Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed

DRAFT FINAL REPORT

South Florida Water Management District
Hydrology and Hydraulics Bureau

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Flood Protection Analysis for the C-4 Watershed --- DRAFT Final Report

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Flood Protection Level of Service Analysis for the C-4 Watershed

EXECUTIVE SUMMARY

The South Florida Water Management District (SFWMD) is evaluating current and future Level of Service (LOS) for flood protection throughout the 16-county region as a means to identify and prioritize long-term infrastructure needs. The LOS program provides a process to establish flood protection thresholds for each basin. These thresholds initiate retrofit and adaptation efforts that will be implemented in conjunction with the District's ongoing structure, canal and levee maintenance programs. Results of this study will be used to identify sea levels at which existing infrastructure can no longer provide flood protection, facilities at risk for flood impacts and the potential need for improvements to operations, canal conveyance or primary and secondary drainage features.

An Adaptive Resilience Planning approach is used to implement changes in three steps, once levels of existing and potential future risks have been defined. The first step is to initiate non-structural and operational changes to reduce the extent and duration of flooding. Second, monitor conditions over time and establish thresholds for hydrologic changes that trigger the need for infrastructure replacement. Third, based on the time needed for construction, initiate infrastructure replacement and upgrades, as thresholds are reached.

An initial study of methods and criteria for such evaluations was conducted in the C-4 Canal watershed in Miami-Dade County, Florida, based on a simplified hydrologic and hydraulic model and a set of performance measures developed for the study. The C-4 canal watershed was selected for this investigation because it has a mixture of residential, commercial and industrial development; environmentally-sensitive wetlands; and a major wellfield used for public water supply; as well as a long history of water management concerns. This area has seen extensive population growth and changes in land use since the design and construction of the original federal flood control project in the 1950s and 1960s. Much of the western portion of the watershed has been dredged to create large lakes and rockpits and the rock has been used to provide fill for development. During the same period, sea level has increased approximately one-half foot, thereby reducing the discharge capacity of the coastal structures.

Although the original water management infrastructure would be inadequate for conditions that exist today, significant improvements have been made to increase stormwater storage, improve drainage of excess surface water into groundwater, reduce groundwater inflow from the Everglades, provide pumping capacity to remove water from developed areas and increase discharges from coastal structures. Because of all these changes, the capacity to protect developed areas from flooding and remove excess water has greatly increased.

Coastal water levels are projected to increase in the future, resulting in greater storm surge and increases of upstream water levels within the watershed. The effects of these changes in water levels on LOS within the watershed were assessed as a basis to identify risk to existing resources and needs for improvements to primary drainage system operations and/or infrastructure. The methodology used in this investigation will subsequently be applied to the remaining watersheds in the Miami River system and to other watersheds throughout the District. The LOS for flood protection is assessed initially by comparing current conditions in the watershed with the original design, to identify changes that have occurred during the past 50 years. LOS is then assessed for a range of future conditions focused on a 50-year planning horizon. A hydraulic mathematical model of the watershed is used to simulate performance of the regional water management system during a series of combined rainfall and storm surge conditions, ranging from 1-in-5 year (20% probability) to 1-in-100 year (1% probability) events. Future conditions consider the additional effects of three possible sea level rise scenarios, with increases of + 4.1 in, + 9.6 in and + 27.1 in (10 cm, 24 cm, and 0.69 cm), respectively. Such increases may be expected to occur between now and the year 2065.

This study finds that the watershed presently has a 1-in-10 LOS for flood protection and is at risk for nuisance flooding at less than design storm conditions. Conditions are expected to worsen somewhat by 2020. Nuisance flooding will increase further by 2025-2030 with increasing sea level and the projected future sea level rise may cause failure of the primary system and significant flooding in six of the 26 developed sub-watersheds by 2050-2060.

