 feet, and for the peak rainfall period eight (8) locations were within tolerance for canal stages. For the statistical analysis of structure discharge, the error varied significantly by location which reflects the simulation performance and the magnitude of flows at each location. The correlation results demonstrated that the model's ability to represent the measured flows varies by location. The groundwater statistics clearly demonstrate that the validation simulation was not as accurate as the calibration simulation. For the peak runoff period, all the monitoring locations had a mean absolute error that exceeded 1.0 foot.

Table 5-32: Validation Model Canal Stage Statistics – Full Model Run

WATERSHED	STATION	30 - DAY SIMULATION (9/01/2017 - 9/30/2017)				
		ME	MAE	RMSE	R	NS
C2	C2SW1	0.19	0.28	0.37	0.93	0.76
	C2SW2	0.17	0.19	0.26	0.97	0.88
	S22H	0.05	0.18	0.24	.096	0.92
C3W	G93H	-0.51	0.62	0.75	0.83	0.28
C4	C4.Coral	-0.07	0.43	0.54	0.57	0.05
	S25BH	-0.16	0.38	0.46	0.84	0.65
	S336T	0.84	0.84	0.93	0.81	-3.26
	S380H	1.25	1.25	1.36	0.86	-7.11
	S380T	1.16	1.16	1.26	0.70	-9.62
	T5W	0.34	0.38	0.51	0.92	0.25
C5	S25H	-0.08	0.25	0.46	0.81	0.55
C6	S26H	0.06	0.23	0.29	0.90	0.80
	S31T	0.25	0.38	0.51	0.80	0.53

Table 5-33: Validation Model Canal Stage Statistics – Peak Rain Period

WATERSHED	STATION	7 - DAY SIMULATION (9/08/2017 - 9/14/2017)				
		ME	MAE	RMSE	R	NS
C2	C2SW1	0.35	0.35	0.41	0.99	0.88
	C2SW2	0.22	0.23	0.28	0.99	0.94
	S22H	0.01	0.20	0.27	0.98	.096
C3W	G93H	-0.21	0.51	0.75	0.75	0.17
C4	C4.Coral	0.45	0.51	0.75	0.75	0.17
	S25BH	0.03	0.23	0.26	0.97	0.94
	S336T	0.89	0.89	1.00	0.95	-8.00
	S380H	0.94	0.94	1.08	0.96	-1.83
	S380T	0.97	0.97	1.04	0.94	-4.40
	T5W	0.44	0.45	0.53	0.99	0.60
C5	S25H	0.14	0.16	0.25	0.98	0.94
C6	S26H	0.01	0.23	0.29	0.96	0.91
	S31T	0.68	0.68	0.77	0.94	0.56

Table 5-34: Validation Model Flow Statistics – Full Model Run

WATERSHED	STATION	30 - DAY SIMULATION (9/01/2017 - 9/30/2017)				
		ME	MAE	RMSE	R	NS
C2	S22Q	192.32	218.32	273.51	0.88	0.54
C3W	G93Q	-37.28	53.73	69.49	0.81	0.40
C4	S25BQ	-291.76	457.43	514.45	0.54	-0.54
	S380Q	82.50	97.69	118.61	0.16	-1.37
	S336Q	0.02	0.07	0.37	0.21958	0.044326
C5	S25Q	18.96	31.51	57.09	0.67	0.35
C6	S26Q	-43.15	272.41	327.12	0.42	0.13
	S31Q	-0.33	12.58	33.51	0.968493	0.93746

Table 5-35: Validation Model Flow Statistics – Peak Rain Period

WATERSHED	STATION	7 - DAY SIMULATION (9/08/2017 – 9/14/2017)				
		ME	MAE	RMSE	R	NS
C2	S22Q	185.52	286.33	357.16	0.89	0.72
C3W	G93Q	-58.92	74.71	100.06	0.89	0.68
C4	S25BQ	-176.24	360.98	421.14	0.81	0.47
	S380Q	-8.43	36.81	52.50	0.74	-0.02
	S336Q	-0.03	0.05	0.06	-1.00	-1.00
C5	S25Q	16.63	52.37	80.77	0.61	0.33
C6	S26Q	176.47	331.24	381.01	0.74	0.40
	S31Q	0.00	0.00	0.00	-1.00	-1.00

Table 5-36: Validation Model Groundwater Stage Statistics – Full Model Run

Watershed	Station	30 - Day Simulation (9/01/2017 – 9/30/2017)				
		ME	MAE	RMSE	R	NS
C2	G-3558	1.17	1.17	1.25	0.85	-7.62
	G-3563	-0.76	1.32	1.48	0.68	-2.61
	G-3572	0.16	0.55	0.91	0.61	-0.12
	G-3565	1.56	1.56	1.80	0.65	-4.04
C3W	G-3570	-1.55	1.69	1.90	0.67	-3.72
C4	C4GW1	2.05	2.05	2.14	0.83	-29.09
	G-3465	-2.58	2.65	3.06	0.45	-14.30
	G-975	0.90	0.90	1.04	0.86	-18.11
	S356GW1	1.19	1.19	1.25	0.71	-15.05
C6	G-3	2.63	2.63	2.84	0.22	-35.06
	G-3567	2.12	2.12	2.19	0.56	-78.29
	S-68	-0.13	0.77	1.00	0.87	-0.01

Table 5-37: Validation Model Groundwater Stage Statistics – Peak Rain Period

Watershed	Station	7 - Day Simulation (9/08/2017 – 9/14/2017)				
		ME	MAE	RMSE	R	NS
C2	G-3558	1.25	1.25	1.37	0.90	-5.65
	G-3563	-0.46	1.84	1.94	0.69	-2.11
	G-3572	0.35	1.40	1.74	0.45	-0.70
	G-3565	1.70	1.70	2.32	0.58	-2.81
C3W	G-3570	-1.46	1.88	2.15	0.74	-2.41
C4	C4GW1	1.99	1.99	2.13	1.00	-14.12
	G-3465	-1.92	2.12	2.51	0.68	-4.67
	G-975	1.54	1.54	1.63	0.98	-28.42
	S356GW1	1.14	1.14	1.21	0.93	-6.02
C6	G-3	3.11	3.11	3.21	0.82	-23.26
	G-3567	2.10	2.10	2.15	0.95	-45.12
	S-68	-0.77	1.27	1.49	0.81	-0.59

Table 5-38 provides a comparison of peak stages for the validation event between measured and simulated. This comparison demonstrates that the peak stages at eight (8) of the eleven (11) locations had an absolute difference less than the 0.5 feet target tolerance and an average discrepancy of 15% as compared with measured peak stages.

Table 5-39 illustrates the difference in simulated and measured peak flow, with a calibration target of ±20% of the peak. The values at S25B and S26 each include the simulated discharge for both the spillway and the forward pump station.

Table 5-38: Validation Model Peak Stage Comparison

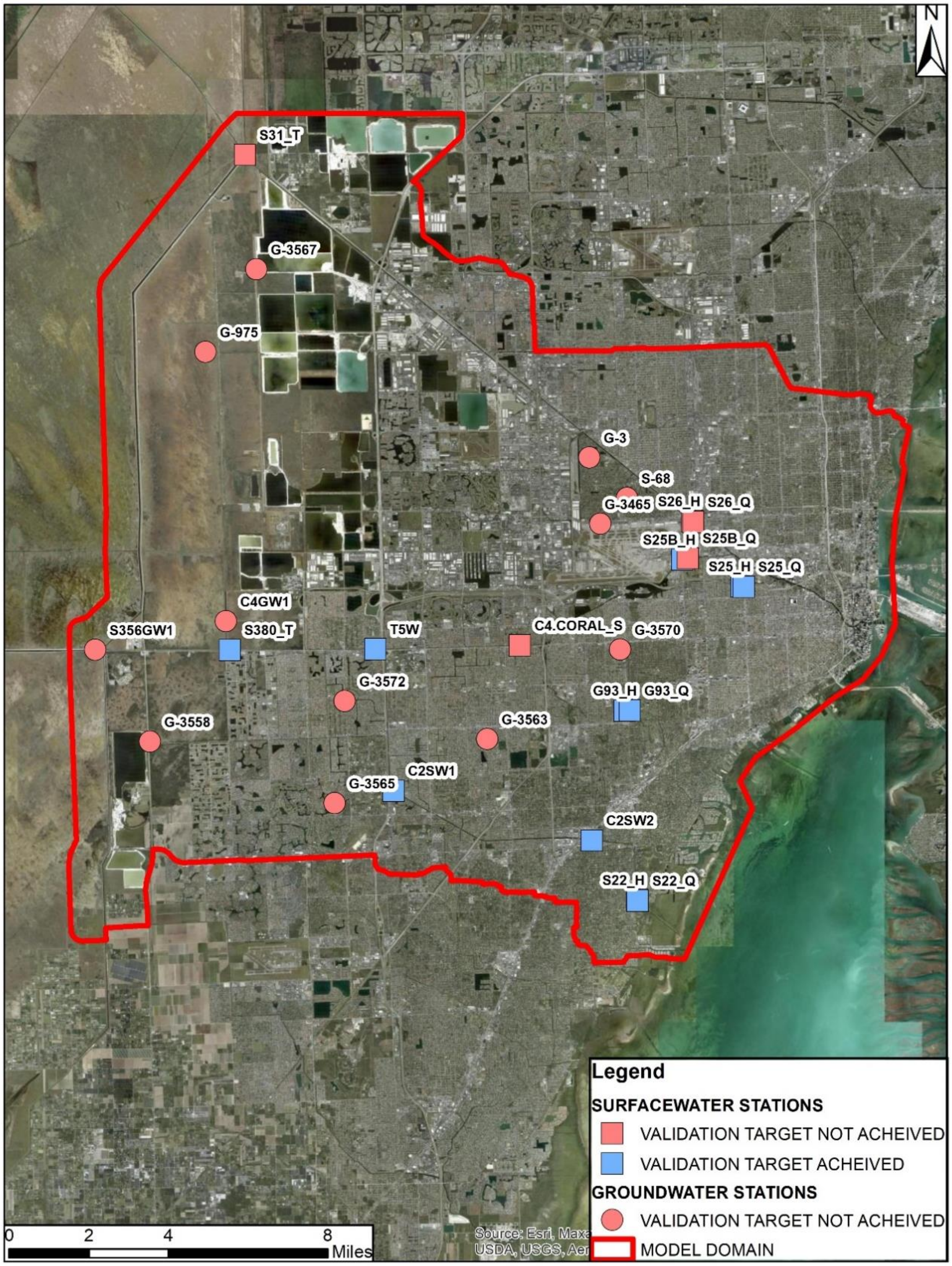
Calibration Point	Peak Stage [FT-NAVD]			
	Measured	Simulated	Absolute Difference	Percent Difference
C2SW1	5.088	4.775	0.313	6%
C2SW2	5.085	4.985	0.1	2%
C4.Canal	3.268	4.442	1.174	36%
G93H	5.618	5.29	0.328	6%
S22H	5.258	4.993	0.265	5%
S25BH	4.131	3.838	0.293	7%
S25H	3.955	3.944	0.011	0%
S26H	4.015	5.329	1.314	33%
S31T	4.768	3.471	1.297	27%
S380T	4.275	3.846	0.429	10%
T5W	4.318	4.622	0.304	7%

Table 5-39: Validation Model Peak Flow Comparison

Calibration Point	Peak Flow [CFS]			
	Measured	Simulated	Absolute Difference	Percent Difference
S25	457.8	510.6	52.8	12%
S25B	1693.7	2056.5	362.7	21%
S26	1365.6	3457.0	2091.4	153%
G93	362.9	360.8	2.1	1%
S22	1690.8	1430.6	260.2	15%

Based on these statistical analyses, a graphic was developed illustrating the locations where the MAE for the calibration simulation was within the targeted tolerance as shown in **Figure 5-127**.

Figure 5-127: Location of Validation Points with MAE within Target Tolerance



5.4.8.3. WATER BUDGET EVALUATION BY WATERSHED

Table 5-40 through **Table 5-54** provide the results of a computation of the water balance for the C2, C3W, C4, C5 and C6 watersheds. Similar to the calibration results these water balance tables describe the water budget for the groundwater and hydrologic simulations associated with MIKE SHE and the water budget for the hydraulic model associated with MIKE 1D.

All water balance calculations were performed in terms of inches in consideration of the significant differences in the contributing areas of each watershed. These results demonstrate that for the validation simulation, the model does not lose or gain water from an unquantifiable or unknown source and the magnitude of movement of water within the budget is within reasonable values.

The MIKE 1D Water Balance tables provide the total 1D inflows and outflows (converted to inches), and when the overland and saturated zone exchanges with the 1D rivers are considered, the total change in storage for the channels is calculated (Channel Δ Storage). However, additional inflows or outflows from channel pumping within the watershed, and not at the watershed boundary, can throw this Channel Δ Storage calculation off, and create large storage values. For example, **Table 5-44** shows a significant Channel Δ Storage value of -12.31 inches for the C3W Watershed. While the inflows do not account for the significant outflows, there is a municipal pump station represented in the watershed for West Miami, which pumps 13 inches into the basin, which when added to the calculation, changes the Channel Δ Storage value to 0.69 inches. It should be noted that the water budget for the calibration does not show the same significant Channel Δ Storage values for the C3W watershed, see **Table 5-21**, as the pump is not on for the majority of the simulation in the calibration simulation.

The C4 Watershed also shows a significant Channel Δ Storage value of -3.29 inches (see **Table 5-47**), which is also due the municipal pump stations (at 7 locations within the C4 Watershed) and the C4 Emergency Detention Basin (which pumps water from the C4 Canal into the basin). Depending on whether pumping into or out of the C4 Canal is dominant will determine the sign and magnitude of the Channel Δ Storage value.

Table 5-40: C2 Watershed (33,511 ac) – MIKE SHE Water Balance

MIKE SHE Water Balance [in]	In	Out
Rainfall	11.89	
ET		1.84
SZ -> River		5.28
River -> SZ	1.48	
OL -> River		0.38
GW Boundary Inflows	1.74	
GW Boundary Outflows		3.45
OL Boundary Inflows	0.00	
OL Boundary Outflows		0.00
GW Pumping		2.85
Total	**15.11**	**13.80**
OL Change in Storage	0.23	
GW Change in Storage	1.12	
Residual	0.06	

Table 5-41: C2 Watershed (33,511 ac) – MIKE 1D Water Balance

Channel Water Balance [in]	In	Out
MIKE 1D Inflows	5.43	
MIKE 1D Outflows		9.54
OL -> River	0.38	
SZ -> River	5.28	
River -> SZ		1.48
Total	**11.10**	**11.02**
Channel ∆ Storage	0.08	

Table 5-42: C2 Watershed (33,511 ac) – MIKE 1D Inflow and Outflow

Inflow Location	Total Flow [in]
Snapper Creek Canal	4.89
SW 132nd Ave Canal	0.60
Coral Way Canal	-0.06

Outflow Location	Total Flow [in]
S121	0.00
S22	9.54

Table 5-43: C3W Watershed (3,580 ac) – MIKE SHE Water Balance

MIKE SHE Water Balance [in]	In	Out
Rainfall	10.88	
ET		1.44
SZ -> River		5.77
River -> SZ	0.09	
OL -> River		0.31
GW Boundary Inflows	2.02	
GW Boundary Outflows		2.75
OL Boundary Inflows	0.00	
OL Boundary Outflows		0.00
GW Pumping		0.00
Total	**12.99**	**10.27**
OL Change in Storage	0.37	
GW Change in Storage	2.29	
Residual	-0.04	

Table 5-44: C3W Watershed (3,580 ac) – MIKE 1D Water Balance

Channel Water Balance [in]	In	Out
MIKE 1D Inflows	-3.03	
MIKE 1D Outflows		15.28
OL -> River	0.31	
SZ -> River	5.77	
River -> SZ		0.09
Total	**3.06**	**15.37**
Channel ∆ Storage	-12.31*	

*see discussion in Section 5.4.8.3

Table 5-45: C3W Watershed (3,580 ac) – MIKE 1D Inflow and Outflow

Inflow Location	Total Flow [in]
Coral Way Canal	-3.03

Outflow Location	Total Flow [in]
G93	15.28

Table 5-46: C4 Watershed (53,904 ac) – MIKE SHE Water Balance

MIKE SHE Water Balance [in]	In	Out
Rainfall	12.18	
ET		2.49
SZ -> River		23.60
River -> SZ	0.31	
OL -> River		-14.74
GW Boundary Inflows	2.70	
GW Boundary Outflows		0.97
OL Boundary Inflows	0.02	
OL Boundary Outflows		0.01
GW Pumping		0.97
Total	**15.21**	**13.30**
OL Change in Storage	0.65	
GW Change in Storage	1.10	
Residual	-0.15	

Table 5-47: C4 Watershed (53,904 ac) – MIKE 1D Water Balance

Channel Water Balance [in]	In	Out
MIKE 1D Inflows	-0.42	
MIKE 1D Outflows		11.42
OL -> River	-14.74	
SZ -> River	23.60	
River -> SZ		0.31
Total	**8.44**	**11.73**
Channel ∆ Storage	-3.29*	

*see discussion in Section 5.4.8.3

Table 5-48: C4 Watershed (53,904 ac) – MIKE 1D Inflow and Outflow

INFLOW LOCATION	TOTAL FLOW [IN]
S-336	0.00
FEC C4 Canal	-0.42

OUTFLOW LOCATION	TOTAL FLOW [IN]
S25A	0.00
S25B	7.16
Snapper Creek Canal	3.04
SW 132 Ave Canal	0.37
Coral Gables Canal	-0.03
Dressels Dairy Canal	0.88

Table 5-49: C5 Watershed (1,215 ac) – MIKE SHE Water Balance

MIKE SHE WATER BALANCE [IN]	IN	OUT
Rainfall	11.96	
ET		1.19
SZ -> River		9.06
River -> SZ	0.41	
OL -> River		1.33
GW Boundary Inflows	5.14	
GW Boundary Outflows		2.65
OL Boundary Inflows	0.02	
OL Boundary Outflows		0.00
GW Pumping		0.00
TOTAL	**17.53**	**14.23**
OL Change in Storage	0.22	
GW Change in Storage	3.06	
Residual	0.03	

Table 5-50: C5 Watershed (1,215 ac) – MIKE 1D Water Balance

CHANNEL WATER BALANCE [IN]	IN	OUT
MIKE 1D Inflows	0.00	
MIKE 1D Outflows		10.54
OL -> River	1.33	
SZ -> River	9.06	
River -> SZ		0.41
TOTAL	13.39	10.96
Channel ∆ Storage	-0.57	

Table 5-51: C5 Watershed (1,215 ac) – MIKE 1D Inflow and Outflow

INFLOW LOCATION	TOTAL FLOW [IN]
S25A	0.00

OUTFLOW LOCATION	TOTAL FLOW [IN]
S25	10.54

Table 5-52: C6 Watershed (33,919 ac) – MIKE SHE Water Balance

MIKE SHE WATER BALANCE [IN]	IN	OUT
Rainfall	13.70	
ET		1.89
SZ -> River		14.14
River -> SZ	1.65	
OL -> River		-7.19
GW Boundary Inflows	1.86	
GW Boundary Outflows		1.94
OL Boundary Inflows	0.01	
OL Boundary Outflows		0.12
GW Pumping		2.70
TOTAL	17.22	13.60
OL Change in Storage	0.89	
GW Change in Storage	2.65	
Residual	-0.05	

Table 5-53: C6 Watershed (33,919 ac) – MIKE 1D Water Balance

CHANNEL WATER BALANCE [IN]	IN	OUT
MIKE 1D Inflows	2.90	
MIKE 1D Outflows		8.29
OL -> River	-7.19	
SZ -> River	14.14	
River -> SZ		1.65
TOTAL	**9.85**	**9.94**
Channel Δ Storage	-0.09	

Table 5-54: C6 Watershed (33,919 ac) – MIKE 1D Inflow and Outflow

INFLOW LOCATION	TOTAL FLOW [IN]
G72	0.00
S31	2.84
S32	0.00
S32A	0.00
Red Road Canal	0.21
NW 87 Ave Canal	-0.15
OUTFLOW LOCATION	**TOTAL FLOW [IN]**
S26	8.96
FEC C4 Canal	-0.67

5.5. ADDENDUM TO THE FINAL CALIBRATION AND VALIDATION

In December 2021, Chen Moore and Associated (CMA) submitted a revised Model Calibration and Validation Report responding to comments on the calibration and validation of the MIKE SHE / MIKE 1D model of the C2, C3W, C4, C5, and C6 watersheds. This report included several revisions to the simulation including allowing overbank spilling, reducing the computational timestep, reducing the overland stability parameters, modifying the separated flow areas and drain codes, and modifications to the tailwater boundary conditions. In January 2022, the District issued additional comments on the revised final calibration report with a focus on four items in particular:

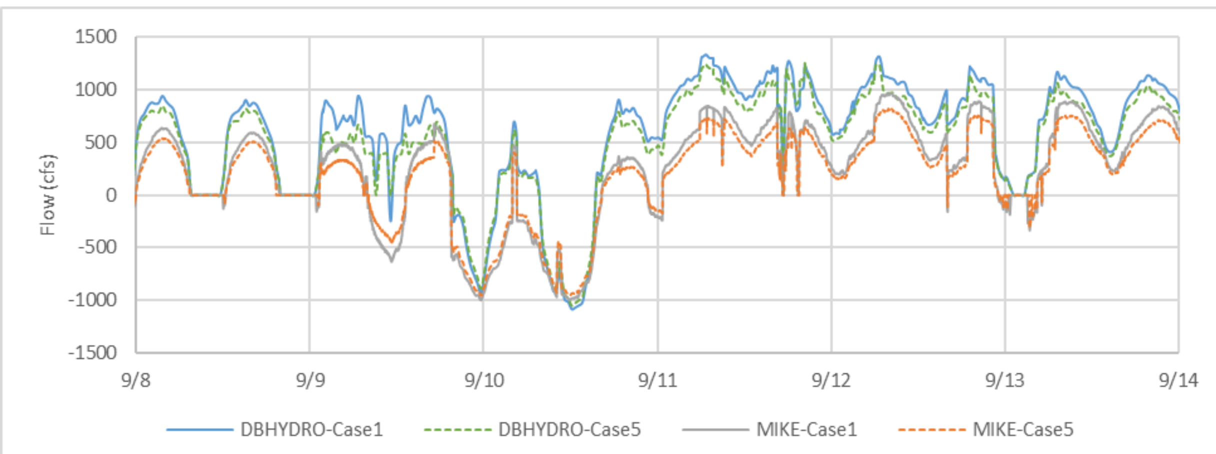
1. Editorial and format improvements.
2. Poor matching between measured and simulated groundwater results.
3. Poor matching between measured and simulated flows at S26.
4. Unusually high negative numbers in the water budget for the C4 Watershed.

At the February 10, 2022, progress meeting it was agreed that the calibration and validation would be revisited with a specific focus on the issues identified. It was also agreed that the results of the model revisions would be documented in a separate memorandum that would serve as an addendum to the report. Accordingly, the following memorandum sections address revisions to the groundwater calibration, verification of the S26 flow calculation and modifications to the calibration, and an evaluation of the water budget methodology.

5.5.1. S26 FLOW CALIBRATION

The mechanism for calculating flow within the model was verified against the Case 1 and Case 5 flow rating equations from the District for S26 (SFWMD, 2015). The stage results were used to calculate Case 1 and Case 5 flow results and were compared with DBHydro calculated flows for Case 1 and Case 5 in **Figure 5-128**. This initial analysis demonstrated that for a given set of head conditions and gate openings, the model flow computation approach provided an accurate calculation of flows.

Figure 5-128. Flow comparison at S26



After demonstrating that the model was correctly calculating the flows based on the simulated head conditions, the focus for improving calibration and validation performance was to develop a better representation of the upstream head condition. When reviewing the reported inflows (S31) and outflows (S26) for the C6 Canal during the validation period, there is a difference of roughly 1,365 CFS at the peak of the event. Because there are only 6 (six) culverts in the upstream reach delivering runoff from the adjacent contributing area and secondary canal, it is assumed that most of that inflow is from groundwater baseflow. Modifications to the model's Saturated Zone parameterization (described in the sections that follow) provided an improvement in the baseflow delivered to the canal and thereby improved the simulated headwater at S26. **Figure 5-129** and

Figure 5-130 demonstrate the comparison of measured and simulated stages for the calibration and validation simulation.

Figure 5-129. Comparison of Updated Validation Results and Measured Data for S26 Gate Flows

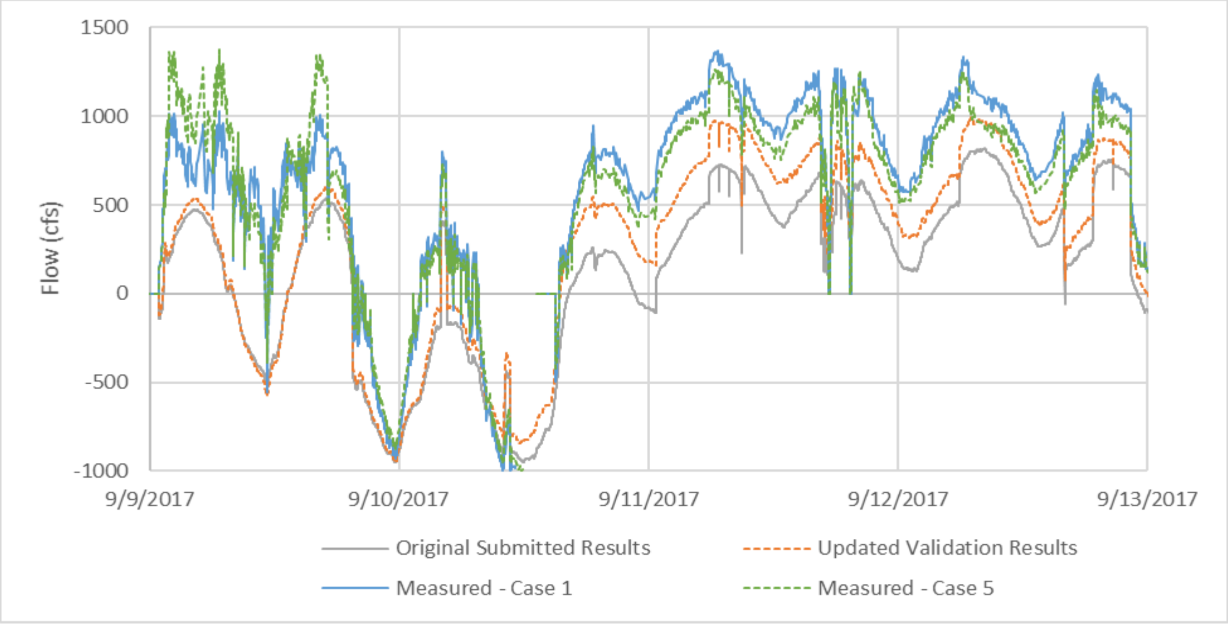
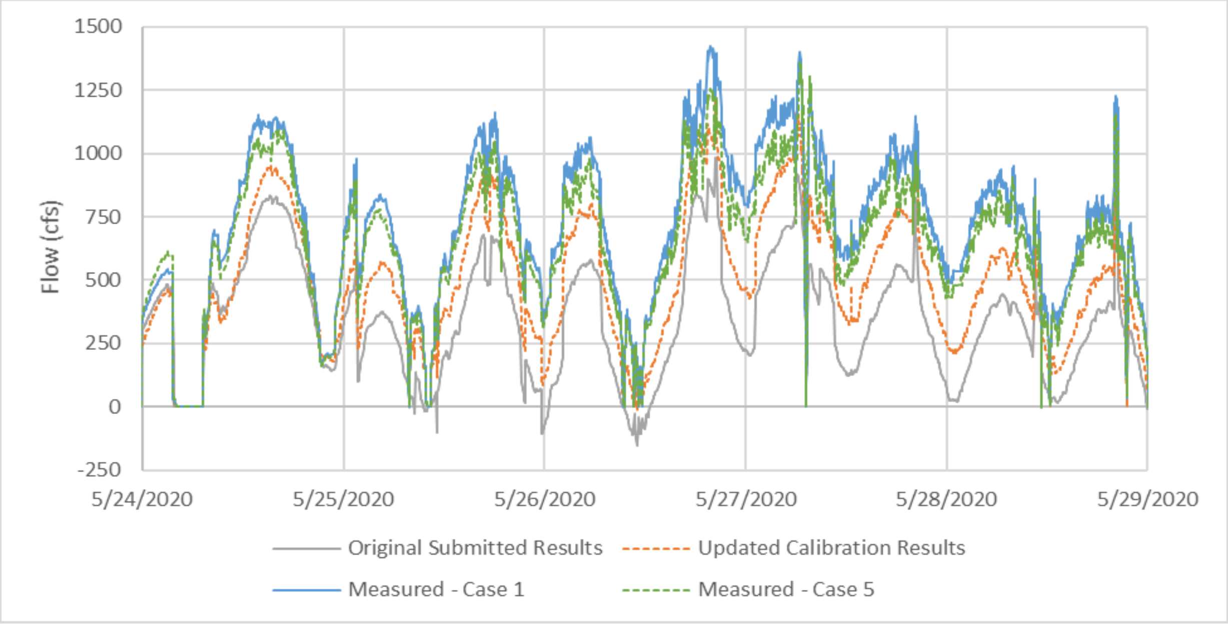


Figure 5-130. Comparison of Updated Calibration Results and Measured Data for S26 Gate Flows



In addition to flows at S26, the Case 5 flows at S25B were also provided by the District. **Figure 5-131** and **Figure 5-132** show the original submitted and updated results for the S25B gate flows and compares them with the Case 5 calculated flows from measured data. In both the calibration and validation simulations the flows increased with the updated results in improved model performance. This improvement is due to the modifications to groundwater and surface water discussed in subsequent sections of this addendum.

Although the DBHYDRO flows are generated using the Case 1 approach, the Case 5 approach is recommended for statistical comparison at both S26 and S25B, as it is more directly comparable to the simulated flows using the MIKE 1D sluice formula.

Figure 5-131. Comparison of Updated Validation Results and Measured Data for S25B Gate Flows

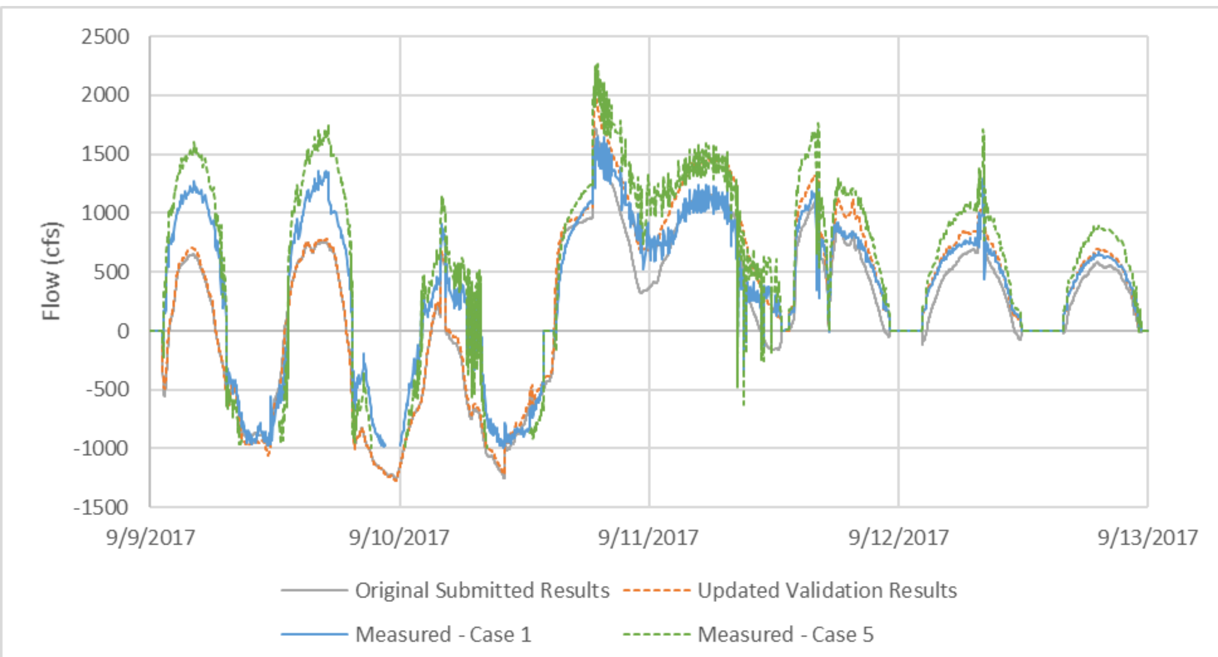
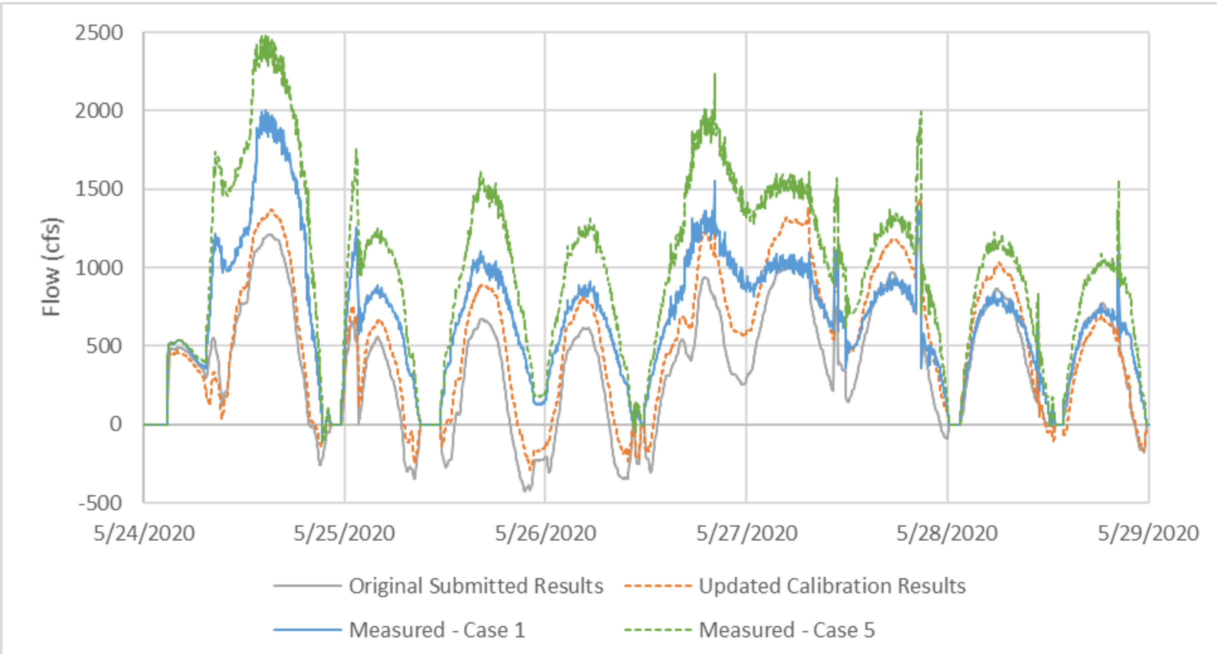


Figure 5-132. Comparison of Updated Calibration Results and Measured Data for S25B Gate Flows



5.5.2. WATER BUDGET FOR C4 EMERGENCY DETENTION BASIN

During initial calibration, the calibration water budget for "Overland to River" in the C4 and C6 Watershed was (-)7.48 inches and (-)4.27 inches, respectively. These values represent the volume of water that moved from model reaches to the land surface over the course of the model simulation as expressed in terms of a depth distributed over the entire water budget calculation area. The validation water budget for the "Overland to River" had similarly negative results, with the C4 and C6 Watershed having values of (-)14.74 inches and (-)7.19 inches, respectively.

To evaluate the source of these negative values, the water budget calculations were altered such that the C4 Watershed was split into 2 halves: areas east of the Turnpike and areas west of the Turnpike. This approach isolated the mining lakes, which are all located in the C4 Watershed west of the Turnpike. **Table 5-55** and **Table 5-56** compare these results within the C4 watershed.

Table 5-55. C4 Watershed – East of Turnpike – MIKE SHE Water Balance

MIKE SHE Water Balance [in]	In	Out
Rainfall	13.74	
ET		1.44
SZ -> River		8.20
River -> SZ	0.22	
OL -> River		0.72
GW Boundary Inflows	1.64	
GW Boundary Outflows		1.40
OL Boundary Inflows	0.02	
OL Boundary Outflows		0.02
GW Pumping		0.00
Total	**15.62**	**11.78**
OL Change in Storage	1.20	
GW Change in Storage	2.54	
Residual	-0.09	

Table 5-56. C4 Watershed – West of Turnpike – MIKE SHE Water Balance

MIKE SHE Water Balance [in]	In	Out
Rainfall	11.50	
ET		2.92
SZ -> River		30.17
River -> SZ	2.45	
OL -> River		-20.73
GW Boundary Inflows	2.77	
GW Boundary Outflows		1.90
OL Boundary Inflows	0.09	
OL Boundary Outflows		0.06
GW Pumping		1.35
Total	**16.81**	**15.67**
OL Change in Storage	0.46	
GW Change in Storage	0.50	
Residual	-0.18	

The results demonstrated in **Table 5-55** and **Table 5-56** illustrate that the negative values evident in the reported water budget analysis for the C4 Watershed were isolated in the areas west of the Turnpike. This region of the watershed is the location where there are several rock mining pits and the C4 Emergency Detention Basin. Additional water budget calculations were performed for the C4 Emergency Detention Basin alone to further explore this issue. **Table 5-57** describes the results of this additional calculation.

Table 5-57. C4 Watershed – Emergency Detention Basin – MIKE SHE Water Balance

MIKE SHE Water Balance [IN]	In	Out
Rainfall	13.05	
ET		3.21
SZ -> River		0.22
River -> SZ	0.65	
OL -> River		-77.77
GW Boundary Inflows	3.12	
GW Boundary Outflows		90.45
OL Boundary Inflows	0.00	
OL Boundary Outflows		0.00
GW Pumping		0.00
Total	**16.82**	**16.11**
OL Change in Storage	0.00	
GW Change in Storage	0.71	
Residual	0.00	

These results demonstrate that the unusual values seen in the initial Calibration Report were due to a large volume of water moving from model reach to the overland area within the C4 Emergency Detention Basin. This result is caused by the representation within the model of the Detention Basin as a river reach with flood codes that allowed water to move to the overland module when the topography was exceeded. **Figure 5-133** illustrates the flood code model parameterization that was used. The areas in red represent the Emergency Detention Basin, while the blue and green colored areas represent the mining lakes.

Reviewing the validation results for the Emergency Detention Basin in detail, the calculation showed 77 inches of River to Overland for the Detention Basin water budget, which is 4,966 acre-feet or 6.5 feet over the footprint of the Detention Basin. **Figure 5-134** shows the flows of water within the water budget components for the Detention Basin.

Based on this review, it is evident that the negative values in the water budget calculations presented in the initial calibration were caused by the mechanics of the model's representation of the detention basin and regional rock mining operations as river reaches with flood codes included. As such, the negative results in the water budget represent storage areas being filled and are not a signal of overbank flooding in developed locations.

Figure 5-133. Flood Codes Parameterization

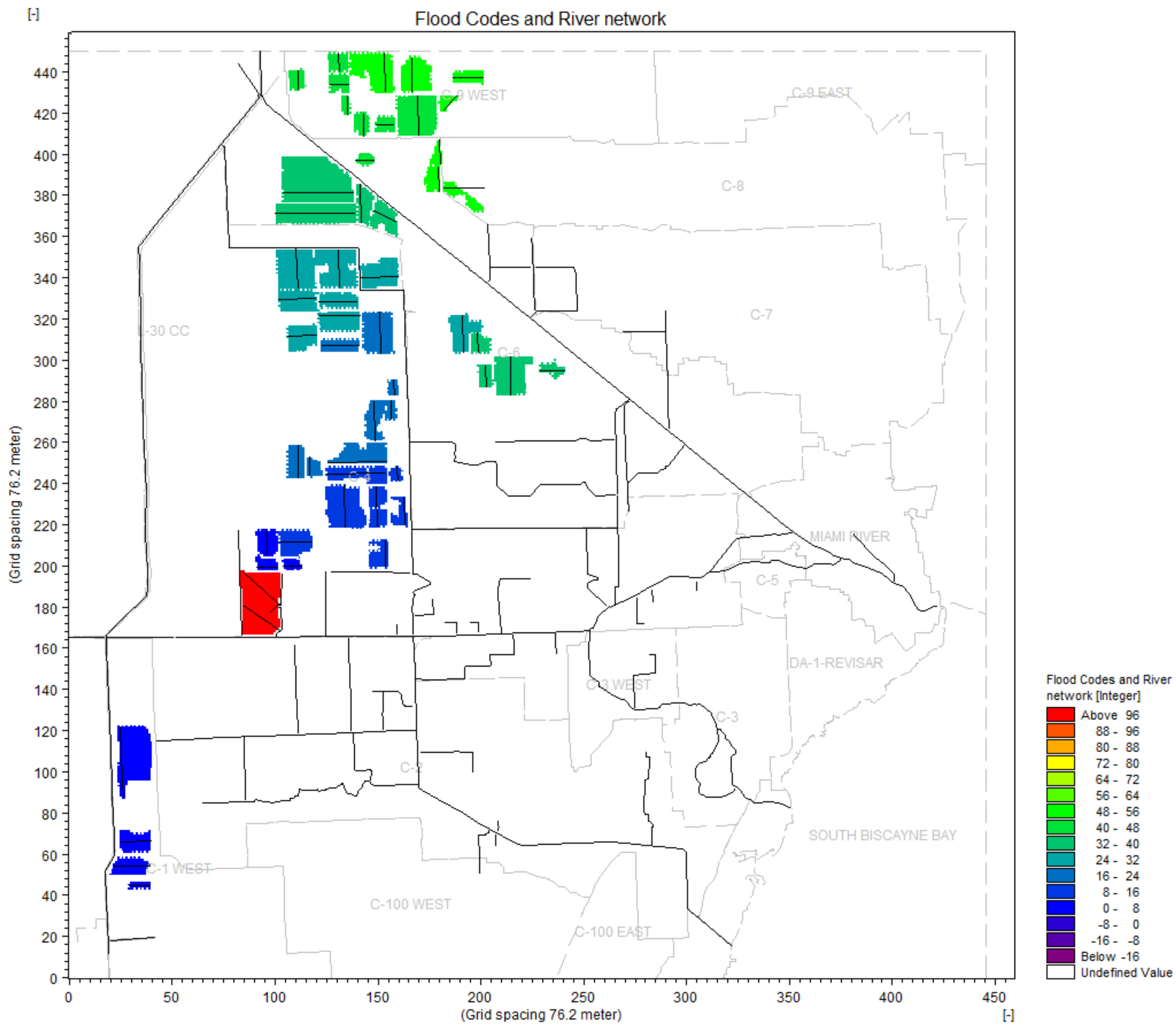
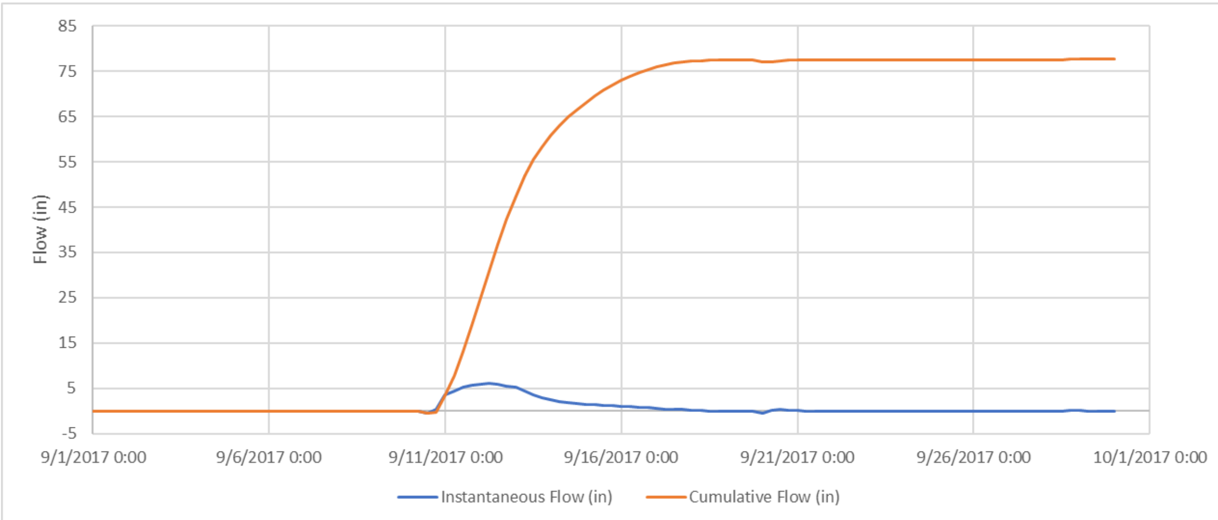


Figure 5-134. Flow within the Detention Basin



5.5.3. MODIFICATIONS TO THE GROUNDWATER MODEL

Upon review of the initial model calibration, there were several issues identified with the results of the groundwater simulation, specifically for the validation run in September 2017. The issues were most evident in the western regions of the model in the Pennsuco Wetlands area as well as in the vicinity of the Miami International Airport (MIA). The monitoring locations where the results were the most concerning were G3, S68, G3327, and G3465. The Calibration Report documented the differences between simulated and measured groundwater and surface water. To improve calibration performance in this addendum effort, two parameters of the groundwater model were modified to evaluate potential improvements: initial conditions and hydraulic conductivity.

5.5.3.1. MODIFYING THE INITIAL CONDITIONS

Upon closer review of the previous validation results, some locations showed an initial drawdown in groundwater elevations and canal stages that are not consistent with measured data. It was determined that initial conditions in the surface water model were more appropriate for dry season conditions (as setup in the calibration run). To address this issue, surface water initial conditions were modified based on the nearest measured canal stages, or groundwater in the case of the mine pits.

In addition to changes in the initial surface water conditions, it was determined that the initial overland depth map used in the validation model was developed for the calibration model and was more appropriate for the dry season simulation. To address this, the map was updated for September 1, 2017, to better estimate initial depths during this wet season simulation. This map was developed by subtracting the topography from the interpolated measured groundwater map for the simulation start date.

5.5.3.2. MODIFYING THE HYDRAULIC CONDUCTIVITY

A variety of modifications to the hydraulic conductivity values were tested during this additional calibration effort. The following process outlines the steps made to improve the groundwater performance throughout the model, specifically during the wet season simulation.

5.5.3.2.1. TESTING UNIFORM VALUES

The initial testing efforts started with global changes to uniform average values which allowed for the identification of areas sensitive to conductivity changes. Improvements were seen to the area north of MIA (G3, G3465, G68, and G3327) with a higher horizontal hydraulic conductivity (Kh) in Layer 1 and a lower Kh in Layer 2.

In addition, during the uniform values tests the ratio between horizontal and vertical conductivity was changed from 1:1 to 10:1. The results showed a reduction in instabilities in some areas as well as improved performance at monitoring stations north of MIA.

5.5.3.2.2. ECSM LAYERING VERSUS USGS LAYERING

As an additional reference for conductivity values, SFWMD staff from the Resource Evaluation Section provided the preliminary groundwater layers and Kh values from the East Coast Surficial Model (ECSM), which is currently under development. The layering was processed and input into the MIKE model for testing and comparison with the existing layering and Kh values.

A comparison of the USGS and ECSM layering was prepared as shown in **Figure 5-135** below. As shown in the comparative figure, the ECSM layering is based on Q Units (see Perkins Q Units in **Figure 5-135**) more closely than USGS model layers, which split Q2. Both models use the bottom of Q1 as the bottom of Layer 3. From there, the ECSM model assumes semi-confined layers 4 and 5 beneath Q1, and the USGS model uses this as the bottom of the model, i.e., no flow between Q1 and the Tamiami Formation.

The USGS model assumes a much lower hydraulic conductivity in the second layer but has higher values in the upper and lower layers 1 and 3. **Table 5-58** provides a summary of the USGS model hydraulic conductivity values side-by-side with the ECSM hydraulic conductivity for each layer. It should be noted that the USGS model hydraulic conductivities are shown with the mining lakes masked out and interpolated, as submitted in the original calibration.

Table 5-58. Comparison of Kh values within the model domain for the USGS model and the ECSM.

Hydraulic Conductivity (ft/day)	USGS Model*			ECSM		
	Layer 1	Layer 2	Layer 3	Layer 1	Layer 2	Layer 3
Max Kh	73,775	256	152,938	10,586	15,000	15,000
Min Kh	317	1	452	39	630	816
Mean Kh	7,512	36	14,053	3,995	9,747	11,217

**USGS Model Kh values are processed with the mine lakes masked out and interpolated*

Figure 5-135. Model layering for the USGS MODFLOW-NWT (Hughes and White, 2016) compared with ECSM layering.

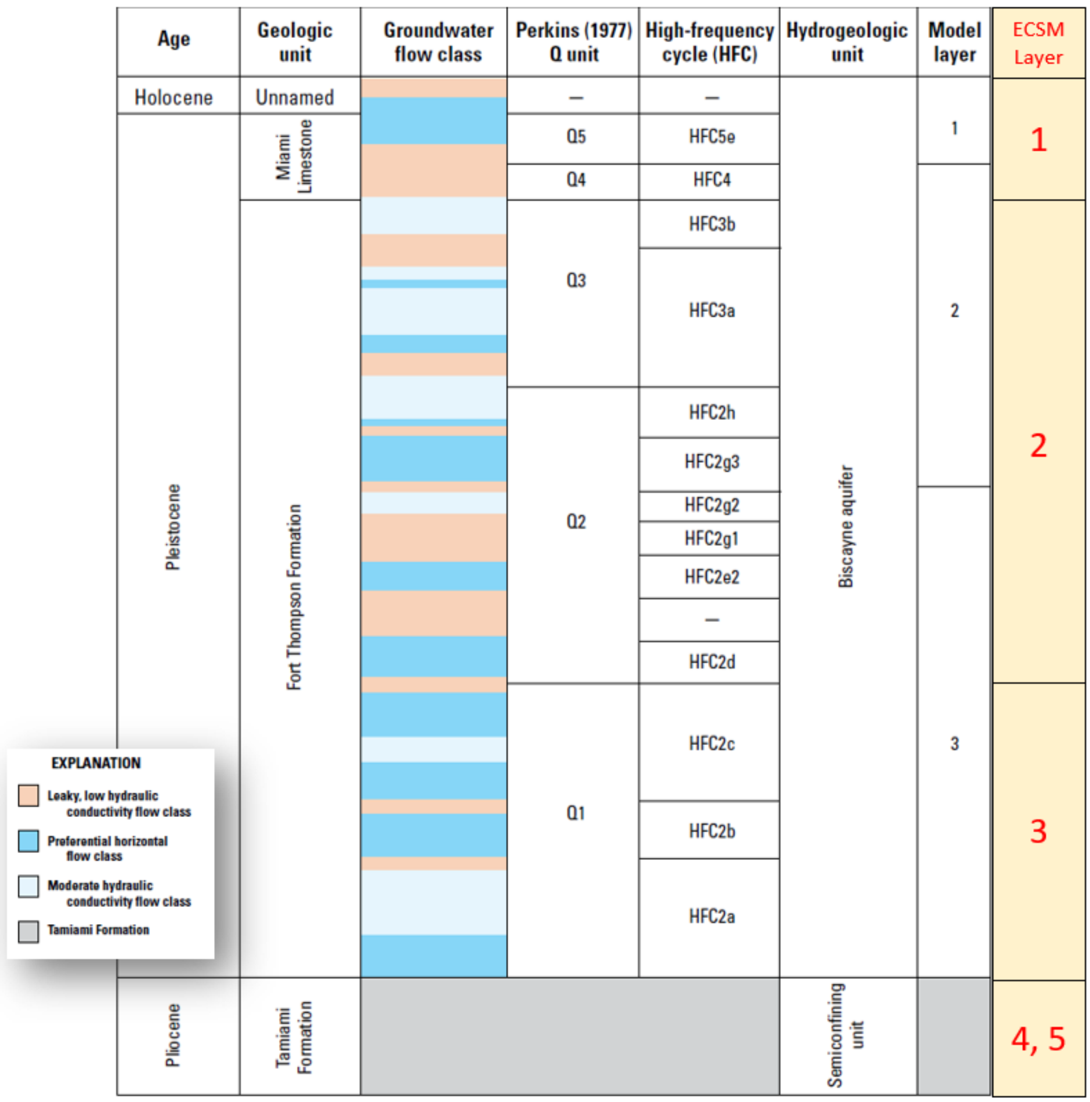
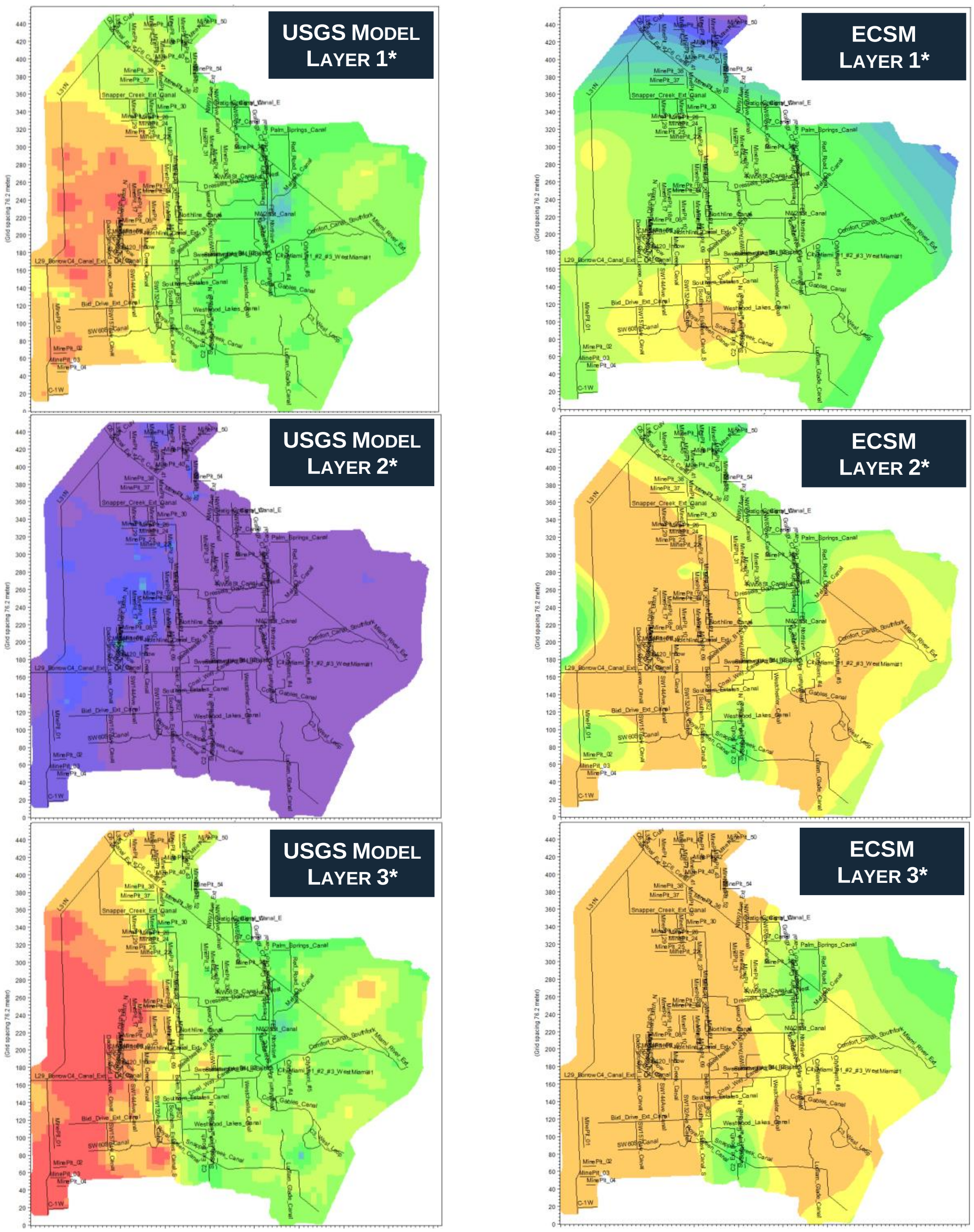
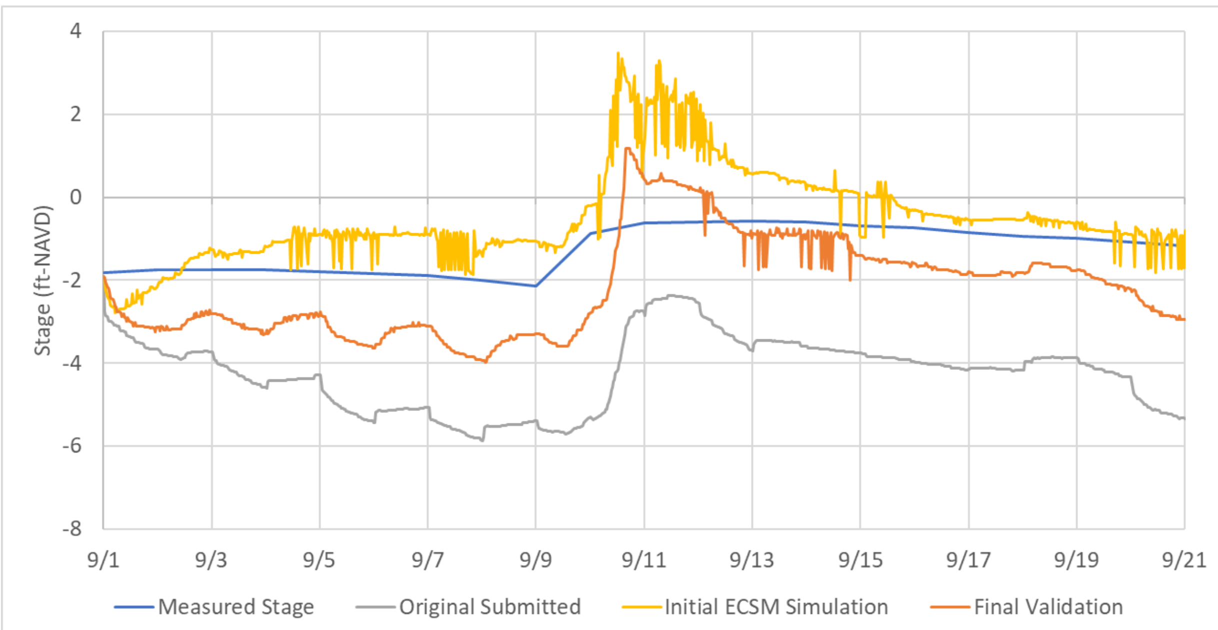


Figure 5-136. Comparison of hydraulic conductivity of USGS model (*values shown are processed with the mine lakes masked out) and the ECSM for the top 3 layers.



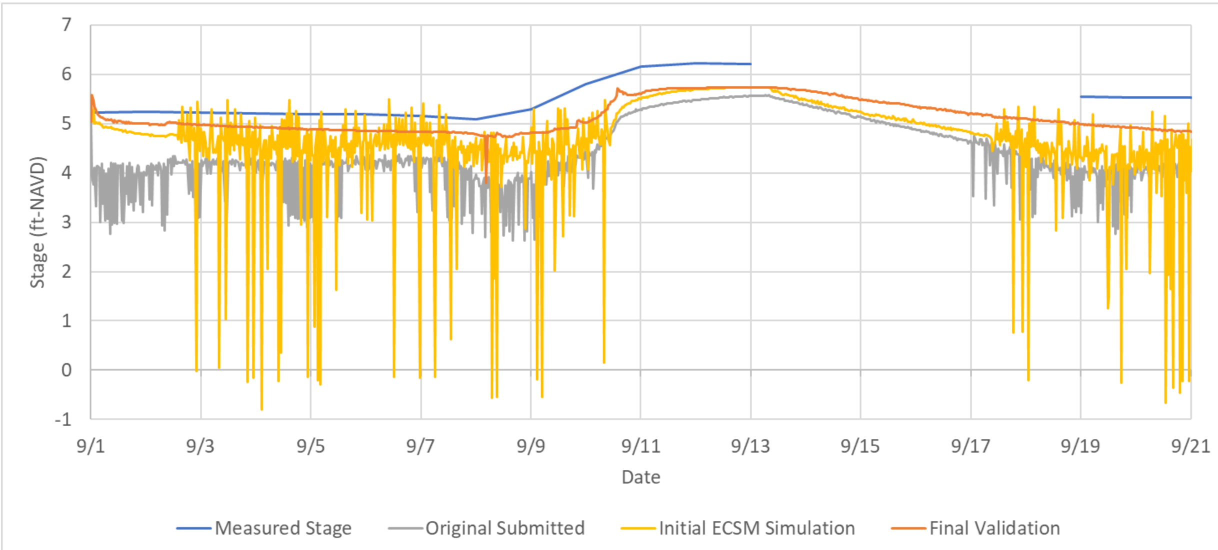
When running the model with the new ECSM layering, there was some improvement at several stations, including in the C2 Watershed; however, station G-3 (north of the MIA and near the C6 Canal) which was previously under estimating groundwater levels was now greatly overestimating, as shown in **Figure 5-137**.

Figure 5-137. Comparison of Original Submitted, Initial ECSM, and Final Validation Simulations at G-3



In addition, many stations now exhibited significant instabilities in the upper groundwater layers. One example is at S356GW1, adjacent to the C4 Canal near S334, as shown in **Figure 5-138**. This figure demonstrates a location where the initial conditions in the surface water caused a sharp drop in water levels at the beginning of the simulation and an adjustment of these conditions will increase levels in the groundwater, specifically before the rainfall event.

Figure 5-138. Comparison of Original, Initial ECSM, and Final Validation Simulations at S356GW1



5.5.3.2.3. LITERATURE REVIEW TO DETERMINE REASONABLE HYDRAULIC CONDUCTIVITY VALUES

Due to the increase in instabilities with the ECSM data, and because the second layer showed such a starkly different overall hydraulic conductivity (see **Table 5-58** in previous section), one concern was that the Kh values from ECSM may be too high for this application of the MIKE model. Therefore, a literature review was performed to check on general Kh values found in the Biscayne aquifer.

One study near the Snapper Creek wellfield performed pneumatic slug tests to determine hydraulic conductivity of three flow zones above the Tamiami Formation (Knowles et. al., 2010). The results showed values ranging from 10,000 feet/day to 40 feet/day, and the geometric means for each layer are shown in **Figure 5-139** below.

Figure 5-139. Geometric mean of each flow zone tested in Knowles et. al., 2010

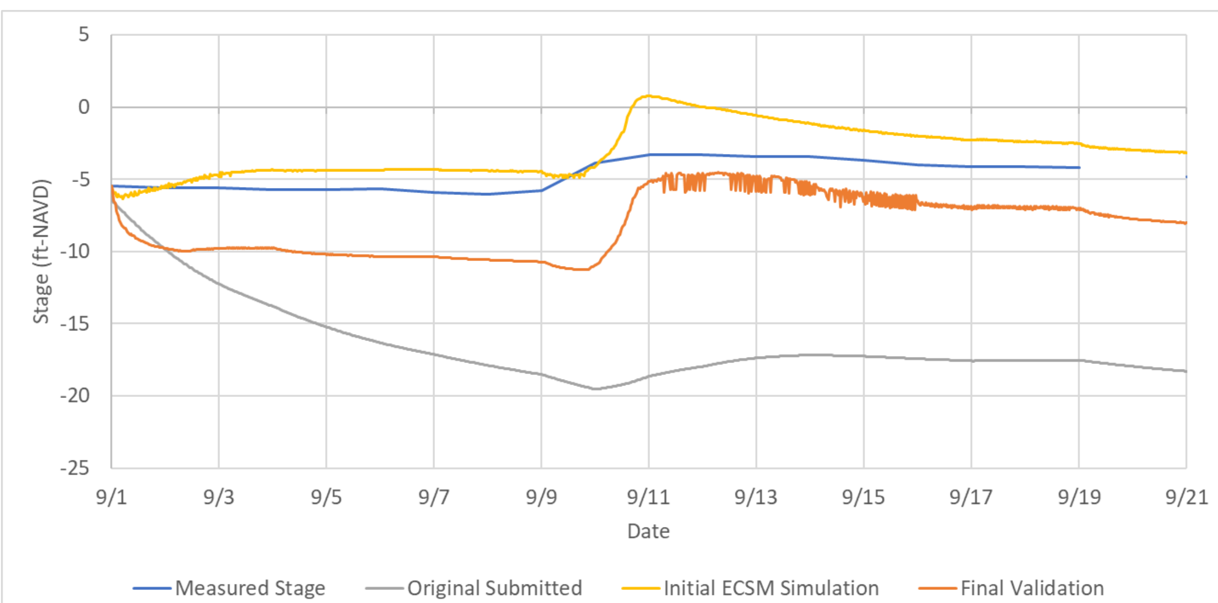
Flow zone	Geometric mean of tests for K, in ft/d
Upper	8,000
Middle	6,000
Lower	4,000
Tamiami Formation	900

The model was tested using the new ECSM layering with maximum and minimum values applied based on the literature values above. Lowering the maximum value in all three of the ECSM layers helped reduce instabilities in many areas. S356GW1 as shown in **Figure 5-138** is a good example of instability reduction from applying a literature review based maximum Kh value. Since there is a lower Kh zone to the north and west of the model, several tests were also done to raise the minimum value in Layer 1 to improve groundwater flow into the C6 Canal headwaters.

5.5.3.2.4. FINAL MODIFICATIONS TO KH VALUES

The model was tested using a combination of ECSM and USGS model Kh values. Since groundwater elevations near MIA overestimate with the higher Kh values from ECSM Layer 2, the USGS Layer 2 values for Kh were used as a test. These lower values reduced groundwater levels at some locations such as G-3 (see **Figure 5-137**). However, some areas were negatively impacted, including the monitoring station near the Alexander Orr well field (G1074B), as shown in **Figure 5-140**. This figure shows how lower Kh values in Layer 2 (as with the original submission in grey) produce very low values due to the nearby pumping wells. Increasing the Kh values here increases stages significantly at this location, as shown with the initial ECSM layering test in yellow and the final submission in orange, which uses a maximum value of 4,000 feet/day.

Figure 5-140. Comparison of Original Submitted, Initial ECSM, and Final Validation Simulations at G-1074B



Stages in the L31N Canal improved with lower Kh values in the upper layer, which led to the reduction of Kh values in the western portion of the model. **Figure 5-141** illustrates stages in the L31N Canal at S335_H with the initial ECSM layering and the final validation results. This station and S337_T showed sensitivity to Kh values in Layer 1 and improved with decreased values. This also improved stability at S356GW1 (see **Figure 5-138**). **Figure 5-142** shows the final Kh in Layer 1, including an assumption that a Kv of 1:10 was used. A maximum value of 8,000 feet/day and a minimum value of 500 feet/day were used for this layer (see **Table 5-59**).

Table 5-59. Comparison of Kh values within the model domain for the final model Kh maps.

Hydraulic Conductivity (ft/day)	USGS Model*			ECSM			Final Modifications		
	Layer 1	Layer 2	Layer 3	Layer 1	Layer 2	Layer 3	Layer 1	Layer 2	Layer 3
Max Kh	73,775	256	152,938	10,586	15,000	15,000	8,000	4,000	6,000
Min Kh	317	1	452	39	630	816	500	10	816
Mean Kh	7,512	36	14,053	3,995	9,747	11,217	3,328	3,781	5,654

Figure 5-141. Comparison of Measured Data with Initial ECSM and Final Validation Results for S335_H

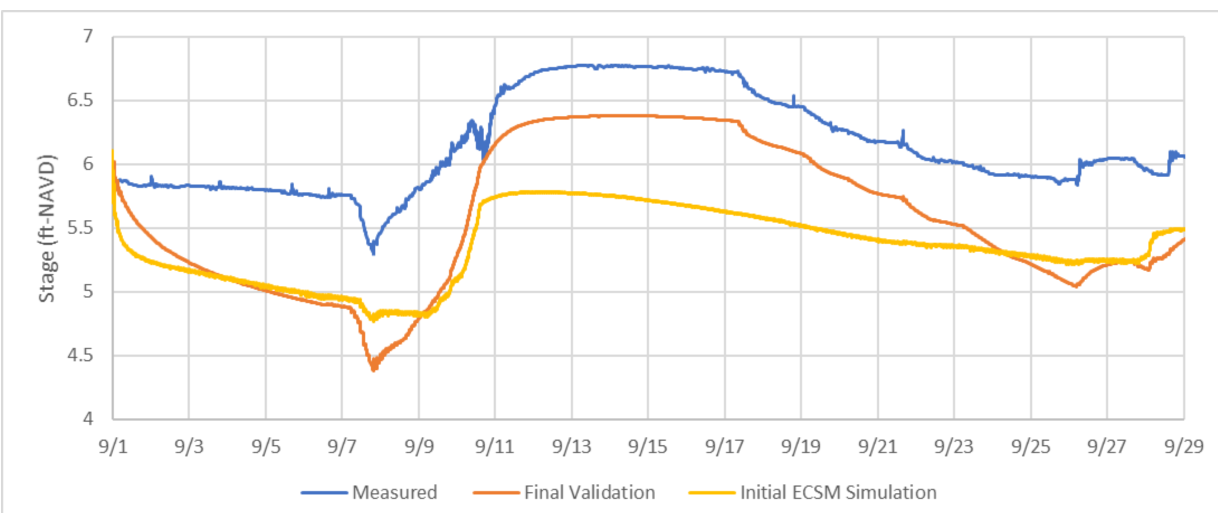
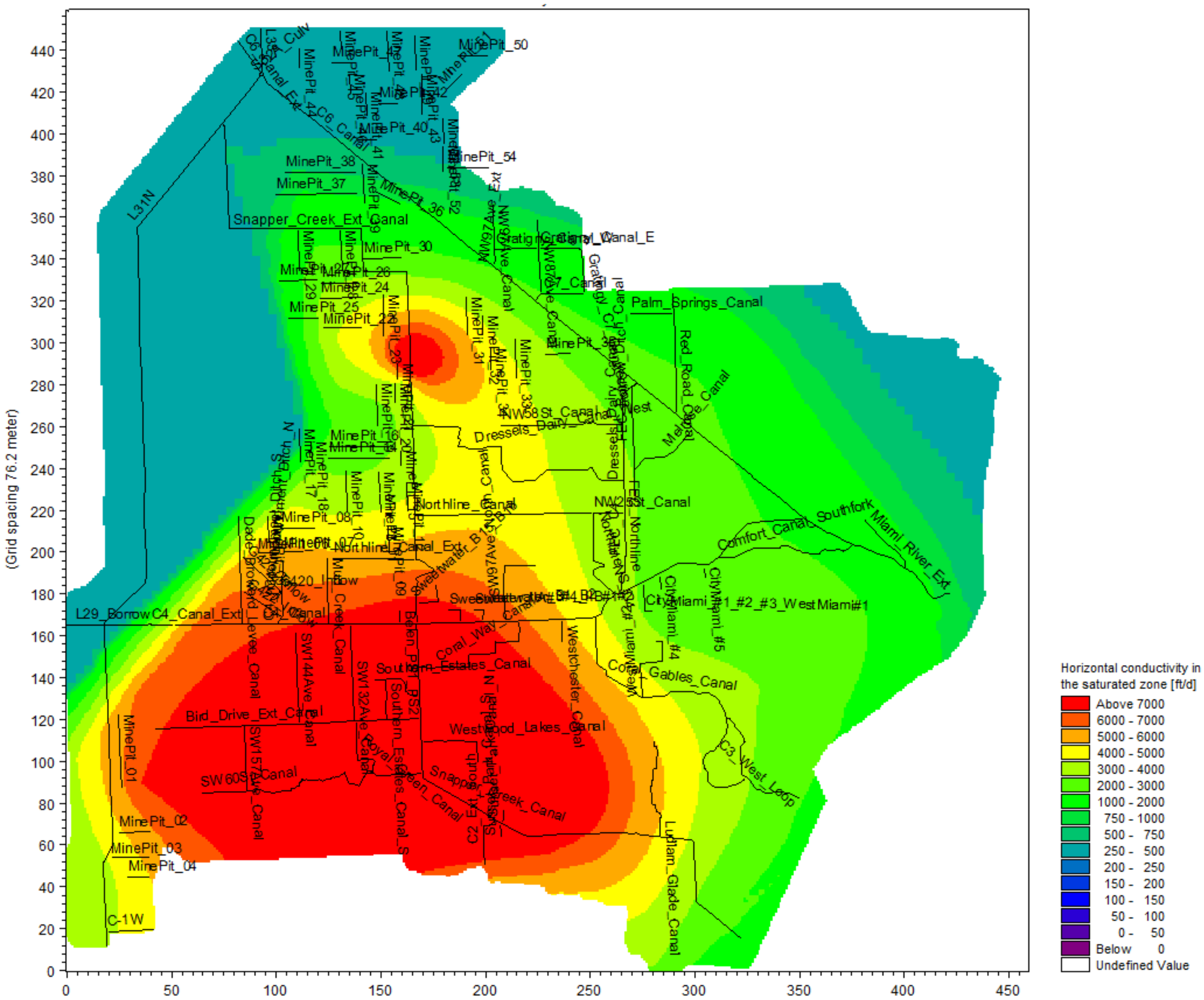


Figure 5-142. Final Kh values for Layer 1



Because the revised model still demonstrated discrepancies in the area around MIA, an effective test for improving performance in a specific region while keeping general performance throughout the model was to modify the ECSM Layer 2 for just the area near MIA. Lowering the Kh values in this specific area proved to be an effective way to lower groundwater levels during the peak at G-3. **Figure 5-143** shows the final Layer 2 Kh values. **Figure 5-144** shows the final Layer 3 Kh values.

Figure 5-143. Final Kh Values for Layer 2

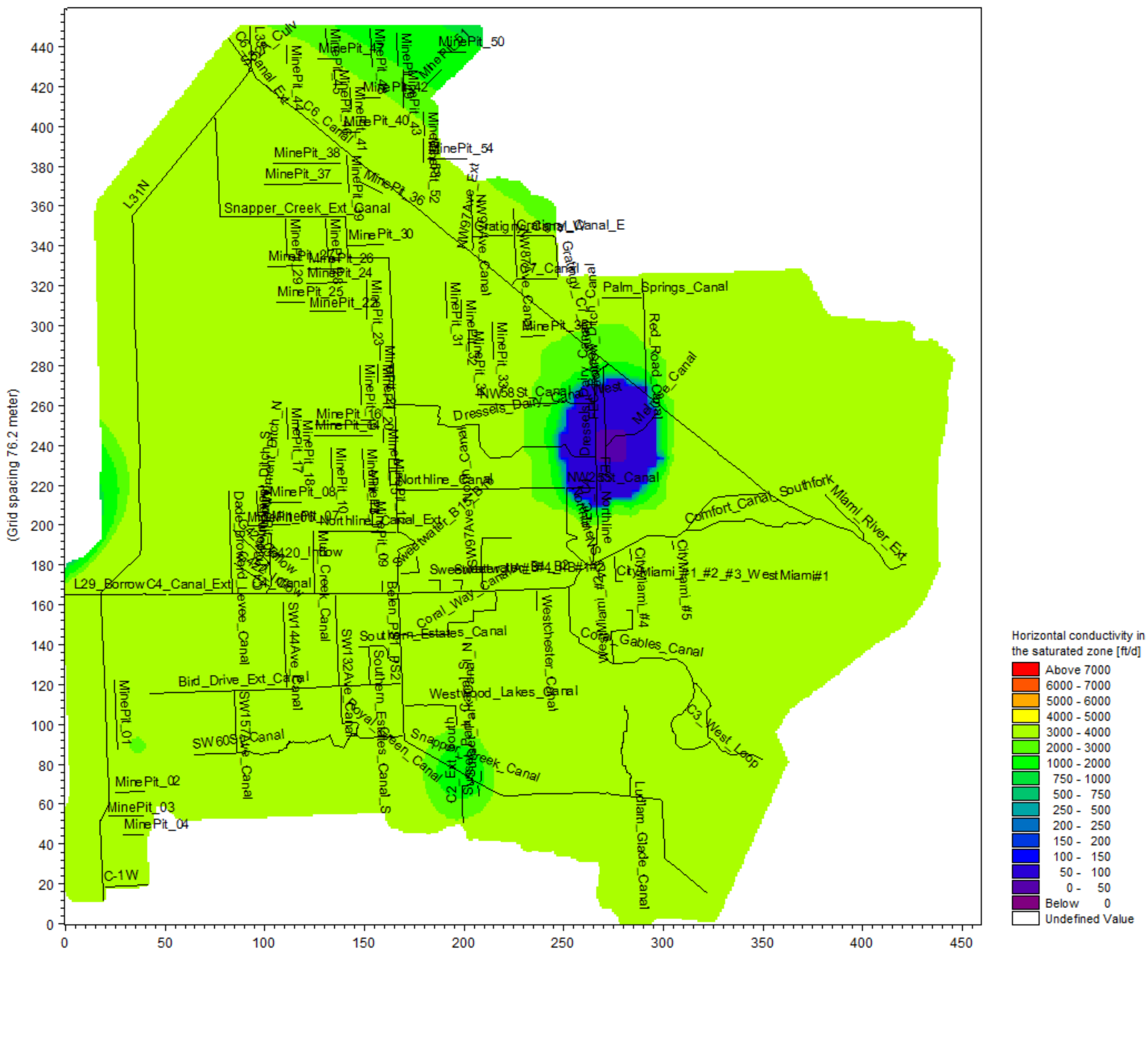
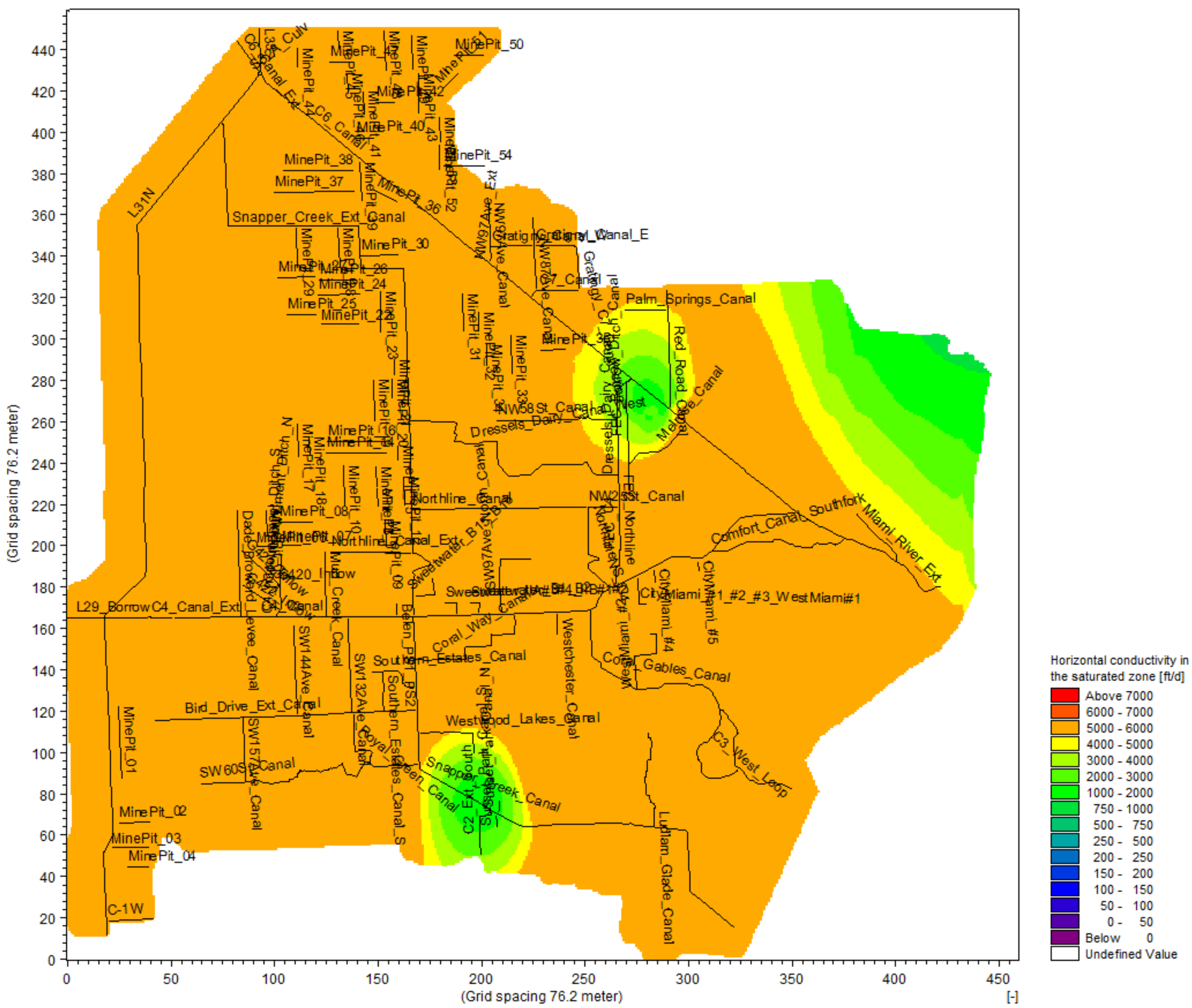


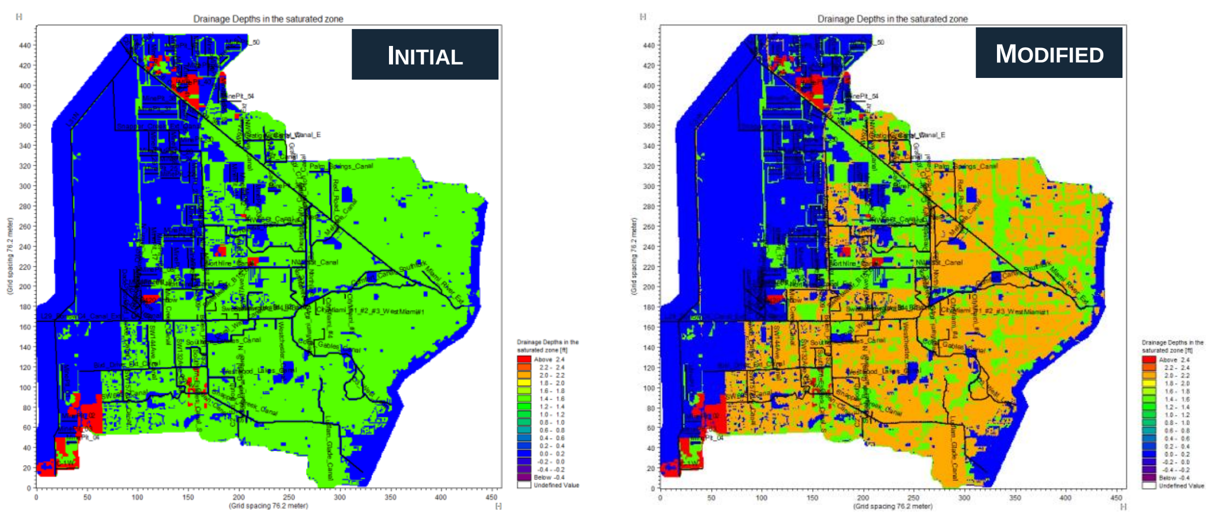
Figure 5-144. Final Kh values for Layer 3



5.5.3.2.5. DRAINAGE LEVELS

All urban areas were initially assigned a drainage depth of 1.5 feet below the ground elevation, based on previous FPLOS studies (i.e., the C8 & C9 study), as described in the calibration report. To help decrease peak stages in groundwater, specifically in areas near the airport and in urban areas such as G3570, the drainage depths were increased to 2.0 feet below the land surface for all medium and high-density urban areas as well as commercial areas (FLUCCS 1200, 1300, and 8100). A depth of 2.0 feet below land surface was chosen as this is less than the agricultural drainage depth of 2.5 feet below land surface, but still deeper than the original 1.5 feet below land surface. Although minimal improvements were shown from this change, the new drainage parameterization was kept as a better representation of drainage in urban areas.

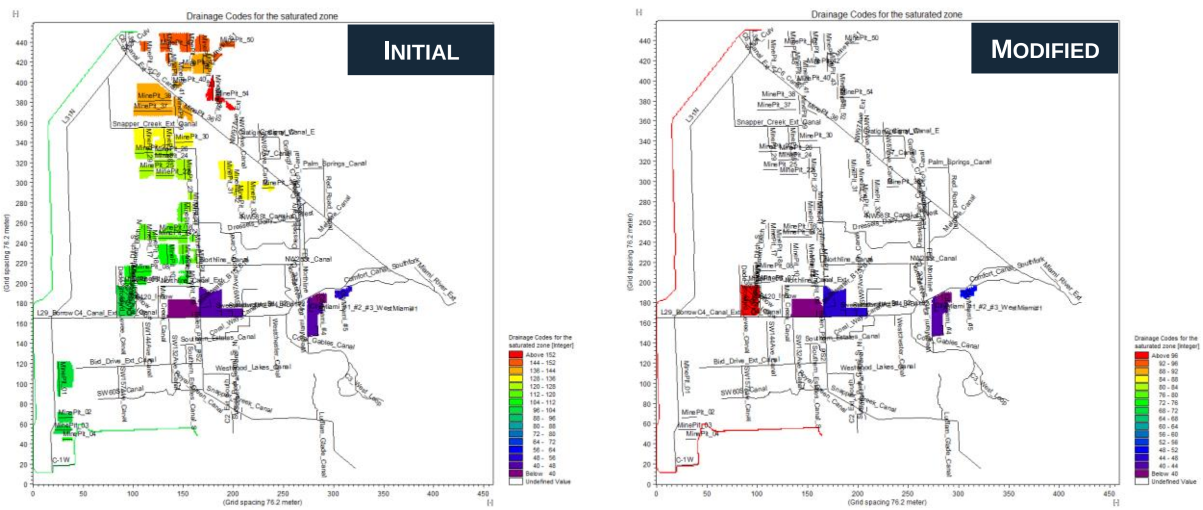
Figure 5-145. Modified drainage depths



5.5.3.2.6. DRAIN CODES

Saturated zone drain codes were also modified to remove the individual mining lakes. After review of previous FPLOS models (i.e., C8 and C9 Watershed Study and South Miami Dade Watershed Study) it was noted that drain codes were not used to represent mining lakes in other recent applications. This approach was determined to be a better representation of the mining lakes, as they are separated from the overland via levees (as defined by the Separated Overland Areas) but should be free to interact with the surrounding groundwater. This change was implemented as a fix for better representation but did not noticeably impact groundwater levels.

Figure 5-146. Submitted versus modified drain codes.



5.5.4. MODIFICATIONS TO THE SURFACE WATER MODEL

In addition to the focus on groundwater hydraulic conductivities and saturated zone drainage, additional modifications to the surface water setup were made to improve model performance and to ensure that flows and stages are not being adversely impacted by incorrect physical parameters. The following changes were made to the surface water model:

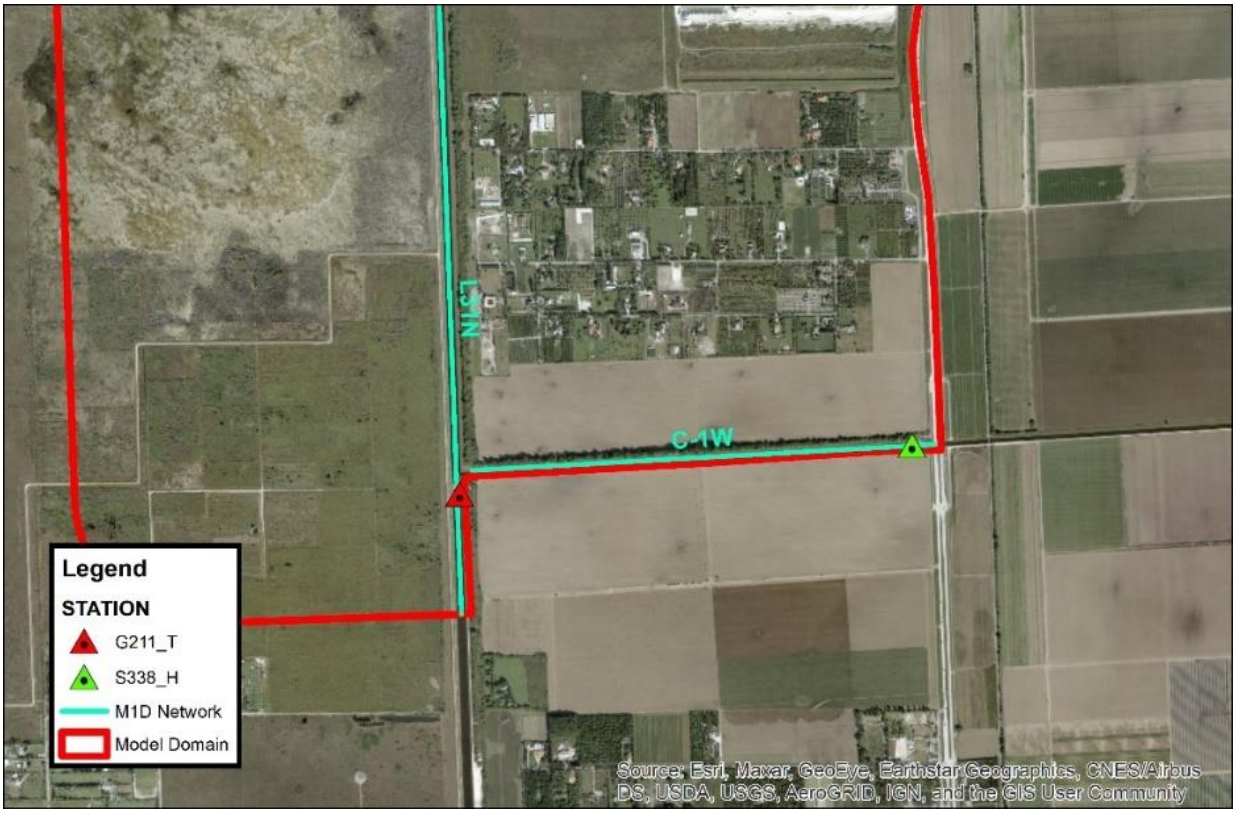
5.5.4.1. INCREASED DENSITY OF H-POINTS ALONG THE CANALS:

The h points along each branch indicate where stages are calculated by the 1D module, and they are also locations where groundwater and surface water are exchanged. The maximum distance of between calculation points was set to 5,000 feet for all canals (the previous max distance was at 10,000 feet); however, some areas were given more calculation points, such as C6 Canal and the C4 Canal upstream of S380 (1,000 feet maximum distance).

5.5.4.2. ADDITIONAL SW BOUNDARIES:

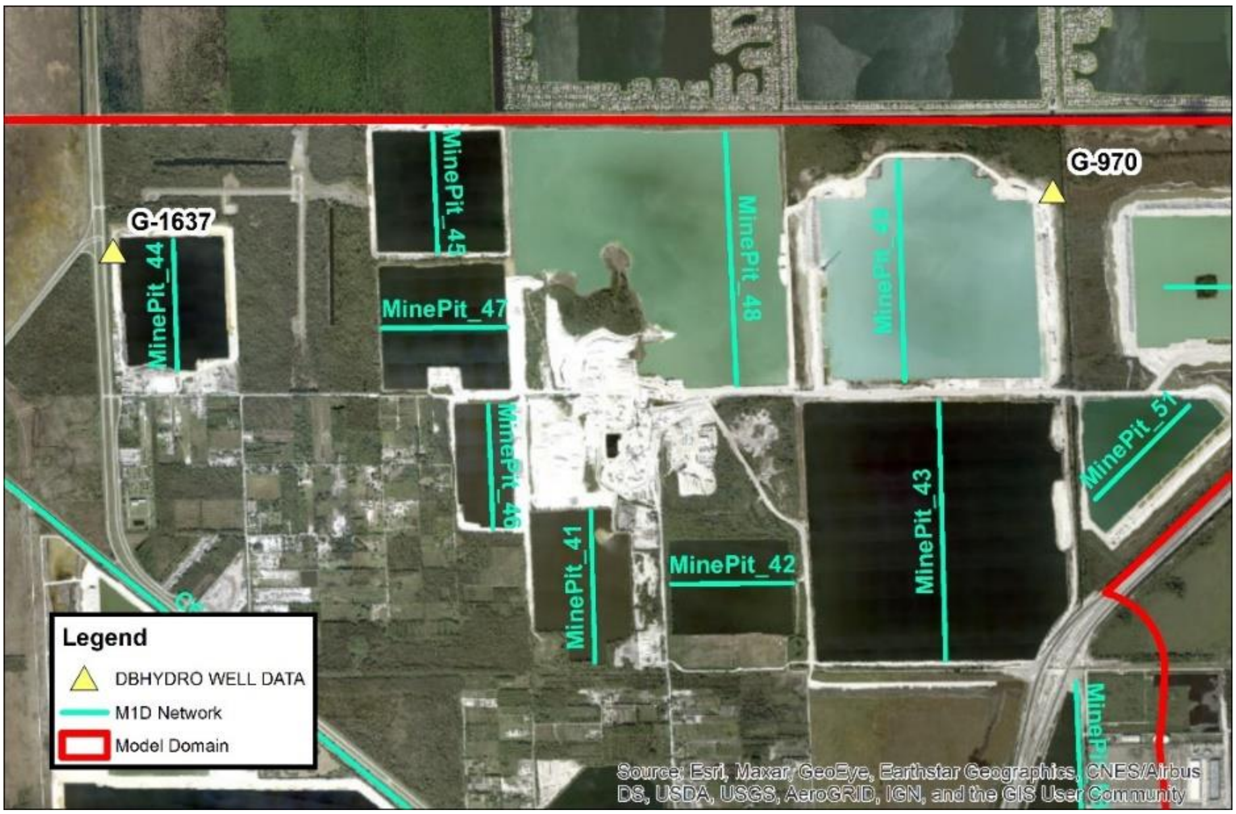
A new surface water boundary was added to represent S338H in the C1-W canal, which helped correct stages in the L31N Canal (see **Figure 5-147** for the physical location).

Figure 5-147. Surface water boundary added to represent S338_H station in the C-1W Canal.



In addition, surface water boundaries were added to two northern mining lakes (named MinePit_44 and MinePit_49 in the model) where groundwater stations are adjacent (G-1637 and G-970 as shown in **Figure 5-148**). These groundwater stations already represent the groundwater boundary, but the addition of the surface water boundary helped establish the mining lake surface elevations at these boundary lakes. This change was not made to any other mining lakes as there is no available monitoring data.

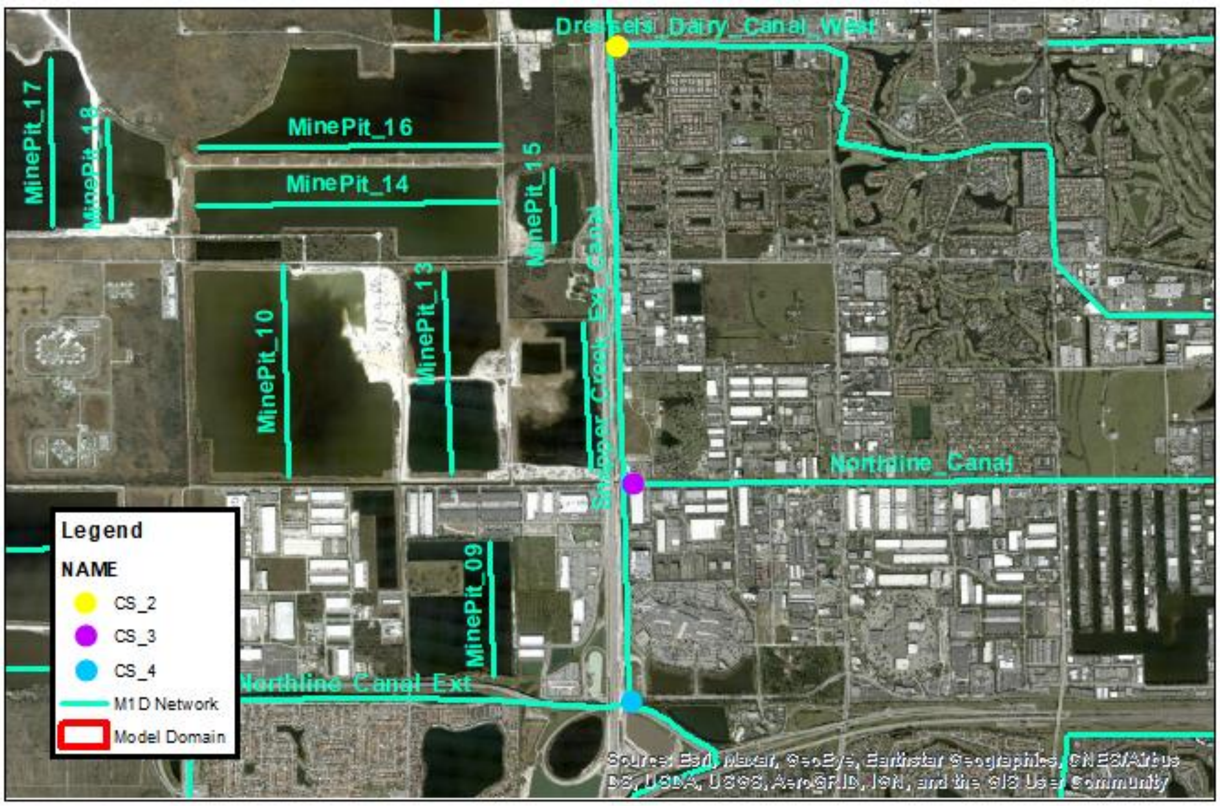
Figure 5-148. Surface water boundary added to the Mine Pit 44 and 49 to represent the GW stages.



5.5.4.3. ADDED MIAMI DADE COUNTY CONTROL STRUCTURES - CS_2 (DRESSELS DAIRY CANAL) AND CS_3 (NORTHLINE CANAL)

The CS_2 and CS_3 gated culverts were added as shown in **Figure 5-149**. A previous C4 Model report (PBS&J, 2004) stated that these structures remain closed "usually", which was justification for these structures to be used as model boundaries, however for this model, these structures were input as closed gated structures during both calibration and validation events.

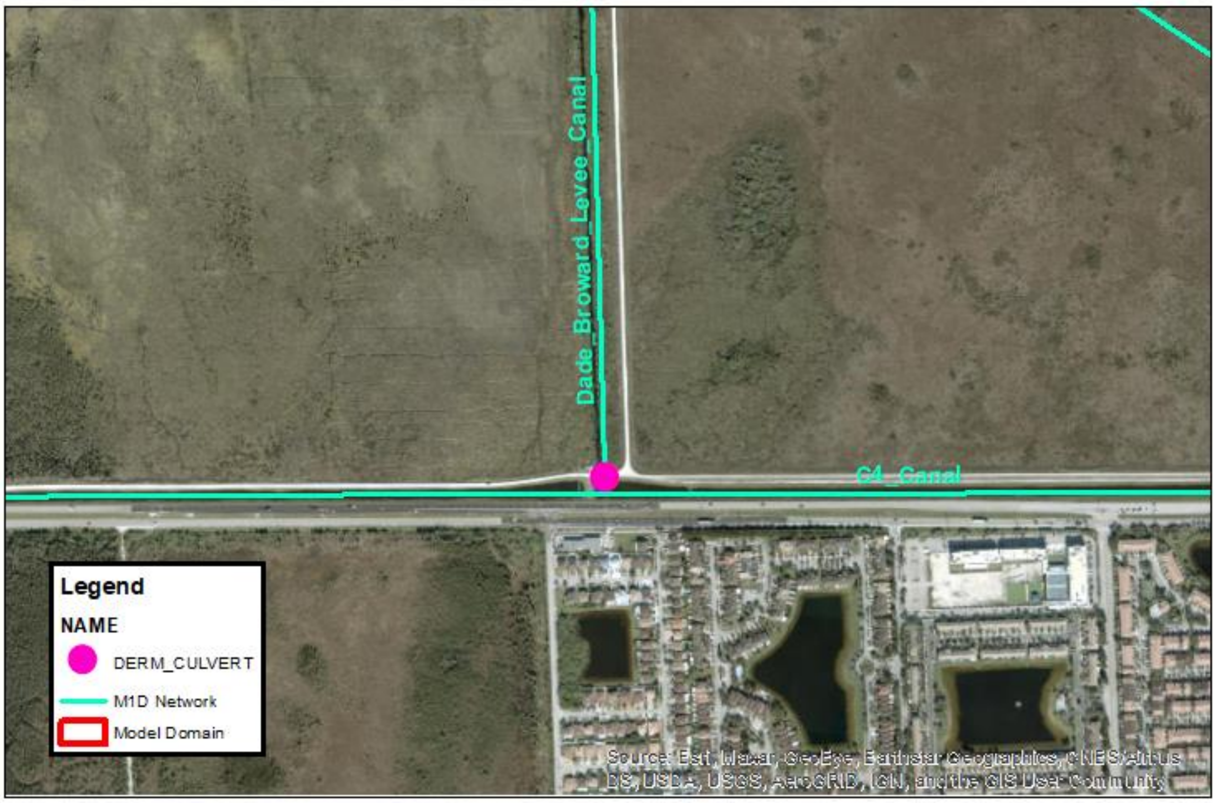
Figure 5-149. Location of closed gated structures CS_2 and CS_3.



5.5.4.4. ADDED GATED CULVERT IN DADE-BROWARD LEVEE CANAL AT C4 CANAL:

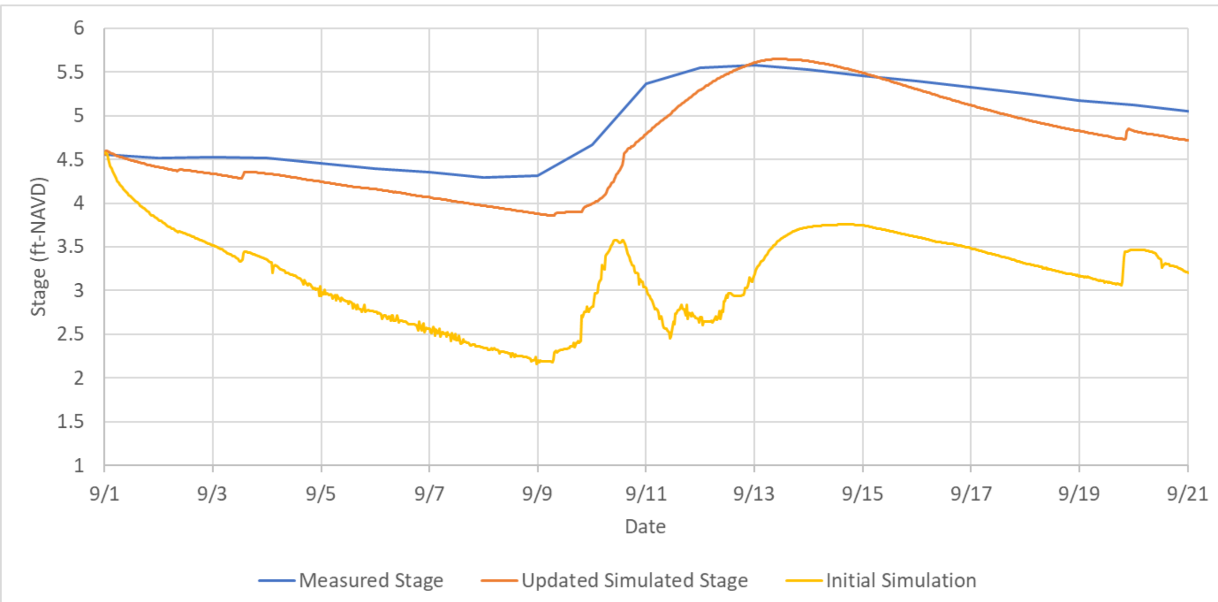
The gated culvert located in the Dade-Broward Levee Canal (shown in **Figure 5-150**) was added to help improve groundwater levels in this area, specifically at C4GW1. The width of this gate was estimated from aerial and ground-based photos. The gate sill was estimated from measured wet season lows at C4GW1, which is adjacent to the canal.

Figure 5-150. Location of DERM Culvert connecting the Dade-Broward Levee Canal with the C4 Canal.



The addition of this gated structure improved groundwater calibration at C4GW1 immediately, and no additional groundwater adjustments were required for this area. **Figure 5-151** illustrates the updated simulation results in orange as compared to earlier simulation results without the structure in yellow. Daily measured groundwater stages are shown in blue.

Figure 5-151. Comparison of Stages at C4GW1



5.5.4.5. ADDED TWO CULVERTS IN THE NORTHWEST WELLFIELD PROTECTION CANAL (SNAPPER_CREEK_EXT_CANAL)

Culverts located within the NWWF Protection Canal (Snapper Creek Ext Canal in the model) were implemented in the model at the location shown in **Figure 5-152**. These are adjacent to the USGS stage and flow monitoring station USGS 02287497. They were added to help improve stages at this location. However, without details regarding these structures, parameters such as dimensions and sill elevations were estimated based on aerials and water levels. These were implemented as 2 feet x 6 feet diameter culverts with a bottom sill of 1.0 feet-NAVD.

Figure 5-152. Location of DERM culverts within the Snapper Creek Ext Canal.

5.5.4.6. MISCELLANEOUS HYDRAULIC GEOMETRY MODIFICATIONS:

- Corrected sill elevation at S335 sluice gate from (-)4.2 to (-)5.762 feet-NAVD.
- Corrected sill elevation at S334 sluice gate from (-)8.68 to (-)8.458 feet-NAVD.
- Corrected maximum discharge at S356 pump station from 0.0 CFS to no max discharge (uses measured pump flows from DBHydro).
- Added “leak” culverts at G119.

The G119 structure is considered permanently closed, but there may be some leaks around or through the structure that could contribute to S380. To improve tailwater at S336 and headwater at S380, three small (1 foot diameter) culverts were modeled at this location to represent leaks through the structure. However, without additional information relating to G119, significant changes to the stages were not achieved.

5.5.5. STATISTICAL COMPARISON WITH ORIGINAL SUBMISSION

During the modification of the validation and calibration parameters, various statistics were evaluated to see how model performance was improving against the previously submitted model. A key metric was the groundwater Mean Absolute Error (MAE)., The results of the final revised validation simulation are shown in **Table 5-60**. The highlighted cells indicate MAE values that are below the 0.5 feet statistical criteria. The new validation run increased the number of stations that now meet these criteria by 33% above the original submission. In addition, the MAE at nearly every groundwater station was improved, with the exception of G-3909 and S-68, which are near well fields.

Table 5-60. Previous Validation Model MAE versus Updated Validation Model MAE for Groundwater

STATION	SUBMITTED VALIDATION MAE	UPDATED VALIDATION MAE	DIFFERENCE
F319	0.849	0.368	0.481
G3	1.607	0.919	0.687
G975	1.385	0.168	1.217
G3439	0.548	0.452	0.097
G3565	1.261	0.898	0.363
G3572	0.443	0.389	0.054
G3465	2.455	1.383	1.072
C4GW1	1.967	0.303	1.664
C2GSW1	2.065	1.486	0.579
G3909	2.114	2.721	-0.607
S356GW1	1.805	0.528	1.277
G3558	1.031	0.450	0.582
G1074B	12.204	2.782	9.422
S68	0.818	1.081	-0.263
G3327	2.350	1.443	0.906
G3918	2.299	0.524	1.775
G3818	0.478	0.161	0.316
G3264AR	0.539	0.494	0.045
G3570	2.017	0.637	1.381
G3563	1.445	1.133	0.311
G3567	1.997	0.145	1.852

Table 5-61 shows the comparison of the previously submitted groundwater statistical results with the updated groundwater results using MAE as the metric. The MAE increased at about 76% of the stations with the new results, and the new results increased the stations meeting the MAE criteria of 0.5 feet by 20%.

Table 5-61. Previous Calibration Model MAE Versus Update Calibration Model MAE for Groundwater

Station	Submitted Calibration MAE	Updated Calibration MAE	Difference
F319	0.849	0.751	0.099
G3	1.607	2.527	-0.920
G975	1.385	0.892	0.493
G3439	0.548	0.377	0.172
G3565	1.261	0.979	0.282
G3572	0.443	0.385	0.059
G3465	2.455	1.574	0.881
C4GW1	1.967	0.265	1.702
C2GSW1	2.065	1.319	0.746
G3909	2.114	0.400	1.714
S356GW1	1.805	0.106	1.699
G3558	1.031	0.835	0.197
G1074B	12.204	2.762	9.442
S68	0.818	1.826	-1.008
G3327	2.350	2.043	0.307
G3918	2.299	0.624	1.675
G3818	0.478	0.872	-0.394
G3264AR	0.539	1.302	-0.762
G3570	2.017	1.216	0.802
G3563	1.445	1.621	-0.176
G3567	1.997	0.325	1.672

In addition, the surface water stages were checked to ensure no adverse effects to surface water were occurring during the groundwater testing phase. **Table 5-62** shows the comparison of the MAE for the previously submitted model and the updated validation for both the full simulation (9/2/2017 – 9/30/2017) and the 7-day rainfall event (9/8/2017 – 9/14/2017). MAE was improved in some areas, specifically to the west, where boundary conditions were adjusted, and some sill corrections were made.

Table 5-62. Surface Water Comparison for Validation

Station	Submitted Validation MAE	Updated Validation MAE	Diff.	Submitted Validation 7-Day MAE	Updated Validation 7-Day MAE	Diff.
G93_H	0.620	0.602	0.018	0.325	0.399	-0.075
G93_T	0.226	0.245	-0.019	0.217	0.278	-0.061
S22_H	0.176	0.187	-0.011	0.201	0.250	-0.048
S22_T	0.129	0.138	-0.009	0.179	0.216	-0.037
S25_H	0.252	0.248	0.004	0.161	0.153	0.008
S25_T	0.123	0.113	0.010	0.146	0.134	0.013
S25B_H	0.385	0.347	0.038	0.229	0.262	-0.033
S25B_T	0.262	0.247	0.016	0.241	0.257	-0.015
S26_H	0.234	0.230	0.004	0.234	0.245	-0.011
S26_T	0.217	0.198	0.020	0.276	0.285	-0.008
S31_T	0.384	0.332	0.053	0.684	0.473	0.211
S334_T	0.688	0.248	0.440	0.773	0.222	0.551
S335_H	1.303	0.585	0.718	1.365	0.531	0.834
S335_T	0.717	0.261	0.456	0.820	0.211	0.609
S336_H	0.688	0.236	0.452	0.773	0.204	0.569
S336_T	0.837	0.337	0.500	0.888	0.286	0.602
S337_T	1.338	0.583	0.756	1.502	0.662	0.840
S380_H	1.248	1.532	-0.284	0.942	1.394	-0.452
S380_T	1.160	1.555	-0.395	0.969	1.833	-0.865
T5W	0.381	0.338	0.043	0.447	0.329	0.118
C4.CORAL_S	0.431	0.376	0.054	0.507	0.524	-0.018
C2SW1	0.285	0.240	0.045	0.354	0.246	0.108
C2SW2	0.190	0.187	0.002	0.232	0.172	0.061
MRMS1	0.201	0.185	0.015	0.253	0.256	-0.003
C2.74	1.625	0.900	0.724	1.645	0.755	0.890
USGS02289500_S	0.428	0.358	0.070	0.249	0.186	0.063
USGS2287497_S	2.770	0.295	2.475	2.560	0.269	2.291

Table 5-63 shows the comparison of the MAE for the previously submitted model and the updated calibration for both the full simulation (5/12/2020 – 5/30/2020) and the 7-day rainfall event (5/24/2020 – 5/30/2020). While calibration at S380 declined with the new simulation, it should be noted that calibration at both T5W and USGS02289500 (which are both downstream of the S380) stayed well within the 0.5 feet criteria for the new simulation.

Stages at C2.74, a station within the Snapper Creek Extension at NW 74th Street, improved for the validation run, but declined with the calibration run. This is likely due to the wet season assumptions that were made regarding the gated DERM culverts in the Northline Canal and Dressel's Dairy Canal as being always closed (see previous section for details).

Table 5-63. Surface Water Comparison for Calibration

Station	Submitted Calibration MAE	Updated Calibration MAE	Diff.	Submitted Calibration 7-Day MAE	Updated Calibration 7-Day MAE	Diff.
G93_H	0.209	0.343	-0.134	0.239	0.367	-0.128
G93_T	0.110	0.149	-0.038	0.141	0.264	-0.122
S22_H	0.185	0.436	-0.252	0.239	0.330	-0.091
S22_T	0.159	0.161	-0.002	0.261	0.320	-0.059
S25_H	0.347	0.454	-0.107	0.402	0.637	-0.235
S25_T	0.141	0.139	0.002	0.166	0.195	-0.029
S25B_H	0.183	0.277	-0.094	0.184	0.191	-0.007
S25B_T	0.159	0.162	-0.003	0.193	0.236	-0.043
S26_H	0.161	0.223	-0.062	0.151	0.198	-0.048
S26_T	0.192	0.202	-0.010	0.232	0.276	-0.044
S31_T	0.126	0.226	-0.100	0.217	0.209	0.008
S334_T	0.245	0.078	0.168	0.510	0.062	0.448
S335_H	0.676	0.518	0.157	0.286	0.252	0.034
S335_T	0.243	0.097	0.146	0.489	0.083	0.406
S336_H	0.263	0.053	0.209	0.515	0.045	0.470
S336_T	0.295	0.250	0.045	0.476	0.394	0.081
S337_T	0.592	0.441	0.152	0.244	0.294	-0.050
S380_H	0.249	0.643	-0.393	0.315	1.176	-0.861
S380_T	0.271	0.887	-0.616	0.272	1.289	-1.017
T5W	0.192	0.385	-0.193	0.216	0.249	-0.032
C4.CORAL_S	0.167	0.461	-0.294	0.237	0.425	-0.188
C2SW1	0.194	0.303	-0.109	0.137	0.153	-0.016
C2SW2	0.122	0.338	-0.216	0.087	0.137	-0.050
MRMS1	0.161	0.158	0.003	0.201	0.236	-0.035
C2.74	0.252	1.093	-0.842	0.298	1.051	-0.753
USGS02289500_S	0.172	0.243	-0.072	0.207	0.185	0.022
USGS2287497_S	0.781	0.165	0.616	0.545	0.305	0.240

5.5.6. ADDENDUM CONCLUSIONS

Investigation into the key concerns from the District led to the following conclusions:

1. Poor matching between measured and simulated groundwater results was addressed by a battery of sensitivity tests and revisions to the groundwater and surface water model, with a heavy focus on the wet season simulation (validation). The final simulation, which used the ECSM vertical layering and a combination of ECSM and USGS modified hydraulic conductivity values, increased model performance with the MAE meeting the 0.5 feet criteria at 33% more stations.
2. Poor matching between measured and simulated flows at S26 was addressed by reviewing Case 1 and Case 5 calculations for flows at this location. After further review of the new data, it was found that just a minor difference in headwater stages can result in high differences in flow at this location. During review with the District, it was concluded that it is best to calibrate to stages rather than flows for the purposes of this analysis.
3. Unusually high negative numbers in the water budget for the C4 Watershed was addressed by reviewing the C4 watershed and blocking out the C4 Emergency Detention Basin, and performing separate water budget calculations for this basin to determine if the exchanges are occurring in that area, as predicted. The results conclude that all the highly negative numbers in the OL to River category are due to exchange of the OL component with the 1D model within the Detention Basin and mining lakes. This model process is working as intended in the model and is accounted for appropriately.

The full results of the model revisions are presented in the sections below.

5.5.7. FINAL CALIBRATION RESULTS

5.5.7.1. UPDATED STATISTICS FOR THE CALIBRATION PERIOD

The following tables provide updated statistics for the calibration period of 5/12/2020 through 5/30/2020 (the first day of the simulation is ignored as it is considered a warmup day) for surface water (**Table 5-64**) and groundwater (**Table 5-66**). The 7-day statistics are also provided to qualify model performance during the rain event (**Table 5-65** and **Table 5-67**).

Table 5-64. Updated Surface Water Statistics for Calibration

Watershed	Station	19 - Day Simulation (5/12/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	C2SW1	0.236	0.303	0.399	0.975	0.603
	C2SW2	0.254	0.338	0.465	0.745	-0.988
	S22H	0.176	0.436	0.518	0.427	-0.297
C3W	G93H	0.036	0.343	0.425	0.723	0.035
C4	USGS 02289500	0.145	0.243	0.302	0.961	0.760
	S25BH	0.163	0.277	0.358	0.765	0.451
	S336T	-0.040	0.250	0.311	0.974	0.840
	S380H	0.642	0.643	1.028	-0.017	-3.398
	S380T	0.887	0.887	1.213	0.534	-1.799
	T5W	0.324	0.385	0.452	0.980	0.703
	USGS 02287497	-0.126	0.165	0.206	0.993	0.910
C5	S25H	-0.284	0.454	0.531	0.773	0.229
C6	S26H	-0.051	0.223	0.262	0.872	0.751
	S31T	0.209	0.226	0.257	0.982	0.844

Table 5-65. Updated 7-Day Surface Water Statistics for Calibration

Watershed	Station	7 - Day Simulation (5/24/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	C2SW1	-0.026	0.153	0.169	0.981	0.910
	C2SW2	-0.046	0.137	0.173	0.928	0.825
	S22H	-0.265	0.330	0.398	0.790	0.321
C3W	G93H	-0.355	0.367	0.473	0.811	-0.329
C4	USGS 02289500	-0.082	0.185	0.234	0.937	0.855
	S25BH	-0.001	0.191	0.260	0.869	0.756
	S336T	-0.315	0.394	0.444	0.935	-0.947
	S380H	1.173	1.176	1.590	-0.270	-41.179
	S380T	1.288	1.289	1.628	0.654	-21.704
	T5W	0.112	0.249	0.272	0.941	0.784
	USGS 02287497	-0.305	0.305	0.315	0.994	0.040
C5	S25H	-0.522	0.637	0.701	0.747	-0.143
C6	S26H	-0.172	0.198	0.235	0.962	0.819
	S31T	0.162	0.209	0.264	0.938	0.655

Table 5-66. Updated Groundwater Statistics for Calibration

Watershed	Station	20 - Day Simulation (5/11/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	G-3558	0.415	0.835	1.154	0.872	0.671
	G-3563	0.522	1.621	2.019	0.934	-0.190
	G-3572	0.031	0.385	0.515	0.980	0.913
	G-3565	-0.255	0.979	1.159	0.909	0.438
	G-3903	0.169	0.528	0.782	0.968	0.663
	G-1074B	-2.761	2.762	3.009	0.975	-0.009
C3W	G-3570	1.090	1.216	1.641	0.981	0.651
C3	F-319	-0.500	0.751	1.555	0.889	0.733
C4	C4GW1	0.156	0.265	0.401	0.993	0.967
	G-3465	1.504	1.574	1.870	0.961	-0.041
	G-975	0.875	0.892	1.009	0.993	0.868
	S356GW1	0.099	0.106	0.156	0.999	0.996
	G-3818	0.872	0.872	0.957	0.995	0.850
	G-3676	-0.207	0.250	0.291	0.993	0.976
	G-3327	1.996	2.043	2.477	0.936	-1.795
C6	G-3	2.389	2.527	3.175	0.022	-8.692
	G-3567	0.086	0.325	0.357	0.990	0.979
	S-68	1.826	1.826	1.965	0.818	0.037
	G-3264AR	1.302	1.302	1.600	0.961	-0.058

Table 5-67. Updated 7-Day Groundwater Statistics for Calibration

Watershed	Station	7 - Day Simulation (5/24/2020 - 5/30/2020)				
		ME	MAE	RMSE	R	NS
C2	G-3558	0.158	0.236	0.314	0.640	0.190
	G-3563	3.060	3.060	3.129	0.823	-12.327
	G-3572	0.512	0.571	0.698	0.984	0.540
	G-3565	0.852	1.177	1.412	0.953	-1.501
	G-3903	0.721	0.792	1.147	0.966	-1.399
	G-1074B	-1.802	1.802	1.903	0.886	-8.150
C3W	G-3570	2.268	2.268	2.512	0.710	-5.995
C3	F-319	-1.504	2.437	3.476	0.921	0.785
C4	C4GW1	0.519	0.534	0.643	0.990	-0.762
	G-3465	2.666	2.666	2.702	0.936	-4.007
	G-975	1.107	1.107	1.120	0.982	-12.453
	S356GW1	0.161	0.170	0.211	0.920	0.592
	G-3818	1.005	1.005	1.006	0.999	-13.351
	G-3676	-0.172	0.209	0.237	0.929	0.318
	G-3327	3.445	3.445	3.527	0.699	-10.773
C6	G-3	4.539	4.539	4.705	0.129	-822.102
	G-3567	-0.334	0.348	0.373	0.997	0.208
	S-68	1.885	1.885	1.949	0.940	-1.124
	G-3264AR	2.395	2.395	2.431	0.000	NA

5.5.7.2. C2 WATERSHED – CANAL STAGE

Figure 5-153. Calibration Model Canal Stage – C2SW1

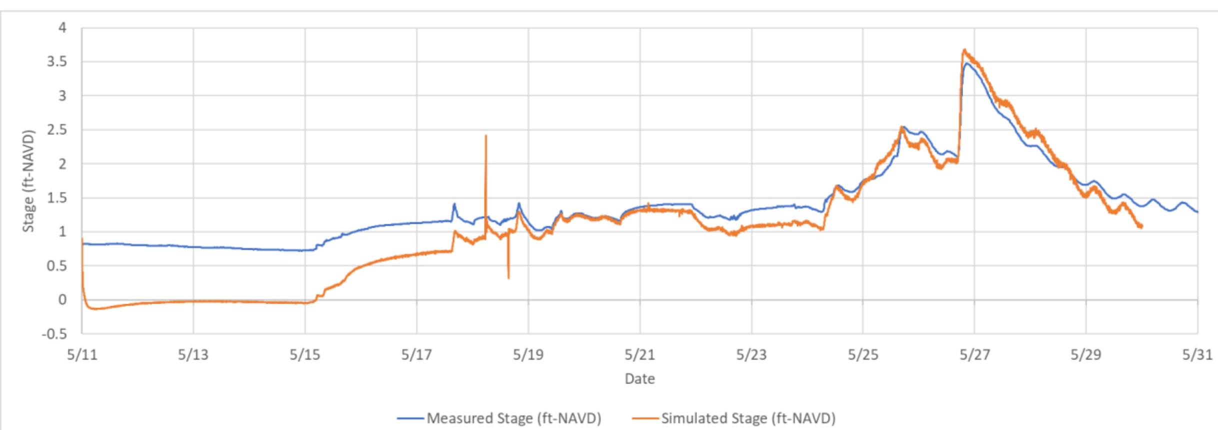


Figure 5-154. Calibration Model Canal Stage – C2SW2

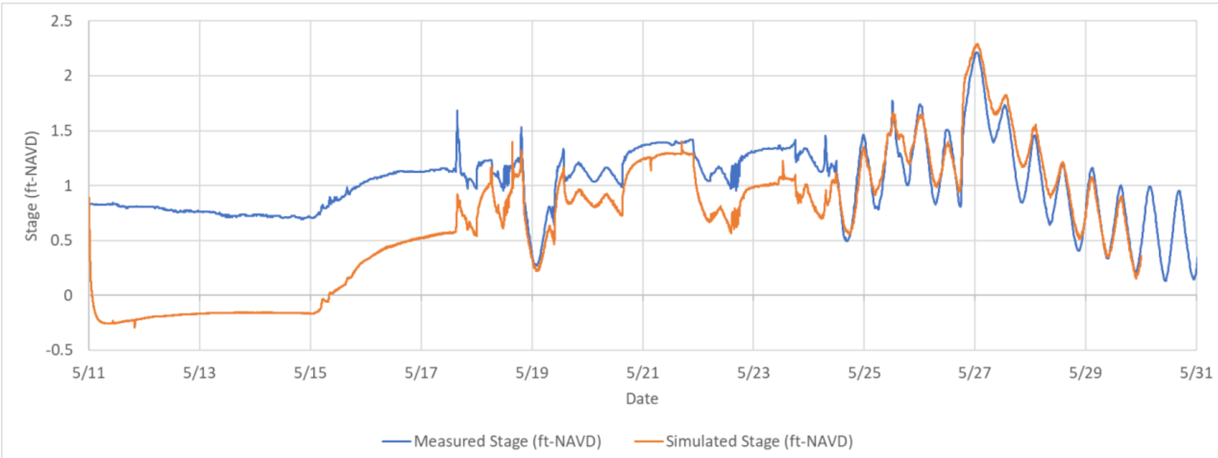
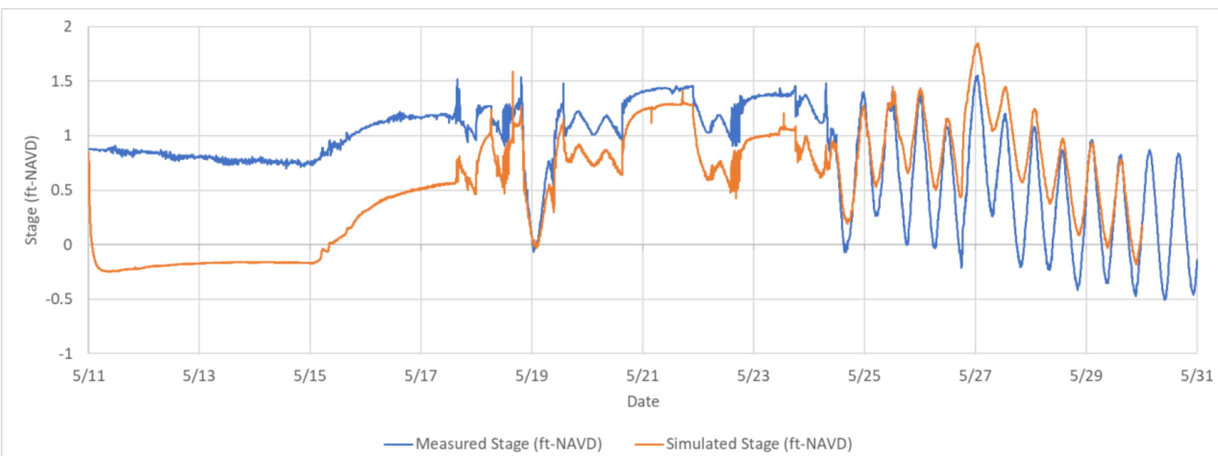
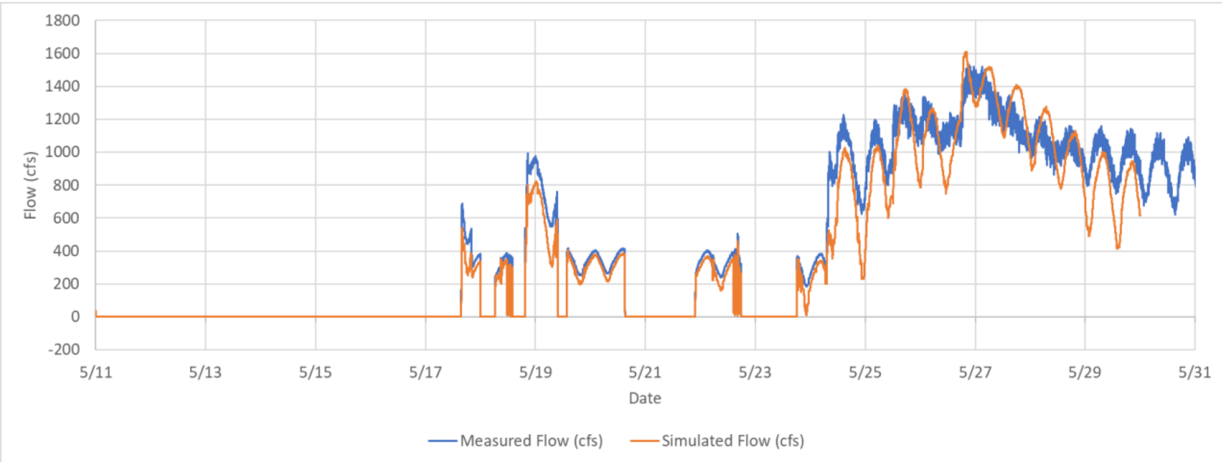


Figure 5-155. Calibration Model Canal Stage – S22-H



5.5.7.3. C2 WATERSHED – CANAL FLOW

Figure 5-156. Calibration Model Canal Flow – S22-Q



5.5.7.4. C2 WATERSHED – GROUNDWATER STAGE

Figure 5-157. Calibration Model Groundwater Stage – G-3558

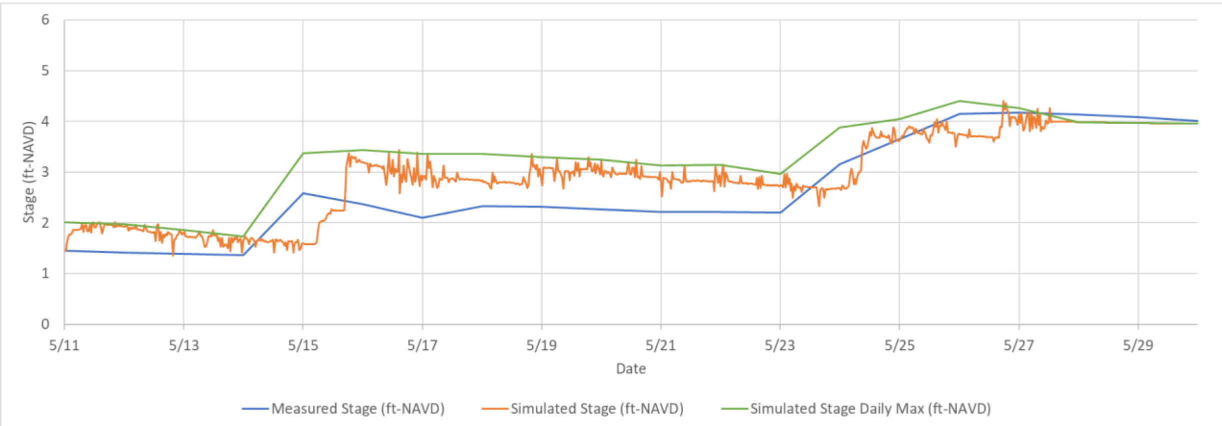


Figure 5-158. Calibration Model Groundwater Stage – G-3563

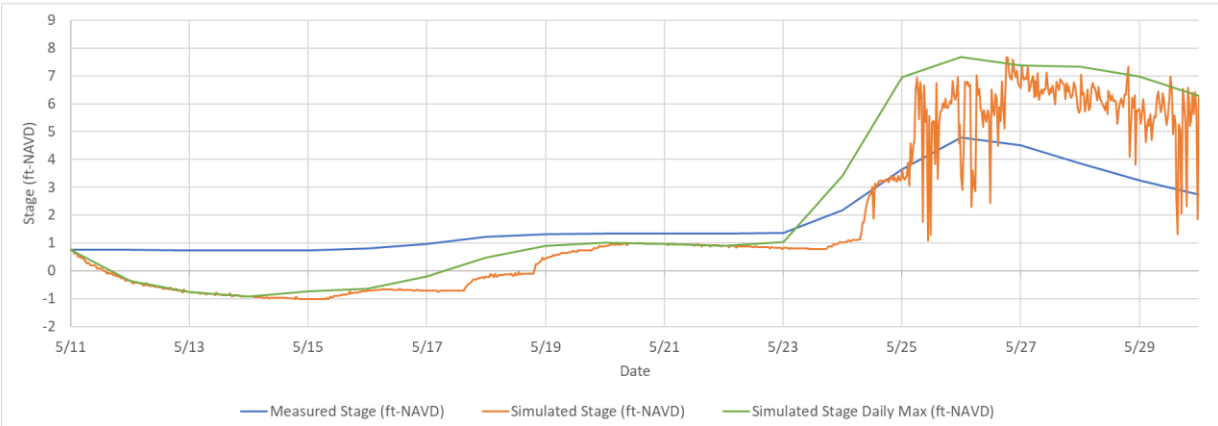


Figure 5-159. Calibration Model Groundwater Stage – G-3572

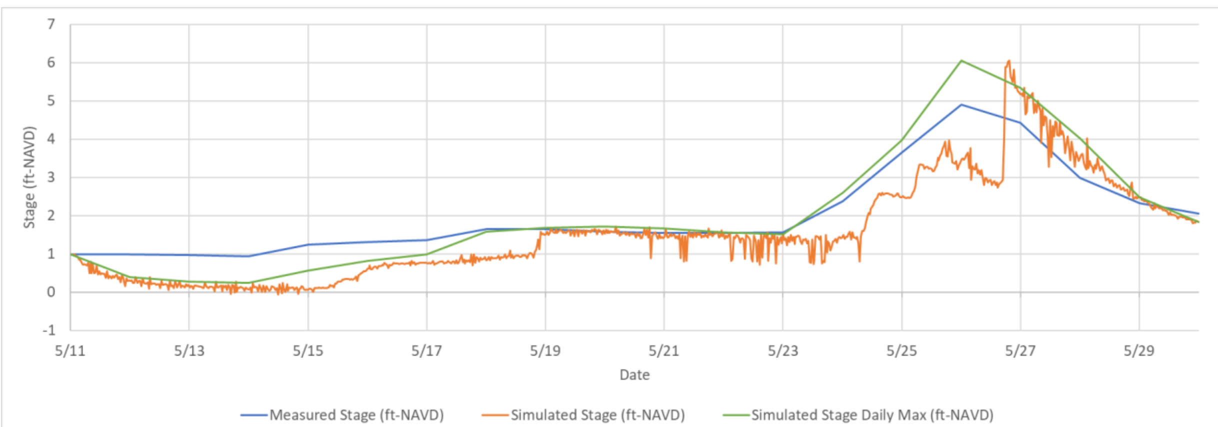


Figure 5-160. Calibration Model Groundwater Stage – G-3565

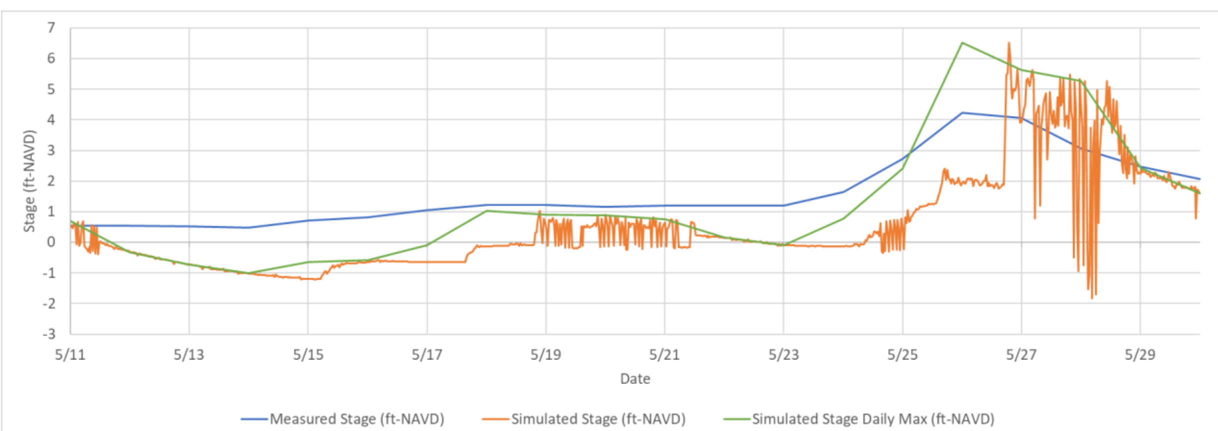


Figure 5-161. Calibration Model Groundwater Stage – F-319

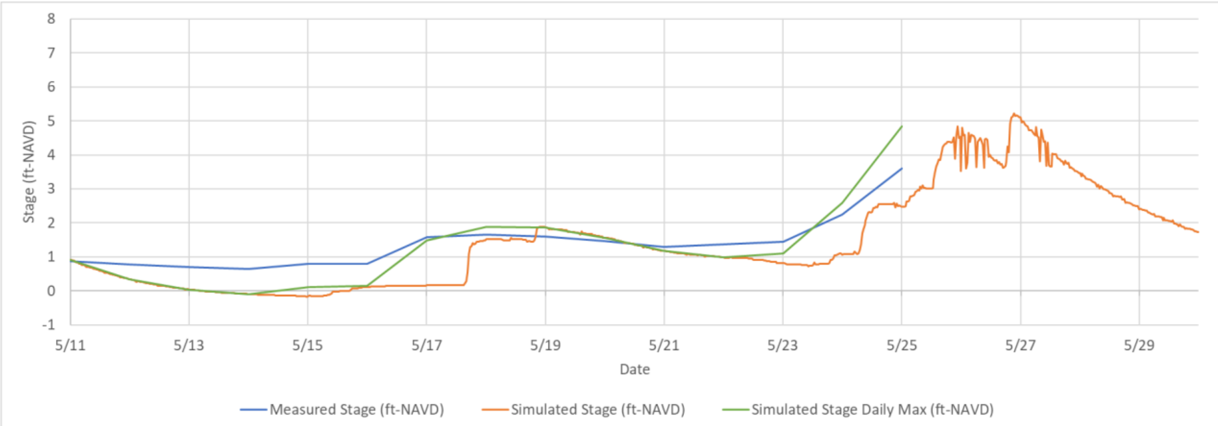


Figure 5-162. Calibration Model Groundwater Stage – G-3439

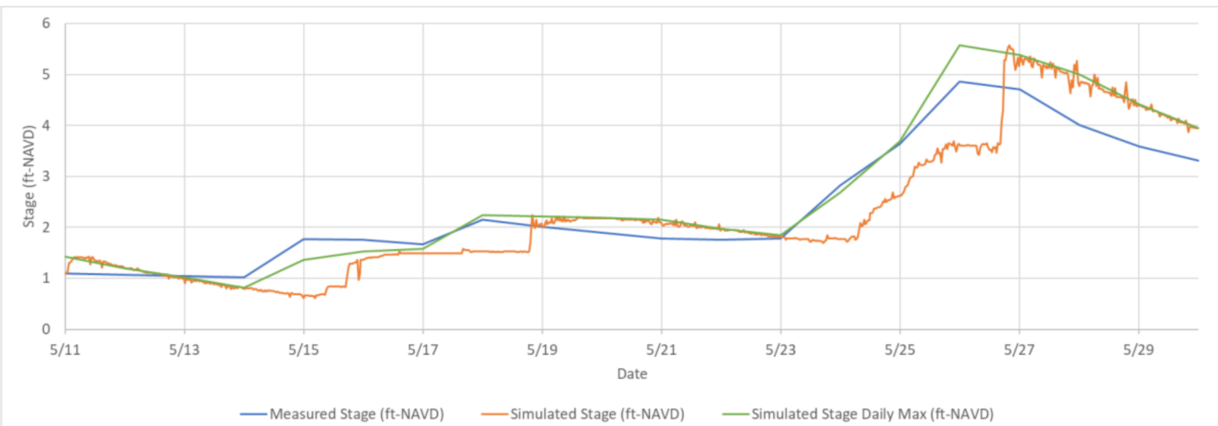


Figure 5-163. Calibration Model Groundwater Stage – C2GSW1

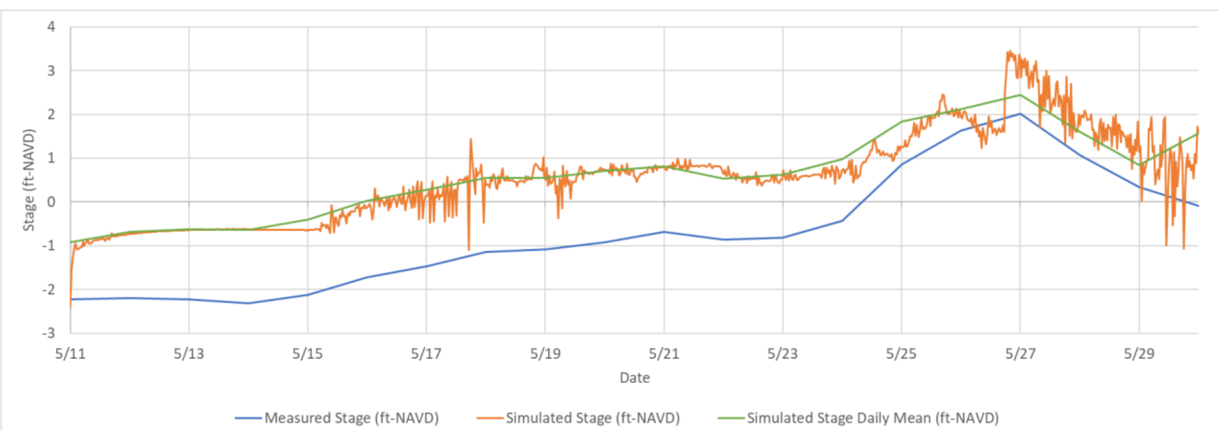


Figure 5-164. Calibration Model Groundwater Stage – G-3909

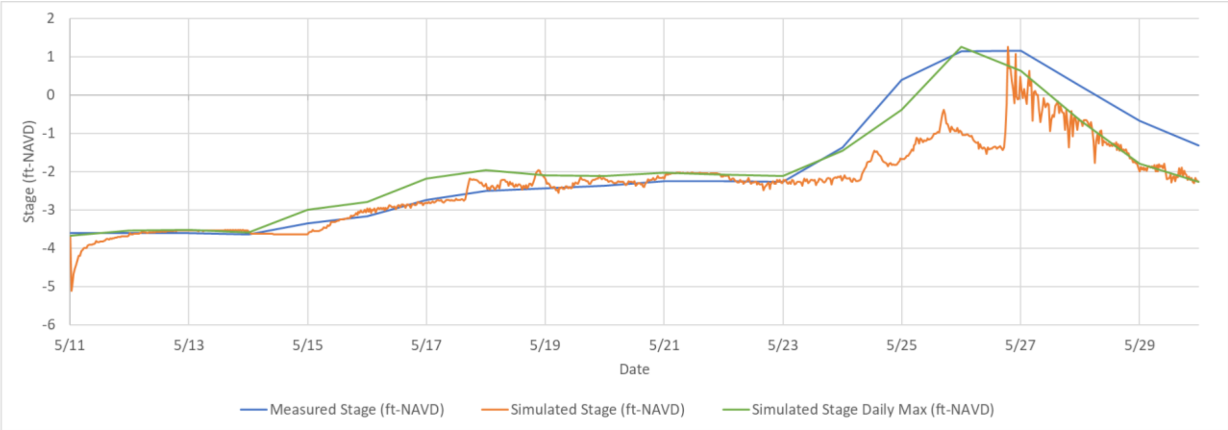


Figure 5-165. Calibration Model Groundwater Stage – G-1074B

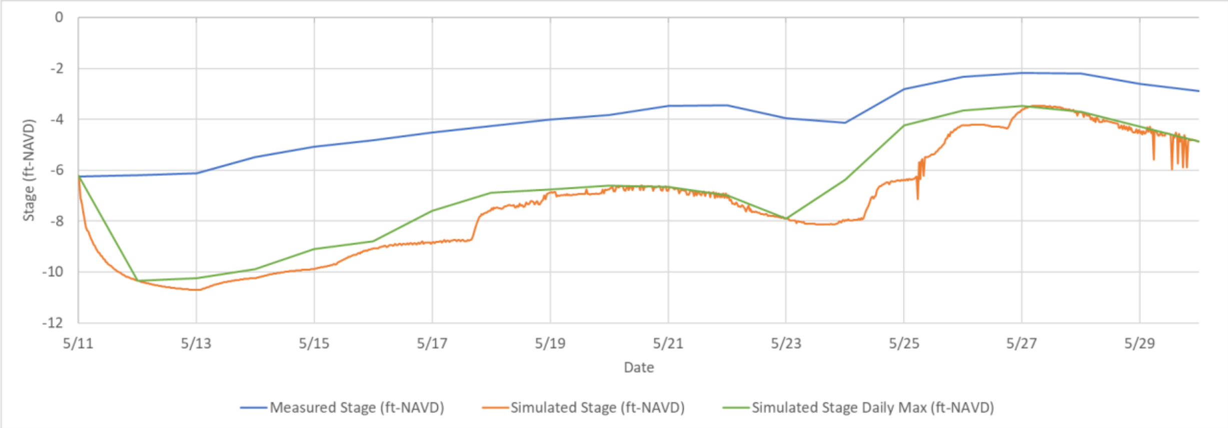
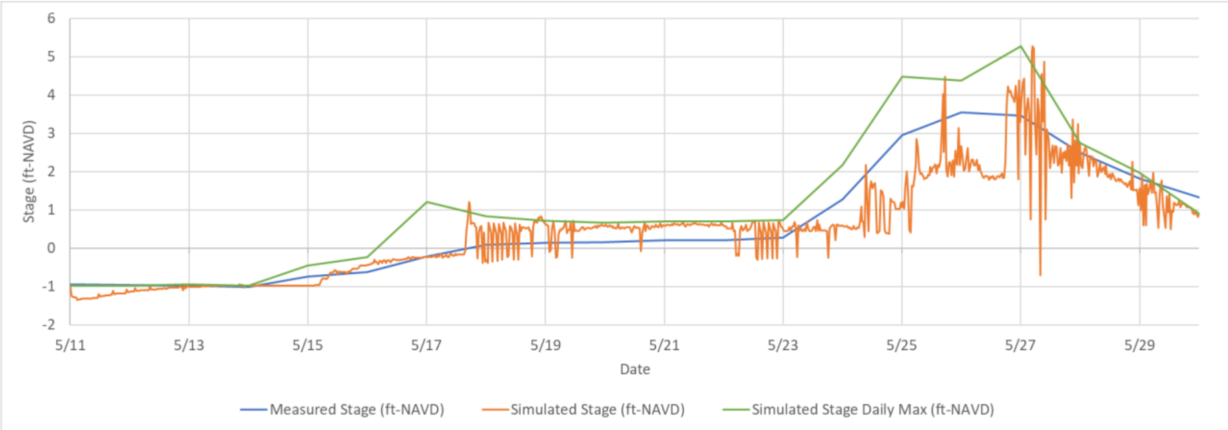
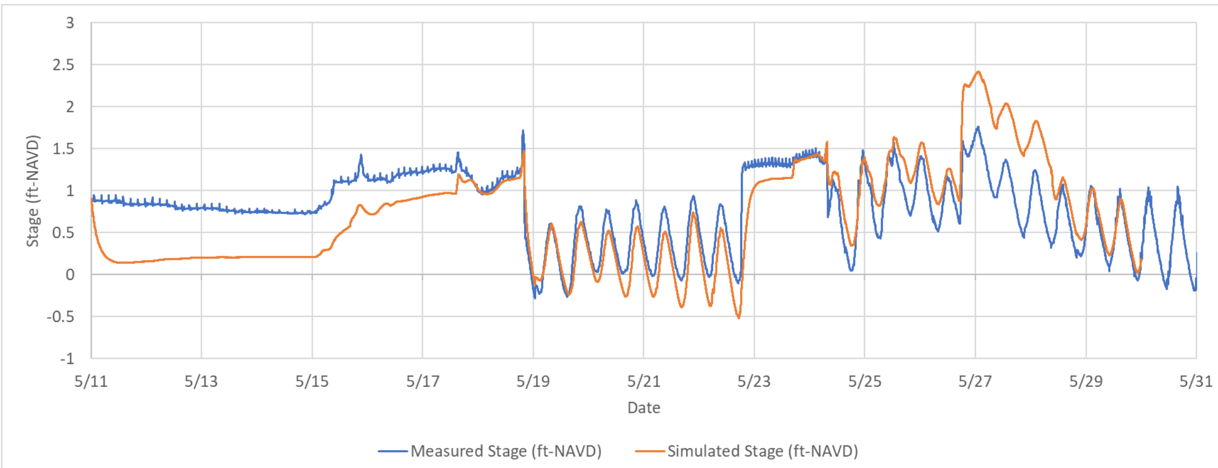


Figure 5-166. Calibration Model Groundwater Stage – G-3918



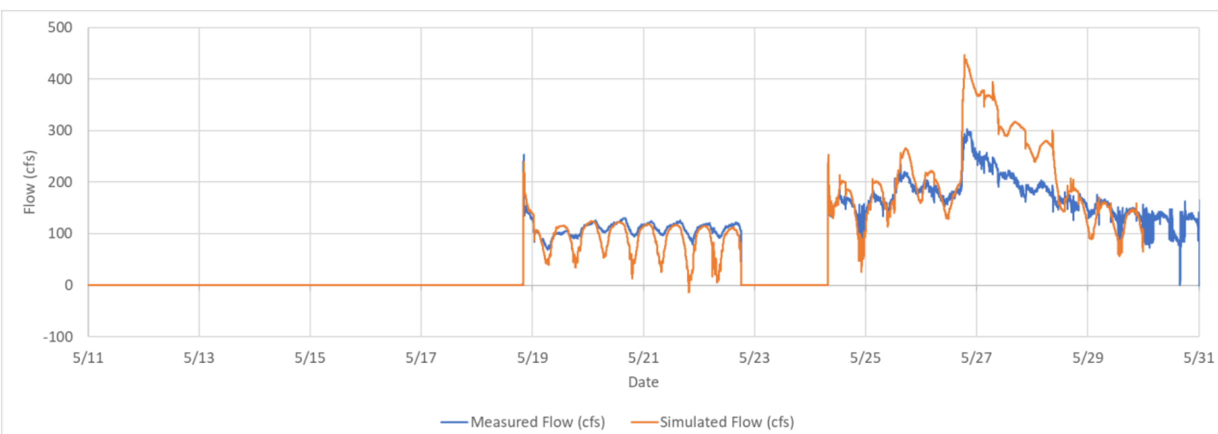
5.5.7.5. C3W WATERSHED – CANAL STAGE

Figure 5-167. Calibration Model Canal Stage – G93-H



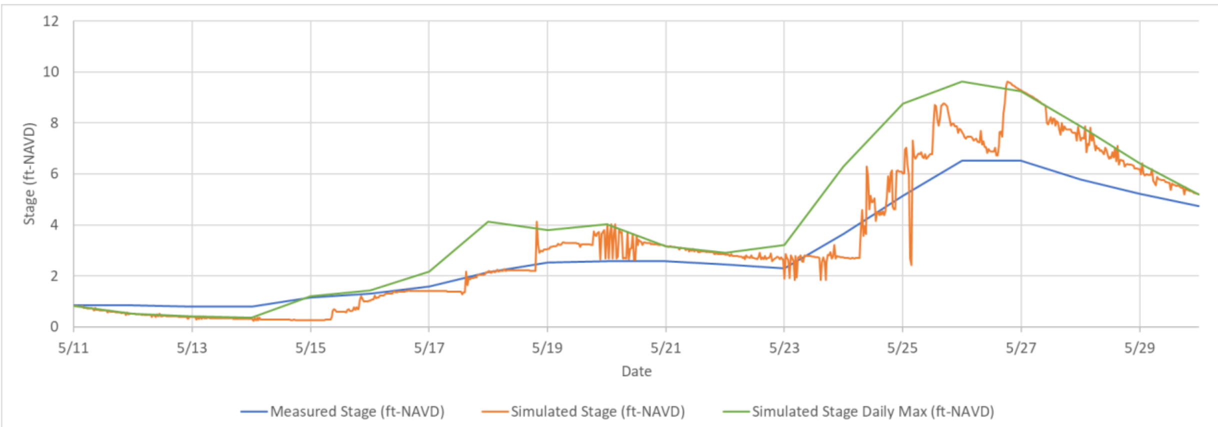
5.5.7.6. C3W WATERSHED – CANAL FLOW

Figure 5-168. Calibration Model Canal Flow – G93-Q



5.5.7.7. C3W WATERSHED – GROUNDWATER STAGE

Figure 5-169. Calibration Model Groundwater Stage – G-3570



5.5.7.8. C4 WATERSHED – CANAL STAGE

Figure 5-170. Calibration Model Canal Stage – S25B-H

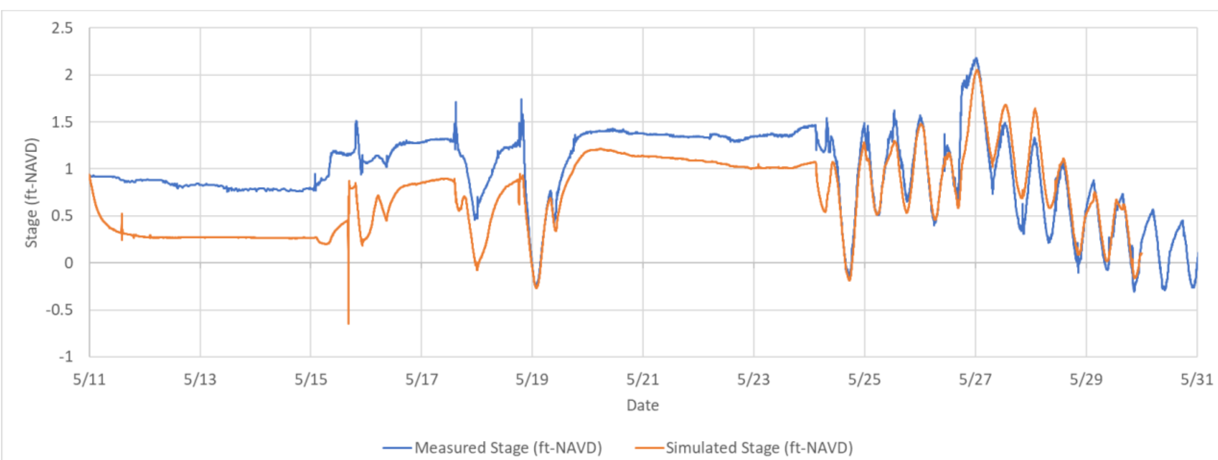


Figure 5-171. Calibration Model Canal Stage – S336-T

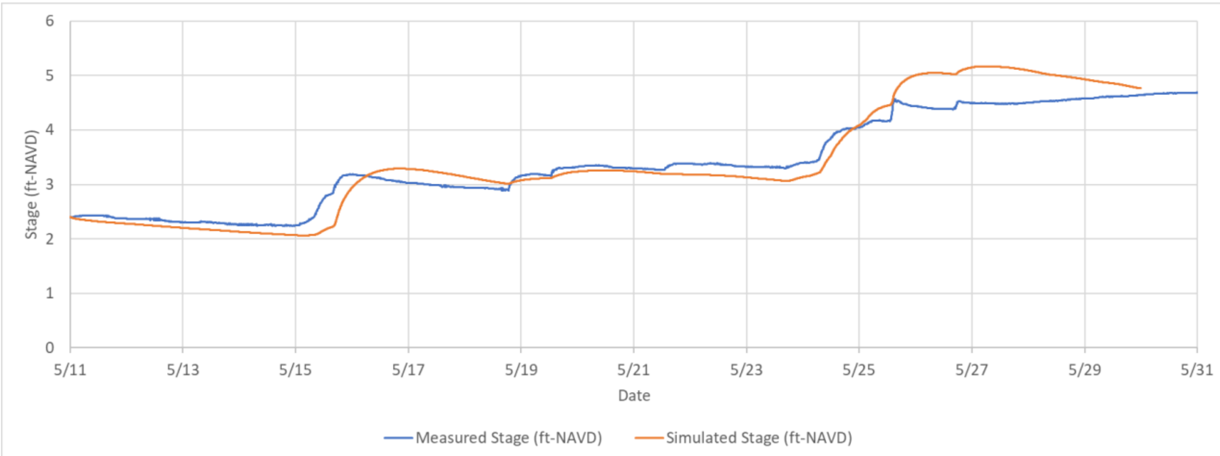


Figure 5-172. Calibration Model Canal Stage – S380-H

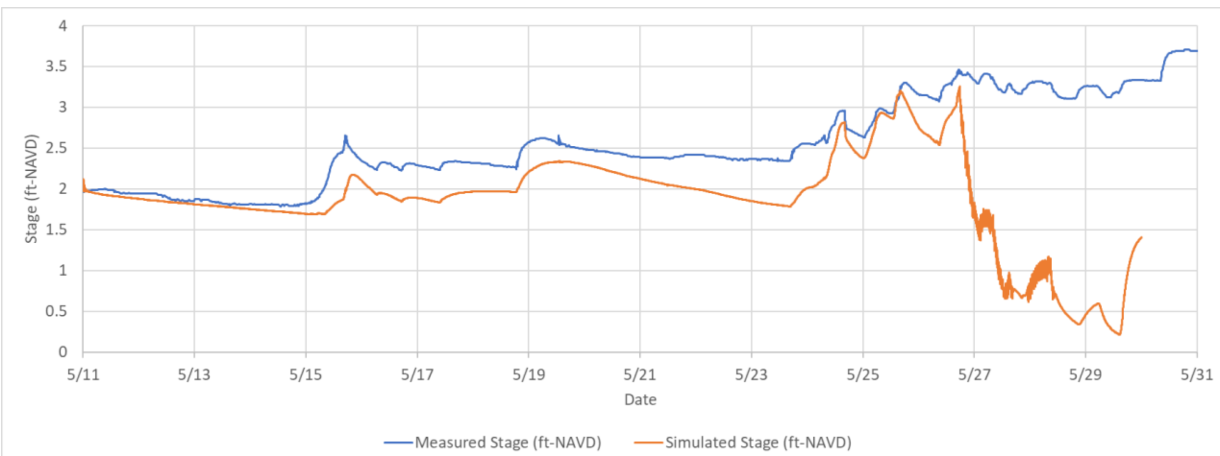


Figure 5-173. Calibration Model Canal Stage – S380-T

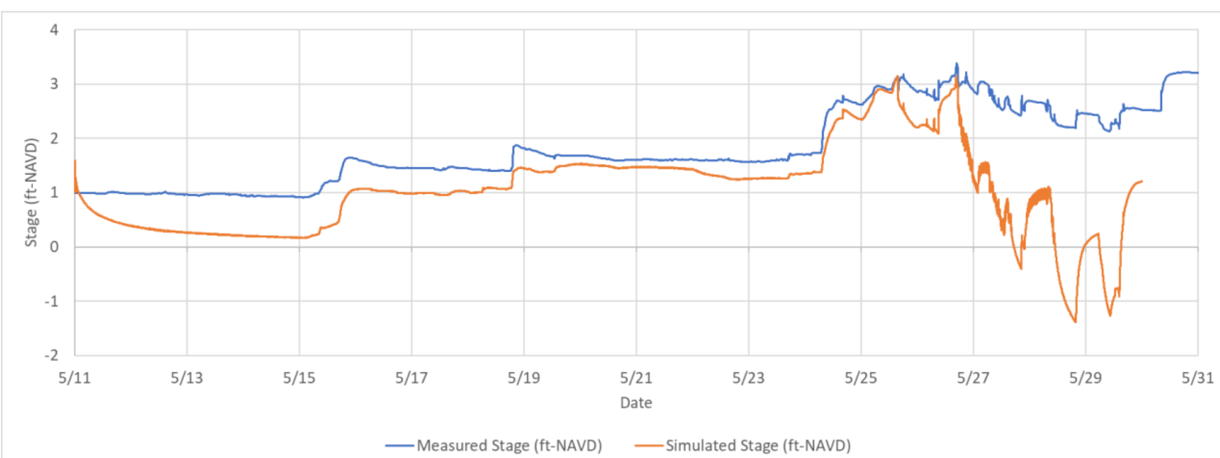


Figure 5-174. Calibration Model Canal Stage – T5W

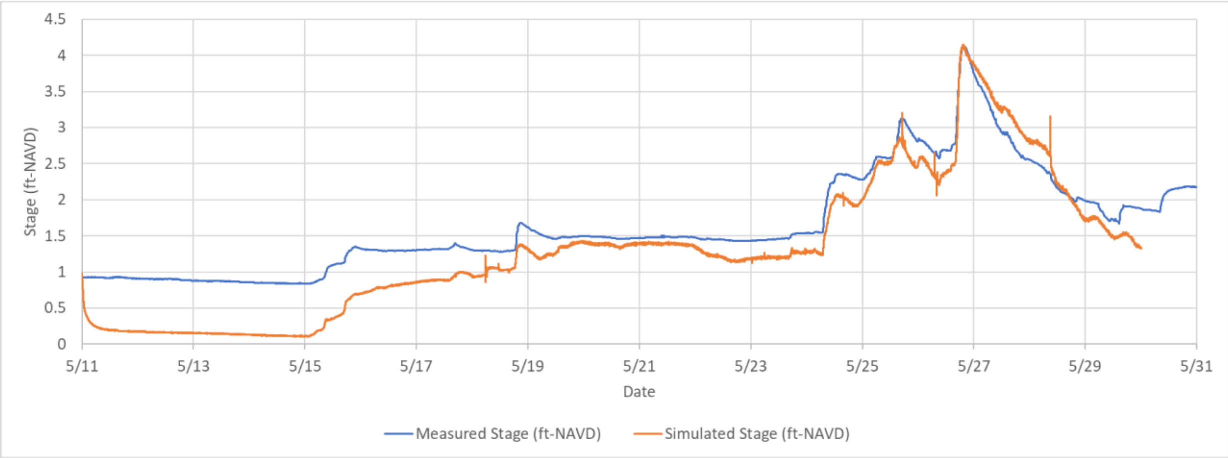


Figure 5-175. Calibration Model Canal Stage – USGS 02287497 (NW Wellfield)

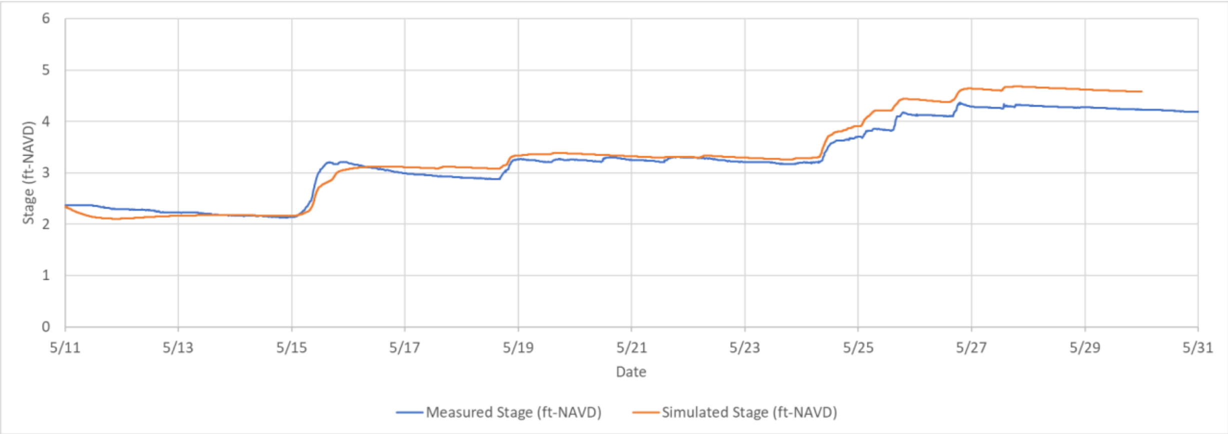
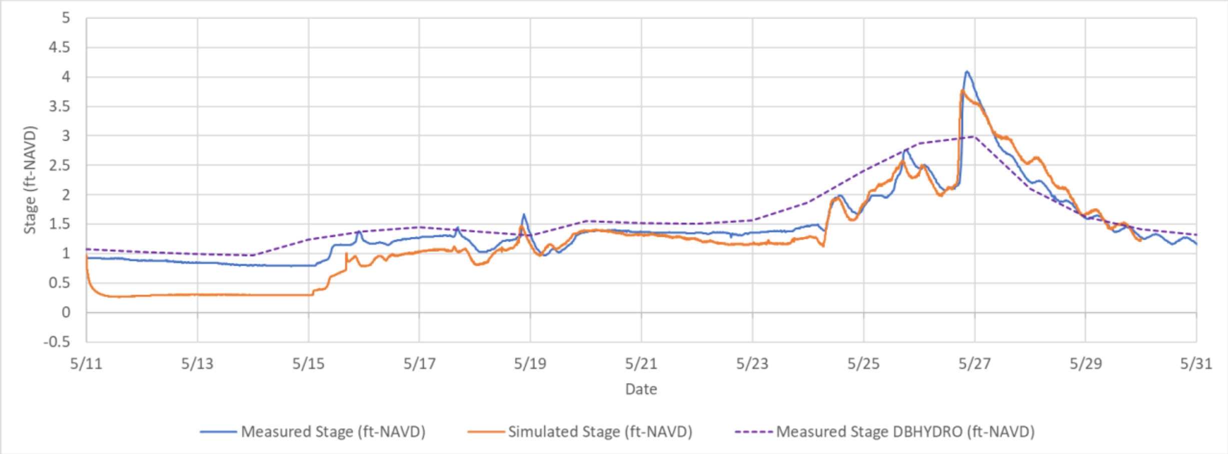


Figure 5-176. Calibration Model Canal Stage – USGS 02289500 (C4.Coral)



5.5.7.9. C4 WATERSHED – CANAL FLOW

Figure 5-177. Calibration Model Canal Flow – S25B-Q

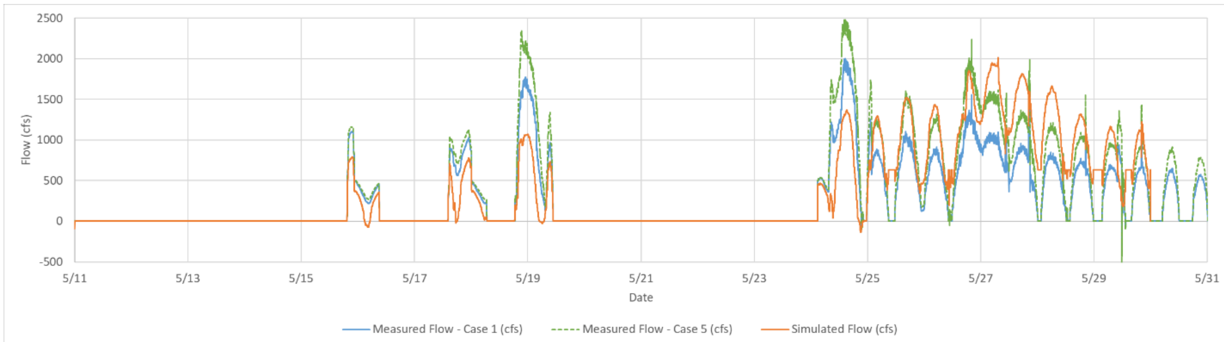


Figure 5-178. Calibration Model Canal Flow – USGS 02289500 (C4.Coral)

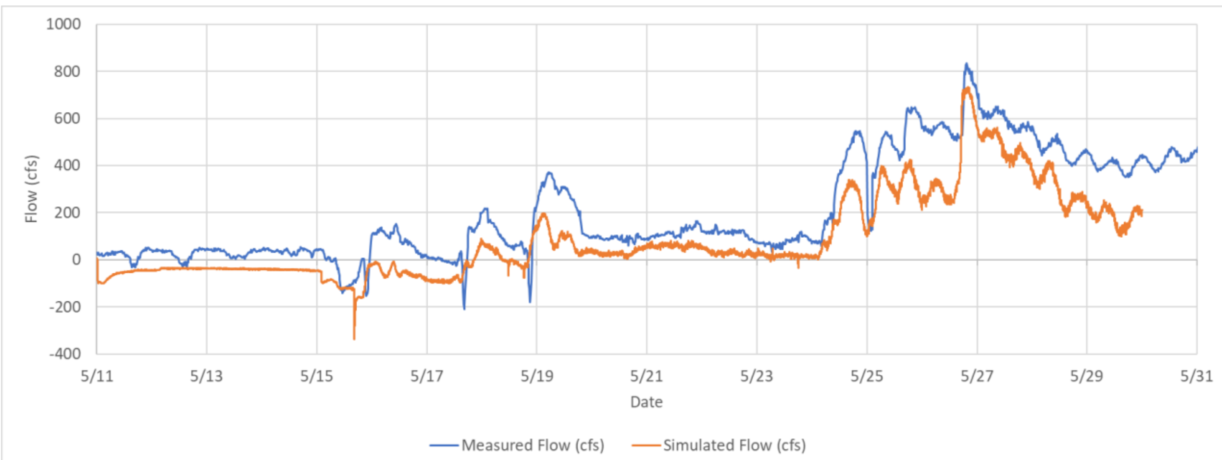


Figure 5-179. Calibration Model Canal Flow – USGS 02287497 (NW Wellfield)

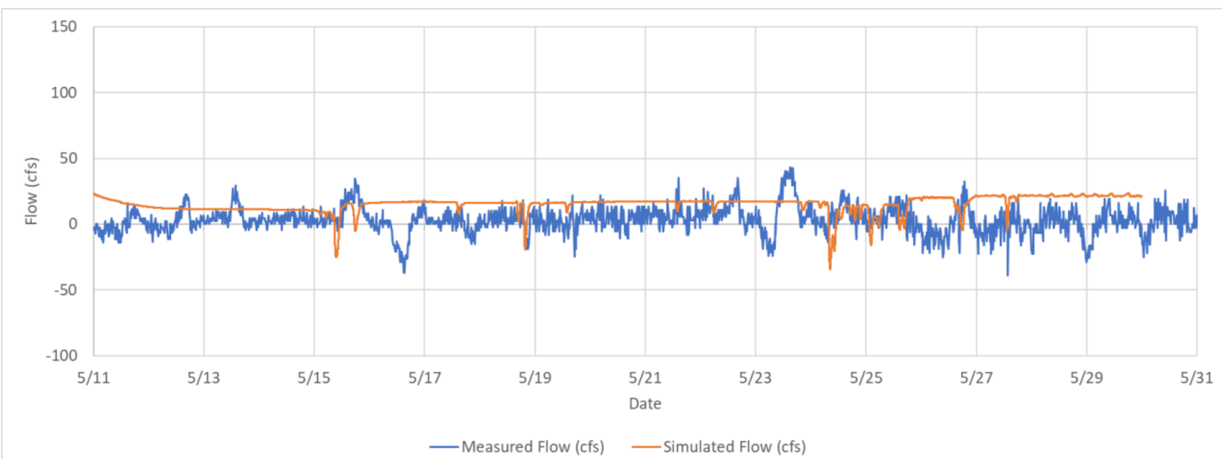


Figure 5-180. Calibration Model Canal Flow – S380-Q

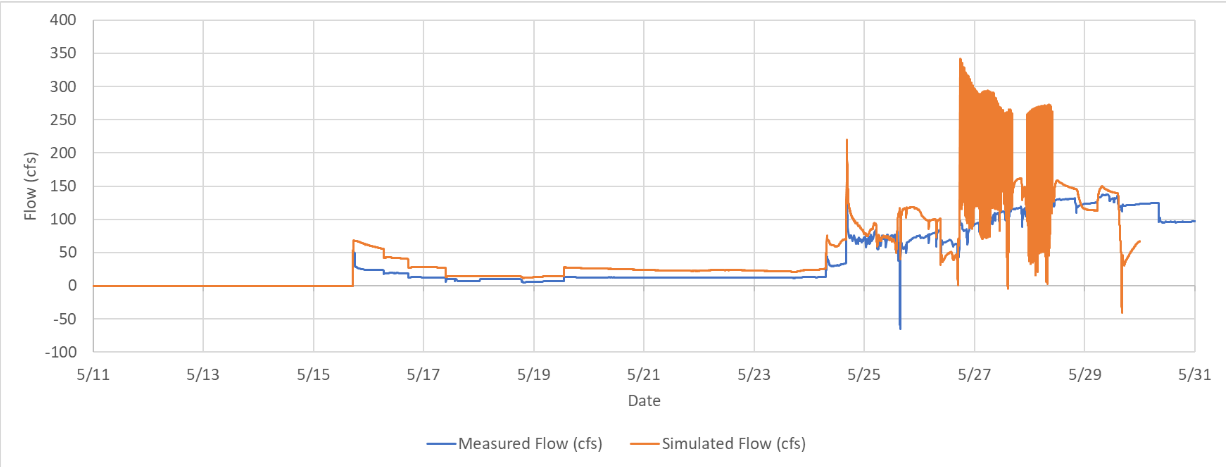
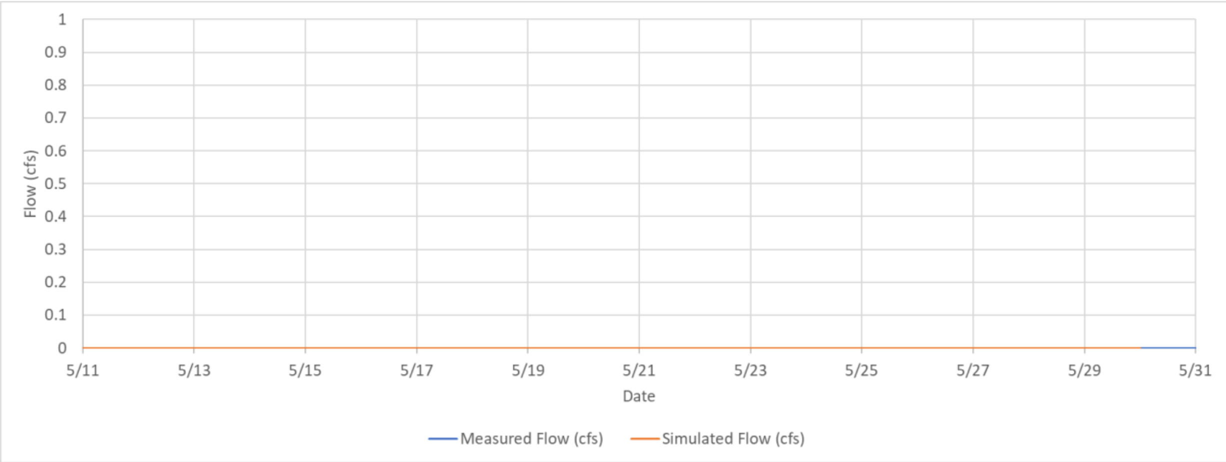


Figure 5-181. Calibration Model Canal Flow – S336-Q



5.5.7.10. C4 WATERSHED – GROUNDWATER STAGE

Figure 5-182. Calibration Model Groundwater Stage – C4GW1

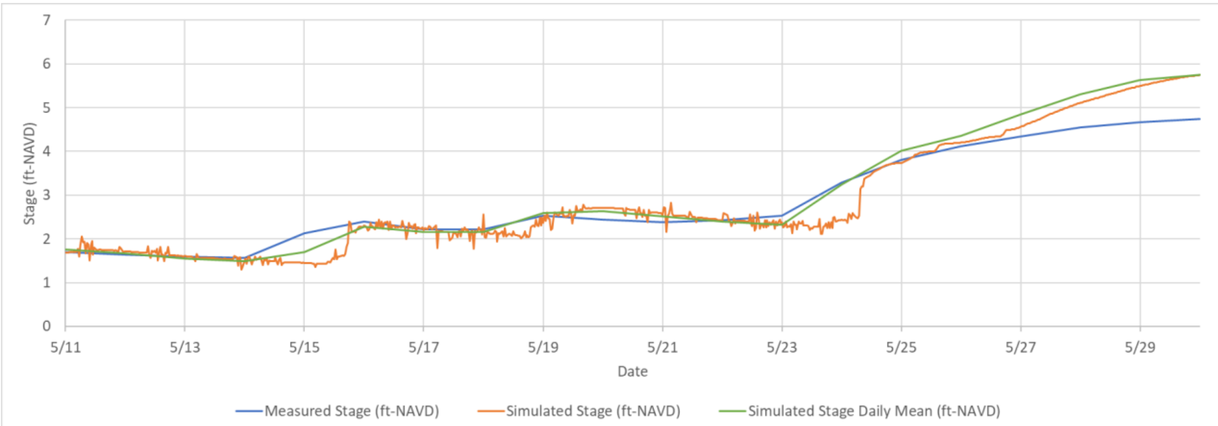


Figure 5-183. Calibration Model Groundwater Stage – G-3465

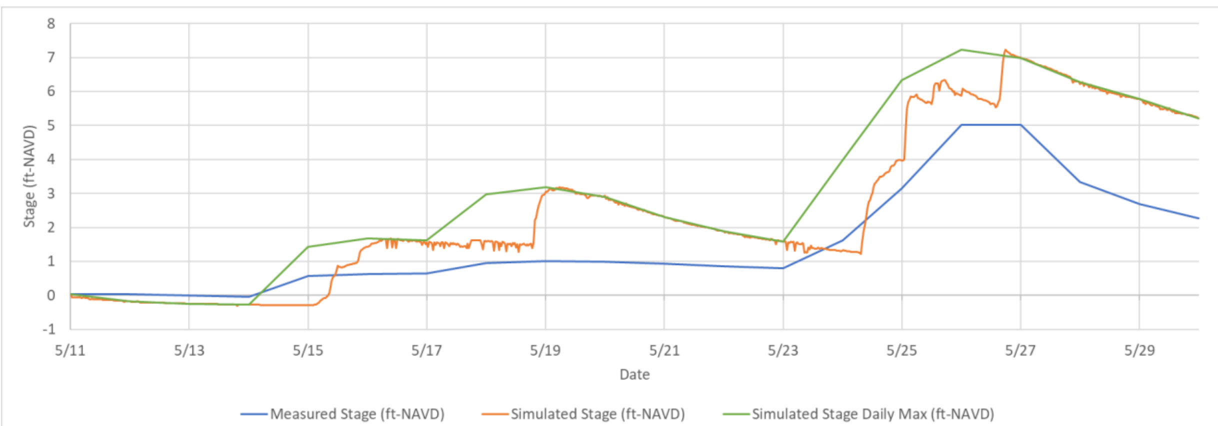


Figure 5-184. Calibration Model Groundwater Stage – G-975

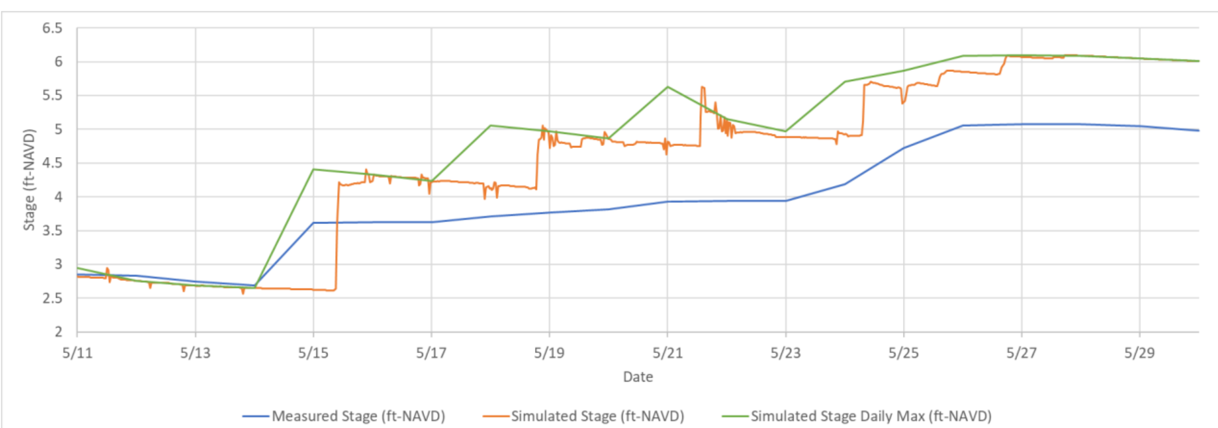


Figure 5-185. Calibration Model Groundwater Stage –S356GW1

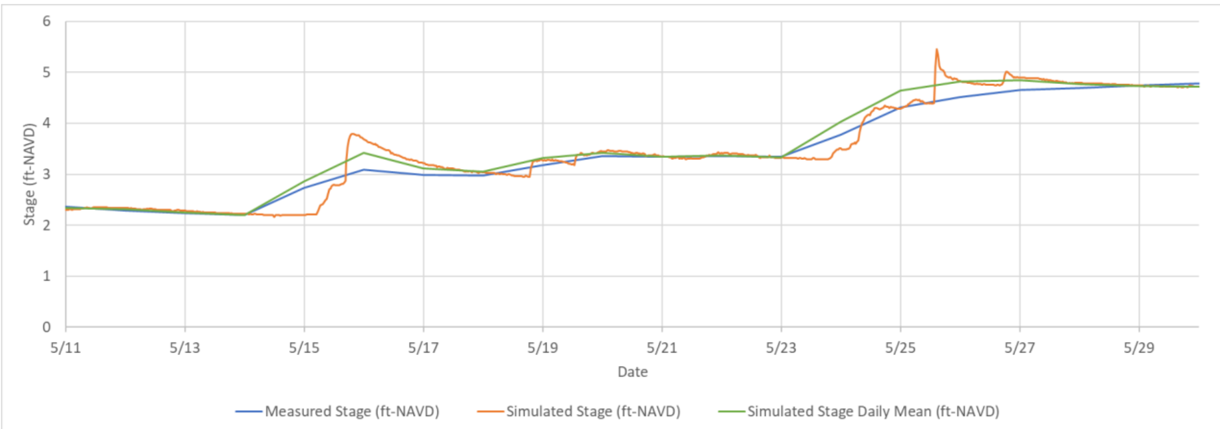
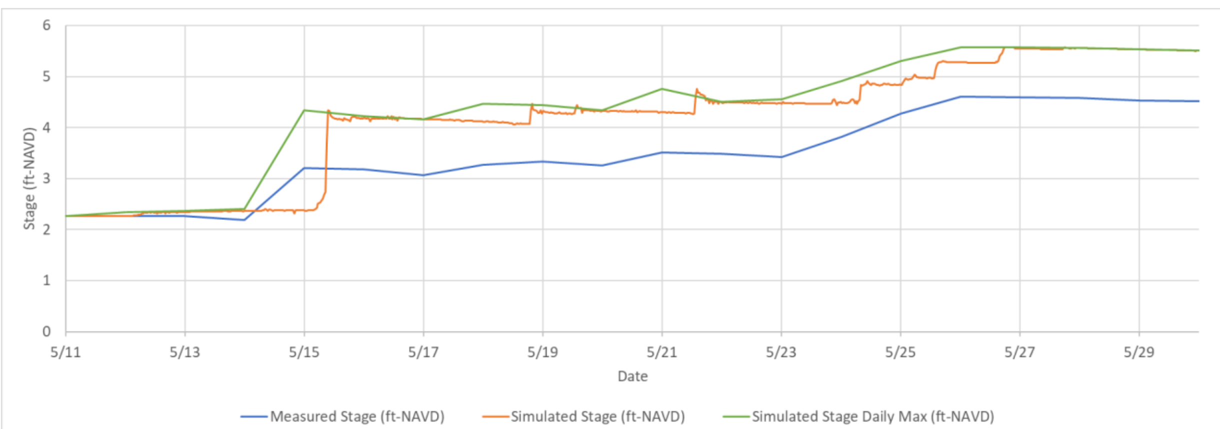
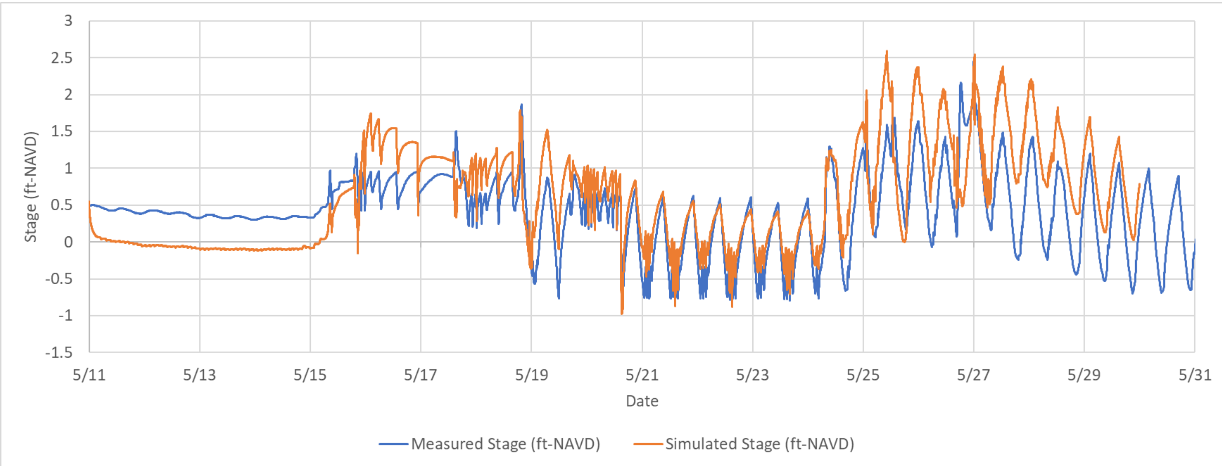


Figure 5-186. Calibration Model Groundwater Stage – G-3818



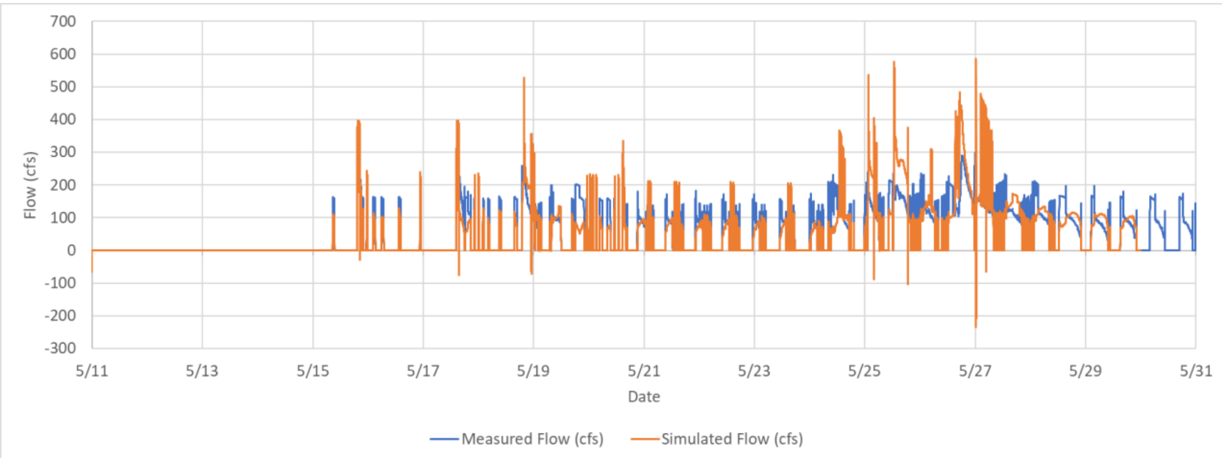
5.5.7.11. C5 WATERSHED – CANAL STAGE

Figure 5-187. Calibration Model Canal Stage – S25-H



5.5.7.12. C5 WATERSHED – CANAL FLOW

Figure 5-188. Calibration Model Canal Flow – S25-Q



5.5.7.13. C6 WATERSHED – CANAL STAGE

Figure 5-189. Calibration Model Canal Stage – S26-H

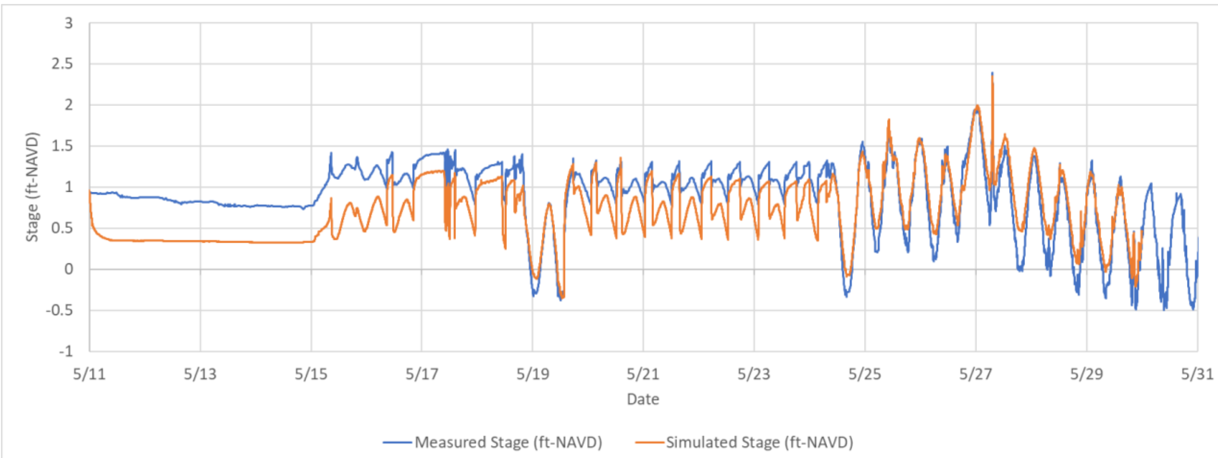
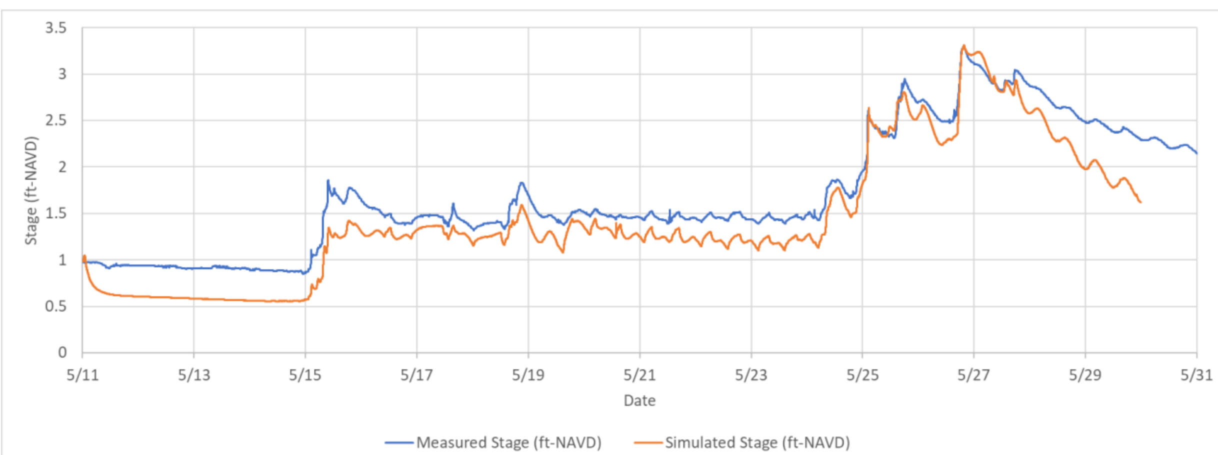


Figure 5-190. Calibration Model Canal Stage – S31-T



5.5.7.14. C6 WATERSHED – CANAL FLOW

Figure 5-191. Calibration Model Canal Flow – S26-Q

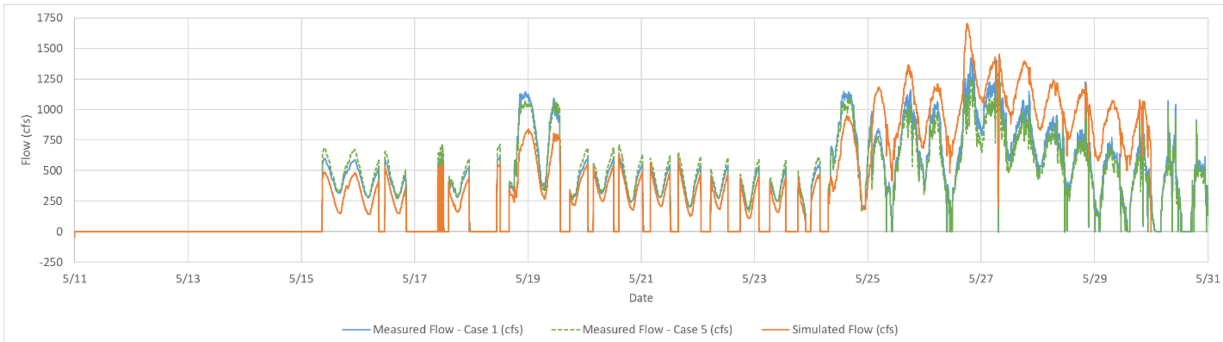
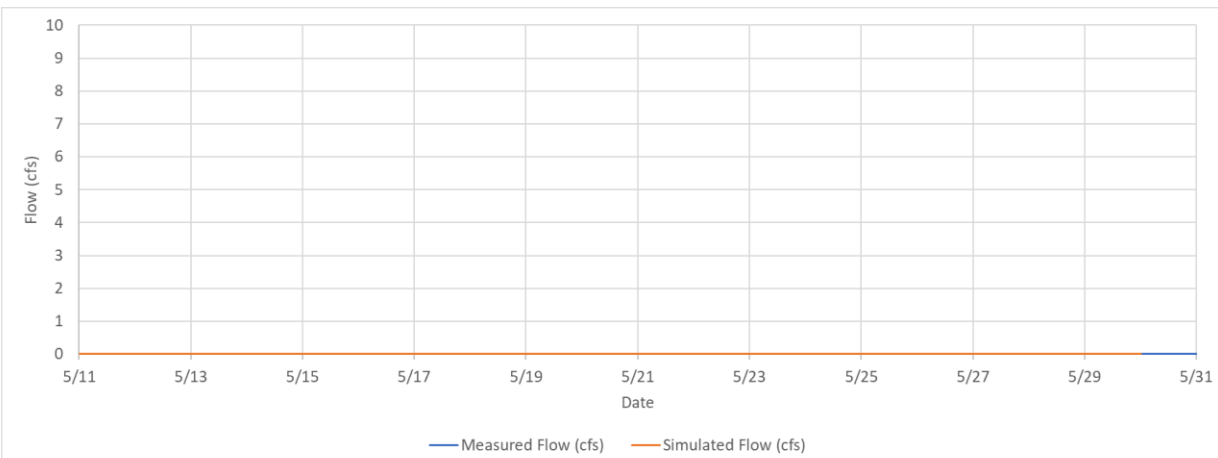


Figure 5-192. Calibration Model Canal Flow – S31-Q



5.5.7.15. C6 WATERSHED – GROUNDWATER STAGE

Figure 5-193. Calibration Model Groundwater Stage – G-3

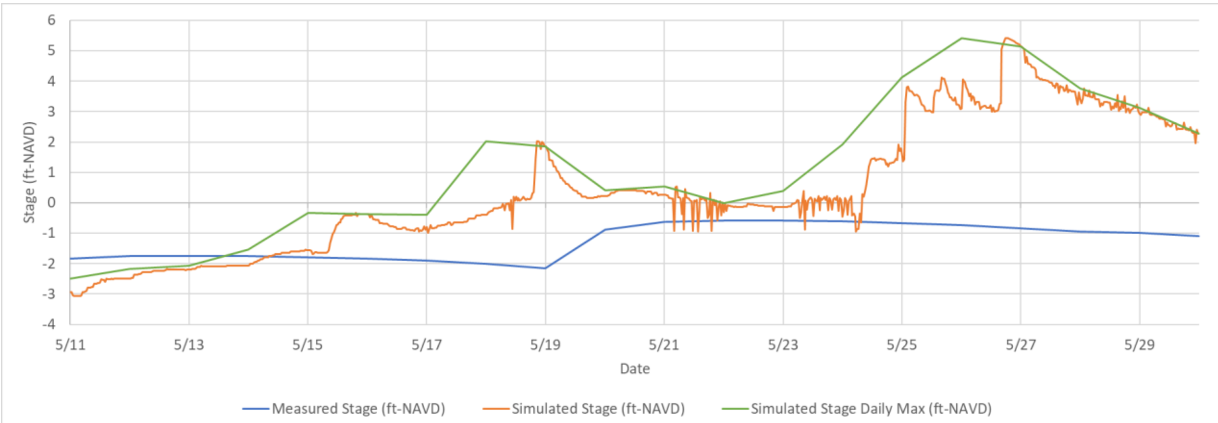


Figure 5-194. Calibration Model Groundwater Stage – G-3567

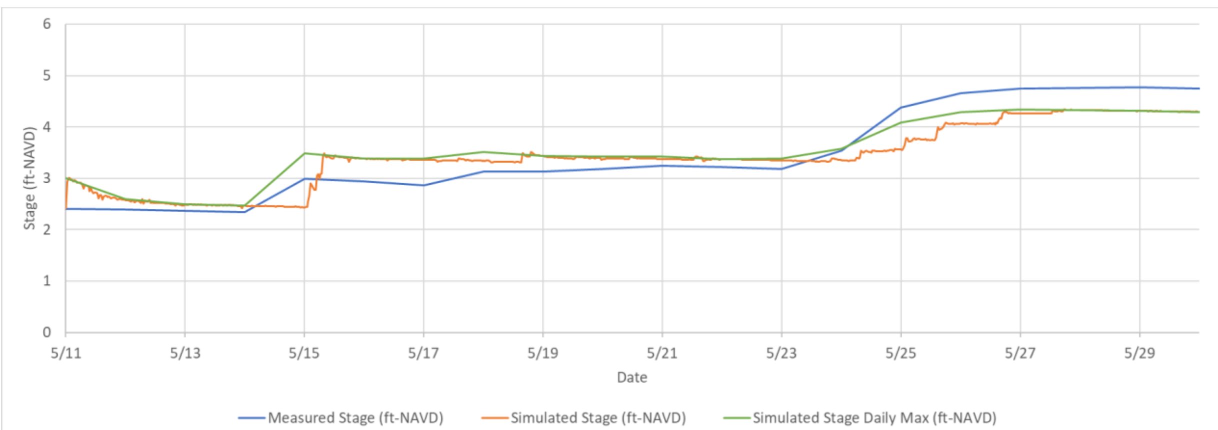


Figure 5-195. Calibration Model Groundwater Stage – S-68

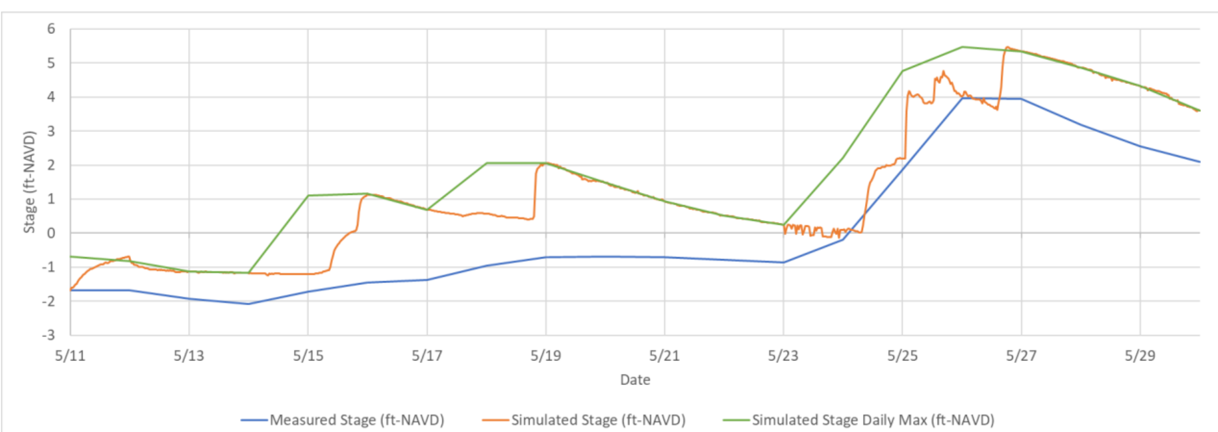
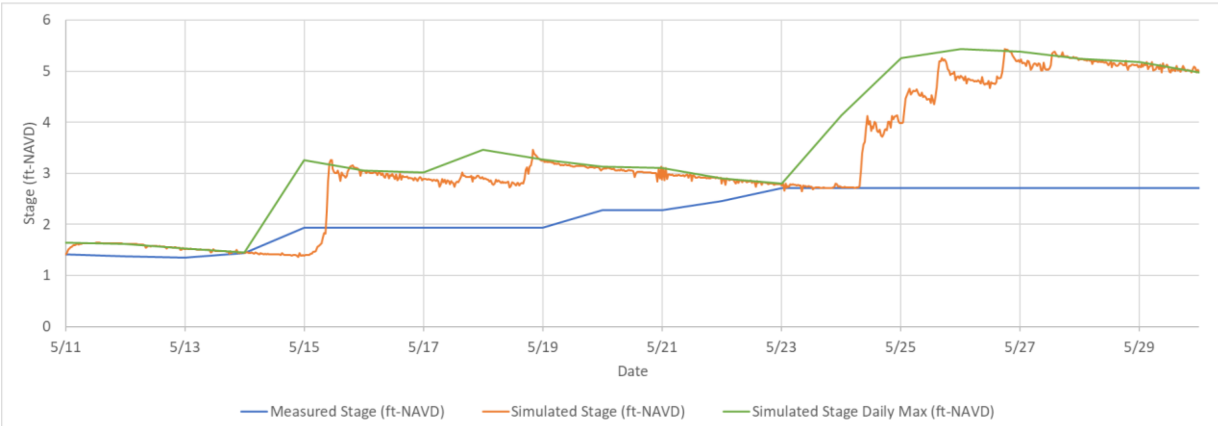


Figure 5-196. Calibration Model Groundwater Stage – G-3264AR



5.5.8. FINAL VALIDATION RESULTS

5.5.8.1. UPDATED STATISTICS FOR THE VALIDATION PERIOD

The following tables provide updated statistics for the validation period of 9/2/2017 through 9/30/2017 (the first day of the simulation is ignored as it is considered a warm-up day) for surface water (**Table 5-68**) and groundwater (**Table 5-70**). The 7-day statistics are also provided to qualify model performance during the rain event (**Table 5-69** and **Table 5-71**).

Table 5-68. Updated Surface Water Statistics for Validation

Watershed	Station	29 - Day Simulation (9/02/2017 - 9/30/2017)				
		ME	MAE	RMSE	R	NS
C2	C2SW1	0.185	0.240	0.318	0.968	0.824
	C2SW2	0.176	0.188	0.244	0.981	0.895
	S22H	0.048	0.187	0.251	0.959	0.914
C3W	G93H	-0.530	0.600	0.713	0.864	0.356
C4	USGS 02289500	-0.214	0.355	0.424	0.860	0.510
	S25BH	-0.162	0.342	0.401	0.891	0.743
	S336T	0.108	0.337	0.394	0.516	0.207
	S380H	1.525	1.525	1.582	0.836	-9.913
	S380T	1.545	1.545	1.650	0.636	-16.719
	T5W	0.301	0.340	0.443	0.963	0.443
	USGS 02287497	0.296	0.296	0.314	0.951	-3.164
C5	S25H	-0.156	0.246	0.427	0.852	0.623
C6	S26H	0.036	0.224	0.283	0.906	0.818
	S31T	0.281	0.348	0.431	0.903	0.677

Table 5-69. Updated 7-Day Surface Water Statistics for Validation

Watershed	Station	7 - Day Simulation (9/08/2017 - 9/14/2017)				
		ME	MAE	RMSE	R	NS
C2	C2SW1	0.217	0.248	0.337	0.995	0.915
	C2SW2	0.140	0.172	0.239	0.991	0.960
	S22H	-0.060	0.250	0.330	0.970	0.937
C3W	G93H	-0.311	0.399	0.489	0.967	0.890
C4	USGS 02289500	0.000	0.186	0.241	0.983	0.943
	S25BH	-0.063	0.262	0.298	0.962	0.922
	S336T	0.286	0.286	0.317	0.926	0.102
	S380H	1.413	1.413	1.515	0.877	-4.529
	S380T	1.827	1.827	1.994	0.523	-18.737
	T5W	0.279	0.330	0.448	0.994	0.711
	USGS 02287497	0.269	0.269	0.290	0.992	-0.733
C5	S25H	0.028	0.153	0.209	0.979	0.957
C6	S26H	-0.068	0.245	0.303	0.953	0.903
	S31T	0.472	0.473	0.557	0.970	0.770

Table 5-70. Updated Groundwater Statistics for the Validation Period

Watershed	Station	29 - Day Simulation (9/1/2017 - 9/29/2017)				
		ME	MAE	RMSE	R	NS
C2	G-3558	0.450	0.450	0.492	0.926	-0.298
	G-3563	0.534	1.133	1.430	0.882	-1.754
	G-3572	-0.223	0.389	0.453	0.981	0.732
	G-3565	-0.293	0.898	1.039	0.935	-0.633
	G-3903	0.070	0.553	0.820	0.960	0.250
	G-1074B	-2.782	2.782	3.043	0.920	-4.958
C3W	G-3570	0.555	0.637	0.892	0.912	-0.013
C3	F-319	-0.106	0.368	0.466	0.961	0.682
C4	C4GW1	-0.296	0.303	0.337	0.948	0.280
	G-3465	1.382	1.383	1.679	0.887	-3.552
	G-975	0.134	0.168	0.246	0.996	0.986
	S356GW1	-0.528	0.528	0.570	0.996	0.949
	G-3818	0.110	0.161	0.191	0.923	0.365
	G-3676	0.676	0.676	0.697	0.914	-2.119
	G-3327	1.443	1.443	1.847	0.821	-6.734
C6	G-3	-0.155	0.919	1.062	0.558	-3.915
	G-3567	-0.029	0.145	0.149	0.837	0.641
	S-68	1.019	1.081	1.392	0.838	-0.951
	G-3264AR	0.494	0.494	0.553	0.882	-0.336

Table 5-71. Updated 7-Day Groundwater Statistics for the Validation Period

Watershed	Station	7 - DAY SIMULATION (9/8/2017 - 9/14/2017)				
		ME	MAE	RMSE	R	NS
C2	G-3558	0.275	0.275	0.361	0.994	0.539
	G-3563	1.599	2.201	2.374	0.973	-3.651
	G-3572	0.005	0.473	0.536	0.996	0.839
	G-3565	0.653	1.422	1.548	0.936	-0.703
	G-3903	0.737	0.999	1.401	0.987	-0.073
	G-1074B	-2.353	2.353	2.807	0.992	-5.323
C3W	G-3570	0.963	1.067	1.393	0.970	-0.436
C3	F-319	0.202	0.582	0.723	0.990	0.561
C4	C4GW1	-0.193	0.222	0.266	0.990	0.764
	G-3465	2.010	2.010	2.205	0.993	-3.393
	G-975	0.308	0.426	0.484	0.998	0.993
	S356GW1	-0.431	0.431	0.435	0.999	0.092
	G-3818	0.090	0.116	0.122	0.974	0.817
	G-3676	0.559	0.559	0.602	0.932	-0.374
	G-3327	2.228	2.228	2.496	0.939	-5.909
C6	G-3	0.264	0.884	1.087	0.844	-1.782
	G-3567	-0.036	0.127	0.136	0.977	0.815
	S-68	1.853	1.853	1.969	0.963	-1.779
	G-3264AR	0.397	0.397	0.415	0.982	0.595

5.5.8.2. C2 WATERSHED – CANAL STAGE

Figure 5-197. Validation Model Canal Stage – C2SW1

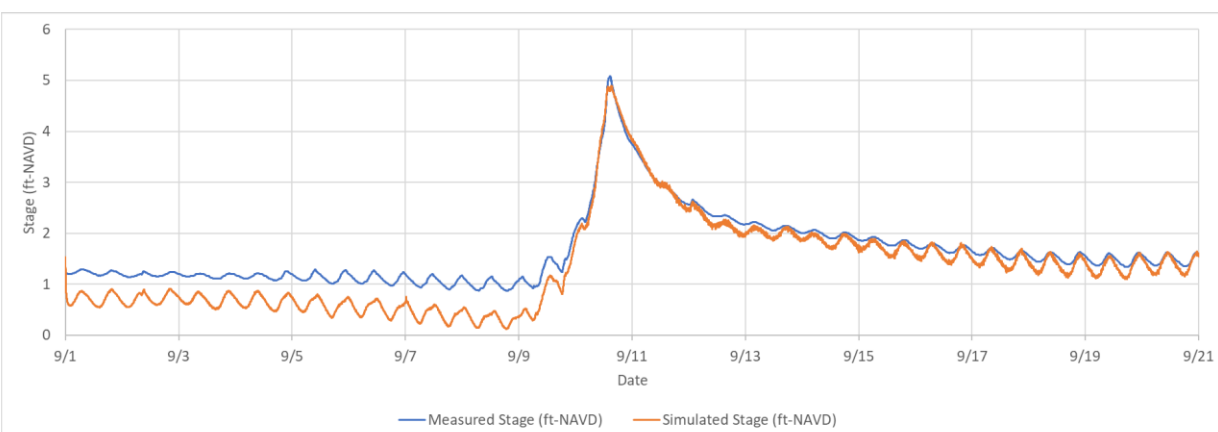


Figure 5-198. Validation Model Canal Stage – C2SW2

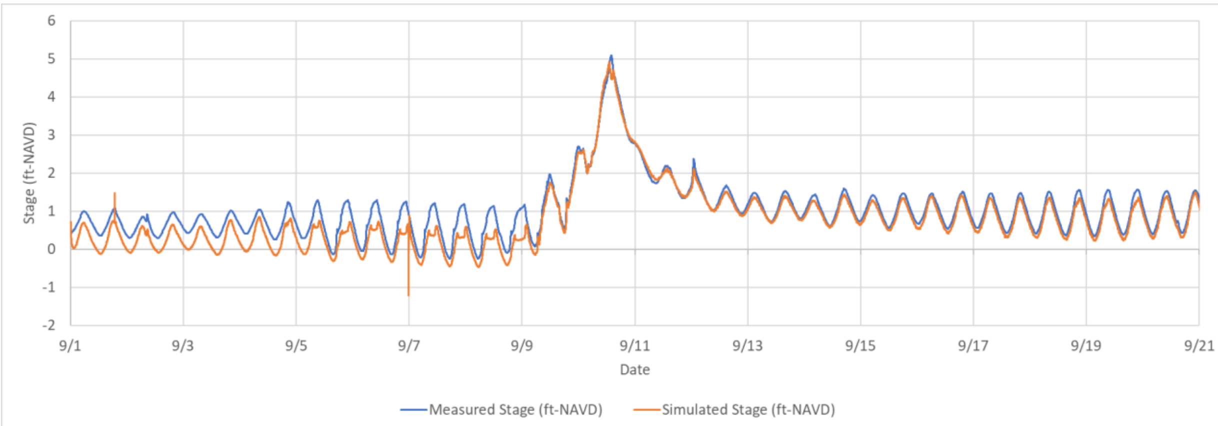
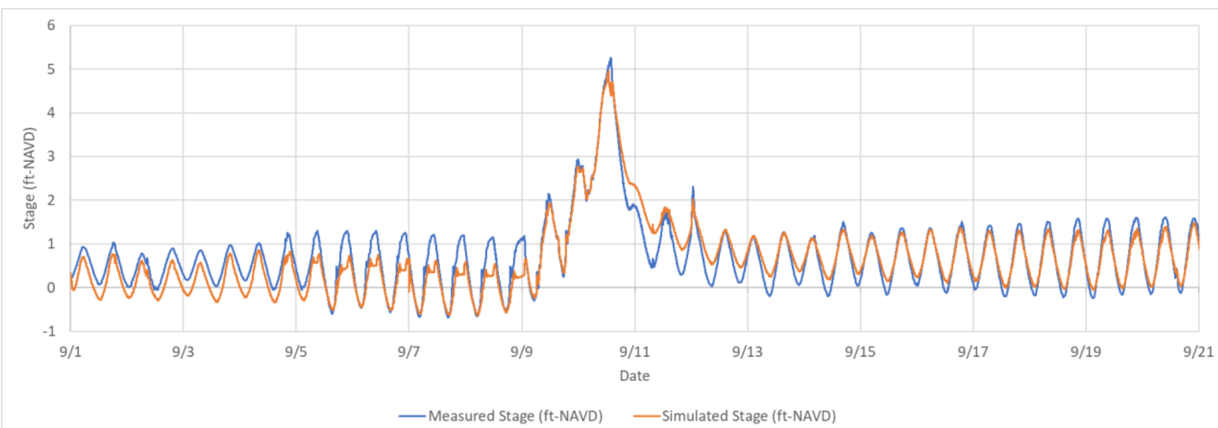
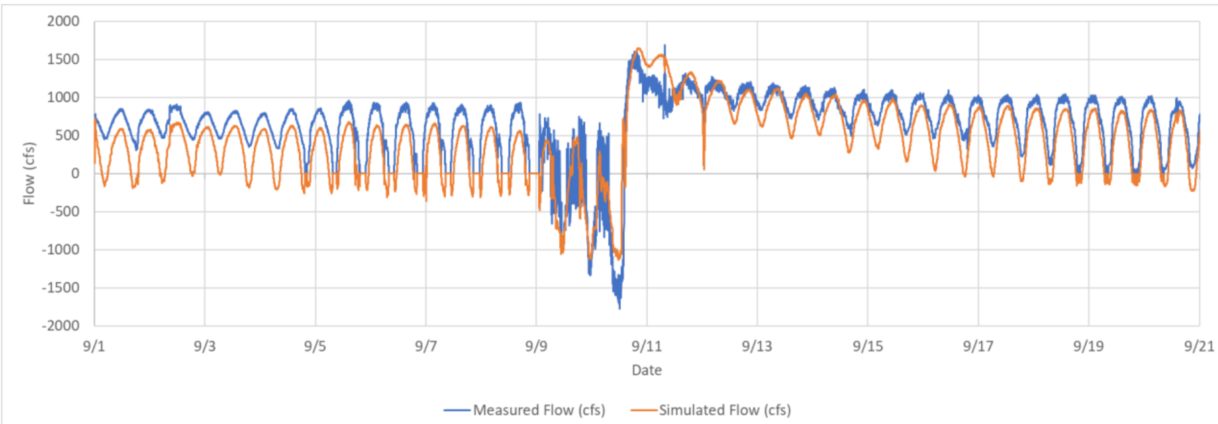


Figure 5-199. Validation Model Canal Stage – S22-H



5.5.8.3. C2 WATERSHED – CANAL FLOW

Figure 5-200. Validation Model Canal Flow – S22-Q



5.5.8.4. C2 WATERSHED – GROUNDWATER STAGE

Figure 5-201. Validation Model Groundwater Stage – G-3558

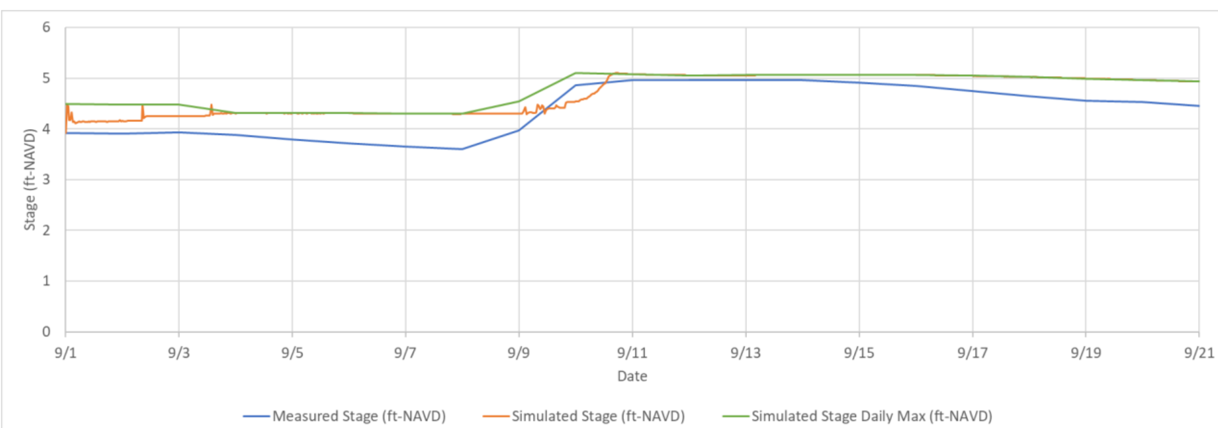


Figure 5-202. Validation Model Groundwater Stage – G-3563

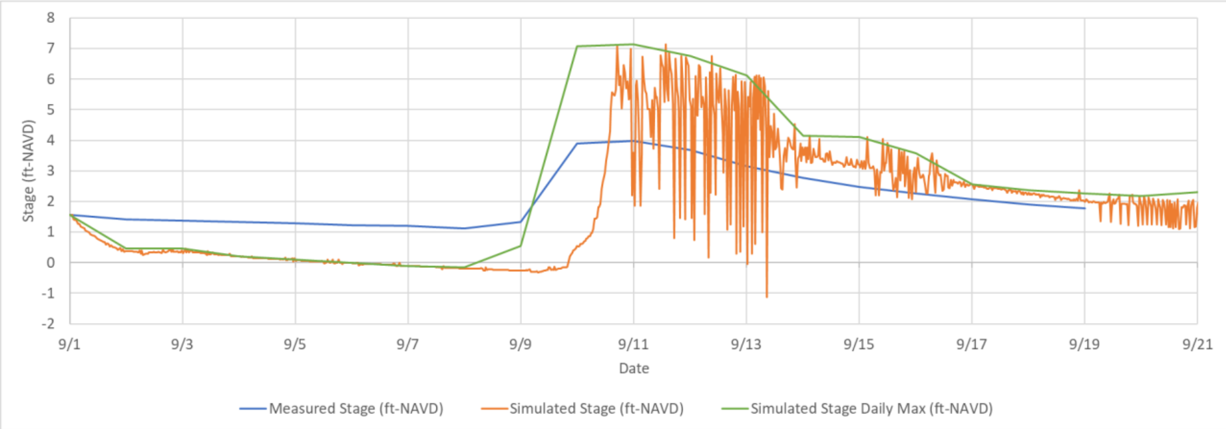


Figure 5-203. Validation Model Groundwater Stage – G-3572

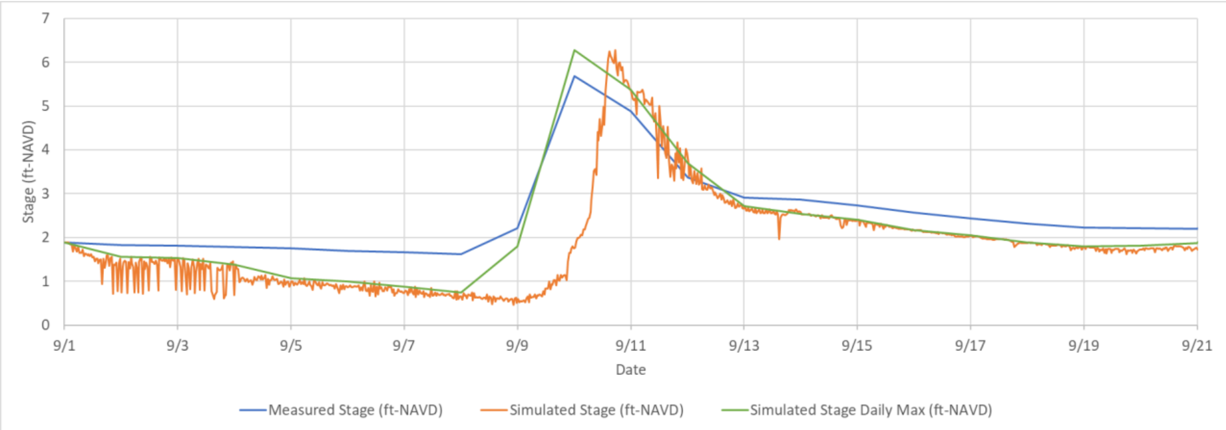


Figure 5-204. Validation Model Groundwater Stage – G-3565

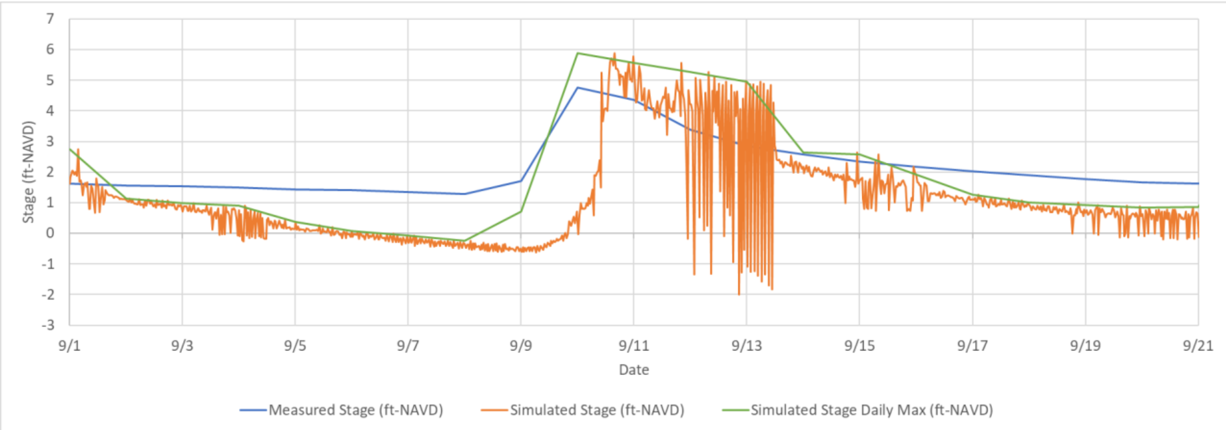


Figure 5-205. Validation Model Groundwater Stage – F-319

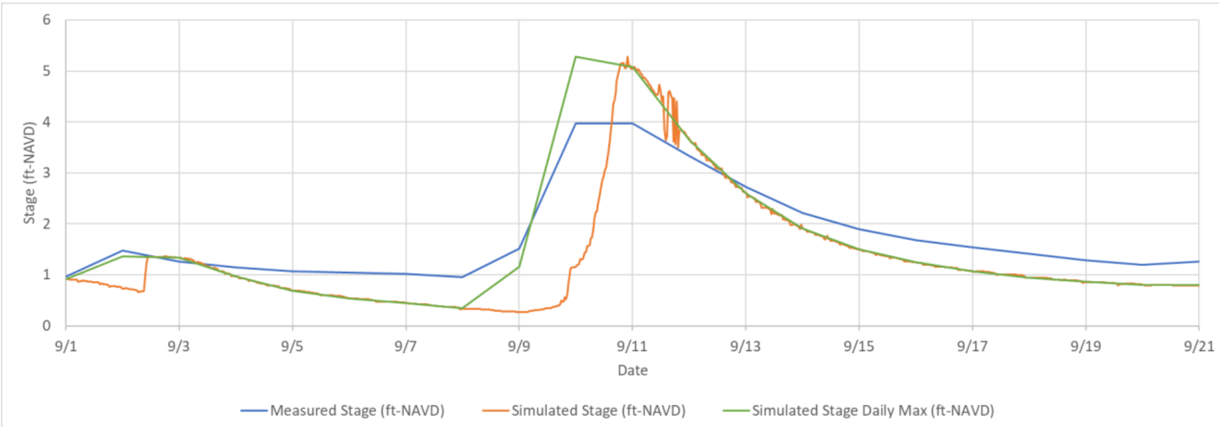


Figure 5-206. Validation Model Groundwater Stage – G-3439

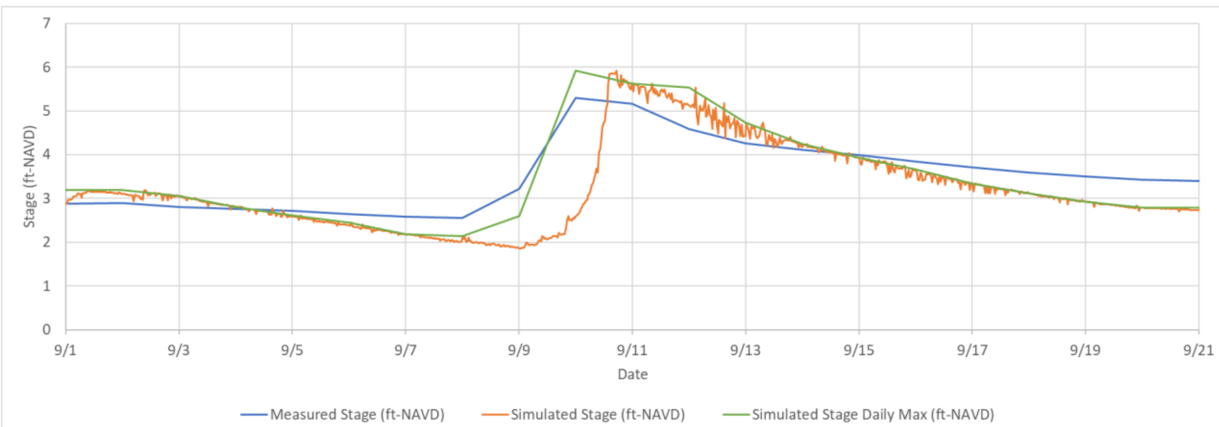


Figure 5-207. Validation Model Groundwater Stage – C2GSW1

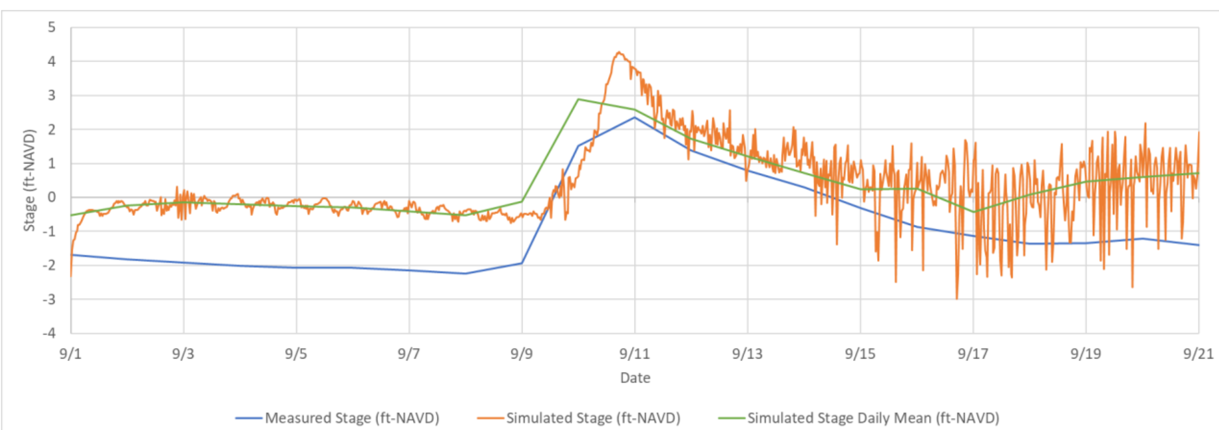


Figure 5-208. Validation Model Groundwater Stage – G-3909

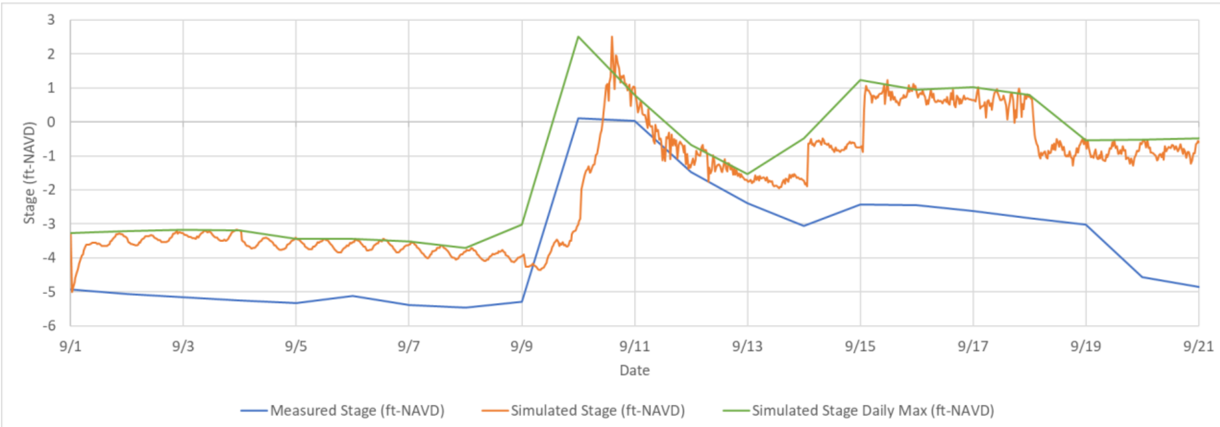


Figure 5-209. Validation Model Groundwater Stage – G-1074B

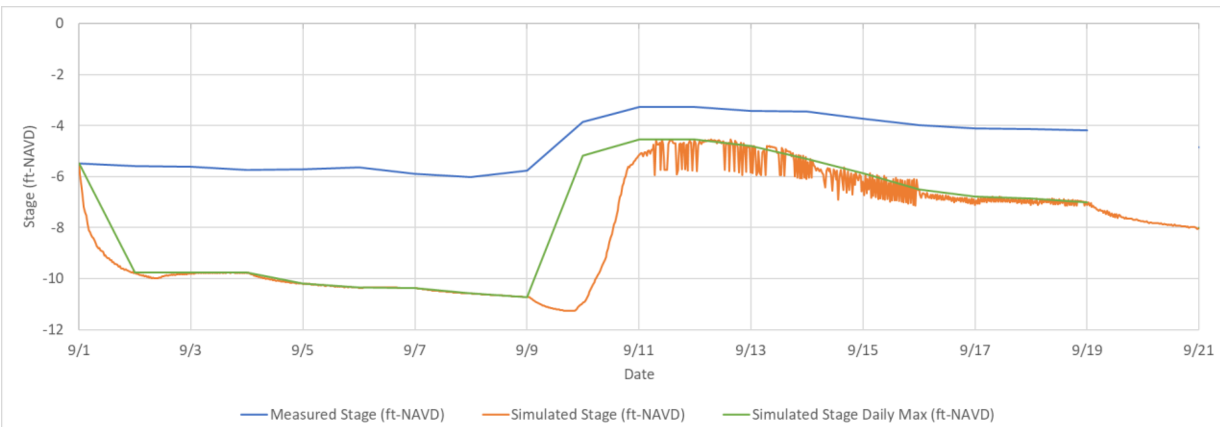
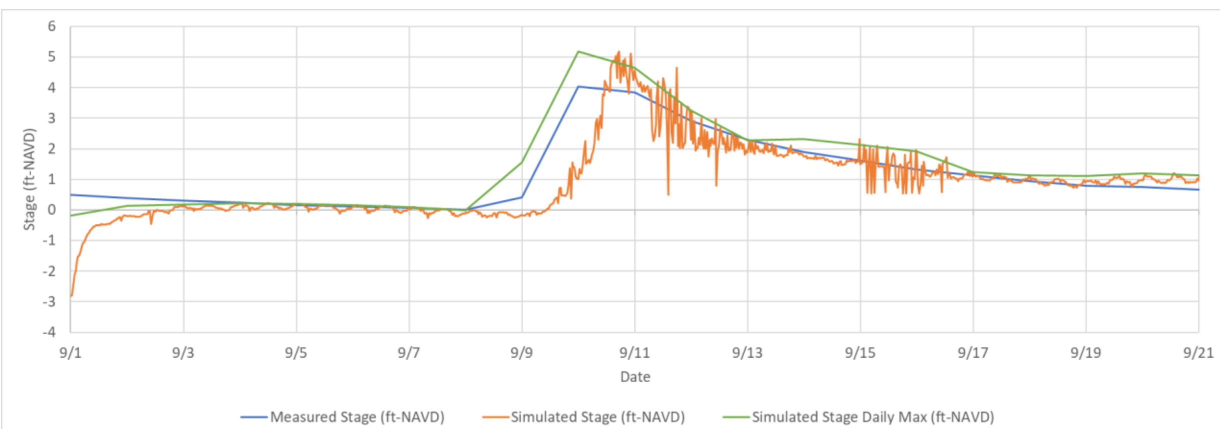
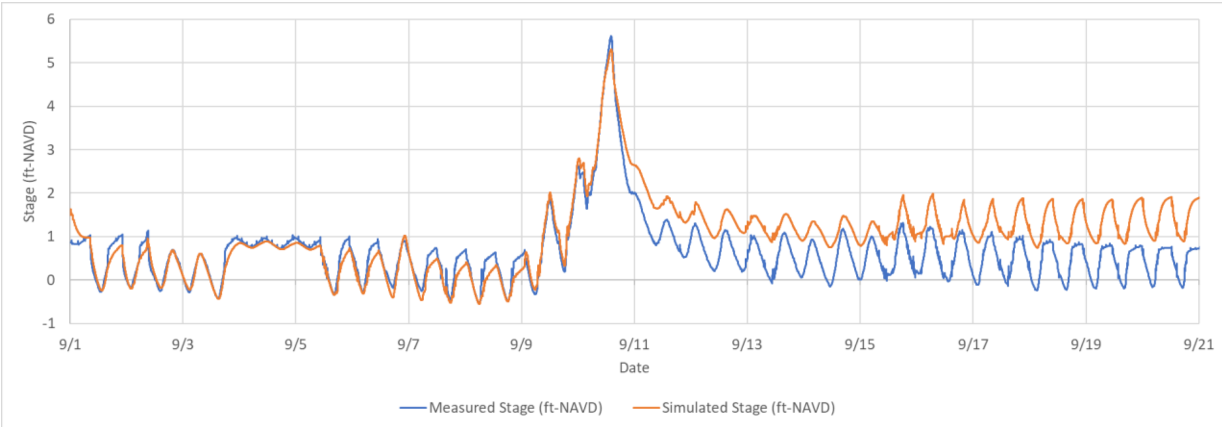


Figure 5-210. Validation Model Groundwater Stage – G-3918



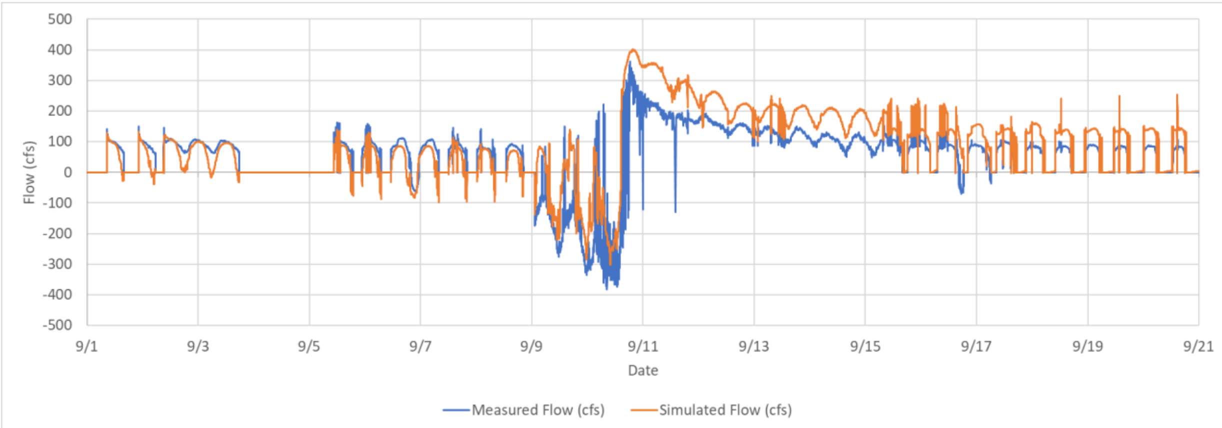
5.5.8.5. C3W WATERSHED – CANAL STAGE

Figure 5-211. Validation Model Canal Stage – G93-H



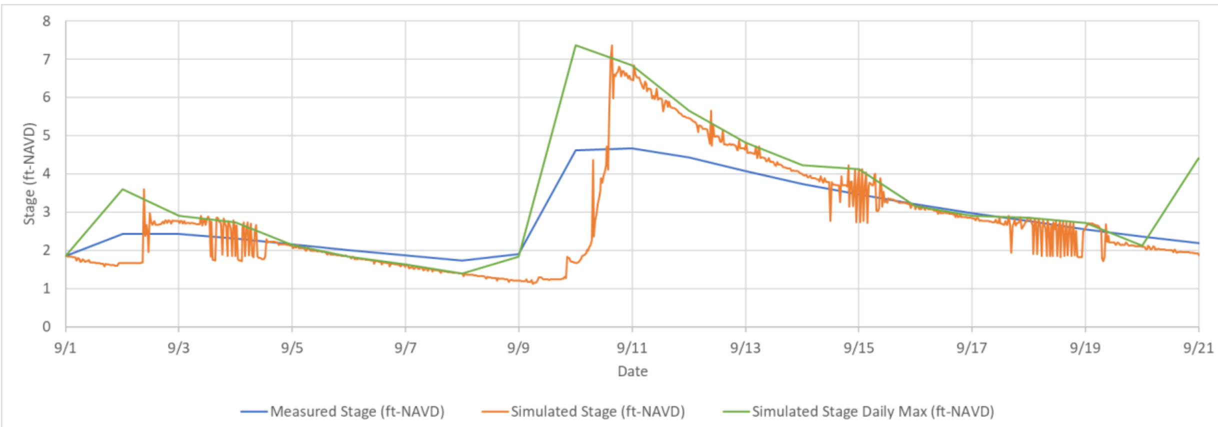
5.5.8.6. C3W WATERSHED – CANAL FLOW

Figure 5-212. Validation Model Canal Flow – G93-Q



5.5.8.7. C3W WATERSHED – GROUNDWATER STAGE

Figure 5-213. Validation Model Groundwater Stage – G-3570



5.5.8.8. C4 WATERSHED – CANAL STAGE

Figure 5-214. Validation Model Canal Stage – S25B-H

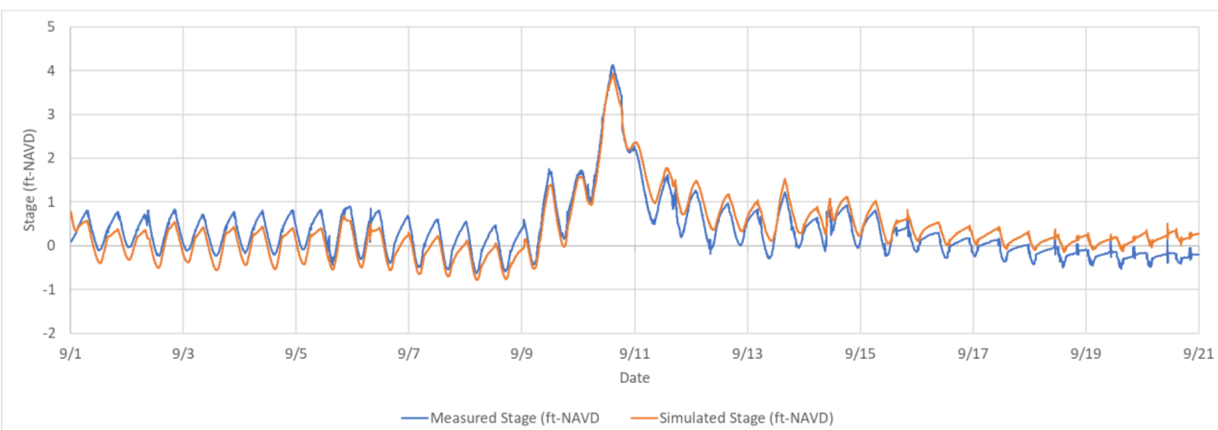


Figure 5-215. Validation Model Canal Stage – S336-T

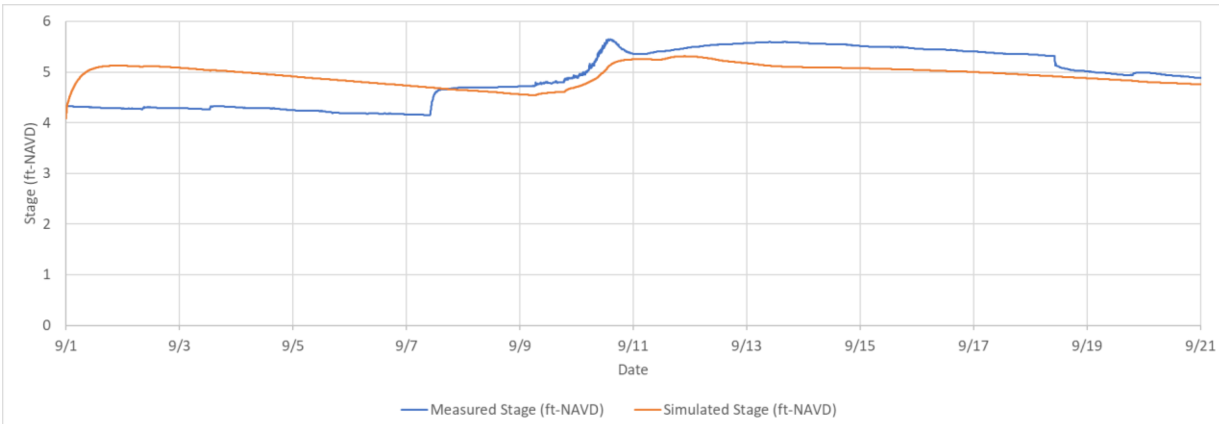


Figure 5-216. Validation Model Canal Stage – S380-H

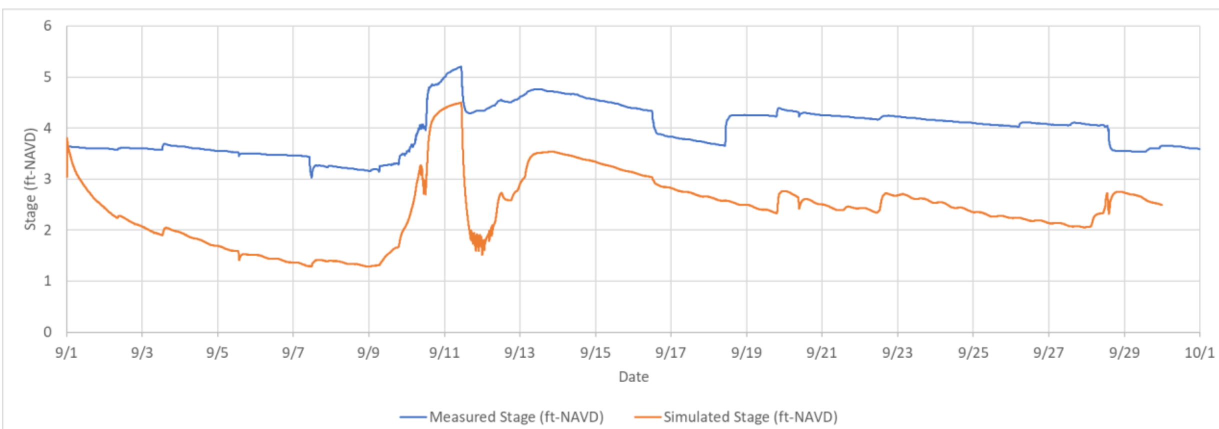


Figure 5-217. Validation Model Canal Stage – S380-T

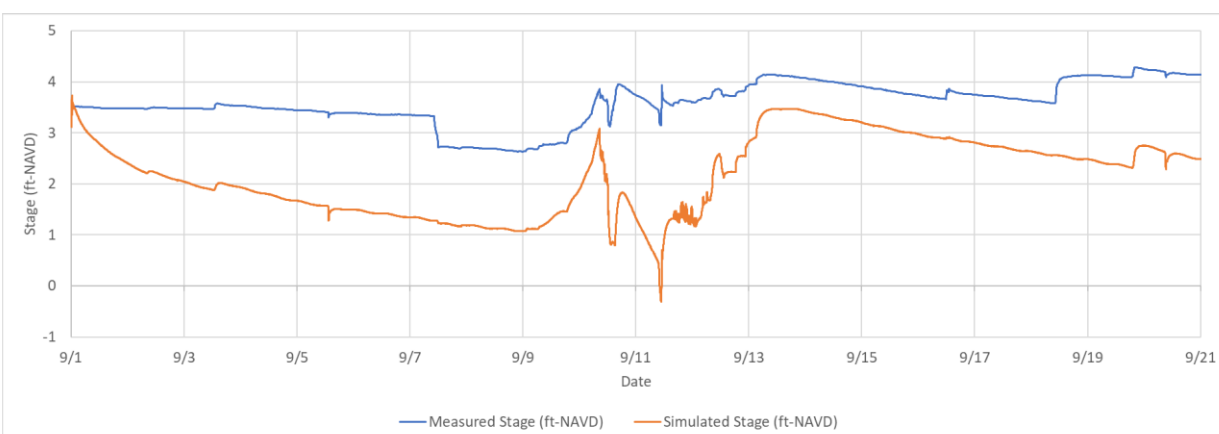


Figure 5-218. Validation Model Canal Stage – T5W

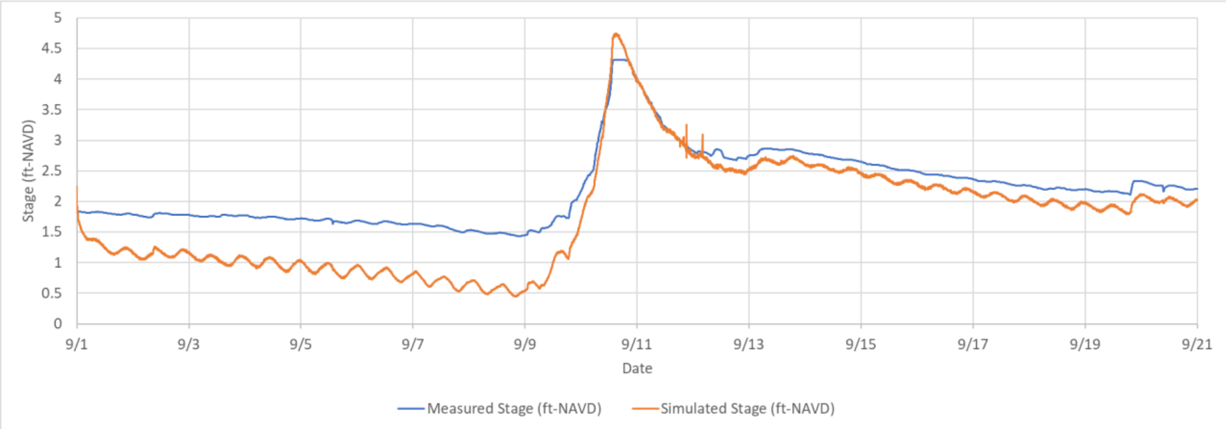


Figure 5-219. Validation Model Canal Stage – USGS 02287497 (NW Wellfield)

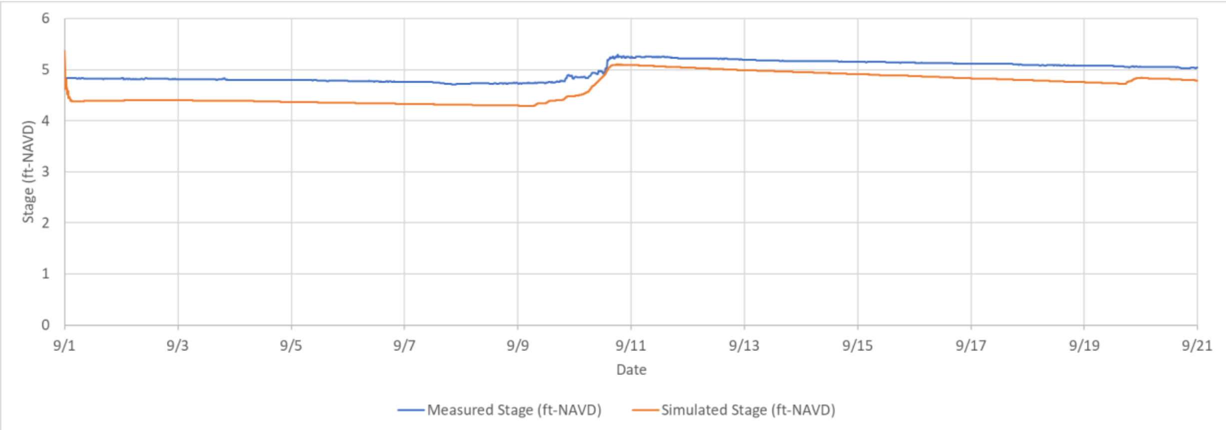
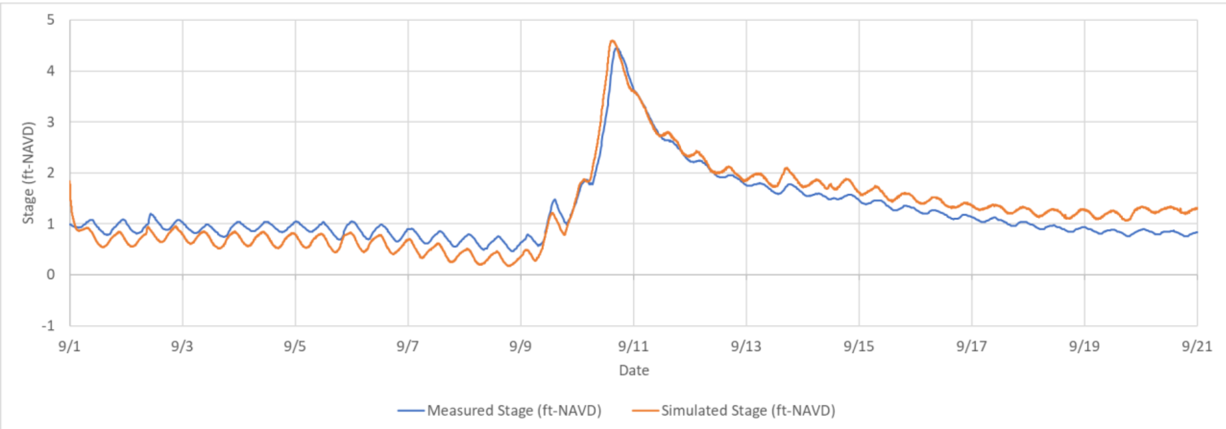


Figure 5-220. Validation Model Canal Stage – USGS 02289500 (C4.Coral)



5.5.8.9. C4 WATERSHED – CANAL FLOW

Figure 5-221. Validation Model Canal Flow – S25B-Q

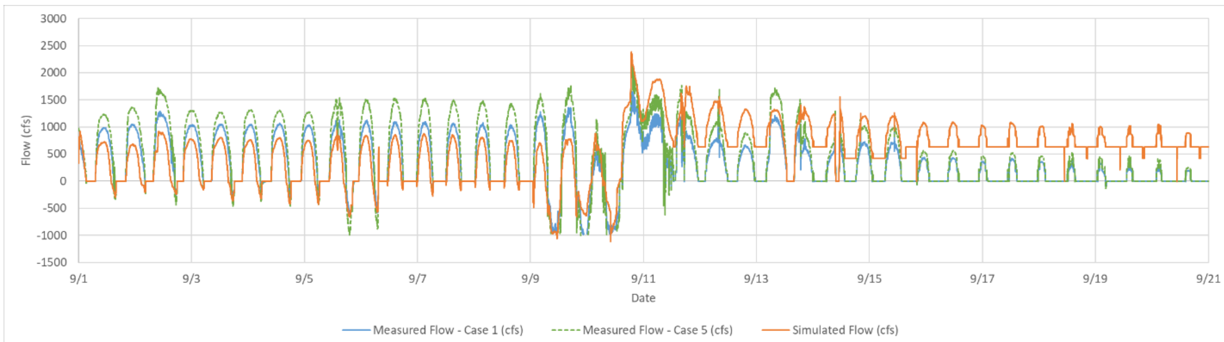


Figure 5-222. Validation Model Canal Flow – USGS 02289500 (C4.Coral)

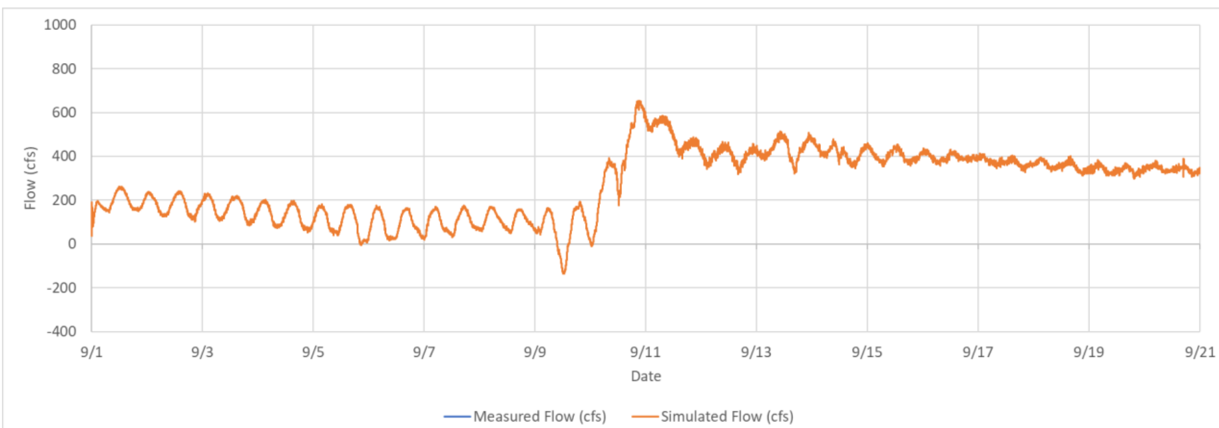


Figure 5-223. Validation Model Canal Flow – USGS 02287497 (NW Wellfield)

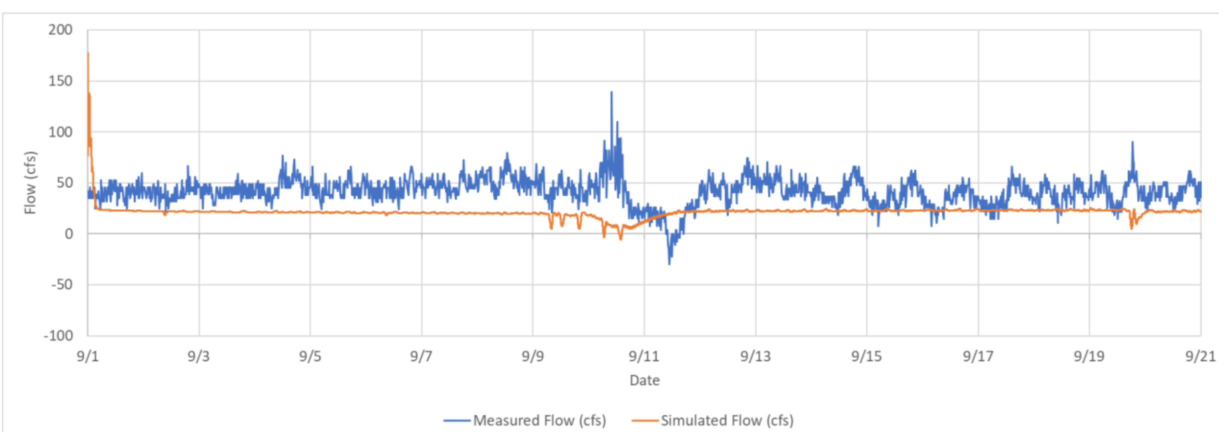


Figure 5-224. Validation Model Canal Flow – S380-Q

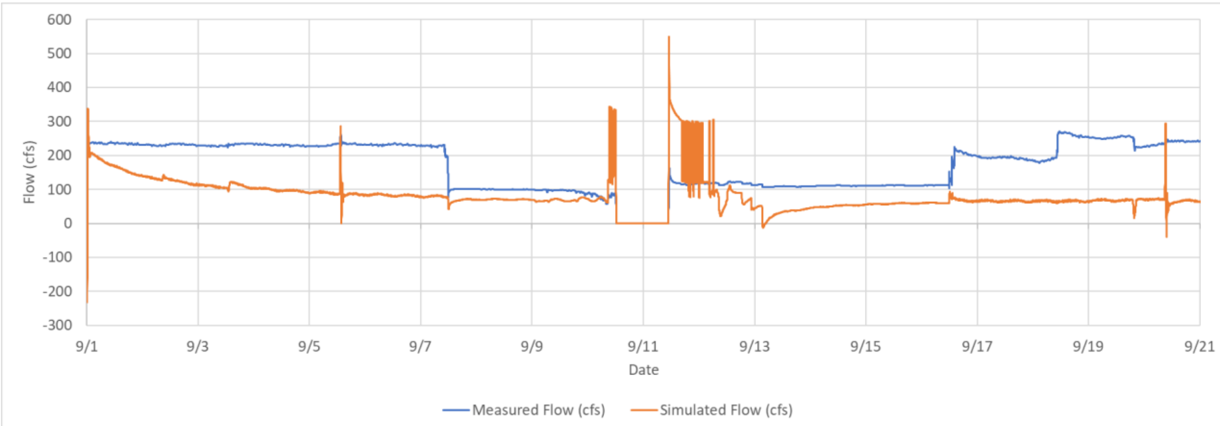
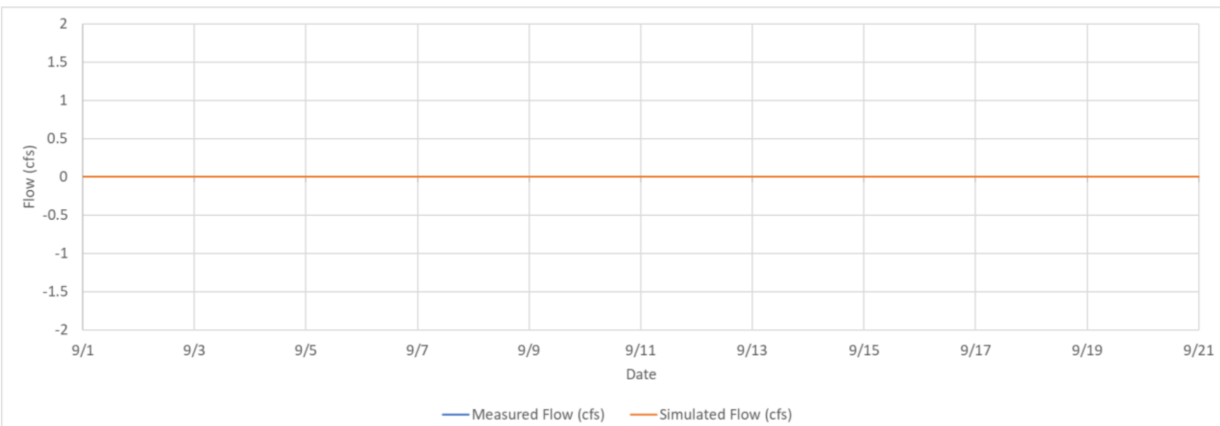


Figure 5-225. Validation Model Canal Flow – S336-Q



5.5.8.10. C4 WATERSHED – GROUNDWATER STAGE

Figure 5-226. Validation Model Groundwater Stage – C4GW1

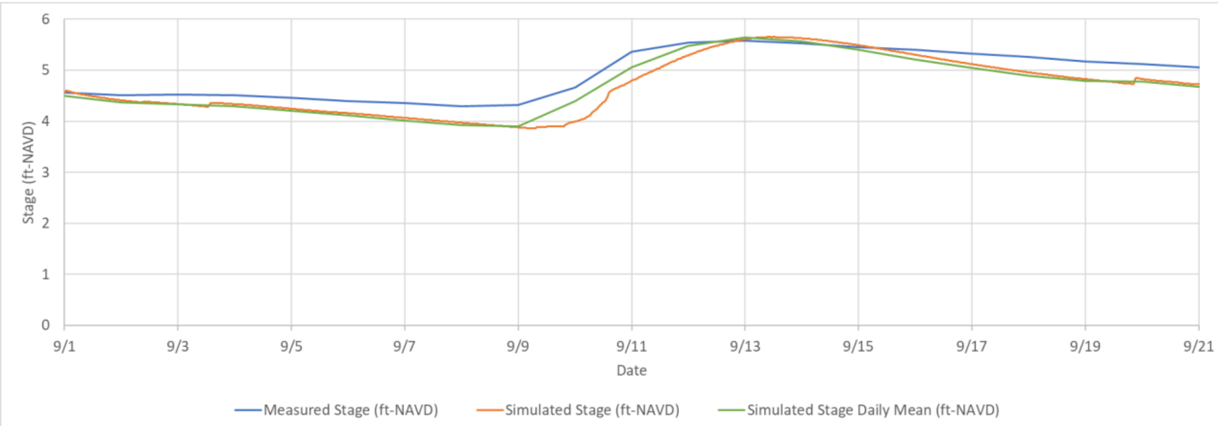


Figure 5-227. Validation Model Groundwater Stage – G-3465

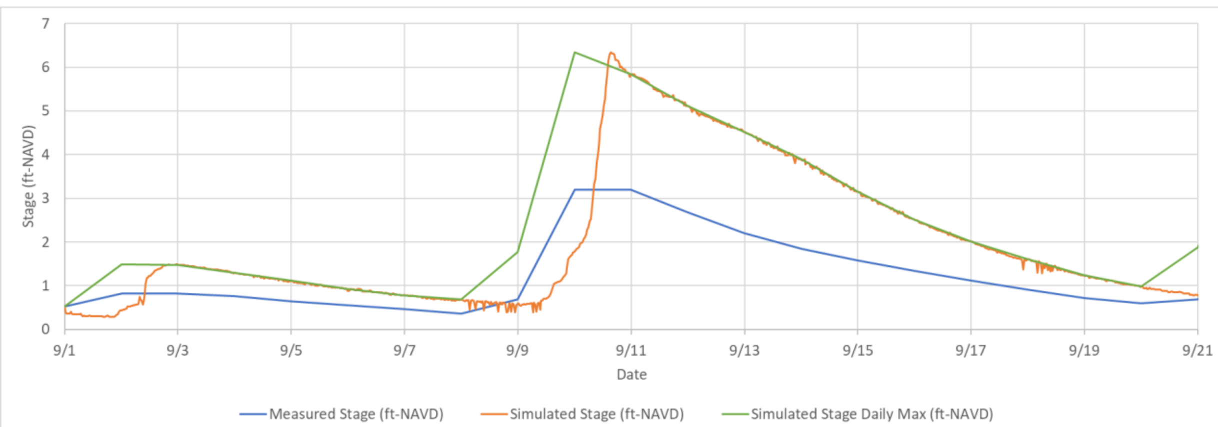


Figure 5-228. Validation Model Groundwater Stage – G-975

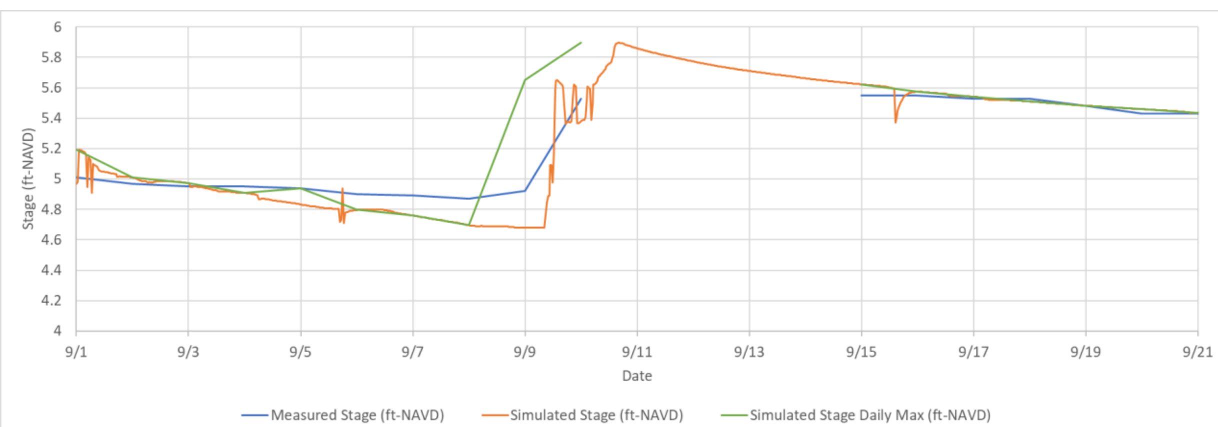


Figure 5-229. Validation Model Groundwater Stage –S356GW1

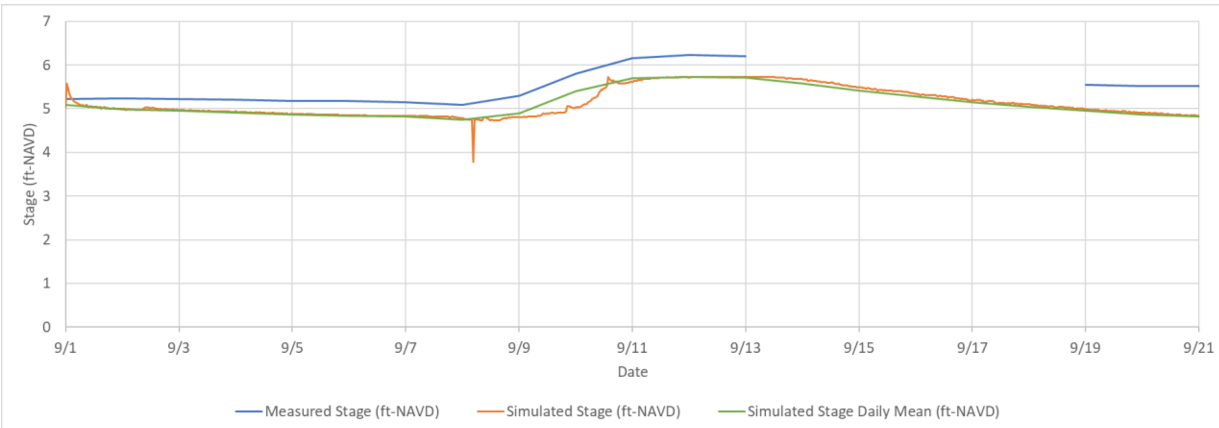
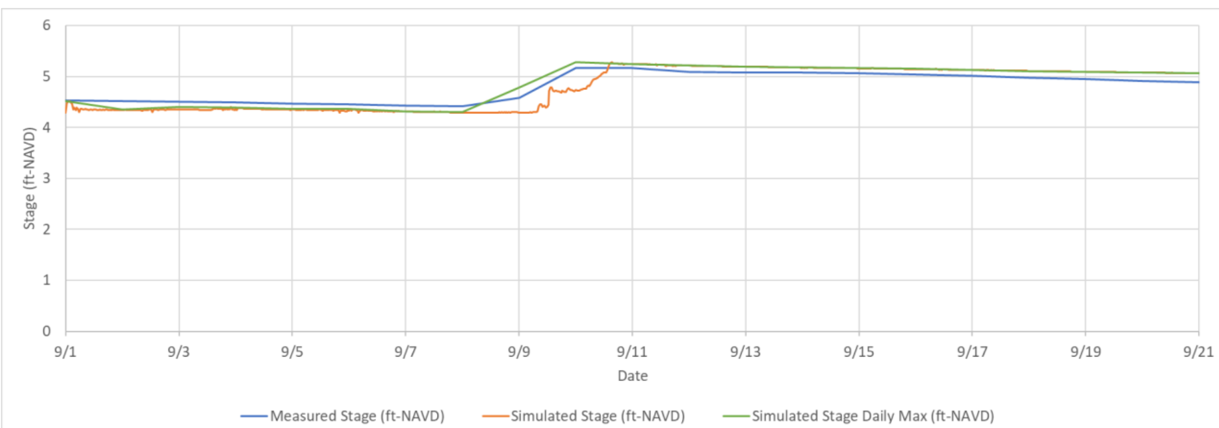
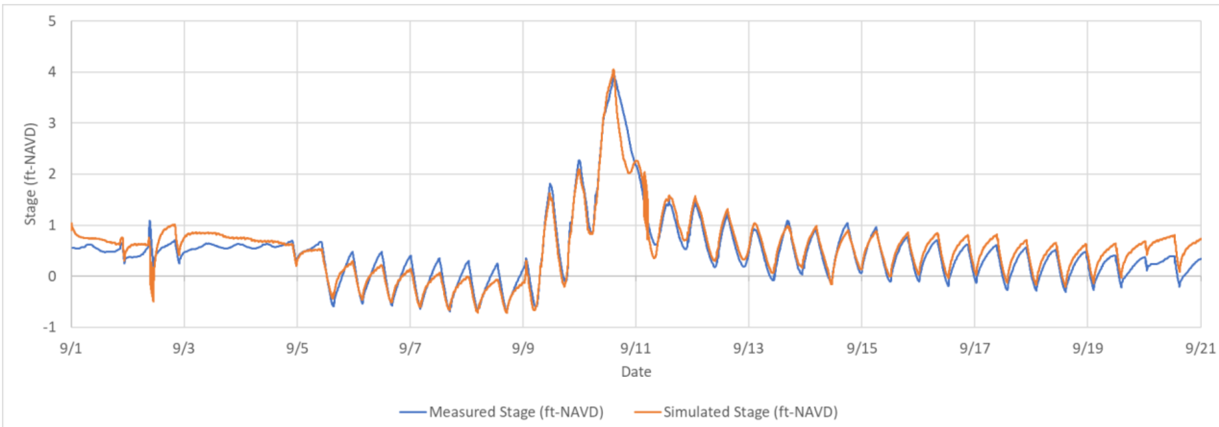


Figure 5-230. Validation Model Groundwater Stage – G-3818



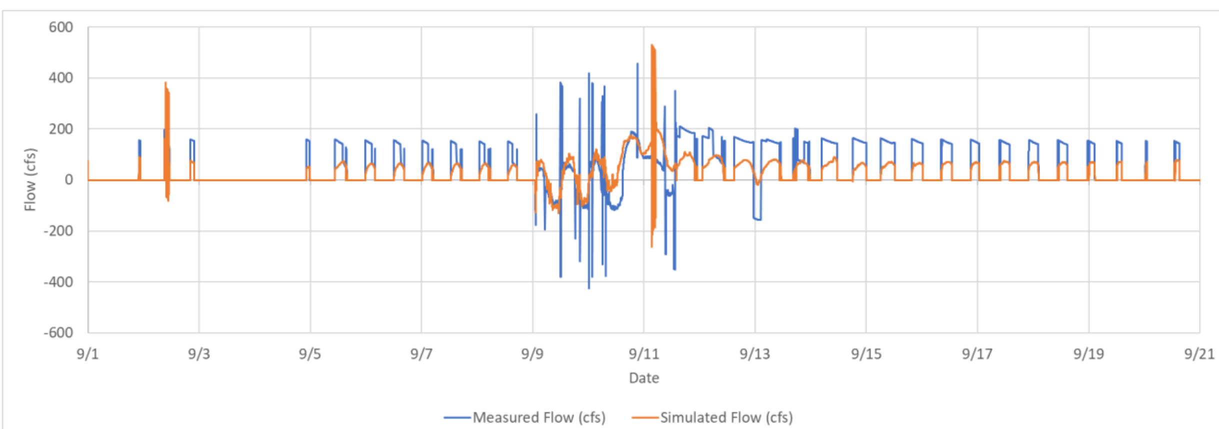
5.5.8.11. C5 WATERSHED – CANAL STAGE

Figure 5-231. Validation Model Canal Stage – S25-H



5.5.8.12. C5 WATERSHED – CANAL FLOW

Figure 5-232. Validation Model Canal Flow – S25-Q



5.5.8.13. C6 WATERSHED – CANAL STAGE

Figure 5-233. Validation Model Canal Stage – S26-H

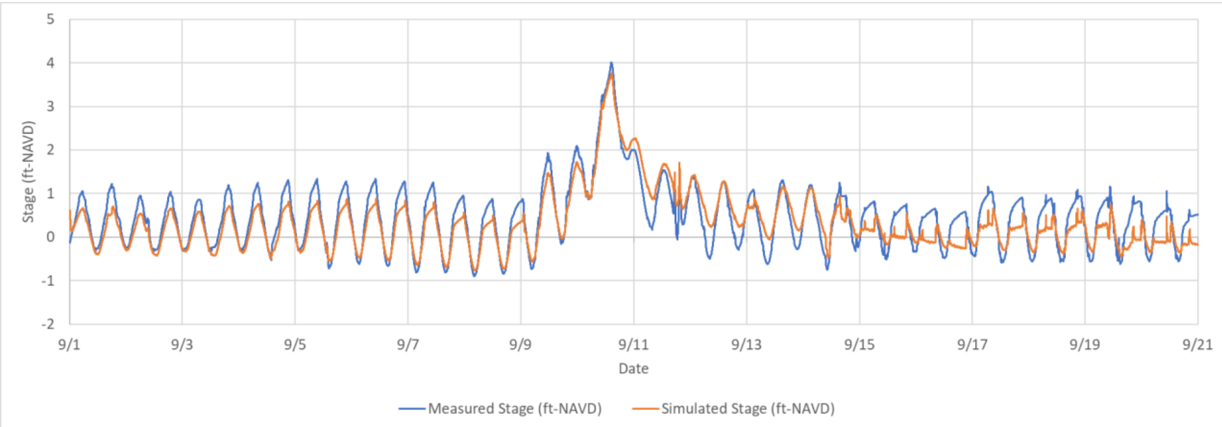
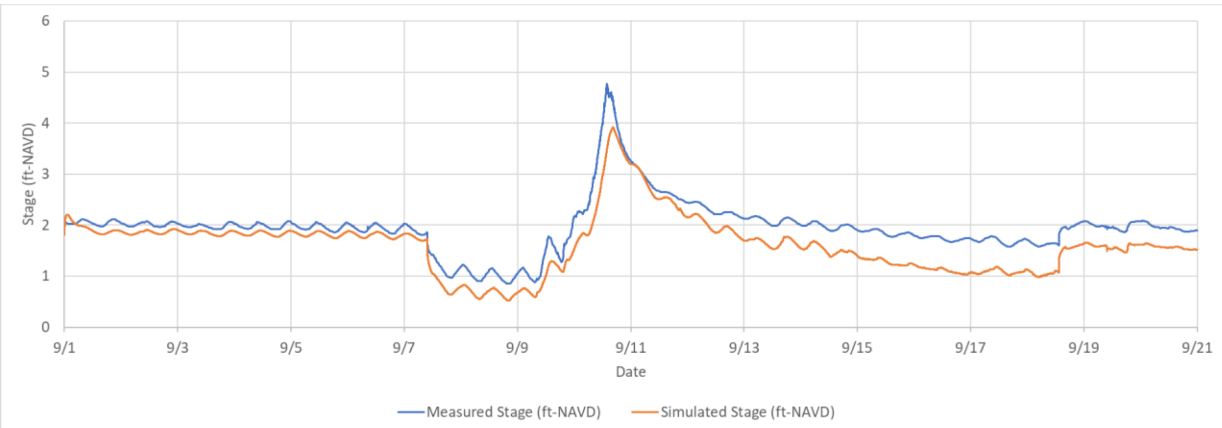


Figure 5-234. Validation Model Canal Stage – S31-T



5.5.8.14. C6 WATERSHED – CANAL FLOW

Figure 5-235. Validation Model Canal Flow – S26-Q

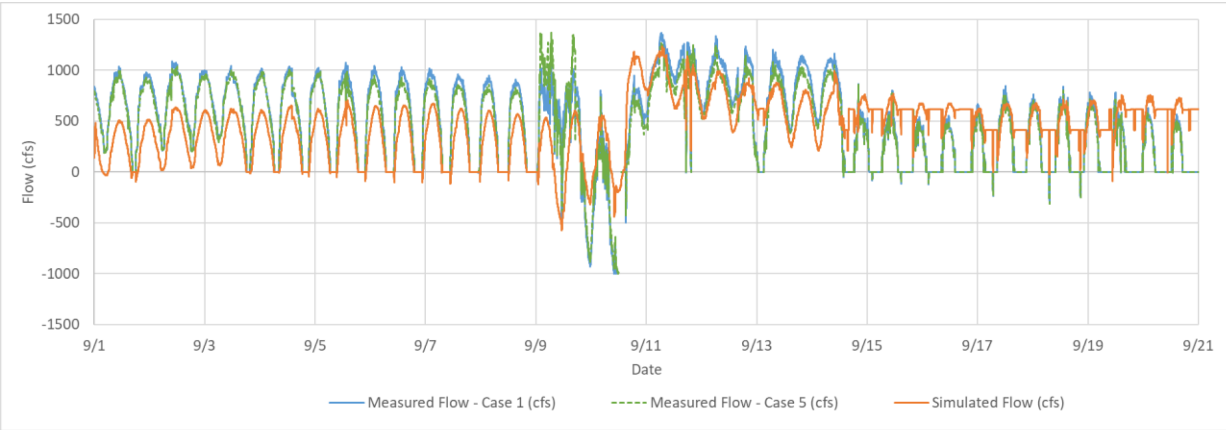
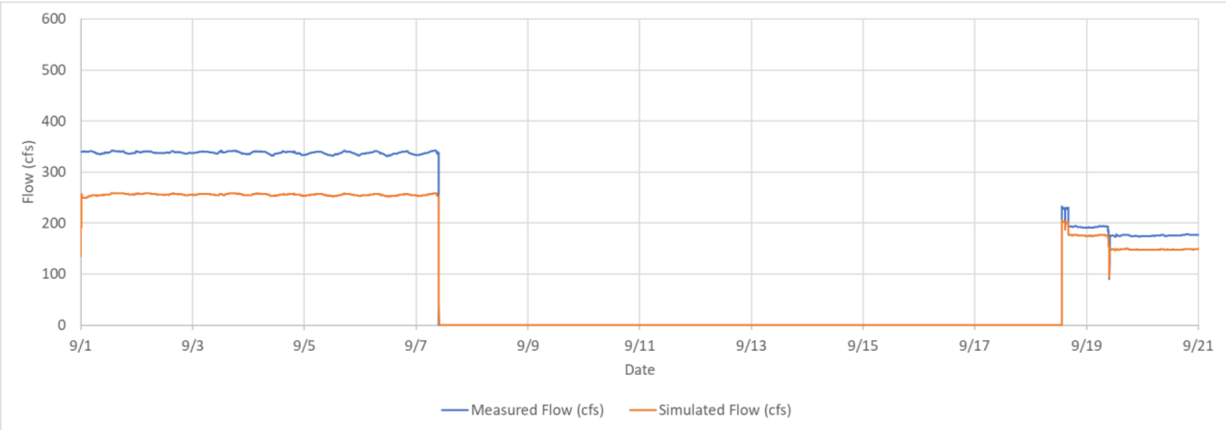


Figure 5-236. Validation Model Canal Flow – S31-Q



5.5.8.15. C6 WATERSHED – GROUNDWATER STAGE

Figure 5-237. Validation Model Groundwater Stage – G-3

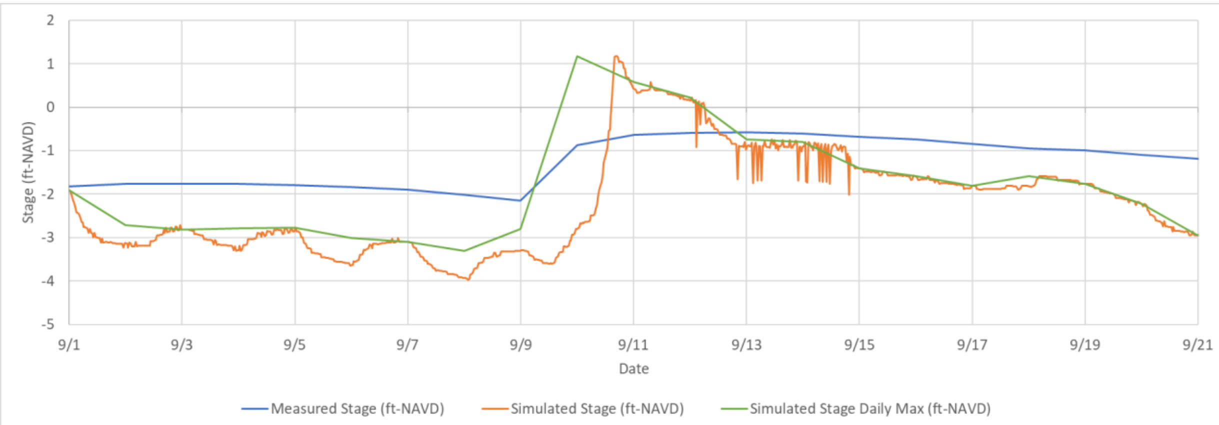


Figure 5-238. Validation Model Groundwater Stage – G-3567

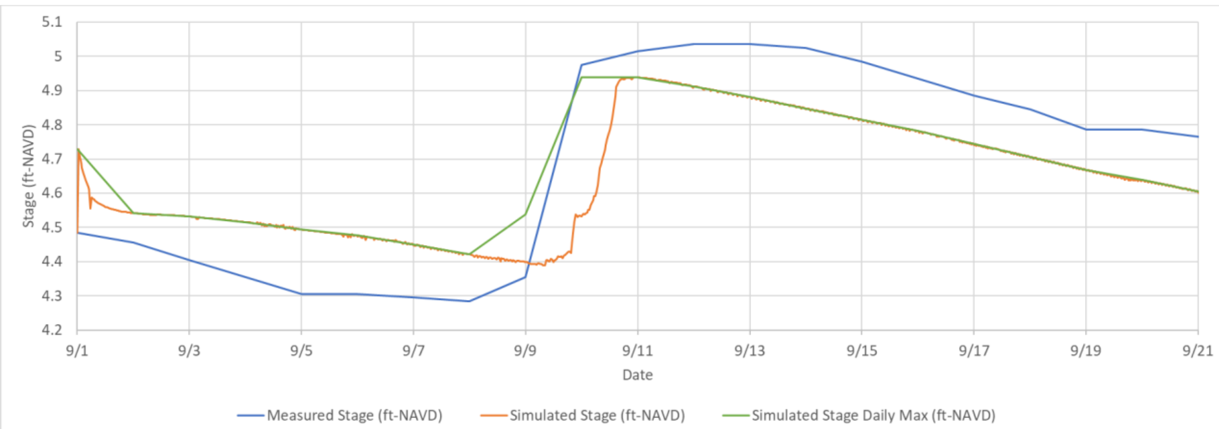


Figure 5-239. Validation Model Groundwater Stage – S-68

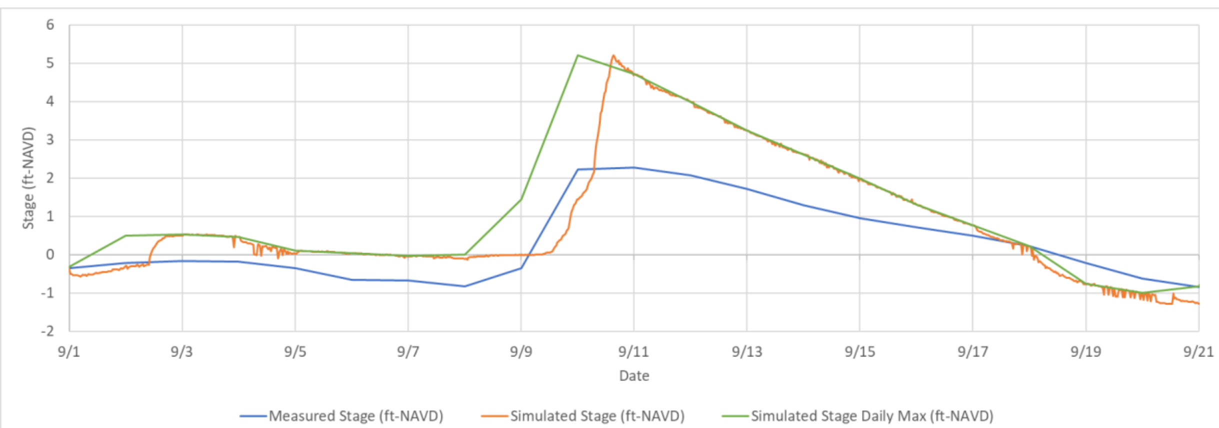
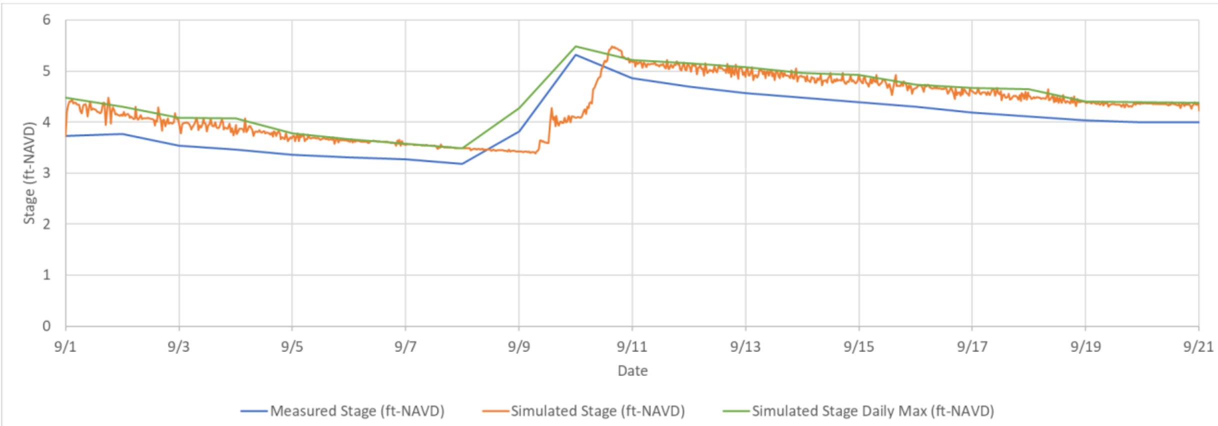


Figure 5-240. Validation Model Groundwater Stage – G-3264AR



5.6. CALIBRATION AND VALIDATION SUMMARY

Developing a model that reflects the hydrologic, hydraulic and groundwater response to rainfall events is critical for performing a Flood Protection Level of Service analysis. The MIKE SHE / MIKE 1D model that was developed for the C2, C3W, C4, C5 and C6 watersheds provided the appropriate tool that was necessary for performing FPLOS assessment on these watersheds. The sensitivity analysis and calibration effort ensure that the model represented the approximate rainfall response of the natural system.

The calibration results demonstrated that at critical locations throughout the model domain, the peak simulated stage was within 0.5 feet of the measured stage. The ability of the calibrated model to accurately reflect groundwater elevations varied throughout the model domain and was generally not as accurate as the simulated surface water stages. During the sensitivity analysis several modifications were made to identify alternative parameterizations that would improve simulation results, however there were no modifications that improved results in all locations. Localized changes to model parameters were not made to improve model performance at individual monitoring locations in consideration that there was no data available to support localized changes.

The validation results demonstrate that the calibrated parameterization for the May 2020 rainfall event did not reflect the natural system response within the model as accurately for the September 2017 rainfall event but was still generally within acceptable tolerances for the hydrologic characteristics of stage and flow. However, at most groundwater monitoring locations the error exceeded the 0.5 feet tolerance. Improvements in the validation model could have been realized by modifying parameters in the calibration configuration or locally within the model, however the intent of the validation simulation was to demonstrate the model's ability to recreate the natural system not for additional calibration efforts.

In the subsequent sections, this report documents how the calibrated model was modified and used to determine the FPLOS of the C2, C3W, C4, C5 and C6 watersheds for the existing and future sea level rise conditions. The relative accuracy of the calibrated model in representing peak canal stages is the most critical component to evaluate the flood protection provided by existing infrastructure under various rainfall and tailwater conditions. Based on the results shown in this report, the calibrated and validated model can be used to evaluate the FPLOS, especially in consideration of the performance of the simulation for canal stages.

6. FLOOD PROTECTION LEVEL OF SERVICE PERFORMANCE METRICS

After developing, calibrating, and validating the MIKE SHE/MIKE 1D model for the C2, C3W, C4, C5, and C6 Watersheds, the model was used to run design storm simulations for the 3-day 100-year, 25-year, 10-year and 5-year rainfall events with current sea level conditions and then compared to the same design storm simulations with future sea level conditions of +1 foot, +2 feet, and +3 feet of sea level rise (referred to as simulations SLR1, SLR2, and SLR3, respectively). To evaluate the impacts of future SLR, the results were analyzed using FPLOS performance metrics in a similar manner as evaluated by the District in other watersheds.

The District has developed six (6) FPLOS Performance Metrics (PM) to quantify the level of flood protection provided within a watershed now and in the future. Four of these measures (PM #1 through #4) assess the performance of the regional drainage systems. PM #5 and #6 assess impacts to local flooding frequency and duration within the communities that these drainage systems serve.

- **PM #1: Maximum stage in primary canals.** Determines the peak stage profile along the primary canal system for design, existing, and future conditions. If stages are above design levels, the ability to drain the adjacent communities is reduced. Further, if water levels exceed the height of the canal banks, overbank flow could cause localized flooding.
- **PM #2: Maximum daily discharge capacity through the primary canals.** Determines the flow capacity of the primary canal systems throughout the District for design, existing, and future conditions. The flow capacity of the canal reaches are assessed with respect to the capacity yielding no adverse impacts.
- **PM #3: Tidal structure flow performance – effects of sea level rise.** Determines the effect of sea level rise on the discharge capacity of tidal structures. This evaluation is based on structure design features and existing operational protocols.
- **PM #4: Peak storm runoff - maximum conveyance capacity of the watershed.** Determines the flows passing at the downstream structure under future conditions. Future scenarios consider the combined effect of storm events, sea level rise, and storm surge averaged over the tidal cycle (for tidal structures).
- **PM #5: Frequency of flooding – stage- based LOS for sub-watersheds.** Determines the overall ability of the water management infrastructure to maintain non-flooding water levels within communities. This PM considers the upper limits of flood stages and depths of water needed to protect the local infrastructure such as homes, commercial buildings, and major roads.

- **PM #6 Duration of flooding – effects of sea level rise.** Determines the time required for water levels to recede to non-flood stages for storm events under future conditions.

In the sections that follow, a detailed evaluation of the PMs is provided for each of the studied watersheds (C2, C3W, C4, C5 and C6).

6.1. NOTES ON RESULTS TIME STEPS AND PROCESSING

This report provides modeling analysis of both 1D channel hydraulics and 2D overland inundation, which are linked numerically during the simulation, but which produce different results outputs at different time scales. The MIKE 1D channel hydraulics output channel flow and stage results every one minute and the MIKE SHE 2D overland depth maps are output every hour. Therefore, there are some differences in how the results should and must be evaluated for the various PMs.

1D results are used to evaluate PM #1 through PM #4, looking at stages and flows in the channels throughout the watershed. However, the coastal structures S26, S25, S25B, G93, S22, etc. experience the effects of tides, which may translate upstream if the gates are open or if structure overtopping is occurring. In addition, gate level changes may cause initial spikes in flows at the moment of operation, which may produce peak flows that are not realistic for the storm event, for example when the 5-year storm peak flow is higher than the 10-year and 25-year storm this does not seem consistent with the total runoff volumes that are expected.

To remove the effects of the tides, and to smooth out sharp peaks caused by a quick gate opening, a 12-hour moving average was implemented for the results timeseries, which can improve the evaluation methods used in PM#2, PM#3, and PM#4. The 12-hour moving average was calculated using an equal number of data on either side of a central value. Given that the output timestep for the 1D results is 1 minute, there are 720 minutes in 12 hours, and the following equation was used to calculate the 12-hour moving average at each timestep ***i***:

$$12\ Hour\ Moving\ Average_i = \frac{1}{720}\sum_{i-\frac{720}{2}}^{i+\frac{720}{2}} Results_i$$

2D Overland Depth Maps were used to evaluate PM #5 and PM #6, looking at the maximum depth and duration of flooding. For the purposes of this analysis overland flooding below three (3) inches of depth is considered nuisance flooding. These depth maps are output hourly; however, the model timestep for the overland module is a maximum of 6 minutes and therefore the first hour of flooding may be anything from 1 minute to 1 hour of flooding. For the purposes of this analysis, the first hour of flooding was considered nuisance flooding.

In addition, it should be noted that the models are run over the simulation period of 10/13/1999 through 10/31/1999; however, these dates are arbitrary and do not represent actual conditions on these dates. This report will provide dates in the month and day (MM/DD) format in order to understand duration from the starting time of 10/13.

7. CURRENT CONDITIONS DESIGN STORM SETUP

This section describes the model setup for the existing conditions scenario in detail including a thorough explanation of the simulation period, design storm rainfall, the initial conditions, the boundary conditions, the public water supply pumping withdrawals, and the water control structure operations. This section also documents the changes to the 1D model network that were implemented to force the tidal conditions downstream of the tidal structures S26, S25B, S25, G93, and S22.

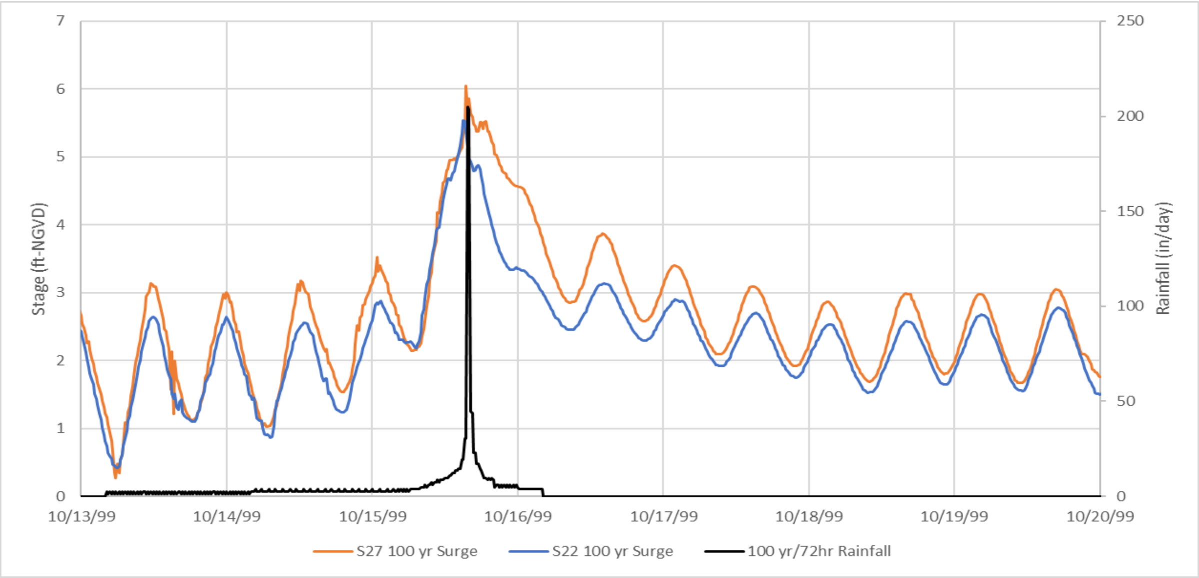
On November 22, 2022, CMA submitted a Technical Memorandum regarding the Mike 1D boundaries downstream of the tidal structures. A sensitivity test was performed to determine the effects of forcing the tidal boundary at the tailwater of the tidal structures, the results of which showed a greater impact of flooding and structure overtopping for the S26 and G93 structures. It was determined that the design storm simulations would then implement a forced tidal condition directly downstream of the tidal structures, based on the surge prescribed by the District. To apply this boundary, the tidal portions of the C6_Canal, C4_Canal, Comfort_Canal_Southfork, Coral_Gables_Canal, and Snapper_Creek_Canal were split from the upstream portions of the canals, with the naming convention "_Tidal" added to the end of the names.

7.1. DESIGN STORM SIMULATION PERIOD

The 72-hour design storm event was established using the October 1999 Hurricane Irene storm surge event. The simulation period is 10/13/1999 to 10/31/1999, with the peak of the rainfall intensity and surge stage occurring on the 15th of October.

The timing of the rainfall event was established to reach peak at the same time as the peak of the storm surge event (as provided by the District for S22_T and S27_T), at about 10/15/99 at 3 PM. **Figure 7-1** shows the storm surge for S27 and S22 during the October 1999 event with the design rainfall established such that the peak of the rain event occurs at roughly the same time as the peak of the tidal surge. The timesteps before and after the 72-hour design event are considered to have zero rainfall. This data was input into the model using a time-varying *dfs2* file. However, the assumptions of coinciding peak rainfall and peak surge along with the same spatial distribution of rainfall being applied over the entire area, makes this analysis very conservative.

Figure 7-1. Rainfall and storm surge comparison.



7.2. INITIAL CONDITIONS

The initial conditions for the model scenarios of existing condition design storms should represent realistic, conservative conditions for high surface water and groundwater conditions. The subsections below describe the selected initial conditions configuration for the model.

7.2.1. ANTECEDENT CONDITIONS FOR SURFACE WATER

For the existing conditions scenarios, the conservative assumption for the initial condition in a flood protection model is a high-water table and wet antecedent condition. These types of environmental conditions are typically present at the end of a wet season in the month of October. The validation simulation period represents a large rainfall event in September 2017. The design storm events use the same initial conditions for surface water as the September 2017 event, which uses measured data from September 1, 2017.

7.2.2. ANTECEDENT CONDITIONS FOR GROUNDWATER

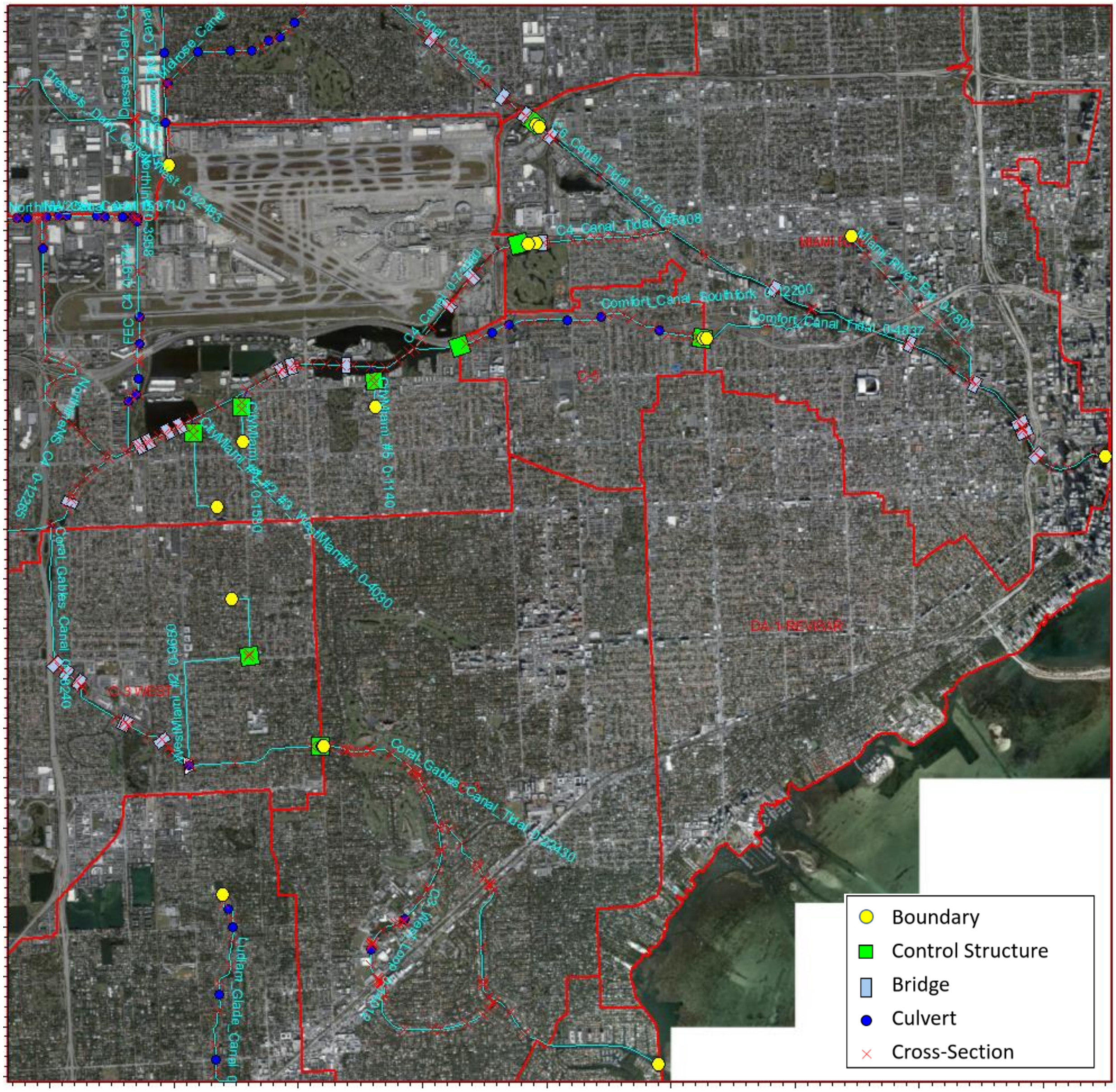
The antecedent conditions for groundwater were established as the seasonal high water table. The USGS Seasonal High-Water Table (USGS, 2016) was used as the initial conditions for groundwater. The USGS data was processed by digitizing contours from the reference map and rasterizing into a format matching the computation grid for the MIKE SHE model of the Saturated Zone. This data was also used as groundwater boundary, as discussed in **Section 7.4.3**.

7.3. MODEL NETWORK

To match the methodology that has been implemented for other FPLOS studies, boundary conditions were forced just downstream of the tidal structures S26, S25B, S25, G93, and S22. To do this, the C6_Canal, C4_Canal, Comfort_Canal_Southfork, Coral_Gables_Canal, and Snapper_Creek_Canal branches were cut into two branches, just downstream of the tidal structure locations. The new downstream branches were named with the suffix _Tidal. All canal cross-sections, connections, structures, and MIKE SHE links were updated with the new _Tidal name and appropriate new location along the branch. In addition, the S25B_P and S26_P reference to MRMS1 in the C6_Canal was updated to match the C6_Canal_Tidal branch name and new location.

The boundary conditions were then added at the tailwater locations for S26, S25B, S25, G93, and S22 structures. These are consistent with the Irene boundary conditions provided by the District. The upstream boundary for Tidal canals were also set to the structure tailwater and downstream boundary was set to the original tidal boundary applied in the model. **Figure 7-2** shows the updated branch and boundary condition setup in the MIKE 1D network.

Figure 7-2. MIKE 1D Network Changes for Design Events



7.4. BOUNDARY CONDITIONS

The following section provides the detailed boundary condition inputs for the existing condition, consistent with the Current Sea Level in 2019 (CSL 2019).

7.4.1. TIDAL BOUNDARIES

To provide a conservative assumption for tailwater (TW) conditions as part of the FPLOS analysis, the tidal boundary stages increased during the event in a manner consistent with storm surge that often occurs concurrently with tropical storm activity. For the model domain, there are five (5) salinity structures where tidal boundary conditions were developed by the District for each design storm:

1. S-26 for the C6 Canal,
2. S-25B for the C4 Canal,
3. S-25 for the C5 Canal,
4. G93 for the C3 Canal, and
5. S-22 for the C2 Canal.

The stage hydrographs representing the tidal boundaries were developed by District staff based on the shape of the observed storm surge during Hurricane Irene in October 1999. The boundaries for each of the structures listed above are shown in **Figure 7-3** through **Figure 7-7**.

Figure 7-3. Design Storm Tailwater Conditions at S26

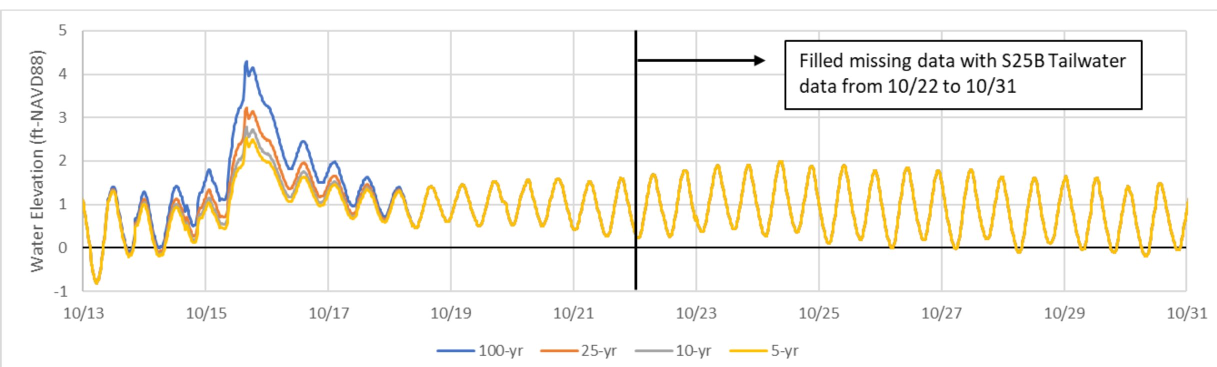


Figure 7-4. Design Storm Tailwater Conditions at S25B

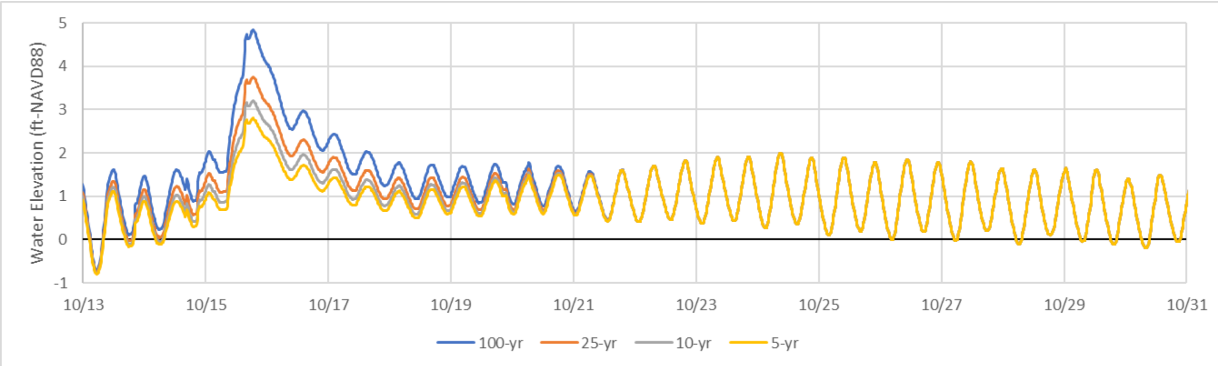


Figure 7-5. Design Storm Tailwater Conditions at S25

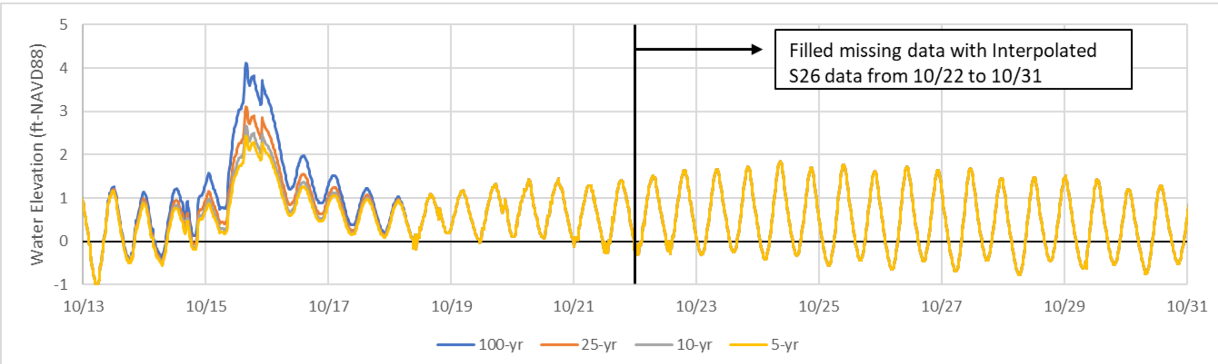


Figure 7-6. Design Storm Tailwater Conditions at G93

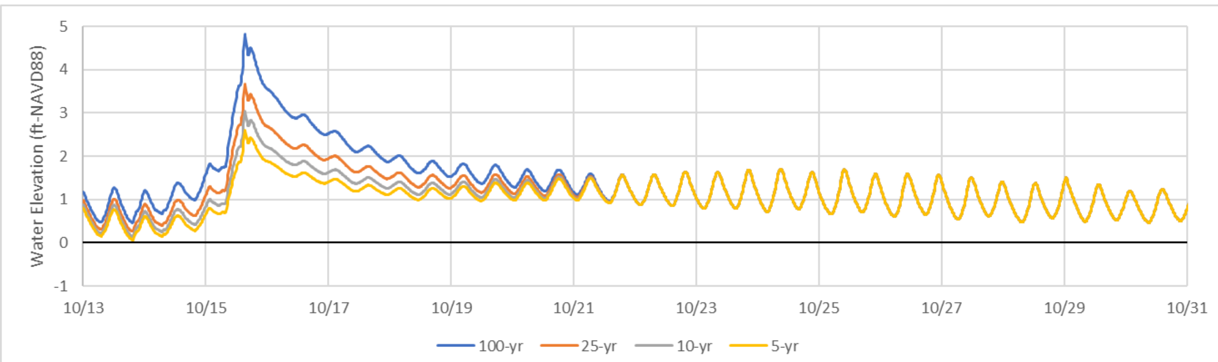
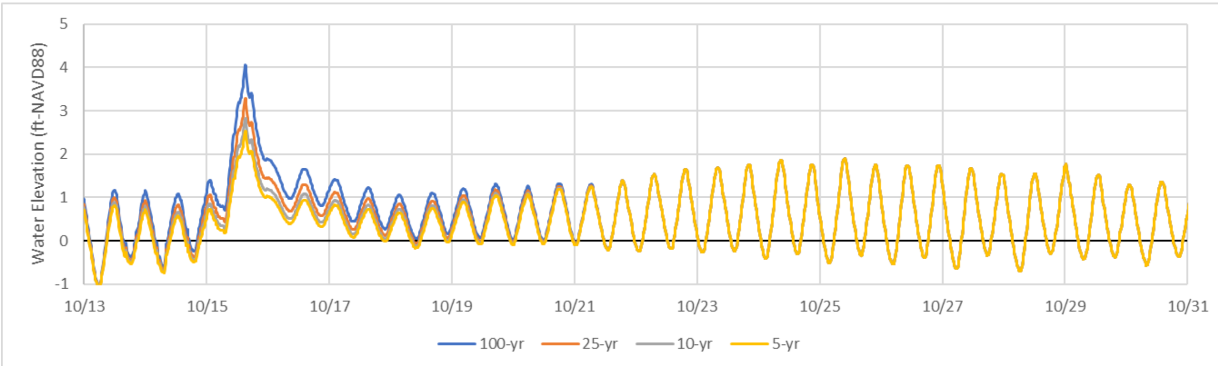


Figure 7-7. Design Storm Tailwater Conditions at S22



7.4.2. DOWNSTREAM TIDAL BOUNDARIES

The downstream boundaries of the C6_Canal_Tidal and Coral_Gables_Canal_Tidal, are far from Biscayne Bay, with the S-25B and S-26 structures located over five (5) miles upstream of the mouth of the Miami River. To develop an appropriate coastal boundary, while accounting for the distance between the TW condition and the model boundary, an interpolation approach was used to estimate the coastal boundary. CMA and the District reviewed three methods for predicting the tidal boundary at Biscayne Bay, as described in the sections below.

7.4.2.1. BOUNDARIES FROM APPROXIMATED DOWNSTREAM HEAD LOSS

The difference between stages in Biscayne Bay and at the tailwater of each salinity structure is a function of the head loss in the downstream channel. Because the head loss in the channel is a function of the conveyance capacity, canal length, and flow rate, the magnitude of the difference from the tailwater to the Bay varies for each location. Because each design storm scenario does not reflect an actual event that can be measured, it is difficult to determine the value of the head loss for each scenario. Therefore to approximate the head loss for each design storm, CMA proposed a six-step process. This process is outlined below:

1. Calculate the difference between measured values at the NOAA tidal gauge at Virginia Key and the tailwater at each structure for the October 1999 Hurricane Irene storm event.
2. Develop a scatter plot for each watershed of calculated head loss from Hurricane Irene and measured discharge at the salinity structure.
3. Calculate a linear regression equation for each watershed to relate flow to head loss

4. Utilize the regression equation to determine the approximated head loss at each timestep for each design storm at each location using the District's HEC-RAS model of the C-4 Watershed to generate the flows.
5. Subtract the approximated head loss at each timestep from the District's tidal timeseries for each salinity structure to generate a projected Biscayne Bay timeseries for each watershed.
6. Average the projected Biscayne Bay boundary conditions to define a single time-series.

Figure 7-8 through **Figure 7-10** below illustrate the scatter plots and the regression lines for the S-22, G-93 and S-25B salinity structures. These figures were generated using measured data collected between October 14th through October 18th of 1999.

A review of the scatter plot results demonstrate that this approach is not a viable method for projecting boundary condition stages at the mouth of each primary canal in the project area due to poor correlation between flow and head loss at each location. In some cases, the head losses are calculated as negative because the stages at Biscayne Bay were higher than stages at the tailwater of the salinity structure. These trends are consistent with times when there is an incoming tide. However, at S-22, the slope of the regression line is negative, indicating that the trend demonstrates more head loss at lower flows than at higher flows. This is physically impossible, and likely indicates that the Virginia Key tide gauge does not accurately portray conditions at the mouth of Snapper Creek, potentially because of the distance between the two locations.

Figure 7-8: Scatter Plot and Regression Line for S-22

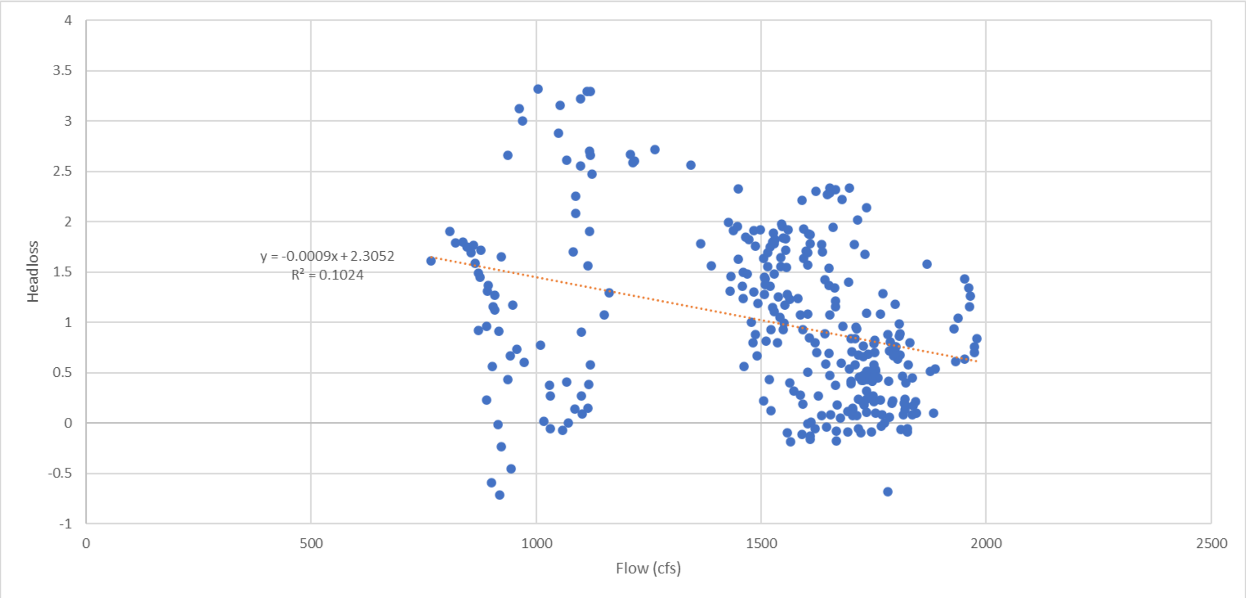


Figure 7-9: Scatter Plot and Regression Line for G-93

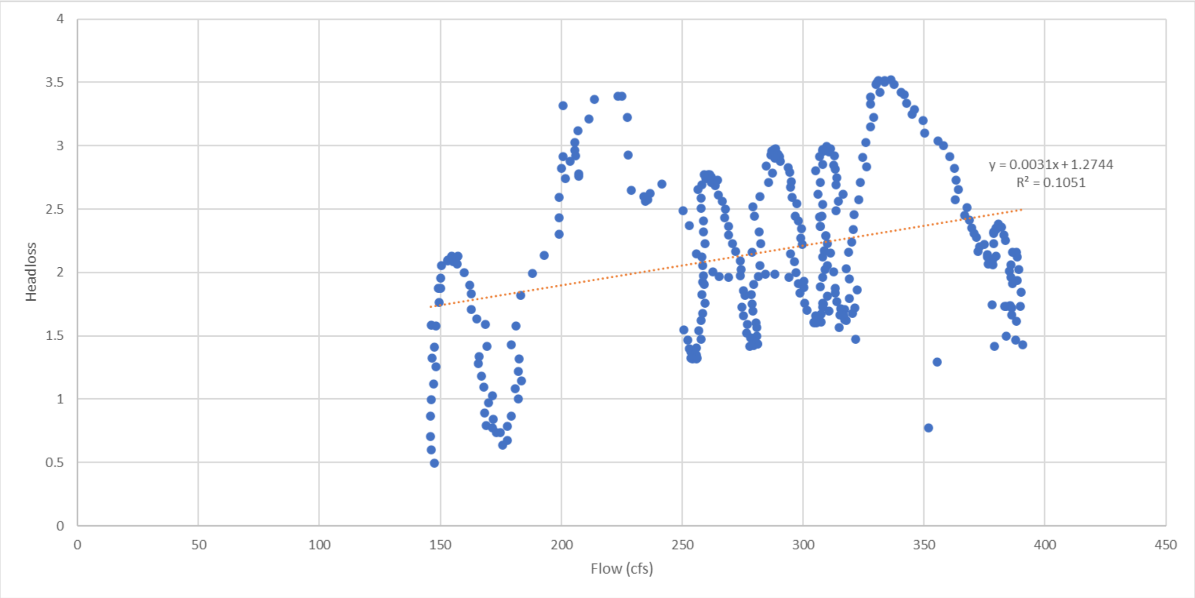
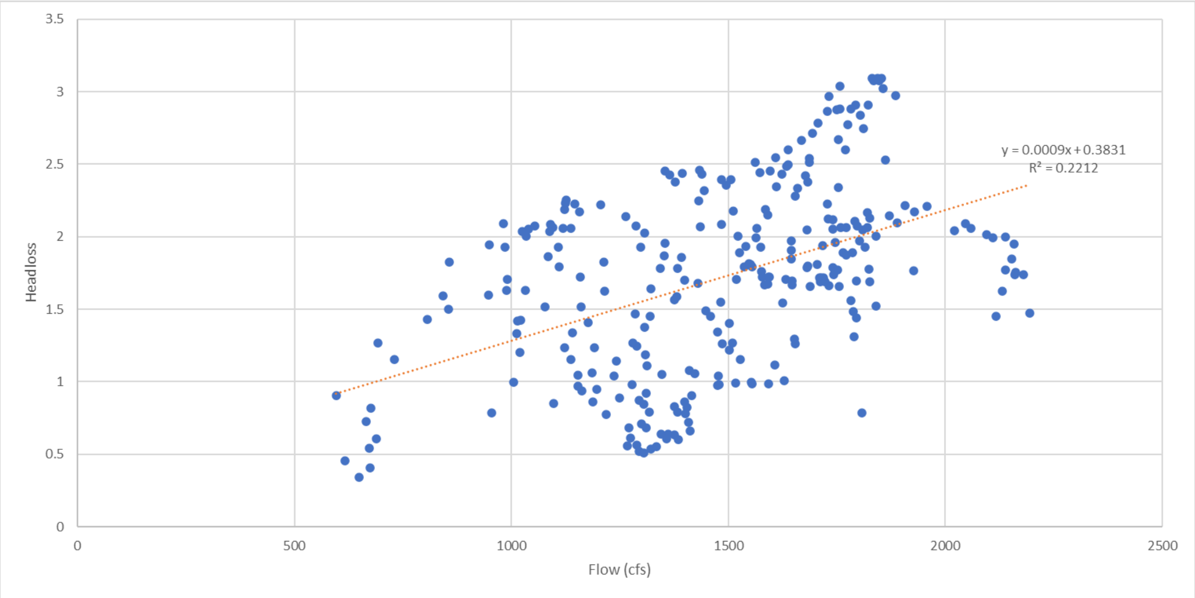


Figure 7-10: Scatter Plot and Regression Line for S-25B



Considering the poor correlation at each location, the regression results were not utilized to project tidal boundary condition stages for each design storm event.

7.4.2.2. BOUNDARIES FROM INTERPOLATED TAILWATER PROJECTIONS

An alternative approach to defining the tidal boundary at Biscayne Bay is to analyze projected data from structures that are closest to Biscayne Bay at either end of the coastline of the model domain and prepare an interpolated hydrograph for each outlet based on the distance between the north and south data points. S22 is the southernmost structure located near Biscayne Bay, approximately 1.43 miles upstream on the C2 Canal or Snapper Creek. S27 is the northernmost structure, located near Biscayne Bay approximately 1.2 miles upstream on the C7 Canal or Little River. **Figure 7-11** compares the tidal boundary provided by the District at the tailwater of the S22 and S27 structures. **Figure 7-14** illustrates the interpolated tidal boundary at the mouth of the Miami River (downstream of the S25, S25B, and S26 structures) as well as the Coral Gables Canal (downstream of the G93 structure). This analysis is based on the data provided for S22 and S27 and the interpolation of projected values at the Miami Canal and Coral Gables Canal is weighted based on the distance of each location from those structures.

Figure 7-11: Tidal Boundaries for the 25-year, 3-day storm at S-22 and S-27

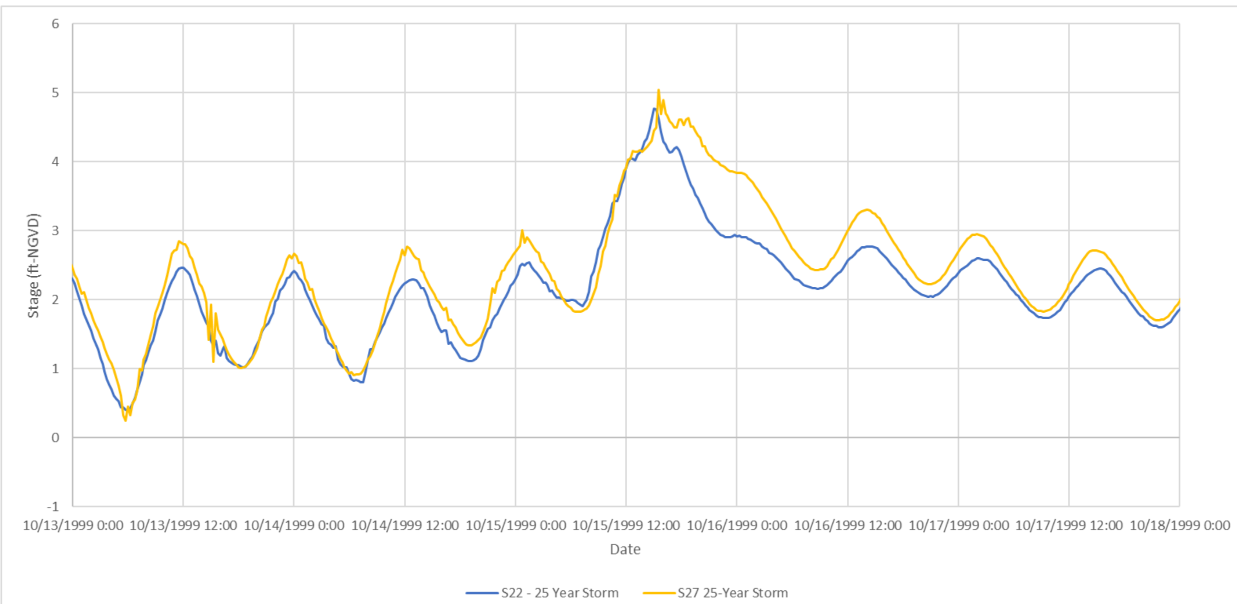
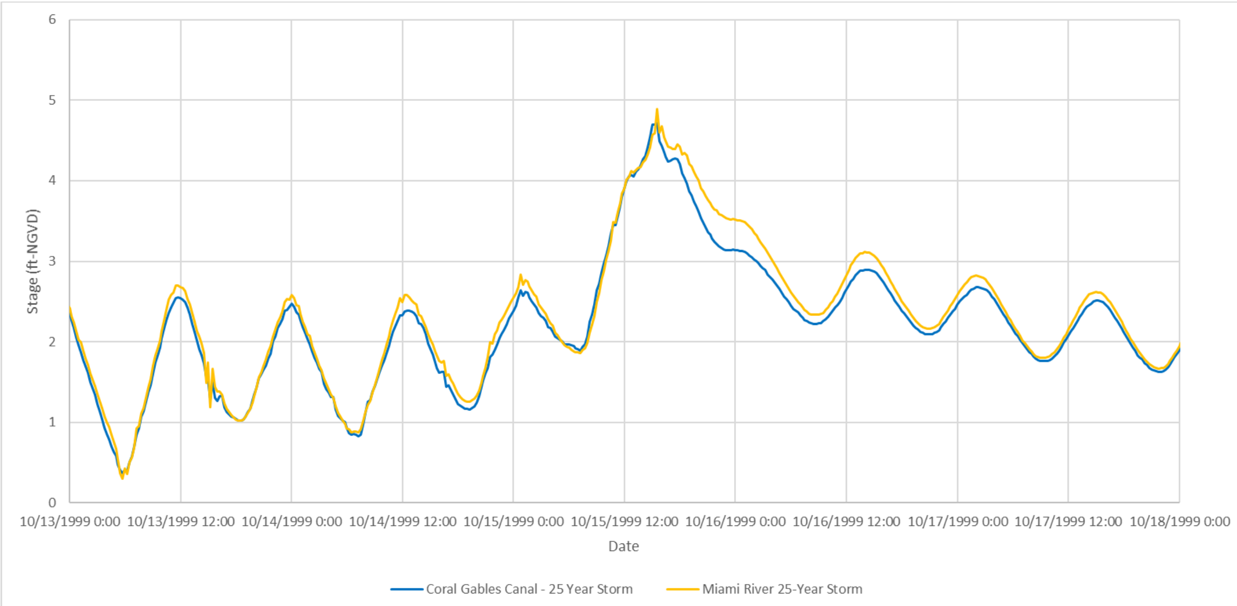


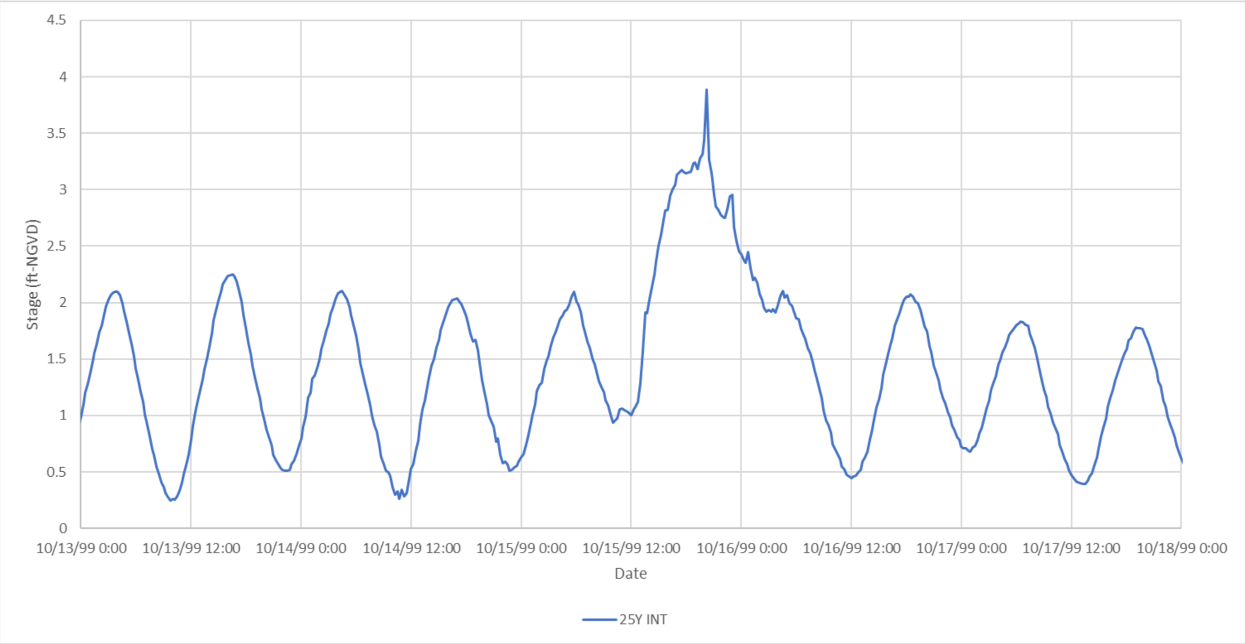
Figure 7-12: Interpolated Tidal Boundaries at Coral Gables Canal (C-3) and the Miami River (C-6)



7.4.2.3. BOUNDARIES FROM RATIO ANALYSIS

The third approach to defining the tidal boundary at Biscayne Bay is to approximate a timeseries for each of the design storm events based on the assumption that data from Hurricane Irene at the Virginia Key station reflects the 50-year storm. This assumption is consistent with the tidal structure tailwater data provided by the District, because the provided stage data matches the values measured during the 1999 Hurricane Irene event. By comparing the proposed timeseries at each coastal structure (S22, G93, S25, S25B, and S26) for each design storm (5-year, 10-year, 25-year, and 100-year), CMA calculated a ratio comparing the 50-year tailwater for each coastal structure. The average ratio for each design event was calculated for every 15-minute timestep and applied to the Virginia Key tidal event data for the 1999 Hurricane Irene event to generate a unique Biscayne Bay timeseries for each design storm. **Figure 7-13** illustrates the 25-year storm ratio applied to the Virginia Key tidal data.

Figure 7-13: Interpolated Tidal Boundaries at Virginia Key



7.4.2.4. FINAL TIDAL BOUNDARIES

Based on the comparison of projected boundary conditions, the interpolation method was considered the most conservative. It achieved the highest peak compared to the other ratio method. The only concern for the interpolation method was that the post storm stages appear to have a slight positive vertical displacement. The ratio method did not reach the same magnitude of peak, but it showed a slight phase shift forward in time while more accurately representing the condition of the Bay after the storm. Based on a comparison of the results, the tidal boundaries at Biscayne Bay for the mouth of the C6_Canal_Tidal and Coral_Gables_Canal_Tidal branches were defined using the interpolation method described in **Section 7.4.2.2**. This method analyzed projected data from structures that are closest to Biscayne Bay at either end of the coastline of the model domain and prepare an interpolated hydrograph for each outlet based on the distance between the north and south data points. The southernmost structure is S-22 near Biscayne Bay, approximately 1.43 miles upstream of the model boundary in the C-2 Canal or Snapper Creek. The northernmost structure is S-27, located near Biscayne Bay approximately 1.2 miles upstream of the model boundary in the C-7 Canal or Little River.

Figure 7-14 illustrates the interpolated tidal boundary at the mouth of the Miami Canal and **Figure 7-15** shows the interpolated tidal boundary at the mouth of the Coral Gables Canal. This analysis was based on the data provided for S-22 and S-27 and the distance of each location from those structures. These interpolated boundaries were also used as the coastal groundwater boundary for these regions.

Figure 7-14: Interpolated Tidal Boundary at the Miami River (C-6)

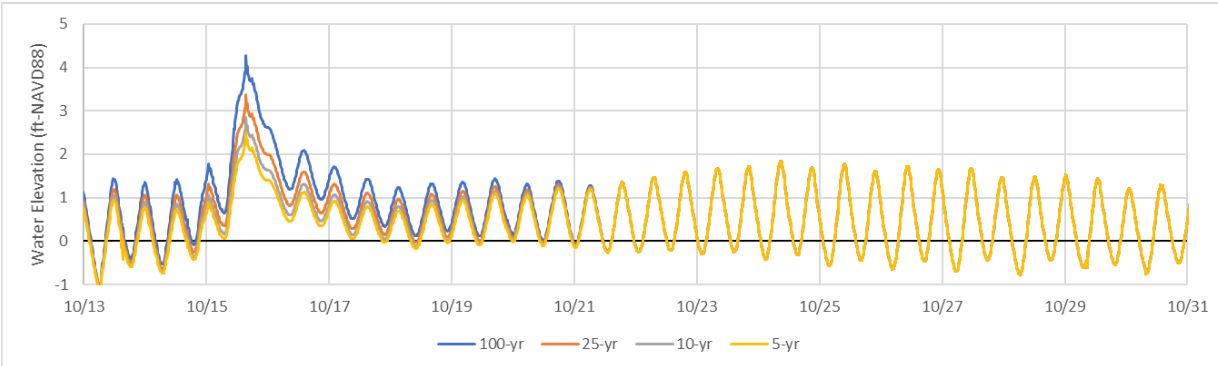
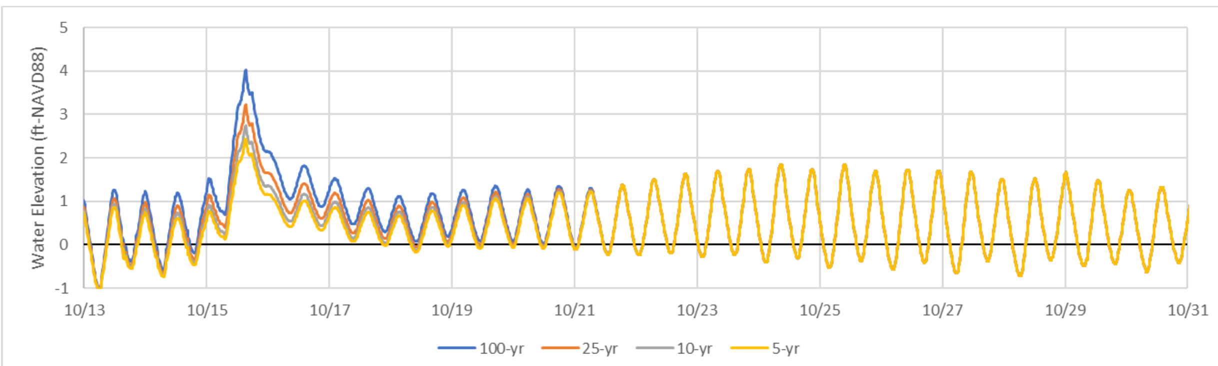


Figure 7-15. Interpolated Tidal Boundary at Coral Gables Canal (C-3)



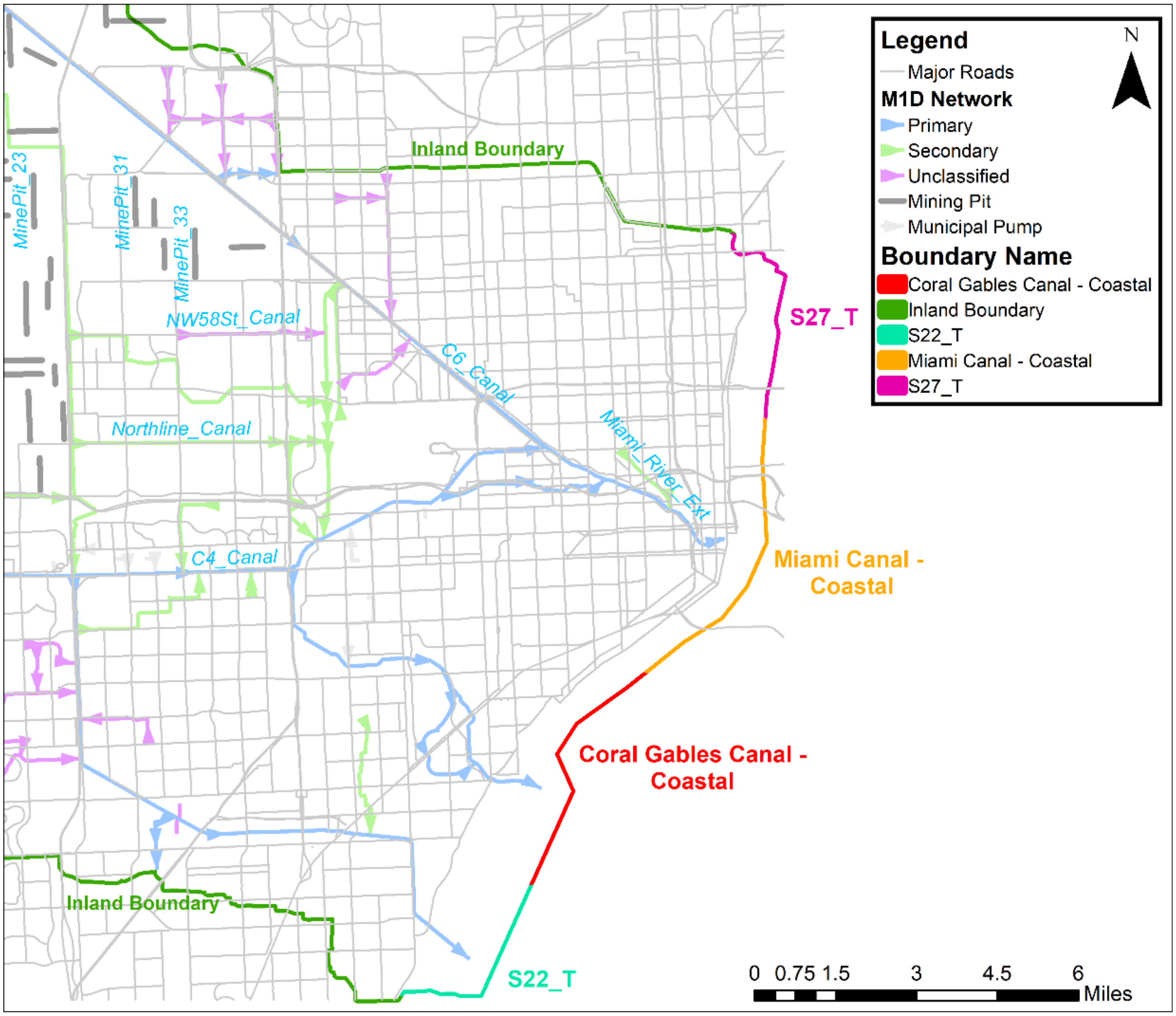
7.4.3. INLAND SURFACE WATER BOUNDARIES

Inland surface water boundaries account for the headwaters (HWs) and upstream conditions of the watersheds. Since no timeseries are available for the HWs, and stages upstream are highly dependent upon the structure operations, constant values were used at these locations. Measured data was used at these locations from the nearest station for September 1, 2017. A full description of these boundaries is provided in **Section 8.5.2.**

7.4.4. GROUNDWATER BOUNDARIES

For the calibration and validation model, the groundwater boundary was a spatial and time-varying head condition imposed along the perimeter of the model domain, as shown in **Figure 7-16**. The values for the groundwater boundary were defined using interpolation techniques to approximate the condition from measured values at monitoring wells. For the existing conditions design storm simulations, it is important to define the groundwater boundaries in a manner that assumes a relatively high-water table condition to be conservative for flooding simulation. The USGS Seasonal high-water table (USGS, 2016) was used as the groundwater boundary for all inland areas.

Figure 7-16. Groundwater Boundary Setup



7.5. CONTROL STRUCTURE OPERATIONS

One of the District's primary objectives is to utilize infrastructure to balance the needs of the environment with developed areas in South Florida. Water control structures are a key component of that infrastructure and enable the storage or conveyance of water to achieve management objectives. For the calibration and validation simulations, control structure operations were based on recorded gate opening and pumping data. For the current conditions event simulations, the gate operations were input as logical operands based on documented operational strategies. The subsections below describe the structure operation assumptions that were used for the current conditions model.

7.5.1. TYPICAL OPERATIONS FOR THE PRIMARY SYSTEM

The primary water management system is the network of State owned and operated facilities that make up the main conveyance canals (Snapper Creek Canal, Coral Gables Canal, C4 Canal, Comfort Canal Southfork, and the C6 Canal) and their related control structures. The primary water management system is constructed to provide for regional water supply needs during dry periods and regional flood protection during wet periods.

During the dry season, the water control structures on the western edge of the model domain open to allow water to move east from WCA 3A. This supports recharge in developed areas and municipal wellfields where water supply withdrawals are made. To facilitate groundwater recharge, structures on the eastern side of the model domain are infrequently opened to maintain water table elevations as high as possible. A specific focus of the structures on the eastern side of the model domain is salinity control operations. The tidal structures of S22, G93, S25, S25B, and S26 all have operational triggers that maintain freshwater to the west and prevent intrusion of saline water from the east.

During the wet season, the water control structures on the western edge of the model domain are closed. This prevents inflows of water from WCA 3A that may diminish the ability of the system to provide flood protection. During this period, structures on the eastern side of the model domain are frequently opened to maintain canal stages and water table elevations at a lower level. This allows for sufficient storage capacity in the watershed to sustain a moderate rainfall event while providing flood protection.

7.5.2. STORM EVENT OPERATIONS FOR THE PRIMARY SYSTEM

During a large rainfall event similar to the design storms, water control infrastructure on the western edge is closed while control structures on the eastern edge are opened on an as needed basis. However, within the model domain, there are unique facilities such as the C4 Emergency Detention Basin (EDB) and several municipal pumps. These facilities make structure operations complex and provide additional mechanisms for regional flood protection. The operating protocol is summarized below but is defined in greater detail in the *MDC County Flood Mitigation Program C-4 Basin Operating Plan* (SFWMD, 2010).

At the beginning of a flooding event, as the stages rise in the canal network, the gates at the coastal structures open to allow inland facilities to discharge to tide. If the stages continue to increase upstream but the tailwater conditions become adverse to gravity discharge due to tidal conditions or storm surge, the forward pumps at S-25B (C4 Canal) and S-26 (C6 Canal) engage. Forward pumping stops at both locations if the gravity discharge capacity exceeds 600 CFS or if the stages in the Miami River exceed 3.18 ft-NAVD, or 4.75 ft-NGVD, to provide downstream flood protection. If the stages upstream continue to rise, the C4 Emergency Detention Basin engages, and the pump station at G-420 begins back pumping from a western reach of the C4 Canal into the detention basin. As the stages increase in the C4 Canal, the G-422 pump station engages to increase the rate of back pumping into the detention basin. At times, canal stages in the C4 Canal are excessively drawn down by the pumps due to the local effects of pump suction. If this occurs, one of the five S-380 culverts to the west opens to facilitate pumping operations. Back pumping continues if the stages in the C4 Canal continue to increase. This continues until the detention basin is filled (8.43 ft-NAVD or 10.0 ft-NGVD). After this point, there is no additional flood protection capacity within the existing primary system.

The Boolean logic tools within MIKE 1D were utilized to recreate the operational strategies within the simulation framework. The MIKE 1D tools recreate regional operations such as the Detention Basin, as well as local operations such as the opening and closing of structures based on differential head criteria. In specific, for control structures an operational definition is added that states that the differential head (dH) at the gate must be greater than or equal to a fixed amount (such as 0.3 feet) for the gate to open and a separate statement is added to close the gate in cases where the dH is less than or equal to a fixed amount (such as 0.1 feet). If neither condition is met, the gate remains in the same position as the previous timestep. For tidal structures the operations utilize a larger threshold of 0.3 feet dH for closure to prevent salinity intrusion. To prevent the types of model instability that often come with gates that open and close too frequently, an additional control definition is added to state that the gate cannot change position more frequently than every 15 minutes.

On May 26, 2022, the operational strategies were discussed at the Weekly Operations Meeting with the water managers from the Office of Operations. During this meeting several items were discussed regarding the assumptions for the operation of spillways, pump stations, and gated culverts. Some of the key elements that changed the operational assumptions of the model were:

- When the structure operations are engaged, the District recommends using an operational logic that sets the gate to “Open” if the head difference across the spillway is 0.3 feet and “Close” if the head difference is less than 0.1 feet for all tidal gates for salinity control purposes.
- District staff described that the typical operations in the Miami Dade region have a normal and low range for canal levels. The operations are shifted from normal to low ranges based on predicted weather from meteorologists and antecedent conditions in the basin. Returning from low range to normal range is based on observing the recovery after a storm and is often based on antecedent or pre-storm stages. For the purposes of analysis of storm events, the model will use low operating ranges for all structures.

- District staff noted that the forward pumps for the C4 and C6 Canals, (S26_P and S25B_P) are operated in a synchronized manner.
- District staff noted that the gated culvert at S25 tends to open and close in short duration cycles when in low range operations, so the engagement of low range operations may be delayed during a storm event.
- The Structure Atlas provides language that describes the rate at which the gates open as six (6) inches per minute for gates S22 and S25B. The District confirmed that this is a reasonable rate for all the model gates.
- For storm event simulations, the model uses "low setting" range of operations at the tidal structures. The “Low Range” settings define the open and close trigger elevations of the structure gates lower than the standard "wet season" operational triggers to provide more aggressive drainage to tide.

Based on these recommendations, updated structure sheets (provided by the District) and an updated C4 Watershed Operations Plan, the hydraulic model parameters were modified for the current conditions design storm simulations. The tables below describe the final assumptions that were used in the modeling effort. Of note, the MIKE 1D network uses logical operands to open and close gates and turn on and off pumps as established by the modeler. Using priorities and logical operands for each control definition, the structure setup can be read as “if-else” statements. **Table 7-1** provides the list of structures and their operational protocol written out as logical statements for the tidal structures and the C4 Impoundment, respectively. As an example, the S25 logic language can be read as follows:

> *“If the time since the last gate change was less than 6 minutes ago, the gate level will remain unchanged. If that is not true, then the gate will close if the head difference across this structure is less than 0.1 feet. If that is not true, then the gate will open fully if the head upstream of the gate is greater than (-)0.345 ft-NAVD. If that is not true, then the gate will close if the head upstream of the structure is less than (-)0.745 ft-NAVD. If that is not true, then the gate will open fully if the head difference across this structure is greater than 0.3 feet. If none of these statements are true, the gate level will remain unchanged.”*

It is noted that all structures at the model boundaries were set to “closed” during the design storm events. This includes the following:

- S31, S32, and S337 (S32A culvert was set to “fully open” during the full simulation)
- S334 and S356 Pump
- G211
- S121

Table 7-1. Structure Operations for the Tidal Structures

Branch Name	Name	Structure Type	Logic Language
Coral_Gables_Canal	G93_G1	Sluice, Formula	Unchanged if TSLGLC<6min, Else Closed if dH<0.1, Else Fully Open if Hups>0.76 ft-NAVD AND dH>0.3ft, Else Closed if Hups<-0.54 ft-NAVD, Otherwise Unchanged
Coral_Gables_Canal	G93_G2	Sluice, Formula	Unchanged if TSLGLC<6min, Else Closed if dH<0.1, Else Fully Open if Hups>0.76 ft-NAVD AND dH>0.3ft, Else Closed if Hups<-0.54 ft-NAVD, Otherwise Unchanged
Snapper_Creek_Canal	S22_G1	Sluice, Formula	Unchanged if TSLGLC<6min, Else Closed if dH<0.1ft, Else Fully Open if Hups > 0.96 ft-NAVD AND dH>0.3ft, Else Closed if Hups < -0.04 ft-NAVD, Otherwise Unchanged
Snapper_Creek_Canal	S22_G2	Sluice, Formula	Unchanged if TSLGLC<6min, Else Closed if dH<0.1ft, Else Fully Open if Hups > 0.96 ft-NAVD AND dH>0.3ft, Else Closed if Hups < -0.04 ft-NAVD, Otherwise Unchanged
Comfort_Canal_Southfork	S25	Underflow	Unchanged if TSLGLC<6min, Else Closed if dh<0.1, Else Fully Open if Hups>-0.345 ft-NAVD, Else Closed if Hups<-0.745 ft-NAVD, Else Fully Open if dh>0.3ft, Otherwise Unchanged
Comfort_Canal_Southfork	S25A_G1	Underflow	Closed
C4_Canal	S25B_G1	Sluice, Formula	Unchanged if TSLGLC<6min, Else Closed if dh<0.1ft, Else Fully Open if Hups>0.451 ft-NAVD, Else Closed if Hups<-0.549 ft-NAVD, Else Fully Open if dH>0.3ft, Otherwise Unchanged
C4_Canal	S25B_G2	Sluice, Formula	Unchanged if TSLGLC<6min, Else Closed if dh<0.1ft, Else Fully Open if Hups>0.451 ft-NAVD, Else Closed if Hups<-0.549 ft-NAVD, Else Fully Open if dH>0.3ft, Otherwise UnchangedTSLGLC
C4_Canal	S25B_P	Discharge	Fully Open if H at T5W > 2.25 ft-NAVD AND Hups > -0.55 ft-NAVD AND H at MRMS1 <3.2 ft-NAVD, Else Closed
C6_Canal	S26_G1	Sluice, Formula	Unchanged if TSLGLC<6min, Else Closed if dh<0.1ft, Else Fully Open if Hups > 0.155 ft-NAVD AND dH>0.3ft, Else Closed if Hups<-0.345 ft-NAVD, Otherwise UnchangedTSLGLC
C6_Canal	S26_G2	Sluice, Formula	Unchanged if TSLGLC<6min, Else Closed if dh<0.1ft, Else Fully Open if Hups > 0.155 ft-NAVD AND dH>0.3ft, Else Closed if Hups<-0.345 ft-NAVD, Otherwise UnchangedTSLGLC
C6_Canal	S26_P	Discharge	Closed if Hups<-0.54 ft-NAVD, Else Fully Open if Q at S25B_P>0 AND Hups> -0.14 ft-NAVD, Else Closed if Q at S25B_P <=0, Else Closed if Q at S26_G1 <300CFS, Otherwise Unchanged

H = head, dH = delta or difference in head between upstream and downstream of the gate,
Hups = head upstream of structure, TSLGLC – Time Since Last Gate Level Change

Table 7-2 provides the list of structures relating to the C4 Impoundment and their operations, as determined from the C4 Operations Manual and structure atlas. In addition to these structures, the S380 structure is operated fully open during the storm event for the full duration of the event.

Table 7-2. Structure Operations for the C4 Impoundment

Branch Name	Name	Structure Type	Logic Language
G420_Inflow	G420_P	Discharge	Closed if Hups<2.628 ft-NAVD, Else Close if T5W < 4.328 ft-NAVD AND C4Imp >6.432 ft-NAVD, Else Close if C4Imp >8.432 ft-NAVD, Else Fully Open if H at T5W is > 3.228 ft-NAVD, Otherwise Unchanged
C4_Impoundment_Ditch_N	G421_S	Sluice, Formula	Fully Open if T5W is <3.928 ft-NAVD AND flow at G420_P>0, Otherwise Close
C4_Impoundment_Ditch_S	G420S_P	Discharge	Closed if Hdws>3.432 ft-NAVD, Else Fully Open if Hups<2.432 ft-NAVD, Otherwise Unchanged
G422_Inflow	G422_P	Discharge	Closed if Hups<2.628 ft-NAVD, Else Close if T5W < 4.328 ft-NAVD AND C4Imp >6.432 ft-NAVD, Else Close if C4Imp >8.432 ft-NAVD, Else start pumps at 267 CFS if H at T5W is > 3.428 ft-NAVD and pump up to 623CFS if H at T5W is >3.628 ft-NAVD, Otherwise Unchanged
G423_Inflow	G423_G1	Underflow	Fully Open if Q>0 at G420P, Otherwise Closed
G423_Inflow	G423_G2	Underflow	Fully Open if Q>0 at G420P, Otherwise Closed
G423_Inflow	G423_G3	Underflow	Fully Open if Q>0 at G420P, Otherwise Closed
G423_Inflow	G423_G4	Underflow	Fully Open if Q>0 at G420P, Otherwise Closed
G423_Inflow	G423_G5	Underflow	Fully Open if Q>0 at G420P, Otherwise Closed
G423_Inflow	G423_G6	Underflow	Fully Open if Q>0 at G420P, Otherwise Closed
H = head, dH = delta or difference in head between upstream and downstream of the gate, Hups = head upstream of structure, Hdws = head downstream of structure, Q = discharge			

7.5.3. STORM EVENT OPERATIONS FOR THE SECONDARY SYSTEM

Several municipalities have separate flood protection infrastructure including canals, gates, and pump stations. Along the C3 and C4 Canals seventeen (17) pump stations that collect runoff from the local stormwater management systems were implemented into the model for the neighborhoods of Belen, Sweetwater, West Miami, and the City of Miami. Each pump station operates in the model according to the permitted wet well on and off triggers as well as a trigger for the C4 Canal at the T5W Station. The pumps will turn on when upstream wet well stages approach flood conditions and off when the well is dry, according to the municipal pump data provided by the District, as shown in **Table 7-3**. In addition, the pumps will turn off if stages at T5W exceed 5 ft-NAVD.

Table 7-3. Municipal Pump On and Off Triggers for the Design Events

Municipality	Municipal Pump Station	Pump Capacity [CFS]	T5W Off Elev. [ft-NAVD]	Wet Well Off Elev. [ft-NAVD]	Wet Well On Elev. [ft-NAVD]
Sweetwater	B1	20	3.43	-6.57	1.43
	B2	20	3.43	-6.57	1.43
	B15	25.4	3.43	-5.57	-1.57
	B16	25.4	3.43	-5.57	-1.57
	IIA #3	27	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS
	IIA #4	27	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS
	IIB #1	27	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS
	IIB #2	27	3.43	-6.57	1.43ft: 13.5CFS 1.93ft: 27CFS
Belen	PS1	100	3.43	-7.32	2.95ft: 25CFS 3.00ft: 100CFS
	PS2	100	3.43	-7.32	2.95ft: 25CFS 3.00ft: 100CFS
West Miami	#1	90	3.43	-9.57	1.43
	#2	100	3.43	-9.57	1.43
City of Miami	#1	30	3.43	-8.57	1.43
	#2	60	3.43	-8.57	1.44
	#3	40	3.43	-8.57	1.44
	#4	54	3.43	-8.57	1.44
	#5	15	3.43	-8.56	1.45

The following logic statement summarizes the imposed operations for all municipal pumps:
Closed if Hups<[Wet Well Off Trigger], Else Closed if H at T5W > 3.43 ft-NAVD, Else Fully Open if Hups>[Wet Well On Trigger], Otherwise Unchanged.

8. FUTURE CONDITIONS DESIGN STORM SETUP

The future conditions models were developed to simulate the effect of Sea Level Rise at +1, +2, and +3 feet for the 5-year, 10-year, 25-year, and 100-year design storm events. In addition to coastal boundaries, which were modified to reflect increasing sea levels, the initial groundwater levels and groundwater boundary conditions and initial surface water levels and surface water boundaries were also modified to reflect this increase.

Since future conditions would also include an increase in development, the increase in infrastructure was represented by modifying topography and land use based on currently undeveloped or older parcels. The future land use file then informed how the maps of overland Manning's roughness, detention storage, percent impervious, and drainage depth would increase. The projected increase in groundwater pumping was also accounted for with a 44% increase in total pumping using the projected 2065 pumping from the Miami-Dade Water and Sewer Department (MD WASD) Permit. In addition, planned canals from Miami Dade were added to the 1D network.

The Future Conditions simulations vary from the Current Conditions model with the following modifications:

1. Topography
   a. Modified to reflect the development of currently vacant parcels and parcels built prior to 1970.
2. Land Use
   a. Modified to reflect the development of currently vacant parcels and parcels built prior to 1970.
3. Overland Parameters
   a. Land use base maps of Manning's M, runoff coefficient, and groundwater drainage levels recreated using future land use.
   b. Modified initial overland depth map based on SLR.
4. Groundwater
   a. Drain level map recreated using future land use.
   b. Modified maps of boundary and initial conditions.
   c. Projected increase in groundwater pumping.
5. MIKE 1D
   a. Modified 1D network to reflect future planned canals from Miami Dade County.
   b. Modified initial conditions and boundary conditions.

The intensity of the rainfall events will remain the same for the future conditions analysis as for the current conditions analysis. The 100-year 3-day design storm will have the same intensity as the current conditions simulations. The rainfall volumes were obtained from NOAA Atlas 14 and the time distribution from the SFWMD, ERP Permit Volume II.

8.1. CLIMATE

Rainfall and evapotranspiration will not be modified for the future conditions analysis. Although rainfall volumes and intensity are expected to change in the future, no consideration for these changes were used in this study due to the uncertainty of the rainfall model predictions.

Long-term trends at rain gauges in Miami may indicate an increase in annual rainfall totals. For example, rainfall at the Miami.FS_R station in Doral, Florida shows an increasing trend in annual rainfall since rainfall records began in January of 1965, as shown in **Figure 8-1**. However, the purpose of this study is to evaluate storm events and their impacts, and annual rainfall trends are not an indicator of whether storm rainfall intensity will be increasing with time. Therefore, an analysis of the frequency of high intensity rainfall events at this same station was performed. **Figure 8-2** shows no discernable trend for the frequency of occurrence of rainfall events greater than the 5-year 3-day design storm (which is an average of 8.69 inches over the C6 Watershed).

As with the current conditions simulation, the design storms were established from the NOAA Atlas 14 Rainfall on the NEXRAD grid. The maximum, mean, minimum, and standard deviation of the rainfall is provided for the four design storm events (i.e., the 100-year 3-day, 25-year 3-day, 10-year 3-day, and 5-year 3-day) for each watershed in **Table 8-1**.

Table 8-1. Rainfall Statistics for Each Watershed

Watershed	Max Rainfall (in)	Mean Rainfall (in)	Min Rainfall (in)	Std. Dev. (in)
100-Year 3-Day				
C2	17.68	17.46	17.15	0.17
C3W	17.66	17.63	17.59	0.02
C4	17.75	17.35	16.50	0.32
C5*	17.61	17.61	17.61	--
C6	17.77	17.44	16.49	0.42
25-Year 3-Day				
C2	13.11	12.93	12.69	0.13
C3W	13.10	13.06	13.03	0.02
C4	13.19	12.86	12.19	0.26
C5*	13.09	13.09	13.09	--
C6	13.20	12.92	12.16	0.34
10-Year 3-Day				
C2	10.54	10.39	10.20	0.11
C3W	10.53	10.50	10.47	0.02
C4	10.60	10.33	9.78	0.21
C5*	10.54	10.54	10.54	--
C6	10.61	10.36	9.74	0.29
5-Year 3-Day				
C2	8.86	8.74	8.57	0.09
C3W	8.86	8.84	8.81	0.02
C4	8.91	8.68	8.21	0.18
C5*	8.86	8.86	8.86	--
C6	8.91	8.69	8.17	0.24
*The C5 Watershed contains only one NEXRAD grid.				

Figure 8-1. Annual Rainfall Trends at Miami.FS_R Station

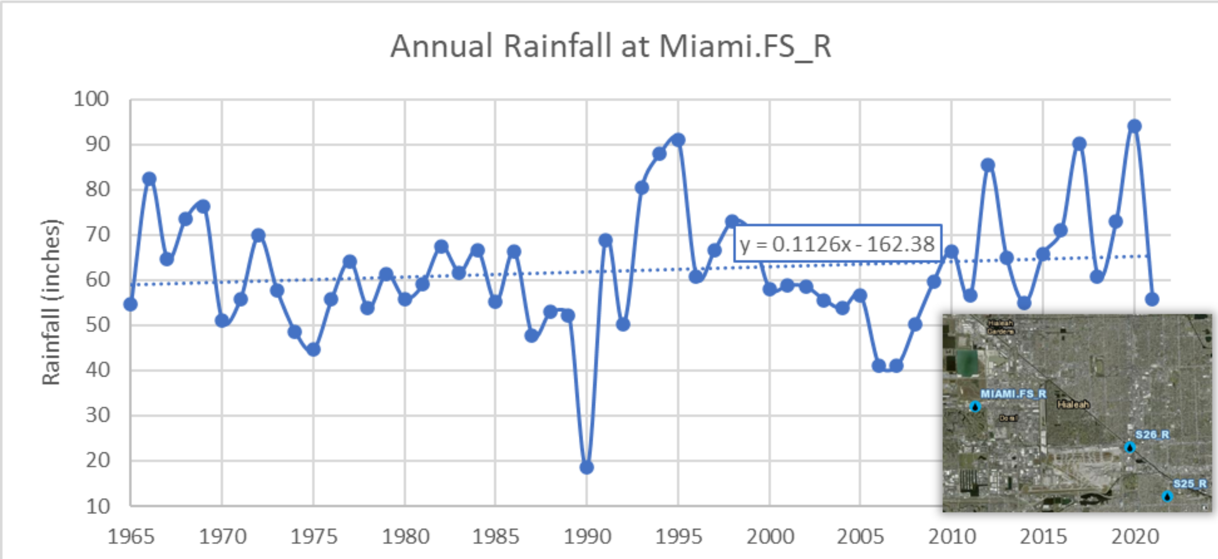
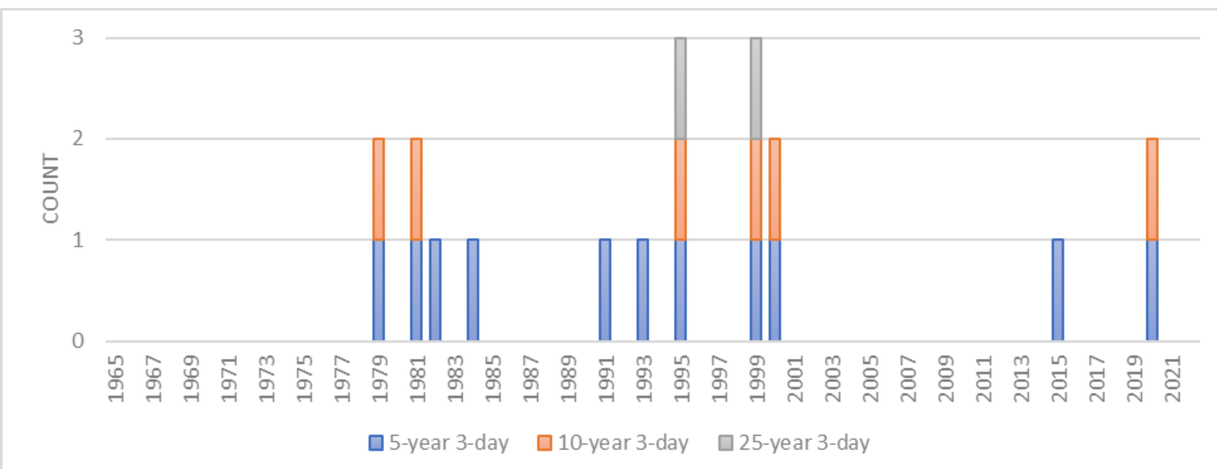


Figure 8-2. Number of Times the Design Rainfall Event is Exceeded Annually at Miami.FS_R



8.2. TOPOGRAPHY

The existing topography is based on the 2018 LiDAR data and will likely change in the next 50 years as more lots are used for residential and commercial development. To represent how the future development of the land will impact the topography, minimum floor elevations were applied to vacant parcels and parcels that were built prior to 1970. Land use data was obtained from the District, based on Level 2 of the Florida Land Use, Cover, and Forms Classification System (FLUCCS) codes.

The Proposed County Flood Criteria developed by Miami Dade County provides contours of the minimum ground surface elevation of developed properties, crown of roads, roadway grades, and secondary canal banks based on a 10-year 24-hour storm event, 2060 scenario with SLR. This data was converted into raster format as shown in **Figure 8-3**.

Parcel data was also obtained and filtered for the following criteria:

- Within the Miami Dade County Development Boundary (shown in yellow in **Figure 8-3**).
- Vacant and/or as-built date prior to 1970.
- Parcel size greater than one (1) acre (Parcels less than 1 acre were excluded since each model cell is 1.43 acres).

The parcel selection was then merged with land use data to select and remove parcel areas that are currently classified as water (i.e., FLUCCS Code 5100 through 5700). Parcels that were identified as Environmentally Endangered Lands (EEL) were also removed from the selection, as these lands are protected by Miami Dade County and will not be developed. The final parcel selection is shown in **Figure 8-3**.

These parcels were assigned the maximum flood criteria elevation on the entire lot. The lots were reduced by 10% using a buffer around each feature to account for the on-site stormwater collection that is required of new development. It should be noted that this buffer did not discriminate between lower or higher topography and reduced the size of the parcel by shrinking the parcel boundaries. The parcels were then exported to a raster and a mosaic was created to extract the maximum of the existing topography and the maximum flood criteria elevation for the selected reduced-size parcels. An example of the process is shown for a small, zoomed-in location within the C2 Watershed in **Figure 8-4**.

8.3. LAND USE

Using the same parcel selection as described in Section 4, a union was created with the current land use data to assign the code to the parcels. These codes were then modified based on the following rules:

- 1100 Residential, Low Density → 1200 Residential, Medium Density
- 1200 Residential, Medium Density → 1300 Residential, High Density
- 1900 through 4400 (Undeveloped) → 1100 Residential, Low Density
- 6100 and 6400 Wetlands (assuming mitigation will occur) →1100 Residential, Low Density
- 7400 Disturbed Lands → 1100 Residential, Low Density

The changes to the current land use are summarized in **Table 8-2** with the total acreage of each land use type for the current and future land use maps.

In many cases, the parcel data did not change, i.e., where parcel data overlapped with land use is defined as water, transportation, etc. The final land use changes for the select parcels is shown in **Figure 8-5**. These changes were copied into the existing land use map to modify only these selected parcel locations.

Future land use data from Miami Dade County was obtained to review and compare with the District land use modification procedure described above. The future land use data from Miami Dade County was taken from the Comprehensive Development Master Plan Land Use Plan. As noted by Miami Dade County, the plan is not an official zoning map and within each map category, numerous land uses, zoning districts and housing types can occur. The Miami Dade County future land data was coarser and more generalized than the land use classifications from the District (i.e., Miami Dade County shapefile contained 1,164 different features in the model domain, while the District's shapefile contained 11,488). Therefore, the District's land use classifications were used to estimate the future land use classifications.

Table 8-2. Future Land Use Changes to Vacant and Old Parcels

FLUCCS Code	Land Use	Current Land Use Area		Future Land Use Area	
		(Acres)	%	(Acres)	%
1100	Residential, Low Density	3,155	1.5	4,191	2.0
1200	Residential, Medium Density	40,001	19.4	39,428	19.2
1300	Residential, High Density	25,077	12.2	26,292	12.8
1400	Commercial and Services	16,446	8.0	16,445	8.0
1500	Industrial	7,833	3.8	7,833	3.8
1600	Extractive	14,459	7.0	14,459	7.0
1700	Institutional	5,725	2.8	5,725	2.8
1800	Recreational	3,810	1.9	3,782	1.8
1900	Open Land	1,187	0.6	832	0.4
2100	Cropland and Pastureland	2,428	1.2	2,419	1.2
2400	Nurseries and Vineyards	1,291	0.6	1,190	0.6
2500	Specialty Farms	5	0.0	5	0.0
2600	Other Open Lands – Rural	138	0.1	138	0.1
3100	Herbaceous (Dry Prairie)	252	0.1	156	0.1
3200	Upland Shrub and Bushland	620	0.3	559	0.3
4100	Upland Coniferous Forest	18	0.0	18	0.0
4200	Upland Hardwood Forest	871	0.4	628	0.3
4300	Upland Mixed Forest	121	0.1	97	0.0
5100	Streams and Waterways	2,187	1.1	2,169	1.1
5200	Lakes	16	0.0	16	0.0
5300	Reservoirs	4,689	2.3	4,588	2.2
5400	Bays and Estuaries	7,358	3.6	7,352	3.6
6100	Wetland Hardwood Forest	17,142	8.3	16,899	8.2
6400	Vegetated Non-Forested Wetlands	35,243	17.1	35,165	17.1
7400	Disturbed Land	3,677	1.8	3,367	1.6
8100	Transportation	9,735	4.7	9,732	4.7
8200	Communications	134	0.1	134	0.1
8300	Utilities	2,244	1.1	2,243	1.1

Figure 8-3. Miami Dade Flood Criteria and Vacant and Old Parcels East of the Urban Development Boundary

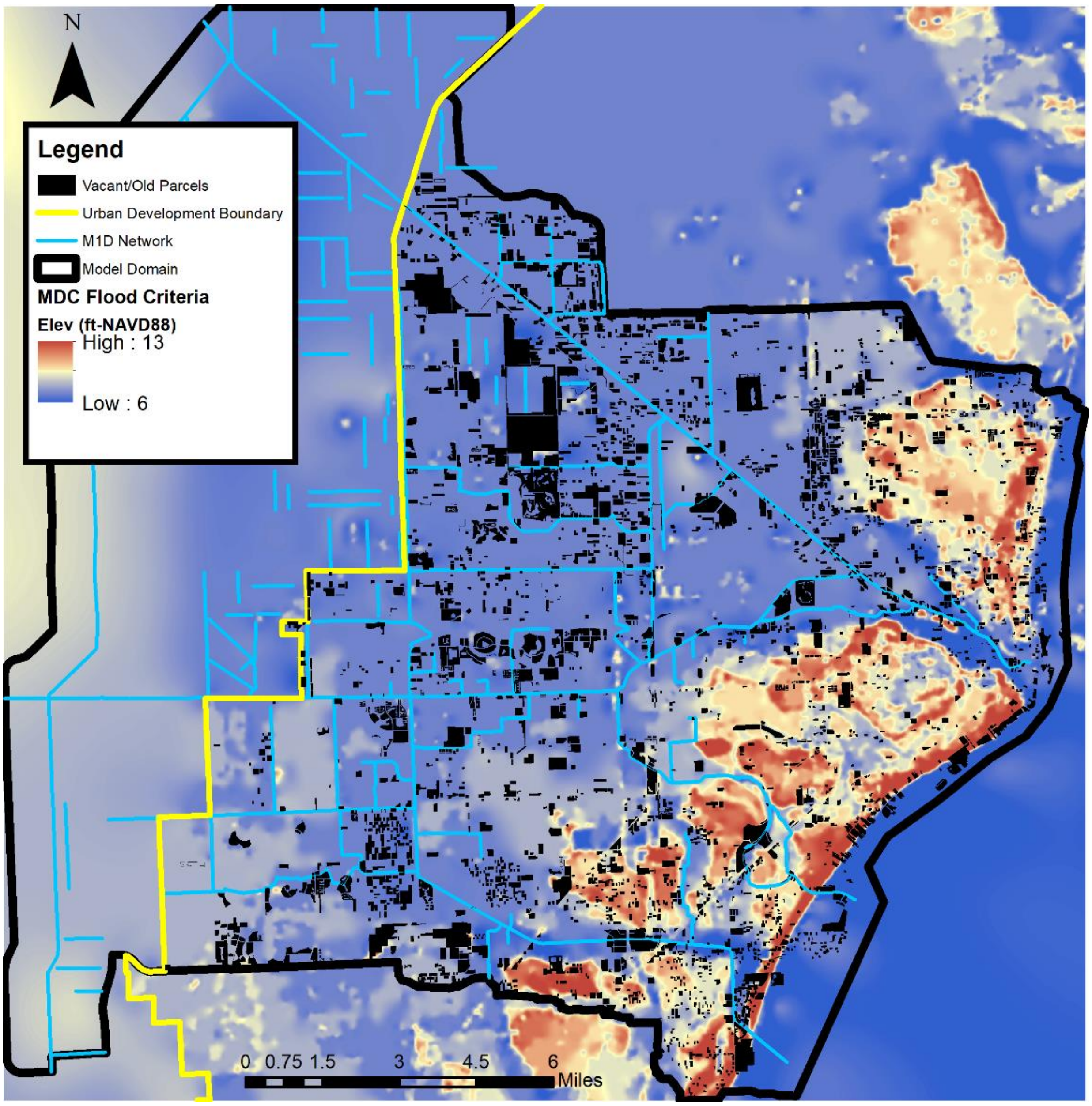


Figure 8-4. Modification to Existing Topography Using Select Parcels and the Miami Dade County Flood Criteria

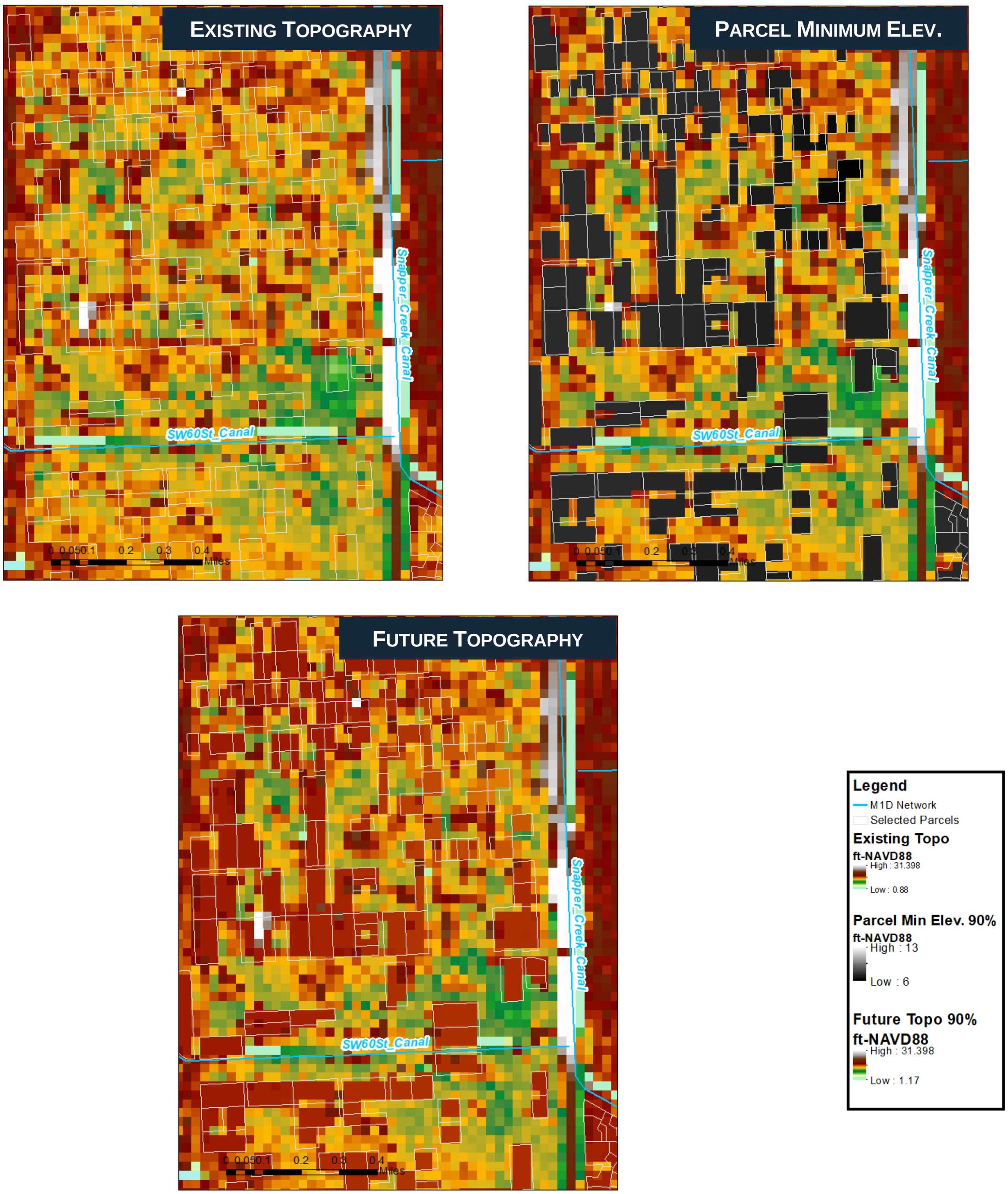
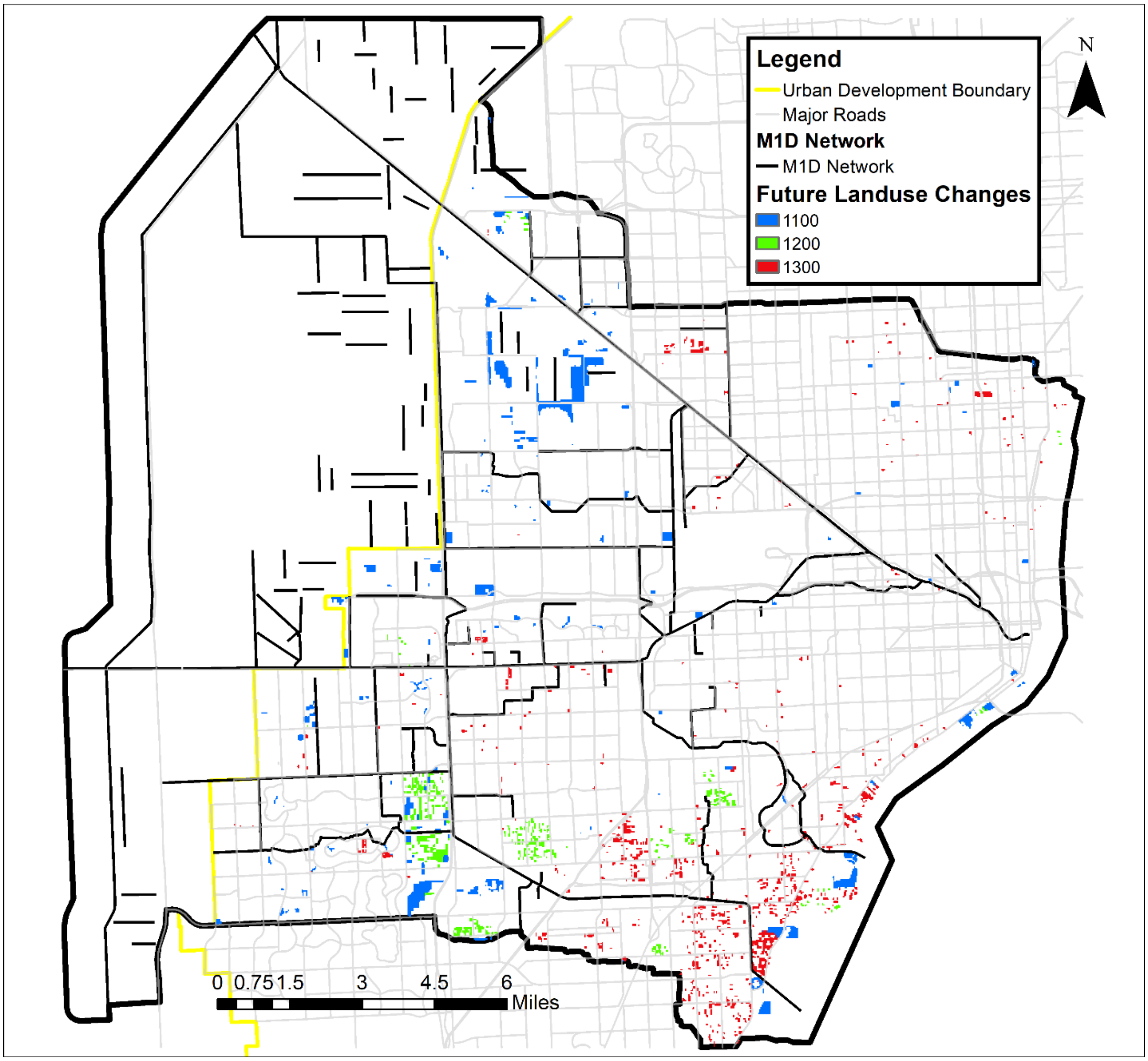


Figure 8-5. Future Land Use Changes for Vacant and Older Parcels East of the Urban Development Boundary



8.4. OVERLAND PARAMETERS

In the current conditions simulations, overland parameters such as Manning's M and the runoff coefficient, as well as the groundwater drainage level, were established by correlating values to land use codes, as shown in **Table 8-3**. For the future conditions' simulations, these same parameters were used with the modified land use map to create updated future conditions Manning's M, runoff coefficient, and drain level.

Table 8-3. Overland and Drainage Parameters for Each Land Use Code

FLUCCS Code	Land Use	Manning's M	Runoff Coefficient	Drain Level (ft)
1100	Residential, Low Density	7.14	0.075	1.5
1200	Residential, Medium Density	8.33	0.22	1.5
1300	Residential, High Density	9.09	0.45	1.5
1400	Commercial and Services	14.29	0.72	1.5
1500	Industrial	14.29	0.4	1.5
1600	Extractive	16.67	0.4	1.5
1700	Institutional	7.69	0.3	1.5
1800	Recreational	7.69	0	0
1900	Open Land	7.14	0	0
2100	Cropland and Pastureland	5.88	0	2.5
2400	Nurseries and Vineyards	5.88	0	2.5
2500	Specialty Farms	5.88	0	2.5
2600	Other Open Lands – Rural	7.14	0	2.5
3100	Herbaceous (Dry Prairie)	7.69	0	0
3200	Upland Shrub and Bushland	3.33	0	0
4100	Upland Coniferous Forest	2.22	0	0
4200	Upland Hardwood Forest	2.22	0	0
4300	Upland Mixed Forest	2.22	0	0
5100	Streams and Waterways	16.67	0	0
5200	Lakes	16.67	0	0
5300	Reservoirs	16.67	0	0
5400	Bays and Estuaries	16.67	0	0
6100	Wetland Hardwood Forest	2.22	0	0
6400	Vegetated Non-Forested Wetlands	3.33	0	0
7400	Disturbed Land	7.14	0	1.5
8100	Transportation	9.09	0.56	1.5
8200	Communications	7.14	0	1.5
8300	Utilities	7.14	0	1.5

8.4.1. DETENTION STORAGE

The future conditions detention storage map uses the same procedure as the existing conditions, with the assumption that all parcels with modified land use were applied the corresponding revised detention storage requirement based on Miami Dade County's Environmental Resource Permit Applicant's Handbook, Vol. II. **Table 8-4** shows the process that was used to modify detention storage values. However, since the only change to future land use is residential, meaning any new development is considered residential, the only revision to detention storage was to modify any land use change to allow for retention/infiltration of at least 3/8 inch per day during the storm event (SFWMD Applicant's Handbook, 2016).

Table 8-4. Detention Storage Values

FLUCCS Code	Land Use	Detention Storage (in)	Runoff Coefficient	ERP Based Det. Stor. (in)
1100	Residential, Low Density	0.1	0.075	0.375
1200	Residential, Medium Density	0.1	0.22	0.375
1300	Residential, High Density	0.1	0.45	0.375
1400	Commercial and Services	0.1	0.72	1.8
1500	Industrial	0.1	0.4	1.0
1600	Extractive	0.1	0.4	Unchanged
1700	Institutional	0.1	0.3	1
1800	Recreational	0.3	0	Unchanged
1900	Open Land	0.15	0	Unchanged
2100	Cropland and Pastureland	0.15	0	Unchanged
2200	Tree Crops	0.25	0	Unchanged
2300	Feeding Operations	0.25	0	Unchanged
2400	Nurseries and Vineyards	0.25	0	Unchanged
2500	Specialty Farms	0.25	0	Unchanged
2600	Other Open Lands - Rural	0.15	0	Unchanged
3100	Herbacous (Dry Prairie)	0.15	0	Unchanged
3200	Upland Shrub and Bushland	0.15	0	Unchanged
3300	Mixed Rangeland	0.15	0	Unchanged
4100	Upland Coniferous Forest	0.4	0	Unchanged
4200	Upland Hardwood Forest	0.4	0	Unchanged
4300	Upland Mixed Forest	0.4	0	Unchanged
4400	Tree Plantations	0.4	0	Unchanged
5100	Streams and Waterways	0	0	Unchanged
5200	Lakes	0	0	Unchanged

FLUCCS Code	Land Use	Detention Storage (in)	Runoff Coefficient	ERP Based Det. Stor. (in)
5300	Reservoirs	0	0	Unchanged
5400	Bays and Estuaries	0	0	Unchanged
5700	Ocean and Gulf	0	0	Unchanged
6100	Wetland Hardwood Forest	0.4	0	Unchanged
6400	Vegetated Non-Forested Wetlands	0.4	0	Unchanged
6500	Non-Vegetated Wetlands	0.4	0	Unchanged
7200	Sand other than Beaches	0.1	0	Unchanged
7400	Disturbed Land	0.1	0	Unchanged
8100	Transportation	0.1	0.56	1.4
8300	Utilities	0.1	0	Unchanged

8.5. SURFACE WATER

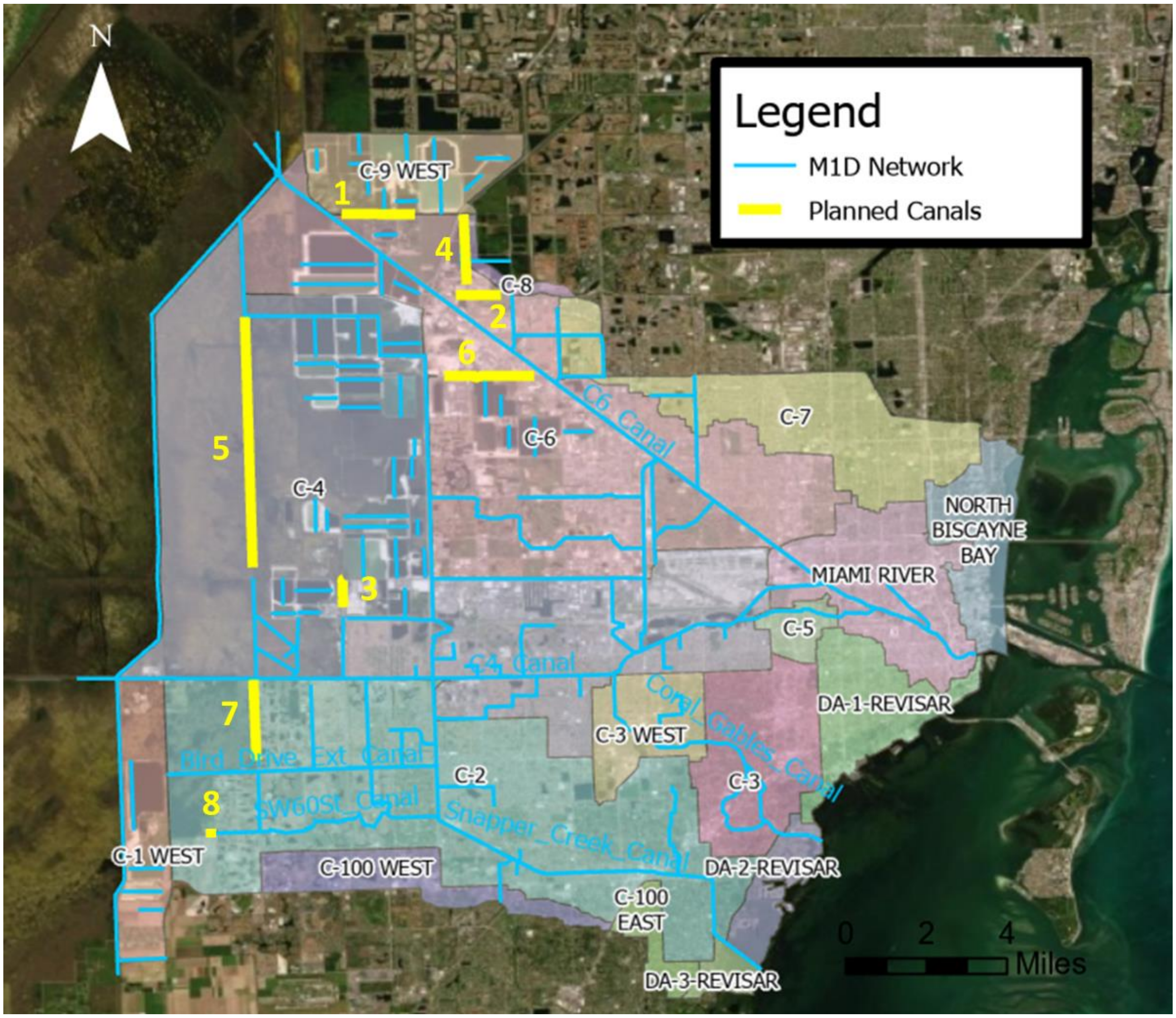
The FPLOS assessment was performed for the future conditions with a downstream boundary consistent with the Current Sea Level in 2019 plus a rise in sea level of +1 foot, +2 feet, and +3 feet. In addition, future conditions included changes to the canal system of the region, based on planned canal construction. To simulate these conditions, the 1D model for the current conditions design storm model was modified to represent changing conditions at each hydraulic boundary, as well as changes to the canal network.

8.5.1. FUTURE CANALS

Miami Dade County's canal database provides details on "Planned Canals" and other planned canal improvements. This information was used to extend existing canals or include new canals. **Figure 8-6** shows the planned canals in yellow with the reference number and the existing canals represented by the 1D network in blue. The following planned canals are listed below with the reference numbers from the figure:

1. Golden Glades Ditch
2. Grahams Dairy Canal
3. Mud Creek Canal Extension
4. NW 107th Avenue Canal
5. NW Wellfield Canal
6. Russian Colony Canal
7. SW 157th Avenue
8. SW 64th Street Canal Extension

Figure 8-6. Planned Canals from Miami Dade County



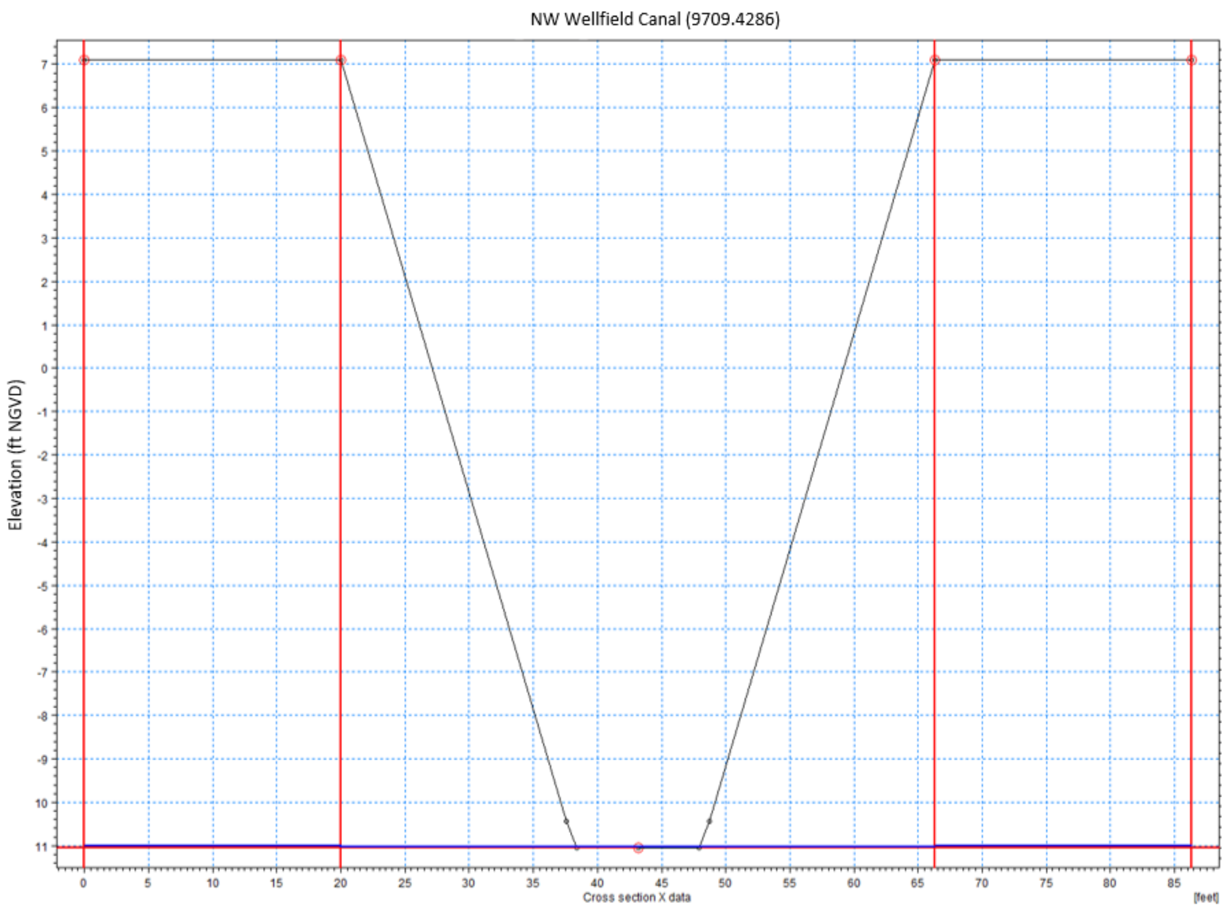
In the C2 Watershed, the SW 64th Street Canal Extension will extend the existing SW60St_Canal in the model to the new SW 170th Avenue Canal. In addition, the SW 157th Avenue Canal was added with a connection to Bird_Drive_Ext_Canal.

In the C4 Watershed, the NW Wellfield Canal will extend the existing Dade_Broward_Levee_Canal north to the Snapper_Creek_Ext_Canal. In addition, Mud Creek Canal Extension will extend the existing Mud_Creek_Canal north to NW 25th Street and the Northline Canal West Extension will extend the existing Northline_Canal west along NW 25th Street, adjacent to a Mine Lake (MinePit_08 in the model). The NW 118th Street Ditch will connect L31N to the NW Wellfield Canal.

In the C6 Watershed, the NW 107th Avenue Canal will extend from the northern model boundary south to the planned Grahams Dairy Canal, which will then go east to connect with the existing NW97Ave_Canal. In addition, the planned Russian Colony Canal and Golden Glades Ditch will connect directly to the C6 Canal.

Future canal geometries were generated using bottom elevations similar to the upstream, downstream, or surrounding canals and standard details as provided in the Miami Dade County Public Works Design Manual. Top of bank elevations were implemented to meet the new criteria defined in the 2021 Miami Dade County Water Control Map. An example of a generated future canal cross section is shown in **Figure 8-7**.

Figure 8-7. Example Future Proposed Canal Cross Section



8.5.2. SURFACE WATER BOUNDARIES

Surface water boundary conditions were modified for SLR using the following criteria:

- +1 foot SLR: For stages less than 1.89 feet-NAVD, add 1 foot (up to a maximum of 1.89 feet-NAVD).
- +2 feet SLR: For stages less than 2.89 feet-NAVD, add 2 feet (up to a maximum of 2.89 feet-NAVD).
- +3 feet SLR: For stages less than 3.89 feet-NAVD, add 3 feet (up to a maximum of 3.89 feet-NAVD).

Table 8-5 provides the modified boundary conditions at each open boundary condition for each SLR scenario. The table provides an identification (ID) for each boundary that can be located in **Figure 8-8**.

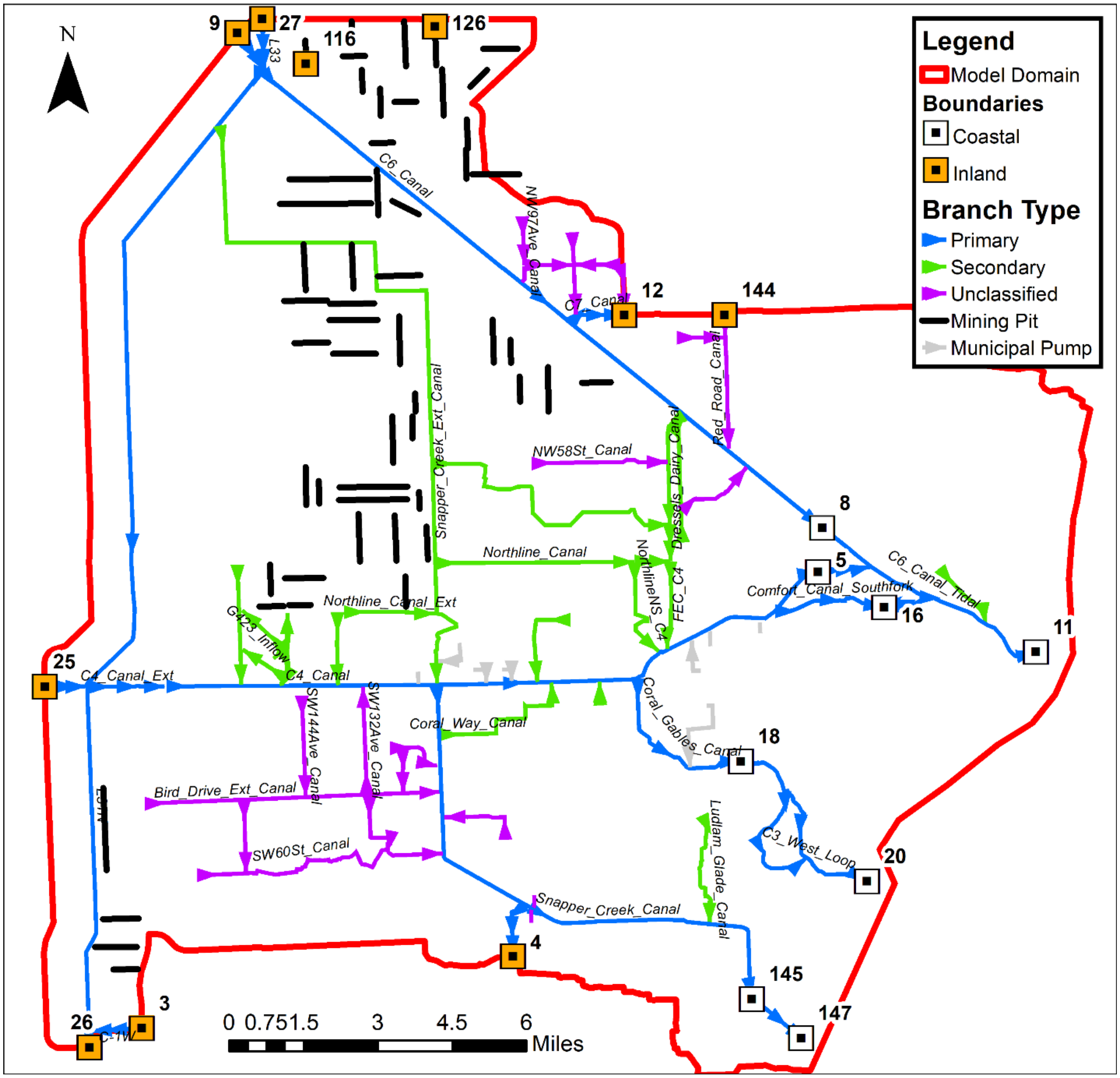
Table 8-5. Future SLR Boundary Conditions

ID**	Branch	Chainage (ft)	Elevation (ft-NAVD)			
			Current Value	SLR +1 ft	SLR +2 ft	SLR + 3 ft
3	C-1W	0	2.32	2.32	2.89	3.89
4	C2_Ext_South	6368	1.06	1.89	2.89	3.89
8	C6_Canal	76840	S26_T	S26 +1	S26 +2	S26 +3
9	C6_Canal_Ext	0	7.50	7.50	7.50	7.50
11	C6_Canal_Tidal	27618	C6 Canal Interp.	C6 Canal Interp. +1	C6 Canal Interp. +2	C6 Canal Interp. +3
12	C7_Canal	6072	1.07	1.89	2.89	3.89
18	Coral_Gables_Canal	18240	G93_T	G93 +1	G93 +2	G93 +3
20	Coral_Gables_Canal_Tidal	22430	Coral Gables Interp	Coral Gables Interp +1	Coral Gables Interp +2	Coral Gables Interp +3
25	L29_Borrow	0	6.69	6.69	6.69	6.69
26	L31N	111948	2.32	2.32	2.89	3.89
27	L33	0	4.83	4.83	4.83	4.83
116	MinePit_44	0	2.60	2.60	2.89	3.89
126	MinePit_49	0	1.40	1.89	2.89	3.89
144	Red_Road_Canal	0	1.02	1.89	2.89	3.89
145	Snapper_Creek_Canal	59908	S22_T	S22 +1	S22 +2	S22 +3
147	Snapper_Creek_Canal_Tidal	7269	S22_T	S22 +1	S22 +2	S22 +3

*Highlighted cells indicate boundaries experiencing changes due to SLR

The Boundary ID is the same number as set-up in the model files (files with the .bnd11 extension) and can be found in **Figure 8-8.

Figure 8-8. Surface Water Boundaries in MIKE 1D Network



The tidal boundaries in the C4 Canal, C6 Canal, Coral Gables Canal, and Snapper Creek Canal, downstream of the tidal structures are shown in **Figure 7-3** through **Figure 7-7**. In addition, the interpolated tidal boundaries located at the mouth of the C6 Canal and Coral Gables Canal are shown in **Figure 7-14** and **Figure 7-15**, respectively. These boundary conditions were modified by adding 1 foot, 2 feet, and 3 feet to each timestep.

8.5.3. INITIAL CONDITIONS FOR SURFACE WATER

For the existing conditions scenarios, the design storm events use the same initial conditions for surface water as the September 2017 event, which uses measured data from September 1, 2017. For the future conditions, these values were modified using the same logic as described in the development of the groundwater initial conditions maps. For future SLR conditions, the surface water initial conditions were modified using the following protocol:

- +1 foot SLR: For stages less than 1.89 feet-NAVD, add 1 feet (up to a maximum of 1.89 feet-NAVD).
- +2 feet SLR: For stages less than 2.89 feet-NAVD, add 2 feet (up to a maximum of 2.89 feet-NAVD).
- +3 feet SLR: For stages less than 3.89 feet-NAVD, add 3 feet (up to a maximum of 3.89 feet-NAVD).

8.6. GROUNDWATER

8.6.1. INITIAL CONDITIONS AND BOUNDARY CONDITIONS

For the current conditions simulations, the USGS seasonal high-water table (USGS, 2016) was used to establish the static groundwater head at the inland boundaries. For the USGS data, contours from the reference map were digitized and rasterized into a format matching the computation grid for the MIKE SHE model of the Saturated Zone.

For future conditions, the initial groundwater conditions were developed using the mean high-water table (MHWT) modified for SLR by using Miami Dade County's future groundwater predictions. The following data was obtained from the Miami Dade Open Data Hub:

- **Groundwater Level Baseline May 2040:** This raster provides the average groundwater elevations in NAVD for the month of May, based on the results of the U.S. Geological Survey groundwater model for Miami Dade – Urban Miami Dade (UMD), used to predict groundwater levels for year 2040, without sea level rise for comparison purposes (Miami Dade, 2021). This dataset assumes zero sea-level rise (or no change) for the period 2011-2040 and uses the sea-level for the year 2009 (-0.9 feet mean sea-level NAVD).
- **Groundwater Level May 2040:** This raster provides the average groundwater elevations in NAVD for the month of May, based on the results of the U.S. Geological Survey groundwater model for Miami Dade – Urban Miami Dade (UMD), used to predict groundwater levels for year 2040, considering sea level rise above the baseline conditions (Miami Dade, 2021). This data uses NRCIII

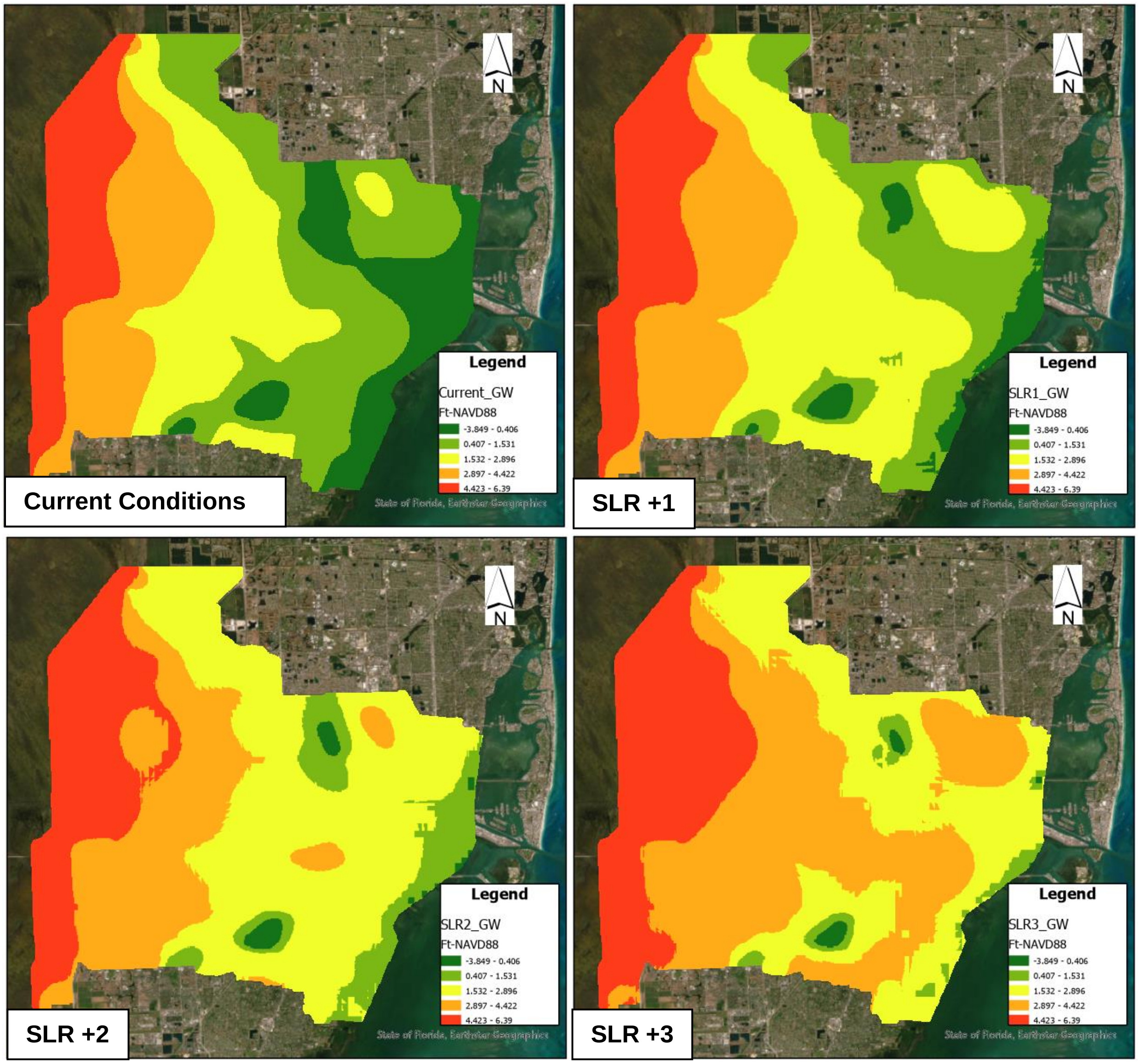
forecast, which assumes a 1-foot sea-level rise increase, from a year 2009 (-)0.9 feet mean sea-level NAVD to a 2040 0.1 feet.

To create the groundwater initial conditions for all SLR conditions, the following steps were taken:

1. Miami Dade County's Groundwater Level Baseline May 2040 raster was subtracted from the Groundwater Level May 2040 raster to obtain the change in groundwater levels for 2040, which assumed 1 foot of SLR.
2. The change in groundwater levels raster was the was multiplied by 2 and 3 to get the change in groundwater elevations for 2 and 3 feet of SLR, respectively.
3. The raster for each SLR scenario was added to the current groundwater initial conditions (USGS MHWT) to obtain the groundwater initial conditions for future scenarios SLR1, SLR2, and SLR3.

The inland groundwater initial conditions were setup in the model as a fixed (use initial value) for each scenario. **Figure 8-9** shows the groundwater initial conditions for current conditions and future conditions (SLR1, SLR2, SLR3).

Figure 8-9. Groundwater Initial Conditions



8.6.2. PUBLIC WATER SUPPLY PUMPED WITHDRAWALS

The following primary public water supply wellfields are owned and operated by Miami Dade County Water and Sewer Department (MD WASD) within the model domain:

- Alexander Orr Wellfield
- Hialeah Wellfield
- Miami Springs Lower Wellfield
- Miami Springs Upper Wellfield
- Northwest Wellfield
- Preston Wellfield
- Snapper Creek Wellfield
- Southwest Wellfield
- West Wellfield

MD WASD provided recorded pumping rate data for each well, which was averaged for September months from 2017 through 2020 for the current conditions design storm simulations. These September average pumping rates were applied at a constant rate throughout the simulation.

The 2020 Water Use Permit Modification for WUP # 13-00017-W, prepared by MD WASD has determined total water supply demand in 2020 for the County to be 332.9 MGD and projected that the demand in 2065 will increase to 479.1 MGD. This represents an increase in water demand of approximately 44%. The total September average was 321 MGD for all wellfields, a 44% increase in the September average pumping rate equals 462 MGD.

To account for this increase in demand in the water supply wells, an increase in production at each of the wells of 44% was introduced, up to the capacity of the individual pump. However, at Preston Wellfield the September average pumping rate was already near the total permitted capacity, so no increase was proposed at this wellfield.

To make up for the remaining demand deficit, wells that pump a lower amount were increased more than 44% at all wellfields except for Preston Wellfield. AO9 pumped only 1% of its total capacity for September (71 GPM out of a total 7500 GPM), so the proposed future pumping rate is 3450 GPM. The existing pumping, pumping capacity, and proposed future pumping are shown in **Table 8-6**.

Table 8-6. Pumping Rates for Current and Future Conditions at all Wellfields

Wellfield	PWS Well ID	Sept. Avg. 2017-2020 (MGD)	Sept. Avg. 2017-2020 (GPM)	Pump Capacity (GPM)	Percent Unused Capacity	Assumed Future Use (GPM)	Assumed Future Use (MGD)
Alexander Orr Permitted Capacity 74.4 MGD	AO1	5.52	3833	4170	8%	4170	6.00
	AO2	6.01	4172	4170	0%	4170	6.00
	AO3	5.21	3621	4170	13%	4170	6.00
	AO4	5.76	4002	4170	4%	4170	6.00
	AO5	4.30	2987	4170	28%	4170	6.00
	AO6	5.85	4066	4170	2%	4170	6.00
	AO7	5.78	4014	4170	4%	4170	6.00
	AO8	10.63	7381	7500	2%	7500	10.80
	AO9	0.10	71	7500	99%	3450	4.97
	AO10	10.63	7380	7500	2%	7500	10.80
	Total	**59.80**	**41526**	**51690**	**1.62**	**47640**	**68.60**
Hialeah Permitted Capacity 12.54 MGD	HIA11	1.57	1090	2500	56%	1150	1.66
	HIA12	1.20	837	3800	78%	1748	2.52
	HIA13	0.79	551	2500	78%	1150	1.66
	Total	**3.57**	**2478**	**8800**	**2.12**	**4048**	**5.83**
Miami Springs Lower Permitted Capacity 35.7 MGD	MS1	0.55	381	3800	90%	1748	2.52
	MS2	0.96	666	2500	73%	1150	1.66
	MS3	0.81	560	2500	78%	1150	1.66
	MS4	1.70	1180	2500	53%	1698	2.45
	MS5	1.47	1019	2500	59%	1150	1.66
	MS6	1.37	955	2500	62%	1150	1.66
	MS7	0.45	309	2500	88%	1150	1.66
	MS8	0.74	511	2500	80%	1150	1.66
	Total	**8.04**	**5581**	**21300**	**5.82**	**10346**	**14.90**
Miami Springs Upper Permitted Capacity 43.6 MGD	MS9	1.37	950	2500	62%	1150	1.66
	MS10	1.98	1373	2500	45%	1975	2.84
	MS14	3.19	2218	4170	47%	3192	4.60
	MS15	2.95	2050	2500	18%	2500	3.60
	MS16	2.67	1852	2500	26%	2500	3.60
	MS17	1.47	1022	2500	59%	1150	1.66
	MS18	2.89	2004	2500	20%	2500	3.60
	MS19	2.65	1839	2500	26%	2500	3.60

Wellfield	PWS Well ID	Sept. Avg. 2017-2020 (MGD)	Sept. Avg. 2017-2020 (GPM)	Pump Capacity (GPM)	Percent Unused Capacity	Assumed Future Use (GPM)	Assumed Future Use (MGD)
	MS20	2.27	1579	2500	37%	2500	3.60
	MS21	1.83	1272	2500	49%	1831	2.64
	MS22	2.09	1452	2500	42%	2500	3.60
	MS23	1.63	1131	2500	55%	1150	1.66
	Total	**26.99**	**18742**	**31670**	**4.86**	**25448**	**36.65**
Preston Permitted Capacity 53.3 MGD	PP1	6.62	4597	6940	34%	4597	6.62
	PP2	8.88	6166	6940	11%	6166	8.88
	PP3	0.00	0	6940	100%	0	0.00
	PP4	9.19	6382	6940	8%	6382	9.19
	PP5	8.59	5968	6940	14%	5968	8.59
	PP6	8.53	5925	6940	15%	5925	8.53
	PP7	9.30	6459	6940	7%	6459	9.30
	Total	**51.12**	**35497**	**48580.00**	**1.89**	**35497**	**51.12**
Northwest Permitted Capacity 149.4 MGD	NW1	2.58	1792	10416.67	83%	4792	6.90
	NW2	2.47	1716	10416.67	84%	4792	6.90
	NW3	4.33	3009	10416.67	71%	4792	6.90
	NW4	0.45	312	10416.67	97%	4792	6.90
	NW5	1.06	737	10416.67	93%	4792	6.90
	NW6	1.27	879	10416.67	92%	4792	6.90
	NW7	0.61	426	10416.67	96%	4792	6.90
	NW8	8.91	6187	10416.67	41%	10417	15.00
	NW9	5.36	3720	10416.67	64%	4792	6.90
	NW10	0.72	502	10416.67	95%	4792	6.90
	NW11	2.36	1641	10420	84%	4793	6.90
	NW12	3.67	2549	10420	76%	4793	6.90
	NW13	1.64	1142	10420	89%	4793	6.90
	NW14	4.29	2981	10420	71%	4793	6.90
	NW15	3.36	2334	10420	78%	4793	6.90
	Total	**43.09**	**29926.62**	**156267**	**12.13**	**77508**	**111.61**
Snapper Creek Permitted Capacity 40 MGD	SC21	5.81	4035	8300	51%	5806	8.36
	SC22	0.02	14	8300	100%	3818	5.50
	SC23	6.47	4490	8300	46%	6461	9.30
	SC24	4.96	3442	8300	59%	3818	5.50
	Total	**17.25**	**11981**	**33200.00**	**2.56**	**19903**	**28.66**

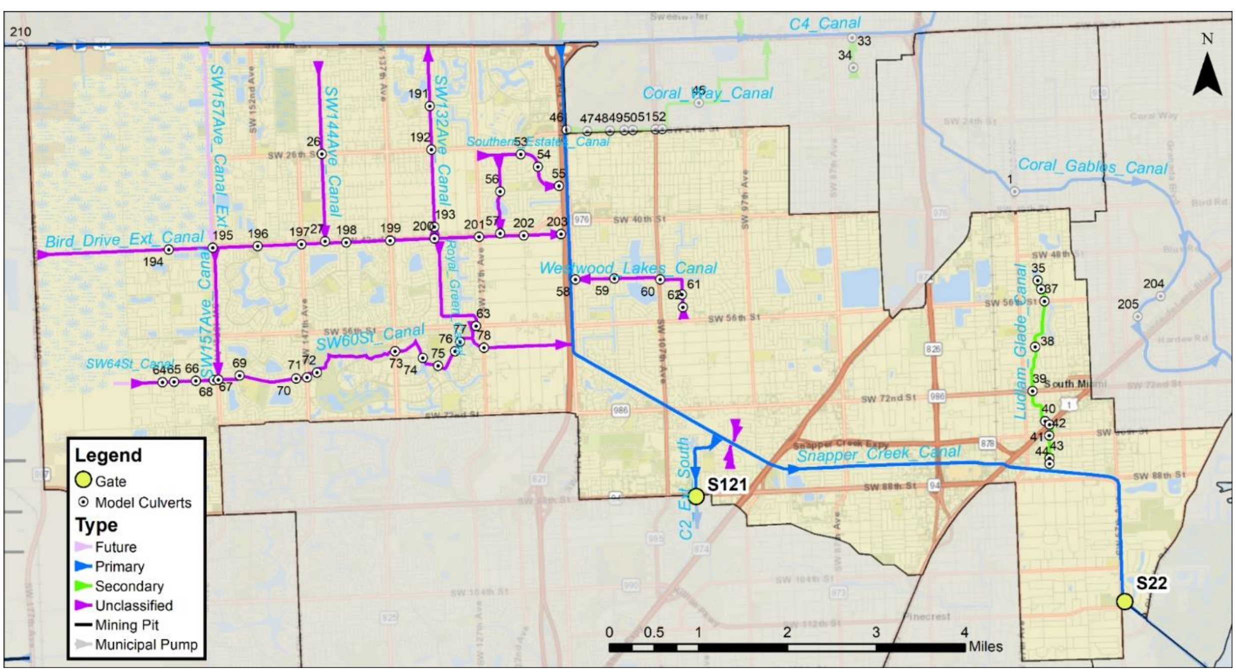
9. FLOOD PROTECTION LEVEL OF SERVICE ASSESSMENT – CURRENT AND FUTURE CONDITIONS

This section documents the results of the FPLOS Assessment for the Current and Future Sea Level Rise Conditions for the C2, C3W, C4, C5, and C6 Watersheds. The section is separated by the watershed, demonstrating the results of both the current and future conditions for each Performance Metric (1 through 6) as described in **Section 6**.

9.1. C2 WATERSHED FLOOD PROTECTION LEVEL OF SERVICE

The C2 Watershed consists of the area south of the C4 Canal and east of the L31N Canal. The primary canal is the Snapper Creek Canal (or C2 Canal), and the secondary canals include Ludlam Glade Canal and Bird Drive Extension Canal. Major water control structures in the C2 Watershed are S121 and S22. S121 controls flows to the C-100 West Watershed and is typically closed during severe storm events and throughout the wet season of June through November. S22 controls flows out of the C2 Watershed to tide and is operated during the wet season to maintain the low range water elevations of (-) 0.04 to 0.96 ft-NAVD (1.5 to 2.5 ft-NGVD), under current conditions. **Figure 9-1** provides a map of the C2 Watershed and the branches that are included in the model, as well as the culverts with the number that corresponds to the numbering in the model.

Figure 9-1. Map of the C2 Watershed



9.1.1. C2 – PM #1 MAXIMUM STAGE IN PRIMARY CANALS

The maximum stage in the primary canals was extracted from the results of each design storm simulation for the C2 Watershed. **Table 9-1** provides the low chord information for each bridge in the Snapper Creek Canal, as well as the peak stage at the nearest H-point (where canal stages are calculated in the model). Bridge low chord and bridge top elevations were established from the C4 HEC RAS model used for the Miami Dade SWMP (SFWMD, 2011). For current conditions, stages in Snapper Creek Canal exceed the low-chord elevation at two bridges: the SW 57th Avenue bridge and at the Bird Drive bridge. In addition to these bridges, for the SLR +2 feet simulation, the SW 77th Avenue bridge low chord is exceeded, and for the SLR +3 feet simulation the Railroad west of SR874 is exceeded. Under no conditions are the bridge top elevations exceeded.

Figure 9-2 shows the stages at the SW 57th Avenue bridge in Snapper Creek Canal and the elevations of the bridge top and bottoms for each SLR simulation for the 100-year storm. The bridge bottom is exceeded for all SLR conditions for the 100-year storm, and for the SLR3 simulation, stages reach the bridge bottom for two weeks with each high tide.

Table 9-2 provides the culvert information with the estimated crown of road at each culvert location in the C2 Watershed, as well as the peak stage at the nearest H-point. The crown of road was estimated from LiDAR data by determining the elevation at the intersection of the center line of the canal and the top of the roadbed. Stages which overtop the estimated crown of road are highlighted in orange. In addition, a culvert number is provided in the first column, which corresponds to the culverts numbered in **Figure 9-1**

For the C2 Watershed, 13 of the 51 culvert locations represented in the model experience road overtopping during a 100-year design storm event for the current conditions, this increases by a few locations as sea level rise increases. The number of locations that are exceeded during each storm event for each SLR condition are evaluated in **Figure 9-3**. While the crown of road is exceeded at five (5) culvert locations during the 25-year storm event for the current conditions simulation, this doubles for the SLR3 simulation. Additionally, while there are no locations exceeded for the 5-year storm under current conditions, this increases to six (6) during the SLR3 simulation. Areas of concern include the Ludlam Glade Canal, Westwood Lakes Canal, and the Southern Estates Canal, which have low crown of roads over the culverts, as interpreted from the LiDAR data.

Table 9-1. Bridge Low Chord and Peak Stage for the Snapper Creek Canal

Location Description	Low Chord	Bridge Top	Current Conditions				SLR1				SLR2				SLR3			
			Elevation (ft-NAVD)															
			100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Turnpike entrance from SW 8th Street	12.16	16.18	5.65	5.08	4.69	4.37	6.06	5.31	5.00	4.73	6.46	5.81	5.32	5.00	6.81	6.25	5.87	5.60
Turnpike north bound exit to SW 9th Street	7.26	10.76	5.67	5.15	4.74	4.42	6.04	5.39	5.07	4.77	6.43	5.80	5.36	5.06	6.78	6.22	5.91	5.63
north bound entrance to Turnpike	9.57	13.93	5.78	5.19	4.75	4.44	6.02	5.48	5.09	4.77	6.41	5.80	5.39	5.10	6.75	6.21	5.90	5.64
SW 40th Ave (Bird Road)	5.57	7.46	5.78	5.19	4.75	4.44	6.02	5.48	5.09	4.78	6.41	5.80	5.38	5.09	6.75	6.21	5.90	5.64
SW 56th Ave (Miller Drive)	8.37	12.06	5.82	5.20	4.73	4.41	6.08	5.50	5.10	4.75	6.38	5.82	5.41	5.10	6.70	6.21	5.89	5.63
SW 117th Ave	8.96	12.86	5.78	5.16	4.69	4.35	6.05	5.48	5.07	4.73	6.34	5.79	5.39	5.07	6.66	6.18	5.87	5.60
SW 107th Ave	8.06	12.26	5.73	5.06	4.60	4.28	6.02	5.45	5.01	4.67	6.29	5.76	5.36	5.04	6.62	6.15	5.84	5.57
SW 72nd (Sunset) Ave	7.08	10.13	5.68	5.00	4.53	4.21	5.98	5.40	4.96	4.62	6.24	5.72	5.32	5.00	6.58	6.12	5.81	5.54
SW 99th Ave	8.06	10.21	5.67	4.97	4.49	4.15	5.98	5.39	4.94	4.60	6.23	5.72	5.32	4.99	6.57	6.11	5.81	5.53
R/R west of SR 874 Express Way	6.43	9.83	5.61	4.88	4.40	4.05	5.94	5.35	4.88	4.54	6.19	5.68	5.28	4.95	6.55	6.08	5.78	5.50
SR 874	10.90	13.90	5.58	4.85	4.36	4.02	5.92	5.32	4.85	4.51	6.17	5.67	5.26	4.93	6.53	6.07	5.77	5.49
SW87 Ave	6.96	8.68	5.52	4.75	4.26	3.92	5.89	5.26	4.80	4.45	6.13	5.65	5.22	4.89	6.51	6.05	5.76	5.47
SW 79th (Kings Creek) Ave	6.68	8.66	5.46	4.67	4.16	3.82	5.87	5.21	4.75	4.40	6.11	5.63	5.19	4.86	6.49	6.03	5.75	5.45
SW 77th Ave	6.07	10.56	5.42	4.63	4.10	3.76	5.85	5.17	4.71	4.36	6.11	5.61	5.17	4.83	6.48	6.02	5.74	5.44
Palmetto Express Way + Ramp (combined)	7.10	13.76	5.38	4.58	4.04	3.70	5.83	5.14	4.68	4.32	6.18	5.59	5.15	4.81	6.65	6.07	5.73	5.49
behind Dadeland Mall	7.46	8.11	5.38	4.57	4.03	3.69	5.82	5.13	4.67	4.32	6.17	5.59	5.15	4.81	6.65	6.06	5.73	5.50
SW 72nd Ave	7.51	12.76	5.34	4.53	3.98	3.64	5.80	5.11	4.64	4.29	6.17	5.58	5.13	4.79	6.65	6.06	5.71	5.48

Location Description	Low Chord	Bridge Top	Current Conditions				SLR1				SLR2				SLR3			
			Elevation (ft-NAVD)															
			100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
SW 70th Ave	8.29	13.60	5.30	4.48	3.93	3.59	5.78	5.07	4.61	4.26	6.15	5.56	5.11	4.77	6.64	6.05	5.70	5.46
US1 + Metro Rail (combined)	9.11	11.96	5.20	4.42	3.80	3.46	5.75	5.04	4.58	4.23	6.14	5.54	5.09	4.76	6.63	6.03	5.69	5.44
SW 67th (Ludlam) Ave	8.78	13.08	5.19	4.29	3.78	3.45	5.72	5.00	4.55	4.19	6.12	5.53	5.06	4.74	6.62	6.02	5.67	5.42
SW 88th Street and SW 57th Ave	4.16	8.89	5.03	4.17	3.61	3.30	5.67	4.92	4.46	4.09	6.09	5.48	5.01	4.69	6.59	6.00	5.65	5.41

*Highlighted cells indicate the stages exceed the bridge low chord

Table 9-2. Estimated Culvert Crown of Road and Peak Stage for the C2 Watershed

Culvert Nunmber	Intersection	Top of Culvert	Estimated Crown of Road	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
SW144Ave_Canal																			
26	**SW 26th St**	3.80	6.5	6.75	6.04	5.56	5.20	6.66	5.99	5.57	5.25	6.85	6.20	5.76	5.42	7.08	6.47	6.07	5.78
27	**SW 42nd St**	5.02	8.9	6.61	5.88	5.39	5.02	6.40	5.68	5.27	4.95	6.63	5.95	5.52	5.20	6.94	6.30	5.91	5.63
Ludlam_Glade_Canal																			
35	**SW 52nd St**	4.44	7.1	6.79	5.48	4.62	4.01	7.09	6.12	5.26	4.71	7.36	6.63	5.81	5.29	7.60	6.88	6.43	5.93
36	**SW 64th Ave**	1.92	5.9	6.67	5.42	4.59	3.97	7.04	6.07	5.23	4.69	7.32	6.53	5.77	5.26	7.58	6.85	6.38	5.89
37	**SW 56th St**	2.39	6.1	6.60	5.39	4.56	3.93	7.01	6.02	5.21	4.67	7.30	6.45	5.74	5.24	7.55	6.83	6.35	5.86
38	**SW 64th St**	4.17	6.9	6.58	5.36	4.53	3.91	6.98	6.00	5.19	4.65	7.27	6.43	5.72	5.22	7.53	6.81	6.33	5.84
39	**SW 72nd St**	2.61	8.6	6.55	5.34	4.50	3.89	6.94	5.98	5.17	4.62	7.20	6.42	5.70	5.21	7.48	6.79	6.32	5.82
40	**SW 64th Ct**	2.56	6.4	6.49	5.30	4.47	3.87	6.89	5.94	5.13	4.60	7.14	6.40	5.68	5.18	7.41	6.75	6.29	5.80
41	**S Dixie Hwy**	2.75	7.2	6.46	5.26	4.44	3.85	6.85	5.91	5.11	4.57	7.09	6.37	5.65	5.16	7.38	6.72	6.26	5.78
42	**SW 80th St**	0.46	7.2	5.49	4.51	3.88	3.48	5.98	5.18	4.65	4.25	6.39	5.70	5.18	4.82	6.82	6.17	5.79	5.50
43	**SW 84th St**	2.86	6.1	5.34	4.41	3.80	3.45	5.86	5.09	4.59	4.20	6.27	5.61	5.12	4.78	6.73	6.10	5.72	5.46
44	**SW 85th St**	4.71	7.0	5.19	4.32	3.74	3.41	5.74	5.01	4.54	4.17	6.16	5.54	5.06	4.74	6.64	6.03	5.68	5.43
Southern_Estates_Canal																			
53	**SW 122nd Ave**	1.51	5.0	6.24	5.49	4.98	4.65	6.41	5.73	5.26	4.92	6.59	6.00	5.52	5.19	6.83	6.32	5.96	5.67
54	**SW 31st St**	2.13	5.2	6.18	5.44	4.96	4.63	6.36	5.68	5.23	4.90	6.56	5.96	5.49	5.17	6.82	6.28	5.94	5.66
55	**FL Turnpike**	2.55	8.0	5.75	5.17	4.75	4.44	6.03	5.45	5.07	4.76	6.42	5.79	5.36	5.09	6.77	6.21	5.90	5.64
Southern_Estates_Canal_S																			
56	**SW 34th St**	11.04	11.5	6.24	5.48	4.98	4.65	6.41	5.73	5.26	4.92	6.59	6.00	5.51	5.19	6.83	6.32	5.95	5.67
57	**SW 42nd St**	2.01	6.0	6.05	5.37	4.90	4.57	6.25	5.60	5.18	4.86	6.48	5.88	5.46	5.15	6.79	6.25	5.92	5.64

Culvert Number	Intersection	Top of Culvert	Estimated Crown of Road	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (FT-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Westwood_Lakes_Canal																			
58	**SW 117th Ave**	1.785	7.0	5.83	5.21	4.75	4.43	6.09	5.51	5.12	4.78	6.39	5.82	5.41	5.11	6.71	6.22	5.89	5.64
59	**SW 112th Ave**	1.775	6.4	6.33	5.66	5.11	4.79	6.56	5.87	5.42	5.08	6.74	6.13	5.66	5.32	6.93	6.42	6.02	5.74
60	**SW 107th Ave**	0.985	5.9	6.77	6.32	5.87	5.45	6.88	6.43	6.12	5.69	6.98	6.52	6.23	5.88	7.11	6.69	6.41	6.21
61	**SW 51st St**	0.995	5.7	6.81	6.39	5.92	5.50	6.91	6.48	6.17	5.74	7.01	6.56	6.29	5.94	7.12	6.72	6.45	6.26
62	**SW 53rd St**	2.408	5.6	6.82	6.39	5.93	5.51	6.91	6.48	6.18	5.75	7.01	6.56	6.30	5.94	7.12	6.72	6.46	6.26
Royal_Green_Canal																			
63	**SW 56th St**	3.39	7.4	6.07	5.37	4.91	4.57	6.25	5.62	5.20	4.86	6.50	5.90	5.47	5.16	6.79	6.25	5.91	5.63
SW60St_Canal																			
64	**SW 163rd Ct**	4.56	8.0	6.95	6.29	5.93	5.68	6.87	6.32	6.00	5.78	6.99	6.41	6.05	5.83	7.03	6.57	6.19	5.94
65	**SW 162nd Ave**	5.31	6.6	6.97	6.29	5.91	5.67	6.84	6.31	5.99	5.76	6.97	6.39	6.04	5.81	7.04	6.56	6.17	5.93
66	**SW 159th Ave**	5.01	7.3	6.94	6.20	5.75	5.47	6.68	6.10	5.78	5.55	6.82	6.23	5.85	5.62	7.01	6.43	6.00	5.77
67	**West of SW 157th Ave**	5.33	6.2	6.93	6.18	5.72	5.45	6.66	6.07	5.75	5.51	6.80	6.21	5.82	5.59	7.00	6.41	5.98	5.74
68	**SW 157th Ave**	5.09	7.3	6.92	6.16	5.66	5.37	6.60	5.98	5.63	5.38	6.75	6.14	5.76	5.47	6.99	6.37	5.93	5.67
70	**SW 148th Ave**	7.94	12.0	6.90	6.11	5.57	5.18	6.48	5.75	5.30	4.96	6.68	5.98	5.54	5.20	7.00	6.31	5.81	5.53
71	**SW 147th Ave**	2.88	8.2	6.91	6.11	5.57	5.18	6.50	5.78	5.33	4.98	6.71	6.01	5.56	5.23	7.02	6.33	5.84	5.56
72	**SW 146th Ave**	1.93	6.9	6.91	6.11	5.57	5.17	6.51	5.80	5.34	4.99	6.72	6.02	5.57	5.23	7.03	6.35	5.87	5.58
73	**SW 137th Ave**	1.79	8.1	6.89	6.09	5.55	5.15	6.59	5.85	5.36	5.01	6.79	6.10	5.60	5.25	7.06	6.39	5.98	5.66
74	**SW 62nd St**	3.27	9.6	6.85	6.04	5.47	5.08	6.51	5.79	5.32	4.97	6.75	6.07	5.57	5.23	7.01	6.36	5.98	5.66
75	**SW 132nd Ave**	2.77	7.0	6.55	5.77	5.23	4.84	6.47	5.76	5.30	4.95	6.72	6.04	5.56	5.22	6.98	6.35	5.97	5.66
76	**SW 129th Ct**	3.90	10.4	6.47	5.70	5.17	4.80	6.30	5.66	5.25	4.90	6.56	5.95	5.50	5.18	6.84	6.28	5.93	5.64
77	**SW 128th Ave**	6.36	10.5	6.13	5.44	4.96	4.62	6.27	5.64	5.23	4.88	6.52	5.92	5.49	5.17	6.82	6.26	5.92	5.63
78	**SW 127th Ave**	2.56	6.9	6.10	5.40	4.94	4.59	6.24	5.62	5.20	4.86	6.50	5.90	5.47	5.16	6.79	6.24	5.91	5.62

Culvert Number	Intersection	Top of Culvert	Estimated Crown of Road	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
SW132Ave_Canal																			
191	**SW 18th St**	3.23	9.5	6.18	5.36	4.84	4.57	6.39	5.60	5.10	4.78	6.64	5.88	5.43	5.05	6.86	6.24	5.84	5.50
192	**SW 26th St**	3.09	6.6	6.39	5.53	4.99	4.67	6.59	5.77	5.25	4.90	6.81	6.06	5.54	5.17	6.92	6.37	5.95	5.61
193	**SW 39th Ter**	3.04	7.3	6.15	5.45	4.98	4.65	6.32	5.63	5.21	4.89	6.55	5.90	5.48	5.17	6.83	6.25	5.92	5.64
Bird_Drive_Ext_Canal																			
194	**SW 162 Ave**	4.04	7.2	6.90	6.09	5.54	5.15	6.23	5.52	5.11	4.81	6.58	5.77	5.36	5.03	6.92	6.23	5.65	5.36
195	**SW 157th Ave**	4.16	7.9	6.89	6.09	5.54	5.15	6.23	5.52	5.11	4.81	6.58	5.77	5.36	5.03	6.92	6.23	5.65	5.36
196	**SW 152nd Ave**	7.12	9.5	6.89	6.10	5.55	5.16	6.23	5.53	5.12	4.81	6.58	5.77	5.37	5.05	6.92	6.24	5.66	5.37
197	**SW 147th Ave**	3.62	7.8	6.87	6.08	5.53	5.15	6.26	5.55	5.14	4.83	6.60	5.79	5.39	5.07	6.93	6.25	5.68	5.40
198	**SW 142nd Ave**	2.96	6.9	6.61	5.88	5.39	5.02	6.40	5.68	5.27	4.95	6.63	5.95	5.52	5.20	6.94	6.30	5.91	5.63
199	**SW 137th Ave**	4.34	7.5	6.31	5.57	5.10	4.76	6.38	5.67	5.25	4.92	6.60	5.93	5.51	5.19	6.88	6.27	5.93	5.64
200	**SW 132nd Ave**	3.77	8.1	6.24	5.52	5.04	4.71	6.36	5.66	5.24	4.91	6.59	5.92	5.49	5.18	6.86	6.27	5.92	5.64
201	**SW 127th Ave**	3.57	7.2	6.14	5.44	4.96	4.63	6.32	5.64	5.21	4.88	6.54	5.91	5.48	5.16	6.83	6.26	5.93	5.64
202	**SW 122nd Ave**	6.69	8.6	6.04	5.36	4.89	4.57	6.24	5.59	5.17	4.85	6.48	5.88	5.46	5.16	6.79	6.25	5.92	5.65
203	**SW 118th Ave**	4.63	7.6	5.94	5.30	4.83	4.51	6.17	5.55	5.14	4.82	6.44	5.85	5.44	5.13	6.76	6.24	5.91	5.64

*Highlighted cells indicate the stages exceed the estimated crown of road.

Figure 9-2. Stages at SW 57th Avenue Bridge in Snapper Creek Canal for the 100-year Storm Event.

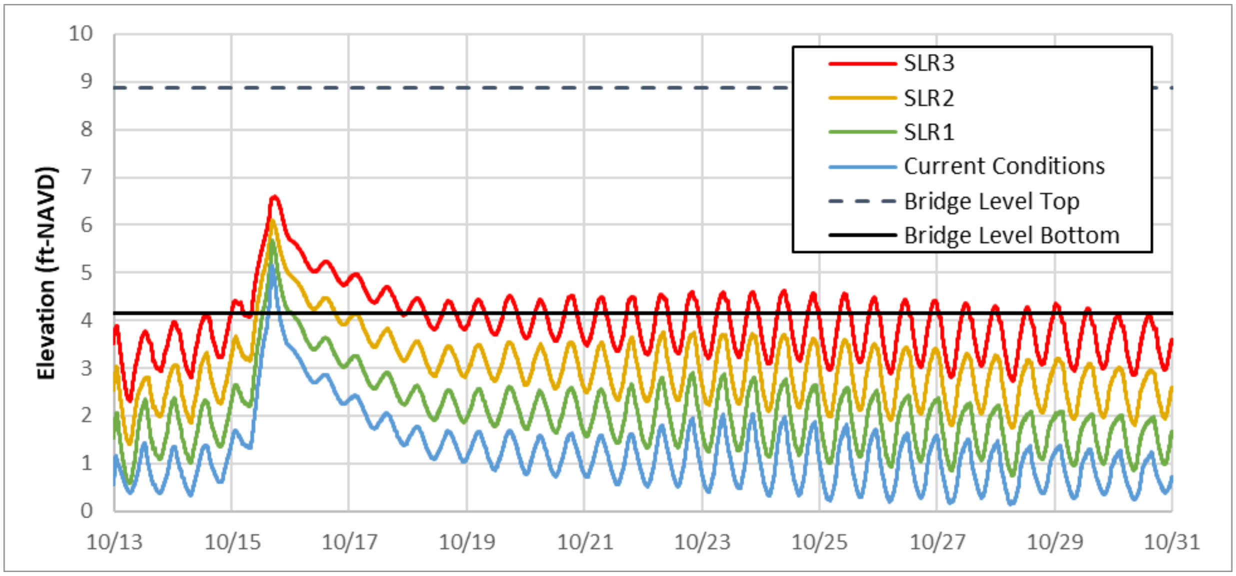
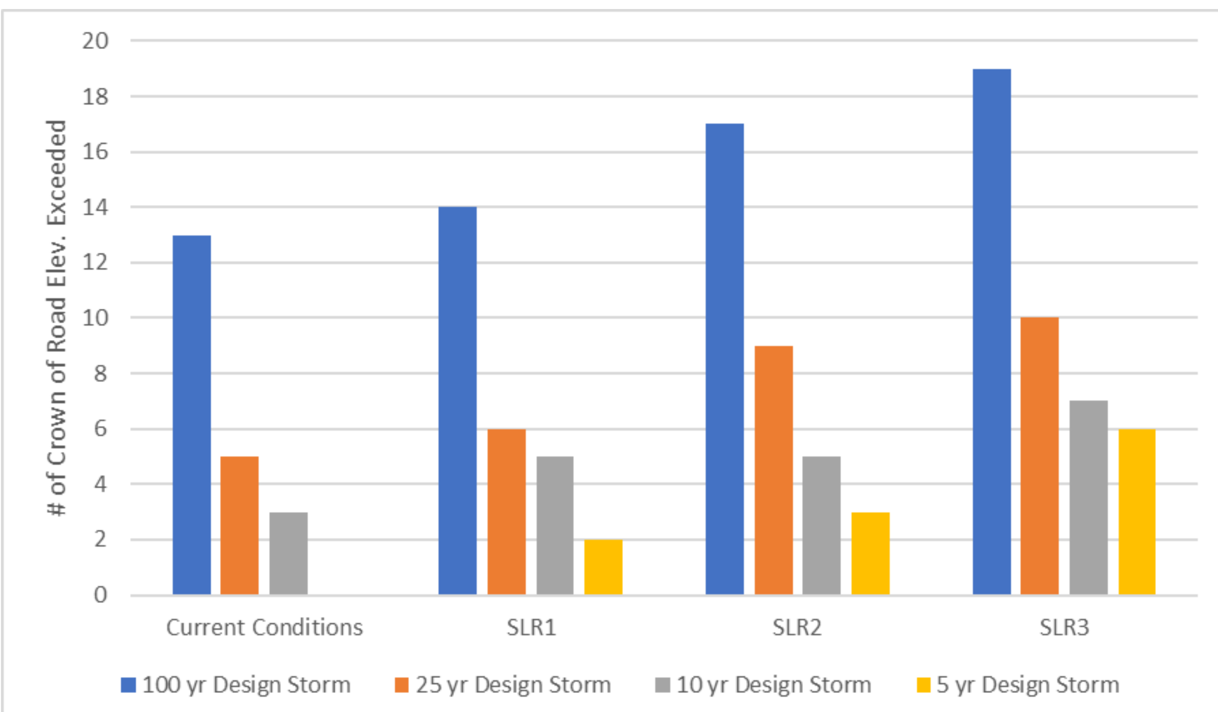


Figure 9-3. Number of Culvert Locations Where the Crown of Road is Exceeded within the C2 Watershed



Canal top of bank was established in the model from cross-section information from previous modeling efforts, including the HEC-RAS models (SFWMD, 2015), and converted to ft-NAVD. Full discussion of the development of these cross-sections is provided in **Section 3.3.1**. Top of bank elevations were extracted from the model and the miles of the evaluated canal segment that is overtopped by the design storm for the Snapper Creek (C2) Canal was estimated upstream of the tidal control structure S22, as summarized in **Table 9-3**.

Table 9-3. Estimated Percentage and Miles of Bank Overtopped per Design Storm

WATERSHED	SIMULATION	100-YEAR	25-YEAR	10-YEAR	5-YEAR
C2 (%)	Current Conditions	21%	7%	3%	0%
	SLR1	41%	18%	10%	5%
	SLR2	45%	23%	18%	13%
	SLR3	52%	43%	31%	20%
C2 (miles)	Current Conditions	2.3	0.8	0.3	0.0
	SLR1	4.5	2.0	1.1	0.6
	SLR2	5.0	2.6	2.0	1.4
	SLR3	5.7	4.7	3.4	2.2

The maximum stage profiles for the Snapper Creek Canal are shown for the Current Conditions in **Figure 9-4**, and for all conditions for the 100-year Design Storm in **Figure 9-5**. These figures show peak stages (bank elevations and major intersection locations) along the entire length of the Snapper Creek Canal from the C4 Canal to the S22 structure. **Figure 9-6** and **Figure 9-7** show the same but for the Bird Drive Extension Canal to the Snapper Creek Canal, to give view of the western portion of the C2 Watershed. **Appendix A** provides a complete set of the C2 Watershed maximum stage profiles for all design events.

Figure 9-4. Maximum Stage Profile for the Snapper Creek Canal in the C2 Watershed for the Current Conditions

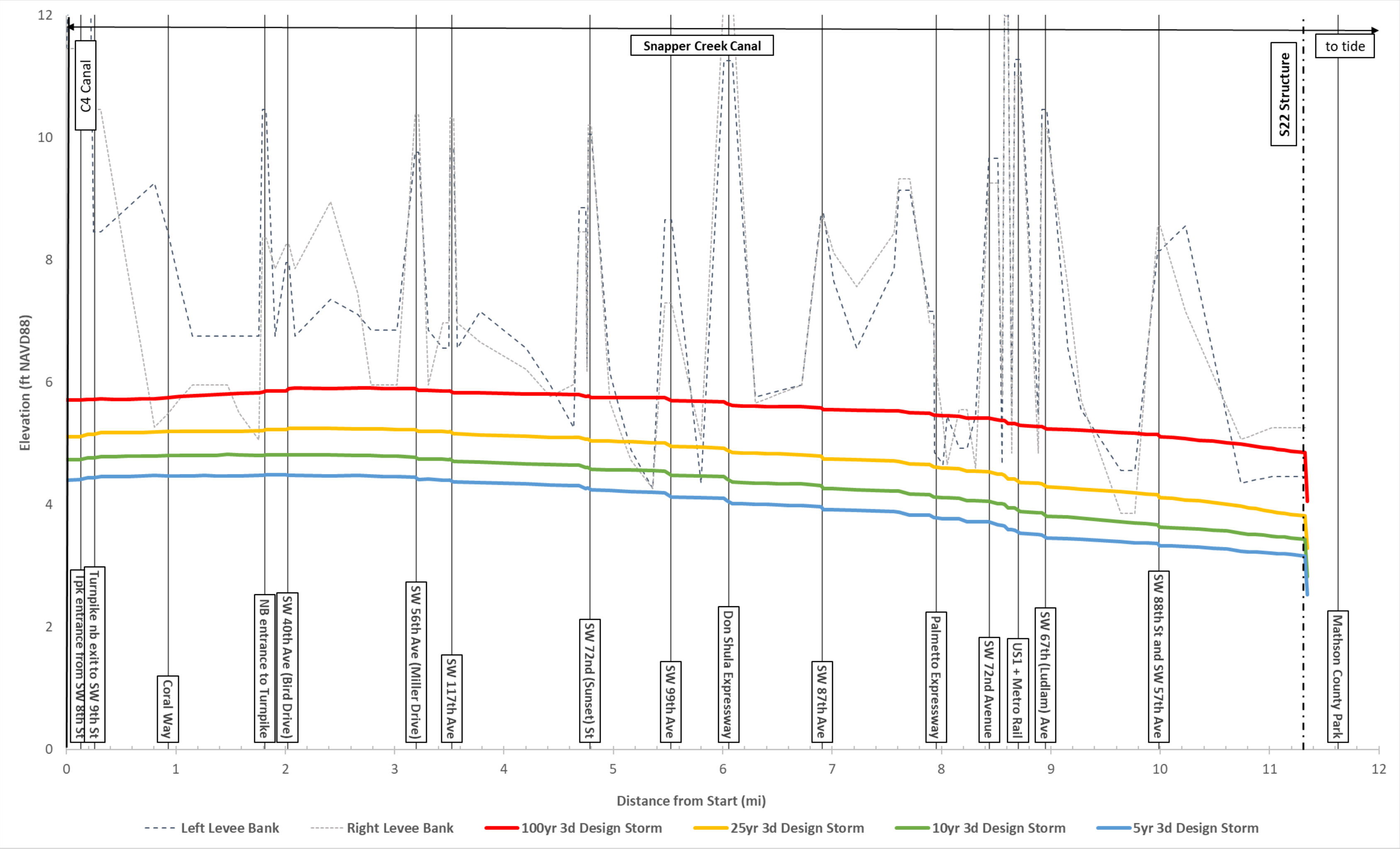


Figure 9-5. Maximum Stage Profile for the Snapper Creek Canal in the C2 Watershed for the 100-year Design Storm

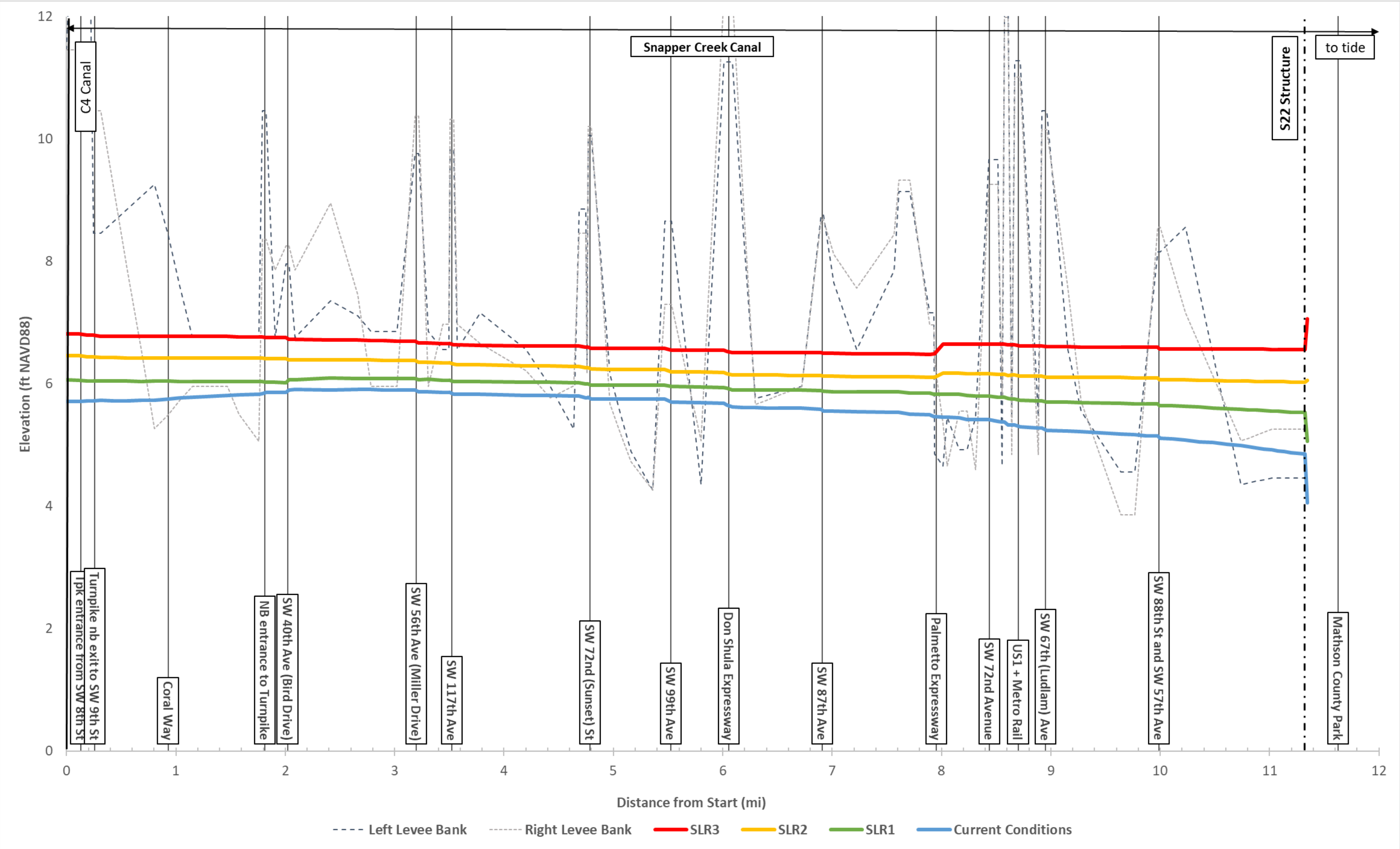


Figure 9-6. Maximum Stage Profile for Bird Drive and Snapper Creek Canals in the C2 Watershed for Current Conditions

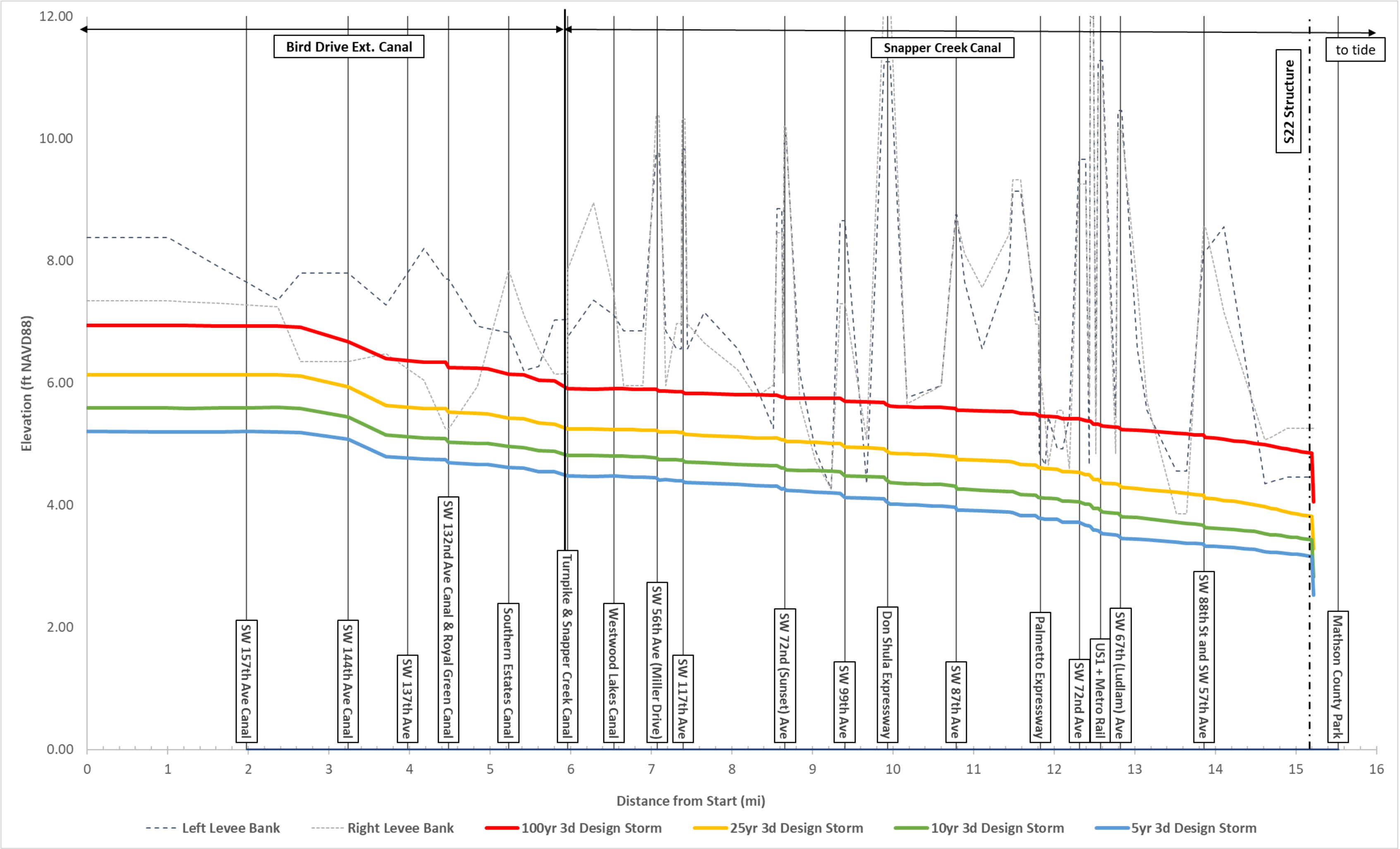
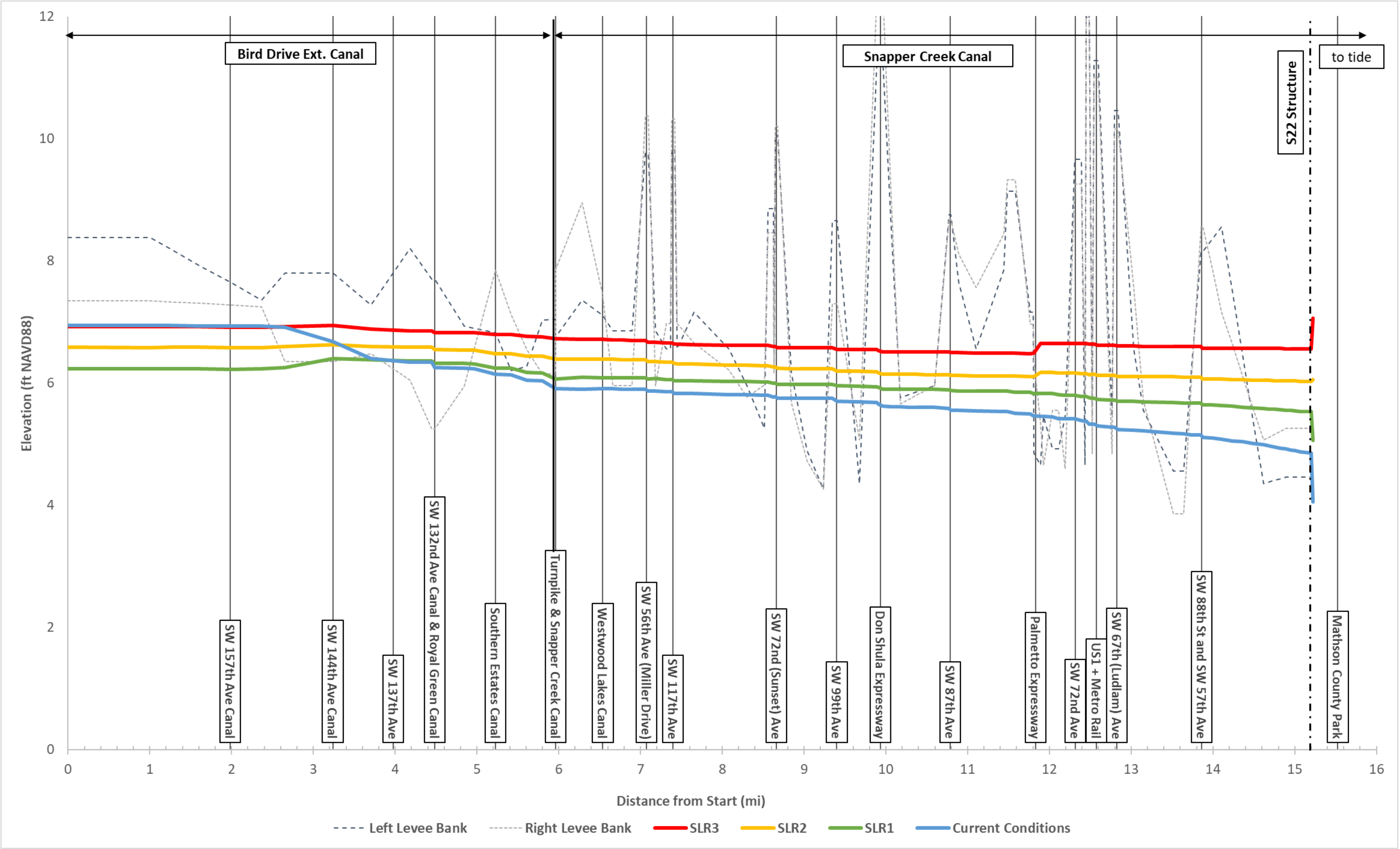
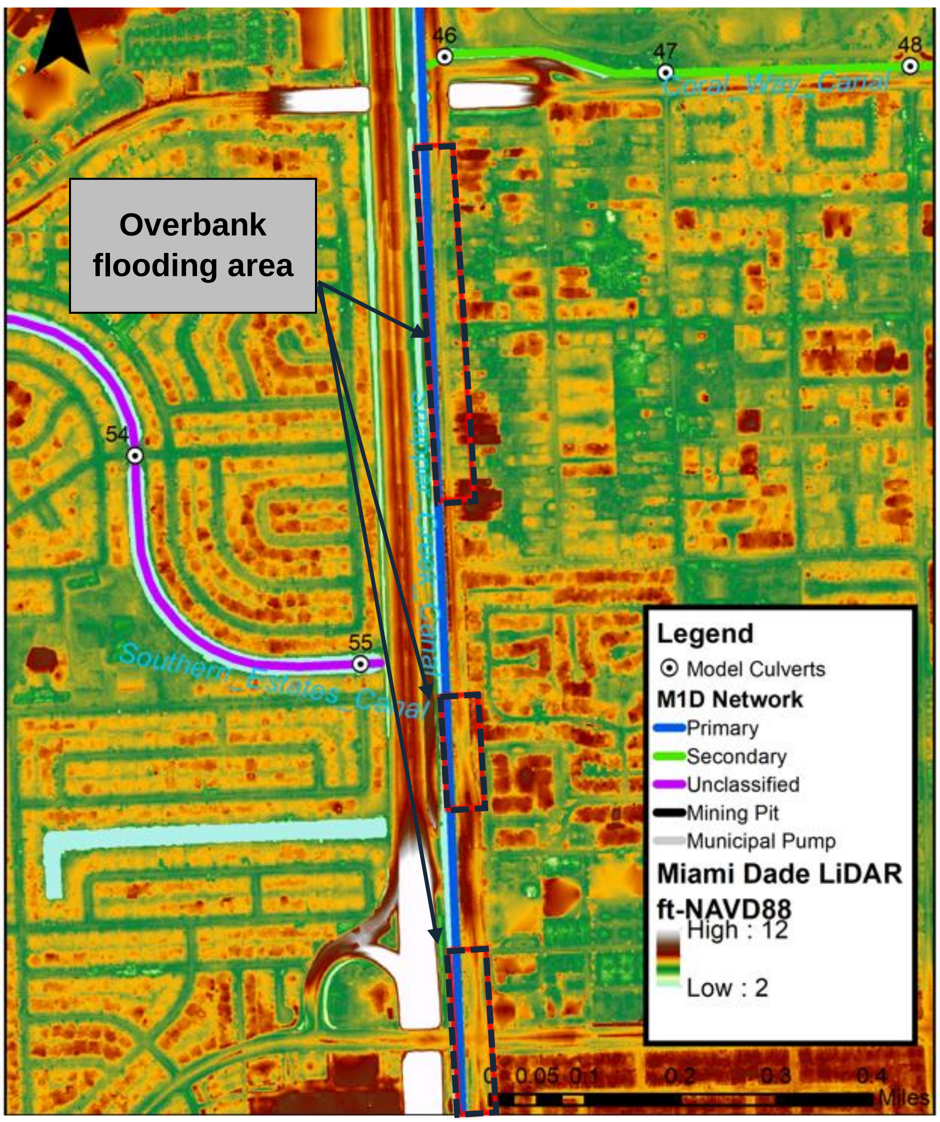


Figure 9-7. Maximum Stage Profile for Bird Drive and Snapper Creek Canals in the C2 Watershed for 100-year Design Storm



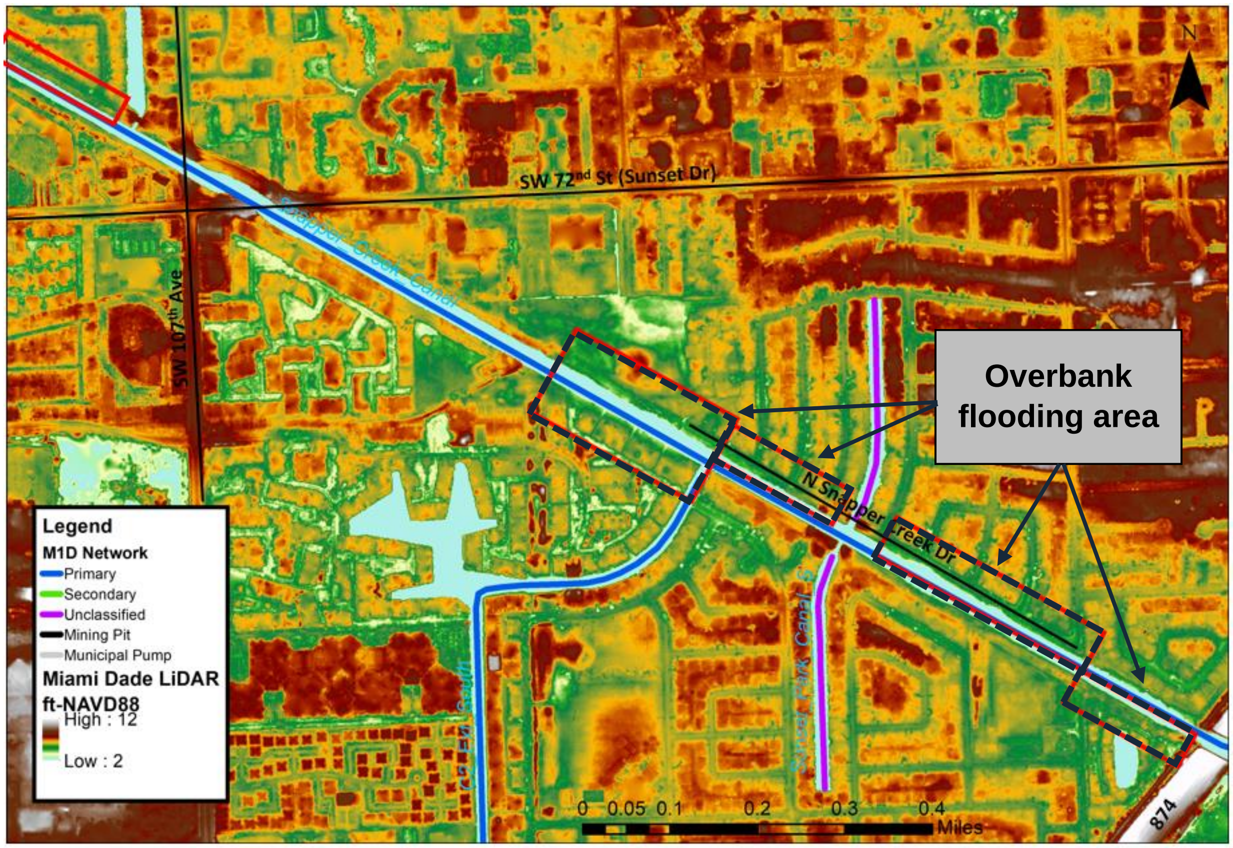
As shown in the current conditions maximum stage profiles in **Figure 9-4**, the top of bank is overtopped in several locations along the Snapper Creek Canal. The 100-year design event exceeded the embankment along the reach of the Snapper Creek Canal near the FIU Campus and extending south to SW 37th Street. This may represent some low spots along SW 117th Avenue, as the LiDAR topography suggests. As further discussed in the PM #6 section, these low canal embankments, along with those in the Westwood Lakes Canal, can reduce the drainage rate of an area draining to the canal. **Figure 9-8** shows a map of this area and the location of the low embankment as indicated by the red outline. In the model, embankments that are low enough to allow overland drainage, but high enough to contain the stages within the canal, are optimal for allowing drainage to occur.

Figure 9-8. Low Snapper Creek Canal Embankment along SW 117th Avenue



The Snapper Creek Wellfield and monitoring wellfield between SW 72nd Avenue and the Don Shula Expressway are also low embankments that are overtopped for the 10- year, 25- year, and 100-year design storm events. In addition, N Snapper Creek Drive, which is adjacent to the canal on the north side has several low spots in these areas. **Figure 9-9** shows a map of this area and the location of the low embankment as indicated by the red outline.

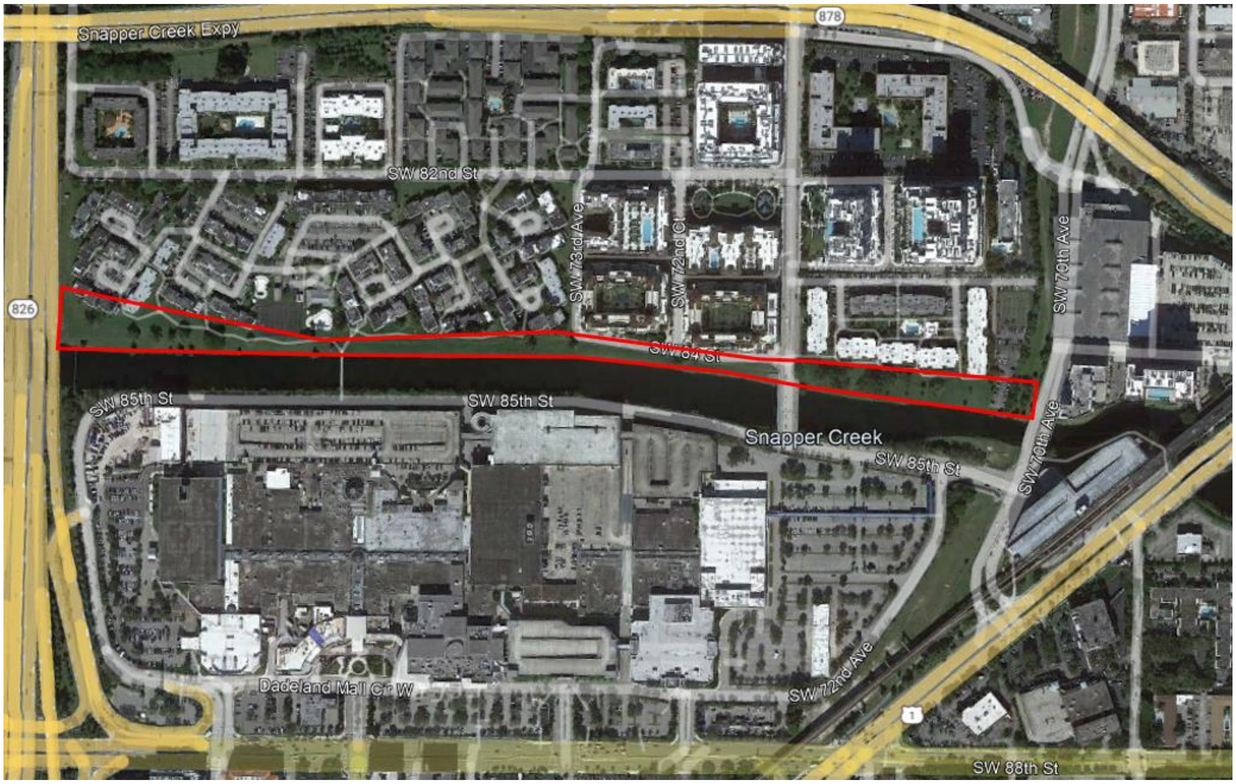
Figure 9-9. Low Snapper Creek Canal Embankment Near Dadeland Mall



There is a low embankment north of Dadeland Mall, that shows overtopping for the 100-year event. **Figure 9-10** shows the location of the low embankment as indicated by the red outline.

Other low areas include the residential area between Ludlam Glade Canal to Old Cutler Road, where little to no levee is present, as residences are adjacent to the canal. This area is overtopped during the 100-year and 25-year design storm. In addition, where Banyan Drive becomes an unpaved path, the topography suggests the embankment is low up to the S22 structure.

Figure 9-10. Low Snapper Creek Canal Embankment Near Dadeland Mall



As shown in the 100-year maximum stage profile in **Figure 9-5**, the top of bank is overtopped in several locations along the Snapper Creek Canal, and this increases throughout the canal at all locations for each sea level rise condition. For the SLR2 and SLR3 simulations, there is a clear influence from the tidal conditions, as the section of the canal between the Palmetto Expressway and the S22 structure are higher than upstream conditions. This is impacting areas such as the low embankments near Dadeland Mall and the low areas from Ludlam Avenue to the SW 57th Avenue bridge.

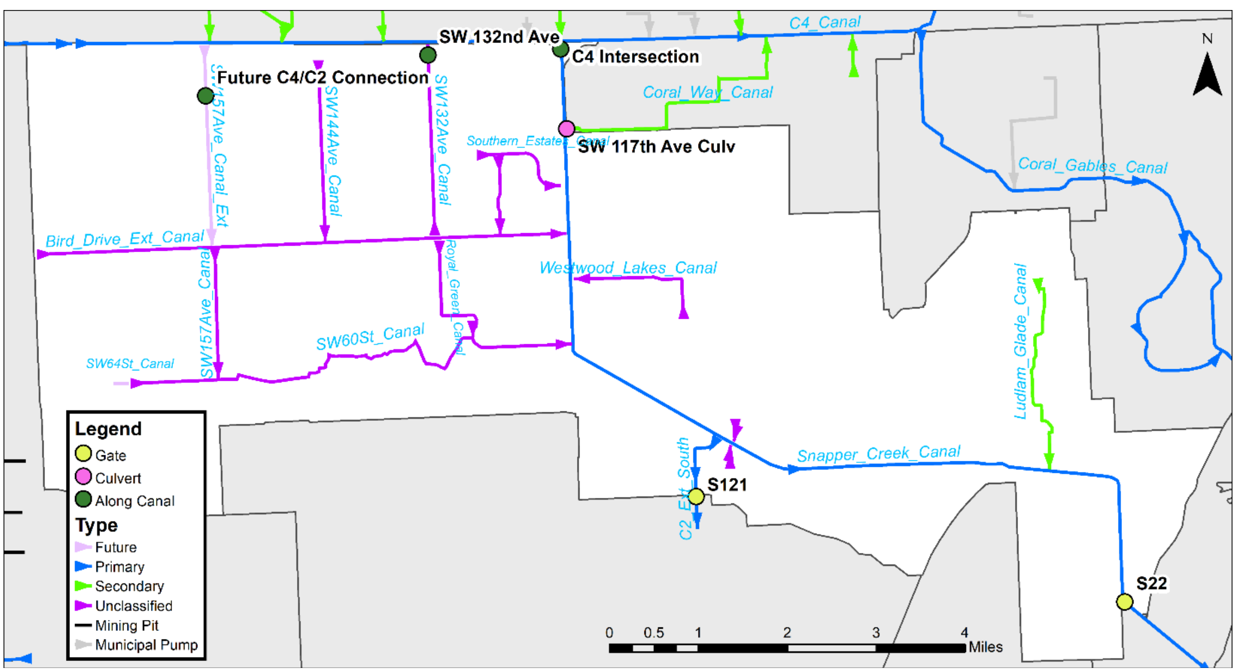
Figure 9-7 shows the 100-year maximum stage profile for the Bird Drive Extension Canal to the Snapper Creek Canal. For areas upstream of SW 144th Avenue, the current conditions simulations show higher water levels than the future conditions. This is due to the addition of future planned canals that were implemented into the model, and specifically the SW 157th Avenue Canal Extension that adds another connection from the C2 to the C4 Watershed just upstream of S380. Flows between the watersheds at this future planned connection are further explored in PM #2.