As a result, projects should be initiated immediately to revise operational procedures rules and regulations and develop mitigation plans to help reduce or eliminate nuisance flooding. Careful monitoring of sea level changes is needed to better predict when threshold water levels, as identified in this report, will occur to trigger initiation of projects to plan, design, refurbish or replace critical infrastructure. In addition, the District needs to consider changes to primary drainage features and surface water management criteria. Projects should also be initiated, in cooperation with local interests and other agencies, to obtain better data for elevations of existing buildings and infrastructure. Longer term projects are needed to examine and upgrade secondary drainage systems and local building standards.

FLOOD PROTECTION LEVEL-OF-SERVICE ANALYSIS FOR THE C-4 WATERSHED

INTRODUCTION

Purpose of this Report

The District is in the process of implementing a new program to evaluate current and future Level of Service for flood protection (LOS). In order to develop the methodology for such program, a pilot study was initiated to evaluate the current and future LOS for flood protection in the C-4 Canal watershed in Miami-Dade County, Florida using a simplified hydrologic and hydraulic model analysis.

Scope

This project was undertaken as an initial step in determining LOS for the Miami River system, to test the development and use of models, define what is meant by LOS, to determine data needs from other agencies and local governments, and to develop and document a standardized approach to these analyses that can be applied to the remaining watersheds in the Miami River system and to other watersheds throughout the District.

LOS is assessed for current conditions in the watershed and for a range of future conditions focused on a 50-year planning horizon. A mathematical model of the watershed was developed by SFWMD to simulate performance of the regional water management system during a series of rainfall events, ranging from a 1-in-5 year storm (20% probability to a 1 in-100 year storm (1% probability). Future conditions consider the effect of three possible sea level rise scenarios. The model simulates water levels and water movement through groundwater, surface flow, and the canal system and provides estimates of water levels over time during the storm and for weeks afterward. Based on results from the model, various performance measures are used to estimate the depth, extent and impact of flooding in the watershed.

Project Objectives

- Define “Level of Service for Flood Protection” as it applies to the C-4 watershed
- Assess changes in rainfall frequency relationships, groundwater and seawater level conditions that may have occurred since the original project deign
- Describe primary, secondary and tertiary systems that work together to provide overall level of service for flood protection within the watershed
- Describe modeling tools used to assess Level of Service
- Describe performance measures and criteria used to quantify level of service
- Apply models, tools and criteria to document current level of service for flood protection within the watershed for a range of design storm events.
- Compare existing level of service to design conditions used in development of the original project
- Conduct additional analyses to determine how level of service will change under future conditions, of sea level rise
- Identify potential actions/responses for further study and evaluation
- Develop an assessment template and guidance documents to provide a format and direction for future watershed flood protection LOS studies.

Issues

Changing Conditions in the Watershed.

The C-4 canal watershed was selected for this pilot study because it has a long history of flood control problems. This area has seen extensive population growth and changes in land use since the construction of the USACE project. Much of the western portion of the watershed has been dredged to create large lakes and rockpits and the rock has been used to provide fill for development. Recently, construction has begun on a seepage barrier that is designed to reduce the flow of groundwater from the Everglades into the C-4 watershed. Since the design of the water management system by the USACE in the 1950s, water levels along the coast have increased approximately ½ foot, and coastal water levels are projected to increase more rapidly in the future. The effects of these historic and potential future changes in sea level on LOS within the watershed need to be assessed.

The C-4 Basin extends from the Everglades along its western boundary eastward to the point where the Miami Canal was constructed through the coastal ridge. West of the ridge, most of the C-4 canal watershed was part of the historical Everglades and has been prone to repeated flooding. Nearly half of the land in the watershed is less than 4 feet above sea level and only about 4% is above 8 feet (**SFWMD, 2014a**). The soils are predominantly wetland peat, marl and sand and the underlying surficial aquifer, consisting of highly porous and transmissive limestone, is at or near the surface throughout much of the watershed (**USACE, 1953**).

The water management infrastructure that was designed and built between 1954 and 1975 would be inadequate for conditions that exist today. Portions of the canal were never constructed to their original design specifications due to the expense of digging through solid rock. Flow through other sections of the canal is constrained by rock outcroppings, debris and bridges. Urban, commercial and industrial development has occurred in areas that originally were intended and designed for agricultural uses. Much of the development occurred during the 1950s through 1970s was based on varying assumptions concerning likely flood conditions and with limited documentation of building elevations.

However, during the intervening years, some significant improvements have been made in to increase stormwater storage, improve drainage of excess surface water into groundwater, reduce groundwater inflow from the Everglades, provide pumping capacity to remove water from developed areas and increase discharges from coastal structures. Because of all these changes, the capacity to remove water has greatly increased, but the current level of flood protection that exists within the system today is uncertain.

Flood Control.

Today, the drainage system in Miami-Dade County is a complex network of interconnected canals that extends from the Broward County Line to south of Homestead and is used to provide flood control and water supply for urban and agricultural lands located east of the Everglades Water Conservation Areas and Everglades National Park. The Miami Canal is still the primary connection to Lake Okeechobee. The canals of Miami County interact with the highly transmissive groundwater, so that rainfall in the watershed is rapidly transferred to the canal system. Excess water moves through the canals and is discharged to the coast, primarily by gravity flow (**SFWMD, 2015a**).

The western portion of C-4 watershed has had a long history of flood control issues. The original USACE project design of the 1950's provided full flood protection for the eastern portion of the watershed, east of 87th Avenue, but only minimal protection for the western basin, with the assumption that land in this portion of the basin would primarily be used for agriculture and other development would be limited (USACE, 1952). This assumption was wrong; extensive urban, commercial and industrial development has occurred in recent decades. Although Dade County has developed flood protection targets for the Level of Service (LOS) that should be provided, the current LOS for flood protection that actually exists in the watershed is not known.

Water Supply.

Seepage from the Everglades through the highly transmissive Biscayne Aquifer is the major source of freshwater for Miami-Dade County (SFWMD, 2013). The SFWMD maintains surface water levels in coastal canals at specific elevations sufficient to recharge local well-fields and has established minimum water levels for coastal canals needed to prevent saltwater intrusion along the coast (SFWMD, 2014c). During drought periods, SFMWD transfers water via gravity-driven flow through canals, from the adjoining Everglades and in severe droughts from Lake Okeechobee to keep water levels near well-fields at their maintenance elevations. District rules and permits limit the amount of water that can be released from the Everglades for wellfield recharge during drought periods.

Water Quality.

The quality of water in the C-4 Canal has been an issue in development of flood control management options and in terms of water supply. The normal movement of surface and groundwater is from west (out of the Everglades) to east (toward saline coastal waters). Water from the Everglades is of high quality and is the preferred source of recharge to urban well-fields. Because of the direct linkage between surface and groundwater in this region, extensive construction of rockpits and canals provides a means for the groundwater to become contaminated with surface water runoff from urban, industrial and agricultural land uses. For this reason, excess surface water cannot be backpumped into the Everglades (**SFWMD, 1982**), and the potential for contamination of drinking water sources limits the options for routing flow in the canals. Stormwater is therefore either routed to storage reservoirs specifically constructed for this purpose or discharged downstream to seawater through coastal structures.

Need for an Integrated Miami River System Study.

In USACE studies conducted during the 1950s (refs), the Corps recognized that due to interconnections and dependencies, it is not possible to fully address issues in any one watershed without considering the entire system and that was the basis used for design of the Miami-Dade County canal system. Water management in the C-4 Canal is directly linked with management of the surrounding connected canal basins. The C-6, C-2, C-3 C-4 and C-5 watersheds can, at times, discharge water into the Miami Canal, and the C-2, C-3, C-7, C-8 and C-9 Canals, under certain conditions, provide opportunities to discharge excess water from the Miami Canal to coastal areas (SFWMD, 2015a). Ultimately, to address LOS problems in eastern Dade County, all of the watersheds must be analyzed as an integrated hydrologic unit. The District will continue to conduct LOS studies during coming years until the entire region has been investigated.

Available Information, Previous and Ongoing Investigations.