9.1.2. C2 – PM #2 MAXIMUM DISCHARGE CAPACITY

The maximum discharge capacity for the C2 Watershed is the sum of the discharges out of the watershed minus the incoming flows weighted by the total area of the watershed. For the C2 Watershed this means flows from the C4 Canal and Coral Way Canal are subtracted from the outflows at the S22 structure and flows leaving the basin at SW 132nd Avenue Canal, these locations are shown in **Figure 9-11**, and the flow direction of the canals as established in the model are shown with arrows. The S121 structure remains closed during design storm simulations, so no flows are considered at that location.

For the future conditions simulations, the SW 157th Avenue Canal Extension was added (SW157Ave_Canal_Ext) to represent the planned canal development in Miami Dade County. The branch includes a connection with the C4 Canal just upstream of the S380 gated structure, which remains open during the simulations. This additional connection with the C4 Watershed is shown in **Figure 9-11** as discharge along the canal. Discharges at this location are also included in the watershed peak discharge capacity calculations as an inflow location.

Figure 9-11. Discharge Locations in the C2 Watershed



The time series at each inflow and outflow location was extracted, and inflows were subtracted from the outflows and divided by the basin area in square miles (sq. mi.) at each timestep, as shown in the equation below.

$$\frac{Outflows[cfs] - Inflows[cfs]}{Basin\ Area\ [sq.mi.]} = Discharge\ Capacity[CSM]$$

The maximum discharge capacity for the C2 Watershed for each design event is shown in **Table 9-4**. The table also provides discharge in cubic feet per second (CFS) at inflow and outflow locations for the watershed. Discharges are not the peak discharge at each location, but rather the discharge at the peak watershed discharge capacity. A negative value at an inflow location means that the flow is leaving the watershed at the peak of the watershed discharge. A negative value at an outflow location means that the flow is entering the watershed at the peak of the watershed discharge.

The regulatory allowable discharge capacity of the C2 watershed northwest of Sunset Drive is 54 CSM (ERP Applicant's Handbook Volume II). The peak discharge capacity for each design storm event is summarized in **Table 9-4** for each simulation (i.e., Current Conditions, SLR +1 foot, SLR +2 feet, and SLR +3 feet). This table indicates that the allowable discharge value of 54 CSM is exceeded for the 100-year and 25-year storm events for both the Current and Future Conditions. However, as the discharge capacity is diminished with increasing SLR, the basin approaches the allowable discharge rate with the 25-year storm for SLR3 (55.4 CSM). This diminished capacity is directly related to the watershed's ability to discharge to tide at the S22 structure, as is further explored in PM #3 and PM #4.

In addition, an increase in discharge capacity is seen for the SLR 1 conditions for all storm events. This is due to the addition of the future connection to the C4 Watershed at SW 157th Avenue. This connection increases the discharge of the entire watershed by moving water into the C4 Impoundment. For all storm events shown, the basin discharge increases with SLR1, but then decreases with each additional foot of SLR.

Figure 9-12 shows the instantaneous discharge capacity in CSM for the C2 Watershed, for each SLR condition simulation for the 100-year, 25-year, 10-year, and 5-year design storm events. To remove the tidal influence, **Figure 9-13** shows the 12-hour moving average discharge capacity in CSM for all SLR simulations and each design storm event. For all storm events in these figures, the discharge capacity for the watershed during SLR1 conditions is higher than the Current Conditions, which is due to increasing flows to the C4 Impoundment. However, as the discharge at S22 reduces (due to higher tidal conditions with increasing sea levels) the inflow to the C4 Impoundment does not increase enough to continue to provide discharge capacity for the watershed, and the watershed discharge decreases again for SLR2 and SLR3 conditions.

Table 9-4. Peak Discharge Capacity (CFS/Square Mile) from the Contributing Drainage Area of the C2 Watershed

	CURRENT CONDITIONS				SLR1				SLR2				SLR3			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Inflow Locations																
Start of Snapper Creek at C4 Canal (CFS)	-566.1	-329.2	-260.0	-347.5	-775.9	-389.8	-204.7	-181.6	-122.2	-372.0	-351.6	-193.8	-66.1	-148.3	-121.9	-64.6
Future Connection at SW 157th Ave (CFS)	--	--	--	--	-403.9	-357.3	-331.3	-317.5	-367.8	-344.8	-318.6	-296.3	-178.7	-296.1	-320.5	-306.7
Outflow Locations																
Coral Way Canal at SW 117th Ave (CFS)	-87.4	-78.7	-75.5	-49.9	-78.2	-80.1	-76.7	-76.0	-18.7	-76.3	-66.6	-71.9	-25.2	-52.5	-46.0	-43.1
SW 132nd Ave Canal into C4 Canal (CFS)	166.4	144.8	155.1	133.1	161.9	118.7	128.3	120.6	106.5	104.1	102.5	106.8	66.3	90.3	85.1	97.3
S112 (CFS)	0	0	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
S22 Total Flow (CFS)	3163.5	2744.8	2428.8	2192.8	2831.6	2535.0	2338.0	2143.6	2734.5	2225.9	1890.1	1877.9	2673.6	2324.1	2061.6	1738.0
Watershed Summary																
Basin Area (sq. mi.)	52.6				52.6				52.6				52.6			
Peak Watershed Discharge (CSM)	72.4	62.7	55.5	51.8	77.9	66.2	58.5	54.0	63.0	59.4	51.9	48.4	56.3	55.4	50.1	42.8

Figure 9-12. Instantaneous Discharge Capacity for the C2 Watershed for 100-year, 25-year, 10-year, and 5-year 3-day Design Storm Events

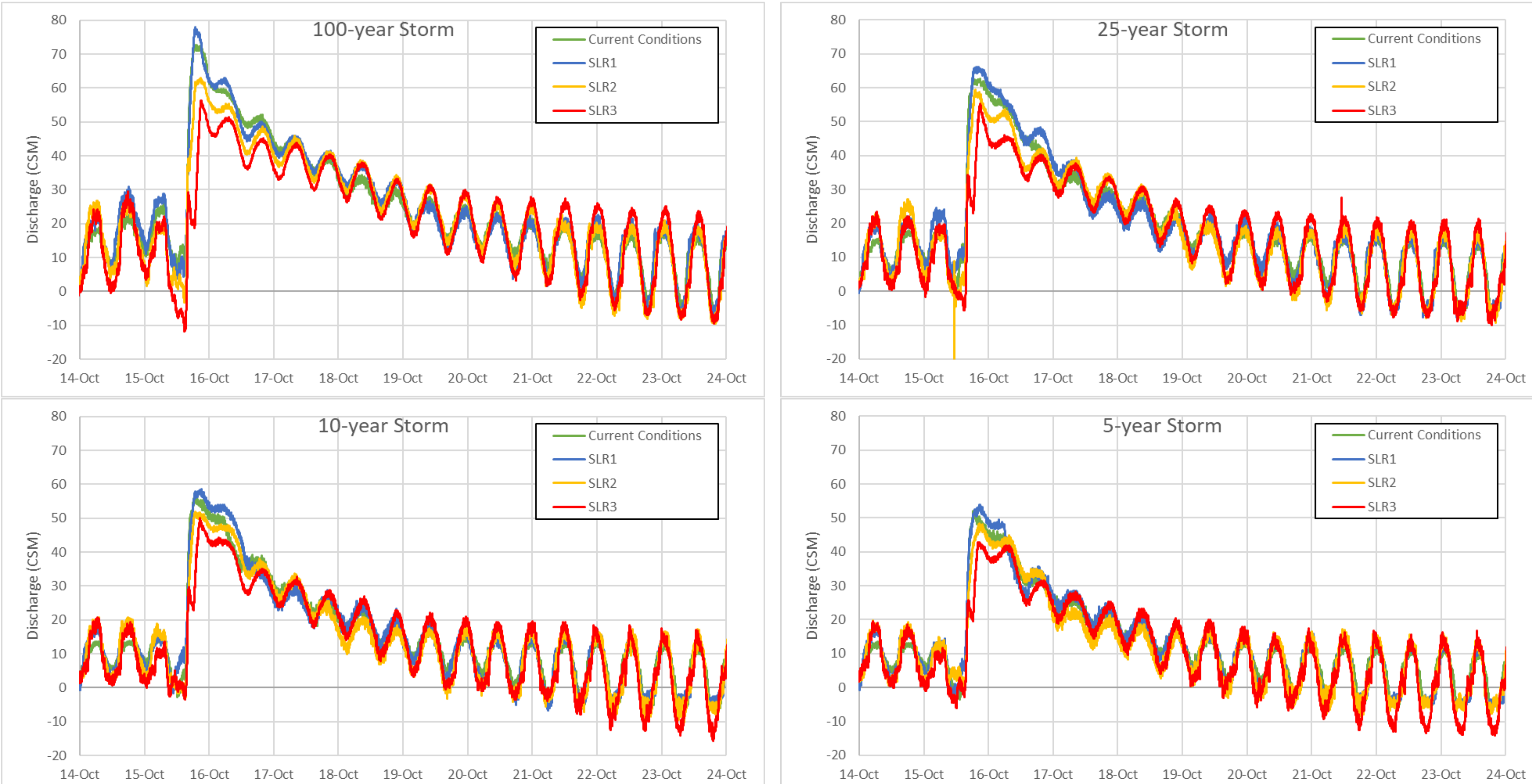
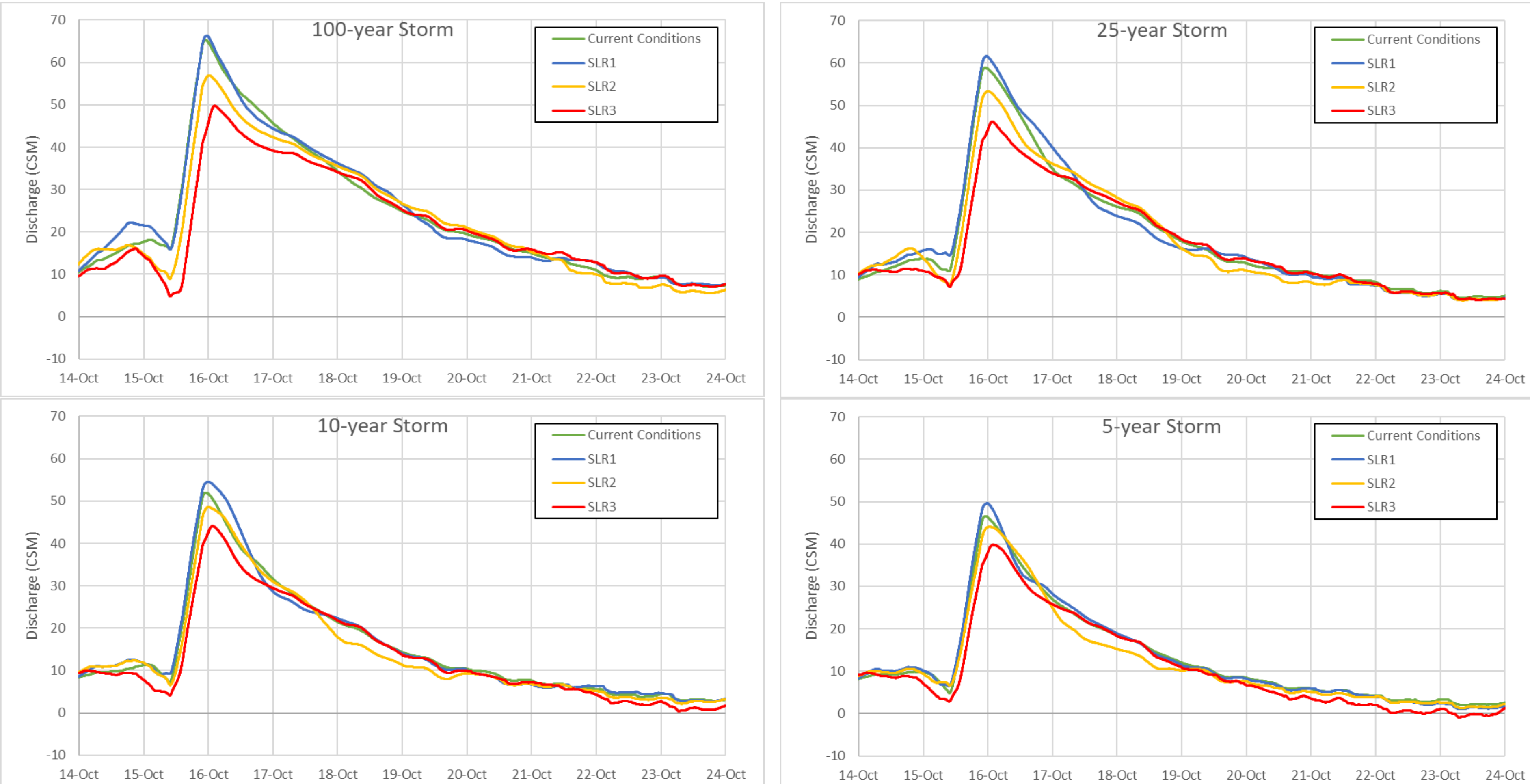


Figure 9-13. 12-Hour Moving Average Discharge Capacity for the C2 Watershed for 100-year, 25-year, 10-year, and 5-year 3-day Design Storm Events



9.1.2.1. INTER-BASIN TRANSFERS

During wet season operations, the C2 Watershed interacts primarily with the C4 Watershed, receiving flows from the C4 Canal at the intersection of the C4 and the Snapper Creek Canal (or SW 8th Street and Florida Turnpike), as well as flows from the Coral Way Canal (at SW 117th Avenue). Normally, wet season flows move from the C4 Watershed into the C2 Watershed or are near zero. However, during the storm event, flows reverse and move from the C2 into the C4 during the peak of the storm. **Figure 9-14** and **Figure 9-15** show the inter-basin transfers for the 100-year and 25-year design storms, respectively. Flows from the C4 Canal into the Snapper Creek Canal, shown in blue, are negative during the storm, indicating that the flows are moving into the C4 Watershed at this time. The same is true for flows in SW 132nd Avenue Canal and at the future connection at SW 157th Avenue positive flows indicate the water is moving from the C2 Watershed into the C4 Watershed. A small amount of flow circulates back into the C2 at the Coral Way Canal intersection with SW 117th Avenue during the peak of the storm event.

Figure 9-14. Exchanges between the C2 and C4 Watershed for the 100-year Design Storm for all Simulations

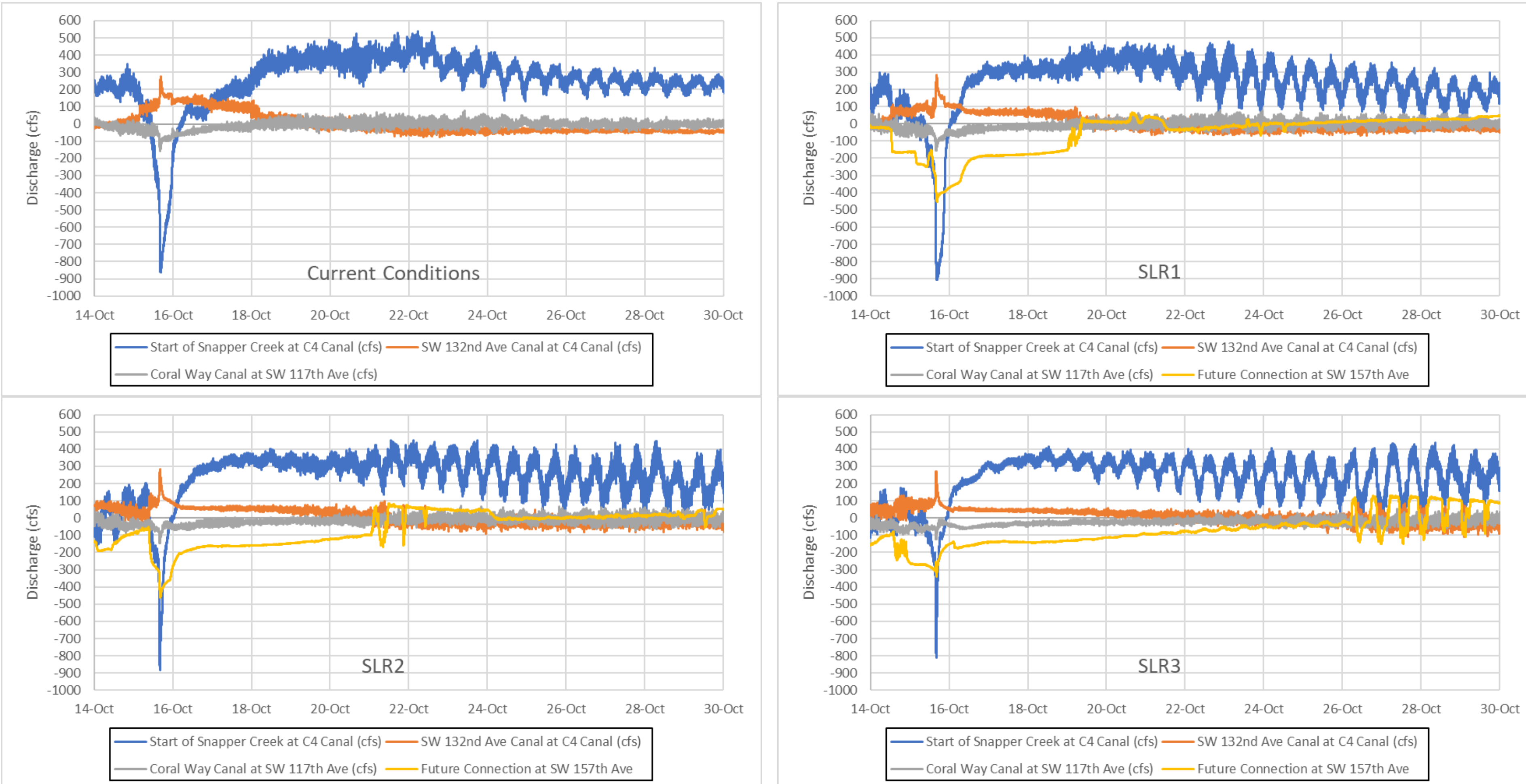
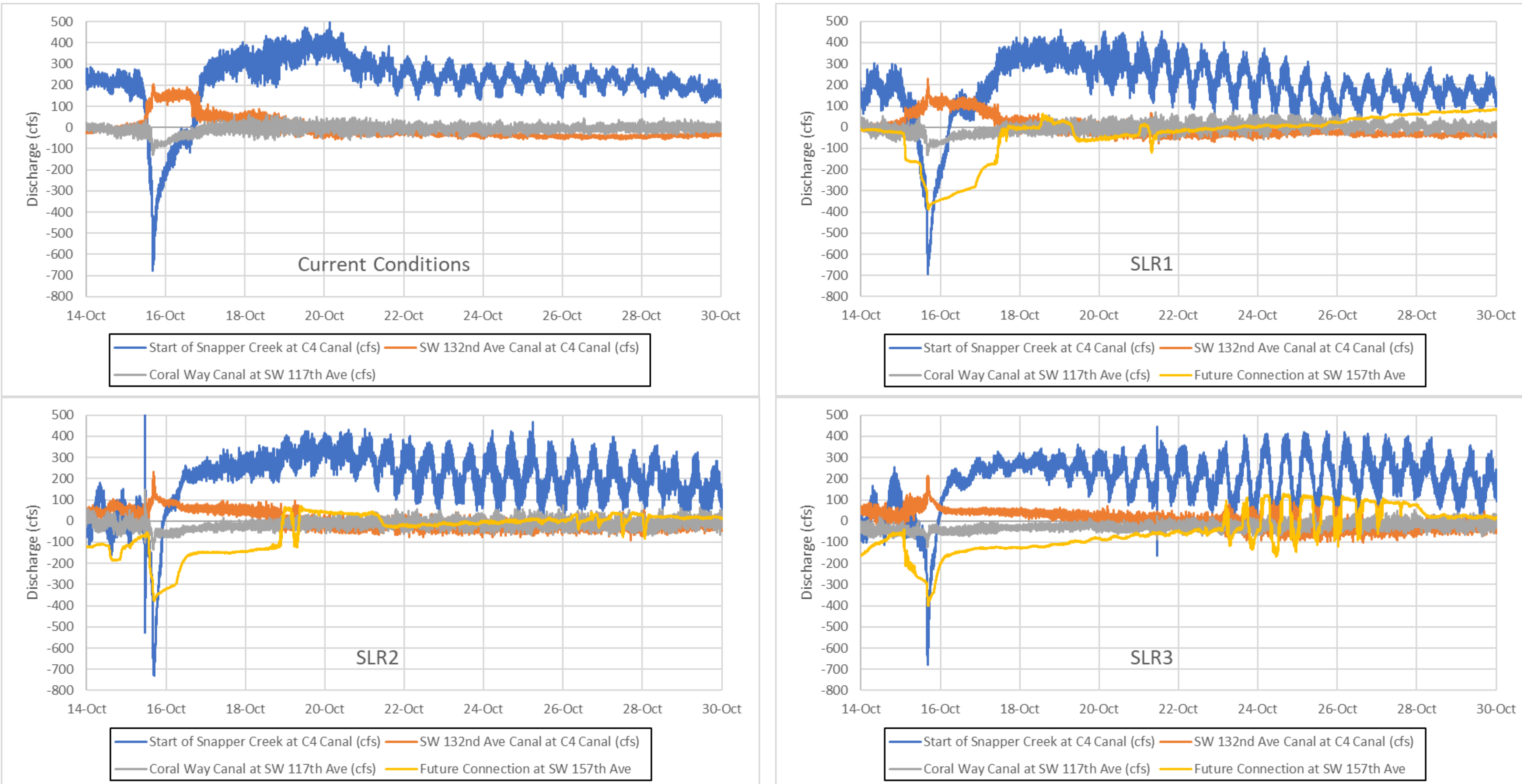


Figure 9-15. Exchanges between the C2 and C4 Watershed for the 25-year Design Storm



9.1.3. C2 – PM #3 STRUCTURE PERFORMANCE

The purpose of PM #3 is to determine the effect of SLR on tidal structure flow performance. For this metric, an evaluation based on structure design features and existing operational protocols was conducted for existing conditions and compared to simulations including SLR over the various storm events. PM #3 only accounts for gravity flows through the structure, where PM #4 will also account for structure flows as well as overtopping, to differentiate between structure capacity and the basic inabilities of the structure to provide tidal protections.

As per the structure data sheet, the S22 tidal structure operates with the intention to maintain optimum water levels in the C2 Watershed and passes 100% of the Standard Project Flood (SPF) without exceeding upstream flood design stage. The structure is also used to restrict downstream flood stages and discharge velocities to protect downstream areas, as well as prevent saline intrusion during periods of high tides. **Table 9-5** shows the design parameters for S22 as provided in the Water Control Operations Atlas.

Table 9-5. Design Parameters for Structure S22

Design Parameters	S22
Design Discharge	1,905 CFS (100% SPF)
Design HW	1.66 ft-NAVD (3.2 ft-NGVD)
Design TW	1.16 ft-NAVD (2.7 ft-NGVD)
Optimum HW	1.36 ft-NAVD (2.9 ft-NGVD)
Optimum TW	Tidal
Maximum Gate Opening	15 ft
Water Level which will Bypass Structure	5.96 ft-NAVD (7.5 ft-NGVD)
Water Level which will Overtop Gates when Closed	2.46 ft-NAVD (4.0 ft-NGVD)
Low Range Operational Triggers	-0.04 ft-NAVD (1.5 ft-NGVD) to 0.96 ft-NAVD (2.5 ft-NGVD)

Figure 9-16 and **Figure 9-17** provide the instantaneous discharge, HW and TW at S22 during the current conditions simulation for the 100-year and 25-year design events, respectively. **Figure 9-18** and **Figure 9-19** provide the 12-hour moving average for the discharge, HW, and TW at S22 during the current conditions simulation for the 100-year and 25-year design events, respectively, which removes the influence of the tides. **Figure 9-20** and **Figure 9-21** provide the instantaneous discharge, HW and TW at S22 during the future conditions SLR1 simulation for the 100-year and 25-year design events, respectively. **Figure 9-22** and **Figure 9-23** provide the 12-hour moving average for the

discharge, HW, and TW at S22 during the future conditions SLR1 simulation for the 100-year and 25-year design events, respectively, which removes the influence of the tides. **Figure 9-24** and **Figure 9-25** provide the instantaneous discharge, HW and TW at S22 during the future conditions SLR2 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-26** and **Figure 9-27** provide the 12-hour moving average for the discharge, HW, and TW at S22 during the future conditions SLR2 simulation for the 100-year and 25-year design events, respectively. **Figure 9-28** and **Figure 9-29** provide the instantaneous discharge, HW and TW at S22 during the future conditions SLR2 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-30** and **Figure 9-31** provide the 12-hour moving average for the discharge, HW, and TW at S22 during the future conditions SLR2 simulation for the 100-year and 25-year design events, respectively. Figures are provided for instantaneous discharge and 12-hour moving discharge for all design storms and SLR simulations in **Appendix B**.

Figure 9-16. Instantaneous Discharge and Stages at S22 for the 100-year 3-day Current Conditions Event

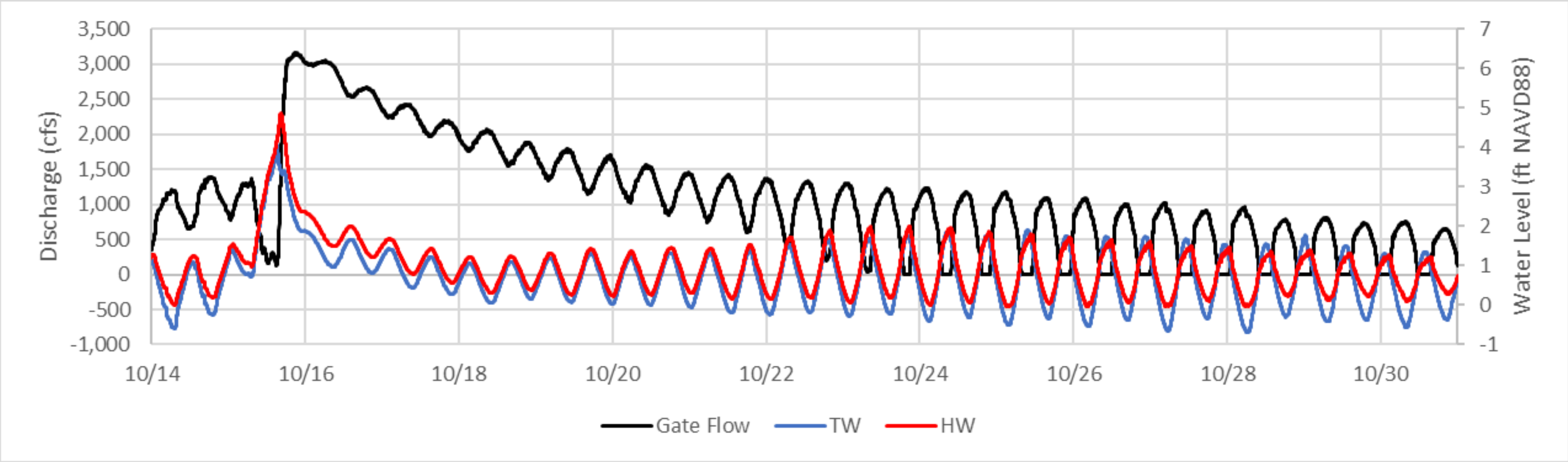


Figure 9-17. Instantaneous Discharge and Stages at S22 for the 25-year 3-day Current Conditions Event

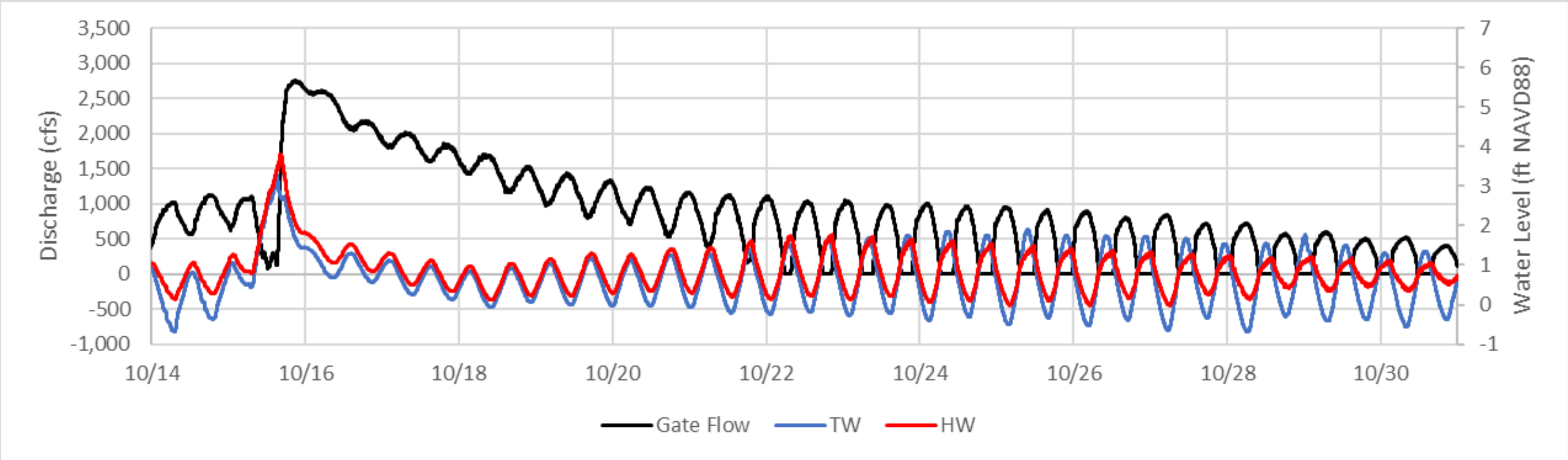


Figure 9-18. 12-Hour Moving Average Discharge and Stages at S22 for the 100-year 3-day Current Conditions Event

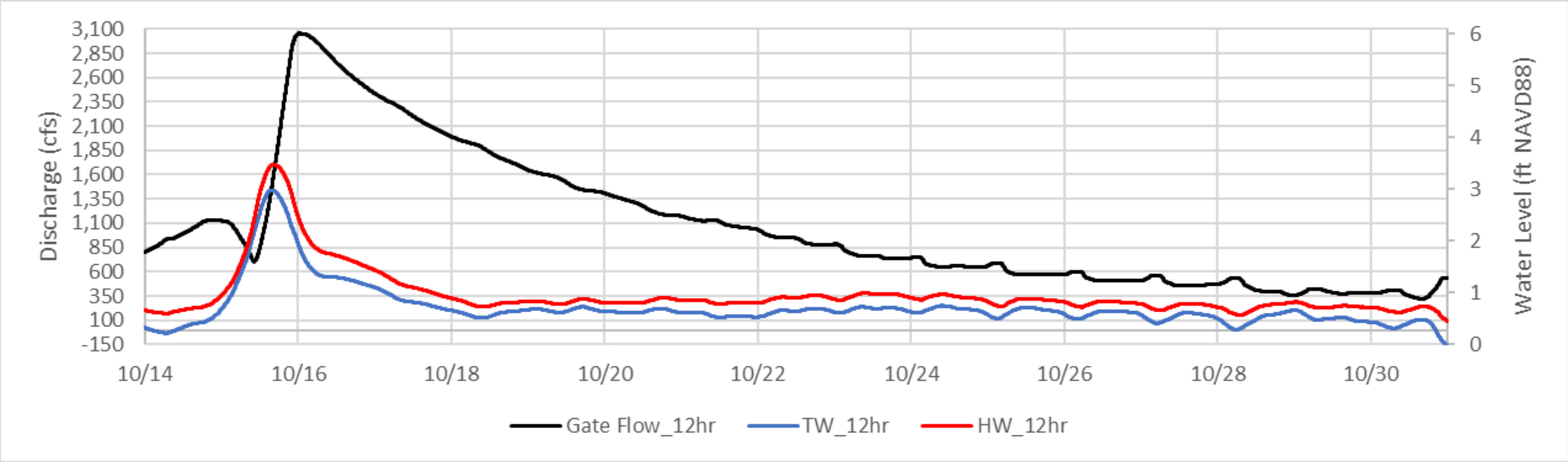


Figure 9-19. 12-Hour Moving Average Discharge and Stages at S22 for the 25-year 3-day Current Conditions Event

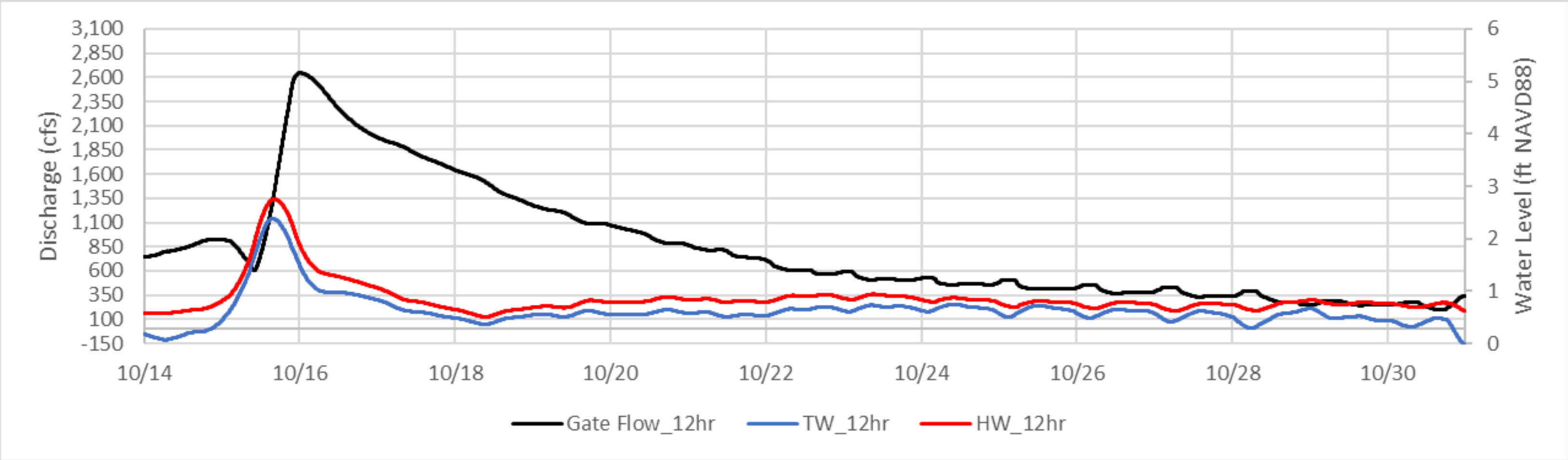


Figure 9-20. Instantaneous Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR1) Event

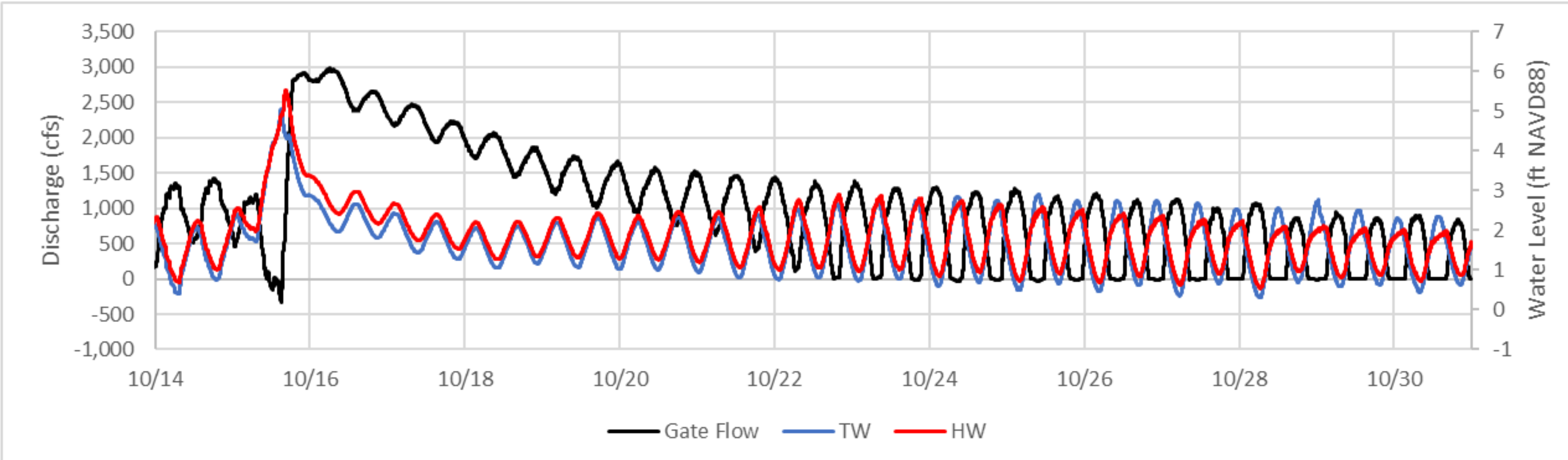


Figure 9-21. Instantaneous Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR1) Event

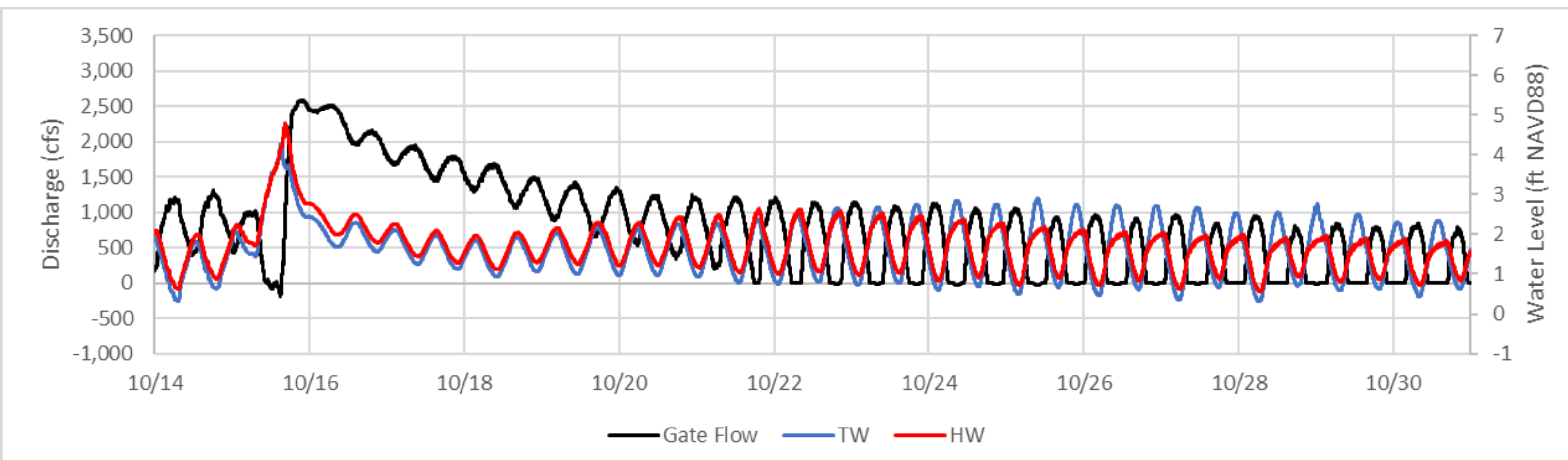


Figure 9-22. 12-Hour Moving Average Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR1) Event

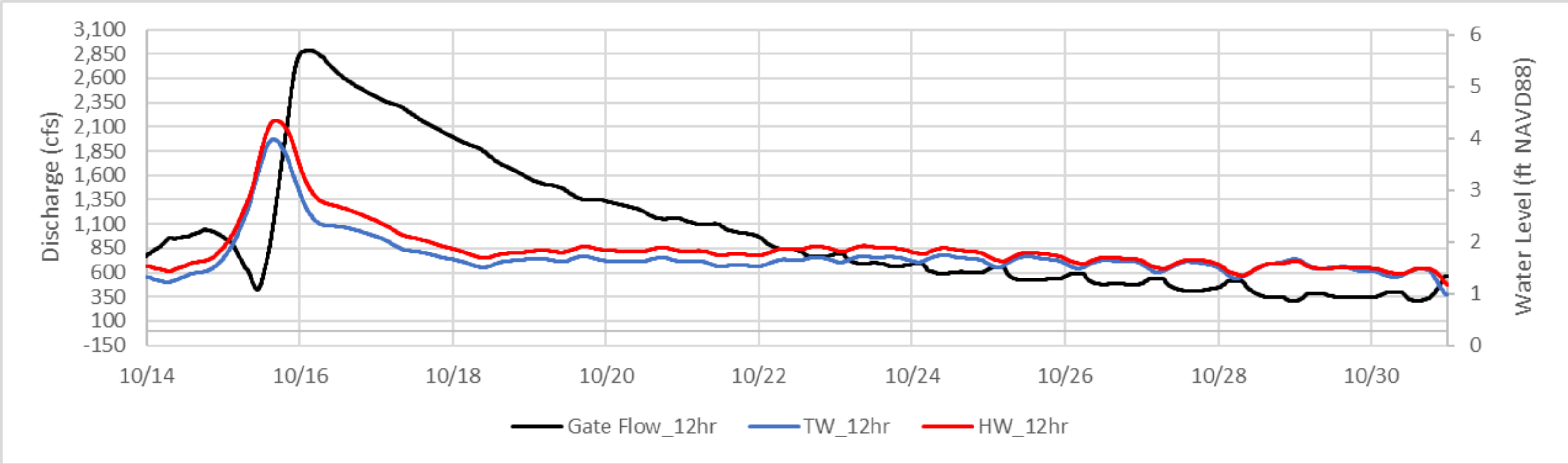


Figure 9-23. 12-Hour Moving Average Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR1) Event

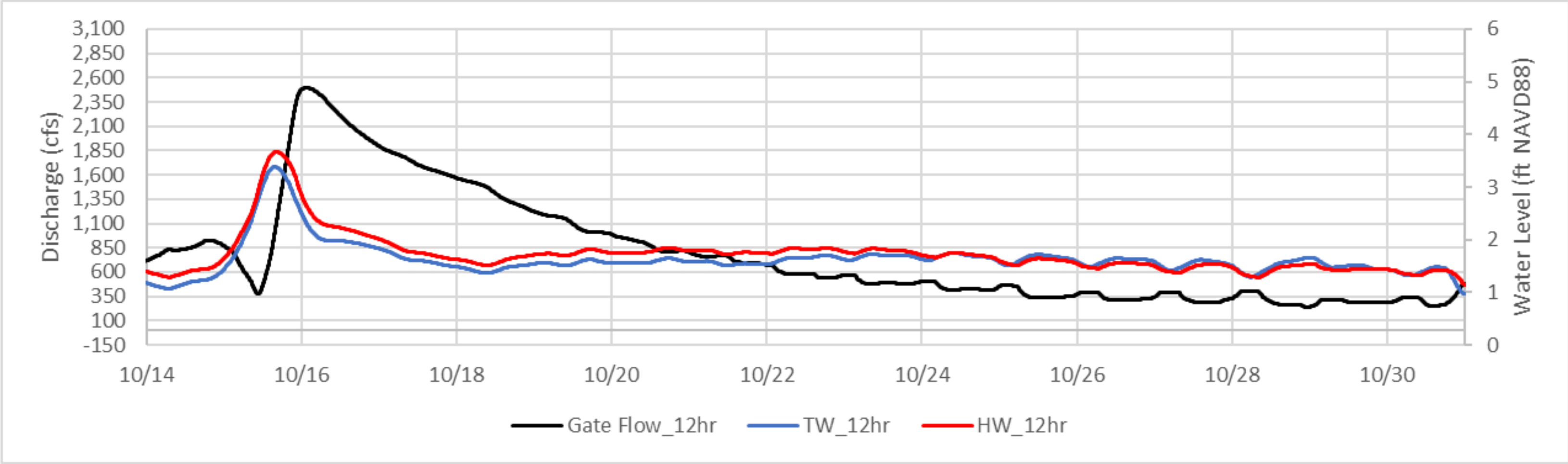


Figure 9-24. Instantaneous Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR2) Event

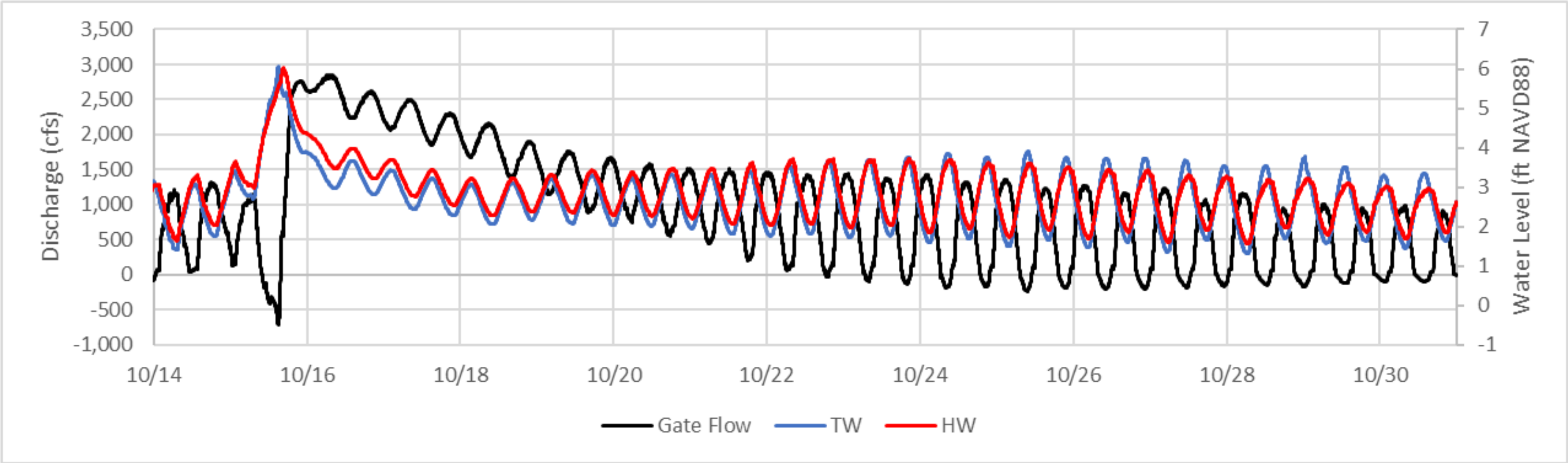


Figure 9-25. Instantaneous Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR2) Event

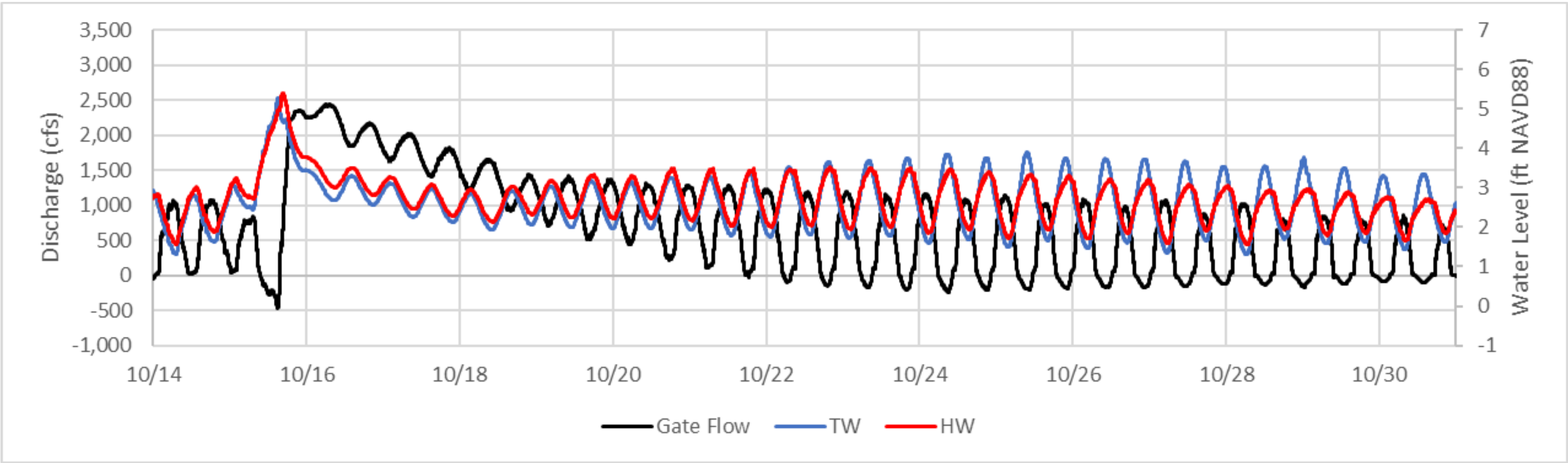


Figure 9-26. 12-Hour Moving Average Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR2) Event

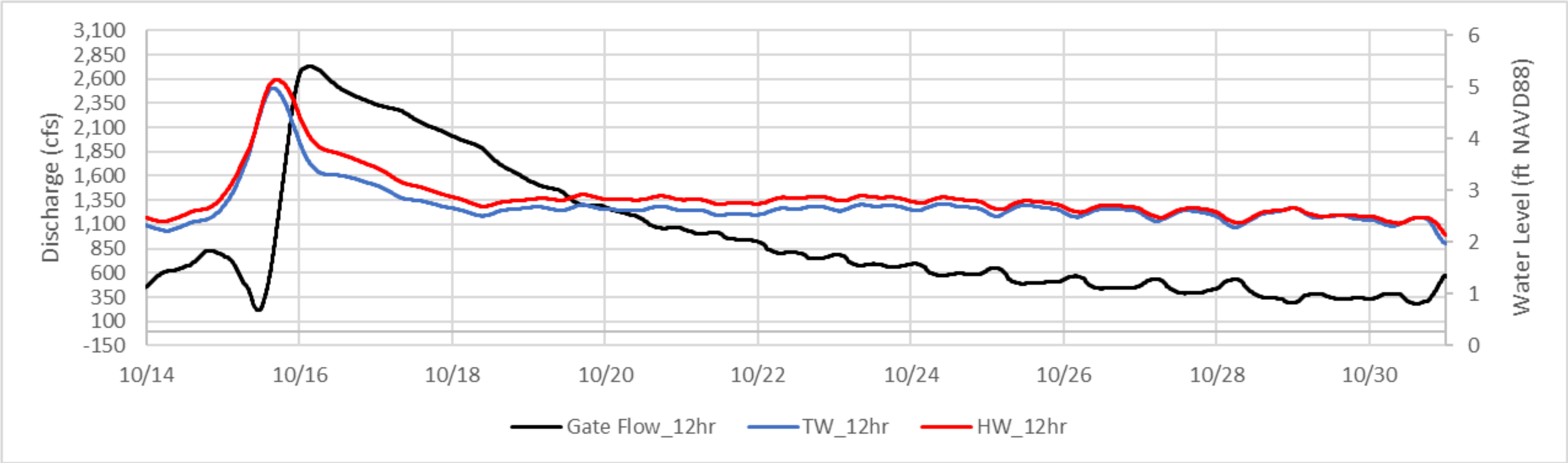


Figure 9-27. 12-Hour Moving Average Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR2) Event

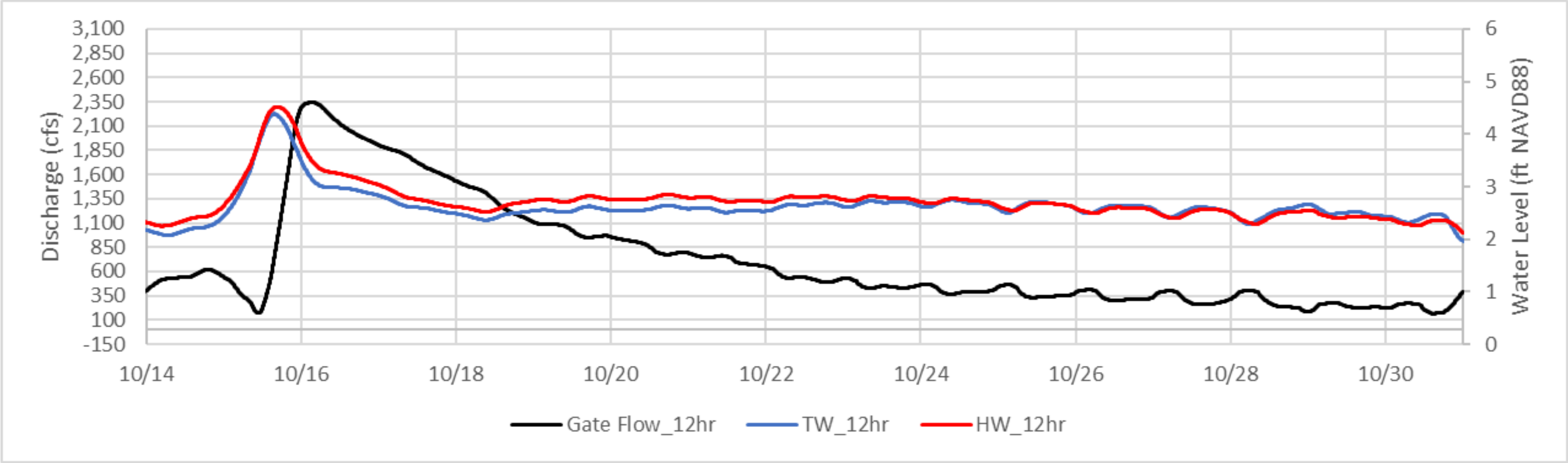


Figure 9-28. Instantaneous Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR3) Event

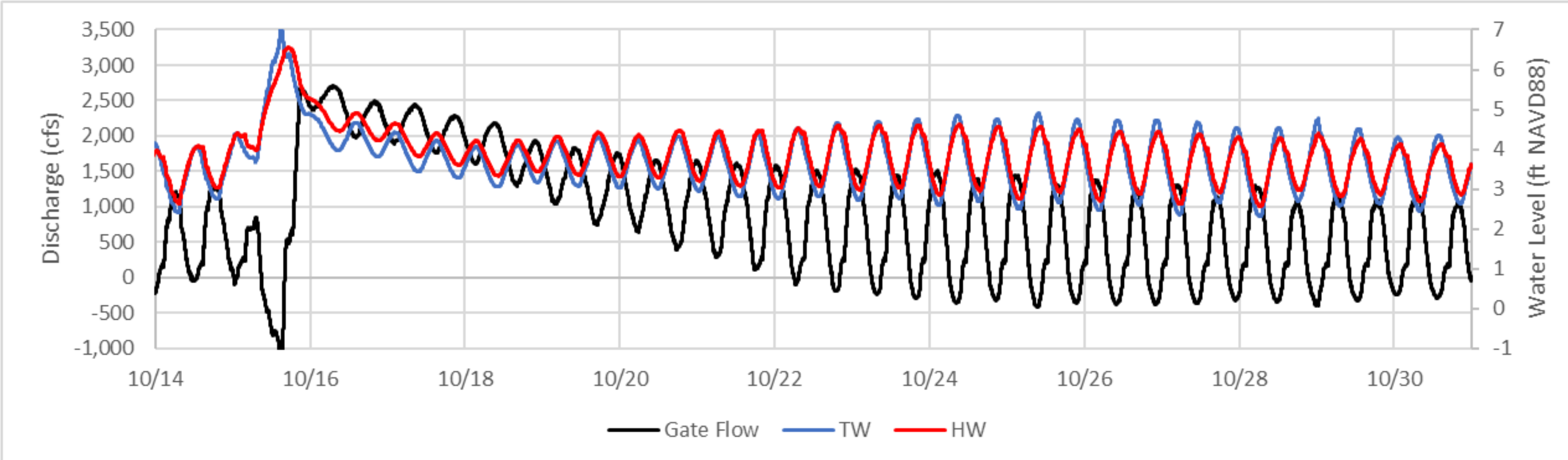


Figure 9-29. Instantaneous Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR3) Event

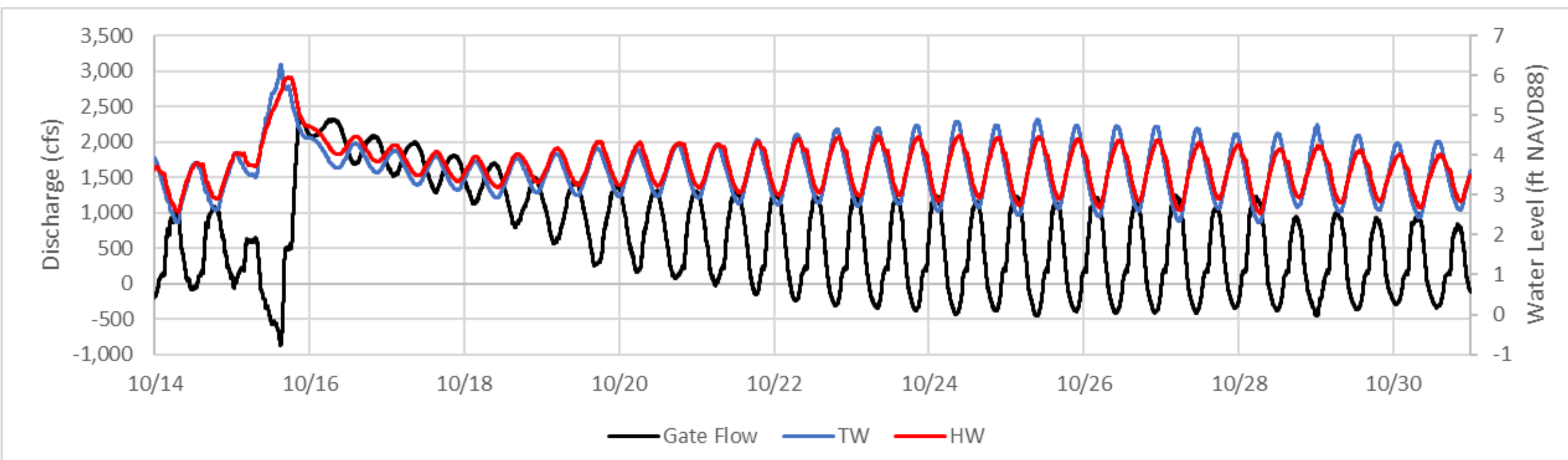


Figure 9-30. 12-Hour Moving Average Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR3) Event

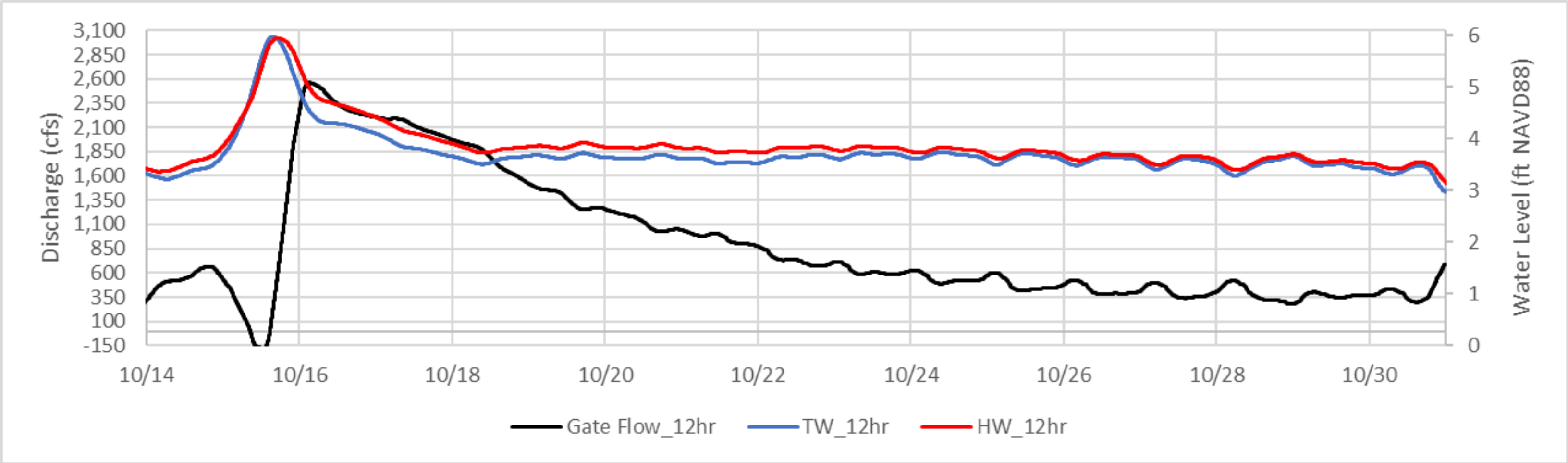
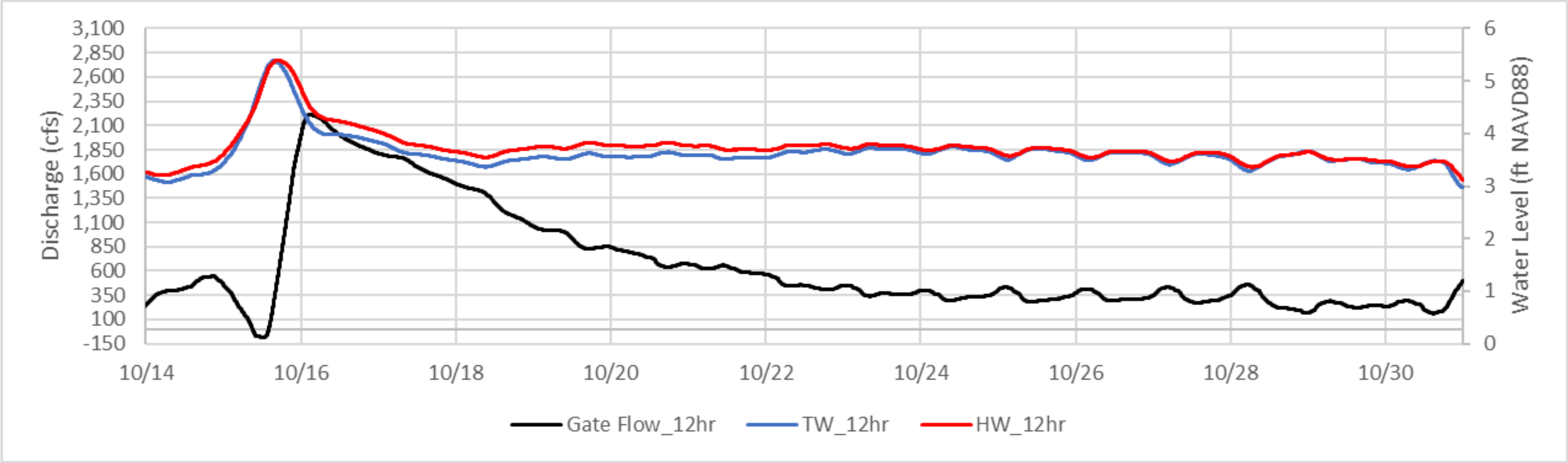


Figure 9-31. 12-Hour Moving Average Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR3) Event



A summary of the instantaneous and 12-hour moving average conditions at the time of peak discharge through S22 are provided in **Table 9-6** and **Table 9-7**, respectively for all design storm return periods and future SLR conditions.

Table 9-6. Summary of Conditions at the Time of Peak Discharge through S22 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak Q	Peak Q (CFS)	HW at Peak Q (ft-NAVD)	TW at Peak Q (ft-NAVD)	ΔH at Peak Q (ft)	Gate Opening at Peak Q* (ft)
100-Year	**Current**	10/15 20:28	3164.77	2.82	2.27	0.55	15
	SLR1	10/16 5:59	2982.90	2.68	2.20	0.48	15
	SLR2	10/16 7:10	2854.69	3.59	3.05	0.54	15
	SLR3	10/16 7:14	2709.43	4.55	4.04	0.51	15
25-Year	**Current**	10/16 0:05	2747.48	2.06	1.67	0.39	15
	SLR1	10/15 21:40	2589.22	2.92	2.55	0.38	15
	SLR2	10/16 6:07	2448.80	3.21	2.85	0.36	15
	SLR3	10/16 7:22	2326.38	4.09	3.73	0.36	15
10-Year	**Current**	10/15 20:58	2459.96	1.70	1.38	0.33	15
	SLR1	10/15 21:43	2354.10	2.55	2.26	0.29	15
	SLR2	10/16 6:16	2183.78	2.90	2.64	0.26	15
	SLR3	10/15 20:38	2063.22	4.73	4.44	0.29	14.5
5-Year	**Current**	10/15 20:43	2244.00	1.51	1.24	0.27	14.5
	SLR1	10/15 20:59	2145.65	2.47	2.19	0.28	14.5
	SLR2	10/16 6:07	1927.47	2.83	2.53	0.30	13.5
	SLR3	10/16 6:21	1834.20	3.81	3.51	0.29	13

*A gate opening of 15 ft represents the gate full open.

Table 9-7. Summary of Conditions at the Time of Peak Discharge through S22 (12-hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak Q	Peak Q (cfs)	HW at Peak Q (ft-NAVD)	TW at Peak Q (ft-NAVD)	ΔH at Peak Q (ft)	Gate Opening at Peak Q* (ft)
100-Year	**Current**	10/16 0:32	3037.97	2.37	1.86	0.50	15.00
	SLR1	10/16 2:51	2886.20	3.06	2.57	0.49	15.00
	SLR2	10/16 3:24	2728.15	4.03	3.52	0.51	15.00
	SLR3	10/16 2:57	2559.77	5.01	4.56	0.45	15.00
25-Year	**Current**	10/16 0:05	2631.27	1.87	1.50	0.38	15.00
	SLR1	10/16 1:38	2494.27	2.64	2.31	0.34	15.00
	SLR2	10/16 3:25	2343.21	3.49	3.14	0.34	15.00
	SLR3	10/16 2:42	2209.76	4.52	4.20	0.32	15.00
10-Year	**Current**	10/16 0:02	2346.71	1.54	1.24	0.30	15.00
	SLR1	10/16 0:41	2256.20	2.42	2.16	0.27	15.00
	SLR2	10/16 2:55	2088.71	3.21	2.95	0.26	15.00
	SLR3	10/16 2:29	1949.21	4.24	3.99	0.26	14.50
5-Year	**Current**	10/16 0:02	2126.41	1.32	1.07	0.26	14.49
	SLR1	10/16 0:37	2033.41	2.24	2.00	0.24	14.49
	SLR2	10/16 2:42	1856.51	3.09	2.82	0.27	13.50
	SLR3	10/16 3:16	1725.02	4.05	3.78	0.27	12.67

*A gate opening of 15 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak HW at S22 are provided in **Table 9-8** and **Table 9-9,** respectively for all design storm return periods and future SLR conditions.

Table 9-8. Summary of Conditions at the Time of Peak HW through S22 (Instantaneous)

Design Storm Return Period	Scenario	Time o Peak HW	Q at Peak HW (CFS)	Peak HW (ft-NAVD)	TW at Peak HW (ft-NAVD)	ΔH at Peak HW (ft)	Gate Opening at Peak HW* (ft)
100-Year	**Current**	10/15 16:25	1402.64	4.78	3.40	1.39	5.5
	SLR1	10/15 16:31	924.63	5.53	4.37	1.16	4
	SLR2	10/15 16:36	590.04	6.03	5.35	0.68	3
	SLR3	10/15 17:28	479.40	6.56	6.40	0.16	0
25-Year	**Current**	10/15 16:15	1553.53	3.87	2.72	1.15	6.5
	SLR1	10/15 16:25	776.02	4.81	3.73	1.08	3.5
	SLR2	10/15 16:34	547.70	5.40	4.69	0.70	3
	SLR3	10/15 17:28	471.17	5.96	5.74	0.22	0
10-Year	**Current**	10/15 16:18	1224.31	3.35	2.35	1.01	5.5
	SLR1	10/15 16:27	755.32	4.33	3.32	1.01	3.5
	SLR2	10/15 16:26	375.69	4.94	4.32	0.62	2
	SLR3	10/15 18:05	446.35	5.62	5.17	0.45	2
5-Year	**Current**	10/15 16:25	1240.71	3.10	2.07	1.04	5.5
	SLR1	10/15 16:27	722.59	3.97	3.06	0.91	3.5
	SLR2	10/15 16:29	331.74	4.62	4.05	0.57	2
	SLR3	10/15 18:05	399.65	5.39	4.93	0.46	2

*A gate opening of 15 ft represents the gate full open.

Table 9-9. Summary of Conditions at the Time of Peak HW through S22 (12-hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak HW	Q at Peak HW (CFS)	Peak HW (ft-NAVD)	TW at Peak HW (ft-NAVD)	ΔH at Peak HW (ft)	Gate Opening at Peak HW* (ft)
100-Year	**Current**	10/15 16:21	1582.17	3.47	2.97	0.49	8.02
	SLR1	10/15 16:28	1276.40	4.34	3.97	0.37	6.62
	SLR2	10/15 16:42	1015.67	5.14	4.96	0.19	6.55
	SLR3	10/15 17:18	614.66	5.95	5.93	0.03	4.76
25-Year	**Current**	10/15 16:15	1404.68	2.75	2.38	0.38	8.10
	SLR1	10/15 16:15	1100.33	3.66	3.38	0.29	6.35
	SLR2	10/15 16:36	916.20	4.52	4.36	0.16	6.47
	SLR3	10/15 16:56	556.38	5.39	5.35	0.04	4.49
10-Year	**Current**	10/15 16:09	1215.55	2.34	2.01	0.33	7.53
	SLR1	10/15 16:06	984.97	3.26	3.01	0.25	6.14
	SLR2	10/15 16:41	825.72	4.11	4.00	0.11	6.31
	SLR3	10/15 16:37	490.80	5.04	5.00	0.04	4.18
5-Year	**Current**	10/15 16:05	1055.41	2.09	1.78	0.31	6.76
	SLR1	10/15 16:03	881.29	3.00	2.78	0.22	5.87
	SLR2	10/15 16:58	769.77	3.85	3.76	0.09	6.01
	SLR3	10/15 16:48	469.80	4.81	4.76	0.05	3.98

*A gate opening of 15 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak TW at S22 are provided in **Table 9-10** and **Table 9-11**, respectively for all design storm return periods and future SLR conditions.

Table 9-10 shows that there are instances when there are negative flows, yet the gate opening is zero, this indicates that the tailwater is higher than the top of the closed gate (2.458 ft-NAVD) and there is a negative head differential across this structure. However, negative flows can also occur when the gate is in the process of closing and the head differential is negative.

Table 9-10. Summary of Conditions at the Time of Peak Tailwater at S22 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (ft-NAVD)	Peak TW (ft-NAVD)	ΔH at Peak TW (ft)	Gate Opening at Peak TW * (ft)
100-Year	**Current**	10/15 14:59	97.12	4.11	4.06	0.05	1
	SLR1	10/15 14:59	-322.46	4.87	5.06	-0.19	0
	SLR2	10/15 14:59	-722.39	5.53	6.06	-0.53	0
	SLR3	10/15 14:59	-1188.93	6.16	7.06	-0.90	0
25-Year	**Current**	10/15 15:42	109.65	3.38	3.29	0.09	1.5
	SLR1	10/15 14:59	-191.52	4.16	4.29	-0.14	0
	SLR2	10/15 14:59	-474.81	4.93	5.29	-0.37	0
	SLR3	10/15 14:59	-874.44	5.59	6.29	-0.70	0
10-Year	**Current**	10/15 14:59	77.57	2.94	2.83	0.11	1
	SLR1	10/15 14:59	-152.20	3.67	3.83	-0.16	0
	SLR2	10/15 14:59	-371.56	4.50	4.83	-0.32	0
	SLR3	10/15 14:59	-694.77	5.26	5.83	-0.57	0
5-Year	**Current**	10/15 14:59	8.02	2.63	2.53	0.10	0
	SLR1	10/15 14:59	-115.95	3.37	3.53	-0.15	0
	SLR2	10/15 14:59	-345.48	4.15	4.53	-0.37	0
	SLR3	10/15 14:59	-592.40	5.03	5.53	-0.50	0

*A gate opening of 15 ft represents the gate full open.

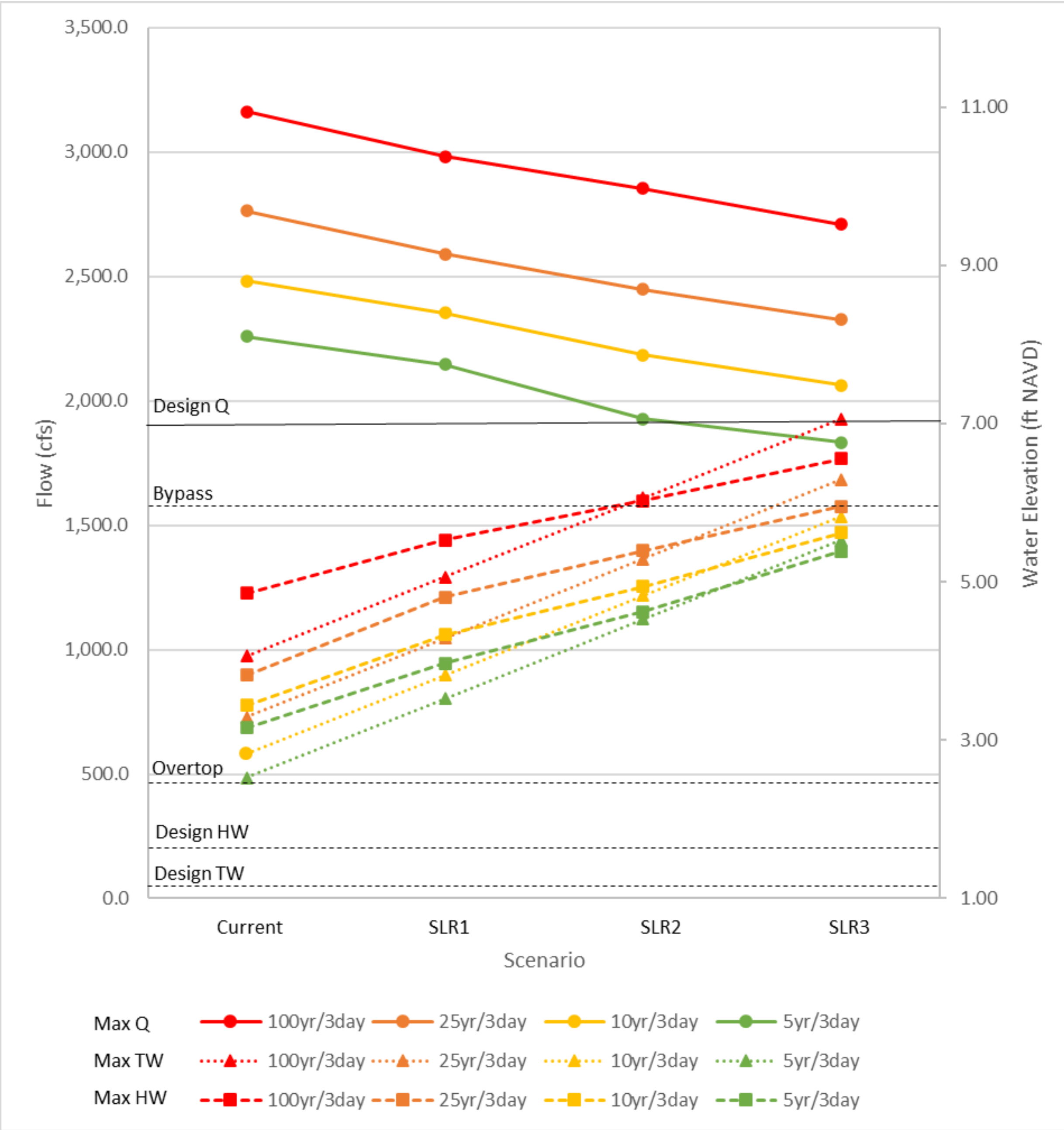
Table 9-11. Summary of Conditions at the Time of Peak Tailwater at S22 (12-hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (FT-NAVD)	Peak TW (FT-NAVD)	ΔH at Peak TW (FT)	Gate Opening at Peak TW* (FT)
100-Year	**Current**	10/15 15:43	1445.87	3.45	2.98	0.47	7.55
	SLR1	10/15 15:43	1102.09	4.33	3.98	0.35	5.78
	SLR2	10/15 15:43	784.37	5.12	4.98	0.14	5.34
	SLR3	10/15 15:43	218.77	5.91	5.98	-0.07	2.79
25-Year	**Current**	10/15 15:42	1301.76	2.75	2.38	0.36	7.66
	SLR1	10/15 15:43	990.55	3.66	3.38	0.28	5.74
	SLR2	10/15 15:43	737.12	4.51	4.38	0.13	5.37
	SLR3	10/15 15:43	300.18	5.37	5.38	-0.01	2.98
10-Year	**Current**	10/15 15:42	1139.46	2.34	2.02	0.32	7.15
	SLR1	10/15 15:42	912.97	3.26	3.02	0.24	5.72
	SLR2	10/15 15:42	644.96	4.10	4.02	0.08	5.08
	SLR3	10/15 15:42	323.81	5.03	5.02	0.01	3.07
5-Year	**Current**	10/15 15:42	999.33	2.09	1.78	0.31	6.45
	SLR1	10/15 15:42	822.66	3.00	2.78	0.21	5.49
	SLR2	10/15 15:42	561.79	3.83	3.78	0.05	4.58
	SLR3	10/15 15:42	292.78	4.80	4.78	0.01	2.84

*A gate opening of 15 ft represents the gate full open.

Figure 9-32 shows a summary of the instantaneous peak discharge, HW, and TW at S22 for all design storm return periods and future SLR conditions. The design parameter values listed in **Table 9-5** are shown graphically in the figure below with the bypass line indicating the water level which will bypass the structure, and the overtop line indicating the water level which will overtop the gates when the gates are closed. Note that the peak discharge, HW, and TW occur at different times for each scenario.

Figure 9-32. Summary of Instantaneous Peak Discharge, HW, and TW at S22

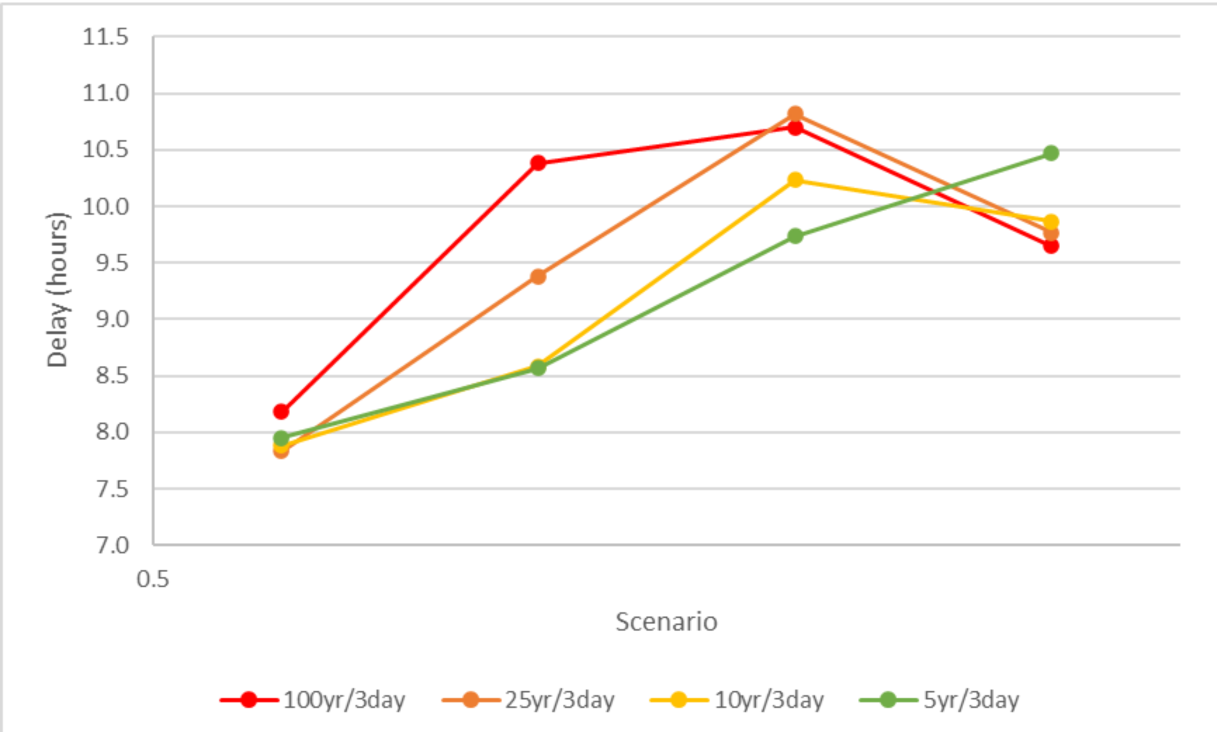


The maximum HW and TW exceed the design HW and TW for all simulations. The maximum HW and TW also exceed the water level which will overtop the gates for all conditions, sometimes occurring when the head differential is less than 0.1 feet, and the gates are therefore closed. TW elevations that exceed this elevation can result in flow entering the basin from storm surge and/or tide. The HW and TW at S22 exceeds the water level that will bypass the structure for the 100-year design storm for SLR2 and SLR3 future scenarios and for the 25-year design storm for the SLR3 future scenario. Flow that bypasses the structure can contribute to flooding of neighborhoods around S22.

The peak discharge at S22 exceeds the design value for all design storms during current conditions and future conditions SLR1 and SLR2. With 3 feet of SLR, S22 is unable to reach the design discharge for the 5-year design storm. The increase in TW levels due to SLR, decreases the head differential at the structure (as shown in **Figure 9-32**) and inhibits the flow out of the basin, potentially creating additional flooding upstream of the structure as shown in PM #1. Not only does high TW conditions inhibit flow from leaving the basin, as shown in **Table 9-10**, during most future conditions simulations, TW elevations exceed HW elevations and the level which will overtop the structure, resulting in flow entering the C2 basin from storm surge.

Without TW limitations, increased HW would generate more flow through a gravity structure (i.e., you would expect to see maximum flow through the structure occurring near the time of maximum HW). The time between the 12-hour moving average peak HW (**Table 9-9**) and the 12-hour moving average peak discharge (**Table 9-7**) is shown in **Figure 9-33** for each design storm. This delay in the structure's ability to discharge water is a result of high TW conditions from the storm surge and is shown to increase with increasing design storm return periods and future SLR scenarios. The delay decreases for the SLR3 simulation for the 10-year, 25-year, and 100-year design storms, most likely as a result of bypassing and overtopping of the structure for these scenarios.

Figure 9-33. Time Between Peak HW and Peak Discharge at S22 for the 12-Hour Moving Average



9.1.4. C2 – PM #4 PEAK STORM RUNOFF

The purpose of PM #4 is to determine the effect of SLR on the maximum peak storm runoff rate, or maximum conveyance capacity of the watershed (in cfs). For this metric, 12-hour moving average flow hydrographs downstream of S22 and the maximum 12-hour moving average total flow was determined for each design storm event and SLR scenario.

The 12-hour moving average discharge hydrographs for each SLR scenario can be found in **Figure 9-34** for the 100-year storm, **Figure 9-35** for the 25-year storm, **Figure 9-36** for the 10-year storm, and **Figure 9-37** for the 5-year storm. Downstream flows for S22 comprise of the discharge from the two gates in addition to overtopping of the structure, if applicable.

Figure 9-34. 12-Hour Moving Average of Flows at S22 for the 100-year Design Storm

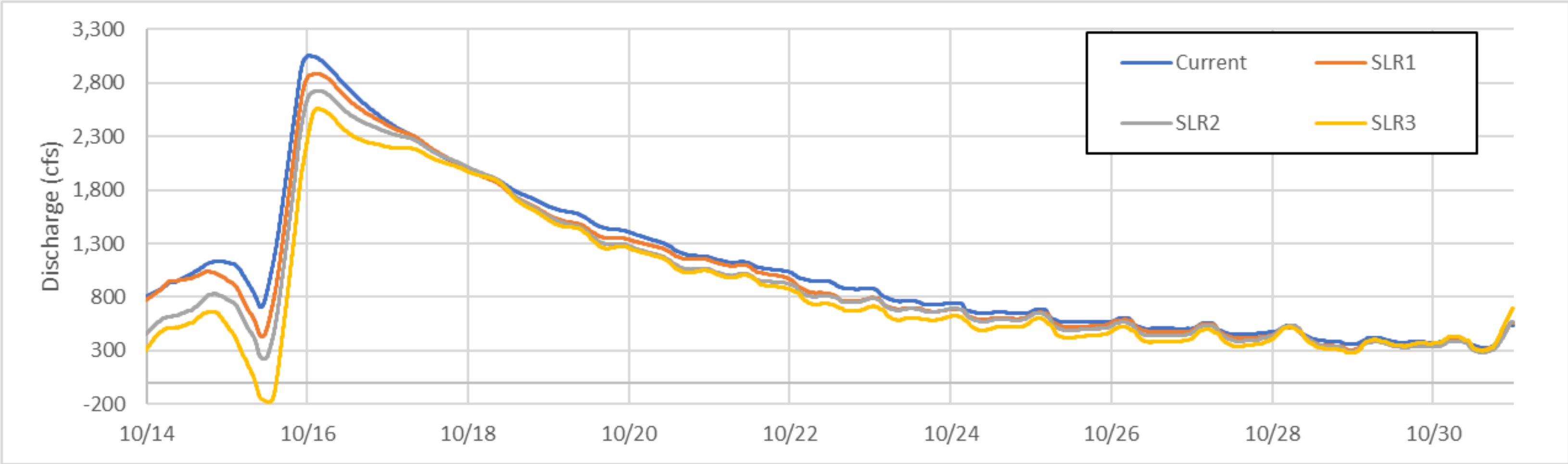


Figure 9-35. 12-Hour Moving Average of Flows at S22 for the 25-year Design Storm

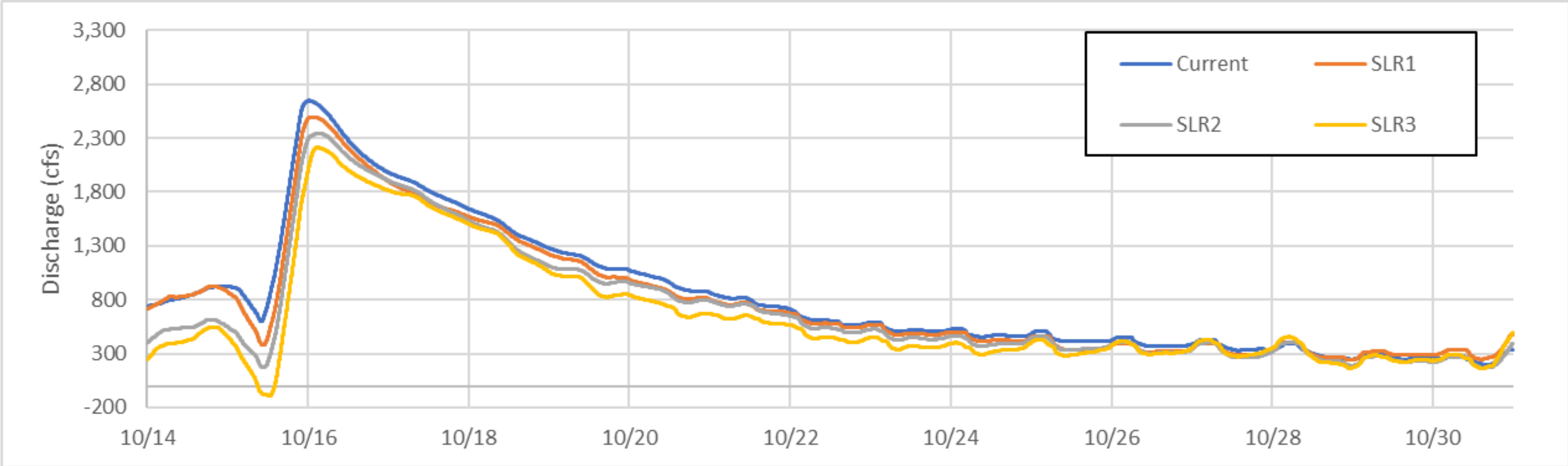


Figure 9-36. 12-Hour Moving Average of Flows at S22 for the 10-year Design Storm

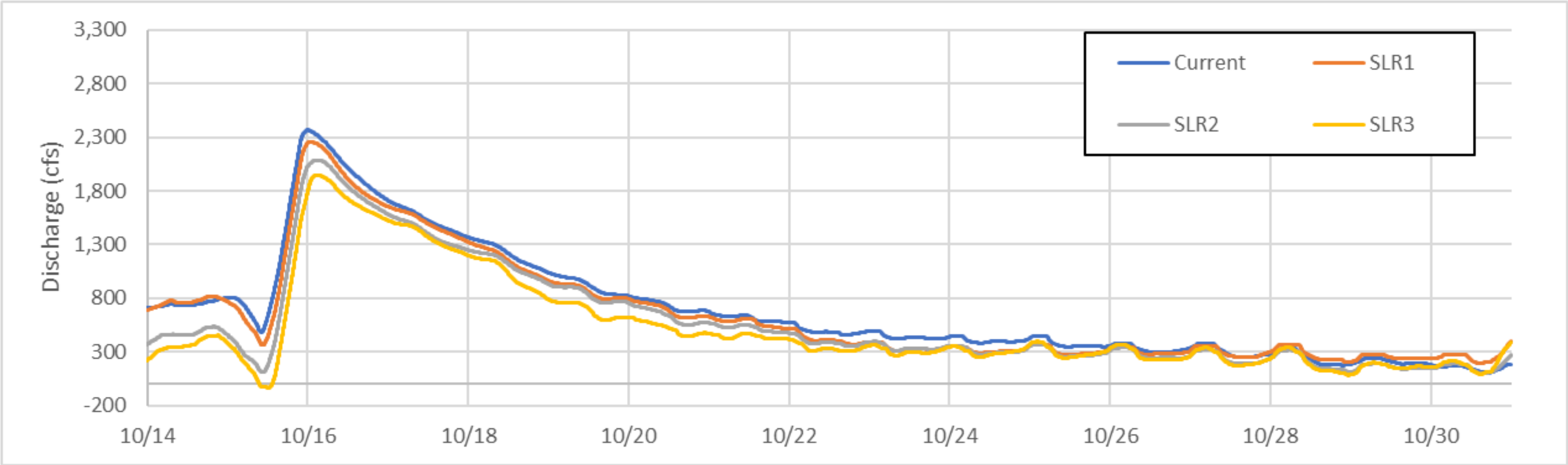
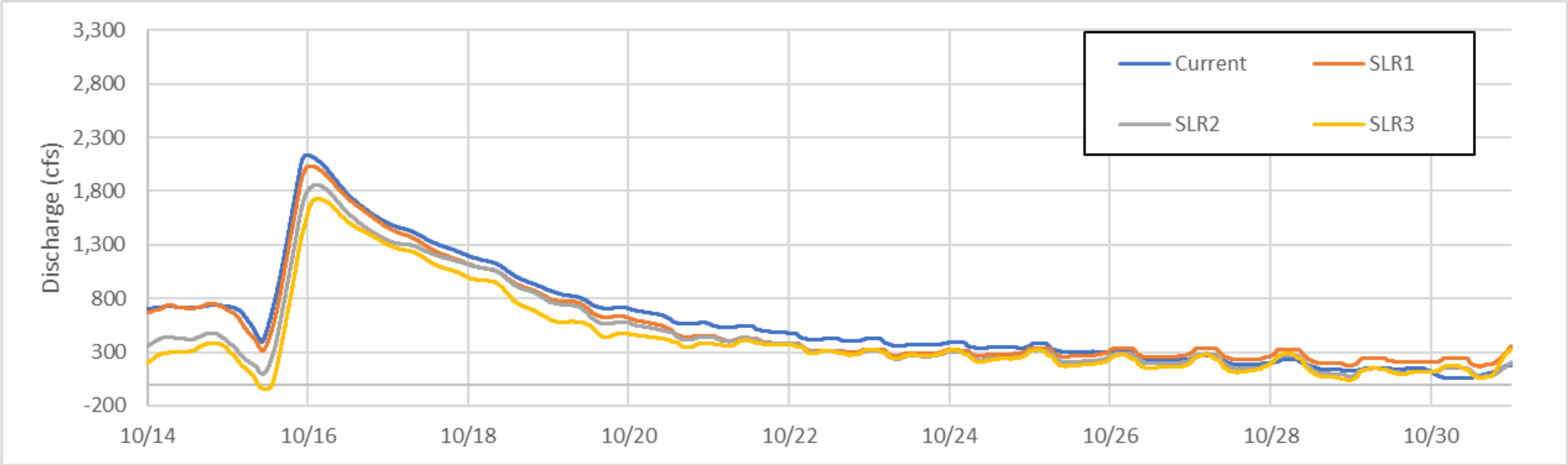


Figure 9-37. 12-Hour Moving Average of Flows at S22 for the 5-year Design Storm

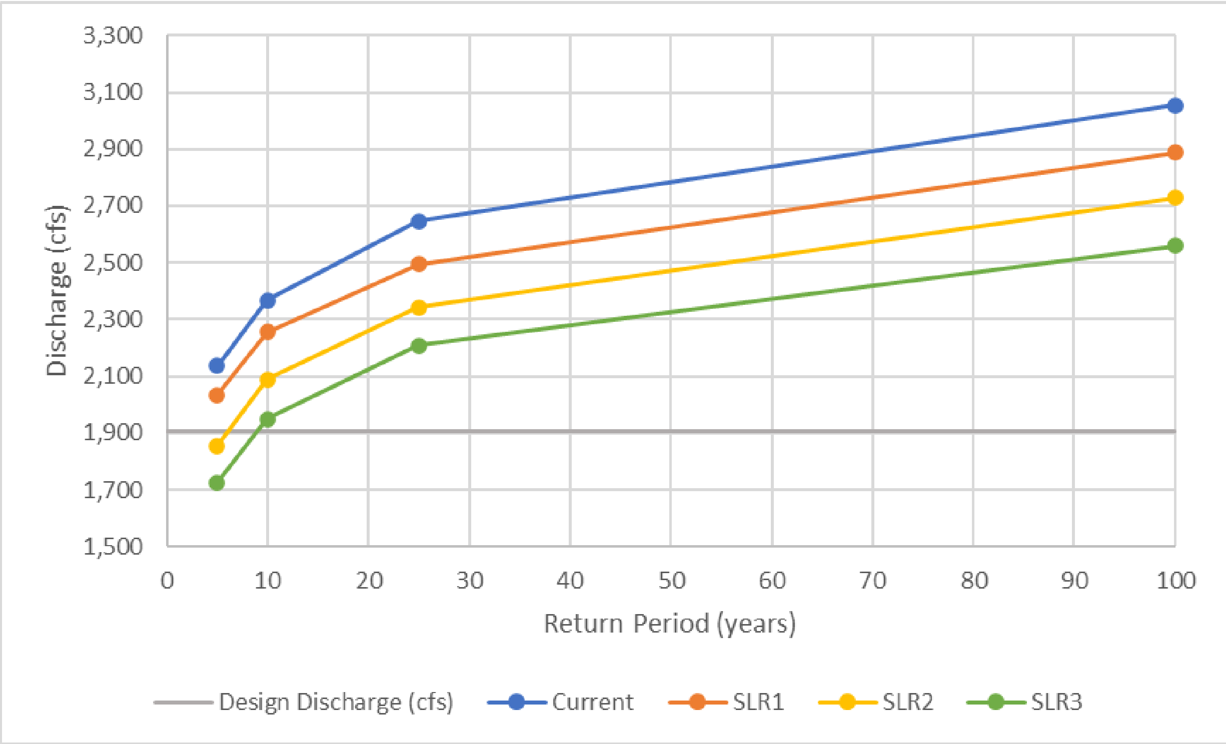


The instantaneous and 12-hour moving average peak discharges for all the design storm event return periods and SLR scenarios are shown in **Table 9-12**. In addition, the percentage difference between the current conditions 12-hour moving average peak discharge and each future conditions SLR scenario is calculated for all simulations. The 12-hour moving average peak discharges are also shown in **Figure 9-38**. The peak 12-hour moving average discharge decreases with increasing SLR for each design storm return period. As discussed previously, certain design storm scenarios are unable to meet the design discharge when simulated under future SLR conditions.

Table 9-12. Peak Discharge Summary at S22

Design Storm Return Period	Scenario	Peak Discharge (CFS)		12-Hour Moving Average Peak Flow Reduction Percentage
		Instantaneous	12-Hour Moving Average	
100-Year	**Current**	3164.77	3037.97	N/A
	SLR1	2982.90	2886.20	5.00%
	SLR2	2854.69	2728.15	10.20%
	SLR3	2709.43	2559.77	15.74%
25-Year	**Current**	2747.48	2631.27	N/A
	SLR1	2589.22	2494.27	5.21%
	SLR2	2448.80	2343.21	10.95%
	SLR3	2326.38	2209.76	16.02%
10-Year	**Current**	2459.96	2346.71	N/A
	SLR1	2354.10	2256.20	3.86%
	SLR2	2183.78	2088.71	10.99%
	SLR3	2063.22	1949.21	16.94%
5-Year	**Current**	2244.00	2126.41	N/A
	SLR1	2145.65	2033.41	4.37%
	SLR2	1927.47	1856.51	12.69%
	SLR3	1834.20	1725.02	18.88%

Figure 9-38. Peak 12-Hour Moving Average Discharge at S22



9.1.5. C2 – PM #5 FREQUENCY OF FLOODING

The maximum overland depth was extracted for each design storm event and SLR condition and evaluated for the C2 Watershed. **Table 9-13** tabulates the flood inundation area in square miles for the C2 Watershed. The total area of the C2 Watershed for this analysis was calculated as 52.4 square miles (slight variations in total area from the District total area are due to estimating basin shape along a coarse grid).

The total area of the C2 Watershed considered Urban is 38.5 square miles, which makes this watershed mostly urban (about 77% urban). **Table 9-14** tabulates the flood inundation area in square miles for the urban areas within the C2 Watershed. This table shows that the greatest distribution of flooding depths in the watershed are between zero and 0.25 feet of flooding, which can be considered nuisance flooding. **Figure 9-39** shows the urban inundation with the same incremental flooding depths as the table for the 100-year storm event, and **Figure 9-40** does the same for the 25-year storm. This provides a clear view of how stages are increasing with sea level rise in this watershed. The graphics show that the area of inundation greater than 1.75 feet does not tend to increase for the SLR1 simulation; however, the area does increase for the SLR2 and SLR3 simulations.

Table 9-15 shows the percentage of the total urban areas within the C2 Watershed that is above the flooding depth. This data shows that less than half of the total urban areas in the C2 Watershed has 3 inches or greater of flooding depth (0.25 feet) for the 100-year design storm event for current conditions, this increases steadily for each SLR simulation by about 3% for each foot of SLR. Approximately a quarter of the watershed experiences 3 inches or greater of flooding during the 25-year storm for current conditions, which increases by about 2.5% for each foot of additional SLR. For the 10-year and 5-year storms, the area of inundation increases by about 2% for each foot of SLR, except for SLR3, which increases by 3% above the SLR2 simulation.

Figure 9-41 and **Figure 9-42** are maps of the maximum overland depth over the entire C2 Watershed for the 25-year and 100-year 3-day design storm events, respectively. Water areas, such as existing lakes and ponds, are masked in black. **Figure 9-43** and **Figure 9-44** provide the same maps with the non-urban areas masked out, to provide a concise picture of how urban areas are impacted within the watershed. As evident in these figures, a majority of the inundated areas in the C2 basin are non-urban.

Figure 9-45, **Figure 9-46**, and **Figure 9-47** show the difference in overland flooding for the C2 Watershed between the current conditions and future sea level rise conditions for the SLR +1 foot, SLR +2 feet, and SLR +3 feet simulations, respectively. Because the rainfall is the same in all the 100-year storm event simulations, these difference maps remove any overland flooding caused by rainfall and show how much is now impacted by rising seas in terms of direct flooding from the canals, or reduced drainage capacity due to higher stages in the primary canal. Where higher canal stages are due to reduced discharge capacity at the structure as well as structure backflow due to overtopping and structure bypass, as discussed in the previous sections.

Maps of the maximum overland depth over the entire C2 Watershed, and only urban areas, for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current conditions and future SLR scenarios are provided in **Appendix C**. Also provided in **Appendix C** are the differences in overland flooding maps for the 25-year, 10-year, and 5-year design storms.

As evident in **Figure 9-43** and **Figure 9-44**, a majority of the inundated areas in the C2 basin are non-urban. There are some areas, primarily north of the Ludlam Glades Canal and in the area of Olympia Heights, east of the Westwood Lakes Canal that experiences 1 to 2 feet of water during the 100-year storm. These locations, and the potential cause of flooding is discussed further in **Section 9.1.6**.

Table 9-13. Incremental Flood Inundation Area (sq. mi.) in the C2 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	23.19	32.30	36.62	39.35	22.36	31.71	36.04	38.82	20.85	30.50	35.01	37.86	19.41	28.79	33.40	36.20
0.25 =< Depth < 0.50	8.23	4.55	3.29	2.59	8.68	4.79	3.71	3.37	9.05	5.03	3.87	3.29	9.14	5.49	4.09	3.50
0.50 =< Depth < 0.75	4.33	3.02	2.49	2.12	4.38	3.48	3.32	3.01	4.61	3.49	3.26	3.16	4.83	3.61	3.09	2.94
0.75 =< Depth < 1.00	2.85	2.33	2.05	2.08	3.07	3.18	2.84	2.45	3.24	2.99	2.78	2.39	3.39	2.66	2.82	2.68
1.00 =< Depth < 1.25	2.13	2.09	2.28	2.40	2.63	2.66	2.16	1.77	2.41	2.69	2.21	1.92	2.43	2.66	2.37	1.92
1.25 =< Depth < 1.50	1.91	2.22	2.36	1.65	2.59	1.97	1.50	0.97	2.44	2.12	1.71	1.35	2.18	2.29	1.83	1.65
1.50 =< Depth < 1.75	1.97	2.20	1.18	0.53	2.34	1.49	0.77	0.38	2.38	1.59	1.16	0.63	2.27	1.85	1.48	1.17
1.75 =< Depth < 2.00	2.30	1.30	0.42	0.18	1.66	0.83	0.39	0.16	1.94	1.23	0.51	0.23	2.12	1.40	0.96	0.49
2.00 =< Depth < 2.25	1.87	0.53	0.17	0.14	1.39	0.42	0.16	0.10	1.38	0.63	0.28	0.12	1.65	0.97	0.45	0.26
2.25 =< Depth < 2.50	1.02	0.21	0.11	0.13	0.82	0.24	0.11	0.15	1.14	0.34	0.15	0.13	1.36	0.57	0.24	0.13
2.50 =< Depth < 2.75	0.57	0.13	0.16	0.10	0.43	0.14	0.14	0.10	0.63	0.19	0.13	0.11	0.87	0.31	0.17	0.14
2.75 =< Depth < 3.00	0.26	0.12	0.09	0.04	0.31	0.13	0.11	0.07	0.41	0.15	0.10	0.08	0.56	0.21	0.13	0.10
3.00 =< Depth	1.80	1.44	1.21	1.12	1.79	1.40	1.21	1.11	1.95	1.48	1.29	1.16	2.24	1.63	1.40	1.27

Total Basin Area = 52.4 square miles

Table 9-14. Incremental Flood Inundation Area (sq. mi.) for Urban Areas in the C2 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	20.68	28.68	32.29	34.42	19.78	27.93	31.57	33.73	18.45	26.93	30.74	33.00	17.21	25.46	29.42	31.76
0.25 =< Depth < 0.50	7.33	3.84	2.49	1.67	7.66	4.07	2.75	1.91	8.02	4.32	3.02	2.06	8.11	4.75	3.41	2.54
0.50 =< Depth < 0.75	3.76	2.29	1.47	0.87	3.82	2.45	1.51	1.06	4.03	2.68	1.69	1.28	4.21	2.98	1.99	1.43
0.75 =< Depth < 1.00	2.32	1.27	0.73	0.49	2.47	1.35	0.90	0.63	2.73	1.49	0.95	0.67	2.91	1.68	1.13	0.82
1.00 =< Depth < 1.25	1.32	0.72	0.40	0.26	1.47	0.81	0.55	0.34	1.55	0.83	0.61	0.44	1.85	0.98	0.64	0.48
1.25 =< Depth < 1.50	0.77	0.39	0.27	0.16	0.83	0.47	0.33	0.21	0.95	0.56	0.38	0.29	1.05	0.64	0.43	0.36
1.50 =< Depth < 1.75	0.50	0.28	0.18	0.09	0.57	0.37	0.20	0.12	0.64	0.37	0.28	0.19	0.67	0.40	0.33	0.28
1.75 =< Depth < 2.00	0.36	0.25	0.10	0.05	0.36	0.23	0.16	0.06	0.39	0.31	0.18	0.09	0.50	0.30	0.27	0.21
2.00 =< Depth < 2.25	0.28	0.15	0.06	0.07	0.31	0.17	0.06	0.04	0.30	0.24	0.14	0.05	0.33	0.28	0.21	0.12
2.25 =< Depth < 2.50	0.23	0.08	0.06	0.05	0.28	0.13	0.05	0.07	0.30	0.17	0.07	0.06	0.30	0.23	0.12	0.07
2.50 =< Depth < 2.75	0.20	0.06	0.07	0.07	0.18	0.07	0.06	0.04	0.25	0.09	0.06	0.06	0.25	0.18	0.09	0.07
2.75 =< Depth < 3.00	0.13	0.07	0.04	0.02	0.16	0.06	0.05	0.04	0.20	0.08	0.04	0.04	0.26	0.10	0.07	0.05
3.00 =< Depth	0.66	0.47	0.37	0.31	0.64	0.44	0.35	0.29	0.73	0.48	0.39	0.32	0.88	0.56	0.44	0.38

Total Basin Area = 38.5 square miles

Table 9-15. Percentage of Total Watershed Area with Inundated Area for Urban Areas in the C2 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0.00	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 0.25	46.3	25.6	16.2	10.7	48.7	27.5	18.1	12.5	52.1	30.1	20.2	14.4	55.4	33.9	23.7	17.6
>= 0.50	27.3	15.6	9.8	6.4	28.8	17.0	10.9	7.5	31.3	18.9	12.4	9.0	34.3	21.6	14.8	11.0
>= 0.75	17.6	9.7	5.9	4.1	18.9	10.6	7.0	4.8	20.9	12.0	8.0	5.7	23.4	13.9	9.7	7.3
>= 1.00	11.5	6.4	4.0	2.8	12.5	7.1	4.7	3.2	13.8	8.1	5.6	4.0	15.8	9.5	6.7	5.2
>= 1.25	8.1	4.5	3.0	2.1	8.7	5.0	3.3	2.3	9.8	5.9	4.0	2.8	11.0	7.0	5.1	3.9
>= 1.50	6.1	3.5	2.3	1.7	6.5	3.8	2.4	1.7	7.3	4.5	3.0	2.1	8.3	5.3	4.0	3.0
>= 1.75	4.8	2.8	1.8	1.5	5.0	2.9	1.9	1.4	5.6	3.5	2.3	1.6	6.5	4.3	3.1	2.3
>= 2.00	3.9	2.1	1.6	1.3	4.1	2.3	1.5	1.3	4.6	2.7	1.8	1.4	5.2	3.5	2.4	1.8
>= 2.25	3.2	1.8	1.4	1.2	3.3	1.8	1.3	1.2	3.8	2.1	1.4	1.3	4.4	2.8	1.8	1.4
>= 2.50	2.6	1.6	1.3	1.0	2.6	1.5	1.2	1.0	3.1	1.7	1.3	1.1	3.6	2.2	1.5	1.3
>= 2.75	2.0	1.4	1.1	0.9	2.1	1.3	1.0	0.9	2.4	1.4	1.1	0.9	3.0	1.7	1.3	1.1
>= 3.00	1.7	1.2	0.9	0.8	1.7	1.1	0.9	0.8	1.9	1.2	1.0	0.8	2.3	1.5	1.1	1.0

Figure 9-39. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C2 Watershed for the 100-year Design Storm

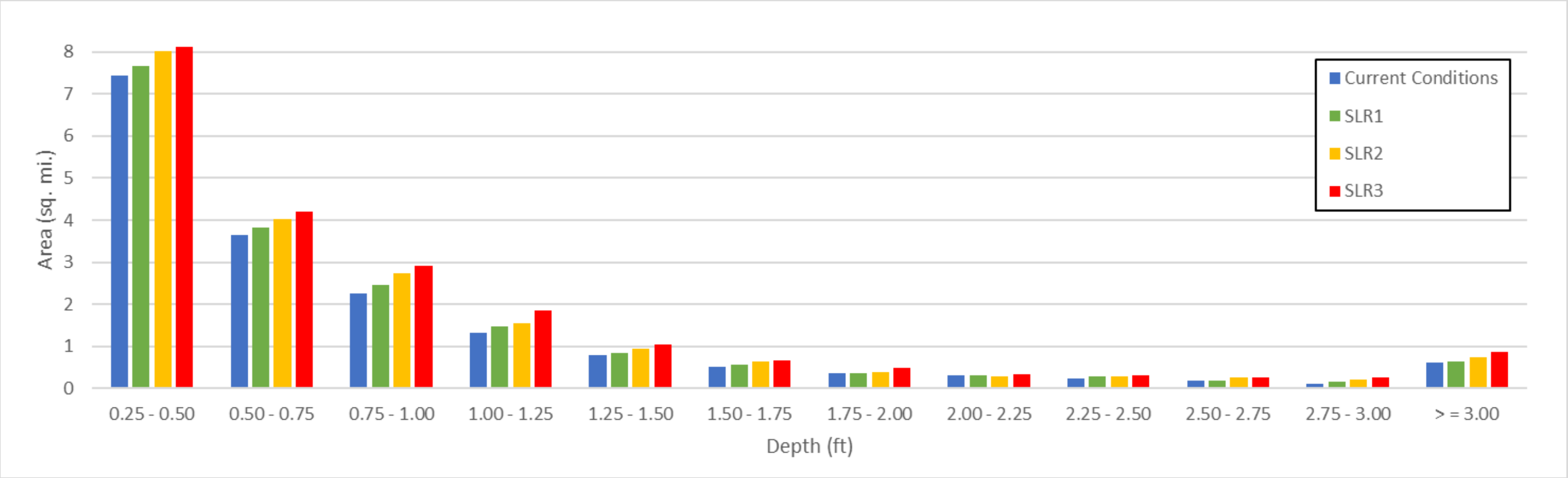


Figure 9-40. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C2 Watershed for the 25-year Design Storm

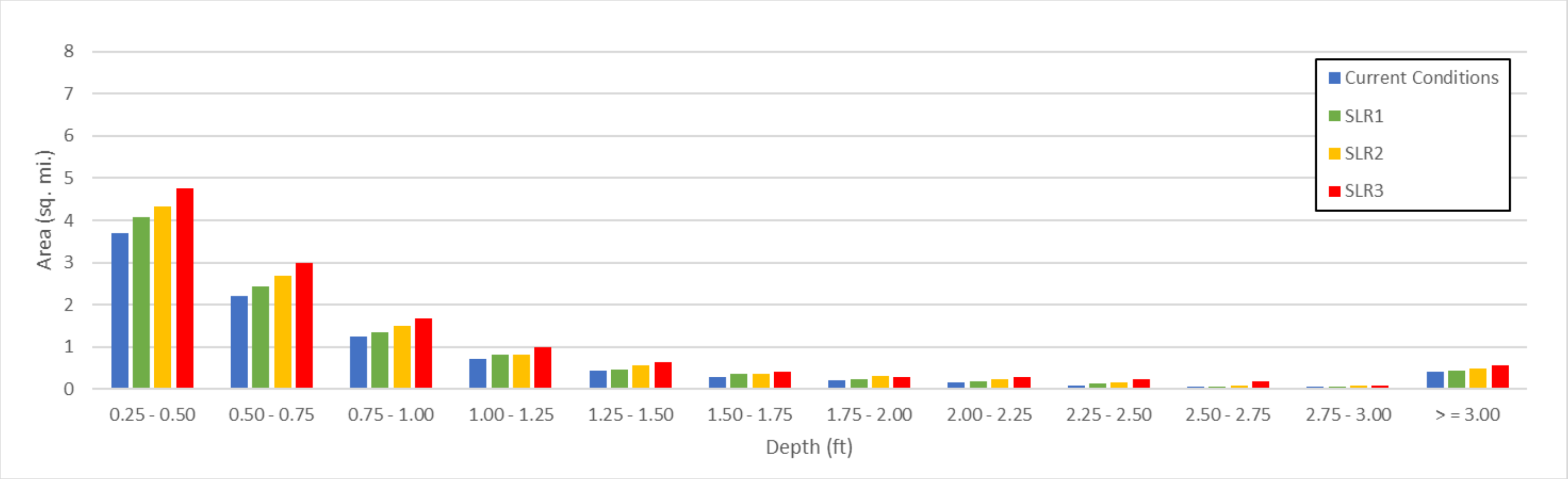


Figure 9-41. Maximum Overland Flood Depth for the Current Conditions 25-year 3-Day Design Storm in the C2 Watershed

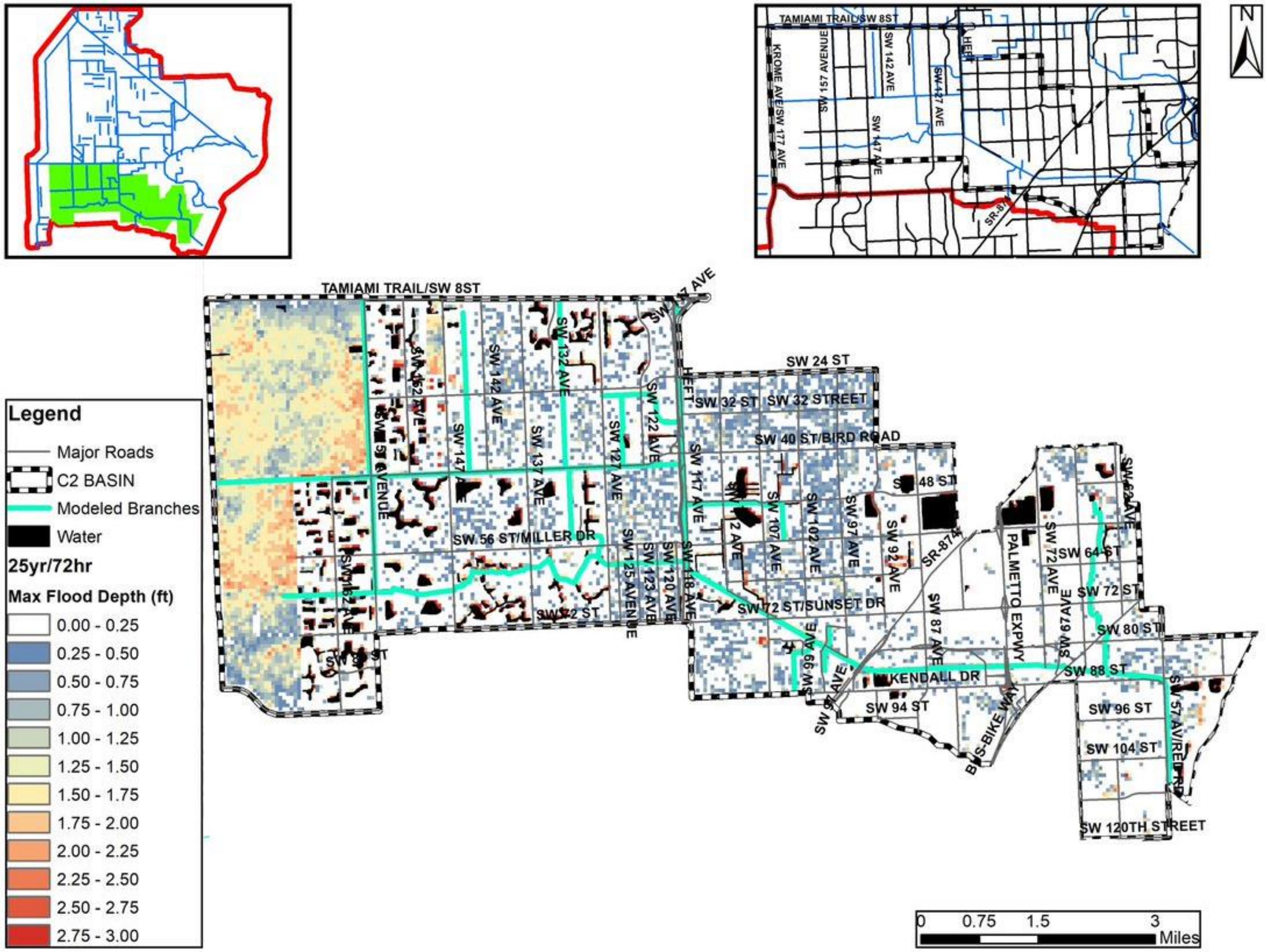


Figure 9-42. Maximum Overland Flood Depth for the Current Conditions 100-year 3-Day Design Storm in the C2 Watershed

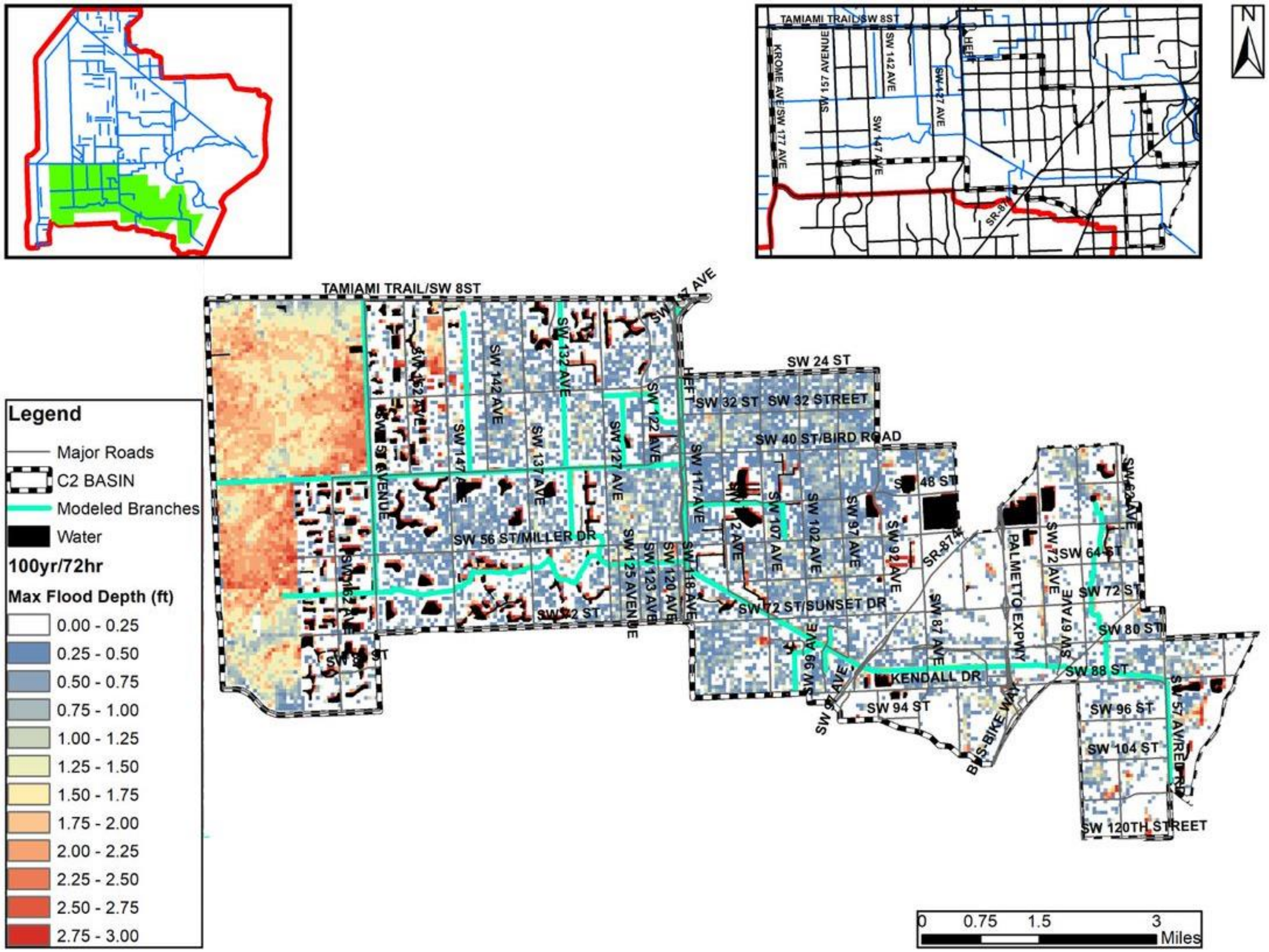


Figure 9-43. Maximum Overland Flood Depth for the Current Conditions 25-year 3-Day Design Storm for Urban Areas in the C2 Watershed

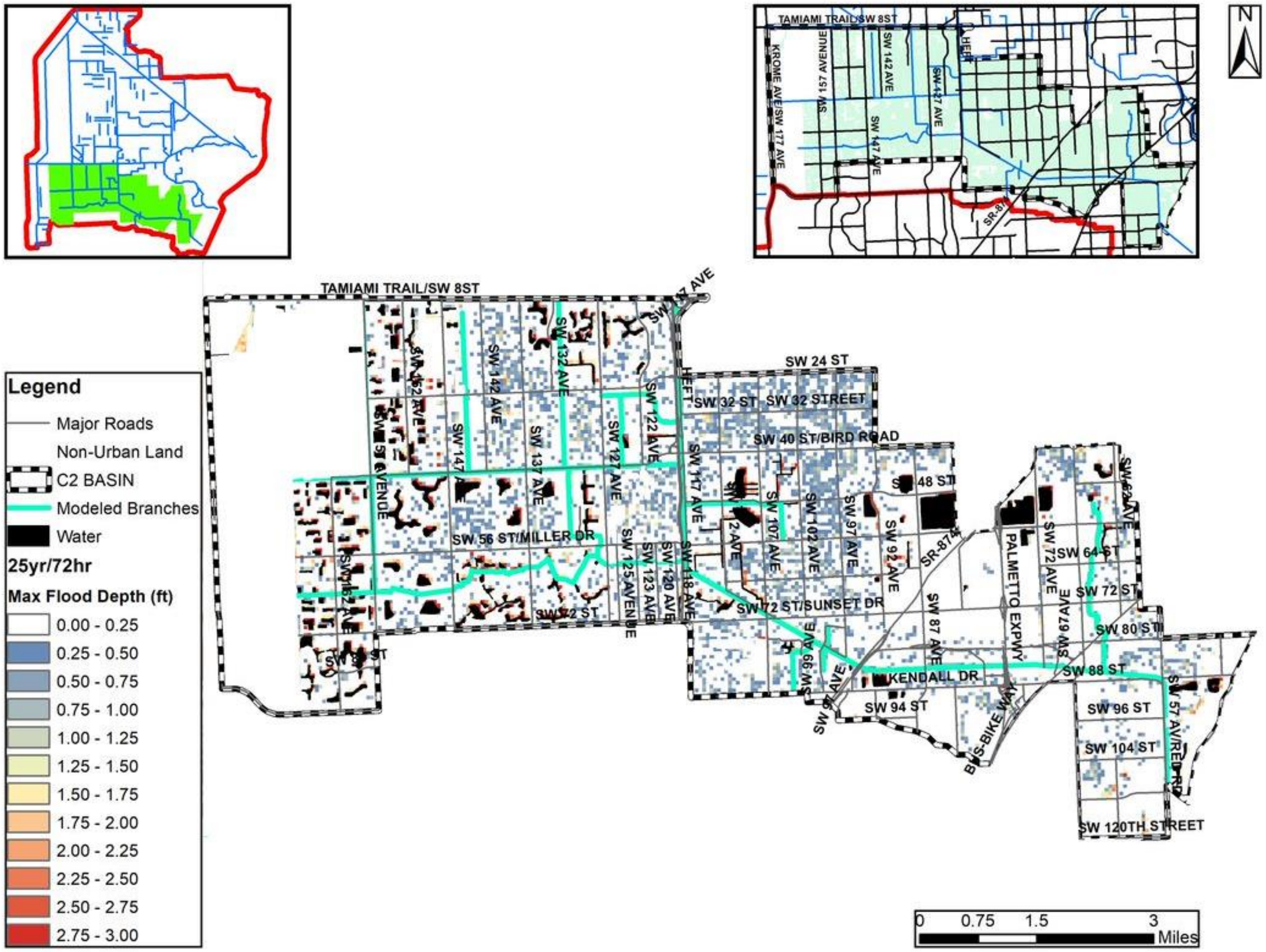


Figure 9-44. Maximum Overland Flood Depth for the Current Conditions 100-year 3-Day Design Storm for Urban Areas in the C2 Watershed

Figure 9-45. Urban Flooding Depth Difference of SLR +1ft and Current Conditions for the 100-year Storm in the C2 Watershed

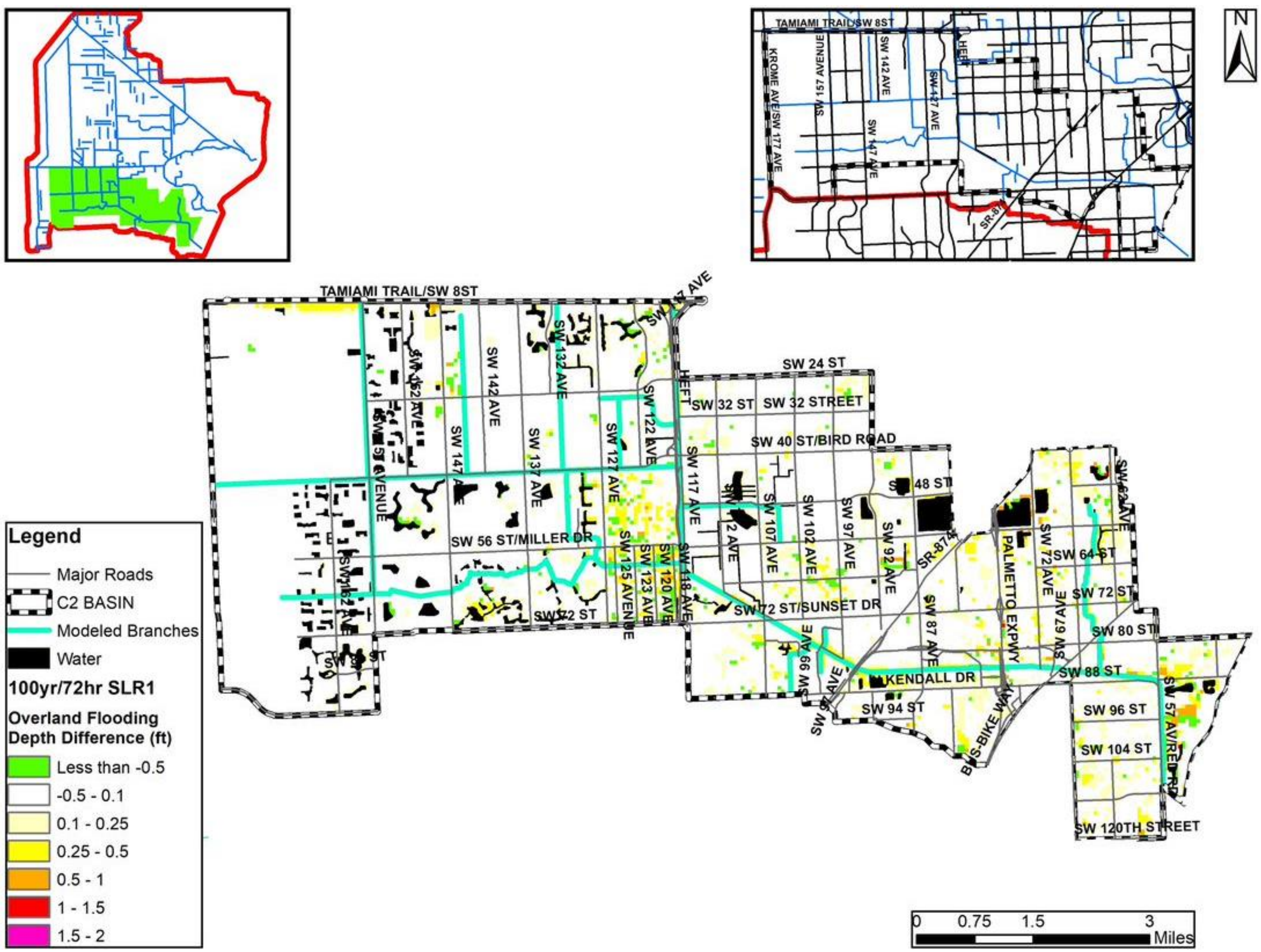


Figure 9-46. Urban Flooding Depth Difference of SLR +2ft and Current Conditions for the 100-year Storm in the C2 Watershed

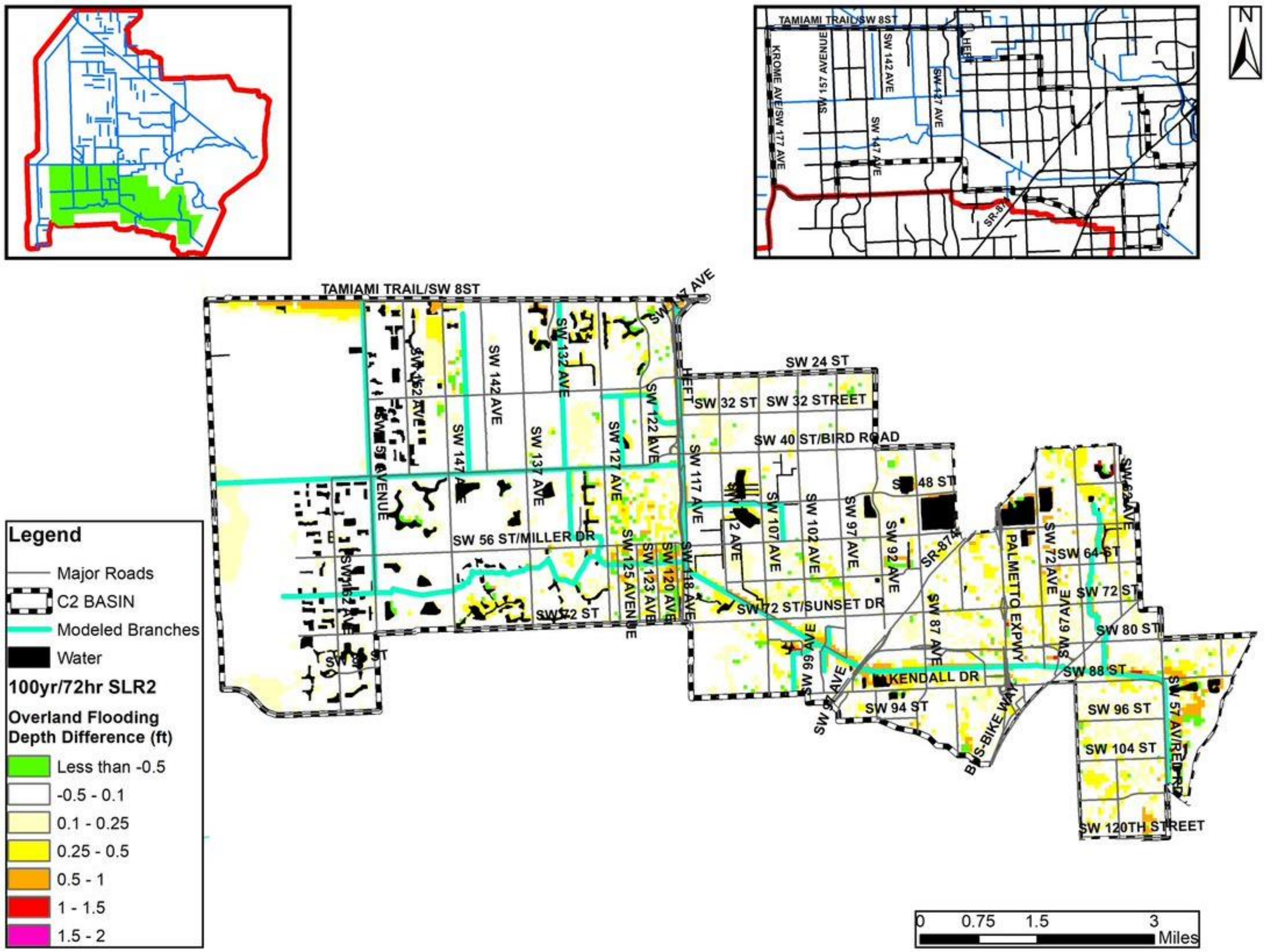
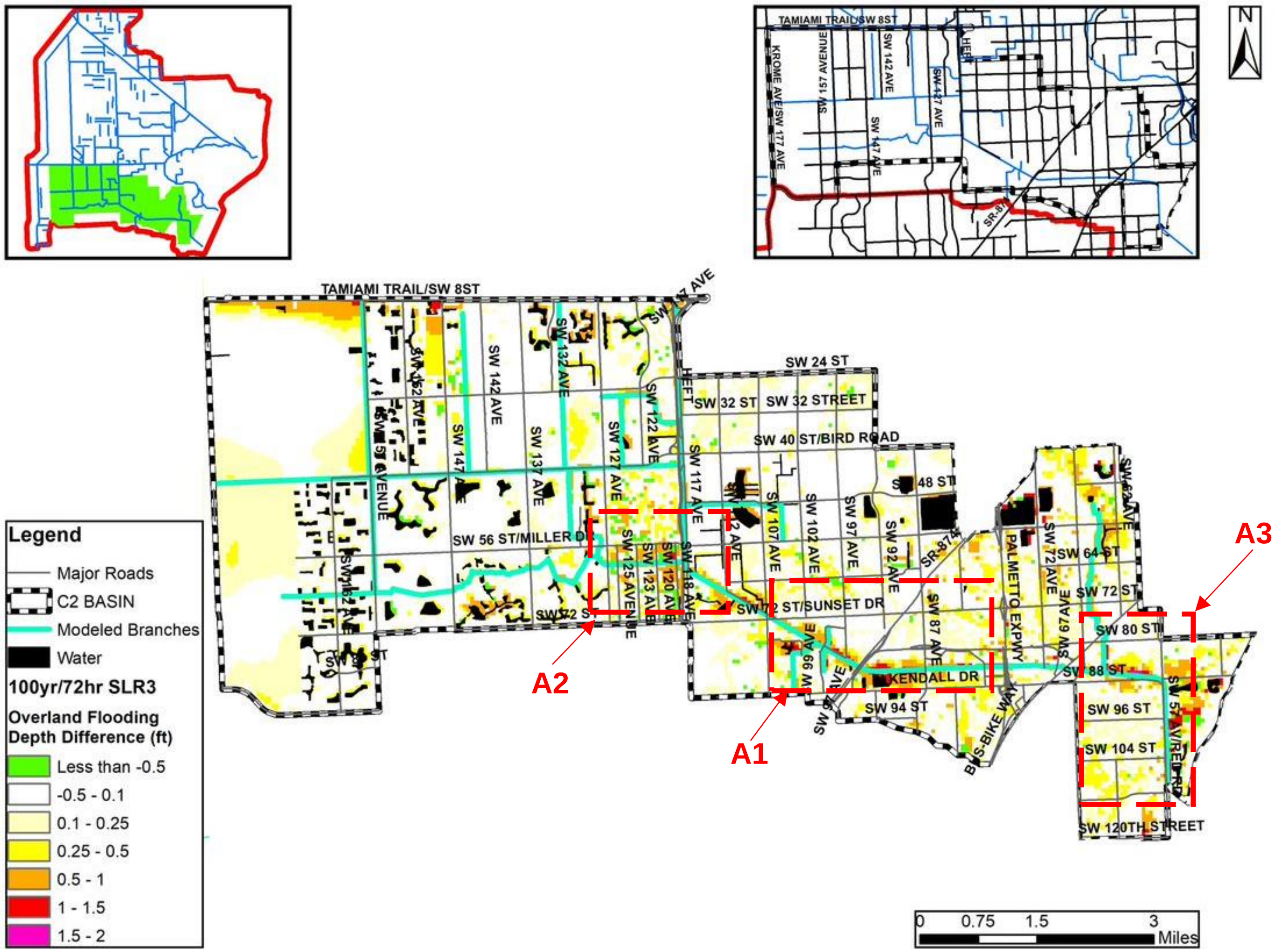


Figure 9-47. Urban Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C2 Watershed



Areas such as the southern bend in the Snapper Creek Canal around Kendall (SW 88th St) and Red Road (SW 57th Avenue) (indicated by 'A3' in **Figure 9-47**) can be impacted by over 1 foot above the current conditions with the SLR3 simulation. A zoomed in look at this neighborhood is shown in **Figure 9-48**. Additional areas, including just east of SR874 (Don Shula Expressway) near the Snapper Creek Canal (indicated by 'A2' in **Figure 9-47**) and areas west of the Canal at SW 60th St (indicated by 'A1' in **Figure 9-47**), may be impacted up to 1.0 to 1.5 feet above the current conditions with the SLR3 simulation. A zoomed in look at these neighborhoods are shown in **Figure 9-49** and **Figure 9-50**.

Figure 9-48. Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C2 Watershed – Kendall and Red Road (A3)

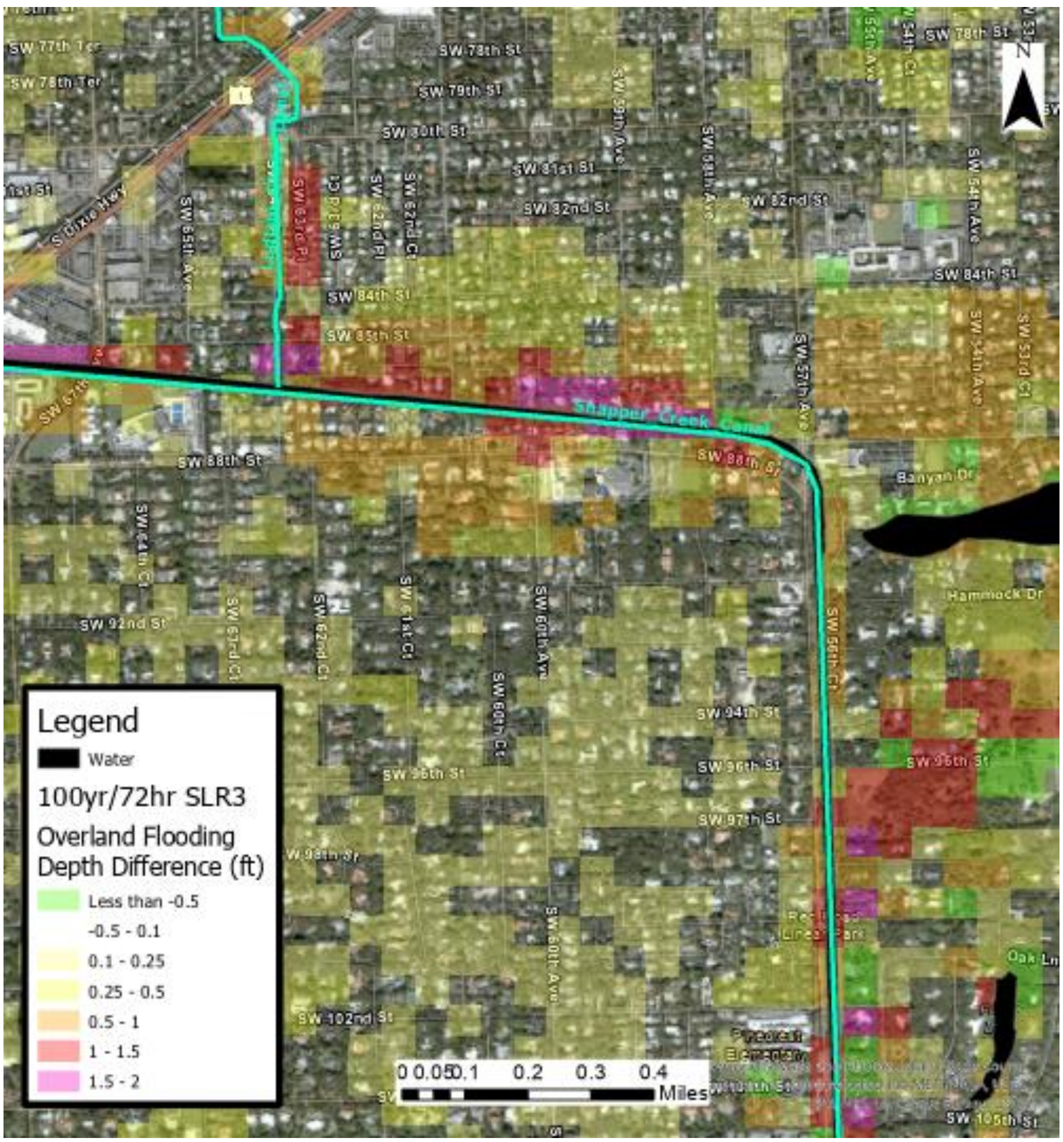


Figure 9-49. Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C2 Watershed – Snapper Creek Canal and SR874 (Don Shula Expressway) (A1)

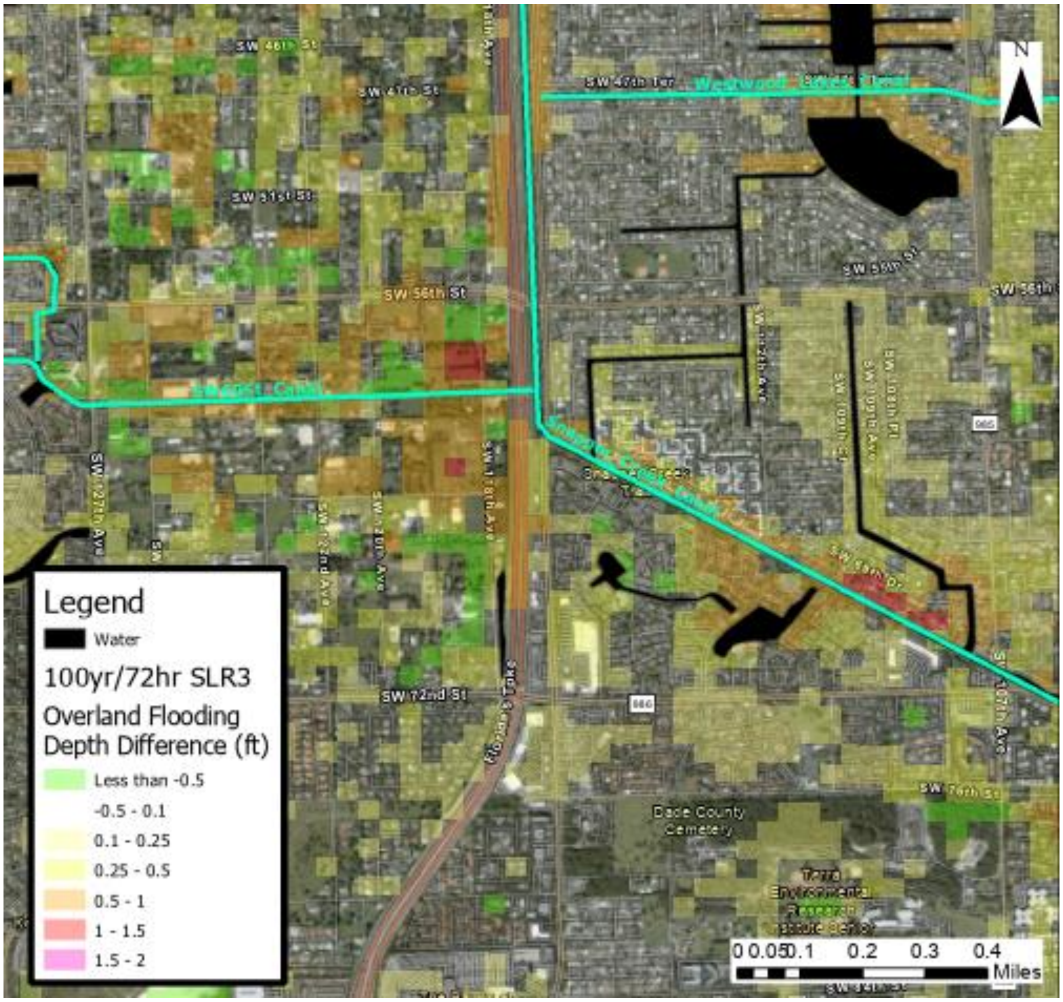
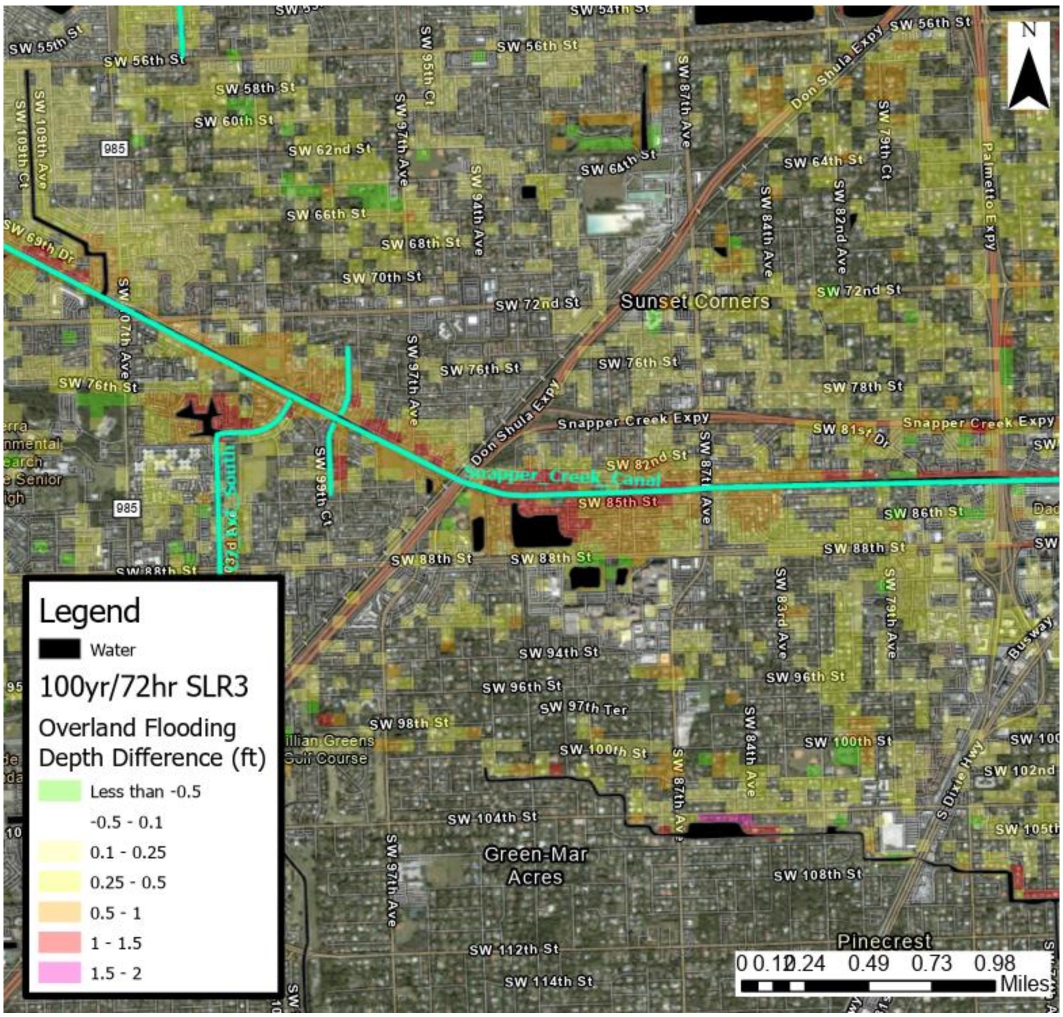


Figure 9-50. Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C2 Watershed – Snapper Creek Canal and SR874 (Don Shula Expressway) (A1)



9.1.6. C2 – PM #6 DURATION OF FLOODING

9.1.6.1. CANAL FLOOD DURATION

In discussions with water managers at SFWMD, it was reported that stages at the T5W station (near the intersection of the C2 and C4 Canals) are an indication of whether the storm has subsided within the areas upstream of the S22, G93, and S25B water control structures. The S25B Forward Pump Station uses stages at T5W as one indicator of when to trigger flood control operations in the study area. If stages at T5W exceed 3.80 ft-NGVD (or 2.228 ft-NAVD using the conversion factor of -1.572 feet), and other conditions are also met, the pump will turn on as described in the C4 Basin Operation Plan (SFWMD, 2019). Under current conditions, stages at T5W for each design storm were evaluated and this 2.228 ft-NAVD trigger was used to identify when the storm would be considered initiated and finalized, establishing the Reference Elevation for the T5W station. However, this reference elevation would not be acceptable under future SLR conditions, as the storms, and even the normal wet season canal elevations may be higher than 2.228 ft-NAVD.

A new reference elevation was established based on the off-trigger elevations used for the municipal pump stations that pump into the C4 Canal. The pumps are meant to turn off if stages in the C4 Canal exceed 5 ft-NGVD, or 3.428 ft-NAVD. Since this trigger has already been established by the County and municipalities as an indicator of when the C4 stages are too high, this makes an appropriate indicator for when the C2, C3W, and C4 watersheds are still in a flood condition for the canals. **Table 9-16** shows the canal flood duration for each storm event and SLR condition at T5W. For the SLR3 for all design storm events, canal stages do not recede past the reference elevation after the storm and therefore the storm duration is longer than the values provided.

Table 9-16. Storm Duration Indicated at T5W

Design Storm	Duration (hours)				
	Current Conditions with 2.23 ft-NAVD Reference Elev.	Current Conditions with 3.43 ft-NAVD Reference Elev.	SLR1	SLR2	SLR3
100-Year	281.8	119.6	162.1	282.6	420.6*
25-Year	184.3	67.6	108.2	217.7	410.4*
10-Year	140.3	40.8	68.6	149.8	408.3*
5-Year	101.6	24.9	47.1	120.1	398.1*
*Canal stages do not recede past the Reference Elevation after the storm and therefore the storm duration is longer than the values provided.					

Figure 9-51 shows the hydrographs at T5W station, comparing the SLR conditions for each design storm. As described in the table and shown in the figure, the Reference Elevation of 3.428 ft-NAVD is shown on the graphs as a grey line. **Figure 9-52** shows the HW stages at S22 comparing the 100-year storm results for each SLR condition, and the vertical lines indicate the start and end of each design storm as determined at T5W. **Figure 9-53** shows the same, but for the 25-year storm.

Figure 9-51. Stages at T5W Station and Storm Duration for C2, C3W and C4 Watersheds

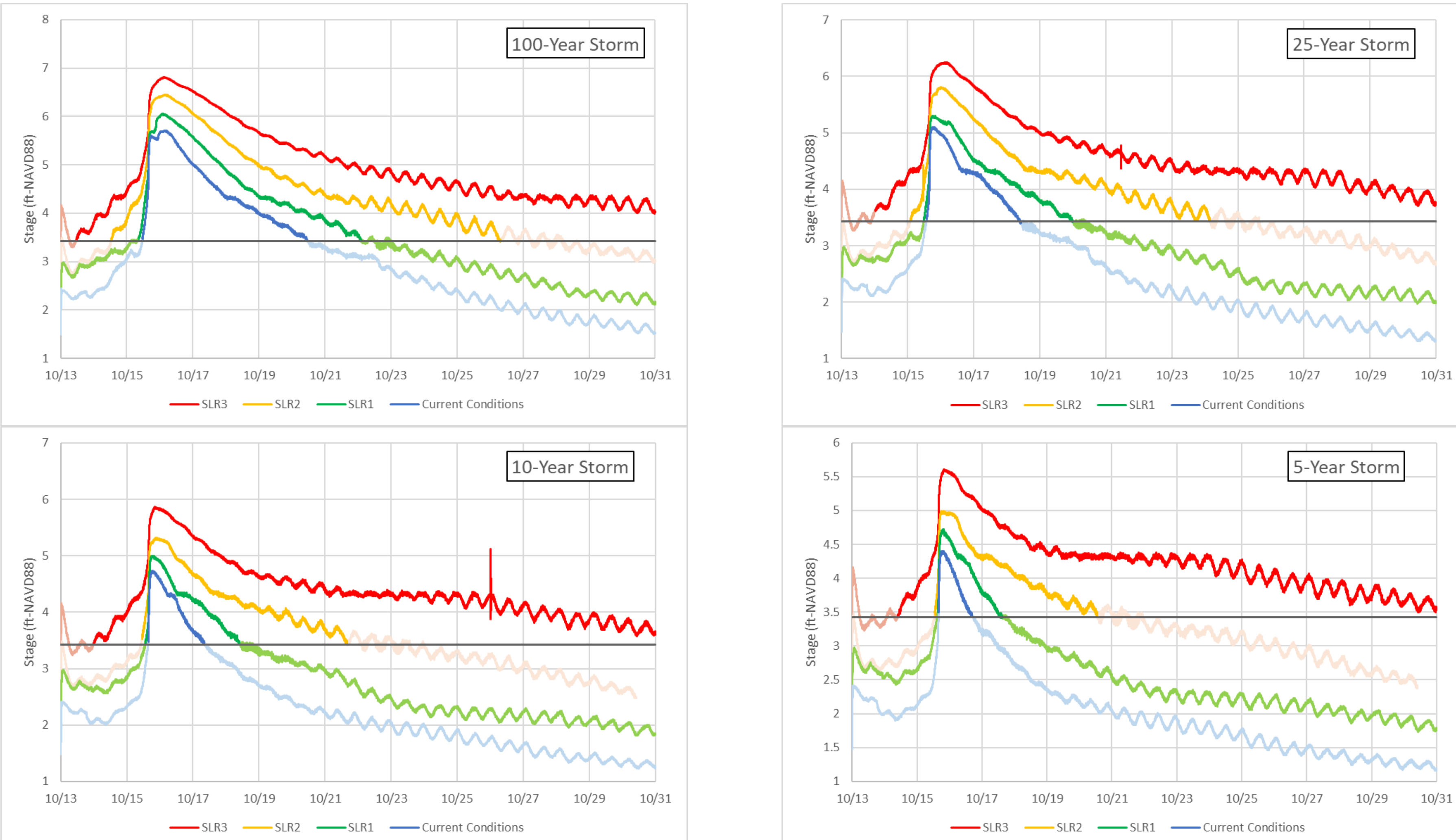


Figure 9-52. T5W Flood Duration Compared with HW at S22 for the 100-year Storm

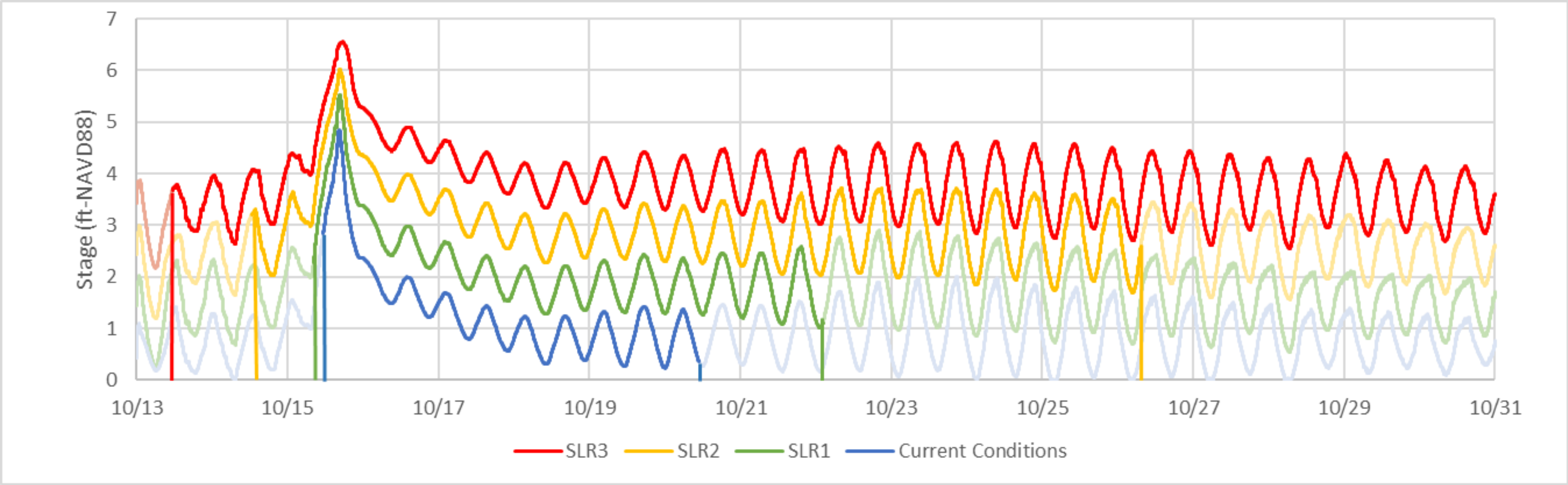
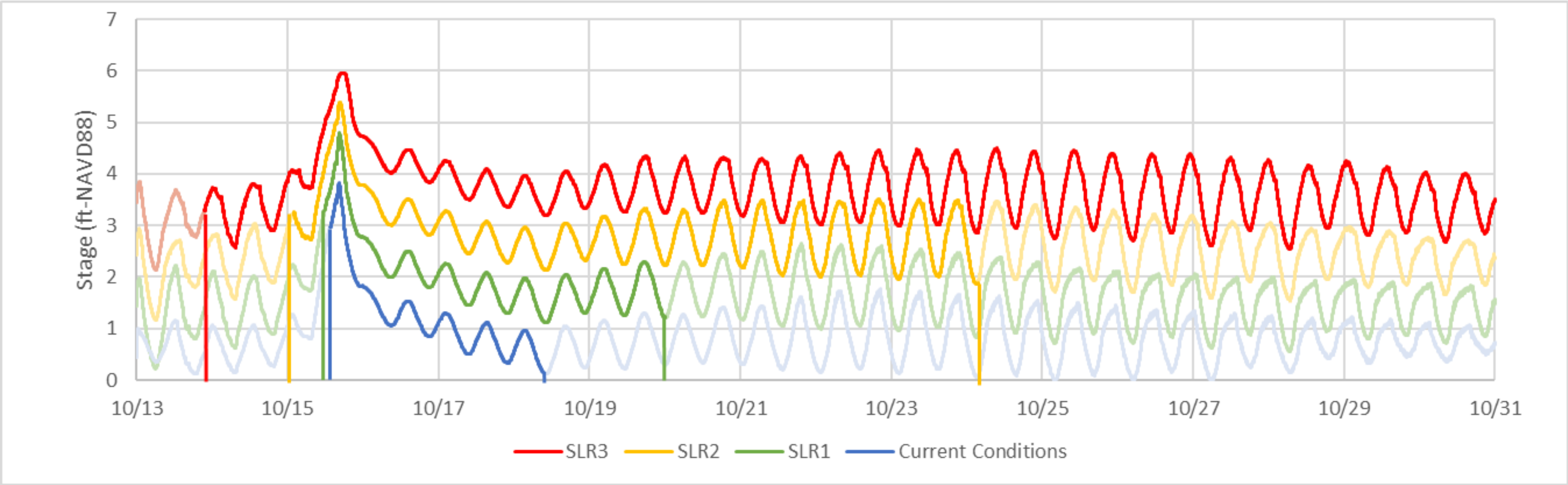


Figure 9-53. T5W Flood Duration Compared with HW at S22 for the 25-year Storm



9.1.6.2. WATERSHED FLOOD DURATION

Table 9-17 tabulates the total area of flood inundation (in square miles) per flood duration range for all areas in the C2 Watershed and **Table 9-18** does this for all urban areas within the C2 Watershed. This table shows that the greatest distribution of flooding duration in the watershed is between zero and one hour of flooding. However, since the model outputs overland depths every hour, the first hour of flooding may be anywhere from 1 minute to 1 hour long. Therefore, the first hour of flooding is considered nuisance flooding for the purposes of this analysis.

Flooding is also distributed in the 12-to-72-hour range, indicating that much of the ponded water in the watershed has a one to three-day journey to a major canal, as represented in the model.

Table 9-19 calculates the percentage of the total urban areas within the C2 Watershed that are inundated by 3 inches or greater for each flood duration. Because the flood duration greater than 1 hour includes all areas inundated with 3 inches or more, this row shows the same percentage as that in **Table 9-15** in PM #5. Additionally, **Figure 9-54, Figure 9-55, Figure 9-56**, and **Figure 9-57** provide a graphical view of this data, plotting the flood duration against the percentage of area inundated for the urban areas of the C2 Watershed for the 100-year, 25-year, 10-year, and 5-year storm events, respectively. These graphics also include the percent increase above the current conditions on the secondary axis, to visualize how the flood duration is changing with each SLR condition. For the 100-year design storms, the percentage of the watershed that is flooded for 48 hours shows the greatest increase above current conditions for all SLR conditions. For the 25-year storm, this increase above current conditions is greatest at around the 24-hour mark.

Table 9-17. Flood Duration per Area of Inundation (in sq. mi.) for the C2 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	23.19	32.30	36.62	39.35	22.36	31.71	36.04	38.82	20.85	30.50	35.01	37.86	19.41	28.79	33.40	36.20
1 to 2 hr.	2.30	0.48	0.33	0.18	2.40	0.50	0.31	0.24	2.55	0.52	0.34	0.23	2.46	0.61	0.35	0.28
2 to 4 hr.	1.46	0.72	0.46	0.42	1.56	0.82	0.50	0.39	1.55	0.74	0.52	0.39	1.46	0.81	0.53	0.37
4 to 8 hr.	1.47	1.02	0.80	0.49	1.42	1.02	0.72	0.56	1.31	1.11	0.68	0.58	1.24	1.06	0.77	0.63
8 to 12 hr.	0.87	0.67	0.52	0.48	0.92	0.71	0.65	0.45	0.90	0.72	0.59	0.42	0.80	0.67	0.52	0.51
12 to 24 hr.	2.77	2.41	1.67	1.19	2.89	2.35	1.86	1.33	2.81	2.21	1.81	1.44	2.78	2.25	1.87	1.37
24 to 48 hr.	3.94	2.60	1.81	1.26	3.86	2.90	1.91	1.66	3.79	2.88	2.18	1.58	3.80	2.89	2.17	1.76
48 to 72 hr.	2.55	1.55	1.03	0.76	2.62	1.48	1.22	1.13	2.69	1.82	1.19	0.91	2.79	1.91	1.37	1.08
72 to 96 hr.	1.61	0.93	0.69	0.52	1.76	1.01	0.94	0.73	2.02	1.12	0.76	0.72	1.93	1.23	0.92	0.75
96 to 144 hr.	1.85	1.21	0.85	0.65	1.98	1.58	1.25	0.98	2.43	1.40	1.20	1.15	2.59	1.56	1.20	0.83
144 to 192 hr.	1.15	0.70	0.54	0.39	1.37	1.14	0.80	0.69	1.29	1.09	0.97	0.64	1.62	1.10	0.71	0.53
192 to 240 hr.	0.86	0.55	0.38	0.42	1.22	0.72	0.63	0.65	1.03	0.85	0.59	0.59	1.09	0.71	0.46	0.46
240 to 336 hr.	1.03	0.73	0.74	0.84	1.50	1.15	1.12	1.02	1.70	1.12	1.06	1.04	1.58	0.97	0.90	0.97
336 to 420 hr.	2.82	2.53	2.39	2.17	3.79	2.84	2.20	1.70	3.74	2.96	2.44	1.95	3.93	3.31	2.91	2.48
420 hr.	4.57	4.03	3.62	3.32	2.78	2.54	2.29	2.07	3.78	3.41	3.13	2.94	4.96	4.58	4.36	4.23

Total Basin Area = 52.4 square miles

Table 9-18. Flood Duration per Area of Inundation (in sq. mi.) for Urban Areas in the C2 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	20.68	28.68	32.29	34.42	19.78	27.93	31.57	33.73	18.45	26.93	30.74	33.00	17.21	25.46	29.42	31.76
1 to 2 hr.	2.06	0.43	0.28	0.13	2.10	0.43	0.26	0.20	2.26	0.42	0.28	0.20	2.19	0.51	0.29	0.24
2 to 4 hr.	1.28	0.61	0.39	0.34	1.36	0.71	0.44	0.33	1.36	0.67	0.45	0.32	1.28	0.71	0.47	0.32
4 to 8 hr.	1.30	0.90	0.65	0.38	1.27	0.89	0.59	0.45	1.17	0.97	0.58	0.50	1.08	0.93	0.67	0.51
8 to 12 hr.	0.75	0.57	0.43	0.36	0.81	0.63	0.56	0.35	0.80	0.62	0.51	0.37	0.72	0.61	0.44	0.44
12 to 24 hr.	2.48	2.00	1.36	0.88	2.58	2.03	1.53	1.07	2.48	1.92	1.56	1.16	2.50	1.98	1.61	1.16
24 to 48 hr.	3.34	2.09	1.34	0.85	3.35	2.36	1.50	1.08	3.33	2.44	1.74	1.16	3.32	2.47	1.87	1.42
48 to 72 hr.	2.07	1.17	0.62	0.36	2.16	1.19	0.76	0.46	2.28	1.46	0.90	0.61	2.37	1.62	1.10	0.80
72 to 96 hr.	1.32	0.61	0.33	0.19	1.47	0.71	0.40	0.25	1.68	0.90	0.46	0.32	1.65	1.02	0.65	0.47
96 to 144 hr.	1.37	0.64	0.28	0.13	1.55	0.72	0.35	0.14	2.00	0.88	0.52	0.28	2.22	1.20	0.73	0.41
144 to 192 hr.	0.76	0.23	0.08	0.04	0.81	0.33	0.10	0.06	0.92	0.48	0.21	0.08	1.28	0.65	0.32	0.17
192 to 240 hr.	0.39	0.09	0.05	0.05	0.50	0.11	0.07	0.07	0.60	0.22	0.07	0.05	0.78	0.35	0.16	0.13
240 to 336 hr.	0.25	0.10	0.07	0.06	0.36	0.13	0.09	0.06	0.60	0.15	0.11	0.10	0.88	0.35	0.20	0.13
336 to 420 hr.	0.13	0.09	0.09	0.08	0.12	0.05	0.04	0.04	0.20	0.11	0.09	0.05	0.47	0.19	0.14	0.11
420 hr.	0.36	0.32	0.28	0.26	0.33	0.30	0.28	0.26	0.42	0.36	0.33	0.33	0.57	0.51	0.48	0.47

Total Basin Urban Area = 38.5 square miles

Table 9-19. Percentage of Total Area Inundated for Urban Areas in the C2 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0 hr.	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 1 hr.	46.3	25.6	16.2	10.7	48.7	27.5	18.1	12.5	52.1	30.1	20.2	14.4	55.4	33.9	23.7	17.6
>= 2 hr.	41.0	24.5	15.5	10.3	43.2	26.4	17.4	12.0	46.3	29.0	19.5	13.8	49.7	32.6	22.9	17.0
>= 4 hr.	37.7	22.9	14.5	9.5	39.7	24.6	16.3	11.1	42.8	27.3	18.3	13.0	46.3	30.8	21.7	16.1
>= 8 hr.	34.3	20.5	12.8	8.5	36.4	22.3	14.7	9.9	39.7	24.8	16.8	11.7	43.5	28.4	20.0	14.8
>= 12 hr.	32.4	19.0	11.7	7.5	34.3	20.6	13.3	9.0	37.6	23.2	15.5	10.7	41.7	26.8	18.8	13.7
>= 24 hr.	25.9	13.9	8.2	5.3	27.6	15.4	9.3	6.3	31.2	18.2	11.5	7.7	35.2	21.7	14.6	10.7
>= 48 hr.	17.2	8.4	4.7	3.1	18.9	9.2	5.4	3.5	22.6	11.8	7.0	4.7	26.6	15.3	9.8	7.0
>= 72 hr.	11.9	5.4	3.1	2.1	13.3	6.1	3.5	2.3	16.6	8.0	4.6	3.1	20.4	11.1	6.9	4.9
>= 96 hr.	8.5	3.8	2.2	1.6	9.5	4.3	2.4	1.6	12.3	5.7	3.4	2.3	16.1	8.4	5.2	3.7
>= 144 hr.	4.9	2.1	1.5	1.3	5.5	2.4	1.5	1.3	7.1	3.4	2.1	1.6	10.4	5.3	3.4	2.6
>= 192 hr.	2.9	1.5	1.3	1.2	3.4	1.5	1.2	1.1	4.7	2.2	1.5	1.4	7.0	3.6	2.5	2.2
>= 240 hr.	1.9	1.3	1.2	1.1	2.1	1.3	1.1	0.9	3.1	1.6	1.4	1.2	5.0	2.7	2.1	1.8
>= 336 hr.	1.3	1.1	1.0	0.9	1.2	0.9	0.8	0.8	1.6	1.2	1.1	1.0	2.7	1.8	1.6	1.5
>= 420 hr.	0.9	0.8	0.7	0.7	0.8	0.8	0.7	0.7	1.1	0.9	0.8	0.8	1.5	1.3	1.3	1.2

Figure 9-54. Percentage and Duration of Inundation for the C2 Watershed for the 100-year Storm Event

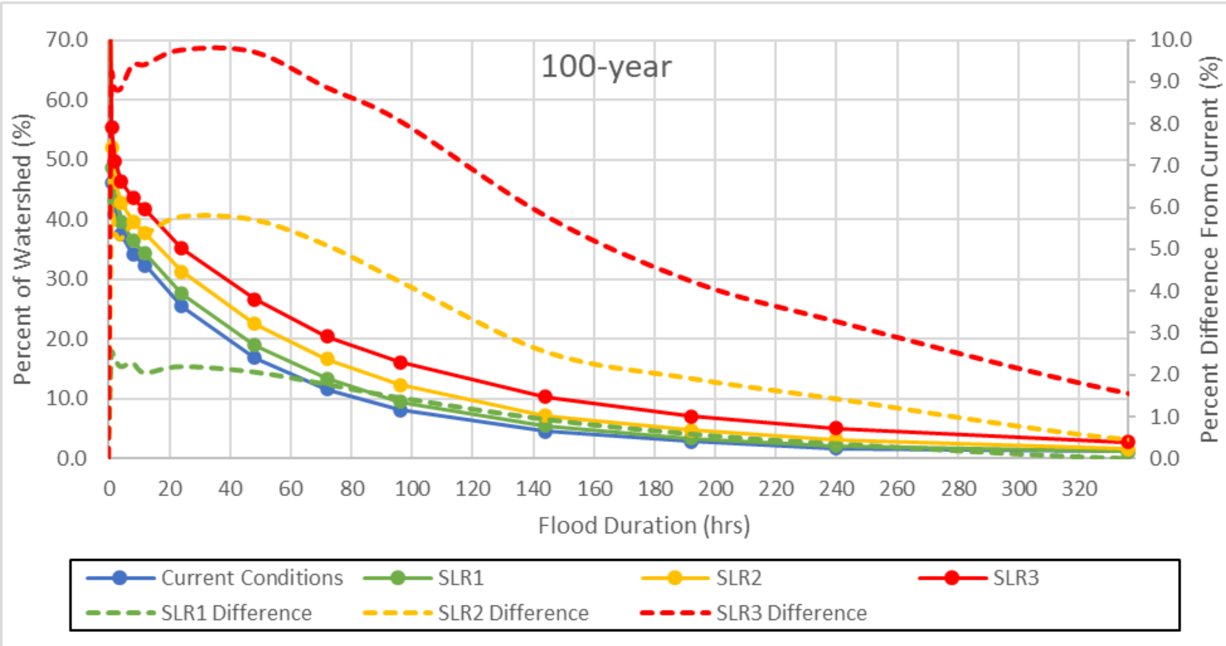


Figure 9-55. Percentage and Duration of Inundation for the C2 Watershed for the 25-year Storm Event

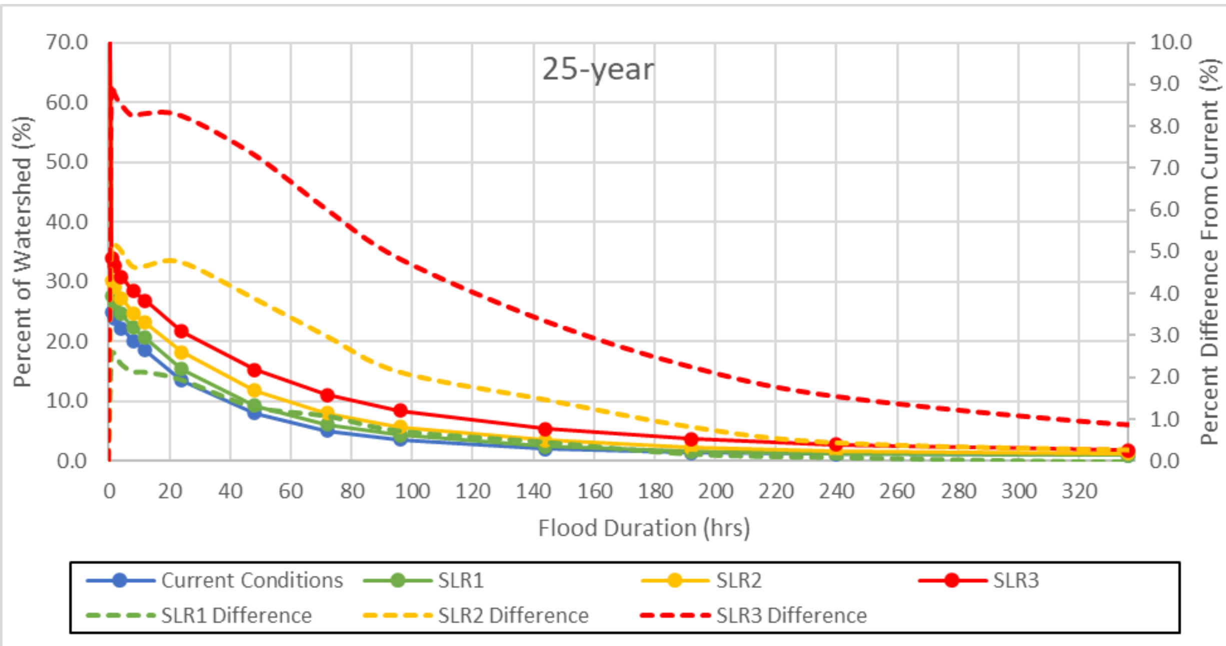


Figure 9-56. Percentage and Duration of Inundation for the C2 Watershed for the 10-year Storm Event

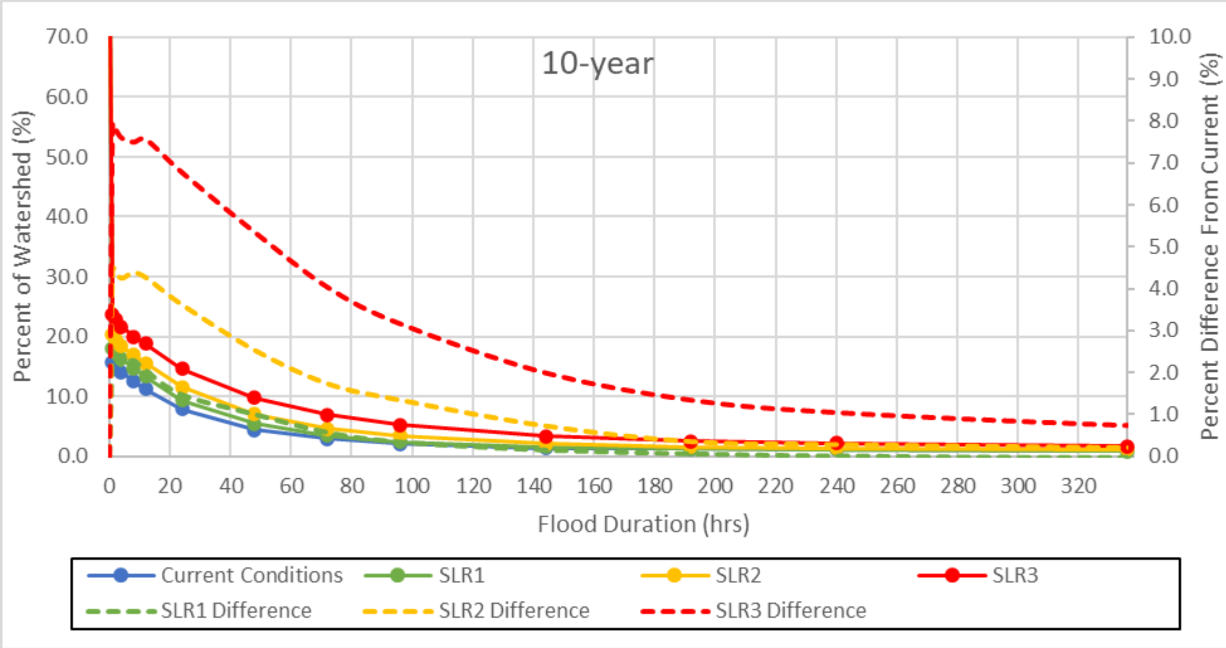


Figure 9-57. Percentage and Duration of Inundation for the C2 Watershed for the 5-year Storm Event

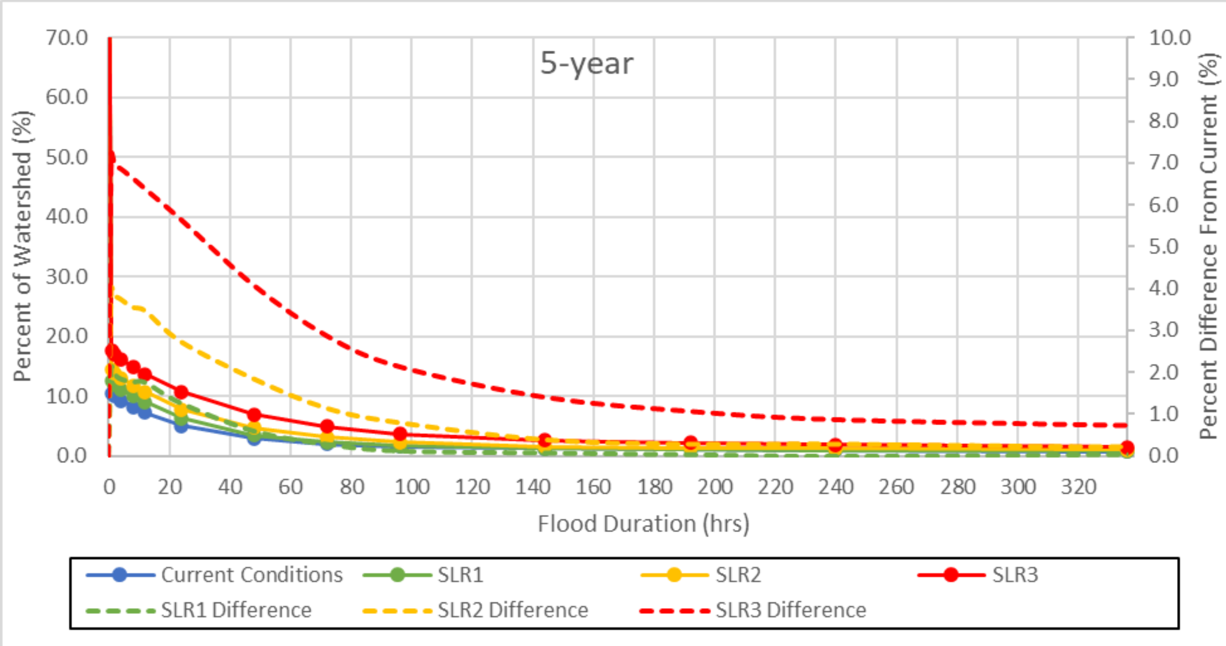


Figure 9-58 and **Figure 9-59** provide flood duration maps for the C2 Watershed for overland flooding depths exceeding 0.25 feet for the current conditions 5-year and 100-year 3-day design storms, respectively. Water areas, such as existing lakes and ponds, are masked in black**. Figure 9-60** and **Figure 9-61** provide the flood duration for only the urban areas within the C2 Watershed (non-urban areas are masked out), to provide a concise picture of how urban areas are impacted within the watershed.

Figure 9-62, **Figure 9-63**, and **Figure 9-64** provide the difference in flood duration between the current conditions and SLR +1 foot, SLR, +2 feet, and SLR +3 feet, respectively, for the 100-year storm event. When compared with the difference maps for flood depth, this can show that even for areas where flood depths are not increasing with future SLR conditions, the duration of flooding may increase due to the reduced ability of the area to drain to the receiving canals that are experiencing higher stages. For example, the large area in the center of the watershed near Bird Road and SW 97th Avenue that continues to increase in flooding duration with each SLR condition does not show as having deeper flooding for these future conditions.

Flood duration maps over the entire C2 Watershed, and only urban areas, for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current conditions and future SLR scenarios are provided in **Appendix D**. Also provided in **Appendix D** are the differences in flood duration maps for the 25-year, 10-year, and 5-year design storms.

Figure 9-58. Flood Duration Map for the Current Conditions 25-year 3-day Design Storm for the C2 Watershed

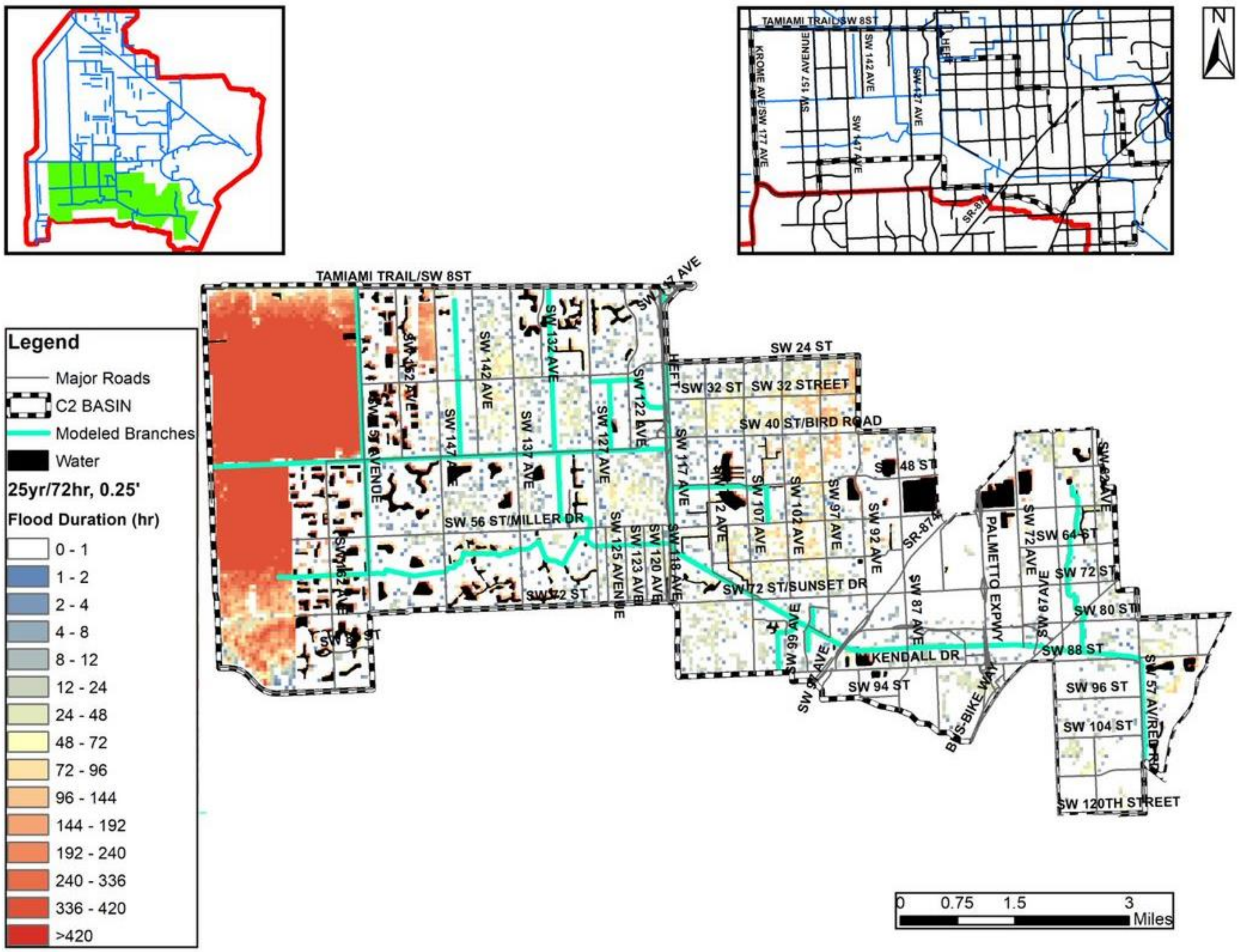


Figure 9-59. Flood Duration Map for the Current Conditions 100-year 3-day Design Storm for the C2 Watershed

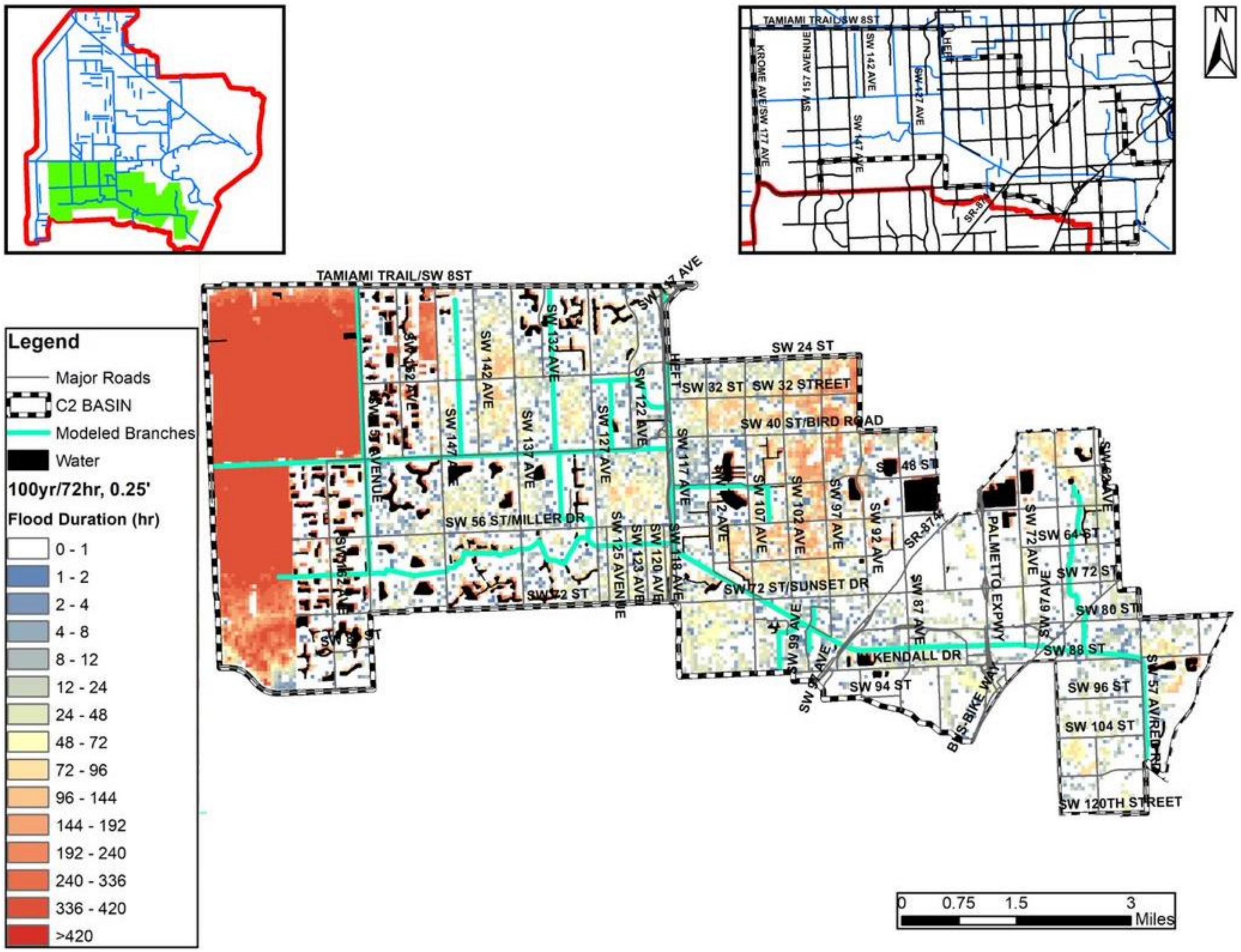


Figure 9-60. Flood Duration Map for the Current Conditions 25-year 3-day Design Storm for Urban Areas in the C2 Watershed

Figure 9-61. Flood Duration Map for the Current Conditions 100-year 3-day Design Storm for Urban Areas in the C2 Watershed

Figure 9-62. Urban Flooding Duration Difference of SLR +1ft and Current Cond. for the 100-year Storm in the C2 Watershed

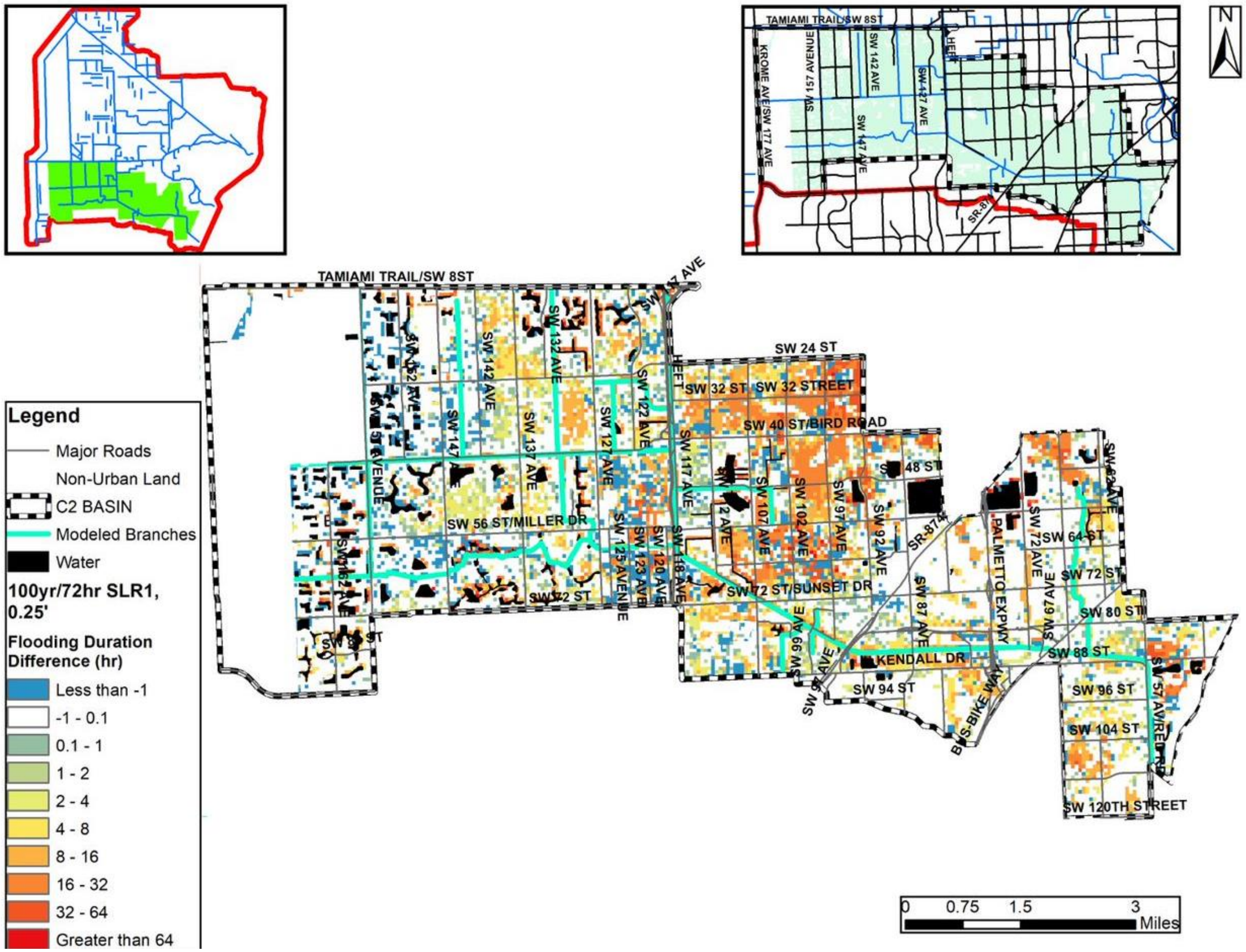


Figure 9-63. Urban Flooding Duration Difference of SLR +2ft and Current Cond. for the 100-year Storm in the C2 Watershed

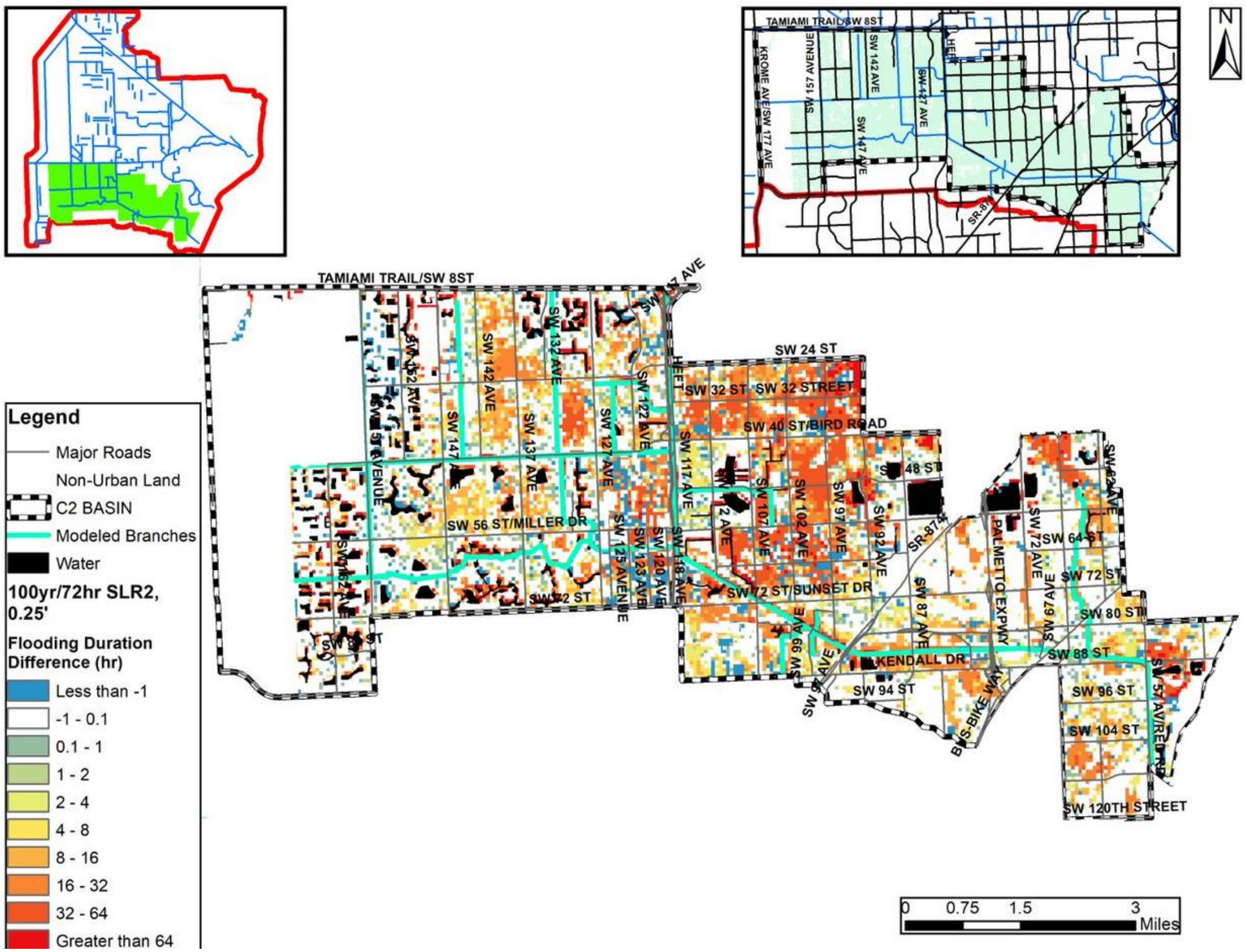
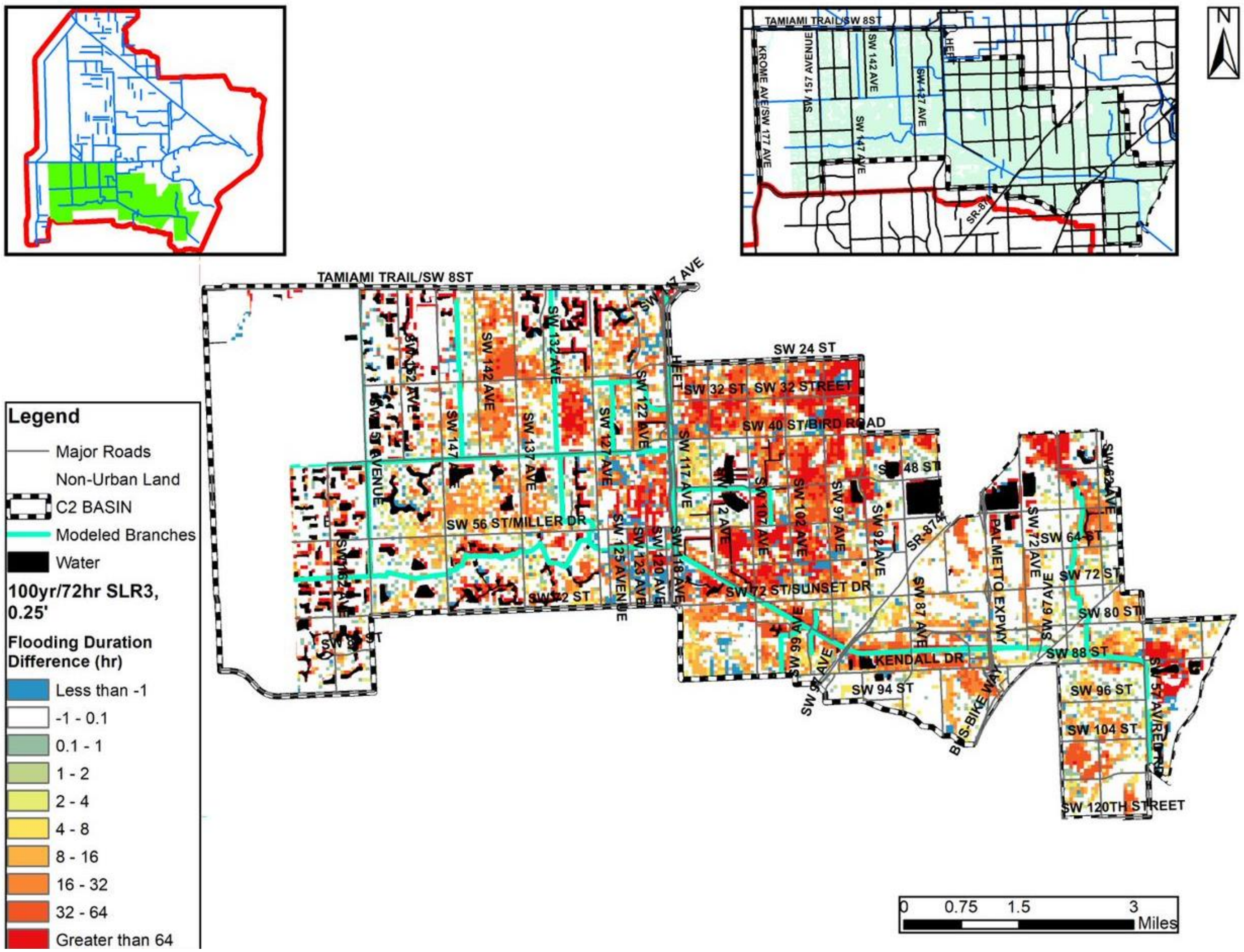
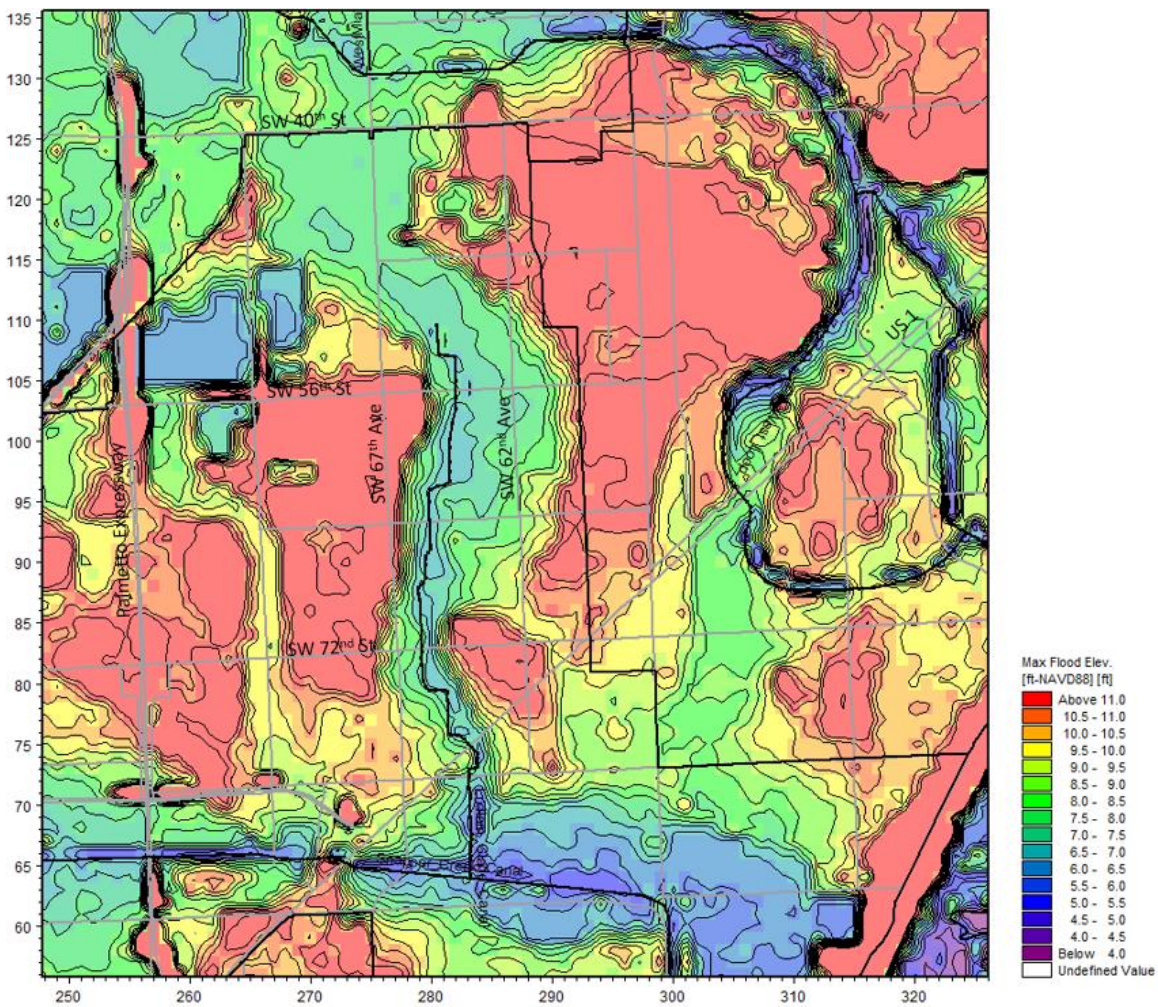


Figure 9-64. Urban Flooding Duration Difference of SLR +3ft and Current Cond. for the 100-year Storm in the C2 Watershed



For the C2 Watershed some areas tend to experience flooding for longer periods. Some areas of concern are near the Ludlam Glades Canal and in the area of Olympia Heights near the Westwood Lakes Canal. Ludlam Glades Canal is nested in one of the finger glades that cut through the Miami Rock Ridge formation, as such the area around the canal has a relatively steep gradient on either side of the canal. **Figure 9-65** shows the overland flood elevation map for this region. Areas of flooding concern tend to coincide with low-lying areas indicated on the map. Concerns of flooding in this region are primarily due to elevation and drainage capacity. Flooding in the finger glade area that is upstream of the canal may be due to lack of access to the drainage canal. This area experiences both deeper depths (PM #5) and increased duration of flooding for the future SLR conditions. With future conditions, direct flooding from the canals is a concern (PM #1) as roads are overtopped for even a 10-year storm for SLR3 conditions.

Figure 9-65. Overland Flood Elevation for the 100-year Design Storm in the area near Ludlam Glades Canal



In the area near Westwood Lakes Canal flood depths do not recede as quickly as water levels in the canals and can get up to 1.25 feet of depth in some areas. This area is higher than the stages in the canal, so direct flooding from the canal is likely not an issue for most of the region, although some areas adjacent to the canal do experience flooding. The overland flood elevation map shown in **Figure 9-66** shows that the region is from 7.5 to 9.0 ft-NAVD, whereas the peak of the flooding in Westwood Lakes Canal reaches 6.8 ft-NAVD for the 100-year design storm. **Figure 9-67** shows the flood elevation at points A and B indicated in the map as compared with the stages in Westwood Lakes Canal.

Furthermore, as we see future SLR overland flooding durations increase (PM #6), but not overland flooding depths (PM #5) for this area, this confirms that this region is perhaps most impacted by tertiary drainage issues.

Figure 9-66. Overland Flood Elevation for the 100-year Design Storm in the Olympia Heights area near Westwood Lakes Canal

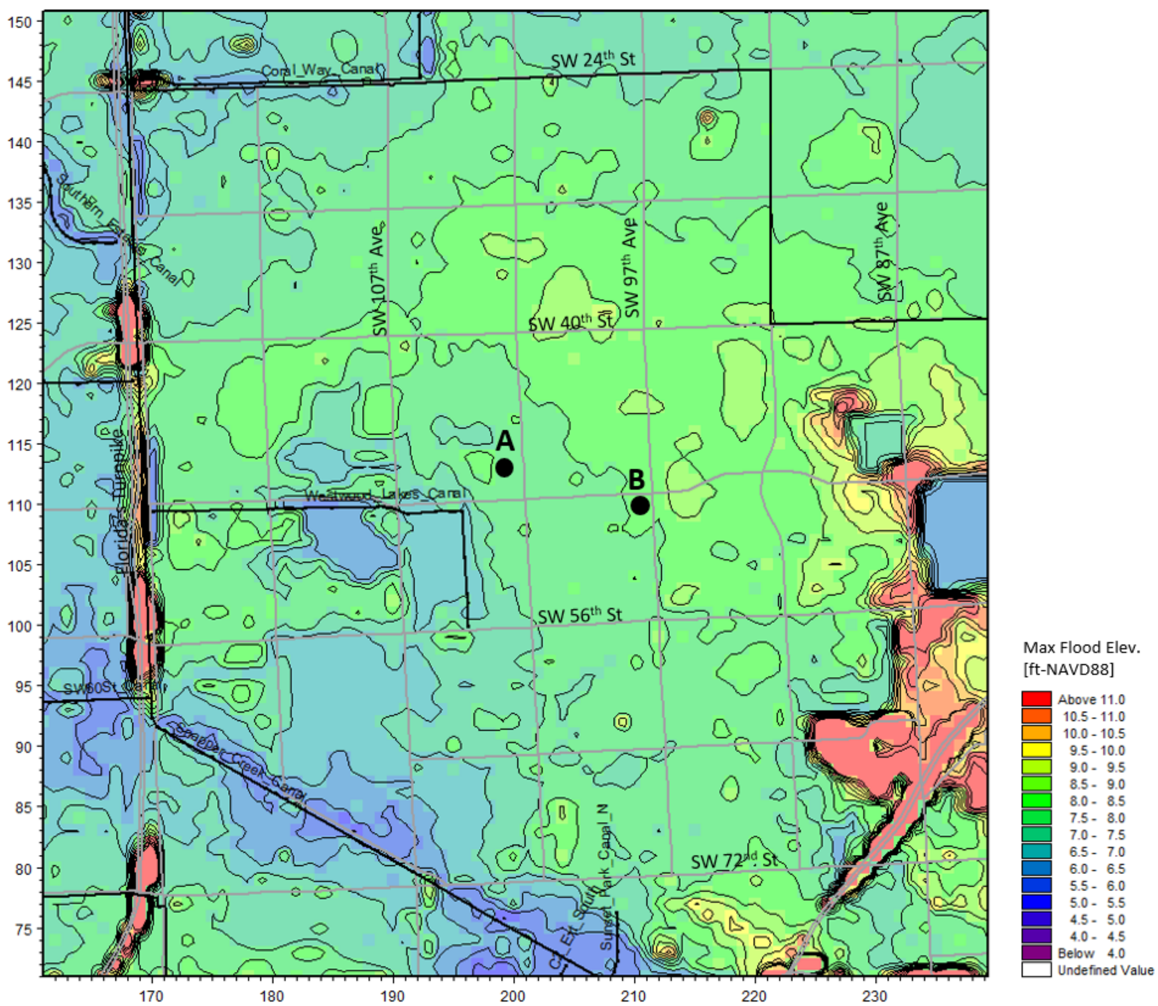
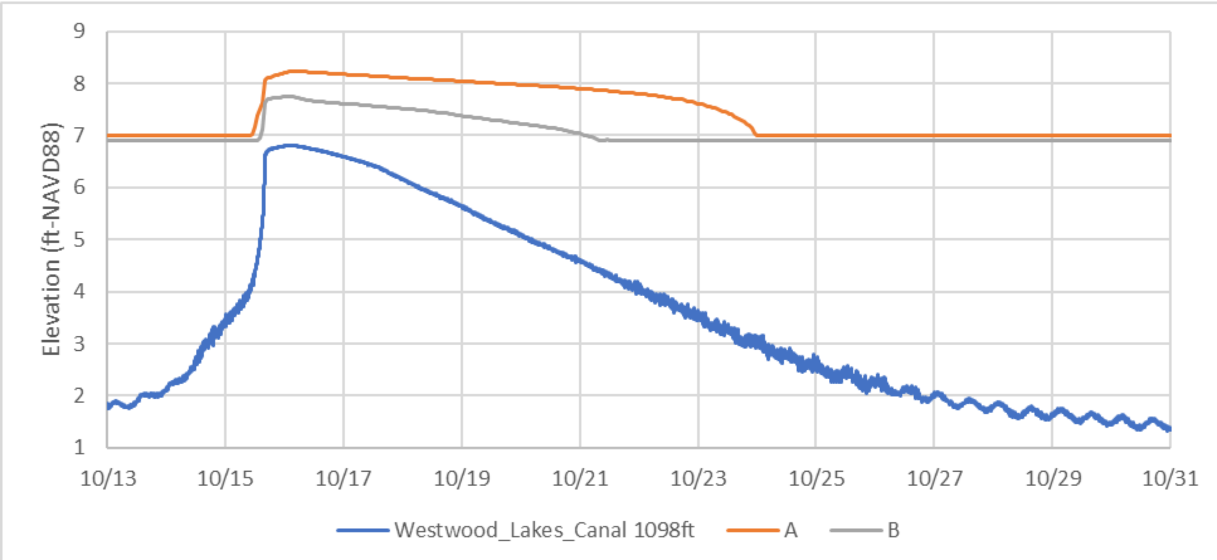


Figure 9-67. Westwood Lakes Canal Stages Compared with Overland Flood Elevation for 100-year Design Storm



This area, while not particularly low in elevation, is a very wide and flat region from the Coral Way Canal to the north and south to the Snapper Creek Canal. The gradient from the maximum elevation in the middle of the region to the Snapper Creek Canal represents a 2 feet elevation difference over 9,000 feet. This low gradient may reduce the speed of drainage. In addition, the Westwood Lakes Canal shows overtopping during the storm event, which may indicate a reduced capacity to carry additional flows from drainage. Additionally, further review of the peak stages along the canal and into the Snapper Creek Canal indicate a drop in stages of 1 foot over the 1.7-mile canal for the 100-year design storm, indicating that there are no flow constraints along this canal. As it is represented in the model, Westwood Lakes Canal may have limited carrying capacity as a drainage canal during storm events.

9.1.7. SUMMARY OF LOS AND RATING FOR THE C2 WATERSHED

The maximum design storm frequency that the C2 Watershed passes without incurring negative impacts is summarized for each performance metric in **Table 9-20**.

Under critical consideration for this watershed is PM #1, PM #5 and PM #6, as these relate directly to flooding in canals and overland flooding depth and duration. For the current conditions, it was concluded that the C2 watershed can handle up to the 10-year storm event, which was primarily due to road exceedance at culvert locations and canal embankment overtopping along the canals, as explained in PM #1. For the SLR1 condition, similar results to the current conditions 10-year storm were found for not only culverts and canal embankments, but also overland flooding depth and duration. All greater SLR conditions and storm events showed significant increases in canal flooding, as discussed in PM #1, and direct flooding from the canals to the overland, as discussed in PM #5 and PM #6. Therefore, it is determined that the overall level of service provided by this watershed for future SLR conditions is the SLR1 5-year storm. No other SLR conditions have passing storms.

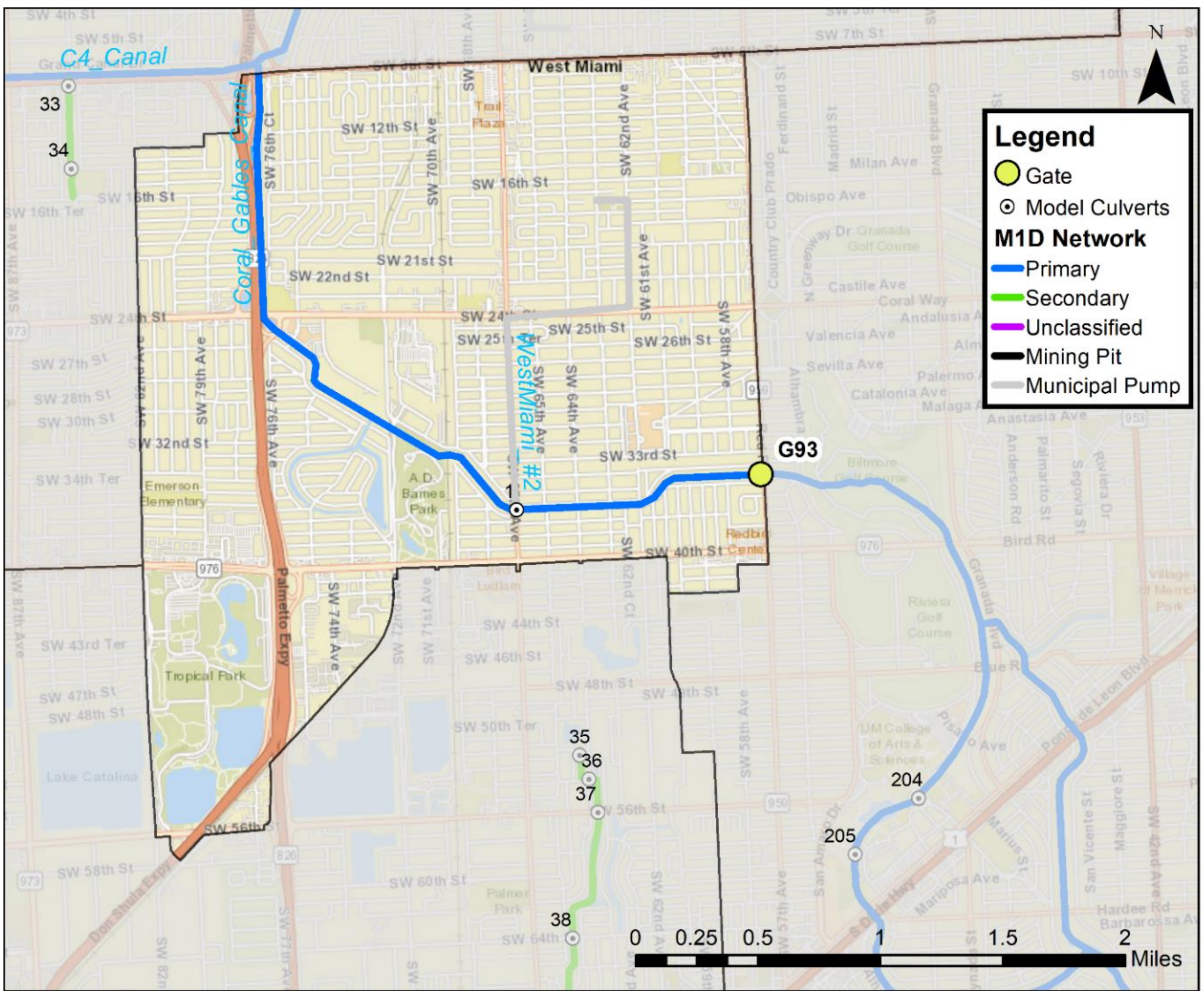
Table 9-20. Performance Metric Summary for the C2 Watershed

Metric	Notes	Current Conds.	SLR +1 ft	SLR +2 ft	SLR +3 ft
PM #1	▪ SLR conditions increase the frequency of bridge low-chord exceedance for the SW 40th Ave and SW 5th Ave bridges. Additionally, the SW 77th Ave bridge low chord is exceeded with the SLR2 condition and the Railroad low chord west of SR874 is exceeded with the SLR3 condition for the 100-year storm. ▪ The number of culvert locations where the crown of road is exceeded increases significantly with each SLR condition. For the SLR1 5-year storm the same two (2) culverts are exceeded that were exceeded for the current conditions 10-year storm. ▪ The length of Snapper Creek that is overtopped is over a mile for all SLR conditions, with the exception of the SLR1 5-year	10-year	5-year	<5-year	<5-year
PM #2	▪ 25-year Allowable Discharge exceeds the ERP value for the current and SLR1 conditions. ▪ With the future canals, a new connection to the C4 Canal increases discharges to the C4 watershed, increasing drainage capacity for SLR1 conditions. SLR2 conditions are similar to current conditions, and SLR3 reduces the discharge capacity at the S22 structure.	--	--	--	--
PM #3	▪ Maximum discharge at S22 falls below design value for the 5-year event with SLR +3. ▪ The HW and TW exceed the water level that will bypass S22 for the 100-year design storm for SLR2 and SLR3 future scenarios and for the 25-year design storm for the SLR3 future scenario.	--	--	--	--
PM #4	▪ Peak 12-hour moving discharge ranges from 1,725 CFS to 3,038 CFS (compared to the design discharge of 1,905 CFS) and decreases with increasing SLR for each design storm return period.	--	--	--	--
PM #5	▪ 17.6% of the watershed is flooded with 0.75 ft of depth or greater for the 100-year, 9.7% for the 25-year storm. Inundated areas at these depths are likely not a result of direct canal flooding, but due to slow drainage from flat areas far away from the canal system. ▪ Percent increases for the current conditions 25-year are comparable to the SLR1 numbers for the 25-year, while the percentages are similar to the 10-year storms for SLR2 and SLR3.	25-year	25-year	10-year	10-year
PM #6	▪ Canal: Stages at the T5W station recede after 68 hours for the 25-year storm for current conditions, and for more than 3 days for the 100-year storm, which is longer than the duration of the storm event itself. For SLR1, the canal recedes in less than 72 hours for the 10-year storm. For SLR2 and SLR3, the canal takes longer than 4 days to recede for all storm events. ▪ Watershed: Percent increases for the current conditions 25-year are comparable to the SLR1 numbers for the 25-year, while the percentages are similar to the 10-year storms for SLR2 and SLR3.	25-year	25-year	10-year	10-year
Overall Level of Service		**10-year**	**5-year**	**<5-year**	**<5-year**

9.2. C3W WATERSHED FLOOD PROTECTION LEVEL OF SERVICE

The C3W Watershed consists of areas south of SW 8th Street, east of SW 82nd Avenue, west of Red Road, and north of Bird Road (with the exception of Tropical Park, which is included in the watershed). The primary discharge canal is the Coral Gables Canal from the intersection with the C4 Canal to the G93 water control structure. G93 is a tidal water control structure that controls water levels in the canal, located at the intersection of Coral Gables Canal and Red Road. G93 controls flows out of the C3W Watershed to tide and is operated during the wet season to maintain the low range water elevations of (-)0.54 to 0.76 ft-NAVD (1.0 to 2.3 ft-NGVD). One municipal pump is also represented in this watershed, West Miami #2, which drains areas north of SW 24th Street in West Miami. **Figure 9-68** provides a map of the watershed and the branches included in the model.

Figure 9-68. Map of the C3W Watershed



9.2.1. C3W – PM #1 MAXIMUM STAGE IN PRIMARY CANALS

The maximum stage in the primary canals was extracted from the results of each design storm simulation for the C3W Watershed. Bank elevations were also extracted from the model, as setup in the MIKE 1D model, to identify areas where peak stages exceed bank elevations, which could be an indication of overbank flooding next to the canal. **Table 9-21** provides the low chord information for each bridge in the Coral Gables Canal, as well as the peak stage at the nearest h-point (where canal stages are calculated in the model). This can be used to establish any issues with bridge low chords; however, low chords in this watershed were not exceeded with any design event, under any SLR condition.

In addition to the bridges above, Coral Gables Canal also includes one culvert located at SW 67th Avenue. **Table 9-22** provides the culvert information with the estimated crown of road, provided from the SWMM model data, as well as the peak stage at the nearest H-point. The estimated crown of road was higher than the peak stages for each design storm, under all SLR conditions. However, the culvert, which consists of two (2) 10-foot diameter corrugated metal pipes, experiences full flow for the 100-year design storms for SLR 2 and SLR3.

The maximum stage profiles for the C3W Watershed are provided in **Appendix A**, with bank elevations and major intersections located along the canals. The figures provided in the Appendix shows the entire length of the Coral Gables Canal from the C4 Canal to tide. **Figure 9-69** shows the maximum stage profile for the C3W Watershed for the Current Conditions. In addition, the maximum stage profile for the C3W Watershed, for the 100-year storm for all conditions, is provided in **Figure 9-70**, with bank elevations and major intersections located along the canal. This figure shows how the water begins to stack up just upstream of the structure as tidal elevations increase. For the SLR2 and SLR3 simulations, water levels downstream of SW 72nd Avenue are showing impacts from the tidal elevation directly. The culverts at Ludlam Avenue and the canal bridges at SW 69th Avenue and SW 72nd Avenue limit flow and reduce the amplitude of the surge at higher stages, damping the tidal effects upstream.

Table 9-21. Bridge Low Chord and Peak Stage for the Coral Gables Canal

Location Description	Low Chord	Bridge Top	Current Conditions				SLR1				SLR2				SLR3			
			Elevation (ft-NAVD)															
			100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
SW 77th Ave (Coral Way)	7.26	9.75	5.47	4.76	4.24	3.90	5.81	5.27	4.84	4.49	6.26	5.67	5.25	4.98	6.78	6.15	5.80	5.55
SR 826 ramp	11.86	14.85	5.49	4.71	4.18	3.84	5.86	5.23	4.79	4.44	6.26	5.67	5.21	4.94	6.74	6.14	5.81	5.55
75th Avenue	7.18	8.86	5.49	4.71	4.18	3.84	5.86	5.23	4.79	4.44	6.26	5.67	5.21	4.94	6.74	6.14	5.81	5.55
RR Trestle just west of SW 72nd Avenue	8.32	11.91	5.50	4.68	4.14	3.80	5.88	5.21	4.76	4.40	6.27	5.67	5.21	4.92	6.76	6.15	5.82	5.55
SW 72nd Avenue	9.03	11.75	5.52	4.62	4.04	3.68	5.93	5.21	4.70	4.31	6.34	5.70	5.23	4.88	6.83	6.22	5.84	5.56
69th Ave	13.86	15.54	5.53	4.60	3.98	3.61	5.95	5.22	4.66	4.26	6.36	5.71	5.23	4.85	6.86	6.24	5.84	5.56
*Highlighted cells indicate the stages exceed the bridge low chord																		

Table 9-22. Estimated Culvert Crown of Road and Peak Stage for the C3W Watershed

Culvert No.	Intersection	Top of Culvert	Estimated Crown of	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Coral_Gables_Canal																			
1	**SW 67th (Ludlam) Avenue**	6.59	8.2	5.56	4.50	3.78	3.29	6.04	5.19	4.56	4.06	6.60	5.76	5.22	4.81	7.28	6.45	5.90	5.55
*Highlighted cells indicate the stages exceed the estimated crown of road.																			

Canal top of bank was established in the model from cross-section information from previous modeling efforts, including the HEC-RAS models (SFWMD, 2015), and converted to ft-NAVD. Full discussion of the development of these cross-sections is provided in **Section 3.3.1**. Top of bank was extracted from the model, and the miles of the evaluated canal segment that is overtopped by the design storm was estimated upstream of the tidal control structure G93, as summarized in **Table 9-23**.

Table 9-23. Estimated Percentage and Miles of Bank Overtopped per Design Storm

Watershed	Simulation	100-Year	25-Year	10-Year	5-Year
C3W (%)	Current Conditions	33%	30%	0%	0%
	SLR1	33%	33%	30%	0%
	SLR2	33%	33%	33%	33%
	SLR3	33%	33%	33%	33%
C3W (miles)	Current Conditions	1.1	1.0	0.0	0.0
	SLR1	1.1	1.1	1.0	0.0
	SLR2	1.1	1.1	1.1	1.1
	SLR3	1.1	1.1	1.1	1.1

As shown in the maximum stage profile in **Figure 9-69** and **Figure 9-70**, the top of bank was overtopped during the 100-year design storm just upstream of the structure where there is a low-lying park (Coral Gables Wayside Park). This area is also overtopped for other storm events during the future SLR conditions. For example, the minimum design storm that overtops for the SLR1 simulation is the 25-year, the minimum storm for the SLR2 simulation is the 10-year, and the minimum storm for the SLR3 is the 5-year.

The golf course upstream of Bird Road has some low embankments that are overtopped during the 25-year and 100-year design storms. In addition, some residential areas upstream of US1, near Blue Road, overtop for the 25-year and 100-year design storms. Both of these areas are also overtopped for the 10-year storm event for the SLR1, SLR2, and SLR3 simulations. The steep gradient drop observed near G93 is a result of the head differential at the structure and is further discussion in PM #3.

Figure 9-69. Maximum Stage Profile for the C3W Watershed for the 100-year Design Storm

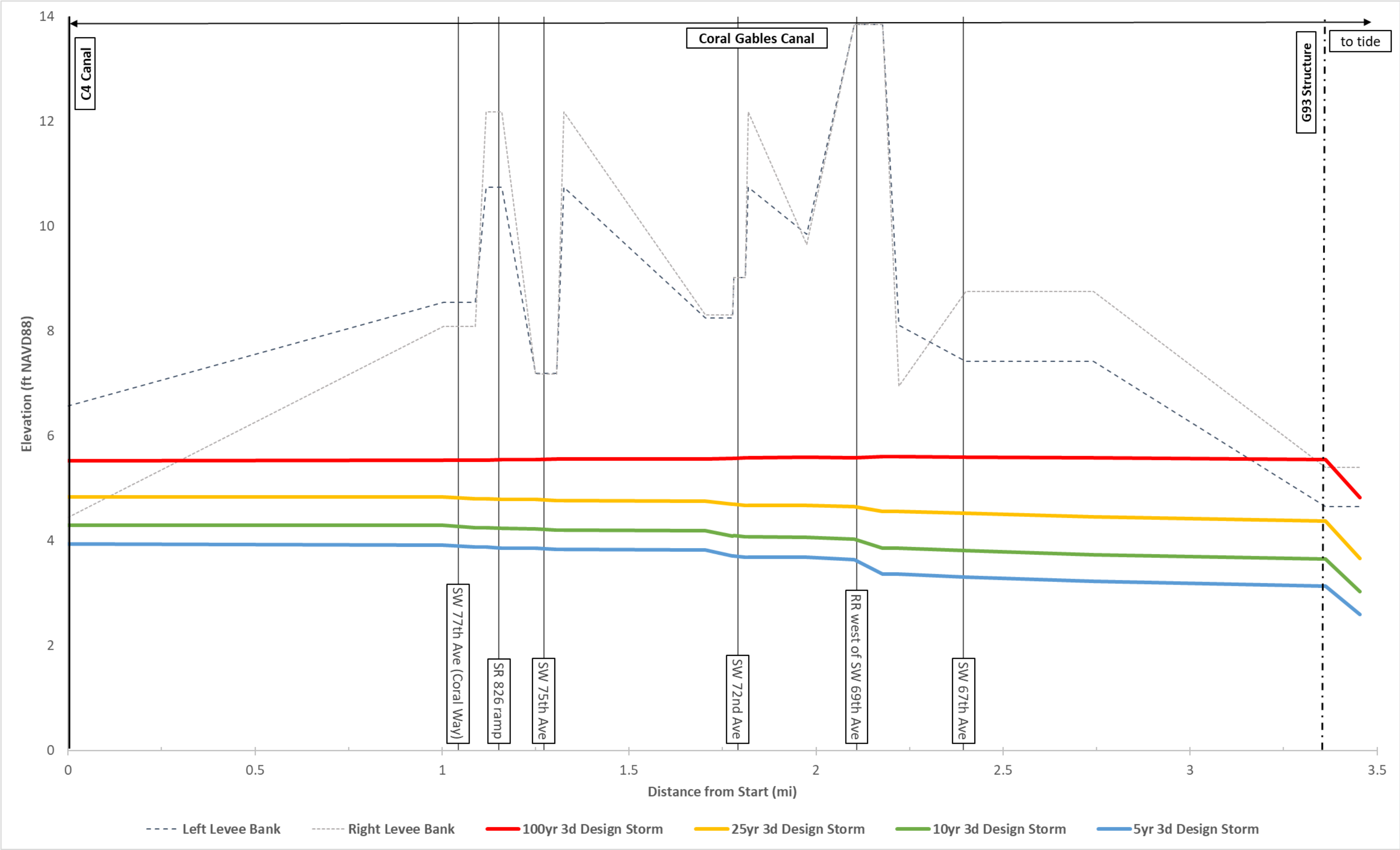
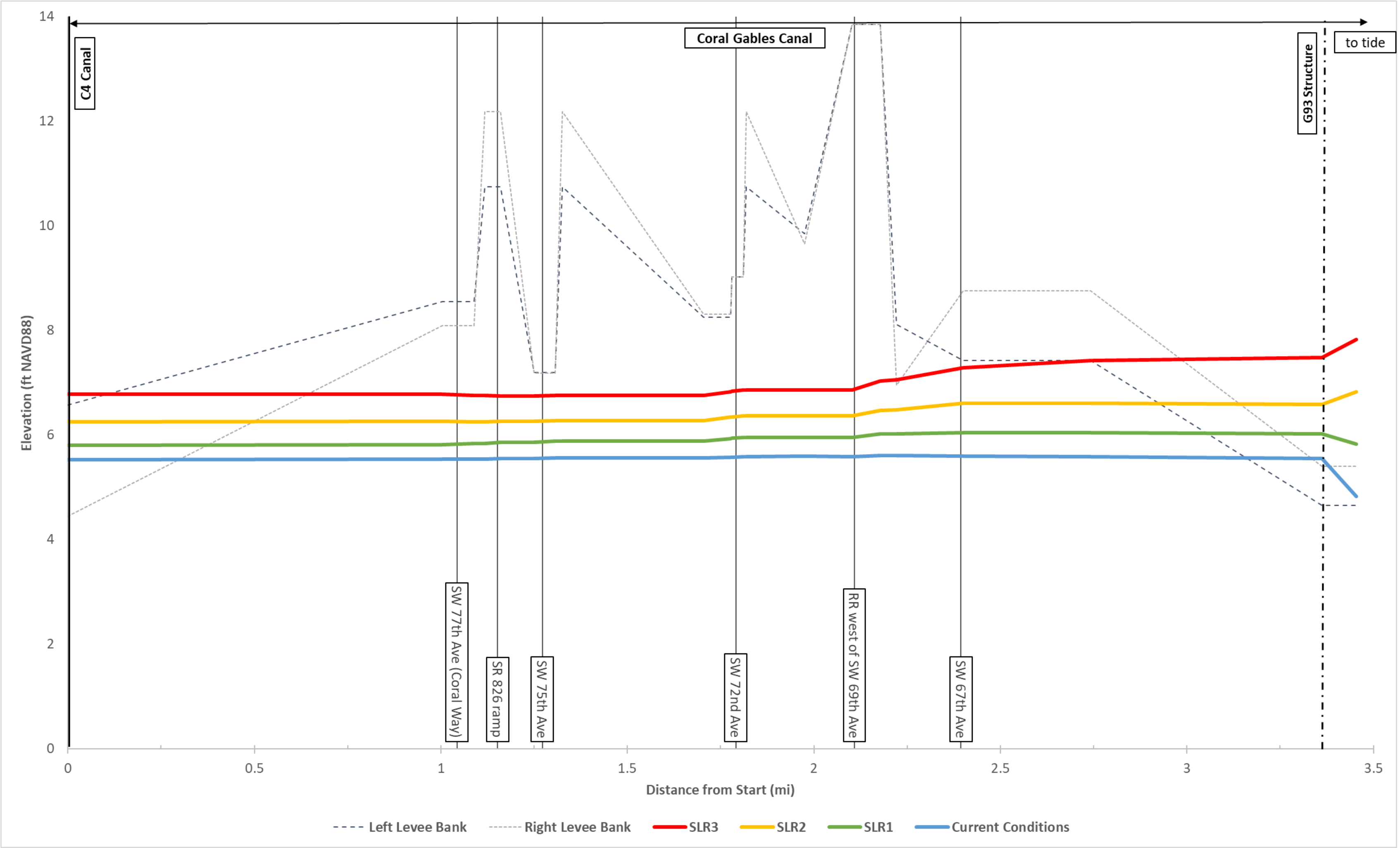


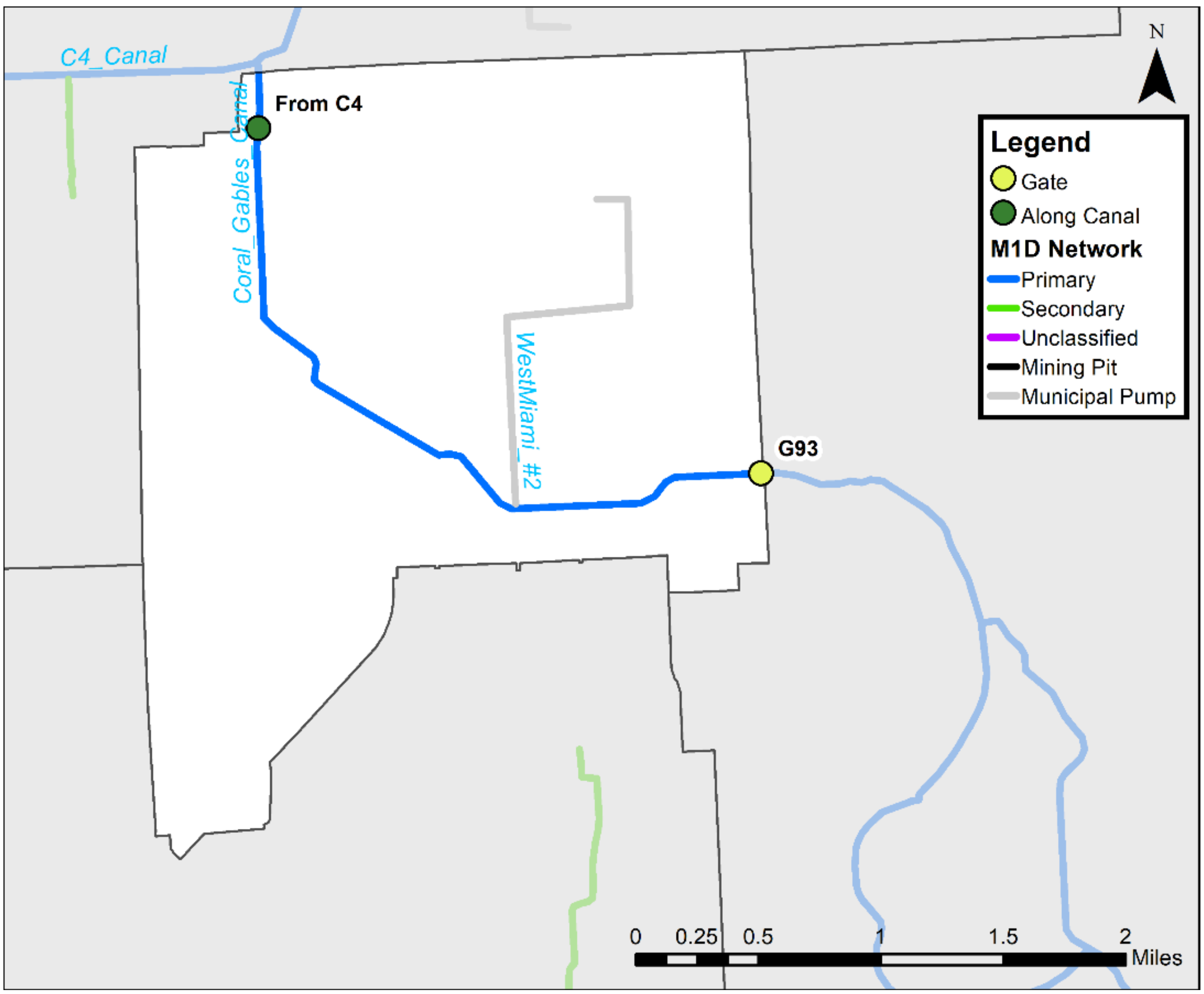
Figure 9-70. Maximum Stage Profile for the C3W Watershed for the 100-year Design Storm



9.2.2. C3W – PM #2 MAXIMUM DISCHARGE CAPACITY

The maximum discharge capacity for the C3W Watershed is the sum of the discharges out of the watershed minus the incoming flows weighted by the total area of the watershed. For the C3W Watershed this means flows from the C4 Canal are subtracted from the outflows at the G93 structure, these locations are shown in **Figure 9-71**.

Figure 9-71. Discharge Inflow Locations in the C3W Watershed



The timeseries at each inflow and outflow location was extracted, and inflows were subtracted from the outflows and divided by the basin area in square miles (sq. mi.) at each timestep as shown in the equation below.

$$\frac{Outflows[cfs] - Inflows[cfs]}{Basin\ Area\ [sq.mi.]} = Discharge\ Capacity[CSM]$$

The maximum discharge capacity for the C5 Watershed for each design event is shown in **Table 9-24** for each simulation (i.e., Current Conditions, SLR +1 foot, SLR +2 feet, and SLR +3 feet). The table also provides discharge in cubic feet per second (CFS) at inflow and outflow locations for the watershed. Discharges are not the peak discharge at each location, but rather the discharge at the peak watershed discharge capacity. A negative value at an inflow location means that the flow is leaving the watershed at the peak of the watershed discharge. A negative value at an outflow location means that the flow is entering the watershed at the peak of the watershed discharge.

The ERP Applicant's Handbook Volume II states that the conveyance system is design to provide flood protection from the 25-year storm event and that the allowable discharge upstream of G-97 (which was later replaced by G-93) for the C3 Basin is 54 CSM. This allowable discharge is exceeded for all conditions and storm events modeled. In all simulations, discharges that reverse back into the C4 Watershed at the intersection with the C4 Canal and the Coral Gables Canal (shown in the table as negative flows) add to the overall discharge capacity of the basin. In the cases of SLR, discharges that reverse into the C4 Watershed tend to increase, alleviating backed-up conditions at the G93 structure, which may not be able to discharge due to higher tailwater conditions. This diminished discharge capacity is directly related to the watershed's inability to discharge to tide at the G93 structure due to higher tidal stages, as is further explored in PM #3 and PM #4.

Figure 9-72 shows the instantaneous discharge capacity in CSM for the C3W Watershed, for each SLR condition simulation for the 100- year, 25- year, 10- year, and 5-year design storm events. To remove the tidal influence, **Figure 9-73** shows the 12-hour moving average discharge capacity in CSM. For all SLR conditions and design storms, the shape of the hydrographs remains the same, indicating that the structure operations may be operating similarly for all conditions.

Table 9-24. Peak Discharge (CFS/sq. mi.) from the Contributing Drainage Area of the C3W Watershed

	Current Conditions				SLR1				SLR2				SLR3			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Inflow Locations																
Coral Gables Canal from C4 Canal (CFS)	-409.0	-206.0	-127.6	-80.4	-457.6	-219.6	-200.5	-159.7	-727.3	-371.6	-159.5	-118.4	-769.7	-627.2	-434.3	-365.9
Outflow Locations																
G93 Total Flow (CFS)	616.3	602.6	552.7	515.1	483.1	529.0	417.4	393.7	341.3	317.6	383.9	343.5	-167.1	225.9	221.1	199.0
Watershed Summary																
Basin Area (Square Mile)	5.9				5.9				5.9				5.9			
Peak Watershed Discharge (CSM)	174.2	137.4	115.6	101.2	159.8	127.2	105.0	94.0	181.5	117.1	92.3	78.5	102.4	144.9	111.3	96.0

Figure 9-72. Instantaneous Discharge Capacity for the C3W Watershed for 100-yera, 25-year, 10-year, and 5-year 3-day Design Storm Events

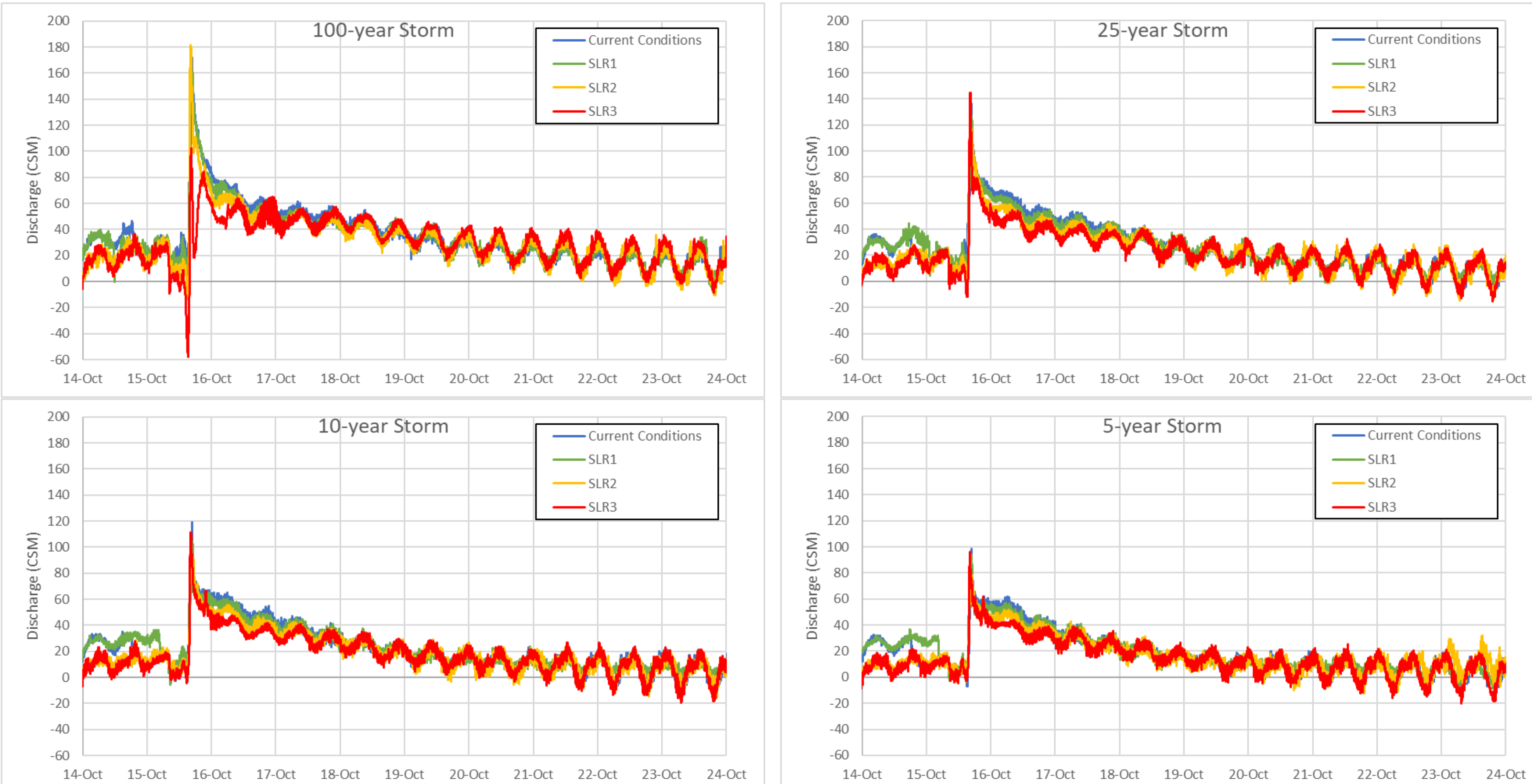
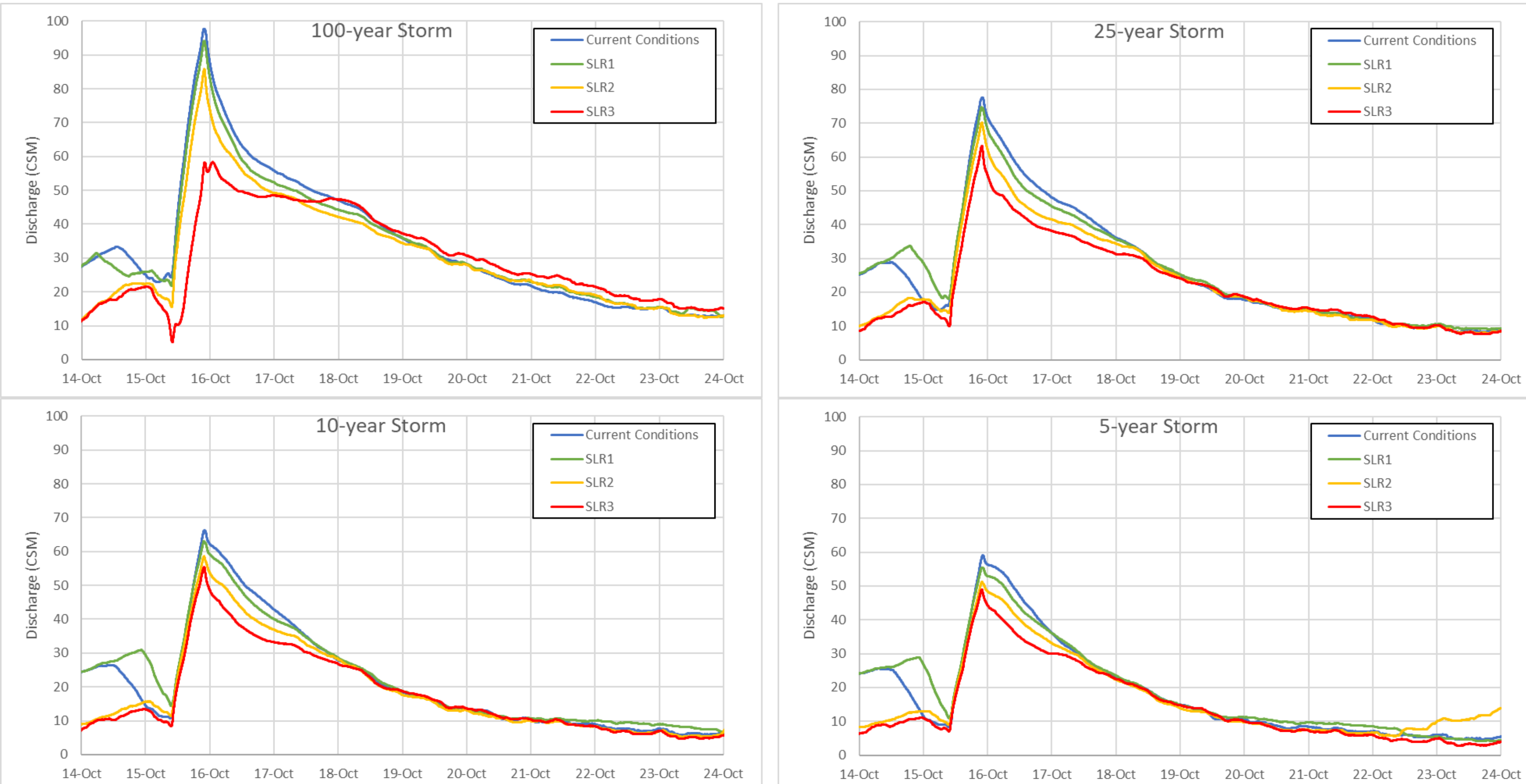


Figure 9-73. 12-Hour Moving Average Discharge Capacity for the C3W Watershed



9.2.2.1. INTER-BASIN TRANSFERS

During wet season operations, the C3W Watershed interacts primarily with the C4 Watershed, receiving flows from the C4 Canal at the intersection of the C4 and the Coral Gables Canal (or SW 8th Street and the Palmetto Expressway). Normally, wet season flows move from the C4 Watershed into the C3W Watershed. However, during the storm event, flows reverse and move from the C3W into the C4 during the peak of the storm. During this flow reversal spike, both the G422 Pump Station and the G420 Pump Station are pumping at maximum capacity, 623 CFS and 669 CFS respectively. Discharge at the start of the Coral Gables Canal is shown in **Figure 9-74** and **Figure 9-75** for the 100-year and 25-year design storms, respectively. Negative flows in these graphs indicate flows are moving into the C4 Watershed.

Figure 9-74. Exchange between the C3W and C4 Watershed for the 100-year Design Storm

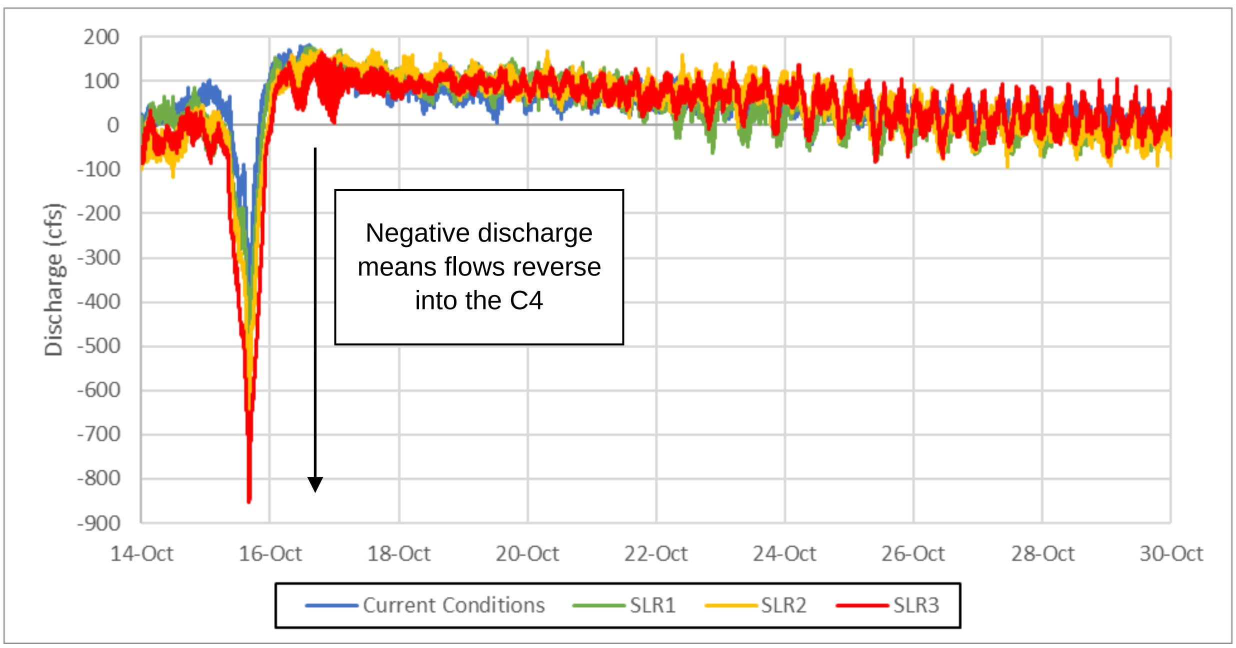
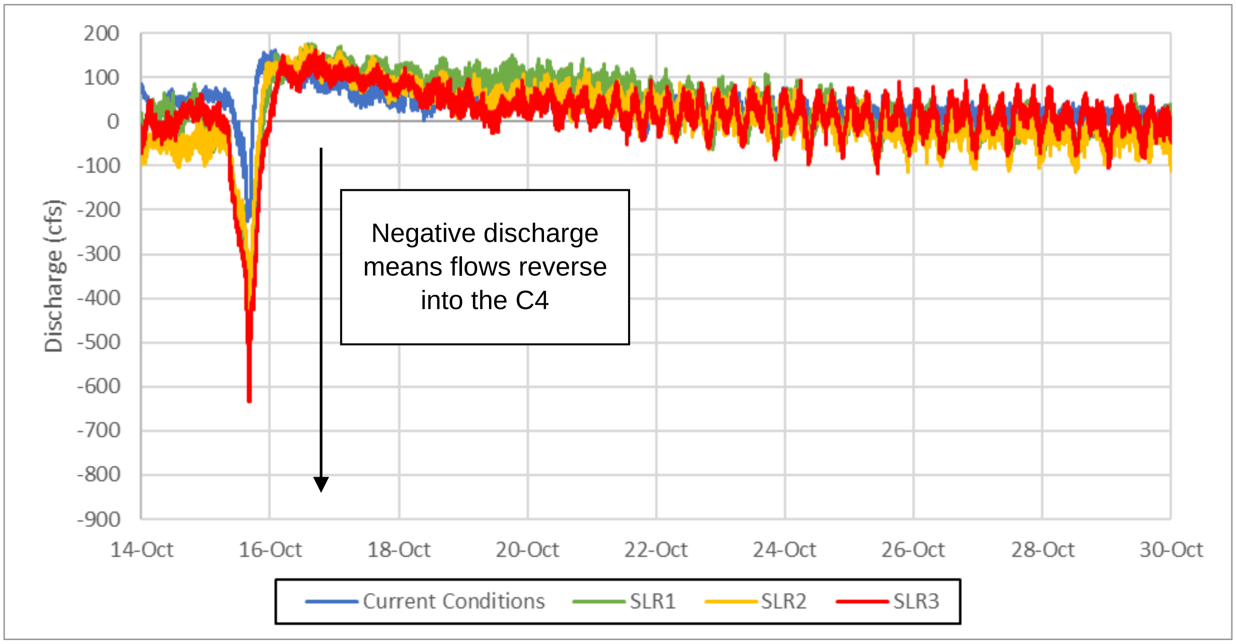


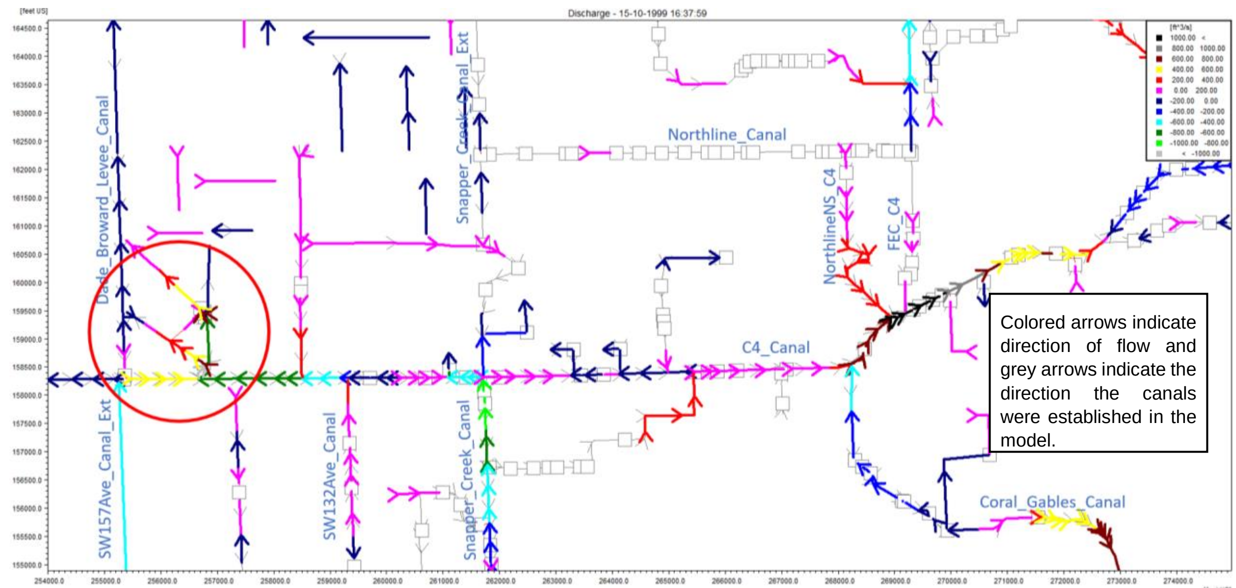
Figure 9-75. Exchange between the C3W and C4 Watershed for the 25-year Design Storm



Since this reversal of flows into the C4 watershed is present in Coral Gables Canal (C3W), Snapper Creek Canal (C2), SW 132nd Avenue Canal Extension (C2), and SW 157th Avenue Canal Extension (C2), further exploration of the cause of this flow reversal was performed. Instantaneous flows were reviewed throughout the model at the peak of the flow reversal into the C4 Watershed to determine where the flows are going. **Figure 9-76** shows the flows and direction of flow at all locations for the peak of the flow reversal (around 10/15 at 4:37PM). At the same time as the flow reversal in the C3W watershed, flows are also reversed in the C2 watershed, going into the C4 at the same time. In addition, flows at S25B (red dashed square in figure) are also reversed during this time due to tidal stages overtopping the closed gated structure. Therefore, the flow reversal is due to the C4 Impoundment (circled in red) pulling 1,292 CFS at that time.

The C4 Impoundment is filling and pulling water from the C4 Canal; however, at the confluence of the C4 and the Coral Gables Canal, the flows are again positive due to contributions from the watershed. However, this dip in flows and water levels causes a pull from the Coral Gables Canal, which has slightly higher water levels just prior to the peak of the storm. These flows from Coral Gables Canal then enter the C4 Canal and are seen as a spike just downstream of the confluence.

Figure 9-76. Instantaneous Flow Direction at Peak Negative Flow for SLR +1ft 100-year Storm



9.2.3. C3W – PM #3 STRUCTURE PERFORMANCE

The purpose of PM #3 is to determine the effect of SLR on tidal structure flow performance. For this metric, an evaluation based on structure design and existing operational protocols was conducted for existing conditions and compared to simulations including SLR over the various storm events. PM #3 will provide only gravity flows through the structure, where PM #4 will also account for structure flows as well as overtopping, to differentiate between structure capacity and the basic inabilities of the structure to provide tidal protections.

As per the structure data sheet, the G93 tidal structure is operated with the intention to maintain water levels in the C3W Watershed and pass the design flood (from a one in ten-year flood), plus a small discharge from the C4 Watershed, without impacting upstream flooding. In addition, the structure is used to restrict flows to decrease stages and velocities that may cause damage to downstream areas, while preventing saline intrusion during high tides. **Table 9-25** provides the design parameters for structure G93 as provided in the Water Control Operations Atlas.

Table 9-25. Design Parameters for Structure G93

DESIGN PARAMETERS	G93
Design Discharge	640 CFS (40% SPF): 540 CFS from C3 and 100 CFS from C4
Design HW	2.94 ft-NAVD (4.5 ft-NGVD)
Design TW	1.44 ft-NAVD (3.0 ft-NGVD)
Optimum HW	1.24 ft-NAVD (2.8 ft-NGVD)
Optimum TW	Tidal
Maximum Gate Opening	5 ft
Water Level which will Bypass Structure	4.44 ft-NAVD (6.00 ft-NGVD)
Water Level which will Overtop Gates when Closed	1.64 ft-NAVD (3.20 ft-NGVD)
Low Range Operational Trigger	0.74 f- NAVD (2.30 ft-NGVD)

Figure 9-77 and **Figure 9-78** provide the instantaneous discharge, HW and TW at G93 during the current conditions simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-79** and **Figure 9-80** provide the 12-hour moving average for the discharge, HW, and TW at G93 during the current conditions simulation for the 100-year and 25-year design events, respectively, which removes the influence of the tides. **Figure 9-81** and **Figure 9-82** provide the instantaneous discharge, HW and TW at G93 during the future conditions SLR1 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-83** and **Figure 9-84** provide the 12-hour moving average for the discharge, HW, and TW at G93 during the future conditions SLR1 simulation for the 100-year and 25-year design events, respectively, which removes the influence of the tides. **Figure 9-85** and **Figure 9-86** provide the instantaneous discharge, HW and TW at G93 during the future conditions SLR2 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-87** and **Figure 9-88** provide the 12-hour moving average for the discharge, HW, and TW at G93 during the future conditions SLR2 simulation for the 100-year and 25-year design events, respectively. **Figure 9-89** and **Figure 9-90** provide the instantaneous discharge, HW and TW at G93 during the future conditions SLR2 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-91** and **Figure 9-92** provide the 12-hour moving average for the discharge, HW, and TW at G93 during the future conditions SLR2 simulation for the 100-year and 25-year design events, respectively. Figures are provided for instantaneous discharge and 12-hour moving discharge for all design storms and SLR simulations in **Appendix B**.

Figure 9-77. Instantaneous Discharge and Stages at G93 for the 100-year 3-day Current Conditions Event

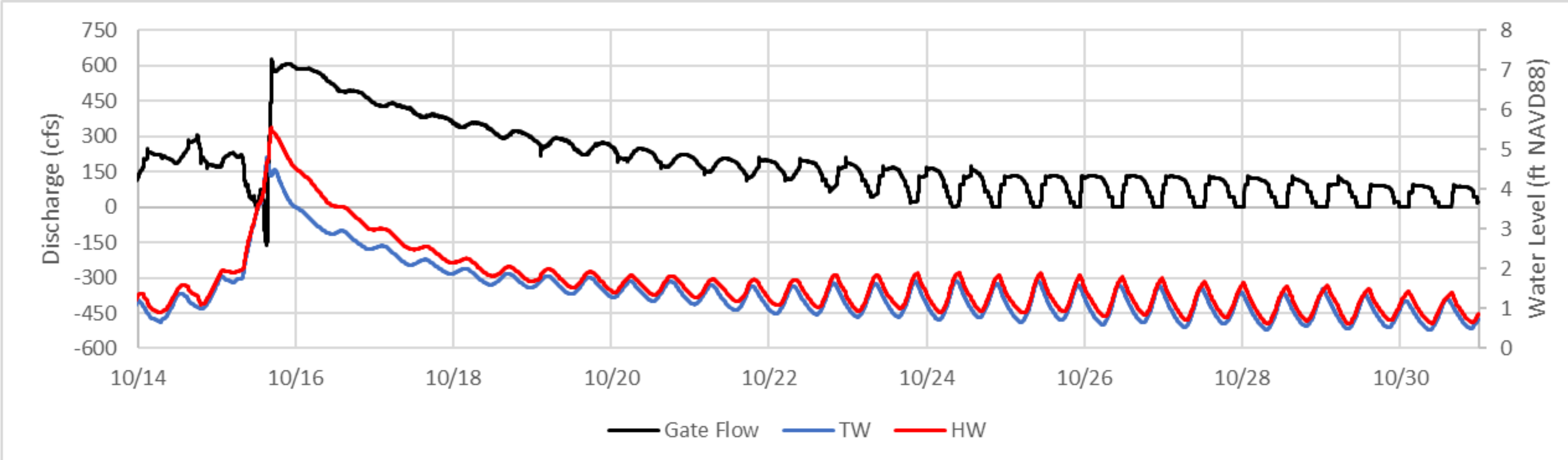


Figure 9-78. Instantaneous Discharge and Stages at G93 for the 25-year 3-day Current Conditions Event

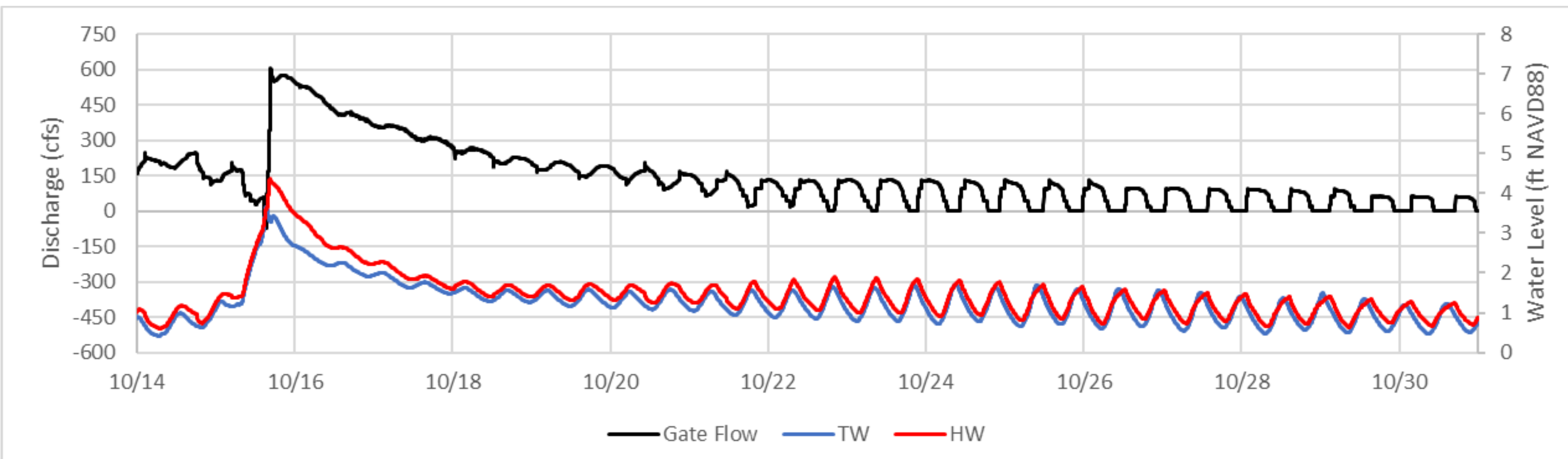


Figure 9-79. 12-Hour Moving Average Discharge and Stages at G93 for the 100-year 3-day Current Conditions Event

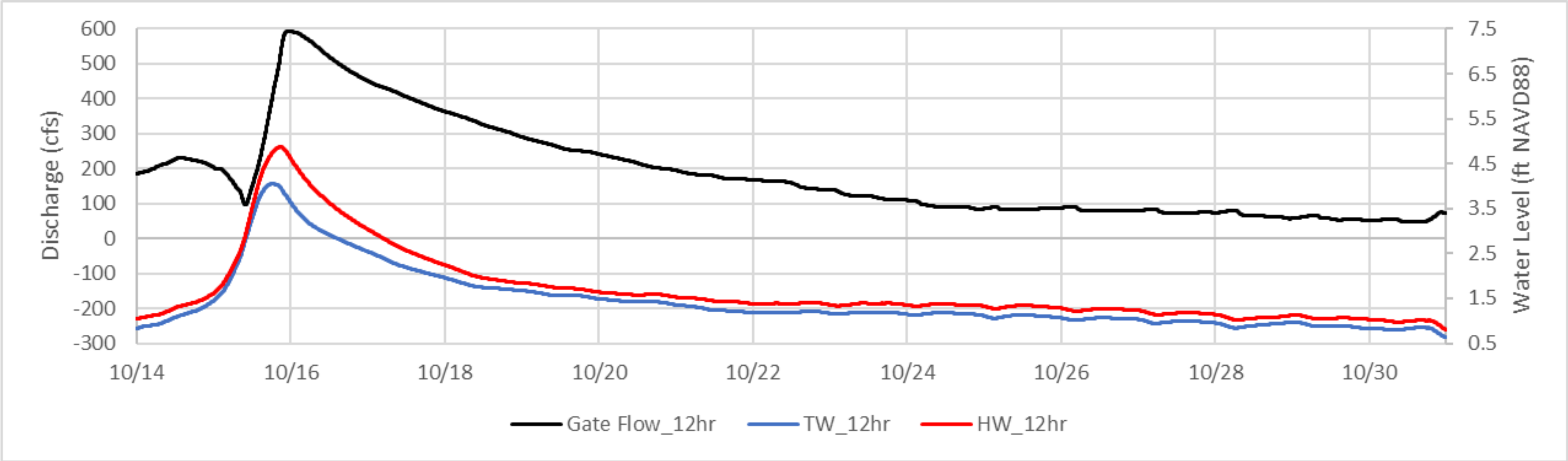


Figure 9-80. 12-Hour Moving Average Discharge and Stages at G93 for the 25-year 3-day Current Conditions Event

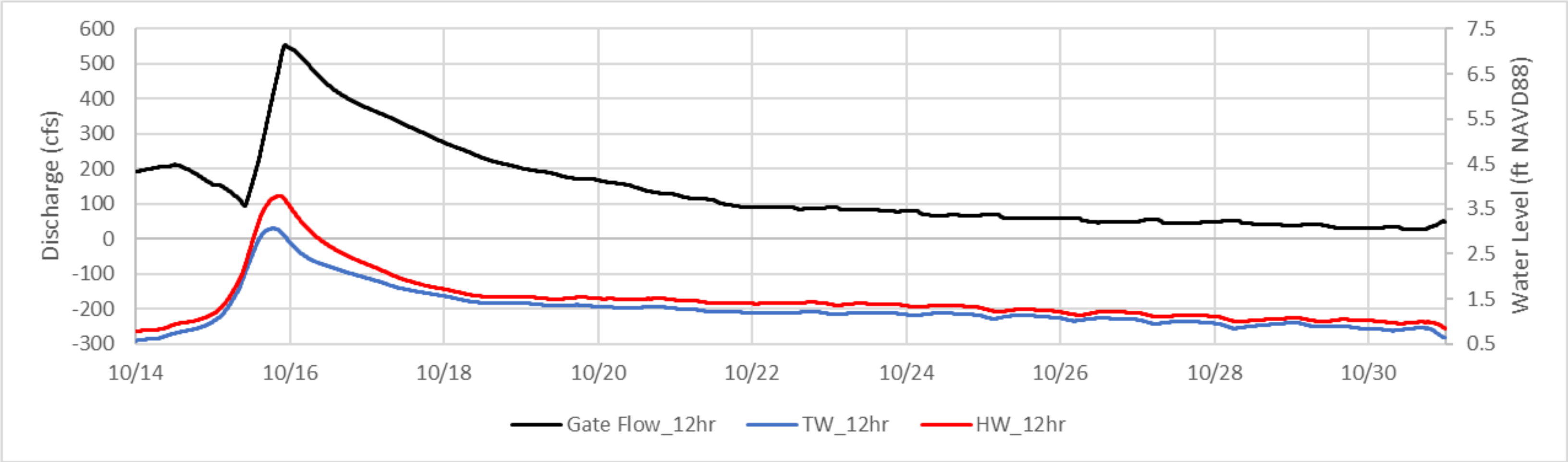


Figure 9-81. Instantaneous Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR1) Event

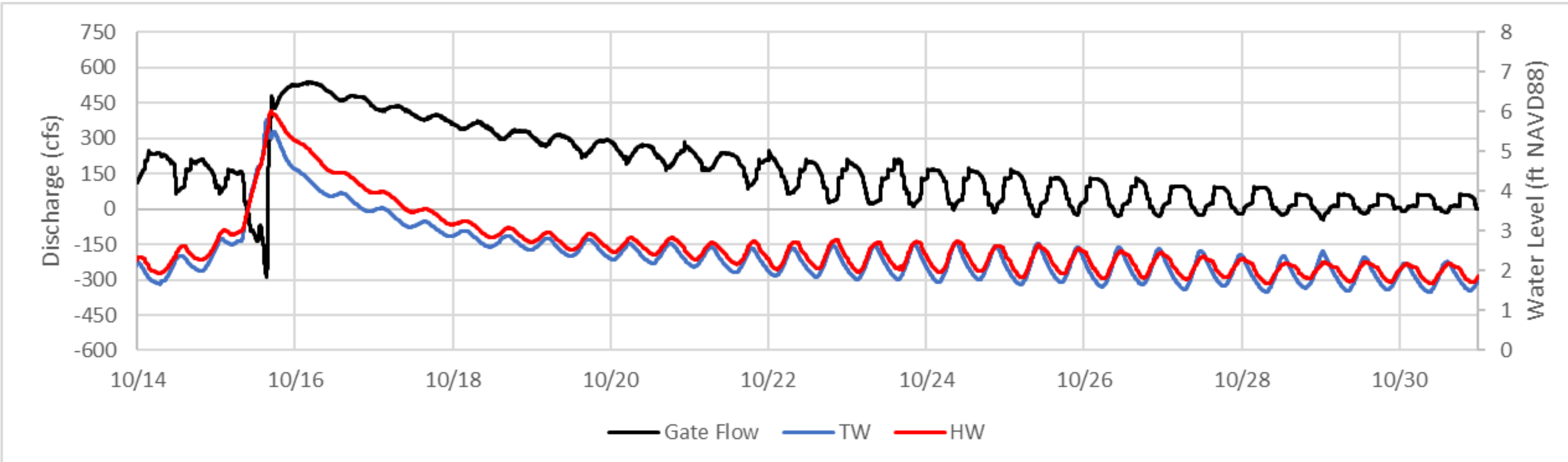


Figure 9-82. Instantaneous Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR1) Event

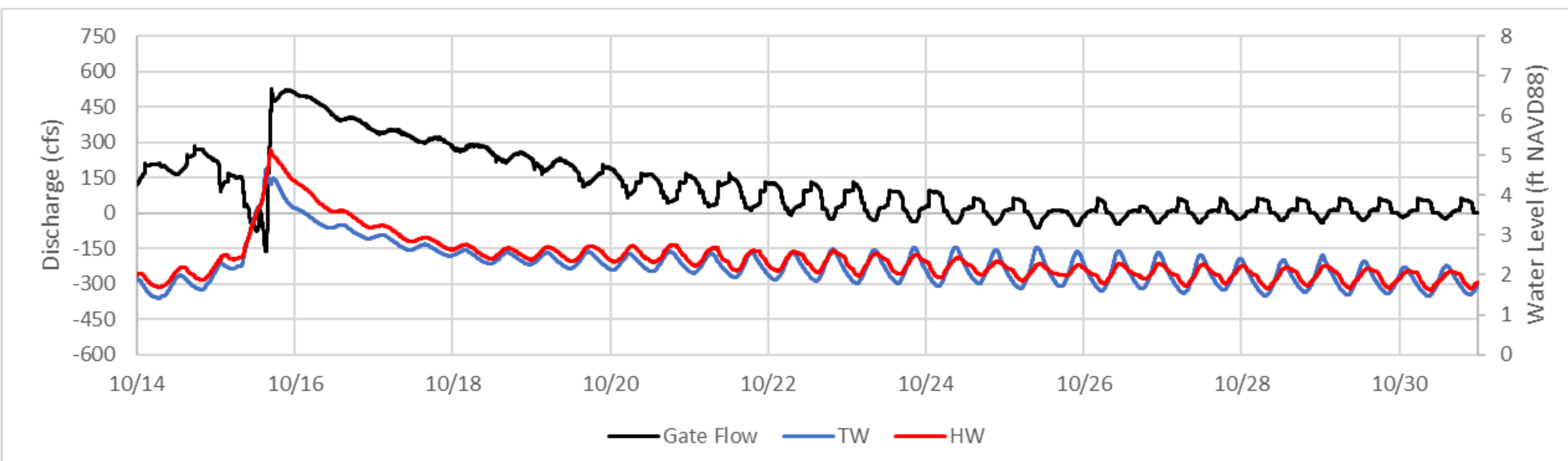


Figure 9-83. 12-Hour Moving Average Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR1) Event

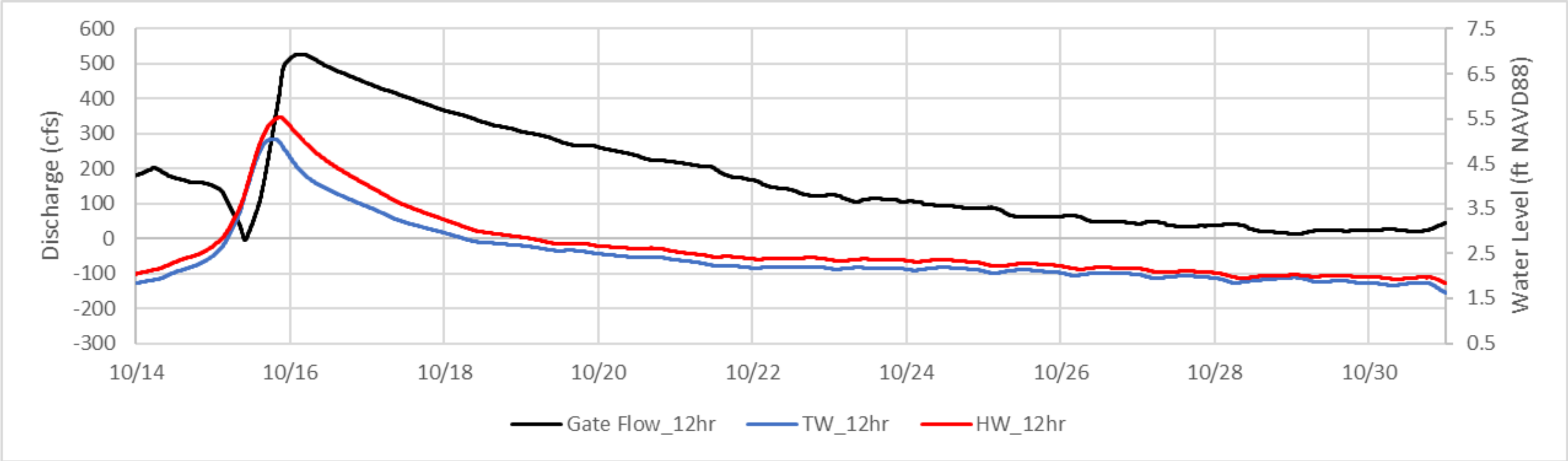


Figure 9-84. 12-Hour Moving Average Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR1) Event

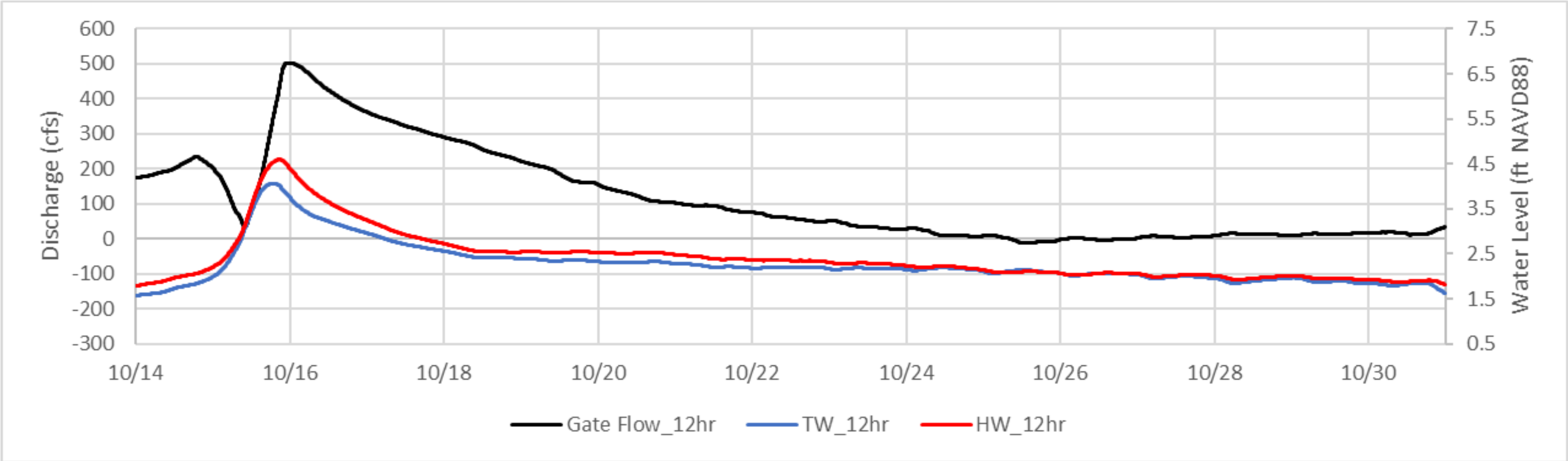


Figure 9-85. Instantaneous Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR2) Event

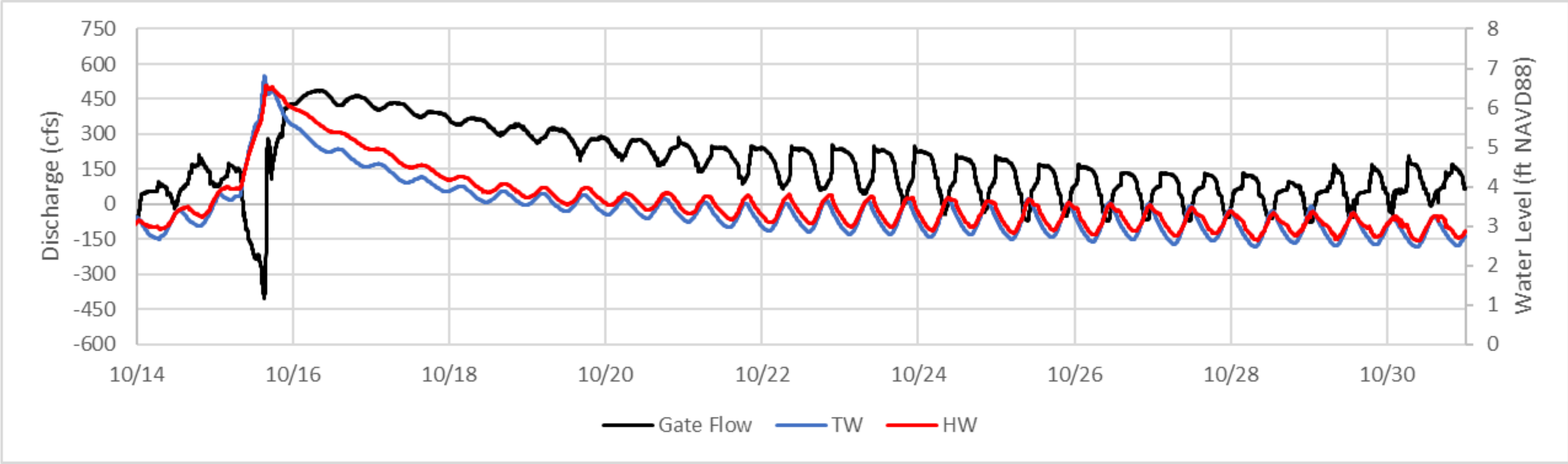


Figure 9-86. Instantaneous Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR2) Event

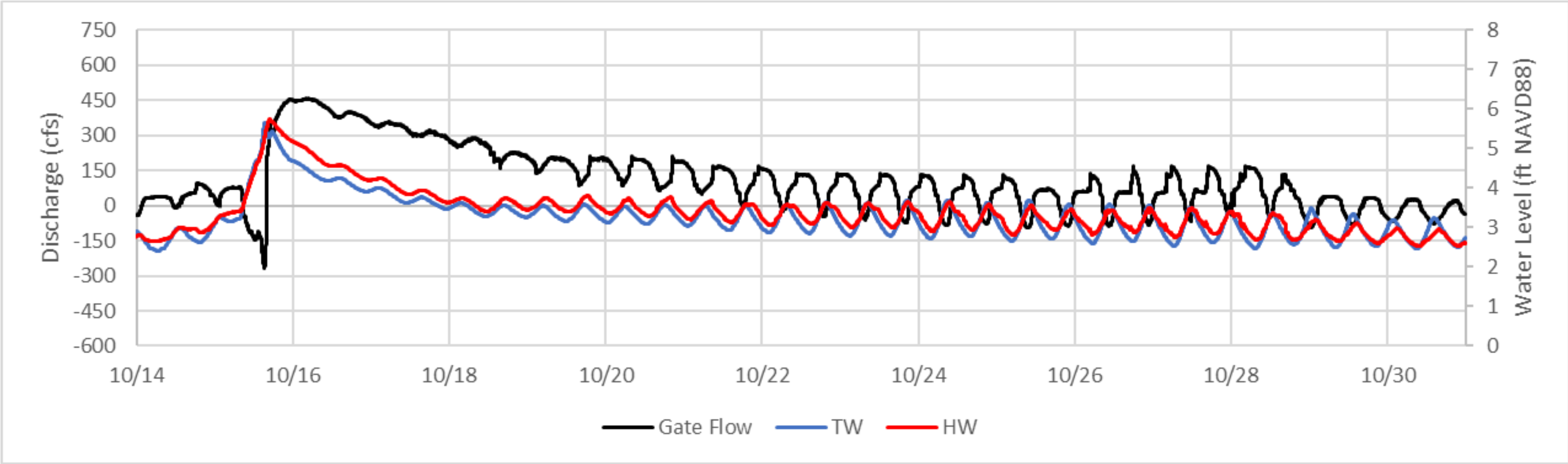


Figure 9-87. 12-Hour Moving Average Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR2) Event

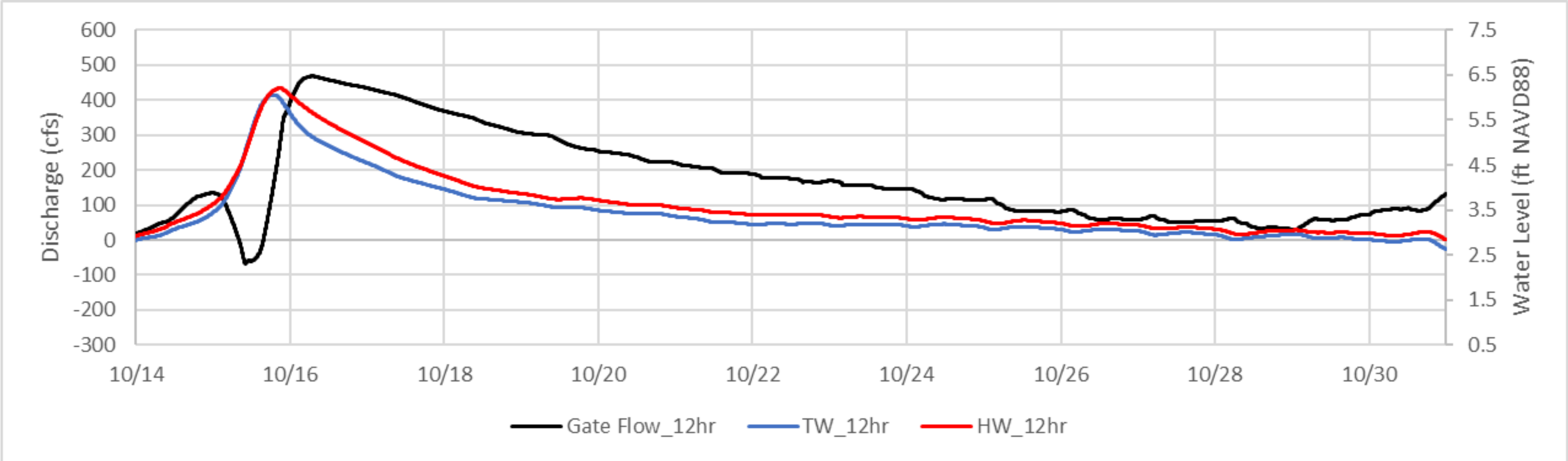


Figure 9-88. 12-Hour Moving Average Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR2) Event

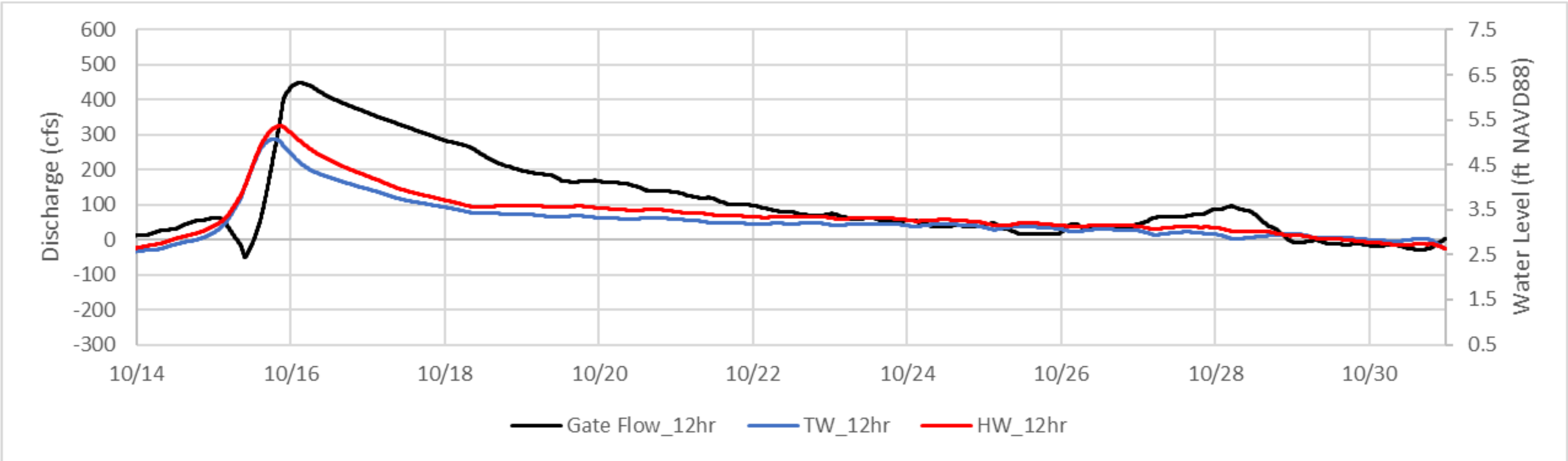


Figure 9-89. Instantaneous Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR3) Event

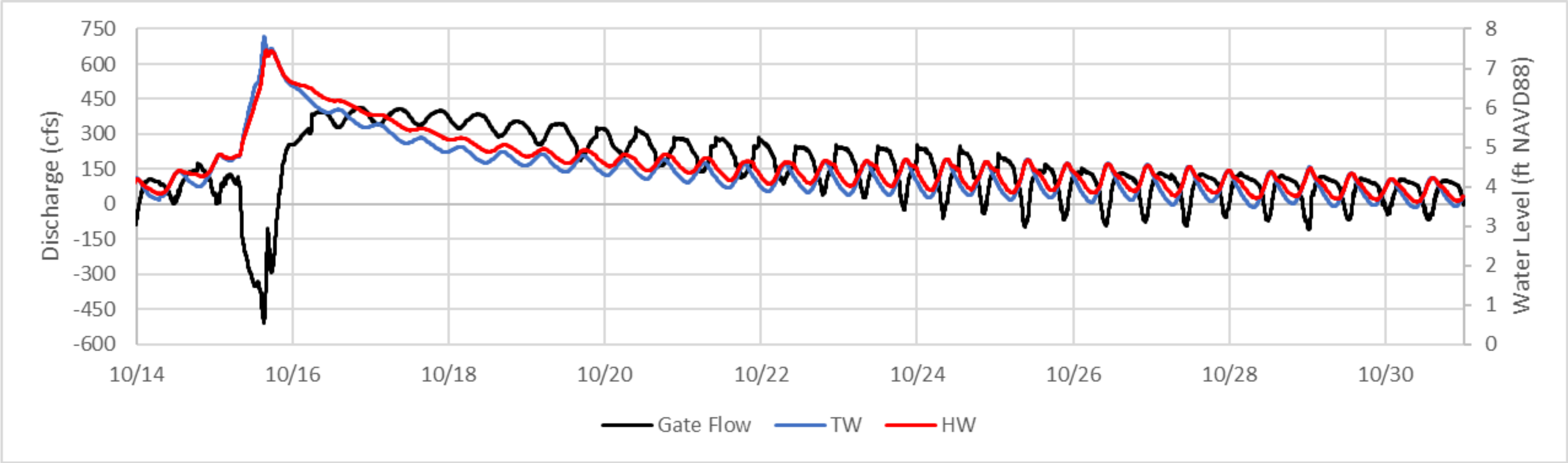


Figure 9-90. Instantaneous Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR3) Event

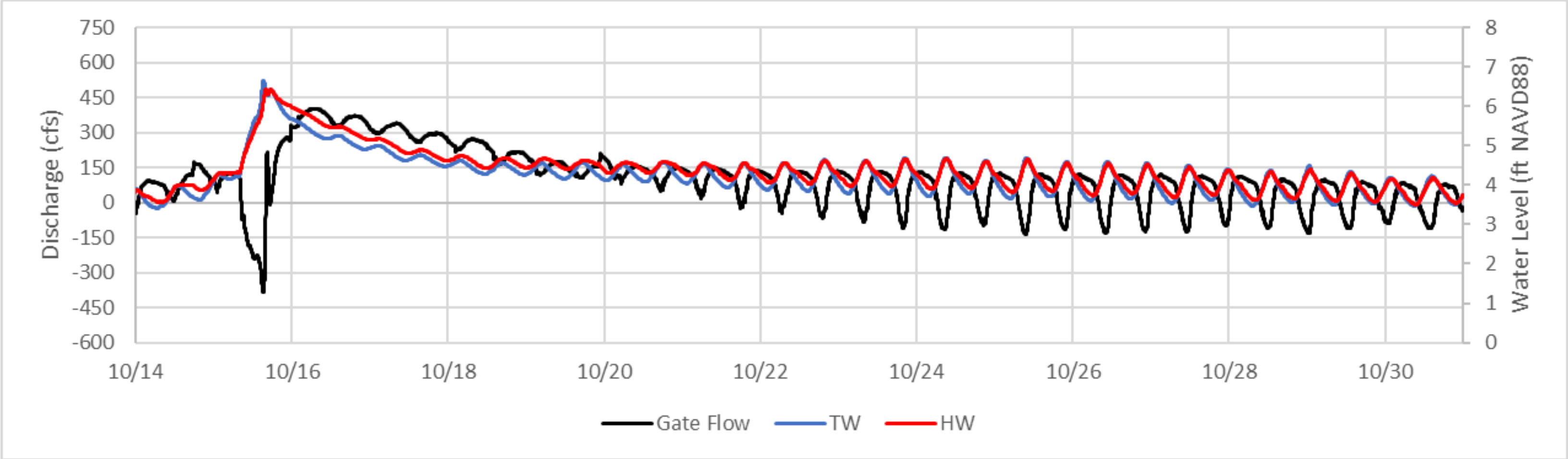


Figure 9-91. 12-Hour Moving Average Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR3) Event

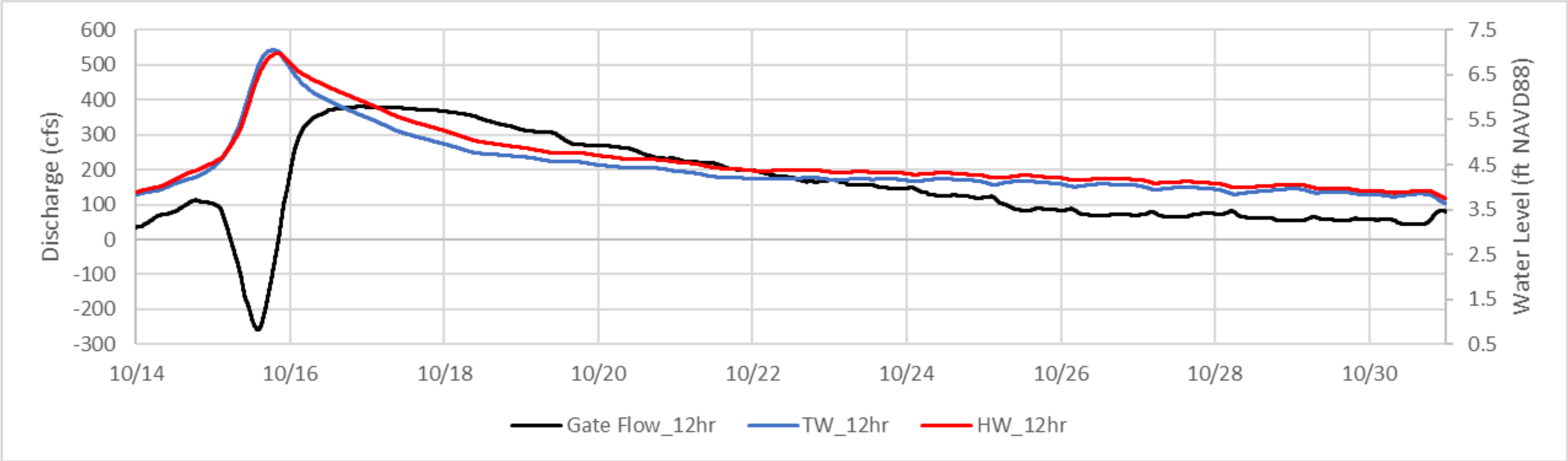
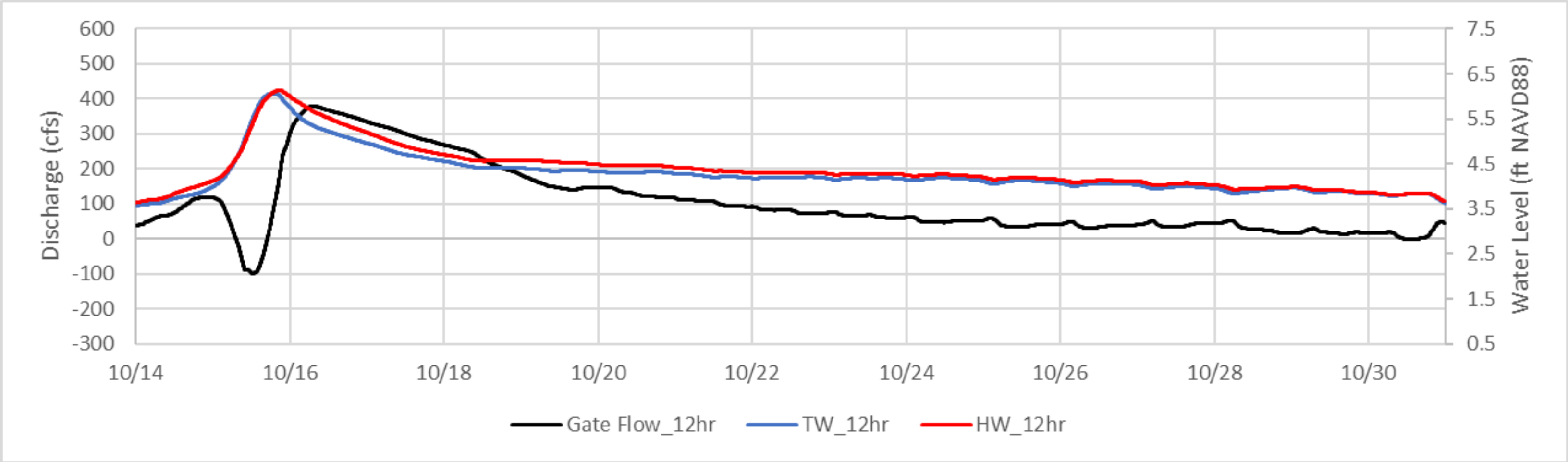


Figure 9-92. 12-Hour Moving Average Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR3) Event



A summary of the instantaneous and 12-hour moving average conditions at the time of peak discharge through G93 are provided in **Table 9-26** and **Table 9-27,** respectively for all design storm return periods and future SLR conditions.

Table 9-26. Summary of Conditions at the Time of Peak Discharge through G93 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak Q	Peak Q (CFS)	HW at Peak Q (ft-NAVD)	TW at Peak Q (ft-NAVD)	ΔH at Peak Q (ft)	Gate Opening at Peak Q* (ft)
100-Year	**Current**	10/15 16:47	616.29	5.43	4.35	1.08	5.00
	SLR1	10/16 4:50	538.85	5.01	4.24	0.77	5.00
	SLR2	10/16 7:49	486.77	5.60	5.01	0.60	5.00
	SLR3	10/16 21:08	413.85	5.93	5.53	0.40	5.00
25-Year	**Current**	10/15 22:36	602.61	4.31	3.29	1.02	5.00
	SLR1	10/15 16:53	528.99	5.05	4.32	0.74	5.00
	SLR2	10/16 4:55	457.22	4.95	4.44	0.51	5.00
	SLR3	10/16 7:32	403.56	5.66	5.29	0.37	5.00
10-Year	**Current**	10/15 16:41	552.67	3.53	2.71	0.82	5.00
	SLR1	10/15 21:15	502.56	4.03	3.38	0.65	5.00
	SLR2	10/15 22:59	436.94	4.70	4.24	0.46	5.00
	SLR3	10/16 6:44	385.92	5.25	4.92	0.33	5.00
5-Year	**Current**	10/15 16:43	515.05	3.00	2.31	0.69	5.00
	SLR1	10/15 21:02	469.80	3.60	3.05	0.55	5.00
	SLR2	10/15 22:51	424.07	4.34	3.91	0.42	5.00
	SLR3	10/15 21:02	367.68	5.35	5.05	0.30	5.00

*A gate opening of 5 ft represents the gate full open.

Table 9-27. Summary of Conditions at the Time of Peak Discharge through G93 (12 hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak Q	Peak Q (CFS)	HW at Peak Q (ft-NAVD)	TW at Peak Q (ft-NAVD)	ΔH at Peak Q (ft)	Gate Opening at Peak Q* (ft)
100-Year	**Current**	10/15 22:47	578.71	4.70	3.78	0.92	5.00
	SLR1	10/16 3:15	526.96	5.08	4.35	0.73	5.00
	SLR2	10/16 7:01	467.72	5.66	5.12	0.54	5.00
	SLR3	10/16 21:56	380.24	5.93	5.60	0.32	5.00
25-Year	**Current**	10/15 22:36	541.18	3.66	2.87	0.78	5.00
	SLR1	10/15 22:53	502.20	4.50	3.85	0.65	5.00
	SLR2	10/16 3:08	448.01	5.02	4.53	0.49	5.00
	SLR3	10/16 7:51	378.84	5.65	5.33	0.32	5.00
10-Year	**Current**	10/15 22:35	488.83	2.98	2.37	0.61	5.00
	SLR1	10/15 22:54	475.09	3.91	3.35	0.56	5.00
	SLR2	10/16 1:59	423.88	4.57	4.15	0.42	5.00
	SLR3	10/16 3:51	366.77	5.35	5.05	0.29	5.00
5-Year	**Current**	10/15 22:31	444.77	2.51	2.02	0.49	4.99
	SLR1	10/16 0:09	441.81	3.40	2.93	0.47	5.00
	SLR2	10/16 1:04	406.15	4.25	3.87	0.38	5.00
	SLR3	10/16 3:02	356.73	5.05	4.78	0.27	5.00

*A gate opening of 5 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak HW at G93 are provided in **Table 9-28** and

Table 9-29, respectively for all design storm return periods and future SLR conditions.

Table 9-28. Summary of Conditions at the Time of Peak HW through G93 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak HW	Q at Peak HW (CFS)	Peak HW (ft-NAVD)	TW at Peak HW (ft-NAVD)	ΔH at Peak HW (ft)	Gate Opening at Peak HW* (ft)
100-Year	**Current**	10/15 16:34	440.22	5.52	4.34	1.18	3.5
	SLR1	10/15 16:45	392.17	6.02	5.34	0.68	3.5
	SLR2	10/15 15:50	-134.33	6.58	6.60	-0.02	0
	SLR3	10/15 15:58	-290.72	7.48	7.54	-0.05	0
25-Year	**Current**	10/15 20:41	412.71	4.35	3.29	1.06	3.5
	SLR1	10/15 16:35	380.58	5.15	4.29	0.86	3.5
	SLR2	10/15 16:47	317.62	5.75	5.30	0.45	3
	SLR3	10/15 15:54	-175.37	6.44	6.47	-0.03	0
10-Year	**Current**	10/15 16:22	327.41	3.65	2.74	0.91	3
	SLR1	10/15 16:41	361.68	4.47	3.71	0.76	3.5
	SLR2	10/15 16:49	275.10	5.20	4.72	0.48	3
	SLR3	10/15 16:22	224.96	5.87	5.74	0.13	0
5-Year	**Current**	10/15 16:24	309.07	3.11	2.33	0.79	3
	SLR1	10/15 16:37	342.30	3.97	3.31	0.66	3.5
	SLR2	10/15 16:52	251.52	4.79	4.33	0.46	3
	SLR3	10/15 17:36	197.60	5.54	5.42	0.12	0

*A gate opening of 5 ft represents the gate full open.

Table 9-29. Summary of Conditions at the Time of Peak HW through G93 (12 hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak HW	Q at Peak HW (CFS)	Peak HW (FT-NAVD)	TW at Peak HW (FT-NAVD)	ΔH at Peak HW (FT)	Gate Opening at Peak HW* (FT)
100-Year	**Current**	10/15 20:48	505.10	4.82	3.98	0.84	4.36
	SLR1	10/15 20:52	424.36	5.53	4.98	0.55	4.31
	SLR2	10/15 20:47	272.85	6.21	5.98	0.23	2.46
	SLR3	10/15 20:14	-5.86	6.98	7.02	-0.04	0.00
25-Year	**Current**	10/15 20:41	475.50	3.75	3.02	0.72	4.35
	SLR1	10/15 20:48	433.23	4.60	4.02	0.58	4.32
	SLR2	10/15 20:47	340.36	5.36	5.02	0.35	4.14
	SLR3	10/15 20:44	171.98	6.13	6.02	0.11	1.26
10-Year	**Current**	10/15 20:31	426.26	3.07	2.50	0.57	4.29
	SLR1	10/15 20:52	414.30	4.00	3.48	0.52	4.35
	SLR2	10/15 20:53	341.87	4.81	4.48	0.33	4.25
	SLR3	10/15 20:44	225.11	5.65	5.49	0.16	2.23
5-Year	**Current**	10/15 20:23	382.11	2.59	2.14	0.45	4.22
	SLR1	10/15 20:52	384.10	3.55	3.12	0.43	4.33
	SLR2	10/15 21:03	338.31	4.43	4.11	0.32	4.25
	SLR3	10/15 20:36	232.38	5.31	5.13	0.18	2.51

*A gate opening of 5 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak TW at G93 are provided in **Table 9-30** and **Table 9-31**, respectively for all design storm return periods and future SLR conditions.

Table 9-30 shows that there are instances when there are negative flows, yet the gate opening is zero, this indicates that the tailwater is higher than the top of the closed gate (1.738 ft-NAVD) and there is a negative head differential across this structure. However, negative flows can also occur when the gate is in the process of closing and the head differential is negative.

Table 9-30. Summary of Conditions at the Time of Peak Tailwater at G93 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (ft-NAVD)	Peak TW (ft-NAVD)	ΔH at Peak TW (ft)	Gate Opening at Peak TW* (ft)
100-Year	**Current**	10/15 15:14	-169.25	4.62	4.82	-0.20	0
	SLR1	10/15 15:14	-287.89	5.36	5.82	-0.46	0
	SLR2	10/15 15:14	-405.47	6.22	6.82	-0.60	0
	SLR3	10/15 15:14	-512.50	7.21	7.82	-0.61	0
25-Year	**Current**	10/15 18:36	-74.68	3.59	3.67	-0.07	0
	SLR1	10/15 15:14	-162.69	4.44	4.67	-0.23	0
	SLR2	10/15 15:14	-268.74	5.25	5.67	-0.42	0
	SLR3	10/15 15:14	-382.47	6.11	6.67	-0.55	0
10-Year	**Current**	10/15 15:14	-43.99	2.97	3.04	-0.07	0
	SLR1	10/15 15:14	-118.80	3.81	4.04	-0.23	0
	SLR2	10/15 15:14	-206.13	4.69	5.04	-0.34	0
	SLR3	10/15 15:14	-305.06	5.60	6.04	-0.44	0
5-Year	**Current**	10/15 15:14	-27.06	2.51	2.60	-0.08	0
	SLR1	10/15 15:14	-91.00	3.36	3.60	-0.24	0
	SLR2	10/15 15:14	-170.86	4.26	4.60	-0.34	0
	SLR3	10/15 15:14	-249.24	5.28	5.60	-0.32	0

*A gate opening of 5 ft represents the gate full open.

Table 9-31. Summary of Conditions at the Time of Peak Tailwater at G93 (12 hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (FT-NAVD)	Peak TW (FT-NAVD)	ΔH at Peak TW (FT)	Gate Opening at Peak TW* (FT)
100-Year	Current	10/15 18:27	395.28	4.71	4.05	0.66	3.38
	SLR1	10/15 18:27	292.47	5.44	5.05	0.39	3.31
	SLR2	10/15 18:27	139.13	6.14	6.05	0.09	1.49
	SLR3	10/15 18:27	-97.13	6.94	7.05	-0.11	0.00
25-Year	Current	10/15 18:36	393.12	3.69	3.06	0.63	3.56
	SLR1	10/15 18:36	333.44	4.52	4.06	0.46	3.40
	SLR2	10/15 18:36	232.97	5.30	5.06	0.24	3.23
	SLR3	10/15 18:36	66.85	6.08	6.06	0.02	0.43
10-Year	Current	10/15 18:44	360.34	3.03	2.52	0.51	3.62
	SLR1	10/15 18:44	326.68	3.94	3.52	0.41	3.46
	SLR2	10/15 18:44	243.25	4.75	4.52	0.23	3.36
	SLR3	10/15 18:44	135.17	5.61	5.52	0.09	1.39
5-Year	Current	10/15 19:03	337.02	2.57	2.15	0.42	3.72
	SLR1	10/15 19:03	316.99	3.51	3.15	0.36	3.58
	SLR2	10/15 19:03	251.59	4.38	4.15	0.23	3.42
	SLR3	10/15 19:03	172.04	5.29	5.15	0.14	1.87

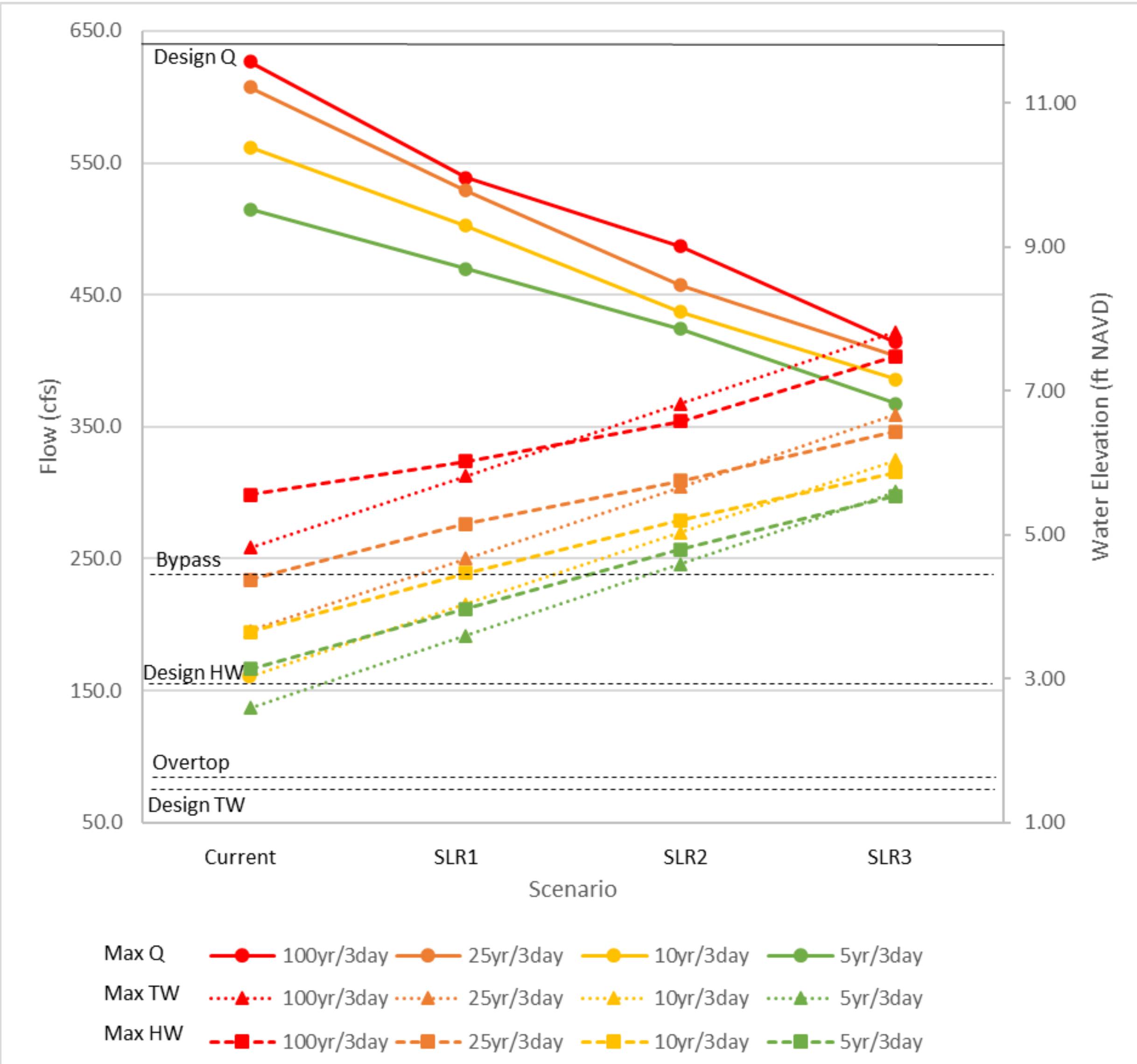
*A gate opening of 5 ft represents the gate full open.

Figure 9-93 shows a summary of the instantaneous peak discharge, HW, and TW at G93 for all design storm return periods and future SLR conditions. The design parameter values listed in **Table 9-25** are shown graphically in the figure below with bypass indicating the water level which will bypass the structure and overtop indicating the water level which will overtop the gates when the gates are closed. Note that the peak discharge, HW, and TW occur at different times for each scenario.

The maximum HW and TW exceed the design HW and TW for all conditions simulations. The maximum HW and TW also exceed the water level which will overtop the gates for all conditions, sometimes occurring when the head differential is less than 0.1 feet, and the gates are therefore closed. TW elevations that exceed the overtopping elevation when the gates are closed can result in flow entering the basin from storm surge and/or tide. The HW at G93 exceeds the water level that will bypass the structure for all future SLR scenarios (SLR1, SLR2, SLR3) except for the 5-year SLR1 scenario. The TW exceeds this bypass elevation during all future SLR scenarios (SLR1, SLR2, SLR3) except for the

5-year and 10-year SLR1 scenarios. Flow that bypasses the structure can contribute to flooding of neighborhoods around G93.

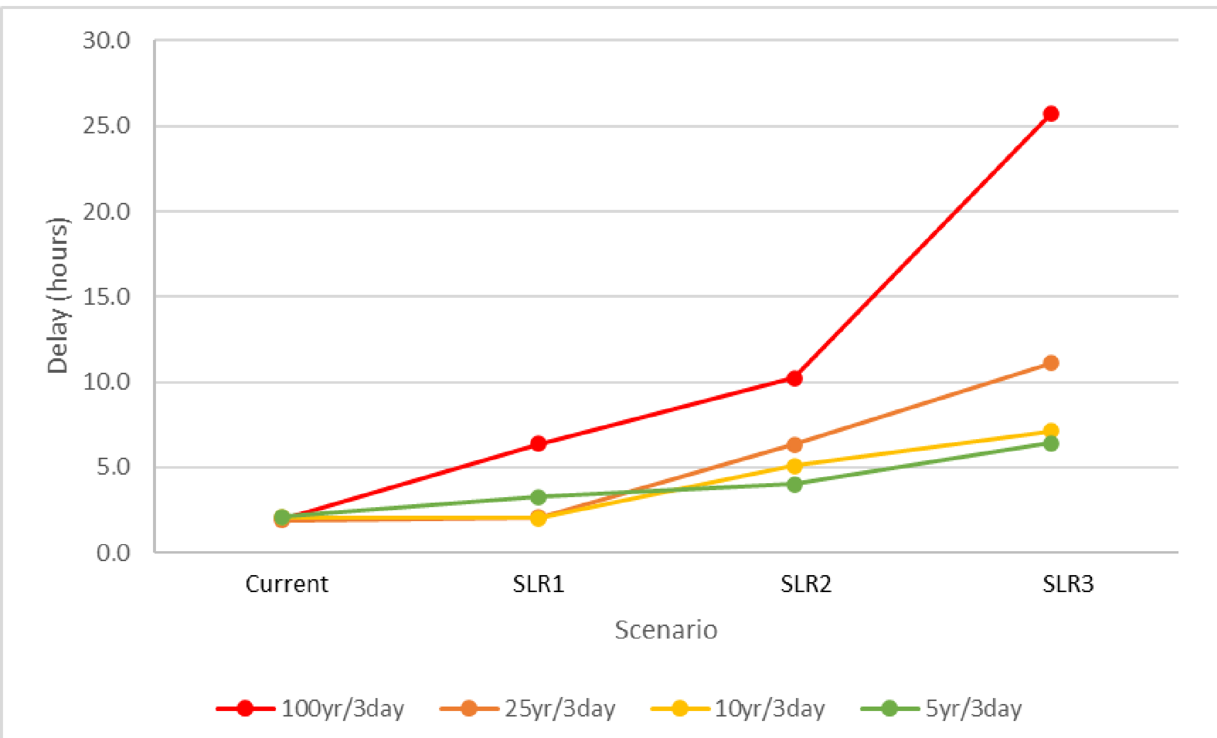
Figure 9-93. Summary of Instantaneous Peak Discharge, HW, and TW at G93



The peak discharge at G93 falls below the design value for all design storms during current and future SLR scenarios. The increase in TW levels due to SLR, decreases the head differential at the structure (as shown in **Figure 9-93**) and inhibits the flow out of the basin, potentially creating additional flooding upstream of the structure as shown in PM #1. Not only does high TW conditions inhibit flow from leaving the basin, as shown in **Table 9-30**, during most future conditions simulations, TW elevations exceed HW elevations and the level which will overtop and/or bypass the structure, resulting in flow entering the C3W basin from storm surge.

Without TW limitations, increased HW would generate more flow through a gravity structure (i.e., expecting to see maximum flow through the structure occurring near the time of maximum HW). The time between the 12-hour moving average peak HW (**Table 9-29**) and the 12-hour moving average peak discharge (**Table 9-27**) is shown in **Figure 9-94** for each design storm. This delay in the structure's ability to discharge water is a result of high TW conditions from the storm surge and is shown to increase with increasing future SLR conditions.

Figure 9-94. Time Between Peak HW and Peak Discharge at G93 for the 12-Hour Moving Average



9.2.4. C3W – PM #4 PEAK STORM RUNOFF

The purpose of PM #4 is to determine the effect of SLR on the maximum peak storm runoff, or maximum conveyance capacity of the watershed. For this metric, 12-hour moving average flow hydrographs downstream of G93 and the maximum 12-hour moving average total flow was determined for each design storm event and SLR scenario.

The 12-hour moving average discharge hydrographs for each SLR scenario can be found in **Figure 9-95** for the 100-year storm, **Figure 9-96** for the 25-year storm, **Figure 9-97** for the 10-year storm, and **Figure 9-98** for the 5-year storm. Downstream flows for G93 comprise of the discharge from the two gates in addition to overtopping of the structure, if applicable.

Figure 9-95. 12-Hour Moving Average of Flows at G93 for the 100-year 3-day Design Storm

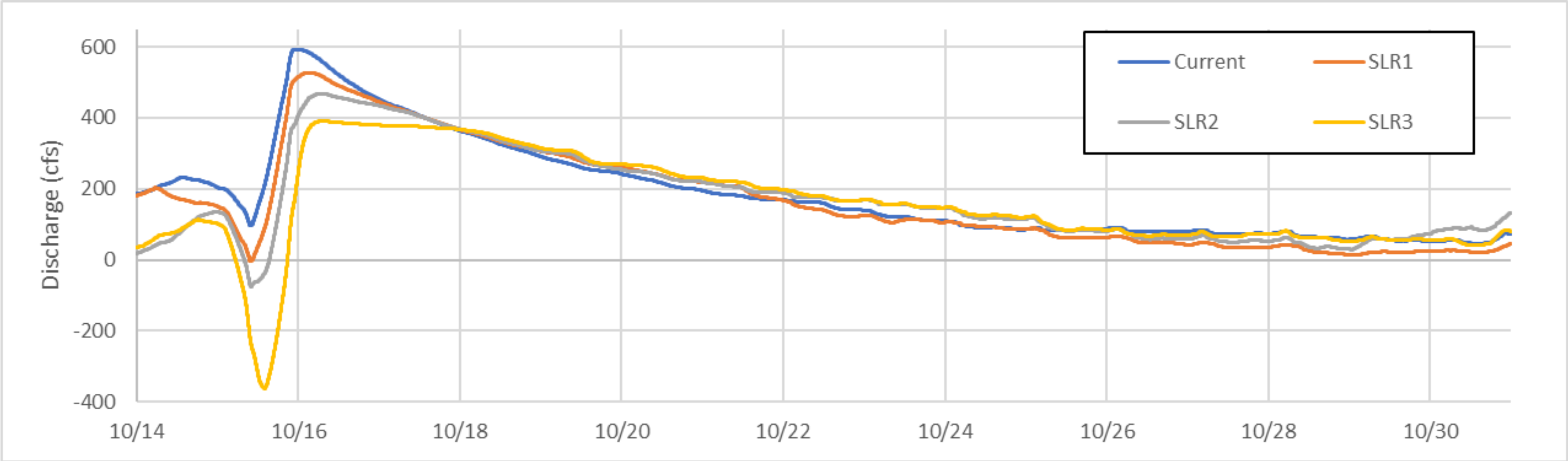


Figure 9-96. 12-Hour Moving Average of Flows at G93 for the 25-year 3-day Design Storm

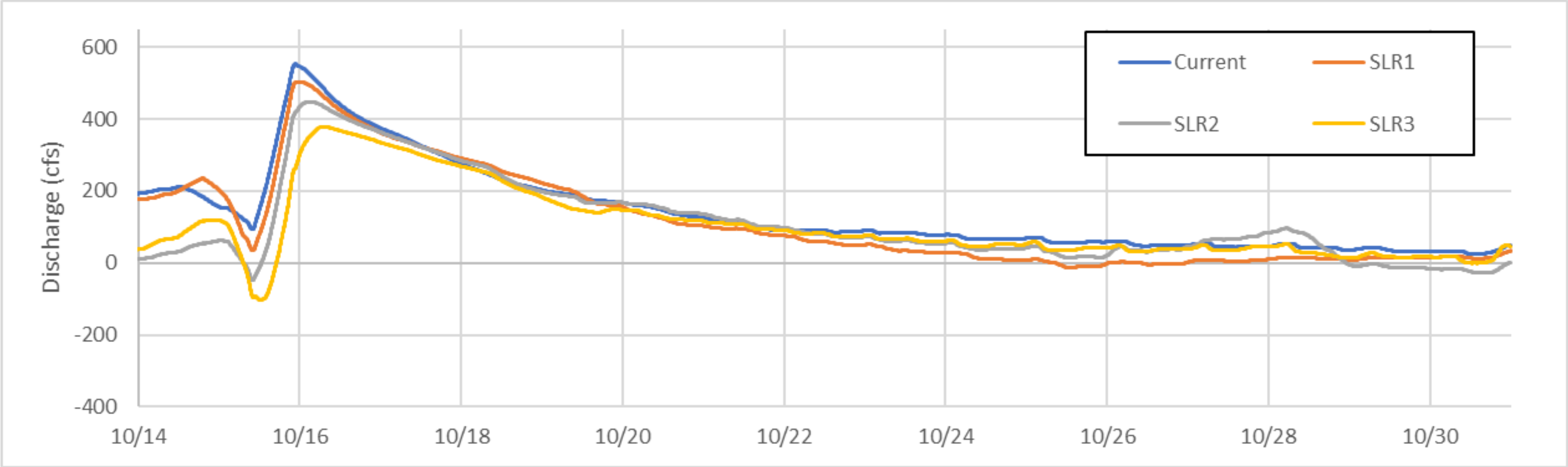


Figure 9-97. 12-Hour Moving Average of Flows at G93 for the 10-year 3-day Design Storm

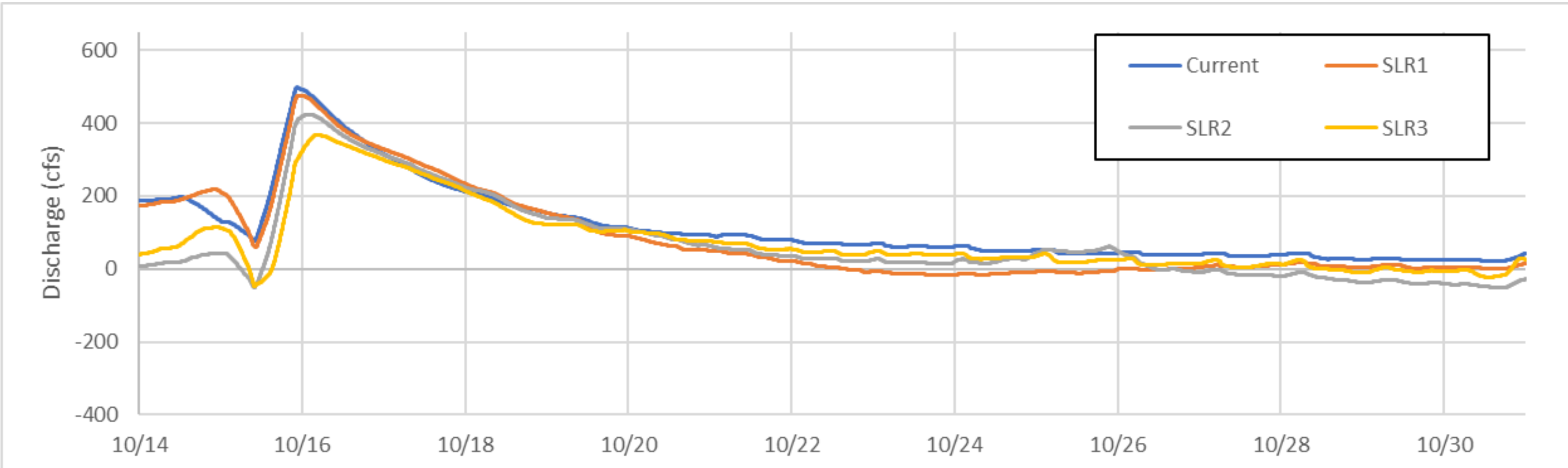
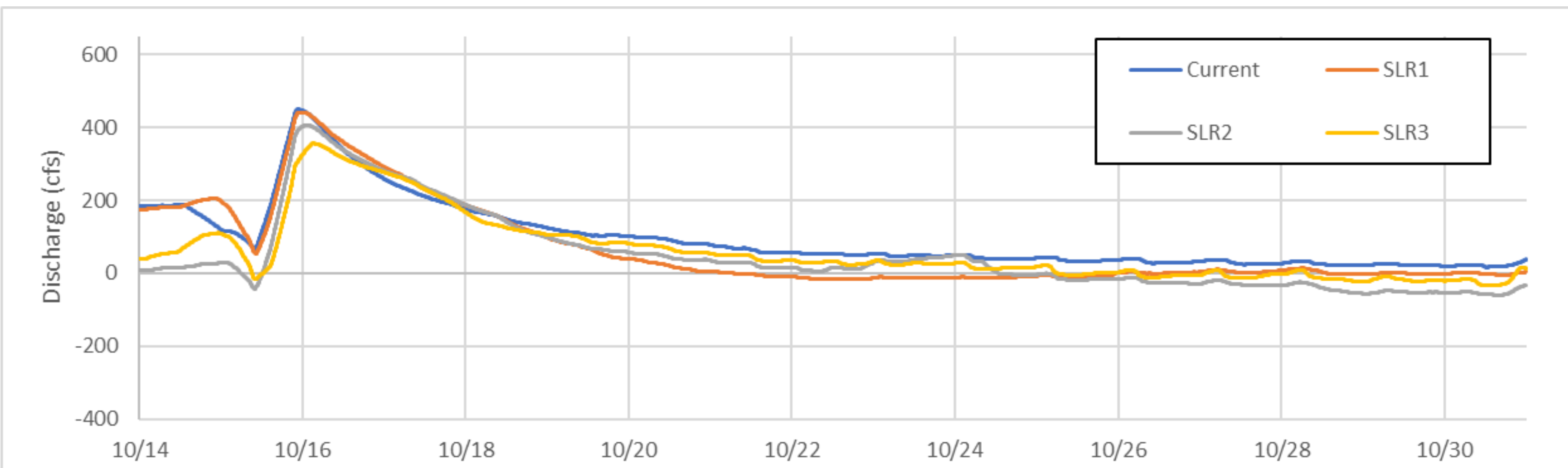


Figure 9-98. 12-Hour Moving Average of Flows at G93 for the 5-year 3-day Design Storm

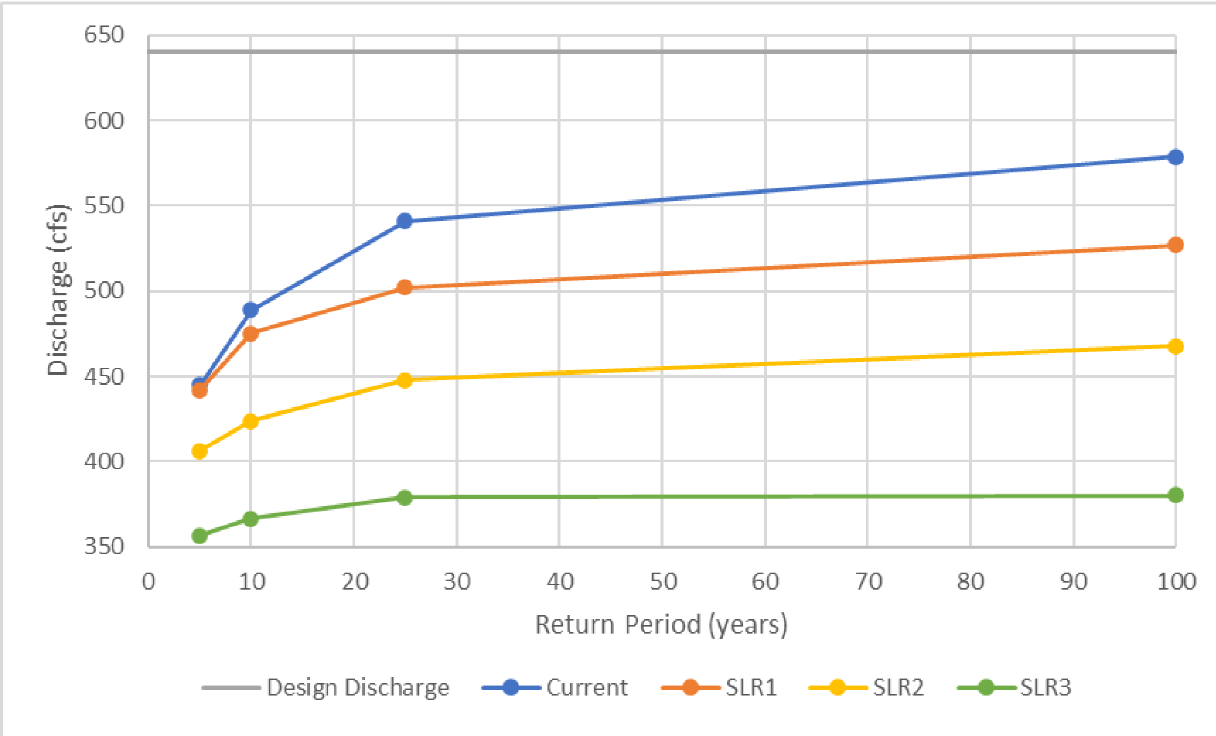


The instantaneous and 12-hour moving average peak discharges for all the design storm event return periods and SLR scenarios are shown in **Table 9-32**. In addition, the percentage difference between the current conditions 12-hour moving average peak discharge and each future conditions SLR scenario is calculated for all simulations. The 12-hour moving average peak discharges are also shown in **Figure 9-99**. The peak 12-hour moving average discharge from the C3W basin falls below the design value for all current and future condition simulations and decreases with increasing SLR for each design storm return period.

Table 9-32. Peak Discharge Summary at G93

Design Storm Return Period	Scenario	Peak Discharge (CFS)		12-Hour Moving Average Peak Flow Reduction Percentage
		Instantaneous	12-Hour Moving Average	
100-Year	Current	616.29	578.71	N/A
	SLR1	538.85	526.96	8.94%
	SLR2	486.77	467.72	19.18%
	SLR3	413.85	380.24	34.30%
25-Year	Current	602.61	541.18	N/A
	SLR1	528.99	502.20	7.20%
	SLR2	457.22	448.01	17.22%
	SLR3	403.56	378.84	30.00%
10-Year	Current	552.67	488.83	N/A
	SLR1	502.56	475.09	2.81%
	SLR2	436.94	423.88	13.29%
	SLR3	385.92	366.77	24.97%
5-Year	Current	515.05	444.77	N/A
	SLR1	469.80	441.81	0.67%
	SLR2	424.07	406.15	8.68%
	SLR3	367.68	356.73	19.80%

Figure 9-99. Peak 12-Hour Moving Discharge at G93



9.2.5. C3W – PM #5 FREQUENCY OF FLOODING

The maximum overland depth was extracted for each design storm event and SLR condition and evaluated for the C3W Watershed. **Table 9-33** tabulates the flood inundation area in square miles for the C3W Watershed. The total area of the C3W Watershed for this analysis was calculated as 5.8 square miles (slight variations in total area from the District total area are due to estimating basin shape along a coarse grid).

The total area of the C3W Watershed considered Urban is 5.1 square miles which makes this watershed mostly urban (about 88% urban). **Table 9-34** tabulates the flood inundation area in square miles for the urban areas within the C3W Watershed. This table shows that the greatest distribution of flooding depths in the watershed are between zero and 0.25 feet of flooding, which can be considered nuisance flooding. **Figure 9-100** shows the urban inundation with the same incremental flooding depths as the table for the 100-year storm event, and **Figure 9-101** does the same for the 25-year storm. This provides a clear view of how stages are increasing with sea level rise in this watershed. The graphics show how the area inundated with more than 2 feet of water tapers off to very small areas. In addition, the SLR3 100-year storm shows a lower area of flooding at the

0.25 feet to 0.5 feet range than the other SLR conditions, which is due to more of this area being inundated at a higher depth than the other SLR conditions.

Table 9-35 shows the percentage of the total urban areas in the C3W Watershed that is above the flooding depth. This data shows that 65% of the total urban areas in the C3W Watershed has greater than 3 inches of flooding depth (0.25 feet) for the 100-year storm for current conditions, this increases steadily for each SLR simulation by about 2% for each foot of SLR. About 35% of the total urban area has greater than 6 inches of flooding depth (0.5 feet) for the 100-year storm for current conditions, which increases by 2-3% for each foot of SLR.

Figure 9-102 and **Figure 9-103** are maps of the maximum overland depth over the entire C3W Watershed for the 25-year and 100-year 3-day design storm events, respectively. Water areas, such as existing lakes and ponds, are masked in black. The C3W watershed is primarily urban, so duplicate maps were not created with non-urban areas masked out.

Figure 9-104, **Figure 9-105**, and **Figure 9-106** show the difference in overland flooding for the C3W Watershed between the current conditions and future sea level rise conditions for the SLR +1 foot, SLR +2 feet, and SLR +3 feet simulations, respectively. Because the rainfall is the same in all the 100-year storm event simulations, these difference maps remove any overland flooding caused by rainfall and show how much is now impacted by rising seas in terms of direct flooding from the canals, or reduced drainage capacity due to higher stages in the primary canal. Where higher canal stages are due to reduced discharge capacity at the structure as well as structure backflow due to overtopping and structure bypass, as discussed in the previous sections. Negative values indicate areas that were elevated in the model topography due to future land development. Areas near the G93 structure along the canal experience the highest increase in flooding. In fact, the increase in flooding seems to start at the canal (for the SLR1 difference) and spread out farther from the canal for the SLR3 difference.

Maps of the maximum overland depth over the entire C3W Watershed for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current conditions and future SLR scenarios are provided in **Appendix C**. Also provided in **Appendix C** are the differences in overland flooding maps for the 25-year, 10-year, and 5-year design storms.

Table 9-33. Incremental Flood Inundation Area (sq. mi.) in the C3W Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	2.11	3.64	4.30	4.73	1.95	3.48	4.17	4.56	1.82	3.30	4.01	4.39	1.68	3.09	3.82	4.20
0.25 =< Depth < 0.50	1.63	0.84	0.60	0.48	1.68	0.90	0.63	0.48	1.66	0.96	0.69	0.54	1.56	0.97	0.71	0.60
0.50 =< Depth < 0.75	0.67	0.50	0.38	0.21	0.69	0.55	0.40	0.34	0.69	0.58	0.39	0.39	0.70	0.62	0.47	0.39
0.75 =< Depth < 1.00	0.51	0.32	0.19	0.13	0.53	0.33	0.24	0.14	0.58	0.36	0.29	0.16	0.64	0.40	0.31	0.22
1.00 =< Depth < 1.25	0.32	0.16	0.09	0.04	0.35	0.18	0.09	0.07	0.38	0.19	0.12	0.08	0.42	0.26	0.13	0.09
1.25 =< Depth < 1.50	0.16	0.07	0.05	0.02	0.17	0.08	0.06	0.03	0.22	0.10	0.07	0.05	0.27	0.11	0.07	0.06
1.50 =< Depth < 1.75	0.08	0.06	0.02	0.02	0.11	0.05	0.03	0.02	0.09	0.07	0.04	0.02	0.15	0.07	0.06	0.04
1.75 =< Depth < 2.00	0.07	0.03	0.02	0.00	0.06	0.04	0.02	0.00	0.08	0.04	0.03	0.01	0.06	0.06	0.04	0.02
2.00 =< Depth < 2.25	0.04	0.01	0.00	0.01	0.04	0.02	0.01	0.01	0.05	0.03	0.00	0.00	0.06	0.03	0.02	0.01
2.25 =< Depth < 2.50	0.03	0.00	0.01	0.01	0.02	0.01	0.00	0.01	0.03	0.02	0.01	0.00	0.05	0.01	0.01	0.01
2.50 =< Depth < 2.75	0.01	0.00	0.00	0.01	0.01	0.00	0.01	0.00	0.01	0.00	0.00	0.01	0.01	0.02	0.00	0.00
2.75 =< Depth < 3.00	0.00	0.00	0.01	0.00	0.01	0.00	0.00	0.00	0.02	0.00	0.01	0.00	0.01	0.01	0.00	0.00
3.00 =< Depth	0.14	0.13	0.11	0.10	0.14	0.12	0.11	0.11	0.14	0.13	0.12	0.11	0.17	0.13	0.13	0.12

Total Basin Area = 5.8 square miles

Table 9-34. Incremental Flood Inundation Area (sq. mi.) for Urban Areas in the C3W Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	1.80	3.18	3.76	4.15	1.67	3.01	3.63	3.97	1.57	2.86	3.49	3.83	1.45	2.67	3.32	3.66
0.25 =< Depth < 0.50	1.50	0.77	0.56	0.45	1.50	0.83	0.59	0.46	1.48	0.88	0.64	0.52	1.40	0.89	0.65	0.57
0.50 =< Depth < 0.75	0.60	0.46	0.35	0.21	0.62	0.52	0.39	0.33	0.62	0.54	0.38	0.36	0.61	0.58	0.46	0.37
0.75 =< Depth < 1.00	0.46	0.30	0.18	0.13	0.50	0.32	0.21	0.13	0.53	0.35	0.27	0.15	0.59	0.38	0.30	0.21
1.00 =< Depth < 1.25	0.30	0.14	0.08	0.04	0.32	0.17	0.09	0.07	0.37	0.18	0.10	0.07	0.39	0.25	0.13	0.08
1.25 =< Depth < 1.50	0.15	0.07	0.04	0.02	0.17	0.07	0.05	0.02	0.20	0.09	0.06	0.04	0.26	0.10	0.06	0.06
1.50 =< Depth < 1.75	0.08	0.04	0.02	0.02	0.10	0.04	0.03	0.02	0.09	0.05	0.03	0.02	0.14	0.06	0.05	0.03
1.75 =< Depth < 2.00	0.05	0.03	0.01	0.00	0.05	0.03	0.01	0.00	0.07	0.03	0.02	0.01	0.06	0.04	0.03	0.02
2.00 =< Depth < 2.25	0.03	0.01	0.00	0.00	0.03	0.02	0.01	0.00	0.04	0.02	0.00	0.00	0.05	0.03	0.00	0.01
2.25 =< Depth < 2.50	0.02	0.00	0.00	0.00	0.02	0.01	0.00	0.00	0.02	0.01	0.00	0.00	0.03	0.00	0.01	0.00
2.50 =< Depth < 2.75	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.01	0.01	0.00	0.00
2.75 =< Depth < 3.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.00 =< Depth	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.03	0.01	0.01	0.01

Total Basin Urban Area = 5.0 square miles

Table 9-35. Percentage of Total Watershed Area with Inundated Area for Urban Areas in the C3W Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0.00	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 0.25	64.1	36.5	25.1	17.3	66.6	40.1	27.7	20.7	68.8	43.0	30.4	23.6	71.1	46.8	33.7	27.0
>= 0.50	34.1	21.2	13.9	8.4	36.7	23.6	15.8	11.5	39.2	25.5	17.7	13.3	43.2	29.1	20.8	15.6
>= 0.75	22.1	12.0	6.9	4.2	24.2	13.3	8.1	5.1	26.9	14.7	10.1	6.1	31.0	17.6	11.7	8.1
>= 1.00	12.9	6.0	3.3	1.7	14.2	6.9	3.9	2.5	16.2	7.8	4.6	3.1	19.1	10.1	5.7	3.9
>= 1.25	6.8	3.2	1.7	1.0	7.8	3.6	2.2	1.2	8.9	4.3	2.6	1.7	11.4	5.1	3.2	2.4
>= 1.50	3.9	1.9	0.9	0.5	4.5	2.2	1.2	0.7	4.9	2.5	1.4	0.9	6.3	3.1	2.1	1.3
>= 1.75	2.4	1.0	0.5	0.2	2.5	1.3	0.6	0.3	3.2	1.4	0.8	0.4	3.6	2.0	1.1	0.8
>= 2.00	1.4	0.4	0.2	0.2	1.5	0.7	0.4	0.2	1.9	0.8	0.4	0.3	2.5	1.1	0.5	0.4
>= 2.25	0.8	0.3	0.2	0.2	0.8	0.4	0.2	0.2	1.1	0.4	0.3	0.2	1.4	0.5	0.4	0.3
>= 2.50	0.4	0.2	0.2	0.2	0.4	0.2	0.2	0.2	0.6	0.3	0.2	0.2	0.8	0.5	0.3	0.2
>= 2.75	0.3	0.2	0.2	0.2	0.4	0.2	0.2	0.2	0.5	0.2	0.2	0.2	0.6	0.3	0.2	0.2
>= 3.00	0.3	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.6	0.2	0.2	0.2

Figure 9-100. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C3W Watershed for the 100-year Design Storm

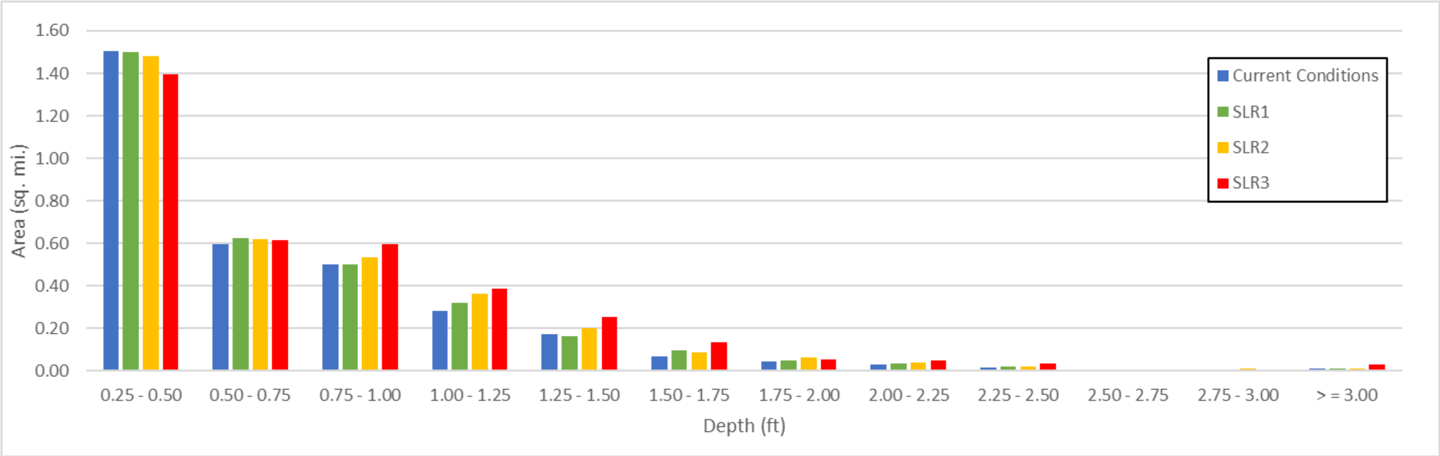


Figure 9-101. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C3W Watershed for the 25-year Design Storm

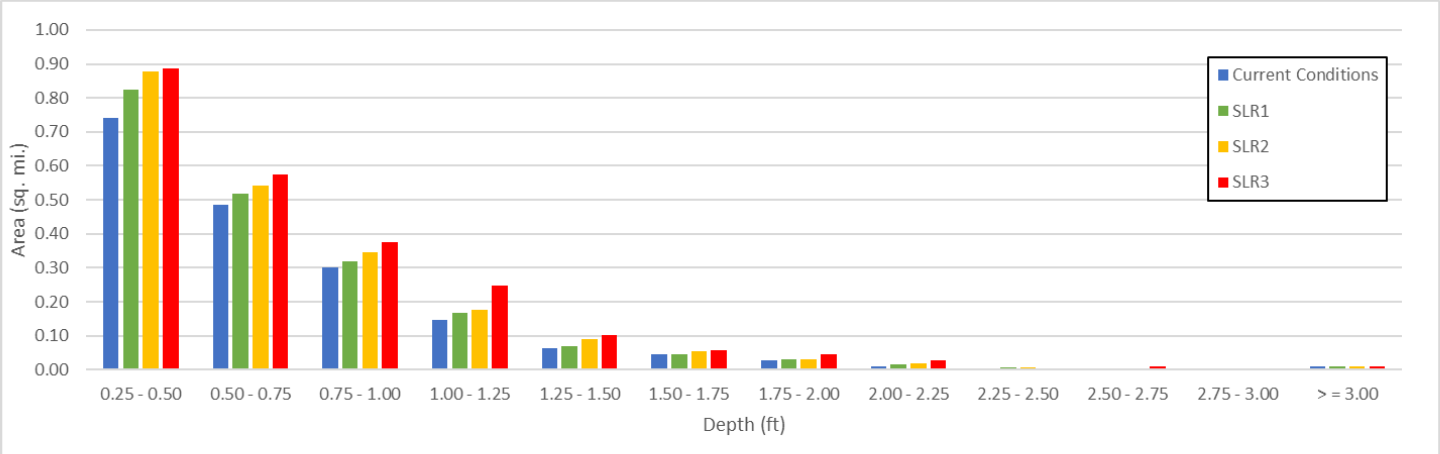


Figure 9-102. Maximum Overland Flood Depth for the Current Conditions 25-year 3-Day Design Storm in the C3W Watershed

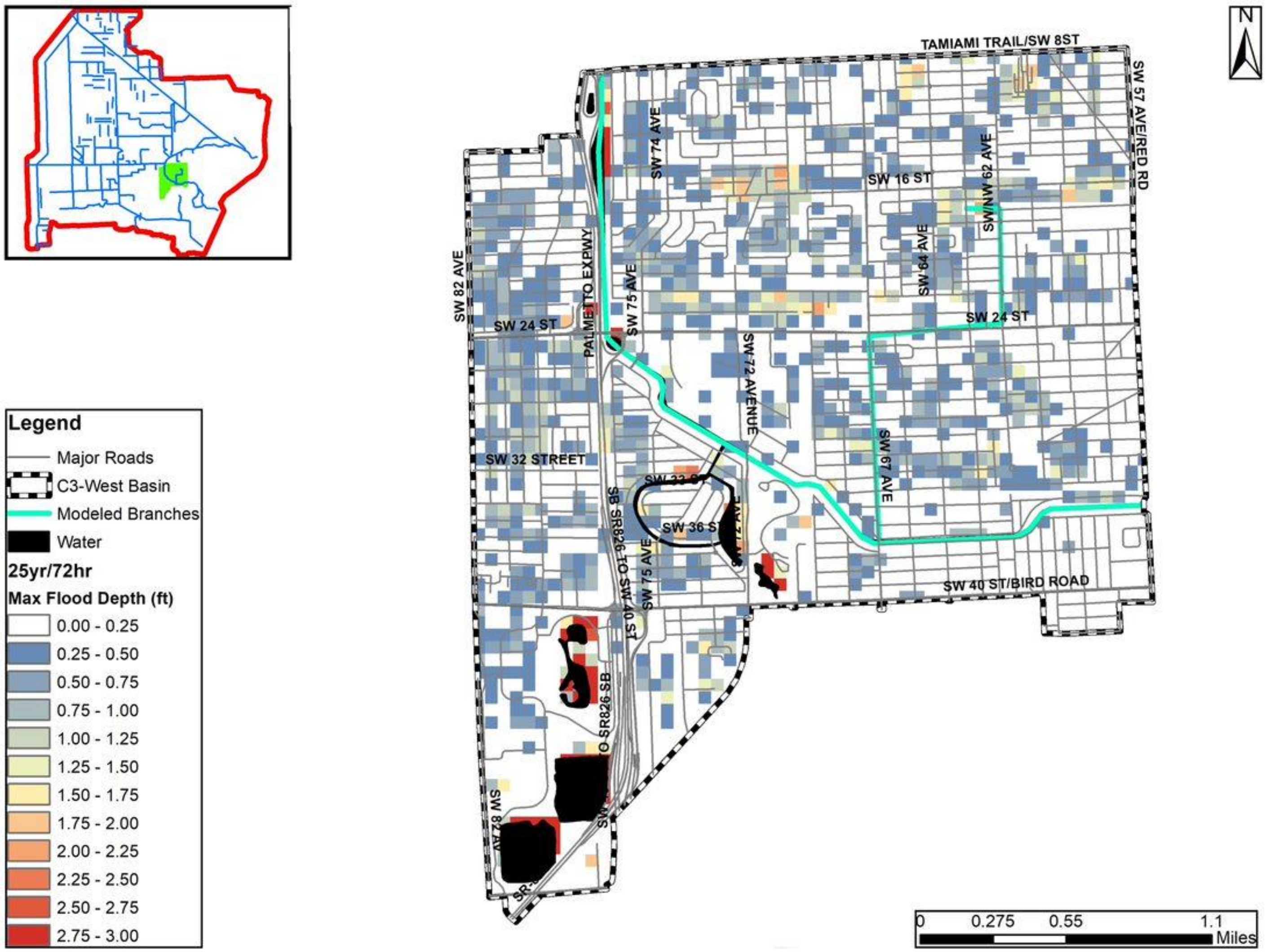


Figure 9-103. Maximum Overland Flood Depth for the Current Conditions 100-year 3-Day Design Storm in the C3W Watershed

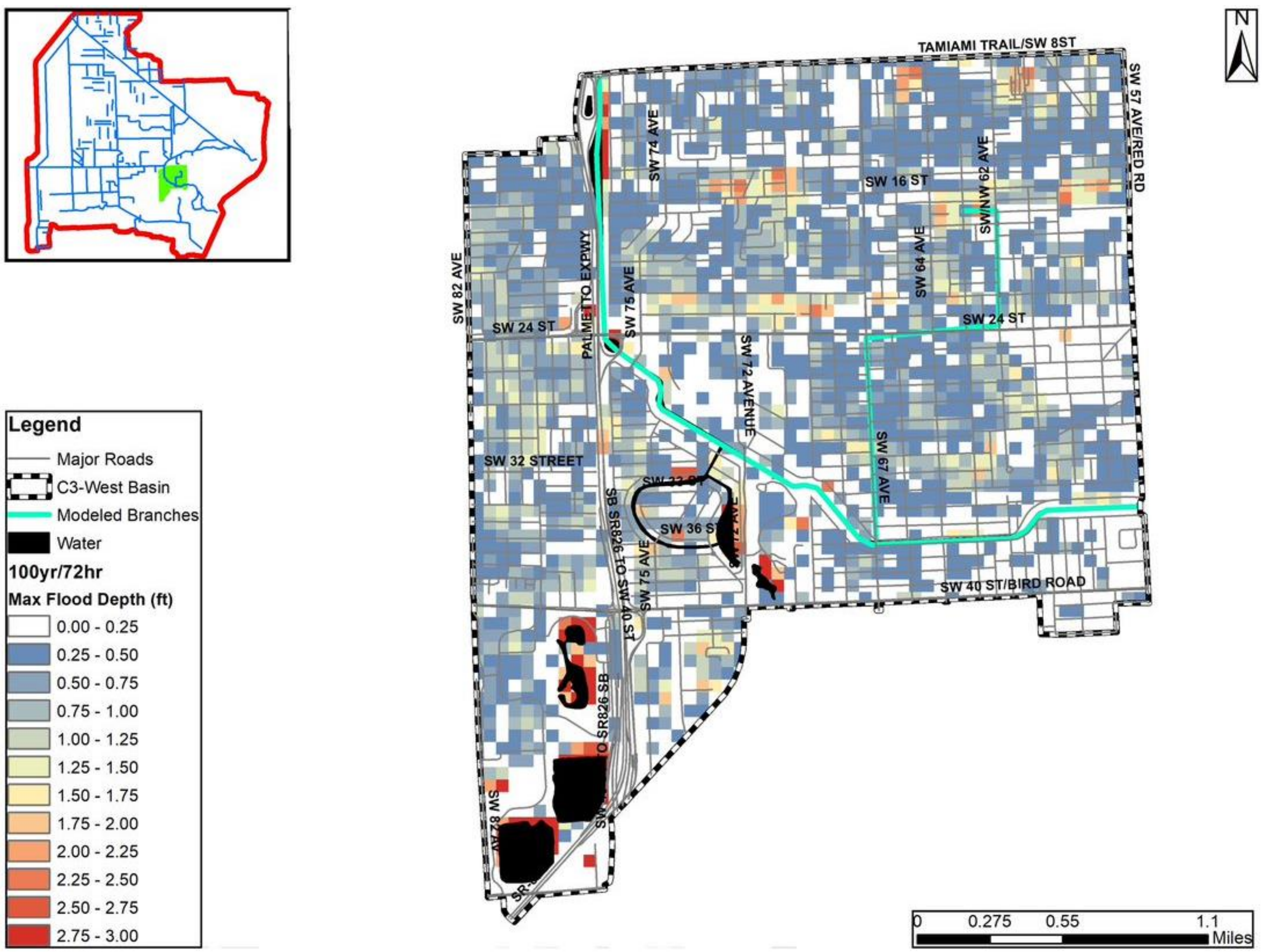


Figure 9-104. Urban Flooding Depth Difference of SLR +1ft and Current Conditions. for the 100-year Storm in the C3W Watershed

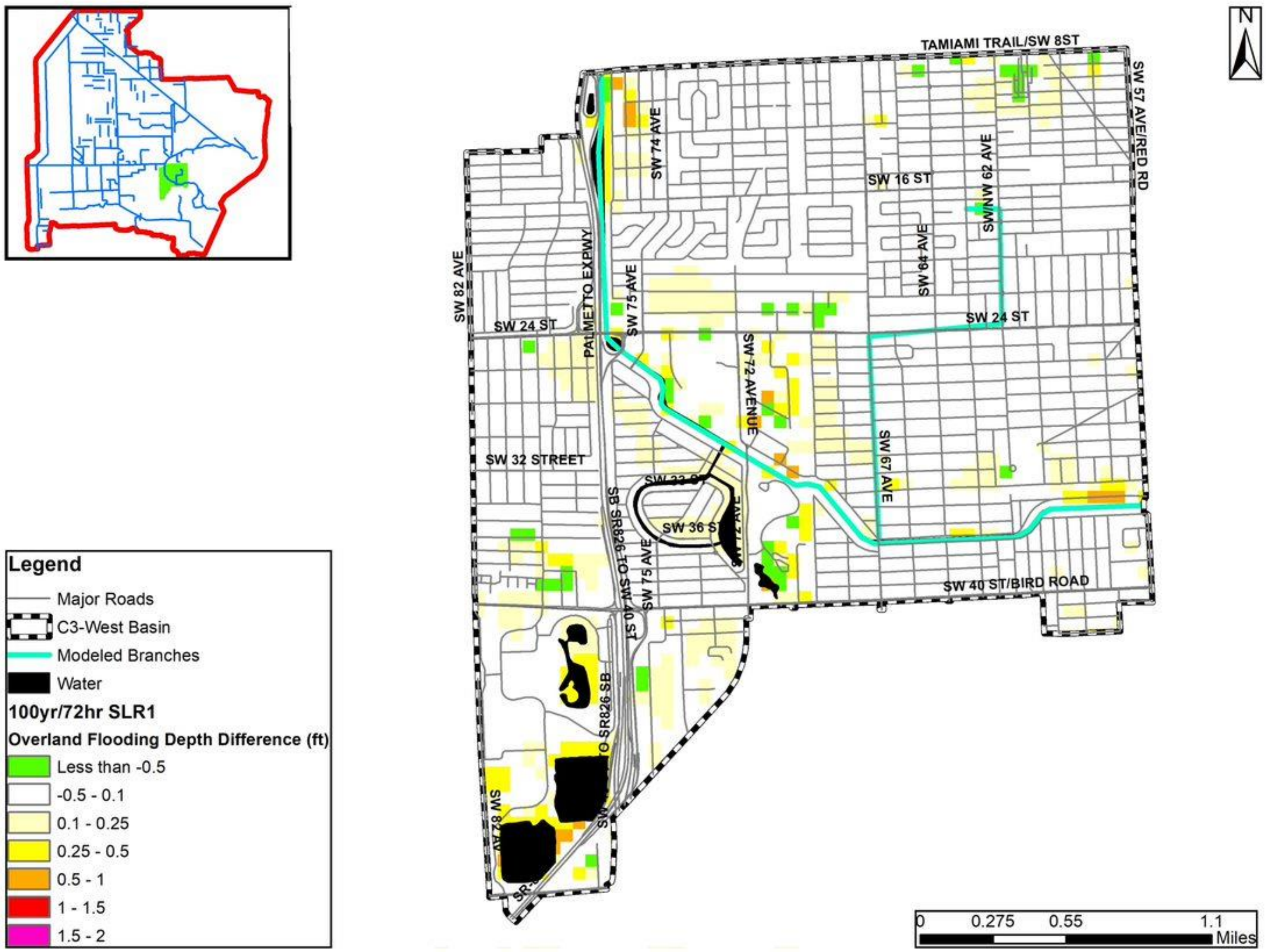


Figure 9-105. Urban Flooding Depth Difference of SLR +2ft and Current Conditions for the 100-year Storm in the C3W Watershed

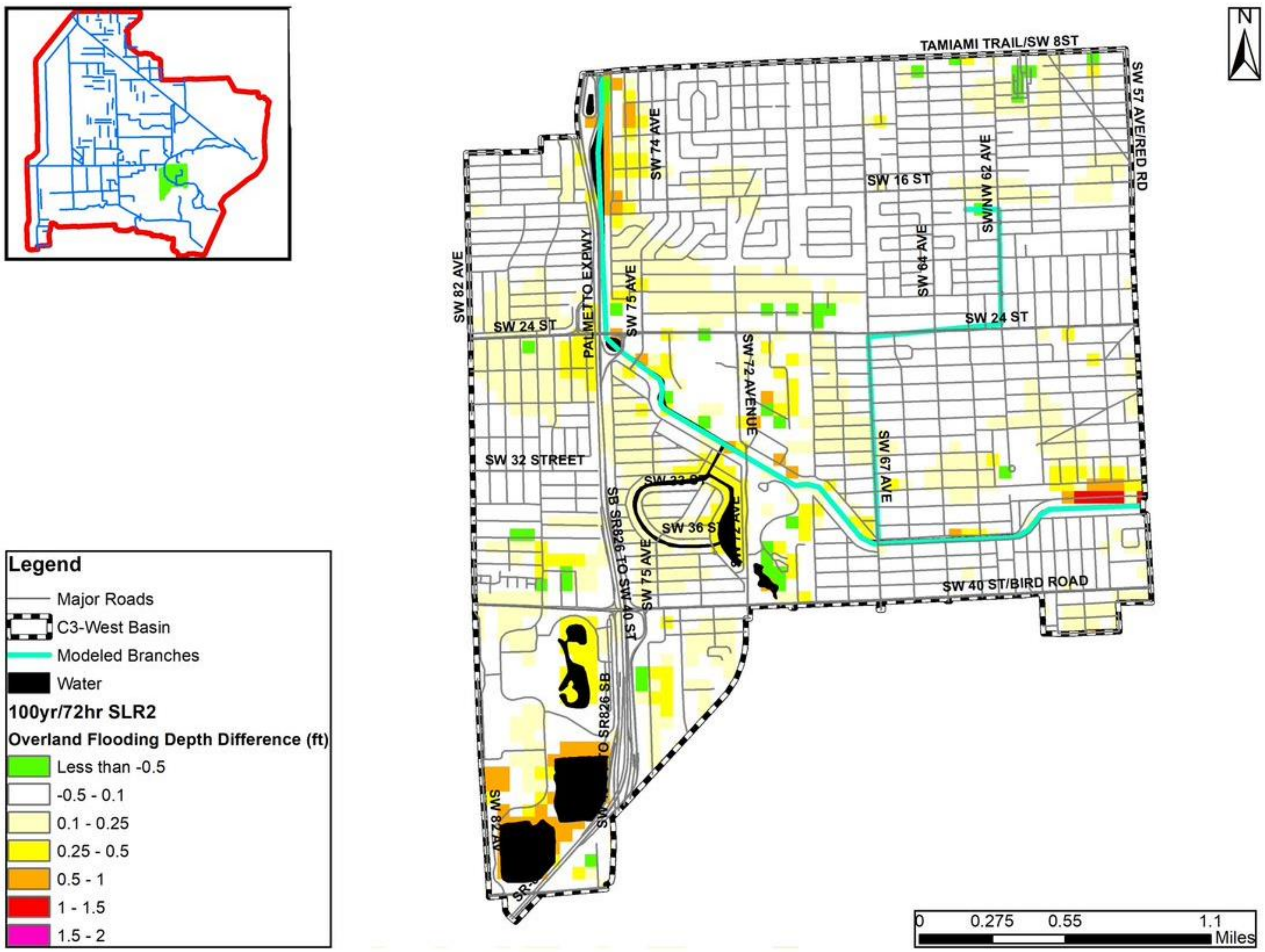
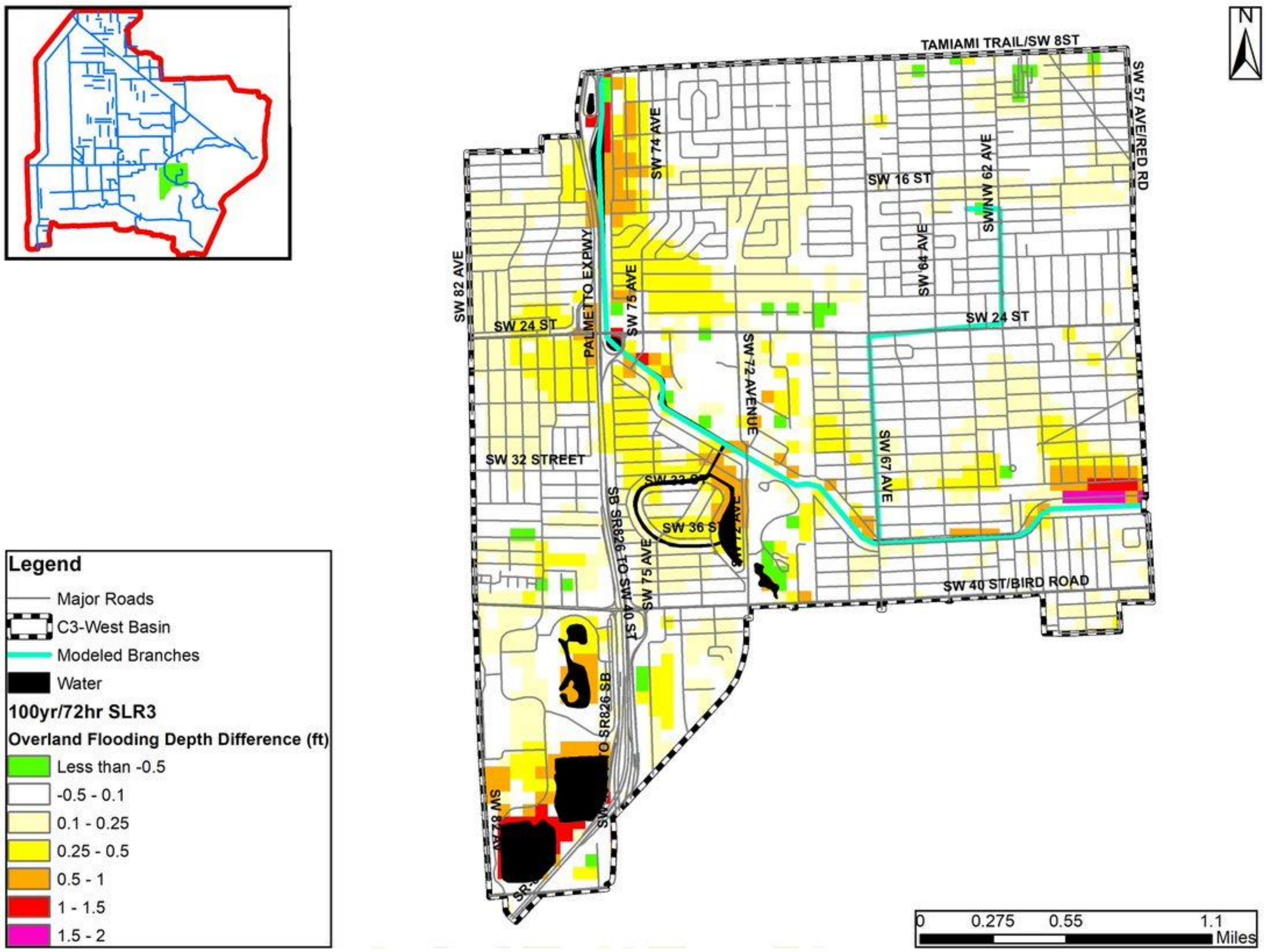


Figure 9-106. Urban Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C3W Watershed



9.2.6. C3W – PM #6 DURATION OF FLOODING

9.2.6.1. CANAL FLOOD DURATION

As discussed in PM #6 of the C2 Watershed, the T5W station was evaluated to determine the duration of flooding for the C2, C3W and C4 Watersheds. **Table 9-36** shows the canal flood duration for each storm event and SLR condition at T5W. For all storm events, under the SLR3 conditions, canal stages do not recede past the Reference Elevation after the storm and therefore the storm duration is longer than the values provided.

Table 9-36. Storm Duration Indicated at T5W

Design Storm	Duration (hours)				
	Current Conditions with 2.23 ft-NAVD Reference Elev.	Current Conditions with 3.43 ft-NAVD Reference Elev.	SLR1	SLR2	SLR3
100-Year	281.8	119.6	162.1	282.6	420.6*
25-Year	184.3	67.6	108.2	217.7	410.4*
10-Year	140.3	40.8	68.6	149.8	408.3*
5-Year	101.6	24.9	47.1	120.1	398.1*

*Canal stages do not recede past the Reference Elevation after the storm and therefore the storm duration is longer than the values provided.

Figure 9-107 shows the HW stages at G93 comparing the 100-year storm results for each SLR condition, and the vertical lines indicate the start and end of each design storm as determined at T5W. **Figure 9-108** shows the same, but for the 25-year storm.

9.2.6.2. WATERSHED FLOOD DURATION

Table 9-37 tabulates the total area of flood inundation (in square miles) per flood duration range for all areas in the C3W Watershed and **Table 9-38** does this for all urban areas within the C3W Watershed. This table shows that the greatest distribution of flooding duration in the watershed is between zero and one hour of flooding, which can be considered insignificant. Flooding is also distributed in the 12 to 72-hour range, indicating that most of the watershed has a one to three-day journey to a major canal, as represented in the model.

Figure 9-107. T5W Flood Duration Compared with HW at G93 for the 100-year Storm

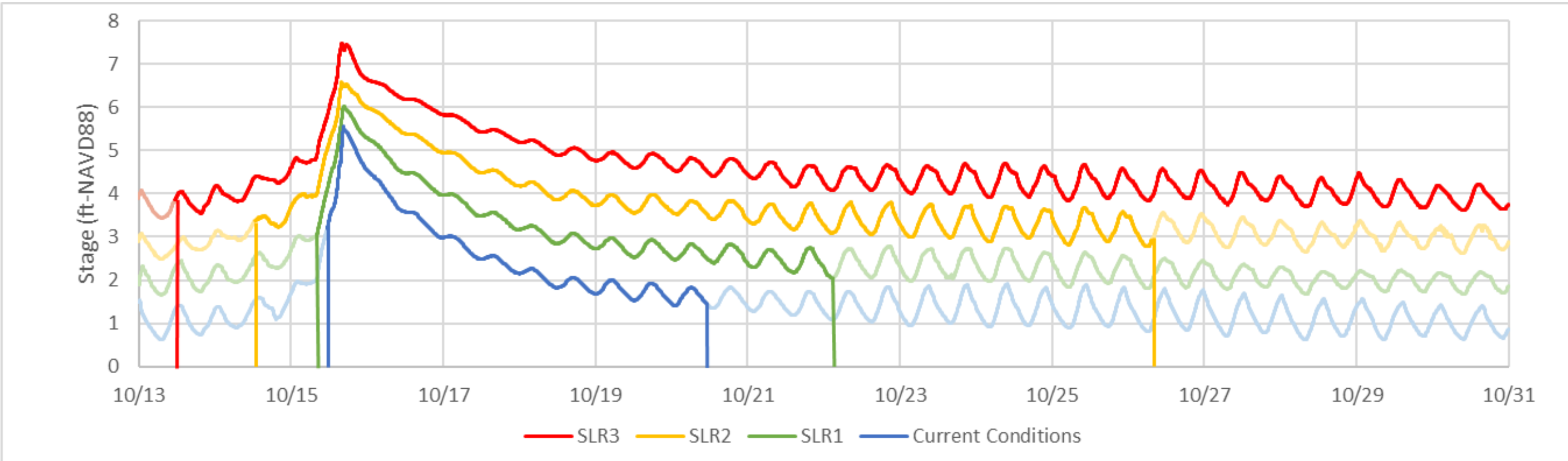


Figure 9-108. T5W Flood Duration Compared with HW at G93 for the 25-year Storm

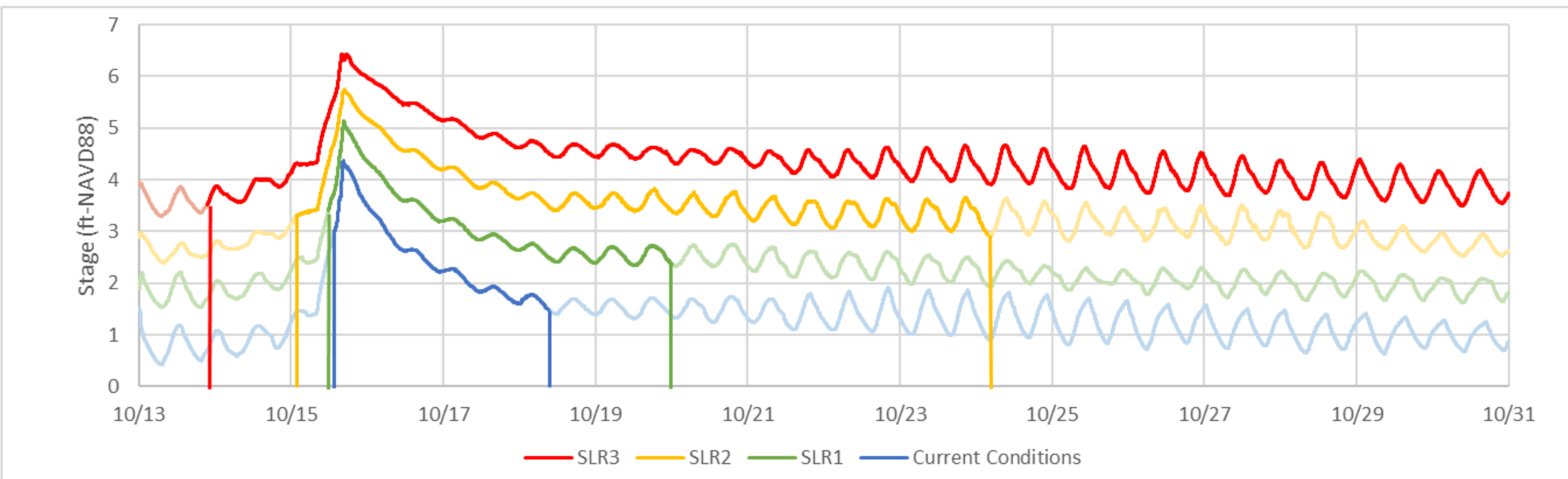


Table 9-39 calculates the percentage of the total urban areas within the C3W Watershed that are inundated by 3 inches or more for the flood duration indicated. Because the flood duration greater than 1 hour includes all areas inundated with 3 inches or more, this row shows the same percentage as that in **Table 9-35** in PM #5. Additionally, **Figure 9-109, Figure 9-110, Figure 9-111**, and **Figure 9-112** provide a graphical view of this data, plotting the flood duration against the percentage of area inundated for the urban areas of the C3W Watershed for the 100-year, 25-year, 10-year, and 5-year storm events, respectively. These graphics also include the percent increase above the current conditions on the secondary axis, to visualize how the flood duration is changing with each SLR condition. For the 100-year design storms, the percentage of the watershed that is flooded for 72-to-96-hours shows the greatest increase above current conditions with each SLR condition. The greatest increase is shown at the 48-hour mark for the 25-year storm, between the 24-to-48-hour mark for the 10-year storm, and the 24-hour mark for the 5-year storm. This indicates a greater impact from both initial rainfall/runoff (due to higher initial conditions) and from reduced drainage capacity of the watershed as a result of higher canal elevations.

Figure 9-113 and **Figure 9-114** provide flood duration maps for the C3W Watershed for overland flooding depths exceeding 0.25 feet for the current conditions 5-year and 100-year 3-day design storms, respectively. Water areas, such as existing lakes and ponds, are masked in black. The C3W watershed is primarily urban, so duplicate maps were not created with non-urban areas masked out.

Figure 9-115, **Figure 9-116**, and **Figure 9-117** provide the difference in flood duration between the Current Conditions and SLR +1 foot, SLR, +2 feet, and SLR +3 feet, respectively, for the 100-year storm event. When compared the difference maps for flood depth, this can show that even for areas where flood depths are not increasing with future SLR conditions, the duration of flooding may increase due to the reduced ability of the area to drain to the receiving canals that are experiencing higher stages.

Flood duration maps over the entire C3W Watershed for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current conditions and future SLR scenarios are provided in **Appendix D**. Also provided in **Appendix D** are the differences in flood duration maps for the 25-year, 10-year, and 5-year design storms.

Table 9-37. Flood Duration per Area of Inundation (in sq. mi.) for the C3W Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	2.11	3.64	4.30	4.73	1.95	3.48	4.17	4.56	1.82	3.30	4.01	4.39	1.68	3.09	3.82	4.20
1 to 2 hr.	0.61	0.12	0.04	0.02	0.61	0.11	0.04	0.03	0.56	0.13	0.05	0.04	0.55	0.12	0.04	0.05
2 to 4 hr.	0.26	0.14	0.09	0.07	0.27	0.15	0.08	0.07	0.27	0.15	0.09	0.07	0.25	0.17	0.09	0.05
4 to 8 hr.	0.23	0.14	0.15	0.10	0.23	0.17	0.11	0.10	0.18	0.17	0.12	0.11	0.21	0.17	0.14	0.13
8 to 12 hr.	0.15	0.13	0.09	0.11	0.14	0.13	0.13	0.09	0.16	0.13	0.12	0.08	0.11	0.13	0.10	0.07
12 to 24 hr.	0.55	0.42	0.40	0.29	0.52	0.41	0.35	0.33	0.53	0.37	0.33	0.34	0.50	0.39	0.33	0.26
24 to 48 hr.	0.60	0.58	0.39	0.26	0.61	0.55	0.47	0.33	0.59	0.54	0.48	0.38	0.56	0.46	0.41	0.41
48 to 72 hr.	0.48	0.27	0.16	0.06	0.45	0.33	0.17	0.12	0.41	0.36	0.24	0.12	0.42	0.37	0.29	0.20
72 to 96 hr.	0.29	0.14	0.03	0.01	0.33	0.15	0.09	0.02	0.33	0.18	0.09	0.07	0.26	0.22	0.14	0.09
96 to 144 hr.	0.25	0.04	0.01	0.01	0.31	0.13	0.02	0.01	0.35	0.18	0.10	0.03	0.39	0.22	0.15	0.10
144 to 192 hr.	0.07	0.01	0.01	0.01	0.14	0.01	0.01	0.01	0.21	0.09	0.00	0.01	0.25	0.15	0.07	0.04
192 to 240 hr.	0.01	0.01	0.01	0.00	0.06	0.00	0.01	0.00	0.11	0.01	0.01	0.00	0.16	0.07	0.03	0.01
240 to 336 hr.	0.02	0.03	0.01	0.01	0.01	0.02	0.01	0.00	0.09	0.02	0.02	0.01	0.19	0.06	0.02	0.03
336 to 420 hr.	0.08	0.06	0.05	0.05	0.04	0.02	0.02	0.02	0.04	0.02	0.01	0.01	0.11	0.04	0.03	0.01
420 hr.	0.05	0.04	0.04	0.04	0.10	0.10	0.09	0.09	0.11	0.11	0.11	0.11	0.12	0.12	0.11	0.12

Total Basin Area = 5.8 square miles

Table 9-38. Flood Duration per Area of Inundation (in sq. mi.) for Urban Areas in the C3W Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	1.80	3.18	3.76	4.15	1.67	3.01	3.63	3.97	1.57	2.86	3.49	3.83	1.45	2.67	3.32	3.66
1 to 2 hr.	0.57	0.10	0.04	0.02	0.54	0.10	0.04	0.03	0.51	0.12	0.04	0.04	0.50	0.11	0.03	0.05
2 to 4 hr.	0.24	0.12	0.08	0.06	0.24	0.13	0.07	0.06	0.25	0.14	0.08	0.07	0.22	0.15	0.07	0.05
4 to 8 hr.	0.20	0.12	0.14	0.09	0.20	0.15	0.10	0.09	0.15	0.15	0.10	0.11	0.18	0.15	0.13	0.12
8 to 12 hr.	0.12	0.12	0.07	0.11	0.13	0.11	0.12	0.08	0.15	0.12	0.11	0.07	0.11	0.11	0.09	0.07
12 to 24 hr.	0.48	0.39	0.38	0.28	0.45	0.38	0.33	0.31	0.48	0.34	0.31	0.33	0.45	0.37	0.30	0.25
24 to 48 hr.	0.56	0.55	0.37	0.24	0.57	0.52	0.46	0.31	0.54	0.51	0.46	0.36	0.52	0.44	0.40	0.40
48 to 72 hr.	0.46	0.26	0.14	0.06	0.43	0.33	0.16	0.12	0.39	0.35	0.23	0.12	0.39	0.36	0.28	0.19
72 to 96 hr.	0.28	0.13	0.02	0.00	0.32	0.15	0.09	0.02	0.31	0.17	0.08	0.06	0.25	0.20	0.14	0.07
96 to 144 hr.	0.23	0.03	0.00	0.00	0.28	0.11	0.01	0.00	0.33	0.17	0.09	0.02	0.36	0.21	0.13	0.09
144 to 192 hr.	0.06	0.00	0.00	0.00	0.13	0.00	0.00	0.00	0.18	0.07	0.00	0.00	0.23	0.13	0.06	0.04
192 to 240 hr.	0.00	0.00	0.00	0.00	0.04	0.00	0.00	0.00	0.08	0.01	0.00	0.00	0.13	0.06	0.03	0.00
240 to 336 hr.	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.06	0.00	0.00	0.00	0.15	0.05	0.01	0.01
336 to 420 hr.	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.06	0.01	0.00	0.00
420 hr.	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01

Total Basin Area = 5.0 square miles

Table 9-39. Percentage of Total Area Inundated for Urban Areas in the C3W Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0 hr.	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 1 hr.	64.1	36.5	25.1	17.3	66.6	40.1	27.7	20.7	68.8	43.0	30.4	23.6	71.1	46.8	33.7	27.0
>= 2 hr.	52.7	34.6	24.3	17.0	55.8	38.0	27.0	20.1	58.6	40.7	29.5	22.9	61.2	44.7	33.0	26.1
>= 4 hr.	47.9	32.2	22.8	15.8	50.9	35.4	25.6	18.8	53.6	37.9	27.8	21.6	56.8	41.7	31.6	25.1
>= 8 hr.	44.0	29.7	20.1	14.0	46.8	32.4	23.6	16.9	50.6	34.9	25.8	19.4	53.2	38.8	29.0	22.7
>= 12 hr.	41.6	27.4	18.6	11.8	44.3	30.1	21.2	15.3	47.7	32.5	23.6	18.0	51.1	36.6	27.2	21.2
>= 24 hr.	32.1	19.6	11.0	6.2	35.3	22.4	14.6	9.1	38.2	25.7	17.4	11.4	42.1	29.3	21.1	16.3
>= 48 hr.	20.9	8.6	3.6	1.4	24.0	12.0	5.4	2.9	27.3	15.6	8.3	4.2	31.7	20.6	13.2	8.3
>= 72 hr.	11.6	3.4	0.8	0.3	15.5	5.5	2.2	0.5	19.6	8.6	3.7	1.9	23.9	13.5	7.7	4.5
>= 96 hr.	6.1	0.9	0.3	0.2	9.1	2.5	0.5	0.2	13.4	5.2	2.1	0.7	18.9	9.5	5.0	3.0
>= 144 hr.	1.5	0.3	0.3	0.2	3.5	0.3	0.2	0.2	6.7	1.8	0.3	0.2	11.7	5.2	2.3	1.2
>= 192 hr.	0.3	0.3	0.3	0.2	1.0	0.2	0.2	0.2	3.1	0.4	0.2	0.2	7.1	2.5	1.1	0.4
>= 240 hr.	0.3	0.3	0.2	0.2	0.2	0.2	0.2	0.2	1.5	0.2	0.2	0.2	4.6	1.3	0.5	0.4
>= 336 hr.	0.3	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	1.5	0.4	0.3	0.2
>= 420 hr.	0.0	0.0	0.0	0.0	0.2	0.2	0.2	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2

Figure 9-109. Percentage and Duration of Inundation for the C3W Watershed for the 100-year Storm Event

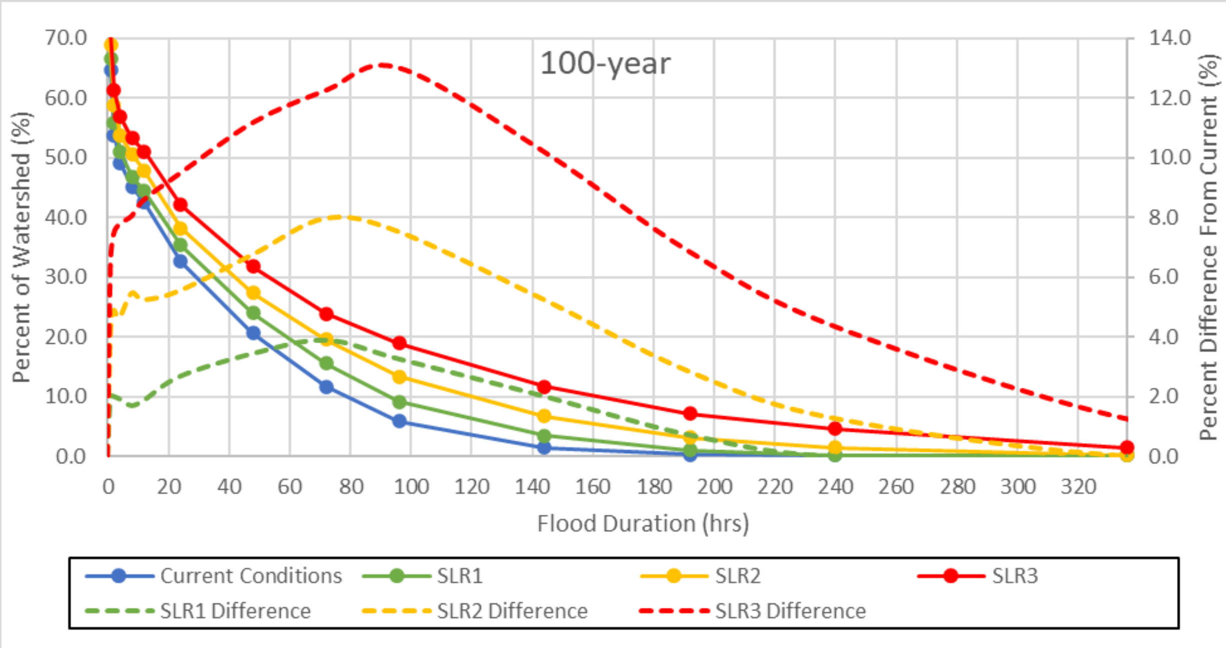


Figure 9-110. Percentage and Duration of Inundation for the C3W Watershed for the 25-year Storm Event

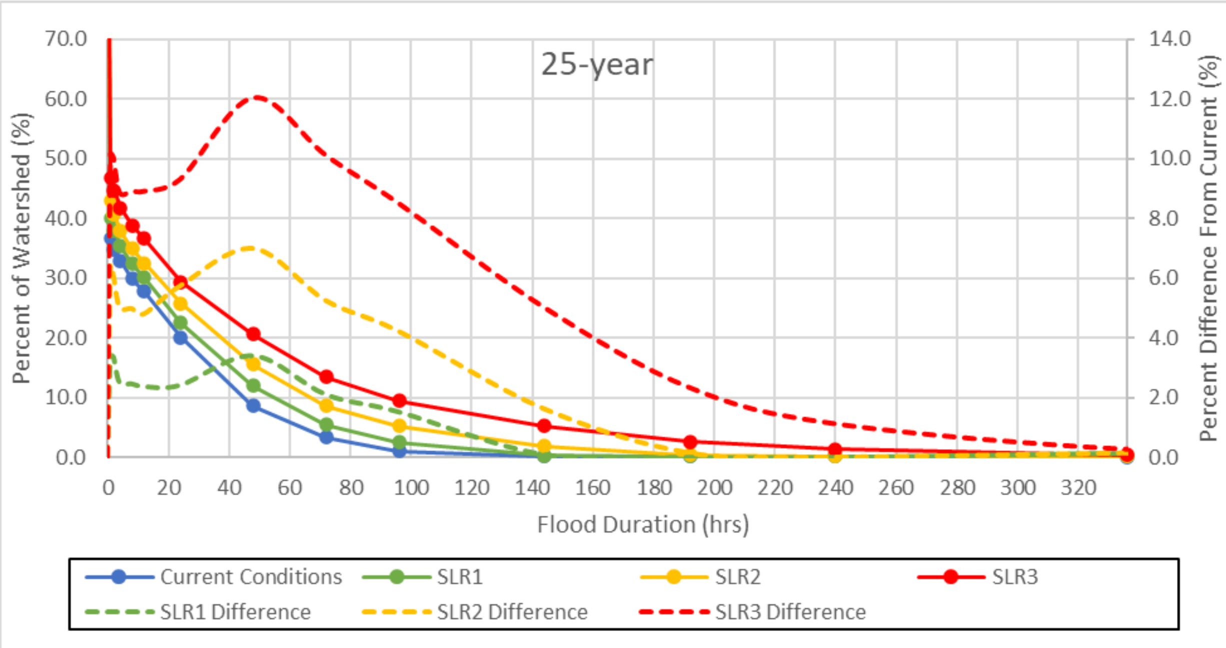


Figure 9-111. Percentage and Duration of Inundation for the C3W Watershed for the 10-year Storm Event

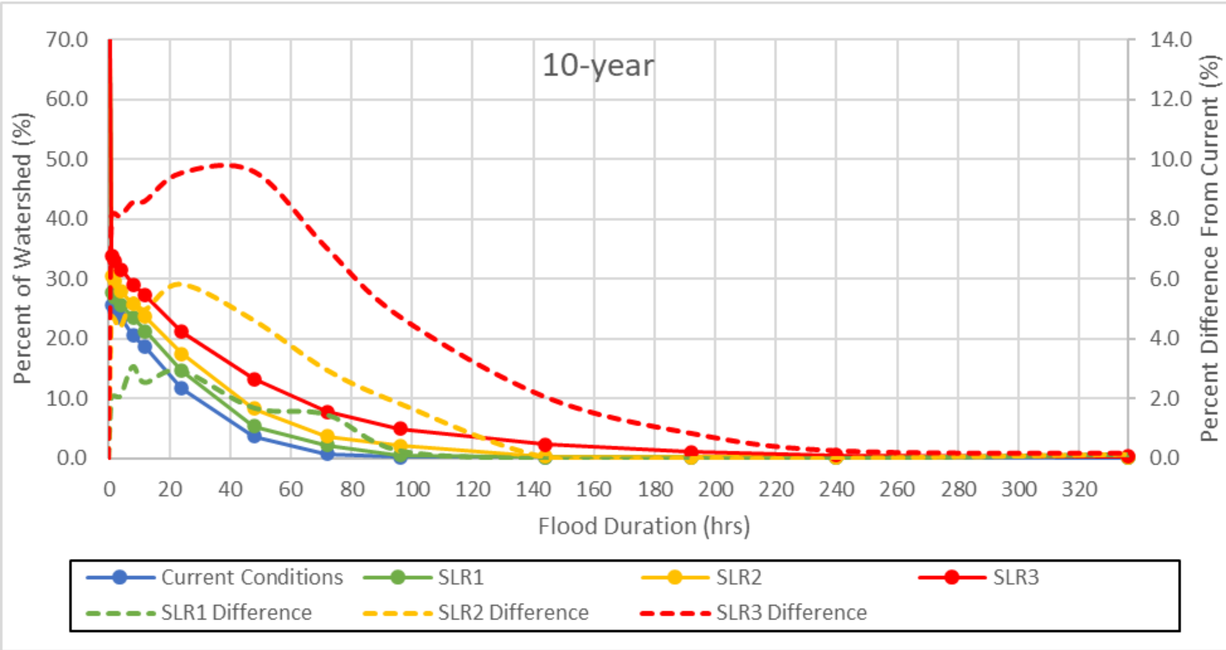


Figure 9-112. Percentage and Duration of Inundation for the C3W Watershed for the 5-year Storm Event

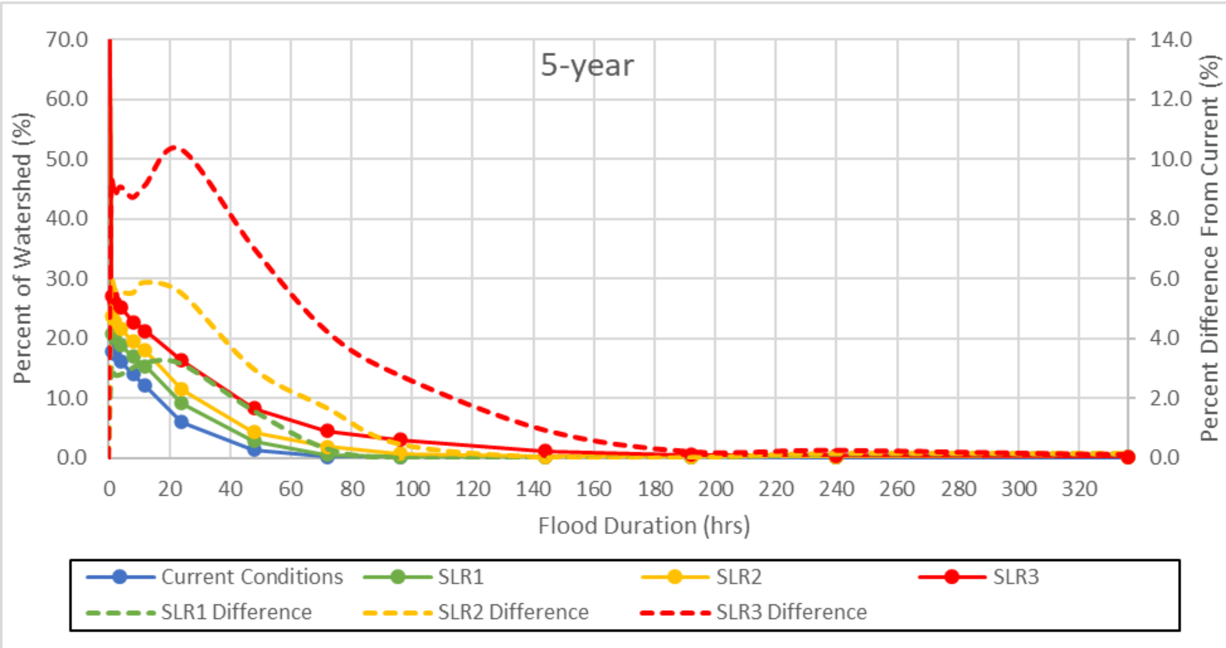


Figure 9-113. Flood Duration Map for the Current Conditions 25-year 3-day Design Storm for the C3W Watershed

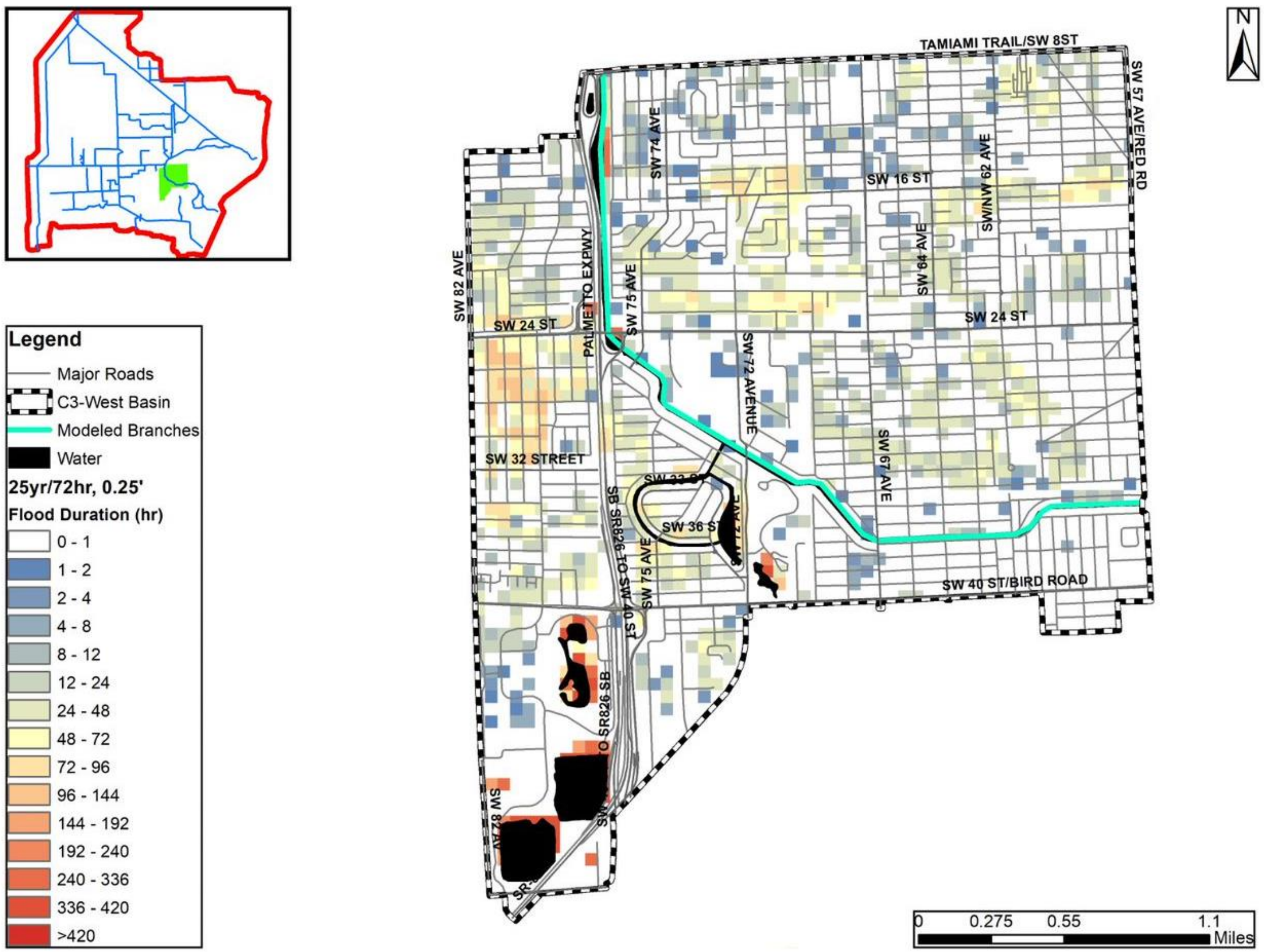


Figure 9-114. Flood Duration Map for the Current Conditions 100-year 3-day Design Storm for the C3W Watershed

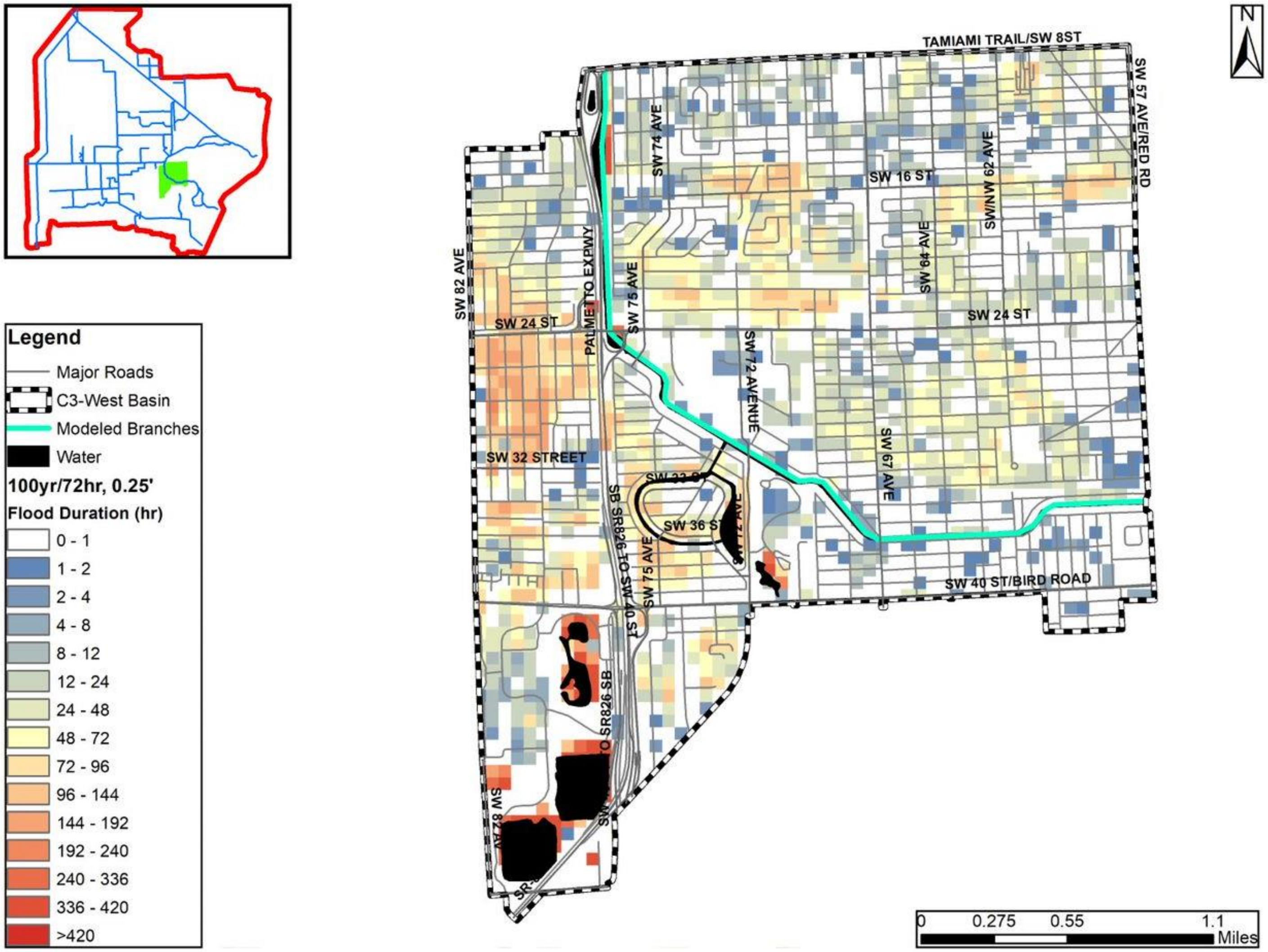


Figure 9-115. Urban Flooding Duration Difference of SLR +1ft and Current Conditions for the 100-year Storm in the C3W Watershed

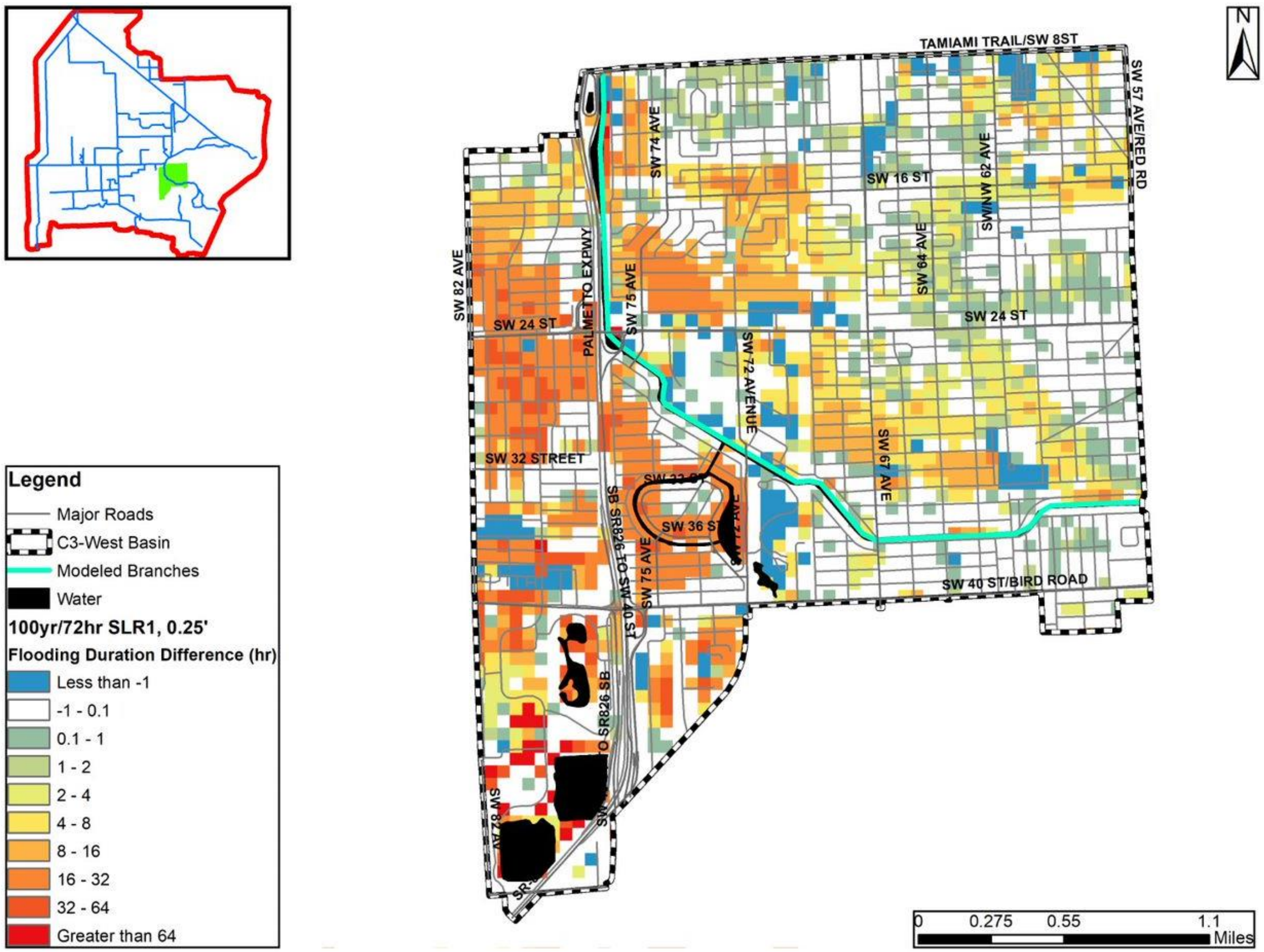


Figure 9-116. Urban Flooding Duration Difference of SLR +2ft and Current Conditions for the 100-year Storm in the C3W Watershed

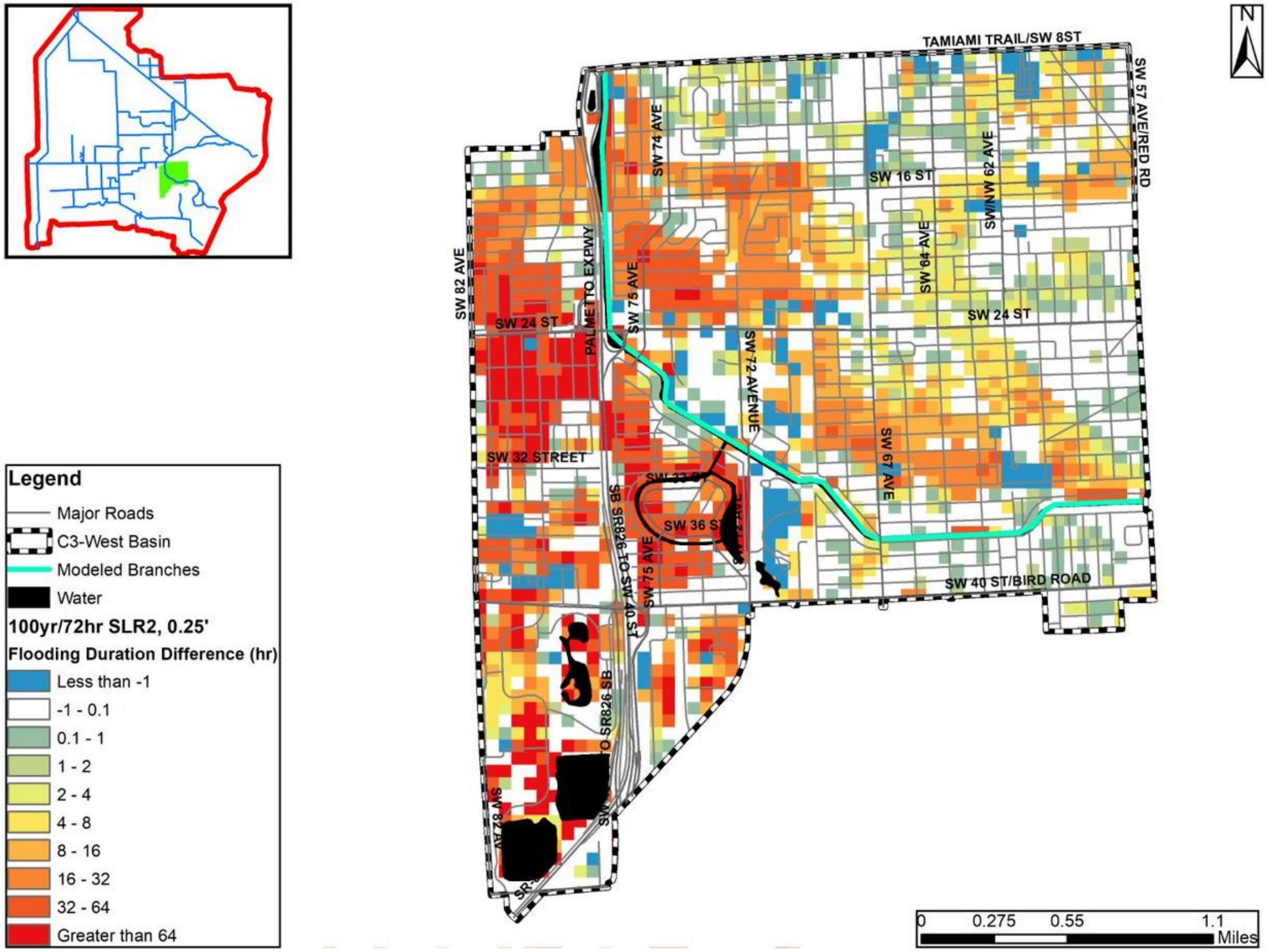
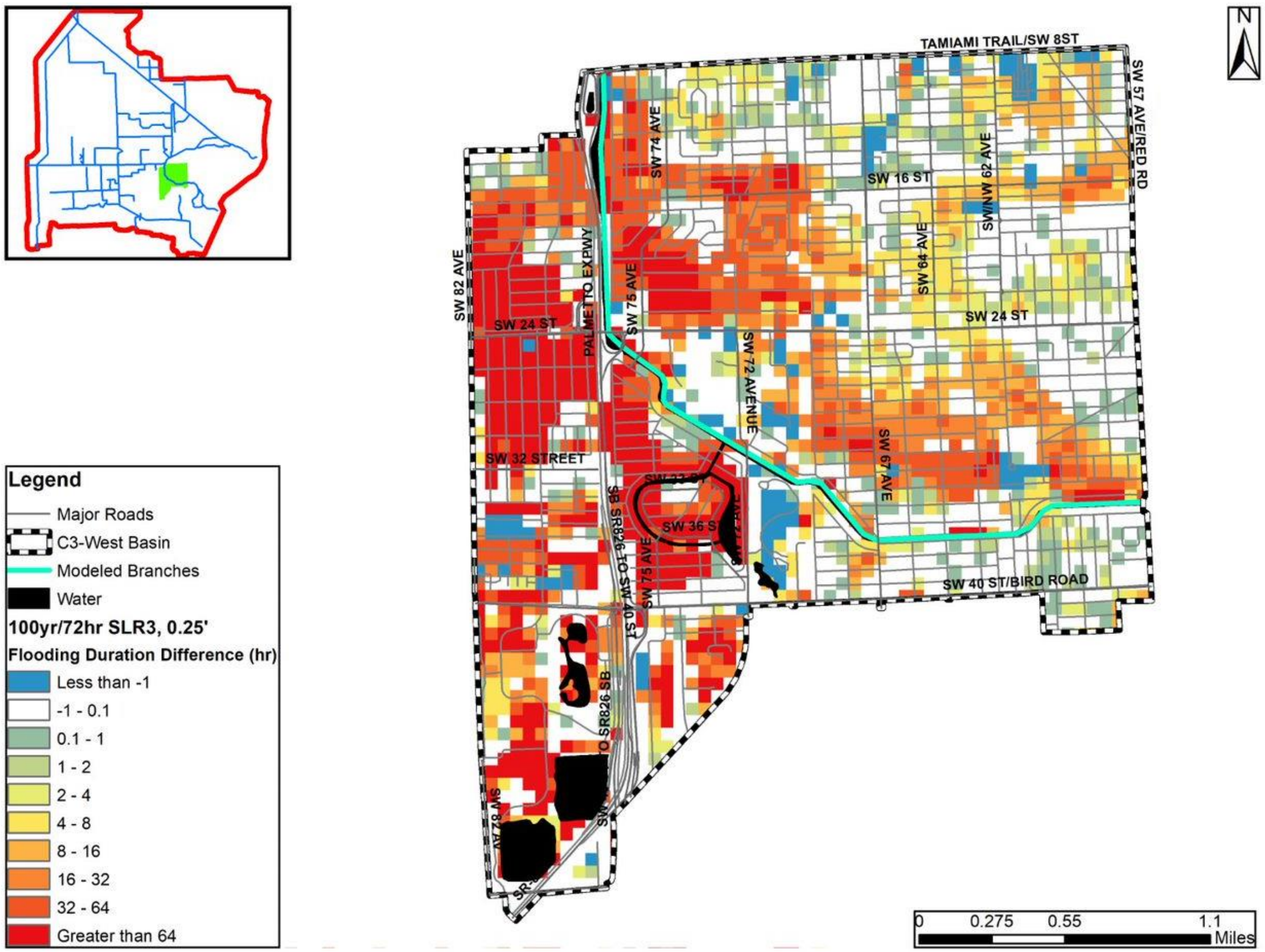
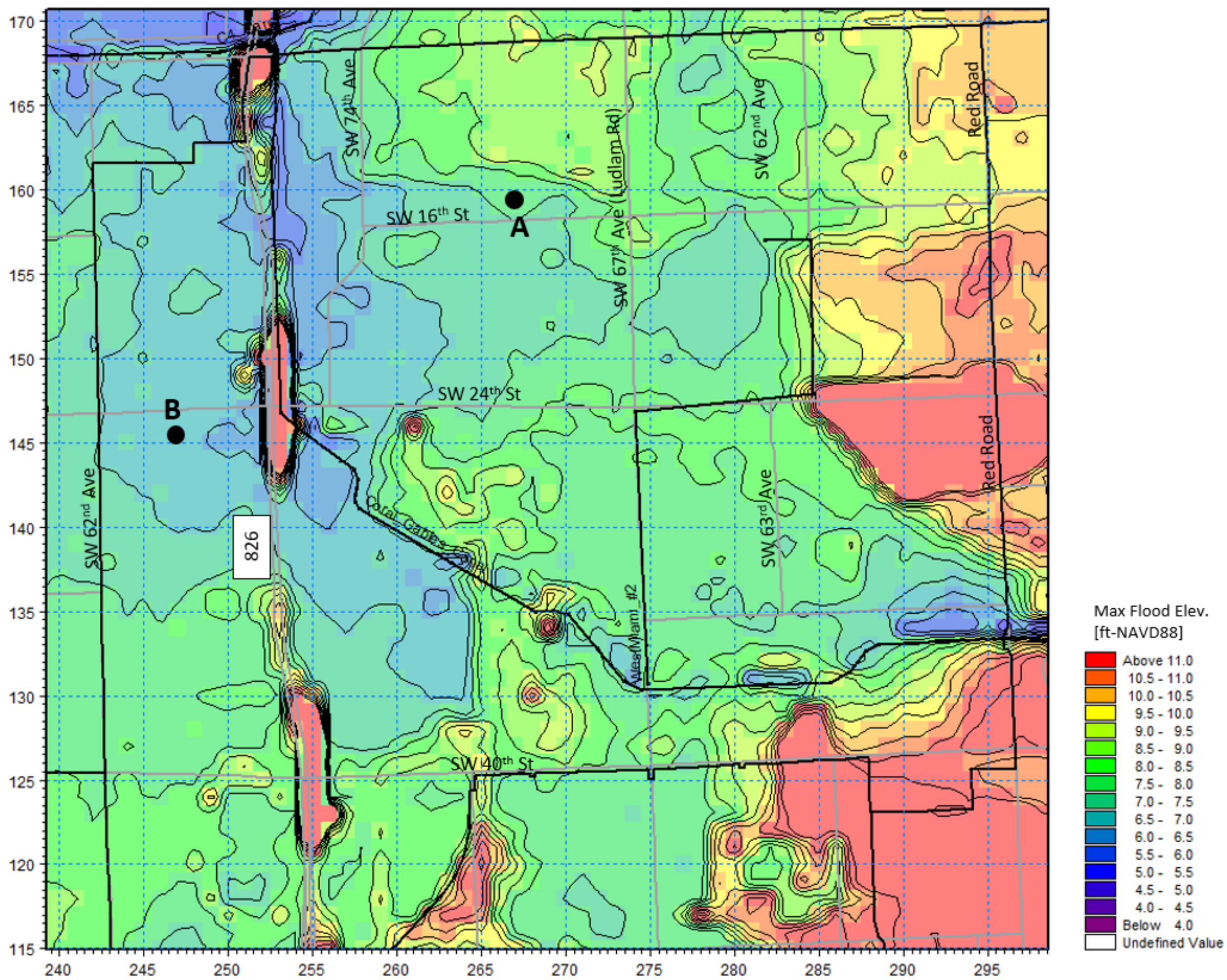


Figure 9-117. Urban Flooding Duration Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C3W Watershed



For the C3W Watershed some areas tend to experience flooding for longer periods. Some areas of concern are along SW 16th Street and in Coral Way Village to the west of the 826 (Palmetto Expressway). **Figure 9-118** provides a map of the overland flooding elevation to compare canal stages with the overland flooding at points A and B shown on the map.

Figure 9-118. Overland Flood Elevation for the 100-year Design Storm for the C3W Watershed



As discussed in PM #1, overtopping of the canal is not a particular concern upstream of the G93 structure. Stages at both point A and point B, shown in **Figure 9-119,** indicate that the elevation of overland water at this location is not directly impacted by the canal stages, as the land surface elevation at both locations are higher than the peak of the storm event. However, both locations are in low-lying basins that may not drain well, and flooding concerns from these areas would be considered a tertiary drainage issue. **Figure 9-120** shows the LiDAR topography with both areas circled in red.

Figure 9-119. Coral Gables Canal Stages Compared with Overland Flood Elevation for 100-year Design Storm

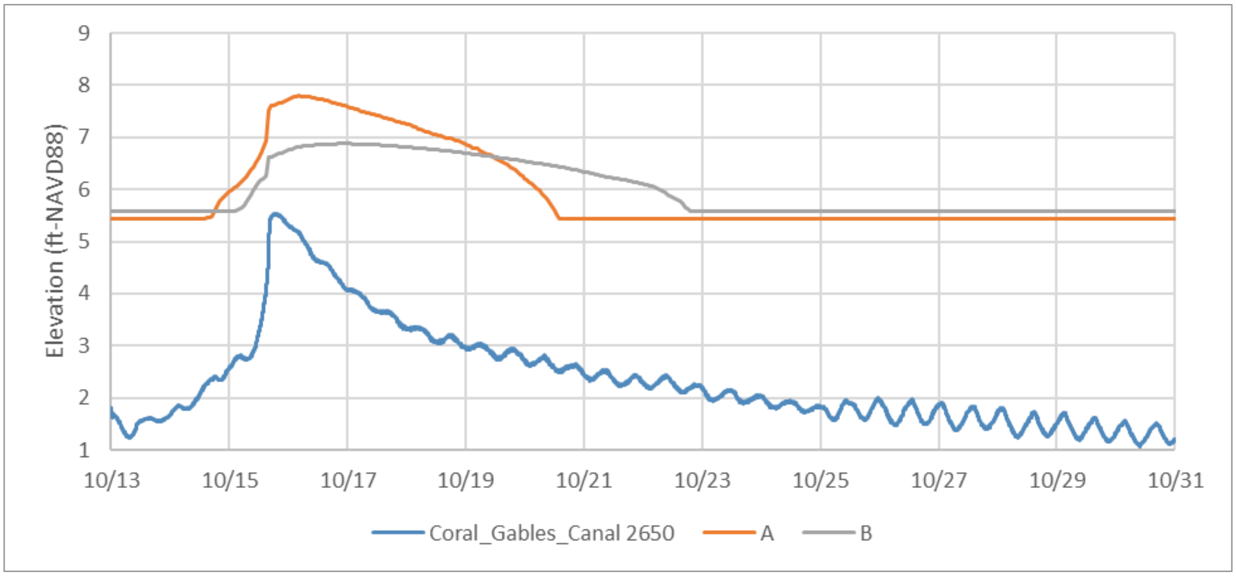
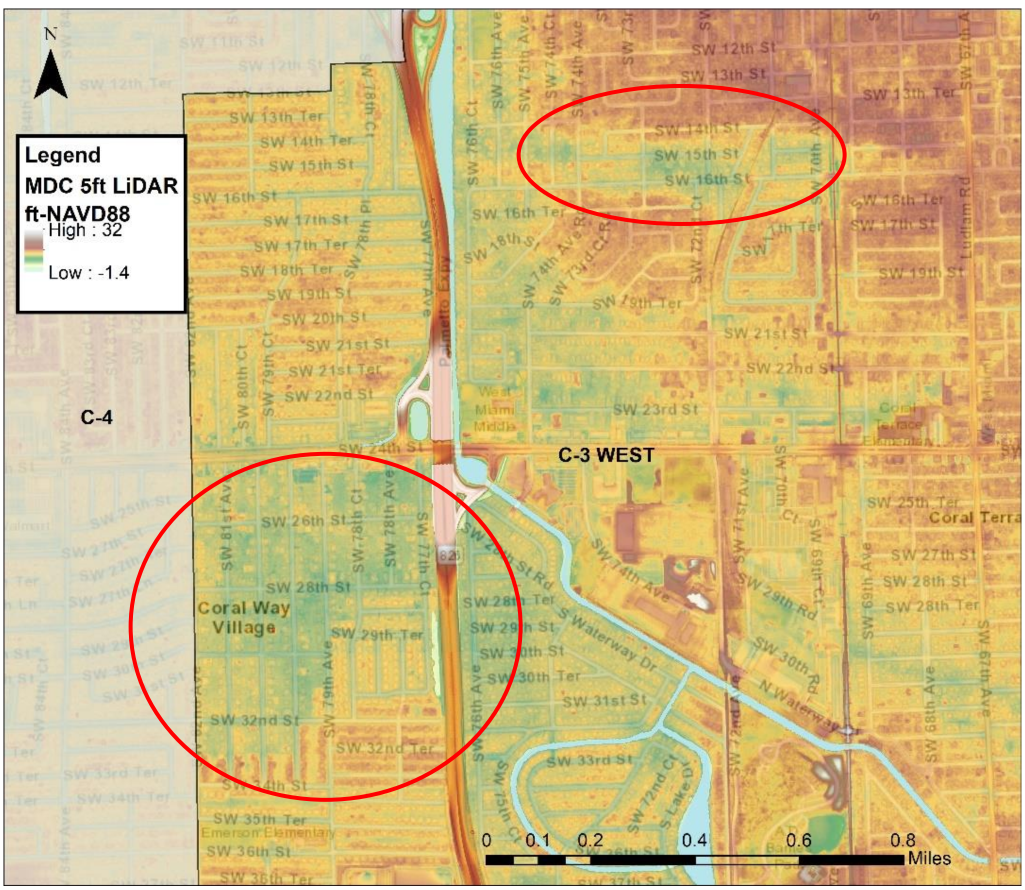


Figure 9-120. LiDAR Topography of Low-Lying Basins in C3W Watershed



9.2.7. SUMMARY OF LOS AND RATING FOR THE C3W WATERSHED

The maximum design storm frequency that the C3W Watershed passes without incurring negative impacts is summarized for each performance metric in **Table 9-40**.

Under critical consideration for this watershed is PM #1, PM #5, and PM #6, as these relate directly to flooding in canals and overland flooding depth and duration. For the current conditions, it was concluded that the drainage provided by the primary system in the C3W basin can handle up to the 25-year event. There is a substantial number of flooded areas likely from tertiary drainage issues due to distance from the canal system, these issues are exacerbated with increasing canal stages, which reduces the ability of the region to drain. Additionally, overtopping of the canal just upstream of the G93 structure increases with SLR, as discussed in PM #1, which then spills into the overland to increase flooding depths and extents, as shown in PM #5. This indicates some amount of primary drainage issues at higher SLR conditions. To account for this, an emphasis on the overall rating was placed on the PM #1 LOS rating, which gives the SLR1 a 10-year, SLR2 a 5-year, and SLR3 does not pass any storms.

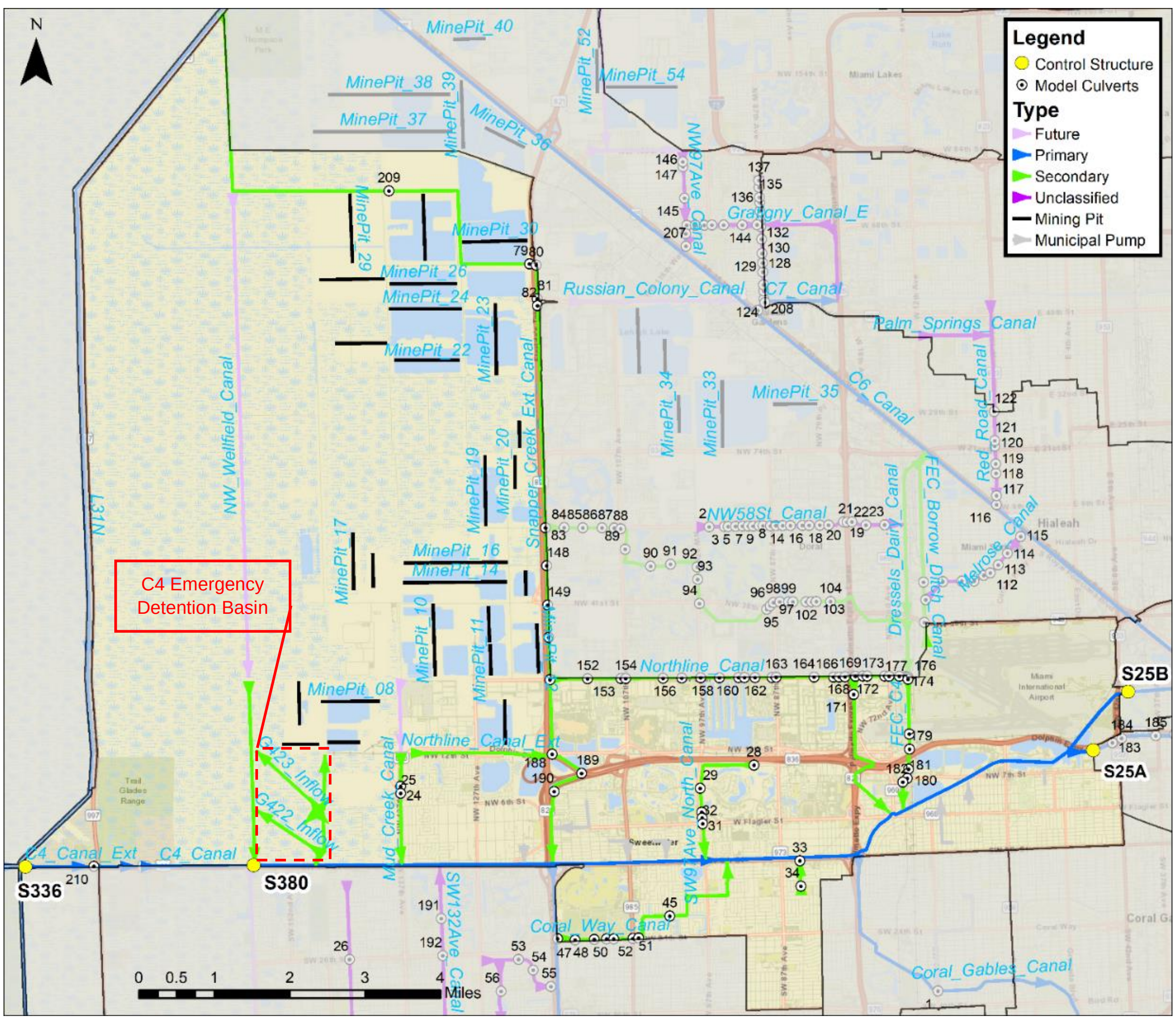
Table 9-40. Performance Metric Summary for the C3W Watershed

Metric	Notes	Current Conds.	SLR +1 ft	SLR +2 ft	SLR +3 ft
PM #1	▪ No design storms exceeded bridge low chords. ▪ No crown of road at culvert locations were overtopped. ▪ Top of bank was only overtopped just upstream of G93 for the 100-yearr storm under current conditions. For SLR1, this area was overtopped for the 25-year. For SLR2, this area was overtopped for the 10-year. And for SLR3, this area was always overtopped.	25-year	10-year	5-year	<5-year
PM #2	▪ No comparable value found for this basin. ▪ Discharge at G93 decreases with each SLR condition and higher storm event, however, reversal of flows into the C4 keep the discharge capacity of the C3W watershed relatively consistent.	--	--	--	--
PM #3	▪ Maximum discharge at G93 falls below design value for all events. ▪ HW exceeds the water level that will bypass G93 for all future SLR scenarios except for the 5-year SLR +1ft scenario. The TW exceeds this bypass elevation during all future SLR scenarios except for the 5-year and 10-year SLR +1ft scenarios.	--	--	--	--
PM #4	▪ Peak 12-hour moving discharge ranges from 357 CFS to 579 CFS, compared to the design discharge of 640 CFS, and decreases with increasing SLR for each design storm return period.	--	--	--	--
PM #5	▪ 22.1% of the watershed was flooded with 0.75 ft or more during the 100-year, and only 12.0% for the 25-year storm. While some areas do show high flood depths, this is likely not due to the canal stages but rather a tertiary drainage issue. ▪ The percentage of area exceeded that was acceptable for current conditions for the 25-year storm is comparable to the 25-year for SLR1 and the 10-year storm for the SLR2 and SLR3 conditions.	25-year	25-year	10-year	10-year
PM #6	▪ Canal: Stages at the T5W station recede after 68 hours for the 25-year storm for current conditions, and for more than 3 days for the 100-year storm, which is longer than the duration of the storm event itself. For SLR1, the canal recedes in less than 72 hours for the 10-year storm. For SLR2 and SLR3, the canal takes longer than 4 days to recede for all storm events. ▪ Watershed: Percent increases for the current conditions 25-year are comparable to the 10-year storms for SLR1 and SLR2, while this is comparable to the 5-year storm for SLR3.	25-year	10-year	10-year	5-year
Overall Level of Service		**25-year**	**10-year**	**5-year**	**<5-year**

9.3. C4 WATERSHED FLOOD PROTECTION LEVEL OF SERVICE

The C4 Watershed consists of areas draining to the C4 Canal from the western wetland regions, the Central Mining Lake Belt region, and the urban developed regions east of NW 137th Avenue to NW42nd Avenue. The primary discharge canal is the C4 Canal from the S336 water control structure to the S25B water control structure. S25A discharges to the C5 Watershed, however, this structure is typically closed in the wet season and is typically used during the dry season to control salinity. S25B is a tidal water control structure that controls water levels in the C4 Canal, located just downstream of NW 42nd Avenue, or LeJeune Rd. During the wet season, the S25B gate is operated to maintain the water elevations of (-)0.55 to 0.45 ft-NAVD (1.0 to 2.0 ft-NGVD). S25B also has a forward pump that is operated when gravity capacity is limited, and the gate must be closed. **Figure 9-121** shows a map of the entire C4 Watershed.

Figure 9-121. Map of the C4 Watershed



The C4 Watershed also contains the C4 Emergency Detention Basin, located just east of the Dade/Broward Levee. The G420 and G422 Pump stations move water from the C4 Canal into above ground storage facilities capable of storing water up to 8.43 ft-NAVD (10 ft-NGVD) during flooding events, with a total pump capacity of 1,292 CFS with all pumps running.

9.3.1. C4 – PM #1 MAXIMUM STAGE IN PRIMARY CANALS

The maximum stage in the primary canals was extracted from the results of each design storm simulation for the C4 Watershed. **Table 9-41** provides the low chord information for each bridge in the C4 Canal, as well as the peak stage at nearest H-point (where canal stages are calculated in the model). This can be used to establish any issues with bridge low chords. Within the C4 Canal, no bridge low-chords were reached at the peak of any storm event simulated. However, the peak of the 100-year design storm for the SLR3 condition came within 0.4 feet of the low chords of the railroad trestle bridges, located near Robert King High Park.

Table 9-42 provides the culvert information with the estimated crown of road from LiDAR data at each culvert location in the C4 Watershed, as well as the peak stage at the nearest H-point. The crown of road was estimated from LiDAR data by determining the elevation at the intersection of the center line of the canal and the top of the roadbed. Stages which overtop the estimated crown of road are highlighted in orange. In addition, a culvert number is provided in the first column, which corresponds to the culverts numbered in **Figure 9-121**. Of the 61 culverts represented in the model for the C4 Watershed, 16 experienced overtopping during the 100-year design storm and only five (5) experienced overtopping during the 25-year design storm for current conditions. However, the number of overtopped culvert locations increases rapidly with each future SLR condition, as shown in **Figure 9-122**. As seen in the graph, the greatest increase in overtopping locations is seen with the increase in SLR from +1 feet to +2 feet for the 25-year storm (with an additional 8 culverts overtopped) and from +2 feet to +3 feet for the 5-year storm (with an additional 12 culverts overtopped).

Table 9-41. Bridge Low Chord and Peak Stage for the C4 Canal

Location Description	Low Chord	Bridge Top	Current Conditions				SLR1				SLR2				SLR3			
			Elevation (ft-NAVD)															
			100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
SW 132nd Ave	8.33	10.33	5.49	4.90	4.53	4.25	5.93	5.14	4.87	4.58	6.43	5.66	5.20	4.87	6.81	6.19	5.68	5.44
SW 127th Ave	8.42	11.33	5.58	4.98	4.60	4.30	6.00	5.19	4.93	4.65	6.46	5.74	5.27	4.94	6.82	6.22	5.79	5.53
122nd Ave	8.00	11.45	5.62	5.02	4.64	4.33	6.04	5.23	4.96	4.69	6.47	5.78	5.30	4.96	6.84	6.25	5.83	5.58
Turnpike SW ramp	8.13	10.12	5.66	5.09	4.69	4.38	6.06	5.31	5.00	4.73	6.46	5.82	5.32	5.00	6.81	6.25	5.86	5.61
SW 109th Ave	7.54	8.85	5.62	5.10	4.66	4.37	6.07	5.36	5.06	4.73	6.47	5.84	5.36	5.06	6.82	6.26	5.88	5.63
SW 107th Ave	7.47	9.96	5.60	5.09	4.66	4.36	6.06	5.41	5.06	4.76	6.45	5.83	5.40	5.10	6.82	6.26	5.89	5.66
SW 97th Ave	8.34	10.33	5.58	5.07	4.63	4.31	6.03	5.39	5.05	4.74	6.44	5.81	5.39	5.10	6.82	6.25	5.88	5.64
SW 94th Ave	7.16	8.45	5.56	5.01	4.56	4.23	5.97	5.37	5.01	4.70	6.40	5.78	5.36	5.08	6.80	6.23	5.86	5.62
SW 92nd Ave	8.38	9.45	5.54	4.96	4.48	4.16	5.92	5.34	4.97	4.65	6.36	5.75	5.33	5.06	6.79	6.20	5.84	5.60
87th (Galloway) Ave	7.46	9.45	5.51	4.88	4.39	4.05	5.86	5.31	4.92	4.59	6.31	5.71	5.30	5.03	6.78	6.18	5.81	5.58
SR 826	25.46	28.45	5.47	4.77	4.25	3.91	5.81	5.26	4.85	4.50	6.25	5.66	5.25	4.99	6.77	6.15	5.79	5.55
Flagler St	7.70	12.95	5.40	4.67	4.12	3.77	5.76	5.22	4.79	4.44	6.21	5.63	5.23	4.96	6.78	6.13	5.77	5.53
Milam Dairy Road	8.12	10.15	5.09	4.31	3.85	3.45	5.51	4.92	4.53	4.18	5.92	5.30	4.93	4.63	6.58	5.90	5.52	5.25
FEC Railroad West Bridge	6.96	8.45	5.07	4.29	3.82	3.42	5.50	4.91	4.52	4.17	5.91	5.30	4.93	4.63	6.58	5.89	5.52	5.25
Old Railroad Bridge	6.96	8.45	4.96	4.14	3.63	3.23	5.43	4.83	4.44	4.08	5.87	5.25	4.88	4.59	6.62	5.88	5.50	5.22
NW 7th Street	8.98	12.27	4.86	3.98	3.44	3.04	5.36	4.75	4.35	4.00	5.83	5.21	4.84	4.56	6.66	5.87	5.47	5.19
Pan American Hospital	7.26	9.20	4.84	3.94	3.39	2.99	5.38	4.73	4.32	3.97	5.85	5.23	4.86	4.56	6.68	5.86	5.49	5.22
NW 57th Ave	7.46	9.65	4.81	3.89	3.33	2.93	5.40	4.71	4.30	3.93	5.87	5.26	4.87	4.57	6.70	5.87	5.52	5.25
SR 836 - Dolphin Expressway	12.96	14.95	4.79	3.84	3.30	2.88	5.42	4.71	4.28	3.90	5.90	5.28	4.88	4.57	6.72	5.89	5.53	5.28
Airport parking lot	11.46	13.45	4.79	3.81	3.26	2.83	5.45	4.71	4.26	3.88	5.94	5.32	4.91	4.59	6.75	5.93	5.56	5.31
*Highlighted cells indicate the stages exceed the bridge low chord																		

Table 9-42. Estimated Culvert Crown of Road and Peak Stage for the C4 Watershed

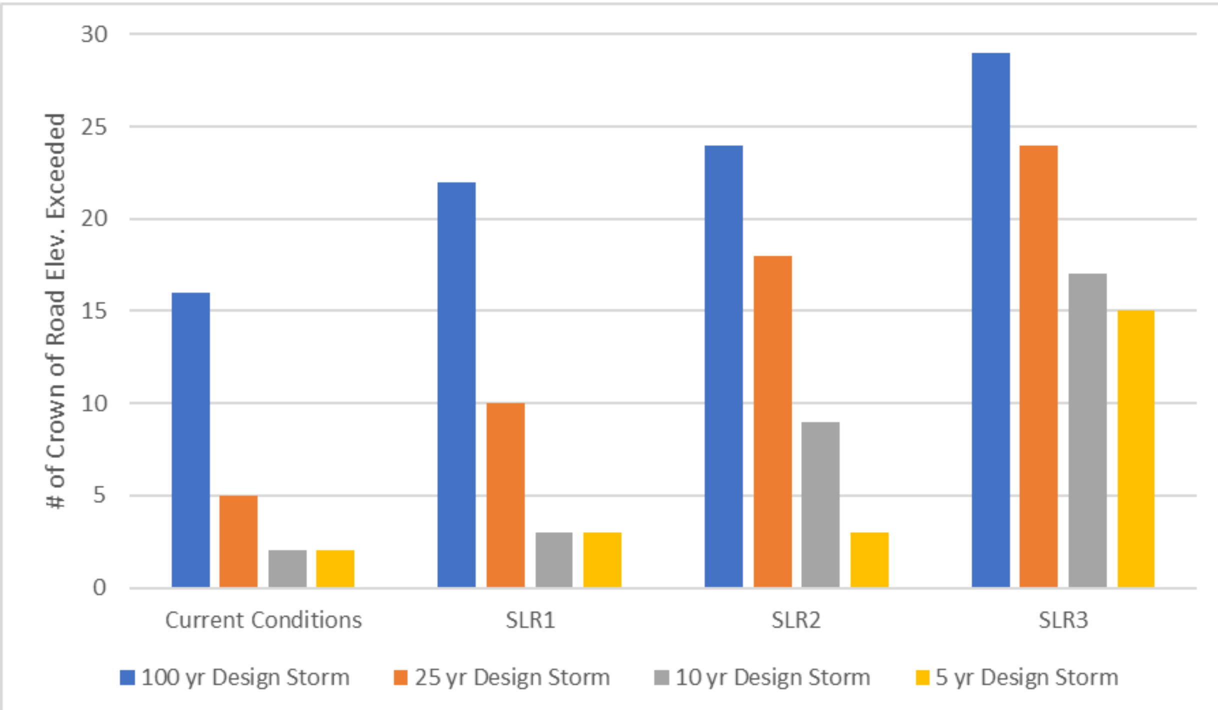
Culvert Number	Intersection	Top of Culvert	Estimated Crown of	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Mud_Creek_Canal																			
24	FPL South Drive1	4.03	6.8	4.95	4.61	4.47	3.74	5.85	4.89	4.64	4.57	6.38	5.50	4.91	4.70	6.80	6.11	5.51	5.10
25	FPL South Drive2	6.87	8.8	4.94	4.61	4.48	3.70	5.83	4.80	4.63	4.56	6.38	5.49	4.81	4.66	6.81	6.11	5.50	5.09
SW97Ave_North_Canal																			
28	NW 8th St	1.94	4.8	5.67	5.12	4.70	4.41	6.09	5.44	5.11	4.80	6.45	5.84	5.40	5.12	6.77	6.25	5.88	5.63
29	Fountainbleau Blvd	2.52	5.6	5.66	5.12	4.69	4.39	6.09	5.43	5.10	4.79	6.45	5.84	5.40	5.11	6.77	6.25	5.88	5.62
30	Blue Riviera Drive	2.03	5.3	5.65	5.11	4.68	4.38	6.08	5.43	5.10	4.79	6.45	5.83	5.40	5.12	6.78	6.25	5.88	5.63
31	Park Hill Plaza Drive	2.45	5.7	5.63	5.10	4.67	4.36	6.07	5.42	5.09	4.78	6.45	5.83	5.40	5.12	6.79	6.25	5.88	5.63
32	W Flagler St	2.20	6.1	5.60	5.09	4.66	4.35	6.06	5.41	5.08	4.76	6.45	5.82	5.40	5.11	6.80	6.25	5.88	5.64
Westchester_Canal																			
33	Tamiami Trl	0.20	8.2	5.94	5.47	4.92	4.43	6.23	5.77	5.42	4.96	6.49	6.01	5.65	5.38	6.81	6.30	6.00	5.78
34	SW 14th St	1.70	4.6	6.19	5.80	5.24	4.65	6.62	6.20	5.84	5.27	6.68	6.33	6.01	5.70	6.89	6.45	6.21	6.01
Coral_Way_Canal																			
45	SW 102nd Ave	5.52	8.9	5.64	5.12	4.69	4.39	6.06	5.43	5.08	4.78	6.46	5.84	5.41	5.12	6.83	6.27	5.90	5.66
46	SW 17th Ave	0.59	5.9	5.69	5.16	4.74	4.43	6.04	5.42	5.07	4.78	6.42	5.79	5.35	5.07	6.77	6.22	5.90	5.64
47	SW 114th Ave	2.85	5.2	5.82	5.24	4.80	4.47	6.09	5.50	5.12	4.82	6.46	5.84	5.39	5.11	6.80	6.25	5.93	5.66
48	SW 112th Ave	3.90	5.8	5.87	5.25	4.82	4.49	6.10	5.52	5.14	4.83	6.48	5.86	5.42	5.13	6.82	6.27	5.94	5.67
49	SW 10900 Blk	3.05	5.5	5.89	5.26	4.83	4.49	6.12	5.54	5.15	4.84	6.49	5.87	5.43	5.15	6.83	6.28	5.94	5.68
50	SW 109th Ave	2.72	5.6	5.90	5.26	4.83	4.50	6.13	5.55	5.15	4.84	6.49	5.88	5.44	5.15	6.83	6.28	5.95	5.68

Culvert Number	Intersection	Top of Culvert	Estimated Crown of	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
51	SW 107th Ave	2.86	6.6	5.90	5.26	4.83	4.50	6.13	5.55	5.15	4.84	6.49	5.88	5.44	5.15	6.84	6.29	5.94	5.69
52	SW 106th Ave	-0.14	6.0	5.87	5.24	4.81	4.48	6.11	5.53	5.14	4.83	6.49	5.88	5.44	5.15	6.84	6.29	5.94	5.68
Snapper_Creek_Ext_Canal																			
79	NW 114th St East	1.75	6.8	6.10	5.58	5.28	5.09	6.18	5.71	5.39	5.18	6.35	5.89	5.61	5.42	6.51	6.07	5.79	5.61
80	NW 114th St West	1.66	7.6	6.09	5.57	5.27	5.09	6.17	5.69	5.38	5.17	6.34	5.88	5.59	5.40	6.50	6.06	5.78	5.59
81	Beacon Station Blvd - Tpk OnRamp	3.72	12.4	6.08	5.57	5.27	5.09	6.16	5.68	5.37	5.16	6.33	5.87	5.58	5.39	6.50	6.05	5.77	5.58
82	Beacon Station Blvd- Tpk OffRamp	4.72	12.5	6.07	5.56	5.26	5.08	6.15	5.66	5.35	5.15	6.33	5.85	5.57	5.37	6.50	6.05	5.76	5.57
83	NW 58th St	1.90	5.4	6.06	5.55	5.26	5.07	6.13	5.64	5.34	5.14	6.32	5.84	5.55	5.35	6.50	6.04	5.75	5.55
148	NW 50th St	1.67	4.4	6.01	5.51	5.23	5.04	6.11	5.59	5.30	5.10	6.33	5.82	5.51	5.31	6.52	6.06	5.75	5.54
149	NW 117th Ave	1.80	6.7	5.99	5.50	5.21	5.03	6.10	5.58	5.29	5.09	6.34	5.82	5.50	5.30	6.53	6.07	5.75	5.54
150	NW 34th St	2.82	5.8	5.98	5.48	5.20	5.02	6.10	5.57	5.28	5.08	6.35	5.82	5.49	5.28	6.55	6.09	5.76	5.55
151	NW 25th St	3.38	7.0	5.95	5.46	5.17	4.99	6.10	5.55	5.26	5.05	6.37	5.81	5.47	5.26	6.58	6.11	5.90	5.90
188	Dolphin Expr Westbound	6.50	8.3	5.70	5.11	4.72	4.40	6.06	5.33	5.02	4.74	6.42	5.79	5.32	5.00	6.72	6.20	5.81	5.56
189	Dolphin Expy to Turnpike	0.56	13.3	5.70	5.11	4.72	4.40	6.06	5.33	5.02	4.74	6.42	5.79	5.32	5.00	6.72	6.20	5.81	5.57
190	Turnpike to Dolphin Expy	2.45	8.3	5.67	5.08	4.69	4.37	6.06	5.31	5.00	4.72	6.44	5.79	5.31	4.99	6.77	6.22	5.83	5.58
209	NW Wellfield Culvert	7	8.6	6.13	5.60	5.29	5.27	6.26	5.84	5.48	5.30	6.40	6.00	5.75	5.55	6.55	6.14	5.91	5.74

Culvert Number	Intersection	Top of Culvert	Estimated Crown of	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Northline_Canal																			
152	NW 112th Ave	3.73	6.5	6.07	5.43	4.83	4.44	6.30	5.80	5.30	4.90	6.47	6.02	5.62	5.29	6.65	6.29	5.99	5.77
153	MBTU Driveway	5.39	7.4	6.06	5.42	4.82	4.43	6.29	5.79	5.29	4.89	6.46	6.02	5.61	5.28	6.65	6.28	5.98	5.76
154	NW 107th Ave	5.20	7.5	6.06	5.42	4.81	4.43	6.28	5.78	5.28	4.89	6.46	6.01	5.61	5.28	6.65	6.28	5.98	5.76
155	NW 102nd Ave	10.38	8.1	6.02	5.38	4.79	4.40	6.25	5.75	5.26	4.86	6.43	5.97	5.58	5.26	6.63	6.26	5.96	5.74
156	NW 99th Ave	10.26	7.7	5.99	5.35	4.76	4.38	6.23	5.72	5.23	4.84	6.41	5.95	5.56	5.24	6.62	6.25	5.95	5.72
157	NW 97th Ave	10.05	8.2	5.96	5.32	4.73	4.35	6.20	5.69	5.21	4.82	6.39	5.93	5.54	5.22	6.61	6.24	5.93	5.70
158	NW 94th Ave	10.16	7.1	5.93	5.27	4.70	4.32	6.17	5.66	5.18	4.79	6.37	5.90	5.51	5.20	6.60	6.22	5.91	5.69
159	NW 92nd Ave	5.81	6.9	5.89	5.23	4.66	4.29	6.14	5.63	5.15	4.77	6.35	5.88	5.48	5.18	6.60	6.21	5.90	5.67
160	MDPD Headquarters Drive	5.61	7.3	5.84	5.18	4.62	4.25	6.11	5.59	5.12	4.74	6.33	5.85	5.46	5.15	6.60	6.20	5.88	5.65
161	NW 89th Ct	5.66	6.8	5.80	5.13	4.58	4.22	6.08	5.55	5.08	4.71	6.31	5.83	5.43	5.13	6.60	6.18	5.86	5.64
162	NW 87th Ave	5.15	6.7	5.77	5.10	4.55	4.19	6.05	5.53	5.06	4.69	6.30	5.81	5.41	5.11	6.61	6.17	5.85	5.62
163	NW 87th Ave	11.56	7.2	5.74	5.07	4.53	4.17	6.03	5.50	5.04	4.67	6.29	5.79	5.39	5.10	6.61	6.17	5.84	5.61
164	NW 82nd Ave	11.17	6.0	5.74	5.06	4.52	4.16	6.03	5.50	5.04	4.67	6.29	5.79	5.39	5.09	6.62	6.17	5.84	5.61
165	NW 79th Ave	11.91	6.8	5.71	5.03	4.48	4.12	6.00	5.47	5.01	4.64	6.28	5.76	5.37	5.07	6.62	6.16	5.83	5.60
166	NW 78th Ave	9.65	7.6	5.67	4.98	4.44	4.07	5.97	5.44	4.97	4.61	6.26	5.74	5.34	5.05	6.63	6.15	5.81	5.58
167	Palmetto Expy W	1.66	5.6	5.67	4.98	4.44	4.07	5.97	5.44	4.97	4.61	6.26	5.74	5.34	5.05	6.63	6.15	5.81	5.58
NW25St_Canal																			
168	Palmetto Expy E	1.78	5.2	5.52	4.80	4.27	3.91	5.85	5.30	4.85	4.50	6.22	5.66	5.26	4.99	6.65	6.12	5.77	5.54
169	NW 75th Ave	2.00	5.1	5.50	4.78	4.25	3.89	5.84	5.28	4.83	4.48	6.20	5.64	5.24	4.97	6.62	6.09	5.75	5.52
170	Utility Access	1.87	5.3	5.48	4.76	4.23	3.88	5.83	5.26	4.81	4.46	6.18	5.62	5.22	4.95	6.59	6.06	5.72	5.50
172	NW 25th St	8.34	6.8	5.46	4.74	4.21	3.85	5.80	5.23	4.79	4.44	6.14	5.60	5.20	4.93	6.53	6.02	5.69	5.48
173	Milam Dairy Rd	0.78	6.3	5.44	4.72	4.19	3.84	5.79	5.22	4.77	4.42	6.13	5.58	5.18	4.91	6.51	6.00	5.68	5.46

Culvert Number	Intersection	Top of Culvert	Estimated Crown of	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
174	NW 25th St Viaduct	1.70	5.0	5.33	4.62	4.12	3.77	5.71	5.13	4.69	4.35	6.01	5.50	5.11	4.85	6.36	5.88	5.60	5.40
175	NW 70th Ave	1.71	6.9	5.38	4.66	4.15	3.80	5.74	5.16	4.72	4.38	6.06	5.53	5.14	4.88	6.43	5.93	5.63	5.43
NorthlineNS_C4																			
171	NW 19th St	2.28	9.7	5.52	4.80	4.27	3.91	5.85	5.30	4.85	4.50	6.22	5.66	5.26	4.99	6.65	6.12	5.77	5.54
FEC_C4																			
176	NW 25th St	6.66	6.3	5.29	4.59	4.09	3.75	5.68	5.09	4.66	4.32	5.98	5.47	5.08	4.82	6.31	5.84	5.57	5.37
177	FEC Railway	1.82	5.6	5.31	4.60	4.10	3.75	5.69	5.11	4.68	4.34	6.00	5.48	5.10	4.84	6.33	5.86	5.58	5.39
178	Under Airport Runway	0.74	5.7	5.35	4.64	4.12	3.77	5.73	5.16	4.73	4.38	6.08	5.54	5.15	4.88	6.49	5.96	5.65	5.44
179	NW 12th St	0.91	6.8	5.35	4.64	4.12	3.77	5.73	5.16	4.73	4.38	6.08	5.54	5.15	4.89	6.49	5.96	5.65	5.44
180	Under Double Tree Hotel	2.87	6.4	5.36	4.65	4.12	3.77	5.74	5.17	4.74	4.40	6.11	5.56	5.17	4.90	6.54	5.99	5.67	5.46
181	Access Road	2.14	5.4	5.38	4.65	4.12	3.77	5.75	5.19	4.76	4.41	6.15	5.59	5.19	4.92	6.63	6.05	5.71	5.48
182	NW 7th St	0.60	10.5	5.38	4.65	4.11	3.76	5.75	5.20	4.77	4.42	6.18	5.61	5.20	4.94	6.69	6.08	5.73	5.50
*Highlighted cells indicate the stages exceed the estimated crown of road.																			

Figure 9-122. Number of Culvert Locations Where the Crown of Road is Exceeded within the C4 Watershed



Problem canal and road intersections that were identified for the C4 Watershed include the following:

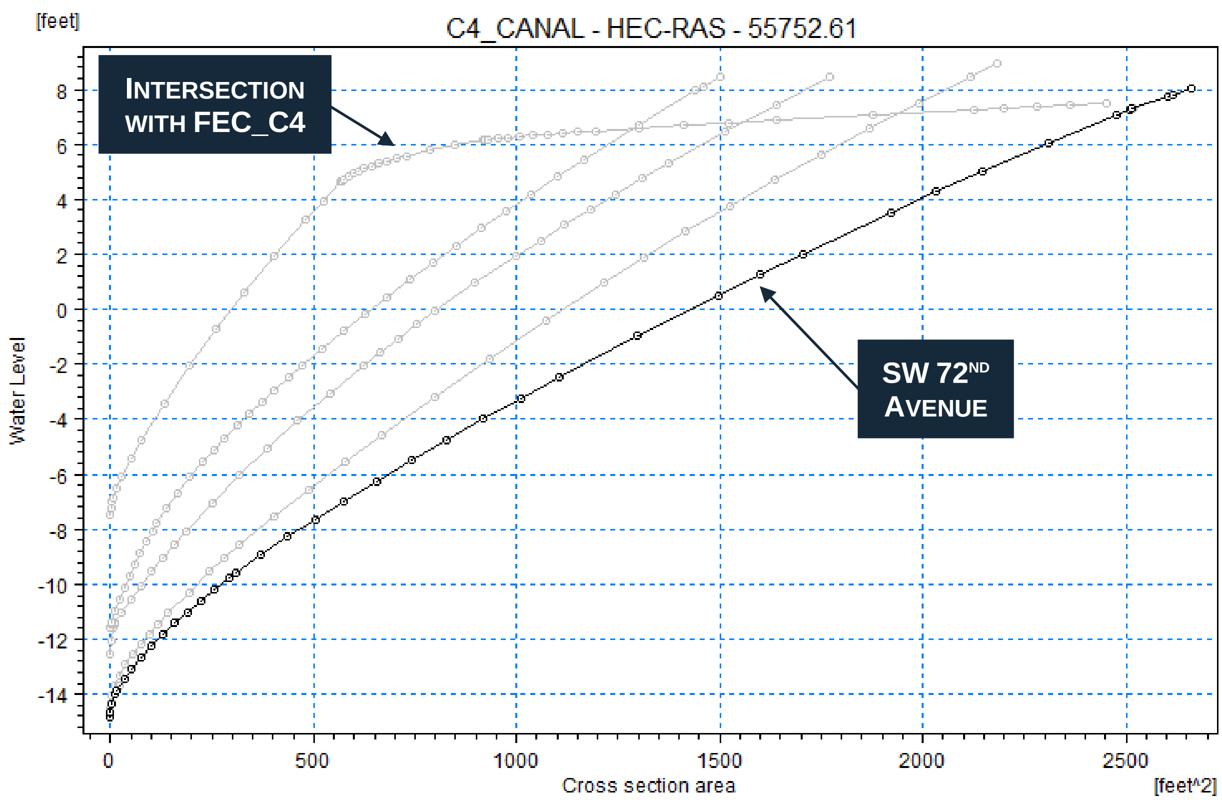
- Fountainbleau – SW 97th Avenue North Canal showed multiple culverts locations where overtopping occurs for the 25-year and 100-year design storms under current conditions.
- Westchester
  - Westchester Canal at 14th Street is flooded for all conditions.
  - Coral Way Canal near The Fair (between the Turnpike and NW 107th Avenue) experiences flooding for the 100-year storm event under current conditions.
- Doral East
  - There are several culverts near NW 25th Avenue Canal near the Palmetto Expressway that experience overtopping during the 100-year storm event under all conditions.

It should be noted that culvert #170 is under a utility access easement within NW 25th Avenue Canal that is not paved or accessible to traffic or pedestrians and does not affect service if flooded.

The maximum stage profiles for the C4 Canal are shown for the Current Conditions in **Figure 9-124**, and for all conditions for the 100-year Design Storm in **Figure 9-125**. These figures show peak stages (with bank elevations and major intersections locations) along the entire length of the C4 Canal from S336 to S25B structures. **Appendix A** provides a complete set of the C4 Canal maximum stage profiles for all design events.

For the Current Conditions, there is a large drop in head near the intersection with W Flagler Street. Upon review of the model data, it was determined that this drop is likely due to the rapid increase in cross-sectional area at this location, as indicated by the HEC-RAS cross-sections. From the intersection with FEC_C4 branch to SW 72nd Avenue, less than 1,000 feet distance the cross-sectional area increases rapidly and the bottom depth decreases from (-)7.5 to (-)14.8 ft-NAVD. **Figure 9-123** shows a plot of the cross-sectional area with water level, as set-up in the model cross-sections. This head drop is also seen for all future conditions' simulations in **Figure 9-125**.

Figure 9-123.Cross-Sectional Area with Water Level for the Cross-Sections Near Flagler St



As shown in **Figure 9-124**, a low spot in the embankment just upstream of SW 94th Avenue is overtopped during the 25-year design storm. This low spot represents the boat access ramp within the utilities easement, as shown in **Figure 9-126**. **Figure 9-125** indicates that other low spots to be overtopped during the 100-yr storm event for future conditions include:

- Between SW 132nd Avenue and SW 127th Avenue – this area represents the low-lying Tamiami Trail Park and may represent existing natural storage.
- Between SW 107th Avenue and SW 97th Avenue – this low-lying area of Sweetwater abuts the canal.
- Between NW 57th Avenue and SR 836 – the natural areas around the Blue Lagoon may represent existing natural storage.

Canal top of bank was established in the model from cross-section information from previous modeling efforts, including the C-4 Pilot Study (SFWMD, 2020), and converted to ft-NAVD. Full discussion of the development of these cross-sections is provided in **Section 3.3.1**. Top of bank was extracted from the model, and the miles of the evaluated canal segment that is overtopped by the design storm was estimated upstream of the tidal control structure S25B, as summarized in **Table 9-43**.

Table 9-43. Estimated Percentage and Miles of Bank Overtopped per Design Storm

WATERSHED	SIMULATION	100-YEAR	25-YEAR	10-YEAR	5-YEAR
C3W (%)	Current Conditions	11%	1%	0%	0%
	SLR1	18%	7%	1%	0%
	SLR2	30%	13%	1%	1%
	SLR3	46%	19%	13%	4%
C3W (miles)	Current Conditions	1.7	0.2	0.0	0.0
	SLR1	2.7	1.1	0.2	0.0
	SLR2	4.7	2.0	0.2	0.2
	SLR3	7.1	2.9	2.0	0.6

Figure 9-124. Maximum Stage Profile for the C4 Canal for the Current Conditions for All Storm Events

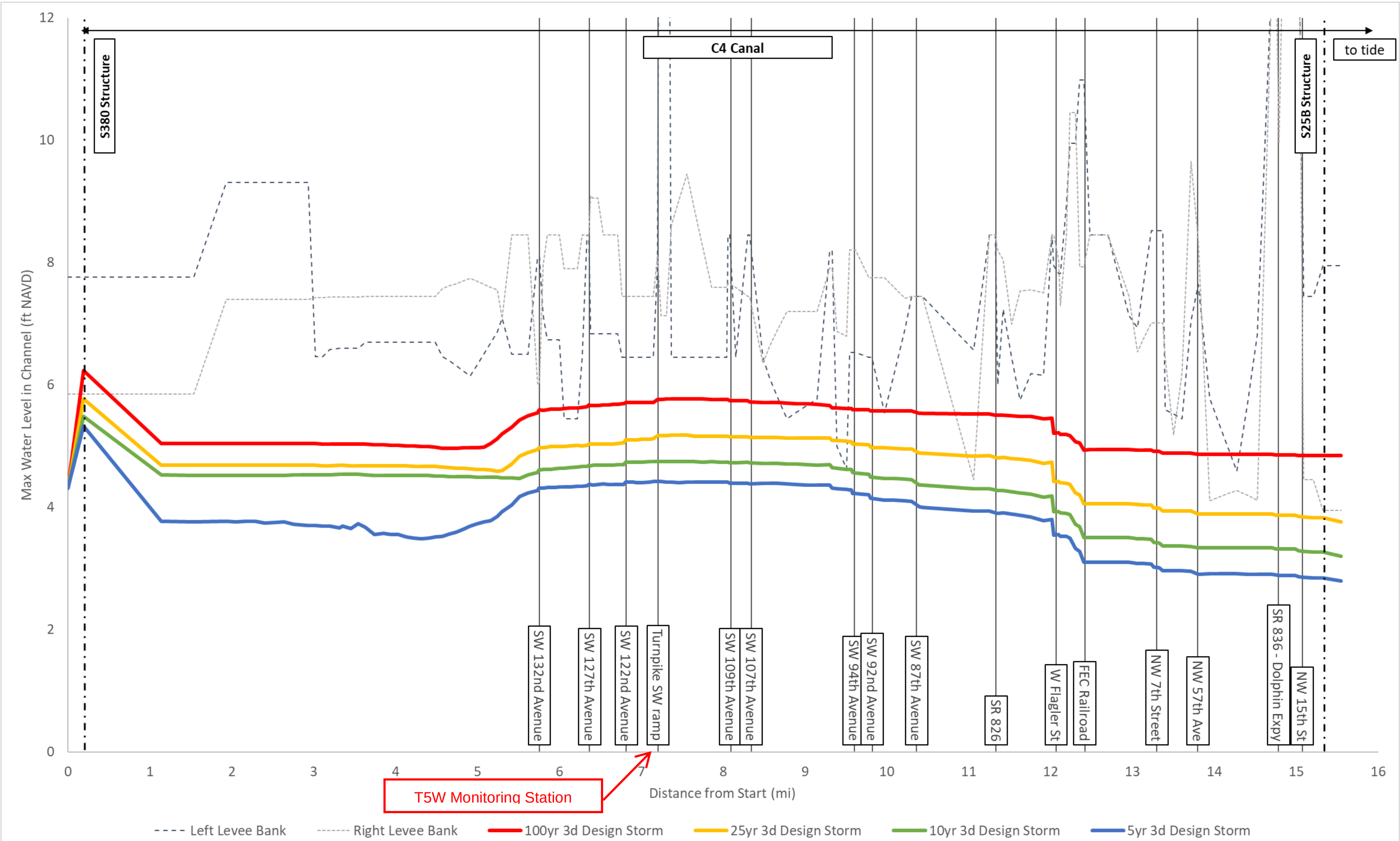


Figure 9-125. Maximum Stage Profile for the C4 Canal for the 100-year Design Storm for all Conditions

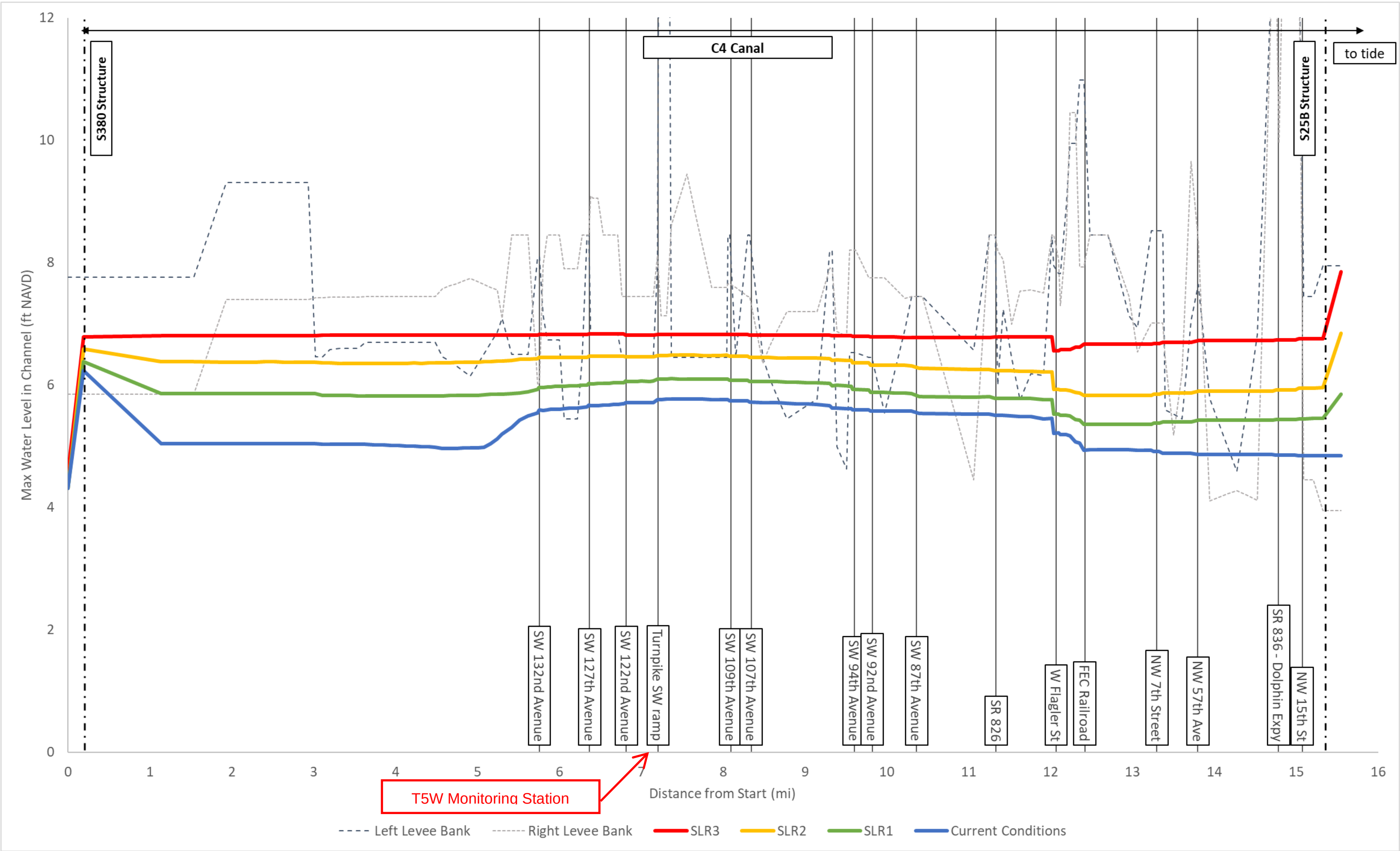
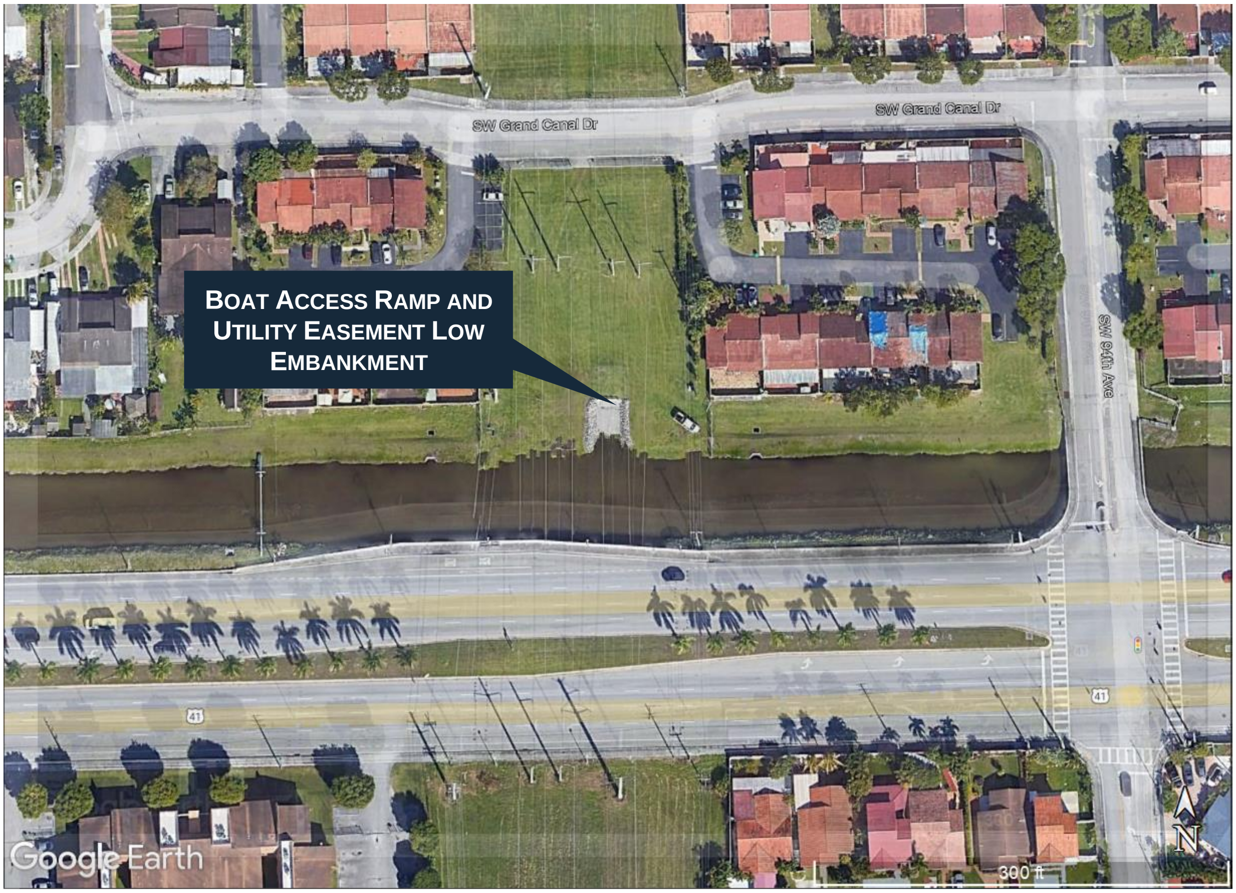


Figure 9-126. Boat Access Ramp along C4 Canal Upstream of SW 94th Avenue

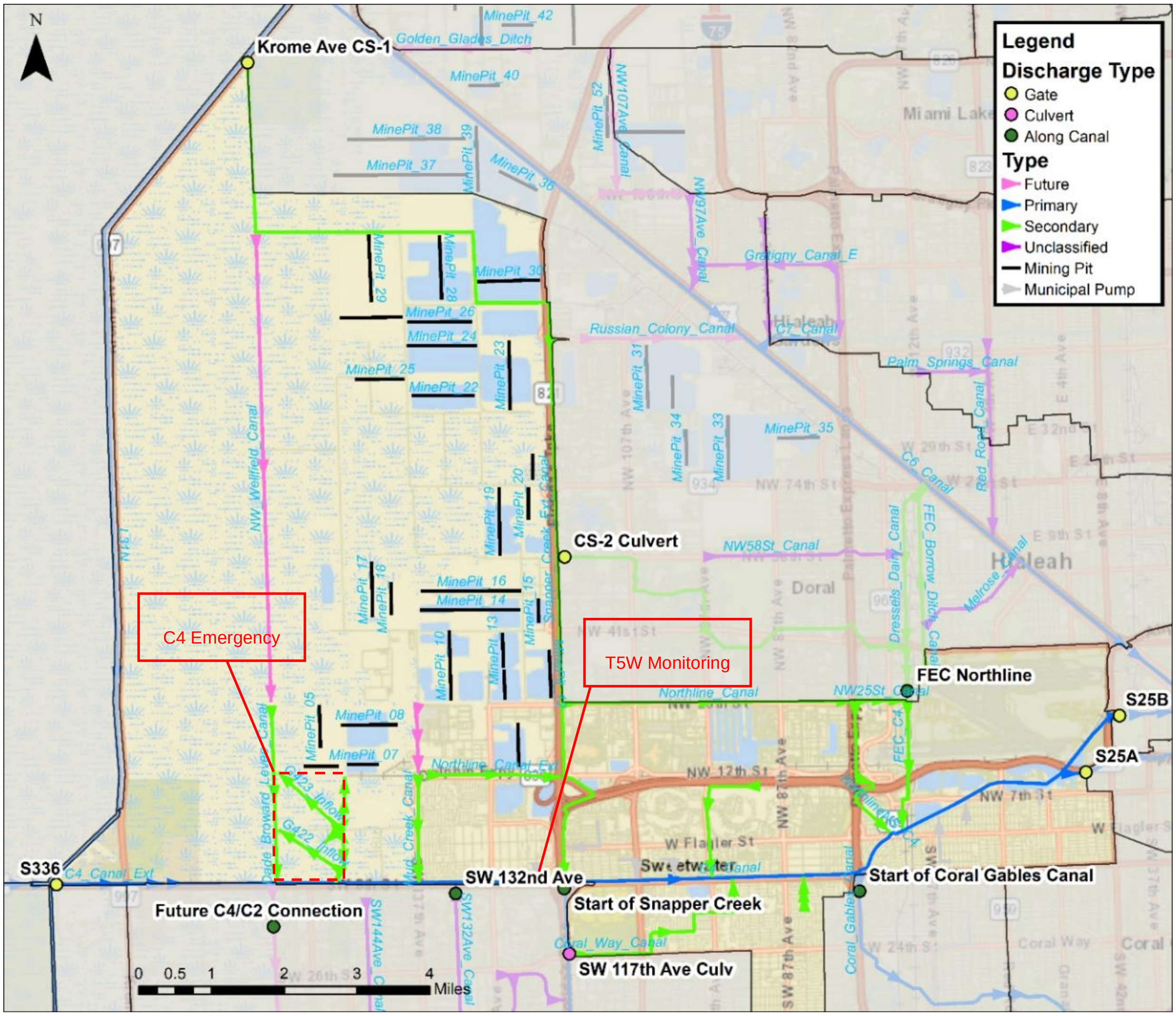


9.3.2. C4 – PM #2 MAXIMUM DISCHARGE CAPACITY

The maximum discharge capacity for the C4 Watershed is the sum of the discharges out of the watershed minus the incoming flows weighted by the total area of the watershed. For the C4 Watershed this means flows coming from the L31N Canal (CS-1 at Krome and S336) and from the C6 Watershed (at FEC Northline canal to the airport) are subtracted from the outflows at the S25B structure and flows discharging to the C2 watershed (at the intersection with Snapper Creek Canal, NW 132nd Avenue, and the future connection at NW 157th Avenue), to the C3W Watershed (at the intersection with Coral Gables Canal), and to the C6 Canal at the CS-2 structure. These locations are shown in **Figure 9-127**. In the figure, green dots indicate areas where there are no structures in the canal and pink dots indicate culverts with no gate controls, yet flows are crossing into the watershed at these locations and must be considered as part of the water budgeting.

No flows cross from the C4 Watershed to the C6 Watershed via the Dressels Dairy Canal West as the DERM CS-2 culvert is considered closed during design storm simulations.

Figure 9-127. Discharge Locations in the C4 Watershed



The timeseries at each inflow and outflow location was extracted, and inflows were subtracted from the outflows and divided by the basin area in square miles (sq. mi.) at each timestep as shown in the equation below.

$$\frac{Outflows[cfs] - Inflows[cfs]}{Basin\ Area\ [sq.mi.]} = Discharge\ Capacity[CSM]$$

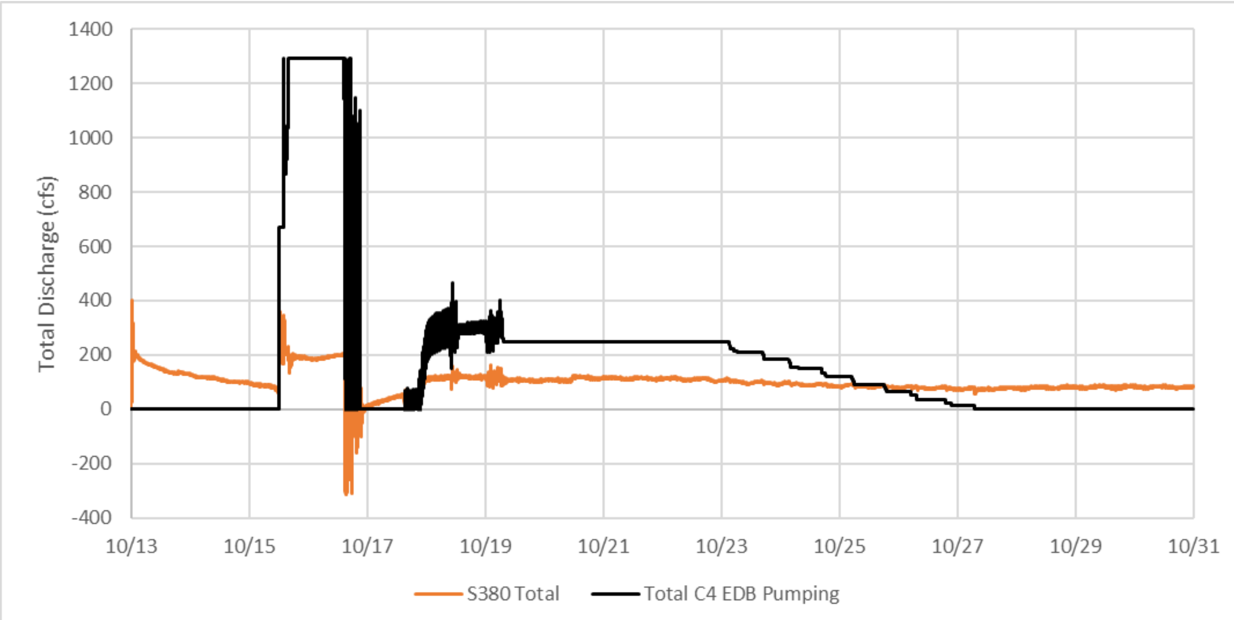
The maximum discharge capacity for the C4 Watershed for each design event is shown in **Table 9-44**. The table also provides discharge in cubic feet per second (CFS) at inflow and outflow locations for the watershed. Because the C4 Watershed is connected with each other watershed, the watershed connections are color coded to keep track of which watershed is exchanging with the C4 Watershed. Discharges are not the peak discharge at each location but rather the discharge at the peak watershed discharge capacity. A negative value at an inflow location means that the flow is leaving the watershed at the peak of the watershed discharge. A negative value at an outflow location means that the flow is entering the watershed at the peak of the watershed discharge.

The ERP Applicant's Handbook Volume II does not indicate an allowable discharge upstream of S25B for the C4 Watershed, and states that there is essentially unlimited inflow by gravity connections east of SW 87th Avenue.

The C4 Watershed contains a more complex system than the other watersheds (i.e., the C2, C3W, C5, and C6 Watersheds) due to the presence of the C4 Emergency Detention Basin (EDB), which pumps water from the C4 Canal when several conditions are met, such as higher water levels in the C4 Canal and if the Detention Basin is not full. **Figure 9-129** shows the discharge in the C4 Emergency Detention Ditch, downstream of both the G420 (669 CFS) and G422 (623 CFS) Pump Stations, which move water into the basin. In general, with increasing SLR, the detention basin seems to pump for longer periods of time and starts pumping earlier in the storm event. In fact, the SLR 2 and SLR3 storms start with both pumps on due to the higher water levels in the C4 Canal, but then decrease pumping before the storm as the Detention Basin reaches its operational depth. When the water levels in the canal shoot up during the peak, the pumps turn back on to fill up the Detention Basin to the emergency depth. However, since the basin is already fairly full, the emergency pumping is reduced or halted by the basin, maxing its capacity earlier in the storm event. The SLR1 scenario does not trigger this earlier pumping; and, therefore, the C4 Emergency Detention Basin will likely still provide sufficient emergency storage with one foot of sea level rise.

It should be noted that the assumptions for operations at S380 were to keep this structure fully open during the design storm events; however, this may not reflect the typical operations that have been implemented at this structure since the development of the C4 EDB, which is to keep the structure partly closed during pumping to maximize pumping from the urban areas and not out of the western portion of the C4 Canal. **Figure 9-128** shows the total flows at S380 compared with the total inflow into the C4 EDB during the 25-year storm event for Current Conditions. While pumping into the C4 EDB does appear to increase flows through S380 structure, this represents around 15% of the total flows into the C4 EDB.

Figure 9-128. Comparison of Flows at S380 and into the C4 Emergency Detention Basin during the Current Conditions 25-year Design Event



The design events also simulate the S336 structure operations according to the structure atlas, which allows this structure to open under low stage conditions at S22 and S25B; however, these triggers only allow the gate to open under lower intensity storms for the current conditions simulations, as shown in **Table 9-44**.

As with other watersheds, there is not a reduction in the discharge capacity of the structure with increasing SLR when looking at the peaks in **Table 9-44** or the instantaneous discharge in **Figure 9-129**. **Figure 9-130** shows the instantaneous discharge capacity in CSM for the C4 Watershed, for each SLR condition simulation for the 100- year, 25- year, 10- year, and 5-year design storm events. Multiple peaks are seen after the peak of the storm, showing the influence of tides, and pumping on discharge from the watershed. To remove the tidal influence, **Figure 9-131** shows the 12-hour moving average discharge capacity in CSM for all SLR simulations and each design storm event. This figure clearly shows that the capacity of the watershed is diminished with increasing SLR, if the tidal and pumping peaks are averaged. However, even with the tidal effects removed, the watershed discharge capacity peaks a full day after the peak of the storm for the SLR conditions.

Table 9-44. Peak Discharge (CFS/sq.mi.) from the Contributing Drainage Area of the C4 Watershed

	Current Conditions				SLR1				SLR2				SLR3			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Inflow Locations																
S336 Total Flow (CFS)	0.0	-39.7	-666.7	-661.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CS-1 Culvert at Krome Ave (CFS)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
FEC Northline Canal upstream of NW 25th St (CFS)	-135.0	-58.5	-19.4	-10.4	-192.0	-82.3	-77.0	-69.9	-200.3	-155.5	-145.2	-47.1	-264.3	-274.2	-179.0	-89.8
SW 132nd Ave Canal at C4 Canal (CFS)	132.2	-12.4	54.2	71.4	108.2	7.6	3.0	63.0	62.6	51.2	83.6	-29.5	46.0	49.6	35.3	28.5
Coral Way Canal at SW 117th Ave (CFS)	-51.5	-30.1	1.3	-16.8	-61.9	-15.9	-39.6	-48.0	-44.6	-38.9	-52.7	-16.7	-43.9	-57.5	-49.1	-42.0
Outflow Locations																
CS-2 Culvert in Dressels Dairy Canal (CFS)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Start of Snapper Creek at C4 Canal (CFS)	50.6	195.6	204.0	232.6	86.7	383.1	317.3	142.5	330.8	226.8	136.8	309.1	314.7	238.5	228.7	209.0
Coral Gables Canal from C4 Canal (CFS)	155.3	61.1	40.7	50.9	134.6	110.3	66.2	95.9	127.3	111.1	112.3	82.5	125.4	116.6	112.3	71.0
S25B Total Flow (CFS)	2551.8	1197.9	1324.4	1280.2	2670.4	1506.9	1369.9	1567.5	2713.8	2126.1	1900.9	1236.7	2502.0	2175.6	1730.2	1499.7
S25A Total Flow (CFS)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1	3.8	0.0	0.0
Future Connection at SW 157th Ave (CFS)	0.0	0.0	0.0	0.0	-340.9	0.5	-34.4	-146.2	-161.9	-148.0	-279.1	29.6	-137.8	-154.5	-128.0	-111.2
Watershed Summary																
Basin Area (sq. mile)	83.3				83.3				83.3				83.3			
Peak Watershed Discharge (CSM)	35.7	31.2	27.3	27.2	32.4	25.1	22.0	20.6	38.3	29.5	23.8	21.0	36.8	32.0	25.6	21.3

Figure 9-129. Instantaneous Discharge at the C4 Emergency Detention Basin for 100 year -, 25- year, 10- year, and 5-year 3-day Design Storm Events

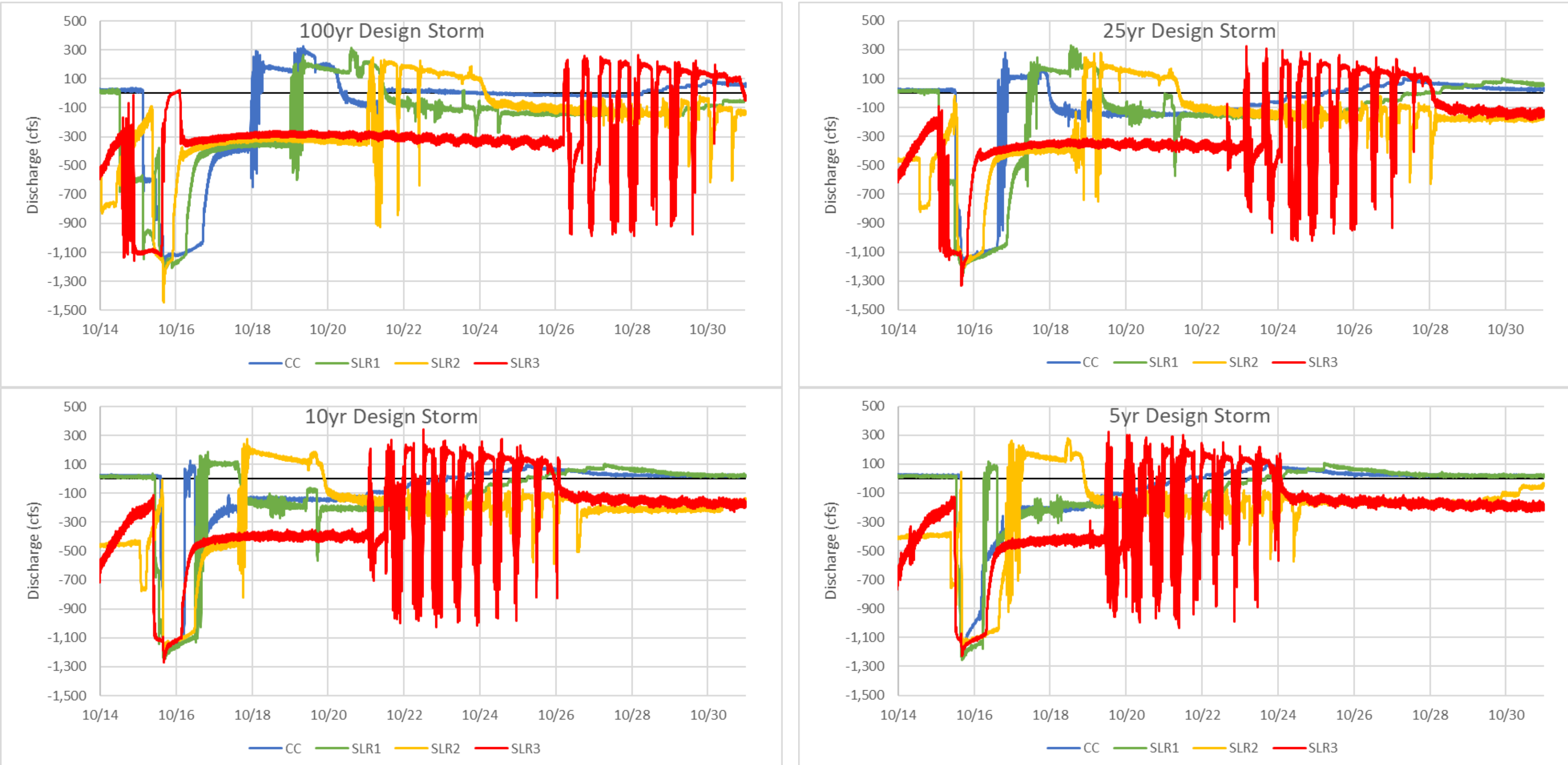


Figure 9-130. Instantaneous Discharge Capacity for the C4 Watershed for 100- year, 25- year, 10- year, and 5-year 3-day Design Storm Events

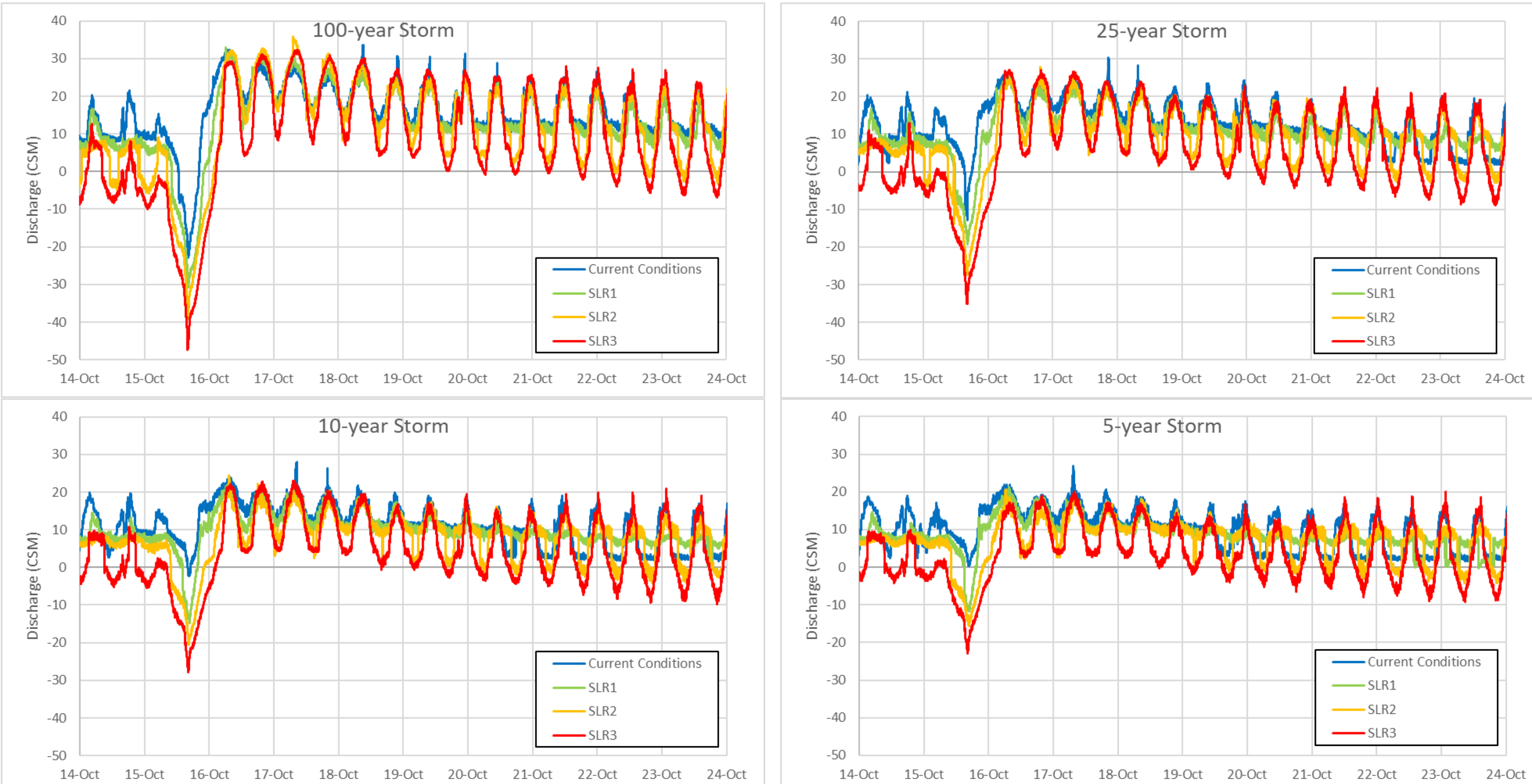
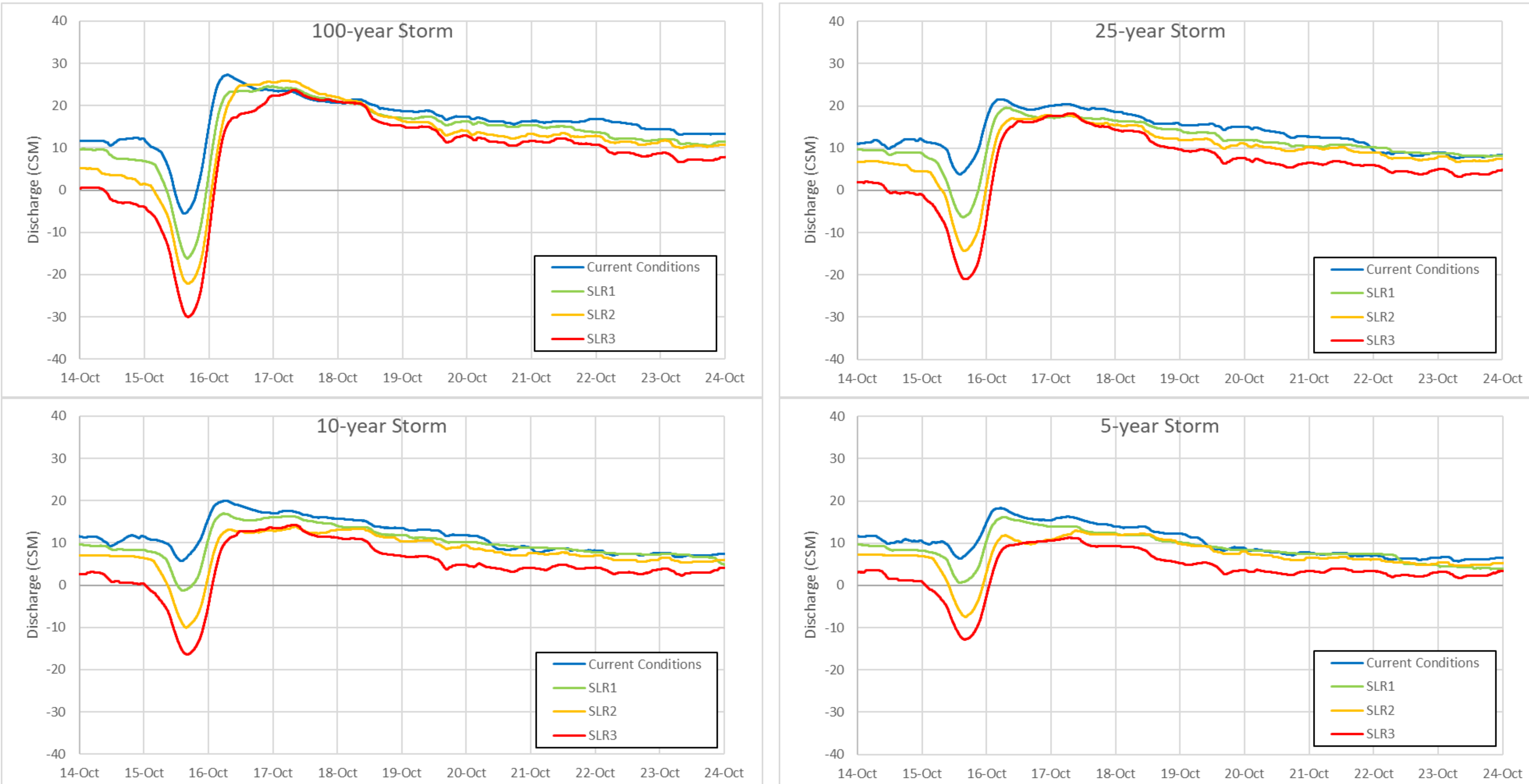


Figure 9-131. 12-Hour Moving Average Discharge Capacity for the C4 Watershed for 100- year, 25- year, 10- year, and 5-year 3-day Design Storm Events



9.3.2.1. INTER-BASIN TRANSFERS

The C4 Watershed is unique among the watersheds in this study, in that it is geographically located in the center and it exchanges with all the other watersheds in this study. The following sections will summarize the interactions between the C4 and the C2, C3W, C5, and C6 watersheds, as explained in detail in the other chapters.

9.3.2.1.1. EXCHANGE WITH THE C2 WATERSHED

During wet season operations, the C2 Watershed receives flows from the C4 Canal at the intersection of the C4 and the Snapper Creek Canal (or SW 8^{th} Street and Florida Turnpike), as well as flows from the Coral Way Canal (at SW 117^{th} Avenue). Normally, wet season flows move from the C4 Watershed into the C2 Watershed or are near zero. However, during the storm event, flows reverse and move from the C2 into the C4 during the peak of the storm. **Figure 9-14** and **Figure 9-15** show the inter-basin transfers for the 100-year and 25-year design storms, respectively.

9.3.2.1.2. EXCHANGE WITH THE C3W WATERSHED

During wet season operations, the C3W Watershed interacts primarily with the C4 Watershed, receiving flows from the C4 Canal at the intersection of the C4 and the Coral Gables Canal (or SW 8^{th} Street and the Palmetto Expressway). Normally, wet season flows move from the C4 Watershed into the C3W Watershed. However, during the storm event, flows reverse and move from the C3W into the C4 during the peak of the storm. During this flow reversal spike, both the G422 Pump Station and the G420 Pump Station are pumping at maximum capacity. Discharge at the start of the Coral Gables Canal is shown in **Figure 9-74** and **Figure 9-75** for the 100-year and 25-year design storms, respectively. Negative flows in these graphs indicate flows are moving into the C4 Watershed.

9.3.2.1.3. EXCHANGE WITH THE C5 WATERSHED

During wet season operations, the C5 Watershed connection with the C4 Watershed at S25A is closed. However, during some of the future SLR conditions, water may exceed the top of the S25A structure, which will act as a weir and flows will exchange between the C5 and C4 Watersheds. The top of this structure is 4.95 ft-NAVD. **Figure 9-194** and **Figure 9-195** show the exchanges between the C5 and C4 Watersheds at the S25A structure for the 100-year and 25-year design storms, respectively. For most of the SLR conditions, the flow tends to move from the C5 into the C4 Watershed, shown as negative flows at S25A. However, during the SLR3 100-year storm, the flows exchange back and forth from both watersheds (or oscillates between negative and positive). These positive flows indicate moments when the stages in the C4 Canal are higher than the Comfort Canal Southfork downstream of S25A and are overtopping the structure.

9.3.2.1.4. EXCHANGE WITH THE C6WATERSHED

Flows between the C6 and C4 Watershed are primarily at the FEC Northline Canal as shown in blue in **Figure 9-256** for the 100-year design storm and in **Figure 9-257** for the 25-year design storm. This is because the CS-2 structure in Dressels Dairy Canal West is closed in the wet season, so flows do not connect from the Snapper Creek Extension to the C6 Watershed. Flows at FEC Northline remain negative for most of the storm event (which indicates water is moving north from the C4 Watershed to the C6 Watershed). The 25-year simulation, **Figure 9-257**, indicates that flows return to near zero or even to positive after the storm subsides (which indicates water flowing south to the C4 Watershed). For the future SLR conditions, the maximum flow into the C6 Watershed do not change significantly; however, the timing and duration of these flow changes, with flows into the C6 lasting longer with each increasing SLR condition.

9.3.3. C4 – PM #3 STRUCTURE PERFORMANCE

The purpose of PM #3 is to determine the effect of SLR on tidal structure flow performance. For this metric, an evaluation based on structure design features and existing operational protocols was conducted for existing conditions and compared to simulations including SLR over the various storm events. PM #3 will provide only gravity flows through the structure and pumping, where PM #4 will also account for structure flows as well as overtopping, to differentiate between structure capacity and the basic inabilities of the structure to provide tidal protections.

As per the structure data sheet, the S25B tidal gravity structure (S25B_S) controls flow from the C4 Canal to the C6 Canal downstream of S26. S25B_S is operated with the intention to maintain water levels in the C4 Canal and pass the standard project flood without impacting upstream flooding. In addition, the structure is used to restrict flows to decrease stages and velocities that may cause damage to downstream areas, while preventing saline intrusion during high tides. **Table 9-45** provides the design parameters for S25B_S as provided in the Water Control Operations Atlas.

Table 9-45. Design Parameters for Structure S25B_S

Design Parameters	S25B_S
Design Discharge	2,000 CFS (100% SPF for easter C4 basin)
Design HW	2.85 ft-NAVD (4.4 ft-NGVD)
Design TW	2.55 ft-NAVD (4.1 ft-NGVD)
Optimum HW	1.25 ft-NAVD (2.8 ft-NGVD)
Optimum TW	Tidal
Maximum Gate Opening	11.9 ft
Water Level which will Bypass Structure	4.15 ft-NAVD (5.7 ft-NGVD)
Water Level which will Overtop Gates when Closed	2.45 ft-NAVD (4.0 ft-NGVD)
Low Range Operational Trigger	-0.55 ft-NAVD (1.0 ft-NGVD) to 0.45 ft-NAVD (2.0 ft-NGVD)

To maintain discharges from the land side to the seaside of S25B_S when gravity capacity is limited, or the gates need to be closed due to the threat of saltwater intrusion, a 600CFS pump station (S25B_P) was added to the S25B spillway (S25B_S) as part of the Miami Dade County Flood Mitigation Program in 2002. As per the structure sheet, S25B_P allows additional discharge capacity during high tide or storm surge events when downstream water levels are elevated up to an elevation of 3.2 ft NAVD88 (4.75 ft NGVD) at gauge MRMS1 on the downstream side of the S25B structure.

Table 9-46. Design Parameters for Structure S25B_P

Design Parameters	S25B_P
Design Discharge Capacity Total	600 CFS (3 x 200 CFS pumps)
Minimum Low Water (HW) Elevation	-0.65 ft-NAVD (0.9 ft-NGVD)

Figure 9-132 and **Figure 9-133** provide the instantaneous discharge, HW and TW at S25B during the current conditions simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-134** and **Figure 9-135** provide the 12-hour moving average for the discharge, HW, and TW at S25B during the current conditions simulation for the 100-year and 25-year design events, respectively, which removes the influence of the tides. **Figure 9-136** and **Figure 9-137** provide the instantaneous discharge, HW and TW at S25B during the future conditions SLR1 simulation for the 100-year 3-day and 25-

year 3-day design events, respectively. **Figure 9-138** and **Figure 9-139** provide the 12-hour moving average for the discharge, HW, and TW at S25B during the future conditions SLR1 simulation for the 100-year and 25-year design events, respectively, which removes the influence of the tides. **Figure 9-140** and **Figure 9-141** provide the instantaneous discharge, HW and TW at S25B during the future conditions SLR2 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-142** and **Figure 9-143** provide the 12-hour moving average for the discharge, HW, and TW at S25B during the future conditions SLR2 simulation for the 100-year and 25-year design events, respectively. **Figure 9-144** and **Figure 9-145** provide the instantaneous discharge, HW and TW at S25B during the future conditions SLR3 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-146** and **Figure 9-147** provide the 12-hour moving average for the discharge, HW, and TW at S25B during the future conditions SLR3 simulation for the 100-year and 25-year design events, respectively. Figures are provided for instantaneous discharge and 12-hour moving discharge for all design storms and SLR simulations in **Appendix B**.

Figure 9-148, **Figure 9-149**, **Figure 9-150**, and **Figure 9-151** show the total instantaneous flow broken into flow from S25B_S and S25B_P for the 100-year design event for existing conditions, future SLR +1 foot, future SLR +2 feet, and future SLR +3 feet conditions, respectively. These figures show a decrease in pump use with increasing SLR conditions over 1 foot. According to structure operations, as further discussed in **Section 7.5**, S25B_P cannot operate unless the water level at MRMS1 is below 3.2 ft-NAVD. **Figure 9-278** shows the water surface elevations during current and future SLR conditions (SLR1, SLR2, SLR3) for the 100-year storm at MRMS1. The 100-year current conditions and future SLR1 scenario are only above the trigger level at MRMS1 during the peak of the storm event, where the water levels during the SLR3 scenario remain above the trigger level for a majority of the simulation, leaving S25B_P unable to activate.

When the gates are overtopped and backflow is occurring, there are some minor instances where the pump turns on, which is creating a short-circuiting effect of both pumping and gate overtopping. **Figure 9-152** provides an example of when the gate flow is reversed, and the pump is operating during the 10-year SLR +2 feet simulation. The pump operational logic was not input into the model with a control to stop pumping if gate overtopping occurs, as this was not strictly written into the structure operations sheet or C4 Basin Operation Plan (SFWMD, 2016).

Figure 9-132. Instantaneous Discharge and Stages at S25B for the 100-year 3-day Current Conditions Event

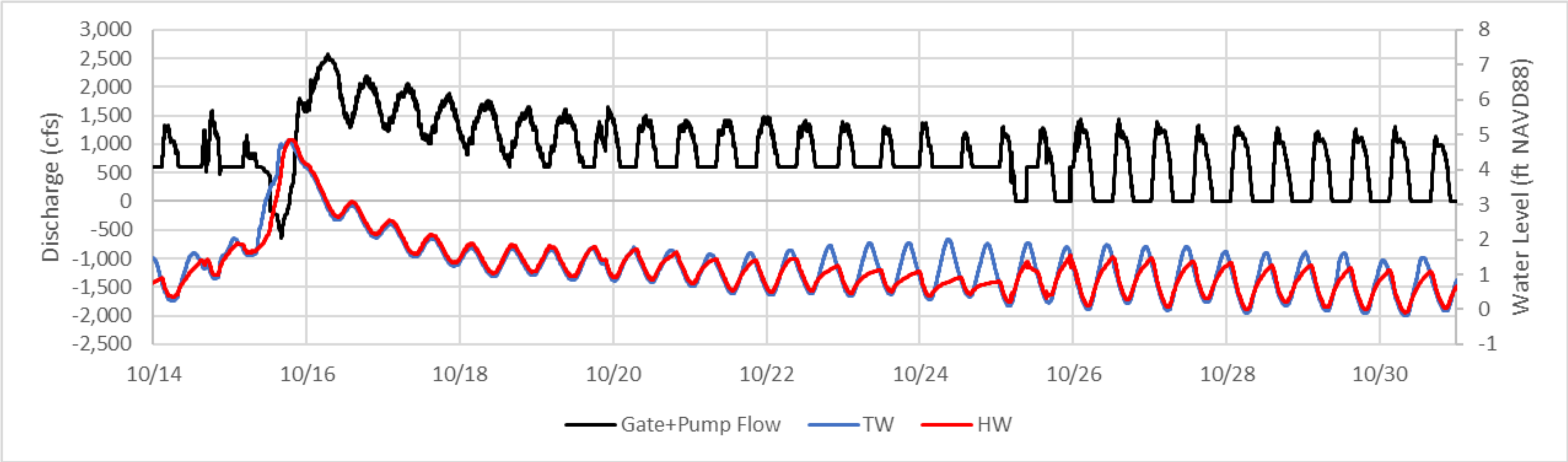


Figure 9-133. Instantaneous Discharge and Stages at S25B for the 25-year 3-day Current Conditions Event

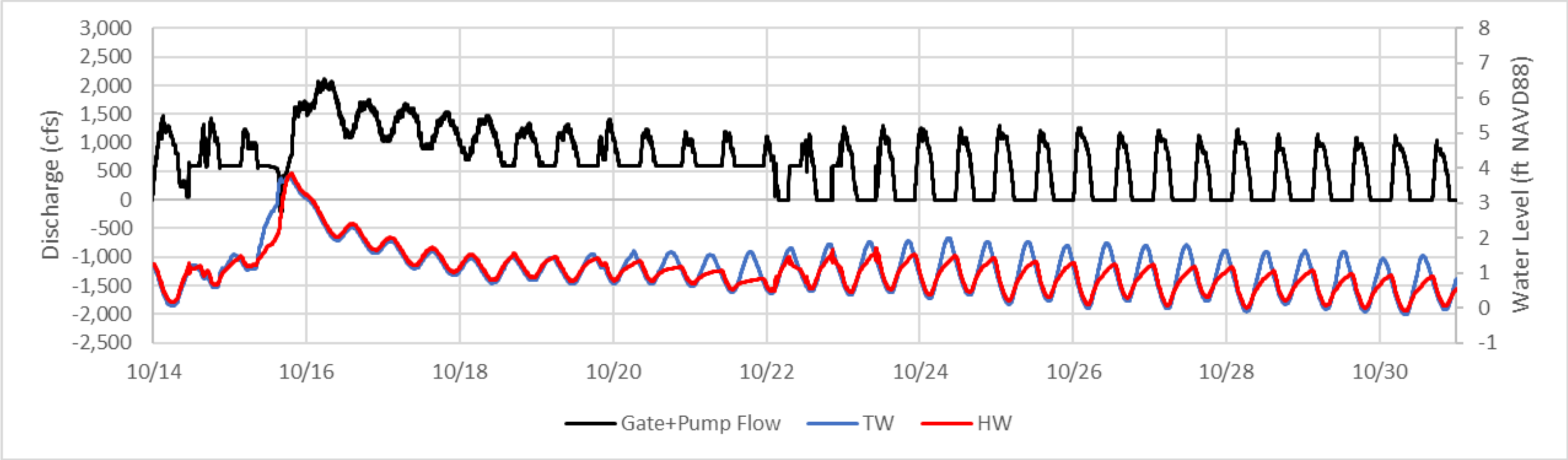


Figure 9-134. 12-Hour Moving Average Discharge and Stages at S25B for the 100-year 3-day Current Conditions Event

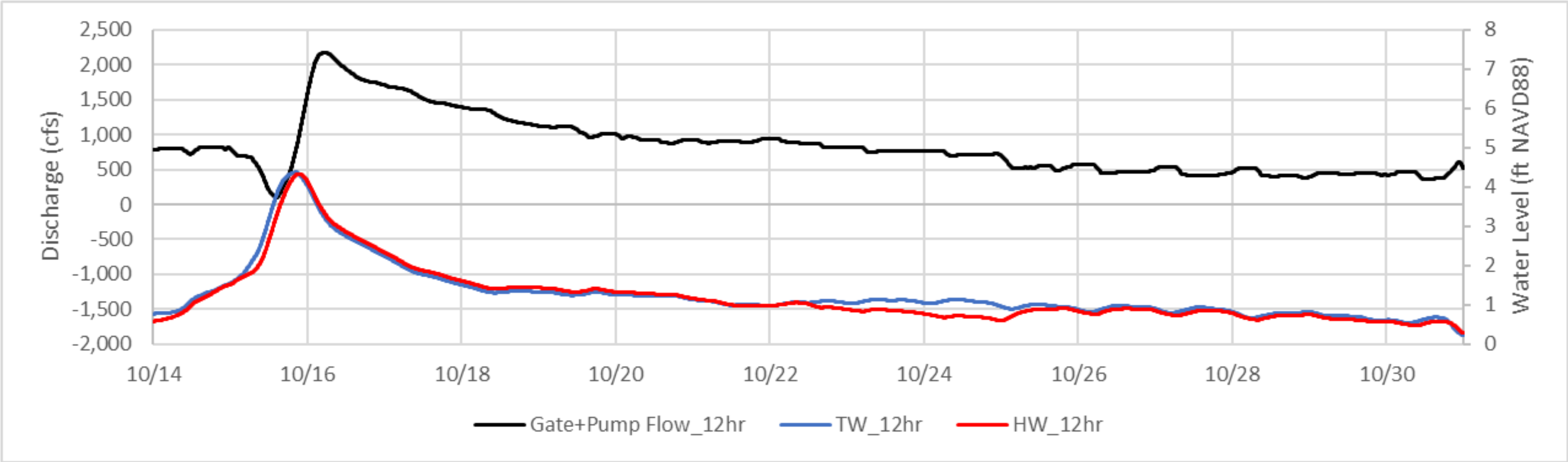


Figure 9-135. 12-Hour Moving Average Discharge and Stages at S25B for the 25-year 3-day Current Conditions Event

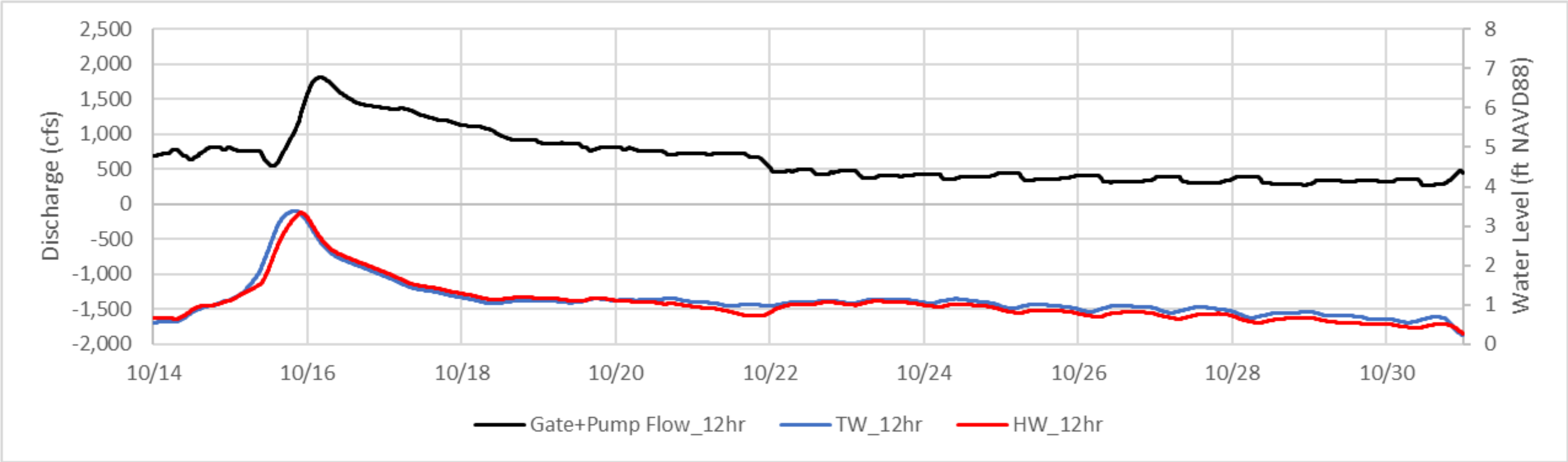


Figure 9-136. Instantaneous Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR1) Event

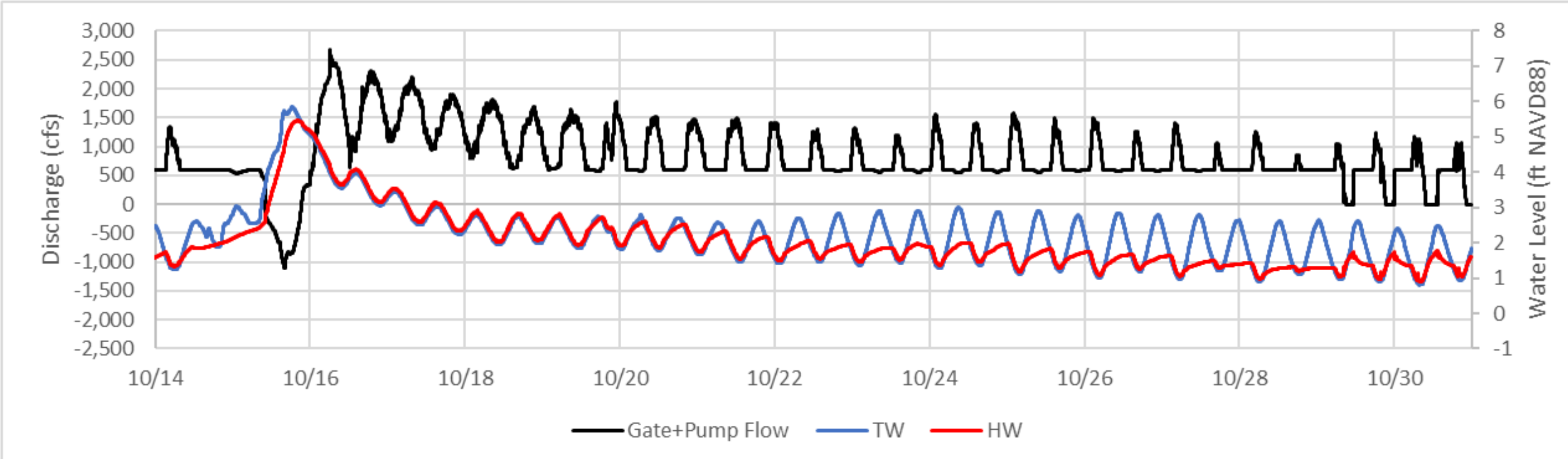


Figure 9-137. Instantaneous Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR1) Event

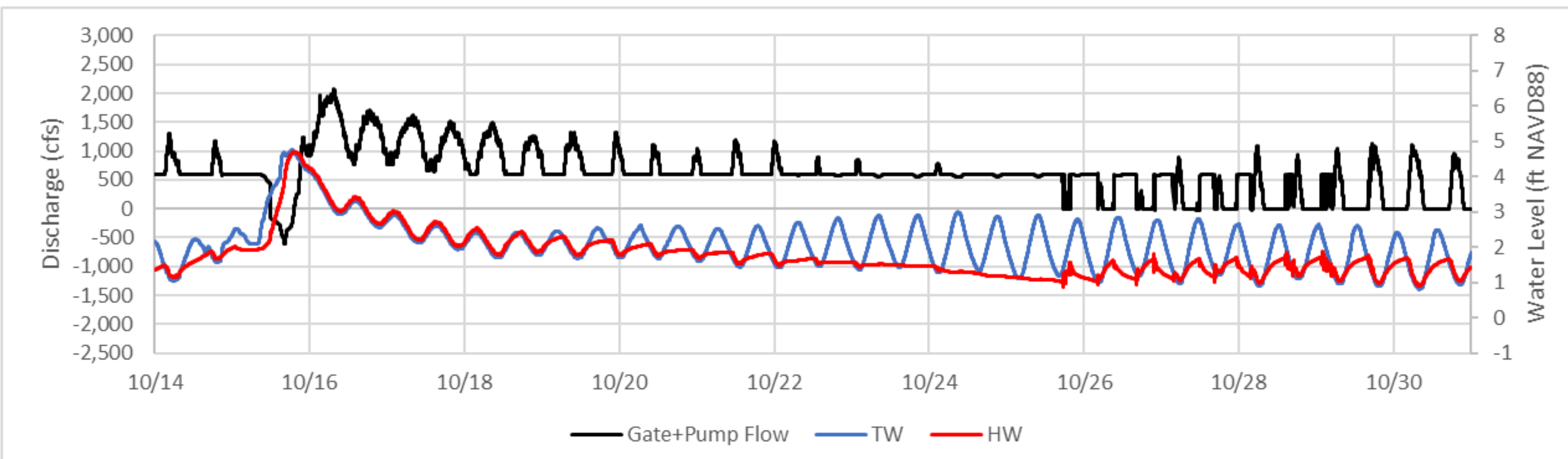


Figure 9-138. 12-Hour Moving Average Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR1) Event

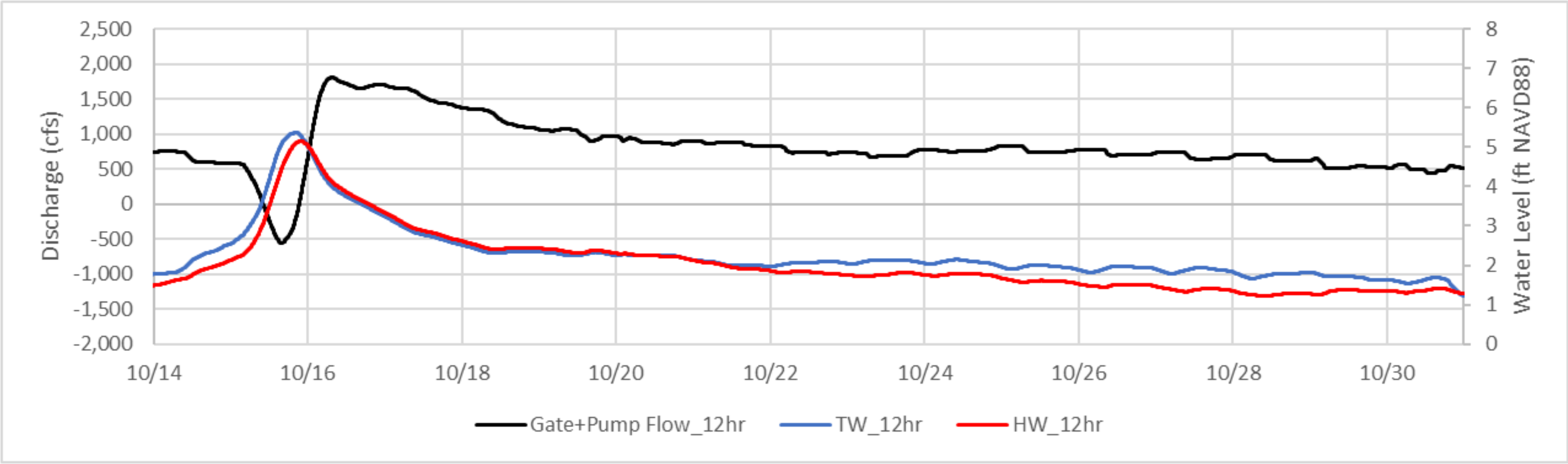


Figure 9-139. 12-Hour Moving Average Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR1) Event

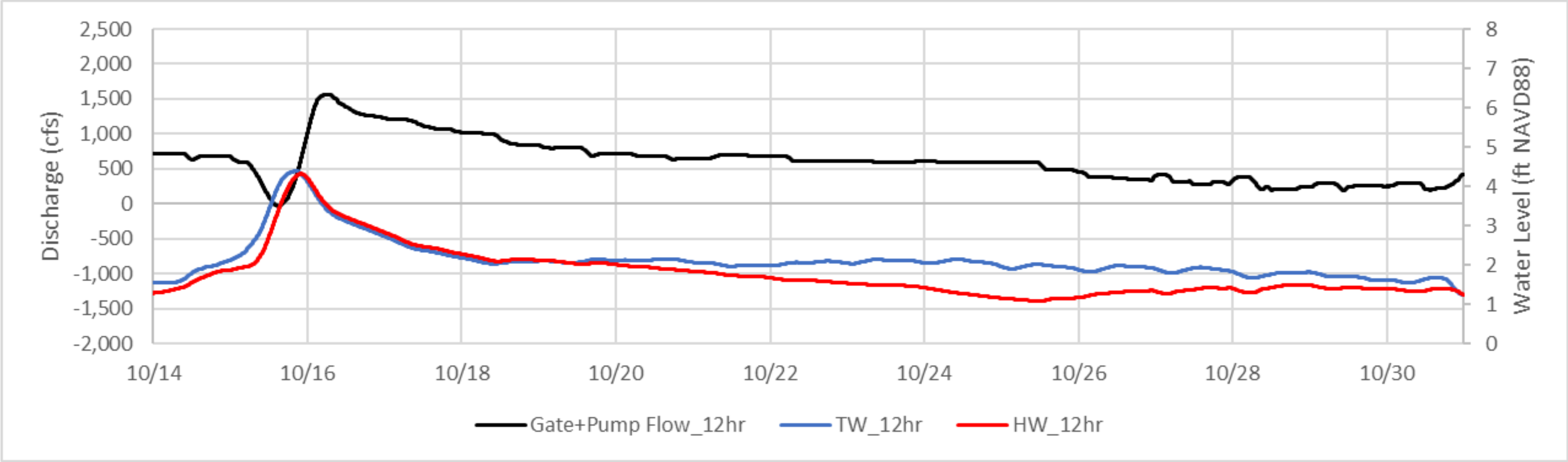


Figure 9-140. Instantaneous Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR2) Event

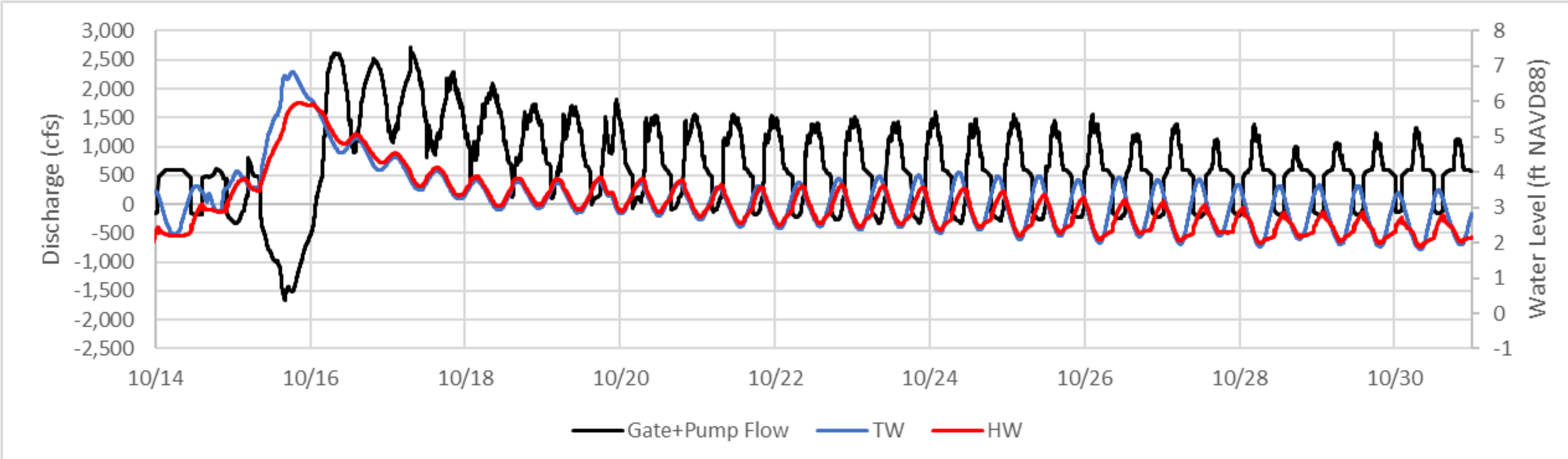


Figure 9-141. Instantaneous Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR2) Event

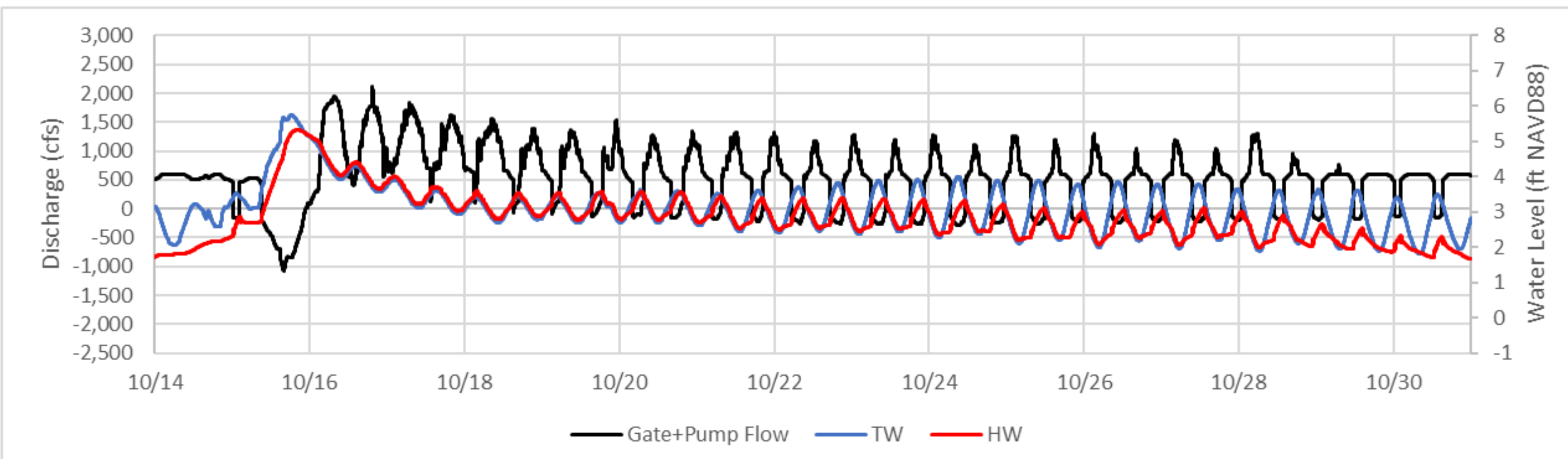


Figure 9-142. 12-Hour Moving Average Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR2) Event

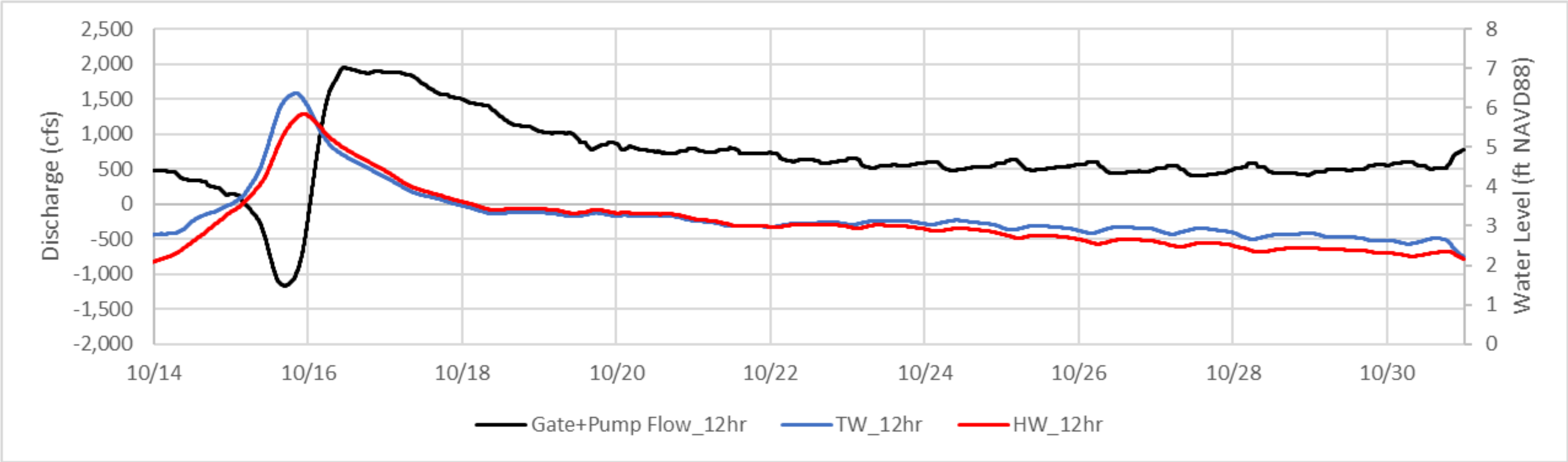


Figure 9-143. 12-Hour Moving Average Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR2) Event

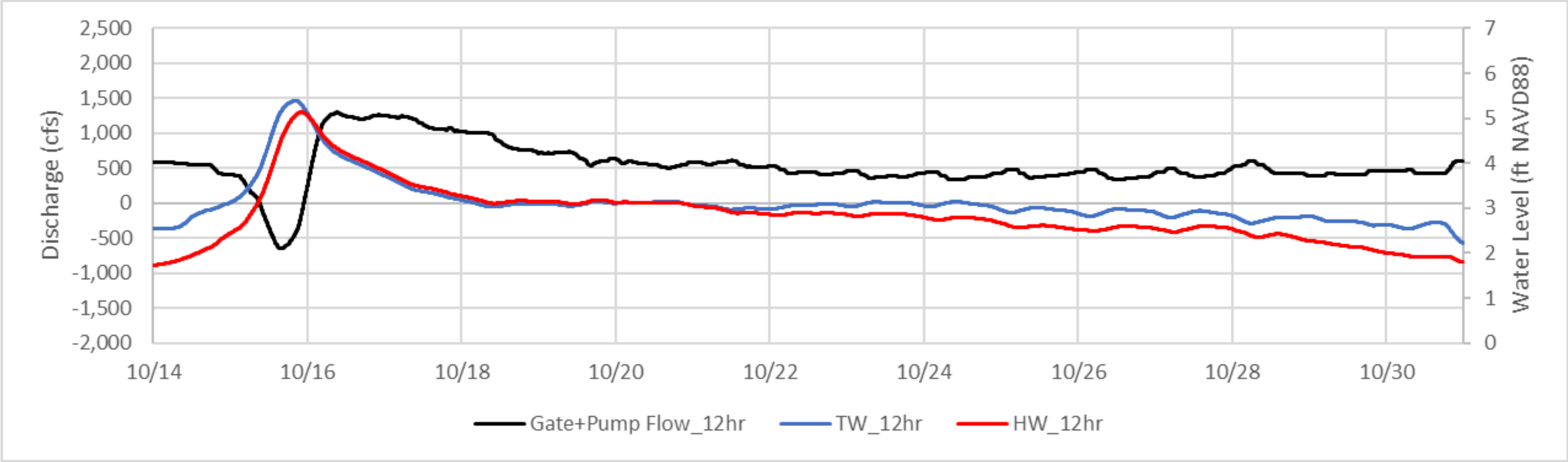


Figure 9-144. Instantaneous Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR3) Event

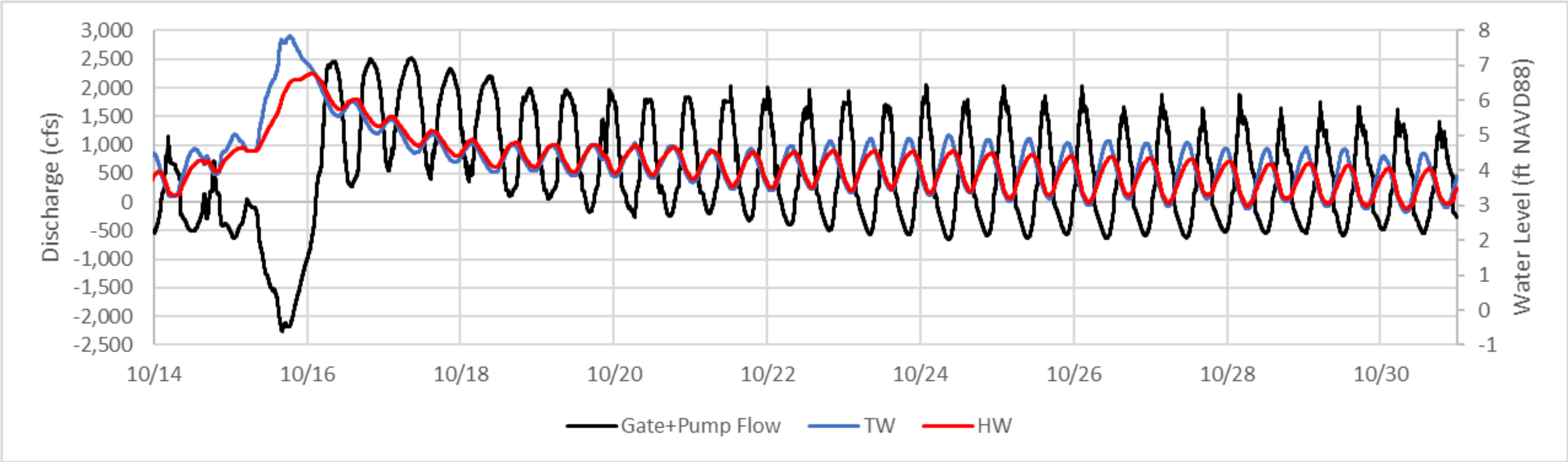


Figure 9-145. Instantaneous Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR3) Event

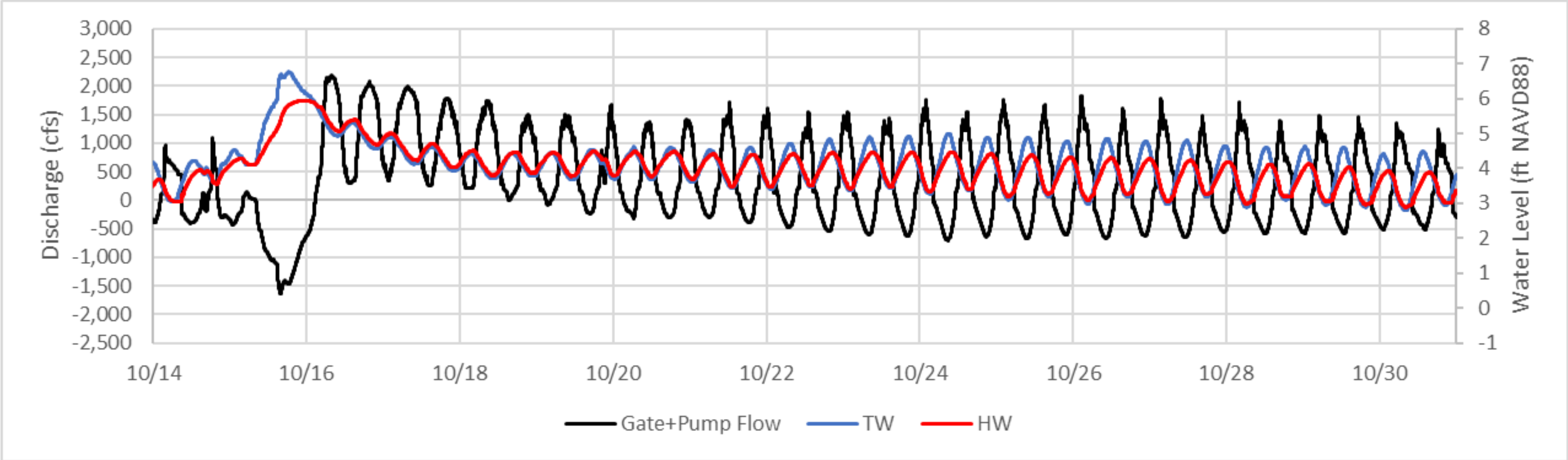


Figure 9-146. 12-Hour Moving Average Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR3) Event

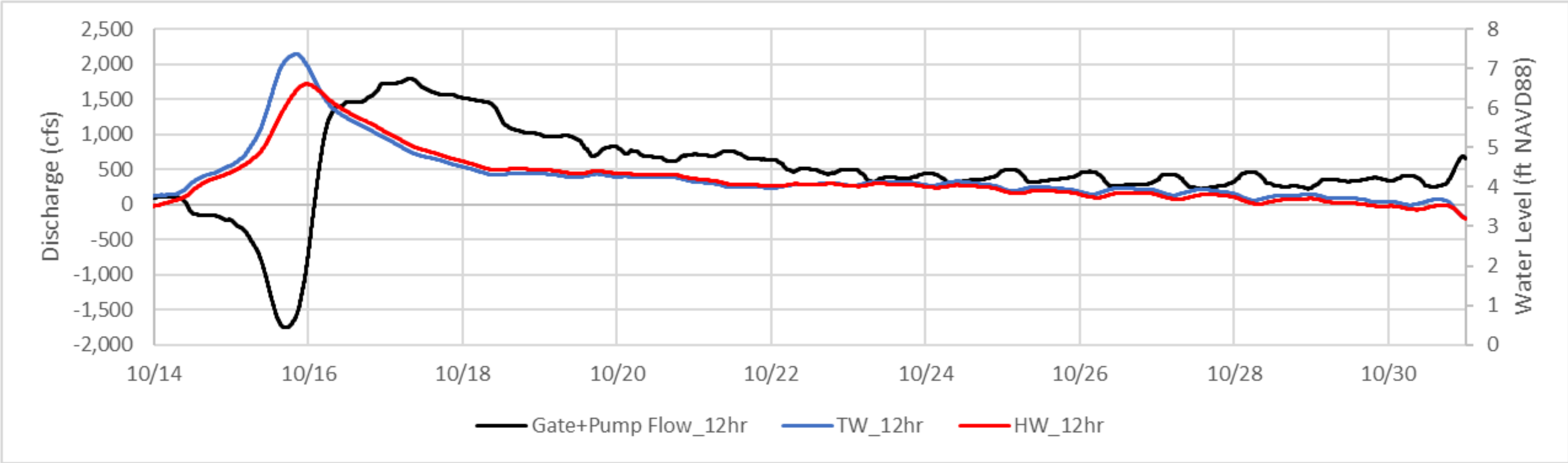


Figure 9-147. 12-Hour Moving Average Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR3) Event

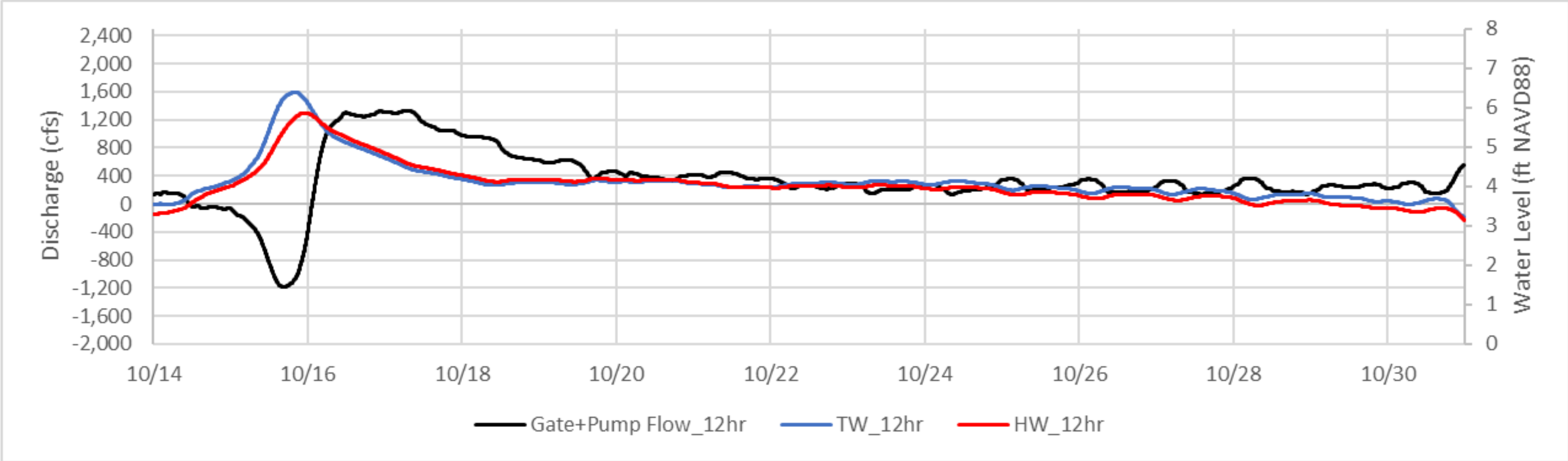


Figure 9-148. Total Instantaneous Flow at S25B for the 100-year 3-day Existing Conditions Event

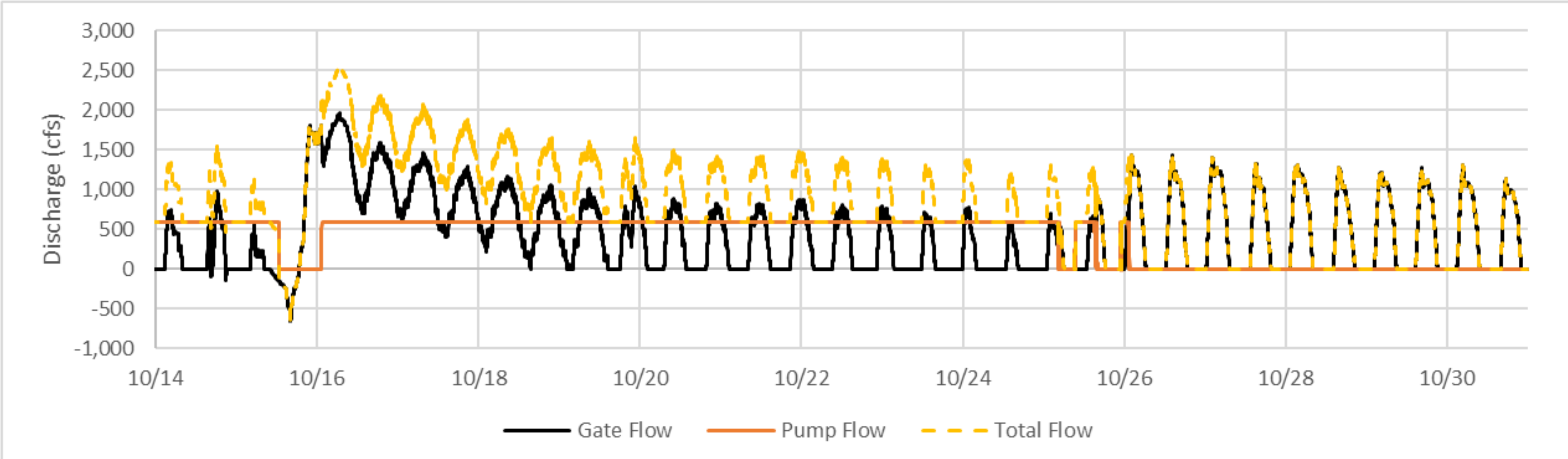


Figure 9-149. Total Instantaneous Flow at S25B for the 100-year 3-day Future Conditions (SLR1) Event

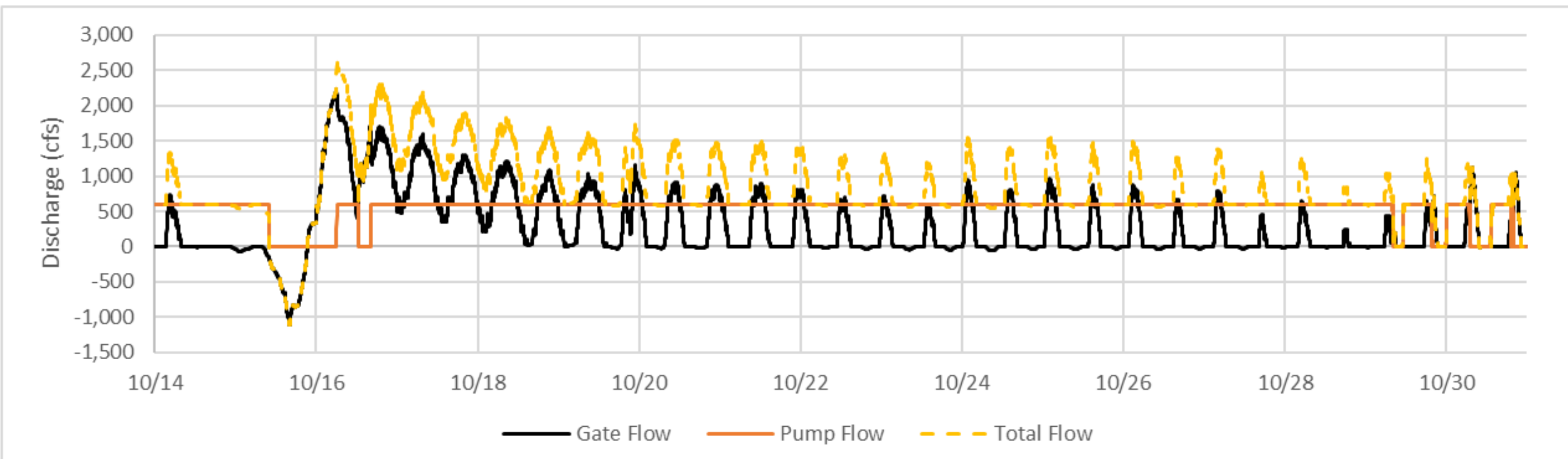


Figure 9-150. Total Instantaneous Flow at S25B for the 100-year 3-day Future Conditions (SLR2) Event

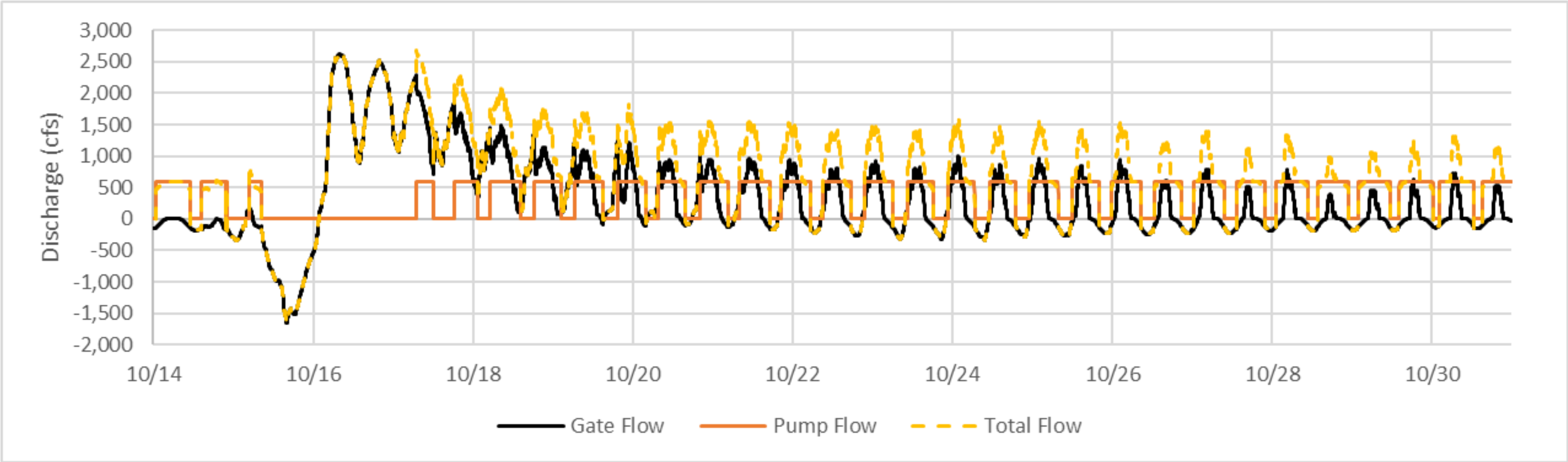


Figure 9-151. Total Instantaneous Flow at S25B for the 100-year 3-day Future Conditions (SLR3) Event

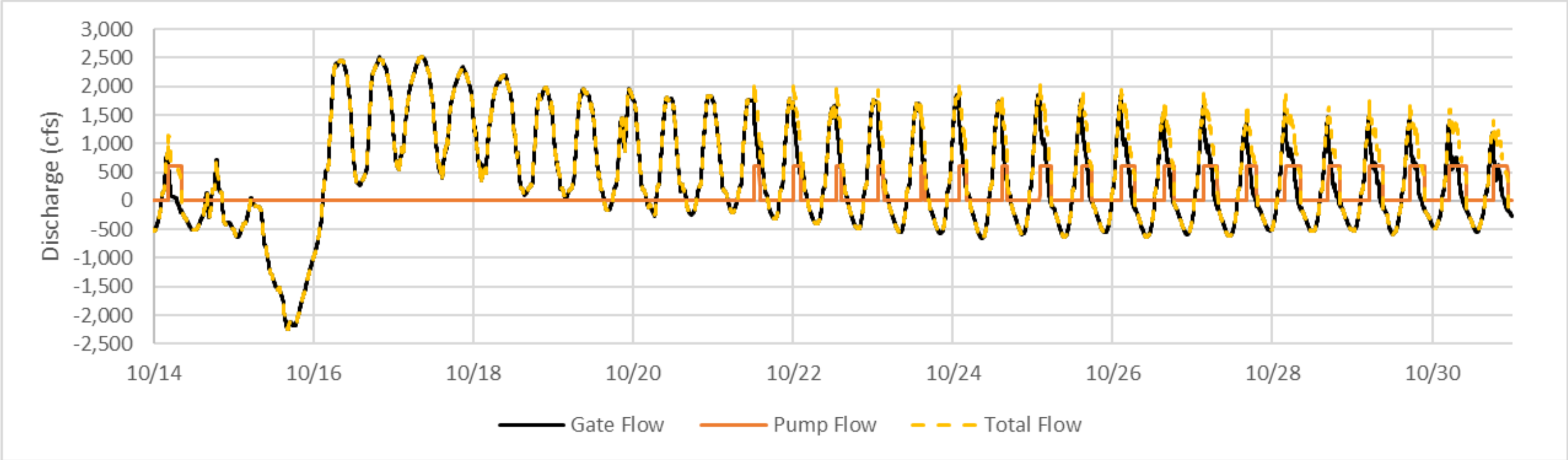
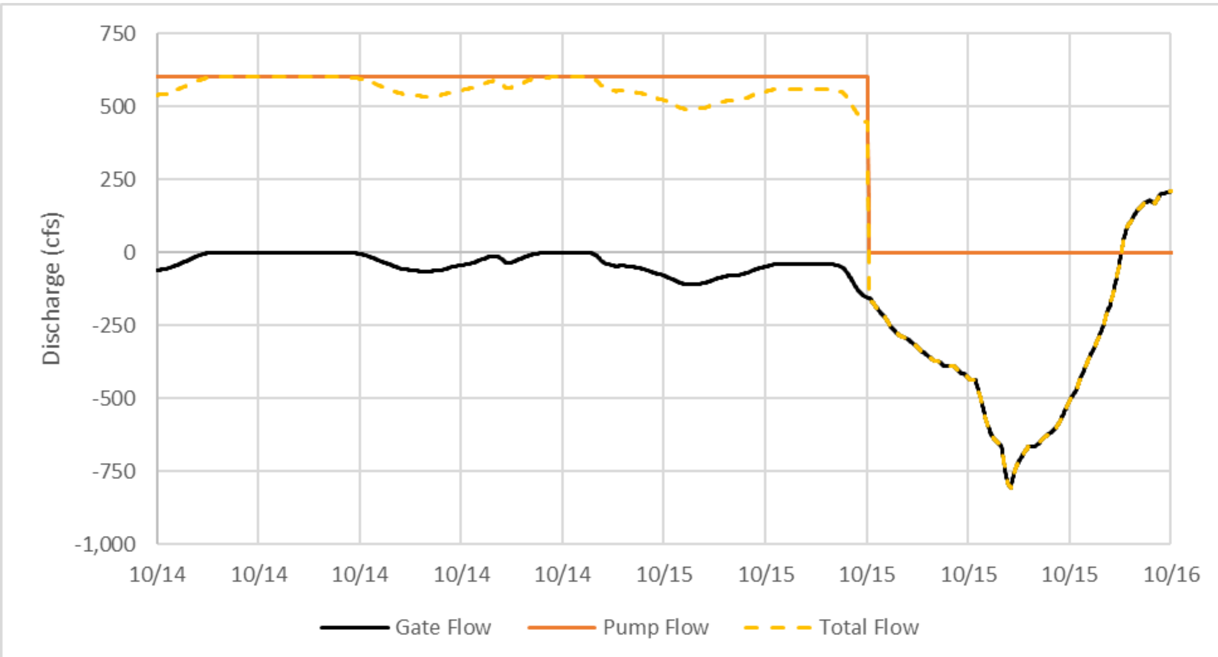


Figure 9-152. Flow Recirculation at the S25B Pump during 10-year 3-day Future Conditions (SLR2) Event



A summary of the instantaneous and 12-hour moving average conditions at the time of peak discharge at S25B are provided in **Table 9-47** and **Table 9-48**, respectively for all design storm return periods and future SLR conditions.

Table 9-47. Summary of Conditions at the Time of Peak Discharge at S25B (Instantaneous)

DESIGN STORM RETURN PERIOD	SCENARIO	TIME OF PEAK Q	PEAK Q (CFS)	HW AT PEAK Q (FT-NAVD)	TW AT PEAK Q (FT-NAVD)	ΔH AT PEAK Q (FT)	GATE OPENING AT PEAK Q* (FT)
100-Year	**Current**	10/16 6:44	2554.38	3.02	2.87	0.15	11.9
	SLR1	10/16 6:08	2670.40	4.17	4.00	0.17	11.9
	SLR2	10/17 7:07	2713.78	3.95	3.77	0.18	11.9
	SLR3	10/17 8:42	2516.10	4.83	4.56	0.28	11.9
25-Year	**Current**	10/16 4:03	2093.14	2.16	2.06	0.10	10.5
	SLR1	10/16 7:40	2079.69	3.16	3.06	0.10	10.5
	SLR2	10/16 19:29	2126.09	3.79	3.70	0.09	11.4
	SLR3	10/16 7:45	2177.79	5.24	5.05	0.19	11.9
10-Year	**Current**	10/16 6:45	1871.00	1.94	1.83	0.11	9
	SLR1	10/13 3:22	1927.70	0.75	0.63	0.12	9
	SLR2	10/16 7:24	1900.91	3.84	3.76	0.08	11
	SLR3	10/13 2:17	1844.53	3.22	3.11	0.12	8.5
5-Year	**Current**	10/16 7:42	1697.83	1.58	1.48	0.10	8
	SLR1	10/13 3:20	1924.72	0.73	0.61	0.12	9
	SLR2	10/13 3:26	1766.74	1.70	1.58	0.12	8
	SLR3	10/13 2:16	1836.91	3.17	3.07	0.10	9

*A gate opening of 11.9 ft represents the gate full open.

Table 9-48. Summary of Conditions at the Time of Peak Discharge at S25B (12 hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak Q	Peak Q (CFS)	HW at Peak Q (FT-NAVD)	TW at Peak Q (FT-NAVD)	ΔH at Peak Q (FT)	Gate Opening at Peak Q* (FT)
100-Year	**Current**	10/16 5:31	2158.51	3.32	3.21	0.11	11.24
	SLR1	10/16 7:35	1814.22	4.13	4.01	0.12	9.90
	SLR2	10/16 11:22	1951.85	4.97	4.79	0.19	10.90
	SLR3	10/17 7:50	1804.43	5.06	4.89	0.17	10.43
25-Year	**Current**	10/16 4:03	1801.79	2.71	2.61	0.10	8.75
	SLR1	10/16 5:57	1567.61	3.52	3.42	0.10	8.29
	SLR2	10/16 9:11	1296.73	4.33	4.22	0.11	8.84
	SLR3	10/17 6:31	1328.63	4.61	4.49	0.11	9.03
10-Year	**Current**	10/16 3:56	1577.74	2.32	2.22	0.10	7.26
	SLR1	10/16 5:25	1387.65	3.18	3.08	0.10	6.16
	SLR2	10/16 7:35	1016.28	4.04	3.94	0.10	6.37
	SLR3	10/17 6:55	1005.39	4.35	4.25	0.10	7.17
5-Year	**Current**	10/16 3:53	1435.60	2.03	1.93	0.10	6.31
	SLR1	10/16 5:22	1246.62	2.91	2.81	0.10	4.99
	SLR2	10/16 6:58	921.02	3.81	3.71	0.10	4.90
	SLR3	10/17 6:23	771.09	4.19	4.10	0.09	5.25

*A gate opening of 11.9 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak HW at S25B are provided in **Table 9-49** and **Table 9-50**, respectively for all design storm return periods and future SLR conditions.

Table 9-49. Summary of Conditions at the Time of Peak HW at S25B (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak HW	Q at Peak HW (CFS)	Peak HW (ft-NAVD)	TW at Peak HW (ft-NAVD)	ΔH at Peak HW (ft)	Gate Opening at Peak HW* (ft)
100-Year	**Current**	10/15 18:58	-112.93	4.79	4.81	-0.02	0
	SLR1	10/15 20:06	-491.09	5.45	5.62	-0.17	0
	SLR2	10/15 20:36	-1113.25	5.96	6.54	-0.58	0
	SLR3	10/16 1:19	-693.81	6.76	6.92	-0.17	0
25-Year	**Current**	10/15 22:08	750.38	3.79	3.67	0.13	1
	SLR1	10/15 18:59	-127.00	4.71	4.73	-0.03	0
	SLR2	10/15 20:12	-576.10	5.32	5.57	-0.25	0
	SLR3	10/15 22:20	-809.77	5.94	6.27	-0.34	0
10-Year	**Current**	10/15 19:55	802.97	3.24	3.08	0.16	1.5
	SLR1	10/15 19:40	220.20	4.26	4.11	0.15	1.5
	SLR2	10/15 19:59	-385.81	4.91	5.07	-0.16	0
	SLR3	10/15 23:37	-439.58	5.56	5.69	-0.13	0
5-Year	**Current**	10/15 19:52	653.53	2.82	2.69	0.12	0.5
	SLR1	10/15 19:37	272.28	3.88	3.73	0.15	2
	SLR2	10/15 20:32	-164.36	4.59	4.62	-0.04	0
	SLR3	10/15 20:10	-683.14	5.32	5.66	-0.35	0

*A gate opening of 11.9 ft represents the gate full open.

Table 9-50. Summary of Conditions at the Time of Peak HW at S25B (12 hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak HW	Q at Peak HW (CFS)	Peak HW (ft-NAVD)	TW at Peak HW (ft-NAVD)	ΔH at Peak HW (ft)	Gate Opening at Peak HW* (ft)
100-Year	Current	10/15 21:41	914.94	4.30	4.31	-0.01	6.21
	SLR1	10/15 21:54	121.49	5.15	5.29	-0.14	2.78
	SLR2	10/15 22:34	-635.95	5.84	6.22	-0.38	0.45
	SLR3	10/15 23:35	-865.46	6.61	7.10	-0.49	0.78
25-Year	Current	10/15 22:08	1249.91	3.32	3.31	0.01	5.09
	SLR1	10/15 22:01	646.65	4.32	4.32	0.00	4.98
	SLR2	10/15 22:12	-166.46	5.14	5.30	-0.16	0.82
	SLR3	10/15 23:10	-639.06	5.86	6.21	-0.35	0.36
10-Year	Current	10/15 22:37	1188.48	2.79	2.78	0.01	4.51
	SLR1	10/15 22:22	763.95	3.83	3.79	0.03	4.30
	SLR2	10/15 22:27	50.60	4.70	4.79	-0.09	1.50
	SLR3	10/15 22:45	-507.26	5.49	5.77	-0.28	0.09
5-Year	Current	10/15 22:43	1116.24	2.42	2.42	0.01	3.98
	SLR1	10/15 22:40	760.01	3.46	3.42	0.04	2.85
	SLR2	10/15 22:39	154.20	4.37	4.42	-0.05	1.86
	SLR3	10/15 22:35	-372.43	5.22	5.43	-0.20	0.17

*A gate opening of 11.9 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak TW at S25B are provided in **Table 9-51** and **Table 9-52**

Table 9-52, respectively for all design storm return periods and future SLR conditions.

Table 9-51 shows that there are instances when there are negative flows, yet the gate opening is zero, this indicates that the tailwater is higher than the top of the closed gate (2.45 ft-NAVD) and there is a negative head differential across this structure. However, negative flows can also occur when the gate is in the process of closing and the head differential is negative.

Since **Table 9-52** shows the averaged values over a 12-hour period, there may be some instances when the average head differential is negative, yet the flows are positive, or the flows are negative, yet the head differential is positive. These disagreements between flows and head differentials are due to the averaging of the values.

Table 9-51. Summary of Conditions at the Time of Peak Tailwater at S25B (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (FT-NAVD)	Peak TW (FT-NAVD)	ΔH at Peak TW (FT)	Gate Opening at Peak TW* (FT)
100-Year	**Current**	10/15 18:30	-282.83	4.75	4.85	-0.10	0
	SLR1	10/15 18:30	-846.49	5.36	5.85	-0.49	0
	SLR2	10/15 18:30	-1521.49	5.86	6.85	-0.98	0
	SLR3	10/15 18:30	-2177.24	6.50	7.85	-1.35	0
25-Year	**Current**	10/15 20:24	530.51	3.73	3.76	-0.02	0
	SLR1	10/15 18:30	-275.81	4.66	4.76	-0.10	0
	SLR2	10/15 18:30	-845.83	5.24	5.76	-0.52	0
	SLR3	10/15 18:30	-1464.47	5.81	6.76	-0.95	0
10-Year	**Current**	10/15 18:30	495.82	3.05	3.20	-0.16	0
	SLR1	10/15 18:30	-51.80	4.19	4.20	-0.01	0
	SLR2	10/15 18:30	-605.49	4.82	5.20	-0.38	0
	SLR3	10/15 18:30	-1118.64	5.48	6.20	-0.72	0
5-Year	**Current**	10/15 18:30	569.45	2.59	2.80	-0.21	0
	SLR1	10/15 18:30	-72.16	3.77	3.80	-0.02	0
	SLR2	10/15 18:30	-473.17	4.48	4.80	-0.32	0
	SLR3	10/15 18:30	-877.63	5.25	5.80	-0.55	0

*A gate opening of 11.9 ft represents the gate full open.

Table 9-52. Summary of Conditions at the Time of Peak Tailwater at S25B (12 hour Moving Average)

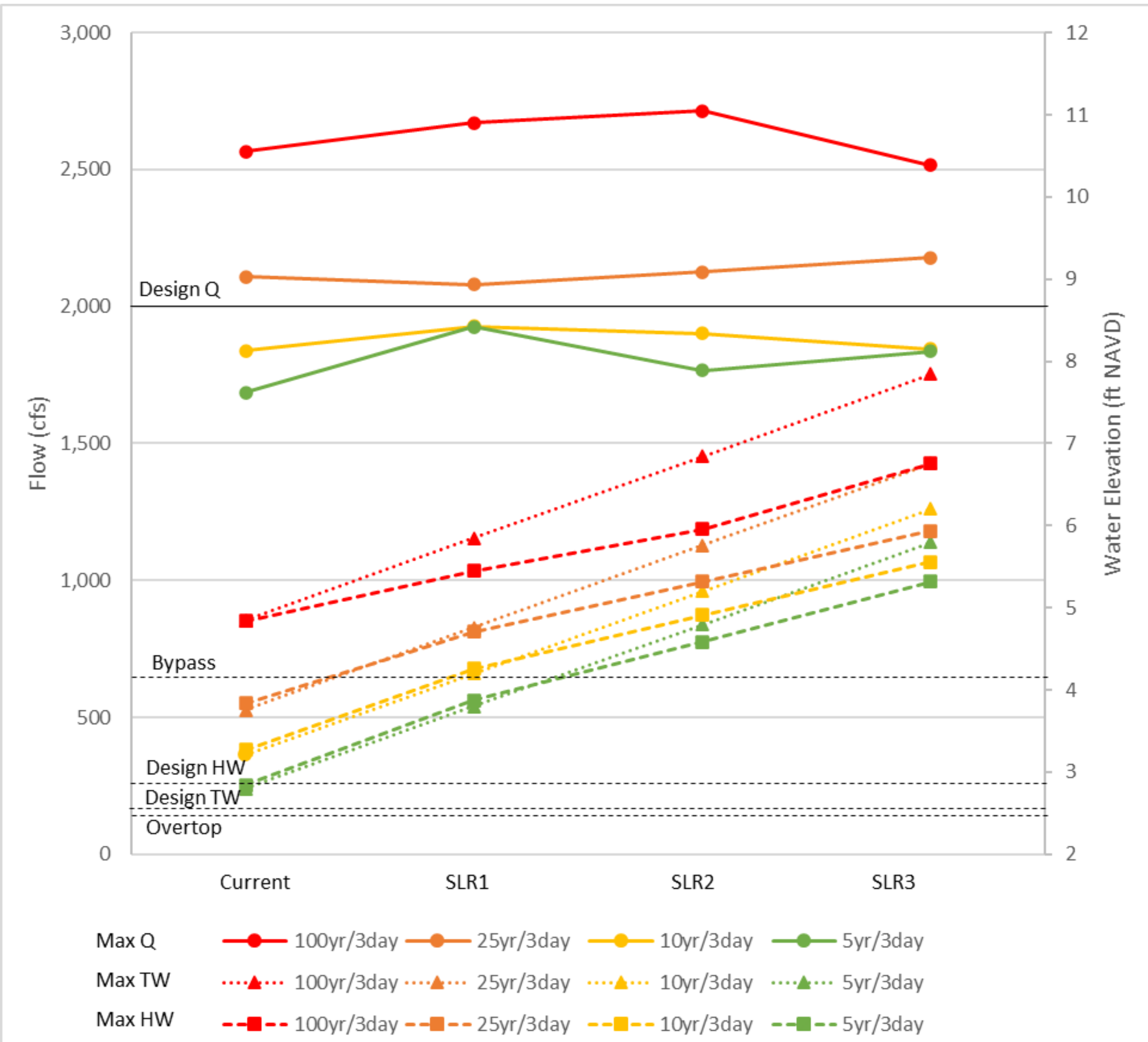
Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (ft-NAVD)	Peak TW (ft-NAVD)	ΔH at Peak TW (ft)	Gate Opening at Peak TW* (ft)
100-Year	**Current**	10/15 20:22	647.83	4.26	4.37	-0.11	5.04
	SLR1	10/15 20:22	-194.72	5.10	5.37	-0.27	1.34
	SLR2	10/15 20:22	-999.00	5.75	6.37	-0.62	0.00
	SLR3	10/15 20:22	-1581.67	6.45	7.37	-0.92	0.00
25-Year	**Current**	10/15 20:24	1019.29	3.23	3.39	-0.16	3.78
	SLR1	10/15 20:24	373.73	4.26	4.39	-0.13	3.66
	SLR2	10/15 20:24	-427.87	5.07	5.39	-0.32	0.01
	SLR3	10/15 20:24	-1055.55	5.76	6.39	-0.63	0.00
10-Year	**Current**	10/15 20:25	990.86	2.69	2.88	-0.20	3.12
	SLR1	10/15 20:25	490.61	3.72	3.88	-0.16	3.29
	SLR2	10/15 20:25	-211.34	4.61	4.88	-0.27	0.31
	SLR3	10/15 20:25	-779.24	5.42	5.88	-0.47	0.00
5-Year	**Current**	10/15 20:26	947.41	2.33	2.52	-0.19	2.73
	SLR1	10/15 20:26	526.48	3.32	3.52	-0.20	1.90
	SLR2	10/15 20:26	-101.13	4.26	4.52	-0.25	0.59
	SLR3	10/15 20:26	-594.61	5.16	5.52	-0.36	0.00

*A gate opening of 11.9 ft represents the gate full open.

Figure 9-153 shows a summary of the instantaneous peak discharge, HW, and TW at S25B for all design storm return periods and future SLR conditions. The design parameter values listed in **Table 9-45** are shown graphically in the figure below with bypass indicating the water level which will bypass the structure and overtop indicating the water level which will overtop the gates when the gates are closed. Note that the peak discharge, HW, and TW occur at different times for each scenario.

The maximum HW at S25B exceeds the design HW for all conditions except for the 5-year current conditions simulation. The maximum TW at S25B exceeds the design TW for all simulations. The maximum HW and TW also exceed the water level which will overtop the gates for all conditions, sometimes occurring when the gates are closed. TW elevations that exceed the overtopping elevation when the gates are closed can result in flow entering the basin from storm surge and/or tide. During the peak of the storm, flow overtops the gate and flows into the basin for all future condition simulations. The amount of flow that enters the basin from storm surge increases with the design storm return period and amount of SLR. The HW at S25B exceeds the water level that will bypass the structure for the 100-year and 25-year current conditions scenarios and all future SLR scenarios (SLR1, SLR2, SLR3) except for the 5-year SLR1 scenario. The TW also exceeds this bypass elevation during the 100-year current conditions scenario and all future SLR scenarios (SLR1, SLR2, SLR3) except for the 5-year SLR1 scenario. Flow that bypasses the structure can contribute to flooding of neighborhoods around S25B.

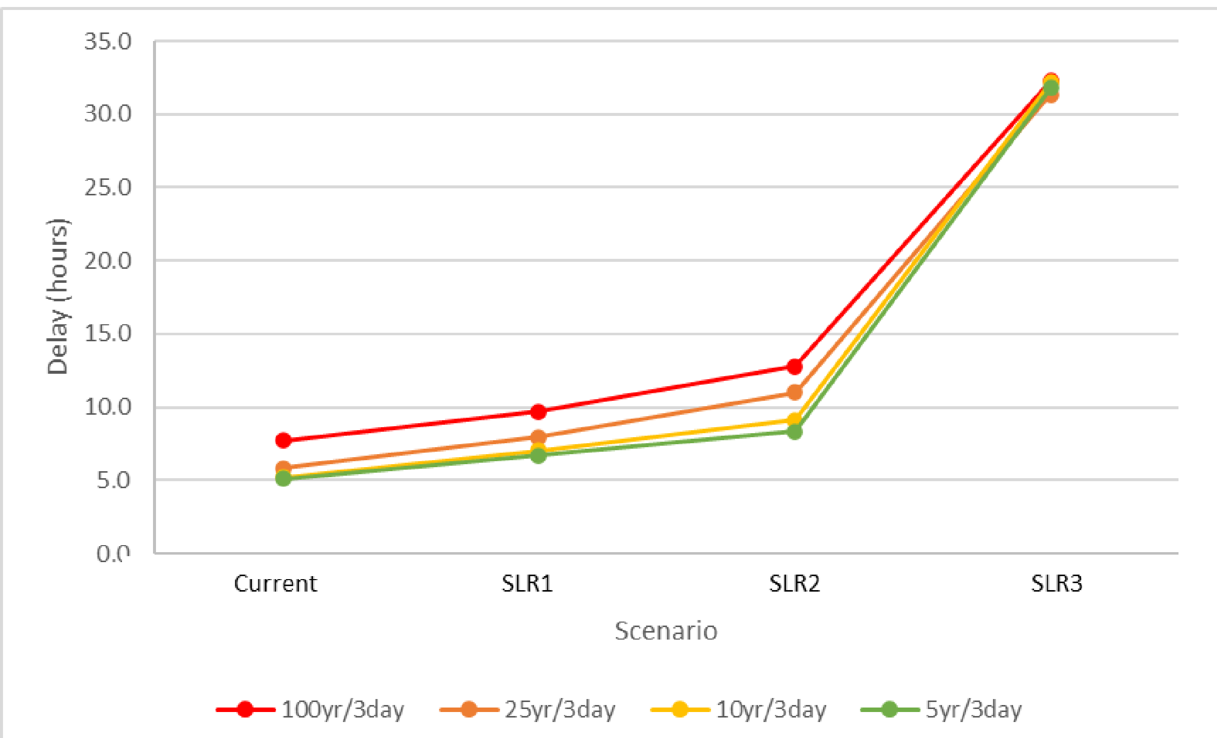
Figure 9-153. Summary of Instantaneous Max Discharge, HW, and TW at S25B



The peak discharges falls below the design discharge of 2,000 CFS for all simulations with the 5-year and 10-year return period storms and does not seem to be affected by the SLR scenarios explored in this report.

Without TW limitations, increased HW would generate more flow through a gravity structure (i.e., expecting to see maximum flow through the structure occurring near the time of maximum HW). The time between the 12-hour moving average peak HW (**Table 9-50**) and the 12-hour moving average peak discharge (**Table 9-48**) is shown in **Figure 9-154** for each design storm. The max delay with respect to the peak discharge at S25B increases by over two-fold from the current conditions and future SLR1 and SLR2 scenarios to the SLR3 scenario. This delay in the structure's ability to discharge water is a result of high TW conditions from the storm surge and is shown to increase with increasing future SLR conditions.

Figure 9-154. Time Between Peak HW and Peak Discharge at S25B for the 12-Hour Moving Average



9.3.4. C4 – PM #4 PEAK STORM RUNOFF

The purpose of PM #4 is to determine the effect of SLR on the maximum peak storm runoff, or maximum conveyance capacity of the watershed. For this metric, 12-hour moving average flow hydrographs downstream of S25B and the maximum 12-hour moving average total flow was determined for each design storm event and SLR scenario.

The 12-hour moving average discharge hydrographs for each SLR scenario can be found in **Figure 9-155** for the 100-year storm, **Figure 9-156** for the 25-year storm, **Figure 9-157** for the 10-year storm, and **Figure 9-158** for the 5-year storm. Downstream flows for S25B comprise of the discharge from S25B_S and S25B_P in addition to overtopping of the structure, if applicable.

The instantaneous and 12-hour moving average peak discharges for all of the design storm event return periods and SLR scenarios are shown in **Table 9-53**. In addition, the percentage difference between the current conditions 12-hour moving average peak discharge and each future conditions SLR scenario is calculated for all simulations. The 12-hour moving average peak discharges are also shown in **Figure 9-159.** The peak 12-hour moving average discharge is decreased for all future SLR scenarios compared to current conditions.

Table 9-53. Peak Discharge Summary at S25B

Design Storm Return Period	Scenario	Peak Discharge (CFS)		12-hour Moving Average Peak Flow Reduction Percentage
		Instantaneous	12-hour Moving Average	
100-Year	**Current**	2554.38	2158.51	N/A
	SLR1	2670.40	1814.22	15.95%
	SLR2	2713.78	1951.85	9.57%
	SLR3	2516.10	1804.43	16.40%
25-Year	**Current**	2093.14	1801.79	N/A
	SLR1	2079.69	1567.61	13.00%
	SLR2	2126.09	1296.73	28.03%
	SLR3	2177.79	1328.63	26.26%
10-Year	**Current**	1871.00	1577.74	N/A
	SLR1	1927.70	1387.65	12.05%
	SLR2	1900.91	1016.28	35.59%
	SLR3	1844.53	1005.39	36.28%
5-Year	**Current**	1697.83	1435.60	N/A
	SLR1	1924.72	1246.62	13.16%
	SLR2	1766.74	921.02	35.84%
	SLR3	1836.91	771.09	46.29%

Figure 9-155. 12-Hour Moving Average of Flows at S25B for the 100- year Design Storm

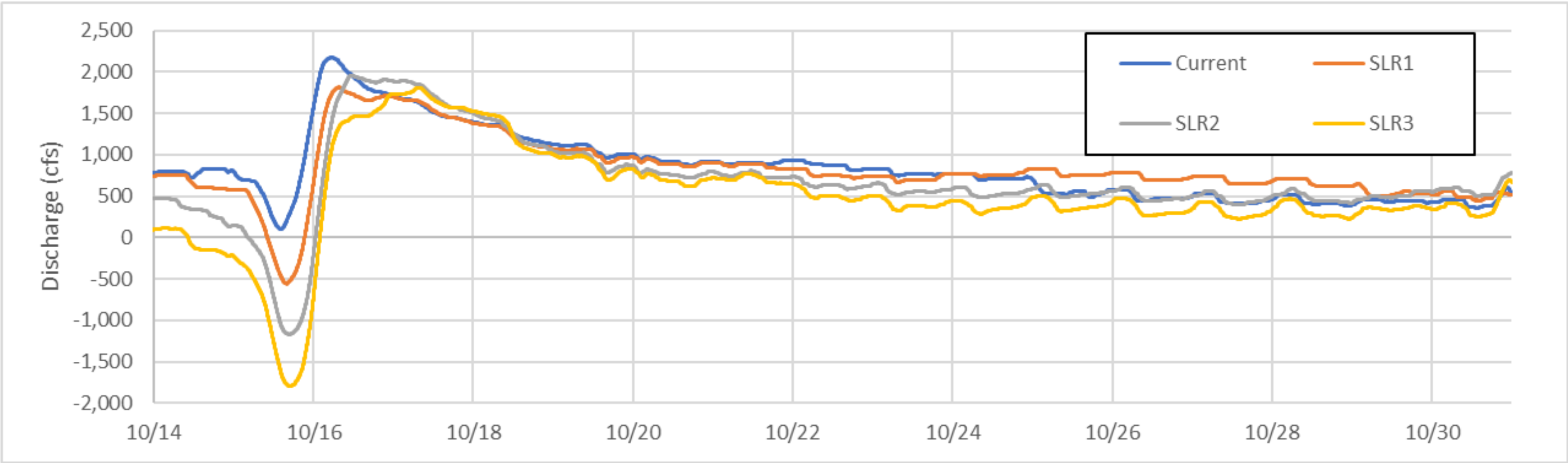


Figure 9-156. 12-Hour Moving Average of Flows at S25B for the 25-year Design Storm

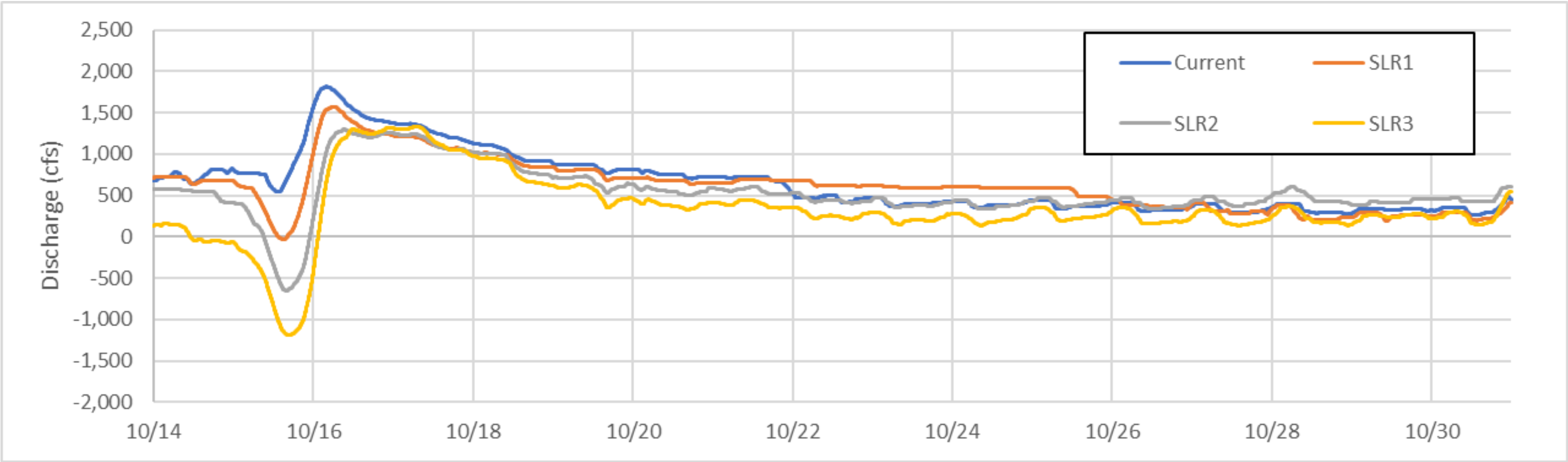


Figure 9-157. 12-Hour Moving Average of Flows at S25B for the 10-year Design Storm

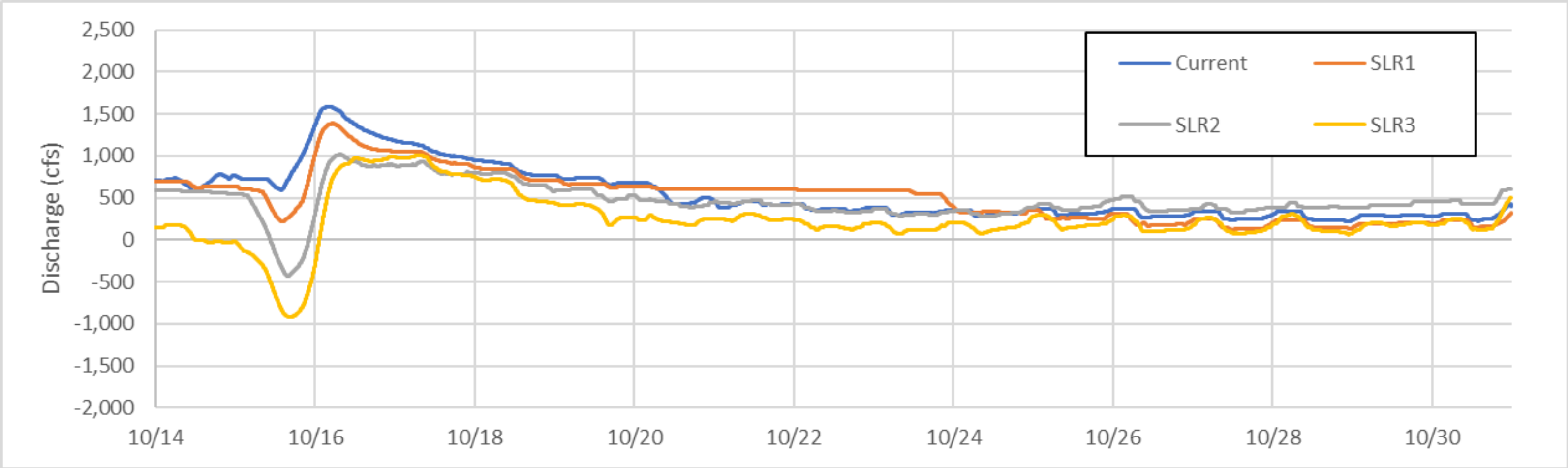


Figure 9-158. 12-Hour Moving Average of Flows at S25B for the 5-year Design Storm

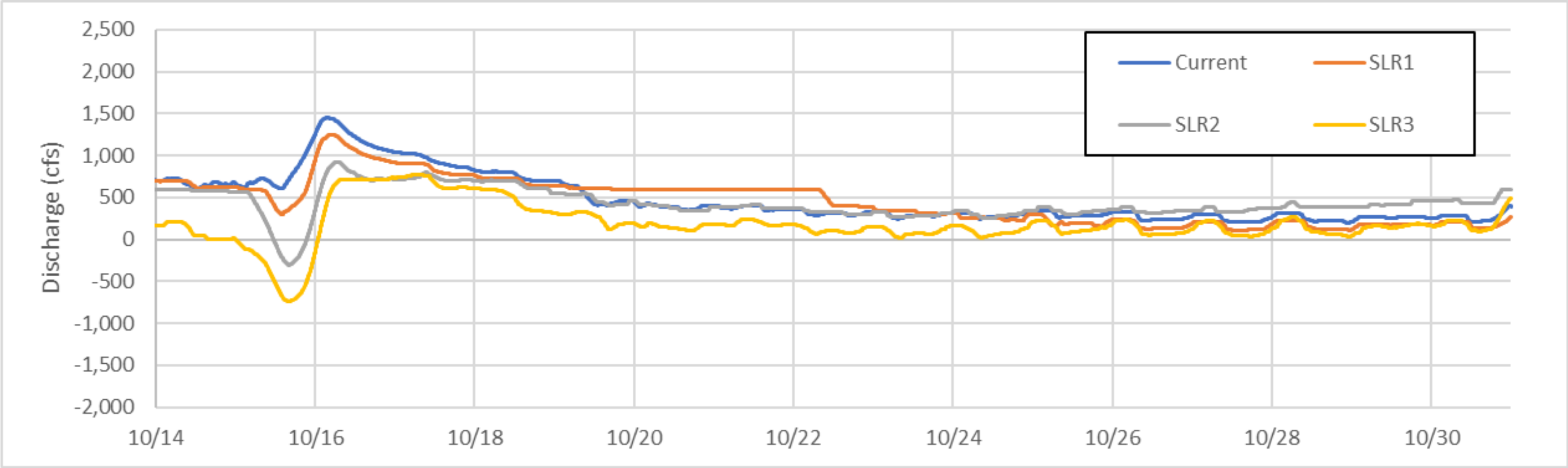
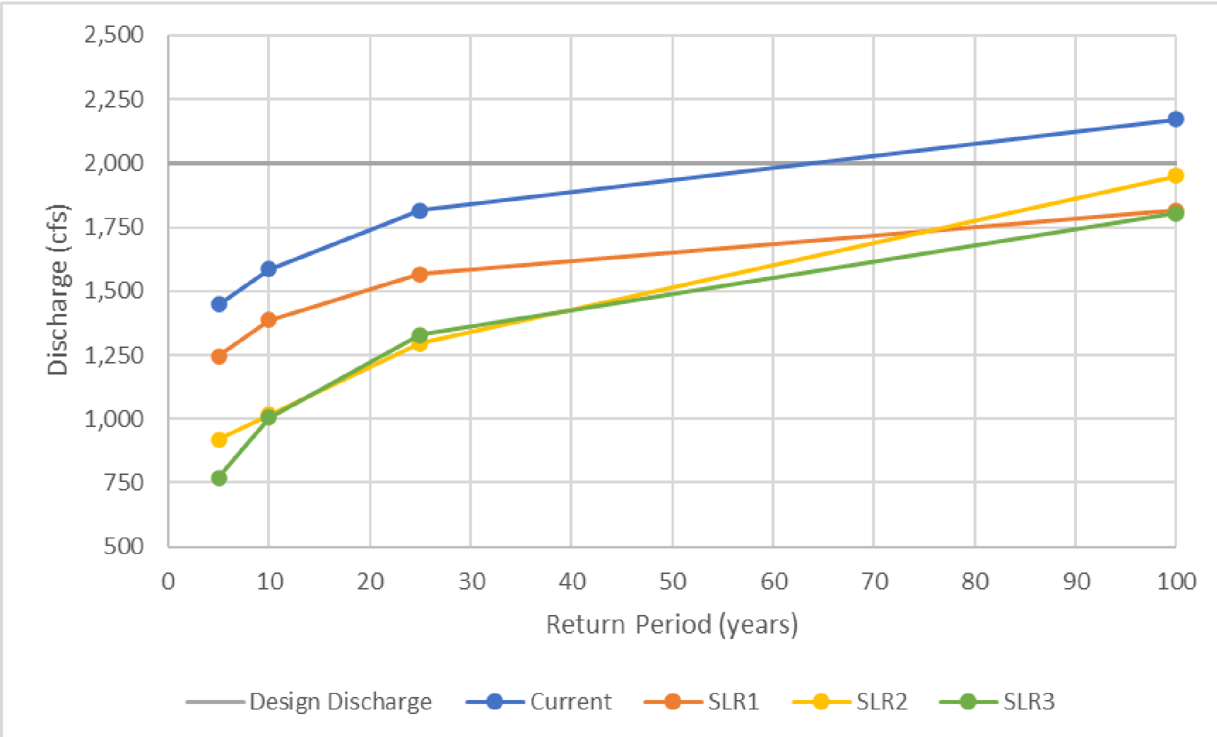


Figure 9-159. Peak 12-Hour Moving Discharge at S25B



9.3.5. C4 – PM #5 FREQUENCY OF FLOODING

The maximum overland depth was extracted for each design storm event and SLR condition and evaluated for the C4 Watershed.

Table 9-54 tabulates the flood inundation area in square miles for the C4 Watershed. The total area of the C4 Watershed for this analysis was calculated as 84.2 square miles (slight variations in total area from the District total area are due to estimating basin shape along a coarse grid).

The total area of the C4 Watershed considered Urban is 27.1 square miles, which makes this watershed mostly urban (about 32% urban). **Table 9-55** tabulates the flood inundation area in square miles for the urban areas within the C4 Watershed. This table shows that the greatest distribution of flooding depths in the watershed are between zero and 0.25 feet of flooding, which can be considered nuisance flooding. **Figure 9-160** shows the urban inundation with the same incremental flooding depths as the table for the 100-year storm event, and **Figure 9-161** does the same for the 25-year storm. This provides a clear view of how flooding depths are increasing with sea level rise in this watershed. The graph of the 100-year storm shows decreasing flooded area between 0.25 feet and 0.5 feet as

SLR increases, indicating that this flooding is moving to higher depths. However, the flooded area increases with SLR at the 0.25 feet to 0.5 feet depth range for the 25-year storm, indicating that while the area of inundation is increasing, it stays at the lower depths for this storm event.

Table 9-56 shows the percentage of the total urban areas within the C4 Watershed that is above the flooding depth. This data shows that 59% of the total urban areas in the C4 Watershed has 3 inches or greater of flooding depth (0.25 feet) for the 100-year design storm event for current conditions. This increases by 1.1% for SLR1, 2.6% for SLR2, and 3.2% for SLR3. Approximately 38% of the watershed experiences 3 inches or greater of flooding during the 25-year storm for current conditions, which increases at a similar rate to the 100-year storm with SLR.

Figure 9-162 and **Figure 9-163** are maps of the maximum overland depth over the entire C4 Watershed for the 25-year and 100-year 3-day design storm events, respectively. Water areas, such as existing lakes and ponds, are masked in black. **Figure 9-164** and **Figure 9-165** provide the same maps with the non-urban areas masked out, to provide a concise picture of how urban areas are impacted within the watershed. As evident in these figures, a majority of the inundated areas in the C4 basin are non-urban.

Figure 9-166, **Figure 9-167**, and **Figure 9-168** show the difference in overland flooding for the C4 Watershed between the current conditions and future sea level rise conditions for the SLR +1 foot, SLR +2 feet, and SLR +3 feet simulations, respectively. Because the rainfall is the same in all the 100-year storm event simulations, these difference maps remove any overland flooding caused by rainfall and show how much is now impacted by rising seas in terms of direct flooding from the canals, or reduced drainage capacity due to higher stages in the primary canal. Where higher canal stages are due to reduced discharge capacity at the structure as well as structure backflow due to overtopping and structure bypass, as discussed in the previous sections.

Maps of the maximum overland depth over the entire C4 Watershed, and only urban areas, for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current and future SLR scenarios are provided in **Appendix C**. Also provided in **Appendix C** are the differences in overland flooding maps for the 25-year, 10-year, and 5-year design storms.

Table 9-54. Incremental Flood Inundation Area (sq. mi.) in the C4 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	14.85	22.14	26.08	28.94	14.49	21.61	25.50	28.14	13.59	20.61	24.30	26.89	12.46	19.26	22.83	25.10
0.25 =< Depth < 0.50	7.53	5.21	4.82	5.00	7.52	5.04	4.31	4.27	7.25	4.91	4.32	4.10	6.90	4.91	4.06	4.00
0.50 =< Depth < 0.75	3.89	4.25	5.66	7.48	3.77	3.81	4.56	5.68	3.84	3.59	4.00	4.62	3.82	3.46	3.69	4.07
0.75 =< Depth < 1.00	3.76	5.96	8.37	7.99	3.21	4.58	6.89	8.08	3.20	3.91	5.53	6.97	3.26	3.39	4.57	5.73
1.00 =< Depth < 1.25	4.32	8.33	7.66	7.61	3.40	7.09	8.50	8.50	2.94	5.89	7.54	7.82	2.96	4.70	6.85	7.40
1.25 =< Depth < 1.50	7.32	7.70	7.48	6.24	5.55	8.43	8.26	7.40	4.37	7.65	7.93	7.65	3.48	7.14	7.40	7.44
1.50 =< Depth < 1.75	7.70	7.13	4.79	3.40	7.96	7.88	6.11	4.26	7.23	7.80	7.02	5.58	5.83	7.44	7.20	6.55
1.75 =< Depth < 2.00	7.33	4.30	2.79	2.47	8.20	5.65	3.44	2.86	7.78	6.85	4.43	3.43	7.68	7.08	5.78	4.38
2.00 =< Depth < 2.25	5.60	2.61	2.22	1.88	7.18	3.25	2.44	2.12	7.32	4.10	2.98	2.69	7.34	5.42	3.64	3.12
2.25 =< Depth < 2.50	3.31	2.08	1.91	1.78	4.13	2.53	1.68	1.49	5.64	2.93	2.32	1.74	6.33	3.52	2.81	2.30
2.50 =< Depth < 2.75	2.39	1.59	1.40	1.48	2.85	1.68	0.93	1.39	3.43	2.25	1.43	0.87	4.43	2.70	2.03	1.53
2.75 =< Depth < 3.00	1.94	0.99	1.56	1.26	2.28	0.90	1.59	1.31	2.66	1.32	0.90	1.24	3.01	2.00	1.13	0.93
3.00 =< Depth	14.29	11.94	9.46	8.70	13.68	11.79	10.02	8.71	14.98	12.41	11.51	10.63	16.74	13.20	12.24	11.68

Total Basin Area = 84.2 square miles

Table 9-55. Incremental Flood Inundation Area (sq. mi.) for Urban Areas in the C4 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	11.14	16.84	19.49	21.28	10.85	16.42	19.13	20.80	10.13	15.70	18.44	20.17	9.26	14.66	17.43	19.07
0.25 =< Depth < 0.50	6.01	3.48	2.63	2.00	5.96	3.54	2.58	2.09	5.81	3.67	2.76	2.22	5.52	3.76	2.86	2.43
0.50 =< Depth < 0.75	2.79	2.11	1.67	1.31	2.81	2.13	1.72	1.36	2.94	2.22	1.78	1.45	2.96	2.36	2.00	1.66
0.75 =< Depth < 1.00	2.08	1.43	1.03	0.72	2.05	1.51	1.12	0.89	2.18	1.58	1.16	0.95	2.41	1.74	1.24	1.02
1.00 =< Depth < 1.25	1.36	0.93	0.63	0.48	1.43	0.95	0.76	0.54	1.51	1.00	0.78	0.63	1.71	1.06	0.91	0.72
1.25 =< Depth < 1.50	0.98	0.65	0.46	0.37	1.00	0.69	0.52	0.46	1.09	0.72	0.62	0.50	1.24	0.82	0.59	0.58
1.50 =< Depth < 1.75	0.68	0.46	0.35	0.33	0.73	0.57	0.40	0.31	0.75	0.57	0.46	0.35	0.82	0.59	0.58	0.49
1.75 =< Depth < 2.00	0.55	0.33	0.28	0.19	0.54	0.40	0.30	0.22	0.58	0.51	0.37	0.29	0.66	0.59	0.45	0.36
2.00 =< Depth < 2.25	0.38	0.28	0.19	0.14	0.56	0.31	0.20	0.14	0.59	0.37	0.25	0.18	0.53	0.43	0.37	0.28
2.25 =< Depth < 2.50	0.37	0.18	0.11	0.05	0.36	0.17	0.11	0.08	0.42	0.28	0.15	0.10	0.51	0.37	0.23	0.16
2.50 =< Depth < 2.75	0.22	0.12	0.05	0.05	0.28	0.12	0.07	0.06	0.38	0.14	0.08	0.04	0.47	0.26	0.14	0.07
2.75 =< Depth < 3.00	0.15	0.07	0.02	0.04	0.16	0.06	0.04	0.03	0.24	0.08	0.04	0.06	0.35	0.14	0.06	0.04
3.00 =< Depth	0.41	0.23	0.20	0.15	0.38	0.24	0.16	0.13	0.48	0.27	0.22	0.17	0.68	0.33	0.26	0.23

Total Basin Urban Area = 27.1 square miles

Table 9-56. Percentage of Total Watershed Area with Inundated Area for Urban Areas in the C4 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0.00	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 0.25	58.9	37.9	28.1	21.5	60.0	39.4	29.4	23.2	62.6	42.1	32.0	25.6	65.8	45.9	35.7	29.6
>= 0.50	36.7	25.0	18.4	14.1	38.0	26.4	19.9	15.5	41.2	28.6	21.8	17.4	45.5	32.0	25.2	20.7
>= 0.75	26.4	17.3	12.2	9.3	27.6	18.5	13.6	10.5	30.3	20.4	15.2	12.0	34.5	23.3	17.8	14.6
>= 1.00	18.8	12.0	8.5	6.6	20.1	12.9	9.5	7.2	22.3	14.5	10.9	8.5	25.7	16.9	13.2	10.8
>= 1.25	13.7	8.5	6.1	4.8	14.8	9.4	6.7	5.2	16.7	10.9	8.0	6.2	19.4	13.0	9.9	8.2
>= 1.50	10.1	6.2	4.4	3.5	11.1	6.9	4.7	3.5	12.7	8.2	5.8	4.4	14.8	10.0	7.7	6.0
>= 1.75	7.6	4.5	3.2	2.3	8.4	4.8	3.3	2.4	9.9	6.1	4.1	3.1	11.8	7.8	5.6	4.2
>= 2.00	5.6	3.2	2.1	1.6	6.4	3.3	2.2	1.6	7.8	4.2	2.7	2.0	9.4	5.6	3.9	2.9
>= 2.25	4.2	2.2	1.4	1.1	4.3	2.2	1.4	1.1	5.6	2.9	1.8	1.3	7.4	4.1	2.5	1.9
>= 2.50	2.8	1.5	1.0	0.9	3.0	1.5	1.0	0.8	4.0	1.8	1.2	1.0	5.5	2.7	1.7	1.3
>= 2.75	2.0	1.1	0.8	0.7	2.0	1.1	0.8	0.6	2.6	1.3	1.0	0.8	3.8	1.7	1.2	1.0
>= 3.00	1.5	0.9	0.7	0.5	1.4	0.9	0.6	0.5	1.8	1.0	0.8	0.6	2.5	1.2	1.0	0.9

Figure 9-160. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C4 Watershed for the 100-year Design Storm

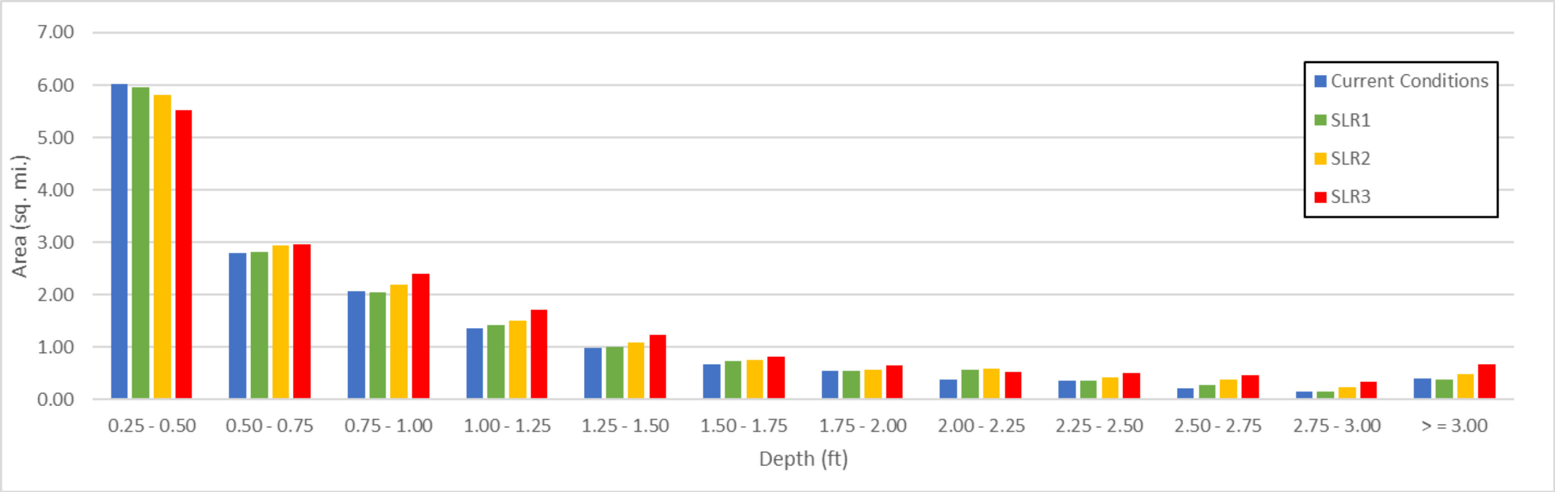


Figure 9-161. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C4 Watershed for the 25-year Design Storm

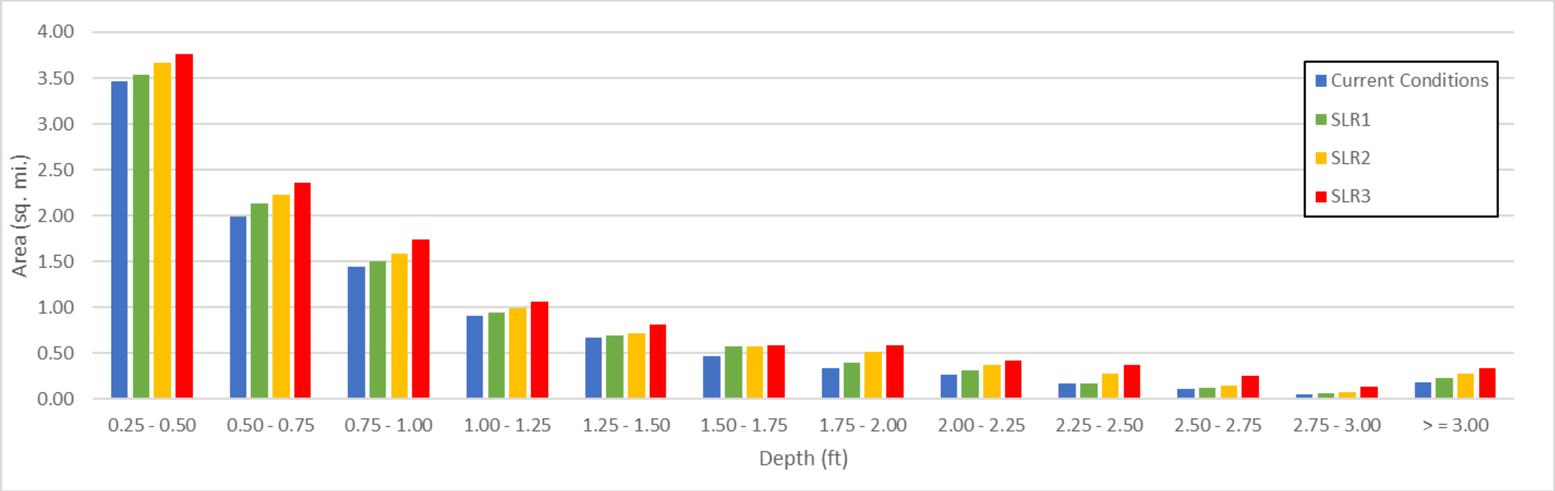


Figure 9-162. Maximum Overland Flood Depth for the Current Conditions 25-year 3-Day Design Storm in the C4 Watershed

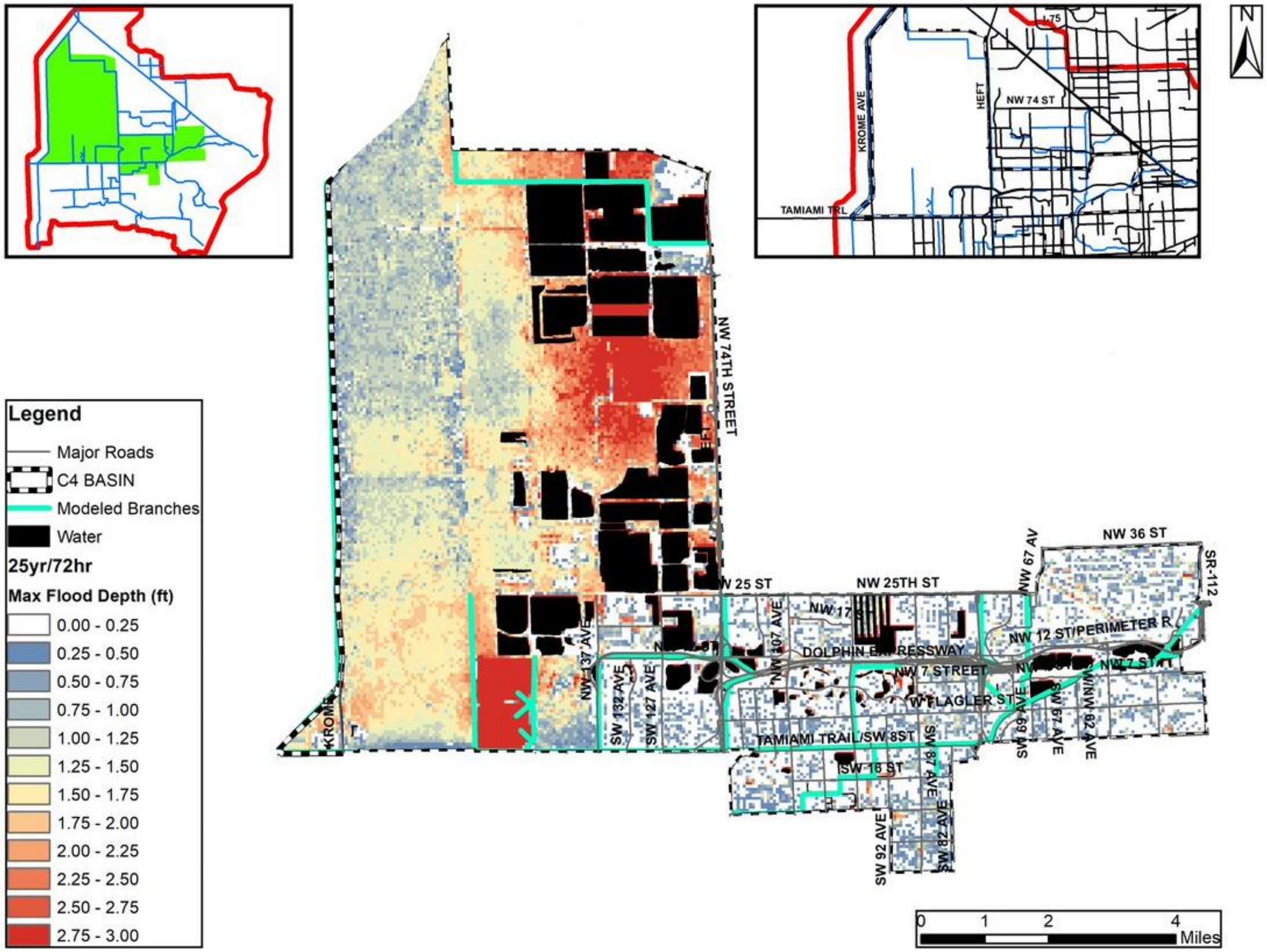


Figure 9-163. Maximum Overland Flood Depth for the Current Conditions 100-year 3-Day Design Storm in the C4 Watershed

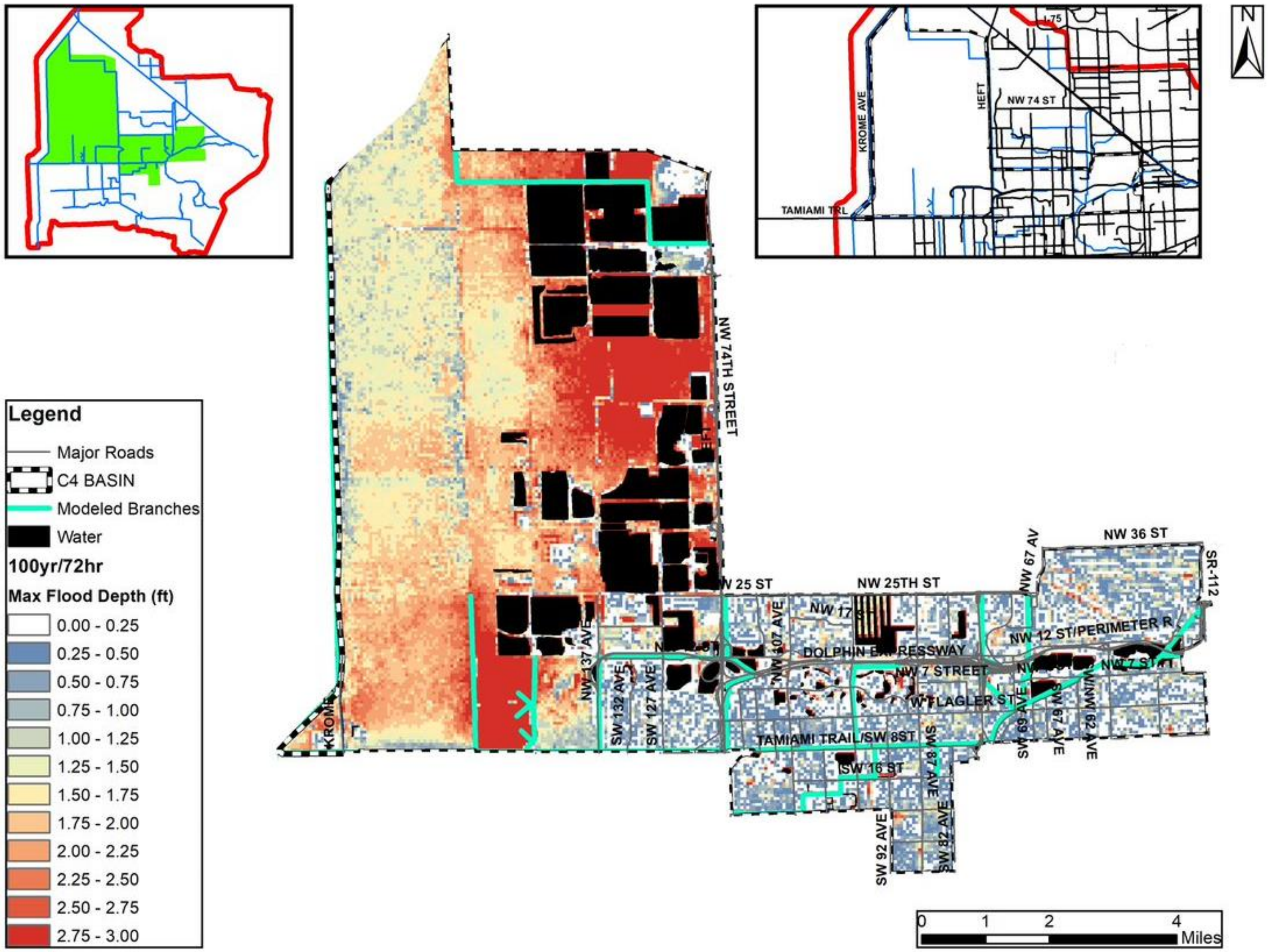


Figure 9-164. Maximum Overland Flood Depth for the Current Conditions 25-year 3-Day Design Storm for Urban Areas in the C4 Watershed

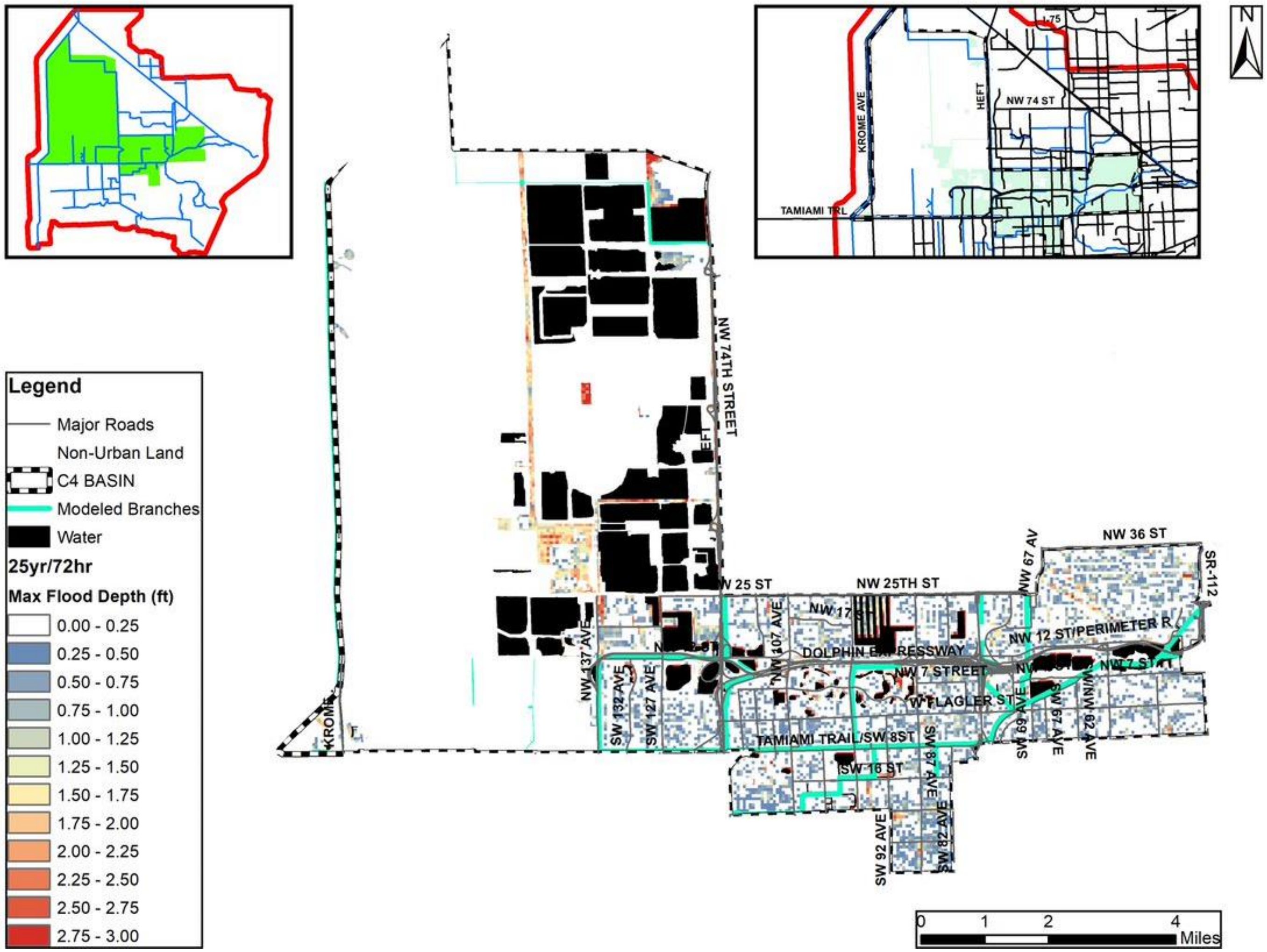


Figure 9-165. Maximum Overland Flood Depth for the Current Conditions 100-year 3-Day Design Storm for Urban Areas in the C4 Watershed

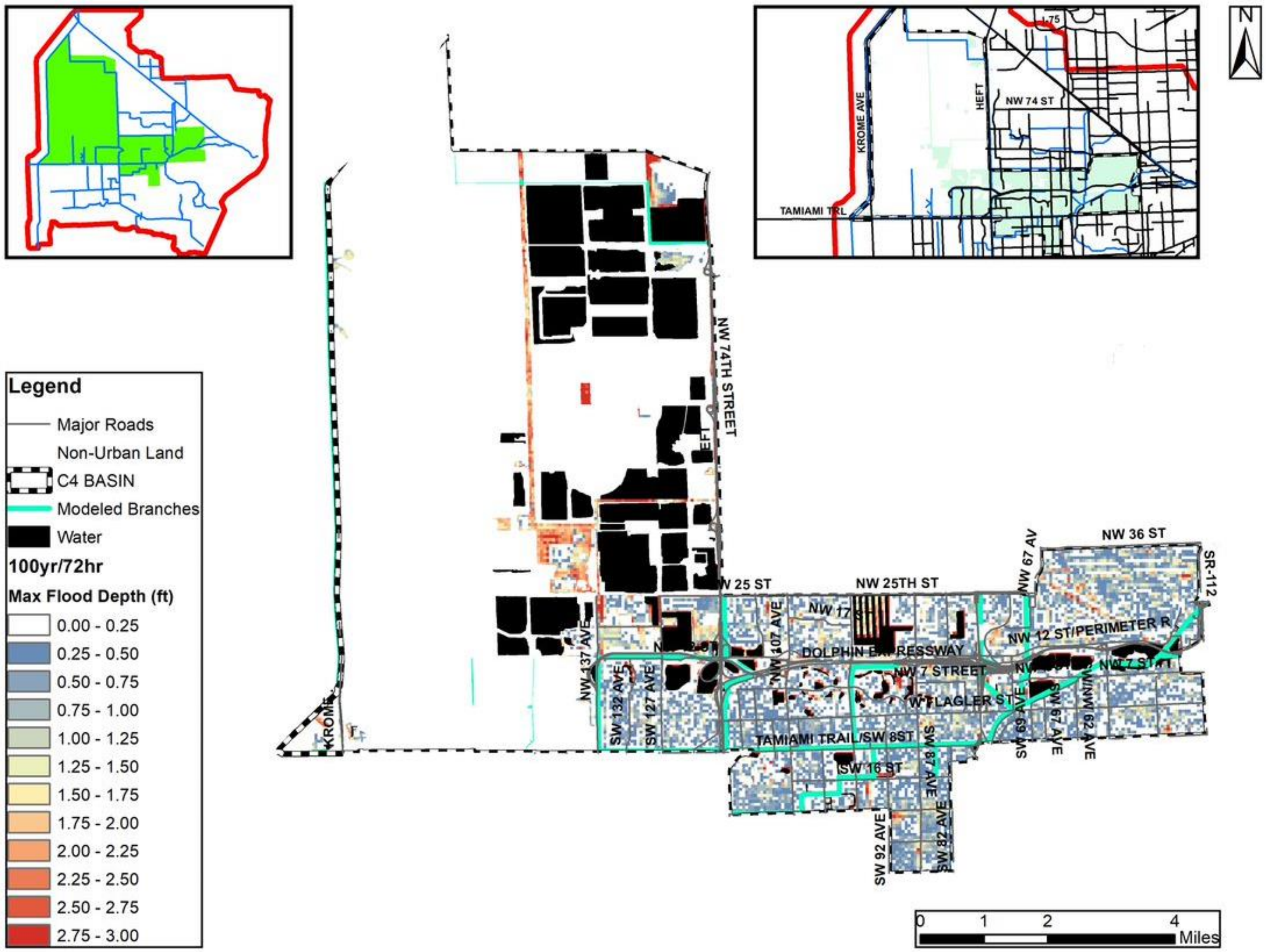


Figure 9-166. Urban Flooding Depth Difference of SLR +1ft and Current Conditions for the 100-year Storm in the C4 Watershed

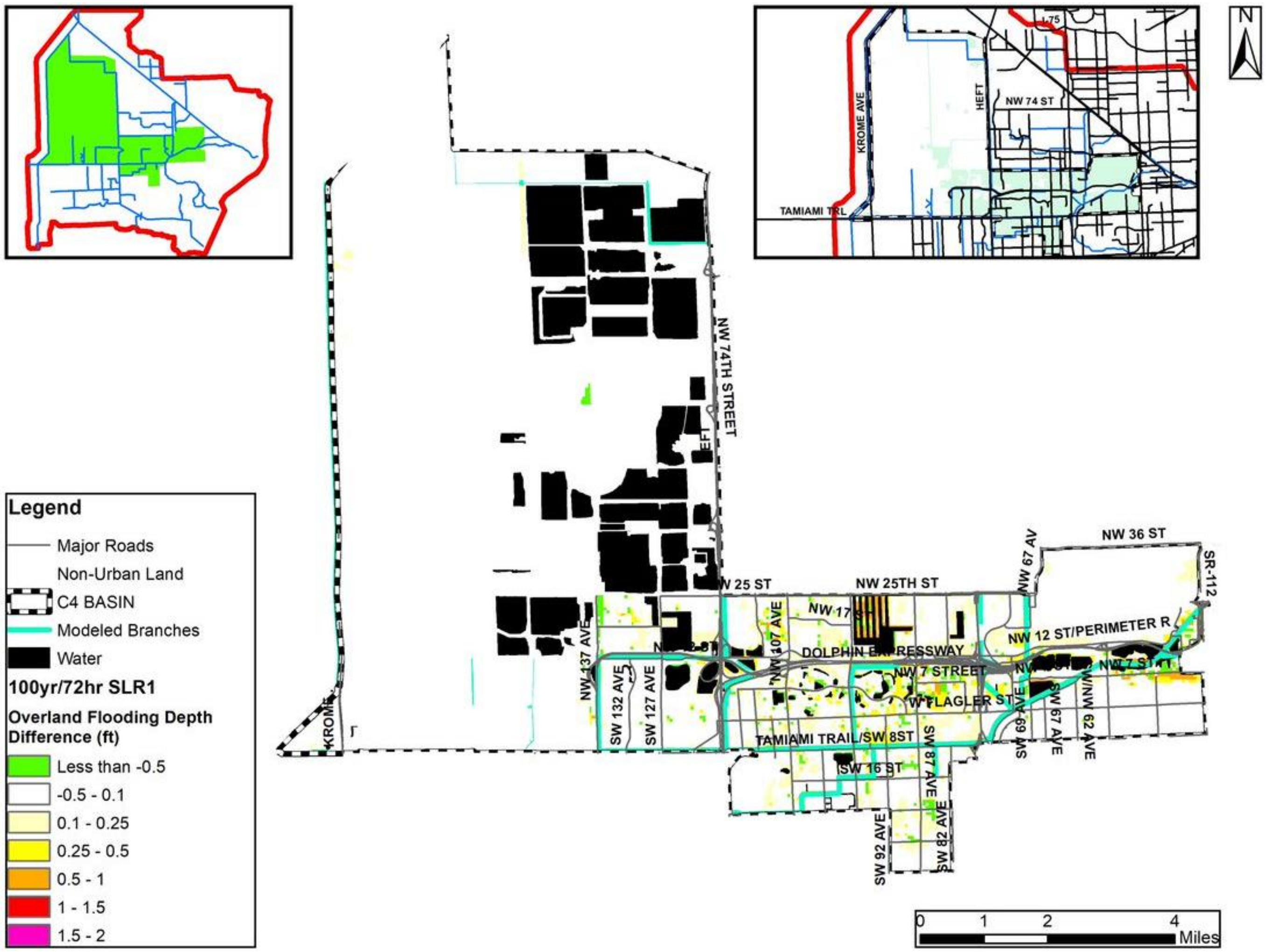


Figure 9-167. Urban Flooding Depth Difference of SLR +2ft and Current Conditions for the 100-year Storm in the C4 Watershed

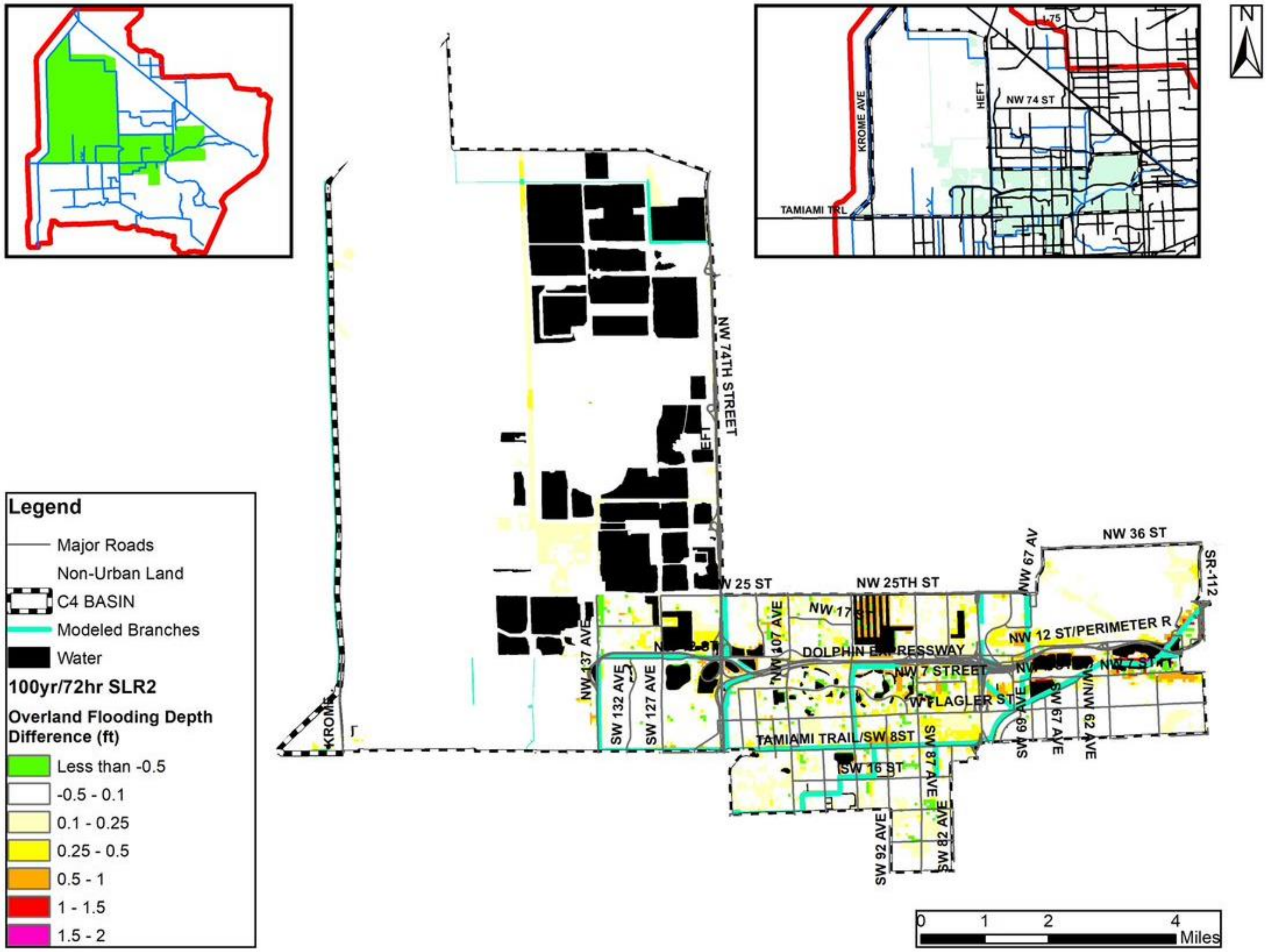
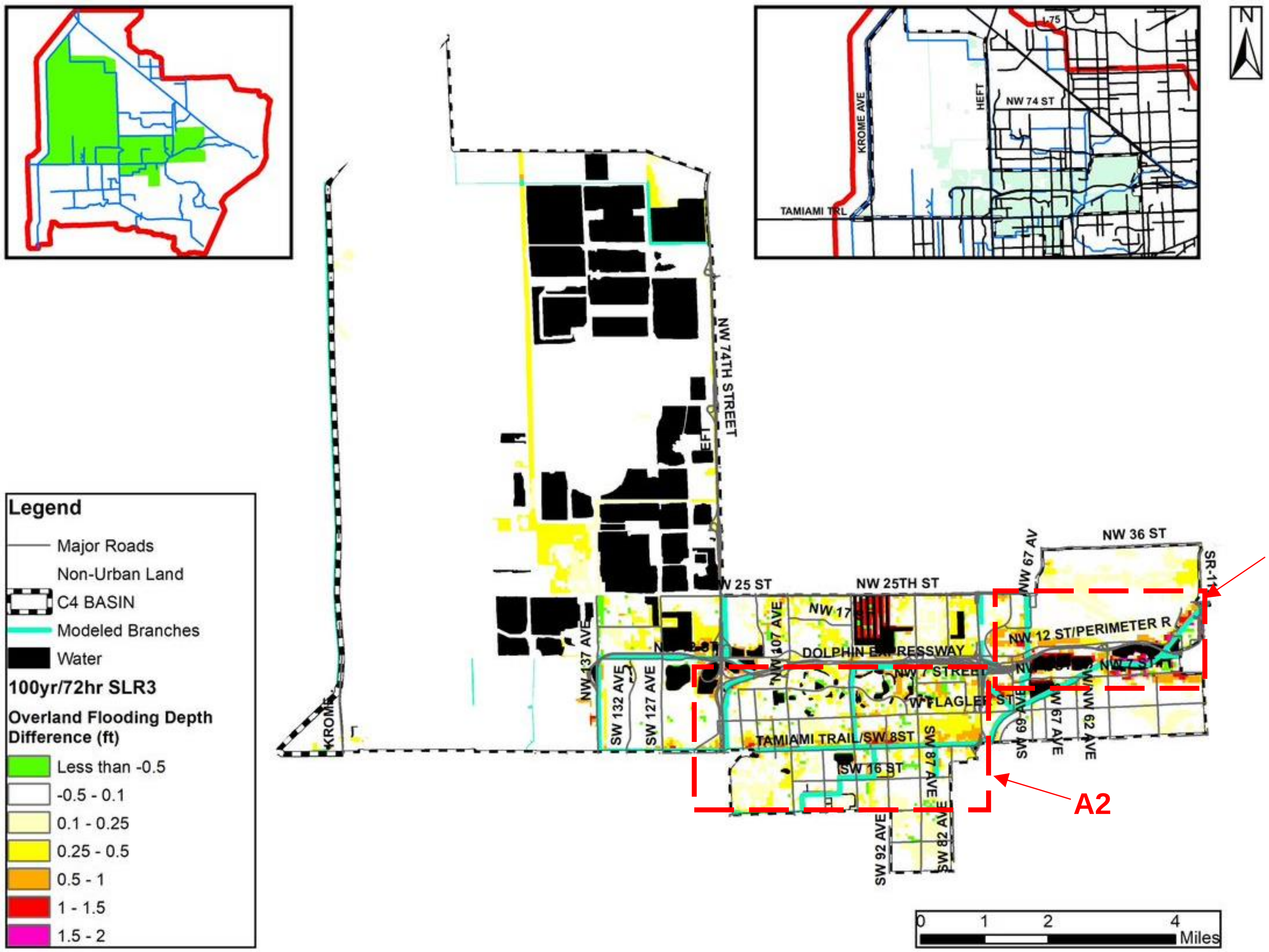


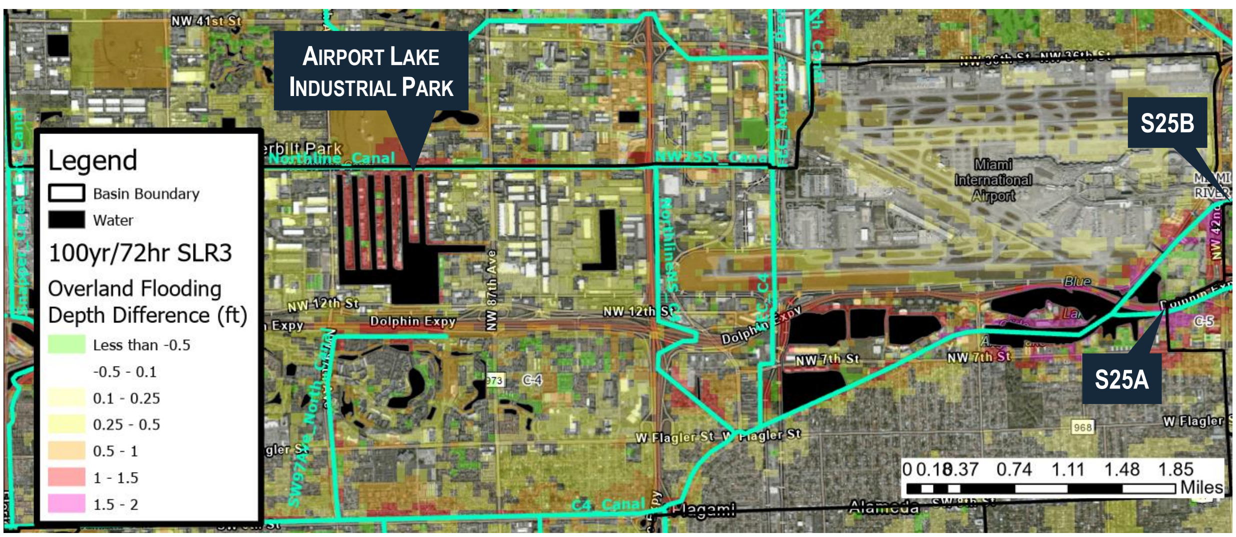
Figure 9-168. Urban Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C4 Watershed



As evident in **Figure 9-164** and **Figure 9-165**, a majority of the inundated areas in the C4 basin are non-urban. There are some areas, such as the Westchester area, that experience up to 2.5 feet of water during the 100-year storm.

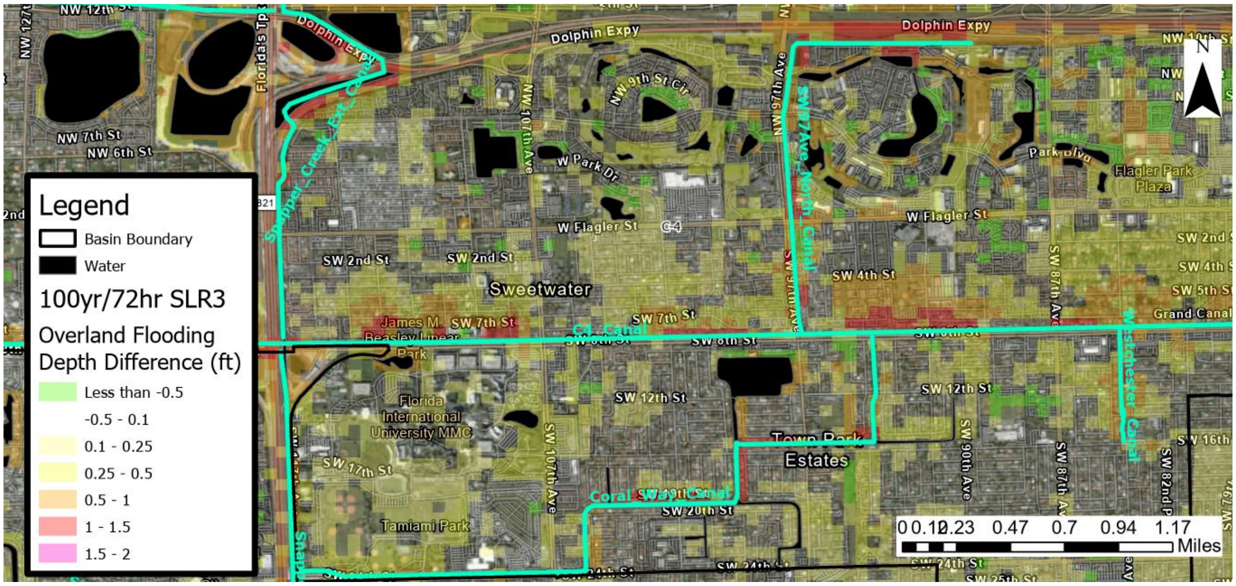
The area just south of Miami International Airport, just upstream of S25A and S25B (indicated by 'A1' in **Figure 9-168**) can be impacted by over 1.5 feet above the current conditions with the SLR3 simulation. A zoomed in look at this neighborhood is shown in **Figure 9-169**. Also shown in this Figure, just south of the Northline Canal is Airport Lake Industrial Park that seems like there is an additional 1.0 to 1.5 feet of flooding above current conditions with the SLR3 simulation. However, the lakes in this area are smaller than the model cell size and are, therefore, lowering the average topography of the cells.

Figure 9-169. Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C4 Watershed – Miami International Airport (A3)



Additional areas, including the north side of the C4 Canal in Sweetwater, along the SW 97th Ave_North Canal, and along the Coral Way canal (indicated by 'A2' in **Figure 9-168**) may be impacted up to 1.5 feet above the current conditions with the SLR3 simulation. A zoomed in look at these neighborhoods are shown in **Figure 9-170.**

Figure 9-170. Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C2 Watershed – Sweetwater (A2)



9.3.6. C4 – PM #6 DURATION OF FLOODING

9.3.6.1. CANAL FLOOD DURATION

In discussions with water managers at SFWMD, it was reported that stages at the T5W station (near the intersection of the C2 and C4 Canals) are an indication of whether the storm has subsided within the areas upstream of the S22, G93, and S25B water control structures. The S25B Forward Pump Station uses stages at T5W as one indicator of when to trigger flood control operations in the study area. If stages at T5W exceed 3.80 ft-NGVD (or 2.228 ft-NAVD using the conversion factor of (-)1.572 feet), and other conditions are also met, the pump will turn on as described in the C4 Basin Operation Plan (SFWMD, 2019). Under current conditions, stages at T5W for each design storm were evaluated and this 2.228 ft-NAVD trigger was used to identify when the storm would be considered initiated and finalized, establishing the Reference Elevation for the T5W station. However, this Reference Elevation would not be acceptable under future SLR conditions, as the storms, and even the normal wet season canal elevations may be higher than 2.228 ft-NAVD.

A new Reference Elevation was established based on the off-trigger elevations used for the municipal pump stations that pump into the C4 Canal. The pumps are meant to turn off if stages in the C4 Canal exceed 5 ft-NGVD, or 3.428 ft-NAVD. Since this trigger has already been established by the County and municipalities as an indicator of when the C4 stages are too high, this makes an appropriate indicator for when the C2, C3W, and C4 watersheds are still in a flood condition for the canals. **Table 9-57** shows the canal flood duration for each storm event and SLR condition at T5W. For all storm events, under the SLR3 conditions, canal stages do not recede past the Reference Elevation after the storm and therefore the storm duration is longer than the values provided.

Table 9-57. Storm Duration Indicated at T5W

Design Storm	Duration (hours)				
	Current Conditions with 2.23 ft-NAVD Reference Elev.	Current Conditions with 3.43 ft-NAVD Reference Elev.	SLR1	SLR2	SLR3
100-Year	281.8	119.6	162.1	282.6	420.6*
25-Year	184.3	67.6	108.2	217.7	410.4*
10-Year	140.3	40.8	68.6	149.8	408.3*
5-Year	101.6	24.9	47.1	120.1	398.1*

*Canal stages do not recede past the Reference Elevation after the storm and therefore the storm duration is longer than the values provided.

Figure 9-51 shows the hydrographs at T5W station, comparing the SLR conditions for each design storm. As described in the table and shown in the figure, the Reference Elevation of 3.428 ft-NAVD is shown on the graphs as a grey line. **Figure 9-171** shows the HW stages at S25B comparing the 100-year storm results for each SLR condition, and the vertical lines indicate the start and end of each design storm as determined at T5W. **Figure 9-172** shows the same, but for the 25-year storm.

9.3.6.2. WATERSHED FLOOD DURATION

Table 9-58 tabulates the total area of flood inundation (in square miles) per flood duration range for all areas in the C4 Watershed and **Table 9-59** does this for all urban areas within the C4 Watershed. This table shows that the greatest distribution of flooding duration in the watershed is between zero and one hour of flooding. However, since the model outputs overland depths every hour, the first hour of flooding may be anywhere from 1 minute to 1 hour long. Therefore, the first hour of flooding is considered nuisance flooding for the purposes of this analysis.

Flooding is also distributed in the 12-to-48-hour range, indicating that much of the ponded water in the watershed has a one to three-day journey to a major canal, as represented in the model.

Table 9-60 calculates the percentage of the total urban areas within the C4 Watershed that are inundated by 3 inches or greater for each flood duration. Because the flood duration greater than 1 hour includes all areas inundated with 3 inches or more, this row shows the same percentage as that in **Table 9-56** in PM #5. Additionally, **Figure 9-173**, **Figure 9-174**, **Figure 9-175**, and **Figure 9-176** provide a graphical view of this data, plotting the flood duration against the percentage of area inundated for the urban areas of the C4 Watershed for the 100-year, 25-year, 10-year, and 5-year storm events, respectively. These graphics also include the percent increase above the current conditions on the secondary axis, to visualize how the flood duration is changing with each SLR condition. For all design storms, the percentage of the watershed that is flooded for 24 hours shows the greatest increase above current conditions for all SLR conditions. However, the SLR1 condition shows only a small increase above current conditions in terms of percentage of flooded area.

Figure 9-177 and **Figure 9-178** provide flood duration maps for the C4 Watershed for overland flooding depths exceeding 0.25 feet for the current conditions 25-year and 100-year 3-day design storms, respectively. Water areas, such as existing lakes and ponds, are masked in black. **Figure 9-179** and **Figure 9-180** provide the flood duration for only the urban areas within the C4 Watershed (non-urban areas are masked out), to provide a concise picture of how urban areas are impacted within the watershed.

Figure 9-181, **Figure 9-182**, and **Figure 9-183** provide the difference in flood duration between the current conditions and SLR +1 foot, SLR +2 feet, and SLR +3 feet, respectively, for the 100-year storm event. When compared with the difference maps for flood depth, this can show that even for areas where flood depths are not increasing with future SLR conditions, the duration of flooding may increase due to the reduced ability of the area to drain to the receiving canals that are experiencing higher stages. For example, the Miami International Airport, the neighborhood at the southeast of the intersection of the Dolphin and Palmetto Expressways (primarily apartments and commercial buildings), and the neighborhood to the south between SW 92nd Avenue and SW 82nd Avenue (primarily medium density residential).

Flood duration maps over the entire C4 Watershed, and only urban areas, for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current conditions and future SLR scenarios are provided in **Appendix D**. Also provided in **Appendix D** are the differences in flood duration maps for the 25-year, 10-year, and 5-year design storms.

Figure 9-171. T5W Flood Duration Compared with HW at S25B for the 100-year Storm

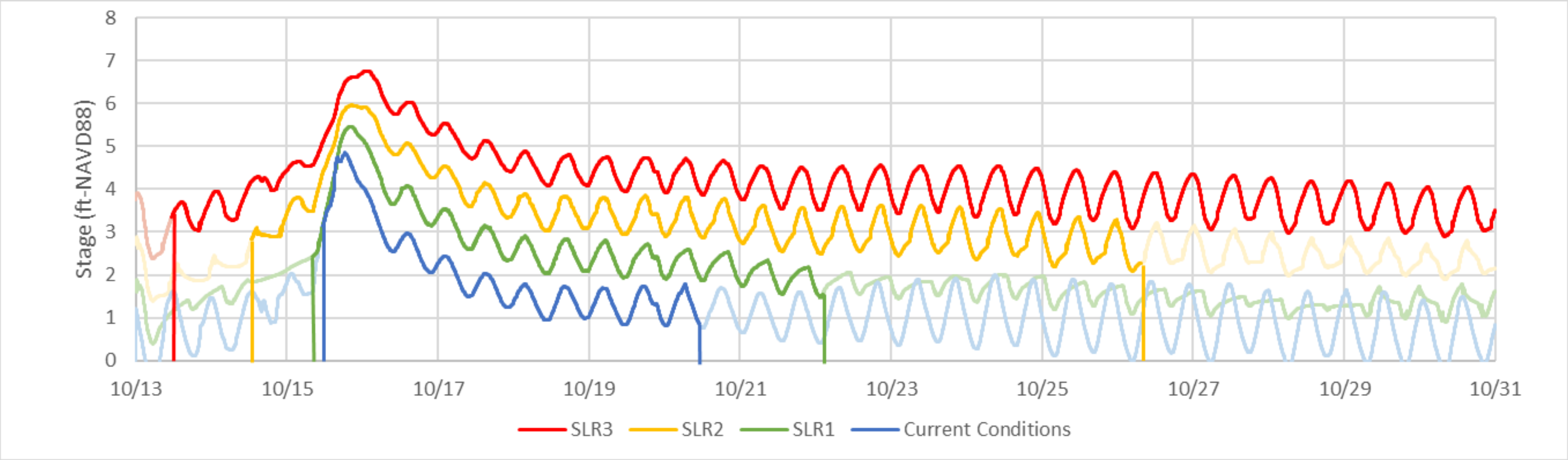


Figure 9-172. T5W Flood Duration Compared with HW at S25B for the 25-year Storm

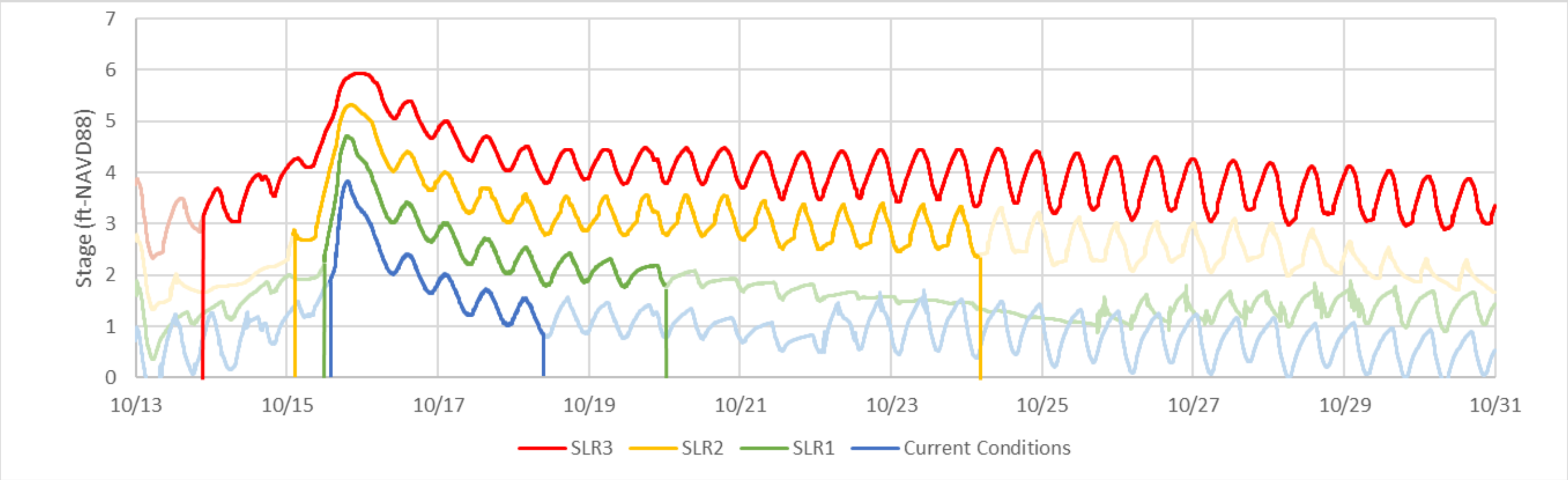


Table 9-58. Flood Duration per Area of Inundation (in sq. mi.) for the C4 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	14.85	22.14	26.08	28.94	14.49	21.61	25.50	28.14	13.59	20.61	24.30	26.89	12.46	19.26	22.83	25.10
1 to 2 hr.	2.55	0.52	0.30	0.28	2.53	0.58	0.29	0.23	2.40	0.58	0.32	0.26	2.18	0.56	0.24	0.25
2 to 4 hr.	1.02	0.72	0.59	0.43	0.97	0.70	0.52	0.39	0.87	0.64	0.47	0.39	0.78	0.53	0.45	0.35
4 to 8 hr.	1.09	1.06	0.89	0.72	0.97	0.97	0.79	0.65	0.79	0.87	0.75	0.62	0.75	0.76	0.71	0.59
8 to 12 hr.	0.84	0.80	0.69	0.54	0.75	0.76	0.67	0.55	0.64	0.64	0.61	0.54	0.51	0.61	0.52	0.50
12 to 24 hr.	2.86	2.29	1.79	1.47	2.67	2.19	1.81	1.45	2.32	2.12	1.72	1.43	1.91	1.84	1.63	1.39
24 to 48 hr.	2.91	2.32	1.78	1.58	2.91	2.22	1.74	1.54	3.02	2.19	1.82	1.52	2.79	2.24	1.74	1.53
48 to 72 hr.	1.79	1.27	1.25	1.06	1.70	1.26	1.11	1.05	1.91	1.39	1.08	0.85	2.11	1.44	1.15	0.95
72 to 96 hr.	1.14	1.09	0.88	0.65	1.32	1.03	0.85	0.76	1.32	0.98	0.71	0.68	1.41	0.98	0.80	0.67
96 to 144 hr.	1.83	1.45	1.08	0.80	1.80	1.51	1.28	1.05	1.90	1.38	1.24	1.09	1.94	1.38	1.08	0.85
144 to 192 hr.	1.45	0.97	0.72	0.66	1.52	1.15	1.00	0.89	1.39	1.19	0.90	0.80	1.48	1.09	0.83	0.67
192 to 240 hr.	1.09	0.68	0.60	0.99	1.11	0.85	0.85	0.85	1.09	0.80	0.82	0.66	1.10	0.79	0.58	0.70
240 to 336 hr.	1.41	1.44	1.53	1.18	1.72	1.52	2.06	2.30	1.80	1.46	1.40	1.49	1.63	1.34	1.40	1.37
336 to 420 hr.	15.95	15.06	14.30	13.65	12.58	12.01	10.83	10.10	9.75	9.11	8.64	8.18	8.83	7.84	7.33	6.84
420 hr.	33.45	32.41	31.72	31.26	37.19	35.86	34.93	34.29	41.46	40.27	39.45	38.85	44.34	43.56	42.94	42.47

Total Basin Urban Area = 84.2 square miles

Table 9-59. Flood Duration per Area of Inundation (in sq. mi.) for Urban Areas in the C4 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	11.14	16.84	19.49	21.28	10.85	16.42	19.13	20.80	10.13	15.70	18.44	20.17	9.26	14.66	17.43	19.07
1 to 2 hr.	2.06	0.37	0.21	0.21	2.04	0.42	0.19	0.15	1.99	0.44	0.22	0.19	1.76	0.43	0.15	0.17
2 to 4 hr.	0.79	0.56	0.43	0.33	0.71	0.53	0.37	0.32	0.64	0.50	0.35	0.31	0.60	0.42	0.34	0.27
4 to 8 hr.	0.93	0.80	0.70	0.52	0.80	0.77	0.62	0.52	0.65	0.70	0.59	0.48	0.62	0.63	0.54	0.48
8 to 12 hr.	0.70	0.63	0.53	0.39	0.61	0.61	0.53	0.38	0.53	0.53	0.50	0.42	0.44	0.50	0.44	0.41
12 to 24 hr.	2.31	1.81	1.32	1.00	2.22	1.72	1.39	1.10	1.90	1.70	1.36	1.09	1.56	1.49	1.34	1.10
24 to 48 hr.	2.34	1.73	1.21	0.92	2.36	1.75	1.27	1.01	2.51	1.74	1.39	1.11	2.32	1.81	1.38	1.16
48 to 72 hr.	1.42	0.87	0.70	0.50	1.34	0.97	0.74	0.57	1.52	1.10	0.81	0.55	1.76	1.13	0.90	0.72
72 to 96 hr.	0.87	0.66	0.47	0.29	1.06	0.73	0.52	0.37	1.06	0.74	0.48	0.45	1.13	0.79	0.62	0.47
96 to 144 hr.	1.28	0.83	0.46	0.25	1.42	0.89	0.58	0.40	1.53	0.97	0.75	0.49	1.52	1.12	0.76	0.57
144 to 192 hr.	0.82	0.39	0.18	0.07	0.92	0.51	0.31	0.15	1.05	0.69	0.39	0.23	1.23	0.79	0.55	0.41
192 to 240 hr.	0.48	0.18	0.06	0.07	0.61	0.29	0.10	0.08	0.73	0.36	0.20	0.13	0.88	0.55	0.36	0.26
240 to 336 hr.	0.49	0.13	0.10	0.09	0.62	0.21	0.15	0.13	0.92	0.40	0.20	0.16	1.18	0.67	0.46	0.30
336 to 420 hr.	0.48	0.35	0.30	0.25	0.52	0.34	0.27	0.23	0.72	0.42	0.35	0.30	1.43	0.74	0.49	0.39
420 hr.	0.99	0.97	0.94	0.93	1.01	0.96	0.93	0.90	1.22	1.12	1.06	1.02	1.41	1.37	1.35	1.32

Total Basin Urban Area = 27.1 square miles

Table 9-60. Percentage of Total Area Inundated for Urban Areas in the C4 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0 hr.	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 1 hr.	58.9	37.9	28.1	21.5	60.0	39.4	29.4	23.2	62.6	42.1	32.0	25.6	65.8	45.9	35.7	29.6
>= 2 hr.	51.3	36.5	27.3	20.7	52.4	37.9	28.7	22.7	55.3	40.5	31.2	24.9	59.3	44.3	35.1	29.0
>= 4 hr.	48.4	34.4	25.7	19.5	49.8	35.9	27.4	21.5	52.9	38.6	29.9	23.8	57.1	42.8	33.9	28.0
>= 8 hr.	44.9	31.5	23.1	17.6	46.8	33.1	25.1	19.6	50.5	36.1	27.7	22.0	54.8	40.5	31.9	26.2
>= 12 hr.	42.3	29.2	21.2	16.2	44.6	30.9	23.1	18.2	48.6	34.1	25.8	20.4	53.2	38.6	30.3	24.7
>= 24 hr.	33.8	22.5	16.3	12.5	36.4	24.5	18.0	14.1	41.6	27.8	20.8	16.4	47.5	33.1	25.3	20.7
>= 48 hr.	25.2	16.1	11.9	9.1	27.7	18.1	13.3	10.4	32.3	21.4	15.7	12.3	38.9	26.4	20.2	16.4
>= 72 hr.	19.9	12.9	9.3	7.2	22.7	14.5	10.5	8.3	26.7	17.4	12.7	10.3	32.4	22.3	16.9	13.7
>= 96 hr.	16.7	10.5	7.6	6.1	18.8	11.8	8.6	6.9	22.8	14.6	10.9	8.6	28.3	19.3	14.6	12.0
>= 144 hr.	12.0	7.4	5.9	5.2	13.6	8.5	6.5	5.5	17.1	11.0	8.2	6.8	22.7	15.2	11.8	9.9
>= 192 hr.	9.0	6.0	5.2	4.9	10.2	6.6	5.3	4.9	13.2	8.5	6.7	5.9	18.1	12.3	9.8	8.4
>= 240 hr.	7.2	5.3	5.0	4.7	7.9	5.6	5.0	4.6	10.6	7.1	5.9	5.5	14.9	10.3	8.5	7.4
>= 336 hr.	5.4	4.9	4.6	4.4	5.7	4.8	4.4	4.2	7.2	5.7	5.2	4.9	10.5	7.8	6.8	6.3
>= 420 hr.	3.7	3.6	3.5	3.4	3.7	3.5	3.4	3.3	4.5	4.1	3.9	3.7	5.2	5.1	5.0	4.9

Figure 9-173. Percentage and Duration of Inundation for the C4 Watershed for the 100-year Storm Event

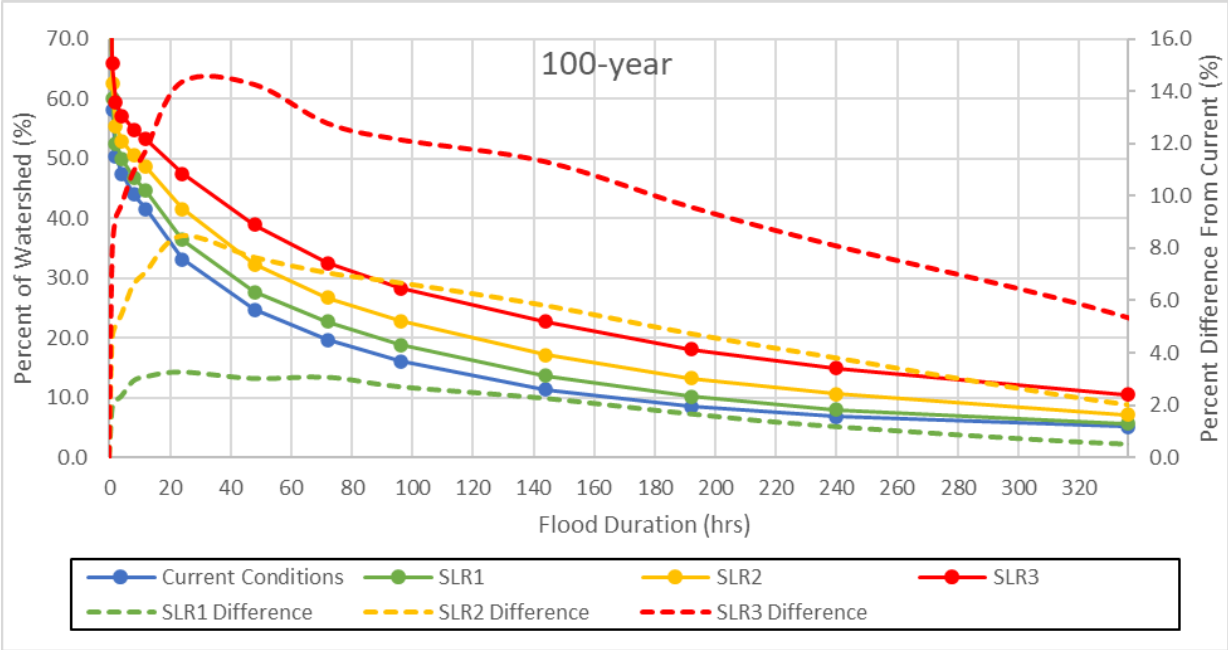


Figure 9-174. Percentage and Duration of Inundation for the C4 Watershed for the 25-year Storm Event

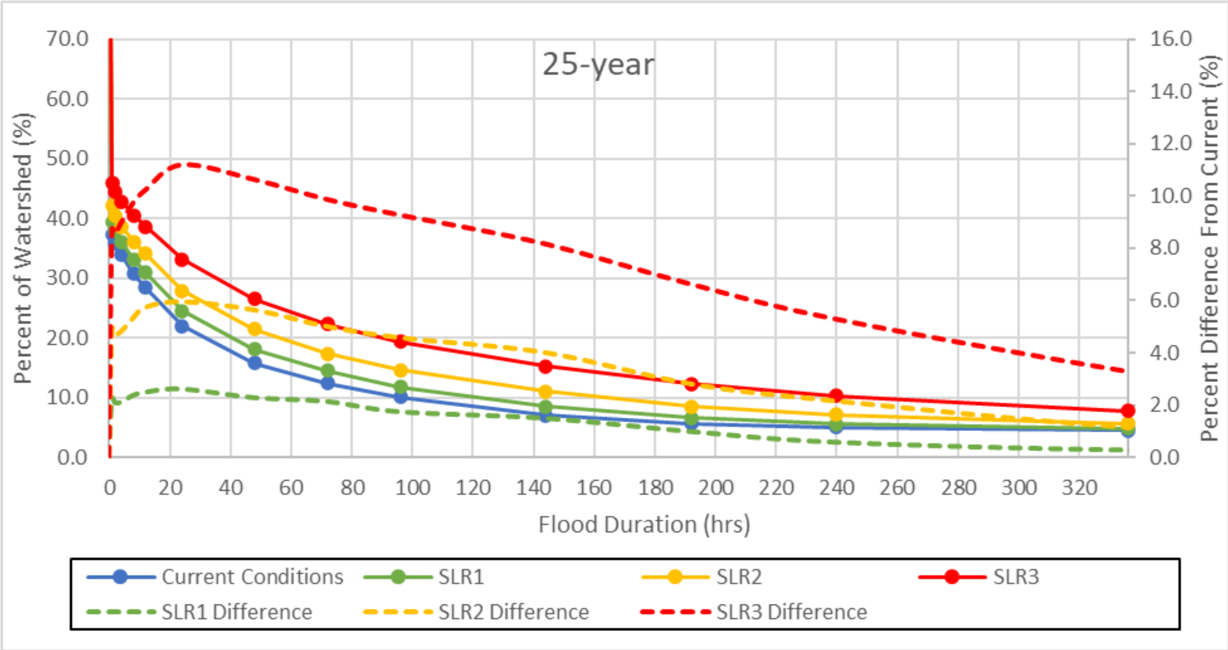


Figure 9-175. Percentage and Duration of Inundation for the C4 Watershed for the 10-year Storm Event

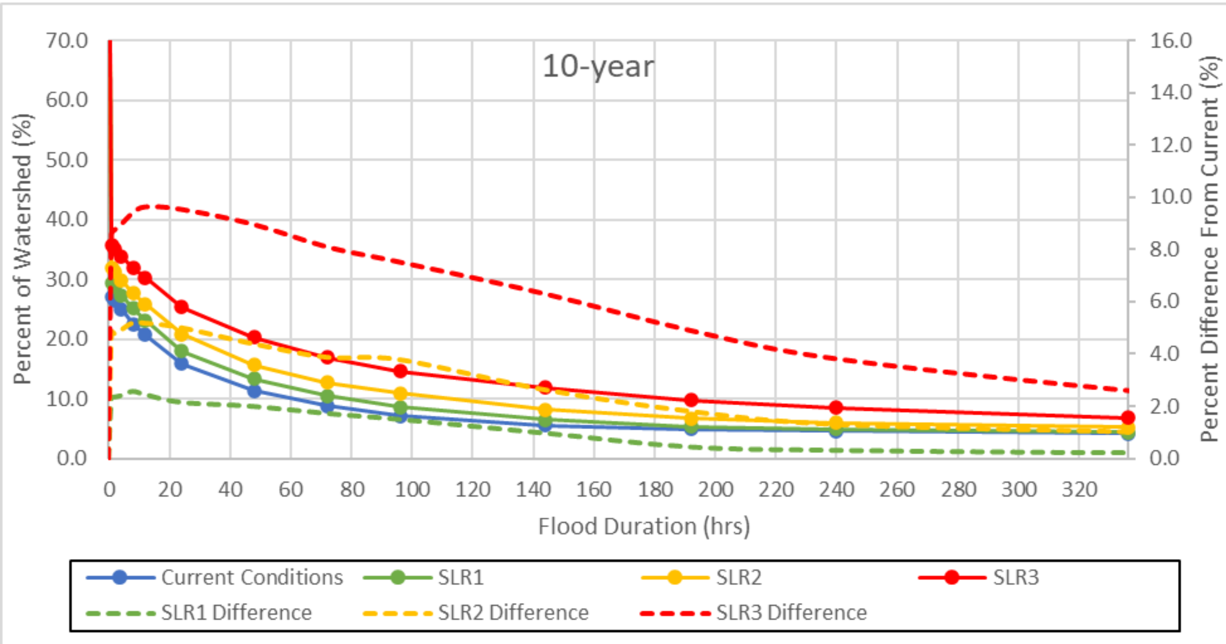


Figure 9-176. Percentage and Duration of Inundation for the C4 Watershed for the 5-year Storm Event

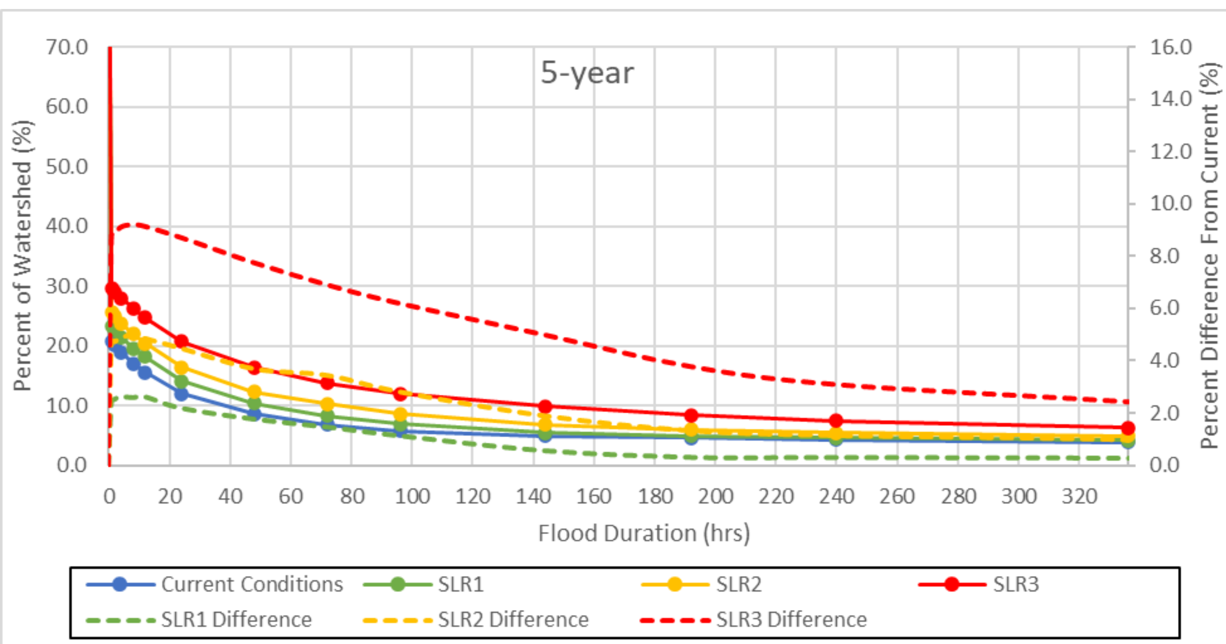


Figure 9-177. Flood Duration Map for the Current Conditions 25-year 3-day Design Storm for the C4 Watershed

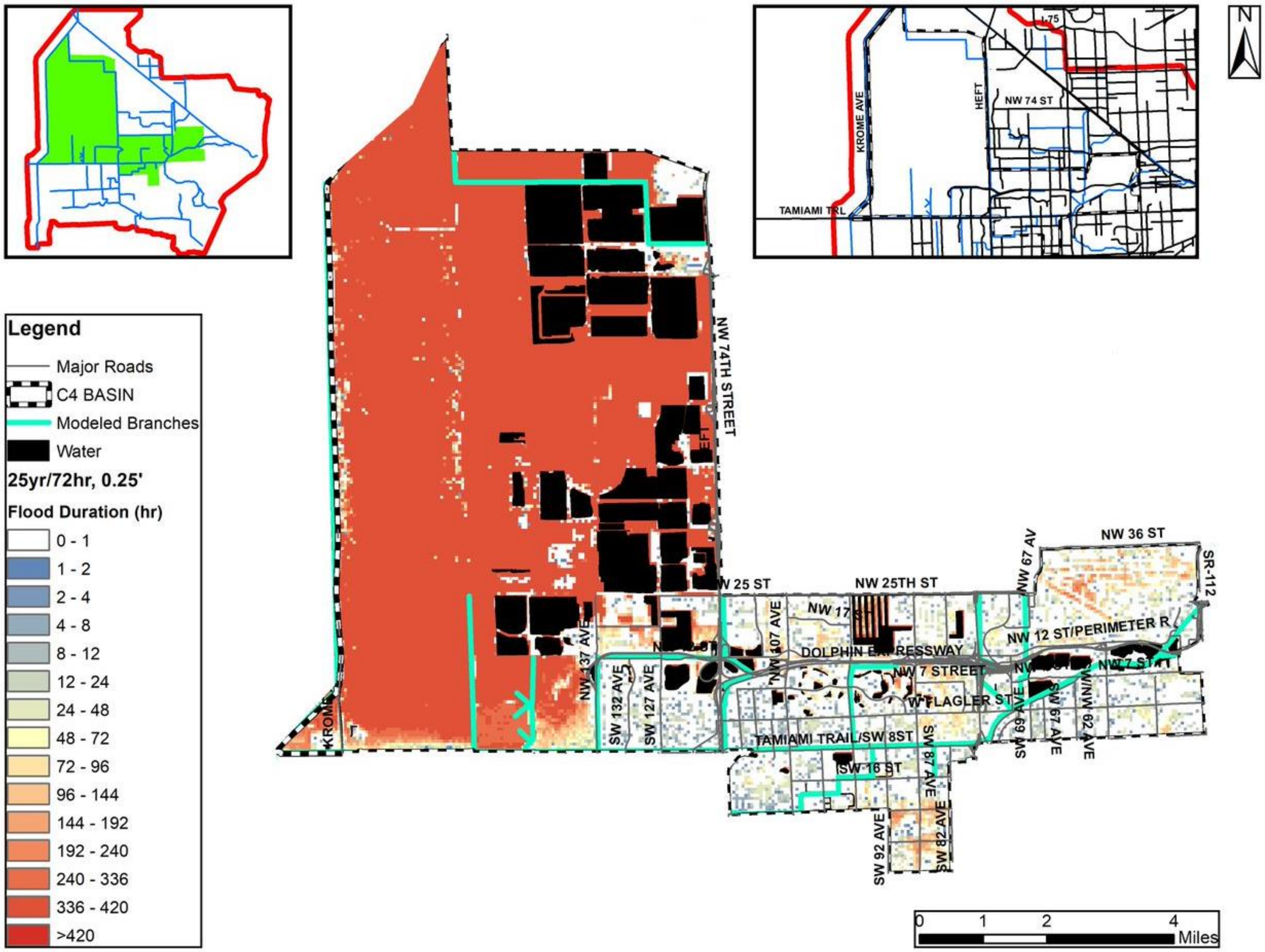


Figure 9-178. Flood Duration Map for the Current Conditions 100-year 3-day Design Storm for the C4 Watershed

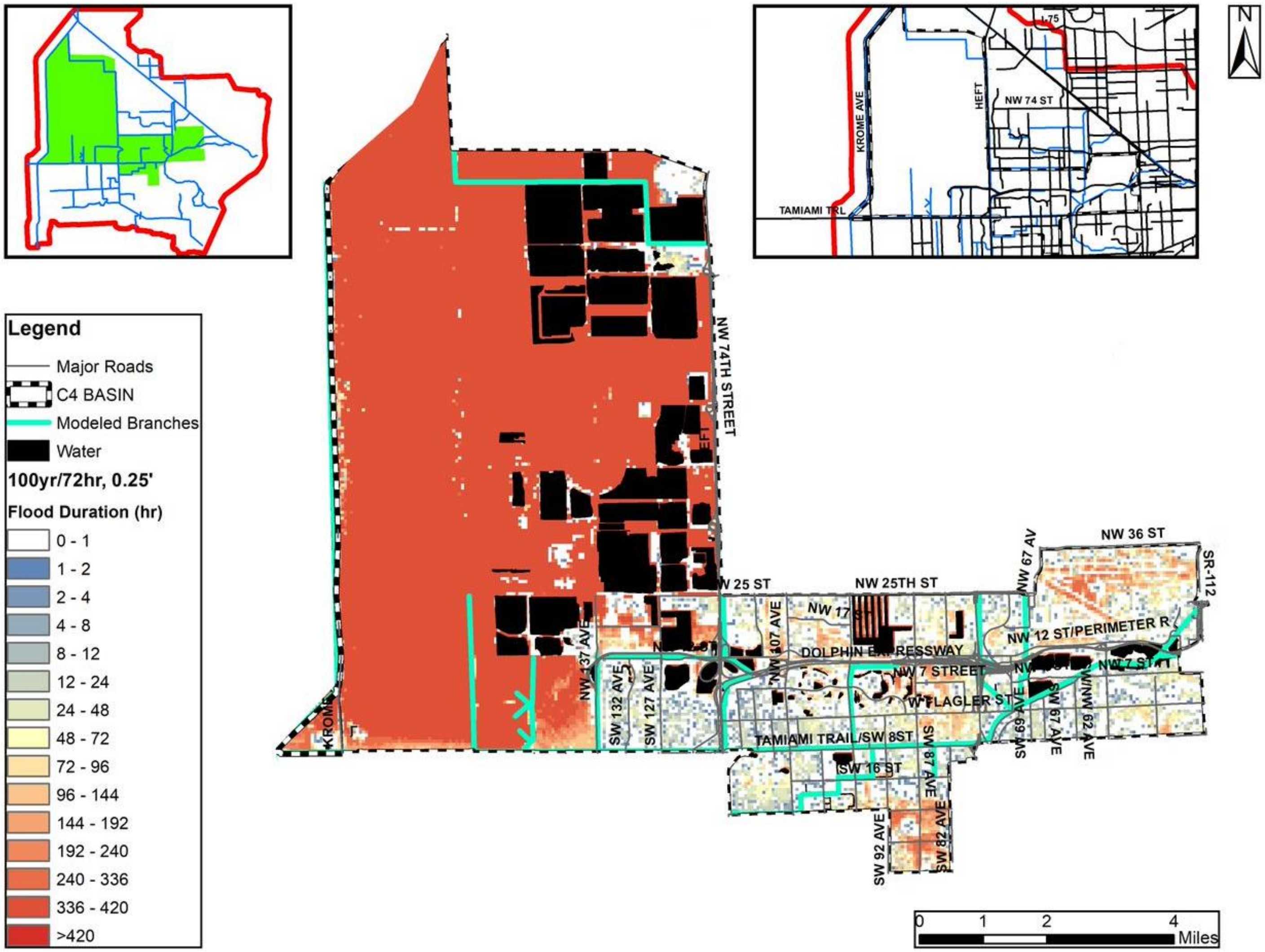


Figure 9-179. Flood Duration Map for the Current Conditions 25-year 3-day Design Storm for Urban Areas in the C4 Watershed

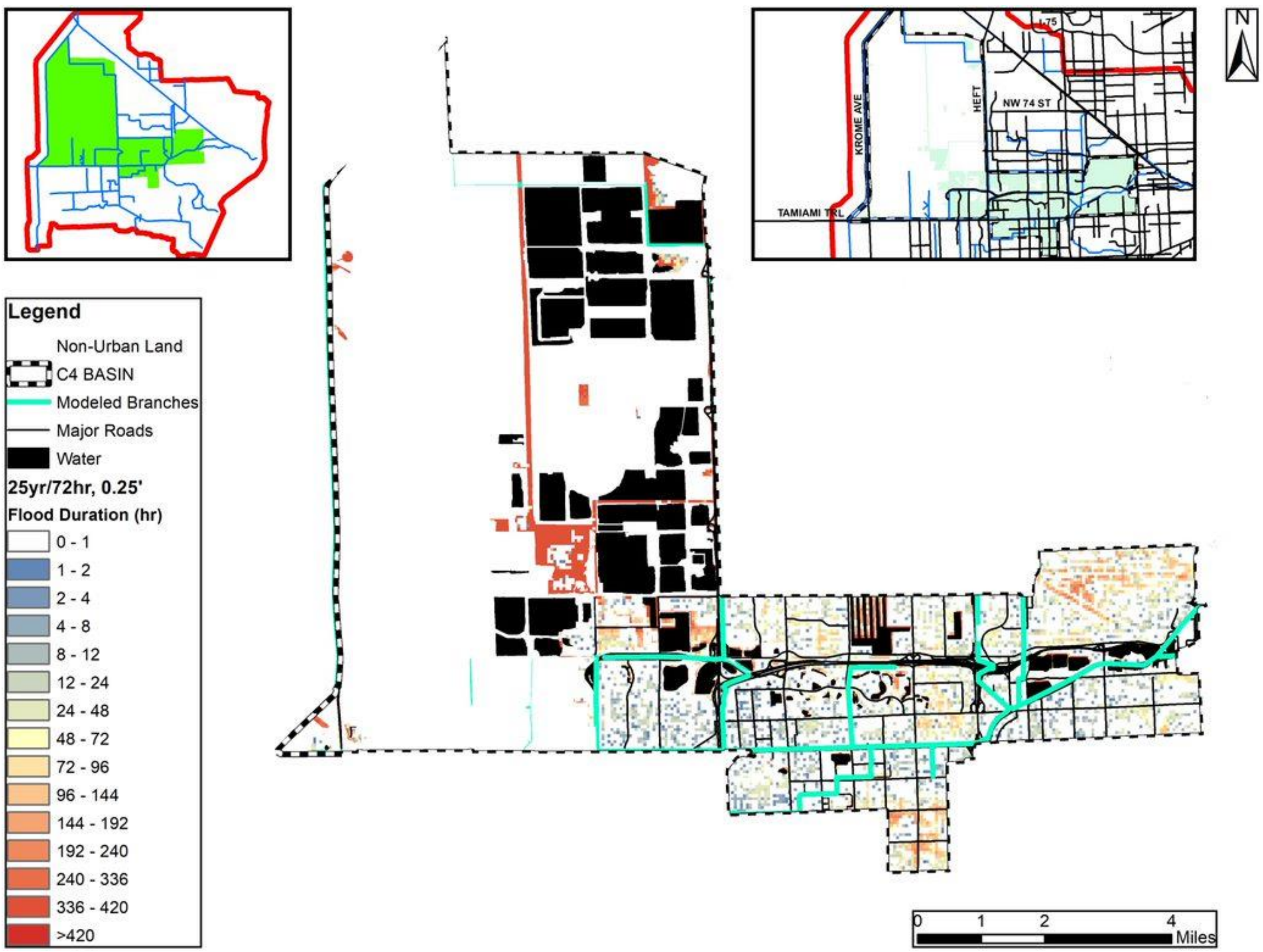


Figure 9-180. Flood Duration Map for the Current Conditions 100-year 3-day Design Storm for Urban Areas in the C4 Watershed

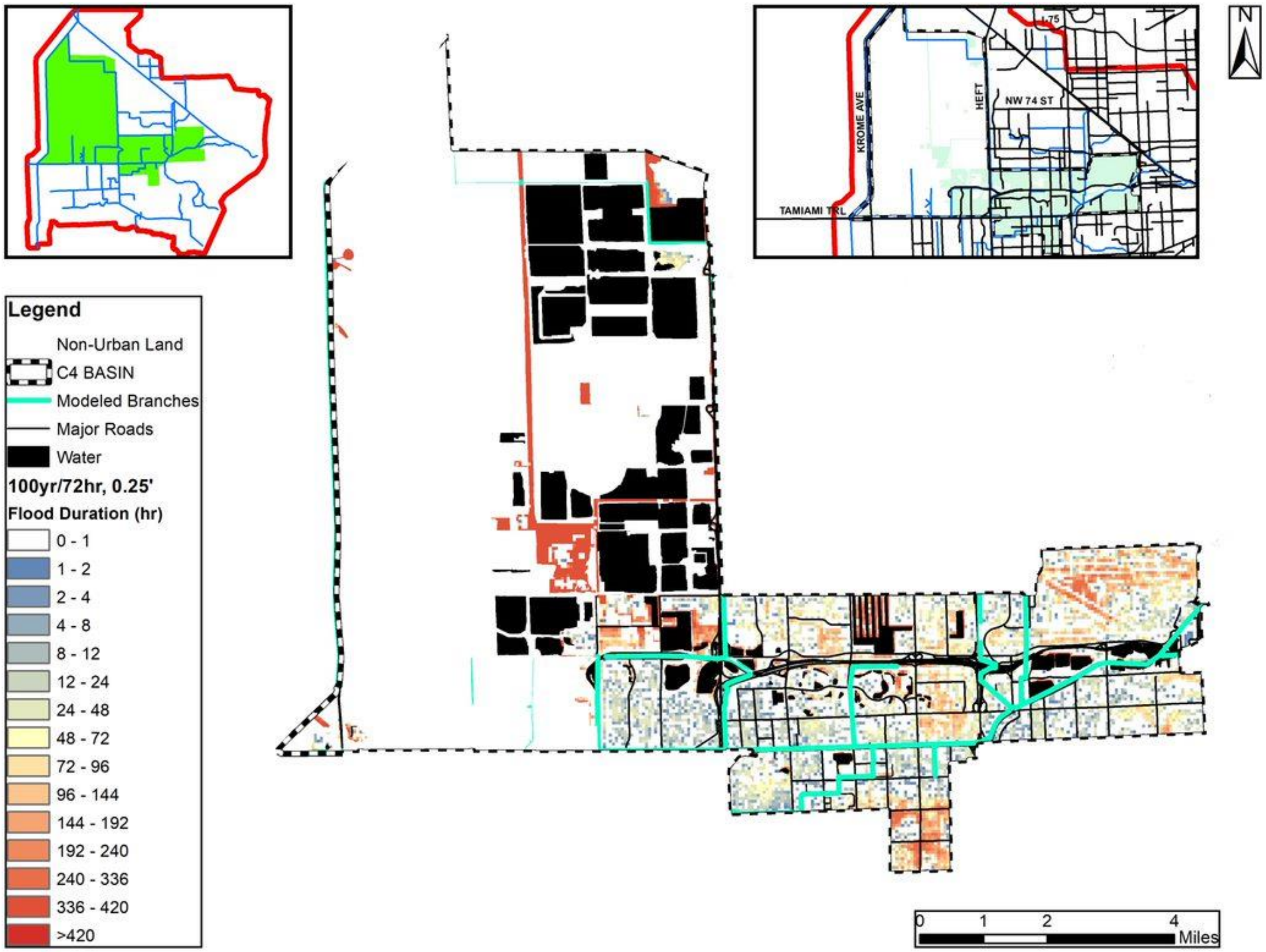


Figure 9-181. Urban Flooding Duration Difference of SLR +1ft and Current Cond. for the 100-year Storm in the C4 Watershed

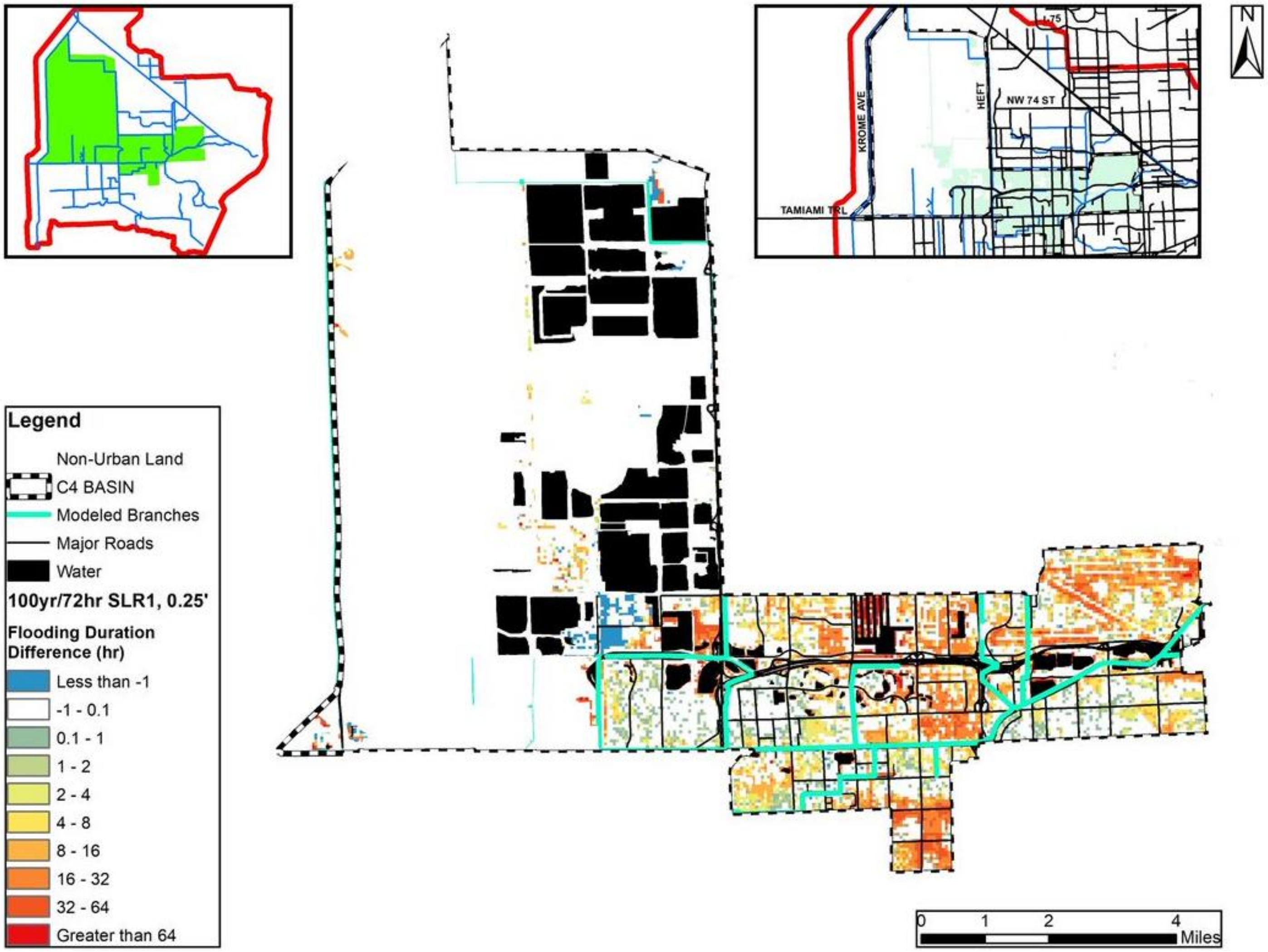


Figure 9-182. Urban Flooding Duration Difference of SLR +2ft and Current Cond. for the 100-year Storm in the C4 Watershed

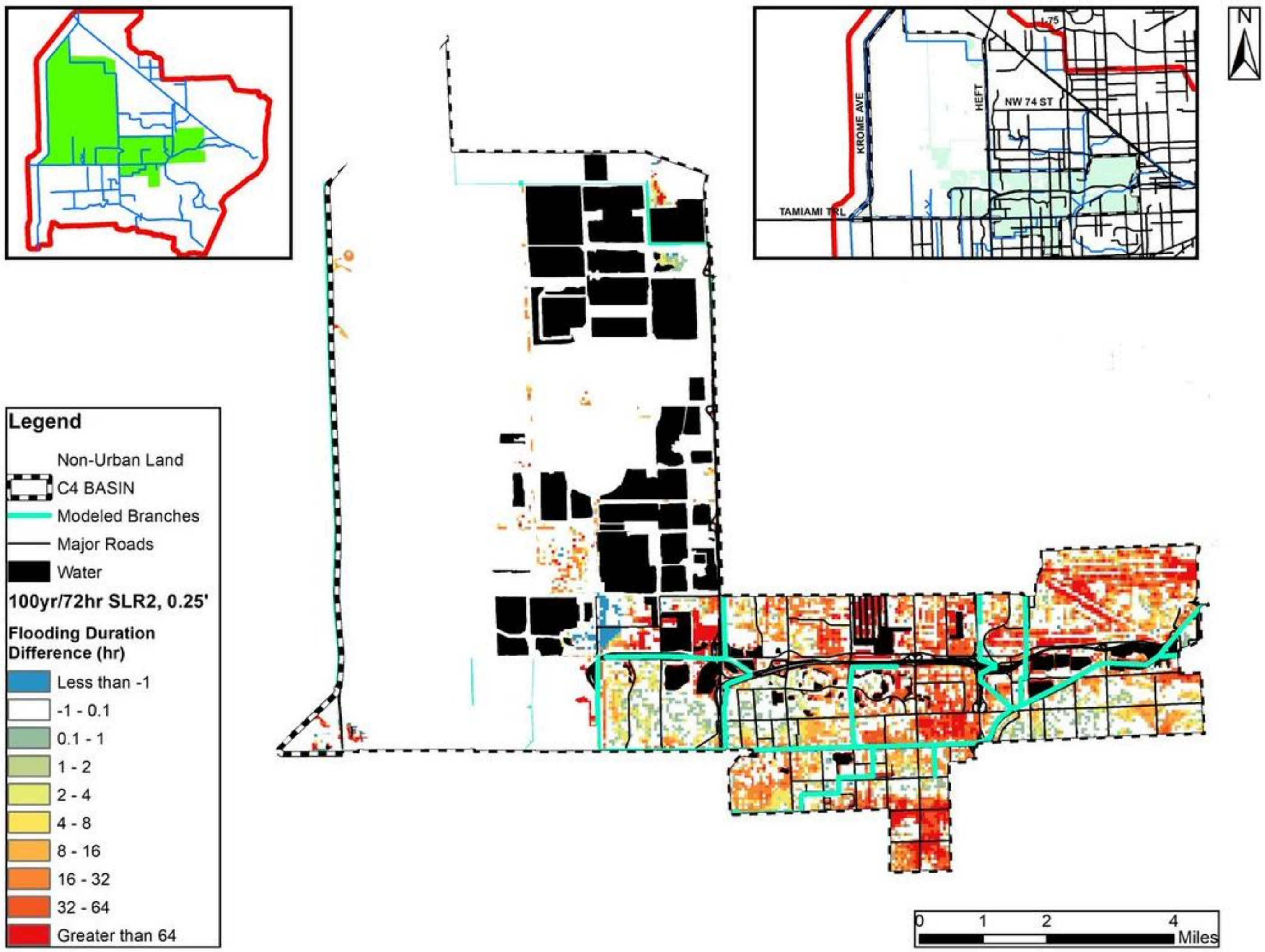
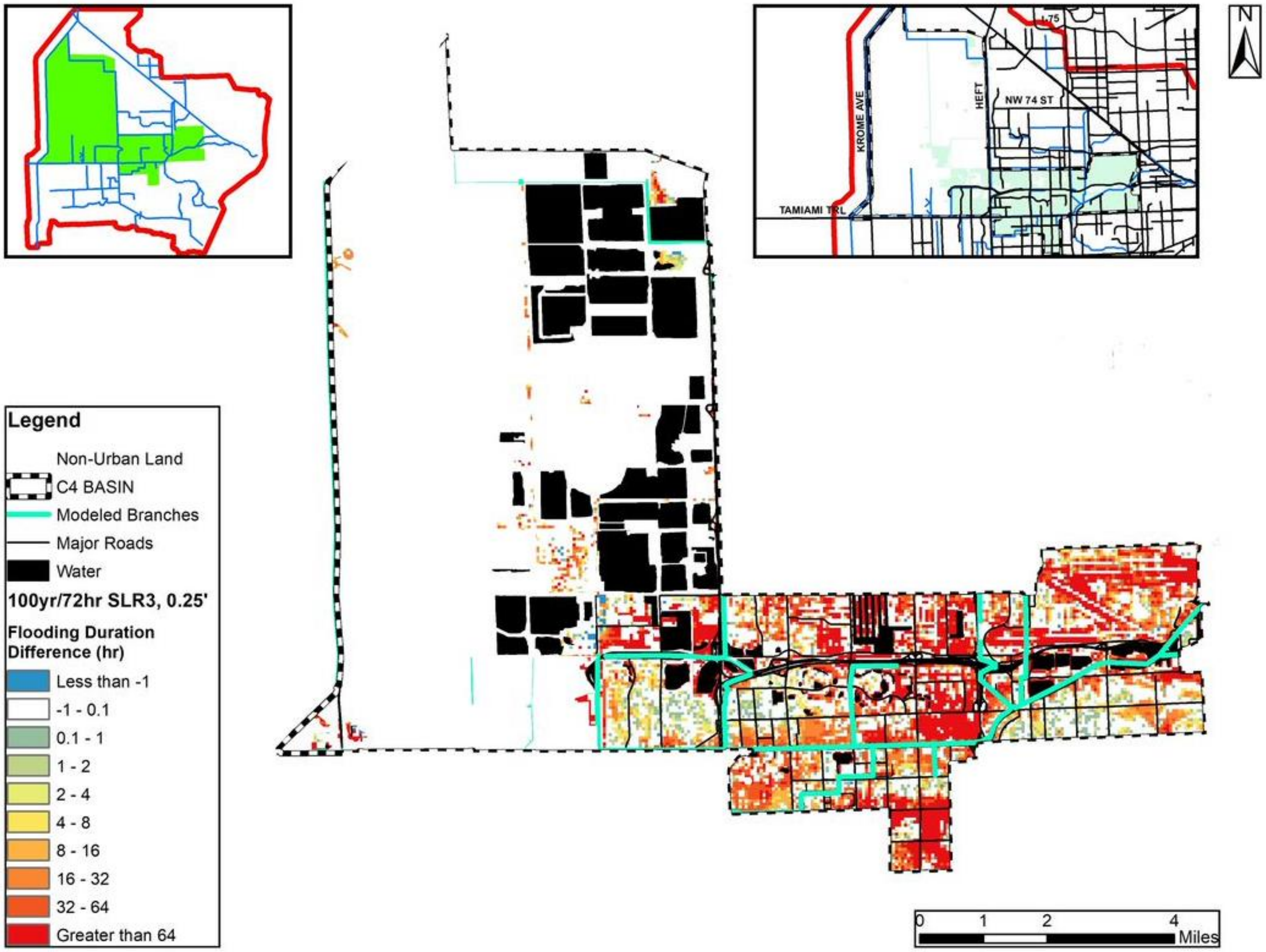


Figure 9-183. Urban Flooding Duration Difference of SLR +3ft and Current Cond. for the 100-year Storm in the C4 Watershed



For the C4 Watershed some areas tend to experience flooding for longer periods. Some areas of concern are 1) directly adjacent to the C4 Canal and just west of the Palmetto Expressway (SR 826), 2) the region jutting south to Bird Road from SW 82nd Avenue to SW 92nd Avenue, and 3) the industrial/commercial area north of the Dolphin Expressway (SR 836) from Miami International Mall to the Palmetto Expressway.

Figure 9-184 shows the overland flooding elevation in the Fontainebleau and Sweetwater region. This zoom in was generated by adding the model topography to the maximum overland water depth for the 100-year design storm simulation under current conditions. Points A and B, in the figure represent regions of flooding in low-lying residential neighborhoods that are unmanaged and serviced by a municipal pump station, respectively. In addition, point C represents a low-lying residential area within the Fontainebleau community that experiences flooding and has reported flooding according to the District's May 28, 2020, presentation titled "C-4 Data Analysis". **Figure 9-185** shows a picture of flooding observed in Sweetwater during the May 2020 storm event.

Figure 9-184. Overland Flood Elevation for the 100-year Design Storm in the area near Sweetwater and Fontainebleau Region

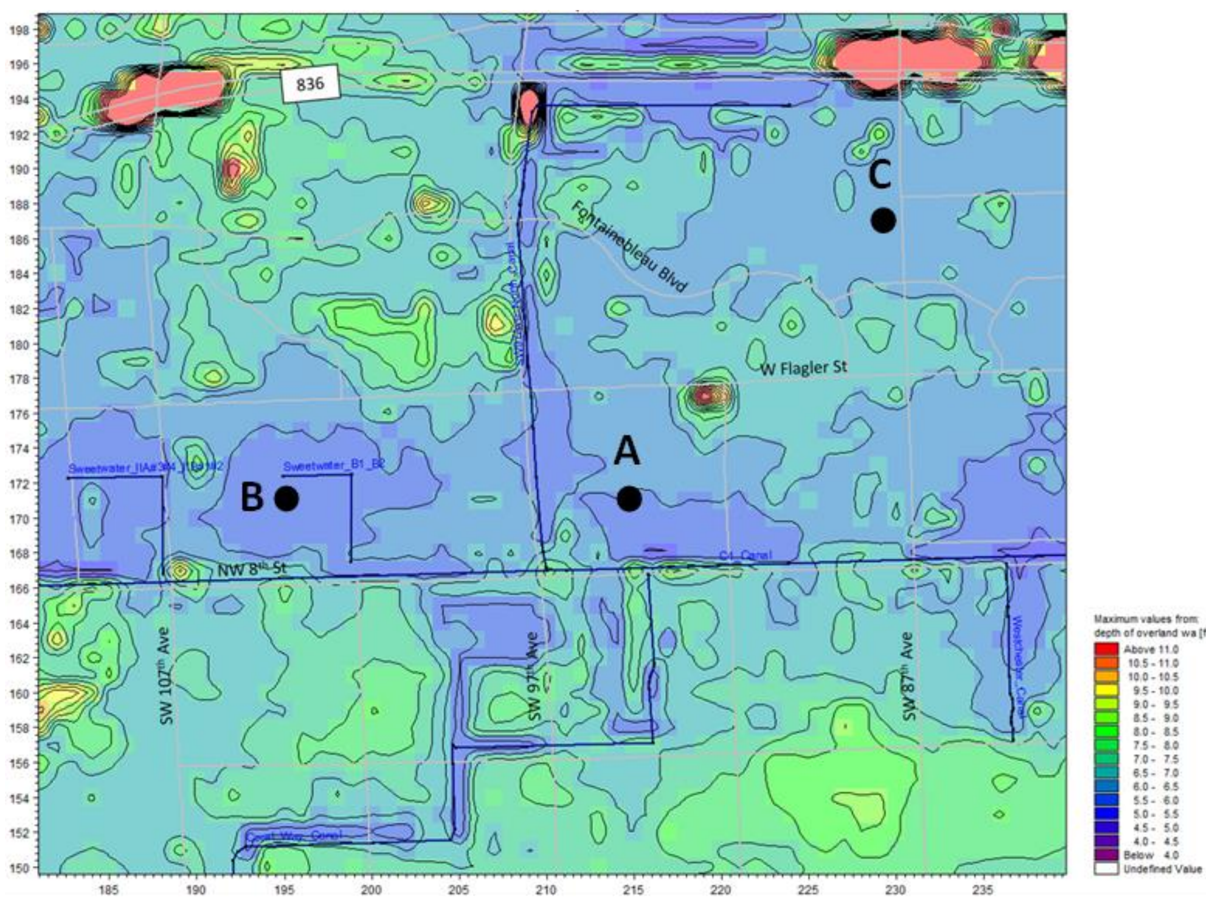
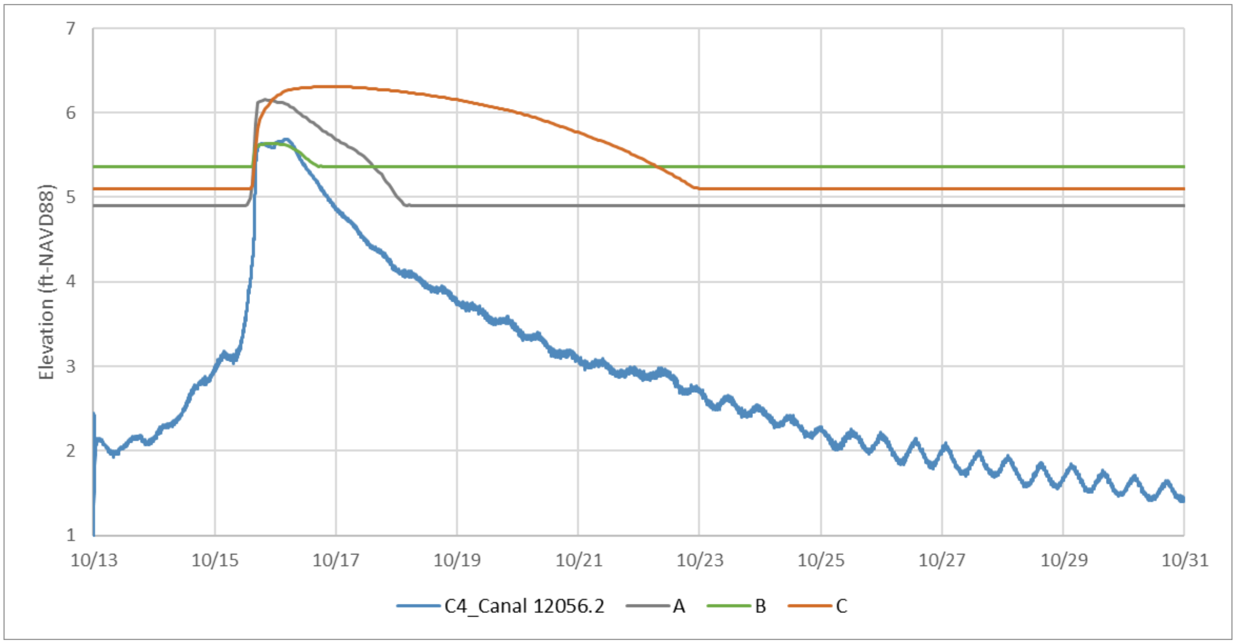


Figure 9-185. Photograph of Flooding in Sweetwater

Figure 9-186 shows a graph of the flooding elevation for the 100-year design storm under current conditions for points A, B and, C with comparative canal levels within the C4_Canal at the intersection with SW97Ave_North_Canal. As expected, point B, which represents an area serviced by a municipal pump stations experiences lower flooding depth and reduced flood duration for the storm event.

Figure 9-186. C-4 Canal Stages Compared with Overland Flood Elevation for 100-year Design Storm



9.3.7. SUMMARY OF LOS AND RATING FOR THE C4 WATERSHED

The maximum design storm frequency that the C4 Watershed passes without incurring negative impacts is summarized for each performance metric in **Table 9-61**.

Under critical consideration for this watershed is PM #1, PM #5, and PM #6, as these relate directly to flooding in canals and overland flooding depth and duration. For the current conditions, it was concluded that the C4 watershed can handle up to the 10-year storm event. The C4 Canal embankments (PM #1) were overtopped for less than a mile for the current conditions 25-year storm and very few culverts were overtopped; however, the overland flooding (PM #5) suggests that more than 0.75 feet (or 9 inches) of flooding may occur in neighborhoods adjacent to the canal, and overland flooding duration is longer than 48 hours for some neighborhoods farther away from the canals, such as the region between SW 92nd Avenue and SW 82nd Avenue north of Bird Road.

For the SLR1 condition, similar results to the current conditions 10-year storm were found for not only culverts and canal embankments, but also overland flooding depth and duration. The number of culverts overtopped doubles from SLR1 to SLR2. For SLR2 the length of canal embankment overtopped (PM #1) is similar to the SLR1 condition. In addition, the flooding depth and duration (PM #5 and PM #6) of the SLR2 5-year storm are similar to the SLR1 10-year storm.

SLR3 conditions and storm events showed significant increases in canal flooding, as discussed in PM #1, and direct flooding from the canals to the overland, as discussed in PM#5 and PM #6. Therefore, the SLR3 condition does not pass any of the simulated storm events.

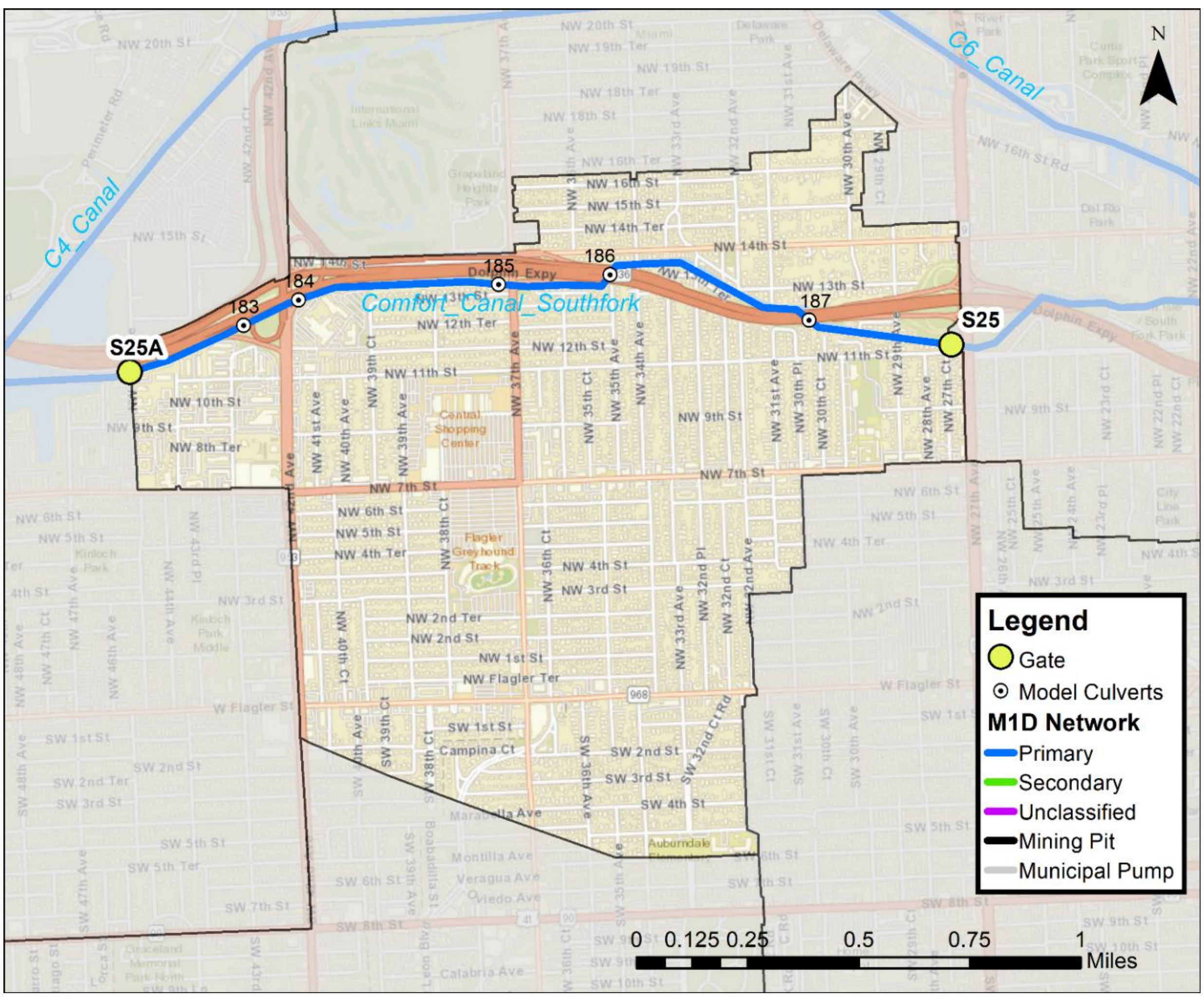
Table 9-61. Performance Metric Summary for the C4 Watershed

METRIC	NOTES	CURRENT CONDS.	SLR +1 FT	SLR +2 FT	SLR +3 FT
PM #1	▪ No design storms exceeded bridge low chords in the C4 Canal. ▪ Five (5) of 61 culvert locations in the watershed were overtopped for the 25-year storm during current conditions, this doubled for SLR1. Three (3) culvert locations were overtopped for the 10-year storm for SLR1, but this increased to nine (9) for SLR2. Three (3) culvert locations were overtopped for the 5-year storm for SLR2, but this increased to 15 for SLR3. ▪ The C4 Canal is overtopped for less than a mile for the current conditions 25-year storm, for the SLR1 and SLR2 conditions for 10-year storm, and for the SLR3 condition for the 5-year storm. Problem areas include Fountainbleau, Westchester, and Doral East.	25-year	10-year	5-year	< 5-year
PM #2	▪ No comparable value found for this basin. ▪ With the future canals, a new connection to the C4 Canal increases discharges from the C2 watershed, reducing drainage capacity for SLR1 conditions. Peak discharge capacity is delayed a full day for SLR2 and SLR3, when tailwater conditions have improved and pumping returns at S25B.	--	--	--	--
PM #3	▪ Maximum discharge at S25B falls below design value for the 5-year and 10-year storm for all current and future conditions. Max discharge is not affected by SLR. ▪ HW exceeds the water level that will bypass S25B for the 100-year and 25-year current conditions scenarios and all future SLR scenarios (SLR1, SLR2, SLR3) except for the 5-year SLR1 scenario. The TW exceeds this bypass elevation during the 100-year current conditions scenario and all future SLR scenarios (SLR1, SLR2, SLR3) except for the 5-year SLR1 scenario.	--	--	--	--
PM #4	▪ Peak 12-hour moving discharge ranges from 771 CFS to 2,159 CFS, compared to the design discharge of 2,000 CFS, and decreases with SLR.	--	--	--	--
PM #5	▪ 26.4% of the watershed is flooded with 0.75 in of depth or greater for the 100-year, 17.3% for the 25-year storm. Inundation at some of these locations is likely related to the canal stages overtopping the canal banks, however, some areas farther away from canals are also experiencing flooding at depths greater than 0.75ft. ▪ Difference maps suggest that OL flooding will increase primarily in areas adjacent to the C4 Canal such as Sweetwater and Fountainbleau.	10-year	10-year	5-year	< 5-year
PM #6	▪ Canal: Stages at the T5W station recede after 68 hours for the 25-year storm for current conditions, and for more than 3 days for the 100-year storm, which is longer than the duration of the storm event itself. For SLR1, the canal recedes in less than 72 hours for the 10-year storm. For SLR2 and SLR3, the canal takes longer than 4 days to recede for all storm events. ▪ Watershed: Percent of area inundated for more than 48 hours for the current conditions 25-year is over 15% and this increases with SLR.	10-year	10-year	5-year	< 5-year
Overall Level of Service		**10-year**	**10-year**	**5-year**	**<5-year**

9.4. C5 WATERSHED FLOOD PROTECTION LEVEL OF SERVICE

The C5 Watershed is a relatively small watershed that consists of areas north of SW 8th Street, west of NW 27th Avenue, east of Red Road, and south of NW 16th Street. The primary discharge canal is the Comfort Canal from the S25A water control structure to the S25 water control structure. S25A is gated culvert that is used for drainage and salinity control, located at NW 45th Avenue. S25 is a tidal water control structure that controls water levels in the canal, located just upstream of NW 27th Avenue. During the wet season the S25 structure is operated to maintain the low range water elevations of (-)0.745 to -0.345 ft-NAVD (0.8 to 1.2 ft-NGVD). During rain events, S25A is closed to prevent runoff from the C4 Watershed from entering the C5 Watershed. **Figure 9-187** shows a map of the C5 Watershed and the branches that are included in the model.

Figure 9-187. Map of the C5 Watershed



9.4.1. C5 – PM #1 MAXIMUM STAGE IN PRIMARY CANALS

The maximum stage in the primary canals was extracted from the results of each design storm simulation for the C5 Watershed. Bank elevations were also extracted from the model, as setup in the MIKE 1D model, to identify areas where peak stages exceed bank elevations, which could be an indication of overbank flooding next to the canal.

Table 9-63 provides the culvert information with the estimated crown of road from LiDAR data at each culvert location in the Comfort Canal, as well as the peak stage at the nearest H-point (where canal stages are calculated in the model). Stages which overtop the estimated crown of road are highlighted. Comfort Canal at NW 37th Avenue, just south of State Road 836, is lower than the 100-year design storm peak stage for current conditions according to the LiDAR data. This canal bank overtopping increase is present for all storm events for the SLR2 and SLR3 conditions. Further investigation of this area has found that NW 37th Avenue is in a low spot, which is discussed in PM #6. Additionally, the NW 42nd Avenue crown of road is overtopped for 100-year storm under SLR3 conditions.

The maximum stage profiles for the C5 Watershed are provided in **Appendix A**, with bank elevations and major intersections located along the canals. The figures provided in the Appendix show the entire length of the Comfort Canal Southfork from the C4 Canal to the S25 structure. **Figure 9-189** shows the maximum stage profile for the current conditions for all design events. In addition, **Figure 9-190** shows the profile for the 100-year storm event under all conditions. Canal top of bank elevations were established in the model from cross-section information from previous modeling efforts, including the HEC-RAS models (SFWMD, 2015), and converted to ft-NAVD. Full discussion of the development of these cross-sections is provided in **Section 3.3.1**. Top of bank elevations were extracted from the model, and the miles of the evaluated canal segment that is overtopped by the design storm was estimated upstream of the tidal control structure S25, as summarized in **Table 9-62**.

Table 9-62. Estimated Percentage and Miles of Bank Overtopped per Design Storm

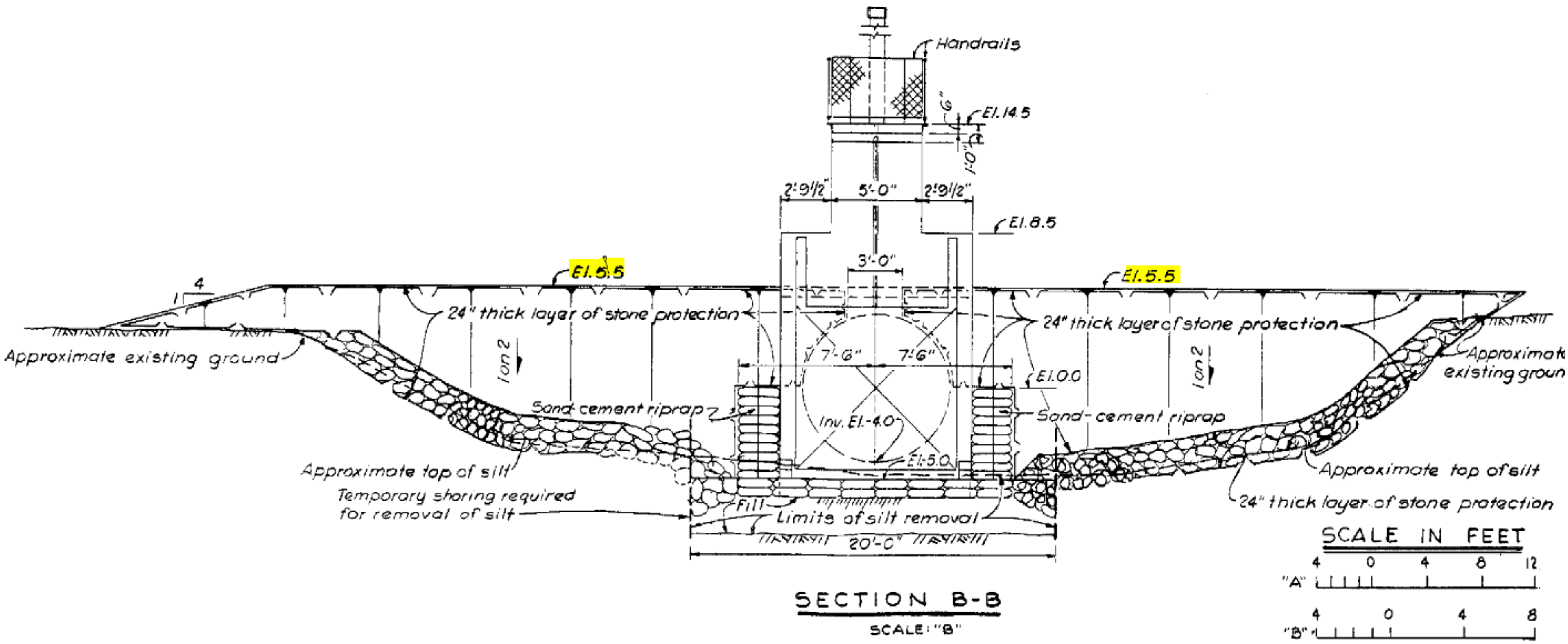
WATERSHED	SIMULATION	100-YEAR	25-YEAR	10-YEAR	5-YEAR
C5 (%)	Current Conditions	83%	0%	0%	0%
	SLR1	83%	83%	0%	0%
	SLR2	83%	83%	83%	83%
	SLR3	85%	83%	83%	83%
C5 (miles)	Current Conditions	1.9	0.0	0.0	0.0
	SLR1	1.9	1.9	0.0	0.0
	SLR2	1.9	1.9	1.9	1.9
	SLR3	2.0	1.9	1.9	1.9

Table 9-63. Estimated Culvert Crown of Road and Peak Stage for the C2 Watershed

Culvert Number	Intersection	Top of Culvert	Estimated Crown of	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Comfort_Canal_Southfork																			
183	836 ramp west	0.77	28.48	4.60	4.25	3.91	3.57	5.00	4.62	4.42	4.17	5.77	4.94	4.62	4.59	6.60	5.80	5.39	5.16
184	NW 42 Ave and 836 ramp east	0.30	5.80	4.62	4.25	3.90	3.56	4.98	4.62	4.41	4.16	5.75	4.93	4.62	4.58	6.52	5.78	5.38	5.16
185	NW 37th Ave	0.51	4.50	4.60	4.23	3.89	3.54	4.97	4.59	4.39	4.14	5.75	4.93	4.60	4.56	6.51	5.77	5.40	5.17
186	836 @ NW 35th Ave	1.87	17.27	4.60	4.20	3.86	3.52	4.94	4.58	4.37	4.12	5.78	4.93	4.61	4.55	6.55	5.80	5.43	5.20
187	836 @ NW 30th Ct	1.77	10.04	4.61	4.15	3.82	3.47	5.06	4.58	4.34	4.09	5.91	4.99	4.65	4.53	6.69	5.90	5.50	5.24

***Highlighted cells indicate the stages exceed the estimated crown of road.**

Figure 9-188. As-built drawing of S25 from 1972



*Elevations in ft-NGVD29

Figure 9-189. Maximum Stage Profile for the C5 Watershed for Current Conditions

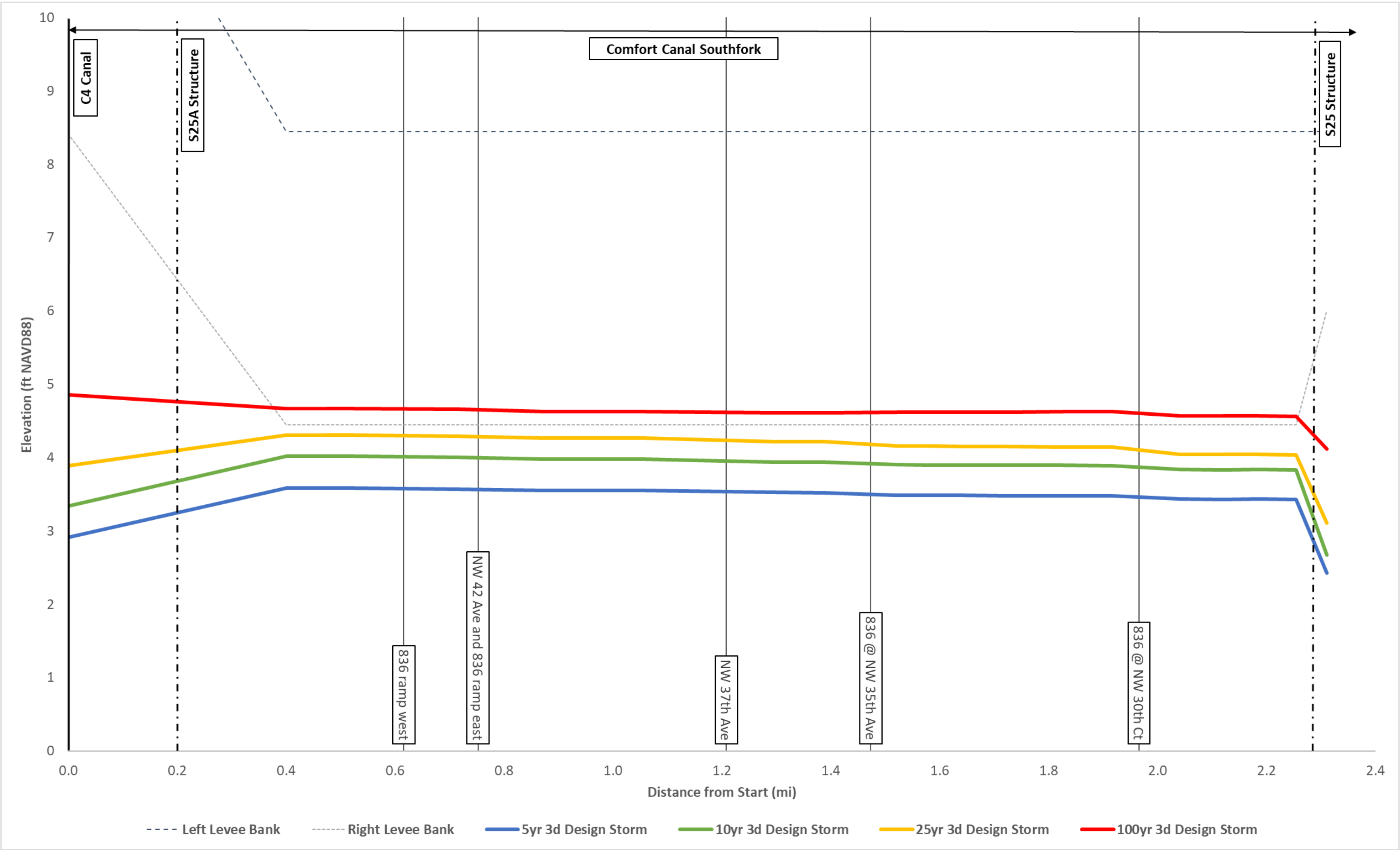
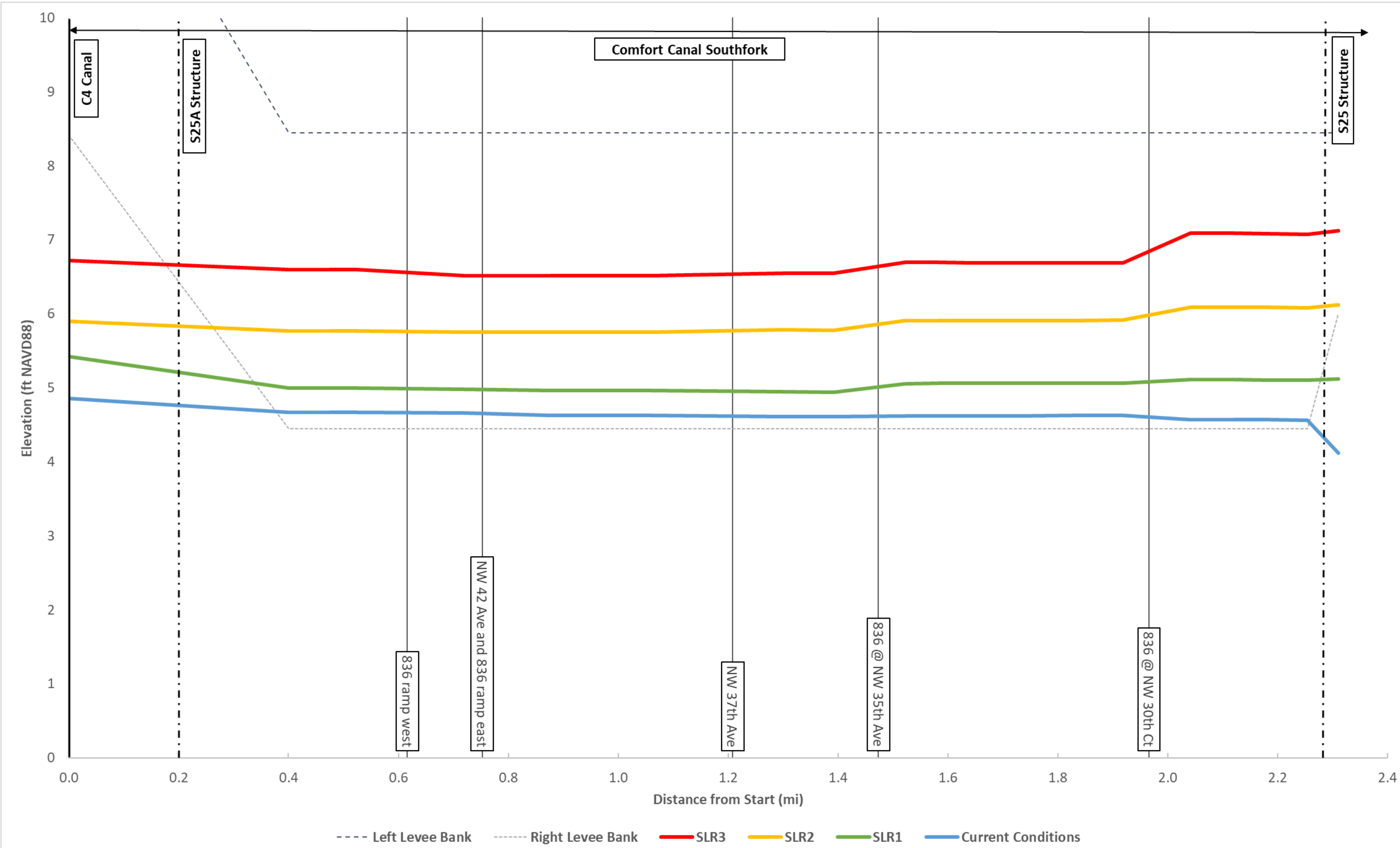


Figure 9-190. Maximum Stage Profile for the C5 Watershed for the 100-year Design Storm



As shown in the maximum stage profile in **Figure 9-190** the top of bank is overtopped for most of the Comfort Canal for the 100-year design event. These cross-sections were obtained from existing models (in the case of Comfort Canal Southfork, the cross-sections are from the HEC-RAS model of the C4 Watershed). In addition, the as-built drawings of the S25 structure indicate that the existing ground and top of bank are at around 3.94 ft-NAVD (5.5 ft-NGVD), as shown in **Figure 9-188**. In addition, the structure sheet provides information about the elevation at which the structure is bypassed, and this was found to be the same elevation.

9.4.2. C5 – PM #2 MAXIMUM DISCHARGE CAPACITY

The maximum discharge capacity for the C5 Watershed is the sum of the discharges out of the watershed minus the incoming flows weighted by the total area of the watershed. For the C5 Watershed this means flows from the S25A structure are subtracted from the outflows at the S25 structure, these locations are shown in **Figure 9-187**. S25A, which connects the C4 Canal to Comfort Canal, remains closed during the event.

The timeseries at each inflow and outflow location was extracted, and inflows were subtracted from the outflows and divided by the basin area in square miles (sq. mi.) at each timestep as shown in the equation below.

$$\frac{Outflows[cfs] - Inflows[cfs]}{Basin\ Area\ [sq.mi.]} = Discharge\ Capacity[CSM]$$

The maximum discharge capacity for the C5 Watershed for each design event is shown in **Table 9-64**. In addition, **Table 9-65** provides the 12-hour moving average of the maximum discharge capacity. The tables also provide discharge in cubic feet per second (CFS) at inflow and outflow locations for the watershed. The C5 Watershed is unique among the watersheds in this study, in that it typically does not experience direct canal interactions with other watersheds, as the S25A structure remains closed in the wet season, meaning that the total flow at the S25 structure is solely from areas contributing to the Comfort Canal Southfork (i.e., the C5 Watershed) and base flow in the canal. However, during higher SLR conditions, the S25A structure is also overtopped and will interact with the C4 Canal.

The ERP Applicant's Handbook Volume II does not indicate an allowable discharge upstream of S25 for the C5 Watershed.

Table 9-64 indicates that the peak discharge is higher for the 10-year design storm than for the 25-year design storm under current conditions. This is due to a spike at the start of the storm event; however, this spike is not as prominent with the future SLR conditions. **Figure 9-192** shows the instantaneous discharge capacity in CSM for the C5 Watershed, which clearly demonstrates the 5-year and 10-year storm events spiking right at the peak of the storm and the 25-year and 100-year storms peaking later after the tailwater recedes. The 5-year and 10-year storms both peak as soon as the gates are fully open, and the initial spike of flow is greater for the 5-year than the peak of the 25-year storm because it not only has a higher head differential at the structure, but also does not experience drowned flow as the tailwater is lower than the culvert/gate opening (i.e., the fully opened gate is at 2.455 ft-NAVD whereas the tailwater for the 5-year storm is at 2.38 ft-NAVD). Further details of the structure discharge can be found in **Section 7.3**, and **Table 9-67** provides the conditions that occur at the time of peak discharge through S25.

Figure 9-193 shows the 12-hour moving average discharge capacity in CSM, which shows that the averaged discharge for the 5-year is less than the averaged discharge for the 10-year storm for current conditions. To further evaluate this, **Table 9-65** shows the 12-hour moving average peak discharge for the C5 Watershed, which clearly demonstrates decreasing discharge with decreasing intensity of storms.

Additionally, the peak discharge capacity for all future SLR conditions are similar in scale to the current conditions; however, the reversal of discharge capacity is much greater for both the SLR2 and SLR3 conditions, due to the increased overtopping flows at S25 as a result of SLR. This creates a second peak for these two SLR conditions, where the canals begin to drain out due to the receding surge that had originally overtopped the structure. Further discussion of the structure overtopping can be found in **Sections 7.3** and **7.4**. In addition, **Figure 9-191** shows the cumulative negative flows at the S25 structure, which are the flows that occur when the structure is overtopped and the tailwater (or tidal side) is higher than the headwater. This creates a condition in which the sea surges into the watershed over S25, this equates to 11 acre-feet for SLR1, 97 acre-ft for SLR2, and 229 acre-ft for SLR3. In addition, tides continue to overflow the structure for SLR3 and add to the flow volume with each high tide.

Figure 9-191. Cumulative negative flows overtopping the S25 structure for the 100-year Design Storm

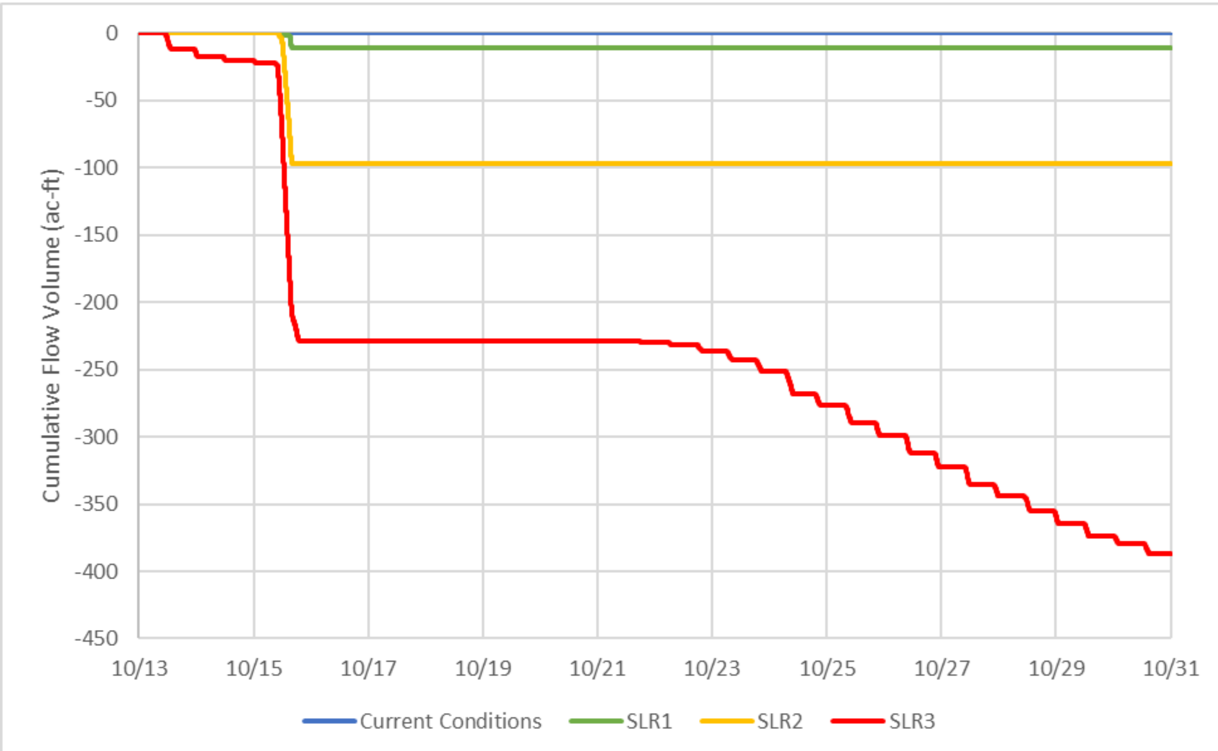


Table 9-64. Peak Discharge (CFS/sq. mi.) from the Contributing Drainage Area of the C5 Watershed

	Current Conditions				SLR1				SLR2				SLR3			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Inflow Locations																
S25A (CFS)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-15.5	0.0	0.0	0.0	-19.3	-0.6	0.0	0.0
Outflow Locations																
S25 Total Flow (CFS)	528.1	387.1	420.2	384.1	534.8	439.6	349.4	278.1	479.4	376.3	351.4	326.3	547.7	429.3	373.4	333.5
Watershed Summary																
Basin Area (Square Mile)	1.9				1.9				1.9				1.9			
Peak Watershed Discharge (CSM)	279.5	204.9	222.4	203.3	283.1	232.7	185.0	147.2	262.0	199.2	186.0	172.7	300.2	227.6	197.6	176.5

Table 9-65. 12-Hour Moving Average Peak Discharge (CFS/sq. mi.) from the Contributing Drainage Area of the C5 Watershed

	Current Conditions				SLR1				SLR2				SLR3			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Inflow Locations																
S25A (CFS)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-15.5	0.0	0.0	0.0	-19.3	-0.6	0.0	0.0
Outflow Locations																
S25 Total Flow (CFS)	259.2	181.9	129.5	100.9	237.3	209.5	173.9	144.4	246.9	188.3	159.7	141.7	297.7	217.7	179.6	162.4
Watershed Summary																
Basin Area (Square Mile)	1.9				1.9				1.9				1.9			
Peak Watershed Discharge (CSM)	137.2	96.3	68.6	53.4	125.6	110.9	92.0	76.5	130.7	99.7	84.6	75.0	157.6	115.2	95.1	86.0

Figure 9-192. Instantaneous Discharge Capacity for the C5 Watershed for 100- year, 25- year, 10- year, and 5-year 3-day Design Storm Events

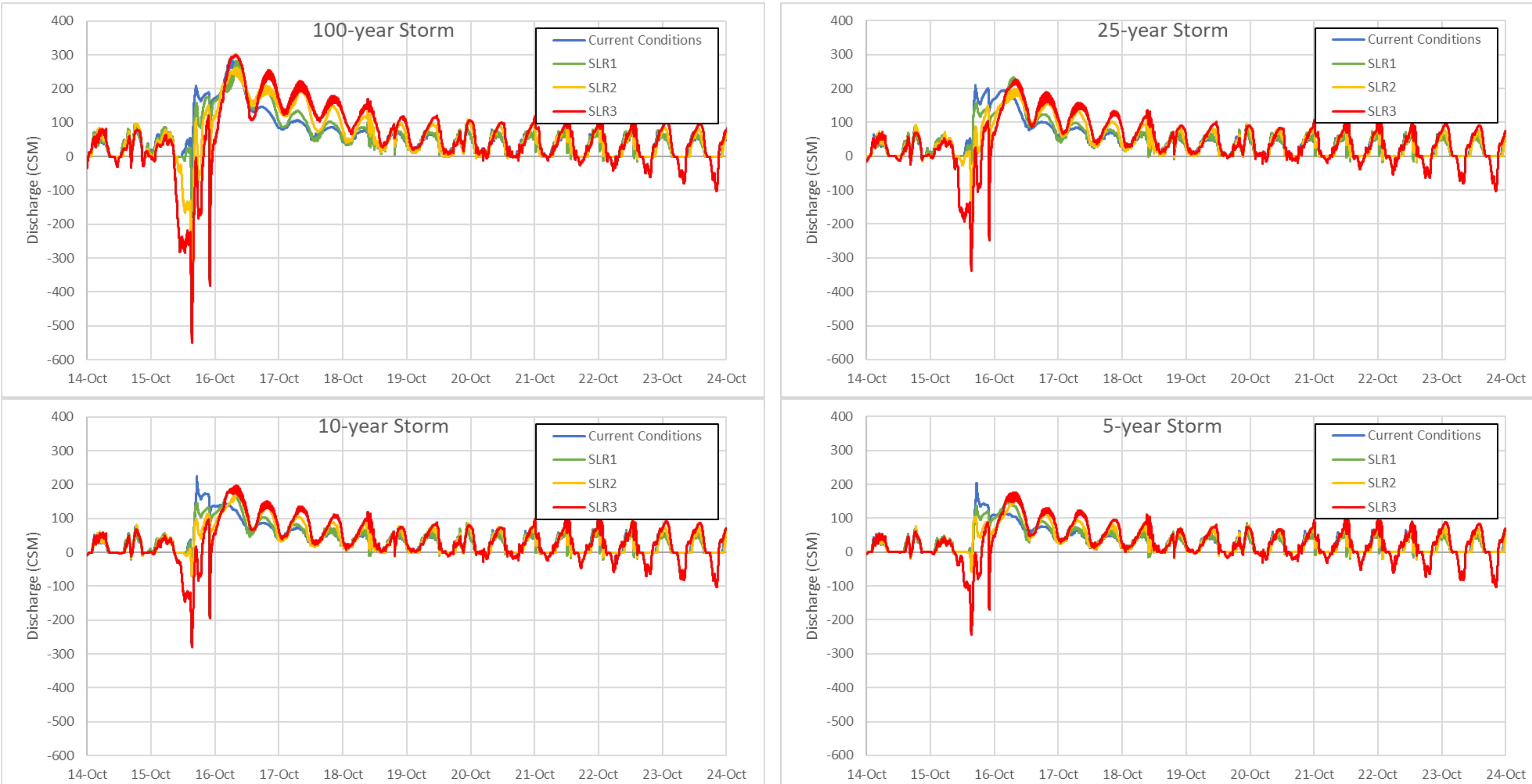
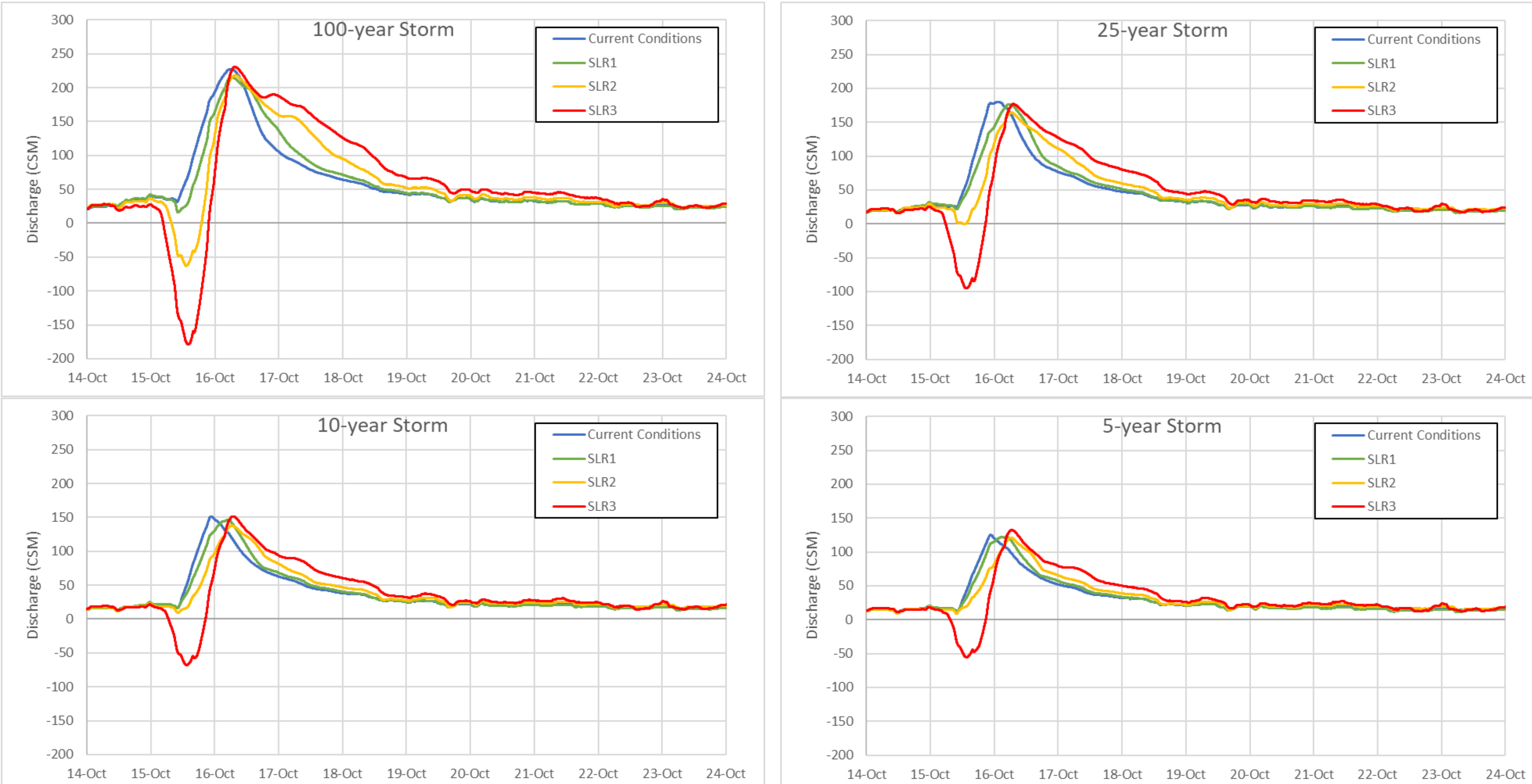


Figure 9-193. 12-Hour Moving Average Discharge Capacity for the C5 Watershed for 100- year, 25- year, 10- year, and 5-year 3-day Design Storm Events



9.4.2.1. INTER-BASIN TRANSFERS

During wet season operations, the C5 Watershed connection with the C4 Watershed at S25A is closed, closing off the watershed to receiving only runoff from the contributing areas to the canal. However, during some of the future SLR conditions, water may exceed the top of the S25A structure, which will act as a weir and flows will exchange between the C5 and C4 Watersheds. The top of this structure is 4.95 ft-NAVD. **Figure 9-194** and **Figure 9-195** show the exchanges between the C5 and C4 Watersheds at the S25A structure for the 100-year and 25-year design storms, respectively. For most of the SLR conditions, the flow tends to move from the C5 into the C4 Watershed, shown as negative flows at S25A. However, during the SLR3 100-year storm, the flows exchange back and forth from both watersheds (or oscillates between negative and positive). These positive flows indicate moments when the stages in the C4 Canal are higher than the Comfort Canal Southfork downstream of S25A and are overtopping the structure.

Figure 9-194. Exchanges between the C5 and C4 Watersheds at S25A for the 100-year Storm

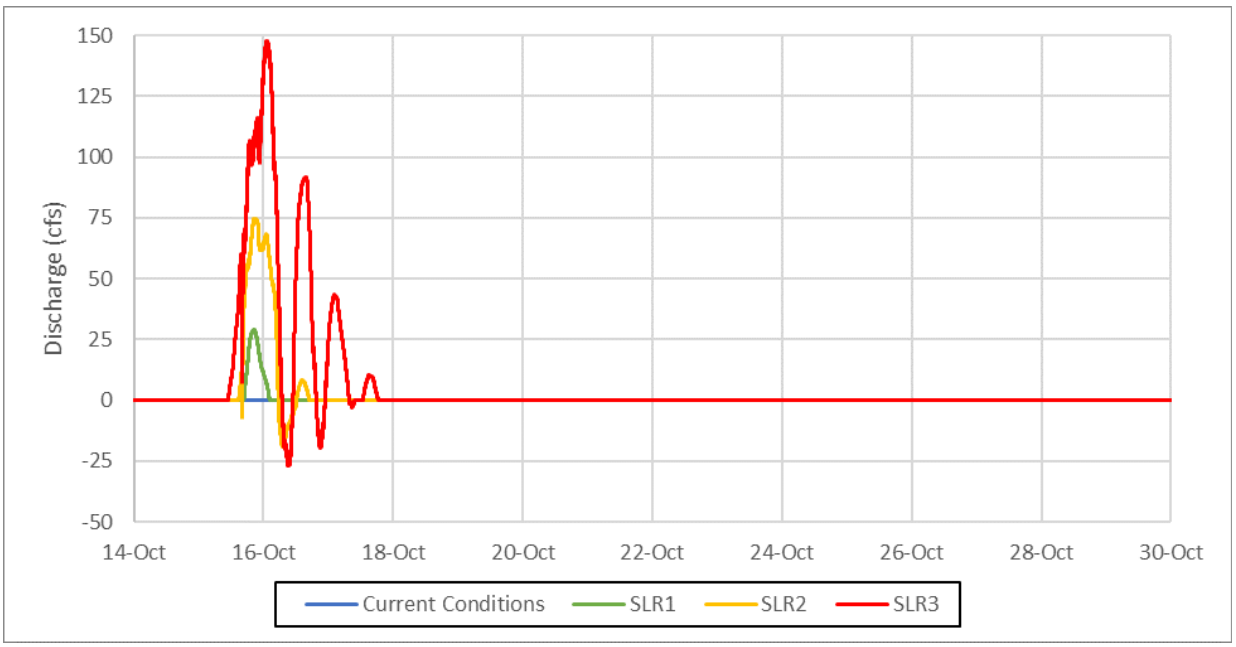
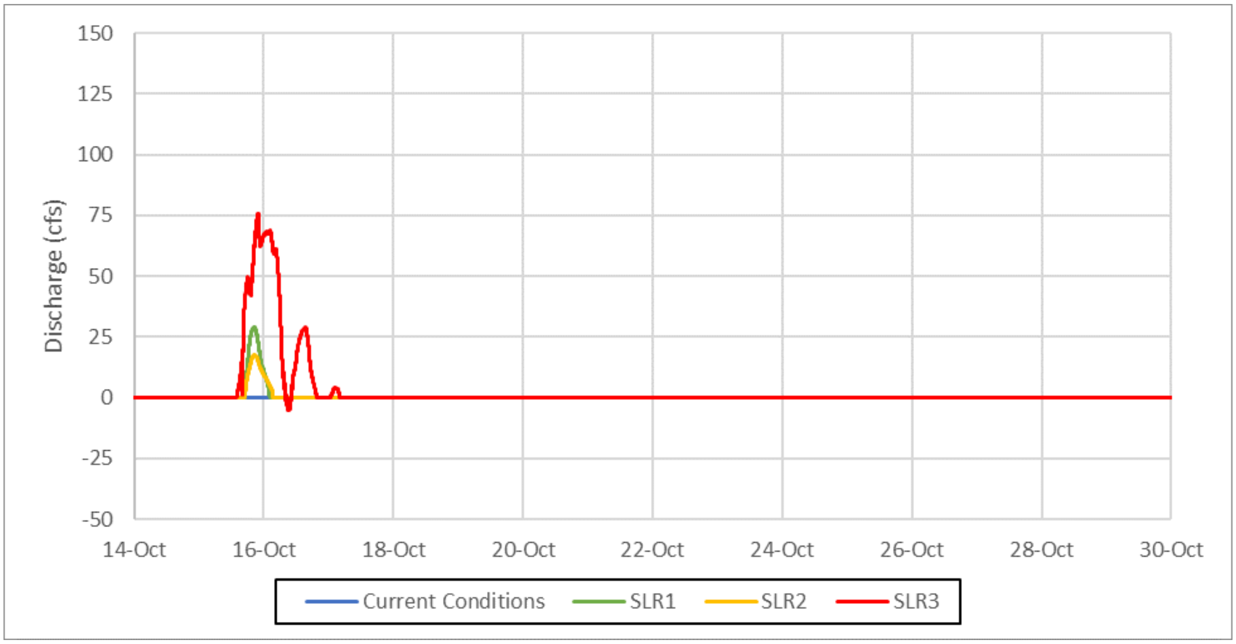


Figure 9-195. Exchanges between the C5 and C4 Watersheds at S25A for the 25-year Storm



9.4.3. C5 – PM #3 STRUCTURE PERFORMANCE

The purpose of PM #3 is to determine the effect of SLR on tidal structure flow performance. For this metric, an evaluation based on structure design features and existing operational protocols was conducted for existing conditions and compared to simulations including SLR over the various storm events. PM #3 will provide only gravity flows through the structure, where PM #4 will also account for structure flows as well as overtopping, to differentiate between structure capacity and the basic inabilities of the structure to provide tidal protections.

As per the structure data sheet, the S25 tidal structure is operated with the intention to maintain water levels in the C5 Watershed and pass the design discharge without impacting upstream flooding. In addition, the structure is used to restrict flows to decrease stages and velocities that may cause damage to downstream areas, while preventing saline intrusion during high tides. **Table 9-66** provides the design parameters for structure S25 as provided in the Water Control Operations Atlas.

Table 9-66. Design Parameters for Structure S25

DESIGN PARAMETERS	S25
Design Discharge	320CFS; 100 CFS (1 in 10-year flood)
Design HW	0.26 ft-NAVD (1.8 ft-NGVD)
Design TW	0.16-ft-NAVD (1.7 ft-NGVD)
Optimum HW	0.46 ft-NAVD (2.0 ft-NGVD)
Optimum TW	N/A
Maximum Gate Opening	8.0 ft
Water Level which will Bypass Structure	3.76 ft-NAVD (5.3 ft-NGVD)
Water Level which will Overtop Gates when Closed	3.76 ft-NAVD (5.3 ft-NGVD)
Low Range Operational Trigger	-0.74 ft-NAVD (0.8 ft-NGVD) to -0.34 ft-NAVD (1.2 ft-NGVD)

Figure 9-196 and **Figure 9-197** provide the instantaneous discharge through the gate, HW and TW at S25 during the current conditions simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-198** and **Figure 9-199** provide the 12-hour moving average for the gate discharge, HW, and TW at S25 during the current conditions simulation for the 100-year and 25-year design events, respectively, which removes the influence of the tides. **Figure 9-200** and **Figure 9-201** provide the instantaneous discharge through the gate, HW and TW at S25 during the future conditions SLR1 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-202** and **Figure 9-203** provide the 12-hour moving average for the gate discharge, HW, and TW at S25 during the future conditions SLR1 simulation for the 100-year and 25-year design events, respectively, which removes the influence of the tides. **Figure 9-204** and **Figure 9-205** provide the instantaneous discharge through the gate, HW and TW at S25 during the future conditions SLR2 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-206** and **Figure 9-207** provide the 12-hour moving average for the gate discharge, HW, and TW at S25 during the future conditions SLR2 simulation for the 100-year and 25-year design events, respectively. **Figure 9-208** and **Figure 9-209** provide the instantaneous discharge through the gate, HW and TW at S25 during the future conditions SLR2 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-210** and **Figure 9-211** provide the 12-hour moving average for the gate discharge, HW, and TW at S25 during the future conditions SLR2 simulation for the 100-year and 25-year design events, respectively. Figures are provided for instantaneous discharge and 12-hour moving discharge for all design storms and SLR simulations in **Appendix B**.

Figure 9-196. Instantaneous Discharge and Stages at S25 for the 100-year 3-day Current Conditions Event

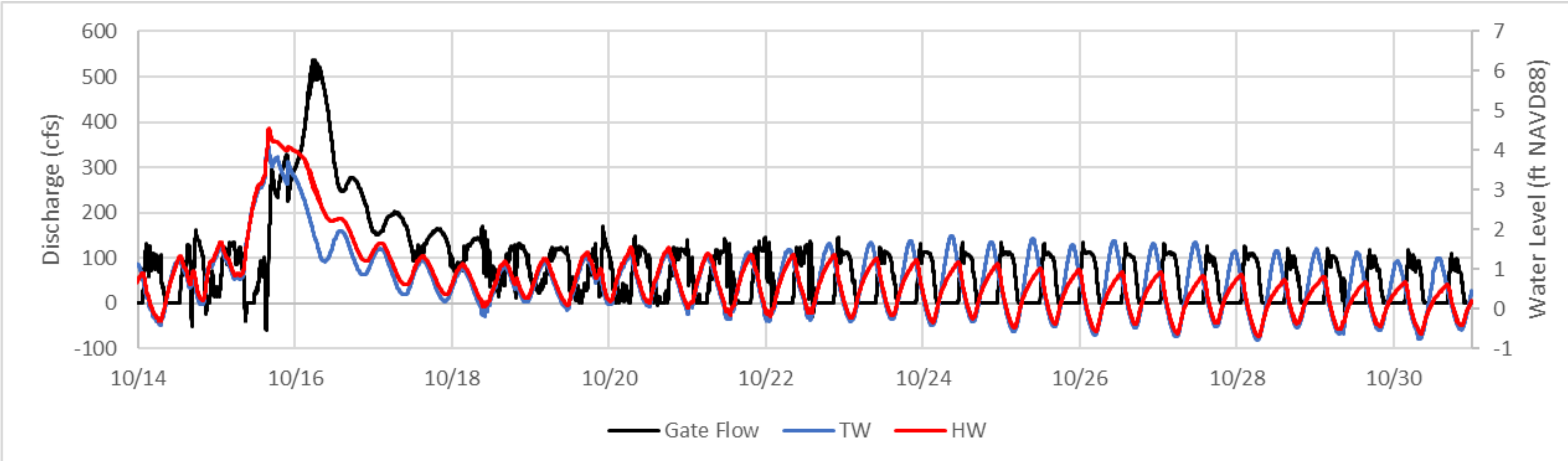


Figure 9-197. Instantaneous Discharge and Stages at S25 for the 25-year 3-day Current Conditions Event

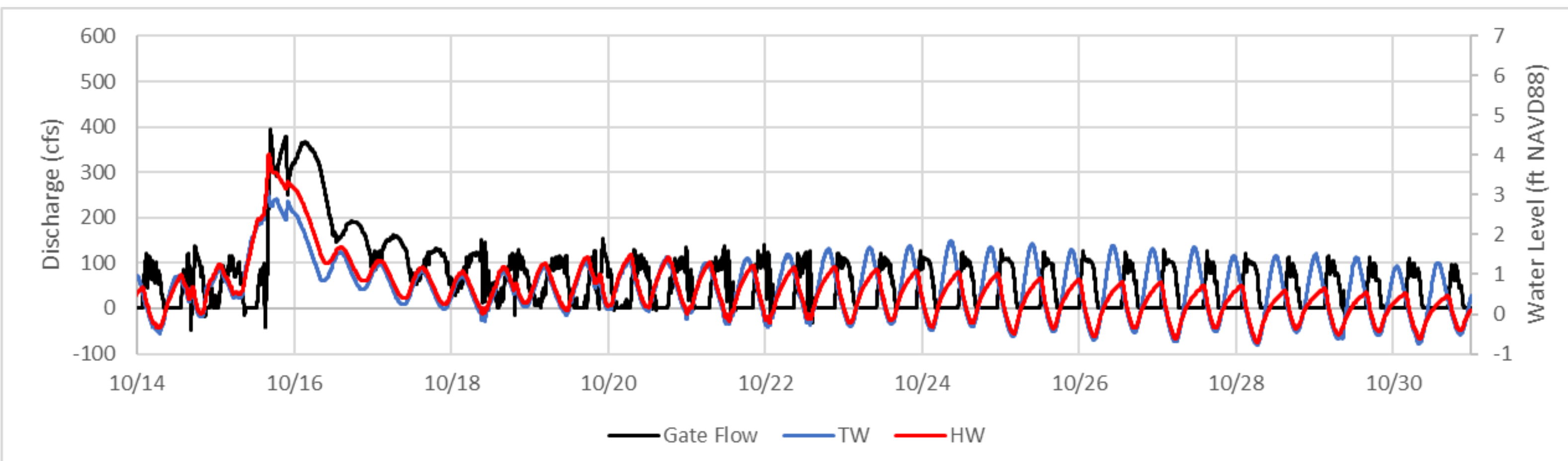


Figure 9-198. 12-Hour Moving Average Discharge and Stages at S25 for the 100-year 3-day Current Conditions Event

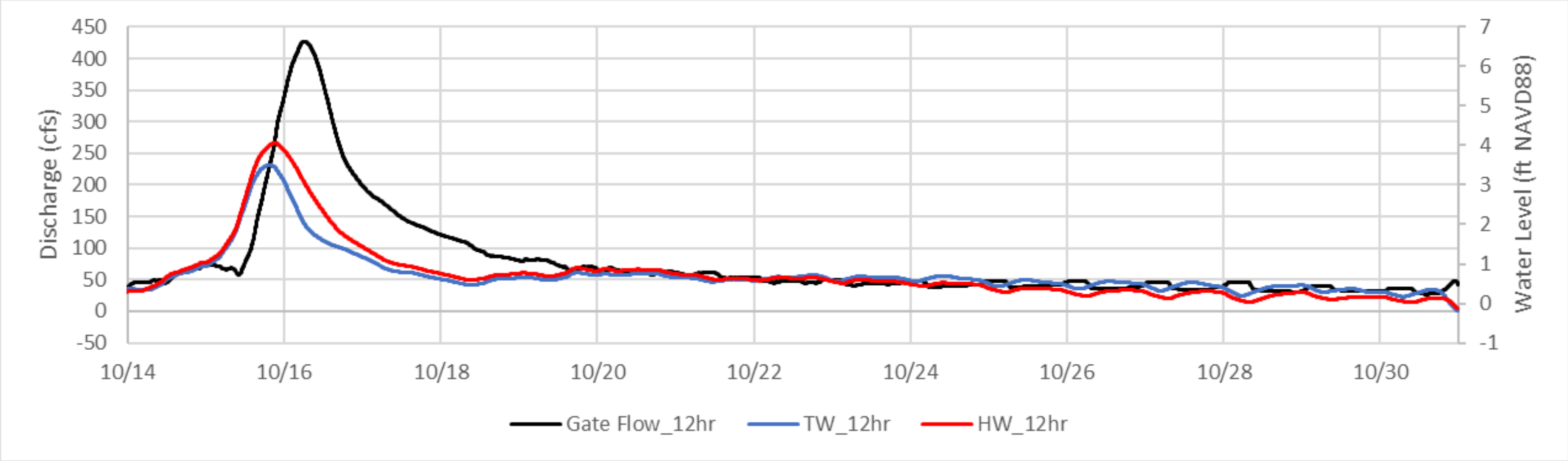


Figure 9-199. 12-Hour Moving Average Discharge and Stages at S25 for the 25-year 3-day Current Conditions Event

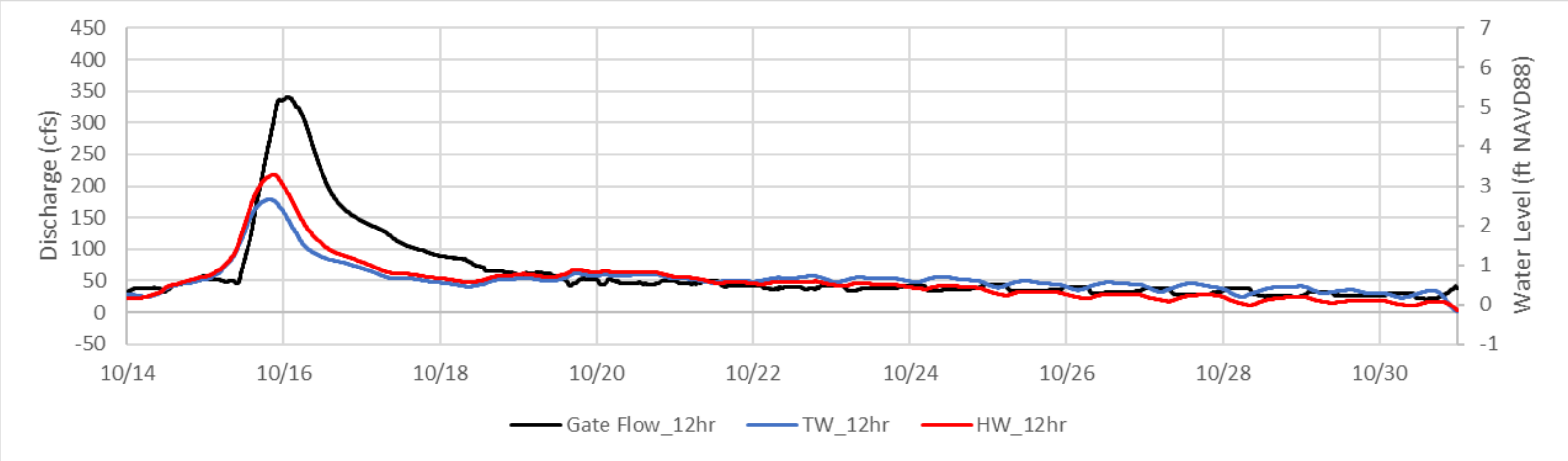


Figure 9-200. Instantaneous Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR1) Event

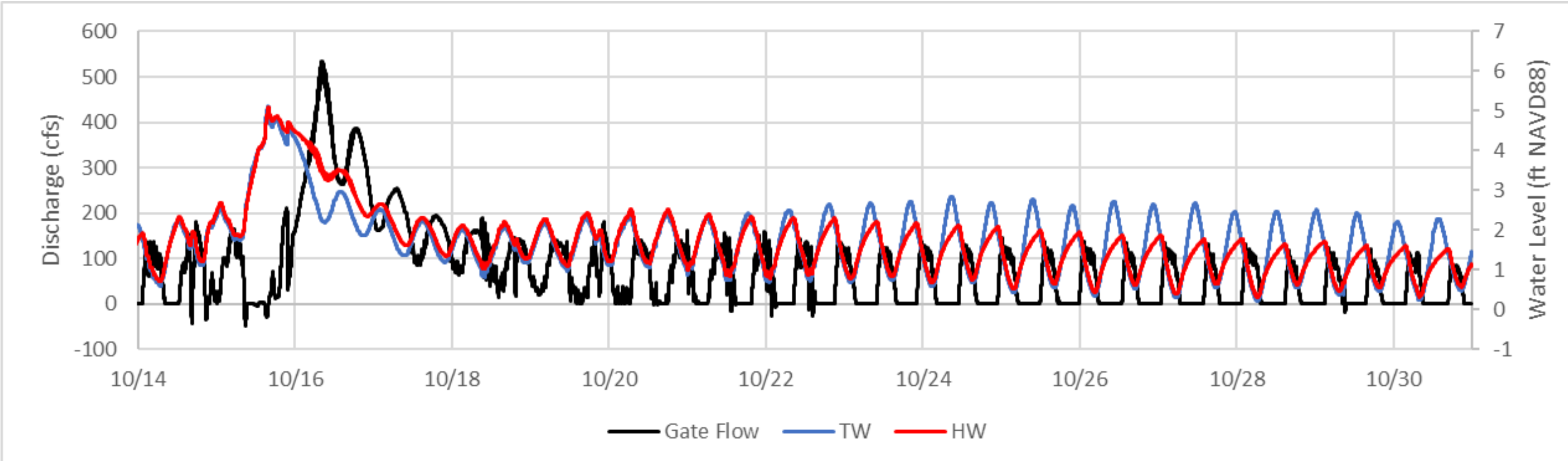


Figure 9-201. Instantaneous Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR1) Event

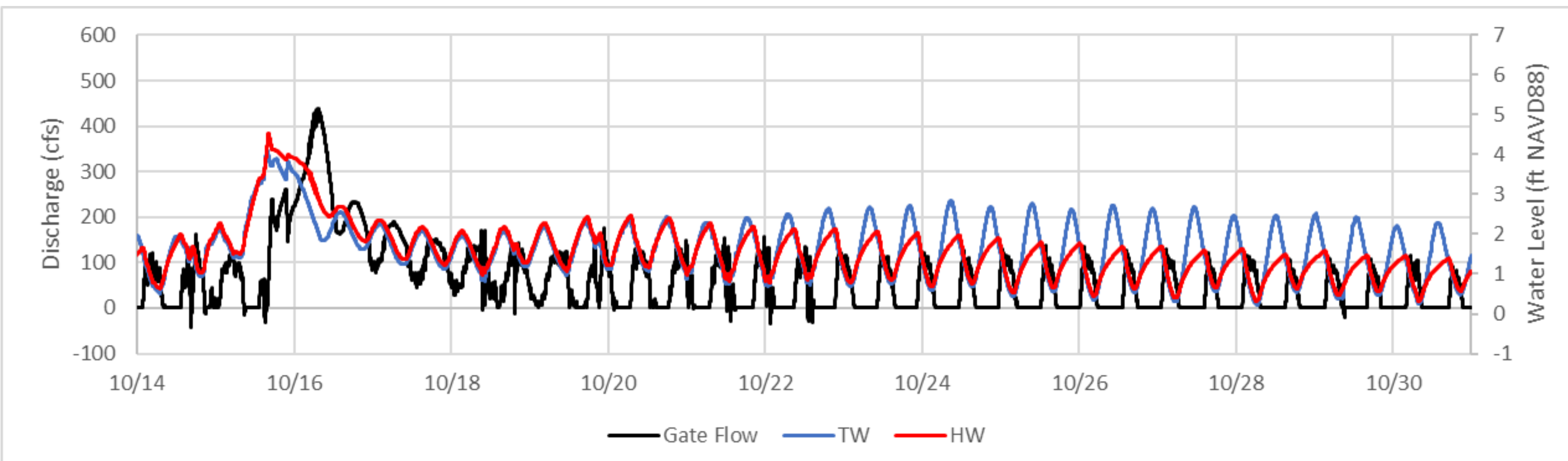


Figure 9-202. 12-Hour Moving Average Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR1) Event

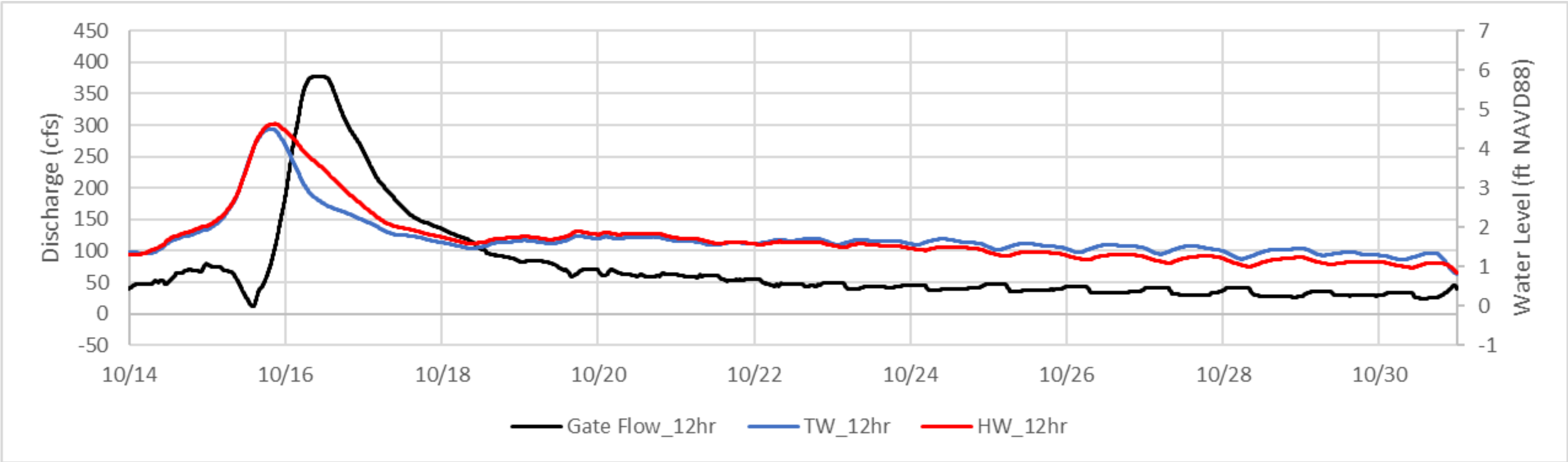


Figure 9-203. 12-Hour Moving Average Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR1) Event

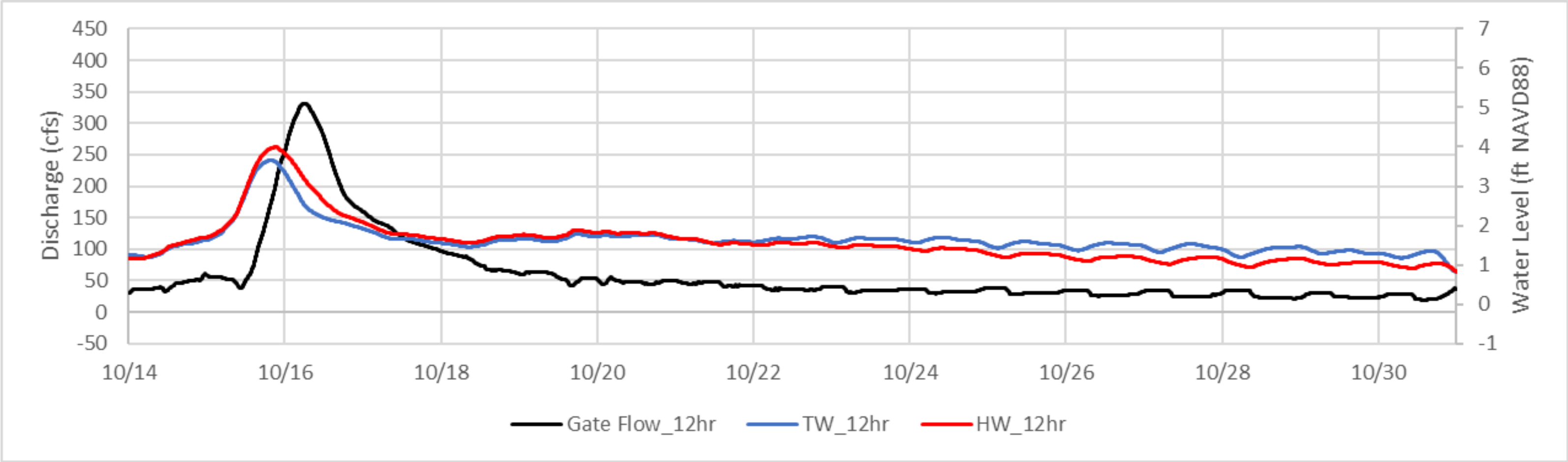


Figure 9-204. Instantaneous Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR2) Event

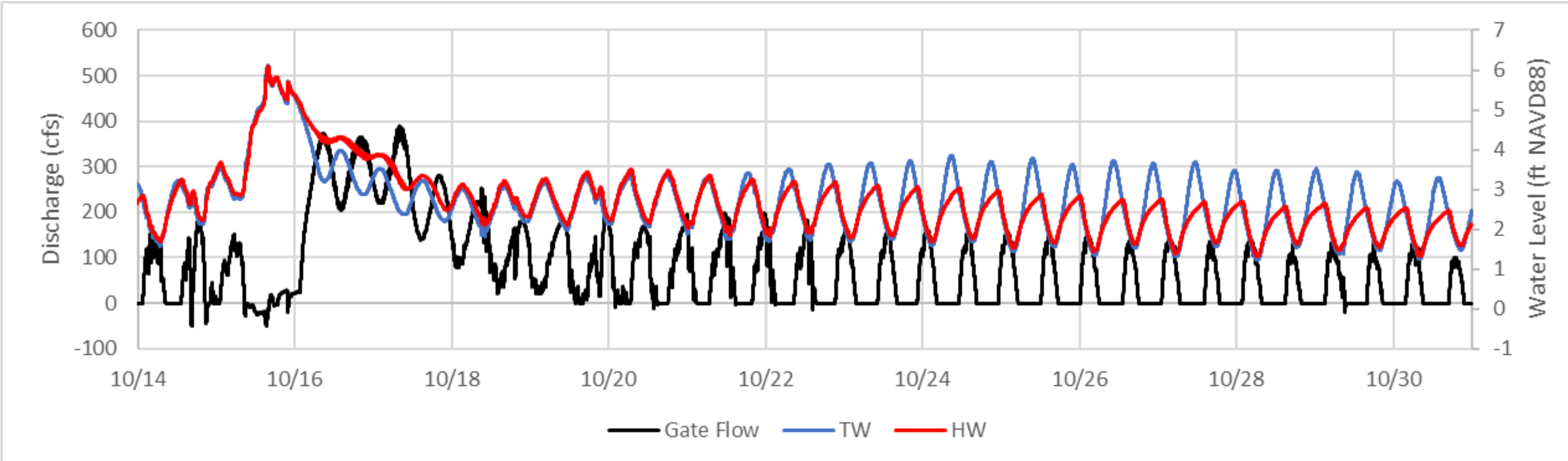


Figure 9-205. Instantaneous Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR2) Event

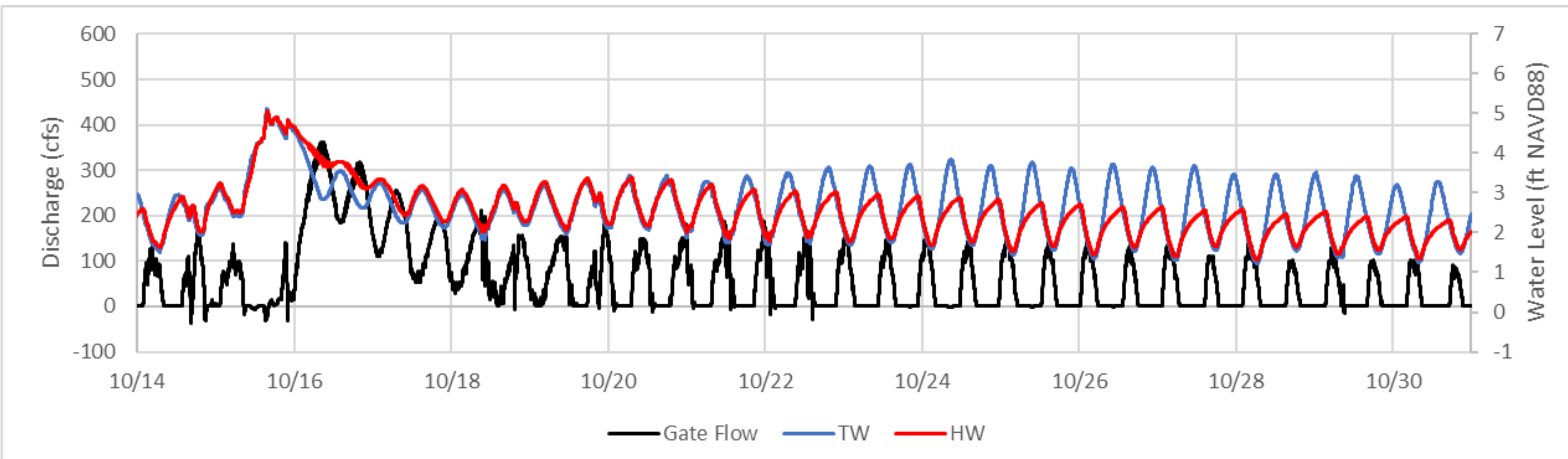


Figure 9-206. 12-Hour Moving Average Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR2) Event

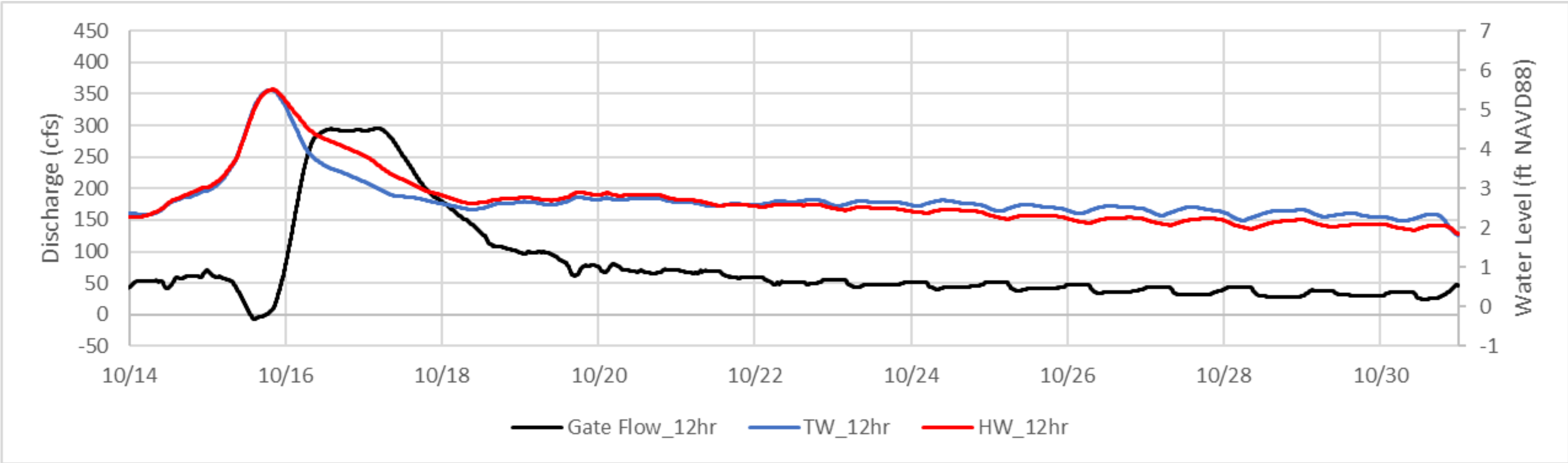


Figure 9-207. 12-Hour Moving Average Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR2) Event

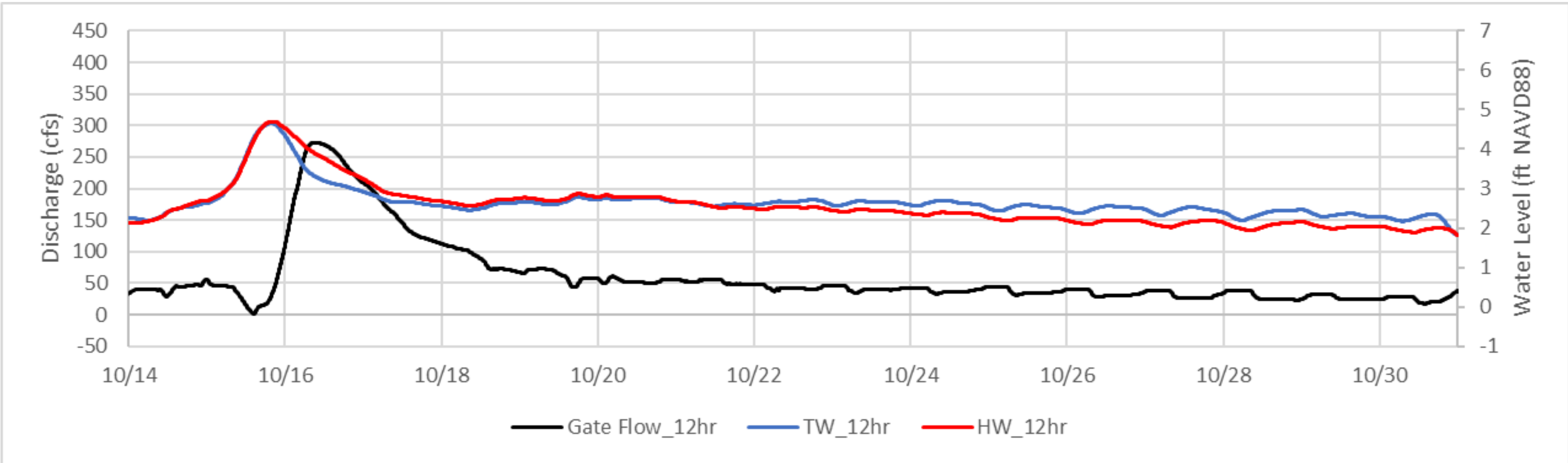


Figure 9-208. Instantaneous Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR3) Event

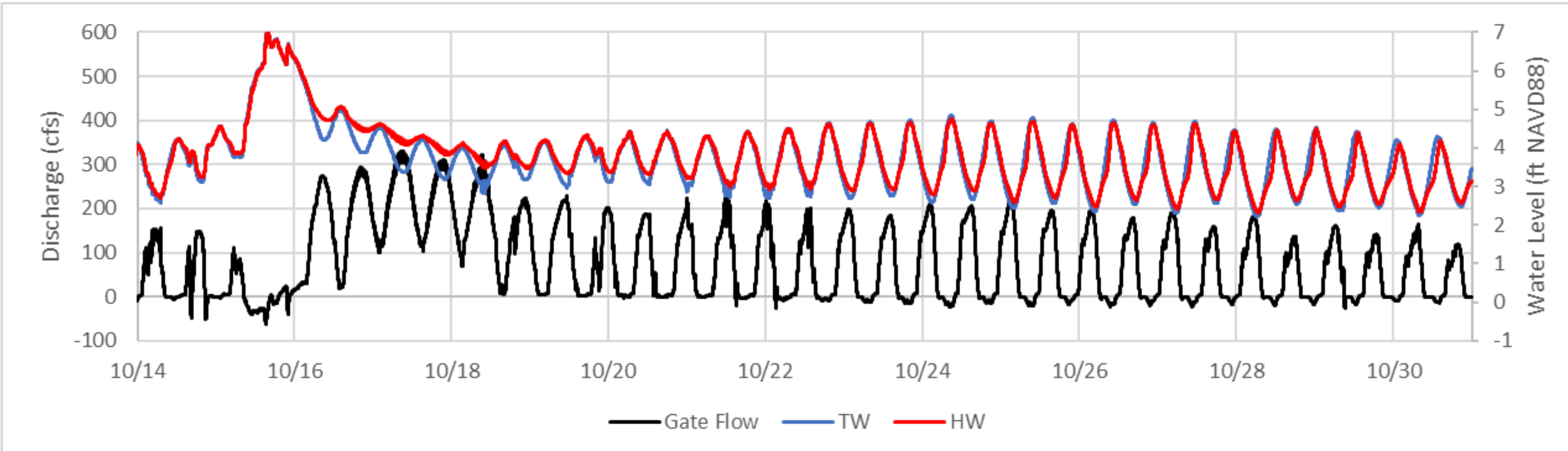


Figure 9-209. Instantaneous Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR3) Event

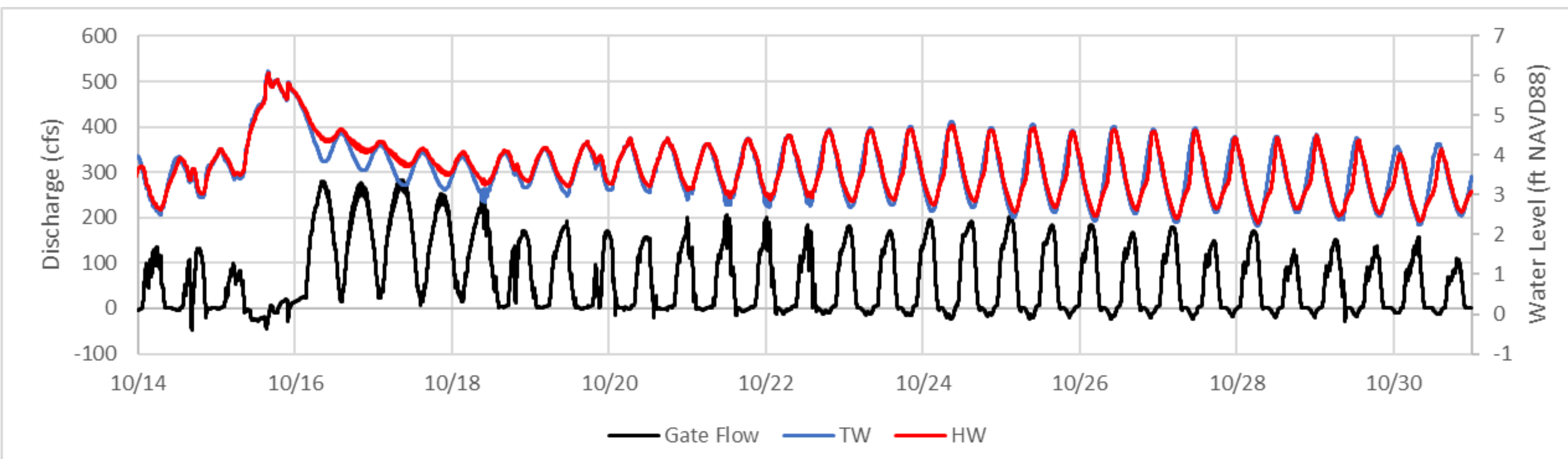


Figure 9-210. 12-Hour Moving Average Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR3) Event

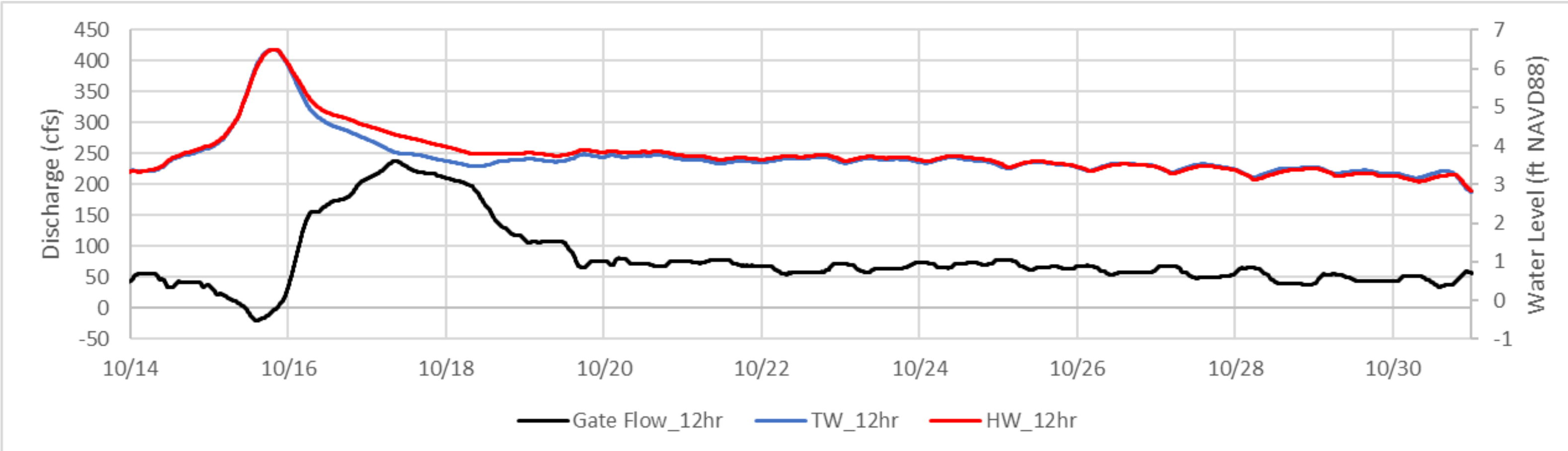
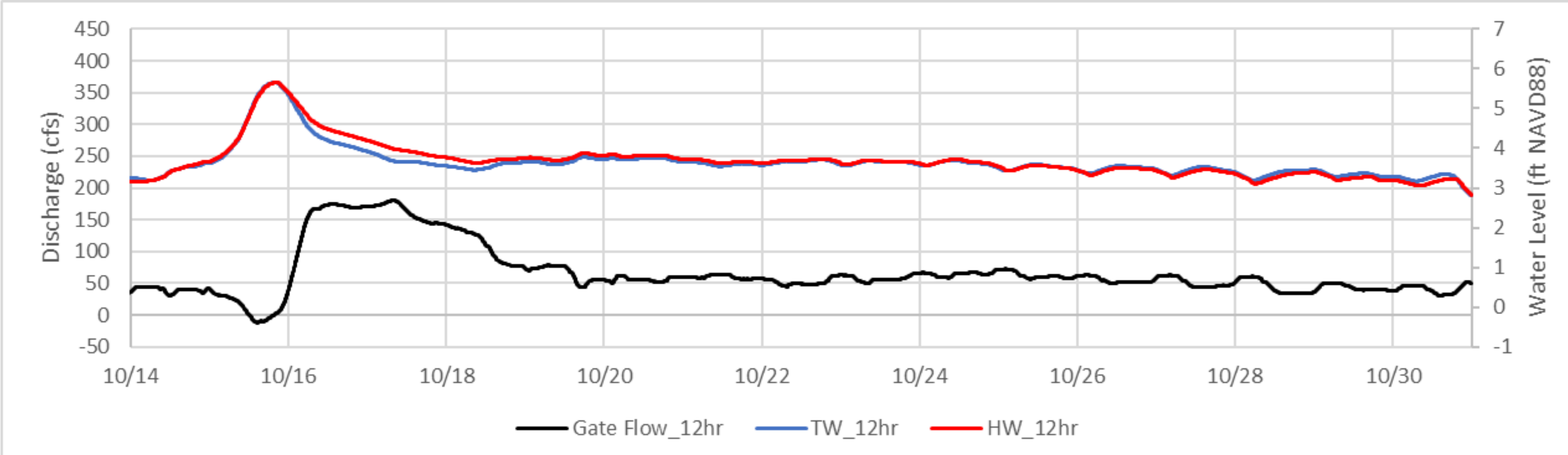


Figure 9-211. 12-Hour Moving Average Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR3) Event



A summary of the instantaneous and 12-hour moving average conditions at the time of peak discharge through S25 are provided in **Table 9-67** and **Table 9-68** respectively for all design storm return periods and future SLR conditions.

Table 9-67. Summary of Conditions at the Time of Peak Discharge through S25 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak Q	Peak Q (CFS)	HW at Peak Q (FT-NAVD)	TW at Peak Q (FT-NAVD)	ΔH at Peak Q (FT)	Gate Opening at Peak Q* (FT)
100-Year	**Current**	10/16 5:30	528.05	3.23	1.99	1.25	8
	SLR1	10/16 8:12	534.80	3.63	2.26	1.37	8
	SLR2	10/17 8:00	389.07	3.30	2.46	0.85	8
	SLR3	10/17 8:48	330.09	4.24	3.39	0.85	8
25-Year	**Current**	10/16 1:37	386.88	3.76	2.73	1.04	8
	SLR1	10/16 7:06	439.56	3.04	2.12	0.92	8
	SLR2	10/16 8:42	361.52	3.87	2.84	1.02	8
	SLR3	10/17 8:48	283.55	3.86	3.26	0.61	8
10-Year	**Current**	10/15 17:07	420.16	3.23	2.31	0.91	8
	SLR1	10/16 6:06	349.40	2.72	2.13	0.60	8
	SLR2	10/16 8:12	351.44	3.56	2.72	0.84	8
	SLR3	10/16 8:36	278.15	4.27	3.69	0.58	8
5-Year	**Current**	10/15 17:17	384.13	2.82	2.11	0.71	8
	SLR1	10/16 5:30	278.11	2.53	2.14	0.38	8
	SLR2	10/16 8:06	326.25	3.31	2.64	0.67	8
	SLR3	10/16 8:24	268.24	4.15	3.61	0.53	8

*A gate opening of 8 ft represents the gate full open.

Table 9-68. Summary of Conditions at the Time of Peak Discharge through S25 (12 hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak Q	Peak Q (CFS)	HW at Peak Q (FT-NAVD)	TW at Peak Q (FT-NAVD)	ΔH at Peak Q (FT)	Gate Opening at Peak Q (FT)
100-Year	Current	10/16 6:45	423.98	2.97	1.95	1.02	8.00
	SLR1	10/16 9:03	377.24	3.67	2.75	0.91	8.00
	SLR2	10/17 4:34	295.40	3.59	2.99	0.59	8.00
	SLR3	10/17 8:31	238.17	4.29	3.84	0.46	7.98
25-Year	Current	10/16 1:37	338.36	2.78	2.13	0.65	8.00
	SLR1	10/16 5:51	331.72	3.20	2.55	0.66	8.00
	SLR2	10/16 9:10	272.66	3.89	3.31	0.59	8.00
	SLR3	10/17 8:13	180.43	3.98	3.67	0.30	6.92
10-Year	Current	10/15 22:37	282.38	2.60	2.15	0.45	7.94
	SLR1	10/16 4:48	276.63	2.90	2.44	0.46	8.00
	SLR2	10/16 7:26	247.35	3.66	3.20	0.46	8.00
	SLR3	10/16 7:35	159.19	4.45	4.19	0.26	6.14
5-Year	Current	10/15 22:35	232.99	2.29	1.97	0.32	7.88
	SLR1	10/16 4:32	230.52	2.65	2.34	0.31	8.00
	SLR2	10/16 6:46	222.71	3.49	3.13	0.36	7.93
	SLR3	10/16 7:10	148.66	4.34	4.10	0.24	5.86

*A gate opening of 8 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak HW at S25 are provided in **Table 9-69** and **Table 9-70**, respectively for all design storm return periods and future SLR conditions.

Table 9-69. Summary of Conditions at the Time of Peak HW through S25 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak HW	Q at Peak HW (CFS)	Peak HW (FT-NAVD)	TW at Peak HW (FT-NAVD)	ΔH at Peak HW (FT)	Gate Opening at Peak HW* (FT)
100-Year	**Current**	10/15 16:01	106.59	4.56	4.08	0.47	3.5
	SLR1	10/15 16:00	1.20	5.10	5.10	0.00	0
	SLR2	10/15 15:49	-25.74	6.08	6.11	-0.03	0
	SLR3	10/15 15:49	-36.63	7.08	7.11	-0.03	0
25-Year	**Current**	10/15 21:05	229.25	4.05	2.92	1.13	5
	SLR1	10/15 16:06	44.58	4.53	4.04	0.49	1.5
	SLR2	10/15 16:00	-8.99	5.08	5.09	-0.02	0
	SLR3	10/15 15:50	-24.79	6.08	6.11	-0.03	0
10-Year	**Current**	10/15 16:28	219.17	3.75	2.45	1.31	4.5
	SLR1	10/15 16:19	113.31	4.28	3.50	0.78	3
	SLR2	10/15 16:00	-2.62	4.65	4.66	-0.01	0
	SLR3	10/15 15:59	-15.66	5.64	5.66	-0.02	0
5-Year	**Current**	10/15 16:28	161.96	3.42	2.22	1.20	3.5
	SLR1	10/15 16:20	94.03	4.06	3.27	0.79	2.5
	SLR2	10/15 16:03	12.25	4.50	4.40	0.10	0.5
	SLR3	10/15 16:00	-17.61	5.39	5.42	-0.03	0

*A gate opening of 8 ft represents the gate full open.

Table 9-70. Summary of Conditions at the Time of Peak HW through S25 (12 hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak HW	Q at Peak HW (CFS)	Peak HW (ft-NAVD)	TW at Peak HW (ft-NAVD)	ΔH at Peak HW (ft)	Gate Opening at Peak HW* (ft)
100-Year	**Current**	10/15 21:10	263.94	4.05	3.45	0.60	7.48
	SLR1	10/15 20:49	105.98	4.63	4.47	0.16	4.78
	SLR2	10/15 20:06	8.81	5.51	5.49	0.02	0.14
	SLR3	10/15 19:47	-2.95	6.49	6.49	0.00	0.00
25-Year	**Current**	10/15 21:05	299.15	3.28	2.63	0.65	7.43
	SLR1	10/15 21:09	201.13	3.99	3.63	0.37	7.10
	SLR2	10/15 20:28	38.86	4.70	4.65	0.05	2.51
	SLR3	10/15 20:11	2.01	5.65	5.65	0.00	0.00
10-Year	**Current**	10/15 20:54	253.82	2.74	2.28	0.46	7.18
	SLR1	10/15 21:08	202.29	3.64	3.27	0.38	7.12
	SLR2	10/15 20:49	56.90	4.36	4.28	0.08	3.50
	SLR3	10/15 20:33	2.21	5.28	5.28	0.00	0.00
5-Year	**Current**	10/15 20:35	204.60	2.42	2.08	0.33	6.79
	SLR1	10/15 21:02	181.74	3.38	3.07	0.31	6.95
	SLR2	10/15 21:02	70.65	4.17	4.07	0.09	4.48
	SLR3	10/15 20:46	0.88	5.08	5.08	0.00	0.00

*A gate opening of 8 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak TW at S25 are provided in **Table 9-71** and **Table 9-72**, respectively for all design storm return periods and future SLR conditions.

Table 9-71 shows that there are instances when there are negative flows, yet the gate opening is zero, this indicates that the tailwater is higher than the overtopping elevation (3.755 ft-NAVD) and there is a negative head differential across this structure. However, negative flows can also occur when the gate is in the process of closing and the head differential is negative.

Table 9-71. Summary of Conditions at the Time of Peak Tailwater at S25 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (ft-NAVD)	Peak TW (ft-NAVD)	ΔH at Peak TW (ft)	Gate Opening at Peak TW* (ft)
100-Year	Current	10/15 15:45	28.36	4.21	4.12	0.09	2
	SLR1	10/15 15:45	-22.55	5.05	5.12	-0.08	0
	SLR2	10/15 15:45	-32.42	6.07	6.12	-0.05	0
	SLR3	10/15 15:45	-43.20	7.08	7.12	-0.04	0
25-Year	Current	10/15 19:52	52.49	3.33	3.11	0.22	2.5
	SLR1	10/15 15:45	5.32	4.15	4.11	0.04	0
	SLR2	10/15 15:45	-25.72	5.01	5.11	-0.10	0
	SLR3	10/15 15:45	-31.02	6.07	6.11	-0.05	0
10-Year	Current	10/15 15:45	18.78	2.86	2.67	0.19	1
	SLR1	10/15 15:45	0.48	3.74	3.67	0.07	0
	SLR2	10/15 15:45	-9.53	4.64	4.67	-0.03	0
	SLR3	10/15 15:45	-27.68	5.61	5.67	-0.06	0
5-Year	Current	10/15 15:45	0.00	2.50	2.43	0.07	0
	SLR1	10/15 15:45	0.00	3.41	3.43	-0.02	0
	SLR2	10/15 15:45	-0.78	4.43	4.43	0.00	0
	SLR3	10/15 15:45	-28.48	5.35	5.43	-0.08	0

*A gate opening of 8 ft represents the gate full open.

Table 9-72. Summary of Conditions at the Time of Peak Tailwater at S25 (12 hour Moving Average)

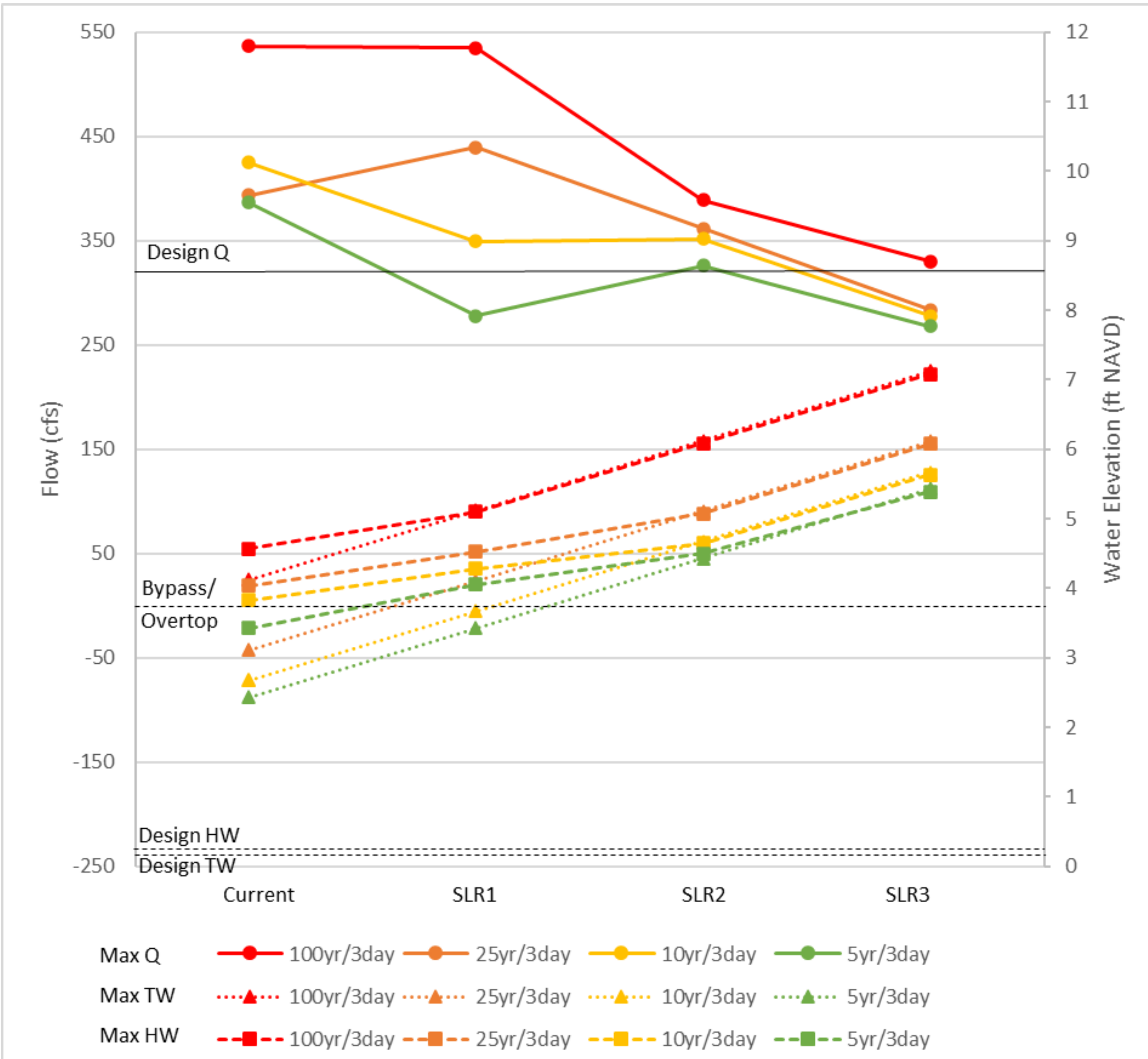
Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (FT-NAVD)	Peak TW (FT-NAVD)	ΔH at Peak TW (FT)	Gate Opening at Peak TW* (FT)
100-Year	**Current**	10/15 19:31	224.58	3.98	3.50	0.49	7.11
	SLR1	10/15 19:31	80.13	4.61	4.50	0.12	3.91
	SLR2	10/15 19:31	5.62	5.51	5.50	0.01	0.01
	SLR3	10/15 19:31	-4.14	6.49	6.50	-0.01	0.00
25-Year	**Current**	10/15 19:52	269.34	3.24	2.65	0.59	7.06
	SLR1	10/15 19:52	175.08	3.97	3.65	0.31	6.57
	SLR2	10/15 19:52	30.87	4.70	4.65	0.04	2.12
	SLR3	10/15 19:52	0.85	5.65	5.65	0.00	0.00
10-Year	**Current**	10/15 20:01	238.94	2.73	2.28	0.44	6.88
	SLR1	10/15 20:01	181.05	3.62	3.28	0.34	6.64
	SLR2	10/15 20:01	45.48	4.35	4.28	0.07	2.97
	SLR3	10/15 20:01	0.47	5.28	5.28	0.00	0.00
5-Year	**Current**	10/15 20:07	196.50	2.41	2.08	0.33	6.49
	SLR1	10/15 20:07	164.09	3.37	3.08	0.28	6.38
	SLR2	10/15 20:07	57.61	4.15	4.08	0.07	3.87
	SLR3	10/15 20:07	-0.92	5.08	5.08	-0.01	0.00

*A gate opening of 8 ft represents the gate full open.

Figure 9-212 shows a summary of the instantaneous peak discharge, HW, and TW at S25 for all design storm return periods and future SLR conditions. The design parameter values listed in **Table 9-66** are shown graphically in the figure below with bypass indicating the water level which will bypass the structure and overtop indicating the water level which will overtop the gates when the gates are closed. Since S25 is a gated culvert built into an embankment, the gate overtopping and bypass elevation are equivalent for this structure. Note that the peak discharge, HW, and TW occur at different times for each scenario.

The maximum HW and TW exceed the design HW and TW for all conditions simulations. The maximum HW at S25 exceeds the water level which will bypass/overtop the structure for all future SLR simulations as well as the 25-year and 100-year current condition simulations (no SLR). The maximum TW at S25 exceeds the water level which will bypass/overtop the structure for the 100-year current conditions (no SLR) simulation and all future SLR scenarios, except for the 5-year and 10-year SLR1 scenarios. TW elevations that exceed this elevation can result in flow entering the basin from storm surge and/or tide and flooding of neighborhoods around S25.

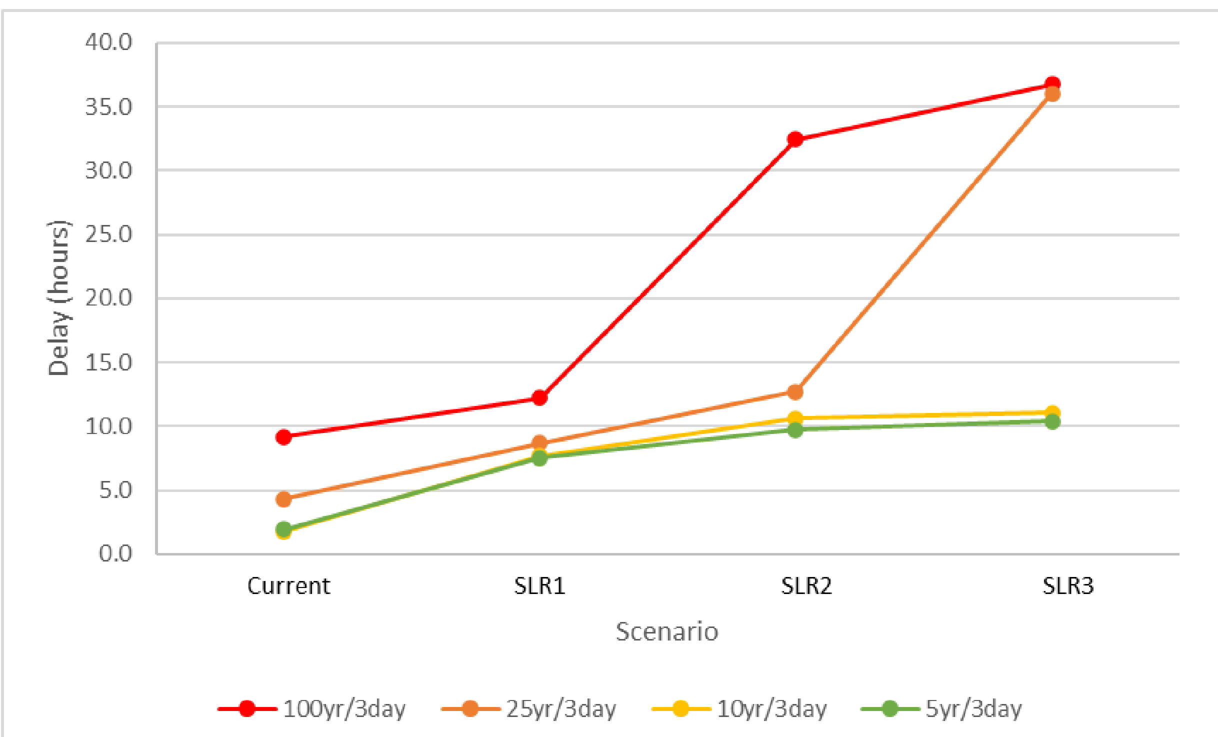
Figure 9-212. Summary of Instantaneous Max Discharge, HW, and TW at S25



The peak discharge at S25 exceeds the design value for all design storms during current conditions. With 1 foot of SLR, the 5-year event is unable to reach the design discharge even with HW conditions exceeding the design value. With 3 feet of SLR, only the 100-year event reaches the design discharge. The increase in TW levels due to SLR decreases the head differential at the structure and inhibits the flow out of the basin, potentially creating additional flooding upstream of the structure as shown in PM #1.

Without TW limitations, increased HW would generate more flow through a gravity structure (i.e., you would expect to see maximum flow through the structure occurring near the time of maximum HW). The time between the 12-hour moving average peak HW (**Table 9-70**) and the 12-hour moving average peak discharge (**Table 9-68**) is shown in **Figure 9-213** for each design storm. All current condition simulations, future SLR1 simulations, future SLR2 5-year, 10-year, and 25-year design storm simulations, and future SLR3 5-year and 10-year design storm simulations reach the peak discharge within 15 hours of the peak HW at S25. However, the 100-year SLR2 simulation and 100-year and 25-year SLR3 scenarios see a delay of more than 30 hours between peak HW and peak discharge. This delay in the structure's ability to discharge water is a result of high TW conditions from the storm surge and can increase duration of inland flooding as shown in PM #6.

Figure 9-213. Time Between Peak HW and Peak Discharge at S25 for the 12-Hour Moving Average



9.4.4. C5 – PM #4 PEAK STORM RUNOFF

The purpose of PM #4 is to determine the effect of SLR on the maximum peak storm runoff, or maximum conveyance capacity of the watershed. For this metric, 12-hour moving average flow hydrographs downstream of S25 and the maximum 12-hour moving average total flow was determined for each design storm event and SLR scenario.

The 12-hour moving average discharge hydrographs for each SLR scenario can be found in **Figure 9-214** for the 100-year storm, **Figure 9-215** for the 25-year storm, **Figure 9-216** for the 10-year storm, and **Figure 9-217** for the 5-year storm. Downstream flows for S25 comprise of the discharge from the single barreled culvert in addition to overtopping of the structure, if applicable.

Figure 9-214. 12-Hour Moving Average of Flows at S25 for the 100-year Design Storm

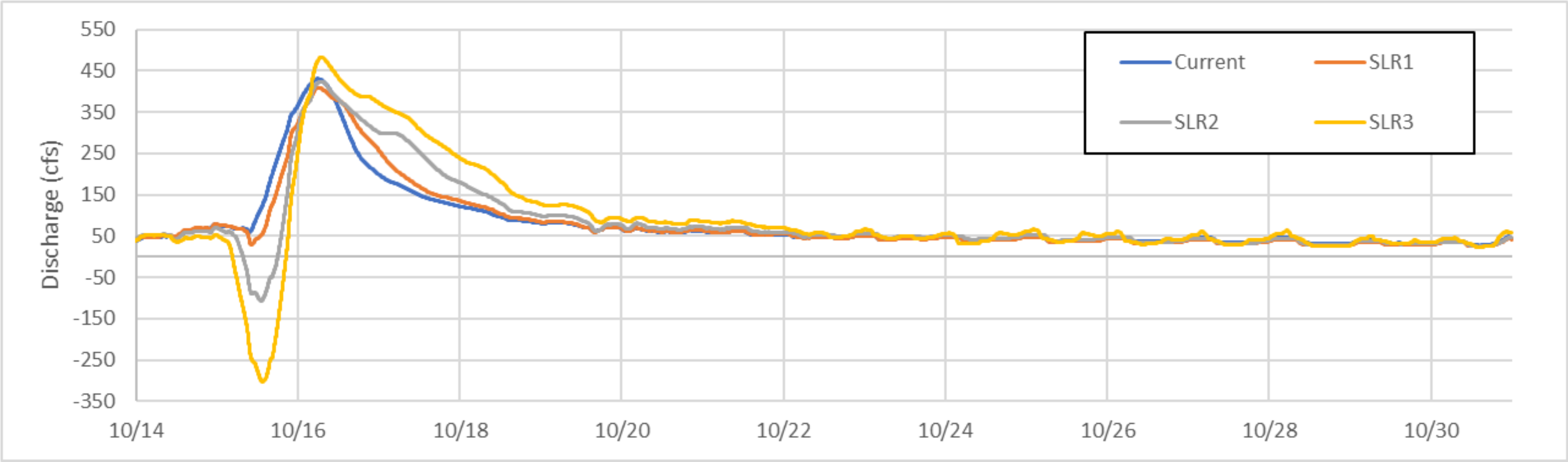


Figure 9-215. 12-Hour Moving Average of Flows at S25 for the 25-year Design Storm

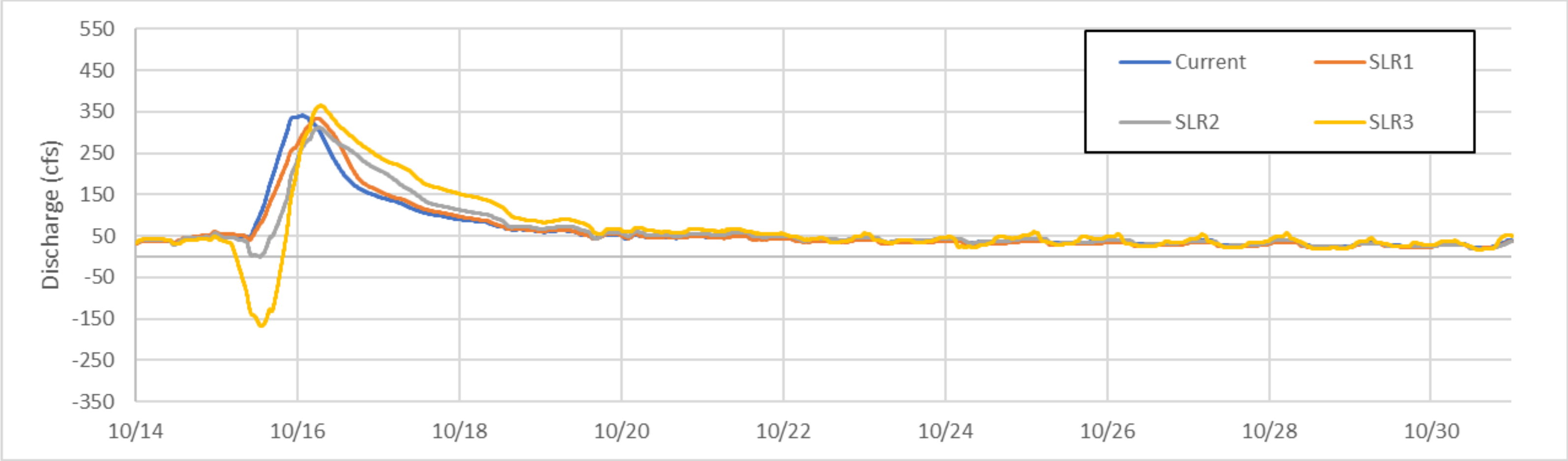


Figure 9-216. 12-Hour Moving Average of Flows at S25 for the 10-year Design Storm

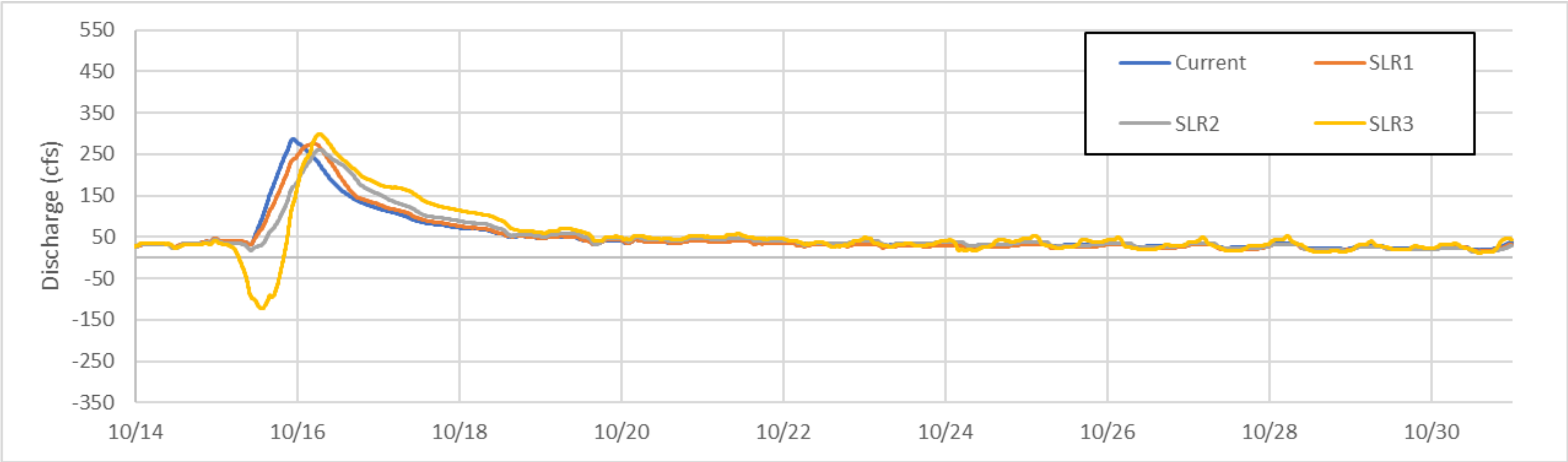
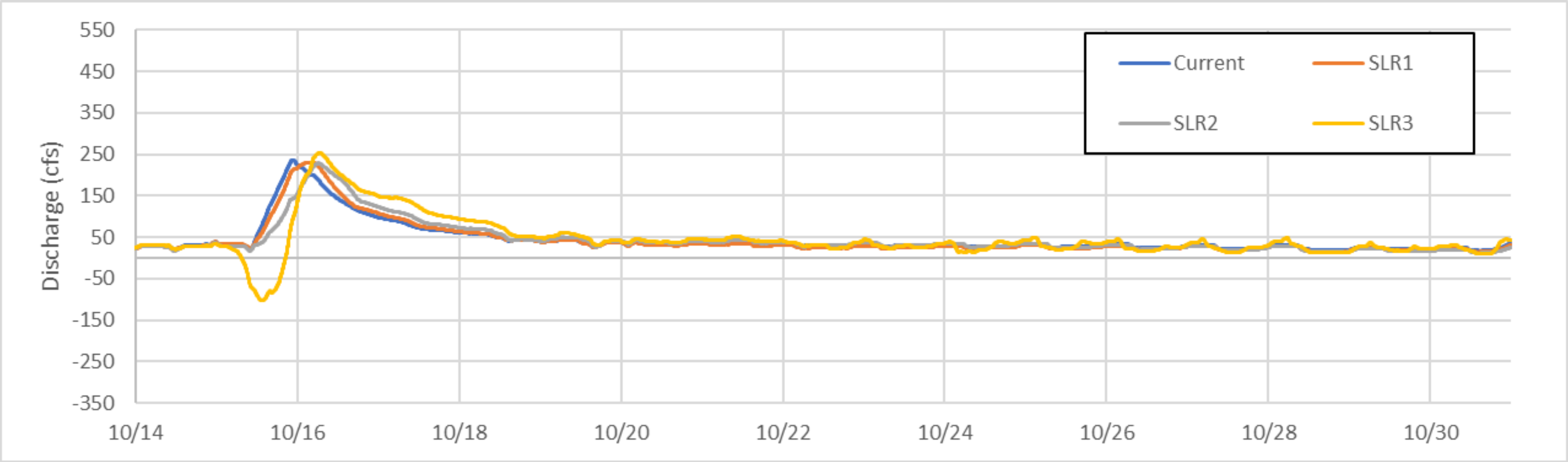


Figure 9-217. 12-Hour Moving Average of Flows at S25 for the 5-year Design Storm

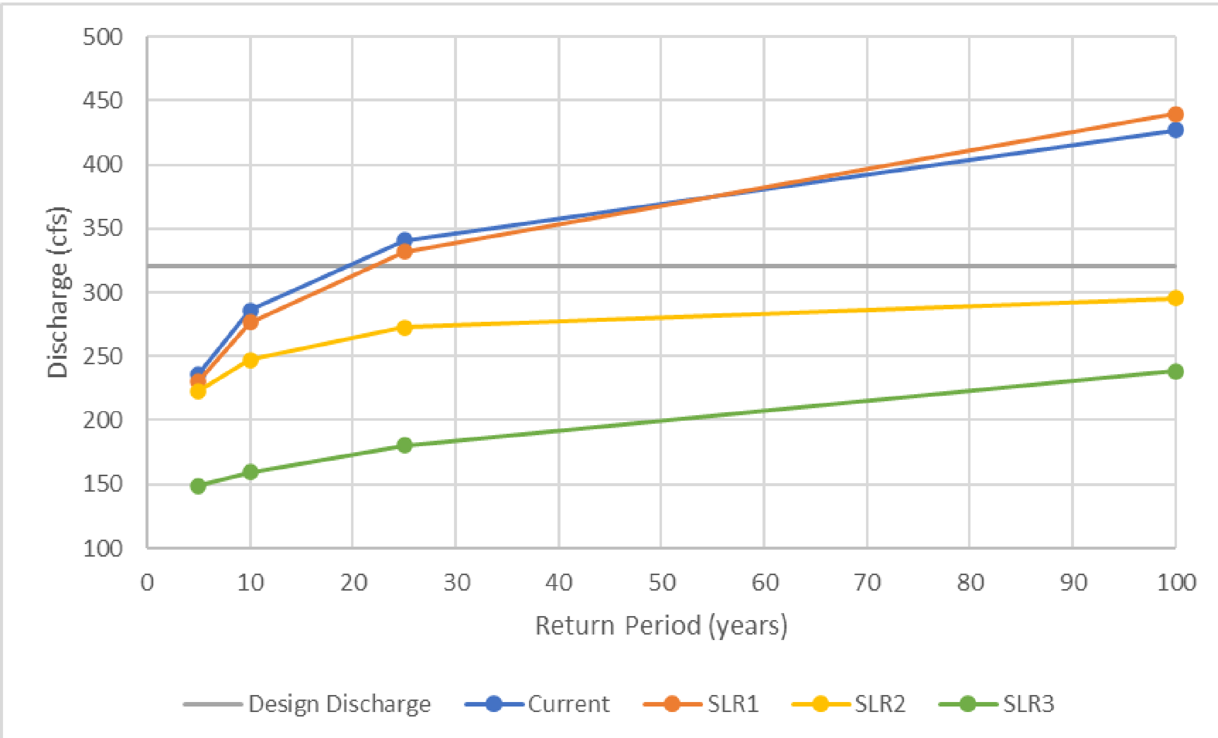


The instantaneous and 12-hour moving average peak discharges for all of the design storm event return periods and SLR scenarios are shown in **Table 9-73**. In addition, the percentage difference between the current conditions 12-hour moving average peak discharge and each future conditions SLR scenario is calculated for all simulations. The 12-hour moving average peak discharges for each design storm event return period and SLR scenario are also shown in **Figure 9-218**. The peak 12-hour moving average discharge decreases with increasing SLR scenario and only reaches the design value for the 100-year and 25-year design storms during current conditions and the SLR1 scenario.

Table 9-73. Peak Discharge Summary at S25

Design Storm Return Period	Scenario	Peak Discharge (CFS)		12-Hour Moving Average Peak Flow Reduction Percentage
		Instantaneous	12-Hour Moving Average	
100-Year	**Current**	536.56	426.73	N/A
	SLR1	534.80	377.24	11.60%
	SLR2	389.07	295.40	30.77%
	SLR3	330.09	238.17	44.19%
25-Year	**Current**	393.73	340.49	N/A
	SLR1	439.56	331.72	2.58%
	SLR2	361.52	272.66	19.92%
	SLR3	283.55	180.43	47.01%
10-Year	**Current**	424.71	286.09	N/A
	SLR1	349.40	276.63	3.31%
	SLR2	351.44	247.35	13.54%
	SLR3	278.15	159.19	44.36%
5-Year	**Current**	386.48	236.28	N/A
	SLR1	278.11	230.52	2.44%
	SLR2	326.25	222.71	5.74%
	SLR3	268.24	148.66	37.08%

Figure 9-218. Peak 12-Hour Moving Discharge at S25



9.4.5. C5 – PM #5 FREQUENCY OF FLOODING

The maximum overland depth was extracted for each design storm event and evaluated for the C5 Watershed. **Table 9-74** tabulates the flood inundation area in square miles for the C5 Watershed. The total area of the C5 Watershed for this analysis was calculated as 1.9 square miles.

The total area of the C5 Watershed considered urban is 1.8 square miles. **Table 9-75** tabulates the flood inundation area in square miles for the urban areas within the C5 Watershed. This table shows that the greatest distribution of flooding depths in the watershed are between zero and 0.25 feet of flooding, which can be considered nuisance flooding. **Figure 9-220** shows the urban inundation with the same incremental flooding depths as the table for the 100-year storm event, and **Figure 9-221** does the same for the 25-year storm. This provides a clear view of how stages are increasing with sea level rise in this watershed. Flooding depths in this watershed actually decrease for the 100-year storm all the way to 1.0 foot for all three SLR conditions and increase after this depth. For the 25-year storm, this switch to increasing begins after 0.75 feet of flooding depth. This depth increase at greater depths than other watersheds is likely an indication of the topography of the watershed, which has a clear depressional area (a remnant finger glade) along the canal and a sharp rise in elevation to the south, as outlined in **Figure 9-219**.

Figure 9-219. Topography within C5

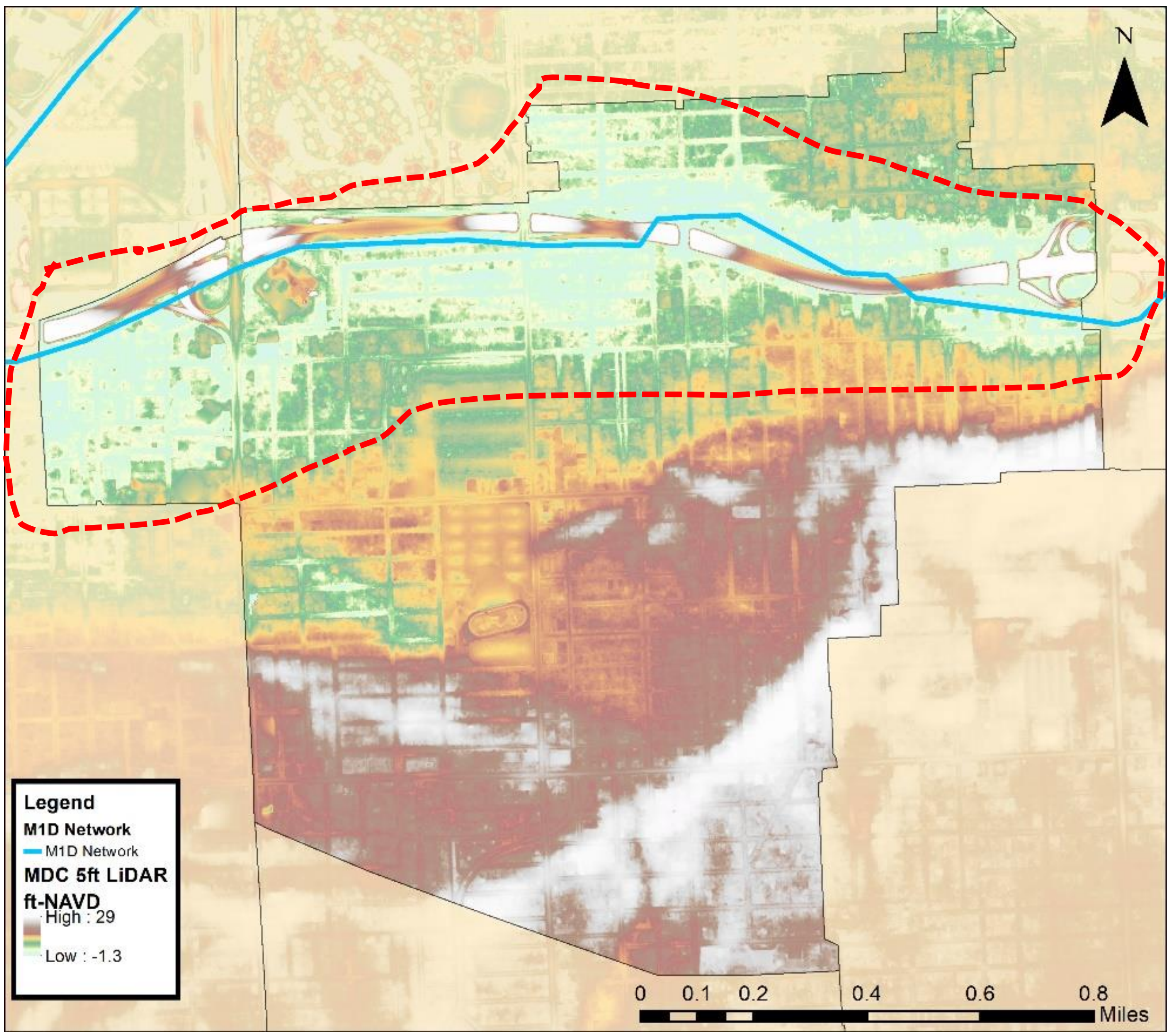


Table 9-76 shows the percentage of the total urban areas within the C5 Watershed that is below the flooding depth. This data shows that 51% of the total urban areas in the C5 Watershed are inundated with 3 inches or greater (0.25 feet) for the current conditions 100-year storm, this percentage increases steadily by about 4% with each foot of SLR. However, the increase in this percentage is greater with higher flood depths, for example at the 1.25 feet mark the SLR 3 condition shows an increase of about 20% above the current conditions. While this watershed is likely experiencing tertiary drainage issues caused by low-lying basins, the primary drainage issues caused by high stages in Comfort Canal Southfork are very pronounced with increasing SLR.

Additionally, the model topography in the urban areas of the C5 watershed was reviewed and it was determined that 21% (or 238 acres) of the land surface is below the peak canal stage for the 100-year, 18% for the 25-year, 13% for the 10-year, and 9% for the 5-year.

Figure 9-222 and **Figure 9-223** are maps of the maximum overland depth over the entire C5 Watershed for the 25-year and 100-year 3-day design storm events, respectively. Water areas, such as existing lakes and ponds, are masked in black.

Figure 9-224, **Figure 9-225**, and **Figure 9-226** show the difference in overland flooding for the C5 Watershed between the current conditions and future sea level rise conditions for the SLR +1 foot, SLR +2 feet, and SLR +3 feet simulations, respectively. Because the rainfall is the same in all the 100-year storm event simulations, these difference maps remove any overland flooding caused by rainfall and show how much is now impacted by rising seas in terms of direct flooding from the canals, or reduced drainage capacity due to higher stages in the primary canal. Higher canal stages are due to reduced discharge capacity at the structure as well as structure backflow due to overtopping and structure bypass, as discussed in the previous sections. Areas directly adjacent to the canal show substantial increase in overland flooding depths, with all three SLR conditions. In addition to the flooding depths, with each SLR condition the extent of flooding increases. Three areas are shown to increase in flooding depth along the canal, 1) the biggest impacted area is around NW 37th Avenue, 2) the area west of NW 42nd Avenue, just downstream of the S25A structure, and 3) the area west of the S25 structure around NW 29th Avenue. However, with the 100-year storm under SLR3 conditions, these distinct areas seem to blend together to form one large floodplain for the canal.

Maps of the maximum overland depth over the entire C5 Watershed for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current conditions and future SLR scenarios are provided in **Appendix C**. Also provided in **Appendix C** are the differences in overland flooding maps for the 25-year, 10-year, and 5-year design storms.

Table 9-74. Incremental Flood Inundation Area (sq. mi.) in the C5 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	0.90	1.29	1.51	1.68	0.83	1.22	1.43	1.57	0.76	1.12	1.34	1.45	0.68	0.98	1.23	1.32
0.25 =< Depth < 0.50	0.33	0.23	0.19	0.08	0.34	0.23	0.17	0.14	0.32	0.24	0.15	0.17	0.27	0.23	0.13	0.14
0.50 =< Depth < 0.75	0.19	0.16	0.07	0.04	0.18	0.15	0.11	0.07	0.16	0.14	0.15	0.09	0.16	0.14	0.13	0.11
0.75 =< Depth < 1.00	0.16	0.08	0.05	0.04	0.16	0.11	0.06	0.04	0.13	0.13	0.08	0.07	0.13	0.12	0.10	0.08
1.00 =< Depth < 1.25	0.13	0.05	0.03	0.02	0.10	0.07	0.03	0.02	0.13	0.08	0.06	0.03	0.11	0.10	0.08	0.07
1.25 =< Depth < 1.50	0.06	0.02	0.01	0.01	0.09	0.03	0.02	0.02	0.09	0.06	0.03	0.02	0.11	0.07	0.05	0.06
1.50 =< Depth < 1.75	0.04	0.02	0.00	0.01	0.07	0.02	0.02	0.01	0.09	0.04	0.02	0.01	0.08	0.06	0.05	0.04
1.75 =< Depth < 2.00	0.01	0.01	0.01	0.00	0.03	0.02	0.01	0.01	0.04	0.03	0.01	0.02	0.07	0.04	0.04	0.03
2.00 =< Depth < 2.25	0.01	0.01	0.00	0.00	0.02	0.01	0.01	0.00	0.06	0.01	0.01	0.00	0.06	0.05	0.03	0.02
2.25 =< Depth < 2.50	0.01	0.00	0.00	0.00	0.01	0.01	0.00	0.00	0.03	0.02	0.01	0.00	0.05	0.02	0.01	0.00
2.50 =< Depth < 2.75	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.04	0.03	0.00	0.00
2.75 =< Depth < 3.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.04	0.00	0.00	0.00
3.00 =< Depth	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.07	0.01	0.01	0.00

Total Basin Area = 1.9 square miles

Table 9-75. Incremental Flood Inundation Area (sq. mi.) for Urban Areas in the C5 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	0.86	1.22	1.42	1.58	0.79	1.14	1.34	1.47	0.73	1.04	1.25	1.36	0.65	0.91	1.14	1.23
0.25 =< Depth < 0.50	0.30	0.21	0.17	0.08	0.31	0.22	0.17	0.14	0.28	0.22	0.14	0.16	0.23	0.21	0.11	0.13
0.50 =< Depth < 0.75	0.18	0.15	0.07	0.04	0.17	0.14	0.11	0.07	0.15	0.13	0.14	0.08	0.15	0.13	0.13	0.11
0.75 =< Depth < 1.00	0.15	0.07	0.04	0.04	0.15	0.10	0.06	0.04	0.12	0.12	0.08	0.07	0.12	0.11	0.10	0.08
1.00 =< Depth < 1.25	0.13	0.04	0.03	0.01	0.10	0.07	0.03	0.02	0.13	0.08	0.06	0.03	0.10	0.10	0.08	0.07
1.25 =< Depth < 1.50	0.06	0.02	0.01	0.01	0.09	0.03	0.02	0.02	0.09	0.06	0.03	0.02	0.11	0.07	0.05	0.06
1.50 =< Depth < 1.75	0.03	0.02	0.00	0.01	0.07	0.02	0.02	0.01	0.09	0.04	0.02	0.01	0.08	0.06	0.05	0.04
1.75 =< Depth < 2.00	0.01	0.00	0.01	0.00	0.03	0.02	0.01	0.01	0.04	0.03	0.01	0.02	0.07	0.04	0.04	0.03
2.00 =< Depth < 2.25	0.01	0.01	0.00	0.00	0.02	0.01	0.01	0.00	0.06	0.01	0.01	0.00	0.06	0.05	0.03	0.02
2.25 =< Depth < 2.50	0.01	0.00	0.00	0.00	0.01	0.01	0.00	0.00	0.03	0.02	0.01	0.00	0.05	0.02	0.01	0.00
2.50 =< Depth < 2.75	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.04	0.03	0.00	0.00
2.75 =< Depth < 3.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.04	0.00	0.00	0.00
3.00 =< Depth	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.06	0.01	0.00	0.00

Total Basin Urban Area = 1.8 square miles

Table 9-76. Percent of Total Watershed Area with Inundated Area for Urban Areas in the C5 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0.00	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 0.25	51.3	30.6	19.1	10.4	55.3	35.4	23.9	16.7	58.7	40.8	28.9	22.8	62.9	48.0	35.0	30.3
>= 0.50	34.1	18.9	9.4	6.1	37.8	22.8	14.5	8.7	42.9	28.2	20.8	13.6	50.1	36.1	28.5	22.9
>= 0.75	24.1	10.1	5.7	3.9	28.2	14.8	8.3	5.0	34.1	20.8	12.6	8.9	41.8	28.5	21.3	16.8
>= 1.00	15.4	5.9	3.2	1.7	19.4	9.0	4.8	2.9	27.5	13.8	8.0	4.7	35.0	22.3	15.8	12.5
>= 1.25	7.9	3.4	1.4	0.9	13.8	5.1	3.2	2.0	20.4	9.2	4.6	2.8	29.4	16.4	11.5	8.8
>= 1.50	4.7	2.2	0.8	0.4	8.7	3.3	1.9	0.9	15.0	5.6	2.9	1.9	23.4	12.7	8.4	5.6
>= 1.75	2.9	1.0	0.6	0.0	4.8	2.2	1.0	0.4	10.1	3.4	1.8	1.1	19.0	9.2	5.4	3.4
>= 2.00	2.2	0.8	0.1	0.0	3.2	1.1	0.4	0.0	7.8	1.8	1.0	0.3	14.8	6.6	3.3	1.7
>= 2.25	1.4	0.3	0.0	0.0	2.0	0.5	0.0	0.0	4.5	1.4	0.4	0.0	11.2	3.8	1.4	0.8
>= 2.50	0.8	0.0	0.0	0.0	1.4	0.1	0.0	0.0	2.9	0.4	0.0	0.0	8.4	2.4	0.8	0.5
>= 2.75	0.3	0.0	0.0	0.0	0.9	0.0	0.0	0.0	1.5	0.3	0.0	0.0	6.1	0.9	0.5	0.3
>= 3.00	0.0	0.0	0.0	0.0	0.1	0.0	0.0	0.0	0.8	0.0	0.0	0.0	3.6	0.6	0.3	0.1

Figure 9-220. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C2 Watershed for the 100-year Design Storm

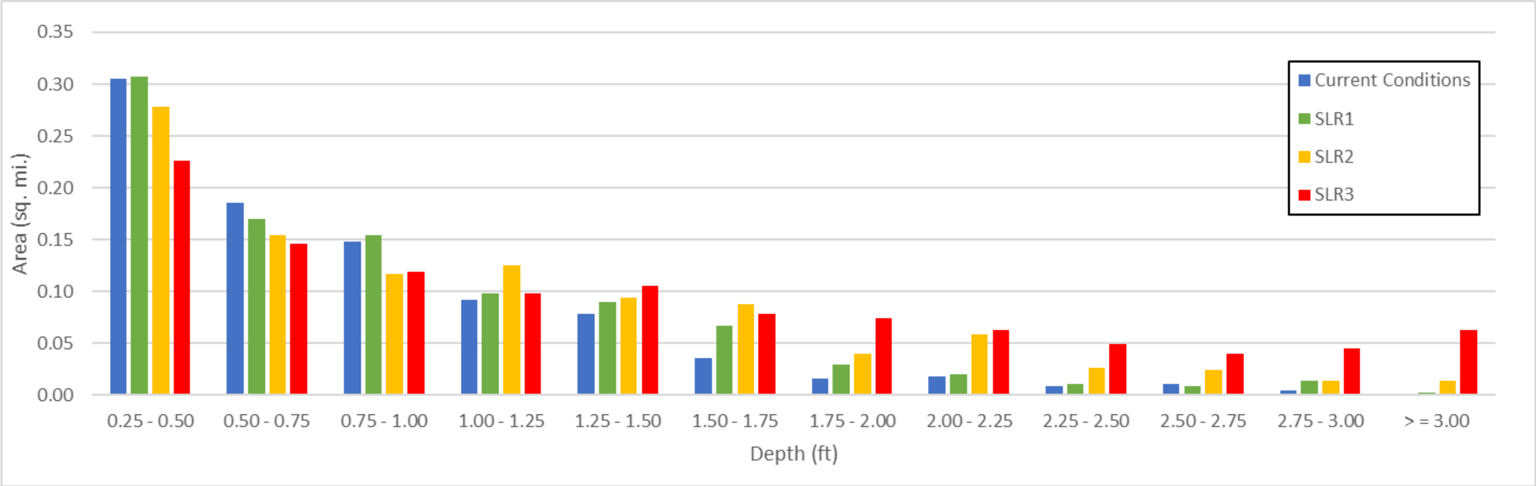


Figure 9-221. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C2 Watershed for the 25-year Design Storm

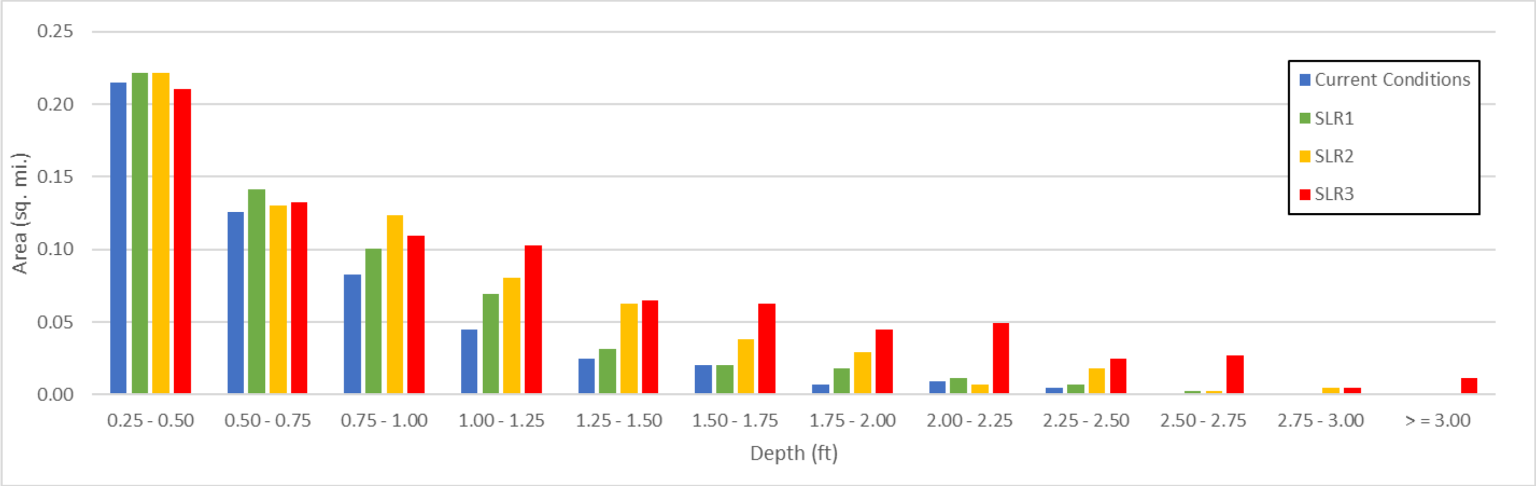


Figure 9-222. Maximum Overland Flood Depth for the Current Conditions 25-year 3-Day Design Storm in the C5 Watershed

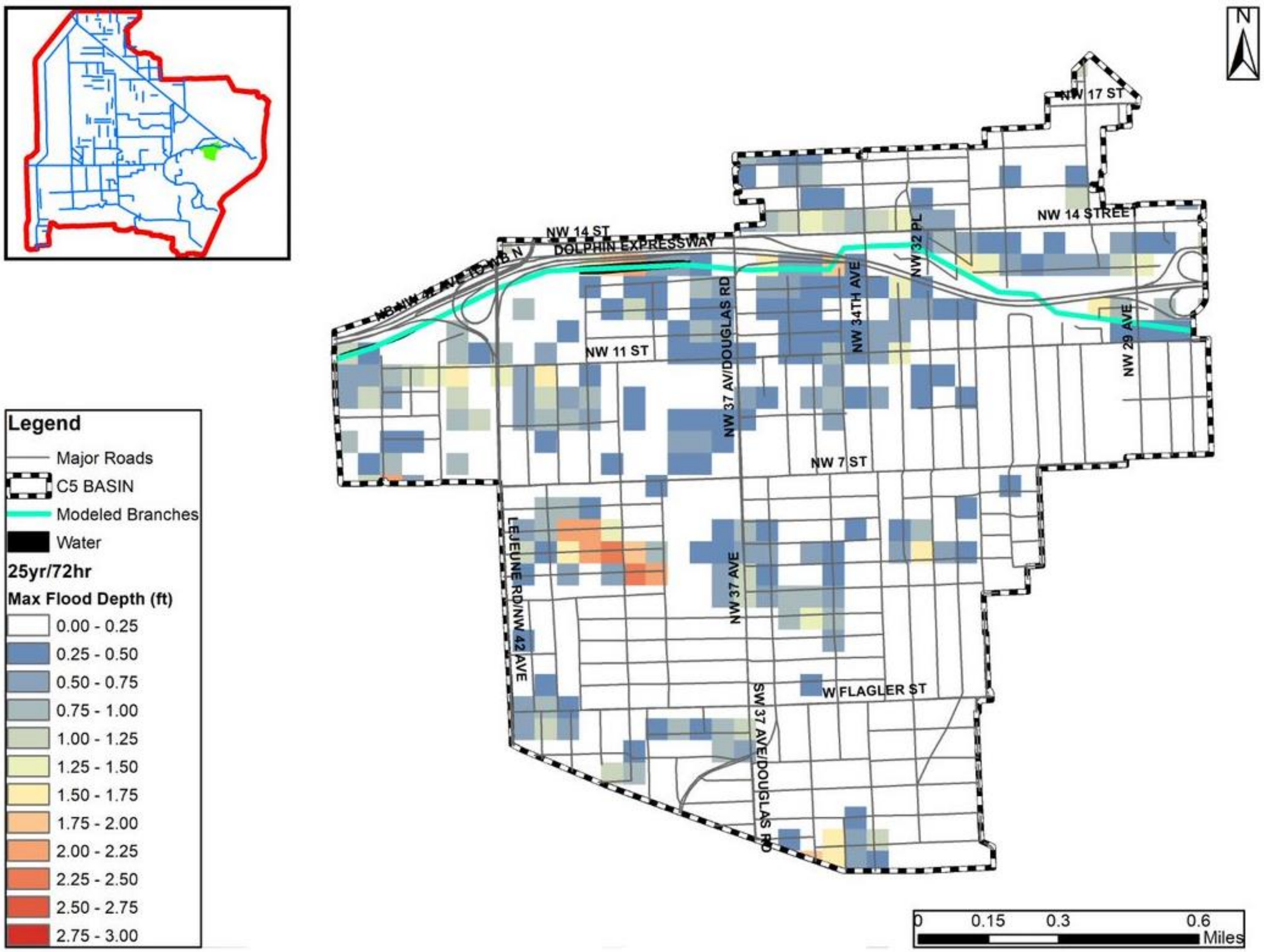


Figure 9-223. Maximum Overland Flood Depth for the Current Conditions 100-year 3-Day Design Storm in the C5 Watershed

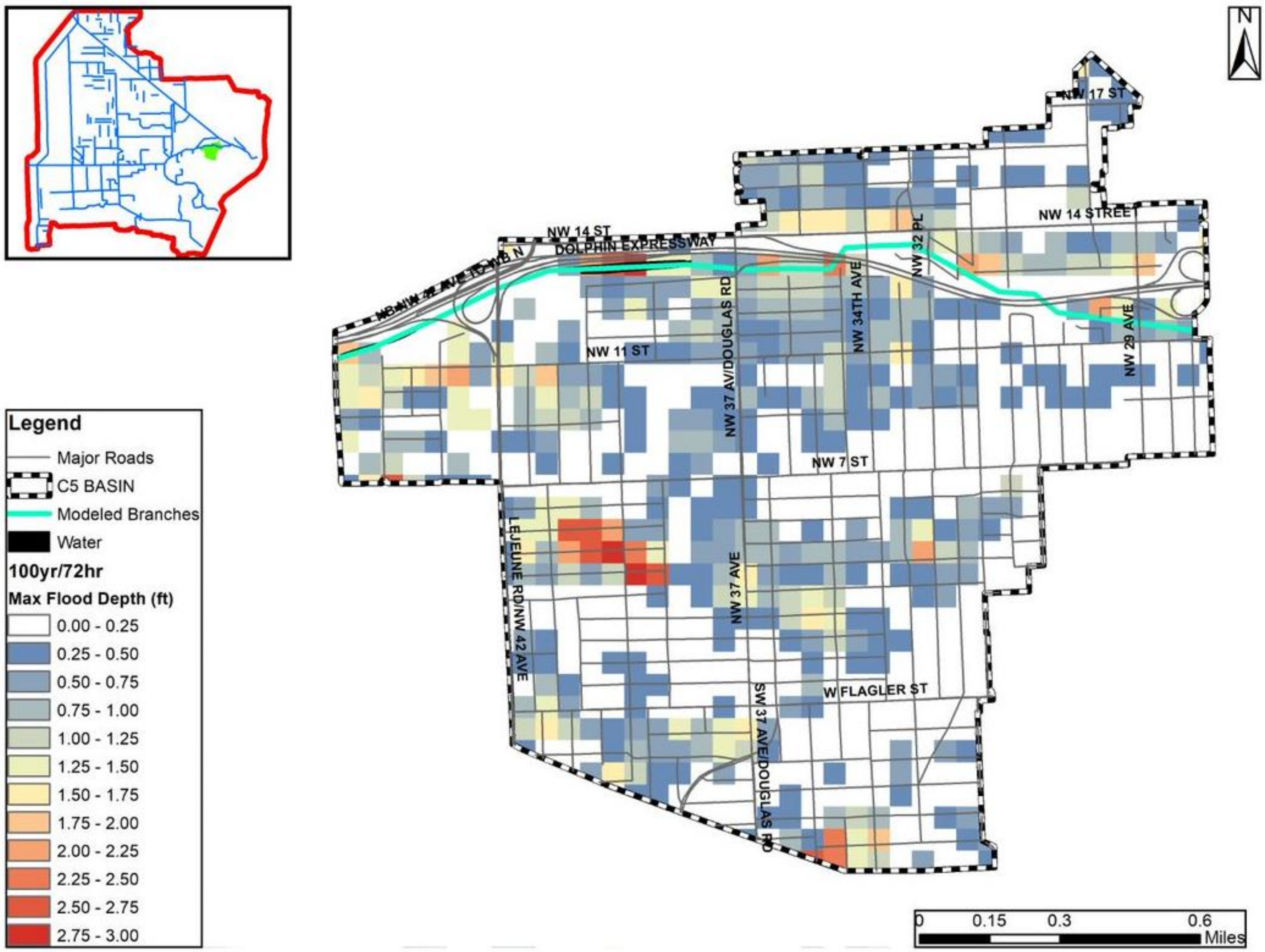


Figure 9-224. Urban Flooding Depth Difference of SLR +1ft and Current Conditions. for the 100-year Storm in the C5 Watershed

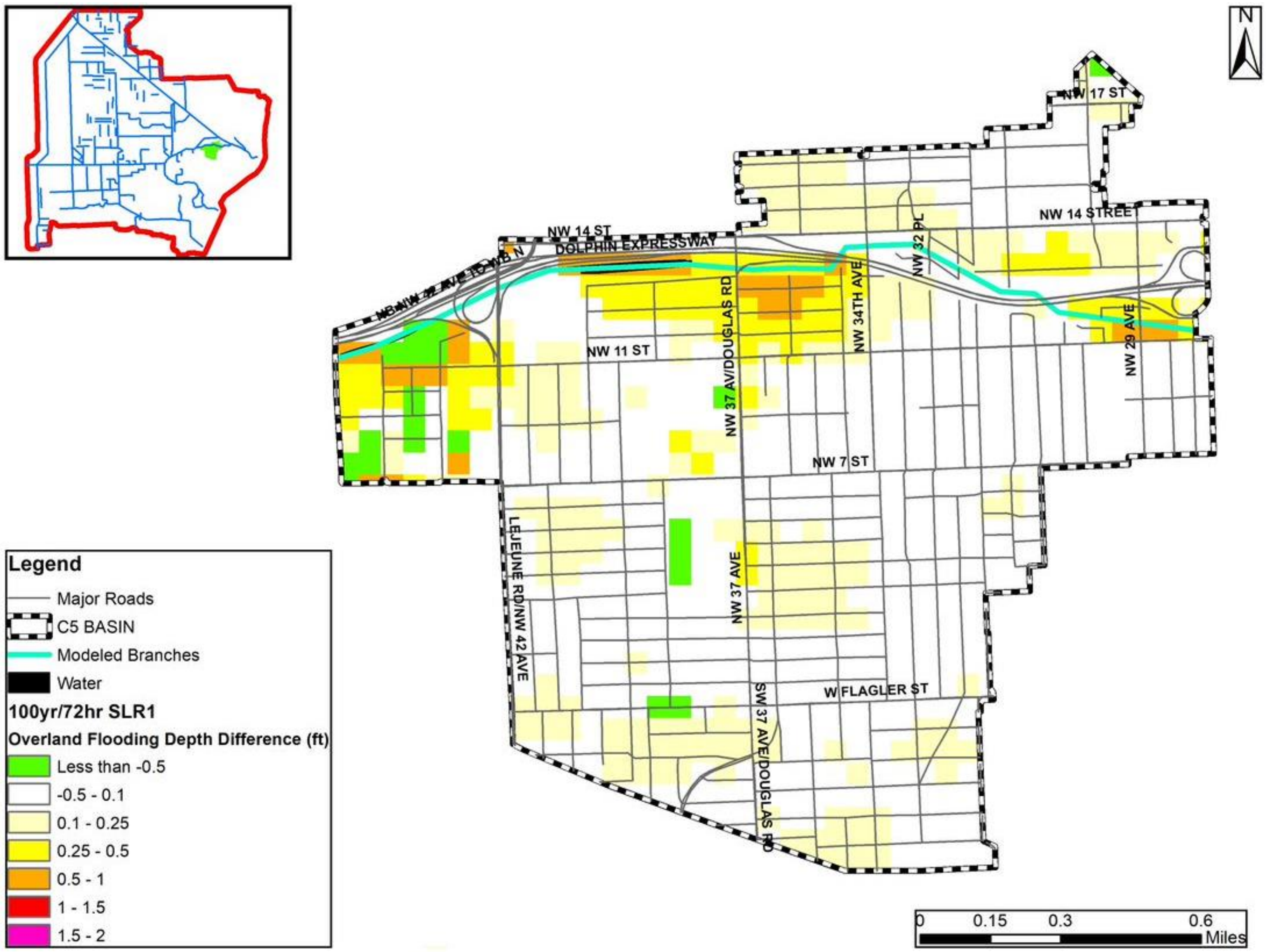


Figure 9-225. Urban Flooding Depth Difference of SLR +2ft and Current Conditions. for the 100-year Storm in the C5 Watershed

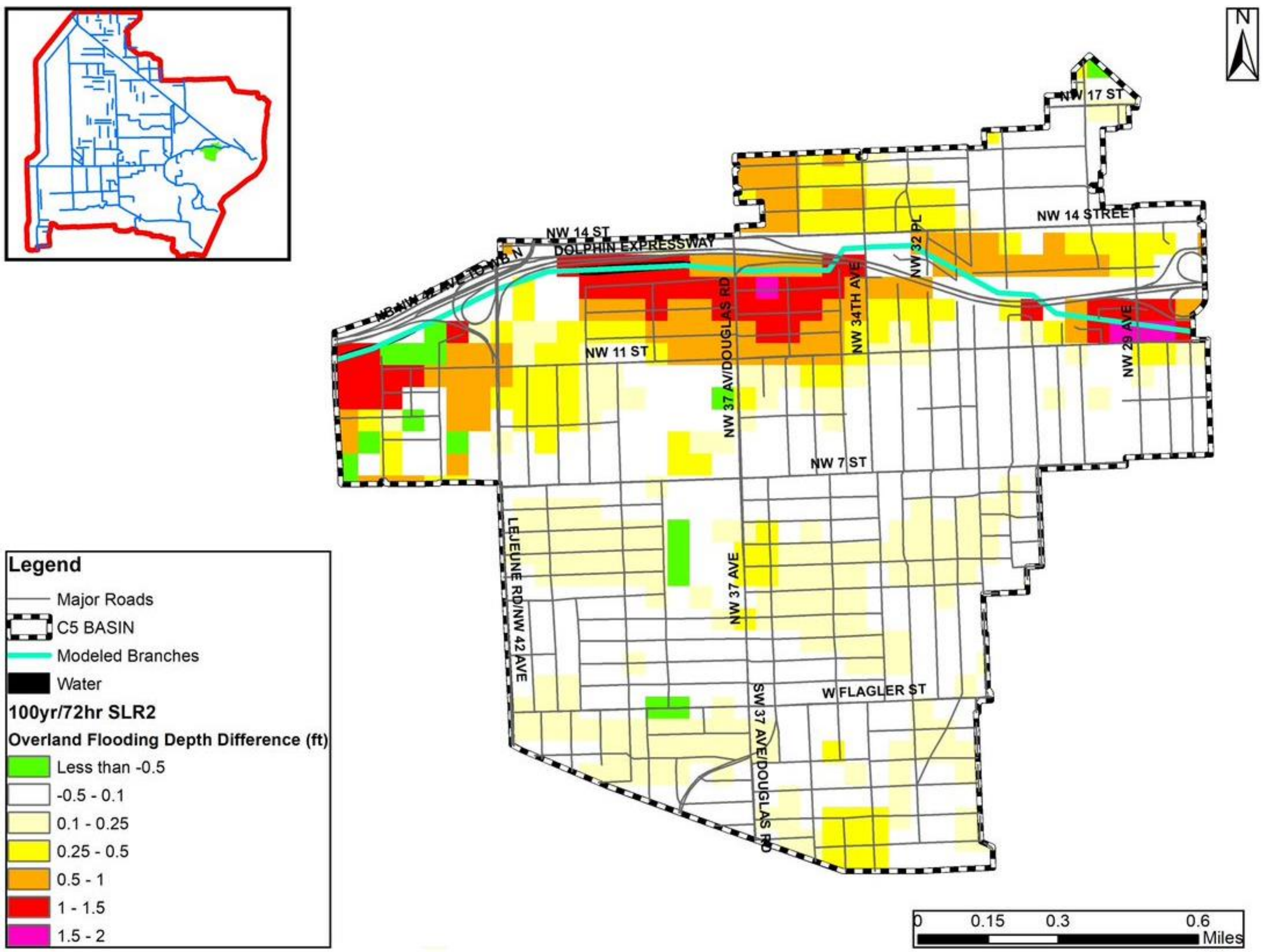
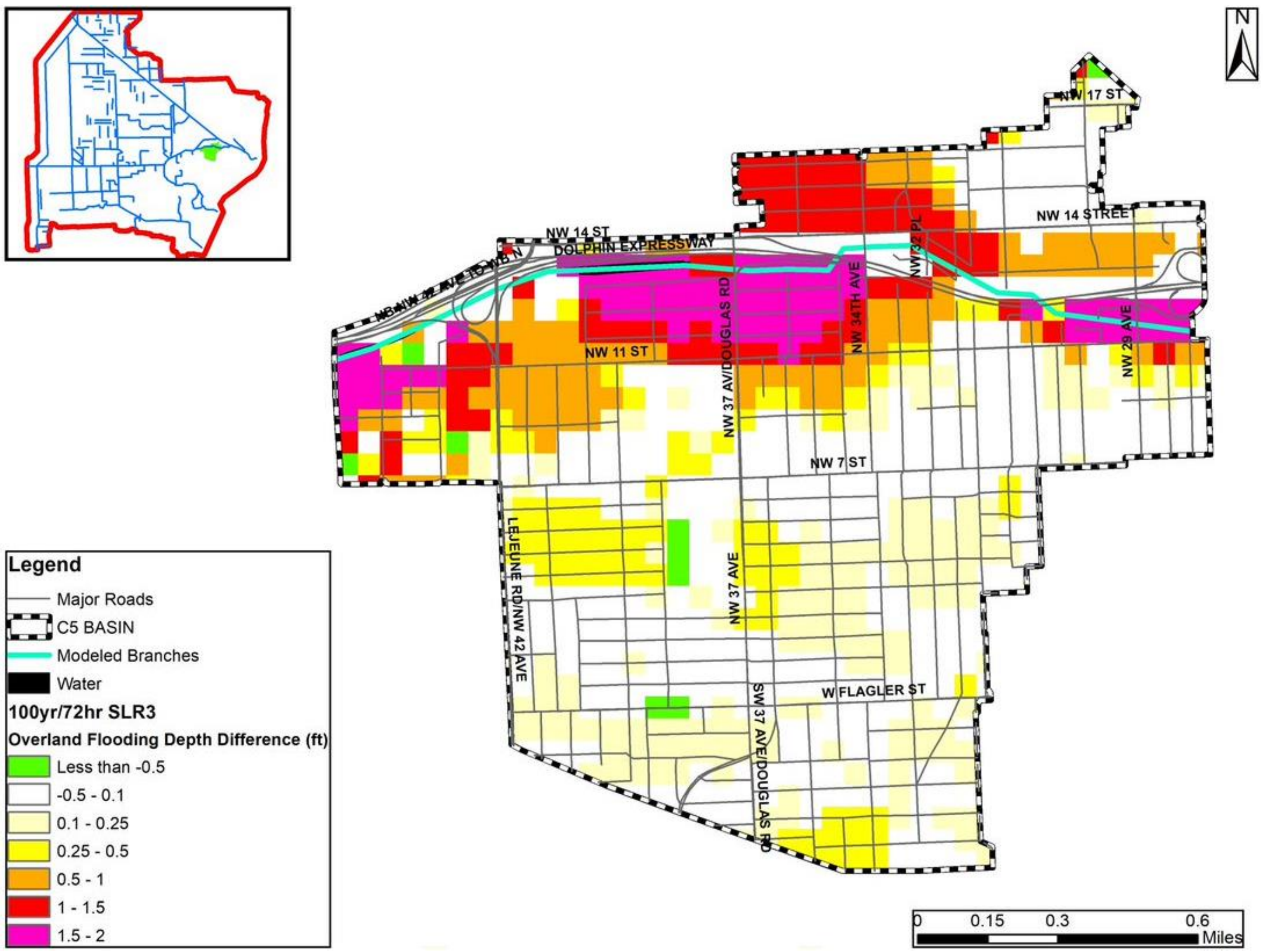


Figure 9-226. Urban Flooding Depth Difference of SLR +3ft and Current Conditions. for the 100-year Storm in the C5 Watershed



9.4.6. C5 – PM #6 DURATION OF FLOODING

9.4.6.1. CANAL FLOOD DURATION

In discussions with water managers at SFWMD, it was found that no target stage is available for S25 to determine duration of flooding in the canal. However, the optimum head elevation, as determined by the structure information sheet, was a reasonable elevation for establishing the duration of each storm event for Comfort Canal Southfork for the current conditions. The optimum head elevation is 2.0 ft-NGVD or 0.46 ft-NAVD using the structure conversion of (-)1.54ft. However, this Reference Elevation would not be acceptable under future SLR conditions, as the storms, and even the normal wet season canal elevations may be higher than 0.46 ft-NAVD.

A new Reference Elevation was established based on the off-trigger elevation used for the S25B Pump and, by proxy, the S26 Pump, which will only run if the S25B pump is on. The trigger is that the stages at MRMS1 in the C6 Canal should be less than 3.2 ft-NAVD. Since this watershed is highly impacted, and flooding will indeed be driven by tidal stages for all future SLR conditions, it would be a reasonable assumption to make tailwater conditions the new Reference Elevation at which storm duration is determined. **Figure 9-227** shows the location of MRMS1, within the C6 Canal just east of NW 27th Avenue, in relation to the S25 structure, just west of NW 27th Avenue within the Comfort Canal Southfork.

Table 9-77 provides the canal flood duration of each storm event, as determined from canal stages at MRMS1 for each SLR condition. For reference, the current conditions canal flood duration using the S25 HW stages above 0.46 ft-NAVD are also provided in the table.

Figure 9-228 shows stages at MRMS1 for each design storm and the duration that the storm is above the Reference Elevation of 3.2 ft-NAVD, the Reference Elevation is shown as a solid black line. The initiation of the flood event is determined to be when the stages exceed the Reference Elevation prior to the peak, and the finalization is determined to be the first time the stages drop below the Reference Elevation after the peak.

Figure 9-227. Location of MRMS1 gage in relation to S25

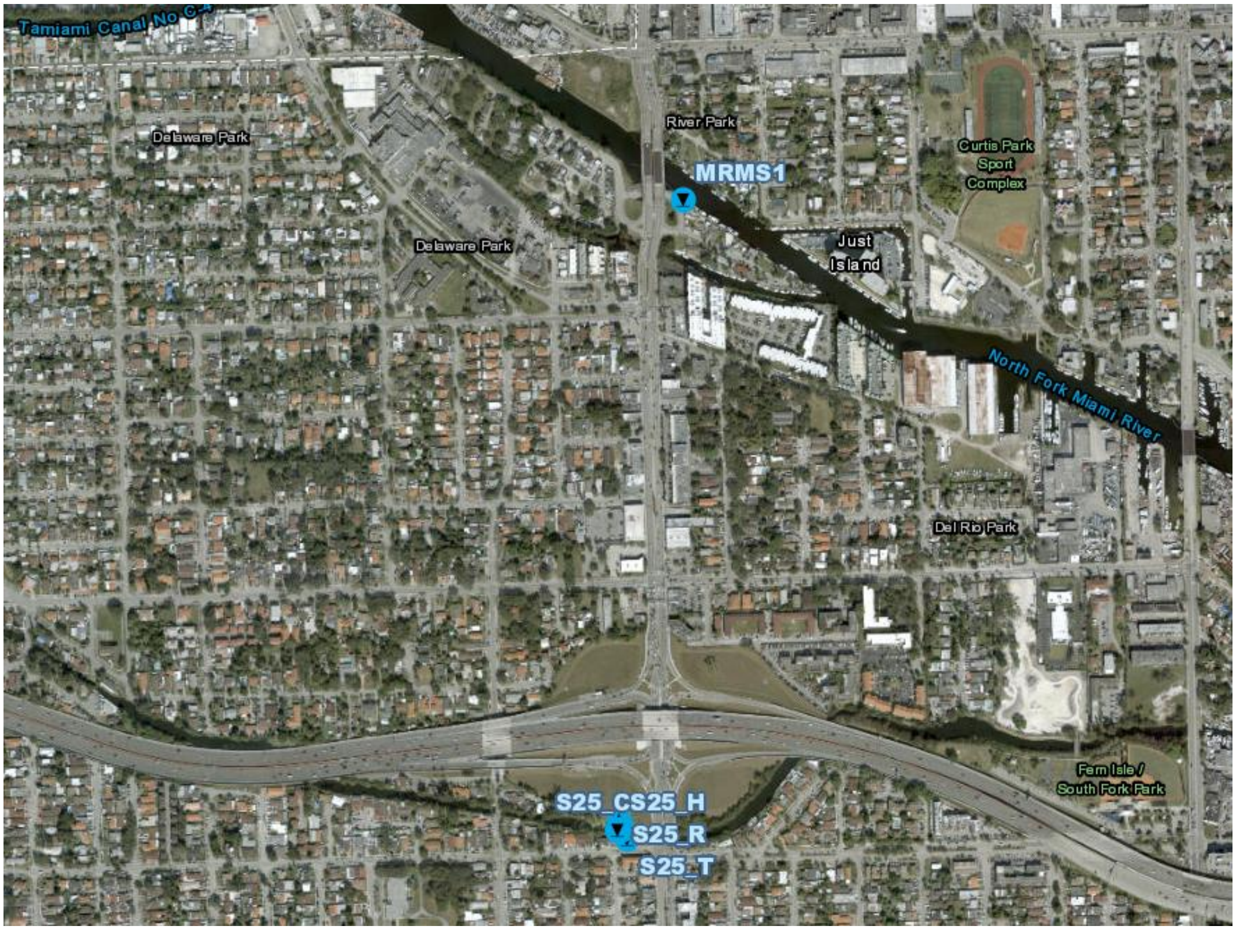


Table 9-77. Storm Duration Indicated at MRMS1 for the C5 Watershed

Design Storm	Duration (hours)				
	Current Conditions with 2.23 ft-NAVD Reference Elev.	Current Conditions with 3.43 ft-NAVD Reference Elev.	SLR1	SLR2	SLR3
100-Year	85.3	12.8	20.0	46.7	182.4
25-Year	83.8	0.9	15.4	34.4	170.4
10-Year	72.9	0.0*	9.5	21.7	159.7
5-Year	60.6	0.0*	5.1	20.1	159.4
*Canal stages do not recede past the Reference Elevation after the storm and therefore the storm duration is longer than the values provided.					

9.4.6.2. WATERSHED FLOOD DURATION

Table 9-78 tabulates the total area of flood inundation (in square miles) per flood duration range for all areas in the C5 Watershed and **Table 9-79** does this for all urban areas within the C5 Watershed. This table shows that the greatest distribution of flooding duration in the watershed is between zero and one hour of flooding, which can be considered insignificant. Flooding is also distributed in the 12 to 48-hour range for the 100- year and 25-year storms, indicating that most of the stormwater runoff from the watershed has a one to two-day journey to Comfort Canal Southfork.

Table 9-80 calculates the percentage of the total urban areas within the C5 Watershed that are inundated by 3 inches or more for the flood duration indicated. Because the flood duration greater than 1 hour includes all areas inundated with 3 inches or more, this row shows the same percentage as that in **Table 9-75** in PM #5. Additionally, **Figure 9-229, Figure 9-230, Figure 9-231**, and **Figure 9-232** provide a graphical view of this data, plotting the flood duration against the percentage of area inundated for the urban areas of the C5 Watershed for the 100-year, 25-year, 10-year, 5-year storm events, respectively. These graphics also include the percent increase above the current conditions on the secondary axis, to visualize how the flood duration is changing with each SLR condition. For the 100-year design storms, the percentage of the watershed that is flooded for 24 hours shows the greatest increase above current conditions for all SLR conditions. For the 25-year storm events, these increases show similar shapes as with the 100-year storms; however, they shift to shorter durations. In addition, SLR shows a secondary peak at around the 48-hour mark. This indicates a greater impact from both initial rainfall/runoff (due to higher initial conditions) and from reduced drainage capacity of the watershed as a result of higher canal elevations.

Figure 9-233 and **Figure 9-234** provide flood duration maps for the C5 Watershed for overland flooding depths exceeding 0.25 feet for the current conditions 5-year and 100-year 3-day design storms, respectively. Water areas, such as existing lakes and ponds, are masked in black.

Figure 9-235, **Figure 9-236**, and **Figure 9-237** provide the difference in flood duration between the Current Conditions and SLR +1 foot, SLR +2 feet, and SLR +3 feet, respectively, for the 100-year storm event. When compared the difference maps for flood depth, this can show that even for areas where flood depths are not increasing with future SLR conditions, the duration of flooding may increase due to the reduced ability of the area to drain to the receiving canals that are experiencing higher stages, for example in the areas south of NW 7th Street and in the area east of the Greyhound Track near NW 4th Street.

Flood duration maps over the entire C5 Watershed for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current conditions and future SLR scenarios are provided in **Appendix D**. Also provided in **Appendix D** are the differences in flood duration maps for the 25-year, 10-year, and 5-year design storms.

For the C5 Watershed some areas tend to experience flooding for longer periods. Some areas of concern are near the Comfort Canal and NW 37th St and in the area east of the Greyhound Track near NW 4th Street.

Figure 9-228. Flood Duration for the C5 Watershed as Determined at MRMS1

Table 9-78. Flood Duration per Area of Inundation (in sq. mi.) for the C5 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	0.90	1.29	1.51	1.68	0.83	1.22	1.43	1.57	0.76	1.12	1.34	1.45	0.68	0.98	1.23	1.32
1 to 2 hr.	0.10	0.02	0.02	0.01	0.13	0.02	0.01	0.00	0.11	0.02	0.01	0.01	0.09	0.04	0.00	0.01
2 to 4 hr.	0.06	0.04	0.03	0.02	0.06	0.04	0.02	0.01	0.06	0.04	0.02	0.02	0.05	0.04	0.03	0.01
4 to 8 hr.	0.06	0.07	0.05	0.03	0.05	0.05	0.05	0.05	0.04	0.04	0.03	0.04	0.04	0.04	0.02	0.02
8 to 12 hr.	0.04	0.06	0.04	0.02	0.05	0.05	0.03	0.04	0.03	0.03	0.02	0.04	0.02	0.02	0.02	0.02
12 to 24 hr.	0.26	0.16	0.11	0.06	0.20	0.20	0.13	0.09	0.16	0.17	0.17	0.11	0.13	0.14	0.09	0.12
24 to 48 hr.	0.22	0.15	0.07	0.04	0.25	0.15	0.13	0.07	0.30	0.22	0.15	0.10	0.24	0.18	0.21	0.15
48 to 72 hr.	0.12	0.05	0.02	0.01	0.13	0.08	0.03	0.02	0.15	0.10	0.06	0.04	0.18	0.17	0.08	0.07
72 to 96 hr.	0.07	0.02	0.01	0.00	0.09	0.03	0.02	0.01	0.08	0.06	0.02	0.02	0.14	0.07	0.05	0.02
96 to 144 hr.	0.03	0.01	0.00	0.00	0.05	0.02	0.01	0.00	0.09	0.04	0.03	0.02	0.10	0.07	0.03	0.04
144 to 192 hr.	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.04	0.02	0.01	0.01	0.07	0.03	0.04	0.02
192 to 240 hr.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.01	0.00	0.03	0.03	0.02	0.01
240 to 336 hr.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.05	0.02	0.01	0.01
336 to 420 hr.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.02	0.02	0.02
420 hr.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.02	0.02	0.02

Total Basin Area = 1.9 square miles

Table 9-79. Flood Duration per Area of Inundation (in sq. mi.) for Urban Areas in the C5 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	0.86	1.22	1.42	1.58	0.79	1.14	1.34	1.47	0.73	1.04	1.25	1.36	0.65	0.91	1.14	1.23
1 to 2 hr.	0.10	0.02	0.02	0.01	0.11	0.02	0.01	0.00	0.09	0.02	0.01	0.01	0.08	0.03	0.00	0.01
2 to 4 hr.	0.05	0.04	0.03	0.02	0.06	0.04	0.02	0.01	0.06	0.04	0.02	0.02	0.04	0.04	0.03	0.01
4 to 8 hr.	0.06	0.06	0.05	0.03	0.05	0.05	0.05	0.05	0.04	0.04	0.03	0.03	0.04	0.04	0.02	0.02
8 to 12 hr.	0.04	0.06	0.04	0.02	0.05	0.05	0.03	0.04	0.03	0.02	0.02	0.04	0.02	0.01	0.02	0.02
12 to 24 hr.	0.25	0.15	0.10	0.06	0.20	0.20	0.12	0.09	0.15	0.17	0.16	0.11	0.13	0.14	0.08	0.11
24 to 48 hr.	0.20	0.13	0.07	0.04	0.24	0.14	0.13	0.07	0.29	0.20	0.14	0.10	0.22	0.17	0.20	0.15
48 to 72 hr.	0.11	0.05	0.02	0.01	0.12	0.08	0.03	0.02	0.13	0.09	0.06	0.04	0.17	0.16	0.08	0.07
72 to 96 hr.	0.06	0.02	0.01	0.00	0.09	0.03	0.02	0.01	0.08	0.06	0.02	0.02	0.13	0.07	0.05	0.02
96 to 144 hr.	0.03	0.01	0.00	0.00	0.05	0.02	0.01	0.00	0.09	0.04	0.03	0.02	0.10	0.07	0.03	0.04
144 to 192 hr.	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.04	0.02	0.01	0.01	0.07	0.03	0.04	0.02
192 to 240 hr.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.01	0.01	0.00	0.03	0.03	0.02	0.01
240 to 336 hr.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.05	0.02	0.01	0.01
336 to 420 hr.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.02	0.02	0.02
420 hr.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.02	0.02	0.02	0.01

Total Basin Urban Area = 1.8 square miles

Table 9-80. Percentage of Total Area Inundated for Urban Areas in the C5 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0 hr.	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 1 hr.	51.3	30.6	19.1	10.4	55.3	35.4	23.9	16.7	58.7	40.8	28.9	22.8	62.9	48.0	35.0	30.3
>= 2 hr.	45.9	29.6	18.2	9.9	48.8	34.4	23.2	16.4	53.9	39.6	28.4	22.4	58.5	46.2	34.8	29.7
>= 4 hr.	42.8	27.1	16.6	9.0	45.6	32.2	22.2	15.8	50.7	37.5	27.5	21.4	55.9	44.1	33.2	29.2
>= 8 hr.	39.4	23.9	13.8	7.5	42.8	29.4	19.2	13.0	48.4	35.2	26.0	19.5	53.8	41.7	32.4	28.0
>= 12 hr.	36.9	20.6	11.7	6.5	39.9	26.6	17.3	10.8	46.5	33.8	24.8	17.1	52.6	40.9	31.1	27.1
>= 24 hr.	22.7	11.8	5.9	3.1	28.8	15.4	10.4	5.9	37.8	24.1	15.7	11.1	45.5	33.1	26.4	20.9
>= 48 hr.	11.5	4.3	2.0	0.8	15.4	7.4	3.2	1.8	21.3	12.6	7.5	5.2	33.1	23.6	15.3	12.2
>= 72 hr.	5.2	1.5	0.6	0.0	8.8	3.1	1.4	0.6	13.6	7.3	3.9	3.1	23.4	14.6	10.6	8.0
>= 96 hr.	1.9	0.5	0.0	0.0	3.8	1.4	0.5	0.0	9.3	3.9	2.7	2.2	15.9	11.0	7.6	6.6
>= 144 hr.	0.1	0.0	0.0	0.0	1.1	0.0	0.0	0.0	3.9	1.9	1.1	0.9	10.3	6.9	6.0	4.5
>= 192 hr.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.9	0.8	0.4	0.1	6.6	5.0	3.6	3.1
>= 240 hr.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.1	0.0	0.0	5.1	3.2	2.4	2.3
>= 336 hr.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.2	2.0	1.8	1.8
>= 420 hr.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.3	1.1	0.9	0.8

Figure 9-229. Percentage and Duration of Inundation for the C5 Watershed for the 100-year Storm Event

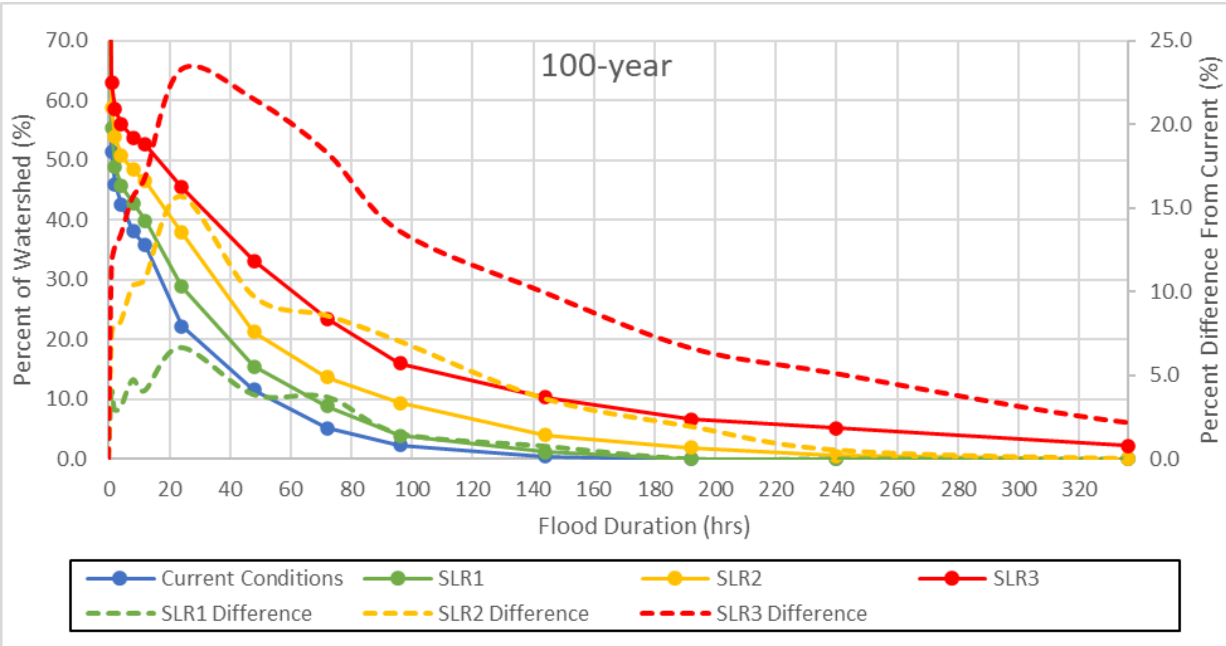


Figure 9-230. Percentage and Duration of Inundation for the C5 Watershed for the 25-year Storm Event

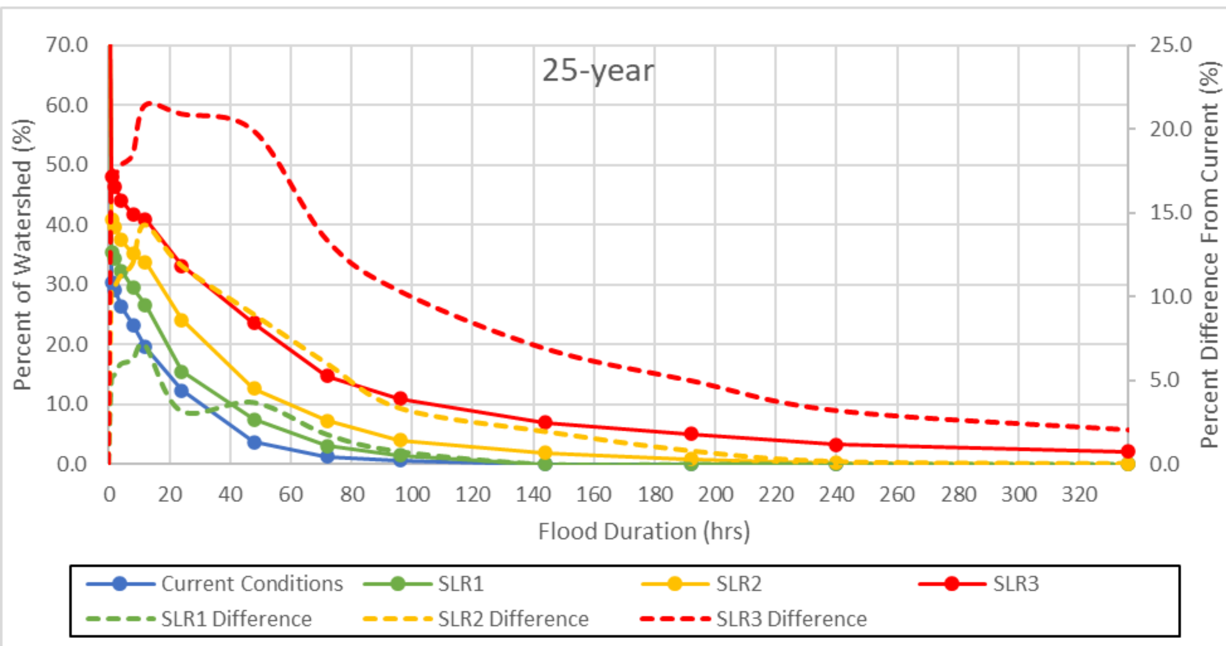


Figure 9-231. Percentage and Duration of Inundation for the C5 Watershed for the 10-year Storm Event

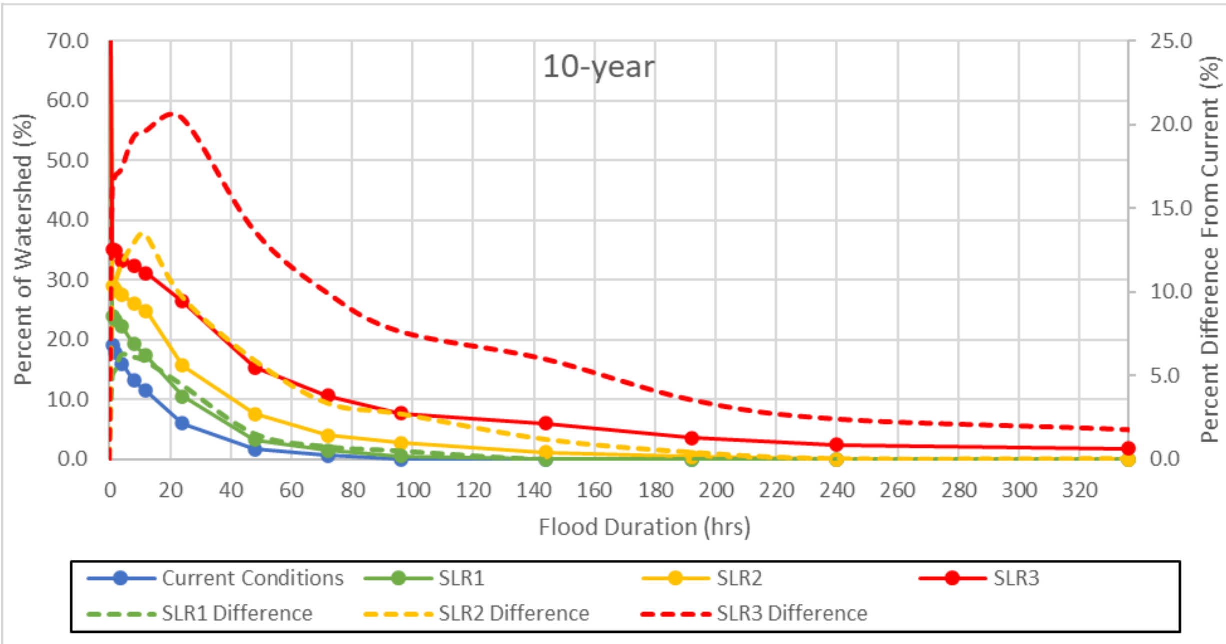


Figure 9-232. Percentage and Duration of Inundation for the C5 Watershed for the 5-year Storm Event

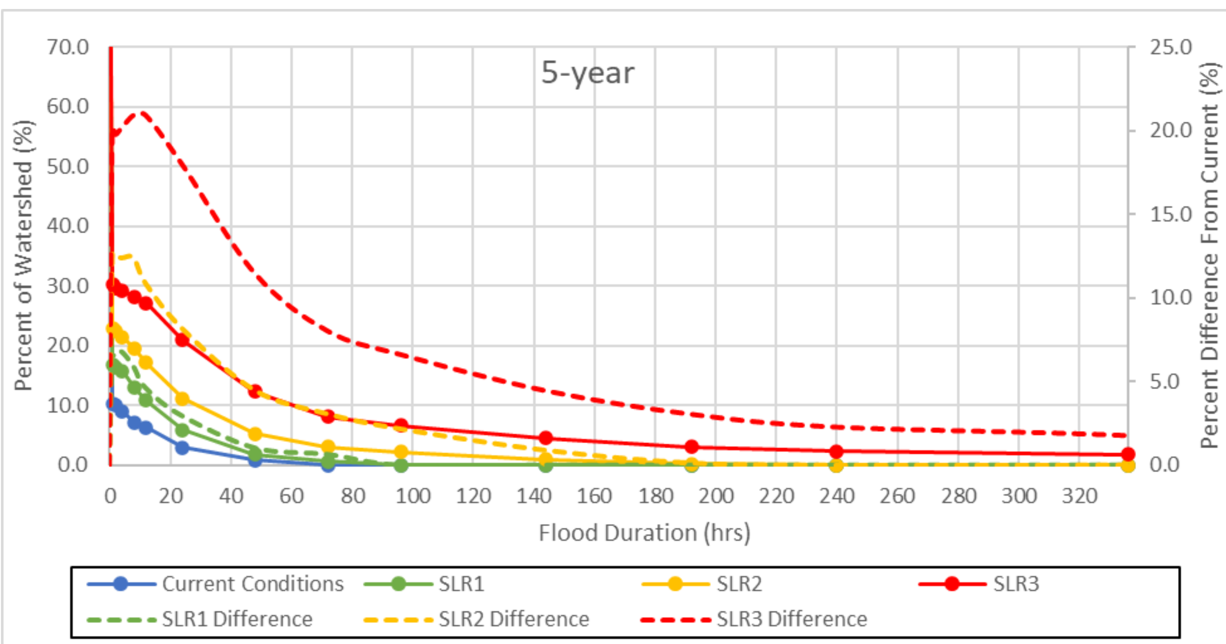


Figure 9-233. Flood Duration Map for the Current Conditions 25-year 3-day Design Storm for the C5 Watershed

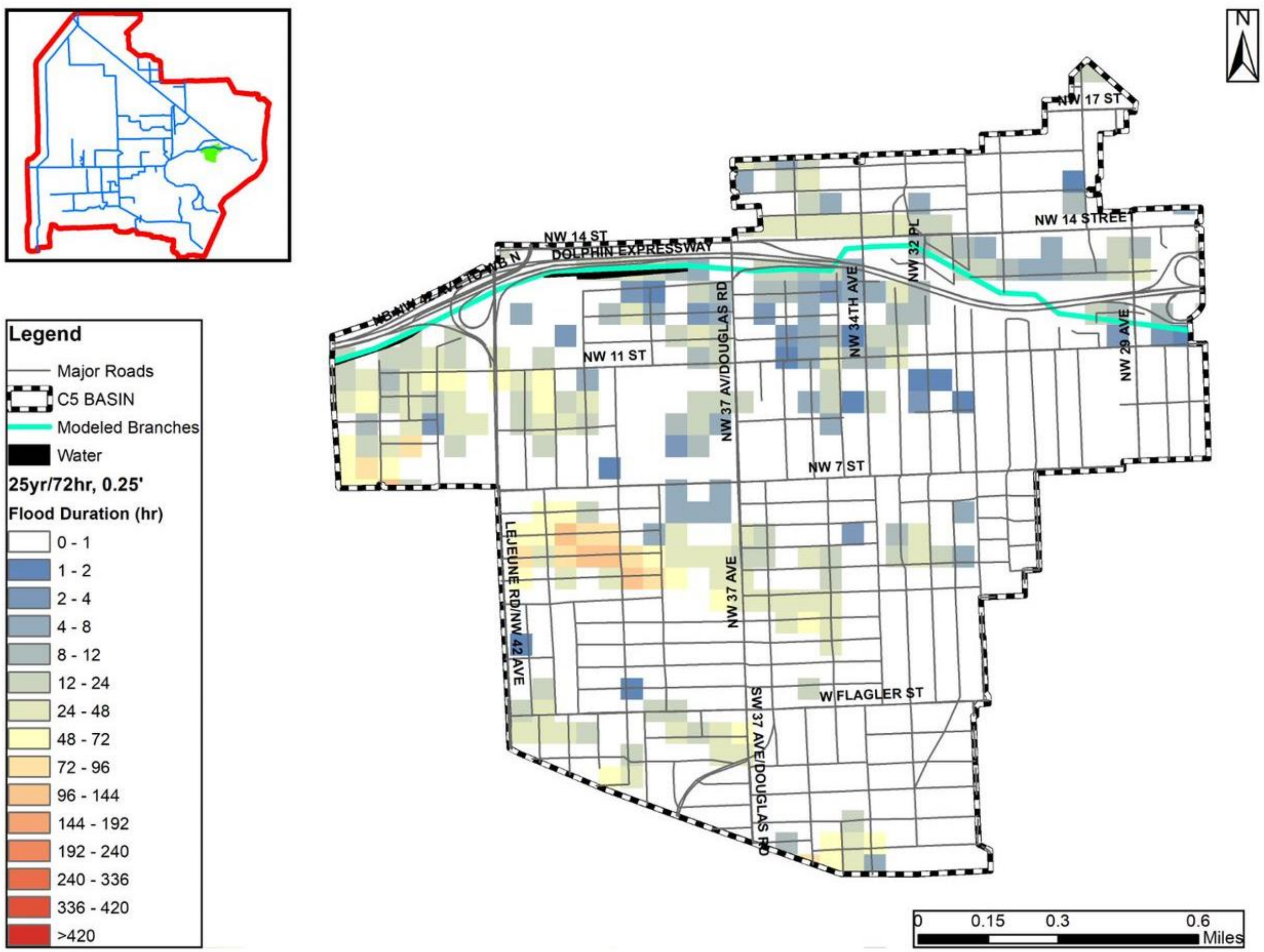


Figure 9-234. Flood Duration Map for the Current Conditions 100-year 3-day Design Storm for the C5 Watershed

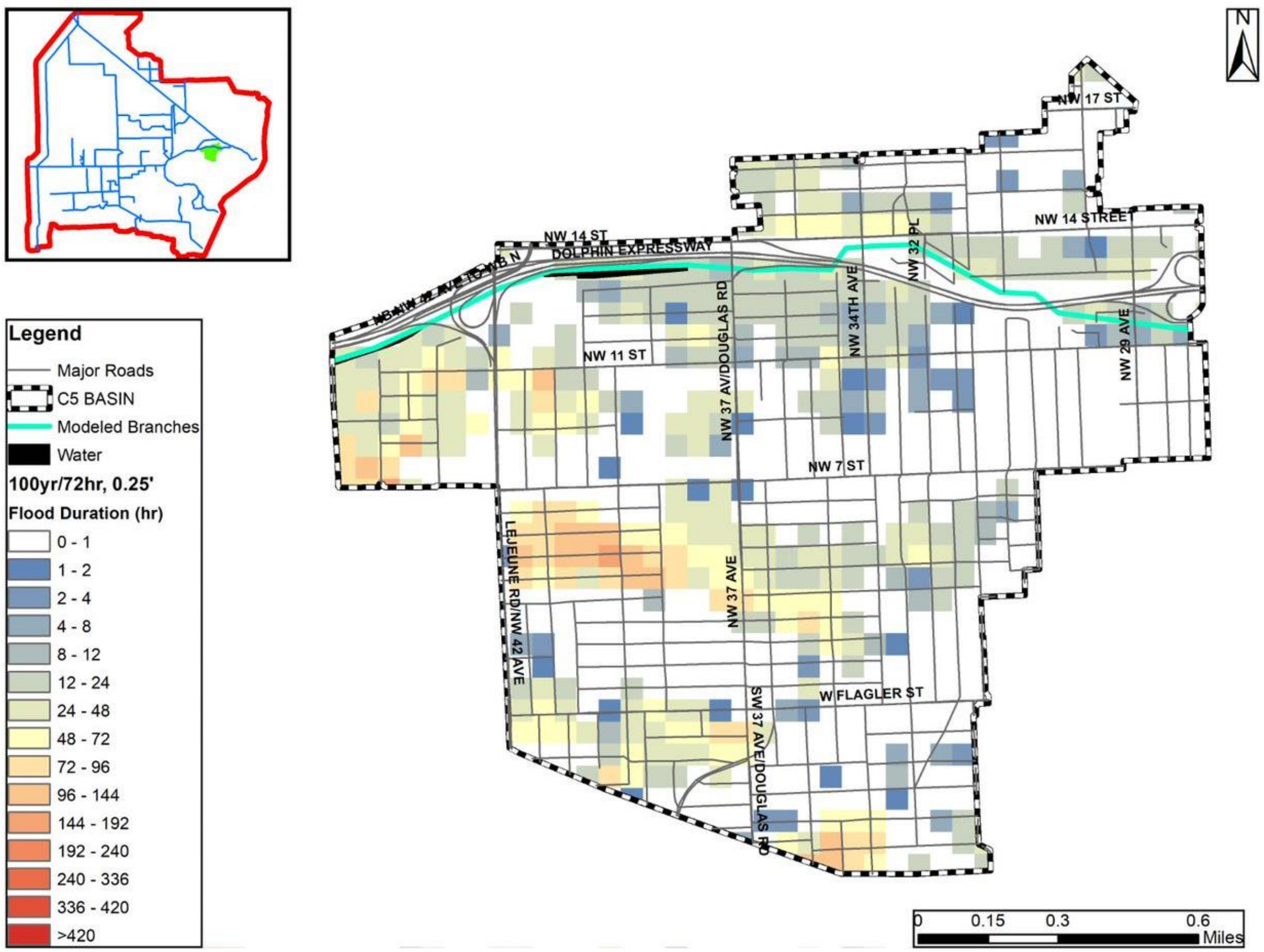


Figure 9-235 Urban Flooding Duration Difference of SLR +1ft and Current Conditions for the 100-year Storm in the C5 Watershed

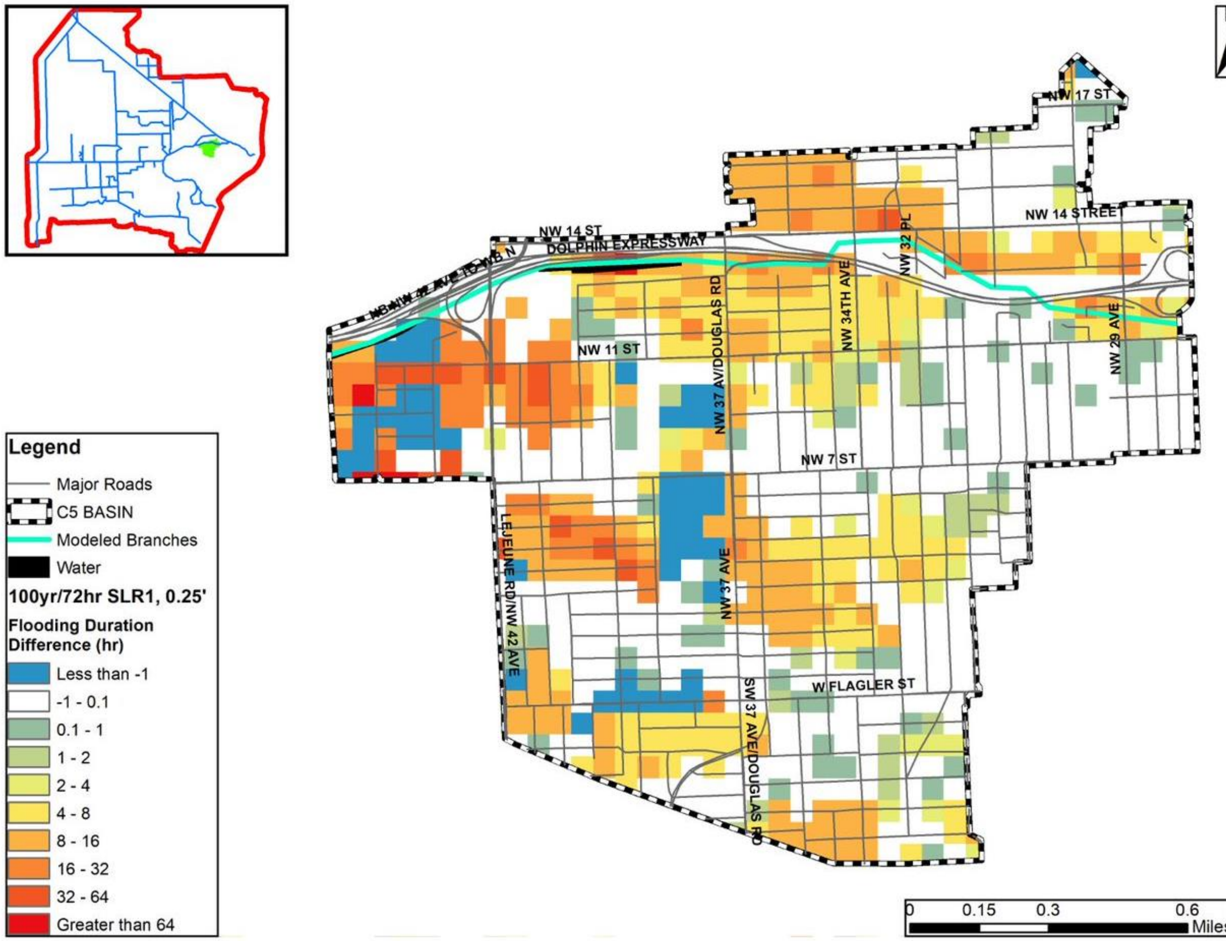


Figure 9-236. Urban Flooding Duration Difference of SLR +2ft and Current Conditions for the 100-year Storm in the C5 Watershed

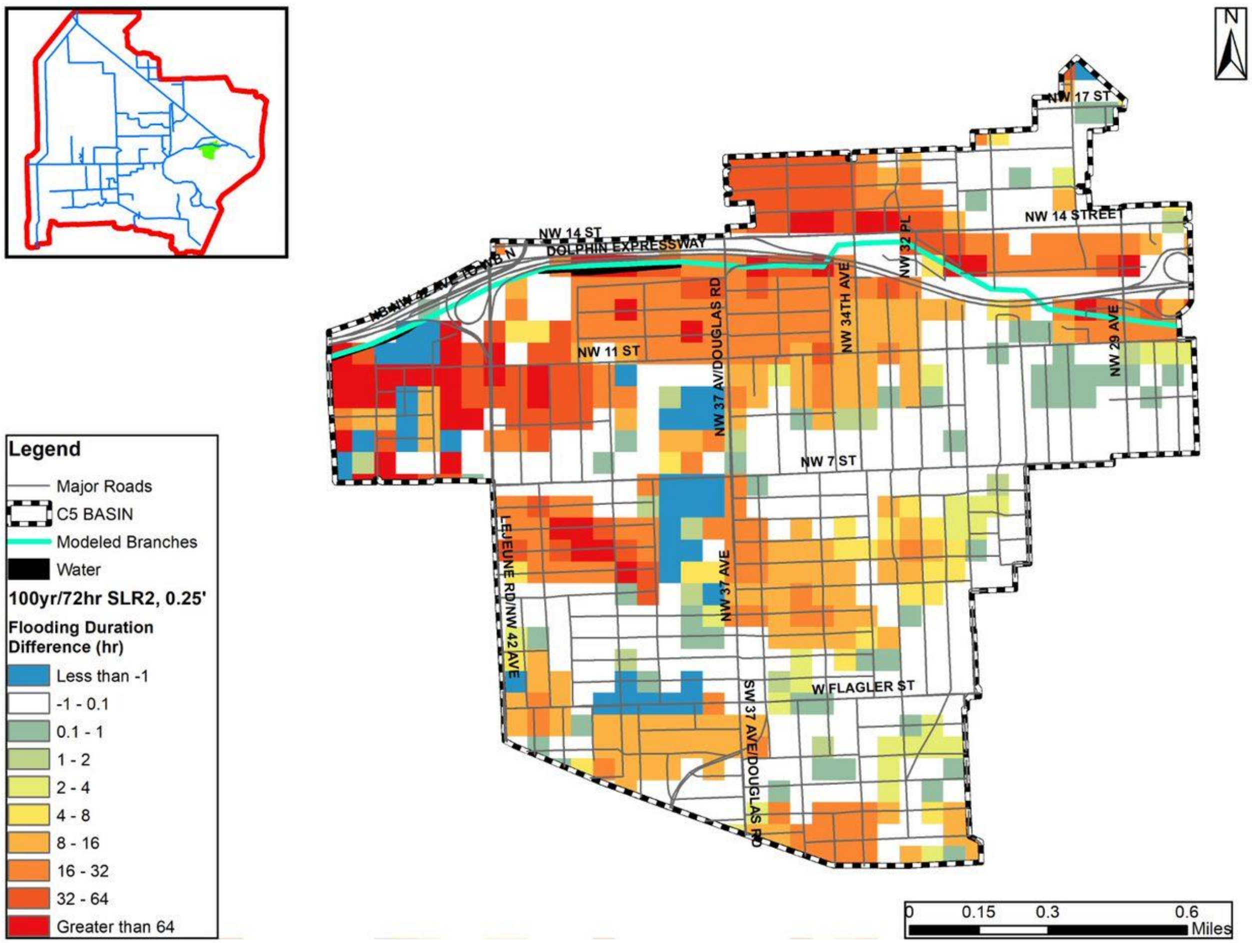
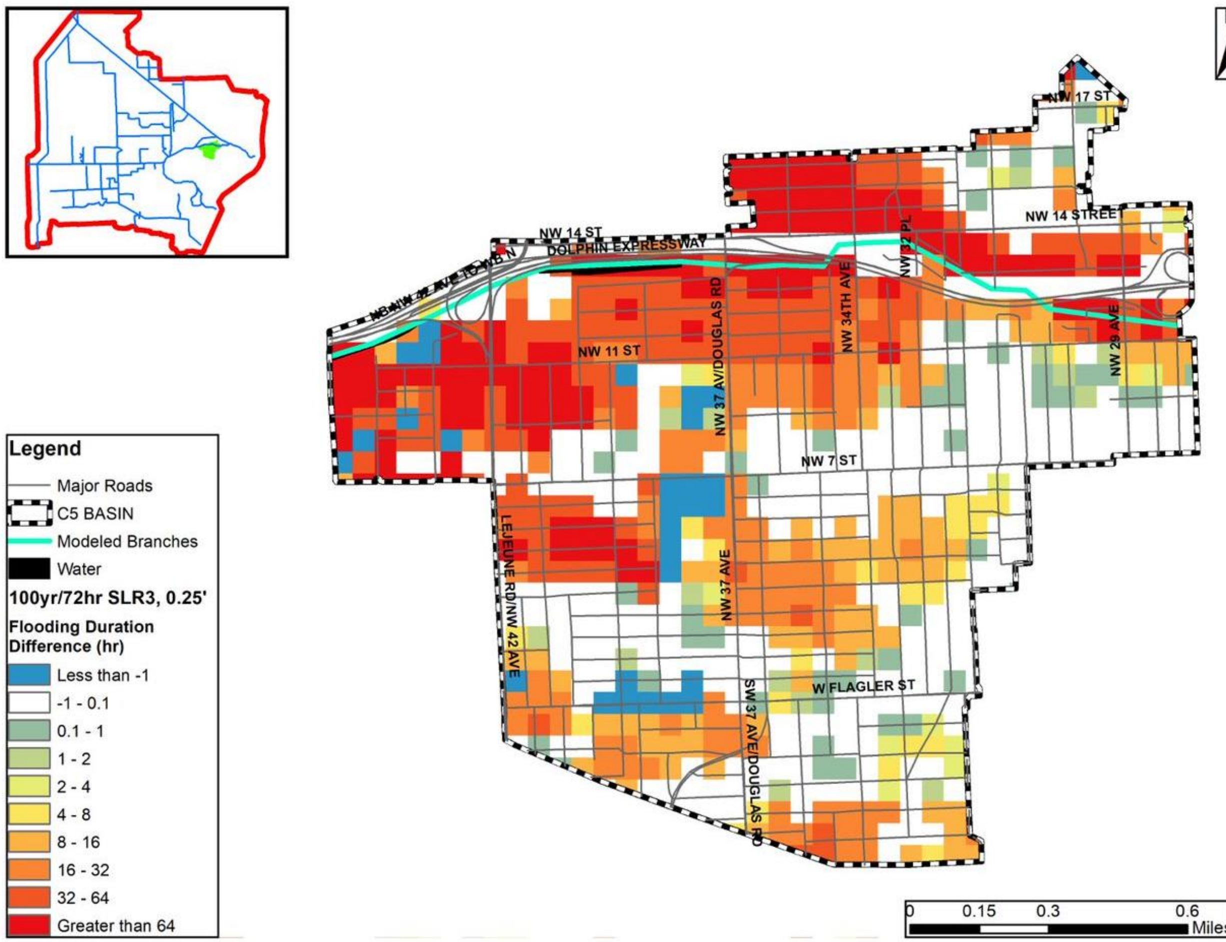
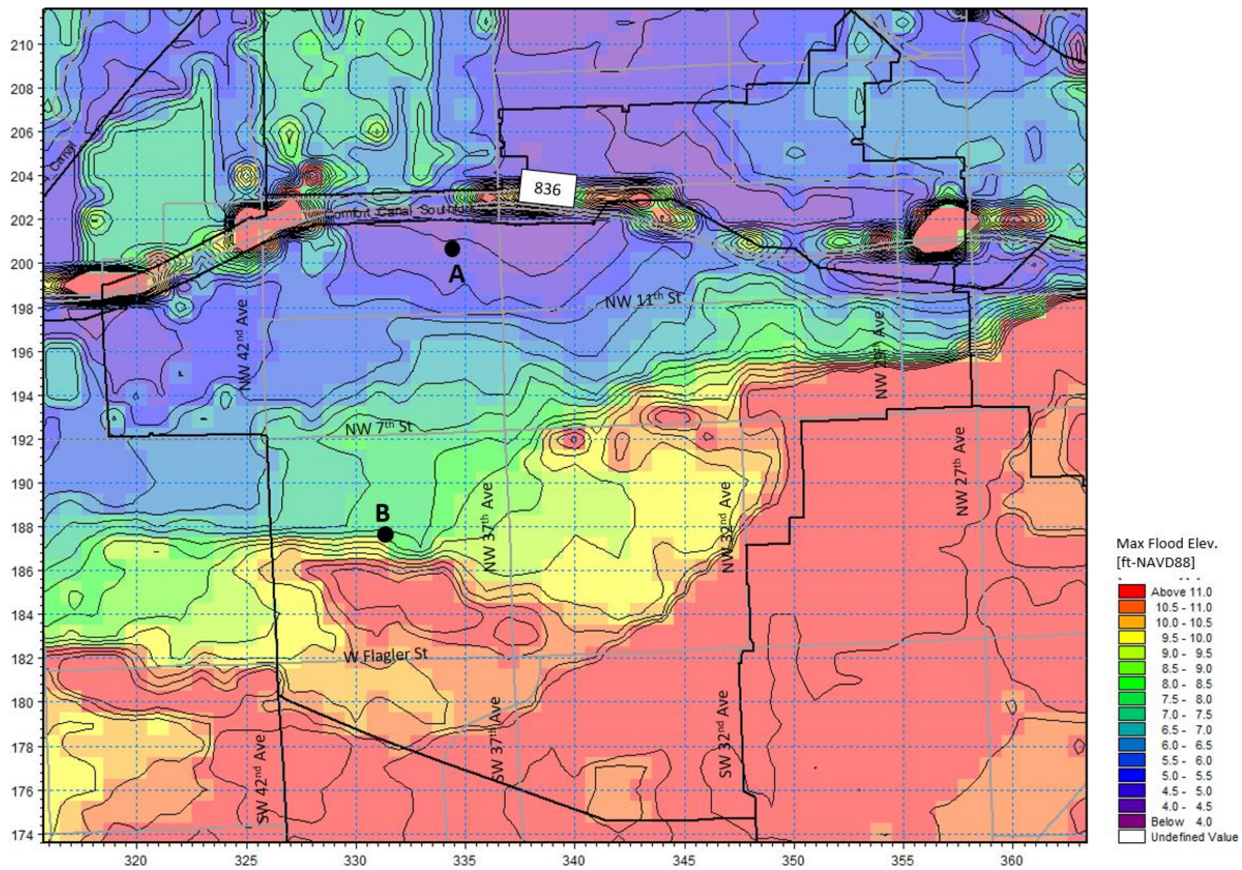


Figure 9-237. Urban Flooding Duration Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C5 Watershed



In the area near the Comfort Canal, flooding in the overland region was reviewed further to determine if canal stages are impacting flooding. **Figure 9-238** provides a map of the overland flooding elevation to compare canal stages with the overland flooding.

Figure 9-238. Overland Flood Elevation for the 100-year Design Storm for the C5 Watershed



As discussed in PM #1, the canal experiences overtopping during the 100-year storm, however it is only overtopped 0.16 feet and the overtopping only lasts for 5 hours. Stages at point A, as indicated in **Figure 9-238**, show that the elevation of overland water at this location is directly impacted by the canal stages, even though the canal is not overtopping the embankment for long, there is nowhere for water to drain so overland flooding rises in unison with the canal stages. However, because point A is so close to the canal, it also experiences a faster recession of flooding depths, as it can drain quickly to the canal. Alternately, stages at point B rise with the storm and drain very slowly due to it being located in a low-lying area surrounded by higher elevation roads, as shown in **Figure 9-240**. This would constitute a tertiary drainage issue for point B.

Figure 9-239. Comfort Canal Southfork Stages Compared with Overland Flood Elevation for 100-year Design Storm

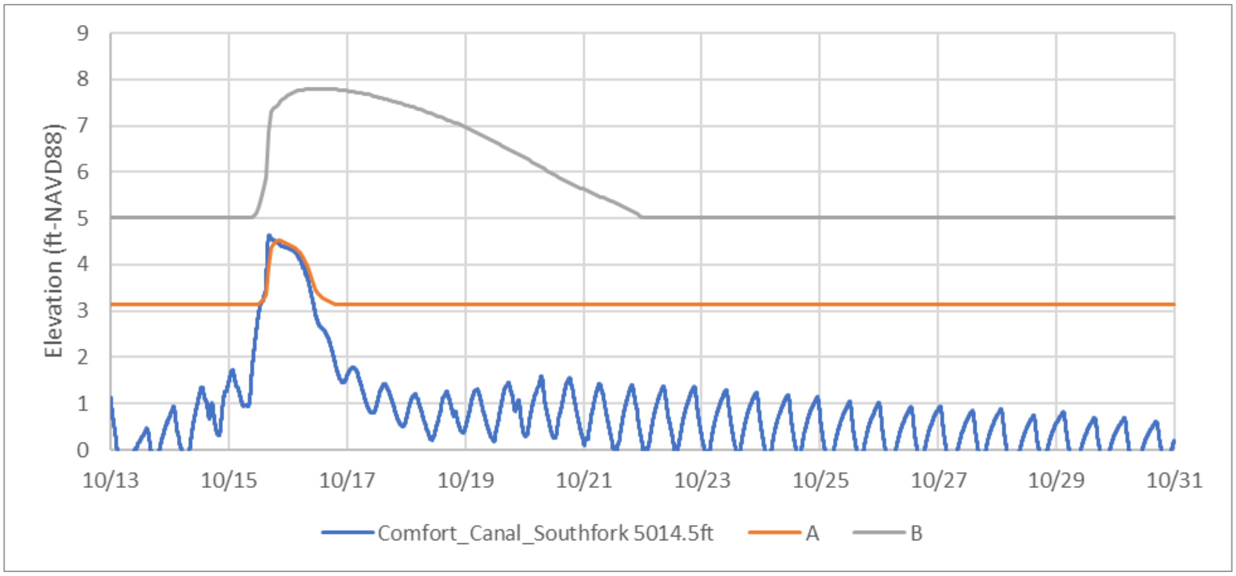
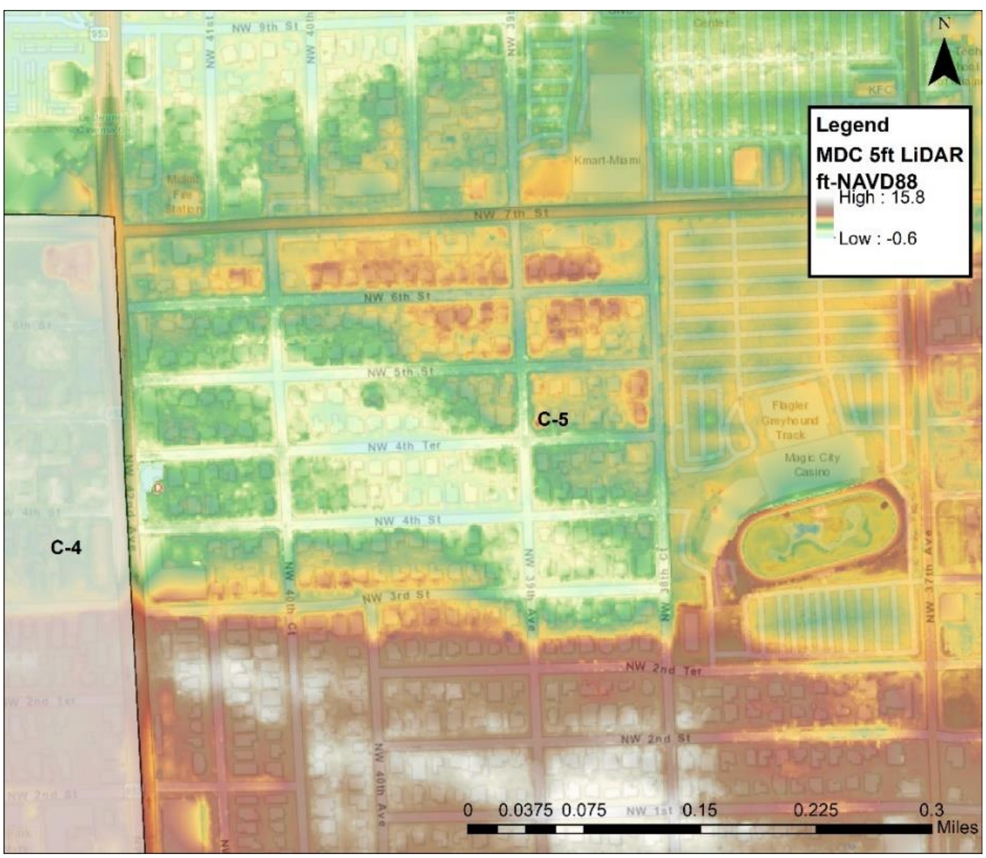


Figure 9-240. LiDAR Topography of Low-Lying Basin in C5 Watershed



9.4.7. SUMMARY OF LOS AND RATING FOR THE C5 WATERSHED

The maximum design storm frequency that the C5 Watershed passes without incurring negative impacts is summarized for each performance metric in **Table 9-81**.

Under critical consideration for this watershed is PM #1, PM #5 and PM #6, as these relate directly to flooding in canals and overland flooding depth and duration. This watershed experiences flooding over the land surface in two ways: 1) due to tertiary drainage issues for low-lying basins far away from the canal and 2) due to stages in the primary canal rising above the land surface elevation. When stages in the canal exceed the land surface elevation for extensive areas of the watershed, this constitutes a no-pass condition for associated design storm. In this way, we must conclude that the C5 Watershed passes the 10-year design storm for current conditions and the 5-year storm for SLR1 conditions. Due to the fact that the majority of the Comfort Canal Southfork embankment (PM #1) was overtopped for all design storm events for SLR2 and SLR3, these conditions do not pass any of the evaluated design events.

Table 9-81. Performance Metric Summary for the C5 Watershed

Metric	Notes	Current Conds.	SLR +1 ft	SLR +2 ft	SLR +3 ft
PM #1	▪ LiDAR indicated a low road just south of State Road 836 on NW 37th Ave that was overtopped for the 100-year design storm. This was overtopped for the 25-year for SLR1, and all storms overtopped this road for the SLR2 and SLR3 conditions. In addition, NW 42nd Ave was overtopped for the SLR3 100-year storm. ▪ 83% of the top-of-bank elevations along the canal were overtopped for the 100-year event and 0% for the 25-year under current conditions. For SLR1, 0% was overtopped for the 10- and 5-year storms. However, all storms show extensive overtopping for the SLR2 and SLR3 conditions.	25-year	10-year	< 5-year	< 5-year
PM #2	▪ No comparable value found for this basin. ▪ Discharge out of the watershed decreased with SLR1 due to reduced ability to discharge to rising tidal conditions, however, discharge increases again for SLR2 and SLR3 but with a shift to after the storm surge resides, due to the watershed draining both the rainfall and surge that overtopped the S25 structure. This produces relatively similar discharge capacities for all SLR conditions.	--	--	--	--
PM #3	▪ Maximum discharge at S25 falls below design value for the 5-year event with SLR +1ft, and the 5-year, 10-year, and 25-year events for SLR +3. ▪ HW exceeds the water level that will bypass S25 for all future SLR simulations as well as the 25-year and 100-year current condition simulations. The TW exceeds this bypass elevation during the 100-year current conditions simulation and all future SLR except for the 5-year and 10-year SLR1 scenarios.	--	--	--	--
PM #4	▪ Peak 12-hour moving discharge ranges from 149 CFS to 427 CFS, compared to the design discharge of 320 CFS, and decreases with increasing SLR above 1 ft.	--	--	--	--
PM #5	▪ For current conditions, canal stages were shown to be impacting the area immediately surrounding Comfort Canal, for the 100- and 25-year storm events. With some deeper flooding in the area east of the Greyhound Track near NW 4th St for all storm events.	10-year	5-year	5-year	< 5-year
PM #6	▪ Canal: Using the new Reference Elevation at MRMS1, stages a recede during the all storm events for current conditions, SLR1, and SLR2 conditions 72 hours. However, it took the canal longer than 4 days to recede for all SLR 3 storm events. ▪ Watershed: The percent of the watershed that is inundated for longer times (i.e., >24hrs) is considerably low for current conditions (i.e., less than 12%). This is comparable to the 10-year for SLR1, the 5-year for SLR2, and does not have a comparison for SLR3 conditions.	25-year	10-year	5-year	< 5-year
Overall Level of Service		**10-year**	**5-year**	**<5-year**	**<5-year**

9.5. C6 WATERSHED FLOOD PROTECTION LEVEL OF SERVICE

The C6 Watershed consists of areas draining to the C6 Canal from the L-30 Levee southeast to Airport Expressway. The primary discharge canal is the C6 or Miami Canal from the S31 water control structure to the S26 water control structure. S31 is gated culvert that is used for dry season water supply to the Miami Canal and to discharge excess water from WCA 3B, located at the L-30 Levee. S26 is a tidal water control structure that controls water levels in the Miami Canal, located just downstream of the Airport Expressway. During the wet season, the S26 is operated to maintain the water elevations of (-)0.345 to 0.155 ft-NAVD (1.2 to 1.7 ft-NGVD). **Figure 9-241** shows a map of the entire C6 Watershed, **Figure 9-242** and **Figure 9-243** provide enlarged views of the Doral and Hialeah Gardens regions, respectively.

Figure 9-241. Map of the C6 Watershed

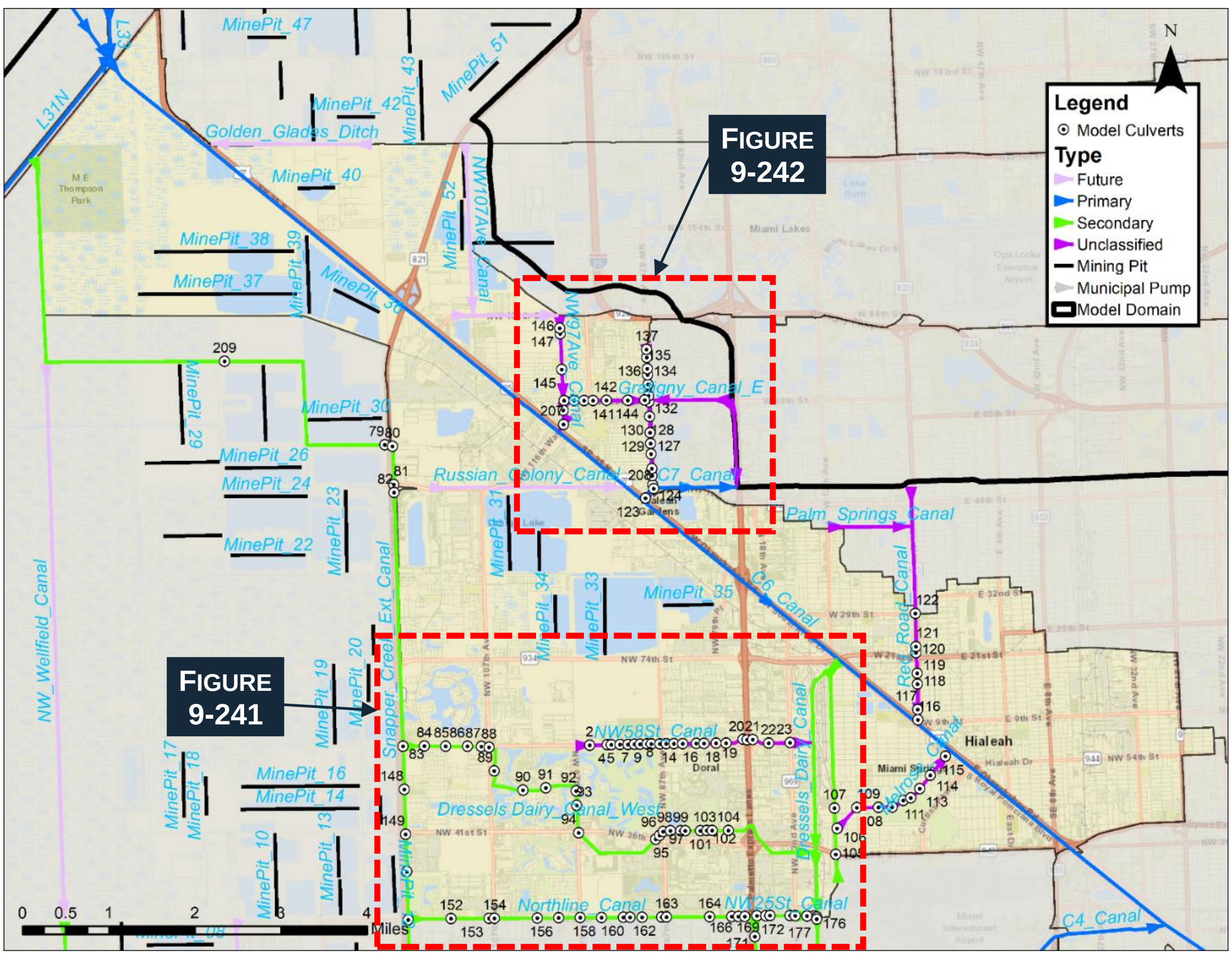


Figure 9-242. Zoomed In Map of C6 Watershed in the Doral Area

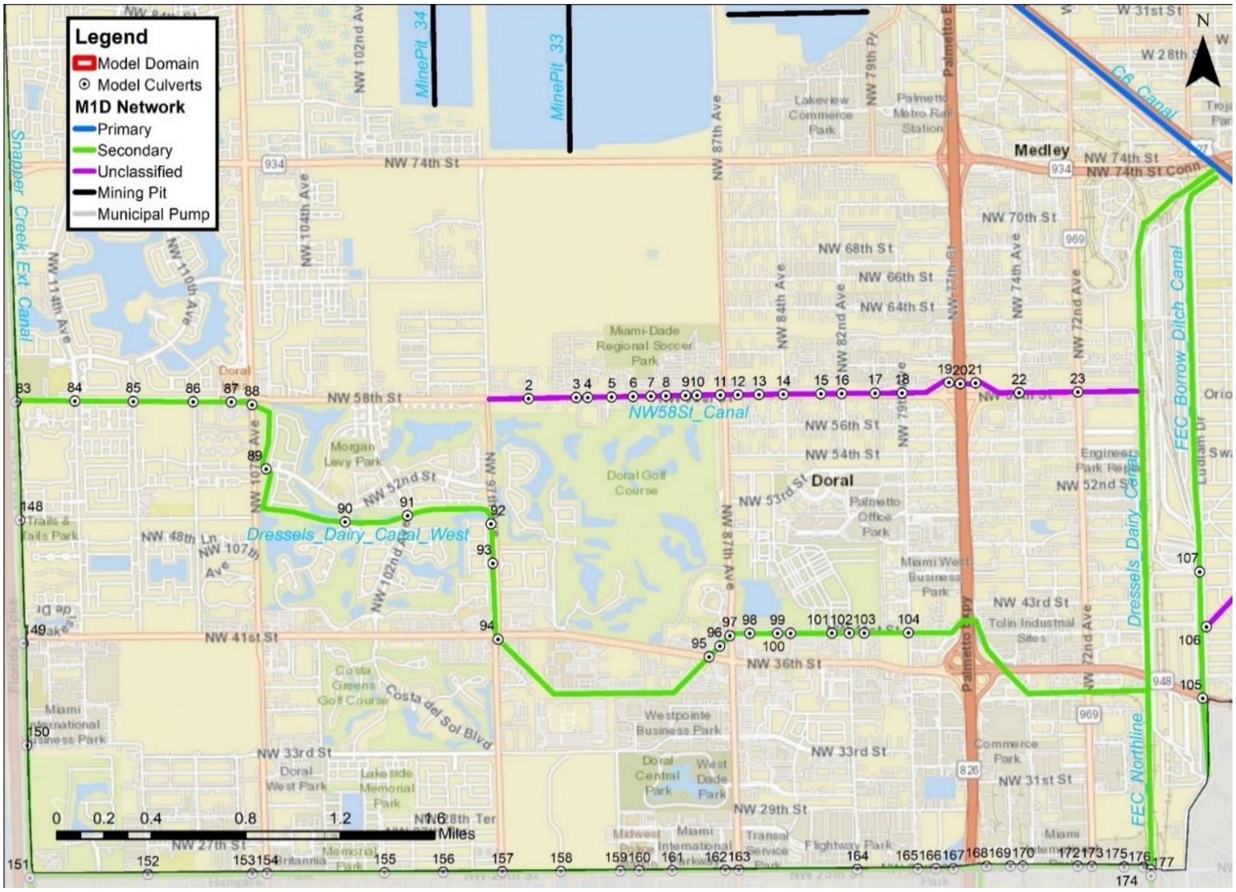
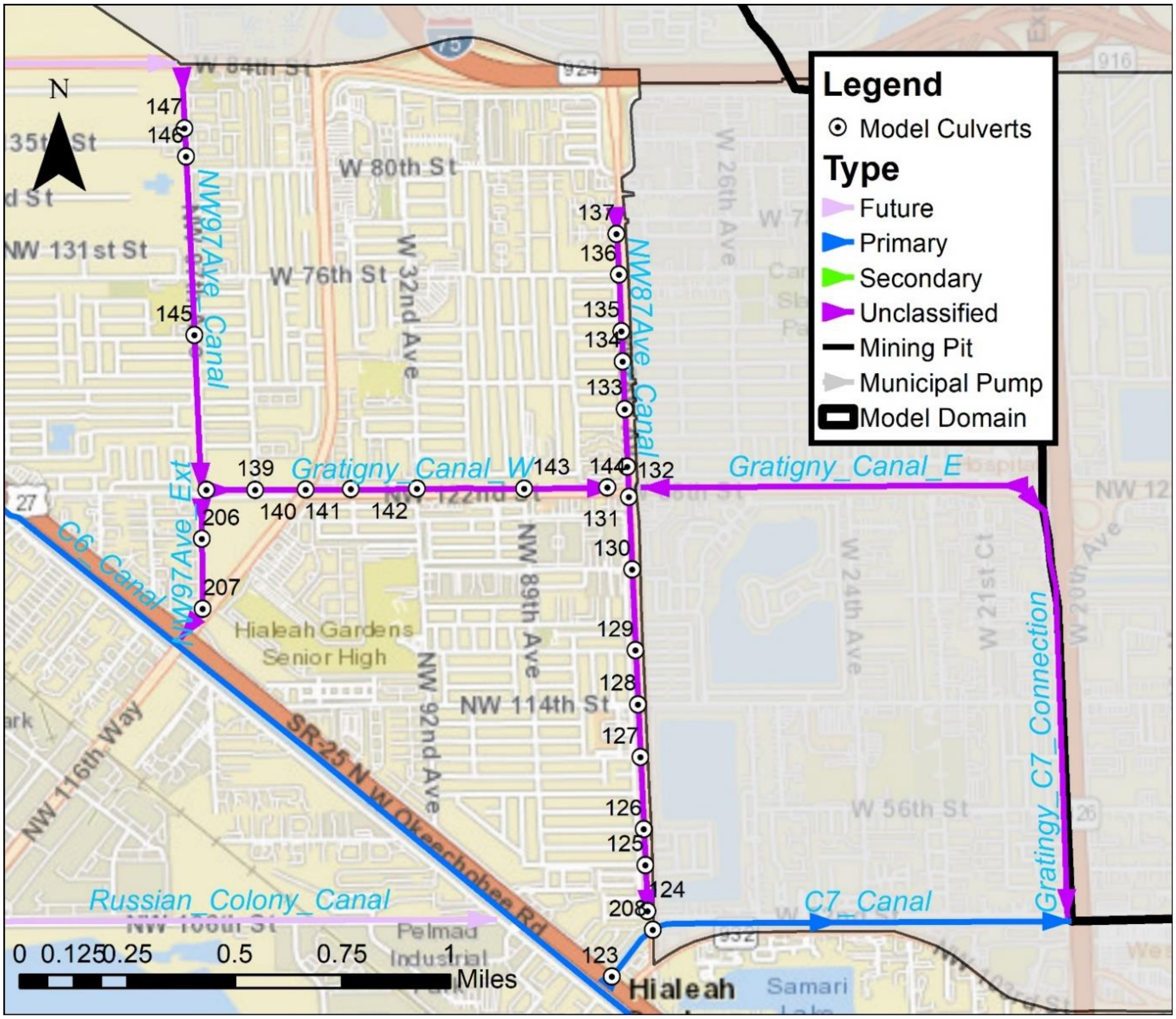


Figure 9-243. Zoomed In Map of the C6 Watershed in the Hialeah Gardens Area



9.5.1. C6 – PM #1 MAXIMUM STAGE IN PRIMARY CANALS

The maximum stage in the primary canals was extracted from the results of each design storm simulation for the C6 Watershed. **Table 9-82** provides the low chord information for each bridge in the C6 or Miami Canal, as well as the peak stage at nearest H-point (where canal stages are calculated in the model). This can be used to establish any issues with bridge low chords. While peak stages do not exceed the low chords in the current conditions simulations for the Miami Canal, two bridge low chords are exceeded for the 100-year storm under SLR +3 feet conditions. These are the Hook Square bridges at Curtiss Parkway and S Hook Square, located in Miami Springs. The Curtiss Parkway bridge is a truss bridge built in 1924 and was widened six feet in 1941, and although the bridge has undergone some upgrades since being built, it is designated as a historic site by the cities of Miami Springs and Hialeah (Suarez, 2022). The S Hook Square bridge was built in 1930 and is also designated as an historic site. In neither instance are the bridges overtopped. The Curtiss Parkway bridge low chord is exceeded during the peak of the storm for approximately 15.5 hours and the S Hook Square bridge low chord is exceeded for 14 hours.

Figure 9-244. Bridges Experiencing Low Chord Exceedance During SLR3 100-year Storm

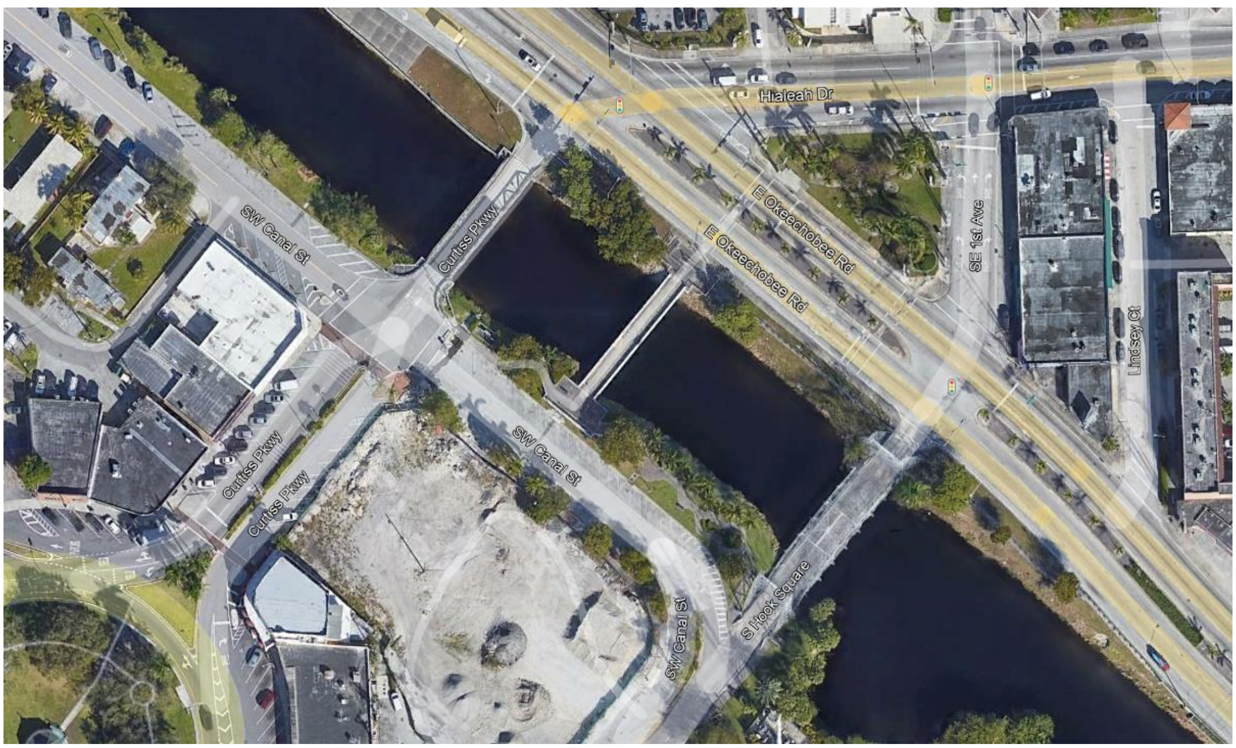


Table 9-83 provides the culvert information with the estimated crown of road from LiDAR data at each culvert location in the C6 Watershed, as well as the peak stage at the nearest H-point. The crown of road was estimated from LiDAR data by determining the elevation at the intersection of the center line of the C6 canal and the top of the roadbed. Stages which overtop the estimated crown of road are highlighted in orange. In addition, a culvert number is provided in the first column, which corresponds to the culverts numbered in **Figure 9-241**, and zoomed in maps provide more details in **Figure 9-242** and **Figure 9-243**. Of the 86 culverts represented in the model for the C6 Watershed, 23 experienced

overtopping during the 100-year design storm and two experienced overtopping during the 25-year design storm for current conditions, while only one culvert experienced overtopping for the 10-year design storm and none for the 5-year design storm. However, the number of overtopped culvert locations increases rapidly with each future SLR condition, as shown in **Figure 9-245**.