A substantial amount of information is currently available to support the LOS analysis in the C-4 Basin. The area has been studied extensively and hydrologic models are available (refs). A number of regulatory and structural means have been developed over the years to constrain flood damages – most significantly the USACE C-4 Coast Canal Structures Critical Project (1999); the FEMA funded Miami Dade County Flood Mitigation Program for the C-4 Basin initiated in response to a severe flood event in October 2-4, 2000 (Post Buckley et al. 2005a,b); and the coordinated flood operations plan of 2011 (City of Miami ID 11-01191, 2011). The report entitled, Water Control Atlas for the Miami River System, which contains essential information for these LOS studies has been recently updated (SFWMD, 2015a).

DESCRIPTION OF THE STUDY AREA

Major Features

Water Management features and operations of the C-4 Basin are described in the Basin Atlas (**SFWMD, 2015a**). The C-4 watershed has an area of approximately 83.3 square miles and is located in eastern Miami-Dade County (**Figure 1**).

Figure 1. C-4 Watershed.

There are two Project canals in the C-4 watershed: C-4 and the L-30 borrow canal that have three functions: to provide flood protection and drainage for the C-4 watershed; to supply water to the C-2, C-3, C-4, and C-5 watersheds; and to maintain a groundwater table elevation near the eastern reach of C-4 adequate to prevent intrusion of saltwater into local groundwater.

The Tamiami (C-4) canal begins in the east at the junction with the Miami Canal (C-6) and extends west to the intersection of the borrow canal of L-29, L-30 and L-31N. The L-30 borrow canal is aligned north-south along the west boundary of the watershed. The south end of L-30 borrow canal is connected to the west end C-4 by S-335. Flow in the canal is normally to the south. C-4 is connected to three other Project canals: C-2, C-3, and C-5, which are bifurcations of C-4.

1. C-2, the Snapper Creek Canal, makes an open channel connection with C-4. Normal flow is from C-4 to C-2. Water from C-2 is discharged to tide through structure S-22.
2. C-3, the Coral Gables Canal makes an open channel connection to C-4 just east of the Palmetto Expressway. Flow is normally from C-4 to C-3, however, during the dry season a stage below 3.0 ft NGVD in C-4 at S-25B can cause flow from C-3 to C-4. Water from C-3 normally discharges to tide through structure G-93.
3. C-5, the Comfort canal branches from C-4 at Blue Lagoon east of Coral Gables and connects downstream to C-6. Normal flow is from C-5 to C-6.

Topography and soils

Table 1 shows the areal distribution of land elevations within the watershed based on recent LIDAR data **(SFWMD, 2014a**). Nearly half of the watershed has an elevation of 4 ft or less A topographic map of the C-4 Basin is shown in **Figure 2.** From Snapper Creek Canal (C-2) west to L-30, the elevation is approximately 4-6 ft. above msl. The top layer consists of organic peat, marl and fine-grained sand

Table 1. Percentages and areas at different elevations within the C-4 Basin

Elevation (ft)	%	Approx. Acres	Cum Acres	Cum %
-3	0.0	0.23	0.	0.0
-1	1.5	818.75	819	1.5
0	3.6	1923.13	2742	5.1
1	7.1	3799.42	6542	12.2
2	5.6	2982.27	9524	17.8
3	8.8	4705.42	14229	26.6
4	21.0	11216.42	25446	47.8
5	23.6	12581.16	38027	71.4
6	12.5	6684.57	44712	83.9
7	8.5	4540.11	49252	92.5
8	3.6	1930.94	51182	96.1
9	1.3	702.81	51886	97.4
10-11	1.1	613.03	52499	98.5
12-14	0.6	309.50	52809	99.1
15-25	0.6	333.61	53143	99.7
26-78	0.3	175.64	53319	100
TOTAL	99.7	53317.01		

53317 acres = 83.31 square miles; 4.2 square miles or 5.1% of the watershed is at or below sea level; 39.76 square miles or 48% is 4ft or less above sea level; 10.22 square miles or 12.2% is 1 ft or less above sea level; 3.9% or 3.24 square miles is above 8 feet.