Figure 9-245. Number of Culvert Locations Where the Crown of Road is Exceeded within the C6 Watershed

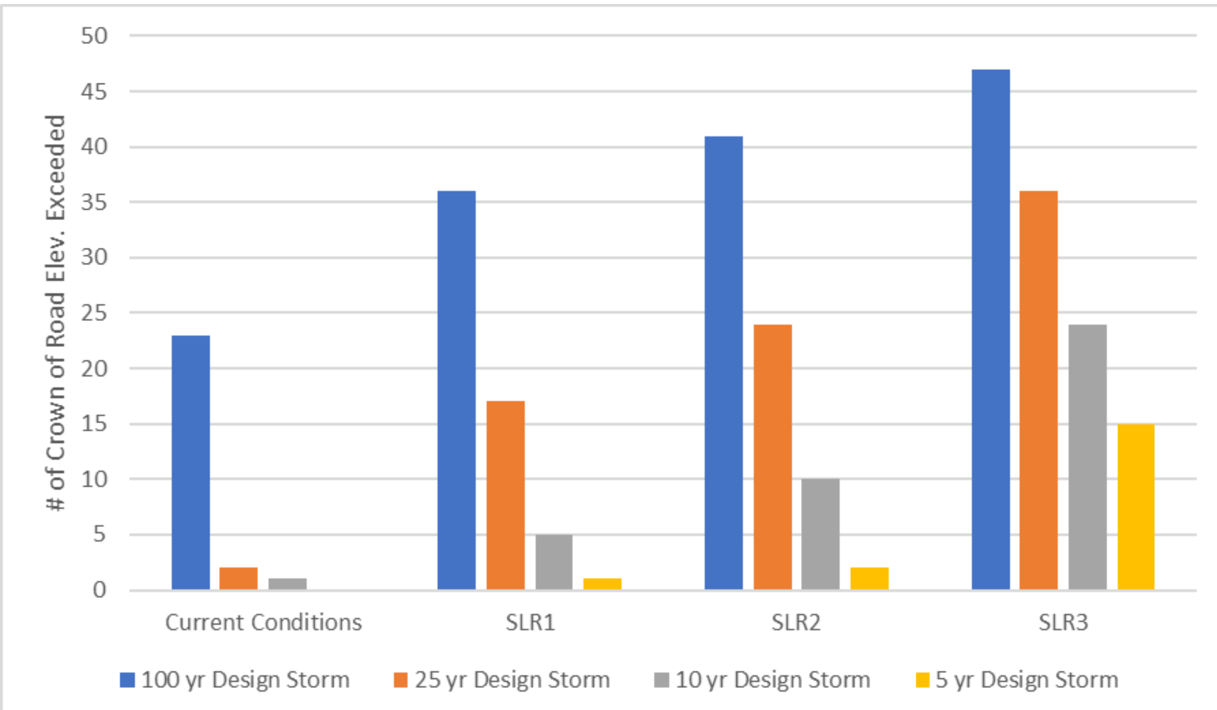


Table 9-82. Bridge Low Chord and Peak Stage for the Miami Canal

Location Description	Low Chord	Bridge Top	Current Conditions				SLR1				SLR2				SLR3			
			Elevation (ft-NAVD)															
			100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Krome Ave (177th Ave)	8.46	11.45	5.15	4.49	4.07	3.77	5.41	4.85	4.48	4.24	5.72	5.20	4.89	4.67	6.02	5.57	5.31	5.14
117th Ave	7.37	9.45	5.14	4.49	4.06	3.76	5.41	4.85	4.48	4.23	5.72	5.20	4.88	4.67	6.02	5.58	5.31	5.14
NW 138th Street	9.17	11.45	5.13	4.48	4.04	3.74	5.42	4.85	4.48	4.22	5.73	5.21	4.89	4.67	6.04	5.59	5.33	5.15
NW 127th Street	8.76	10.75	5.12	4.46	4.02	3.72	5.43	4.85	4.48	4.20	5.74	5.22	4.89	4.67	6.06	5.61	5.34	5.17
Smith Crossing	5.36	7.35	5.11	4.45	4.01	3.70	5.43	4.86	4.47	4.19	5.75	5.23	4.89	4.66	6.07	5.62	5.35	5.18
NW 116th Way	8.86	11.85	5.08	4.41	3.96	3.65	5.45	4.86	4.46	4.16	5.77	5.25	4.90	4.66	6.11	5.65	5.38	5.21
NW 79th Ave	8.17	11.16	5.07	4.38	3.92	3.60	5.46	4.86	4.45	4.14	5.80	5.27	4.91	4.67	6.15	5.68	5.41	5.24
12th Ave	7.46	12.45	5.07	4.37	3.91	3.58	5.46	4.86	4.45	4.14	5.81	5.27	4.91	4.67	6.16	5.70	5.42	5.25
Hialeah Expressway	26.76	29.73	5.06	4.35	3.88	3.56	5.47	4.86	4.44	4.13	5.84	5.29	4.93	4.68	6.19	5.72	5.45	5.27
FEC Railroad	6.43	11.55	5.05	4.33	3.86	3.53	5.48	4.86	4.44	4.12	5.86	5.31	4.94	4.69	6.21	5.74	5.47	5.29
Miami Metrorail	35.06	38.45	5.05	4.33	3.86	3.53	5.48	4.86	4.44	4.12	5.86	5.31	4.94	4.69	6.21	5.74	5.47	5.29
concrete ped. Bridge	23.76	27.35	4.94	4.17	3.69	3.37	5.42	4.77	4.35	4.05	5.85	5.27	4.90	4.65	6.24	5.77	5.48	5.29
Hook Square/Curtiss Pkwy	5.83	8.22	4.89	4.08	3.59	3.29	5.39	4.72	4.30	4.01	5.87	5.25	4.88	4.63	6.29	5.79	5.49	5.31
Hook Square ped. Bridge	6.96	11.75	4.86	4.04	3.54	3.24	5.36	4.68	4.26	3.98	5.90	5.24	4.86	4.62	6.36	5.83	5.52	5.33
S Hook Square	5.96	7.25	4.83	4.00	3.51	3.19	5.34	4.65	4.23	3.95	5.92	5.23	4.84	4.60	6.40	5.85	5.54	5.35
SE 4th Ave	7.26	13.55	4.79	3.93	3.47	3.13	5.32	4.61	4.19	3.91	5.95	5.21	4.82	4.58	6.49	5.89	5.55	5.37
LeJune Road	7.36	12.25	4.74	3.85	3.40	3.07	5.29	4.56	4.13	3.88	5.98	5.20	4.80	4.56	6.60	5.92	5.58	5.39
NW 36th Ave	8.36	14.45	4.63	3.71	3.34	2.97	5.22	4.48	4.01	3.82	6.06	5.16	4.75	4.52	6.80	6.02	5.65	5.45

*Highlighted cells indicate the stages exceed the bridge low chord

Table 9-83. Estimated Culvert Crown of Road and Peak Stage for the C6 Watershed

Culvert Number	Intersection	Top of Culvert	Estimated Crown of Road	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Dressels_Dairy_Canal_West																			
84	NW 114th Ave	3.96	6.6	6.05	5.38	4.81	4.39	6.31	5.74	5.27	4.81	6.45	5.95	5.54	5.21	6.62	6.21	5.91	5.68
85	NW 112th Ave	2.25	5.4	6.04	5.38	4.81	4.39	6.30	5.74	5.26	4.80	6.44	5.94	5.54	5.21	6.62	6.20	5.90	5.68
86	NW 109th Ave	2.07	5.5	6.02	5.36	4.79	4.38	6.27	5.72	5.25	4.79	6.44	5.93	5.53	5.20	6.62	6.18	5.89	5.67
87	Delia Plaza	2.84	6.2	5.99	5.34	4.78	4.37	6.26	5.71	5.24	4.79	6.43	5.91	5.52	5.19	6.61	6.17	5.88	5.65
88	NW 107th Ave	3.09	8.0	5.95	5.32	4.76	4.36	6.25	5.69	5.22	4.78	6.42	5.89	5.50	5.17	6.61	6.17	5.86	5.64
89	NW 52nd St	2.21	6.8	5.90	5.29	4.74	4.35	6.24	5.67	5.21	4.77	6.42	5.87	5.49	5.16	6.60	6.16	5.84	5.63
90	NW 104th Ave	3.37	7.2	5.89	5.27	4.73	4.34	6.23	5.65	5.19	4.76	6.41	5.86	5.47	5.15	6.59	6.15	5.84	5.62
91	NW 102nd Ave	2.27	6.5	5.88	5.25	4.71	4.33	6.22	5.64	5.19	4.75	6.40	5.85	5.47	5.14	6.58	6.15	5.83	5.62
92	NW 49th Ter	2.39	5.4	5.87	5.23	4.70	4.32	6.21	5.63	5.18	4.74	6.39	5.84	5.46	5.13	6.57	6.14	5.82	5.61
93	NW 46th Ln	2.48	5.8	5.84	5.17	4.66	4.30	6.18	5.62	5.15	4.73	6.36	5.82	5.44	5.12	6.55	6.12	5.81	5.60
94	NW 97th Ave	1.27	6.7	5.81	5.12	4.63	4.27	6.16	5.60	5.13	4.71	6.34	5.81	5.43	5.10	6.53	6.10	5.80	5.58
95	NW 36th St	6.13	8.1	5.78	5.09	4.60	4.25	6.13	5.58	5.11	4.69	6.32	5.79	5.41	5.09	6.52	6.08	5.78	5.57
96	private drive	5.94	7.9	5.77	5.08	4.58	4.24	6.12	5.57	5.10	4.68	6.30	5.78	5.40	5.08	6.51	6.07	5.77	5.56
97	NW 87th Ave	5.24	7.6	5.76	5.07	4.57	4.23	6.11	5.56	5.10	4.68	6.30	5.78	5.40	5.08	6.50	6.07	5.77	5.56
98	Leon Medical Ctr 1	5.75	7.0	5.74	5.05	4.55	4.21	6.09	5.54	5.08	4.66	6.28	5.76	5.38	5.06	6.49	6.05	5.76	5.55
99	Leon Medical Ctr 2	6.21	6.8	5.73	5.04	4.54	4.20	6.08	5.53	5.07	4.65	6.27	5.75	5.37	5.06	6.48	6.05	5.75	5.54
100	Leon Medical Ctr 3	7.99	8.8	5.72	5.03	4.53	4.19	6.07	5.52	5.06	4.65	6.26	5.75	5.37	5.05	6.48	6.04	5.74	5.54
101	NW 83rd Ave	7.65	8.1	5.71	5.02	4.51	4.17	6.06	5.51	5.05	4.63	6.25	5.73	5.35	5.04	6.47	6.03	5.74	5.53
102	NW 82nd Ave	6.29	6.8	5.69	5.00	4.49	4.15	6.04	5.49	5.03	4.62	6.24	5.72	5.34	5.03	6.46	6.02	5.73	5.52
103	NW 81st Ave	2.44	4.8	5.63	4.94	4.42	4.07	5.99	5.44	4.98	4.57	6.20	5.69	5.31	5.00	6.43	5.99	5.70	5.49
104	NW 79th Ave	3.69	6.1	5.43	4.74	4.23	3.89	5.81	5.24	4.80	4.43	6.07	5.56	5.18	4.90	6.36	5.90	5.62	5.42

Culvert Number	Intersection	Top of Culvert	Estimated Crown of Road	Current Conditions 100-Year	25-Year	10-Year	5-Year	SLR1 100-Year	25-Year	10-Year	5-Year	SLR2 100-Year	25-Year	10-Year	5-Year	SLR3 100-Year	25-Year	10-Year	5-Year
				Elevation (ft-NAVD)															
FEC_Borrow_Ditch_Canal																			
105	NW 36th St	-0.37	7.1	5.31	4.54	4.03	3.66	5.65	4.99	4.53	4.19	5.97	5.41	5.04	4.76	6.28	5.81	5.53	5.34
107	Rosedale Dr	0.87	6.7	5.20	4.47	3.98	3.62	5.57	4.94	4.49	4.16	5.90	5.36	5.00	4.73	6.23	5.77	5.49	5.31
C7_Canal																			
123	Okeechobee Rd and C7	3.76	7.5	5.08	4.40	3.95	3.64	5.44	4.85	4.45	4.15	5.77	5.25	4.90	4.66	6.11	5.65	5.38	5.21
Gratigny_Canal_W																			
138	NW 97th Ave	1.05	5.7	5.10	4.44	3.98	3.67	5.45	4.84	4.44	4.13	5.75	5.23	4.87	4.63	6.07	5.62	5.35	5.17
140	NW 92nd Ave	2.82	7.5	4.98	4.22	3.75	3.44	5.22	4.35	3.65	3.20	5.54	4.91	4.44	4.06	5.78	5.42	5.10	4.86
141	NW 93rd Ct	1.81	6.0	4.97	4.20	3.73	3.41	5.17	4.27	3.56	3.09	5.51	4.86	4.38	3.99	5.77	5.40	5.07	4.82
142	NW 92nd Ave	2.40	4.9	4.96	4.17	3.69	3.38	5.10	4.18	3.47	3.00	5.48	4.80	4.31	3.93	5.74	5.36	5.03	4.78
143	NW 89th Ave	2.33	5.7	4.93	4.14	3.66	3.34	4.81	3.90	3.24	2.81	5.27	4.57	4.10	3.76	5.63	5.20	4.86	4.65
144	Walgreens Driveway	4.98	6.5	4.78	4.02	3.56	3.25	4.13	3.39	2.89	2.56	4.72	4.09	3.74	3.50	5.34	4.82	4.55	4.40
Melrose_Canal																			
106	NW 67th Ave	-0.88	4.8	5.20	4.47	3.98	3.62	5.57	4.94	4.49	4.16	5.90	5.36	5.00	4.73	6.23	5.77	5.49	5.31
108	Hammond Dr	-1.09	5.8	5.52	4.99	4.50	4.02	5.75	5.34	4.95	4.47	5.94	5.59	5.28	5.00	6.19	5.80	5.61	5.45
109	Lenape Dr	-0.86	6.3	5.65	5.17	4.59	4.05	5.96	5.60	5.17	4.54	6.05	5.75	5.47	5.17	6.18	5.89	5.70	5.56
110	Golf Cart Path 1	1.05	5.7	5.64	5.14	4.57	4.03	6.02	5.61	5.15	4.53	6.14	5.77	5.47	5.16	6.26	5.92	5.72	5.57
111	Golf Cart Path 2	1.05	5.5	5.62	5.11	4.54	4.01	6.04	5.60	5.13	4.51	6.17	5.77	5.46	5.14	6.31	5.94	5.72	5.56
112	Golf Cart Path 3	-0.55	6.2	5.60	5.08	4.52	3.99	6.04	5.58	5.10	4.49	6.17	5.76	5.44	5.12	6.30	5.94	5.70	5.55
113	Deer Run	-0.55	7.2	5.57	5.06	4.48	3.97	6.02	5.55	5.07	4.47	6.15	5.73	5.42	5.09	6.28	5.92	5.68	5.53
114	Westward Dr	-1.55	6.5	5.53	5.01	4.44	3.93	5.92	5.48	5.00	4.44	6.06	5.67	5.37	5.05	6.16	5.88	5.63	5.50
115	N Royal Poinciana Blvd	-1.45	5.4	5.37	4.81	4.26	3.78	5.65	5.23	4.77	4.31	5.82	5.42	5.19	4.91	6.14	5.75	5.54	5.39

Culvert Number	Intersection	Top of Culvert	Estimated Crown of Road	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Red_Road_Canal																			
116	Okeechobee Rd	1.41	6.8	4.94	4.17	3.69	3.37	5.42	4.77	4.35	4.05	5.85	5.27	4.90	4.65	6.24	5.77	5.48	5.29
117	Northside Church	2.09	3.4	4.84	4.14	3.59	3.22	5.23	4.66	4.21	3.89	5.44	5.09	4.74	4.50	5.84	5.40	5.24	5.11
118	W 16th St	1.41	4.7	4.83	4.13	3.57	3.20	5.21	4.65	4.19	3.87	5.42	5.06	4.72	4.48	5.81	5.38	5.22	5.09
119	W 18th St	2.09	4.9	4.70	4.00	3.43	3.05	5.14	4.54	4.06	3.72	5.33	4.99	4.62	4.36	5.65	5.30	5.17	5.02
120	Metrorail	2.22	7.8	4.29	3.64	3.08	2.71	4.81	4.17	3.73	3.37	5.12	4.71	4.36	4.09	5.36	5.13	4.98	4.84
121	W 23rd St	1.57	6.1	3.45	2.81	2.35	2.06	4.10	3.45	3.10	2.79	4.69	4.16	3.86	3.67	5.19	4.84	4.65	4.51
122	W 29th St	1.87	5.5	2.17	1.73	1.52	1.37	3.00	2.50	2.32	2.20	3.89	3.37	3.21	3.13	4.47	4.27	4.16	4.09
NW58St_Canal																			
2	MD Public Safety Training Drive	2.13	6.1	6.12	5.40	4.87	4.49	6.68	6.09	5.55	5.12	6.81	6.26	5.80	5.43	6.95	6.45	6.11	5.83
3	Road, Bridge, and Canal Maint. #1	1.90	6.3	6.12	5.40	4.87	4.49	6.68	6.09	5.55	5.12	6.81	6.26	5.80	5.43	6.95	6.45	6.11	5.83
4	Road, Bridge, and Canal Maint. #2	1.67	5.8	6.11	5.39	4.87	4.48	6.67	6.08	5.54	5.10	6.80	6.26	5.80	5.42	6.94	6.45	6.11	5.82
5	SFWMD Drive #1	2.52	5.8	6.11	5.39	4.86	4.47	6.67	6.08	5.54	5.09	6.79	6.26	5.79	5.42	6.93	6.44	6.10	5.82
6	SFWMD Drive #2	2.38	5.4	6.10	5.38	4.84	4.44	6.66	6.07	5.53	5.08	6.78	6.25	5.79	5.41	6.92	6.44	6.10	5.82
7	Gated Parking Lot	1.47	5.4	6.09	5.37	4.82	4.42	6.64	6.06	5.52	5.07	6.76	6.24	5.78	5.40	6.90	6.42	6.09	5.81
8	NW 89th Ave	2.28	5.9	6.07	5.35	4.81	4.40	6.62	6.04	5.49	5.05	6.74	6.21	5.76	5.38	6.88	6.41	6.07	5.80
9	MD Solid Waste 1	1.96	6.3	6.05	5.32	4.78	4.38	6.59	6.00	5.46	5.02	6.71	6.18	5.73	5.35	6.85	6.38	6.05	5.78
10	MD Solid Waste 2	2.96	6.2	6.02	5.29	4.75	4.35	6.54	5.95	5.42	4.99	6.66	6.14	5.70	5.32	6.81	6.35	6.02	5.75
11	NW 87th Ave	1.79	5.4	6.00	5.27	4.72	4.33	6.50	5.92	5.39	4.96	6.63	6.11	5.67	5.30	6.77	6.32	5.99	5.73
12	Driveway	2.99	5.5	5.90	5.18	4.63	4.26	6.31	5.72	5.21	4.82	6.45	5.95	5.54	5.20	6.62	6.19	5.89	5.65
13	Power Corp Drive	1.90	5.7	5.88	5.16	4.62	4.25	6.29	5.70	5.19	4.81	6.44	5.94	5.53	5.19	6.60	6.18	5.87	5.64
14	NW 84th Ave	1.58	5.4	5.81	5.10	4.57	4.19	6.17	5.59	5.09	4.70	6.34	5.84	5.45	5.12	6.52	6.10	5.81	5.59

Culvert Number	Intersection	Top of Culvert	Estimated Crown of Road	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
15	Kelly Tractor Drive	2.81	5.5	5.69	5.06	4.51	4.09	6.00	5.43	4.98	4.59	6.21	5.71	5.34	5.04	6.43	6.00	5.72	5.52
16	NW 82nd Ave	3.37	5.7	5.68	5.04	4.50	4.08	5.97	5.41	4.96	4.58	6.19	5.69	5.32	5.03	6.42	5.99	5.71	5.51
17	Tractor Rental Dr	1.70	5.2	5.66	5.03	4.48	4.06	5.96	5.40	4.94	4.56	6.17	5.68	5.31	5.02	6.41	5.98	5.70	5.50
18	NW 79th Ave	2.12	5.7	5.62	5.00	4.45	4.03	5.91	5.36	4.91	4.53	6.14	5.64	5.28	4.99	6.39	5.95	5.67	5.47
19	826 SB Exit Ramp	2.78	16.4	5.58	4.97	4.43	4.00	5.87	5.32	4.88	4.50	6.11	5.61	5.25	4.97	6.37	5.93	5.65	5.45
20	Palmetto Exp (826)	-0.78	8.2	5.56	4.95	4.41	3.99	5.86	5.30	4.86	4.48	6.10	5.60	5.23	4.95	6.36	5.91	5.64	5.44
21	826 NB Entrance Ramp	2.64	14.4	5.56	4.95	4.41	3.99	5.86	5.30	4.86	4.48	6.10	5.60	5.23	4.95	6.36	5.91	5.64	5.44
22	NW 74th Ave	0.63	6.9	5.49	4.87	4.33	3.92	5.79	5.24	4.80	4.43	6.05	5.54	5.18	4.91	6.33	5.88	5.60	5.41
23	NW 72nd Ave	1.02	7.0	5.22	4.53	4.03	3.69	5.62	5.01	4.59	4.26	5.94	5.41	5.04	4.79	6.27	5.80	5.53	5.35
NW87Ave_Canal																			
124	NW 106th Ter	2.29	6.0	4.98	4.34	3.90	3.60	5.28	4.70	4.31	4.01	5.57	5.08	4.75	4.53	5.88	5.48	5.23	5.07
125	NW 108th St	4.77	5.9	4.97	4.33	3.89	3.59	5.27	4.68	4.29	4.00	5.56	5.06	4.73	4.51	5.87	5.46	5.22	5.06
126	NW 109th Ter	2.18	6.0	4.94	4.29	3.85	3.56	5.24	4.62	4.18	3.88	5.52	5.02	4.65	4.42	5.80	5.41	5.16	4.99
127	NW 112th St	1.74	5.1	4.87	4.20	3.75	3.49	5.13	4.51	3.92	3.61	5.43	4.91	4.55	4.22	5.75	5.34	5.08	4.87
128	NW 114th St	1.35	5.2	4.83	4.14	3.66	3.40	5.07	4.39	3.70	3.33	5.40	4.82	4.45	4.07	5.73	5.30	5.02	4.79
129	NW 116th St	2.55	4.8	4.80	4.09	3.60	3.34	5.03	4.30	3.59	3.15	5.37	4.76	4.37	4.00	5.70	5.27	4.98	4.75
130	NW 119th St	2.51	5.3	4.66	3.90	3.46	3.20	4.69	3.87	3.23	2.81	5.09	4.50	4.07	3.75	5.49	5.05	4.78	4.59
131	W 68th St	0.41	6.4	4.31	3.70	3.37	3.18	4.05	3.33	2.85	2.53	4.66	4.04	3.70	3.48	5.29	4.78	4.52	4.37
132	Walgreens Drwy	5.64	6.1	4.34	3.72	3.39	3.19	4.08	3.36	2.87	2.54	4.68	4.06	3.72	3.49	5.31	4.80	4.54	4.38
133	W 71st St	2.76	5.3	4.57	3.86	3.50	3.25	4.37	3.52	2.97	2.60	4.94	4.20	3.82	3.57	5.43	4.94	4.63	4.45
134	W 72nd Pl	2.43	5.2	4.66	3.90	3.55	3.28	4.49	3.60	3.02	2.63	5.05	4.25	3.85	3.61	5.49	5.00	4.66	4.48
135	W 74th St	1.90	4.9	4.83	4.02	3.68	3.36	4.79	3.81	3.16	2.70	5.27	4.40	3.96	3.72	5.65	5.16	4.76	4.55
136	W 76th St	1.71	5.2	4.85	4.03	3.70	3.38	4.82	3.83	3.19	2.72	5.29	4.41	3.97	3.74	5.68	5.18	4.78	4.56

Culvert Number	Intersection	Top of Culvert	Estimated Crown of Road	Current Conditions				SLR1				SLR2				SLR3			
				Elevation (ft-NAVD)															
				100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
137	Martha's Apts Driveway	1.52	5.9	4.86	4.04	3.72	3.39	4.86	3.86	3.21	2.74	5.30	4.43	3.99	3.76	5.71	5.20	4.79	4.57
NW97Ave_Canal																			
145	NW 130th St	1.70	5.7	5.18	4.54	4.06	3.71	5.74	5.07	4.60	4.21	5.95	5.42	4.98	4.71	6.17	5.73	5.44	5.23
146	13600 NW 97th Ave Private Dr	2.79	5.0	5.18	4.55	4.08	3.73	5.82	5.12	4.65	4.24	6.03	5.46	5.01	4.74	6.23	5.78	5.47	5.25
147	13700 NW 97th Ave Private Dr	2.00	5.6	5.18	4.56	4.09	3.74	5.87	5.15	4.67	4.26	6.09	5.48	5.03	4.76	6.31	5.81	5.49	5.26
NW97Ave_Ext																			
206	NW 122nd St	4.52	6.4	5.10	4.44	3.98	3.67	5.45	4.84	4.44	4.13	5.75	5.23	4.87	4.63	6.07	5.62	5.35	5.17
207	NW 97th Ave	5.16	7.6	5.10	4.43	3.98	3.67	5.44	4.85	4.45	4.15	5.75	5.24	4.88	4.64	6.08	5.63	5.36	5.19

*Highlighted cells indicate the stages exceed the estimated crown of road.

In the Doral area, the Dressels Dairy Canal West experiences crown of road overtopping for the 100-year event at two culvert locations closer to the Turnpike where it runs along NW 58th Street at culverts 85 and 86, where LiDAR data indicates the highest road elevation at these bridges is almost six (6) inches lower than the peak of the 100-year storm. Additionally, entrance roads into The Greens at Doral at NW 49th Terrace and NW 46th Lane experience overtopping for the 100-year storm. The canal then experiences overtopping of the road at the NW 81st Avenue crossing (culvert 103) for the 100-year and 25-year storms. With future SLR conditions, overtopping at these locations increases. This reduces the minimum passing storm with each foot of SLR, such that the culverts are overtopped for even the 5-year storm under SLR3 conditions.

The NW 58th Street Canal experiences overtopping extensively along the culvert intersections for the 100-year storm event under current conditions. However, many of these locations will experience overtopping during the SLR3 condition for even a 5-year storm.

The maximum stage profiles for the C6 Watershed are provided in **Appendix A**, with bank elevations and major intersections located along the canals. The 100-year storm events for each profile are provided below in **Figure 9-246** through **Figure 9-249** with bank elevations and major intersections located along the canal. This shows how the water begins to stack up just upstream of the structure as tidal elevations increase. For the SLR2 and SLR3 simulations, the max water levels downstream of the S26 structure exceed the max water levels upstream, as tidal conditions reverse the flows into the watershed.

As shown in the maximum stage profile in **Figure 9-246**, the top of bank is overtopped in several locations along the C6 Canal (Miami Canal). Wetland areas upstream of the FL Turnpike will often have low embankments that are overtopped for the 100- and 25-year design storm events. Two areas between NW 107th Avenue and NW 79th Avenue overtop during the 100-year design storm, which is exacerbated by SLR conditions, indicating the low spots along NW South River Drive; however, recent development may have raised the land elevation in this area. One low spot between the FEC Railroad and Hook Square bridge near Lake Louise on N Royal Poinciana Blvd is also overtopped during the 100-year design storm.

In addition, the canal near the S26 structure experiences overtopping during the 100- and 25-year design storms, with a top of bank elevation of 3.96 ft-NAVD (5.5 ft-NGVD) on the north side, as documented in the structure handbook. This was also confirmed with cross-section information from S26 and S26_P as-built drawings, as shown in **Figure 9-250**. This elevation is exceeded at this location for all storm events under all SLR conditions, with the exception of the 5-year storm for SLR1 conditions.

A resident complaint from the May 2020 flood event pinpointed NW 58th Street Canal as experiencing flooding: "The Canal on NW 58th Street west of the palmetto expressway in Miami Dade was overflowing." Further review of canal embankments in the NW 58th Street Canal and the maximum stages along the canal profile was performed, as shown in **Figure 9-248**. Almost the entire canal may experience overtopping during the 100-year design storm. The NW 58th Street Canal experiences road overtopping at twelve culvert locations during the 100-year design storm. In addition, there are low embankments between the SFWMD Building Driveway (Culvert #5) and NW 89th Avenue (Culvert #8) that show overtopping for all design storms. These conditions are exacerbated by sea level rise, with the canal and culvert locations experiencing flooding for less intense storms as the sea level increases.

Canal top of bank was established in the model from cross-section information from previous modeling efforts, including the C-6 HEC-RAS model (SFWMD, 2020), and converted to ft-NAVD. Full discussion of the development of these cross-sections is provided in **Section 3.3.1**. Top of bank was extracted from the model, and the miles of the evaluated canal segment that is overtopped by the design storm was estimated upstream of the tidal control structure S26, as summarized in **Table 9-84**

Table 9-84. Estimated Percentage and Miles of Bank Overtopped per Design Storm

WATERSHED	SIMULATION	100-YEAR	25-YEAR	10-YEAR	5-YEAR
C6 (%)	Current Conditions	29%	5%	0%	0%
	SLR1	45%	18%	4%	2%
	SLR2	50%	32%	19%	11%
	SLR3	56%	47%	43%	35%
C6 (miles)	Current Conditions	4.2	0.7	0.0	0.0
	SLR1	6.5	2.7	0.6	0.2
	SLR2	7.2	4.7	2.8	1.6
	SLR3	8.1	6.8	6.3	5.0

Figure 9-246. Maximum Stage Profile for the C6 Canal for Current Conditions

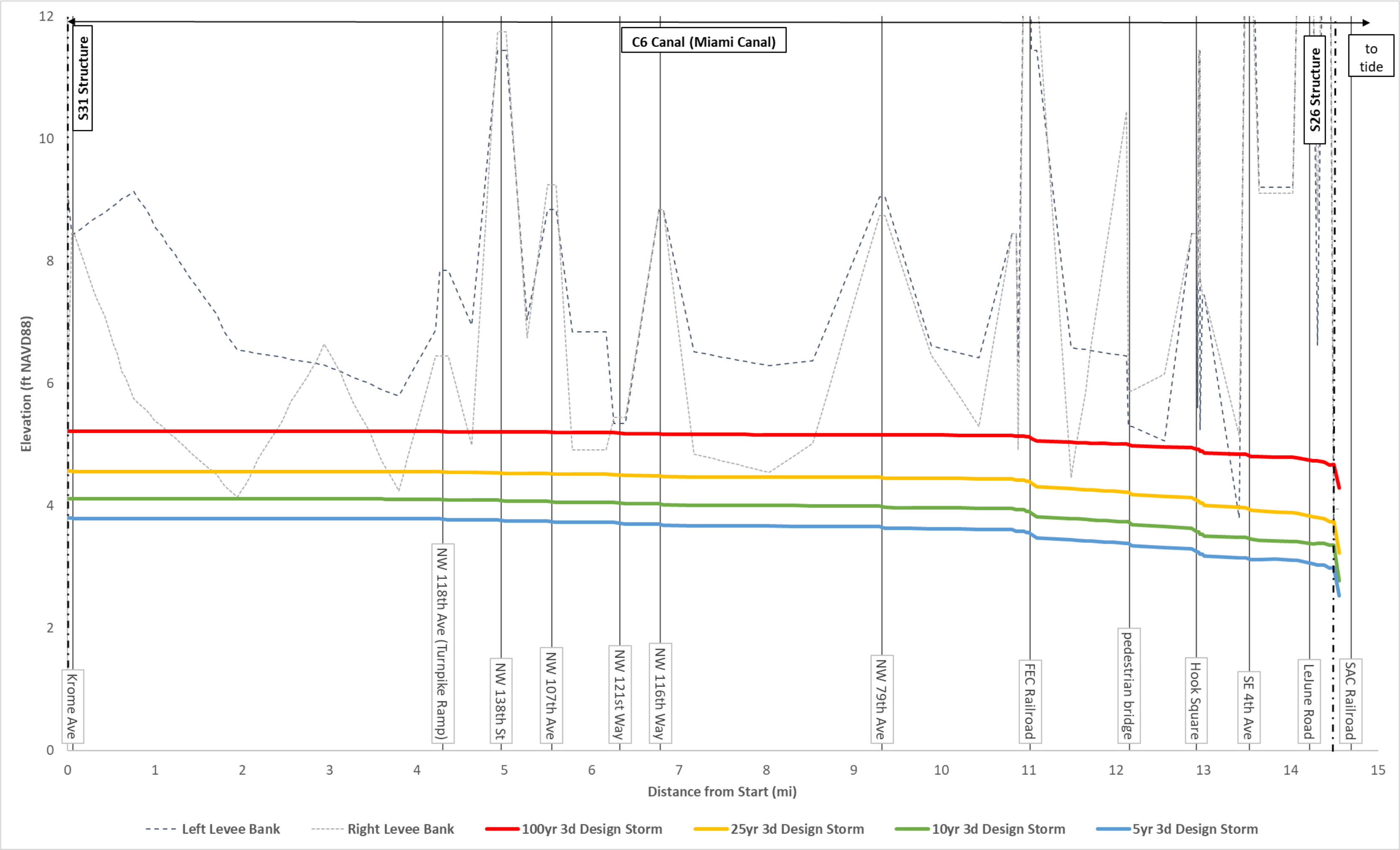


Figure 9-247. Maximum Stage Profile for the C6 Canal for the 100-year Design Storm

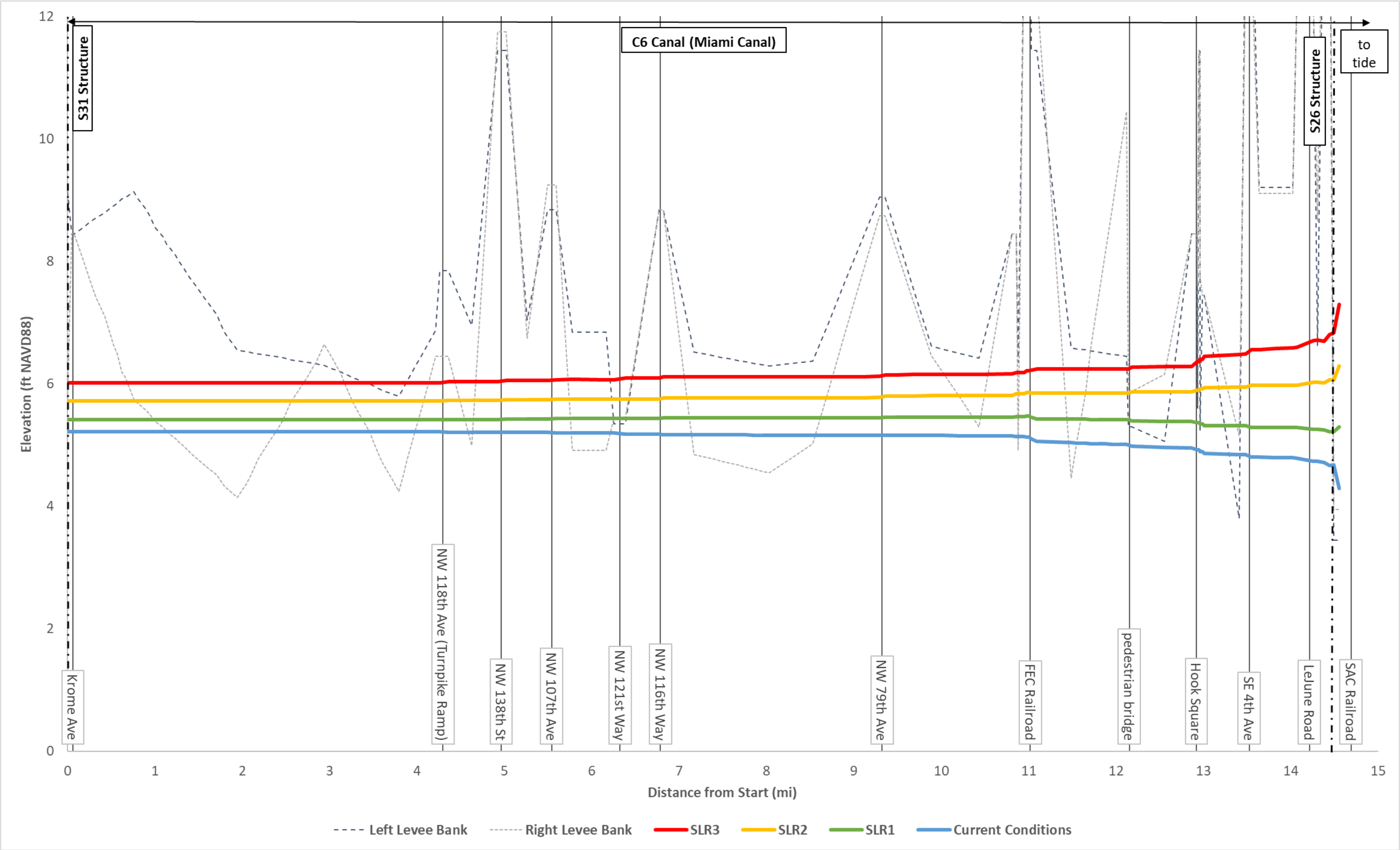


Figure 9-248. Maximum Stage Profile for the NW 58th St Canal and the C6 Canal for the Current Conditions

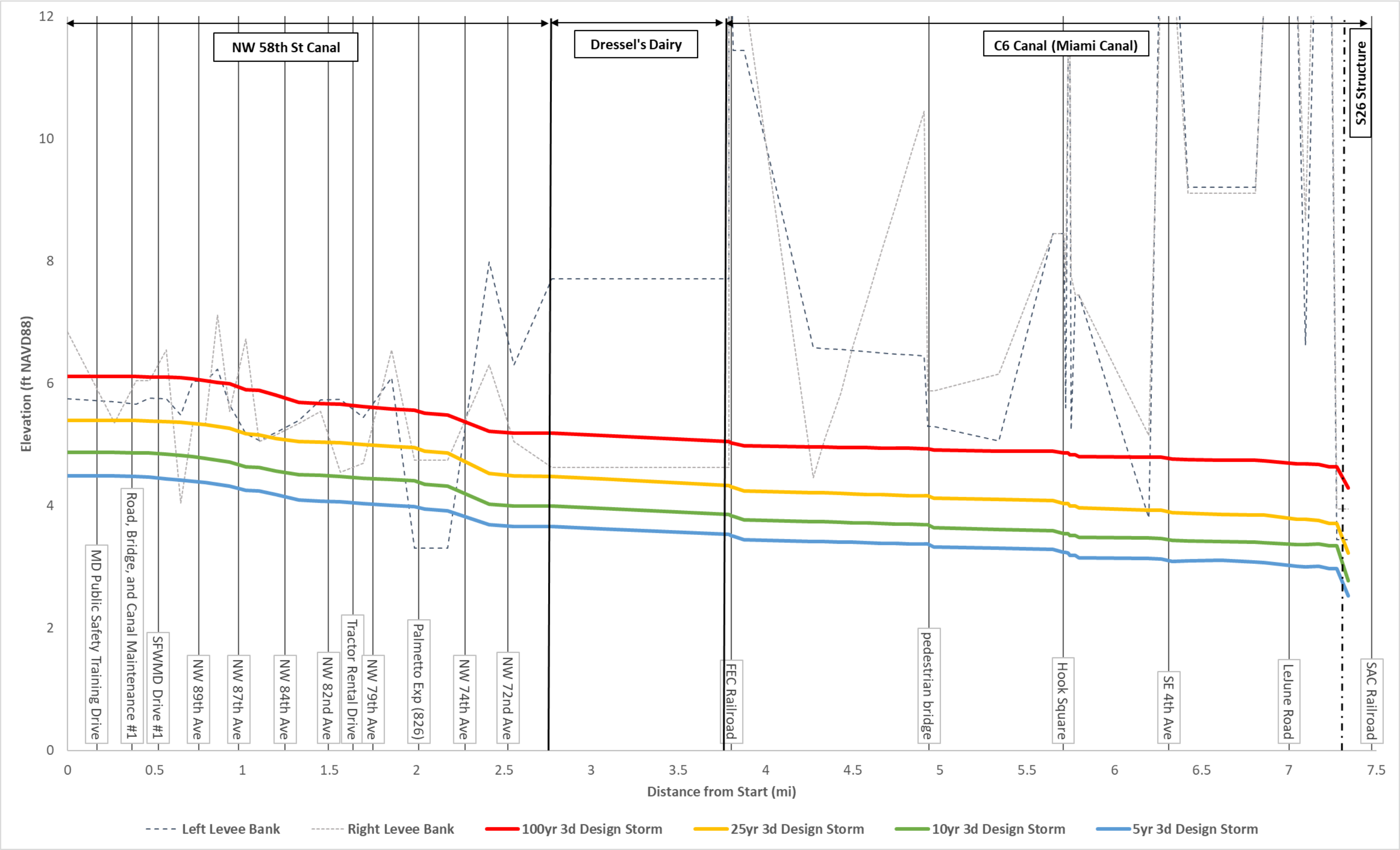


Figure 9-249. Maximum Stage Profile for the NW 58th St Canal and the C6 Canal for the 100-year Design Storm

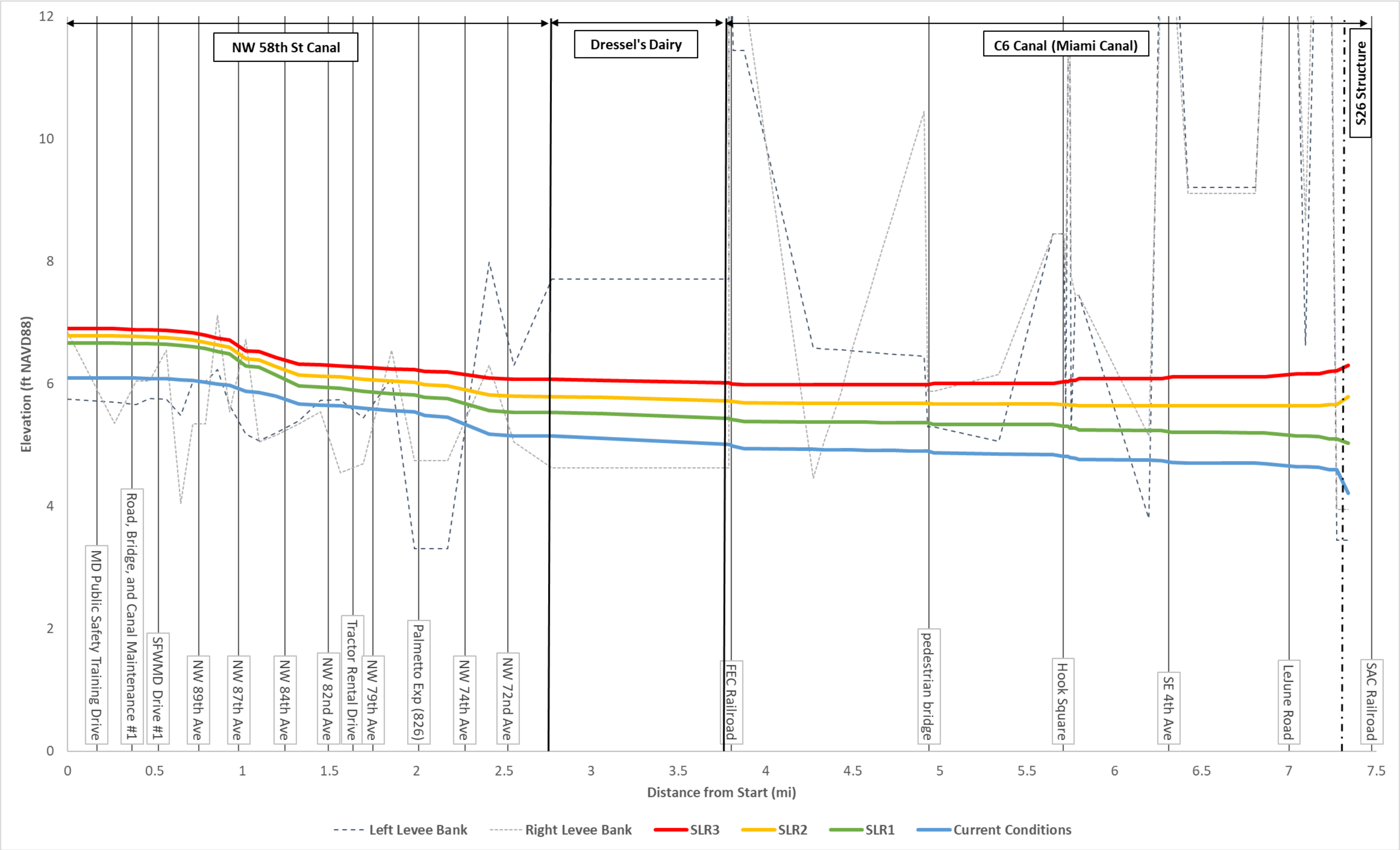
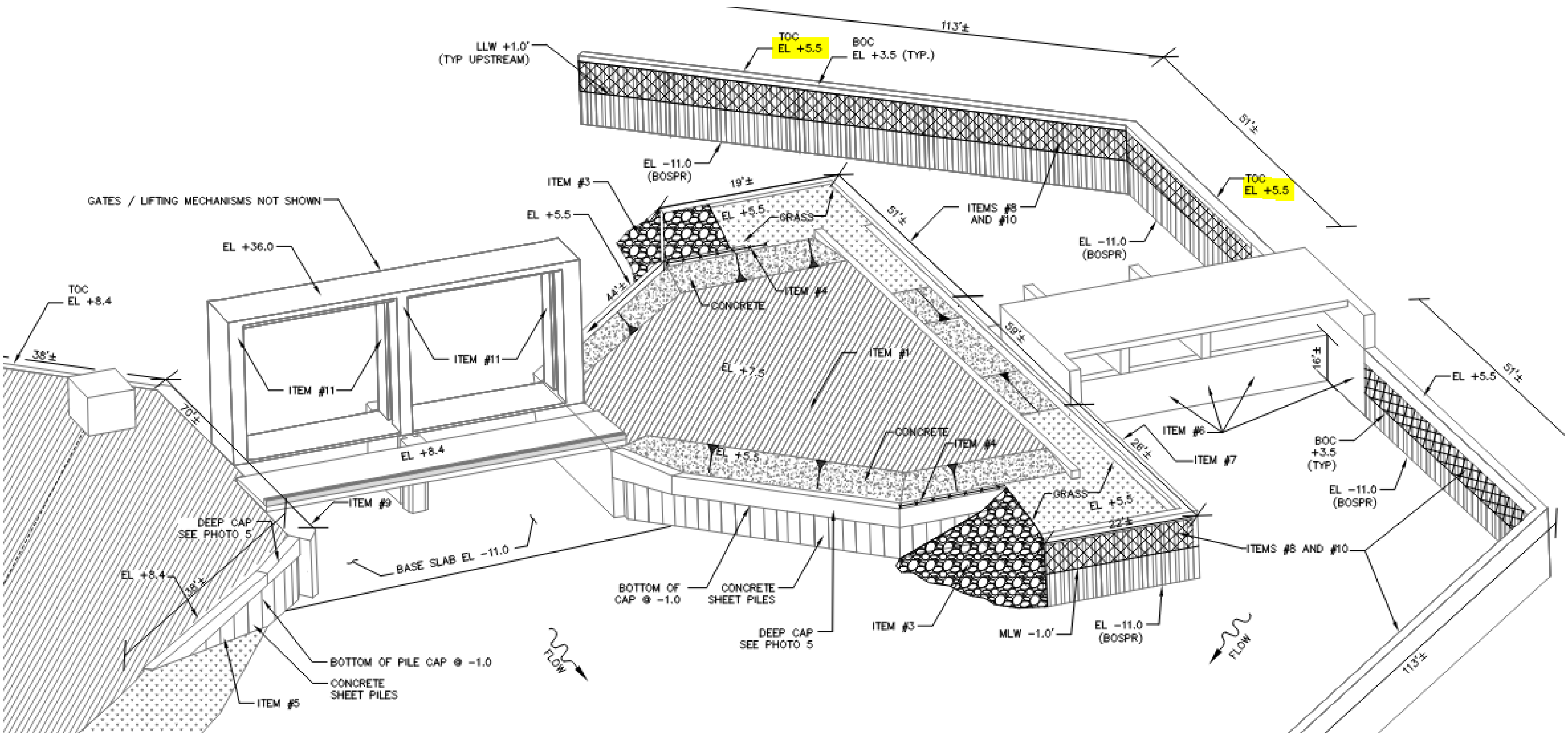


Figure 9-250. As-built Drawing of the S26 Pump Station

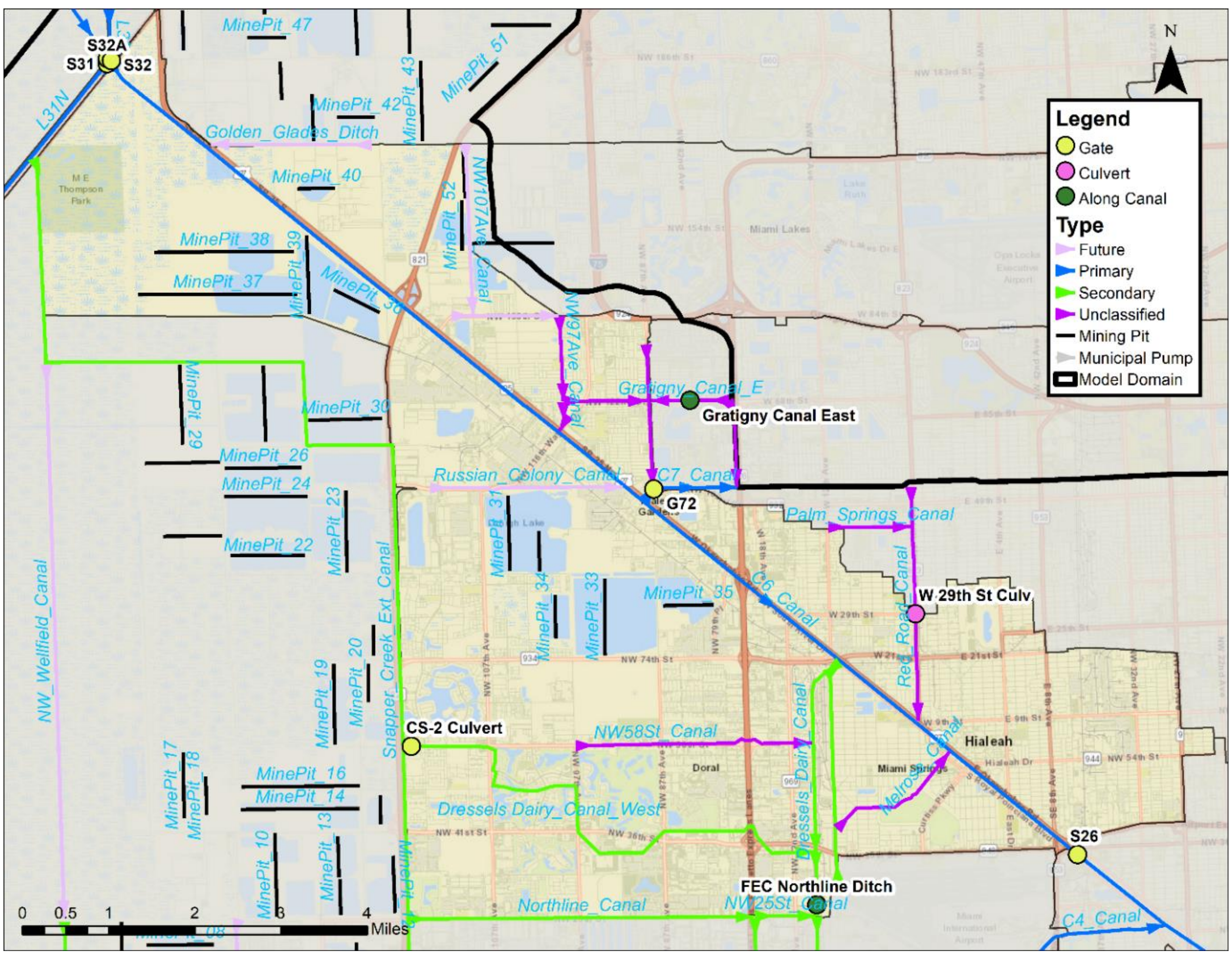


*Elevations in ft-NGVD29

9.5.2. C6 – PM #2 MAXIMUM DISCHARGE CAPACITY

The maximum discharge capacity for the C6 Watershed is the sum of the discharges out of the watershed minus the incoming flows weighted by the total area of the watershed. For the C6 Watershed this means flows from the WCA 3B (via S31, S32, and S32A), Red Road Canal, and Gratigny Canal are subtracted from the outflows at the S26 structure and flows leaving the basin at G72 in the C7 Canal and the FEC Northline canal to the airport, these locations are shown in **Figure 9-251**. In the figure, green dots indicate areas where there are no structures in the canal and pink dots indicate culverts with no gate controls, yet flows are crossing into the watershed at these locations and must be considered as part of the water budgeting.

Figure 9-251. Inflow/Outflow Locations in the C6 Watershed



No flows cross from the C4 Watershed via the Dressels Dairy Canal West as the DERM CS-2 culvert is considered closed during design storm simulations, as are S31, S32, S32A, and G72 (however, G72 still experiences flows due to overtopping).

The timeseries at each inflow and outflow location was extracted, and inflows were subtracted from the outflows and divided by the basin area in square miles (sq. mi.) at each timestep as shown in the equation below.

$$\frac{Outflows[cfs] - Inflows[cfs]}{Basin\ Area\ [sq.mi.]} = Discharge\ Capacity[CSM]$$

The maximum discharge capacity for the C6 Watershed for each design event is shown in **Table 9-85**. The table also provides discharge in cubic feet per second (CFS) at inflow and outflow locations for the watershed. Discharges are not the peak discharge at each location, but rather the discharge at the peak watershed discharge capacity. A negative value at an inflow location means that the flow is leaving the watershed at the peak of the watershed discharge. A negative value at an outflow location means that the flow is entering the watershed at the peak of the watershed discharge.

The ERP Applicant's Handbook Volume II does not indicate an allowable discharge upstream of S26 for the C6 Watershed.

The discharge capacity is reduced for each future SLR condition; however, the SLR1 condition is only slightly less than the current conditions for each design storm, indicating that a foot of sea level rise will not greatly impact the discharge capacity of the watershed. The diminished capacity for the SLR2 and SLR3 conditions is directly related to the watershed's inability to discharge to tide at the S26 structure due to higher tidal conditions, as is further explored in PM #3.

Figure 9-252 shows the instantaneous discharge capacity in CSM for the C6 Watershed, for each SLR condition simulation for the 100- year, 25- year, 10- year, and 5-year design storm events. To remove the tidal influence, **Figure 9-253** shows the 12-hour moving average discharge capacity in CSM for all SLR simulations and each design storm event. This shows that the peak for the SLR3 condition for the higher storm events shifts to after the peak storm surge occurs, when the S26 structure can begin discharging again.

Table 9-85. Peak Discharge (CFS/sq.mi.) from the Contributing Drainage Area of the C6 Watershed

	Current Conditions				SLR1				SLR2				SLR3			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
Inflow Locations																
S31 Total Flow (CFS)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
S32 Total Flow (CFS)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
S32A Total Flow (CFS)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
CS-2 Culvert in Dressels Dairy Canal (CFS)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Red Road Canal at W 29th St (CFS)	-171.0	-145.0	-135.5	-130.7	-165.1	-140.6	-126.2	-119.8	-152.5	-130.2	-114.9	-106.5	-118.8	-116.8	-101.8	-94.9
Gratigny Canal at W 24th Ave (CFS)	-300.0	-223.9	-190.4	-169.3	-355.7	-309.2	-277.9	-261.1	-344.2	-287.8	-255.7	-237.1	-255.0	-269.2	-230.1	-215.1
Outflow Locations																
FEC Northline Canal upstream of NW 25th St (CFS)	-133.3	-73.3	-72.1	-71.9	-158.6	-105.5	-86.2	-71.0	-273.5	-188.4	-143.1	-109.5	-257.5	-274.4	-213.6	-178.5
G72 Total Flow (CFS)	125.8	49.9	26.8	15.9	171.5	101.1	62.1	45.2	227.7	158.2	117.6	96.8	227.4	223.3	184.9	168.8
S26 Total Flow (CFS)	2088.6	1704.3	1500.4	1359.0	2101.1	1645.5	1416.2	1268.9	1850.3	1511.7	1281.0	1123.8	1687.2	1268.1	1127.6	944.9
Watershed Summary																
Basin Area (Square Mile)	53.4				53.4				53.4				53.4			
Peak Watershed Discharge (CSM)	47.8	38.4	33.3	30.0	49.3	39.1	33.6	30.4	43.1	35.6	30.4	27.2	38.0	30.0	26.8	23.3

Figure 9-252. Instantaneous Discharge Capacity for the C6 Watershed for 100- year, 25- year, 10- year, and 5-year 3-day Design Storm Events

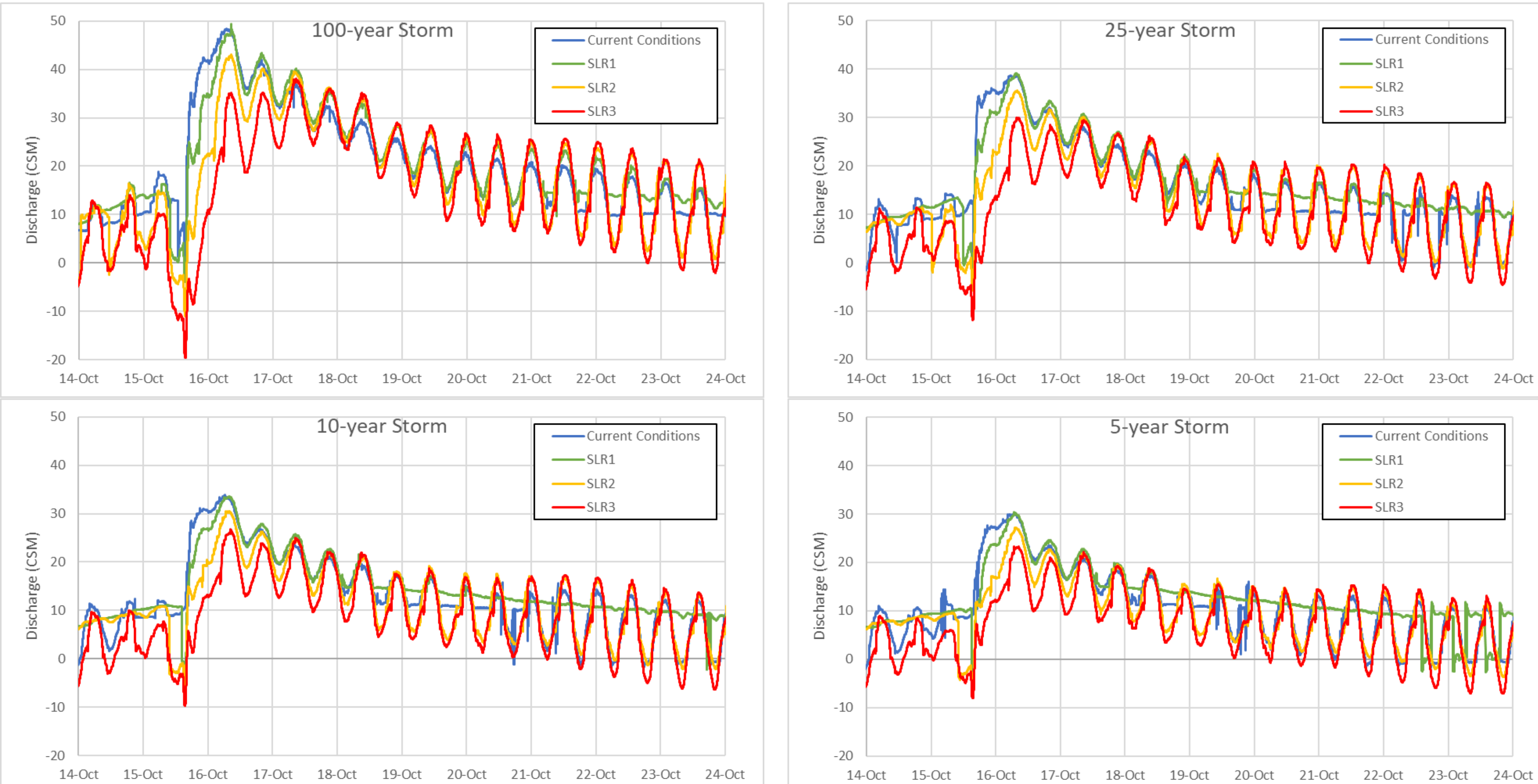
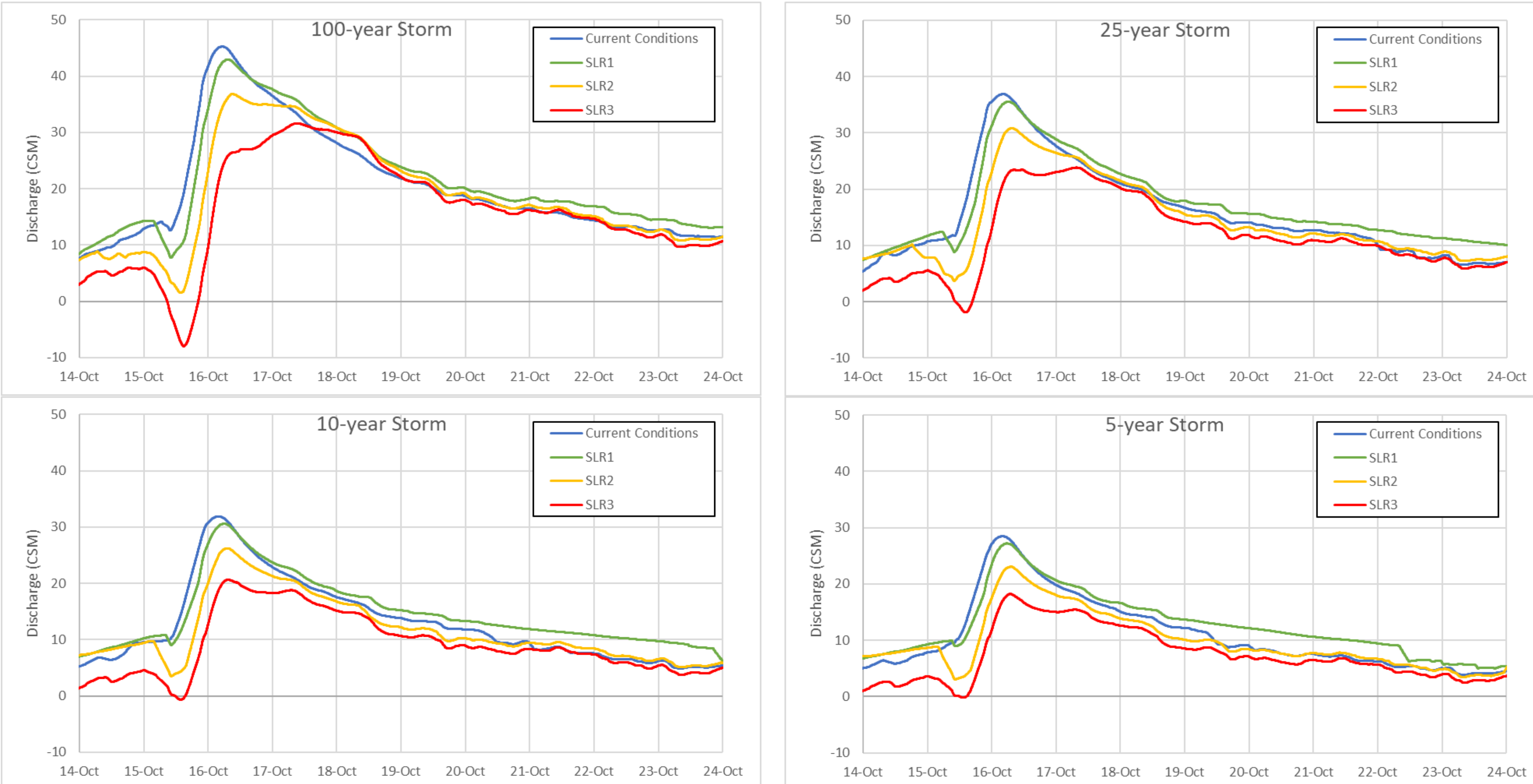


Figure 9-253. 12-Hour Moving Average Discharge Capacity for the C6 Watershed for 100- year, 25- year, 10- year, and 5-year 3-day Design Storm Events



9.5.2.1. INTER-BASIN TRANSFERS

Discharge leaving the C6 Watershed exchanges with the C7 Watershed and the C4 Watershed. **Figure 9-254** shows the flows at G72 structure (positive flows are leaving C6 Watershed), in Red Road Canal at W 29th St (negative flows are leaving C6 Watershed), and in Gratigny Canal East at W 24th Avenue (negative flows are leaving C6 Watershed) for the 100-year design storm. **Figure 9-255** shows these flows for the 25-year design storm. All of these flows represent exchanges to the C7 Watershed. Since the model is established with static boundary conditions, or conditions that do not change over the course of the simulation (see **Table 8-5** for full list of boundary conditions), at these locations, the exchange is largest during the peak of the storm when the water levels in the C6 exceed the boundary conditions.

Flows out of the watershed at G72 tend to increase with future SLR conditions, flows out of the basin at Red Road Canal tend to decrease with increasing SLR conditions, and flows out of the watershed at Gratigny Canal East tend to hover around the same value with each condition.

It should be noted that the flows at G72 represent overtopping of the structure, and not gate flows, as this structure is closed during these simulations.

Flows between the C6 and C4 Watershed are primarily at the FEC Northline Canal as shown in blue in **Figure 9-256** for the 100-year design storm and in **Figure 9-257** for the 25-year design storm. This is because the CS-2 structure in Dressels Dairy Canal West is closed in the wet season, so flows do not connect from the Snapper Creek Extension to the C6 Watershed. Flows at FEC Northline remain negative for most of the storm event (which indicates water is moving north from the C4 Watershed to the C6 Watershed). The 25-year simulation, **Figure 9-257**, indicates that flows return to near zero or even to positive after the storm subsides (which indicates water flowing south to the C4 Watershed). For the future SLR conditions, the maximum flow into the C6 Watershed do not change significantly; however, the timing and duration of these flow changes, with flows into the C6 lasting longer with each increasing SLR condition.

Figure 9-254. Exchanges between the C6 and C7 Watershed for the 100-year Design Storm for all Simulations

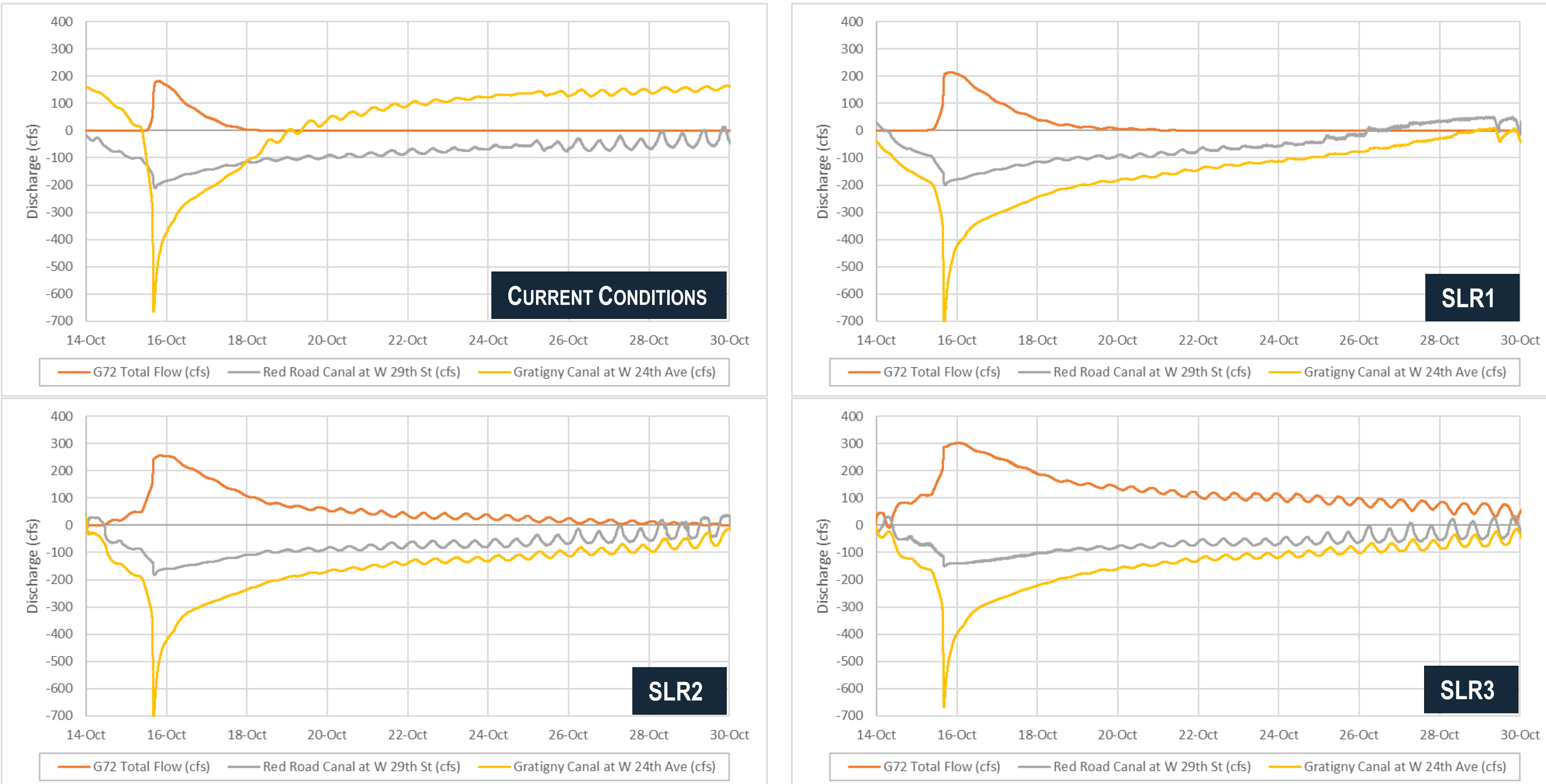


Figure 9-255. Exchanges between the C6 and C7 Watershed for the 25-year Design Storm

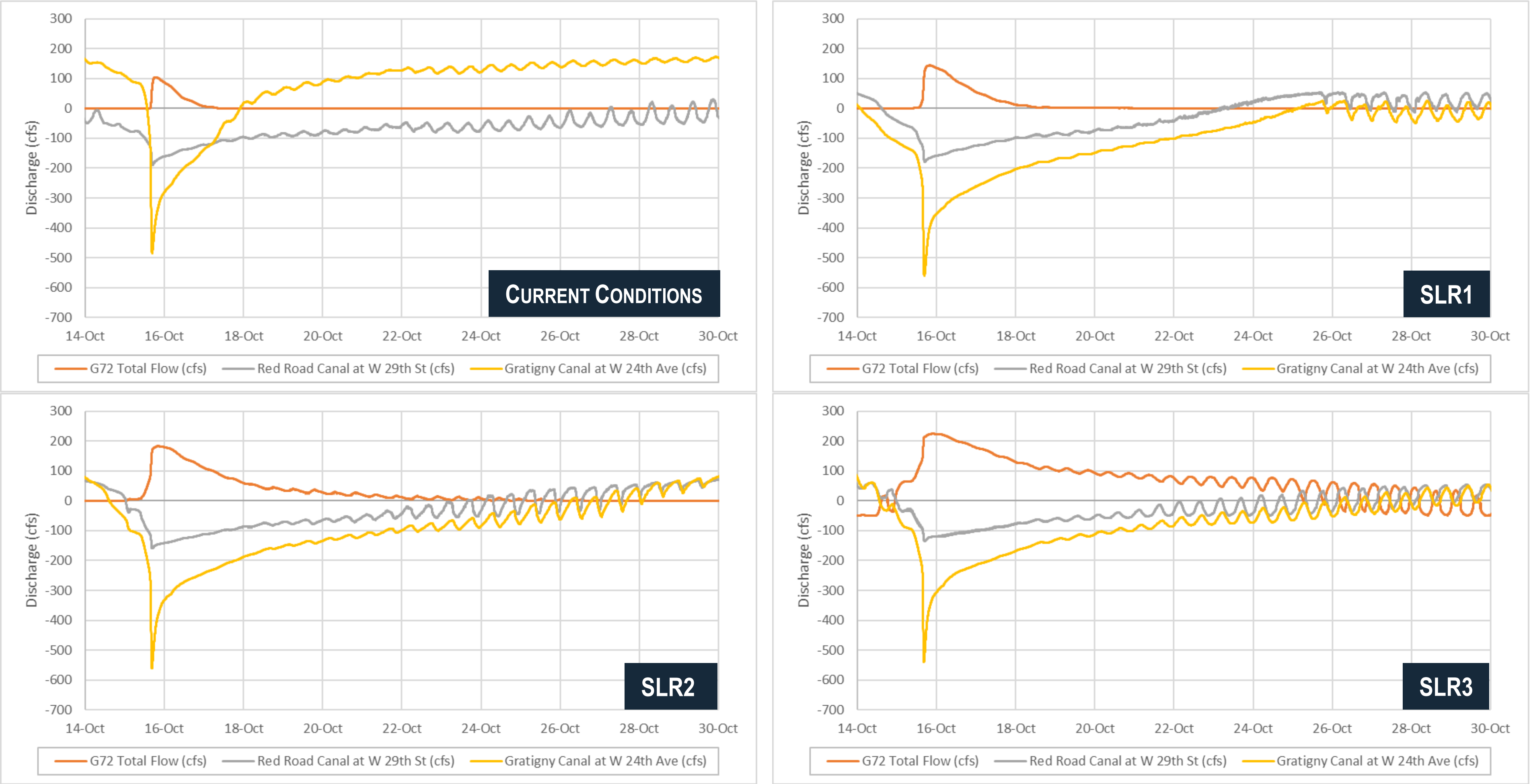


Figure 9-256. Exchange Between C6 Watershed and C4 Watershed During 100-year Design Storm

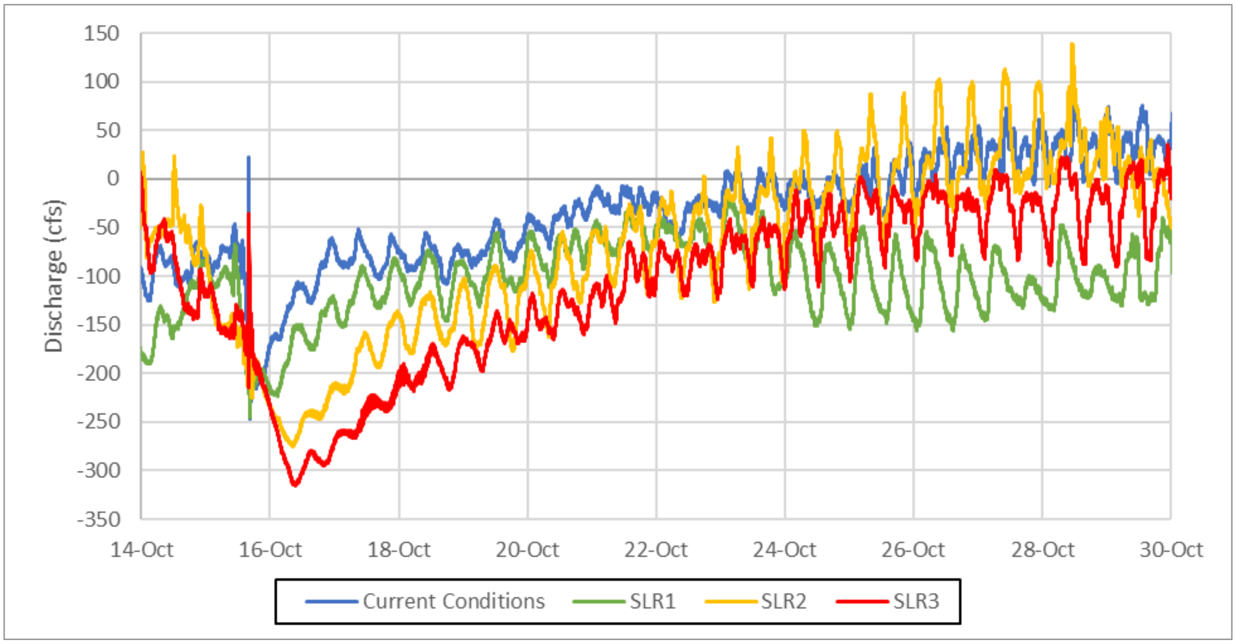
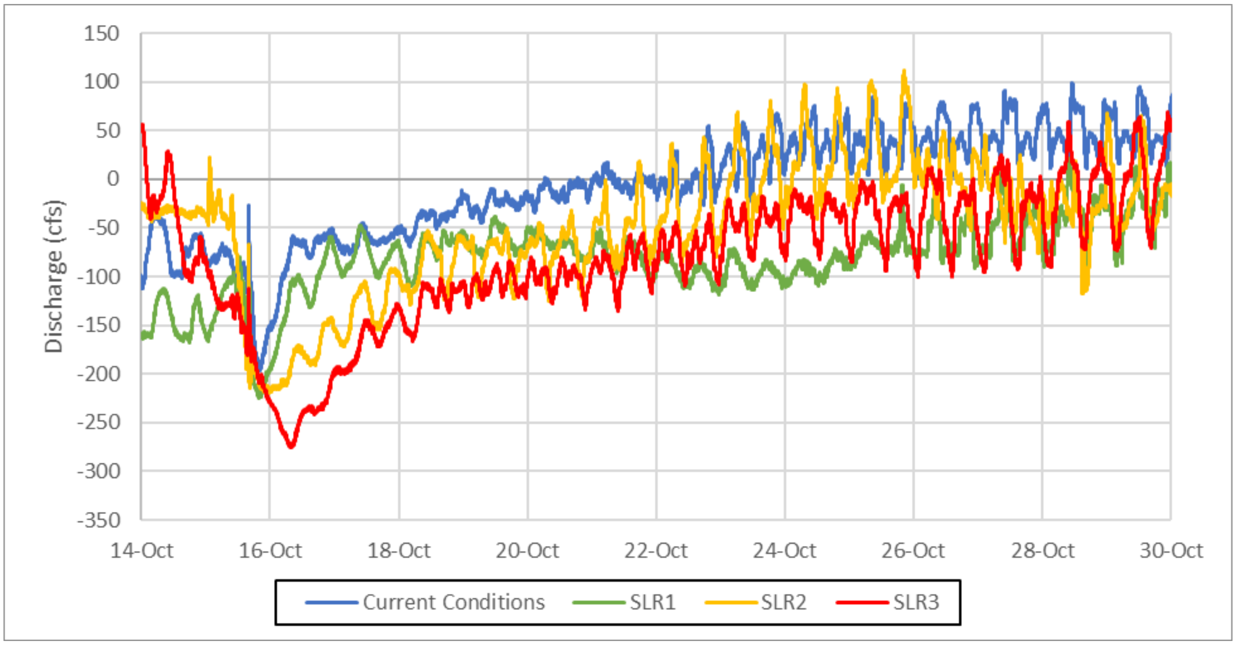


Figure 9-257. Exchange Between C6 Watershed and C4 Watershed During 25-year Design Storm



9.5.3. C6 – PM #3 STRUCTURE PERFORMANCE

The purpose of PM #3 is to determine the effect of SLR on tidal structure flow performance. For this metric, an evaluation based on structure design features and existing operational protocols was conducted for existing conditions and compared to simulations including SLR over the various storm events. PM #3 will provide only gravity flows through the structure and pumping, where PM #4 will also account for structure flows as well as overtopping, to differentiate between structure capacity and the basic inabilities of the structure to provide tidal protections.

As per the structure data sheet, the S26 tidal gravity structure (S26_S) is operated with the intention to maintain water levels in the C6 Watershed and pass the standard project flood without impacting upstream flooding. In addition, the structure is used to restrict flows to decrease stages and velocities that may cause damage to downstream areas, while preventing saline intrusion during high tides. **Table 9-86** provides the design parameters for S26_S as provided in the Water Control Operations Atlas.

Table 9-86. Design Parameters for Structure S26_S

DESIGN PARAMETERS	S26_S
Design Discharge	3,470 CFS (100% SPF)
Design HW	2.86 ft-NAVD (4.4 ft-NGVD)
Design TW	2.36 ft-NAVD (3.9 ft-NGVD)
Optimum HW	0.96 ft-NAVD (2.5 ft-NGVD)
Optimum TW	Tidal
Maximum Gate Opening	14.1 feet
Water Level which will Bypass Structure	3.96 ft-NAVD (5.5 ft-NGVD)
Water Level which will Overtop Gates when Closed	2.46 ft-NAVD (4.0 ft-NGVD)
Low Range Operational Trigger	-0.34 ft-NAVD (1.2 ft-NGVD) to 0.16 ft-NAVD (1.7 f- NGVD)

To maintain flood protection for the C6 basin, a 600CFS pump station (S26_P, **Table 9-87**) was added to the S26 spillway (S26_S) as part of the Miami Dade County Flood Mitigation Program in 2004. As per the structure sheet, S26_P allows additional discharge capacity during high tide or storm surge events when downstream water levels are elevated. Also, when the S25B forward pump station is operating, and the S26_S capacity is reduced, S26_P is operated to maintain upstream flood control prevention.

Table 9-87. Design Parameters for Structure S26_P

Design Parameters	S26_P
Design Discharge Capacity Total	600 CFS (3 x 200 CFS pumps)
Minimum Low Water (HW) Elevation	-0.64 ft-NAVD (0.9 ft-NGVD)

Figure 9-258 and **Figure 9-259** provide the instantaneous discharge, HW and TW at S26 during the current conditions simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-260** and **Figure 9-261** provide the 12-hour moving average for the discharge, HW, and TW at S26 during the current conditions simulation for the 100-year and 25-year design events, respectively, which removes the influence of the tides. **Figure 9-262** and **Figure 9-263** provide the instantaneous discharge, HW and TW at S26 during the future conditions SLR1 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-264** and **Figure 9-265** provide the 12-hour moving average for the discharge, HW, and TW at S26 during the future conditions SLR1 simulation for the 100-year and 25-year design events, respectively. **Figure 9-266** and **Figure 9-267** provide the instantaneous discharge, HW and TW at S26 during the future conditions SLR2 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-268** and **Figure 9-269** provide the 12-hour moving average for the discharge, HW, and TW at S26 during the future conditions SLR2 simulation for the 100-year and 25-year design events, respectively. **Figure 9-270** and **Figure 9-271** provide the instantaneous discharge, HW and TW at S26 during the future conditions SLR2 simulation for the 100-year 3-day and 25-year 3-day design events, respectively. **Figure 9-272** and **Figure 9-273** provide the 12-hour moving average for the discharge, HW, and TW at S26 during the future conditions SLR2 simulation for the 100-year and 25-year design events, respectively. Figures are provided for instantaneous discharge and 12-hour moving discharge for all design storms and SLR simulations in **Appendix B**.

Figure 9-274, **Figure 9-275**, **Figure 9-276**¸and **Figure 9-277** show the total instantaneous flow broken into flow from S26_S and S26_P for the 100-year design event for existing conditions, future SLR +1 foot, future SLR +2 feet , and future SLR +3 feet conditions, respectively. These figures show a decrease in pump use with SLR over 1 foot. According to structure operations, as further discussed in the Design Storm Setup section of this report, S26_P cannot operate unless S25B pumps (S25B_P) are already on. S25B_P is dependent on the elevation at MRMS1 tidal gauge station, with pumps only operating if the water level at MRMS1 is below 3.2 ft-NAVD. Therefore, S26_P can only operate if the water elevation at MRMS1 is less than 3.2 ft-NAVD. **Figure 9-278** shows the water surface elevations during current and future SLR conditions (SLR1, SLR2, SLR3) for the 100-year storm at MRMS1. The 100-year current conditions and future SLR1 scenario are only above the trigger level at MRMS1 during the peak of the storm event, where the water levels during the SLR3 scenario remain above the trigger level for a majority of the simulation, leaving S26_P unable to activate.

When the gates are overtopped and backflow is occurring, there are some minor instances where the pump turns on, which is creating a short-circuiting effect of both pumping and gate overtopping. **Figure 9-279** provides an example of when the gate flow is reversed, and the pump is operating during the 10-year SLR +2 feet simulation. The pump operational logic was not input into the model with a control to stop pumping if gate overtopping occurs, as this was not strictly written into the structure operations sheet or C4 Basin Operation Plan (SFWMD, 2016).

Figure 9-258. Instantaneous Discharge and Stages at S26_S for the 100-year 3-day Current Conditions Event

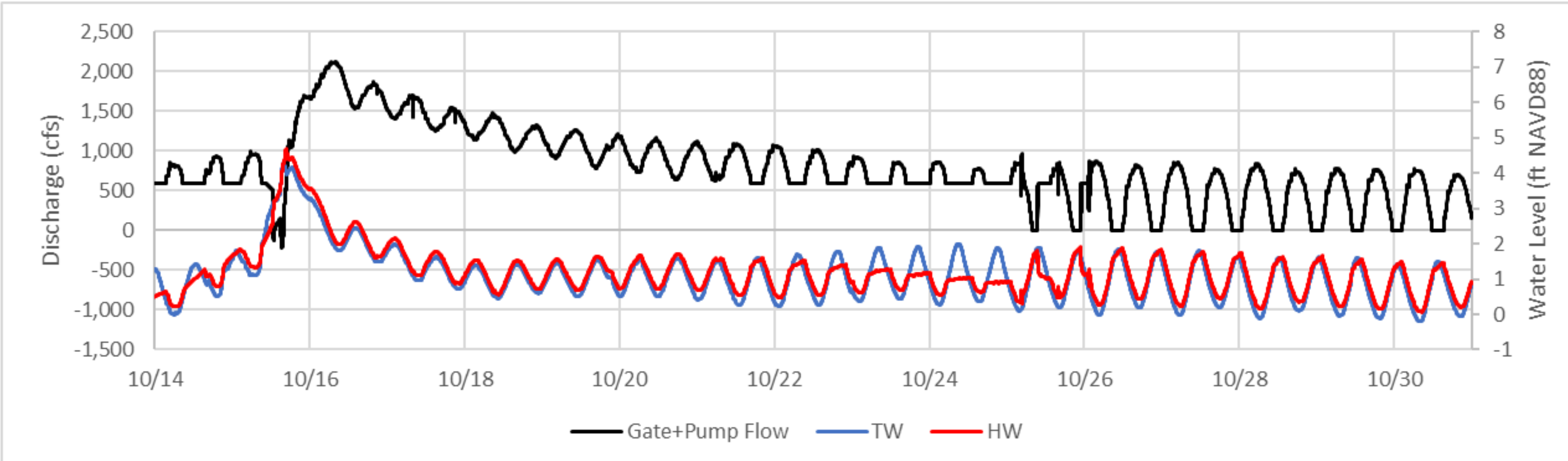


Figure 9-259. Instantaneous Discharge and Stages at S26_S for the 25-year 3-day Current Conditions Event

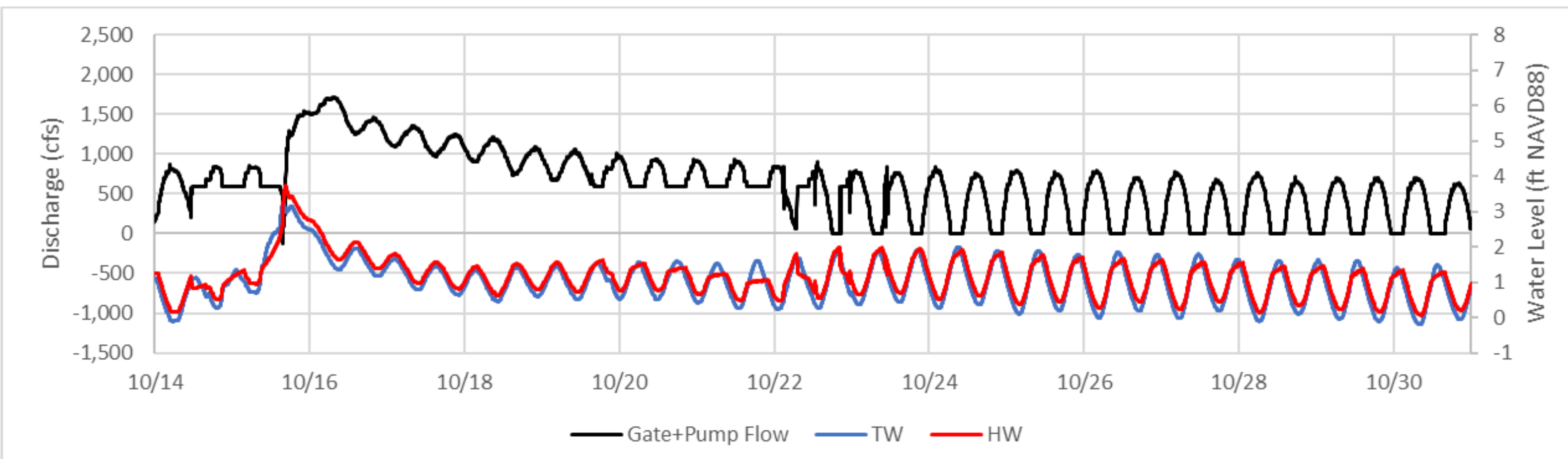


Figure 9-260. 12-Hour Moving Average Discharge and Stages at S26_S for the 100-year 3-day Current Conditions Event

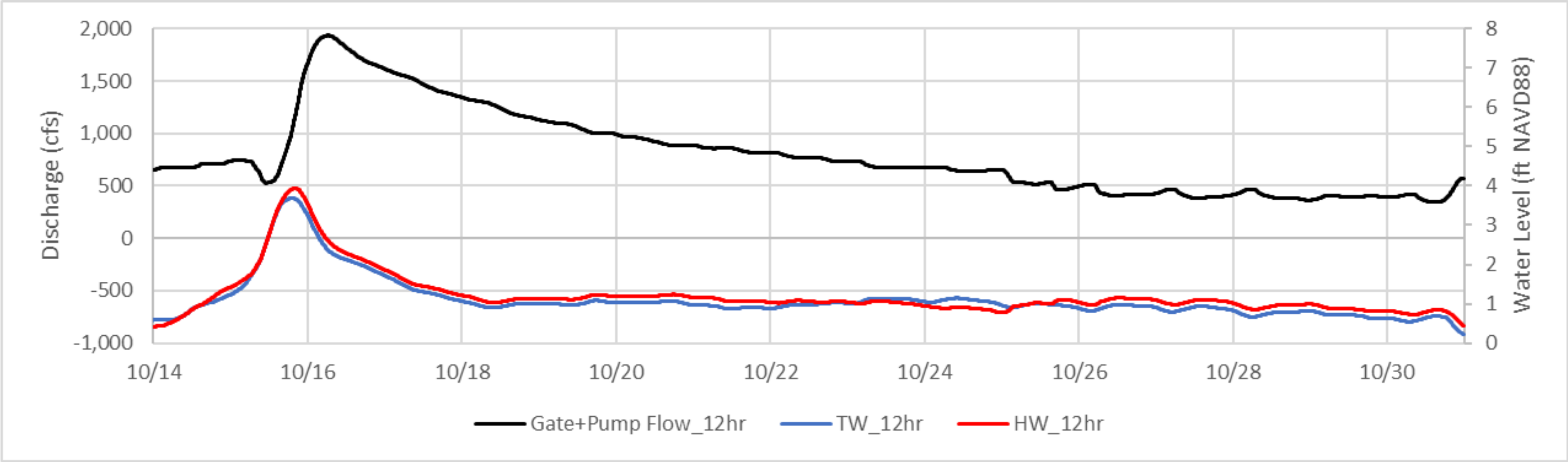


Figure 9-261. 12-Hour Moving Average Discharge and Stages at S26_S for the 25-year 3-day Current Conditions Event

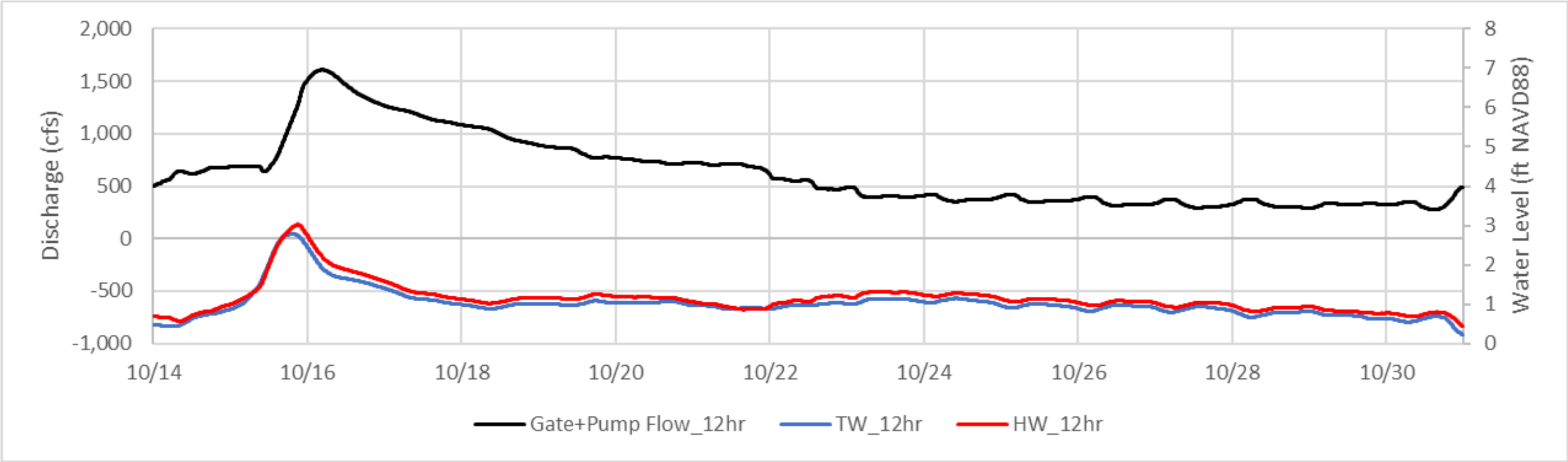


Figure 9-262. Instantaneous Discharge and Stages at S26_S for the 100-year 3-day Future Conditions (SLR1) Event

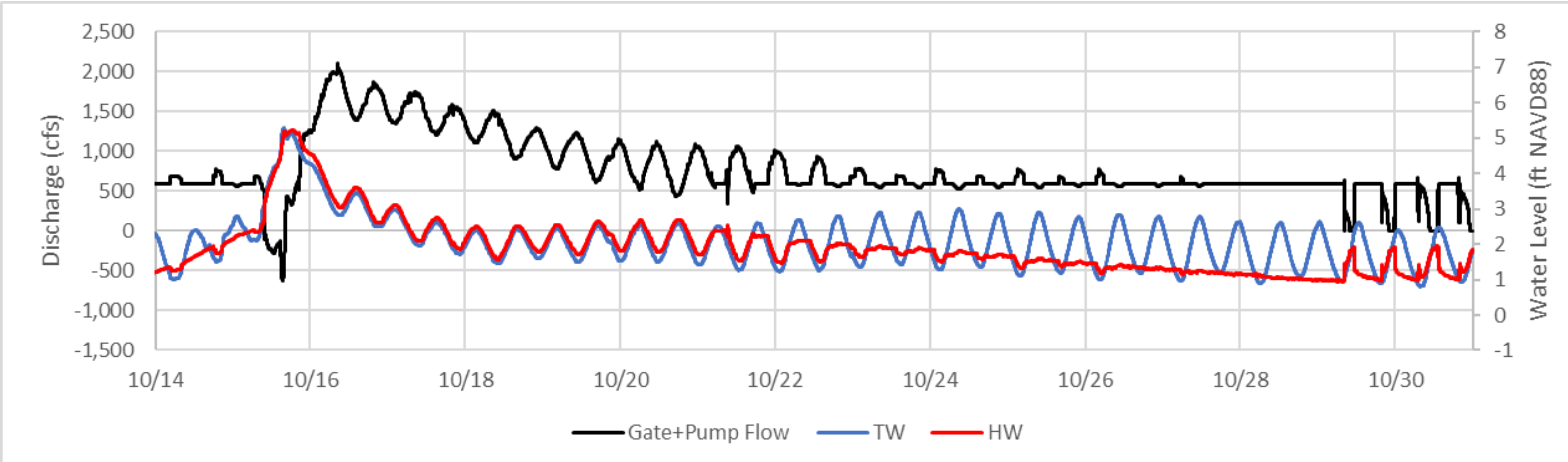


Figure 9-263. Instantaneous Discharge and Stages at S26_S for the 25-year 3-day Future Conditions (SLR1) Event

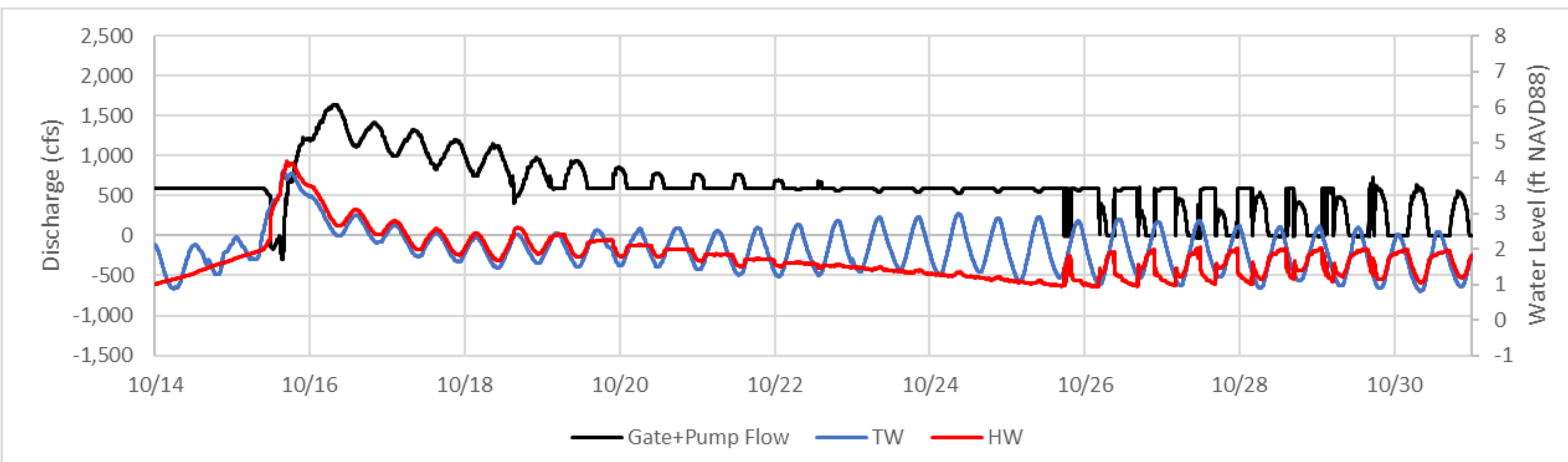


Figure 9-264. 12-Hour Moving Average Discharge and Stages at S26_S for the 100-year 3-day Future Conditions (SLR1) Event

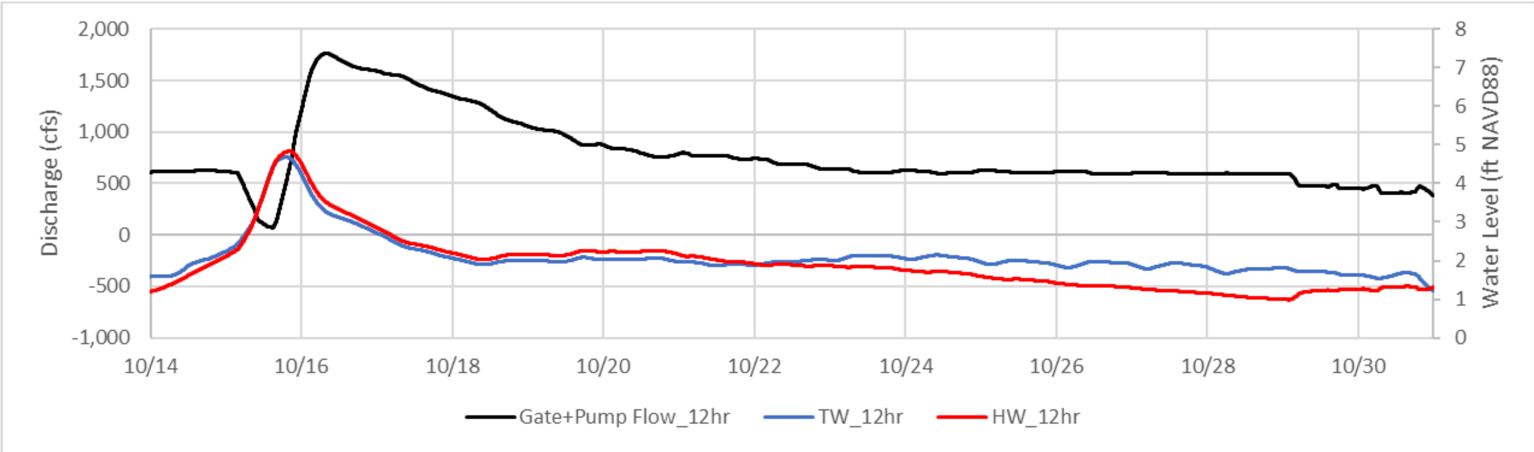


Figure 9-265. 12-Hour Moving Average Discharge and Stages at S26_S for the 25-year 3-day Future Conditions (SLR1) Event

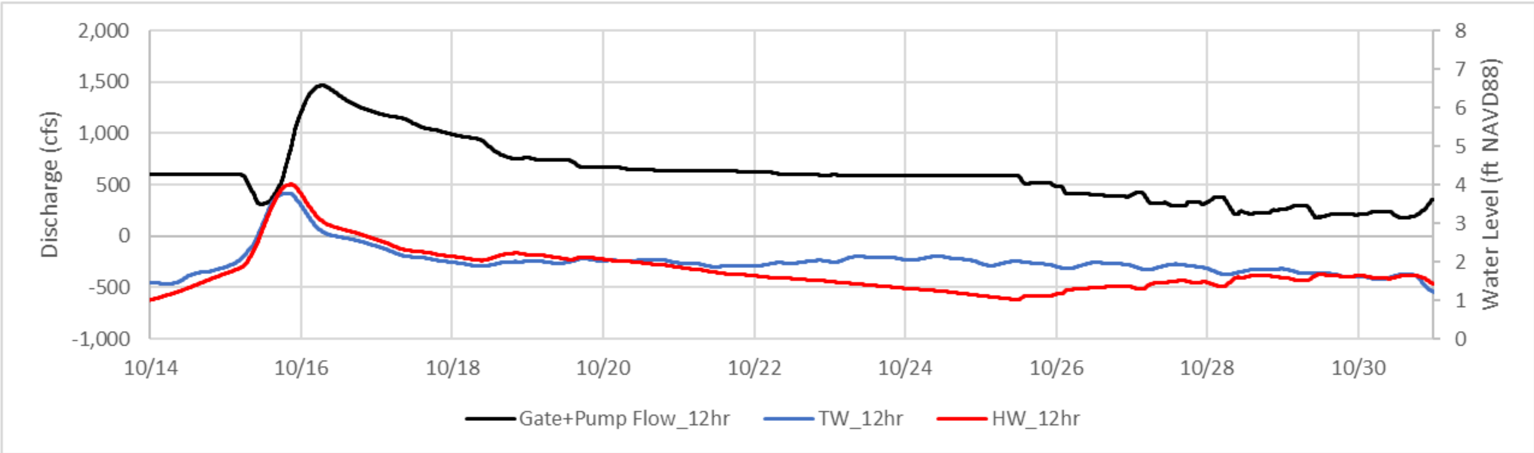


Figure 9-266. Instantaneous Discharge and Stages at S26_S for the 100-year 3-day Future Conditions (SLR2) Event

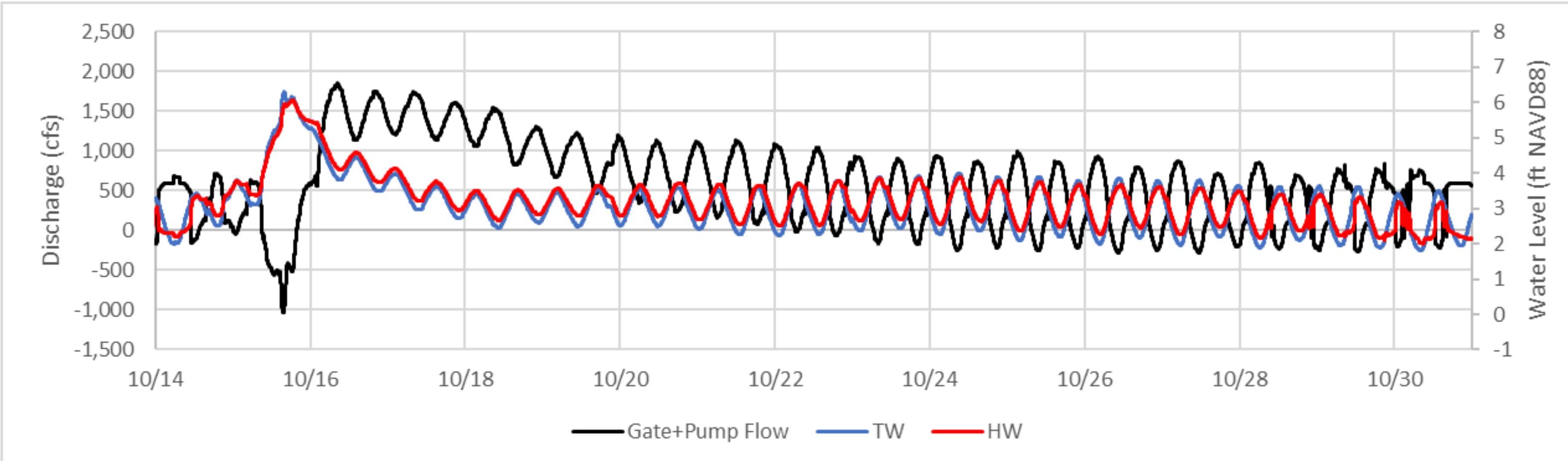


Figure 9-267. Instantaneous Discharge and Stages at S26_S for the 25-year 3-day Future Conditions (SLR2) Event

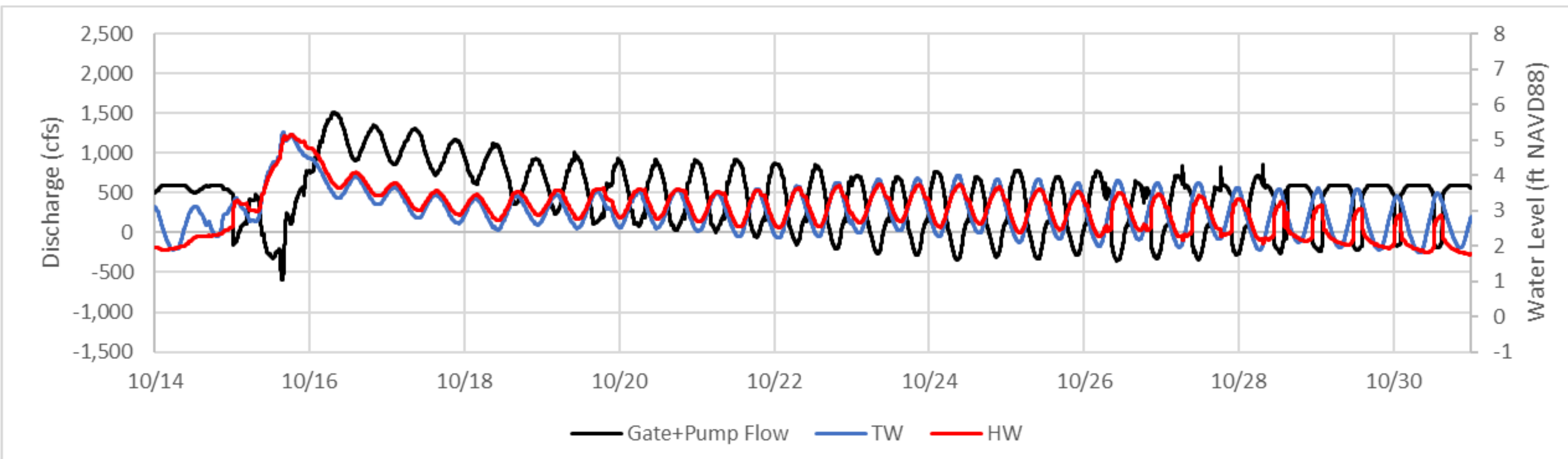


Figure 9-268. 12-Hour Moving Average Discharge and Stages at S26_S for the 100-year 3-day Future Conditions (SLR2) Event

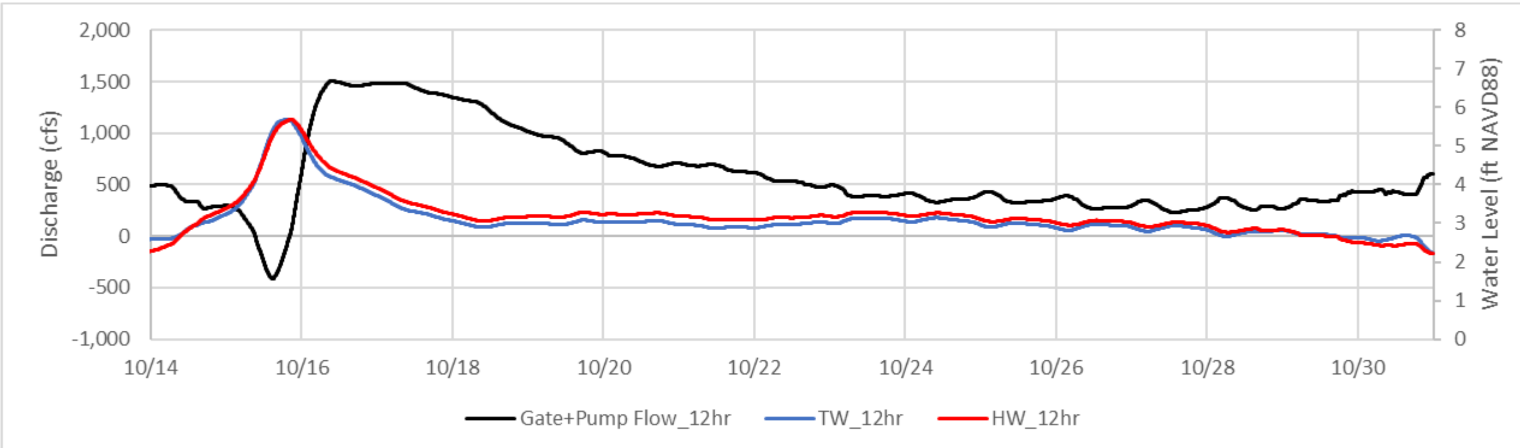


Figure 9-269. 12-Hour Moving Average Discharge and Stages at S26_S for the 25-year 3-day Future Conditions (SLR2) Event

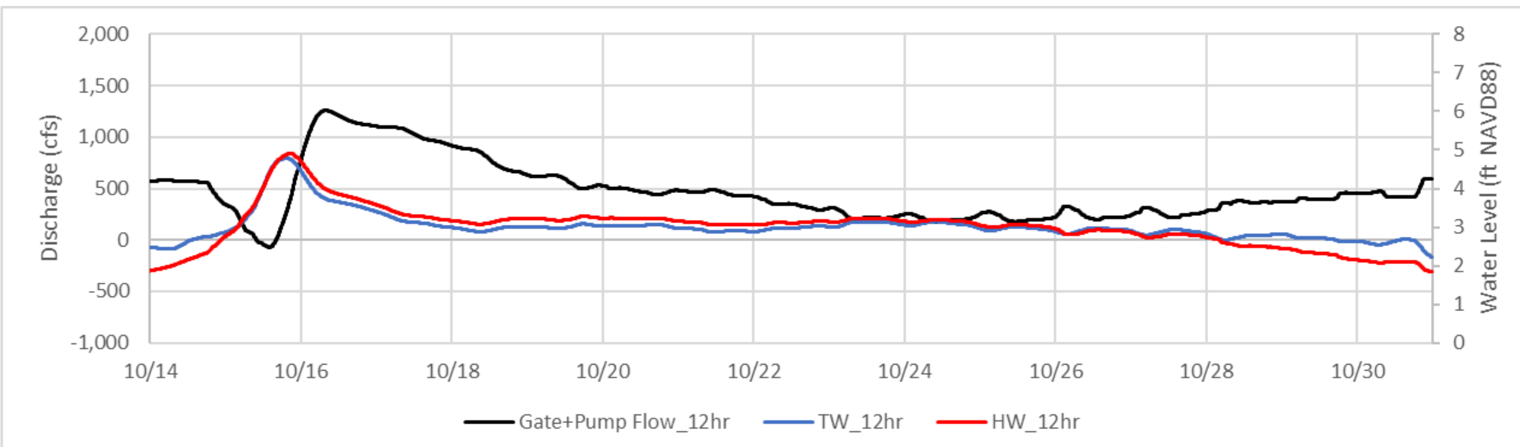


Figure 9-270. Instantaneous Discharge and Stages at S26_S for the 100-year 3-day Future Conditions (SLR3) Event

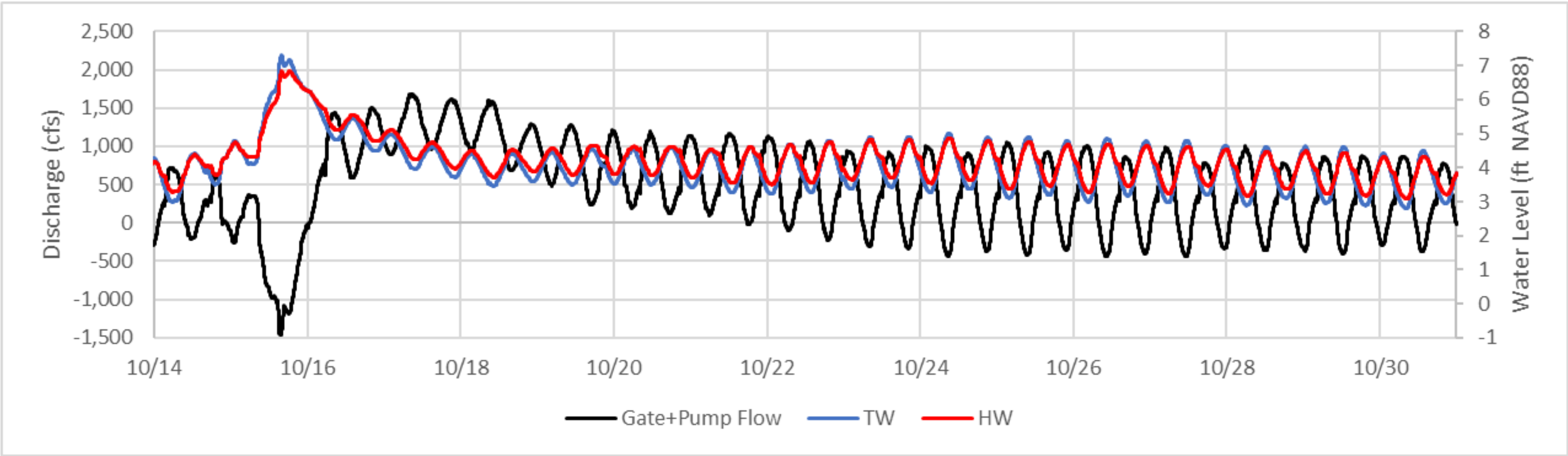


Figure 9-271. Instantaneous Discharge and Stages at S26_S for the 25-year 3-day Future Conditions (SLR3) Event

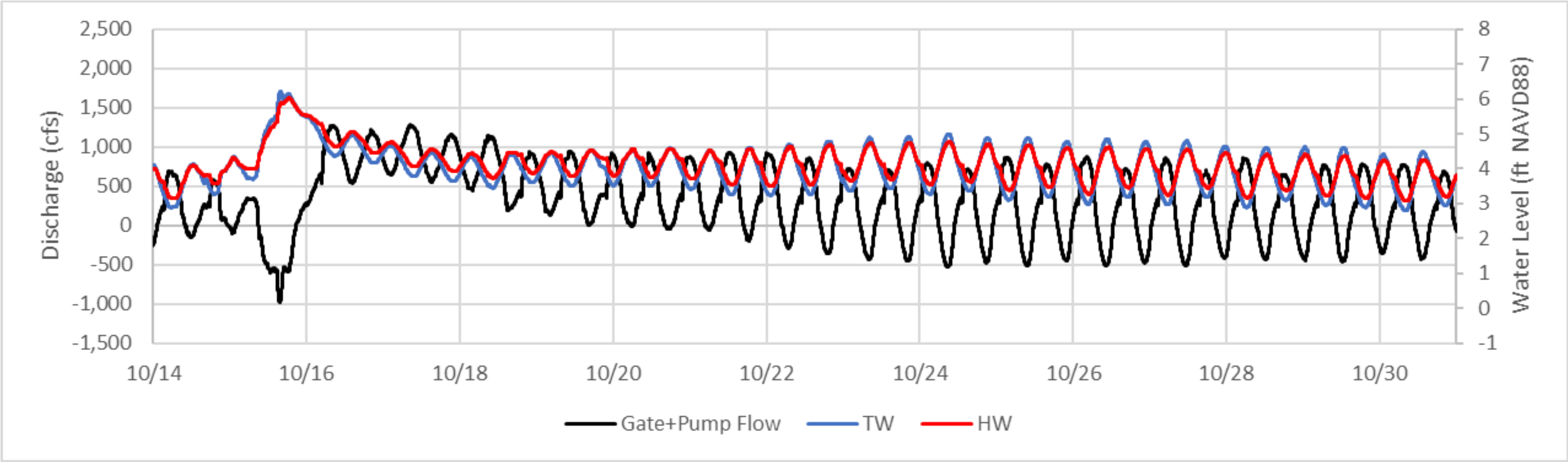


Figure 9-272. 12-Hour Moving Average Discharge and Stages at S26_S for the 100-year 3-day Future Conditions (SLR3) Event

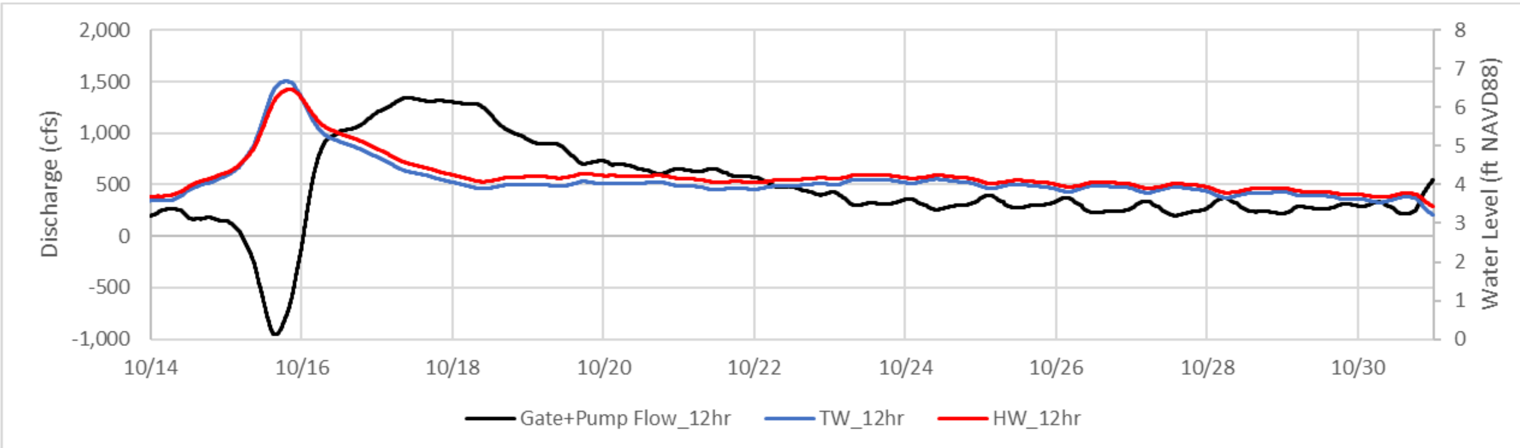


Figure 9-273. 12-Hour Moving Average Discharge and Stages at S26_S for the 25-year 3-day Future Conditions (SLR3) Event

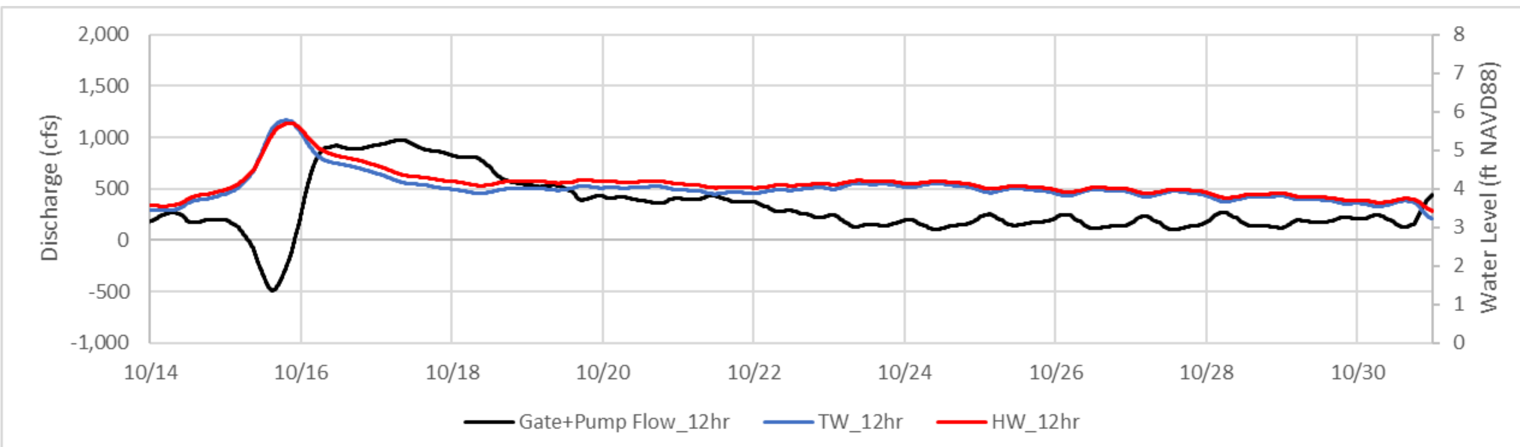


Figure 9-274. Total Instantaneous Flow at S26 for the 100-year 3-day Existing Conditions Event

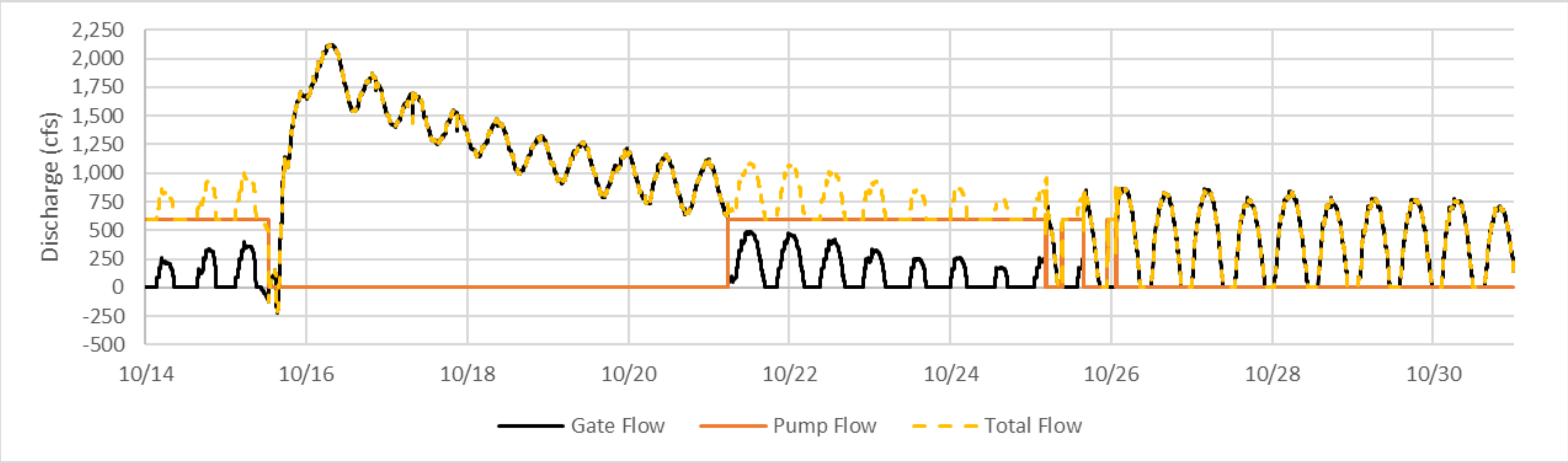


Figure 9-275. Total Instantaneous Flow at S26 for the 100-year 3-day Future Conditions (SLR1) Event

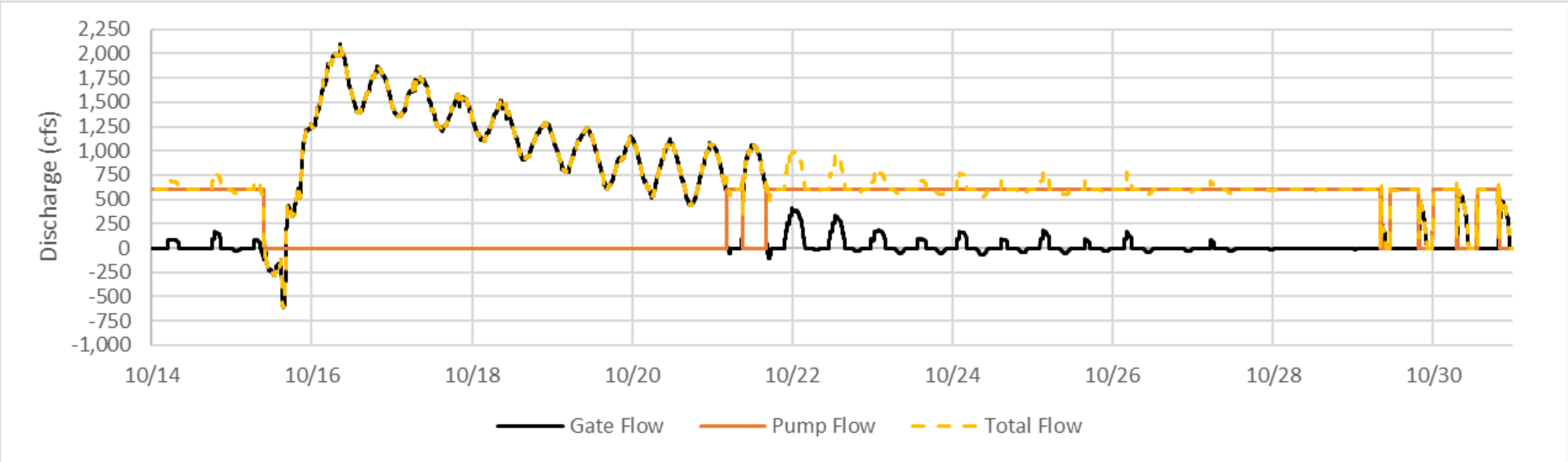


Figure 9-276. Total Instantaneous Flow at S26 for the 100-year 3-day Future Conditions (SLR2) Event

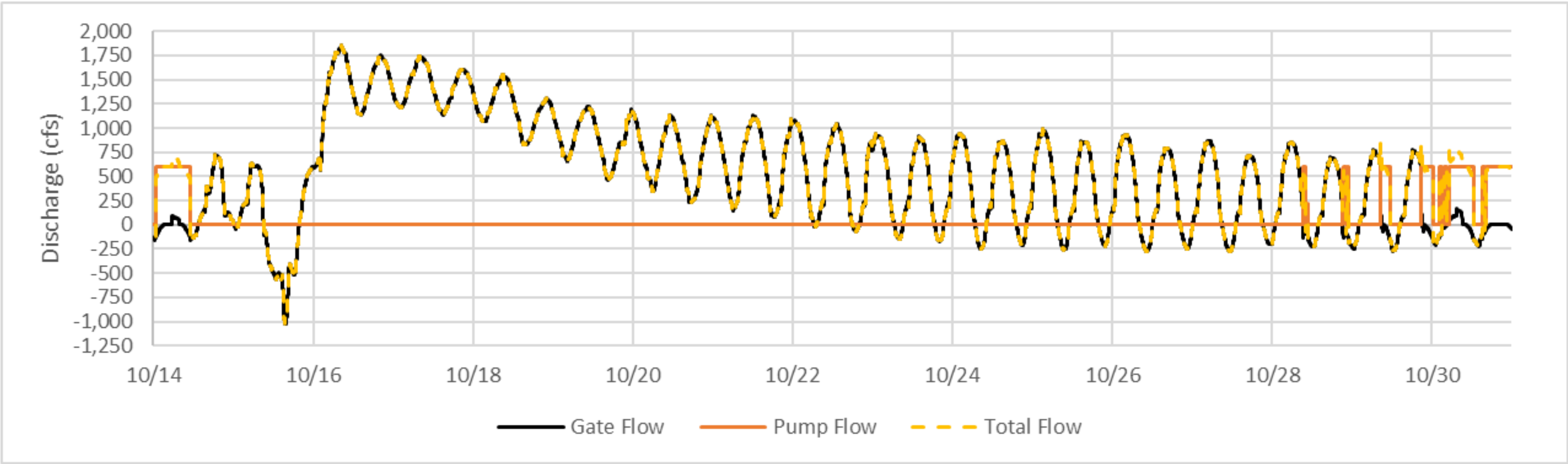


Figure 9-277. Total Instantaneous Flow at S26 for the 100-year 3-day Future Conditions (SLR3) Event

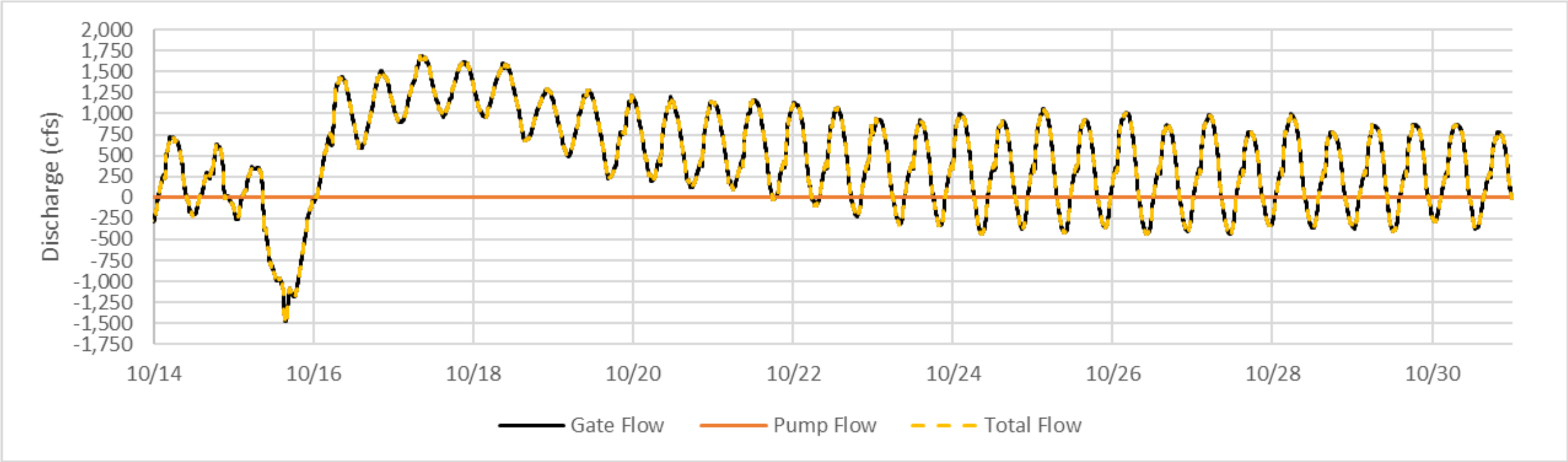


Figure 9-278. Water Surface Elevation at MRMS1 during 100-year Storm

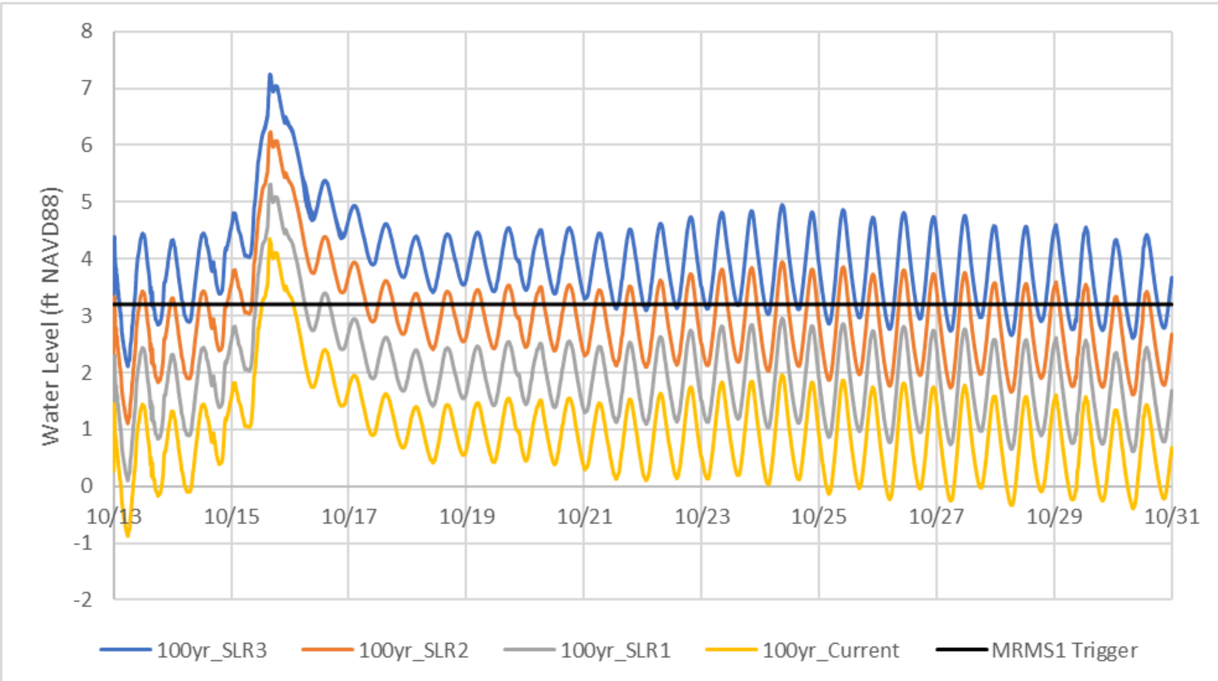
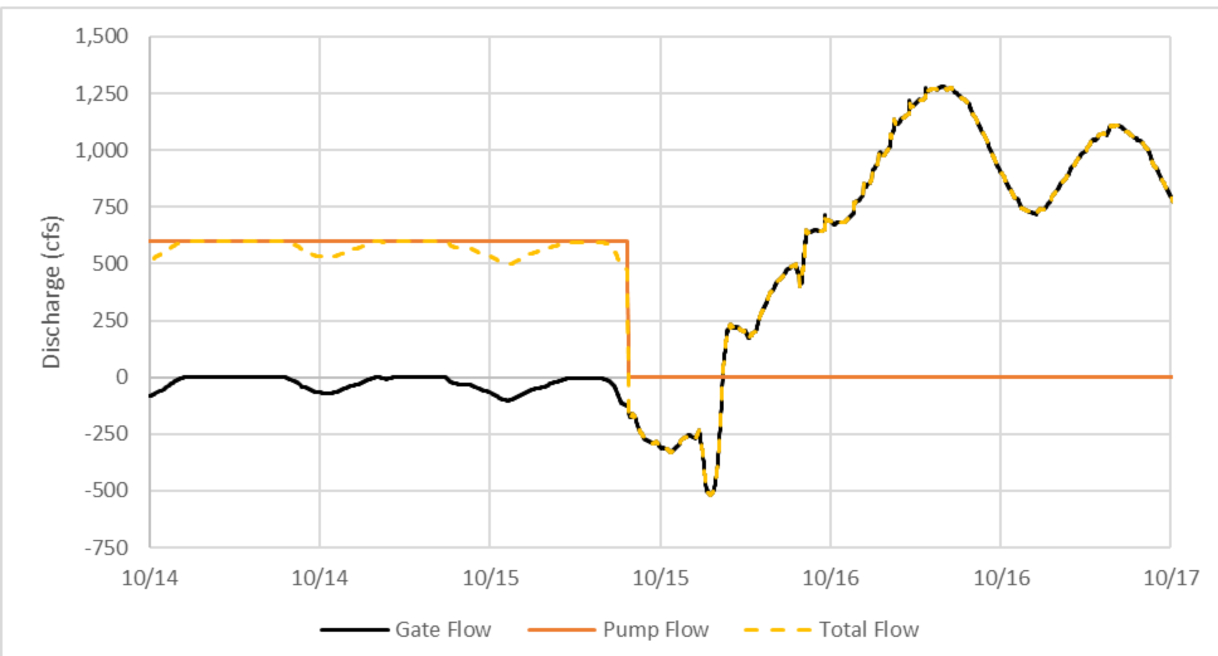


Figure 9-279. Flow Recirculation during 10-year 3-day Future Conditions (SLR2) Event



A summary of the instantaneous and 12-hour moving average conditions at the time of peak discharge at S26 are provided in **Table 9-88** and **Table 9-89**, respectively for all design storm return periods and future SLR conditions.

Table 9-88. Summary of Conditions at the Time of Peak Discharge at S26 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak Q	Peak Q (CFS)	HW at Peak Q (FT-NAVD)	TW at Peak Q (FT-NAVD)	ΔH at Peak Q (FT)	Gate Opening at Peak Q* (FT)
100-Year	**Current**	10/16 8:00	2095.31	2.06	1.88	0.18	13.5
	SLR1	10/16 8:28	2101.14	3.08	2.84	0.25	14
	SLR2	10/16 8:30	1850.34	4.13	3.83	0.30	12.5
	SLR3	10/17 8:29	1688.27	4.30	4.01	0.29	11.5
25-Year	**Current**	10/16 5:07	1706.72	1.74	1.45	0.29	11.5
	SLR1	10/16 7:45	1645.51	2.73	2.43	0.30	11
	SLR2	10/16 7:53	1511.67	3.72	3.42	0.30	10
	SLR3	10/17 8:30	1282.36	4.11	3.82	0.29	8.5
10-Year	**Current**	10/16 6:12	1500.38	1.71	1.42	0.30	10
	SLR1	10/16 7:45	1416.90	2.52	2.23	0.29	9.5
	SLR2	10/16 7:58	1281.29	3.50	3.21	0.29	8.5
	SLR3	10/16 8:29	1127.64	4.46	4.17	0.28	7.5
5-Year	**Current**	10/16 4:27	1358.96	1.83	1.54	0.29	9
	SLR1	10/16 6:45	1269.66	2.51	2.22	0.29	8.5
	SLR2	10/16 7:14	1123.80	3.45	3.17	0.28	7.5
	SLR3	10/16 6:46	944.86	4.52	4.22	0.30	6

*A gate opening of 14.1 ft represents the gate full open.

Table 9-89. Summary of Conditions at the Time of Peak Discharge through S26 (12 hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak Q	Peak Q (CFS)	HW at Peak Q (FT-NAVD)	TW at Peak Q (FT-NAVD)	ΔH at Peak Q (FT)	Gate Opening at Peak Q* (FT)
100-Year	**Current**	10/16 6:43	1903.10	2.56	2.34	0.22	13.06
	SLR1	10/16 7:39	1766.11	3.53	3.28	0.25	12.96
	SLR2	10/16 10:14	1505.11	4.40	4.17	0.23	12.09
	SLR3	10/17 9:59	1344.77	4.55	4.33	0.21	11.06
25-Year	**Current**	10/16 5:07	1587.32	2.17	1.89	0.28	10.93
	SLR1	10/16 6:39	1464.17	3.06	2.79	0.27	10.27
	SLR2	10/16 7:57	1259.10	3.98	3.73	0.25	9.23
	SLR3	10/17 7:50	975.86	4.38	4.17	0.20	8.03
10-Year	**Current**	10/16 4:34	1398.19	1.96	1.68	0.28	9.54
	SLR1	10/16 6:17	1261.03	2.84	2.57	0.27	8.79
	SLR2	10/16 7:39	1060.77	3.76	3.51	0.25	7.75
	SLR3	10/16 7:54	786.33	4.73	4.51	0.23	5.17
5-Year	**Current**	10/16 4:54	1259.76	1.79	1.51	0.28	8.59
	SLR1	10/16 6:13	1112.80	2.71	2.44	0.27	7.73
	SLR2	10/16 7:24	917.03	3.64	3.39	0.25	6.67
	SLR3	10/16 7:16	663.98	4.62	4.40	0.22	3.94

*A gate opening of 14.1 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak HW at S26 are provided in **Table 9-90** and **Table 9-91**, respectively for all design storm return periods and future SLR conditions.

Table 9-90. Summary of Conditions at the Time of Peak HW at S26 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak HW	Q at Peak HW (CFS)	Peak HW (ft-NAVD)	TW at Peak HW (ft-NAVD)	ΔH at Peak HW (ft)	Gate Opening at Peak HW* (ft)
100-Year	**Current**	10/15 16:45	670.00	4.64	3.97	0.67	2.5
	SLR1	10/15 18:19	336.94	5.22	5.15	0.07	0
	SLR2	10/15 18:23	-502.08	6.07	6.15	-0.09	0
	SLR3	10/15 16:00	-1420.14	6.83	7.30	-0.47	0
25-Year	**Current**	10/15 21:14	550.52	3.71	3.03	0.68	2
	SLR1	10/15 16:53	424.70	4.49	3.98	0.51	1.5
	SLR2	10/15 18:23	122.19	5.16	5.14	0.02	0
	SLR3	10/15 18:44	-561.32	6.03	6.14	-0.11	0
10-Year	**Current**	10/15 16:43	315.86	3.35	2.56	0.79	1
	SLR1	10/15 18:23	574.69	4.02	3.72	0.30	3.5
	SLR2	10/15 18:38	199.85	4.75	4.72	0.03	0
	SLR3	10/15 18:45	-384.00	5.65	5.72	-0.06	0
5-Year	**Current**	10/15 17:13	393.80	2.97	2.37	0.60	1.5
	SLR1	10/15 18:09	239.33	3.82	3.47	0.36	1
	SLR2	10/15 18:45	179.20	4.52	4.48	0.03	0
	SLR3	10/15 18:45	-270.52	5.45	5.48	-0.04	0

*A gate opening of 14.1 ft represents the gate full open.

Table 9-91. Summary of Conditions at the Time of Peak HW at S26 (12 hour Moving Average)

Design Storm Return Period	Scenario	Time of Peak HW	Q at Peak HW (CFS)	Peak HW (FT-NAVD)	TW at Peak HW (FT-NAVD)	ΔH at Peak HW (FT)	Gate Opening at Peak HW* (FT)
100-Year	Current	10/15 20:02	1095.55	3.92	3.67	0.26	7.19
	SLR1	10/15 20:15	652.22	4.84	4.66	0.17	3.74
	SLR2	10/15 20:48	55.66	5.68	5.64	0.04	0.26
	SLR3	10/15 20:26	-647.75	6.48	6.66	-0.18	0.00
25-Year	Current	10/15 21:14	1243.14	3.02	2.74	0.28	8.04
	SLR1	10/15 20:47	864.92	4.01	3.76	0.25	5.69
	SLR2	10/15 20:52	410.29	4.90	4.76	0.15	2.00
	SLR3	10/15 20:50	-140.90	5.71	5.76	-0.05	0.00
10-Year	Current	10/15 21:46	1161.74	2.61	2.33	0.28	7.44
	SLR1	10/15 21:09	788.82	3.61	3.37	0.24	5.19
	SLR2	10/15 21:04	398.71	4.54	4.38	0.16	1.95
	SLR3	10/15 20:57	-17.11	5.37	5.38	-0.01	0.00
5-Year	Current	10/15 22:03	1049.92	2.38	2.11	0.27	6.56
	SLR1	10/15 21:43	708.33	3.38	3.13	0.25	4.43
	SLR2	10/15 21:15	343.19	4.32	4.16	0.16	1.63
	SLR3	10/15 20:59	20.18	5.17	5.18	0.00	0.00

*A gate opening of 14.1 ft represents the gate full open.

A summary of the instantaneous and 12-hour moving average conditions at the time of peak TW at S26 are provided in **Table 9-92** and **Table 9-93**, respectively for all design storm return periods and future SLR conditions.

Table 9-92 shows that there are instances when there are negative flows, yet the gate opening is zero, this indicates that the tailwater is higher than the top of the closed gate (2.458 ft-NAVD) and there is a negative head differential across this structure. However, negative flows can also occur when the gate is in the process of closing and the head differential is negative.

Since **Table 9-93** shows the averaged values over a 12-hour period, there may be some instances when the average head differential is negative, yet the flows are positive, or the flows are negative, yet the head differential is positive. These disagreements between flows and head differentials are due to the averaging of the values.

Table 9-92. Summary of Conditions at the Time of Peak Tailwater at S26 (Instantaneous)

Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (ft-NAVD)	Peak TW (ft-NAVD)	ΔH at Peak TW (ft)	Gate Opening at Peak TW* (ft)
100-Year	**Current**	10/15 16:00	66.54	4.31	4.30	0.01	0
	SLR1	10/15 16:00	-455.02	5.17	5.30	-0.13	0
	SLR2	10/15 16:00	-980.29	5.95	6.30	-0.35	0
	SLR3	10/15 16:00	-1420.14	6.83	7.30	-0.47	0
25-Year	**Current**	10/15 19:34	120.86	3.33	3.23	0.11	0
	SLR1	10/15 16:00	-96.01	4.20	4.23	-0.02	0
	SLR2	10/15 16:00	-473.68	5.08	5.23	-0.15	0
	SLR3	10/15 16:00	-937.67	5.89	6.23	-0.33	0
10-Year	**Current**	10/15 16:00	568.08	2.34	2.78	-0.44	0
	SLR1	10/15 16:00	-200.79	3.65	3.78	-0.13	0
	SLR2	10/15 16:00	-412.58	4.61	4.78	-0.17	0
	SLR3	10/15 16:00	-715.16	5.53	5.78	-0.24	0
5-Year	**Current**	10/15 16:00	596.75	1.86	2.53	-0.67	0
	SLR1	10/15 16:00	-240.42	3.18	3.53	-0.35	0
	SLR2	10/15 16:00	-417.14	4.30	4.53	-0.22	0
	SLR3	10/15 16:00	-636.84	5.30	5.53	-0.23	0

*A gate opening of 14.1 ft represents the gate full open.

Table 9-93. Summary of Conditions at the Time of Peak Tailwater at S26_S (12 hour Moving Average)

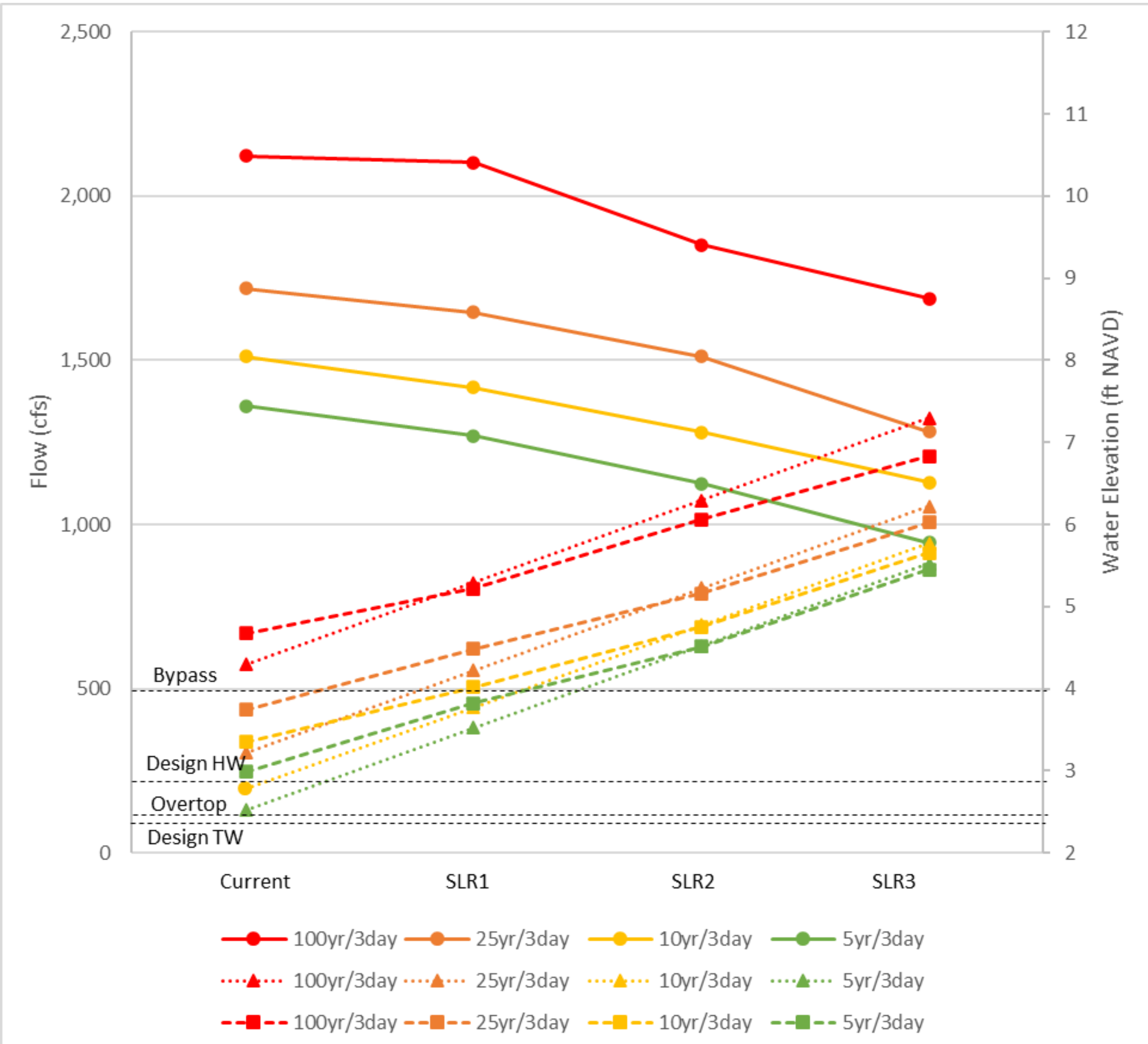
Design Storm Return Period	Scenario	Time of Peak TW	Q at Peak TW (CFS)	HW at Peak TW (ft-NAVD)	Peak TW (ft-NAVD)	ΔH at Peak TW (ft)	Gate Opening at Peak TW* (ft)
100-Year	**Current**	10/15 19:03	961.14	3.91	3.68	0.23	6.25
	SLR1	10/15 19:03	495.18	4.82	4.68	0.14	2.83
	SLR2	10/15 19:03	-119.75	5.64	5.68	-0.03	0.00
	SLR3	10/15 19:03	-778.00	6.45	6.68	-0.22	0.00
25-Year	**Current**	10/15 19:34	1115.62	2.97	2.77	0.19	6.63
	SLR1	10/15 19:34	727.42	3.99	3.77	0.21	4.81
	SLR2	10/15 19:34	282.10	4.88	4.77	0.11	1.34
	SLR3	10/15 19:34	-247.98	5.69	5.77	-0.08	0.00
10-Year	**Current**	10/15 19:47	1036.22	2.54	2.40	0.14	5.93
	SLR1	10/15 19:47	671.26	3.54	3.40	0.15	4.32
	SLR2	10/15 19:47	281.32	4.51	4.40	0.12	1.38
	SLR3	10/15 19:47	-102.75	5.36	5.40	-0.04	0.00
5-Year	**Current**	10/15 19:58	941.18	2.28	2.19	0.09	5.14
	SLR1	10/15 19:58	604.14	3.25	3.19	0.06	3.44
	SLR2	10/15 19:58	233.46	4.29	4.19	0.11	1.14
	SLR3	10/15 19:58	-46.73	5.16	5.19	-0.02	0.00

*A gate opening of 14.1 ft represents the gate full open.

Figure 9-280 shows a summary of the instantaneous peak discharge, HW, and TW at S26 for all design storm return periods and future SLR conditions. The design parameter values listed in **Table 9-86** are shown graphically in the figure below with bypass indicating the water level which will bypass the structure and overtop indicating the water level which will overtop the gates when the gates are closed. The design discharge (3470 CFS) is not shown in **Figure 9-280** as it is significantly higher than the peak discharge as further discussed. Note that the peak discharge, HW, and TW occur at different times for each scenario.

The maximum HW and TW at S26 exceeds the design HW and TW for all simulations. The maximum HW exceeds the water level which will overtop the gates for all conditions except for the current conditions 5-year design storm. The maximum TW exceeds the water level which will overtop the gates for all conditions, sometimes occurring when the gates are closed. TW elevations that exceed the overtopping elevation when the gates are closed can result in flow entering the basin from storm surge and/or tide. During the peak of the storm, flow overtops the gate and flows into the basin for all future condition simulations. The amount of flow that enters the basin from storm surge increases with design storm return period and amount of SLR. The HW at S26 exceeds the water level that will bypass the structure for the 100-year current conditions scenario and all future SLR scenarios (SLR1, SLR2, SLR3) except for the 5-year SLR1 scenario. The TW also exceeds this bypass elevation during the 100-year current conditions scenario and all future SLR scenarios (SLR1, SLR2, SLR3) except for the 5-year and 10-year SLR1 scenarios. Flow that bypasses the structure can contribute to flooding of neighborhoods around S26.

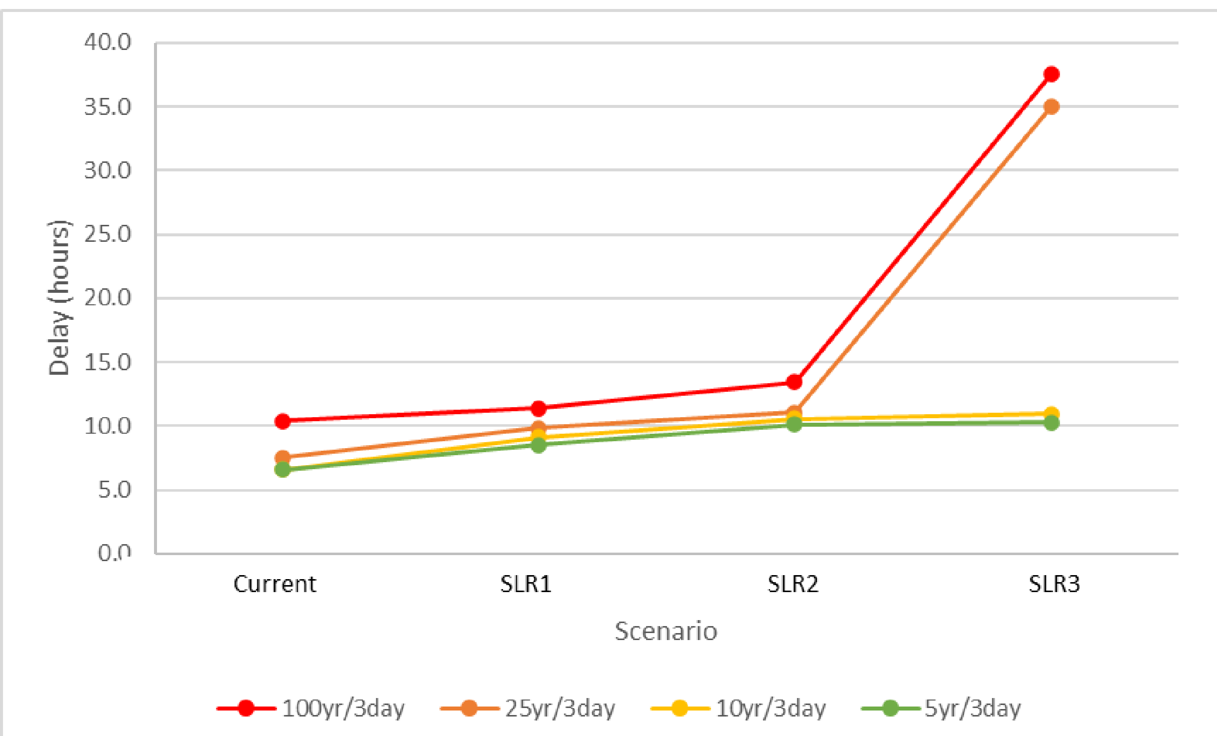
Figure 9-280. Summary of Instantaneous Max Discharge, HW, and TW at S26_S



The peak discharges for all design storms during current and future conditions fall significantly under the design discharge of 3,470 CFS. This correlates with Canal Conveyance Capacity Project – C6 Canal Study (C6 Report) which found that the design flows could not be conveyed through S26_S while satisfying the water surface elevation criteria set by the original Central and Southern Florida Project (SFWMD, 2020). According to the C6 Report, the original design of the C6 Canal was not implemented completely, and continued urbanization now limits the scope of further implementation of the original plan, limiting the ability to discharge the design flow through S26_S.

Without TW limitations, increased HW would generate more flow through a gravity structure (i.e., you would expect to see maximum flow through the structure occurring near the time of maximum HW). The time between the 12-hour moving average peak HW (**Table 9-88**) and the 12-hour moving average peak discharge (**Table 9-89**) is shown in **Figure 9-281** for each design storm. All current condition simulations and most future SLR scenarios reach the peak discharge within 15 hours of the peak HW at S26_S. However, the 100-year and 25-year SLR3 scenarios see a delay between peak HW and peak discharge of more than double the other scenarios. This delay in the structure's ability to discharge water is a result of high TW conditions.

Figure 9-281. Time Between Peak HW and Peak Discharge at S26_S for the 12-Hour Moving Average



9.5.4. C6 – PM #4 PEAK STORM RUNOFF

The purpose of PM #4 is to determine the effect of SLR on the maximum peak storm runoff, or maximum conveyance capacity of the watershed. For this metric, 12-hour moving average flow hydrographs downstream of S26 and the maximum 12-hour moving average total flow was determined for each design storm event and SLR scenario.

The 12-hour moving average discharge hydrographs for each SLR scenario can be found in **Figure 9-282** for the 100-year storm, **Figure 9-283** for the 25-year storm, **Figure 9-284** for the 10-year storm, and **Figure 9-285** for the 5-year storm. Downstream flows for S26 comprise of the discharge from S26_S and S26_P in addition to overtopping of the structure, if applicable.

Figure 9-282. 12-Hour Moving Average of Flows at S26 for the 100- year Design Storm

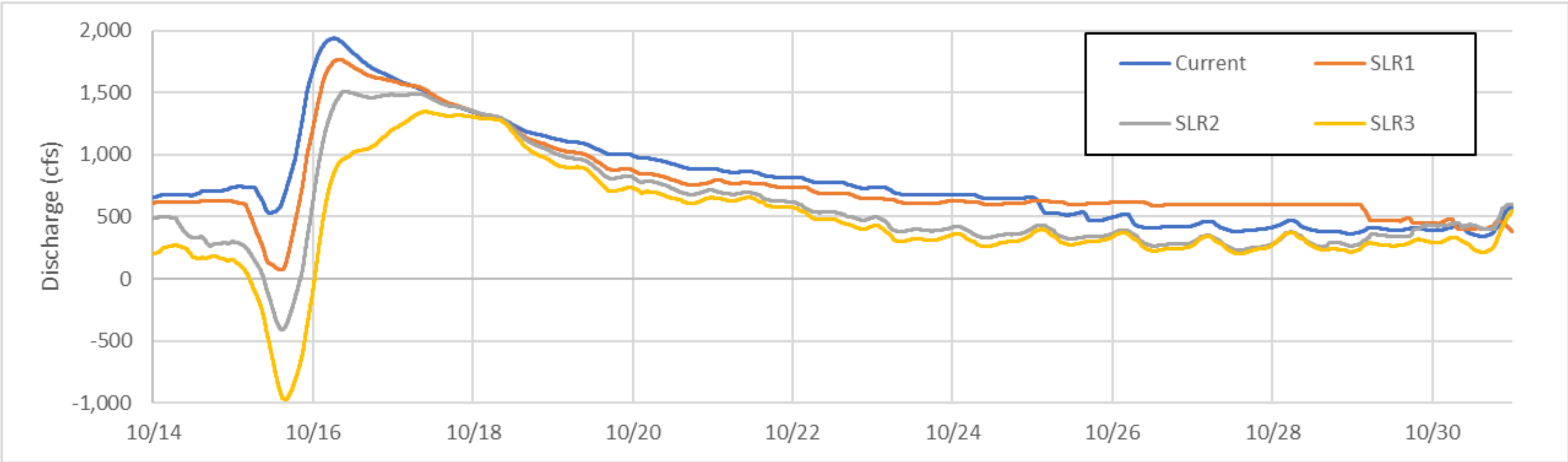


Figure 9-283. 12-Hour Moving Average of Flows at S26 for the 25-year Design Storm

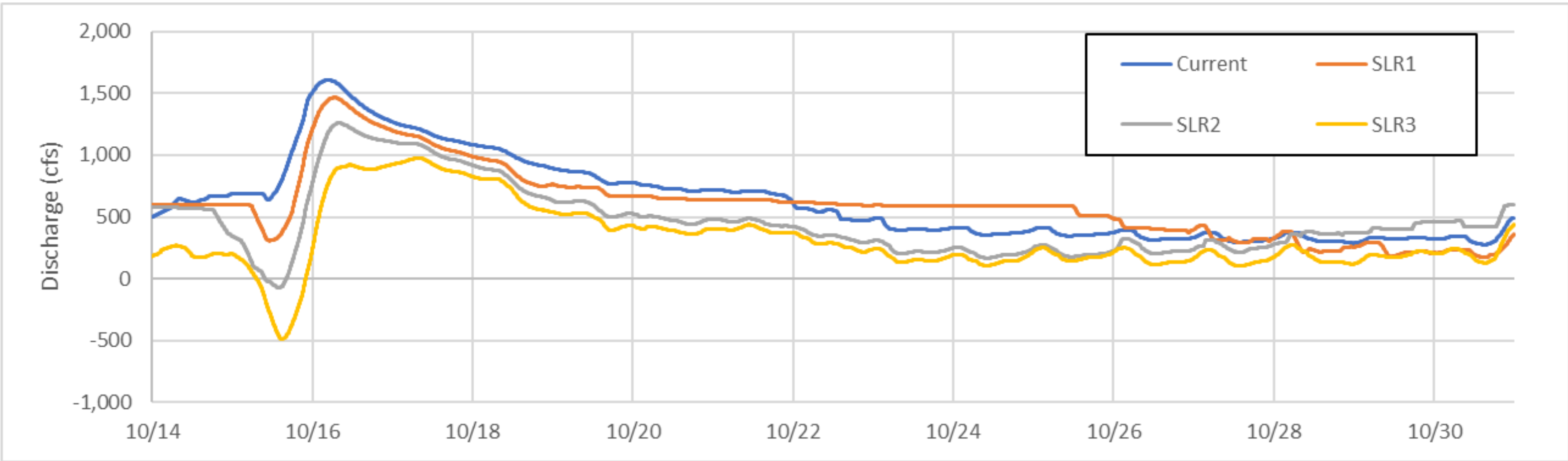


Figure 9-284. 12-Hour Moving Average of Flows at S26 for the 10-year Design Storm

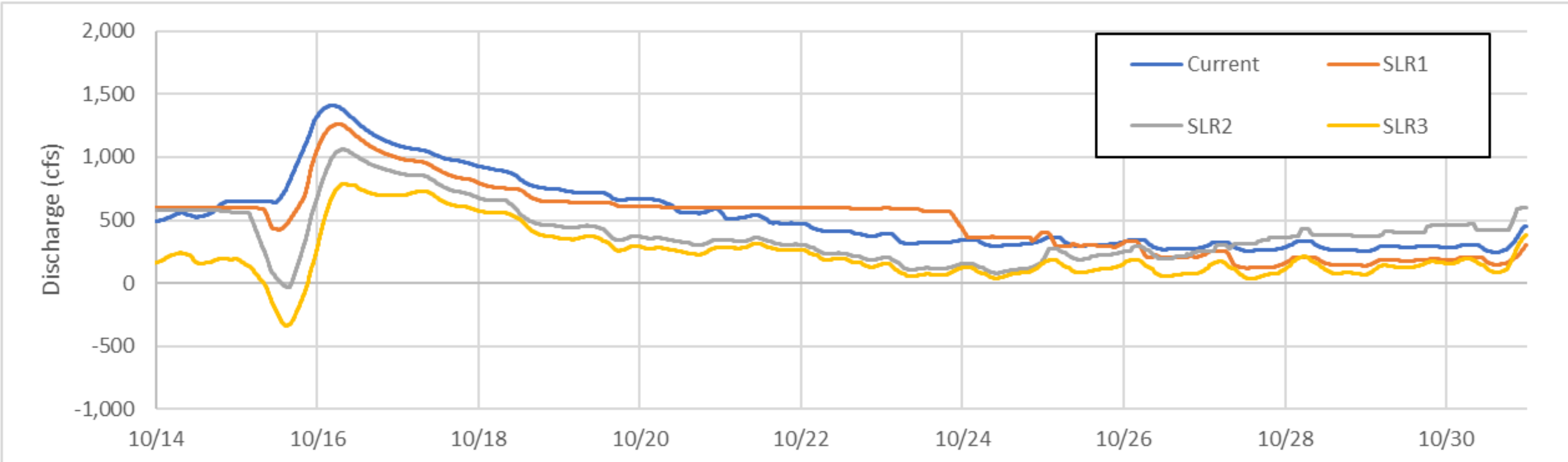
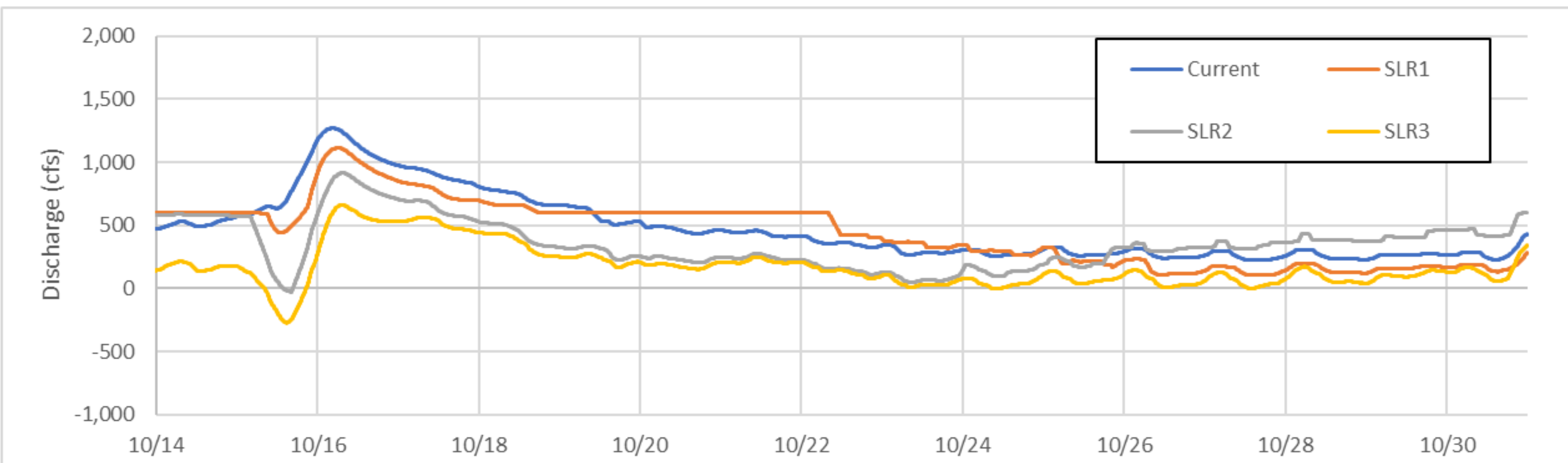


Figure 9-285. 12-Hour Moving Average of Flows at S26 for the 5-year Design Storm

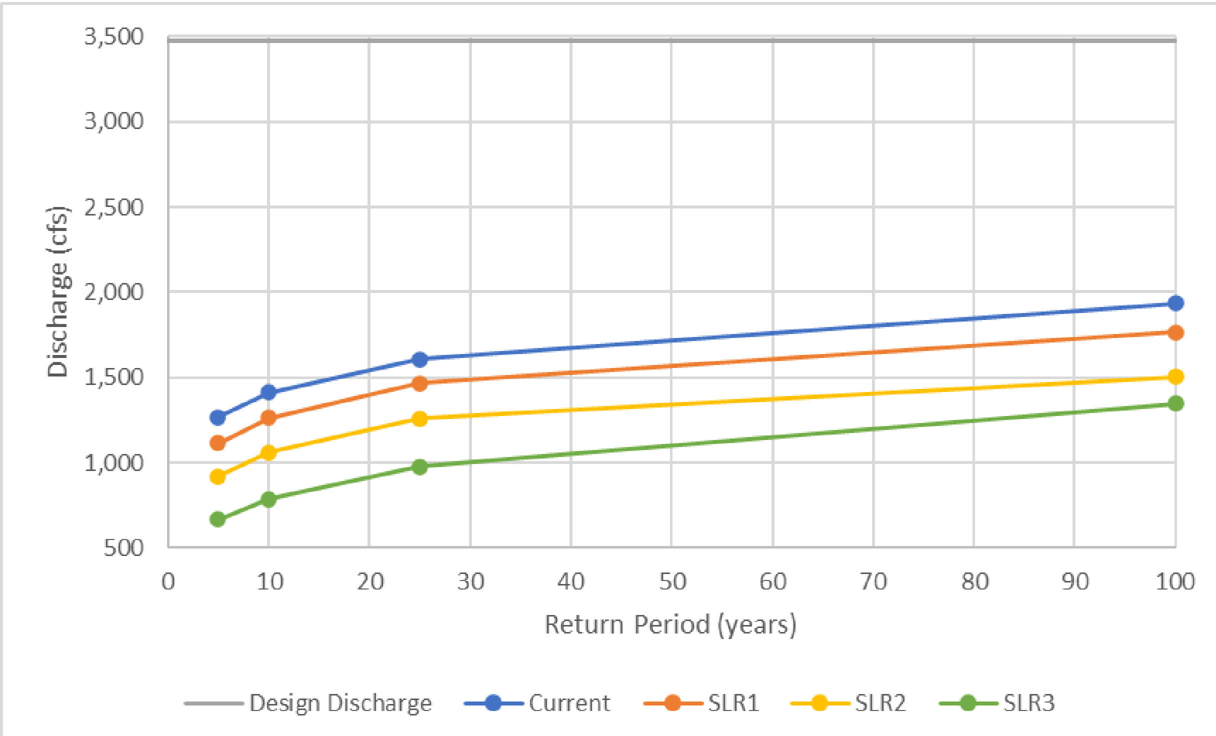


The instantaneous and 12-hour moving average peak discharges for all the design storm event return periods and SLR scenarios are shown in **Table 9-94**. In addition, the percentage difference between the current conditions 12-hour moving average peak discharge and each future conditions SLR scenario is calculated for all simulations. The 12-hour moving average peak discharges are also shown in **Figure 9-286**. The peak 12-hour moving average discharge decreases with increasing SLR scenario. As discussed previously, all peak discharges from these simulations fall significantly short of the design discharge at S26.

Table 9-94. Peak Discharge Summary at S26

Design Storm Return Period	Scenario	Peak Discharge (CFS)		12-hour Moving Average Peak Flow Reduction Percentage
		Instantaneous	12-hour Moving Average	
100-Year	**Current**	2095.31	1903.10	N/A
	SLR1	2101.14	1766.11	7.20%
	SLR2	1850.34	1505.11	20.91%
	SLR3	1688.27	1344.77	29.34%
25-Year	**Current**	1706.72	1587.32	N/A
	SLR1	1645.51	1464.17	7.76%
	SLR2	1511.67	1259.10	20.68%
	SLR3	1282.36	975.86	38.52%
10-Year	**Current**	1500.38	1398.19	N/A
	SLR1	1416.90	1261.03	9.81%
	SLR2	1281.29	1060.77	24.13%
	SLR3	1127.64	786.33	43.76%
5-Year	**Current**	1358.96	1259.76	N/A
	SLR1	1269.66	1112.80	11.67%
	SLR2	1123.80	917.03	27.21%
	SLR3	944.86	663.98	47.29%

Figure 9-286. Peak 12-Hour Moving Discharge at S26



9.5.5. C6 – PM #5 FREQUENCY OF FLOODING

The maximum overland depth was extracted for each design storm event and evaluated for the C6 Watershed. **Table 9-95** tabulates the flood inundation area in square miles for the C6 Watershed. The total area of the C6 Watershed for this analysis was calculated as 52.9 square miles (slight variations in total area from the District total area are due to estimating basin shape along a coarse grid).

The total area of the C6 Watershed considered Urban is 33.4 square miles, or 63% urban. **Table 9-96** tabulates the flood inundation area in square miles for the urban areas within the C6 Watershed. This table shows that the greatest distribution of flooding depths in the watershed are between zero and 0.25 feet of flooding, which can be considered nuisance flooding. **Figure 9-287** shows the urban inundation with the same incremental flooding depths as the table for the 100-year storm event, and **Figure 9-288** does the same for the 25-year storm. This provides a clear view of how stages are increasing with sea level rise in this watershed. In addition, all three SLR conditions for the 100-year storm show a lower area of flooding at the 0.25 to 0.5 feet range than the current conditions, which is due to more areas being inundated at greater depths.

Table 9-97 shows the percentage of the total urban areas in the C6 Watershed that are below the flooding depth. This data shows that 59% of the total urban areas in the C6 Watershed have 3 inches or greater of flooding depth (0.25 feet) for current conditions, which only increases by 1% for SLR1, and then jumps up to 2.5 % increase for the SLR2 and 3% for the SLR3 condition. For current conditions 38 % of the watershed is inundated by 6 inches or greater (0.5 feet) for the 100-year design storm event, this also shows an accelerating increase with increasing SLR conditions.

This watershed is likely experiencing both tertiary drainage issues caused by low-lying basins and secondary drainage issues caused by high stages in secondary canals such as NW 58th Street Canal, as discussed further in PM #6.

Figure 9-289 and **Figure 9-290** are maps of the maximum overland depth over the entire C6 Watershed for the 25-year and 100-year 3-day design storm events, respectively. Water areas, such as existing lakes and ponds, are masked in black. **Figure 9-291** and **Figure 9-292** provide the same maps with the non-urban areas masked out, to provide a concise picture of how urban areas are impacted within the watershed. As evident in these figures, a majority of the inundated areas in the C6 basin are non-urban.

Figure 9-293, **Figure 9-294**, and **Figure 9-295** show the difference in overland flooding for the C6 Watershed between the current conditions and future sea level rise conditions for the SLR +1 foot, SLR +2 feet, and SLR +3 feet simulations, respectively. Because the rainfall is the same in all the 100-year storm event simulations, these difference maps remove any overland flooding caused by rainfall and show how much is now impacted by rising seas in terms of direct flooding from the canals, or reduced drainage capacity due to higher stages in the primary and secondary canals. Where higher canal stages are due to reduced discharge capacity at the structure as well as structure backflow due to overtopping and structure bypass, as discussed in the previous sections.

Maps of the maximum overland depth over the entire C6 Watershed, and only urban areas, for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current conditions and future SLR scenarios are provided in **Appendix C**. Also provided in **Appendix C** are the differences in overland flooding maps for the 25-year, 10-year, and 5-year design storms.

Table 9-95. Incremental Flood Inundation Area (sq. mi.) in the C6 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	16.03	24.02	28.12	30.86	16.44	24.41	28.64	31.28	15.33	23.17	27.47	30.04	14.05	21.47	25.68	28.19
0.25 =< Depth < 0.50	8.19	5.19	4.19	3.53	8.06	5.39	4.14	3.58	7.89	5.58	4.28	3.75	7.48	5.65	4.54	3.99
0.50 =< Depth < 0.75	4.28	3.60	2.89	2.39	4.43	3.54	2.97	2.54	4.56	3.63	3.08	2.58	4.71	3.89	3.28	2.84
0.75 =< Depth < 1.00	3.34	2.58	2.08	1.80	3.44	2.59	2.25	2.24	3.58	2.71	2.22	2.06	3.65	2.90	2.32	2.08
1.00 =< Depth < 1.25	2.59	1.82	1.63	1.40	2.63	2.11	2.21	2.11	2.77	2.02	1.94	2.05	3.13	2.16	1.84	1.58
1.25 =< Depth < 1.50	1.81	1.52	1.54	1.66	1.94	2.10	2.15	2.32	2.10	1.78	2.14	2.20	2.19	1.58	1.63	1.99
1.50 =< Depth < 1.75	1.46	1.70	1.79	1.67	1.68	2.19	2.38	1.77	1.53	2.09	2.36	2.24	1.66	1.70	2.24	2.52
1.75 =< Depth < 2.00	1.48	1.85	1.66	1.64	2.07	2.38	1.32	0.71	1.70	2.47	1.93	1.23	1.39	2.23	2.42	1.94
2.00 =< Depth < 2.25	1.84	1.65	1.55	1.27	2.39	1.22	0.54	0.46	2.14	1.83	0.88	0.53	1.82	2.38	1.50	1.03
2.25 =< Depth < 2.50	1.81	1.45	1.00	0.61	1.94	0.58	0.42	0.26	2.43	0.92	0.45	0.38	2.39	1.44	0.87	0.54
2.50 =< Depth < 2.75	1.66	0.96	0.49	0.38	0.95	0.39	0.26	0.30	1.41	0.45	0.35	0.25	1.97	0.82	0.46	0.36
2.75 =< Depth < 3.00	1.09	0.50	0.31	0.25	0.52	0.32	0.28	0.18	0.76	0.37	0.33	0.30	1.15	0.48	0.34	0.33
3.00 =< Depth	7.36	6.06	5.69	5.44	6.43	5.71	5.36	5.18	6.74	5.92	5.50	5.30	7.33	6.21	5.81	5.54

Total Basin Area = 52.9 square miles

Table 9-96. Incremental Flood Inundation Area (sq. mi.) for Urban Areas in the C6 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 =< Depth < 0.25	13.40	20.18	23.53	25.68	13.41	19.89	23.28	25.38	12.53	18.93	22.37	24.41	11.51	17.61	20.96	22.96
0.25 =< Depth < 0.50	7.12	4.34	3.35	2.74	6.68	4.35	3.29	2.84	6.54	4.49	3.42	3.02	6.25	4.55	3.67	3.24
0.50 =< Depth < 0.75	3.60	2.92	2.23	1.78	3.62	2.90	2.39	1.88	3.70	3.00	2.54	2.02	3.79	3.20	2.73	2.32
0.75 =< Depth < 1.00	2.79	1.95	1.48	1.18	2.83	2.04	1.51	1.23	2.97	2.18	1.65	1.36	3.02	2.42	1.81	1.58
1.00 =< Depth < 1.25	1.98	1.24	0.93	0.62	2.10	1.37	1.05	0.77	2.27	1.50	1.09	0.89	2.59	1.66	1.31	0.95
1.25 =< Depth < 1.50	1.22	0.82	0.52	0.34	1.44	0.89	0.66	0.46	1.58	0.96	0.77	0.61	1.73	1.07	0.80	0.73
1.50 =< Depth < 1.75	0.87	0.53	0.32	0.26	0.91	0.65	0.44	0.28	0.99	0.70	0.54	0.39	1.22	0.79	0.64	0.54
1.75 =< Depth < 2.00	0.64	0.36	0.26	0.26	0.67	0.46	0.24	0.14	0.75	0.53	0.32	0.22	0.81	0.59	0.48	0.33
2.00 =< Depth < 2.25	0.41	0.28	0.25	0.17	0.54	0.26	0.13	0.12	0.54	0.38	0.22	0.13	0.62	0.46	0.29	0.24
2.25 =< Depth < 2.50	0.35	0.23	0.15	0.08	0.38	0.16	0.13	0.05	0.46	0.23	0.11	0.09	0.52	0.33	0.22	0.15
2.50 =< Depth < 2.75	0.30	0.17	0.06	0.08	0.24	0.10	0.06	0.05	0.35	0.13	0.09	0.06	0.41	0.23	0.12	0.09
2.75 =< Depth < 3.00	0.18	0.07	0.07	0.05	0.14	0.08	0.04	0.05	0.21	0.09	0.06	0.05	0.27	0.13	0.09	0.05
3.00 =< Depth	0.56	0.34	0.26	0.20	0.45	0.27	0.22	0.16	0.52	0.33	0.24	0.20	0.70	0.40	0.29	0.25

Total Basin Urban Area = 33.4 square miles

Table 9-97. Percentage of Total Watershed Area with Inundated Area for Urban Areas in the C6 Watershed

Flooding Depth (ft)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0.00	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 0.25	59.9	39.6	29.6	23.2	59.9	40.5	30.4	24.1	62.5	43.3	33.1	27.0	65.6	47.3	37.3	31.3
>= 0.50	38.6	26.6	19.6	15.0	39.9	27.5	20.5	15.6	42.9	29.9	22.8	18.0	46.9	33.7	26.3	21.6
>= 0.75	27.8	17.9	12.9	9.7	29.1	18.8	13.4	9.9	31.9	21.0	15.2	11.9	35.6	24.1	18.2	14.7
>= 1.00	19.5	12.1	8.5	6.1	20.6	12.7	8.9	6.3	23.0	14.4	10.3	7.8	26.5	16.9	12.7	9.9
>= 1.25	13.6	8.4	5.7	4.3	14.3	8.6	5.7	4.0	16.2	10.0	7.0	5.2	18.8	12.0	8.8	7.1
>= 1.50	9.9	5.9	4.1	3.3	10.0	5.9	3.7	2.6	11.5	7.1	4.7	3.4	13.6	8.8	6.4	4.9
>= 1.75	7.3	4.3	3.2	2.5	7.3	4.0	2.4	1.7	8.5	5.0	3.1	2.2	10.0	6.4	4.5	3.3
>= 2.00	5.4	3.3	2.4	1.7	5.2	2.6	1.7	1.3	6.3	3.4	2.2	1.6	7.5	4.6	3.1	2.3
>= 2.25	4.2	2.4	1.6	1.2	3.6	1.8	1.3	0.9	4.6	2.3	1.5	1.2	5.7	3.2	2.2	1.6
>= 2.50	3.1	1.7	1.1	1.0	2.5	1.4	0.9	0.8	3.3	1.6	1.2	0.9	4.1	2.3	1.5	1.2
>= 2.75	2.2	1.2	1.0	0.7	1.8	1.1	0.8	0.6	2.2	1.2	0.9	0.7	2.9	1.6	1.2	0.9
>= 3.00	1.7	1.0	0.8	0.6	1.4	0.8	0.7	0.5	1.6	1.0	0.7	0.6	2.1	1.2	0.9	0.7

Figure 9-287. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C6 Watershed for the 100-year Design Storm

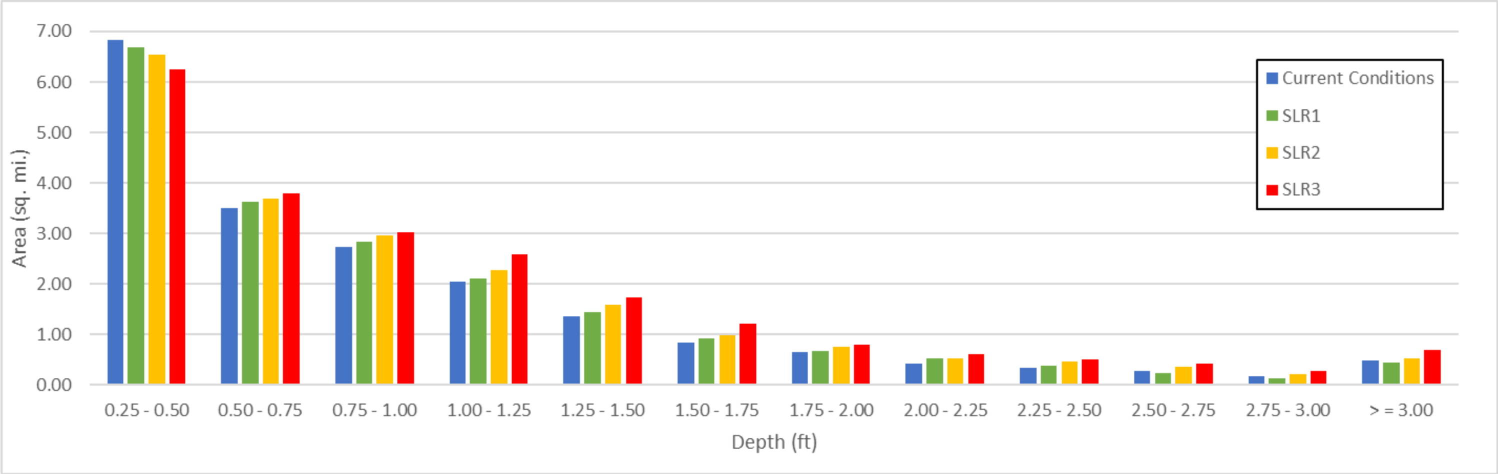


Figure 9-288. Incremental Flood Inundation Above 0.25ft for Urban Areas in the C6 Watershed for the 25-year Design Storm

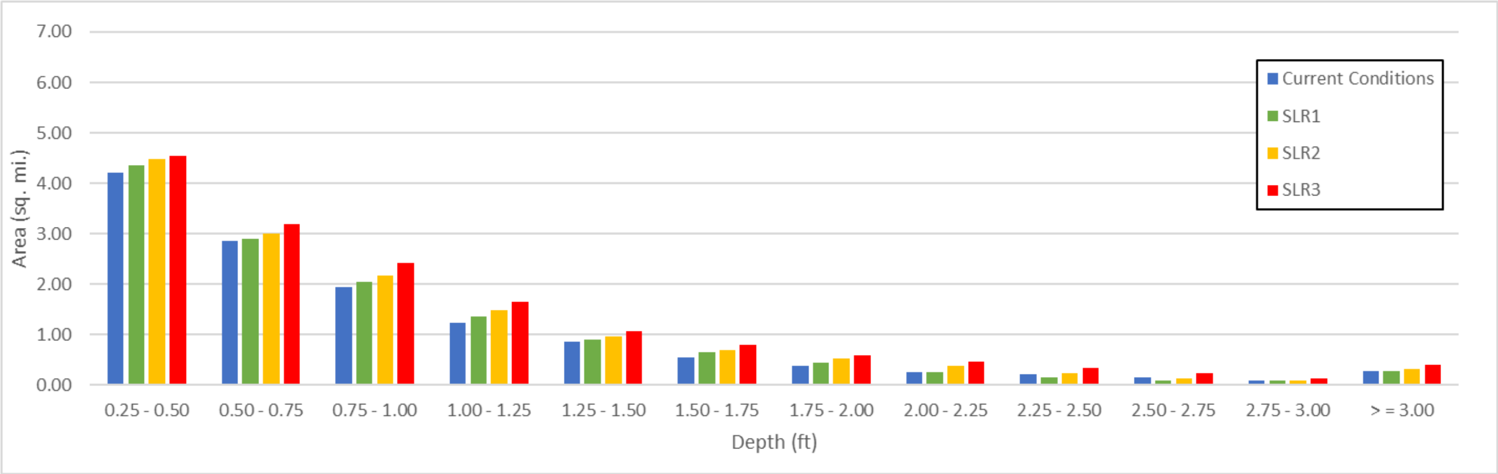


Figure 9-289. Maximum Overland Flood Depth for the Current Conditions 25-year 3-Day Design Storm in the C6 Watershed

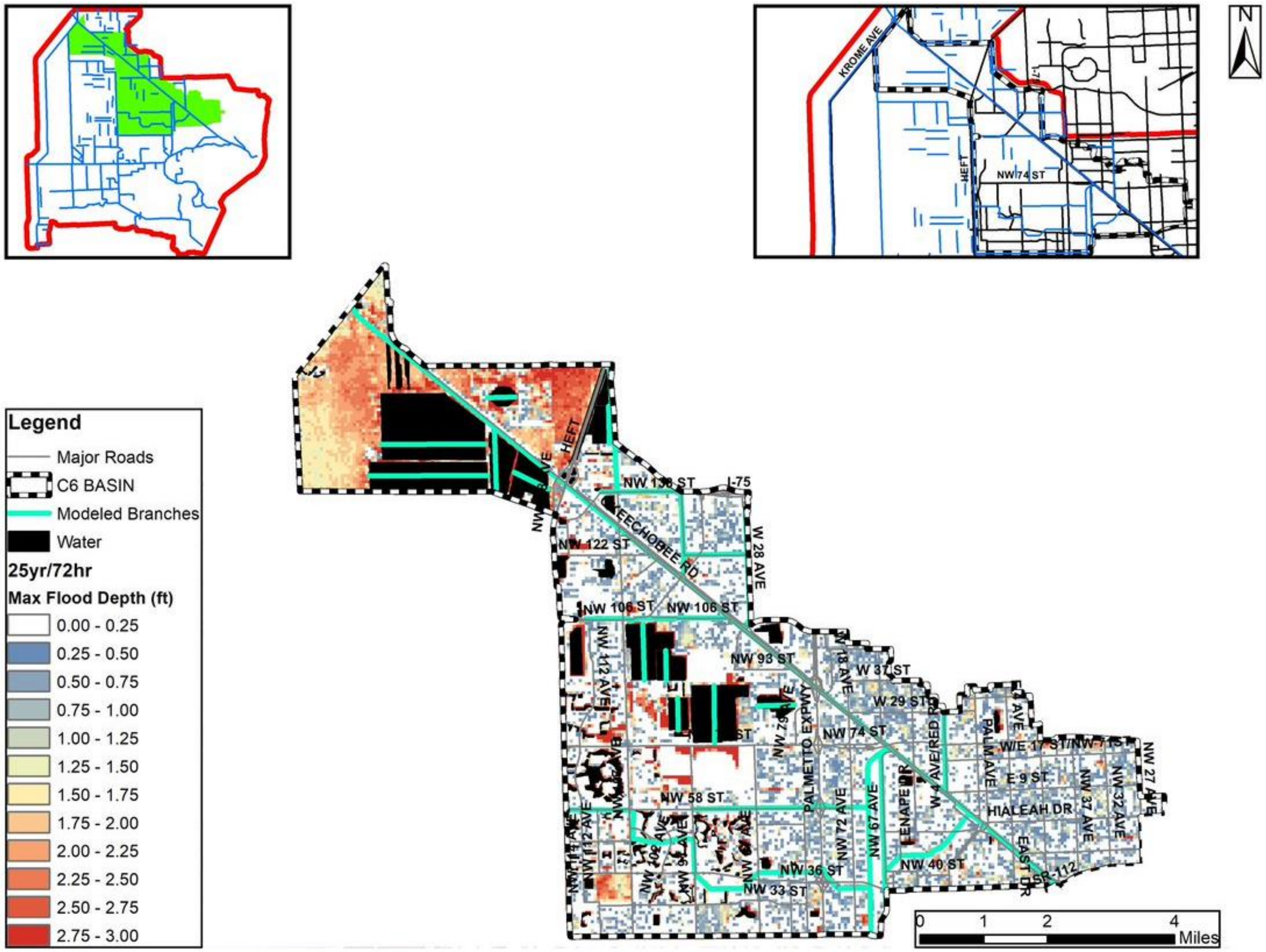


Figure 9-290. Maximum Overland Flood Depth for the Current Conditions 100-year 3-Day Design Storm in the C6 Watershed

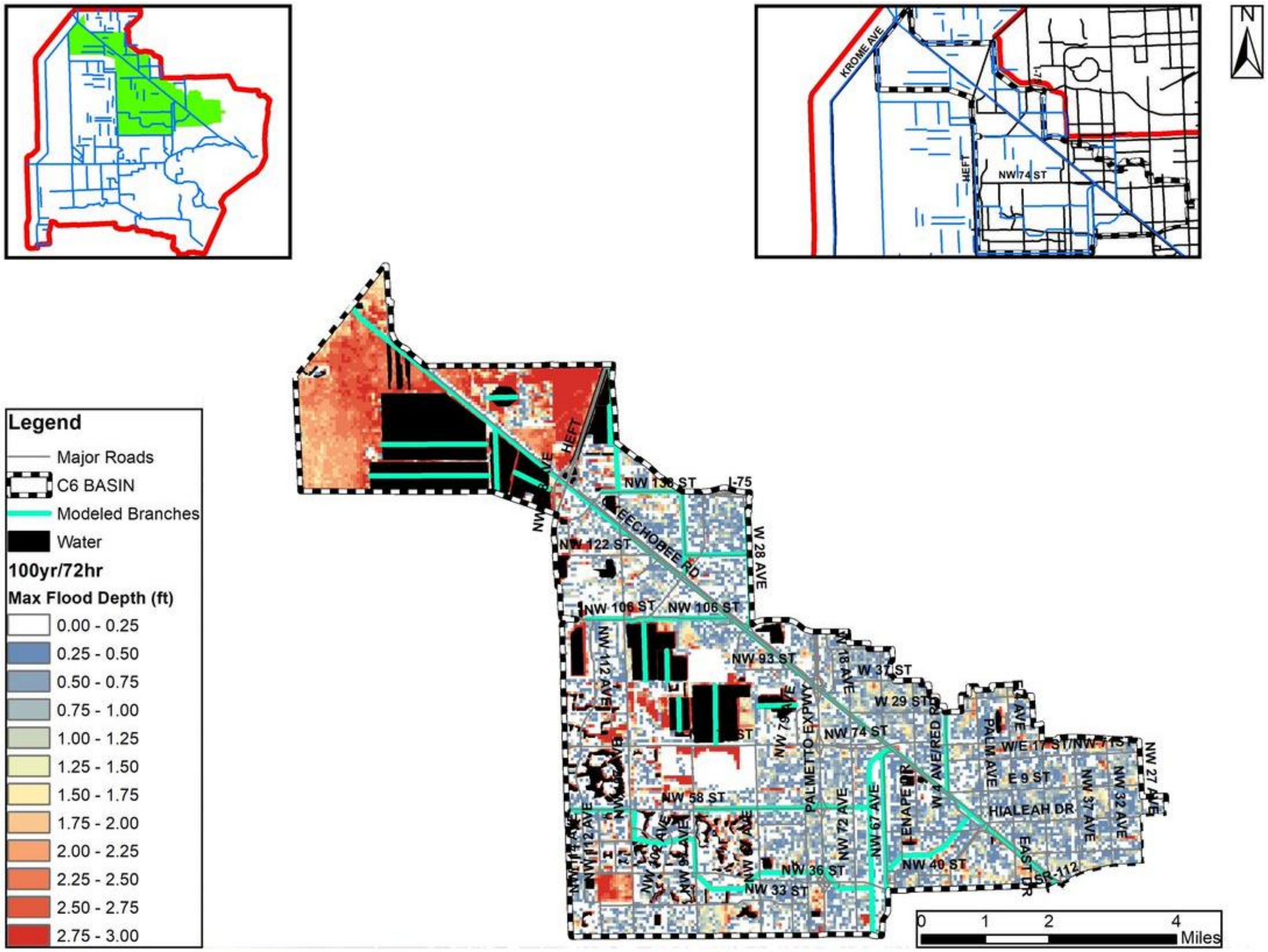


Figure 9-291. Maximum Overland Flood Depth for the Current Conditions 25-year 3-Day Design Storm for Urban Areas in the C6 Watershed

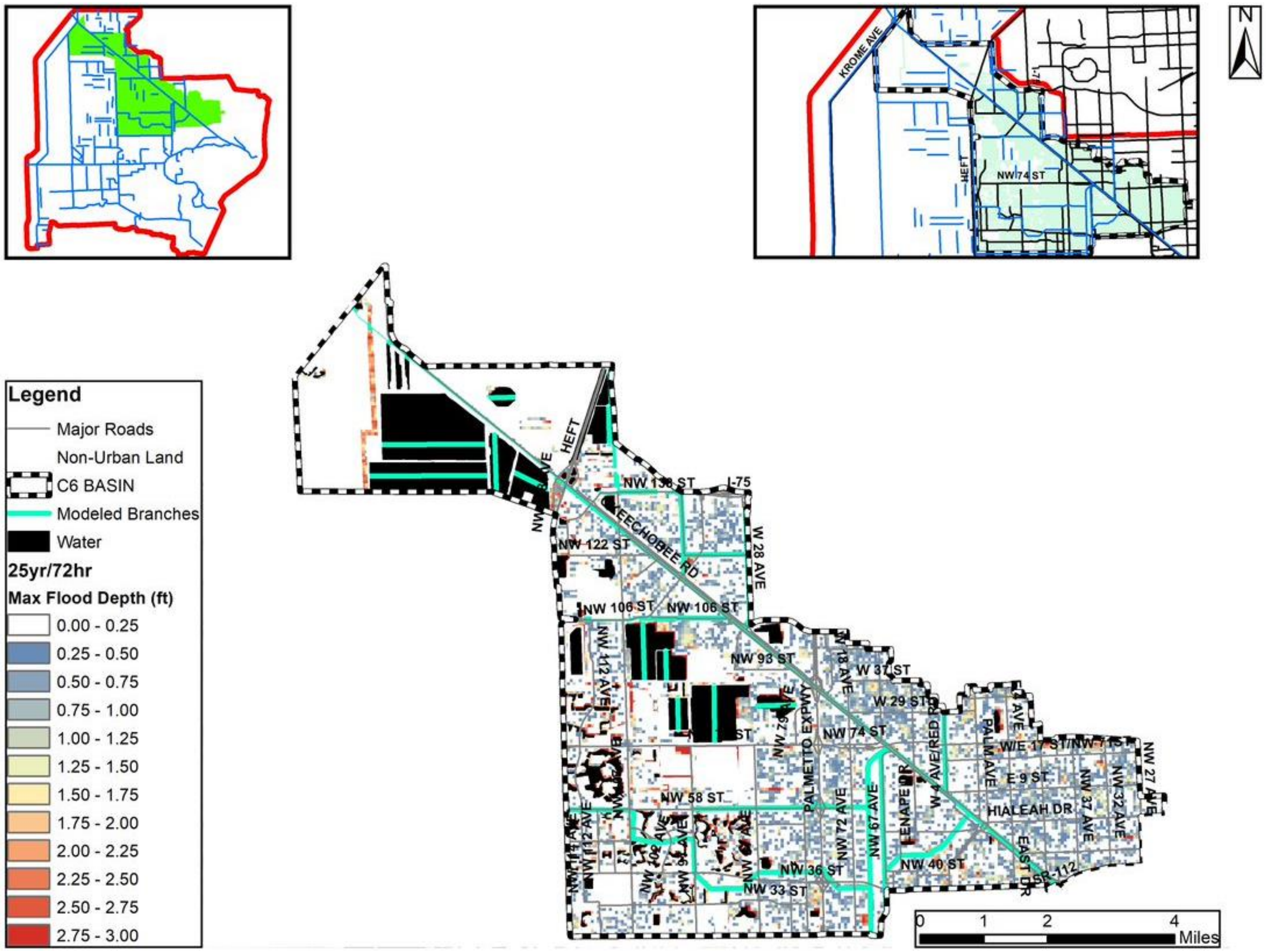


Figure 9-292. Maximum Overland Flood Depth for the Current Conditions 100-year 3-Day Design Storm for Urban Areas in the C6 Watershed

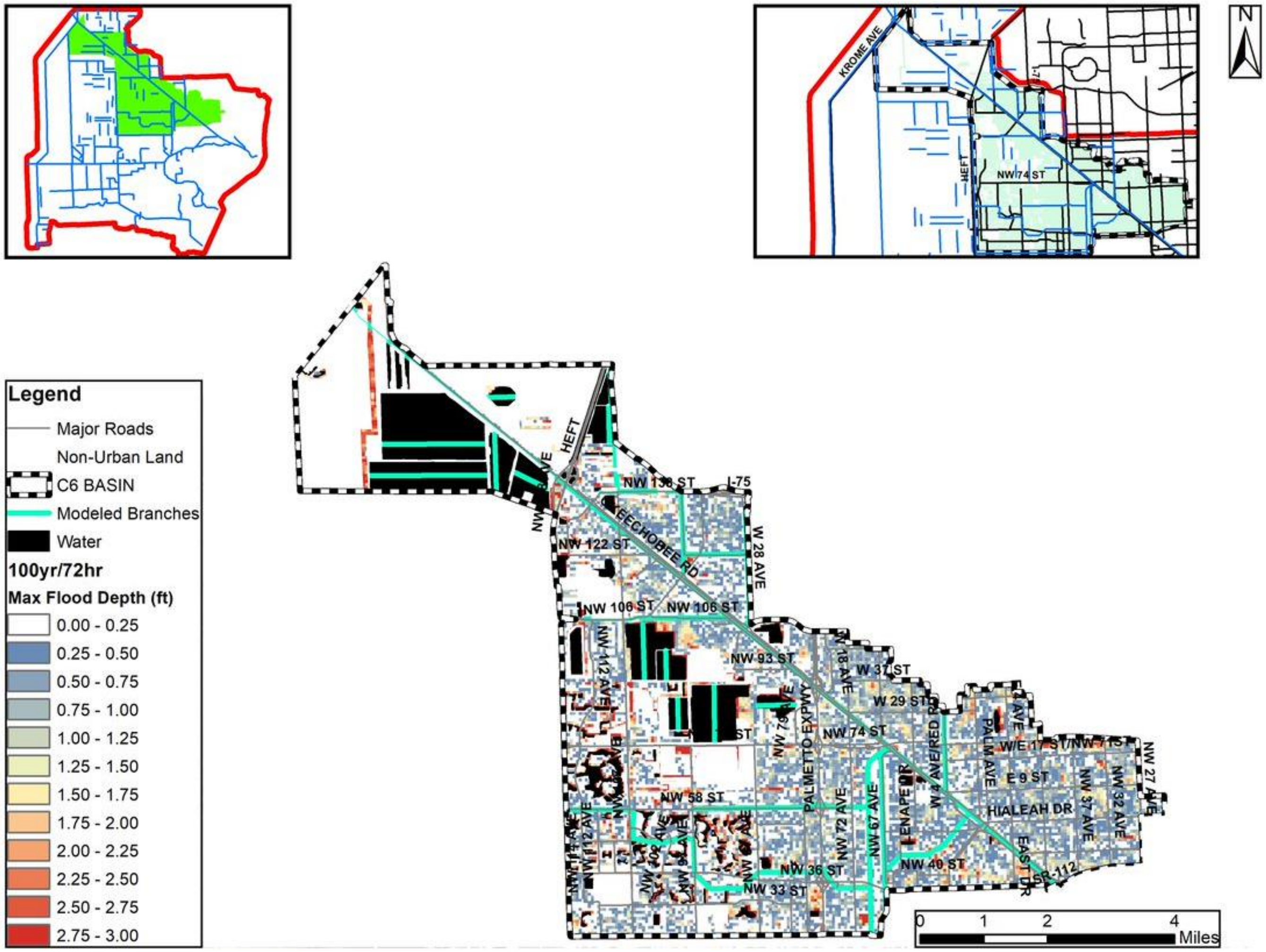


Figure 9-293. Urban Flooding Depth Difference of SLR +1ft and Current Conditions for the 100-year Storm in the C6 Watershed

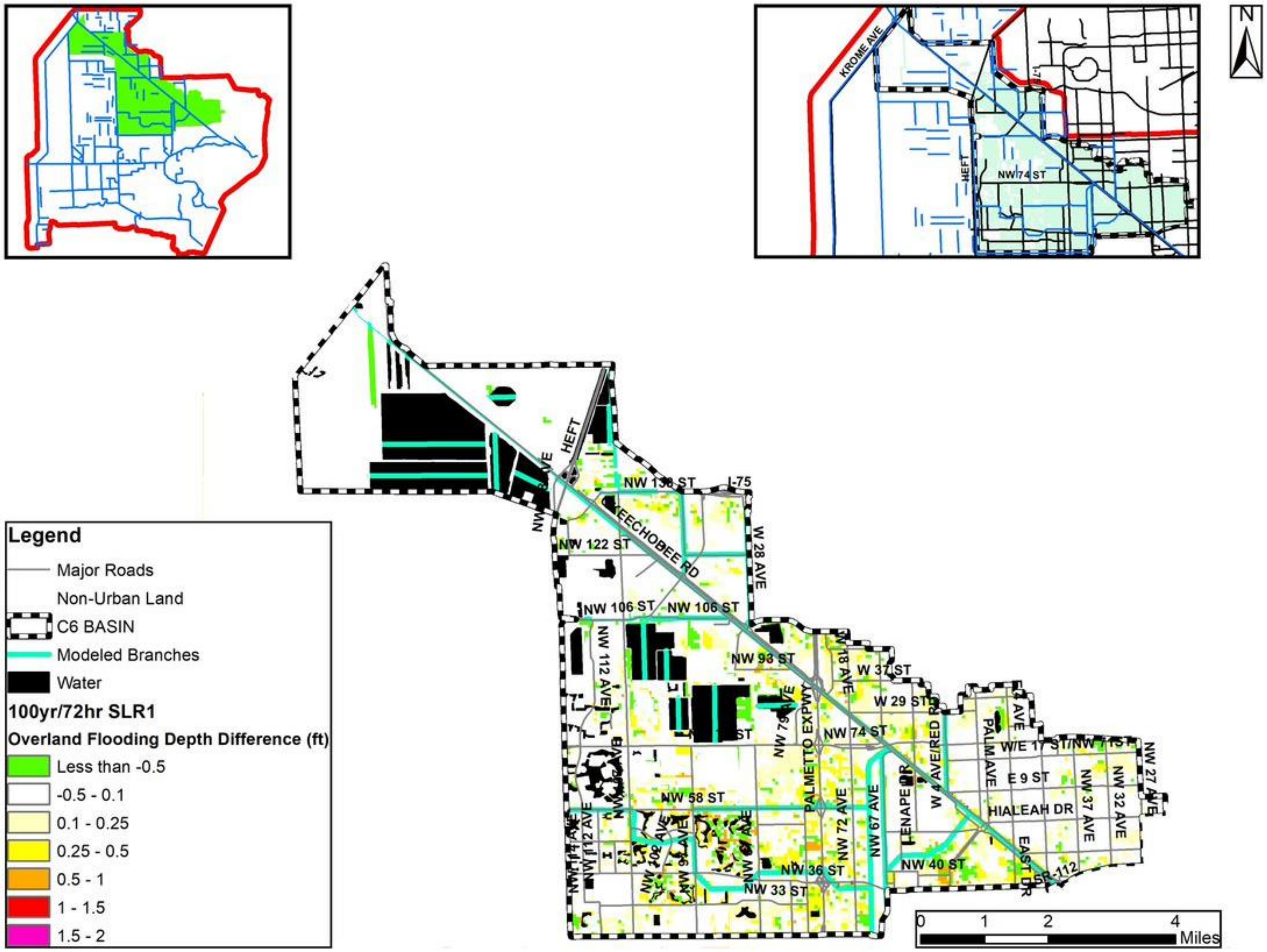


Figure 9-294. Urban Flooding Depth Difference of SLR +2ft and Current Conditions for the 100-year Storm in the C6 Watershed

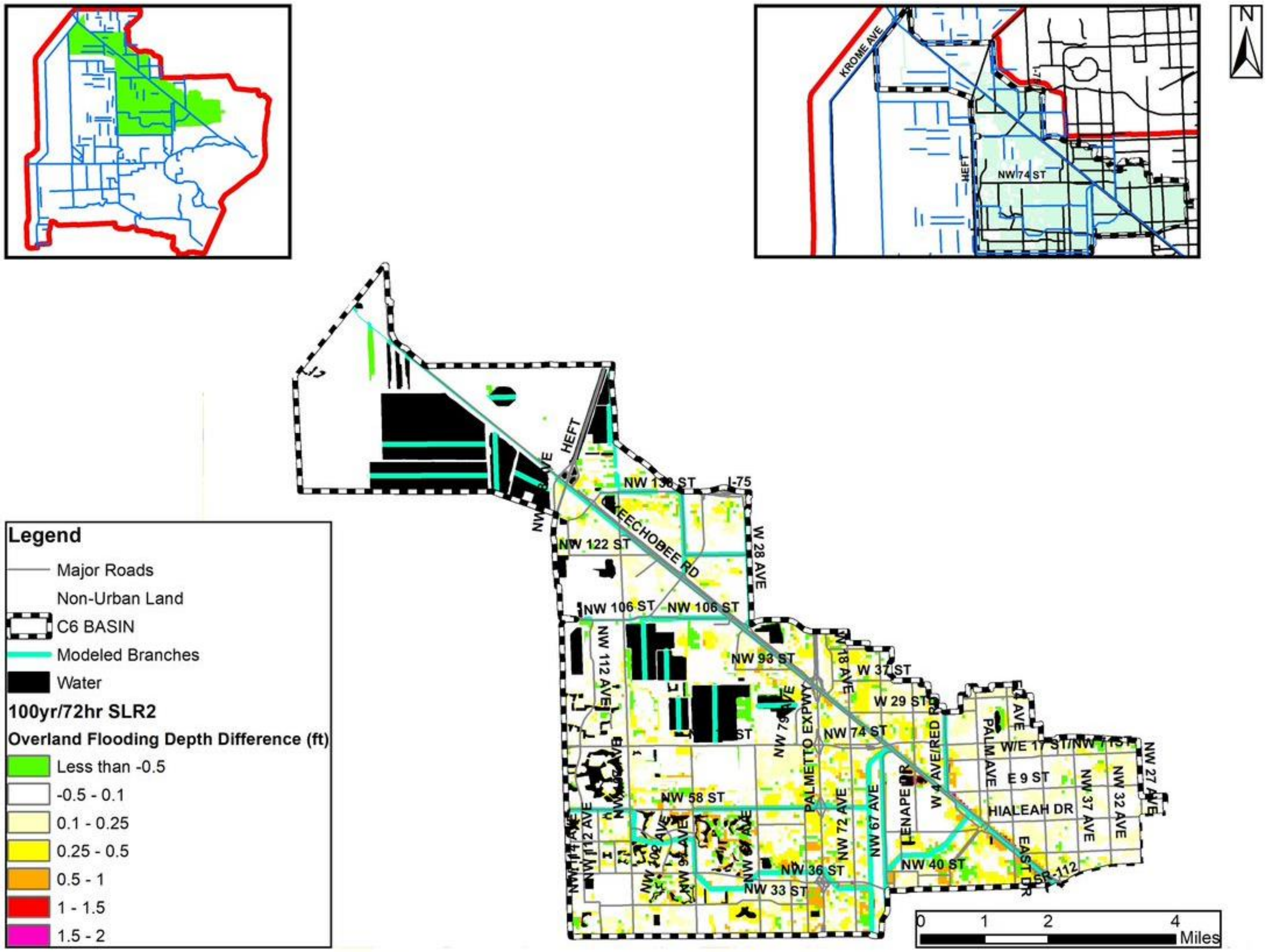
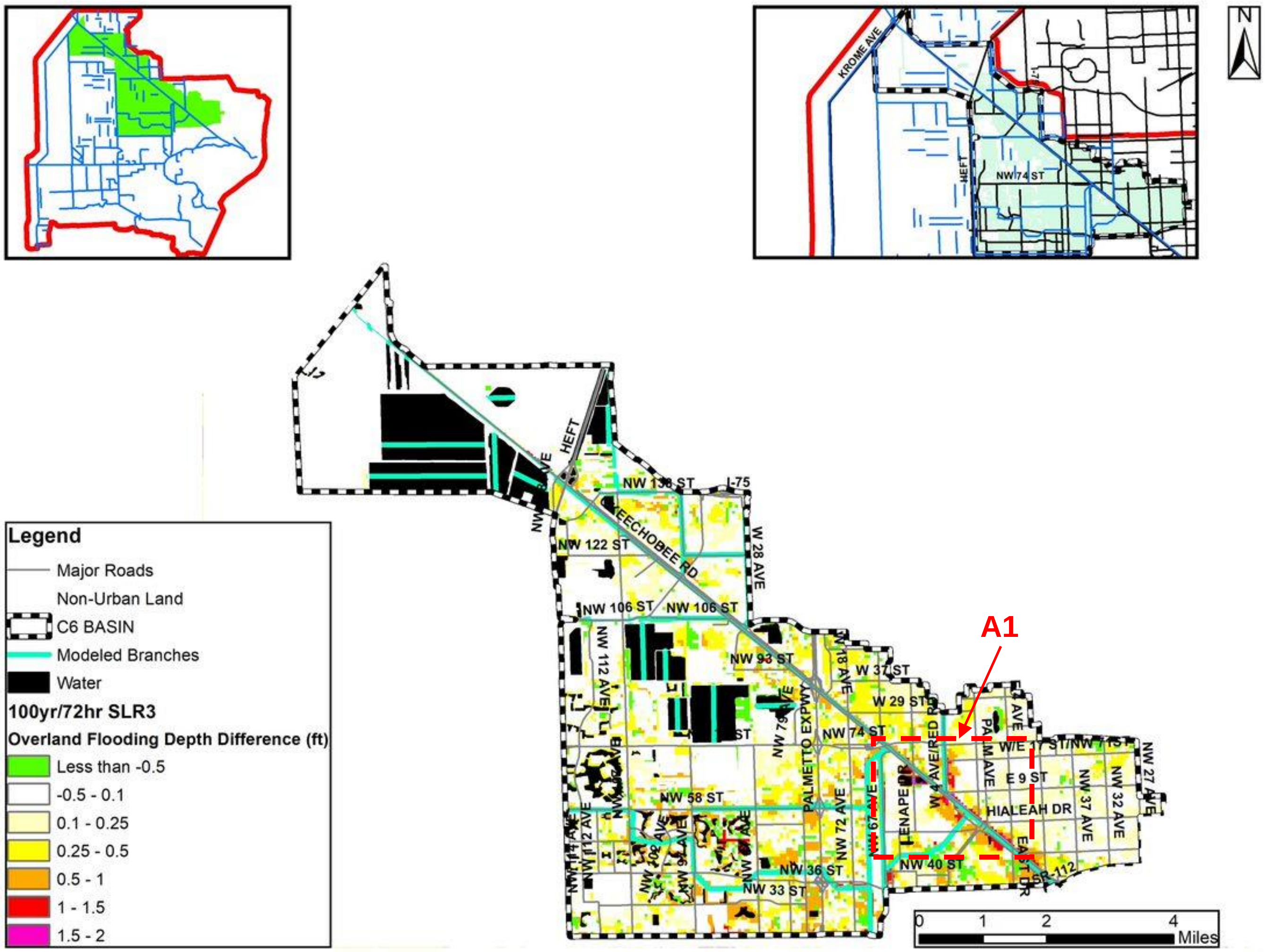
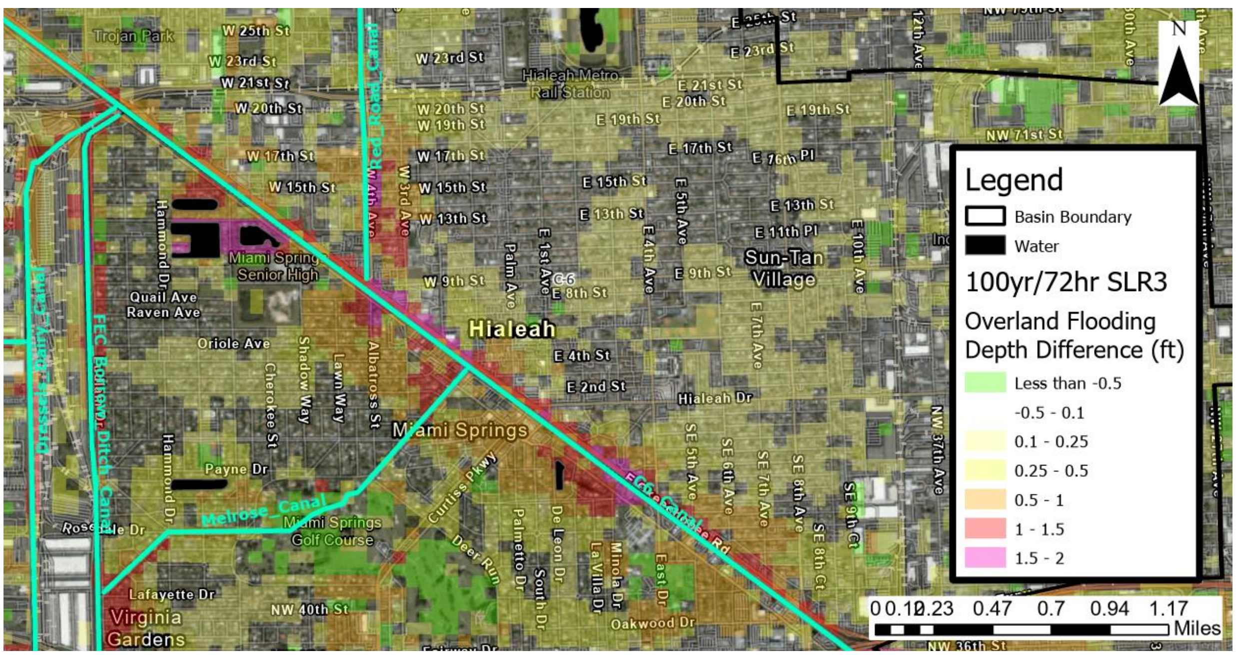


Figure 9-295. Urban Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C6 Watershed



Similar to the results seen in the C2 Basin, a majority of the most inundated areas in the C6 basin are non-urban. Additionally, there are parks and golf courses in the urban areas of the C6 basin that are shown to have higher maximum flood depths than surrounding residential areas such as Miami Springs Golf Course and Stafford Park, just north of Miami International Airport. Areas of concern that show flooding for extended periods of time are discussed in **Section 9.5.6**. The area in Hialeah near the intersection of the Red Road Canal and the C6 Canal (indicated by 'A1' in **Figure 9-295**) may be impacted by over 2 feet above the current conditions with the SLR3 simulation. An enlarged view of this neighborhood is shown in **Figure 9-296**

Figure 9-296. Flooding Depth Difference of SLR +3ft and Current Conditions for the 100-year Storm in the C6 Watershed – Hialeah



9.5.6. C6 – PM #6 DURATION OF FLOODING

9.5.6.1. CANAL FLOOD DURATION

In discussions with water managers at SFWMD, it was found that no target stage is available for S26 to determine duration of flooding in the canal. However, the optimum head elevation, as determined by the structure information sheet, is a reasonable elevation for establishing the duration of each storm event for the C6 Canal under current conditions. The optimum head elevation is 2.5 ft-NGVD or 0.96 ft-NAVD using the structure conversion of (-)1.54 feet. However, this Reference Elevation would not be acceptable under future SLR conditions, as the storms, and even the normal wet season canal elevations may be higher than 0.96 ft-NAVD.

A new Reference Elevation was established based on the off-trigger elevation used for the S25B Pump and, by proxy, the S26 Pump, which will only run if the S25B pump is on. The trigger is that the stages at MRMS1 in the C6 Canal should be less than 3.2 ft-NAVD. **Table 9-98** provides the canal flood duration of each storm event, as determined from canal stages, for each SLR condition. The initiation of the flood event is determined to be when the stages exceed the optimum HW prior to the peak, and the finalization is determined to be the first time the stages drop below the optimum HW after the peak.

Table 9-98. Storm Duration Indicated at S26

Design Storm	Duration (hours)				
	Current Conditions with 2.23 ft-NAVD Reference Elev.	Current Conditions with 3.43 ft-NAVD Reference Elev.	SLR1	SLR2	SLR3
100-Year	71.6	13.9	21.9	70.3	308.9
25-Year	60.2	4.2	16.0	46.5	334.2
10-Year	47.6	0.5	11.8	44.8	333.8
5-Year	44.6	0.0*	9.5	43.6	333.8

*Canal stages do not recede past the Reference Elevation after the storm and therefore the storm duration is longer than the values provided.

Figure 9-297 shows the hydrographs at S26_H station, comparing the SLR conditions for each design storm. As described in the table and shown in the figure, the Reference Elevation of 3.2 ft-NAVD is shown on the graphs as a grey line.

Figure 9-297. Stages at S26_H and Storm Duration for the C6 Watershed

9.5.6.2. WATERSHED FLOOD DURATION

Table 9-99 tabulates the total area of flood inundation (in square miles) per flood duration range for all areas in the C6 Watershed and **Table 9-100** does this for all urban areas within the C6 Watershed. This table shows that the greatest distribution of flooding duration in the watershed is between zero and one hour of flooding, which can be considered insignificant. Flooding is also distributed in the 12 to 144-hour range for the 100-year and 25-year storms, indicating a wide range of flood durations for this watershed, depending on the location to major canals, as represented in the model.

Table 9-101 calculates the percentage of the total urban areas within the C6 Watershed that are inundated by 3 inches or more for the flood duration indicated. Because the flood duration greater than 1 hour includes all areas inundated with 3 inches or more, this row shows the same percentage as that in **Table 9-97** in PM #5. Additionally, **Figure 9-298, Figure 9-299, Figure 9-300,** and **Figure 9-301** provide a graphical view of this data, plotting the flood duration against the percentage of area inundated for the urban areas of the C6 Watershed for the 100-year, 25-year, 10-year, 5-year storm events, respectively. These graphics also include the percent increase above the current conditions on the secondary axis, to visualize how the flood duration is changing with each SLR condition. For the 100-year design storms, the percentage of the watershed that is flooded for 72 hours shows the greatest increase above current conditions with each SLR condition. However, the greatest increase is shown at the 48-hour mark for the 25-year storm events. This indicates a greater impact from reduced drainage capacity of the watershed as a result of higher canal elevations.

Figure 9-302 and **Figure 9-303** provide flood duration maps for the C6 Watershed for overland flooding depths exceeding 0.25 feet for the current conditions 5-year and 100-year 3-day design storms, respectively. Water areas, such as existing lakes and ponds, are masked in black. **Figure 9-304** and **Figure 9-305** provide the flood duration for only the urban areas within the C2 Watershed (non-urban areas are masked out), to provide a concise picture of how urban areas are impacted within the watershed.

Figure 9-306, **Figure 9-307**, and **Figure 9-308** provide the difference in flood duration between the current conditions and SLR +1 foot, SLR +2 feet, and SLR +3 feet, respectively, for the 100-year storm event. When compared with the difference maps for flood depth, this can show that even for areas where flood depths are not increasing with future SLR conditions, the duration of flooding may increase due to the reduced ability of the area to drain to the receiving canals that are experiencing higher stages.

Table 9-99. Flood Duration per Area of Inundation (in sq. mi.) for the C6 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	16.03	24.02	28.12	30.86	16.44	24.41	28.64	31.28	15.33	23.17	27.47	30.04	14.05	21.47	25.68	28.19
1 to 2 hr.	2.60	0.60	0.27	0.19	2.43	0.61	0.27	0.20	2.21	0.57	0.26	0.18	1.97	0.61	0.23	0.21
2 to 4 hr.	0.86	0.58	0.40	0.41	0.92	0.63	0.46	0.35	0.81	0.65	0.46	0.40	0.79	0.61	0.34	0.33
4 to 8 hr.	0.93	0.85	0.75	0.57	0.93	0.87	0.73	0.59	0.85	0.78	0.66	0.59	0.67	0.78	0.67	0.54
8 to 12 hr.	0.60	0.65	0.58	0.56	0.63	0.65	0.54	0.50	0.59	0.62	0.56	0.49	0.52	0.45	0.43	0.45
12 to 24 hr.	2.48	2.17	1.94	1.53	2.53	2.17	1.86	1.59	2.22	2.10	1.68	1.45	1.86	1.83	1.64	1.41
24 to 48 hr.	3.30	2.84	2.32	1.83	3.39	2.81	2.19	1.94	3.20	2.58	2.12	1.84	2.75	2.45	2.06	1.75
48 to 72 hr.	2.49	1.77	1.36	1.25	2.44	1.84	1.59	1.18	2.51	1.80	1.47	1.16	2.40	1.83	1.46	1.21
72 to 96 hr.	1.76	1.34	1.07	0.77	1.84	1.42	1.02	0.87	1.88	1.29	1.04	0.85	1.93	1.22	0.99	0.86
96 to 144 hr.	2.55	1.88	1.30	0.88	2.65	1.81	1.50	1.07	2.53	1.87	1.45	1.27	2.52	1.88	1.50	1.23
144 to 192 hr.	1.83	1.13	0.73	0.58	1.84	1.26	0.78	0.56	1.92	1.33	1.10	0.81	2.01	1.37	1.10	0.83
192 to 240 hr.	1.33	0.64	0.49	0.33	1.28	0.74	0.48	0.37	1.36	1.03	0.68	0.52	1.46	1.06	0.72	0.66
240 to 336 hr.	1.39	0.94	0.65	0.50	1.70	0.97	0.73	0.55	2.19	1.33	0.88	0.66	2.31	1.55	1.20	0.90
336 to 420 hr.	2.64	1.53	1.05	0.85	4.39	3.37	2.89	2.67	4.78	3.47	2.91	2.55	5.72	3.99	3.15	2.68
420 hr.	12.14	12.01	11.90	11.81	9.54	9.37	9.26	9.17	10.56	10.32	10.19	10.12	11.97	11.83	11.76	11.69

Total Basin Area = 52.9 square miles

Table 9-100. Flood Duration per Area of Inundation (in sq. mi.) for Urban Areas in the C6 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
0 to 1 hr.	13.40	20.18	23.53	25.68	13.41	19.89	23.28	25.38	12.53	18.93	22.37	24.41	11.51	17.61	20.96	22.96
1 to 2 hr.	2.29	0.50	0.22	0.16	2.06	0.50	0.20	0.14	1.87	0.45	0.18	0.13	1.64	0.49	0.17	0.15
2 to 4 hr.	0.73	0.46	0.29	0.31	0.74	0.45	0.34	0.28	0.63	0.49	0.33	0.30	0.62	0.43	0.24	0.27
4 to 8 hr.	0.76	0.69	0.64	0.45	0.70	0.65	0.56	0.47	0.65	0.58	0.51	0.47	0.50	0.59	0.50	0.41
8 to 12 hr.	0.51	0.53	0.47	0.50	0.49	0.51	0.44	0.43	0.45	0.48	0.46	0.41	0.40	0.35	0.34	0.37
12 to 24 hr.	2.16	1.86	1.63	1.25	2.08	1.81	1.59	1.36	1.77	1.73	1.42	1.26	1.47	1.45	1.39	1.20
24 to 48 hr.	2.81	2.39	1.90	1.50	2.79	2.41	1.82	1.60	2.68	2.17	1.83	1.55	2.32	2.07	1.75	1.53
48 to 72 hr.	2.10	1.43	1.05	0.93	2.01	1.50	1.27	0.90	2.09	1.56	1.21	0.96	1.99	1.59	1.29	1.04
72 to 96 hr.	1.42	1.03	0.80	0.56	1.56	1.11	0.76	0.65	1.56	1.08	0.84	0.65	1.62	1.06	0.85	0.73
96 to 144 hr.	2.02	1.44	0.91	0.53	2.08	1.39	1.13	0.76	2.11	1.47	1.13	0.98	2.15	1.57	1.24	1.00
144 to 192 hr.	1.38	0.77	0.41	0.30	1.39	0.96	0.55	0.35	1.54	1.04	0.83	0.57	1.71	1.13	0.91	0.67
192 to 240 hr.	0.93	0.37	0.26	0.15	0.98	0.52	0.28	0.17	1.06	0.80	0.50	0.37	1.17	0.89	0.56	0.50
240 to 336 hr.	0.94	0.48	0.28	0.17	1.24	0.58	0.37	0.21	1.68	0.95	0.59	0.37	1.88	1.21	0.91	0.66
336 to 420 hr.	1.23	0.61	0.37	0.28	1.39	0.68	0.37	0.30	2.13	1.06	0.64	0.43	3.44	2.03	1.37	1.02
420 hr.	0.74	0.71	0.67	0.65	0.51	0.47	0.45	0.43	0.67	0.62	0.58	0.56	1.02	0.97	0.94	0.93

Total Basin Urban Area = 33.4 square miles

Table 9-101. Percentage of Total Area Inundated for Urban Areas in the C6 Watershed

Flooding Duration (hrs.)	Current Conditions Area (sq. mi.)				SLR1 Area (sq. mi.)				SLR2 Area (sq. mi.)				SLR3 Area (sq. mi.)			
	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year	100-Year	25-Year	10-Year	5-Year
>= 0 hr.	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
>= 1 hr.	59.9	39.6	29.6	23.2	59.9	40.5	30.4	24.1	62.5	43.3	33.1	27.0	65.6	47.3	37.3	31.3
>= 2 hr.	53.1	38.2	28.9	22.7	53.7	39.0	29.8	23.7	56.9	42.0	32.5	26.6	60.7	45.9	36.8	30.9
>= 4 hr.	50.9	36.8	28.1	21.8	51.5	37.6	28.7	22.8	55.0	40.5	31.5	25.7	58.8	44.6	36.1	30.1
>= 8 hr.	48.6	34.7	26.2	20.4	49.4	35.7	27.0	21.4	53.1	38.8	30.0	24.3	57.4	42.8	34.6	28.8
>= 12 hr.	47.1	33.1	24.8	18.9	48.0	34.2	25.7	20.2	51.7	37.4	28.6	23.1	56.2	41.8	33.6	27.7
>= 24 hr.	40.6	27.5	19.9	15.2	41.7	28.8	21.0	16.1	46.4	32.2	24.4	19.3	51.8	37.4	29.4	24.1
>= 48 hr.	32.2	20.4	14.2	10.7	33.4	21.6	15.5	11.3	38.4	25.7	18.9	14.6	44.8	31.2	24.2	19.6
>= 72 hr.	25.9	16.1	11.1	7.9	27.4	17.1	11.7	8.6	32.2	21.0	15.3	11.8	38.8	26.5	20.3	16.5
>= 96 hr.	21.7	13.1	8.7	6.2	22.7	13.8	9.4	6.6	27.5	17.8	12.8	9.8	34.0	23.3	17.8	14.3
>= 144 hr.	15.6	8.8	6.0	4.6	16.5	9.6	6.1	4.4	21.2	13.4	9.4	6.9	27.6	18.6	14.1	11.3
>= 192 hr.	11.5	6.5	4.7	3.7	12.3	6.7	4.4	3.3	16.6	10.3	6.9	5.2	22.5	15.3	11.3	9.3
>= 240 hr.	8.7	5.4	4.0	3.3	9.4	5.2	3.6	2.8	13.4	7.9	5.4	4.1	19.0	12.6	9.6	7.8
>= 336 hr.	5.9	3.9	3.1	2.8	5.7	3.4	2.5	2.2	8.4	5.0	3.6	3.0	13.3	9.0	6.9	5.8
>= 420 hr.	2.2	2.1	2.0	2.0	1.5	1.4	1.3	1.3	2.0	1.9	1.7	1.7	3.1	2.9	2.8	2.8

Figure 9-298. Percentage and Duration of Inundation for the C6 Watershed for the 100-year Storm Event

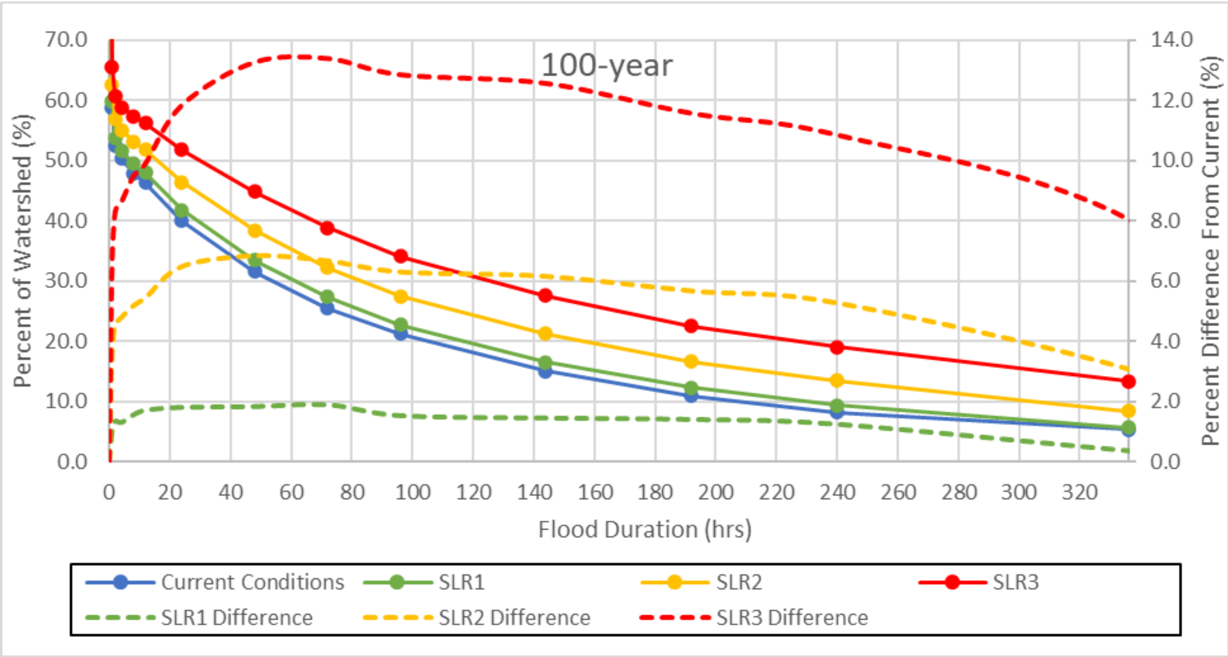


Figure 9-299. Percentage and Duration of Inundation for the C6 Watershed for the 25-year Storm Event

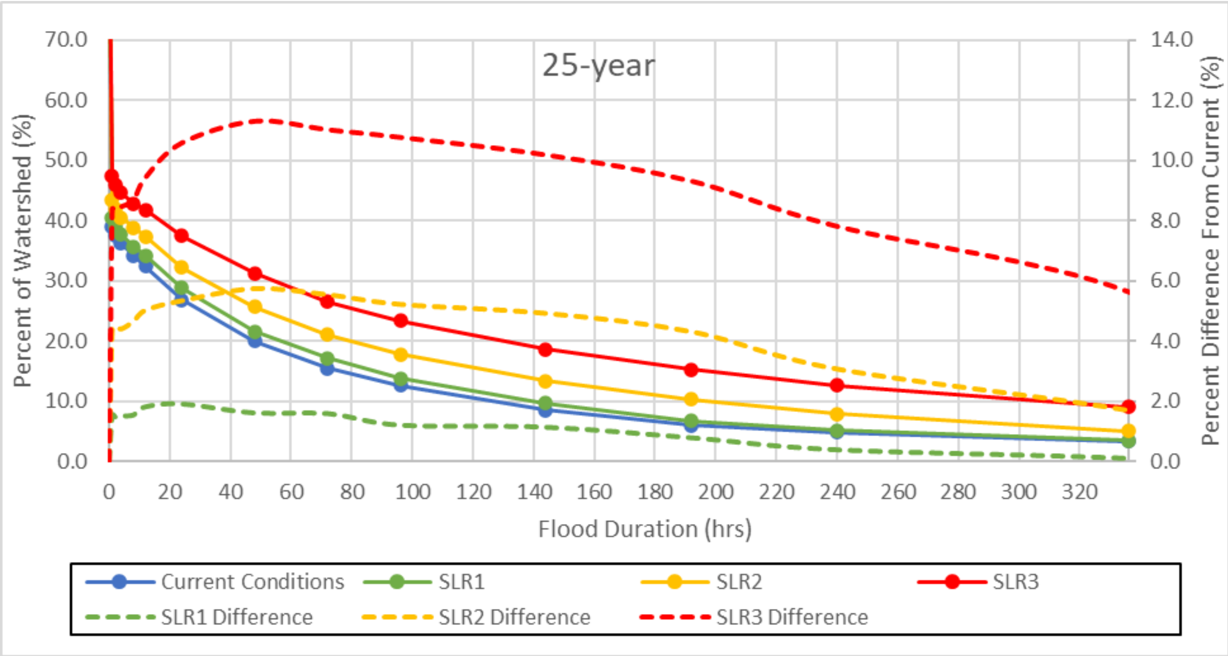


Figure 9-300. Percentage and Duration of Inundation for the C6 Watershed for the 10-year Storm Event

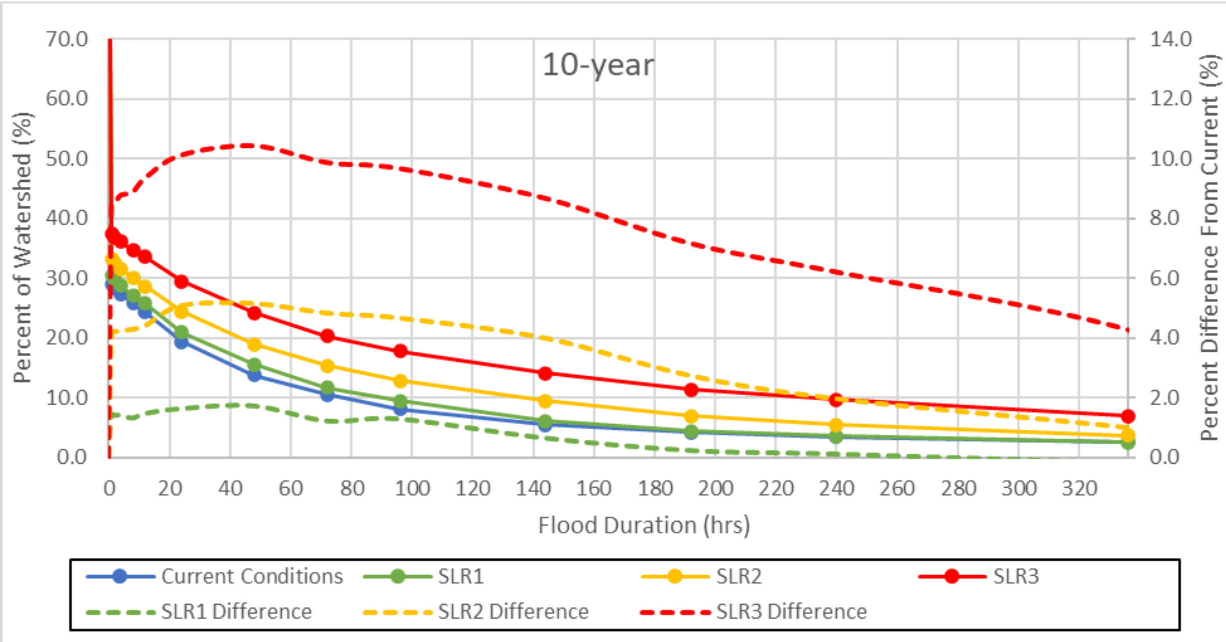


Figure 9-301. Percentage and Duration of Inundation for the C6 Watershed for the 5-year Storm Event

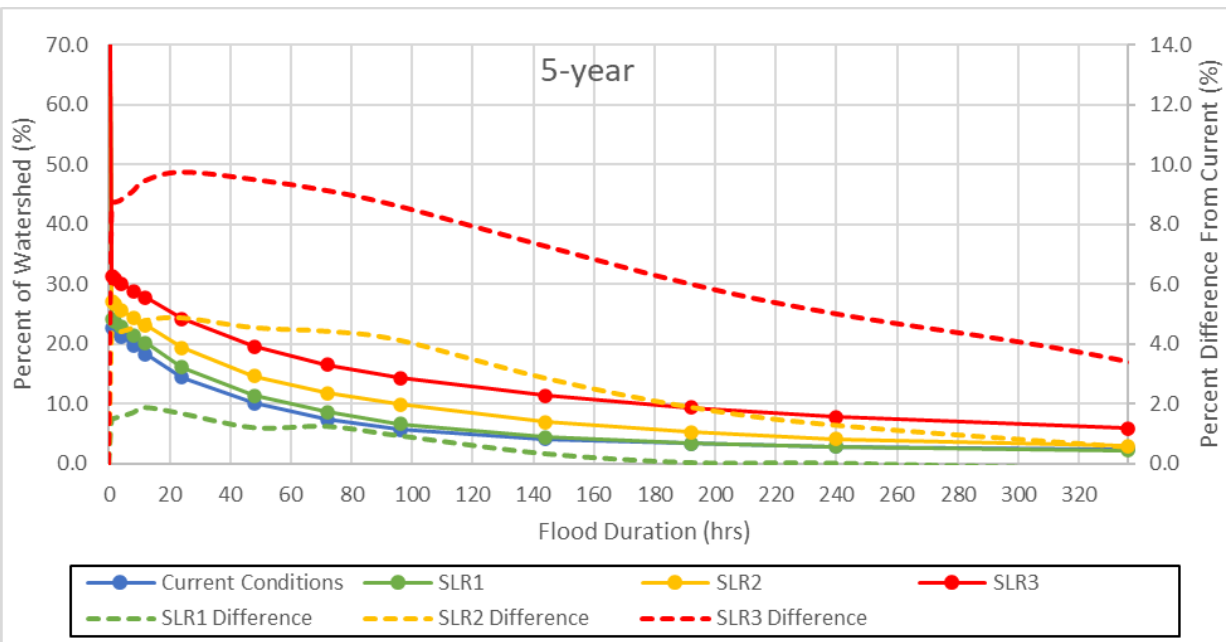


Figure 9-302. Flood Duration Map for the Current Conditions 25-year 3-day Design Storm for the C6 Watershed

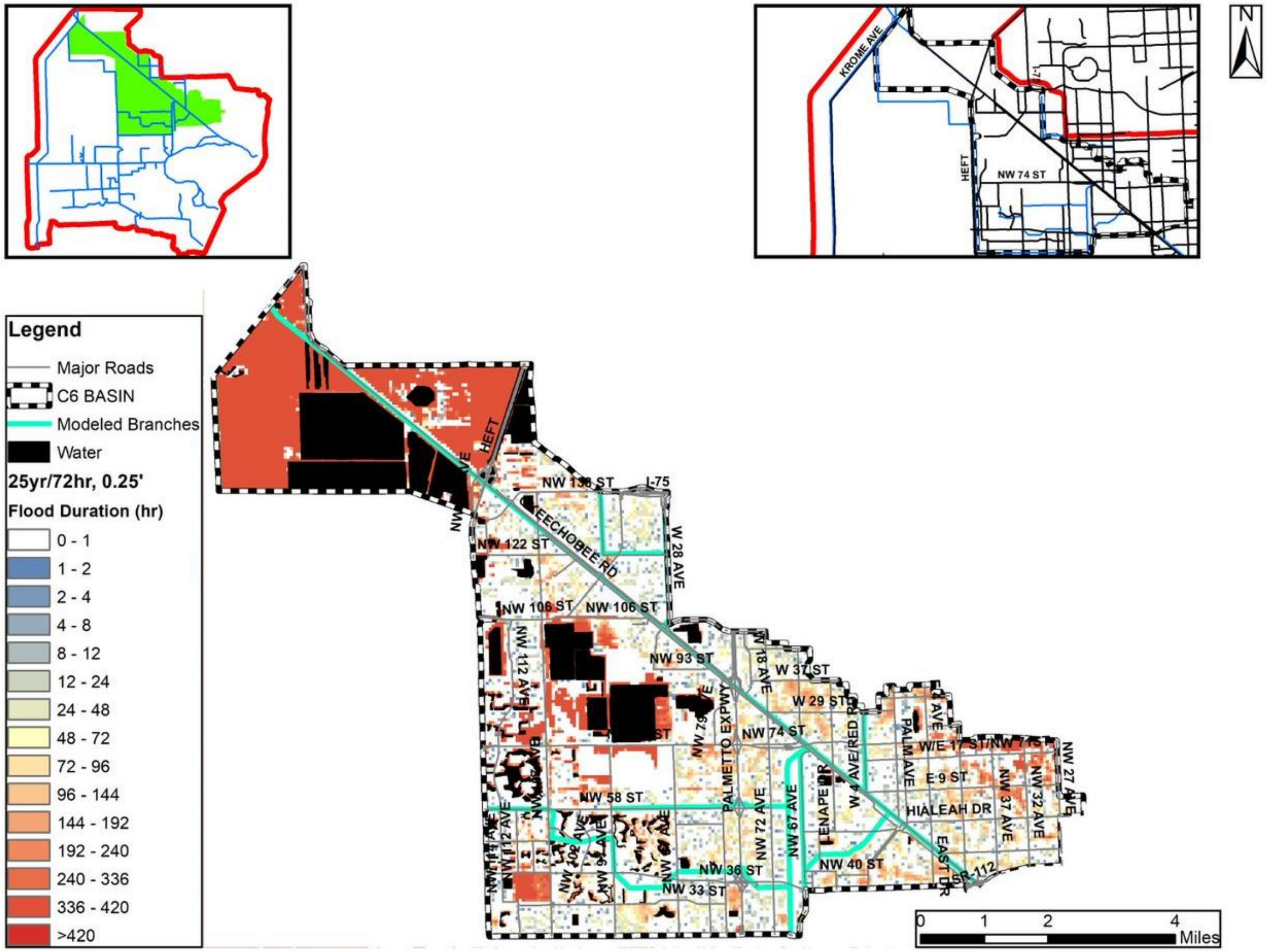


Figure 9-303. Flood Duration Map for the Current Conditions 100-year 3-day Design Storm for the C6 Watershed

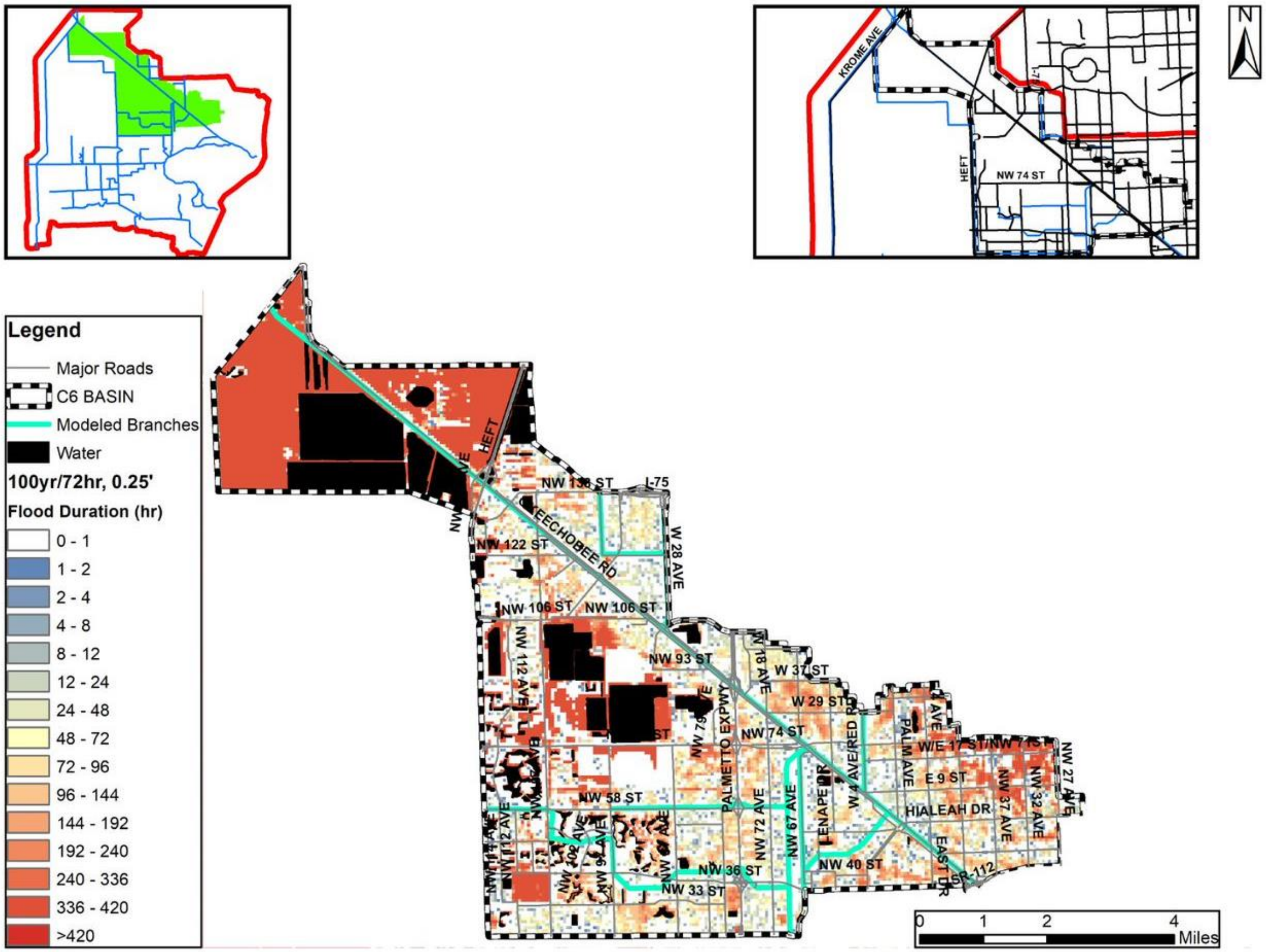


Figure 9-304. Flood Duration Map for the Current Conditions 25-year 3-day Design Storm for Urban Areas in the C6 Watershed

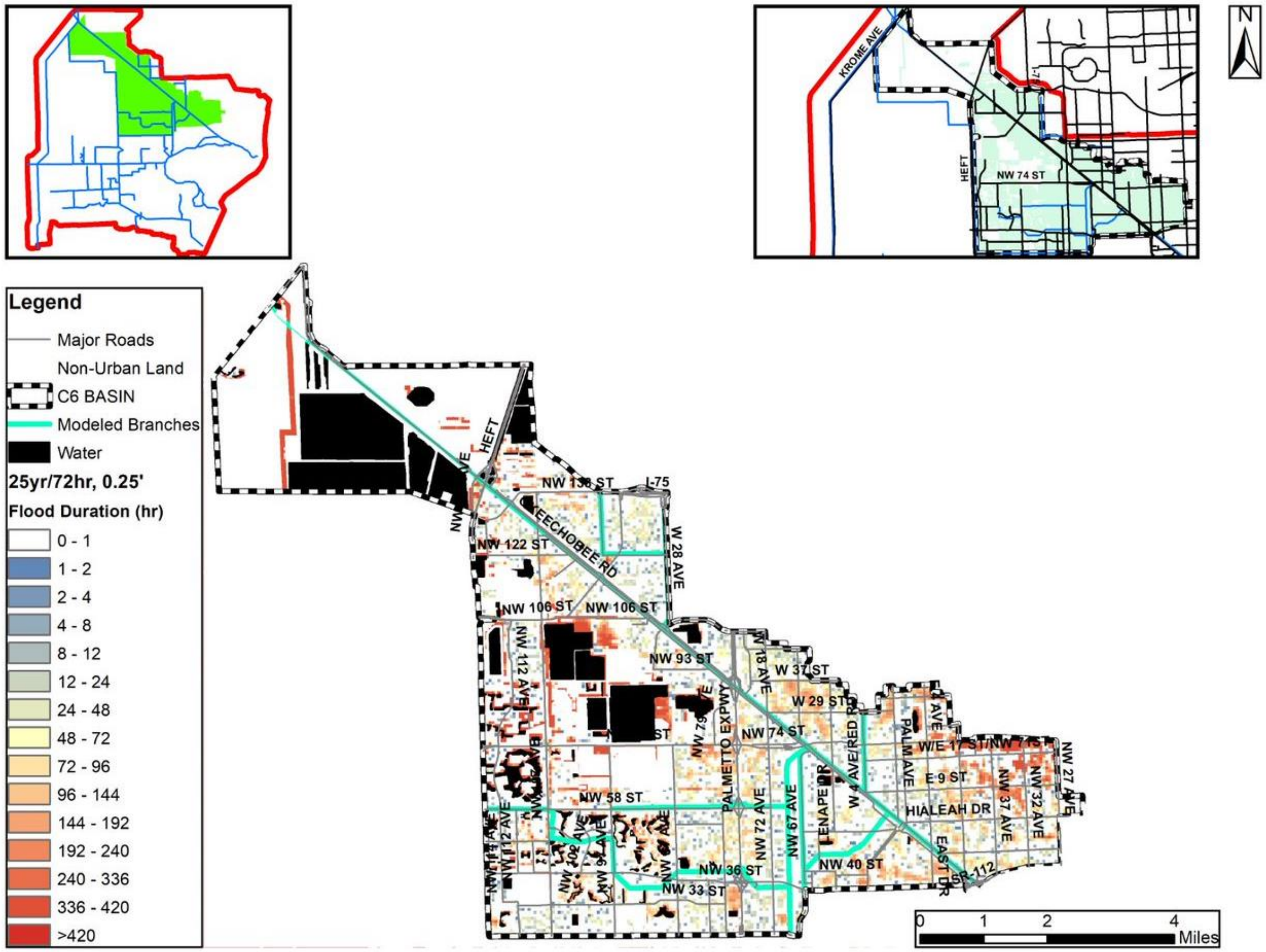


Figure 9-305. Flood Duration Map for the Current Conditions 100-year 3-day Design Storm for Urban Areas in the C6 Watershed

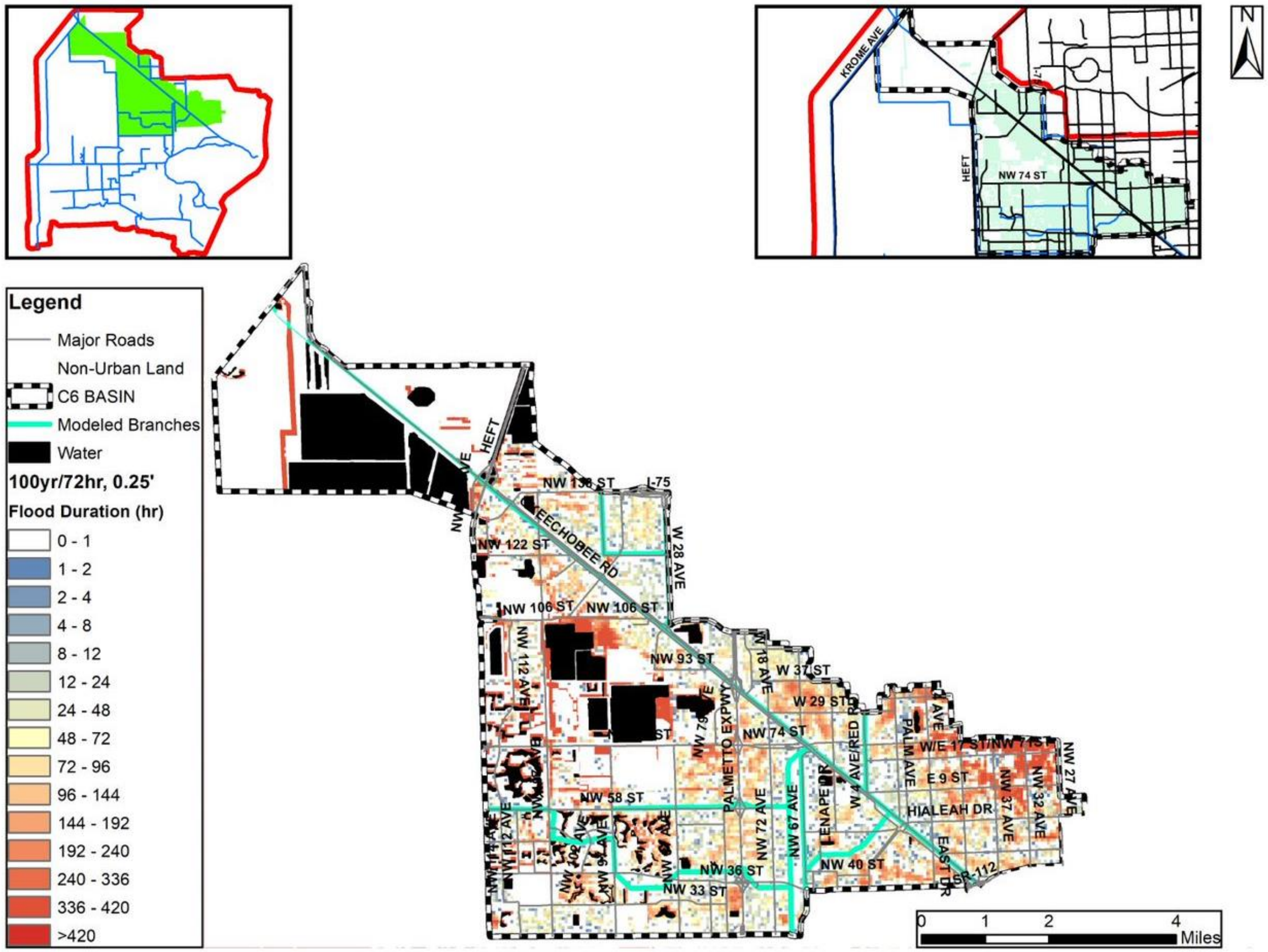


Figure 9-306. Urban Flooding Duration Difference of SLR +1ft and Current Cond. for the 100-year Storm in the C6 Watershed

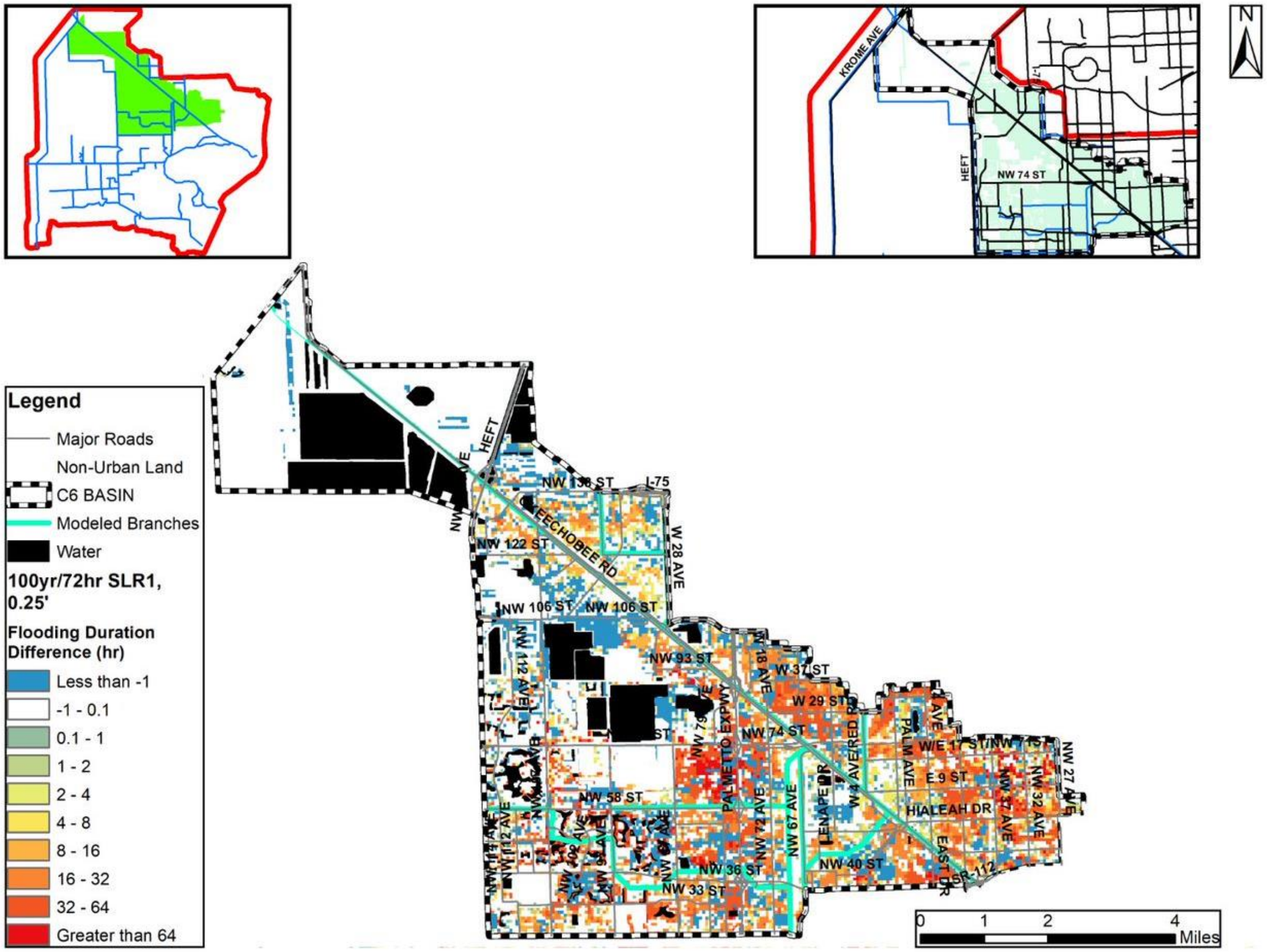


Figure 9-307. Urban Flooding Duration Difference of SLR +2ft and Current Cond. for the 100-year Storm in the C6 Watershed

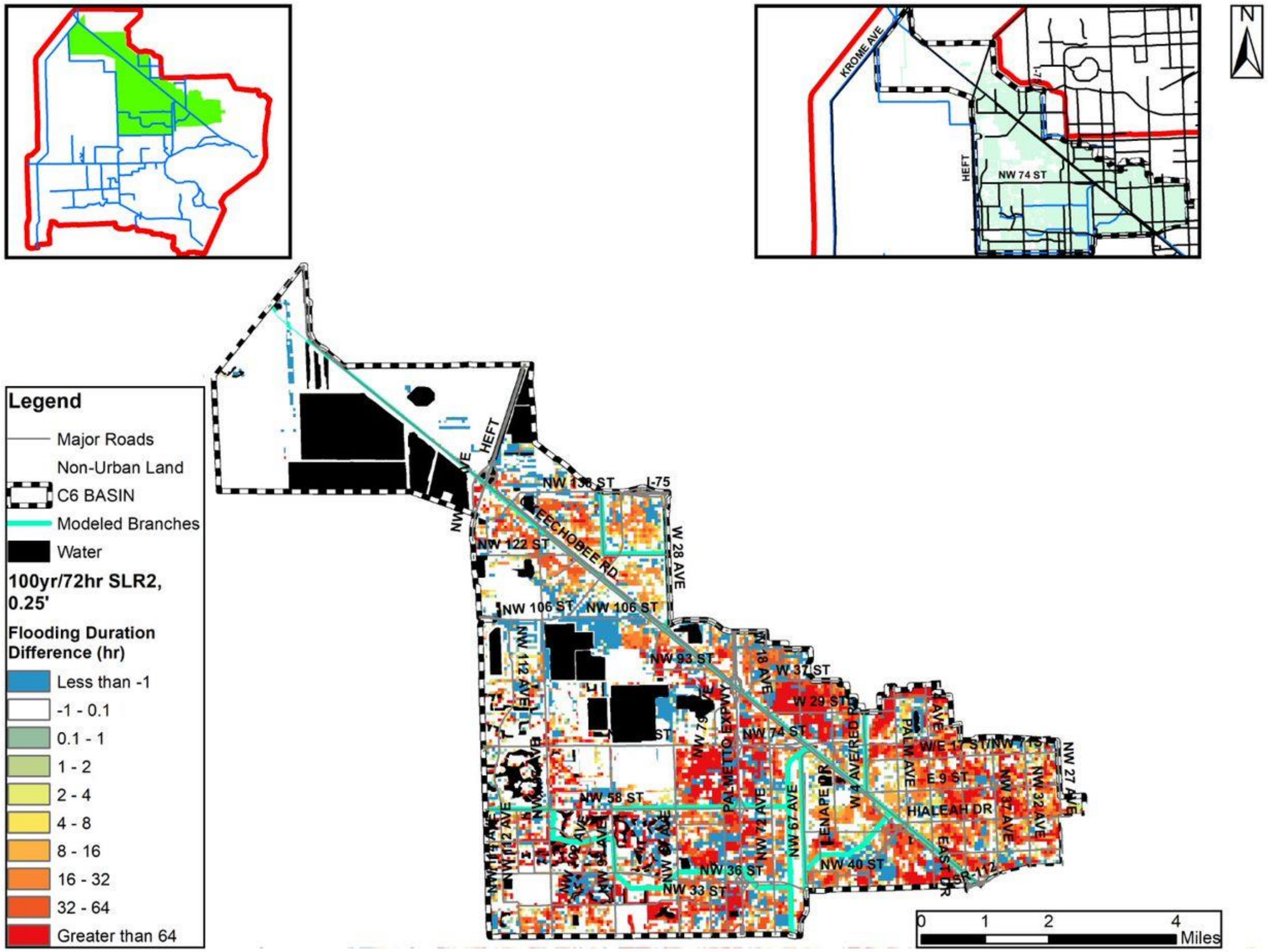
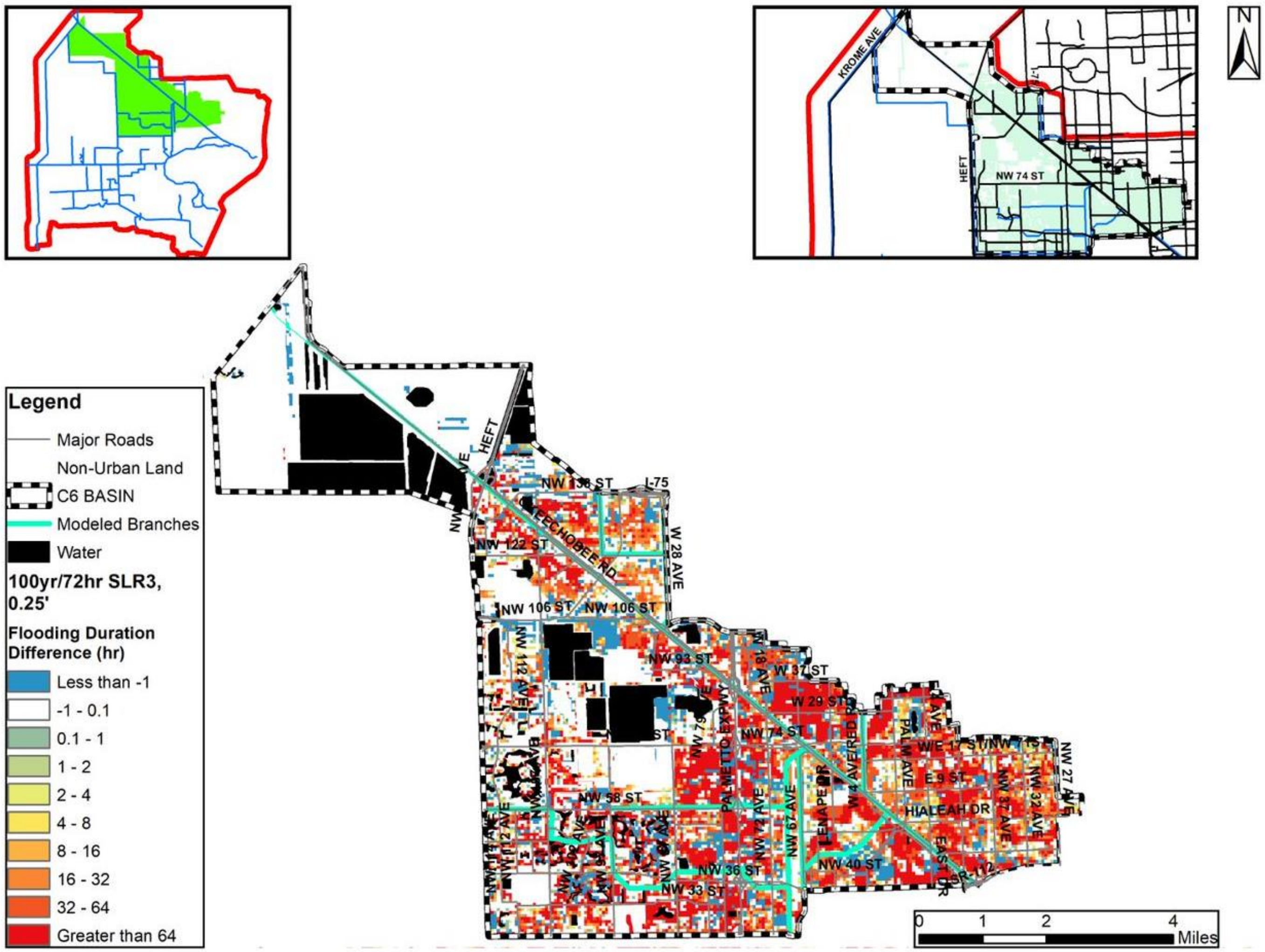


Figure 9-308. Urban Flooding Duration Difference of SLR +3ft and Current Cond. for the 100-year Storm in the C6 Watershed



Flood duration maps over the entire C6 Watershed, and only urban areas, for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for all current conditions and future SLR scenarios are provided in **Appendix D**. Also provided in **Appendix D** are the differences in flood duration maps for the 25-year, 10-year, and 5-year design storms.

For the C6 Watershed some areas tend to experience flooding for longer periods. Some areas of concern are near the NW 58th Street Canal and in the area of Hialeah near W 12th Avenue. In the area near NW 58th Street Canal, flooding in the overland region was reviewed further to determine if canal stages are impacting flooding. As discussed in PM #1, there are several areas along NW 58th Street Canal where the canal embankment is overtopped for the 100-year storm event, see **Figure 9-309** for the peak stage profile along NW 58th Street Canal.

Two complaints regarding the May 2020 rainfall event from the commercial areas of this region were reviewed and it was determined that both complaints were in areas that drain directly to the NW 58th Street Canal. Nearby cells were taken at points **A** and **B**, as indicated in **Figure 9-309**, for comparison with the stages in NW 58th Street.

As shown in **Figure 9-310**, both points have surface elevations that are lower than canal stages and are actually at or below the canal embankment of 4.7 ft-NAVD. Point **A** takes much longer for flooding to recede due to low topography and distance from the canal (about 3,500 feet away), whereas Point **B** is only one grid cell away (250 feet). Both points are impacted by reduced drainage to this canal, which is not classified as either primary or secondary, but would likely be considered a secondary canal. One final observation is that the canal peak stage profile (**Figure 9-249**) shows a head drop across NW 74th Avenue of about 0.3 feet, which is input as a single-barreled culvert and may be causing flow constrictions.

The area in Hialeah near the NW 58th Street Canal experiences reduced drainage or tertiary drainage issues during current conditions, with some areas directly adjacent to the canal receiving direct impacts from the canal stages for higher storm events. During future SLR scenarios, this area was observed to be highly impacted by increased duration of flooding, with greater than 2 days more than current conditions for the SLR1 100-year storm.

Figure 9-309. Overland Flood Elevation for the 100-year Design Storm in the area near NW 58th St Canal

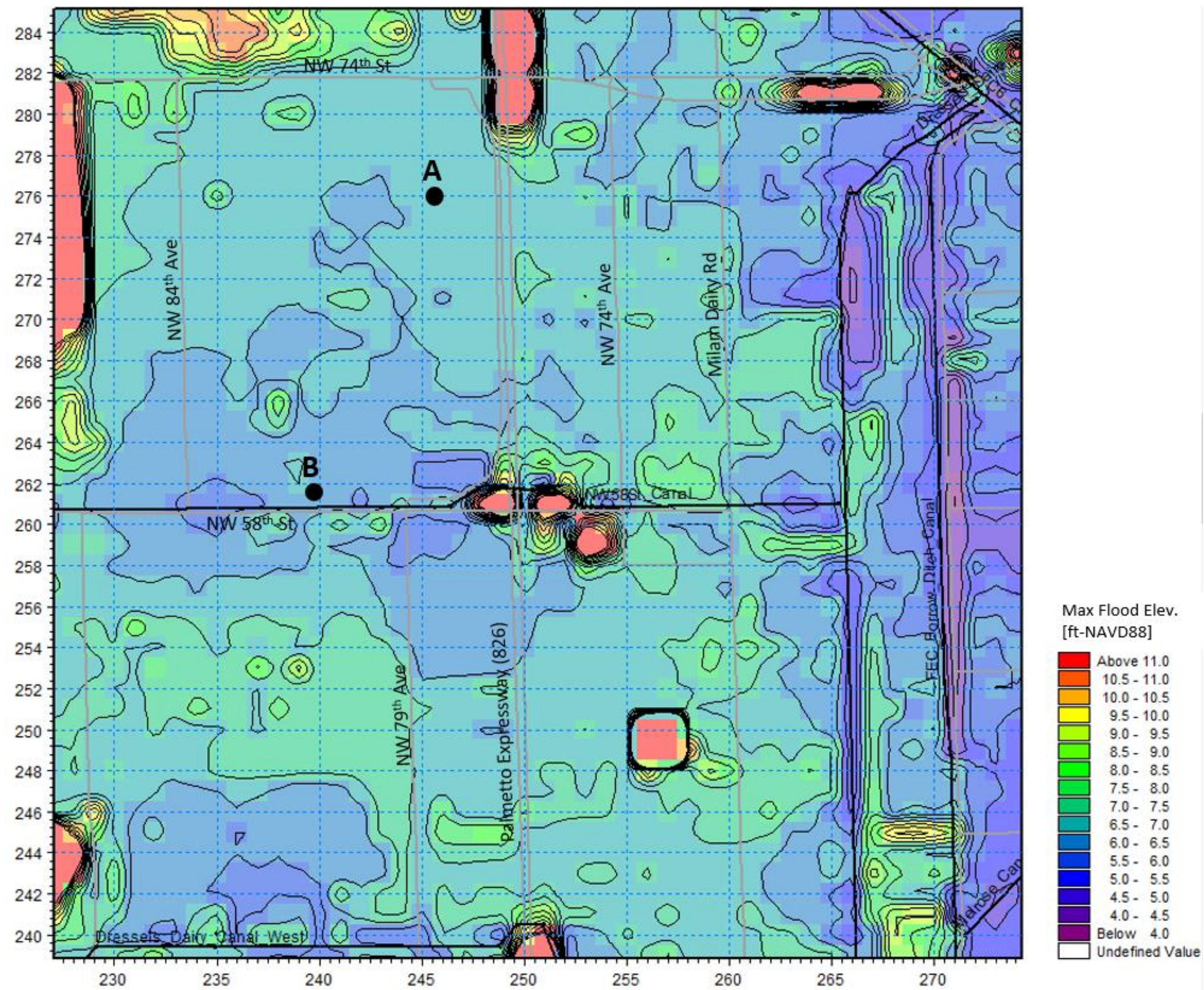
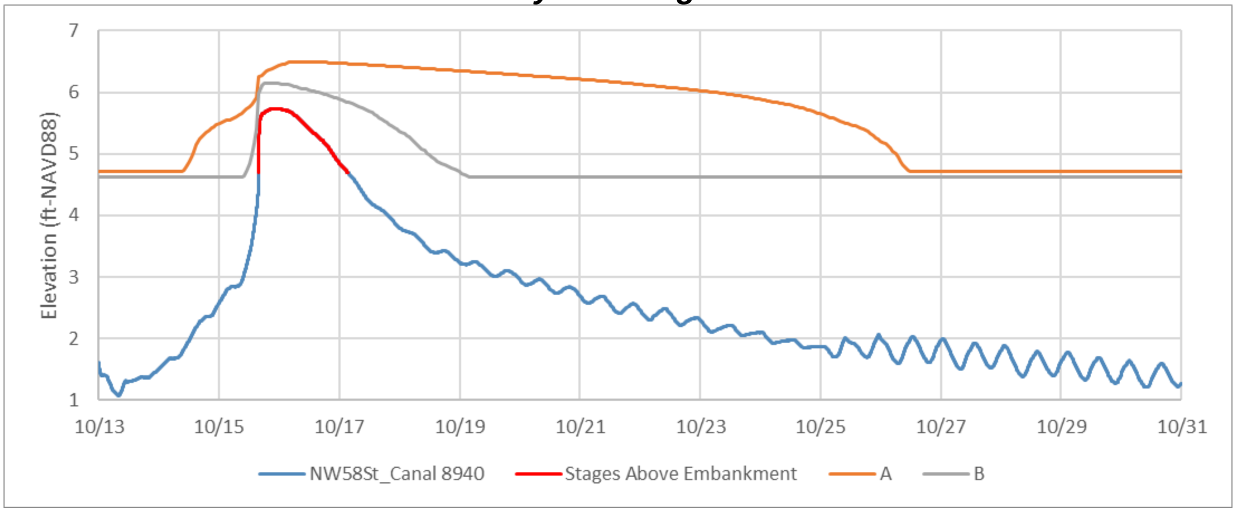


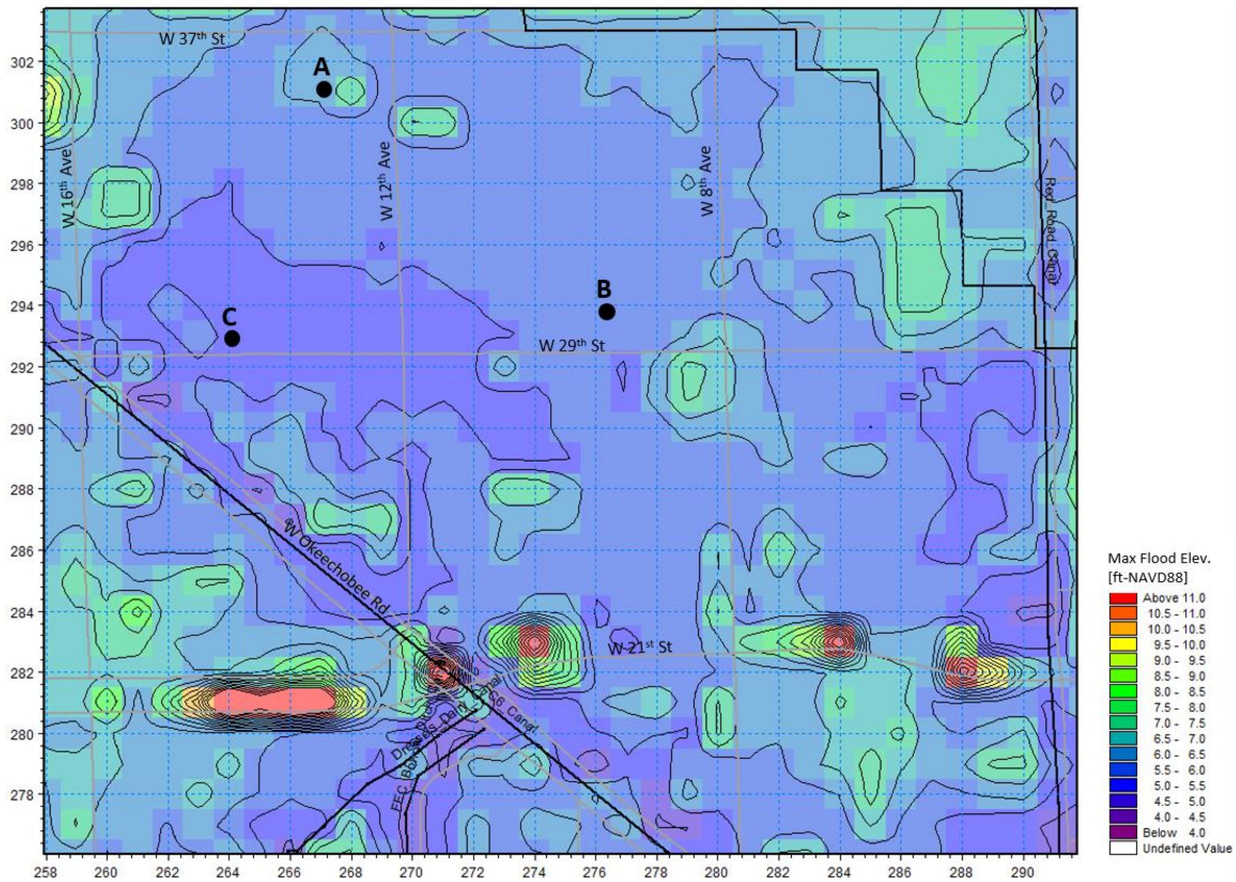
Figure 9-310. NW 58th St Canal Stages Compared with Overland Flood Elevation for 100-year Design Storm



In the area of Hialeah near W 12th Avenue, topography suggests that there is a low-lying flat area north of the C6 Canal between W 16th Avenue and the Red Road Canal. This area experiences high flooding depths as well as long flooding durations. Flooding in this area is corroborated with anecdotal evidence of flooding along W 34th Street in Hialeah where video showed extended flooding in a house after Hurricane Eta passed, as shown **Figure 9-311**. Overland flooding elevation timeseries at a cell nearby this location, point A as indicated in **Figure 9-312**, was extracted along with two (2) other low-lying areas for comparison with the stages in the C6 Canal to determine if canal stages are impacting flooding.

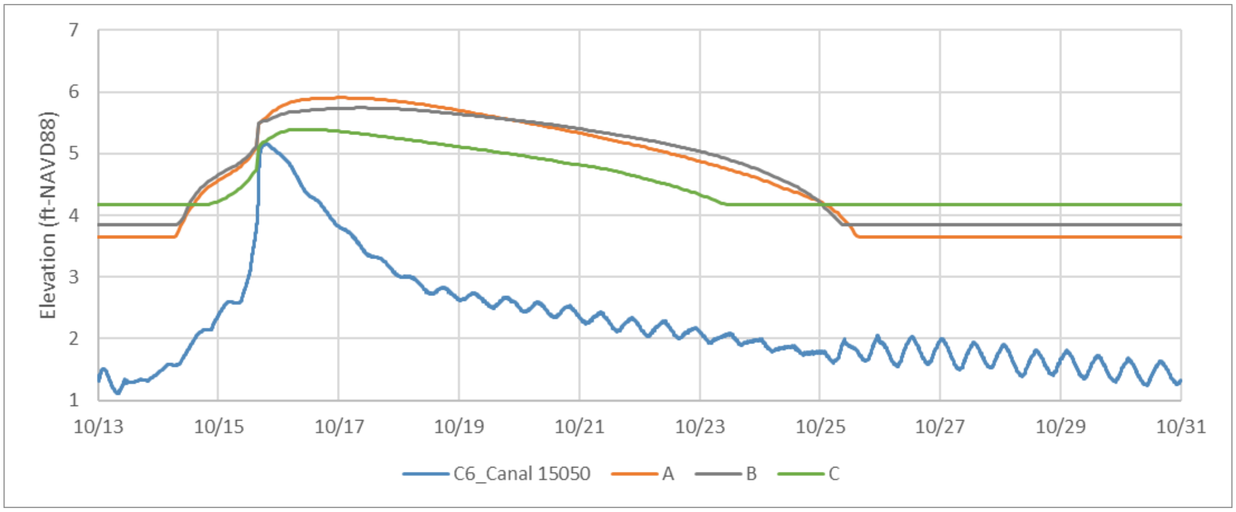
Figure 9-311. House flooding along W 34th St in Hialeah after Hurricane Eta

Figure 9-312. Overland Flood Elevation for the 100-year Design Storm in the Hialeah area near W 12th Avenue



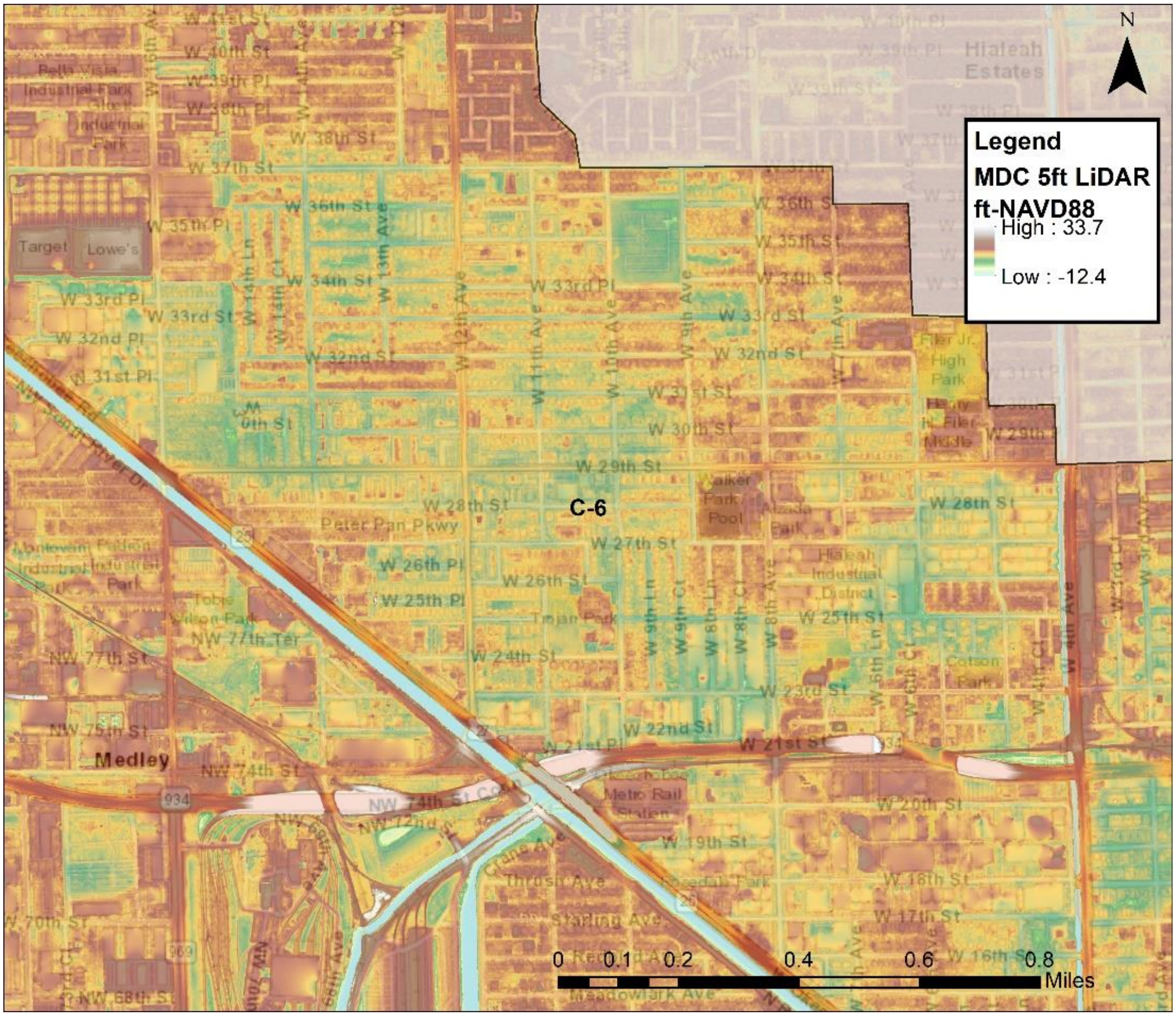
As shown in **Figure 9-313**, all three points (**A**, **B**, and **C**) have surface elevations that are lower than canal stages during the 100-year storm event. Point C, which is closest to the canal, recedes faster than Point **A** and **B**; however, all of these points are flooded for almost a week. All points are impacted by very low topography and reduced drainage to the C6 Canal, which is classified as a primary canal. Additionally, the peak of the 25-year storm is 4.35 ft-NAVD and the peak of the 10-year storm is 3.82 ft-NAVD at this location along the C6 Canal, which is higher than the land elevation at point **A**.

Figure 9-313. C6 Canal Stages Compared with Overland Flood Elevation for 100-year Design Storm



This area shows up to 0.5 feet of additional flooding with the SLR3 100-year simulation, which does not indicate direct impacts from the canal, however the duration of inundation increases significantly, indicating that the area will experience extended flooding of about 2 days even with SLR1.

Figure 9-314. LiDAR Topography of Low-Lying Basin in C6 Watershed



9.5.7. SUMMARY OF LOS AND RATING FOR THE C6 WATERSHED

The maximum design storm frequency that the C6 Watershed passes without incurring negative impacts is summarized for each performance metric in **Table 9-102.**

Under critical consideration for this watershed is PM #1, PM #5, and PM #6, as these relate directly to flooding in canals and overland flooding depth and duration. For current conditions it was determined that this watershed experiences flooding primarily due to tertiary drainage issues due to distance from the canal system. Some flooding due to peak stages in the primary and secondary canals is present in some areas for all design storms except the 5-year, which allowed for a 5-year rating for the current conditions. While SLR will impact flooding in the area, no substantial increase above current conditions in embankment or road overtopping (PM #1) was found with the SLR1 5-year storm above. The SLR1 condition, in general, did not significantly impact the watershed's ability to discharge during the peak of the storm (PM #2), nor did it cause a significant increase in overland flooding depths and durations (PM #5 and PM #6) across the watershed for the lower storms. However, the SLR2 and SLR3 conditions caused significant increases in these areas for all storm events. Therefore, the SLR1 condition received the same LOS rating as the current conditions of a 5-year storm, while the higher SLR conditions did not pass any of the storms.

Table 9-102. Performance Metric Summary for the C6 Watershed

Metric	Notes	Current Conds.	SLR +1ft	SLR +2ft	SLR +3ft
PM #1	▪ Two bridge low chords are exceeded only for the SLR3 100-year simulation (Hook Square/Curtiss Pkwy & S Hook Square). ▪ For current conditions, 18 of 61 culverts experienced overtopping during the 100-year storm. Only two (2) experienced overtopping during the 25-year design storm. The number of culvert locations where the crown of road is exceeded increases significantly with each SLR condition, with the exception of the 5-year storm event that doesn't increase significantly until SLR3. ▪ 29% of the top-of-bank elevations along the C6 Canal were overtopped during the 100-year design storm, and only 5% during the 25-year. However, secondary canals such as NW 58th St Canal experience overtopping in some areas for all storm events, with majority of the canal passing the 25-year. Overtopping is comparable (and less than one mile) to the 25-year for current conditions for the SLR1 10-year. (See **Table 9-83**)	25-year	10-year	< 5-year	< 5-year
PM #2	▪ No comparable value found for this basin. ▪ There is a general decrease in discharge capacity for the watershed with each higher SLR condition due to reduced discharge at the S26 structure. However, SLR1 shows similar discharge capacity as the current conditions, due to increased pumping at S26_P	--	--	--	--
PM #3	▪ Maximum discharge at S26 falls significantly below design value for all current and future conditions. ▪ HW exceeds the water level that will bypass S22 for 100-year current conditions and all future SLR scenarios except for the 5-year SLR1 scenario. The TW exceeds this bypass elevation during 100-year current conditions and all future SLR except for the 5-year and 10-year SLR1 scenarios.	--	--	--	--
PM #4	▪ Peak 12-hour moving discharge ranges from 664 CFS to 1903 CFS, compared to the design discharge of 3,470 CFS, and decreases with increasing SLR for each design storm return period.	--	--	--	--
PM #5	▪ The percentage of the watershed that is inundated with 3 inches or greater is still relatively high for the 5-year storm at 23.2% of the total urban area, however, this number drops rapidly to 15.0% for areas inundated with 6 inches or more. ▪ Areas such as Hialeah near W 12th Ave are low-lying and experience flooding due to peak stages in the C6 Canal for all storms except the 5-year for current conditions, however, inundation is comparable to the SLR1 5-year for higher depths. No other storms have comparable inundation.	5-year	5-year	<5-year	< 5-year
PM #6	▪ Canal: Using the new Reference Elevation, stages at the S26_H recede during the current conditions, SLR1, and SLR2 in less than 72 hours for all storm events. For SLR 3 this lasts for a week or more. ▪ Watershed: Portions of the watershed (such as Hialeah) show high flooding duration (i.e., > 72 hours) for all storm events.	<5-year	<5-year	<5-year	< 5-year
Overall Level of Service		**5-year**	**5-year**	**<5-year**	**<5-year**

10. CONCLUSIONS

As described in this report, CMA has prepared the FPLOS analysis for the C2, C3W, C4, C5, and C6 watersheds in central Miami Dade County. To perform this evaluation, an H&H model was calibrated and validated and then modified for design storm analysis of existing and future conditions. The calibration and validation procedure was extensive and lengthy given the complexity of the system, including highly managed drainage canals, unmonitored municipal pumping, a highly transmissive and non-uniform surficial aquifer, and a highly interconnected canal and surficial aquifer system. A battery of sensitivity analyses were performed for the calibration and validation, as well as a complex review of the hydrogeology in the region, which served to calibrate various parameters in the model, including horizontal hydraulic conductivity and channel roughness.

Once calibration was finalized, the FPLOS analysis was performed for the 100- year, 25-year, 10- year, and 5-year 3-day design storm events as a comparison of current conditions with future conditions under sea level rise of +1 foot, +2 feet and +3 feet. The future conditions models were modified to account for changes such as increased development, new canals, increased groundwater withdrawals, and increasing boundary stages. Other components remained static across the current and future conditions models, such as the rainfall intensities and structure operations for all canal gates and pumps.

Performance metrics (PM) provided a standardized guideline for LOS evaluation and led to a better understanding of the potential issues that individual watersheds may be experiencing. As noted in this report, each PM addresses a different characteristic of the watersheds LOS from aspects of canal and structure conveyance to flooding extent and duration. To focus the overall findings of each watershed, the design storms identified as the LOS for all PMs were aggregated with additional weight given to PM #1, PM #5, and PM #6 due to their direct relationship to flooding. Specifically, PM #1 focused on canal stages and overtopping of canal banks and structures and had more weight when determining the overall level of service for the watershed, in general this determined the minimum LOS if it was the lowest. PM #5 and PM #6, which focused on the inundation depth and duration, are also critical for understanding watershed flooding impacts and could lower the minimum LOS provided by PM #1.

In general, the PM#5 and PM#6 metrics are weighed differently for each watershed to determine what is the most critical factor for the watershed. However, PM #1 is a pass-or-fail criteria that must hold the most weight when determining the minimum LOS, as this considers infrastructure and direct canal overtopping.

Some critical components such as number of roads overtopped, number of bridge low chords exceeded, and direct flooding from a primary canal can also affect the assignment of the LOS. For example, in the C2 watershed the overland flooding from PM #5 and PM #6 may have not been a major concern for SLR1 25-year storms, however, the length of canal embankment that was overtopped jumped up to more than a mile and the SW 57th Avenue bridge low-chord was exceeded with the 10-year storm for SLR1. These factors prohibited the watershed from receiving a higher ranking than the 5-year storm for SLR1.

A summary of the current design storm LOS for each watershed as evaluated using the method described above is shown in **Table 10-1**.

Table 10-1. Summary of Current LOS Provided by Each Watershed

CONDITION	PM	C2	C3W	C4	C5	C6
Current Conditions	PM #1	10-year	25-year	25-year	25-year	25-year
	PM #5	25-year	25-year	10-year	10-year	5-year
	PM #6	25-year	25-year	10-year	25-year	<5-year
	Overall LOS	**10-year**	**25-year**	**10-year**	**10-year**	**5-year**
Future Conditions SLR +1 foot	PM #1	5-year	10-year	10-year	10-year	10-year
	PM #5	25-year	25-year	10-year	5-year	5-year
	PM #6	25-year	10-year	10-year	10-year	<5-year
	Overall LOS	**5-year**	**10-year**	**10-year**	**5-year**	**5-year**
Future Conditions SLR +2 feet	PM #1	<5-year	5-year	5-year	<5-year	<5-year
	PM #5	10-year	10-year	5-year	5-year	<5-year
	PM #6	10-year	10-year	5-year	5-year	<5-year
	Overall LOS	**<5-year**	**5-Year**	**5-year**	**<5-year**	**<5-year**
Future Conditions SLR +3 feet	PM #1	<5-year	<5-year	<5-year	<5-year	<5-year
	PM #5	10-year	10-year	<5-year	<5-year	<5-year
	PM #6	10-year	5-year	<5-year	<5-year	<5-year
	Overall LOS	**<5-year**	**<5-year**	**<5-year**	**<5-year**	**<5-year**

The performance metrics are a useful tool for understanding the critical areas of the subject watersheds that will be impacted by flooding with high-intensity storm events, storm surge, future development, and rising sea levels. In this analysis, no watershed is completely able to protect against flooding from high-intensity rainfall and storm surge, and as can be seen in the results, all future sea level rise conditions increased these impacts substantially.

An additional value of this FPLOS analysis will be in future efforts to prioritize potential flood mitigation projects and strategies.

Using the insight gleaned from this FPLOS study regarding the current and potential flooding risks associated with each watershed, preliminary mitigation strategies are being developed to help mitigate the effects of extreme rainfall and sea level rise (Deliverable 6.1). These mitigation strategies will undergo initial cost estimation and, as part of Phase II, will be implemented into the model as a means of understanding the effectiveness of the strategy at reducing flooding depths and durations and protecting critical infrastructure. The FPLOS evaluated in this study provides invaluable baseline understanding of current and future flooding conditions expected in the C2, C3W, C4, C5, and C6 watersheds and is the first step toward developing an adaptation plan that addresses short-term and long-term needs.

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APPENDIX A

PM1 Maximum Stage Profiles

Contents

Figures

Appendix A – PM#1 Maximum Stage Profiles

The figures provided in this appendix include maximum stage profiles. The figures compare the maximum stage within each primary canal of the watershed for each SLR condition (i.e., Current Conditions, SLR1, SLR2, and SLR3) for all design storm events (i.e. 5-year, 10-year, 25-year, and 100-year design storms).

1 C2 Watershed

Figure A 1-1 shows the maximum stage profile for the Snapper Creek Canal within the C2 Watershed from the C4 Canal to the S22 water control structure for the Current Conditions simulations for all design storm events. **Figure A 1-2** through **Figure A 1-5** show the maximum stage profiles for all Current and Future Conditions for the 5-year, 10-year, 25-year, and 100-year storm events.

Figure A 1-6 shows the maximum stage profile for the NW 58th St Canal, Dressel's Dairy Canal, and the C6 Canal (or the Miami Canal) within the C6 Watershed to the S26 water control structure for the Current Conditions simulations for all design storm events. **Figure A 1-7** through **Figure A 1-10** show the maximum stage profiles for all Current and Future Conditions for the 5-year, 10-year, 25-year, and 100-year storm events.

Figure A 1-1. Maximum Stage Profile for the Snapper Creek Canal in the C2 Watershed for the Current Conditions

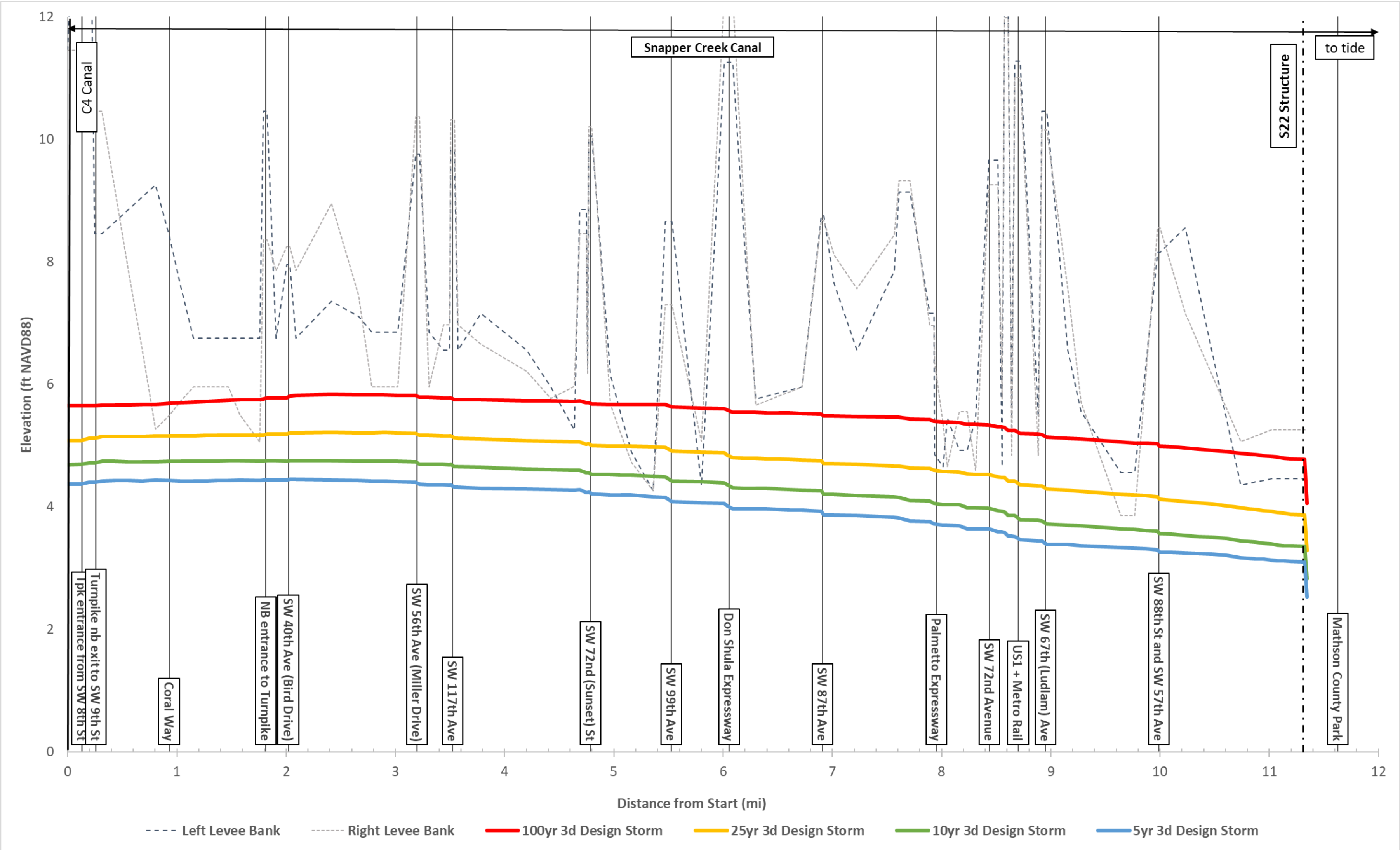


Figure A 1-2. Maximum Stage Profile for the Snapper Creek Canal in the C2 Watershed for the 5-year Design Storm

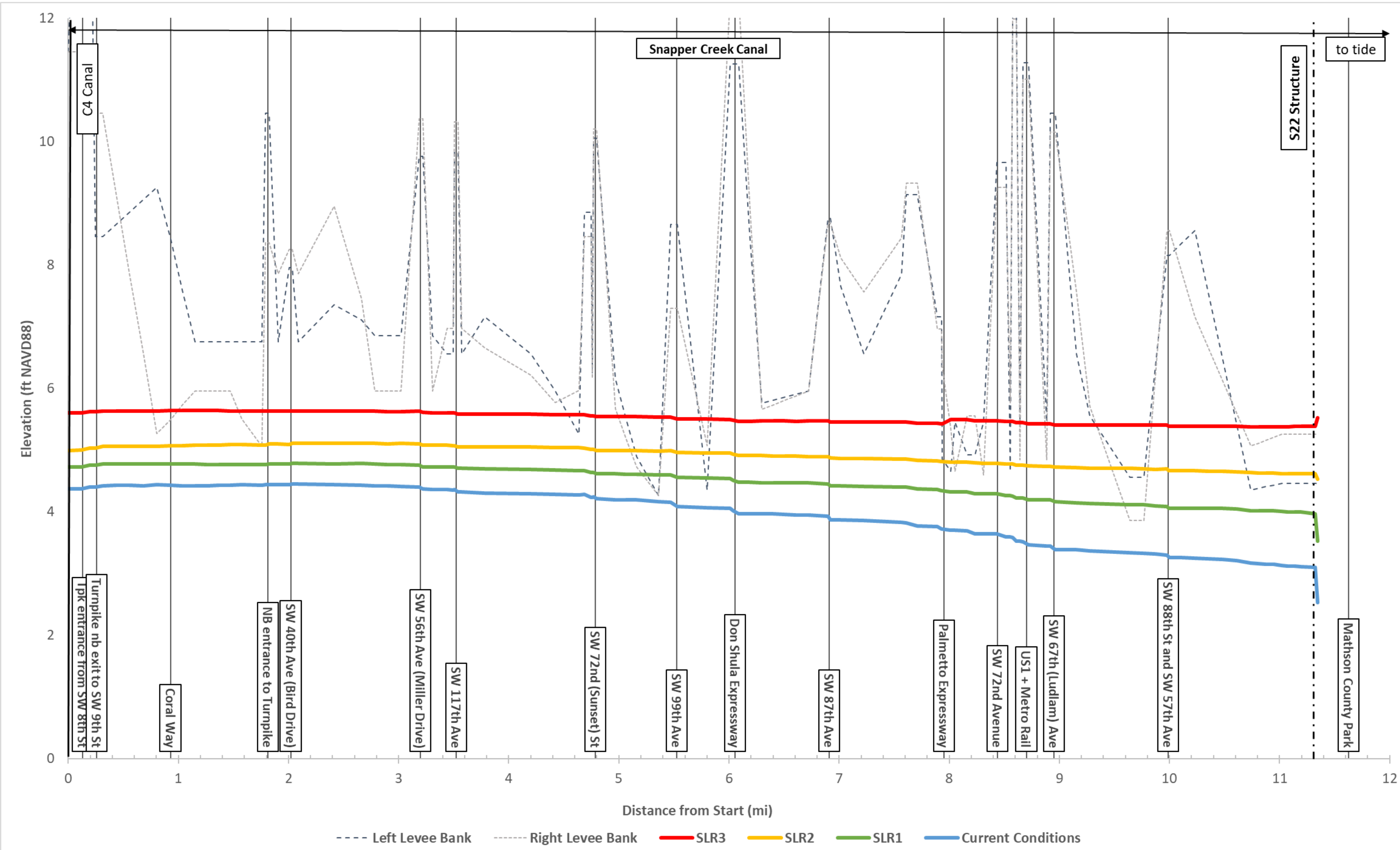


Figure A 1-3. Maximum Stage Profile for the Snapper Creek Canal in the C2 Watershed for the 10-year Design Storm

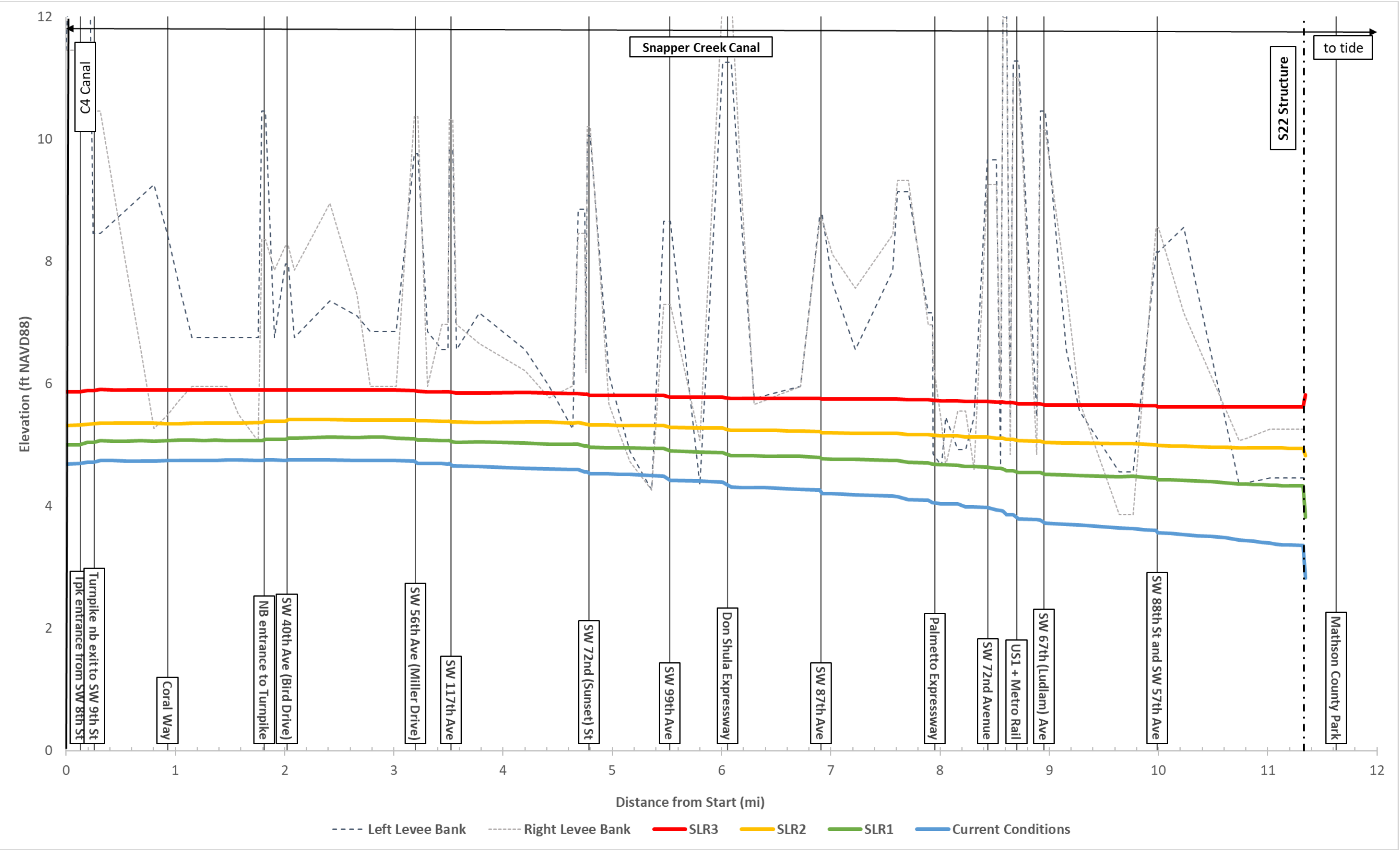


Figure A 1-4. Maximum Stage Profile for the Snapper Creek Canal in the C2 Watershed for the 25-year Design Storm

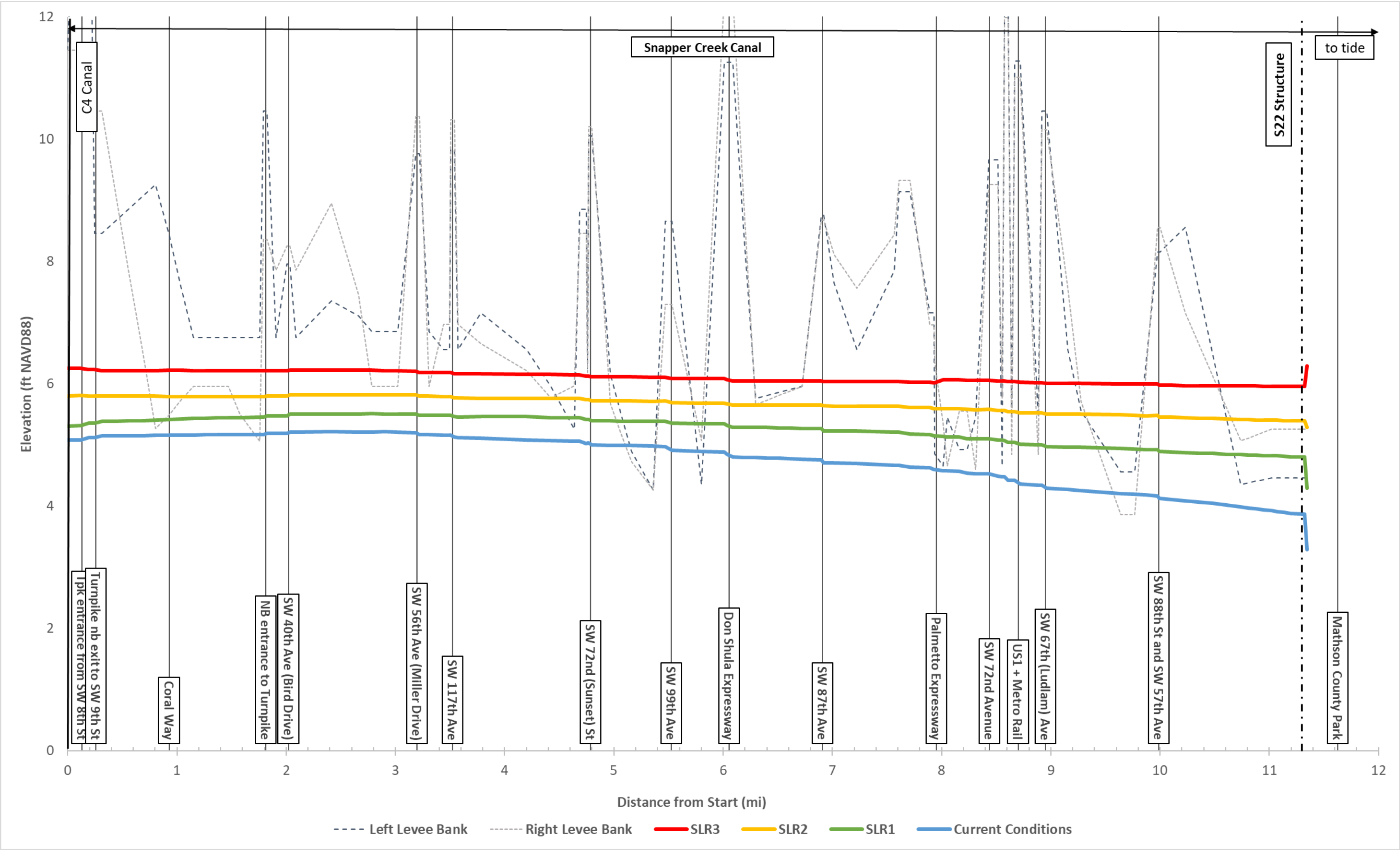


Figure A 1-5. Maximum Stage Profile for the Snapper Creek Canal in the C2 Watershed for the 100-year Design Storm

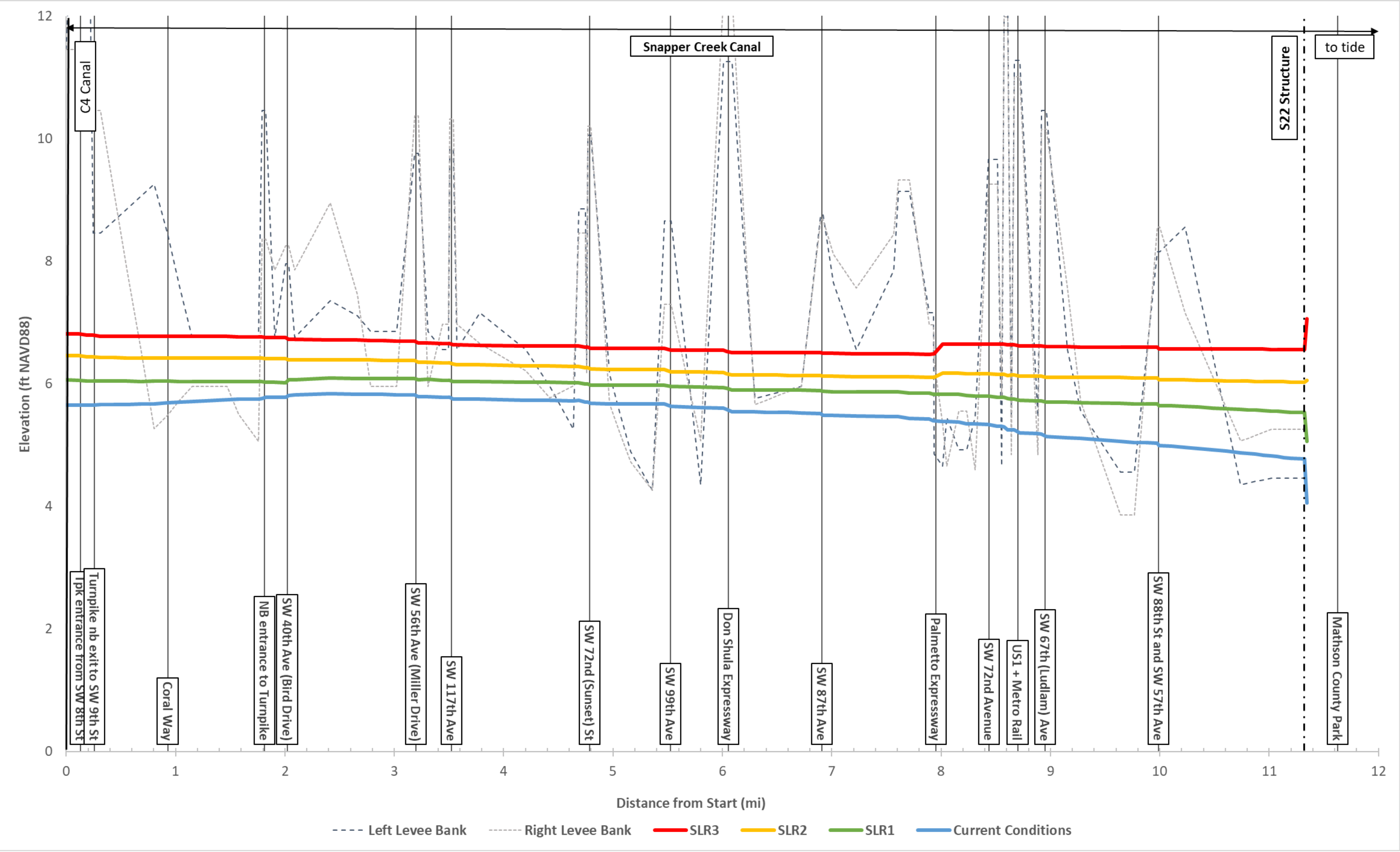


Figure A 1-6. Maximum Stage Profile for the Bird Drive Ext. Canal & Snapper Creek Canal in the C2 Watershed for the Current Conditions

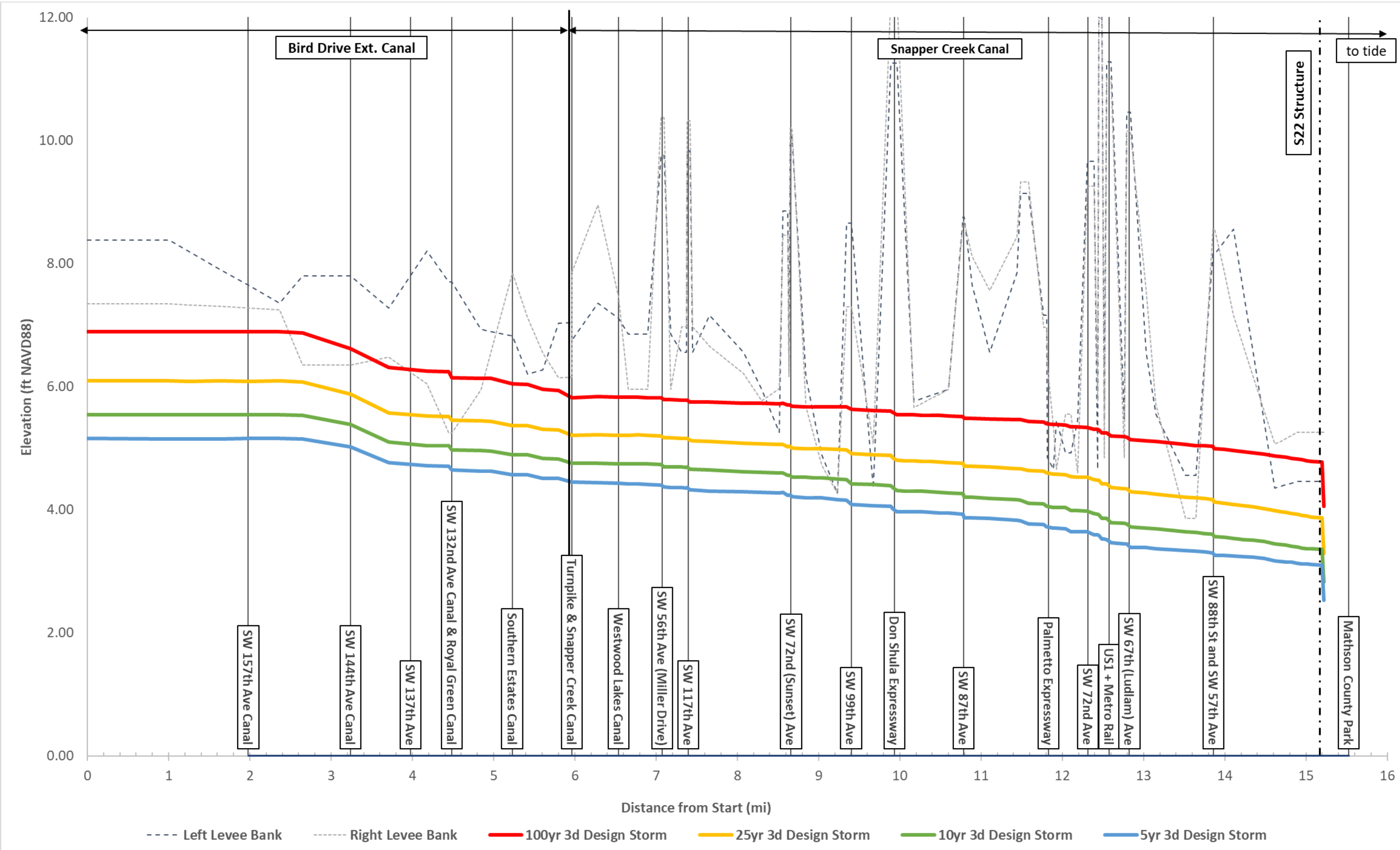


Figure A 1-7. Maximum Stage Profile for the Bird Drive Ext. Canal & Snapper Creek Canal in the C2 Watershed for the 5-year Design Storm

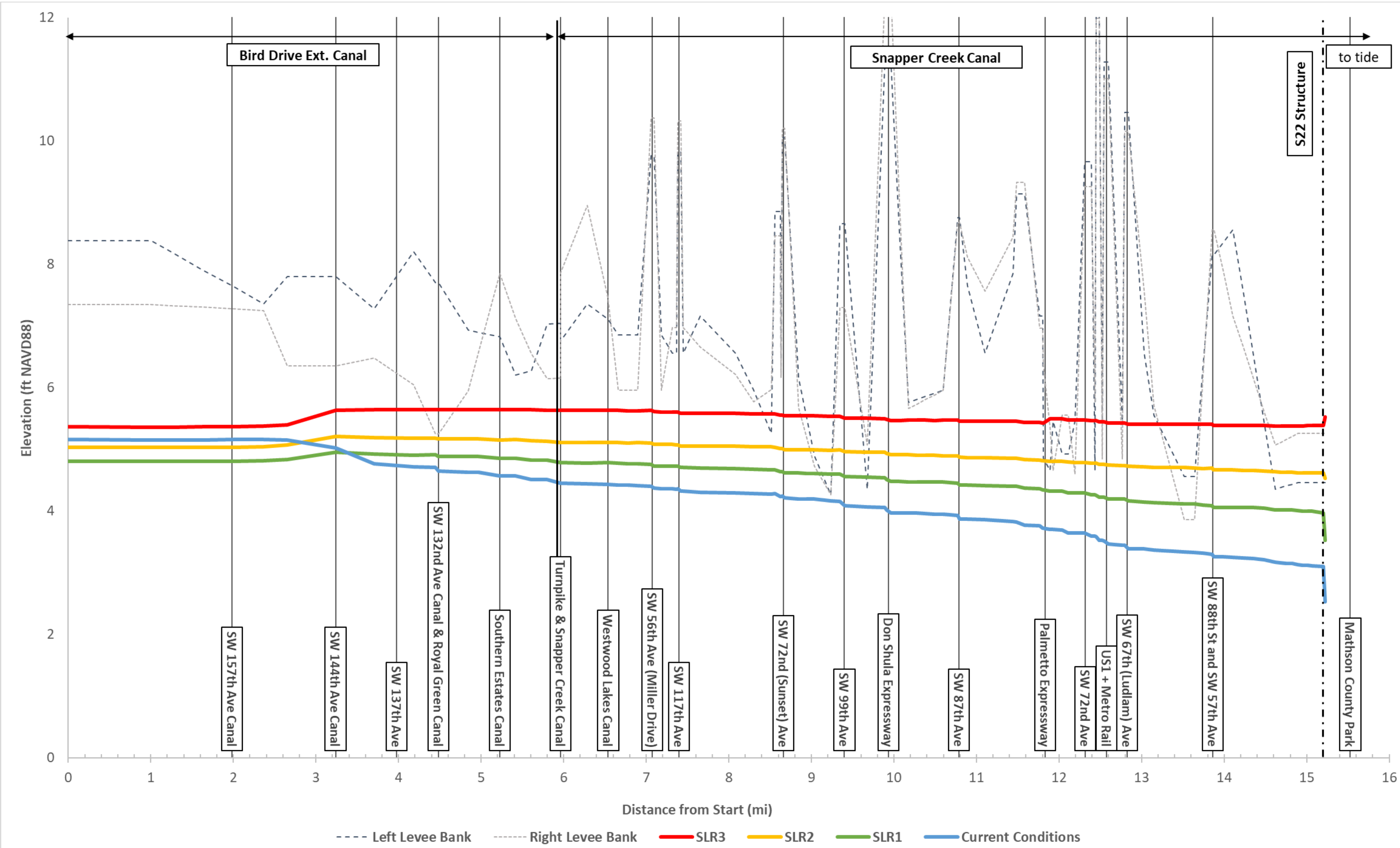


Figure A 1-8. Maximum Stage Profile for the Bird Drive Ext. Canal & Snapper Creek Canal in the C2 Watershed for the 10-year Design Storm

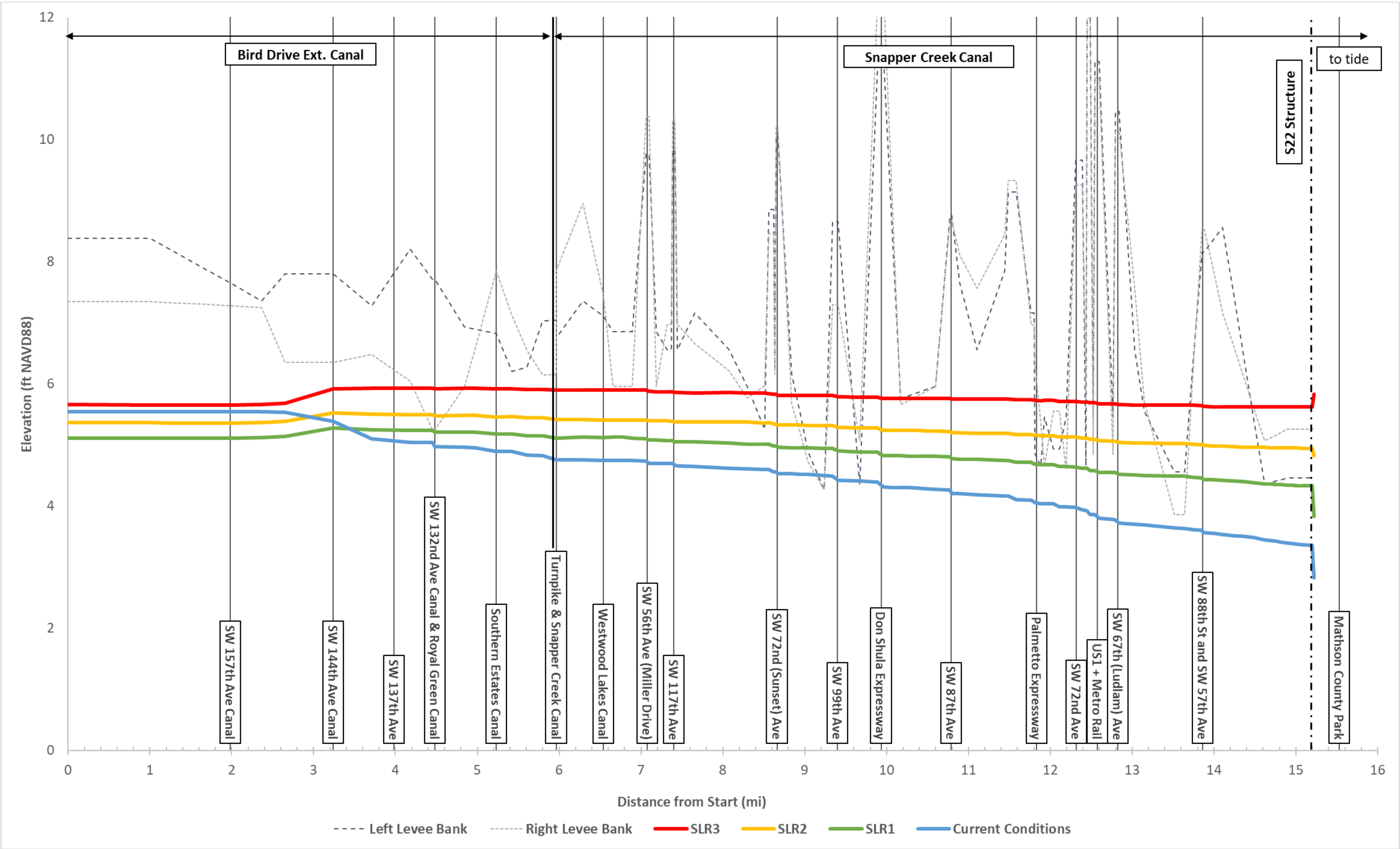


Figure A 1-9. Maximum Stage Profile for the Bird Drive Ext. Canal & Snapper Creek Canal in the C2 Watershed for the 25-year Design Storm

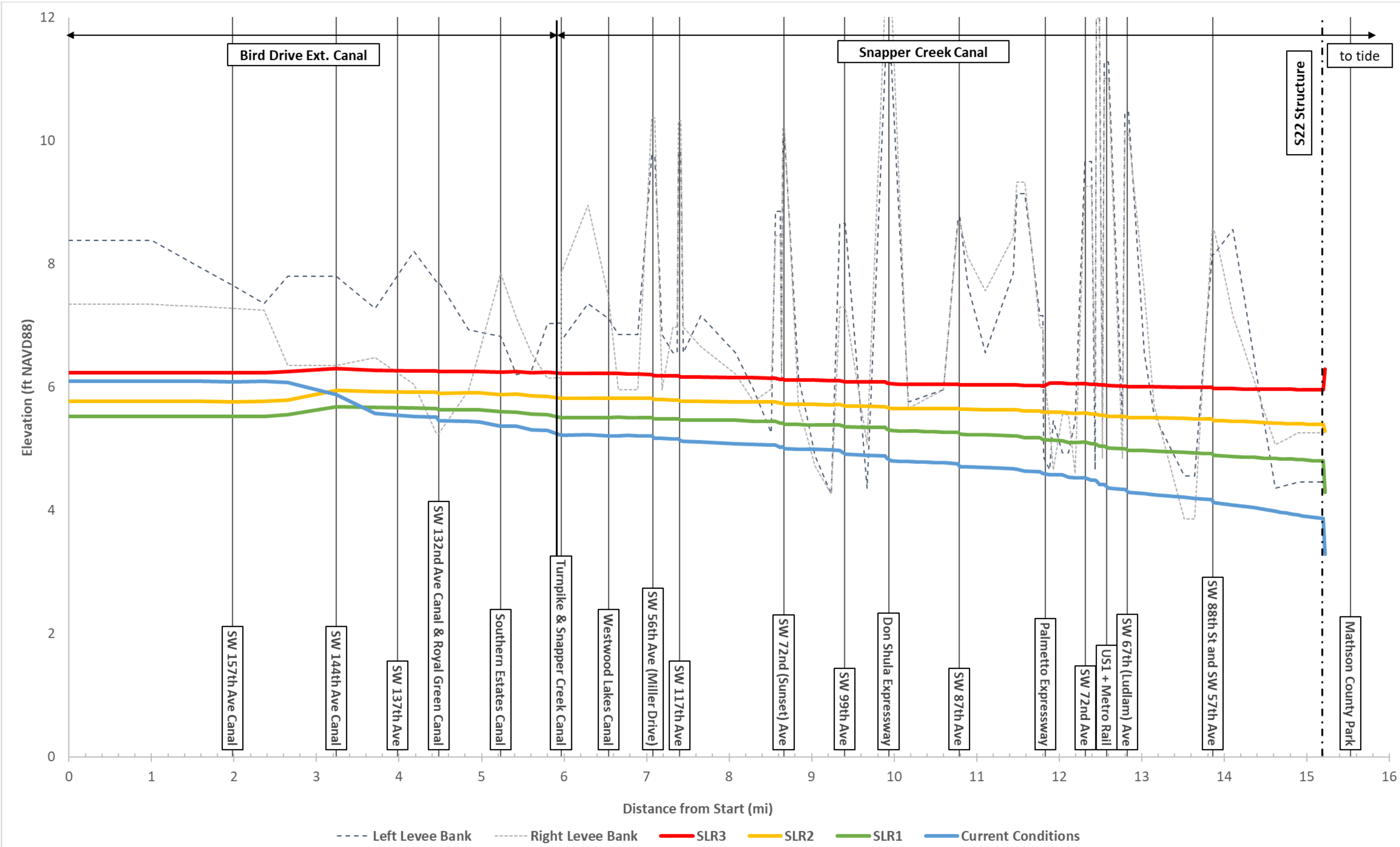
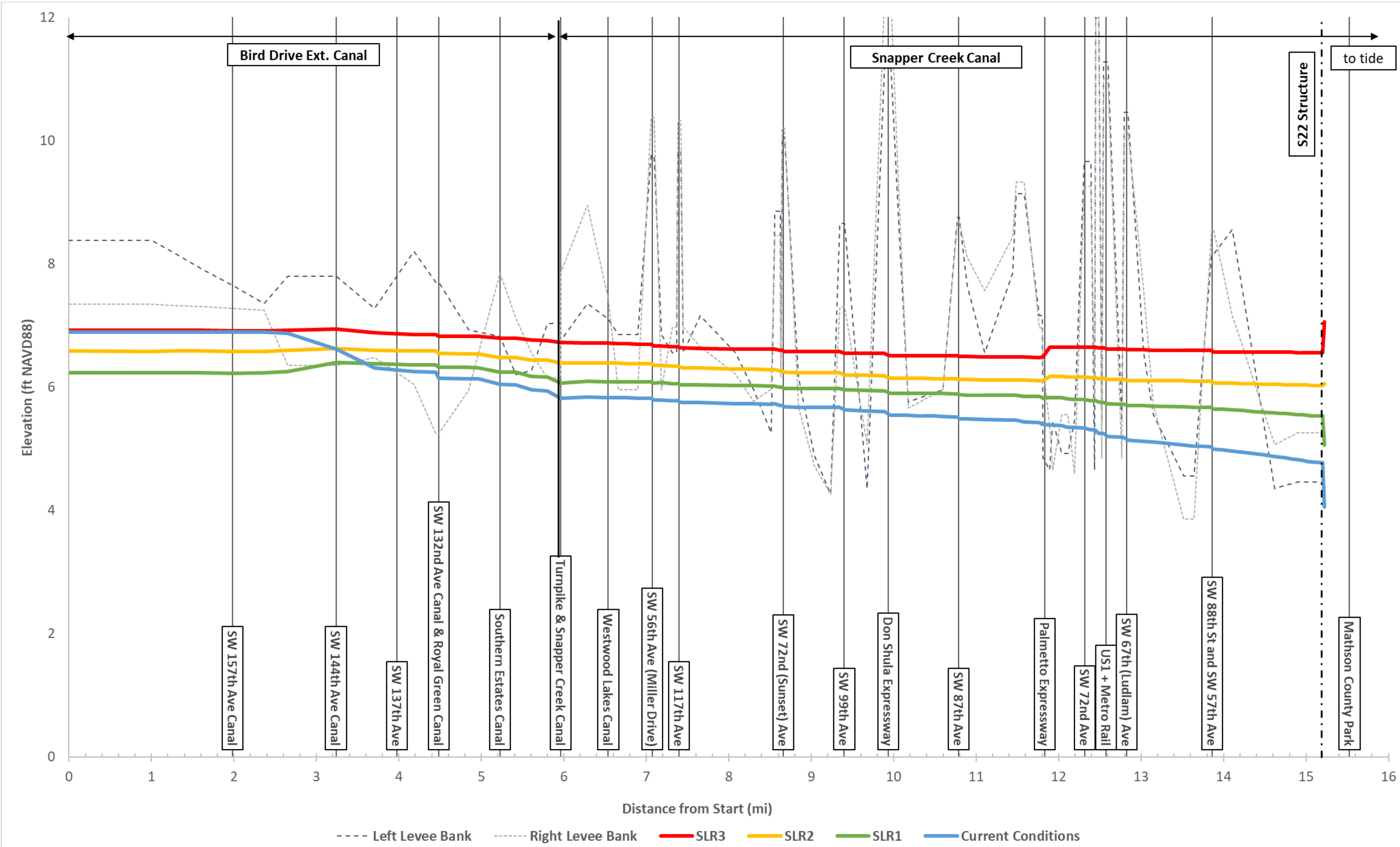


Figure A 1-10. Maximum Stage Profile for the Bird Drive Ext. Canal & Snapper Creek Canal in the C2 Watershed for the 100-year Design Storm



2 C3W Watershed

Figure A 2-1 shows the maximum stage profile for the Coral Gables Canal within the C3W Watershed from the C4 Canal to the G93 water control structure for the Current Conditions simulations for all design storm events. **Figure A 2-2** through **Figure A 2-5** show the maximum stage profiles for all Current and Future Conditions for the 5-year, 10-year, 25-year, and 100-year storm events.

Figure A 2-1. Maximum Stage Profile for the Coral Cables Canal in the C3 Watershed for the Current Conditions

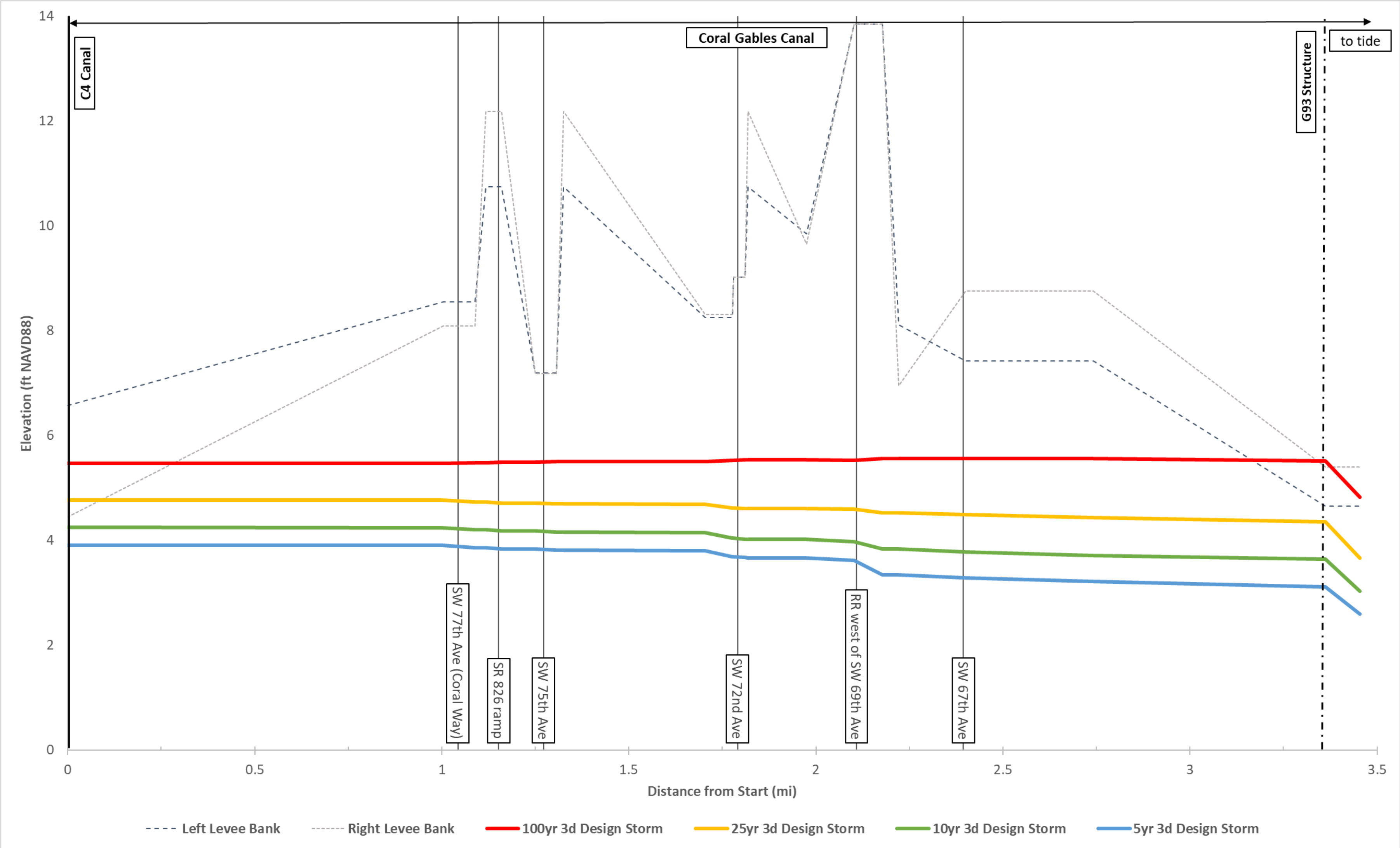


Figure A 2-2. Maximum Stage Profile for the Coral Cables Canal in the C3 Watershed for the 5-year Design Storm

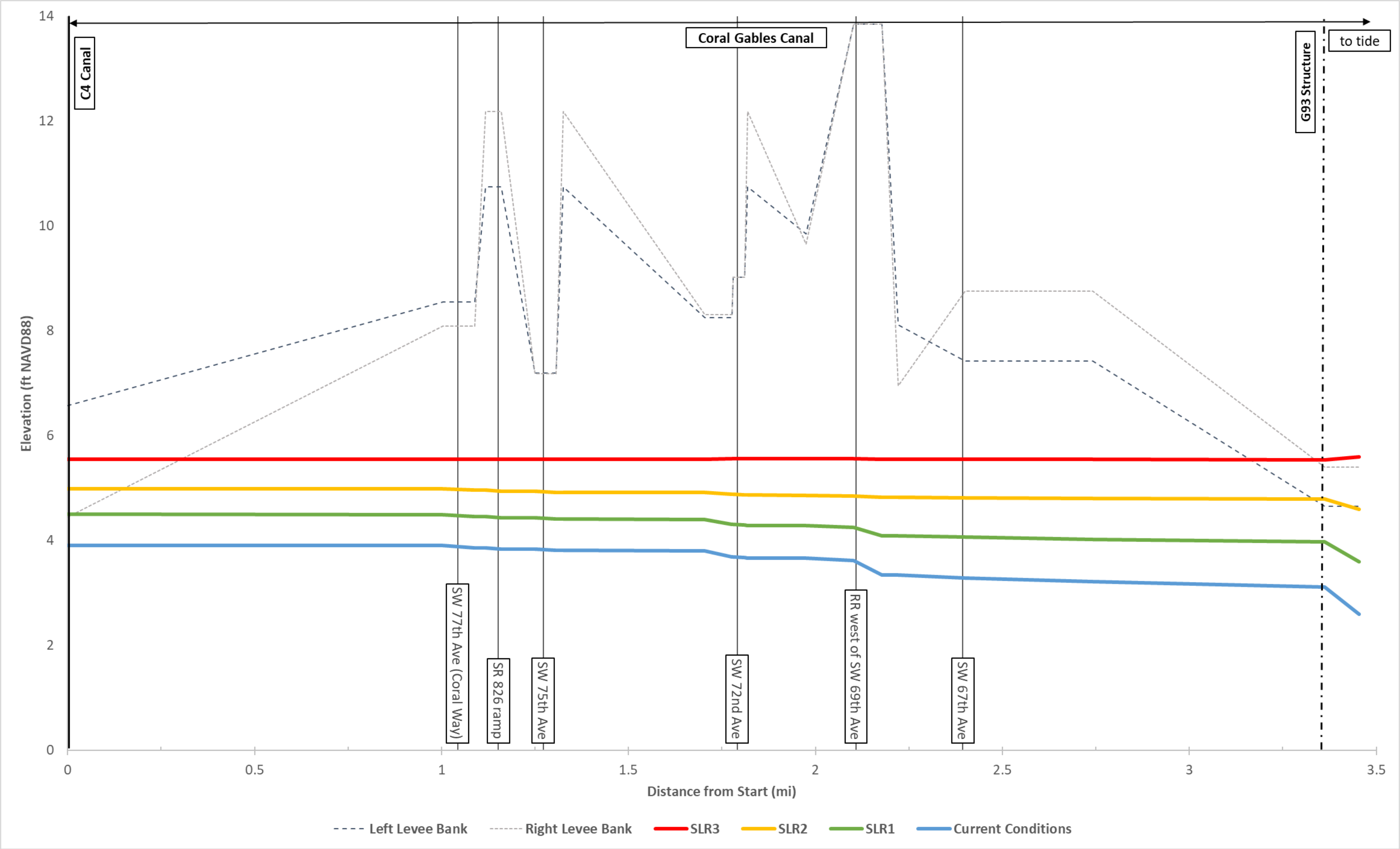


Figure A 2-3. Maximum Stage Profile for the Coral Cables Canal in the C3 Watershed for the 10-year Design Storm

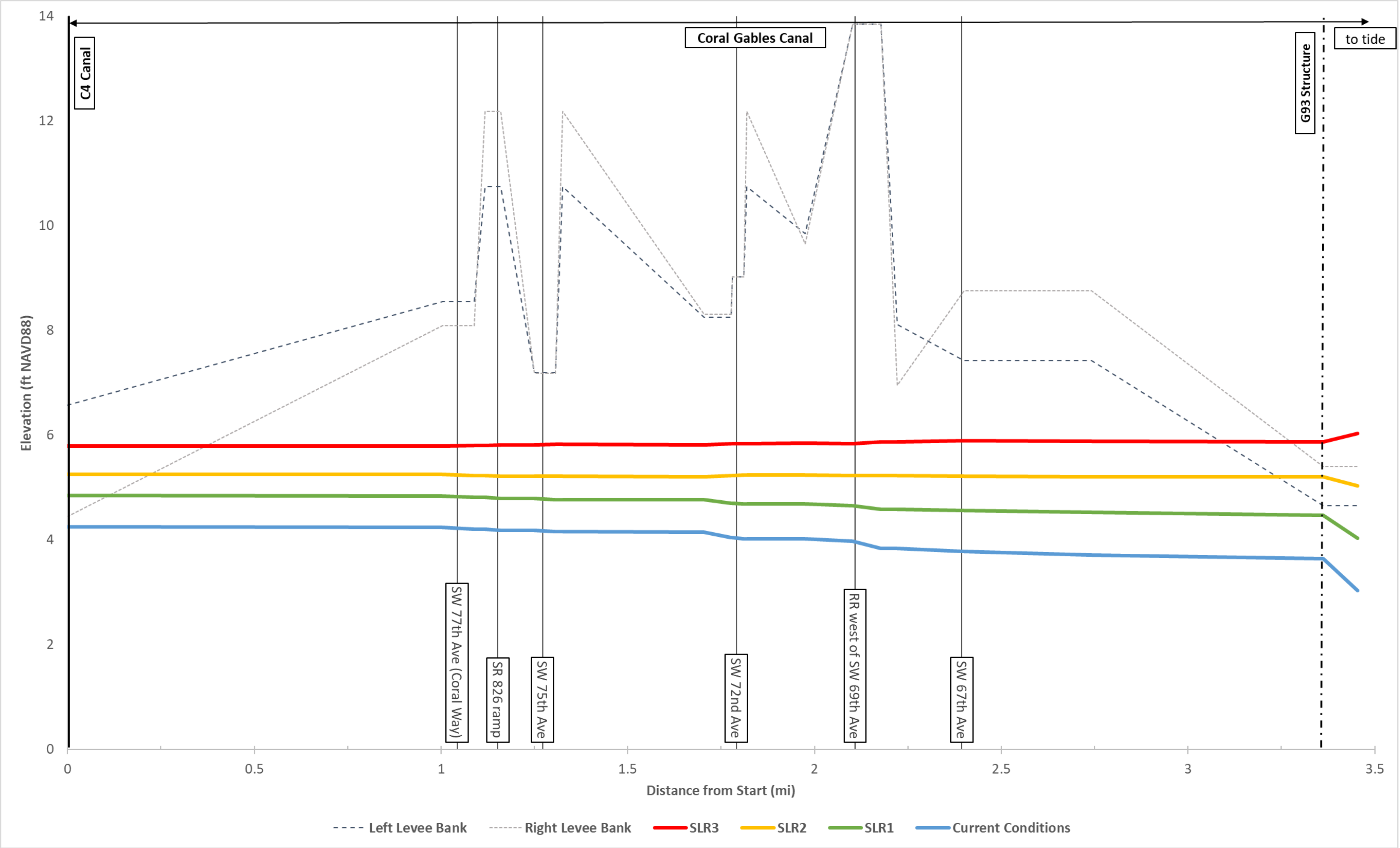


Figure A 2-4. Maximum Stage Profile for the Coral Cables Canal in the C3 Watershed for the 25-year Design Storm

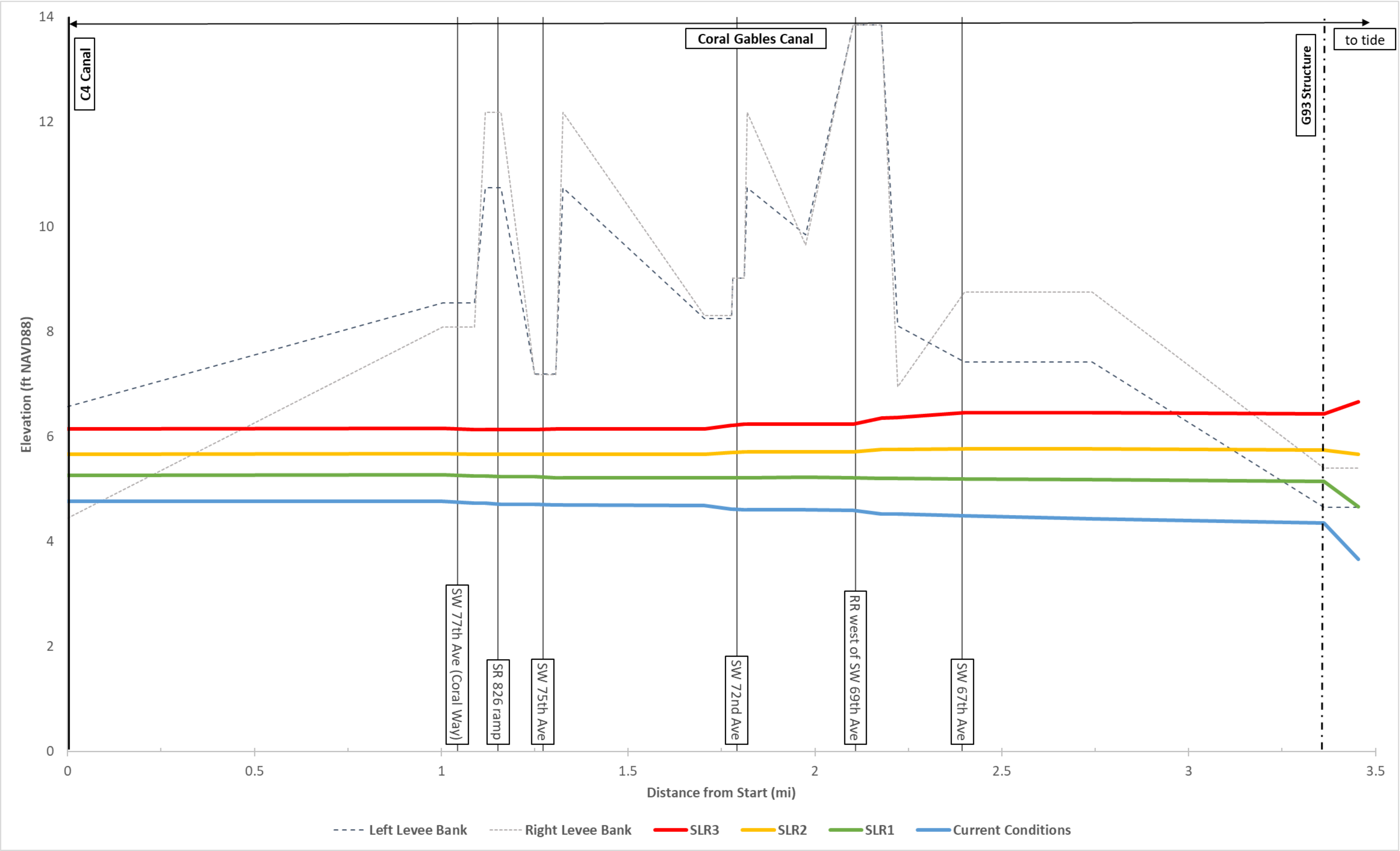
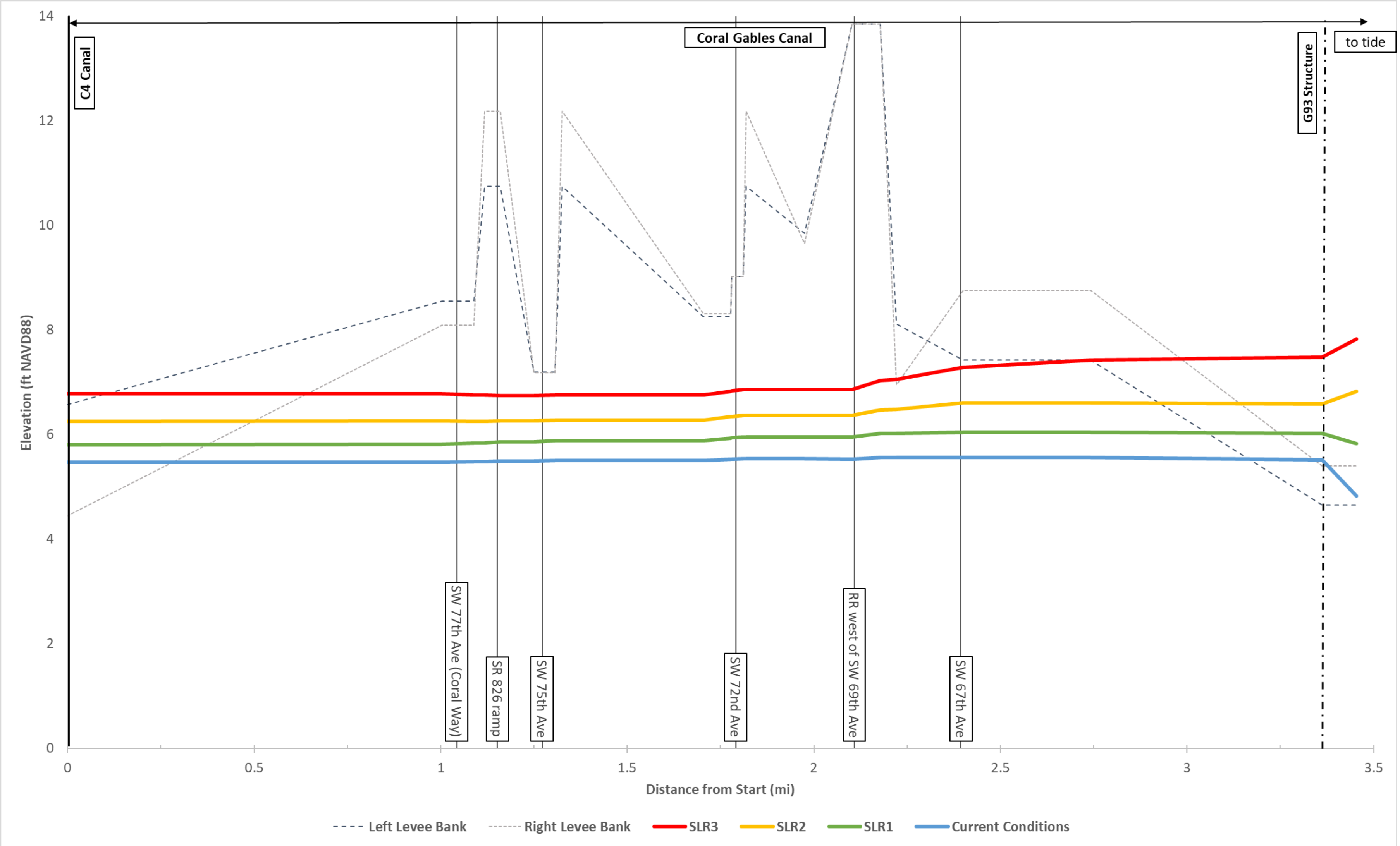


Figure A 2-5. Maximum Stage Profile for the Coral Cables Canal in the C3 Watershed for the 100-year Design Storm



3 C4 Watershed

Figure A 3-1 shows the maximum stage profile for the C4 Canal (or the Tamiami Canal) within the C4 Watershed from the S380 to the S25B water control structures for the Current Conditions simulations for all design storm events. **Figure A 3-2** through **Figure A 3-5** show the maximum stage profiles for all Current and Future Conditions for the 5-year, 10-year, 25-year, and 100-year storm events.

Figure A 3-1. Maximum Stage Profile for the C4 Canal in the C4 Watershed for the Current Conditions

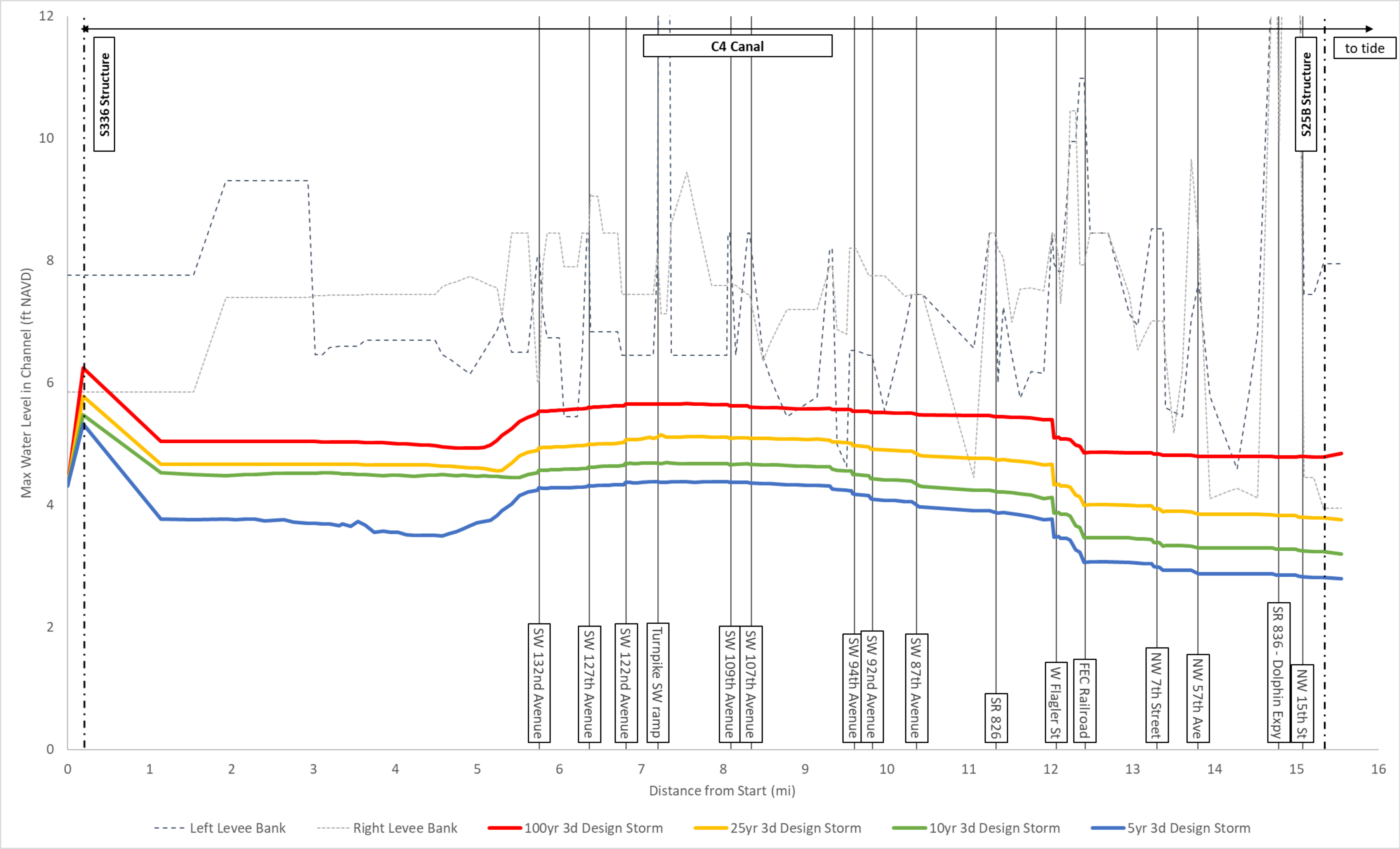


Figure A 3-2. Maximum Stage Profile for the C4 Canal in the C4 Watershed for the 5-year Design Storm

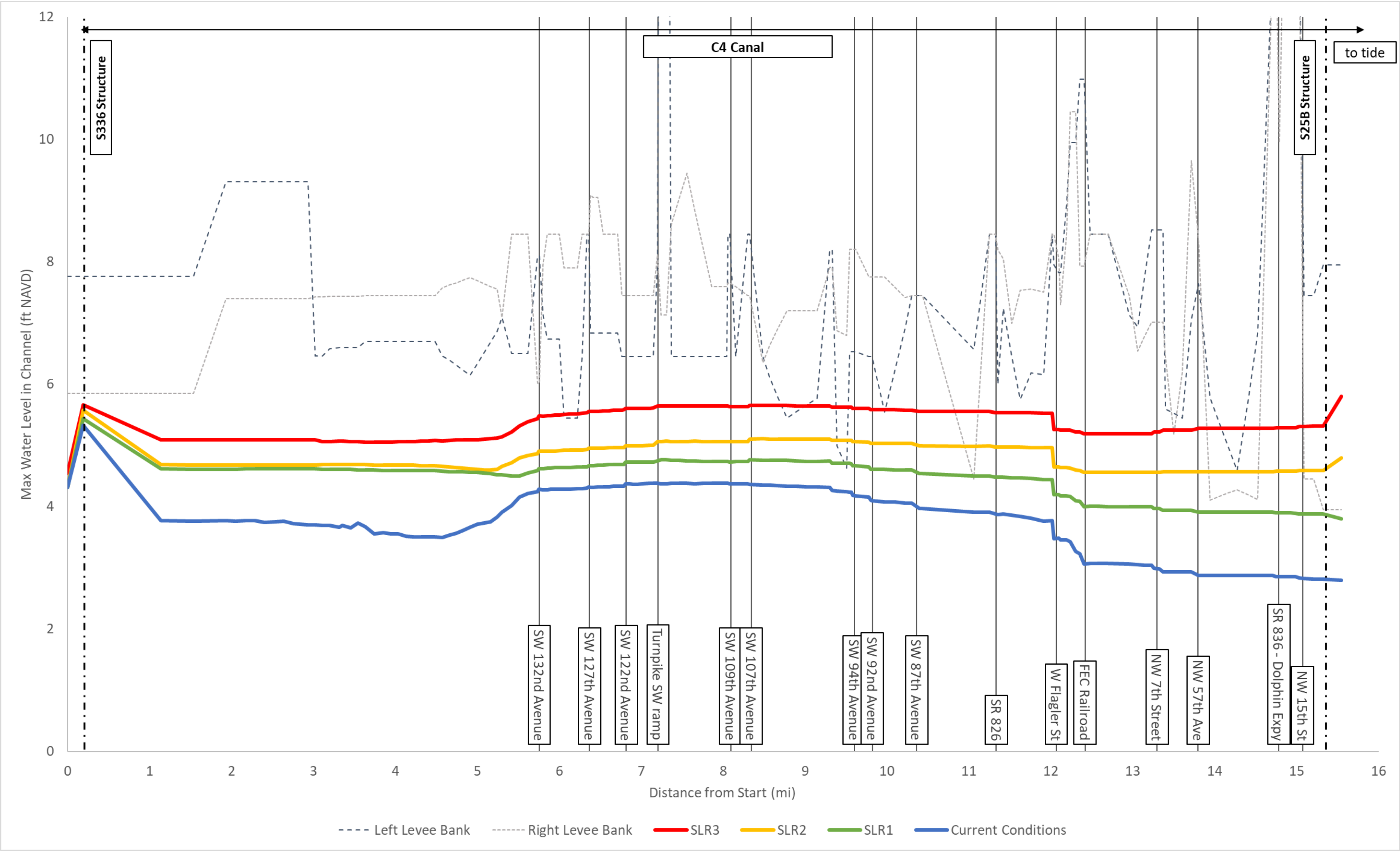


Figure A 3-3. Maximum Stage Profile for the C4 Canal in the C4 Watershed for the 10-year Design Storm

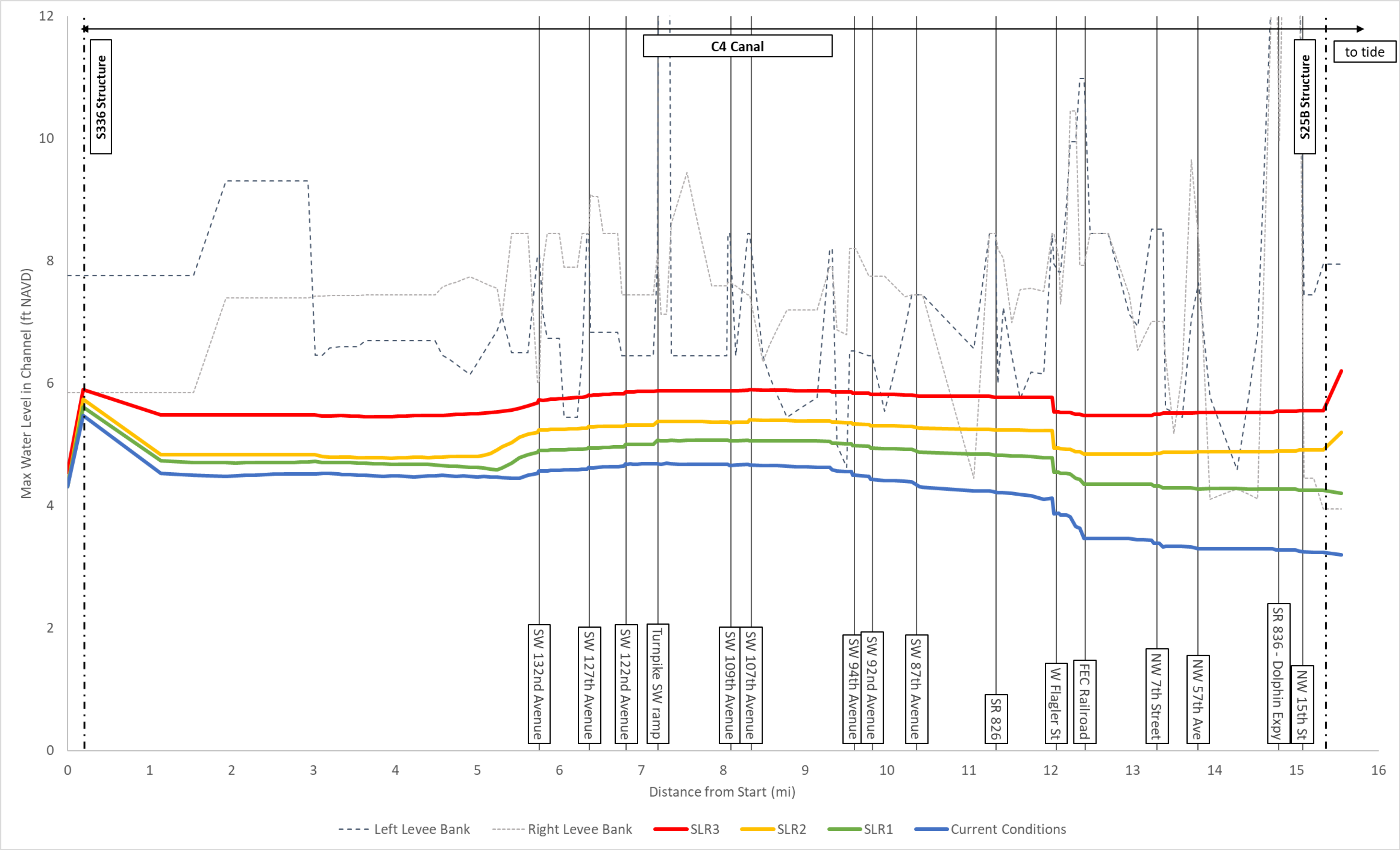


Figure A 3-4. Maximum Stage Profile for the C4 Canal in the C4 Watershed for the 25-year Design Storm

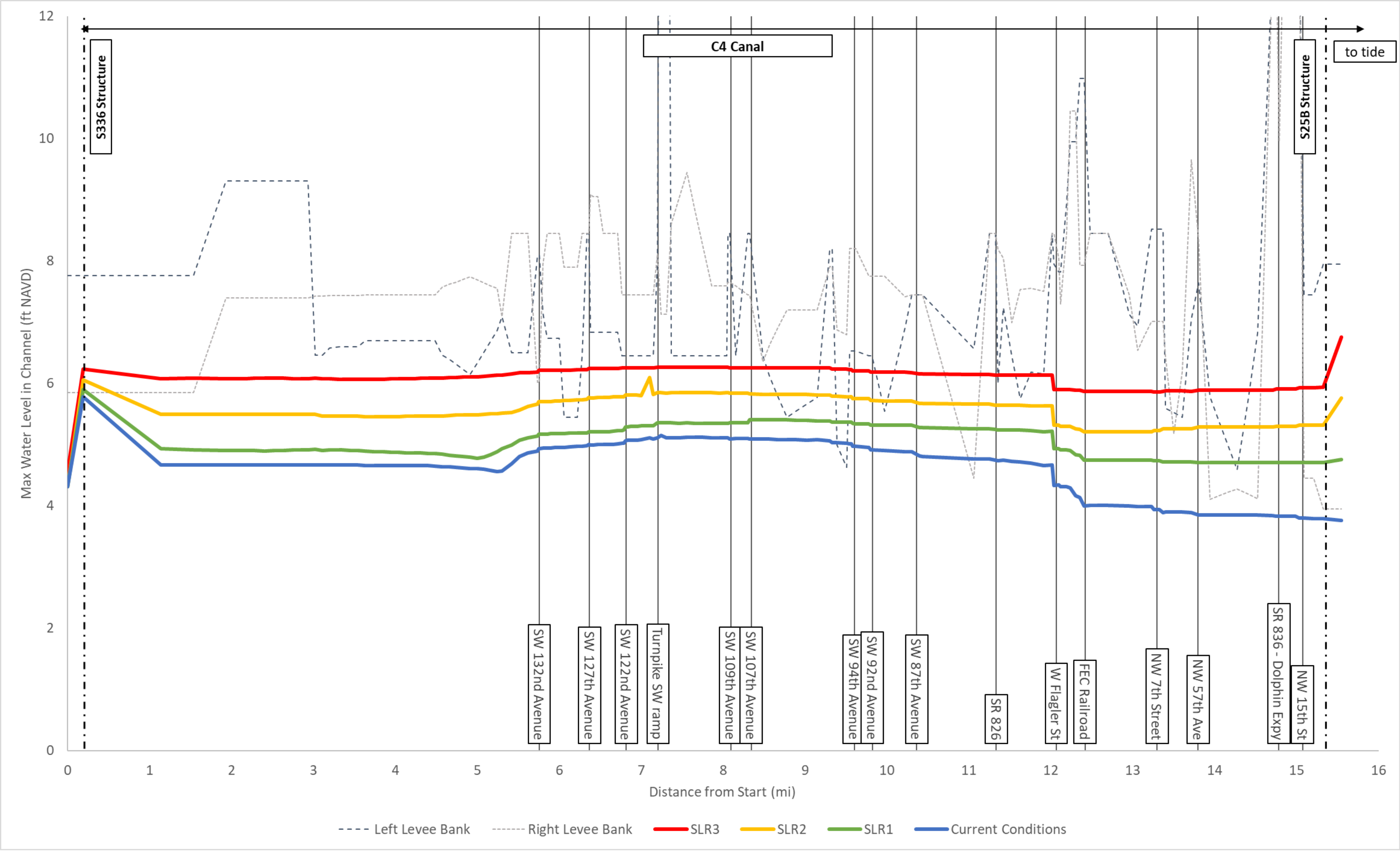
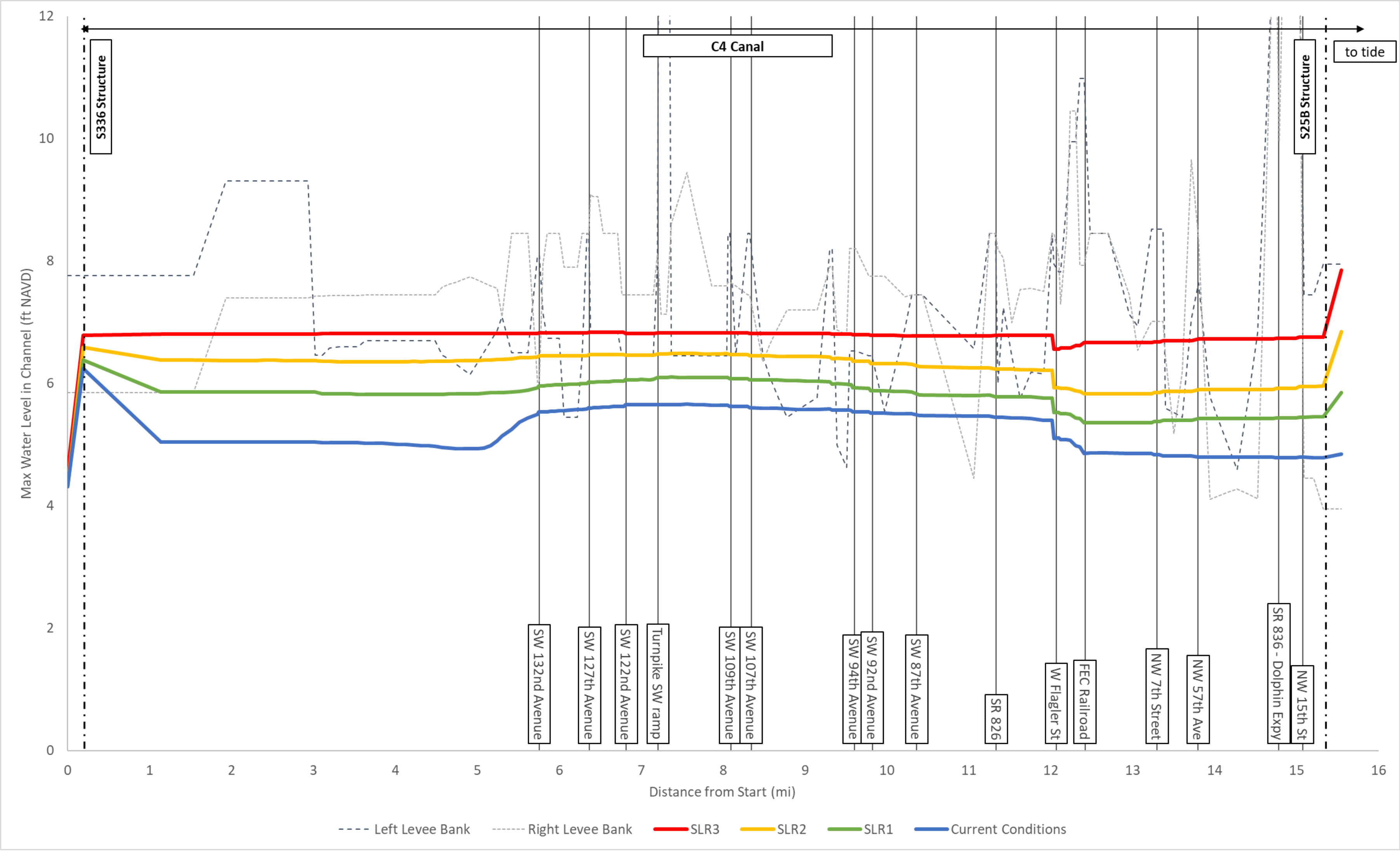


Figure A 3-5. Maximum Stage Profile for the C4 Canal in the C4 Watershed for the 100-year Design Storm



4 C5 Watershed

Figure A 4-1 shows the maximum stage profile for the Comfort Canal Southfork within the C5 Watershed from the S25A to the S25 water control structures for the Current Conditions simulations for all design storm events. **Figure A 4-2** through **Figure A 4-5** show the maximum stage profiles for all Current and Future Conditions for the 5-year, 10-year, 25-year, and 100-year storm events.

Figure A 4-1. Maximum Stage Profile for the C5 Canal in the C5 Watershed for the Current Conditions

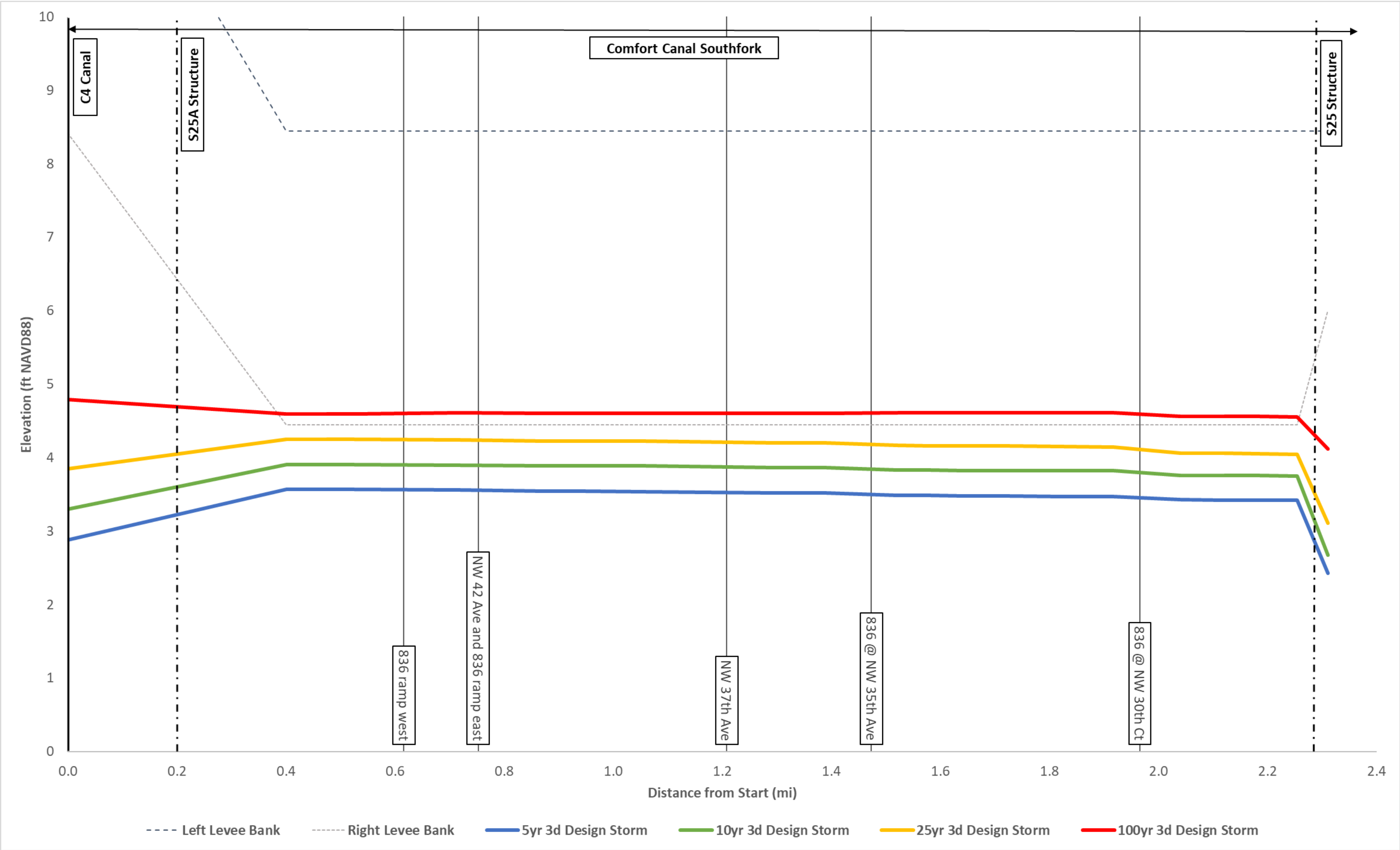


Figure A 4-2. Maximum Stage Profile for the C5 Canal in the C5 Watershed for the 5-year Design Storm

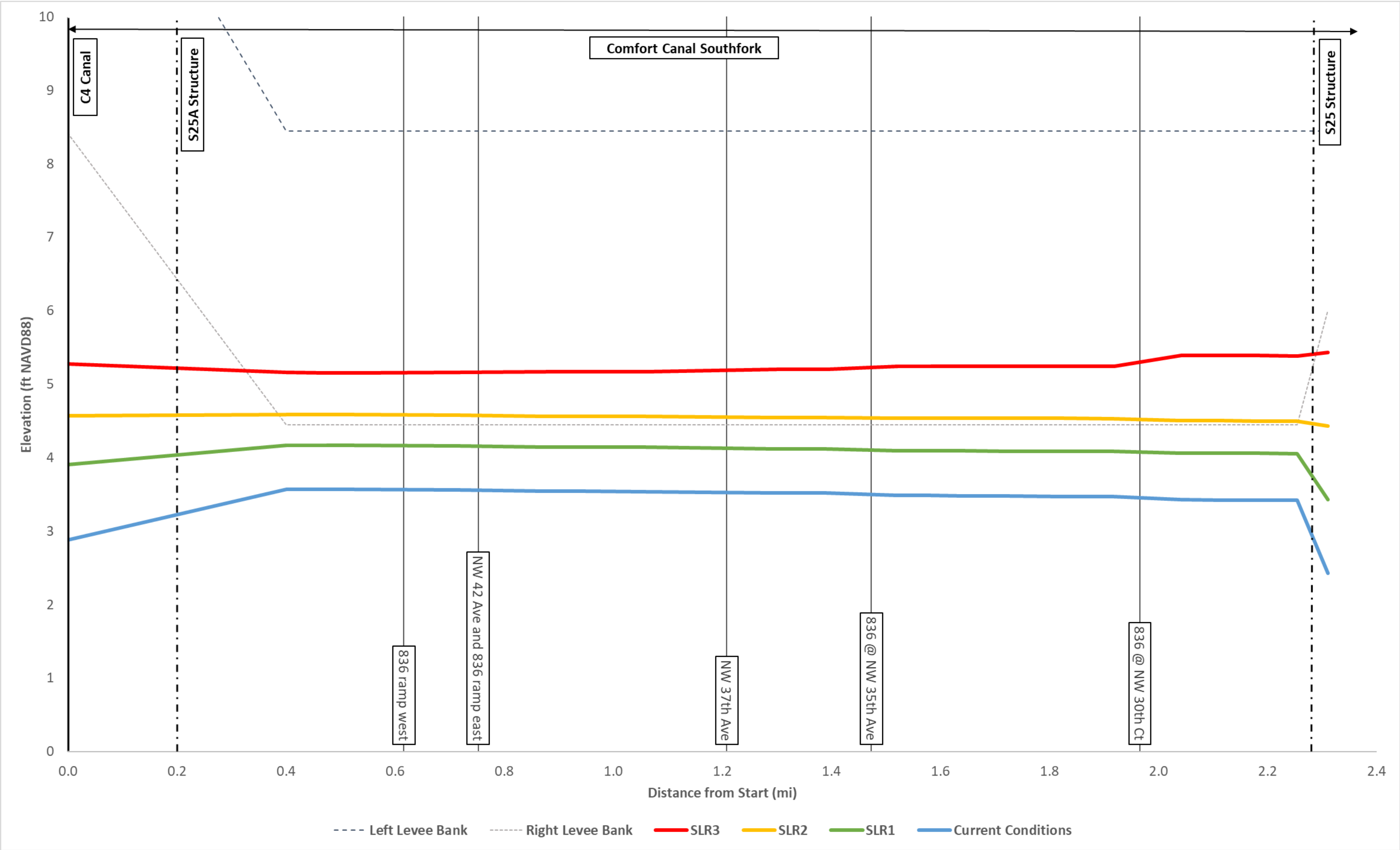


Figure A 4-3. Maximum Stage Profile for the C5 Canal in the C5 Watershed for the 10-year Design Storm

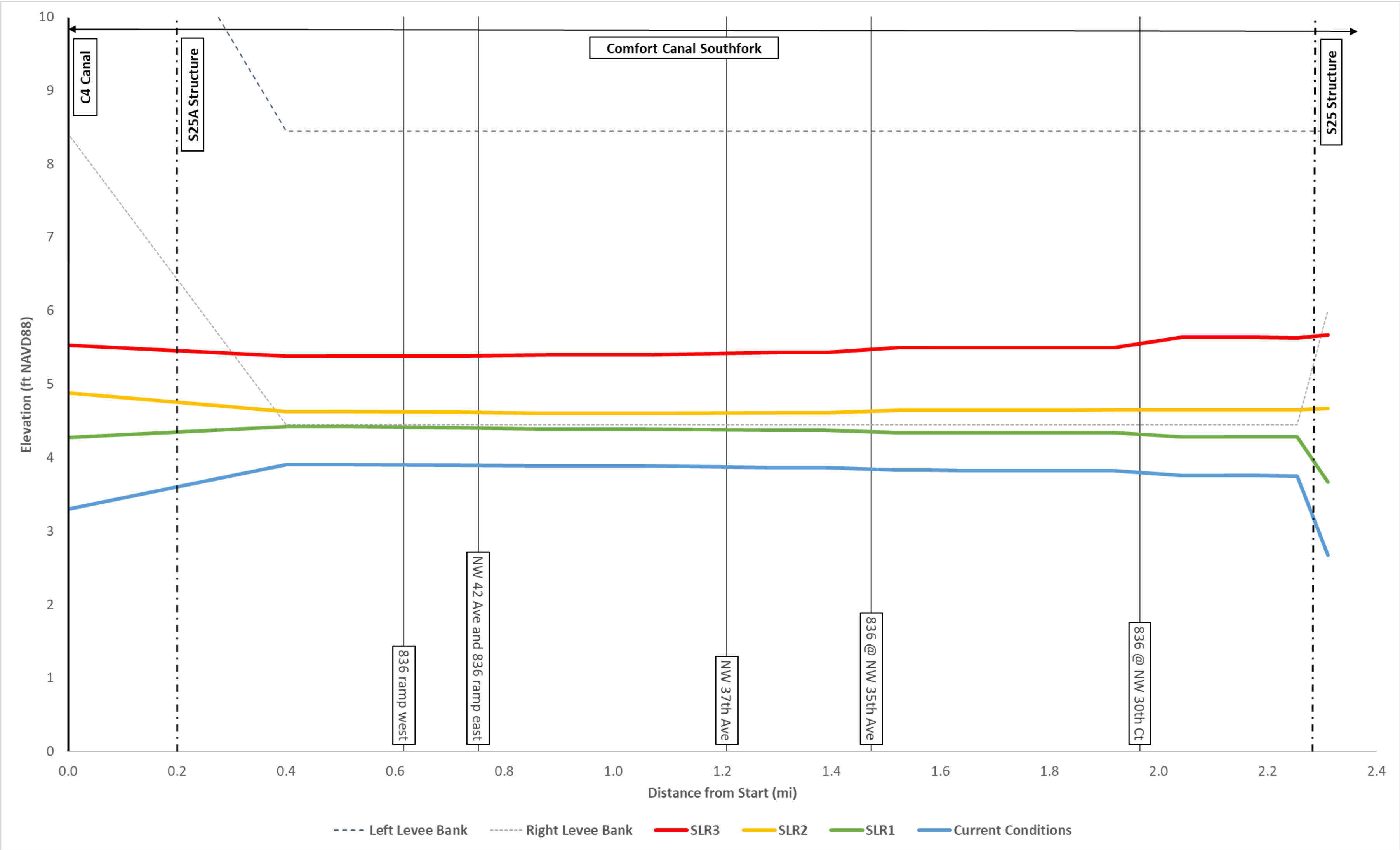


Figure A 4-4. Maximum Stage Profile for the C5 Canal in the C5 Watershed for the 25-year Design Storm

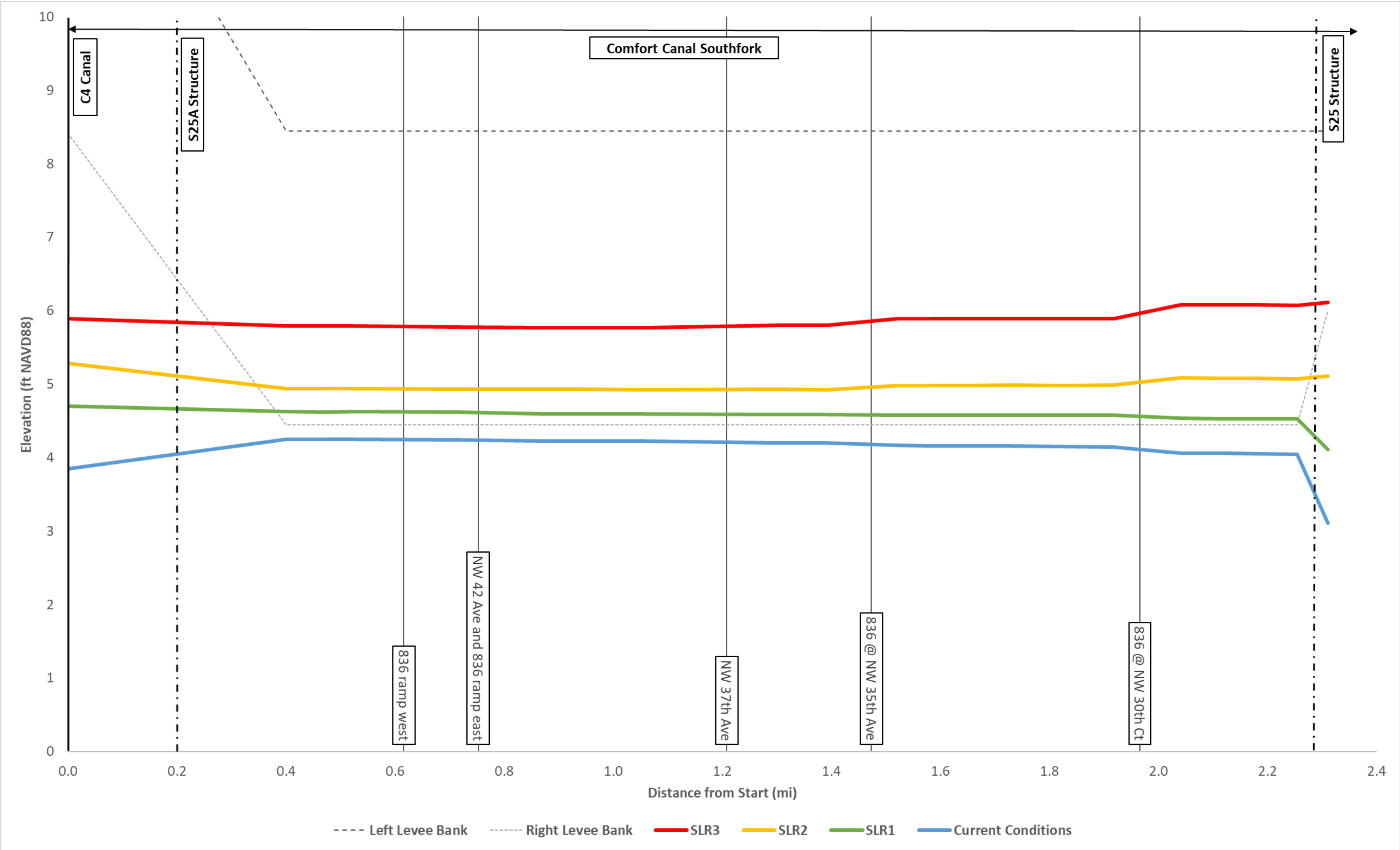
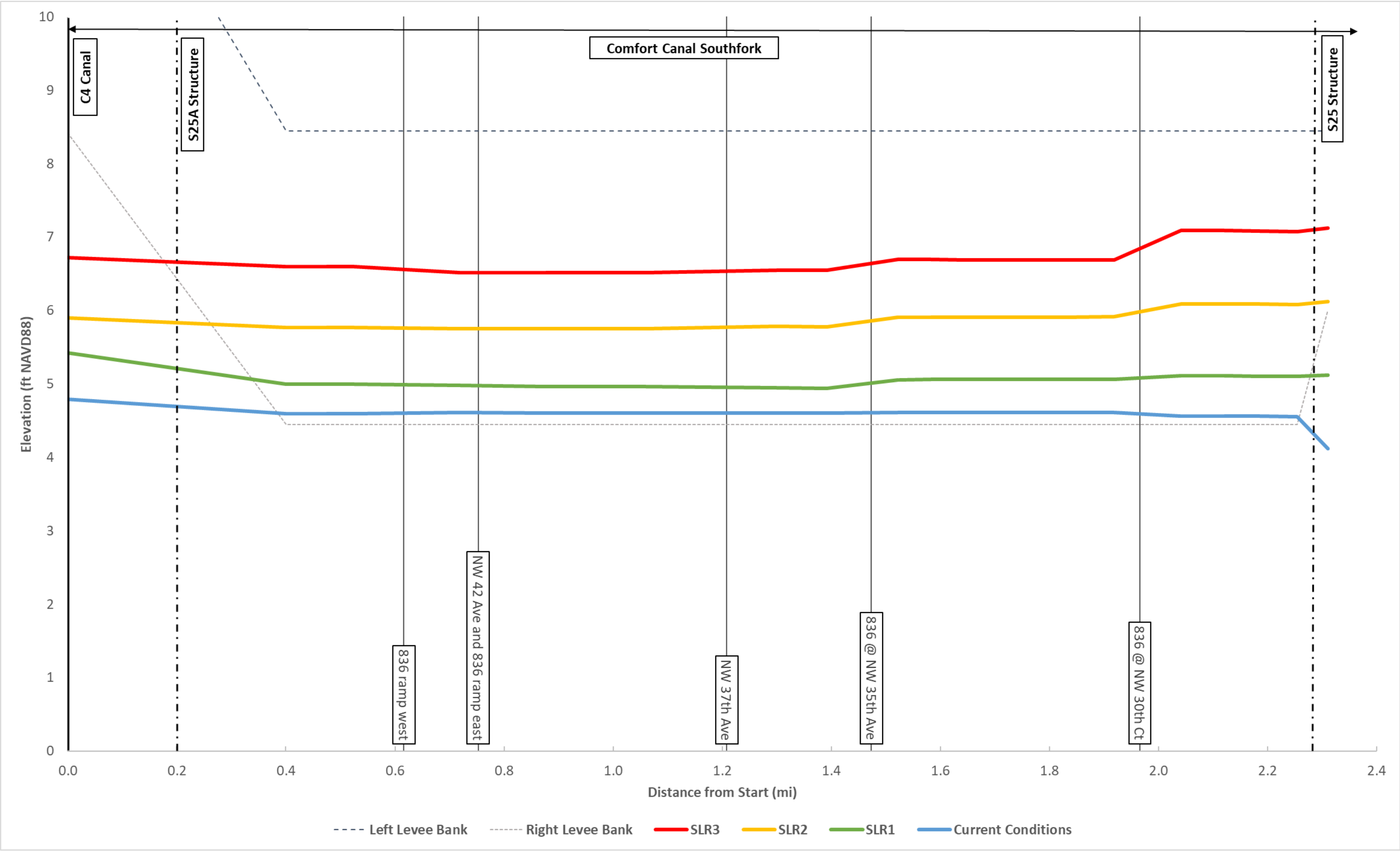


Figure A 4-5. Maximum Stage Profile for the C5 Canal in the C5 Watershed for the 100-year Design Storm



5 C6 Watershed

Figure A 5-1 shows the maximum stage profile for the C6 Canal (or the Miami Canal) within the C6 Watershed from the S31 to the S26 water control structures for the Current Conditions simulations for all design storm events. **Figure A 5-2**, **Figure A 5-3**, **Figure A 5-4**, and **Figure A 5-5** show the maximum stage profiles for all Current and Future Conditions for the 5-year, 10-year, 25-year, and 100-year storm events.

Figure A 5-6 shows the maximum stage profile for the NW 58th St Canal, Dressel's Dairy Canal, and the C6 Canal (or the Miami Canal) within the C6 Watershed to the S26 water control structure for the Current Conditions simulations for all design storm events. **Figure A 5-7**, **Figure A 5-8**, **Figure A 5-9**, and **Figure A 5-10** show the maximum stage profiles for all Current and Future Conditions for the 5-year, 10-year, 25-year, and 100-year storm events.

Figure A 5-1. Maximum Stage Profile for the C6 Canal in the C6 Watershed for the Current Conditions

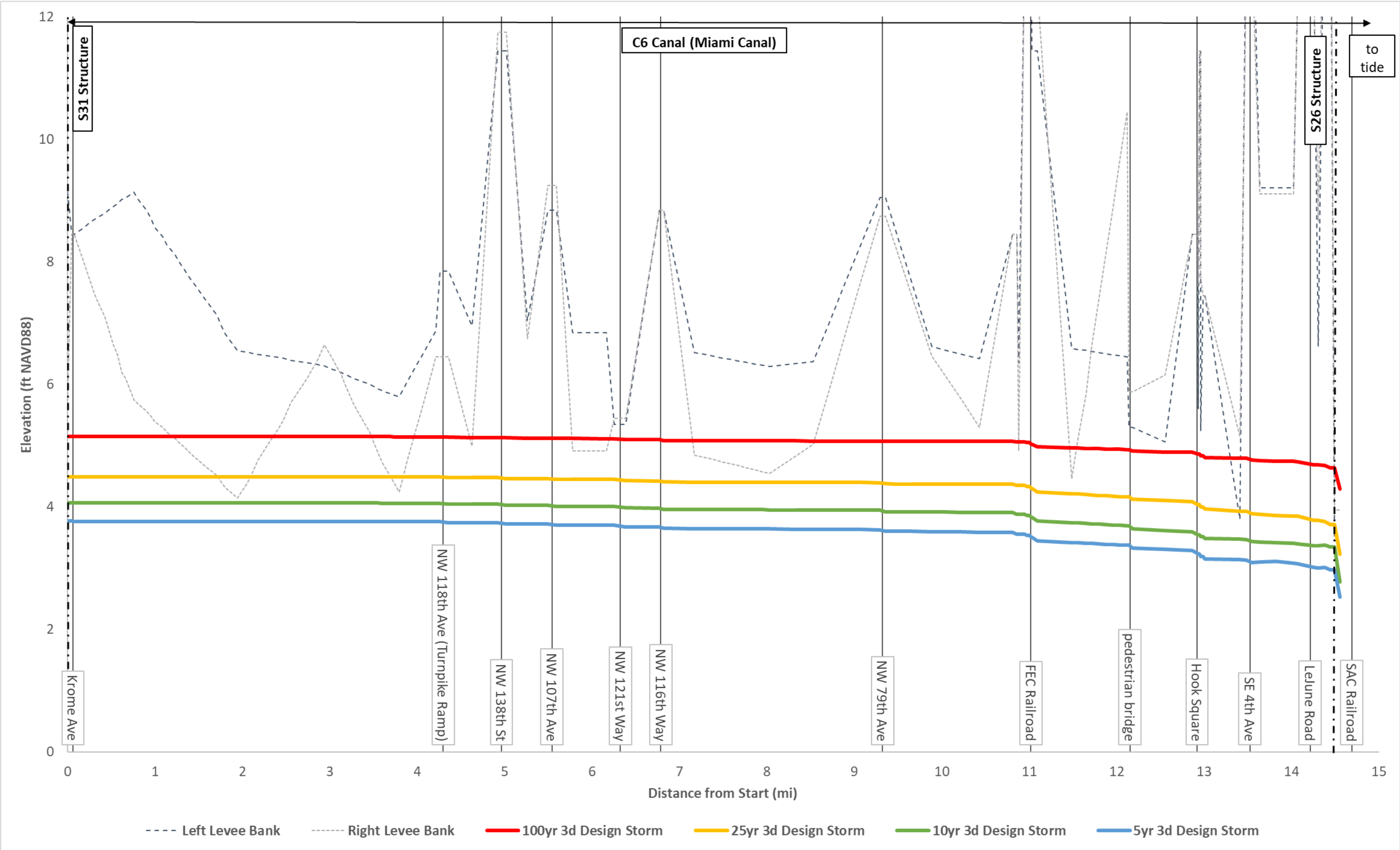


Figure A 5-2. Maximum Stage Profile for the C6 Canal in the C6 Watershed for the 5-year Design Storm

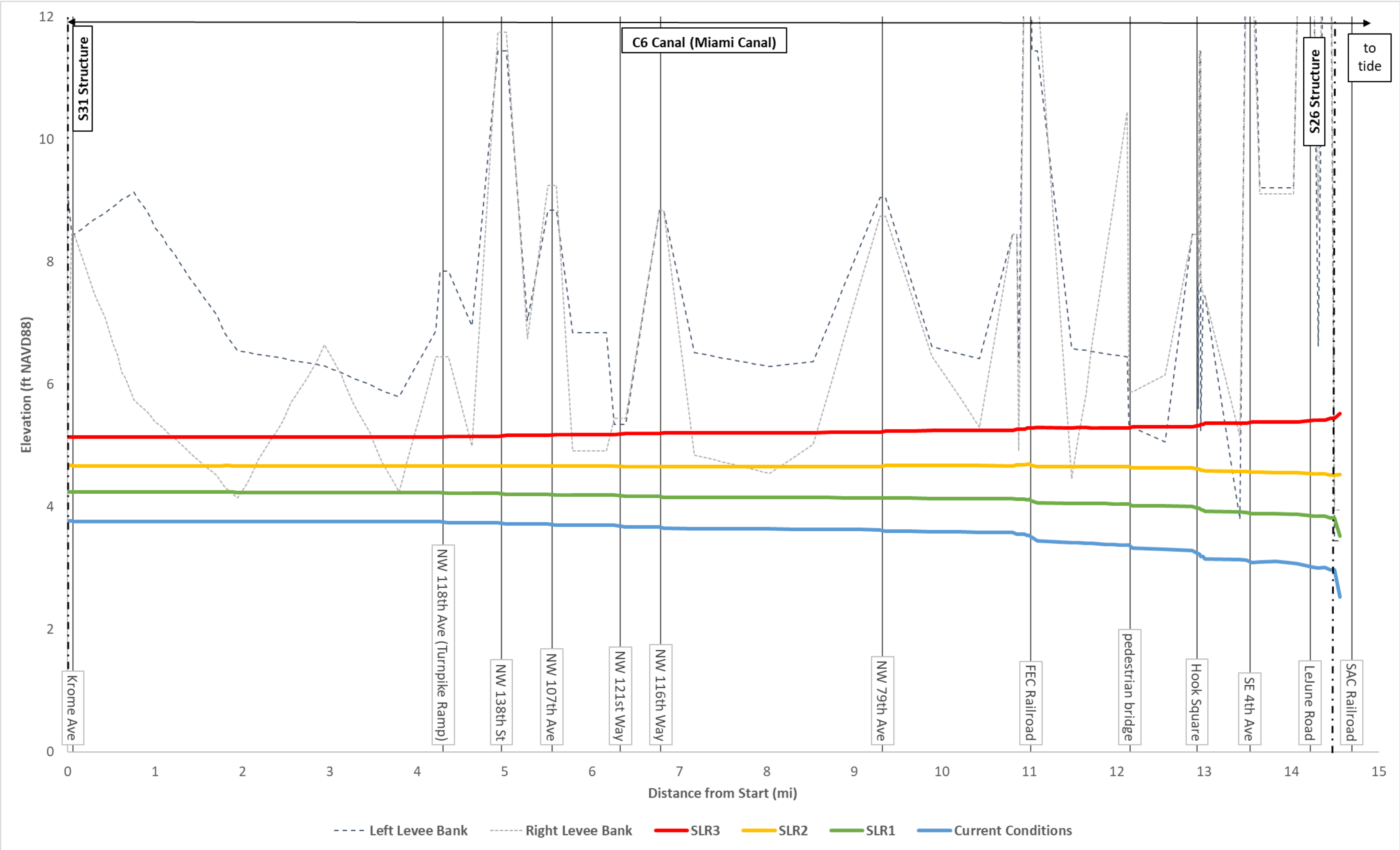


Figure A 5-3. Maximum Stage Profile for the C6 Canal in the C6 Watershed for the 10-year Design Storm

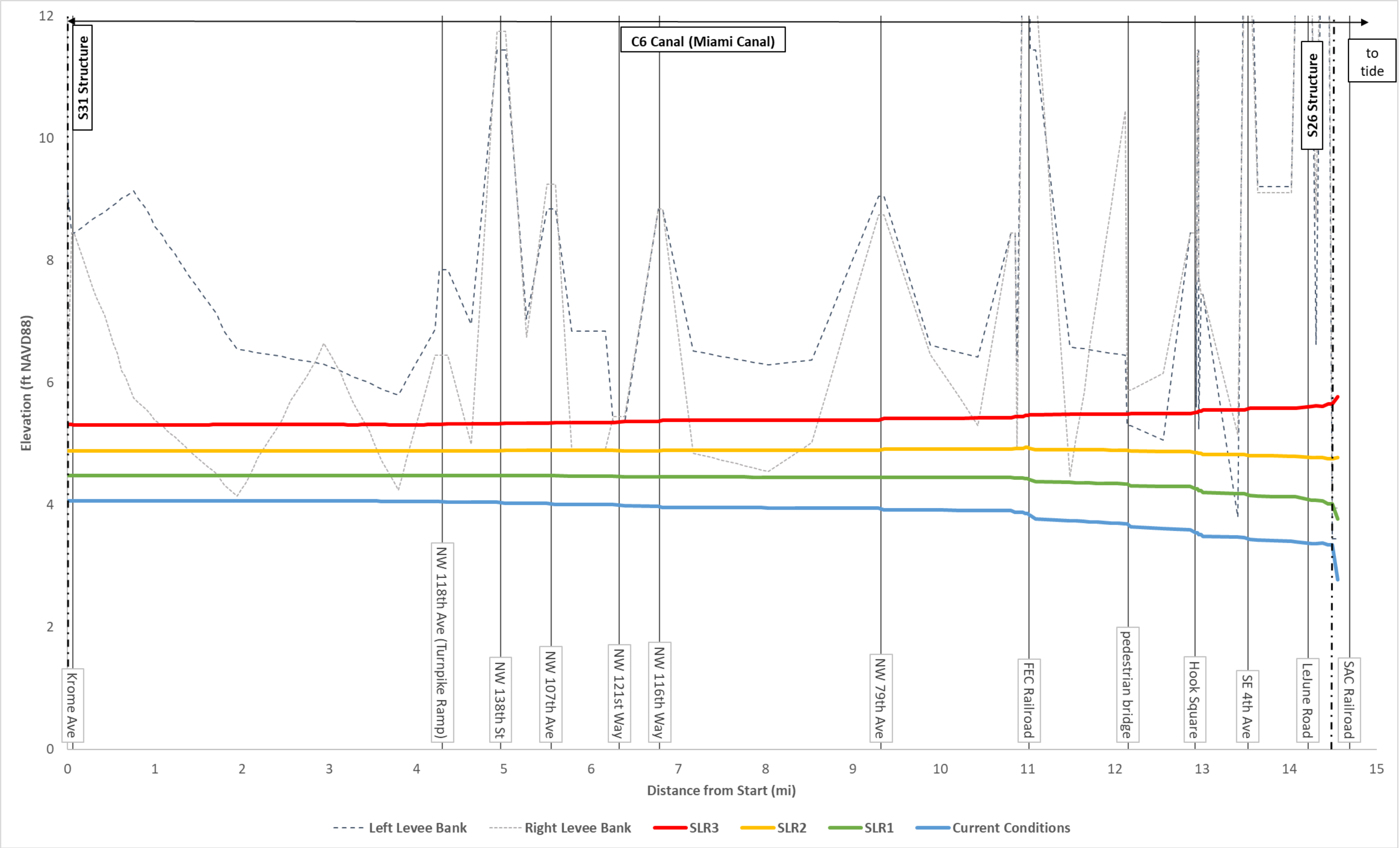


Figure A 5-4. Maximum Stage Profile for the C6 Canal in the C6 Watershed for the 25-year Design Storm

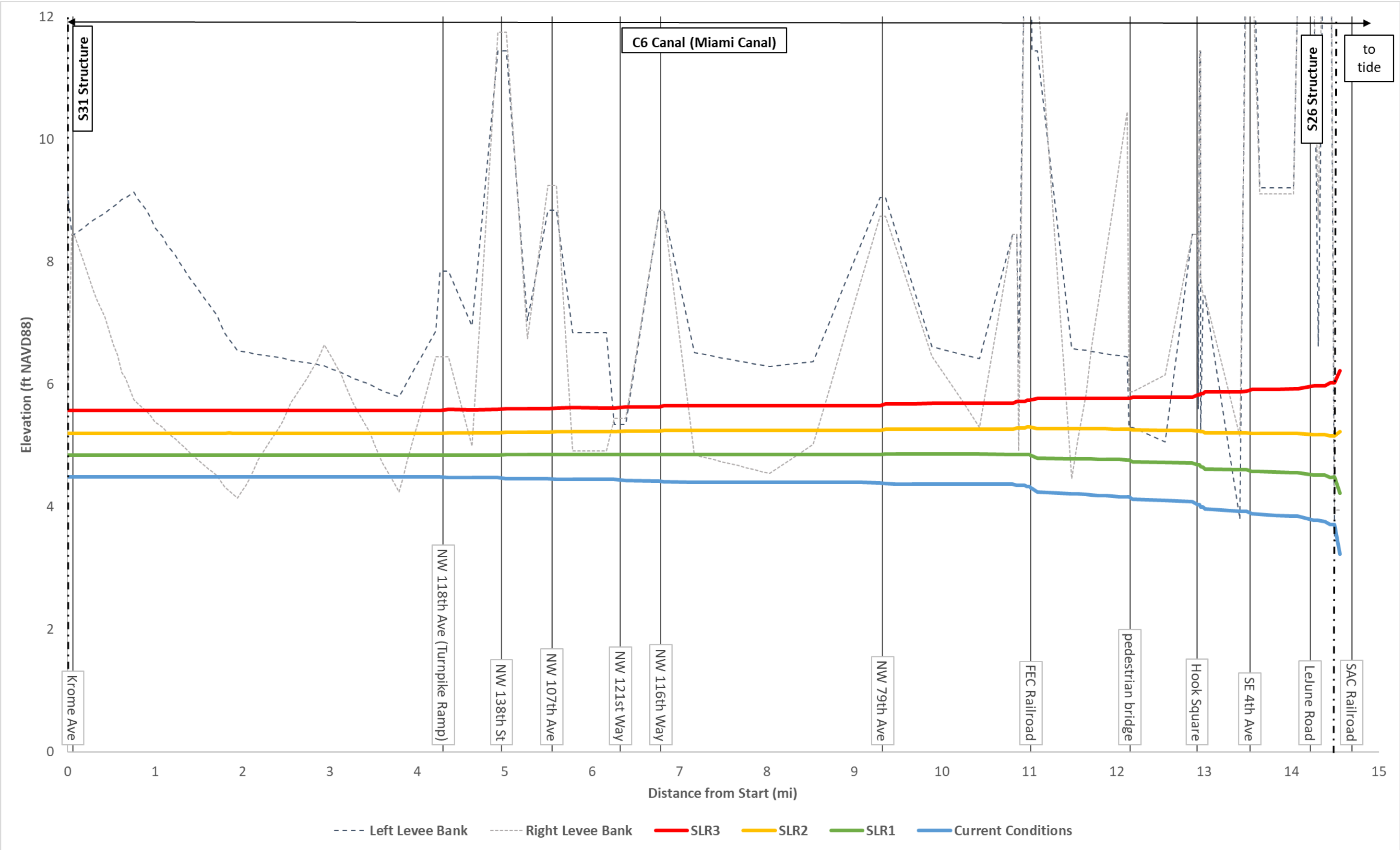


Figure A 5-5. Maximum Stage Profile for the C6 Canal in the C6 Watershed for the 100-year Design Storm

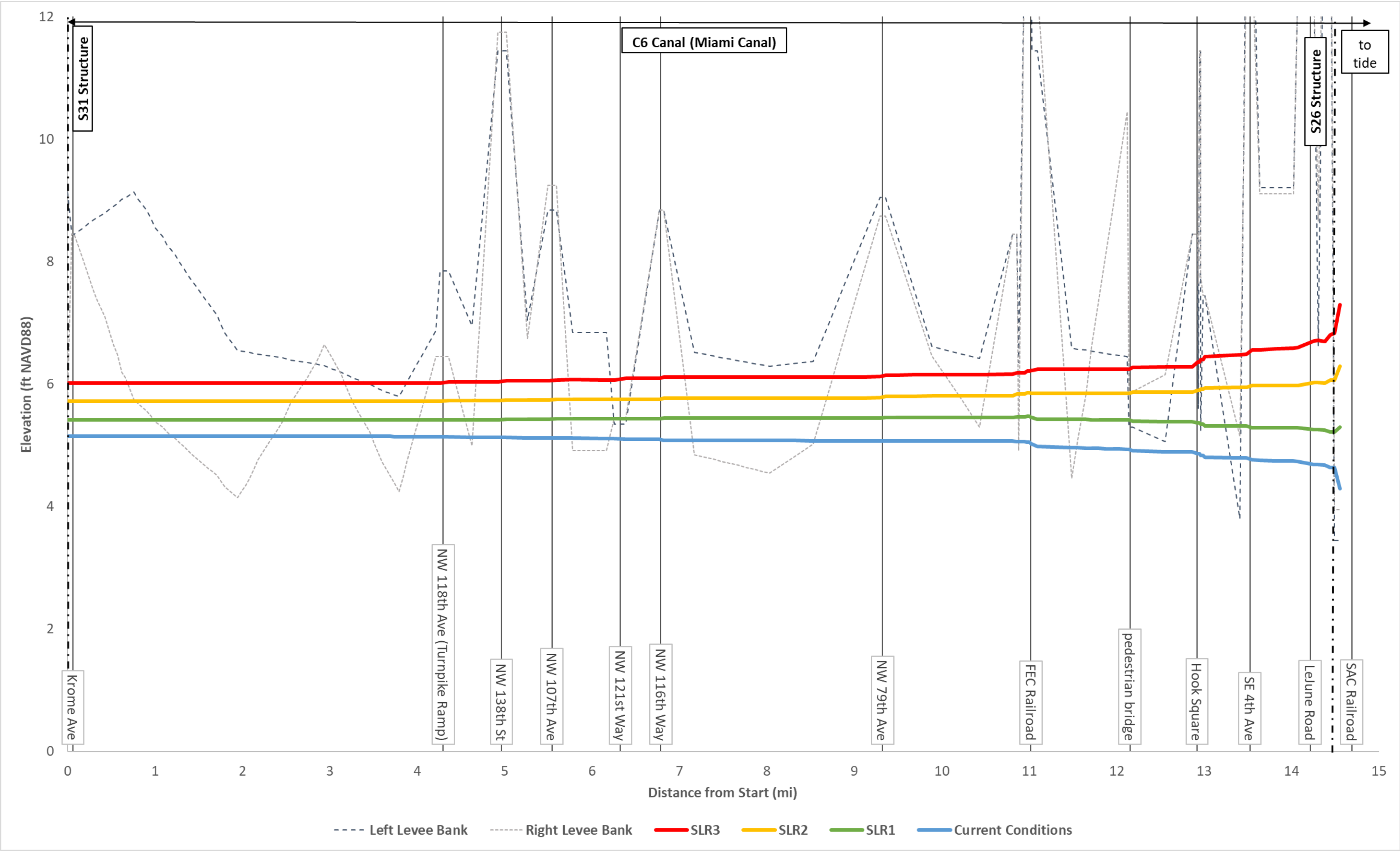


Figure A 5-6. Maximum Stage Profile for the NW 58th St Canal & C6 Canal in the C6 Watershed for the Current Conditions

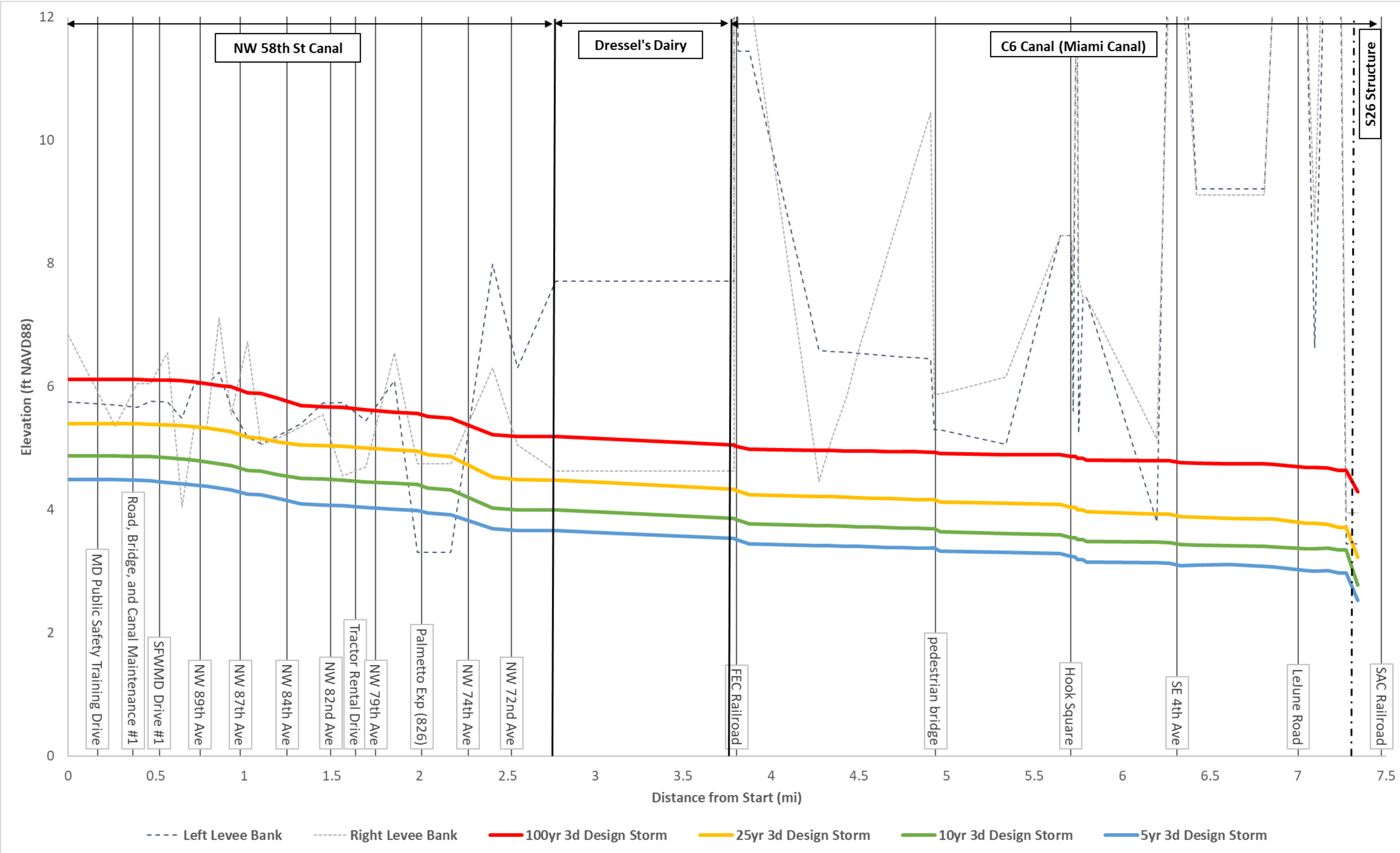


Figure A 5-7. Maximum Stage Profile for the NW 58th St Canal & C6 Canal in the C6 Watershed for the 5-year Design Storm

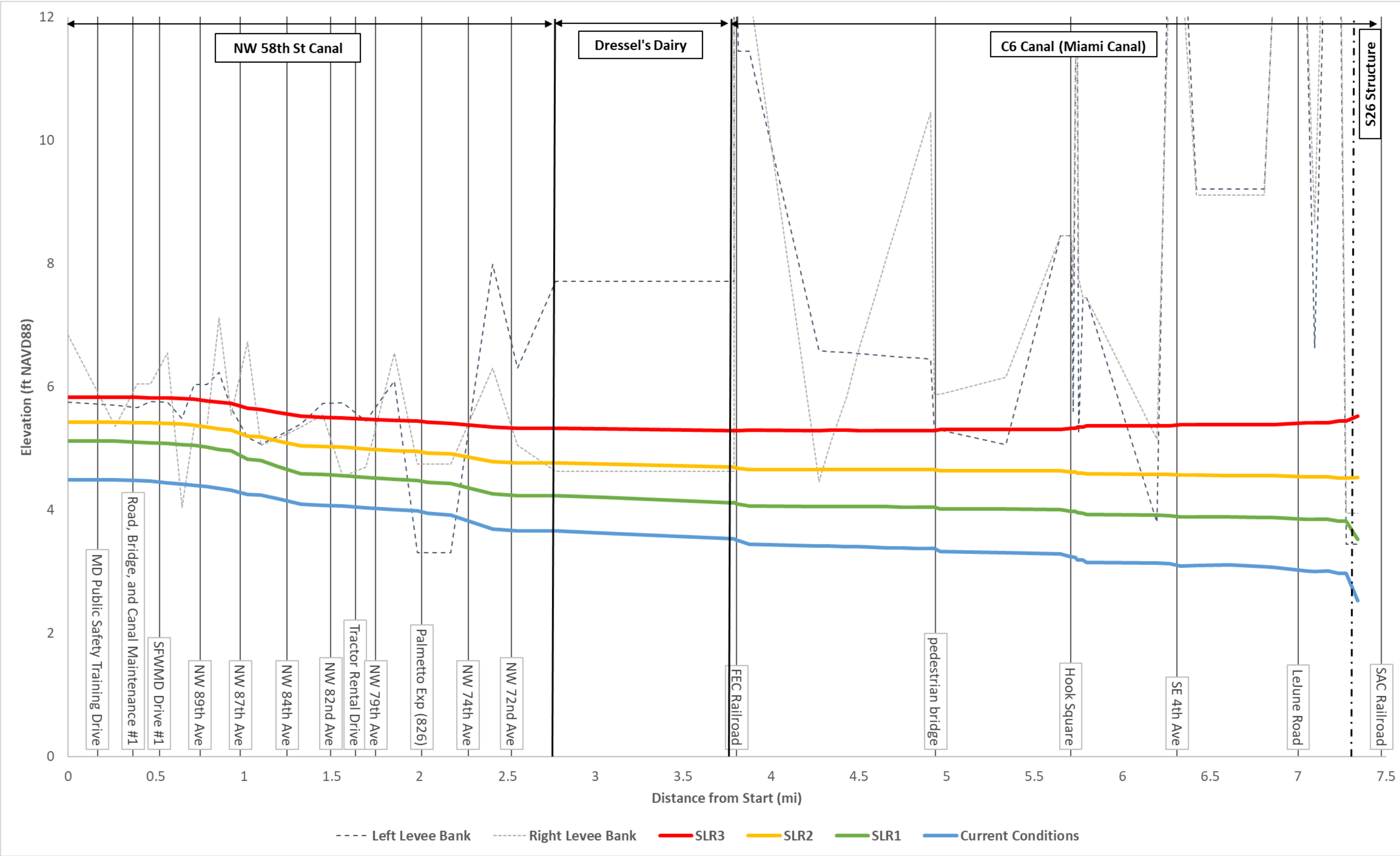


Figure A 5-8. Maximum Stage Profile for the NW 58th St Canal & C6 Canal in the C6 Watershed for the 10-year Design Storm

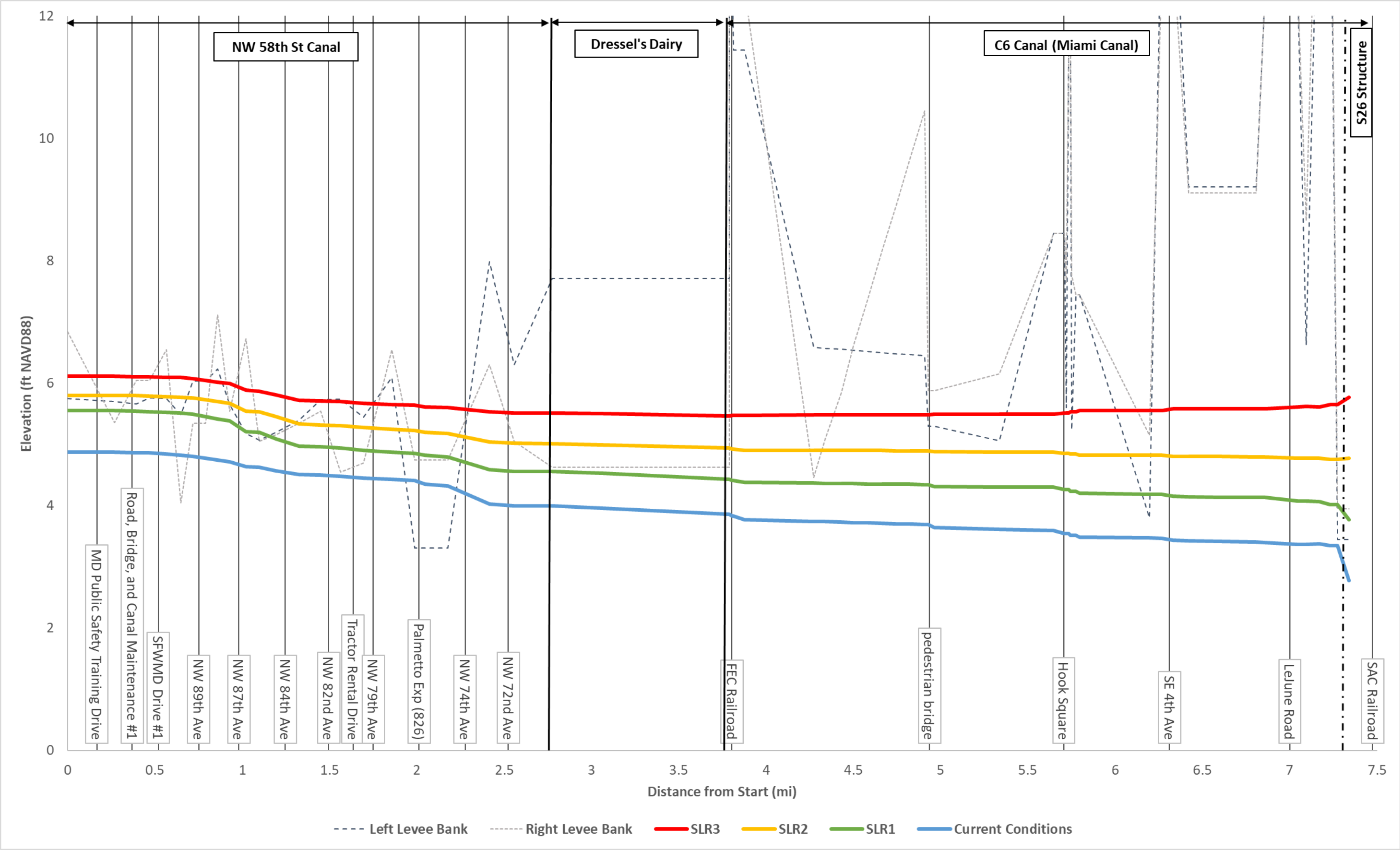


Figure A 5-9. Maximum Stage Profile for the NW 58th St Canal & C6 Canal in the C6 Watershed for the 25-year Design Storm

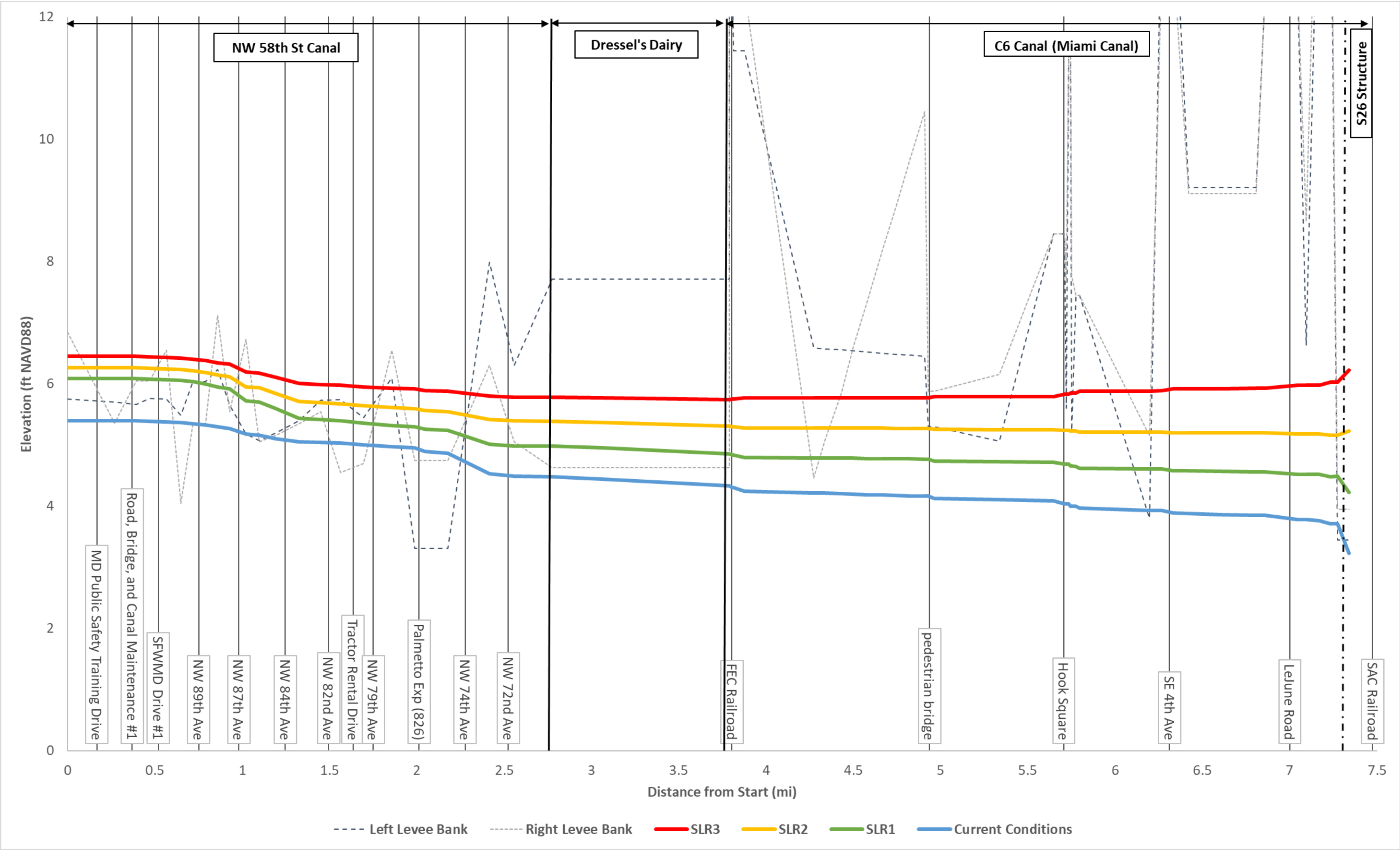
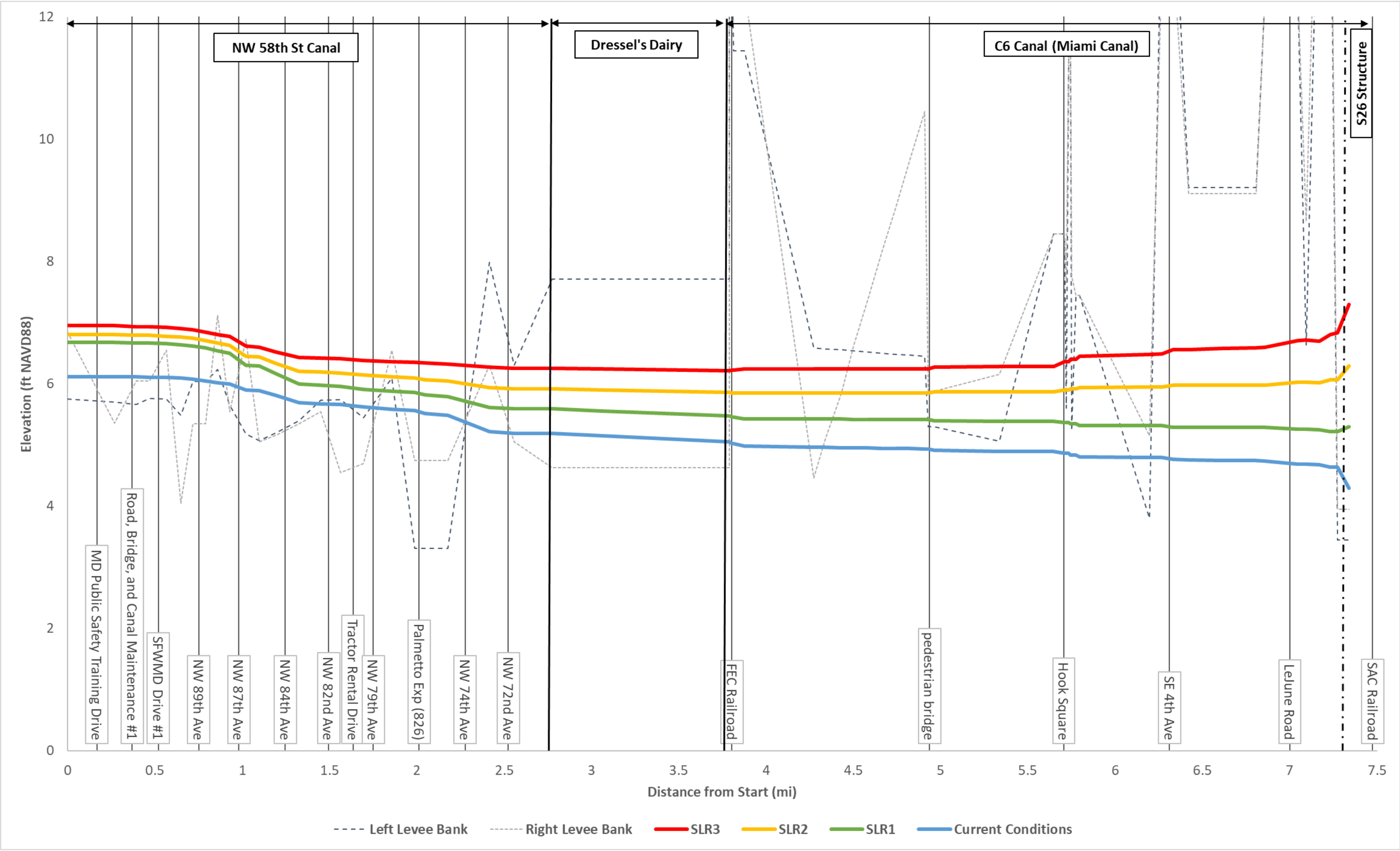


Figure A 5-10. Maximum Stage Profile for the NW 58th St Canal & C6 Canal in the C6 Watershed for the 100-year Design Storm



APPENDIX B

PM3 Structure Performance Figures

Contents

Figures

Appendix B – PM#3 Structure Performance

The figures provided in this appendix include instantaneous and 12-hour average discharge, HW, and TW at the tidal structures for each watershed. Included are profiles for each design storm return period (5-year, 10-year, 25-year, and 100-year) and each SLR condition (i.e. Current Conditions, SLR1, SLR2, and SLR3).

1 C2 Watershed

Figure B 1-1 through **Figure B 1-32** show the instantaneous and 12-hour average discharge, HW, and TW at S22 for each SLR condition for the 100-, 25-, 10-, and 5-year design storm events.

Figure B 1-1. Instantaneous Discharge and Stages at S22 for the 100-year 3-day Current Conditions Event

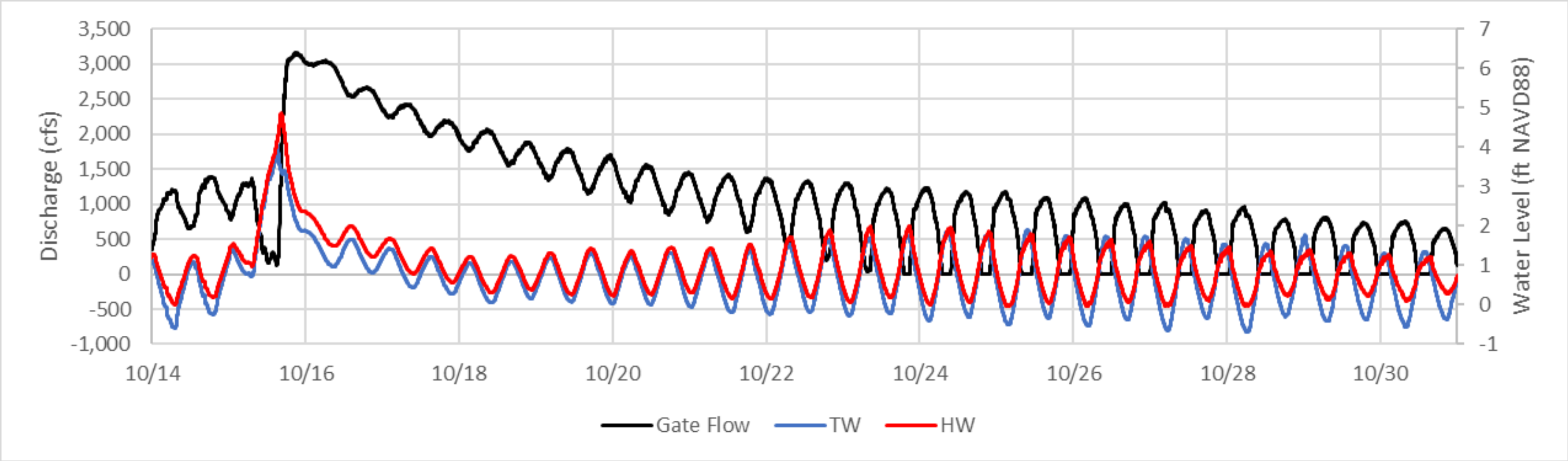


Figure B 1-2. Instantaneous Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR1) Event

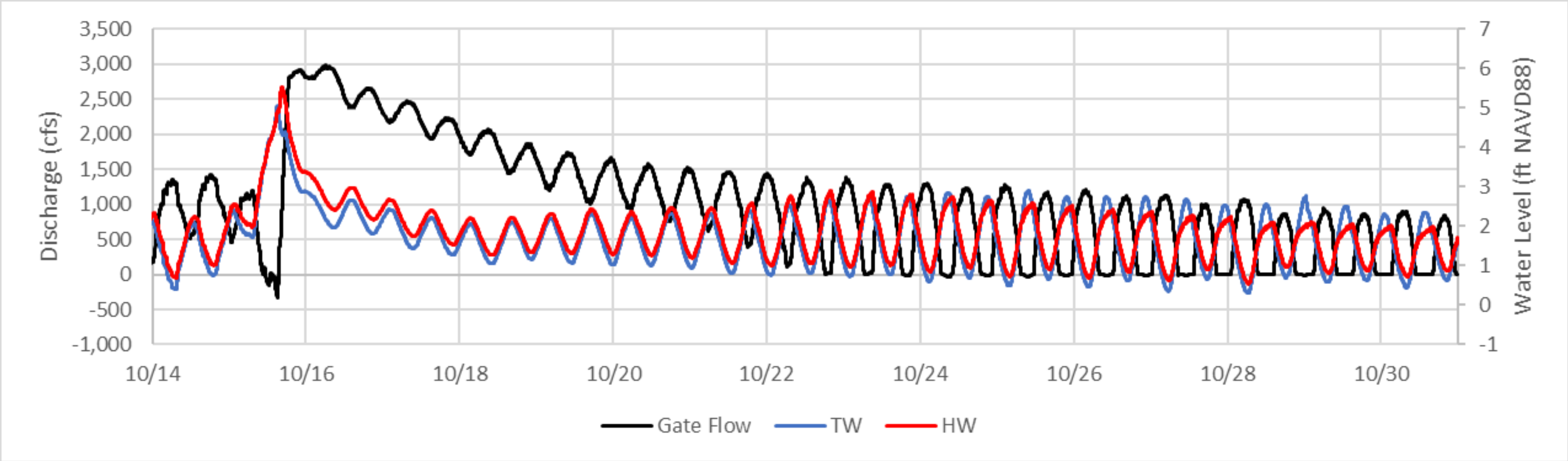


Figure B 1-3. Instantaneous Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR2) Event

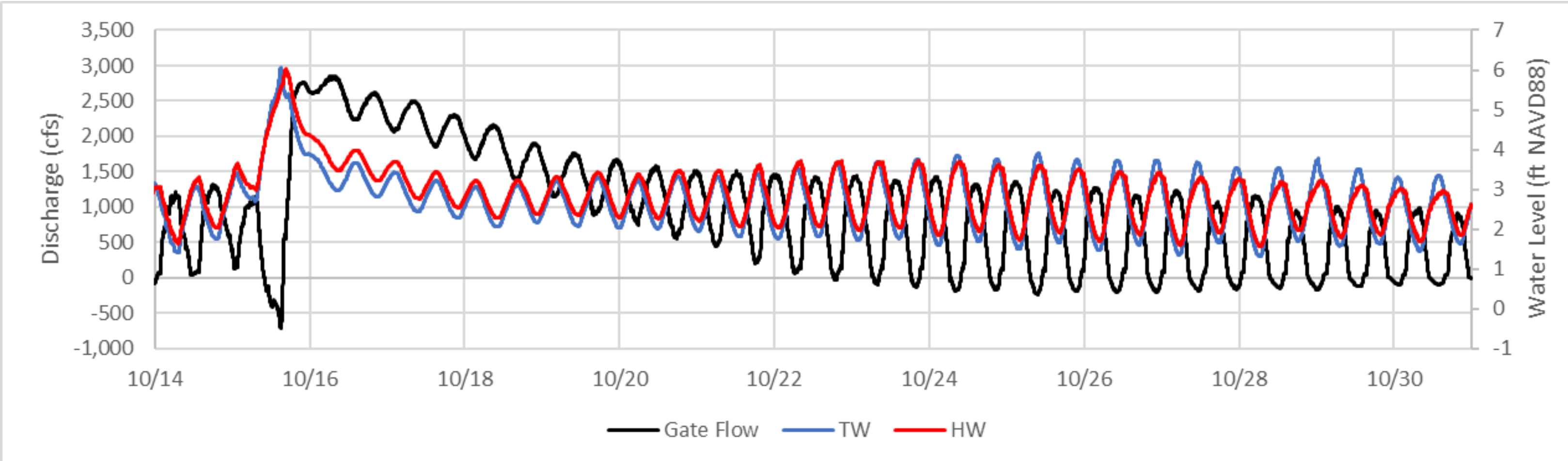


Figure B 1-4. Instantaneous Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR3) Event

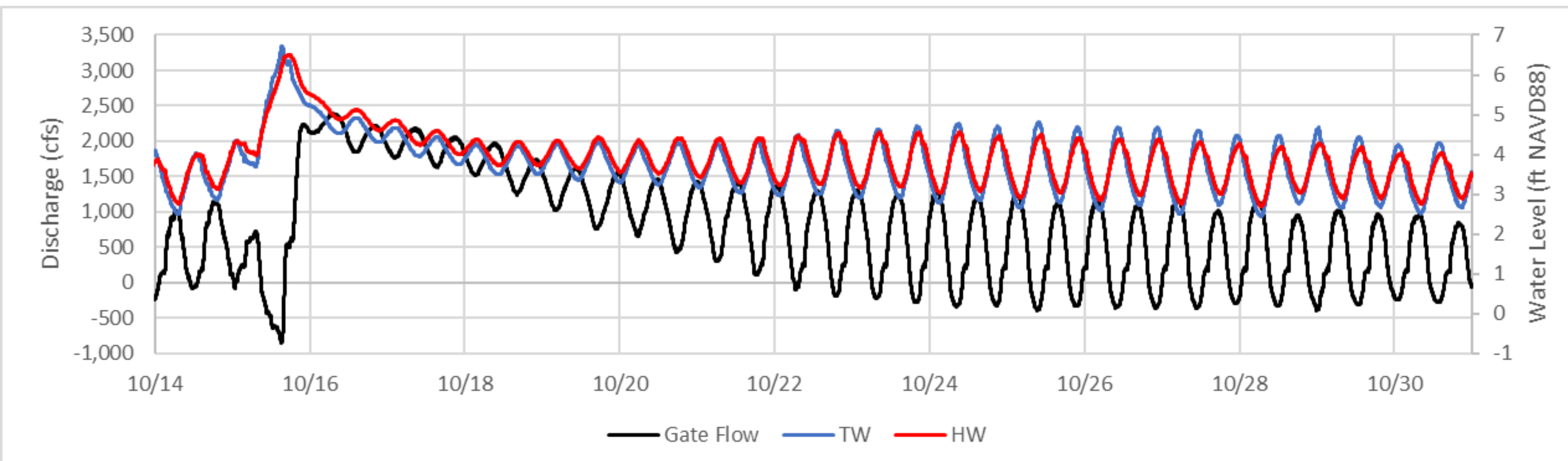


Figure B 1-5. 12-Hour Moving Average Discharge and Stages at S22 for the 100-year 3-day Current Conditions Event

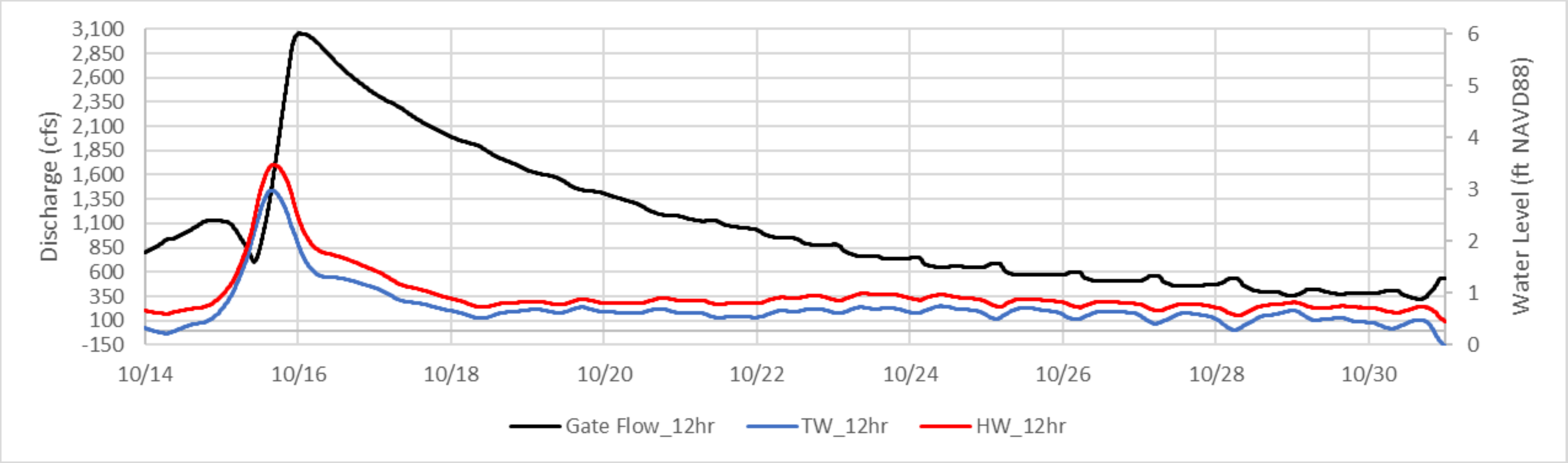


Figure B 1-6. 12-Hour Moving Average Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR1) Event

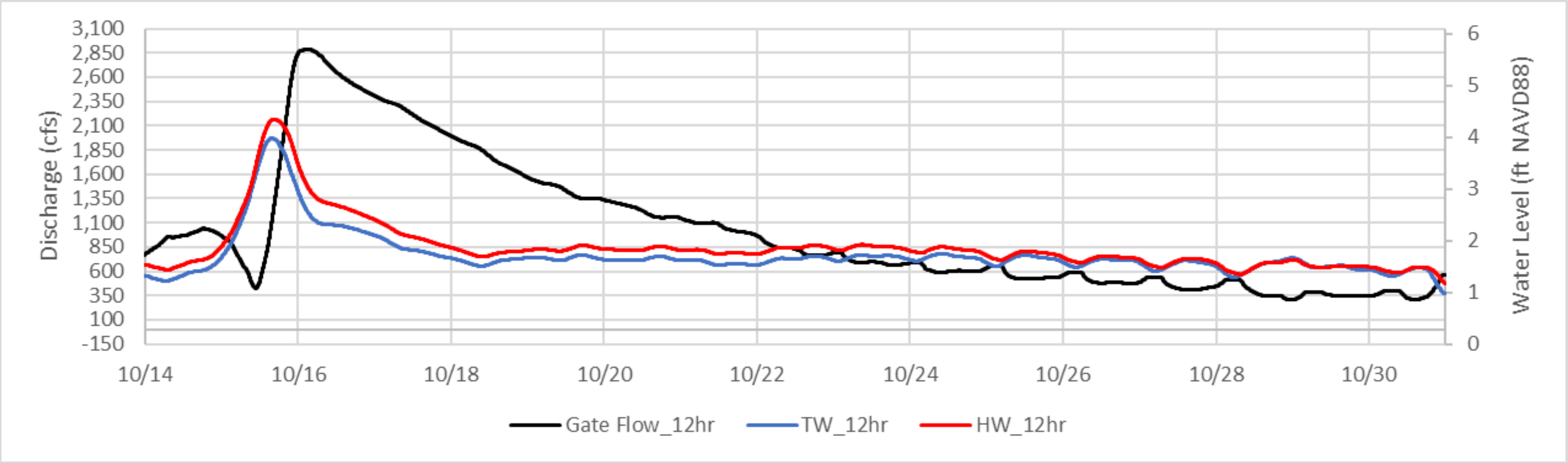


Figure B 1-7. 12-Hour Moving Average Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR2) Event

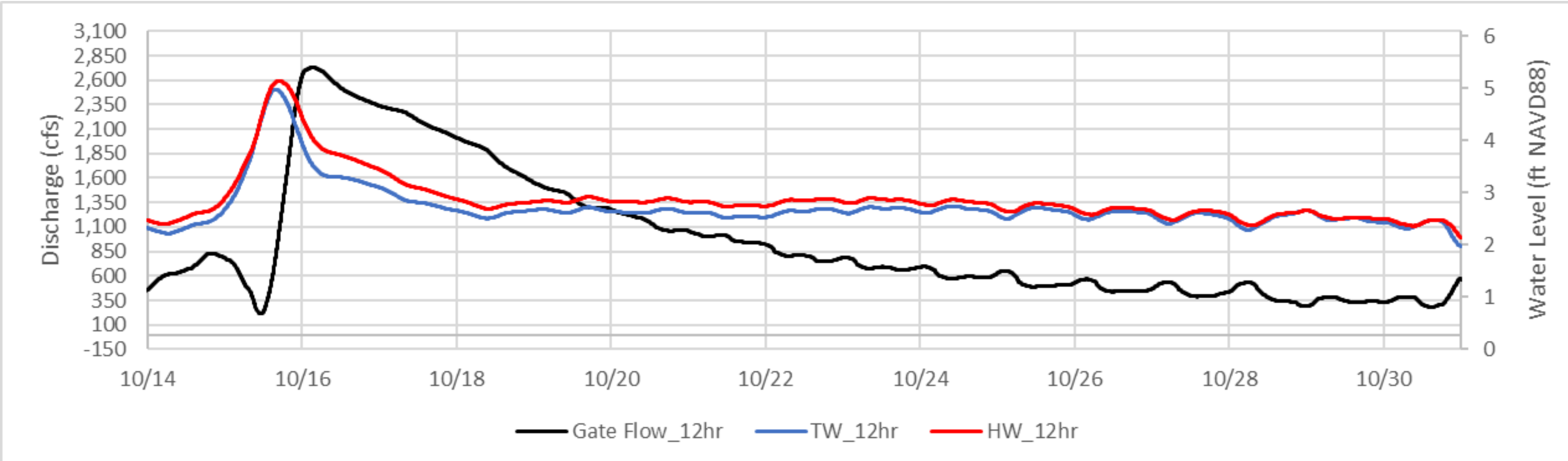


Figure B 1-8. 12-Hour Moving Average Discharge and Stages at S22 for the 100-year 3-day Future Conditions (SLR3) Event

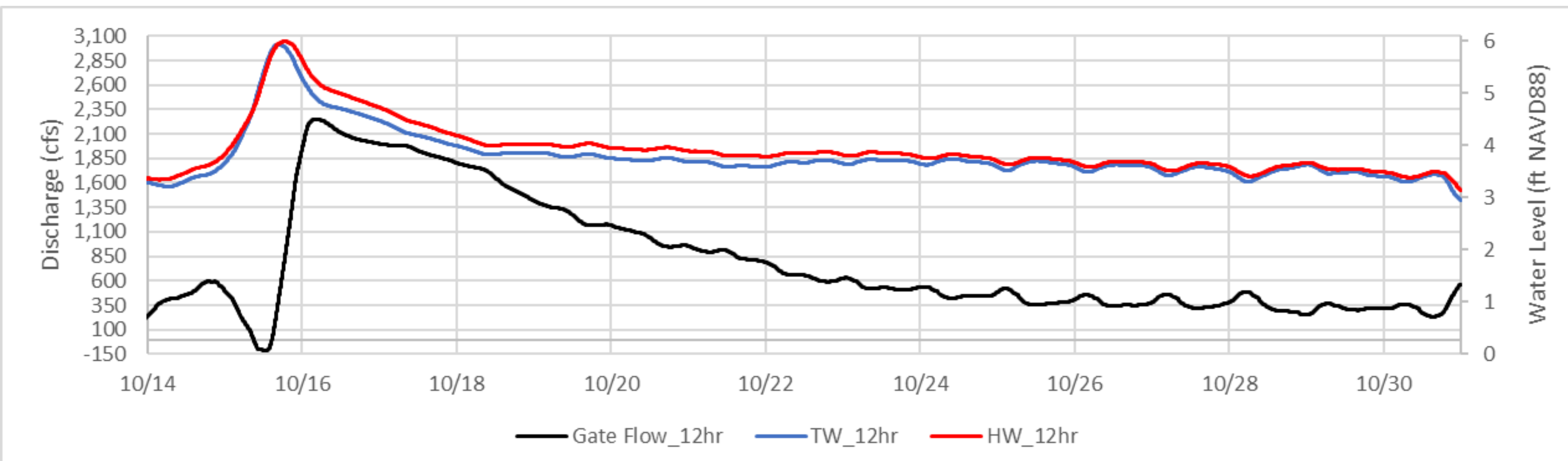


Figure B 1-9. Instantaneous Discharge and Stages at S22 for the 25-year 3-day Current Conditions Event

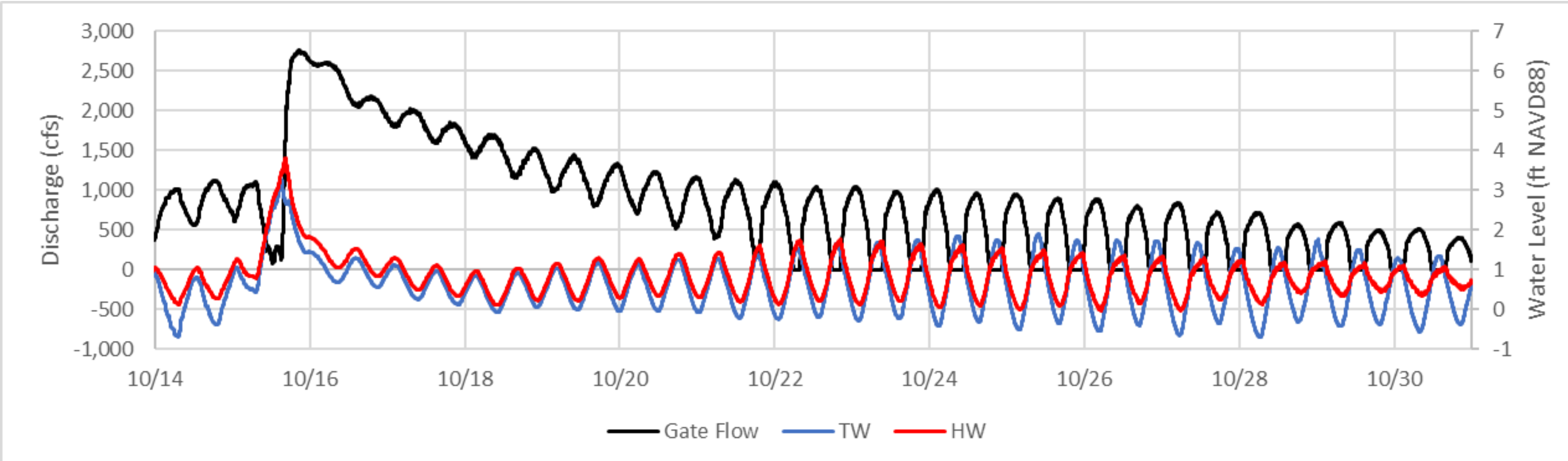


Figure B 1-10. Instantaneous Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR1) Event

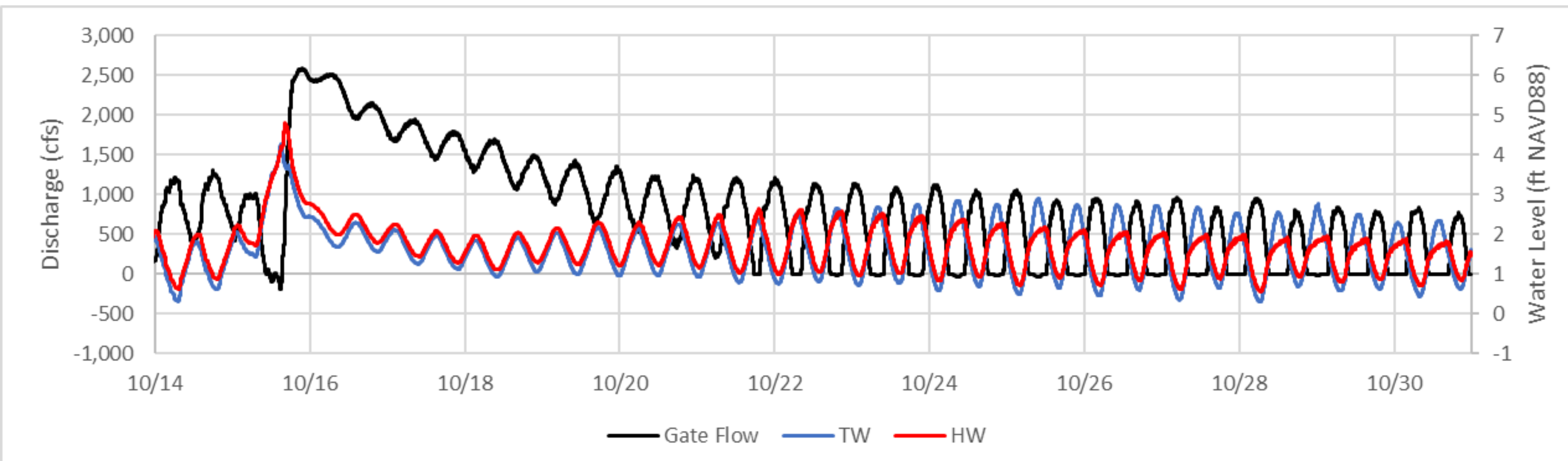


Figure B 1-11. Instantaneous Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR2) Event

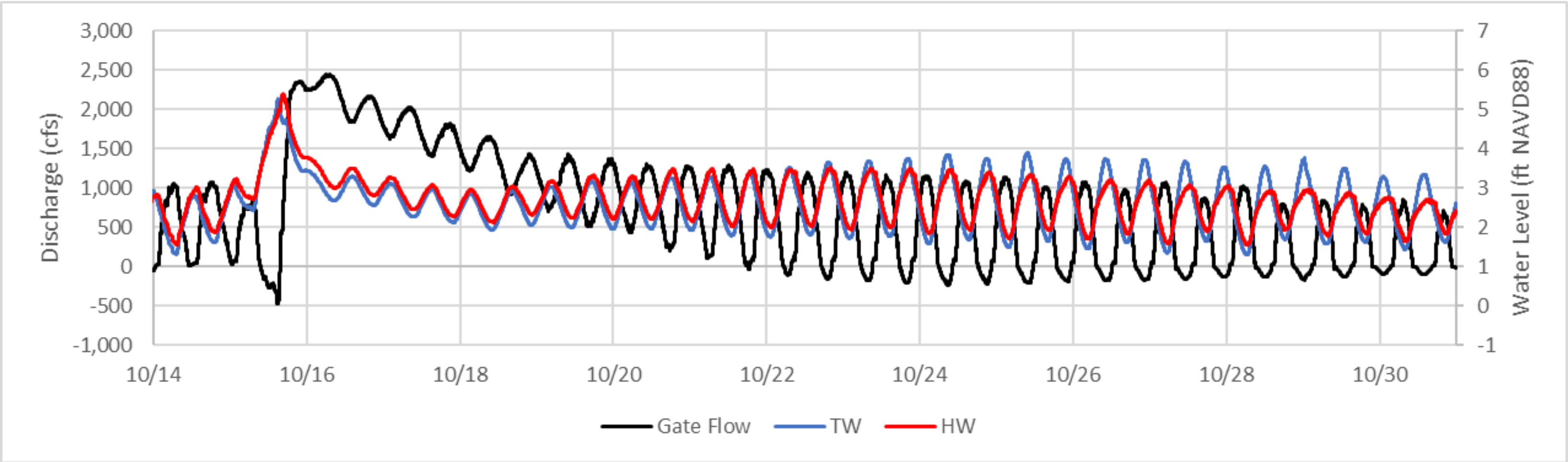


Figure B 1-12. Instantaneous Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR3) Event

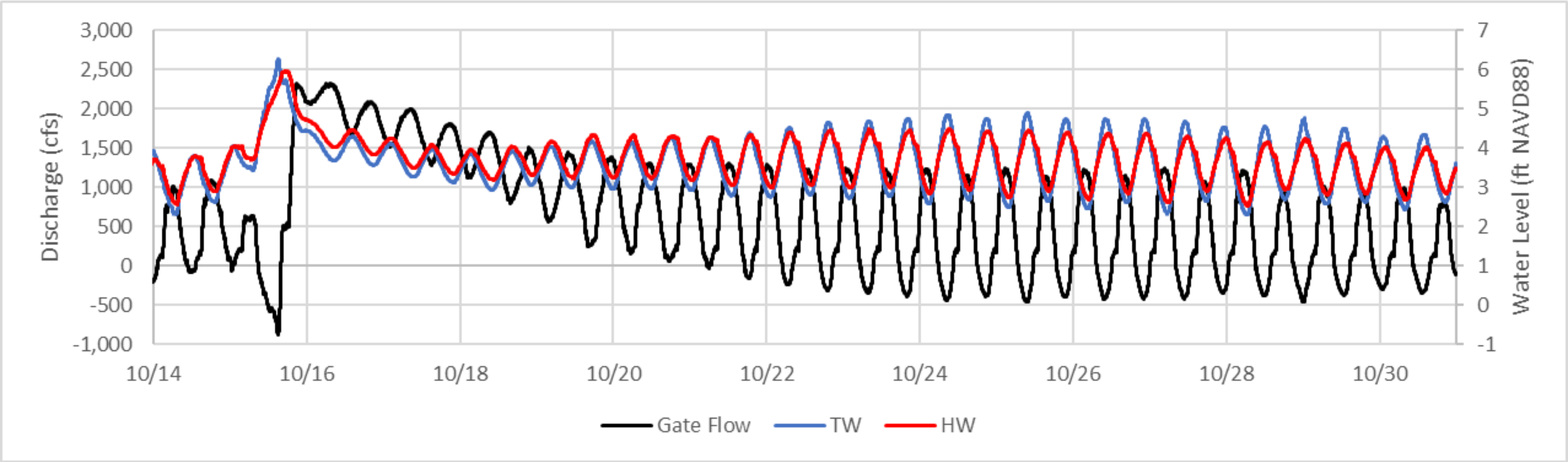


Figure B 1-13. 12-Hour Moving Average Discharge and Stages at S22 for the 25-year 3-day Current Conditions Event

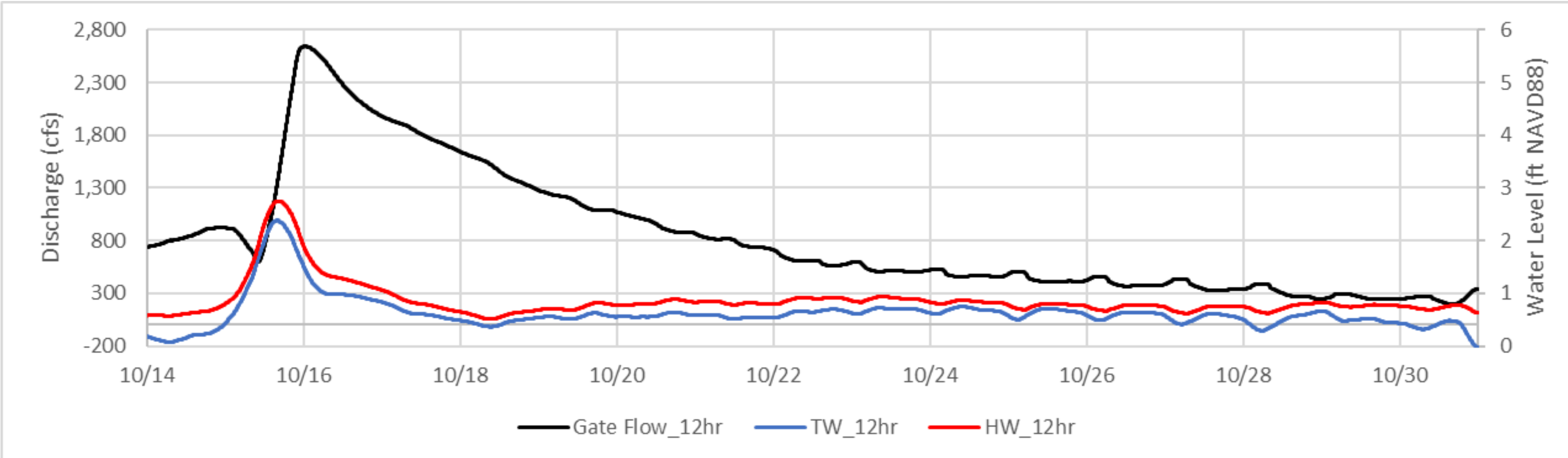


Figure B 1-14. 12-Hour Moving Average Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR1) Event

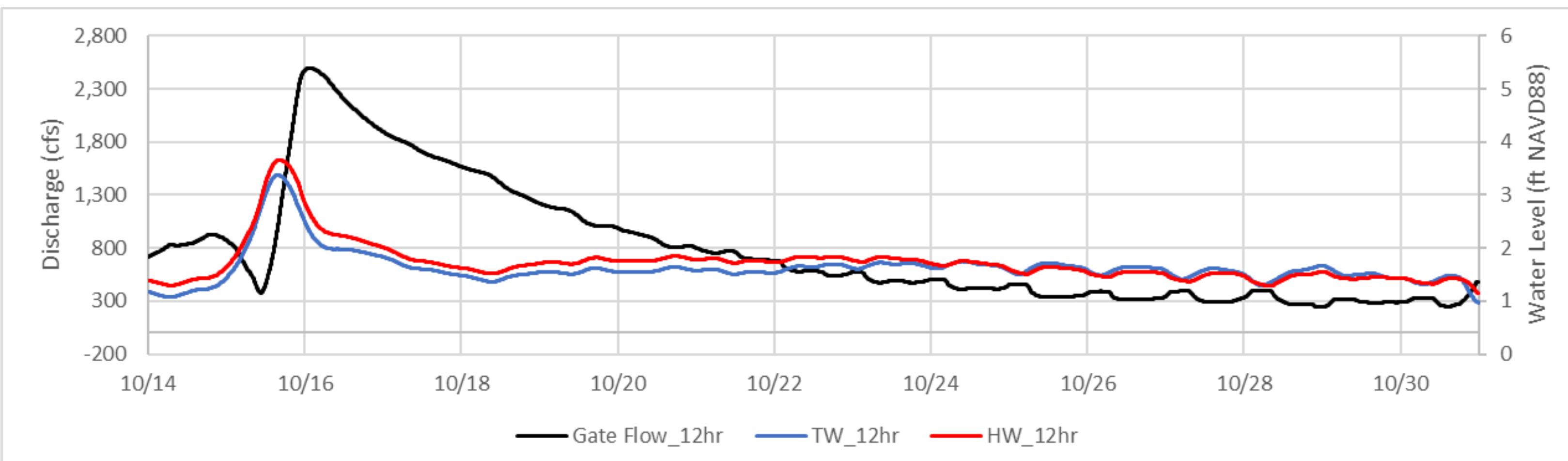


Figure B 1-15. 12-Hour Moving Average Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR2) Event

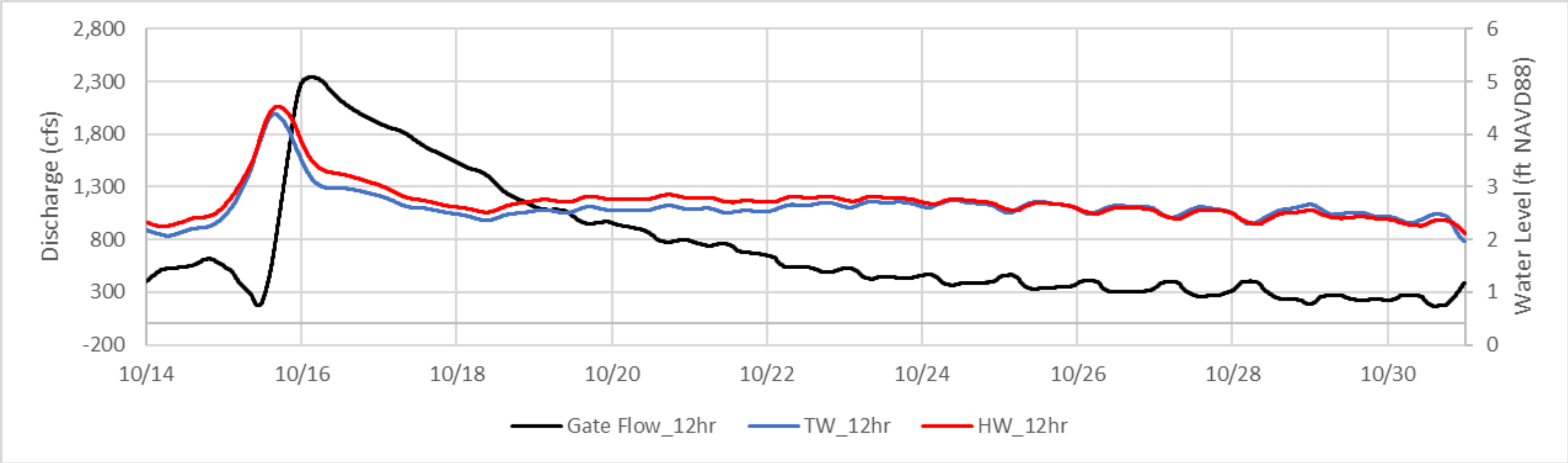


Figure B 1-16. 12-Hour Moving Average Discharge and Stages at S22 for the 25-year 3-day Future Conditions (SLR3) Event

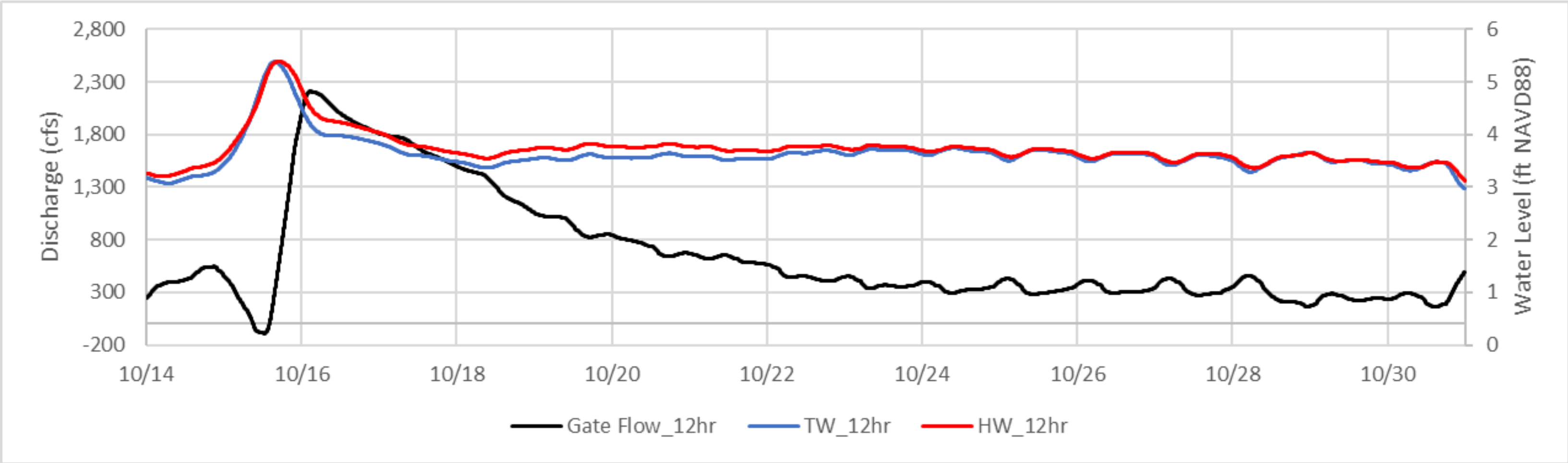


Figure B 1-17. Instantaneous Discharge and Stages at S22 for the 10-year 3-day Current Conditions Event

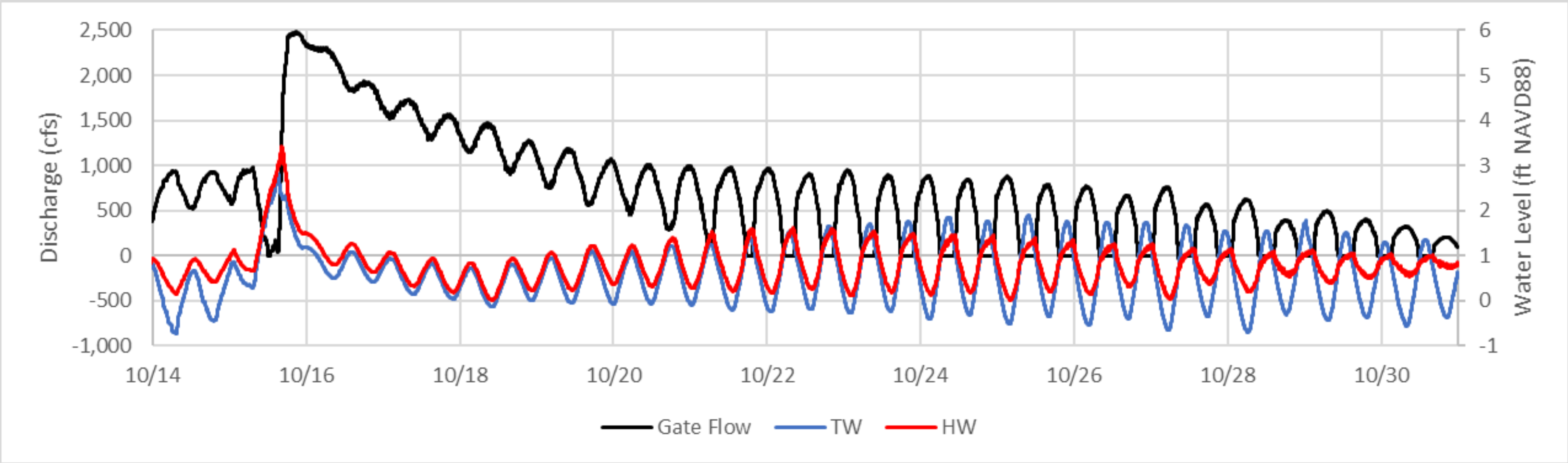


Figure B 1-18. Instantaneous Discharge and Stages at S22 for the 10-year 3-day Future Conditions (SLR1) Event

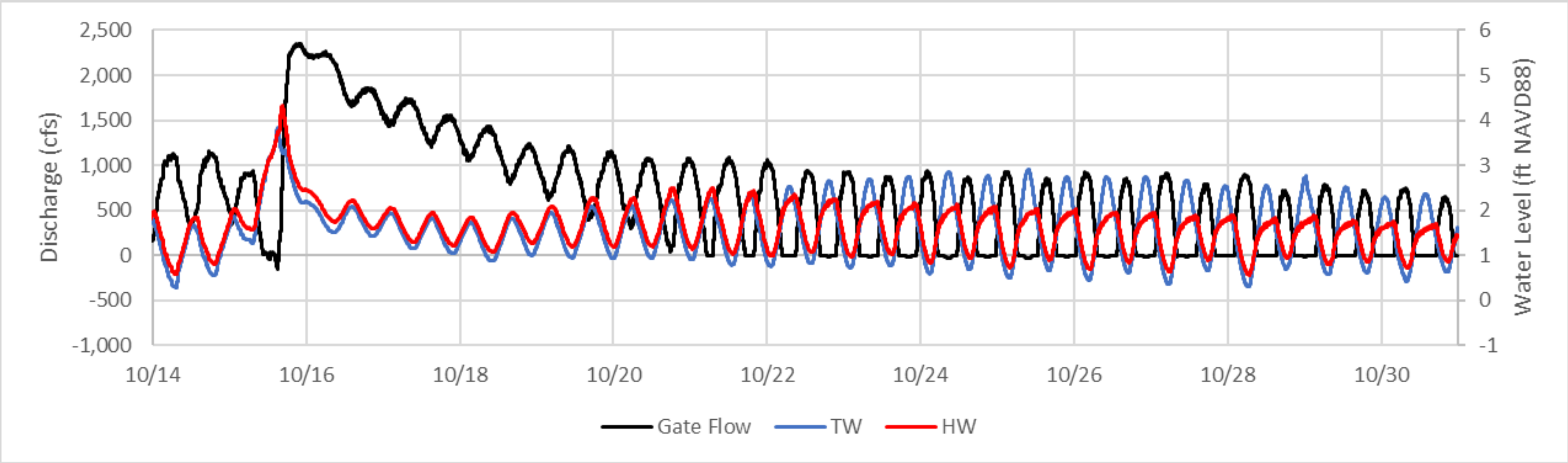


Figure B 1-19. Instantaneous Discharge and Stages at S22 for the 10-year 3-day Future Conditions (SLR2) Event

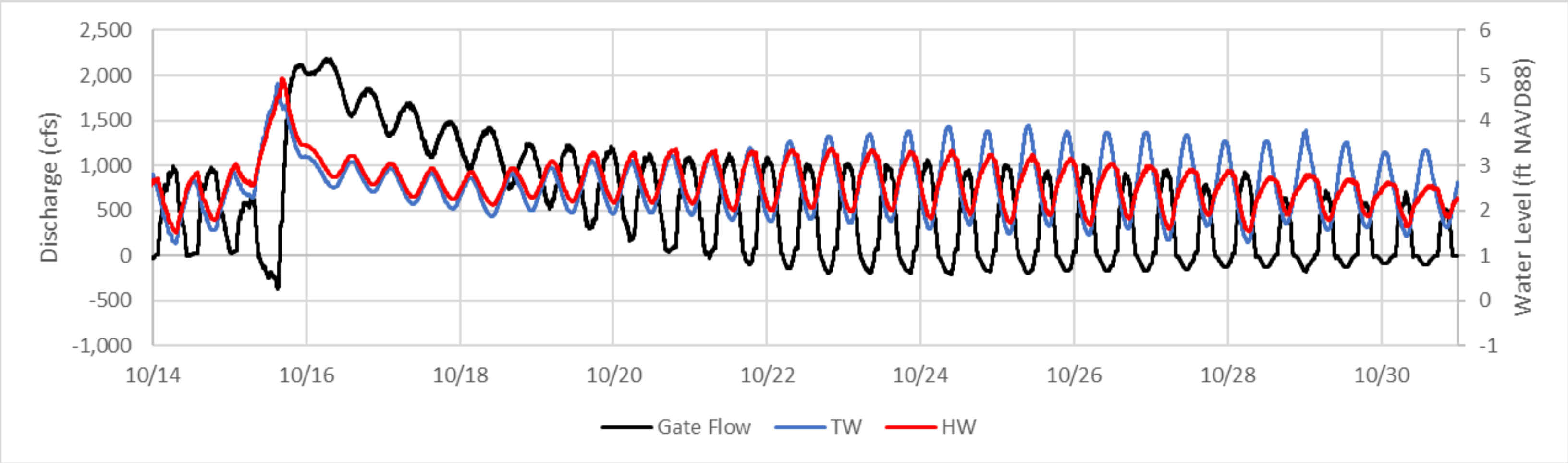


Figure B 1-20. Instantaneous Discharge and Stages at S22 for the 10-year 3-day Future Conditions (SLR3) Event

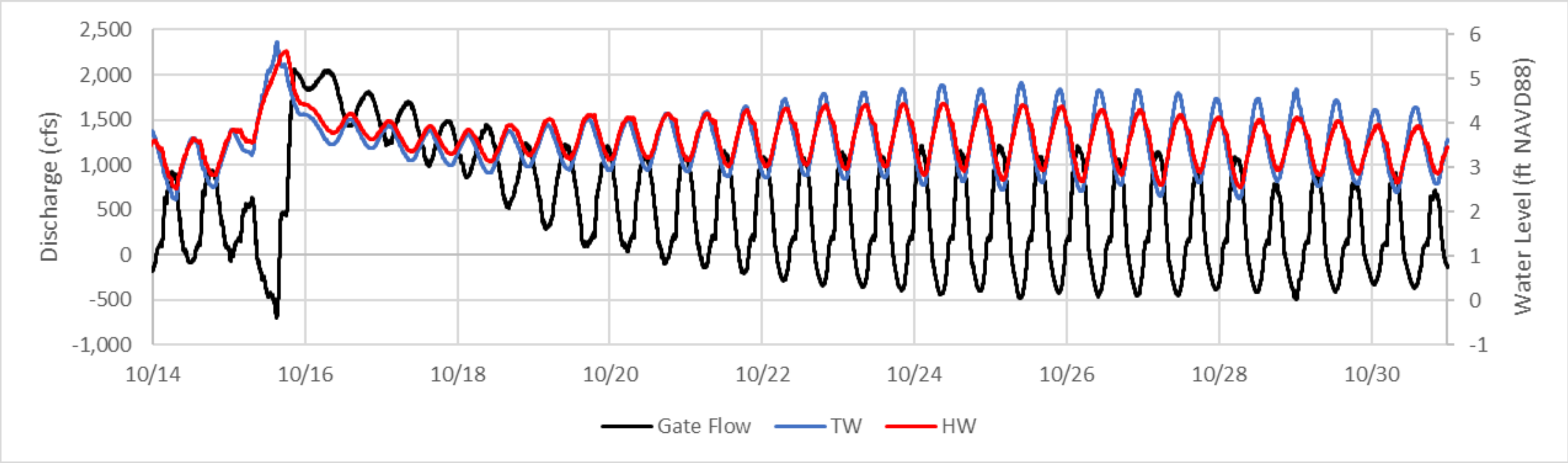


Figure B 1-21. 12-Hour Moving Average Discharge and Stages at S22 for the 10-year 3-day Current Conditions Event

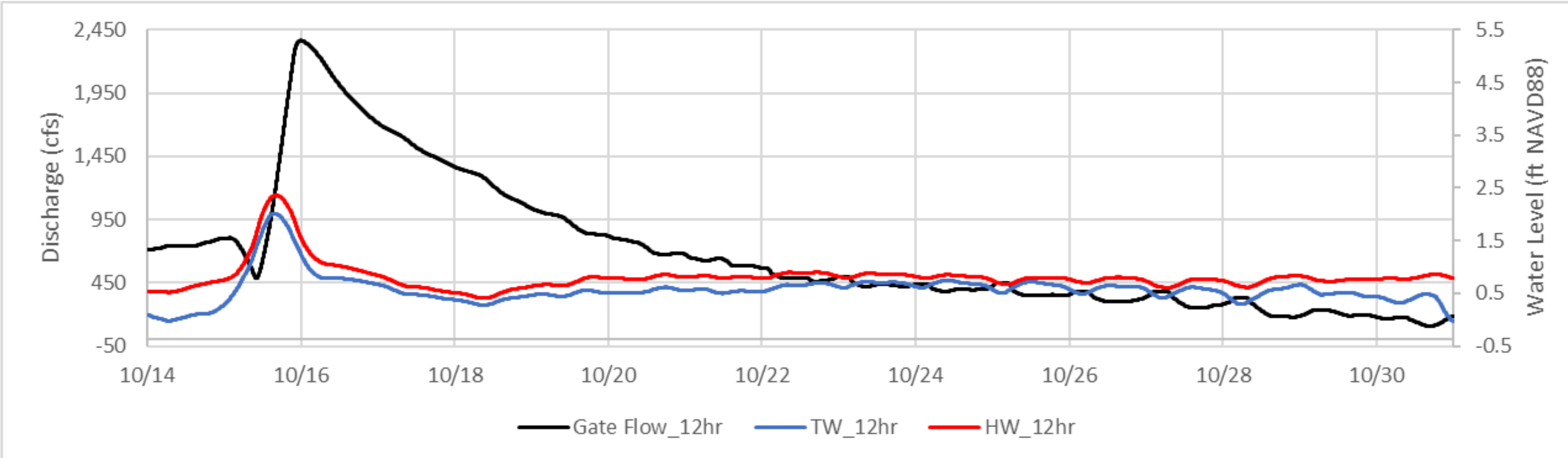


Figure B 1-22. 12-Hour Moving Average Discharge and Stages at S22 for the 10-year 3-day Future Conditions (SLR1) Event

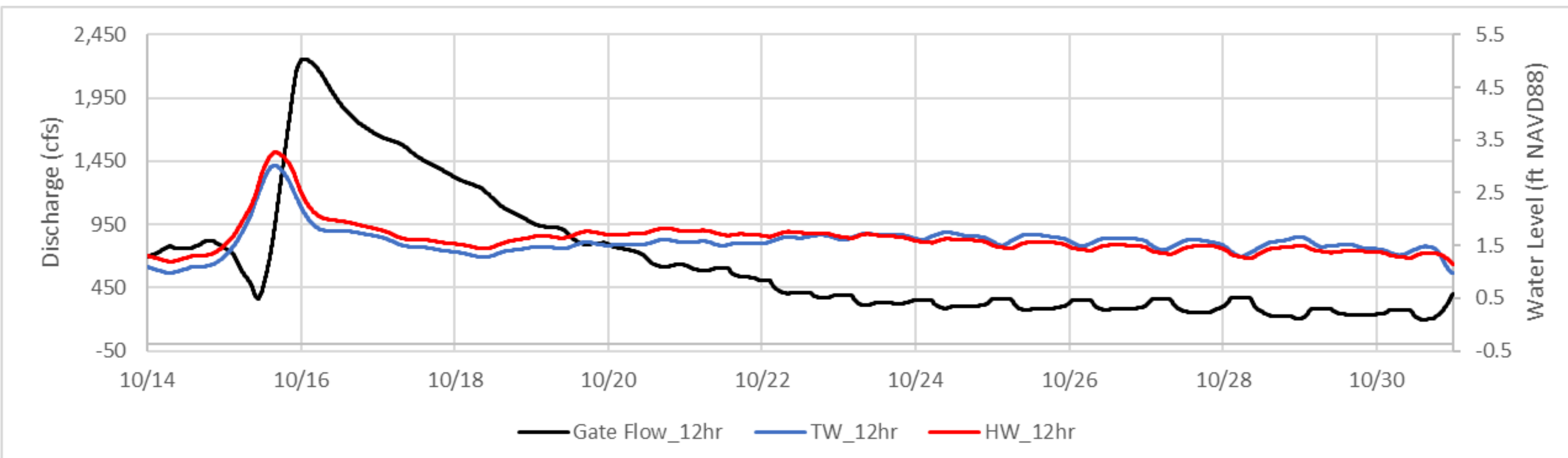


Figure B 1-23. 12-Hour Moving Average Discharge and Stages at S22 for the 10-year 3-day Future Conditions (SLR2) Event

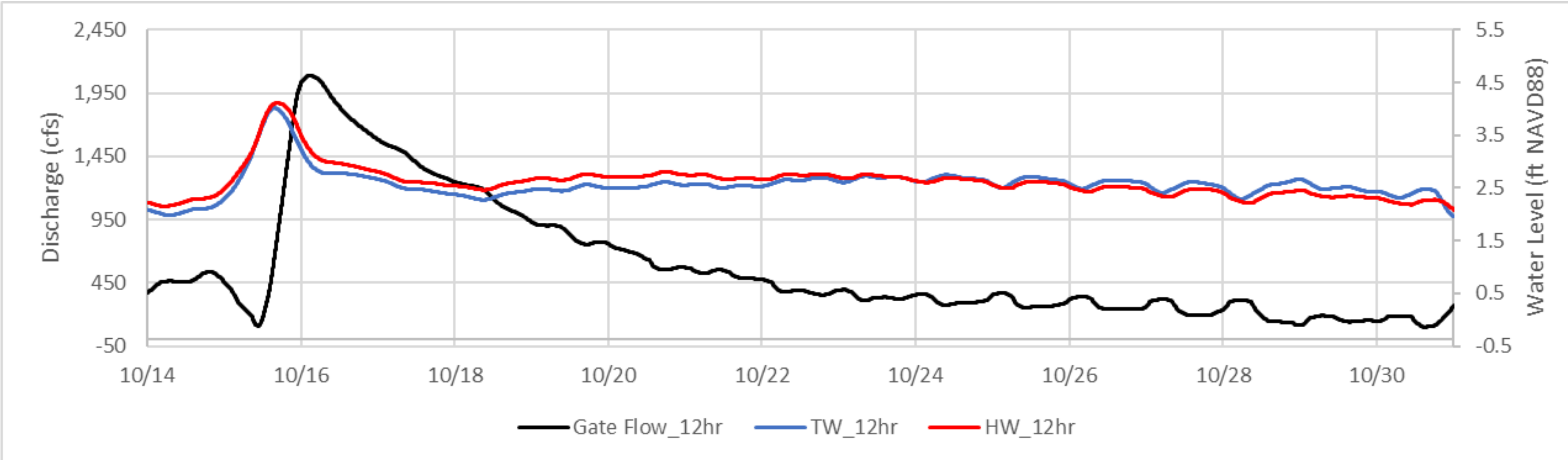


Figure B 1-24. 12-Hour Moving Average Discharge and Stages at S22 for the 10-year 3-day Future Conditions (SLR3) Event

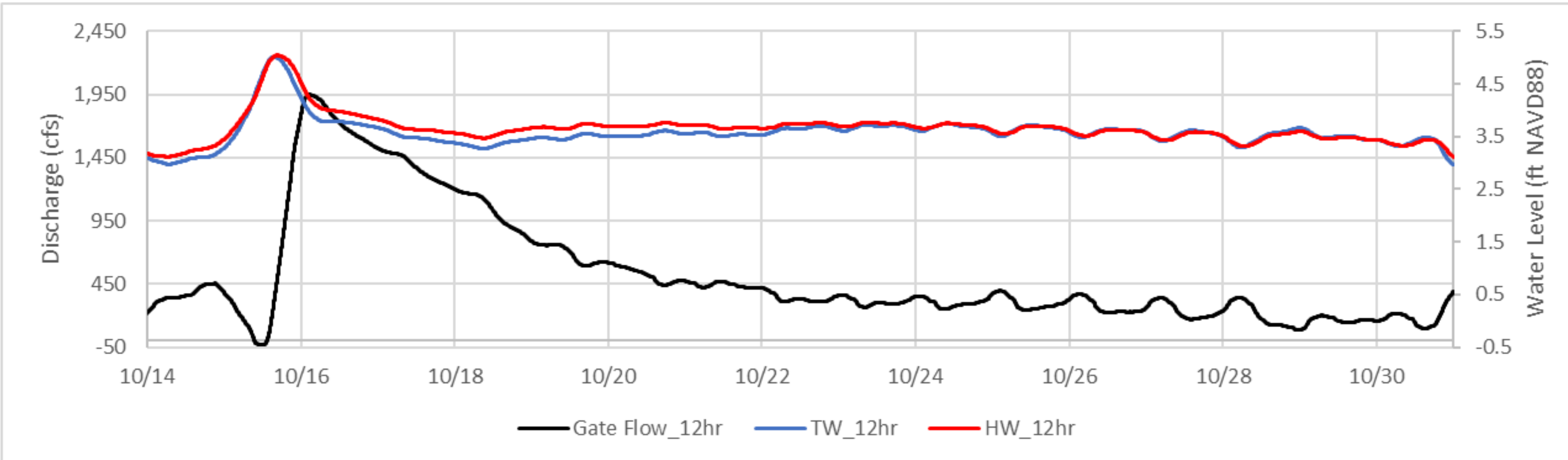


Figure B 1-25. Instantaneous Discharge and Stages at S22 for the 5-year 3-day Current Conditions Event

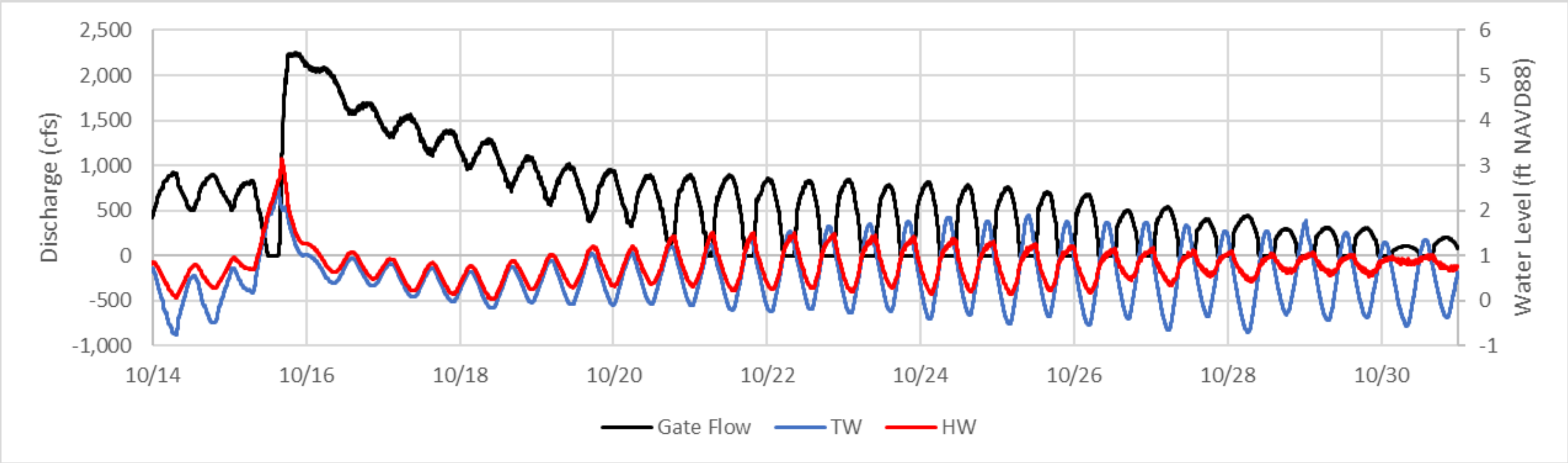


Figure B 1-26. Instantaneous Discharge and Stages at S22 for the 5-year 3-day Future Conditions (SLR1) Event

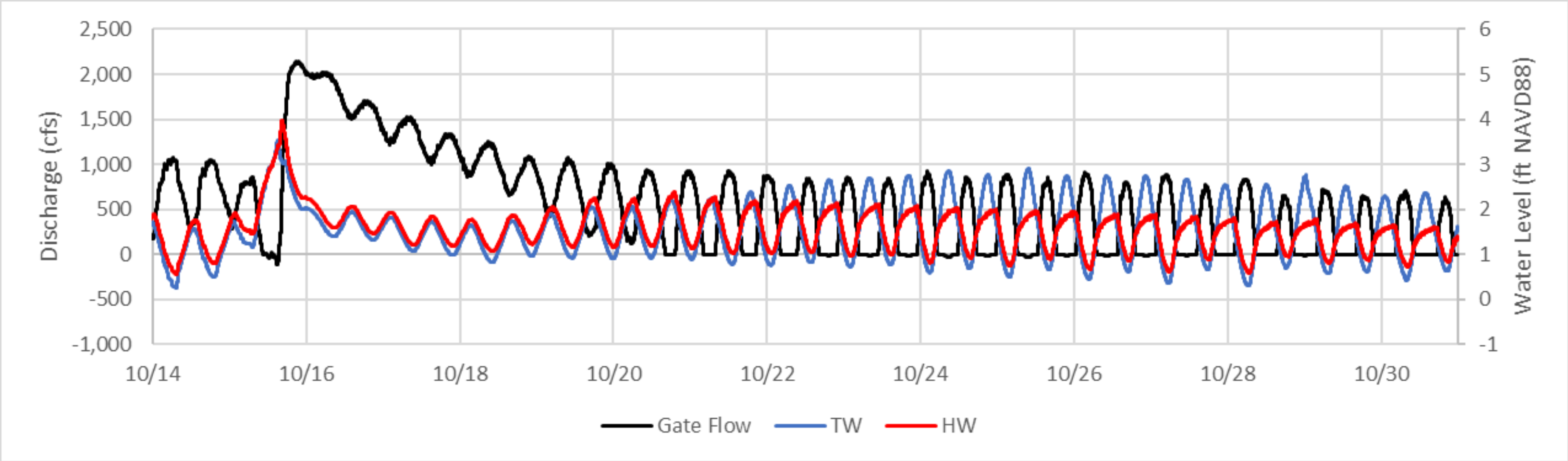


Figure B 1-27. Instantaneous Discharge and Stages at S22 for the 5-year 3-day Future Conditions (SLR2) Event

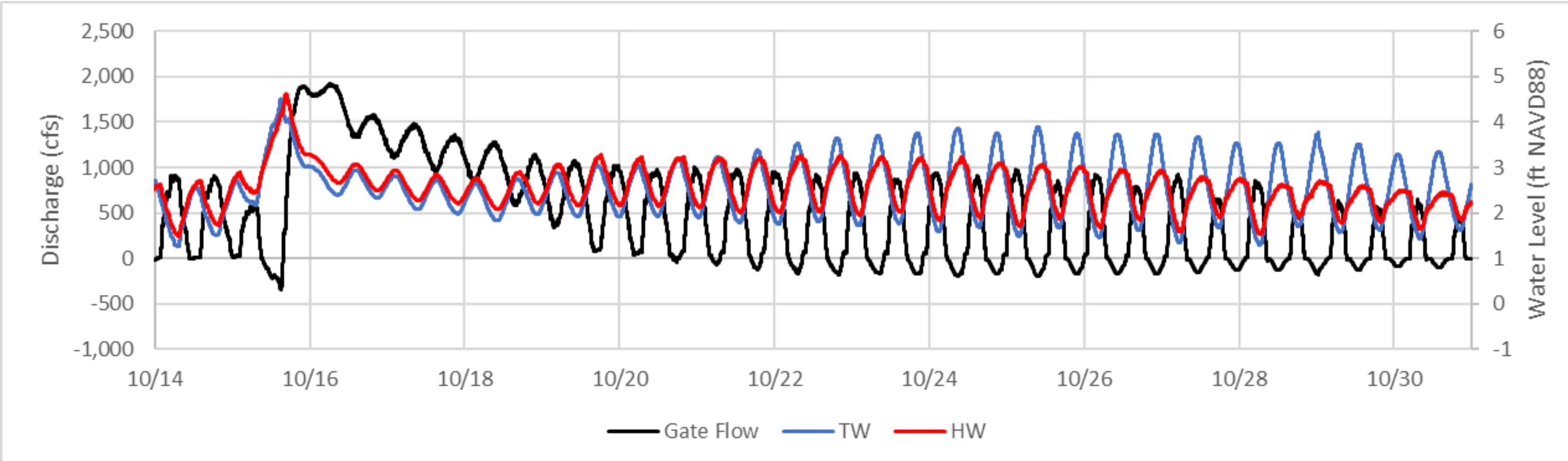


Figure B 1-28. Instantaneous Discharge and Stages at S22 for the 5-year 3-day Future Conditions (SLR3) Event

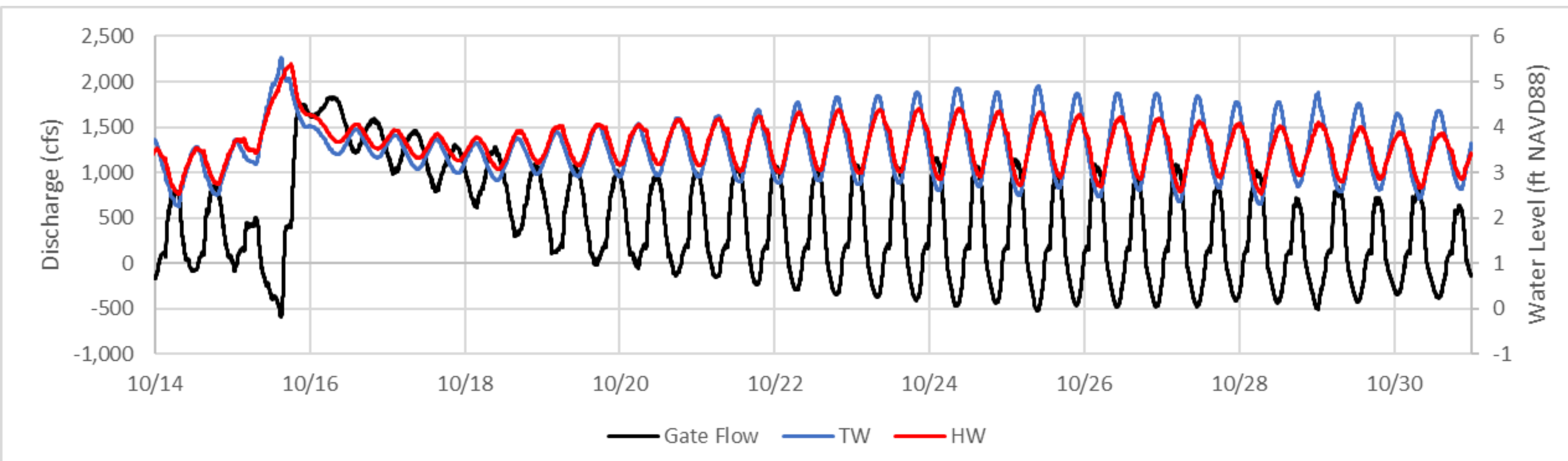


Figure B 1-29. 12-Hour Moving Average Discharge and Stages at S22 for the 5-year 3-day Current Conditions Event

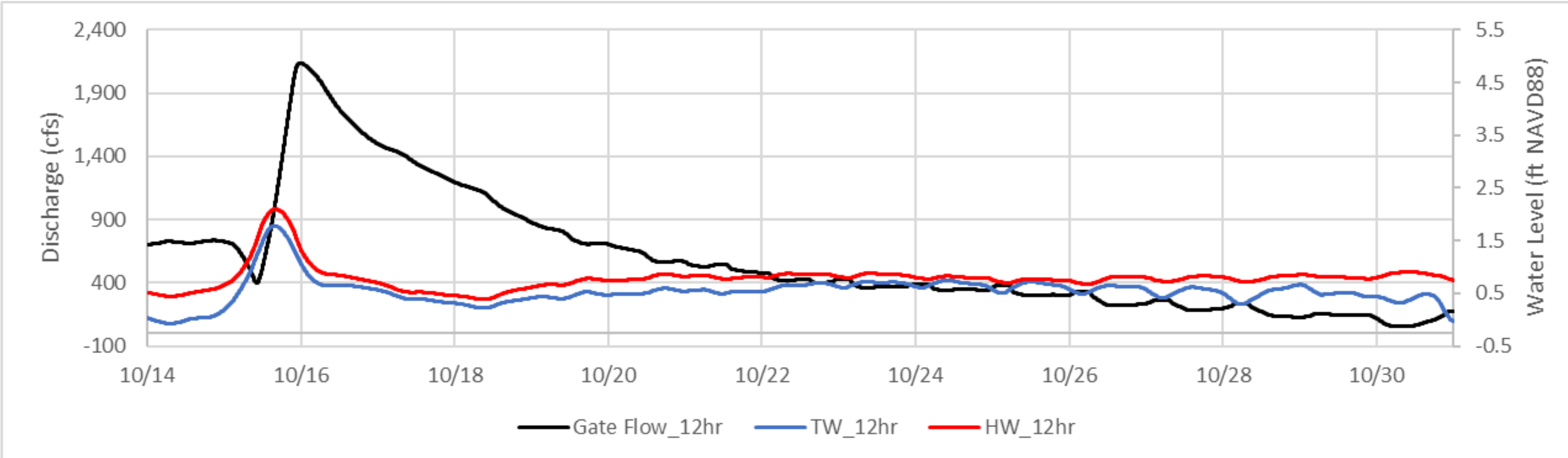


Figure B 1-30. 12-Hour Moving Average Discharge and Stages at S22 for the 5-year 3-day Future Conditions (SLR1) Event

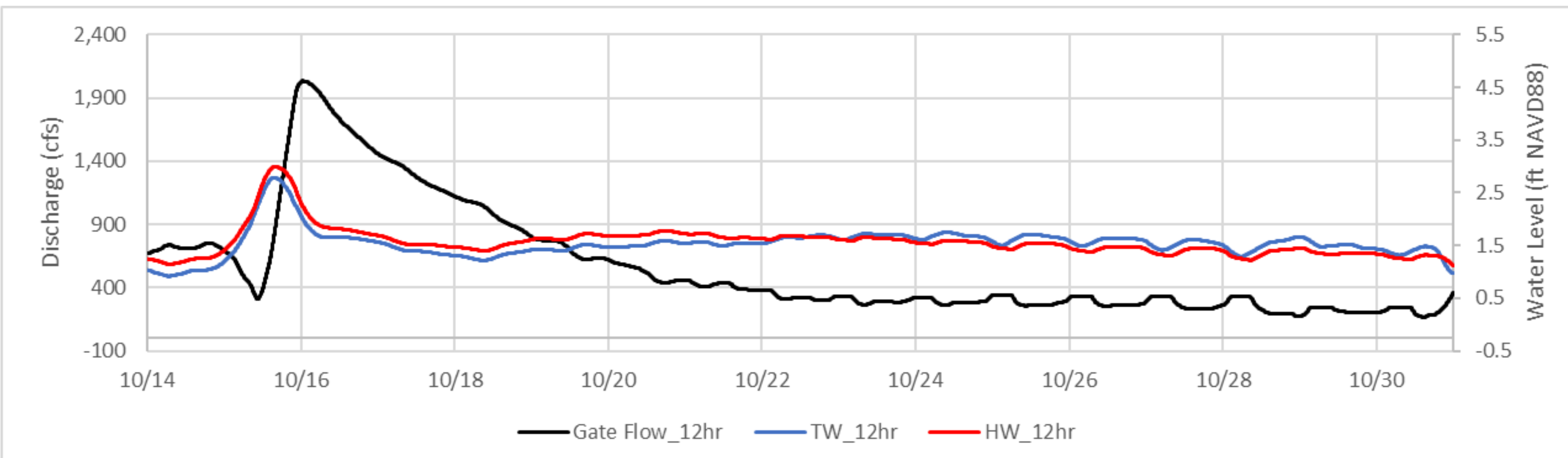


Figure B 1-31. 12-Hour Moving Average Discharge and Stages at S22 for the 5-year 3-day Future Conditions (SLR2) Event

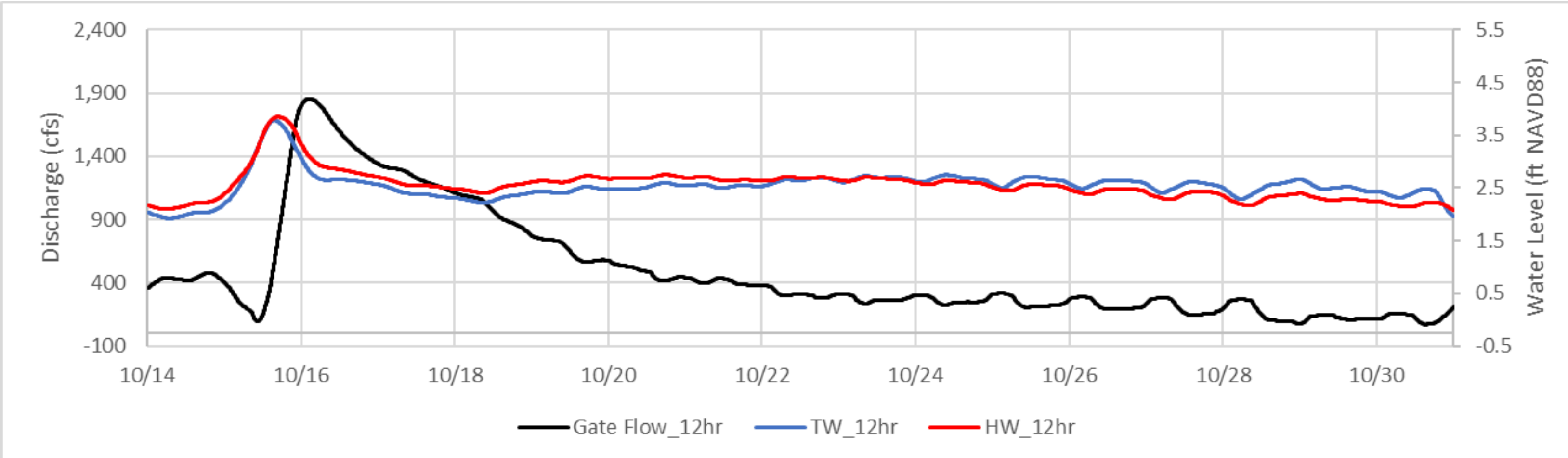
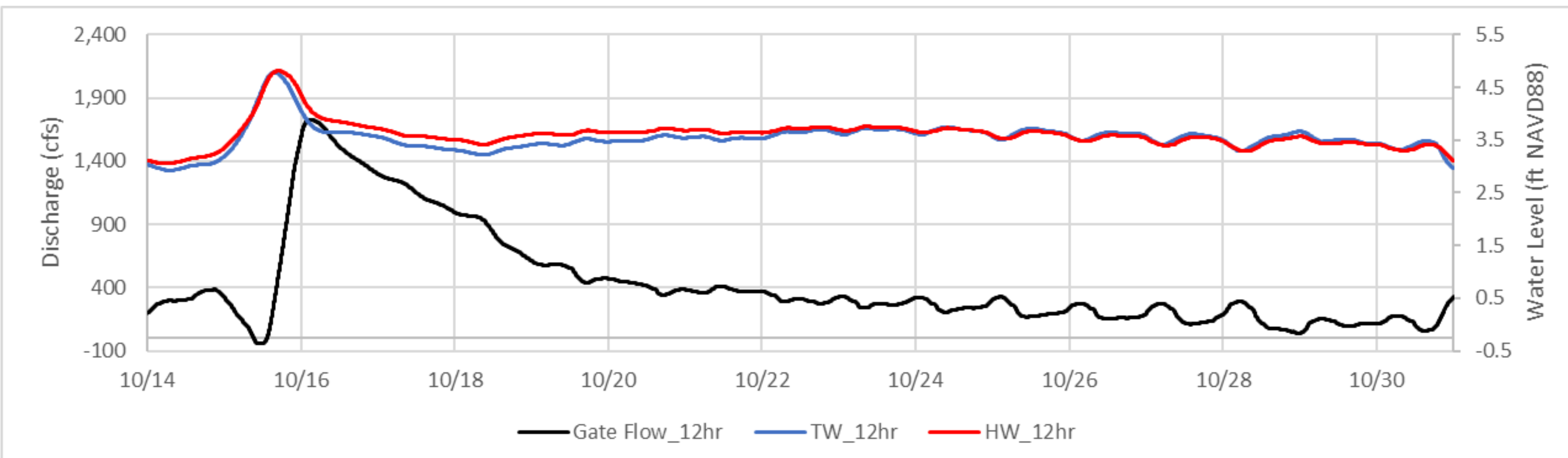


Figure B 1-32. 12-Hour Moving Average Discharge and Stages at S22 for the 5-year 3-day Future Conditions (SLR3) Event



2 C3W Watershed

Figure B 2-1 through **Figure B 2-32** show the instantaneous and 12-hour average discharge, HW, and TW at G93 for each SLR condition for the 100-, 25-, 10-, and 5-year design storm events.

Figure B 2-1. Instantaneous Discharge and Stages at G93 for the 100-year 3-day Current Conditions Event

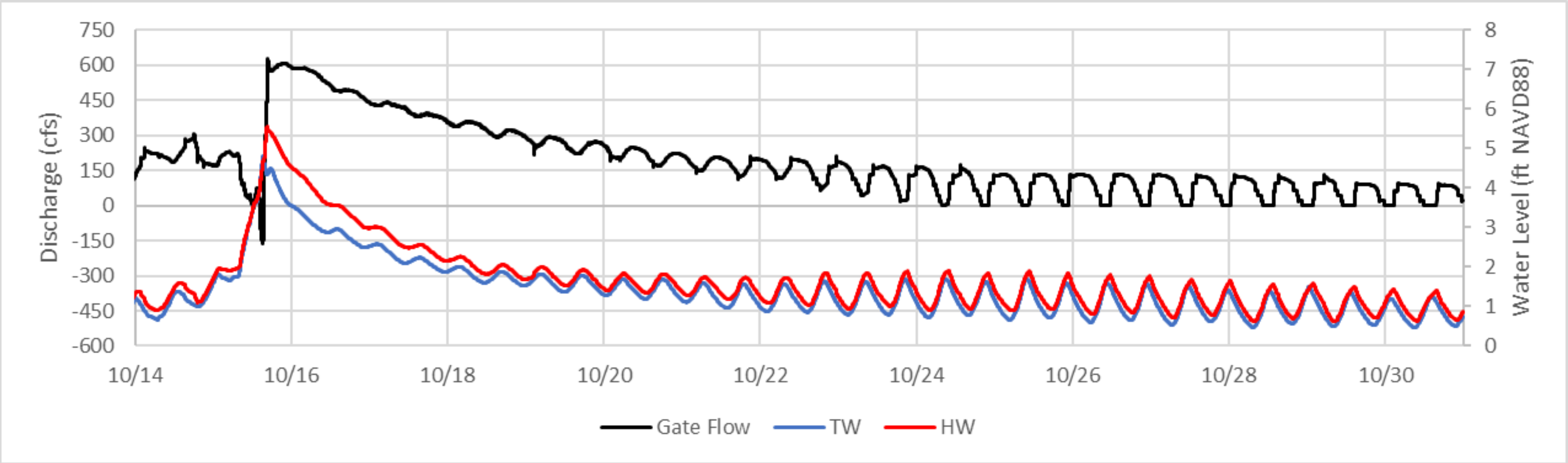


Figure B 2-2. Instantaneous Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR1) Event

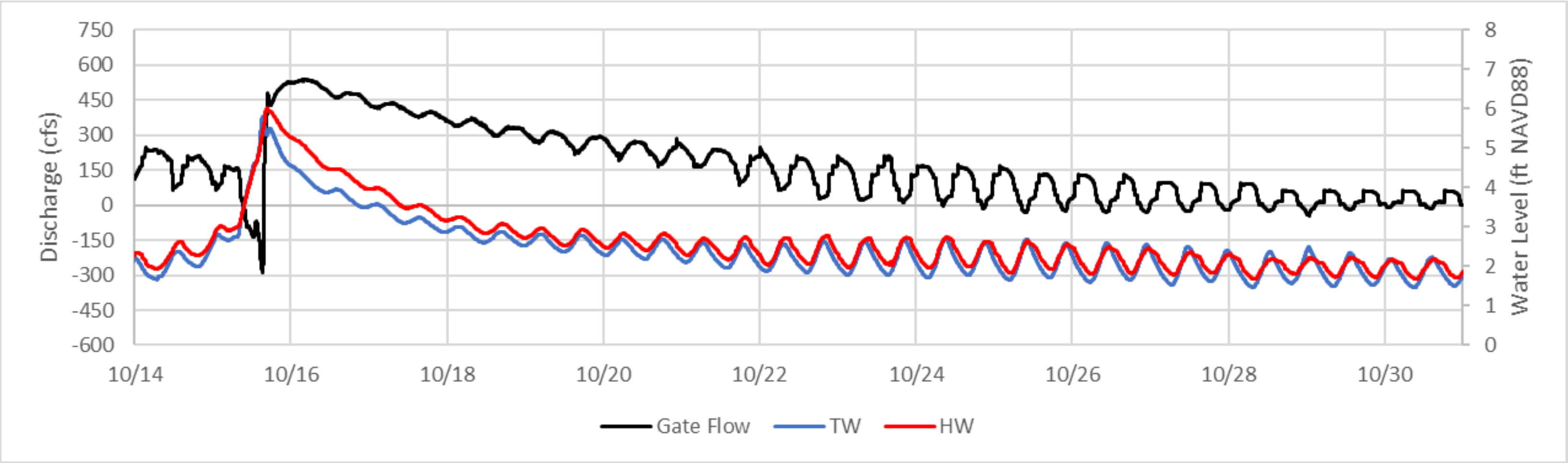


Figure B 2-3. Instantaneous Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR2) Event

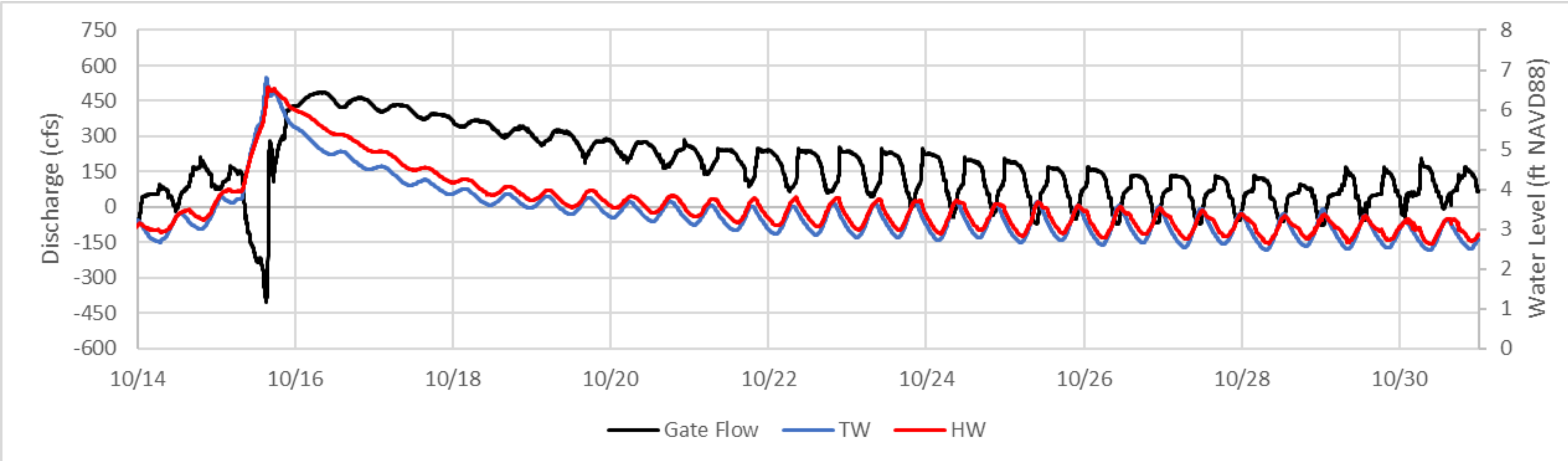


Figure B 2-4. Instantaneous Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR3) Event

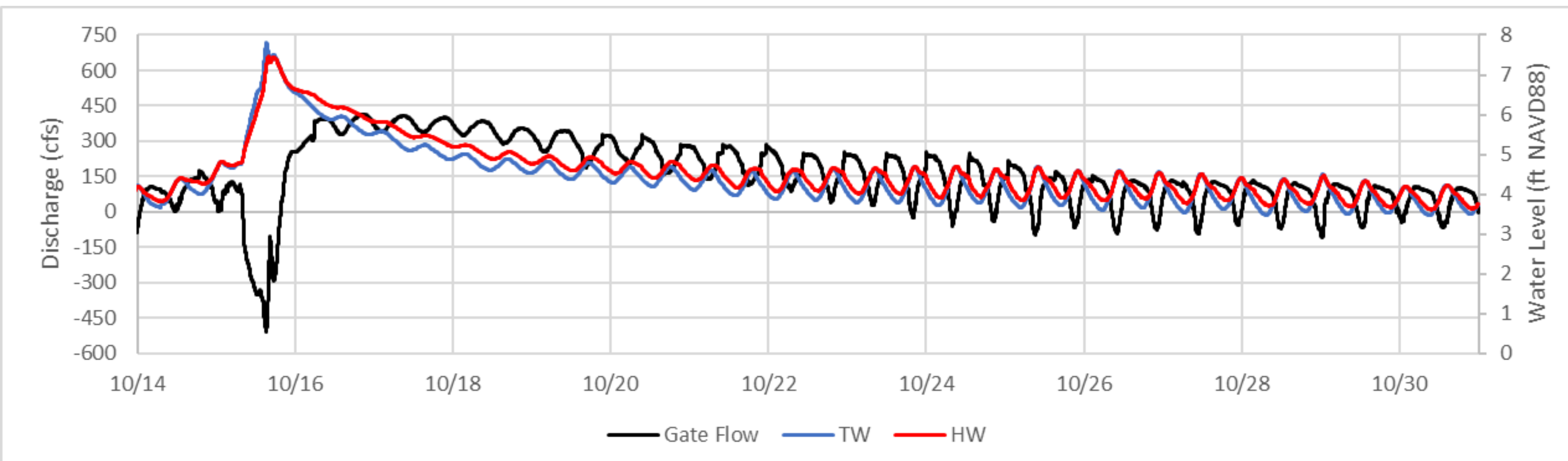


Figure B 2-5. 12-Hour Moving Average Discharge and Stages at G93 for the 100-year 3-day Current Conditions Event

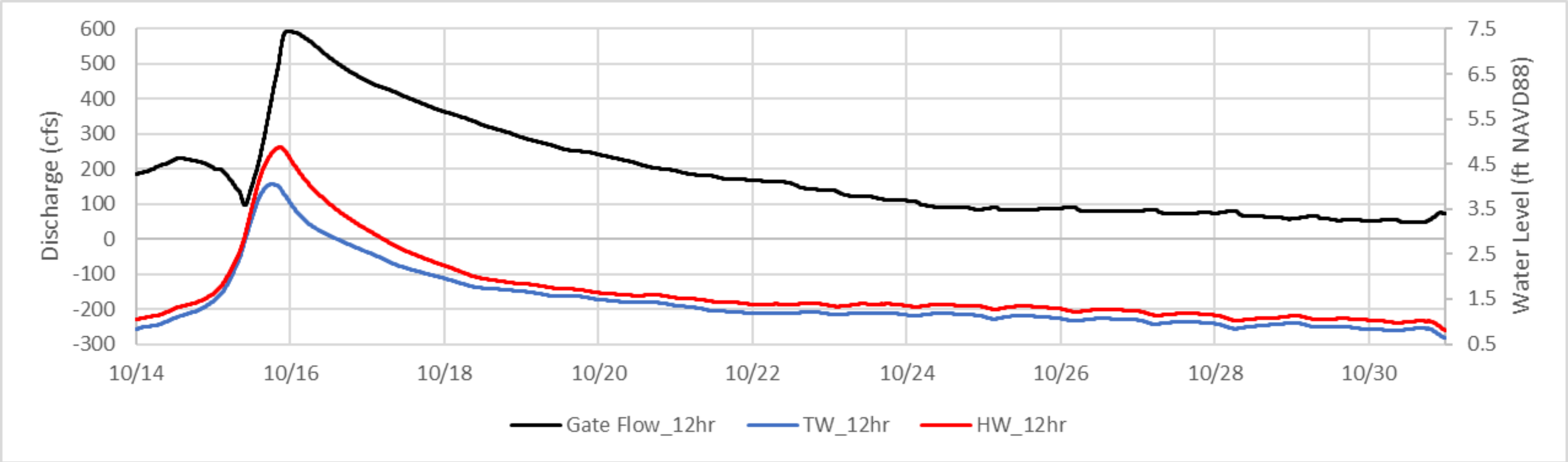


Figure B 2-6. 12-Hour Moving Average Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR1) Event

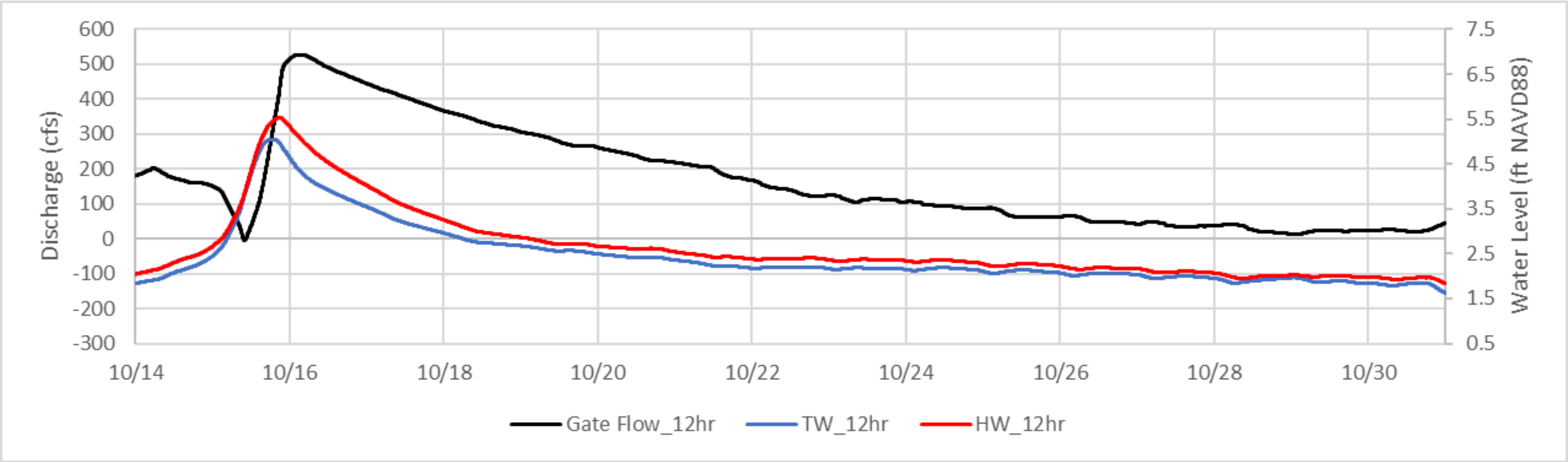


Figure B 2-7. 12-Hour Moving Average Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR2) Event

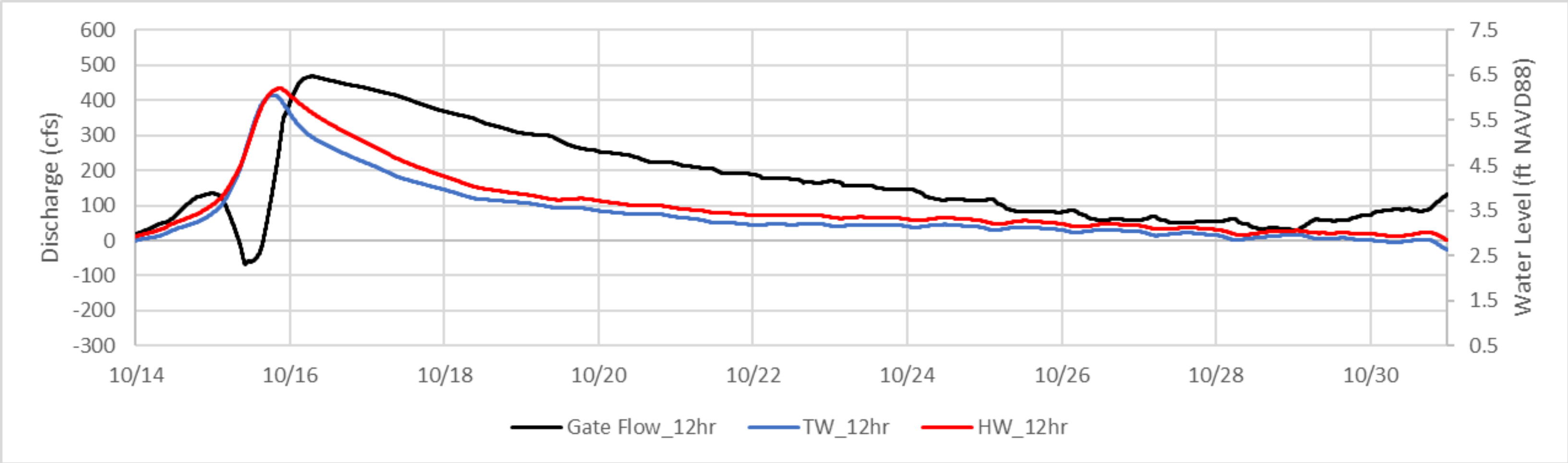


Figure B 2-8. 12-Hour Moving Average Discharge and Stages at G93 for the 100-year 3-day Future Conditions (SLR3) Event

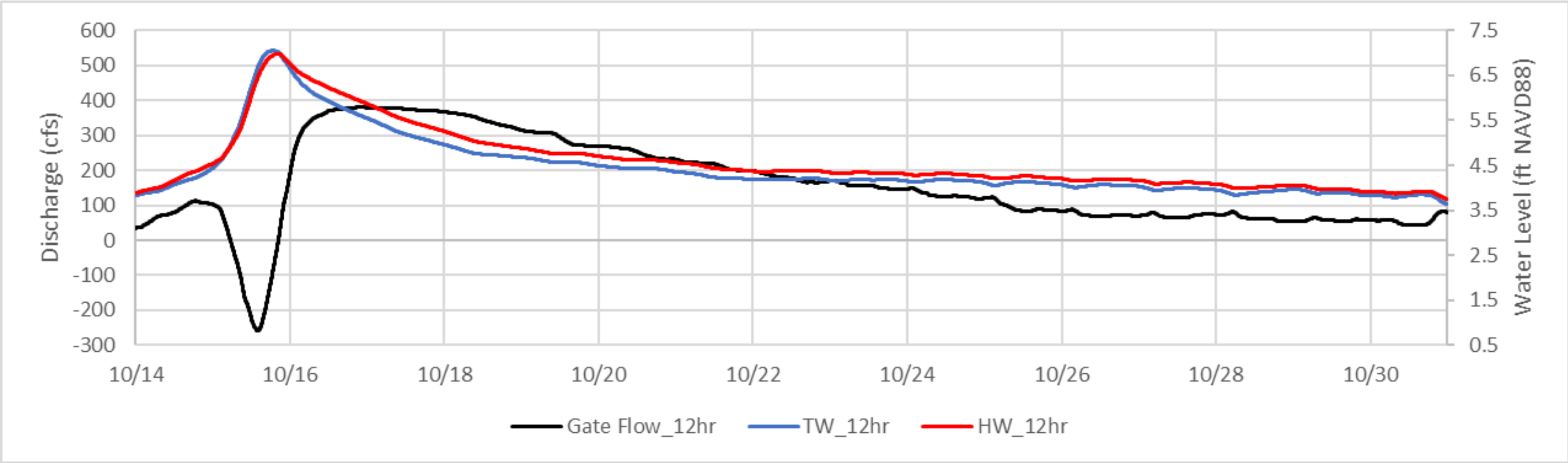


Figure B 2-9. Instantaneous Discharge and Stages at G93 for the 25-year 3-day Current Conditions Event

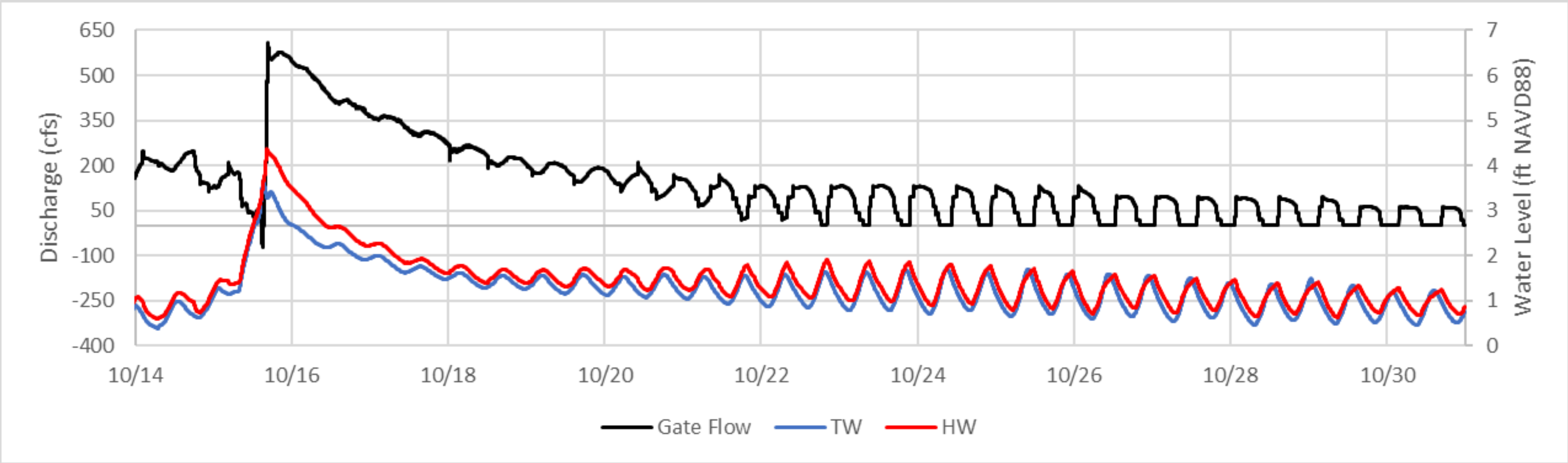


Figure B 2-10. Instantaneous Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR1) Event

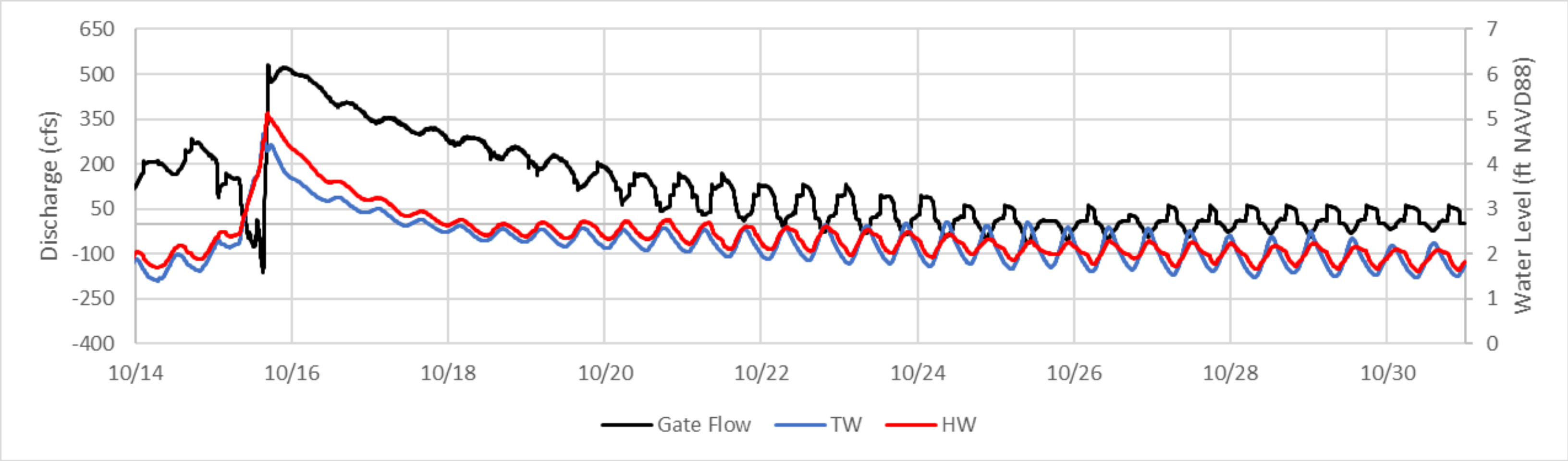


Figure B 2-11. Instantaneous Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR2) Event

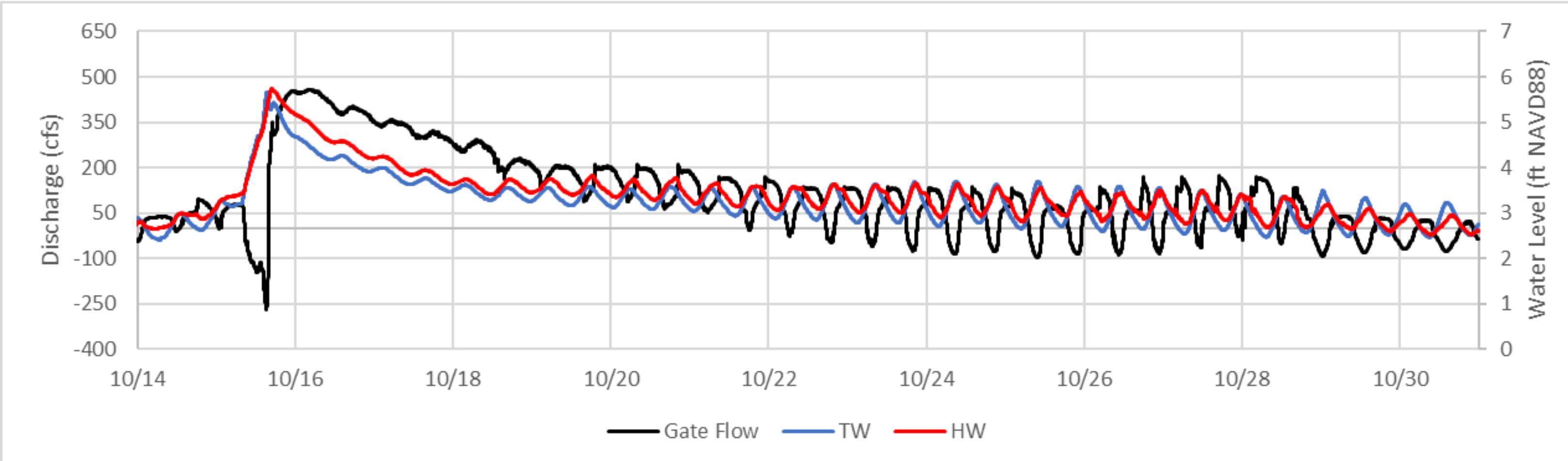


Figure B 2-12. Instantaneous Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR3) Event

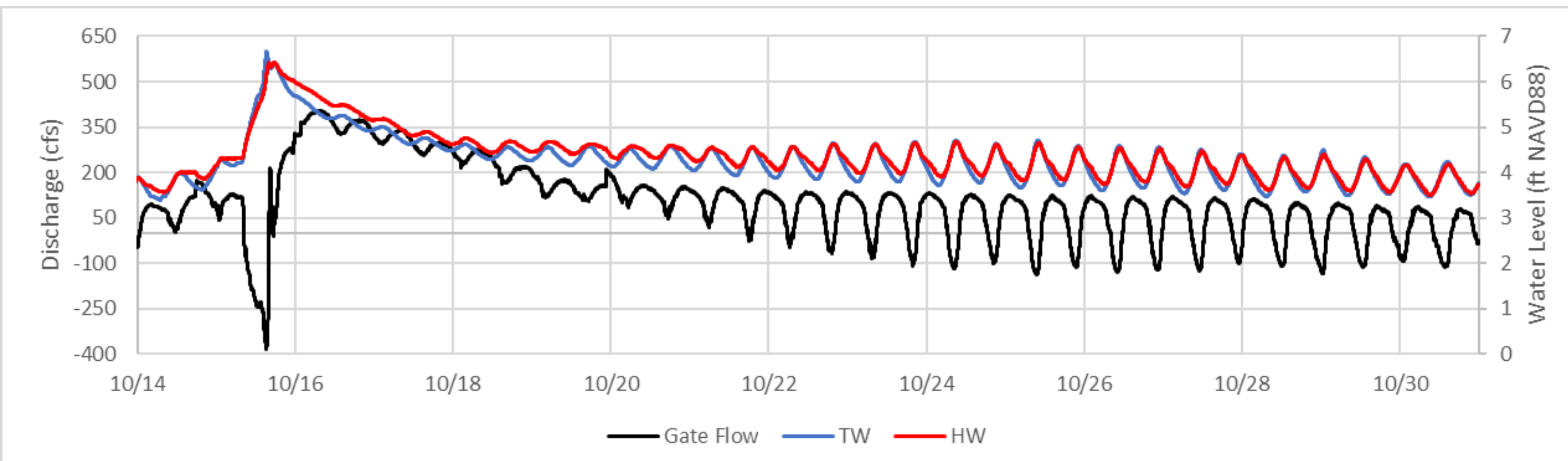


Figure B 2-13. 12-Hour Moving Average Discharge and Stages at G93 for the 25-year 3-day Current Conditions Event

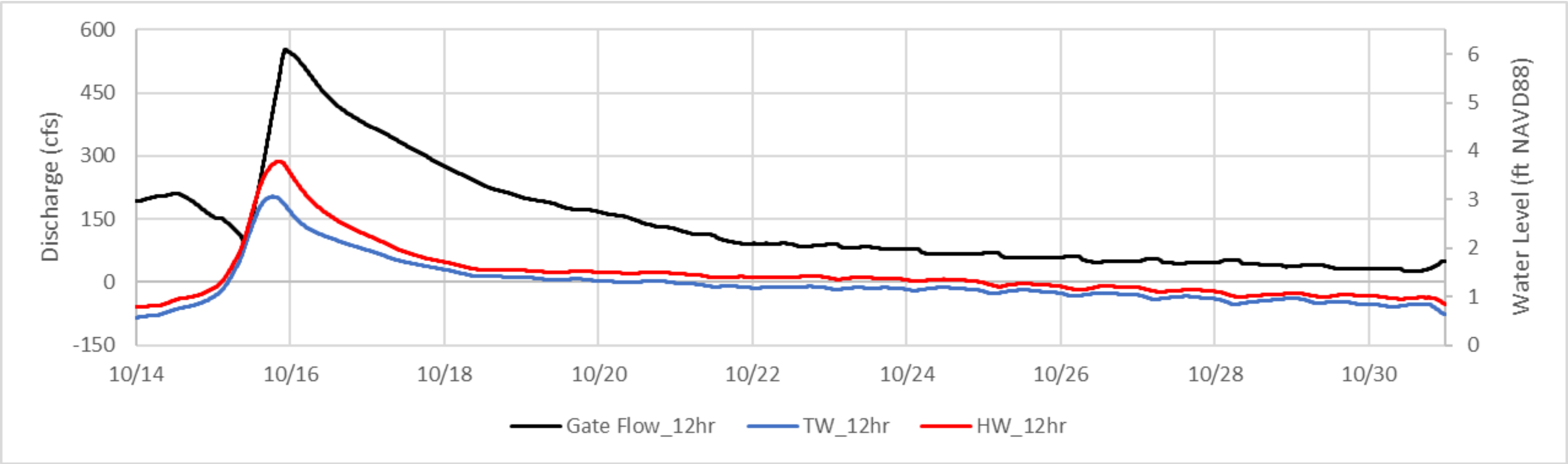


Figure B 2-14. 12-Hour Moving Average Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR1) Event

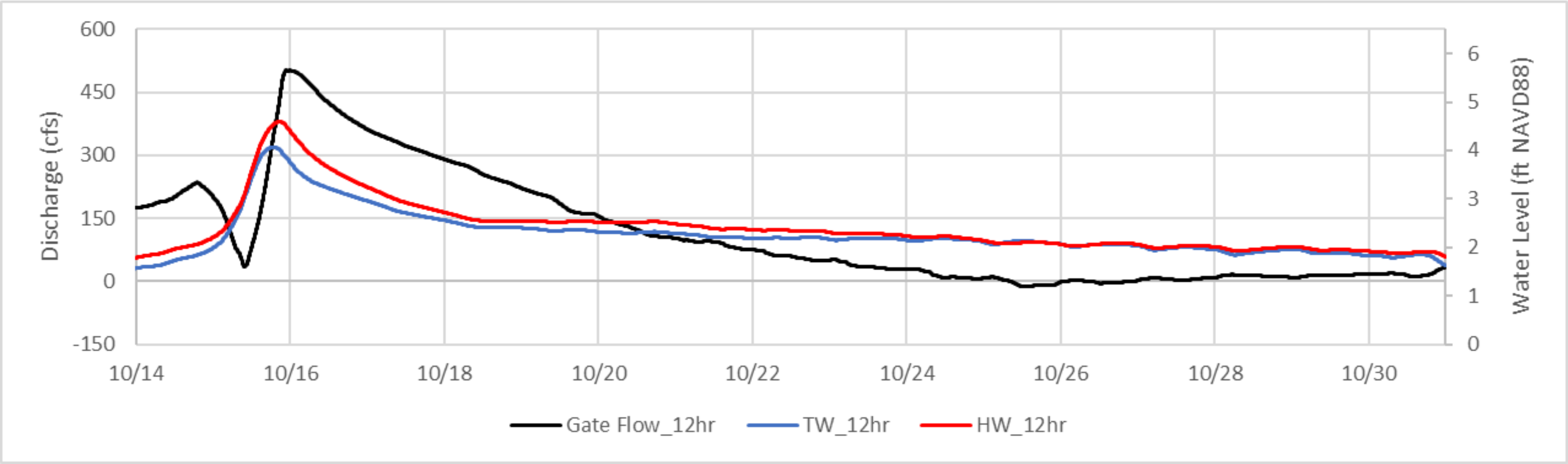


Figure B 2-15. 12-Hour Moving Average Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR2) Event

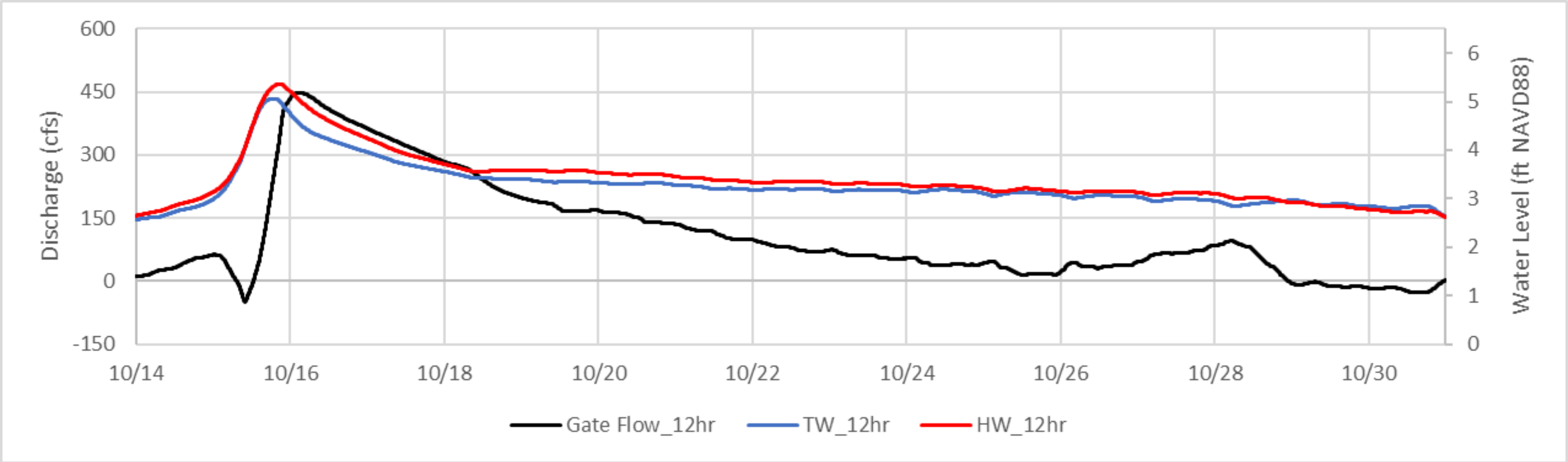


Figure B 2-16. 12-Hour Moving Average Discharge and Stages at G93 for the 25-year 3-day Future Conditions (SLR3) Event

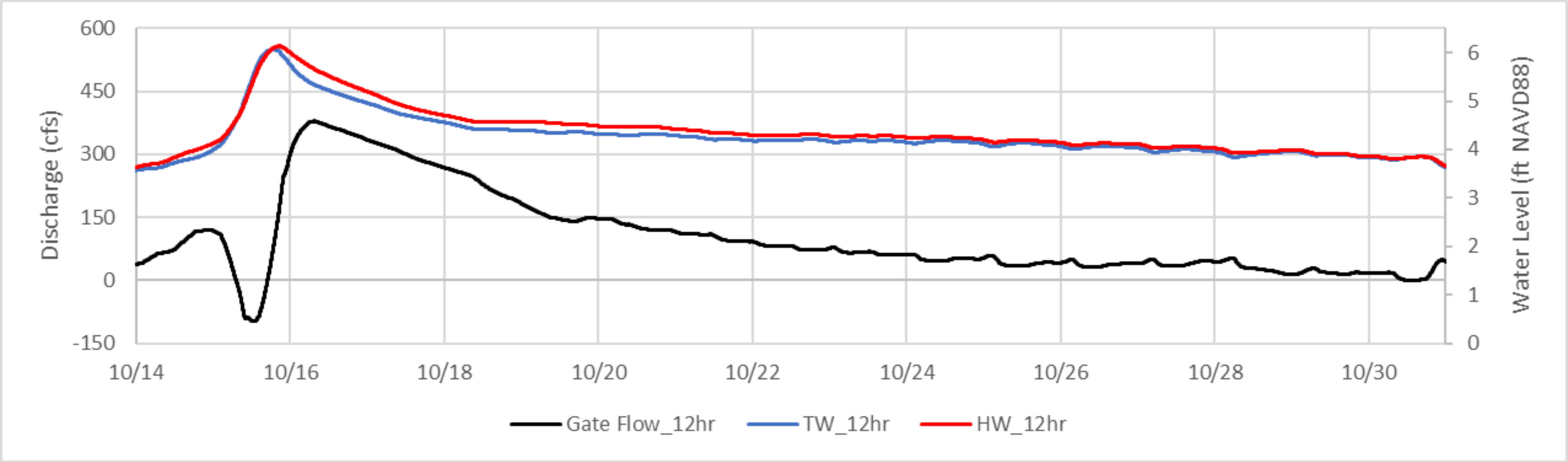


Figure B 2-17. Instantaneous Discharge and Stages at G93 for the 10-year 3-day Current Conditions Event

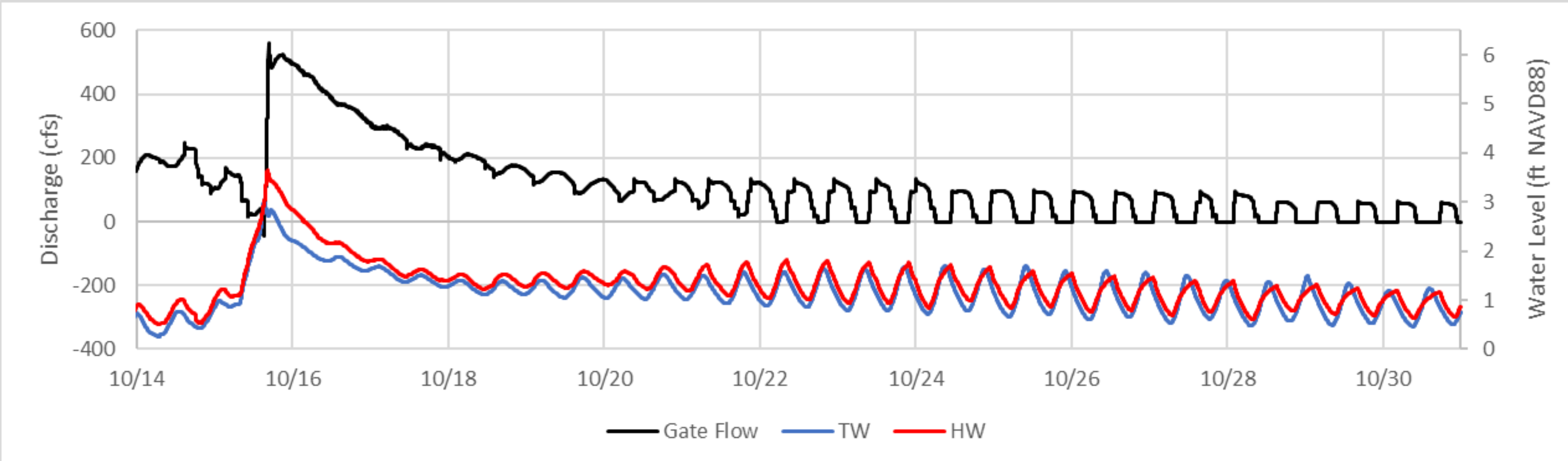


Figure B 2-18. Instantaneous Discharge and Stages at G93 for the 10-year 3-day Future Conditions (SLR1) Event

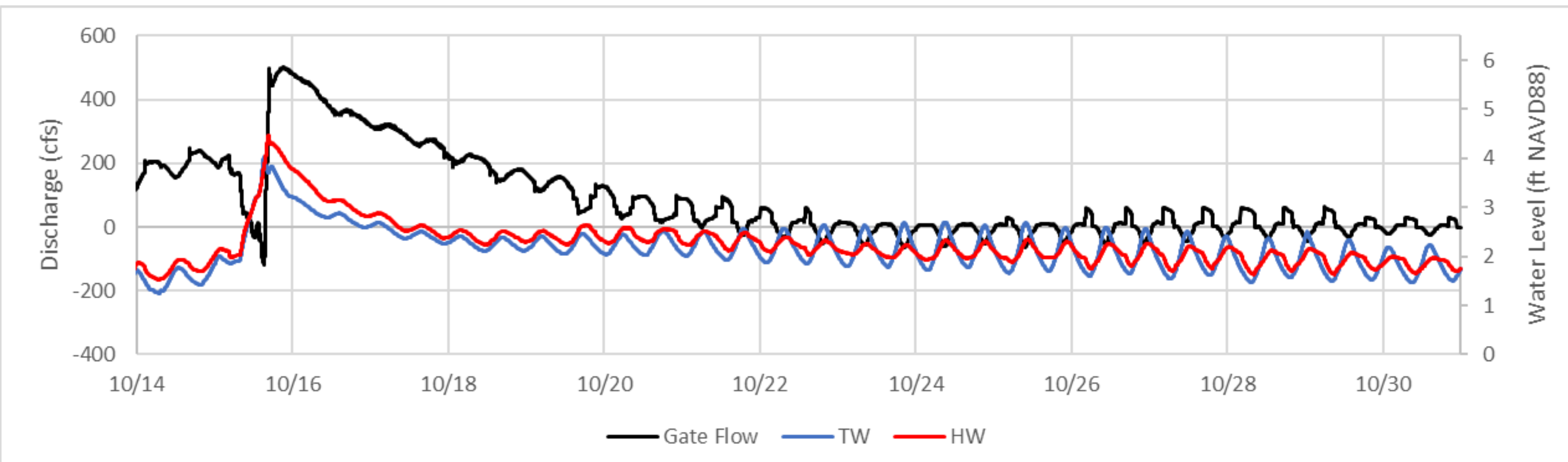


Figure B 2-19. Instantaneous Discharge and Stages at G93 for the 10-year 3-day Future Conditions (SLR2) Event

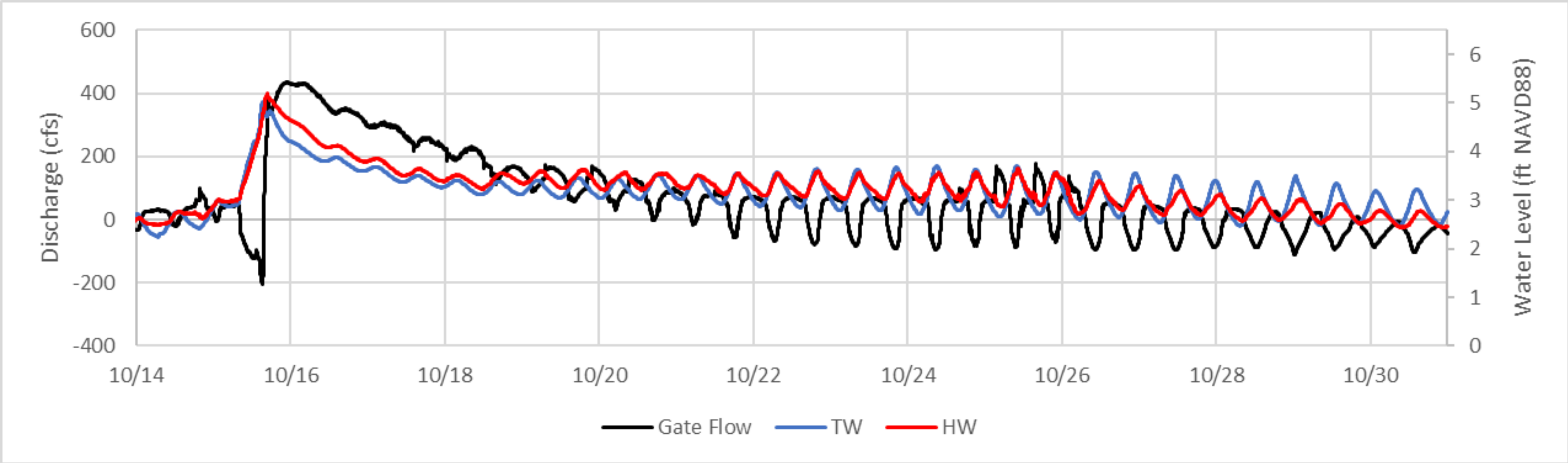


Figure B 2-20. Instantaneous Discharge and Stages at G93 for the 10-year 3-day Future Conditions (SLR3) Event

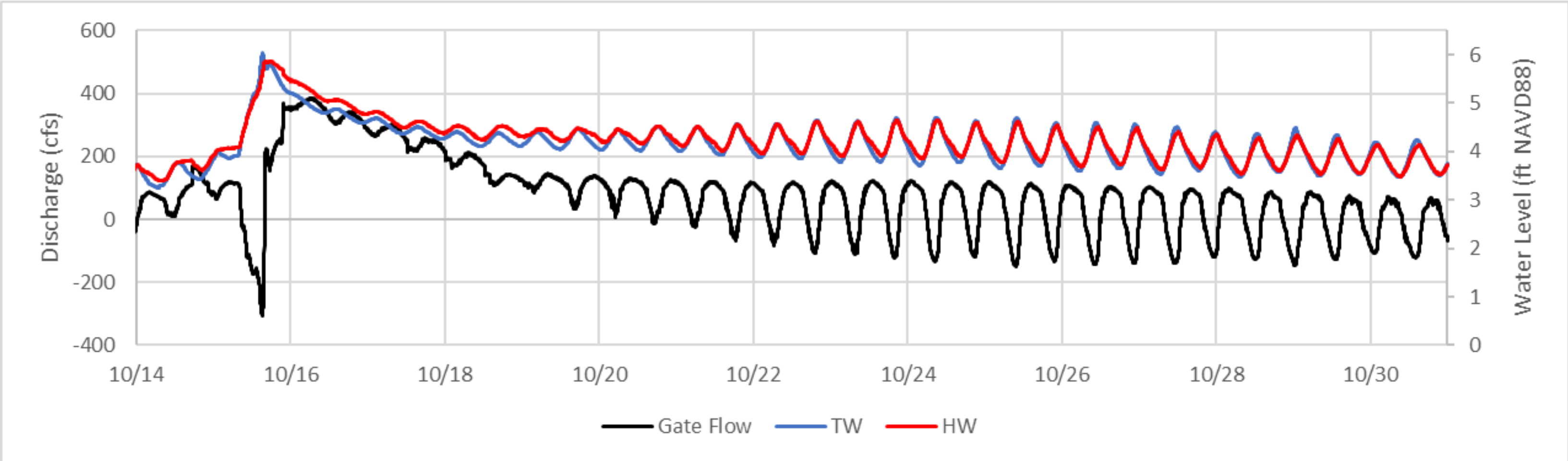


Figure B 2-21. 12-Hour Moving Average Discharge and Stages at G93 for the 10-year 3-day Current Conditions Event

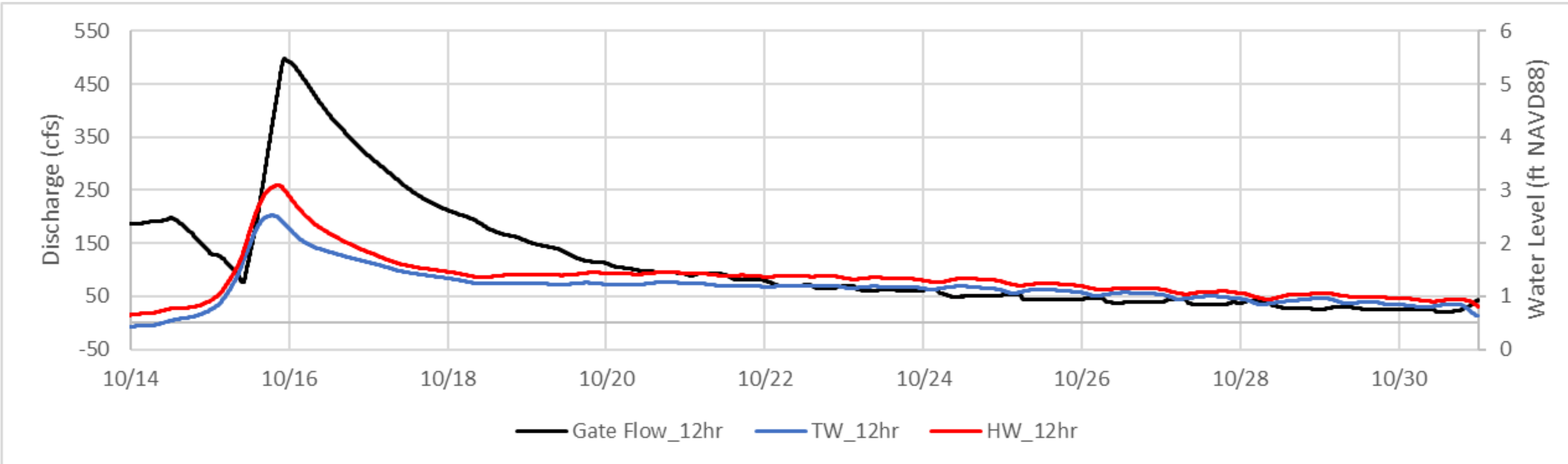


Figure B 2-22. 12-Hour Moving Average Discharge and Stages at G93 for the 10-year 3-day Future Conditions (SLR1) Event

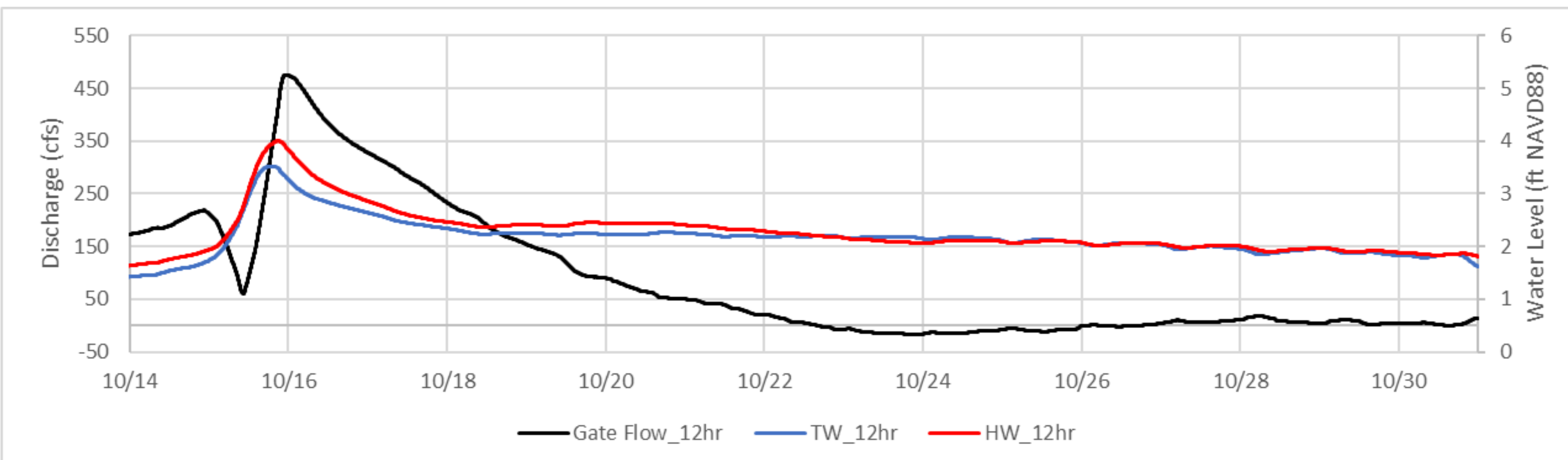


Figure B 2-23. 12-Hour Moving Average Discharge and Stages at G93 for the 10-year 3-day Future Conditions (SLR2) Event

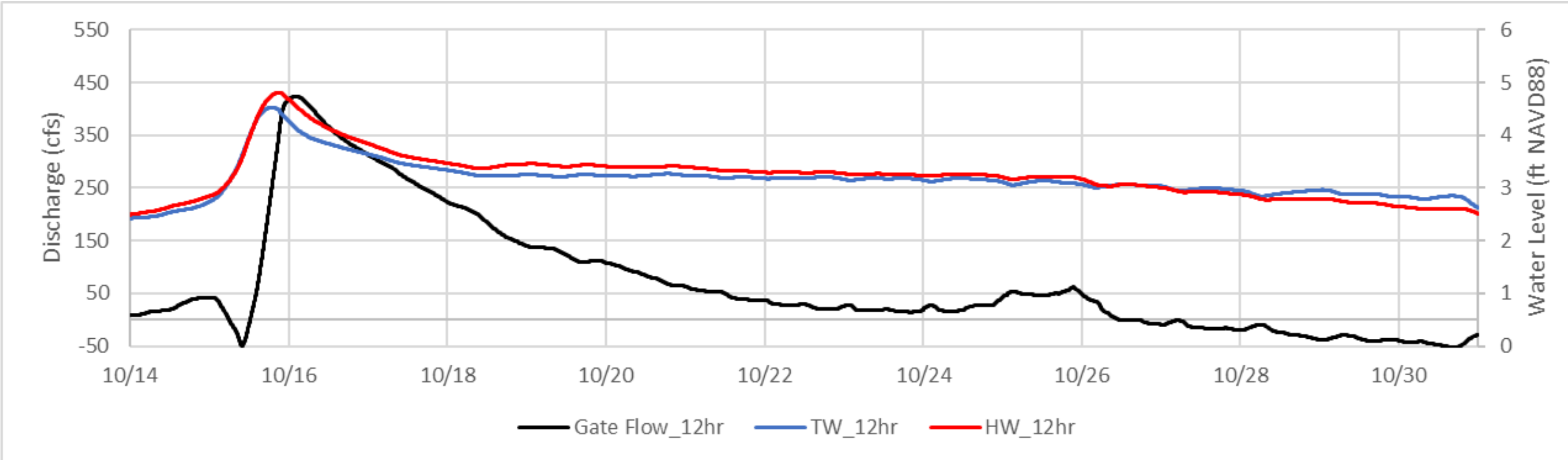


Figure B 2-24. 12-Hour Moving Average Discharge and Stages at G93 for the 10-year 3-day Future Conditions (SLR3) Event

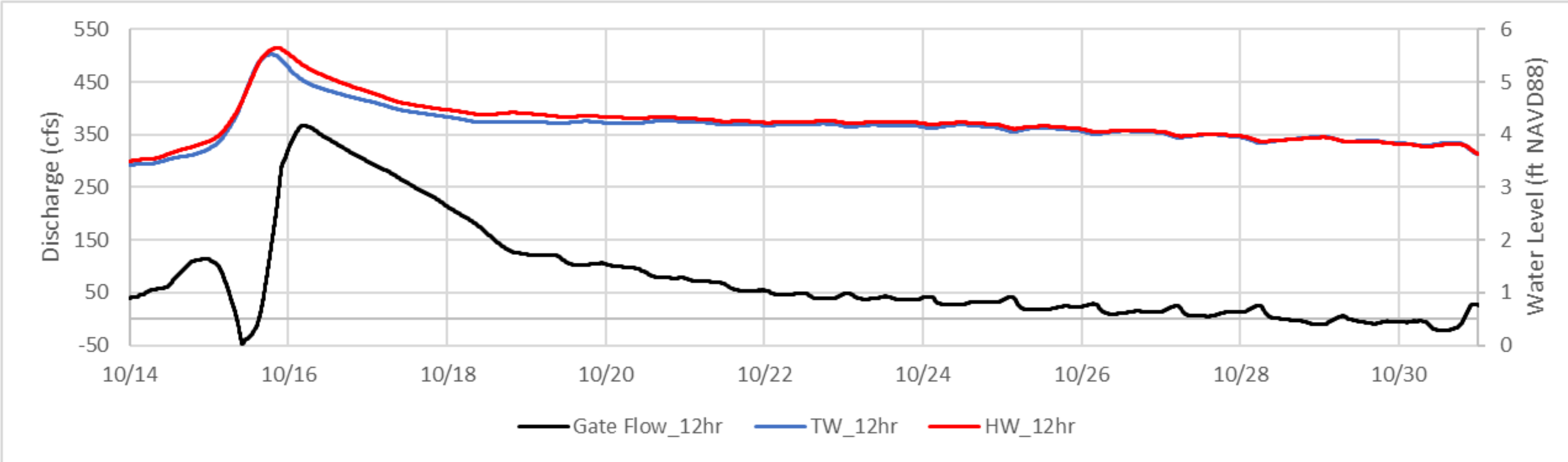


Figure B 2-25. Instantaneous Discharge and Stages at G93 for the 5-year 3-day Current Conditions Event

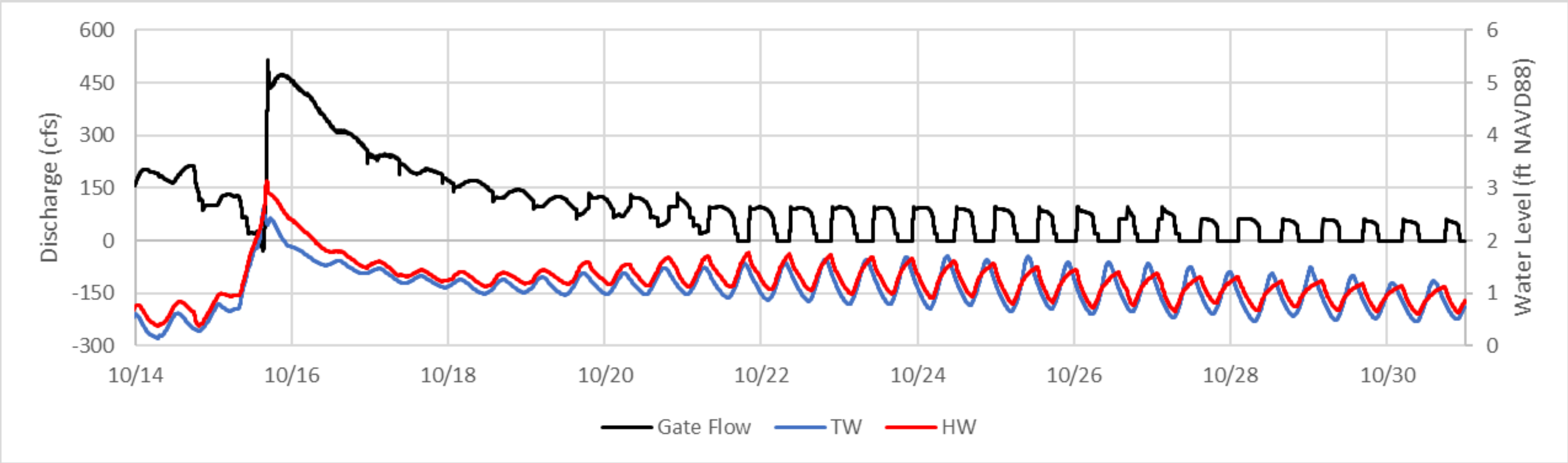


Figure B 2-26. Instantaneous Discharge and Stages at G93 for the 5-year 3-day Future Conditions (SLR1) Event

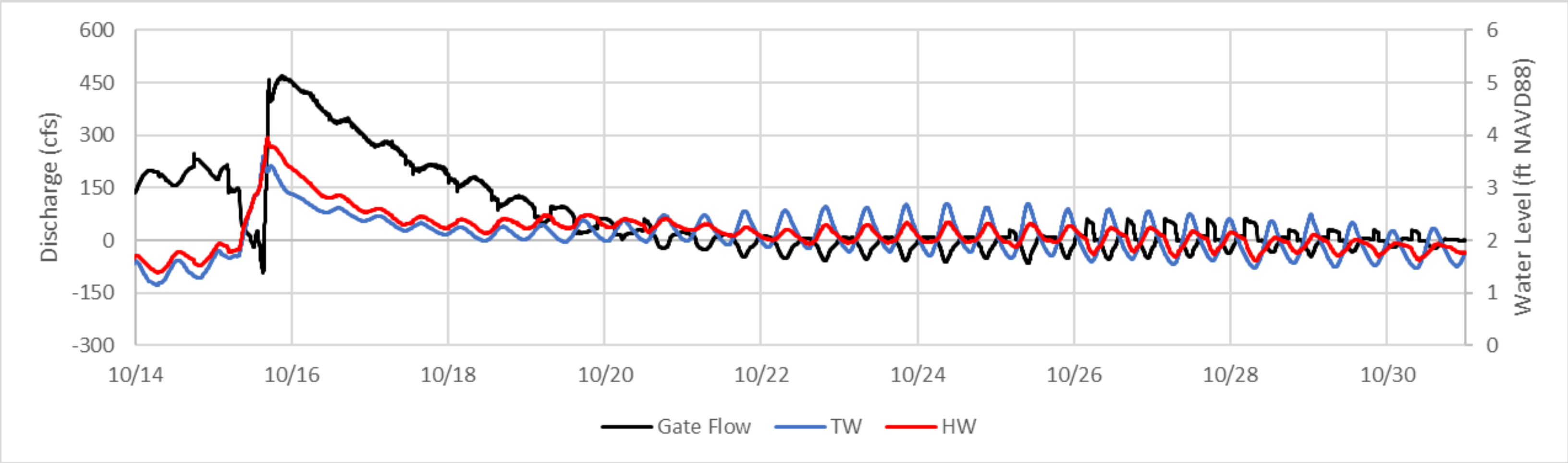


Figure B 2-27. Instantaneous Discharge and Stages at G93 for the 5-year 3-day Future Conditions (SLR2) Event

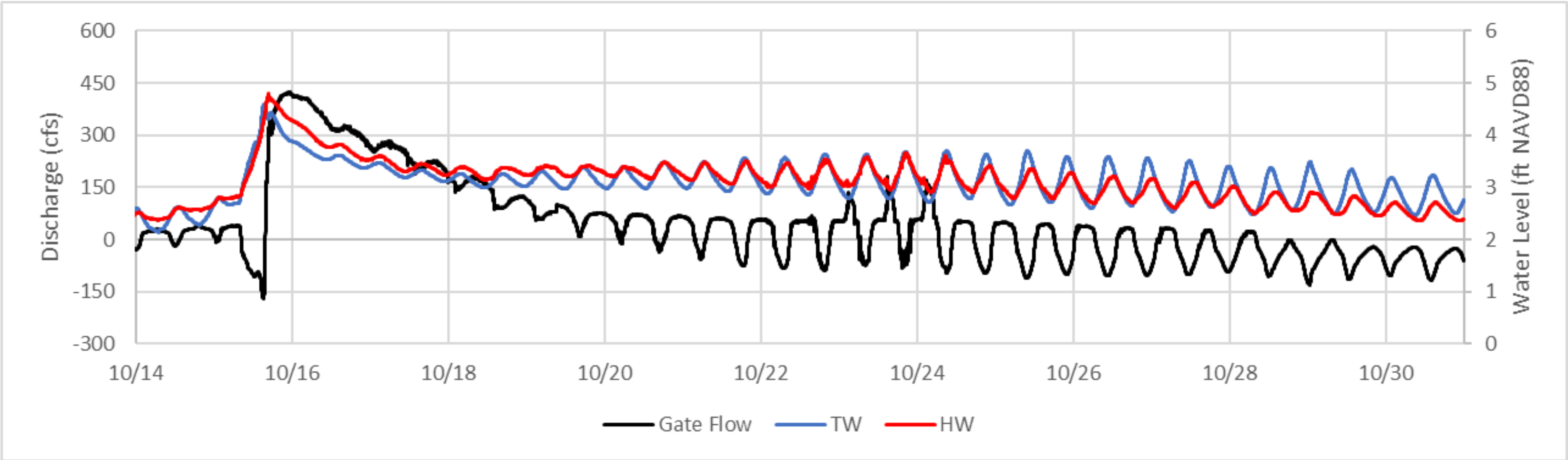


Figure B 2-28. Instantaneous Discharge and Stages at G93 for the 5-year 3-day Future Conditions (SLR3) Event

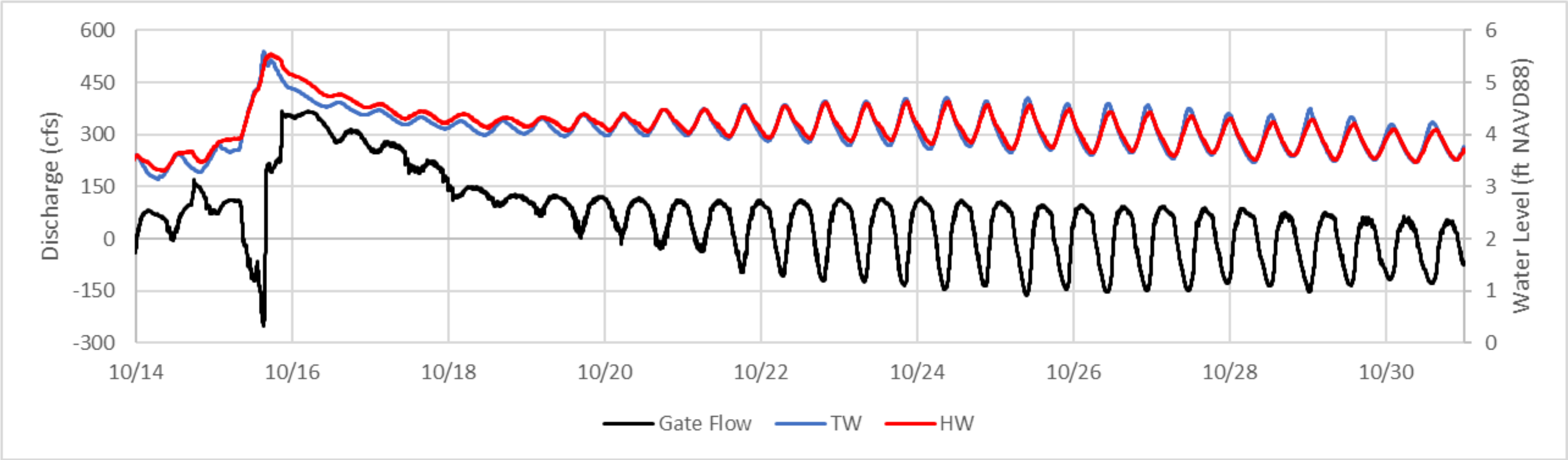


Figure B 2-29. 12-Hour Moving Average Discharge and Stages at G93 for the 5-year 3-day Current Conditions Event

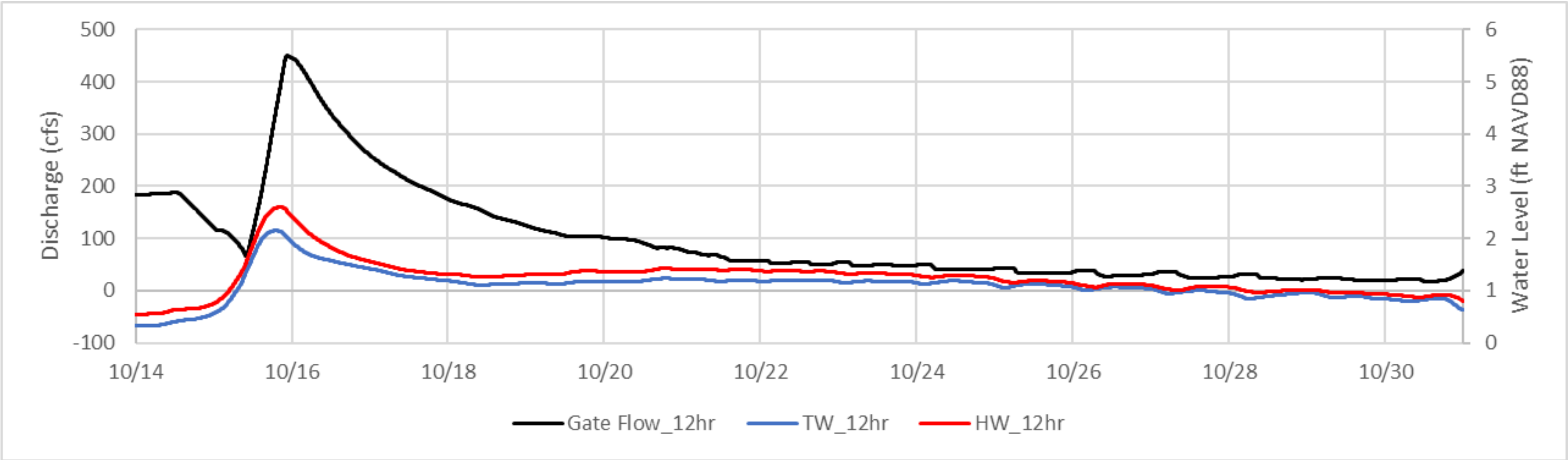


Figure B 2-30. 12-Hour Moving Average Discharge and Stages at G93 for the 5-year 3-day Future Conditions (SLR1) Event

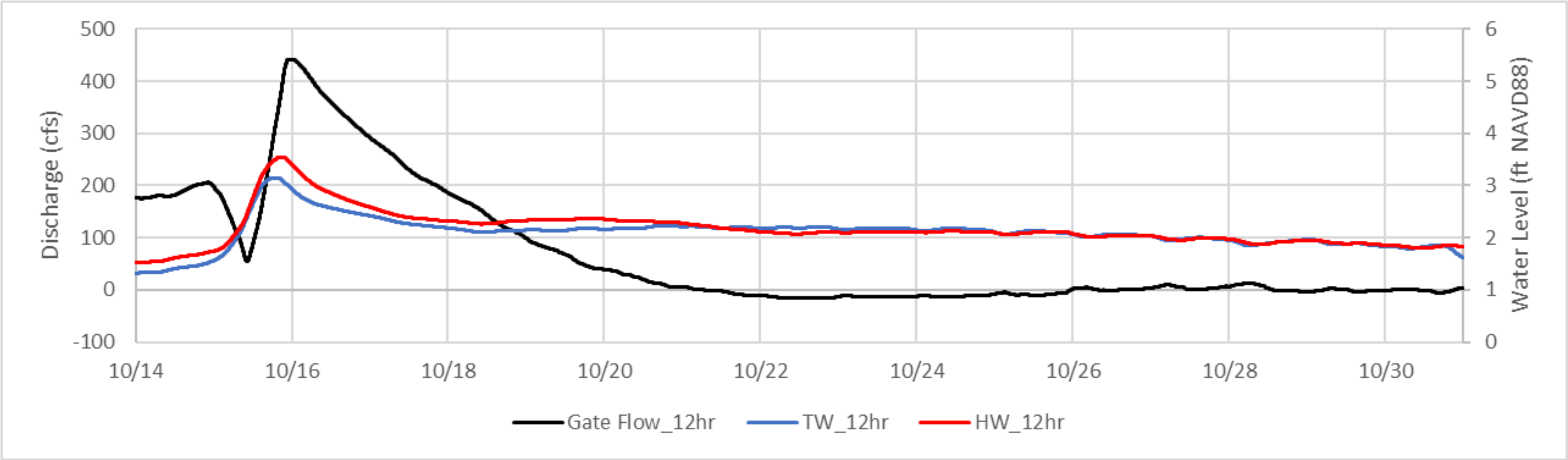


Figure B 2-31. 12-Hour Moving Average Discharge and Stages at G93 for the 5-year 3-day Future Conditions (SLR2) Event

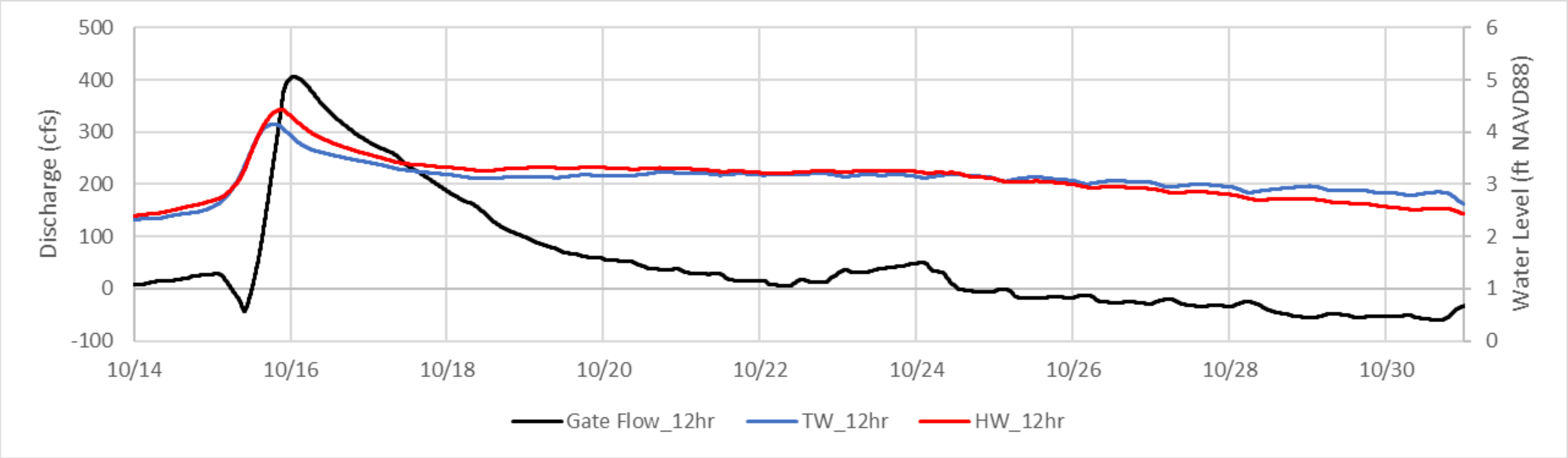
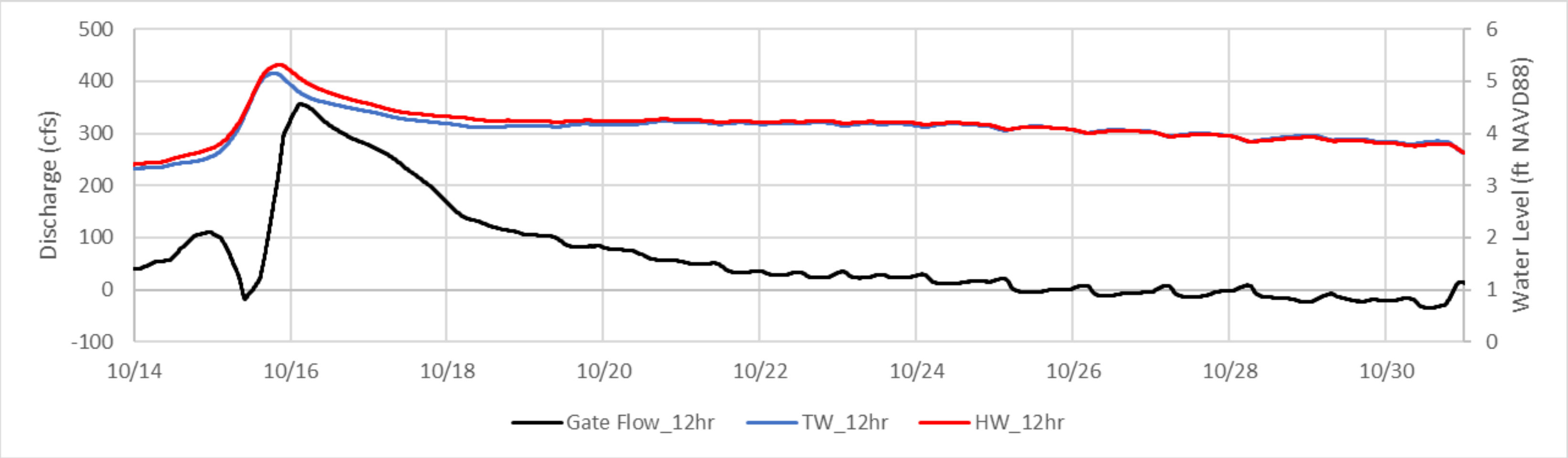


Figure B 2-32. 12-Hour Moving Average Discharge and Stages at G93 for the 5-year 3-day Future Conditions (SLR3) Event



3 C4 Watershed

Figure B 3-1 through **Figure B 3-32** show the instantaneous and 12-hour average discharge, HW, and TW at S25B for each SLR condition for the 100-, 25-, 10-, and 5-year design storm events.

Figure B 3-1. Instantaneous Discharge and Stages at S25B for the 100-year 3-day Current Conditions Event

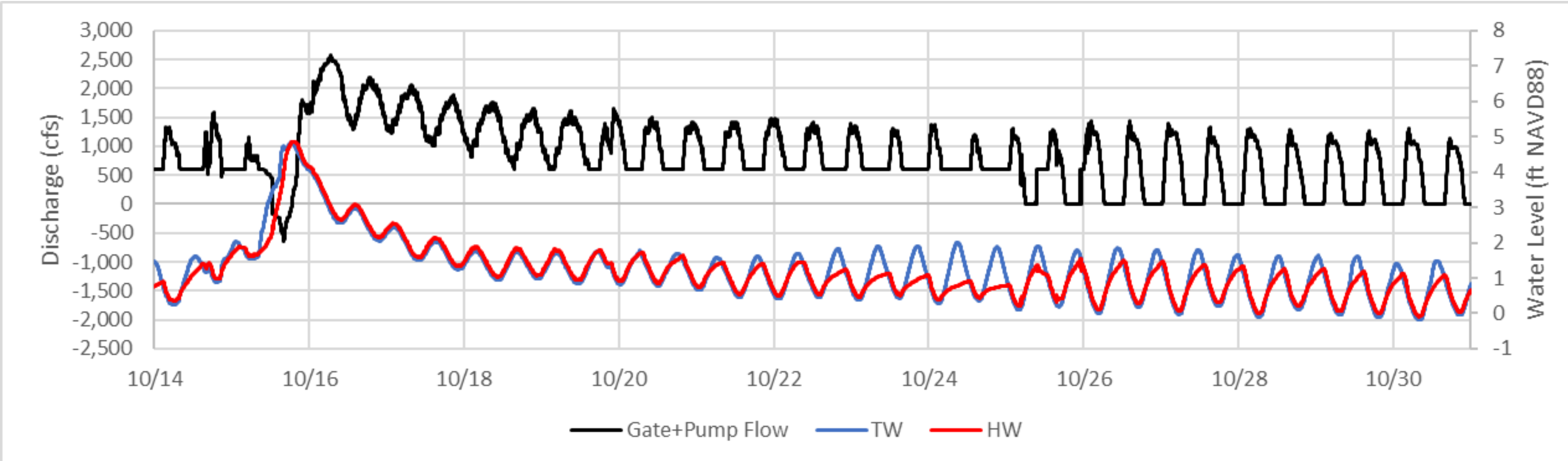


Figure B 3-2. Instantaneous Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR1) Event

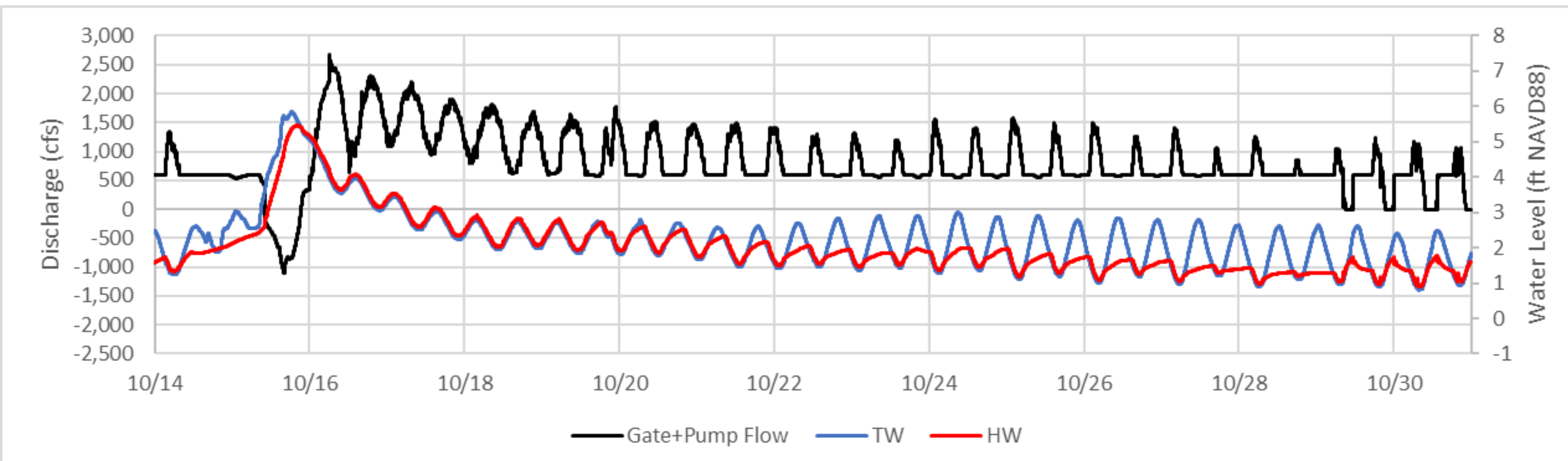


Figure B 3-3. Instantaneous Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR2) Event

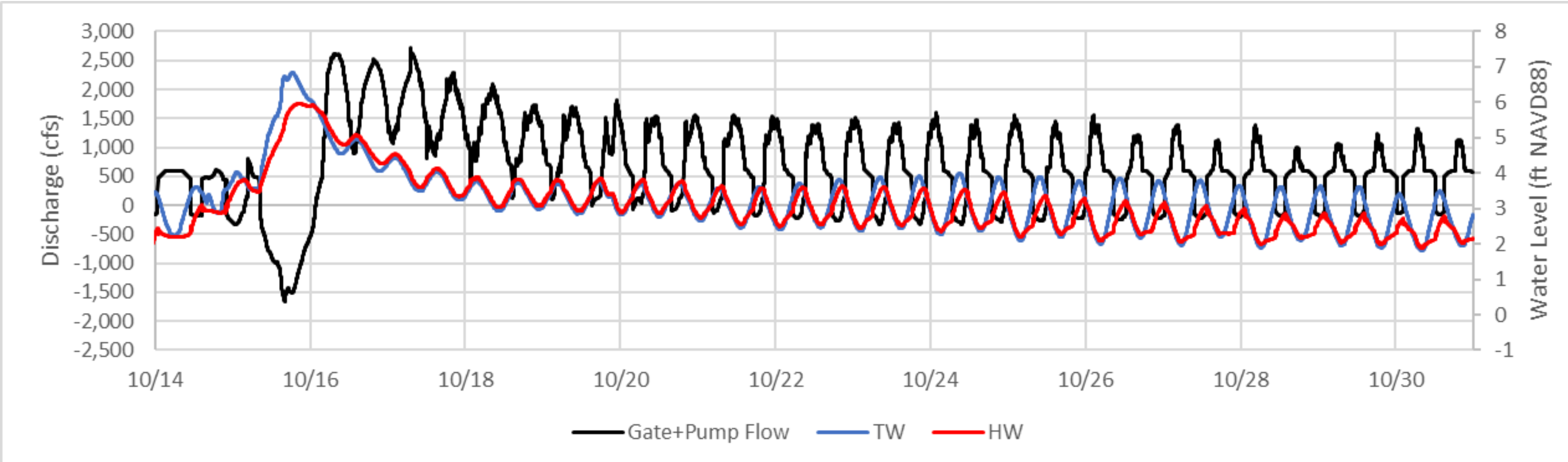


Figure B 3-4. Instantaneous Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR3) Event

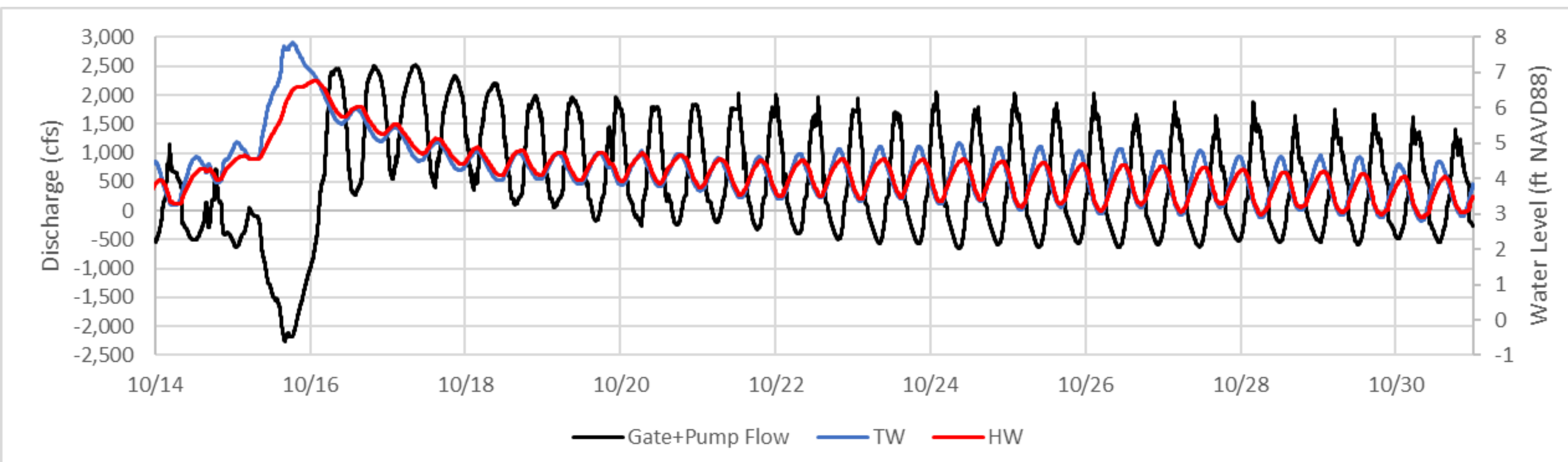


Figure B 3-5. 12-Hour Moving Average Discharge and Stages at S25B for the 100-year 3-day Current Conditions Event

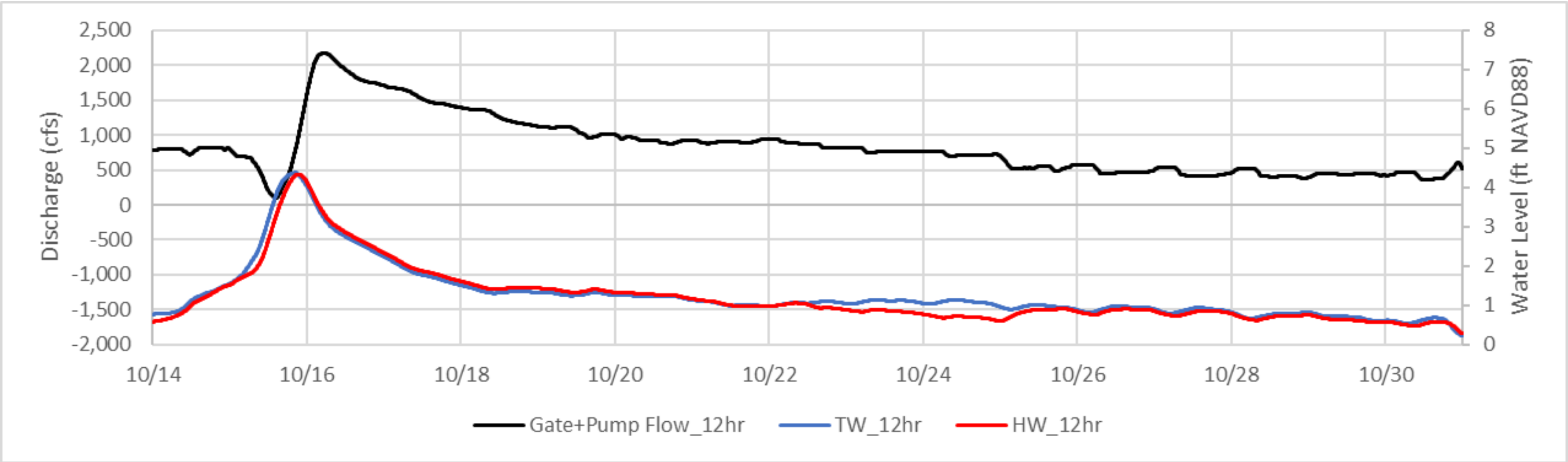


Figure B 3-6. 12-Hour Moving Average Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR1) Event

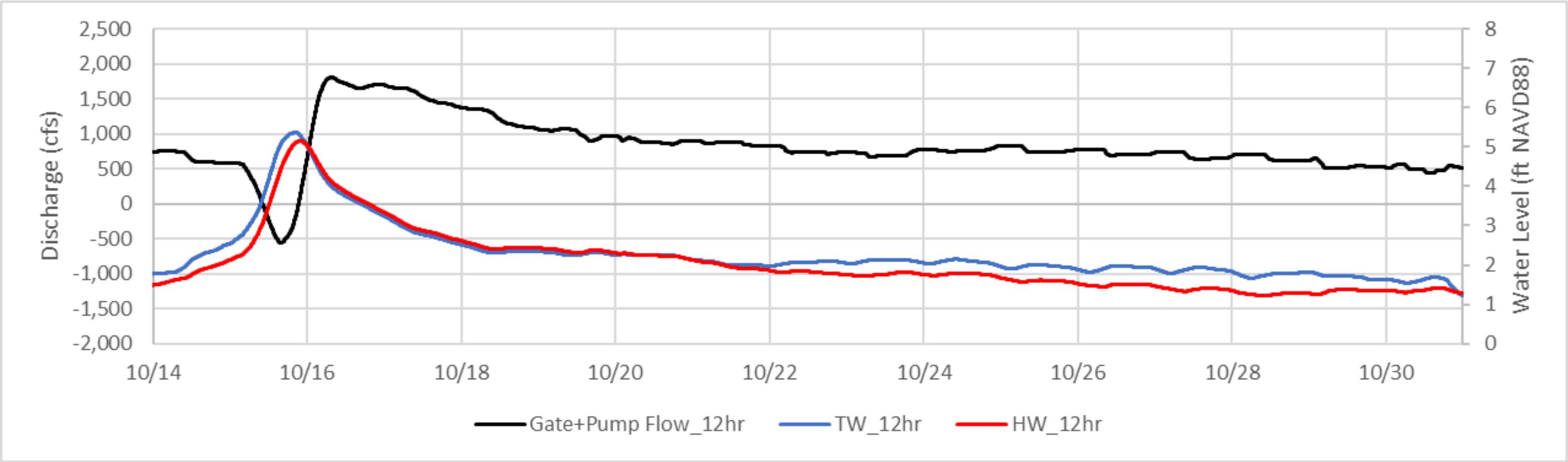


Figure B 3-7. 12-Hour Moving Average Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR2) Event

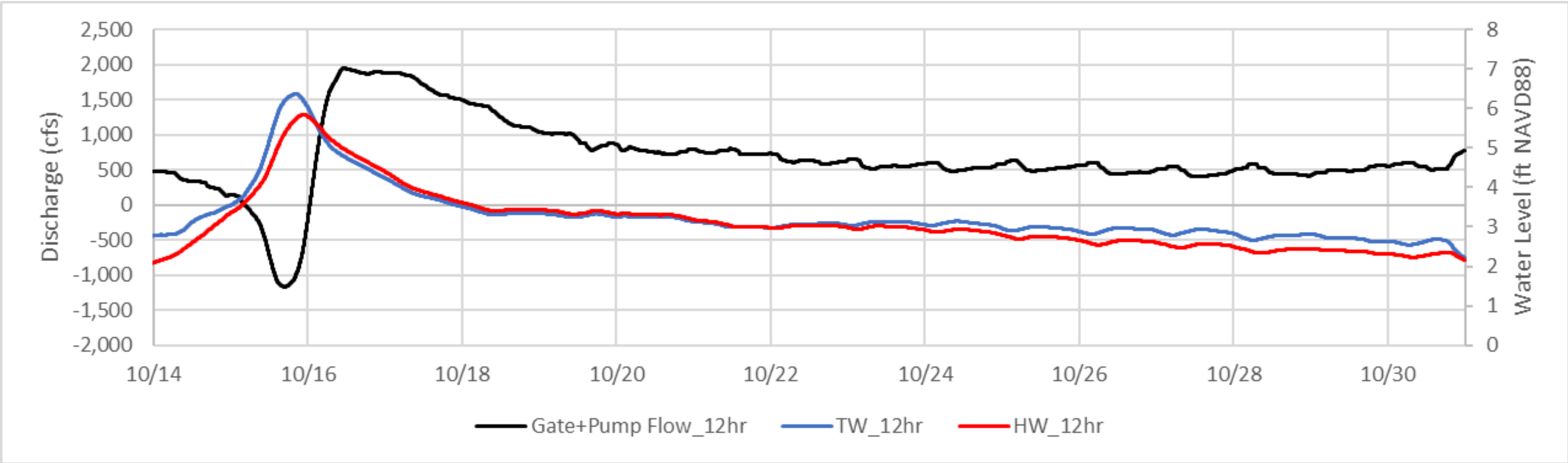


Figure B 3-8. 12-Hour Moving Average Discharge and Stages at S25B for the 100-year 3-day Future Conditions (SLR3) Event

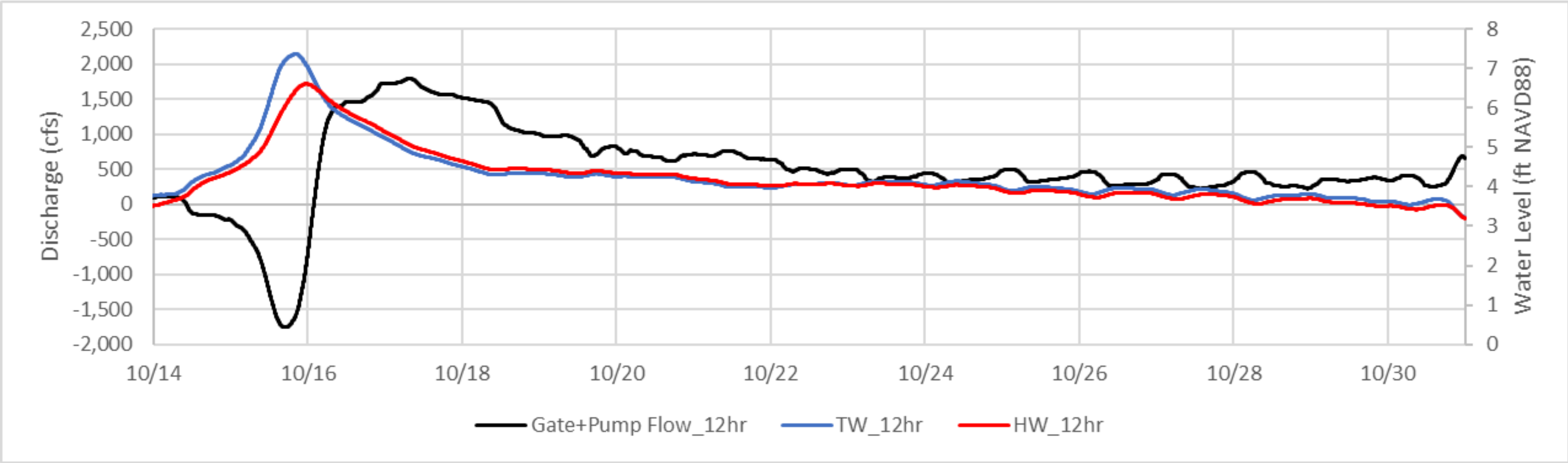


Figure B 3-9. Instantaneous Discharge and Stages at S25B for the 25-year 3-day Current Conditions Event

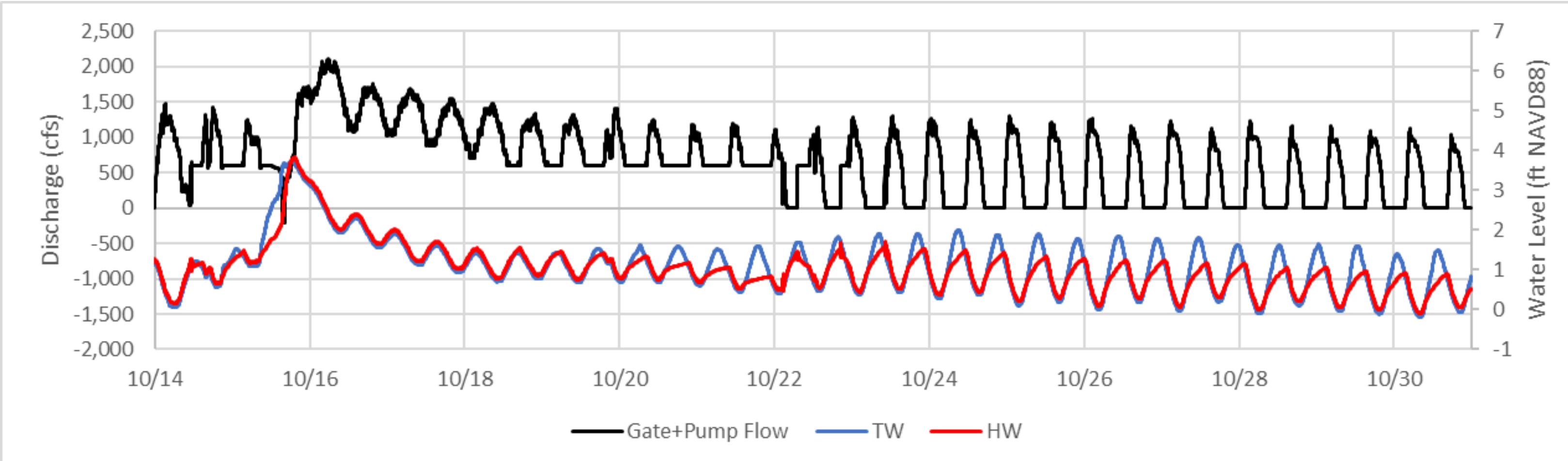


Figure B 3-10. Instantaneous Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR1) Event

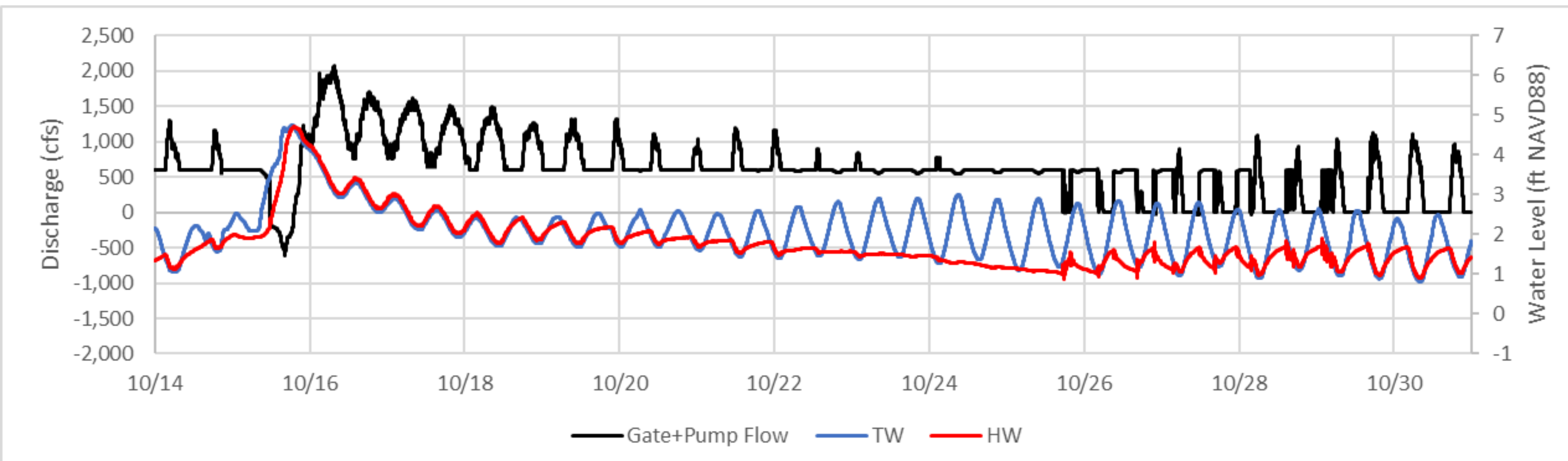


Figure B 3-11. Instantaneous Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR2) Event

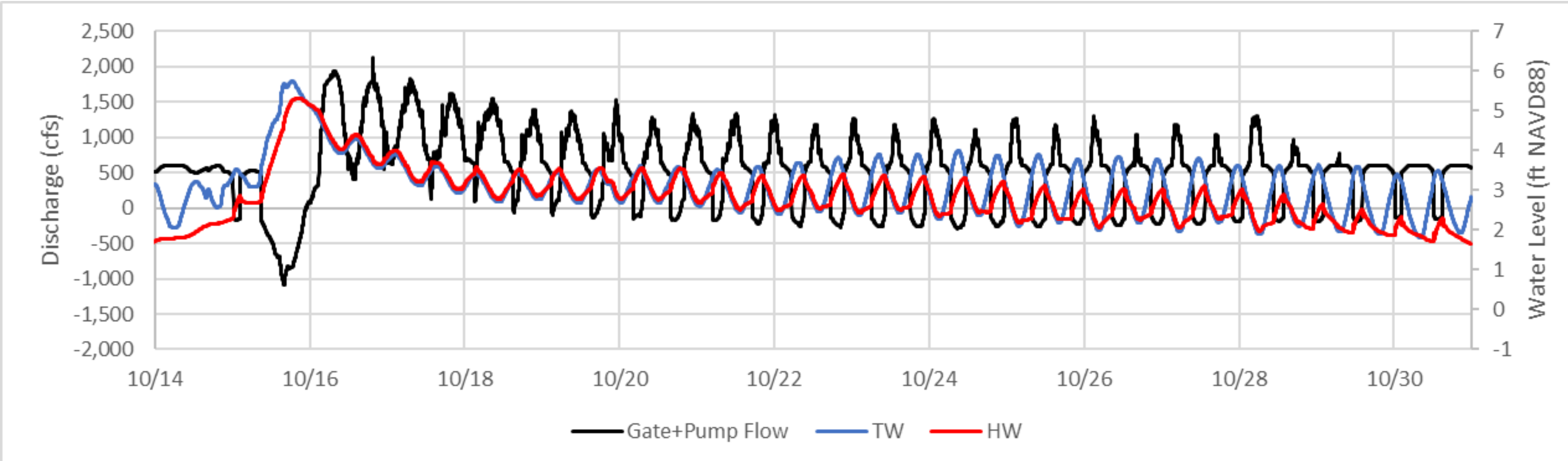


Figure B 3-12. Instantaneous Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR3) Event

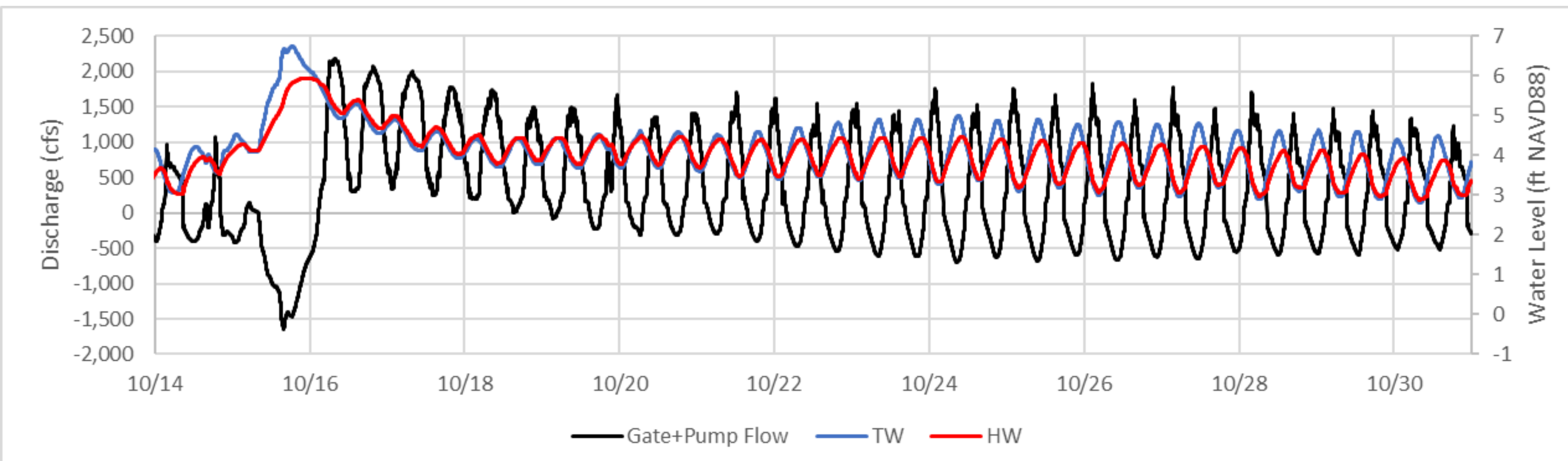


Figure B 3-13. 12-Hour Moving Average Discharge and Stages at S25B for the 25-year 3-day Current Conditions Event

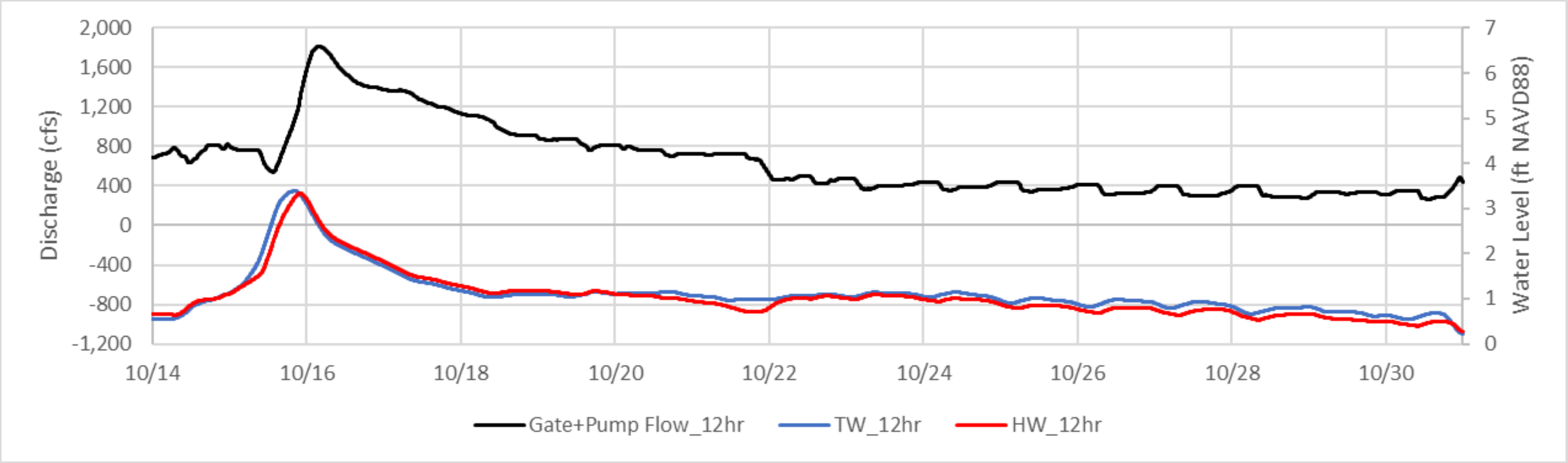


Figure B 3-14. 12-Hour Moving Average Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR1) Event

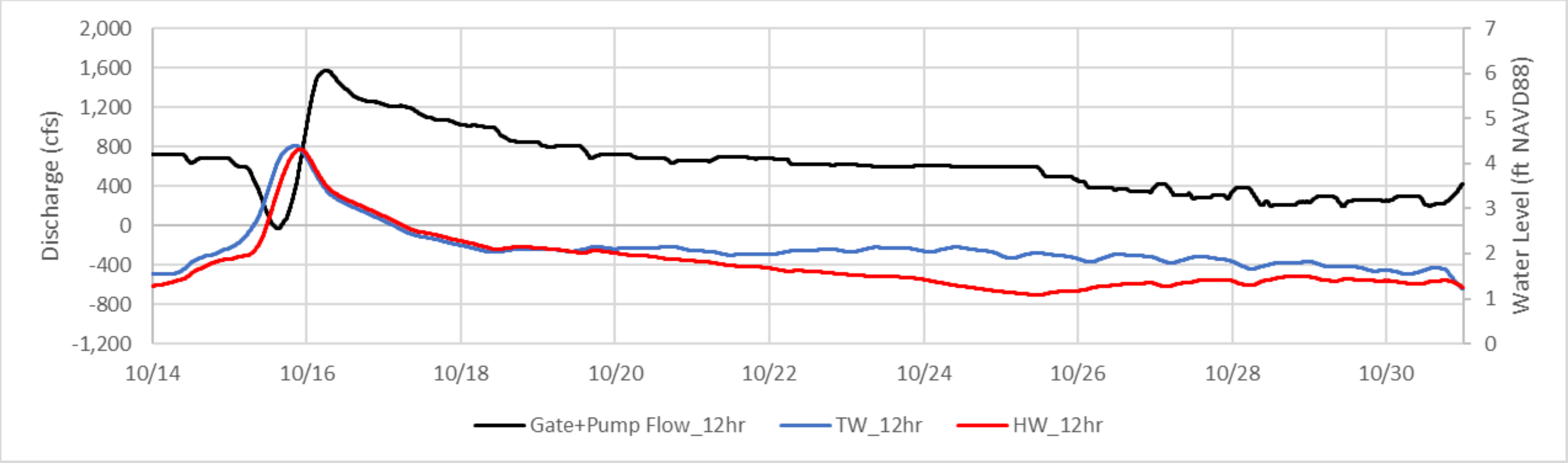


Figure B 3-15. 12-Hour Moving Average Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR2) Event

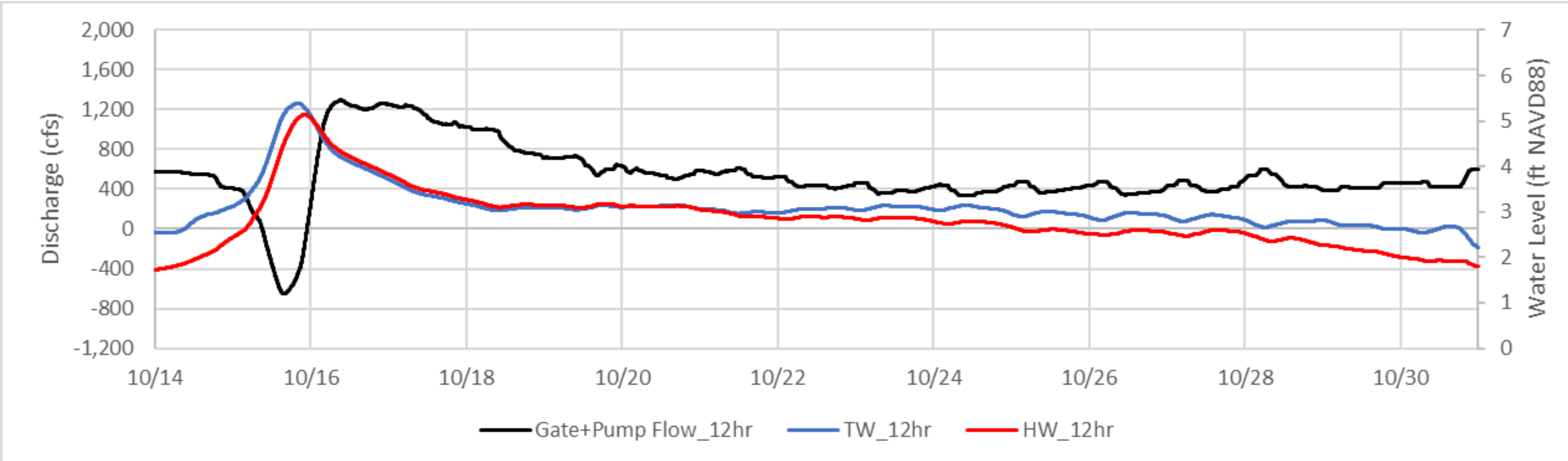


Figure B 3-16. 12-Hour Moving Average Discharge and Stages at S25B for the 25-year 3-day Future Conditions (SLR3) Event

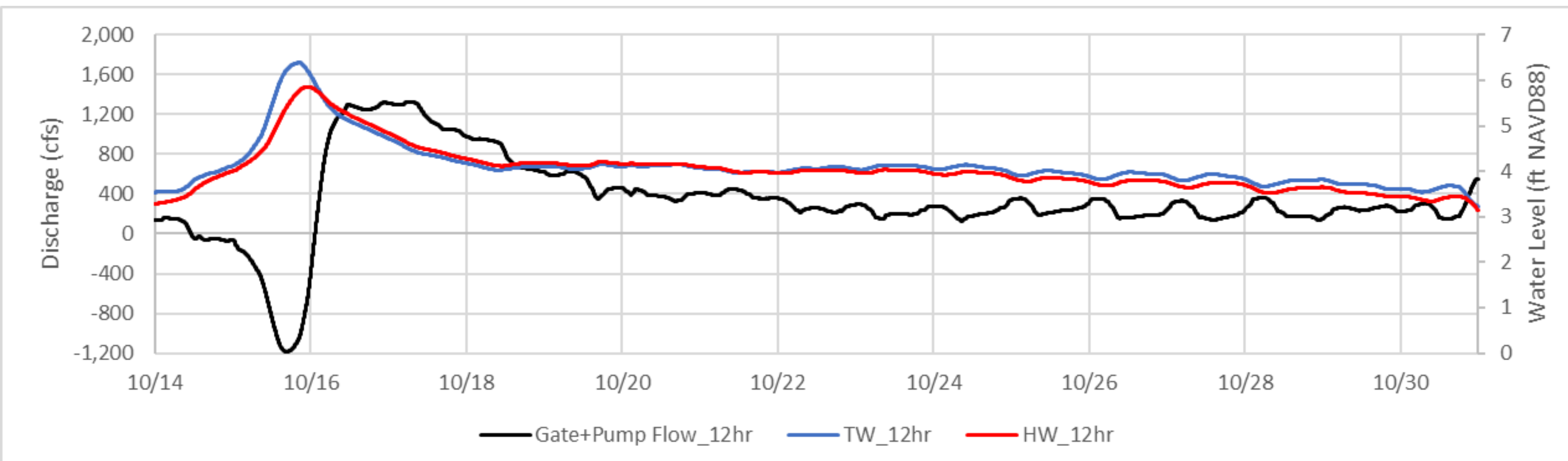


Figure B 3-17. Instantaneous Discharge and Stages at S25B for the 10-year 3-day Current Conditions Event

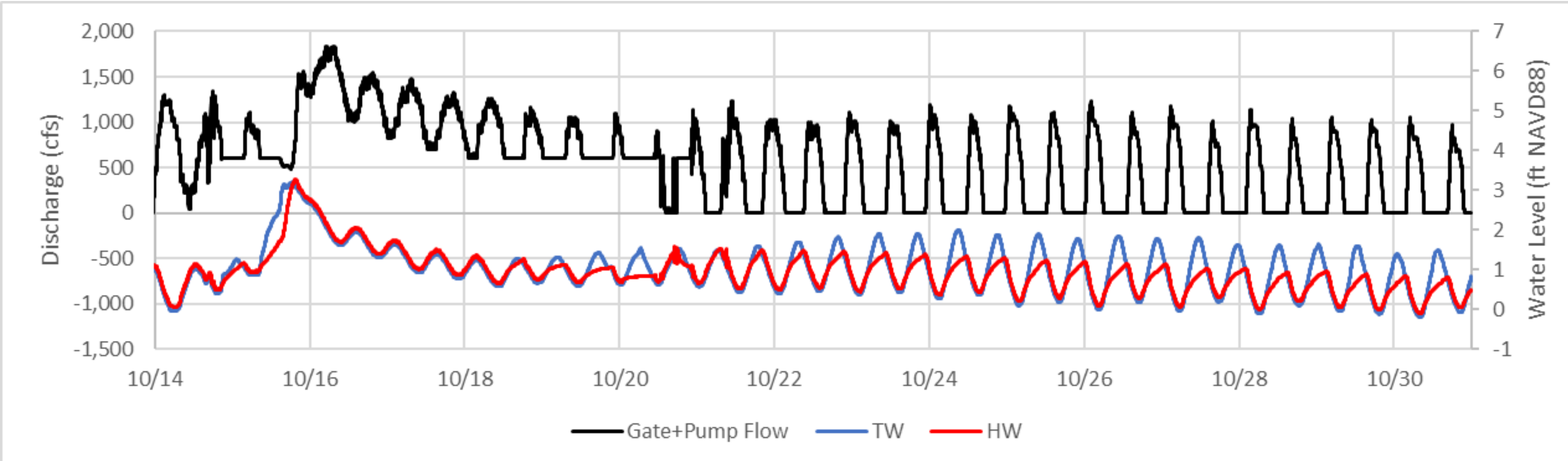


Figure B 3-18. Instantaneous Discharge and Stages at S25B for the 10-year 3-day Future Conditions (SLR1) Event

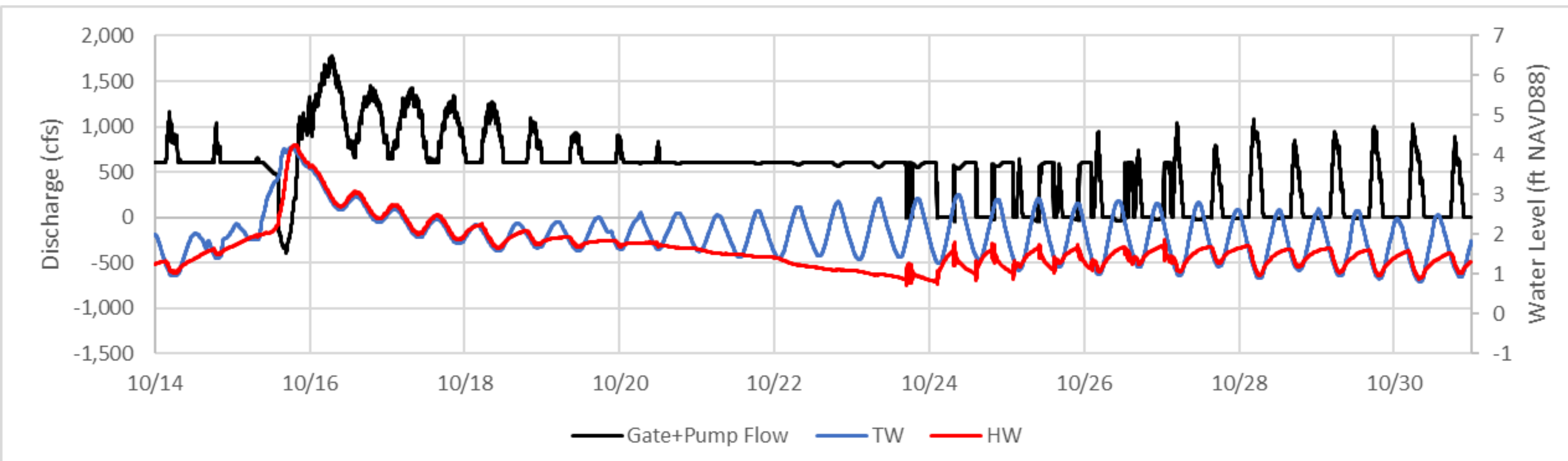


Figure B 3-19. Instantaneous Discharge and Stages at S25B for the 10-year 3-day Future Conditions (SLR2) Event

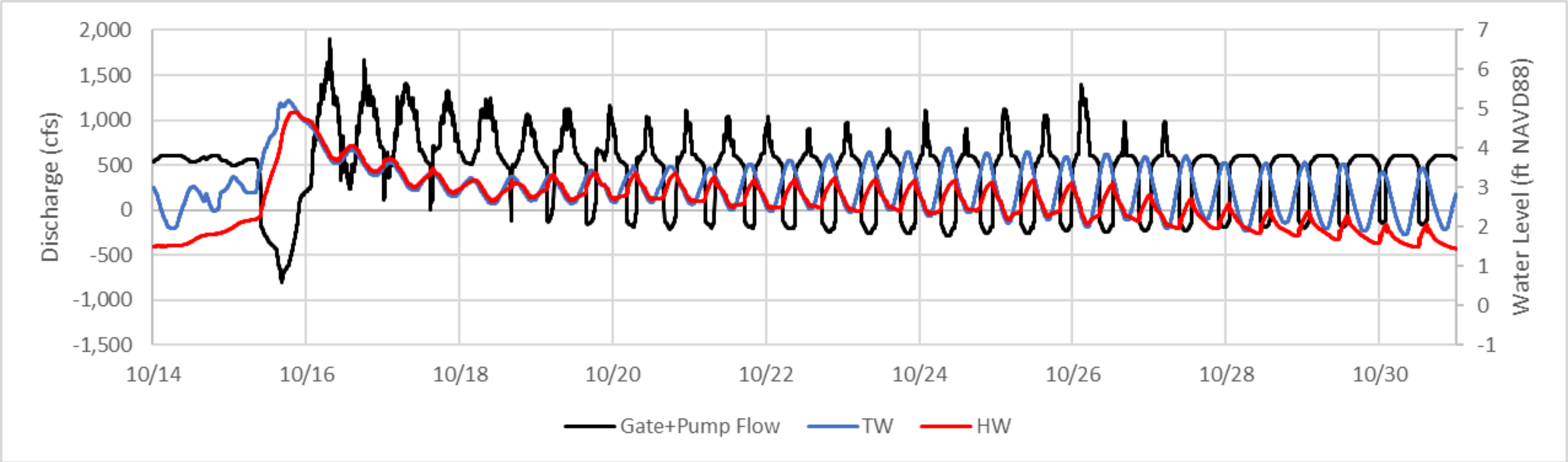


Figure B 3-20. Instantaneous Discharge and Stages at S25B for the 10-year 3-day Future Conditions (SLR3) Event

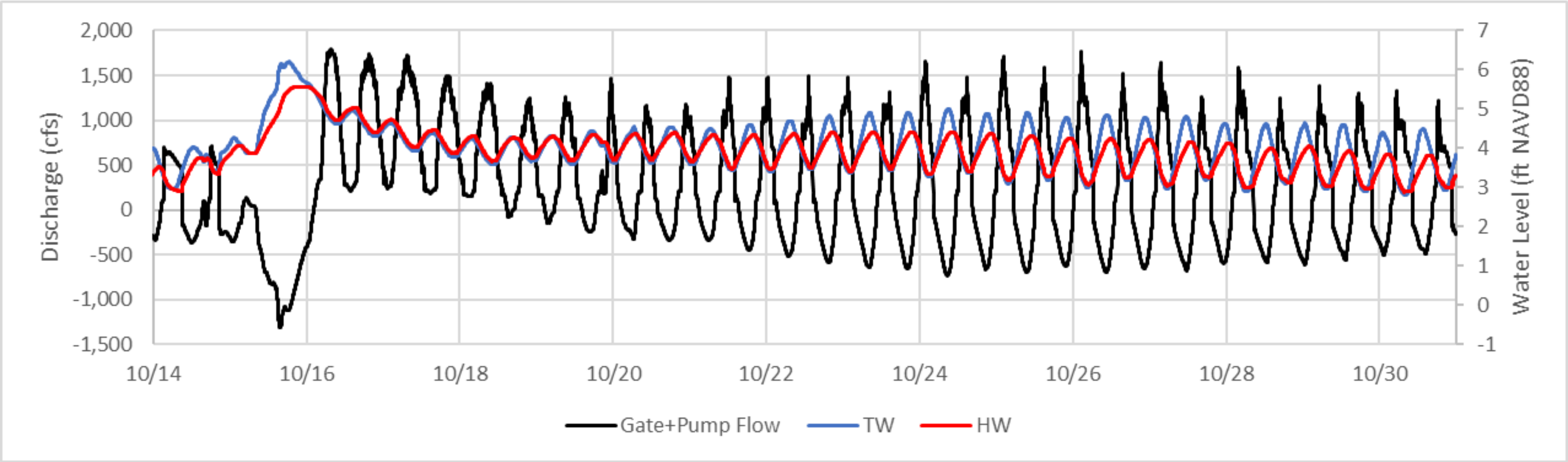


Figure B 3-21. 12-Hour Moving Average Discharge and Stages at S25B for the 10-year 3-day Current Conditions Event

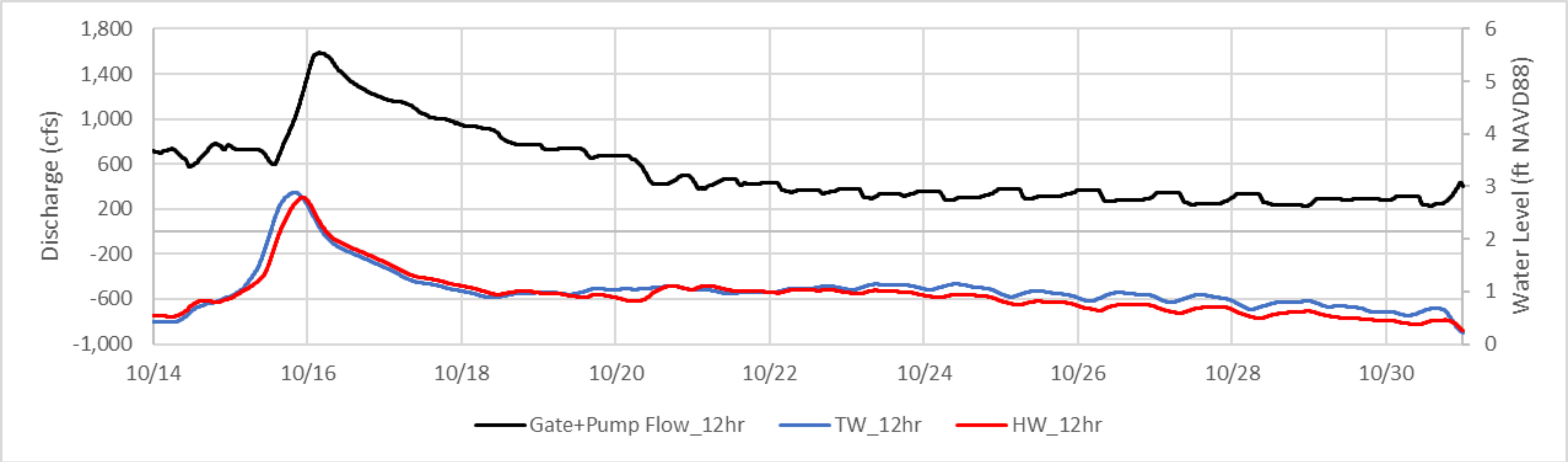


Figure B 3-22. 12-Hour Moving Average Discharge and Stages at S25B for the 10-year 3-day Future Conditions (SLR1) Event

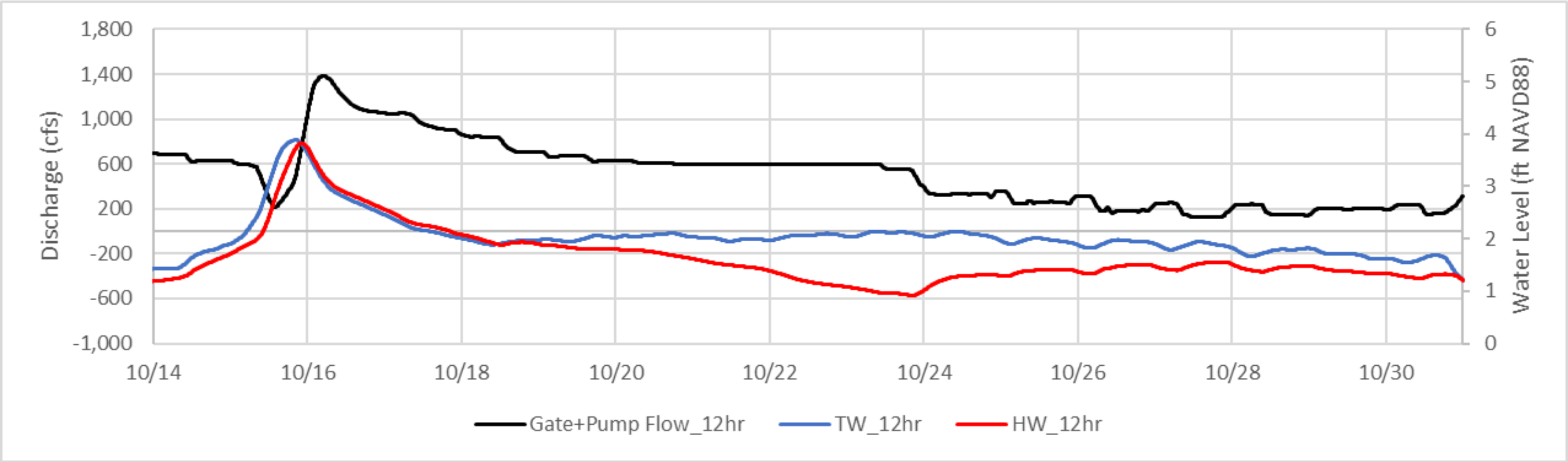


Figure B 3-23. 12-Hour Moving Average Discharge and Stages at S25B for the 10-year 3-day Future Conditions (SLR2) Event

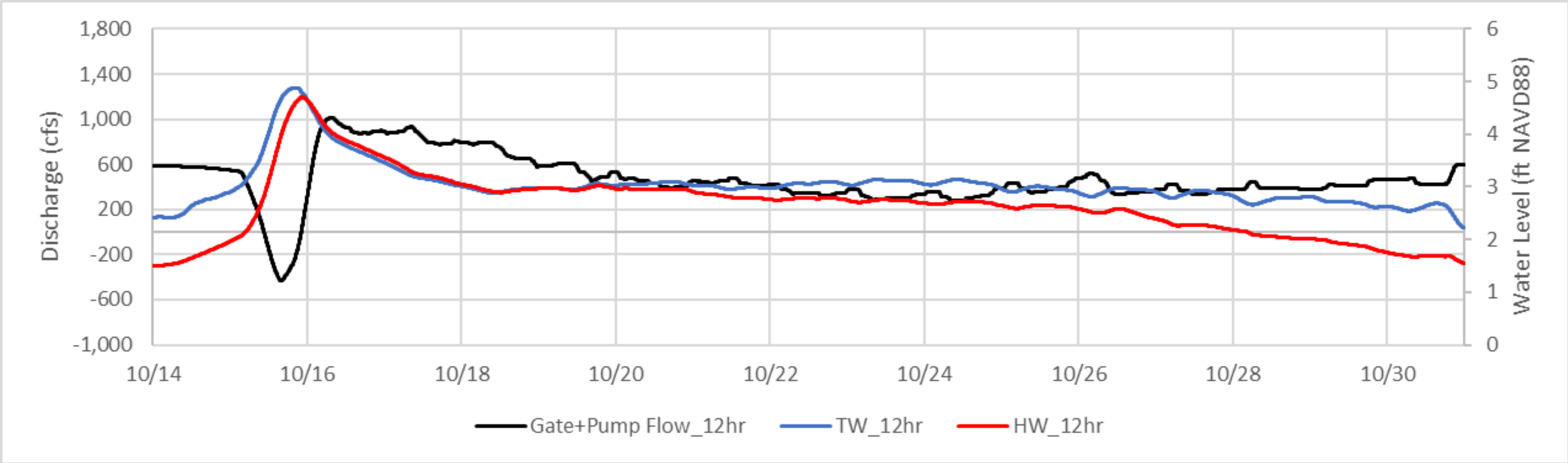


Figure B 3-24. 12-Hour Moving Average Discharge and Stages at S25B for the 10-year 3-day Future Conditions (SLR3) Event

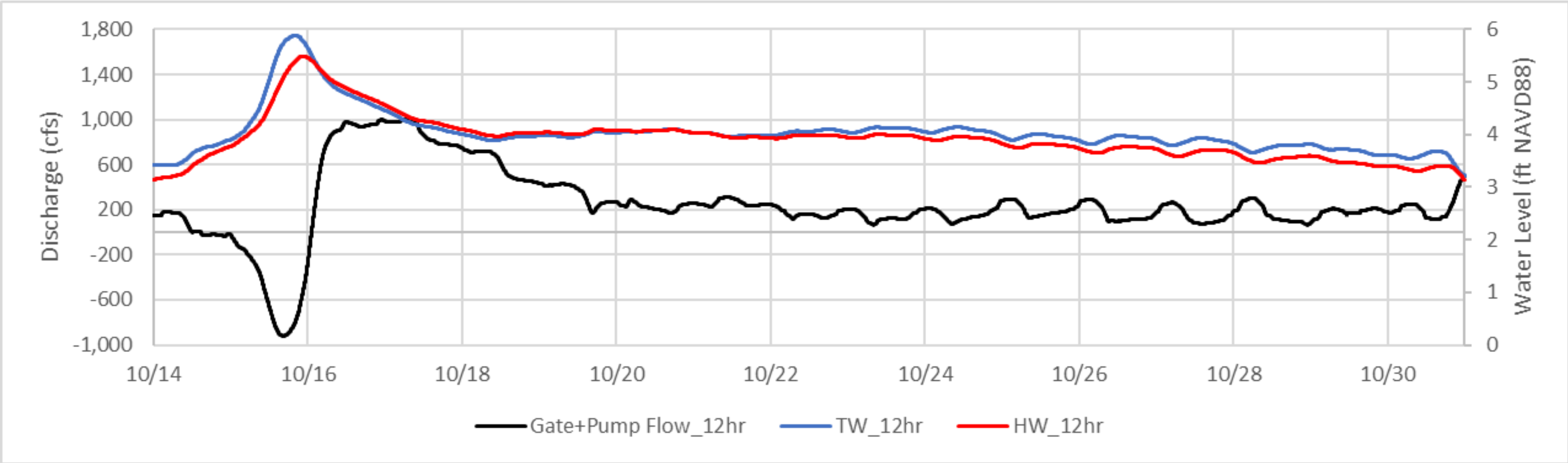


Figure B 3-25. Instantaneous Discharge and Stages at S25B for the 5-year 3-day Current Conditions Event

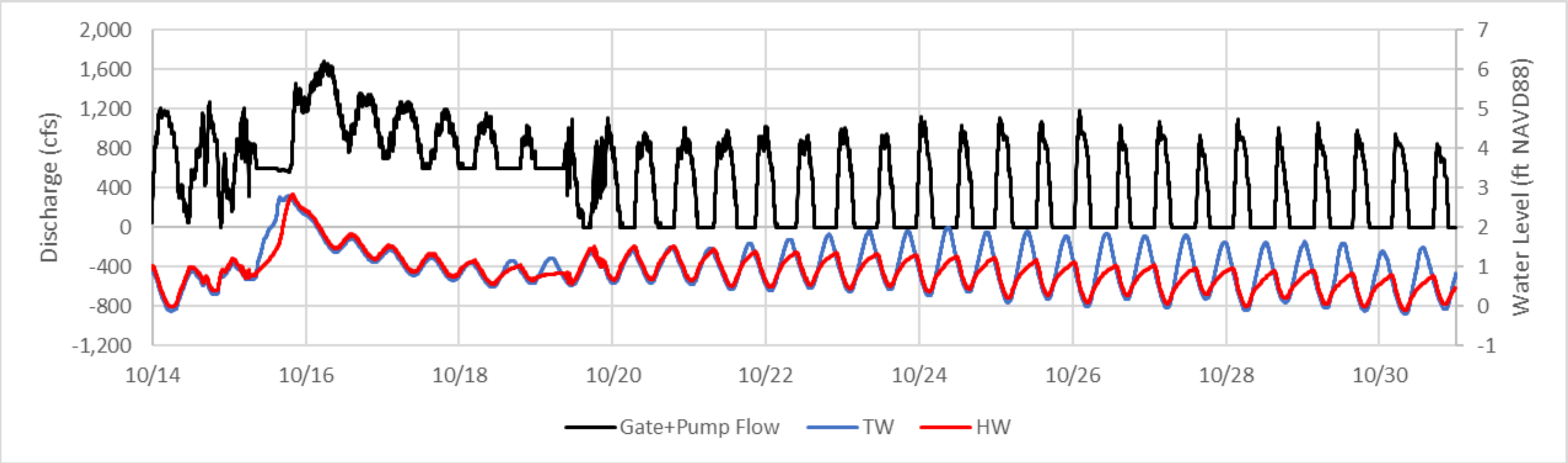


Figure B 3-26. Instantaneous Discharge and Stages at S25B for the 5-year 3-day Future Conditions (SLR1) Event

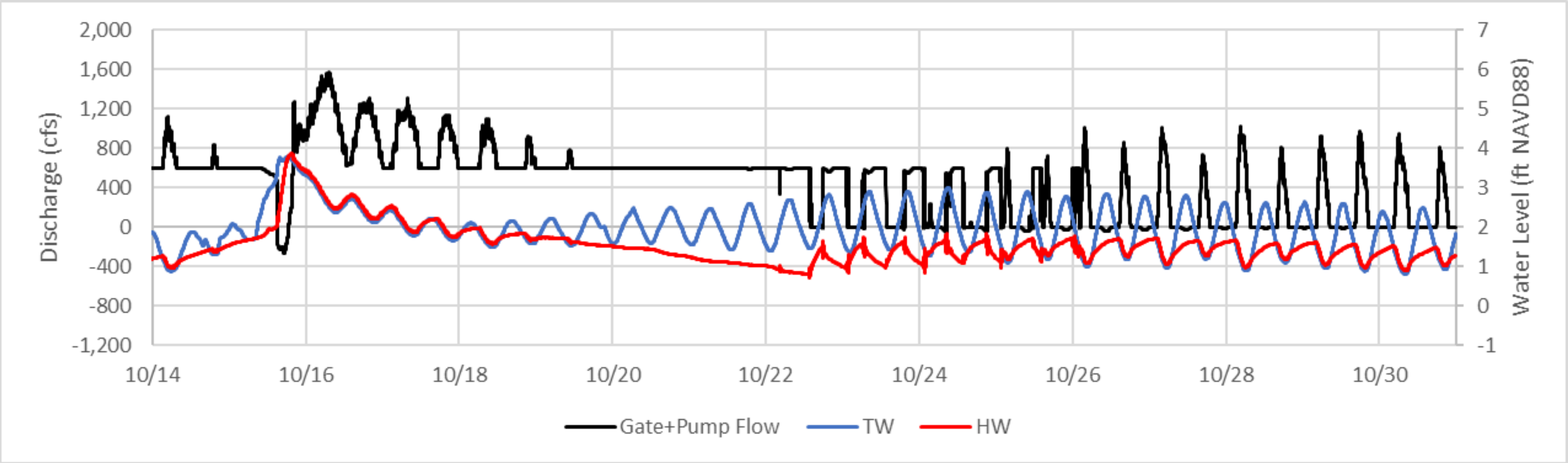


Figure B 3-27. Instantaneous Discharge and Stages at S25B for the 5-year 3-day Future Conditions (SLR2) Event

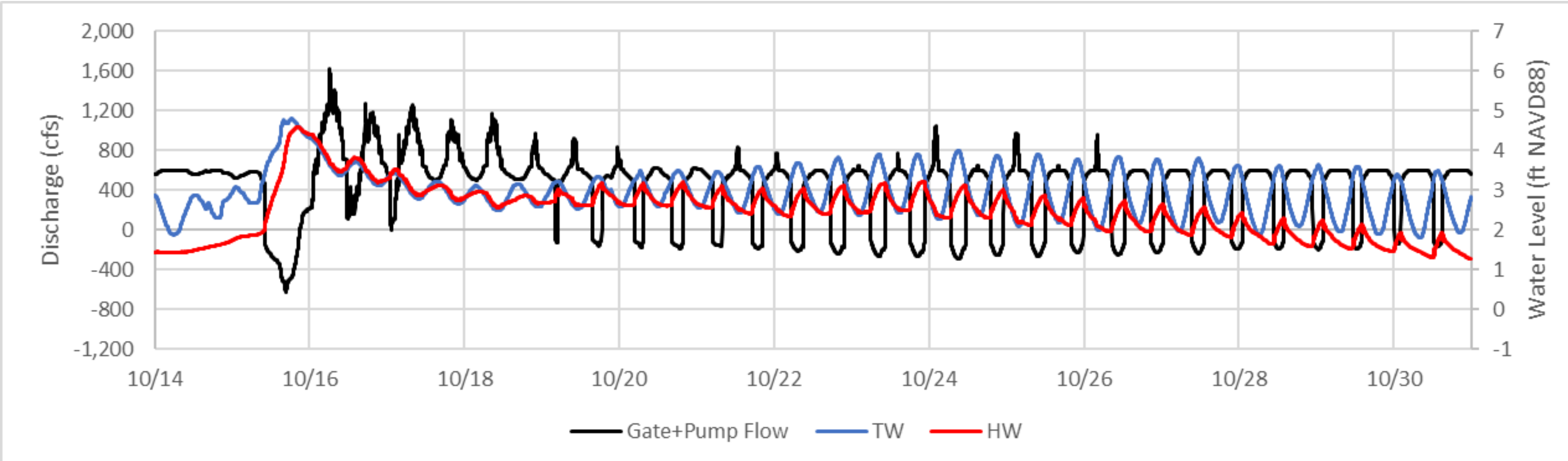


Figure B 3-28. Instantaneous Discharge and Stages at S25B for the 5-year 3-day Future Conditions (SLR3) Event

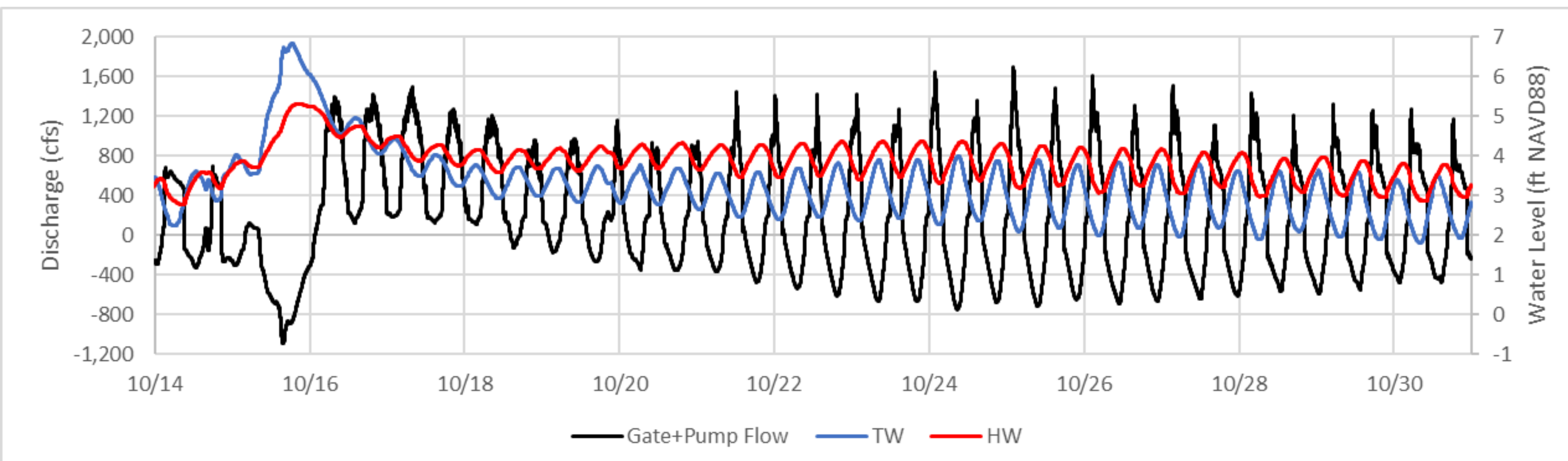


Figure B 3-29. 12-Hour Moving Average Discharge and Stages at S25B for the 5-year 3-day Current Conditions Event

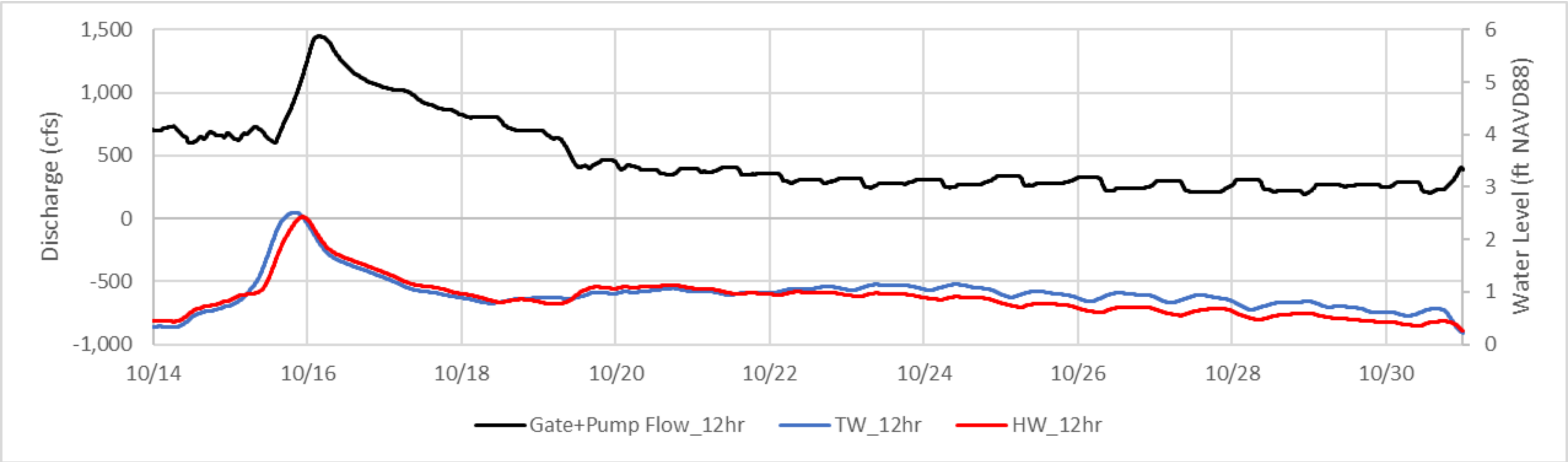


Figure B 3-30. 12-Hour Moving Average Discharge and Stages at S25B for the 5-year 3-day Future Conditions (SLR1) Event

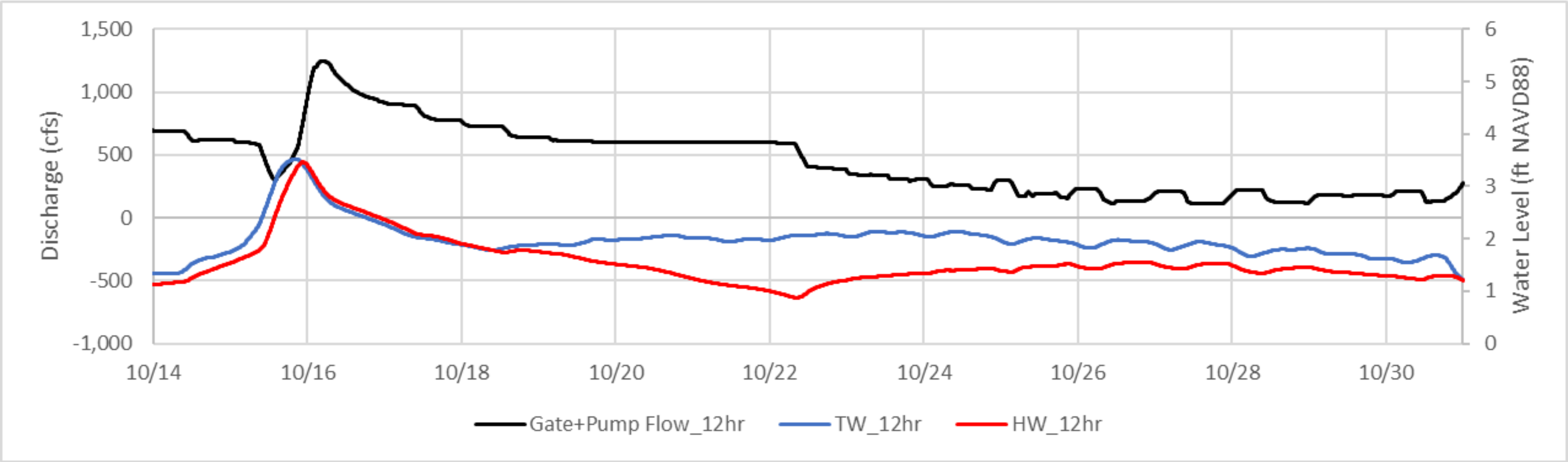


Figure B 3-31. 12-Hour Moving Average Discharge and Stages at S25B for the 5-year 3-day Future Conditions (SLR2) Event

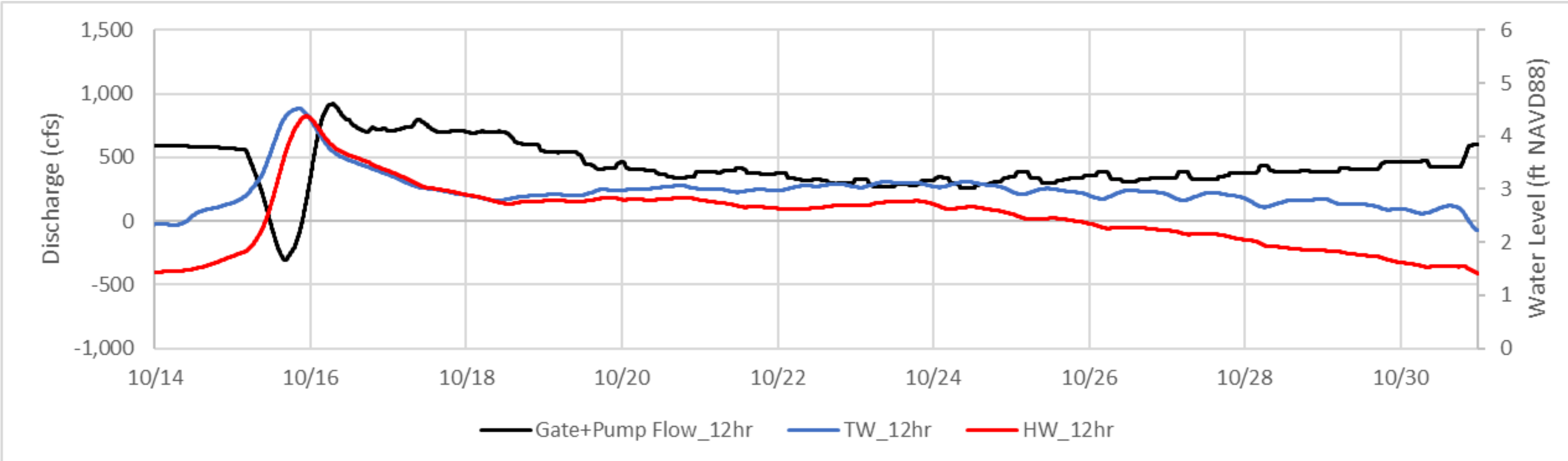


Figure B 3-32. 12-Hour Moving Average Discharge and Stages at S25B for the 5-year 3-day Future Conditions (SLR3) Event

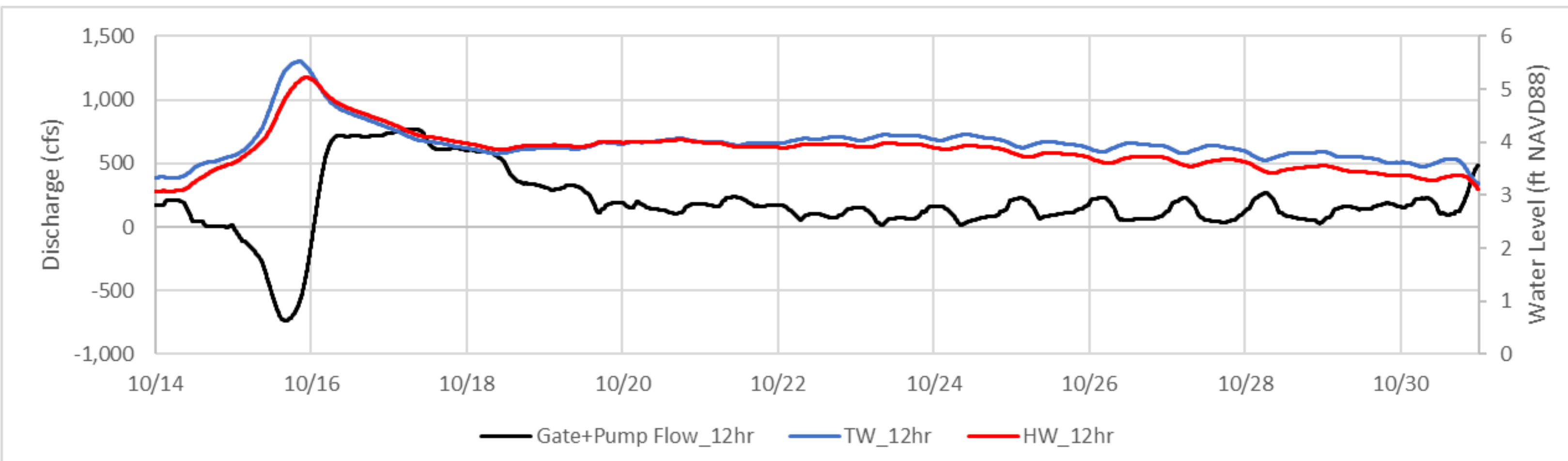


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Figure B 4-1 through **Figure B 4-32** show the instantaneous and 12-hour average discharge, HW, and TW at S25 for each SLR condition for the 100-, 25-, 10-, and 5-year design storm events.

Figure B 4-1. Instantaneous Discharge and Stages at S25 for the 100-year 3-day Current Conditions Event

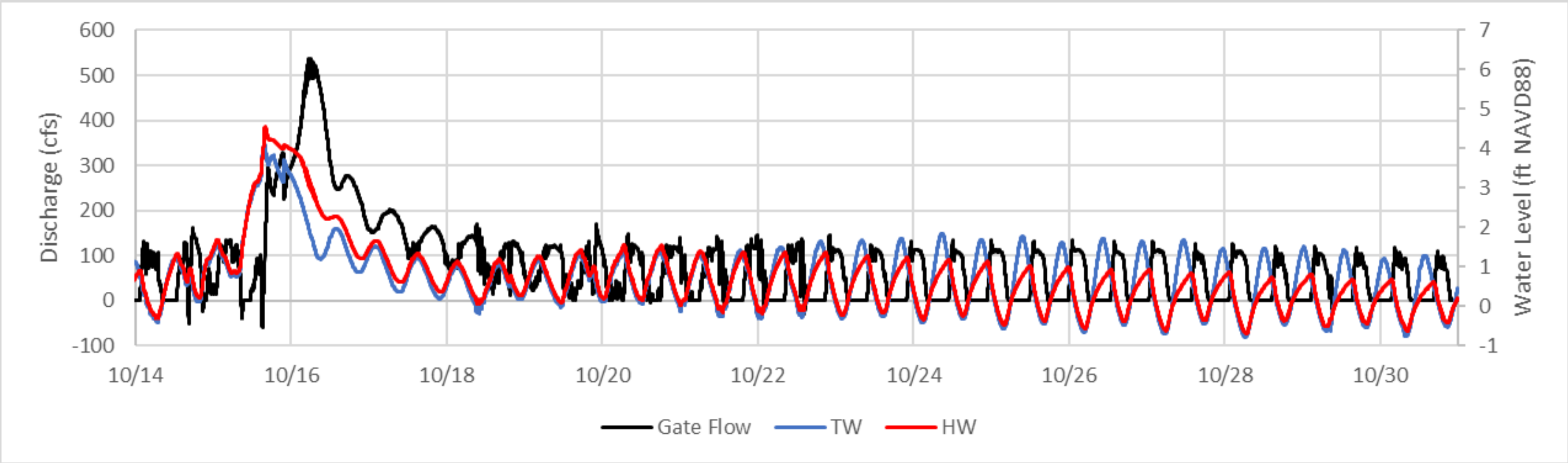

Figure B 4-2. Instantaneous Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR1) Event

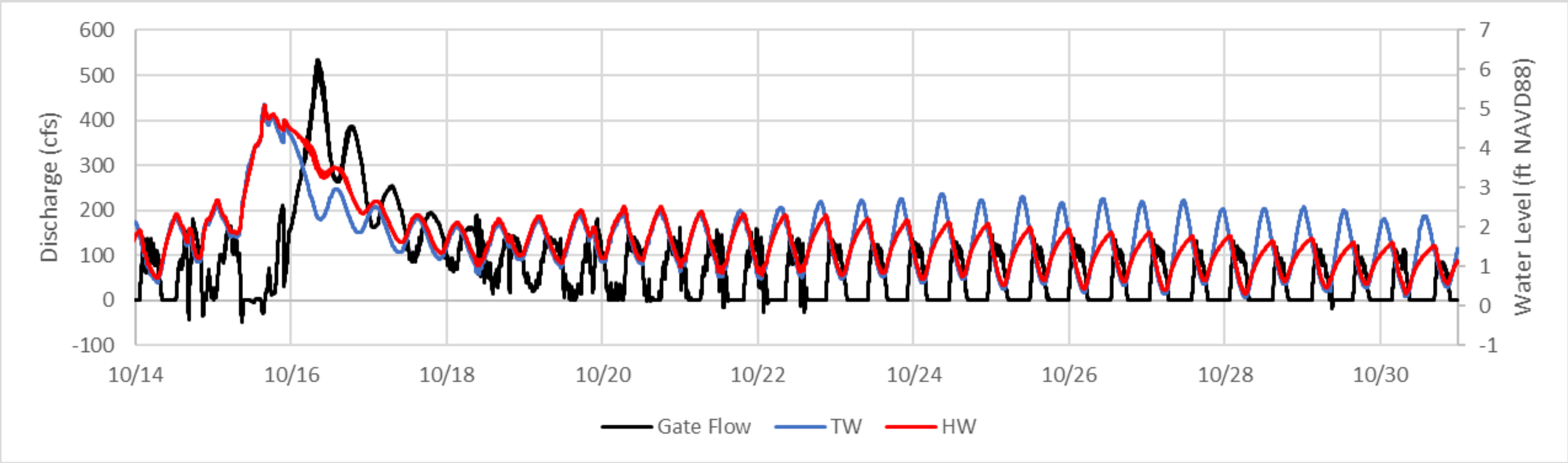

Figure B 4-3. Instantaneous Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR2) Event

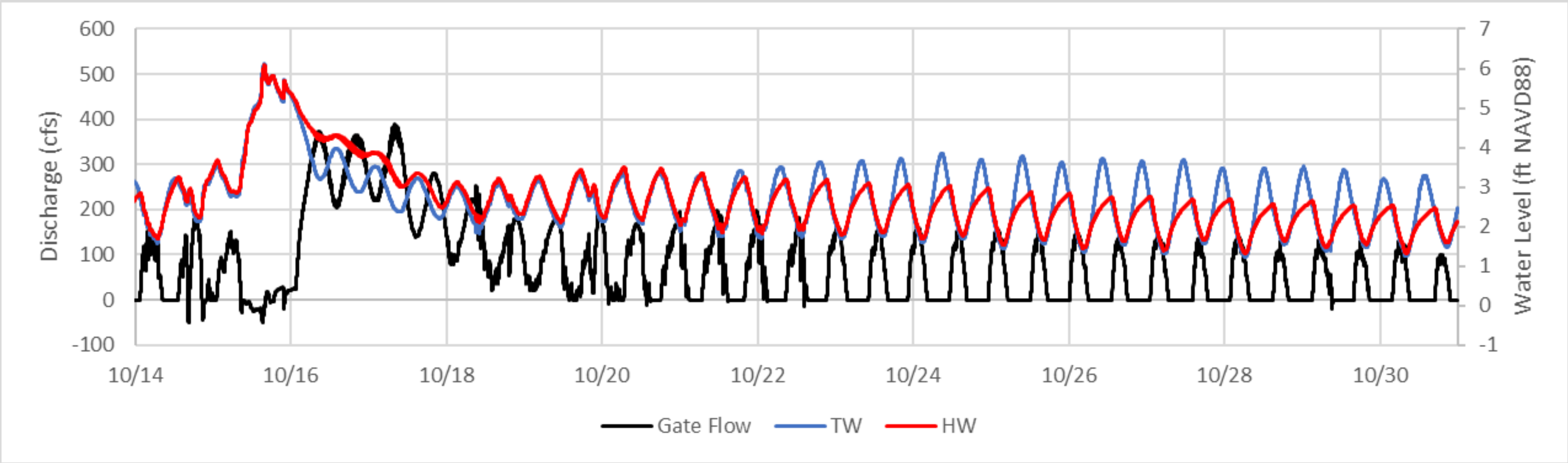

Figure B 4-4. Instantaneous Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR3) Event

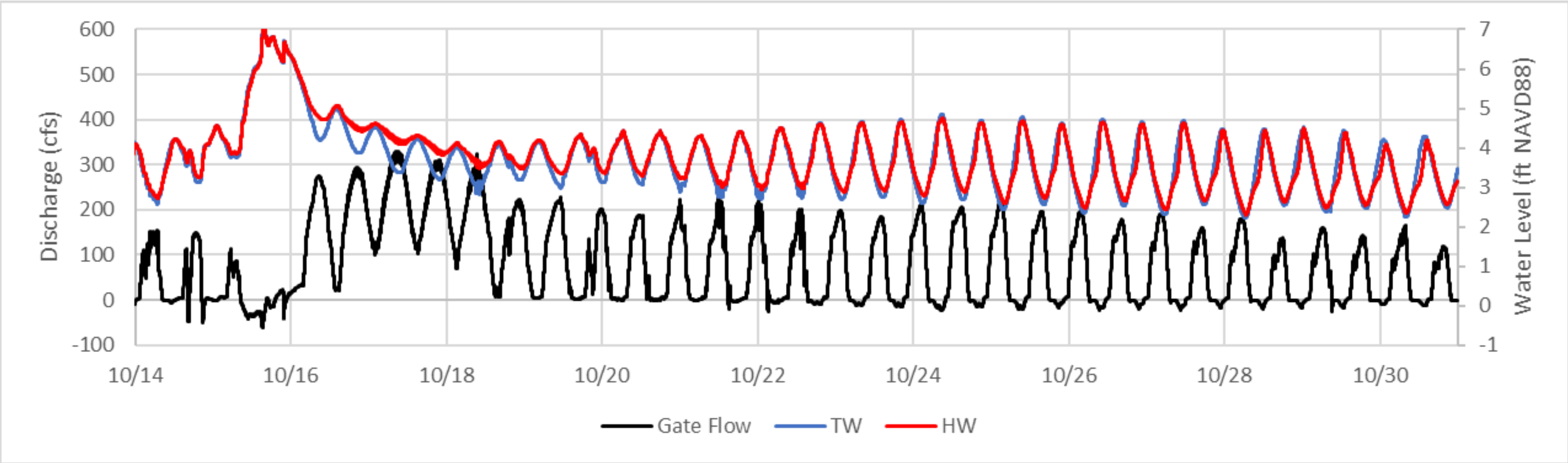

Figure B 4-5. 12-Hour Moving Average Discharge and Stages at S25 for the 100-year 3-day Current Conditions Event

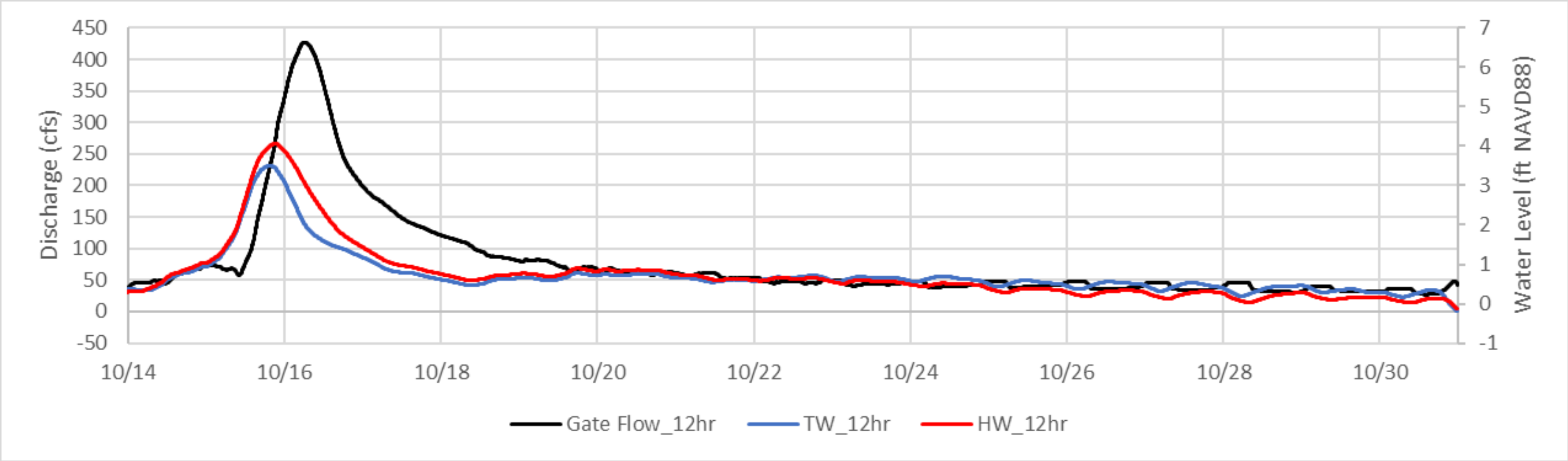

Figure B 4-6. 12-Hour Moving Average Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR1) Event

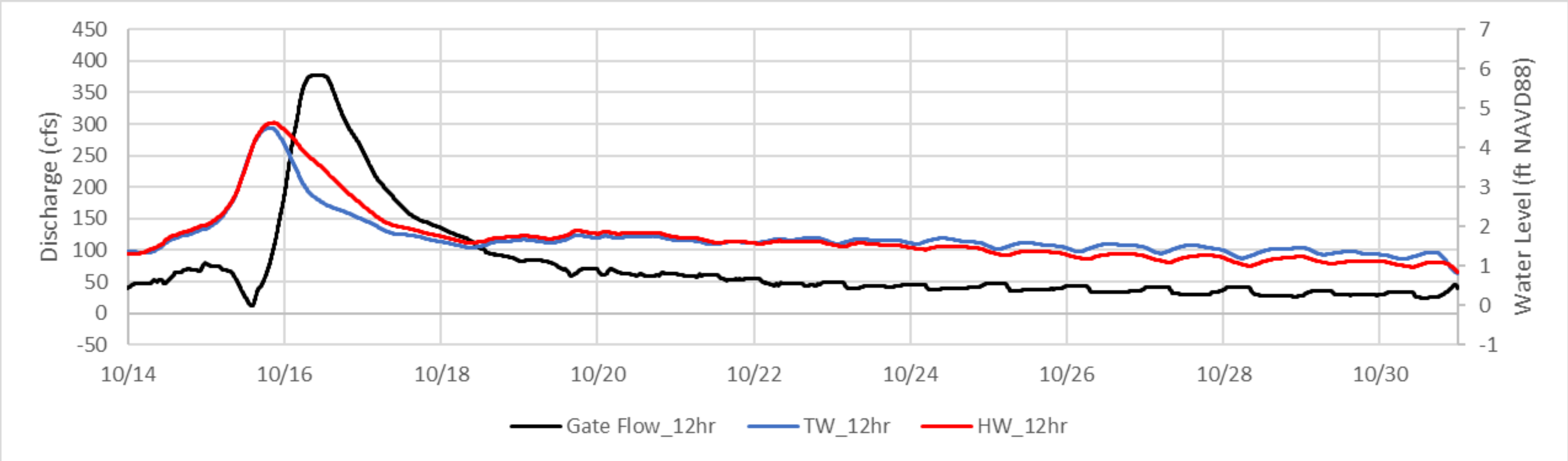

Figure B 4-7. 12-Hour Moving Average Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR2) Event

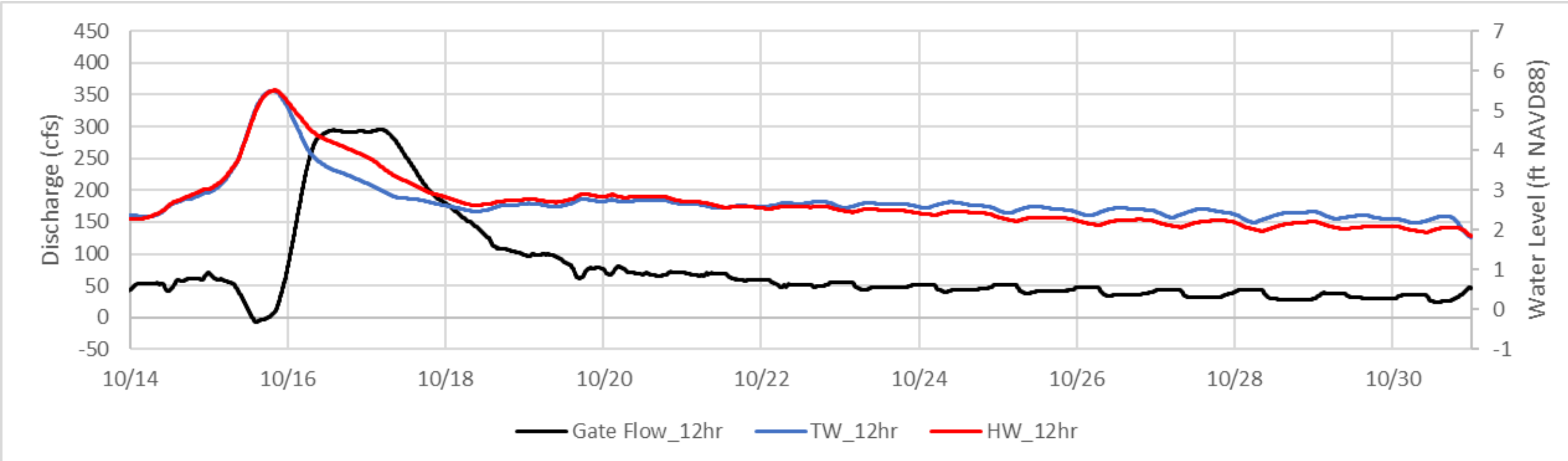

Figure B 4-8. 12-Hour Moving Average Discharge and Stages at S25 for the 100-year 3-day Future Conditions (SLR3) Event

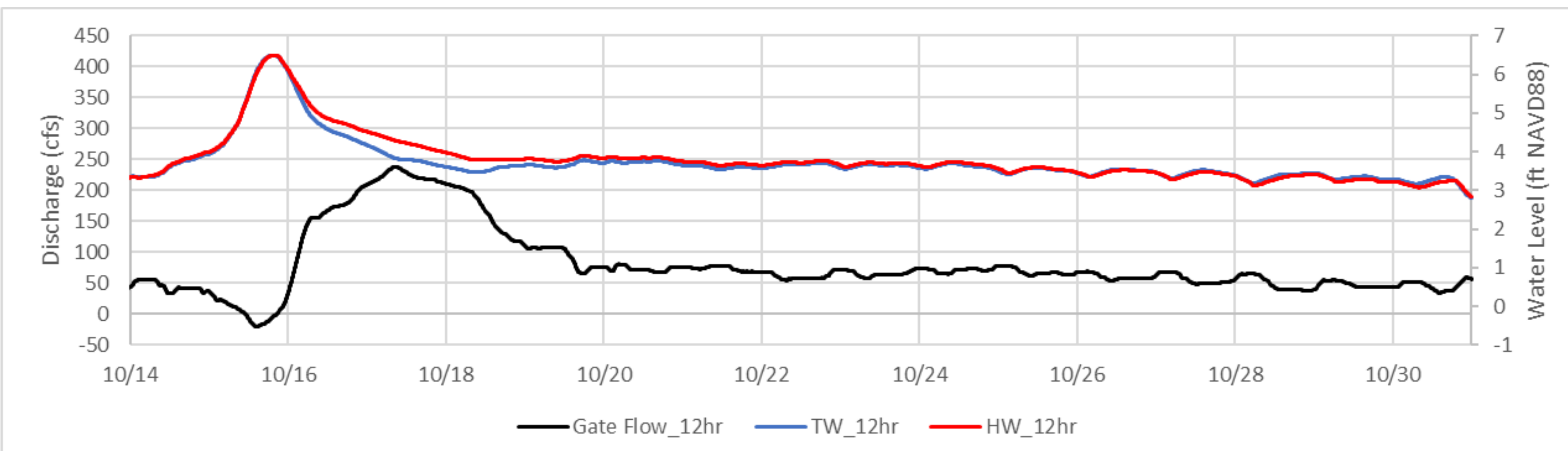

Figure B 4-9. Instantaneous Discharge and Stages at S25 for the 25-year 3-day Current Conditions Event

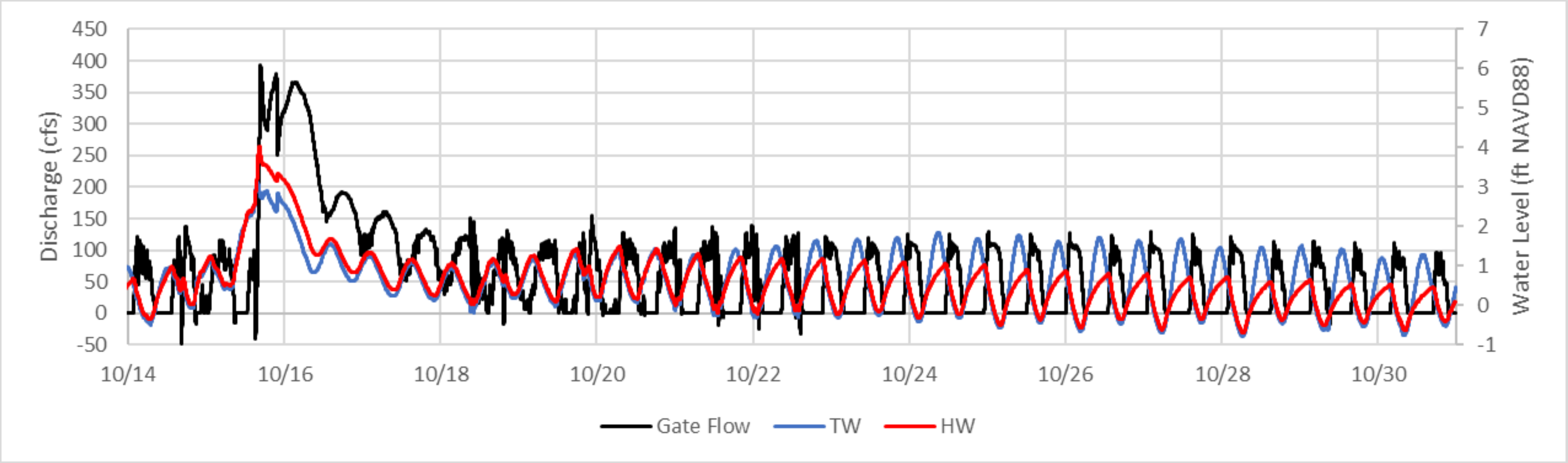

Figure B 4-10. Instantaneous Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR1) Event

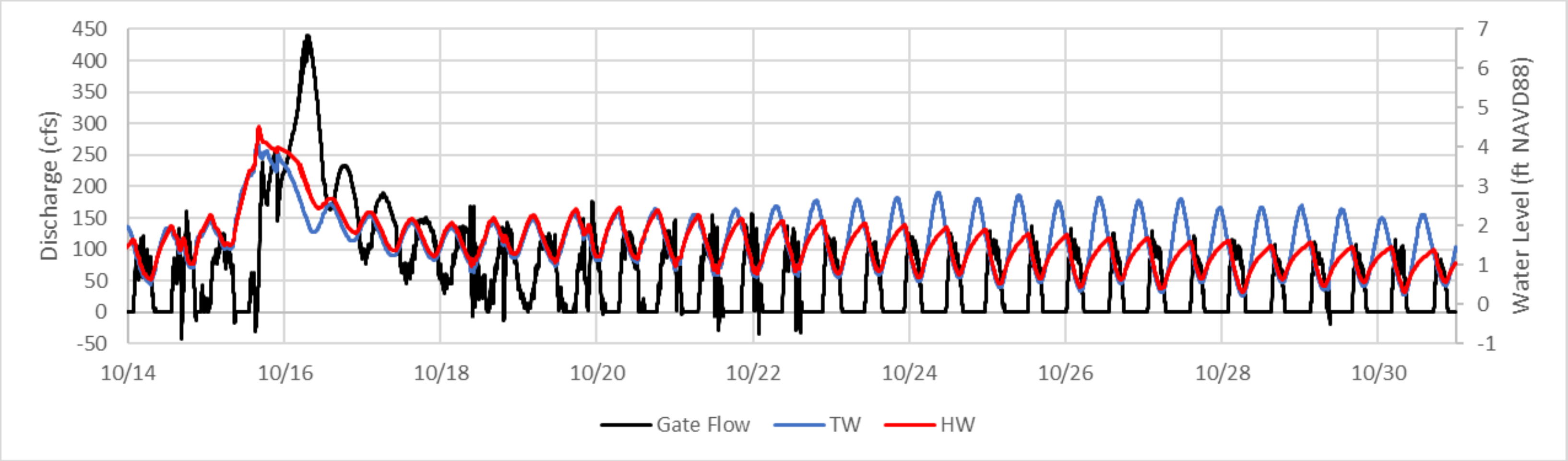

Figure B 4-11. Instantaneous Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR2) Event

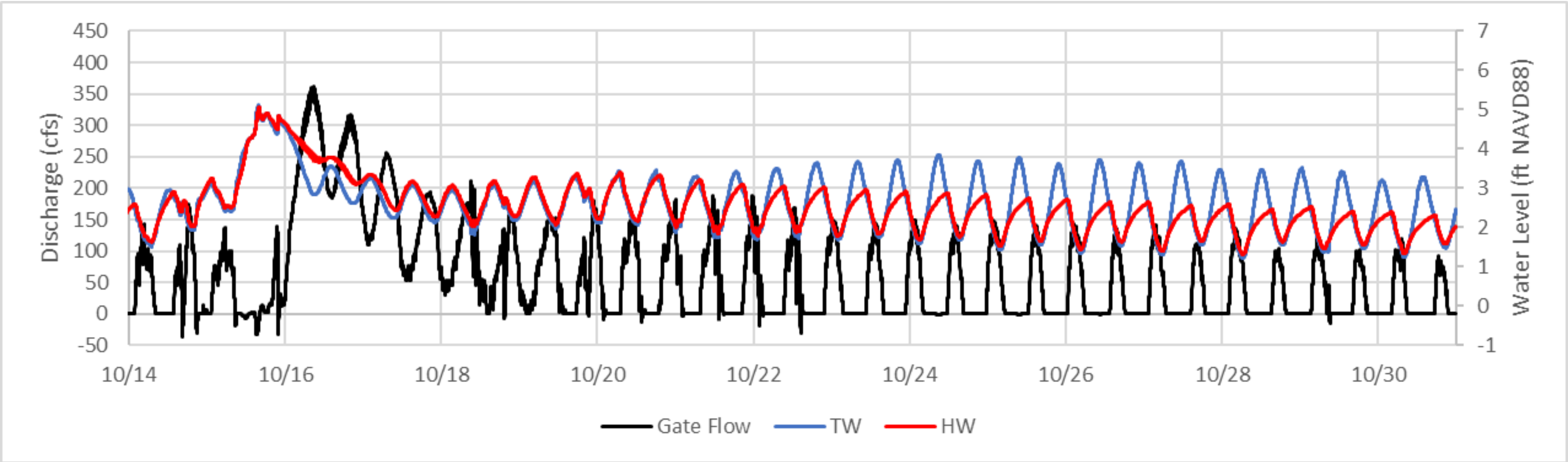

Figure B 4-12. Instantaneous Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR3) Event

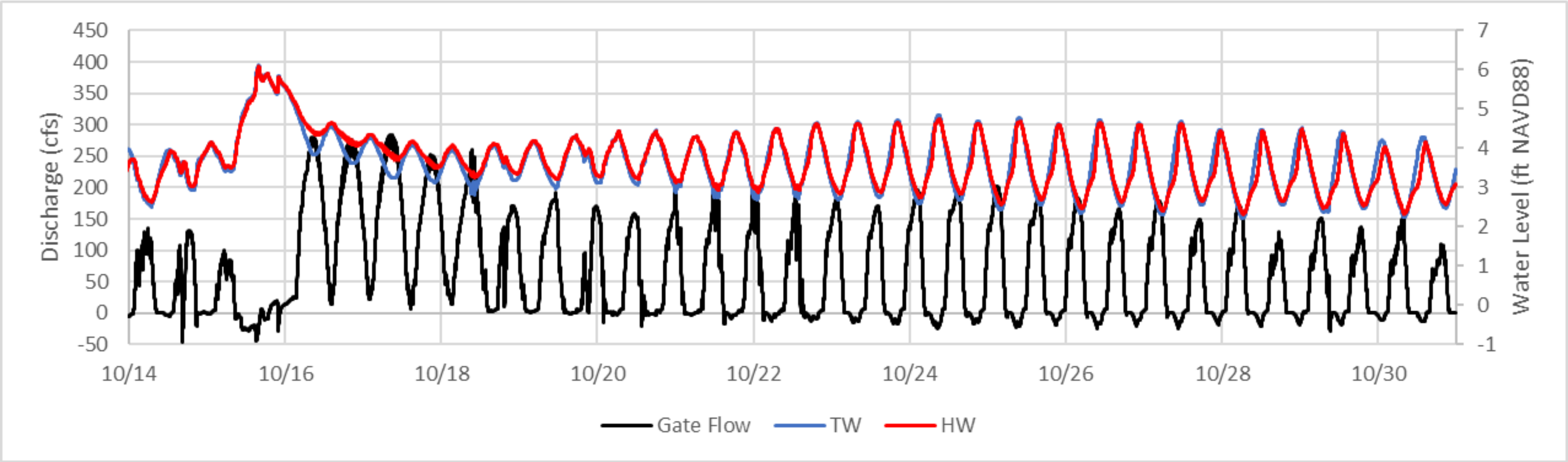

Figure B 4-13. 12-Hour Moving Average Discharge and Stages at S25 for the 25-year 3-day Current Conditions Event

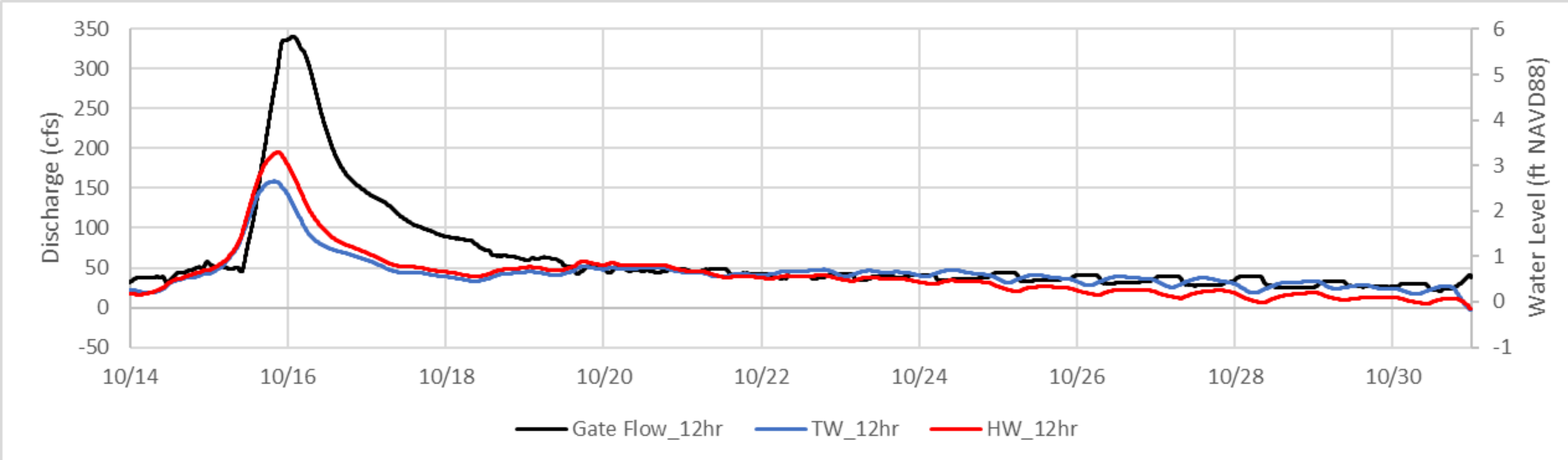

Figure B 4-14. 12-Hour Moving Average Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR1) Event

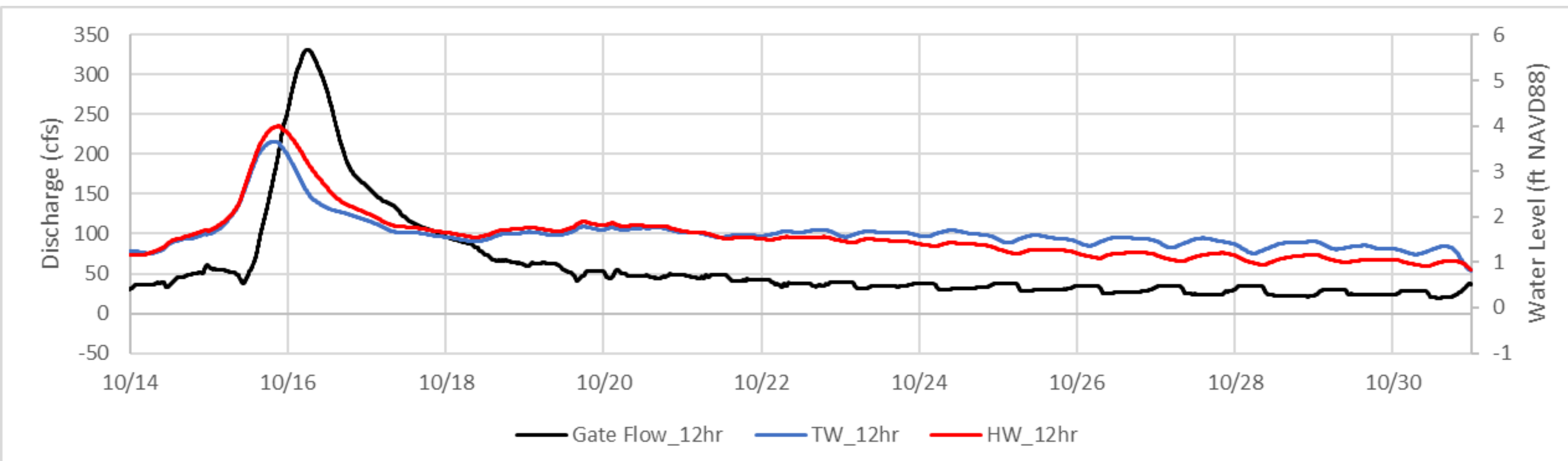

Figure B 4-15. 12-Hour Moving Average Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR2) Event

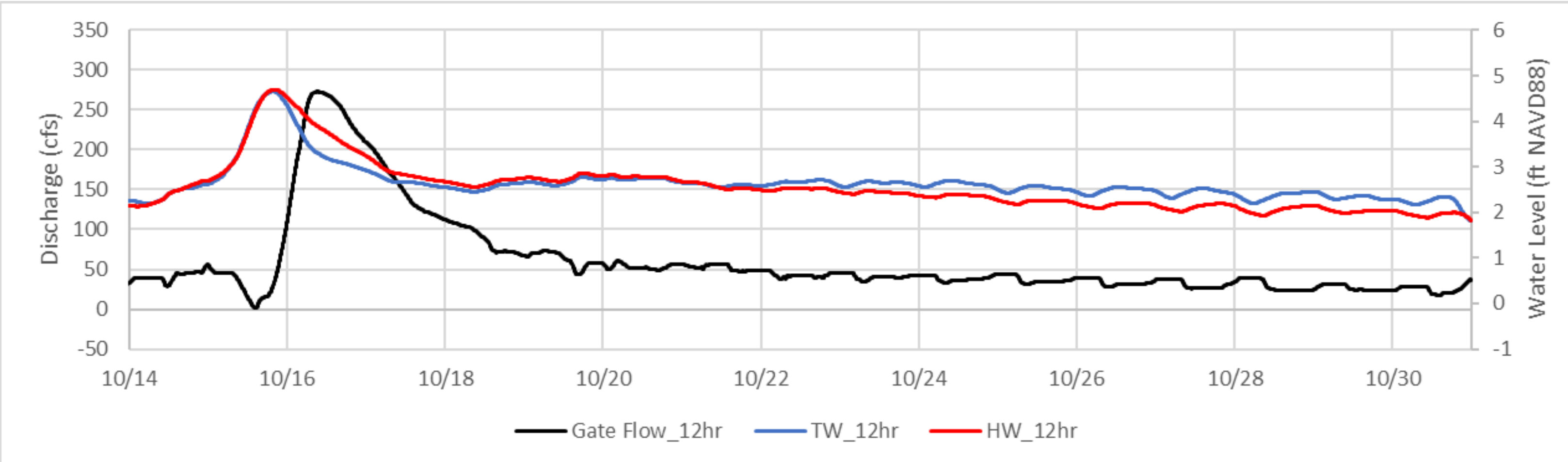

Figure B 4-16. 12-Hour Moving Average Discharge and Stages at S25 for the 25-year 3-day Future Conditions (SLR3) Event

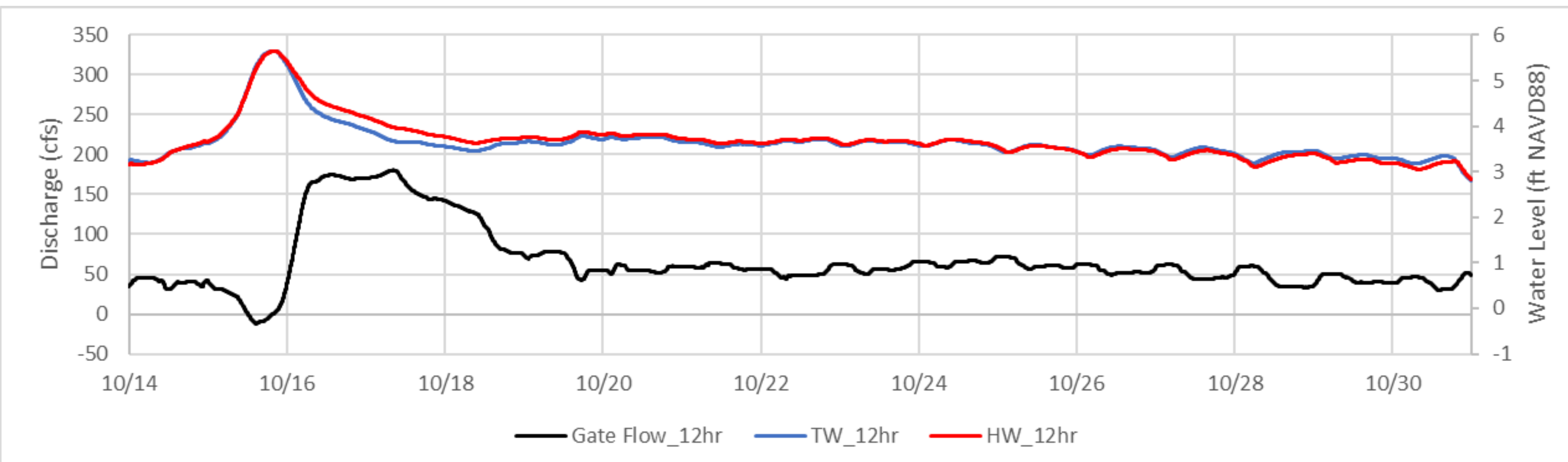

Figure B 4-17. Instantaneous Discharge and Stages at S25 for the 10-year 3-day Current Conditions Event

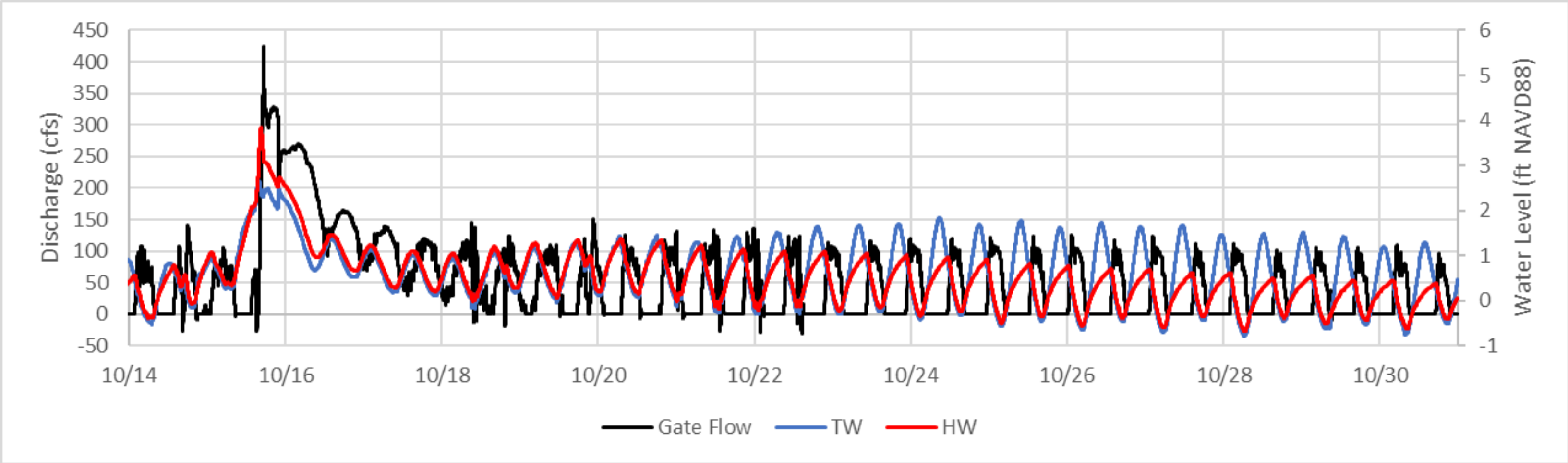

Figure B 4-18. Instantaneous Discharge and Stages at S25 for the 10-year 3-day Future Conditions (SLR1) Event

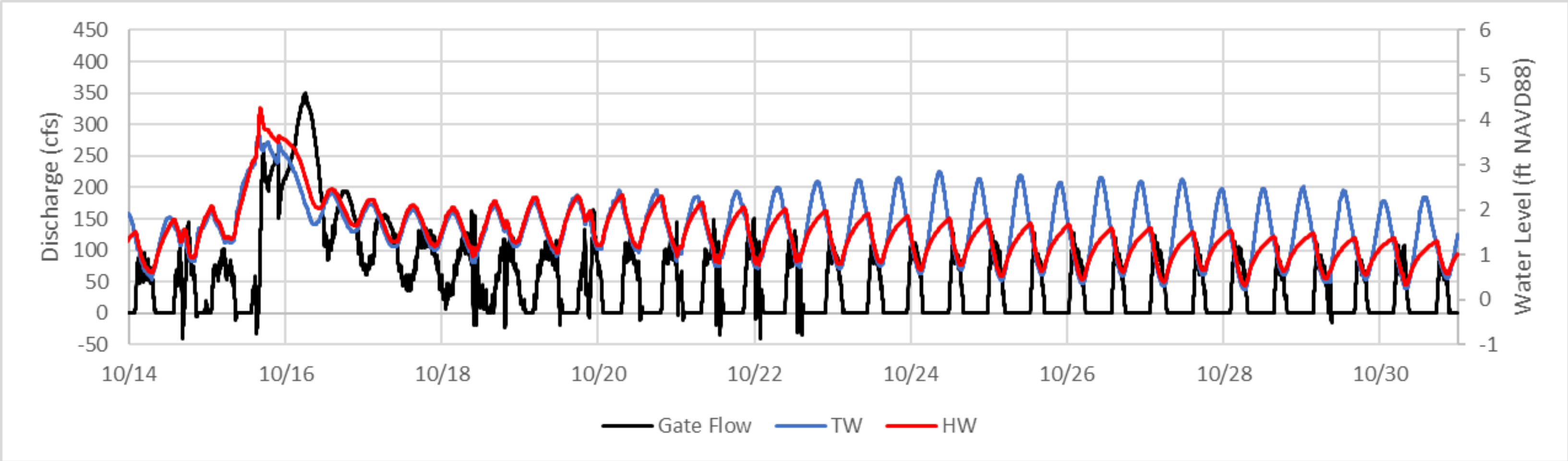

Figure B 4-19. Instantaneous Discharge and Stages at S25 for the 10-year 3-day Future Conditions (SLR2) Event

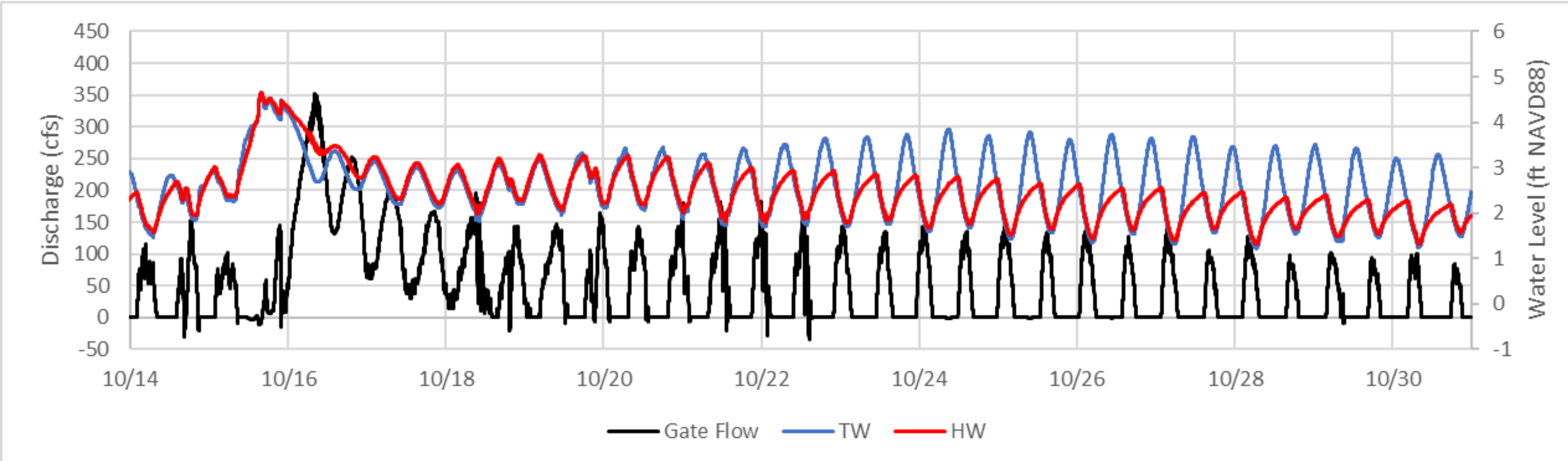

Figure B 4-20. Instantaneous Discharge and Stages at S25 for the 10-year 3-day Future Conditions (SLR3) Event

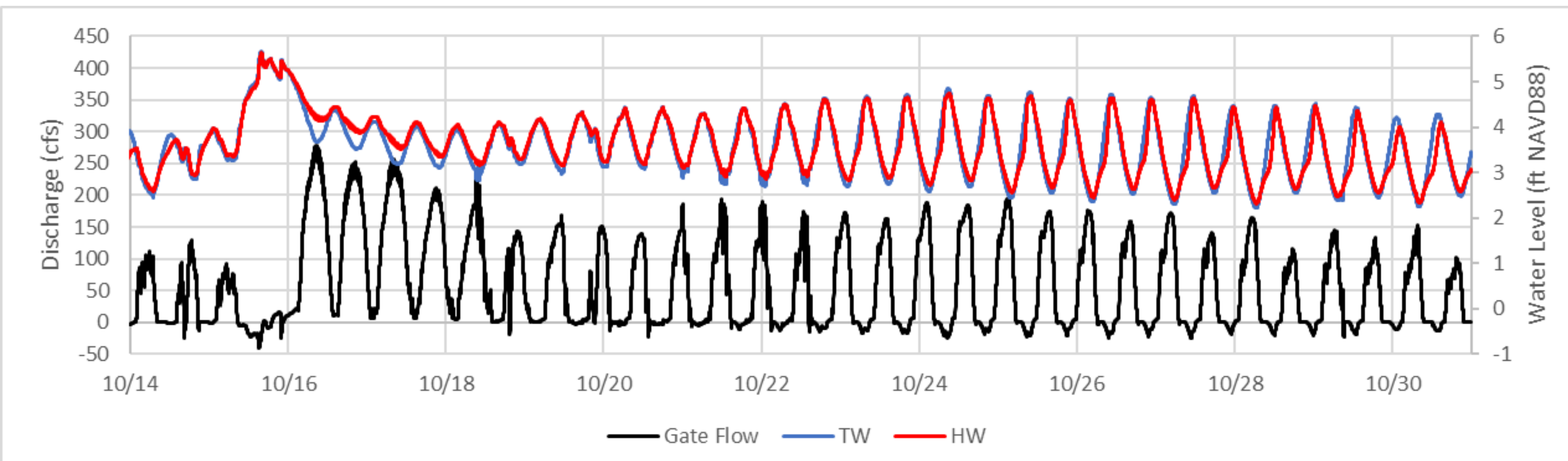

Figure B 4-21. 12-Hour Moving Average Discharge and Stages at S25 for the 10-year 3-day Current Conditions Event

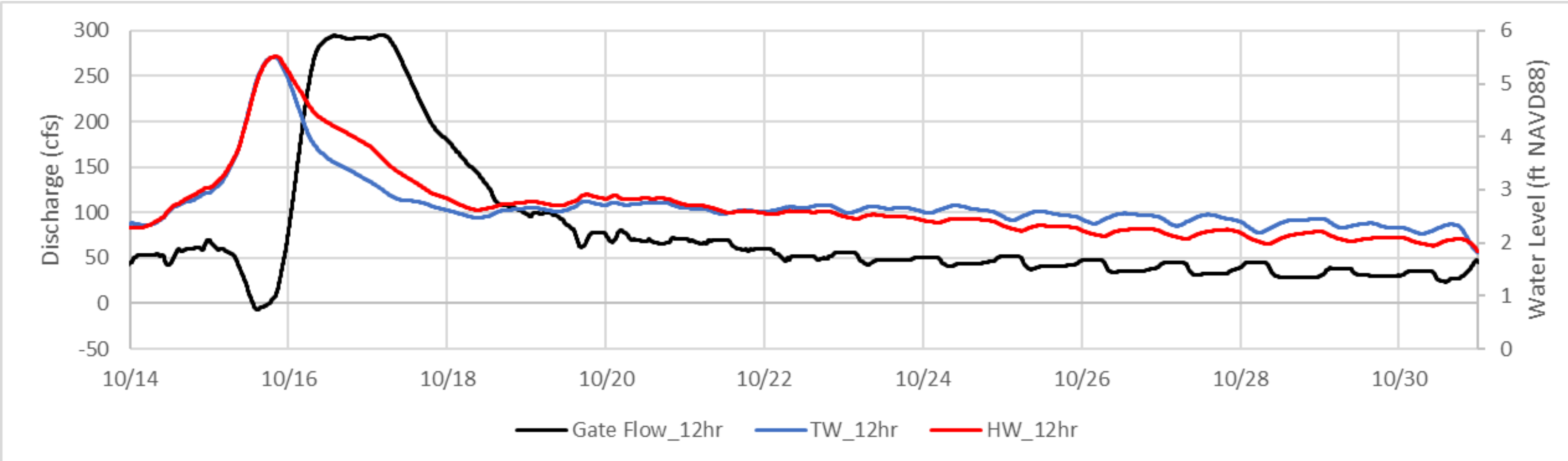

Figure B 4-22. 12-Hour Moving Average Discharge and Stages at S25 for the 10-year 3-day Future Conditions (SLR1) Event

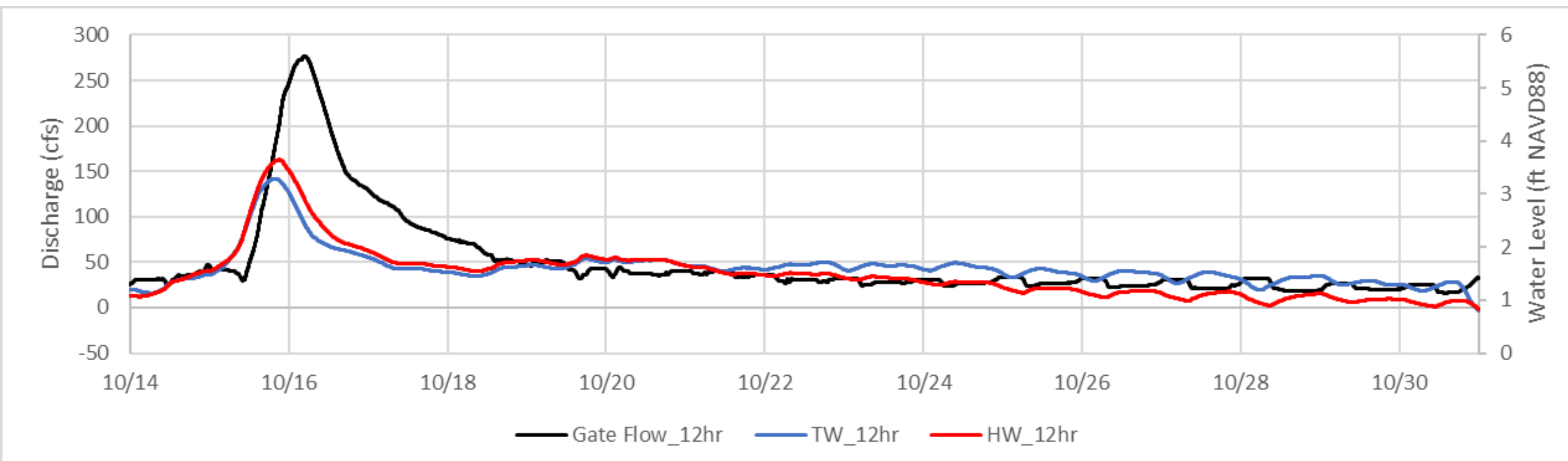

Figure B 4-23. 12-Hour Moving Average Discharge and Stages at S25 for the 10-year 3-day Future Conditions (SLR2) Event

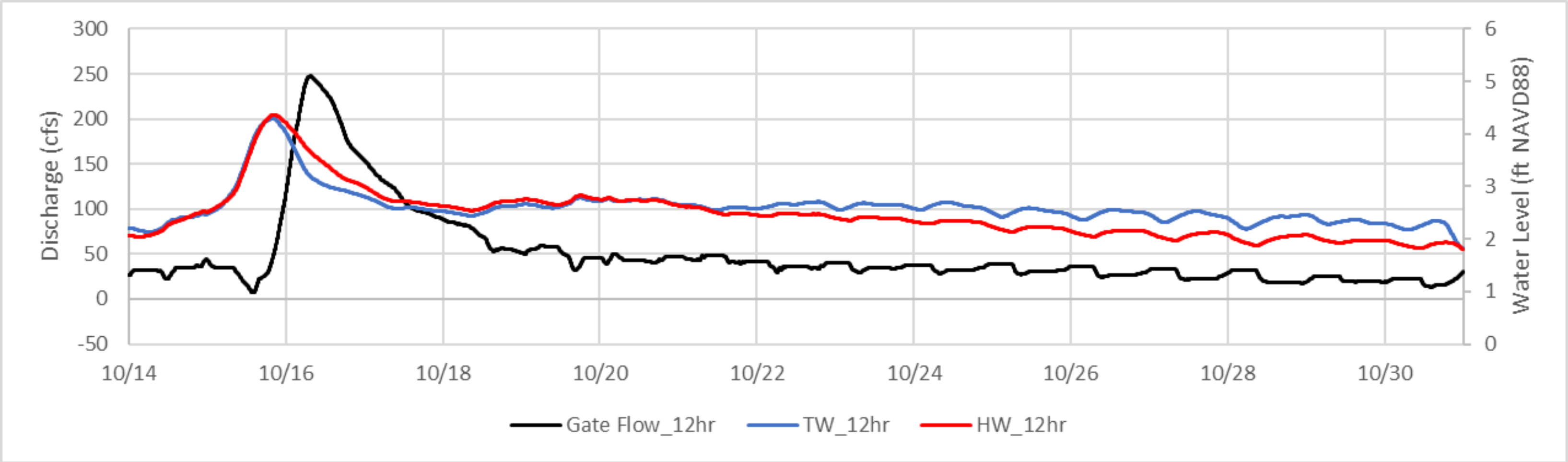

Figure B 4-24. 12-Hour Moving Average Discharge and Stages at S25 for the 10-year 3-day Future Conditions (SLR3) Event

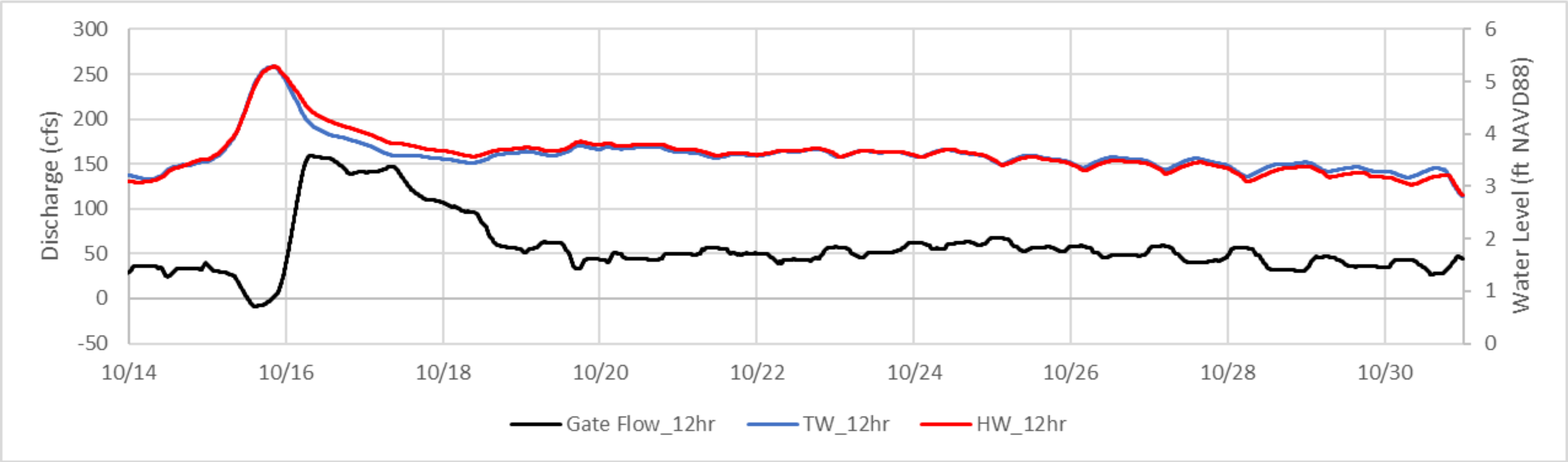

Figure B 4-25. Instantaneous Discharge and Stages at S25 for the 5-year 3-day Current Conditions Event

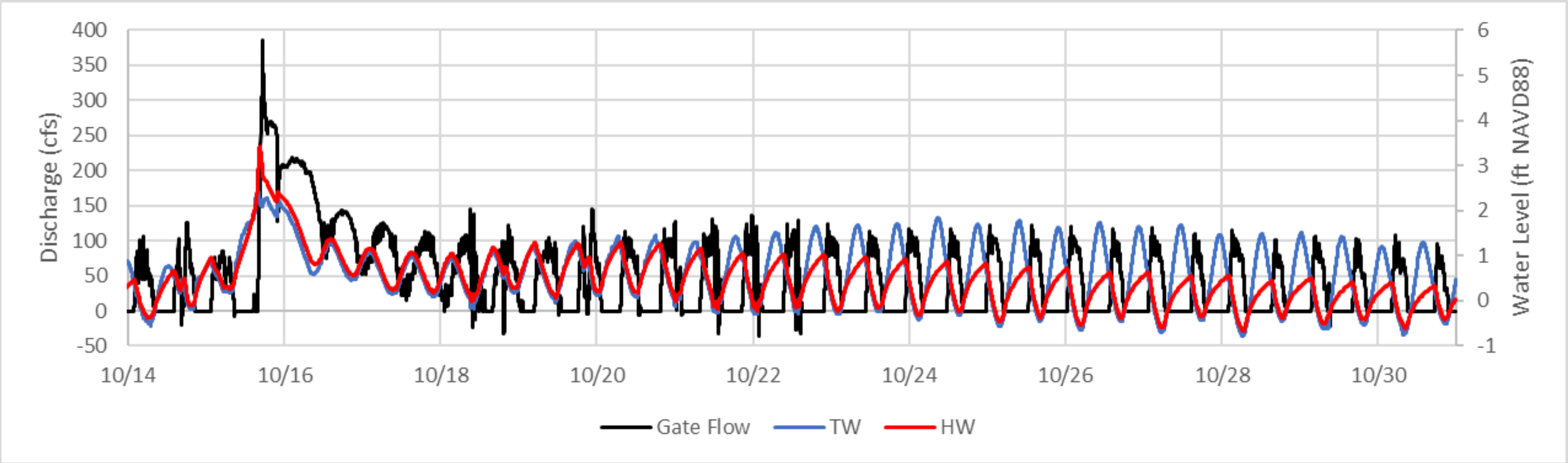

Figure B 4-26. Instantaneous Discharge and Stages at S25 for the 5-year 3-day Future Conditions (SLR1) Event

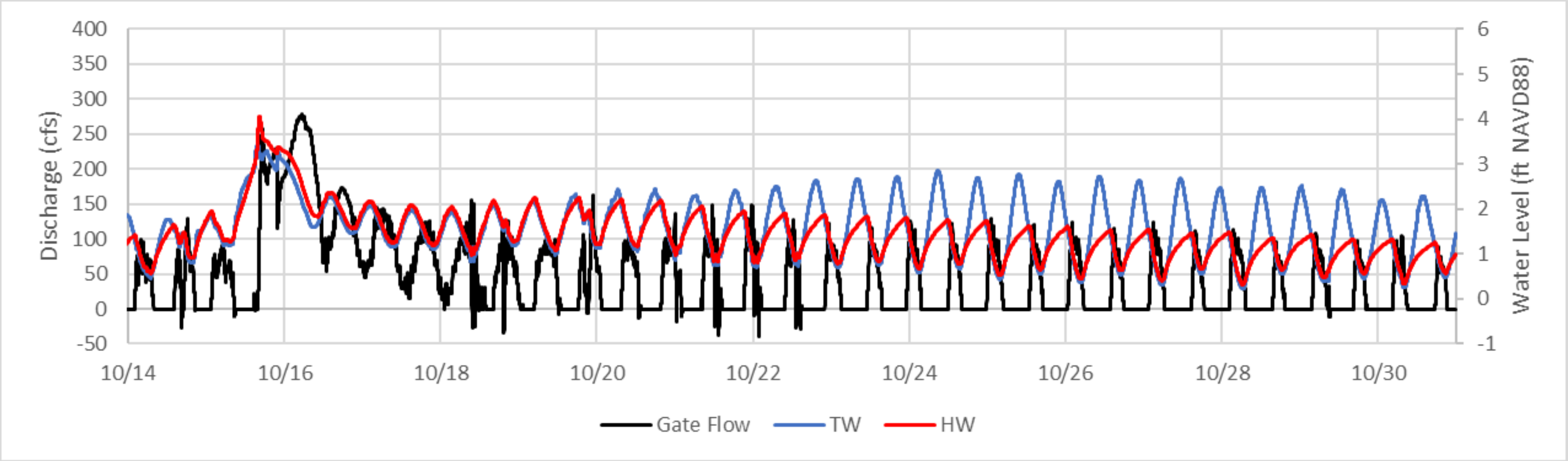

Figure B 4-27. Instantaneous Discharge and Stages at S25 for the 5-year 3-day Future Conditions (SLR2) Event

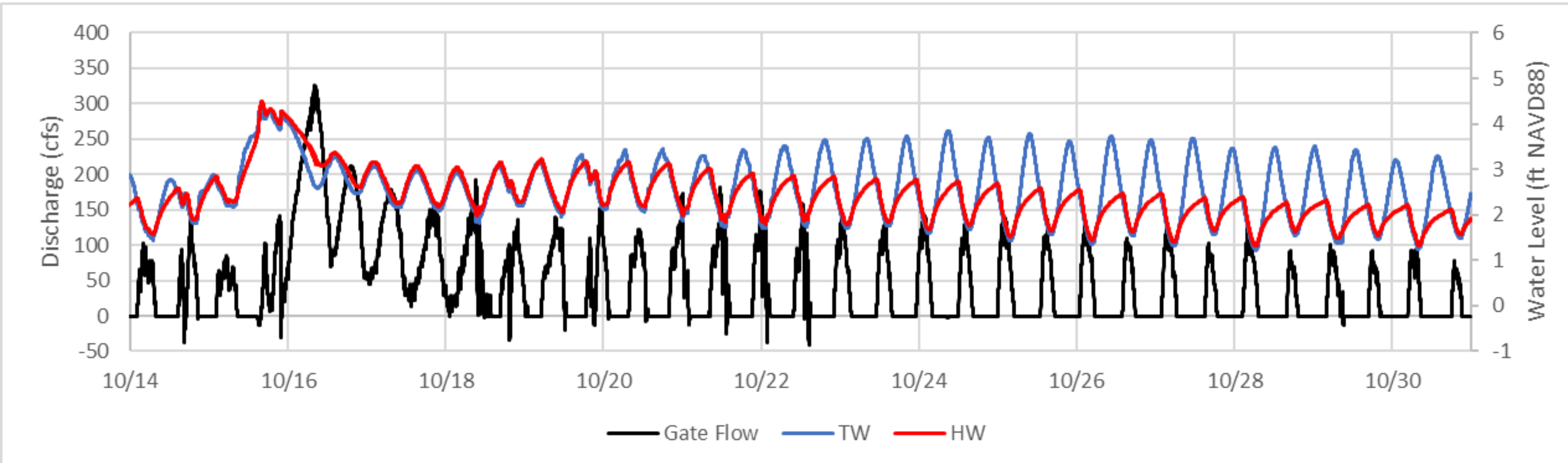

Figure B 4-28. Instantaneous Discharge and Stages at S25 for the 5-year 3-day Future Conditions (SLR3) Event

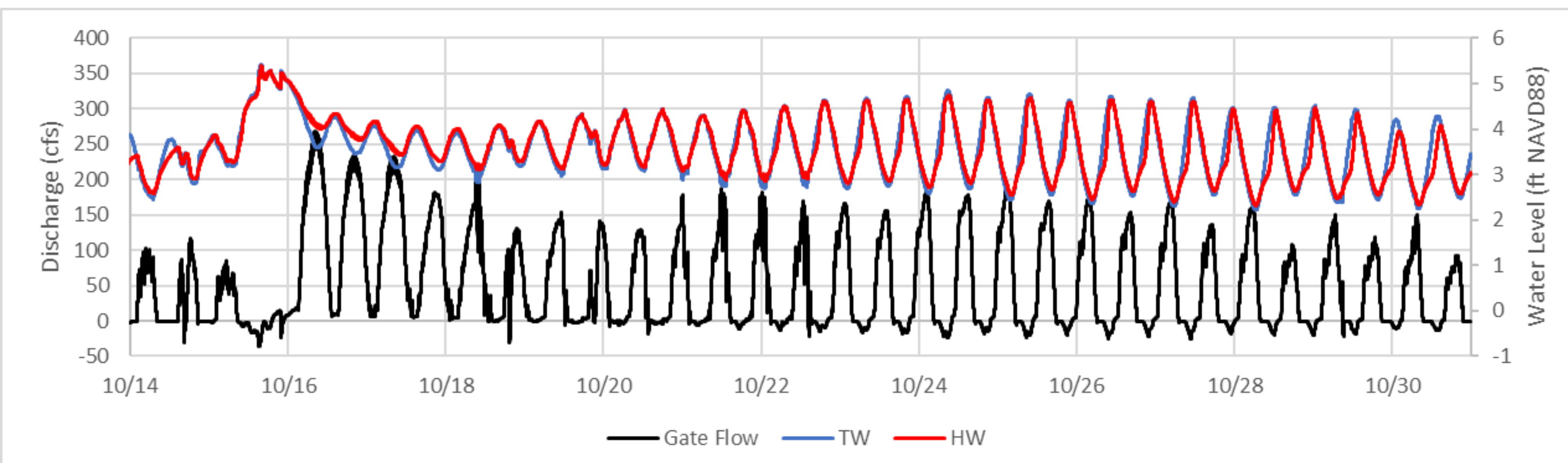

Figure B 4-29. 12-Hour Moving Average Discharge and Stages at S25 for the 5-year 3-day Current Conditions Event

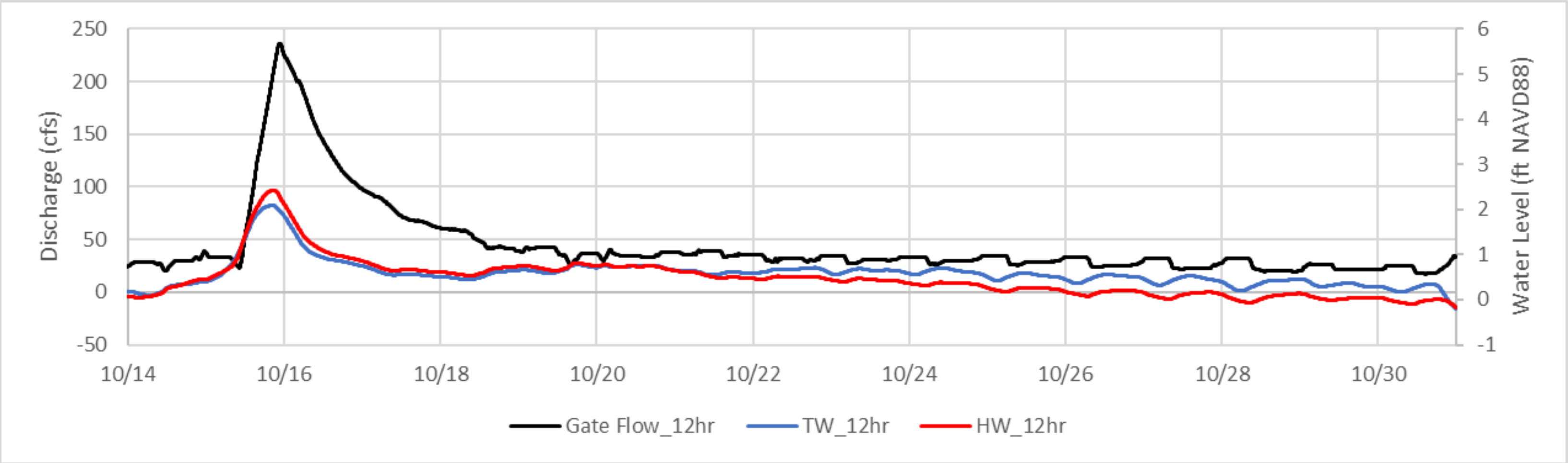

Figure B 4-30. 12-Hour Moving Average Discharge and Stages at S25 for the 5-year 3-day Future Conditions (SLR1) Event

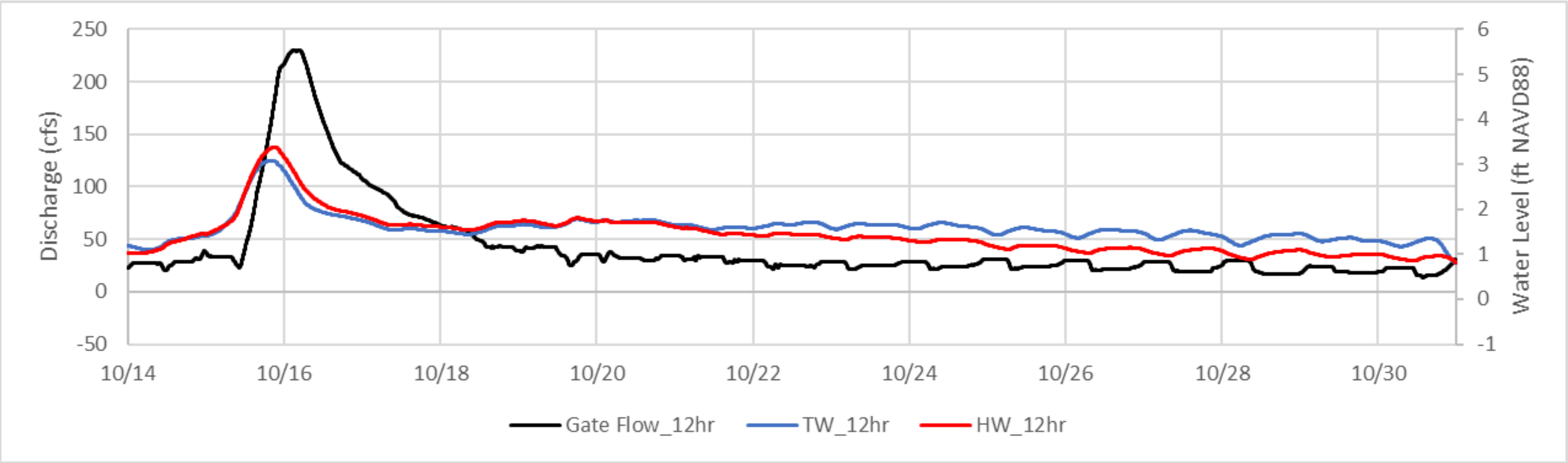

Figure B 4-31. 12-Hour Moving Average Discharge and Stages at S25 for the 5-year 3-day Future Conditions (SLR2) Event

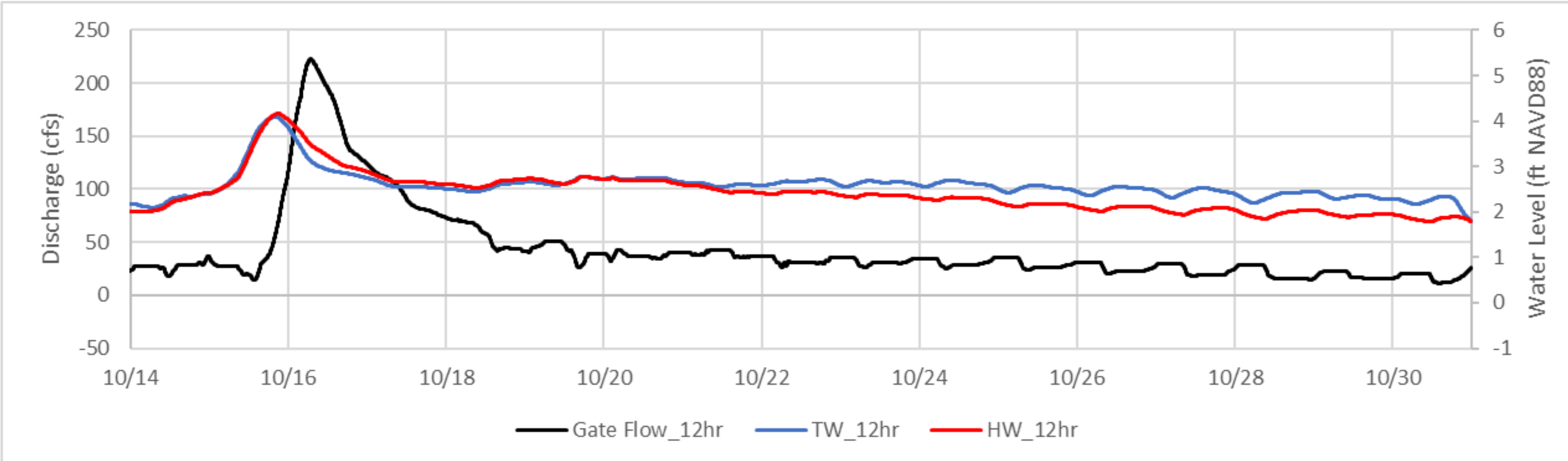

Figure B 4-32. 12-Hour Moving Average Discharge and Stages at S25 for the 5-year 3-day Future Conditions (SLR3) Event

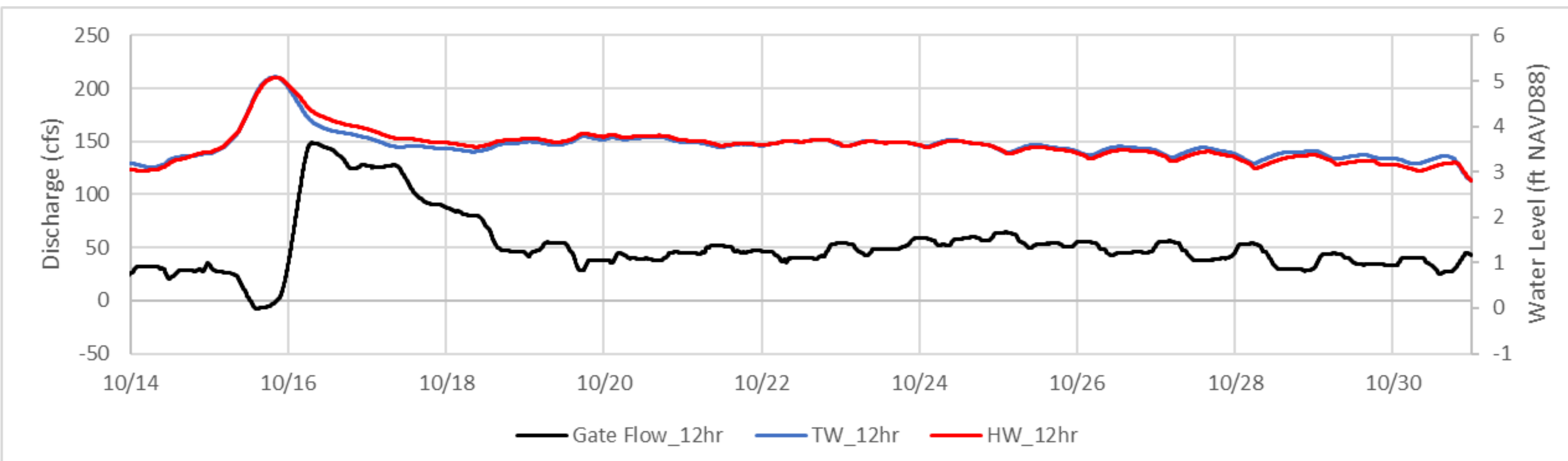

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Figure B 5-1 through **Figure B 5-32** show the instantaneous and 12-hour average discharge, HW, and TW at S26 for each SLR condition for the 100-, 25-, 10-, and 5-year design storm events.

Figure B 5-1. Instantaneous Discharge and Stages at S26 for the 100-year 3-day Current Conditions Event

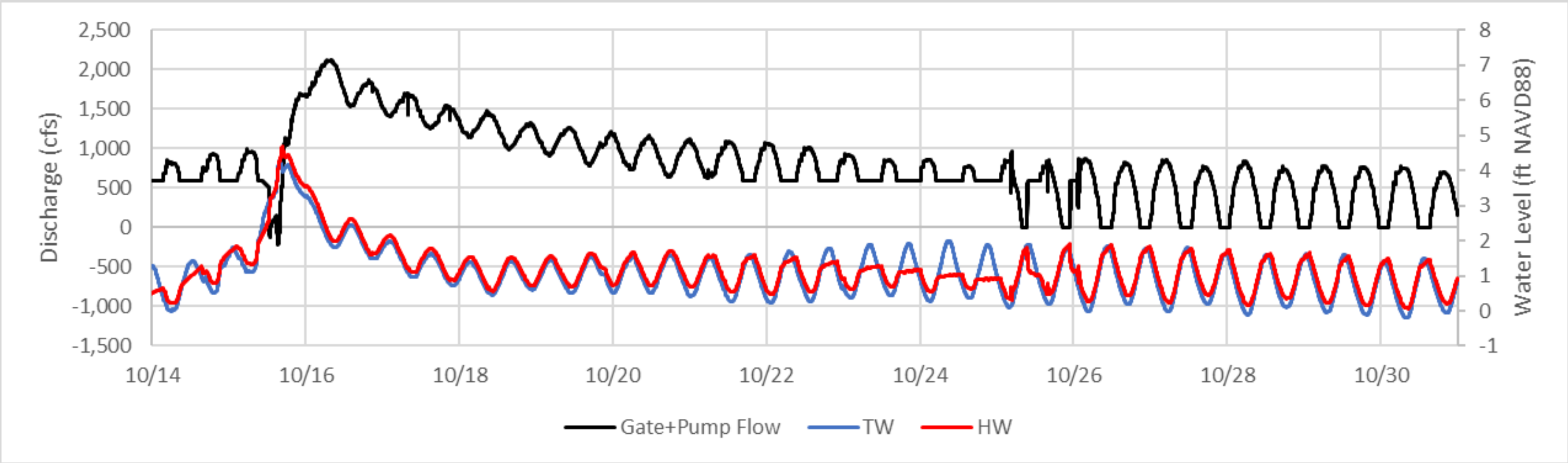


Figure B 5-2. Instantaneous Discharge and Stages at S26 for the 100-year 3-day Future Conditions (SLR1) Event

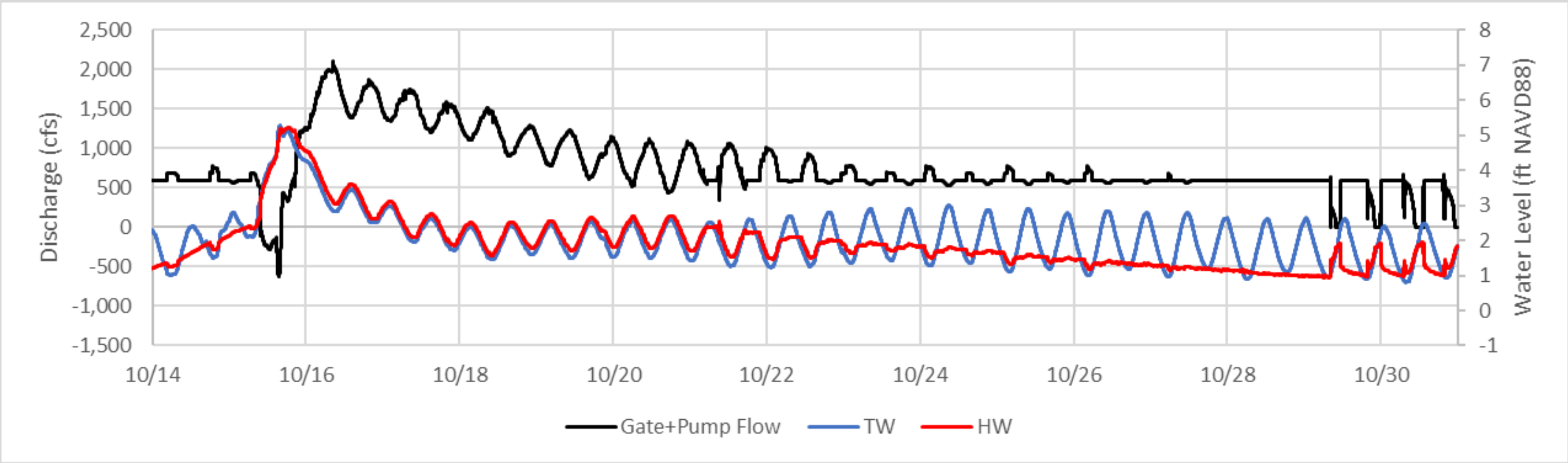


Figure B 5-3. Instantaneous Discharge and Stages at S26 for the 100-year 3-day Future Conditions (SLR2) Event

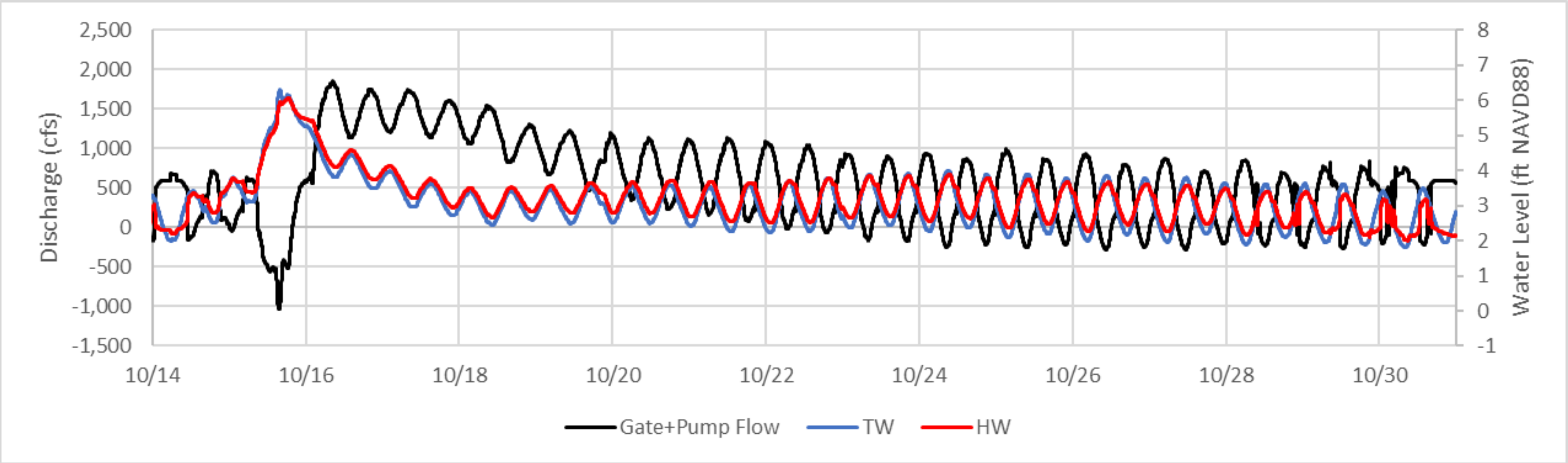


Figure B 5-4. Instantaneous Discharge and Stages at S26 for the 100-year 3-day Future Conditions (SLR3) Event

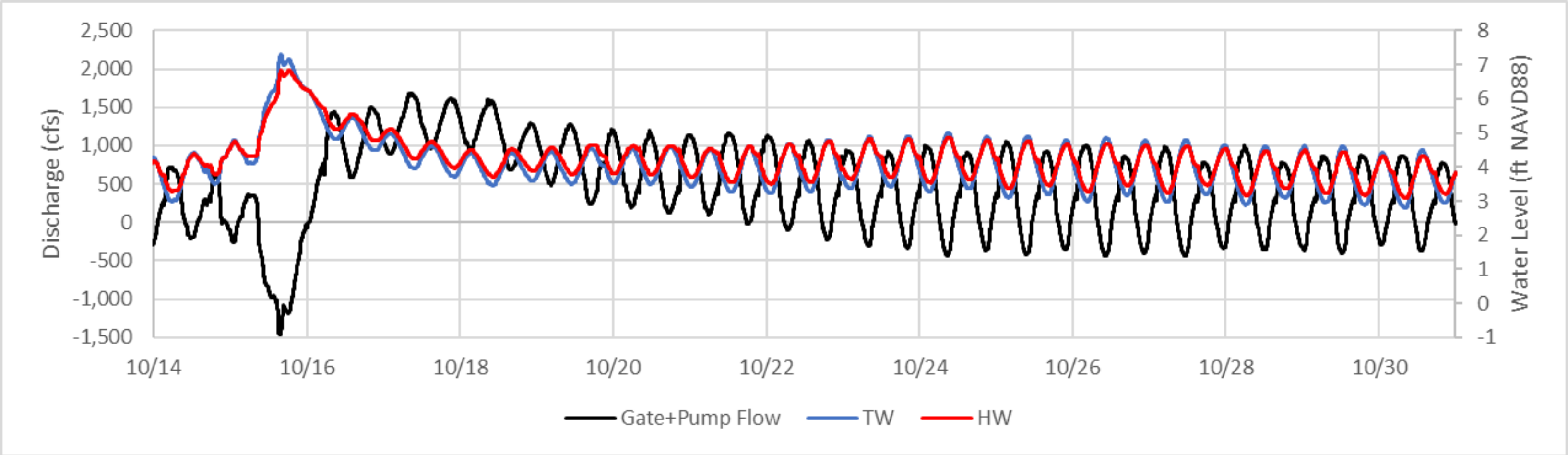


Figure B 5-5. 12-Hour Moving Average Discharge and Stages at S26 for the 100-year 3-day Current Conditions Event

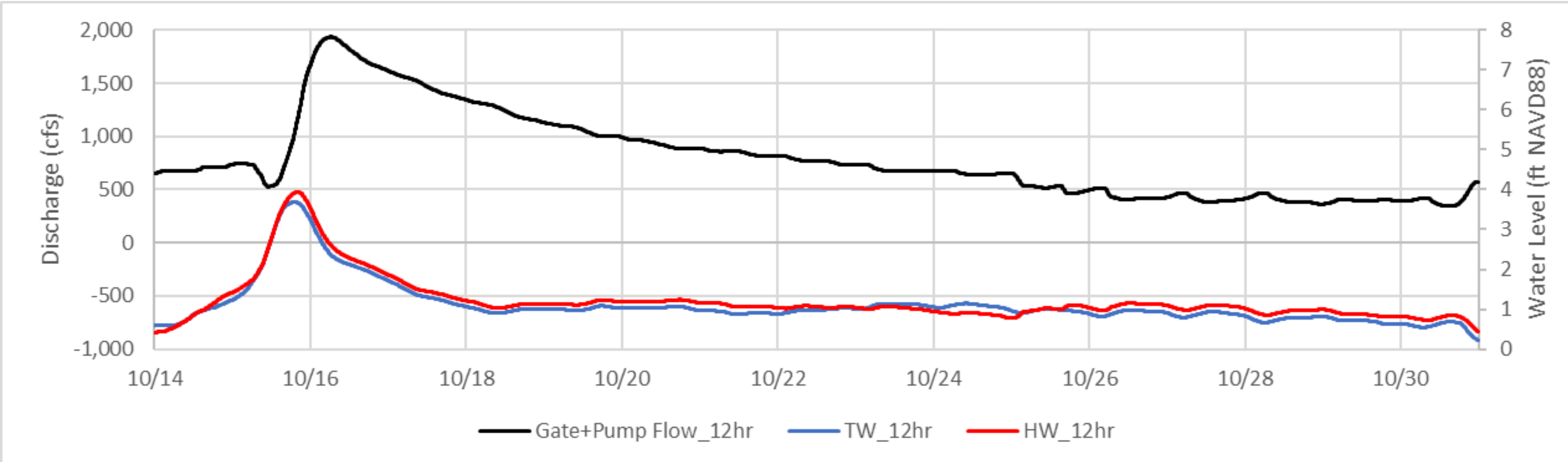


Figure B 5-6. 12-Hour Moving Average Discharge and Stages at S26 for the 100-year 3-day Future Conditions (SLR1) Event

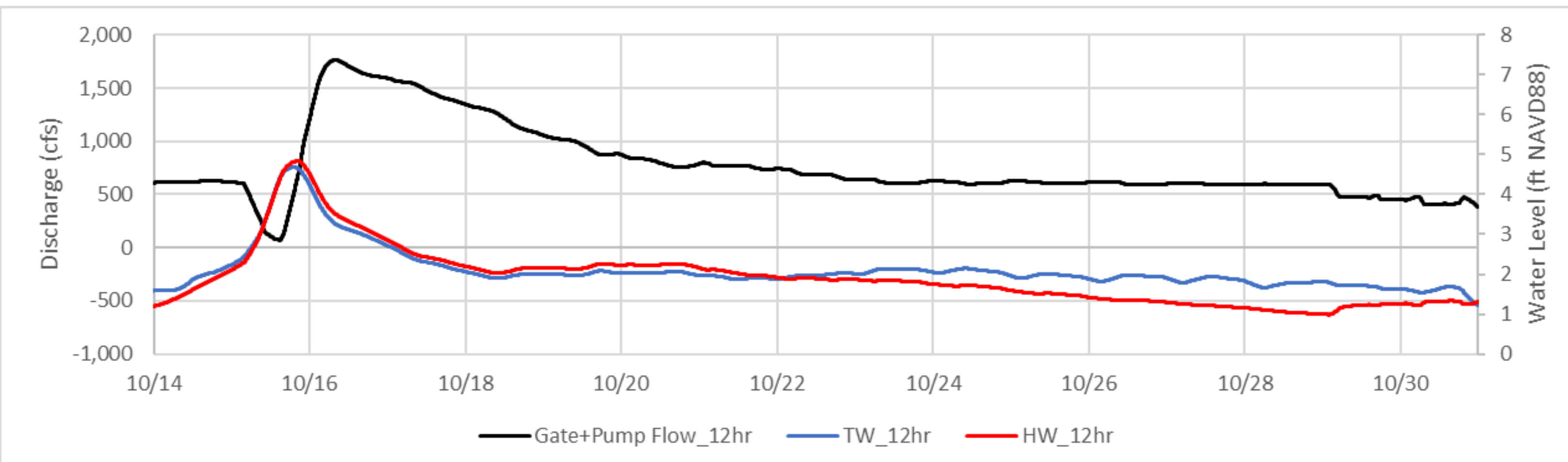


Figure B 5-7. 12-Hour Moving Average Discharge and Stages at S26 for the 100-year 3-day Future Conditions (SLR2) Event

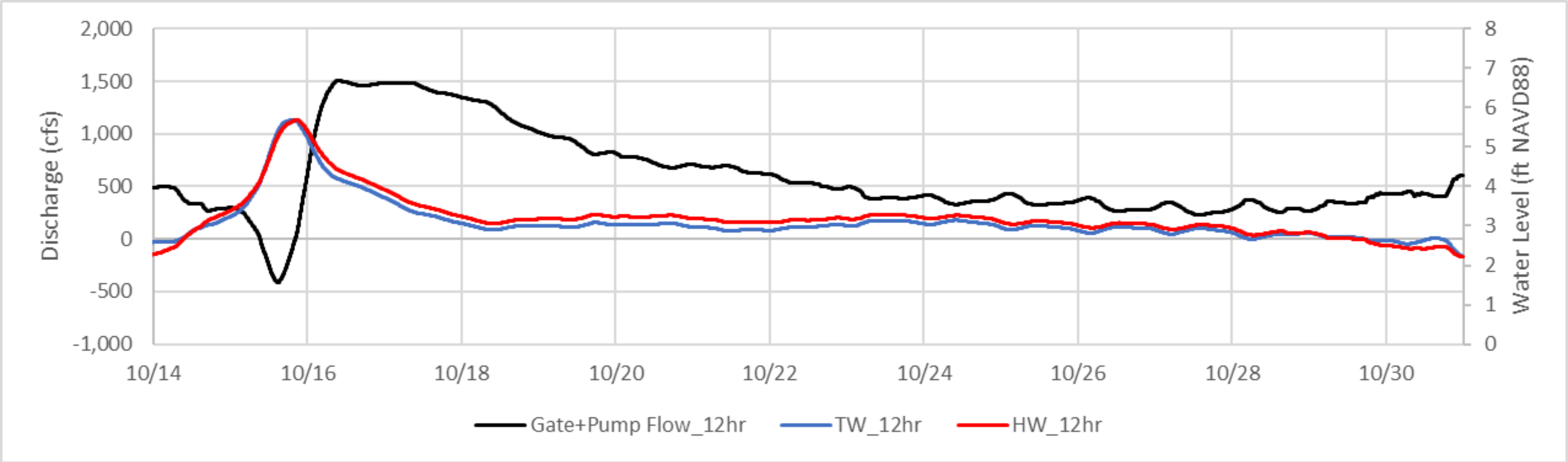


Figure B 5-8. 12-Hour Moving Average Discharge and Stages at S26 for the 100-year 3-day Future Conditions (SLR3) Event

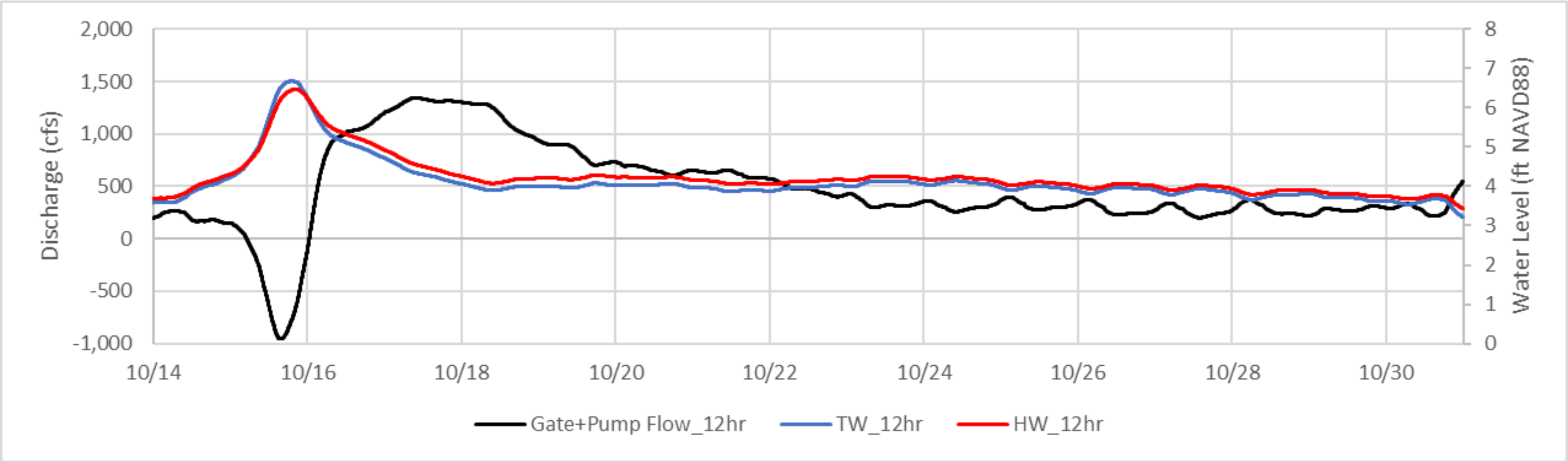


Figure B 5-9. Instantaneous Discharge and Stages at S26 for the 25-year 3-day Current Conditions Event

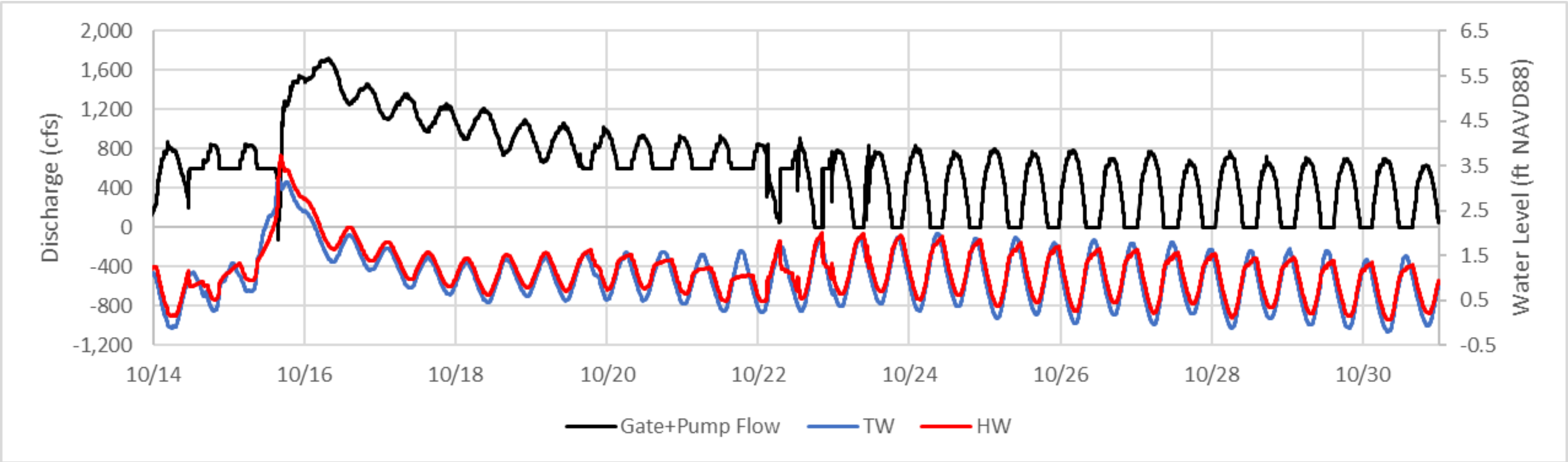


Figure B 5-10. Instantaneous Discharge and Stages at S26 for the 25-year 3-day Future Conditions (SLR1) Event

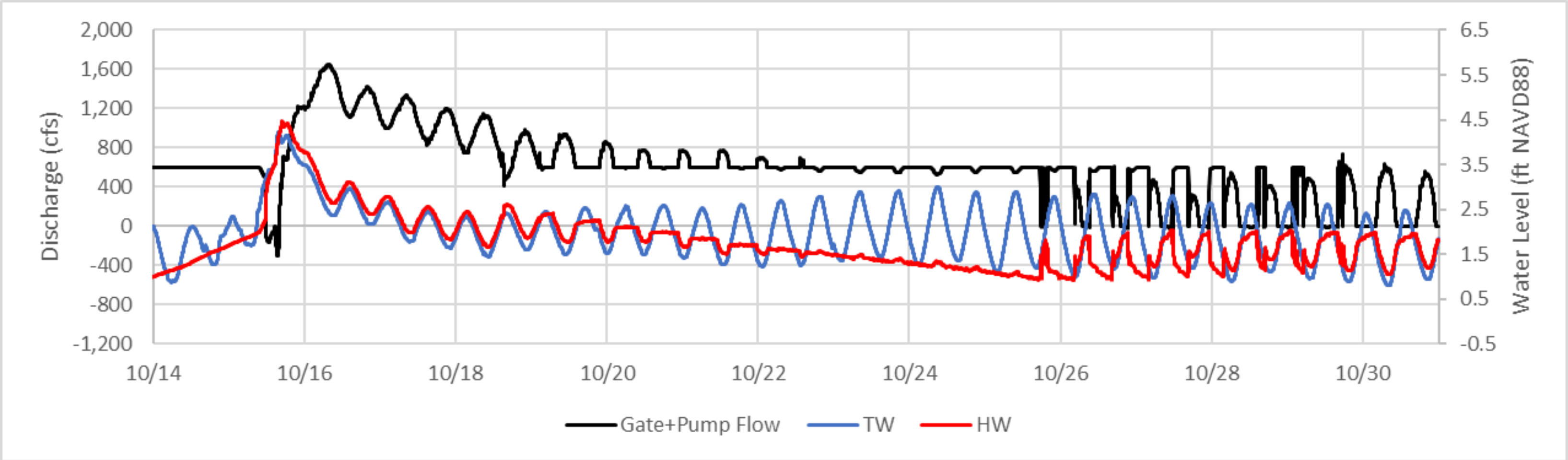


Figure B 5-11. Instantaneous Discharge and Stages at S26 for the 25-year 3-day Future Conditions (SLR2) Event

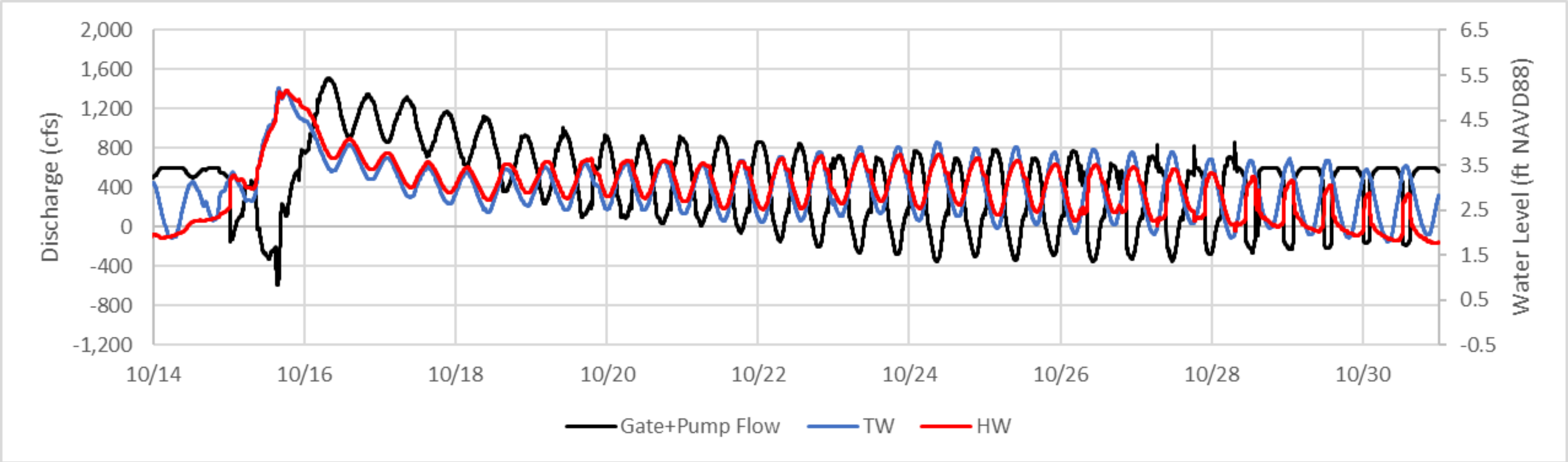


Figure B 5-12. Instantaneous Discharge and Stages at S26 for the 25-year 3-day Future Conditions (SLR3) Event

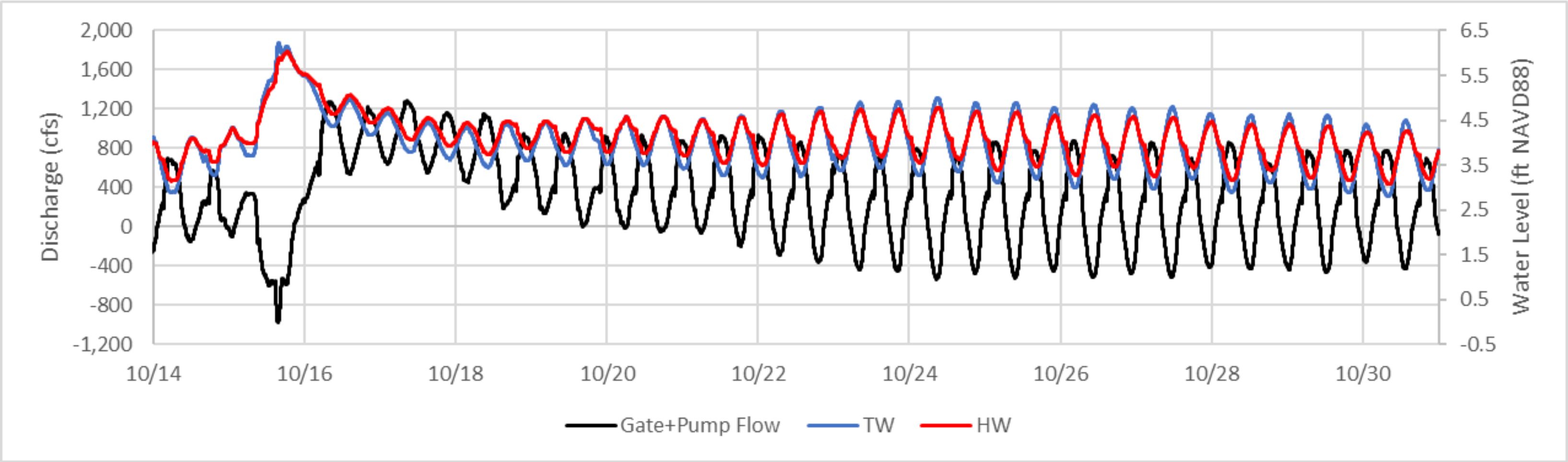


Figure B 5-13. 12-Hour Moving Average Discharge and Stages at S26 for the 25-year 3-day Current Conditions Event

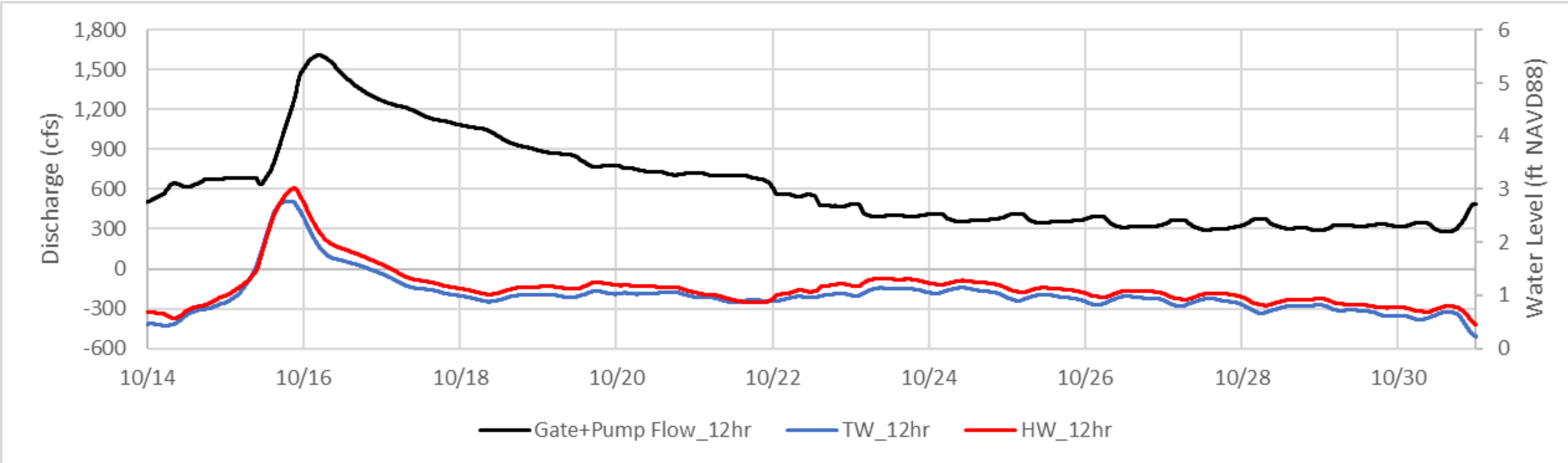


Figure B 5-14. 12-Hour Moving Average Discharge and Stages at S26 for the 25-year 3-day Future Conditions (SLR1) Event

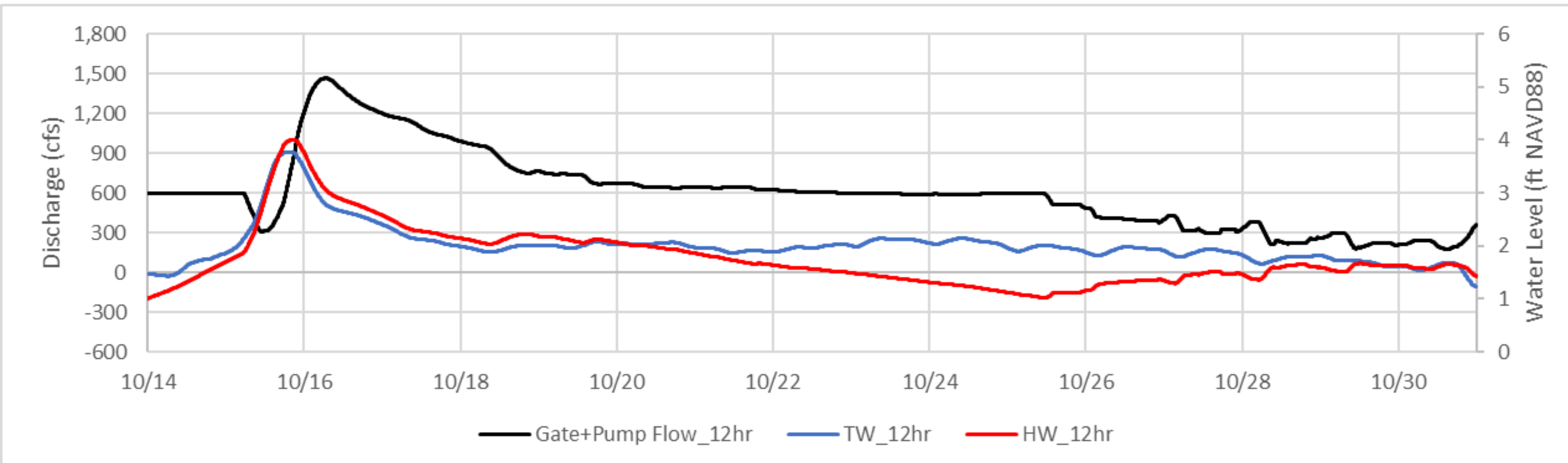


Figure B 5-15. 12-Hour Moving Average Discharge and Stages at S26 for the 25-year 3-day Future Conditions (SLR2) Event

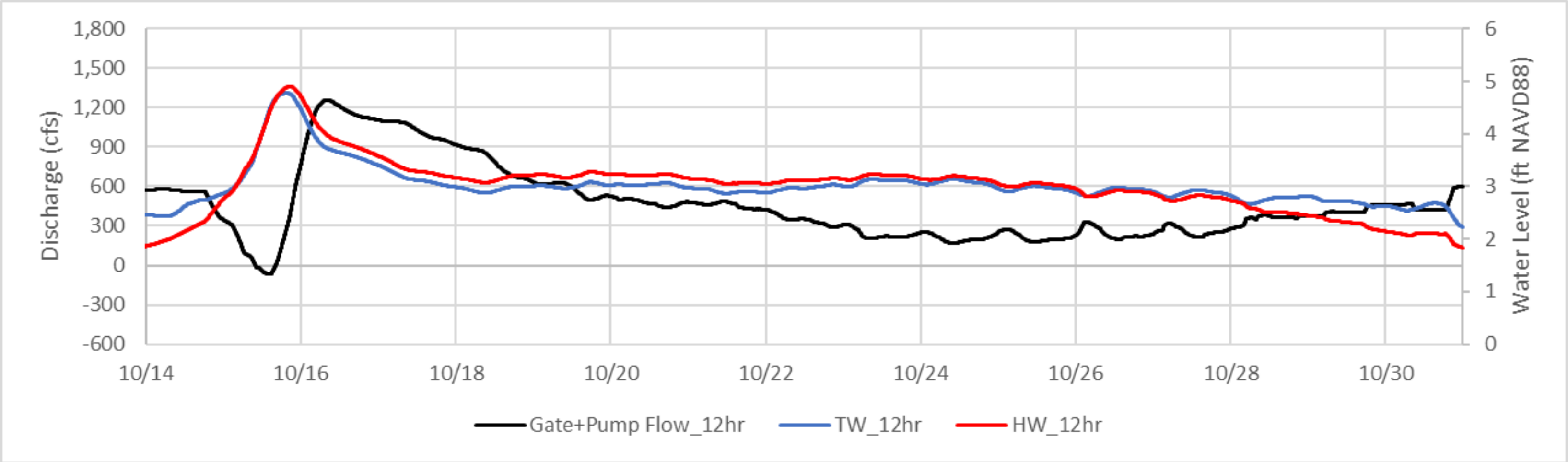


Figure B 5-16. 12-Hour Moving Average Discharge and Stages at S26 for the 25-year 3-day Future Conditions (SLR3) Event

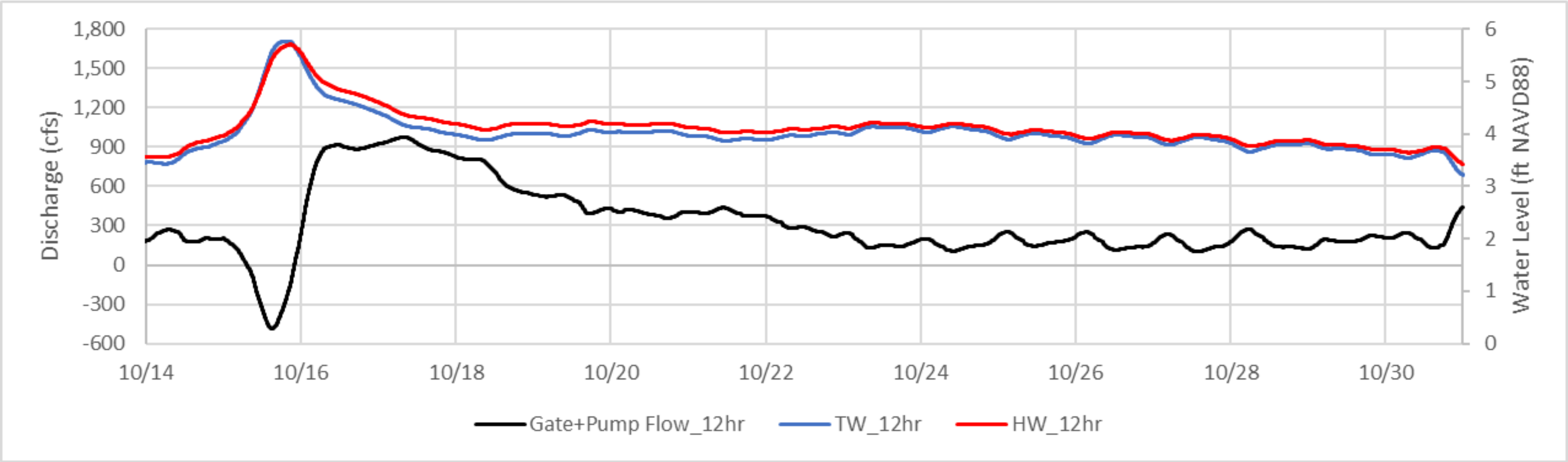


Figure B 5-17. Instantaneous Discharge and Stages at S26 for the 10-year 3-day Current Conditions Event

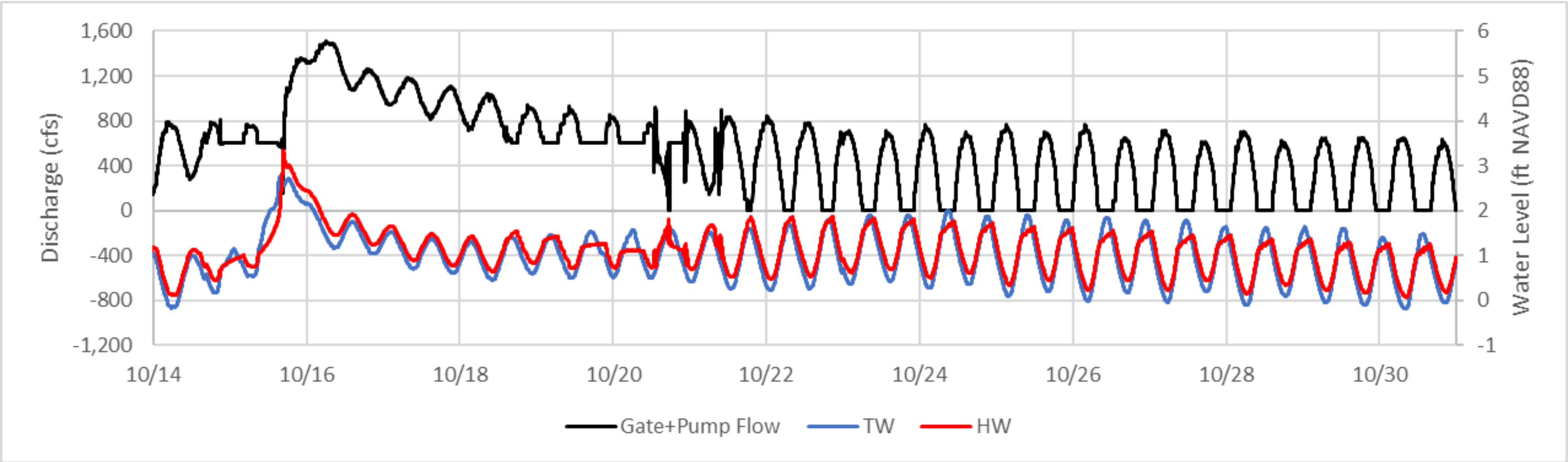


Figure B 5-18. Instantaneous Discharge and Stages at S26 for the 10-year 3-day Future Conditions (SLR1) Event

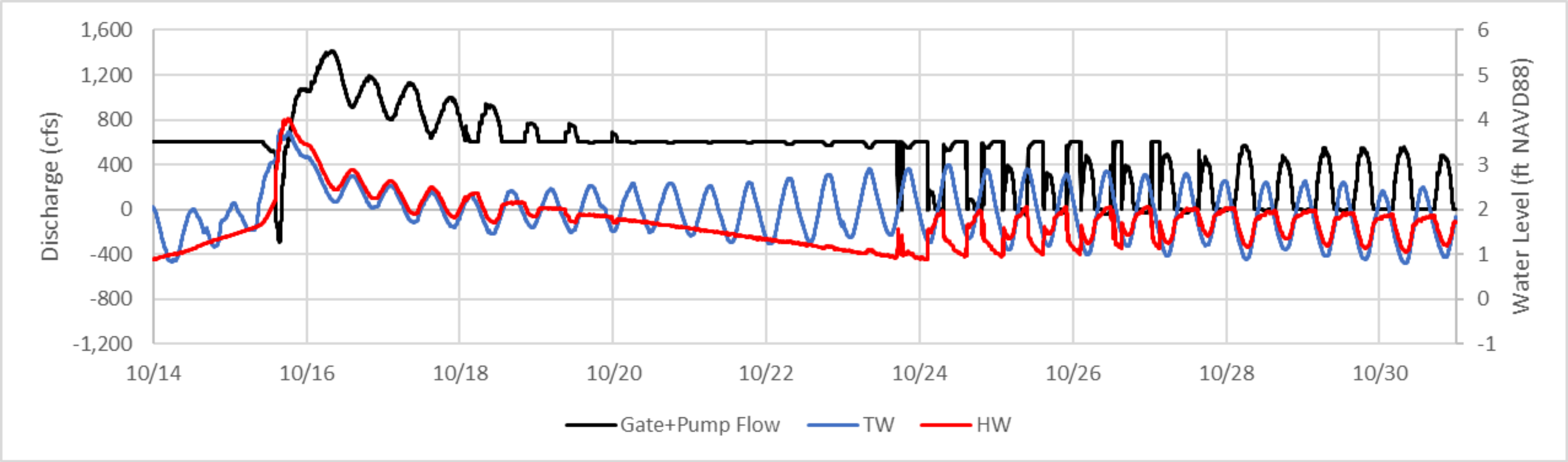


Figure B 5-19. Instantaneous Discharge and Stages at S26 for the 10-year 3-day Future Conditions (SLR2) Event

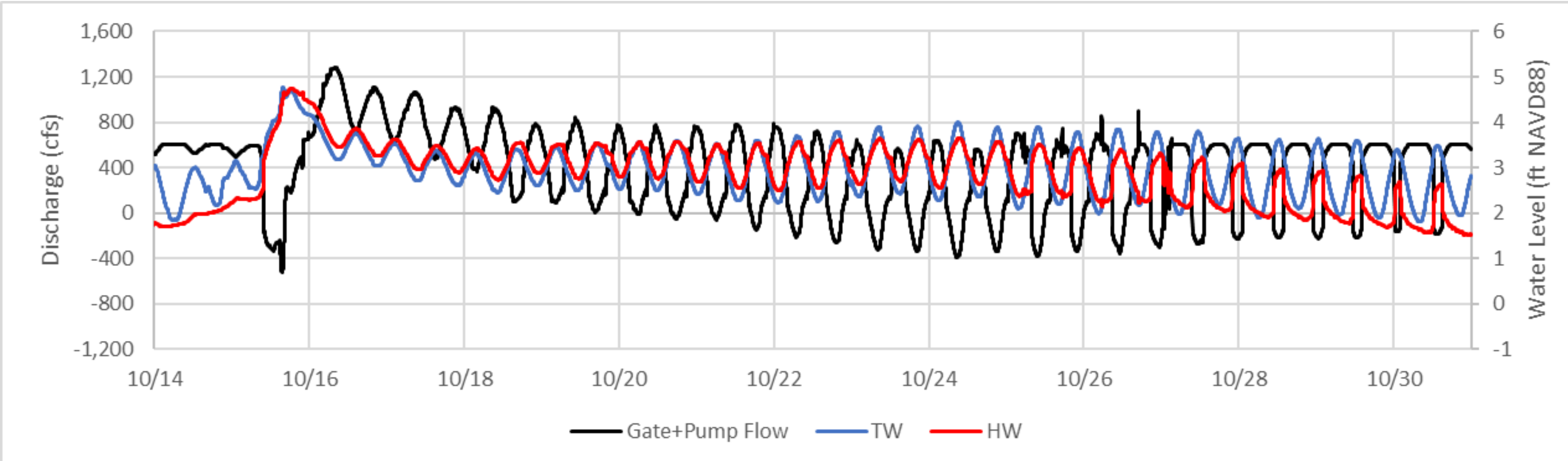


Figure B 5-20. Instantaneous Discharge and Stages at S26 for the 10-year 3-day Future Conditions (SLR3) Event

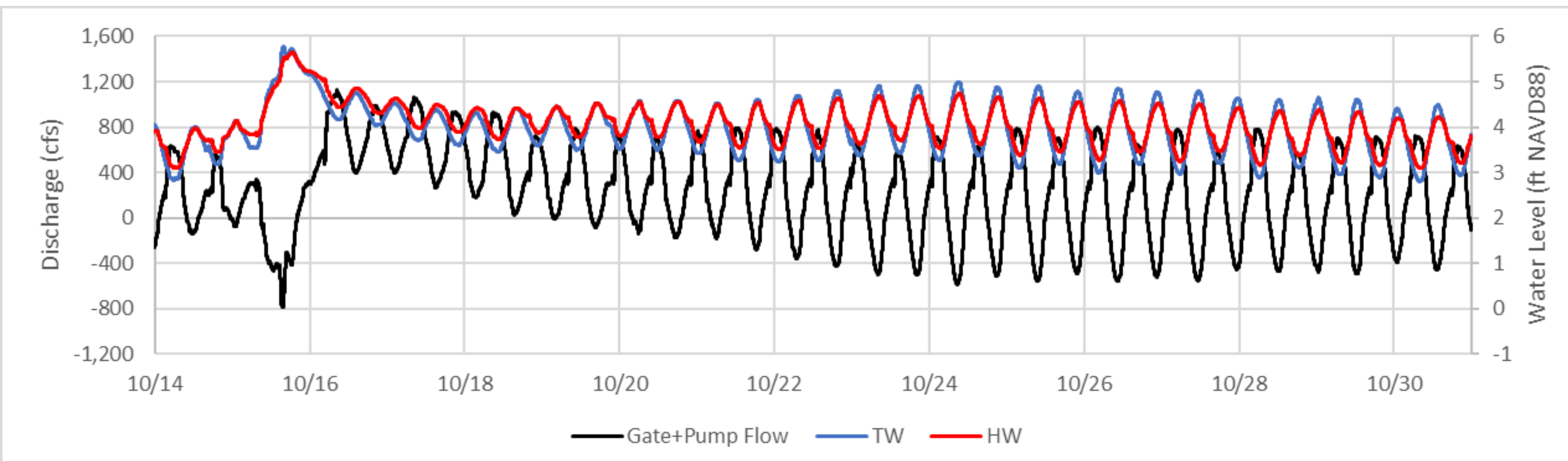


Figure B 5-21. 12-Hour Moving Average Discharge and Stages at S26 for the 10-year 3-day Current Conditions Event

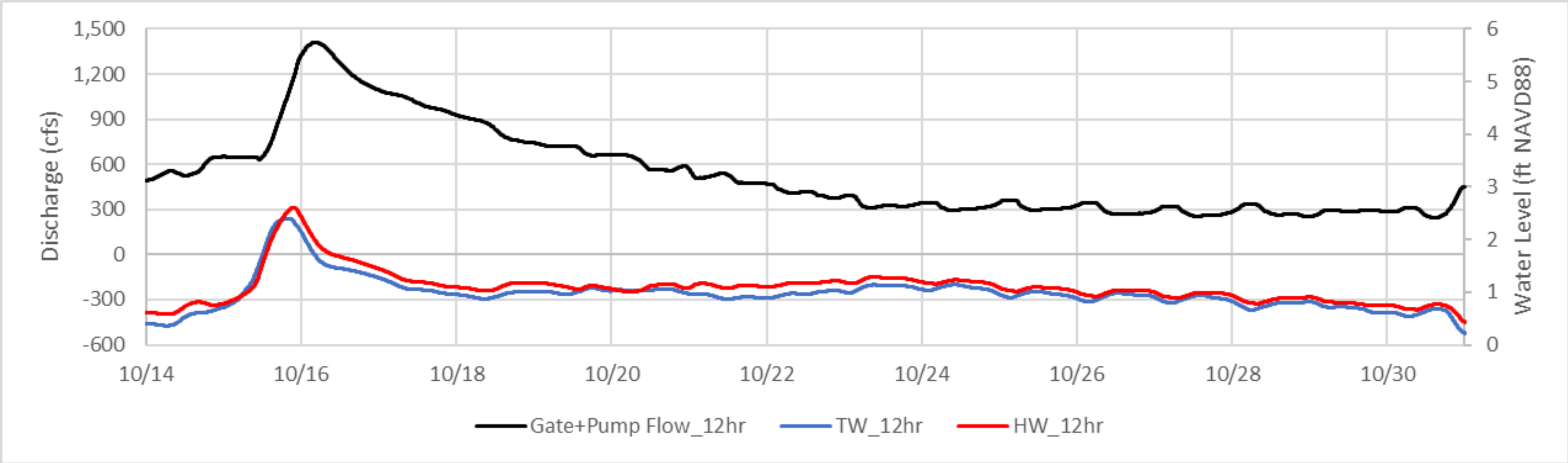


Figure B 5-22. 12-Hour Moving Average Discharge and Stages at S26 for the 10-year 3-day Future Conditions (SLR1) Event

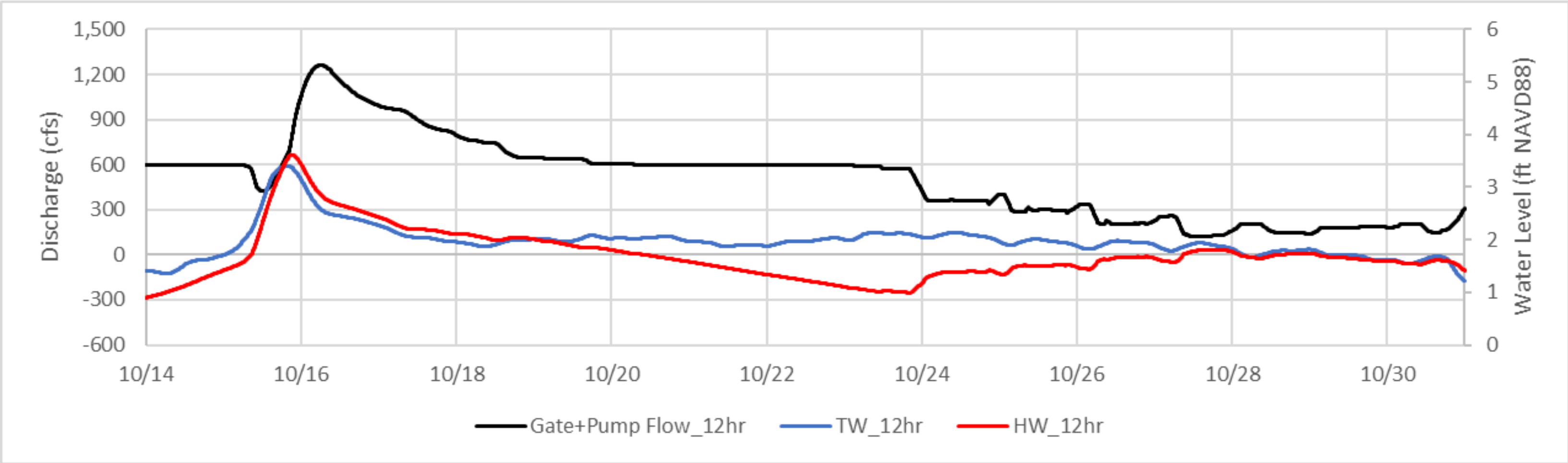


Figure B 5-23. 12-Hour Moving Average Discharge and Stages at S26 for the 10-year 3-day Future Conditions (SLR2) Event

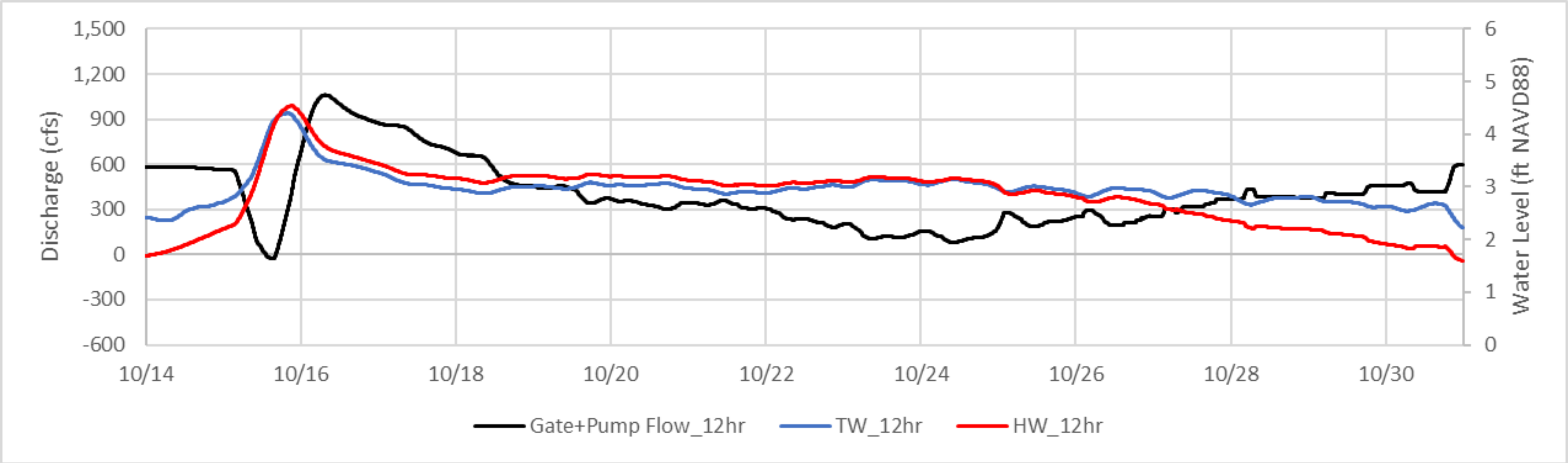


Figure B 5-24. 12-Hour Moving Average Discharge and Stages at S26 for the 10-year 3-day Future Conditions (SLR3) Event

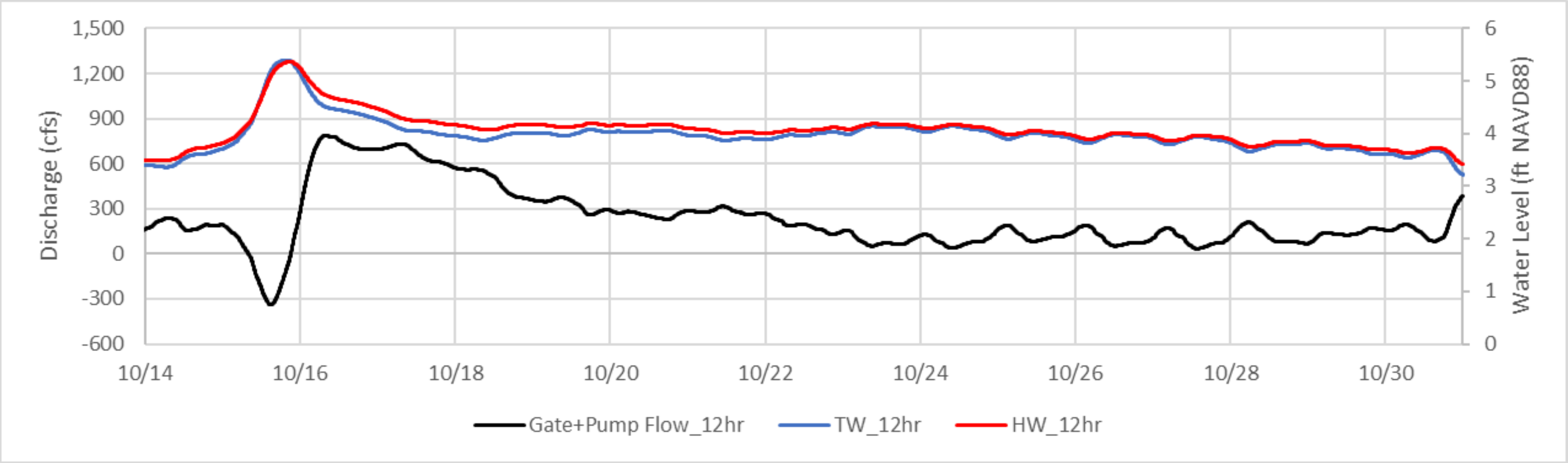


Figure B 5-25. Instantaneous Discharge and Stages at S26 for the 5-year 3-day Current Conditions Event

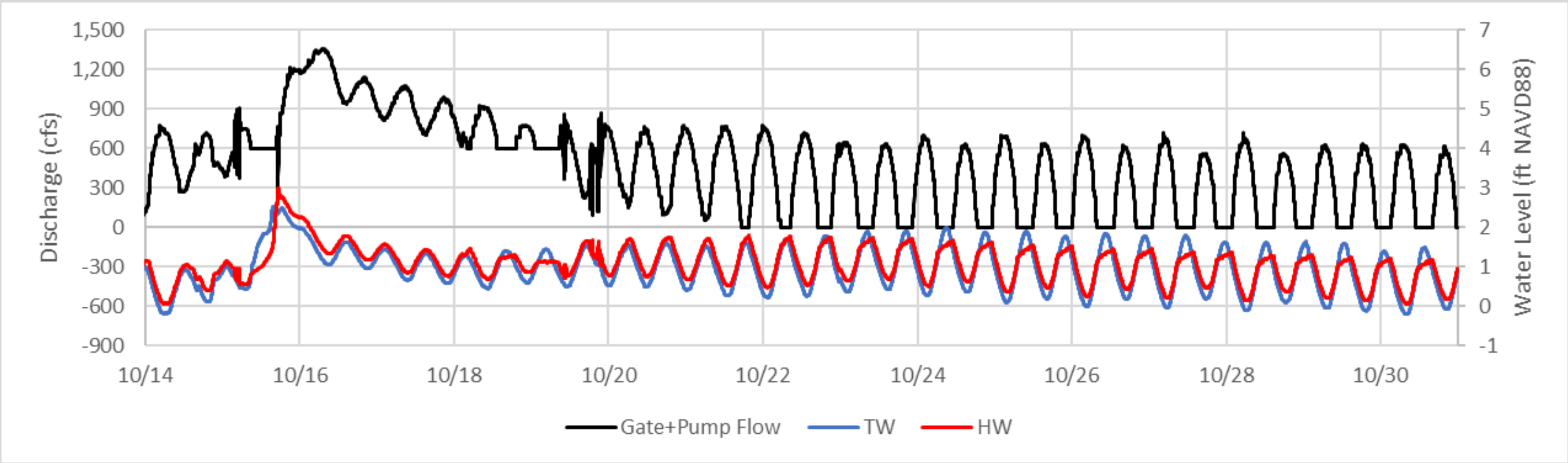


Figure B 5-26. Instantaneous Discharge and Stages at S26 for the 5-year 3-day Future Conditions (SLR1) Event

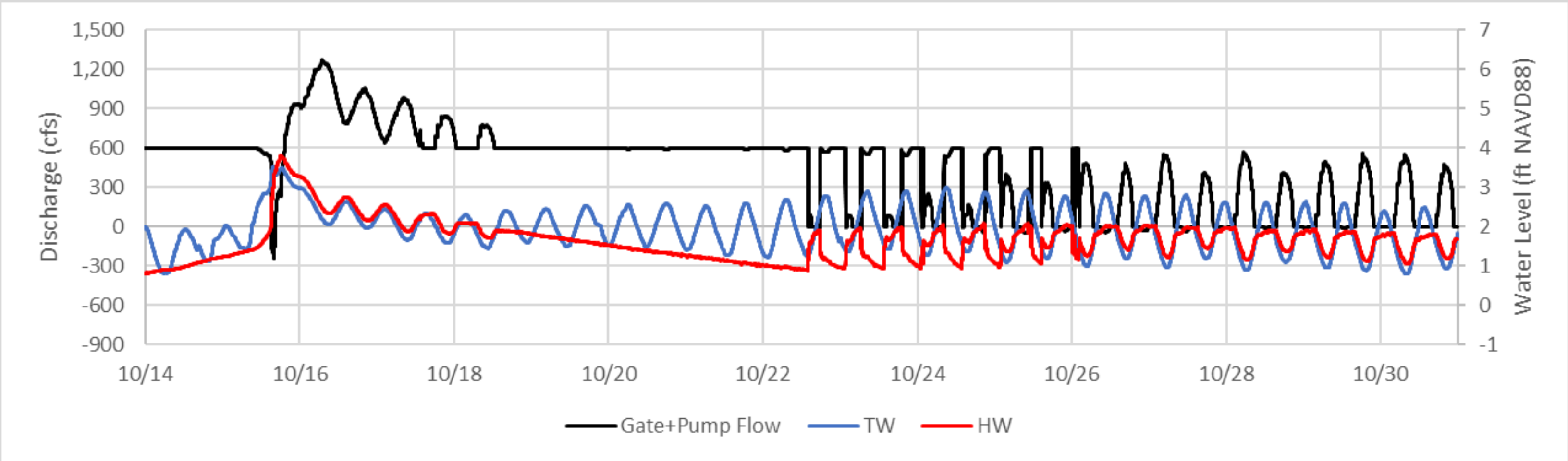


Figure B 5-27. Instantaneous Discharge and Stages at S26 for the 5-year 3-day Future Conditions (SLR2) Event

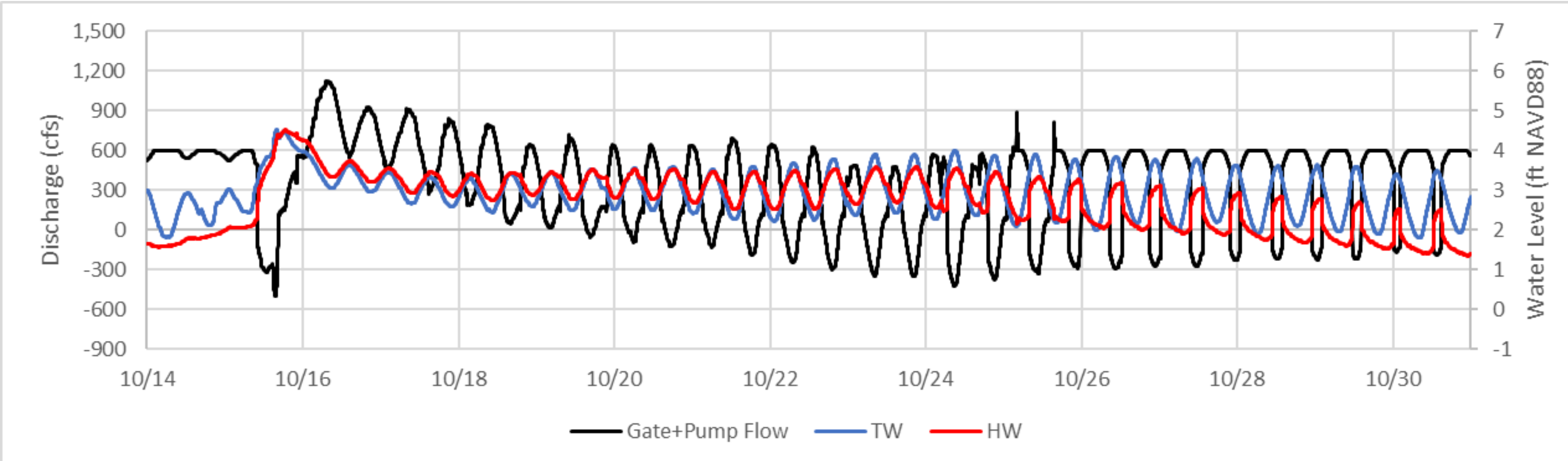


Figure B 5-28. Instantaneous Discharge and Stages at S26 for the 5-year 3-day Future Conditions (SLR3) Event

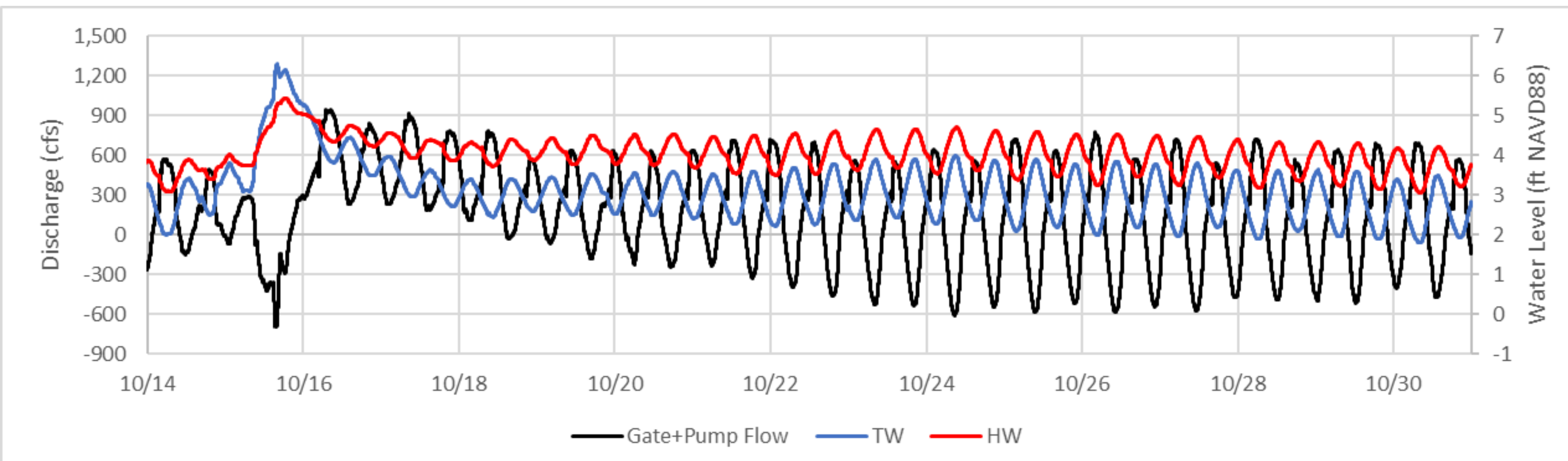


Figure B 5-29. 12-Hour Moving Average Discharge and Stages at S26 for the 5-year 3-day Current Conditions Event

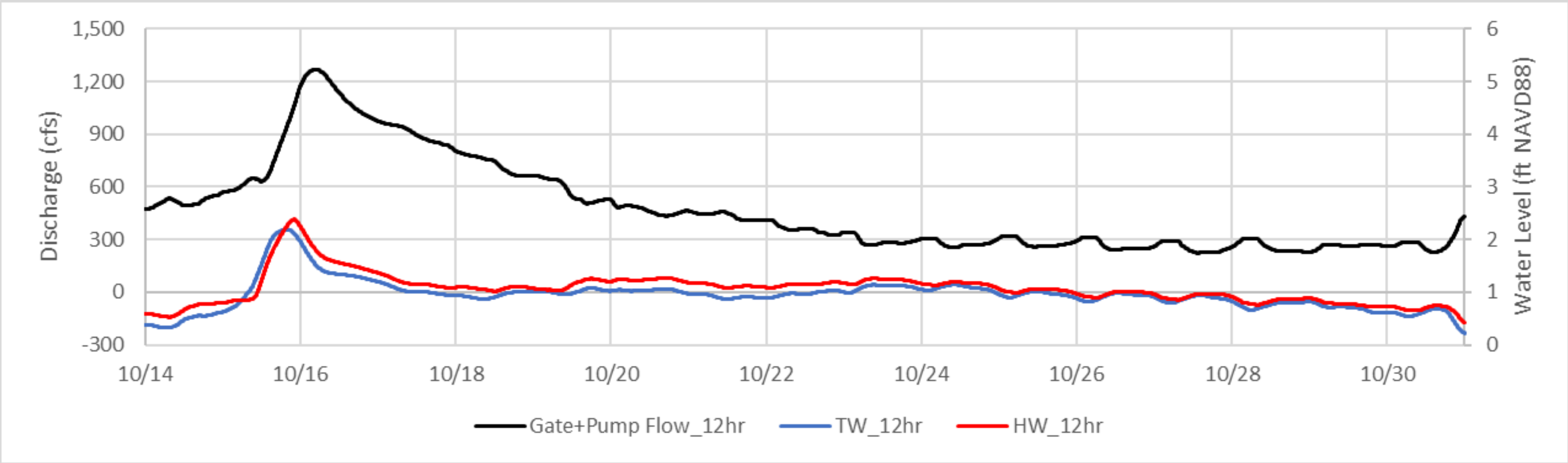


Figure B 5-30. 12-Hour Moving Average Discharge and Stages at S26 for the 5-year 3-day Future Conditions (SLR1) Event

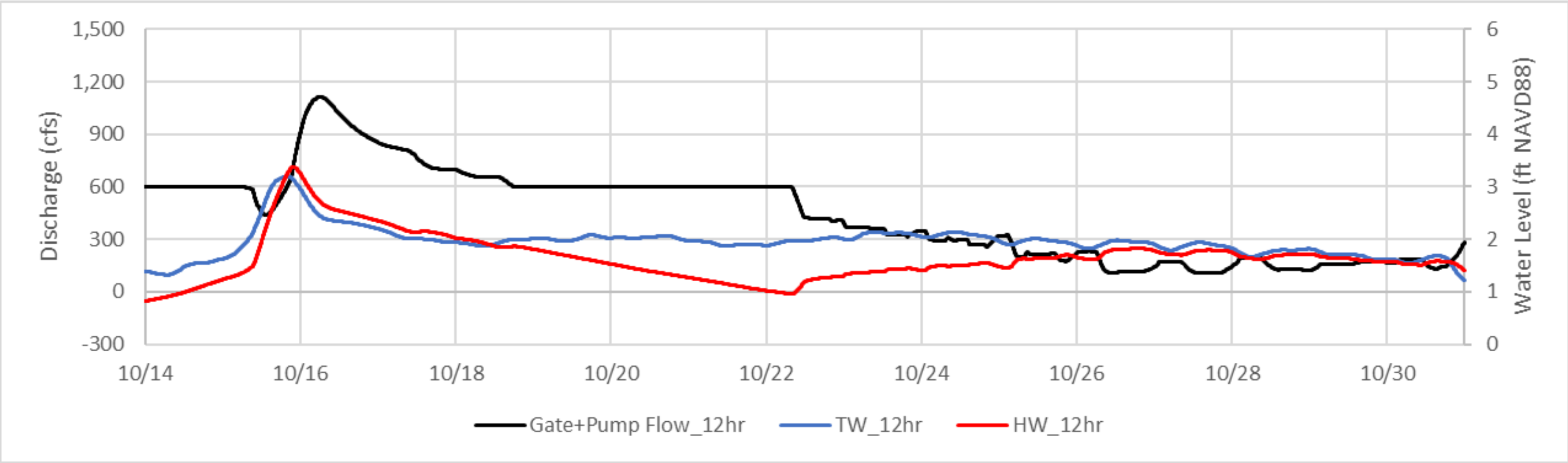


Figure B 5-31. 12-Hour Moving Average Discharge and Stages at S26 for the 5-year 3-day Future Conditions (SLR2) Event

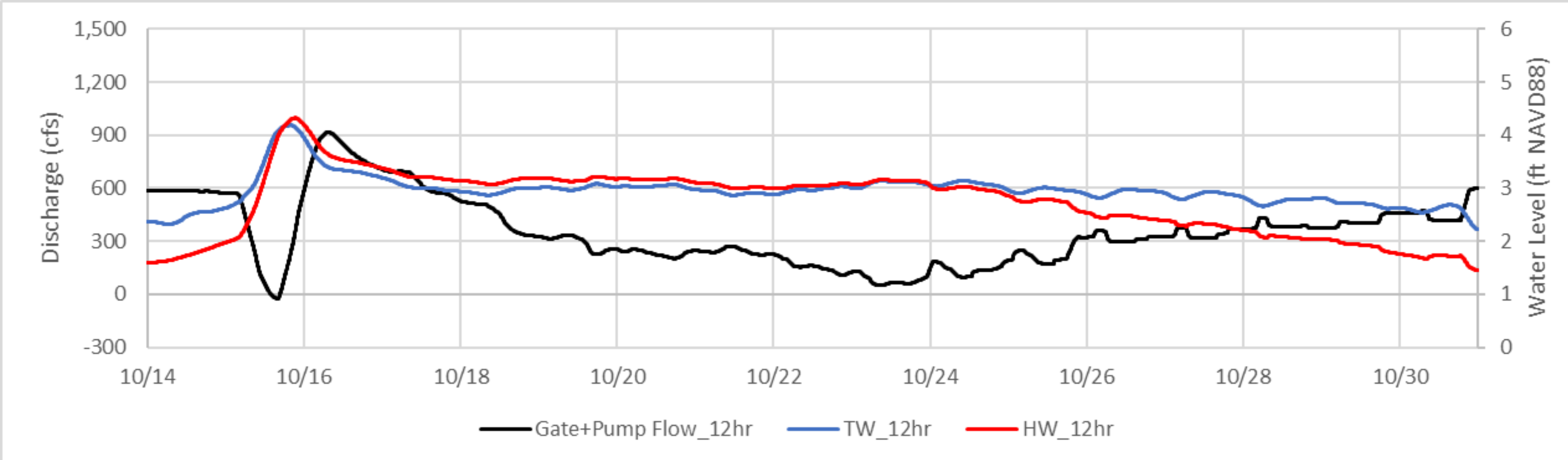
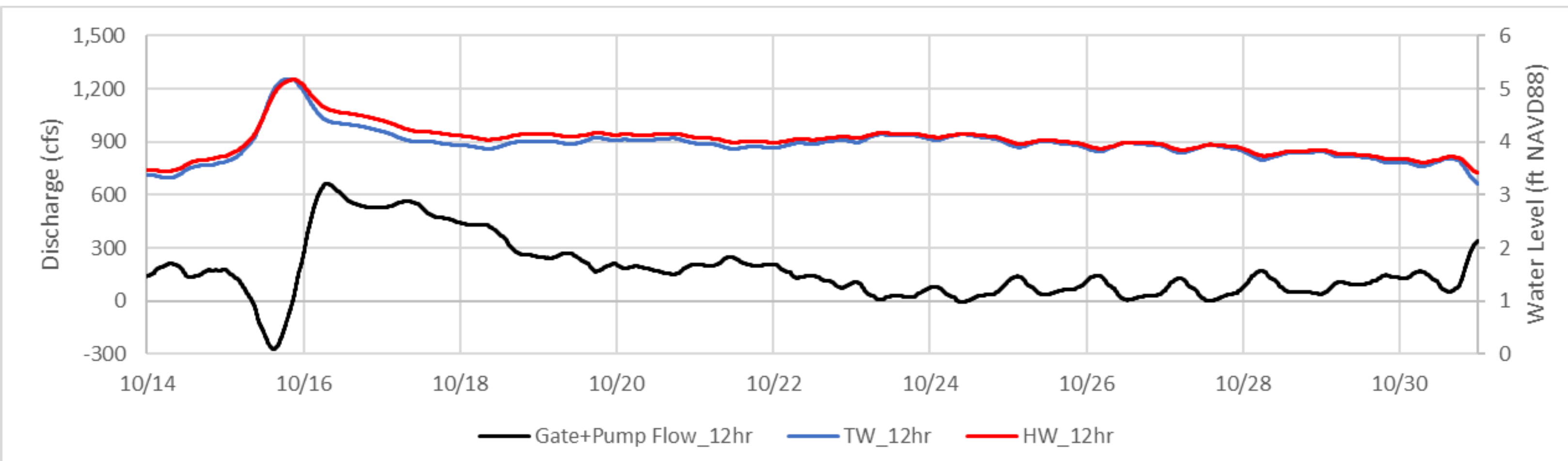


Figure B 5-32. 12-Hour Moving Average Discharge and Stages at S26 for the 5-year 3-day Future Conditions (SLR3) Event



APPENDIX C

PM5 Maximum Depth Figures

Contents

Figures

Appendix C – PM#5 Maximum Depth Figures

The figures provided in this appendix include maximum depth figures for each watershed. The figures compare the maximum depth of flooding for each SLR condition (i.e., Current Conditions, SLR1, SLR2, and SLR3).

1 C2 Watershed

Figure C 1-1 through **Figure C 1-15** are maps of the maximum overland depth over the entire C2 Watershed for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for each SLR condition. Water areas are masked in black. **Figure C 1-16** through **Figure C 1-31** provide the same maps with the non-urban areas masked out, to provide a concise picture of how urban areas are impacted within the watershed.

Figure C 1-32 through **Figure C 1-43** show the difference in overland flooding for the C2 Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively. **Figure C 1-44** through **Figure C 1-55** show the same maps with the non-urban areas masked out.

Figure C 1-1. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for Current Conditions in the C2 Watershed

Figure C 1-1. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR1 in the C2 Watershed

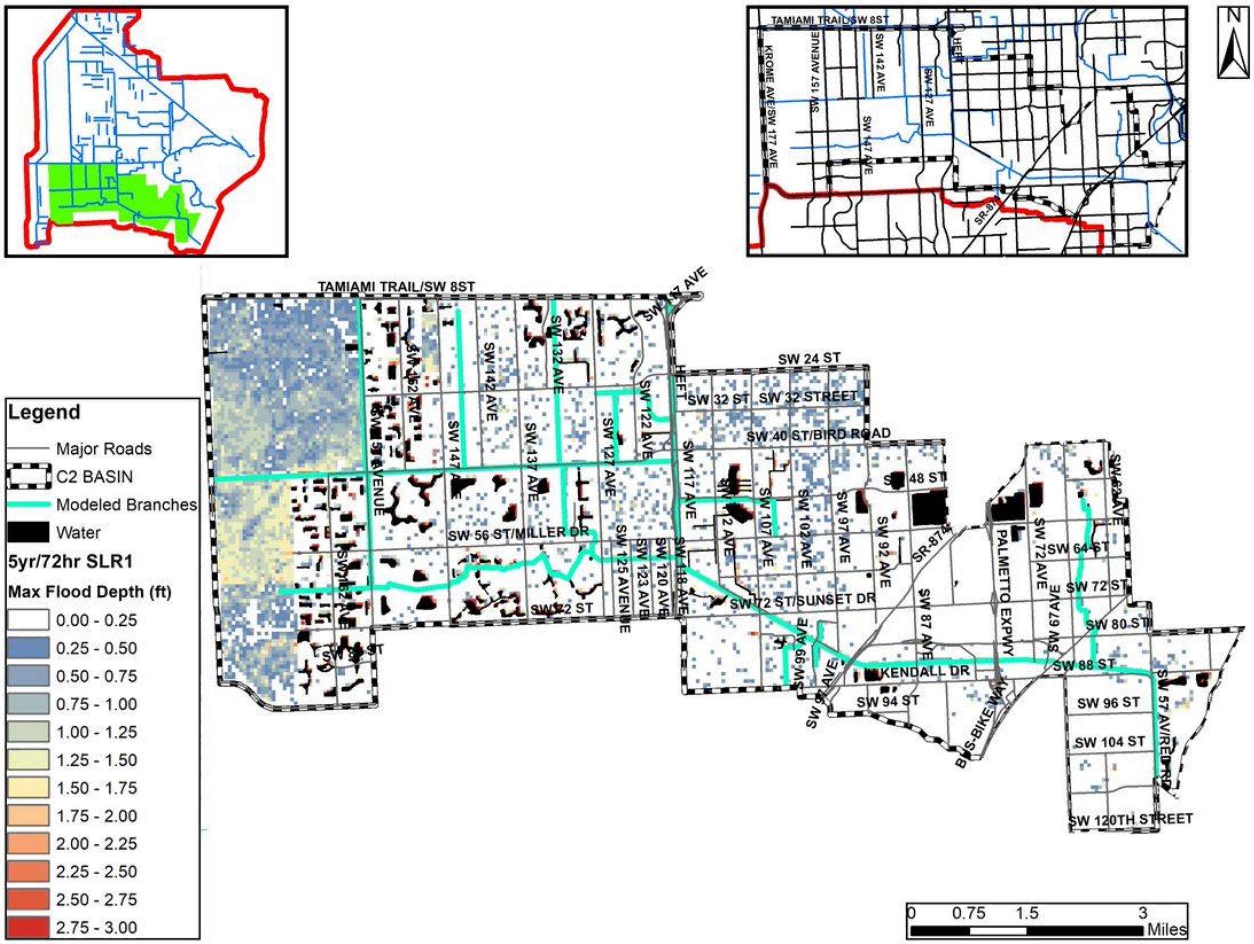


Figure C 1-2. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR2 in the C2 Watershed

Figure C 1-3. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR3 in the C2 Watershed

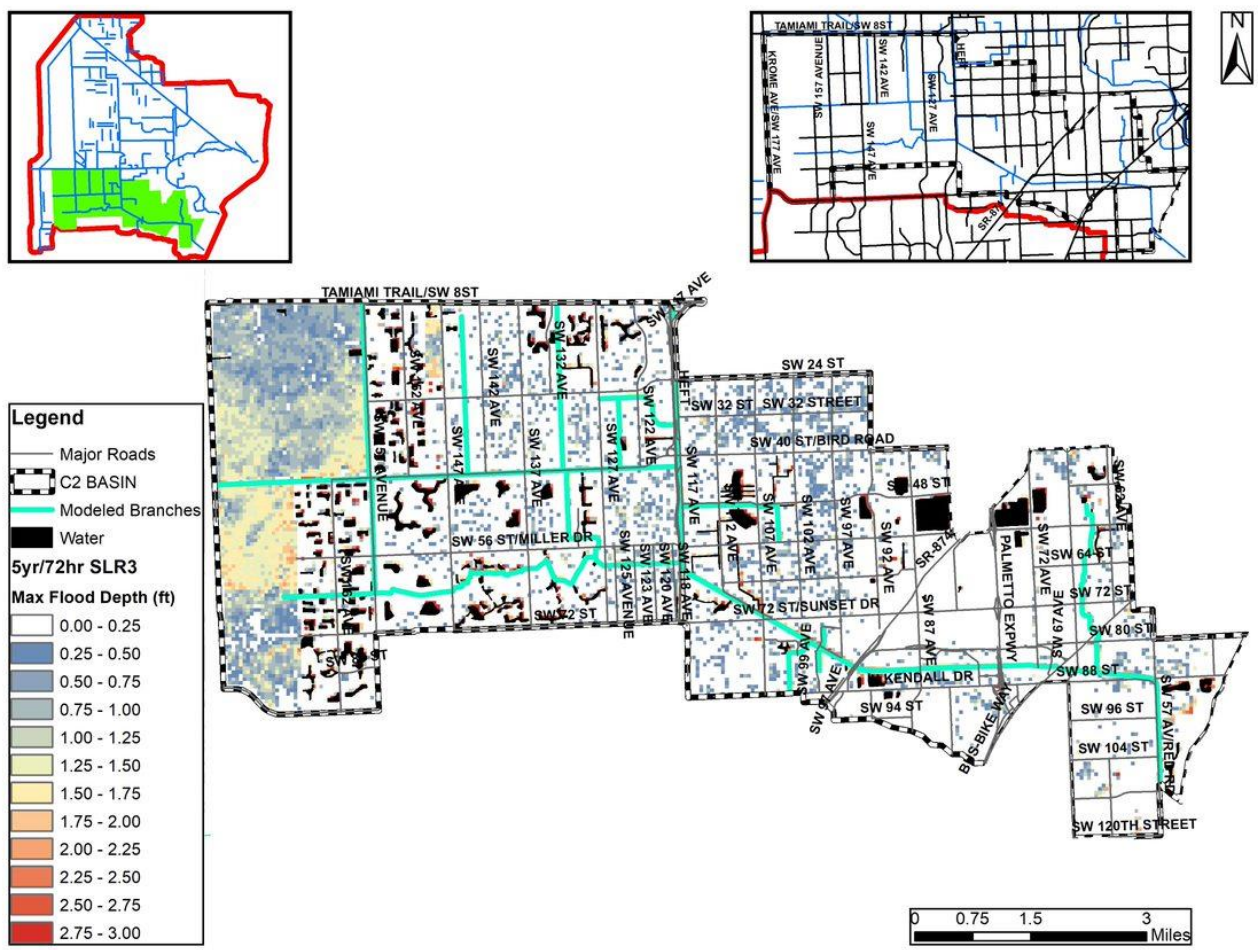


Figure C 1-4. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for Current Conditions in the C2 Watershed

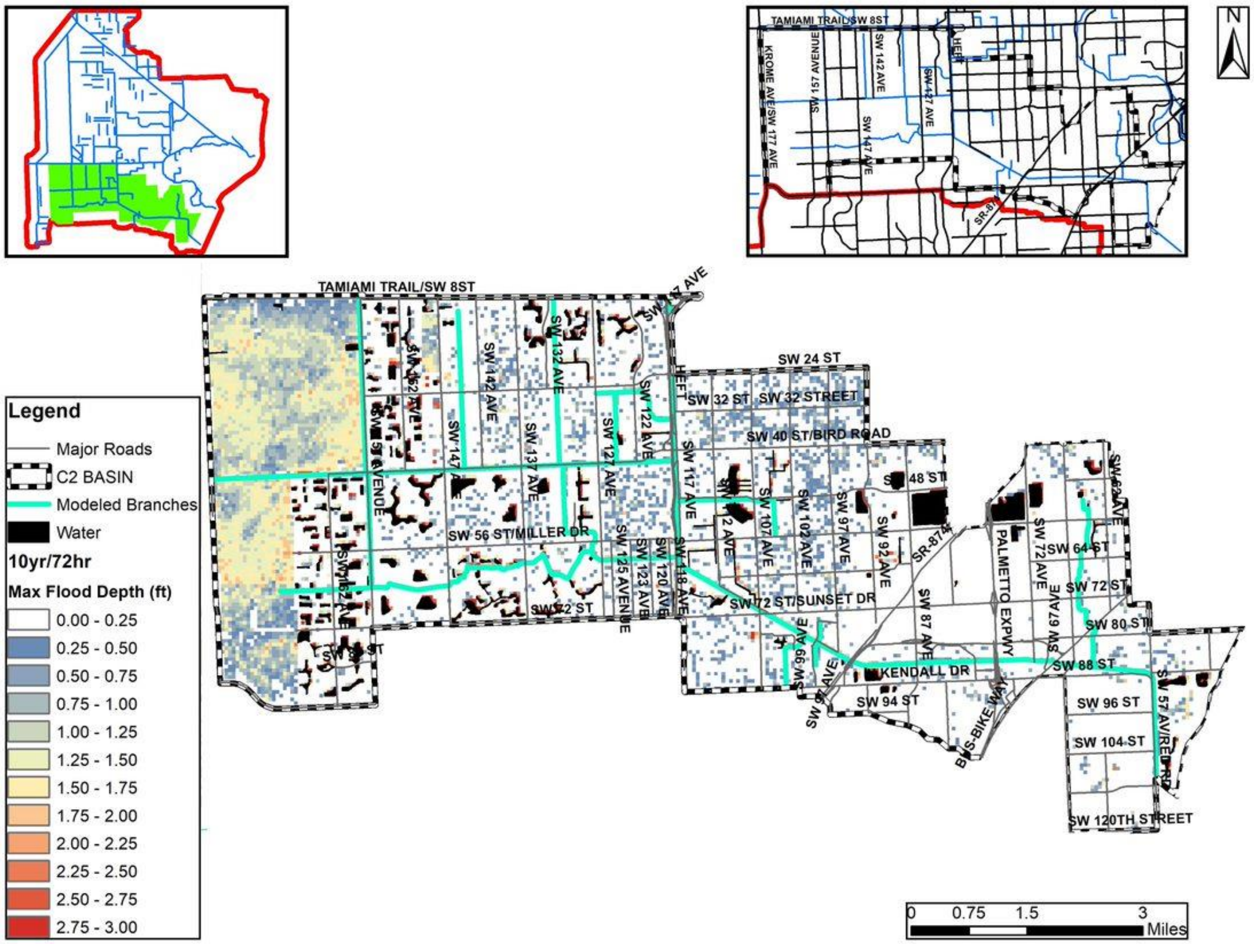


Figure C 1-5. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR1 in the C2 Watershed

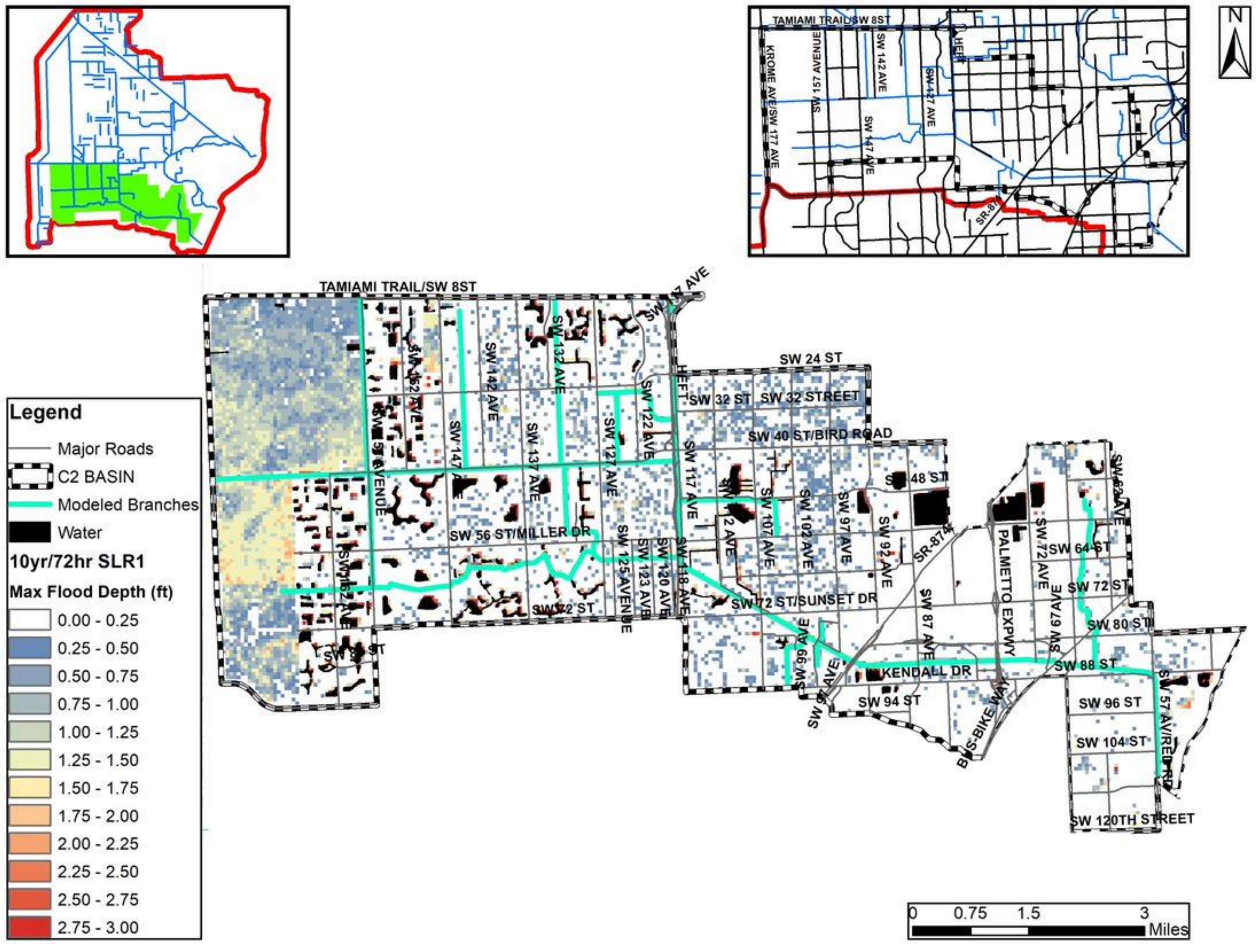


Figure C 1-6. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR2 in the C2 Watershed

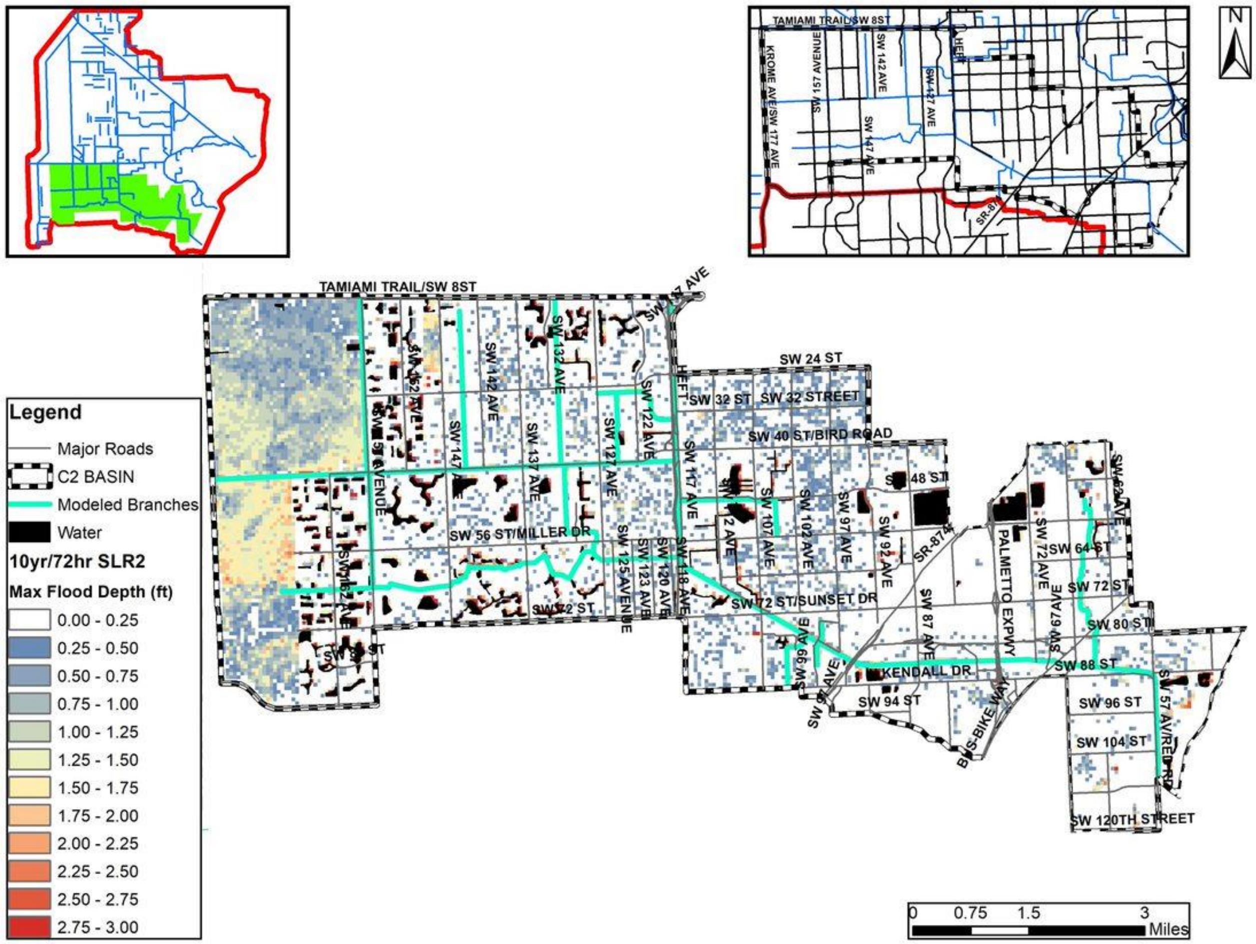


Figure C 1-7. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR3 in the C2 Watershed

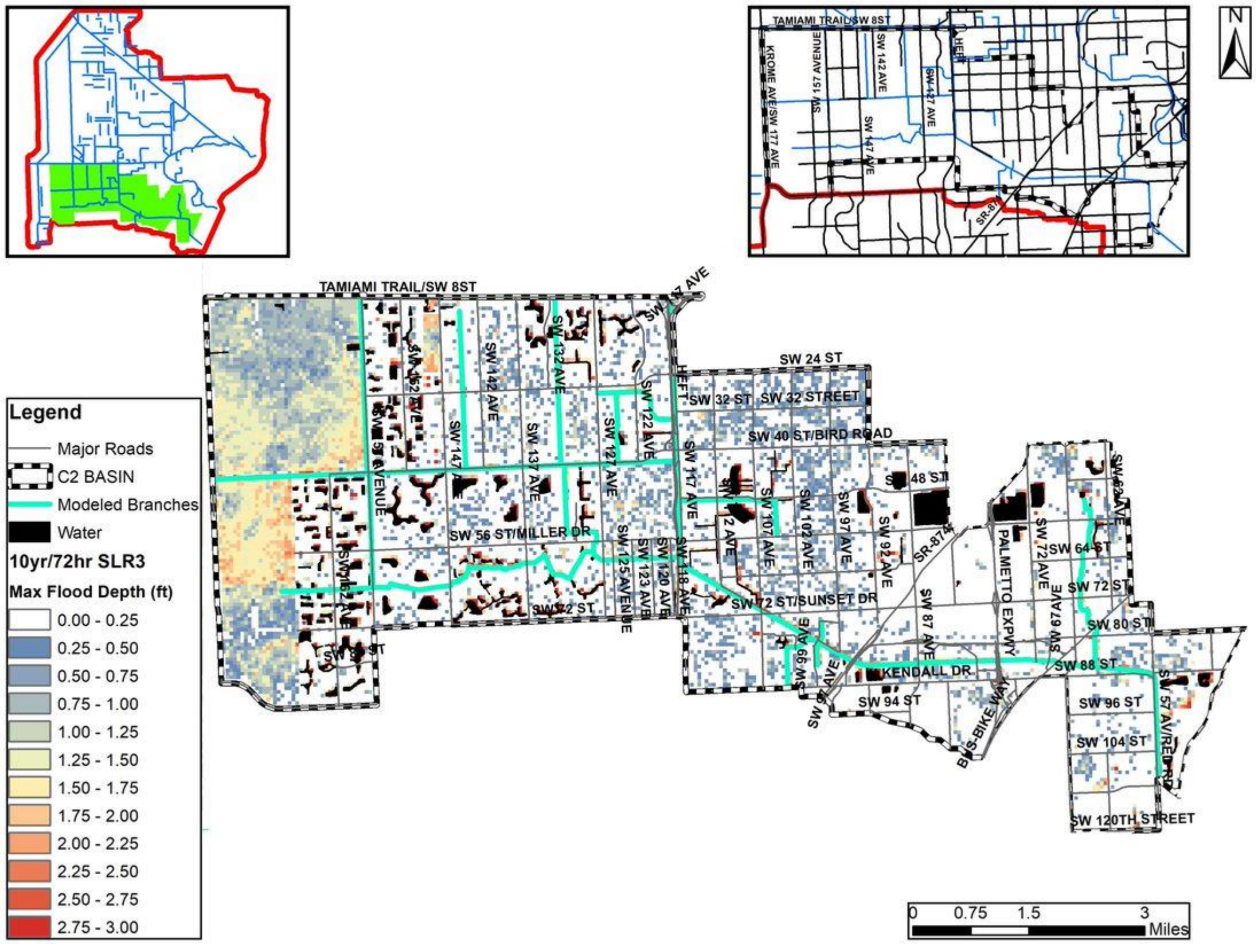


Figure C 1-8. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for Current Conditions in the C2 Watershed

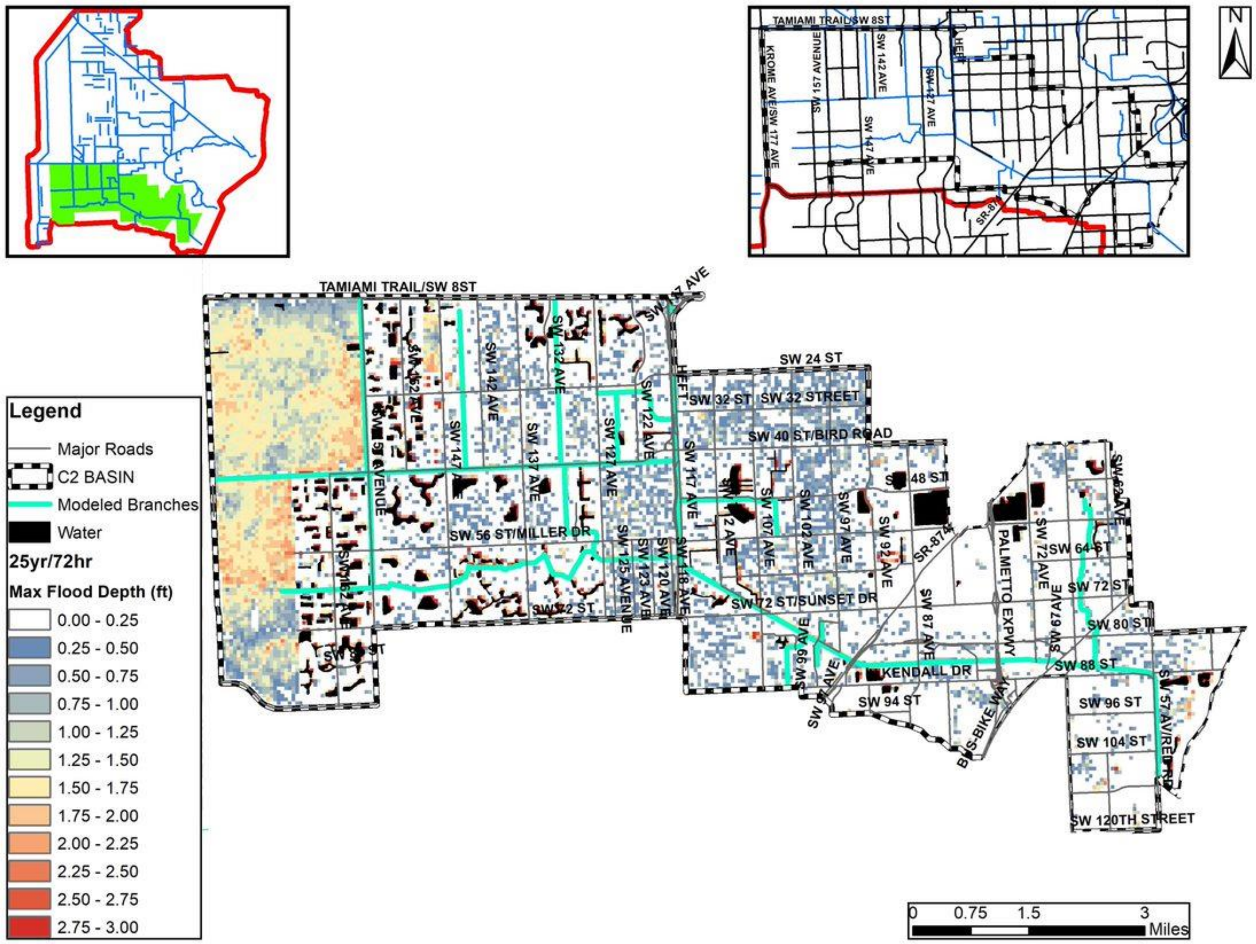


Figure C 1-9. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR1 in the C2 Watershed

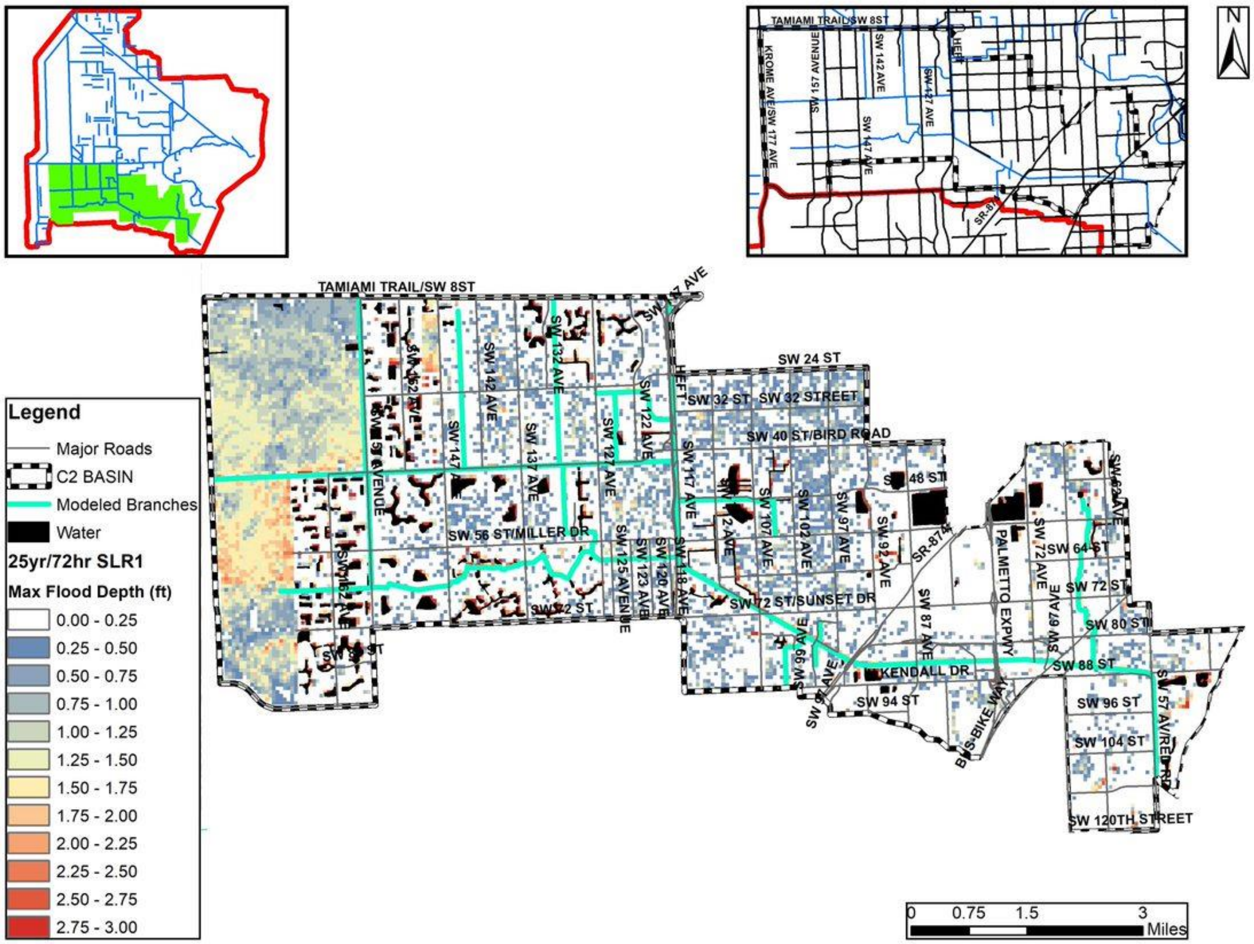


Figure C 1-10. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR2 in the C2 Watershed

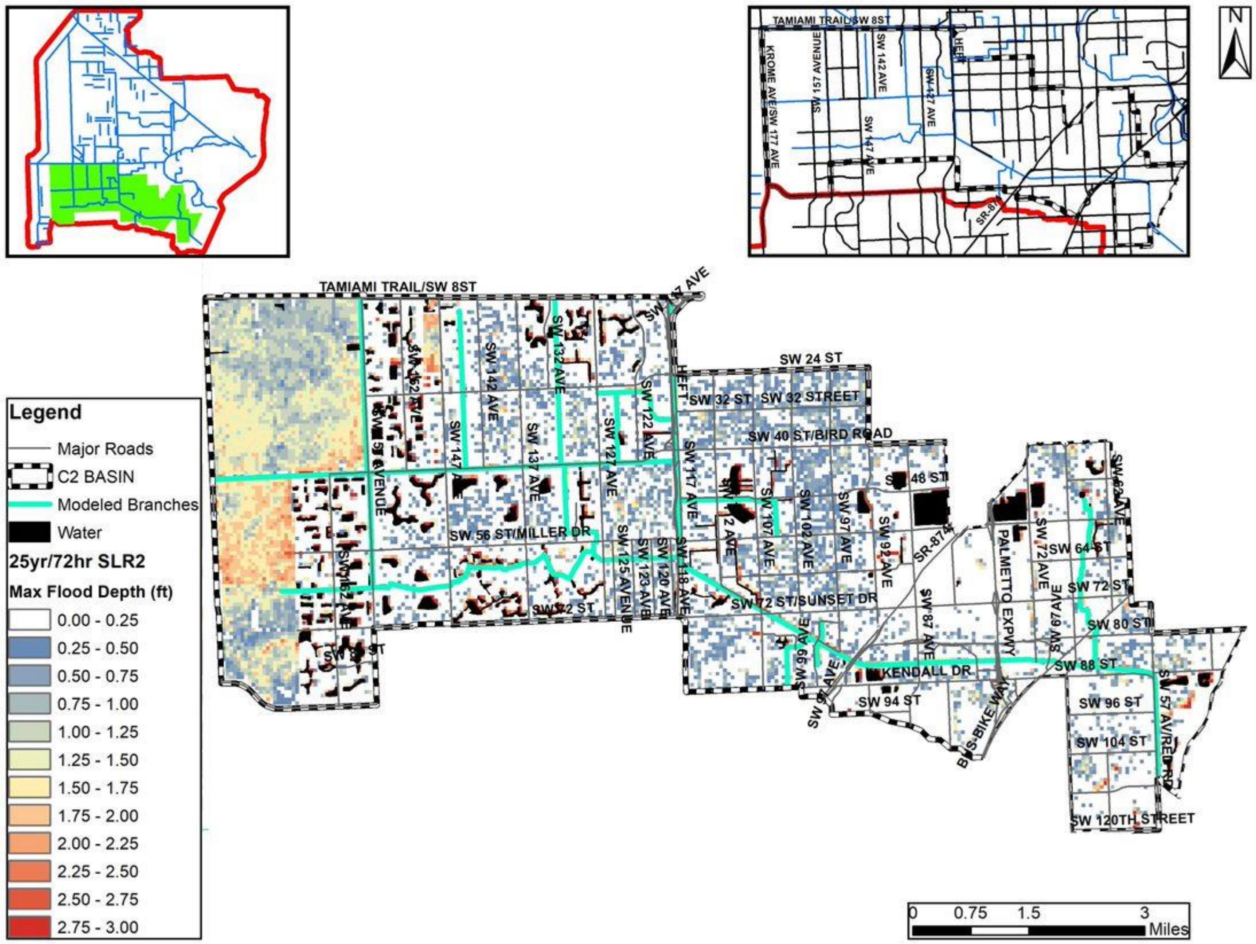


Figure C 1-11. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR3 in the C2 Watershed

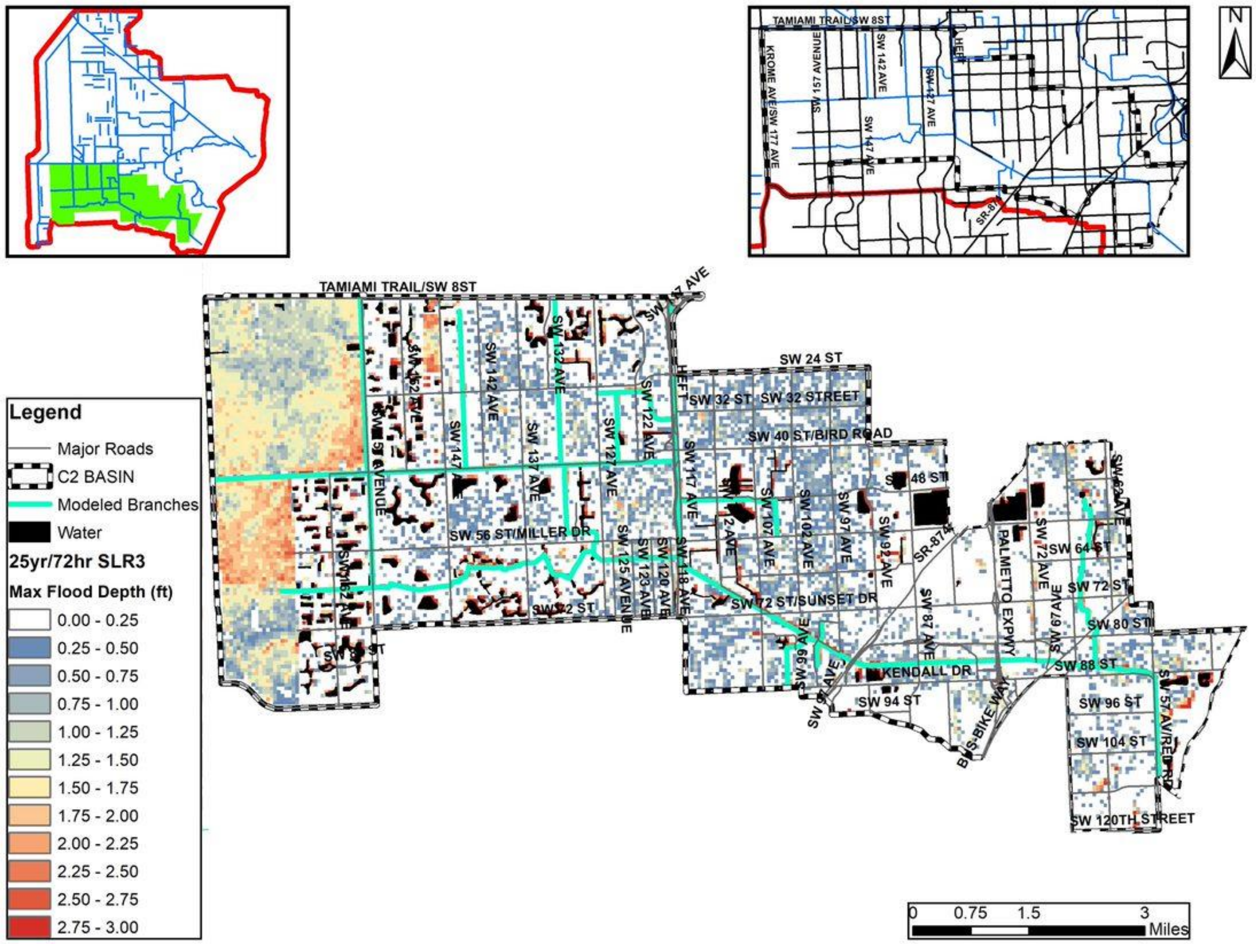


Figure C 1-12. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for Current Conditions in the C2 Watershed

Figure C 1-13. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR1 in the C2 Watershed

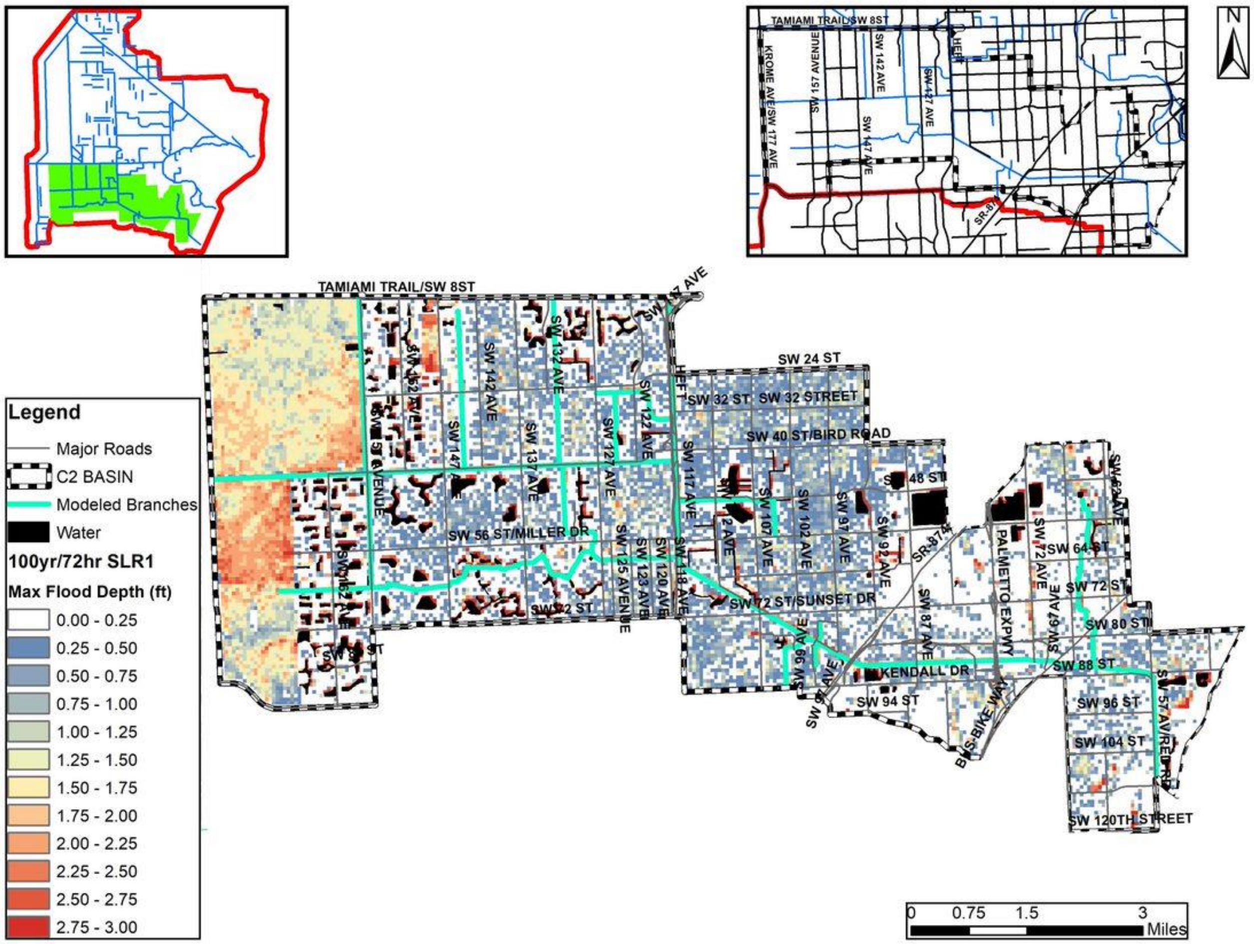


Figure C 1-14. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR2 in the C2 Watershed

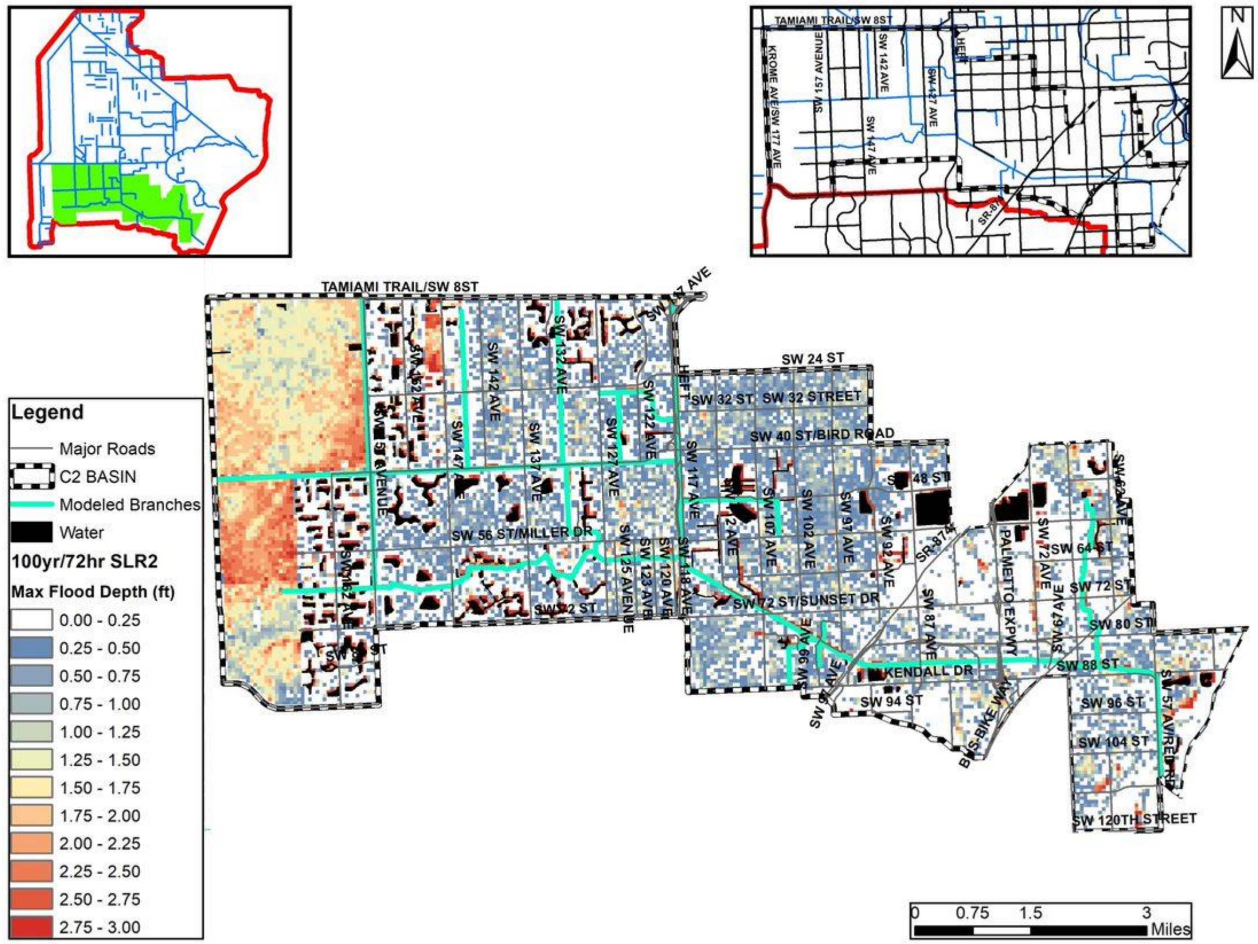


Figure C 1-15. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR3 in the C2 Watershed

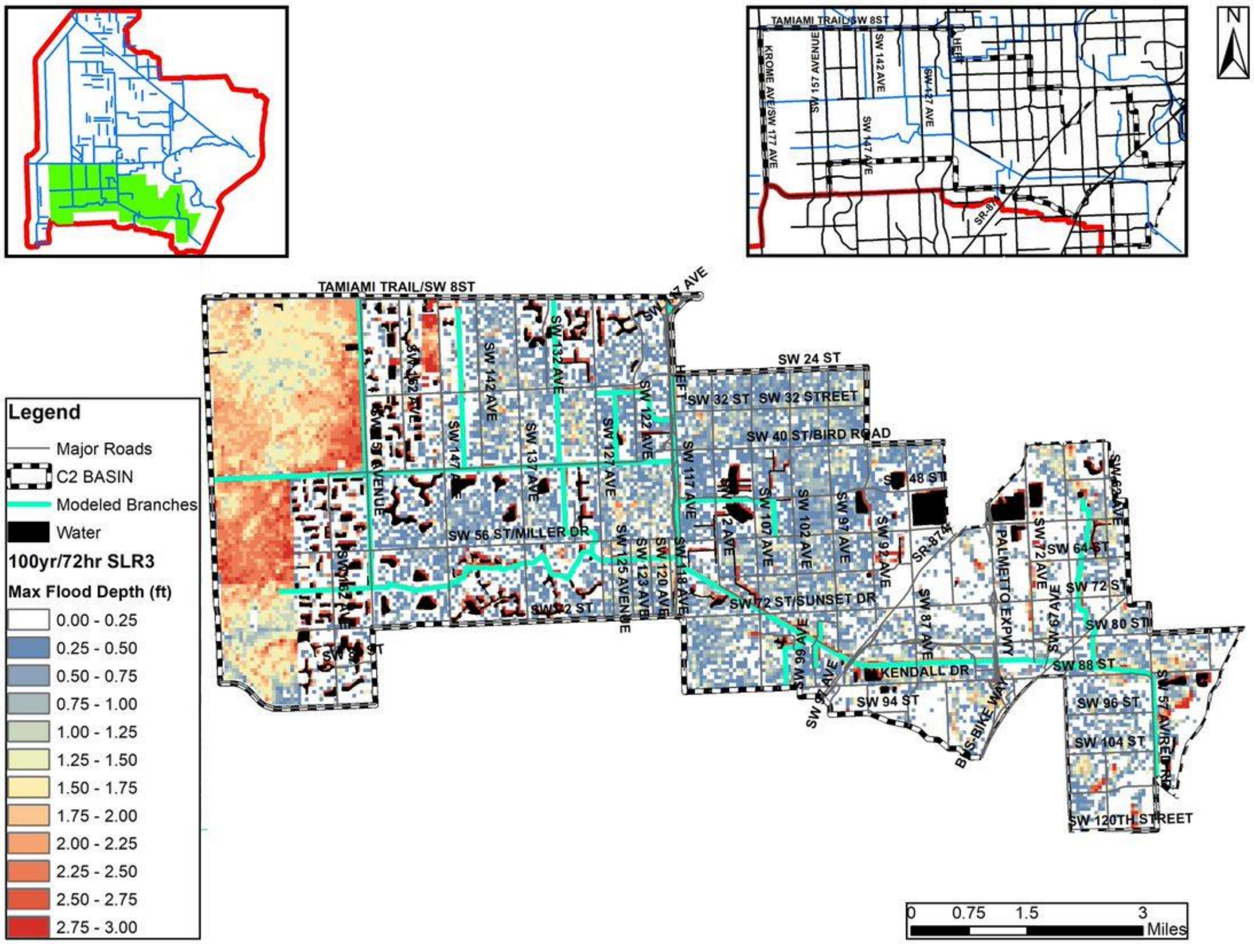


Figure C 1-16. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for Current Conditions in Urban Areas in the C2 Watershed

Figure C 1-17. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR1 in Urban Areas in the C2 Watershed

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Figure C 1-24. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for Current Conditions in Urban Areas in the C2 Watershed

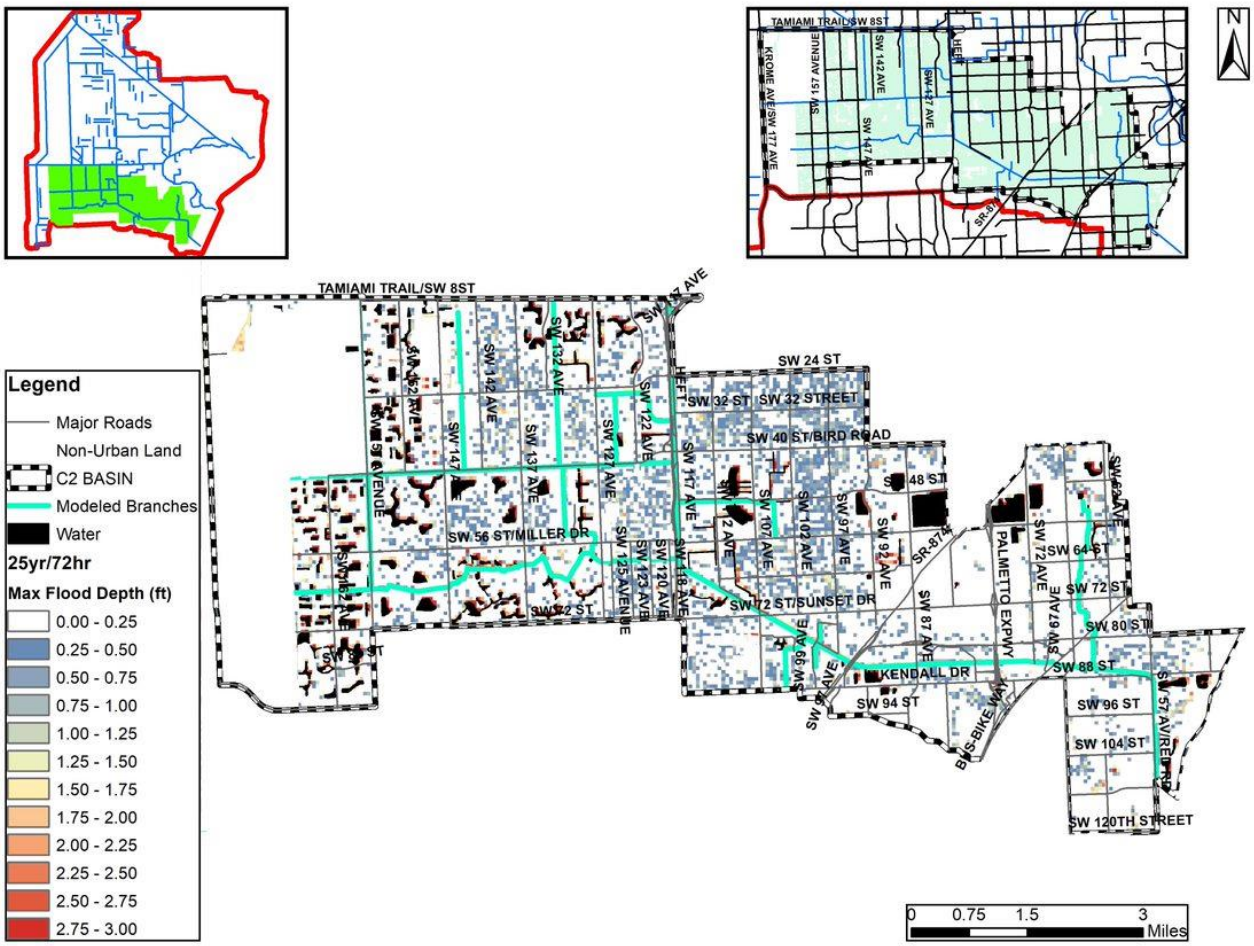

Figure C 1-25. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR1 in Urban Areas in the C2 Watershed

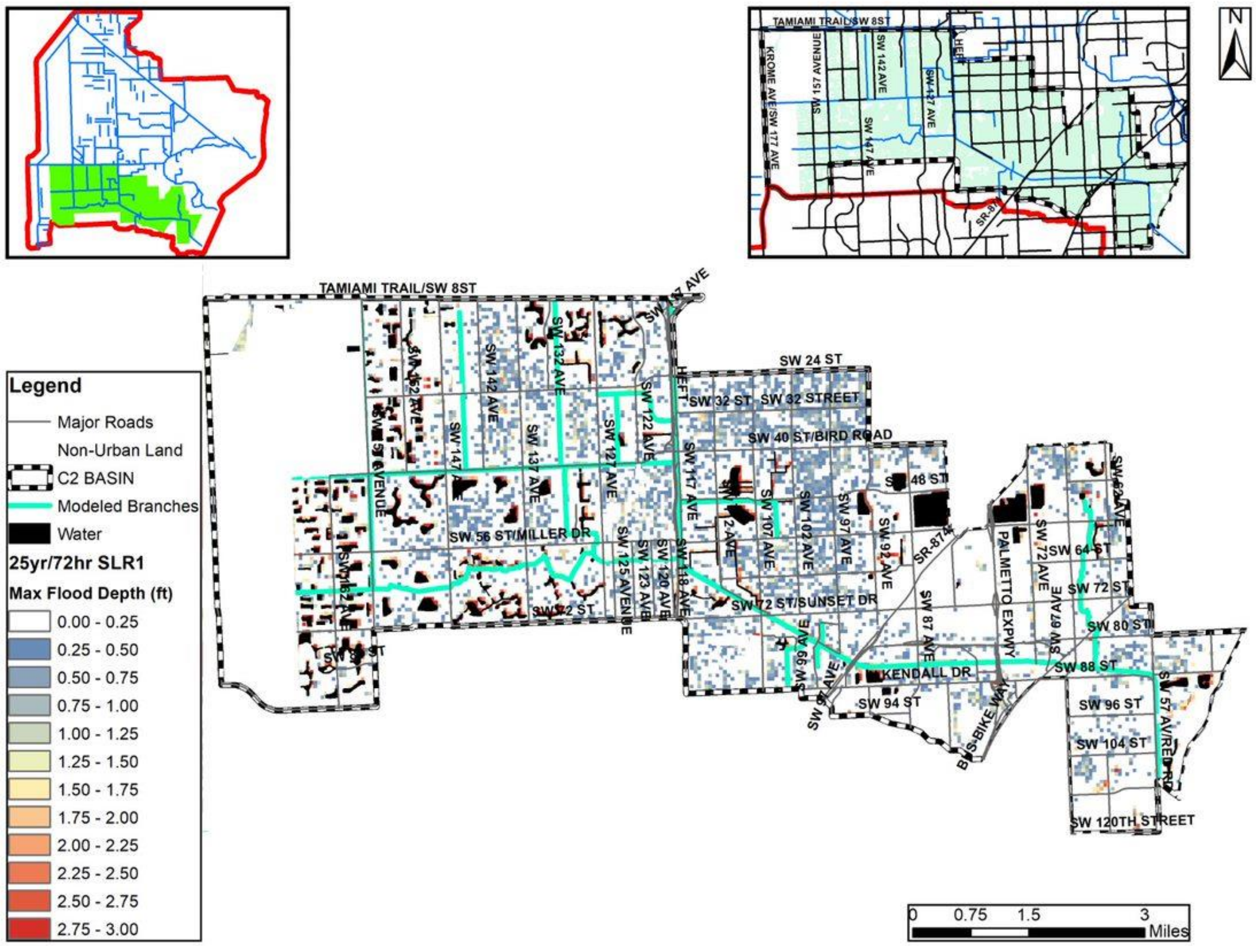

Figure C 1-26. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR2 in Urban Areas in the C2 Watershed

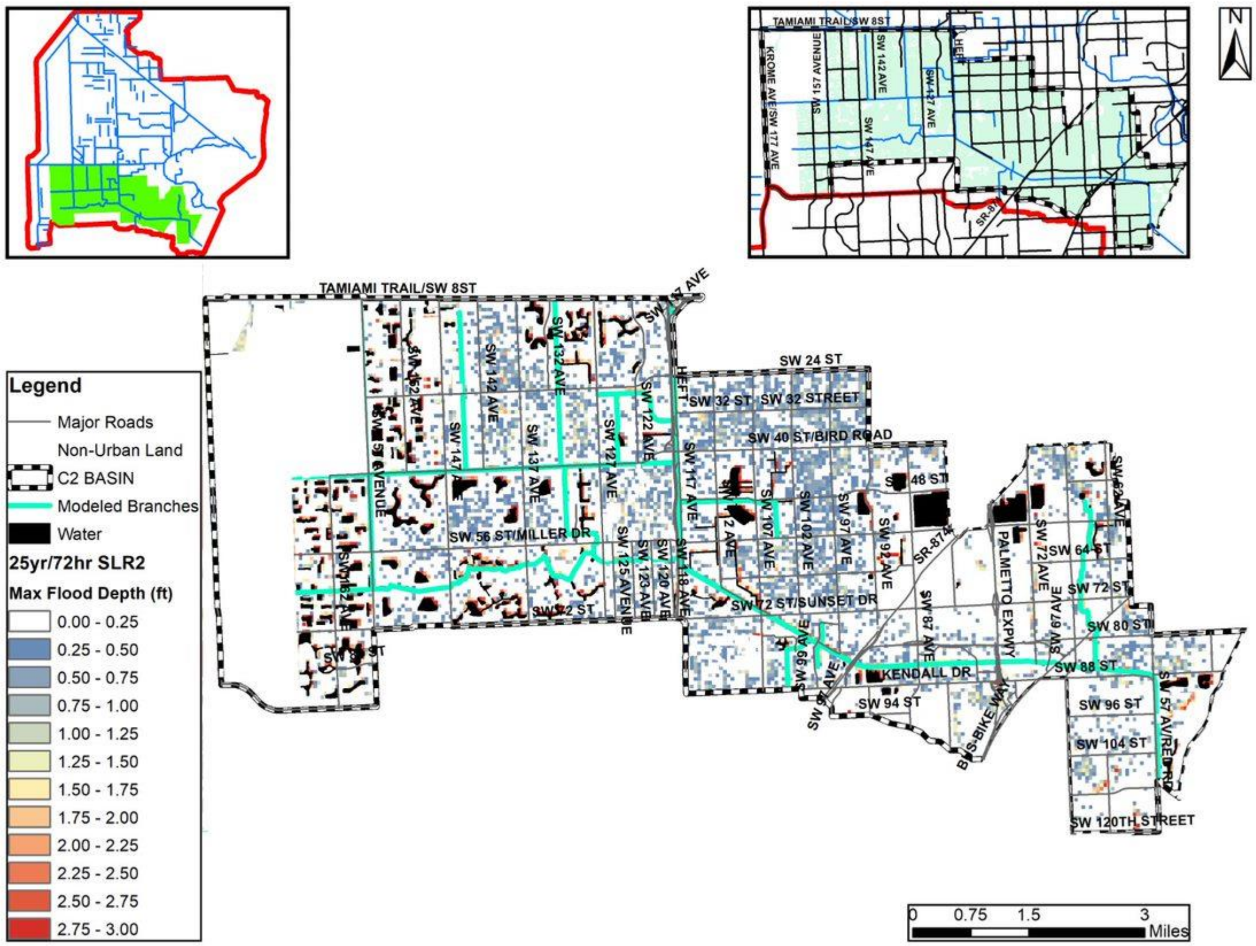

Figure C 1-27. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR3 in Urban Areas in the C2 Watershed

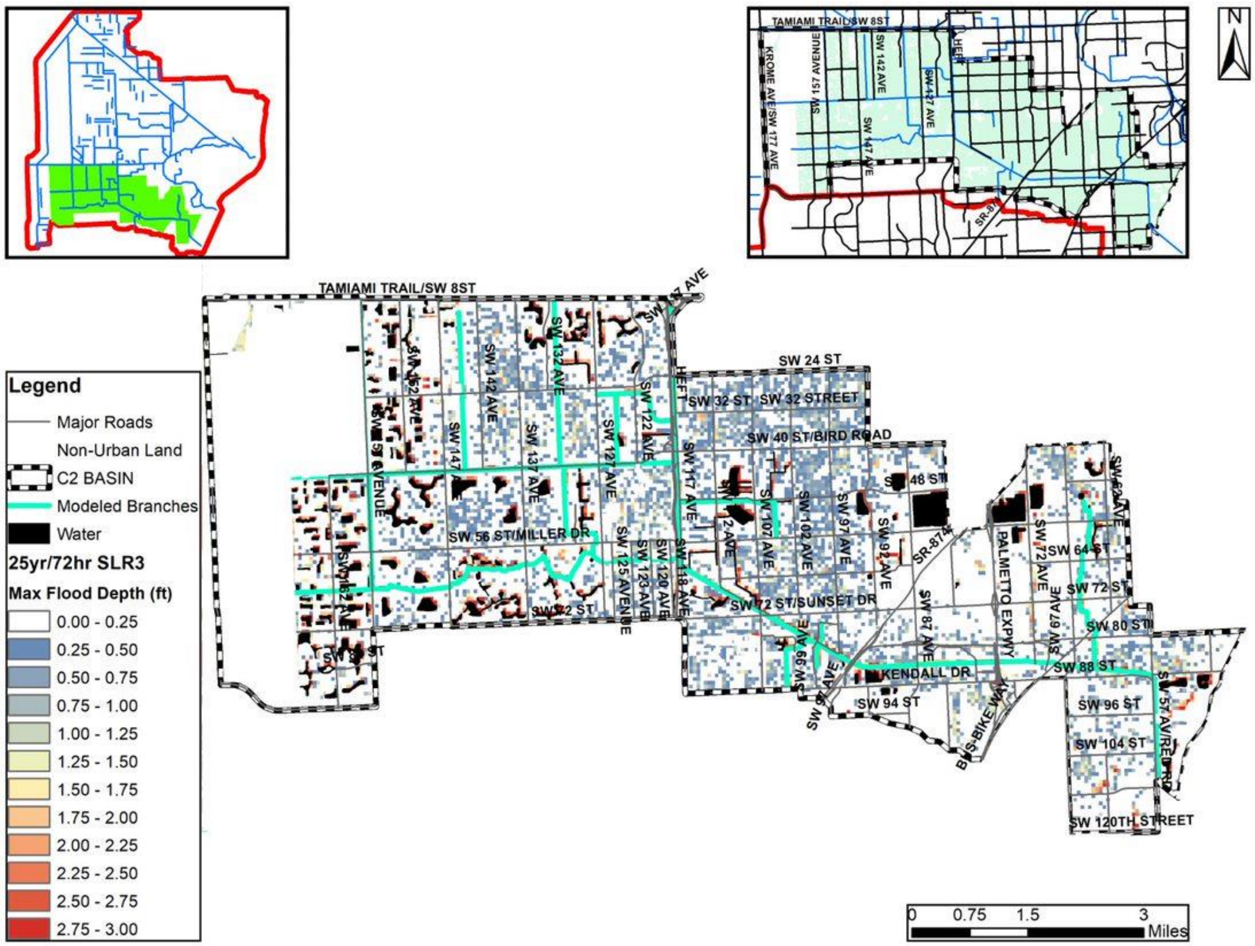

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Figure C 1-29. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR1 in Urban Areas in the C2 Watershed

Figure C 1-30. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR2 in Urban Areas in the C2 Watershed

Figure C 1-31. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR3 in Urban Areas in the C2 Watershed

Figure C 1-32. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C2 Watershed

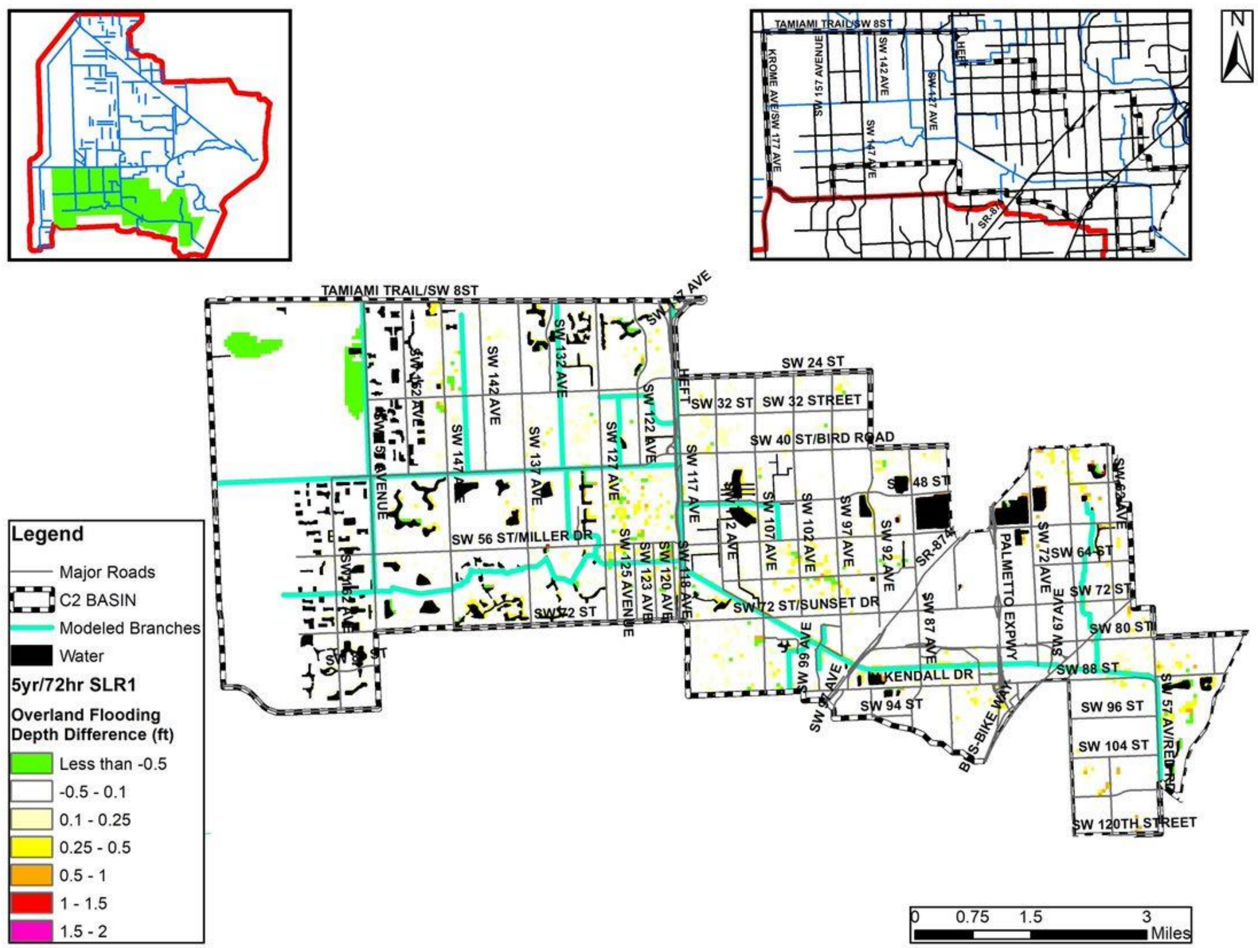


Figure C 1-33. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C2 Watershed

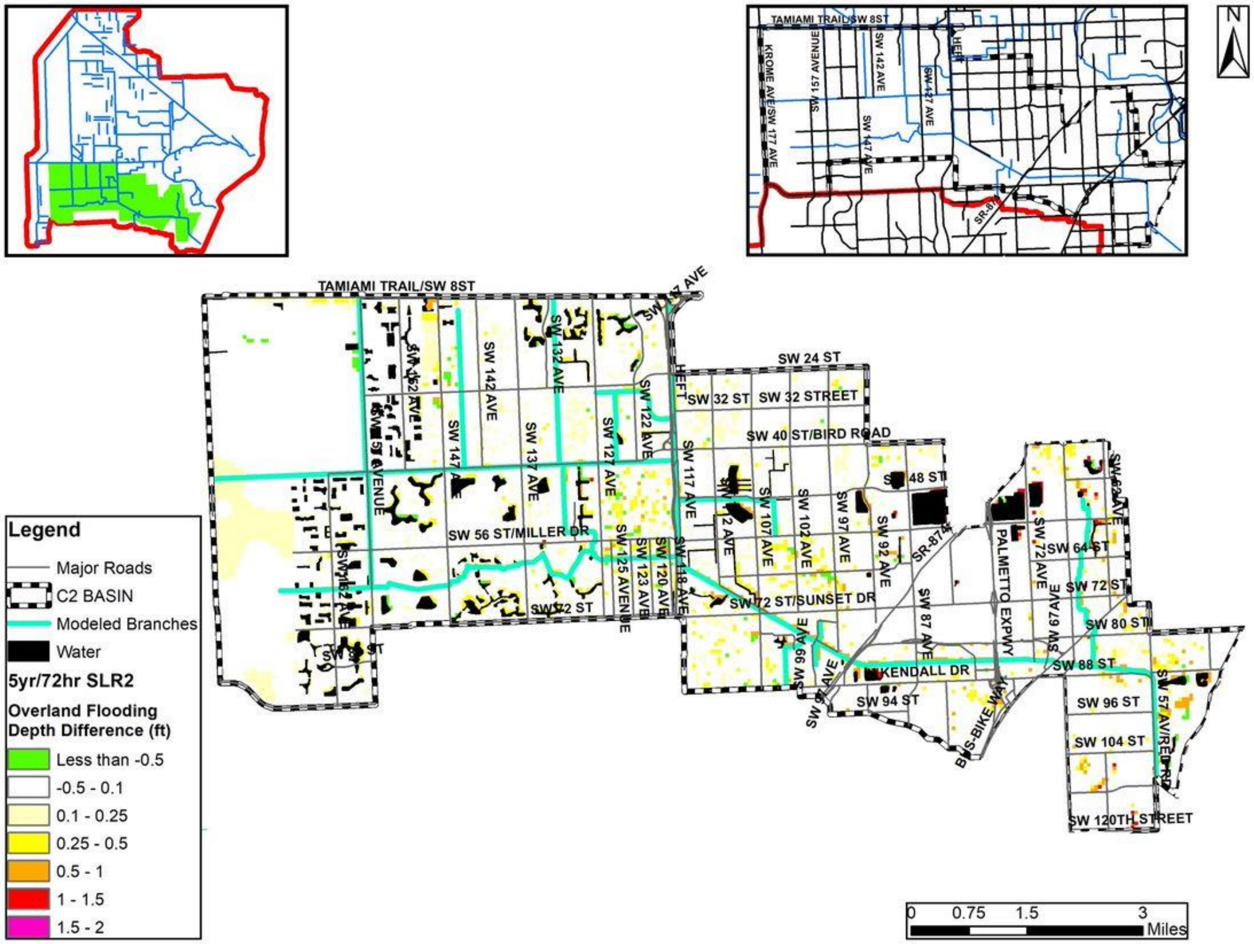


Figure C 1-34. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C2 Watershed

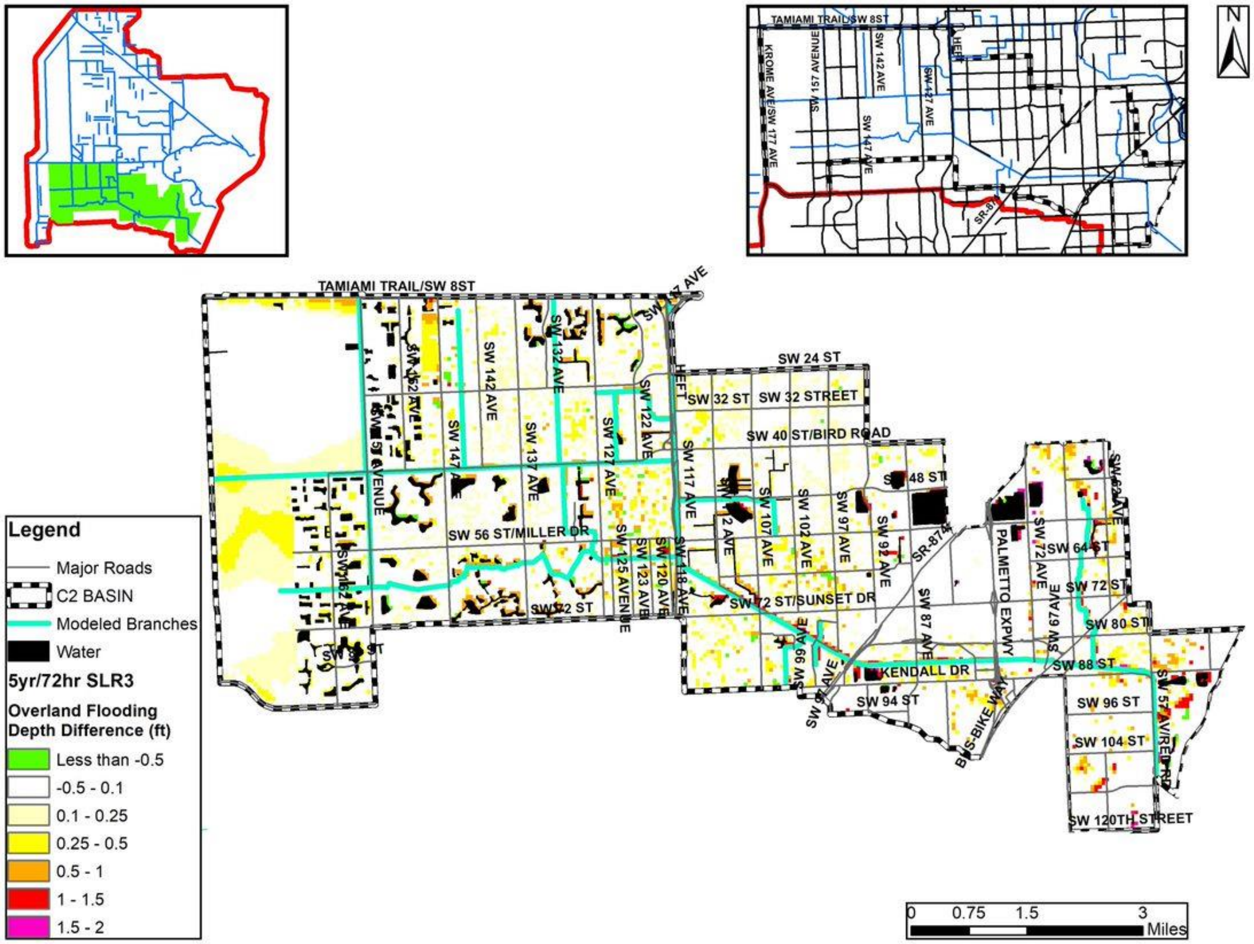


Figure C 1-35. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C2 Watershed

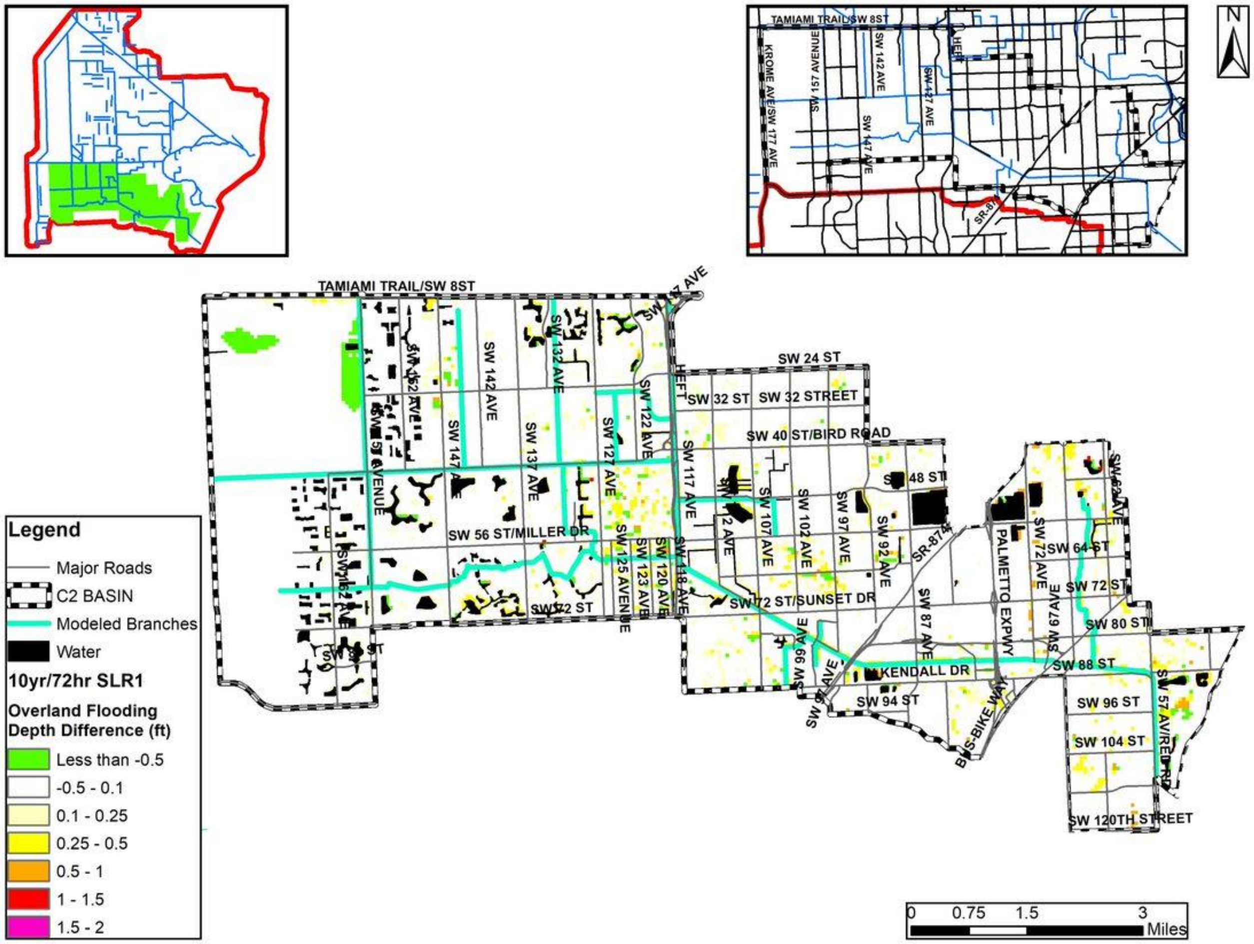


Figure C 1-36. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C2 Watershed

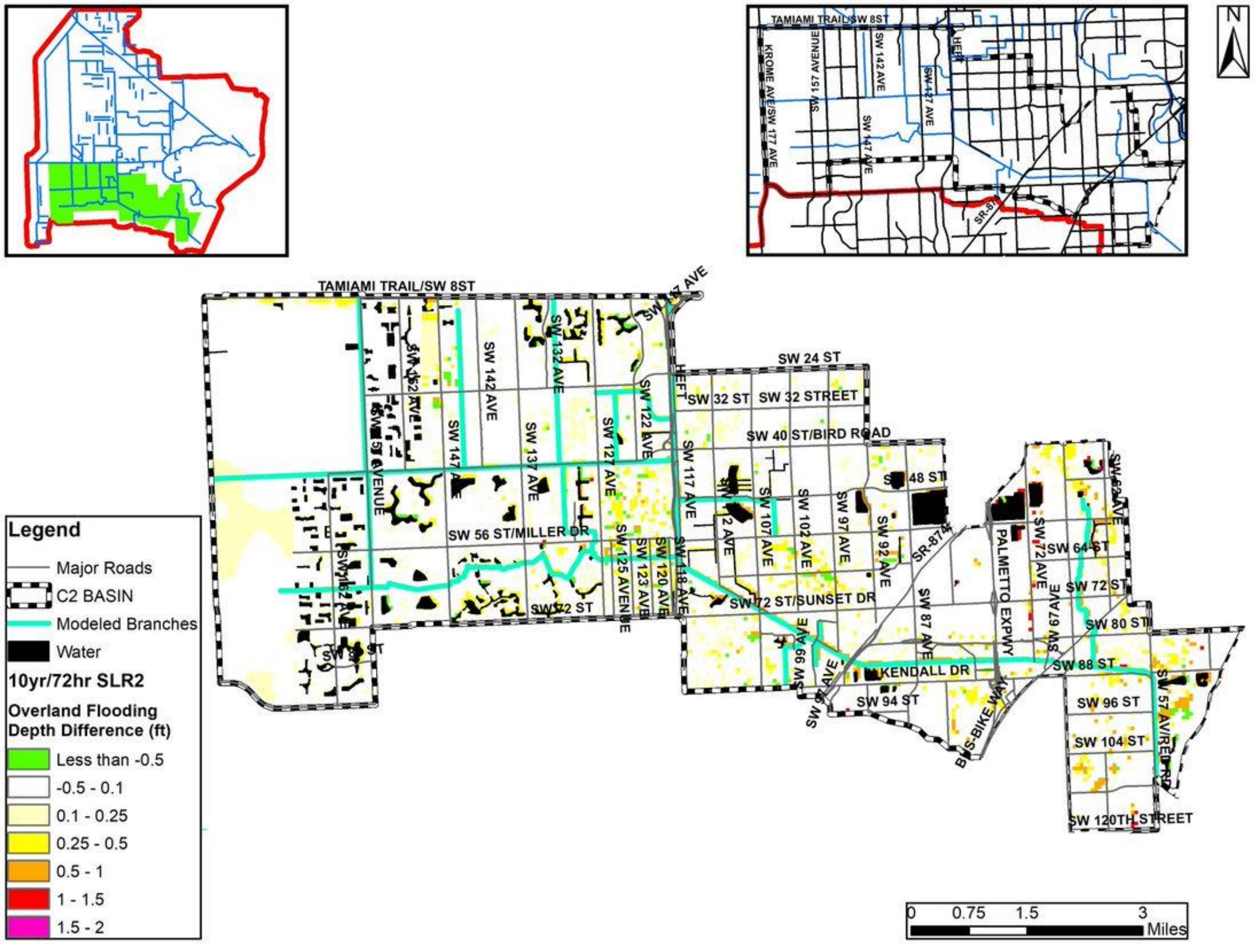


Figure C 1-37. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C2 Watershed

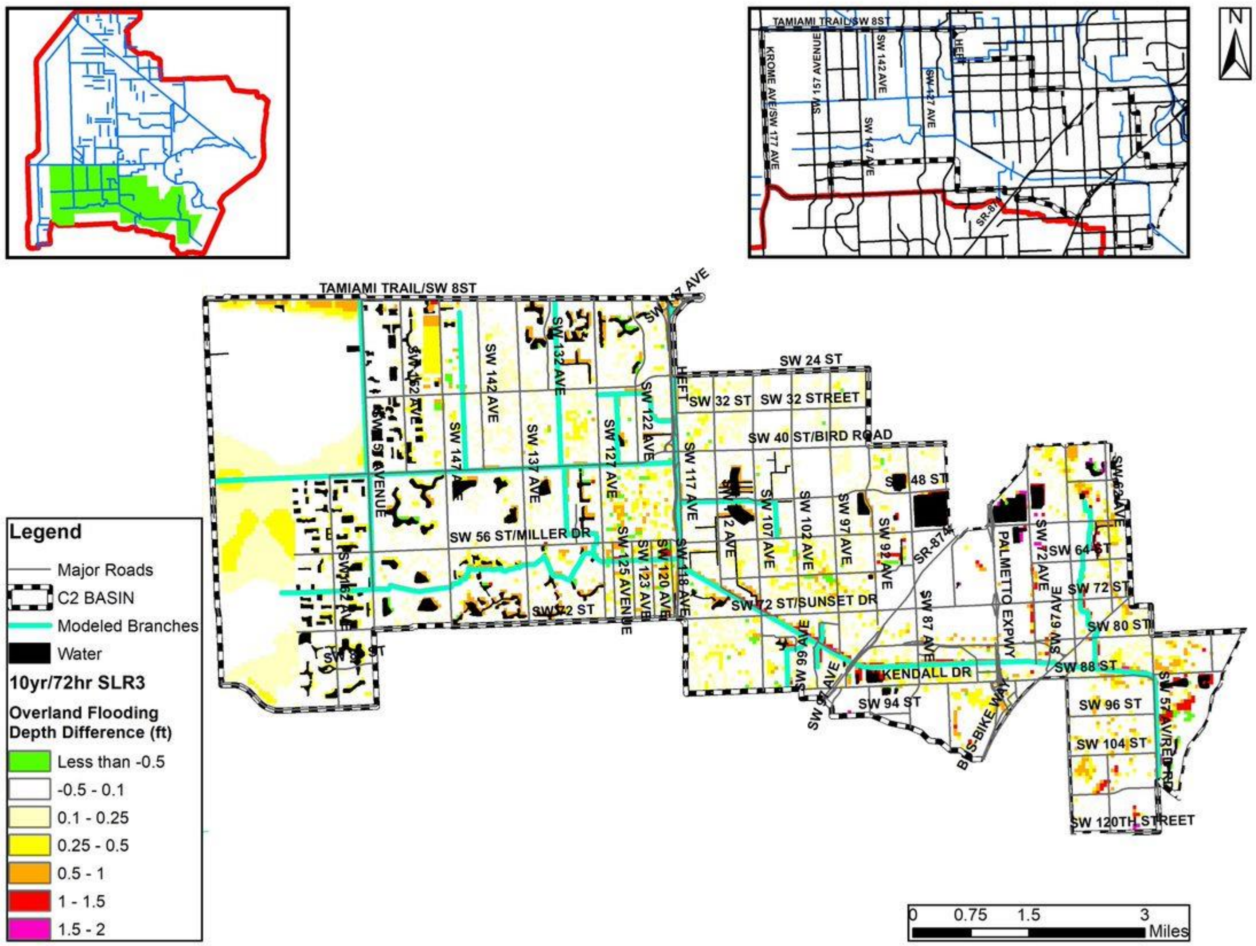


Figure C 1-38. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C2 Watershed

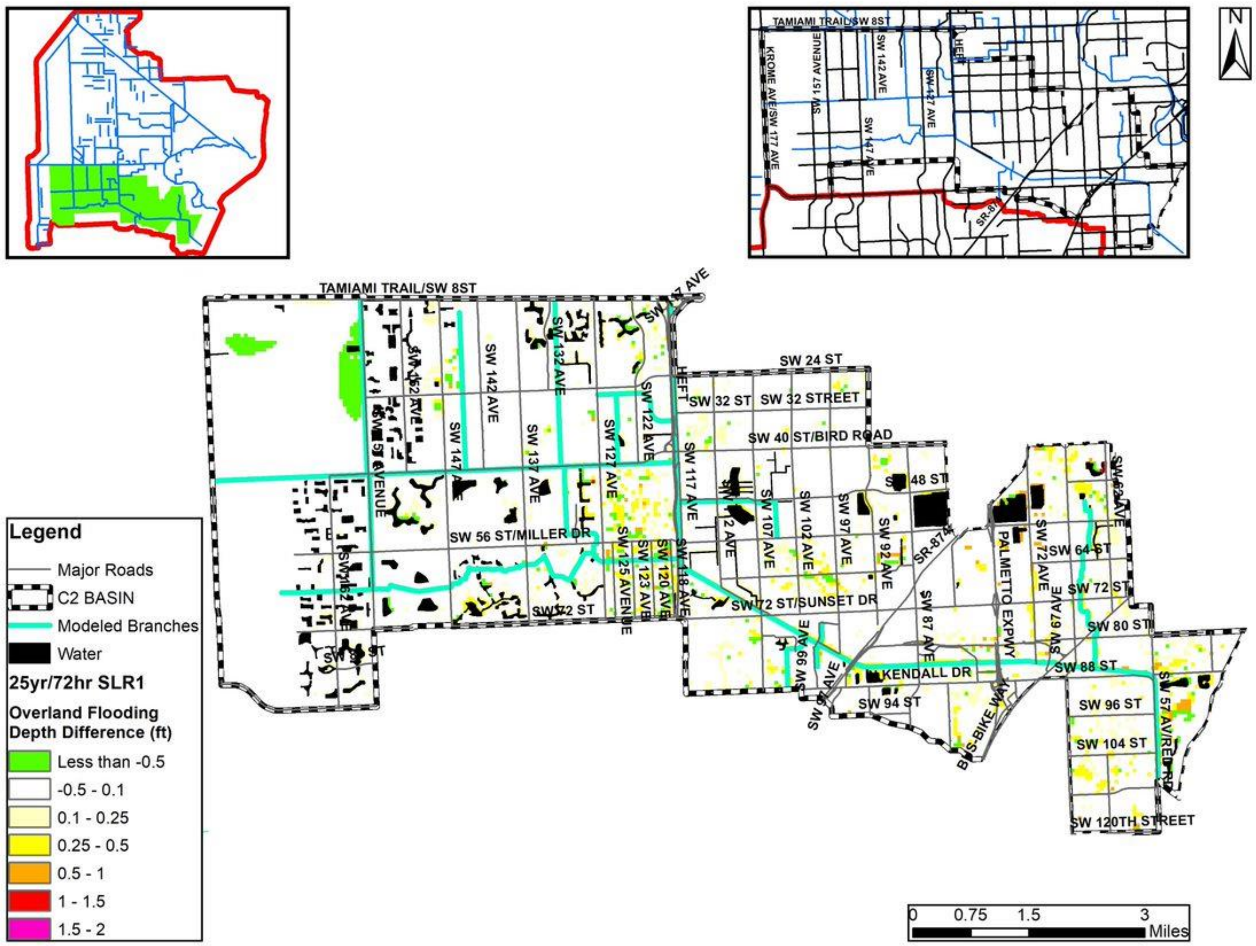


Figure C 1-39. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C2 Watershed

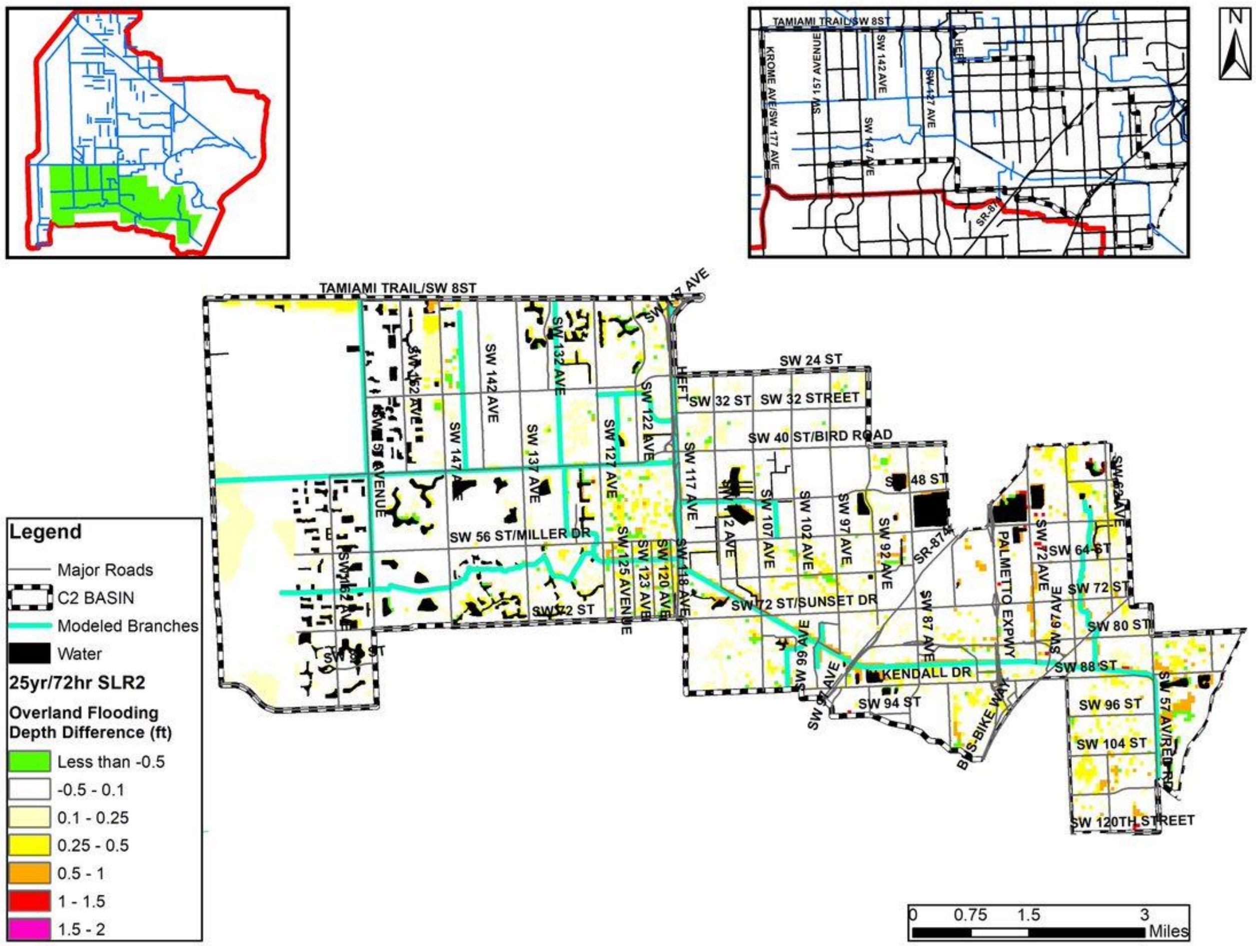


Figure C 1-40. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C2 Watershed

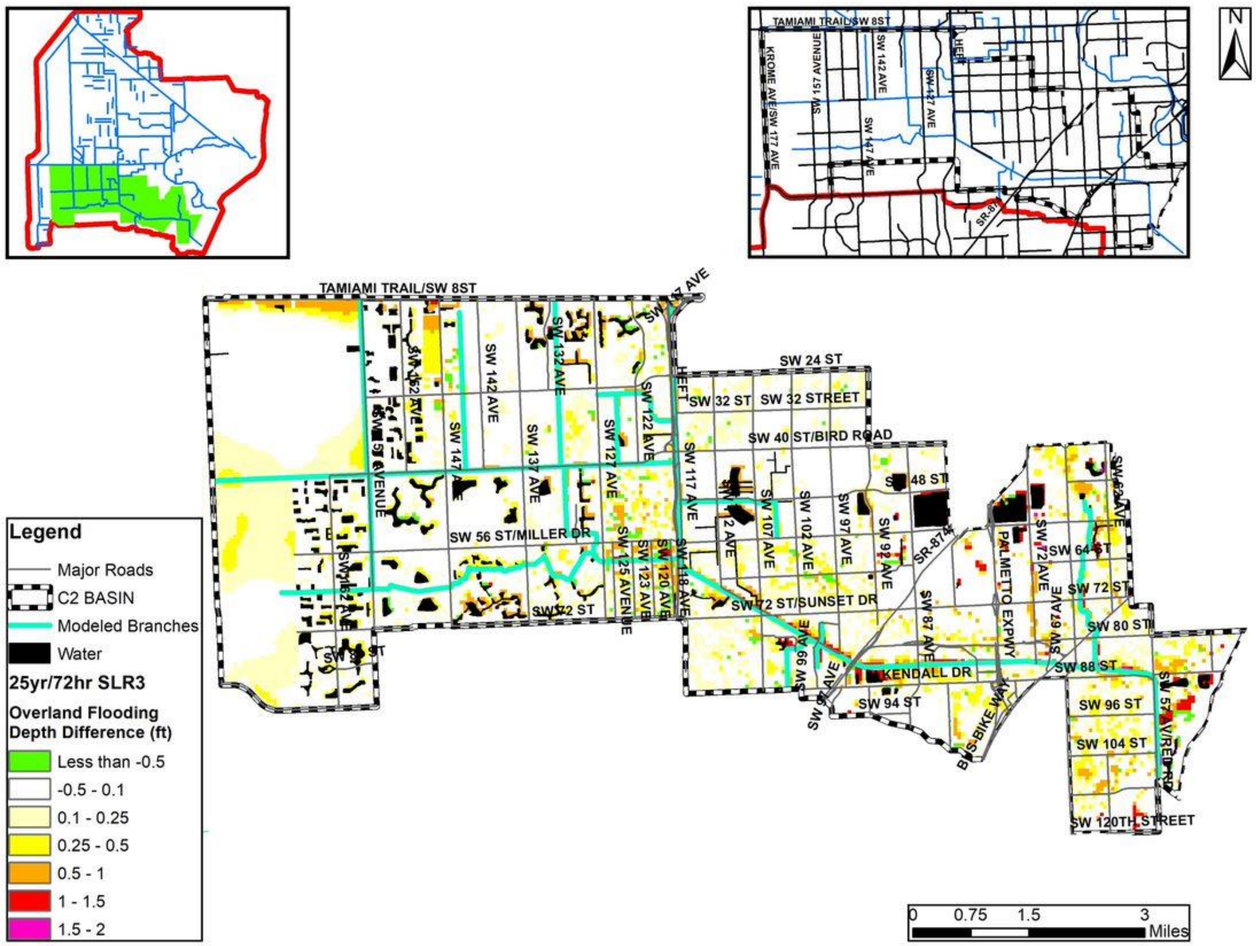


Figure C 1-41. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C2 Watershed

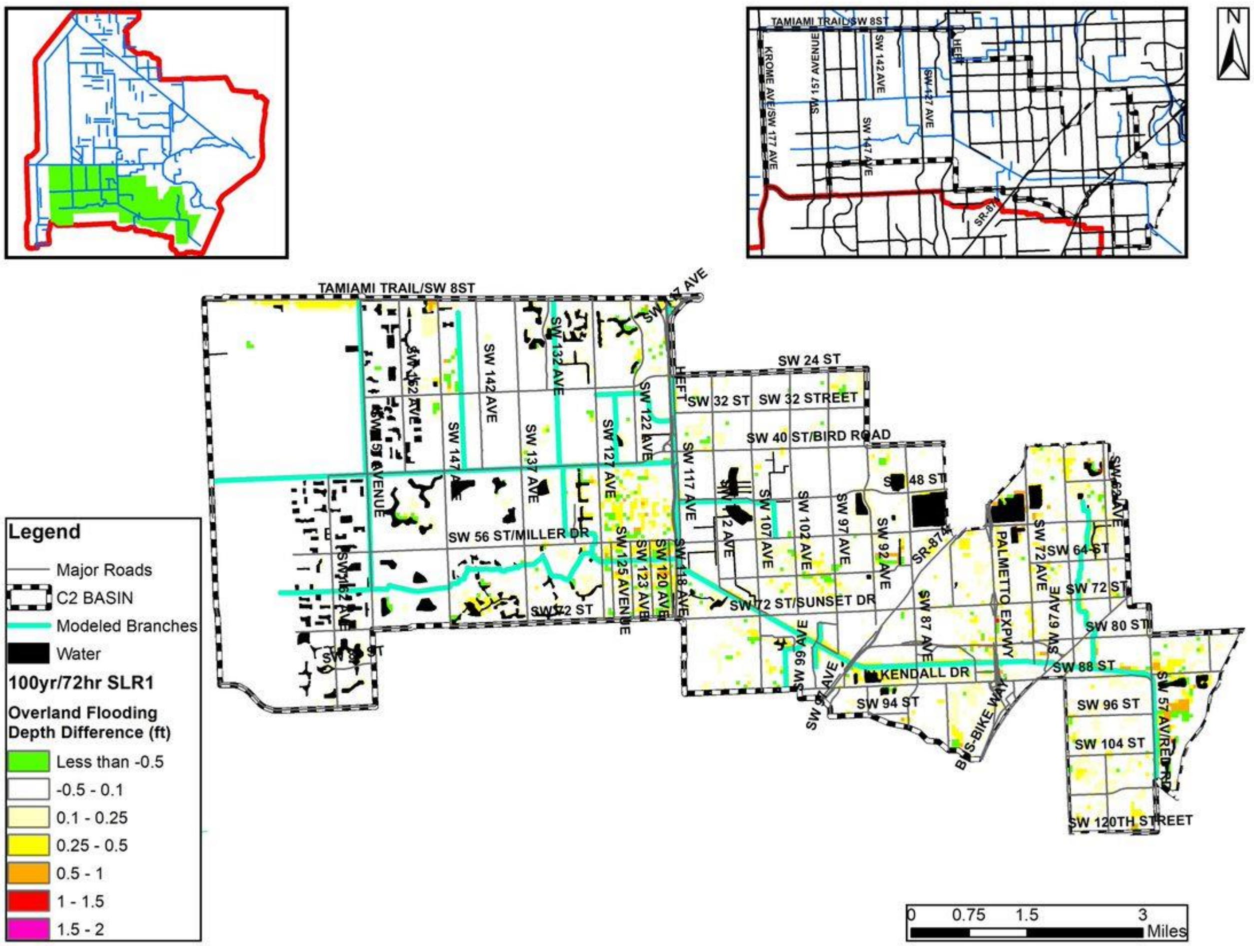


Figure C 1-42. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C2 Watershed

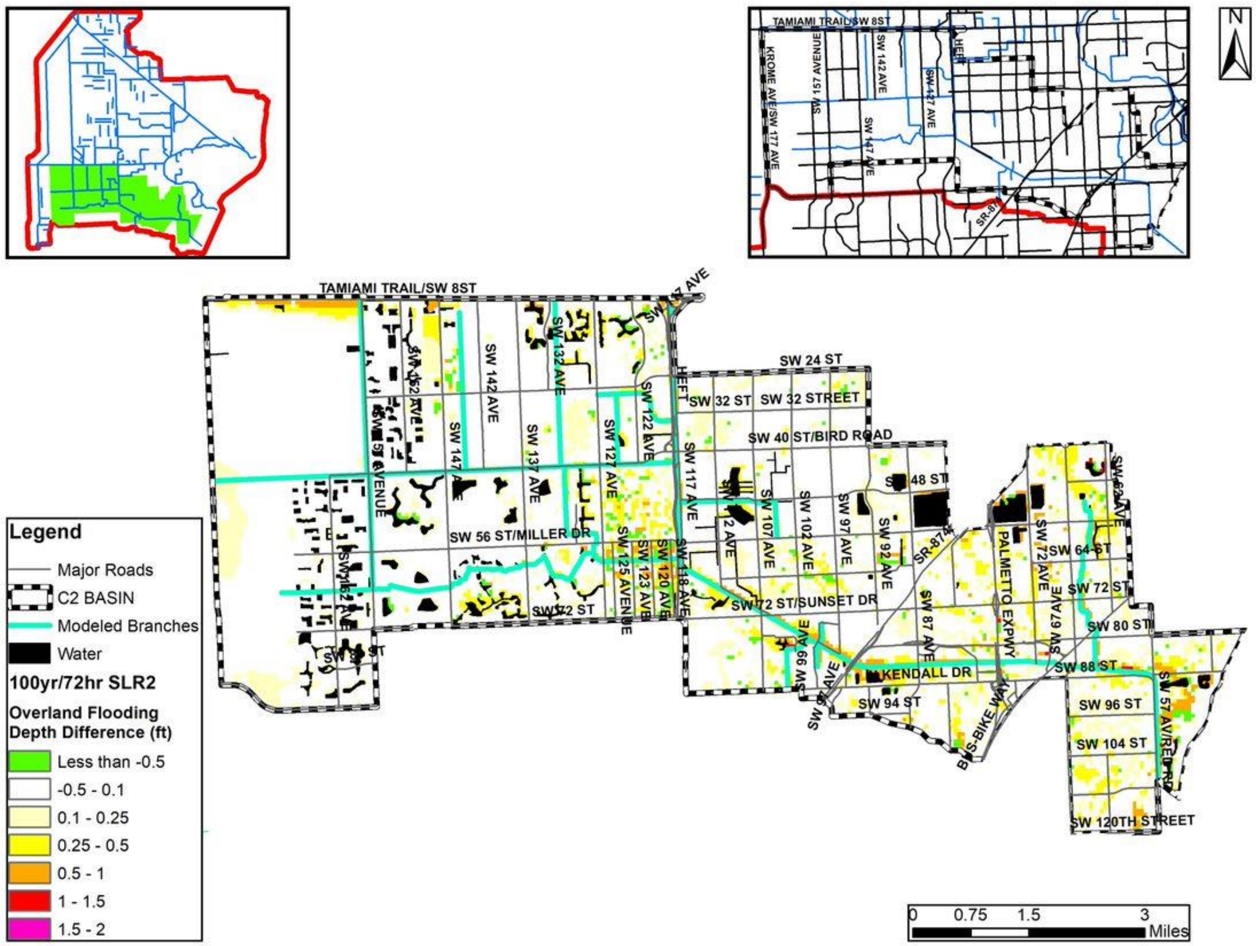


Figure C 1-43. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C2 Watershed

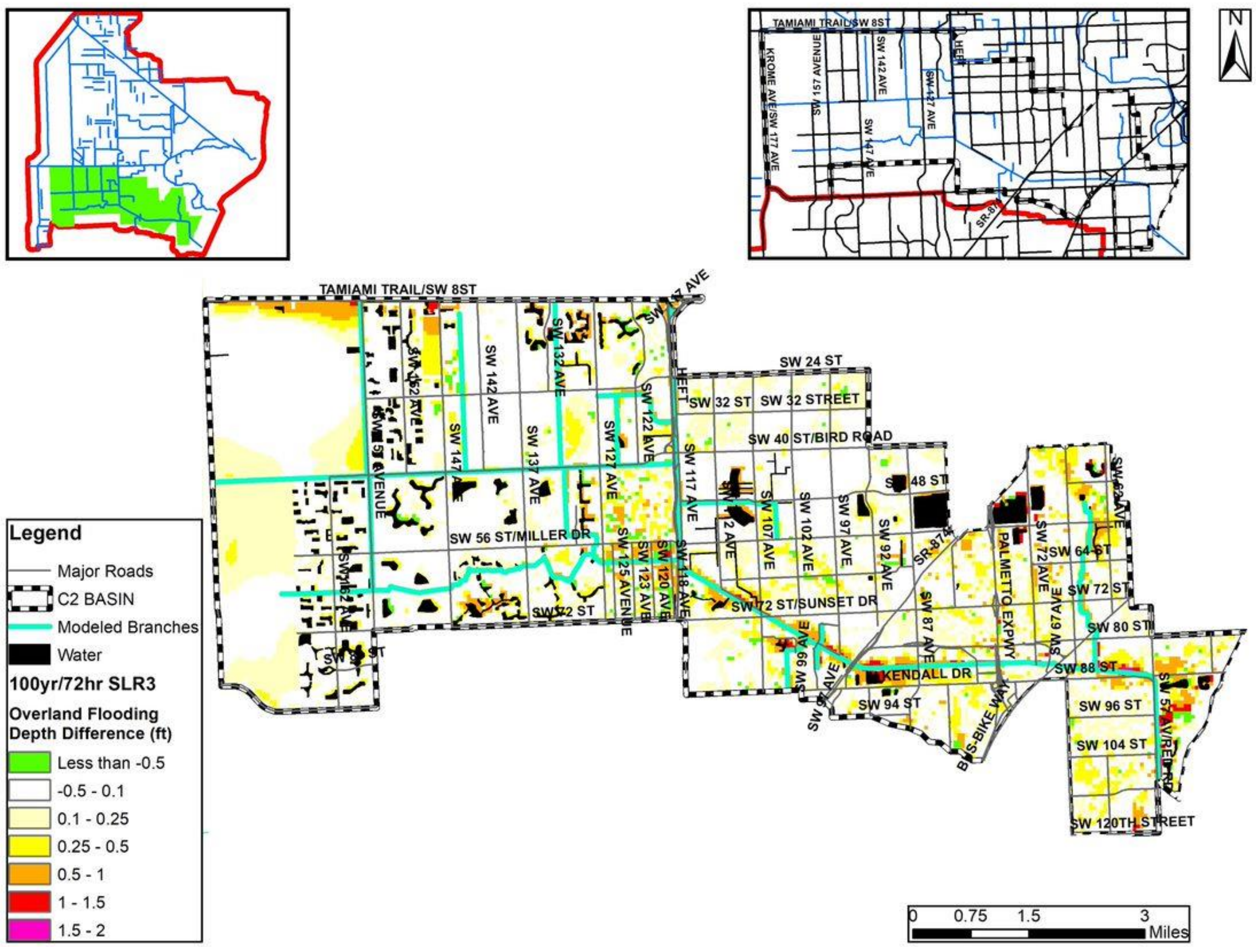


Figure C 1-44. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure C 1-45. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C2 Watershed

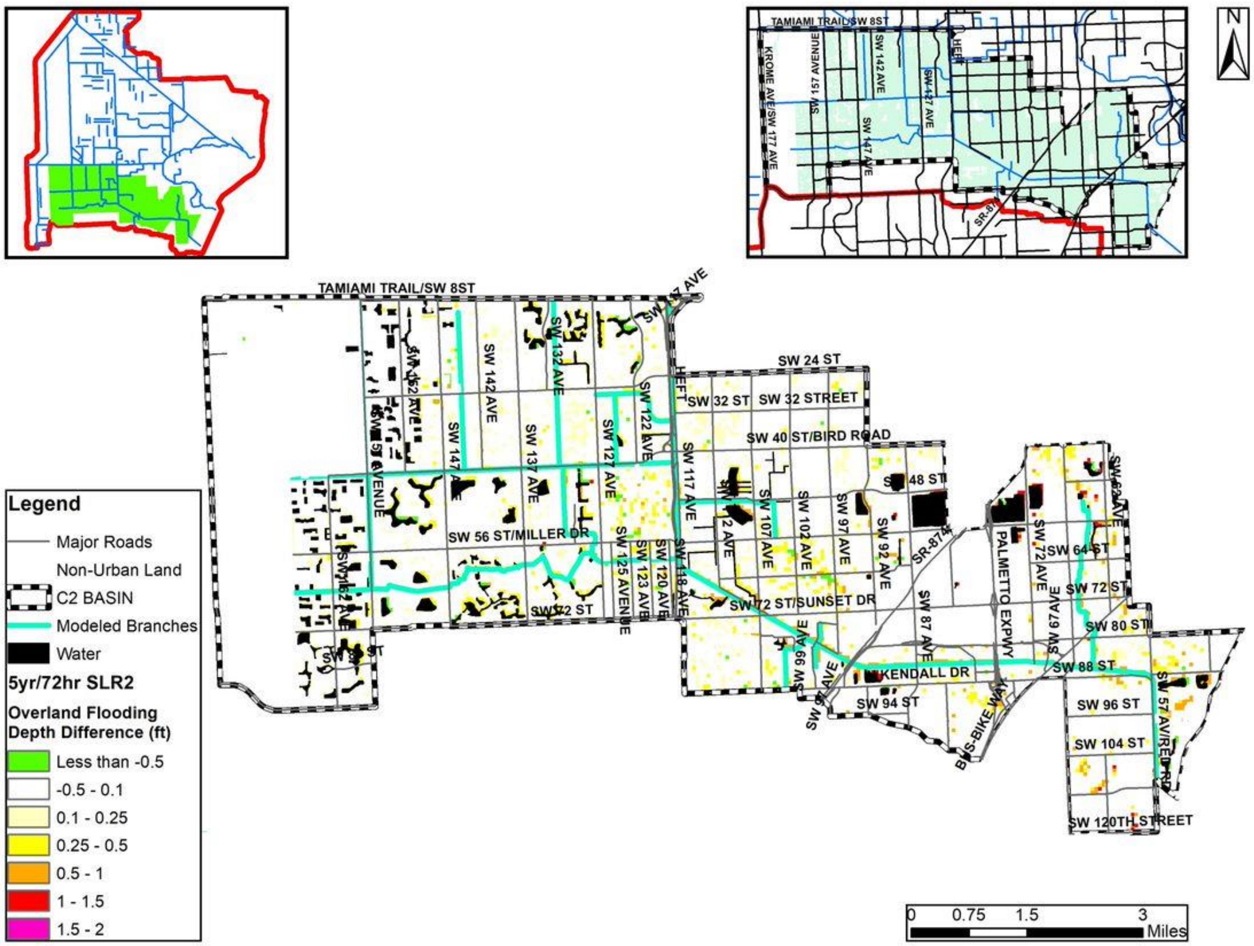


Figure C 1-46. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C2 Watershed

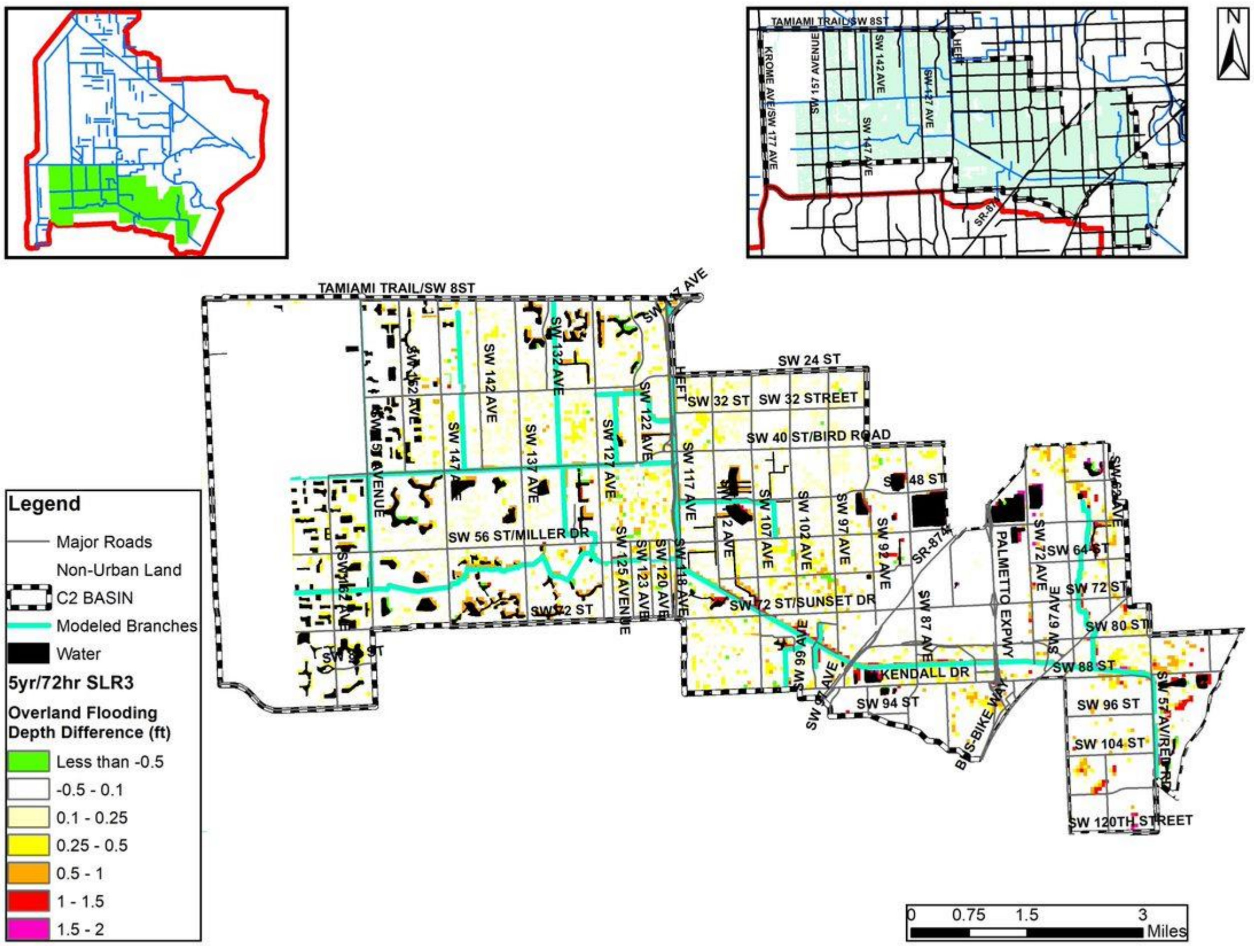


Figure C 1-47. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure C 1-48. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C2 Watershed

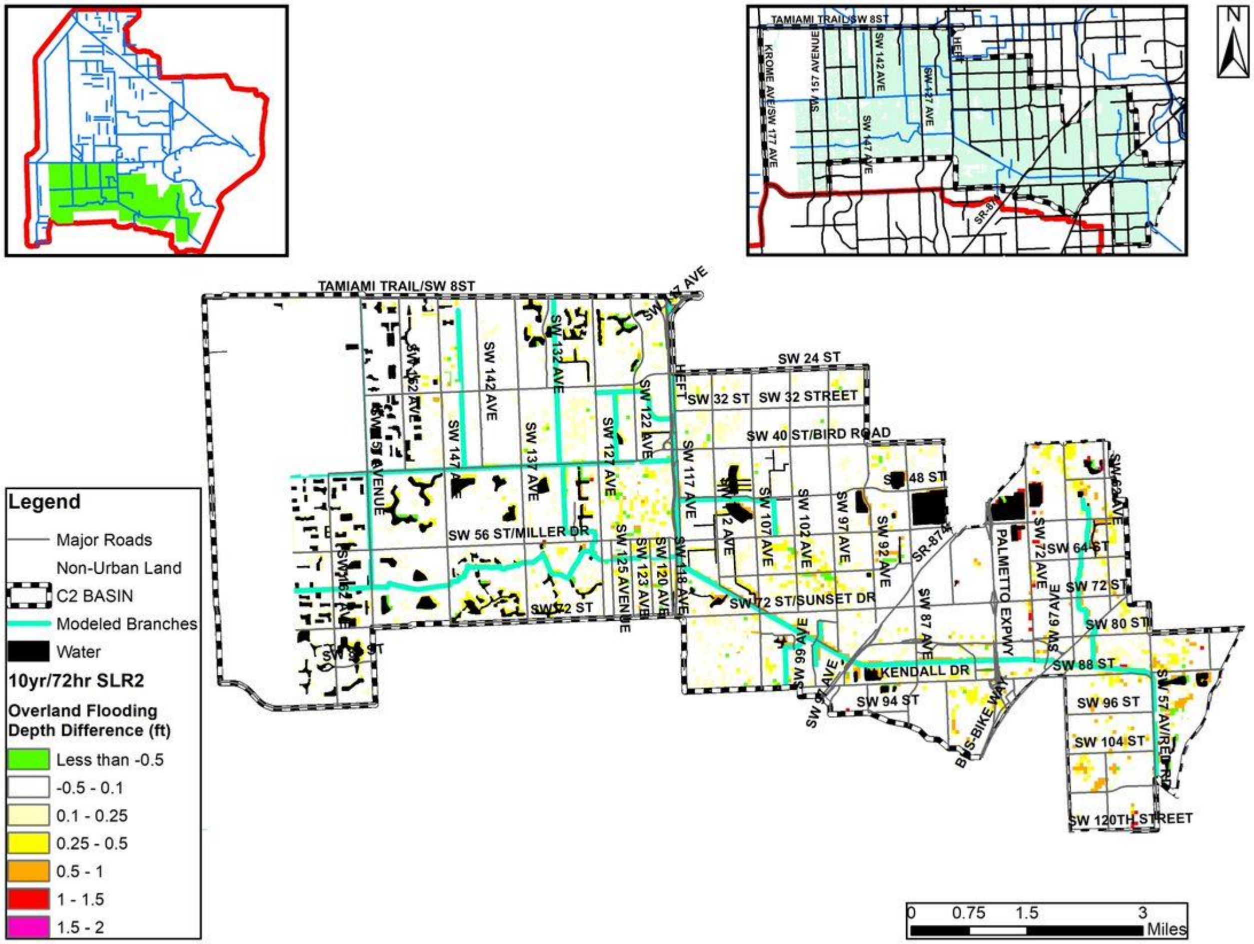


Figure C 1-49. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C2 Watershed

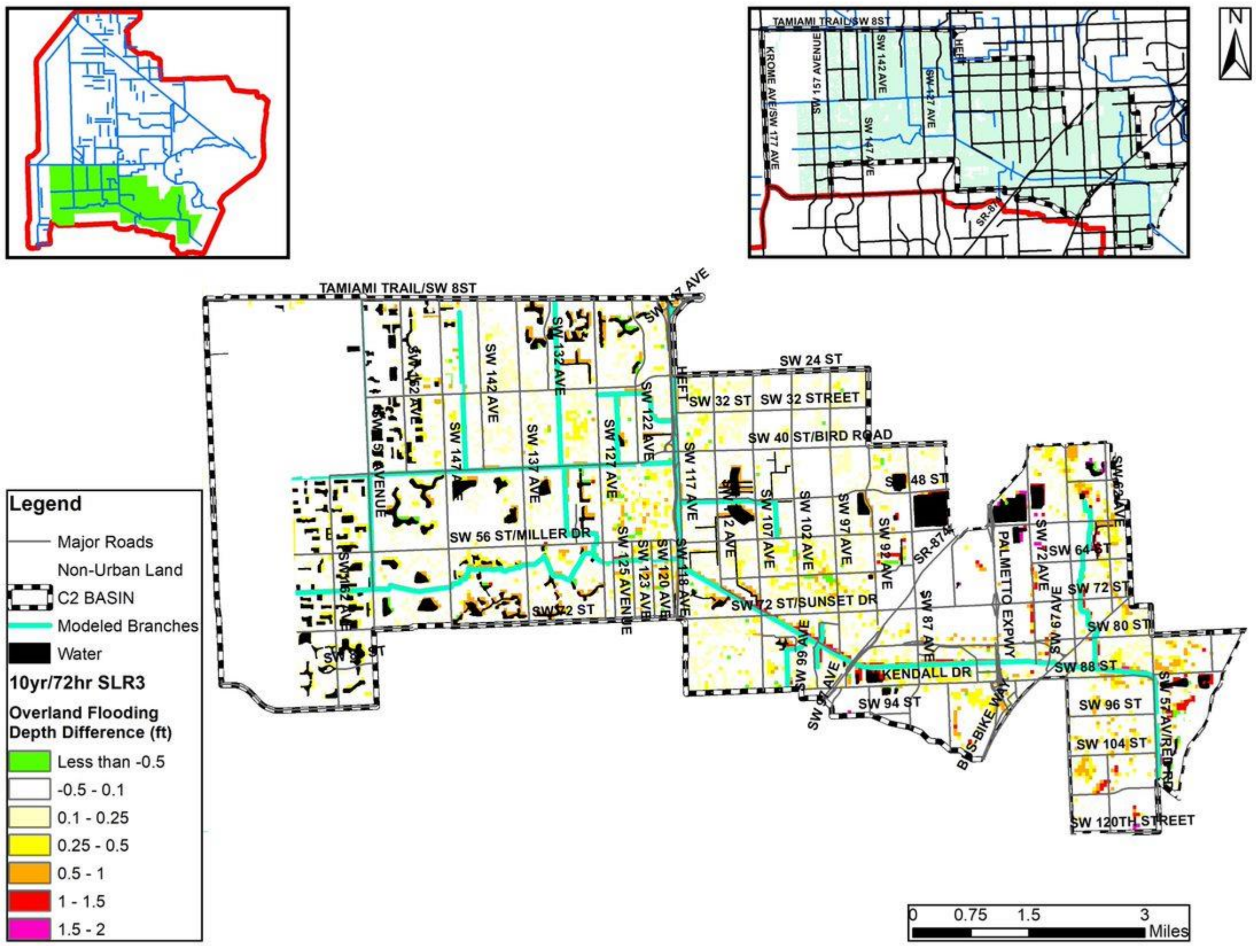


Figure C 1-50. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure C 1-51. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure C 1-52. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure C 1-53. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C2 Watershed

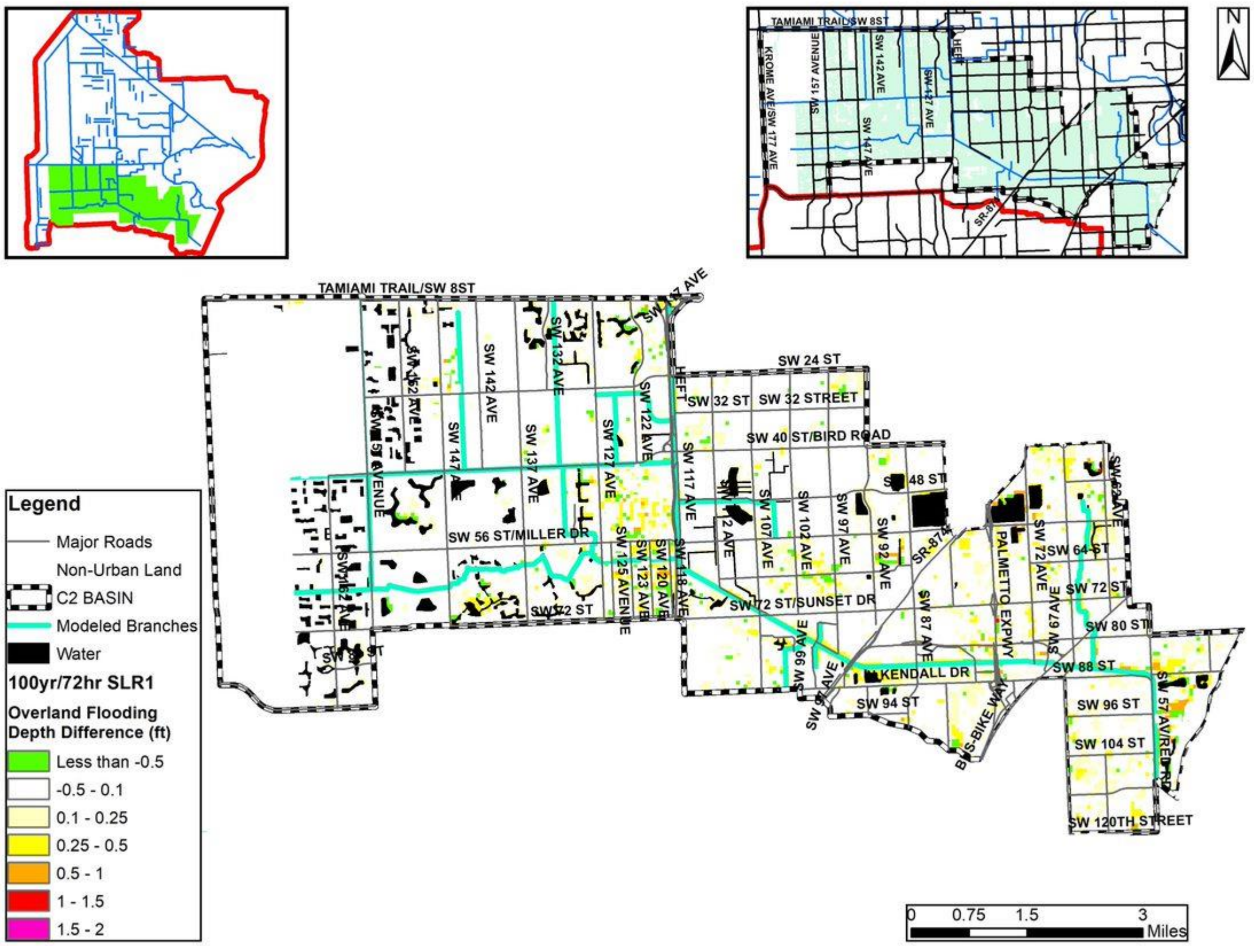
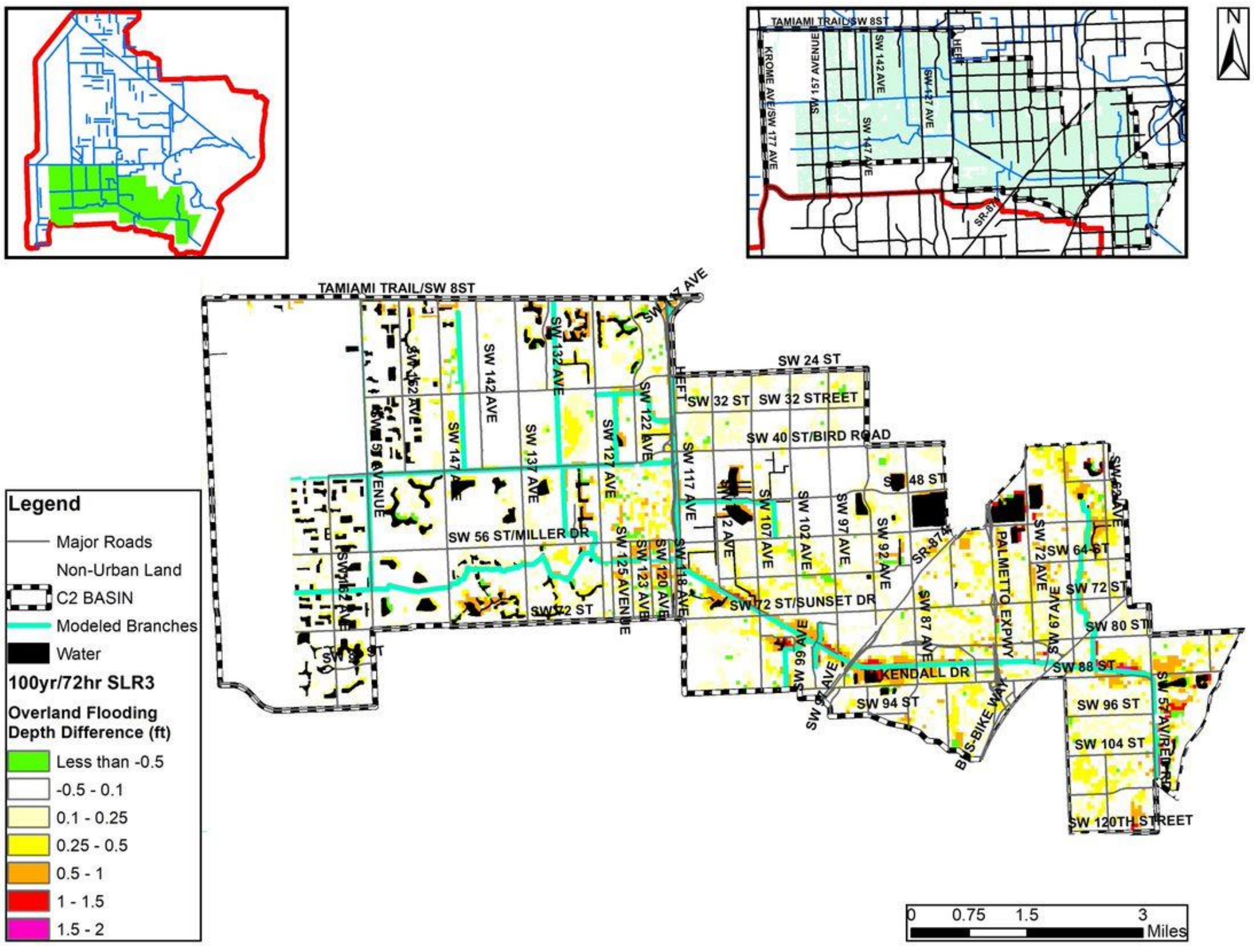


Figure C 1-54. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure C 1-55. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C2 Watershed



2 C3W Watershed

Figure C 2-1 through **Figure C 2-16** are maps of the maximum overland depth over the entire C3W Watershed for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for each SLR condition. Water areas are masked in black.

Figure C 2-17 through **Figure C 2-28** show the difference in overland flooding for the C3W Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively.

Figure C 2-1. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for Current Conditions in the C3W Watershed

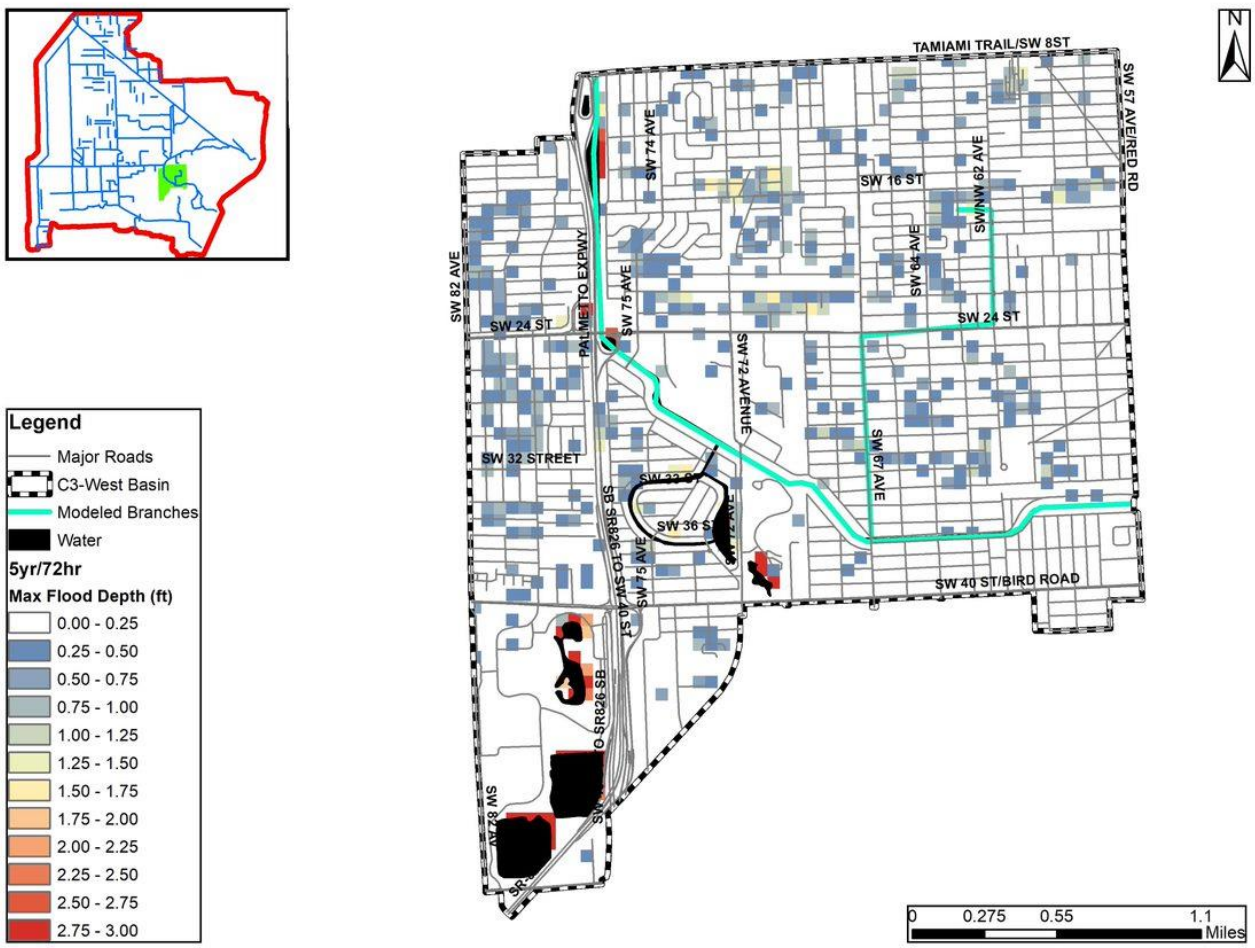


Figure C 2-2. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR1 in the C3W Watershed

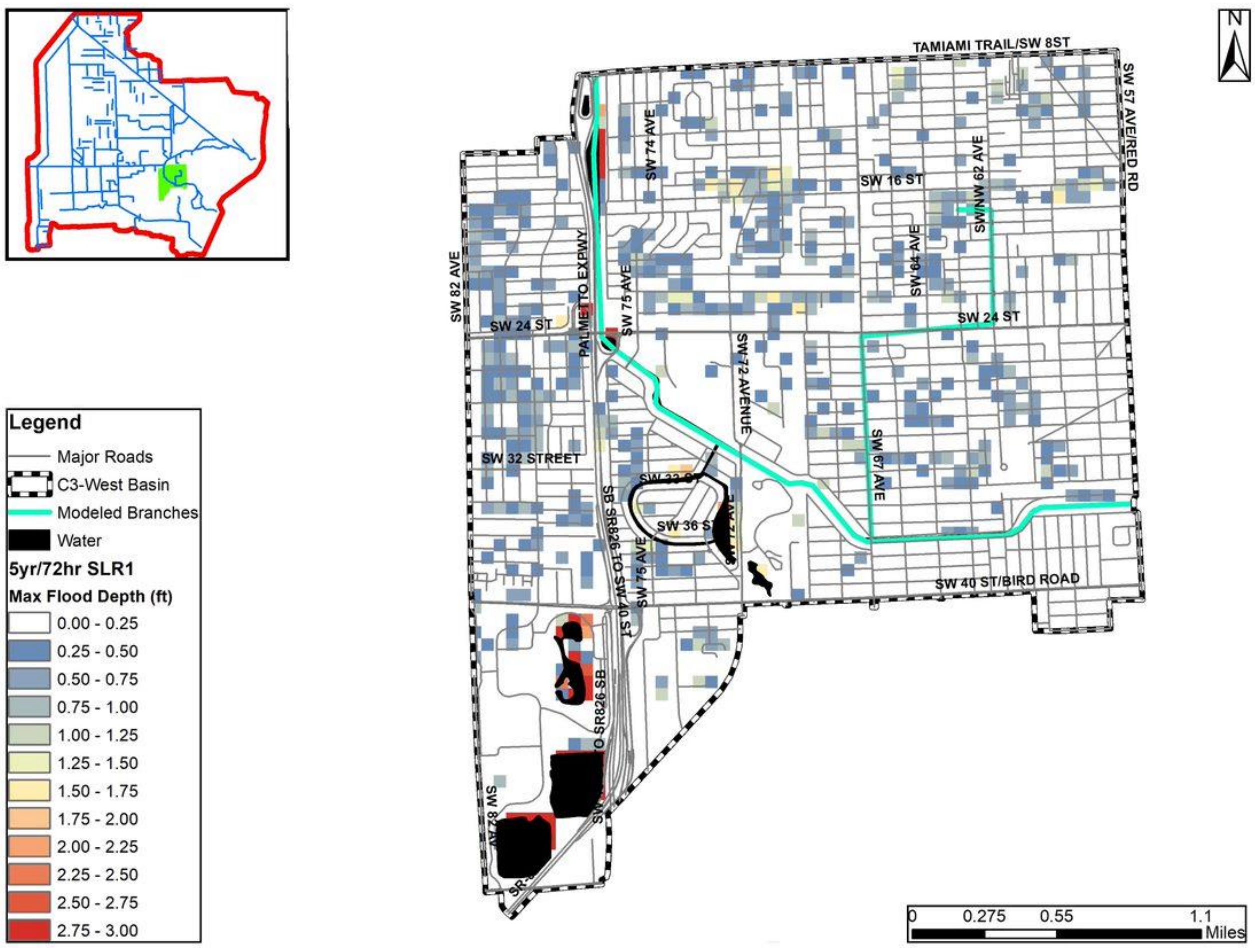


Figure C 2-3. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR2 in the C3W Watershed

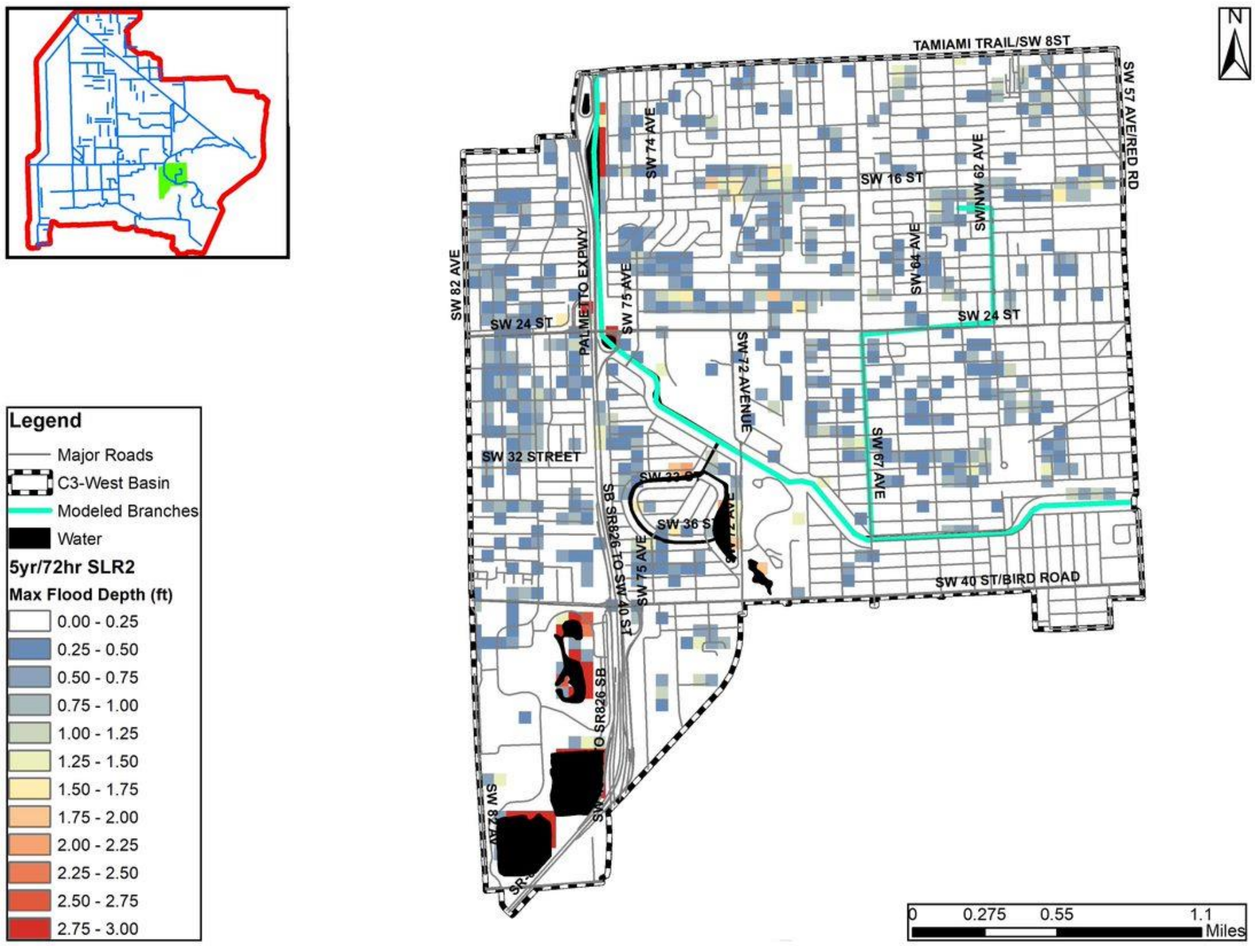


Figure C 2-4. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR3 in the C3W Watershed

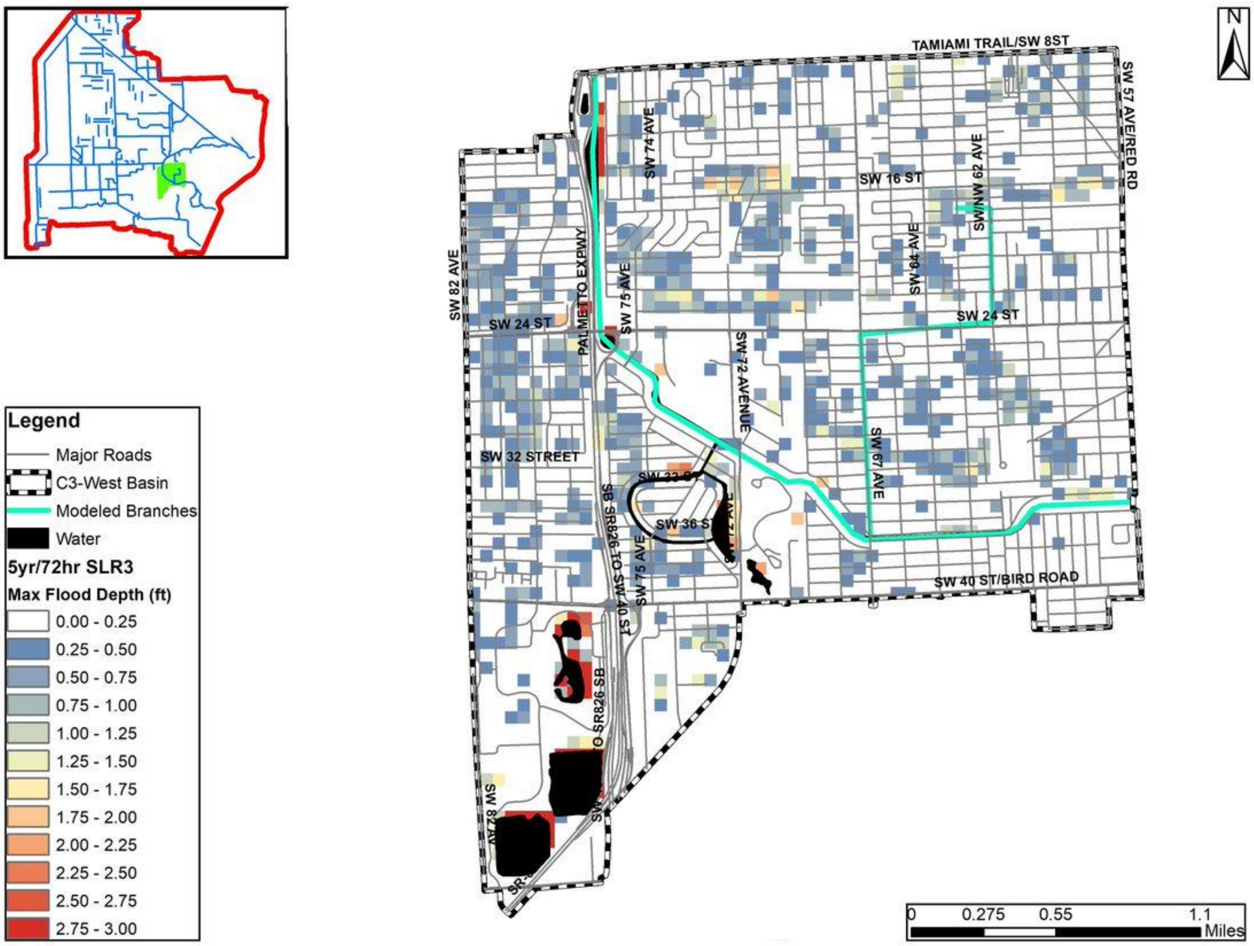


Figure C 2-5. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for Current Conditions in the C3W Watershed

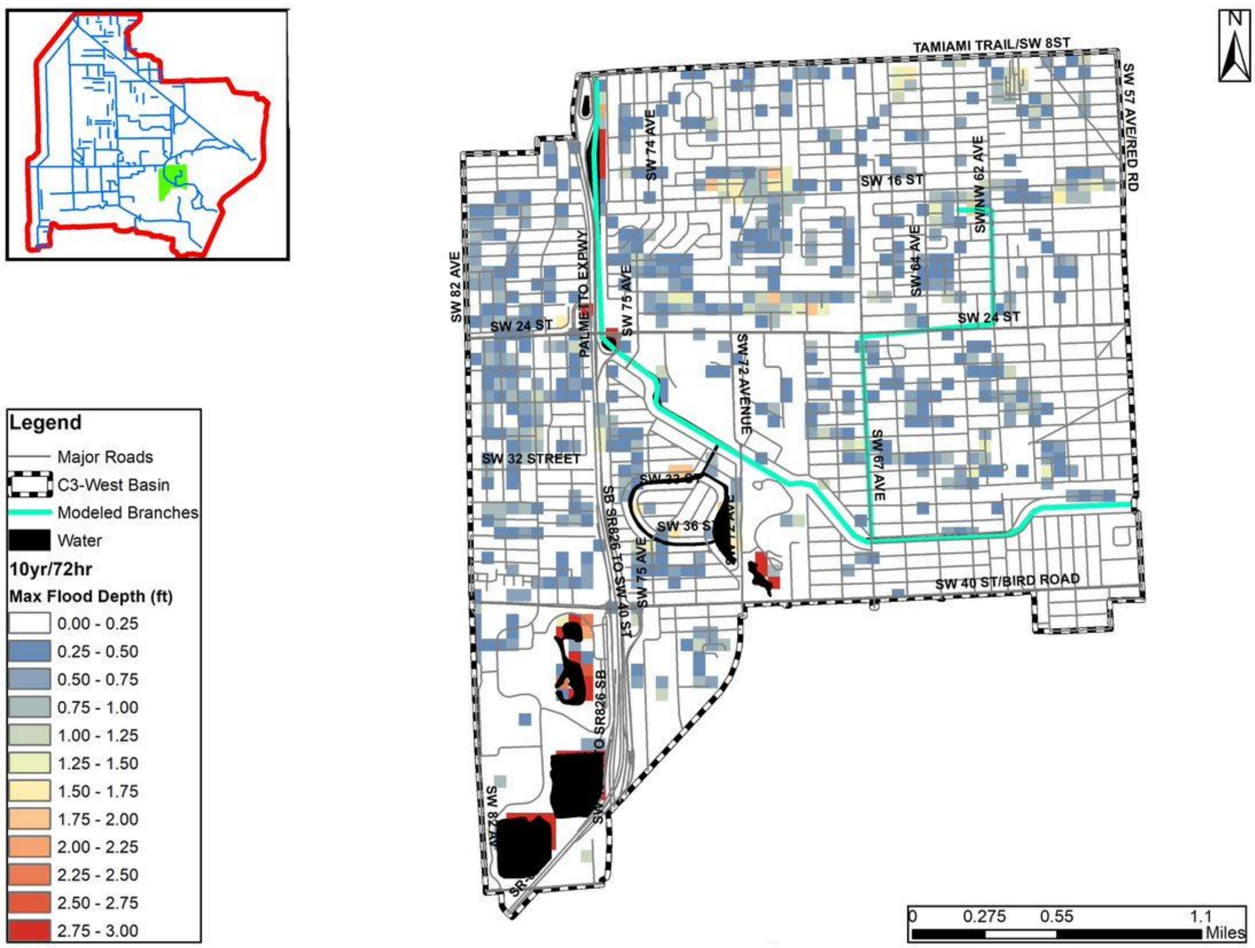


Figure C 2-6. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR1 in the C3W Watershed

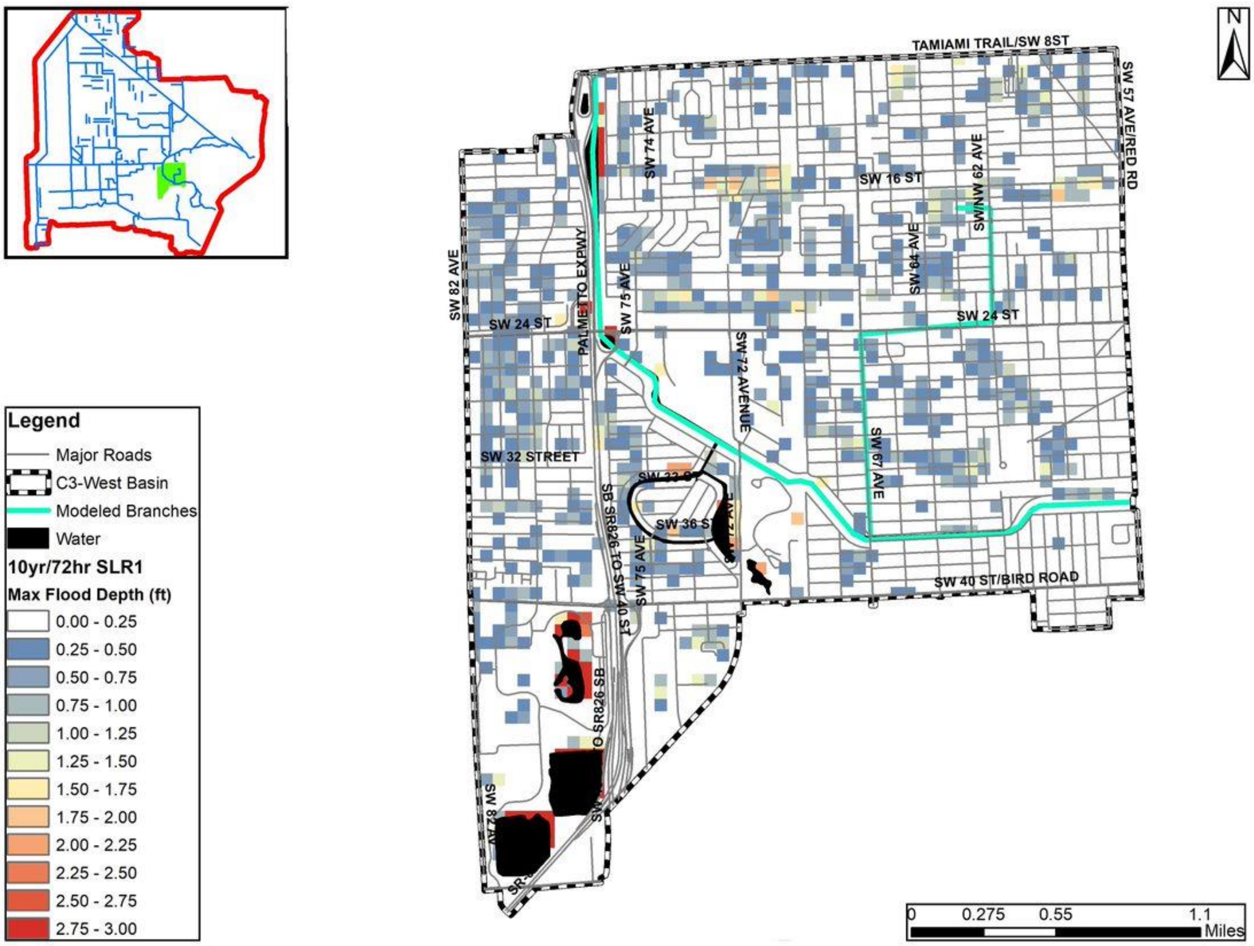


Figure C 2-7. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR2 in the C3W Watershed

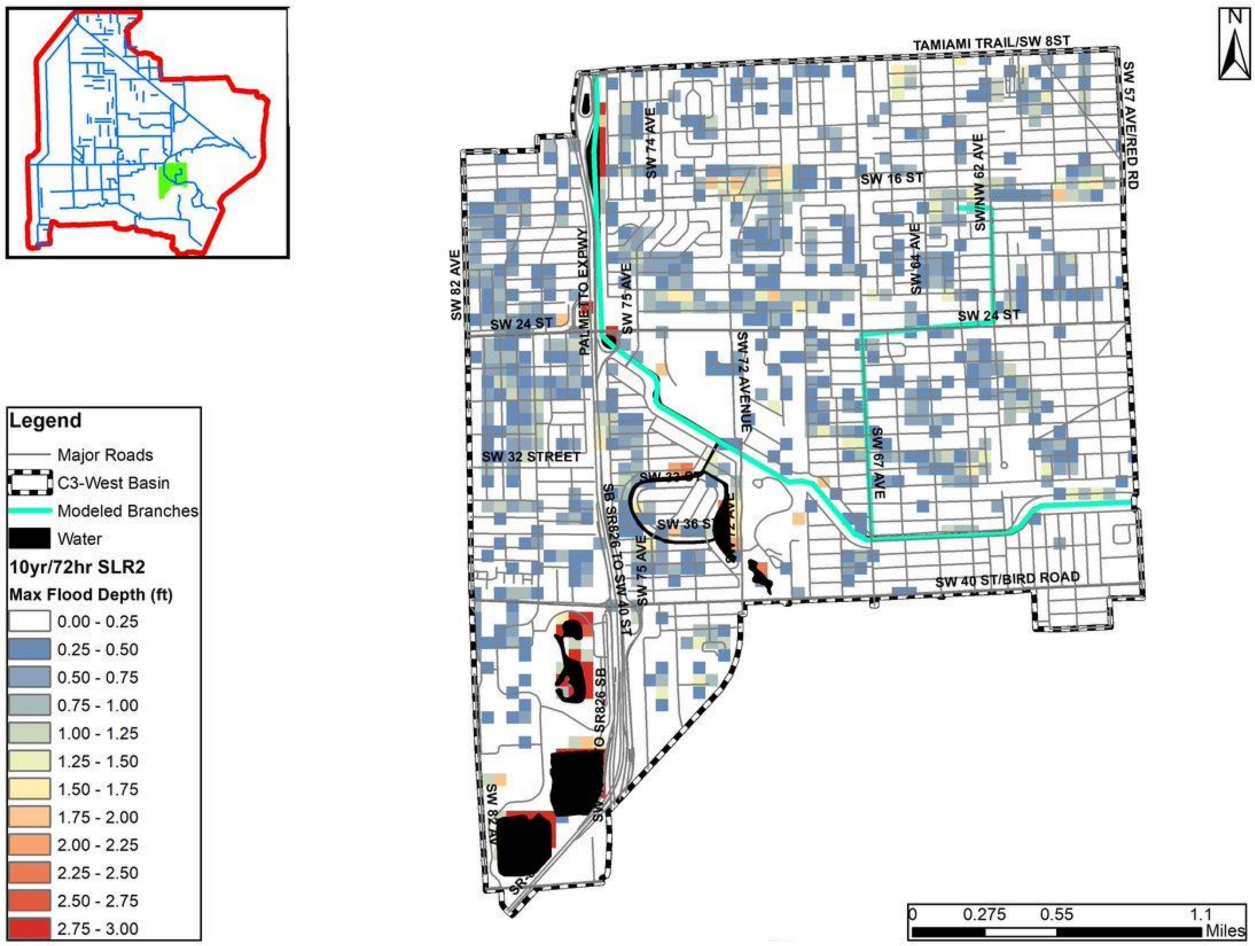


Figure C 2-8. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR3 in the C3W Watershed

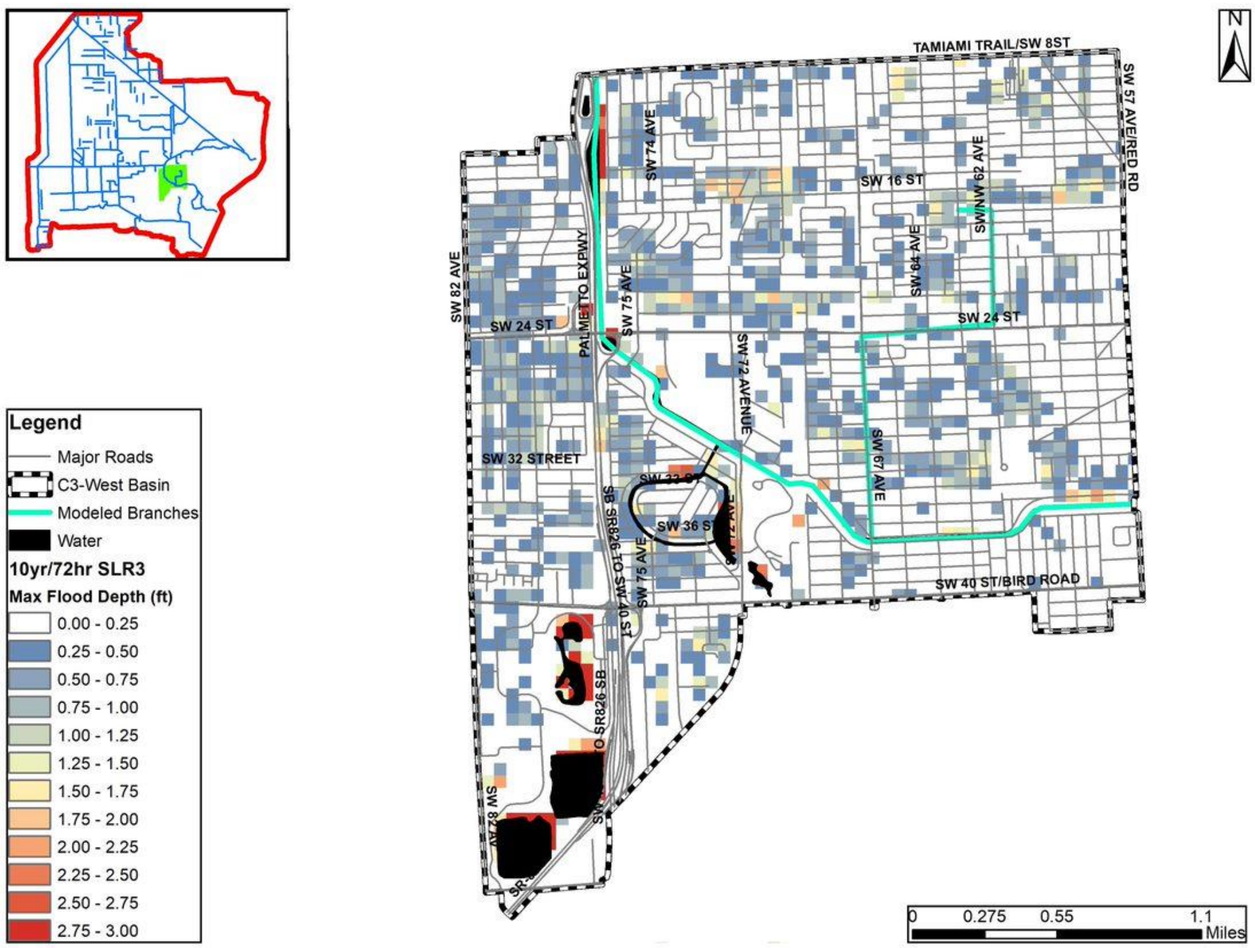


Figure C 2-9. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for Current Conditions in the C3W Watershed

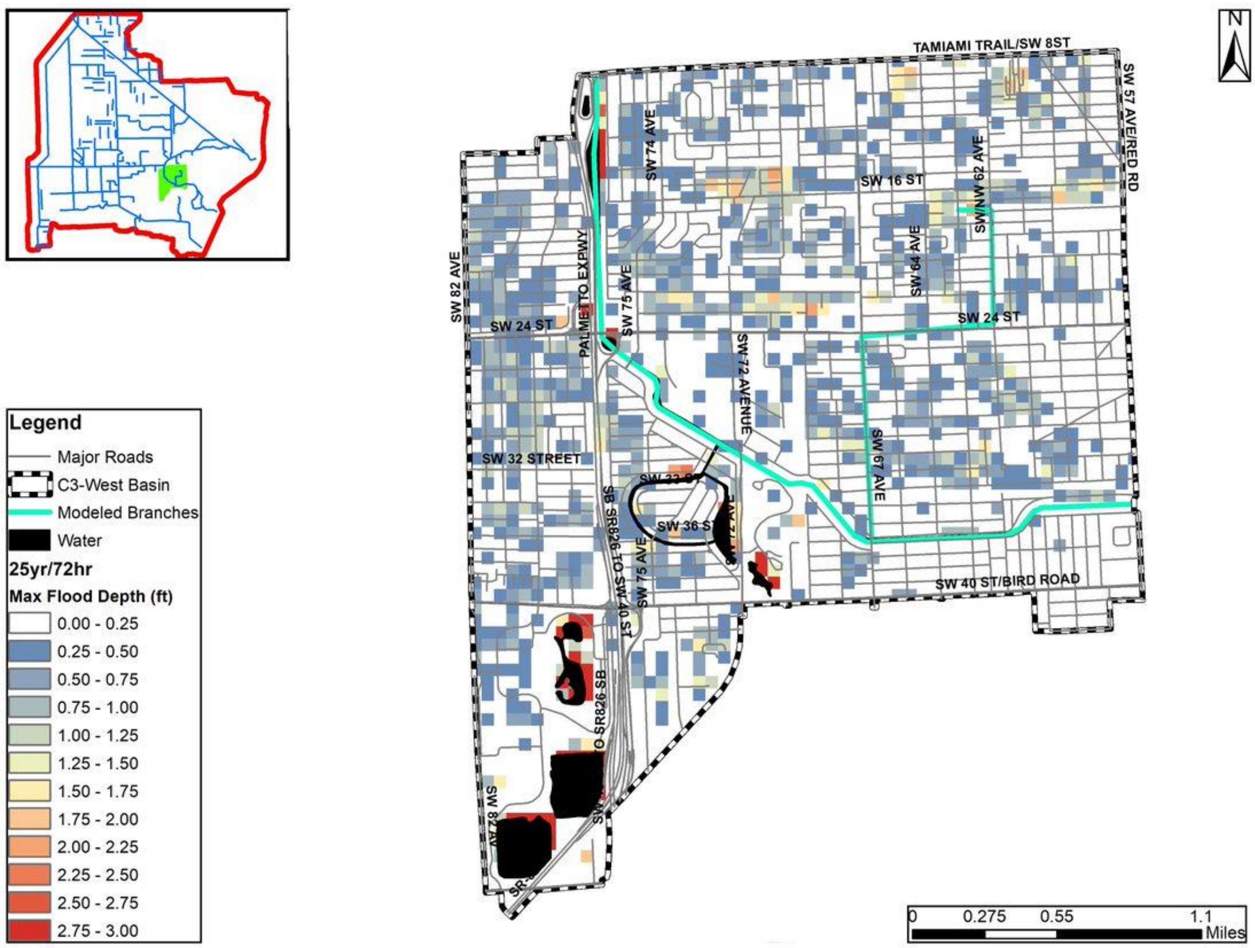


Figure C 2-10. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR1 in the C3W Watershed

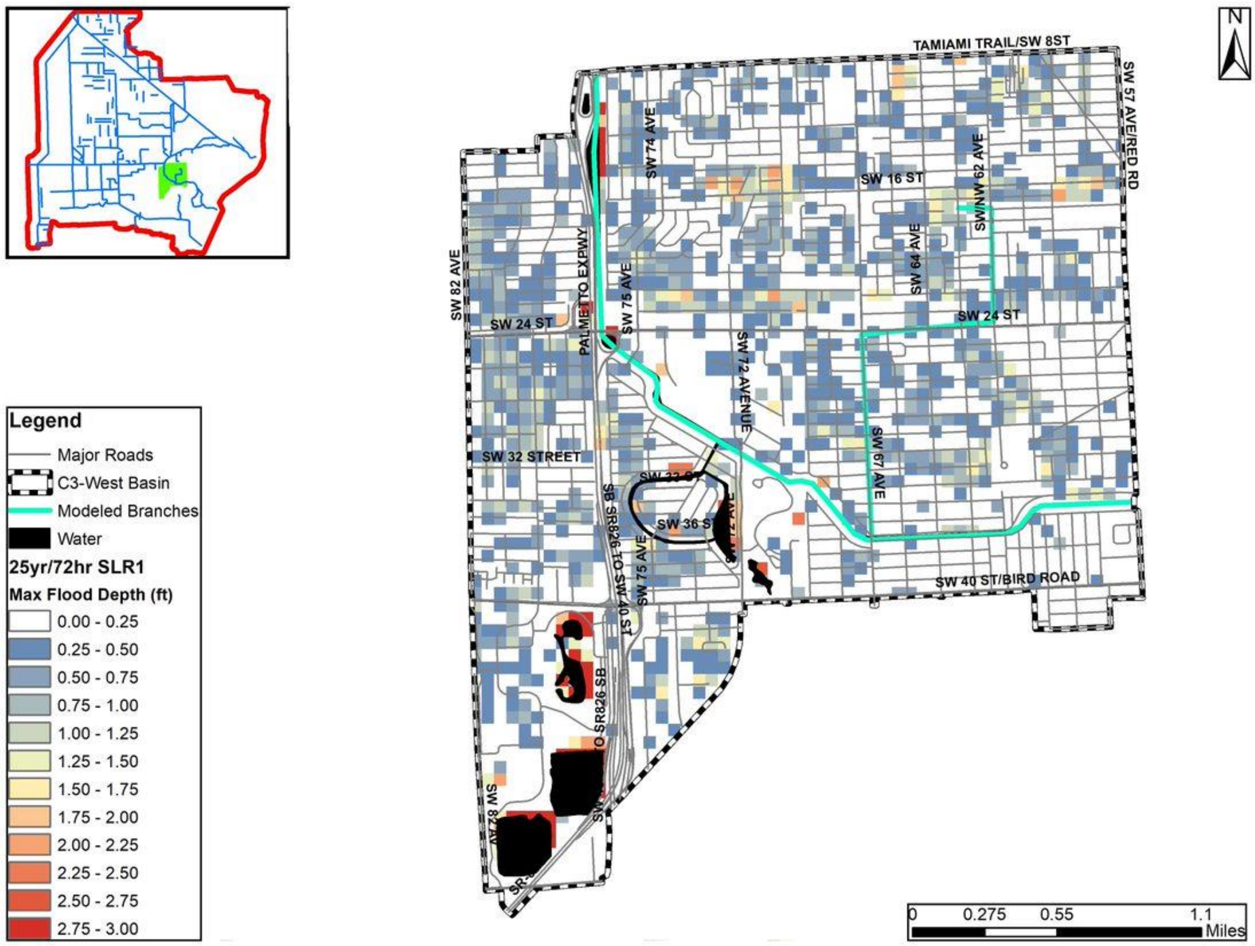


Figure C 2-11. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR2 in the C3W Watershed

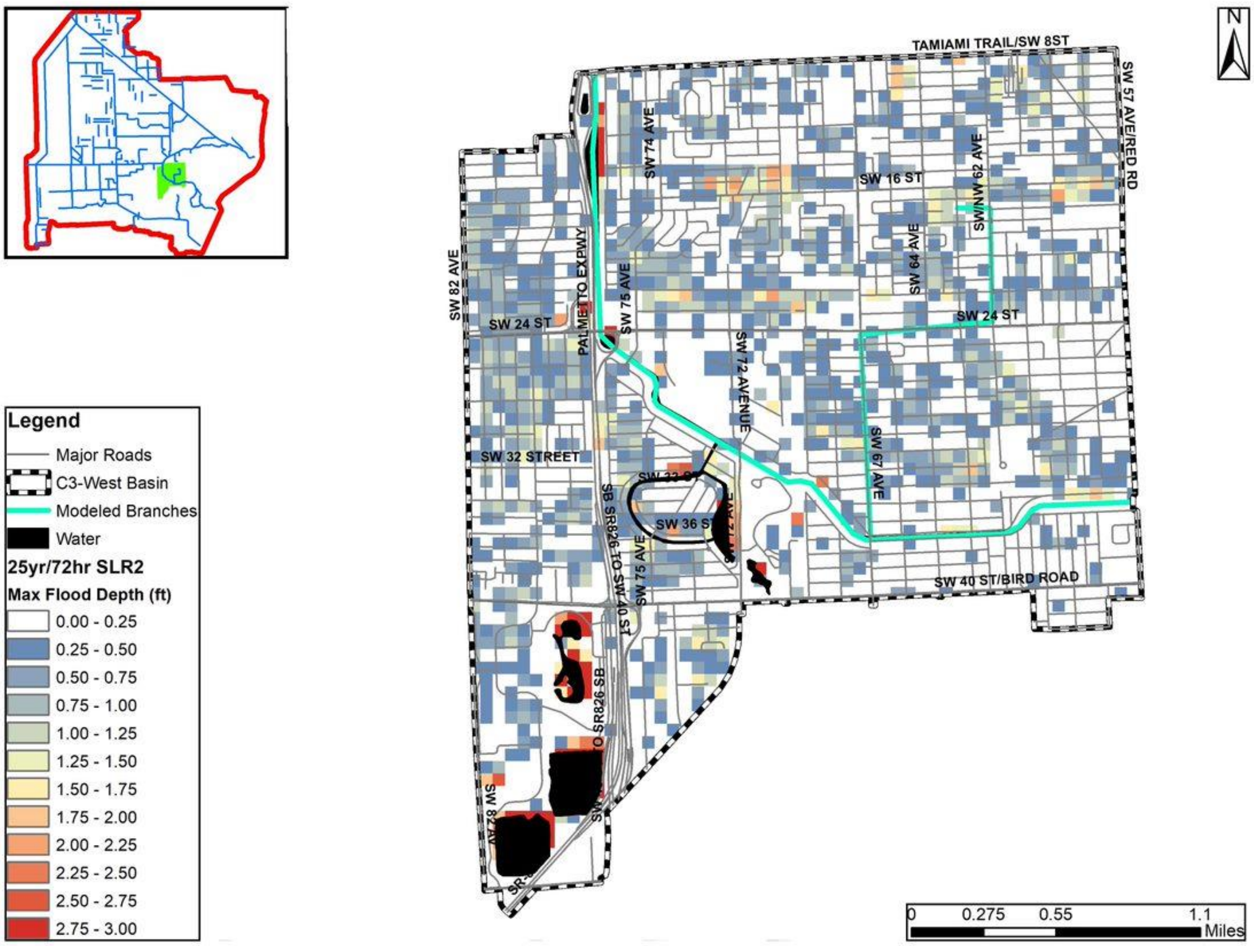


Figure C 2-12. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR3 in the C3W Watershed

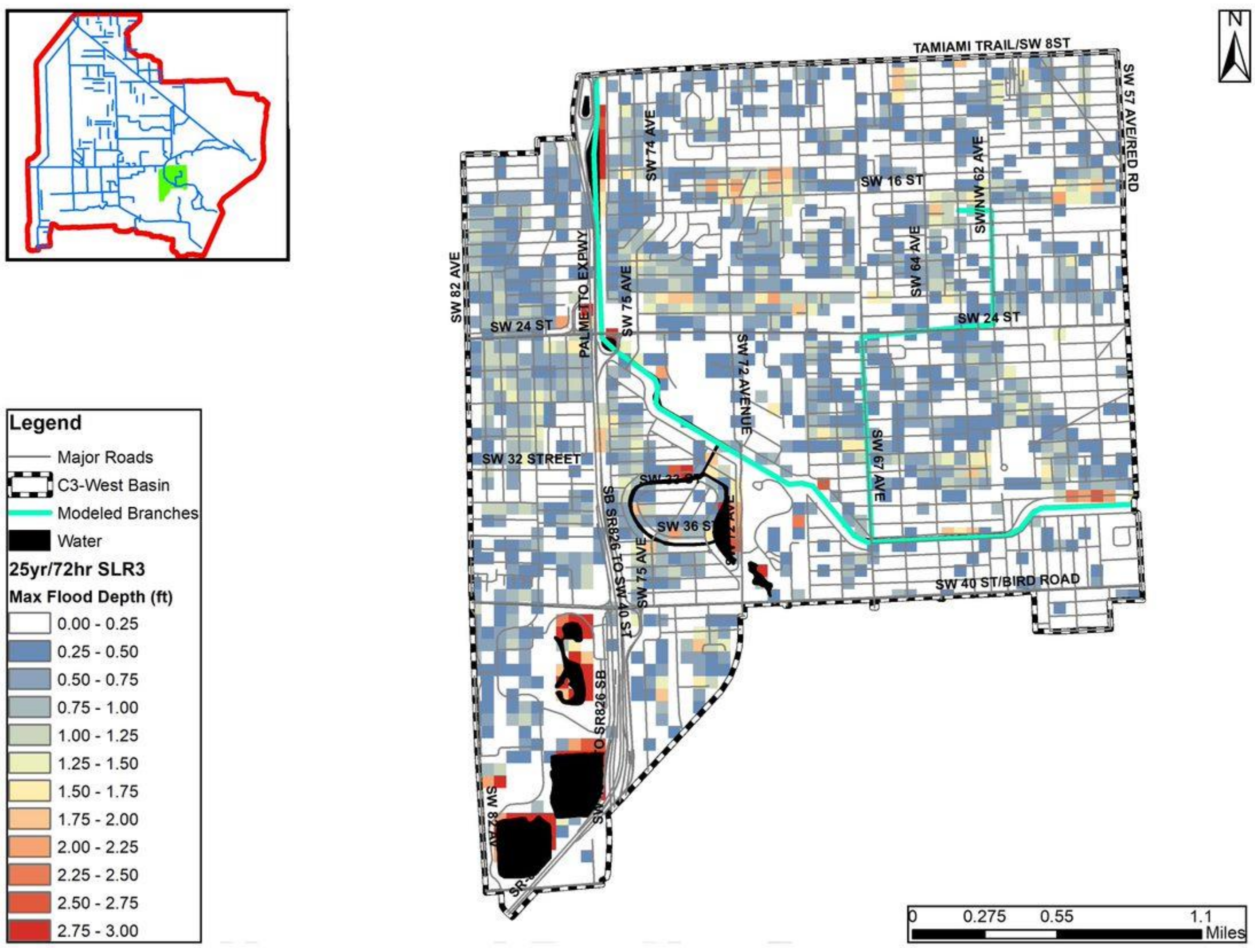


Figure C 2-13. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for Current Conditions in the C3W Watershed

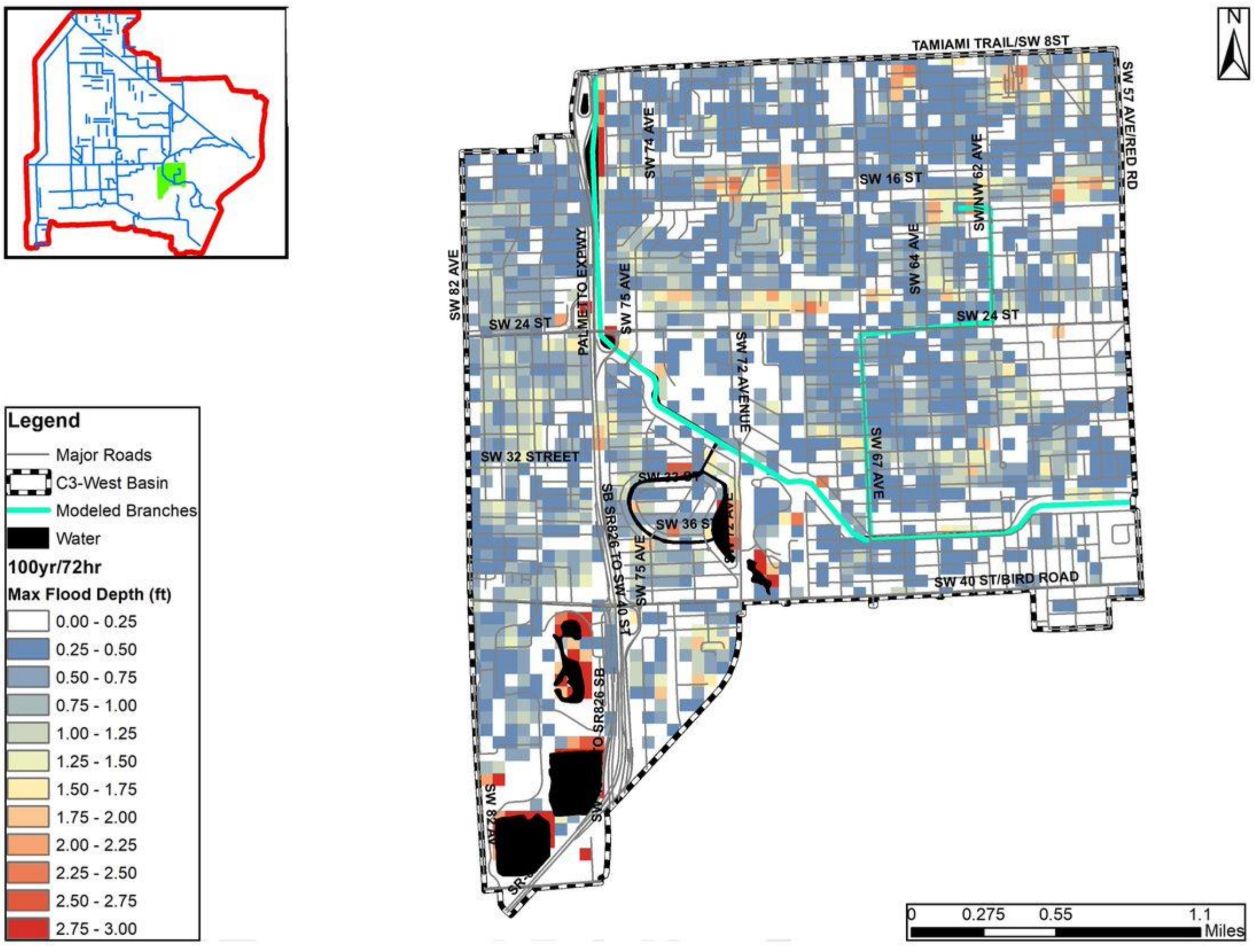


Figure C 2-14. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR1 in the C3W Watershed

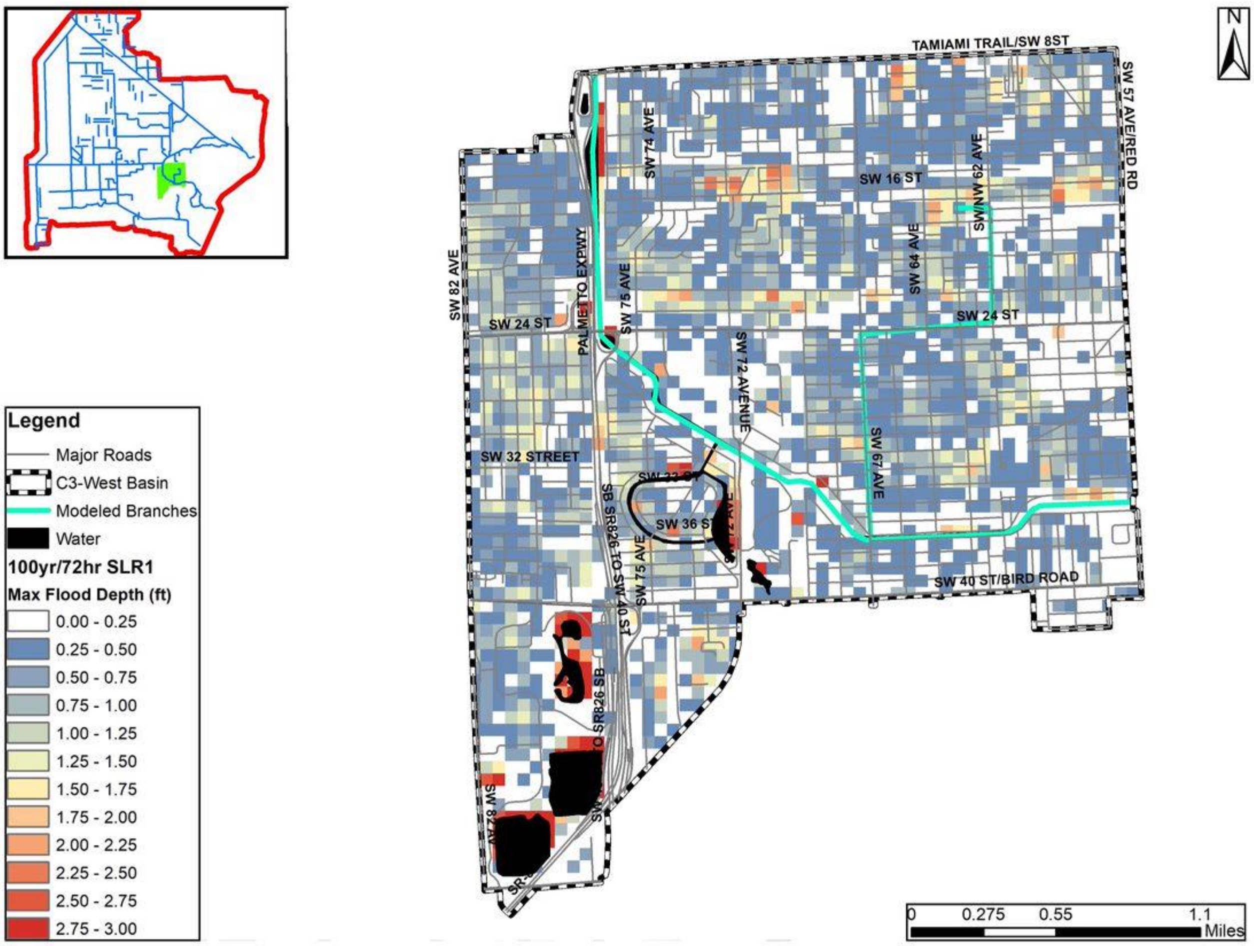


Figure C 2-15. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR2 in the C3W Watershed

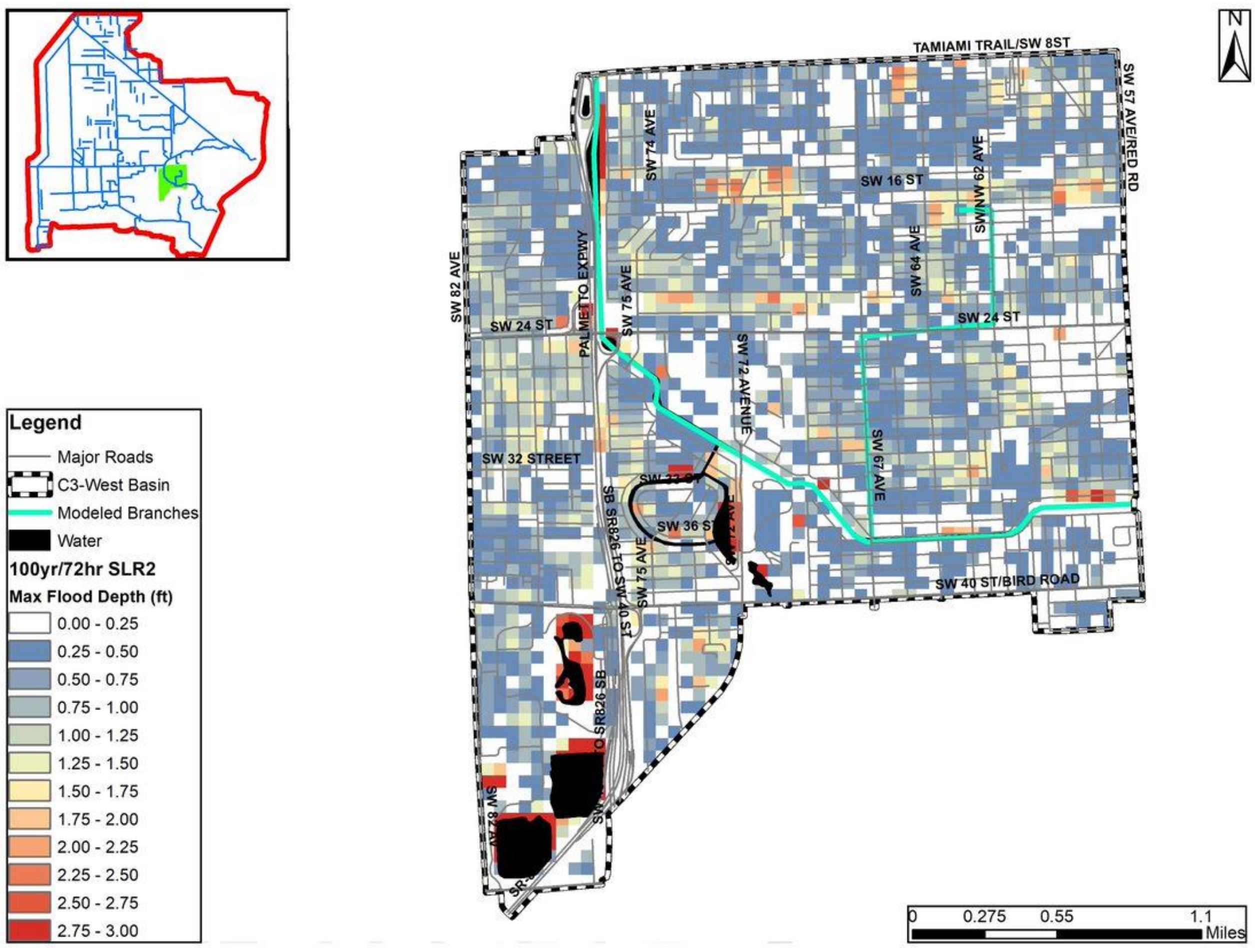


Figure C 2-16. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR3 in the C3W Watershed

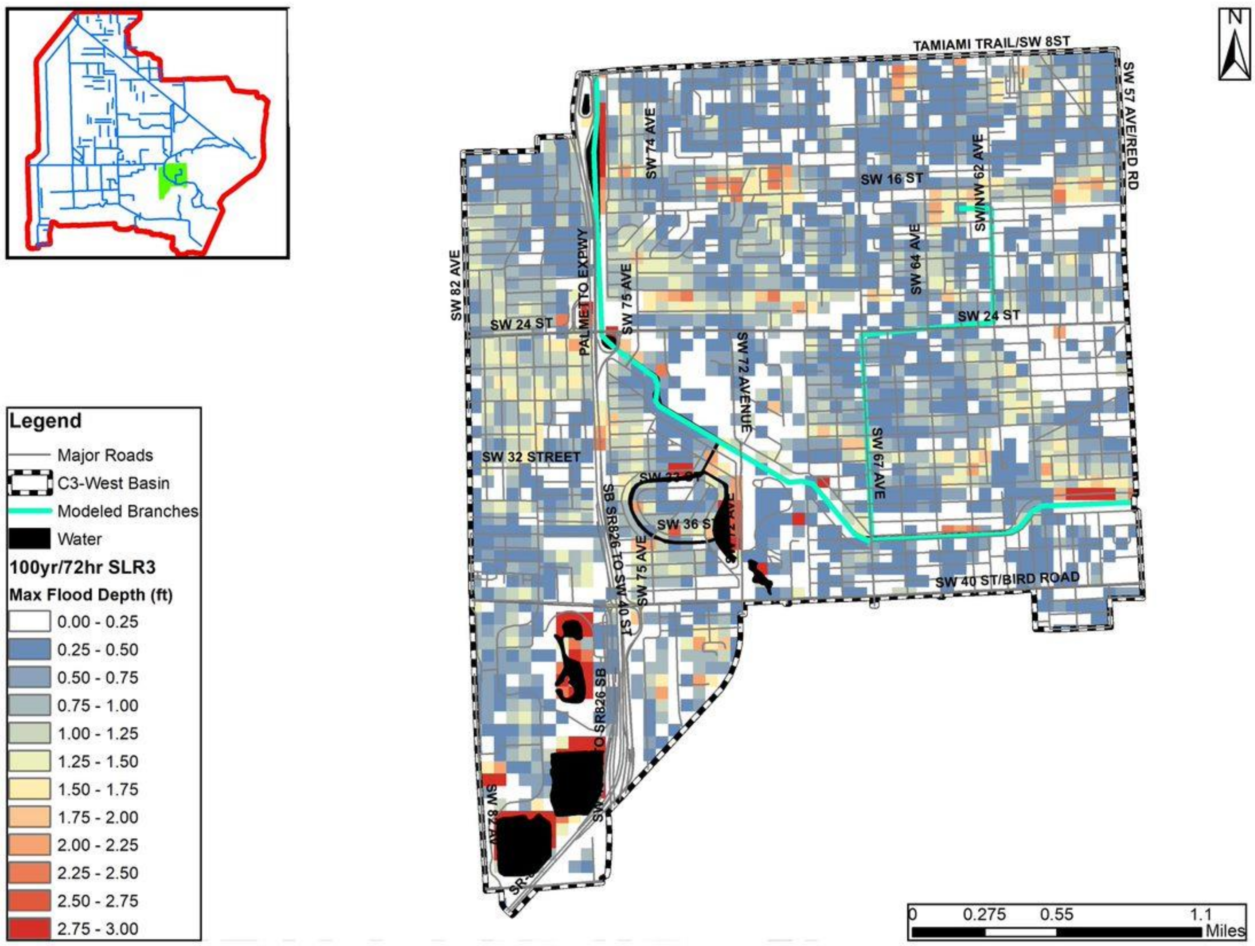

Legend
Major Roads
C3-West Basin
Modeled Branches
Water
100yr/72hr SLR3
Max Flood Depth (ft)
0.00 - 0.25
0.25 - 0.50
0.50 - 0.75
0.75 - 1.00
1.00 - 1.25
1.25 - 1.50
1.50 - 1.75
1.75 - 2.00
2.00 - 2.25
2.25 - 2.50
2.50 - 2.75
2.75 - 3.00
TAMIAMI TRAIL/SW 8ST
SW 57 AVE/RED RD
SW 74 AVE
SW 16 ST
SW/NW 62 AVE
SW 64 AVE
SW 82 AVE
PALMETTO EXPWY
SW 75 AVE
SW 24 ST
SW 24 ST
SW 72 AVENUE
SW 32 STREET
SW 67 AVE
SW 36 ST
SB SR826 TO SW 40 ST
SW 40 ST/BIRD ROAD
SW 78 AV
N
0 0.275 0.55 1.1
Miles

Figure C 2-17. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C3W Watershed

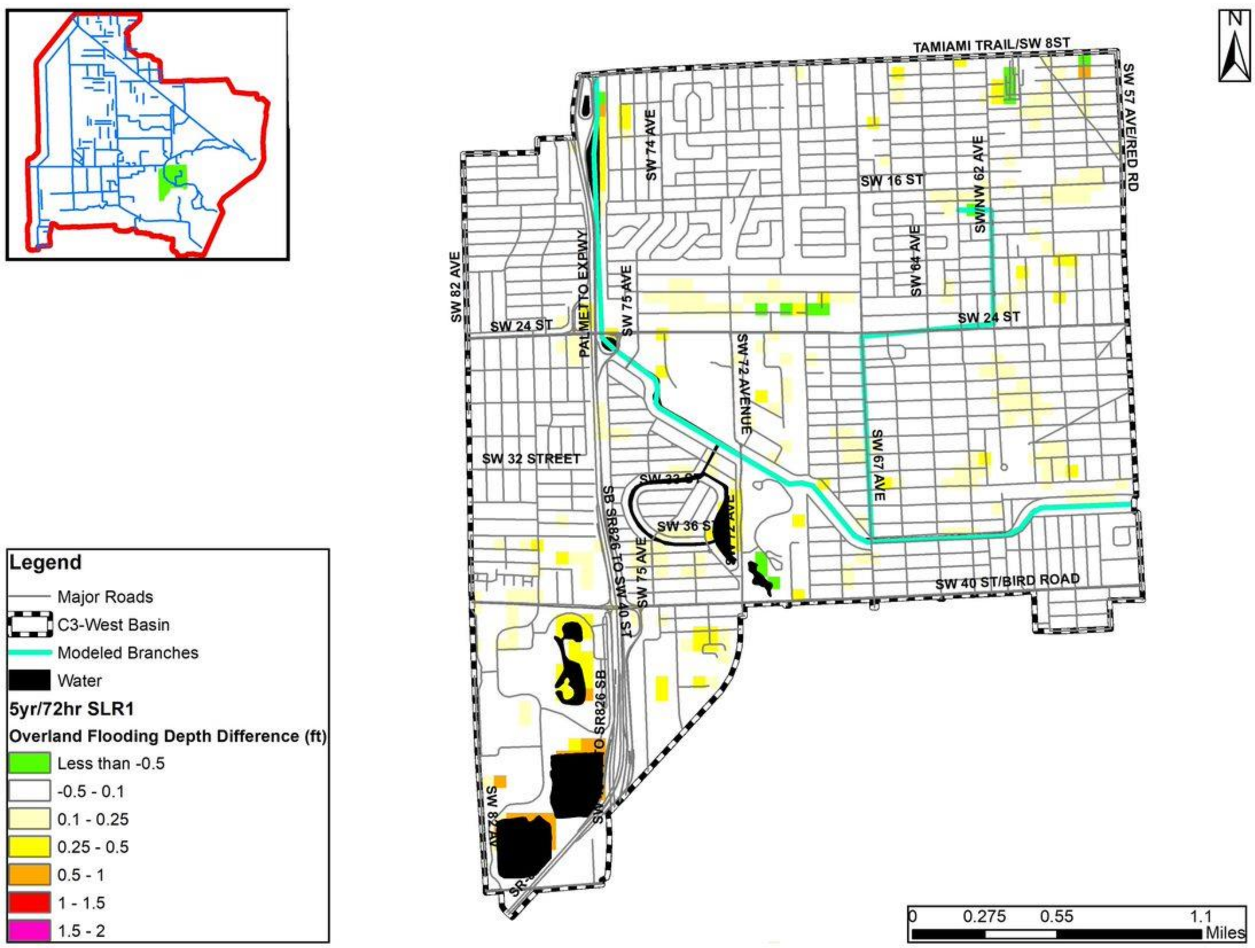


Figure C 2-18. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C3W Watershed

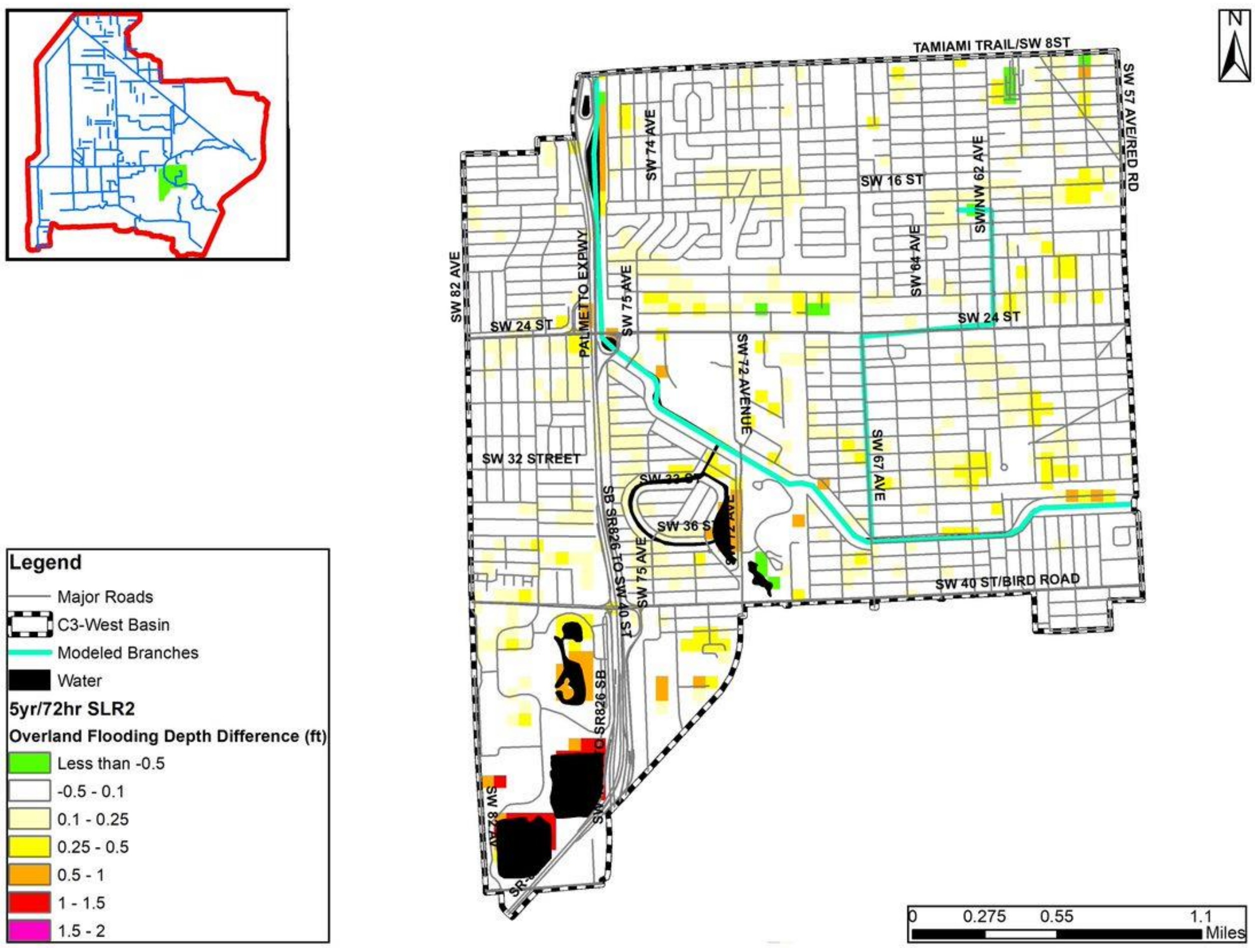


Figure C 2-19. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C3W Watershed

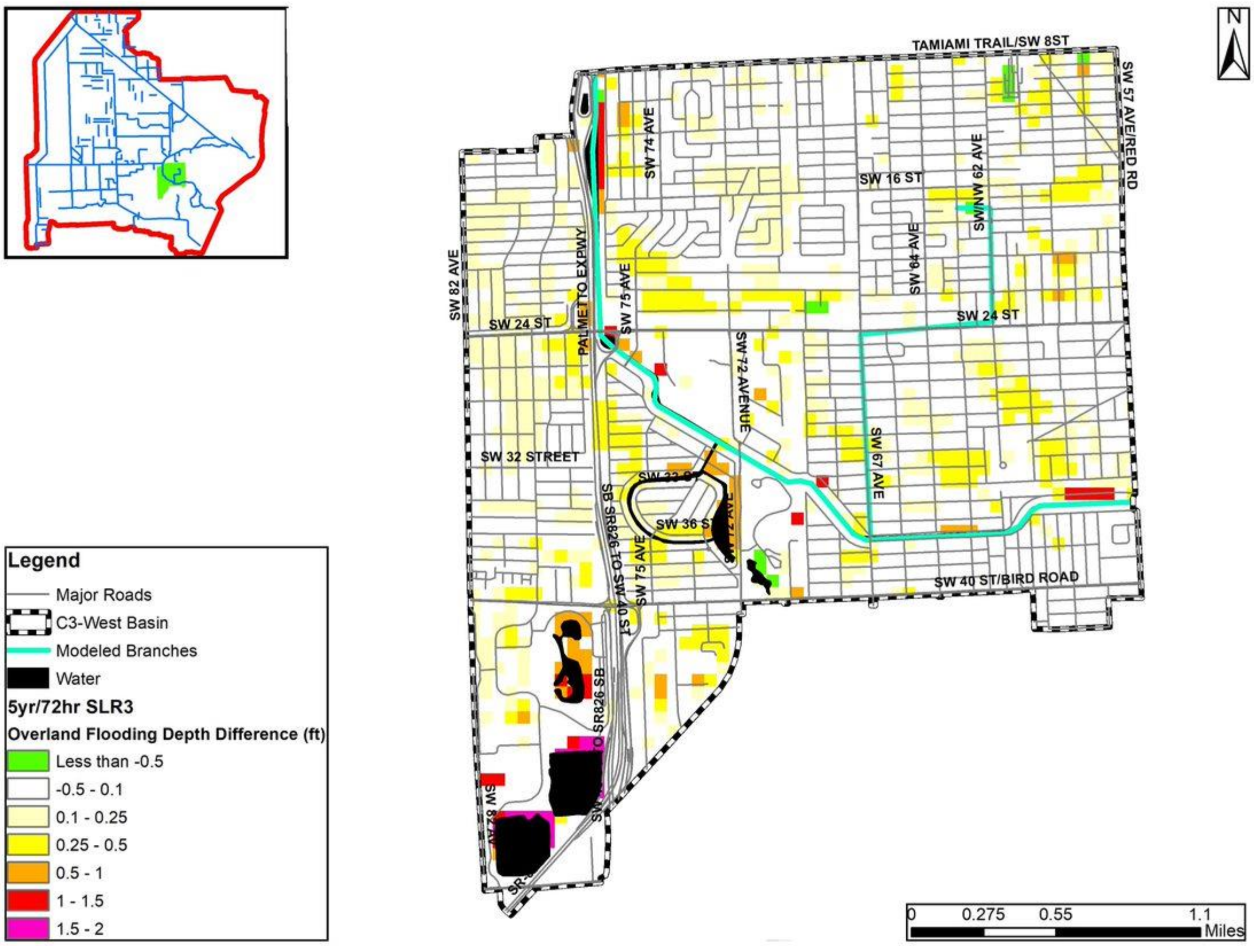


Figure C 2-20. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C3W Watershed

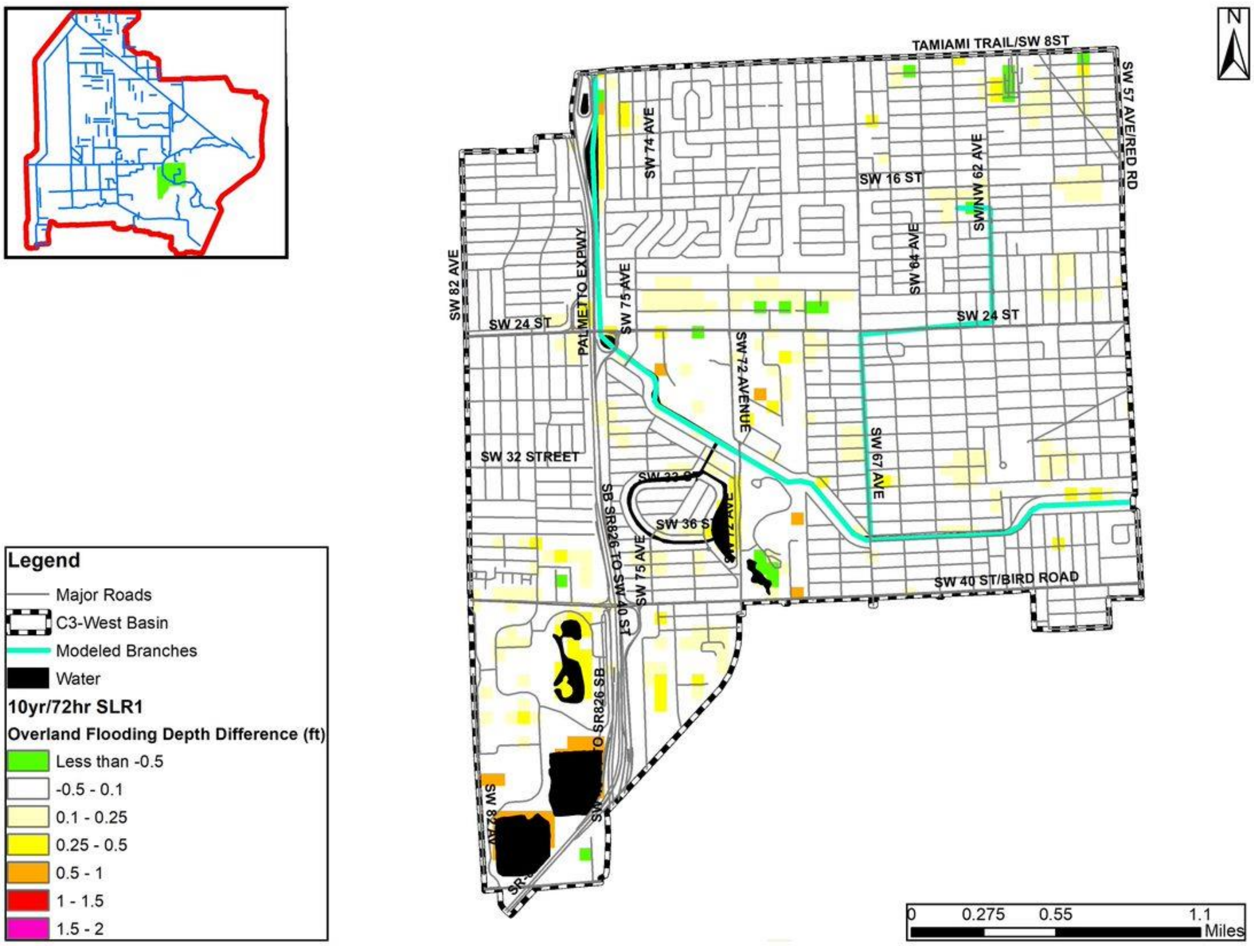


Figure C 2-21. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C3W Watershed

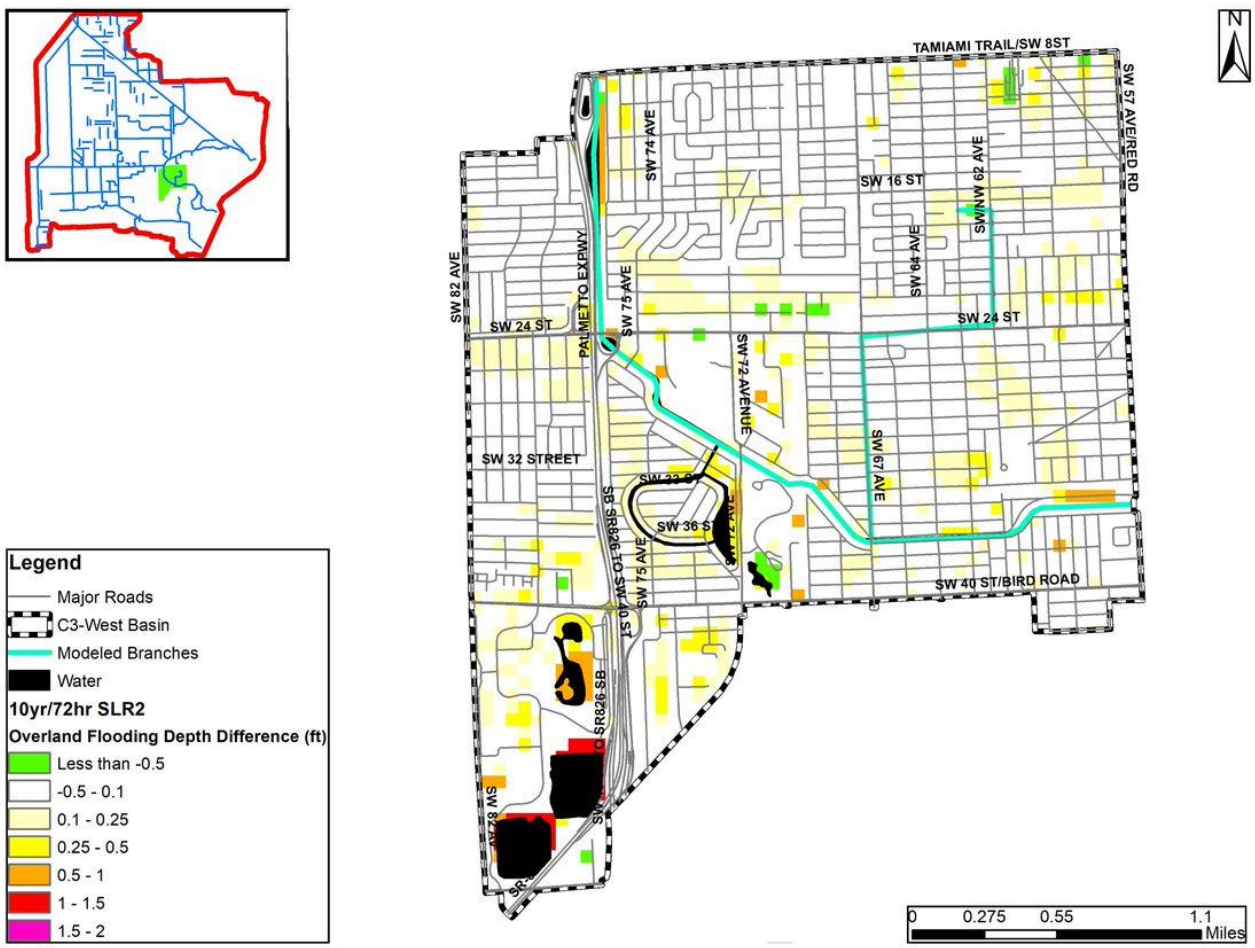


Figure C 2-22. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C3W Watershed

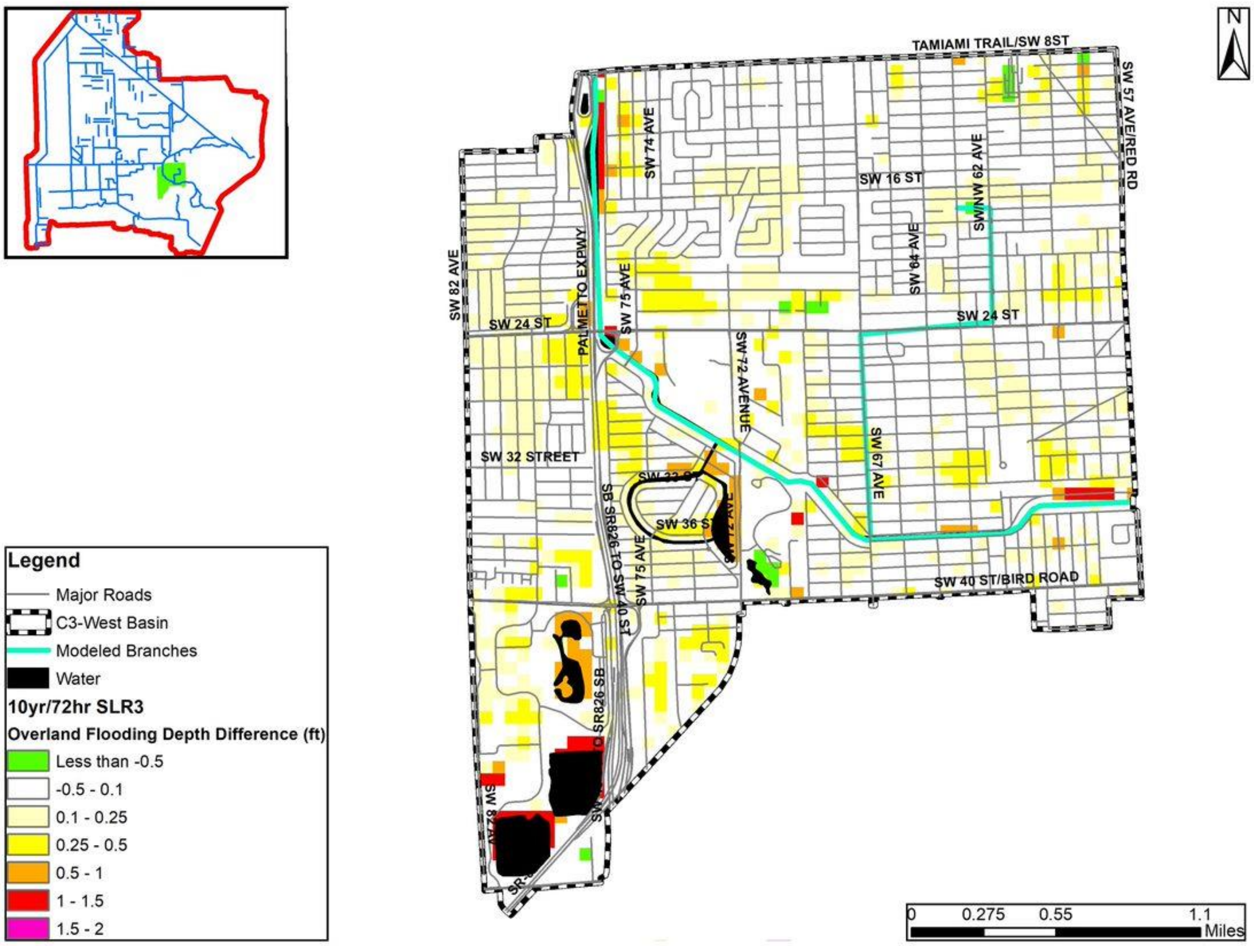


Figure C 2-23. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C3W Watershed

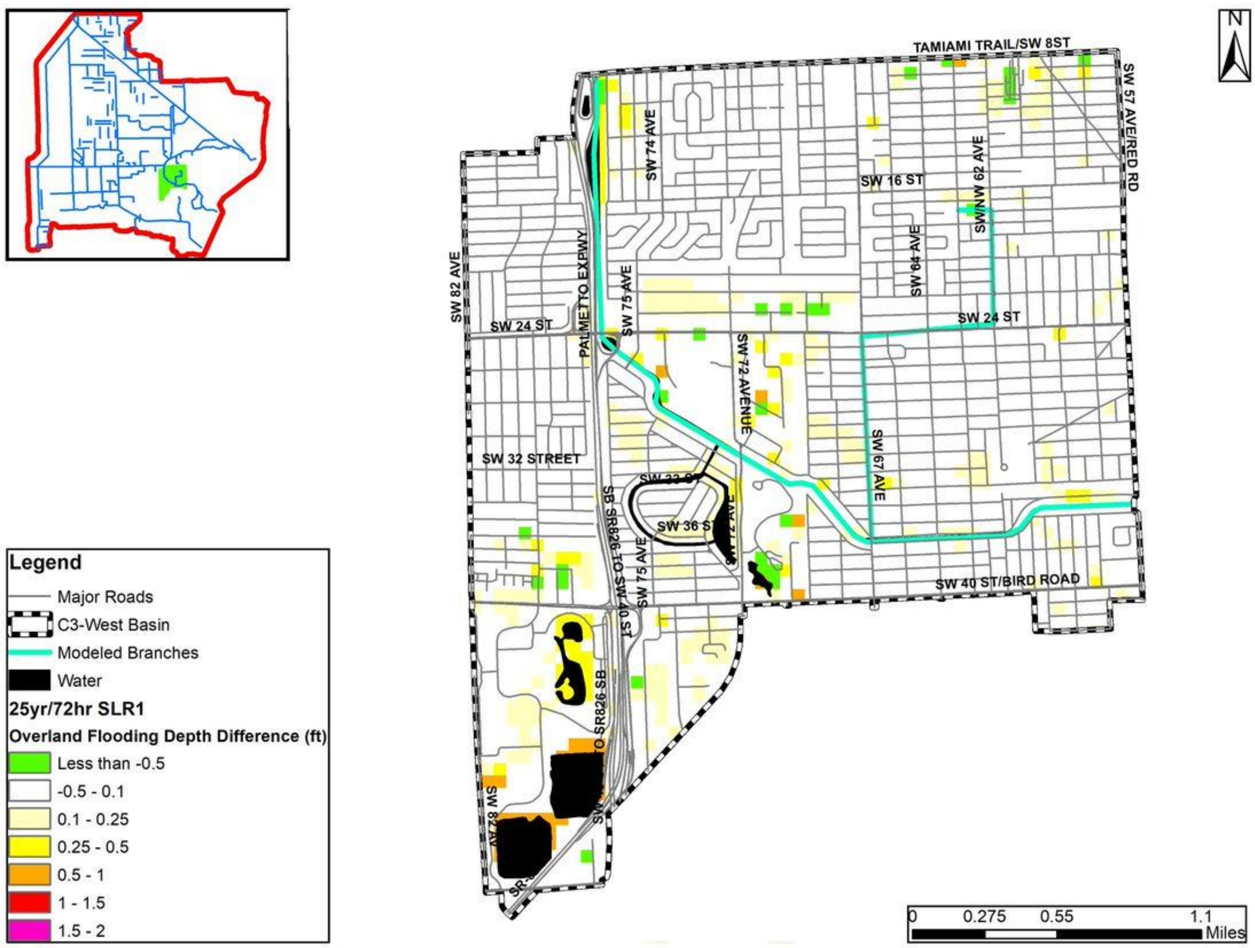


Figure C 2-24. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C3W Watershed

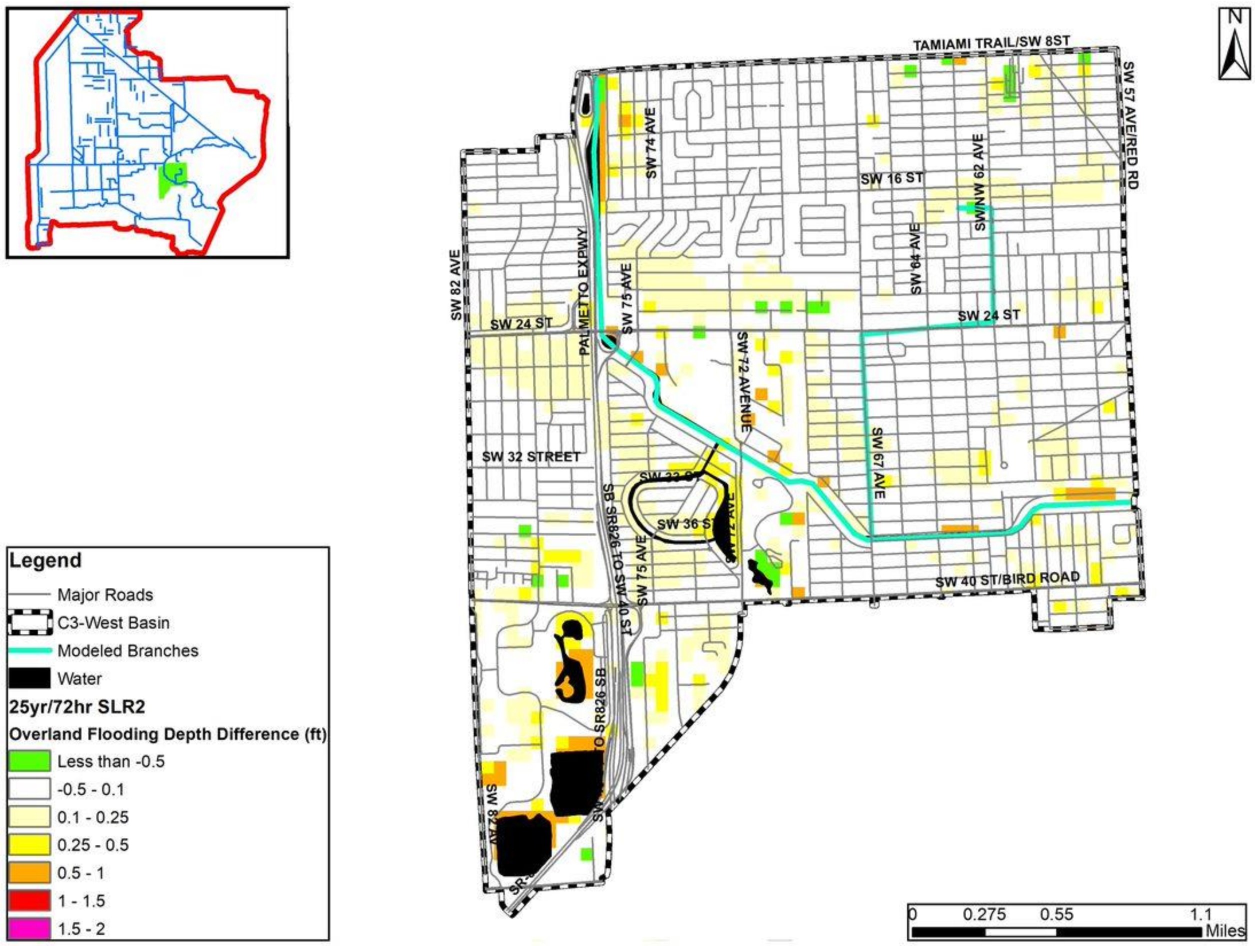


Figure C 2-25. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C3W Watershed

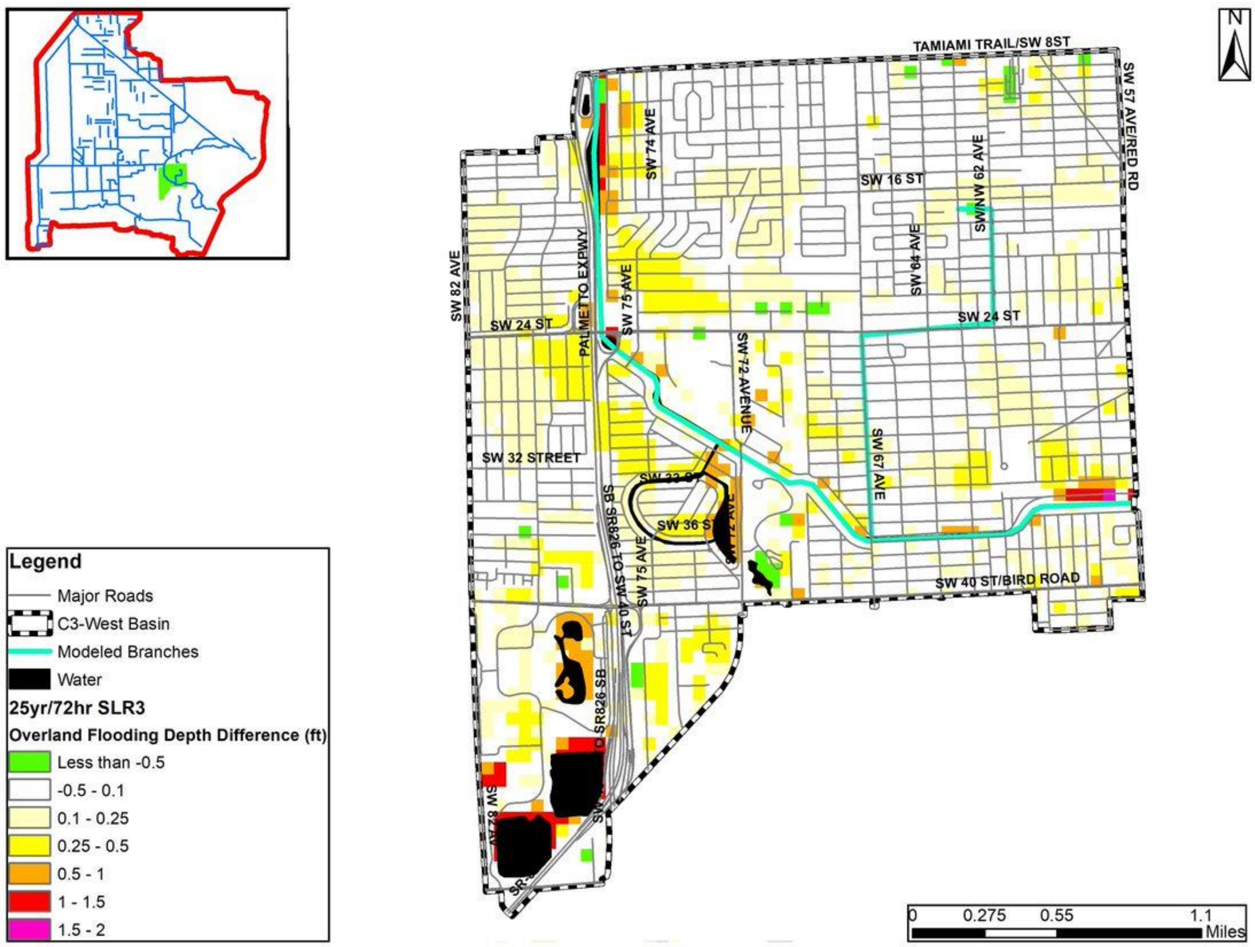


Figure C 2-26. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C3W Watershed

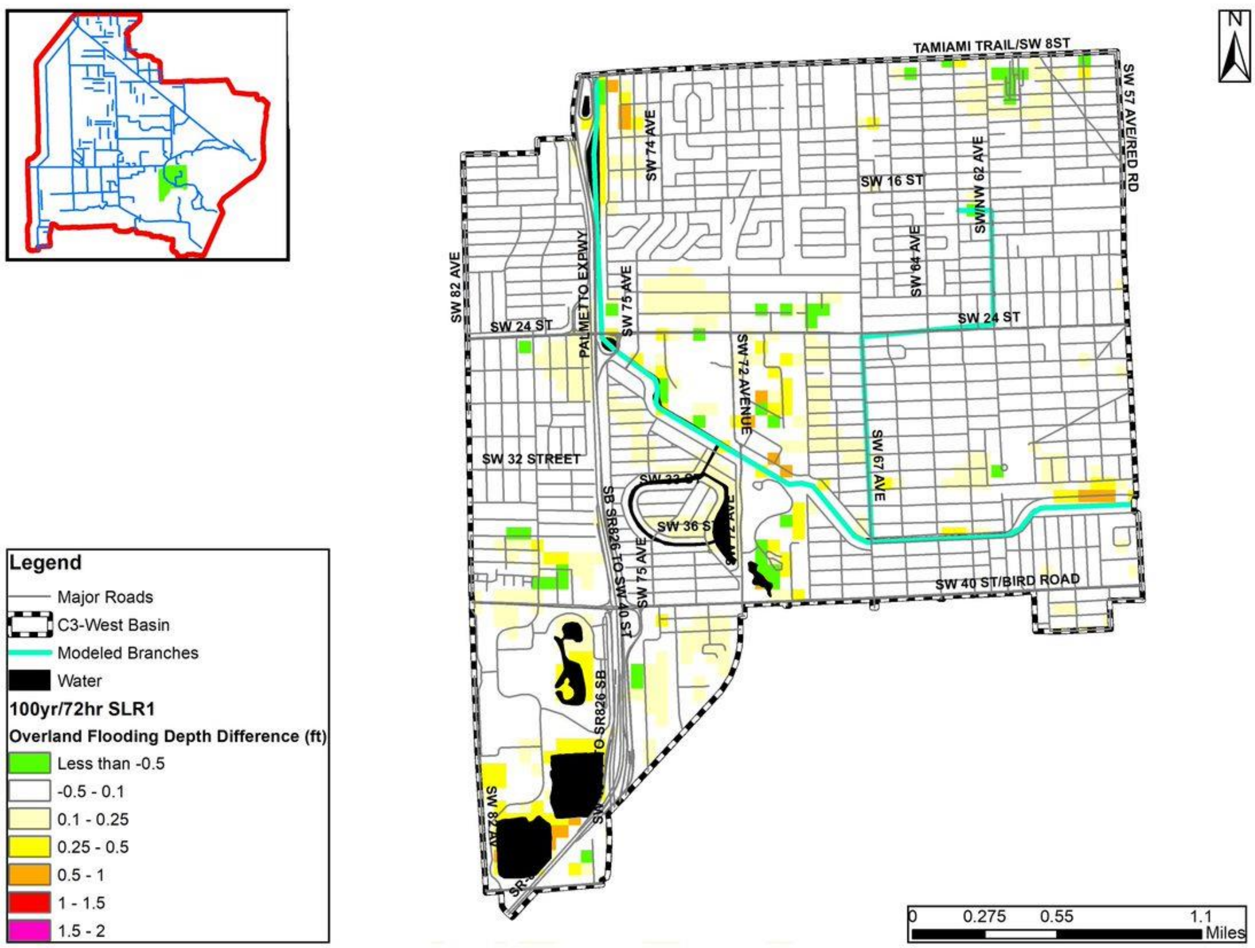


Figure C 2-27. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C3W Watershed

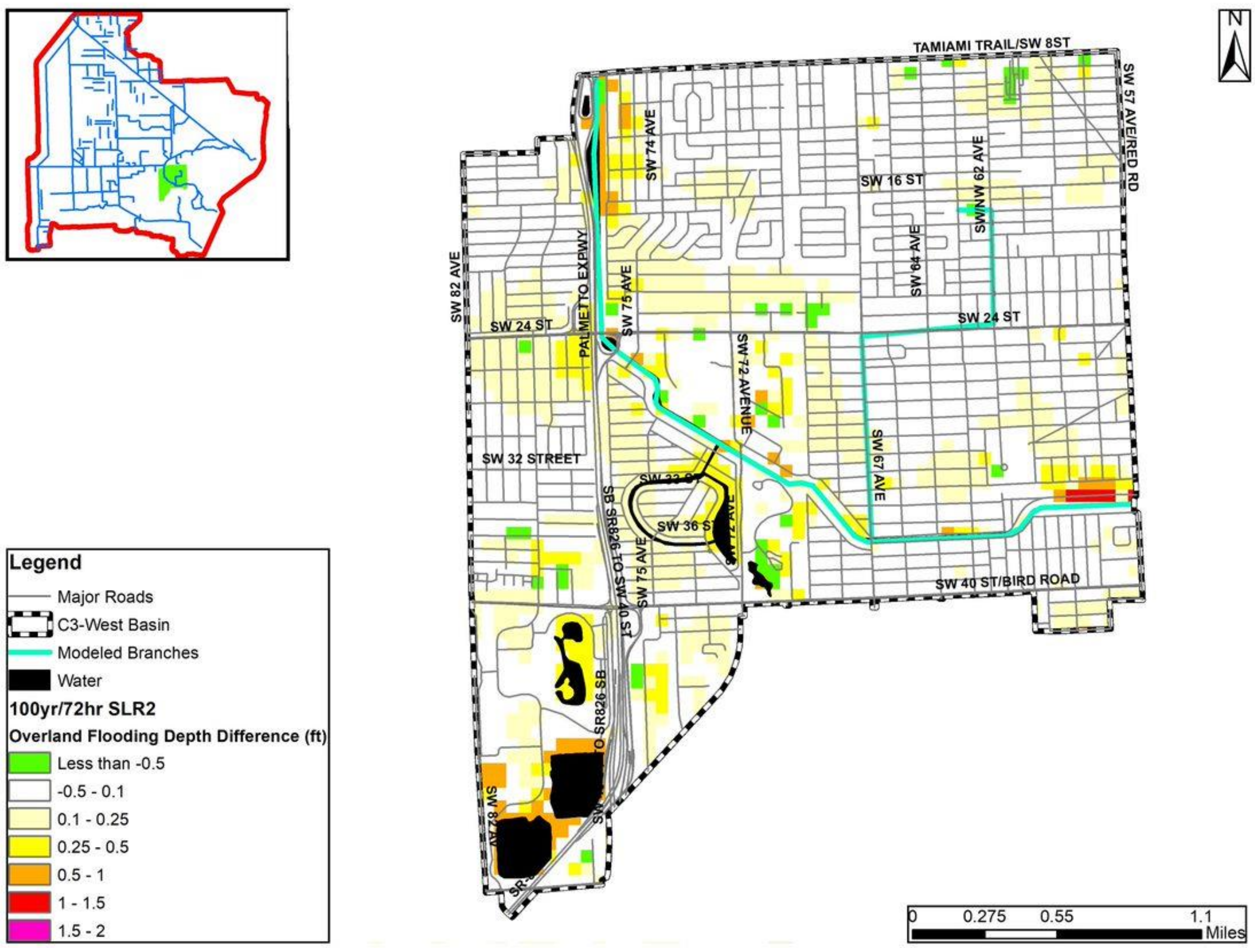
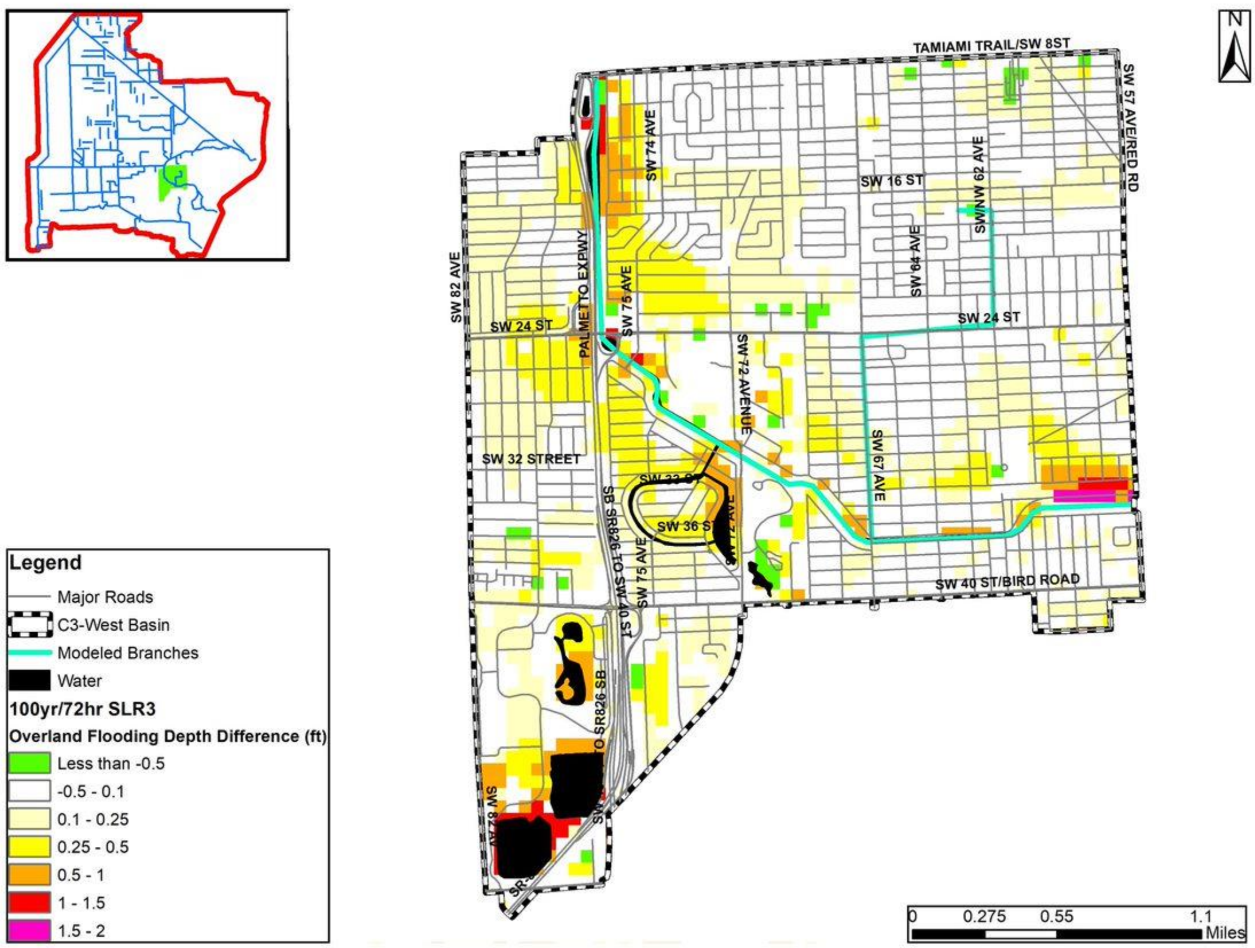


Figure C 2-28. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C3W Watershed



3 C4 Watershed

Figure C 3-1 through **Figure C 3-16** are maps of the maximum overland depth over the entire C4 Watershed for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for each SLR condition. Water areas are masked in black. **Figure C 3-17** through **Figure C 3-32** provide the same maps with the non-urban areas masked out, to provide a concise picture of how urban areas are impacted within the watershed.

Figure C 3-33 through **Figure C 3-44** show the difference in overland flooding for the C4 Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively. **Figure C 3-45** through **Figure C 3-56** show the same maps with the non-urban areas masked out.

Figure C 3-1. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for Current Conditions in the C4 Watershed

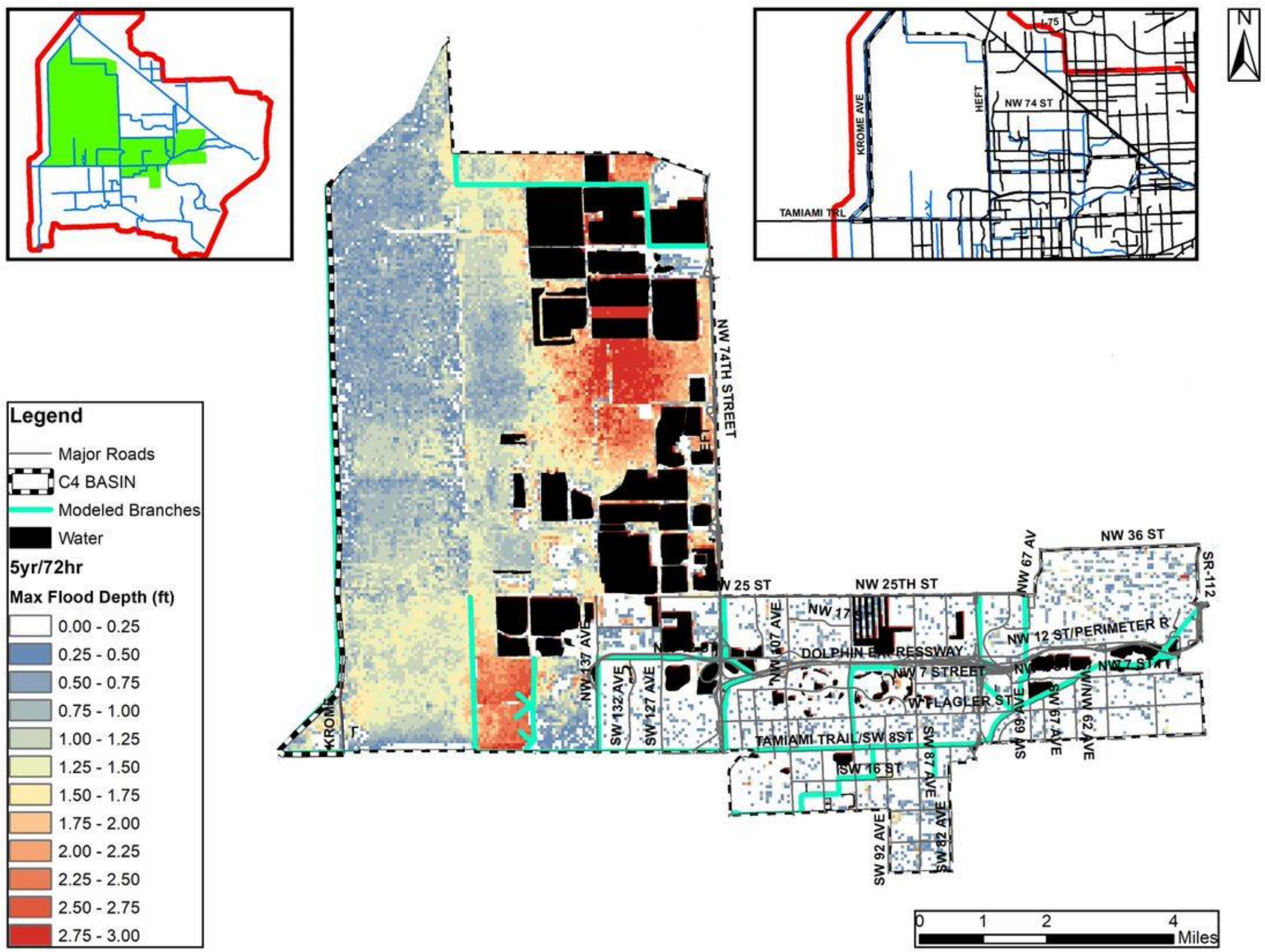


Figure C 3-2. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR1 in the C4 Watershed

Figure C 3-3. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR2 in the C4 Watershed

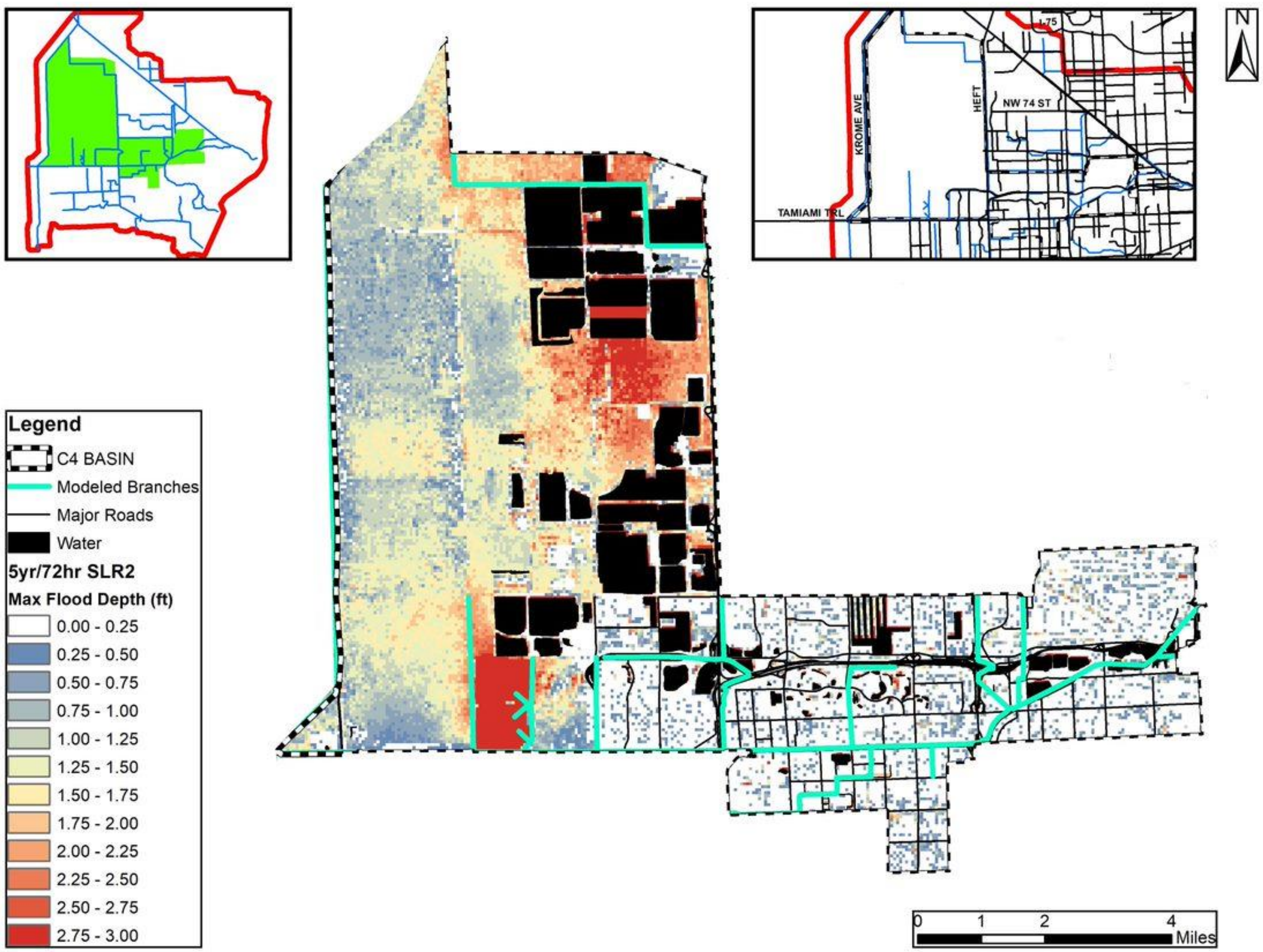


Figure C 3-4. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR3 in the C4 Watershed

Figure C 3-5. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for Current Conditions in the C4 Watershed

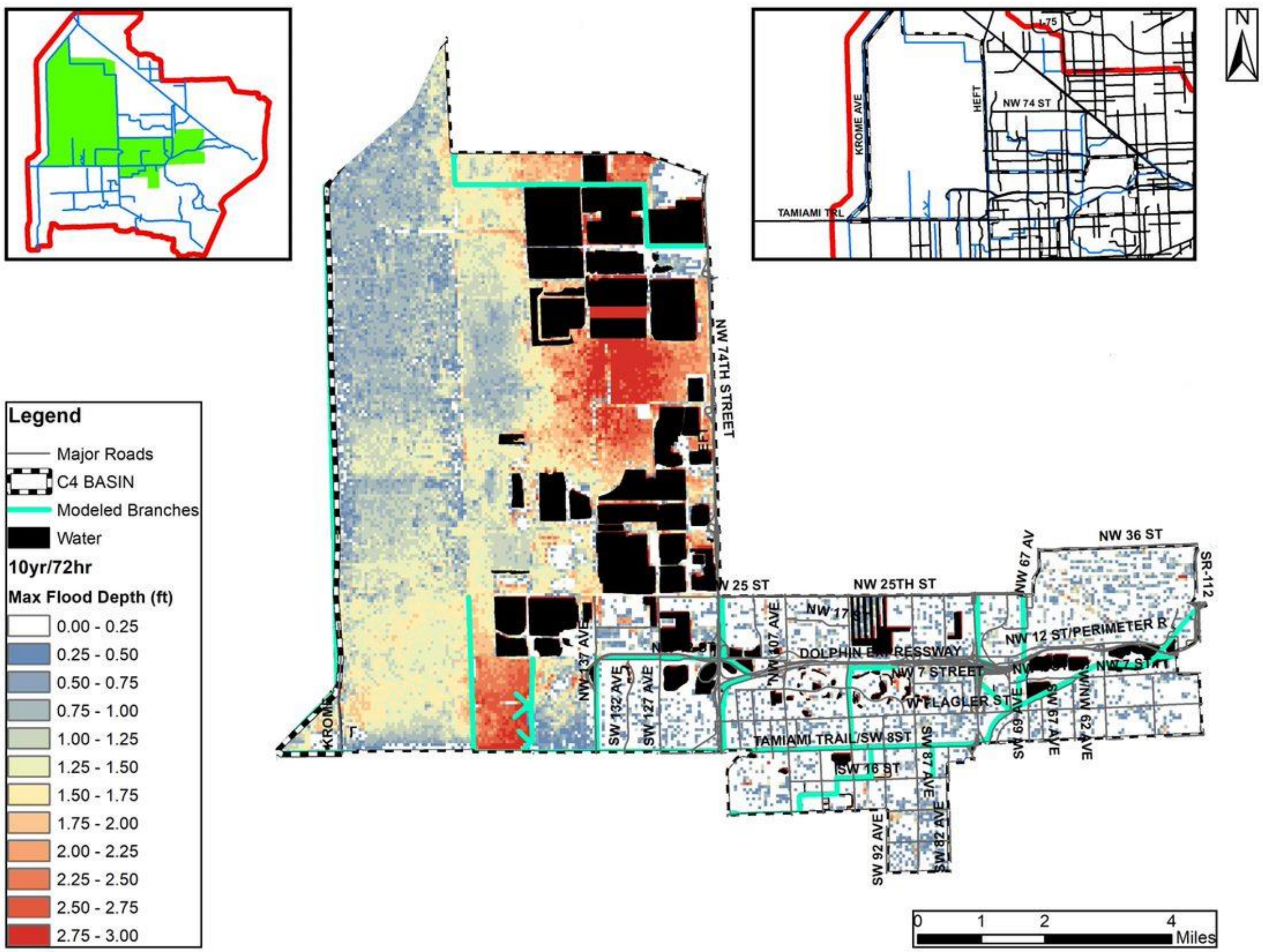


Figure C 3-6. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR1 in the C4 Watershed

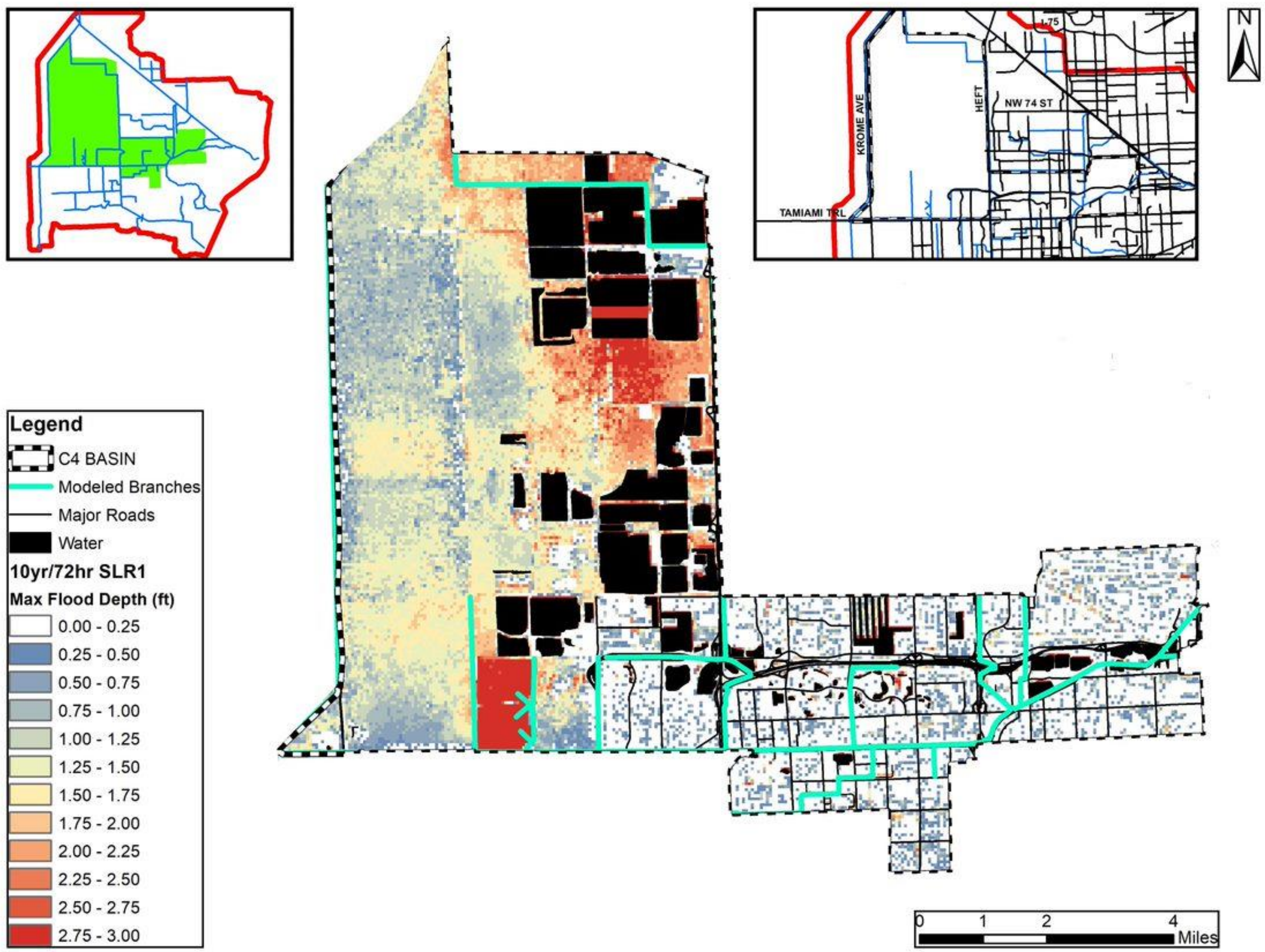


Figure C 3-7. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR2 in the C4 Watershed

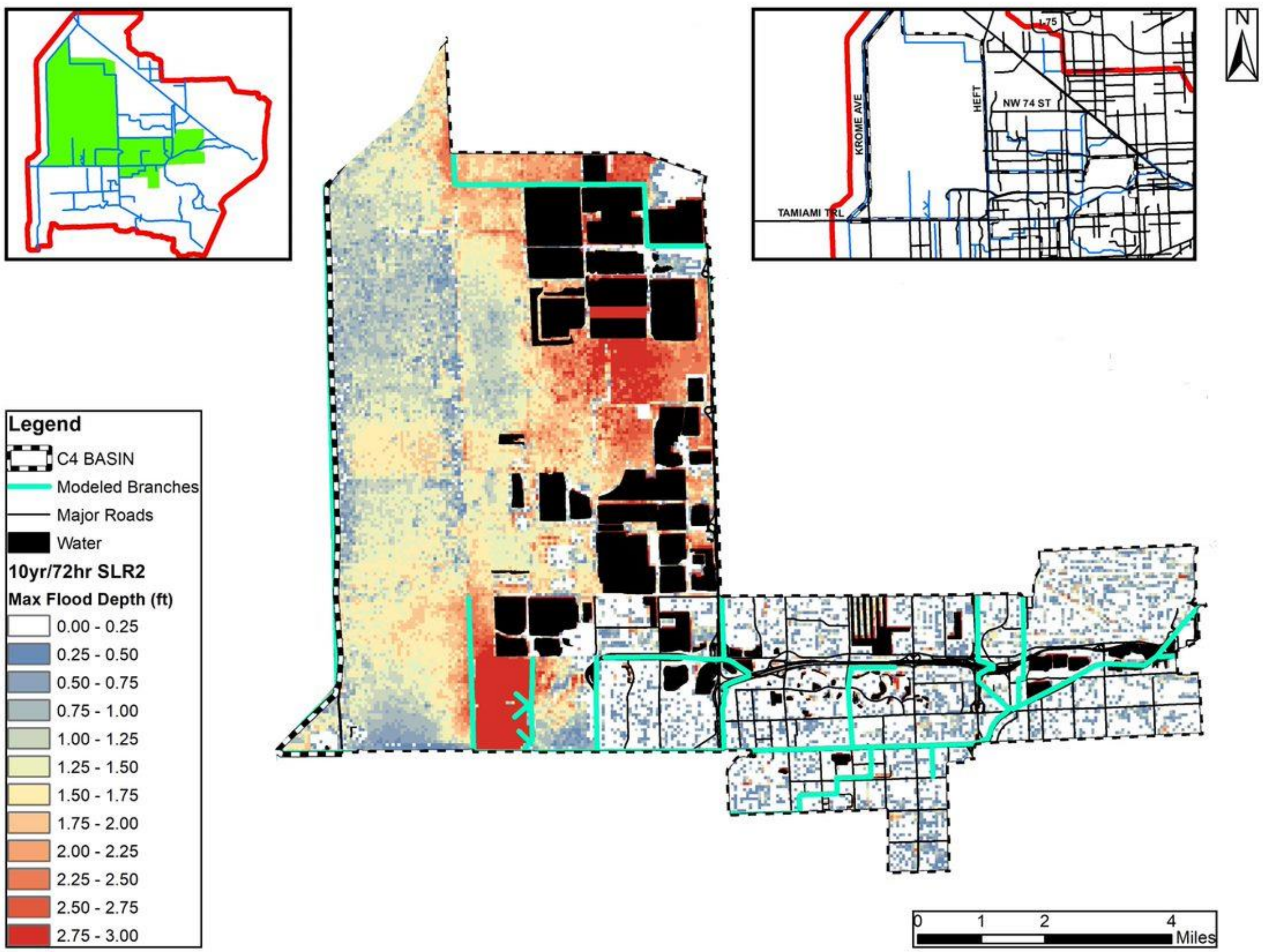


Figure C 3-8. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR3 in the C4 Watershed

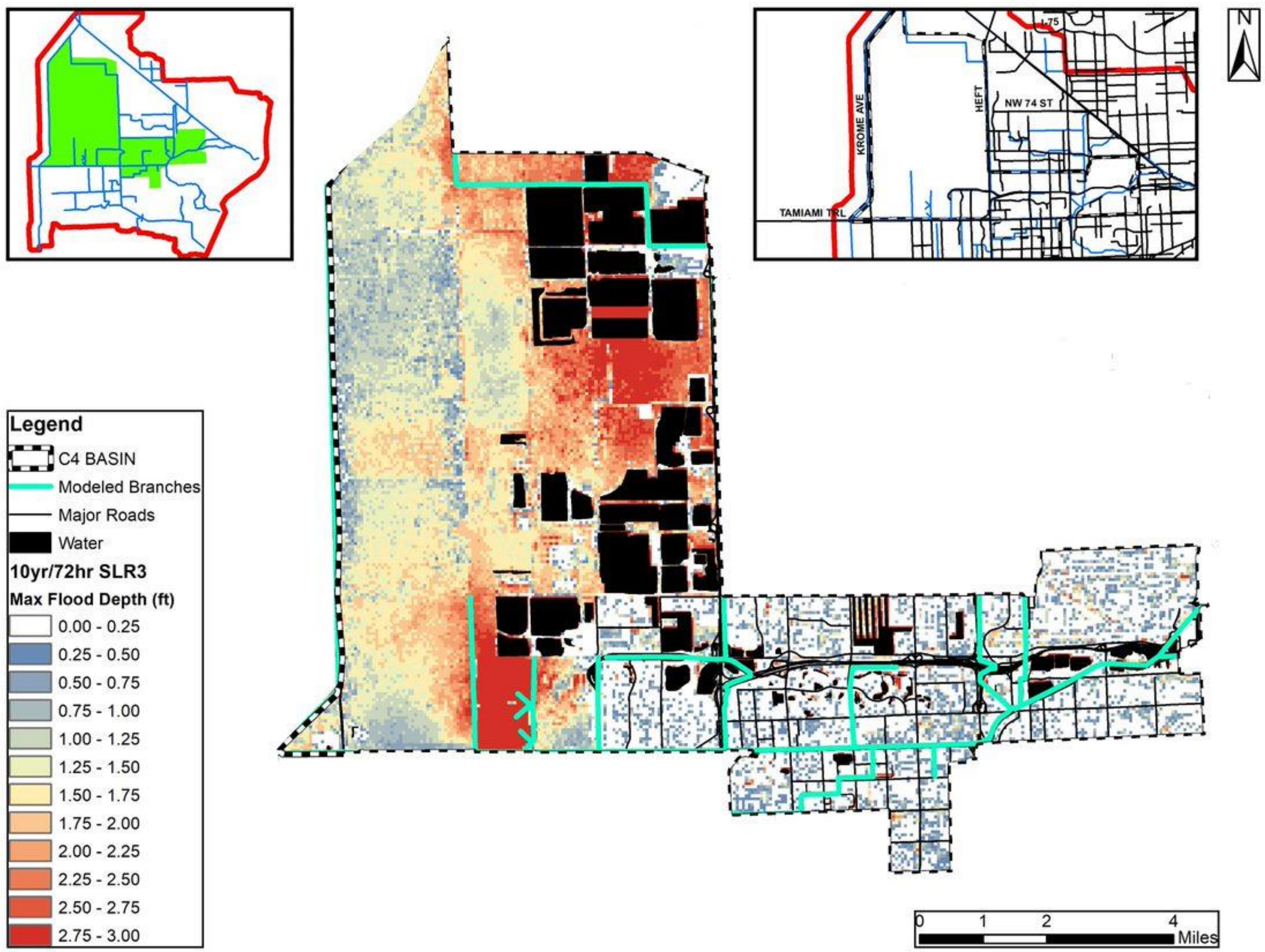


Figure C 3-9. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for Current Conditions in the C4 Watershed

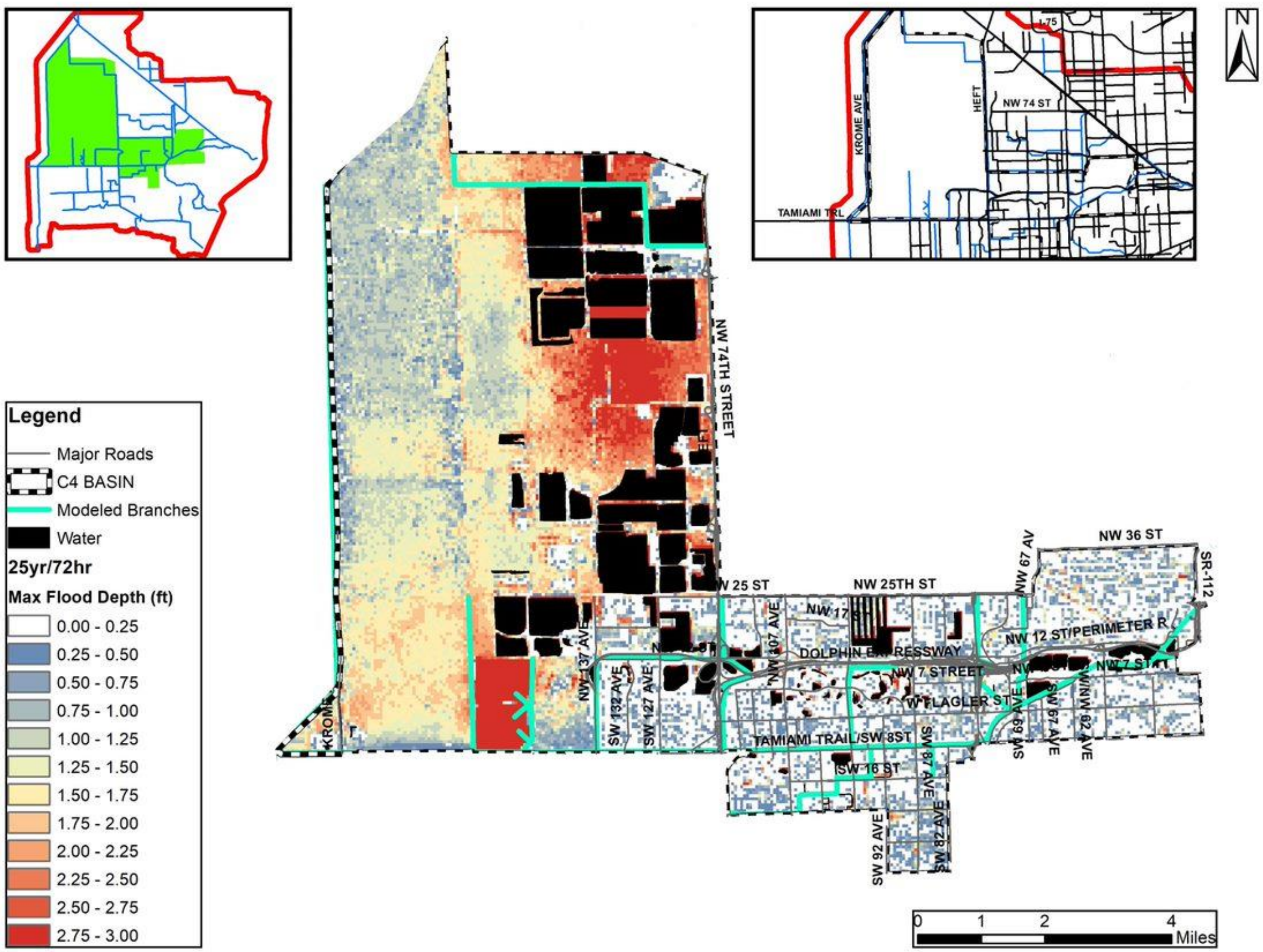


Figure C 3-10. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR1 in the C4 Watershed

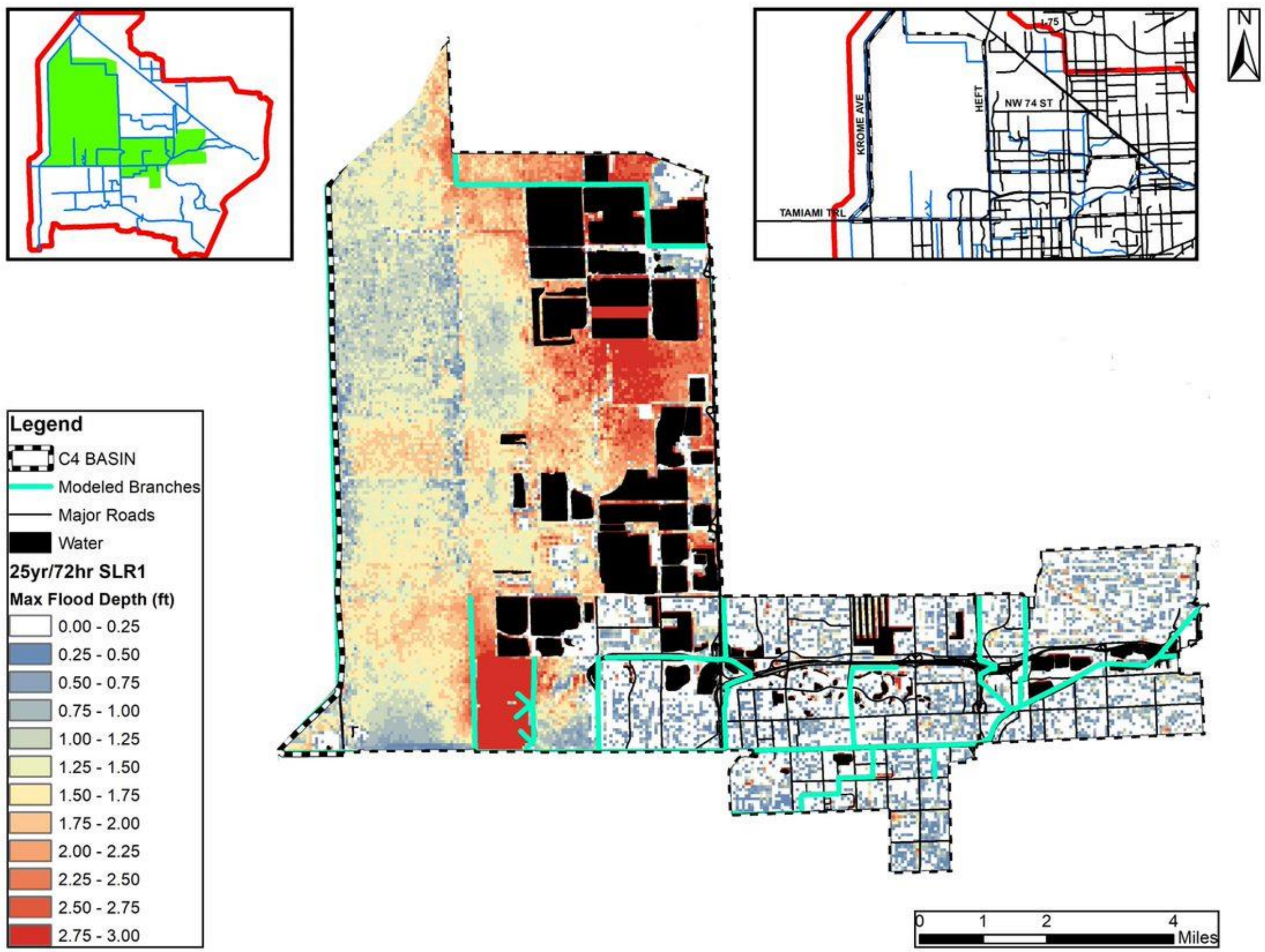


Figure C 3-11. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR2 in the C4 Watershed

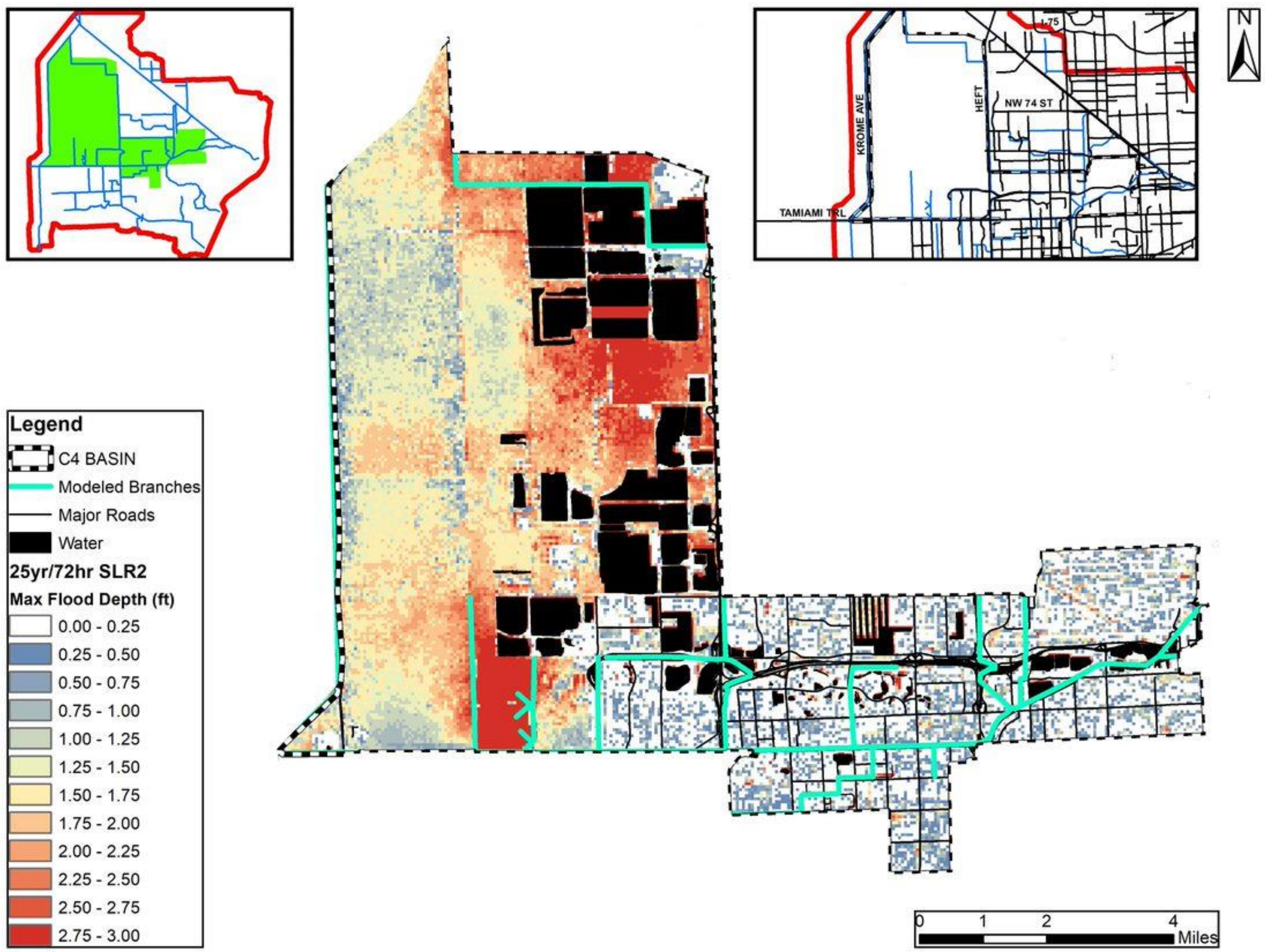


Figure C 3-12. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR3 in the C4 Watershed

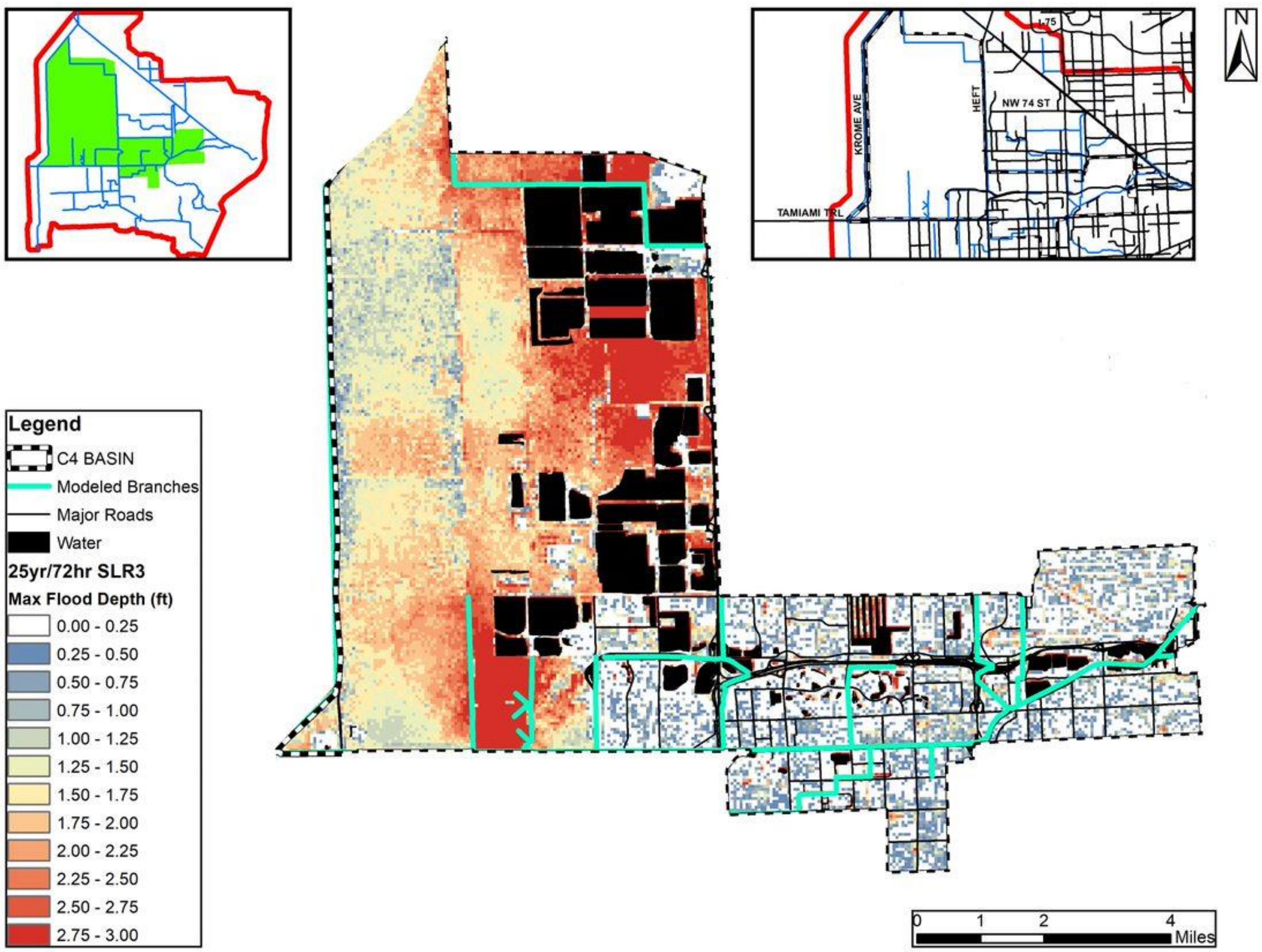


Figure C 3-13. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for Current Conditions in the C4 Watershed

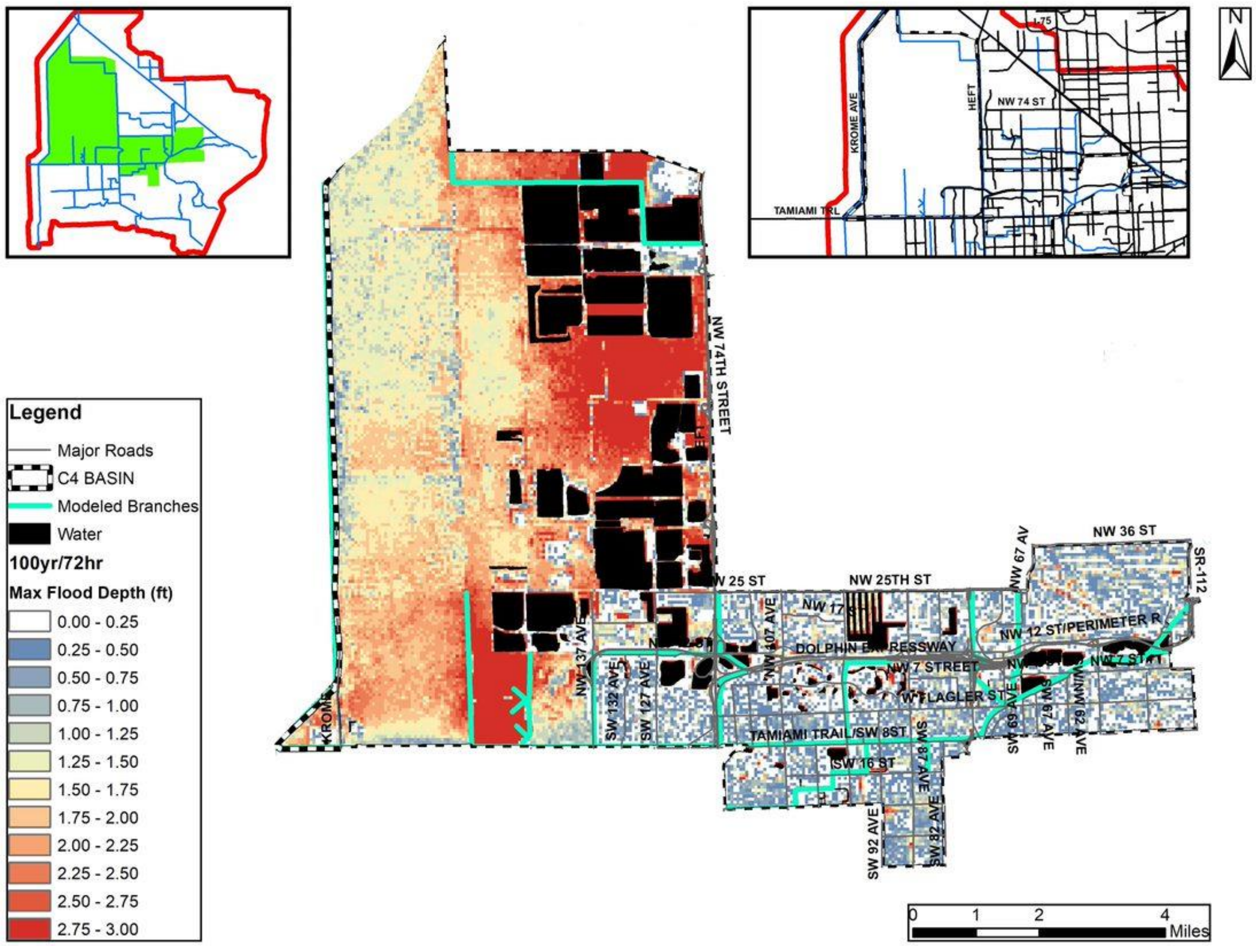


Figure C 3-14. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR1 in the C4 Watershed

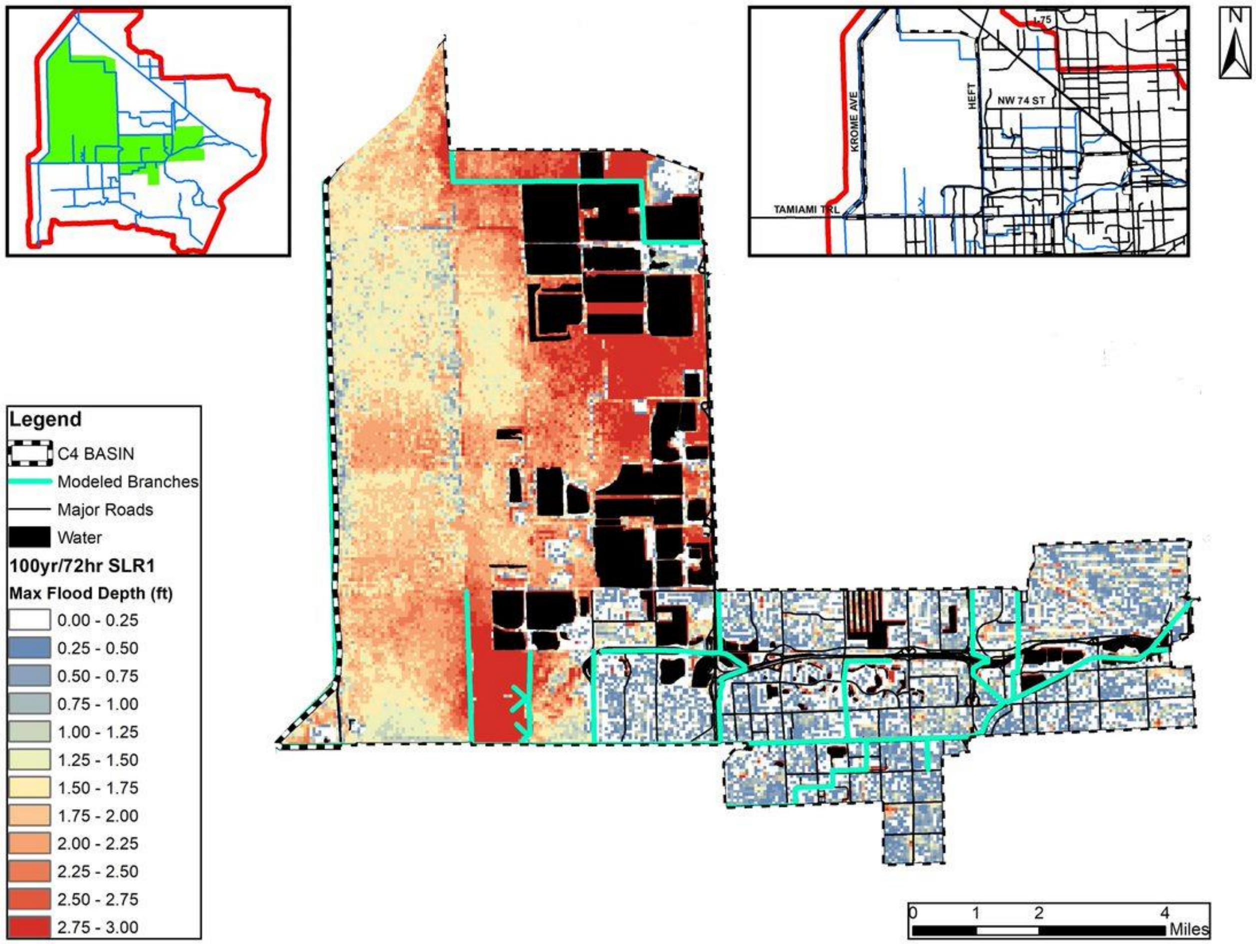


Figure C 3-15. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR2 in the C4 Watershed

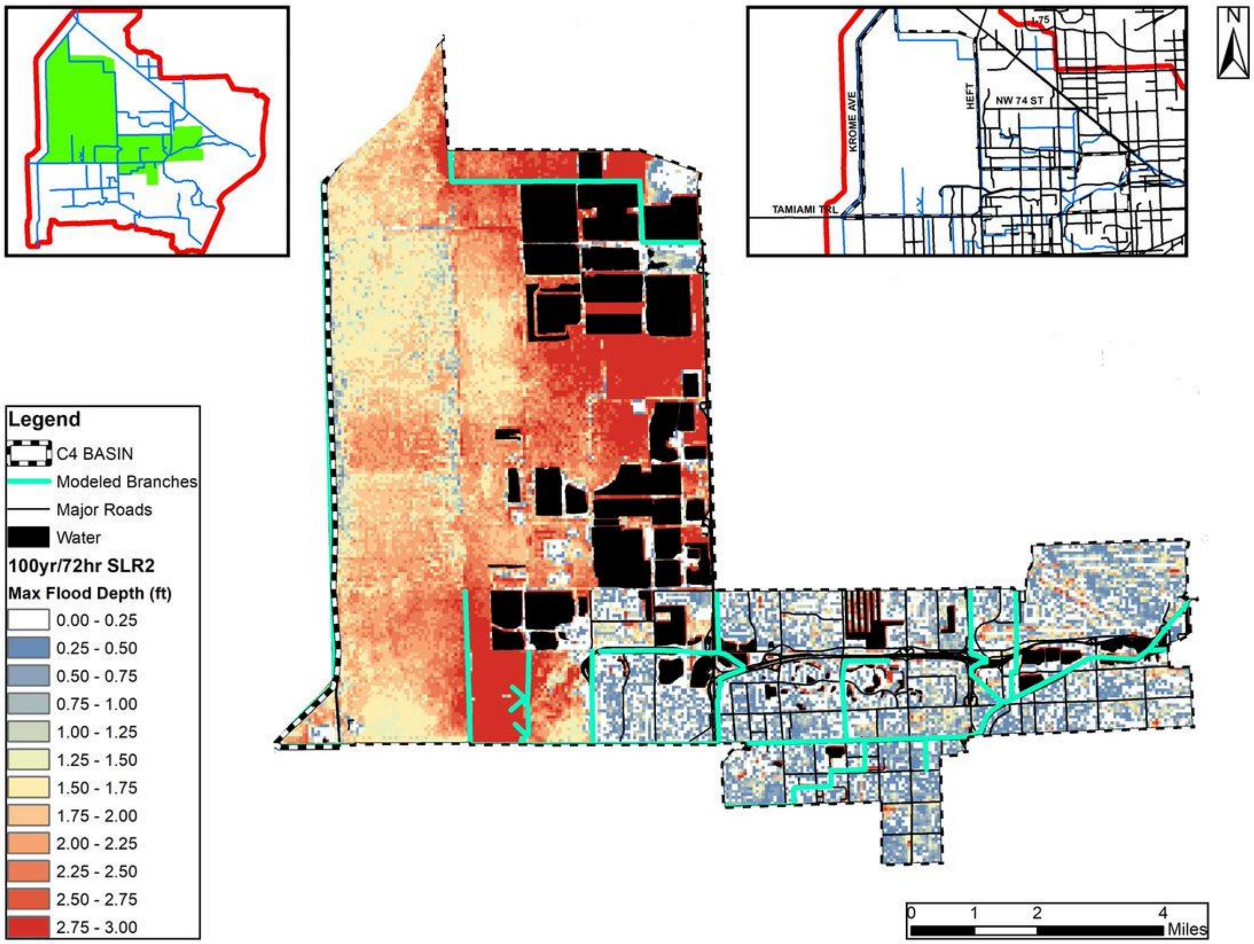


Figure C 3-16. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR3 in the C4 Watershed

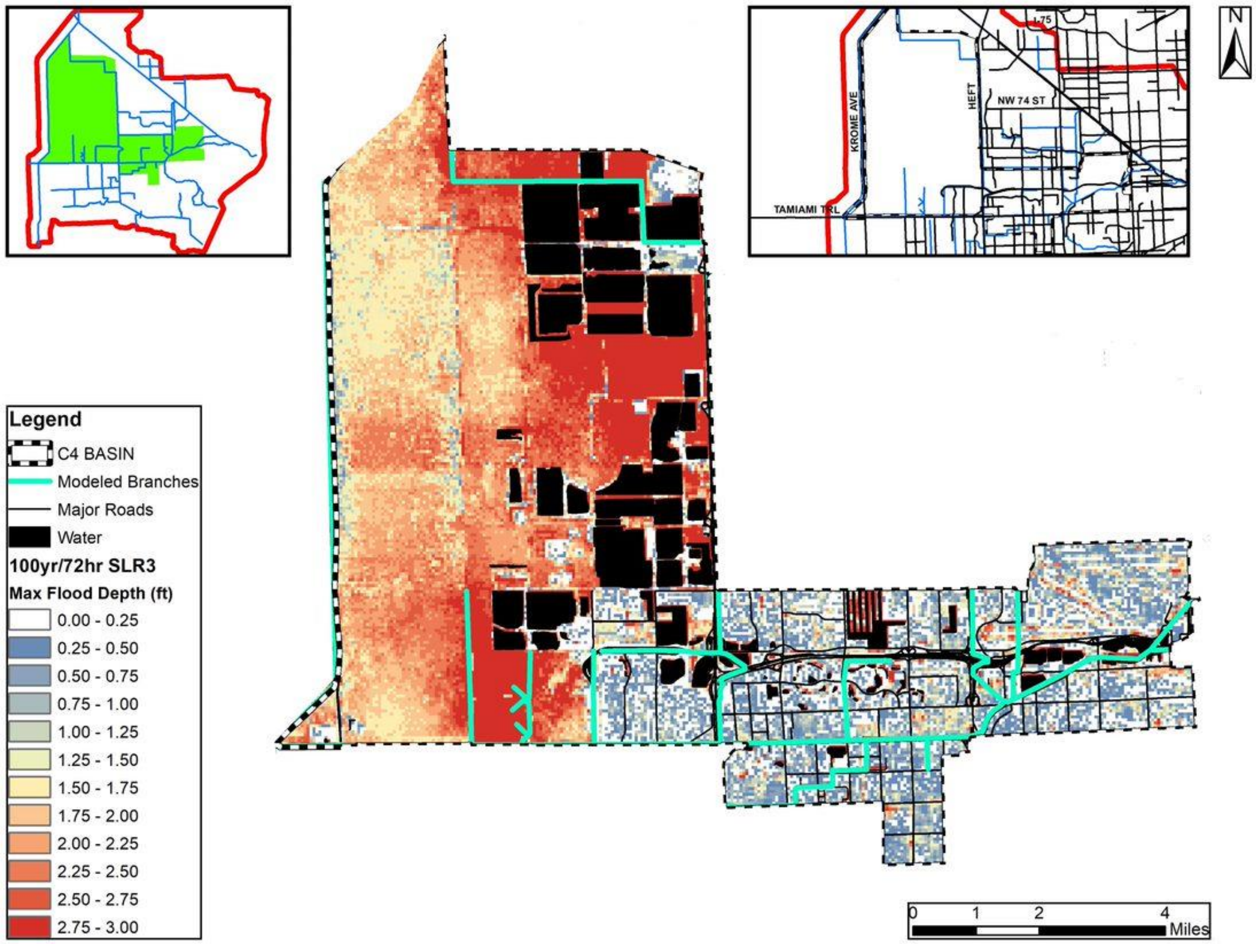


Figure C 3-17. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for Current Conditions in Urban Areas in the C4 Watershed

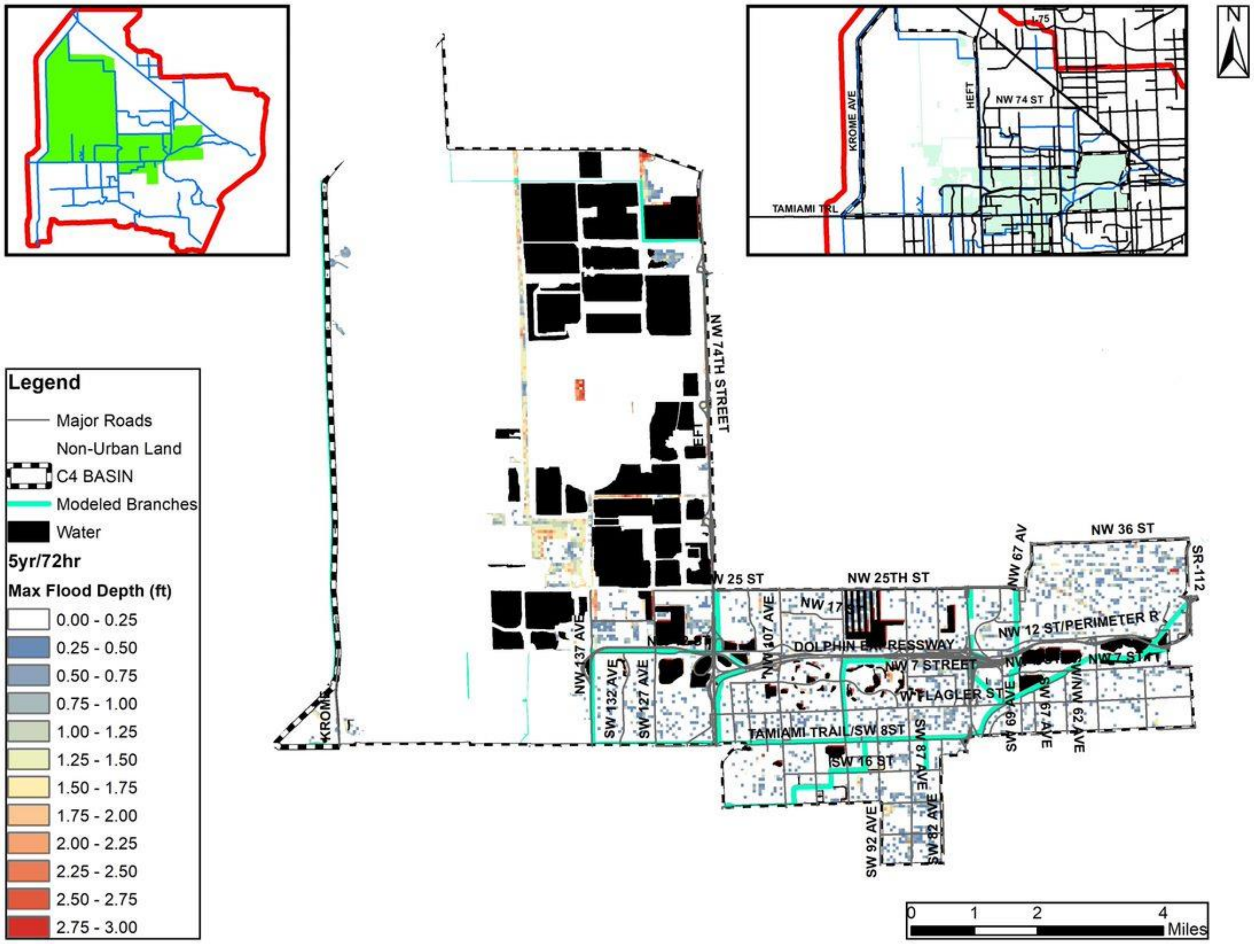


Figure C 3-18. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR1 in Urban Areas in the C4 Watershed

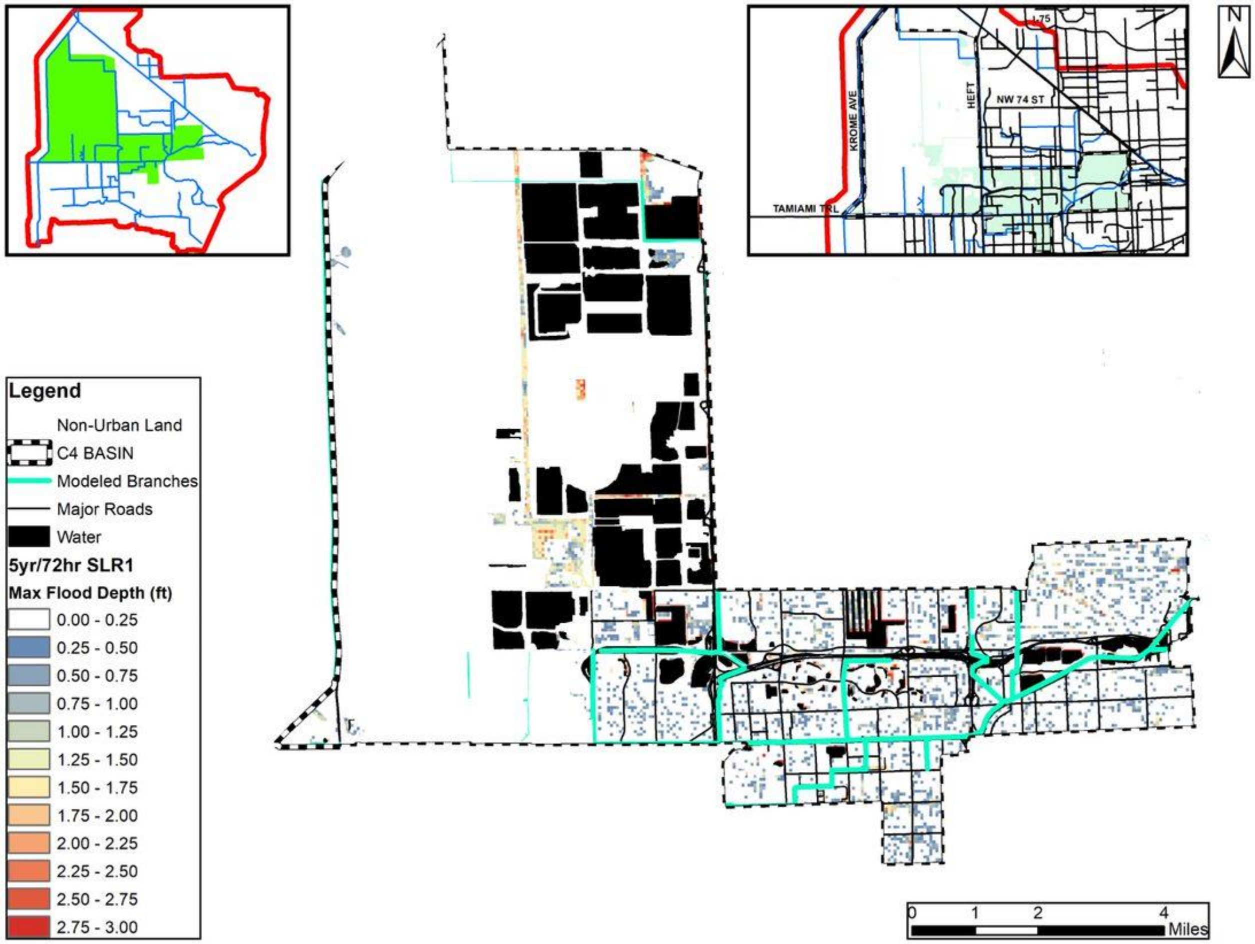


Figure C 3-19. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR2 in Urban Areas in the C4 Watershed

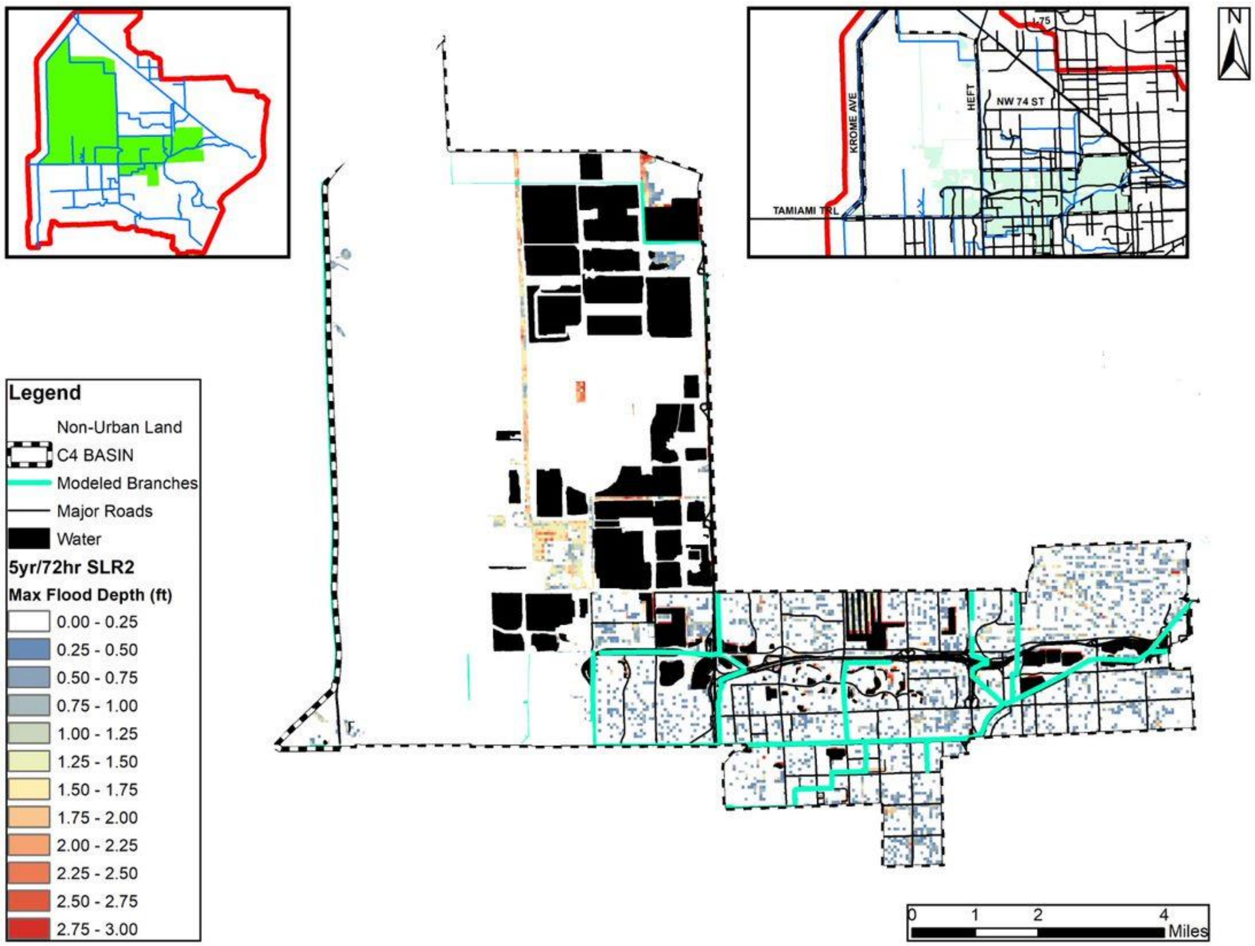


Figure C 3-20. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR3 in Urban Areas in the C4 Watershed

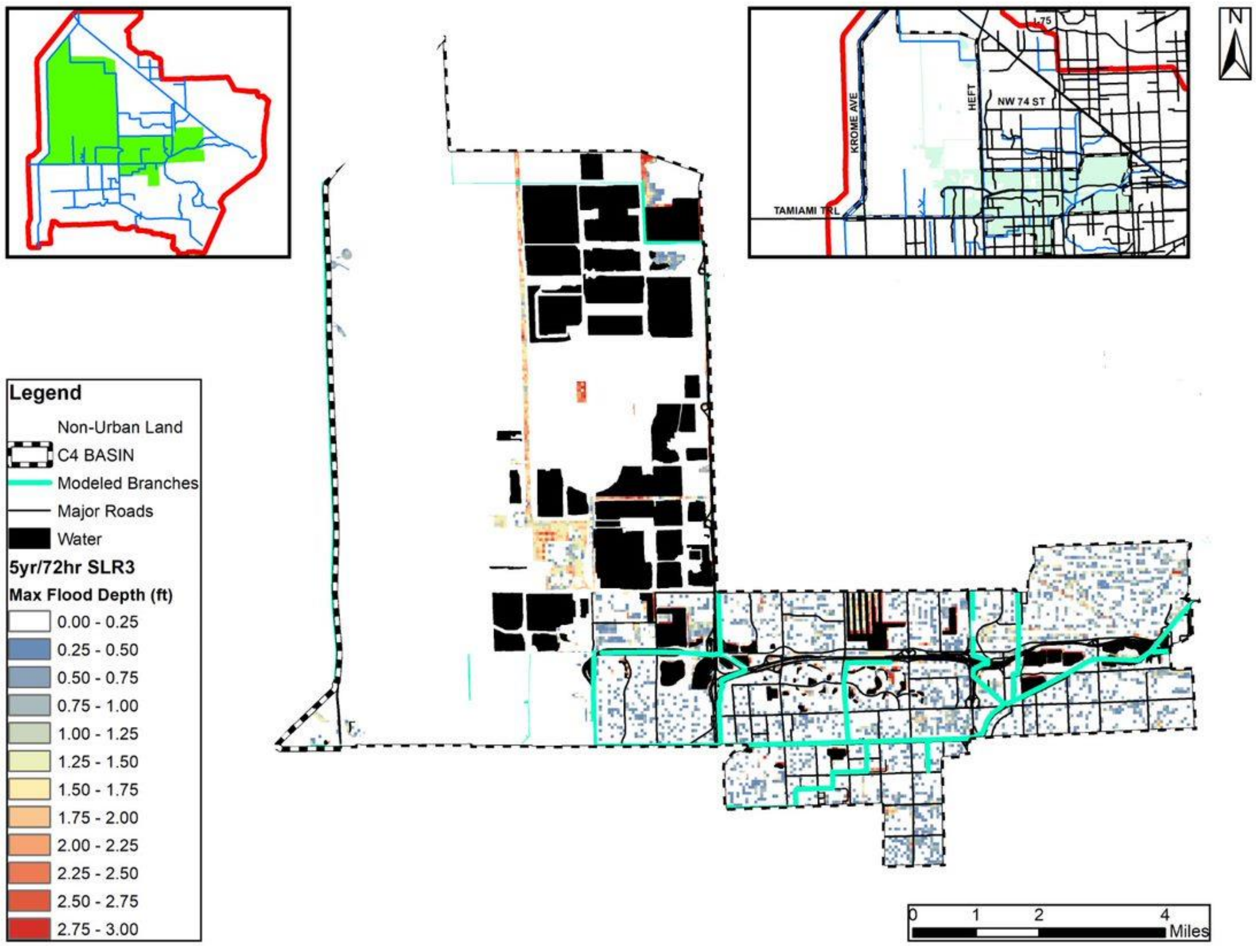


Figure C 3-21. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for Current Conditions in Urban Areas in the C4 Watershed

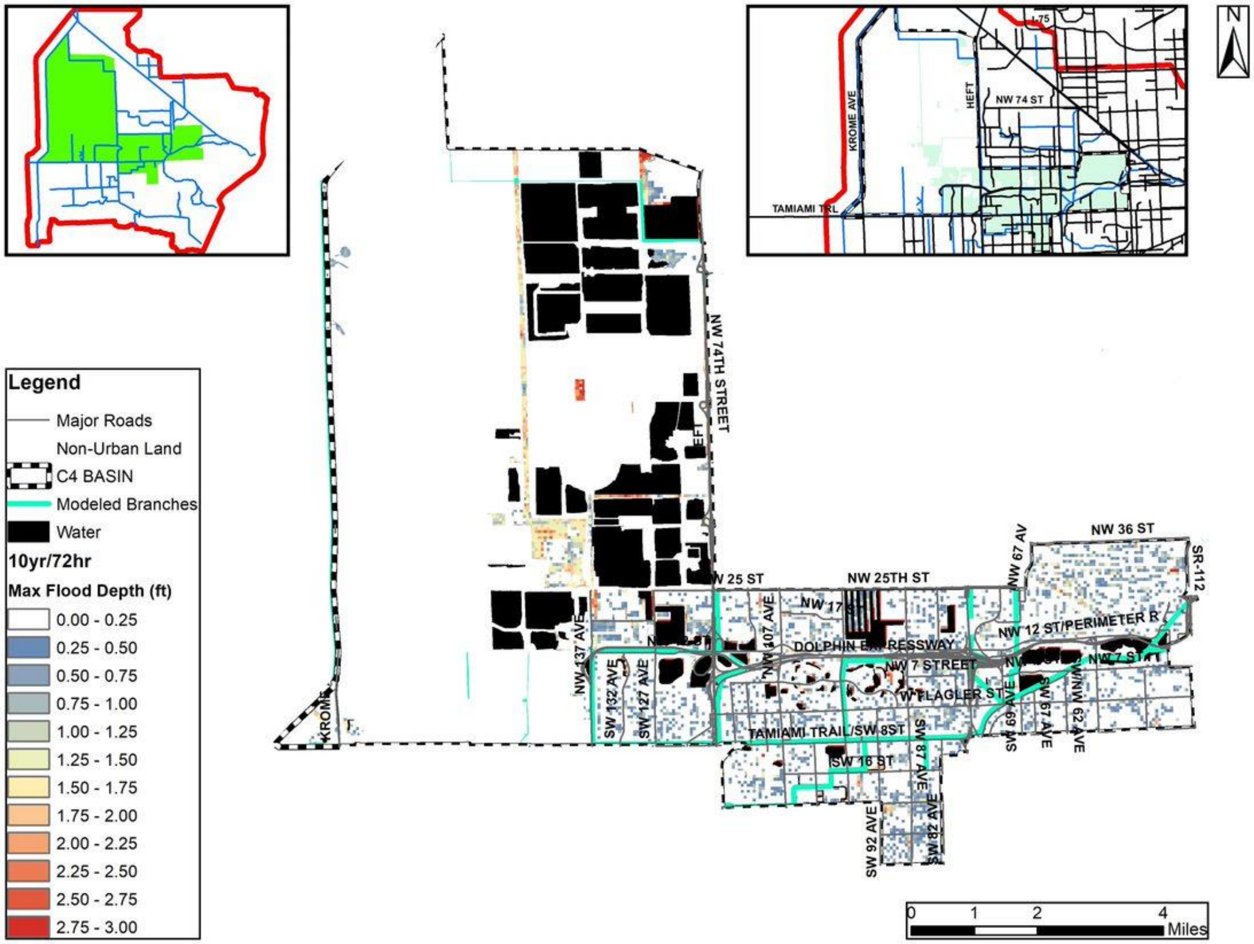


Figure C 3-22. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR1 in Urban Areas in the C4 Watershed

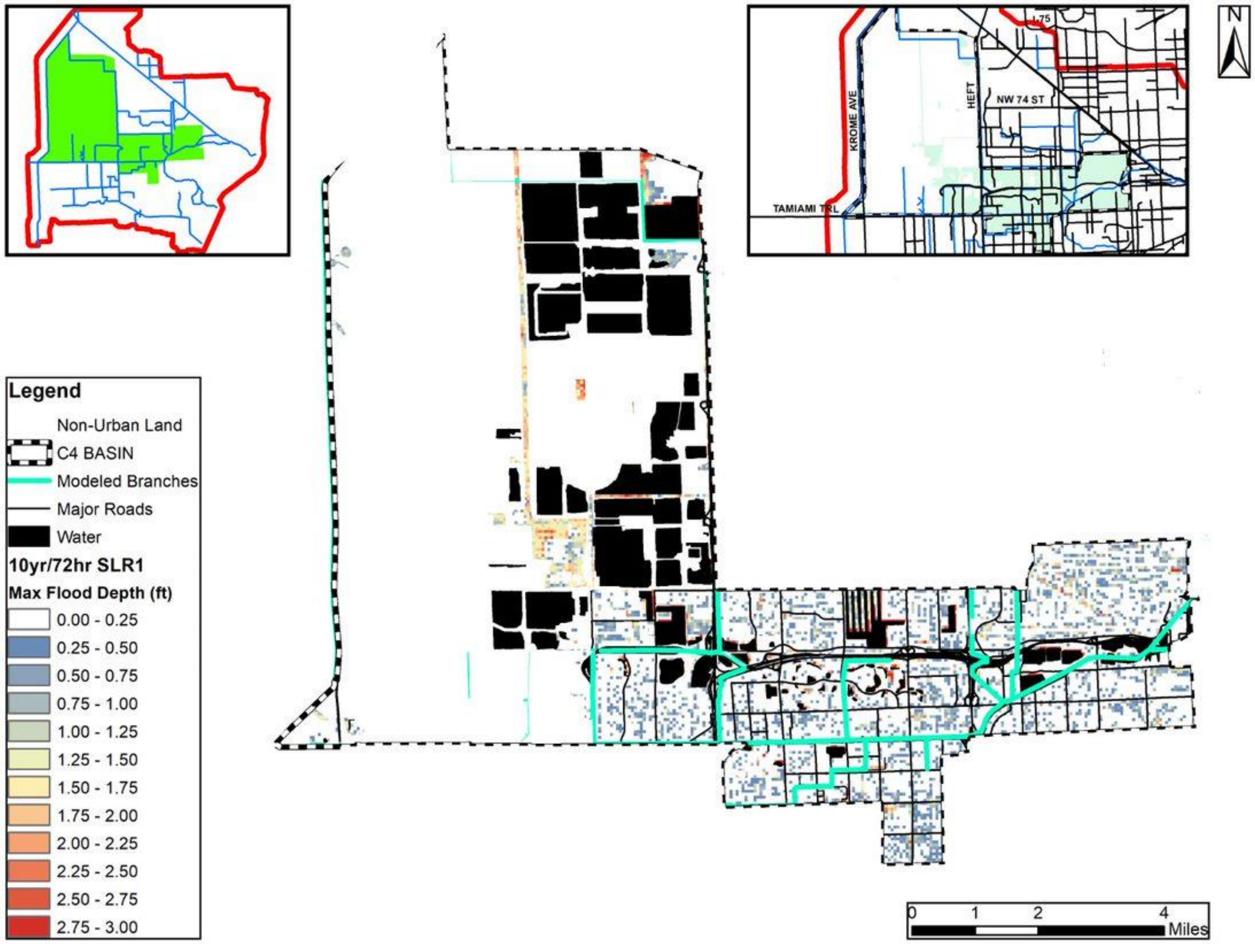


Figure C 3-23. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR2 in Urban Areas in the C4 Watershed

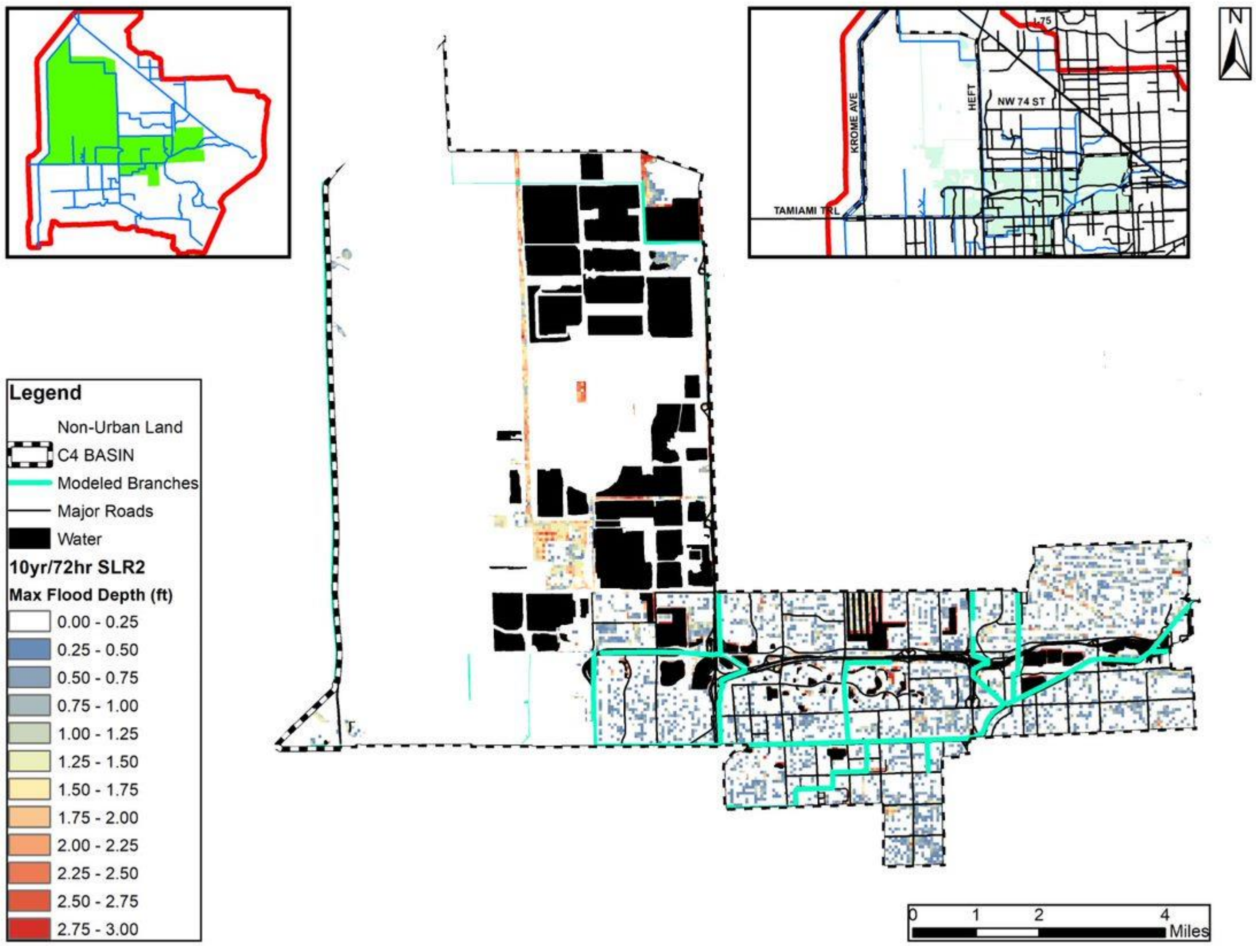


Figure C 3-24. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR3 in Urban Areas in the C4 Watershed

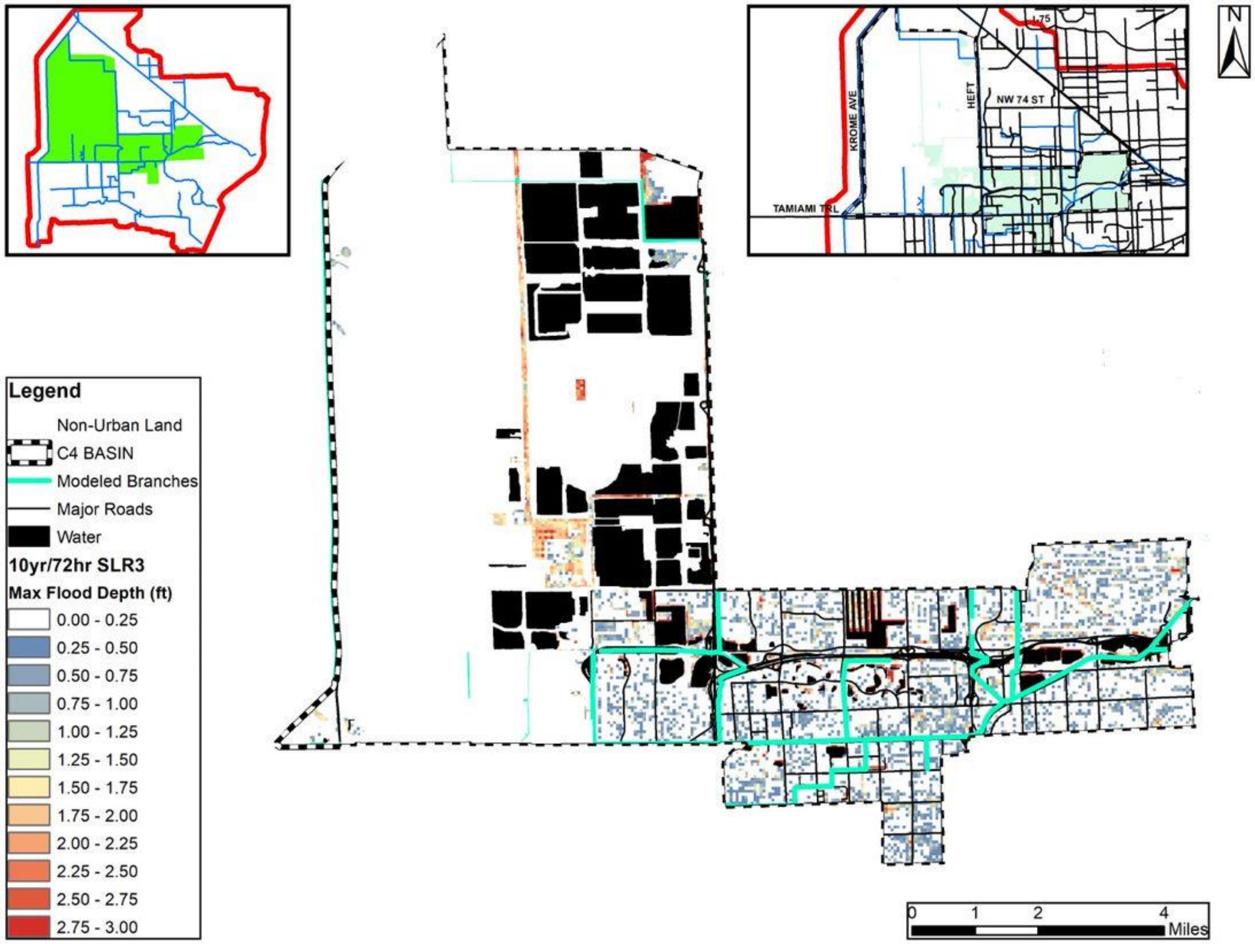


Figure C 3-25. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for Current Conditions in Urban Areas in the C4 Watershed

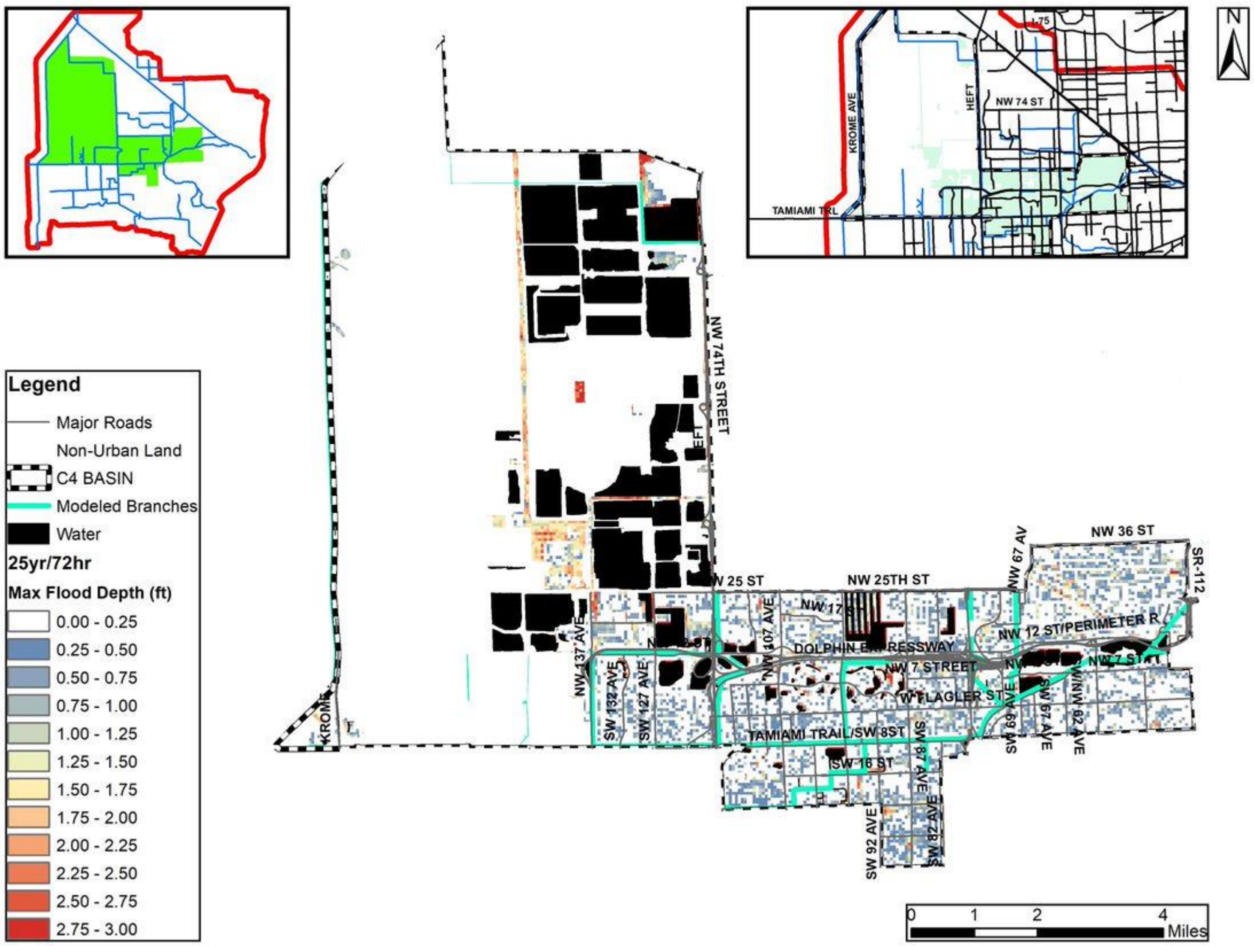


Figure C 3-26. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR1 in Urban Areas in the C4 Watershed

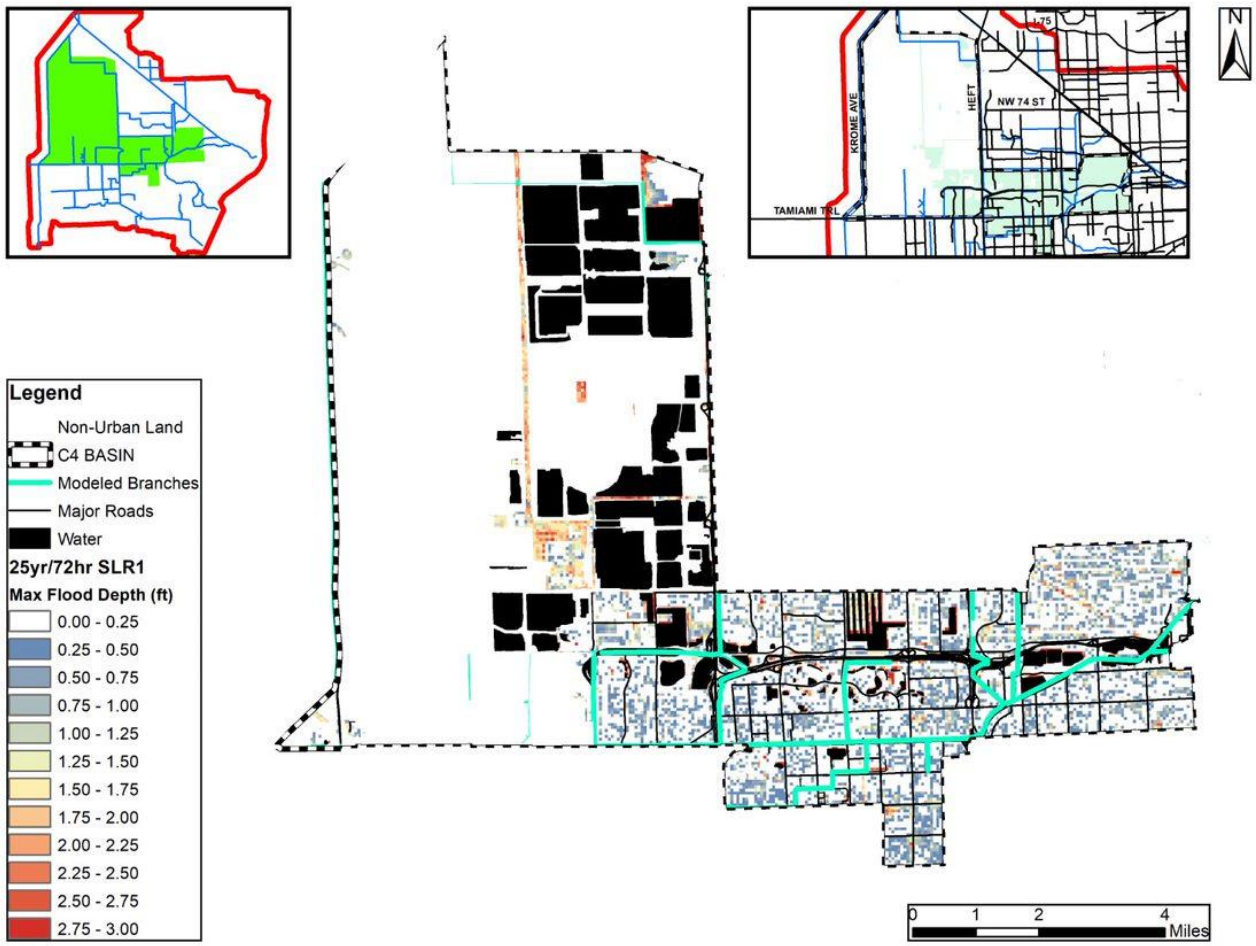


Figure C 3-27. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR2 in Urban Areas in the C4 Watershed

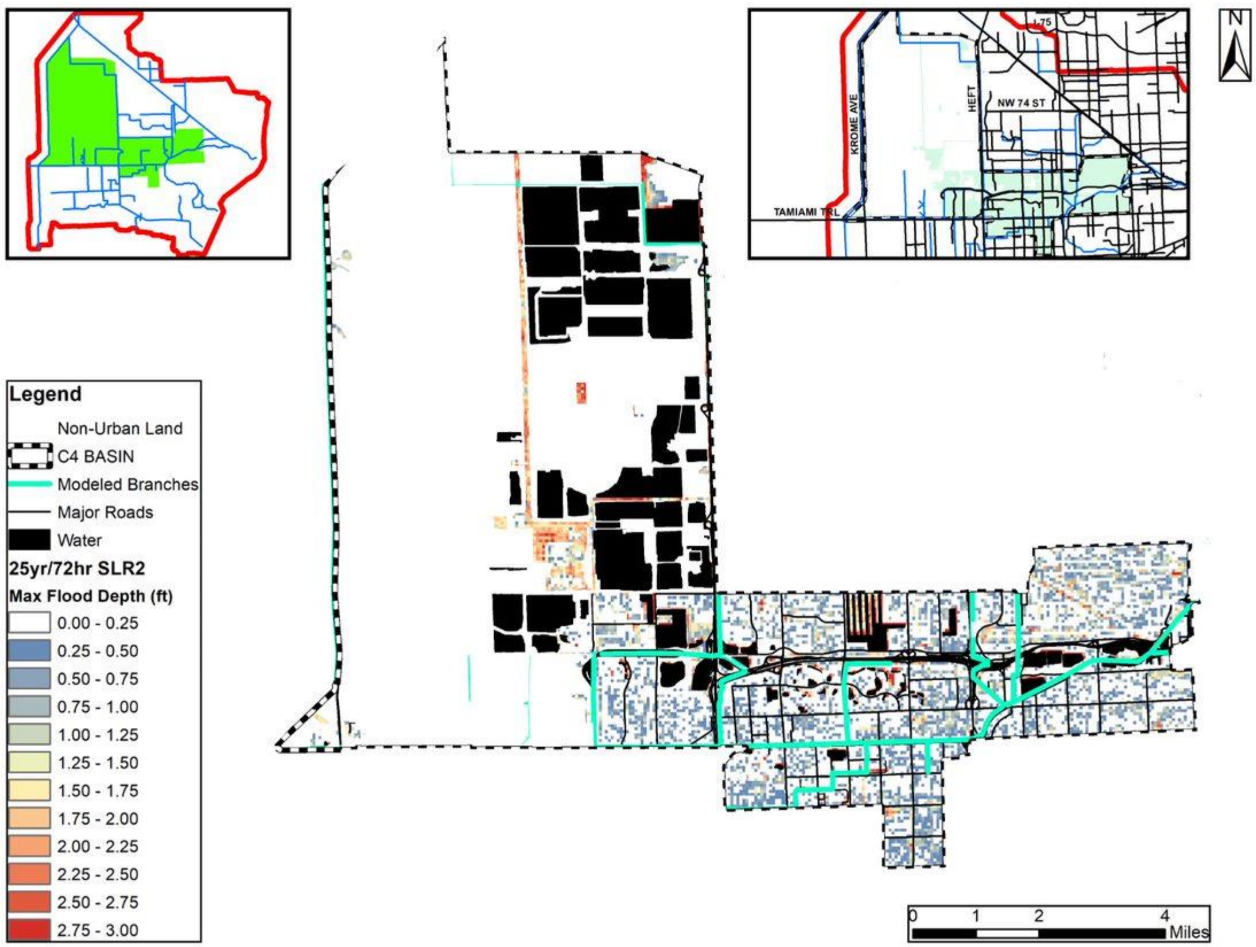


Figure C 3-28. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR3 in Urban Areas in the C4 Watershed

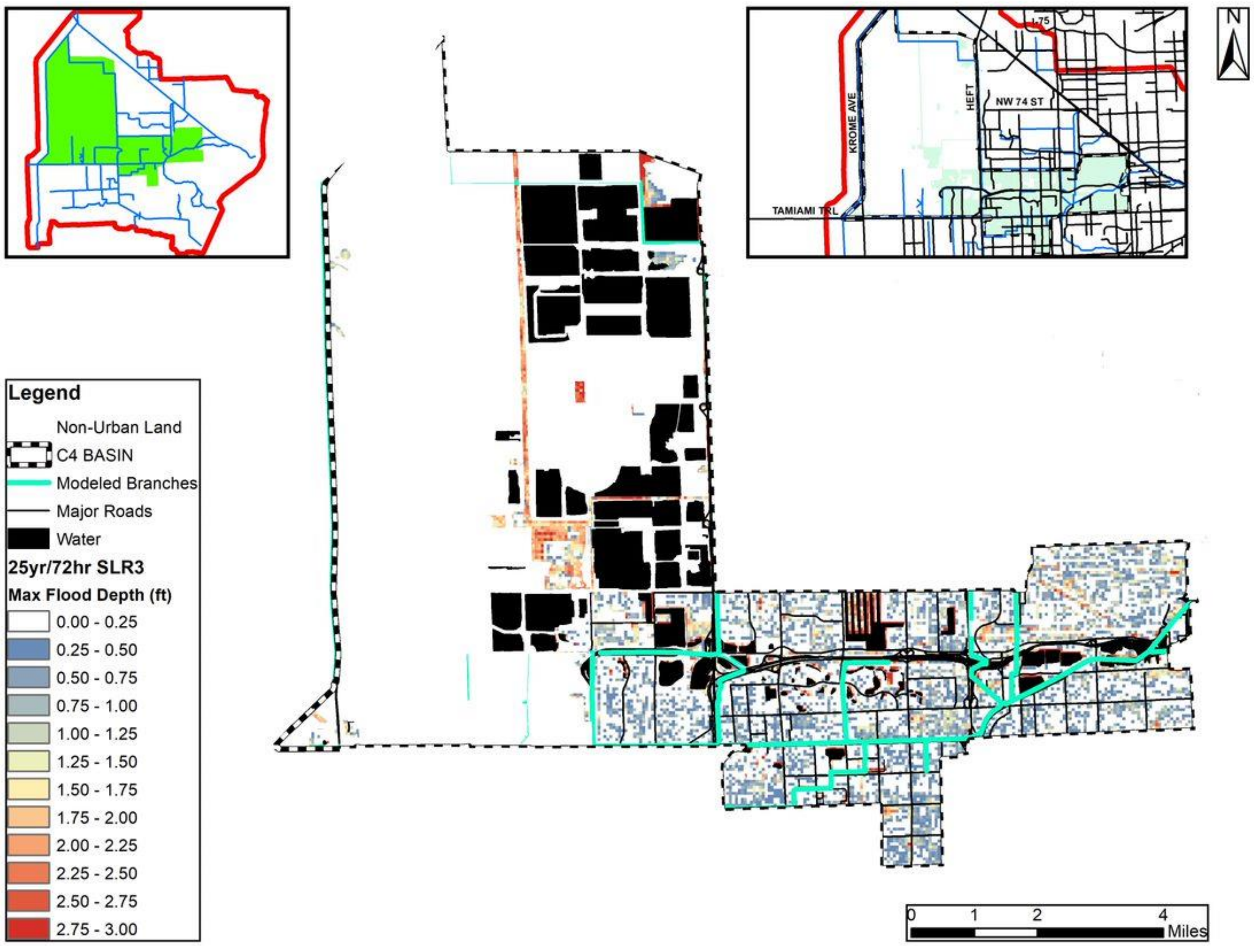


Figure C 3-29. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for Current Conditions in Urban Areas in the C4 Watershed

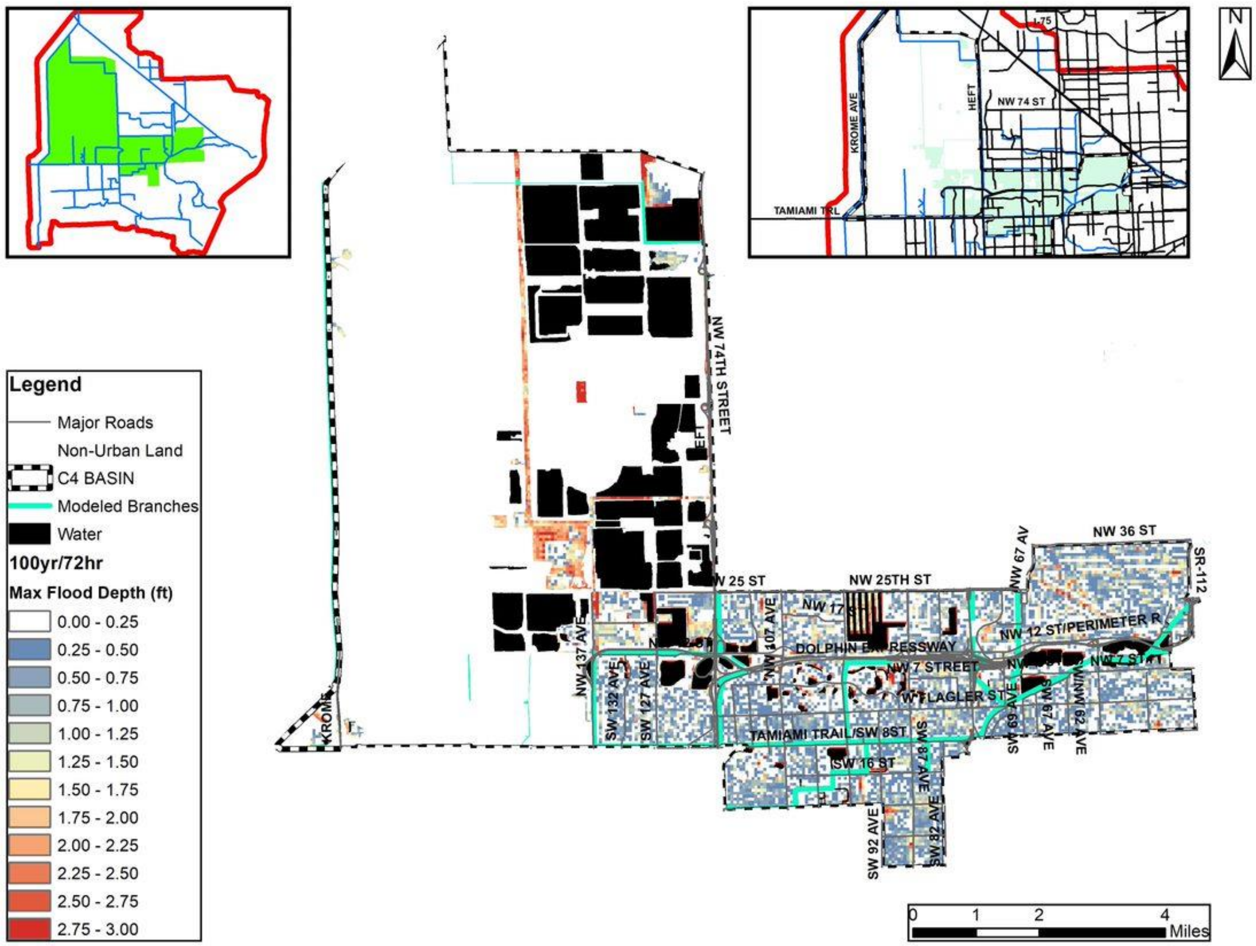


Figure C 3-30. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR1 in Urban Areas in the C4 Watershed

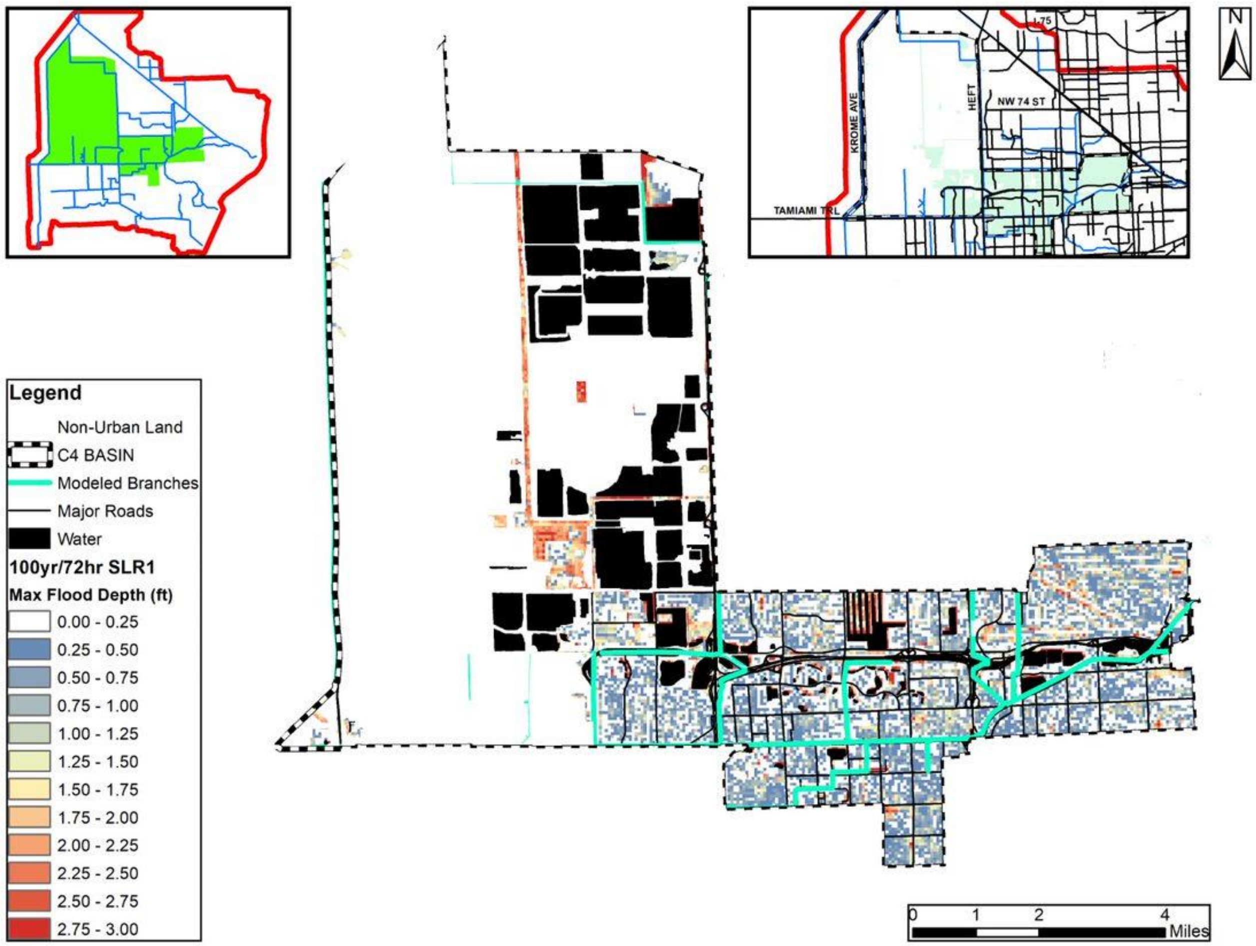


Figure C 3-31. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR2 in Urban Areas in the C4 Watershed

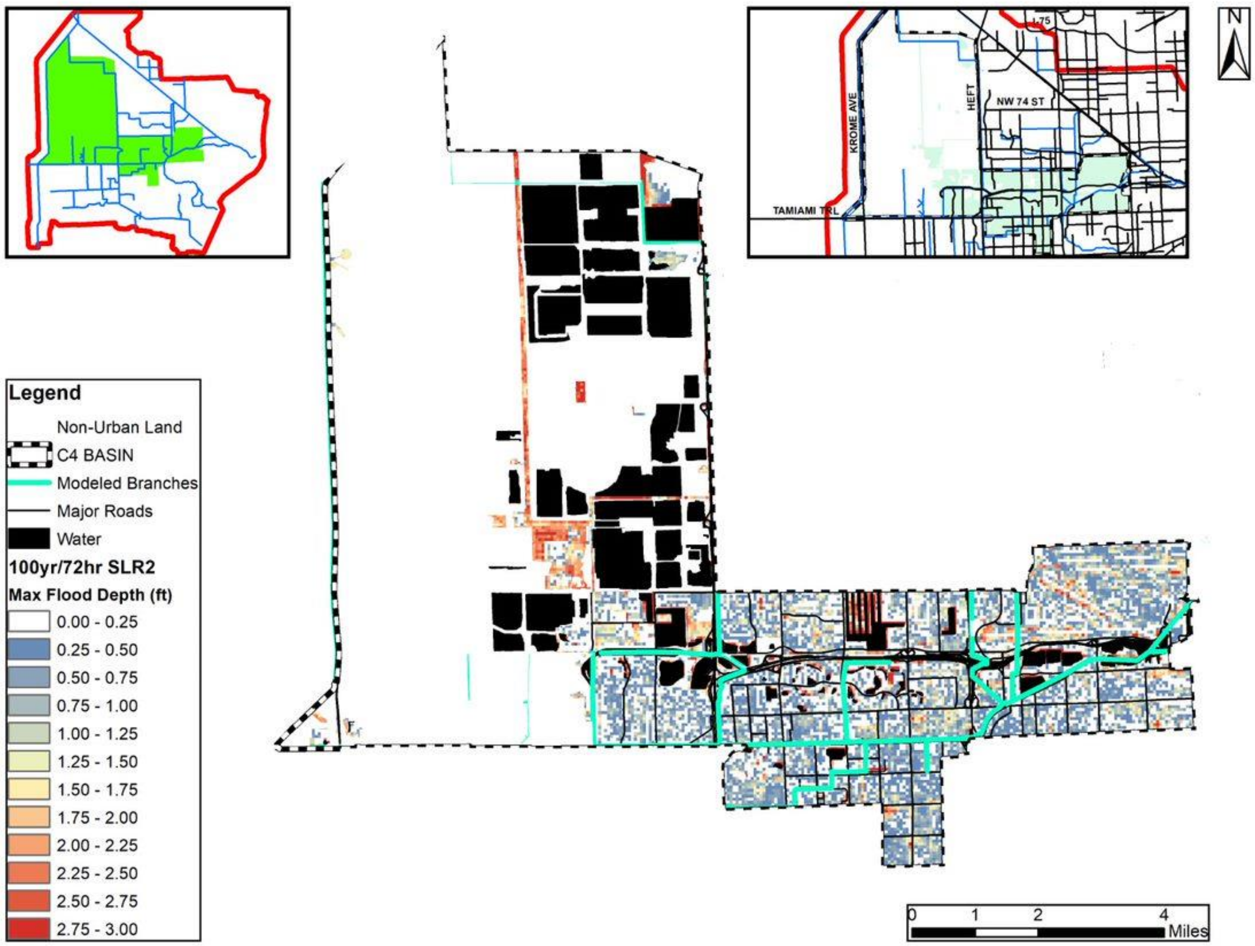


Figure C 3-32. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR3 in Urban Areas in the C4 Watershed

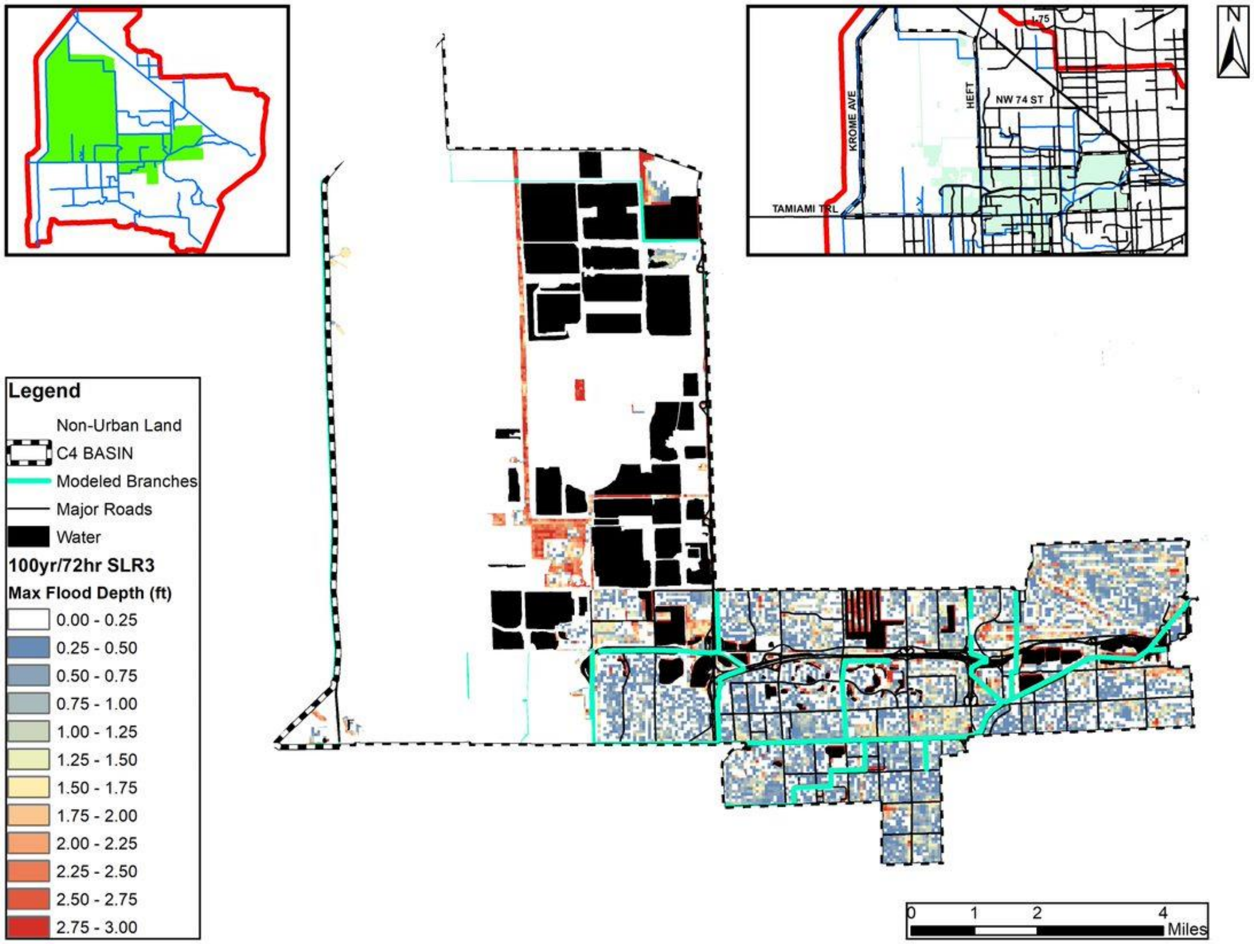


Figure C 3-33. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C4 Watershed

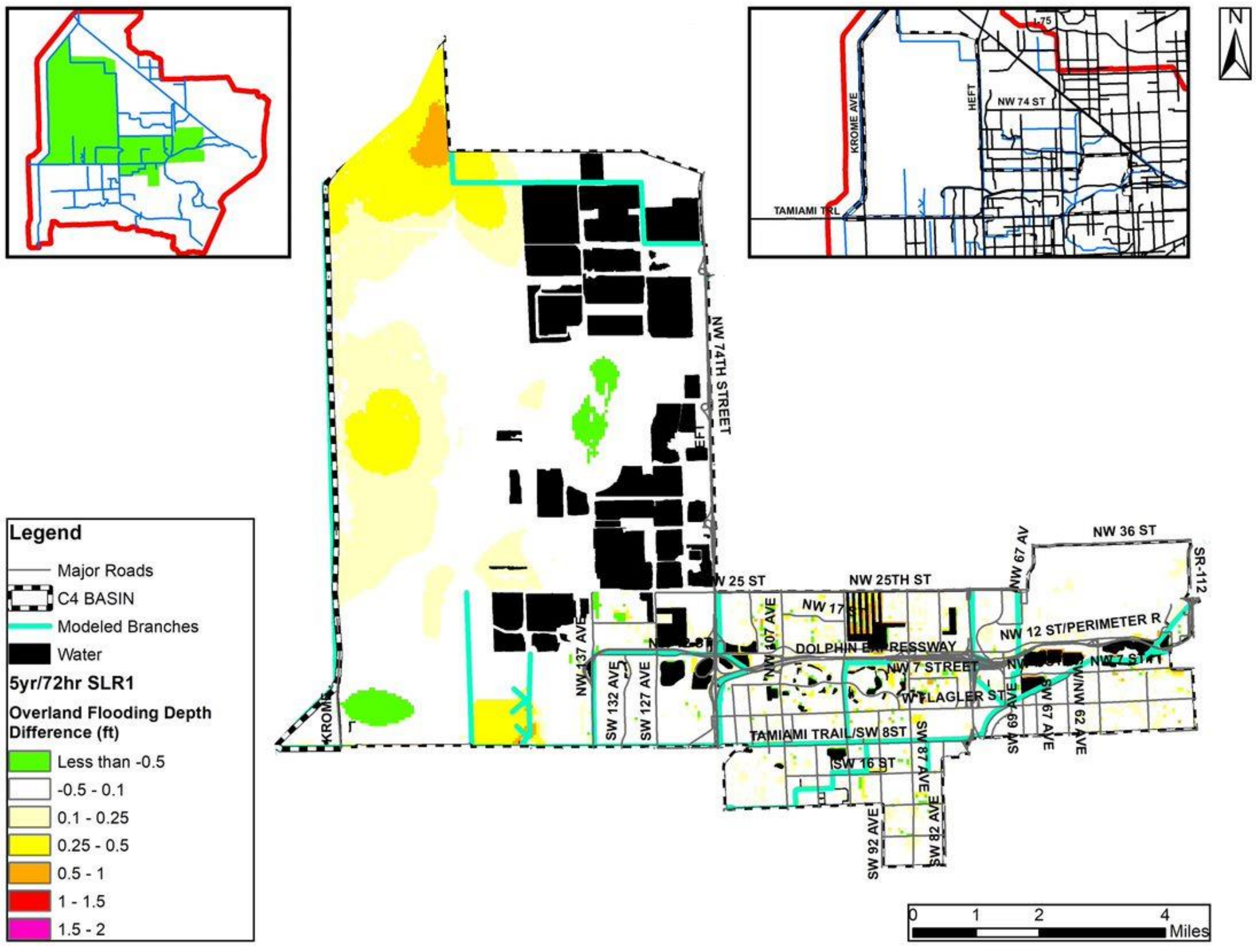


Figure C 3-34. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C4 Watershed

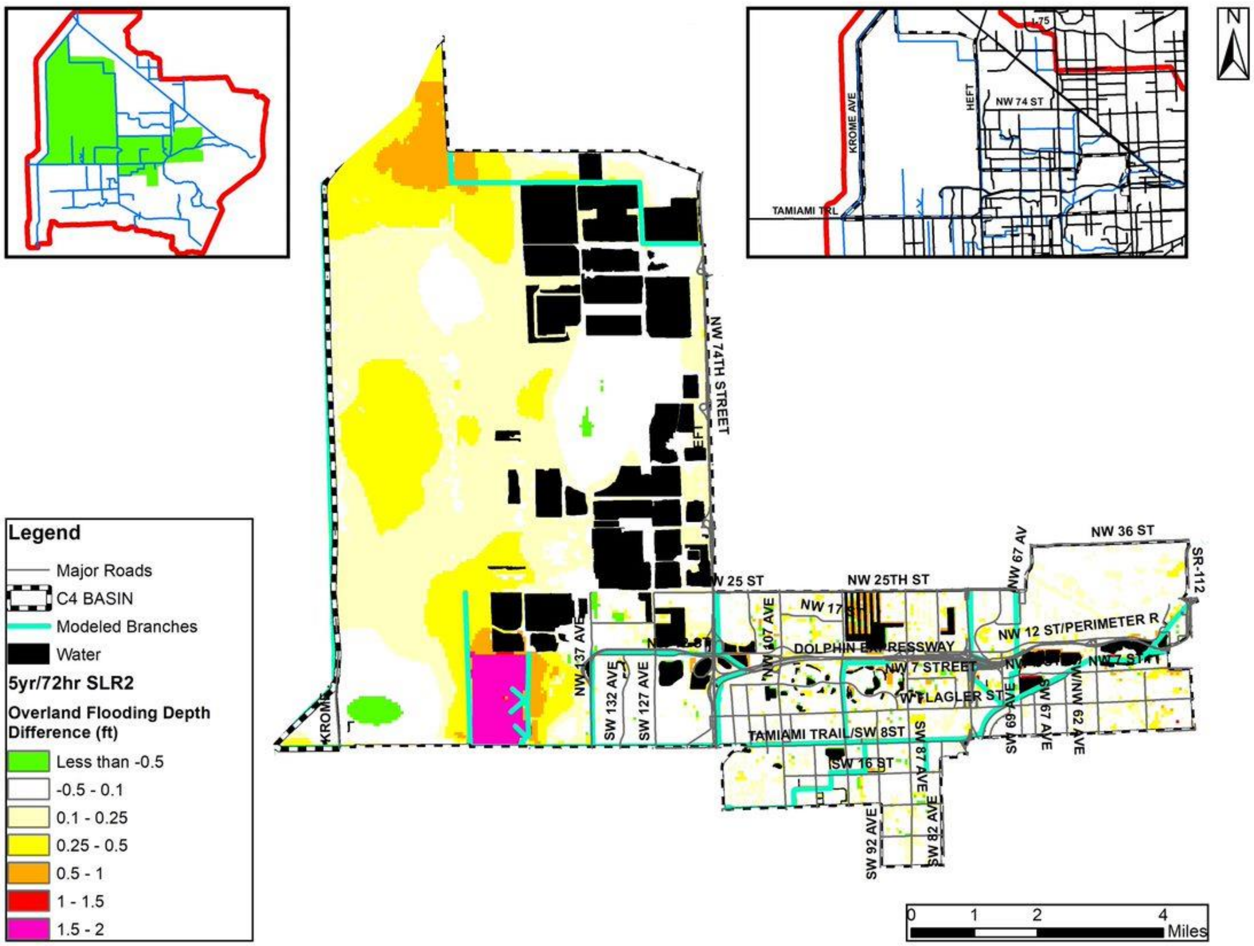


Figure C 3-35. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C4 Watershed

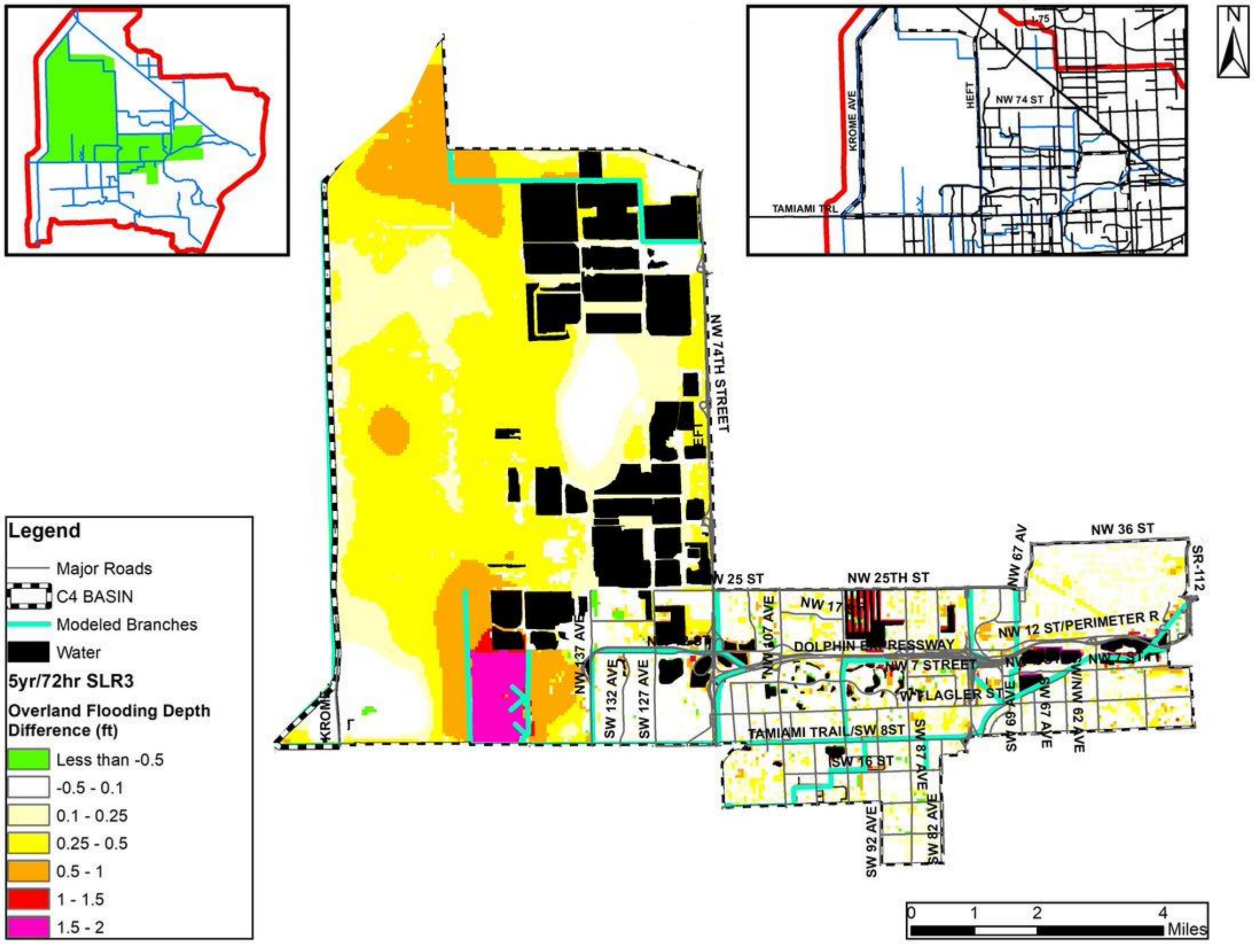


Figure C 3-36. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C4 Watershed

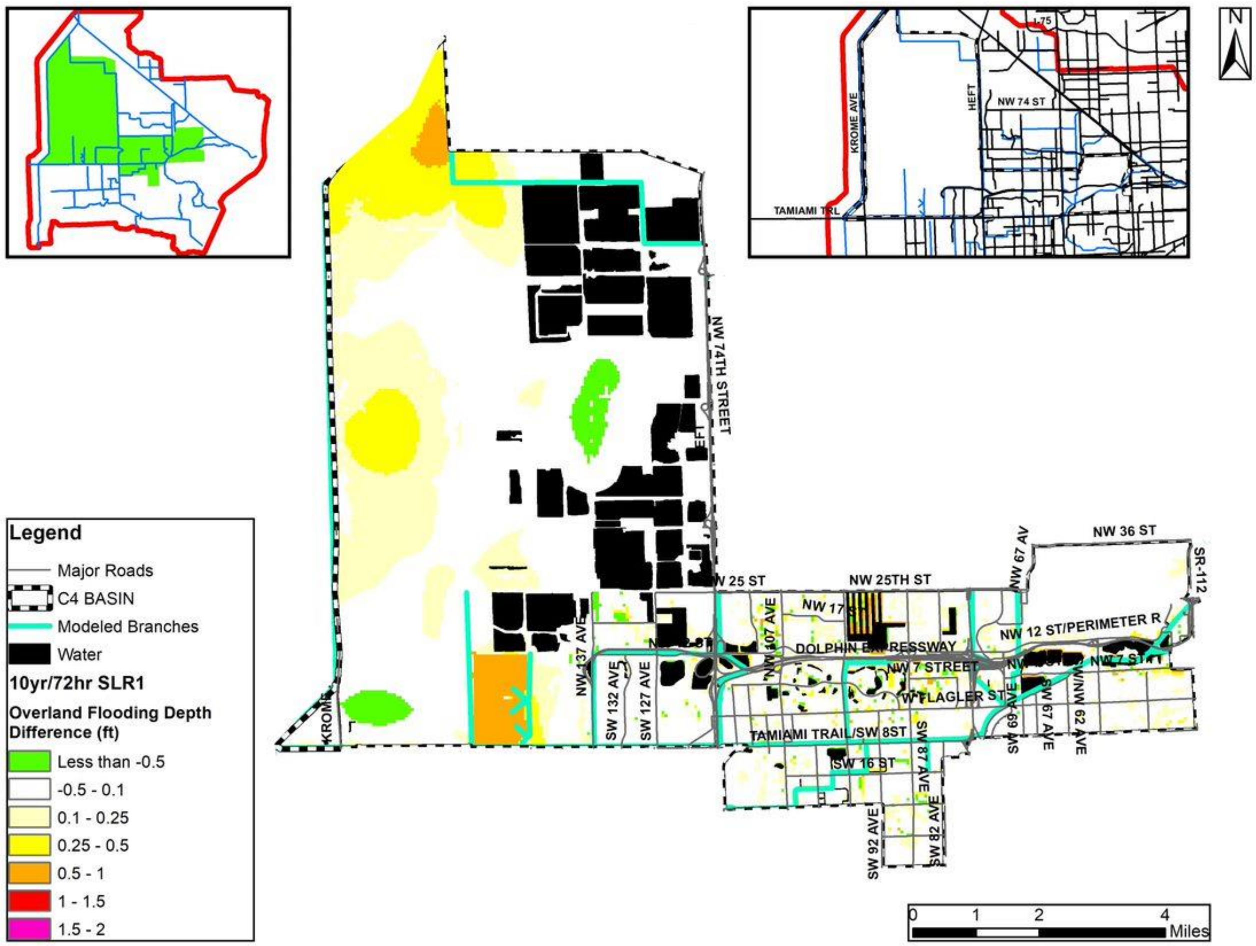


Figure C 3-37. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C4 Watershed

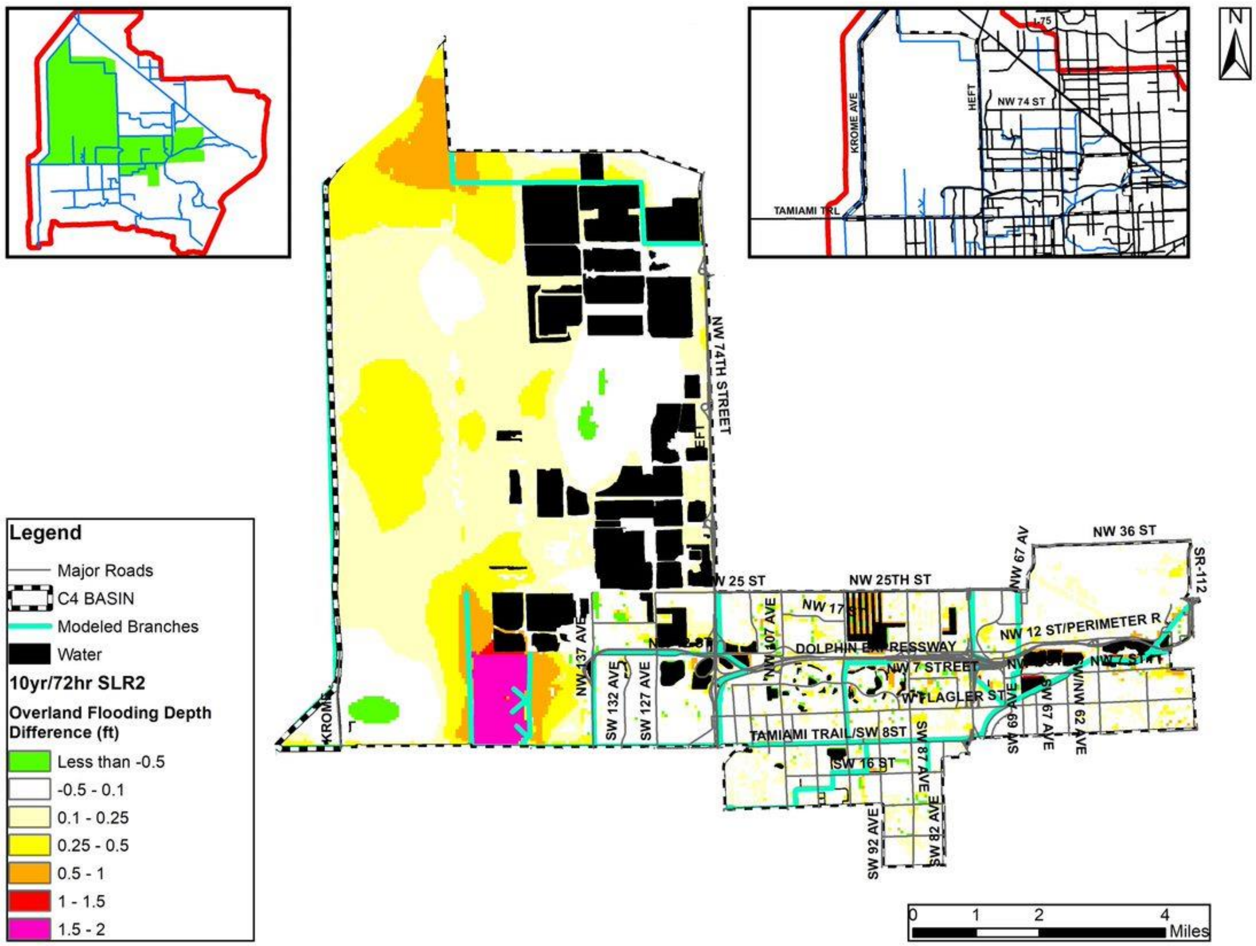


Figure C 3-38. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C4 Watershed

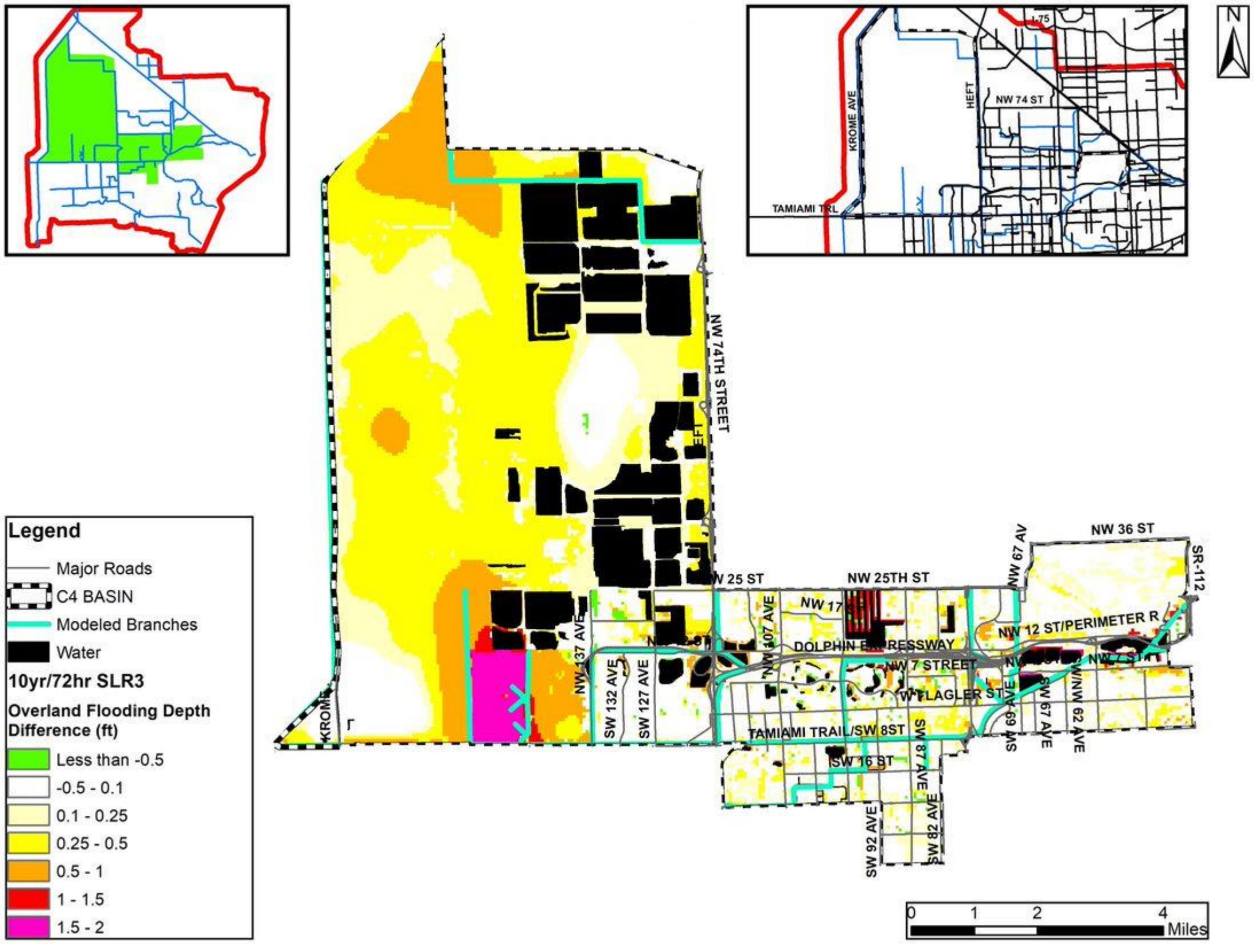


Figure C 3-39. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C4 Watershed

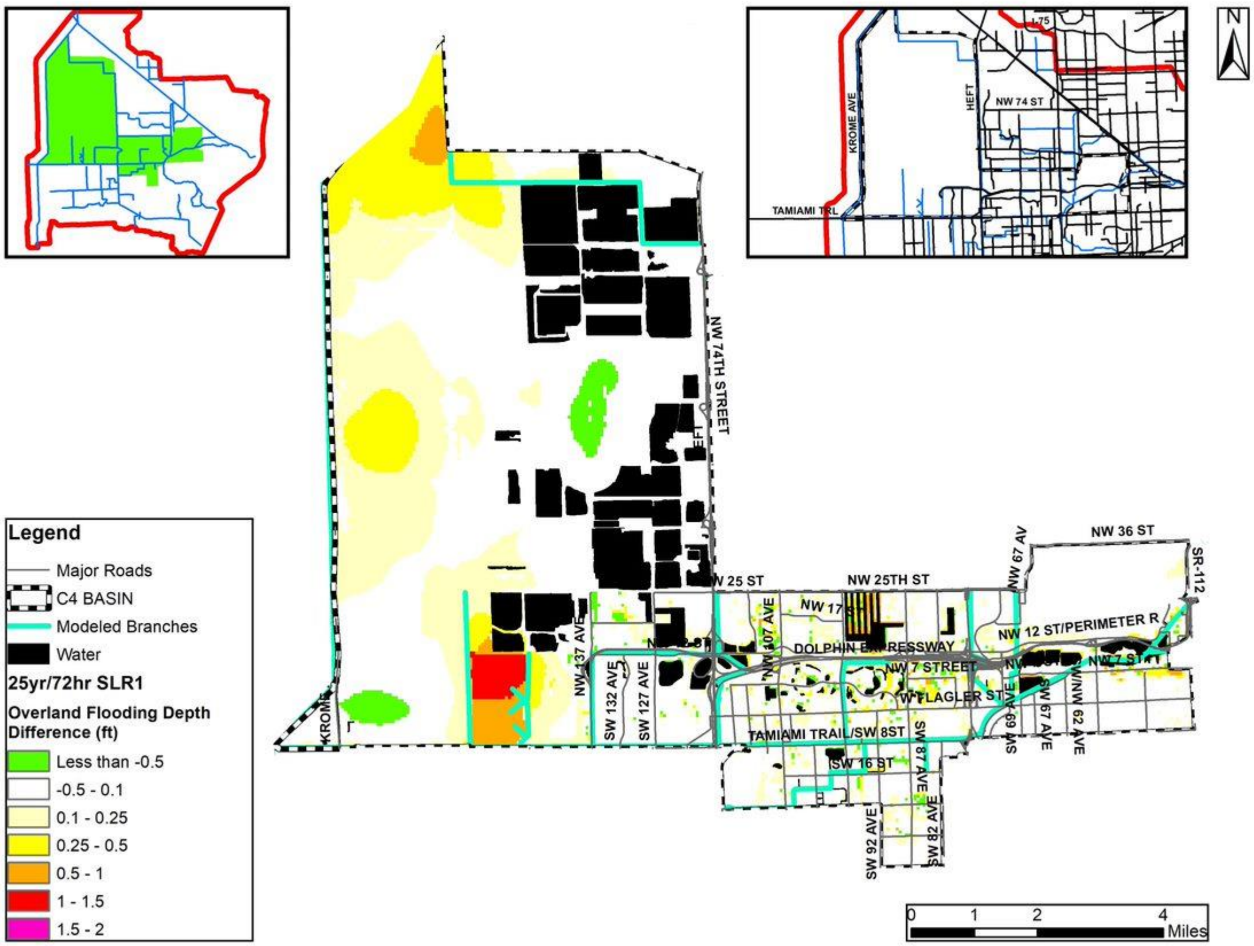


Figure C 3-40. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C4 Watershed

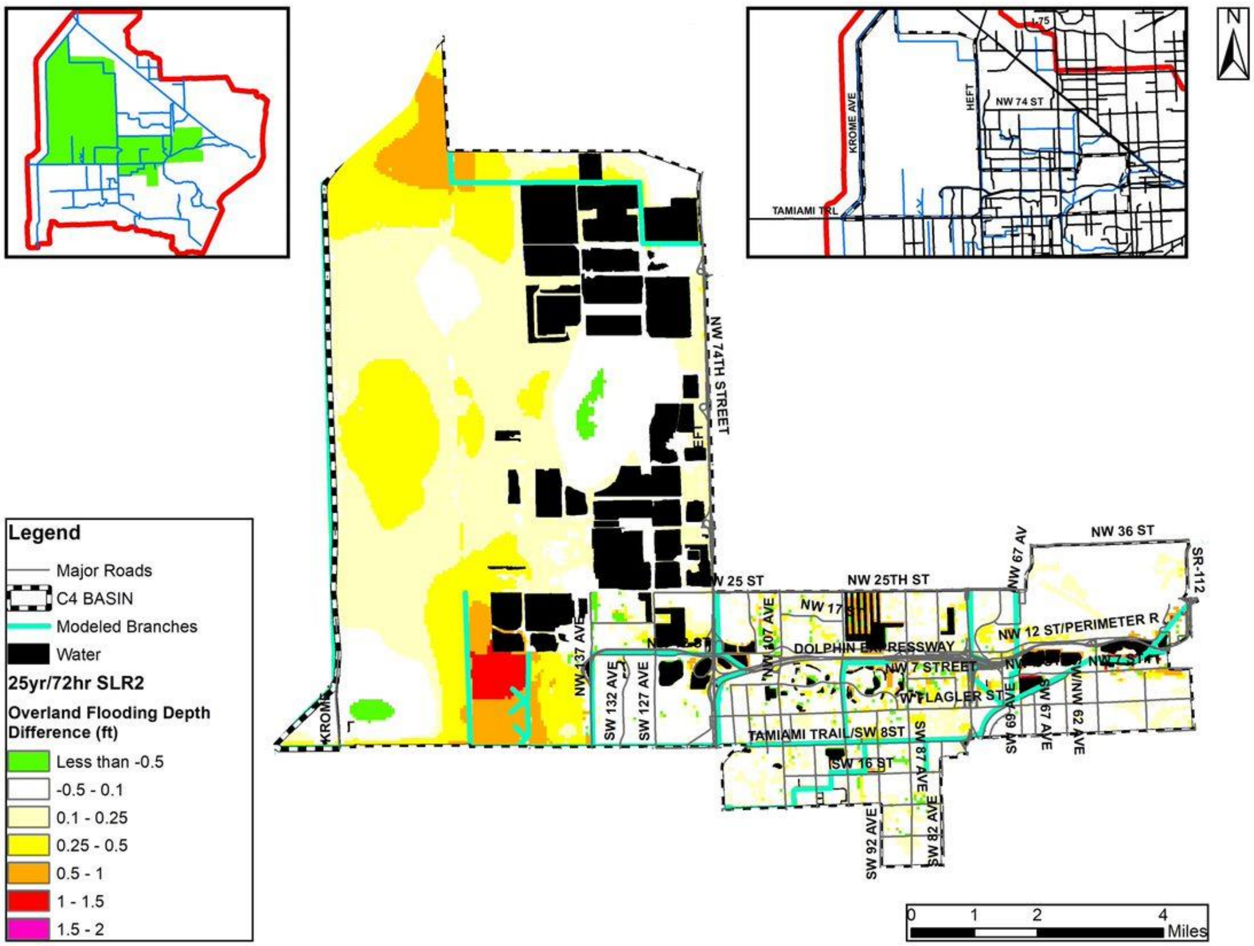


Figure C 3-41. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C4 Watershed

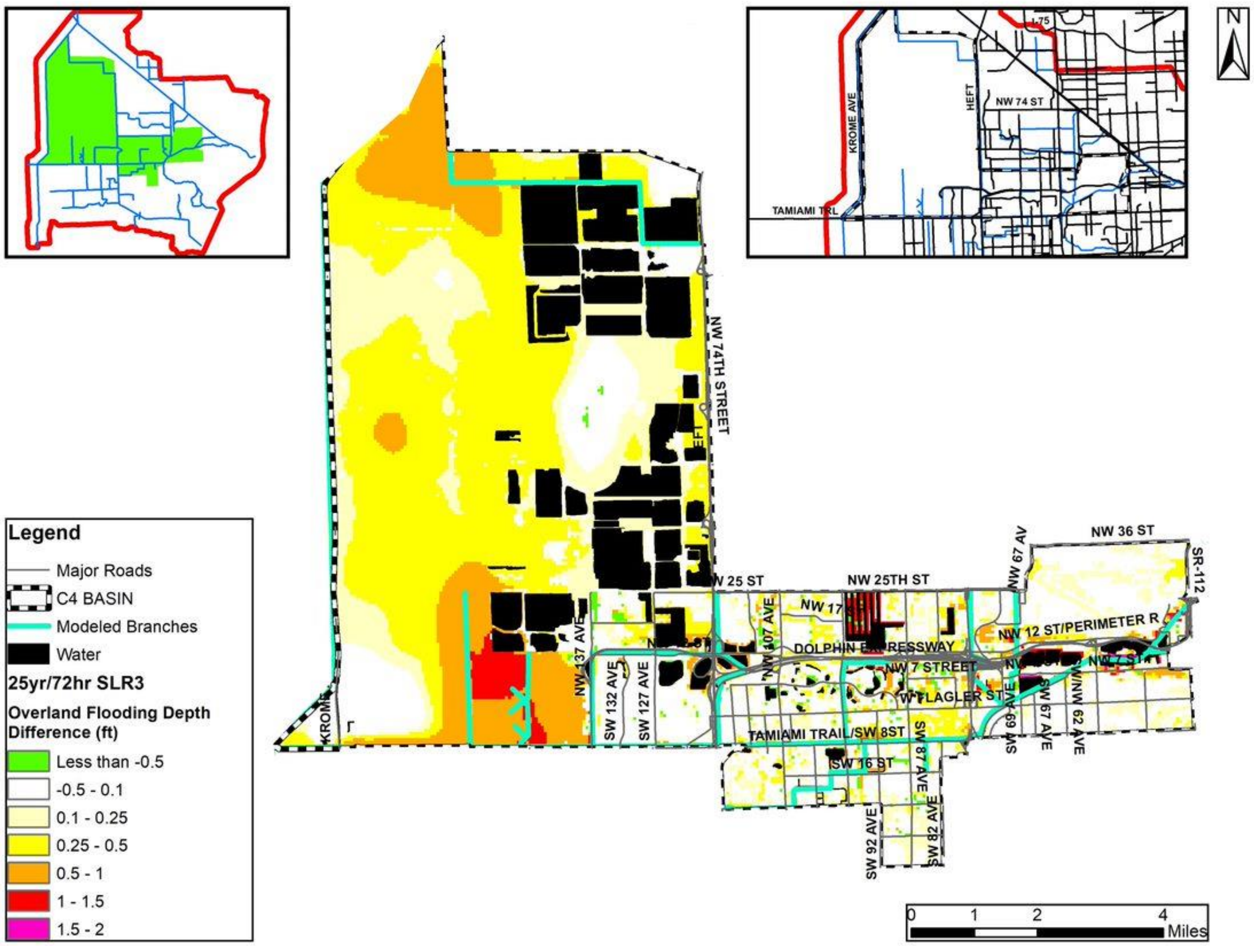


Figure C 3-42. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C4 Watershed

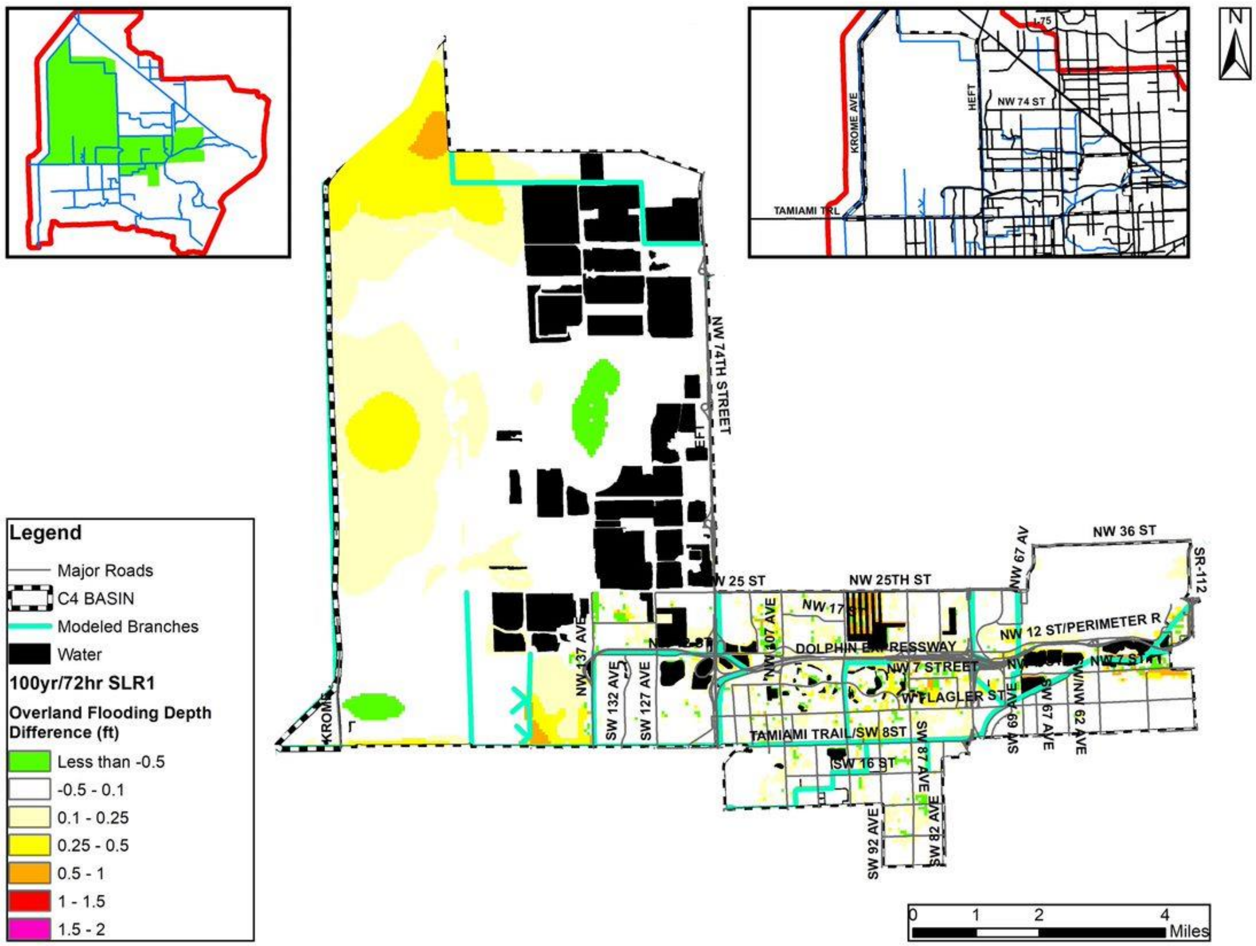


Figure C 3-43. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C4 Watershed

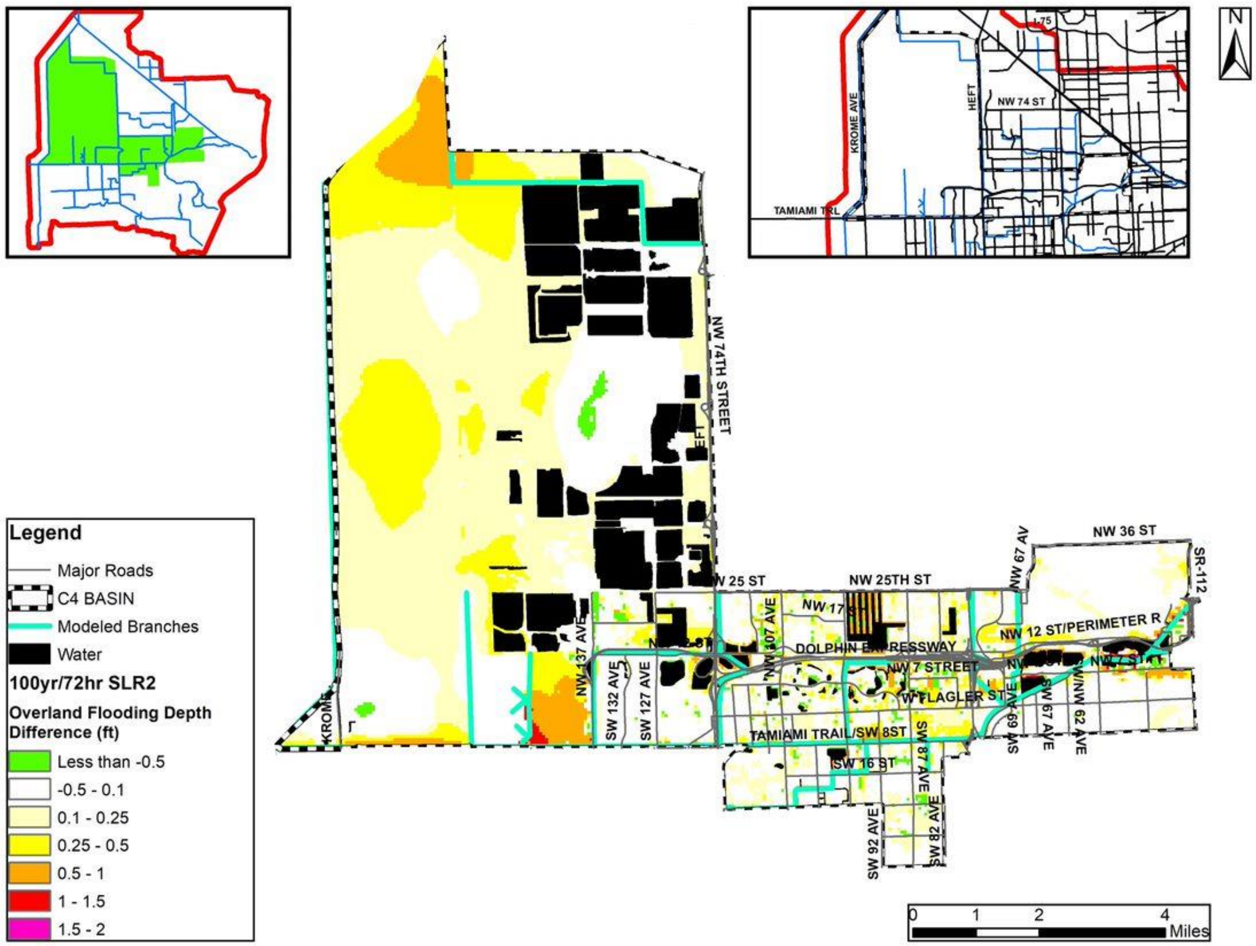


Figure C 3-44. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C4 Watershed

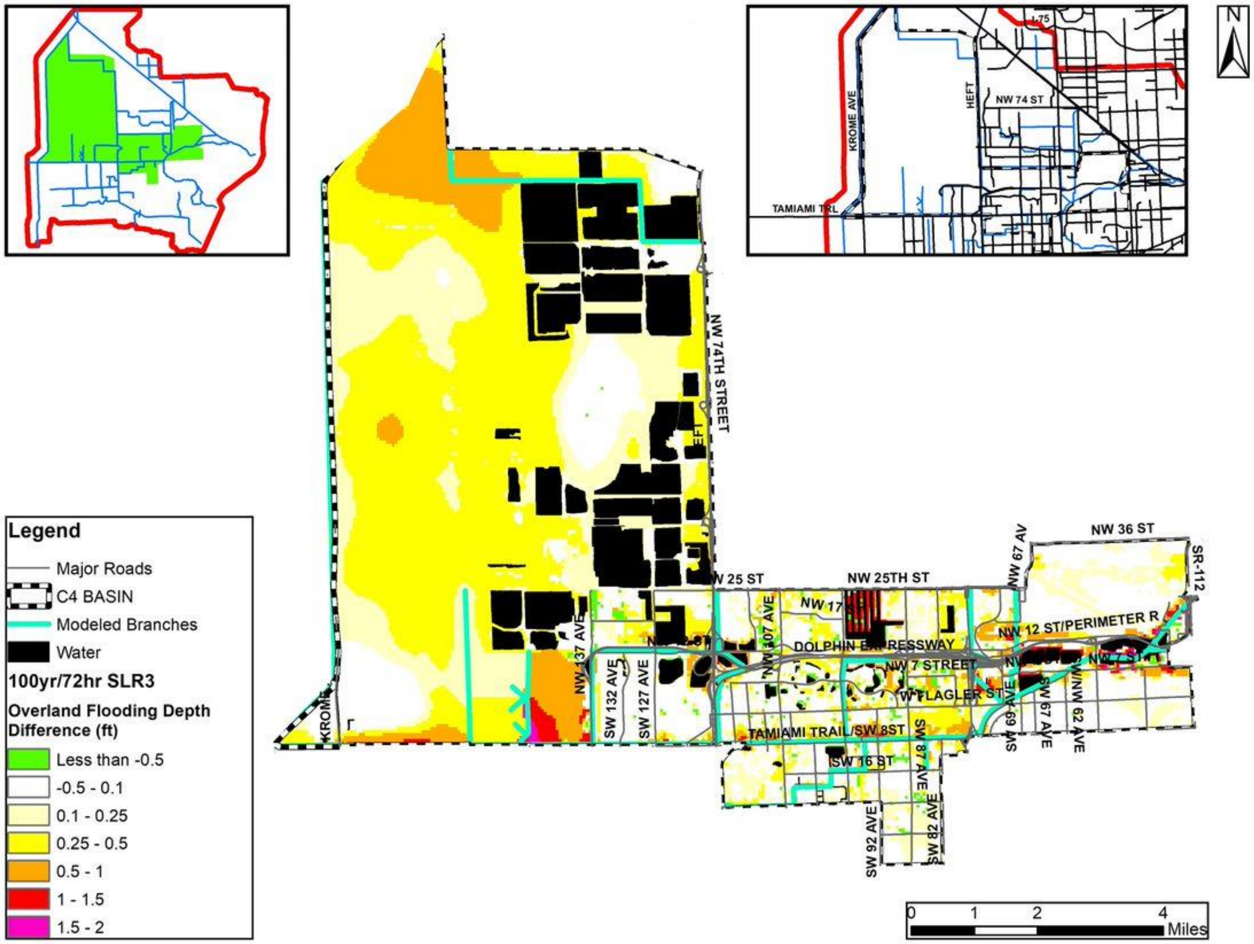


Figure C 3-45. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C4 Watershed

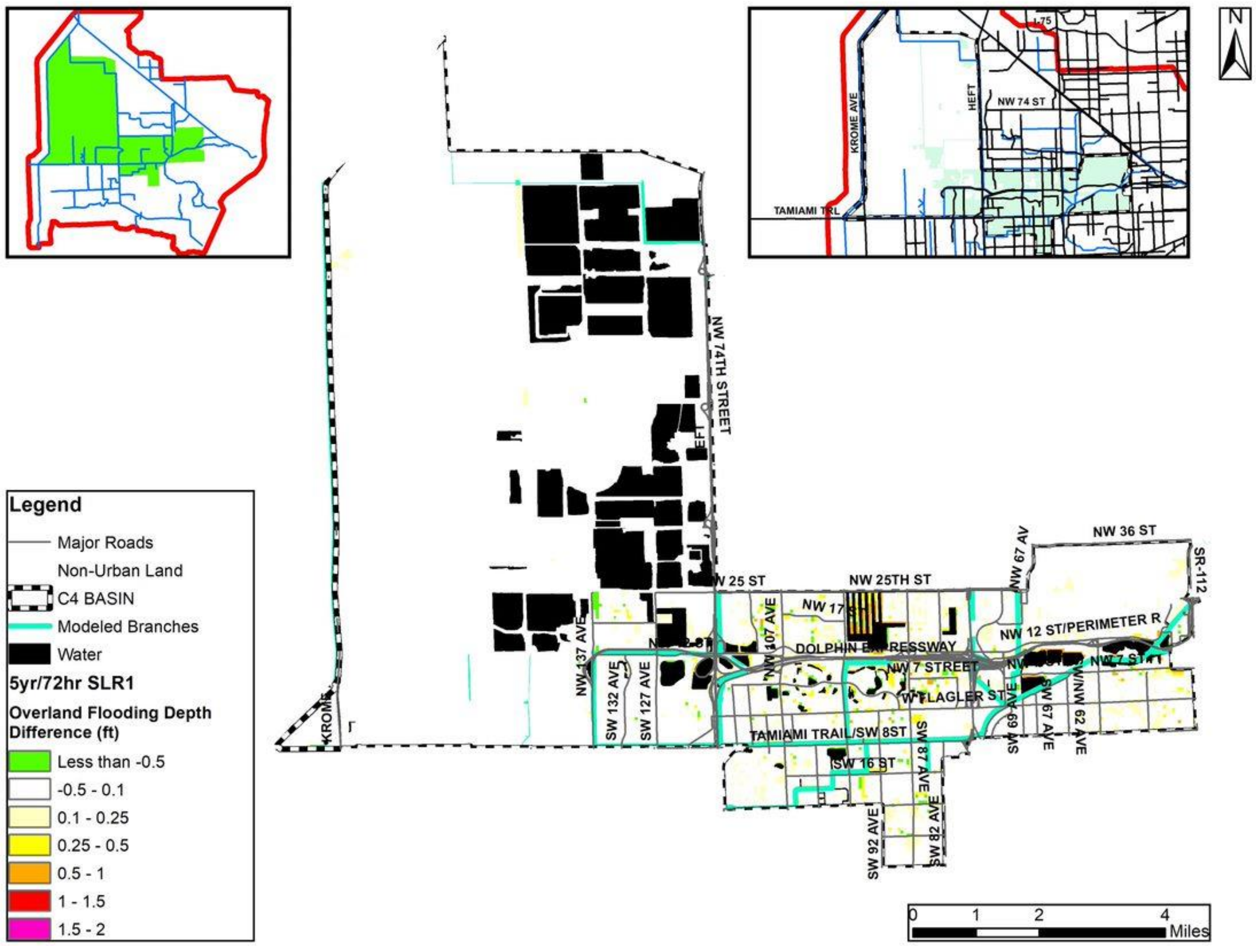


Figure C 3-46. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C4 Watershed

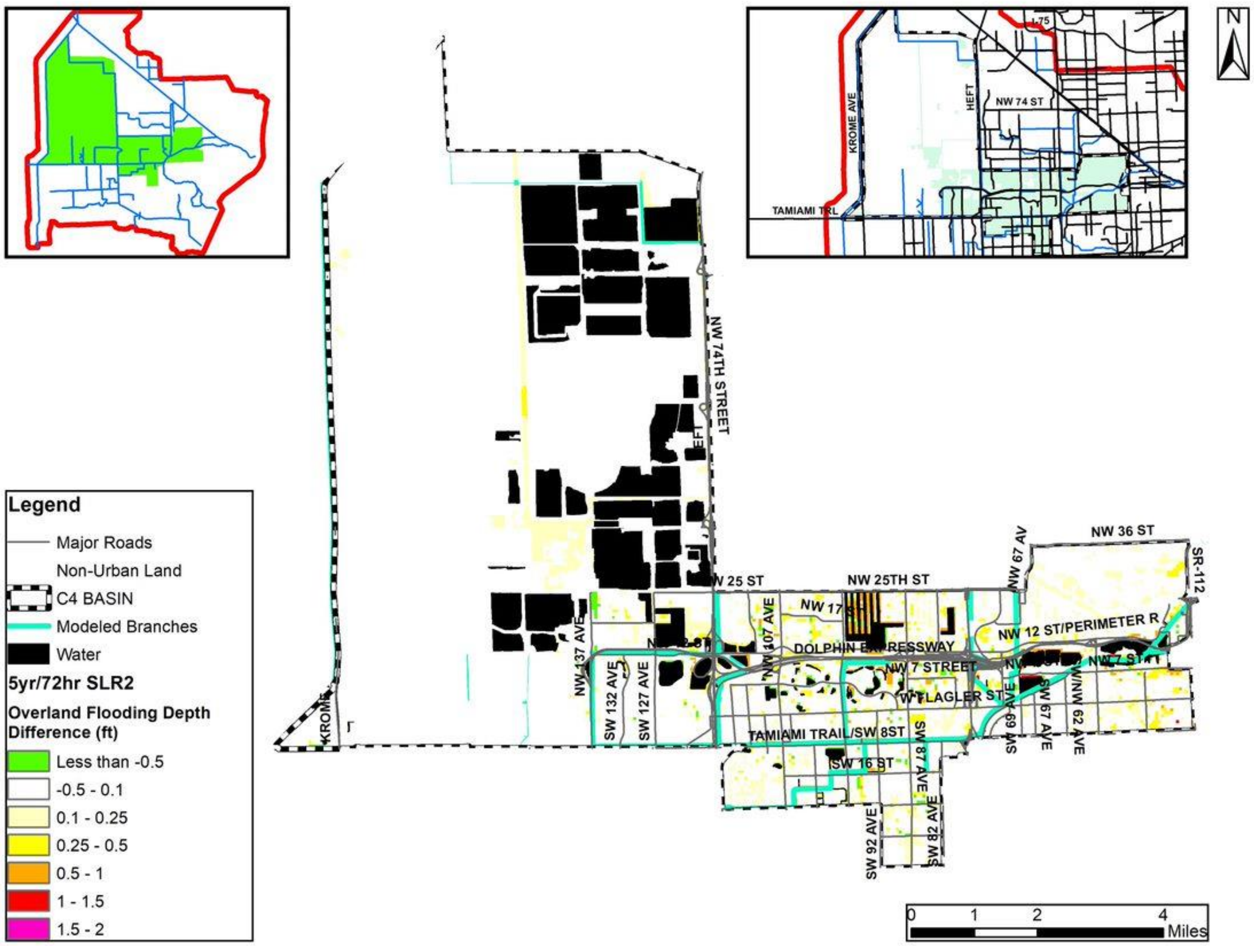


Figure C 3-47. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C4 Watershed

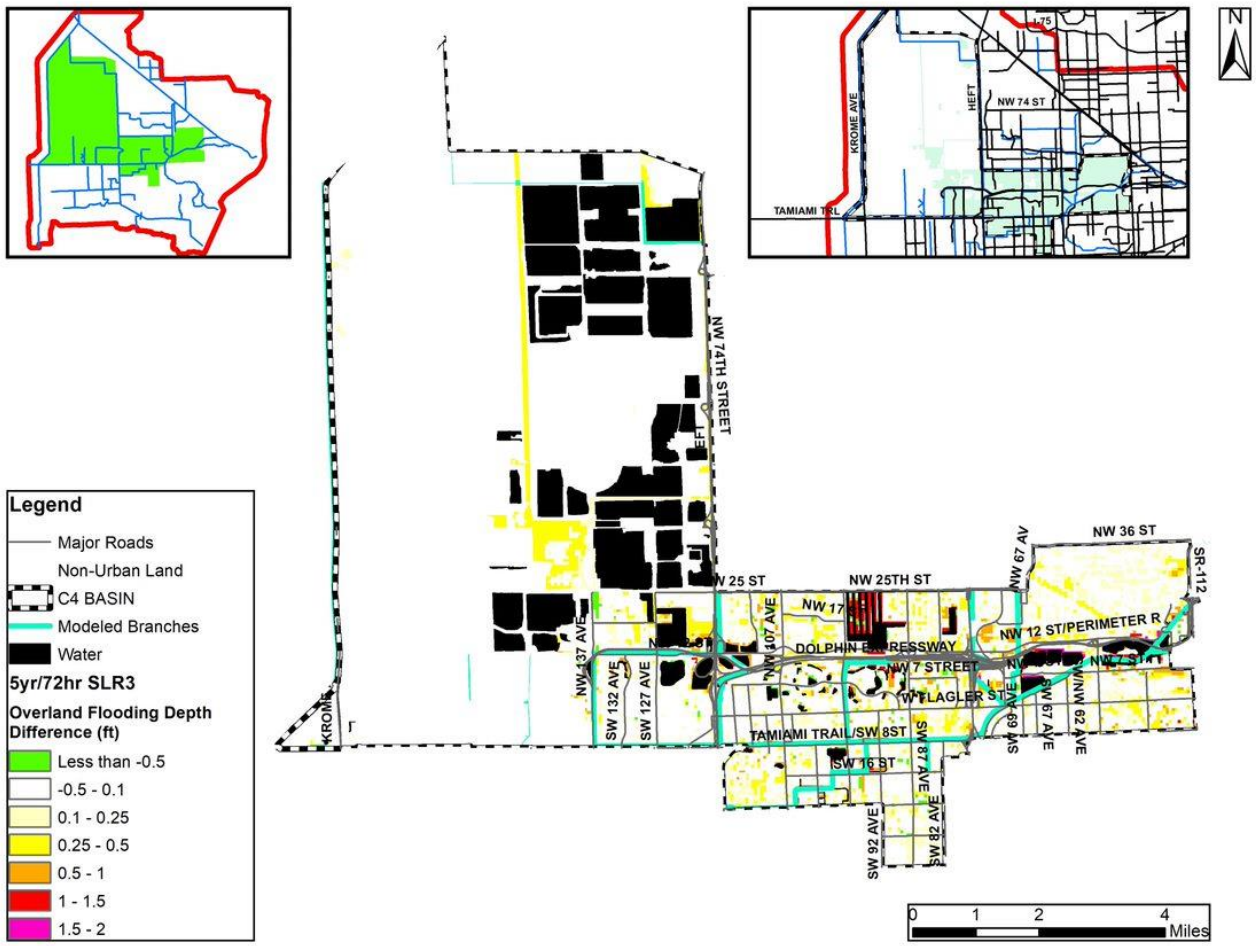


Figure C 3-48. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C4 Watershed

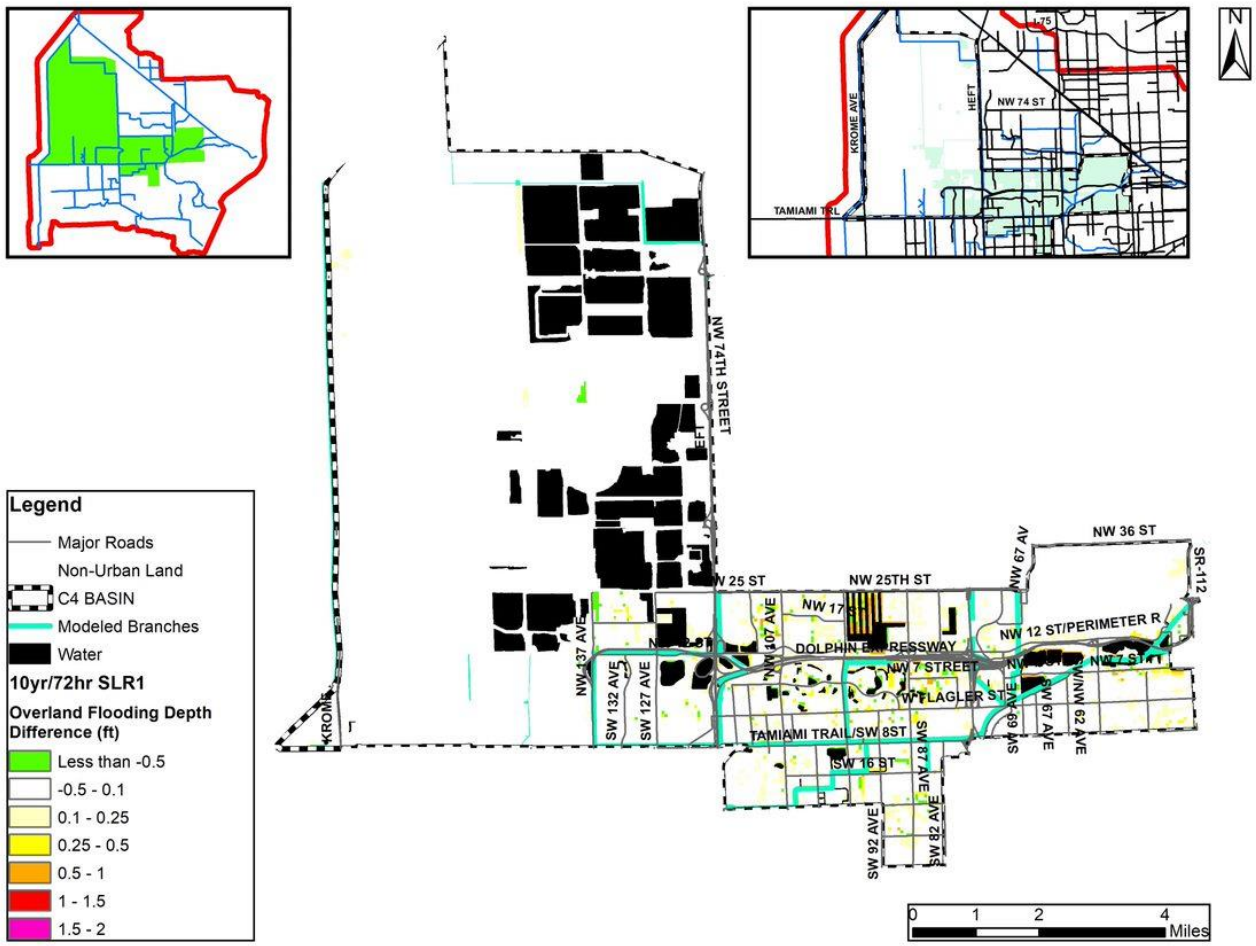


Figure C 3-49. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C4 Watershed

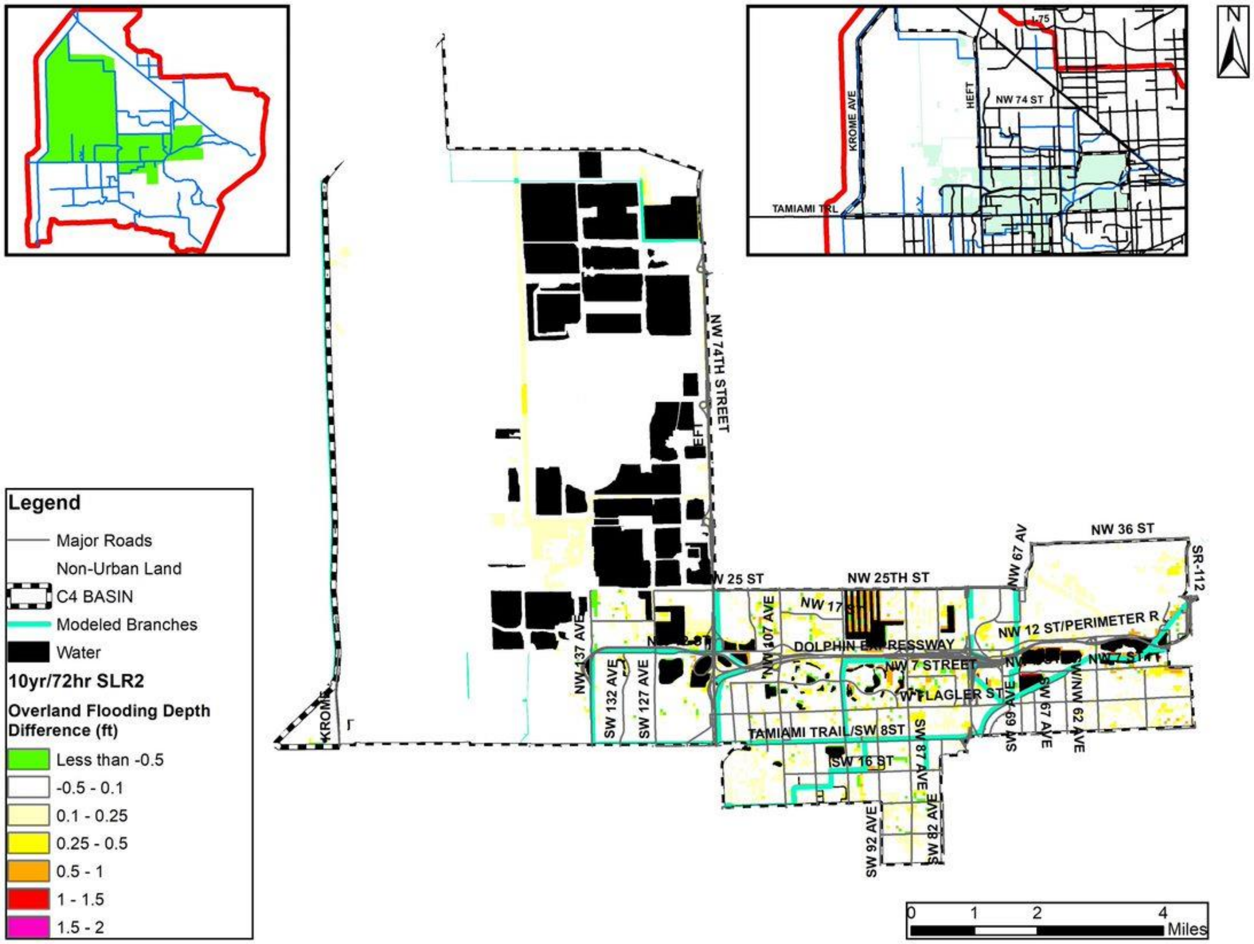


Figure C 3-50. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C4 Watershed

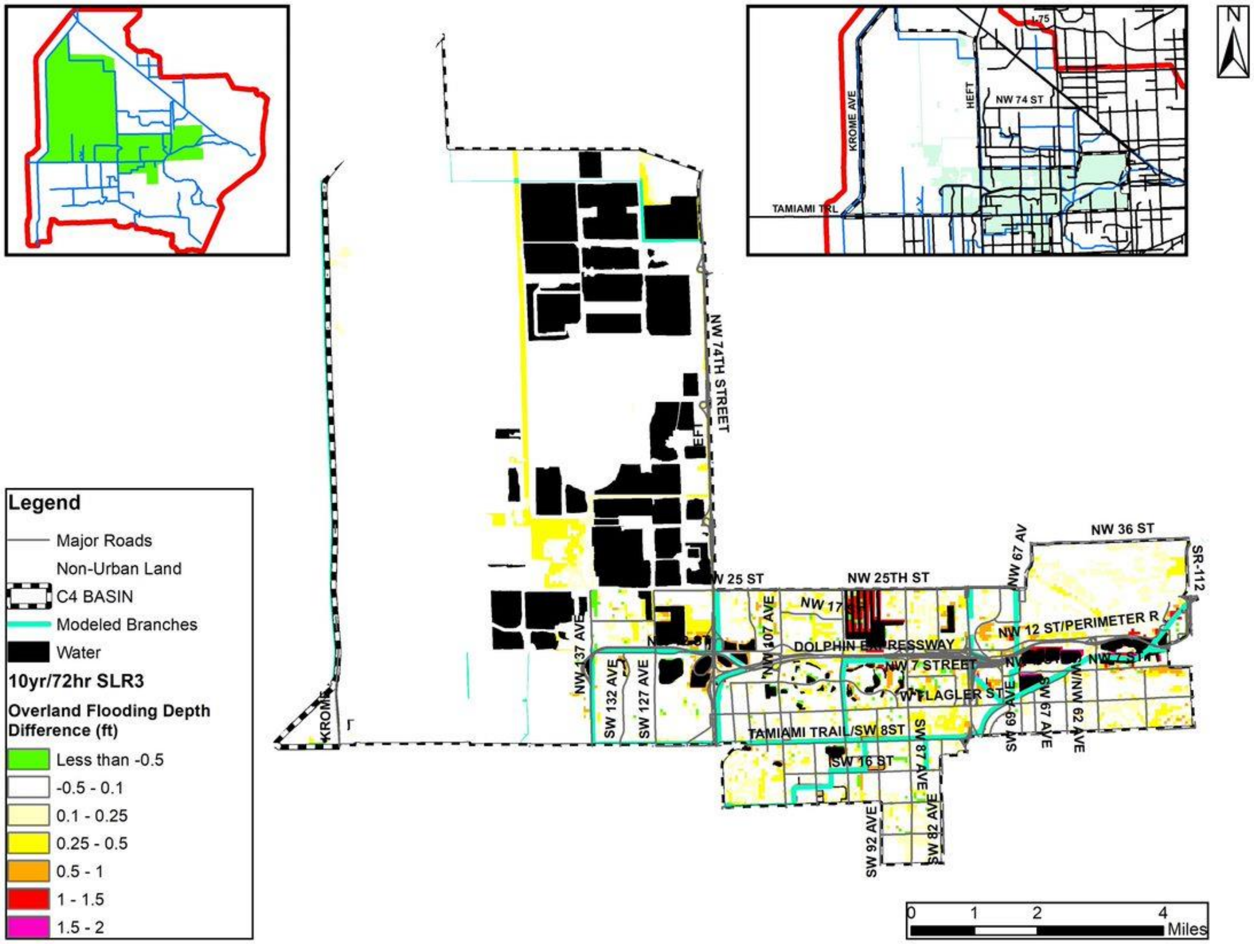


Figure C 3-51. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C4 Watershed

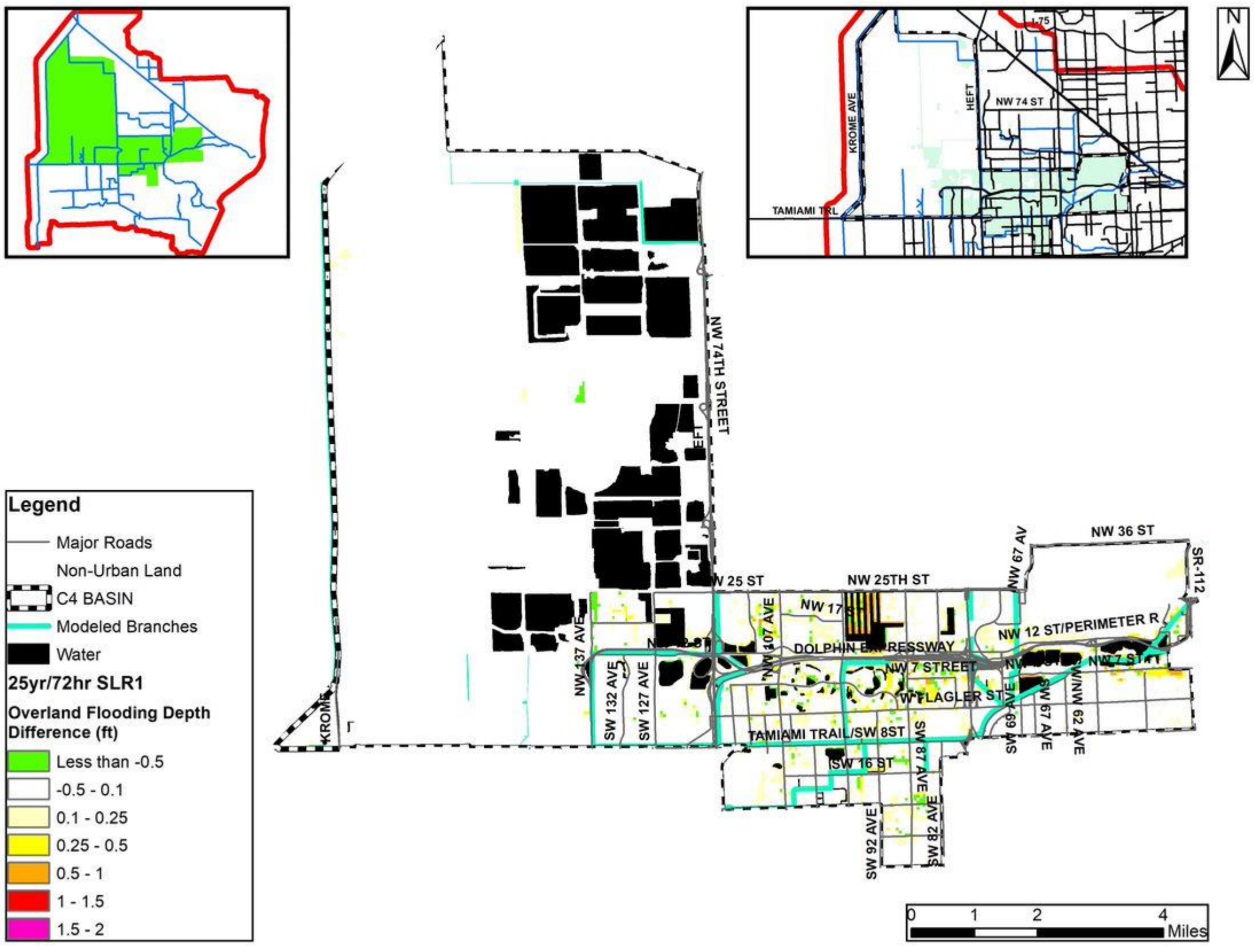


Figure C 3-52. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C4 Watershed

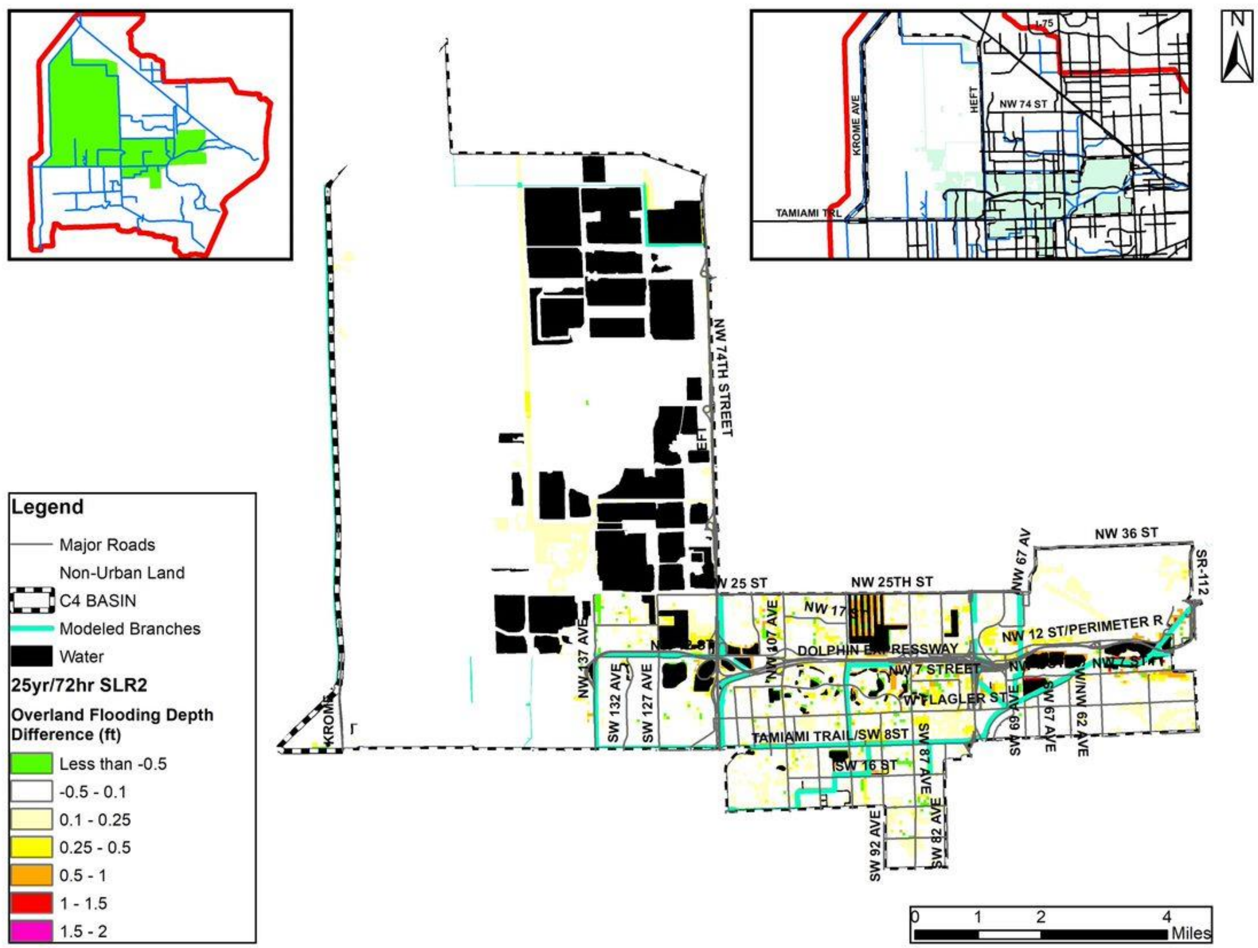


Figure C 3-53. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C4 Watershed

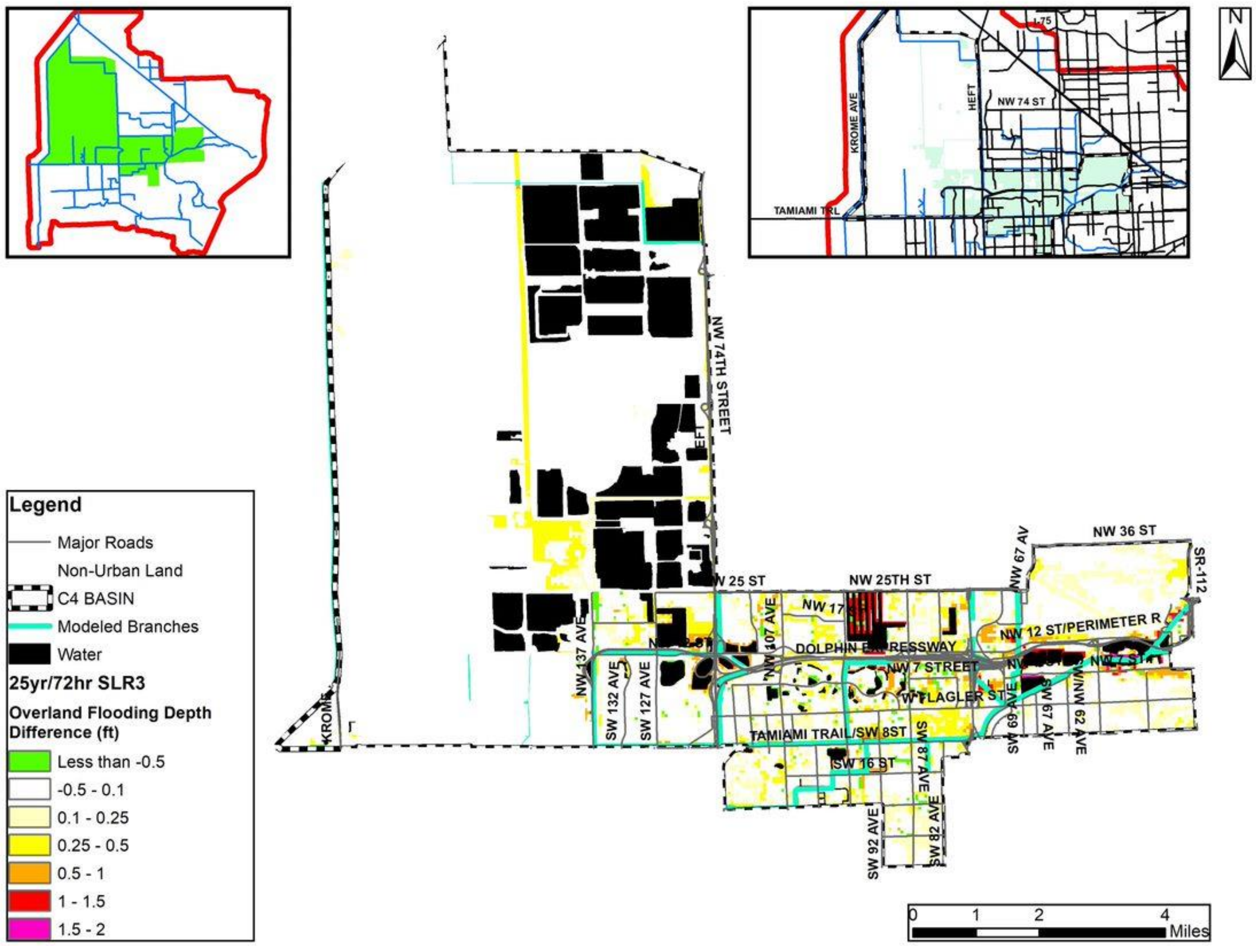


Figure C 3-54. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C4 Watershed

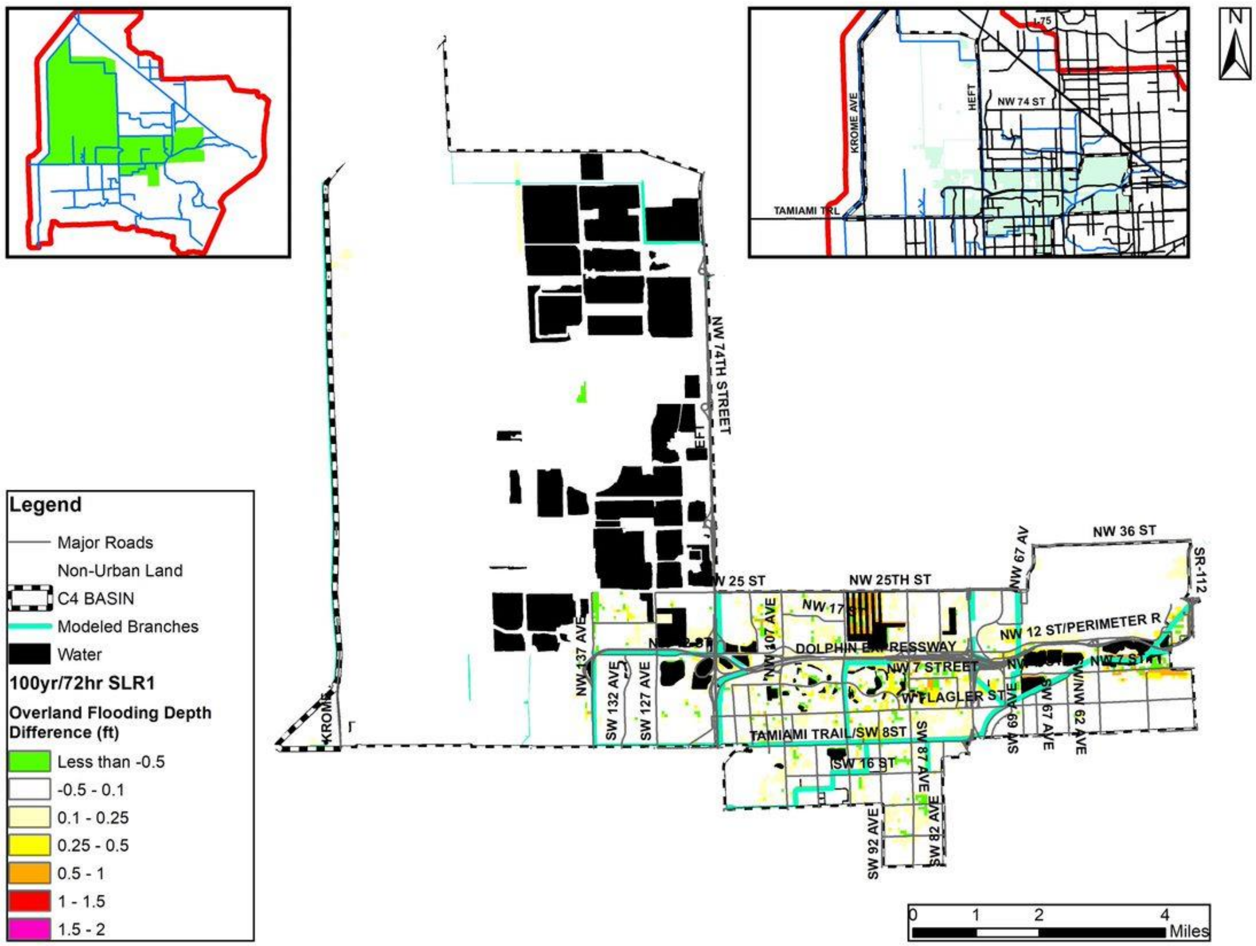


Figure C 3-55. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C4 Watershed

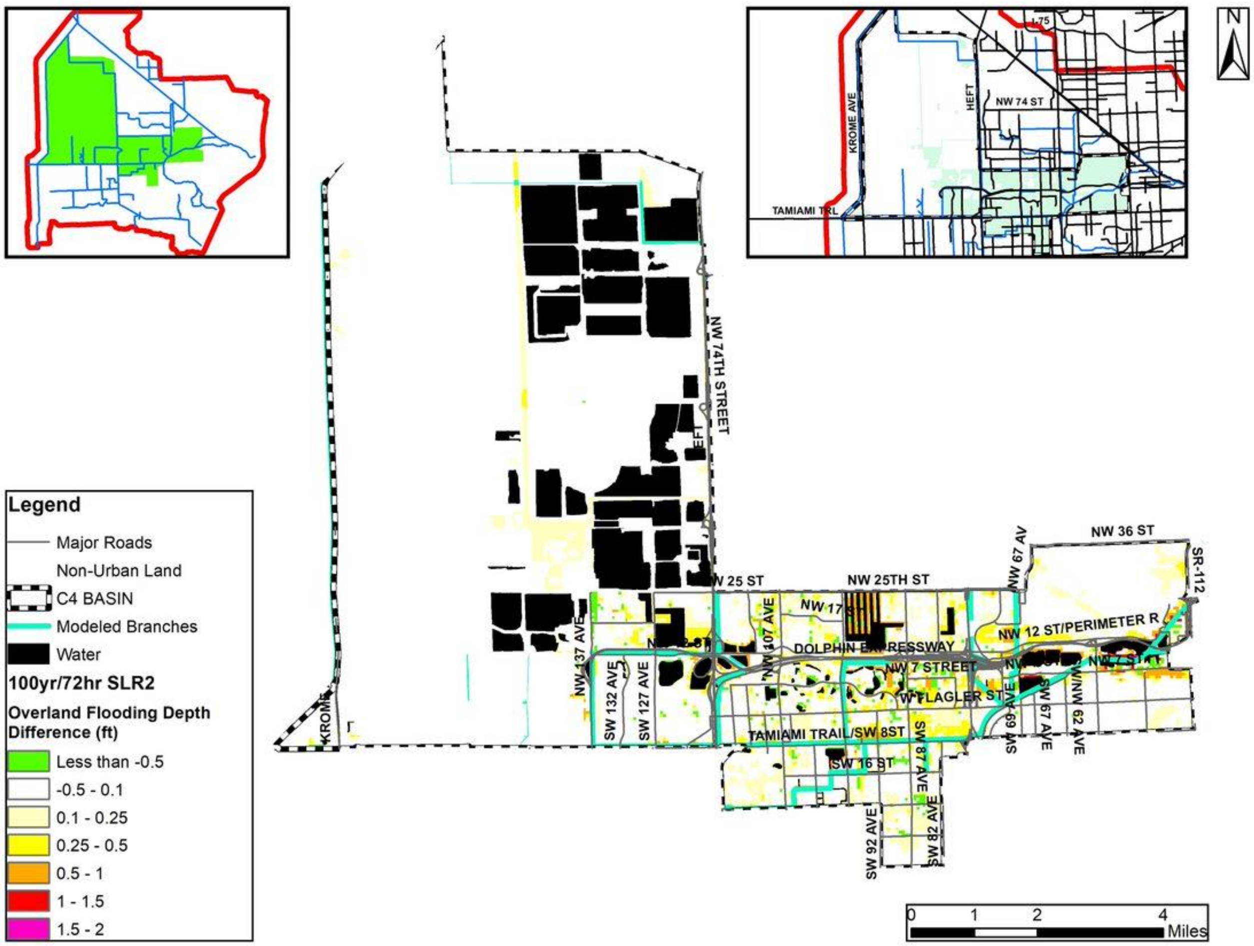
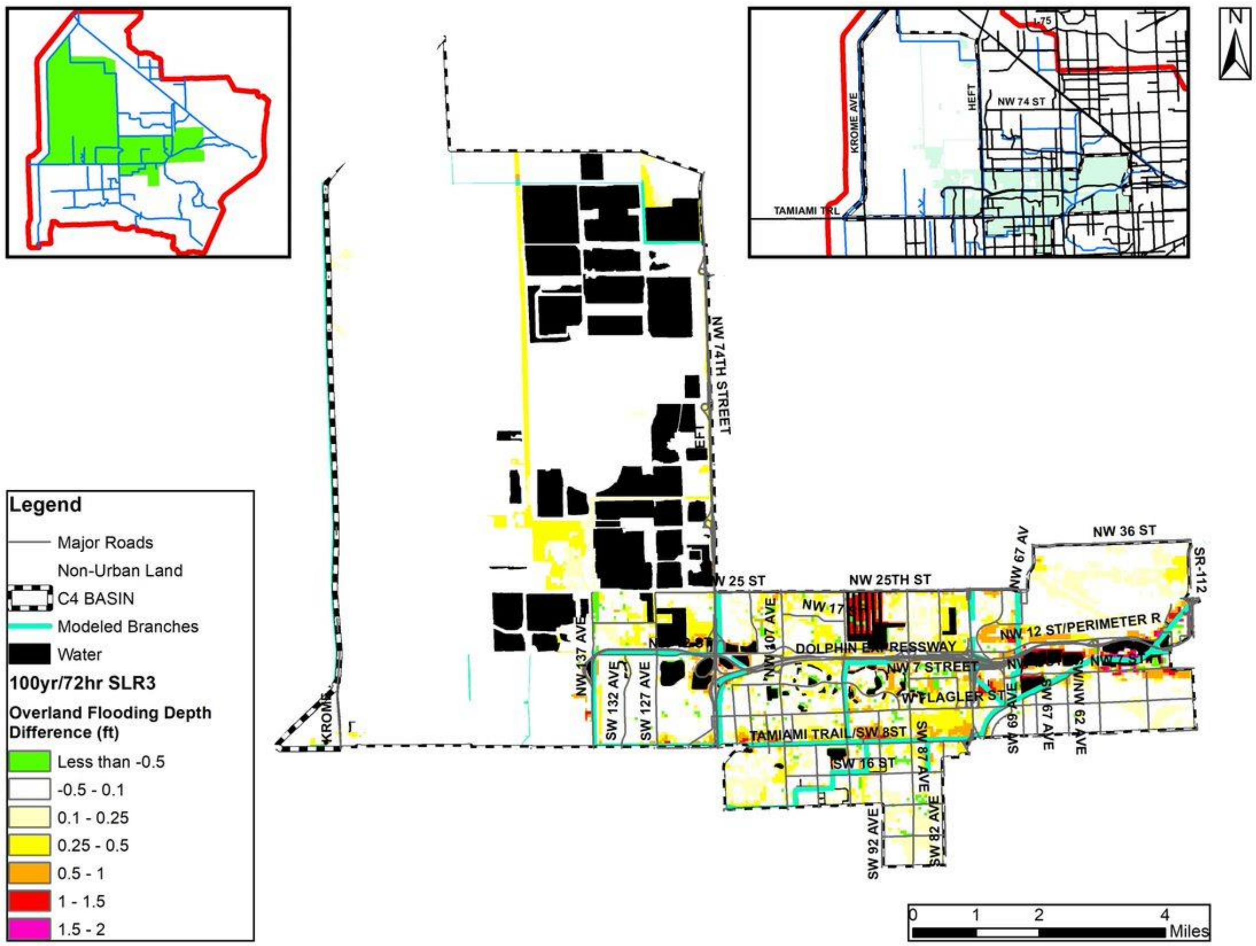


Figure C 3-56. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C4 Watershed



4 C5 Watershed

Figure C 4-1 through **Figure C 4-16** are maps of the maximum overland depth over the entire C5 Watershed for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for each SLR condition. Water areas are masked in black.

Figure C 4-17 through **Figure C 4-28** show the difference in overland flooding for the C5 Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively.

Figure C 4-1. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for Current Conditions in the C5 Watershed

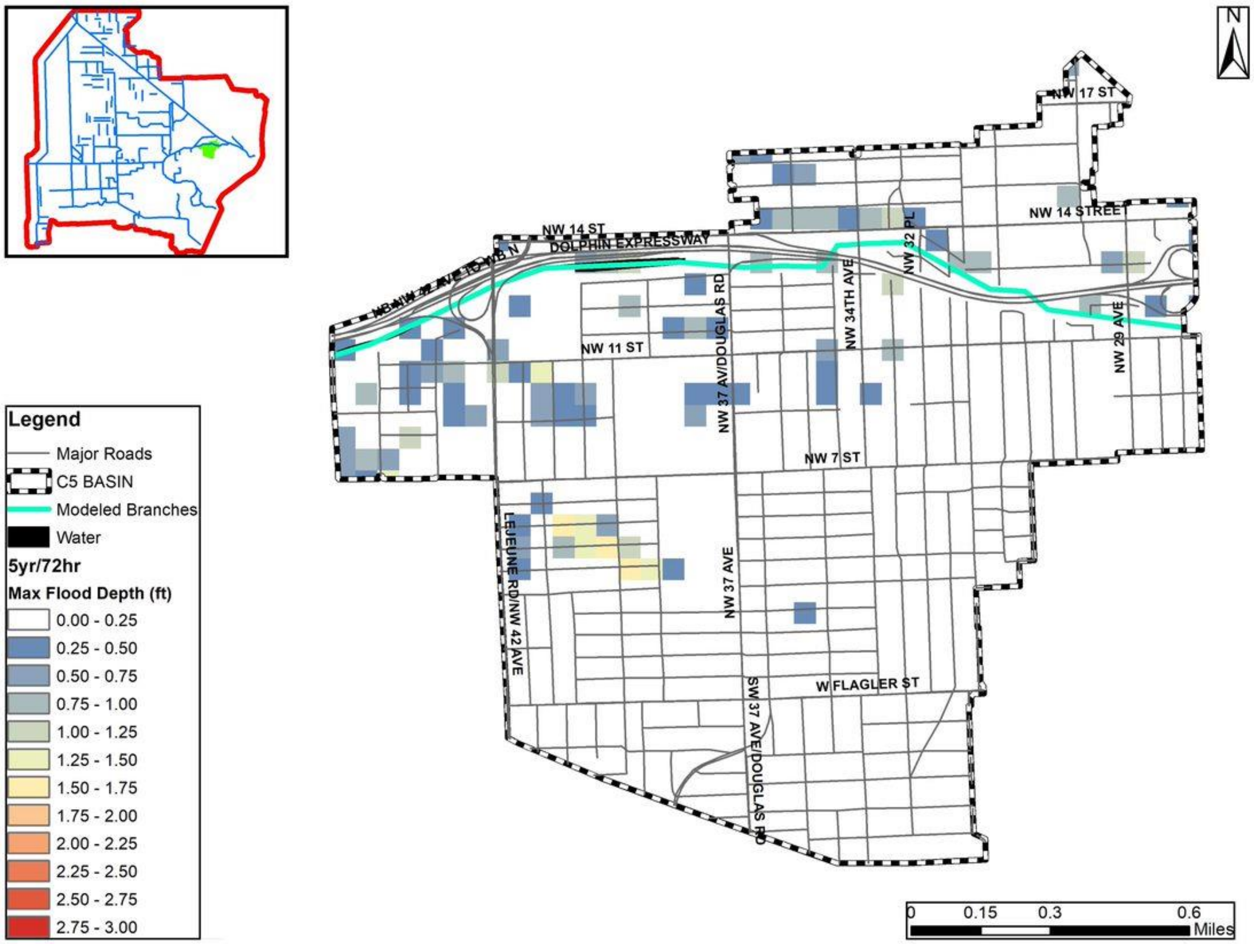


Figure C 4-2. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR1 in the C5 Watershed

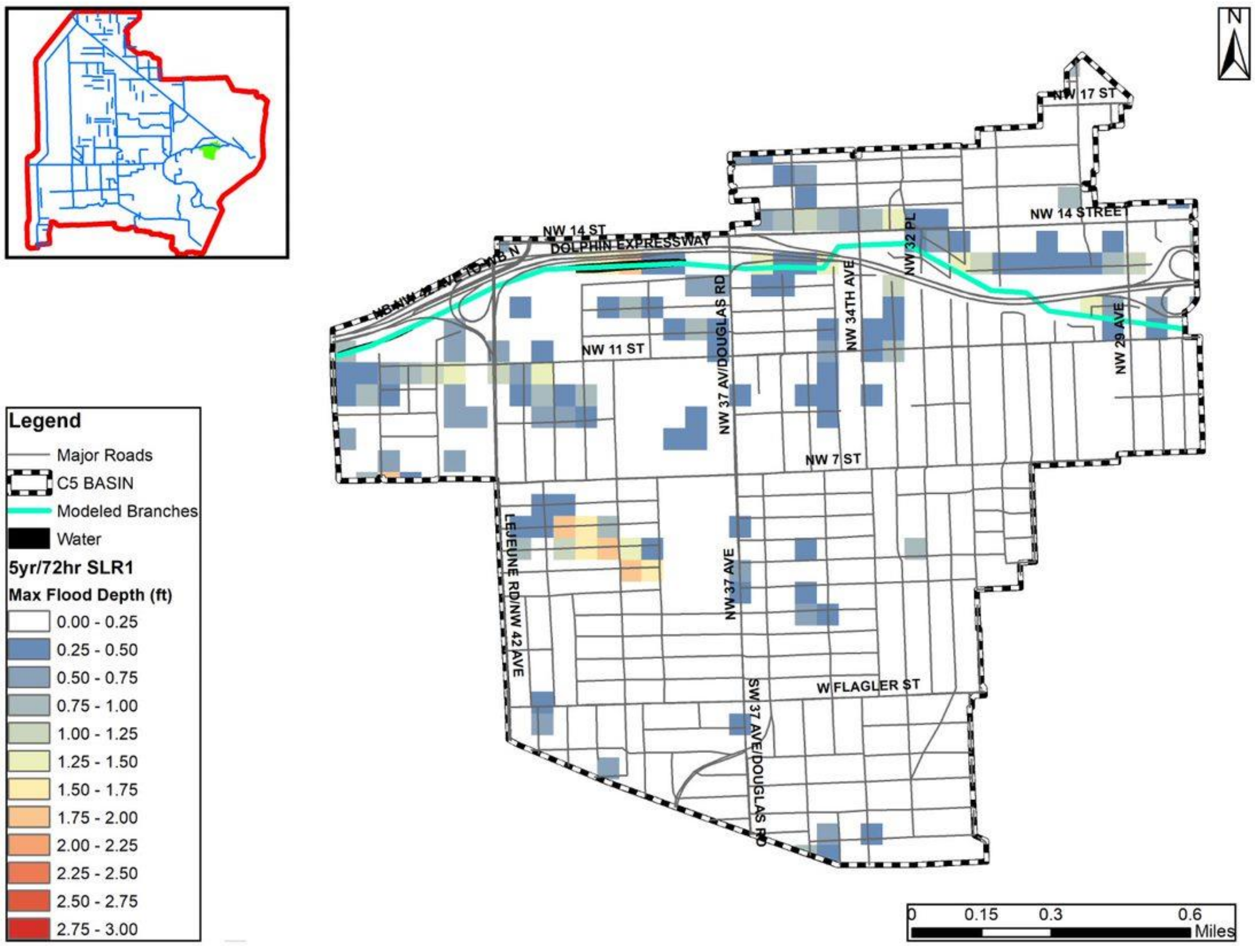


Figure C 4-3. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR2 in the C5 Watershed

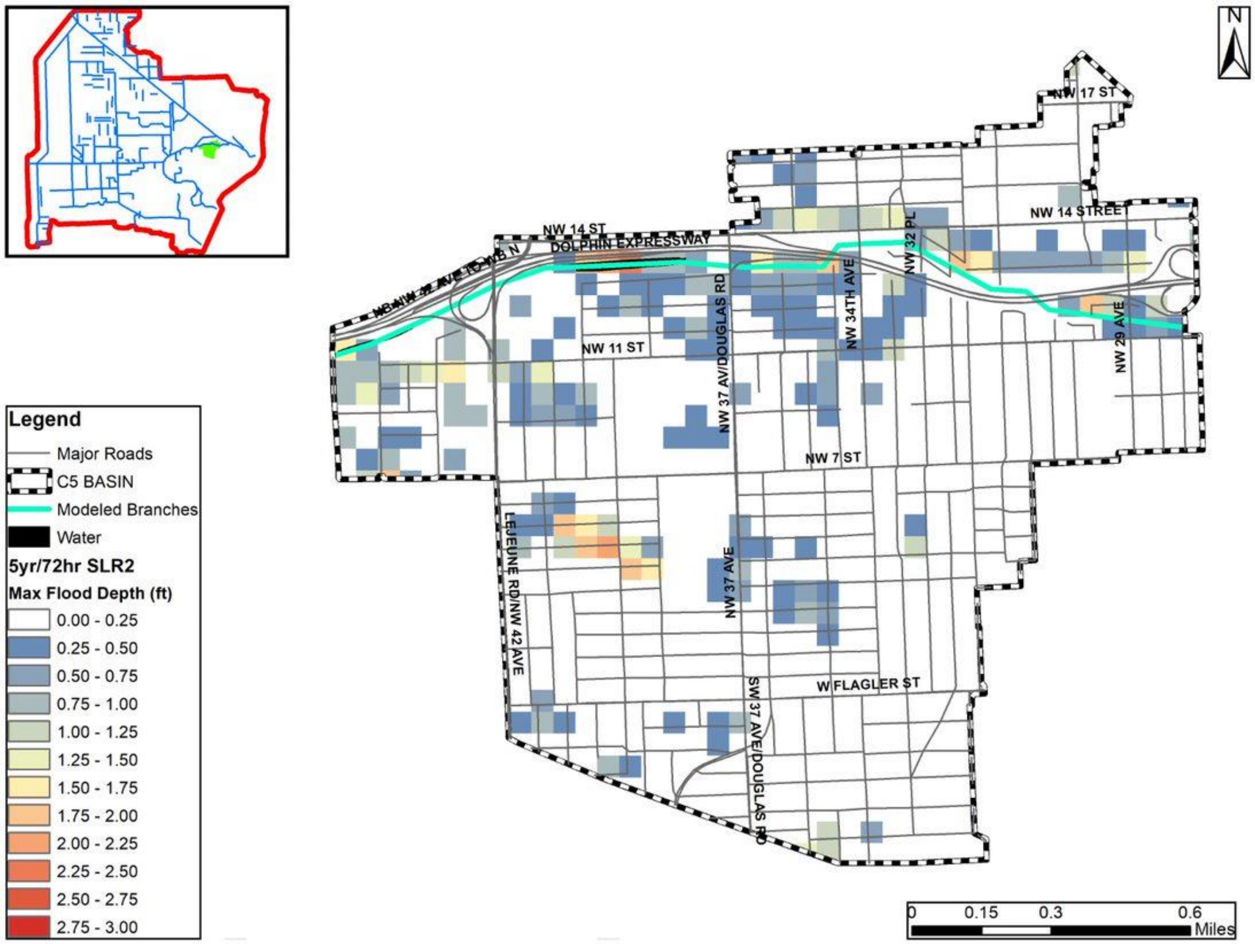


Figure C 4-4. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR3 in the C5 Watershed

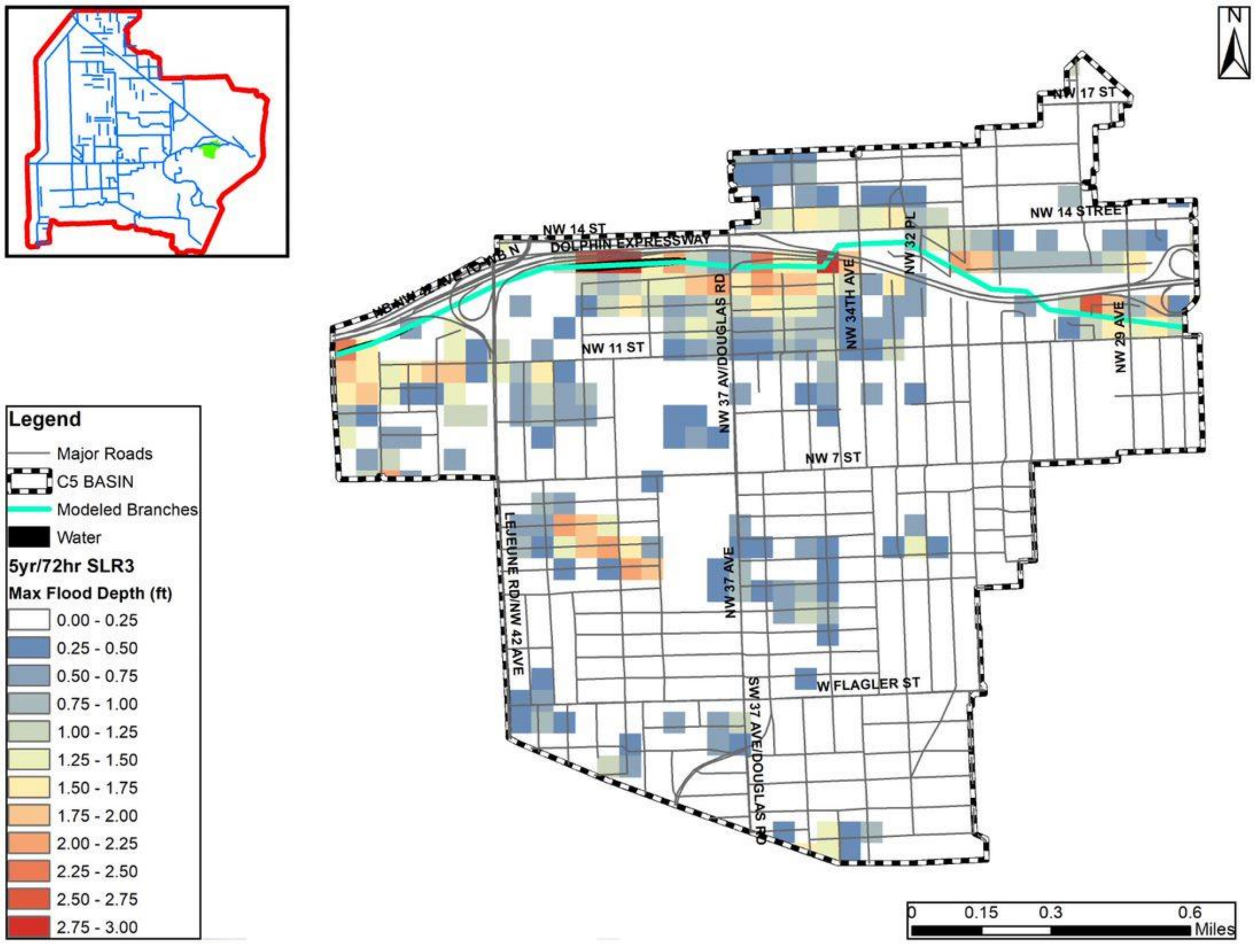


Figure C 4-5. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for Current Conditions in the C5 Watershed

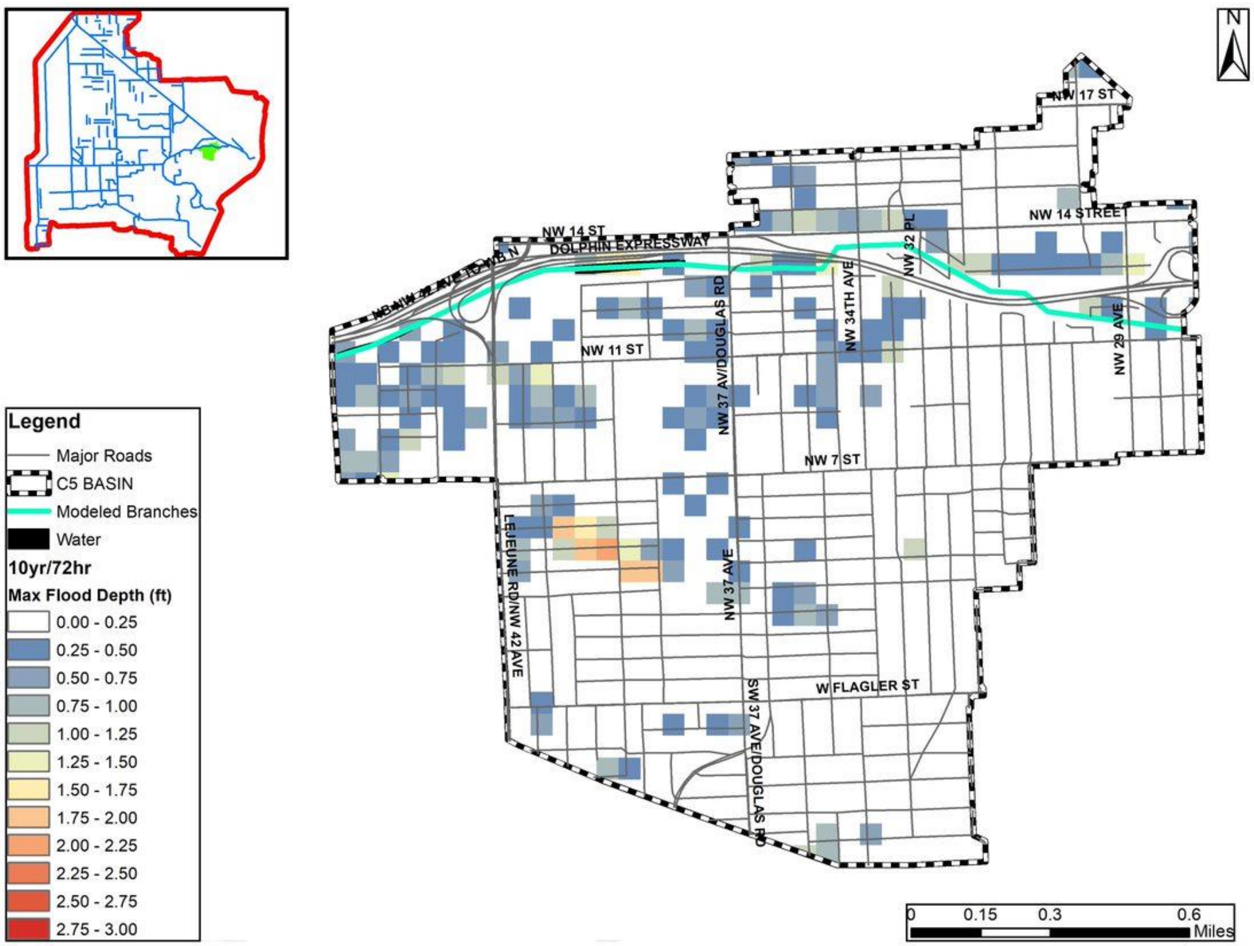


Figure C 4-6. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR1 in the C5 Watershed

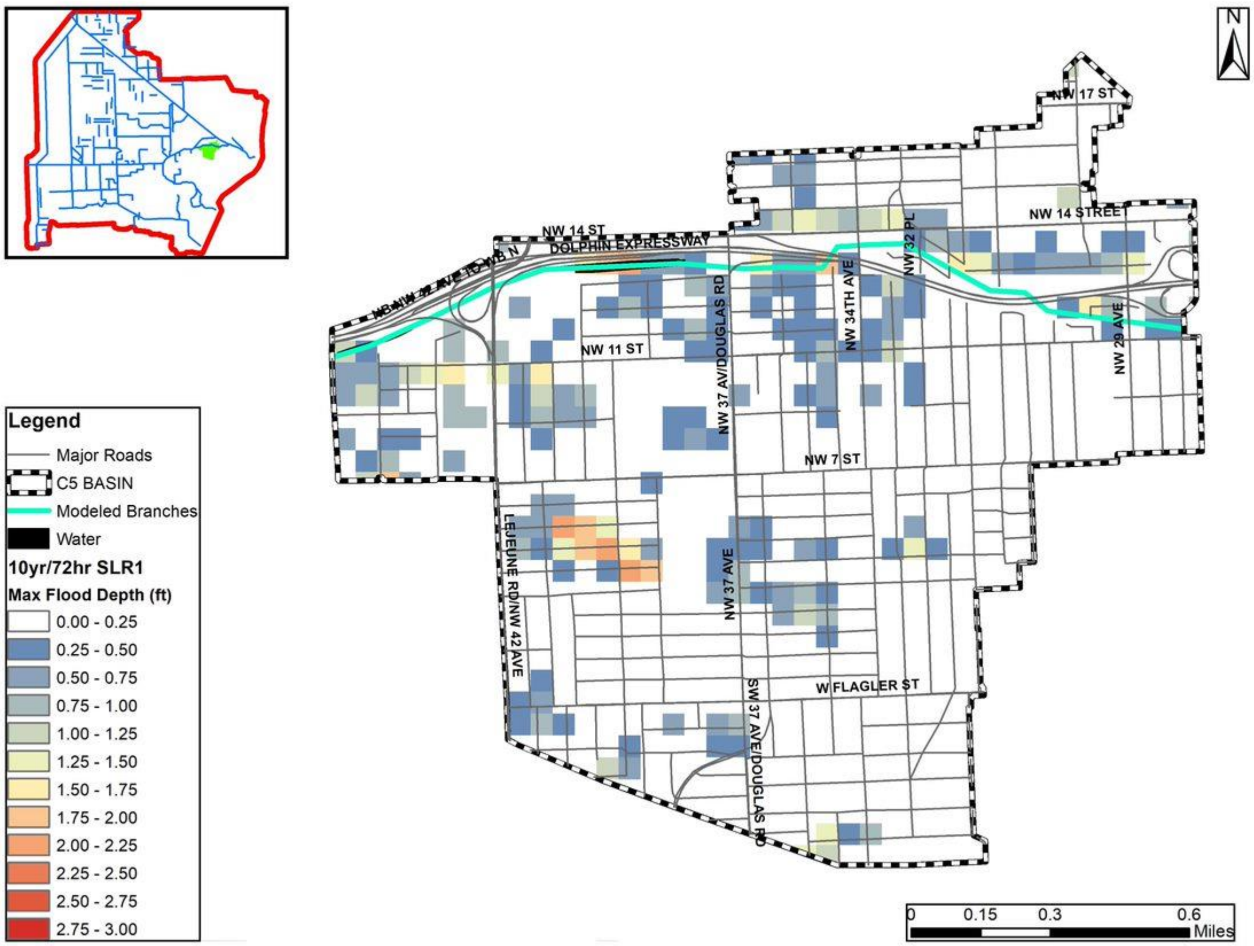


Figure C 4-7. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR2 in the C5 Watershed

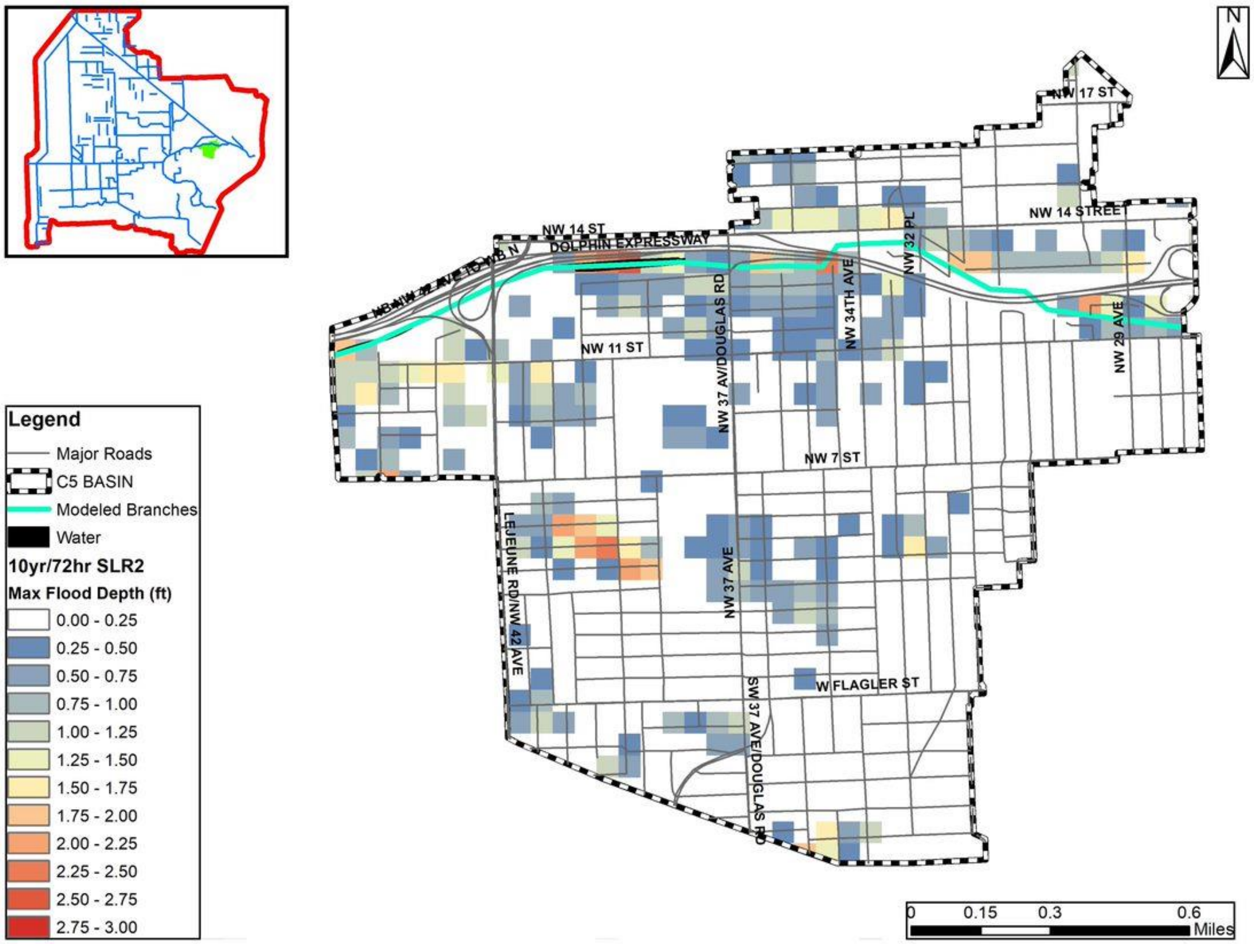


Figure C 4-8. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR3 in the C5 Watershed

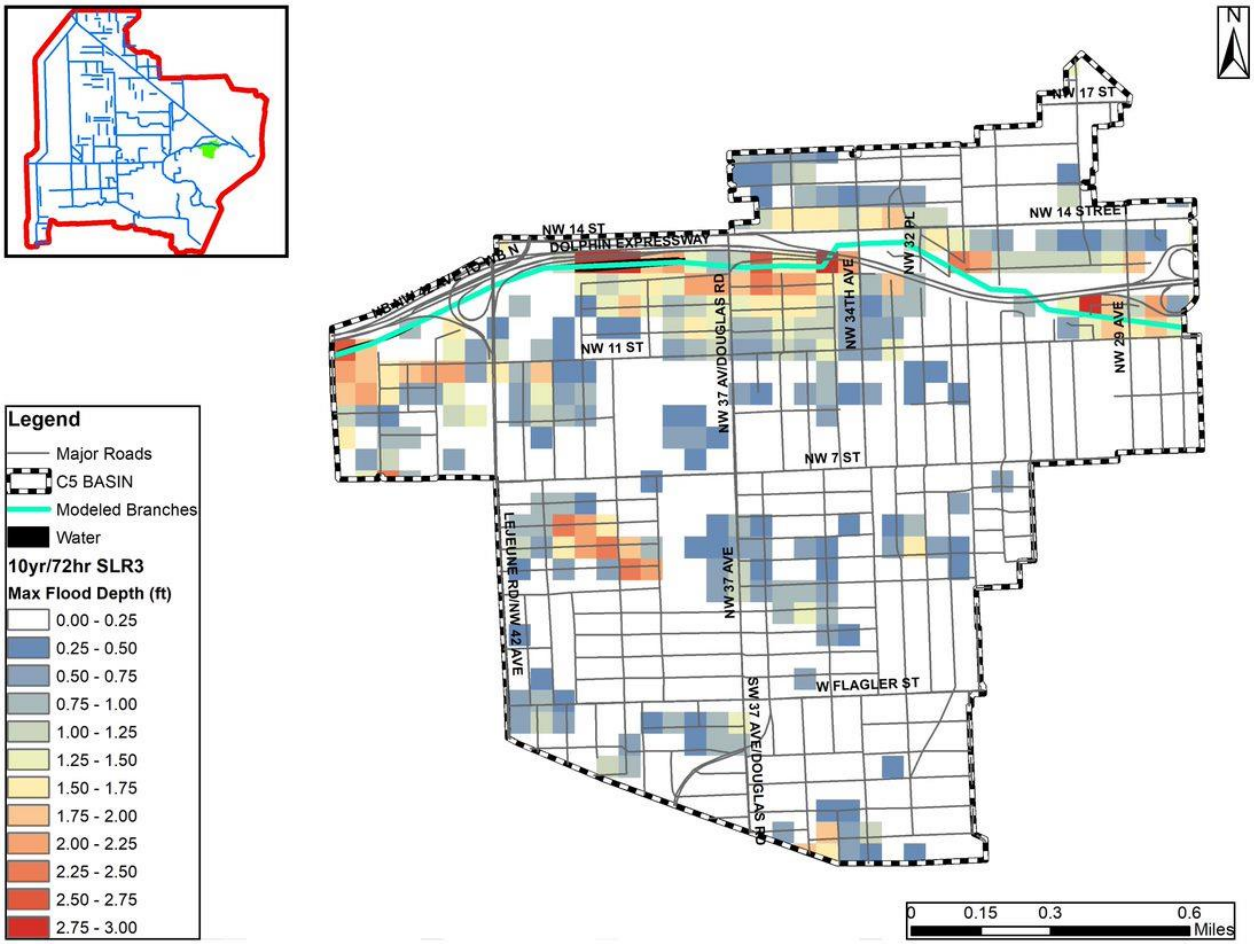


Figure C 4-9. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for Current Conditions in the C5 Watershed

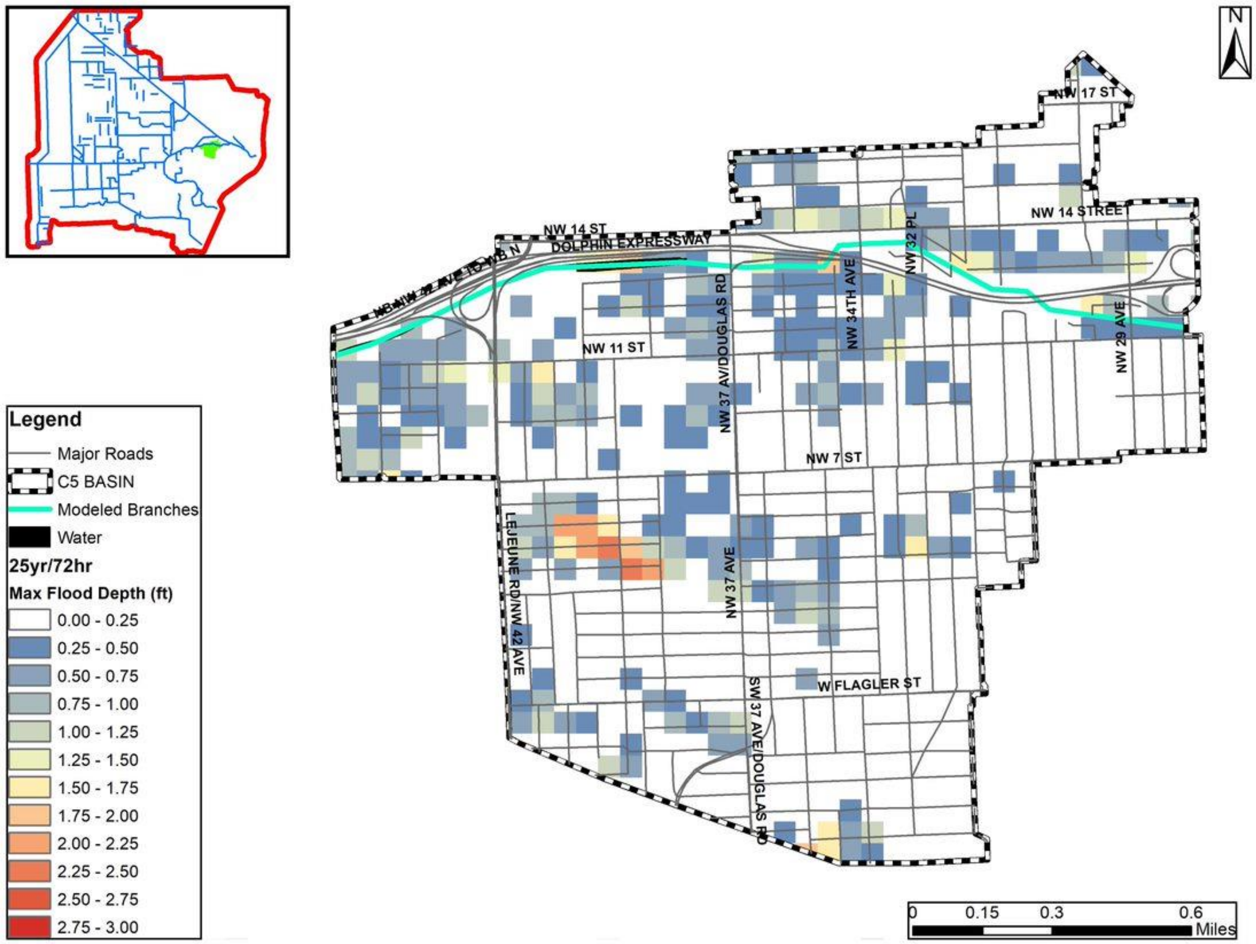


Figure C 4-10. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR1 in the C5 Watershed

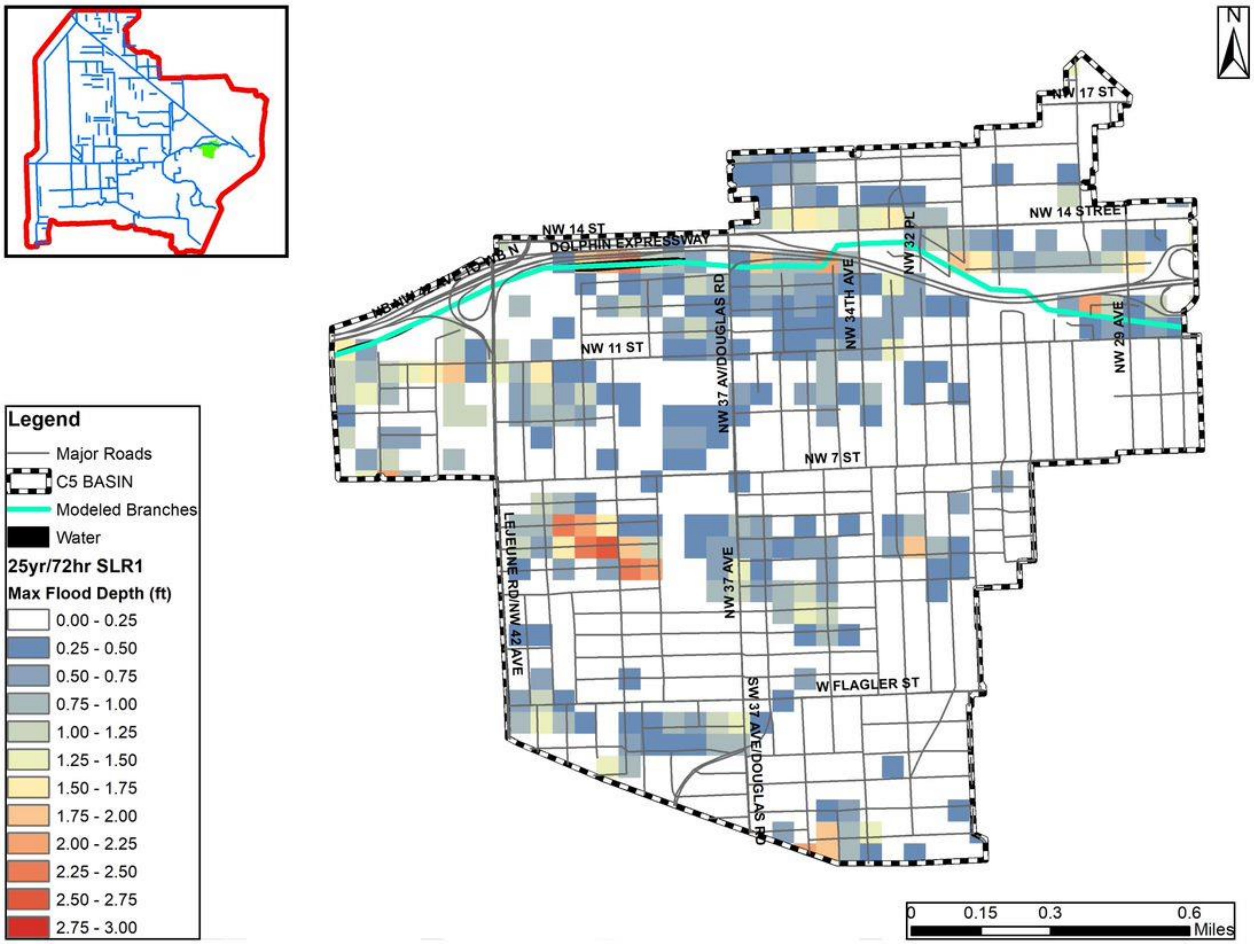


Figure C 4-11. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR2 in the C5 Watershed

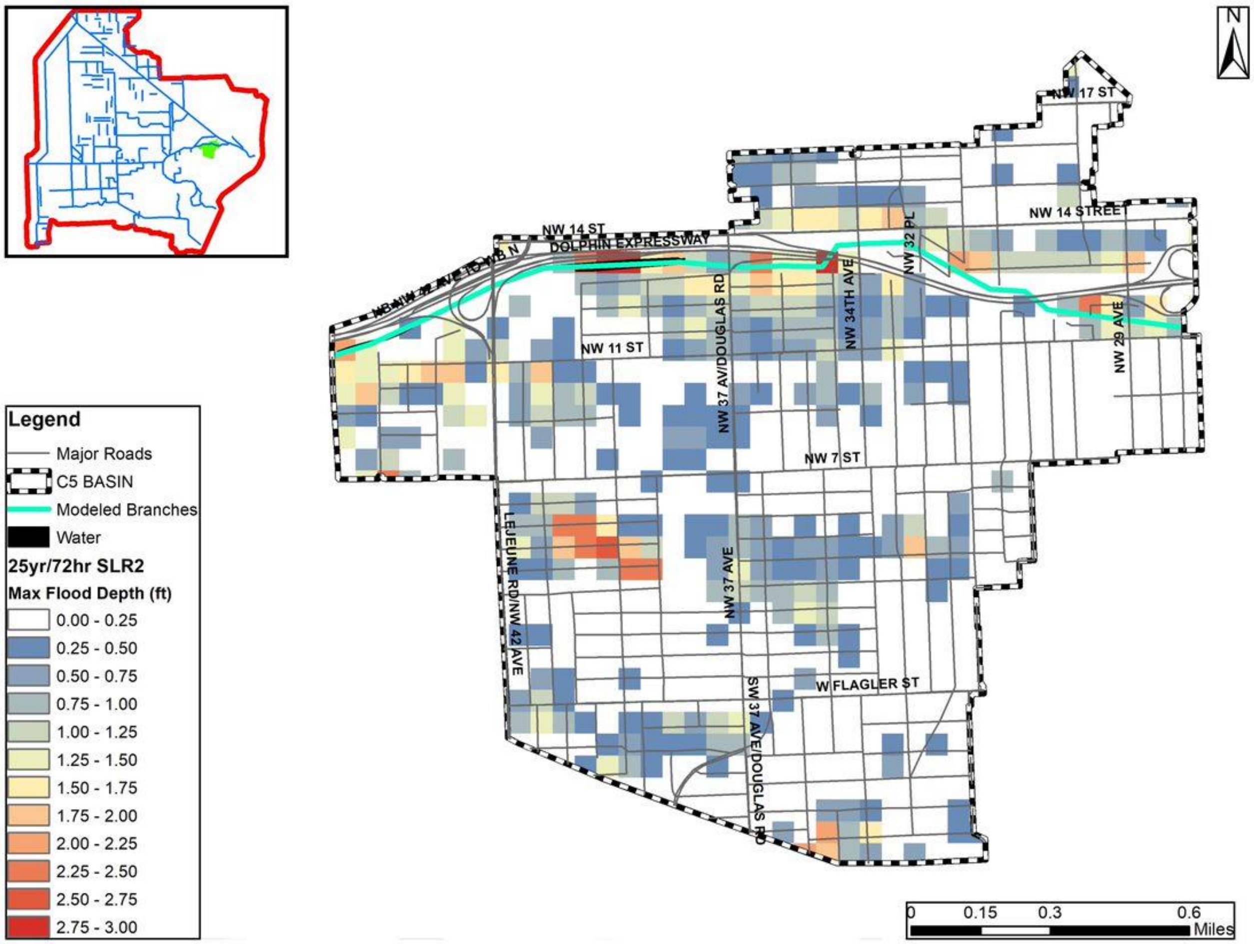


Figure C 4-12. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR3 in the C5 Watershed

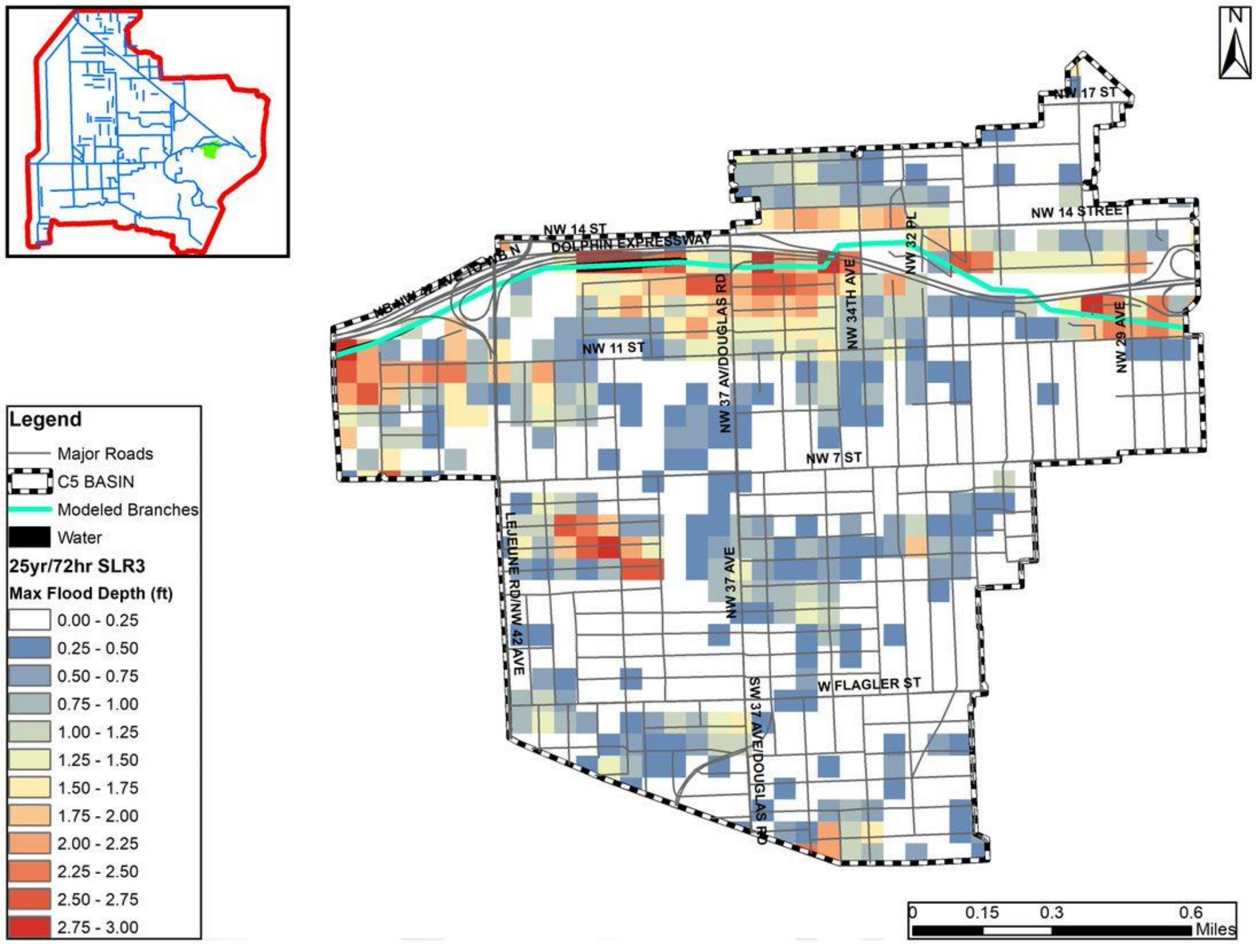


Figure C 4-13. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for Current Conditions in the C5 Watershed

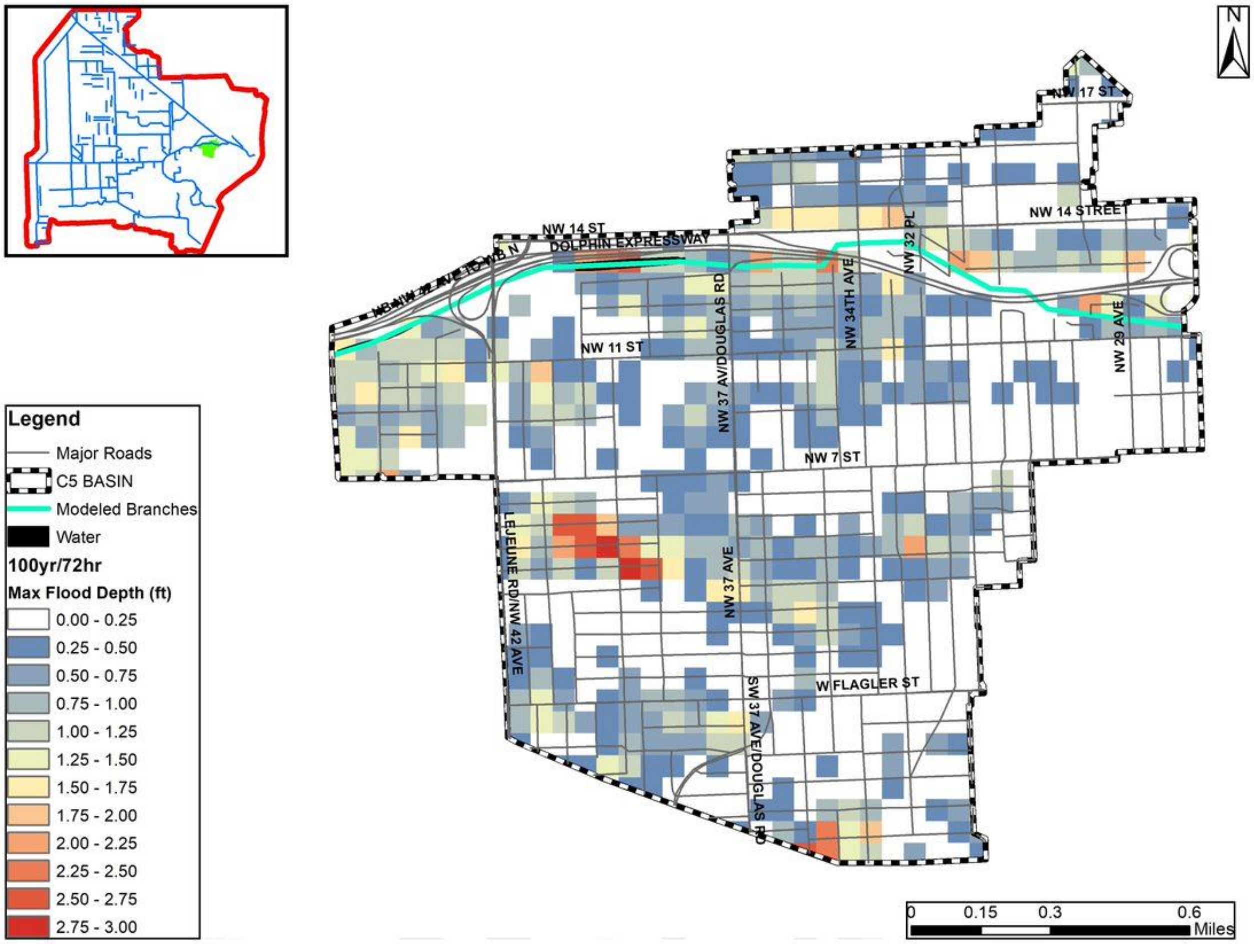


Figure C 4-14. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR1 in the C5 Watershed

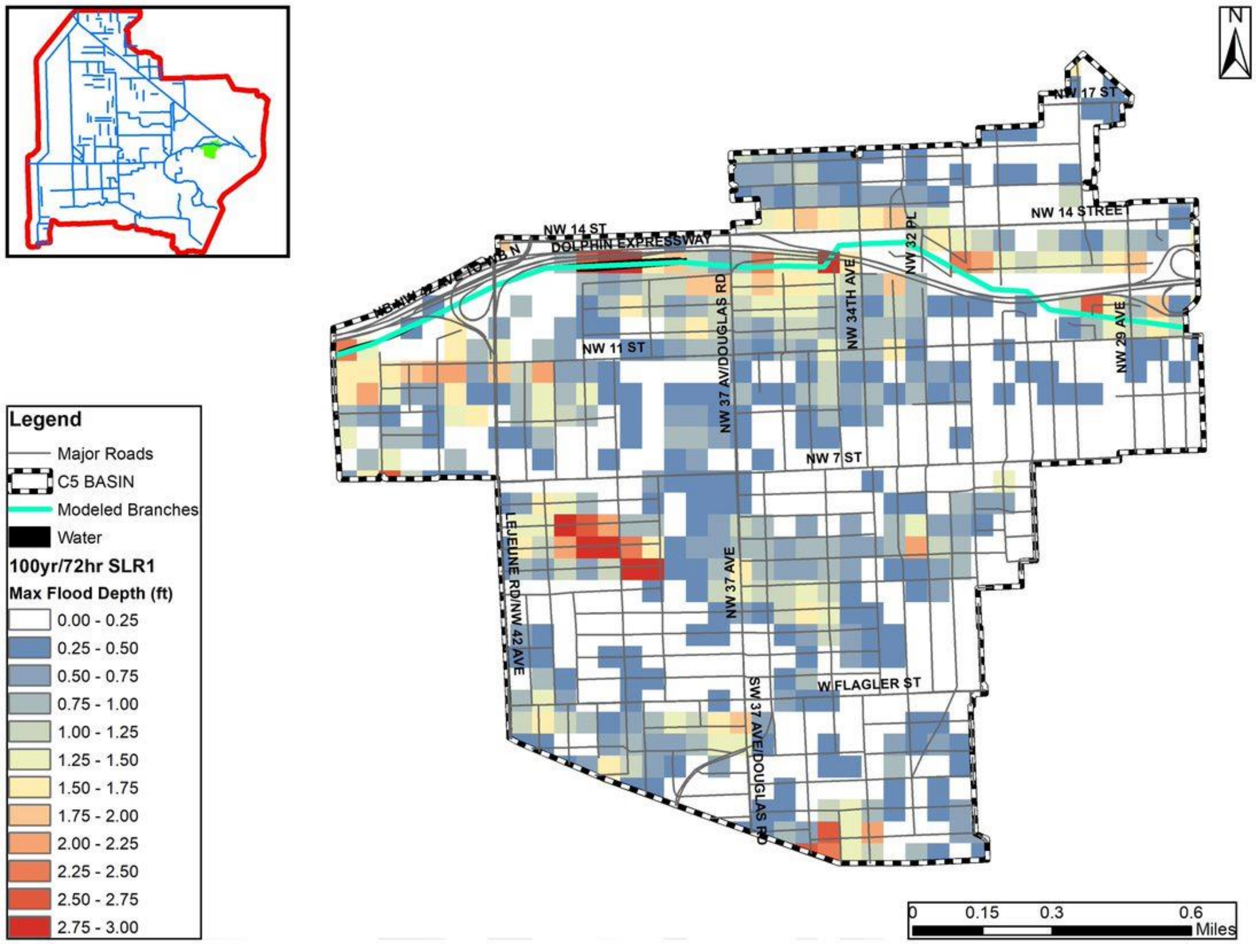


Figure C 4-15. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR2 in the C5 Watershed

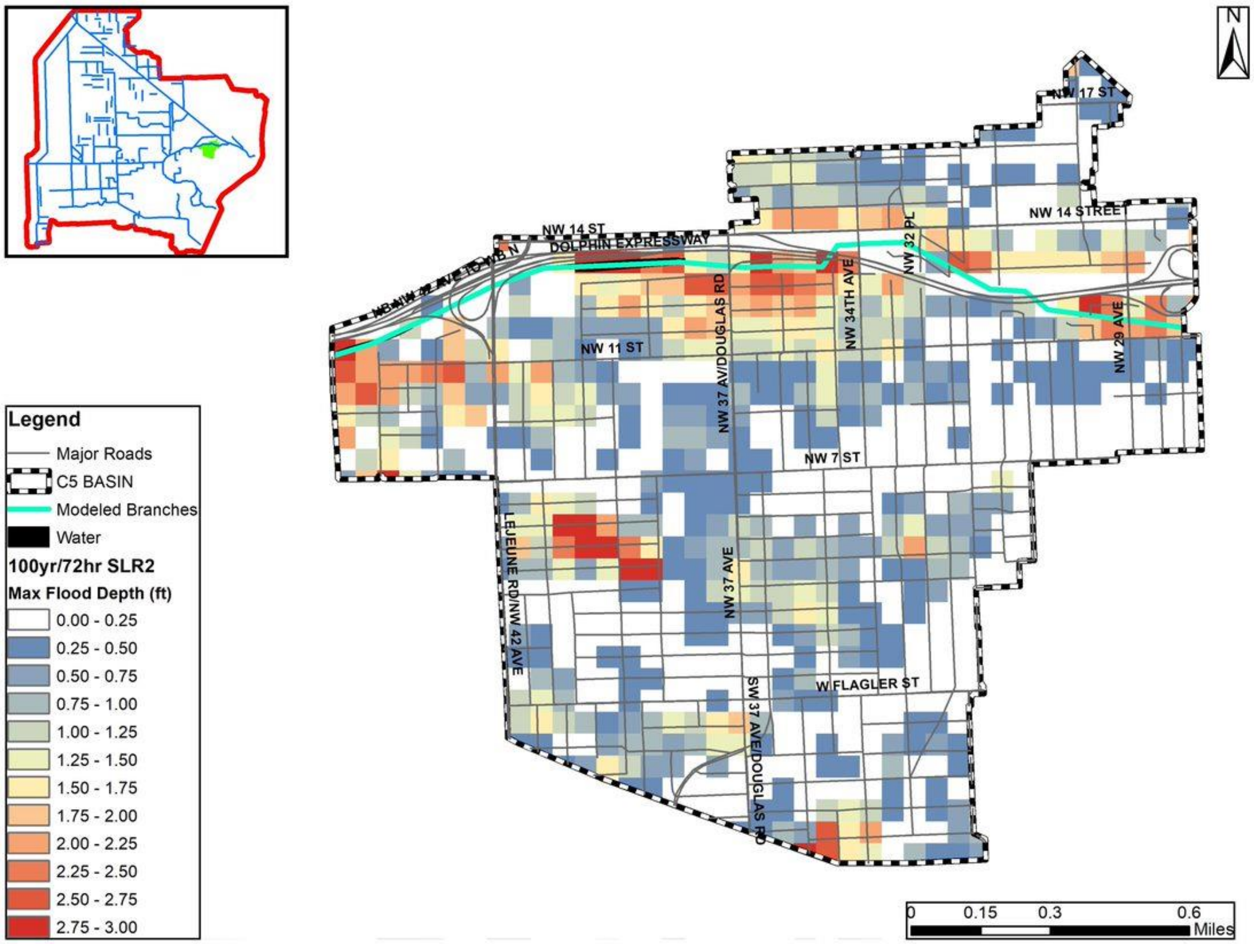


Figure C 4-16. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR3 in the C5 Watershed

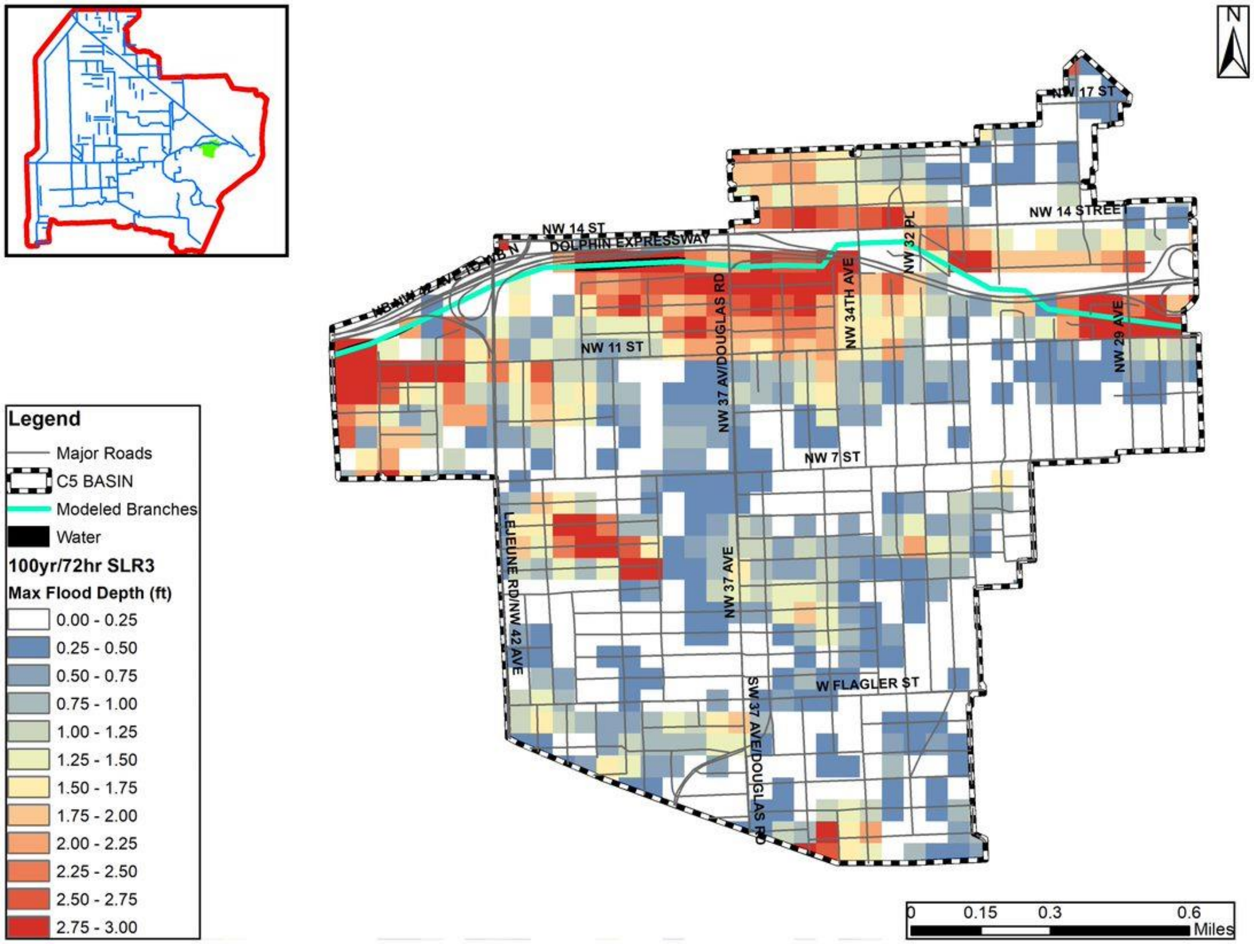


Figure C 4-17. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C5 Watershed

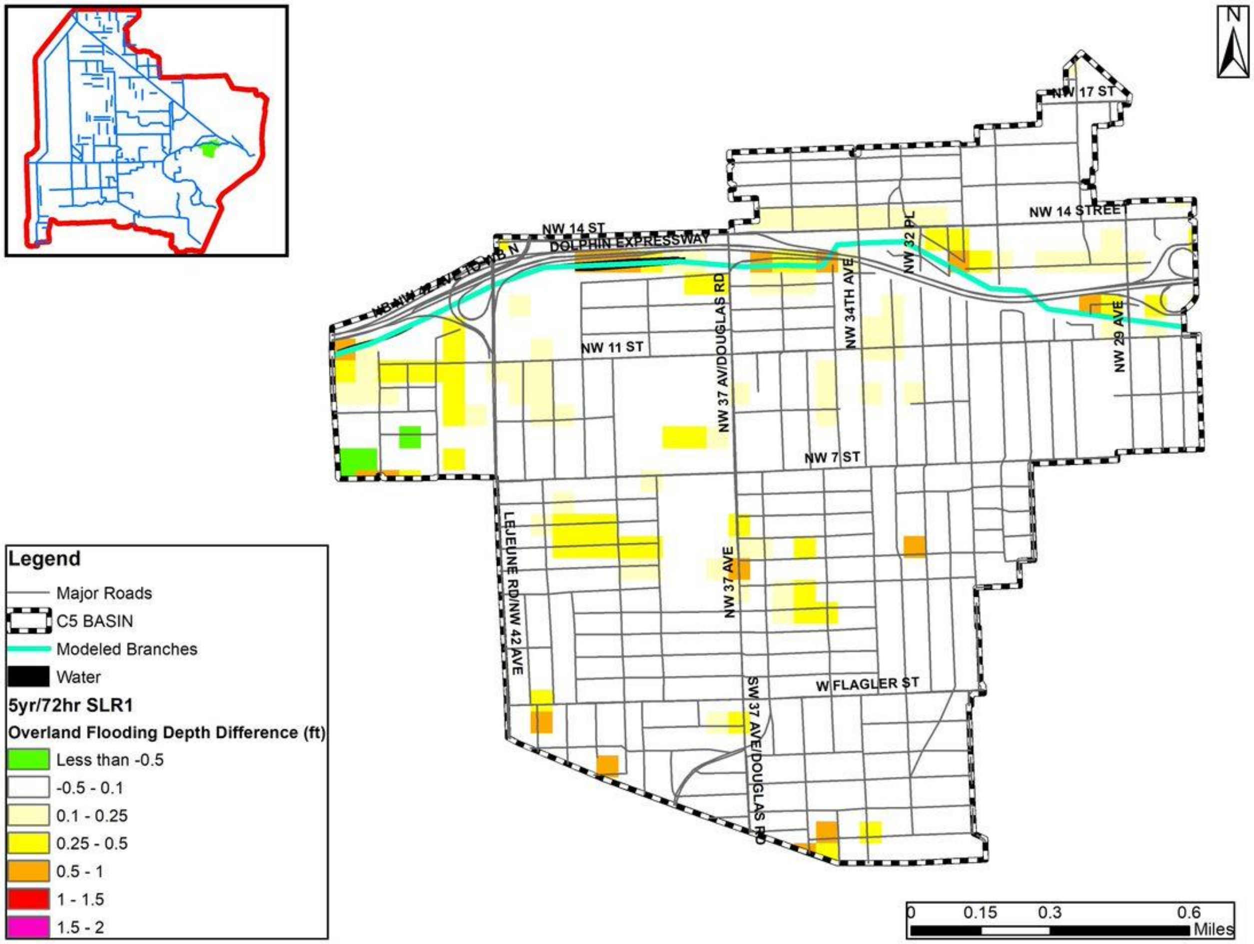


Figure C 4-18. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C5 Watershed

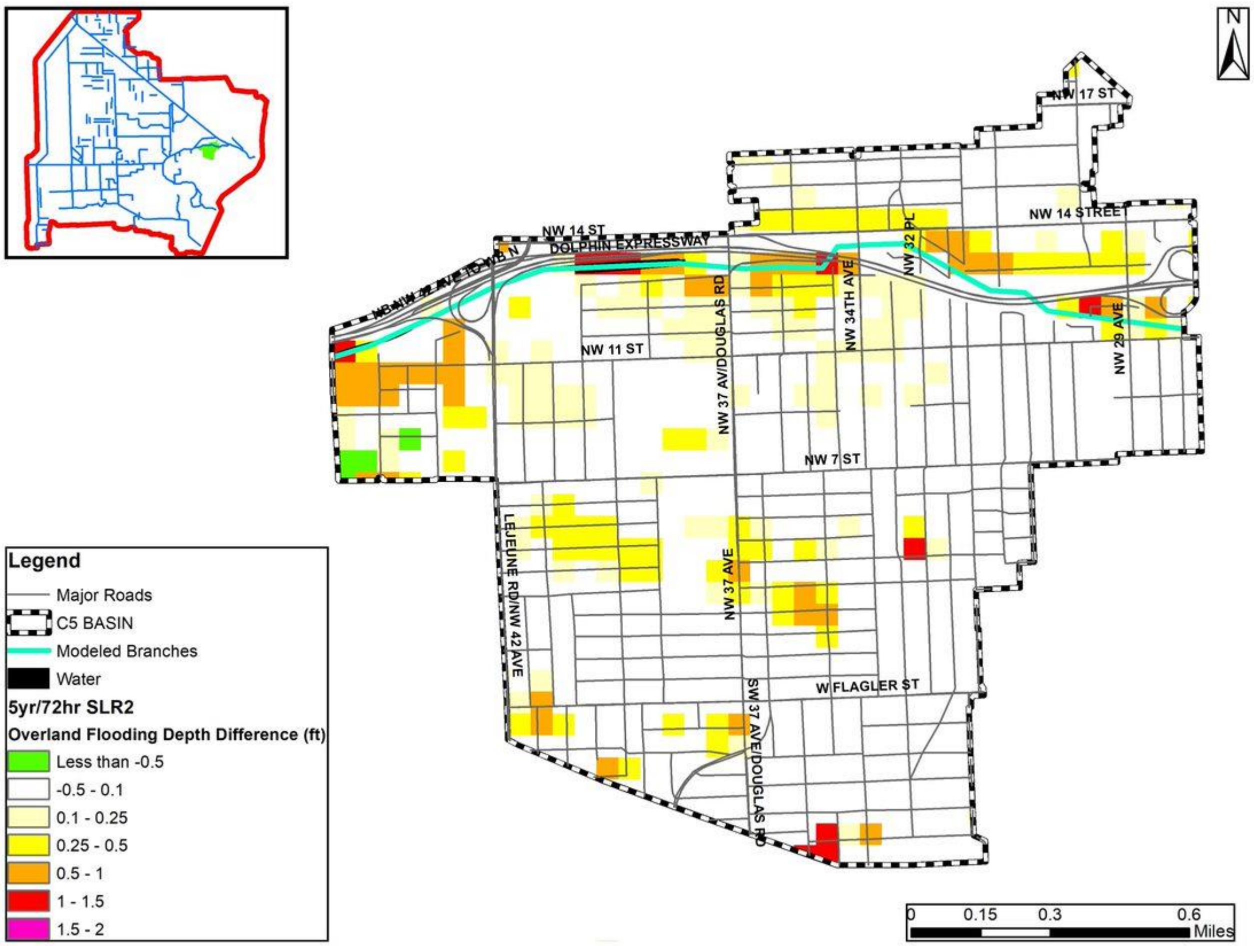


Figure C 4-19. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C5 Watershed

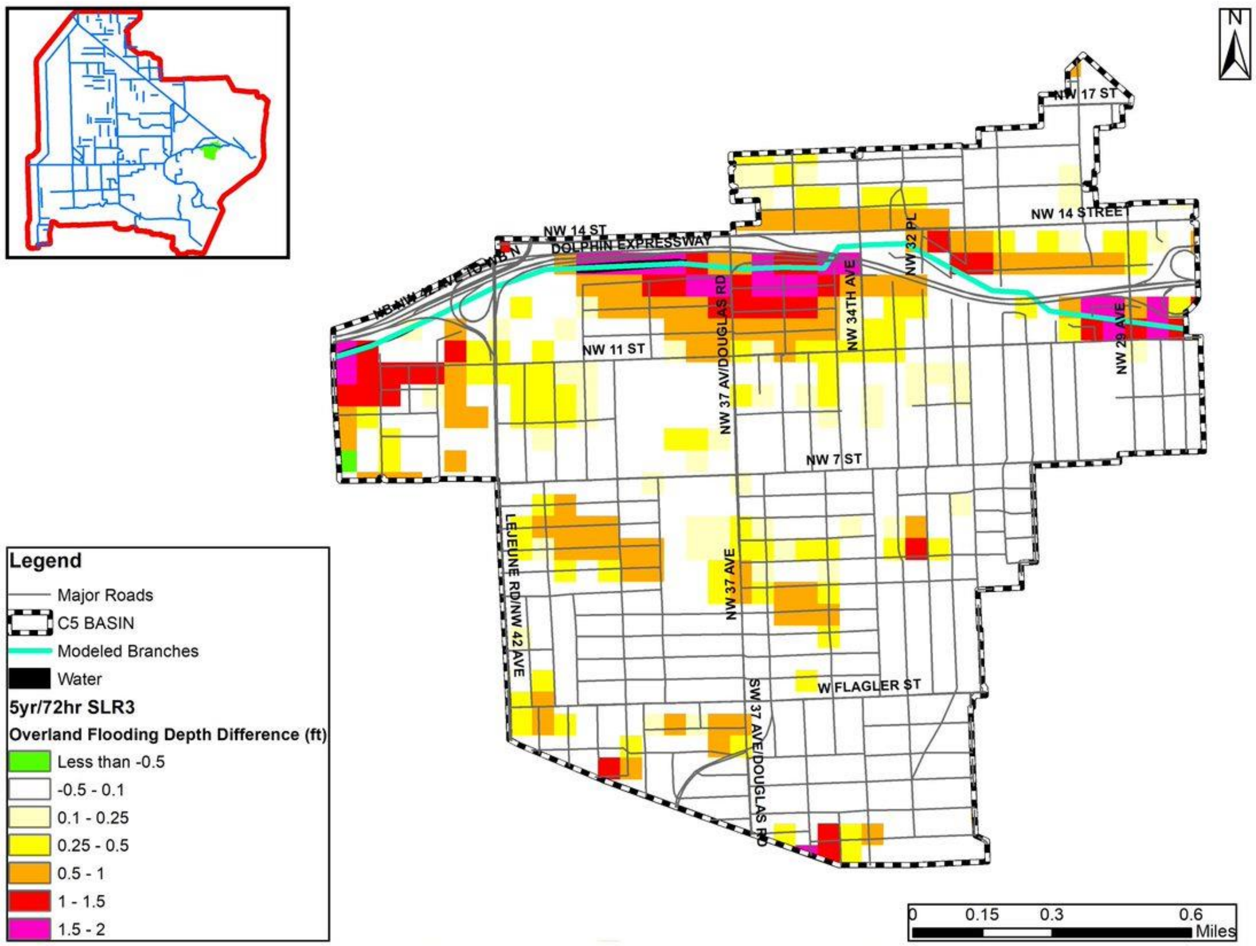


Figure C 4-20. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C5 Watershed

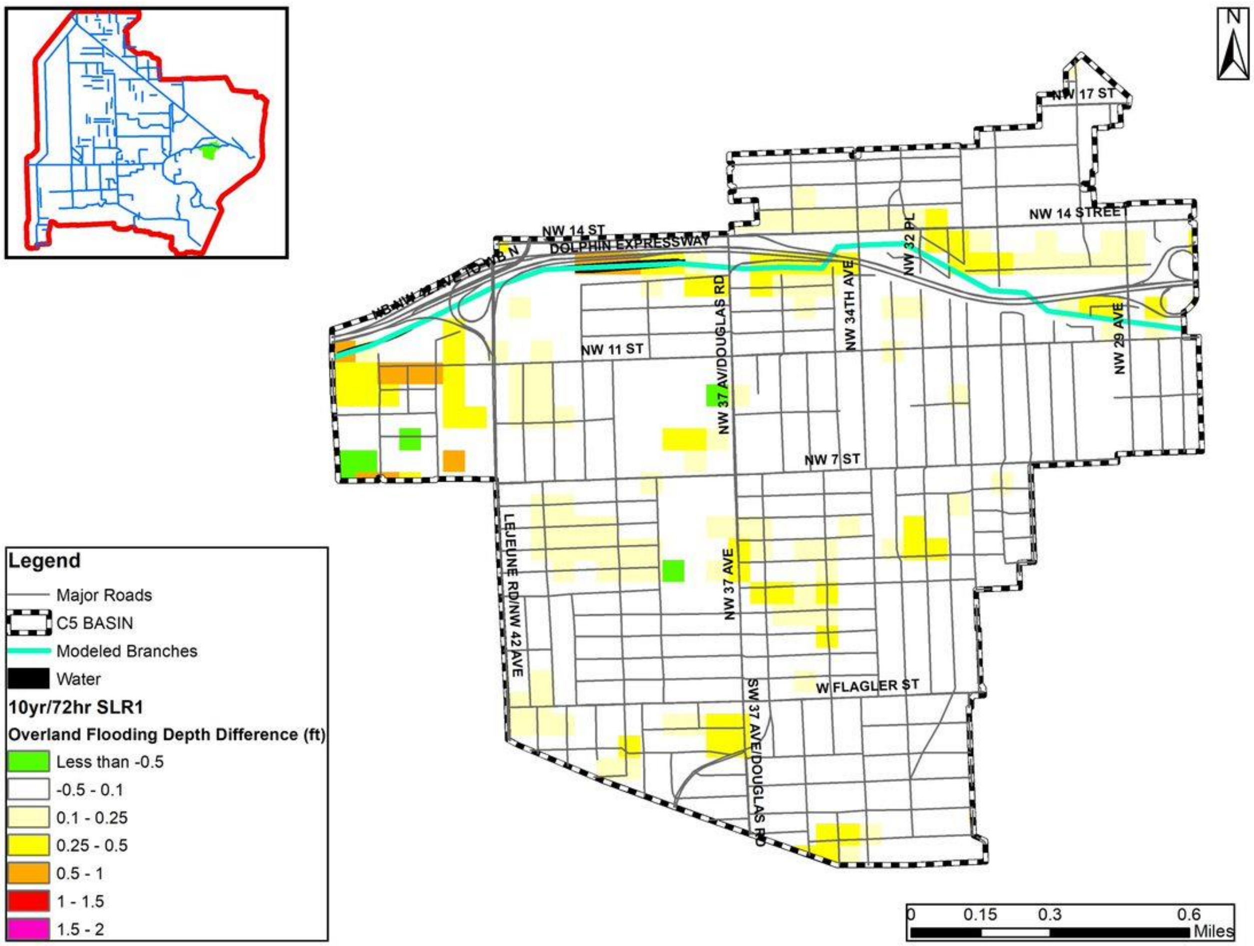


Figure C 4-21. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C5 Watershed

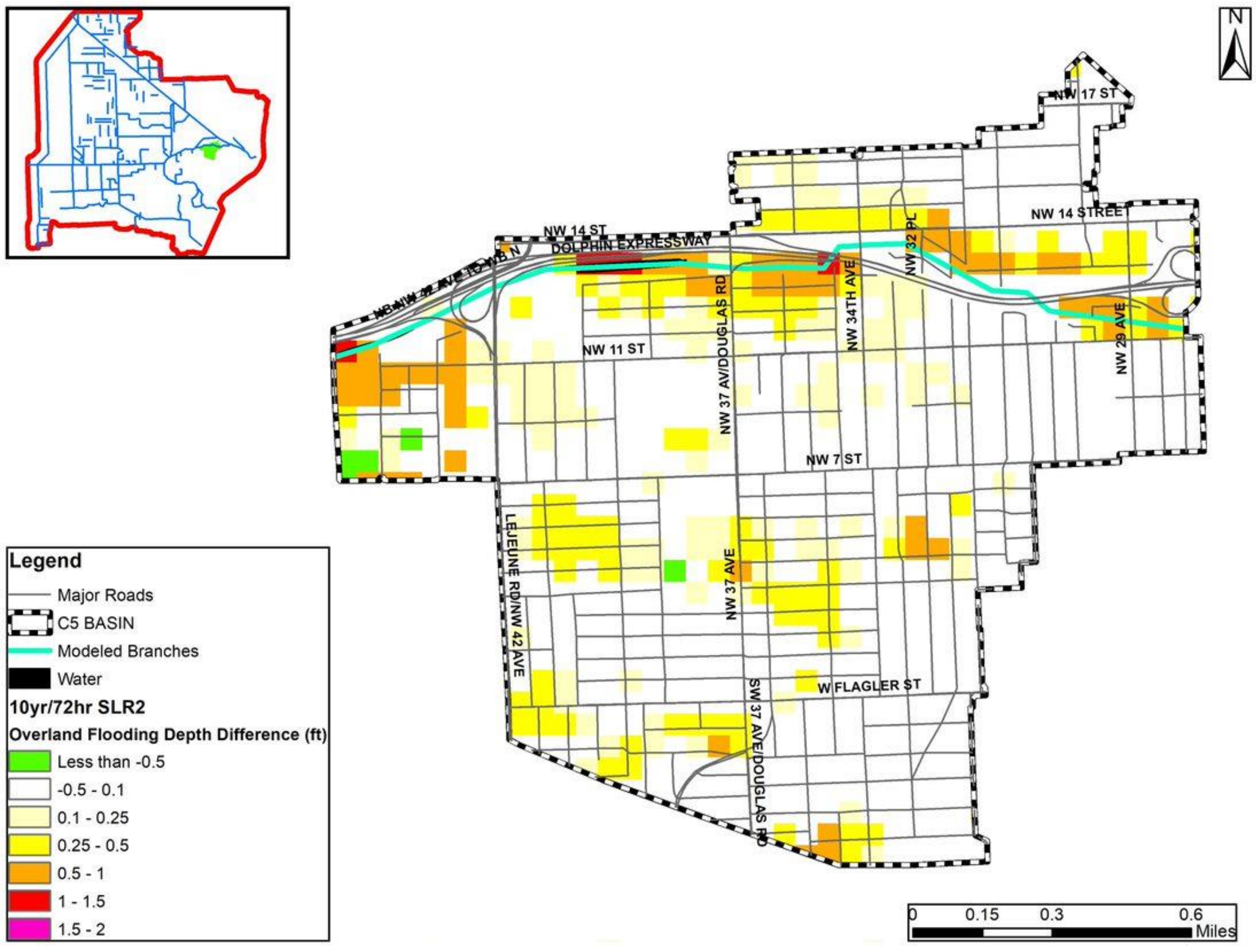


Figure C 4-22. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C5 Watershed

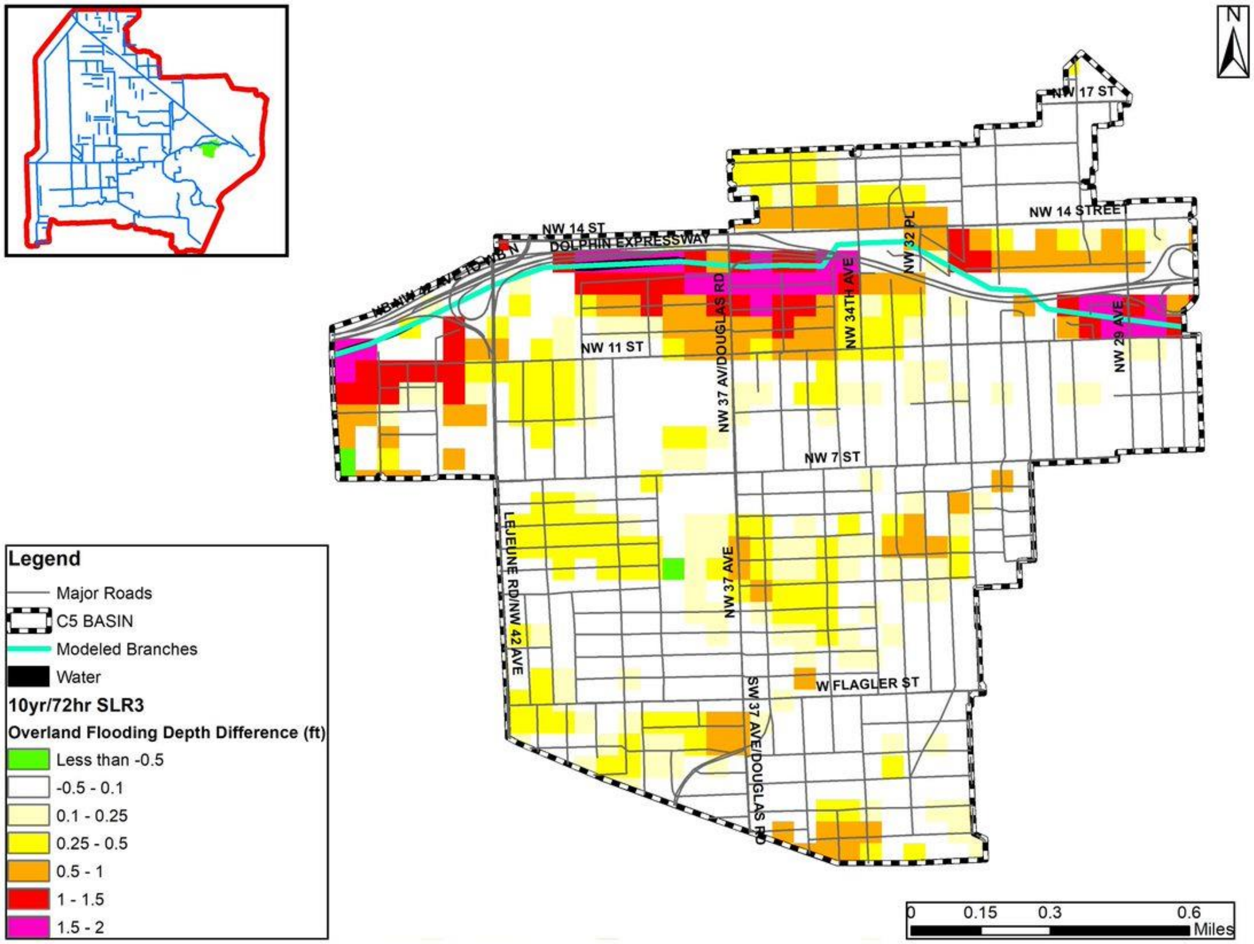


Figure C 4-23. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C5 Watershed

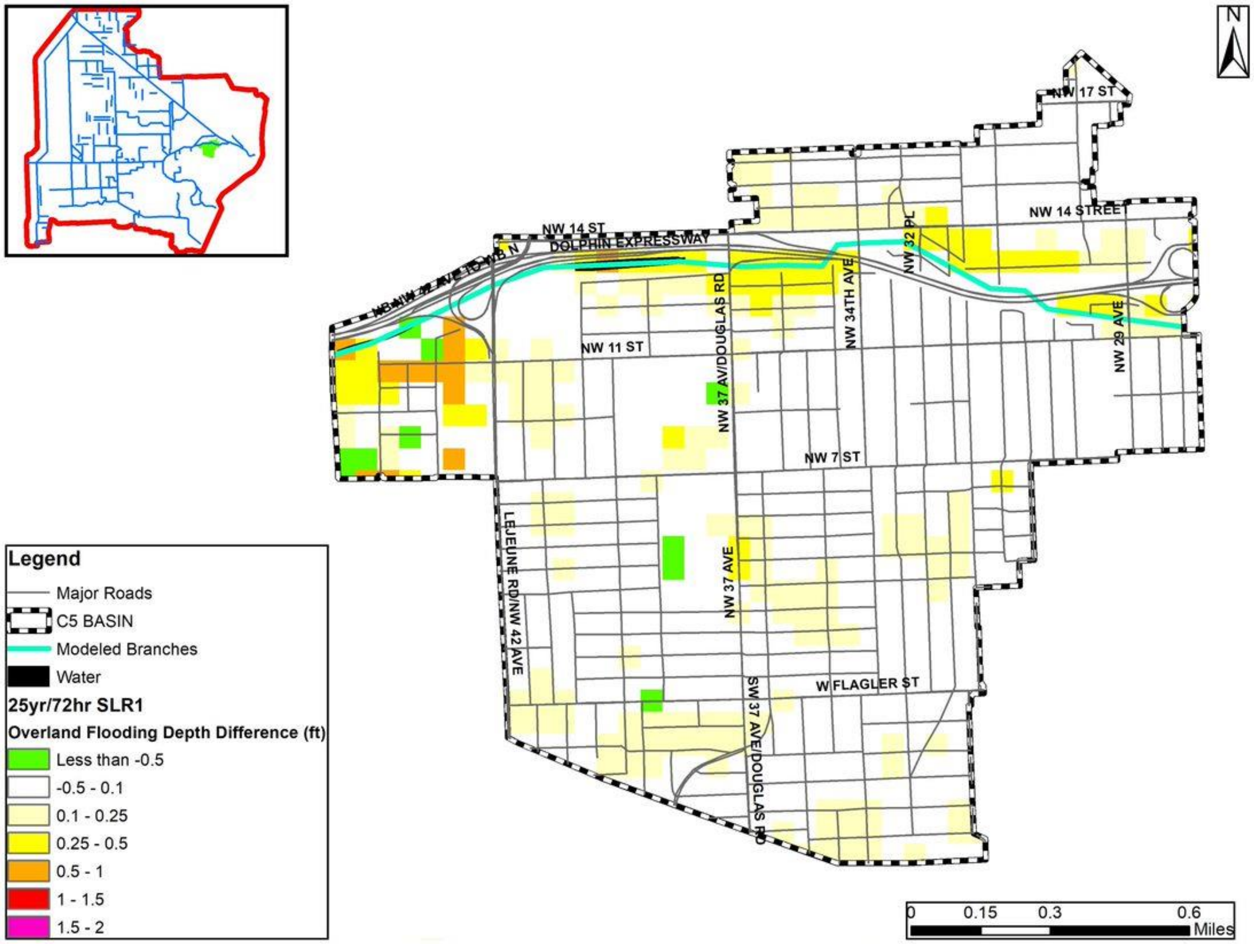


Figure C 4-24. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C5 Watershed

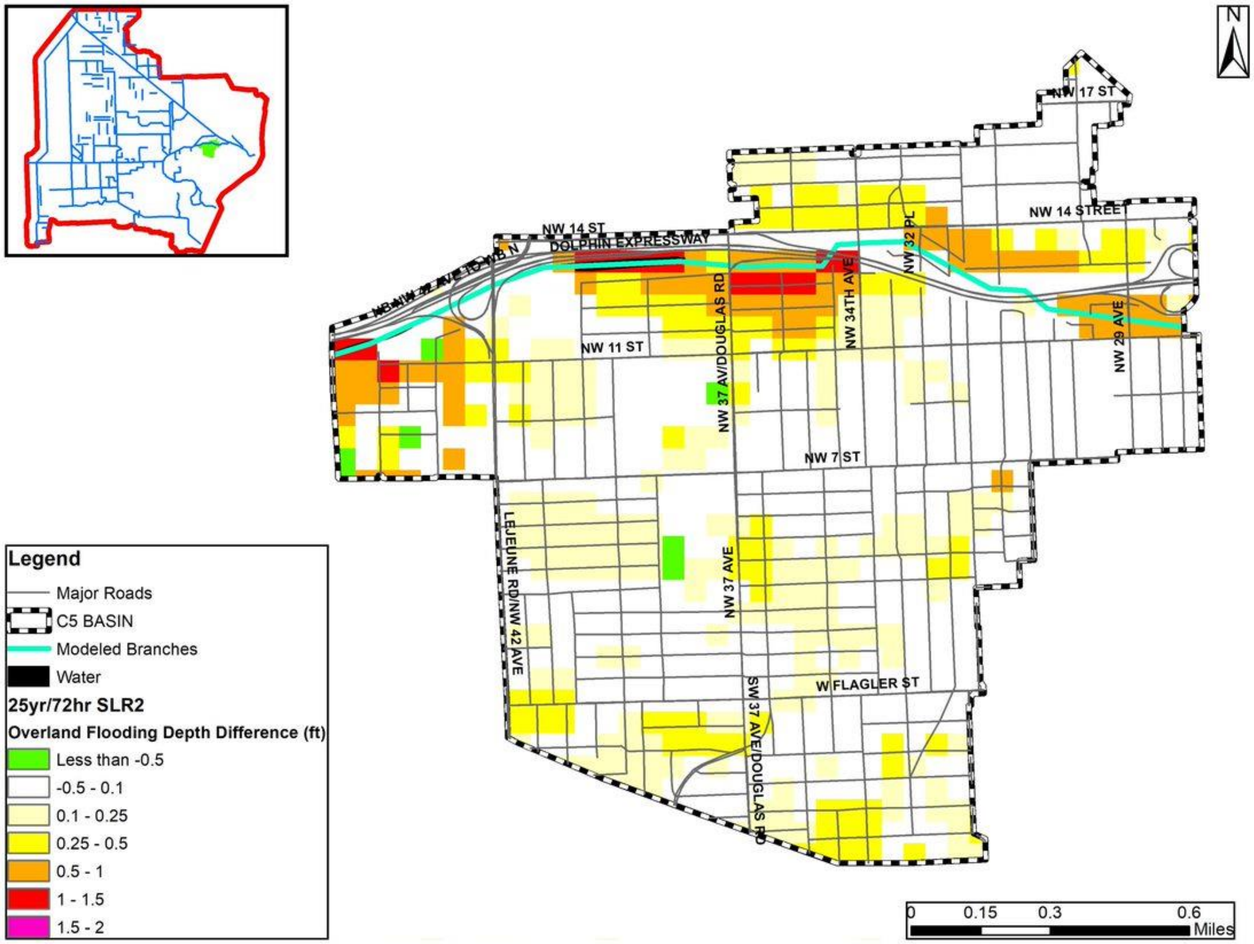


Figure C 4-25. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C5 Watershed

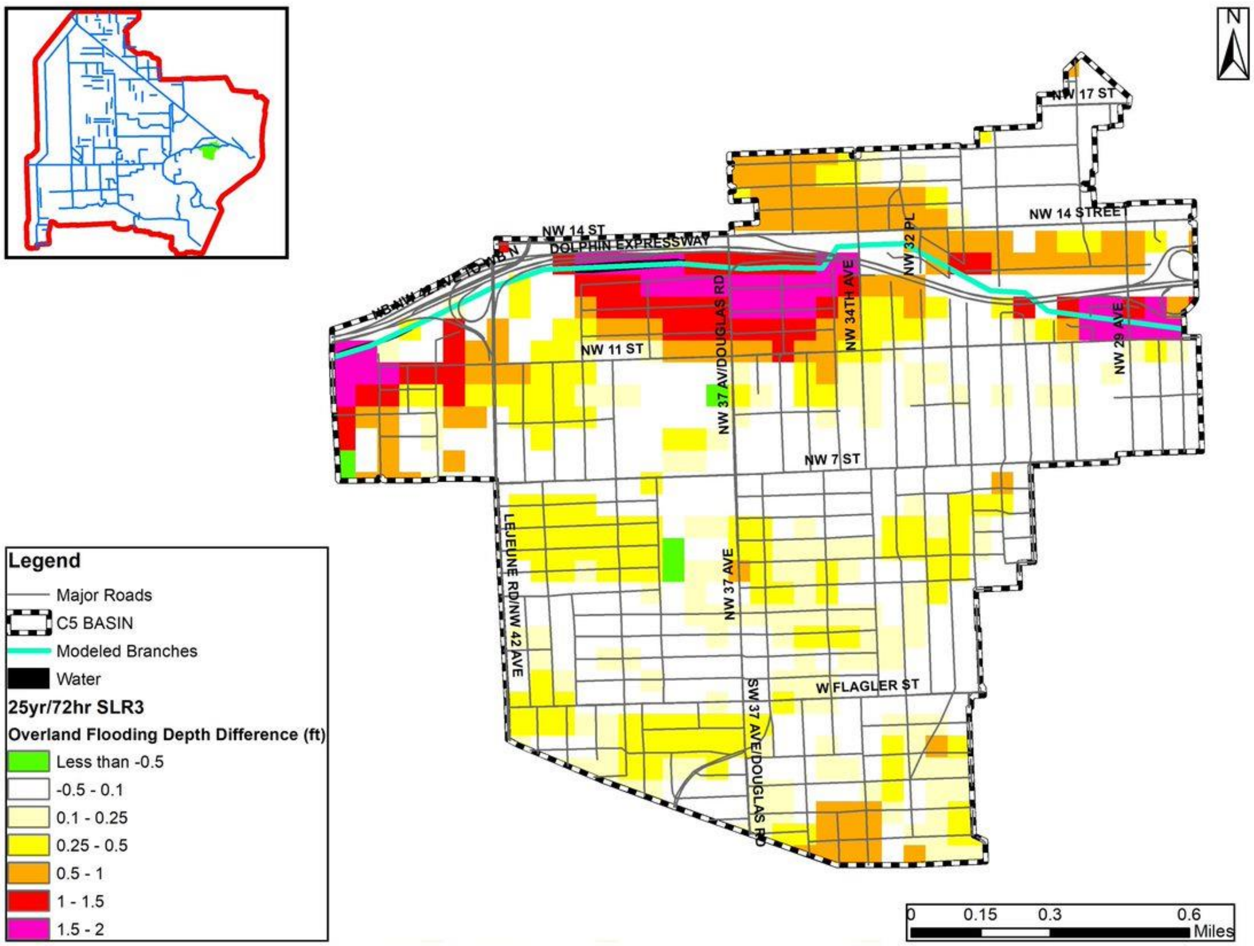


Figure C 4-26. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C5 Watershed

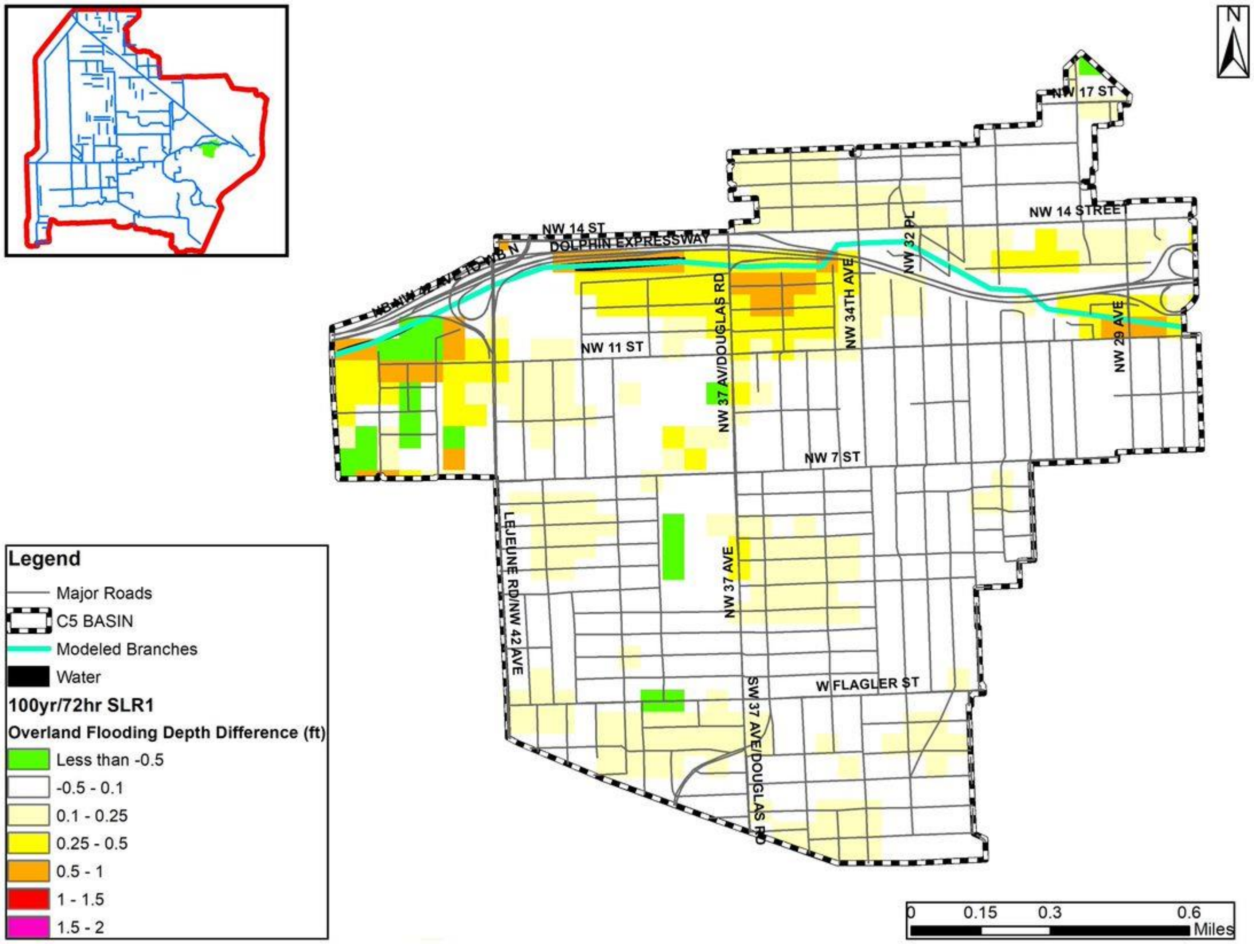


Figure C 4-27. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C5 Watershed

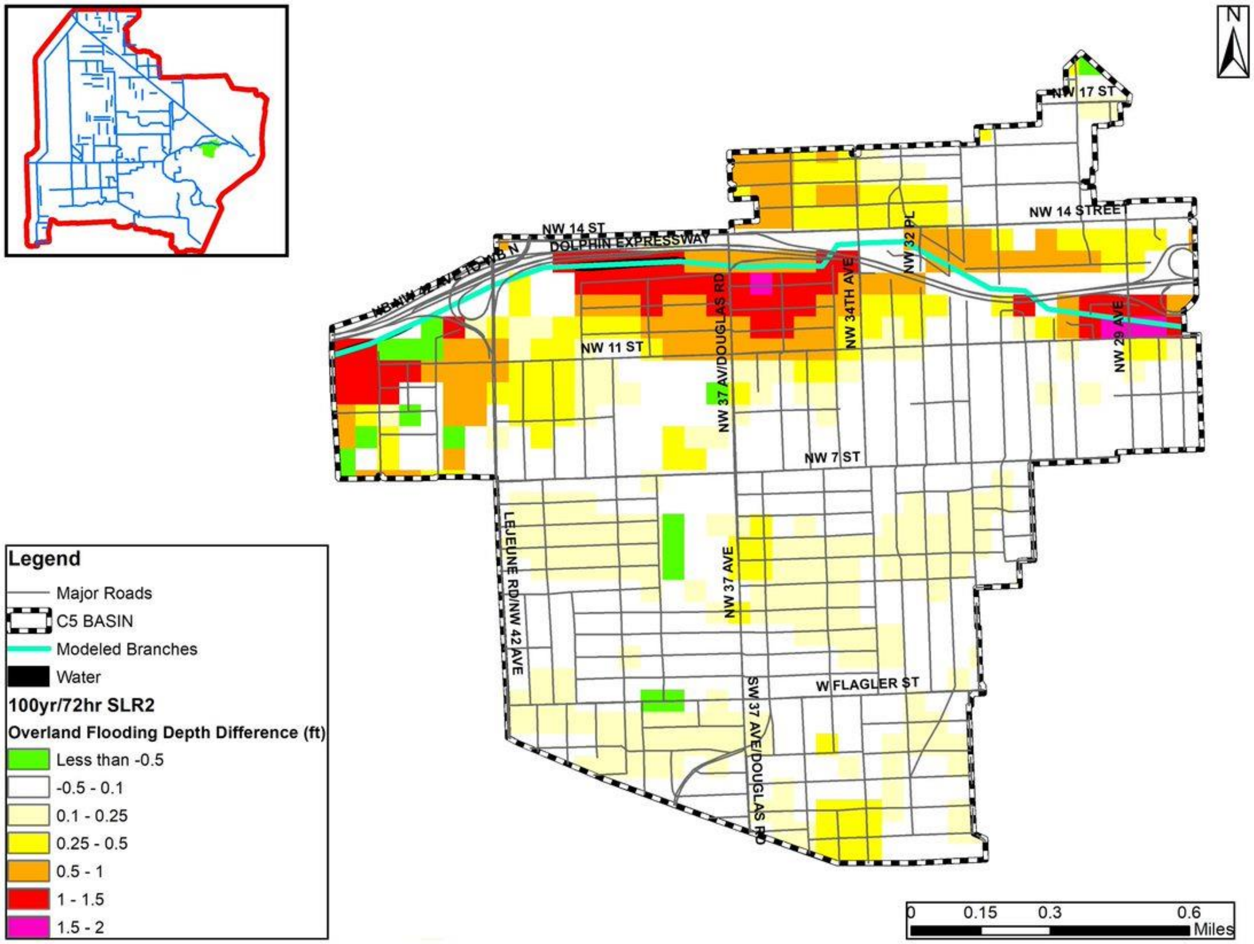
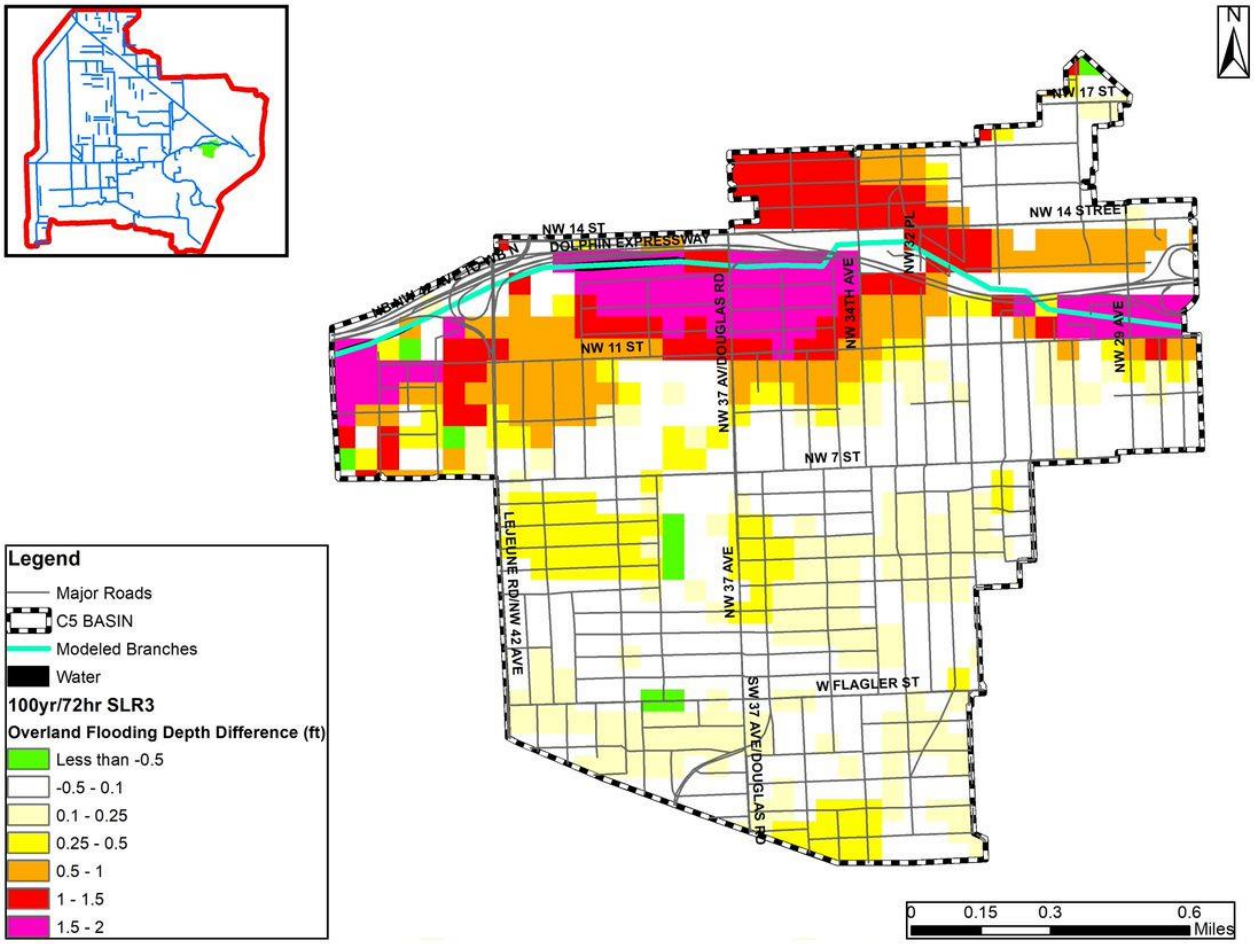


Figure C 4-28. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C5 Watershed



5 C6 Watershed

Figure C 5-1 through **Figure C 5-16** are maps of the maximum overland depth over the entire C6 Watershed for the 5-year, 10-year, 25-year, and 100-year 3-day design storm events for each SLR condition. Water areas are masked in black. **Figure C 5-17** through **Figure C 5-32** provide the same maps with the non-urban areas masked out, to provide a concise picture of how urban areas are impacted within the watershed.

Figure C 5-33 through **Figure C 5-44** show the difference in overland flooding for the C6 Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively. **Figure C 5-45** through **Figure C 5-56** show the same maps with the non-urban areas masked out.

Figure C 5-1. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for Current Conditions in the C6 Watershed

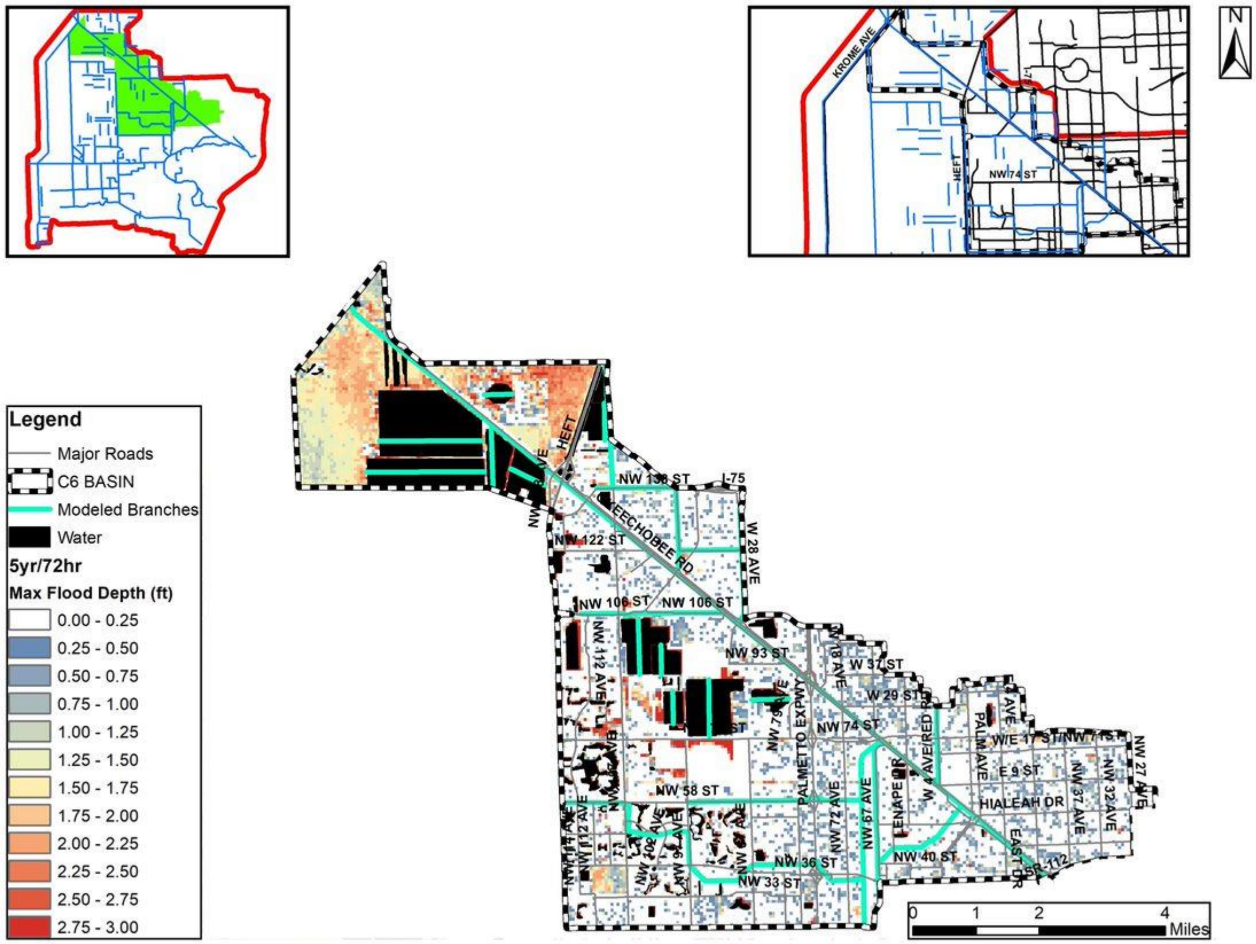


Figure C 5-2. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR1 in the C6 Watershed

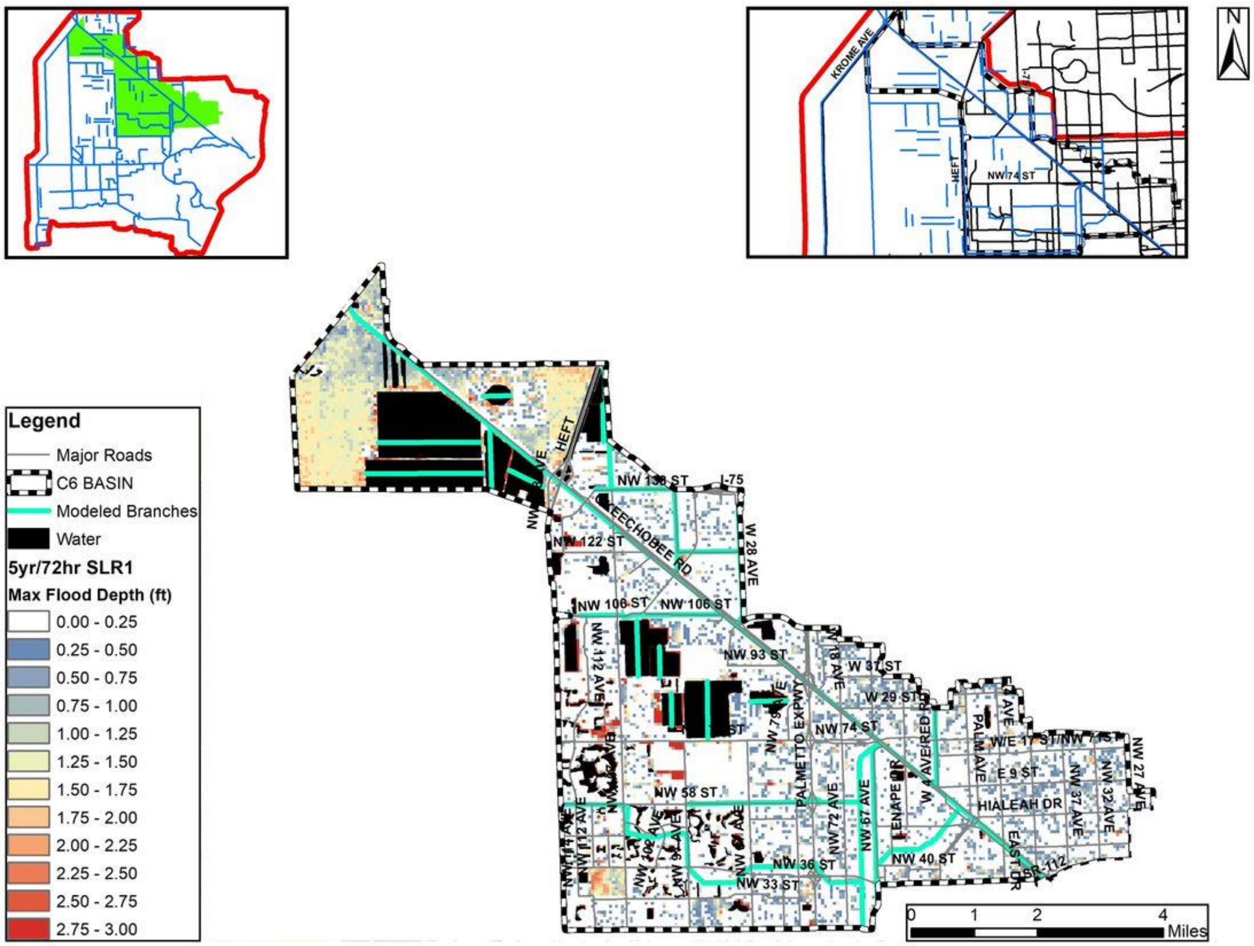


Figure C 5-3. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR2 in the C6 Watershed

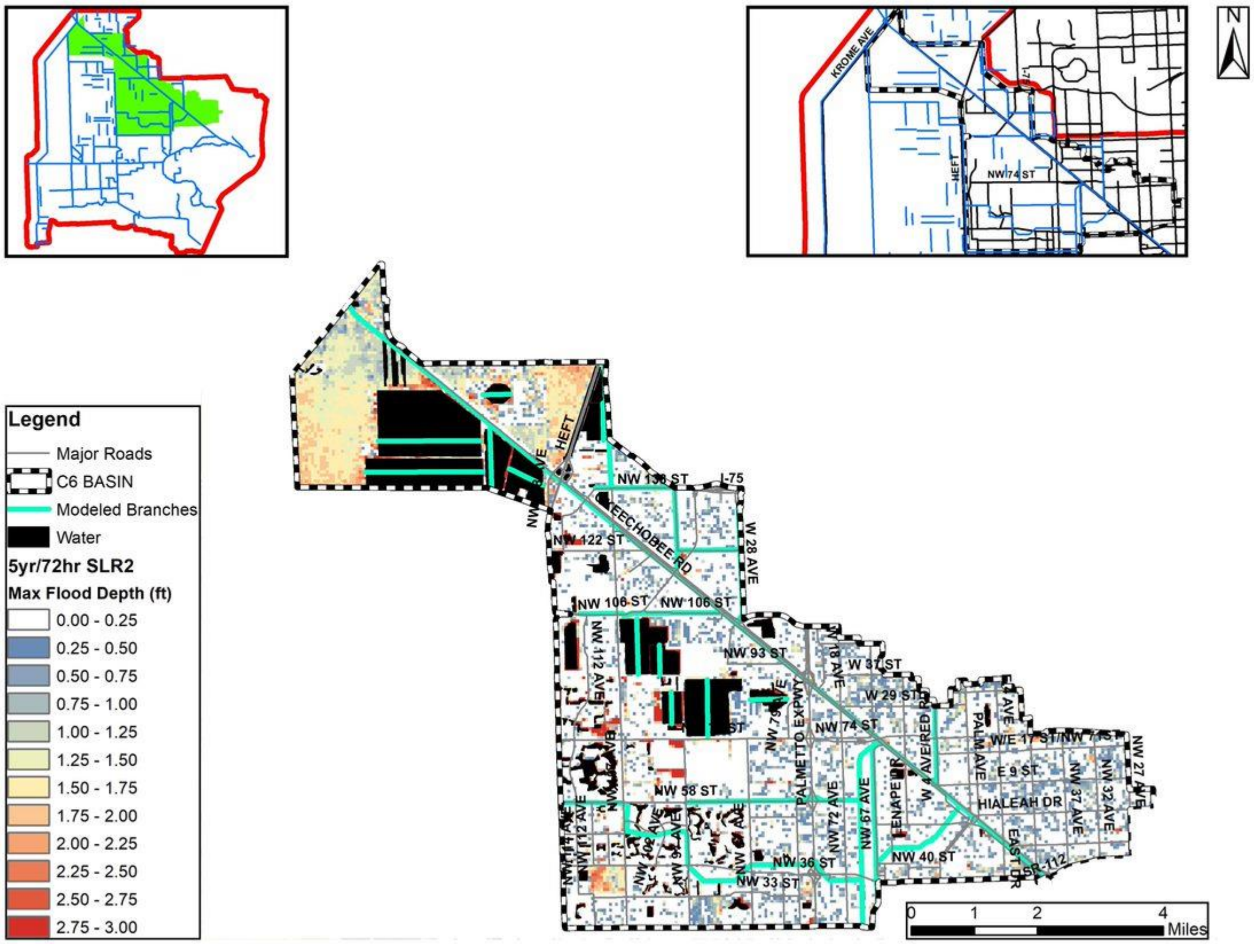


Figure C 5-4. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR3 in the C6 Watershed

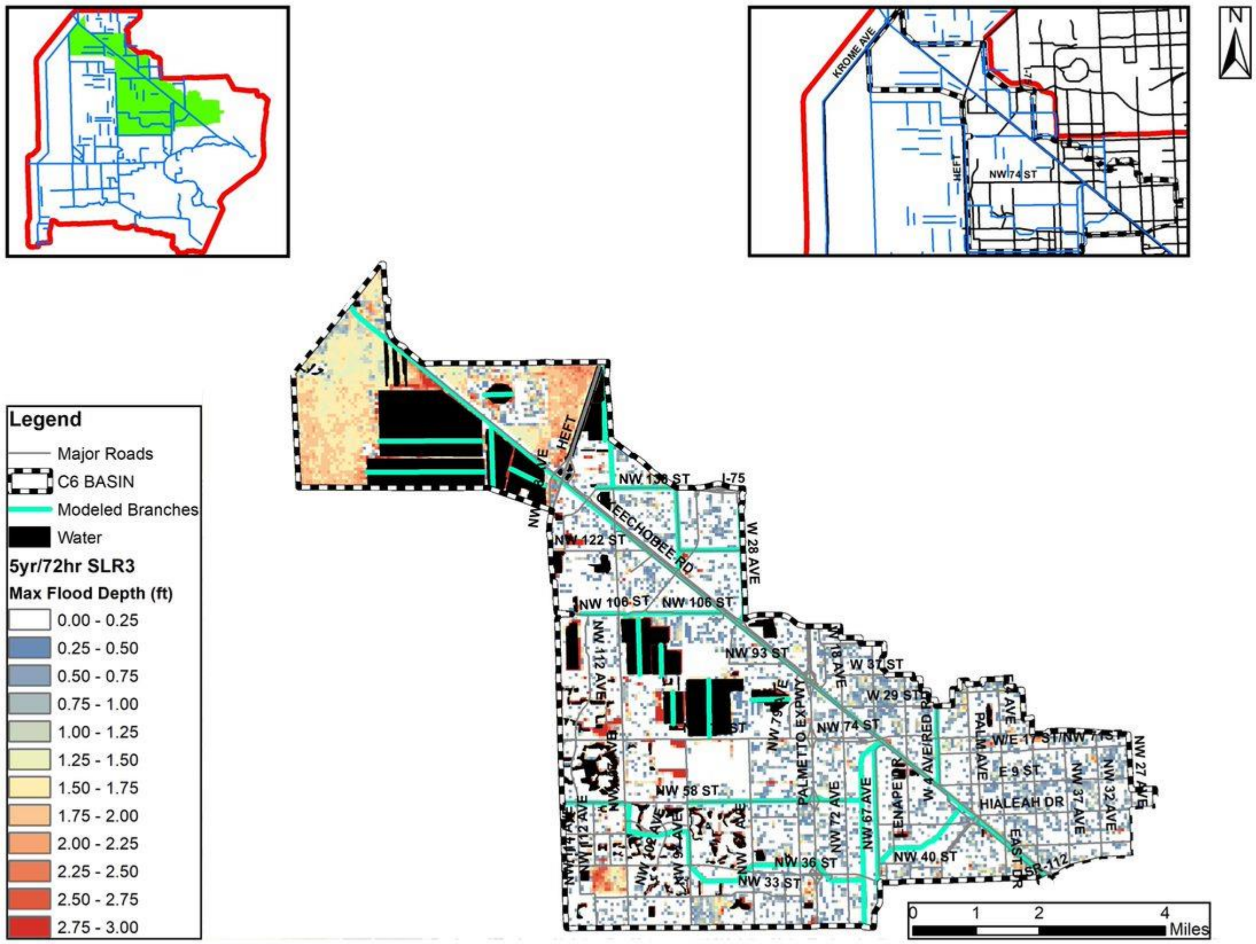


Figure C 5-5. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for Current Conditions in the C6 Watershed

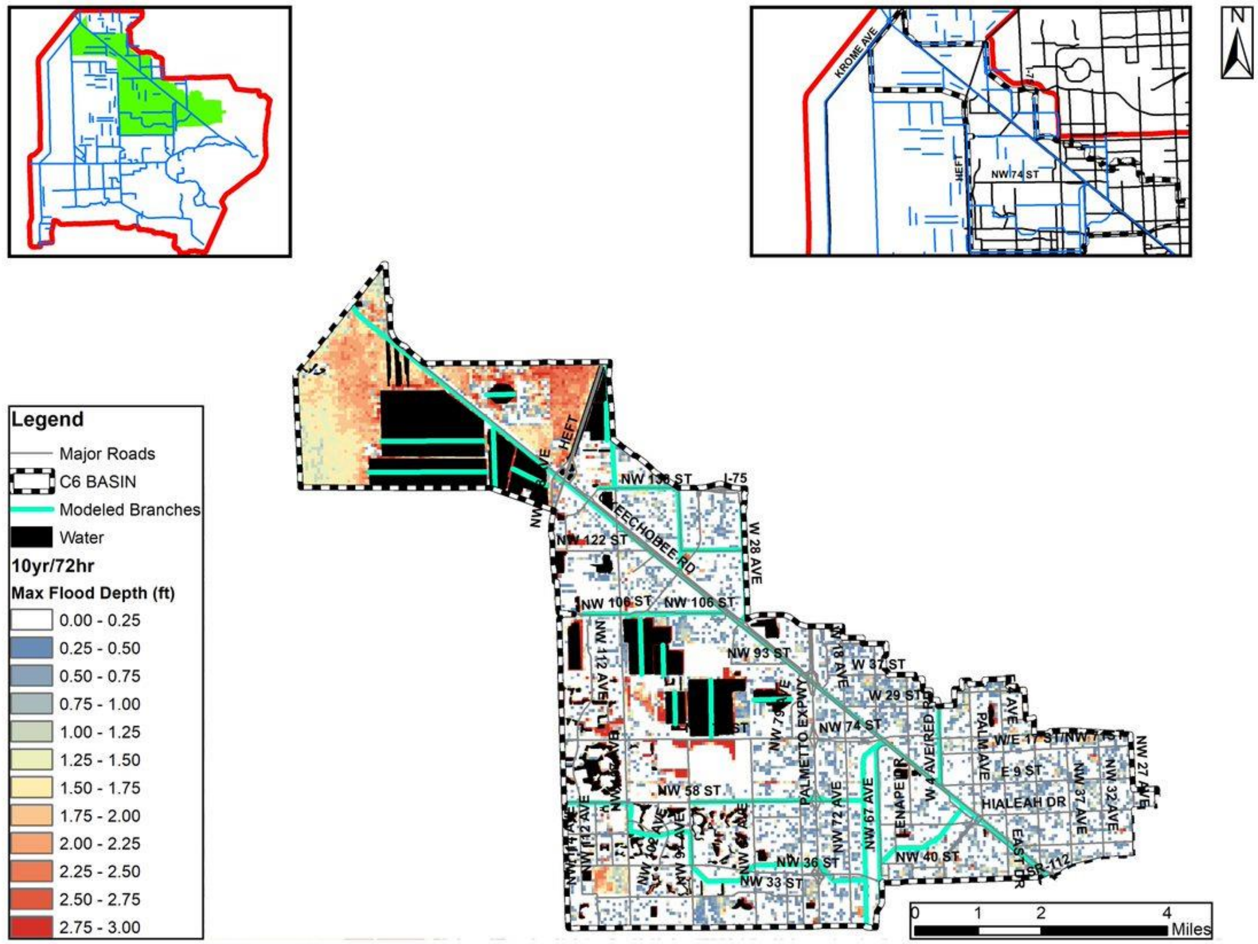


Figure C 5-6. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR1 in the C6 Watershed

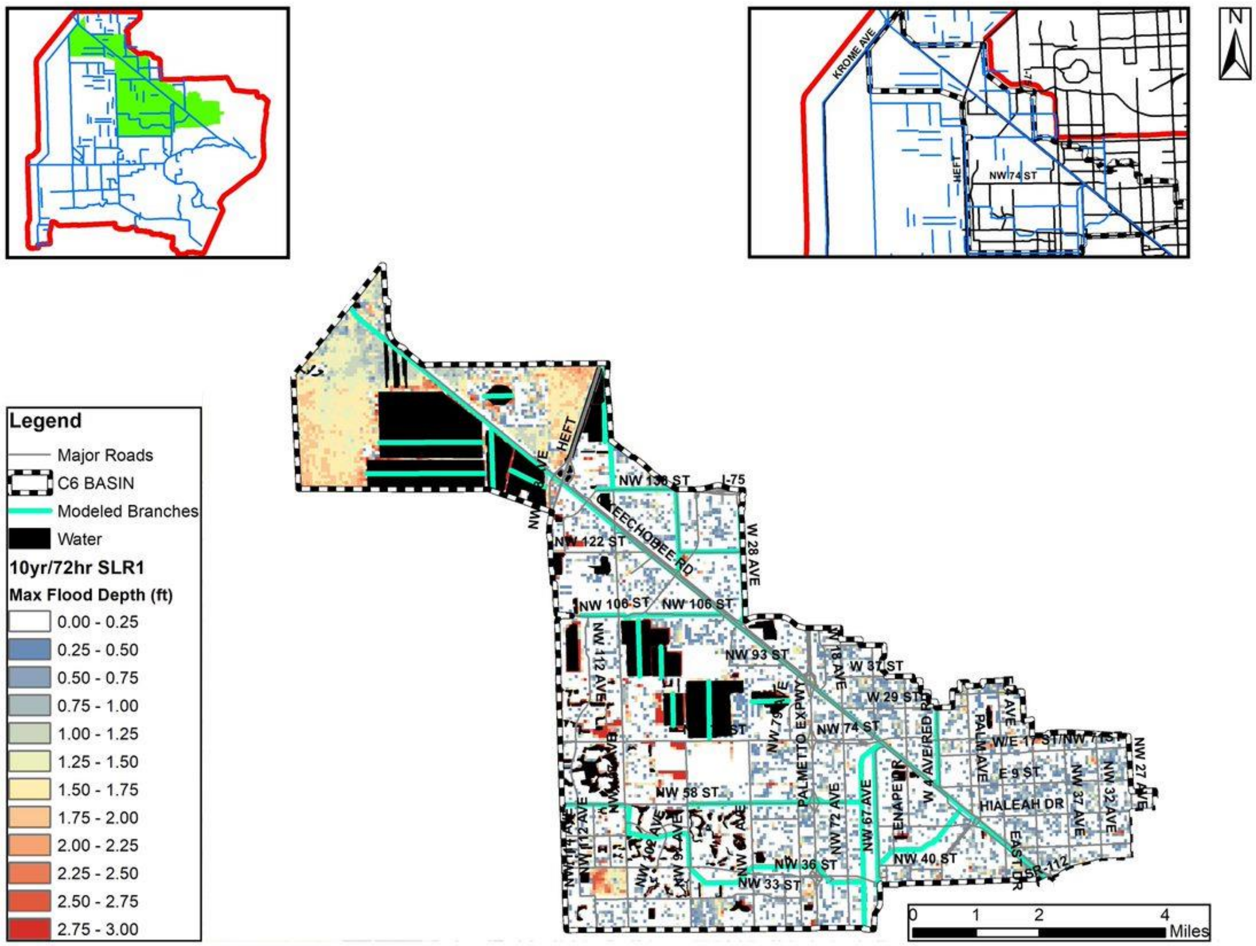


Figure C 5-7. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR2 in the C6 Watershed

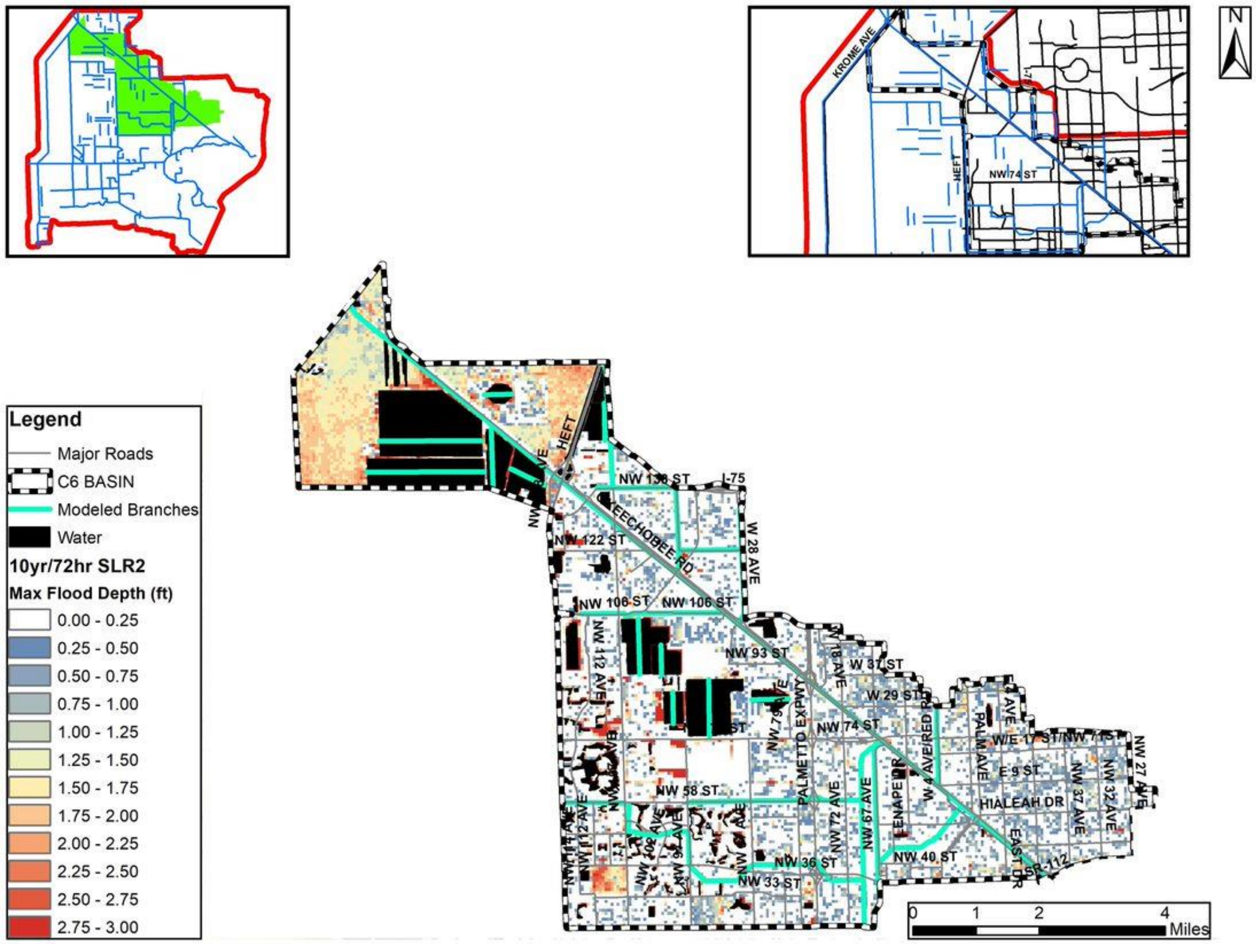


Figure C 5-8. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR3 in the C6 Watershed

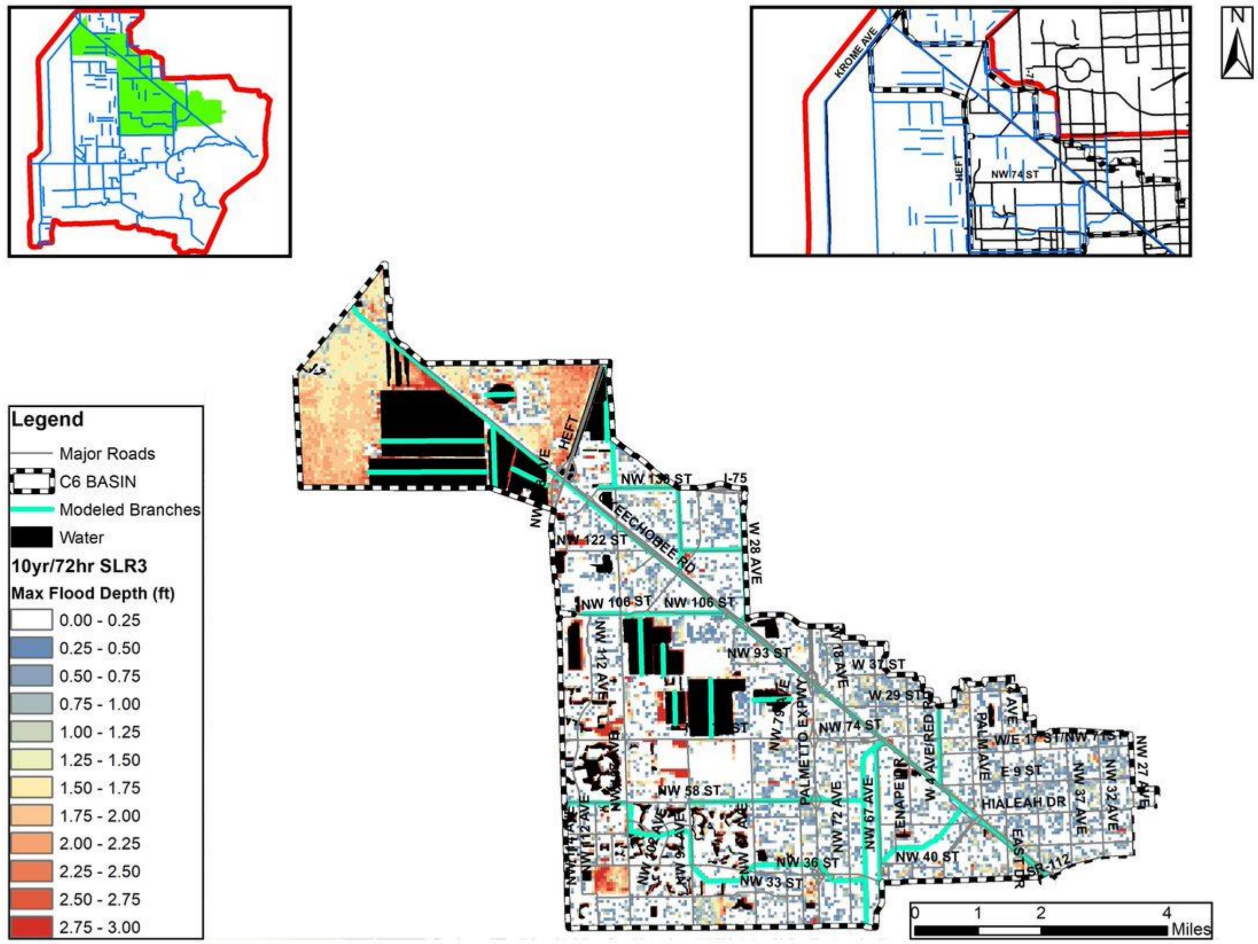


Figure C 5-9. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for Current Conditions in the C6 Watershed

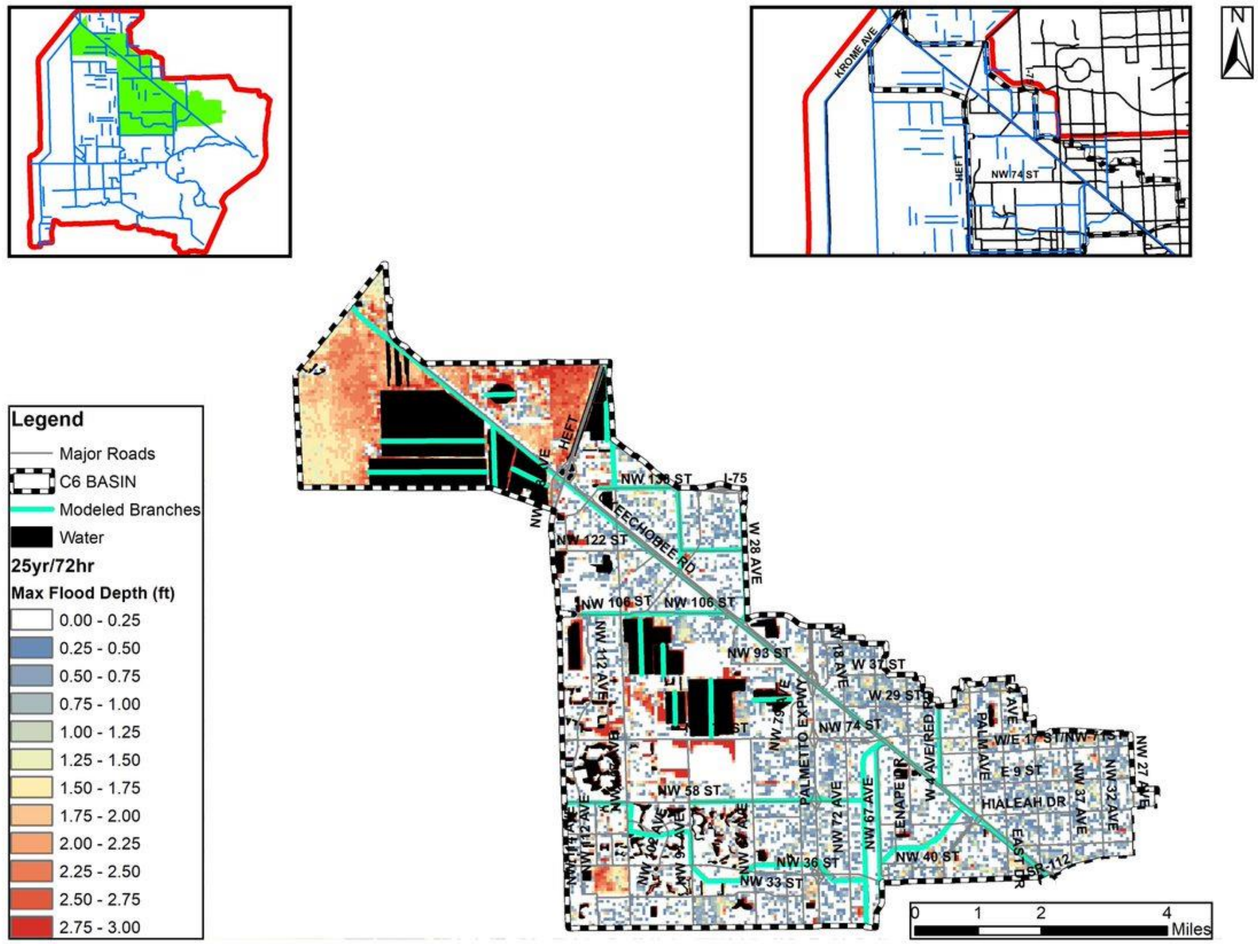


Figure C 5-10. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR1 in the C6 Watershed

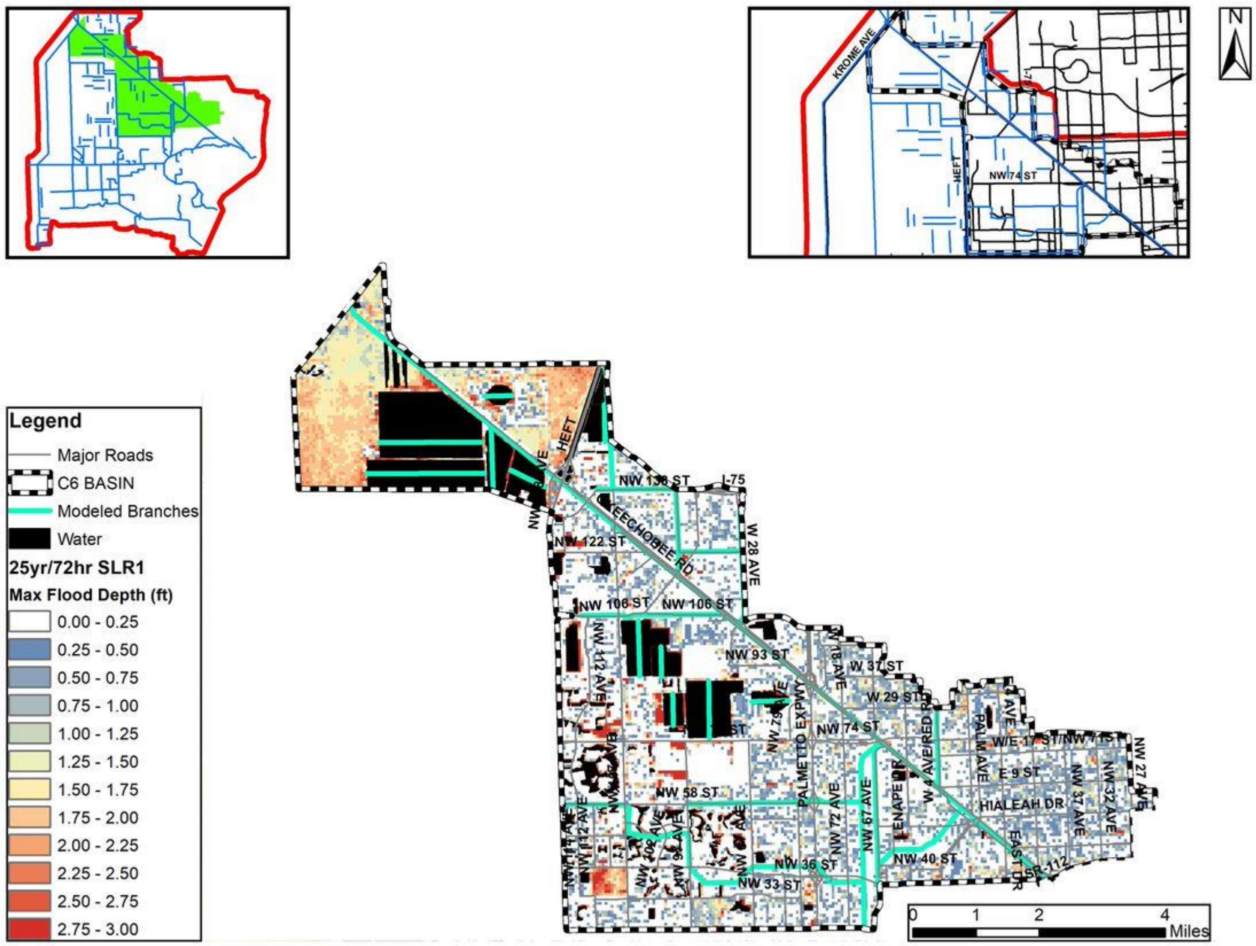


Figure C 5-11. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR2 in the C6 Watershed

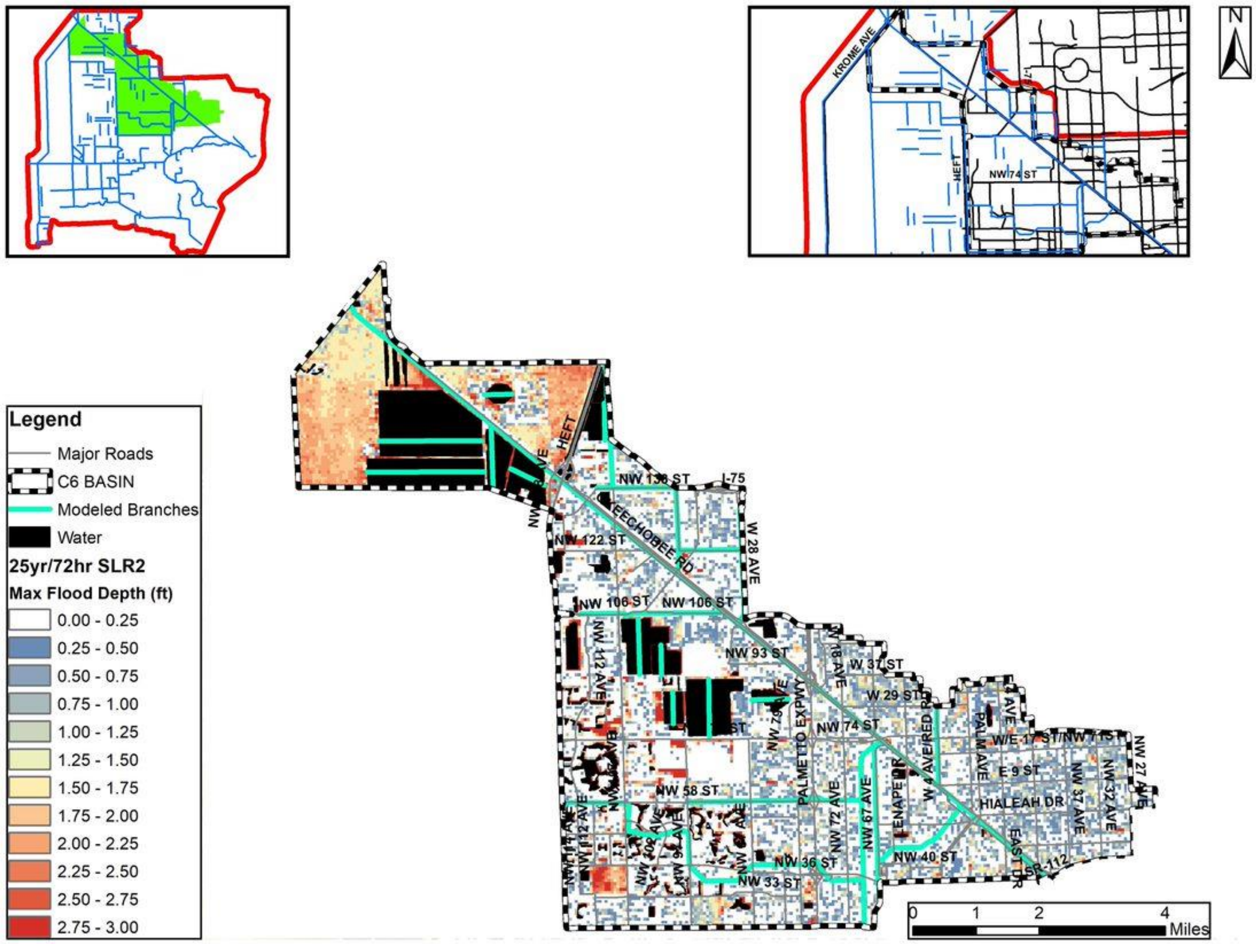


Figure C 5-12. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR3 in the C6 Watershed

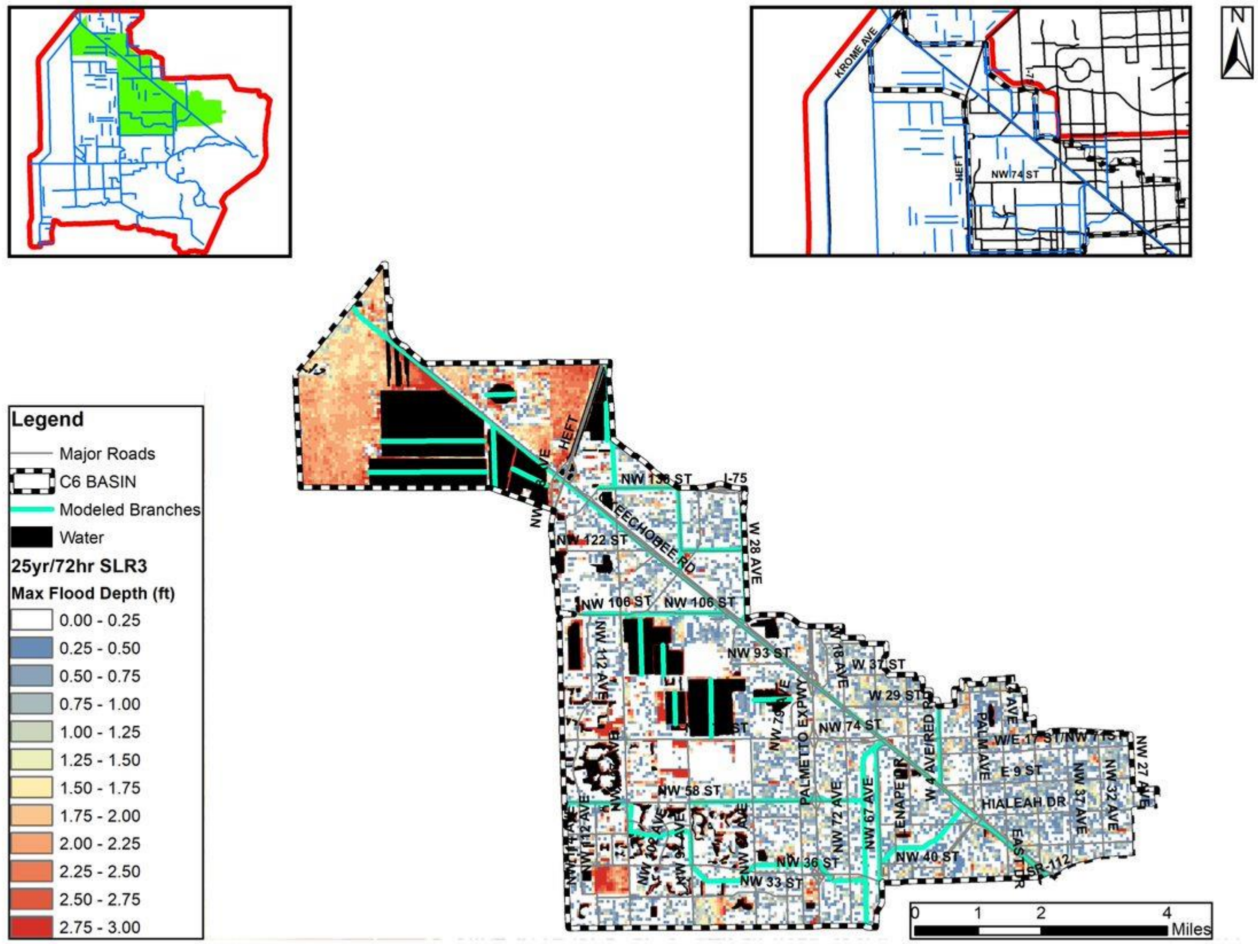


Figure C 5-13. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for Current Conditions in the C6 Watershed

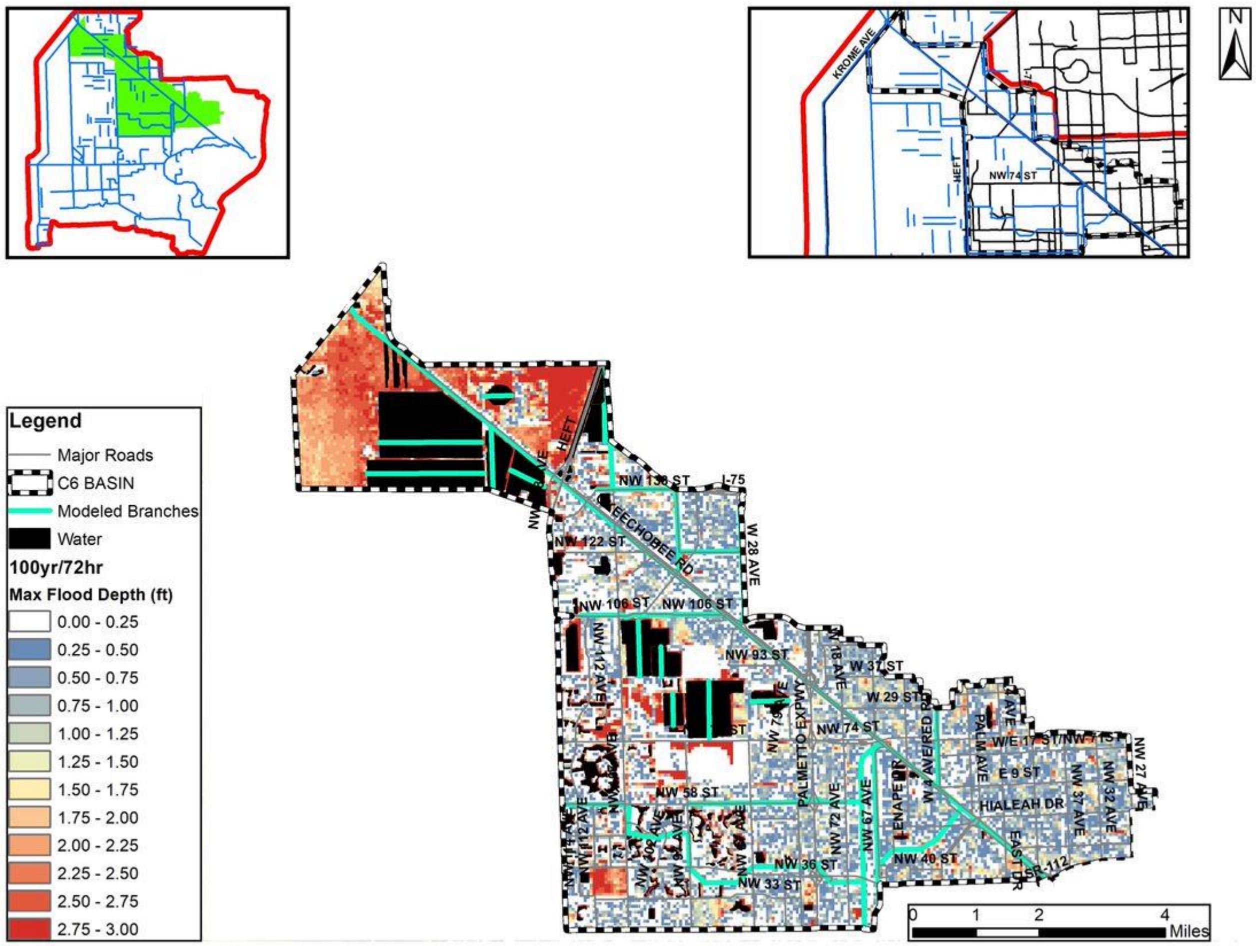


Figure C 5-14. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR1 in the C6 Watershed

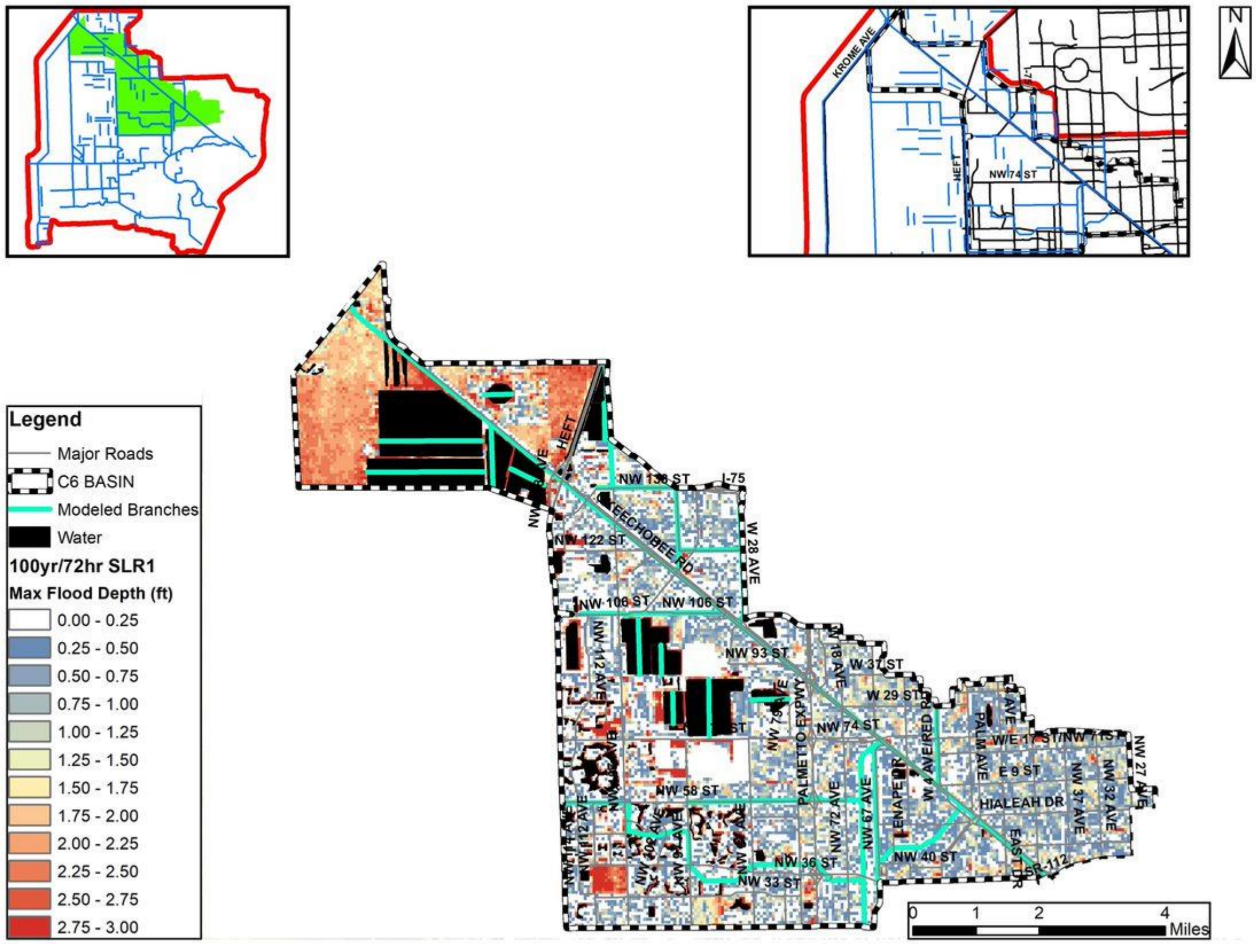


Figure C 5-15. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR2 in the C6 Watershed

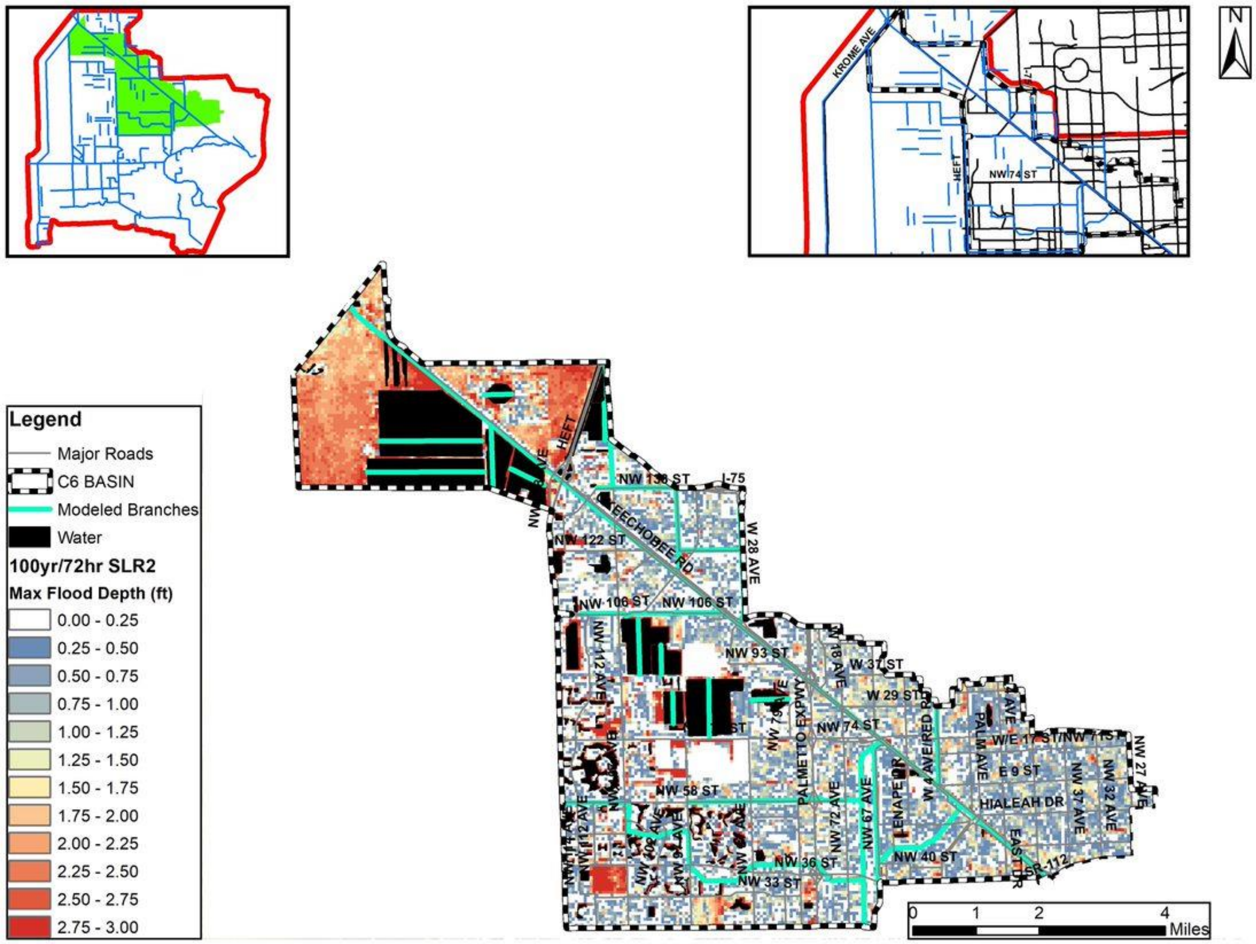


Figure C 5-16. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR3 in the C6 Watershed

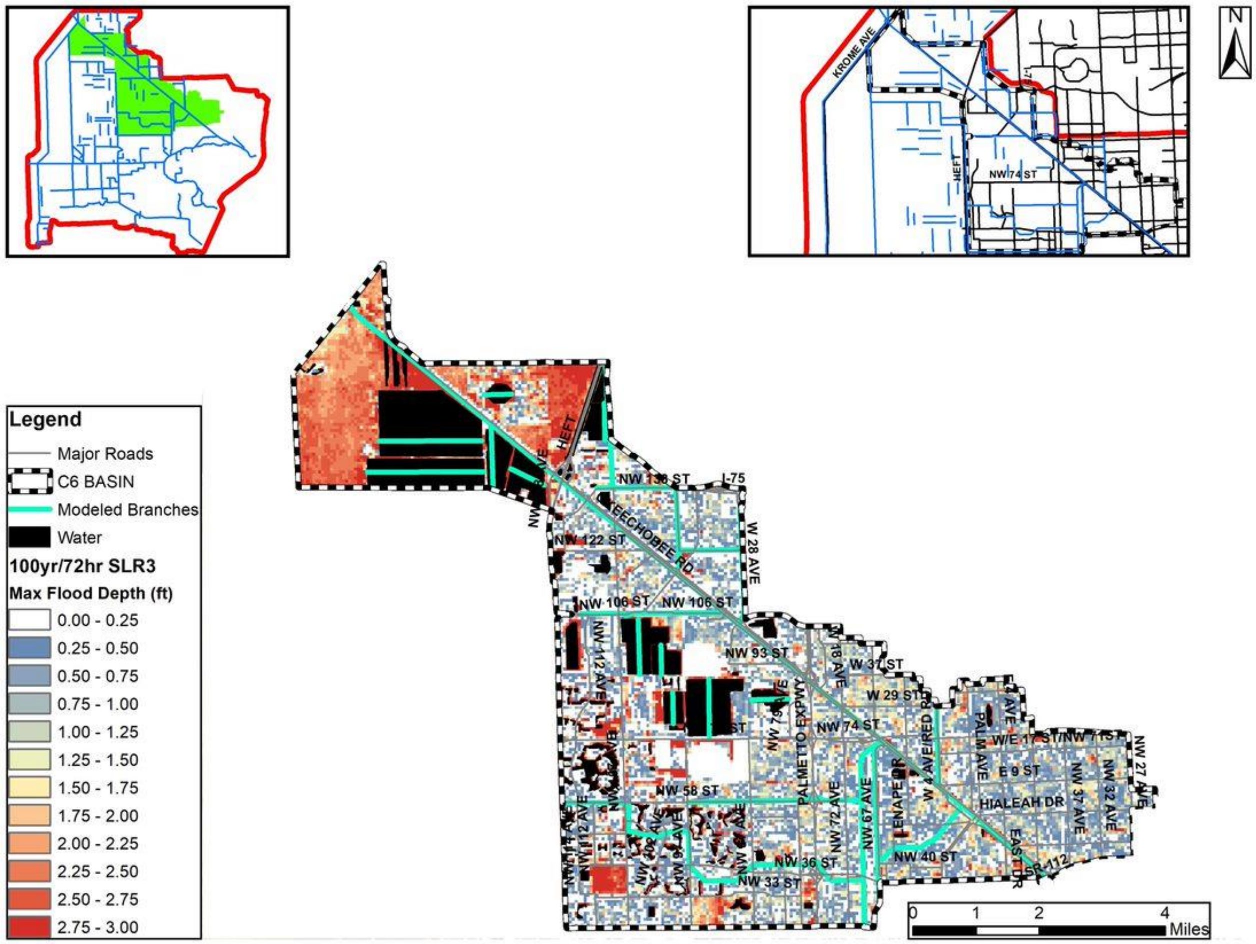


Figure C 5-17. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for Current Conditions in Urban Areas in the C6 Watershed

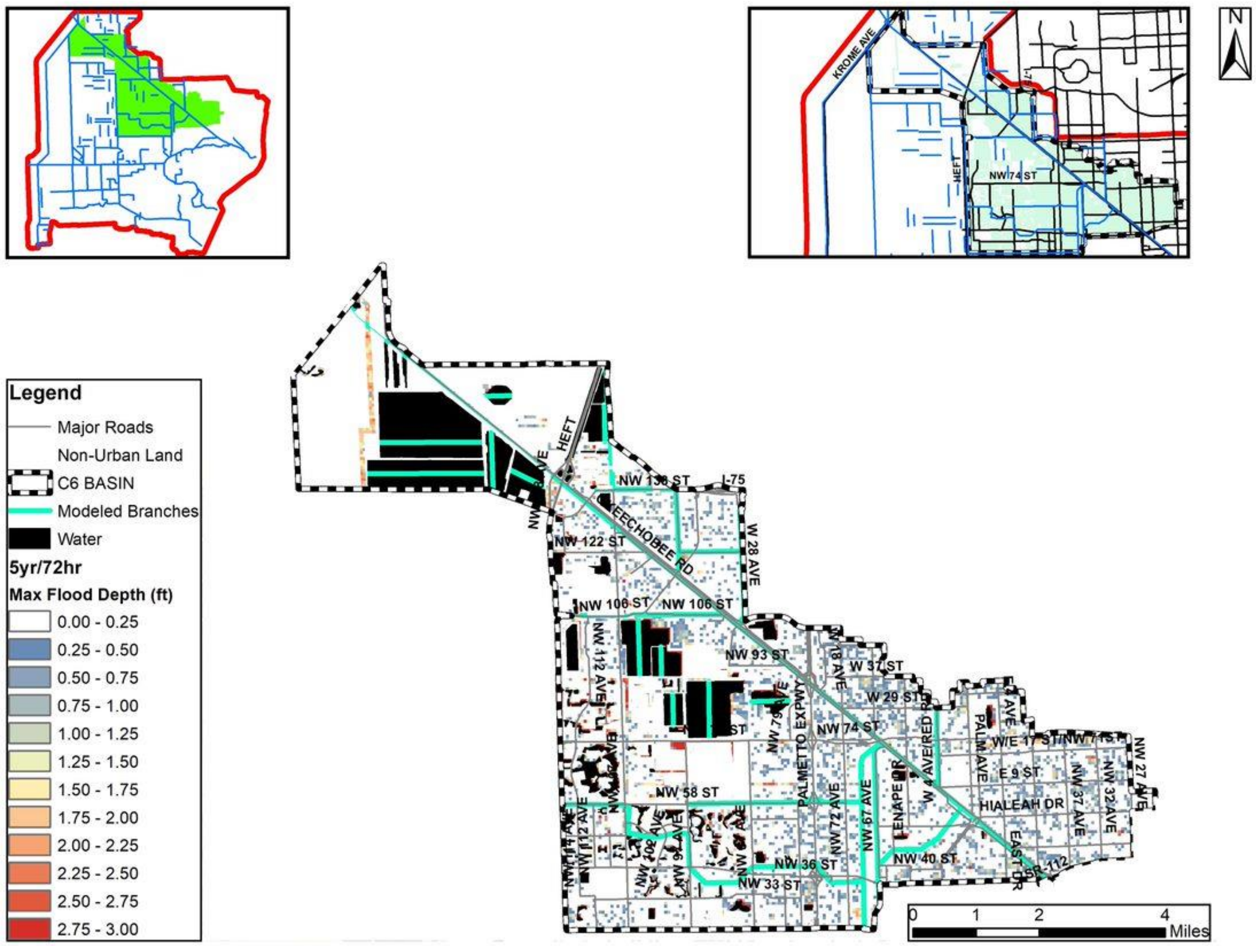


Figure C 5-18. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR1 in Urban Areas in the C6 Watershed

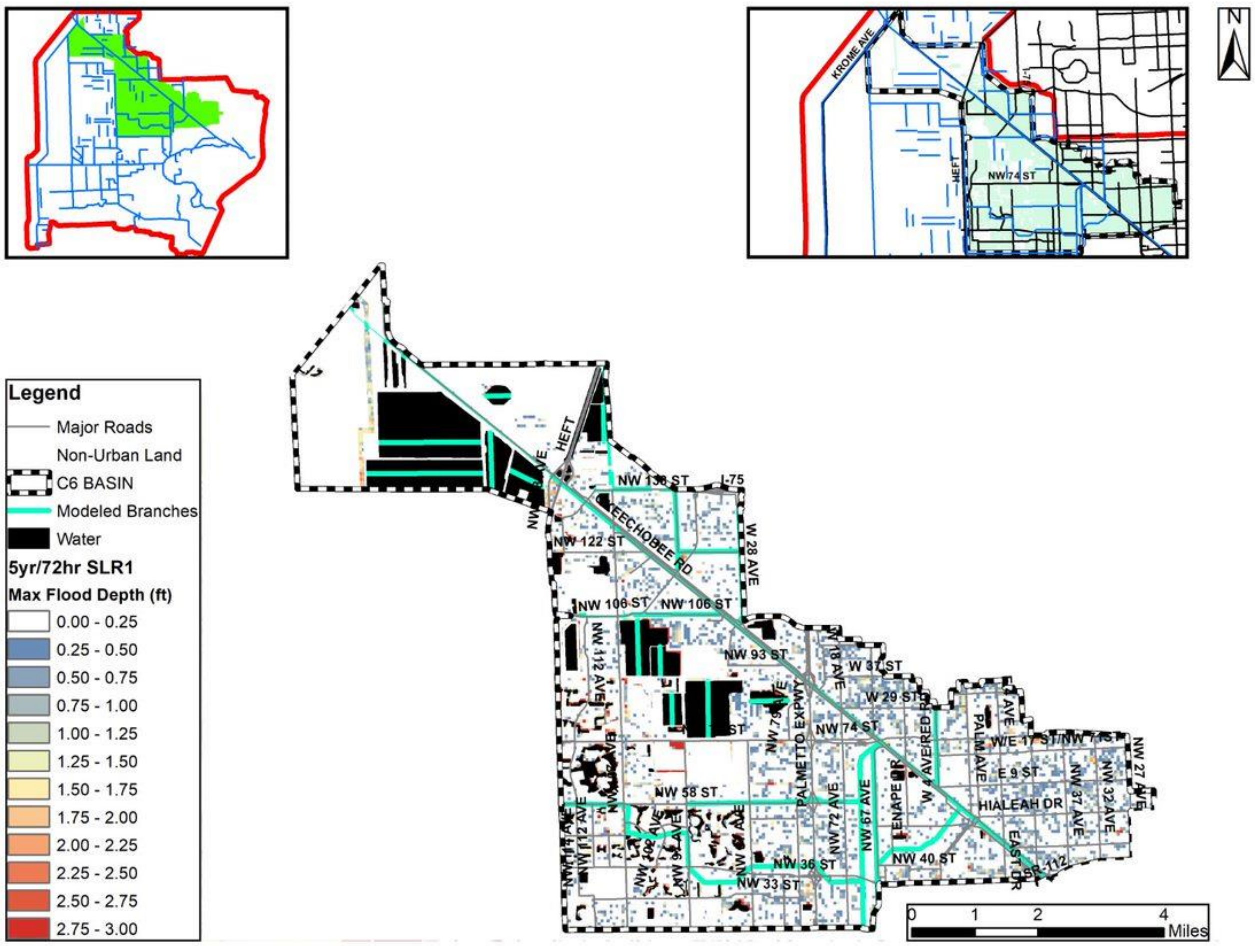


Figure C 5-19. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR2 in Urban Areas in the C6 Watershed

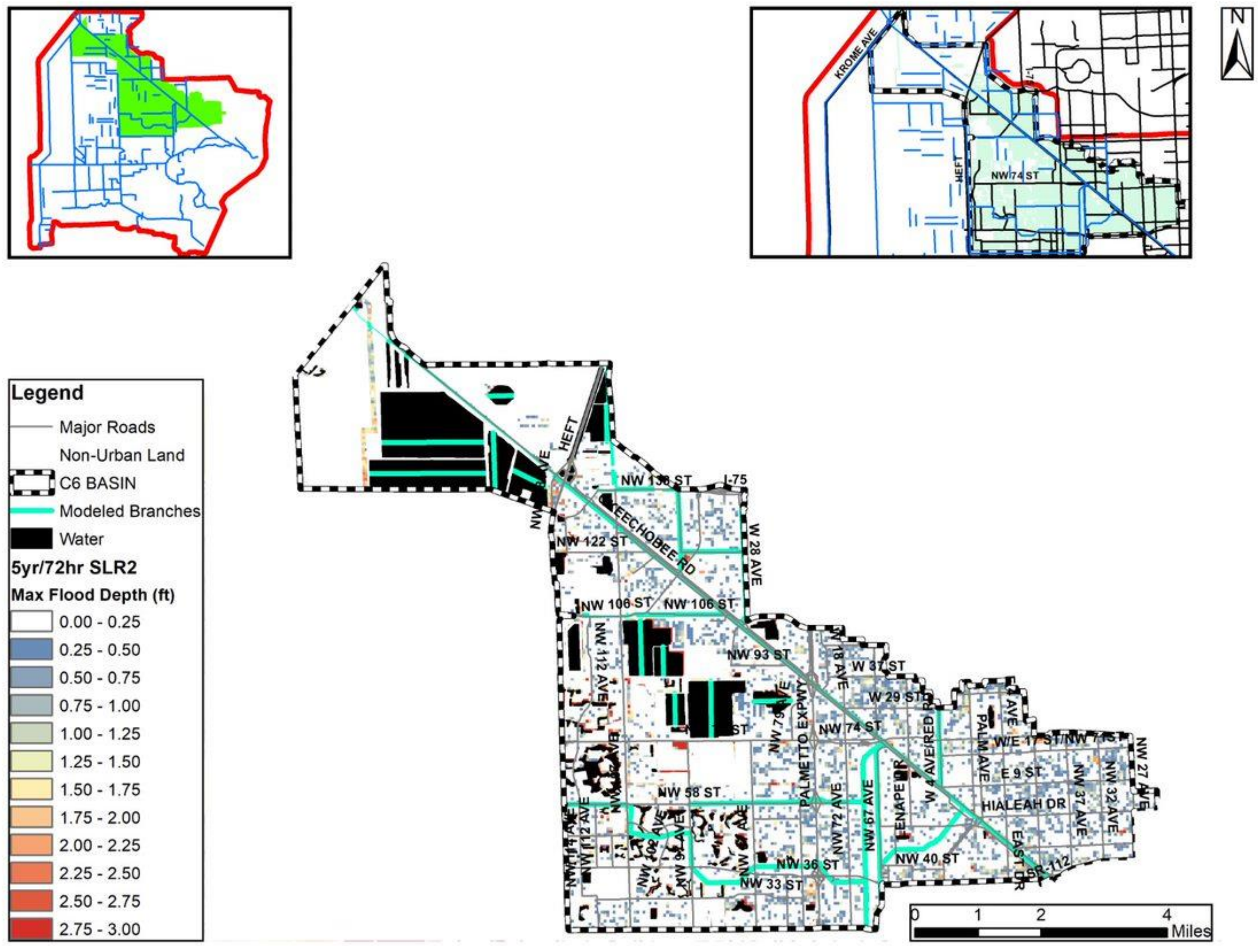


Figure C 5-20. Maximum Overland Flood Depth for the 5-year 3-Day Design Storm for SLR3 in Urban Areas in the C6 Watershed

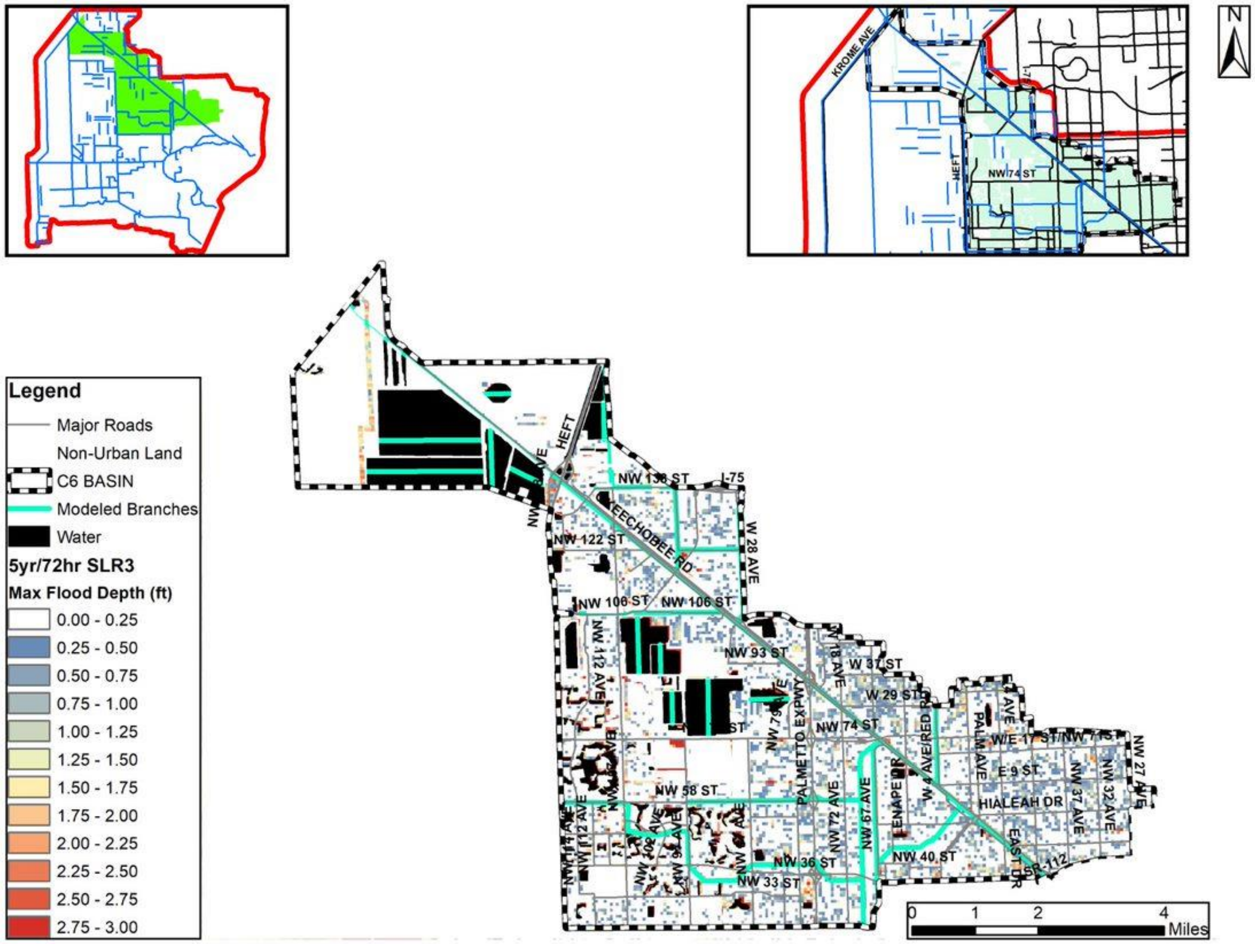


Figure C 5-21. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for Current Conditions in Urban Areas in the C6 Watershed

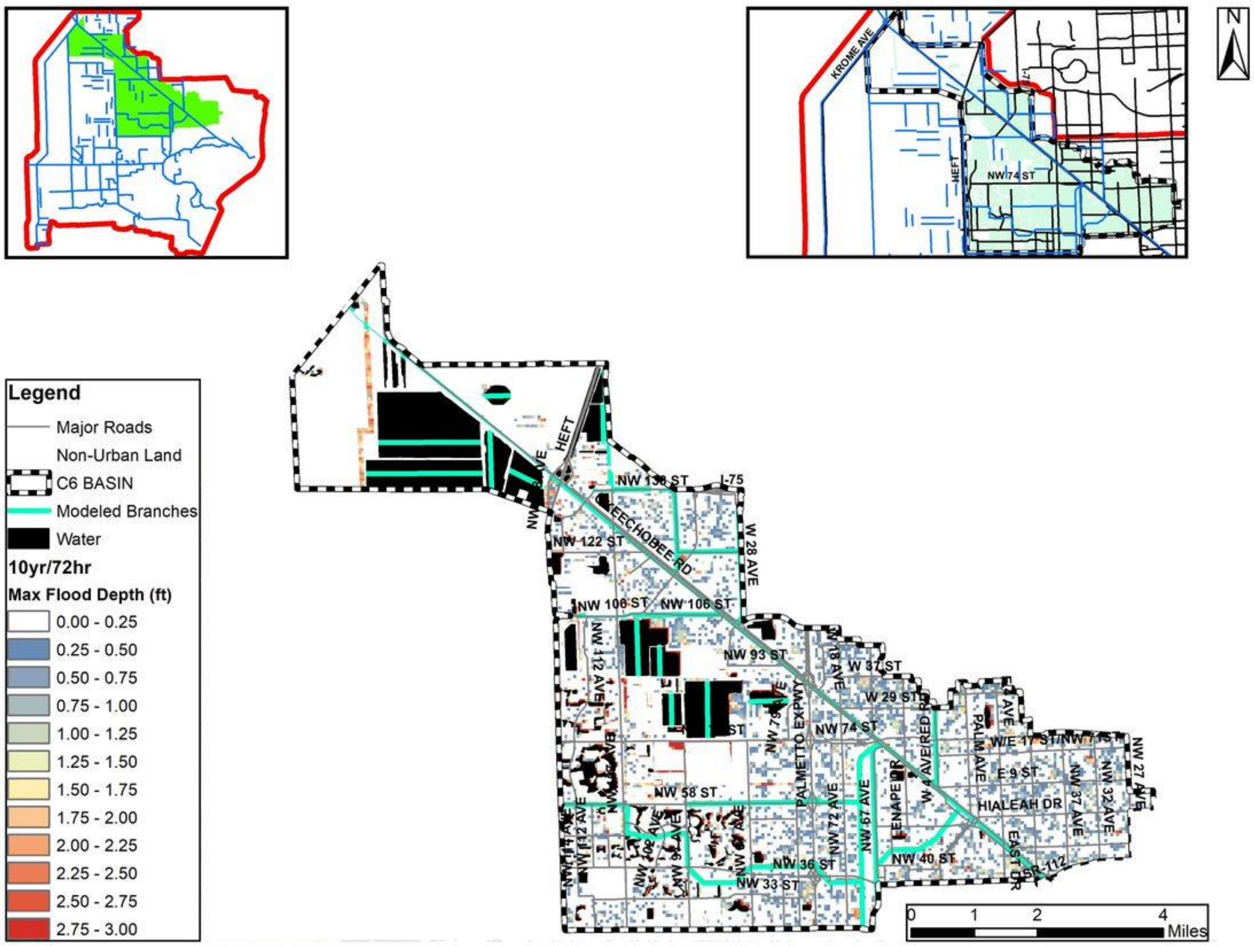


Figure C 5-22. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR1 in Urban Areas in the C6 Watershed

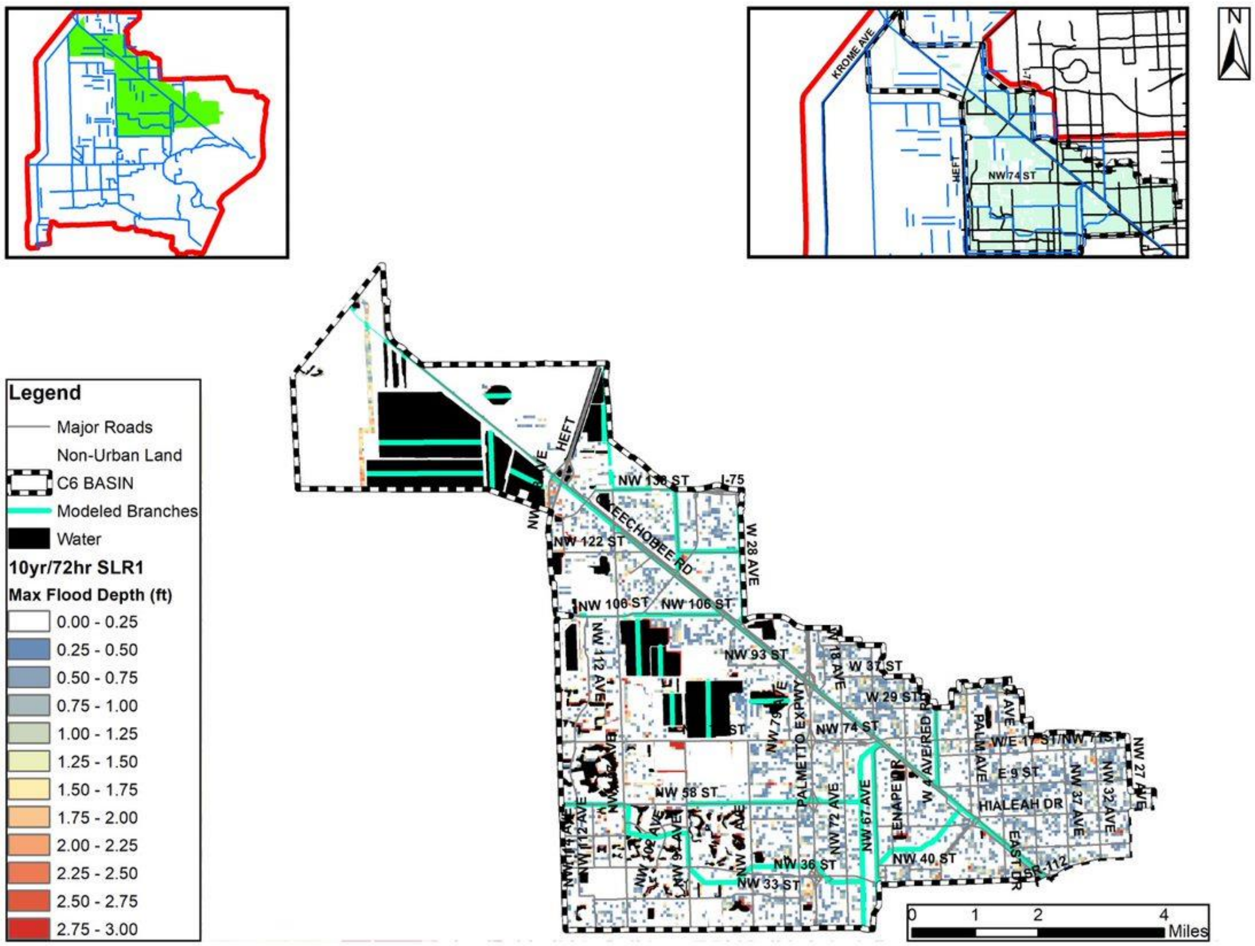


Figure C 5-23. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR2 in Urban Areas in the C6 Watershed

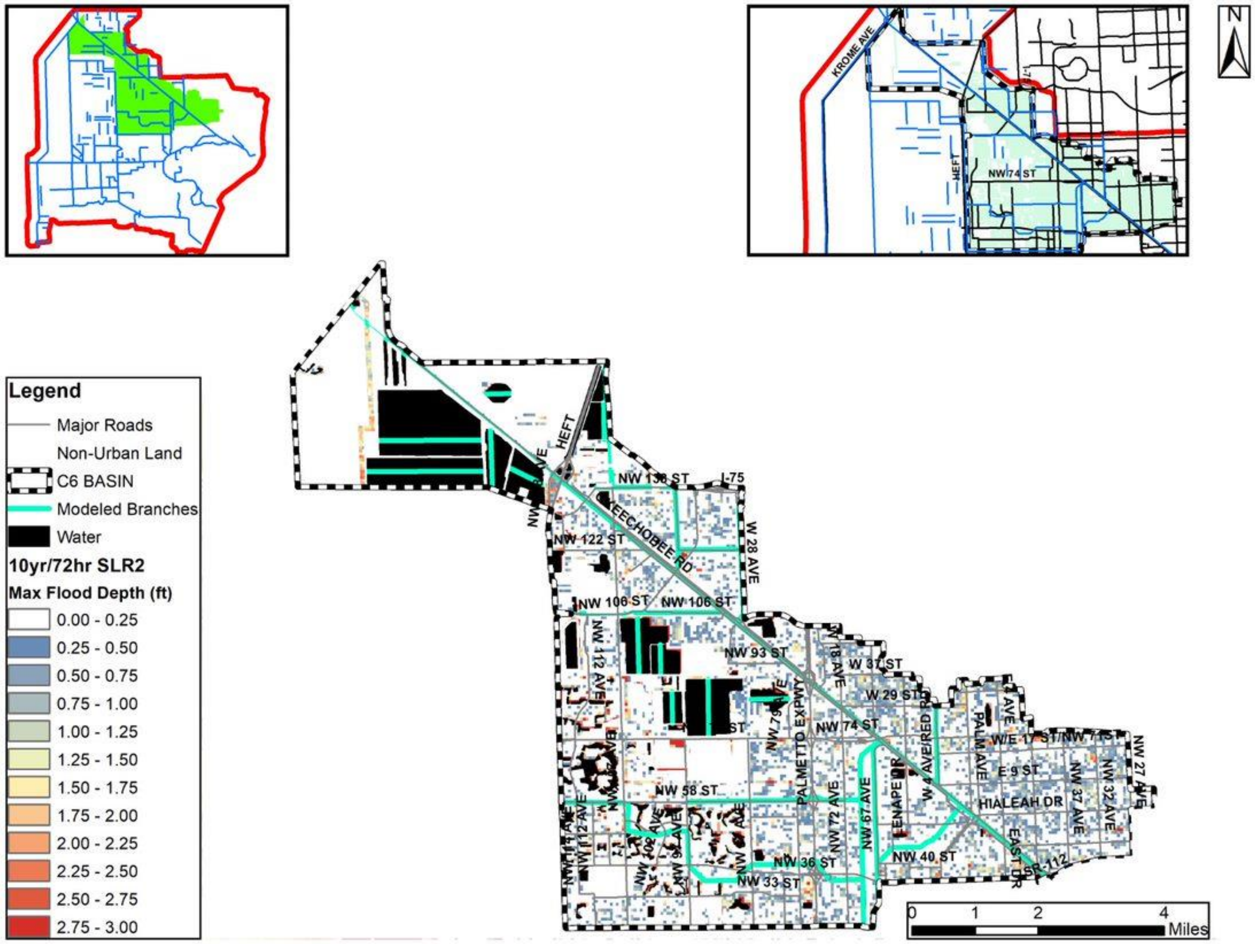


Figure C 5-24. Maximum Overland Flood Depth for the 10-year 3-Day Design Storm for SLR3 in Urban Areas in the C6 Watershed

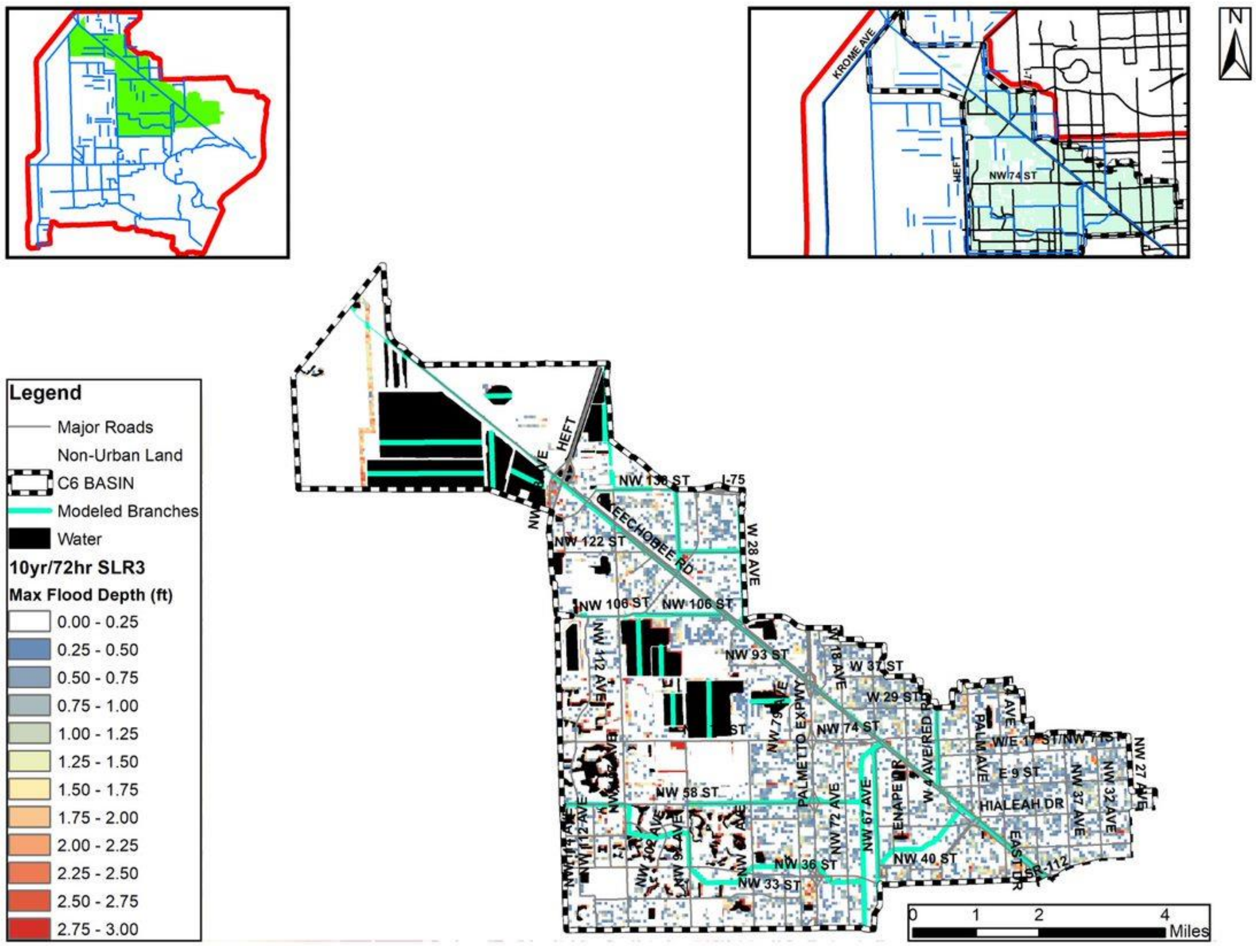


Figure C 5-25. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for Current Conditions in Urban Areas in the C6 Watershed

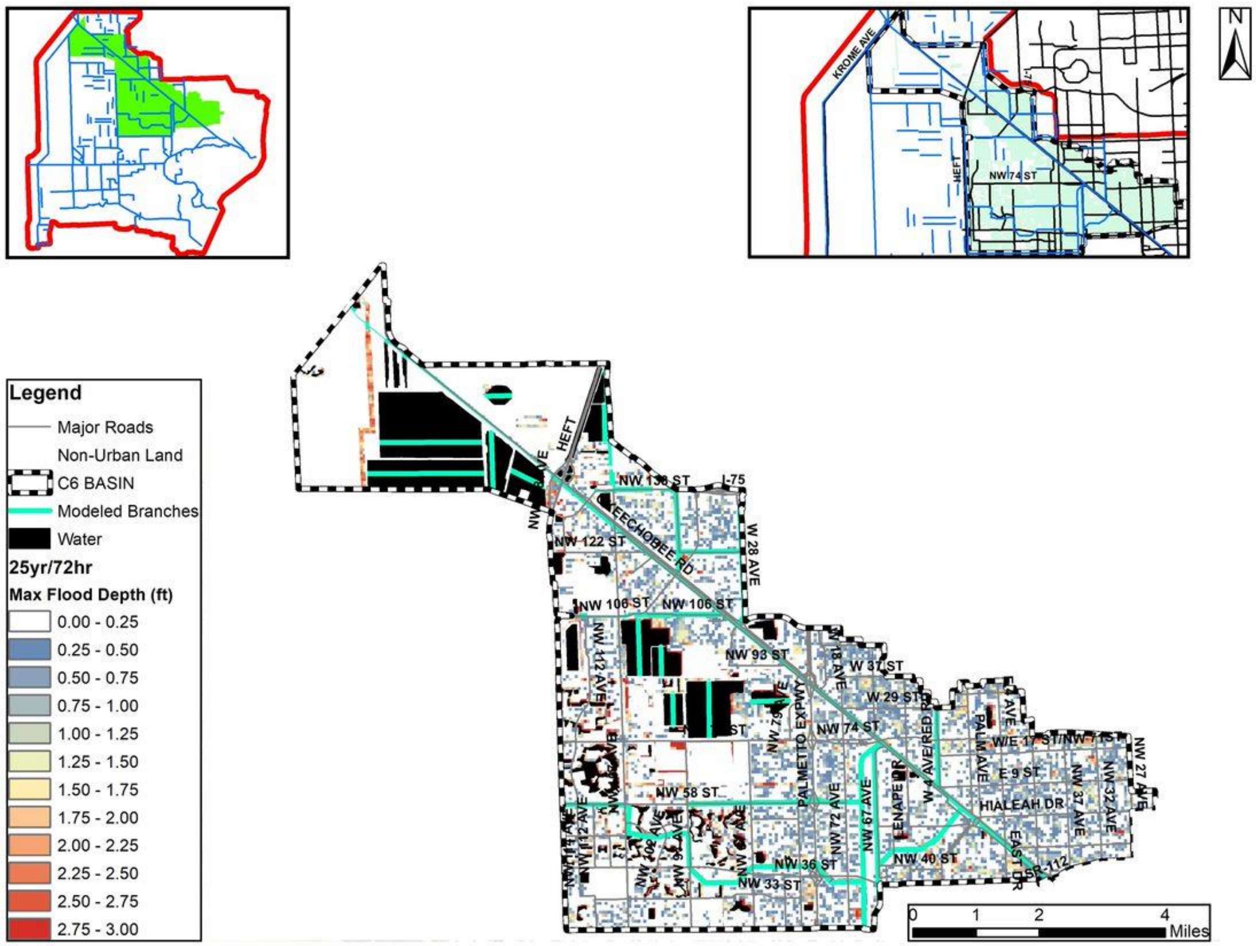


Figure C 5-26. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR1 in Urban Areas in the C6 Watershed

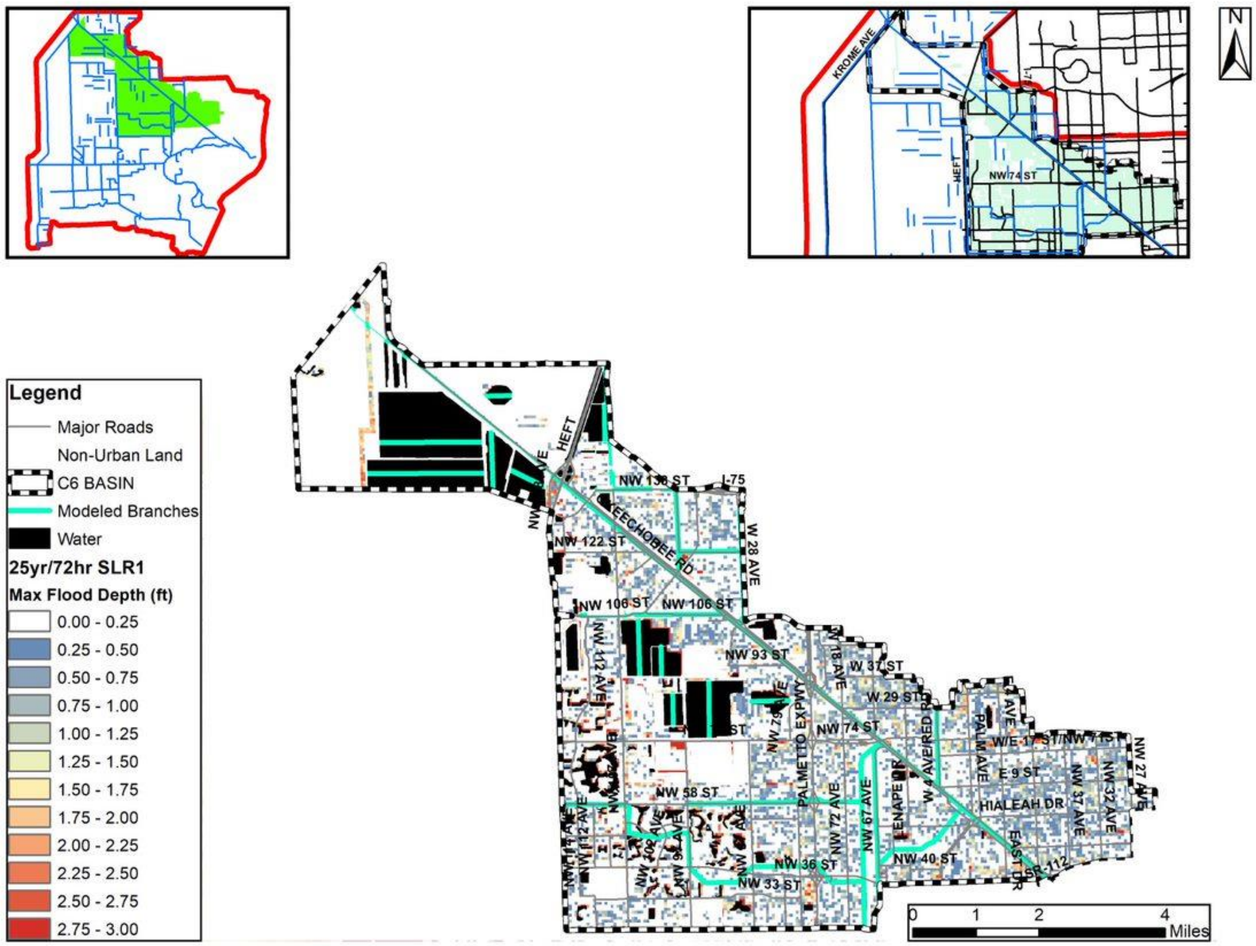


Figure C 5-27. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR2 in Urban Areas in the C6 Watershed

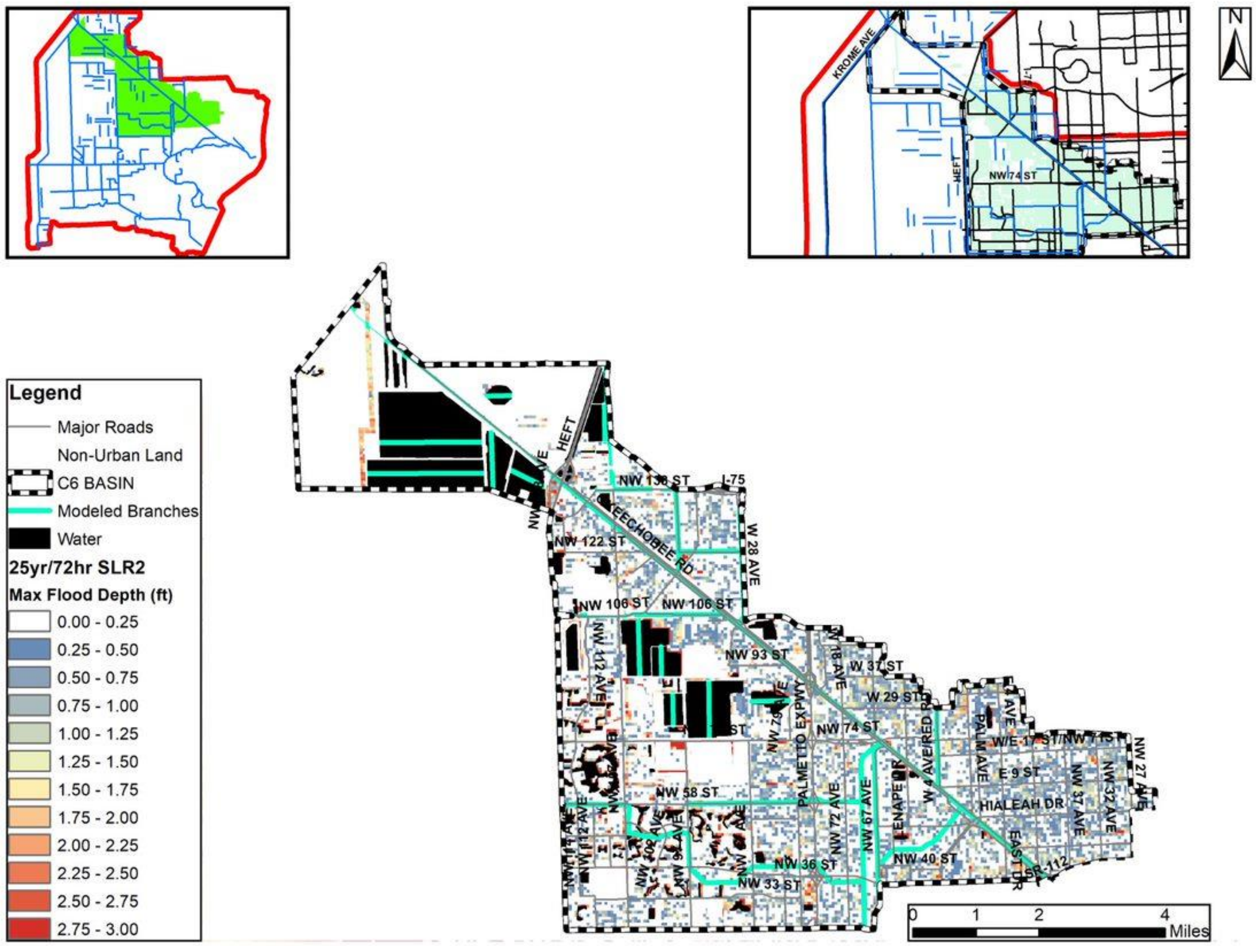


Figure C 5-28. Maximum Overland Flood Depth for the 25-year 3-Day Design Storm for SLR3 in Urban Areas in the C6 Watershed

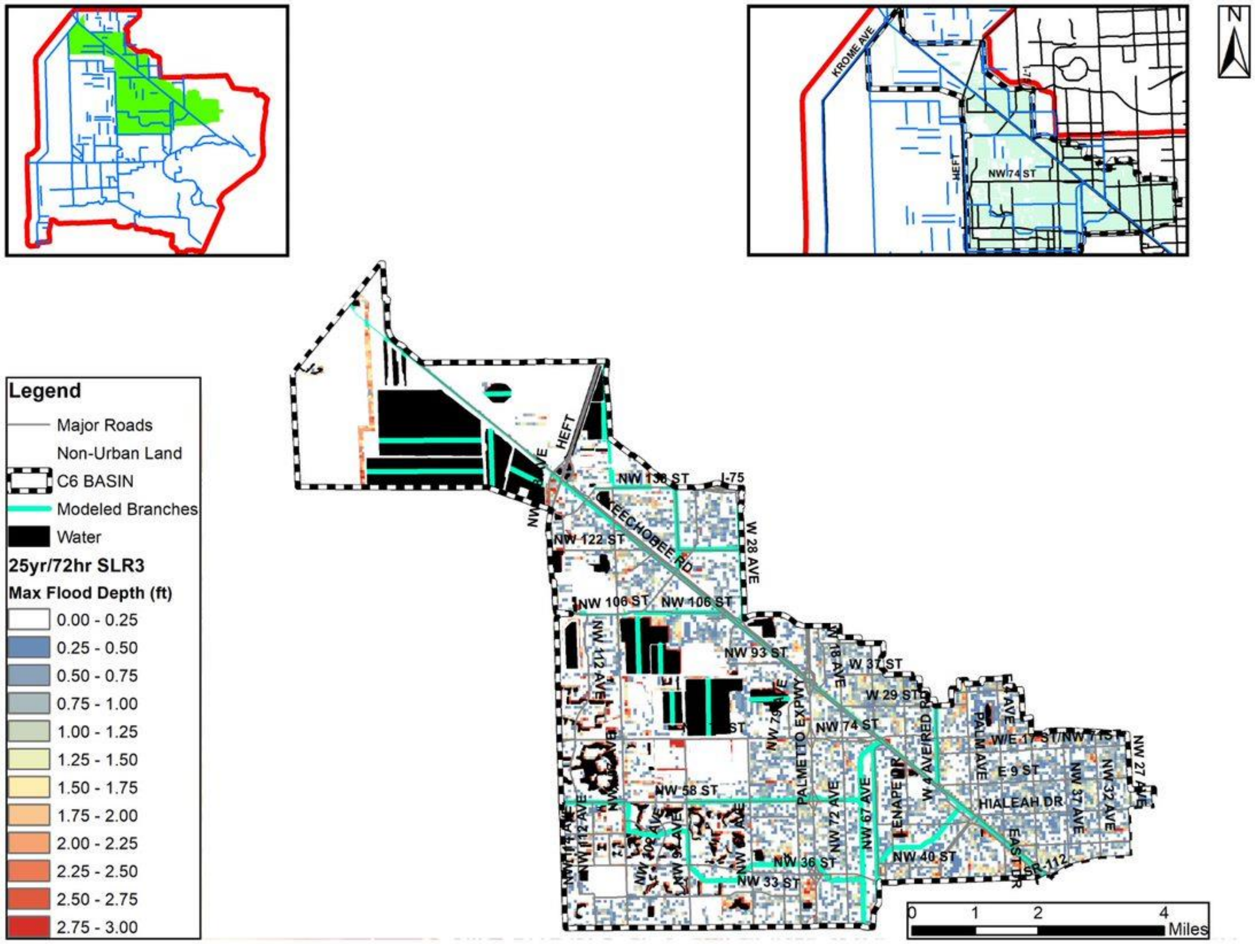


Figure C 5-29. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for Current Conditions in Urban Areas in the C6 Watershed

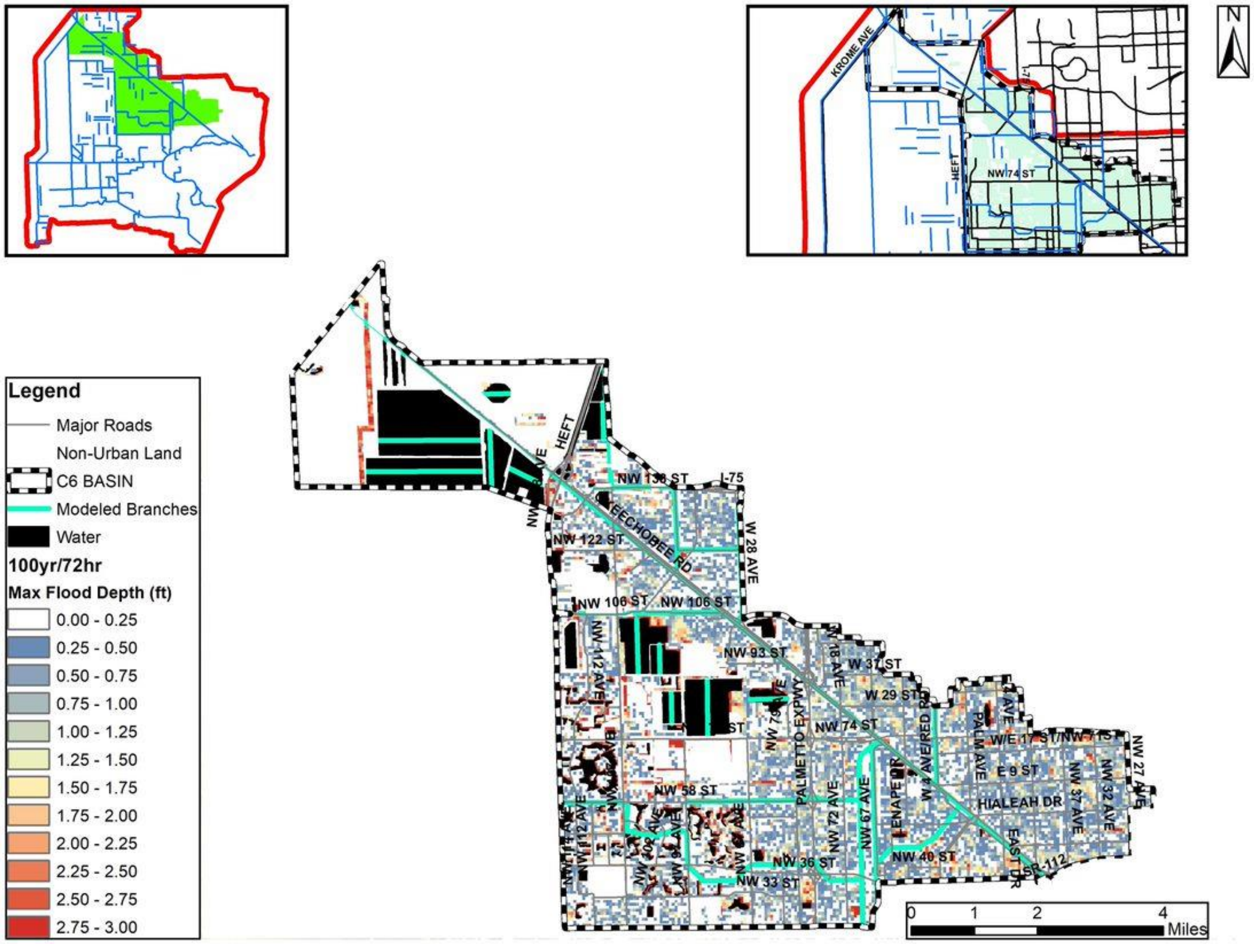


Figure C 5-30. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR1 in Urban Areas in the C6 Watershed

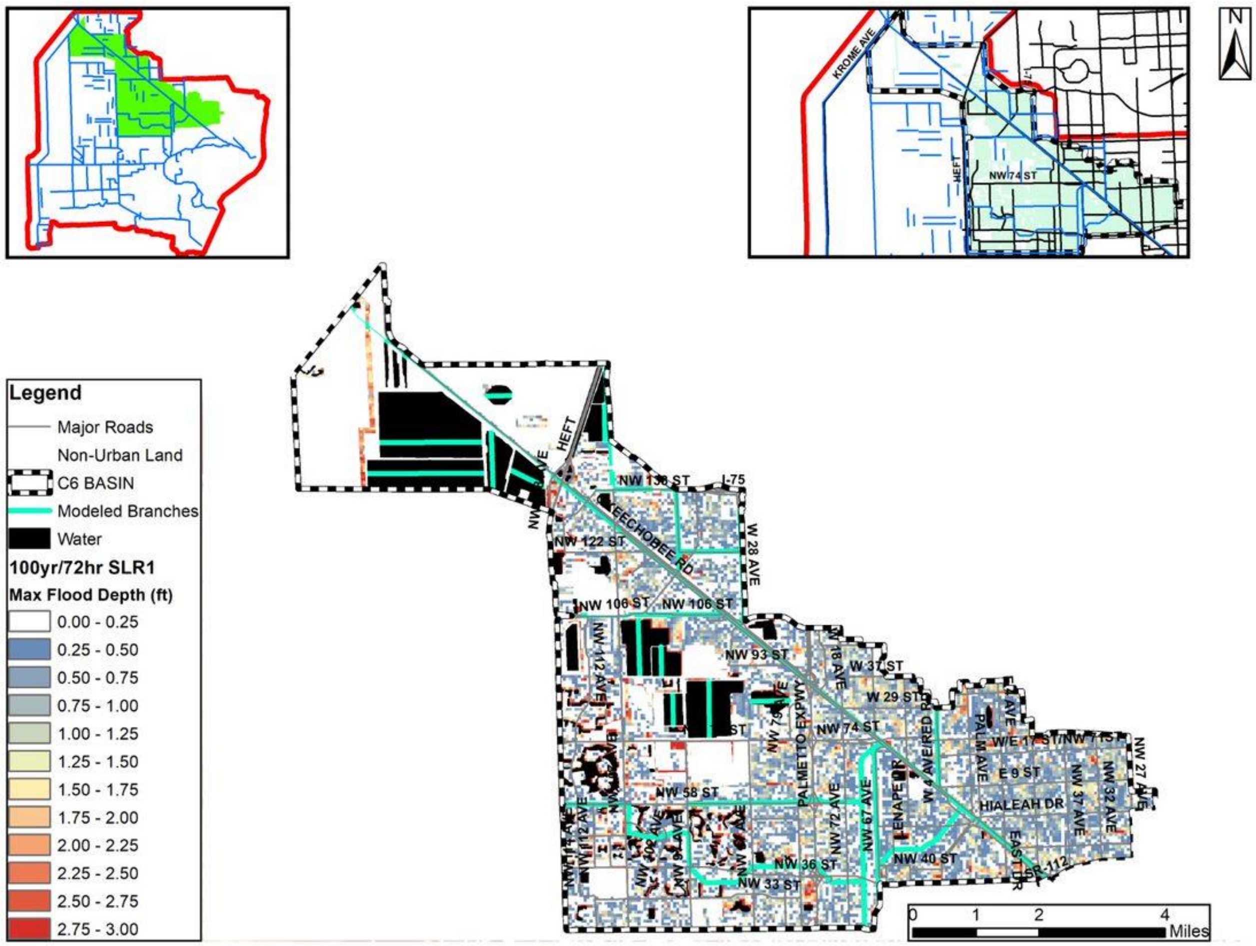


Figure C 5-31. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR2 in Urban Areas in the C6 Watershed

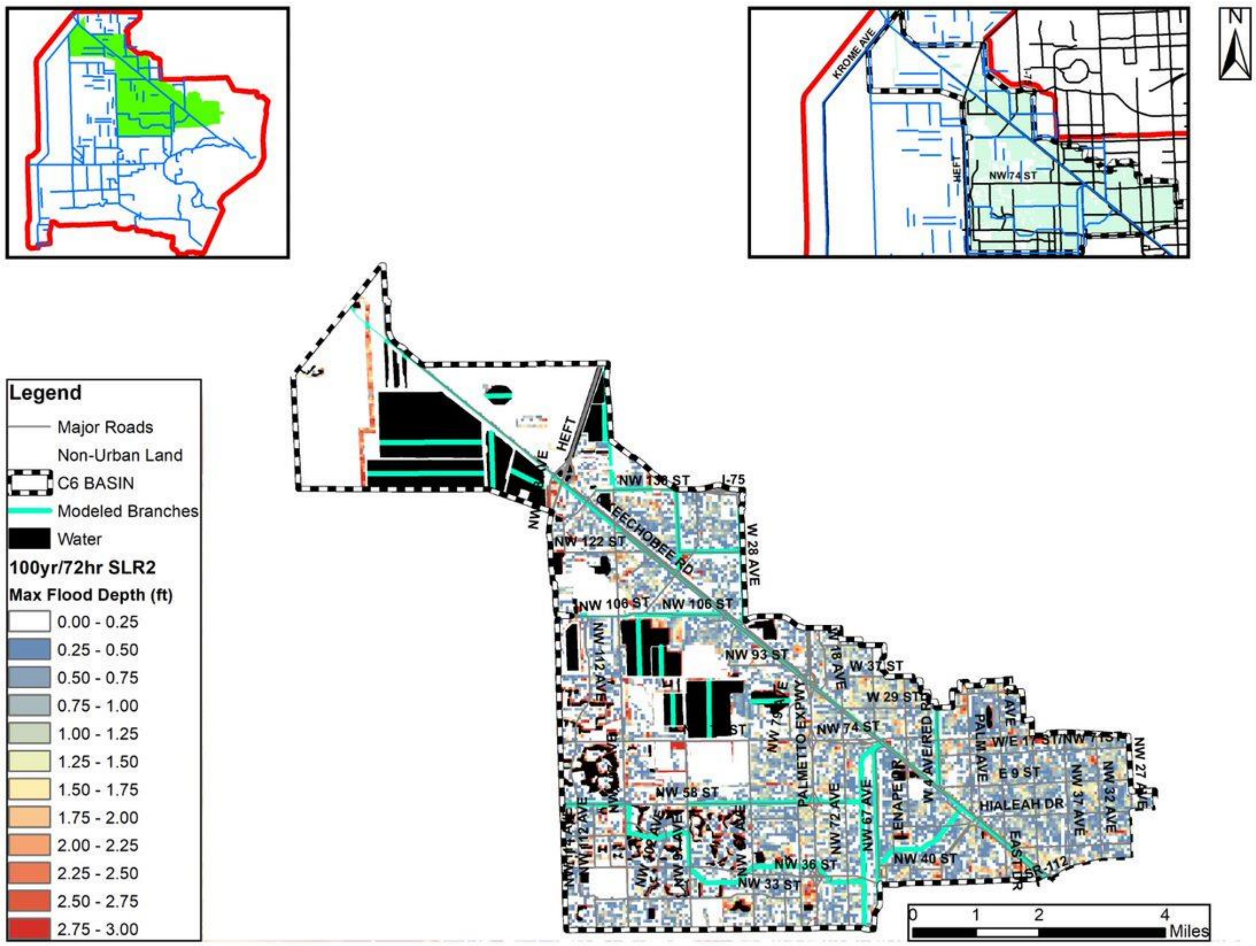


Figure C 5-32. Maximum Overland Flood Depth for the 100-year 3-Day Design Storm for SLR3 in Urban Areas in the C6 Watershed

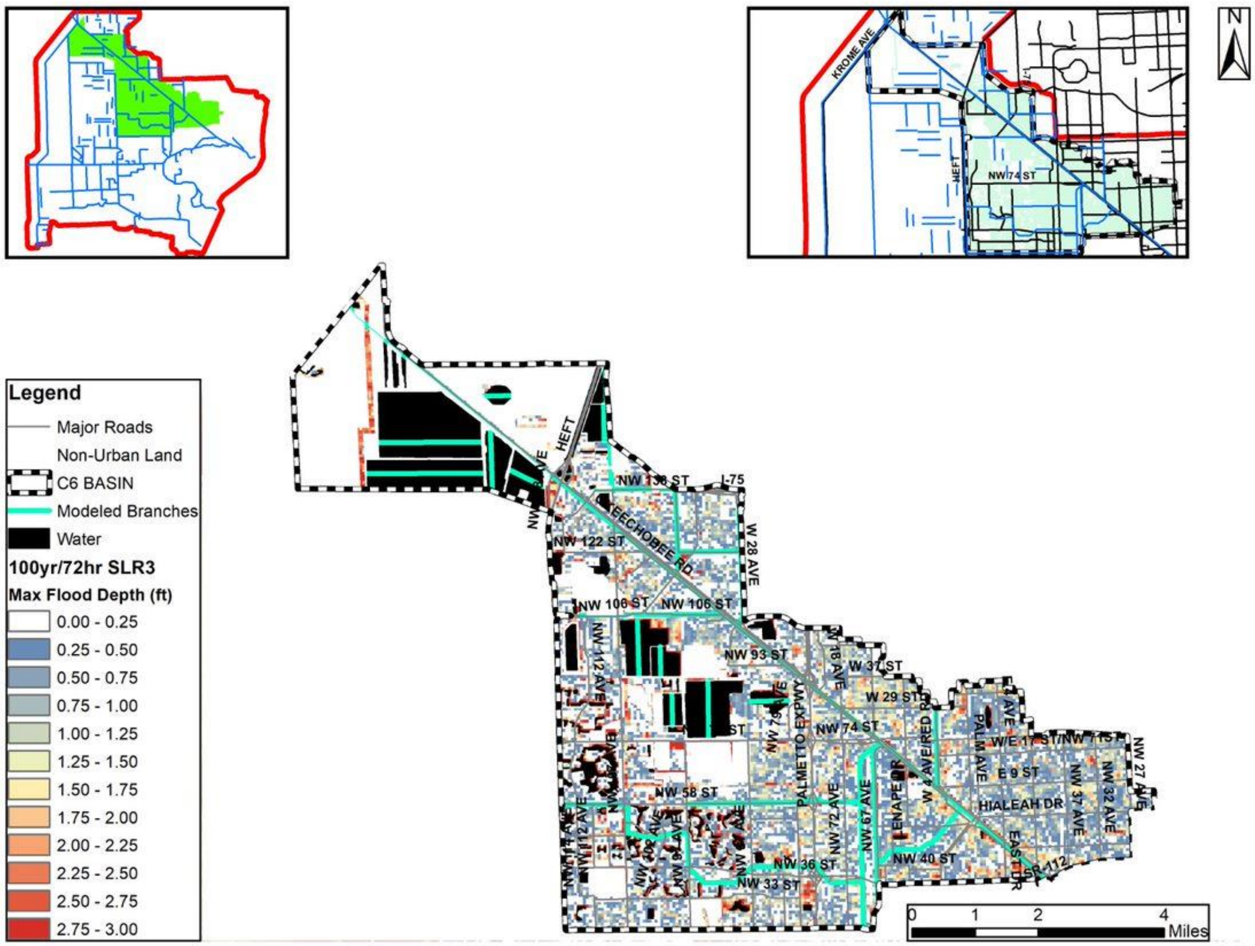


Figure C 5-33. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C6 Watershed

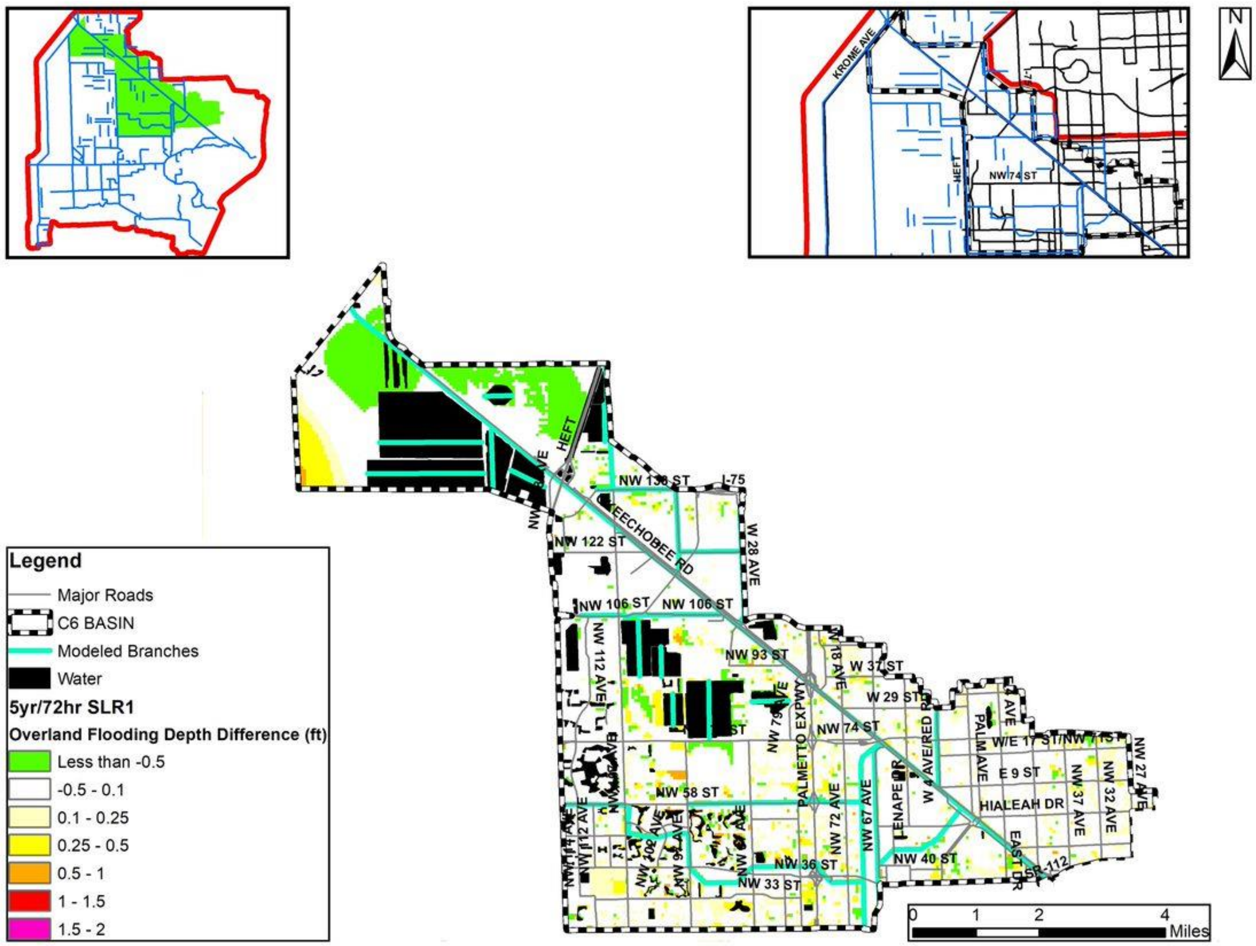


Figure C 5-34. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C6 Watershed

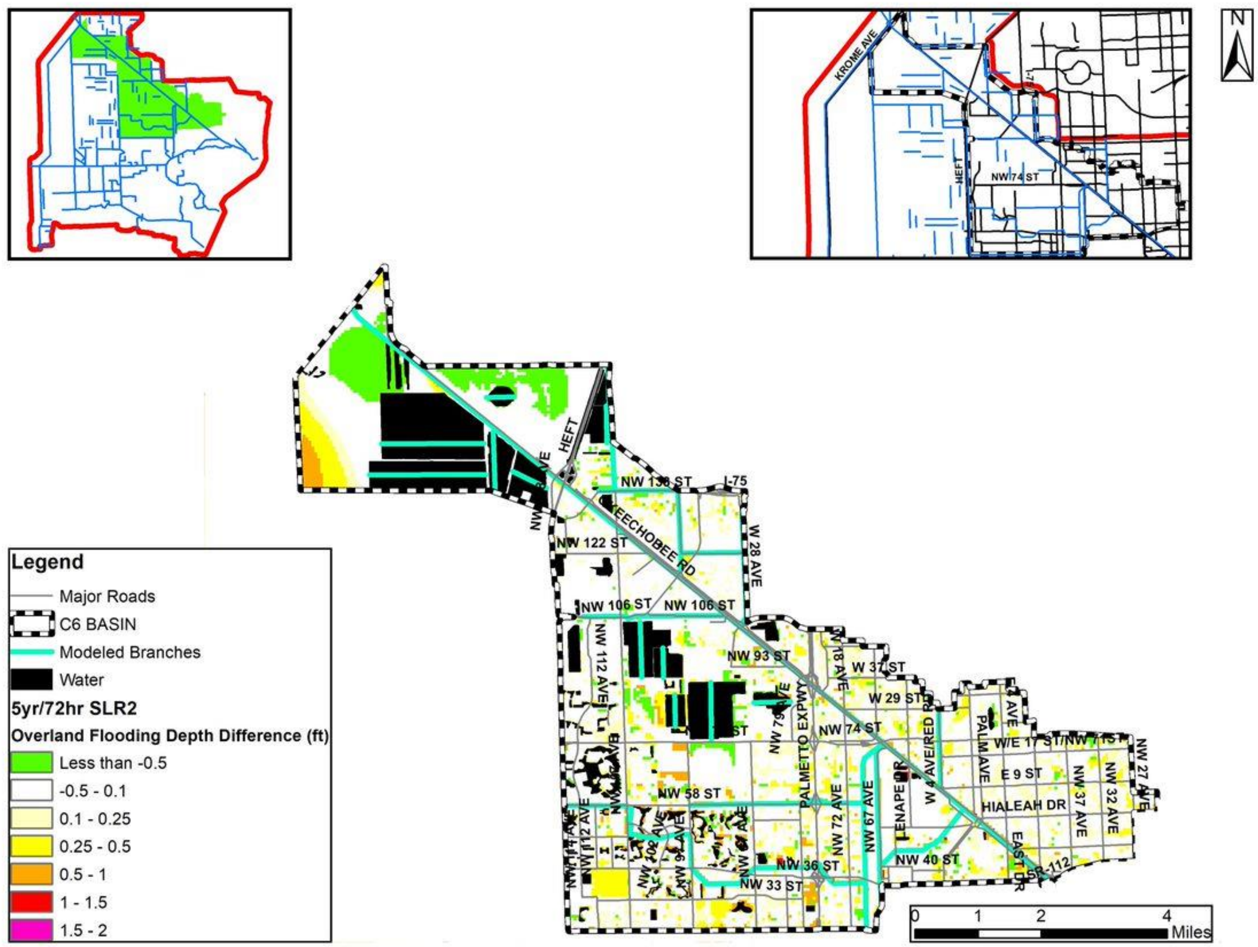


Figure C 5-35. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C6 Watershed

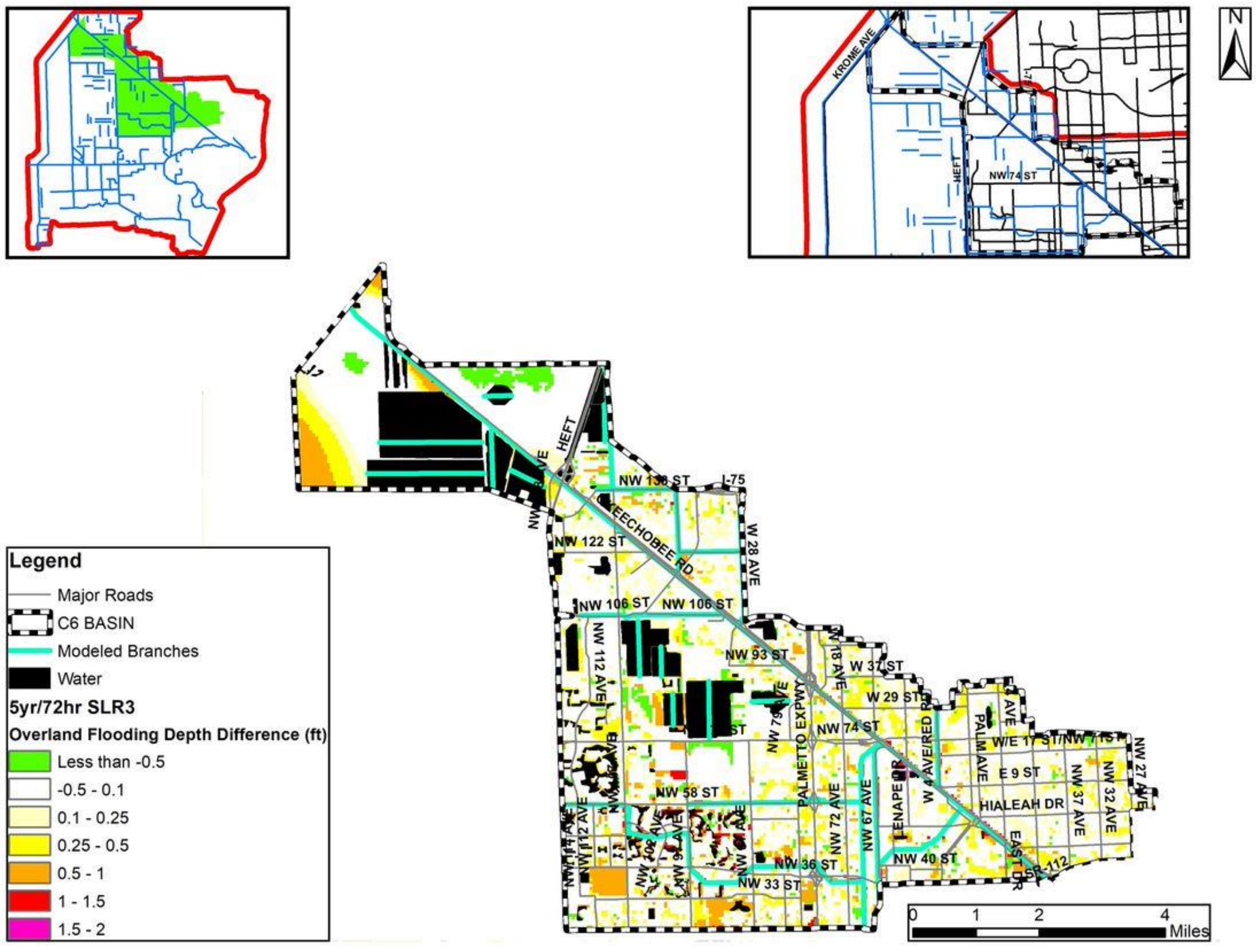


Figure C 5-36. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C6 Watershed

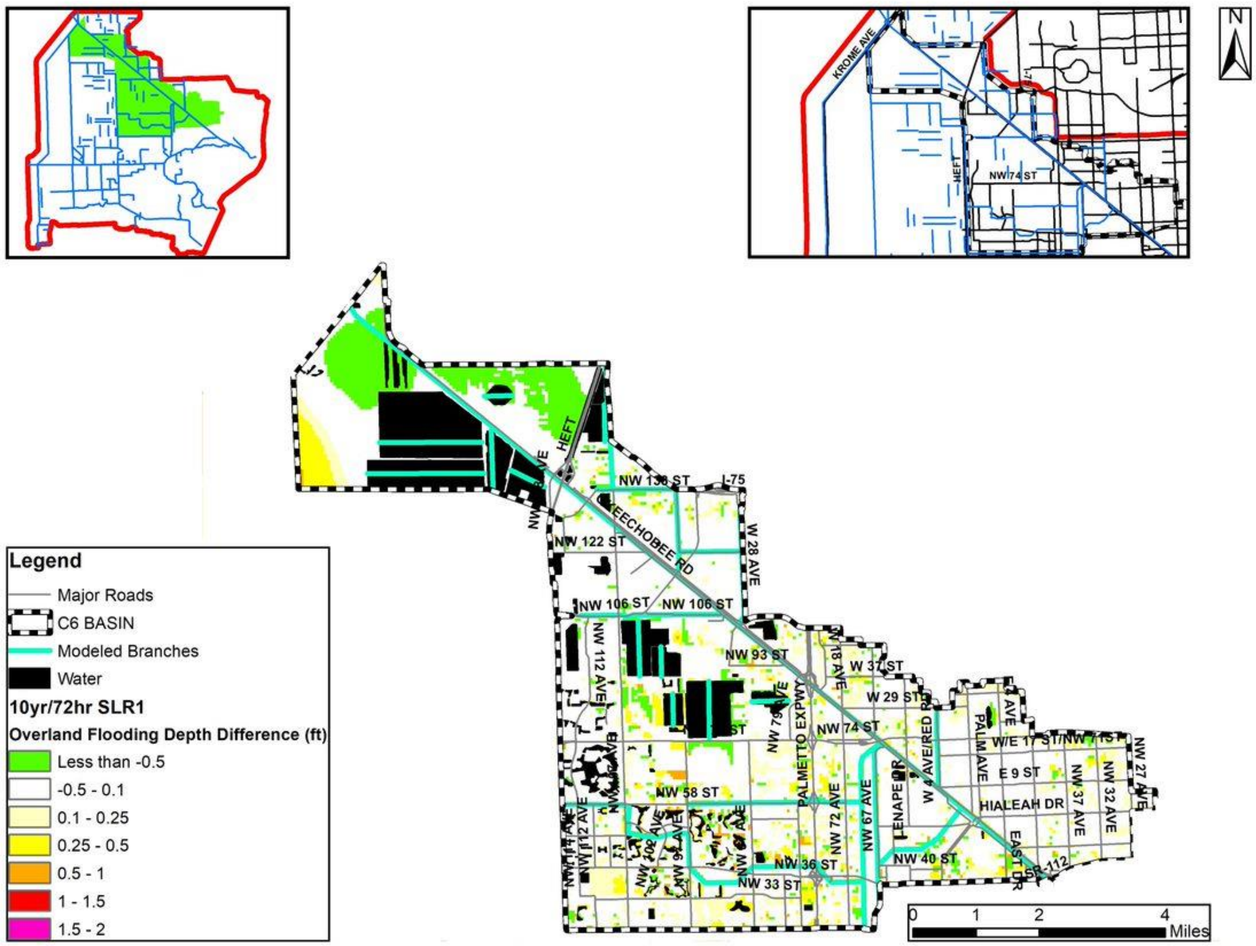


Figure C 5-37. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C6 Watershed

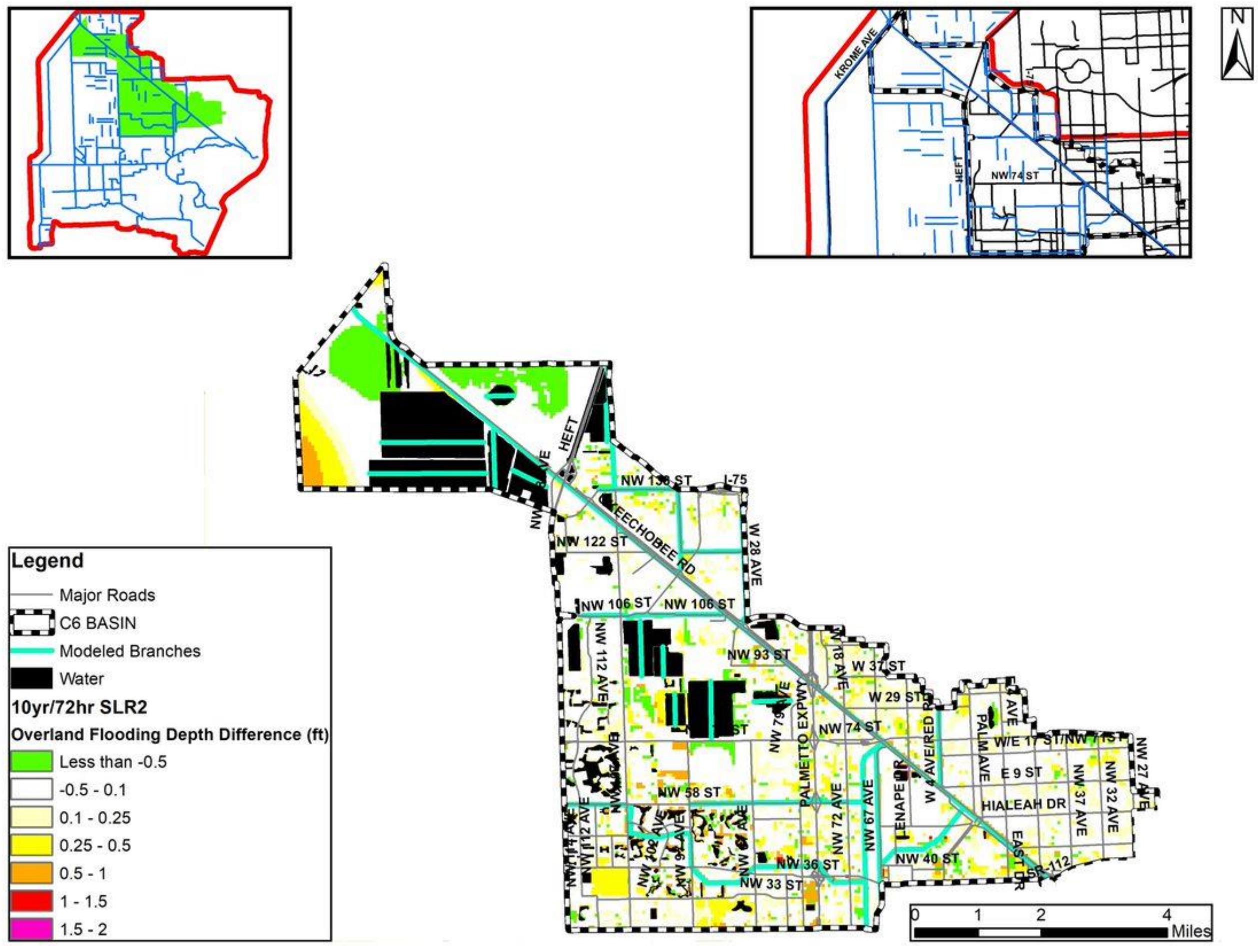


Figure C 5-38. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C6 Watershed

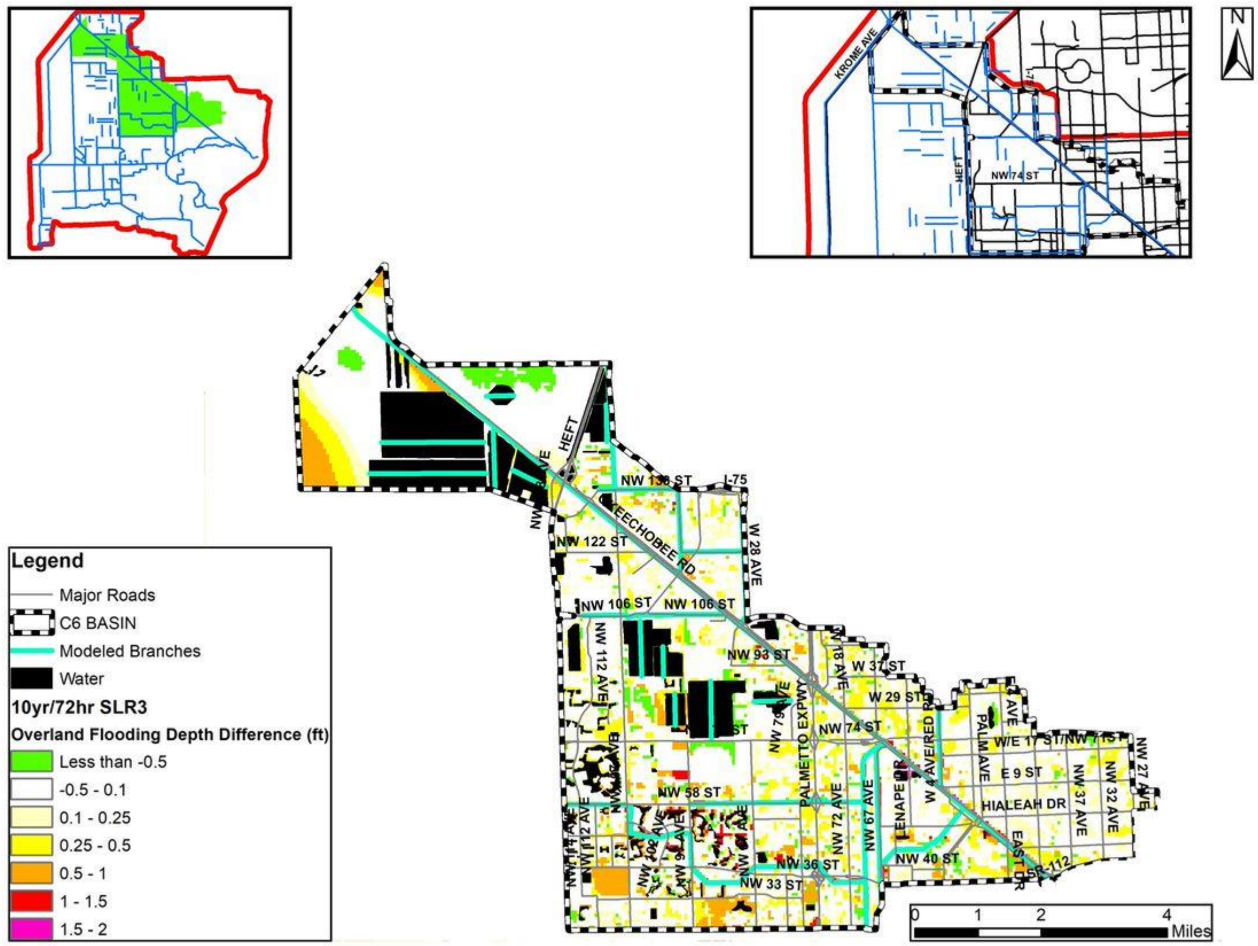


Figure C 5-39. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C6 Watershed

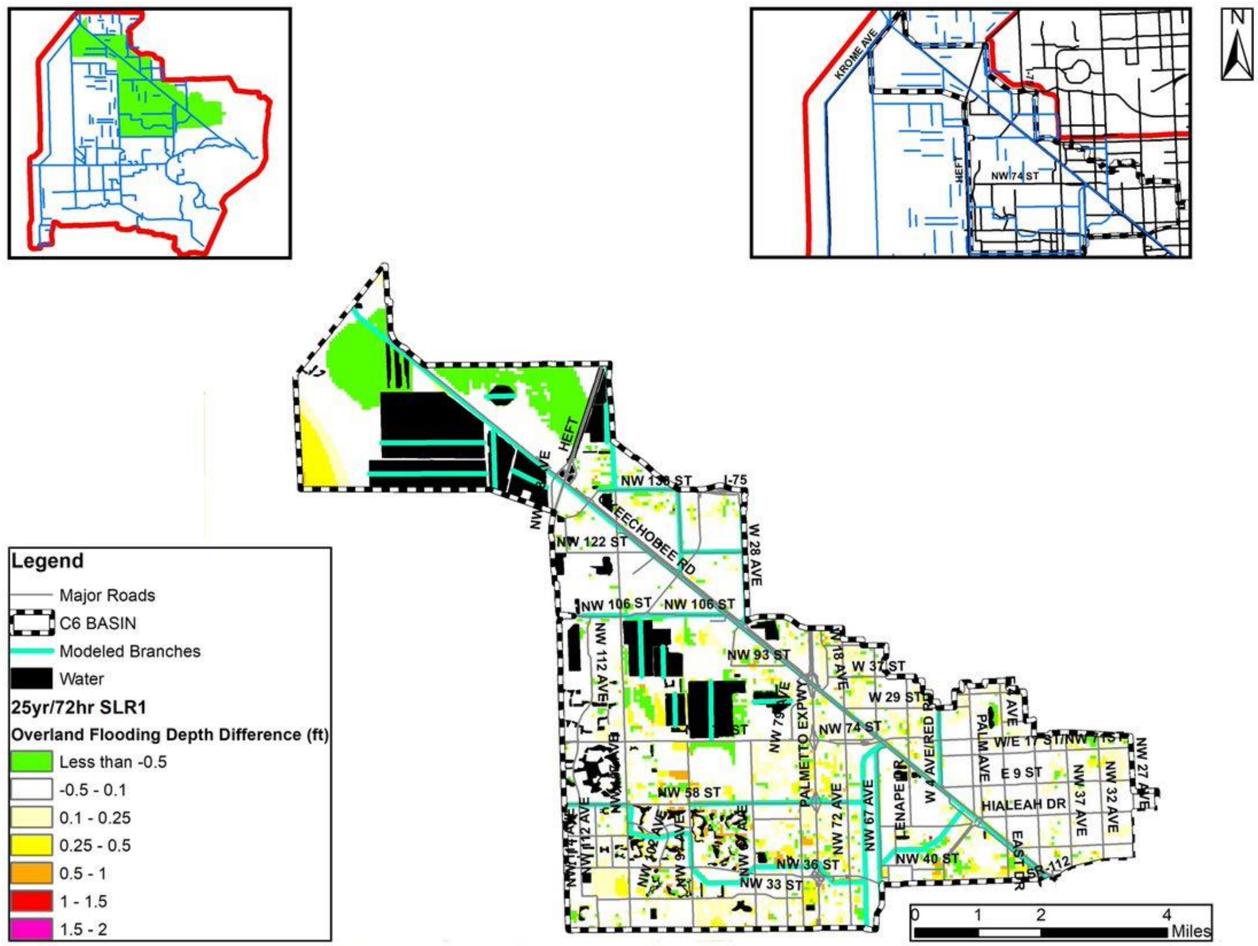


Figure C 5-40. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C6 Watershed

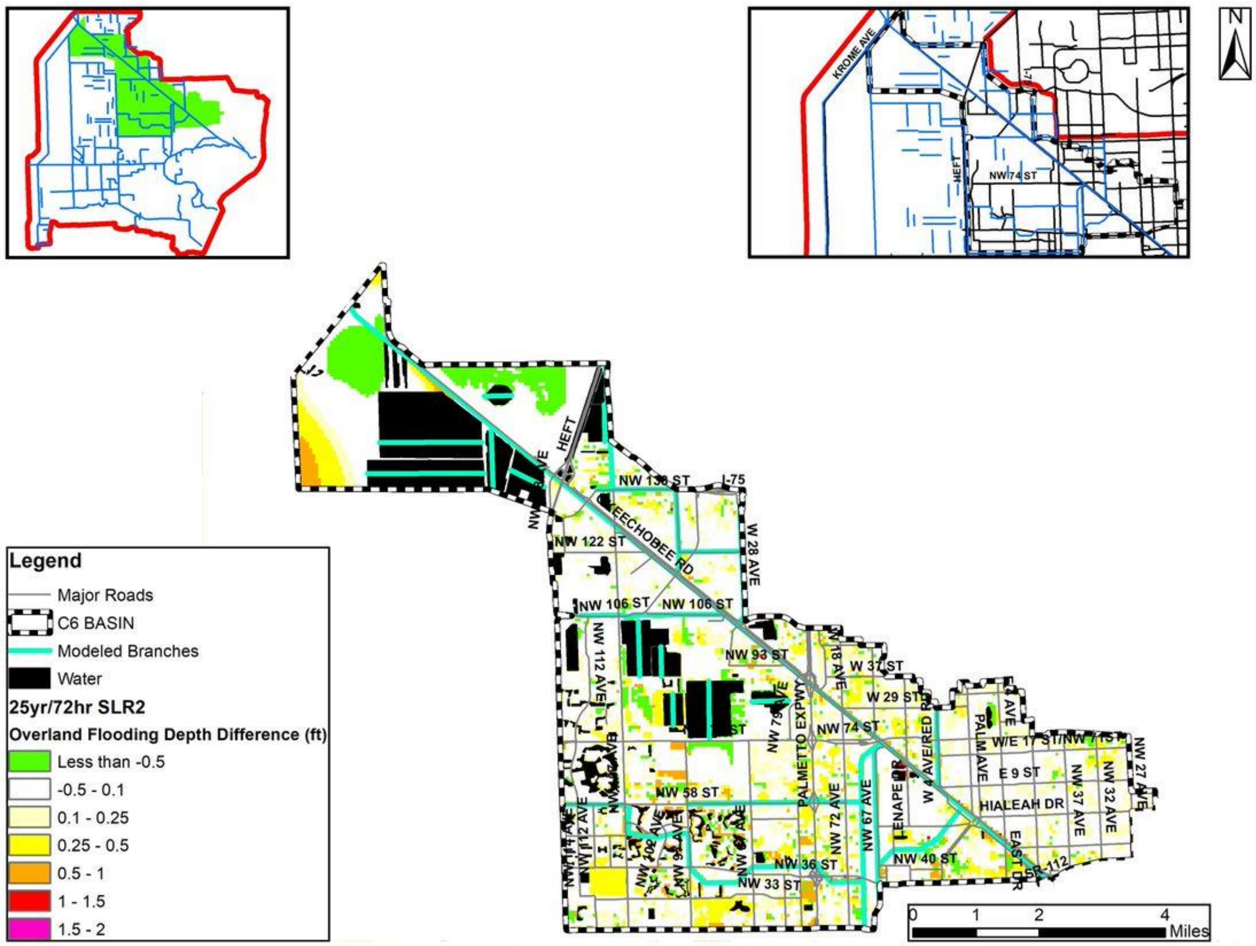


Figure C 5-41. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C6 Watershed

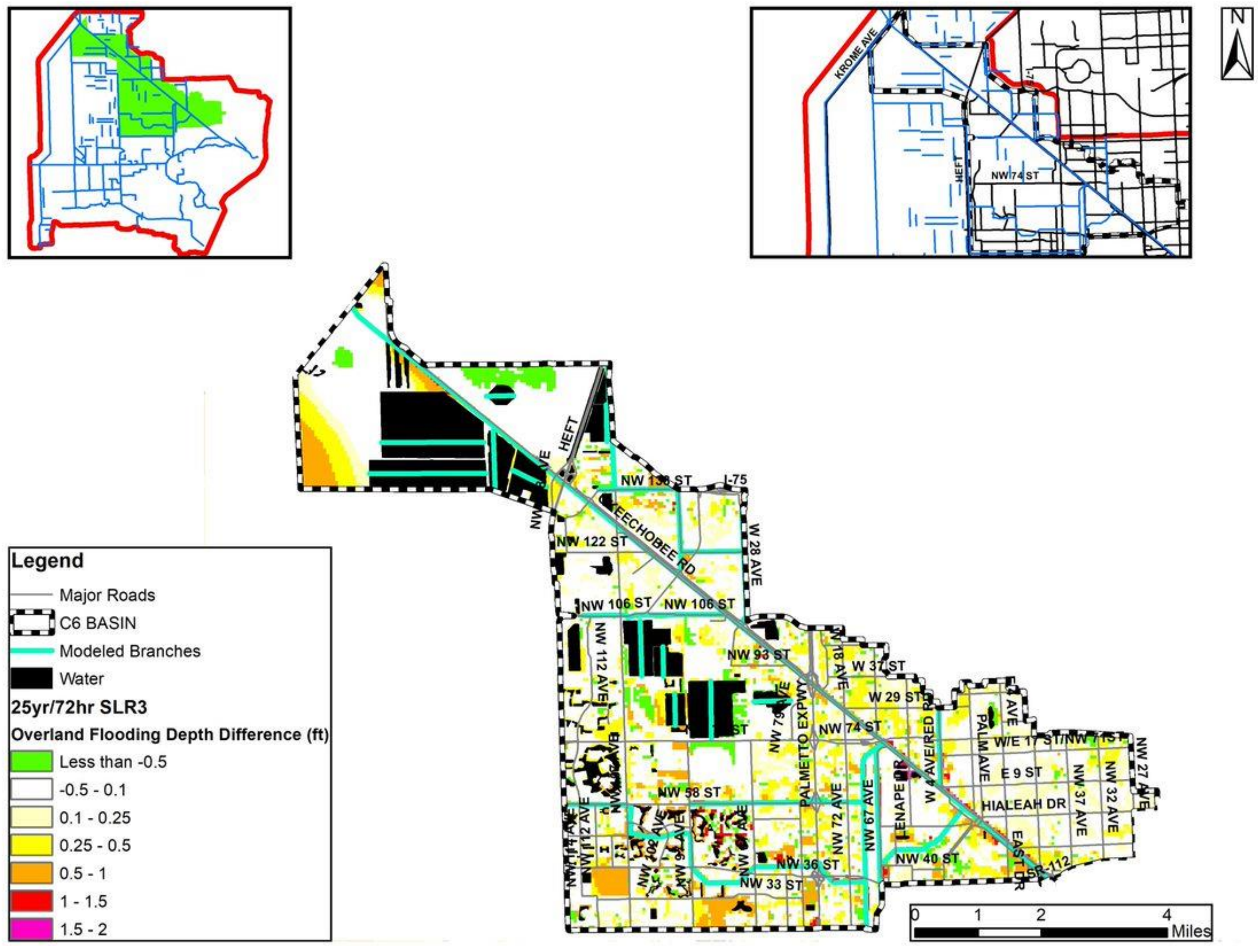


Figure C 5-42. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C6 Watershed

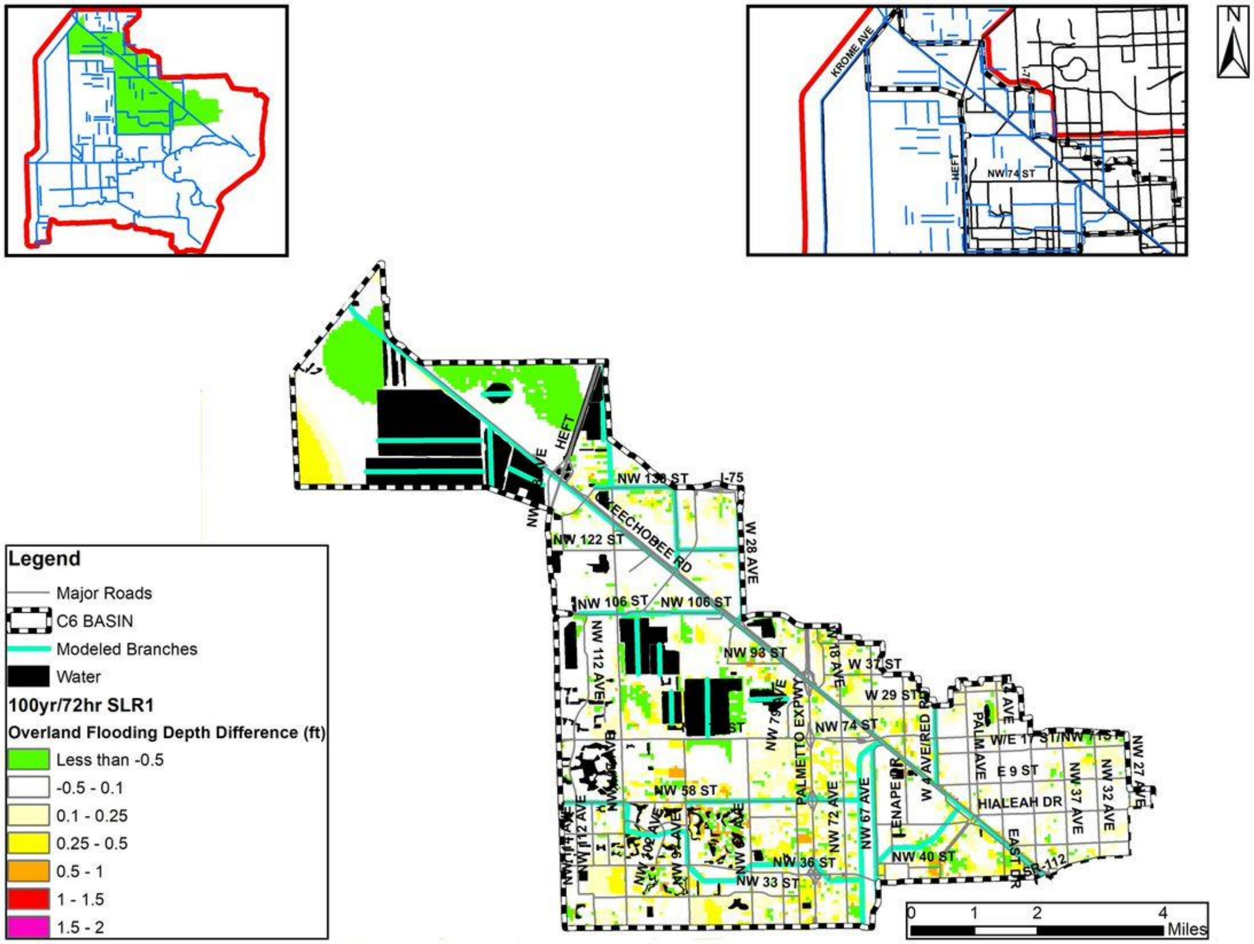


Figure C 5-43. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C6 Watershed

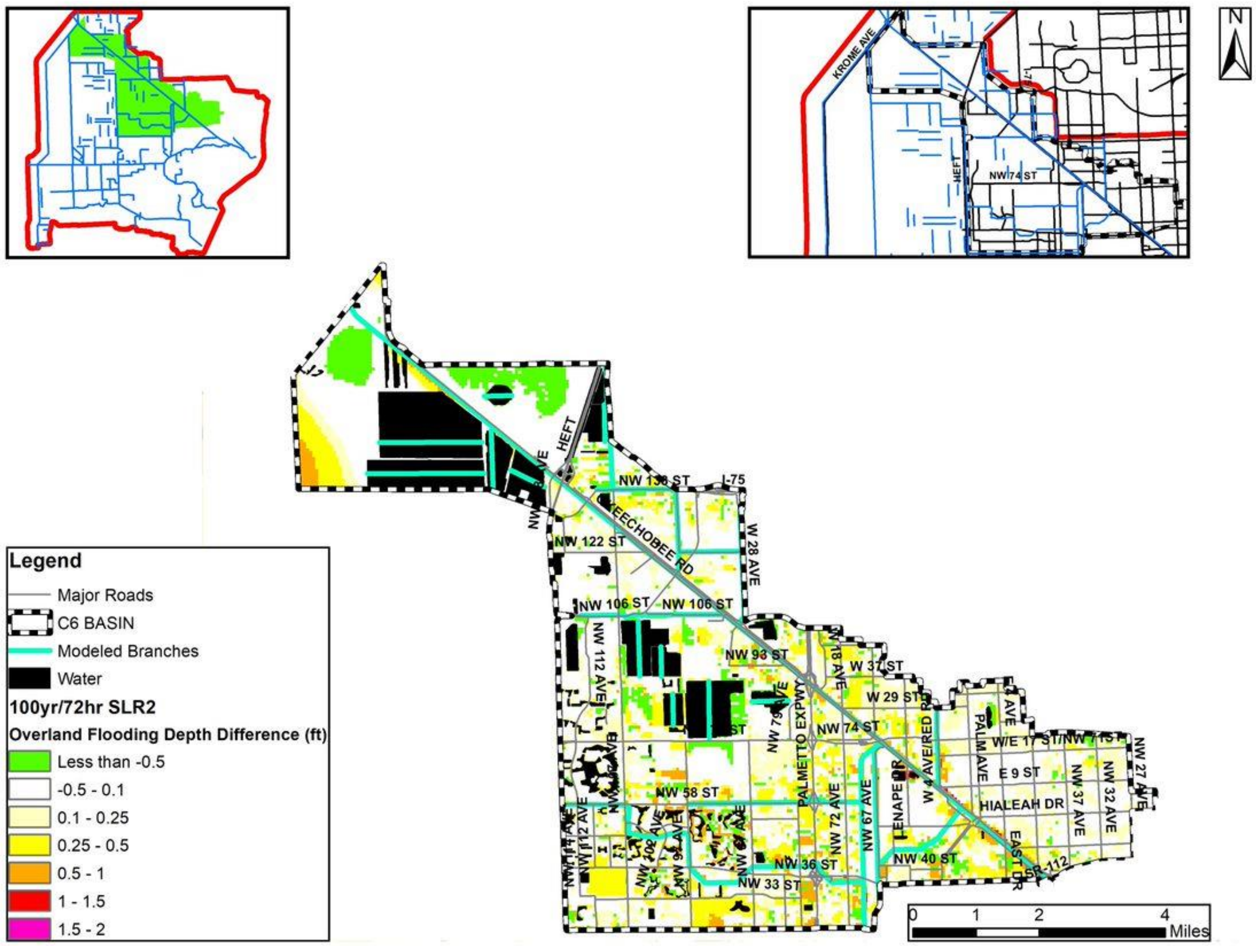


Figure C 5-44. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C6 Watershed

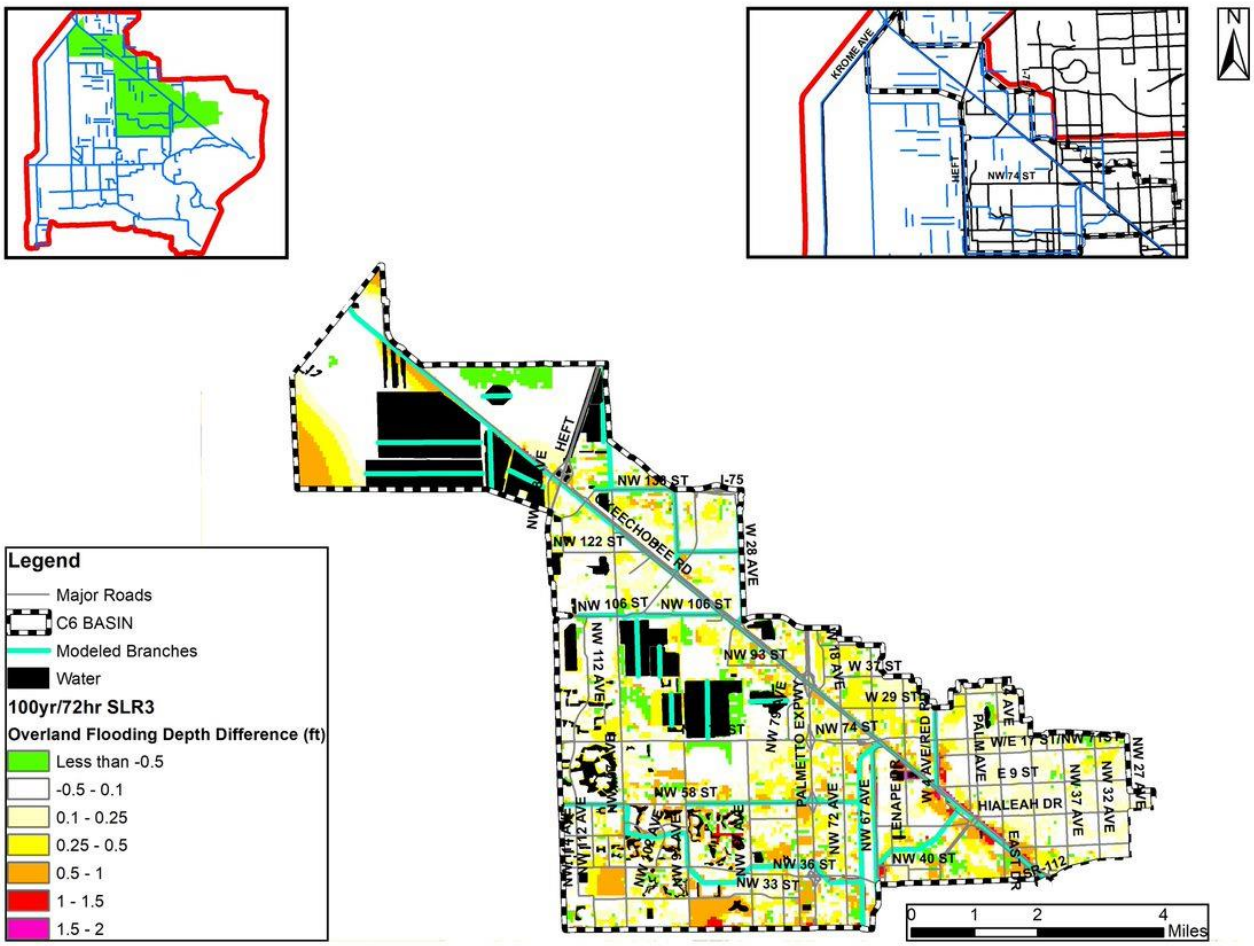


Figure C 5-45. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C6 Watershed

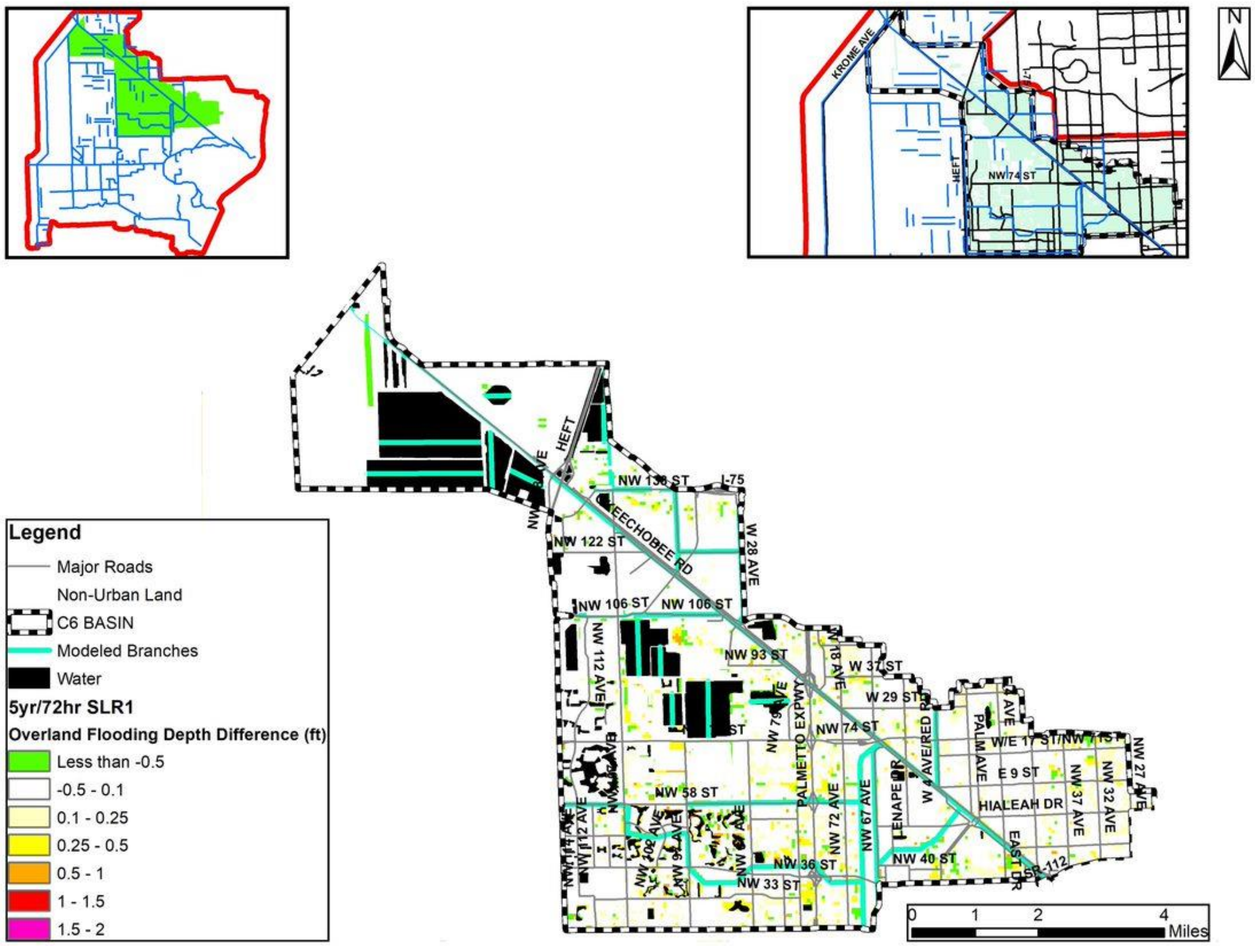


Figure C 5-46. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C6 Watershed

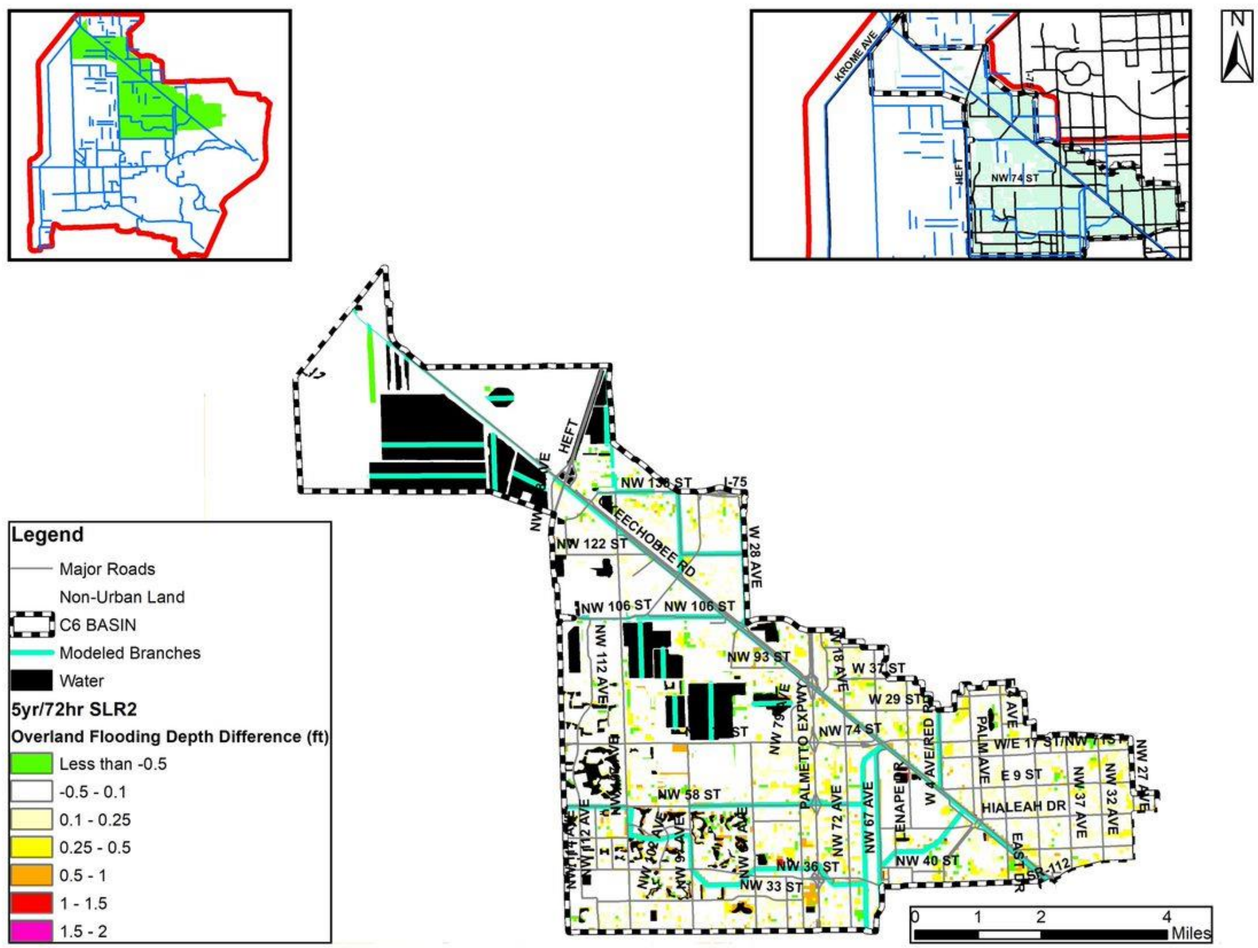


Figure C 5-47. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C6 Watershed

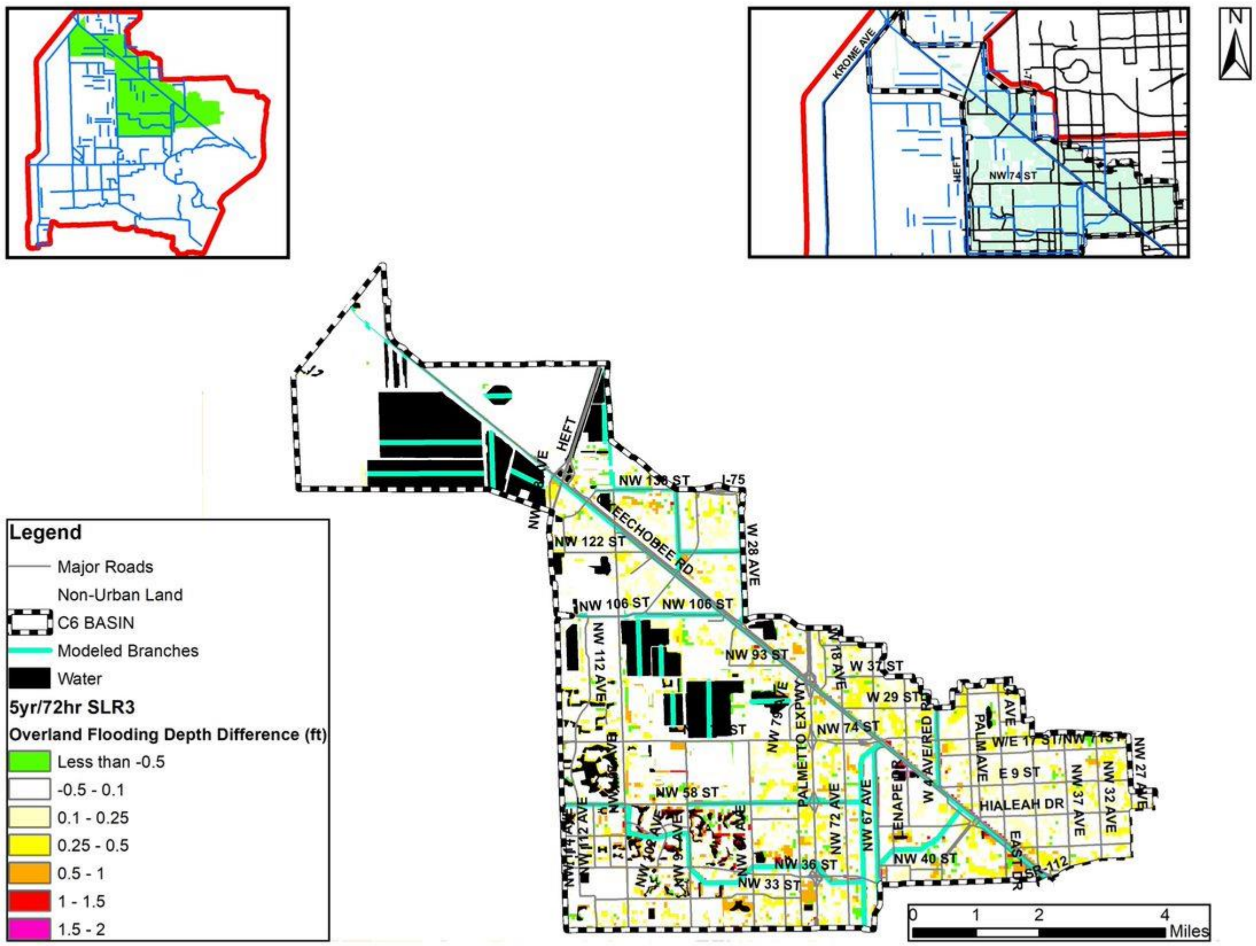


Figure C 5-48. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C6 Watershed

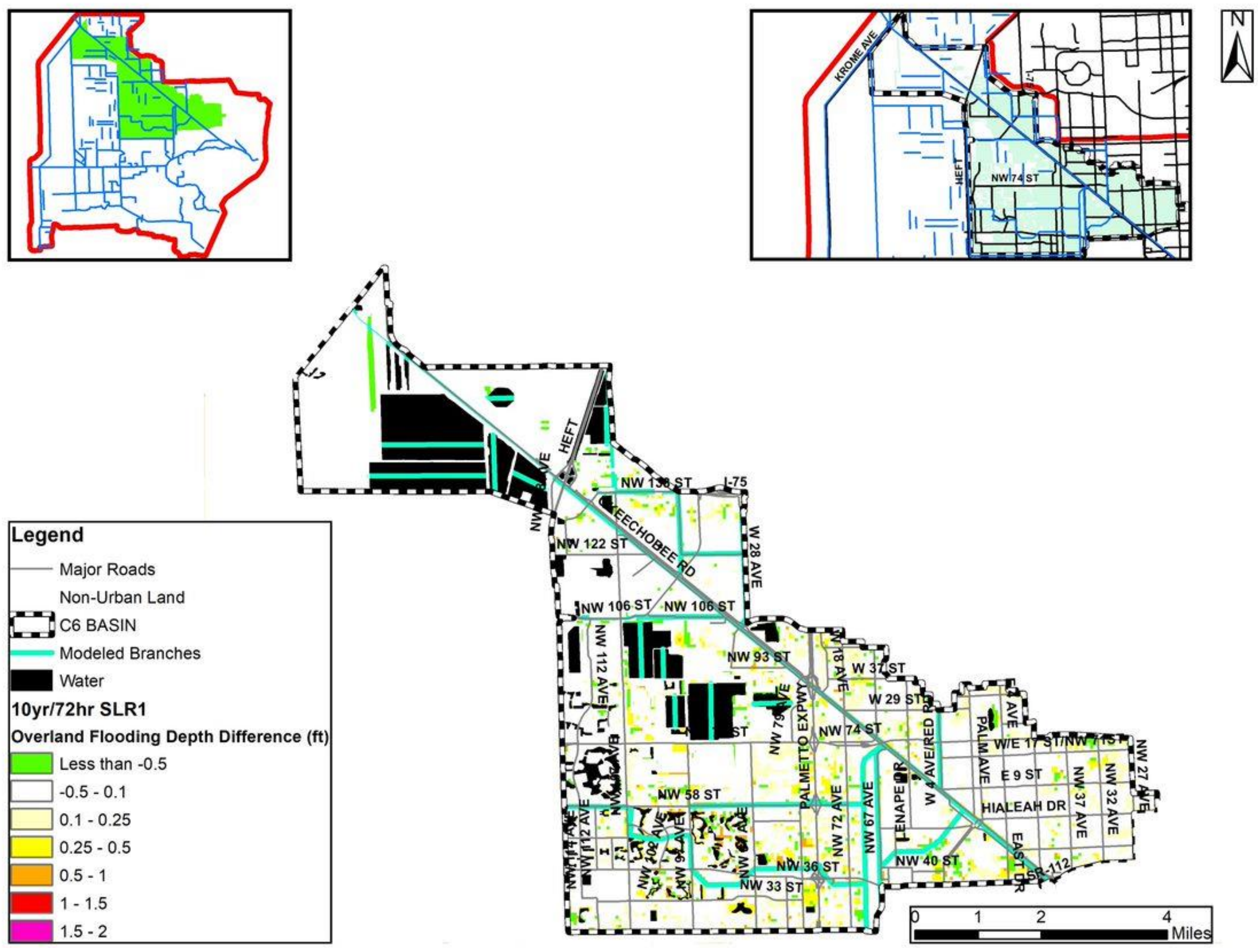


Figure C 5-49. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C6 Watershed

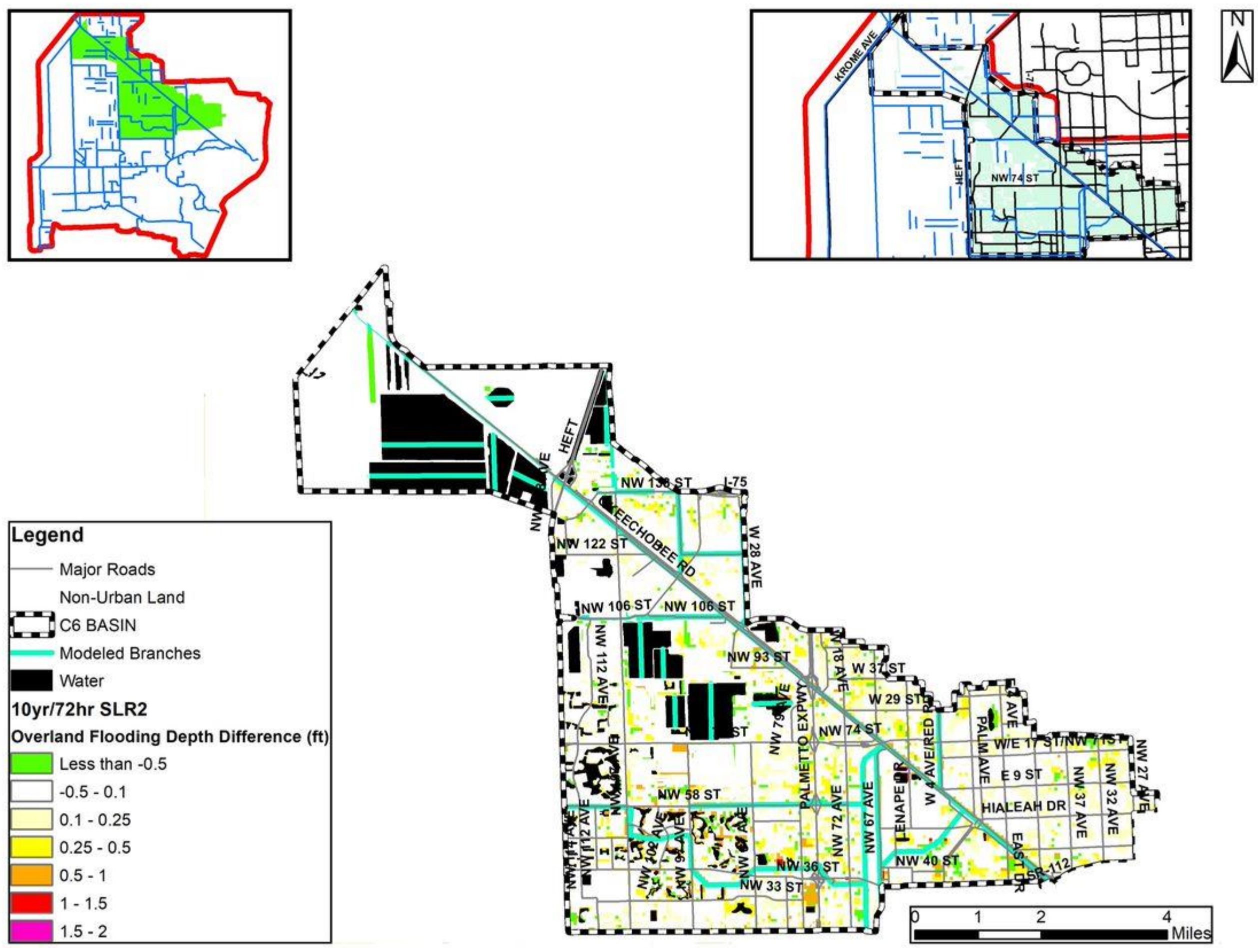


Figure C 5-50. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C6 Watershed

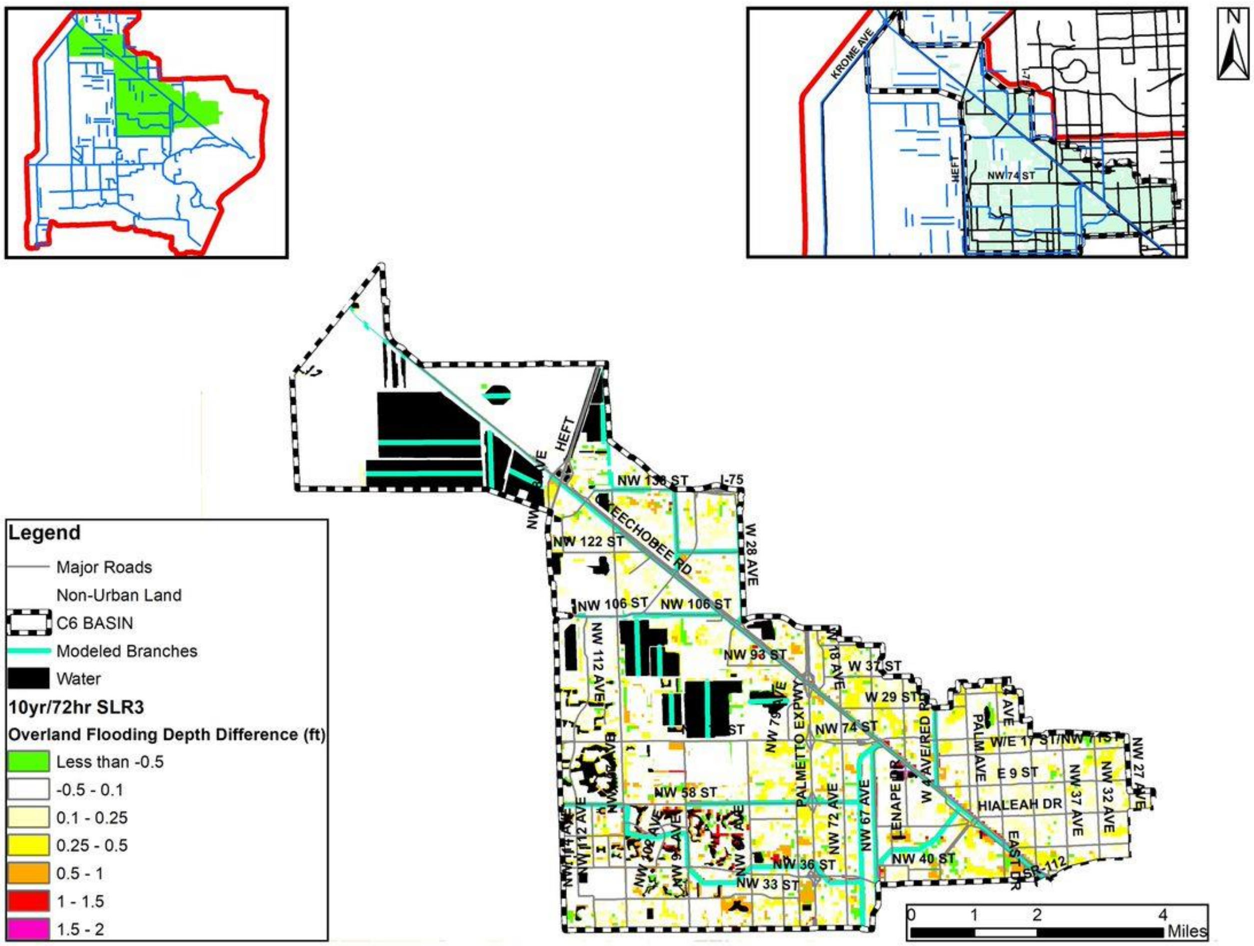


Figure C 5-51. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C6 Watershed

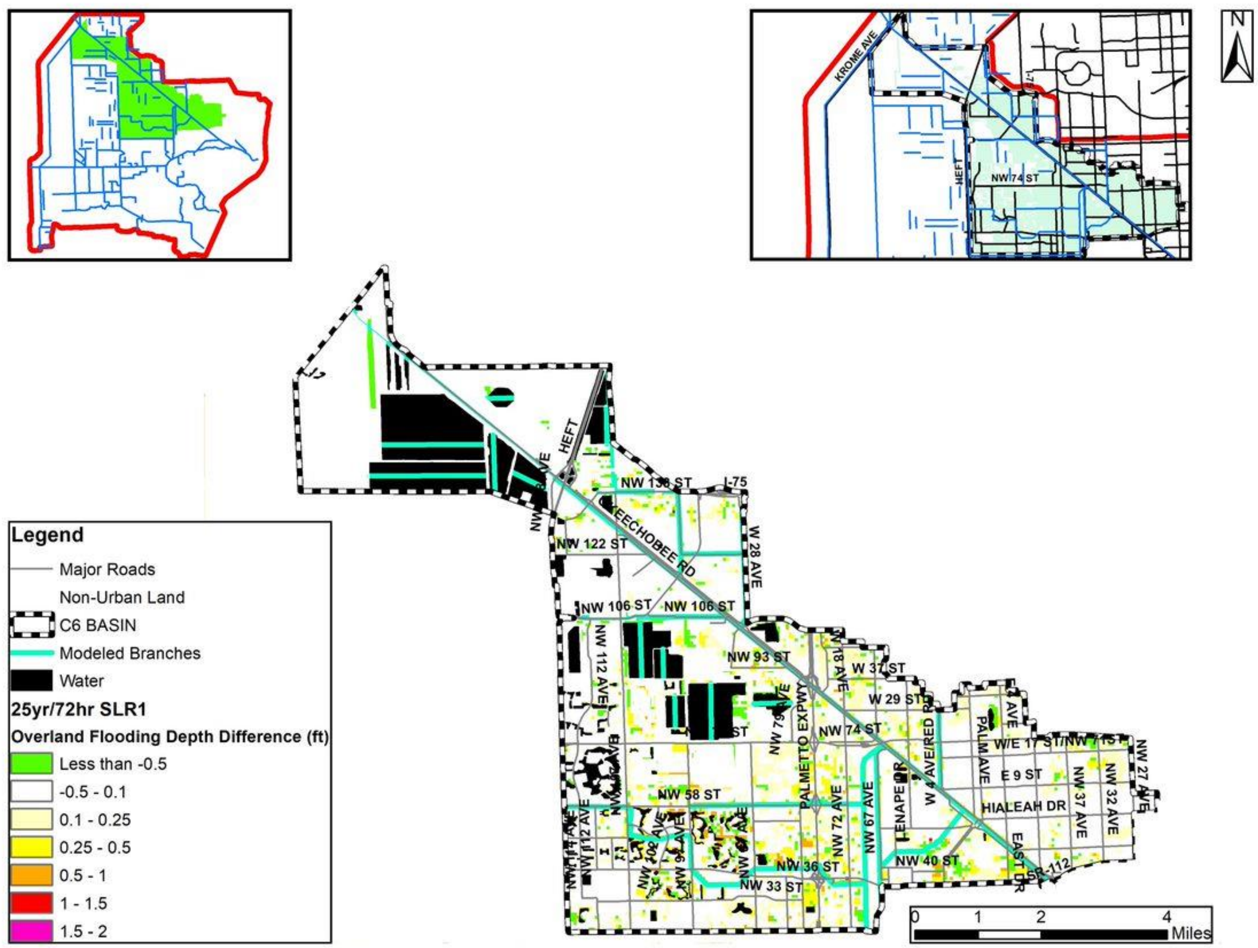


Figure C 5-52. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C6 Watershed

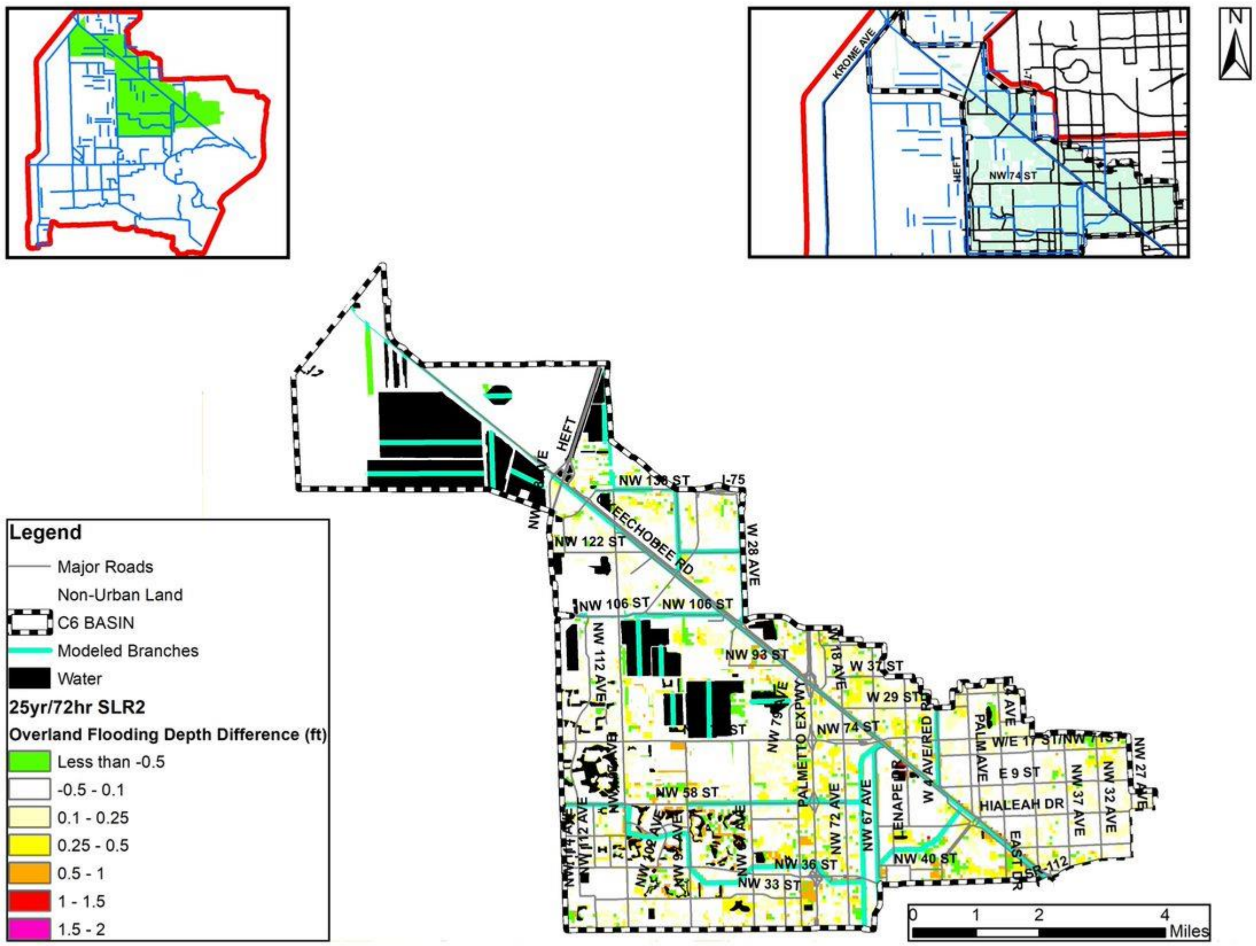


Figure C 5-53. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C6 Watershed

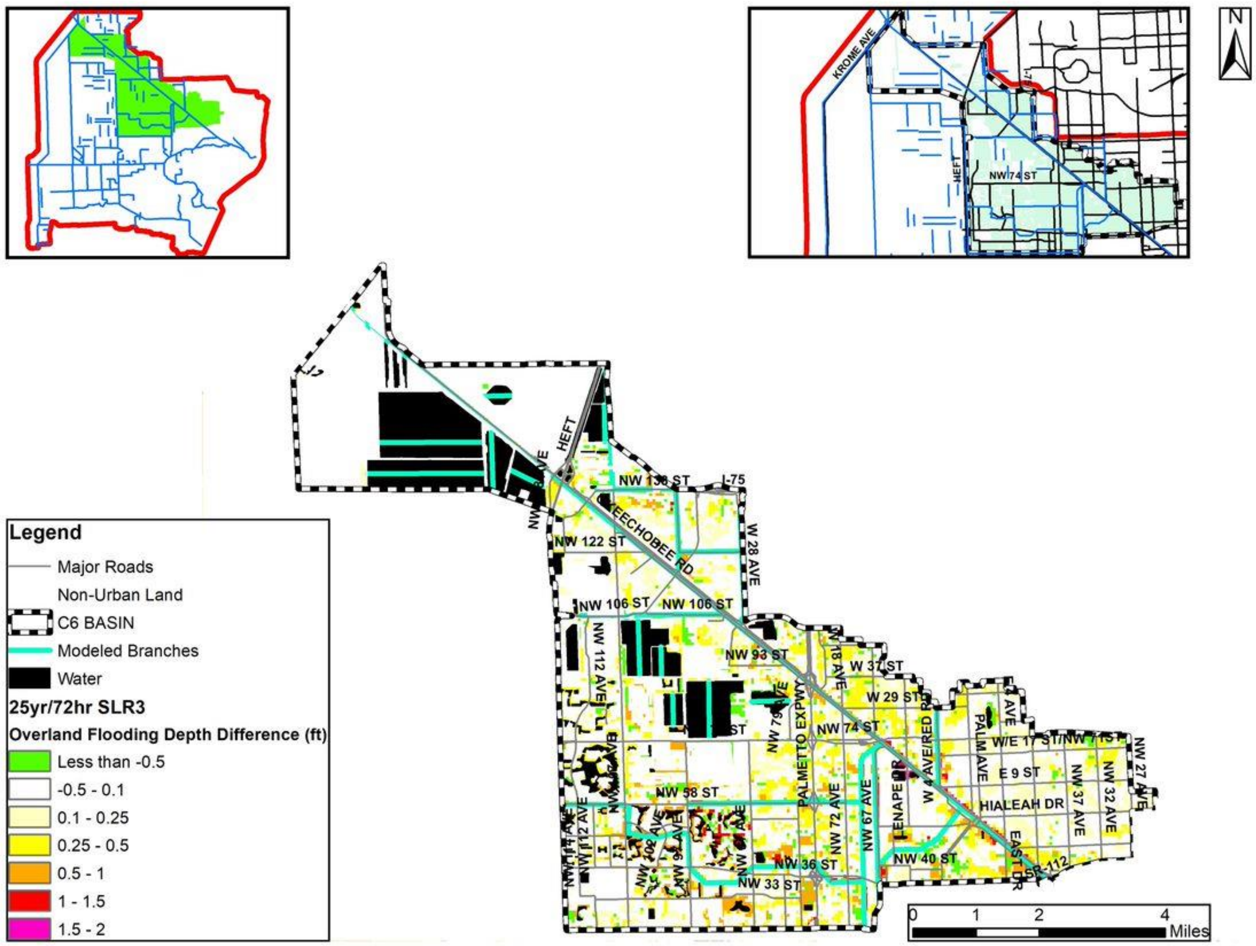


Figure C 5-54. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C6 Watershed

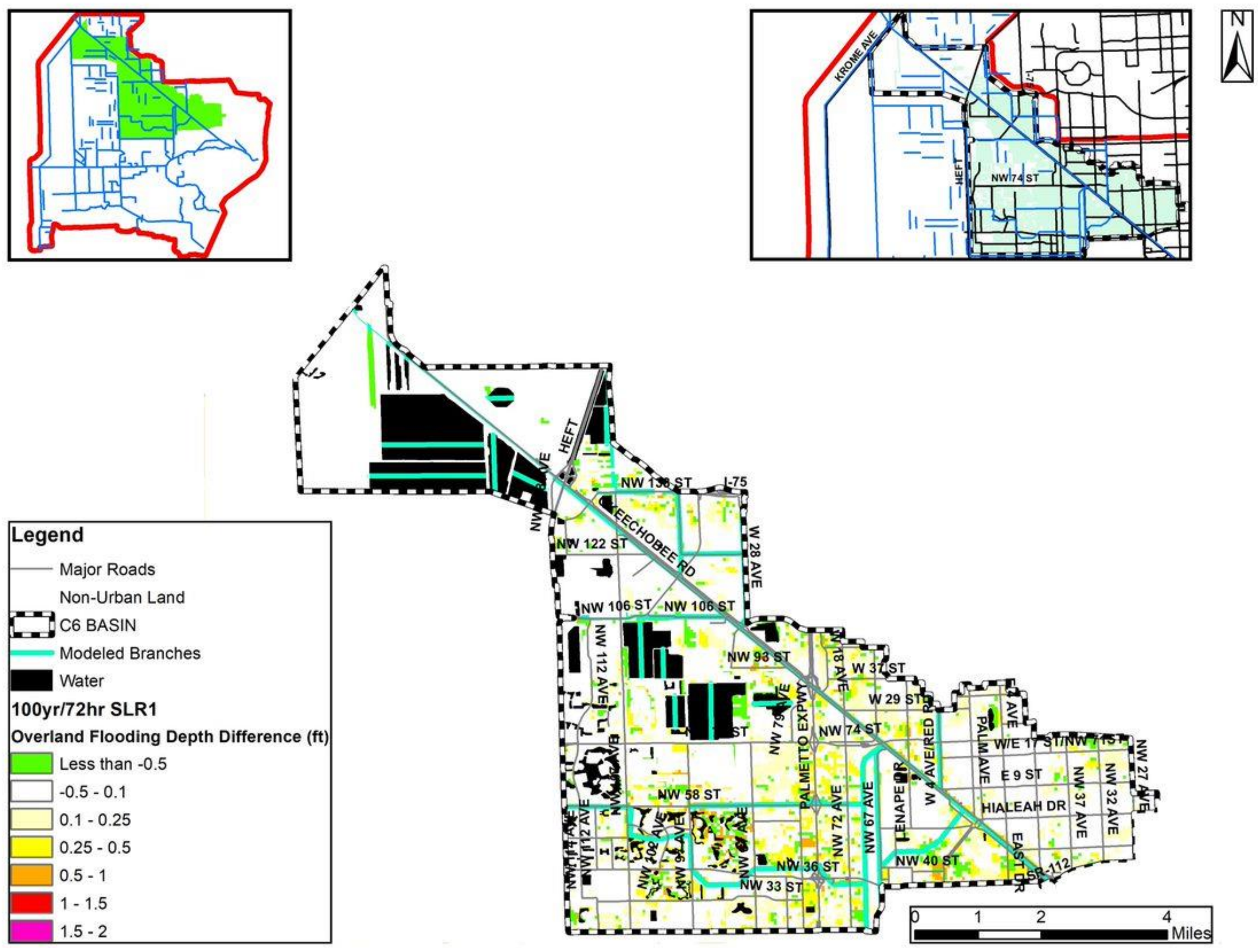


Figure C 5-55. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C6 Watershed

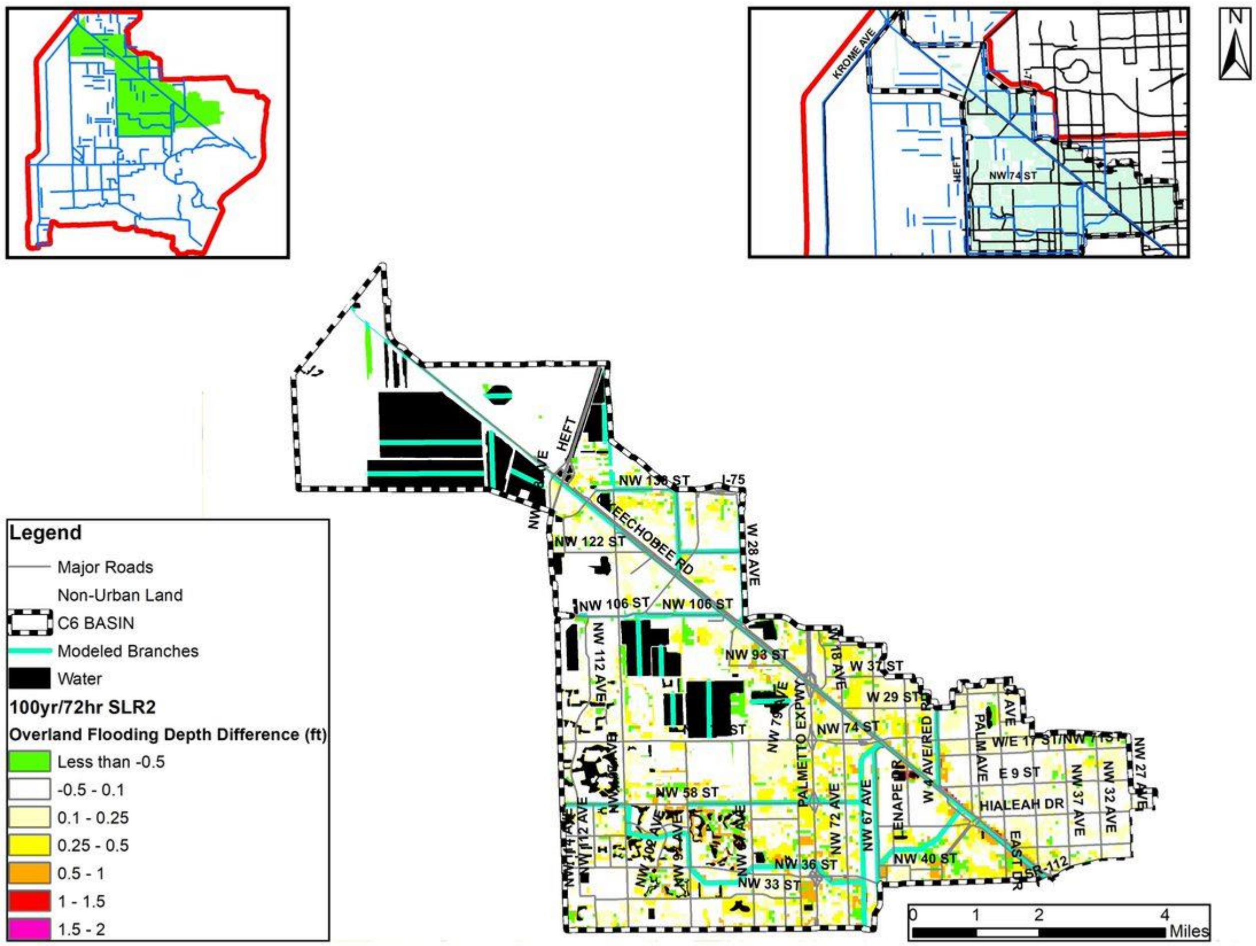
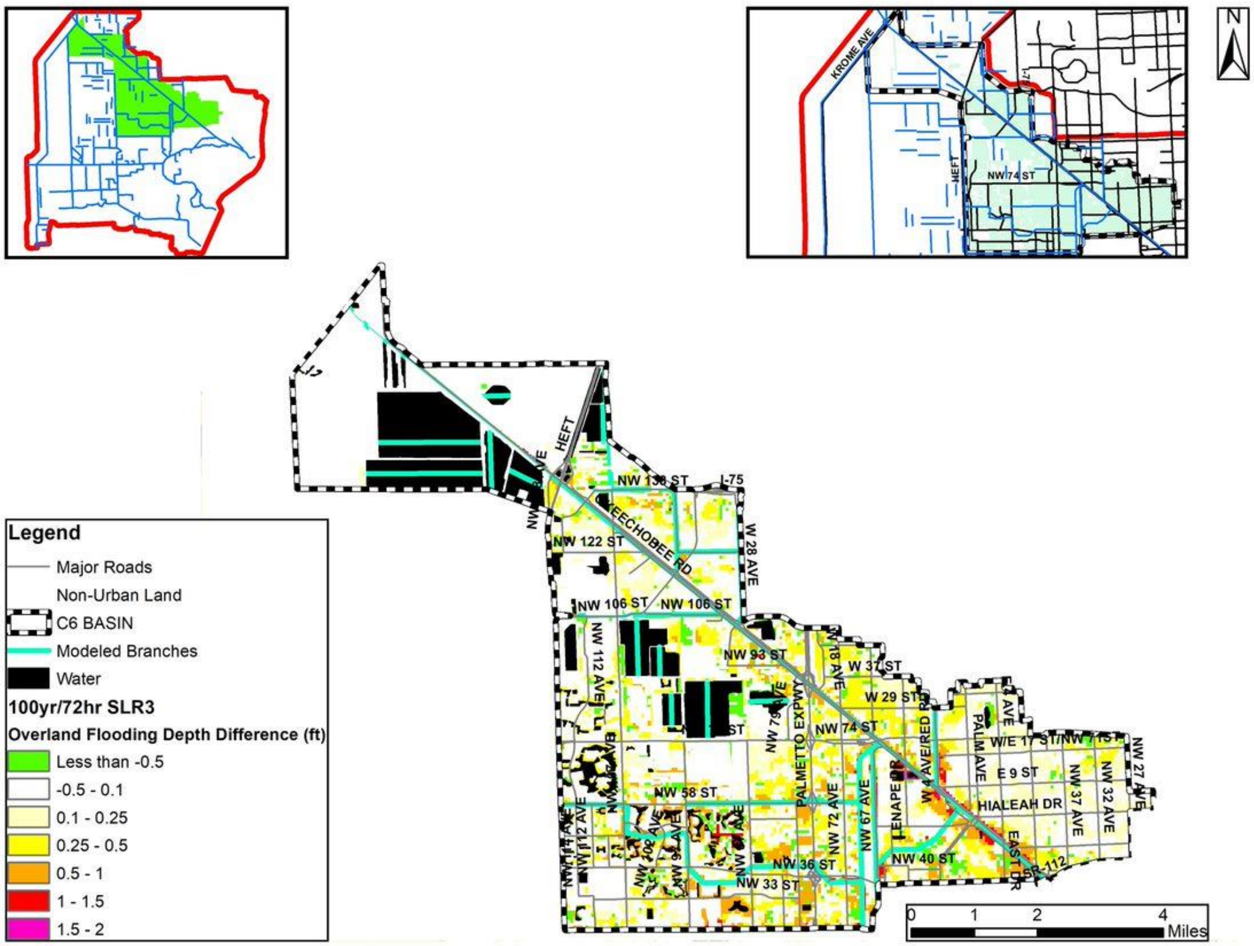


Figure C 5-56. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C6 Watershed



APPENDIX D

PM6 Flood Duration Figures

Contents

Figures

Appendix D – PM#6 Flood Duration Figures

The figures provided in this appendix include flood duration figures for each watershed. The figures compare the duration of flooding for each SLR condition (i.e., Current Conditions, SLR1, SLR2, and SLR3).

1 C2 Watershed

Figure D 1-1 through **Figure D 1-16** provides flood duration maps for the C2 Watershed for overland flooding depths exceeding 0.25 ft for the 5-, 10-, 25-, and 100-year 3-day design storms, respectively, for each SLR condition. Water areas are masked in black. Figure D 1-17 through **Figure D 1-32** provide the flood duration for only the urban areas within the C2 Watershed (non-urban areas are masked out).

Figure D 1-33 through **Figure D 1-44** show the difference in overland flooding for the C2 Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively. **Figure D 1-45** through **Figure D 1-56** show the same maps with the non-urban areas masked out.

Figure D 1-1. Flood Duration Map for the 5-year 3-Day Design Storm for Current Conditions in the C2 Watershed

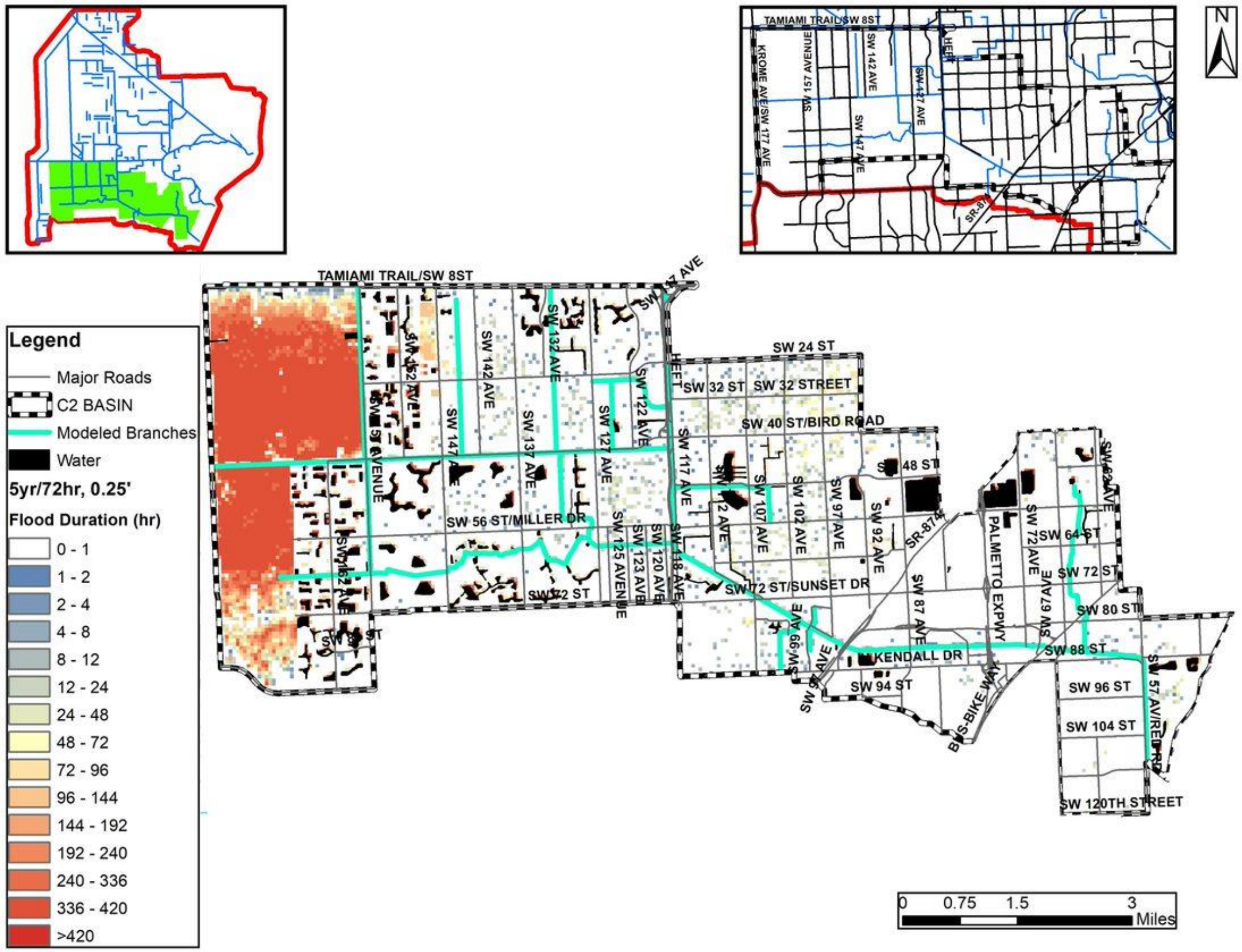


Figure D 1-2. Flood Duration Map for the 5-year 3-Day Design Storm for SLR1 in the C2 Watershed

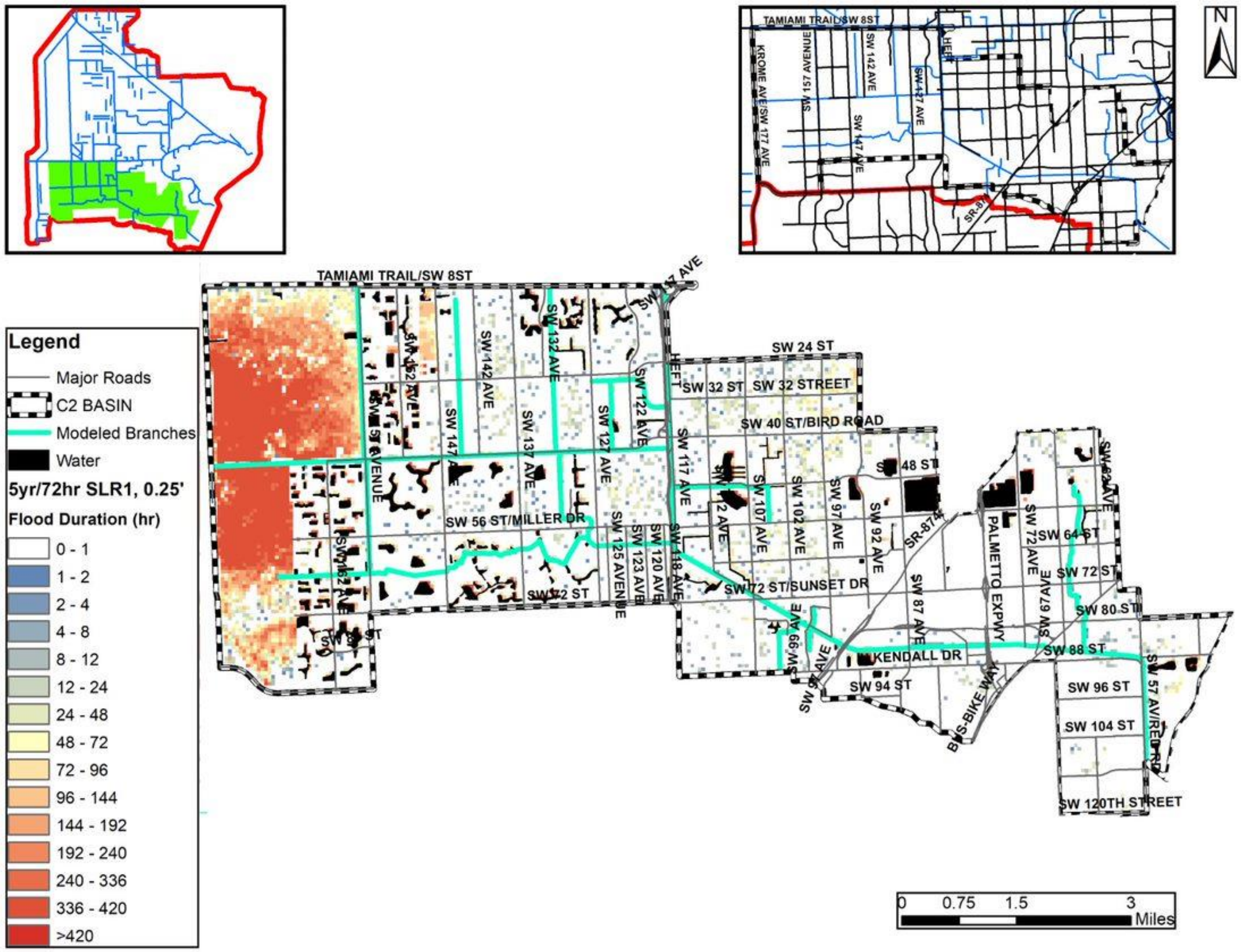


Figure D 1-3. Flood Duration Map for the 5-year 3-Day Design Storm for SLR2 in the C2 Watershed

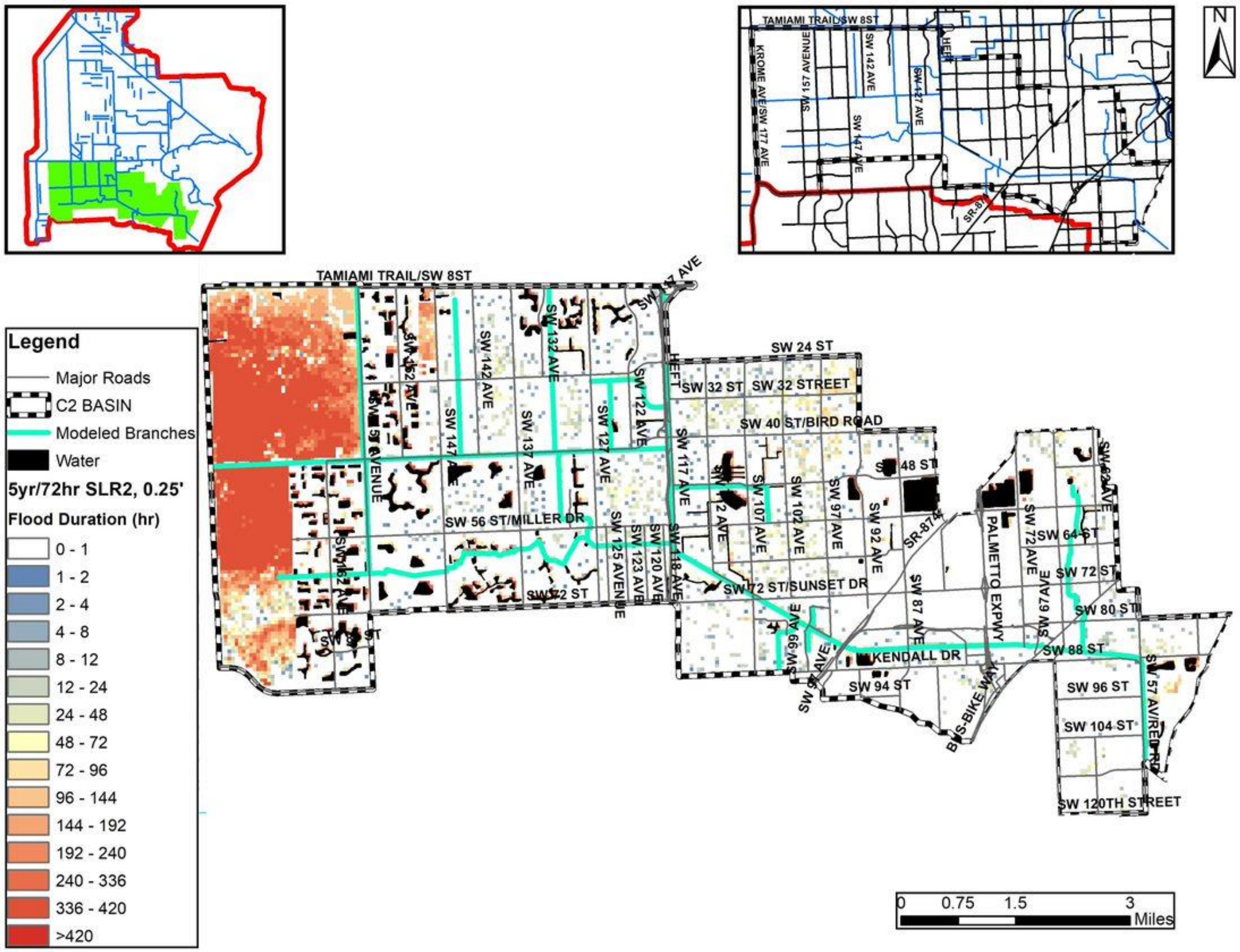


Figure D 1-4. Flood Duration Map for the 5-year 3-Day Design Storm for SLR3 in the C2 Watershed

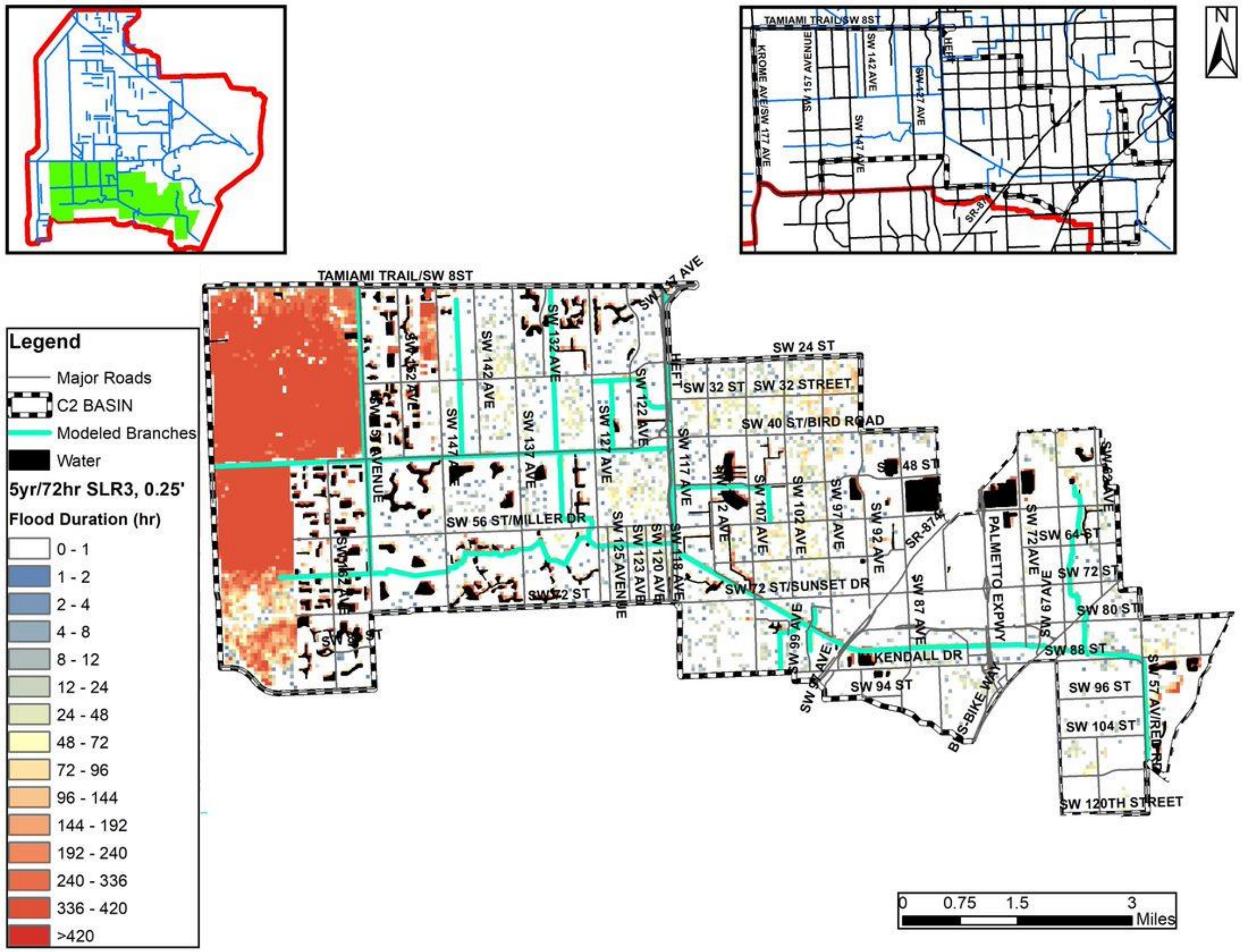


Figure D 1-5. Flood Duration Map for the 10-year 3-Day Design Storm for Current Conditions in the C2 Watershed

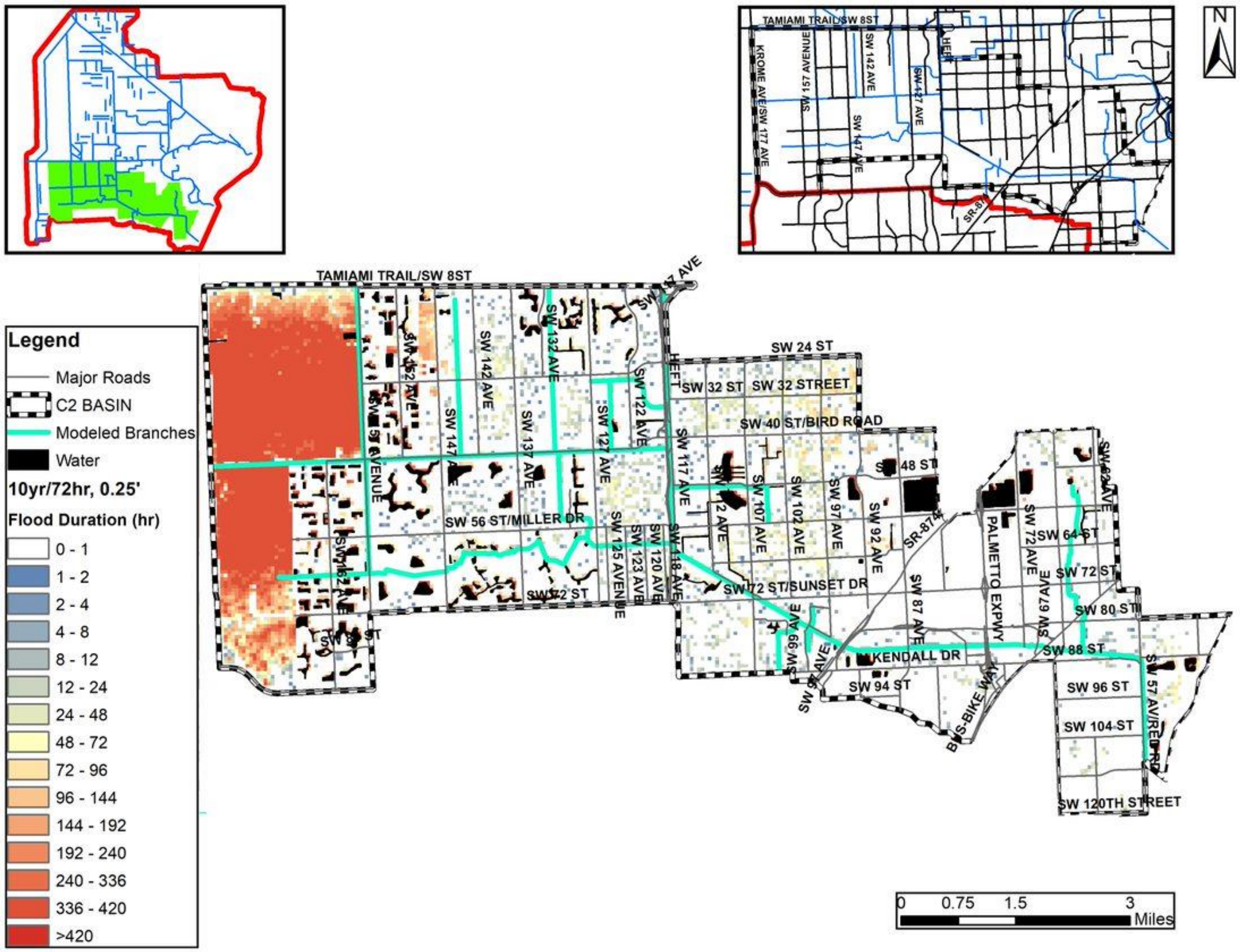


Figure D 1-6. Flood Duration Map for the 10-year 3-Day Design Storm for SLR1 in the C2 Watershed

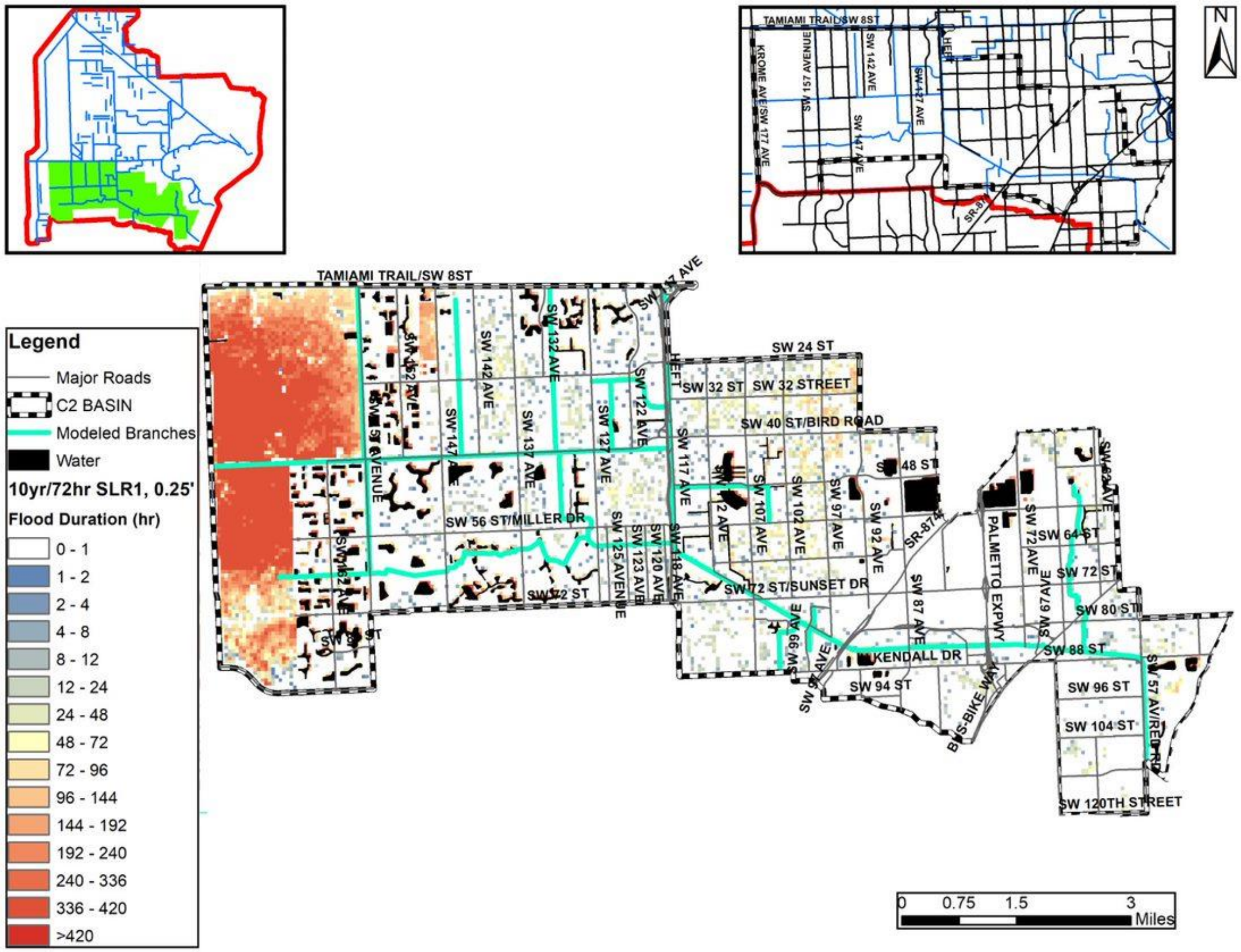


Figure D 1-7. Flood Duration Map for the 10-year 3-Day Design Storm for SLR2 in the C2 Watershed

Figure D 1-8. Flood Duration Map for the 10-year 3-Day Design Storm for SLR3 in the C2 Watershed

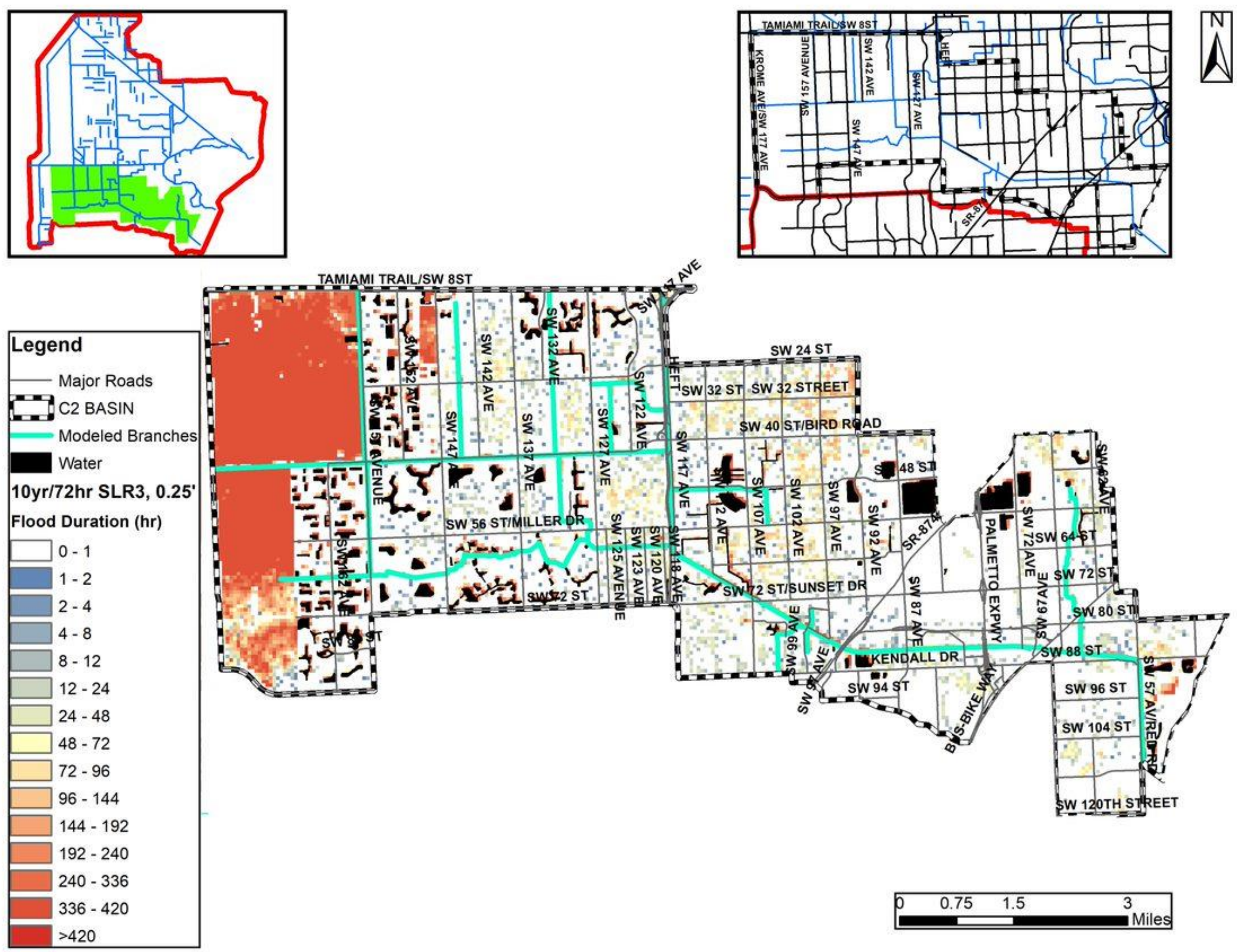


Figure D 1-9. Flood Duration Map for the 25-year 3-Day Design Storm for Current Conditions in the C2 Watershed

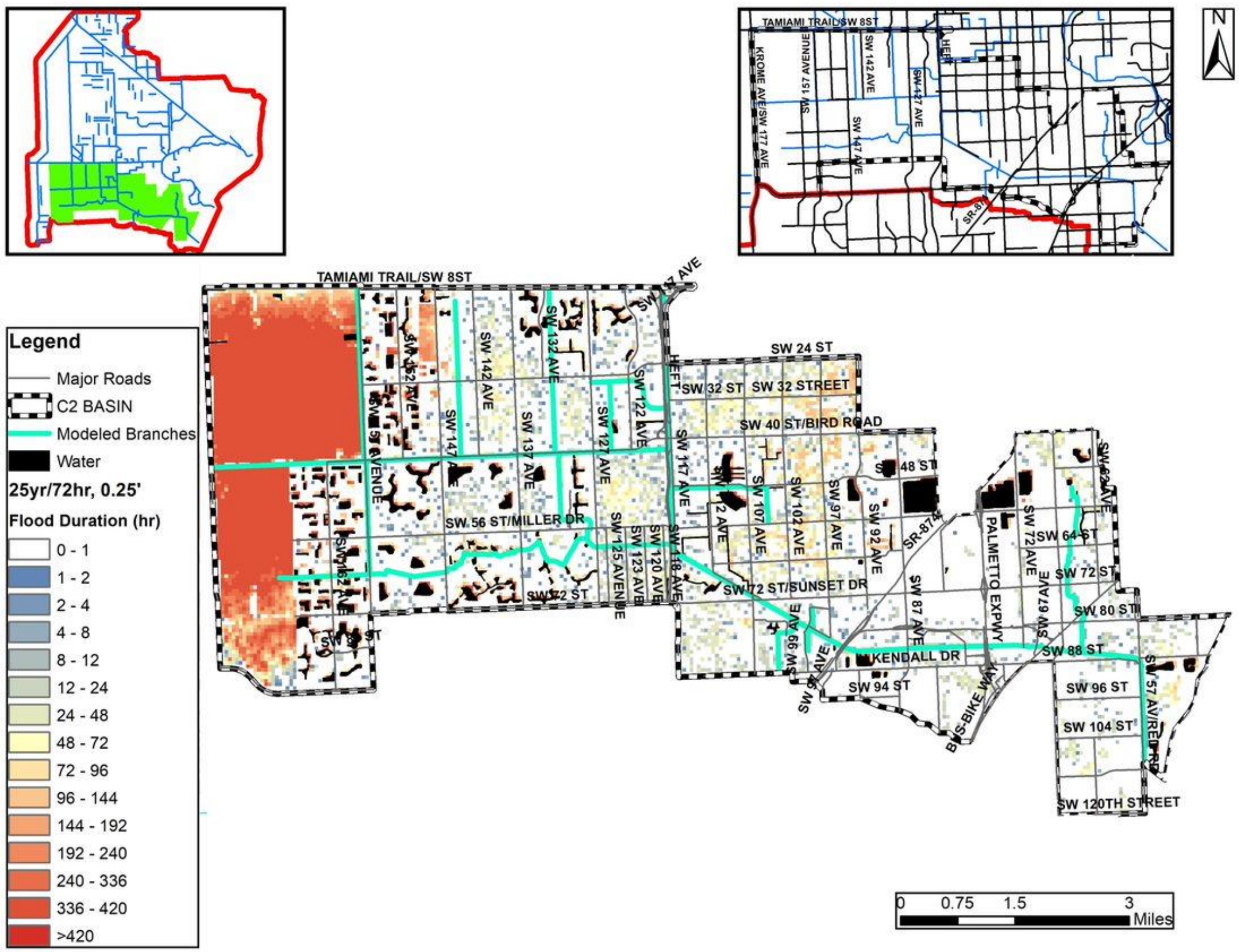


Figure D 1-10. Flood Duration Map for the 25-year 3-Day Design Storm for SLR1 in the C2 Watershed

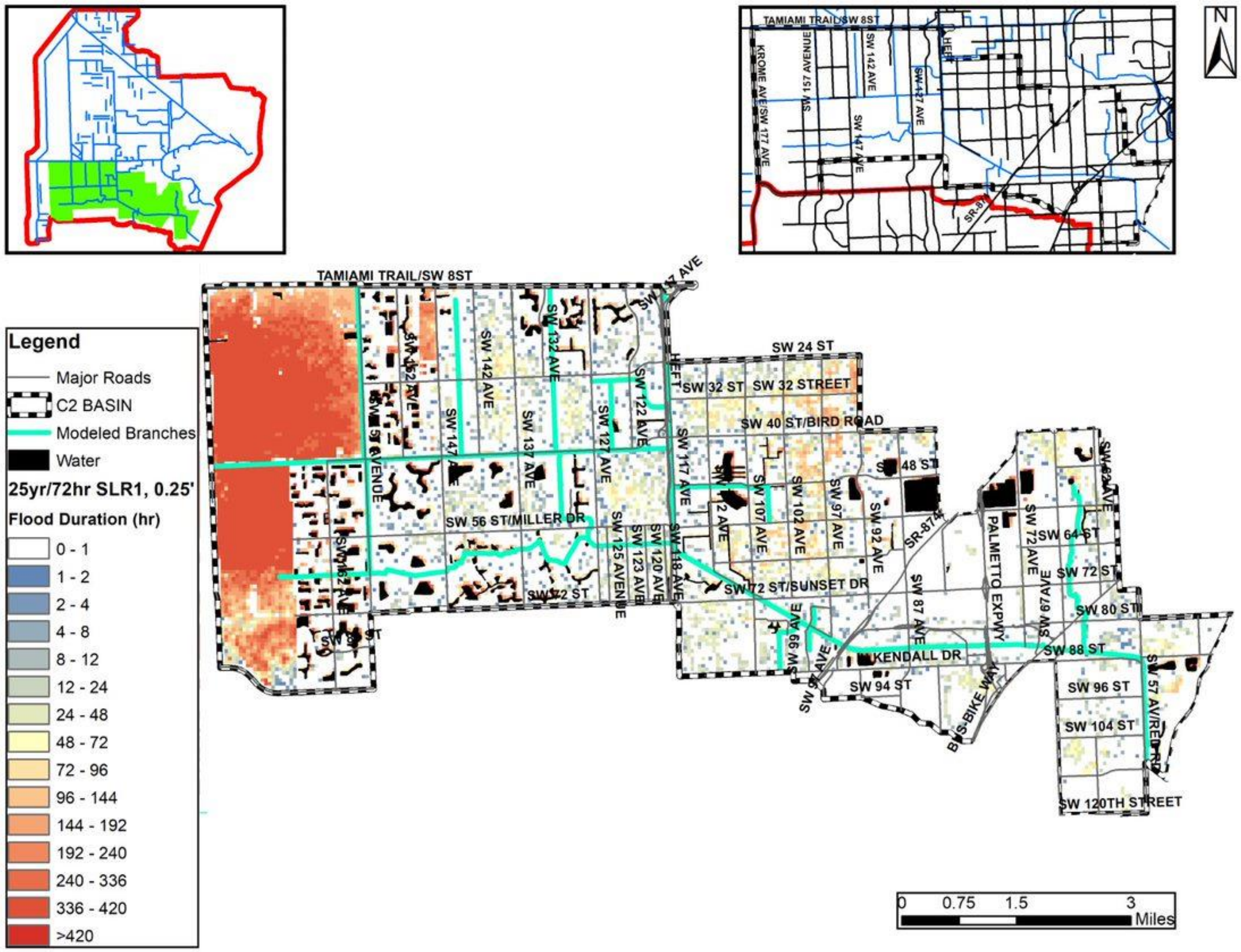


Figure D 1-11. Flood Duration Map for the 25-year 3-Day Design Storm for SLR2 in the C2 Watershed

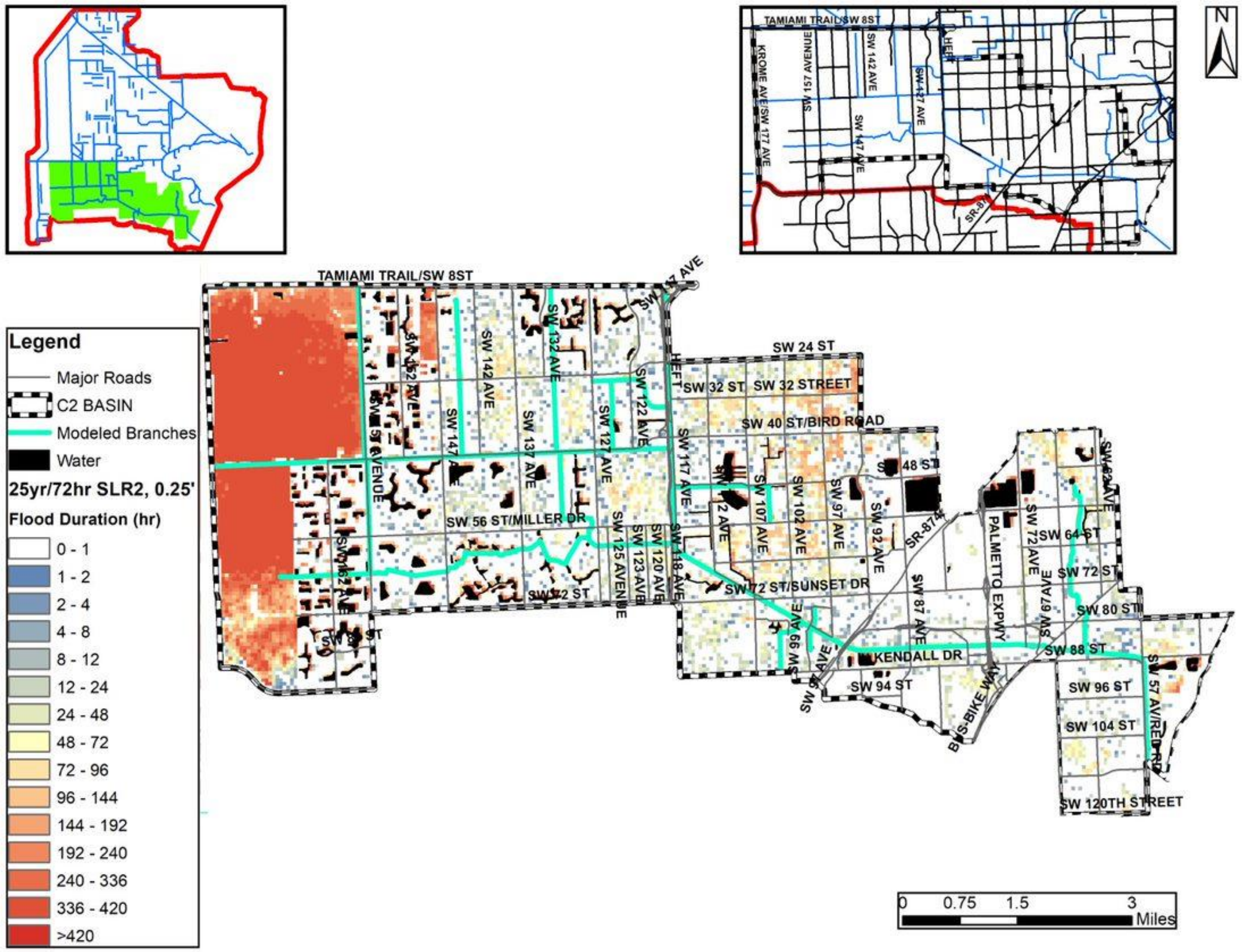


Figure D 1-12. Flood Duration Map for the 25-year 3-Day Design Storm for SLR3 in the C2 Watershed

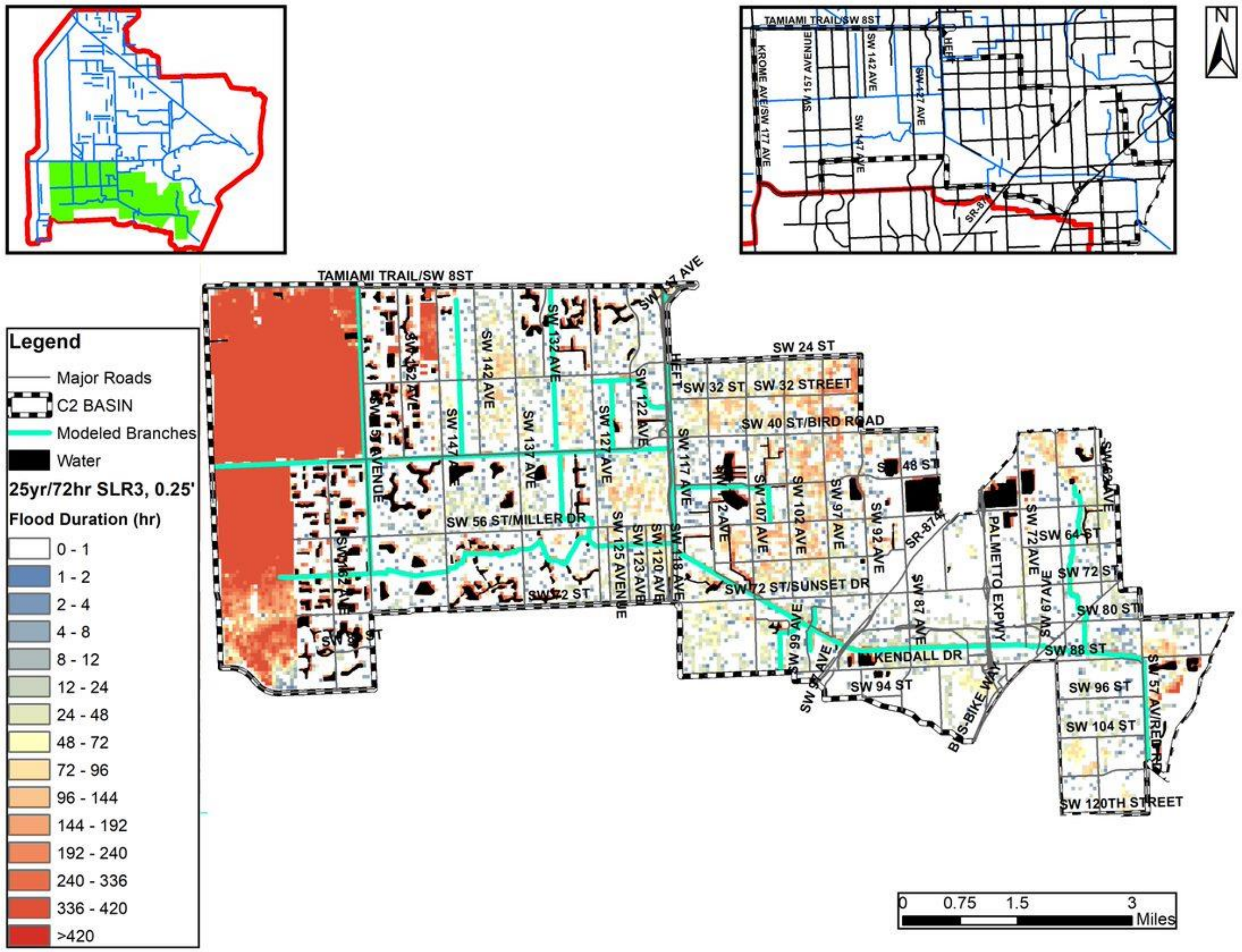


Figure D 1-13. Flood Duration Map for the 100-year 3-Day Design Storm for Current Conditions in the C2 Watershed

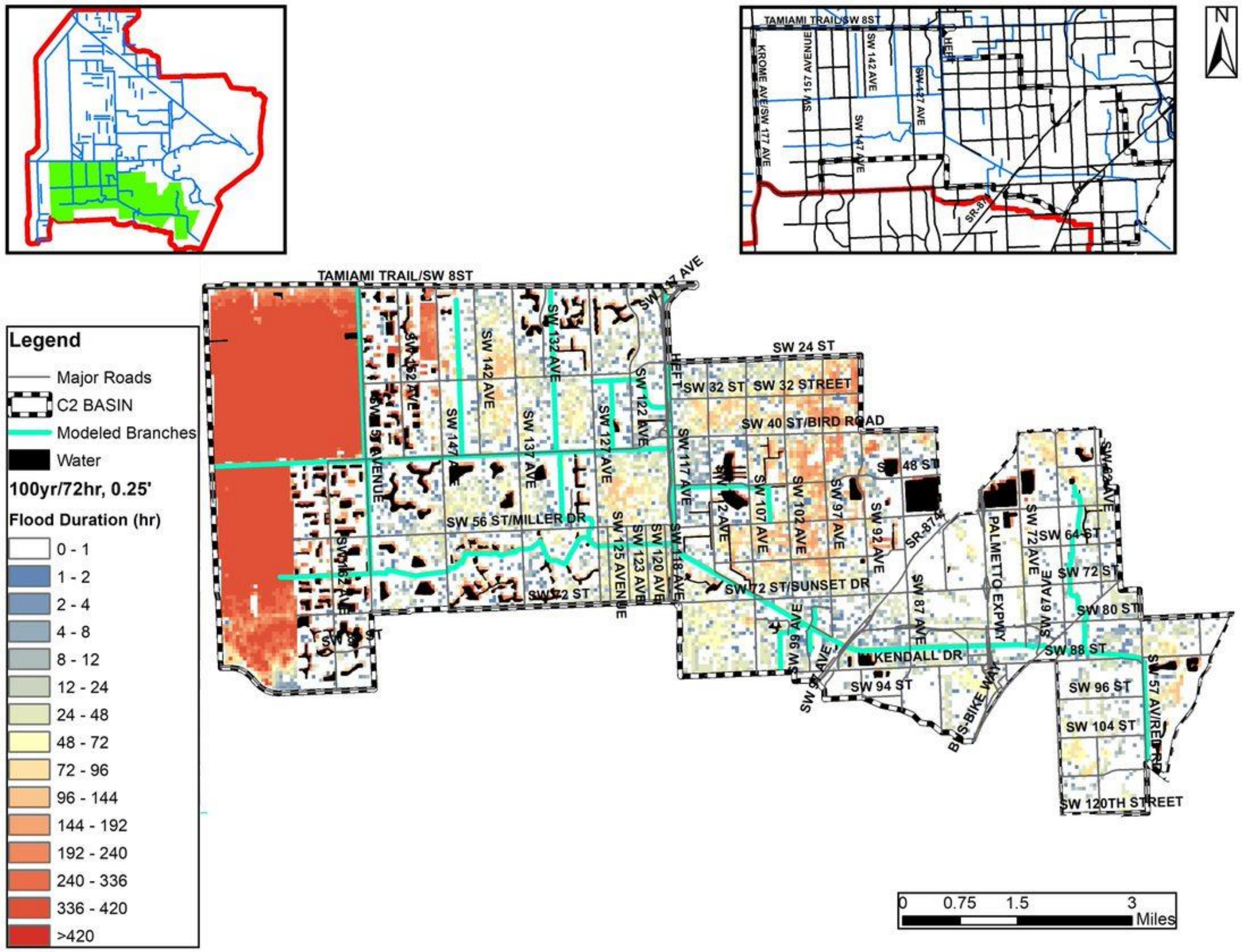


Figure D 1-14. Flood Duration Map for the 100-year 3-Day Design Storm for SLR1 in the C2 Watershed

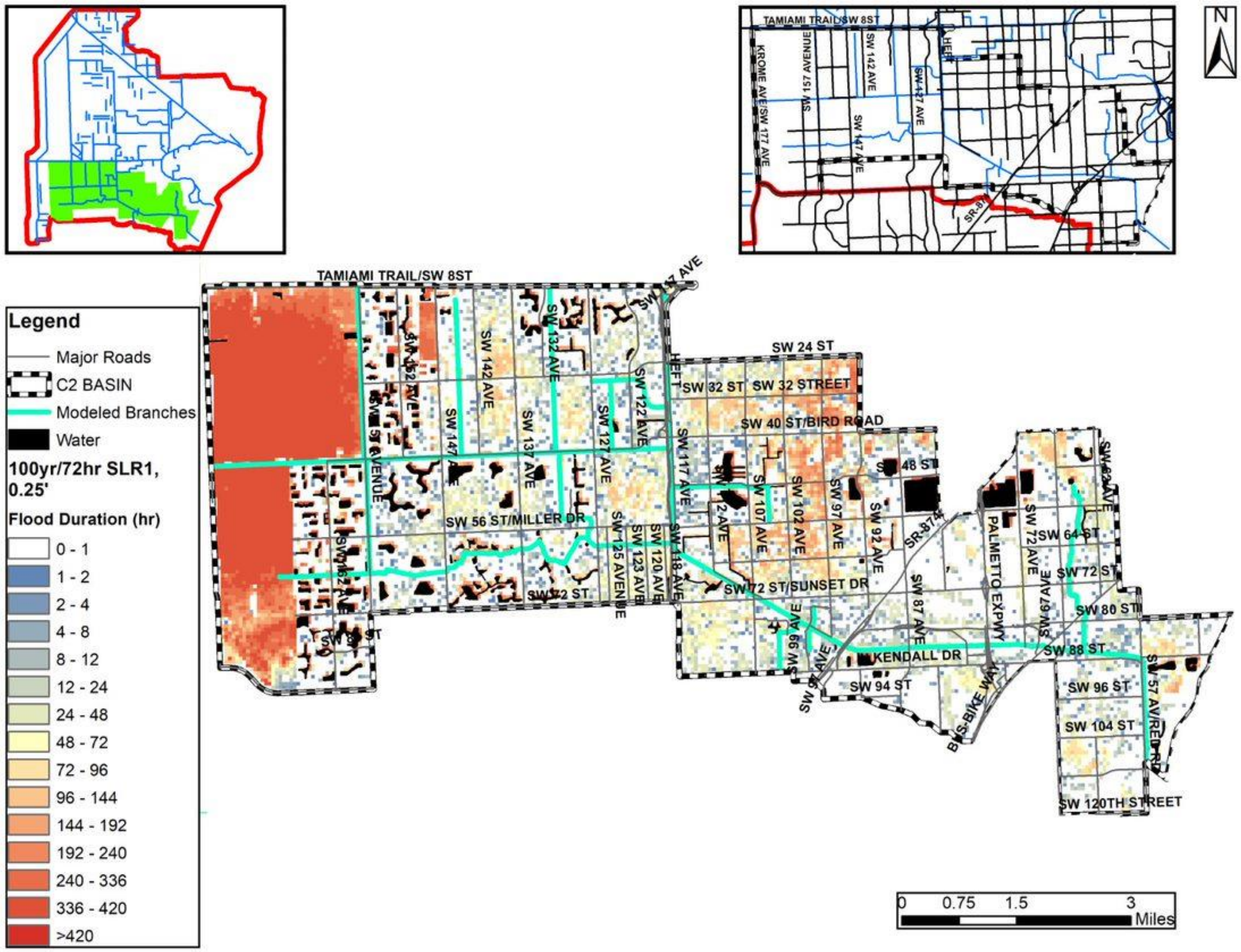


Figure D 1-15. Flood Duration Map for the 100-year 3-Day Design Storm for SLR2 in the C2 Watershed

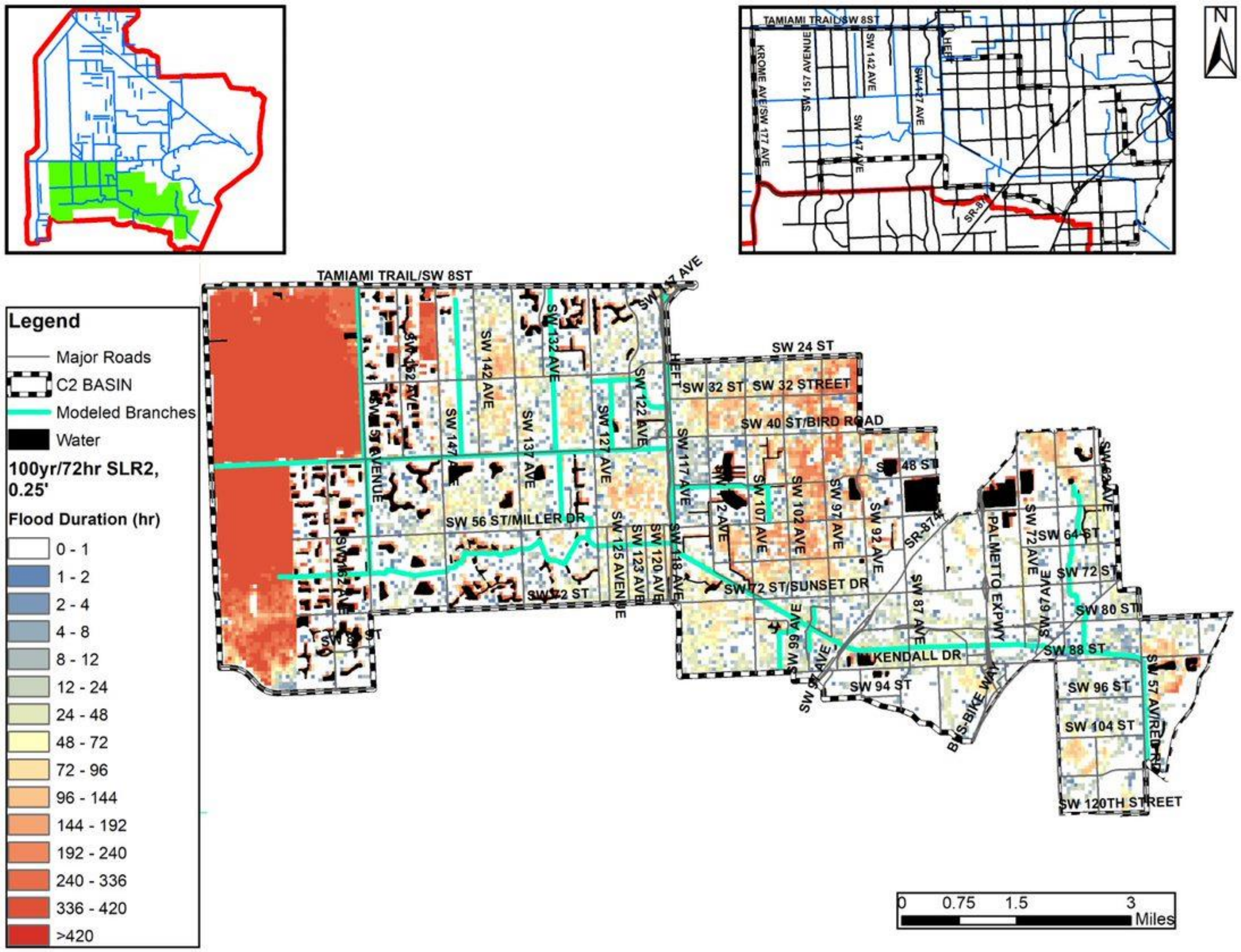


Figure D 1-16. Flood Duration Map for the 100-year 3-Day Design Storm for SLR3 in the C2 Watershed

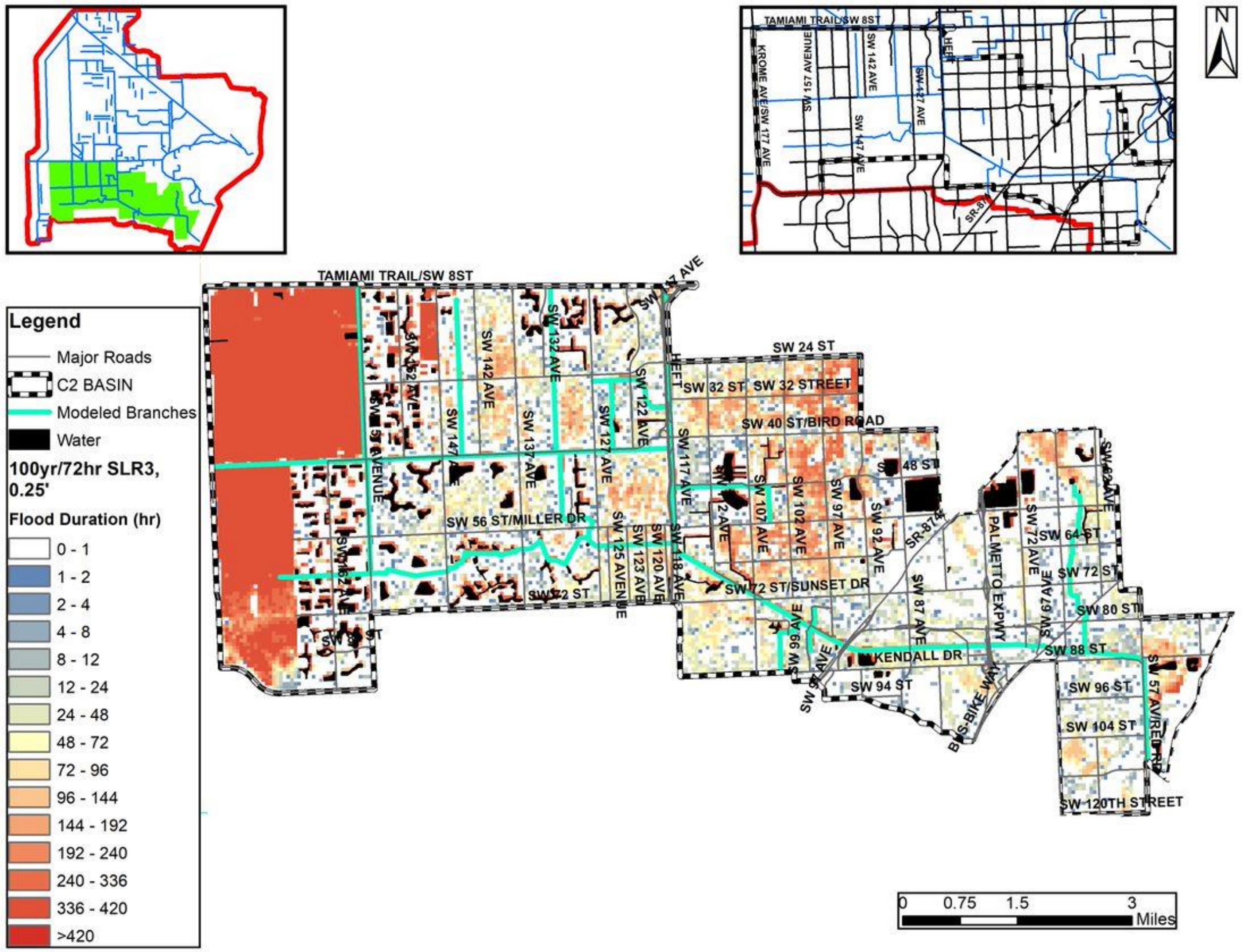


Figure D 1-17. Flood Duration Map for the 5-year 3-Day Design Storm for Current Conditions in Urban Areas in the C2 Watershed

Figure D 1-18. Flood Duration Map for the 5-year 3-Day Design Storm for SLR1 in Urban Areas in the C2 Watershed

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Figure D 1-29. Flood Duration Map for the 100-year 3-Day Design Storm for Current Conditions in Urban Areas in the C2 Watershed

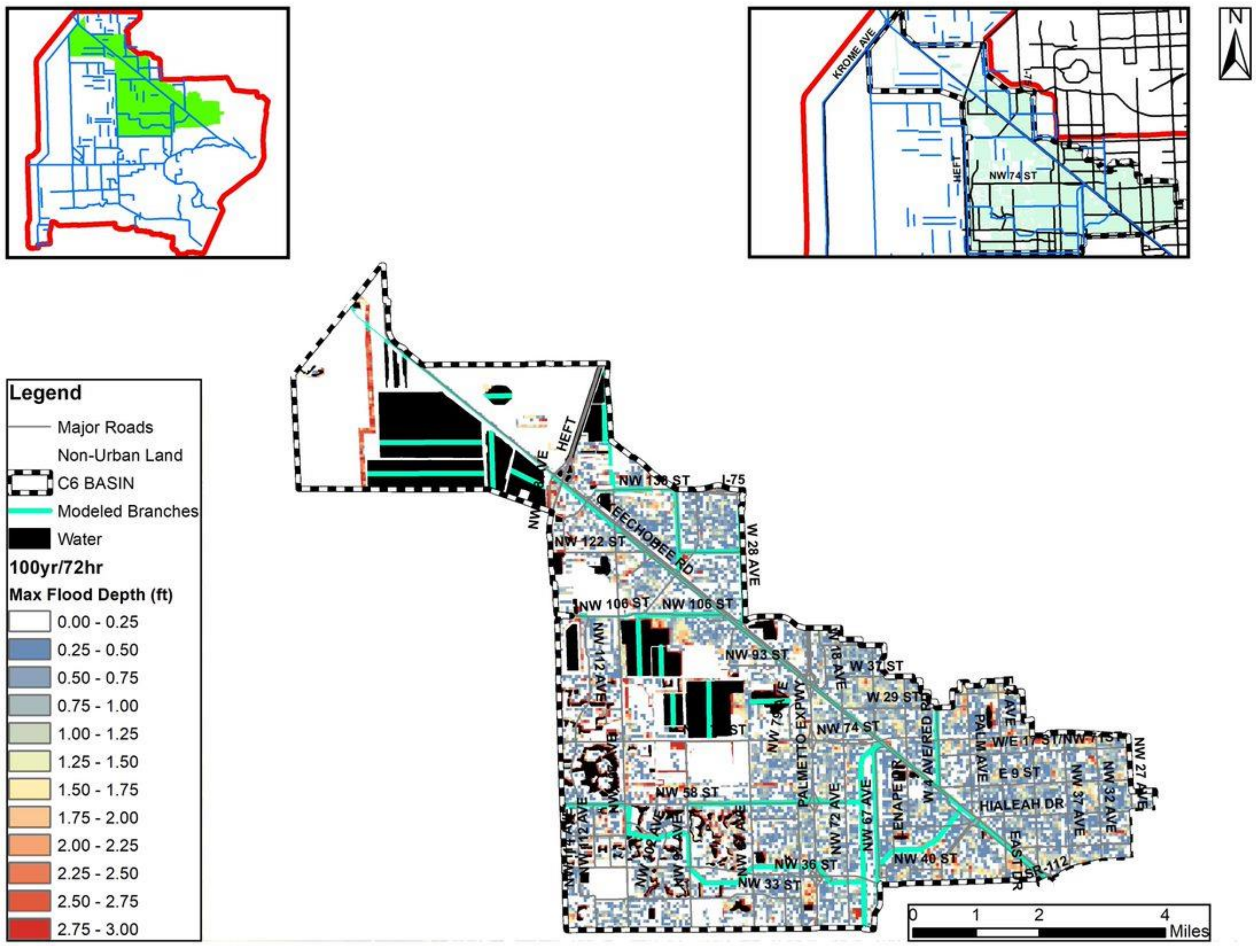

Figure D 1-30. Flood Duration Map for the 100-year 3-Day Design Storm for SLR1 in Urban Areas in the C2 Watershed

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Figure D 1-32. Flood Duration Map for the 100-year 3-Day Design Storm for SLR3 in Urban Areas in the C2 Watershed

Figure D 1-33. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C2 Watershed

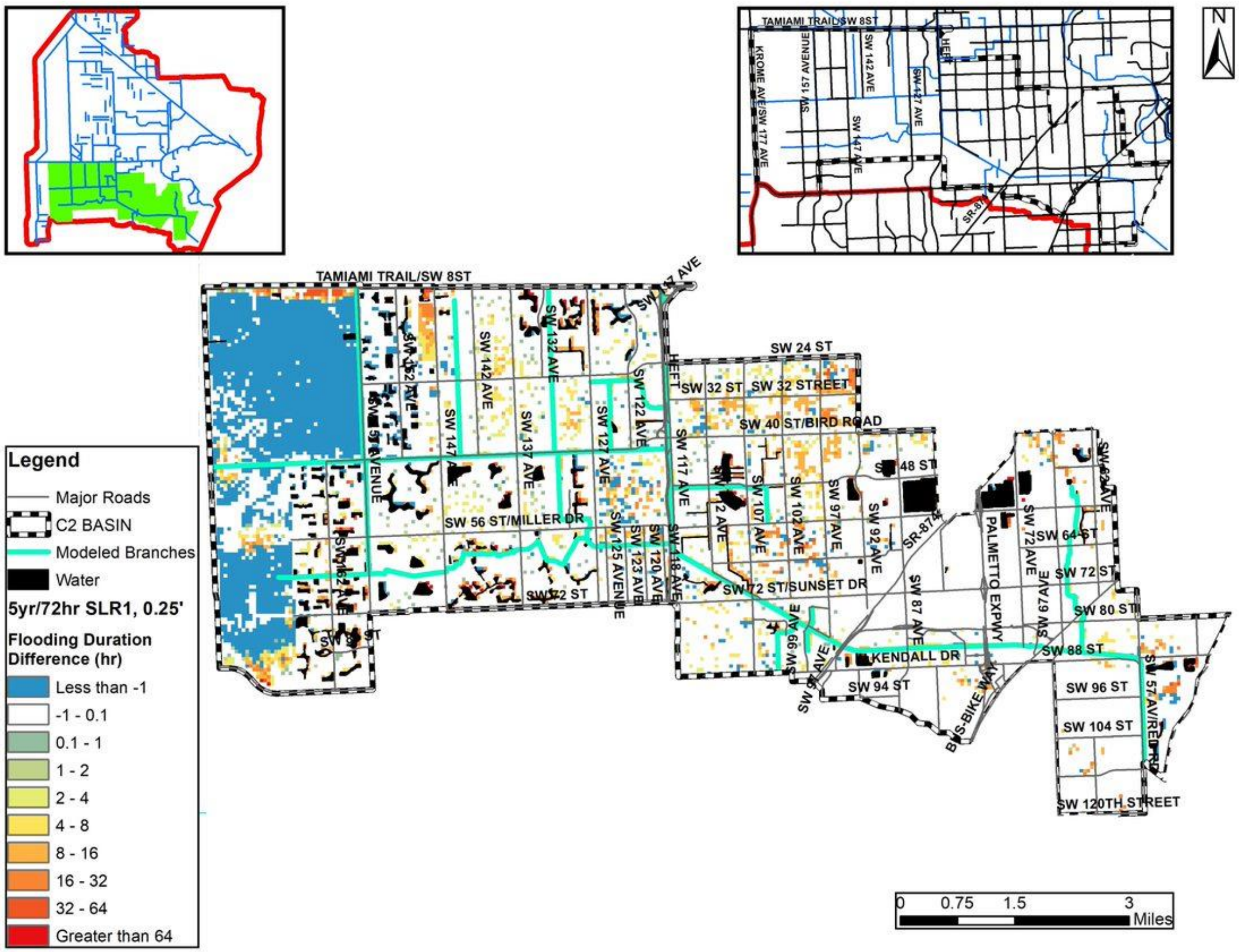


Figure D 1-34. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C2 Watershed

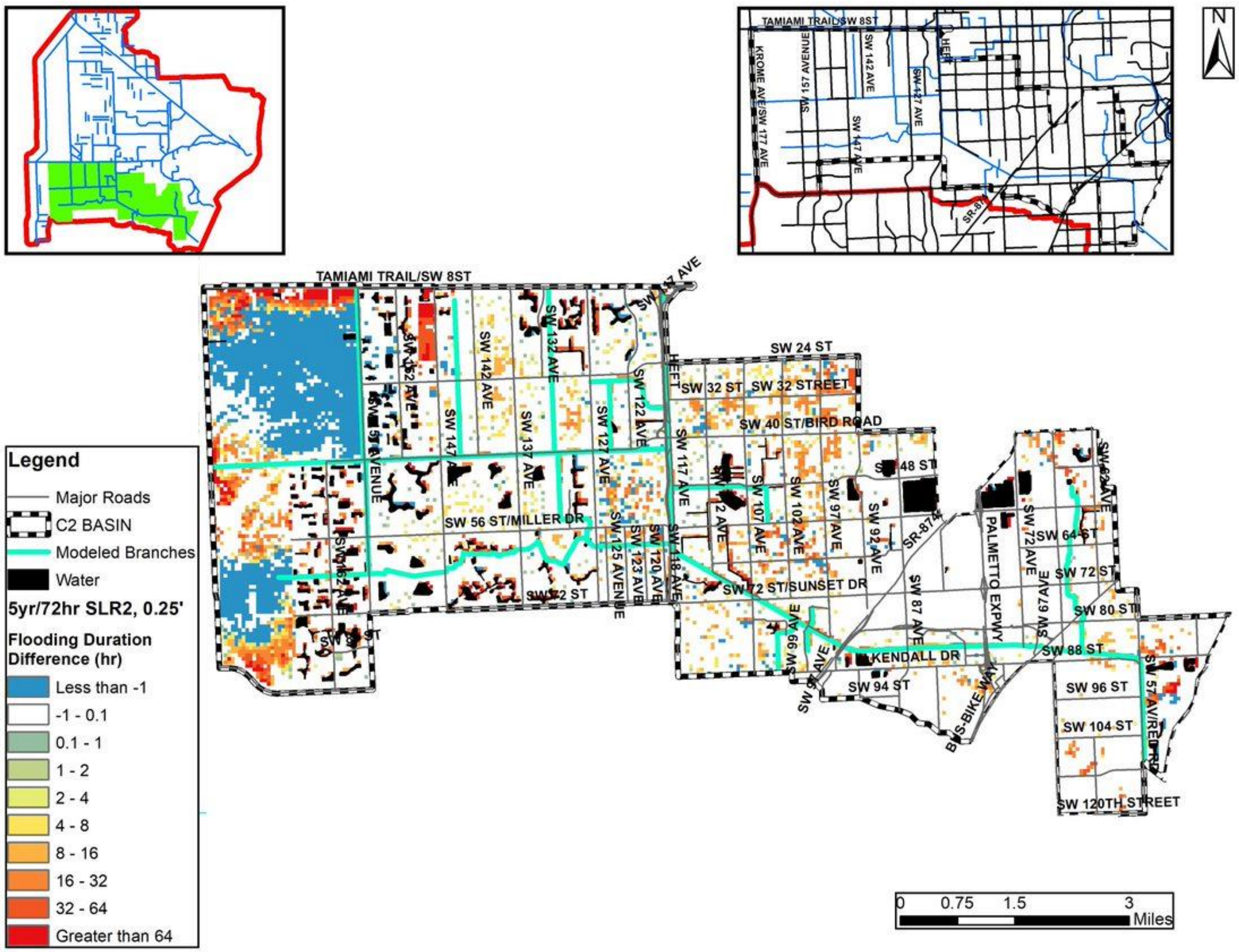


Figure D 1-35. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C2 Watershed

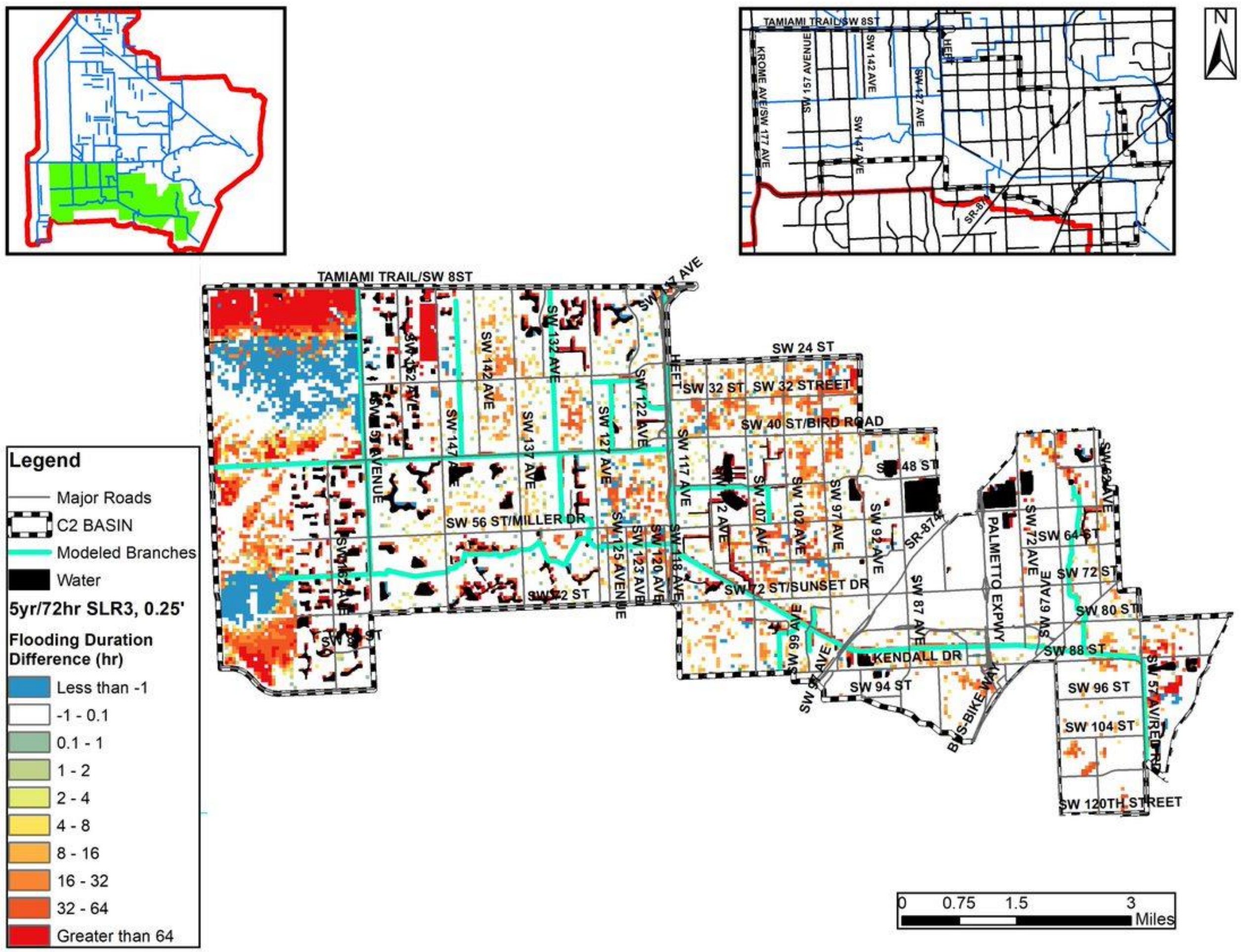


Figure D 1-36. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C2 Watershed

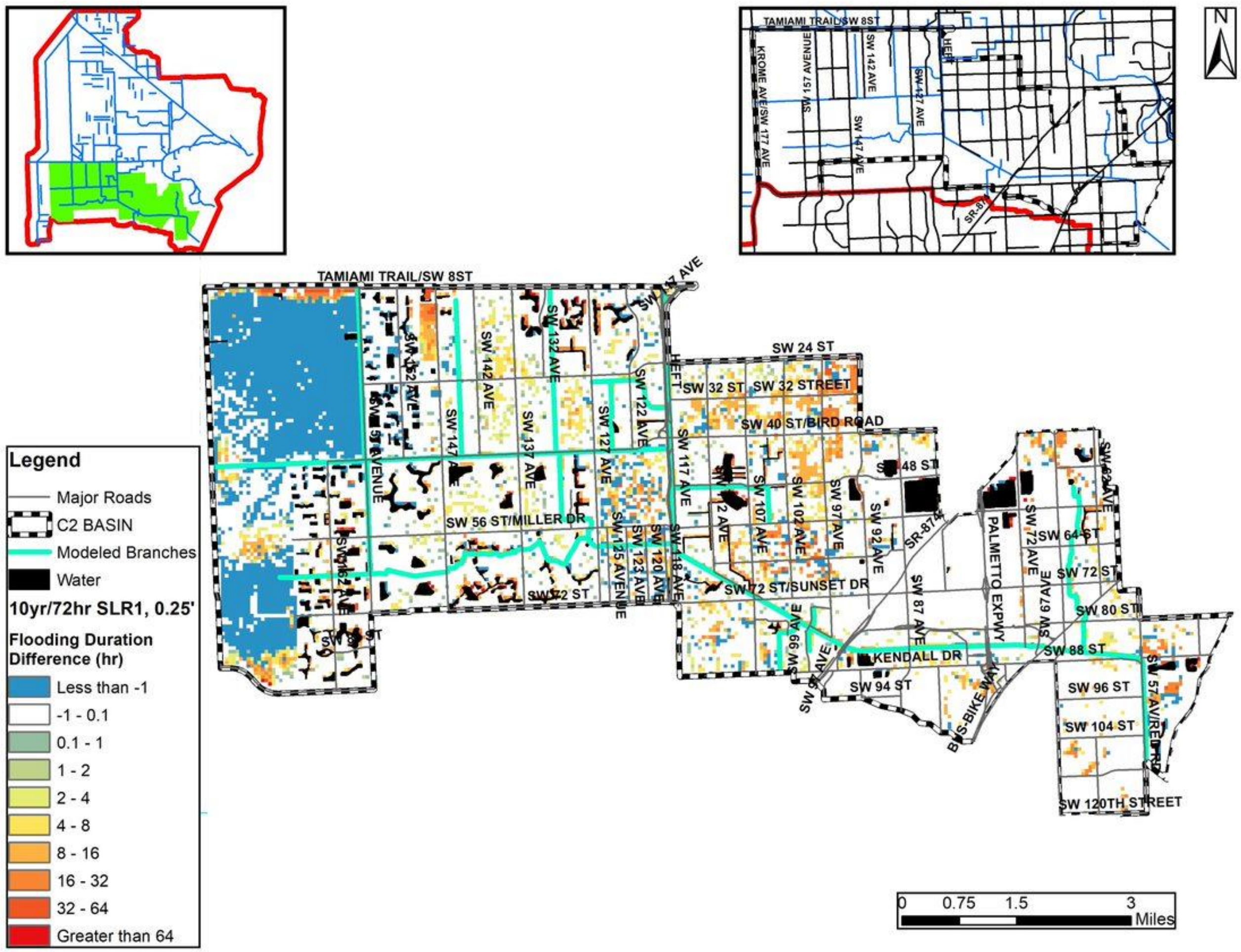


Figure D 1-37. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C2 Watershed

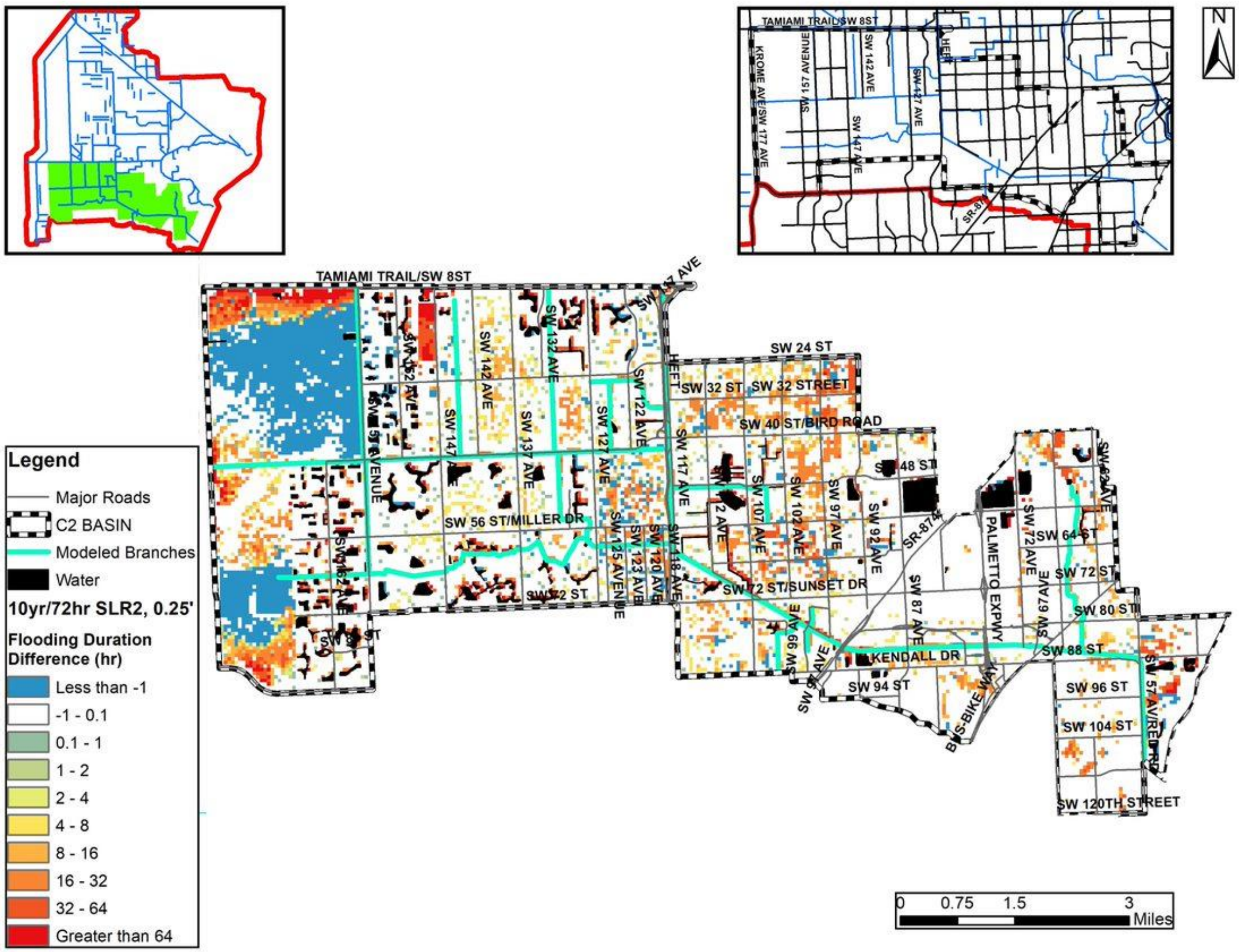


Figure D 1-38. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C2 Watershed

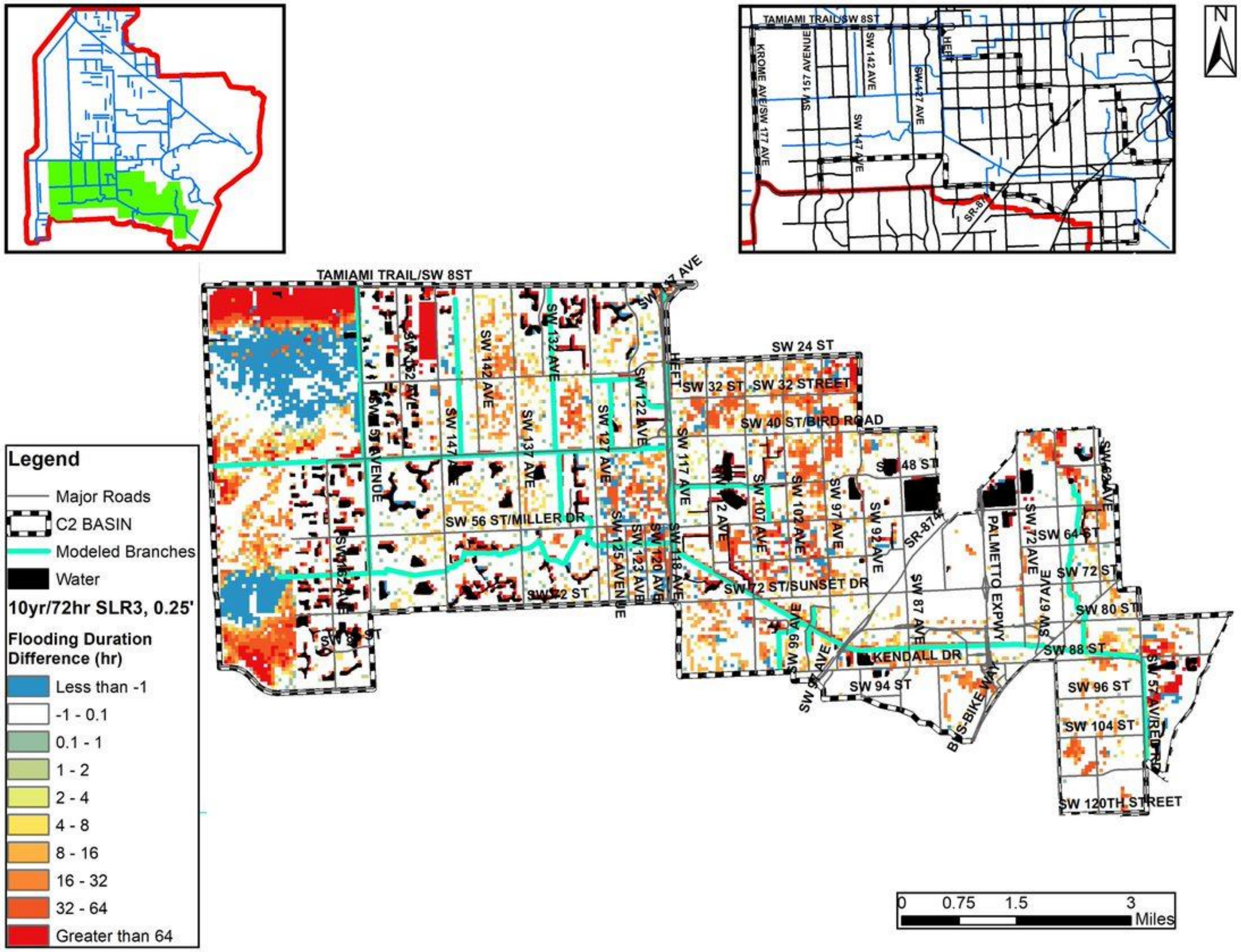


Figure D 1-39. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C2 Watershed

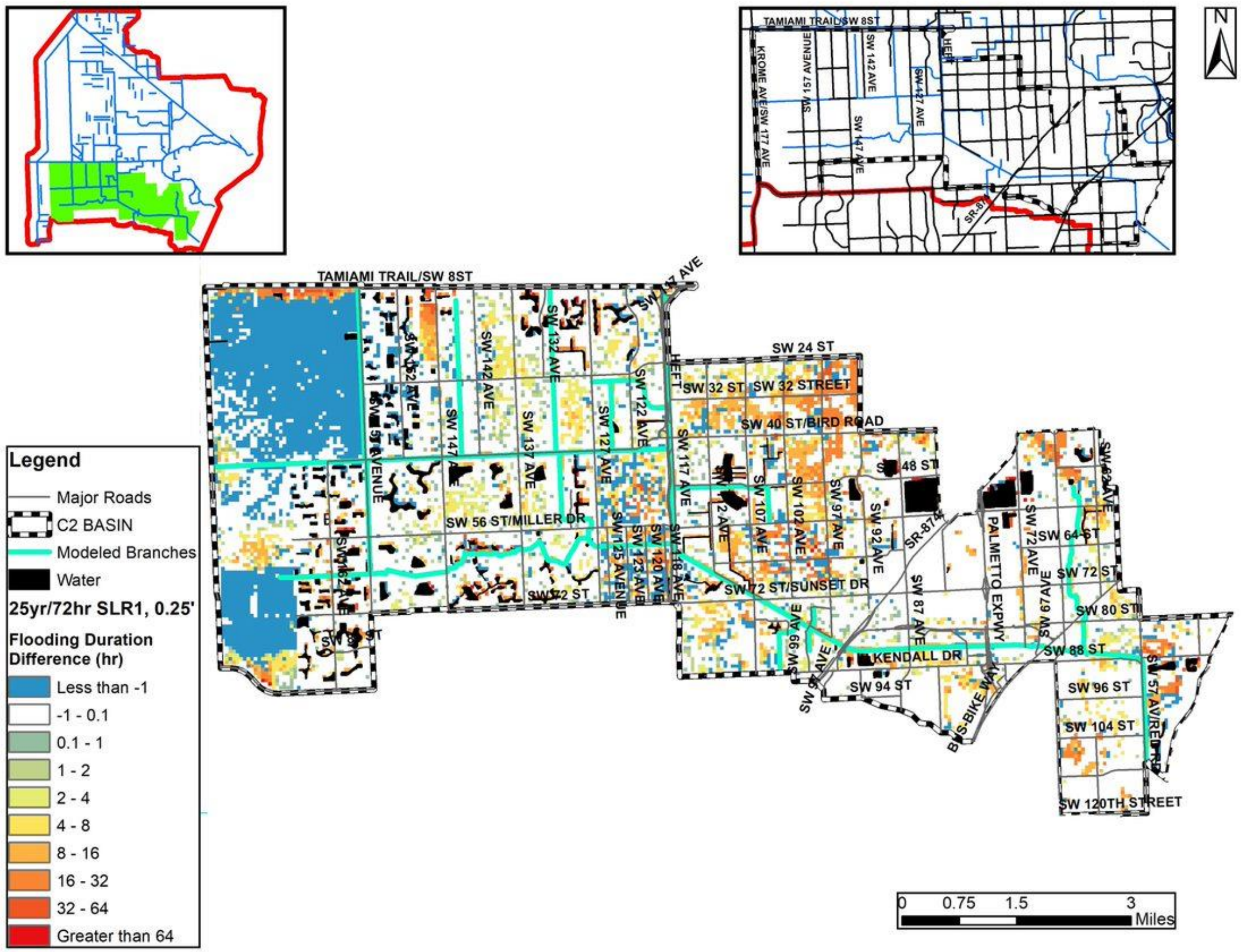


Figure D 1-40. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C2 Watershed

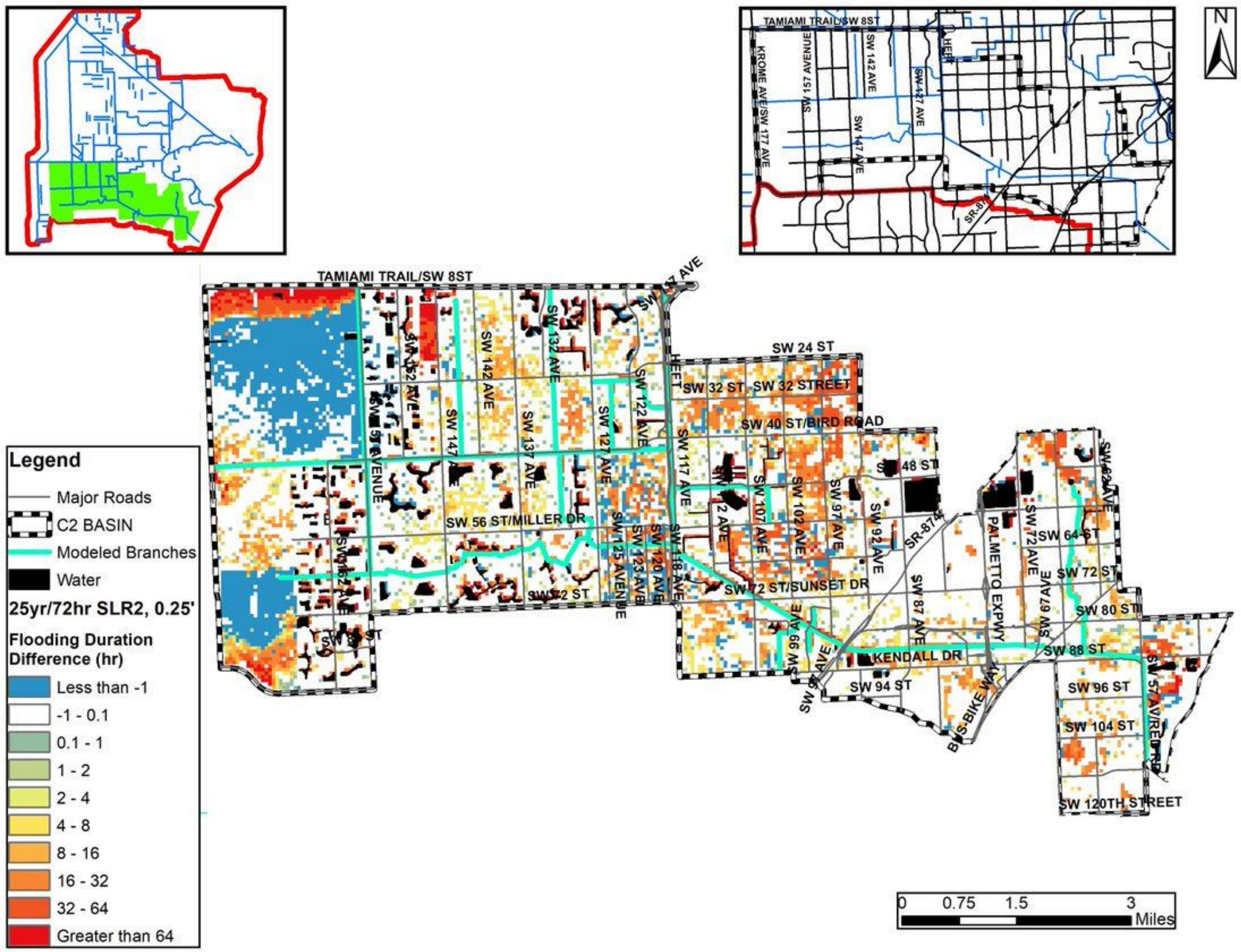


Figure D 1-41. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C2 Watershed

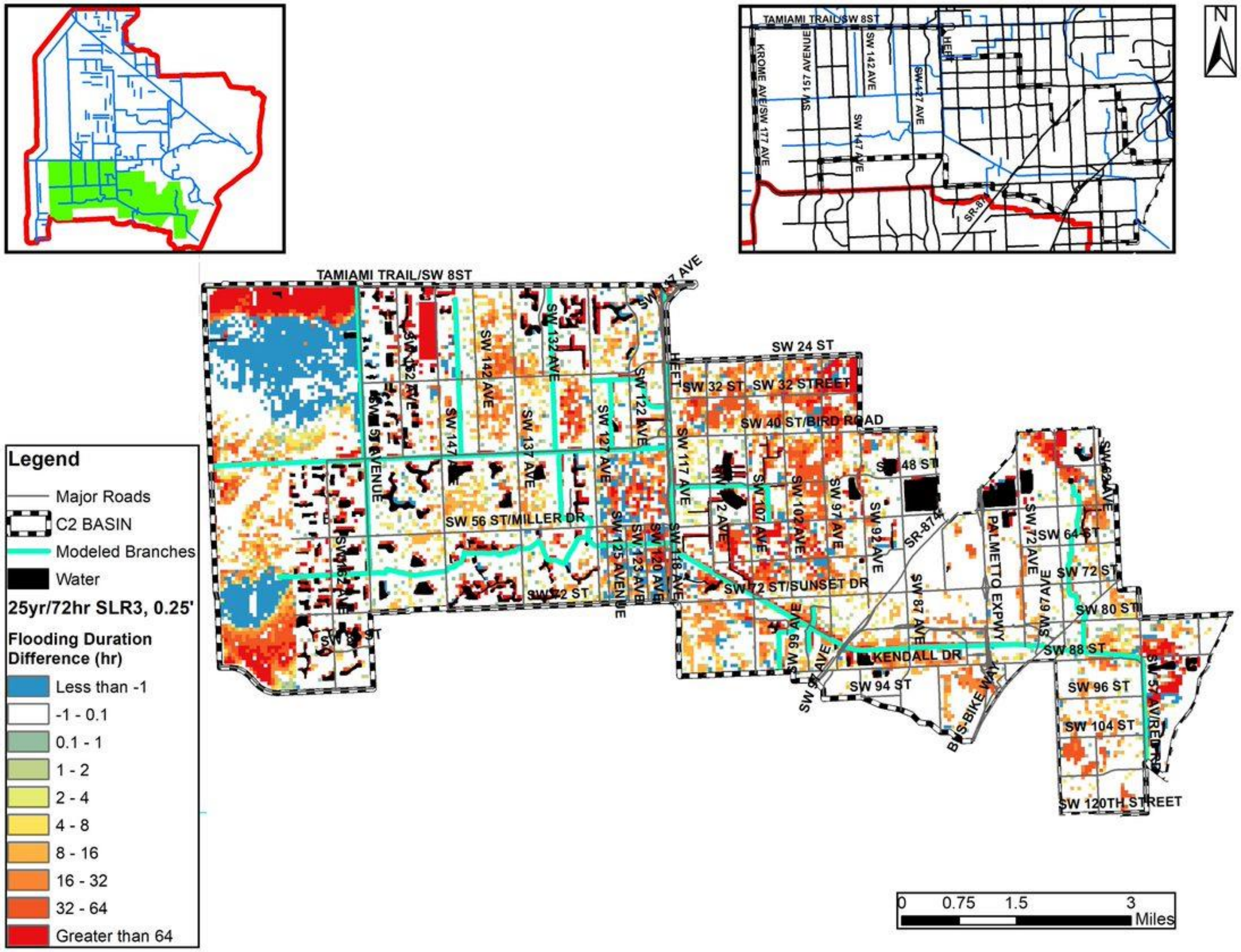


Figure D 1-42. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C2 Watershed

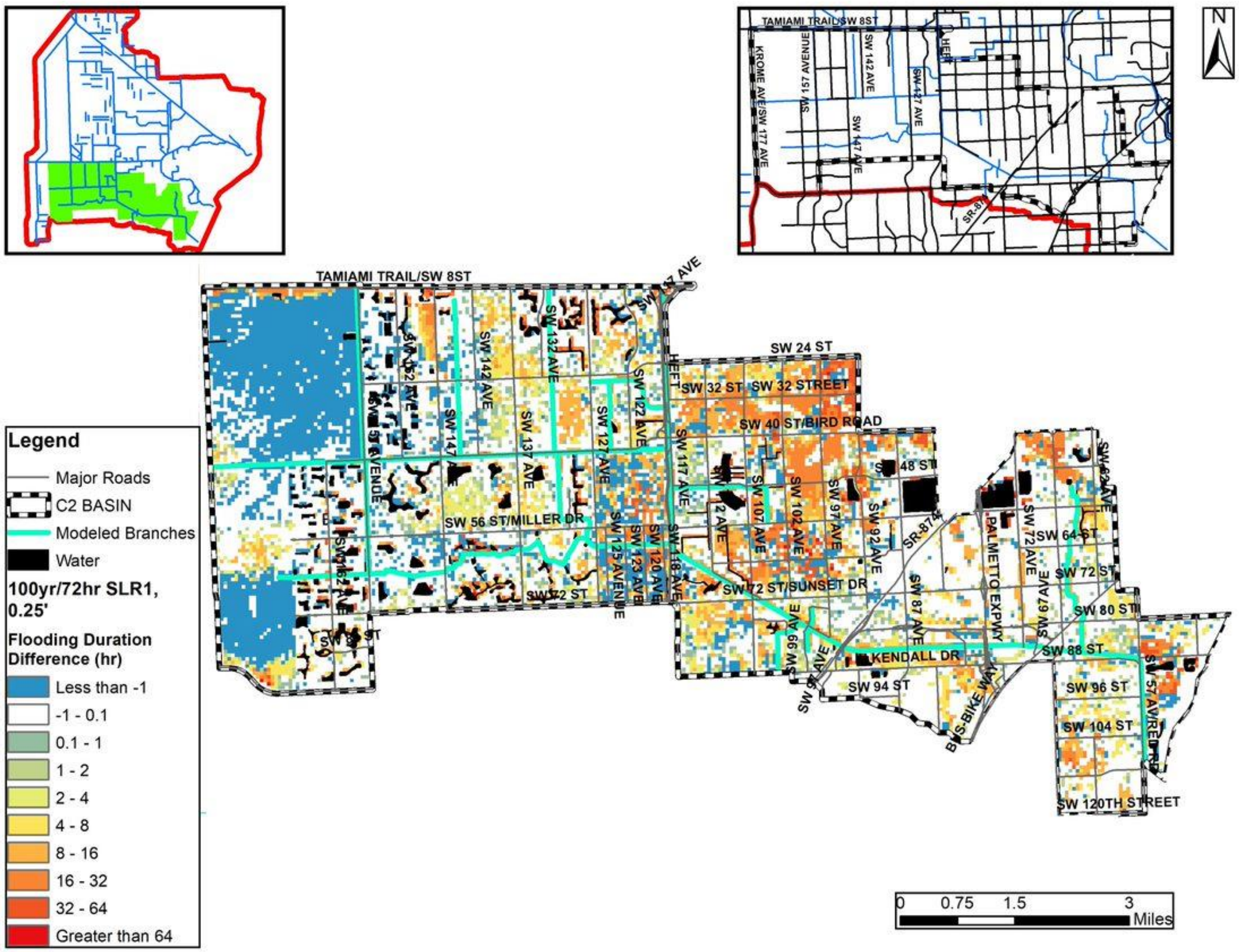


Figure D 1-43. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C2 Watershed

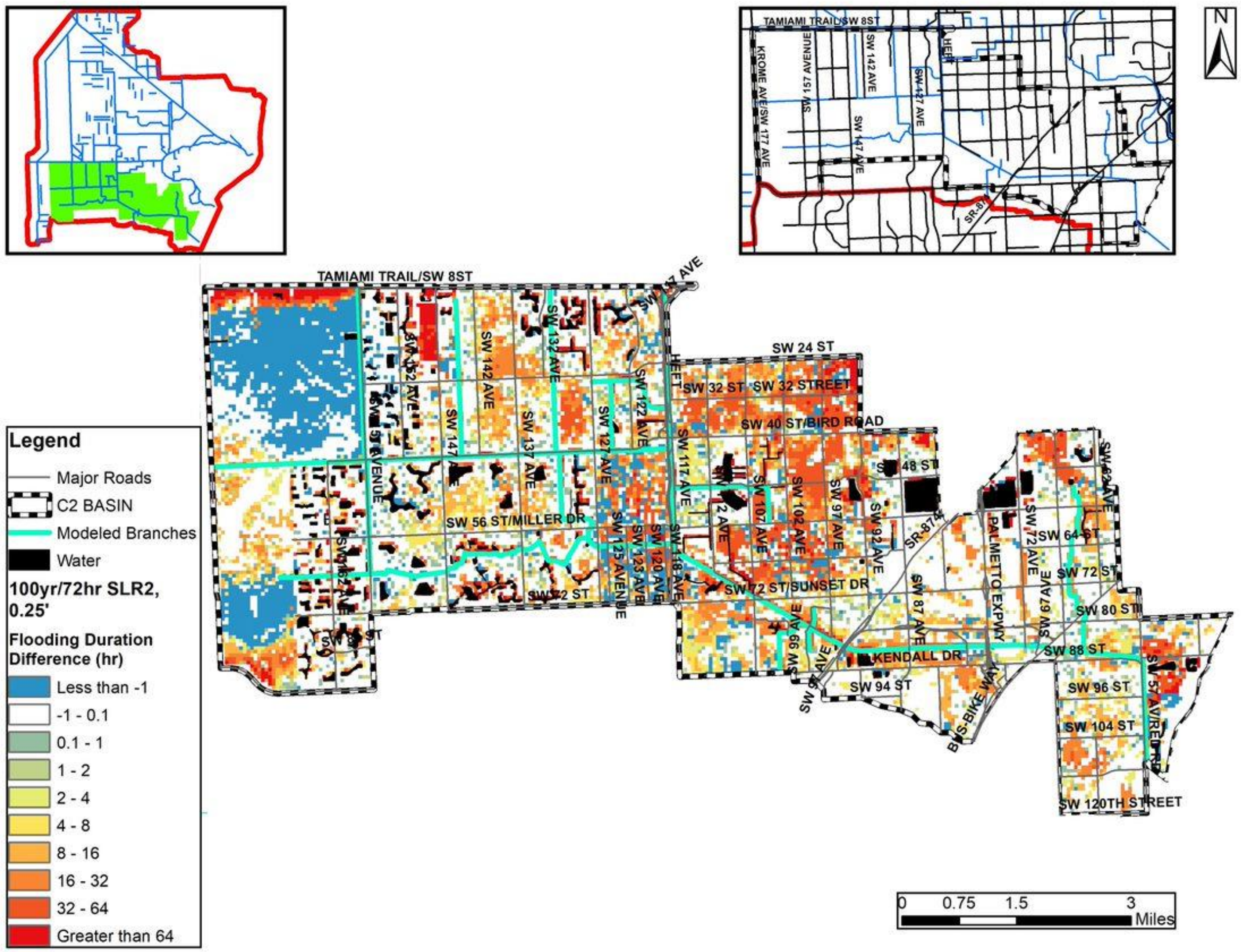


Figure D 1-44. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C2 Watershed

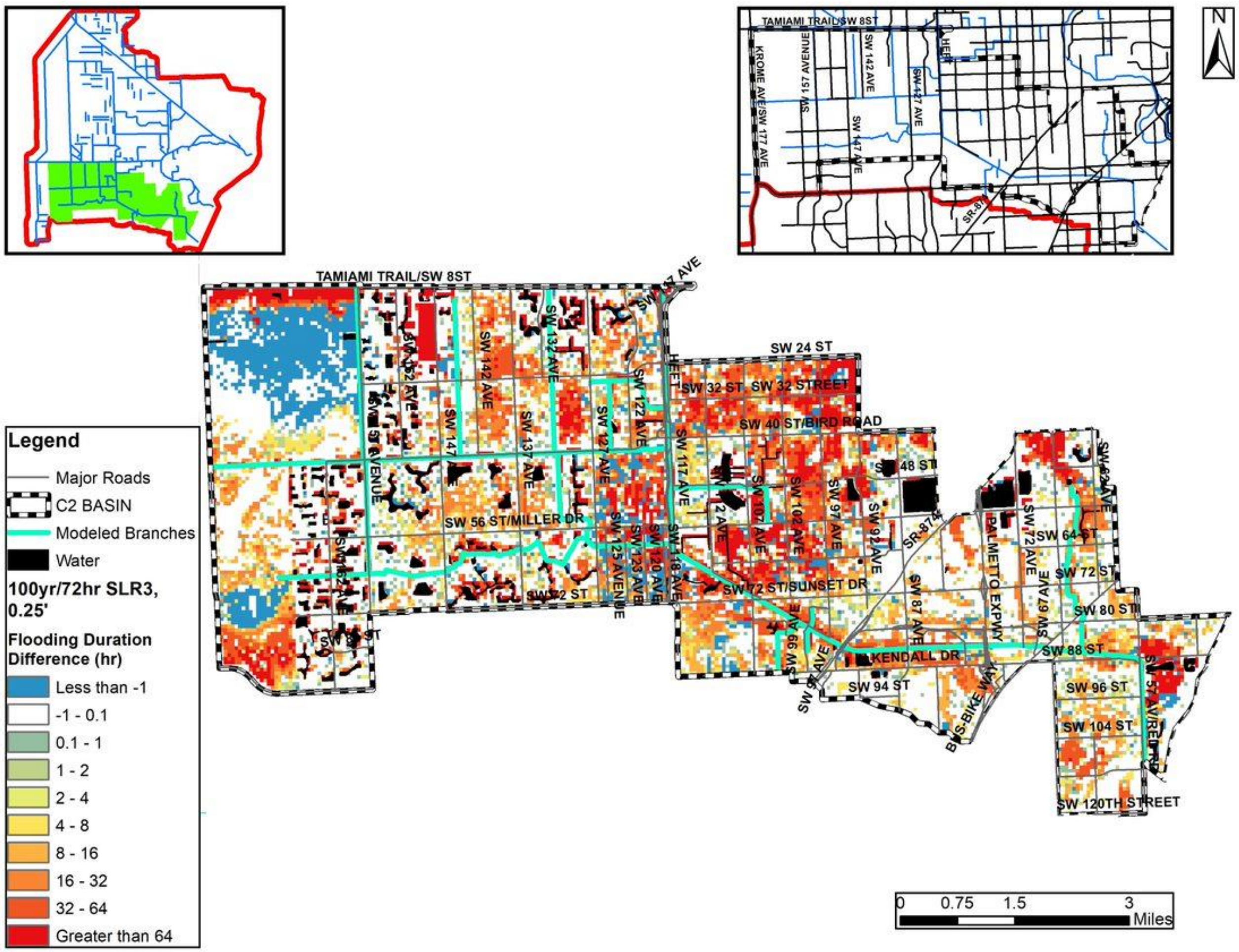


Figure D 1-45. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure D 1-46. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure D 1-47. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure D 1-48. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure D 1-49. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C2 Watershed

Figure D 1-50. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C2 Watershed

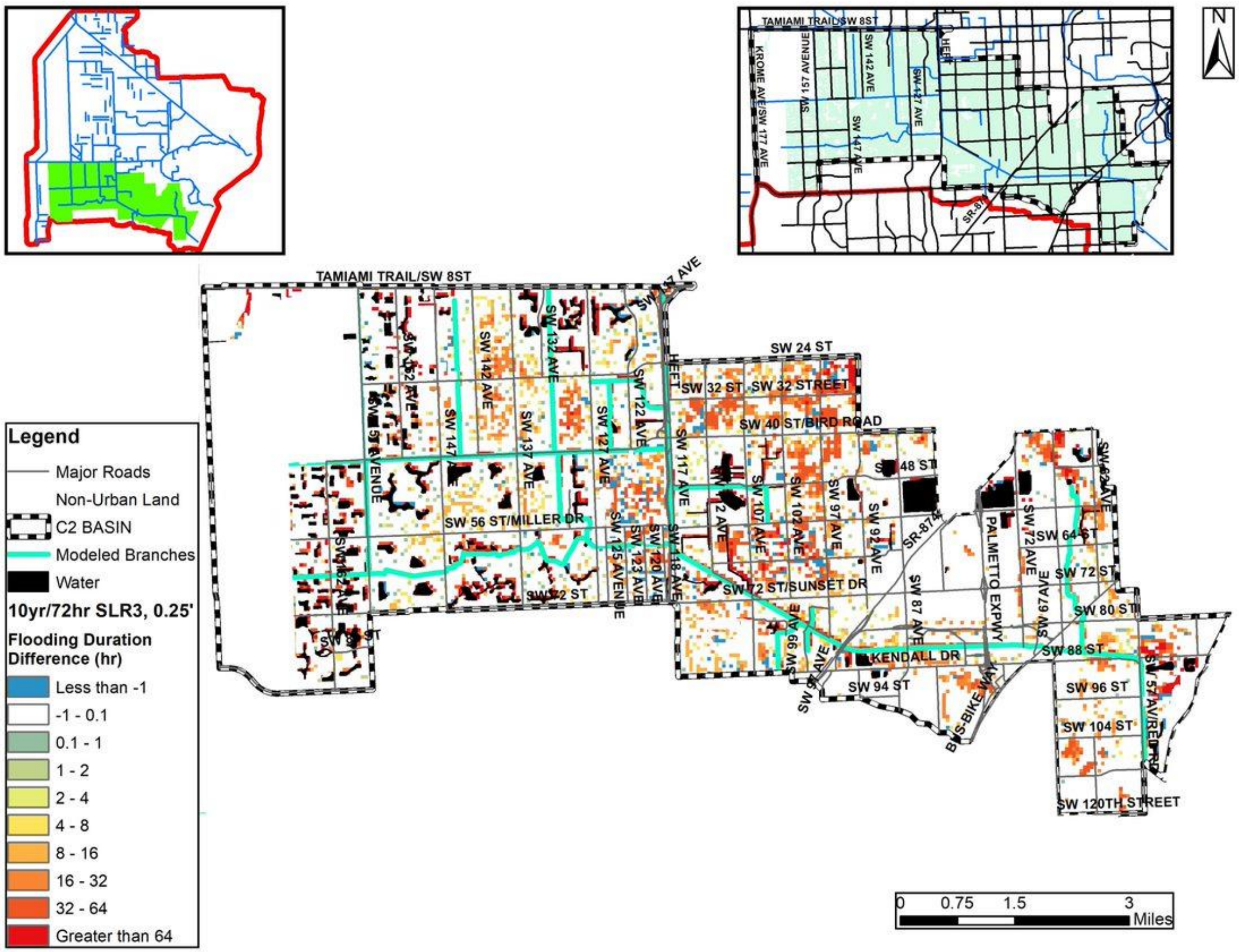


Figure D 1-51. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C2 Watershed

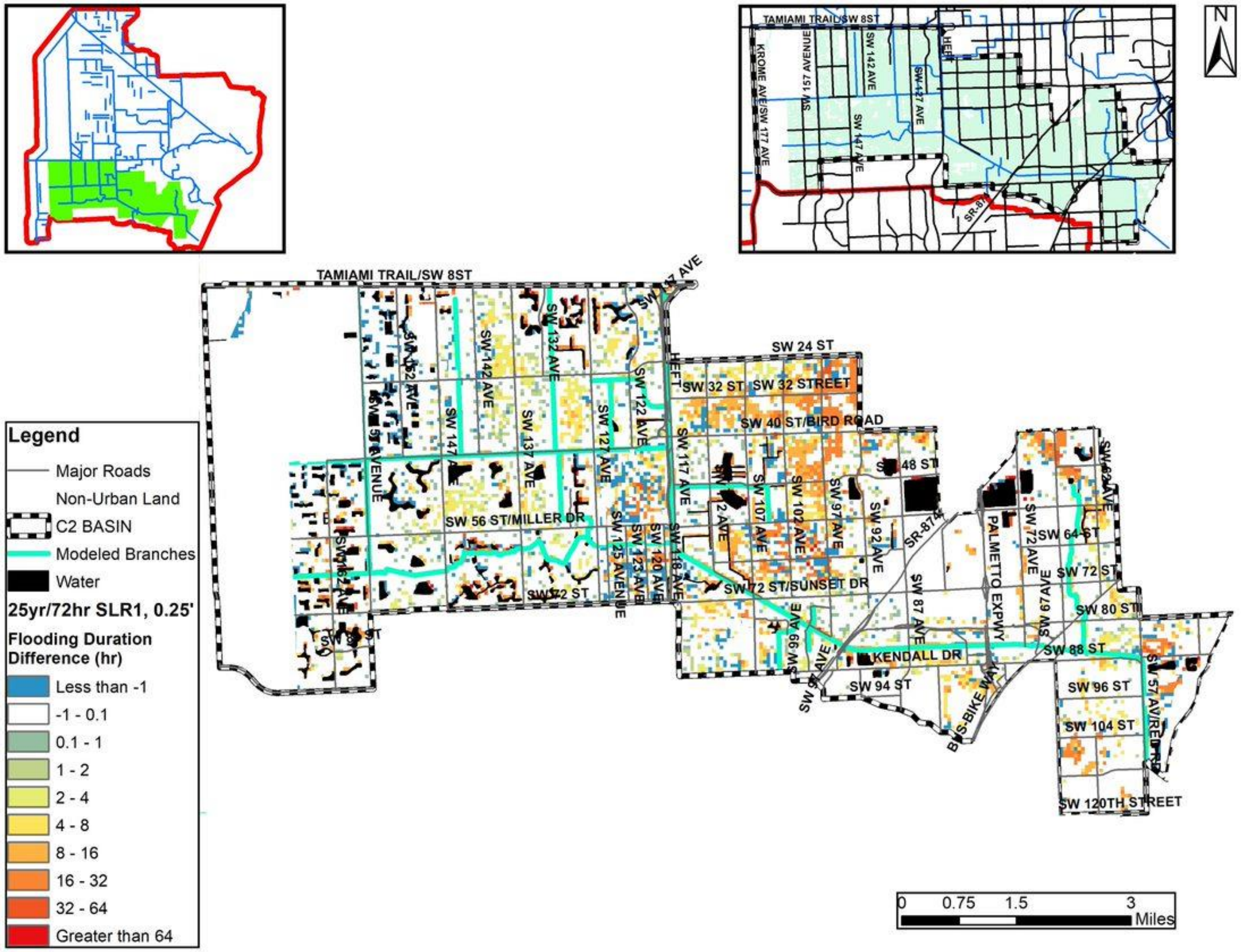


Figure D 1-52. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C2 Watershed

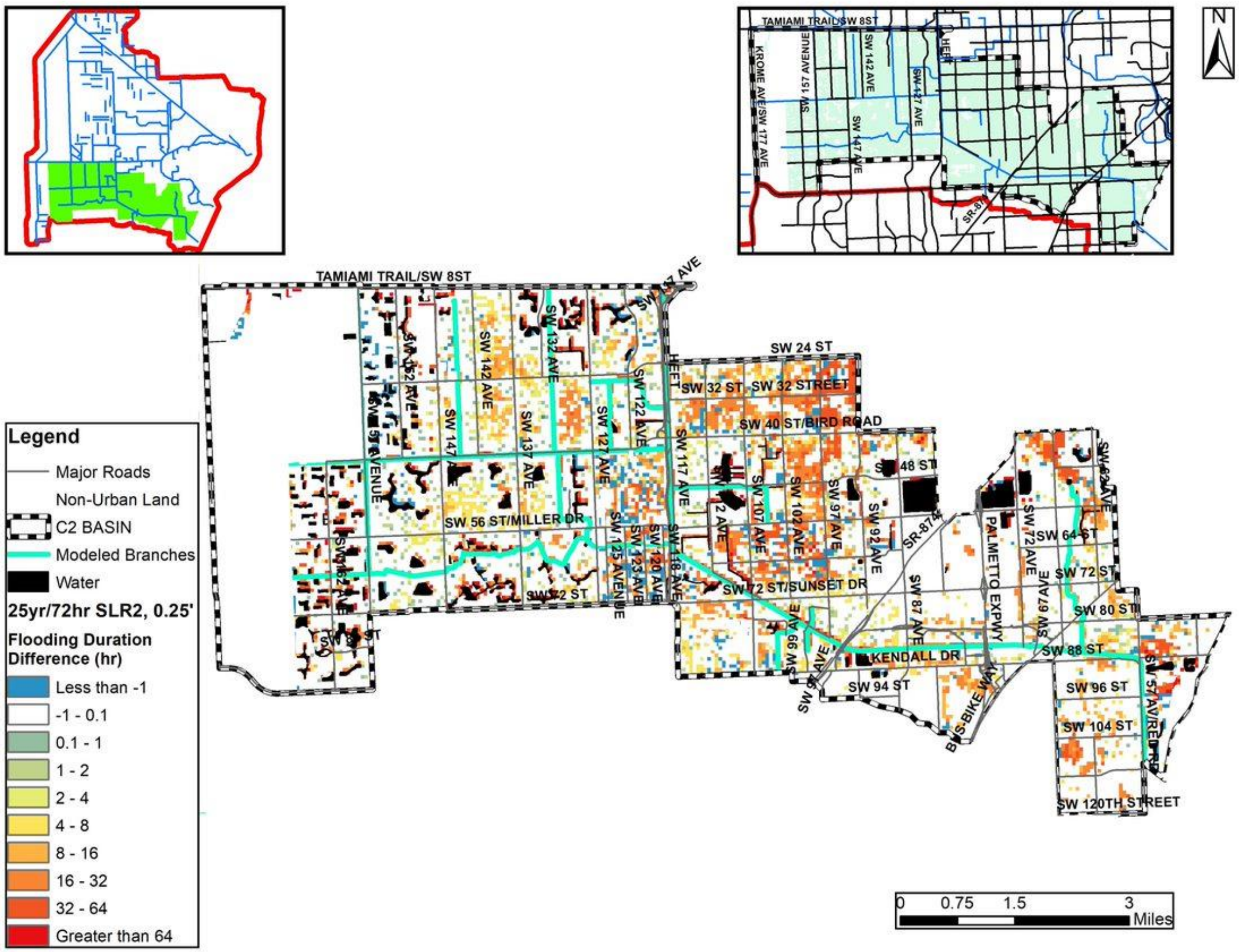


Figure D 1-53. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C2 Watershed

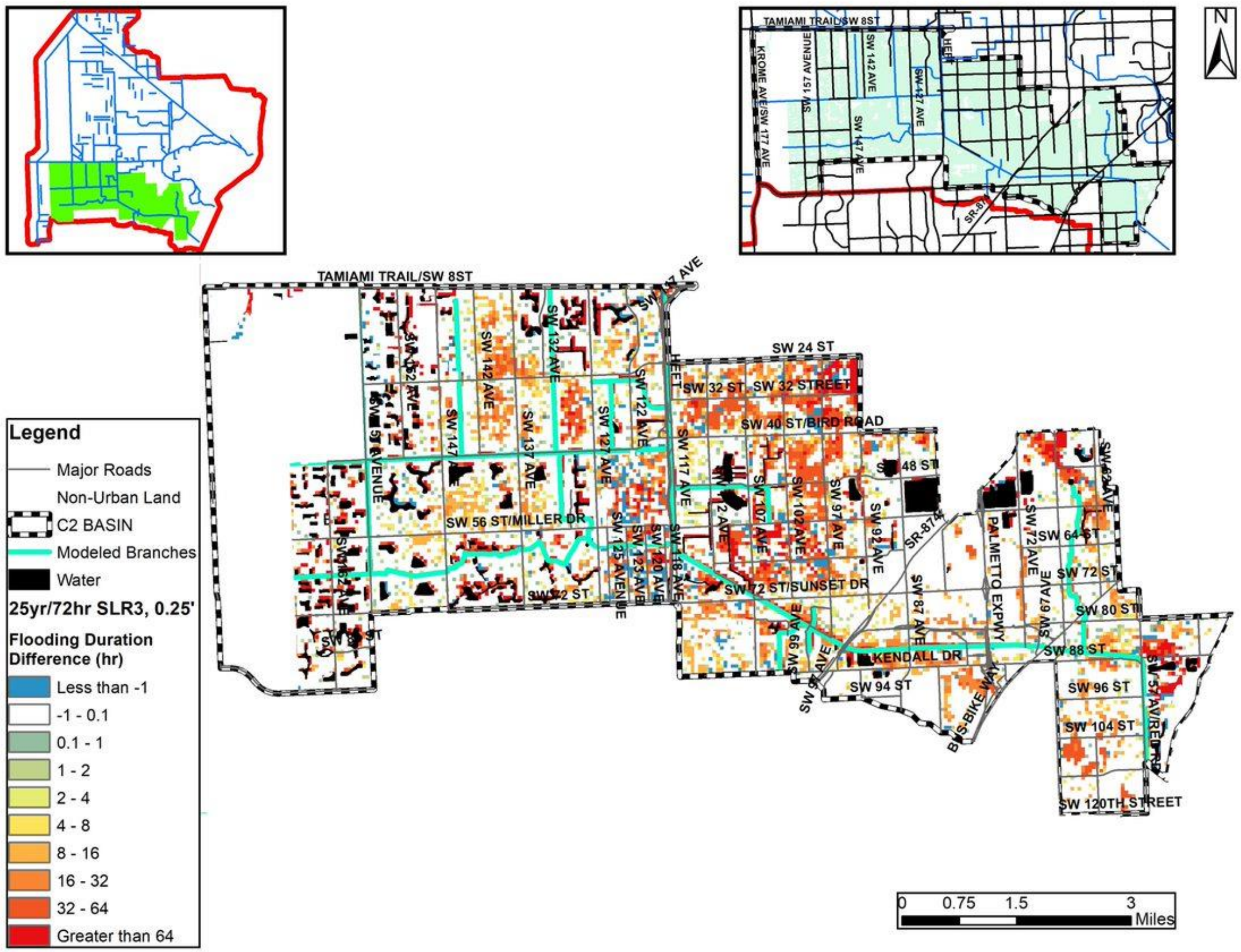


Figure D 1-54. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C2 Watershed

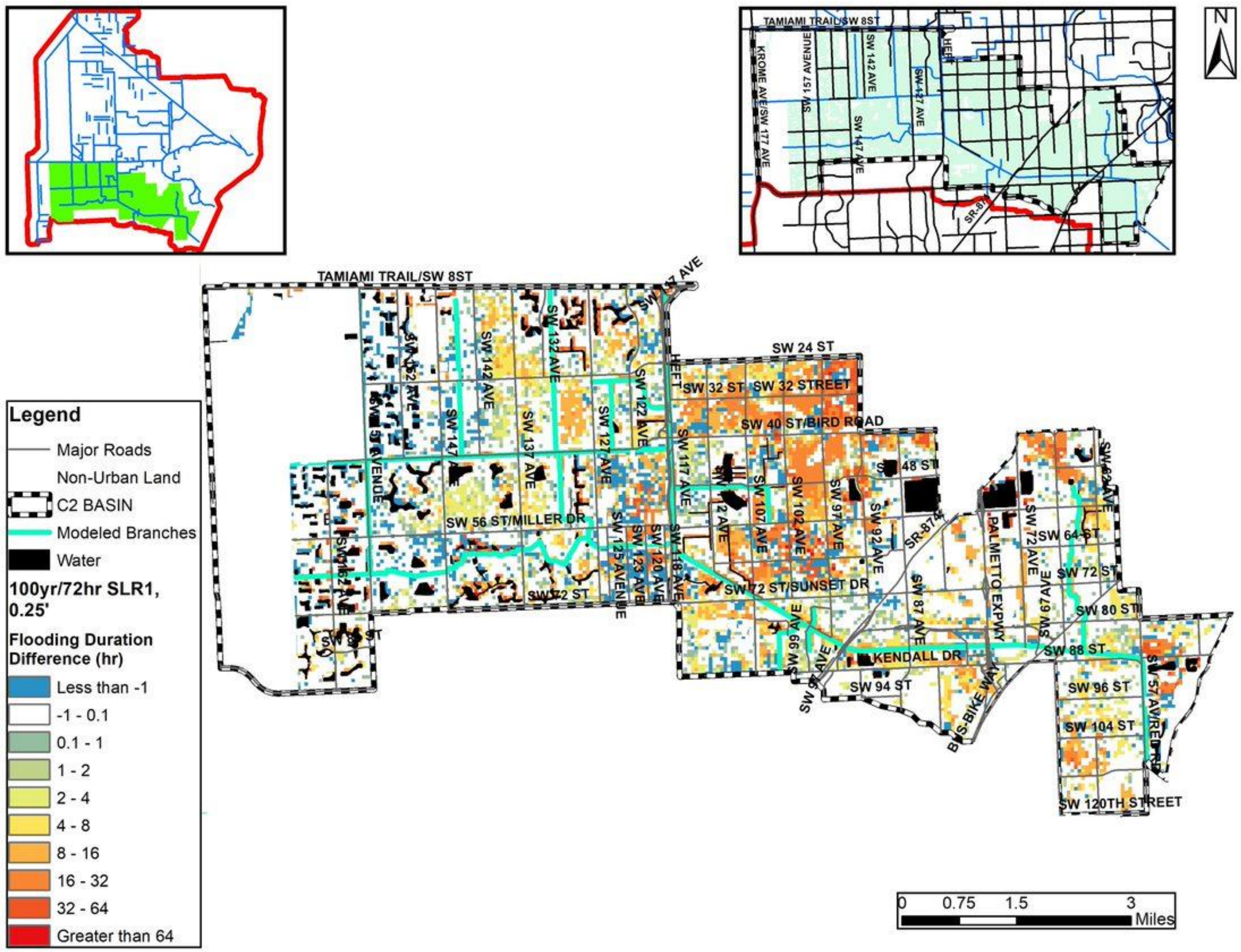


Figure D 1-55. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C2 Watershed

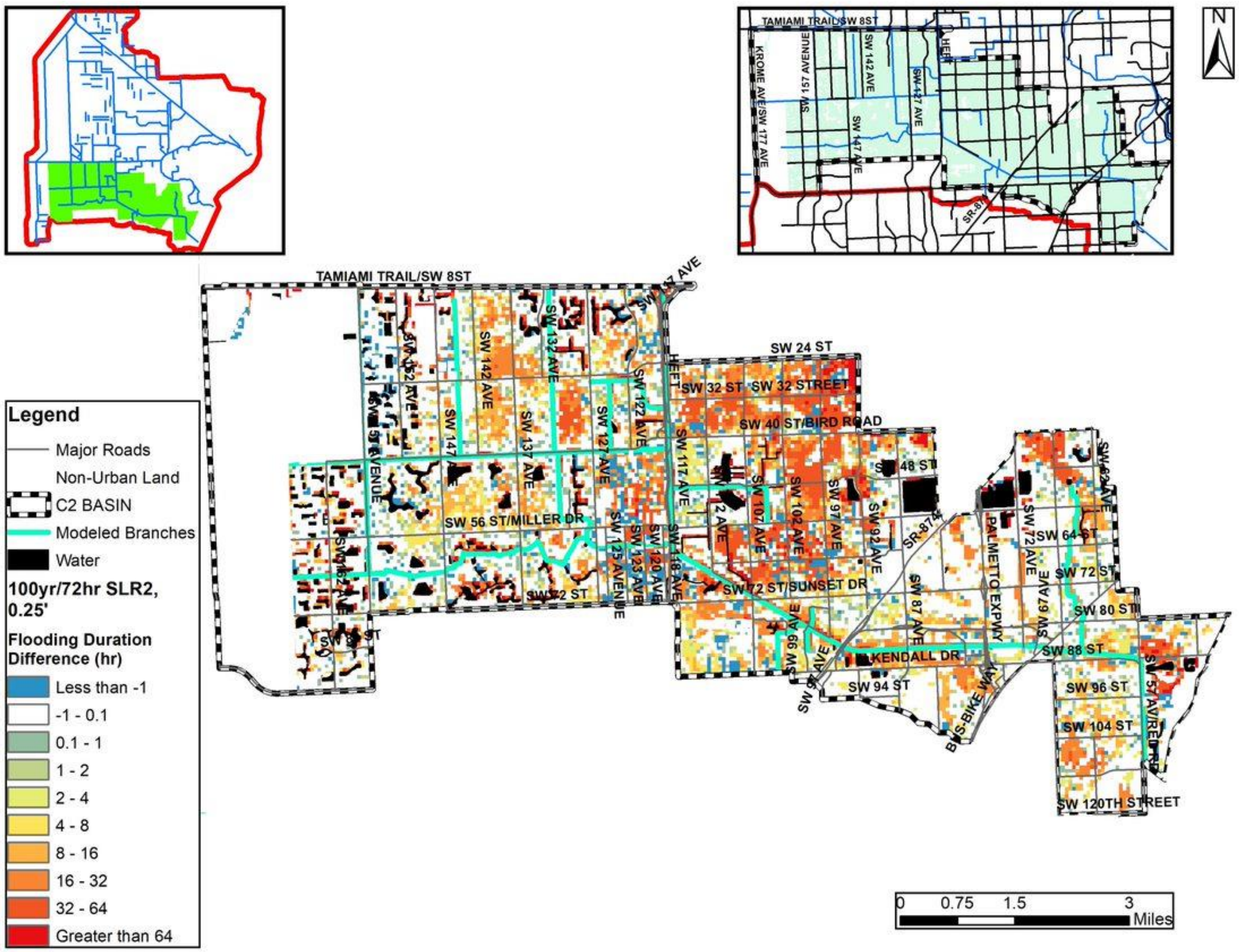
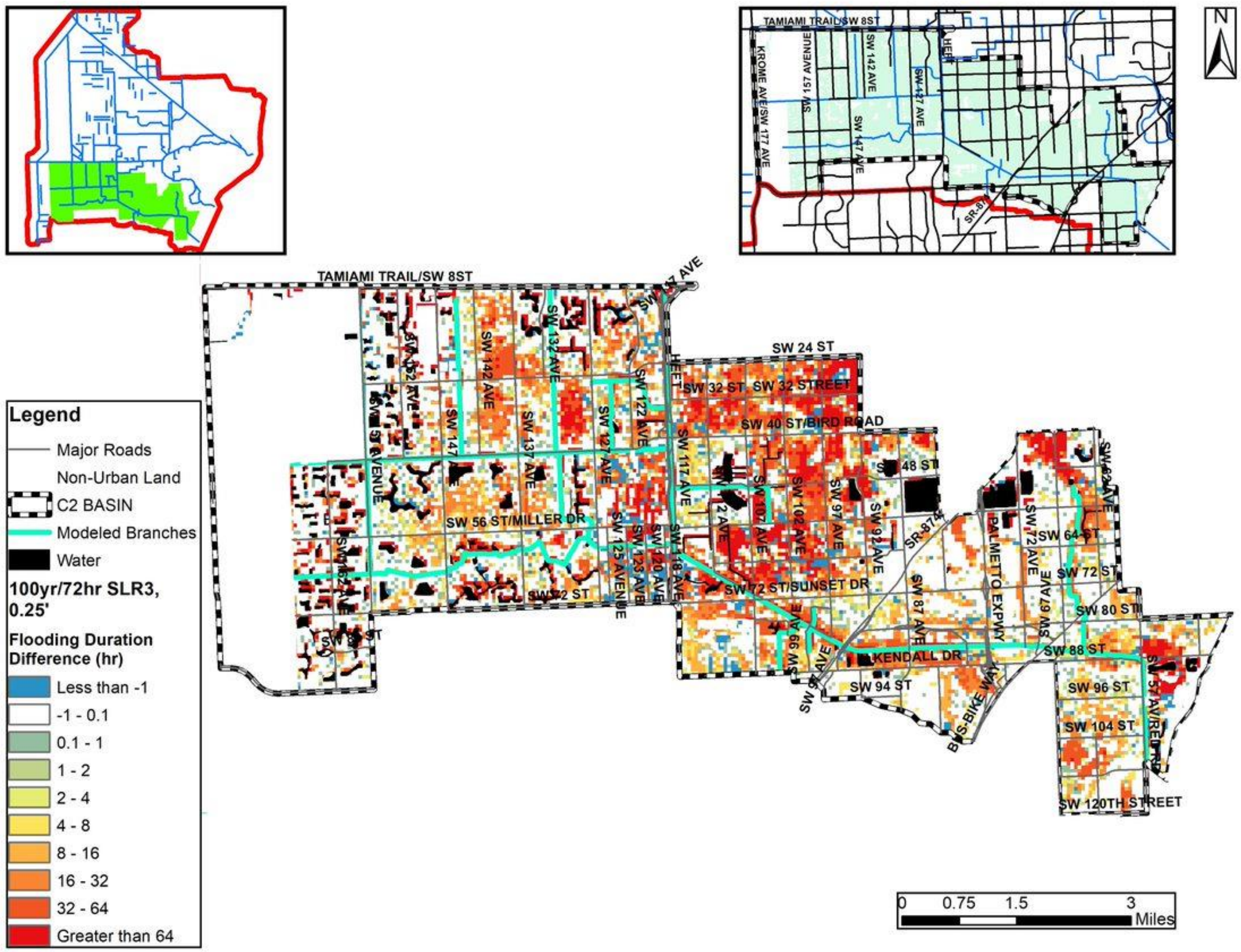


Figure D 1-56. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C2 Watershed



2 C3W Watershed

Figure D 2-1 through **Figure D 2-16** provides flood duration maps for the C3W Watershed for overland flooding depths exceeding 0.25 ft for the 5-, 10-, 25-, and 100-year 3-day design storms, respectively, for each SLR condition. Water areas are masked in black.

Figure D 2-17 through **Figure D 2-28** show the difference in overland flooding for the C3W Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively.

Figure D 2-1. Flood Duration Map for the 5-year 3-Day Design Storm for Current Conditions in the C3W Watershed

Figure D 2-2. Flood Duration Map for the 5-year 3-Day Design Storm for SLR1 in the C3W Watershed

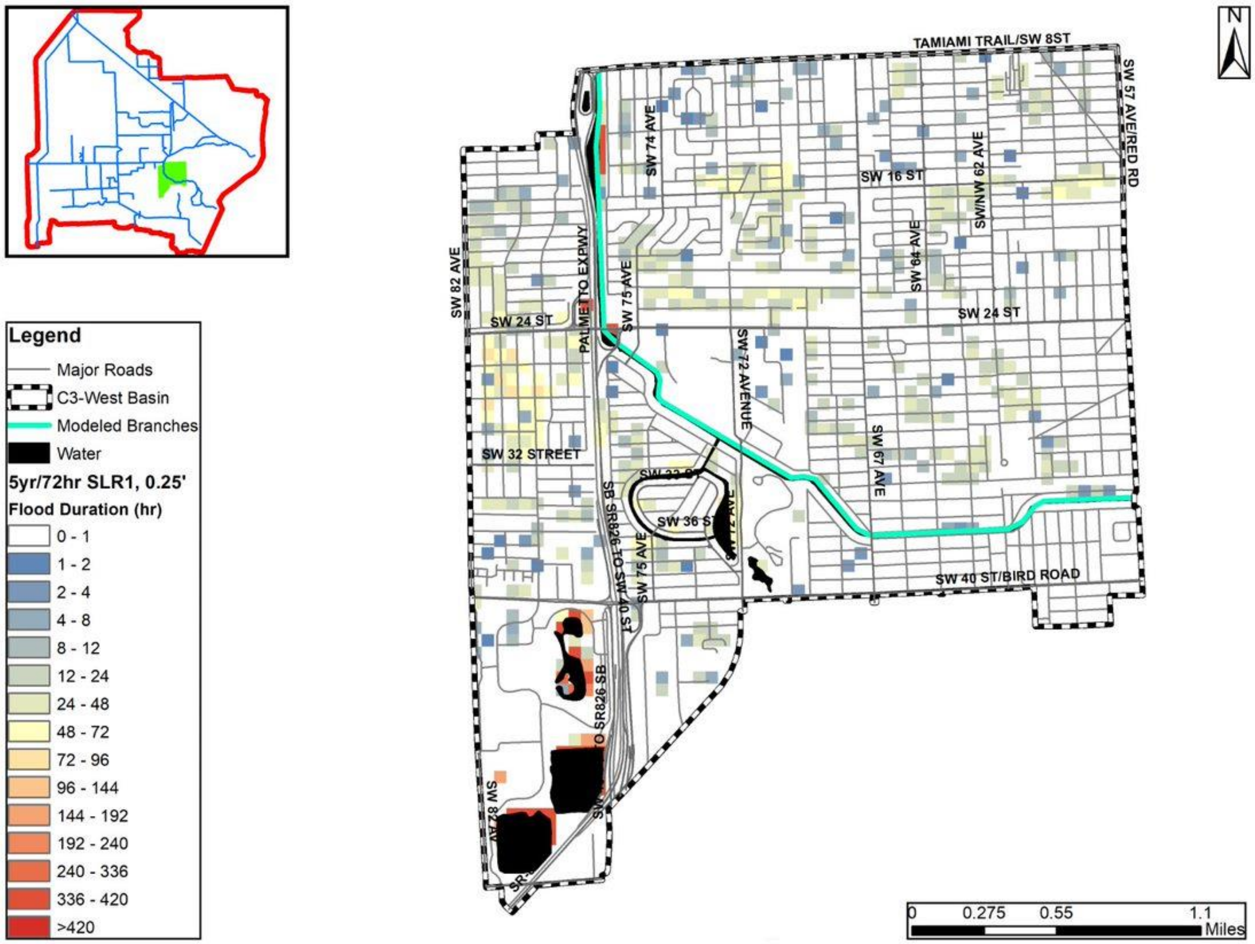


Figure D 2-3. Flood Duration Map for the 5-year 3-Day Design Storm for SLR2 in the C3W Watershed

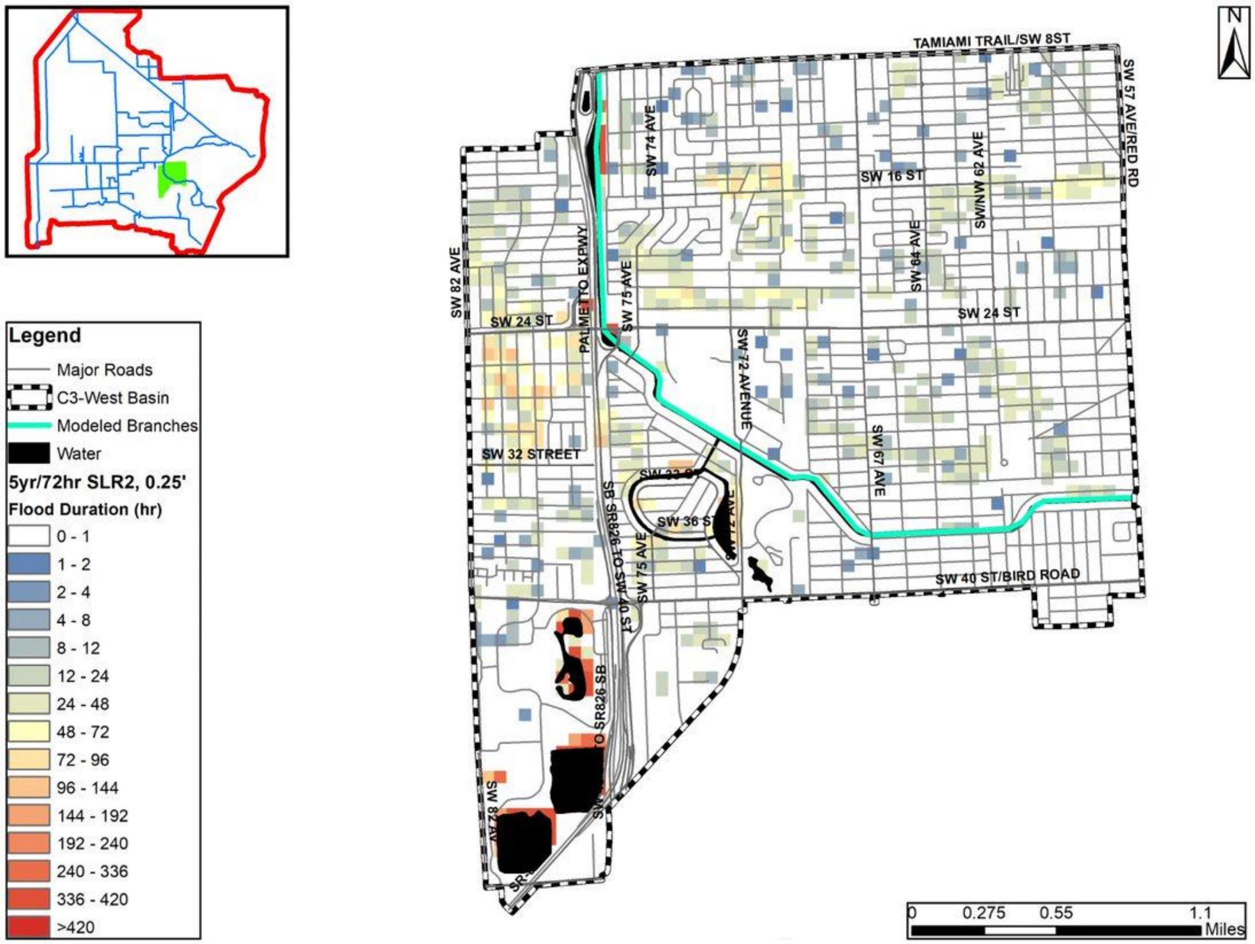


Figure D 2-4. Flood Duration Map for the 5-year 3-Day Design Storm for SLR3 in the C3W Watershed

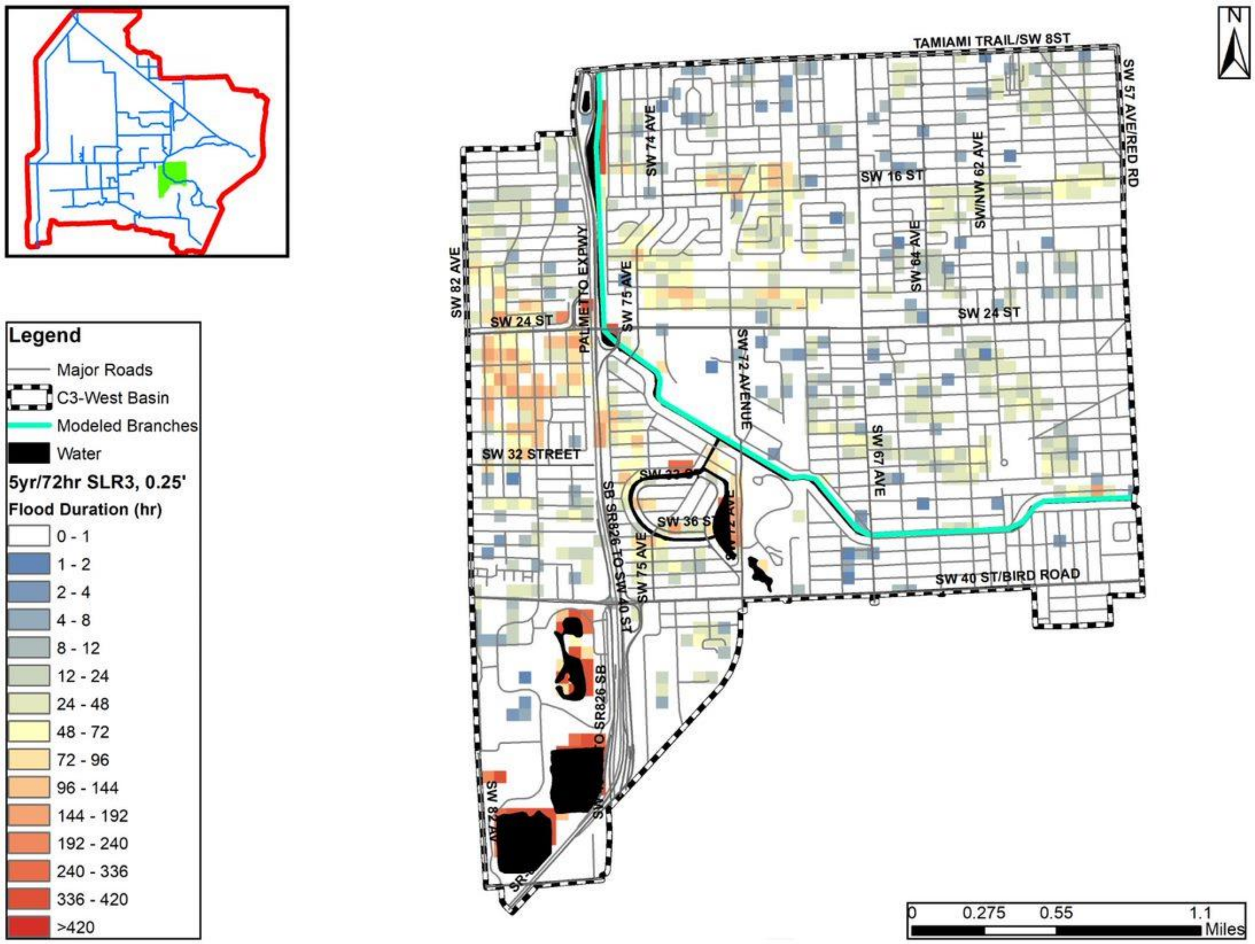


Figure D 2-5. Flood Duration Map for the 10-year 3-Day Design Storm for Current Conditions in the C3W Watershed

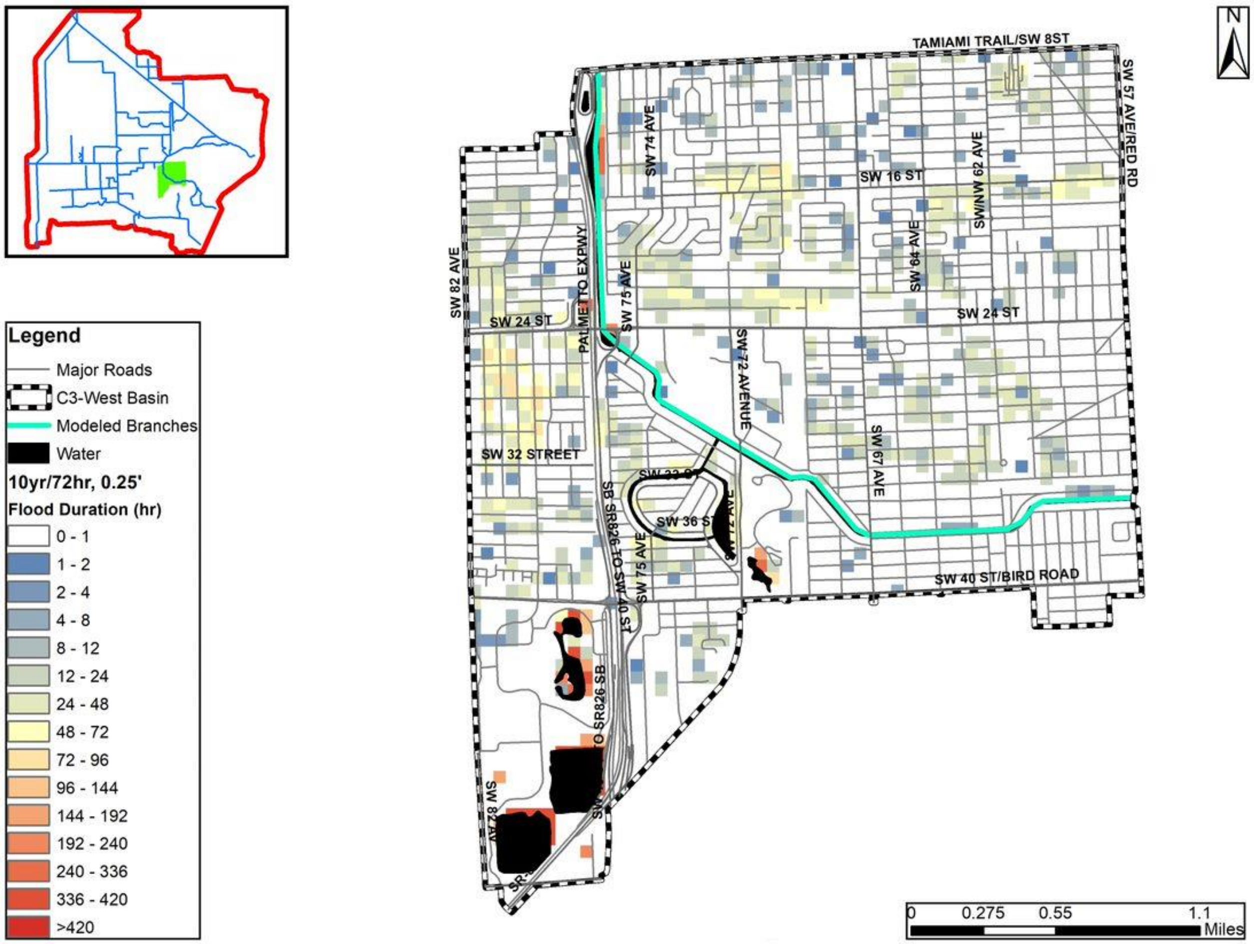


Figure D 2-6. Flood Duration Map for the 10-year 3-Day Design Storm for SLR1 in the C3W Watershed

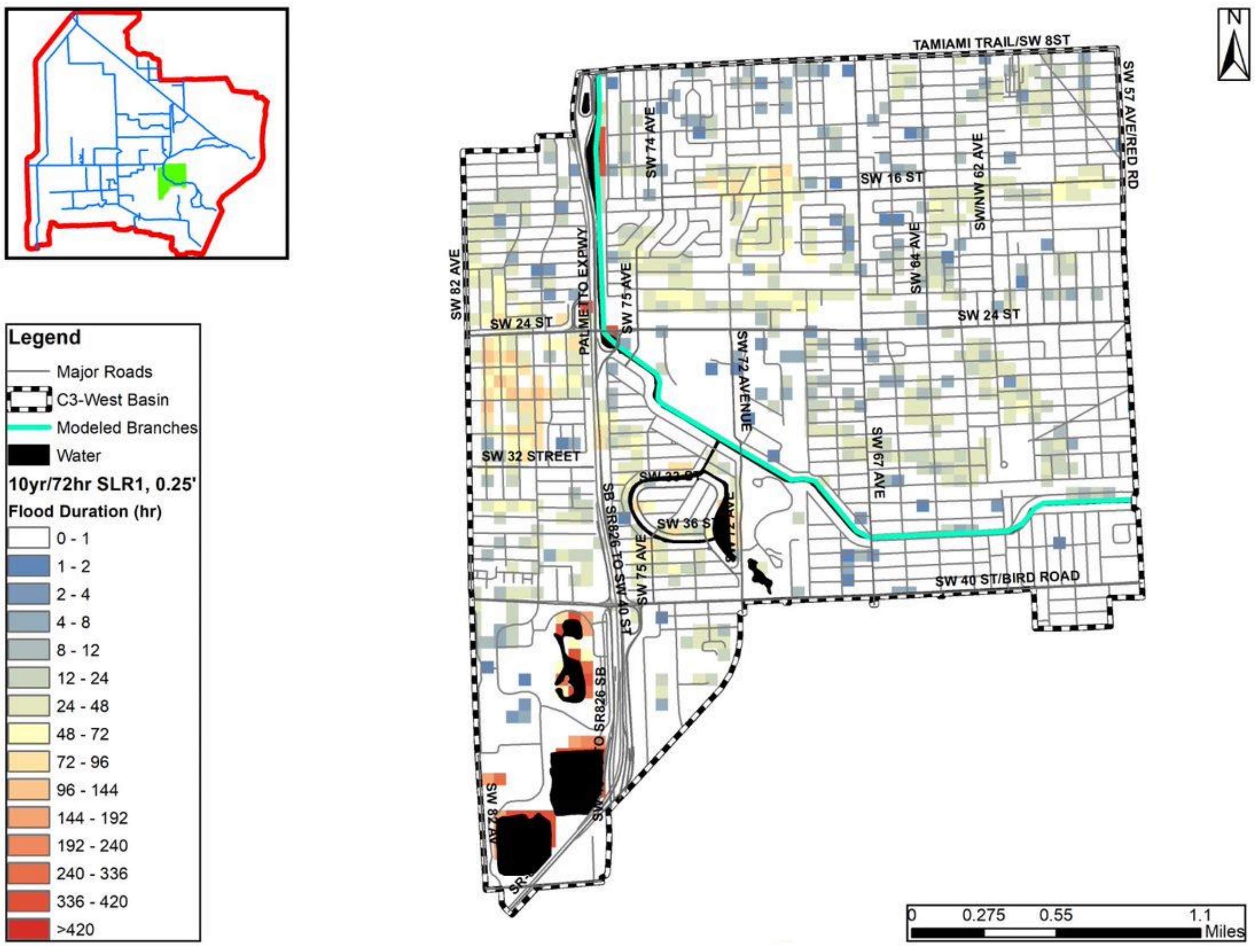


Figure D 2-7. Flood Duration Map for the 10-year 3-Day Design Storm for SLR2 in the C3W Watershed

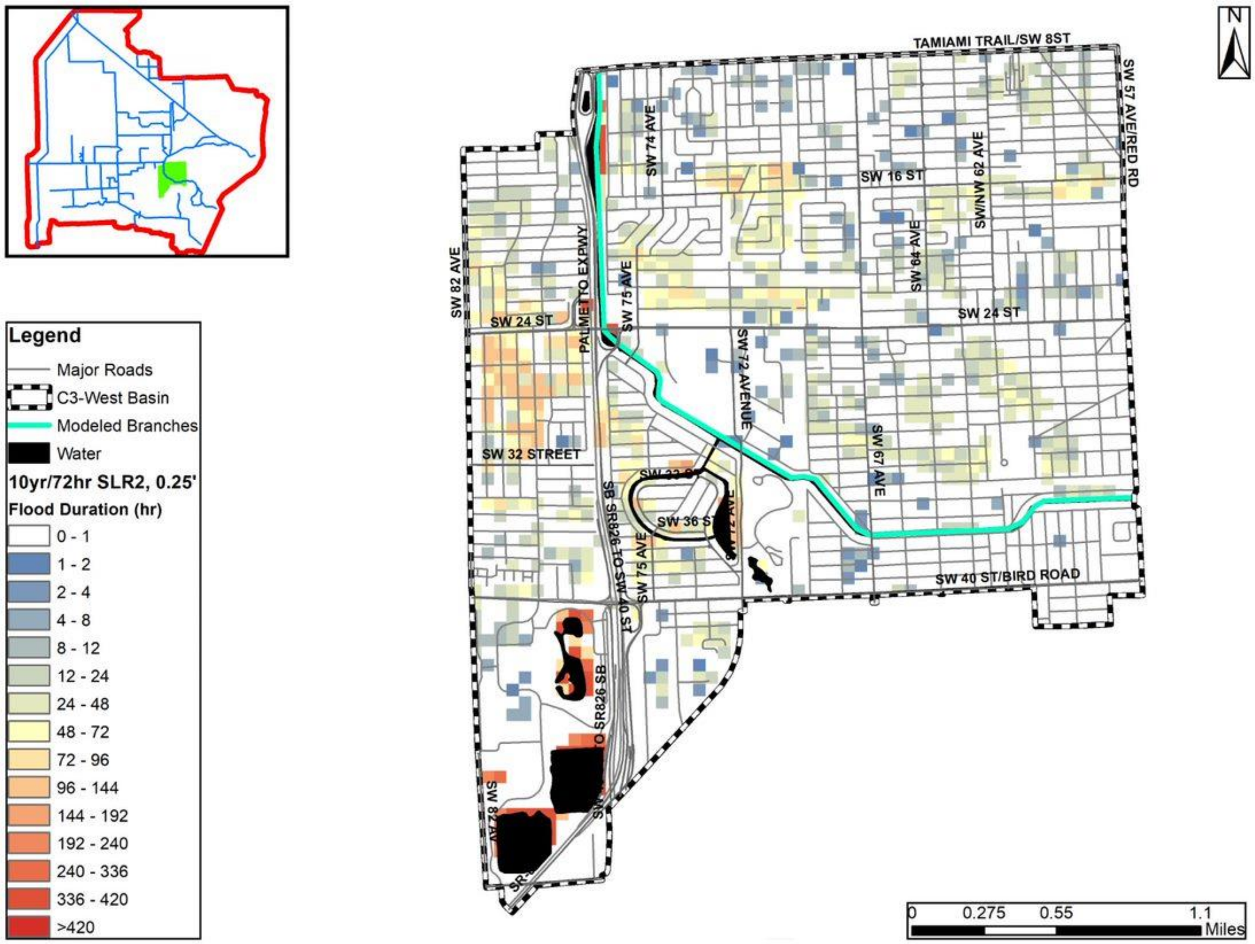


Figure D 2-8. Flood Duration Map for the 10-year 3-Day Design Storm for SLR3 in the C3W Watershed

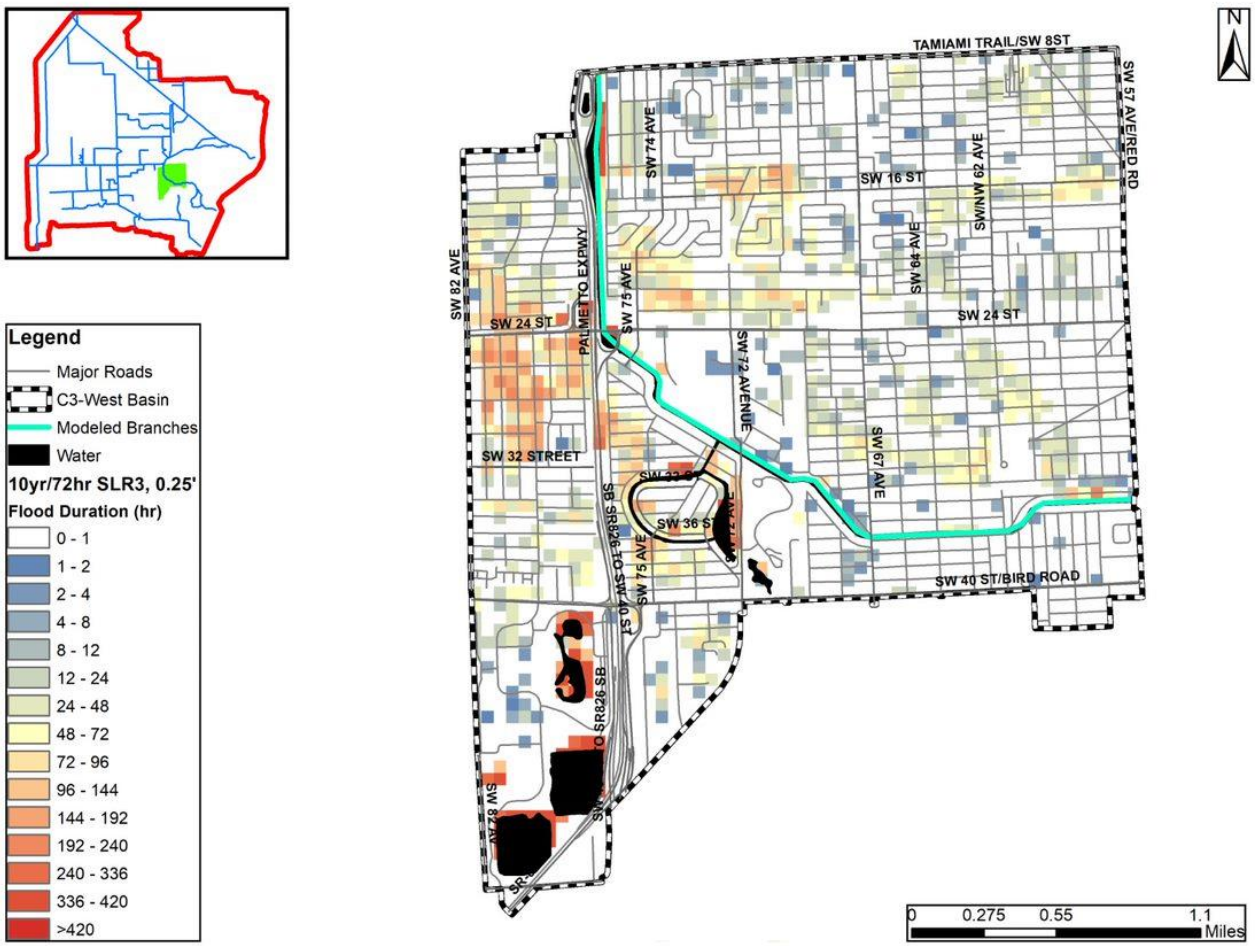


Figure D 2-9. Flood Duration Map for the 25-year 3-Day Design Storm for Current Conditions in the C3W Watershed

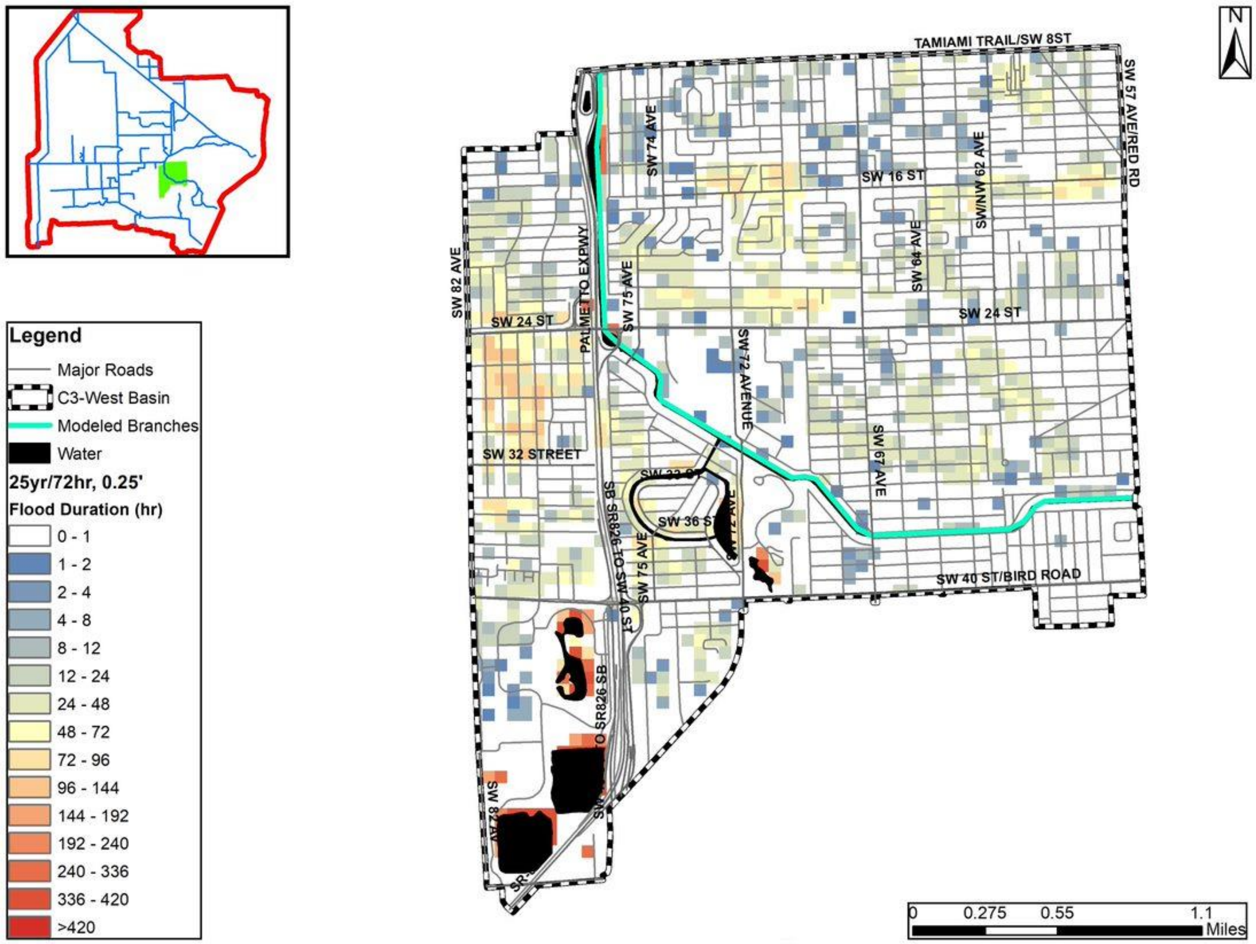


Figure D 2-10. Flood Duration Map for the 25-year 3-Day Design Storm for SLR1 in the C3W Watershed

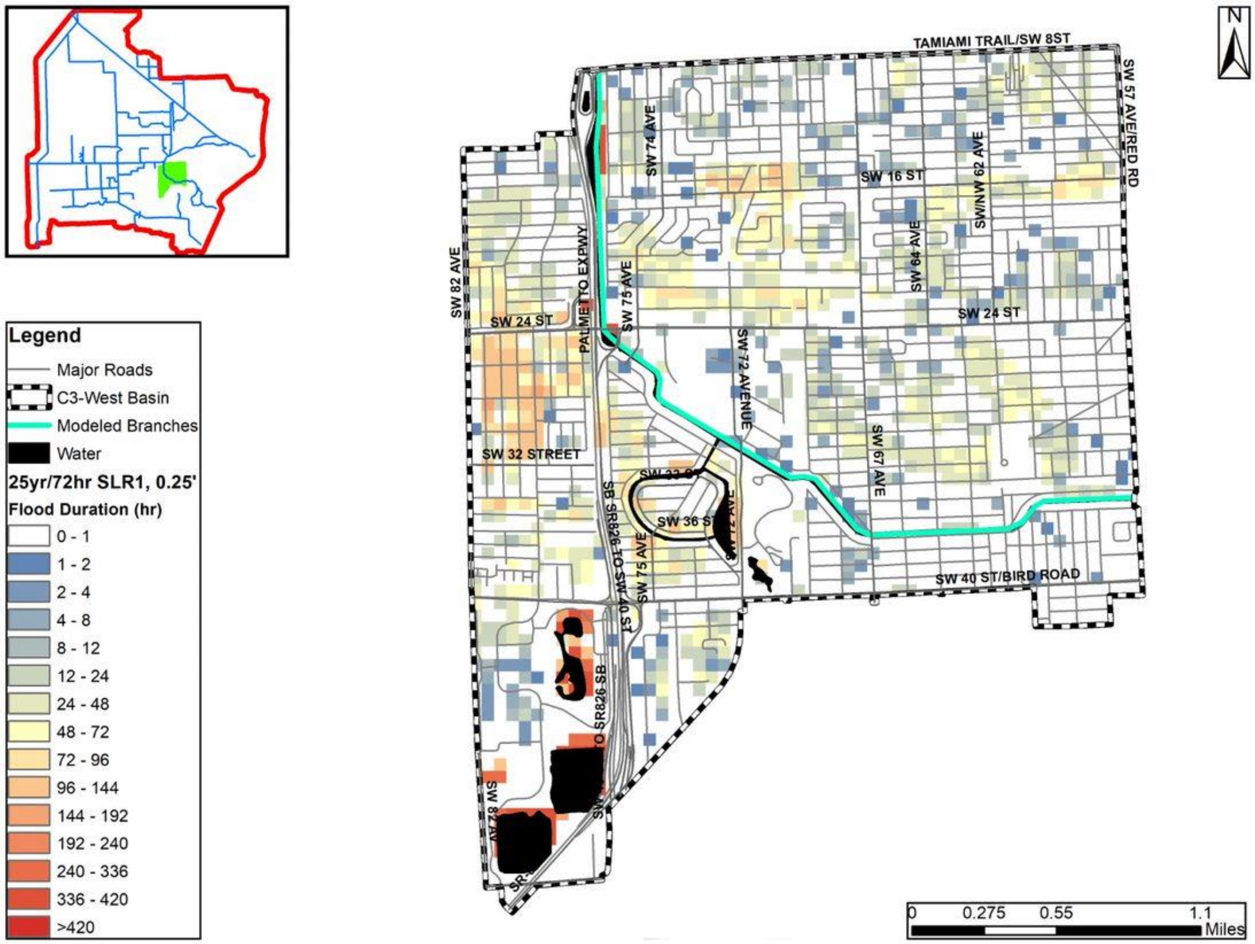


Figure D 2-11. Flood Duration Map for the 25-year 3-Day Design Storm for SLR2 in the C3W Watershed

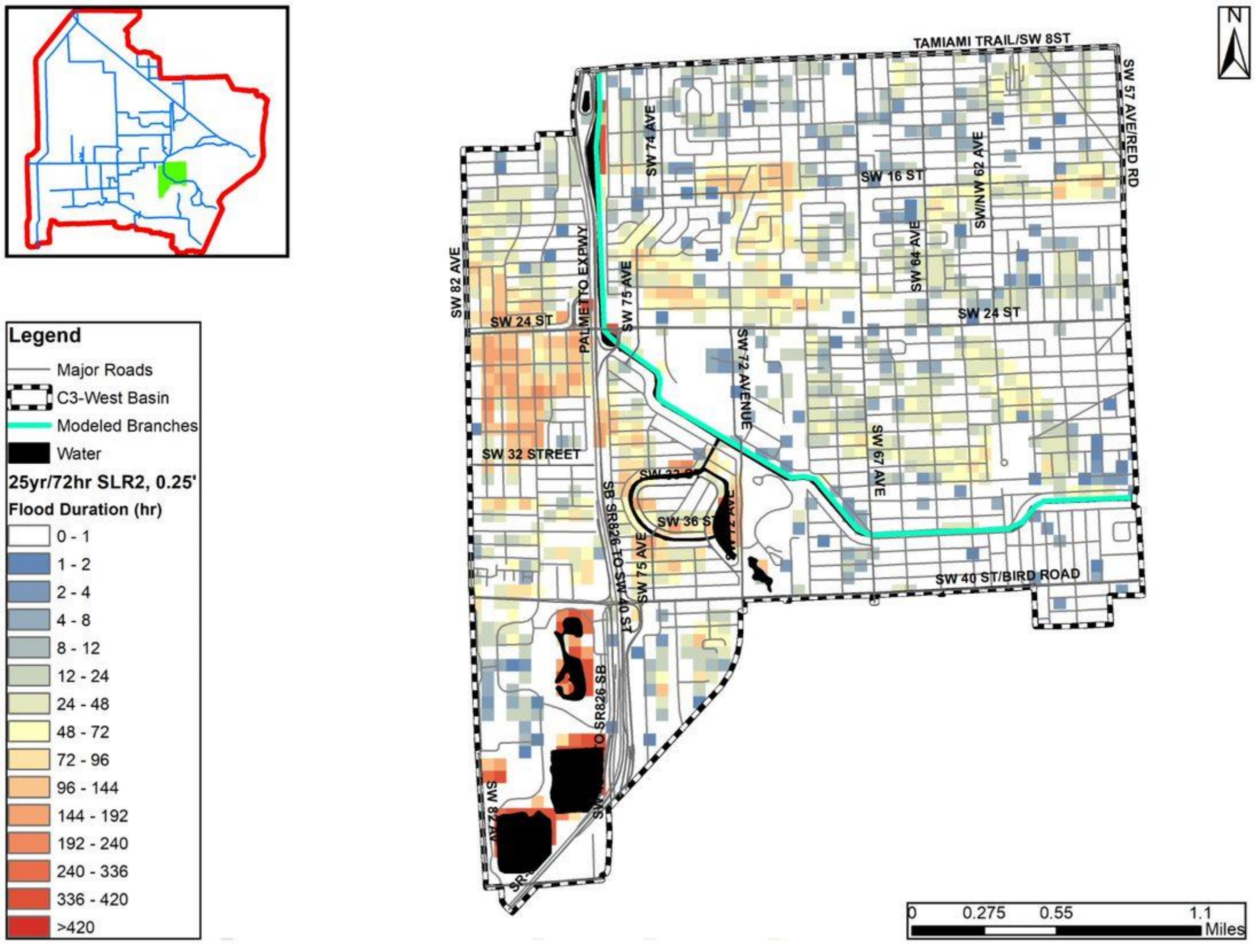


Figure D 2-12. Flood Duration Map for the 25-year 3-Day Design Storm for SLR3 in the C3W Watershed

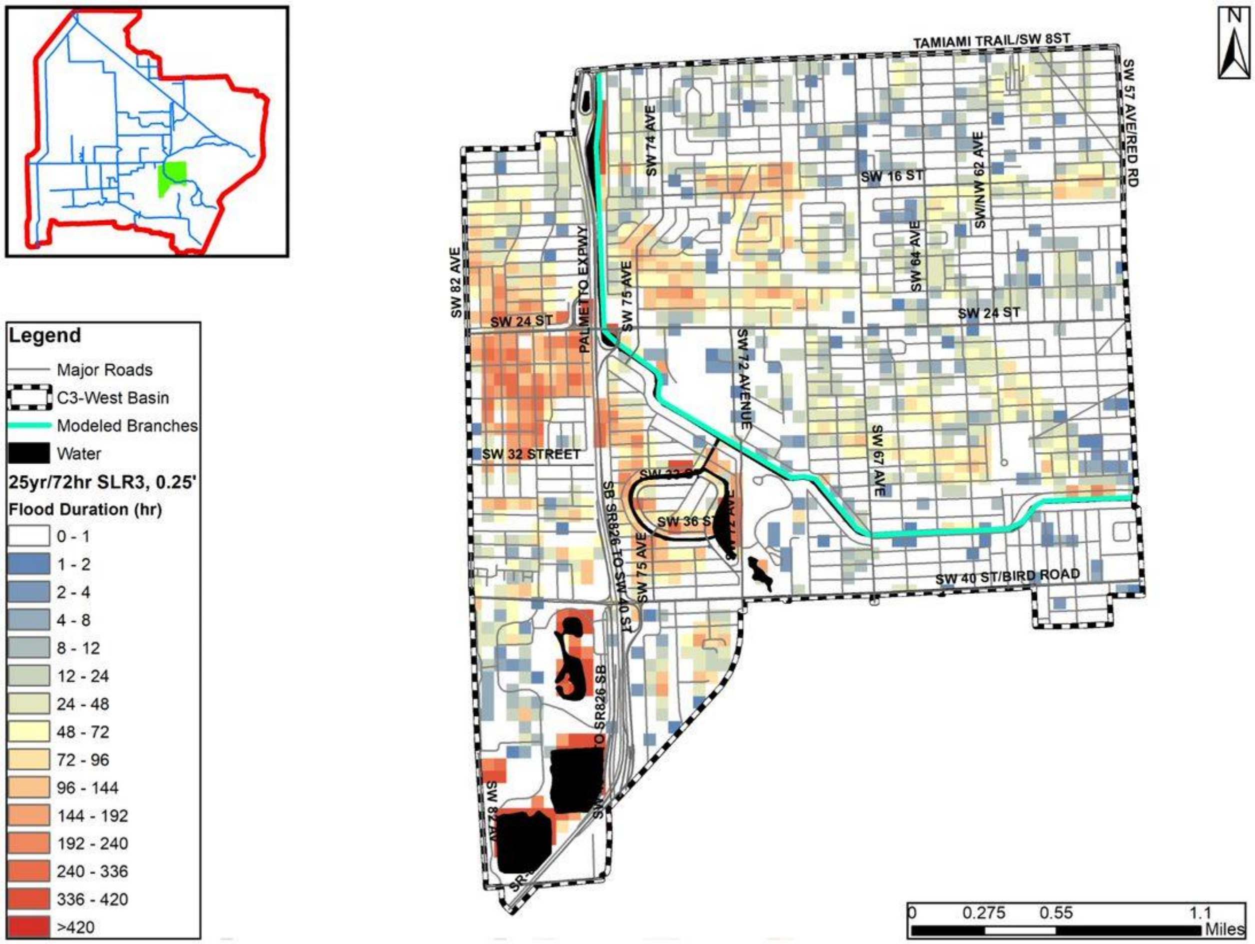


Figure D 2-13. Flood Duration Map for the 100-year 3-Day Design Storm for Current Conditions in the C3W Watershed

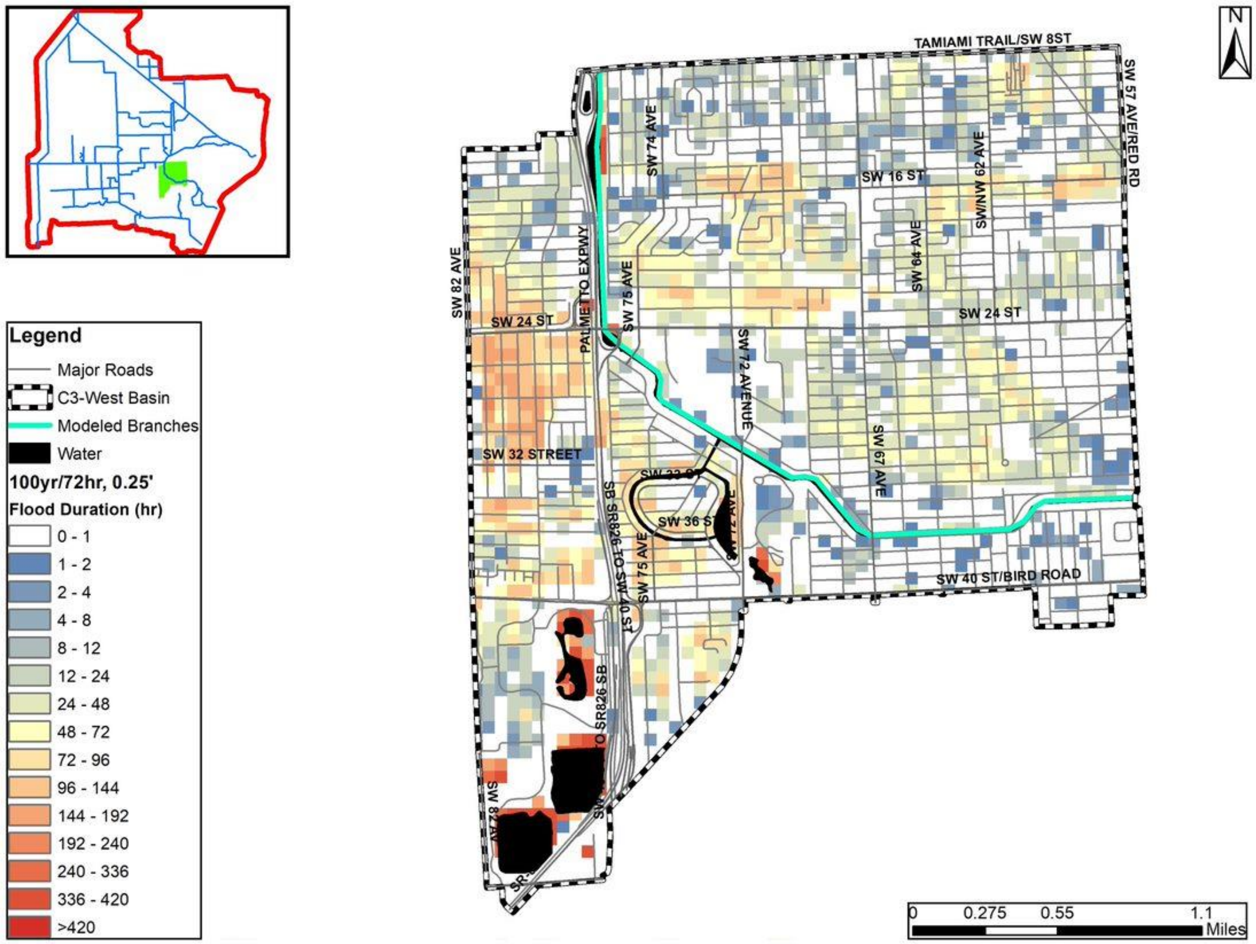


Figure D 2-14. Flood Duration Map for the 100-year 3-Day Design Storm for SLR1 in the C3W Watershed

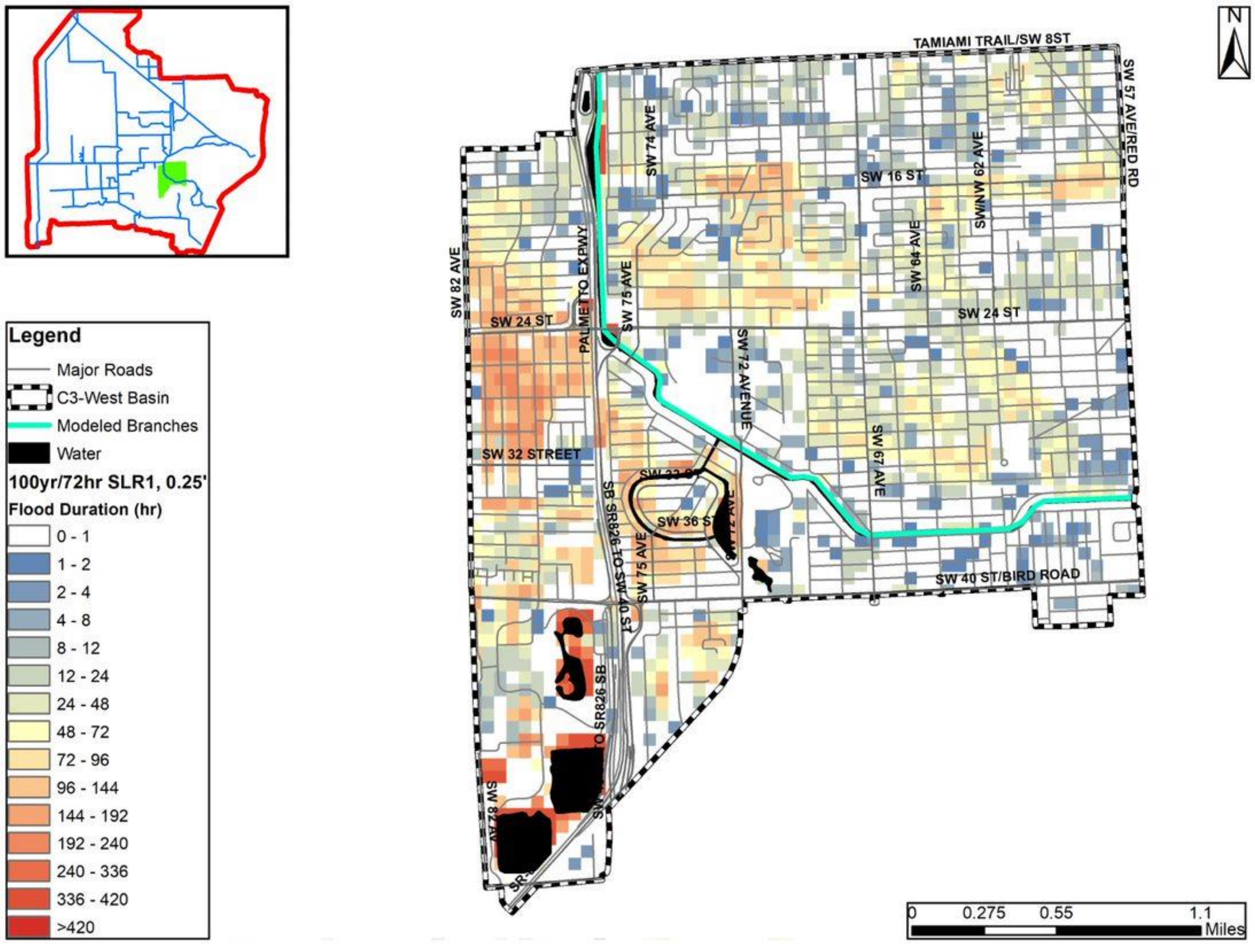


Figure D 2-15. Flood Duration Map for the 100-year 3-Day Design Storm for SLR2 in the C3W Watershed

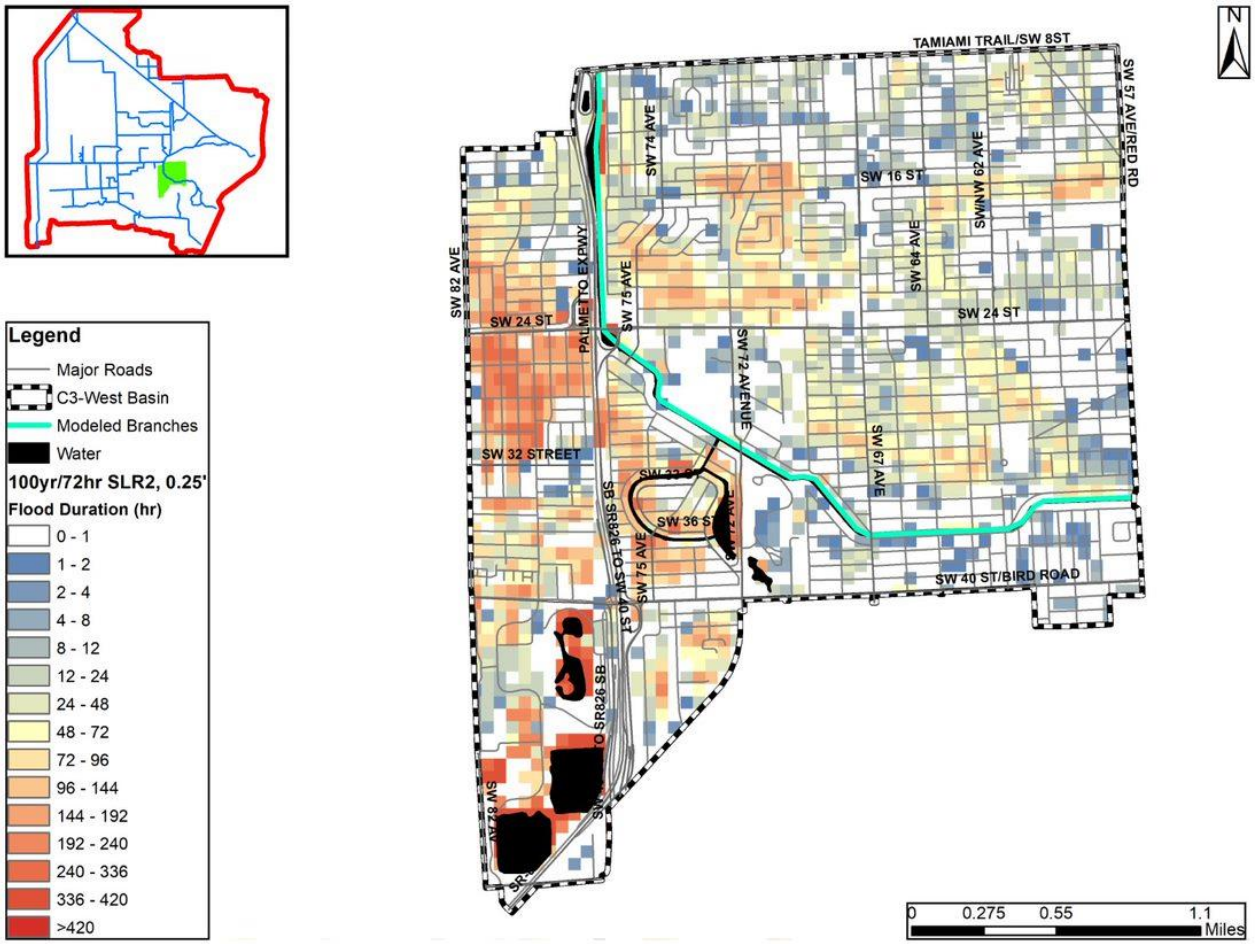


Figure D 2-16. Flood Duration Map for the 100-year 3-Day Design Storm for SLR3 in the C3W Watershed

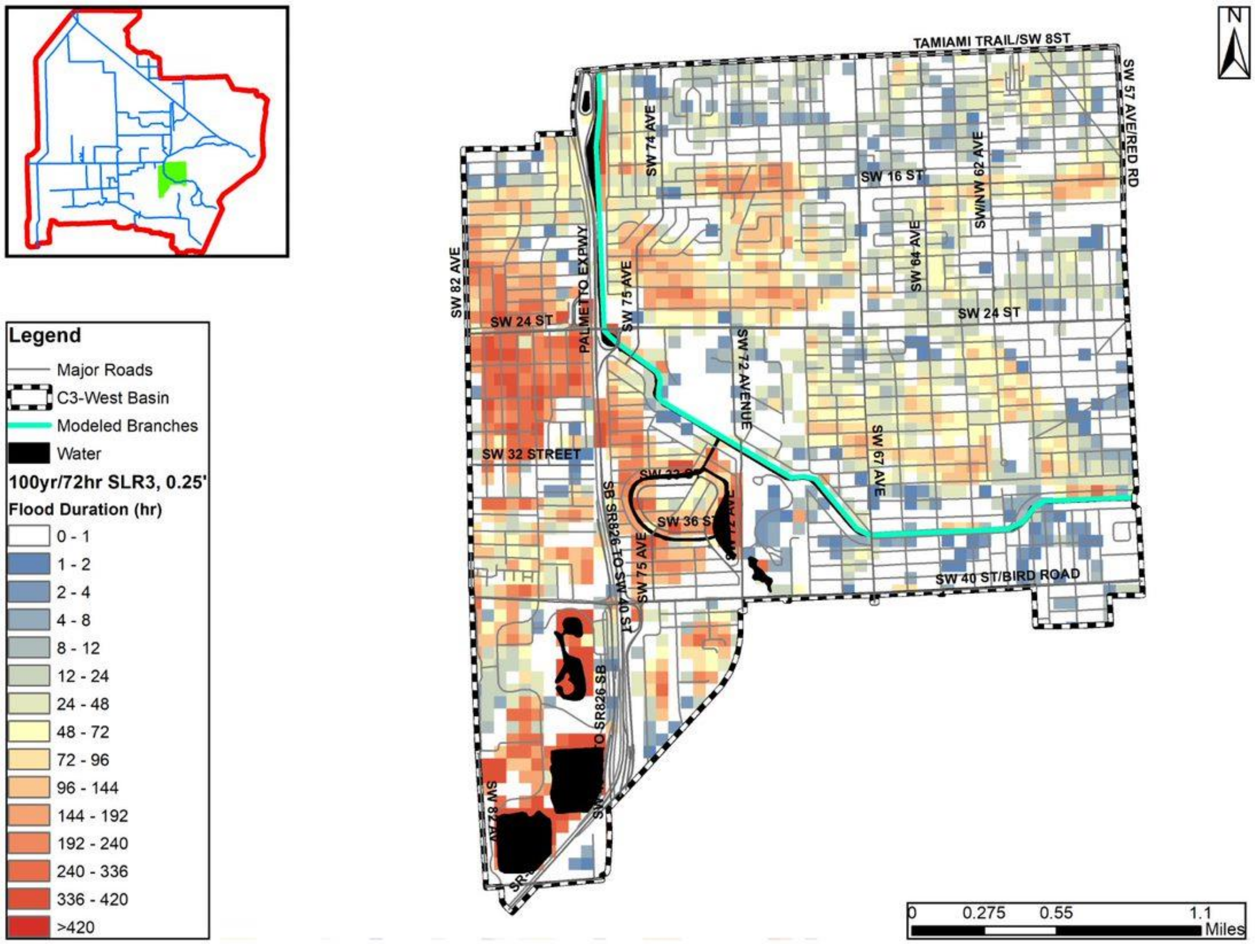


Figure D 2-17. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C3W Watershed

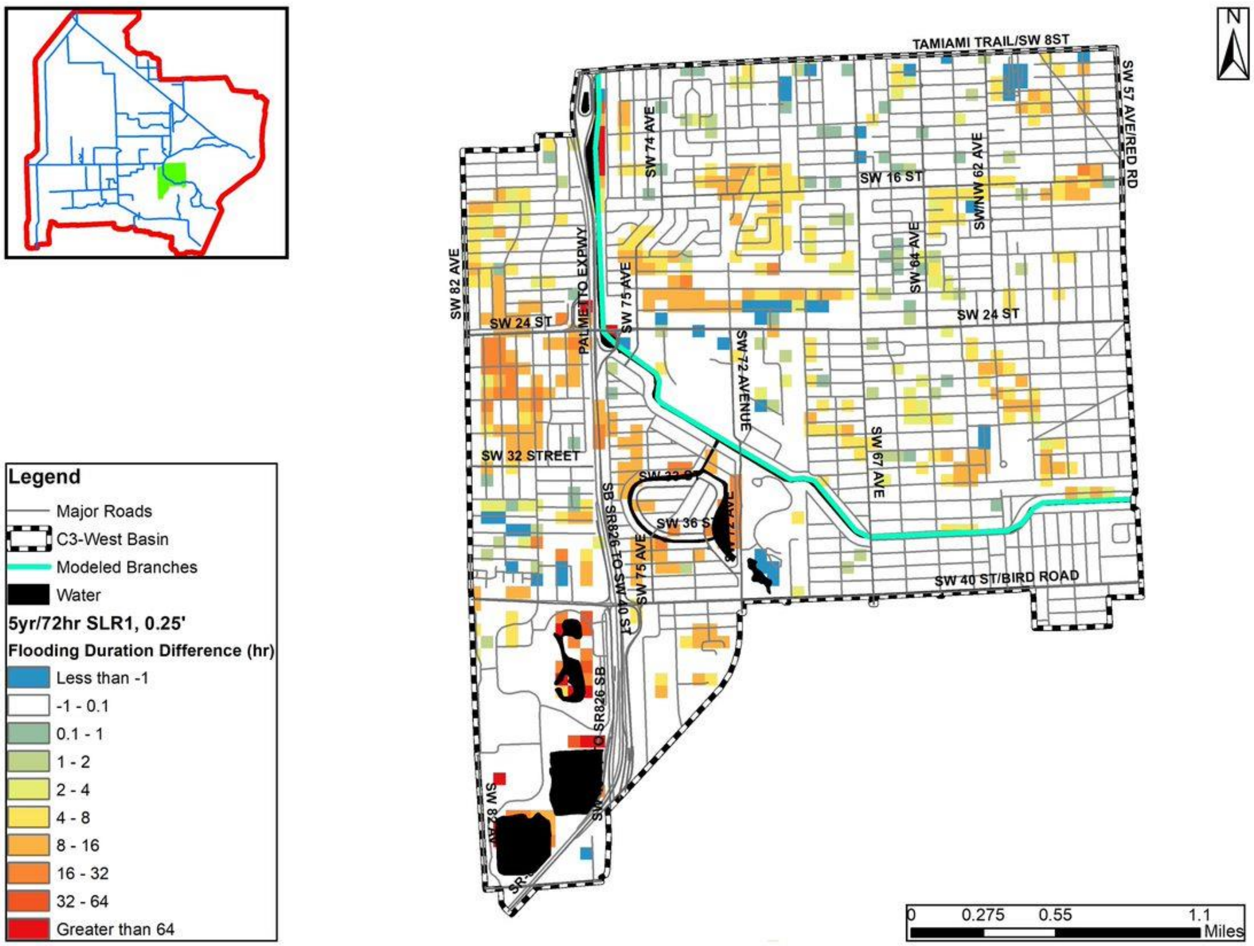


Figure D 2-18. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C3W Watershed

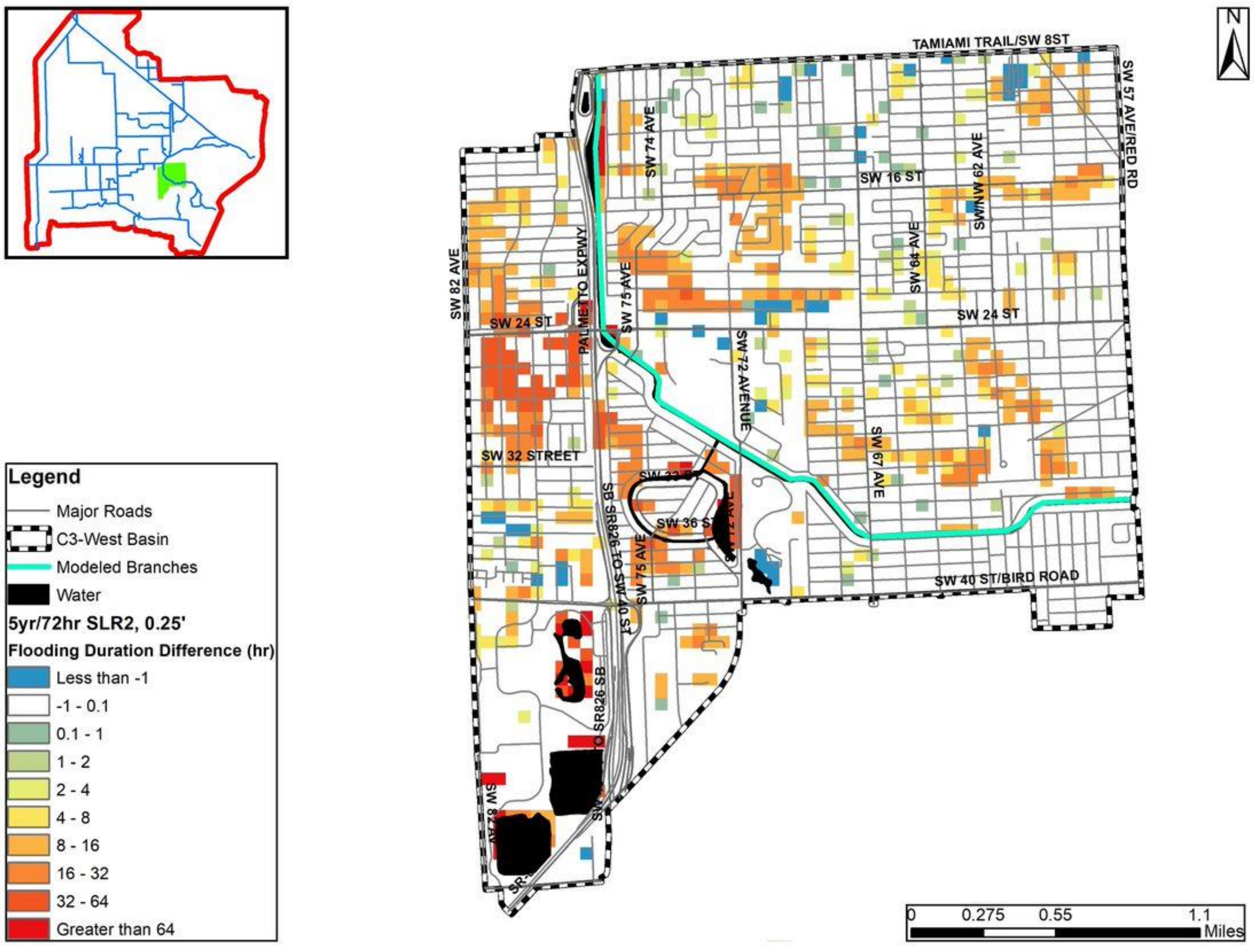


Figure D 2-19. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C3W Watershed

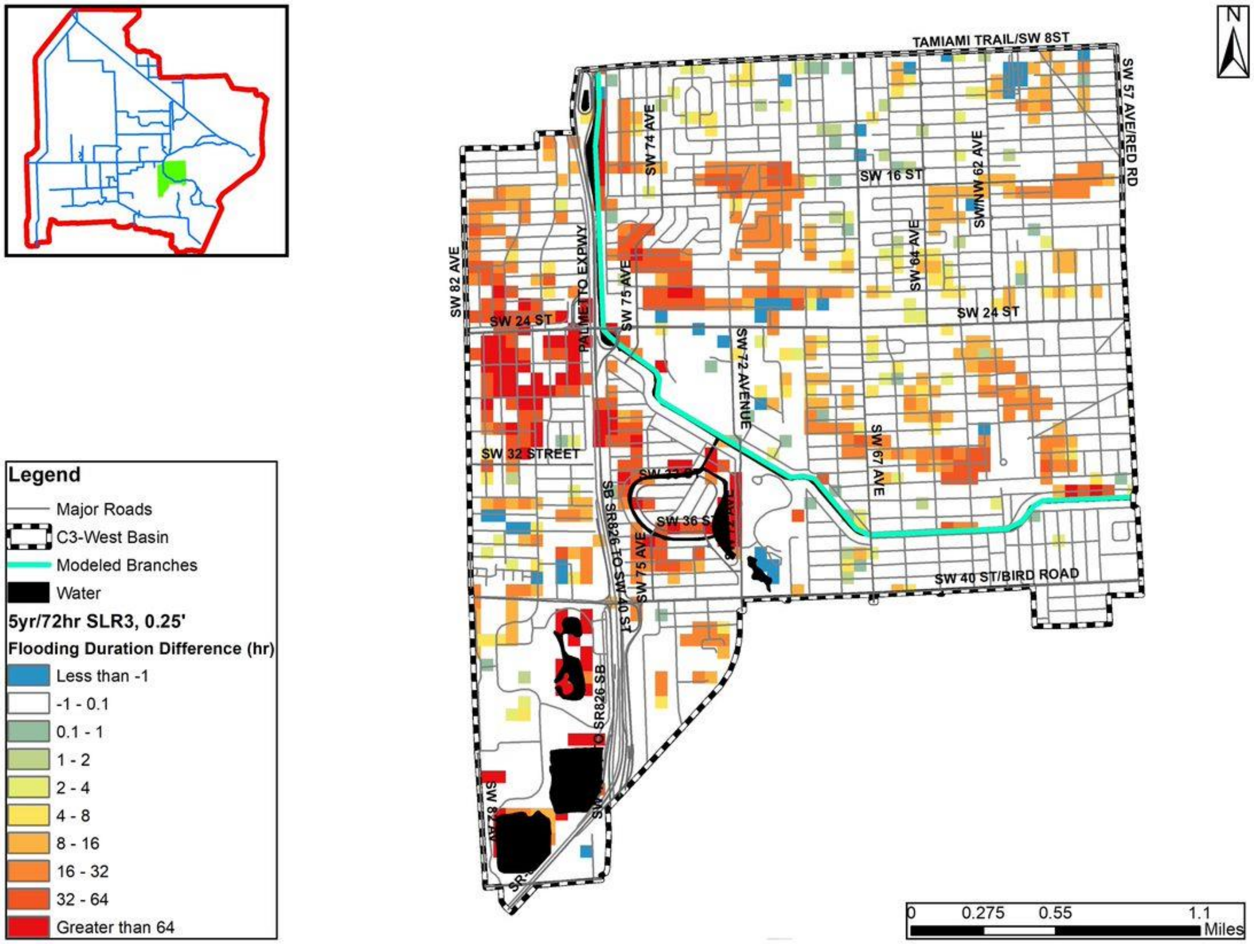


Figure D 2-20. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C3W Watershed

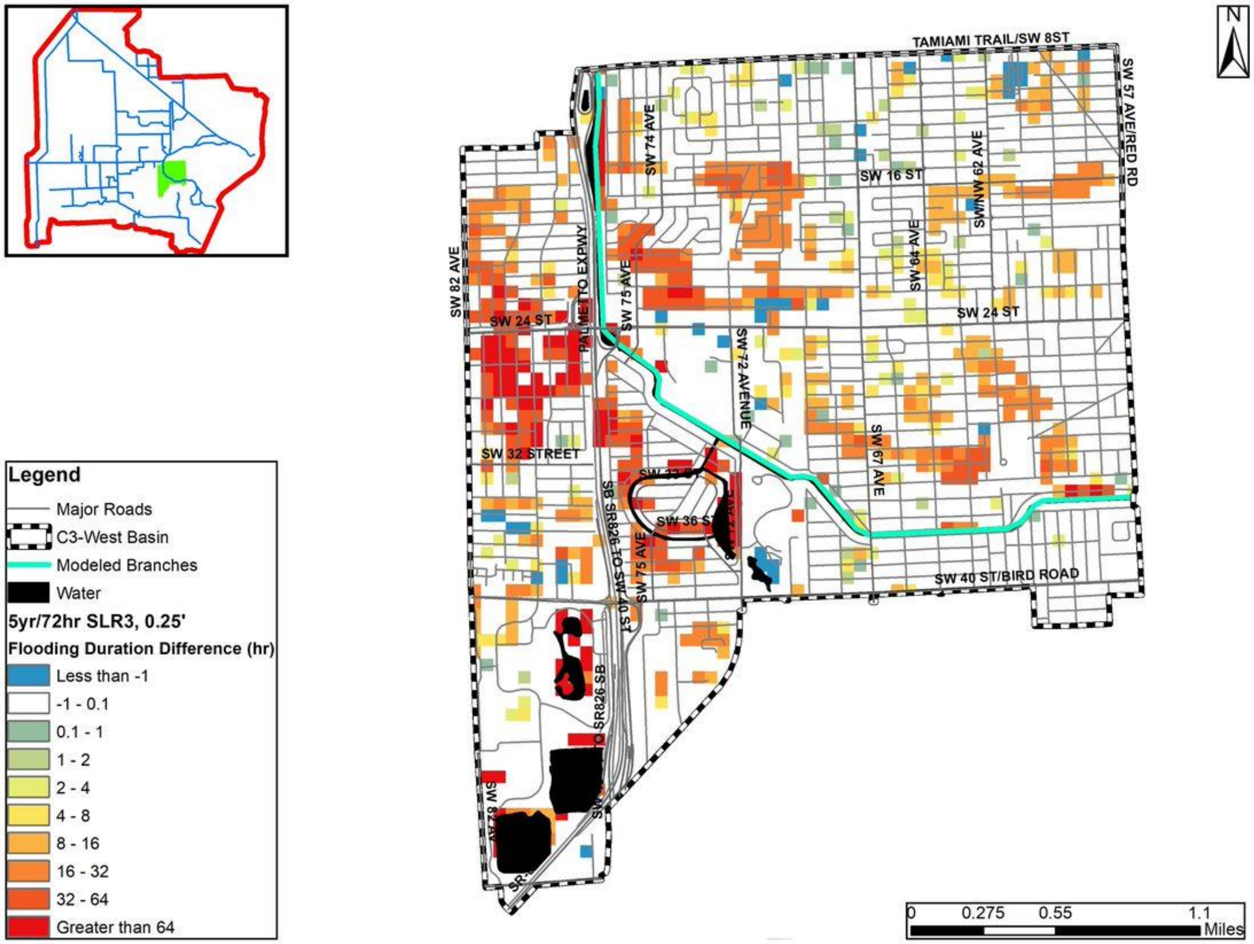


Figure D 2-21. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C3W Watershed

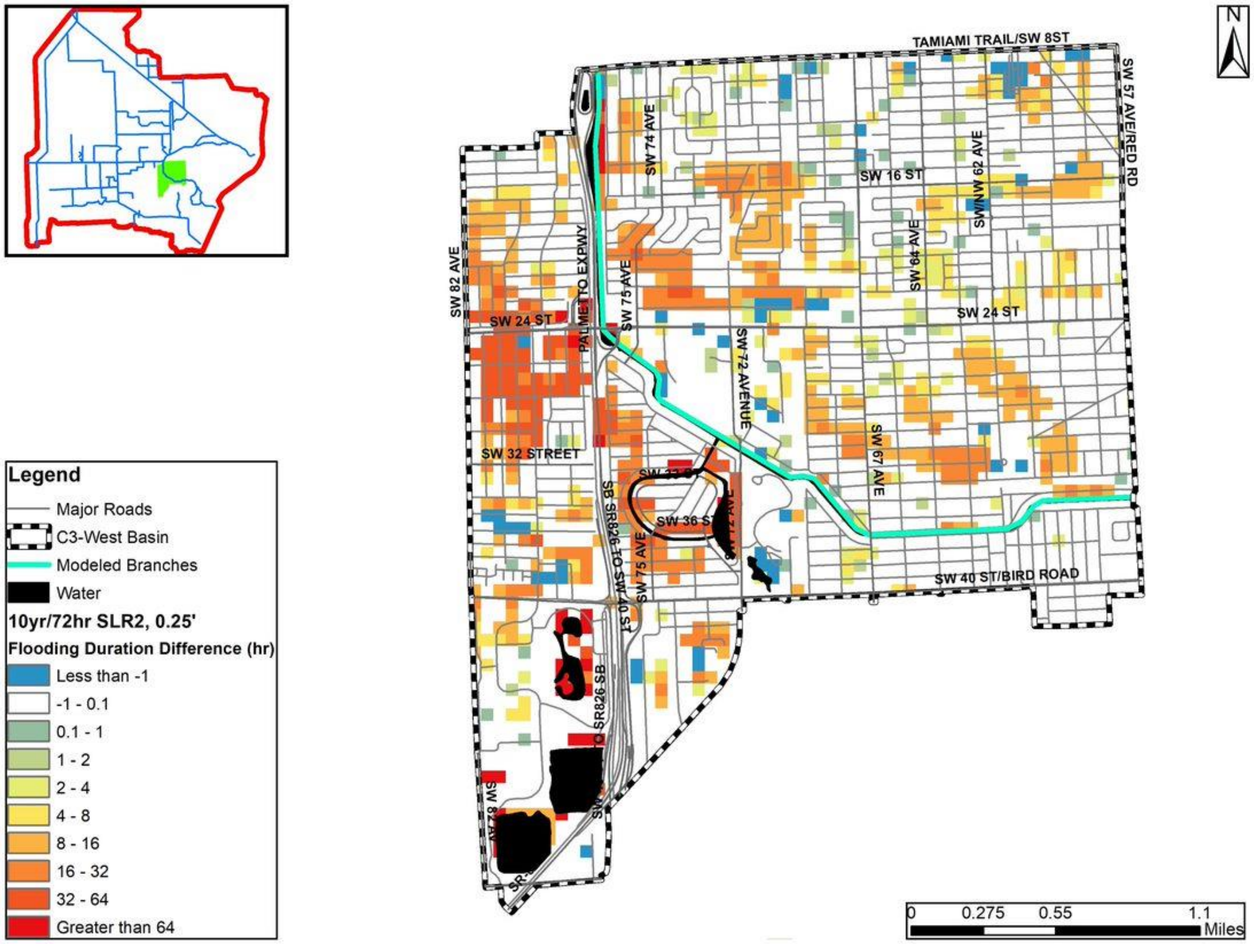


Figure D 2-22. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C3W Watershed

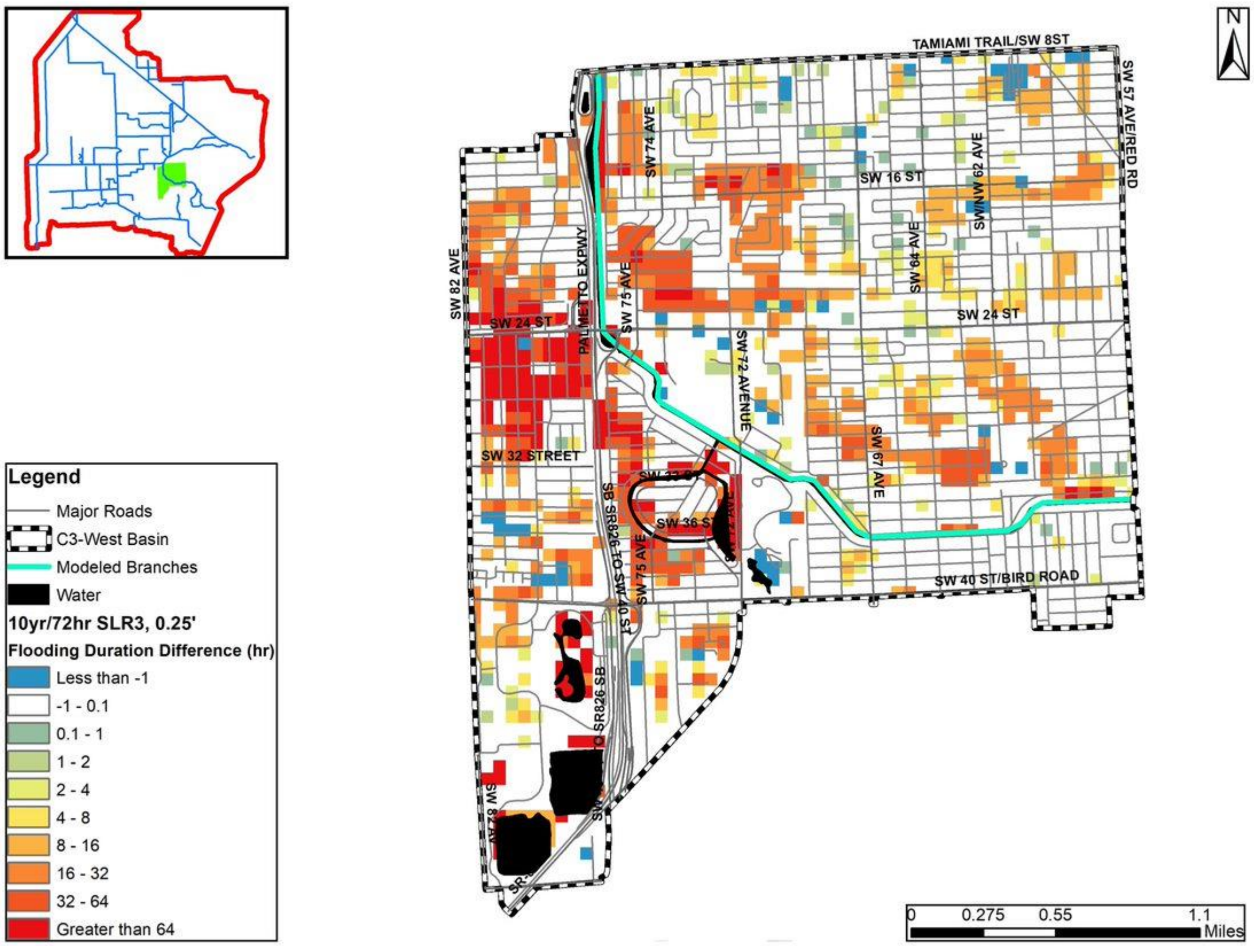


Figure D 2-23. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C3W Watershed

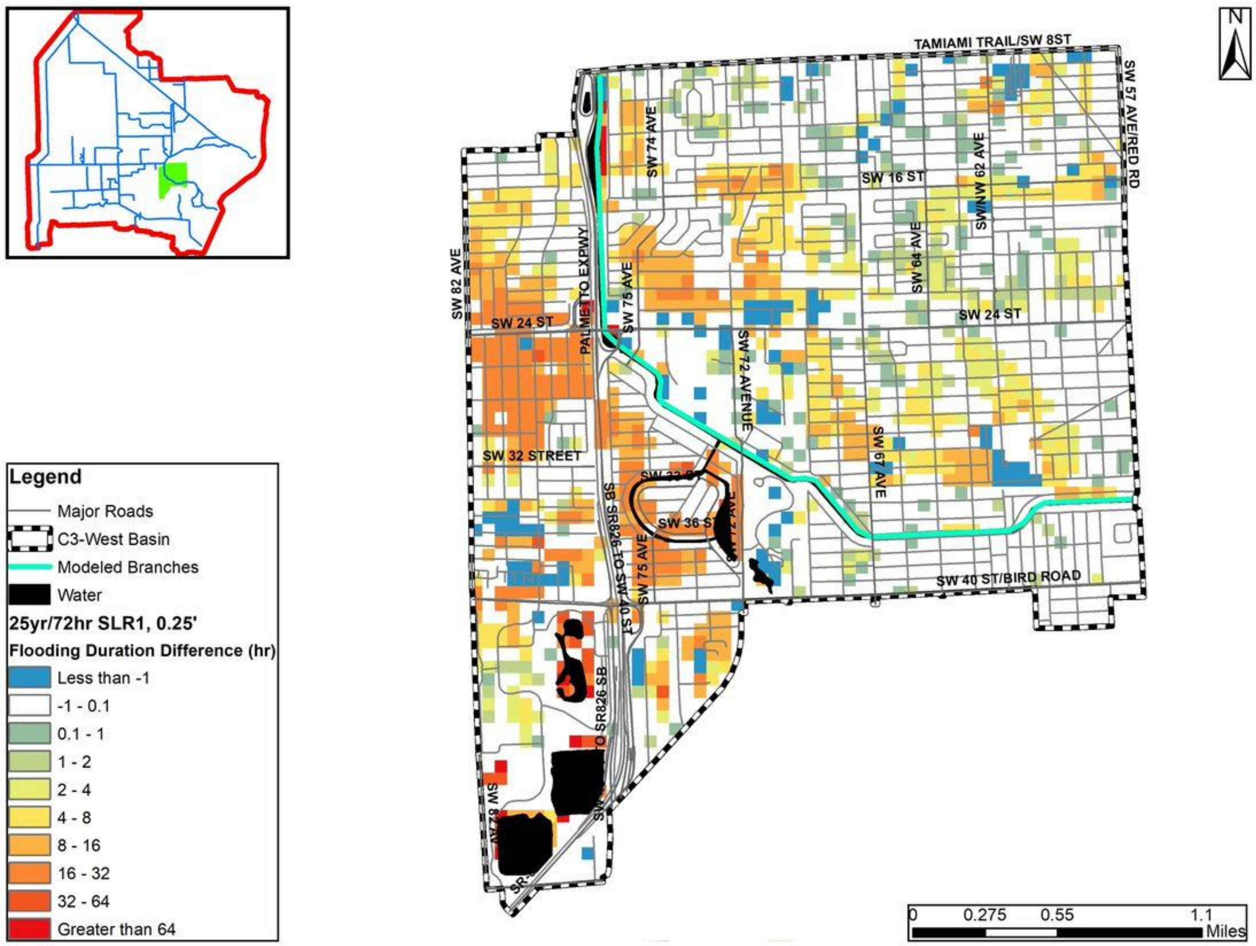


Figure D 2-24. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C3W Watershed

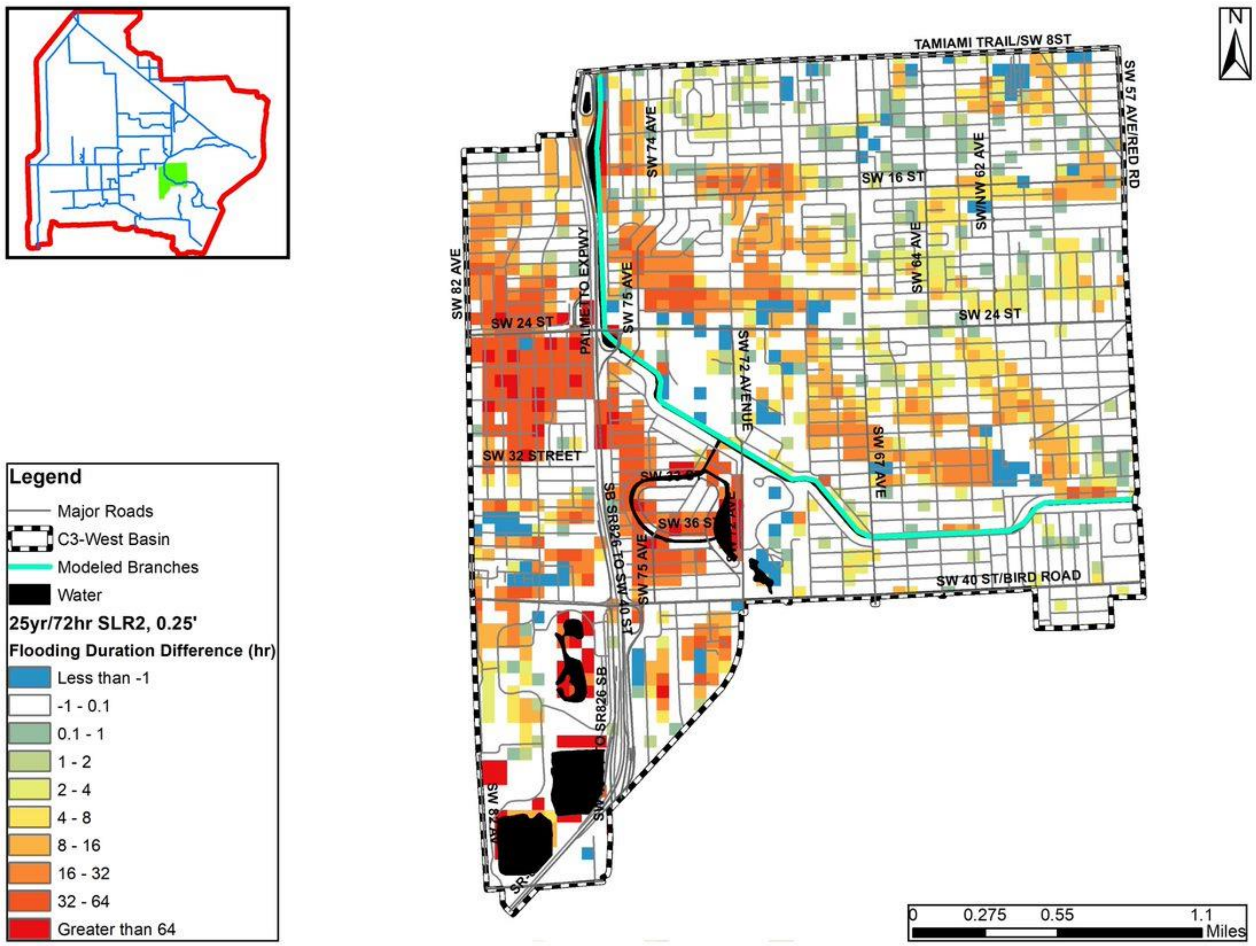


Figure D 2-25. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C3W Watershed

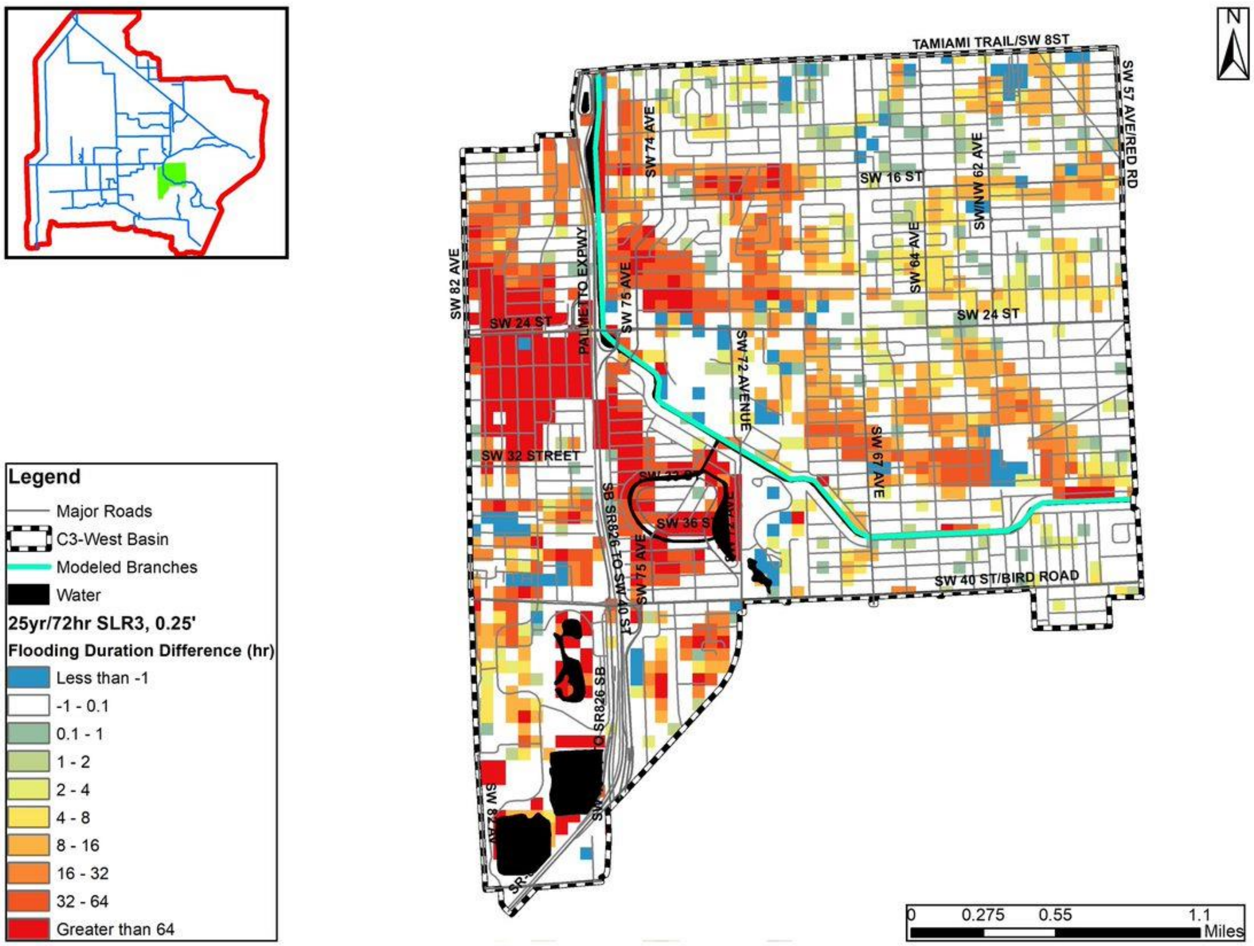


Figure D 2-26. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C3W Watershed

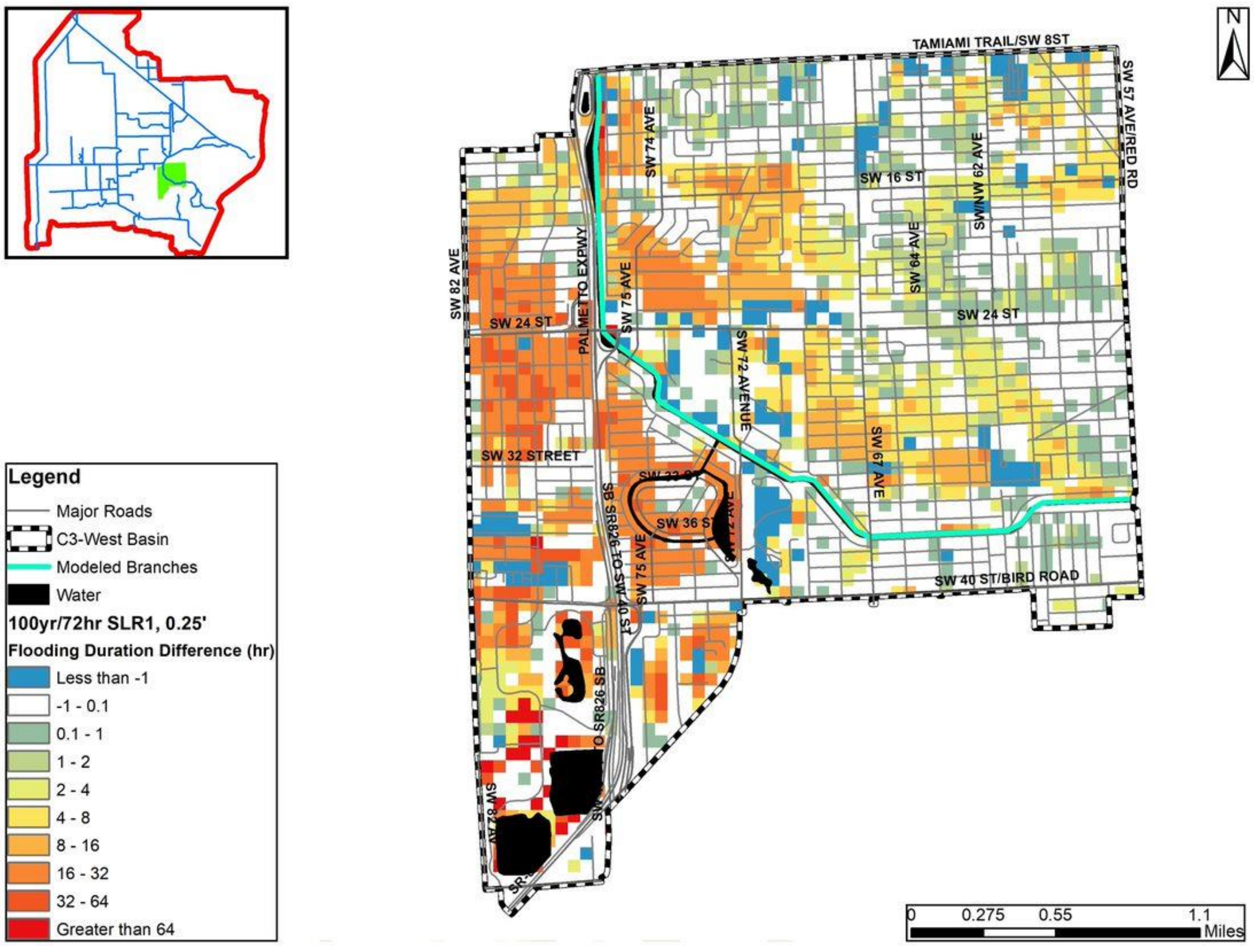


Figure D 2-27. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C3W Watershed

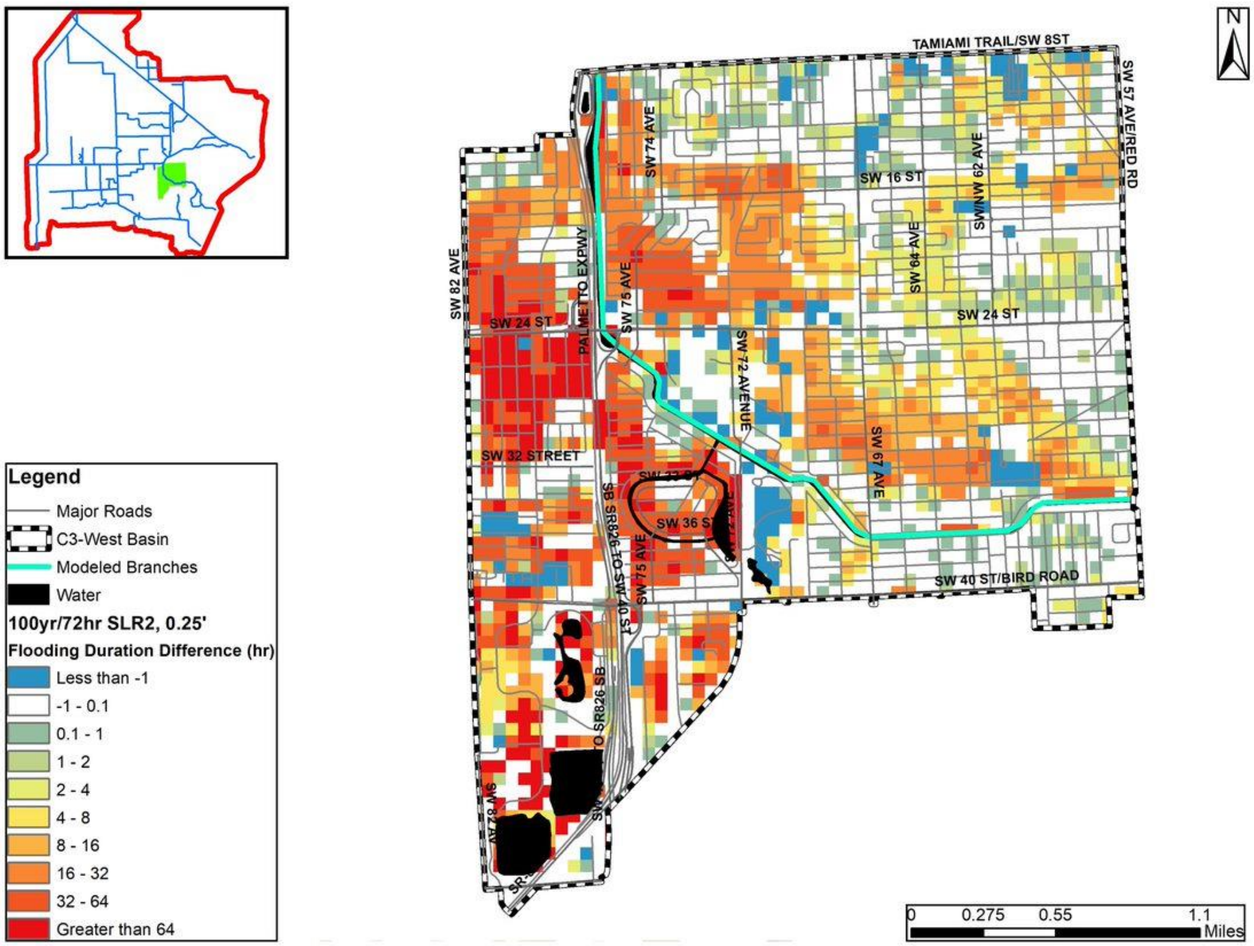
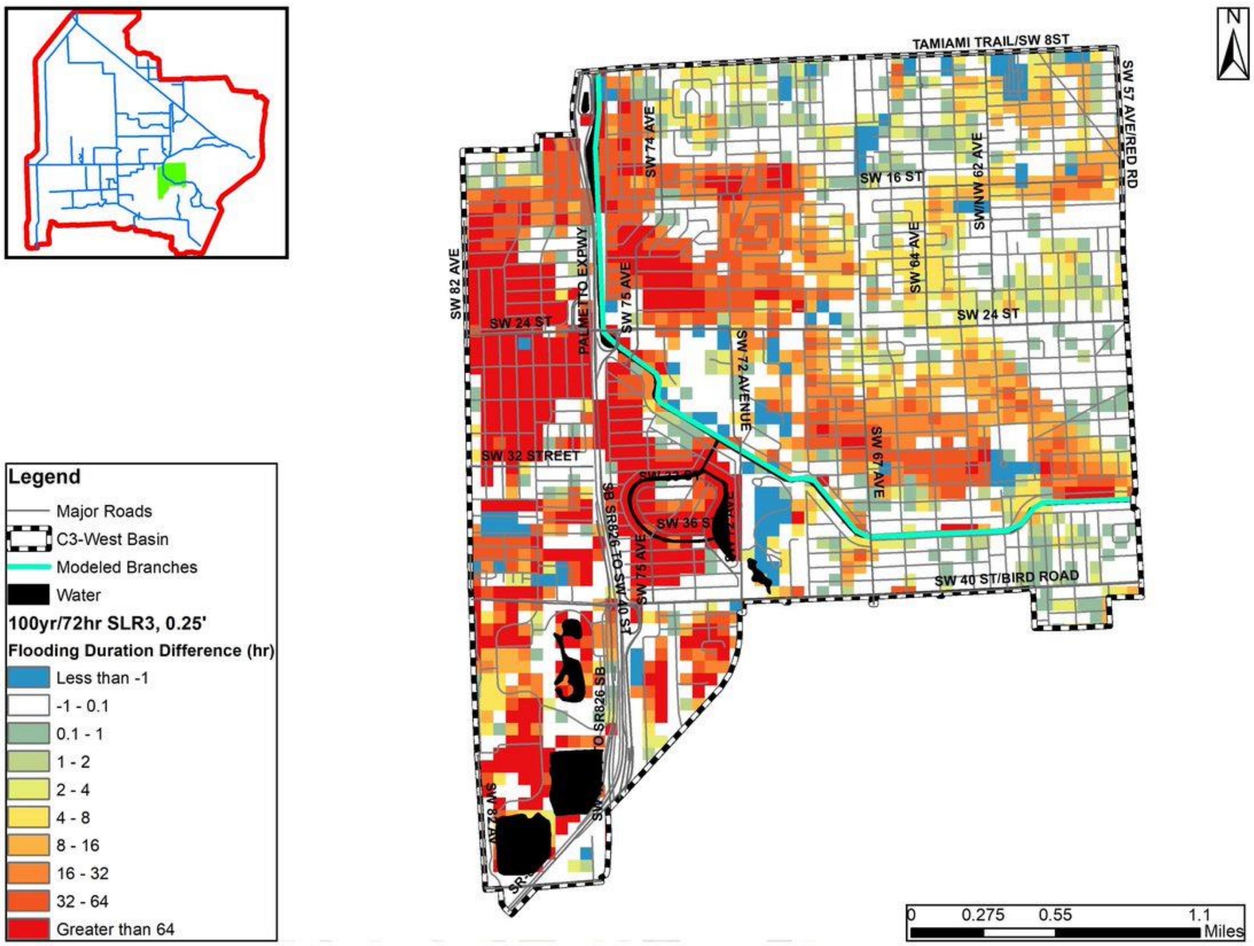


Figure D 2-28. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C3W Watershed



3 C4 Watershed

Figure D 3-1 through **Figure D 3-16** provides flood duration maps for the C4 Watershed for overland flooding depths exceeding 0.25 ft for the 5-, 10-, 25-, and 100-year 3-day design storms, respectively, for each SLR condition. Water areas are masked in black. Figure D 3-17 through **Figure D 3-32** provide the flood duration for only the urban areas within the C4 Watershed (non-urban areas are masked out).

Figure D 3-33 through **Figure D 3-44** show the difference in overland flooding for the C4 Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively. **Figure D 3-45** through **Figure D 3-56** show the same maps with the non-urban areas masked out.

Figure D 3-1. Flood Duration Map for the 5-year 3-Day Design Storm for Current Conditions in the C4 Watershed

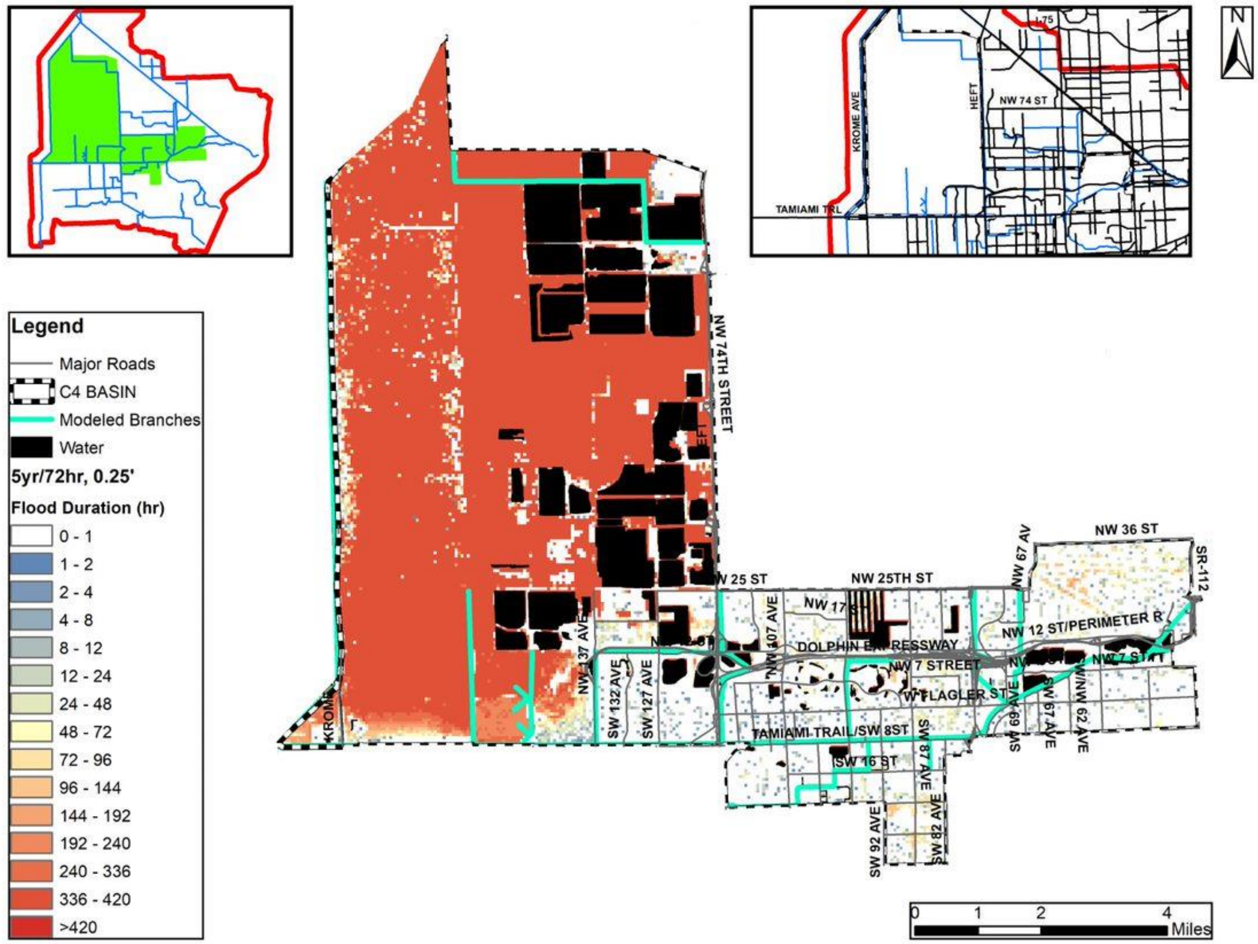


Figure D 3-2. Flood Duration Map for the 5-year 3-Day Design Storm for SLR1 in the C4 Watershed

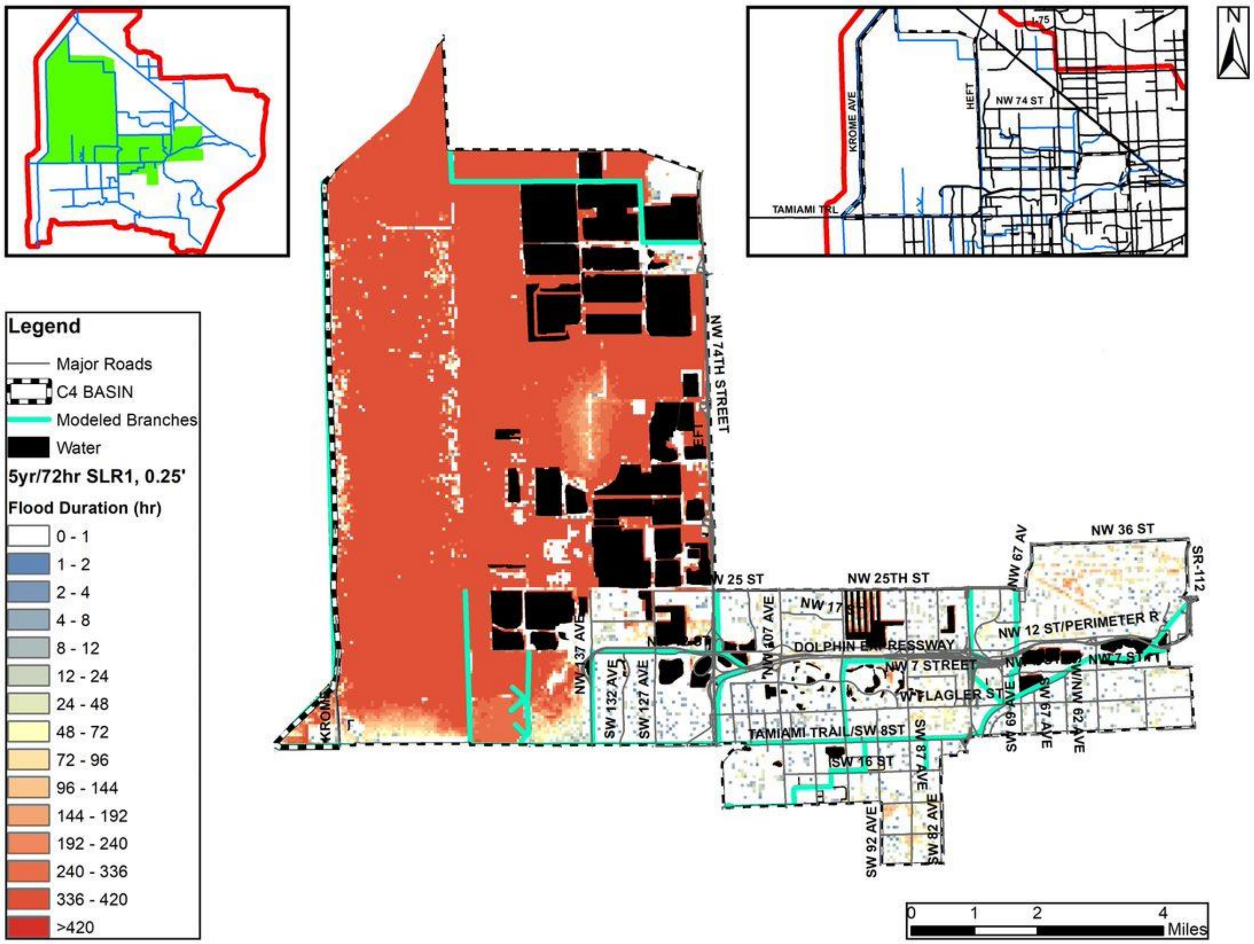


Figure D 3-3. Flood Duration Map for the 5-year 3-Day Design Storm for SLR2 in the C4 Watershed

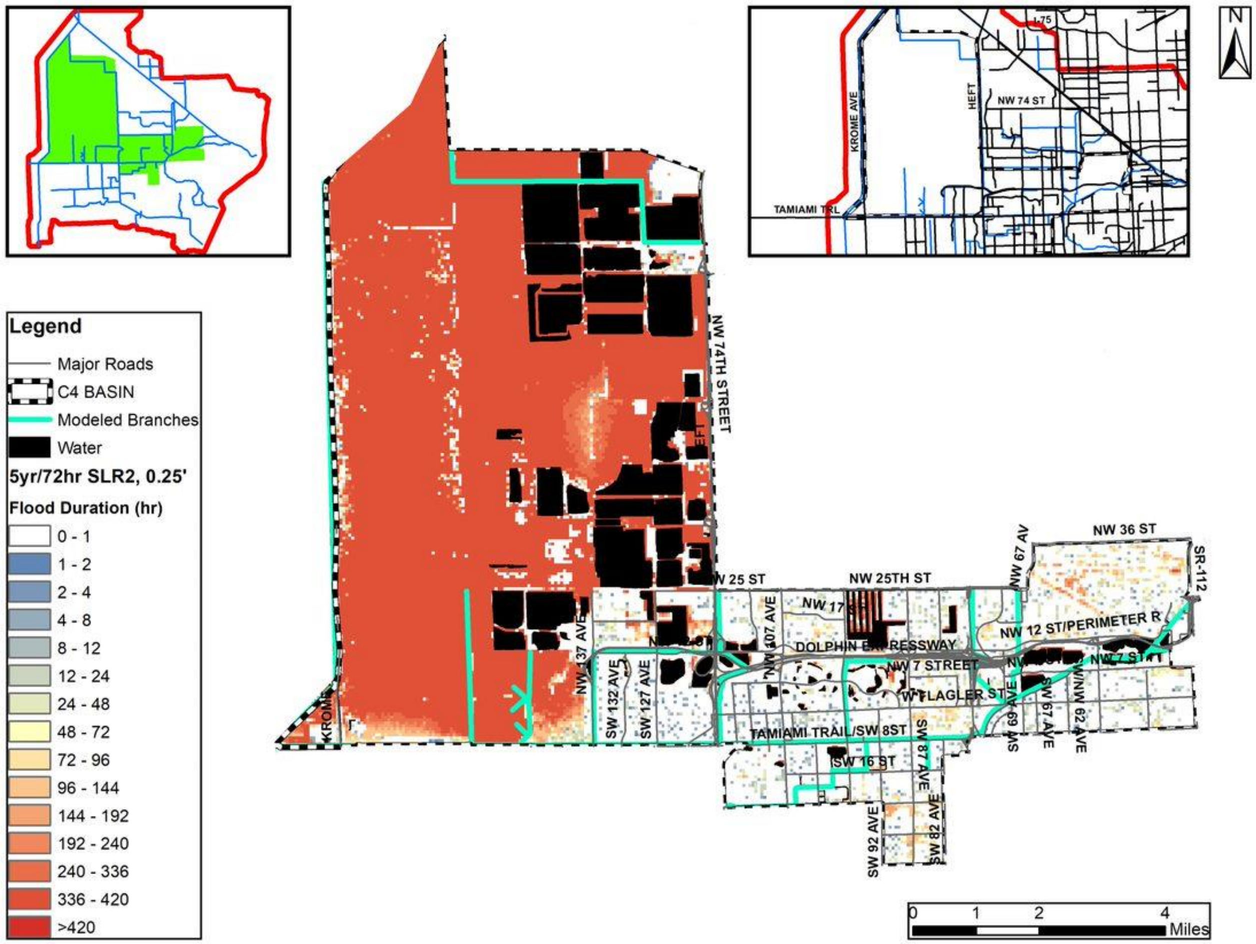


Figure D 3-4. Flood Duration Map for the 5-year 3-Day Design Storm for SLR3 in the C4 Watershed

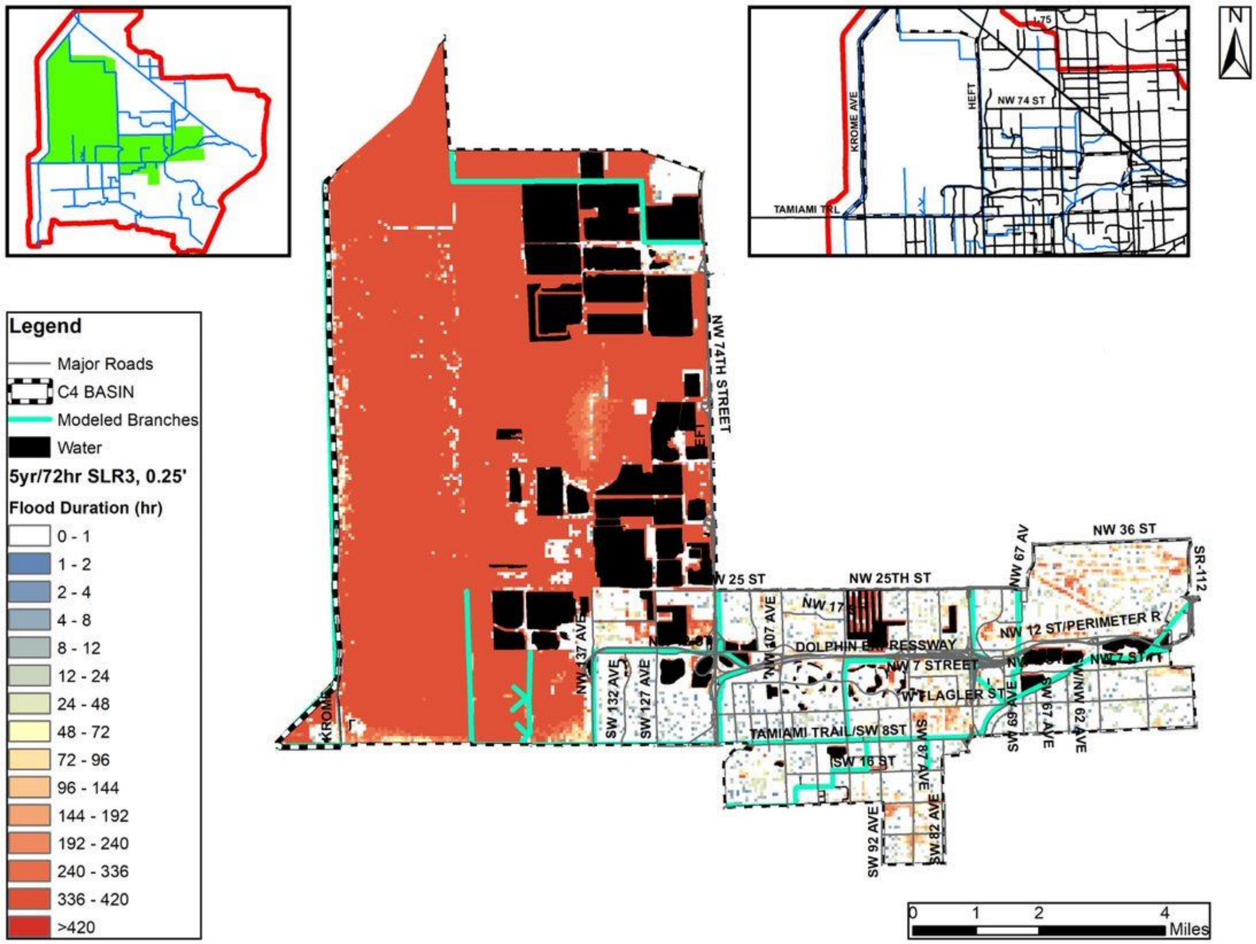


Figure D 3-5. Flood Duration Map for the 10-year 3-Day Design Storm for Current Conditions in the C4 Watershed

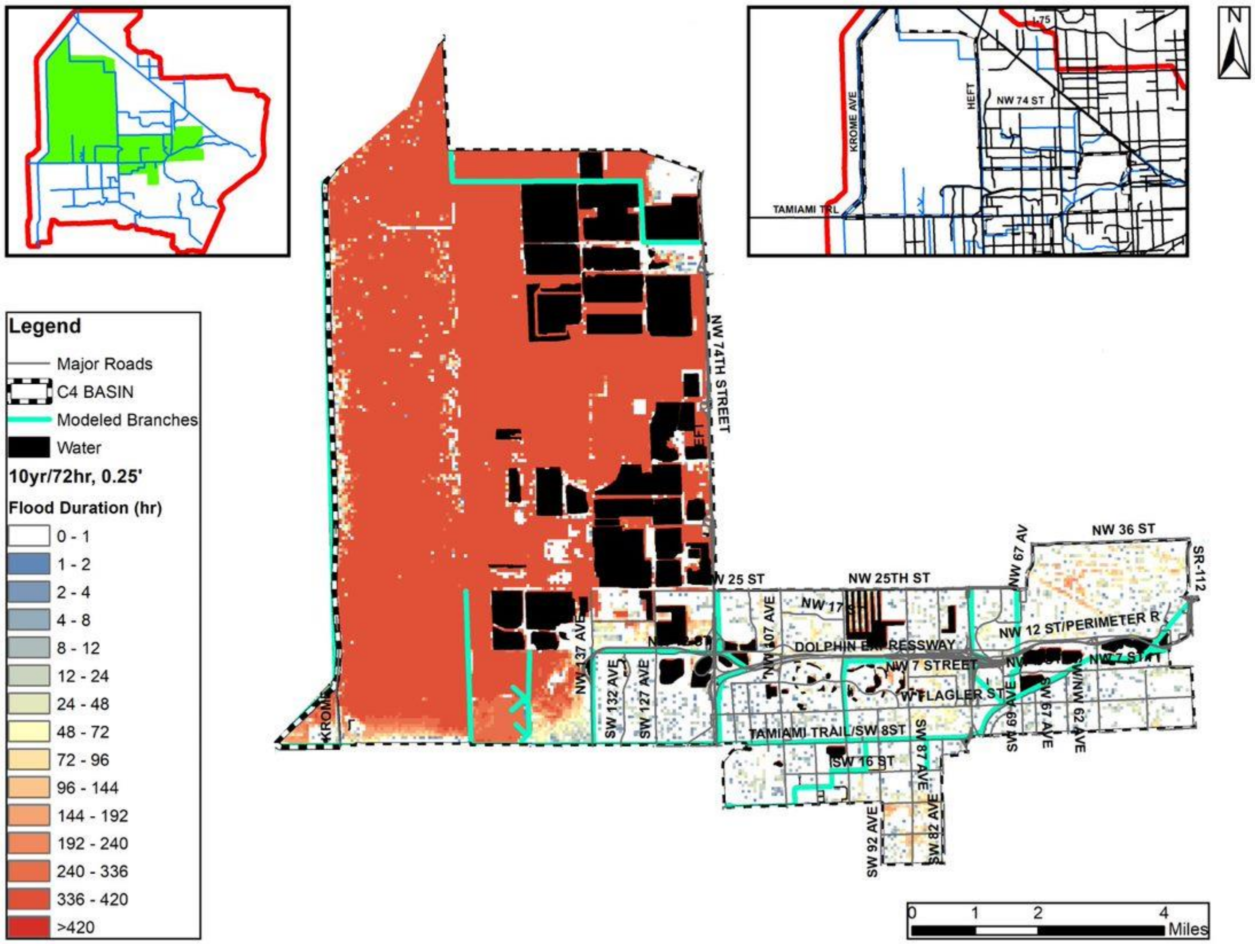


Figure D 3-6. Flood Duration Map for the 10-year 3-Day Design Storm for SLR1 in the C4 Watershed

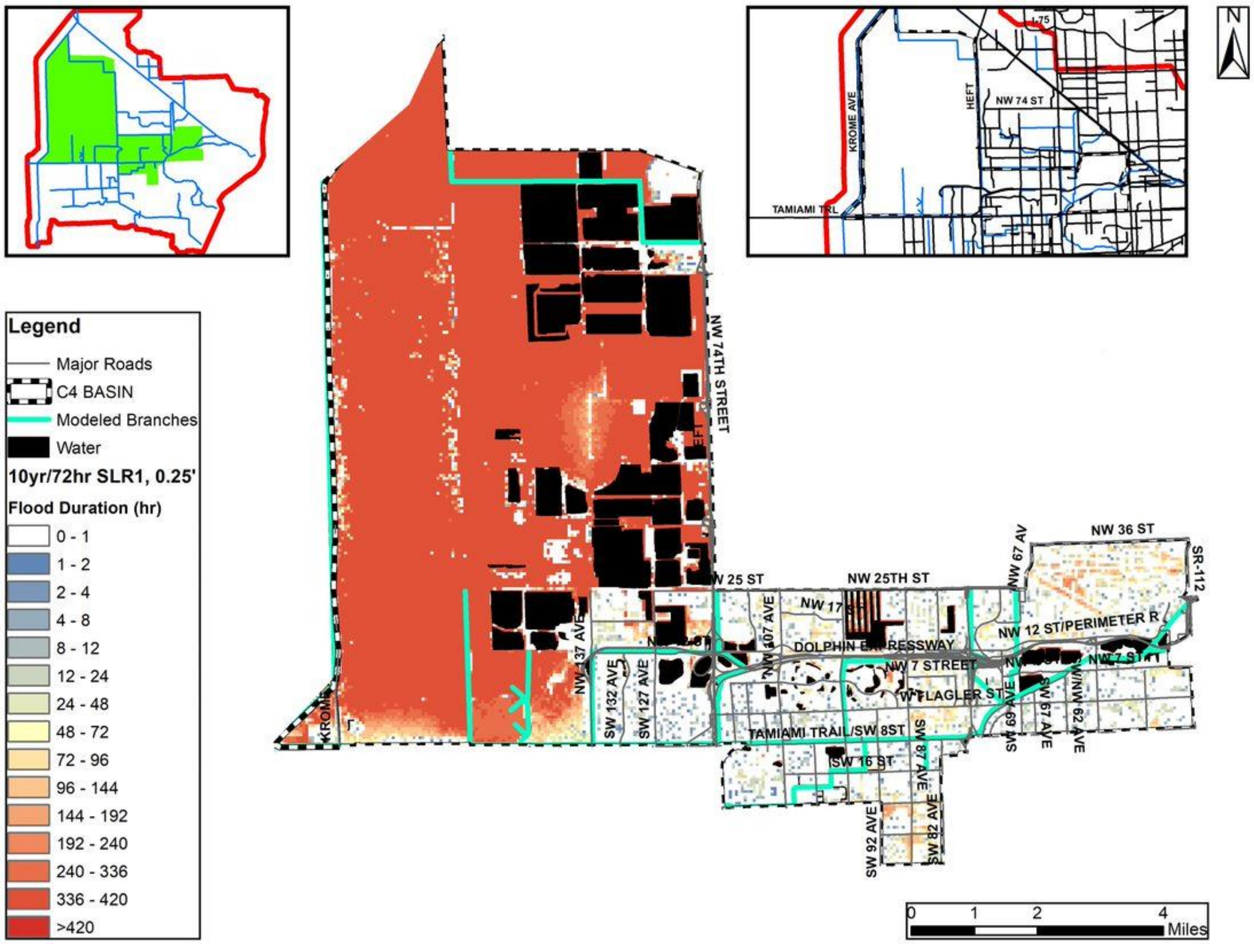


Figure D 3-7. Flood Duration Map for the 10-year 3-Day Design Storm for SLR2 in the C4 Watershed

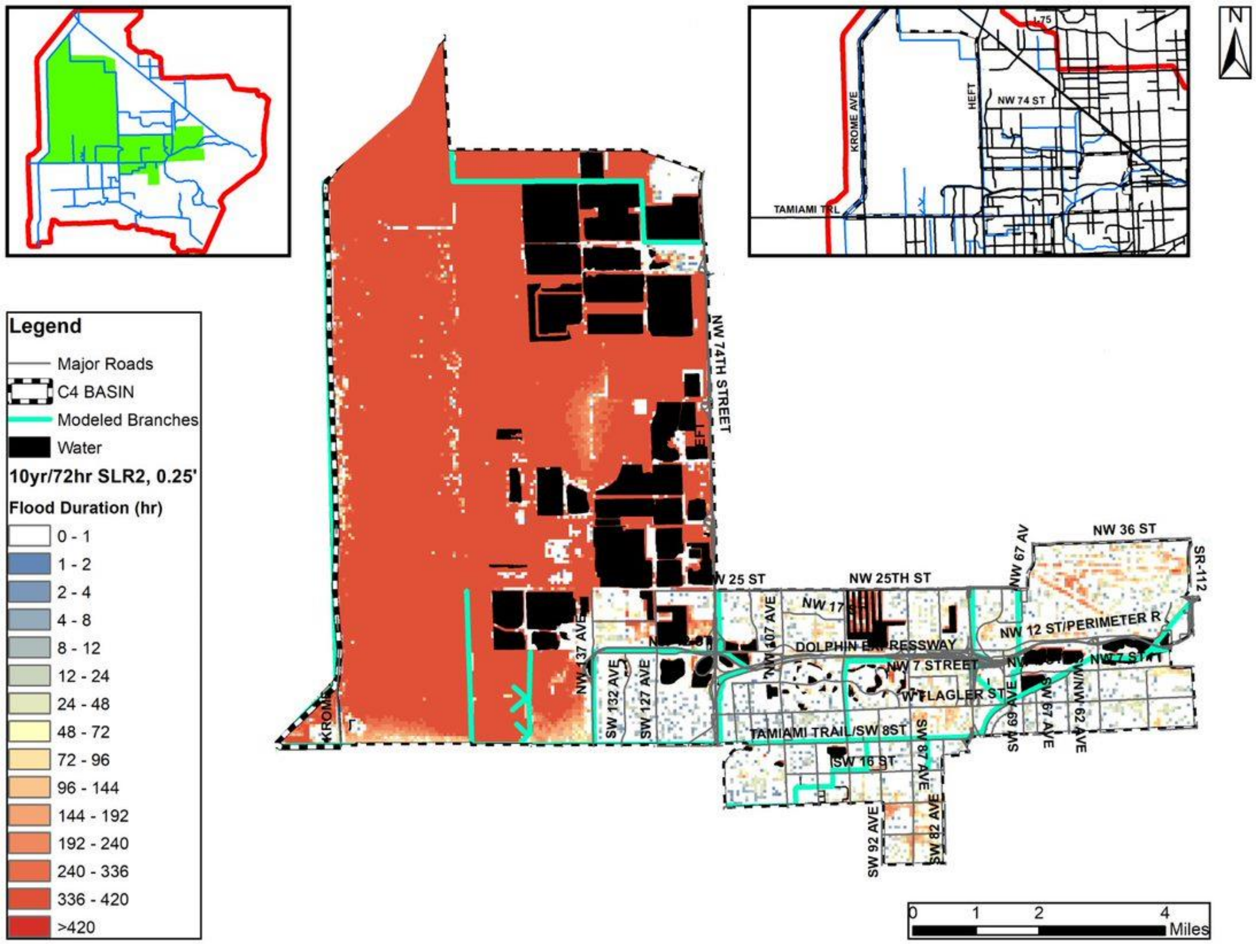


Figure D 3-8. Flood Duration Map for the 10-year 3-Day Design Storm for SLR3 in the C4 Watershed

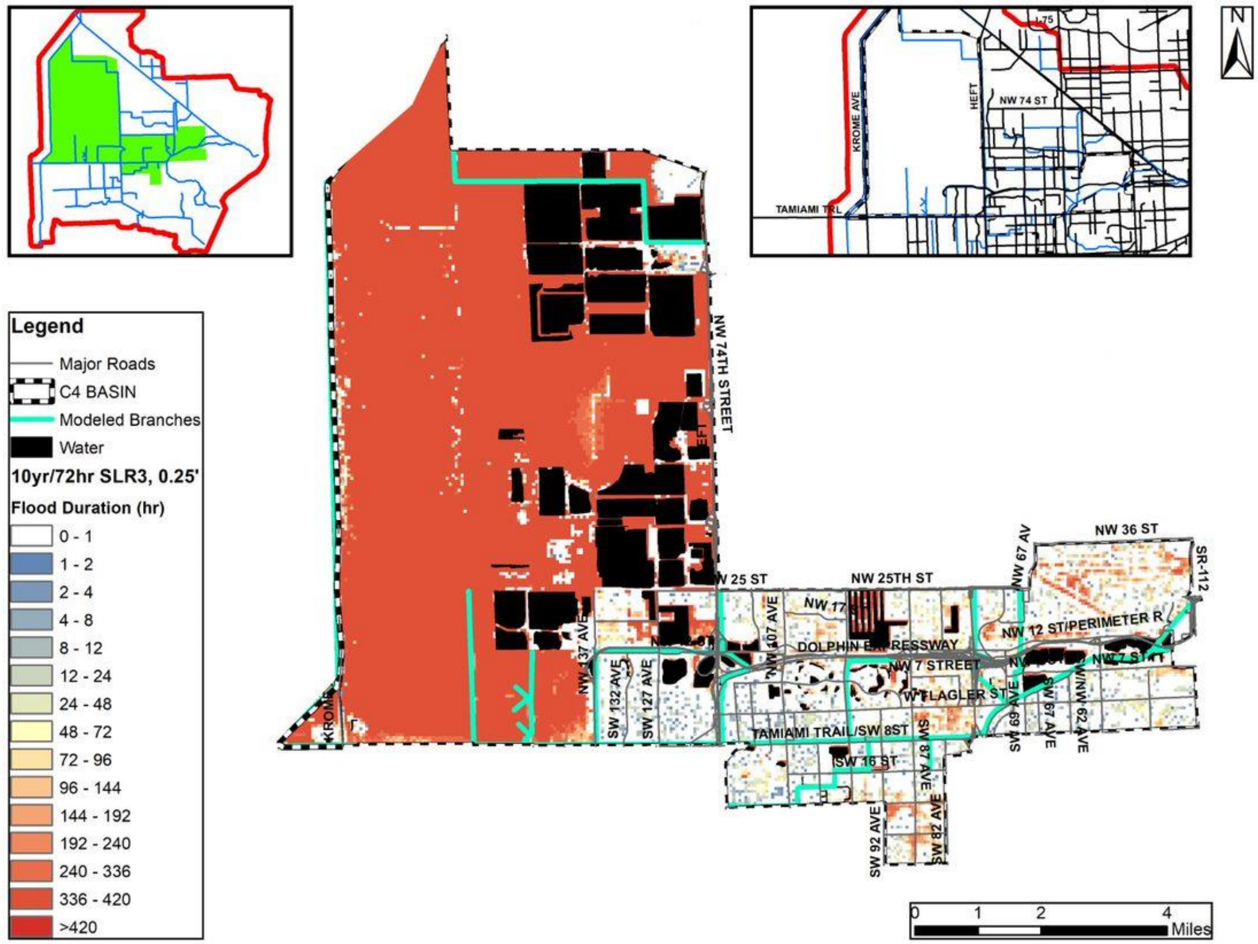


Figure D 3-9. Flood Duration Map for the 25-year 3-Day Design Storm for Current Conditions in the C4 Watershed

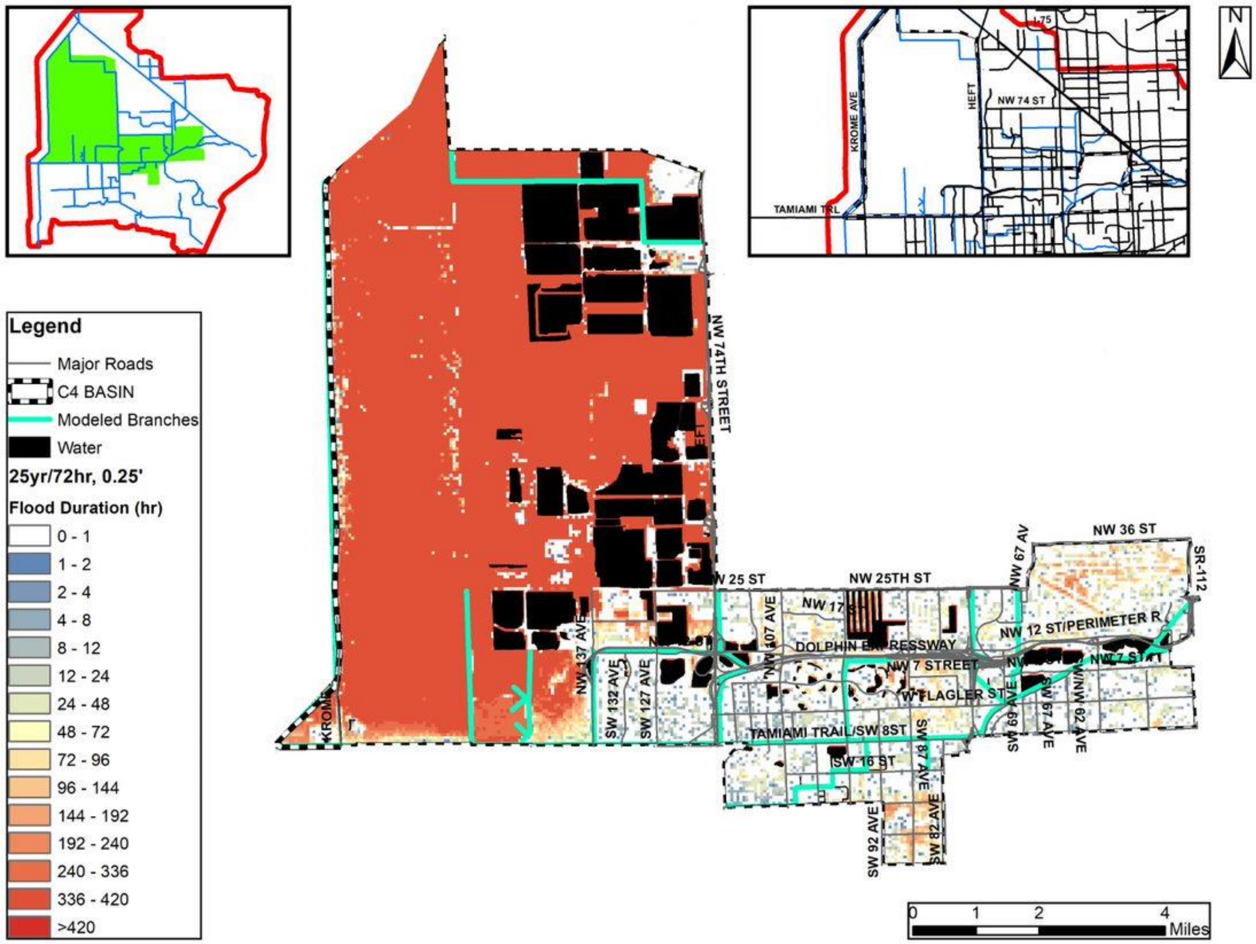


Figure D 3-10. Flood Duration Map for the 25-year 3-Day Design Storm for SLR1 in the C4 Watershed

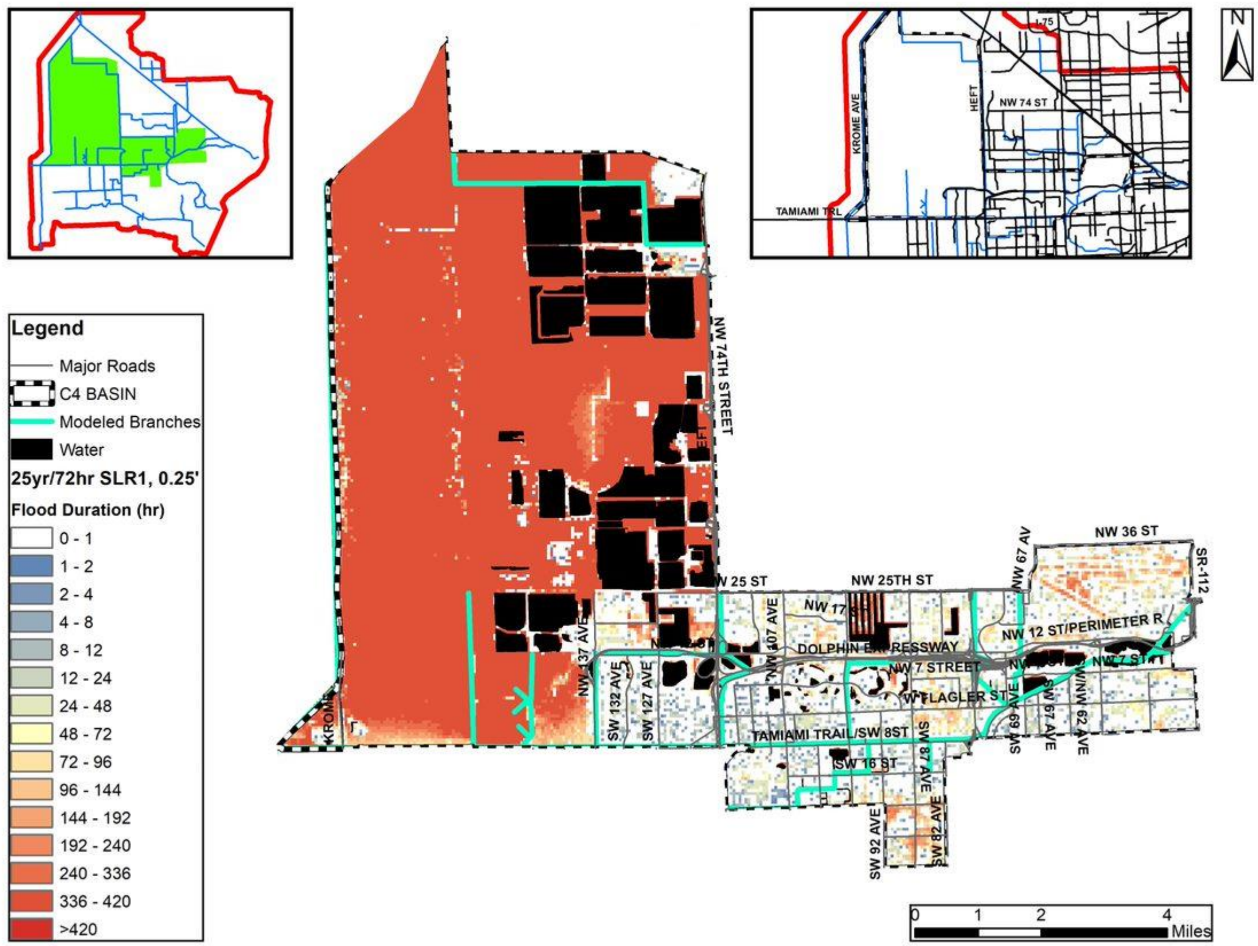


Figure D 3-11. Flood Duration Map for the 25-year 3-Day Design Storm for SLR2 in the C4 Watershed

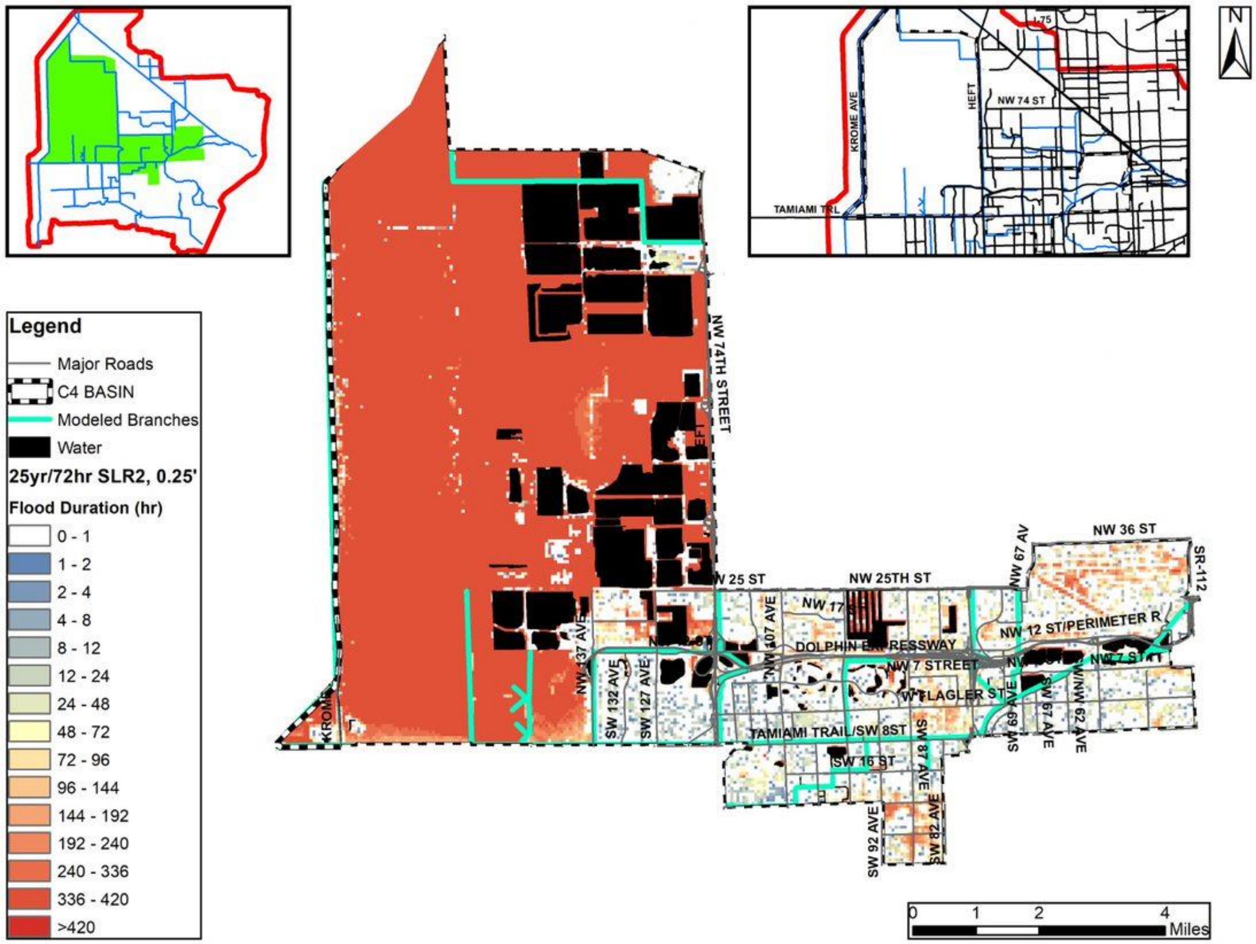


Figure D 3-12. Flood Duration Map for the 25-year 3-Day Design Storm for SLR3 in the C4 Watershed

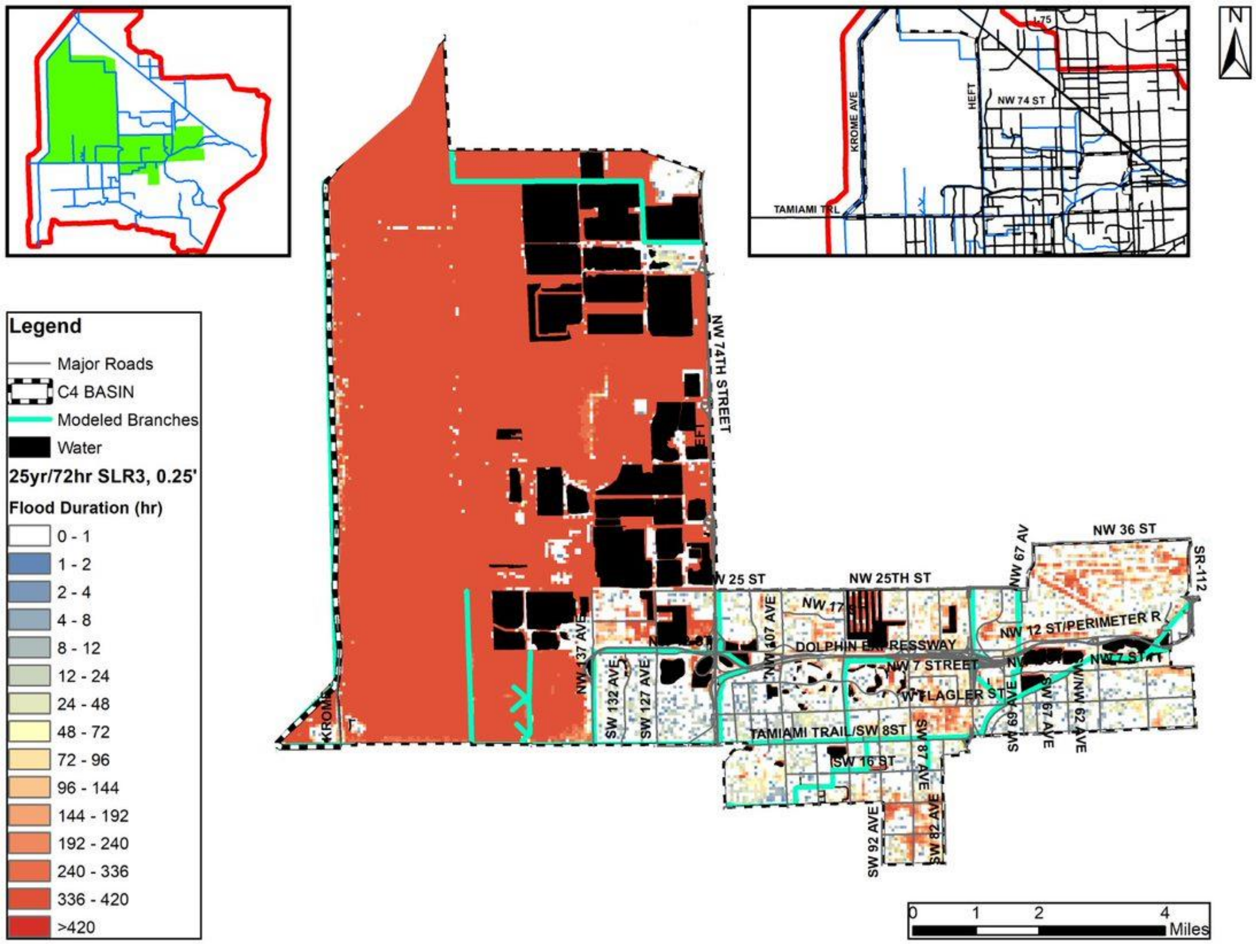


Figure D 3-13. Flood Duration Map for the 100-year 3-Day Design Storm for Current Conditions in the C4 Watershed

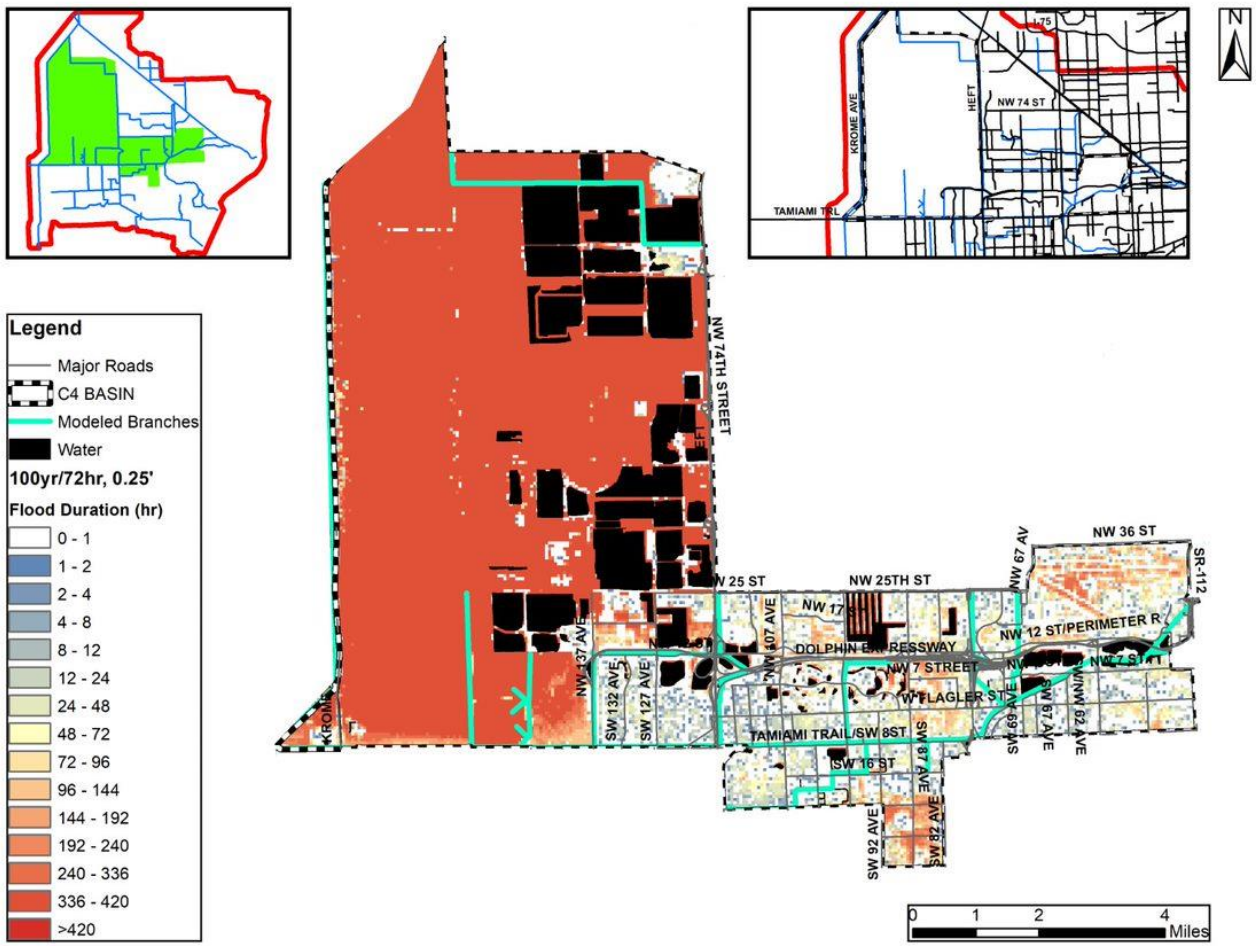


Figure D 3-14. Flood Duration Map for the 100-year 3-Day Design Storm for SLR1 in the C4 Watershed

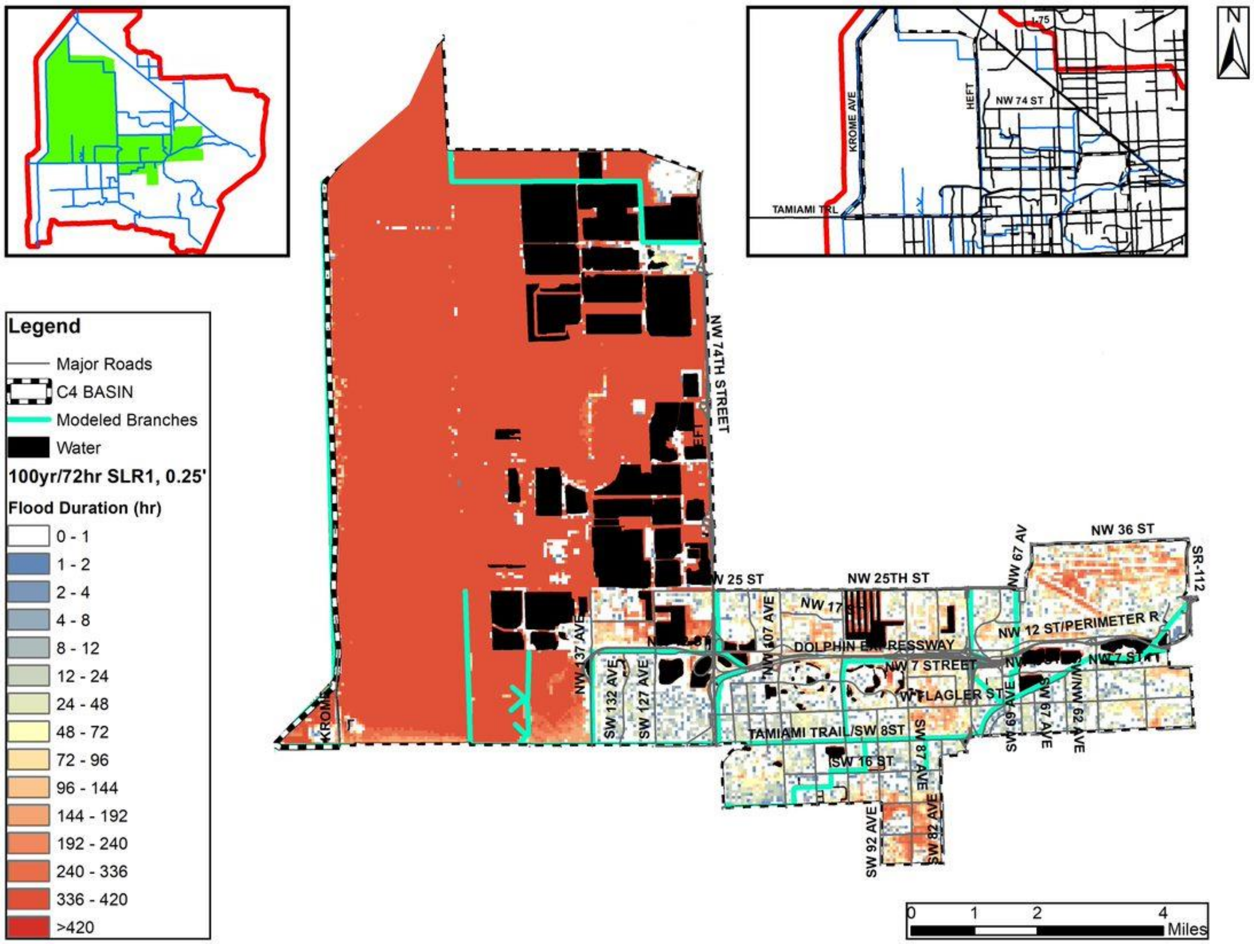


Figure D 3-15. Flood Duration Map for the 100-year 3-Day Design Storm for SLR2 in the C4 Watershed

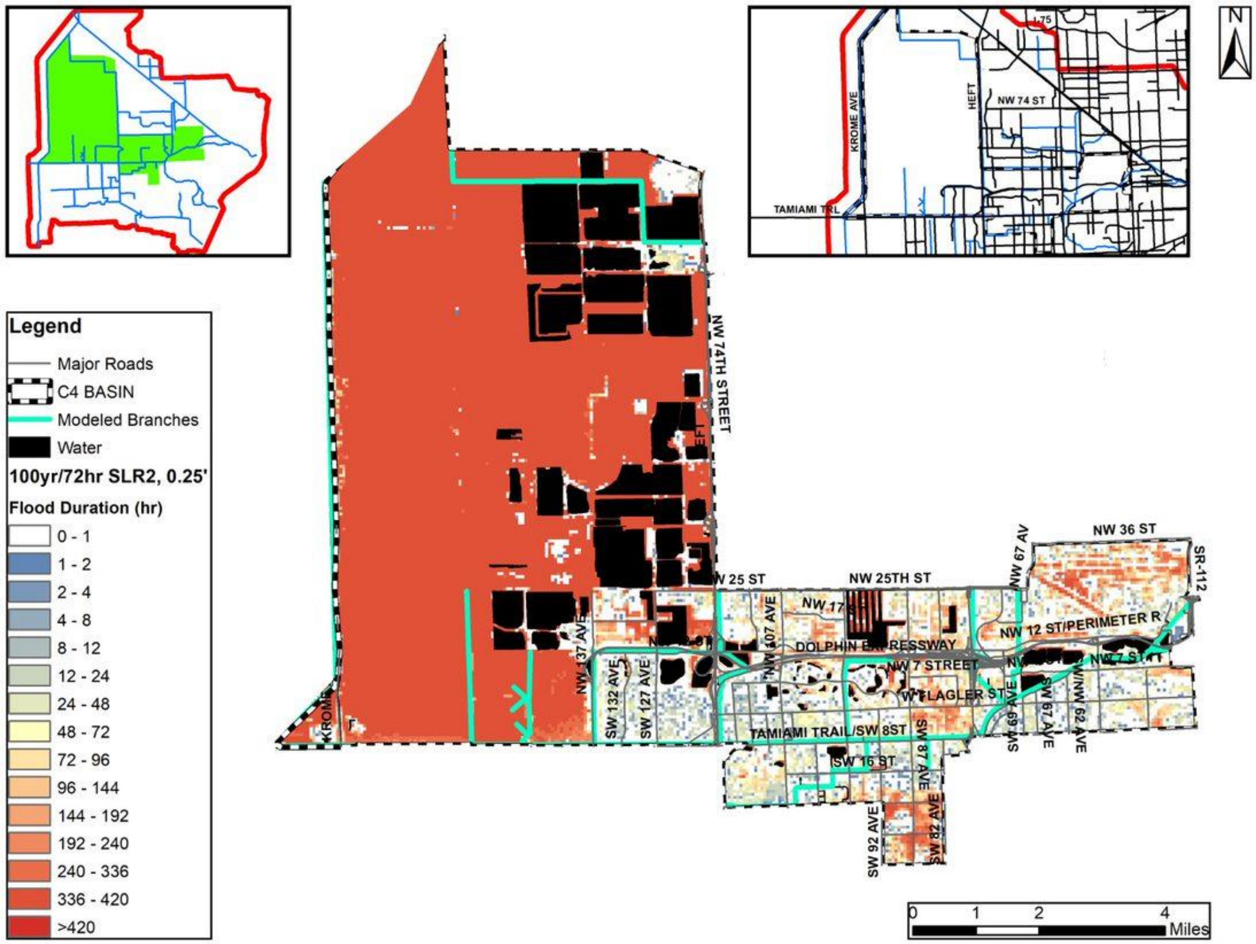


Figure D 3-16. Flood Duration Map for the 100-year 3-Day Design Storm for SLR3 in the C4 Watershed

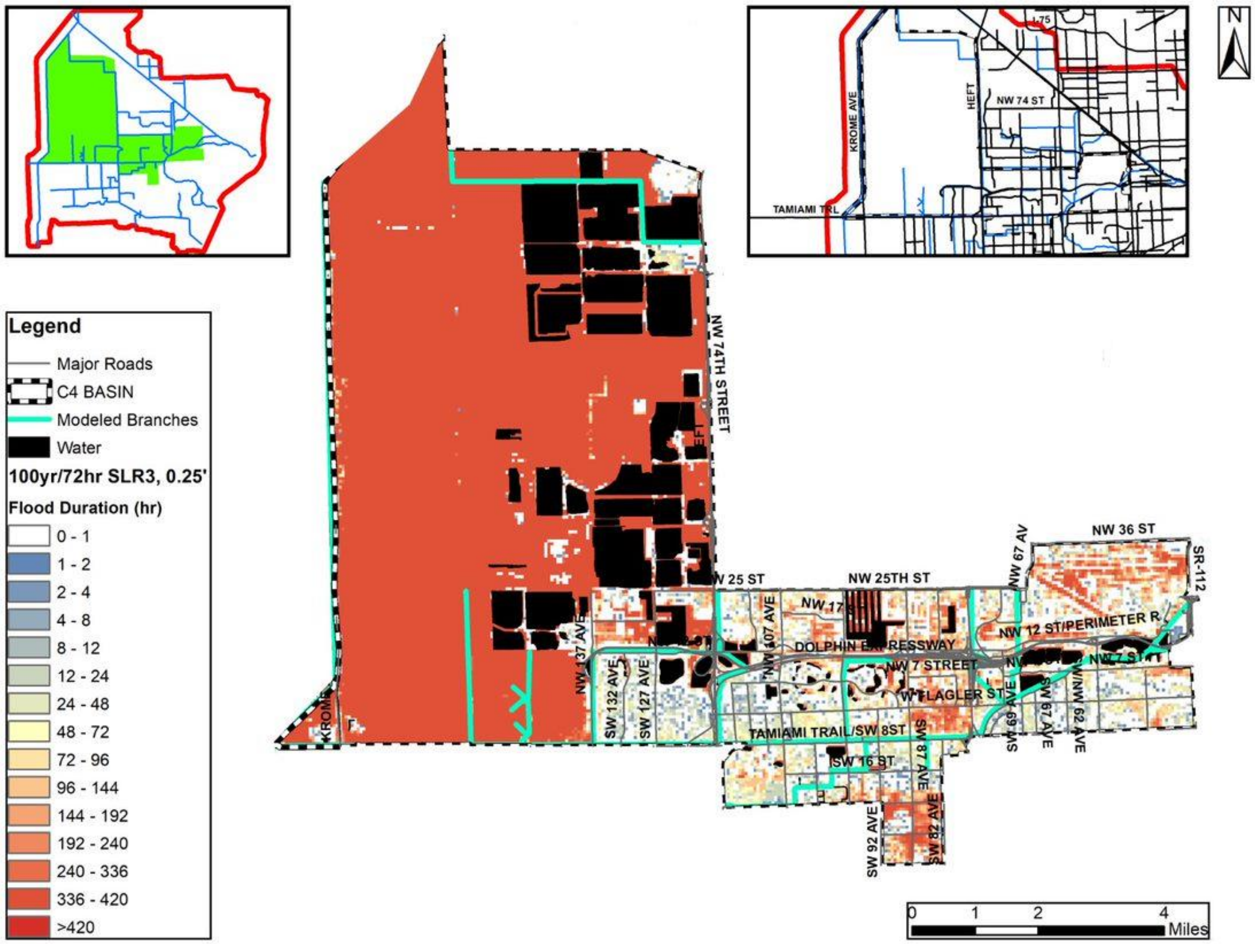


Figure D 3-17. Flood Duration Map for the 5-year 3-Day Design Storm for Current Conditions in Urban Areas in the C4 Watershed

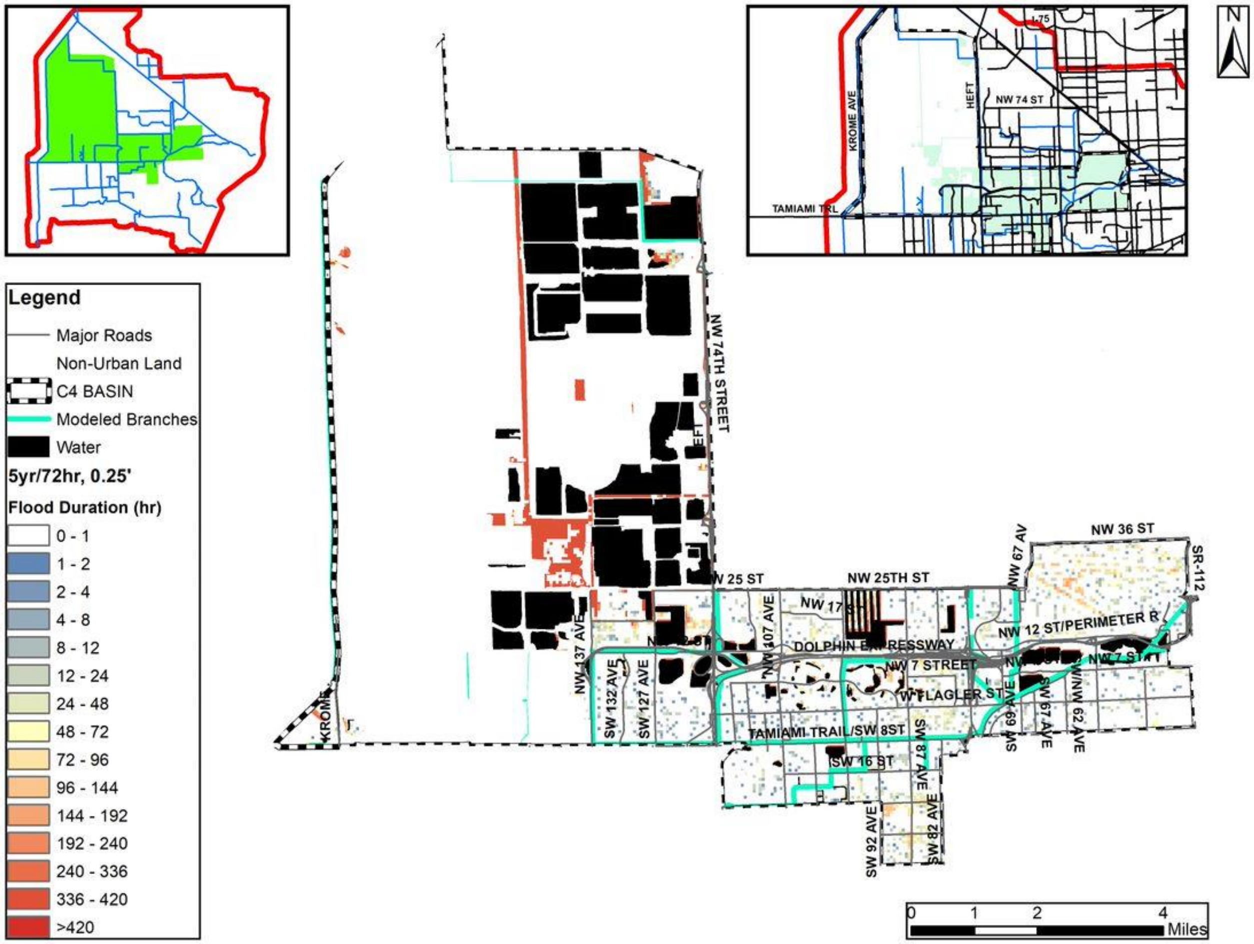


Figure D 3-18. Flood Duration Map for the 5-year 3-Day Design Storm for SLR1 in Urban Areas in the C4 Watershed

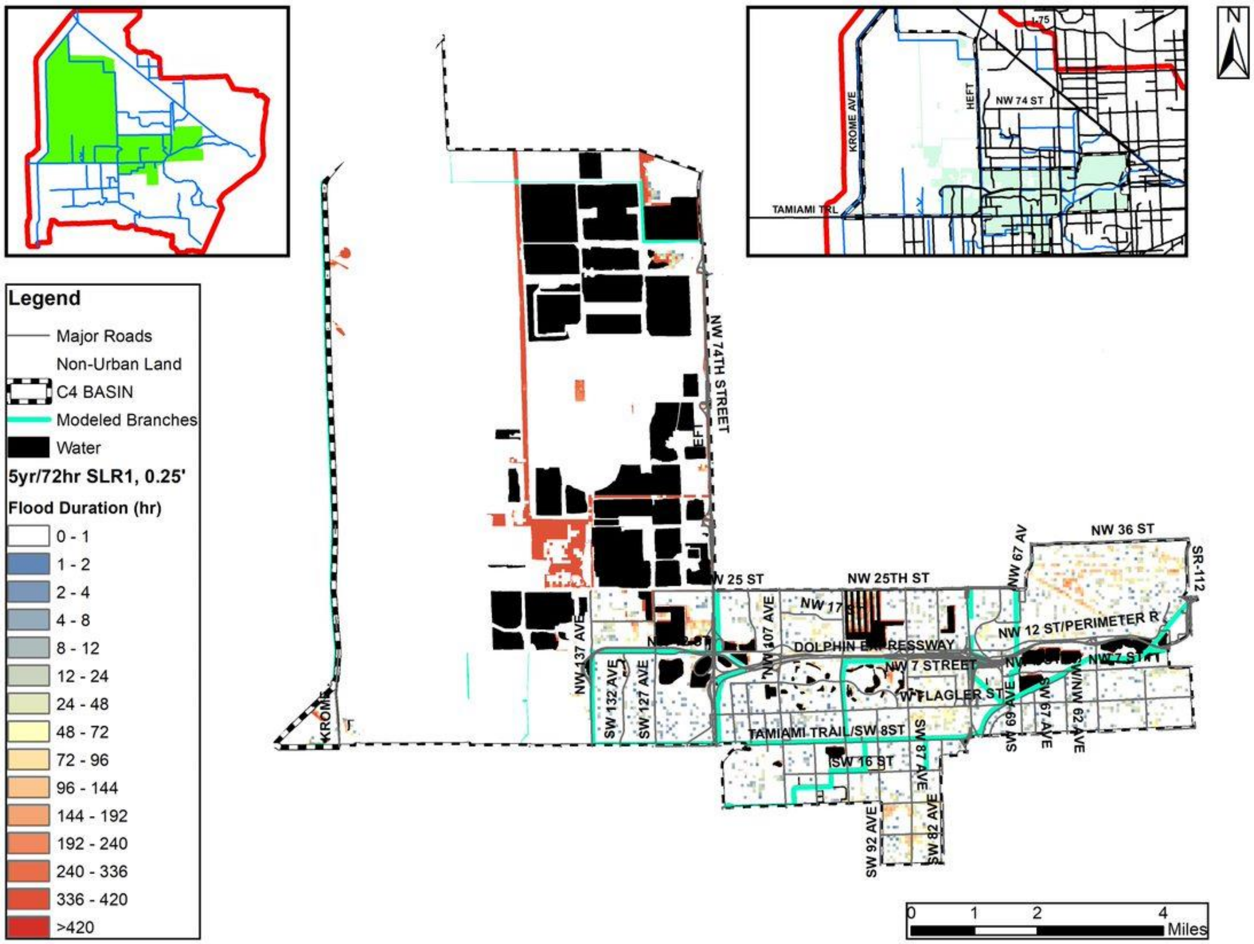


Figure D 3-19. Flood Duration Map for the 5-year 3-Day Design Storm for SLR2 in Urban Areas in the C4 Watershed

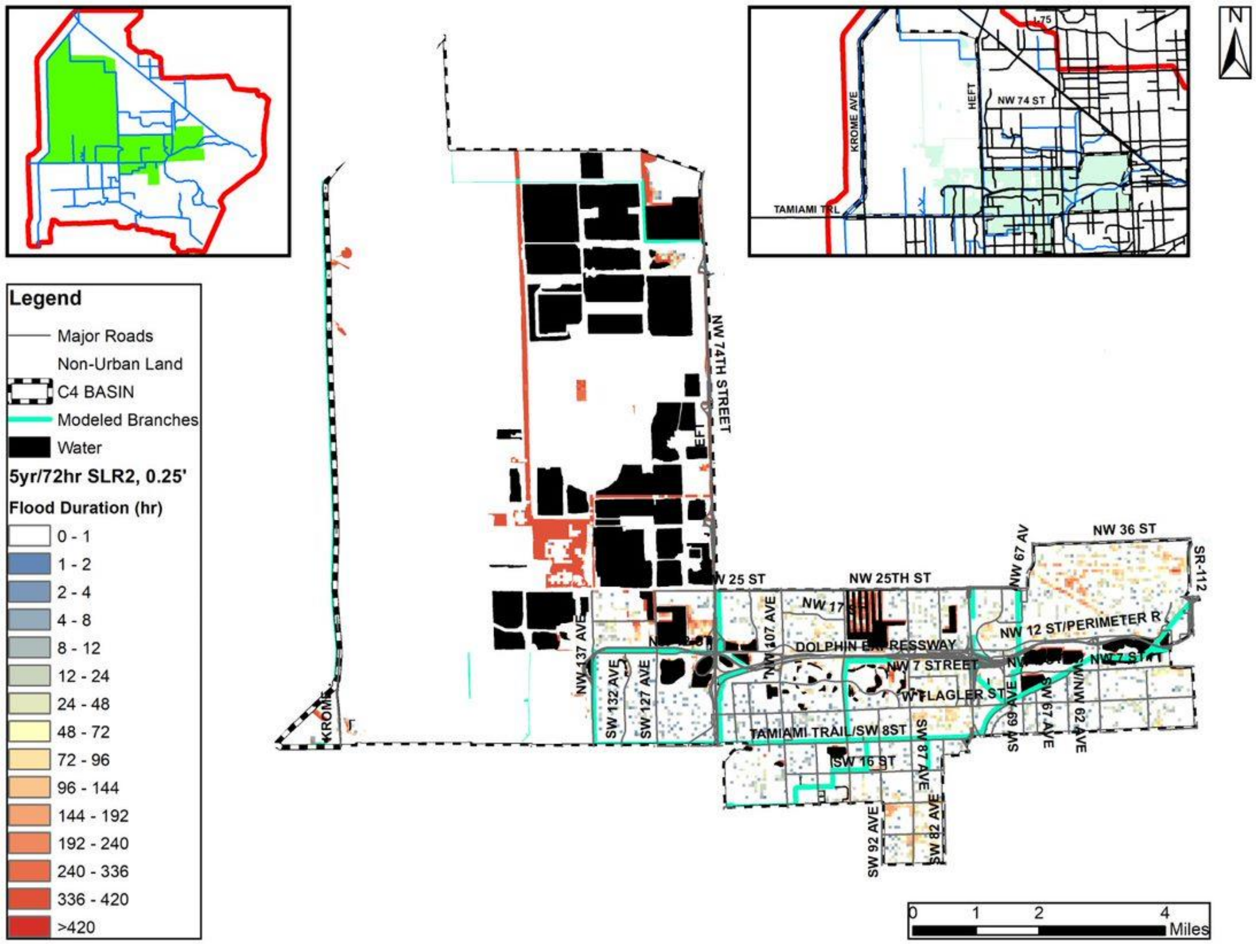


Figure D 3-20. Flood Duration Map for the 5-year 3-Day Design Storm for SLR3 in Urban Areas in the C4 Watershed

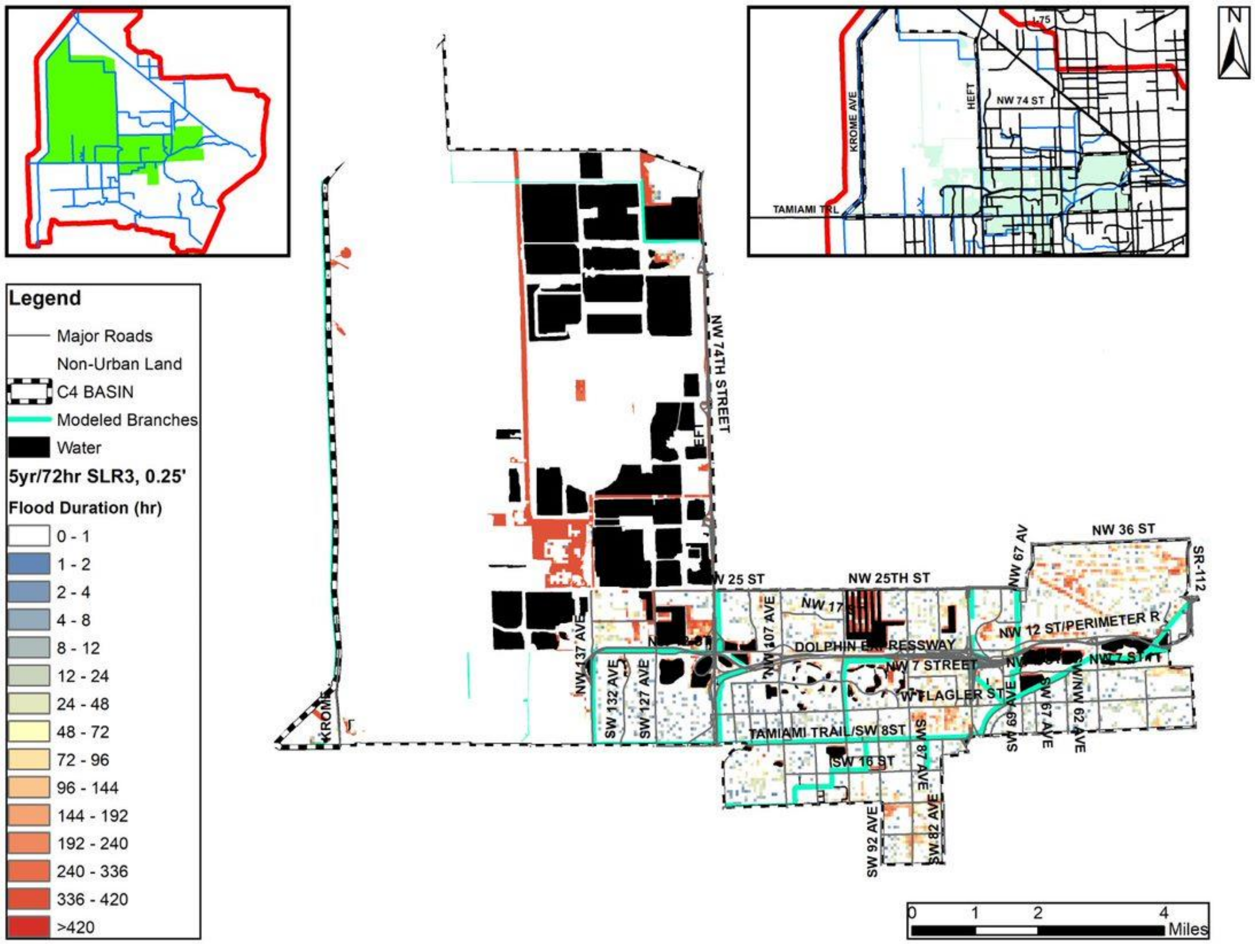


Figure D 3-21. Flood Duration Map for the 10-year 3-Day Design Storm for Current Conditions in Urban Areas in the C4 Watershed

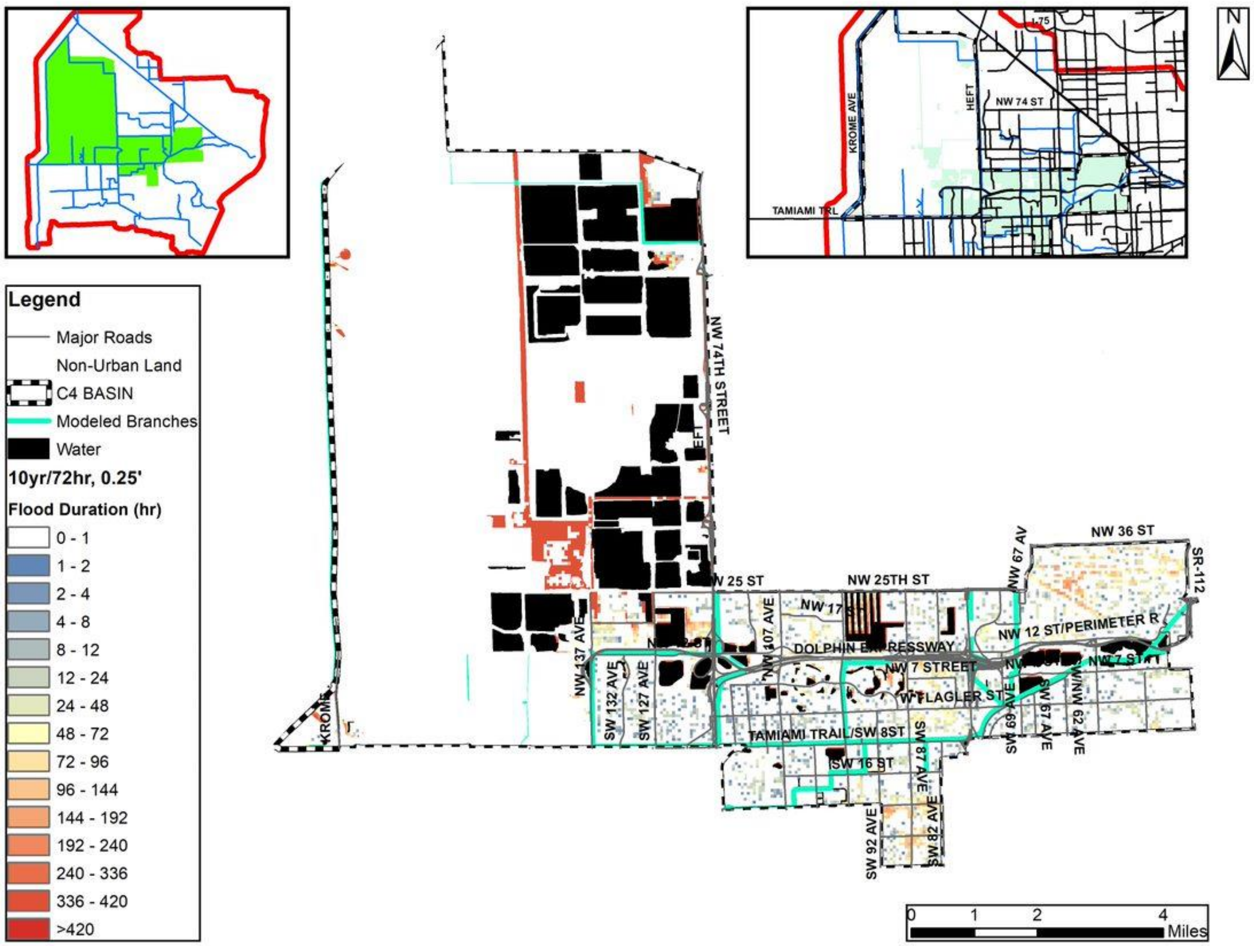


Figure D 3-22. Flood Duration Map for the 10-year 3-Day Design Storm for SLR1 in Urban Areas in the C4 Watershed

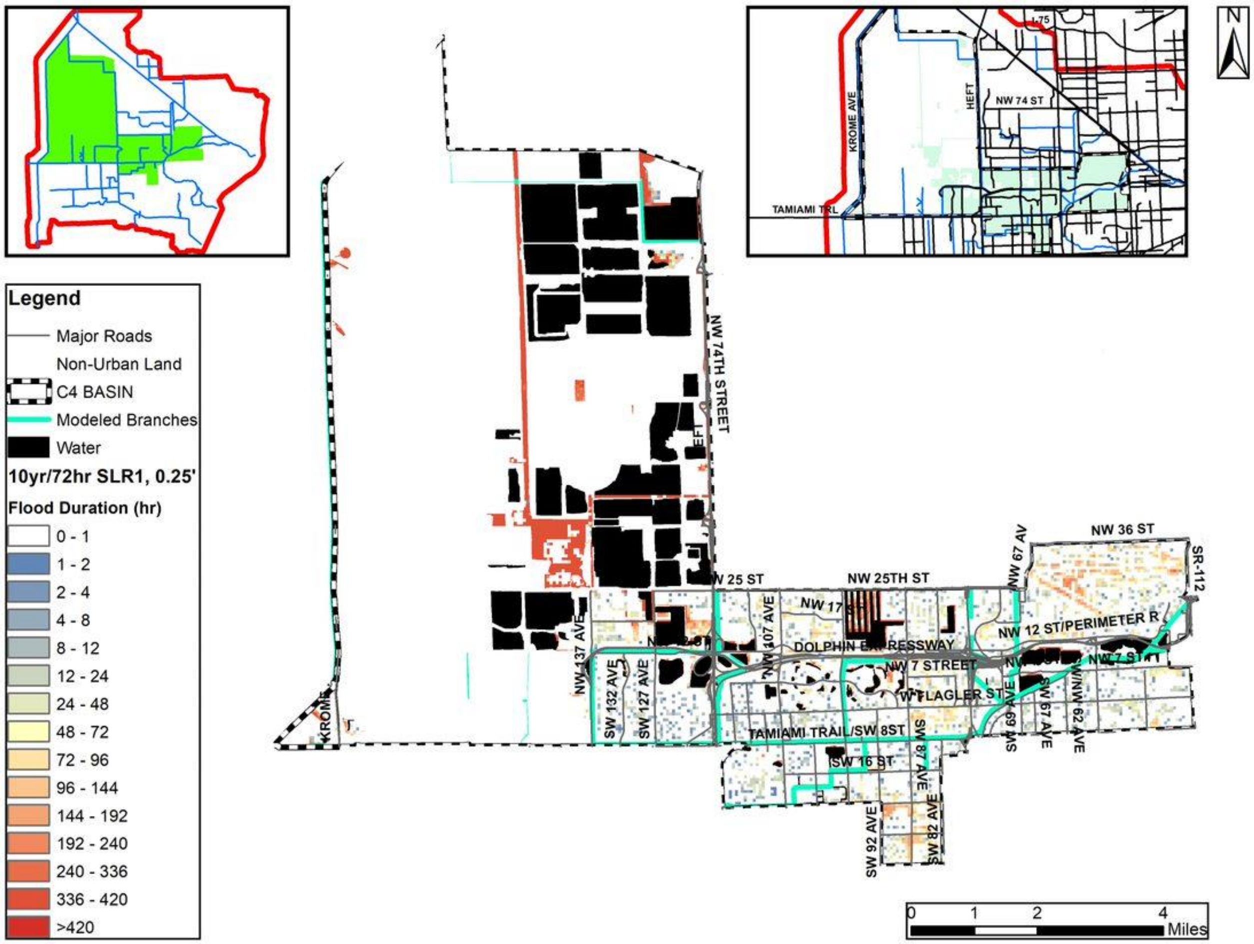


Figure D 3-23. Flood Duration Map for the 10-year 3-Day Design Storm for SLR2 in Urban Areas in the C4 Watershed

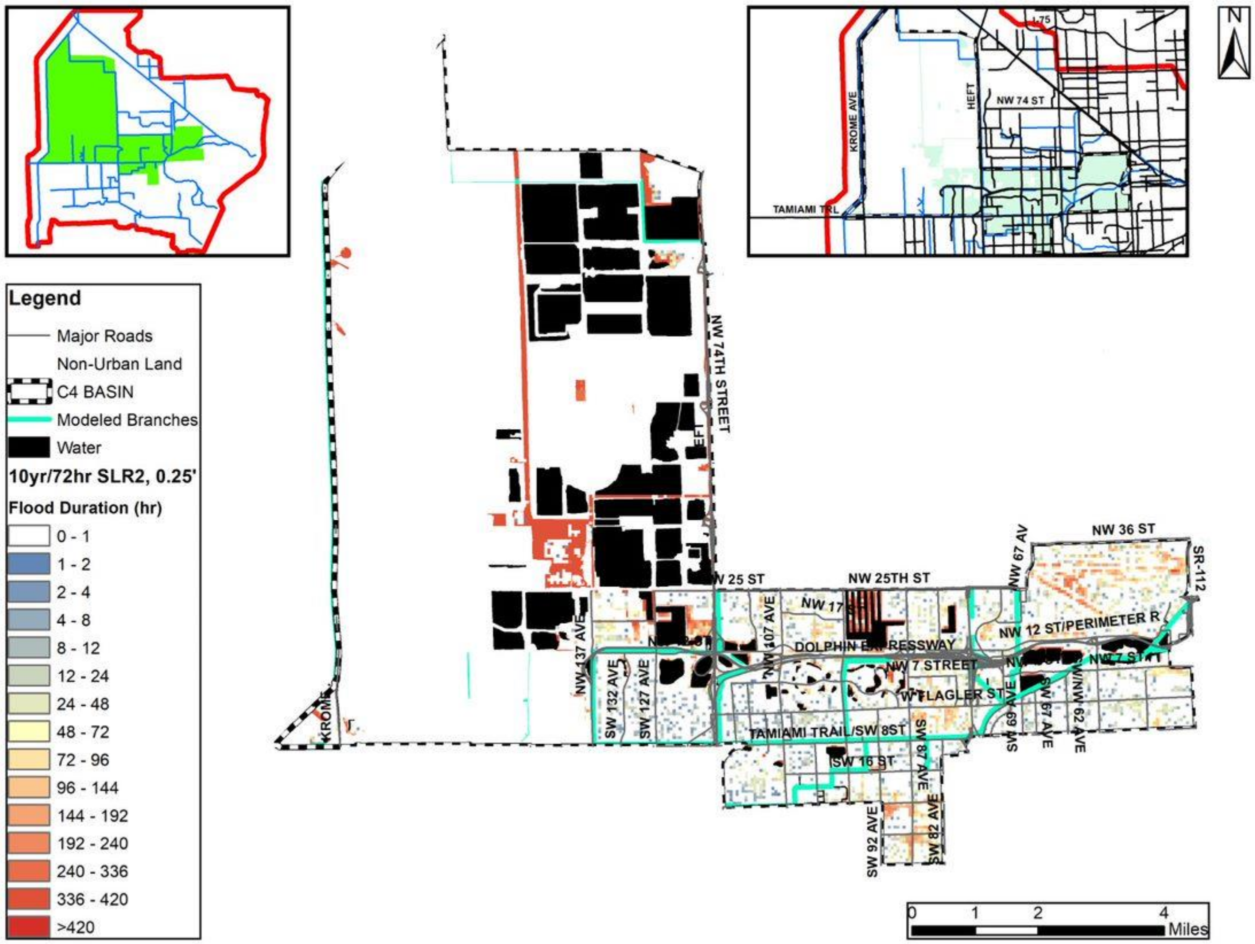


Figure D 3-24. Flood Duration Map for the 10-year 3-Day Design Storm for SLR3 in Urban Areas in the C4 Watershed

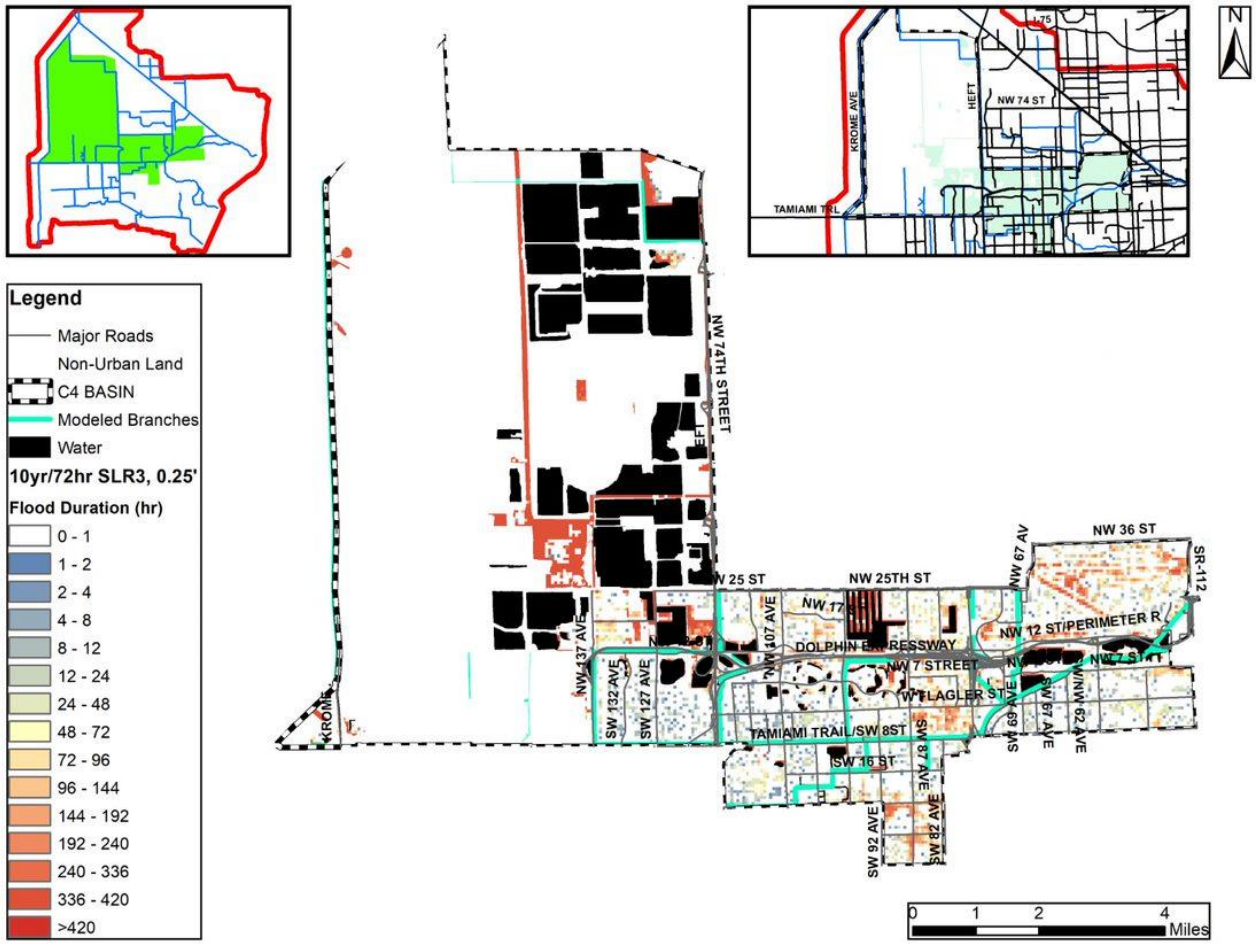


Figure D 3-25. Flood Duration Map for the 25-year 3-Day Design Storm for Current Conditions in Urban Areas in the C4 Watershed

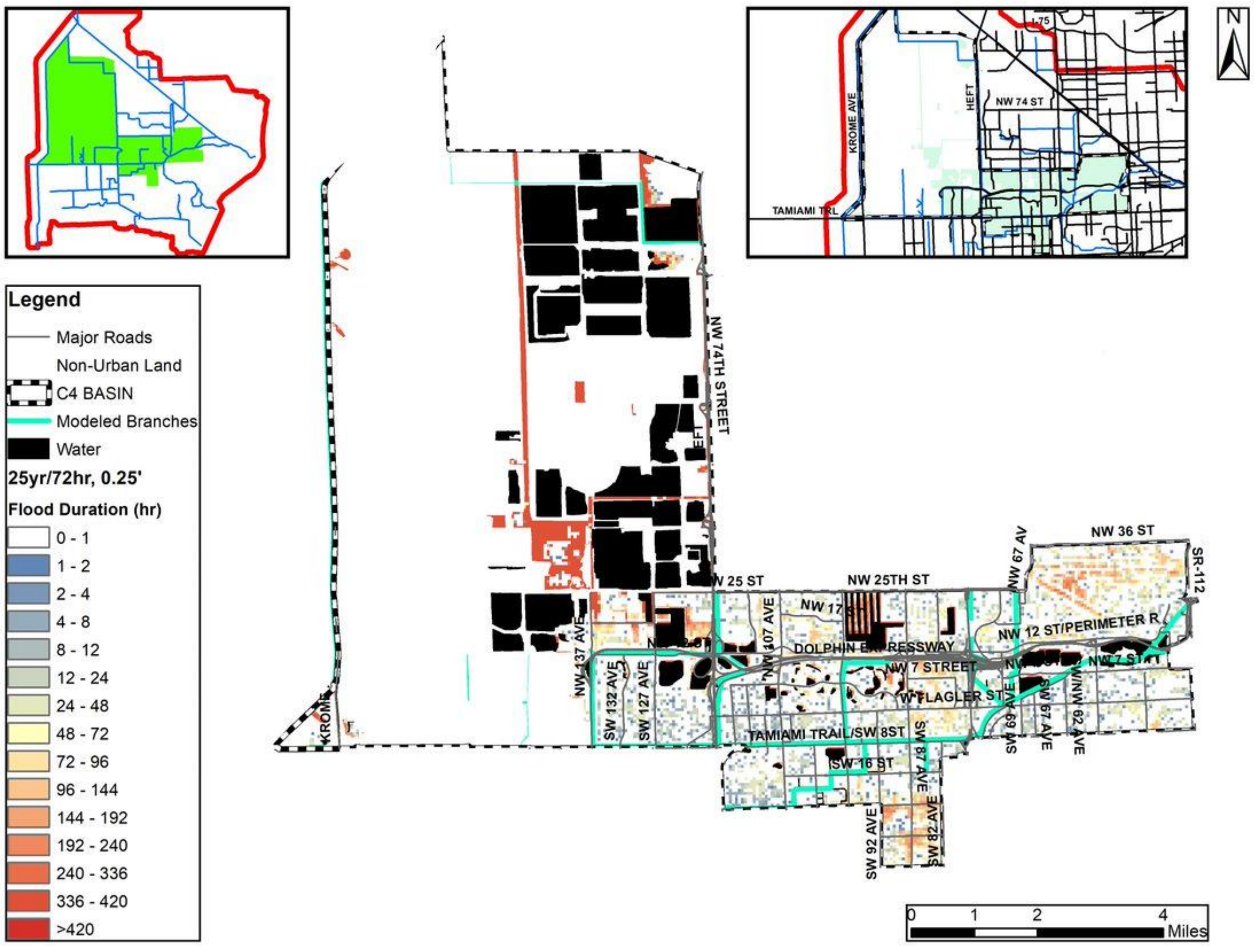


Figure D 3-26. Flood Duration Map for the 25-year 3-Day Design Storm for SLR1 in Urban Areas in the C4 Watershed

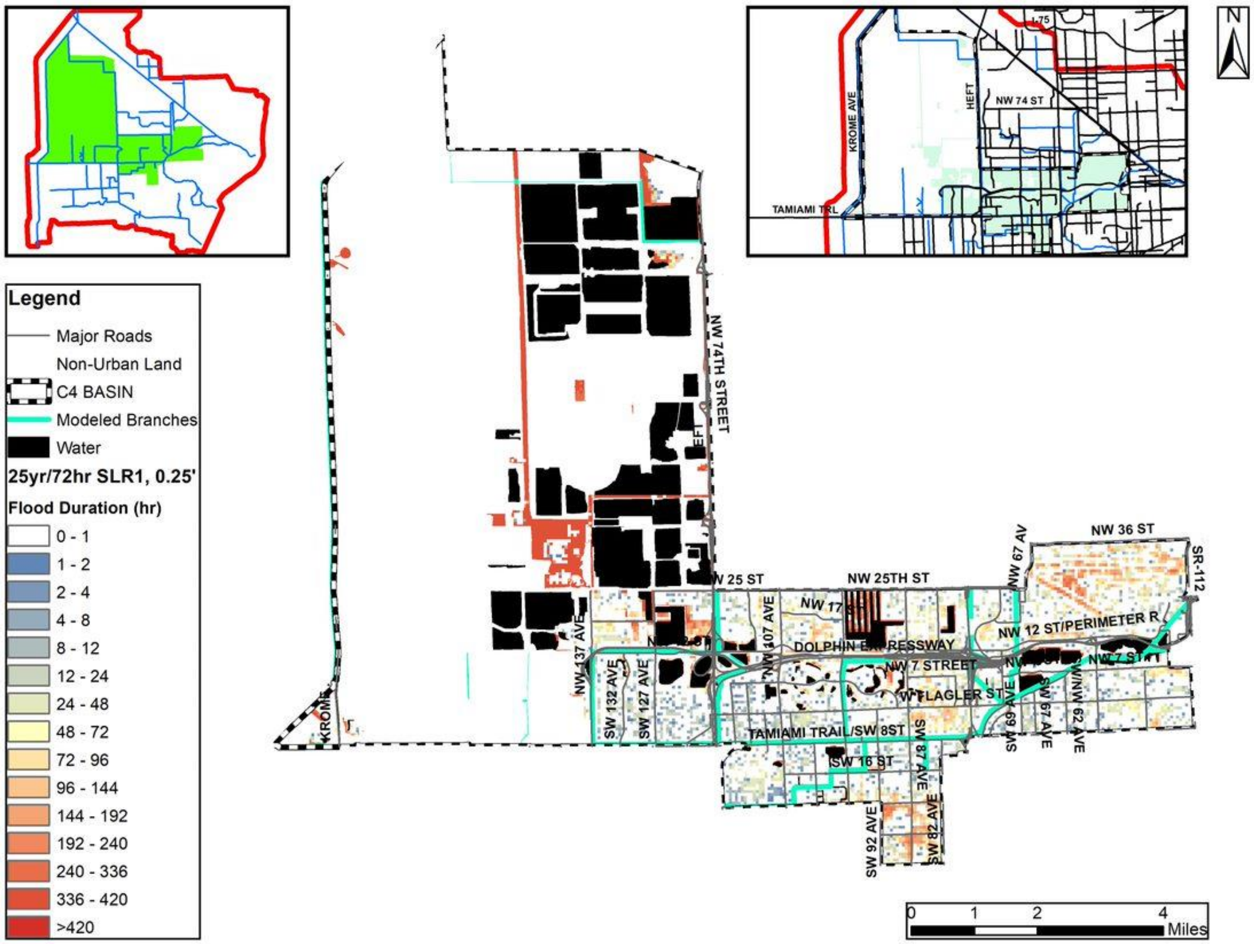


Figure D 3-27. Flood Duration Map for the 25-year 3-Day Design Storm for SLR2 in Urban Areas in the C4 Watershed

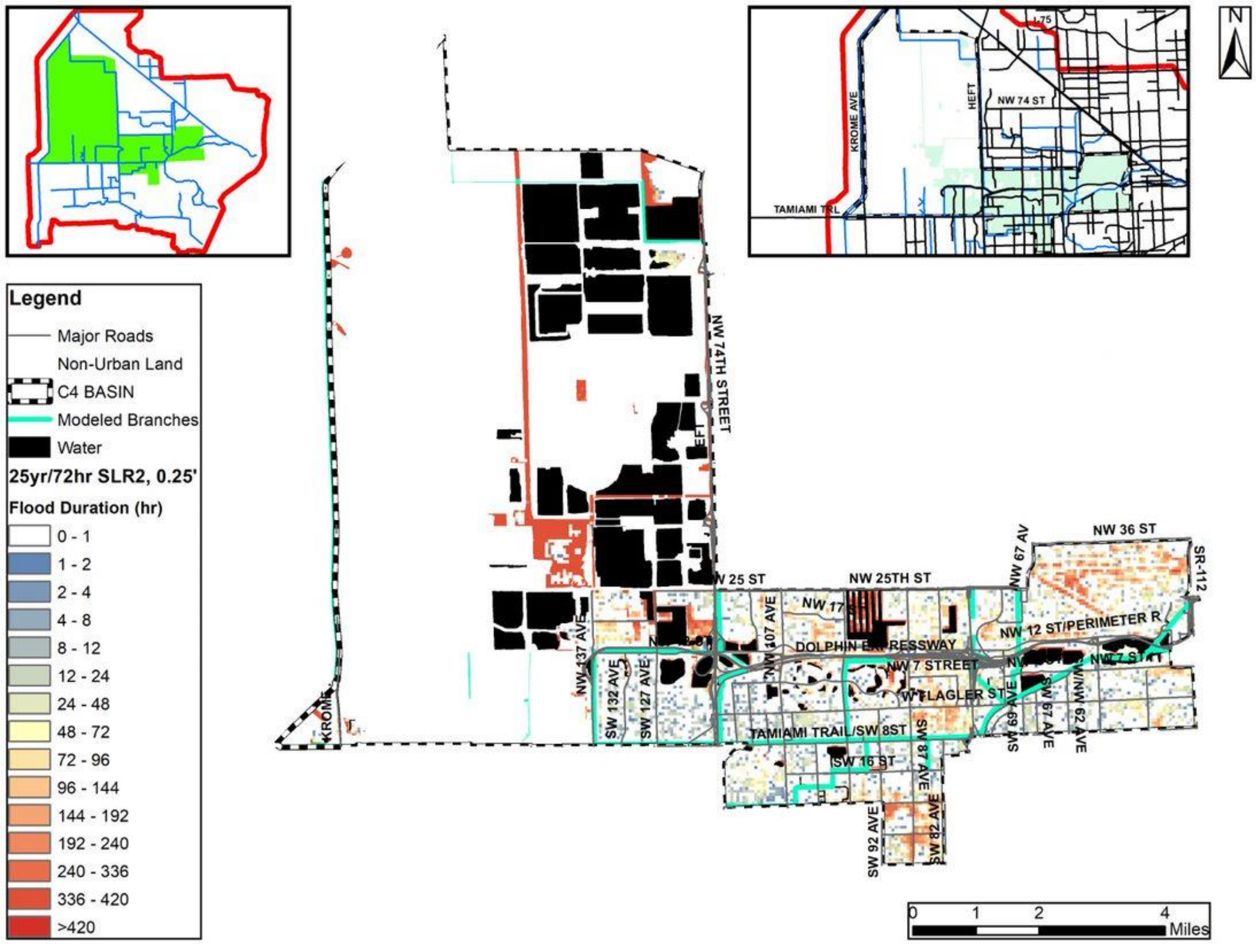


Figure D 3-28. Flood Duration Map for the 25-year 3-Day Design Storm for SLR3 in Urban Areas in the C4 Watershed

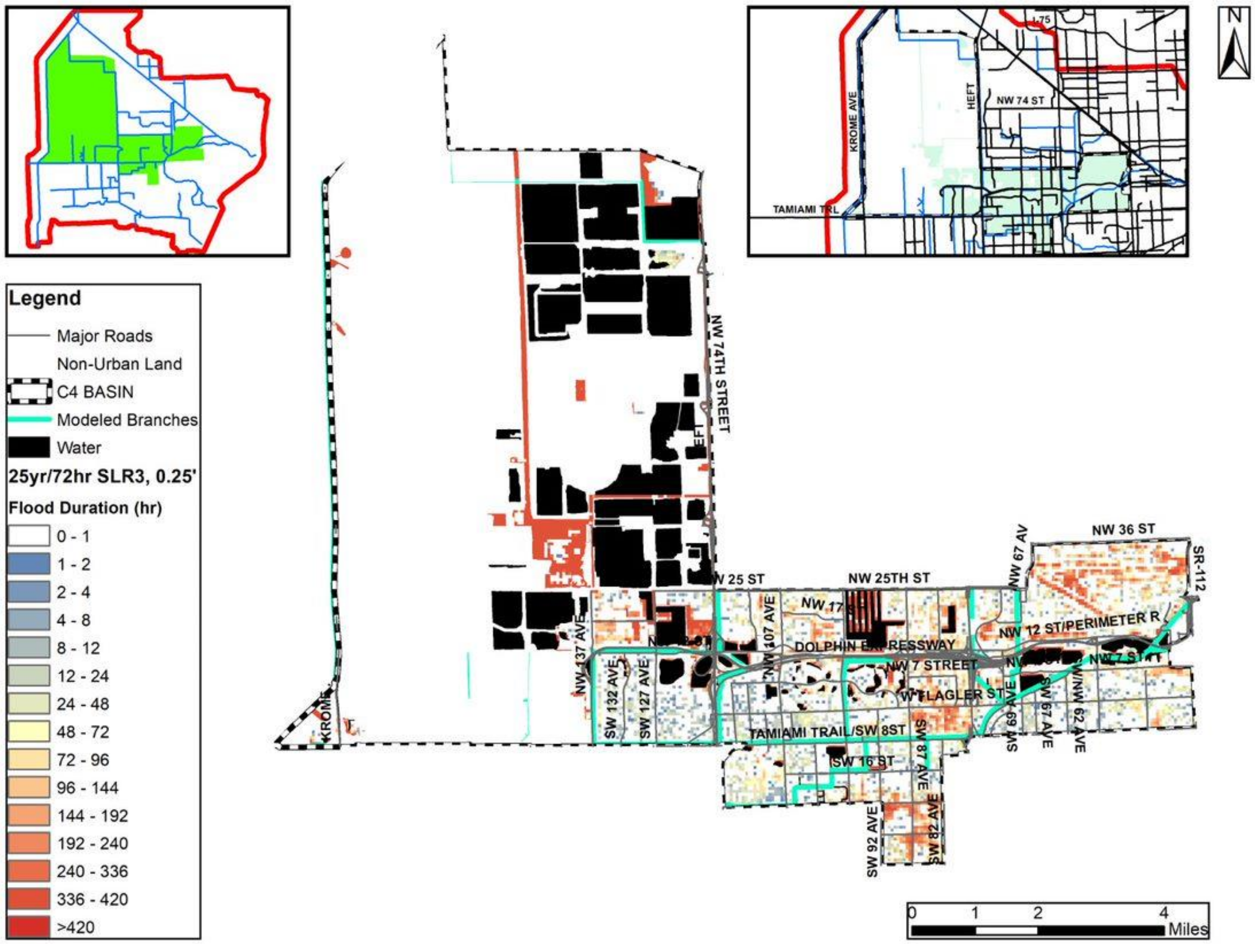


Figure D 3-29. Flood Duration Map for the 100-year 3-Day Design Storm for Current Conditions in Urban Areas in the C4 Watershed

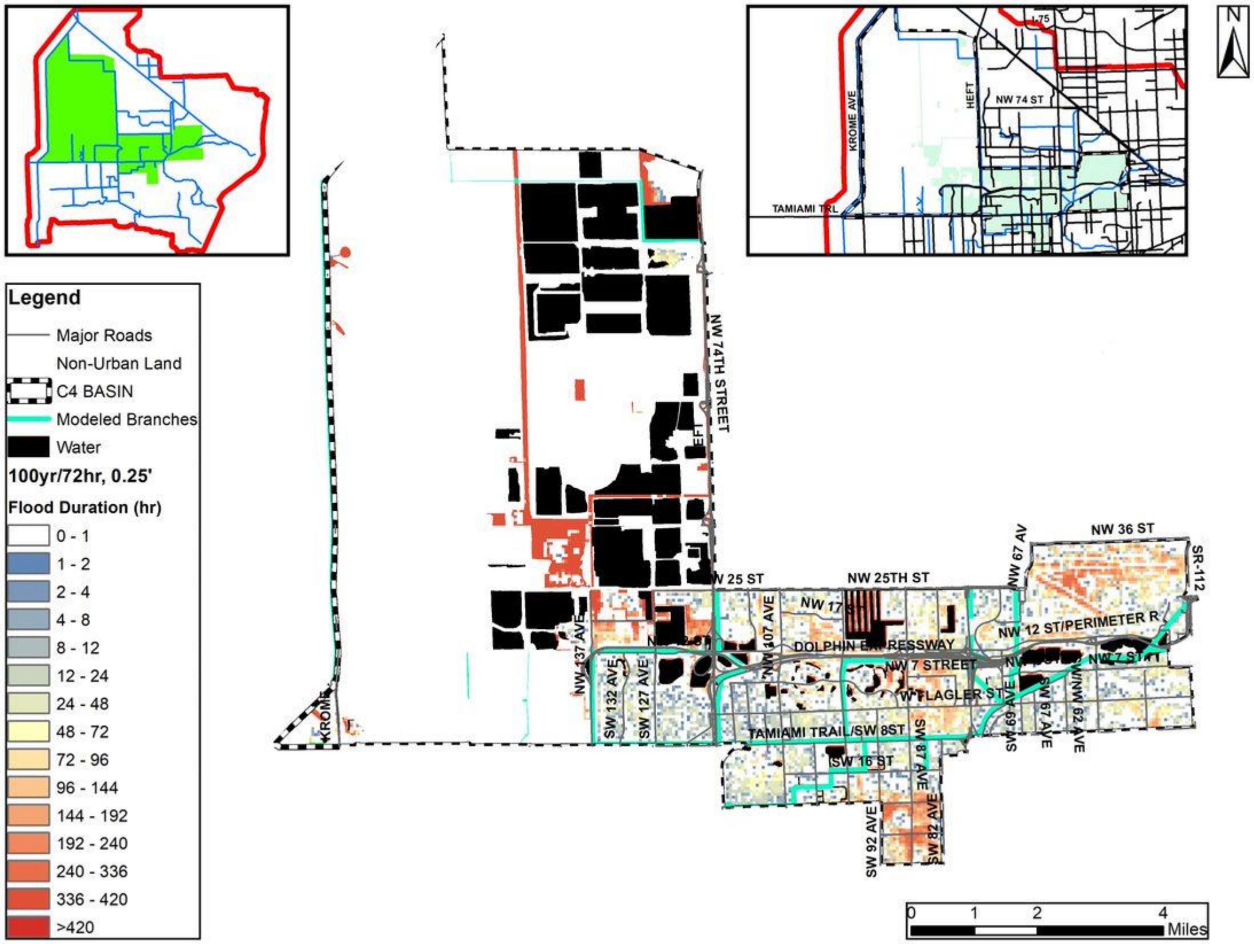


Figure D 3-30. Flood Duration Map for the 100-year 3-Day Design Storm for SLR1 in Urban Areas in the C4 Watershed

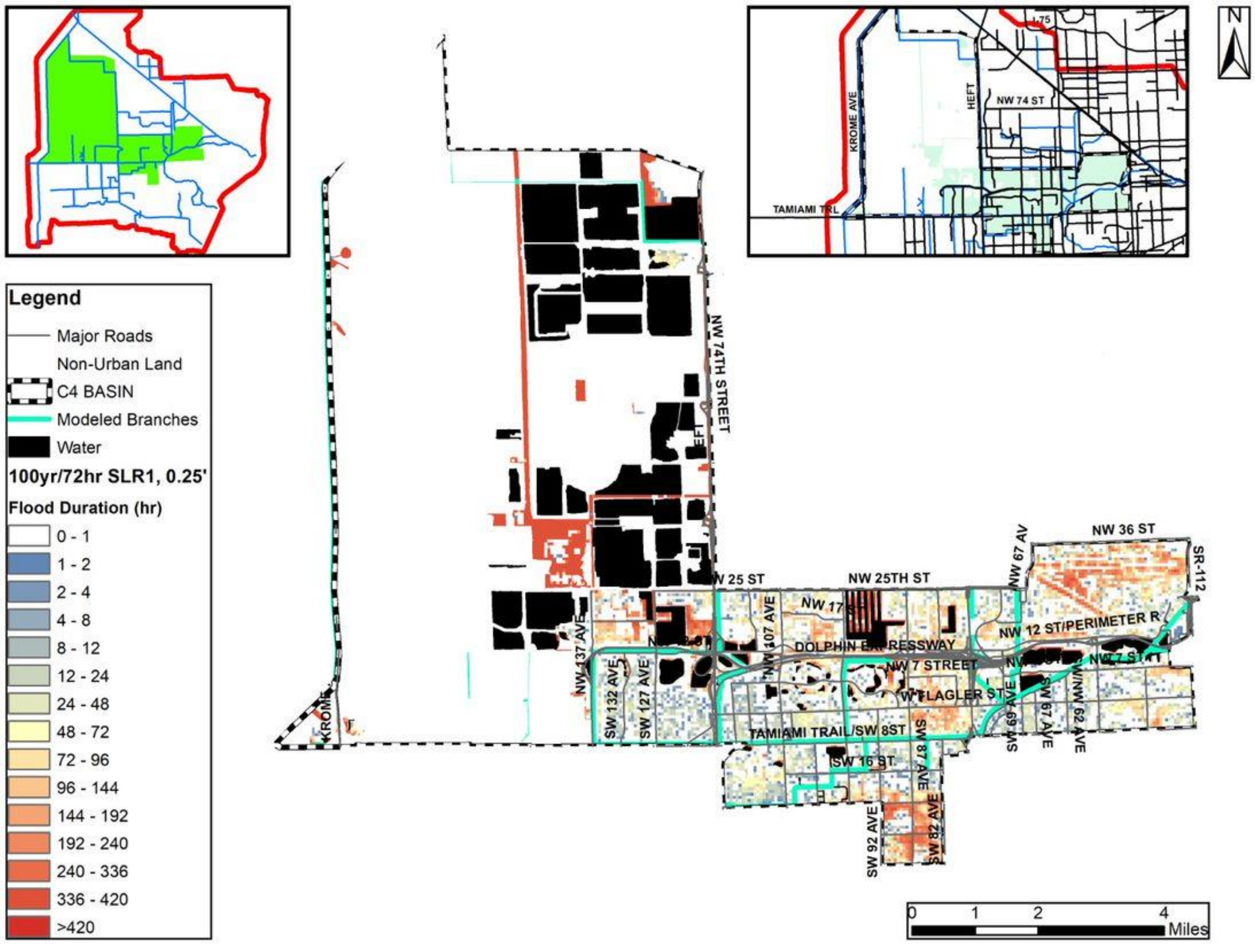


Figure D 3-31. Flood Duration Map for the 100-year 3-Day Design Storm for SLR2 in Urban Areas in the C4 Watershed

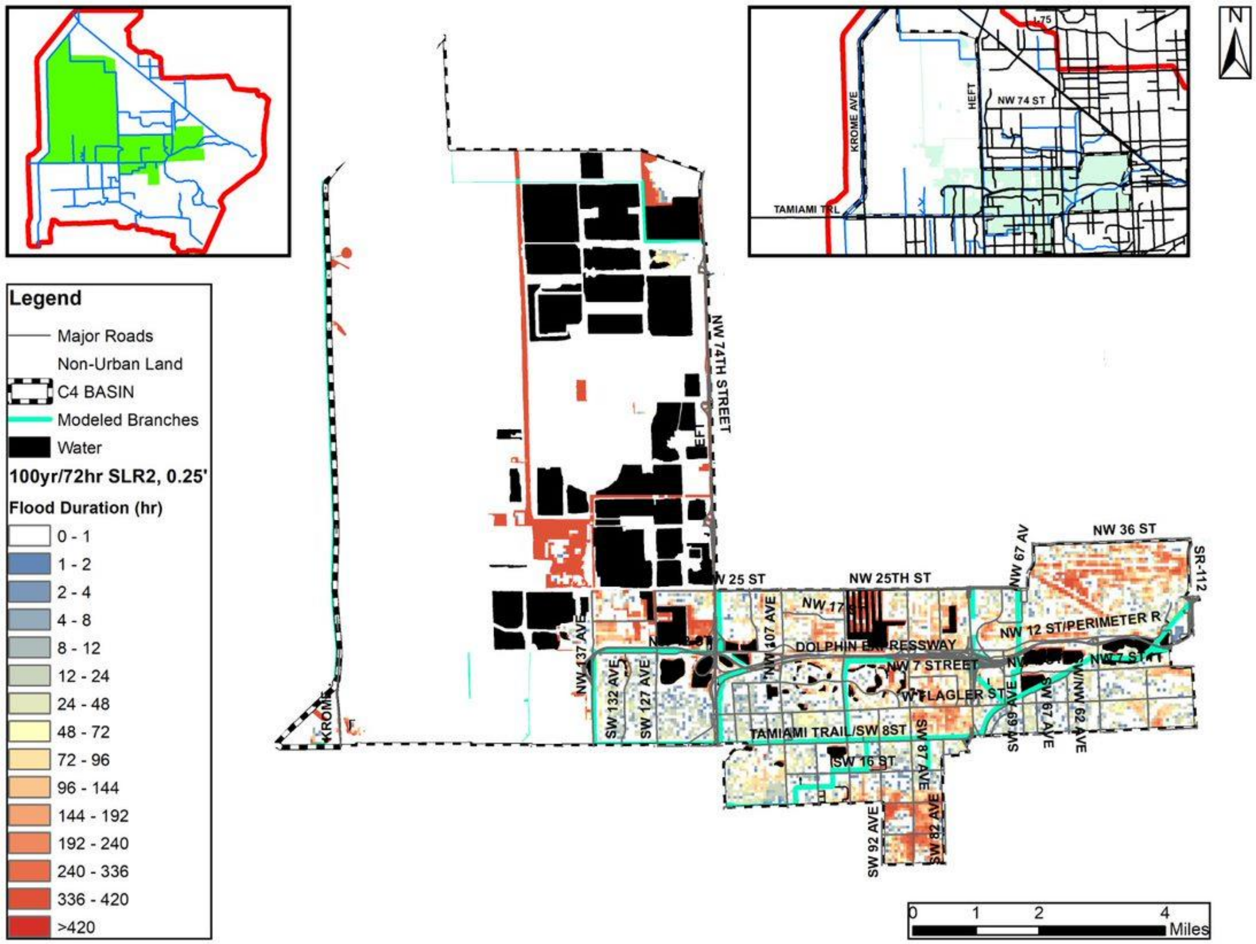


Figure D 3-32. Flood Duration Map for the 100-year 3-Day Design Storm for SLR3 in Urban Areas in the C4 Watershed

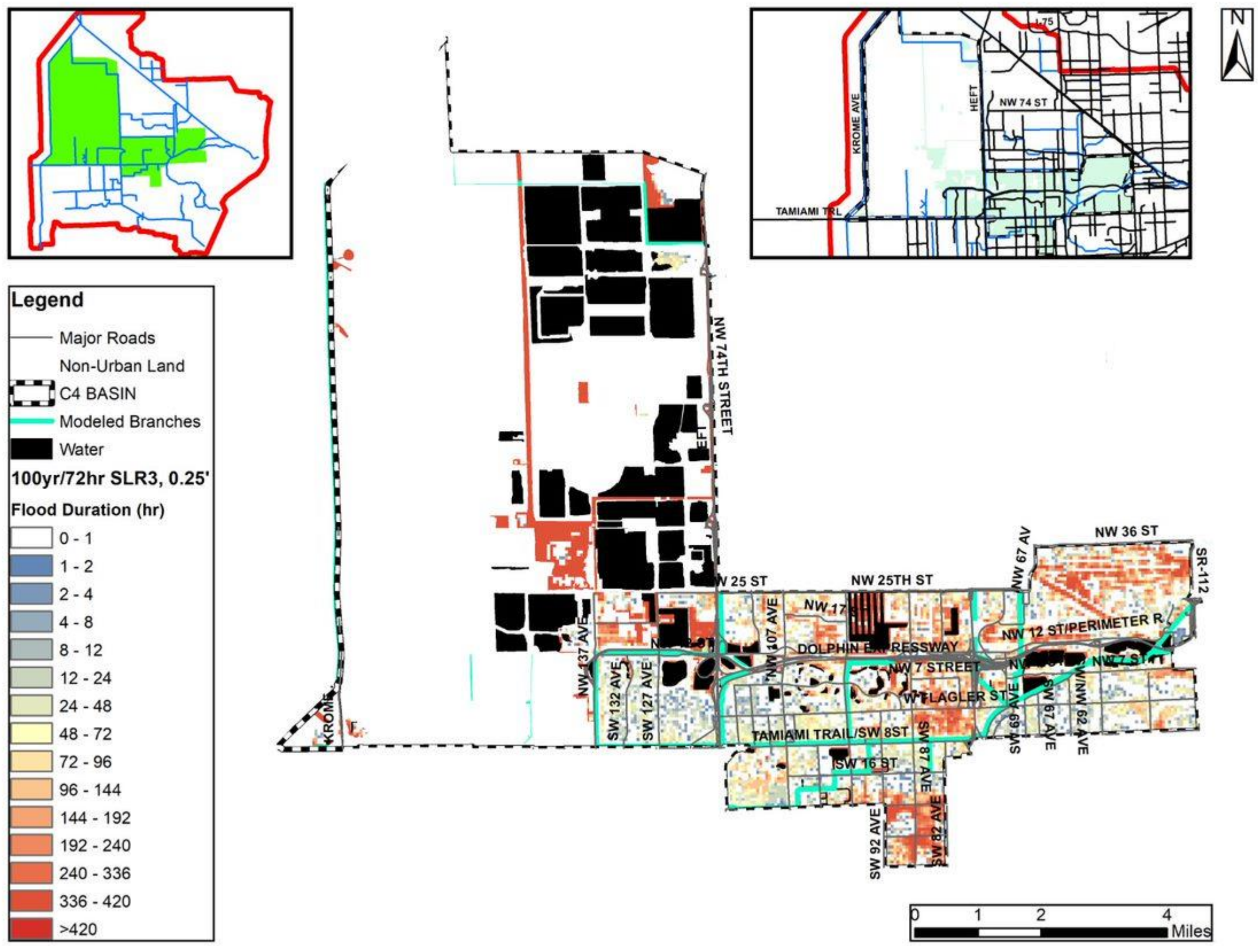


Figure D 3-33. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C4 Watershed

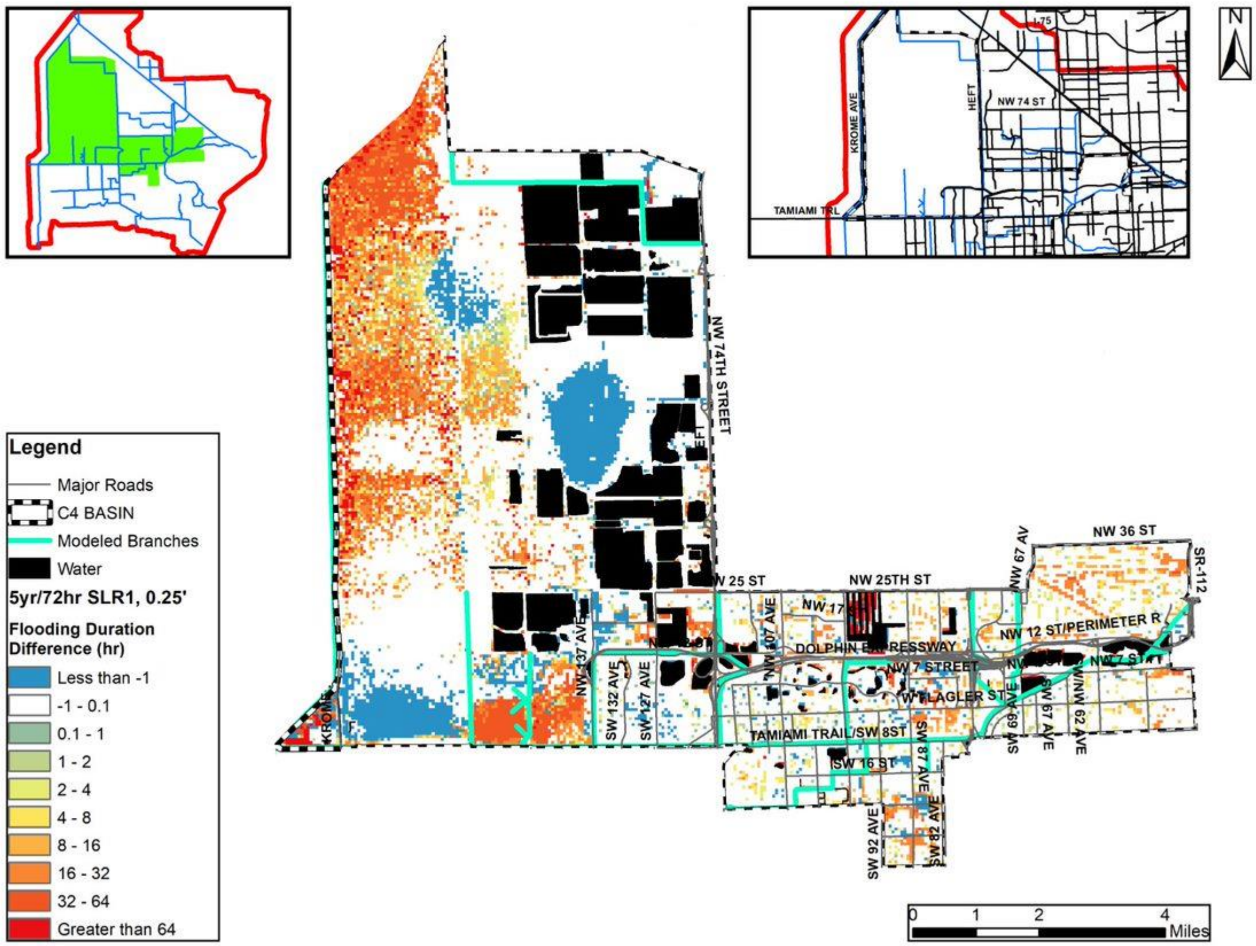


Figure D 3-34. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C4 Watershed

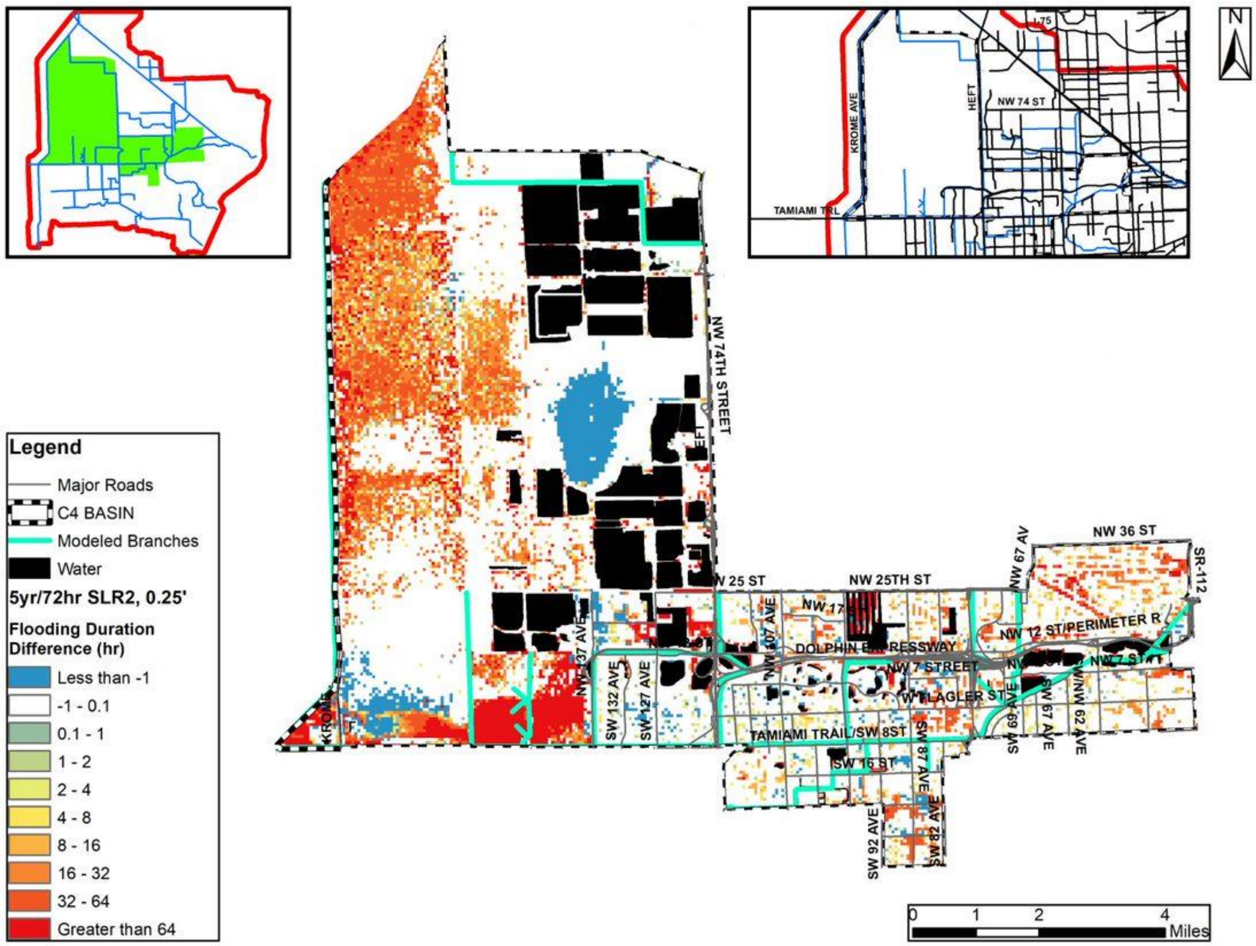


Figure D 3-35. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C4 Watershed

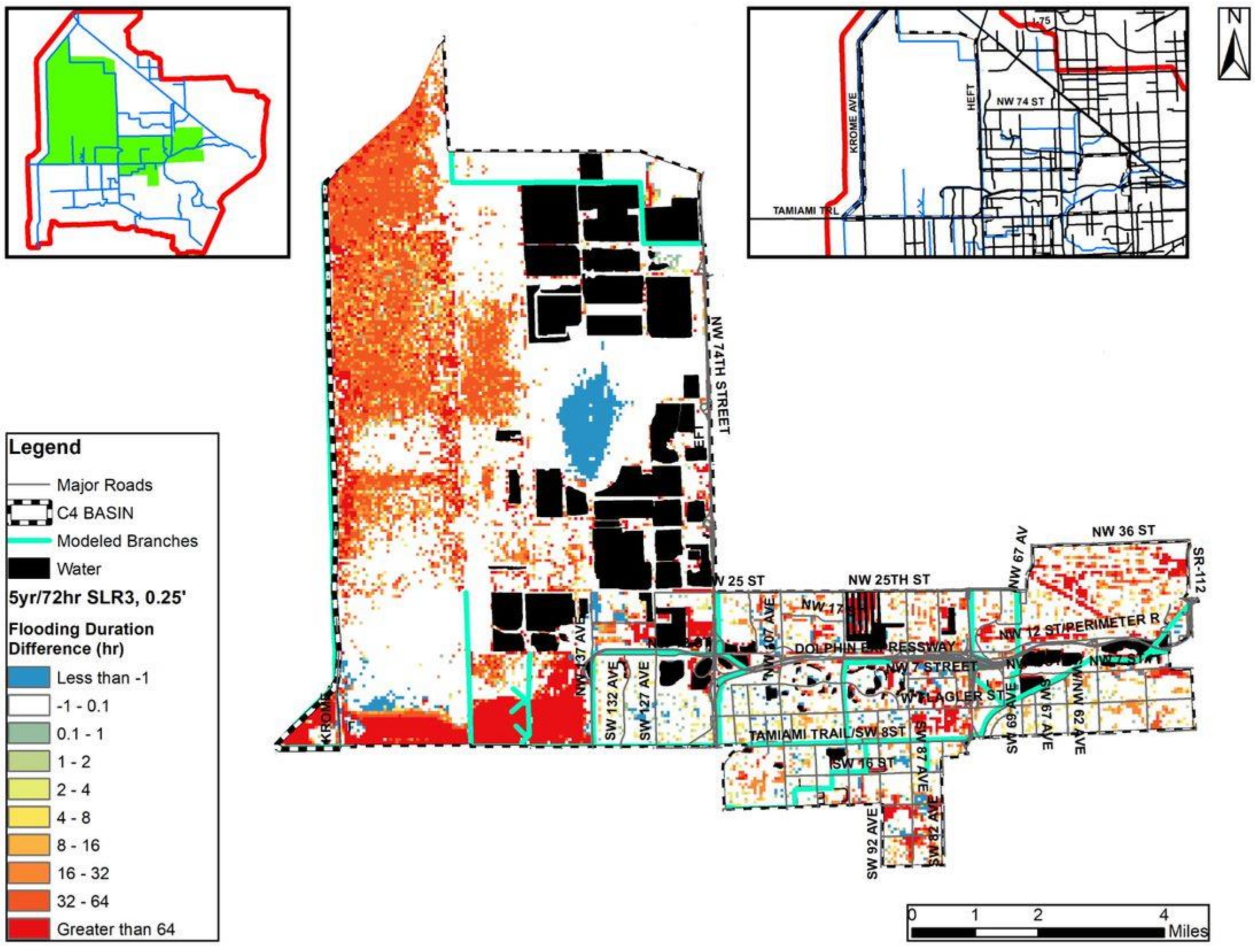


Figure D 3-36. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C4 Watershed

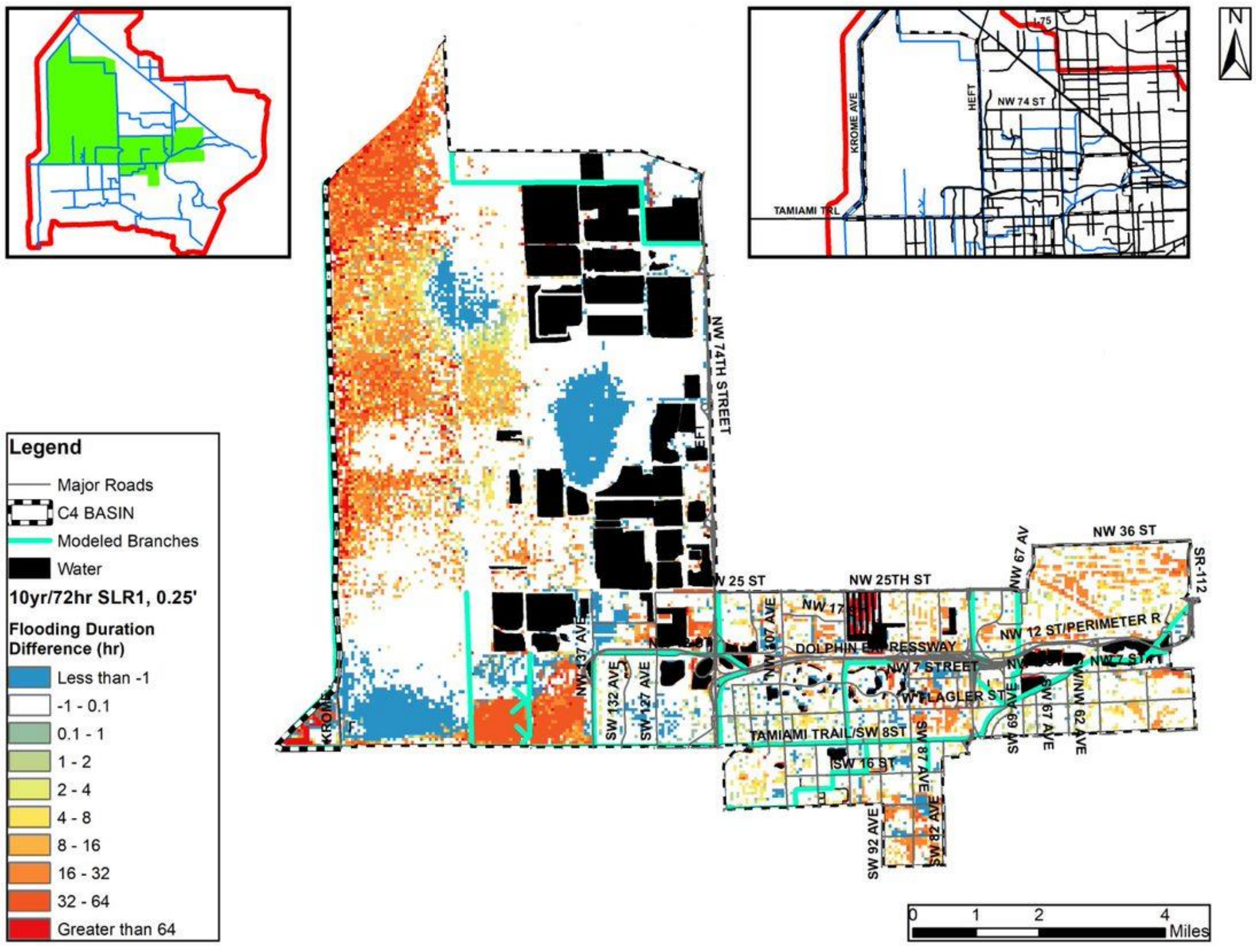


Figure D 3-37. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C4 Watershed

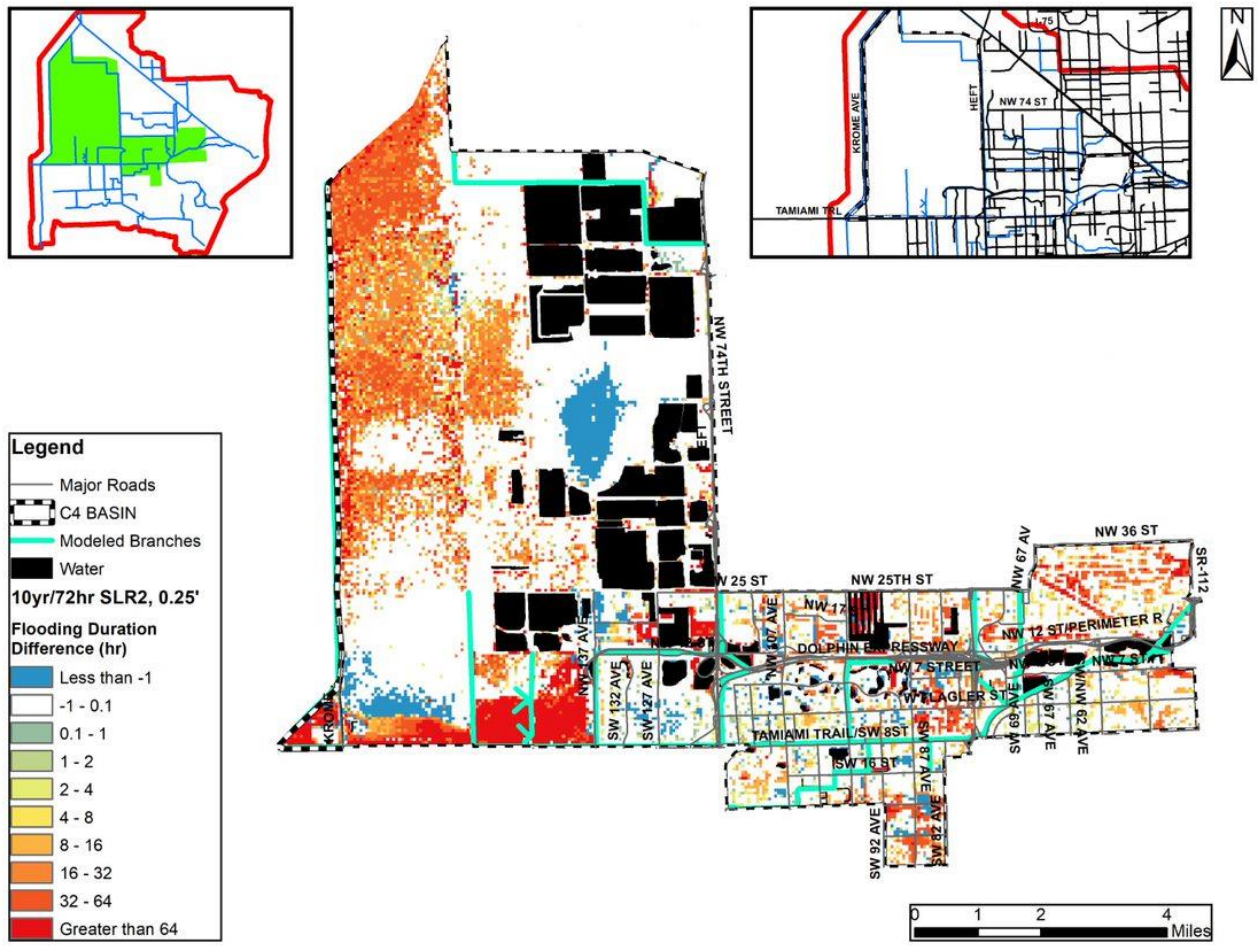


Figure D 3-38. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C4 Watershed

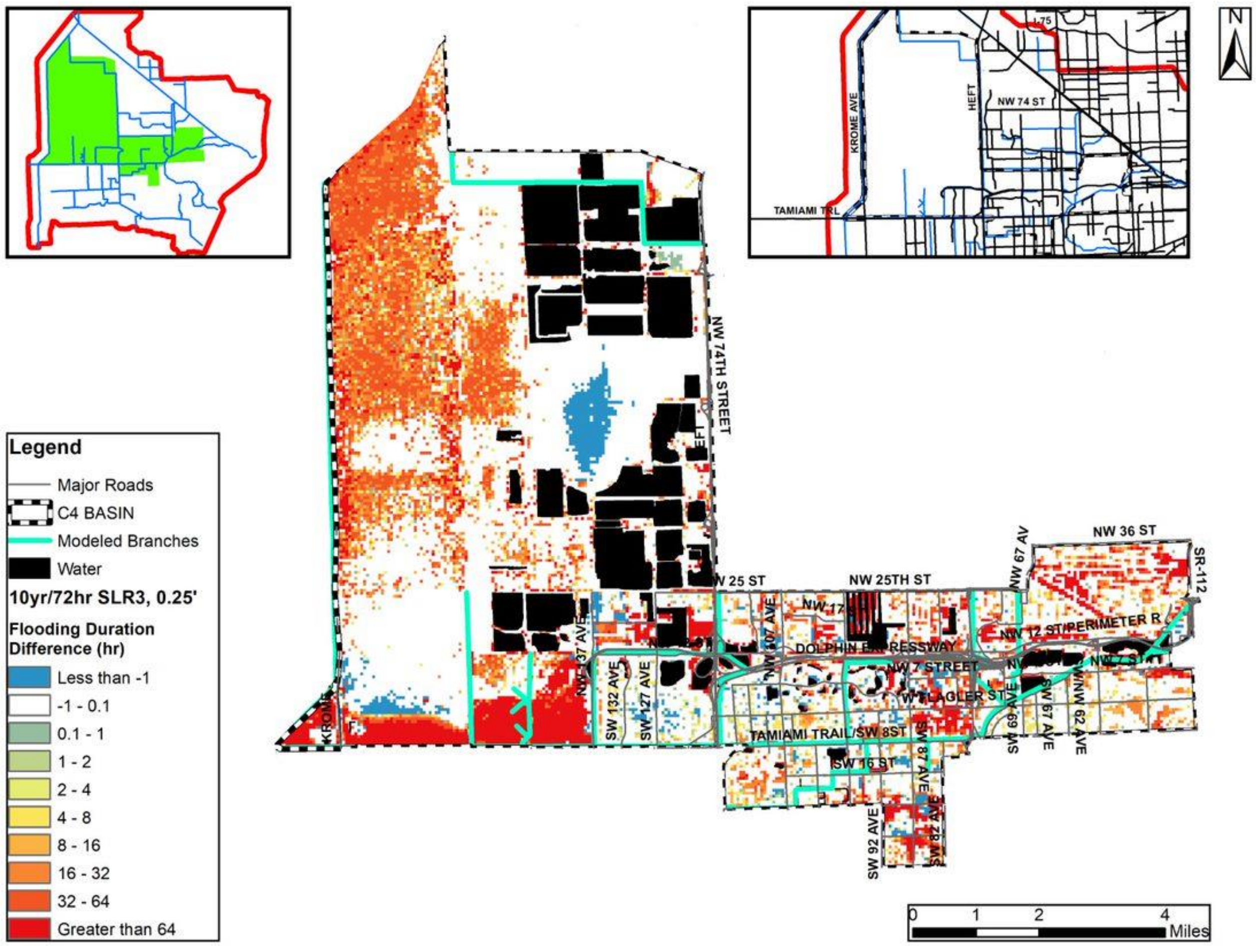


Figure D 3-39. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C4 Watershed

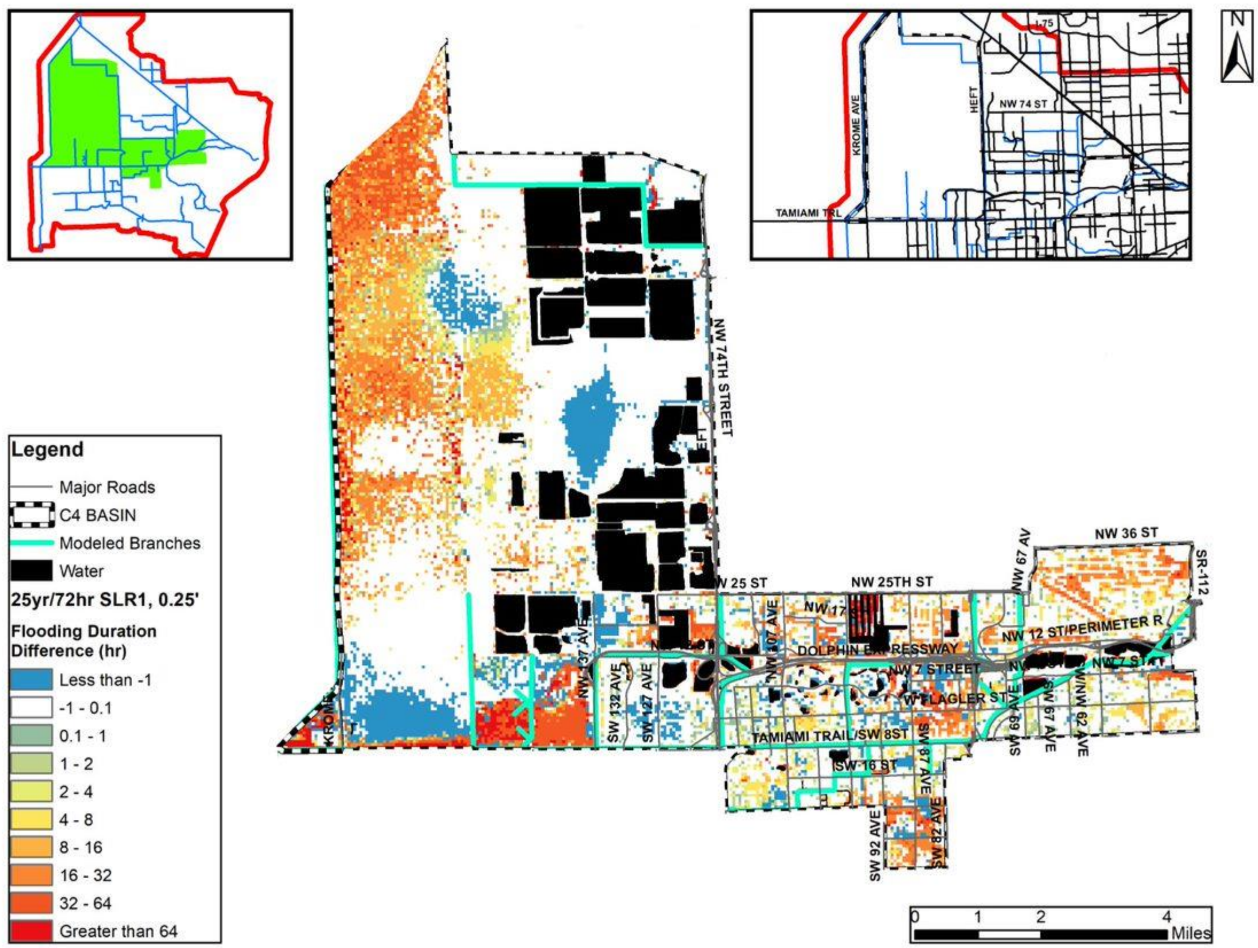


Figure D 3-40. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C4 Watershed

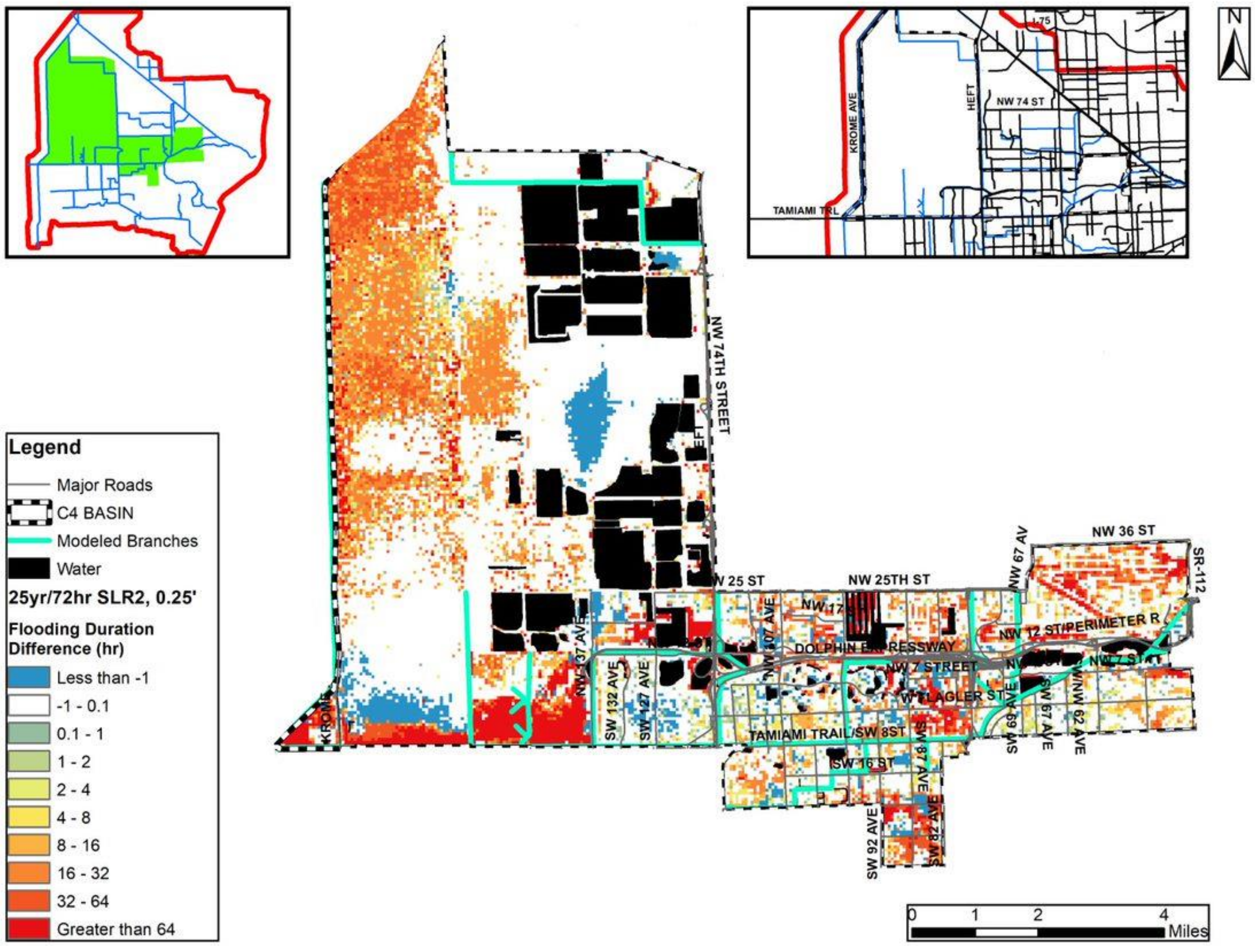


Figure D 3-41. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C4 Watershed

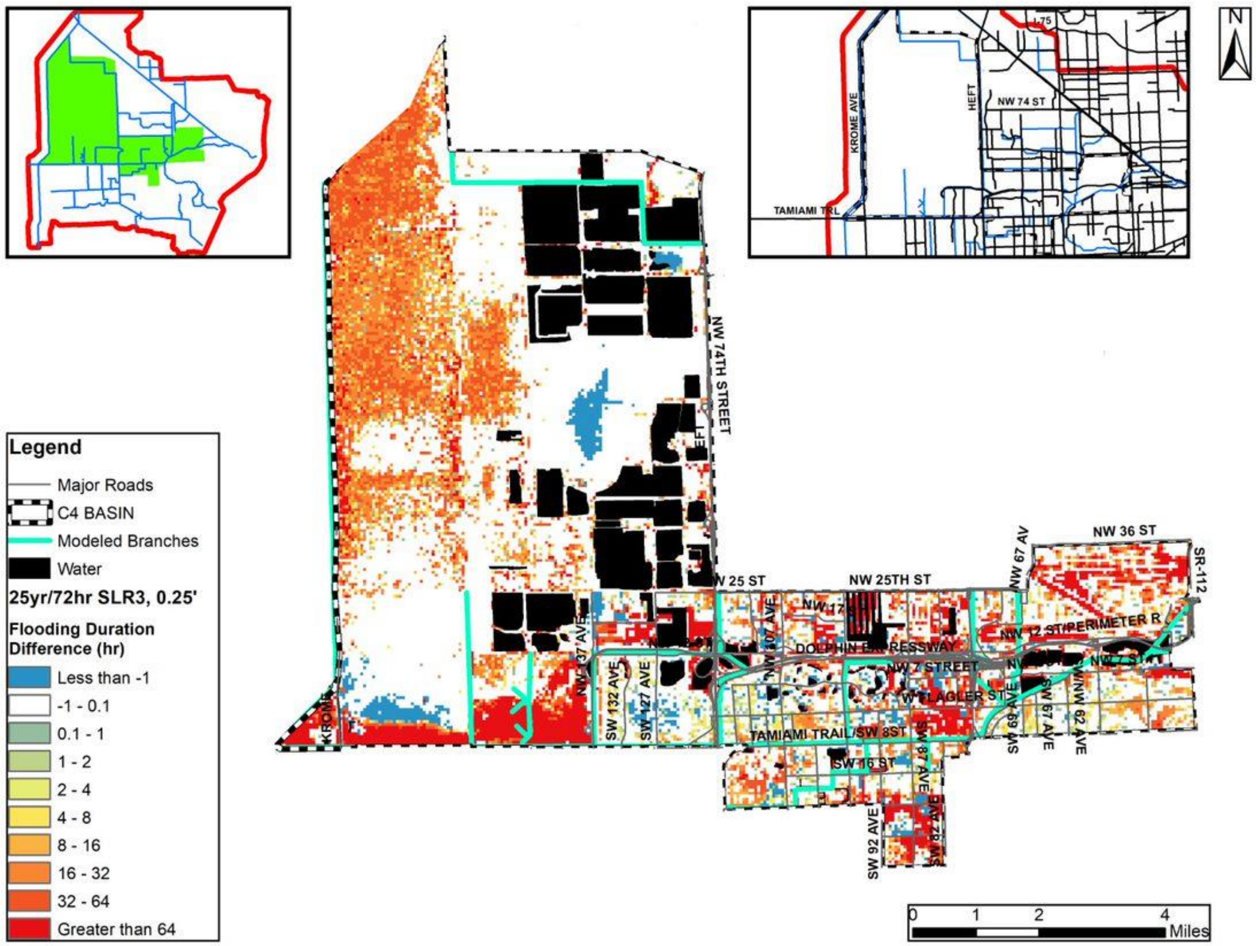


Figure D 3-42. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C4 Watershed

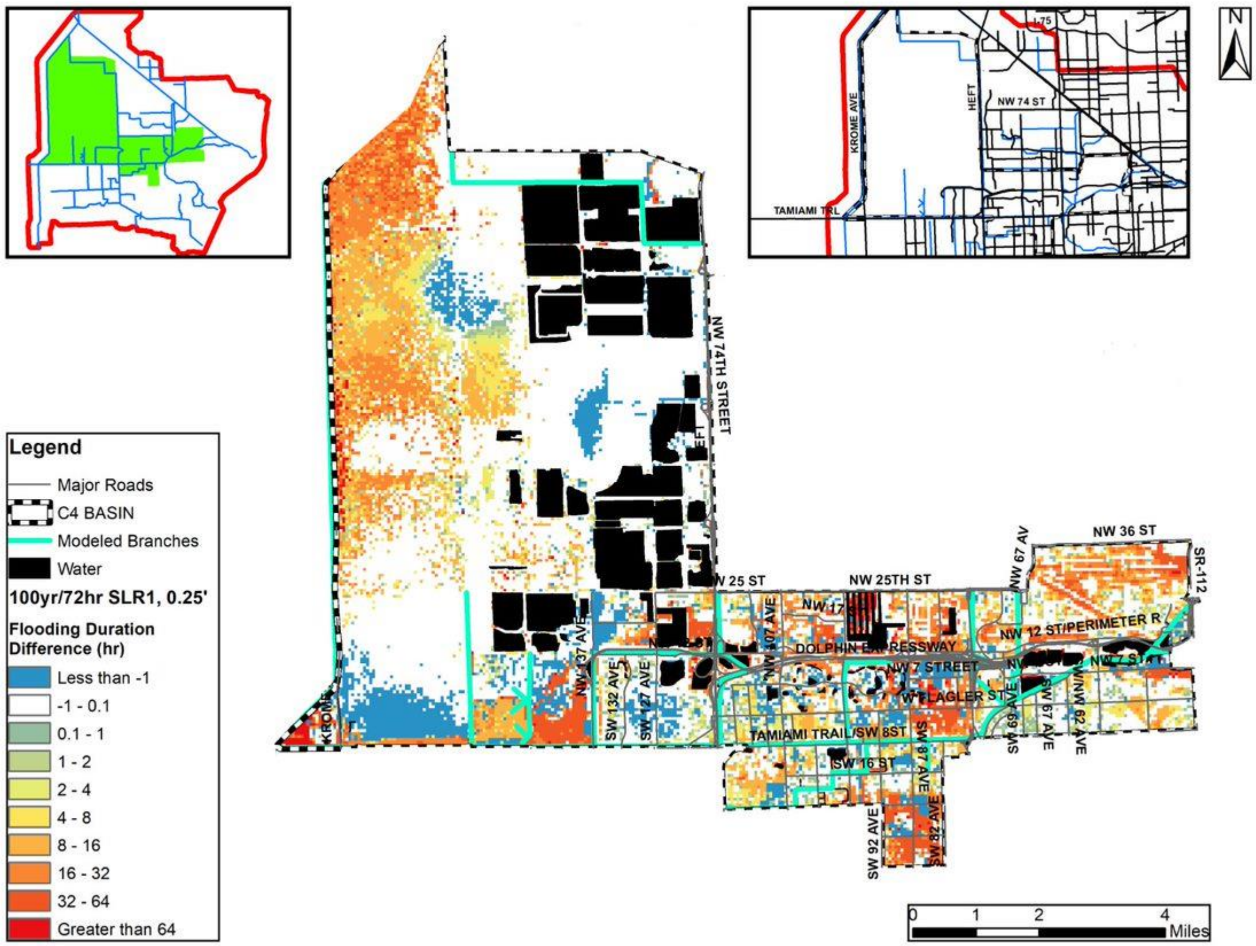


Figure D 3-43. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C4 Watershed

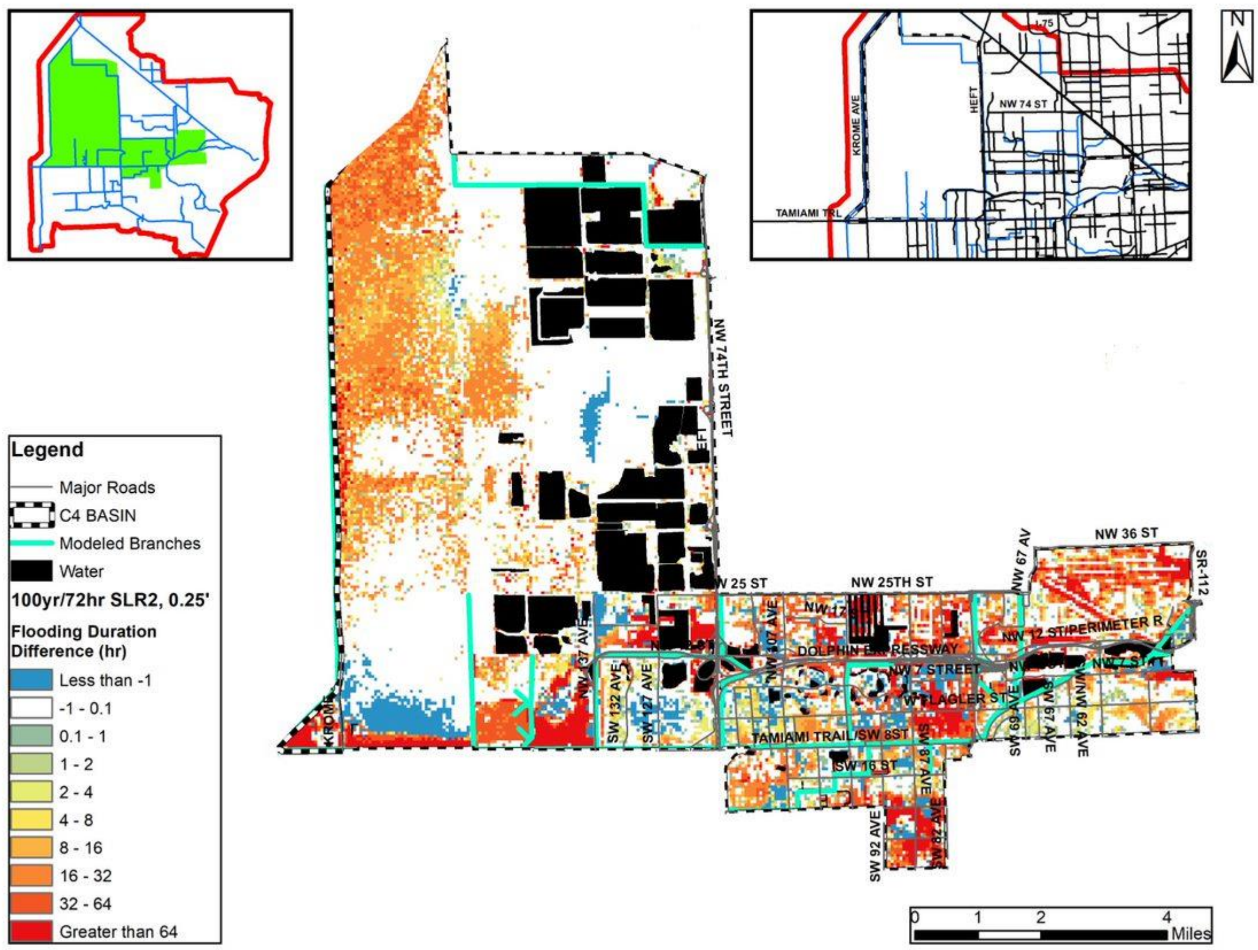


Figure D 3-44. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C4 Watershed

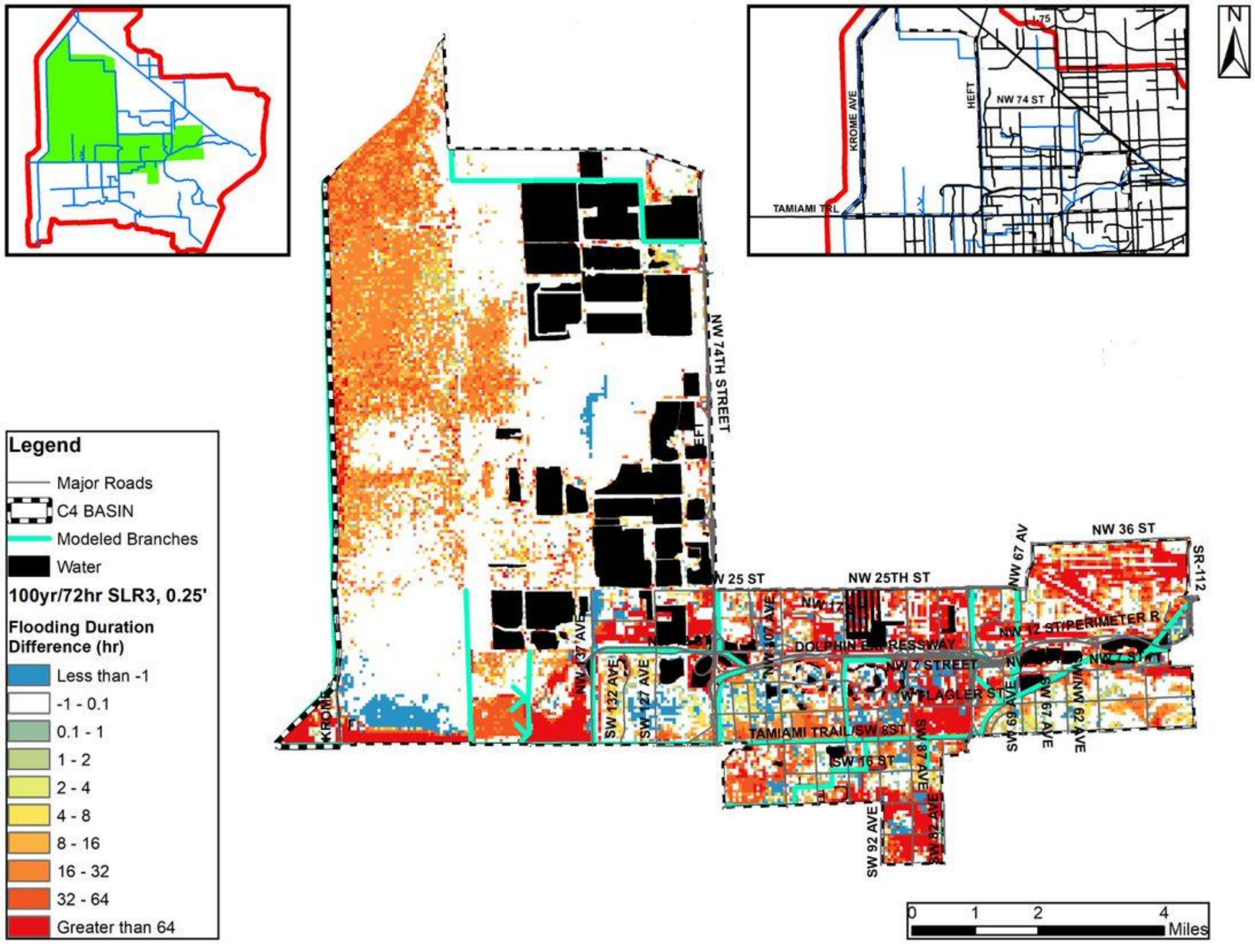


Figure D 3-45. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C4 Watershed

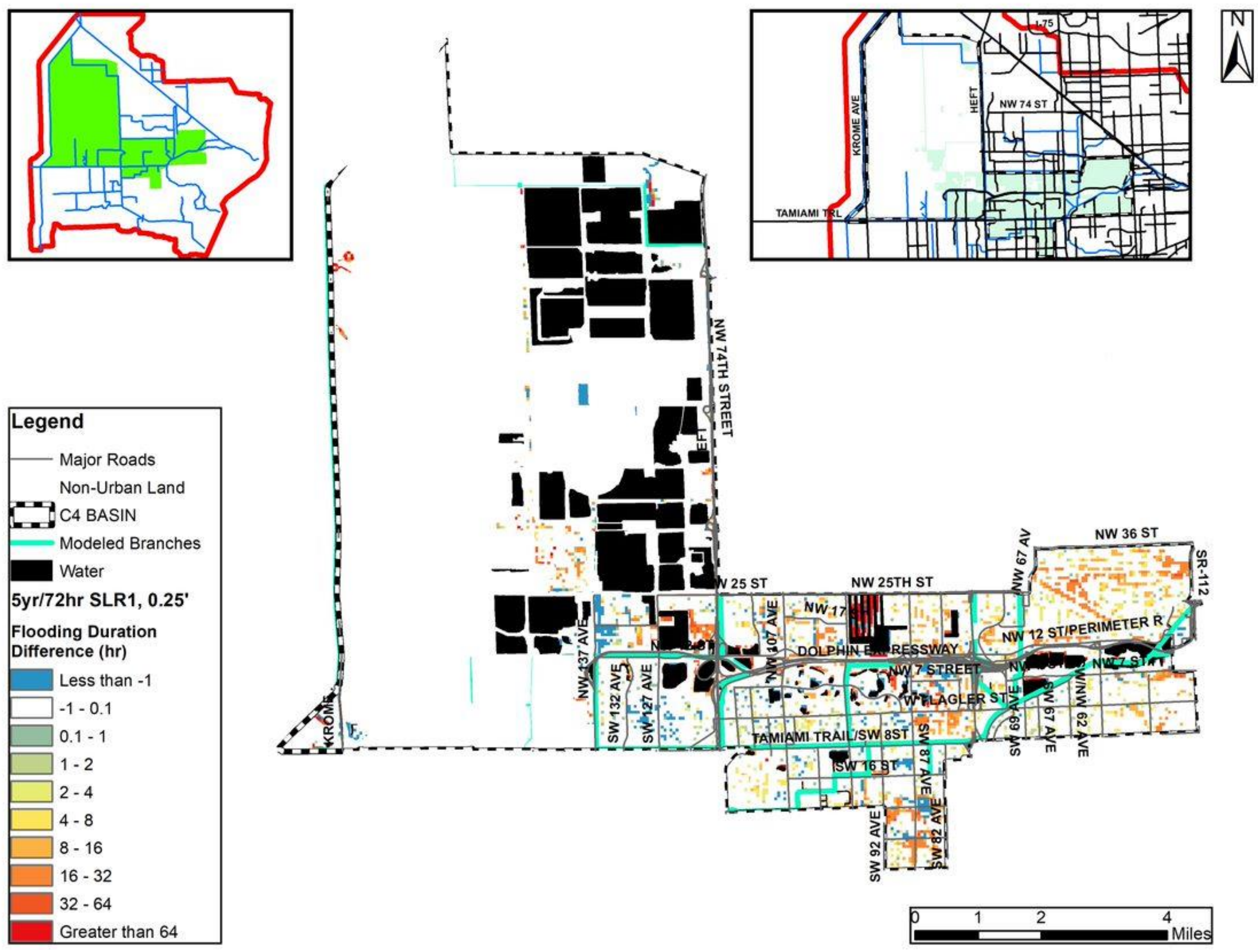


Figure D 3-46. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C4 Watershed

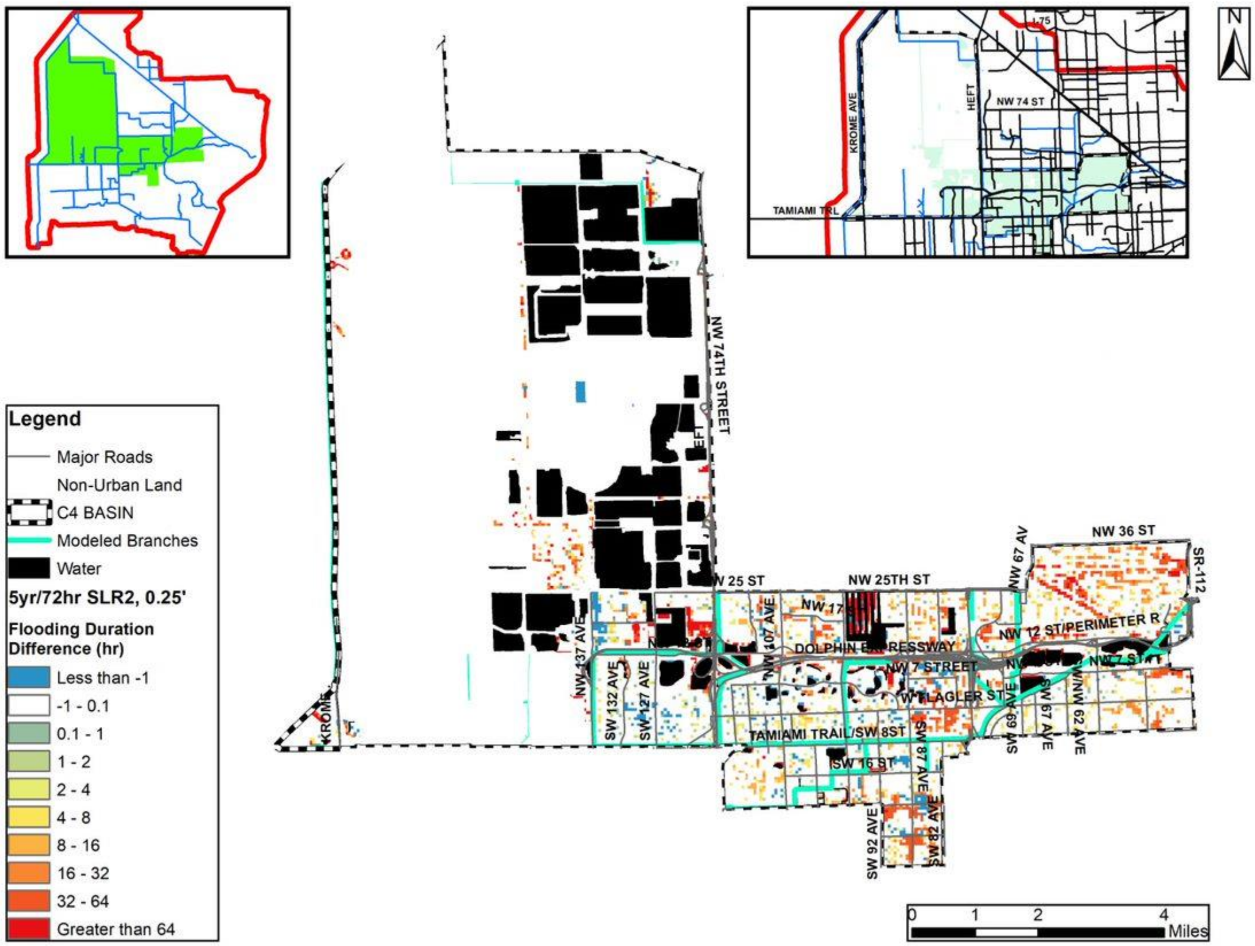


Figure D 3-47. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C4 Watershed

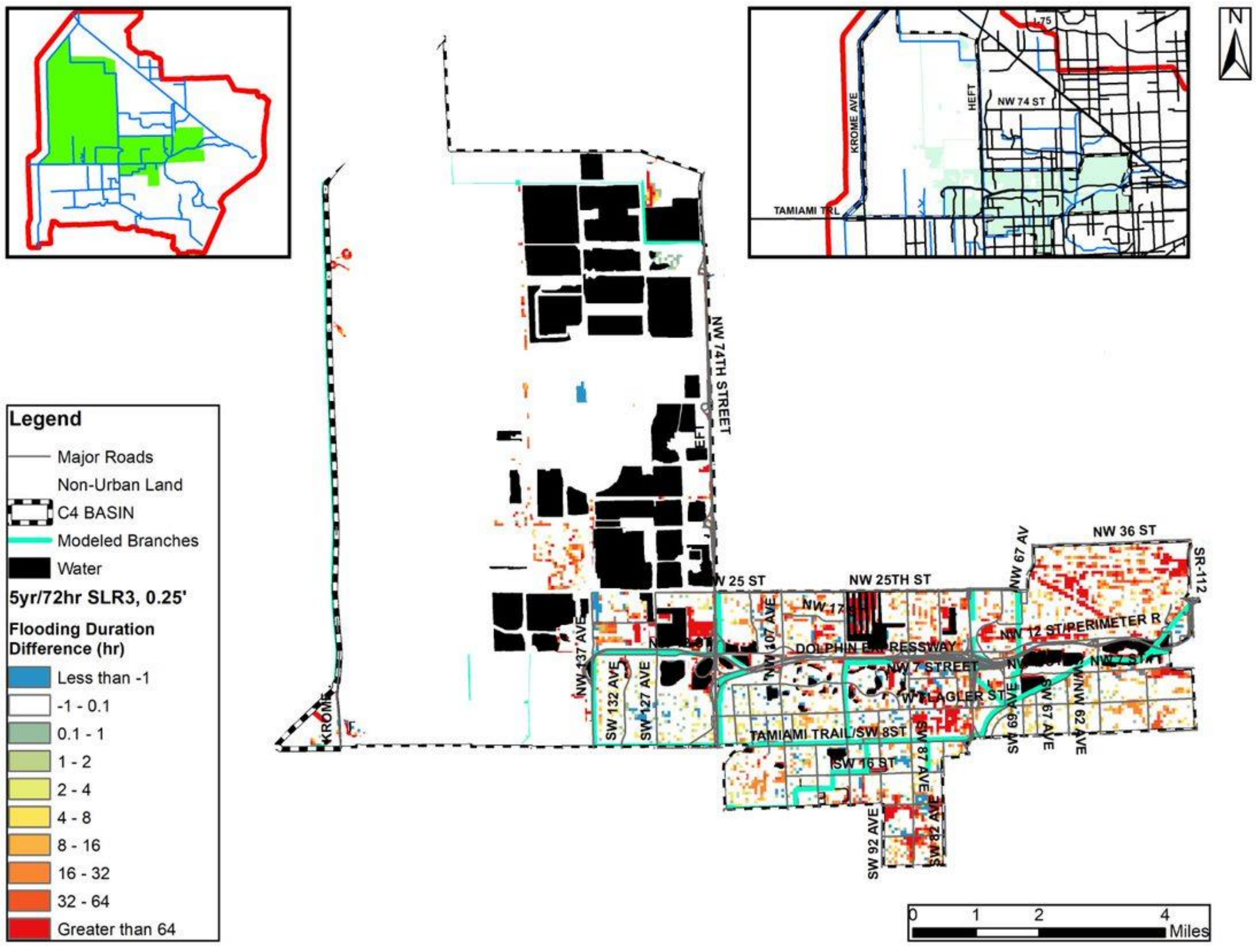


Figure D 3-48. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C4 Watershed

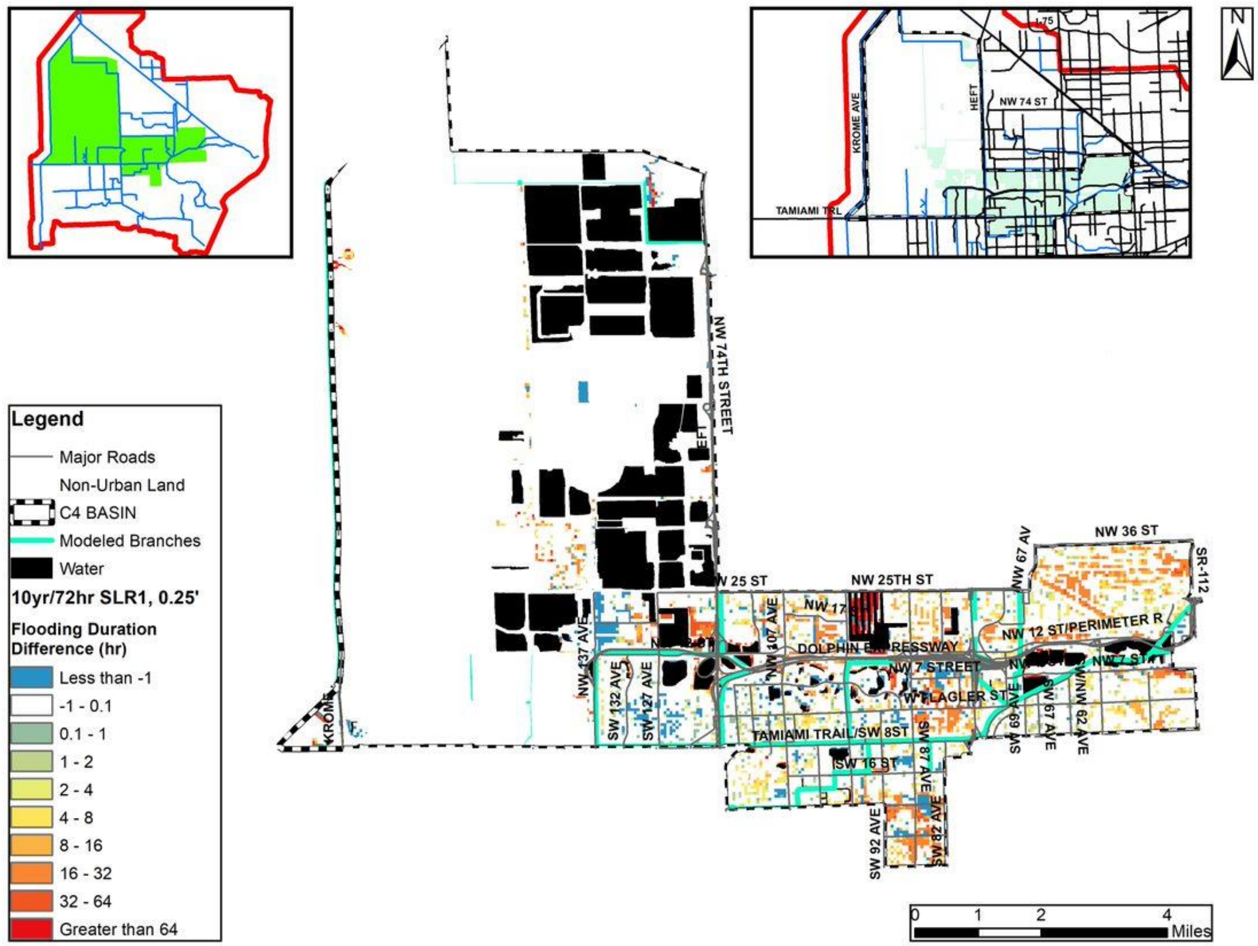


Figure D 3-49. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C4 Watershed

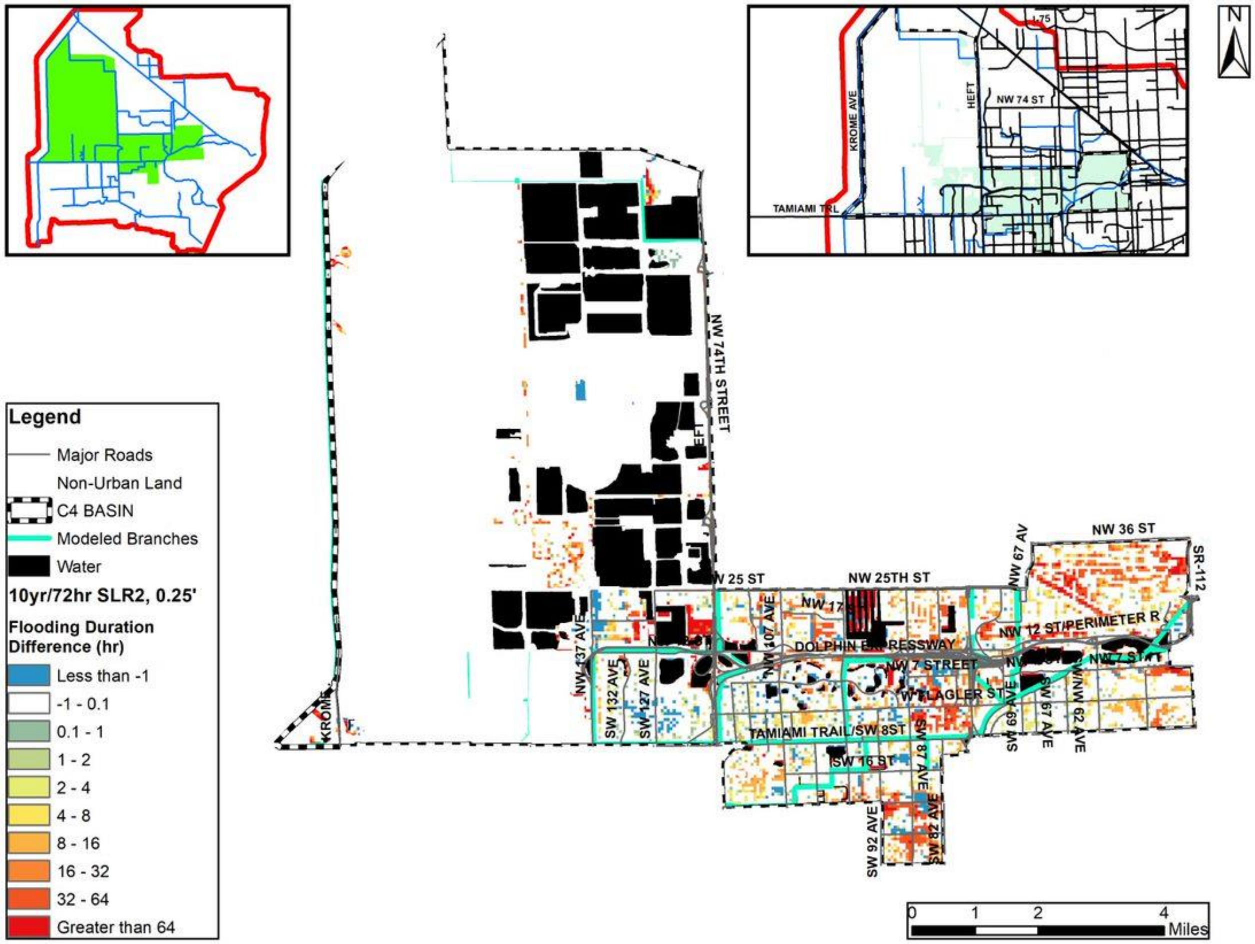


Figure D 3-50. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C4 Watershed

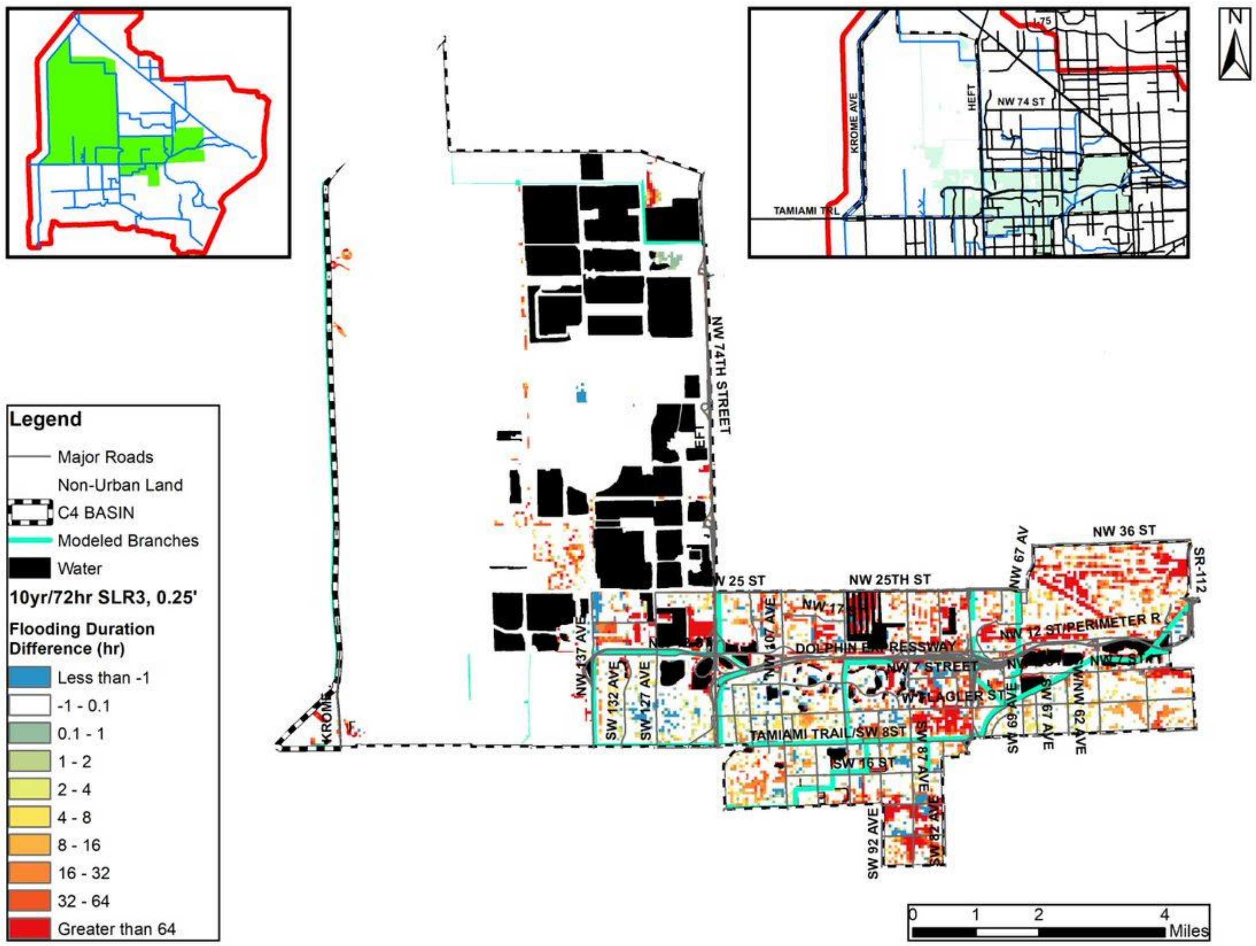


Figure D 3-51. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C4 Watershed

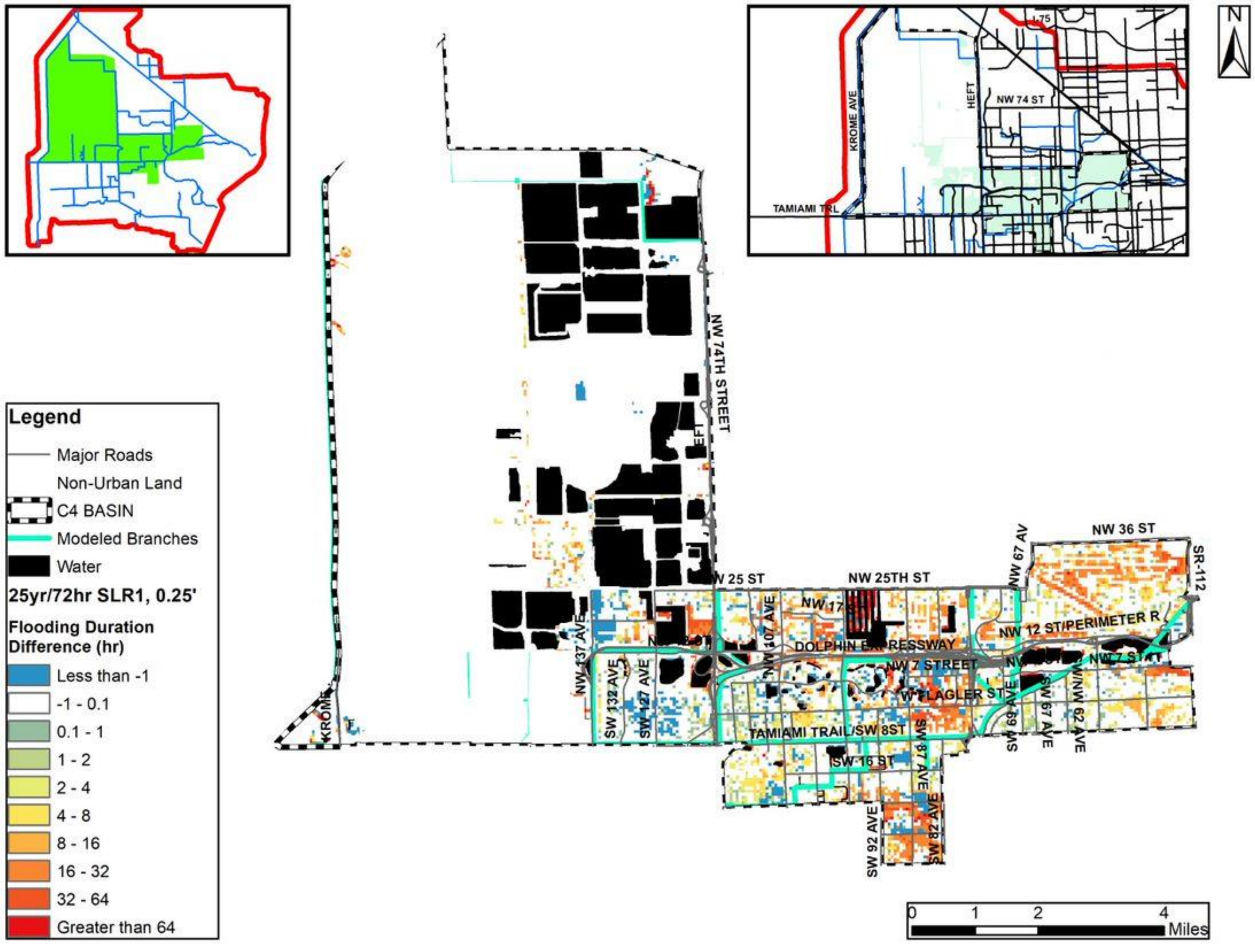


Figure D 3-52. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C4 Watershed

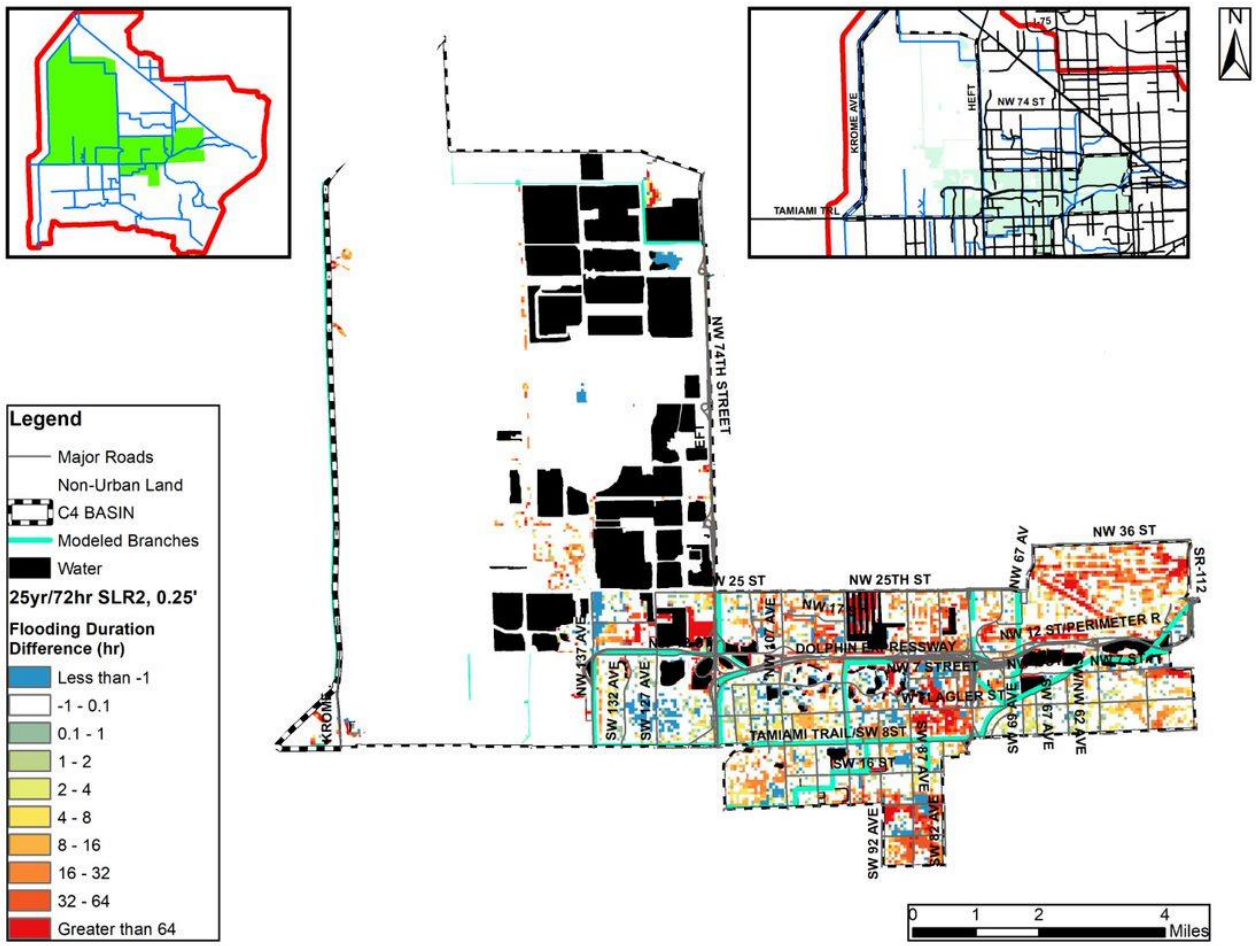


Figure D 3-53. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C4 Watershed

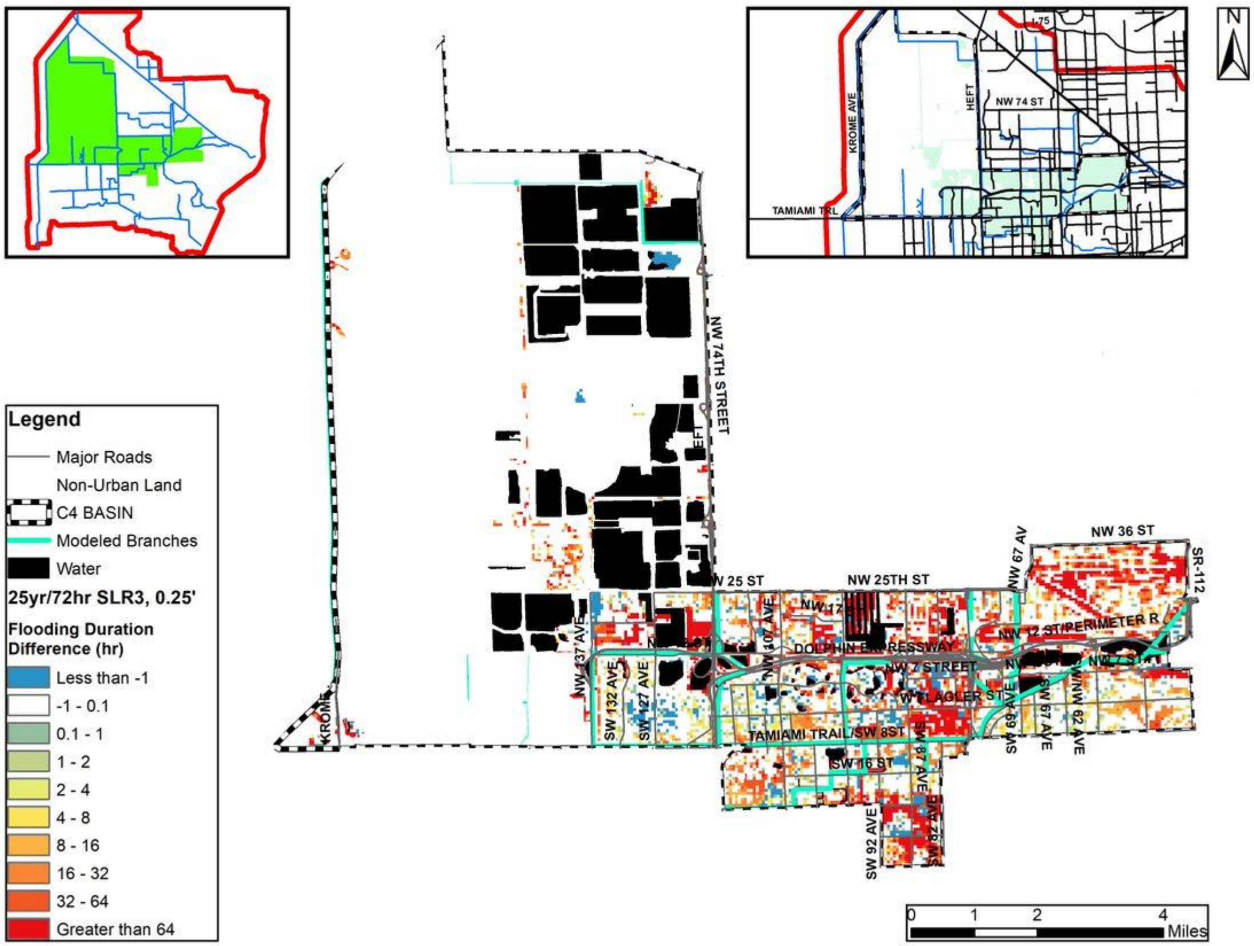


Figure D 3-54. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C4 Watershed

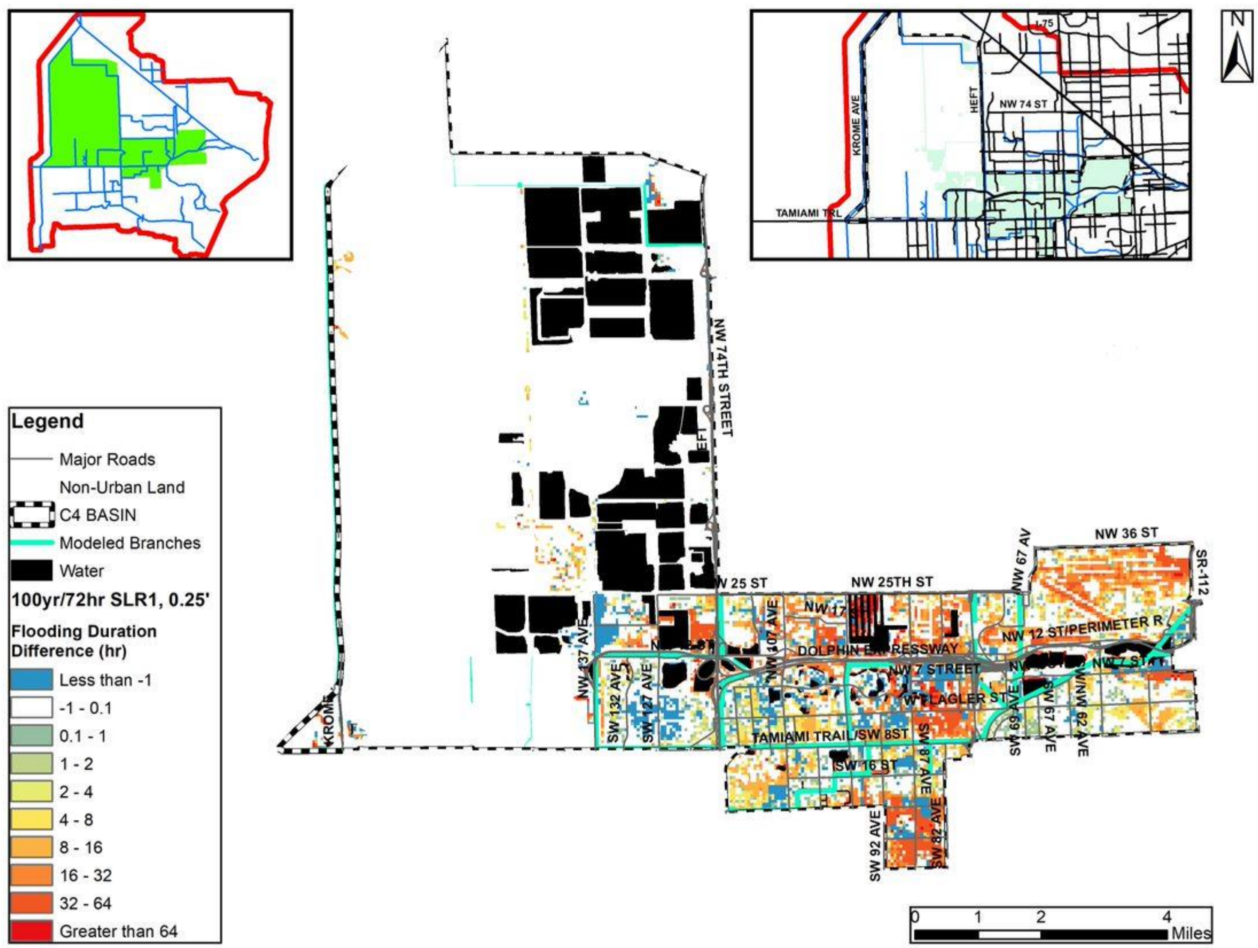


Figure D 3-55. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C4 Watershed

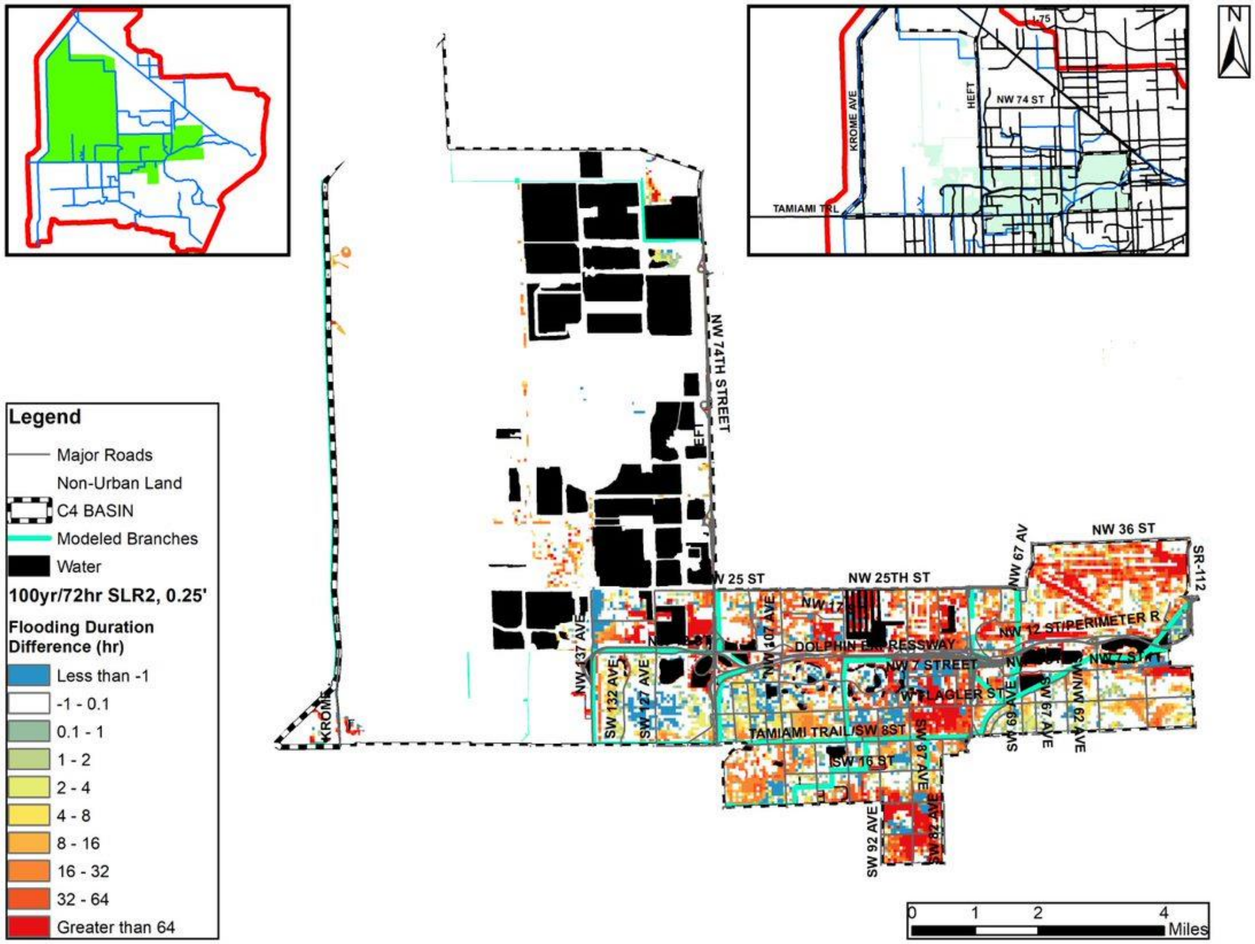
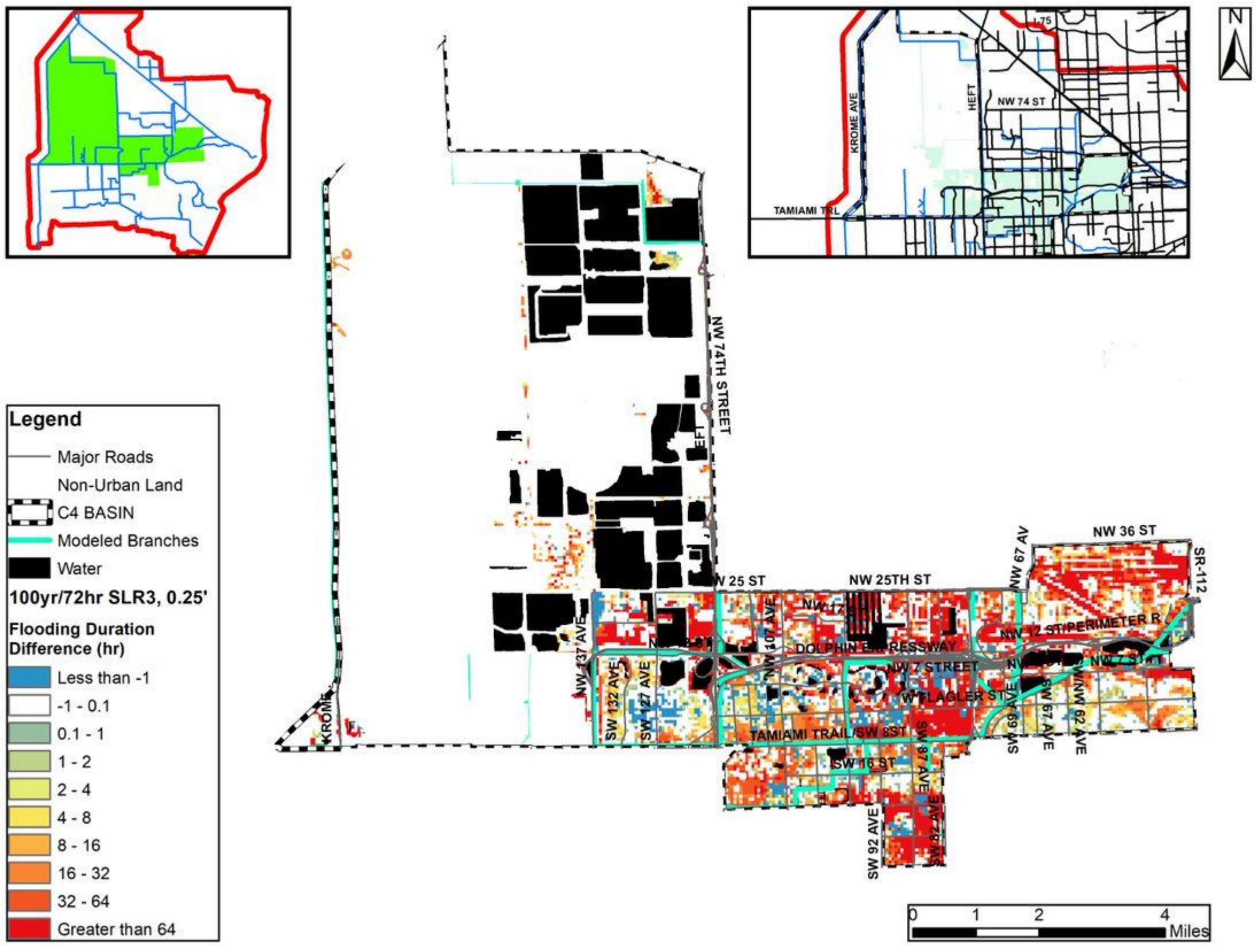


Figure D 3-56. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C4 Watershed



4 C5 Watershed

Figure D 4-1 through **Figure D 4-16** provides flood duration maps for the C5 Watershed for overland flooding depths exceeding 0.25 ft for the 5-, 10-, 25-, and 100-year 3-day design storms, respectively, for each SLR condition. Water areas are masked in black.

Figure D 4-17 through **Figure D 4-28** show the difference in overland flooding for the C5 Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively.

Figure D 4-1. Flood Duration Map for the 5-year 3-Day Design Storm for Current Conditions in the C5 Watershed

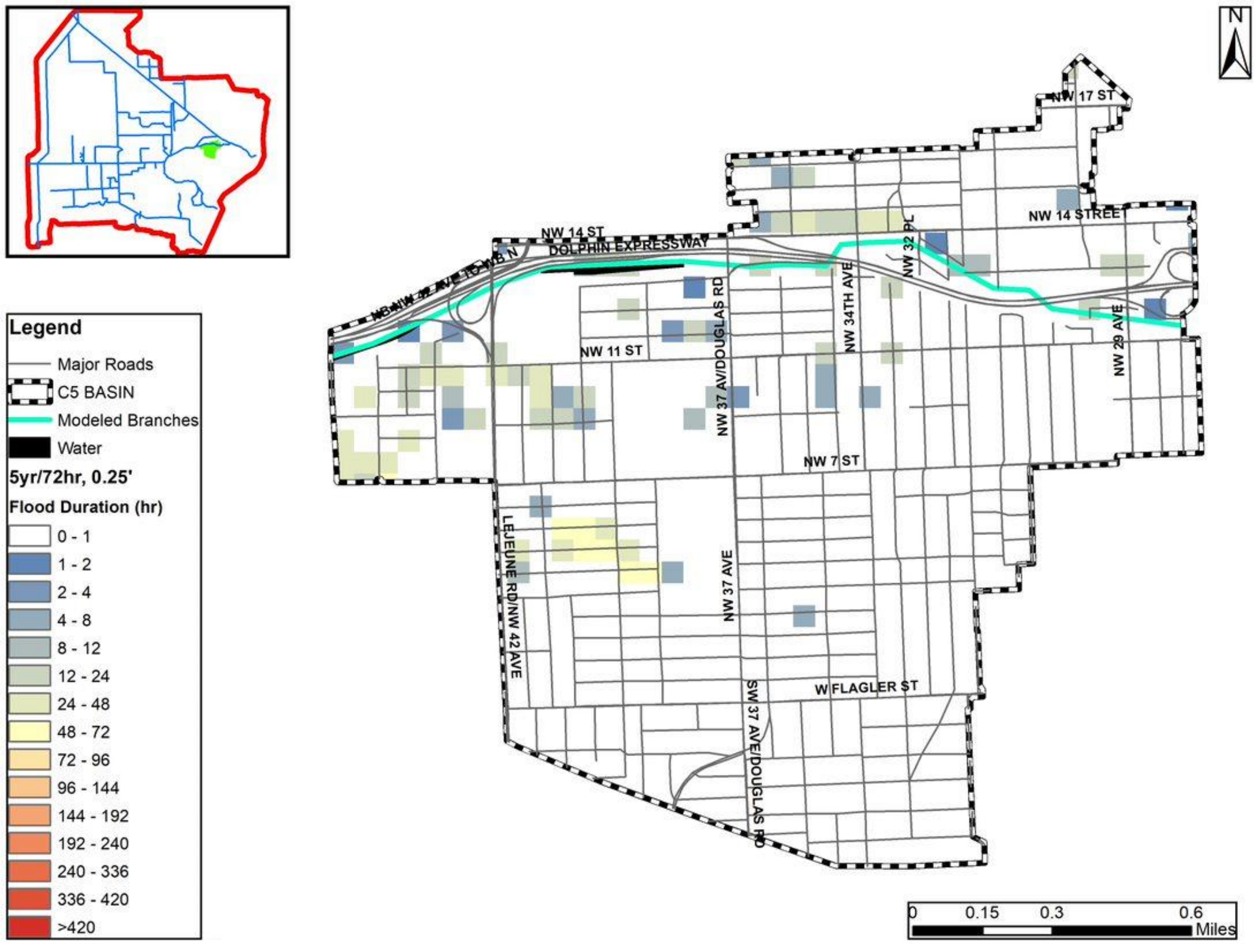


Figure D 4-2. Flood Duration Map for the 5-year 3-Day Design Storm for SLR1 in the C5 Watershed

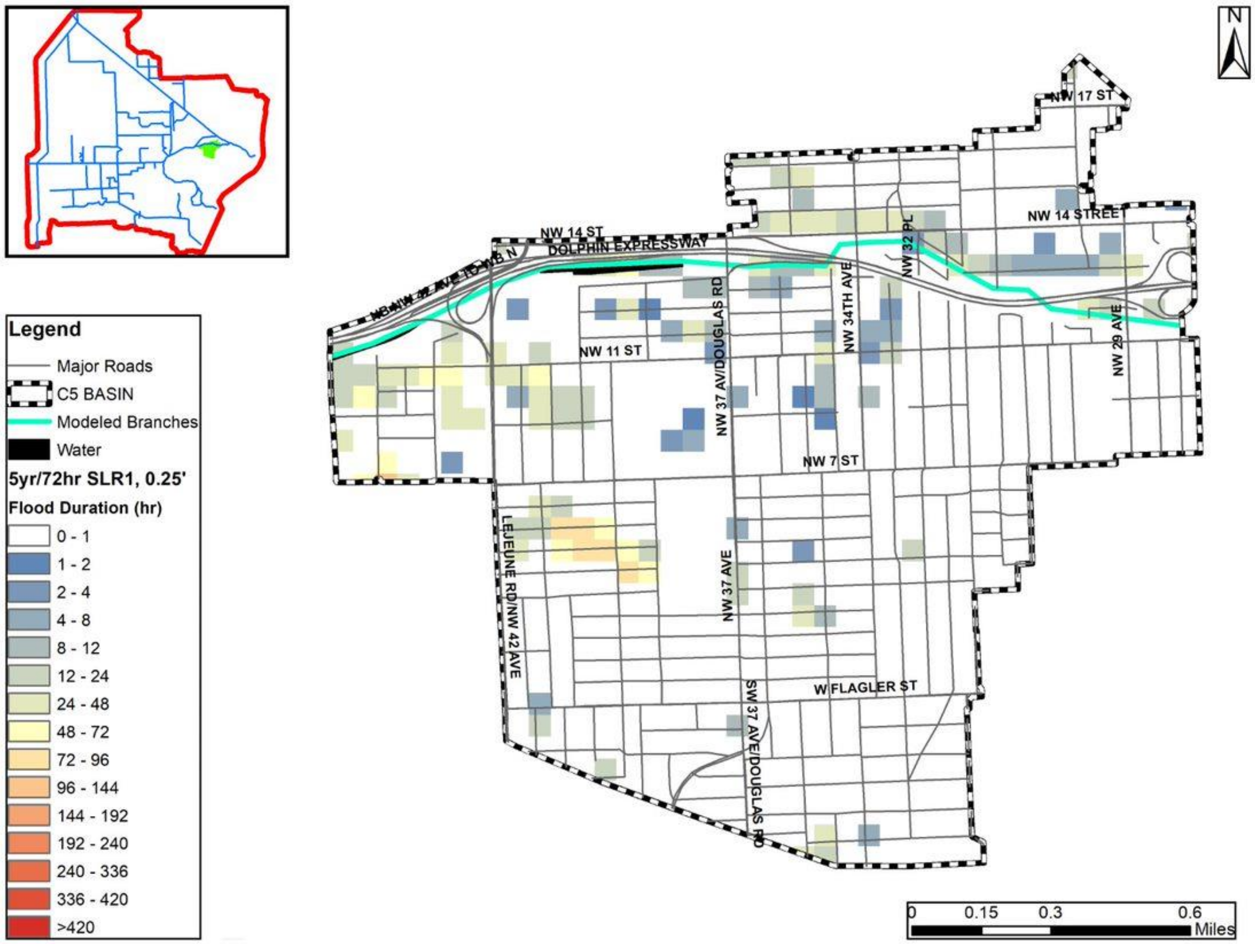


Figure D 4-3. Flood Duration Map for the 5-year 3-Day Design Storm for SLR2 in the C5 Watershed

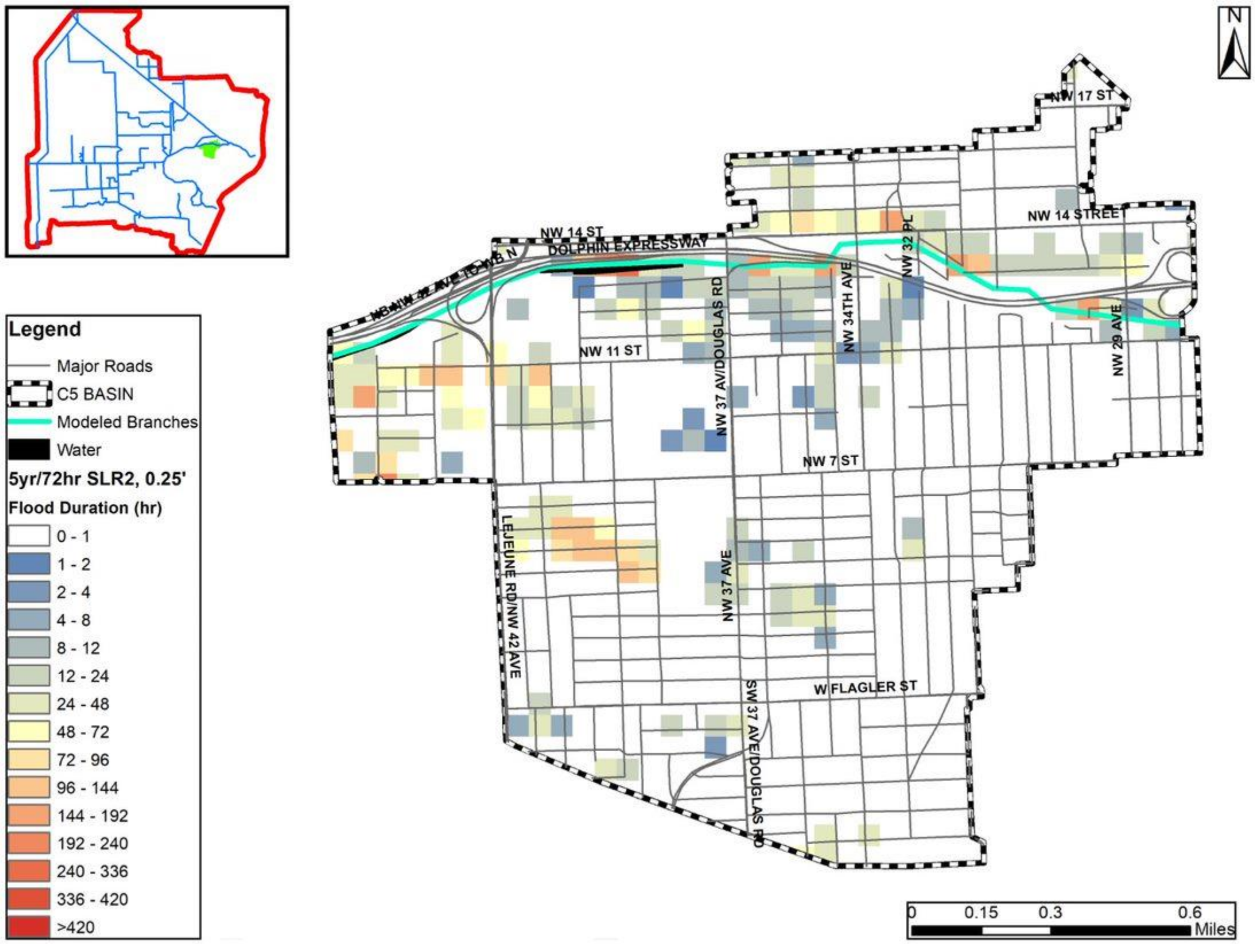


Figure D 4-4. Flood Duration Map for the 5-year 3-Day Design Storm for SLR3 in the C5 Watershed

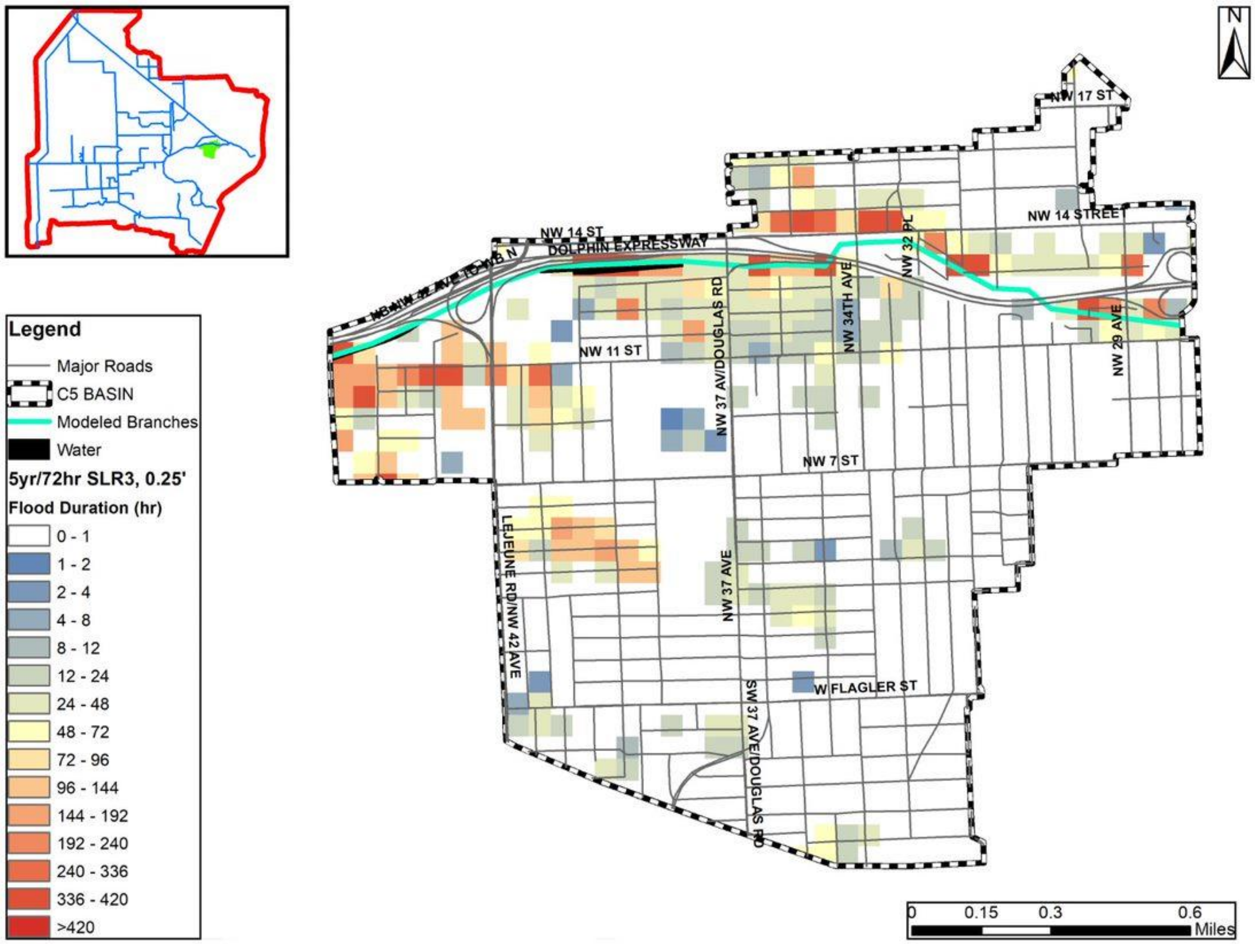


Figure D 4-5. Flood Duration Map for the 10-year 3-Day Design Storm for Current Conditions in the C5 Watershed

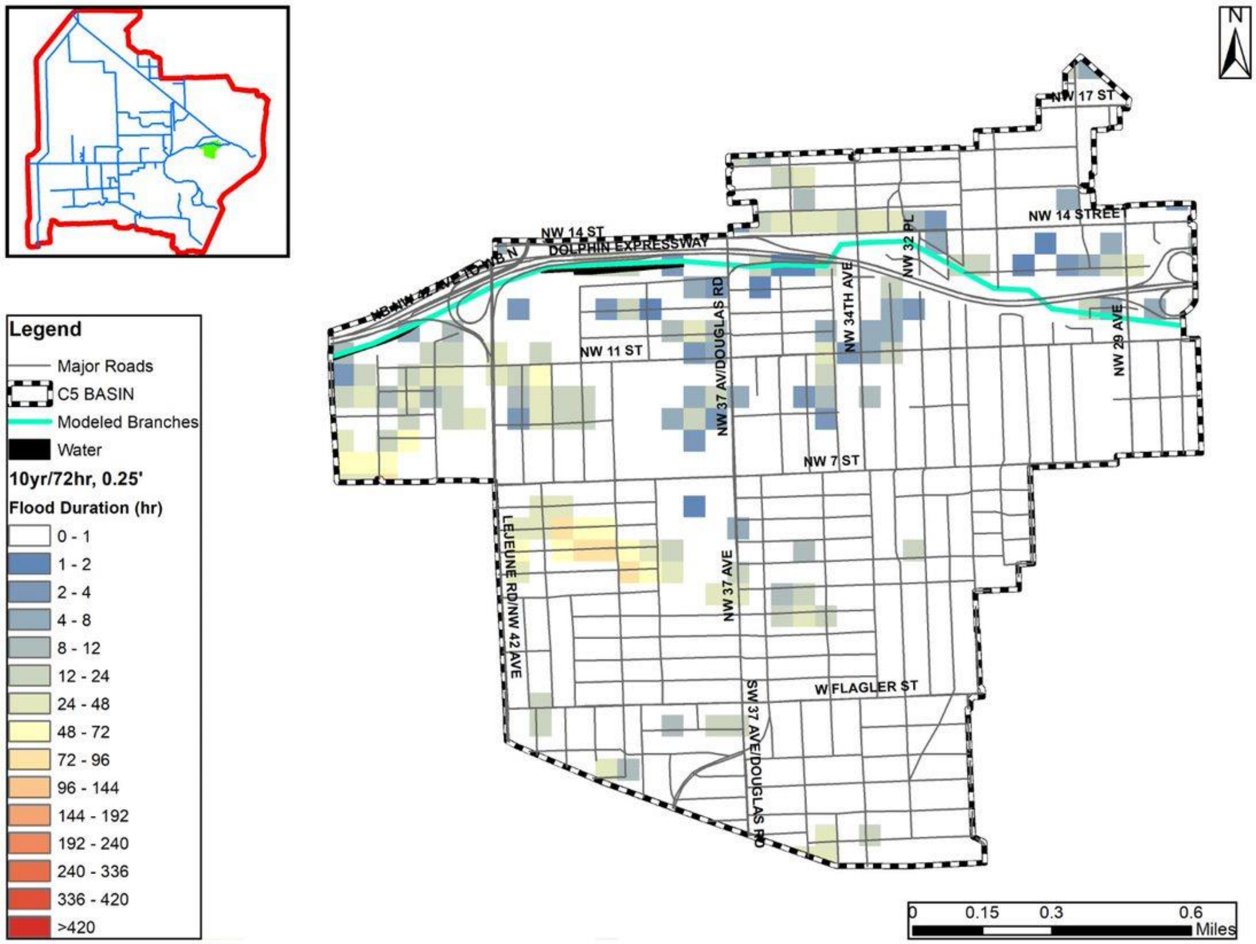


Figure D 4-6. Flood Duration Map for the 10-year 3-Day Design Storm for SLR1 in the C5 Watershed

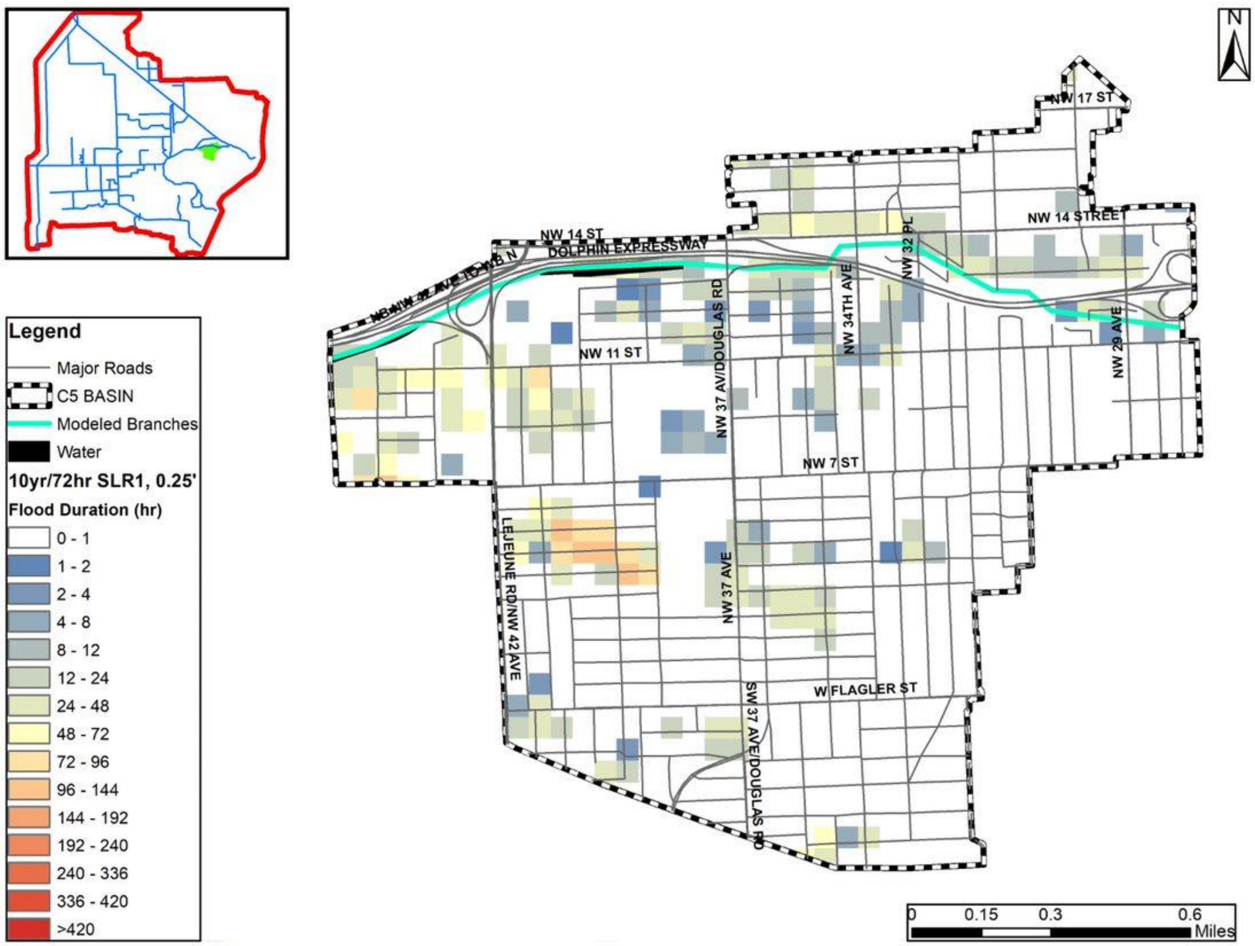


Figure D 4-7. Flood Duration Map for the 10-year 3-Day Design Storm for SLR2 in the C5 Watershed

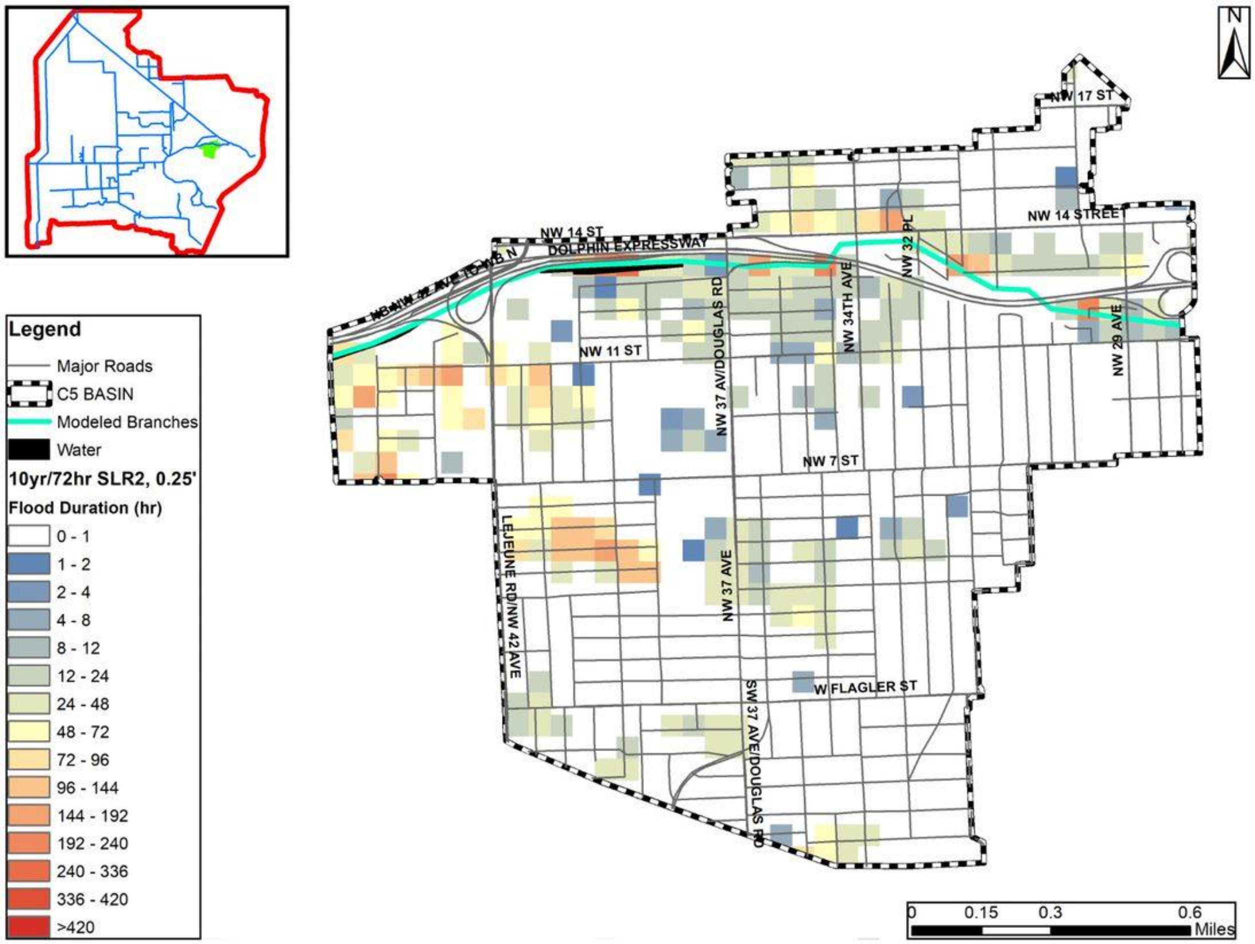


Figure D 4-8. Flood Duration Map for the 10-year 3-Day Design Storm for SLR3 in the C5 Watershed

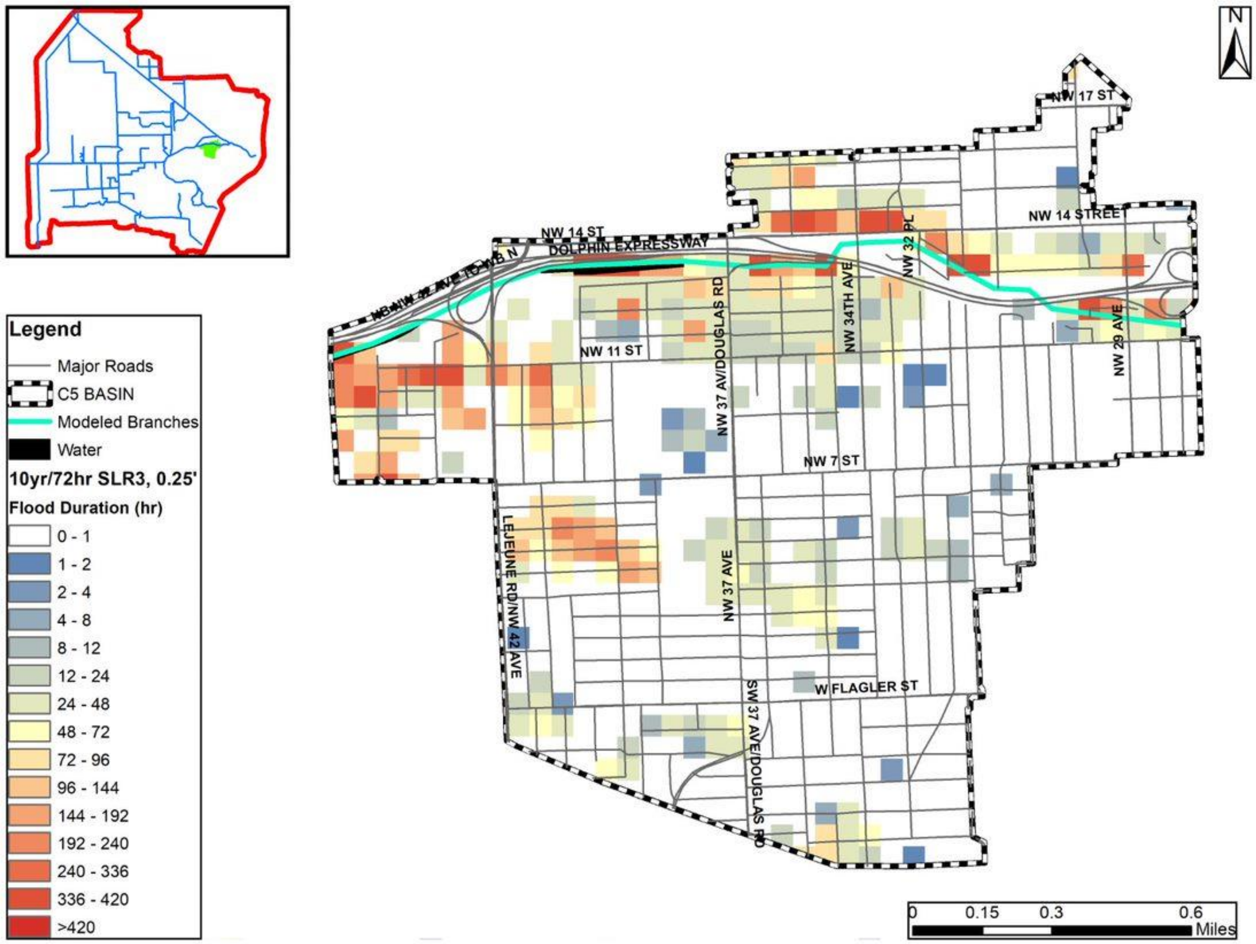


Figure D 4-9. Flood Duration Map for the 25-year 3-Day Design Storm for Current Conditions in the C5 Watershed

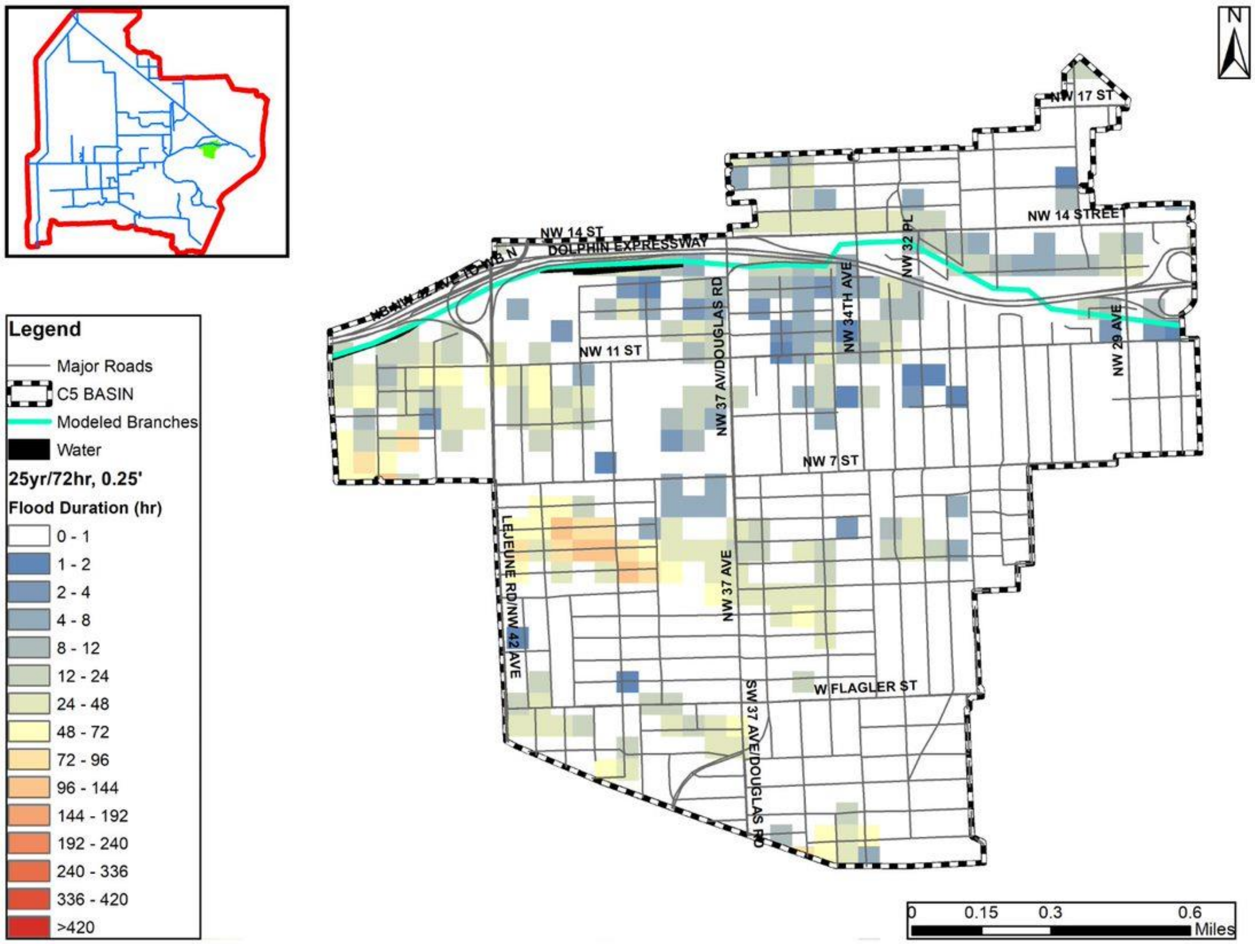


Figure D 4-10. Flood Duration Map for the 25-year 3-Day Design Storm for SLR1 in the C5 Watershed

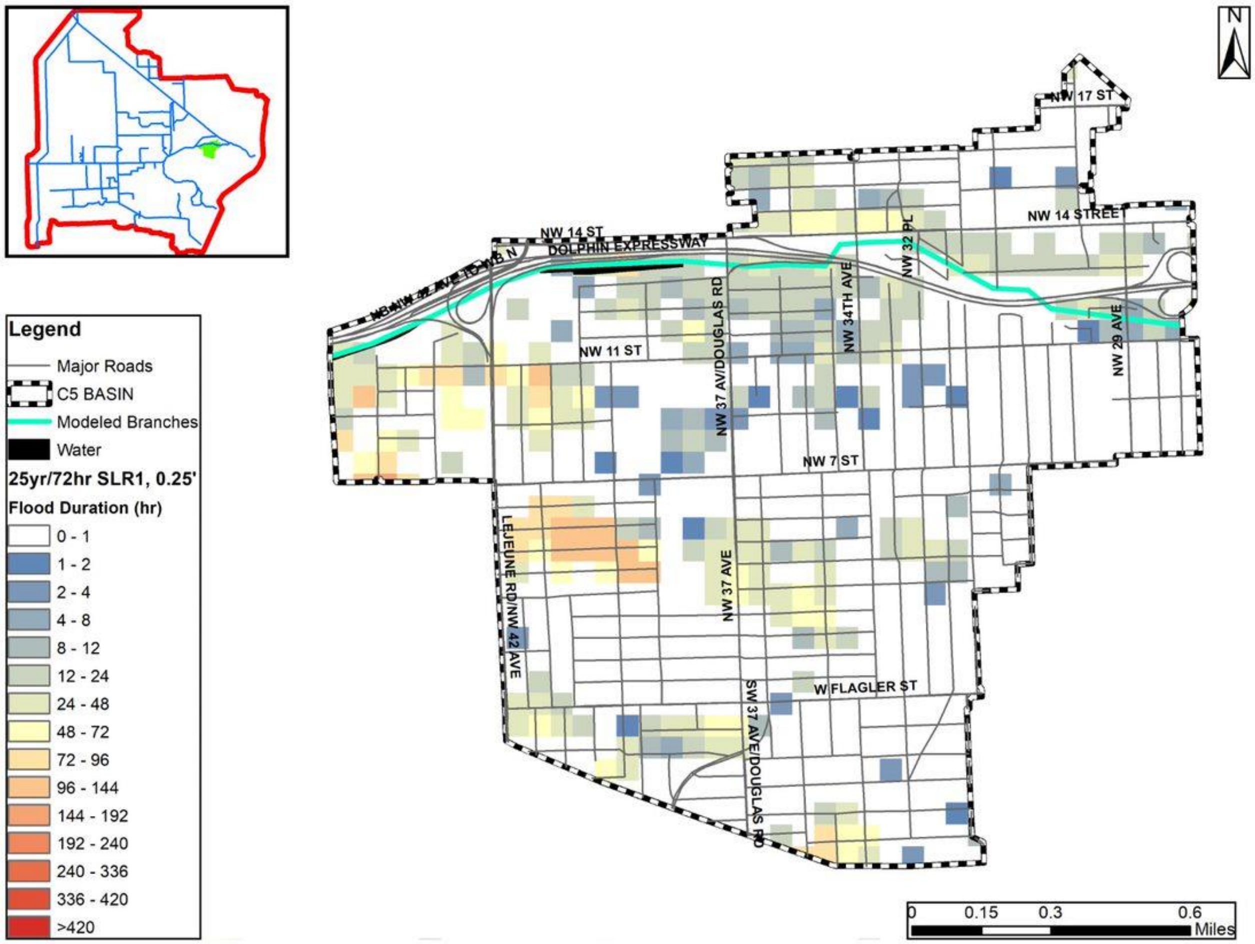


Figure D 4-11. Flood Duration Map for the 25-year 3-Day Design Storm for SLR2 in the C5 Watershed

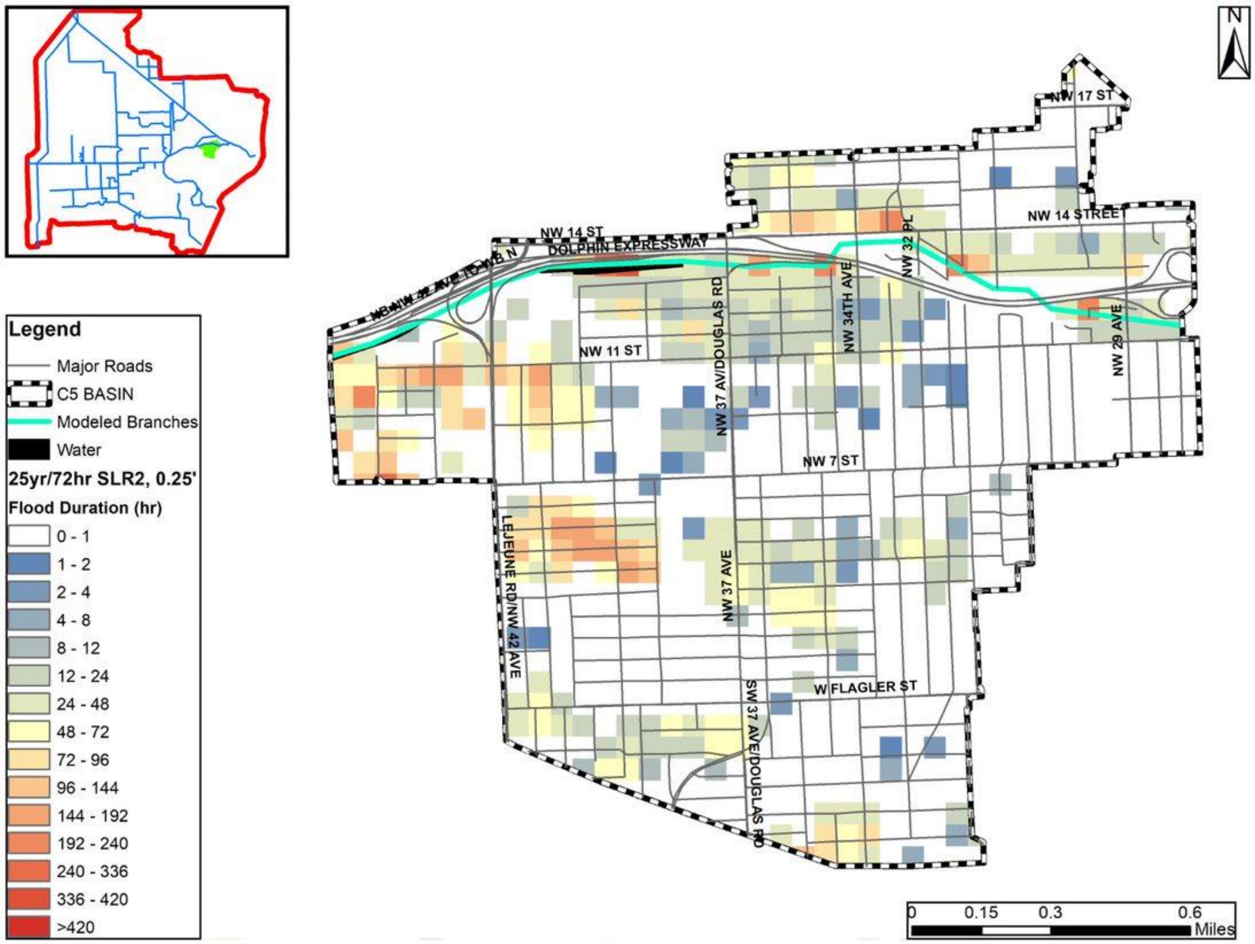


Figure D 4-12. Flood Duration Map for the 25-year 3-Day Design Storm for SLR3 in the C5 Watershed

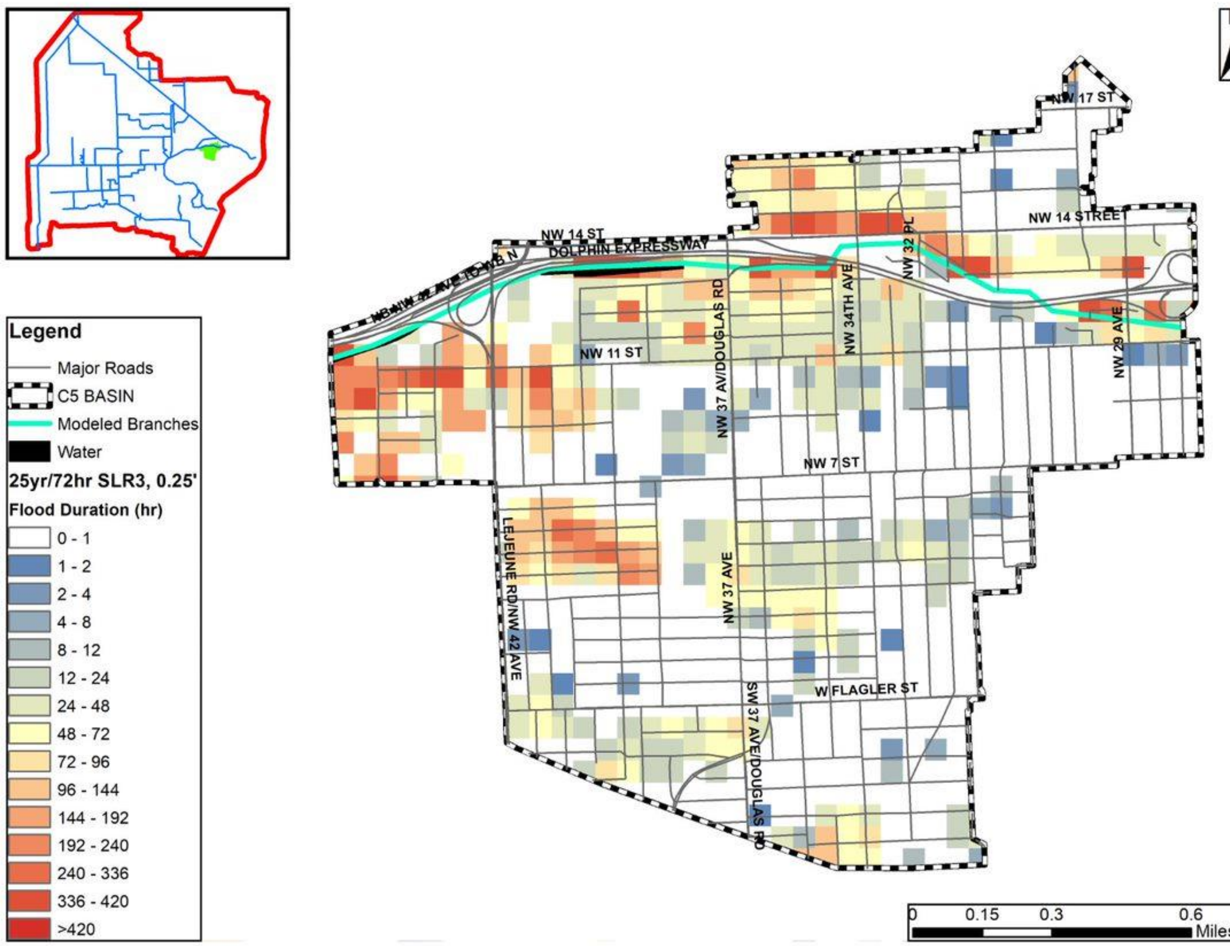


Figure D 4-13. Flood Duration Map for the 100-year 3-Day Design Storm for Current Conditions in the C5 Watershed

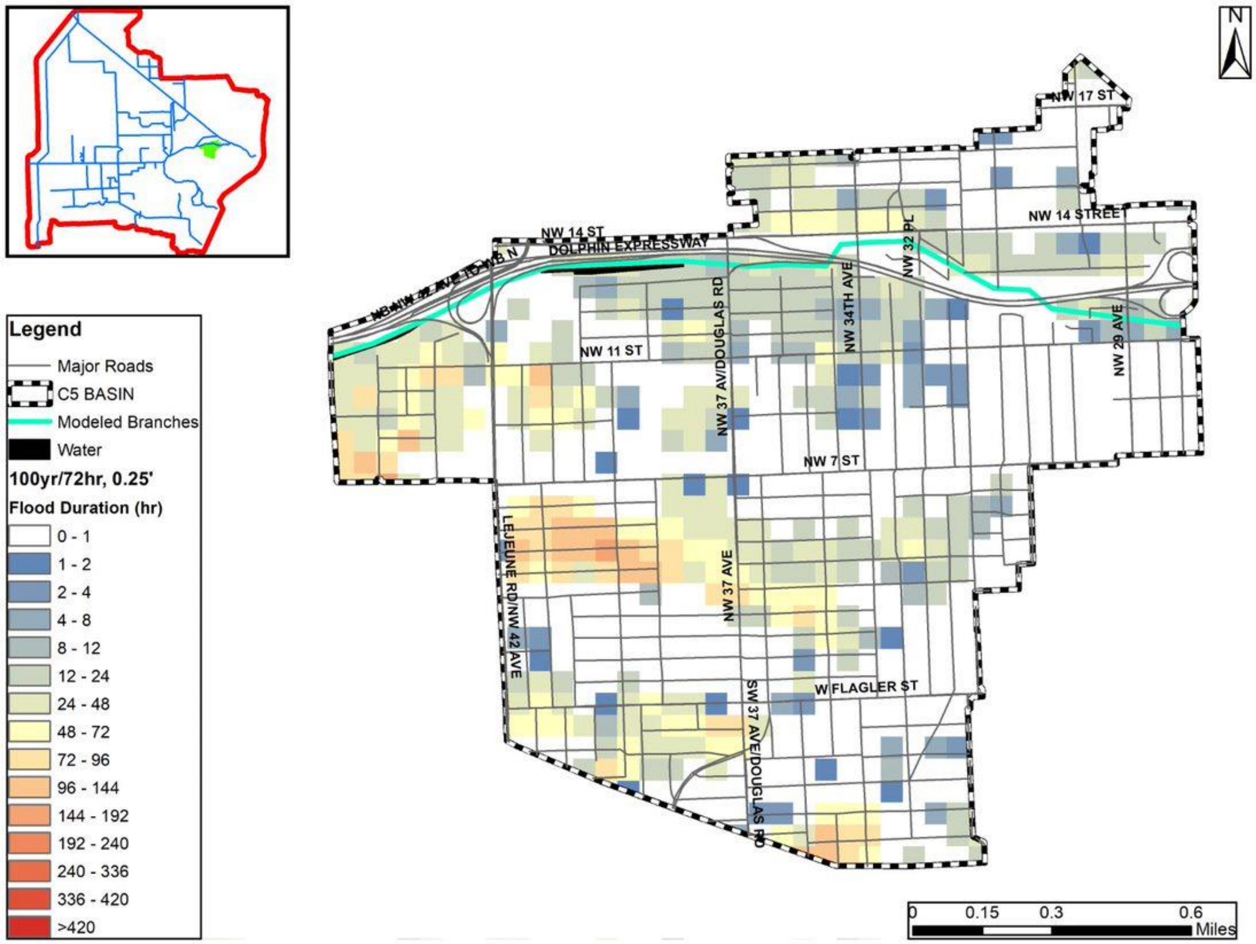


Figure D 4-14. Flood Duration Map for the 100-year 3-Day Design Storm for SLR1 in the C5 Watershed

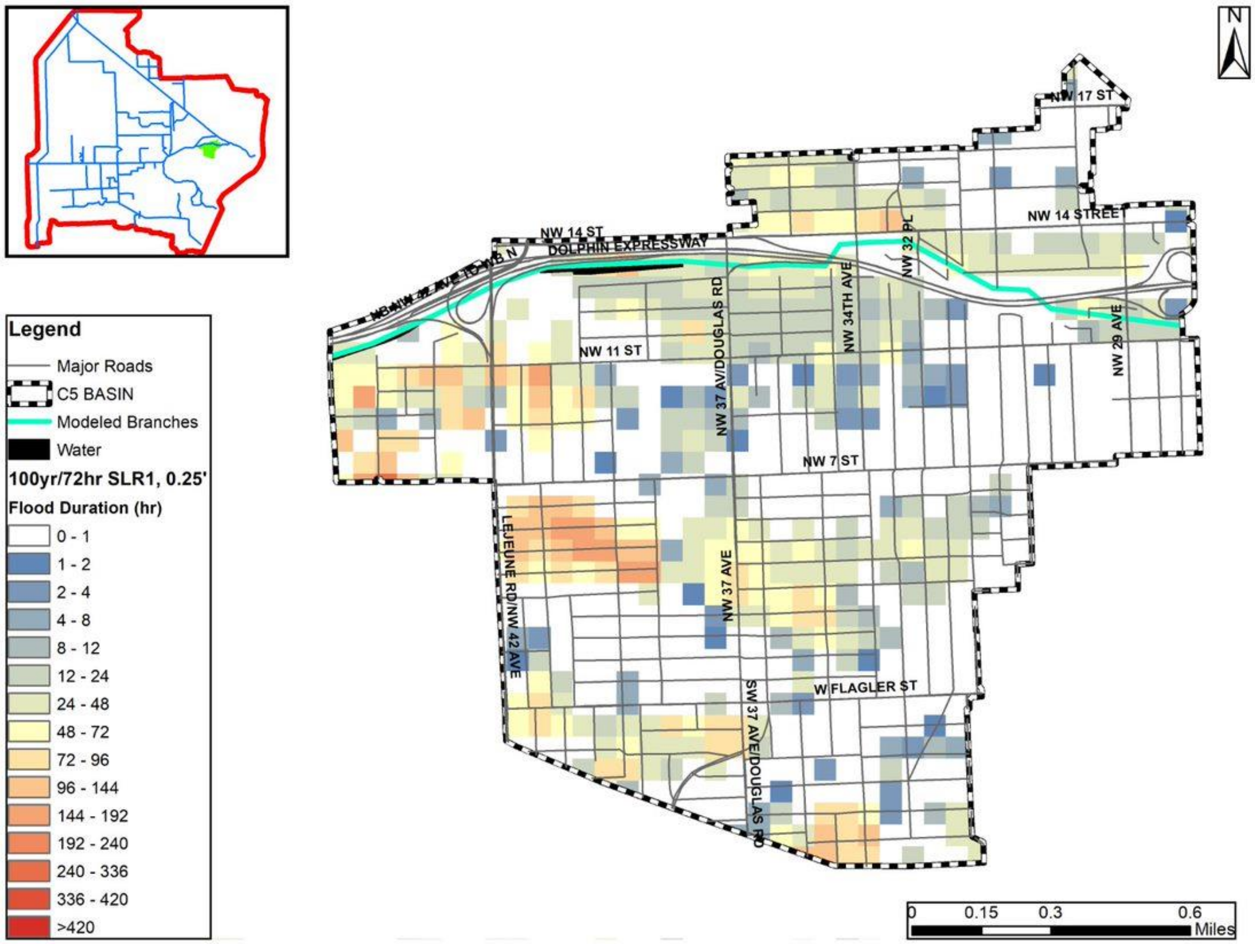


Figure D 4-15. Flood Duration Map for the 100-year 3-Day Design Storm for SLR2 in the C5 Watershed

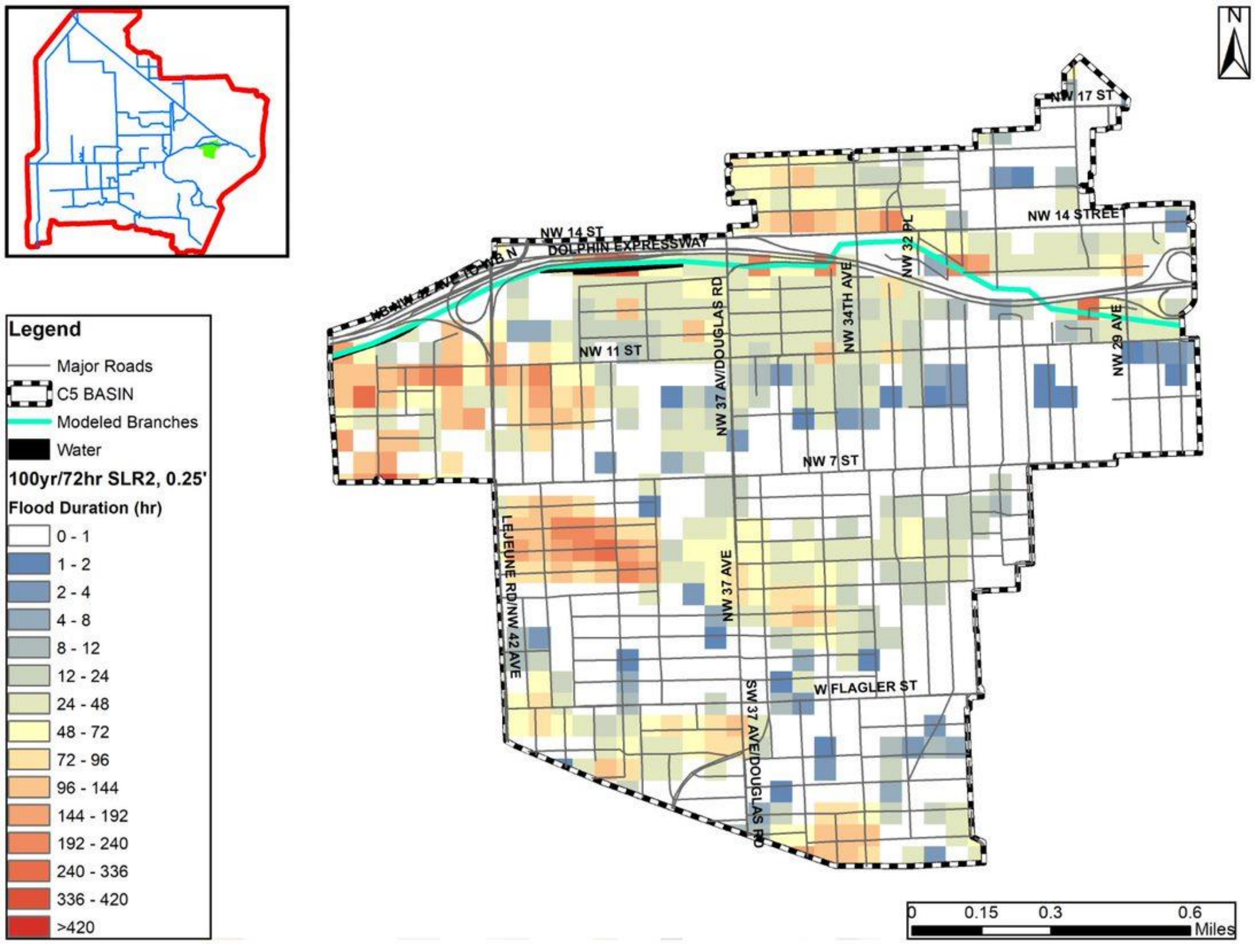


Figure D 4-16. Flood Duration Map for the 100-year 3-Day Design Storm for SLR3 in the C5 Watershed

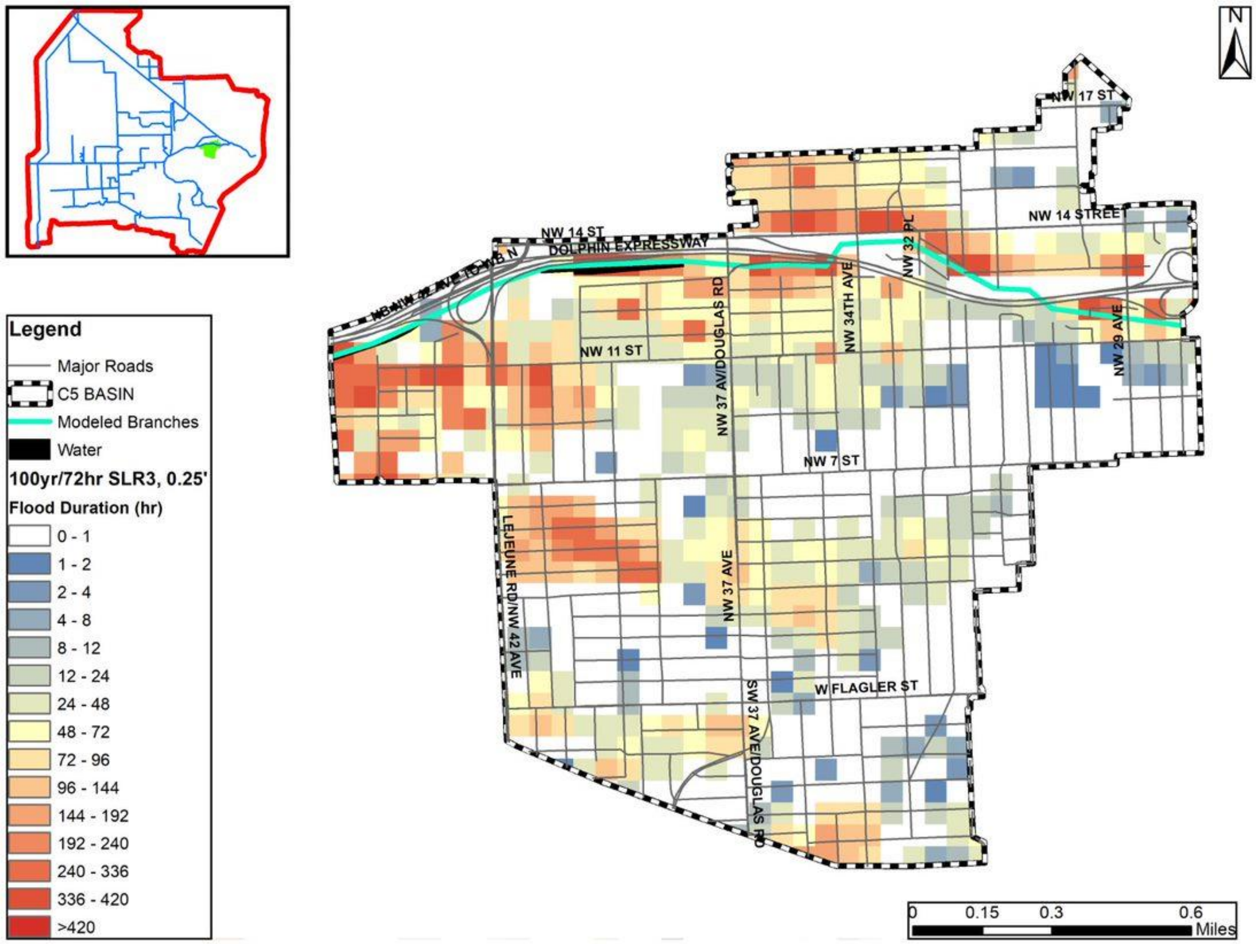


Figure D 4-17. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C5 Watershed

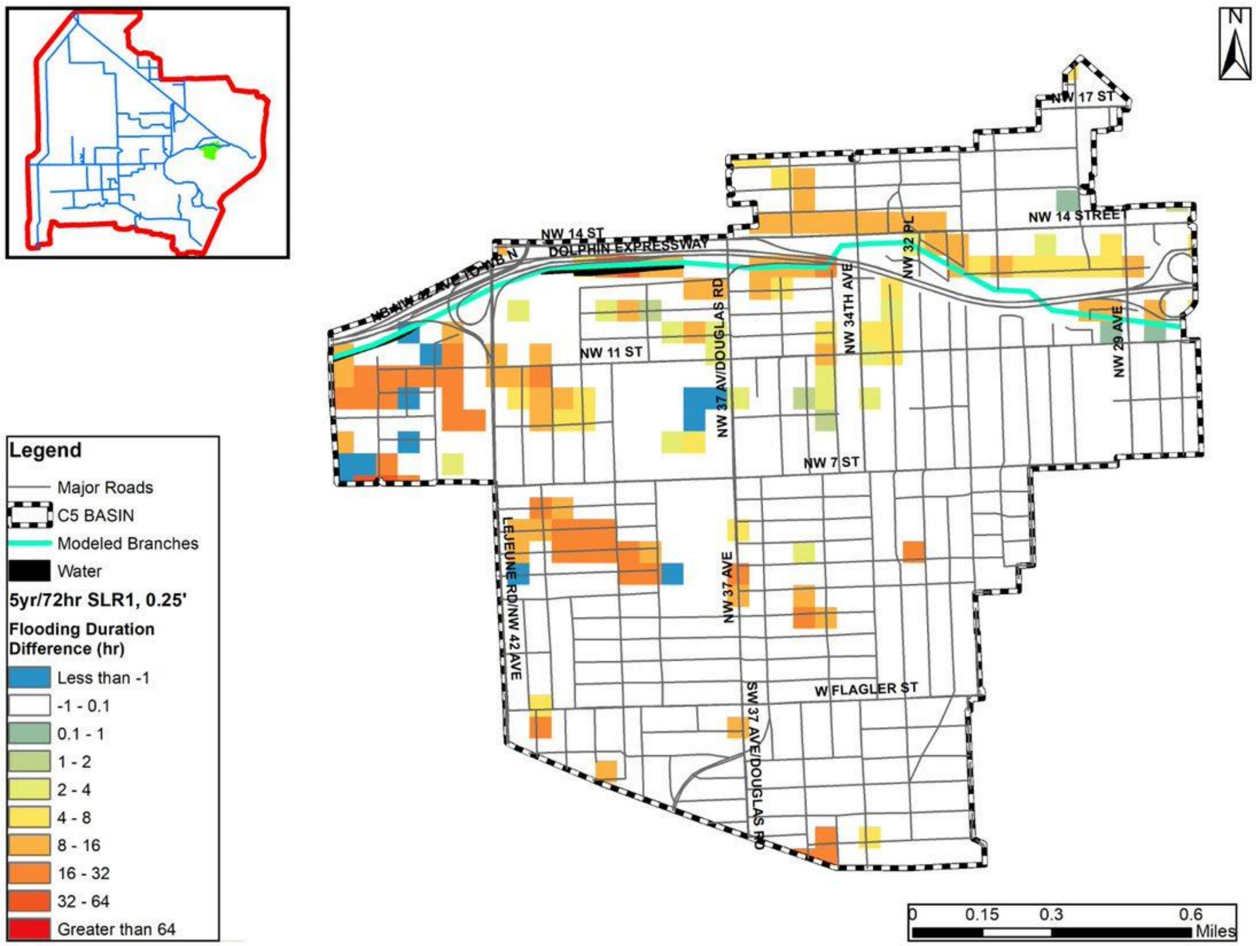


Figure D 4-18. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C5 Watershed

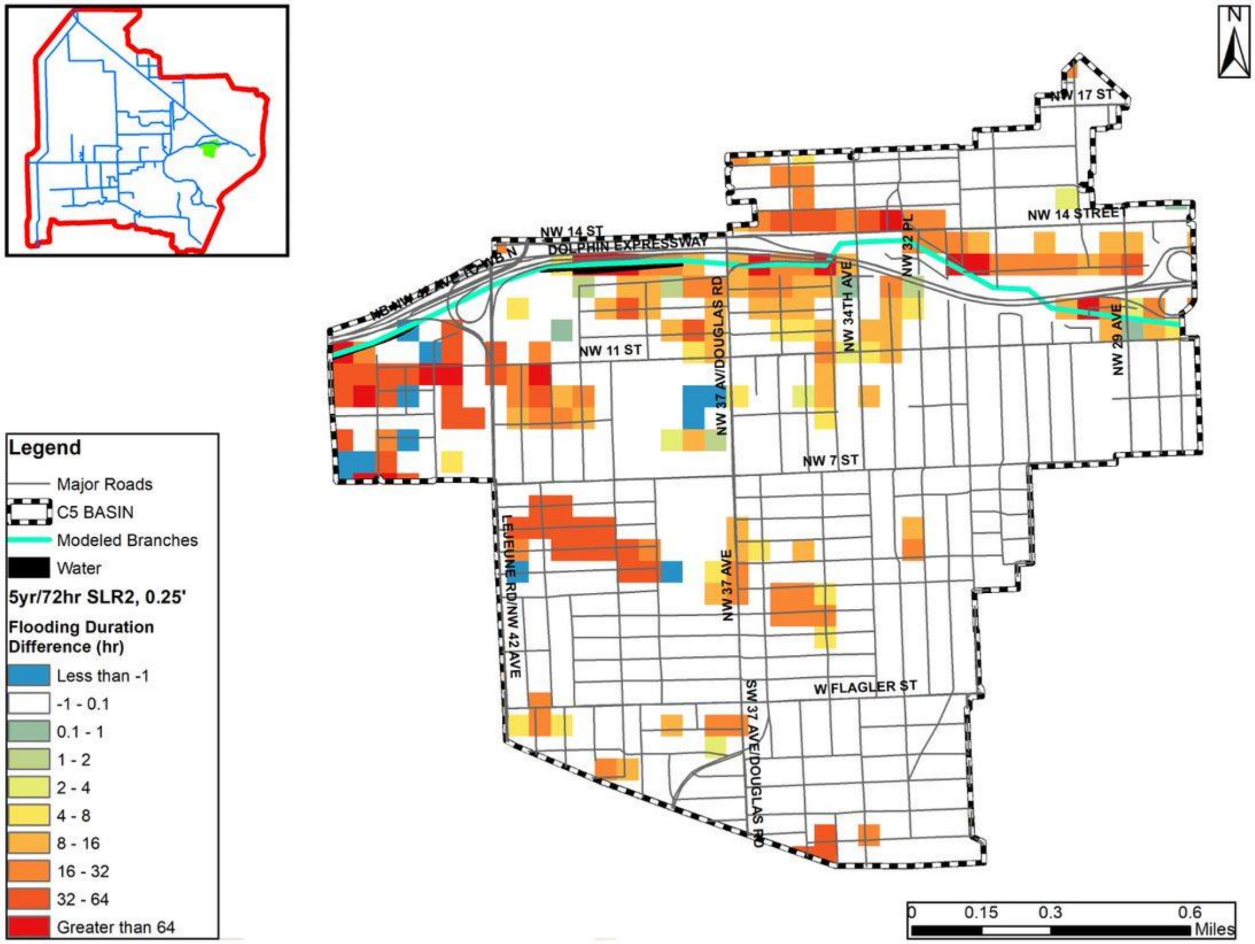


Figure D 4-19. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C5 Watershed

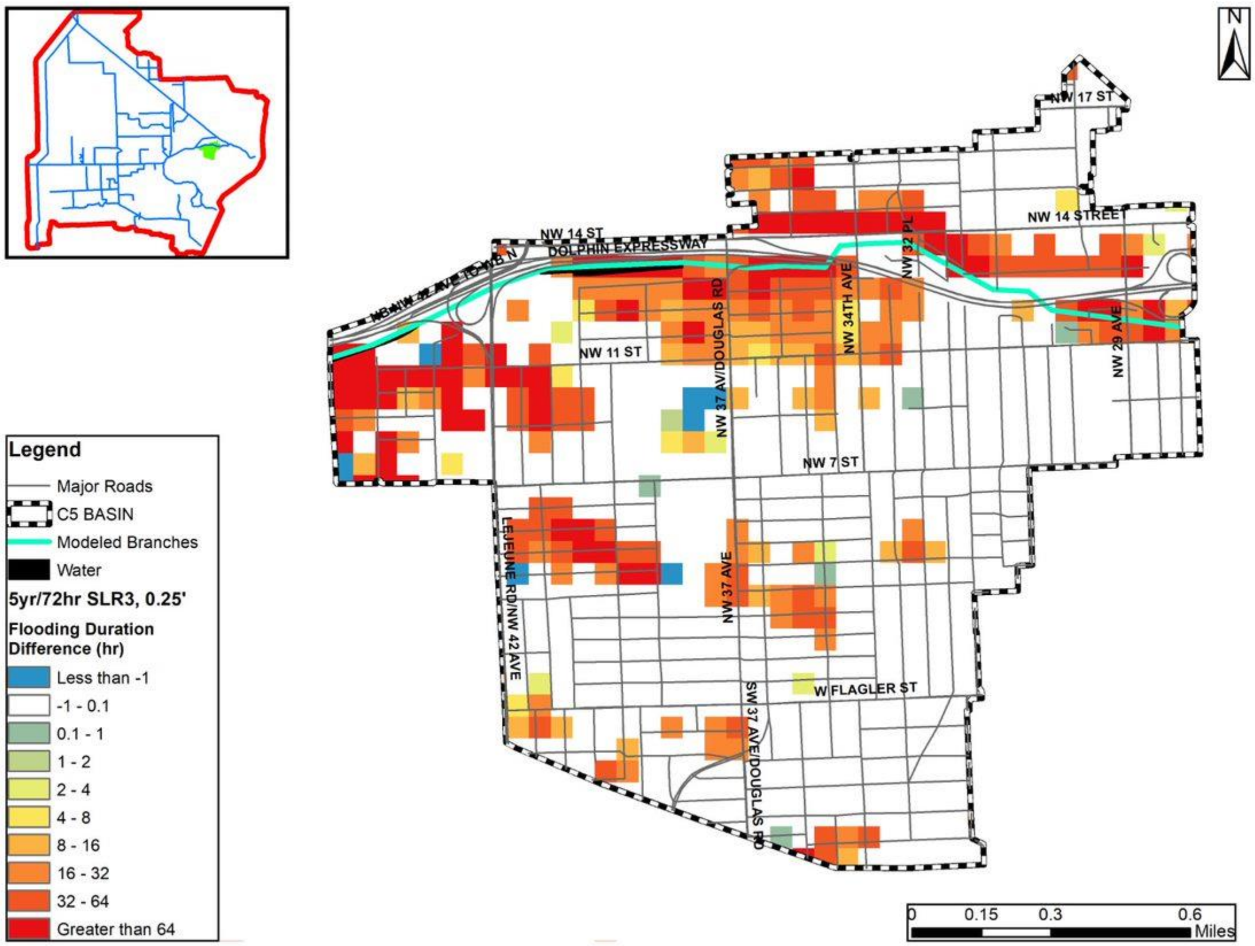


Figure D 4-20. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C5 Watershed

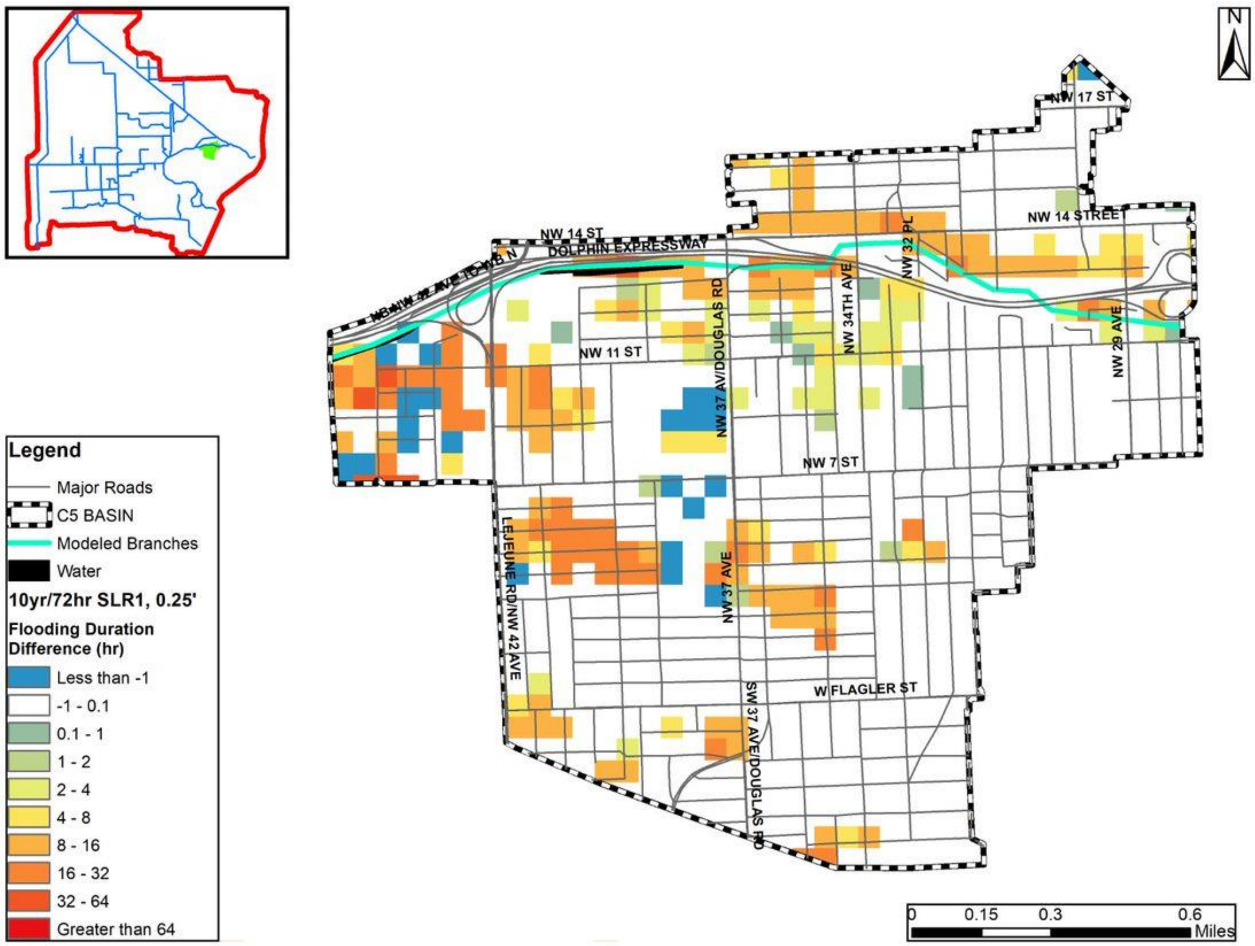


Figure D 4-21. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C5 Watershed

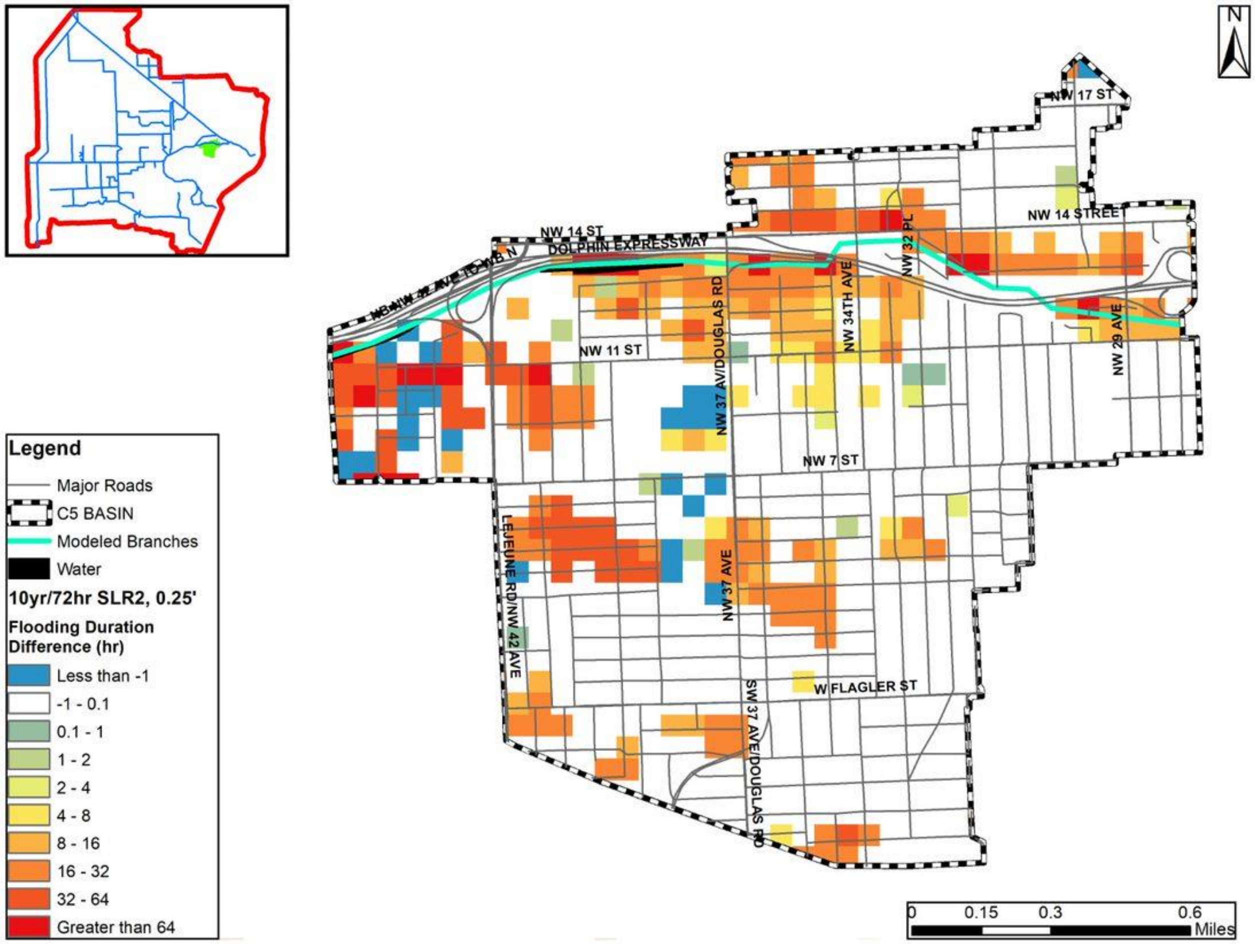


Figure D 4-22. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C5 Watershed

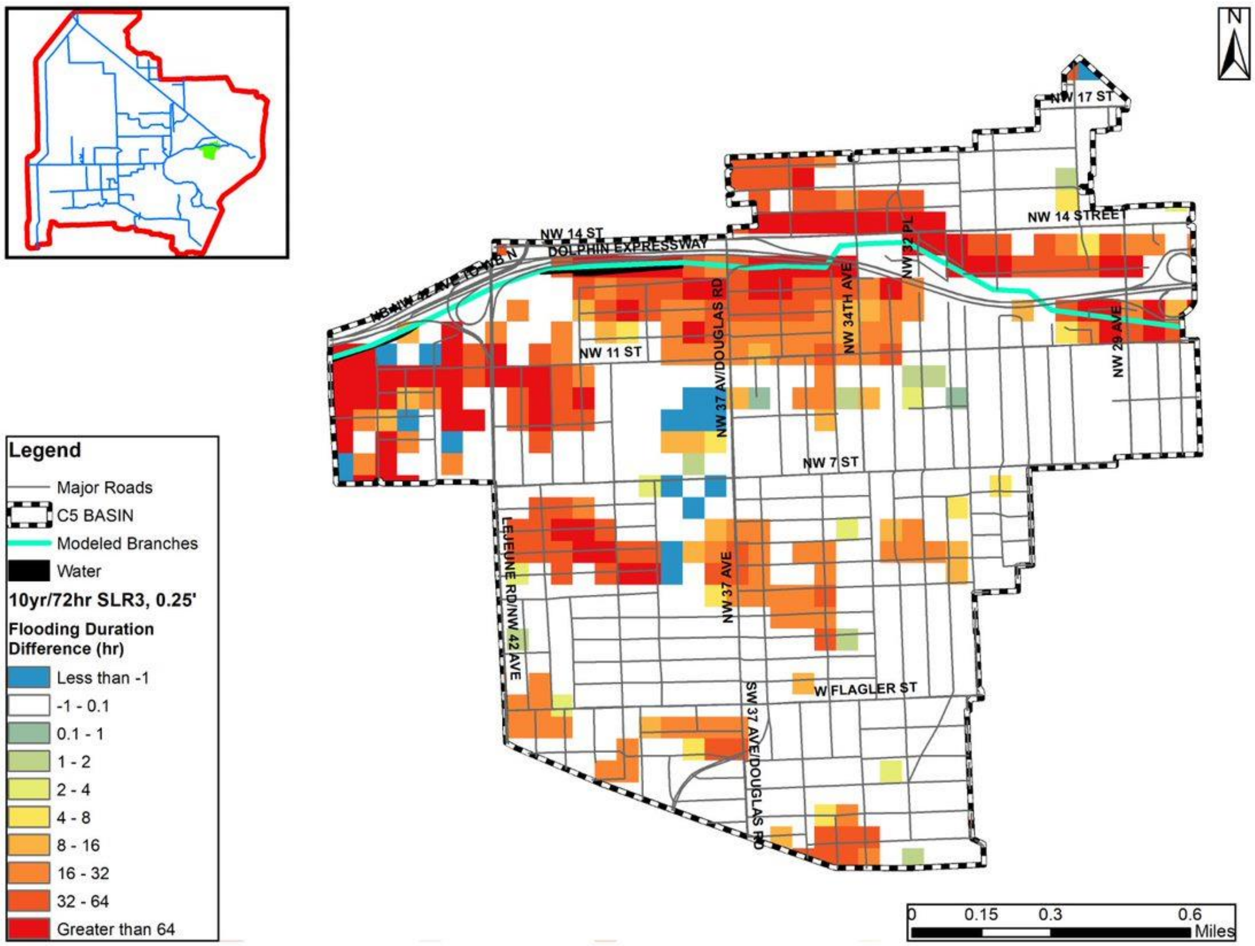


Figure D 4-23. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C5 Watershed

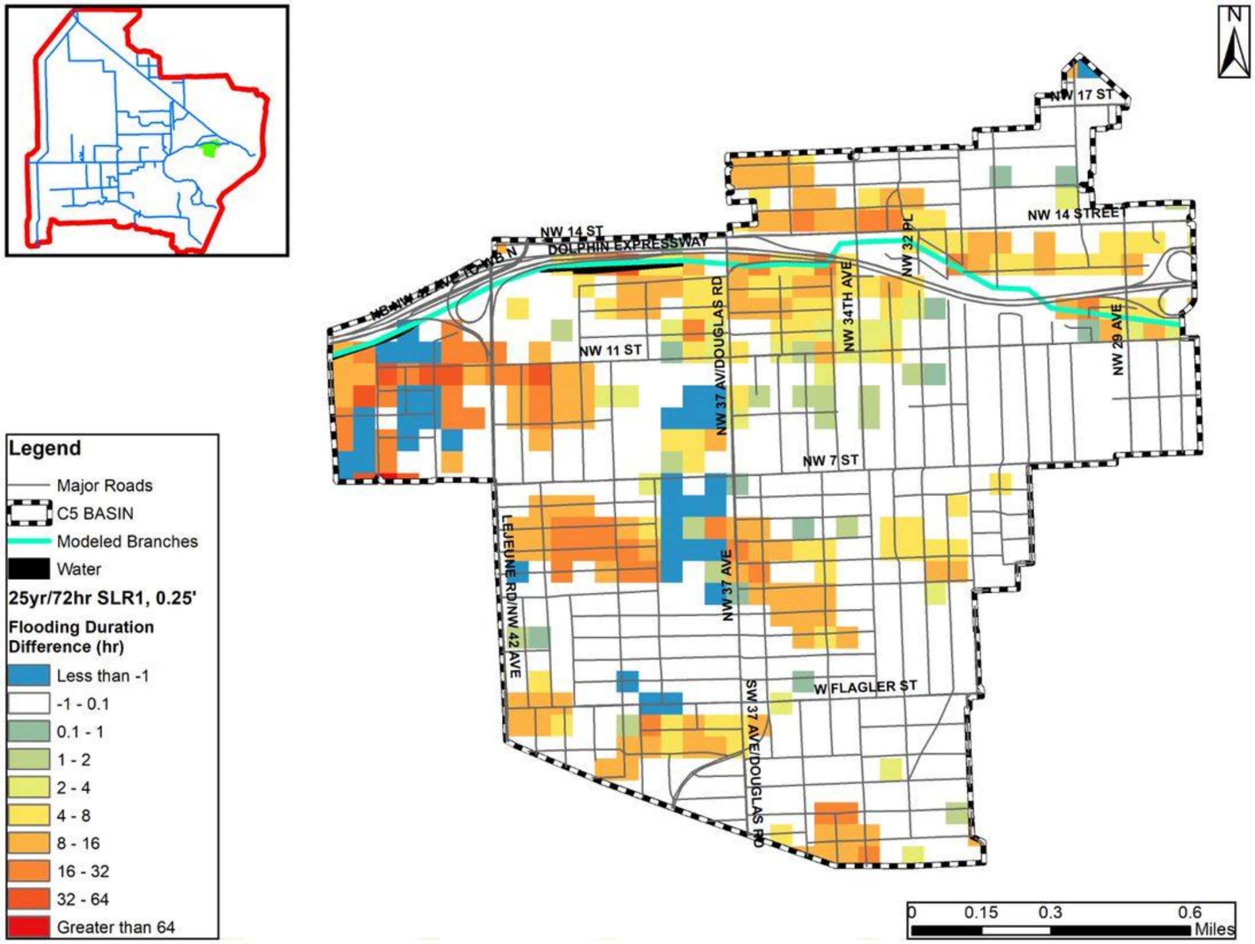


Figure D 4-24. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C5 Watershed

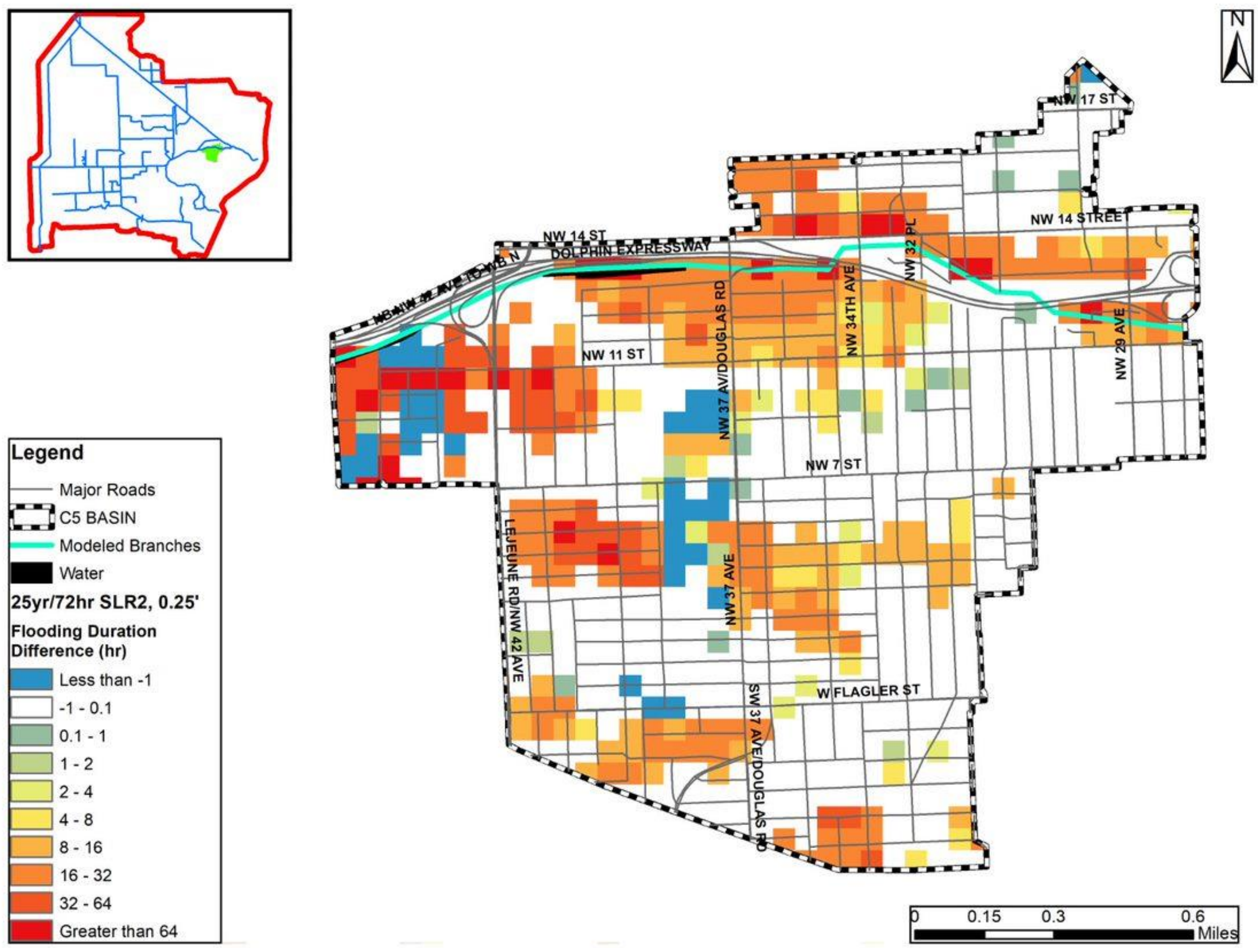


Figure D 4-25. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C5 Watershed

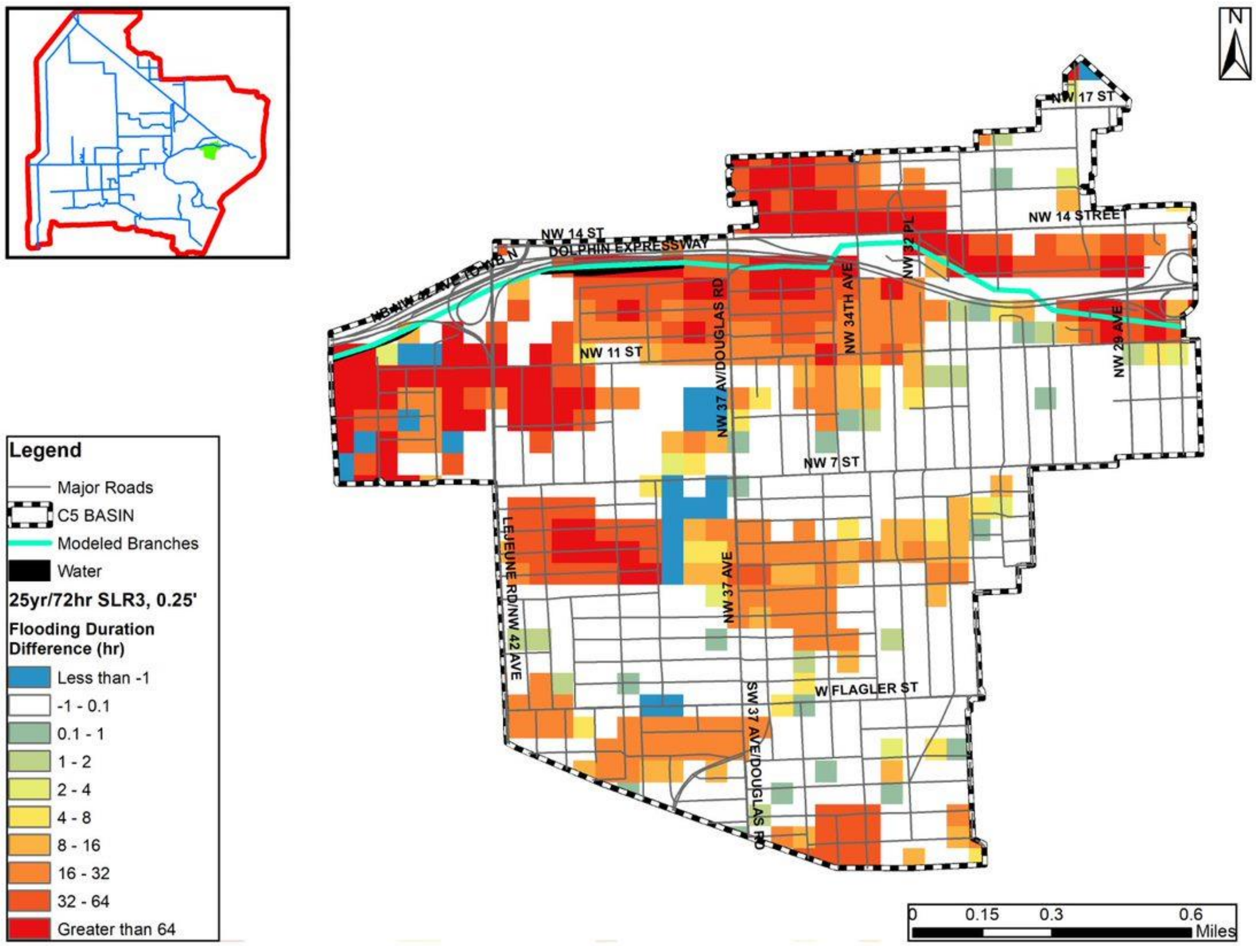


Figure D 4-26. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C5 Watershed

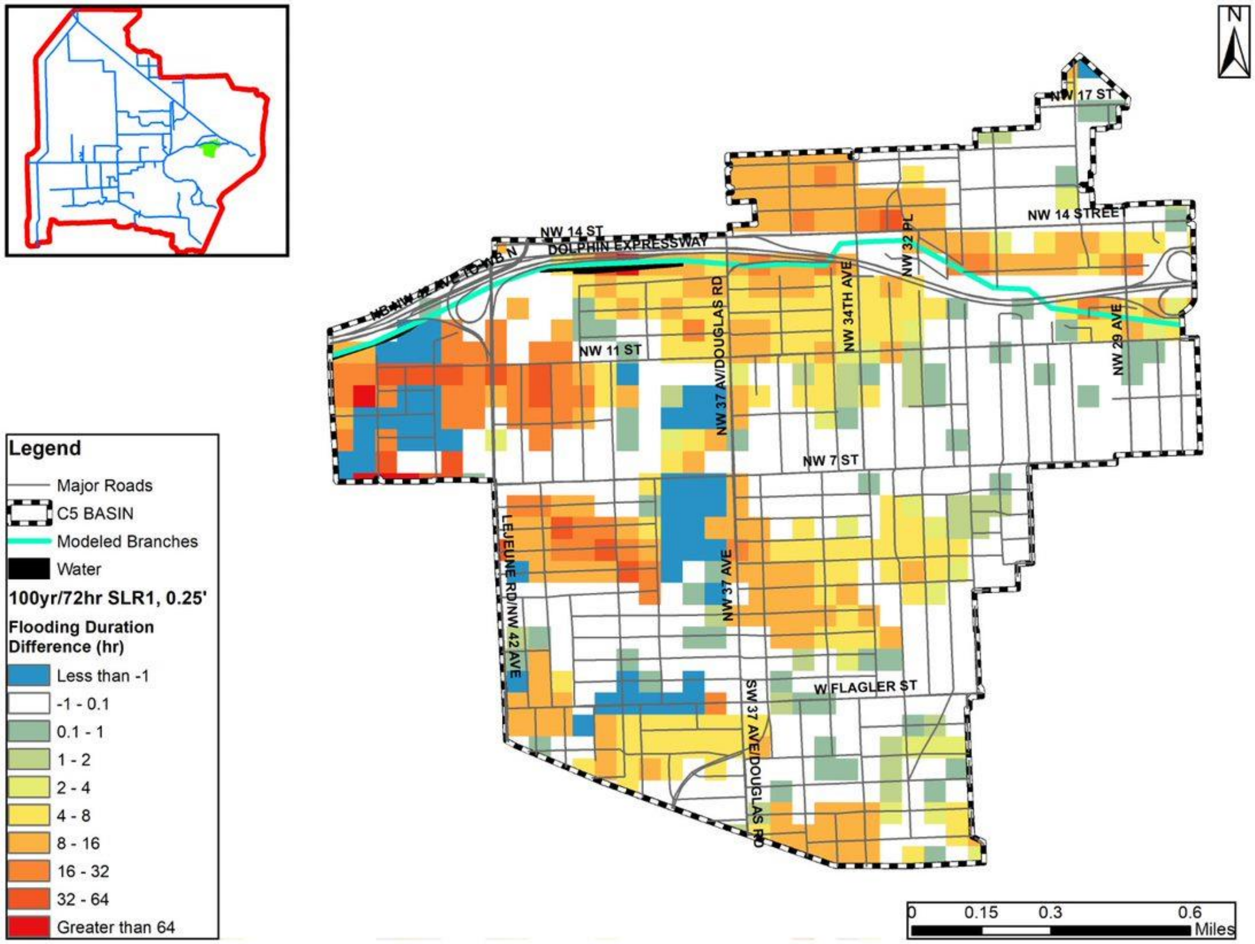


Figure D 4-27. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C5 Watershed

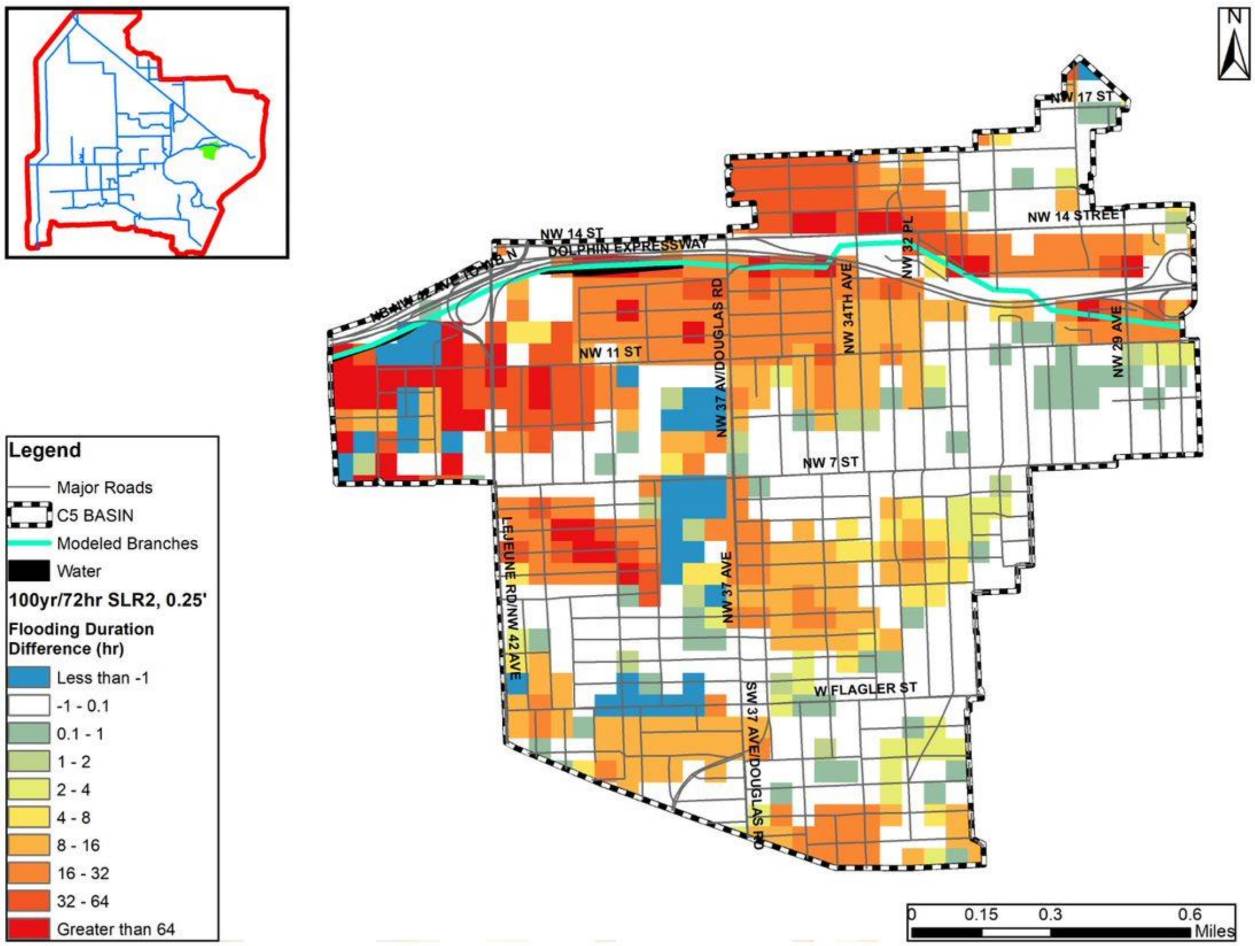
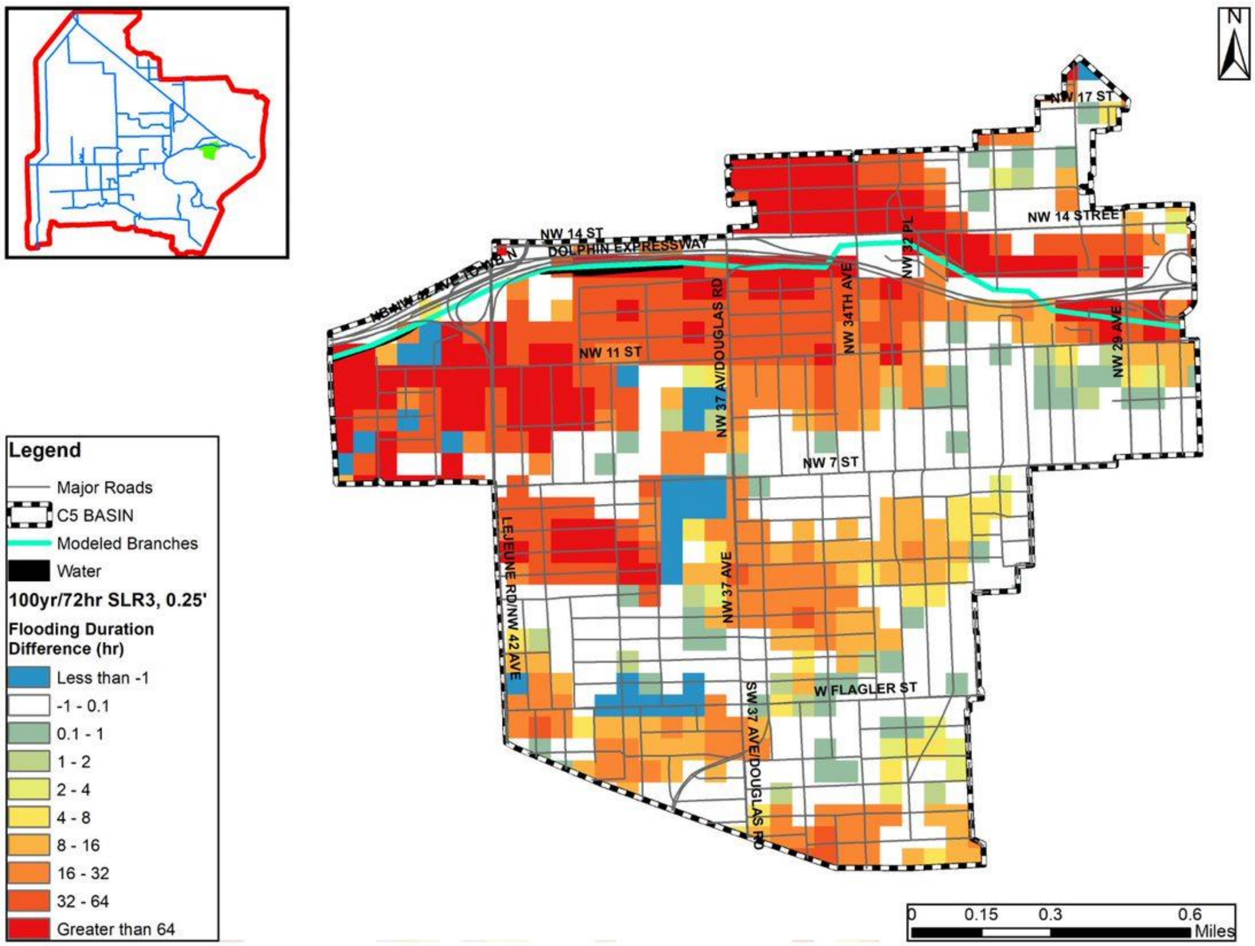


Figure D 4-28. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C5 Watershed



5 C6 Watershed

Figure D 5-1 through Figure D 5-16 provides flood duration maps for the C6 Watershed for overland flooding depths exceeding 0.25 ft for the 5-, 10-, 25-, and 100-year 3-day design storms, respectively, for each SLR condition. Water areas are masked in black. Figure D 5-17 through **Figure D 5-32** provide the flood duration for only the urban areas within the C2 Watershed (non-urban areas are masked out).

Figure D 5-33 through **Figure D 5-44** show the difference in overland flooding for the C6 Watershed between the current conditions and future sea level rise conditions for the SLR +1ft, SLR +2ft, and SLR +3ft simulations, respectively. Figure D 5-45 through **Figure D 5-56** show the same maps with the non-urban areas masked out.

Figure D 5-1. Flood Duration Map for the 5-year 3-Day Design Storm for Current Conditions in the C6 Watershed

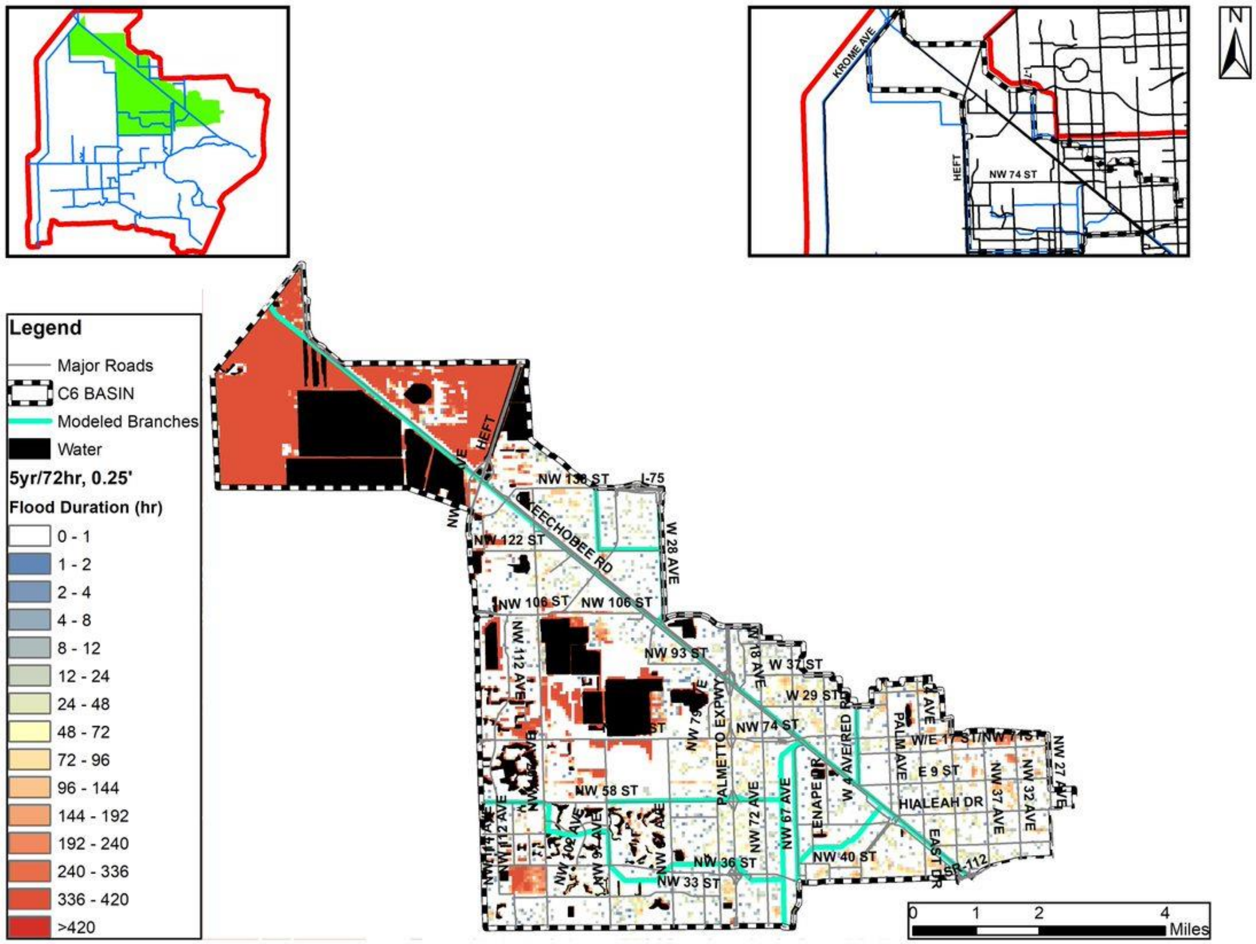


Figure D 5-2. Flood Duration Map for the 5-year 3-Day Design Storm for SLR1 in the C6 Watershed

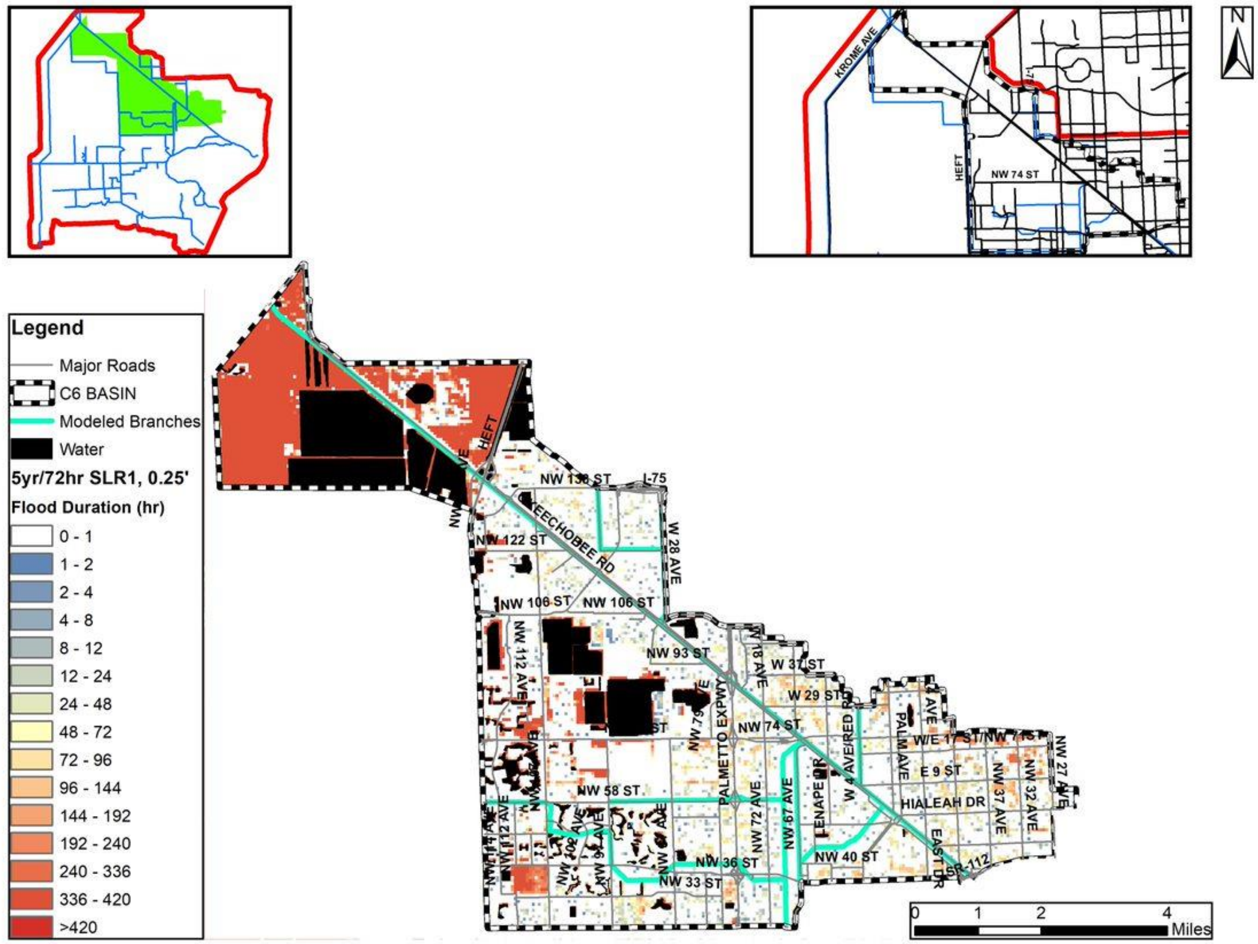


Figure D 5-3. Flood Duration Map for the 5-year 3-Day Design Storm for SLR2 in the C6 Watershed

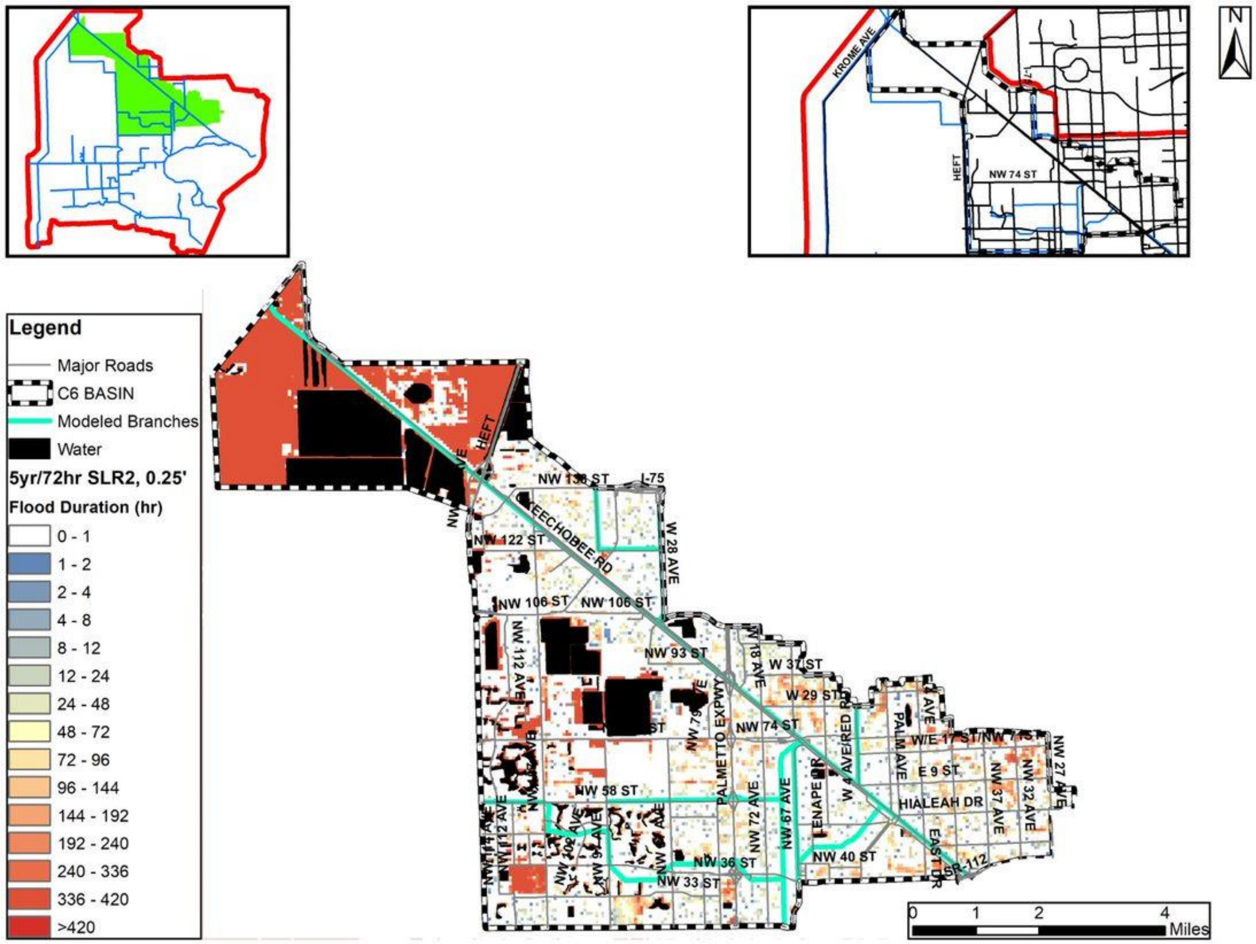


Figure D 5-4. Flood Duration Map for the 5-year 3-Day Design Storm for SLR3 in the C6 Watershed

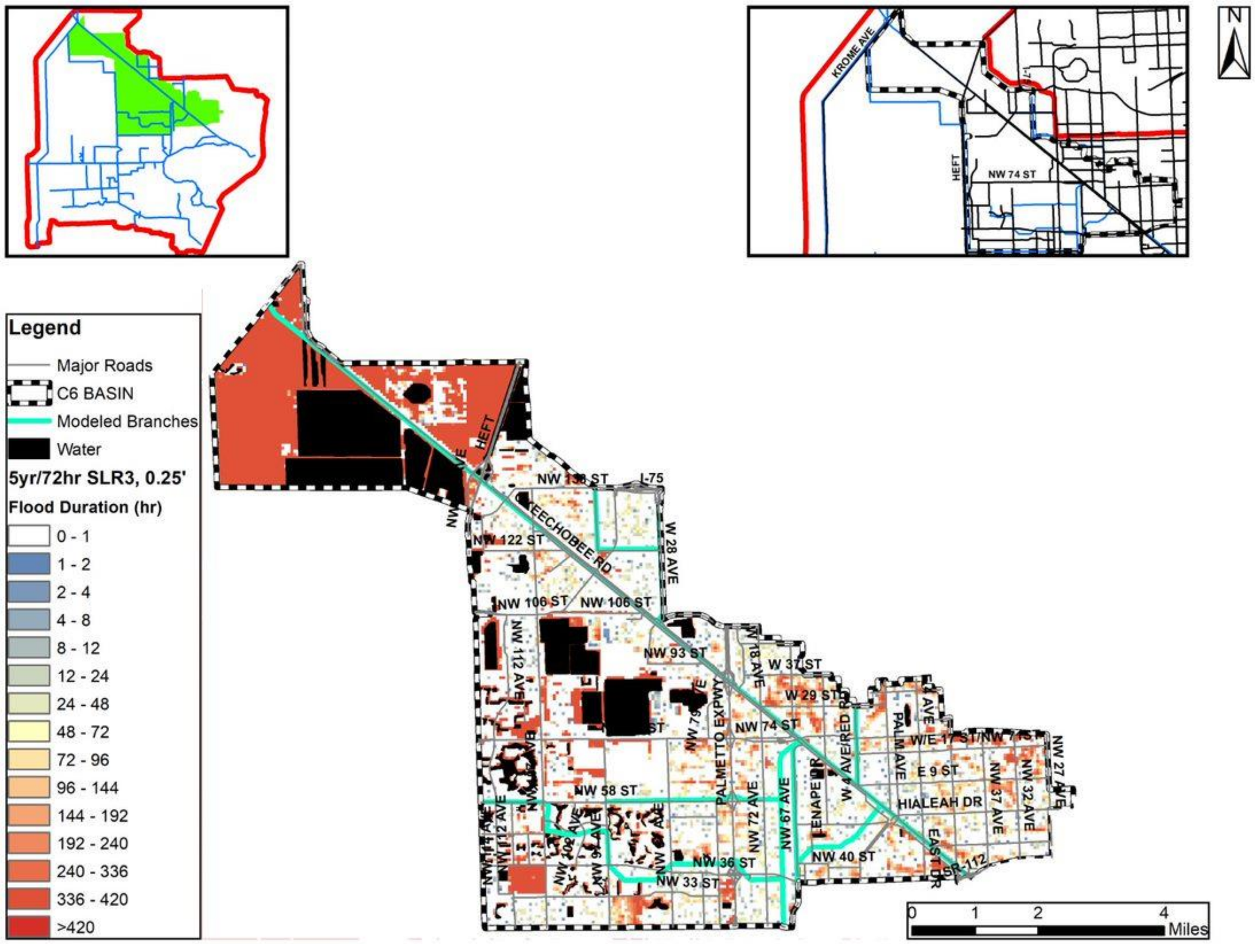


Figure D 5-5. Flood Duration Map for the 10-year 3-Day Design Storm for Current Conditions in the C6 Watershed

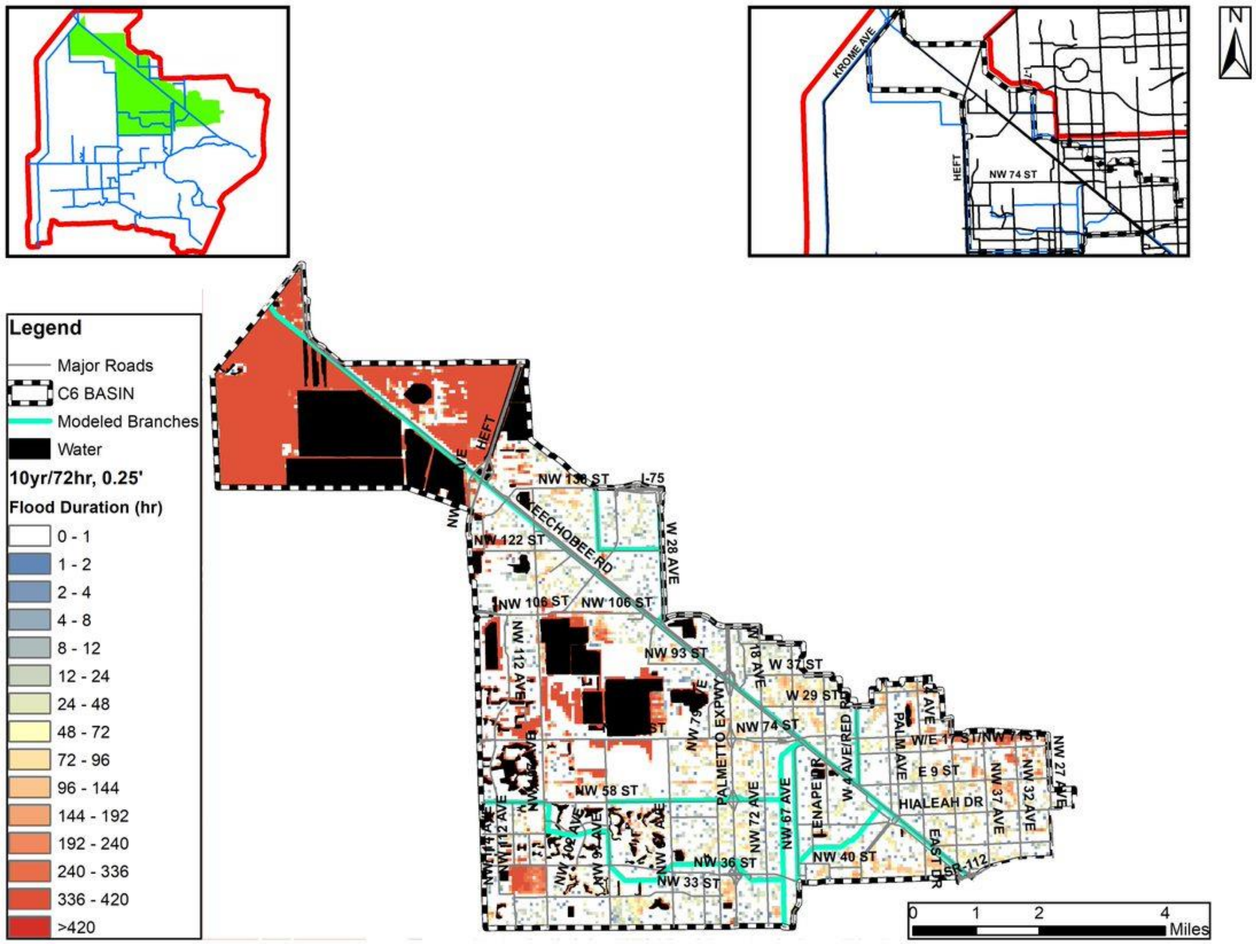


Figure D 5-6. Flood Duration Map for the 10-year 3-Day Design Storm for SLR1 in the C6 Watershed

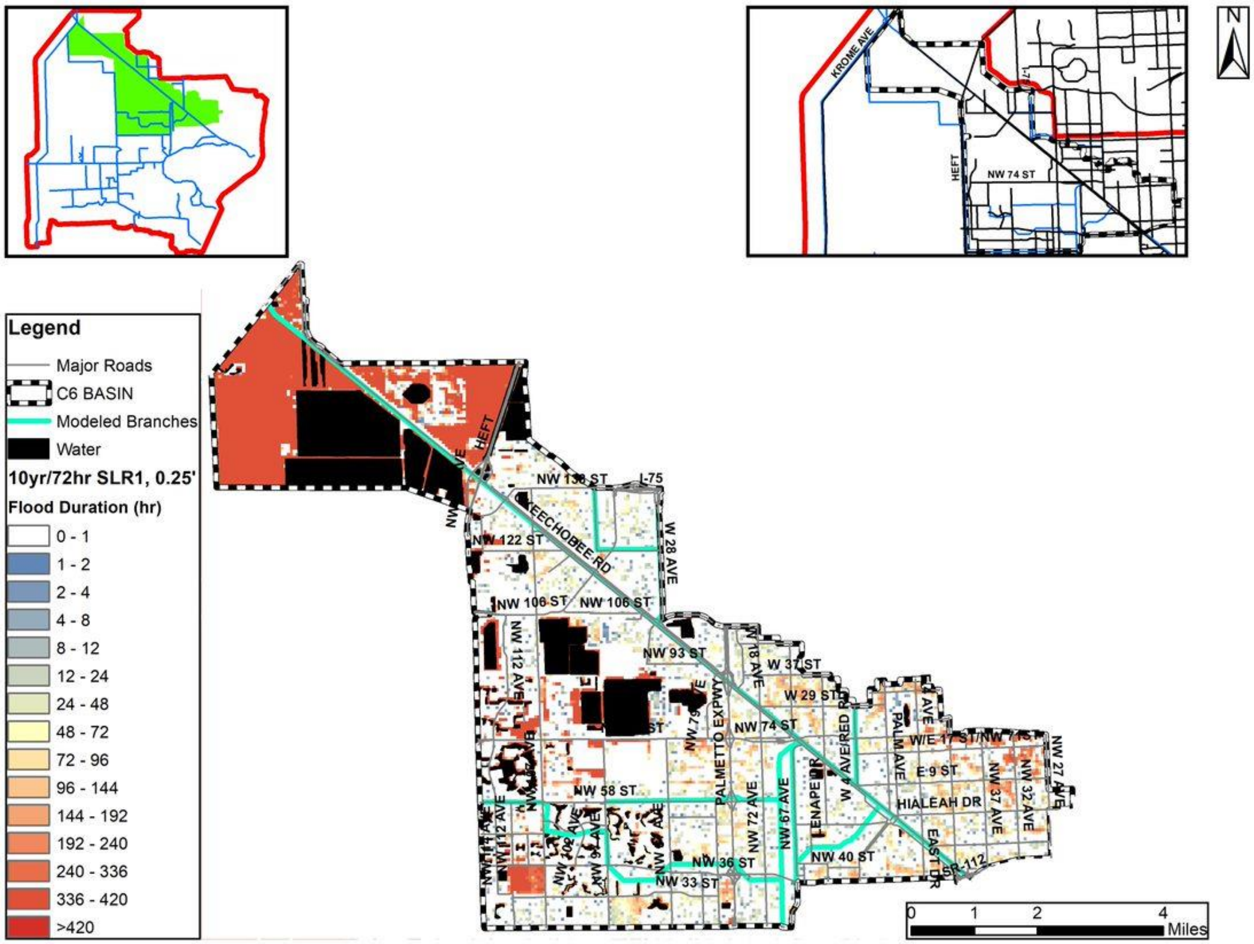


Figure D 5-7. Flood Duration Map for the 10-year 3-Day Design Storm for SLR2 in the C6 Watershed

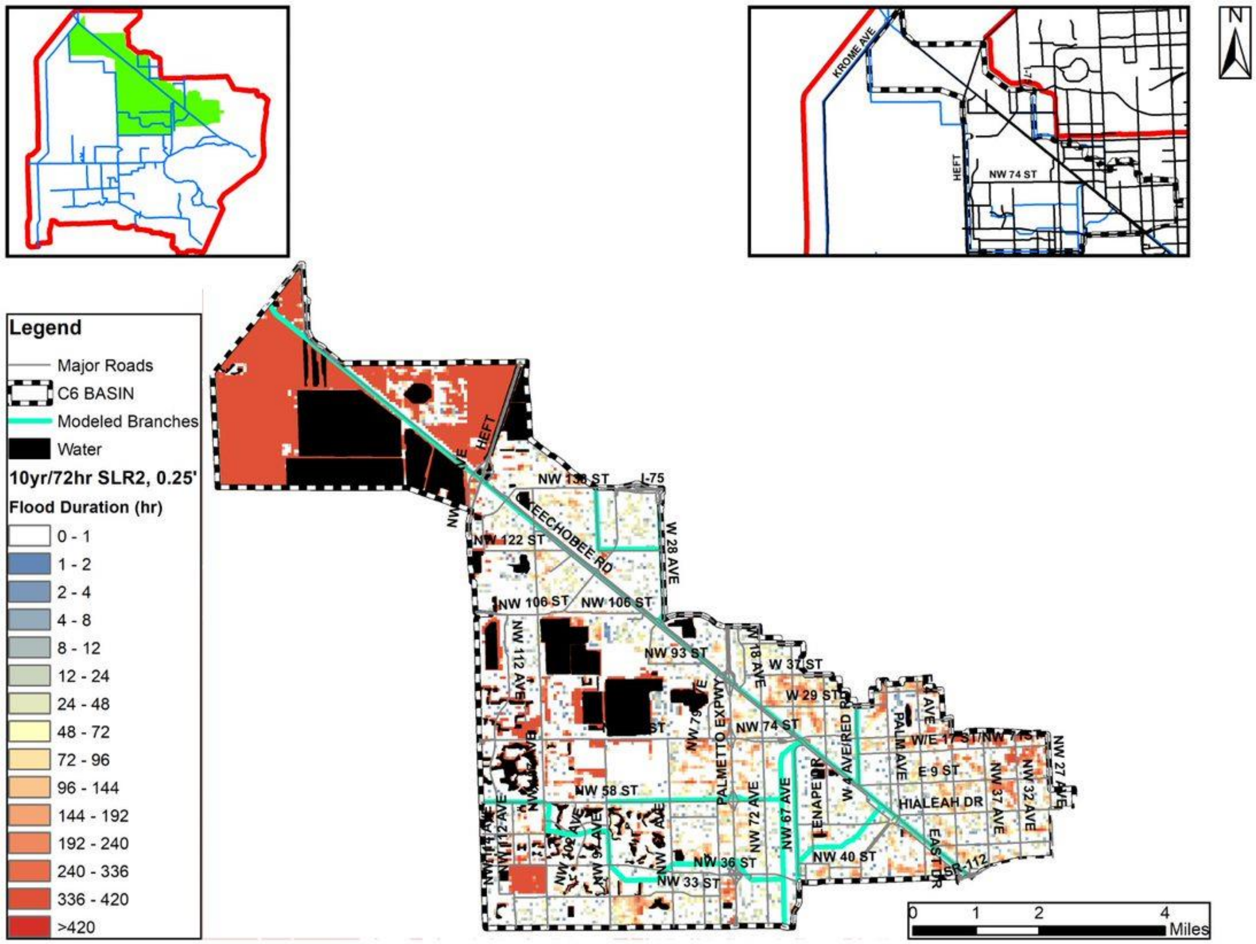


Figure D 5-8. Flood Duration Map for the 10-year 3-Day Design Storm for SLR3 in the C6 Watershed

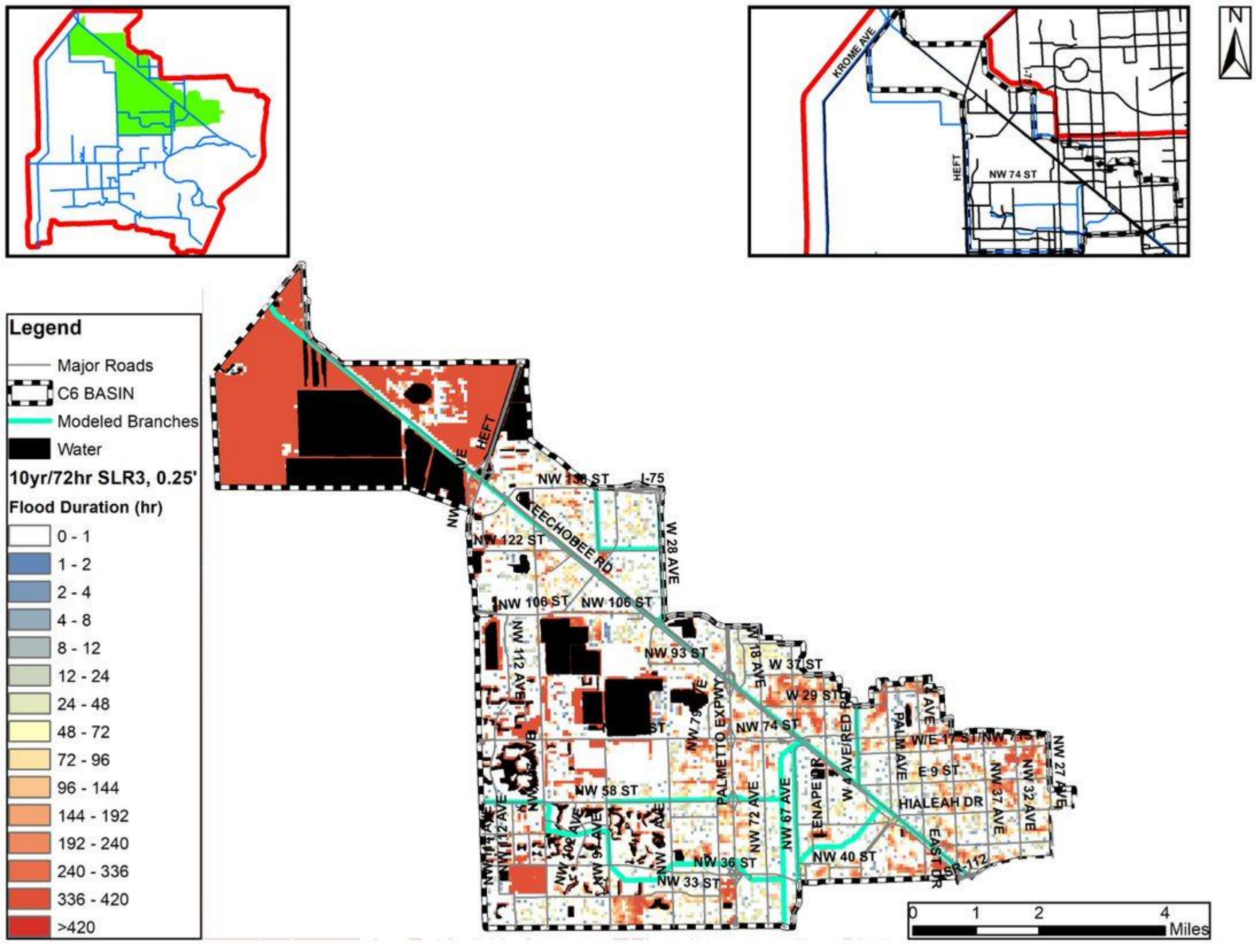


Figure D 5-9. Flood Duration Map for the 25-year 3-Day Design Storm for Current Conditions in the C6 Watershed

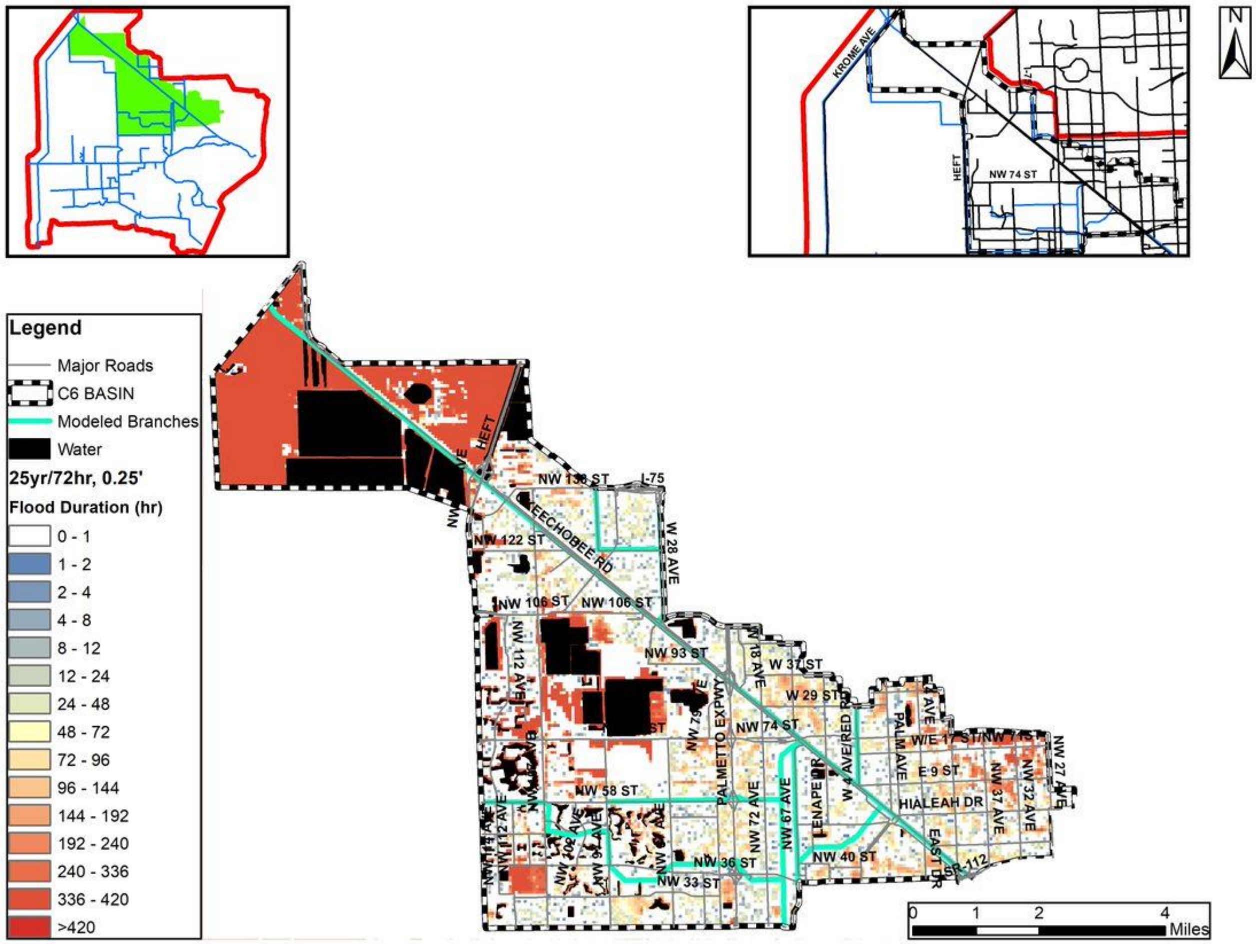


Figure D 5-10. Flood Duration Map for the 25-year 3-Day Design Storm for SLR1 in the C6 Watershed

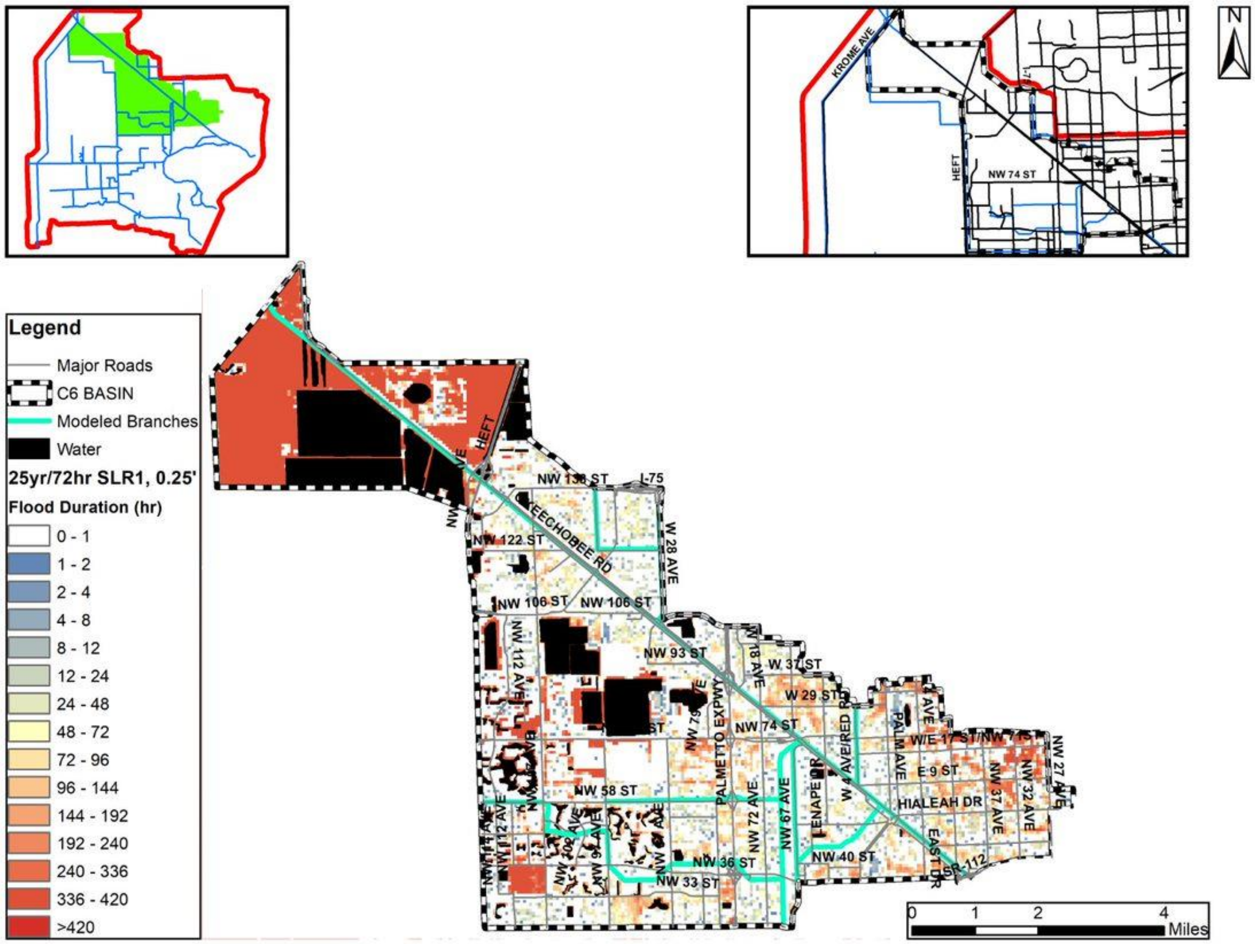


Figure D 5-11. Flood Duration Map for the 25-year 3-Day Design Storm for SLR2 in the C6 Watershed

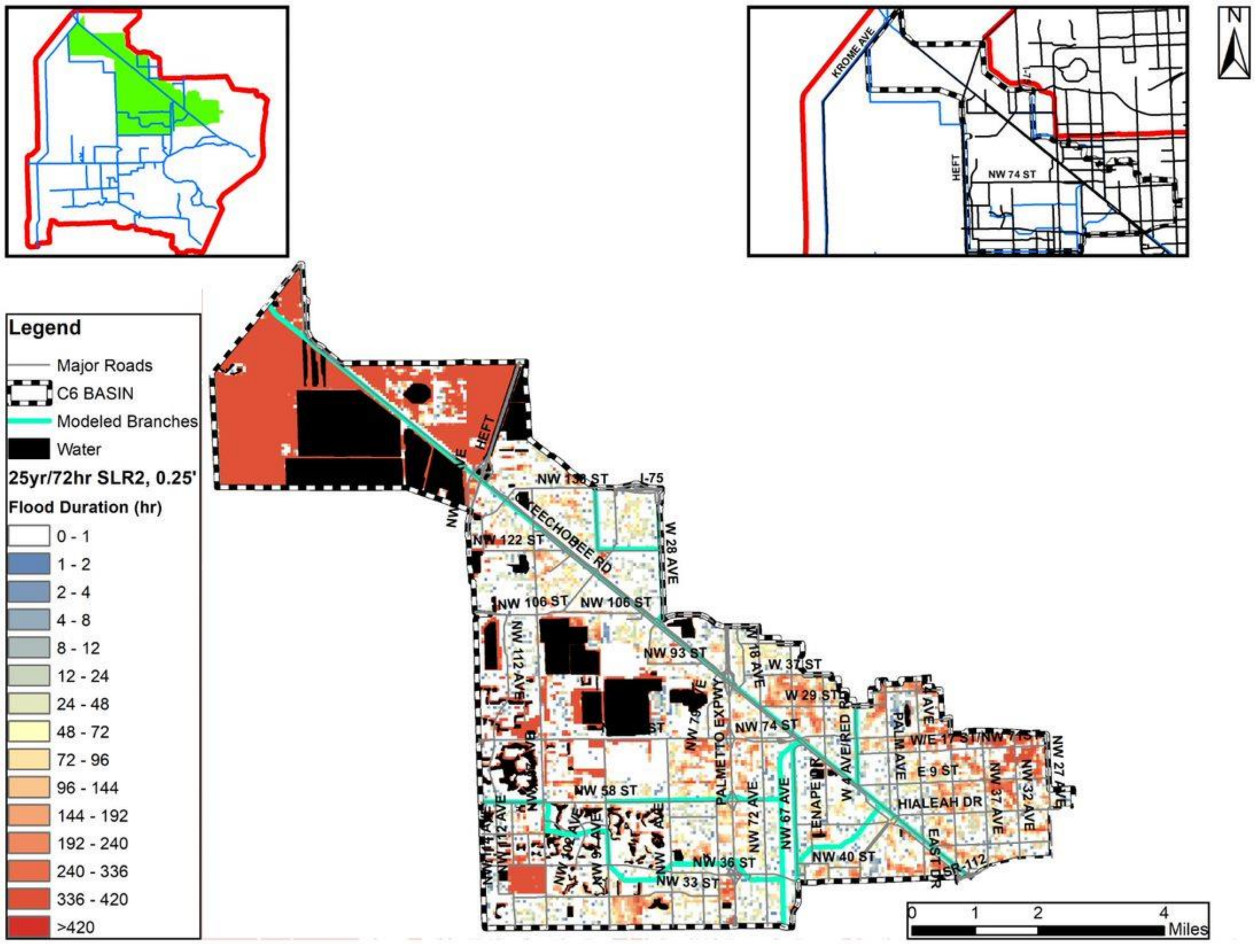


Figure D 5-12. Flood Duration Map for the 25-year 3-Day Design Storm for SLR3 in the C6 Watershed

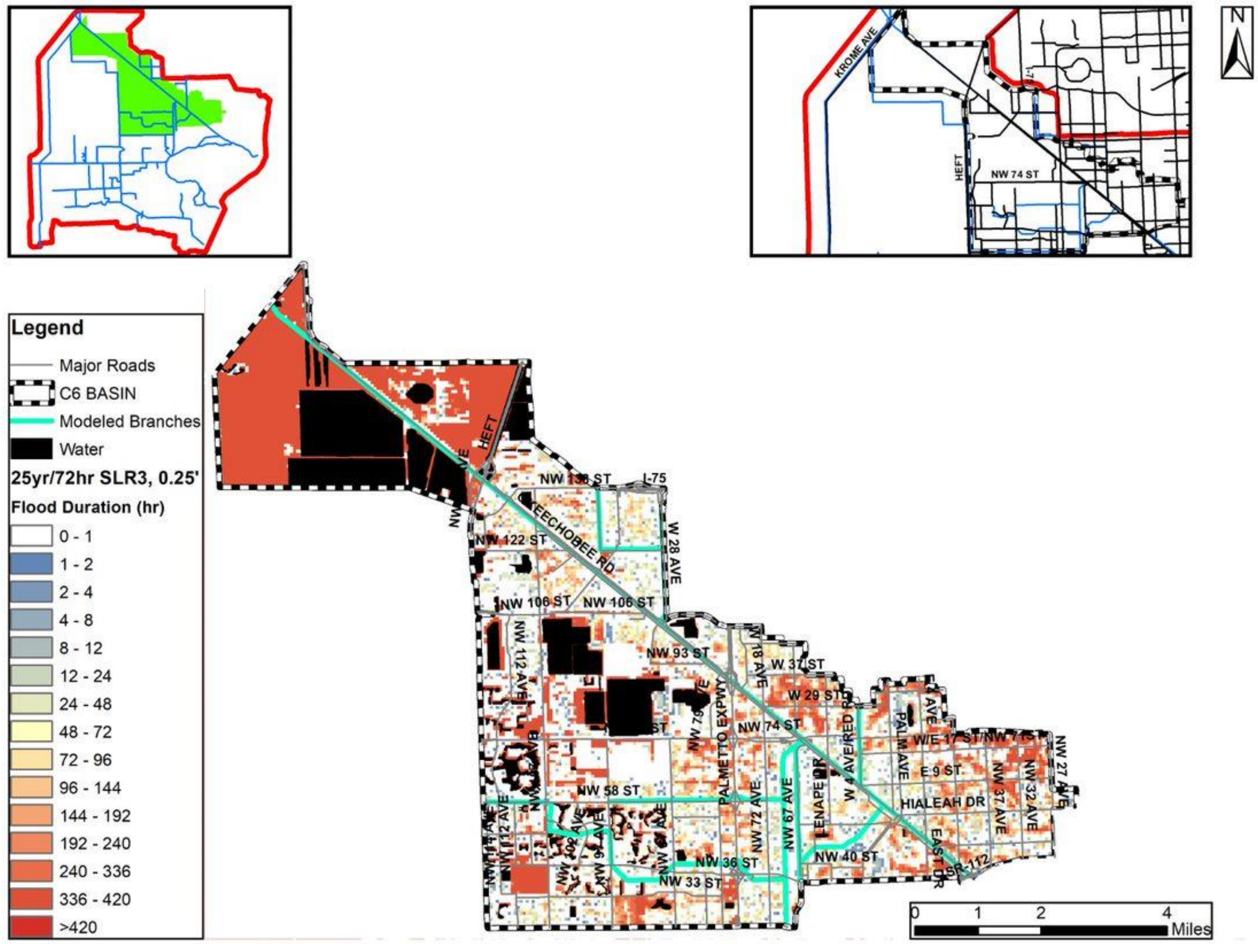


Figure D 5-13. Flood Duration Map for the 100-year 3-Day Design Storm for Current Conditions in the C6 Watershed

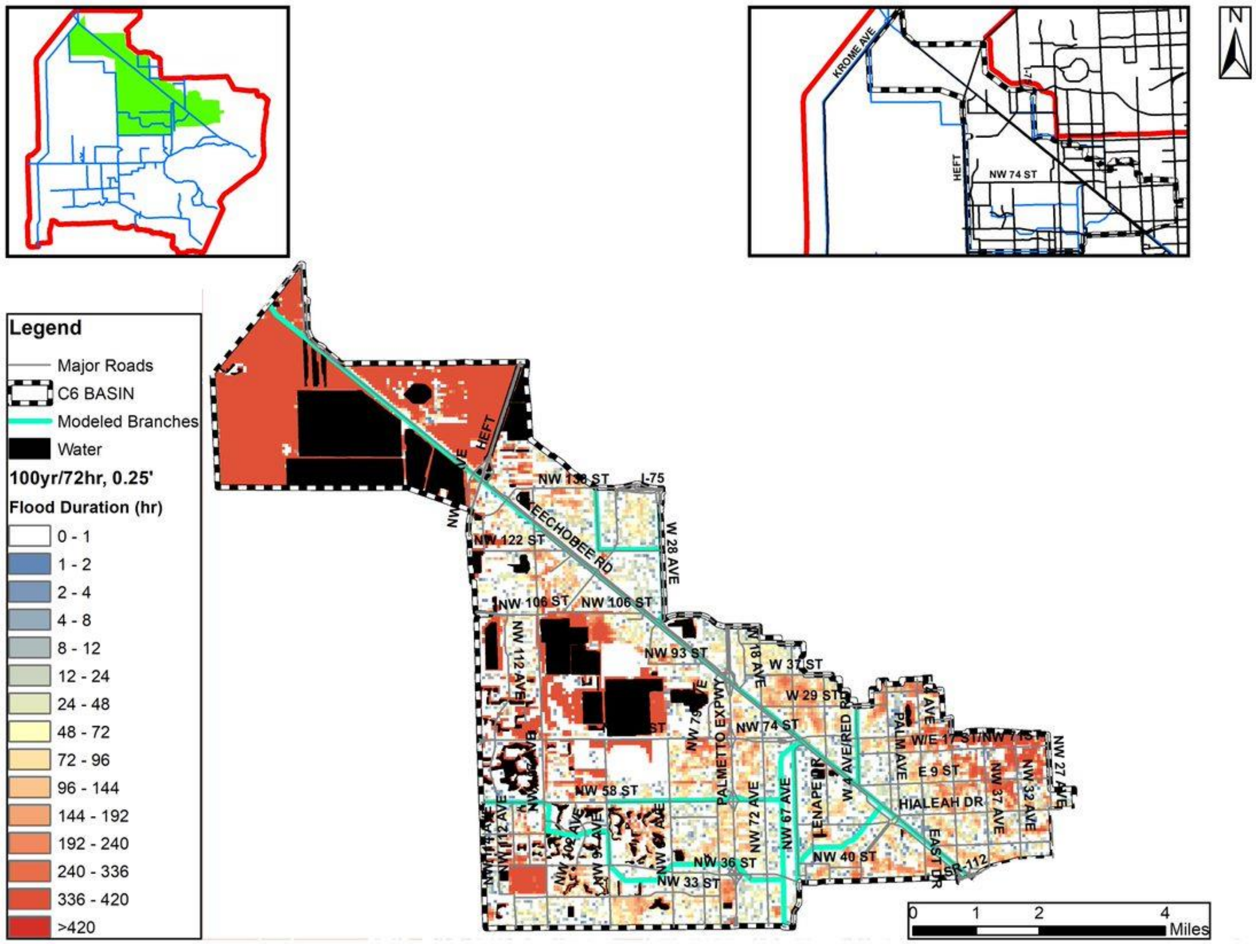


Figure D 5-14. Flood Duration Map for the 100-year 3-Day Design Storm for SLR1 in the C6 Watershed

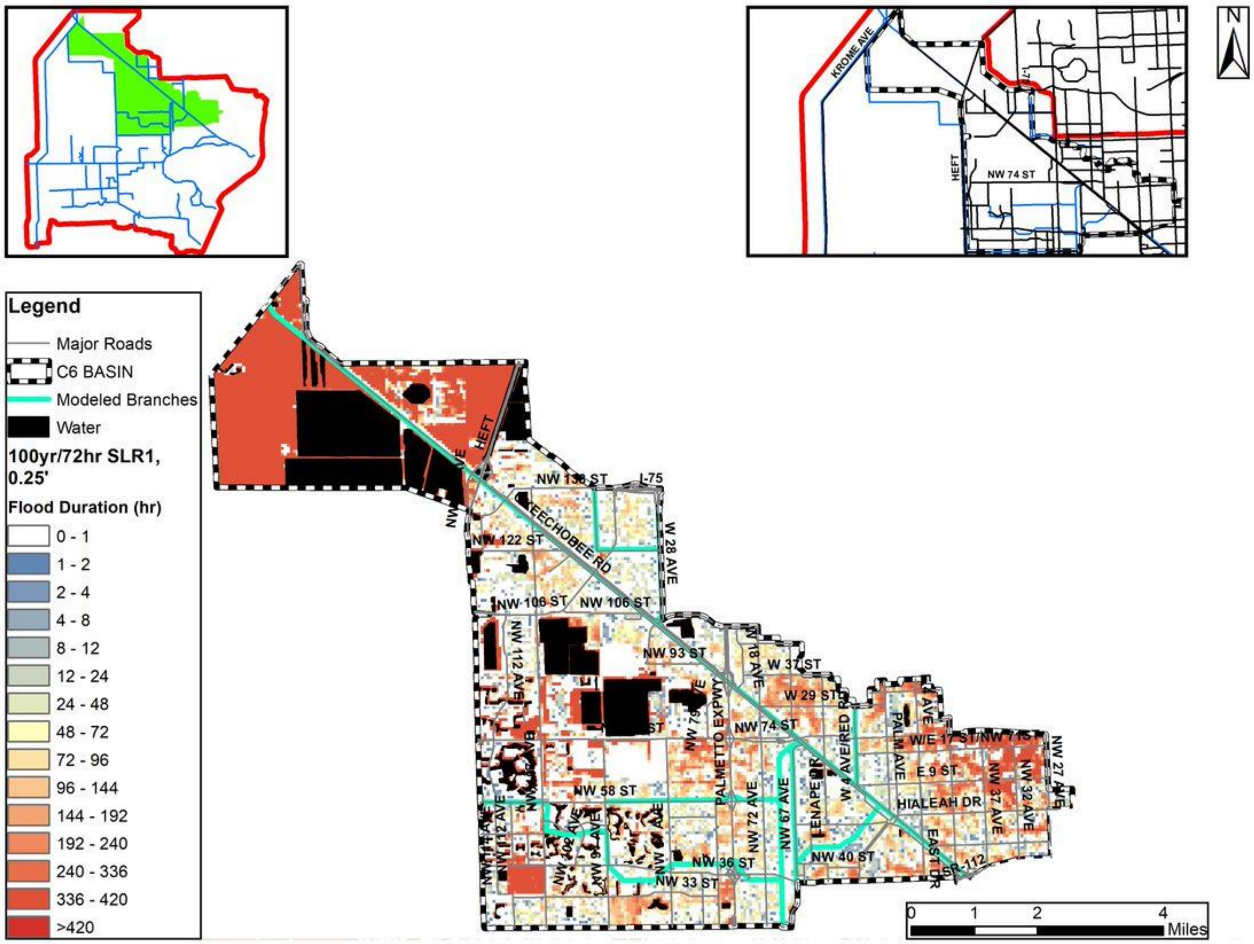


Figure D 5-15. Flood Duration Map for the 100-year 3-Day Design Storm for SLR2 in the C6 Watershed

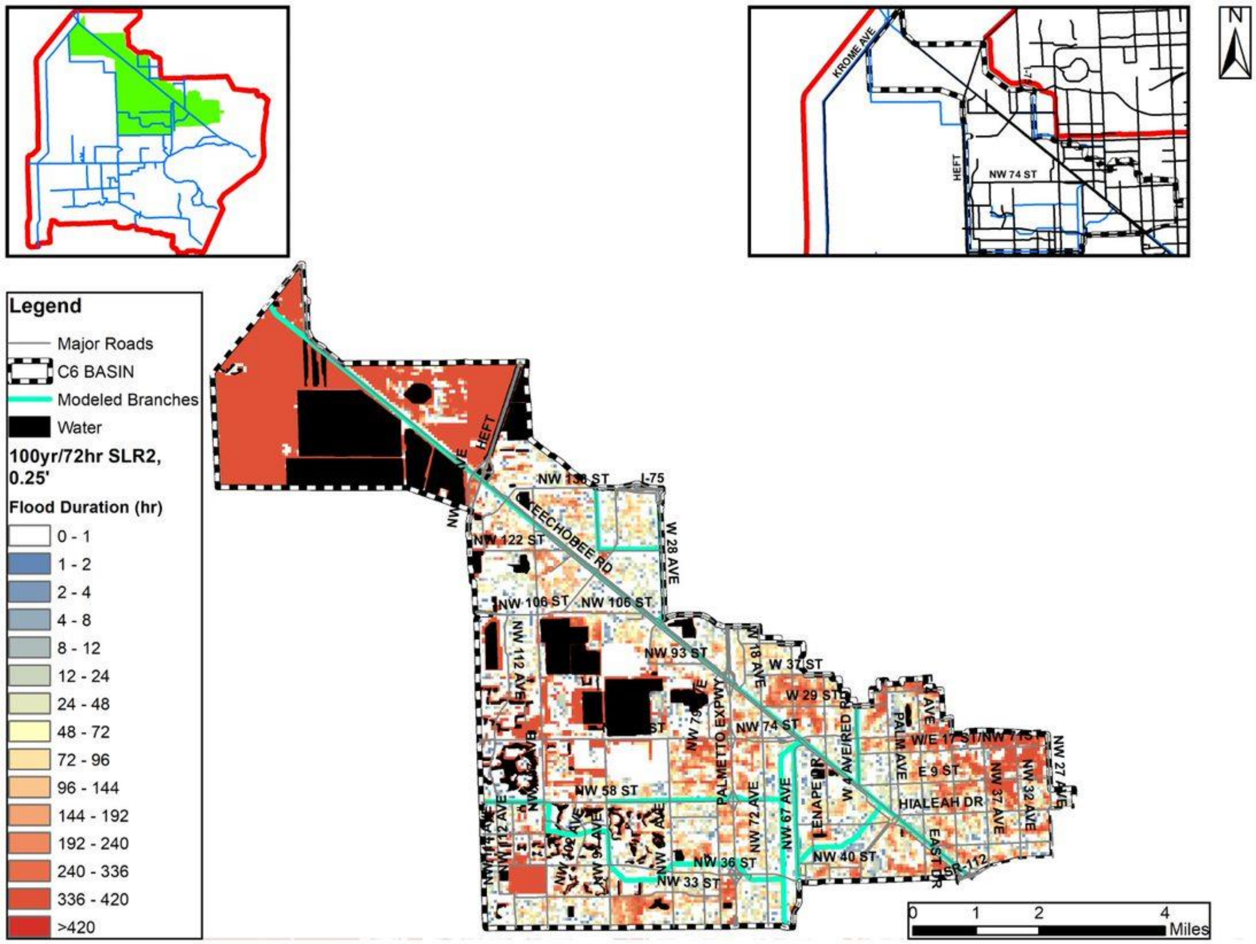


Figure D 5-16. Flood Duration Map for the 100-year 3-Day Design Storm for SLR3 in the C6 Watershed

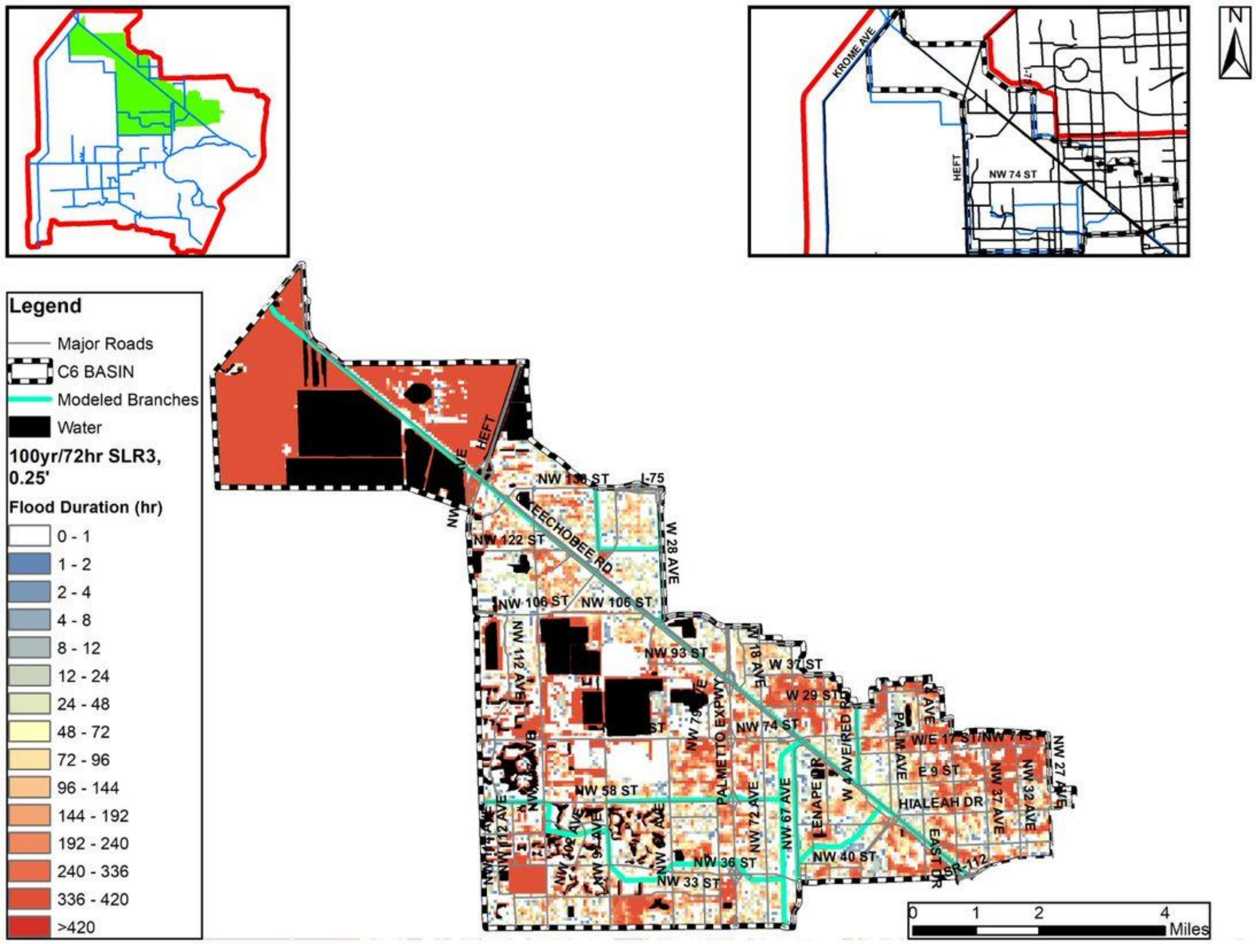


Figure D 5-17. Flood Duration Map for the 5-year 3-Day Design Storm for Current Conditions in Urban Areas in the C6 Watershed

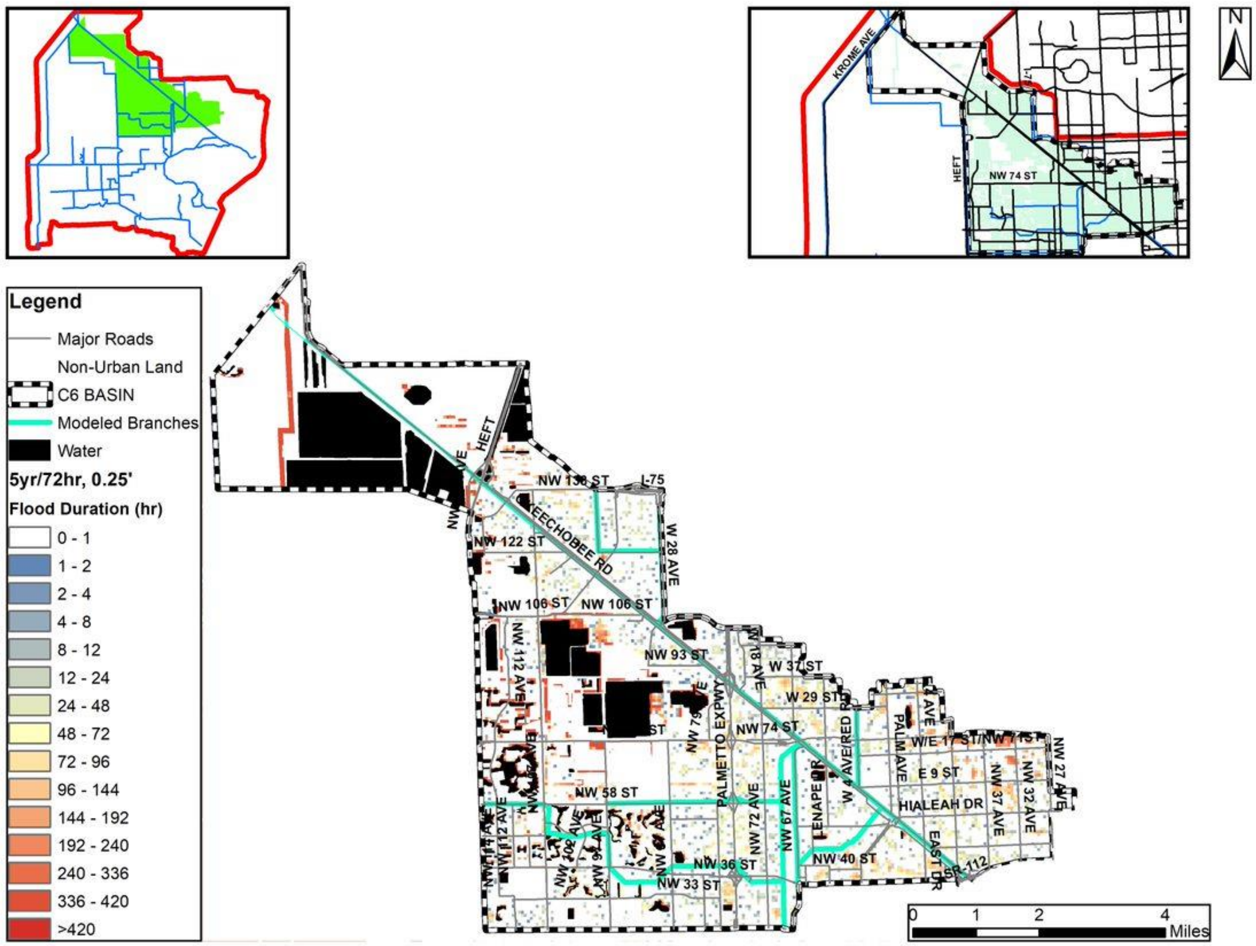


Figure D 5-18. Flood Duration Map for the 5-year 3-Day Design Storm for SLR1 in Urban Areas in the C6 Watershed

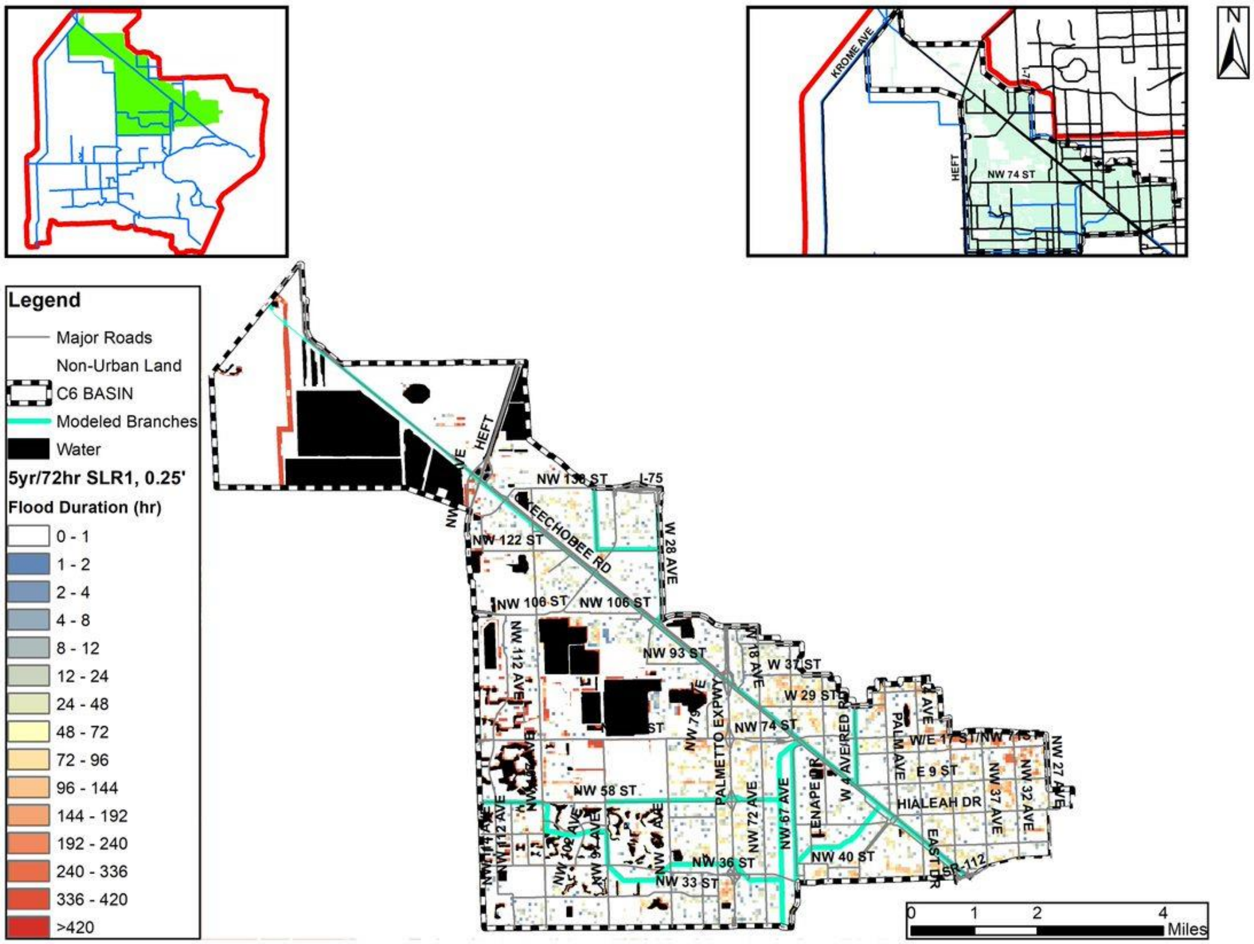


Figure D 5-19. Flood Duration Map for the 5-year 3-Day Design Storm for SLR2 in Urban Areas in the C6 Watershed

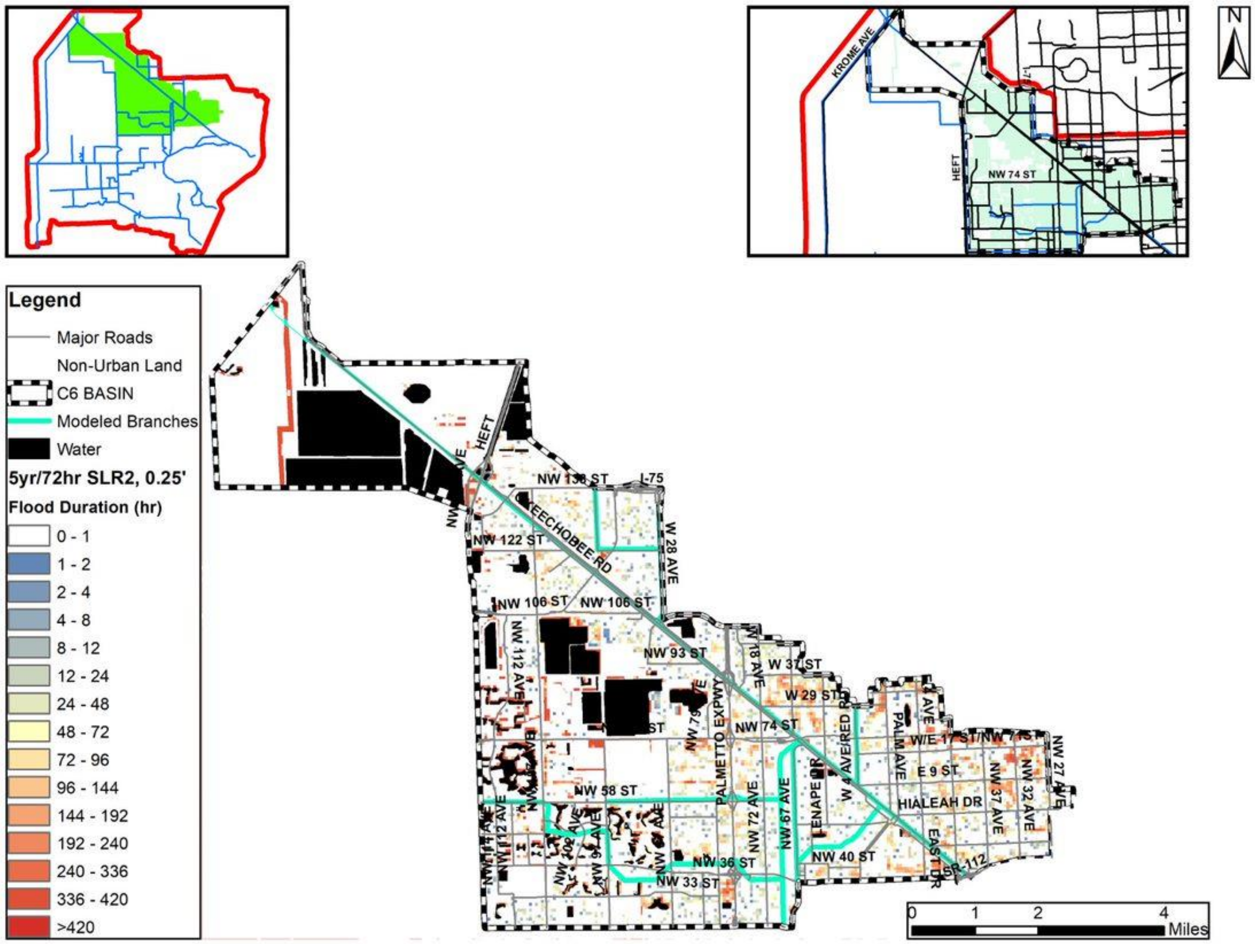


Figure D 5-20. Flood Duration Map for the 5-year 3-Day Design Storm for SLR3 in Urban Areas in the C6 Watershed

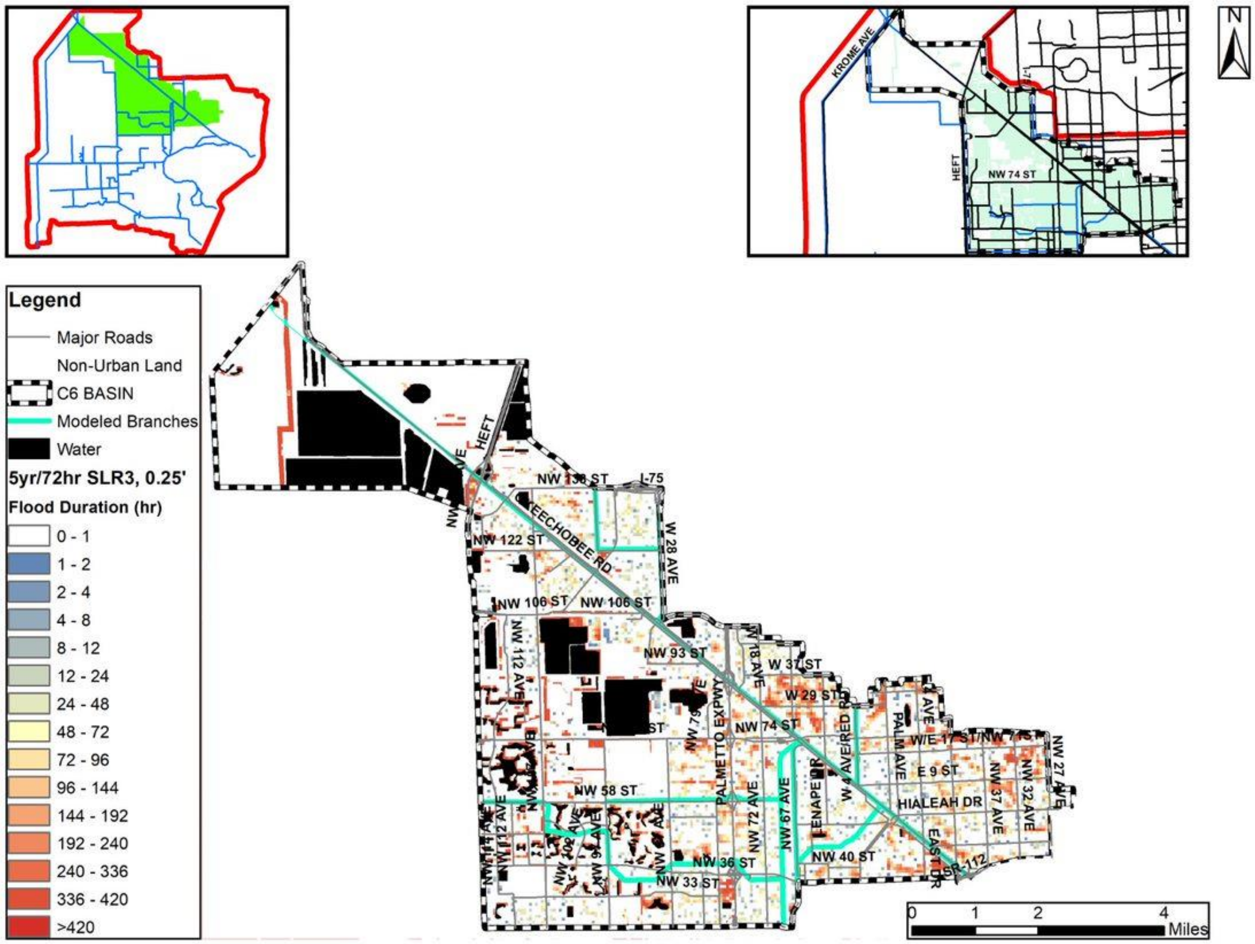


Figure D 5-21. Flood Duration Map for the 10-year 3-Day Design Storm for Current Conditions in Urban Areas in the C6 Watershed

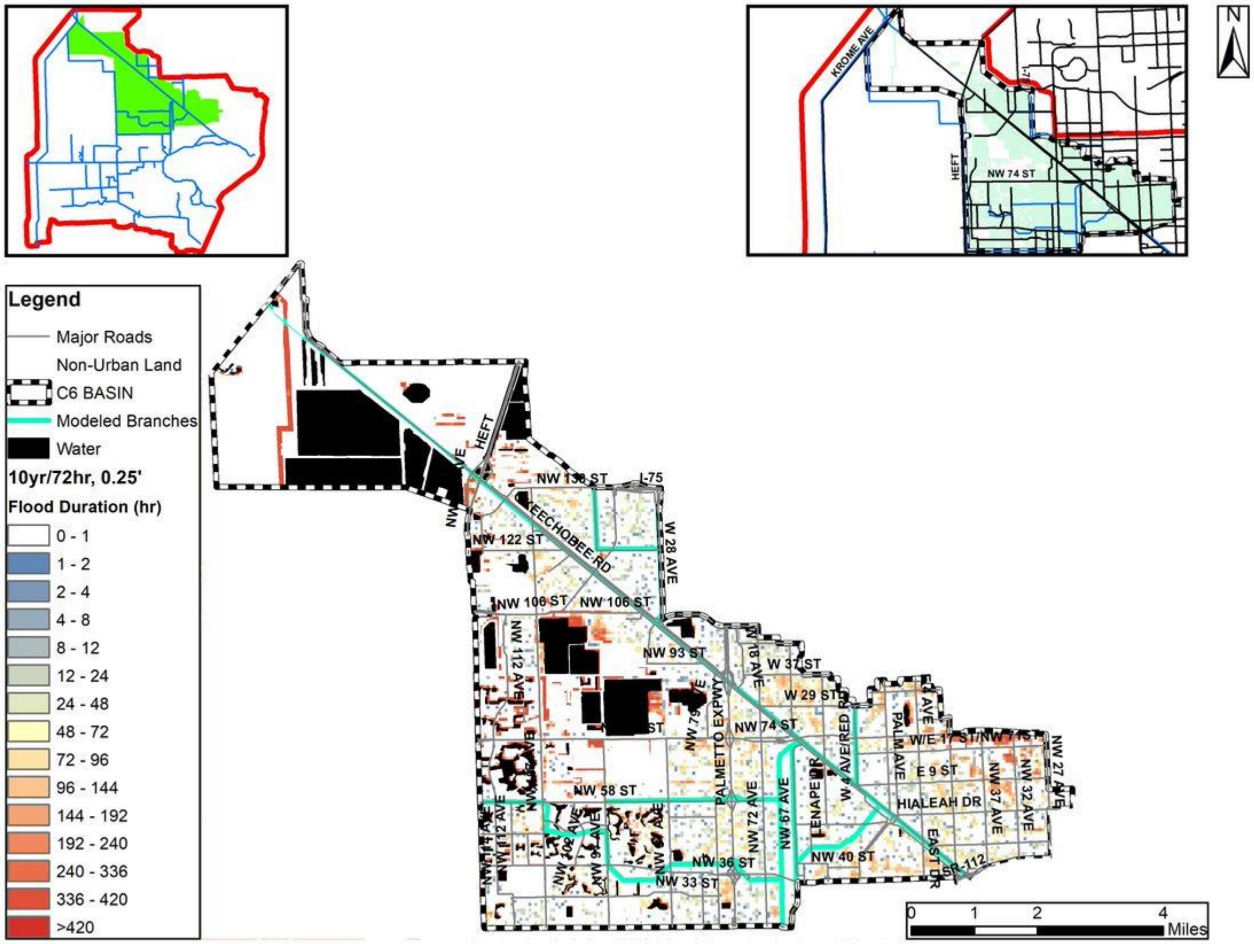


Figure D 5-22. Flood Duration Map for the 10-year 3-Day Design Storm for SLR1 in Urban Areas in the C6 Watershed

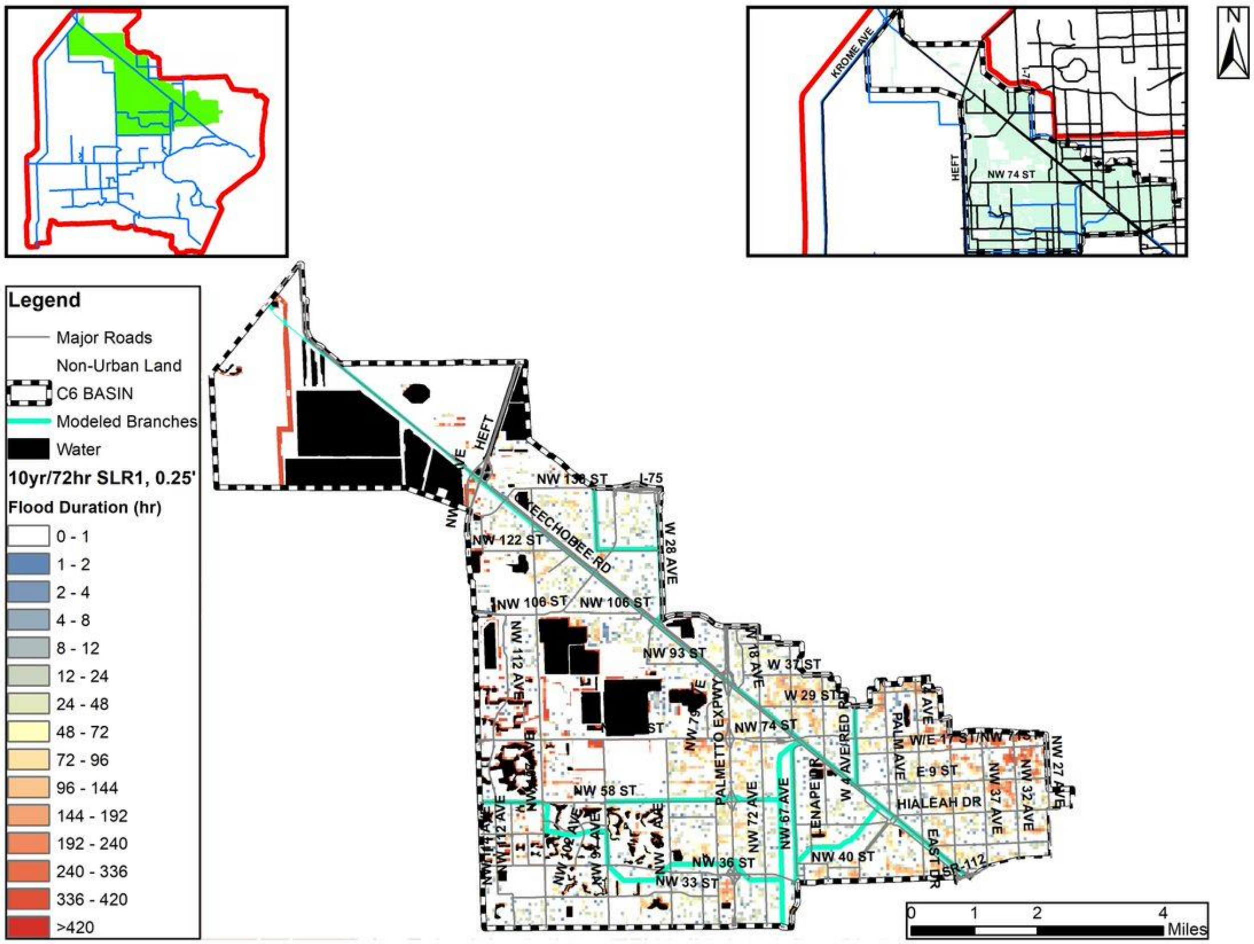


Figure D 5-23. Flood Duration Map for the 10-year 3-Day Design Storm for SLR2 in Urban Areas in the C6 Watershed

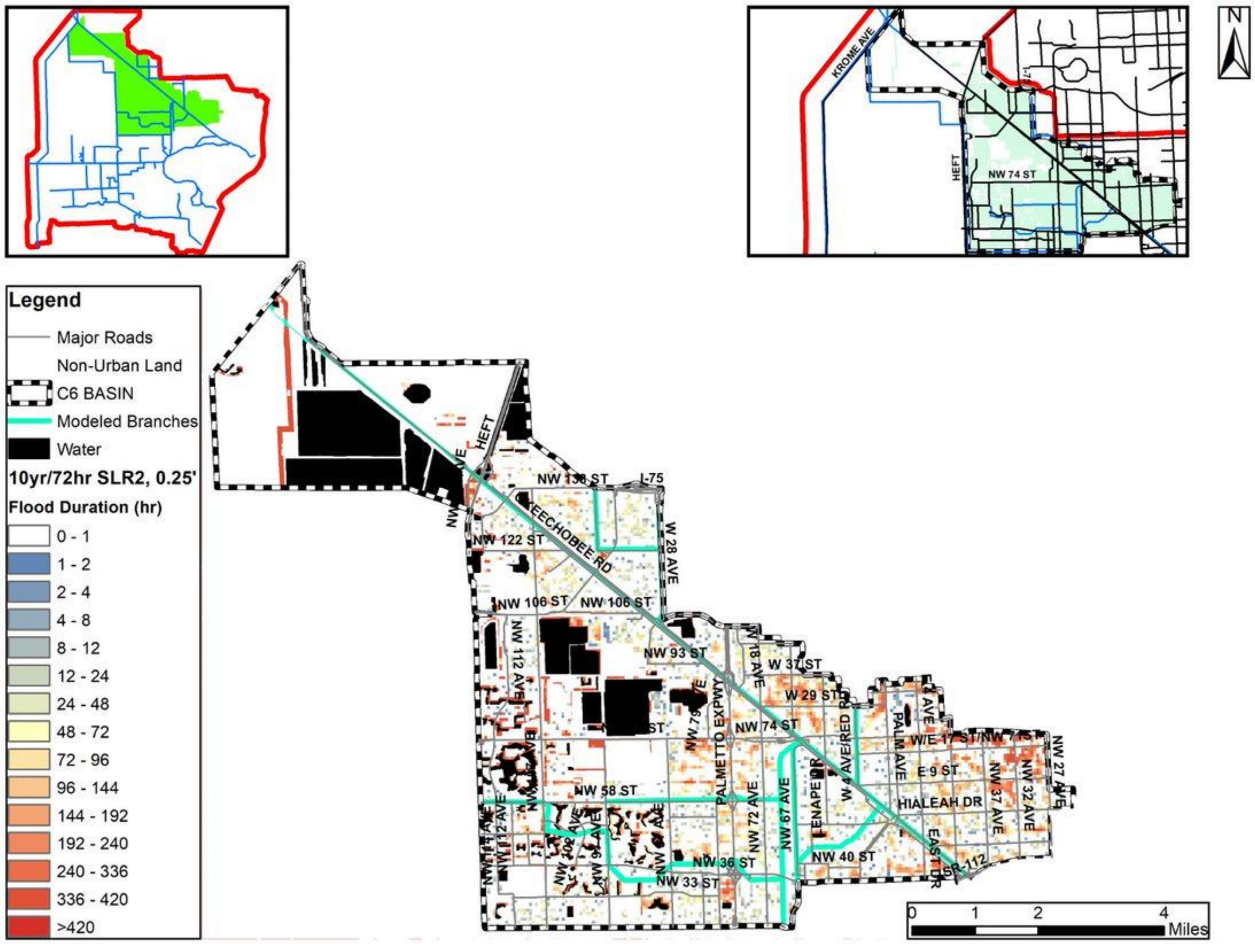


Figure D 5-24. Flood Duration Map for the 10-year 3-Day Design Storm for SLR3 in Urban Areas in the C6 Watershed

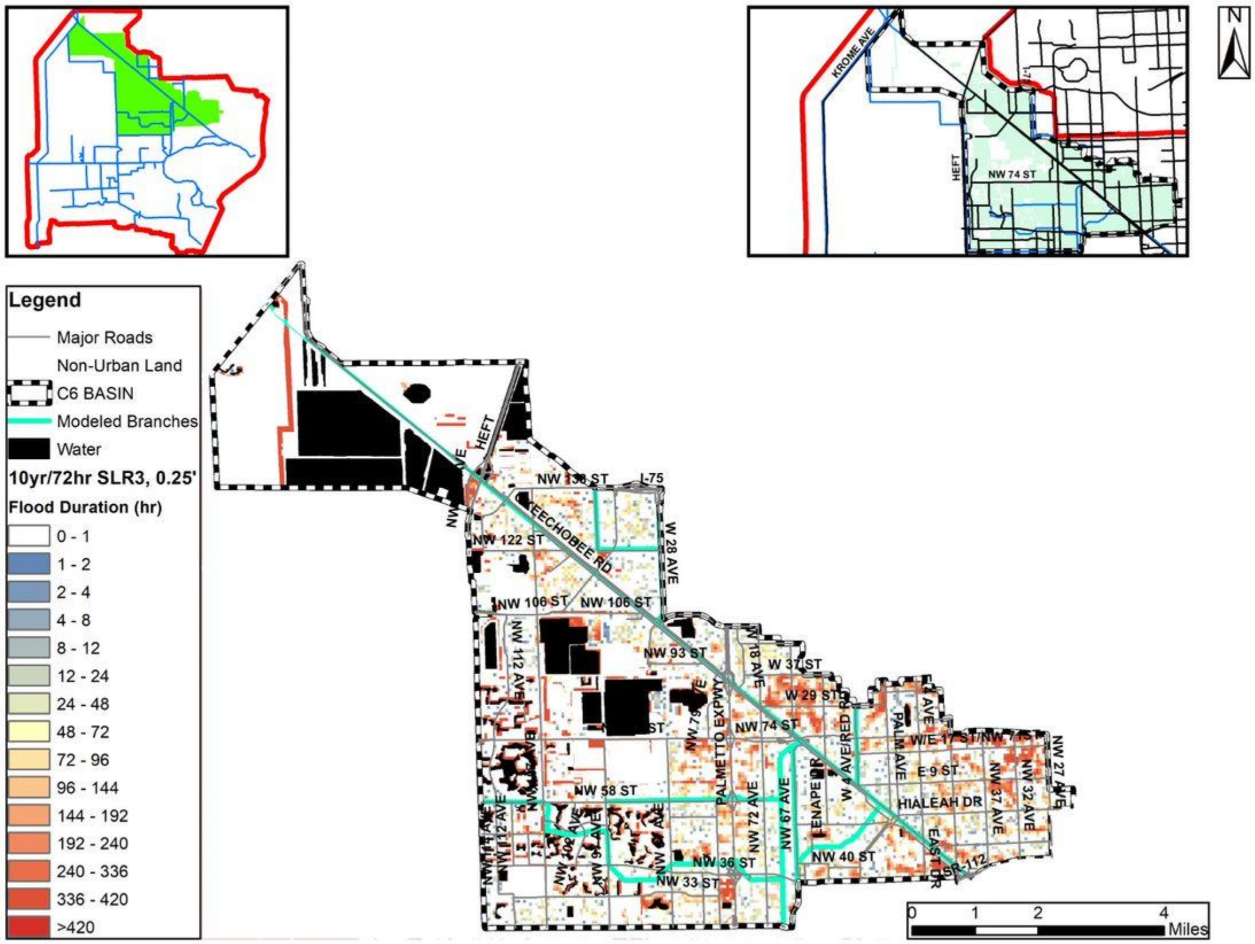


Figure D 5-25. Flood Duration Map for the 25-year 3-Day Design Storm for Current Conditions in Urban Areas in the C6 Watershed

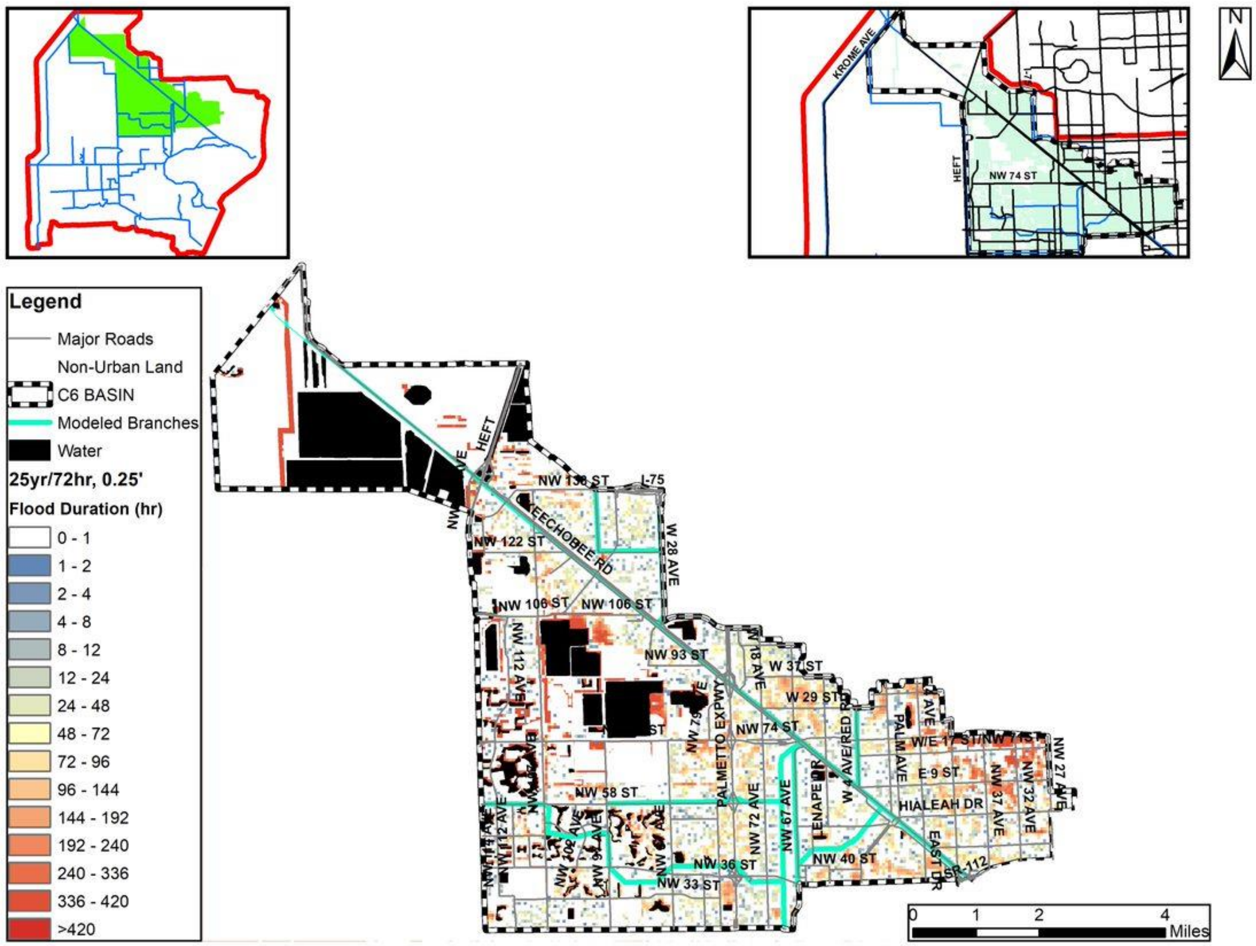


Figure D 5-26. Flood Duration Map for the 25-year 3-Day Design Storm for SLR1 in Urban Areas in the C6 Watershed

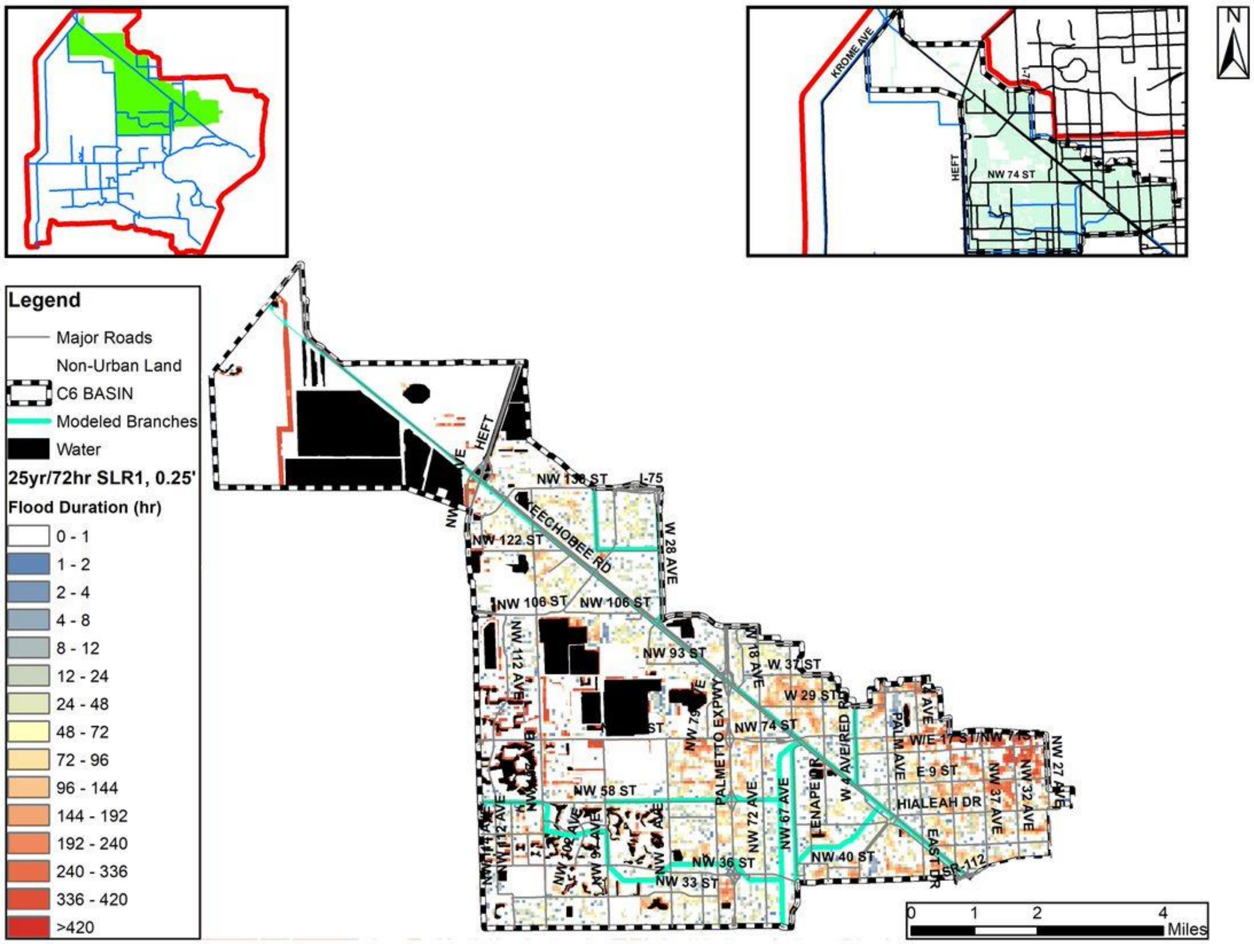


Figure D 5-27. Flood Duration Map for the 25-year 3-Day Design Storm for SLR2 in Urban Areas in the C6 Watershed

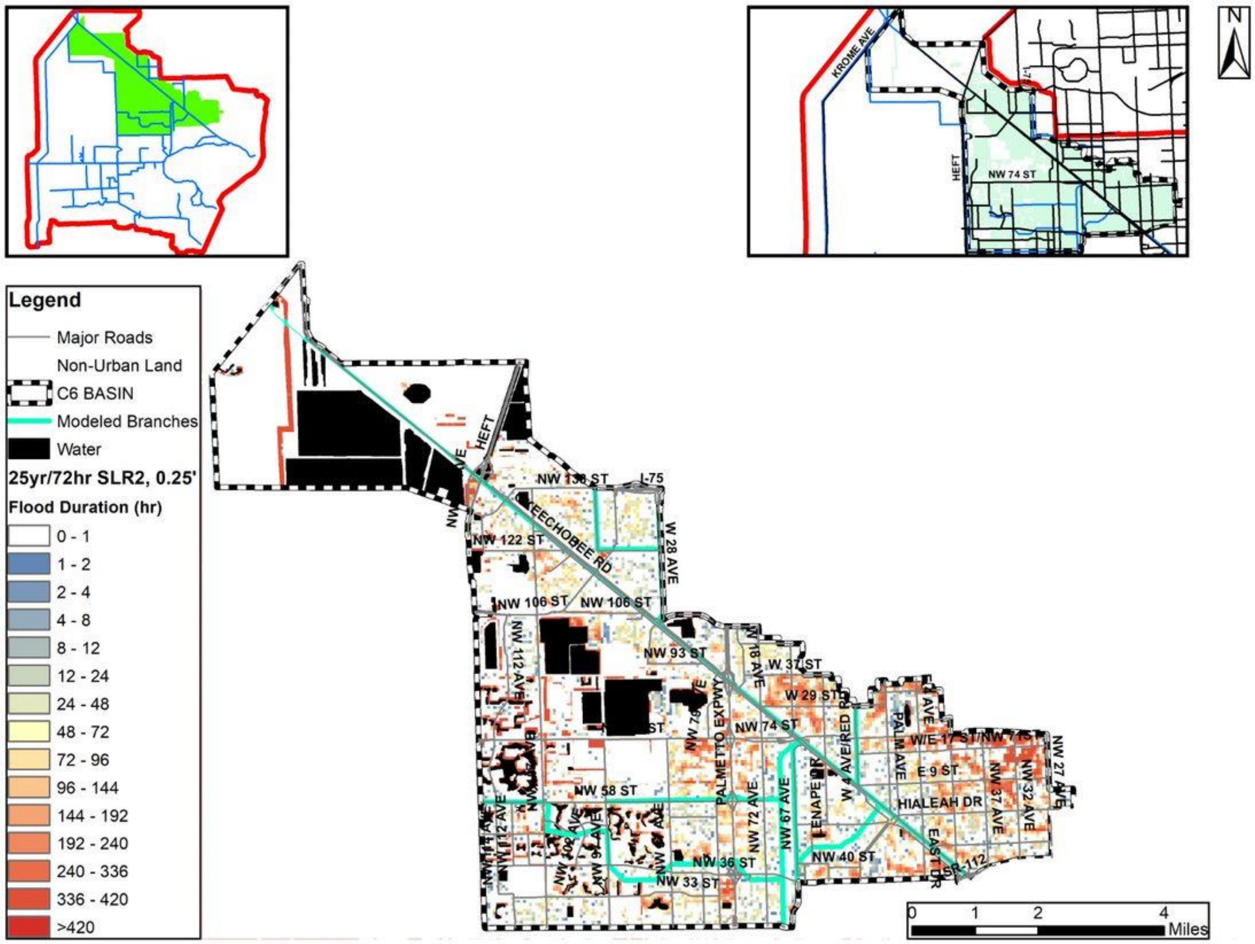


Figure D 5-28. Flood Duration Map for the 25-year 3-Day Design Storm for SLR3 in Urban Areas in the C6 Watershed

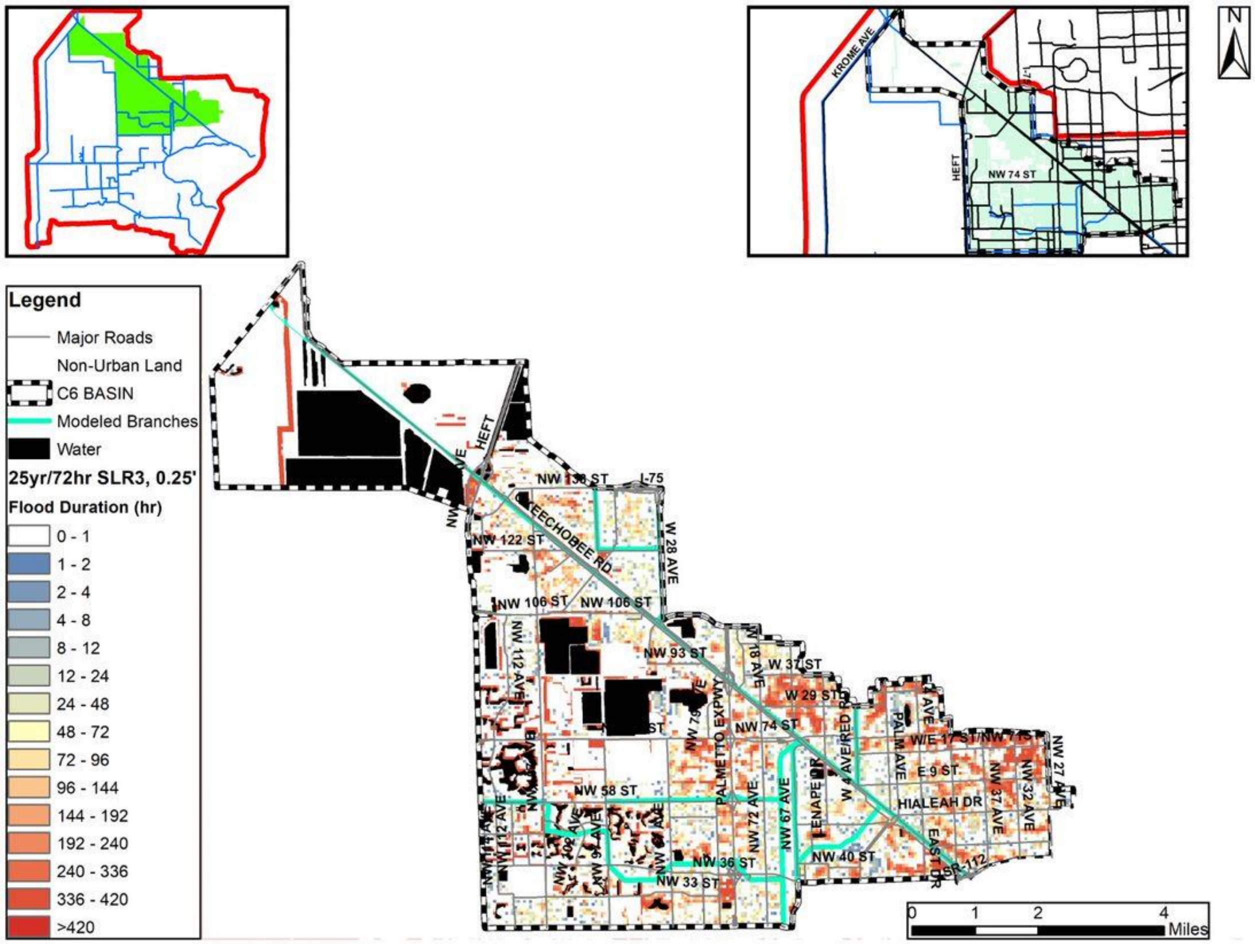


Figure D 5-29. Flood Duration Map for the 100-year 3-Day Design Storm for Current Conditions in Urban Areas in the C6 Watershed

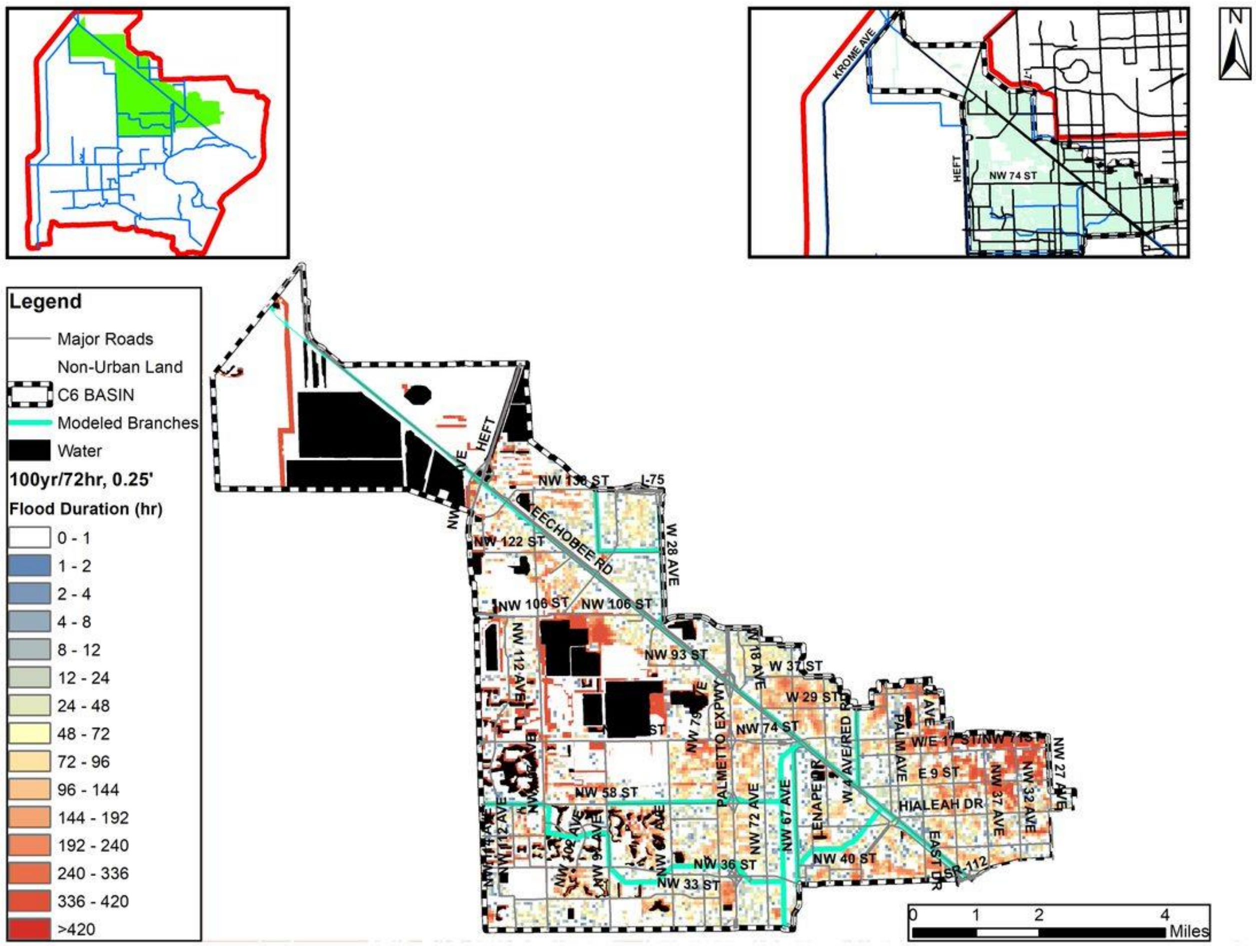


Figure D 5-30. Flood Duration Map for the 100-year 3-Day Design Storm for SLR1 in Urban Areas in the C6 Watershed

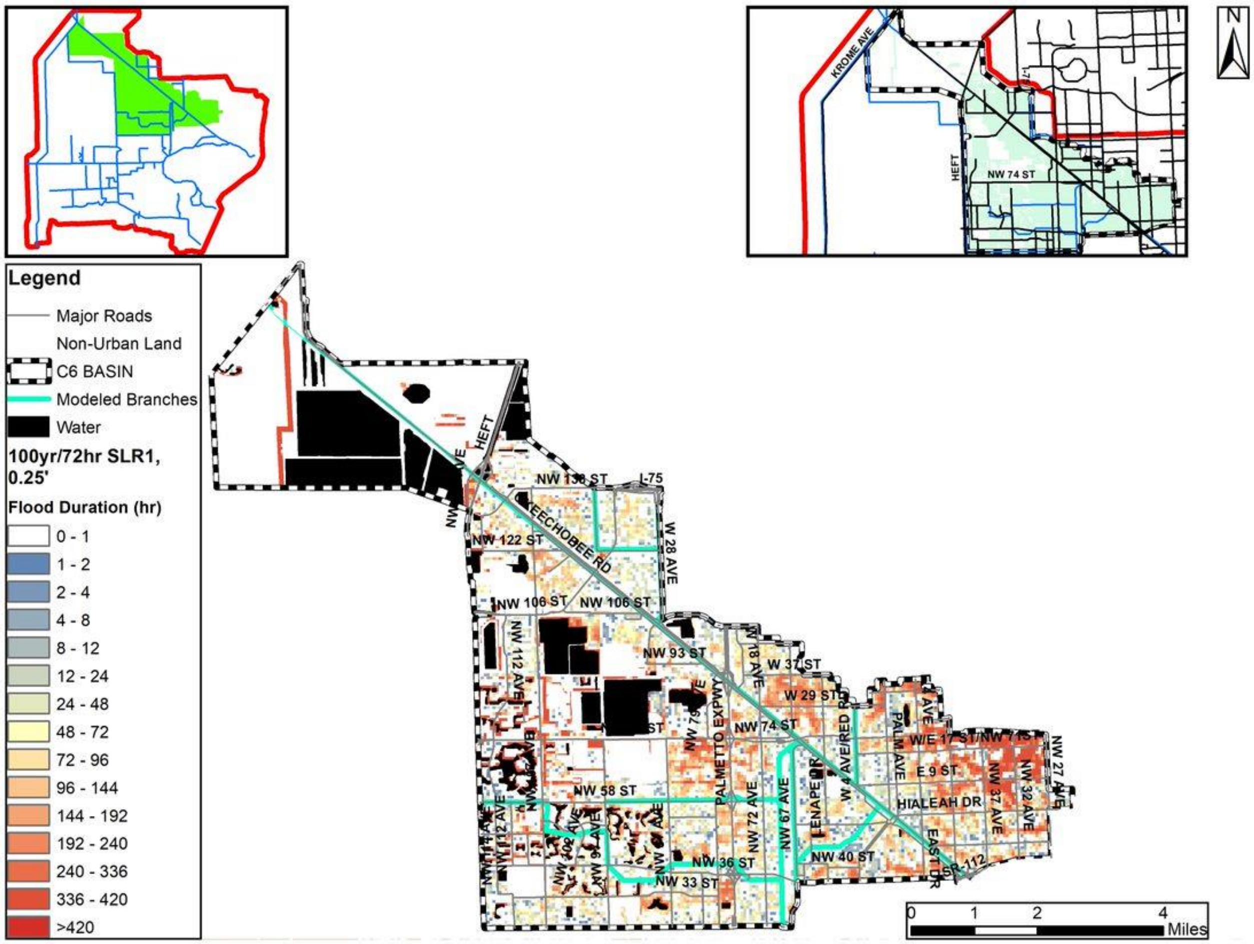


Figure D 5-31. Flood Duration Map for the 100-year 3-Day Design Storm for SLR2 in Urban Areas in the C6 Watershed

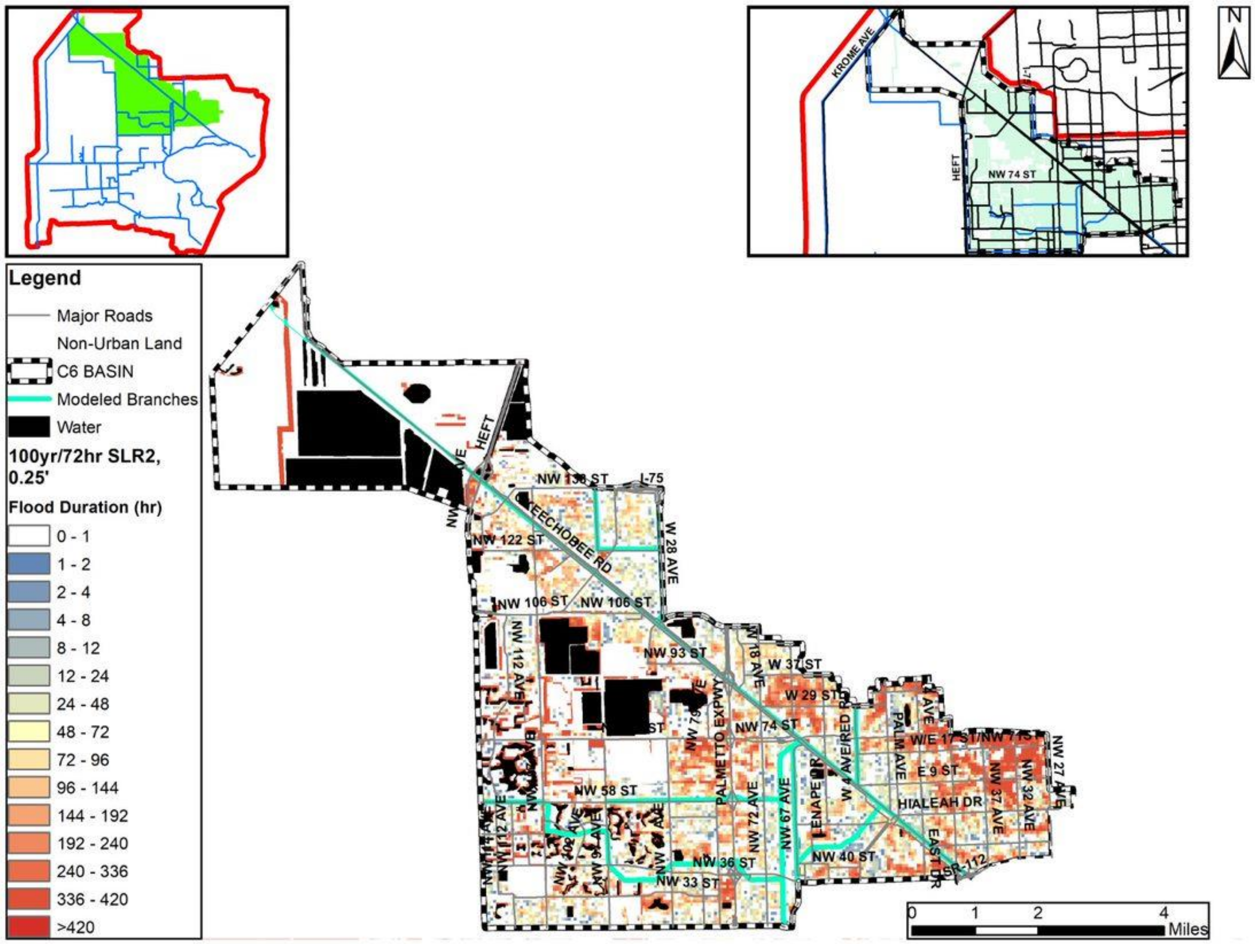


Figure D 5-32. Flood Duration Map for the 100-year 3-Day Design Storm for SLR3 in Urban Areas in the C6 Watershed

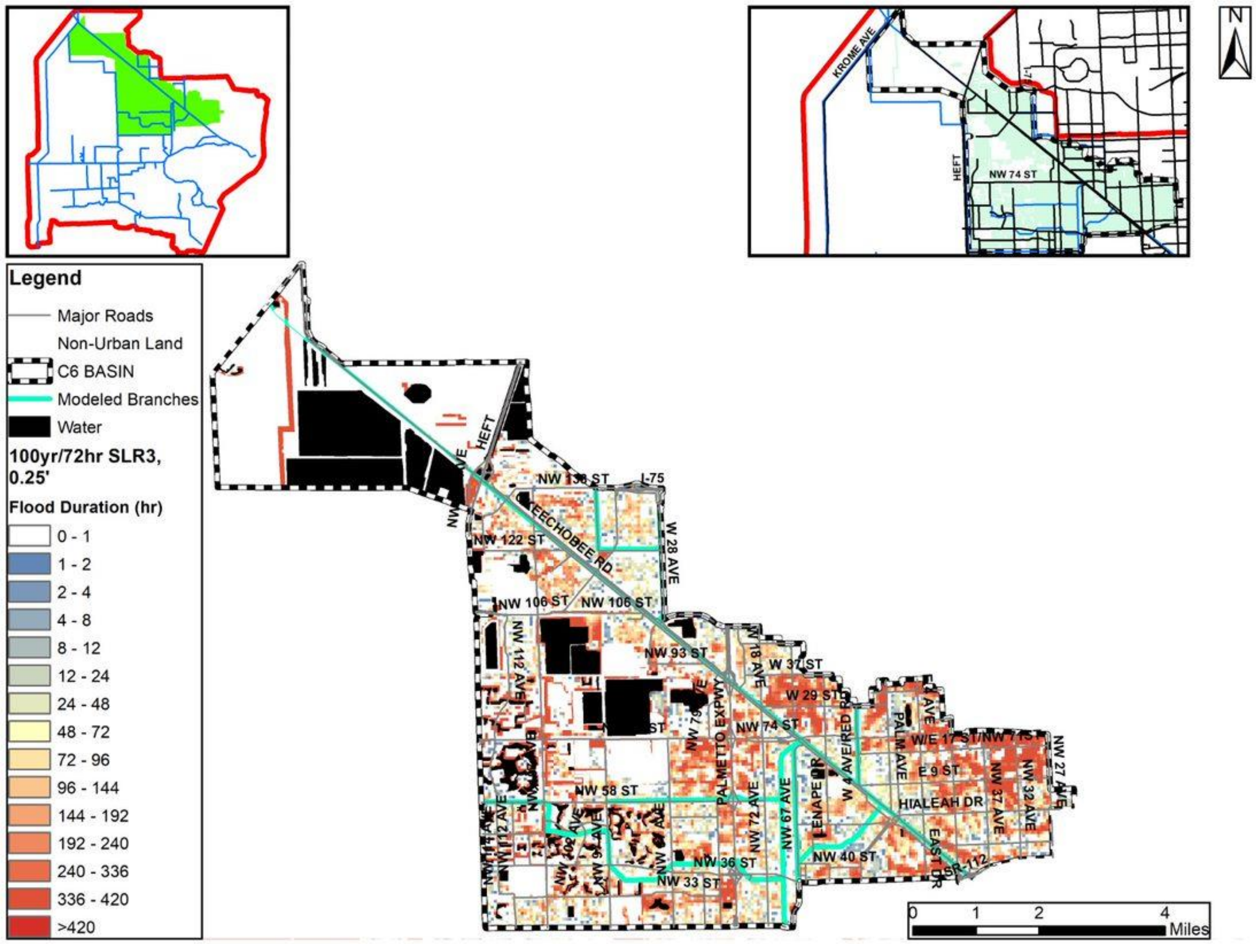


Figure D 5-33. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in the C6 Watershed

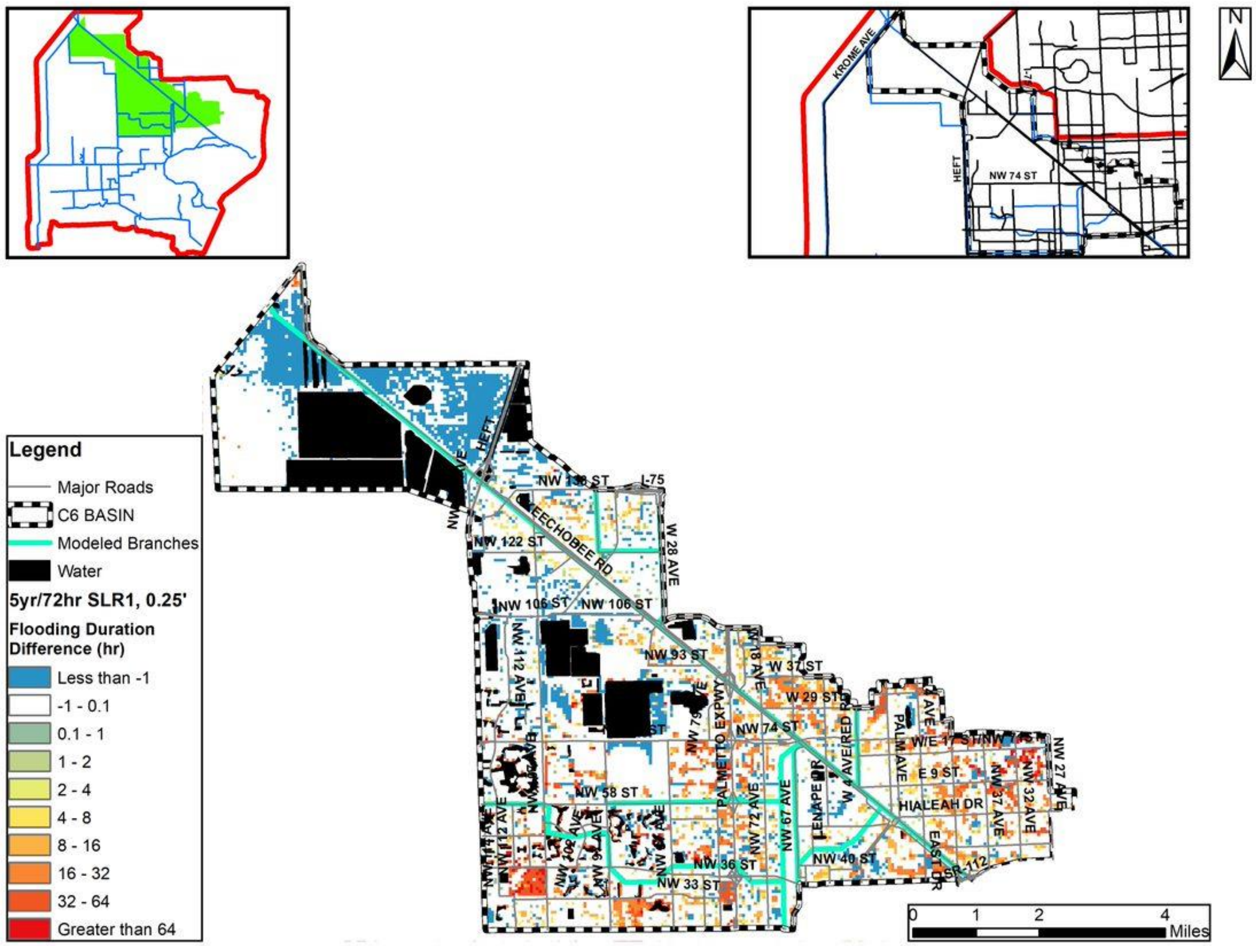


Figure D 5-34. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in the C6 Watershed

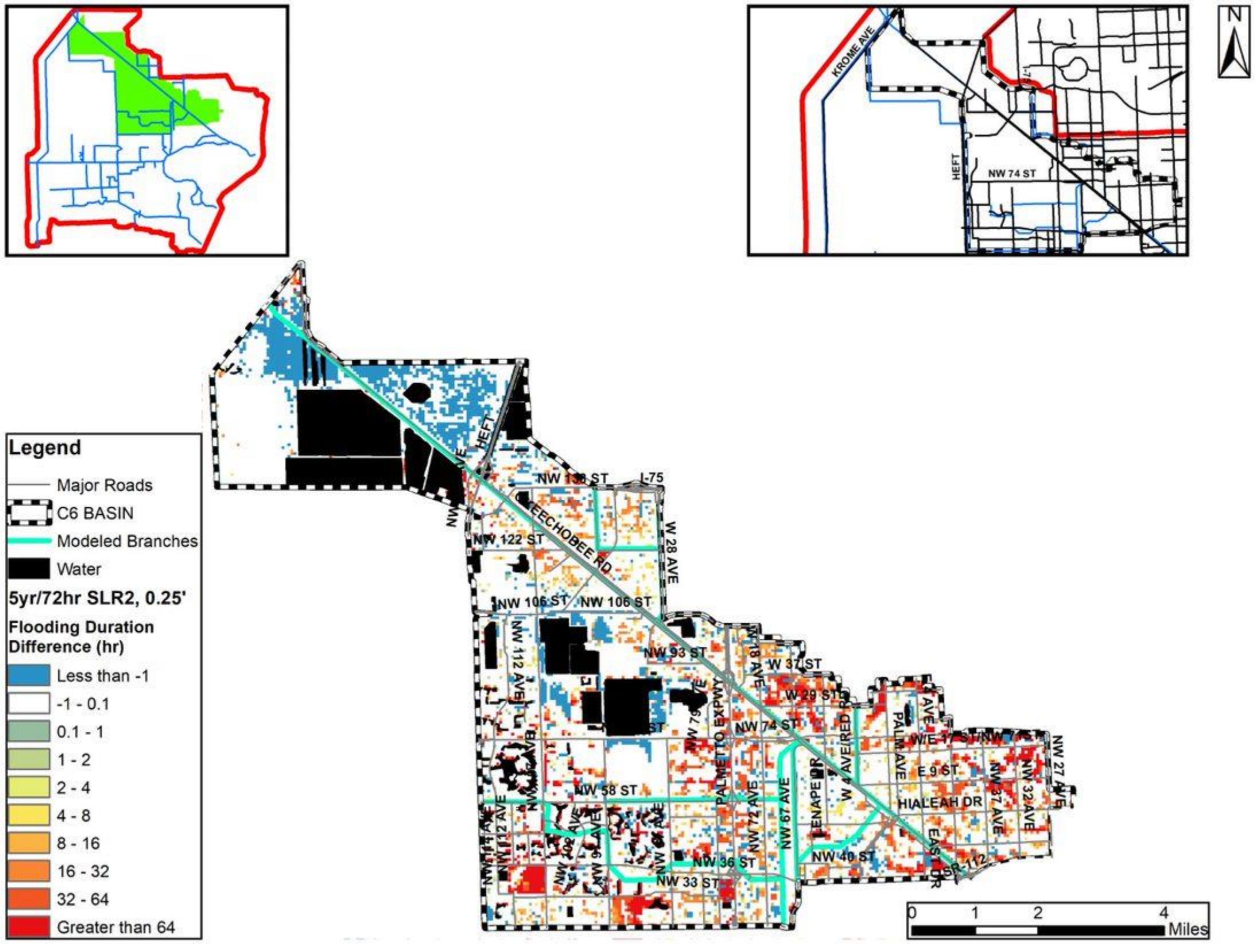


Figure D 5-35. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in the C6 Watershed

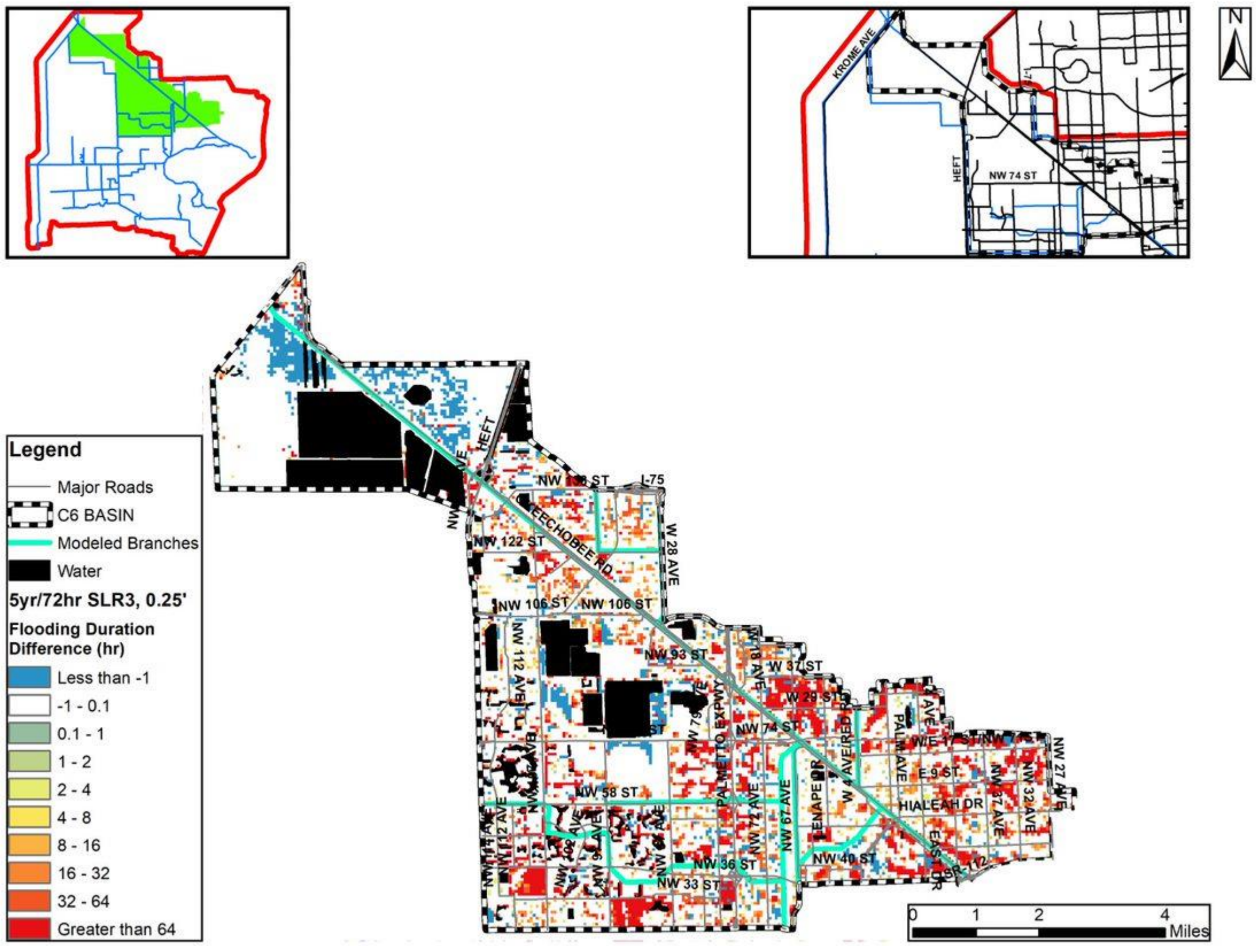


Figure D 5-36. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in the C6 Watershed

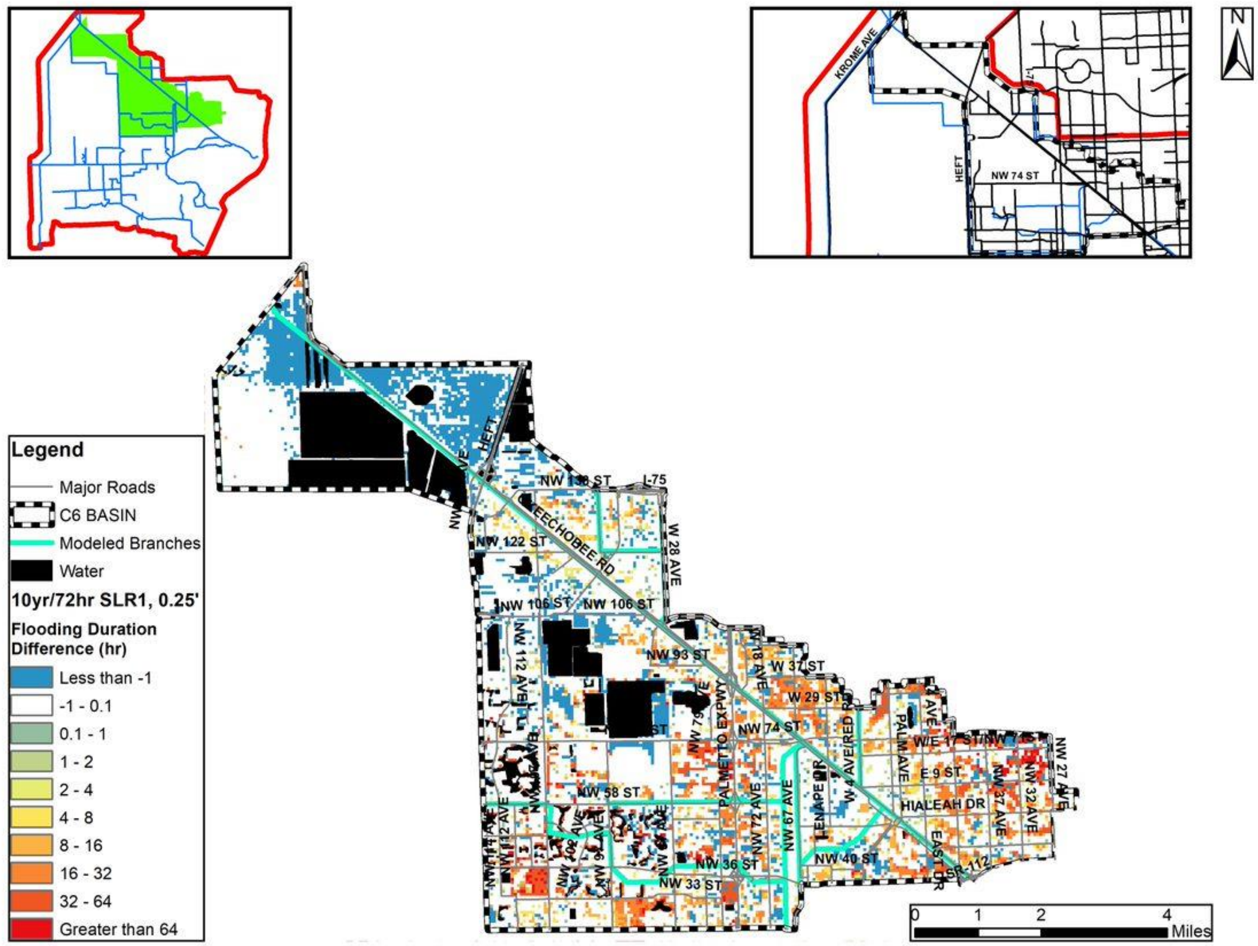


Figure D 5-37. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in the C6 Watershed

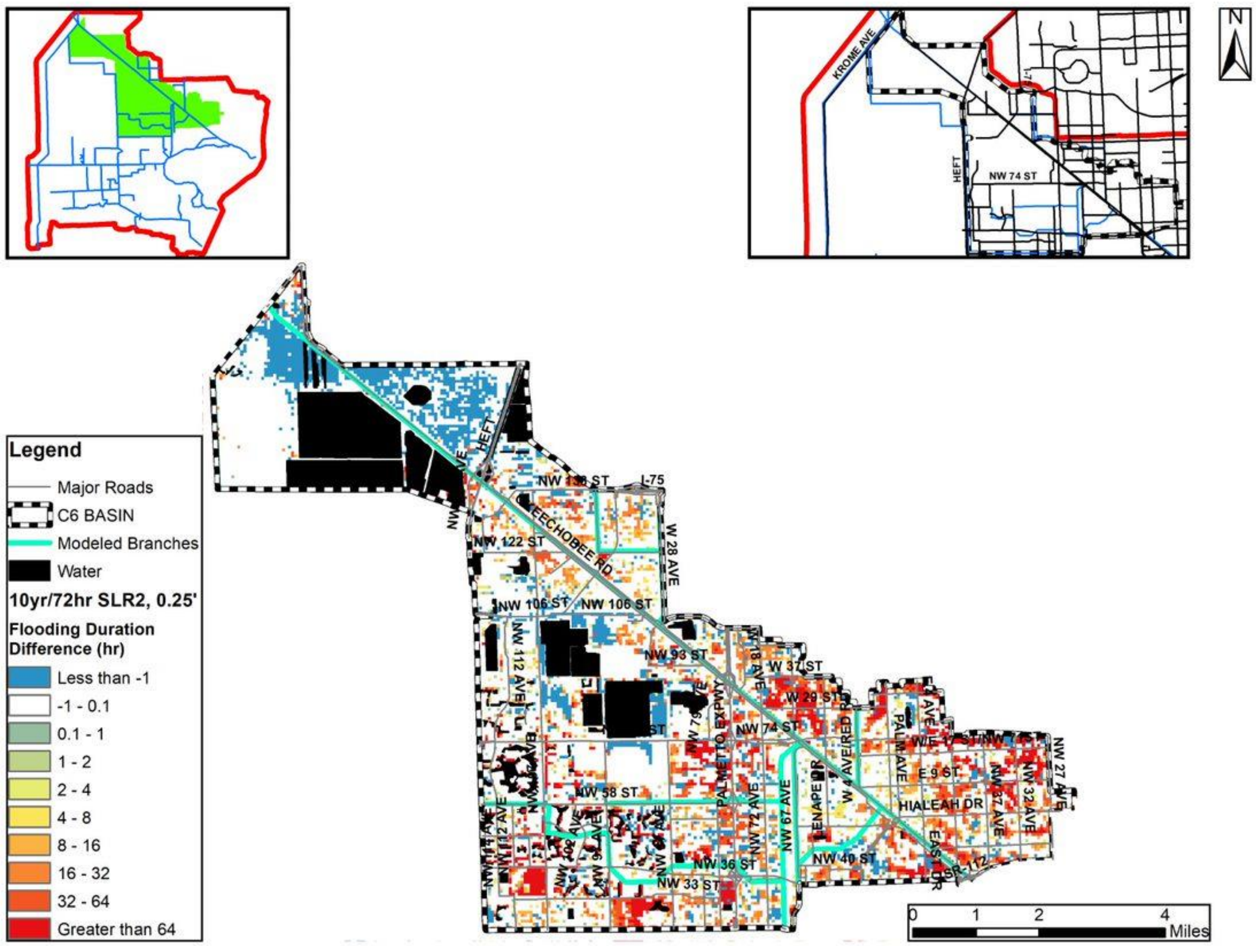


Figure D 5-38. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in the C6 Watershed

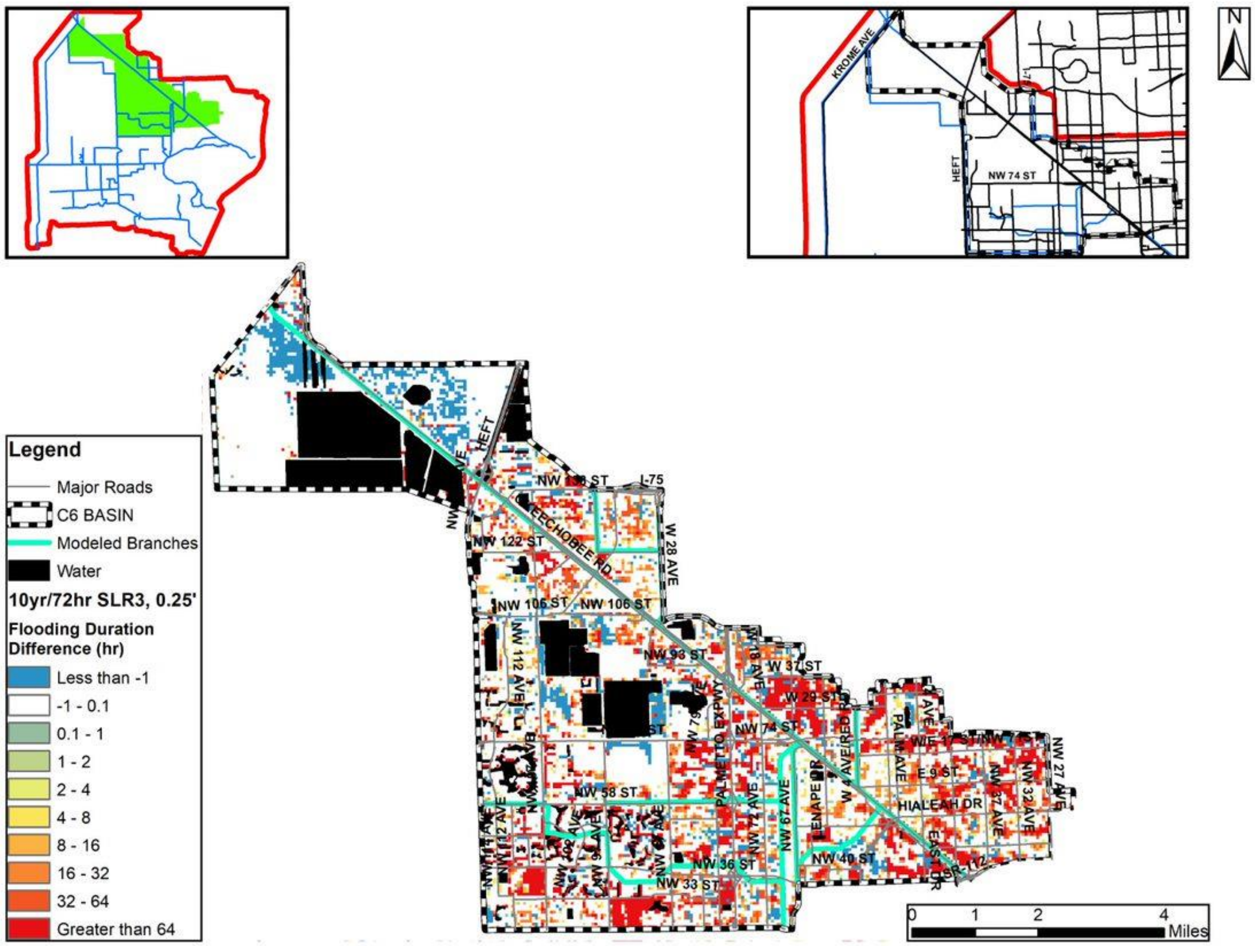


Figure D 5-39. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in the C6 Watershed

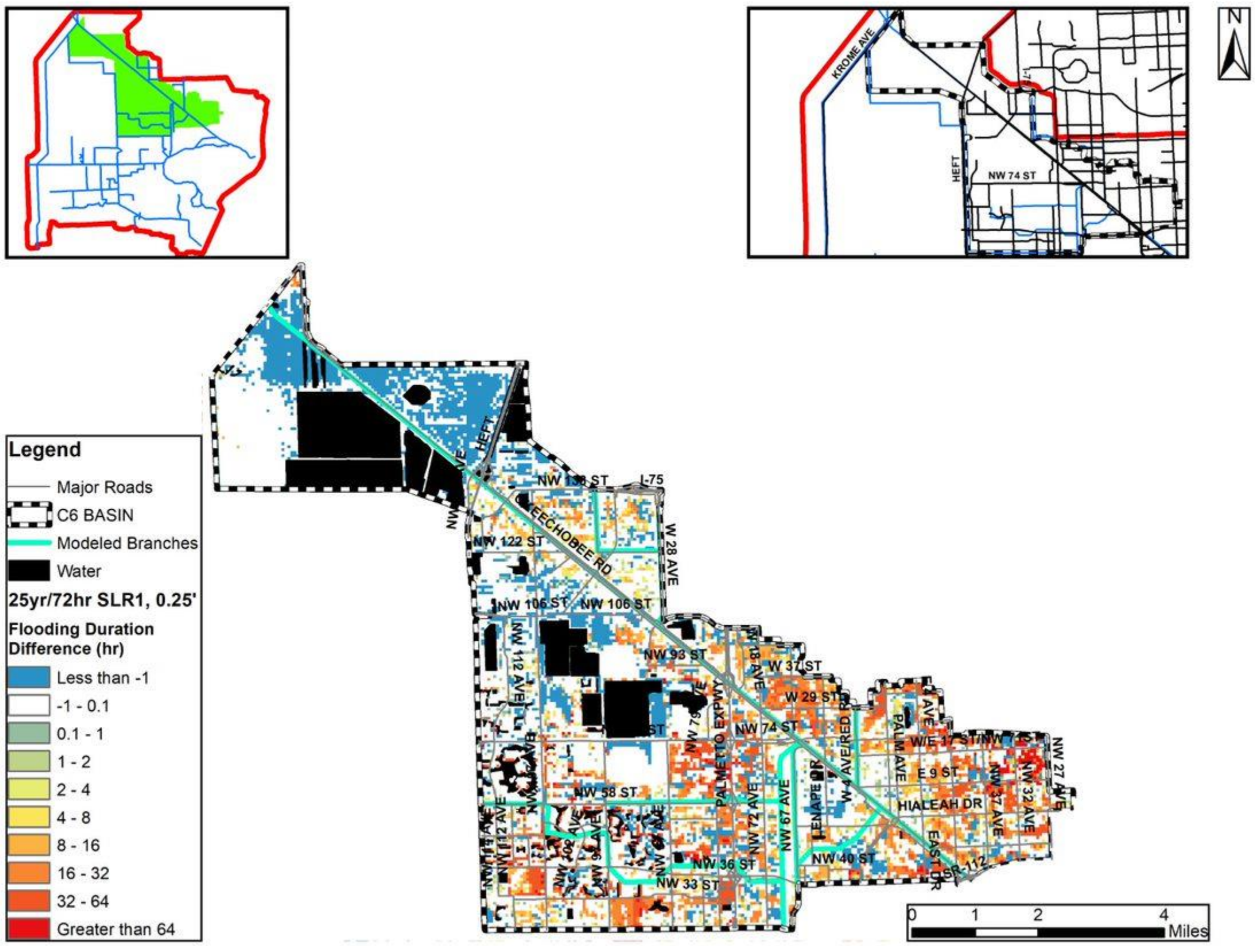


Figure D 5-40. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in the C6 Watershed

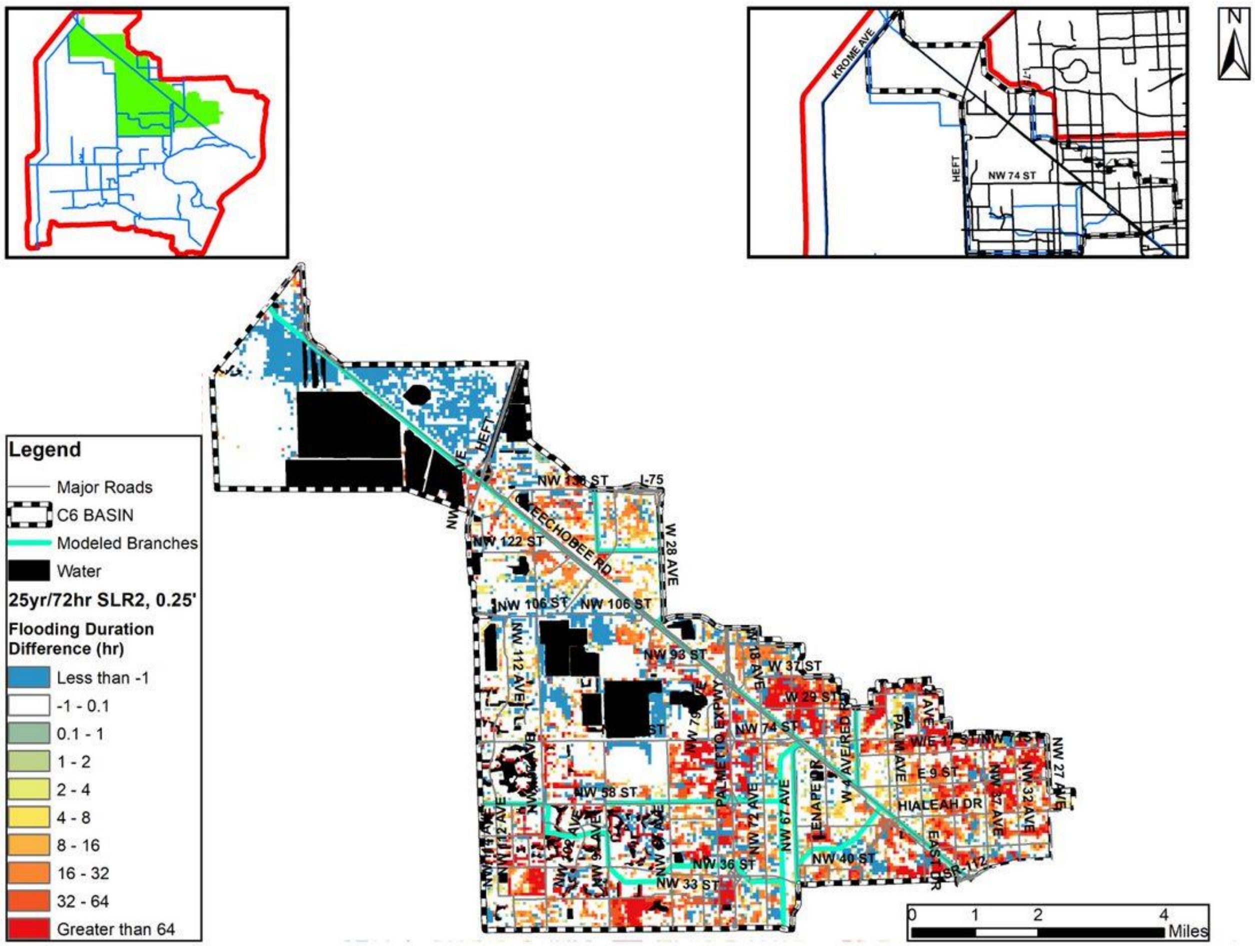


Figure D 5-41. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in the C6 Watershed

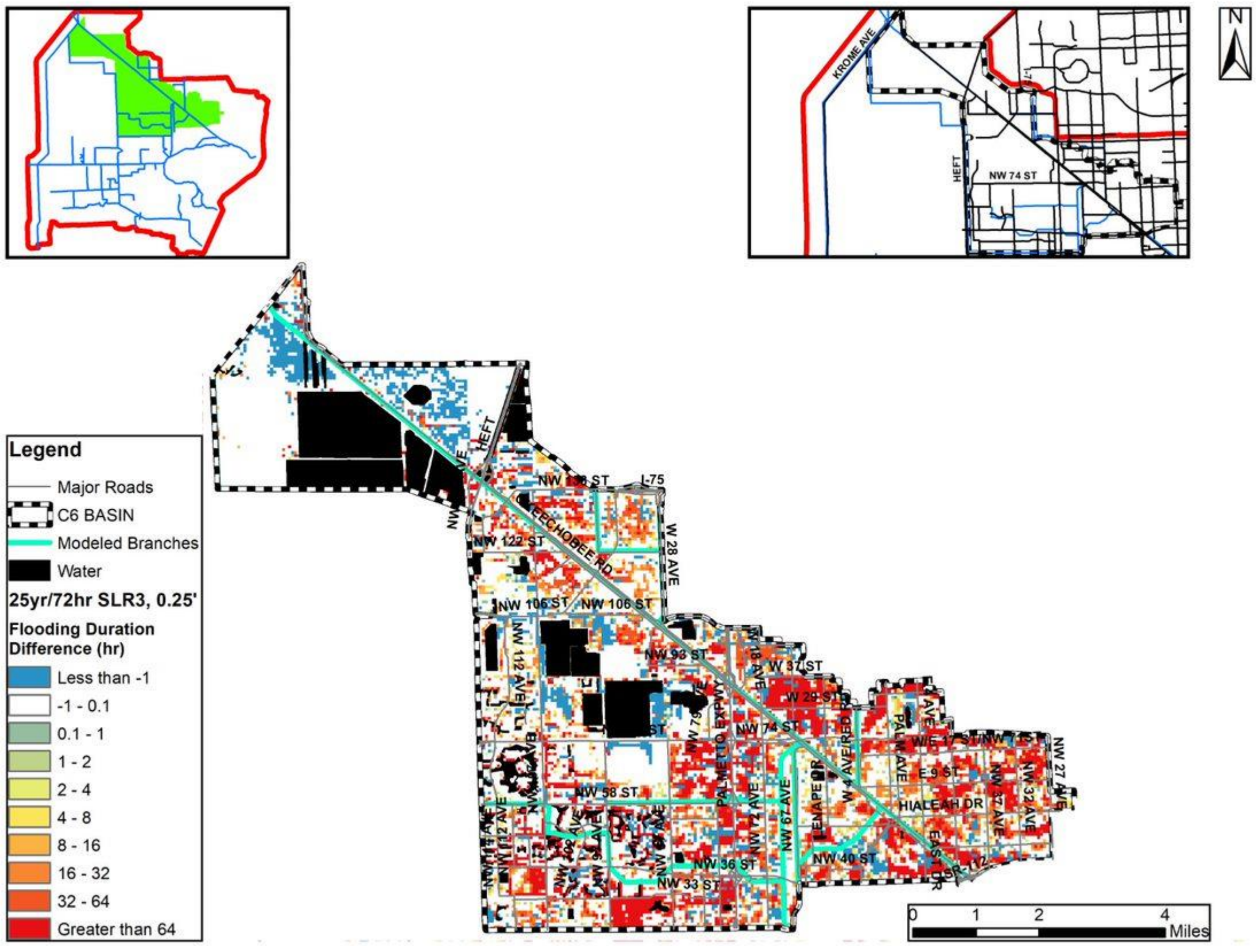


Figure D 5-42. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in the C6 Watershed

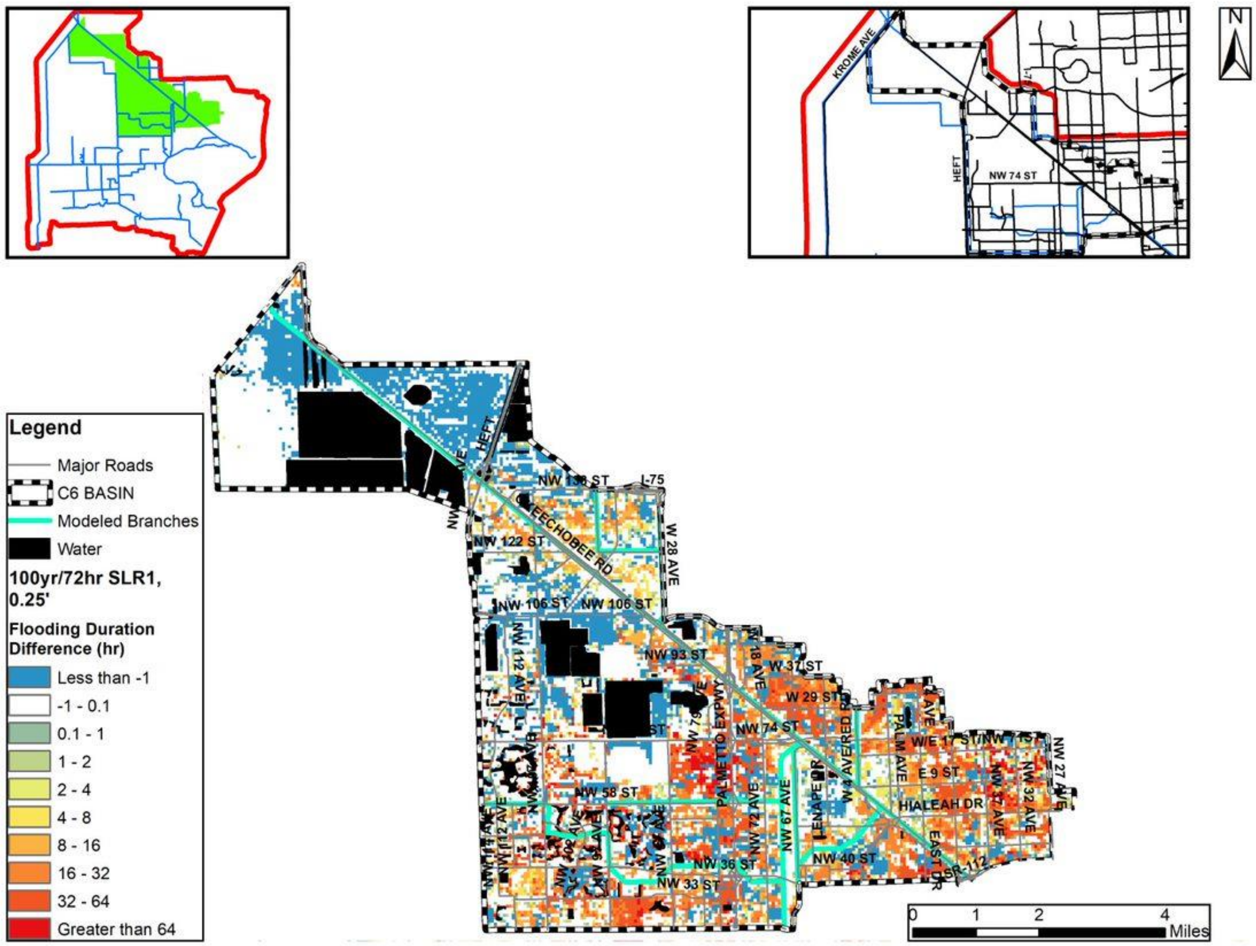


Figure D 5-43. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in the C6 Watershed

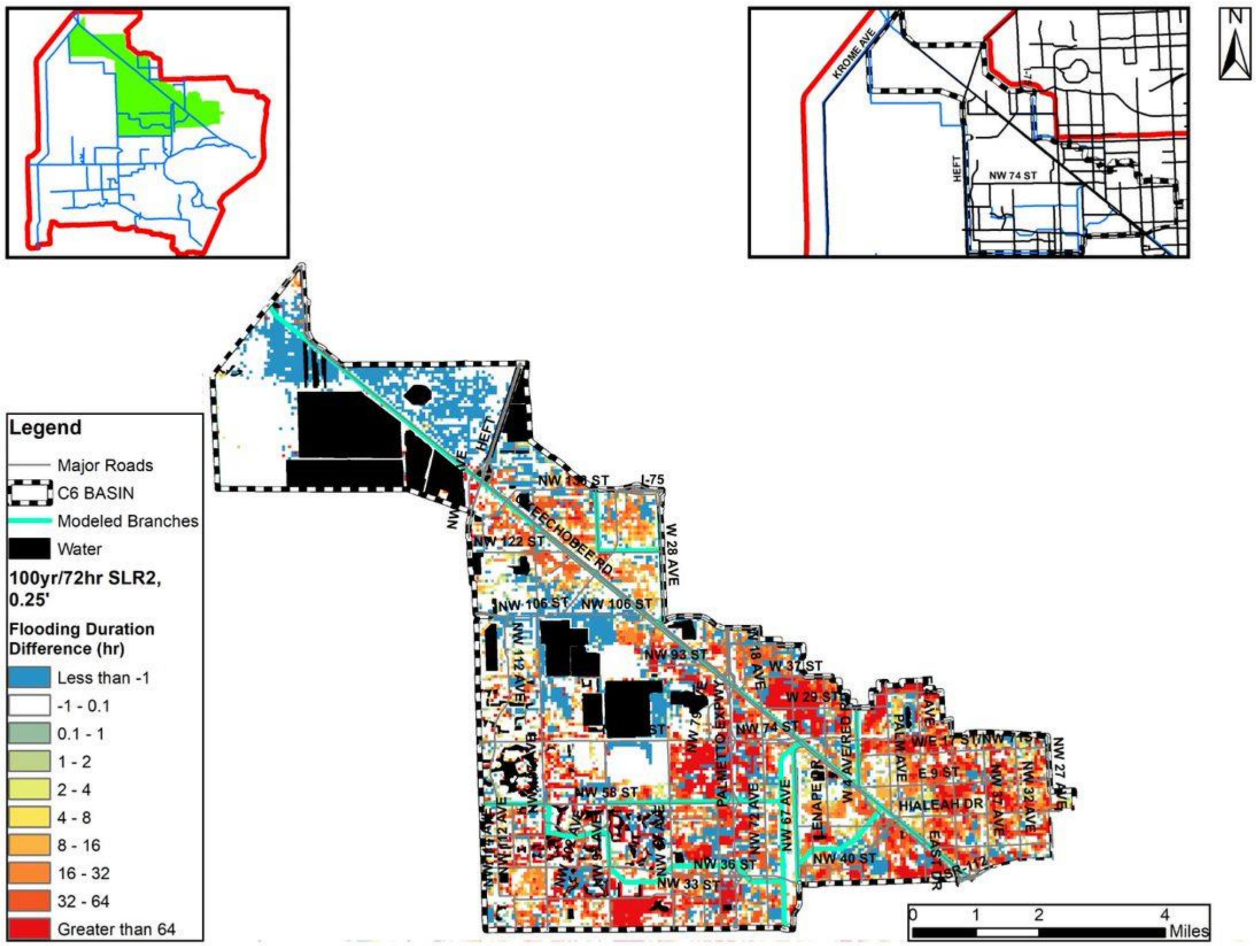


Figure D 5-44. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in the C6 Watershed

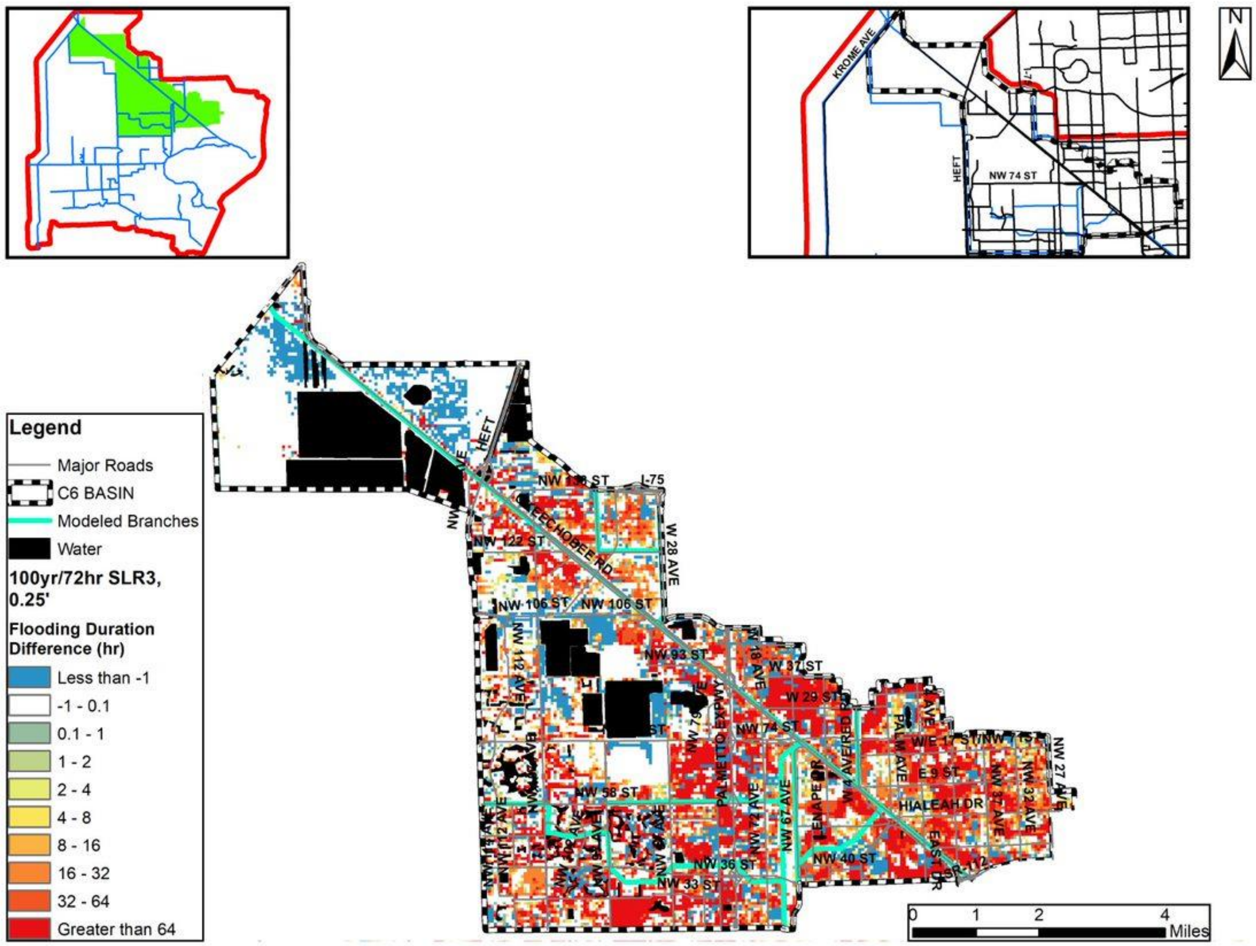


Figure D 5-45. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C6 Watershed

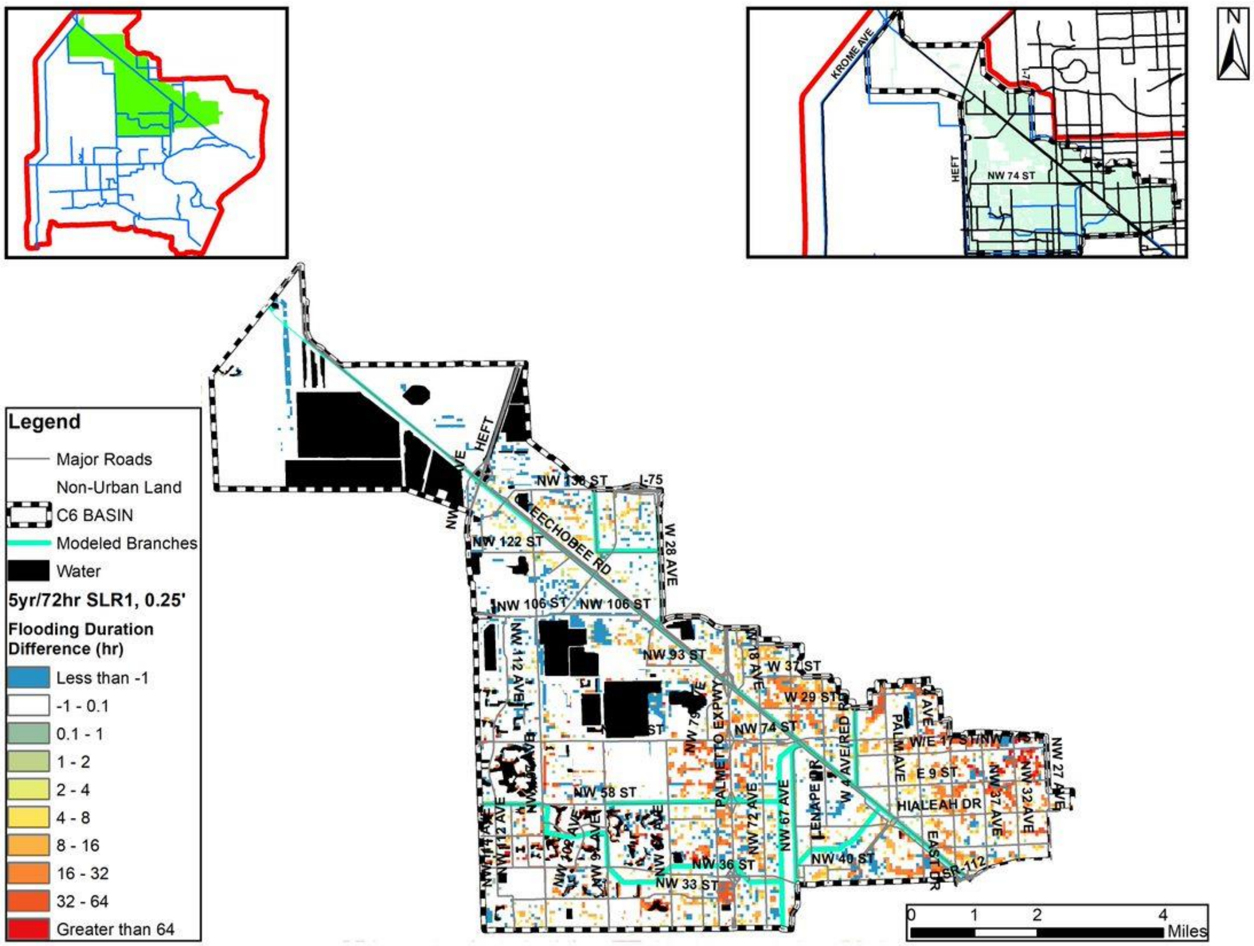


Figure D 5-46. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C6 Watershed

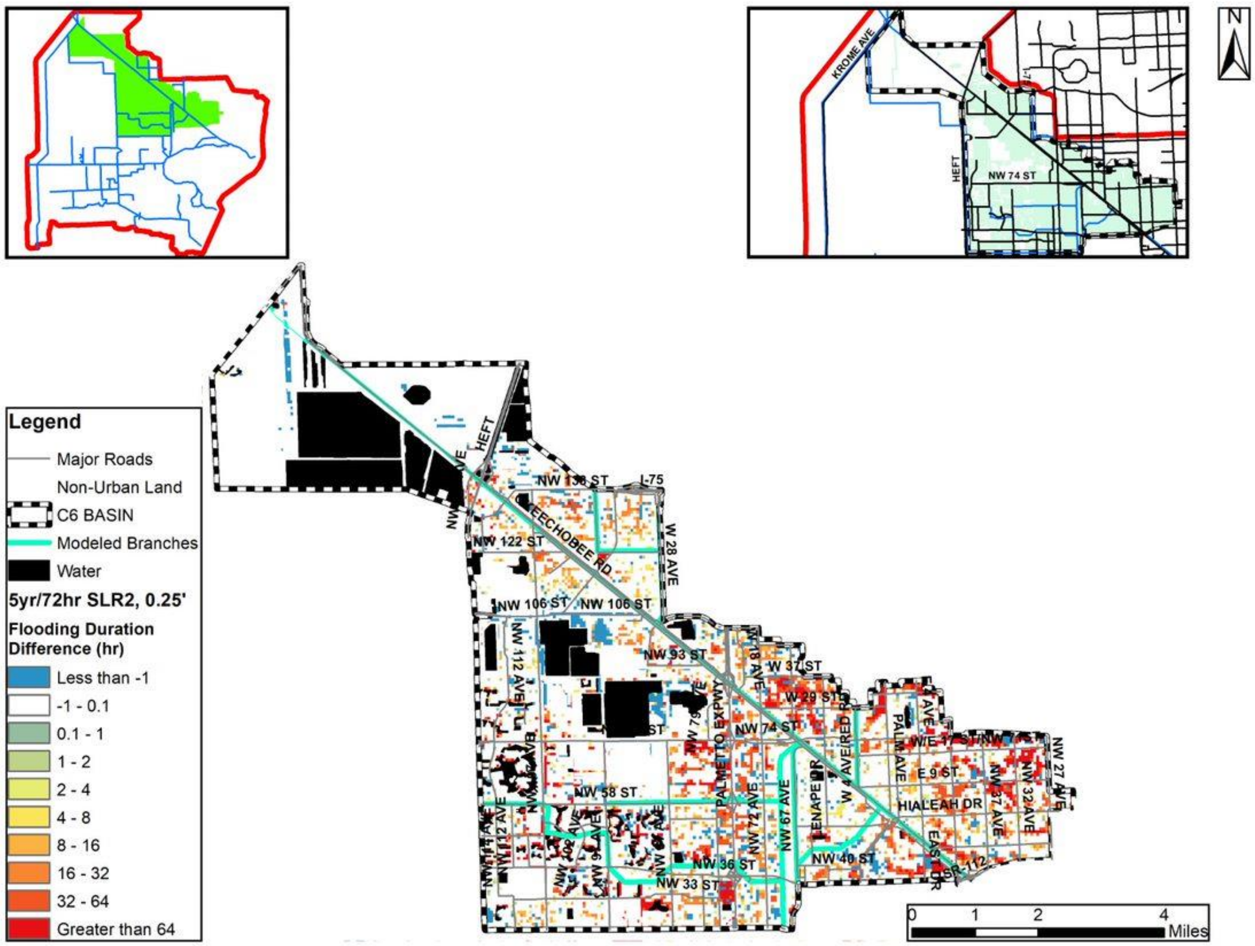


Figure D 5-47. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 5-year 3-Day Design Storm in Urban Areas in the C6 Watershed

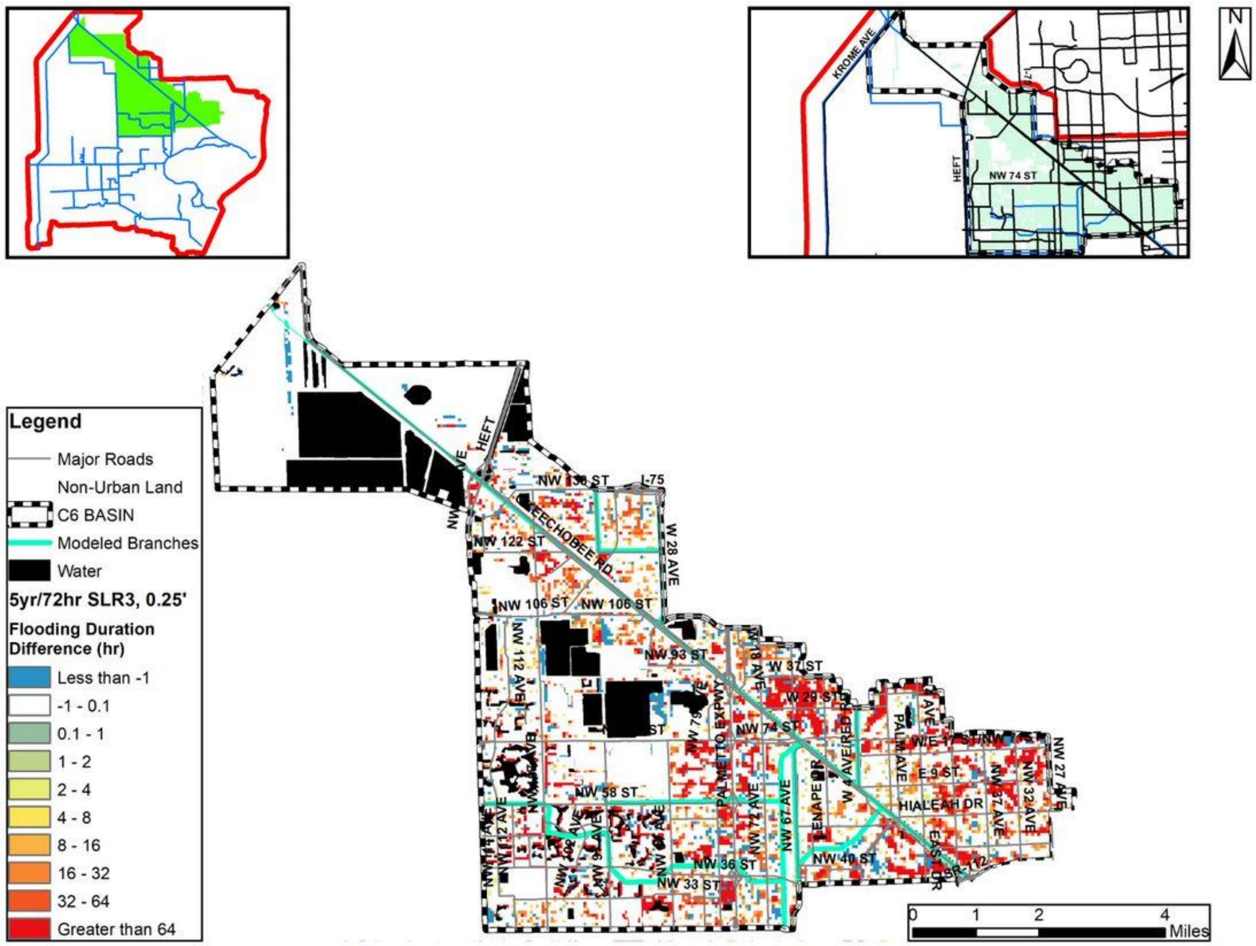


Figure D 5-48. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C6 Watershed

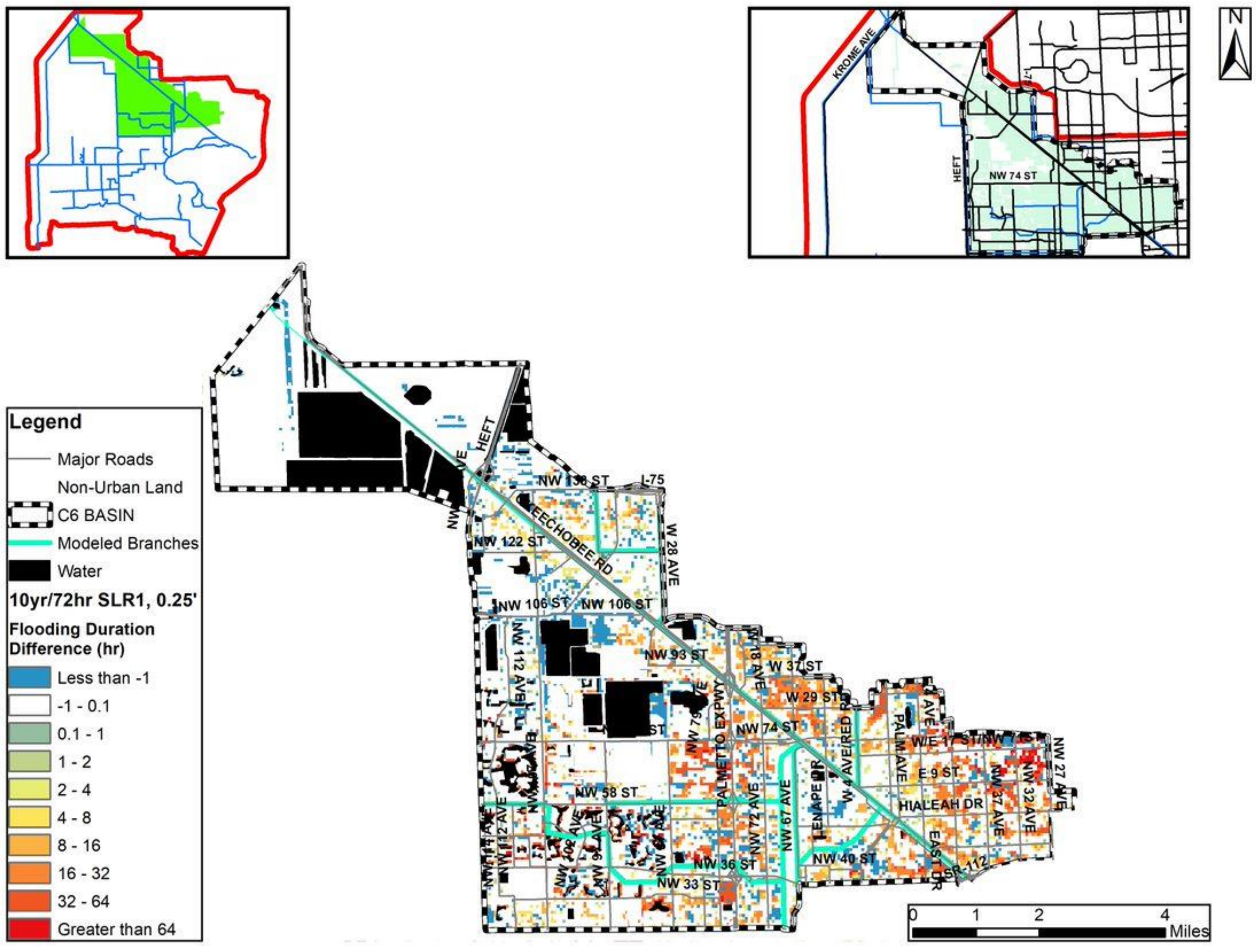


Figure D 5-49. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C6 Watershed

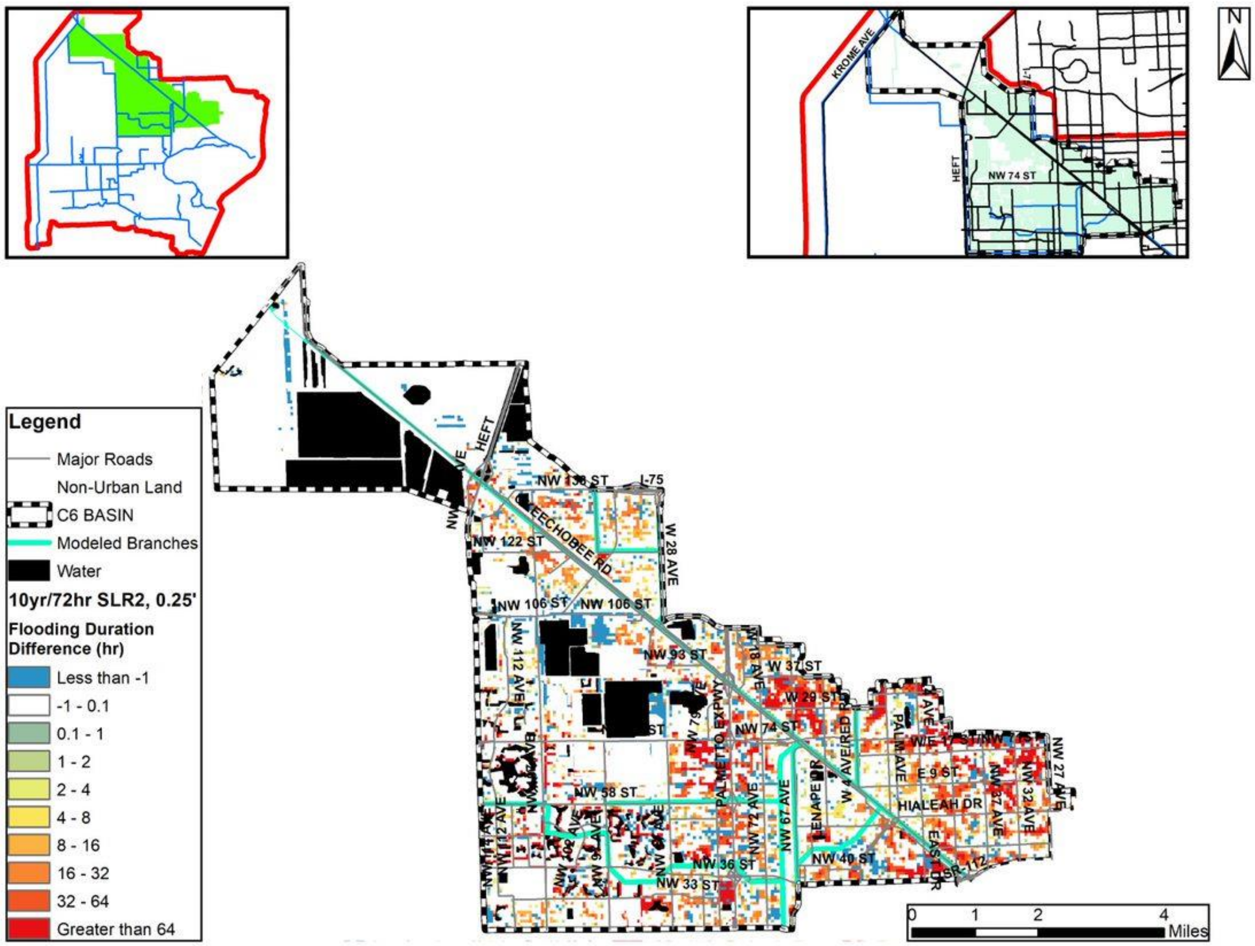


Figure D 5-50. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 10-year 3-Day Design Storm in Urban Areas in the C6 Watershed

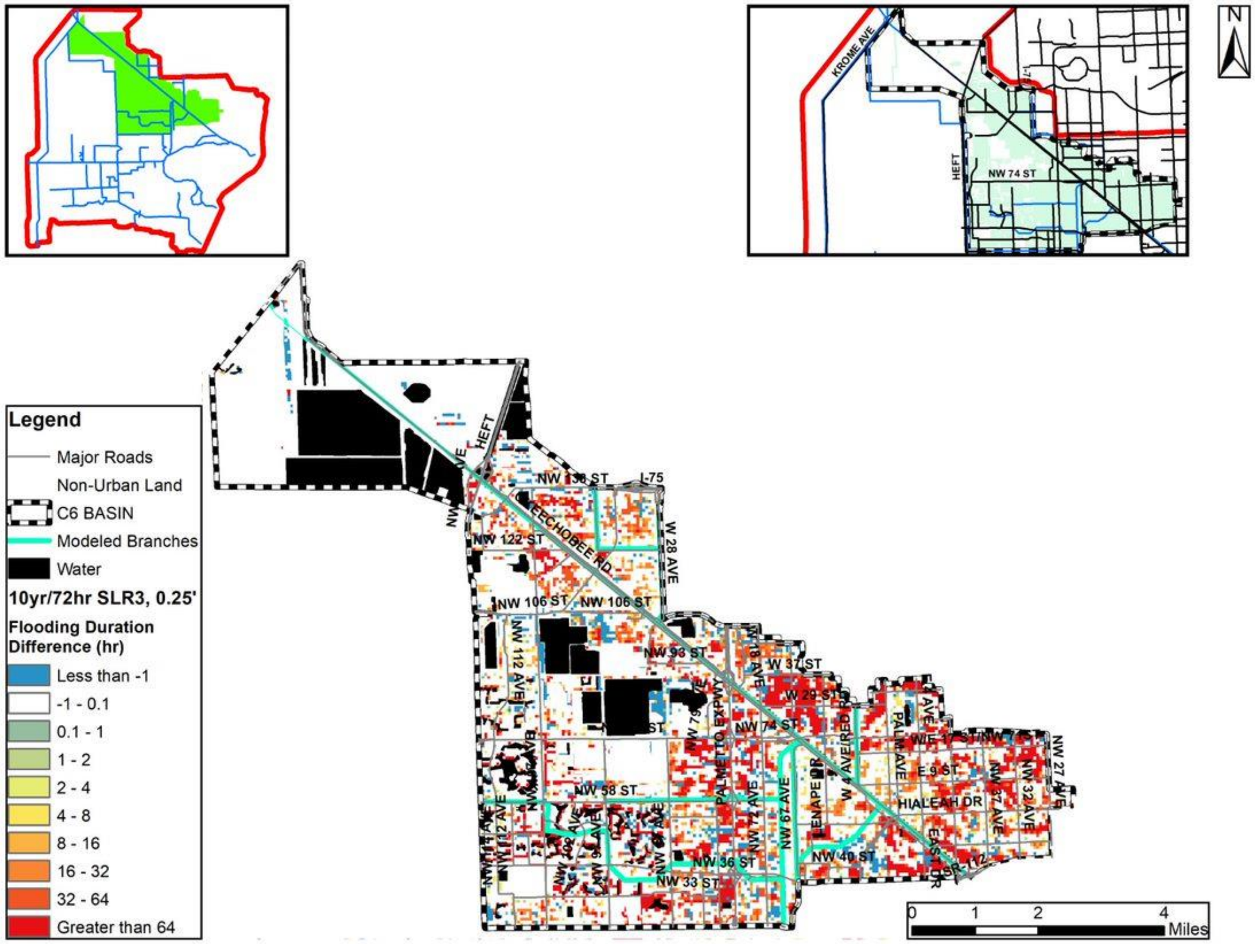


Figure D 5-51. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C6 Watershed

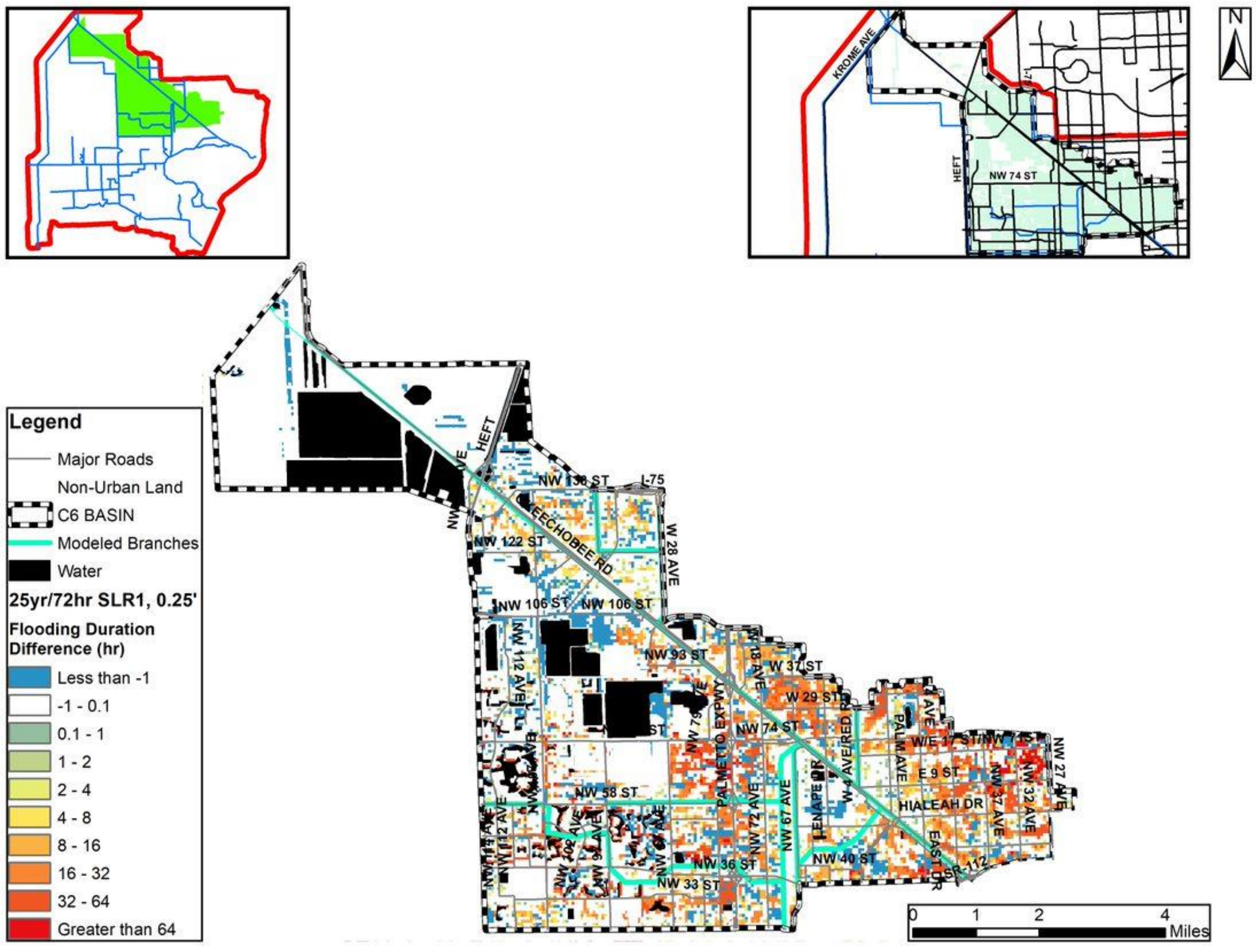


Figure D 5-52. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C6 Watershed

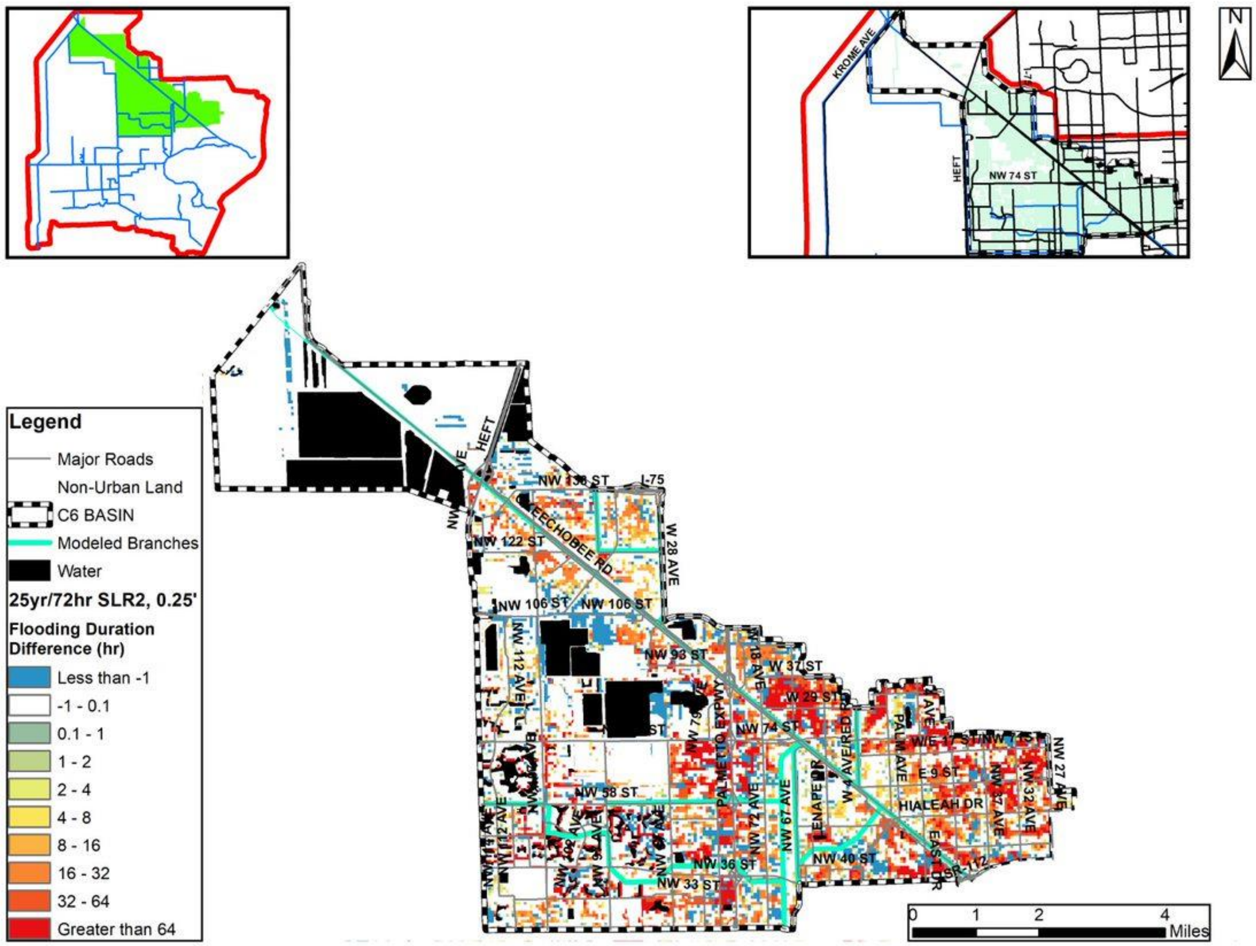


Figure D 5-53. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 25-year 3-Day Design Storm in Urban Areas in the C6 Watershed

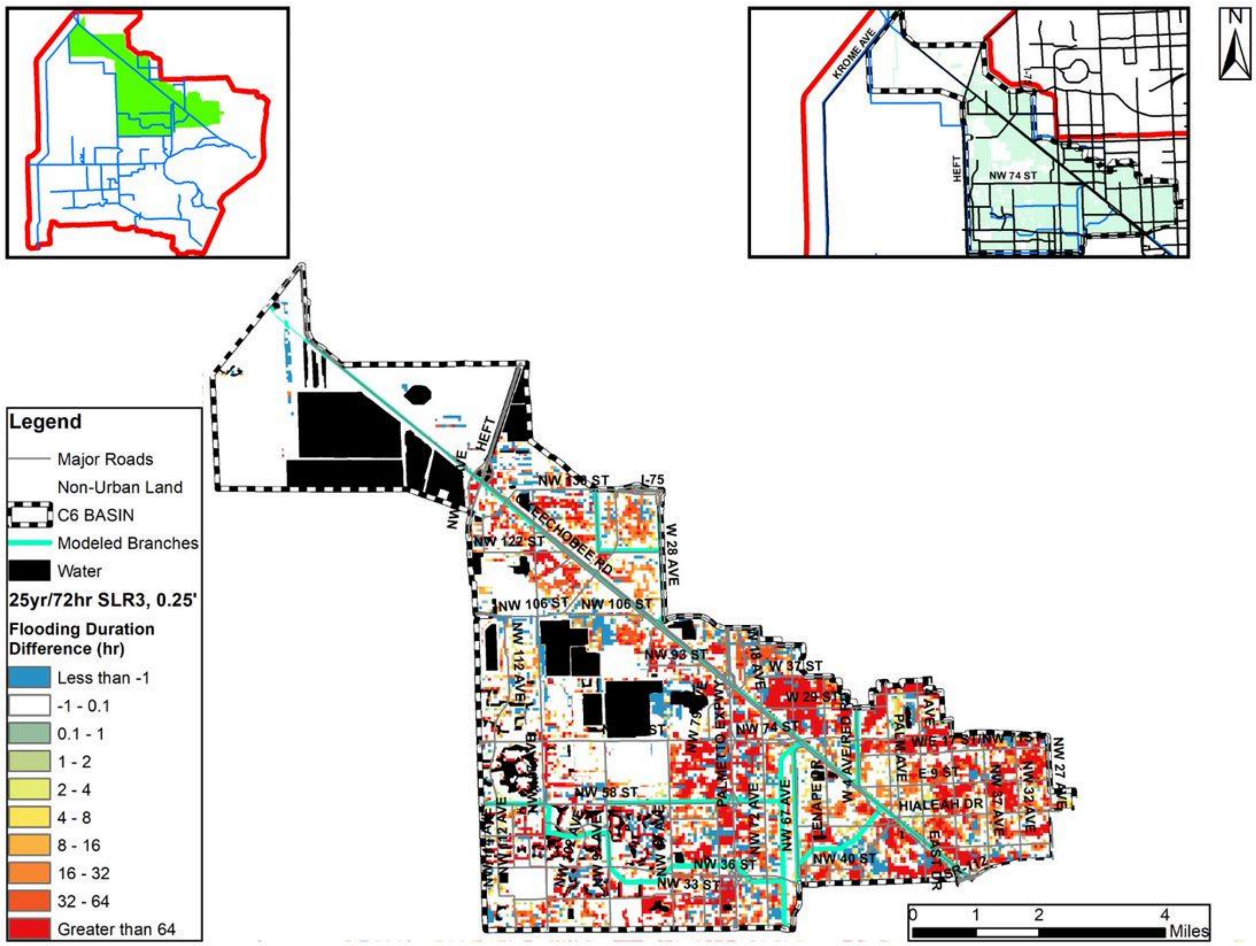


Figure D 5-54. Flooding Depth Difference of SLR +1 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C6 Watershed

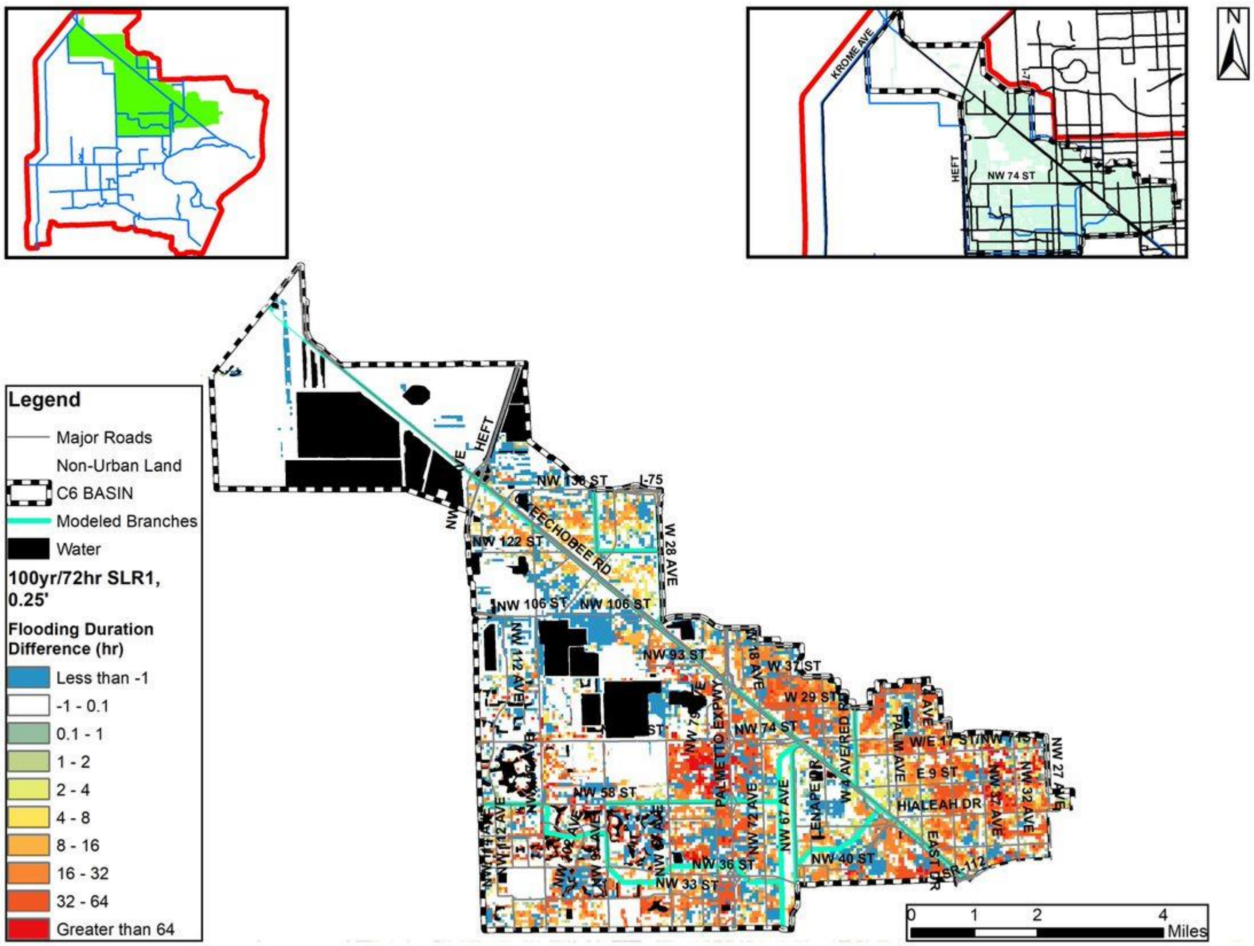


Figure D 5-55. Flooding Depth Difference of SLR +2 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C6 Watershed

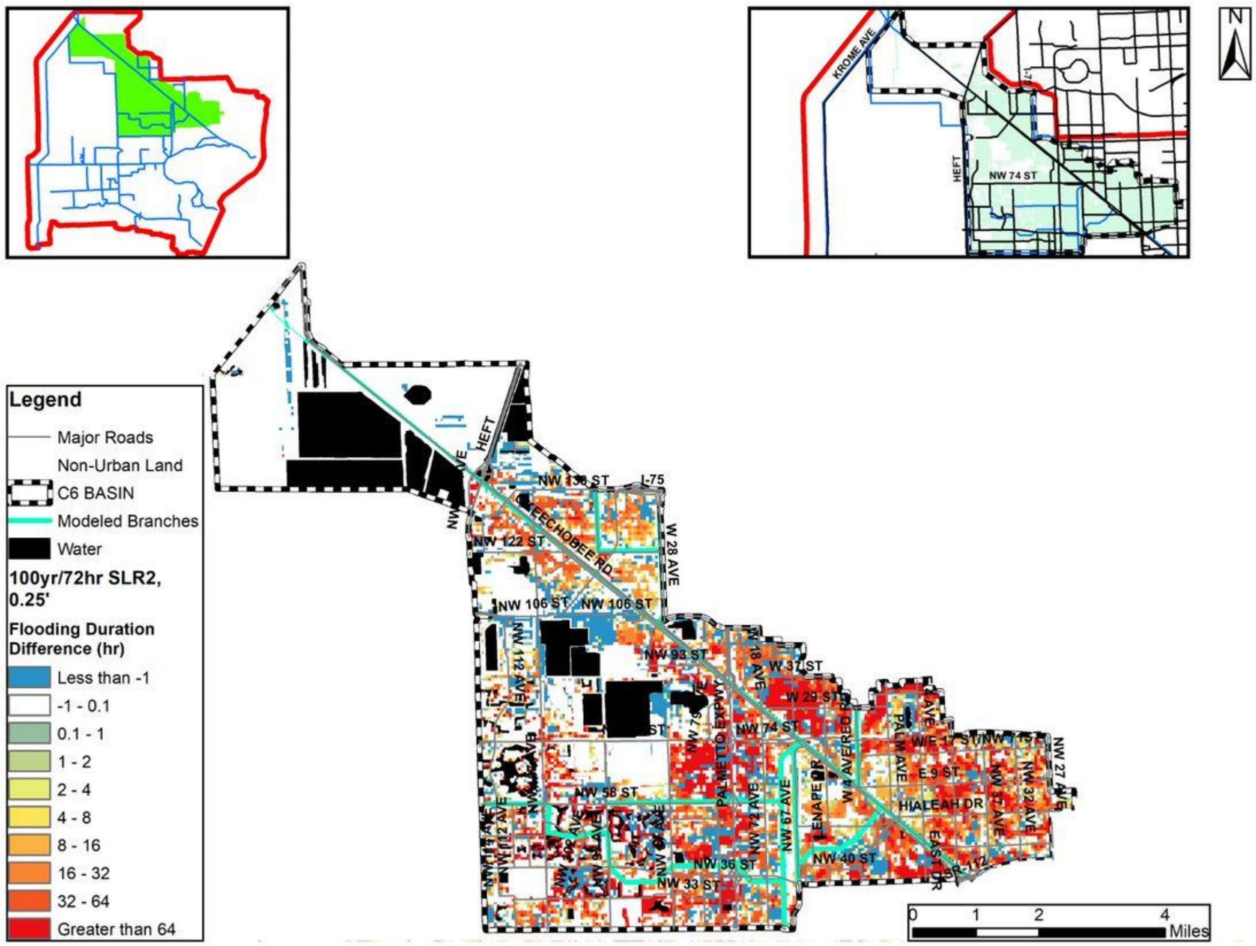
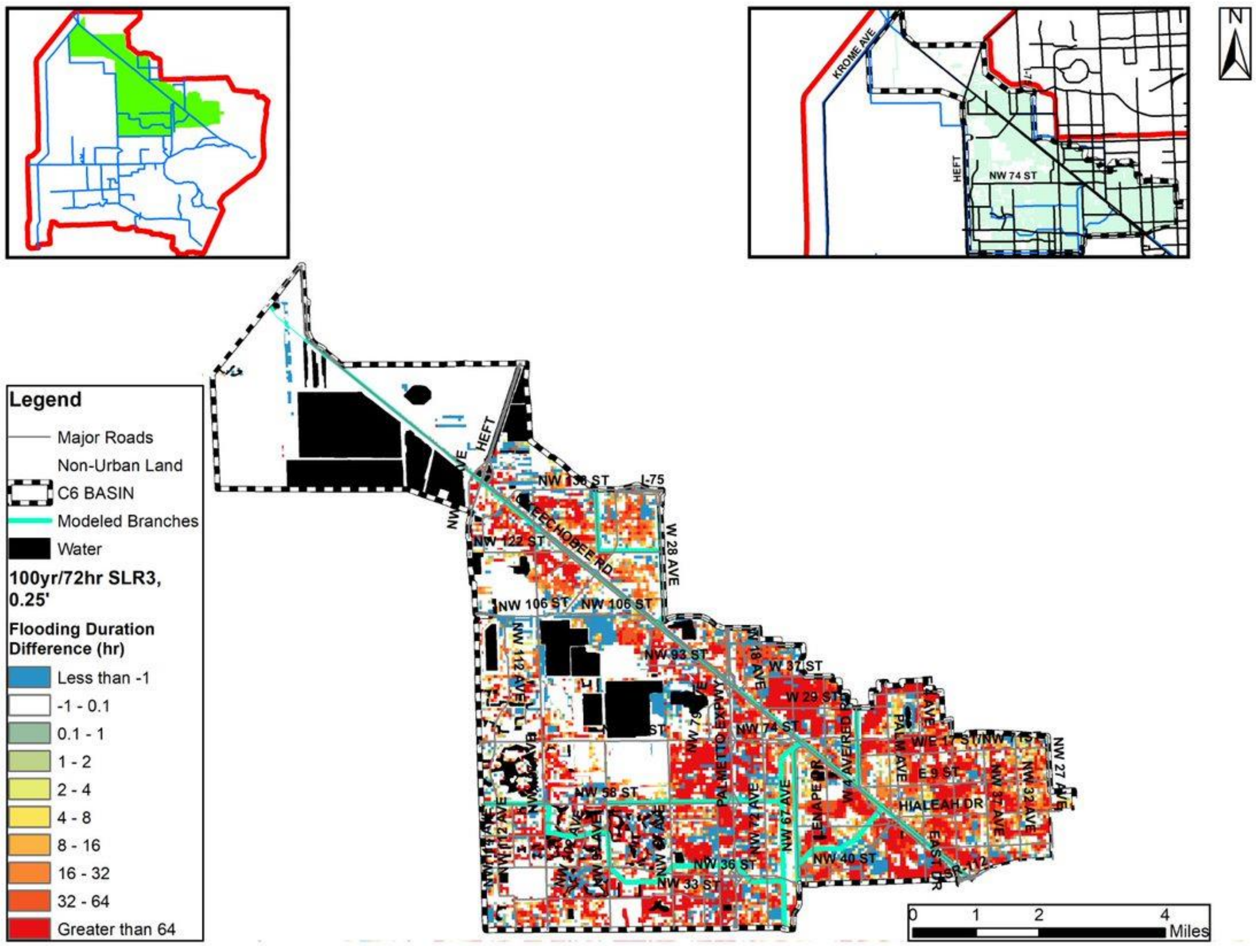


Figure D 5-56. Flooding Depth Difference of SLR +3 ft and Current Conditions for the 100-year 3-Day Design Storm in Urban Areas in the C6 Watershed



APPENDIX E

Water Budgets for All Simulations

Appendix E – Water Budgets by Watershed

Water budgets were performed for each watershed using the MIKE SHE Water Balance Calculation tool. The tables below show the total inches for each water budget input (shown as positive values) and output (shown as negative values) and for each design storm. In addition, the change in storage for the overland and groundwater modules are summed and the remaining residual indicates any water that remains unaccounted. The tables compare the water budget for each SLR condition (i.e., Current Conditions, SLR1, SLR2, and SLR3). For the Current Conditions simulations, the watershed residuals are low and considered to be performing as intended.

Table E 1. C2 Watershed (33,511 ac) – MIKE SHE Water Balance

	Current Conditions				SLR +1ft				SLR +2ft				SLR +3ft			
MIKE SHE Water Balance [IN]	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**
Rainfall	17.46	12.94	10.39	8.74	17.46	12.94	10.39	8.74	17.46	12.94	10.39	8.74	17.46	12.94	10.39	8.74
ET	-0.94	-0.90	-0.86	-0.84	-0.94	-0.91	-0.88	-0.86	-0.98	-0.95	-0.92	-0.90	-0.99	-0.97	-0.95	-0.94
SZ -> River	-10.25	-8.50	-7.45	-6.68	-10.64	-8.90	-7.81	-7.08	-10.55	-8.82	-7.80	-7.07	-10.54	-8.71	-7.74	-7.09
River -> SZ	1.64	1.63	1.68	1.70	2.17	2.32	2.32	2.35	3.02	3.02	3.00	2.99	3.86	4.05	4.20	4.30
OL -> River	-1.96	-1.05	-0.63	-0.42	-2.26	-1.30	-0.84	-0.58	-2.52	-1.53	-1.03	-0.74	-2.62	-1.87	-1.42	-1.13
GW Boundary Inflows	1.38	1.24	1.15	1.09	1.46	1.42	1.36	1.31	1.51	1.47	1.45	1.45	1.49	1.47	1.46	1.46
GW Boundary Outflows	-4.60	-3.76	-3.26	-2.94	-4.44	-3.68	-3.23	-2.93	-4.40	-3.72	-3.32	-3.06	-4.26	-3.63	-3.28	-3.05
OL Boundary Inflows	0.02	0.01	0.00	0.00	0.02	0.01	0.00	0.00	0.03	0.01	0.01	0.00	0.04	0.02	0.01	0.01
OL Boundary Outflows	-0.02	0.00	0.00	0.00	-0.02	0.00	0.00	0.00	-0.02	-0.01	0.00	0.00	-0.02	-0.01	0.00	0.00
GW Pumping	-1.68	-1.68	-1.68	-1.68	-2.15	-2.29	-2.29	-2.29	-2.29	-2.29	-2.29	-2.29	-2.29	-2.29	-2.29	-2.29
TOTAL	1.05	-0.07	-0.66	-1.03	0.66	-0.39	-0.98	-1.34	1.26	0.12	-0.51	-0.88	2.13	1.00	0.38	0.01
OL Change in Storage	1.45	0.71	0.32	0.08	1.05	0.41	0.08	-0.12	1.29	0.58	0.20	-0.02	1.78	0.98	0.56	0.30
GW Change in Storage	-0.37	-0.75	-0.94	-1.05	-0.37	-0.77	-1.02	-1.17	0.00	-0.41	-0.68	-0.83	0.34	0.05	-0.14	-0.27
Residual	0.02	0.01	0.01	0.02	0.01	-0.01	-0.01	0.00	0.03	0.00	-0.02	-0.02	0.04	0.04	0.02	0.00

Table E 2. C3W Watershed – MIKE SHE Water Balance

	Current Conditions				SLR +1ft				SLR +2ft				SLR +3ft			
MIKE SHE Water Balance [IN]	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**
Rainfall	16.99	12.59	10.12	8.52	16.99	12.59	10.12	8.52	16.99	12.59	10.12	8.52	16.99	12.59	10.12	8.52
ET	-0.71	-0.65	-0.61	-0.57	-0.74	-0.69	-0.65	-0.62	-0.77	-0.72	-0.70	-0.68	-0.79	-0.75	-0.73	-0.71
SZ -> River	-15.88	-11.98	-9.66	-8.08	-15.83	-11.96	-9.48	-8.00	-15.24	-11.58	-9.18	-7.84	-14.80	-11.25	-8.75	-7.48
River -> SZ	0.06	0.08	0.08	0.08	0.07	0.09	0.11	0.12	0.15	0.22	0.29	0.32	0.32	0.43	0.66	0.81
OL -> River	-2.03	-1.10	-0.72	-0.54	-2.22	-1.22	-0.86	-0.64	-2.63	-1.50	-1.05	-0.78	-3.02	-1.85	-1.34	-1.01
GW Boundary Inflows	6.92	5.01	3.94	3.23	7.03	5.31	4.80	4.06	6.85	5.41	5.05	4.41	6.58	5.22	4.86	4.30
GW Boundary Outflows	-4.73	-3.88	-3.36	-3.00	-4.81	-4.13	-4.18	-3.76	-4.52	-4.19	-4.51	-4.15	-4.26	-3.90	-4.43	-4.24
OL Boundary Inflows	0.10	0.03	0.01	0.00	0.12	0.03	0.10	0.05	0.14	0.04	0.11	0.07	0.17	0.05	0.13	0.08
OL Boundary Outflows	-0.04	-0.01	0.00	0.00	-0.05	-0.02	-0.11	-0.06	-0.06	-0.03	-0.13	-0.08	-0.08	-0.03	-0.16	-0.10
GW Pumping	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TOTAL	0.68	0.09	-0.20	-0.36	0.56	0.00	-0.15	-0.33	0.91	0.24	0.00	-0.21	1.11	0.51	0.36	0.17
OL Change in Storage	0.64	0.41	0.30	0.24	0.59	0.34	0.24	0.16	0.69	0.41	0.28	0.20	0.80	0.51	0.38	0.29
GW Change in Storage	-0.06	-0.34	-0.46	-0.57	-0.04	-0.29	-0.41	-0.46	0.18	-0.09	-0.25	-0.35	0.19	0.07	0.04	0.01
Residual	0.05	0.01	0.02	0.00	0.10	0.06	0.02	0.03	0.10	0.11	0.07	0.07	0.04	0.08	0.11	0.13

Table E 3. C4 Watershed – MIKE SHE Water Balance

	Current Conditions				SLR +1ft				SLR +2ft				SLR +3ft			
MIKE SHE Water Balance [IN]	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**
Rainfall	16.69	12.37	9.93	8.34	16.69	12.37	9.93	8.34	16.69	12.37	9.93	8.34	16.69	12.37	9.93	8.34
ET	-1.23	-1.21	-1.20	-1.19	-1.24	-1.22	-1.20	-1.19	-1.25	-1.23	-1.22	-1.21	-1.25	-1.24	-1.23	-1.22
SZ -> River	-19.33	-16.84	-15.31	-14.31	-19.44	-16.96	-15.39	-14.31	-19.56	-17.27	-16.01	-15.12	-20.24	-17.69	-16.25	-15.30
River -> SZ	0.43	0.42	0.41	0.42	0.60	0.55	0.52	0.51	1.05	0.92	0.86	0.83	1.72	1.67	1.63	1.61
OL -> River	8.81	8.53	8.30	8.21	9.43	9.19	8.81	8.54	9.68	9.67	9.65	9.54	10.62	10.21	9.97	9.81
GW Boundary Inflows	2.24	1.97	1.77	1.63	2.00	1.75	1.61	1.48	1.83	1.64	1.55	1.47	1.62	1.42	1.35	1.28
GW Boundary Outflows	-1.22	-1.01	-0.89	-0.83	-1.36	-1.19	-1.09	-1.00	-1.30	-1.18	-1.12	-1.06	-1.23	-1.12	-1.07	-1.02
OL Boundary Inflows	0.16	0.10	0.07	0.06	0.17	0.11	0.08	0.06	0.20	0.12	0.10	0.07	0.24	0.15	0.12	0.09
OL Boundary Outflows	-0.19	-0.12	-0.09	-0.07	-0.22	-0.14	-0.11	-0.08	-0.25	-0.16	-0.13	-0.11	-0.30	-0.19	-0.16	-0.13
GW Pumping	-0.50	-0.50	-0.50	-0.50	-1.32	-1.32	-1.32	-1.32	-1.32	-1.32	-1.32	-1.32	-1.32	-1.32	-1.32	-1.32
TOTAL	5.86	3.71	2.49	1.76	5.31	3.14	1.84	1.03	5.77	3.56	2.29	1.43	6.55	4.26	2.97	2.14
OL Change in Storage	5.86	3.88	2.77	2.07	5.35	3.32	2.14	1.41	5.61	3.60	2.47	1.70	6.16	4.09	2.91	2.15
GW Change in Storage	-0.15	-0.27	-0.33	-0.36	-0.14	-0.25	-0.33	-0.38	0.02	-0.10	-0.19	-0.25	0.11	0.04	-0.01	-0.04
Residual	-0.01	-0.01	-0.01	-0.01	0.01	0.00	0.00	0.01	0.01	0.01	0.01	0.01	-0.02	-0.01	0.00	0.01

Table E 4. C5 Watershed – MIKE SHE Water Balance

	Current Conditions				SLR +1ft				SLR +2ft				SLR +3ft			
MIKE SHE Water Balance [IN]	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**
Rainfall	16.97	12.63	10.16	8.54	16.97	12.63	10.16	8.54	16.97	12.63	10.16	8.54	16.97	12.63	10.16	8.54
ET	-0.54	-0.50	-0.47	-0.45	-0.56	-0.52	-0.50	-0.47	-0.60	-0.56	-0.53	-0.51	-0.64	-0.60	-0.58	-0.56
SZ -> River	-19.91	-16.65	-14.50	-12.91	-19.05	-15.72	-13.56	-12.01	-19.37	-16.16	-14.12	-12.67	-23.97	-20.99	-19.23	-18.09
River -> SZ	0.10	0.04	0.02	0.01	0.34	0.24	0.20	0.17	1.78	1.71	1.70	1.72	5.98	6.38	6.79	7.09
OL -> River	-5.36	-2.96	-1.77	-1.07	-6.47	-3.98	-2.69	-1.91	-8.79	-5.97	-4.51	-3.56	-9.02	-6.08	-4.50	-3.62
GW Boundary Inflows	15.00	12.13	10.31	9.08	15.74	12.53	10.51	9.08	16.73	13.49	11.46	10.02	17.78	14.67	12.49	11.14
GW Boundary Outflows	-5.66	-4.56	-3.80	-3.31	-4.84	-4.05	-3.48	-3.05	-4.32	-3.66	-3.18	-2.89	-4.48	-3.87	-3.40	-3.07
OL Boundary Inflows	0.11	0.05	0.03	0.02	0.16	0.08	0.05	0.03	0.34	0.12	0.08	0.05	0.78	0.36	0.15	0.11
OL Boundary Outflows	-0.01	0.00	0.00	0.00	-0.01	0.00	0.00	0.00	-0.02	0.00	0.00	0.00	-0.03	0.00	0.00	0.00
GW Pumping	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
TOTAL	0.70	0.18	-0.02	-0.09	2.28	1.21	0.69	0.38	2.72	1.60	1.06	0.70	3.37	2.50	1.88	1.54
OL Change in Storage	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.04	0.04	0.03	0.03
GW Change in Storage	0.62	0.24	0.06	-0.03	0.56	0.18	0.03	-0.07	0.45	0.12	-0.05	-0.13	0.37	0.15	0.02	-0.07
Residual	0.00	0.01	-0.01	0.00	0.00	0.00	0.01	0.00	0.00	-0.01	-0.01	0.01	0.00	-0.01	0.01	0.00

Table E 5. C6 Watershed – MIKE SHE Water Balance

	Current Conditions				SLR +1ft				SLR +2ft				SLR +3ft			
MIKE SHE Water Balance [IN]	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**	**100-yr**	**25-yr**	**10-yr**	**5-yr**
Rainfall	16.84	12.48	10.01	8.40	16.84	12.48	10.01	8.40	16.84	12.48	10.01	8.40	16.84	12.48	10.01	8.40
ET	-0.96	-0.93	-0.90	-0.88	-0.96	-0.92	-0.90	-0.88	-0.98	-0.94	-0.92	-0.90	-1.00	-0.97	-0.95	-0.93
SZ -> River	-12.88	-11.14	-10.06	-9.32	-13.47	-11.62	-10.48	-9.66	-12.84	-11.10	-10.09	-9.39	-13.31	-11.38	-10.28	-9.55
River -> SZ	1.03	1.12	1.17	1.23	1.21	1.33	1.41	1.44	1.60	1.68	1.72	1.75	2.32	2.51	2.66	2.78
OL -> River	1.19	2.27	2.78	3.06	1.35	2.40	2.90	3.19	1.11	2.09	2.62	2.93	1.88	2.57	2.96	3.18
GW Boundary Inflows	1.65	1.50	1.42	1.37	1.99	1.91	1.82	1.76	1.88	1.82	1.76	1.72	1.76	1.69	1.63	1.57
GW Boundary Outflows	-2.70	-2.32	-2.04	-1.82	-2.52	-2.14	-1.87	-1.67	-2.37	-2.02	-1.79	-1.60	-2.16	-1.87	-1.67	-1.51
OL Boundary Inflows	0.60	0.44	0.36	0.31	0.48	0.33	0.25	0.20	0.58	0.41	0.32	0.27	0.69	0.51	0.42	0.35
OL Boundary Outflows	-0.60	-0.40	-0.31	-0.26	-0.52	-0.32	-0.23	-0.18	-0.60	-0.39	-0.29	-0.23	-0.69	-0.47	-0.36	-0.30
GW Pumping	-1.70	-1.70	-1.70	-1.70	-2.06	-2.06	-2.06	-2.06	-2.06	-2.06	-2.06	-2.06	-2.06	-2.06	-2.06	-2.06
TOTAL	2.47	1.32	0.73	0.39	2.34	1.39	0.85	0.54	3.16	1.97	1.28	0.89	4.27	3.01	2.36	1.93
OL Change in Storage	1.81	1.02	0.63	0.41	2.00	1.33	0.99	0.79	2.44	1.65	1.23	0.98	3.10	2.24	1.78	1.50
GW Change in Storage	0.38	0.15	0.00	-0.09	0.14	-0.01	-0.13	-0.22	0.36	0.14	-0.03	-0.15	0.47	0.36	0.27	0.21
Residual	0.01	0.01	0.01	0.02	0.03	0.03	0.04	0.05	0.02	0.03	0.03	0.04	0.00	0.01	0.00	0.01