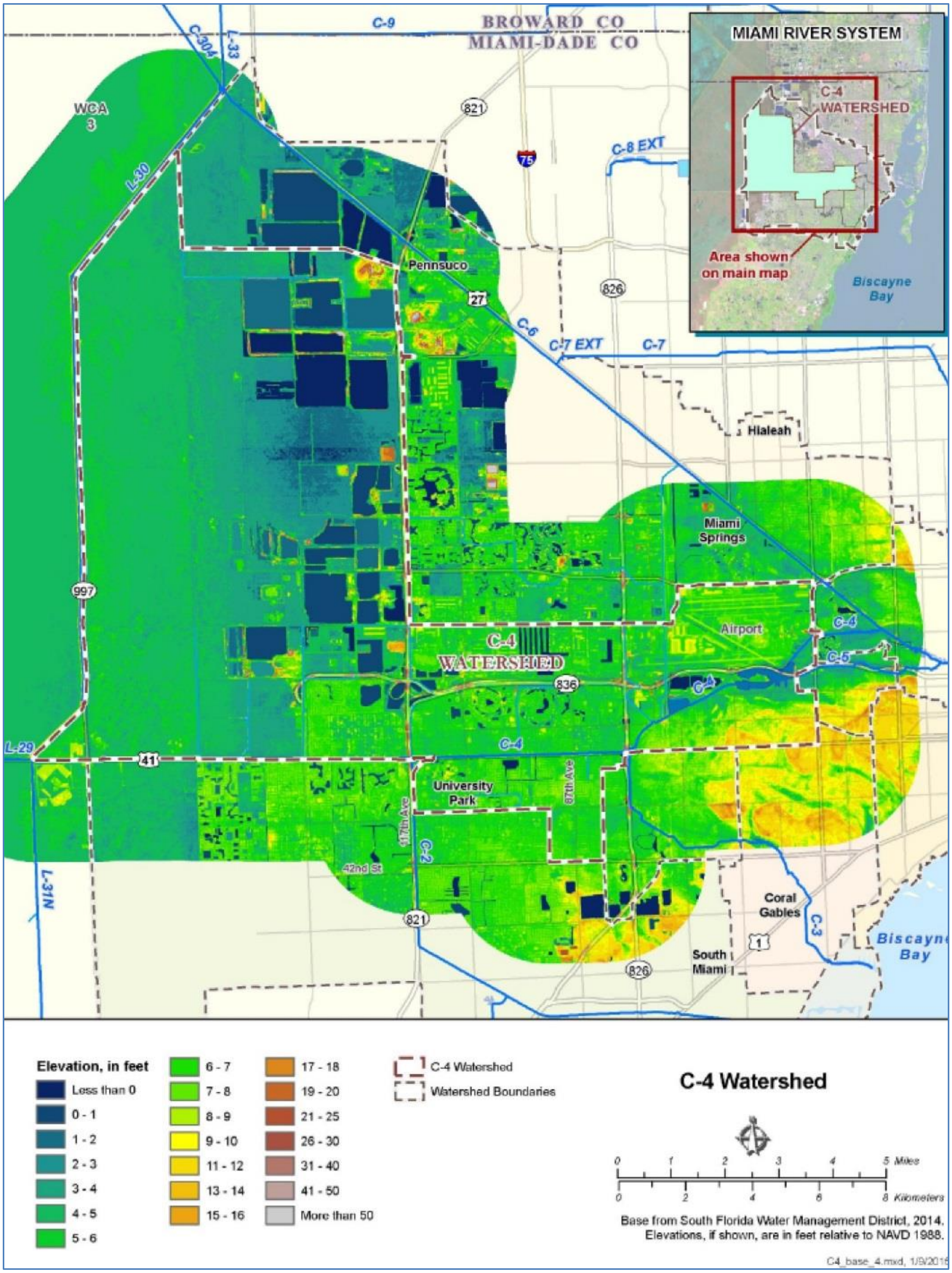


Figure 2. C-4 Basin Topography (SFWMD, 2014a)

with limestone pebbles. The peat overlies a thin marl bed and thins irregularly eastward from a depth of 4 feet at the Dade-Broward levee to a feather edge near Snapper Creek Canal. From Snapper Creek to Coral Gables Canal (C-3), the surface elevation is 6-7 ft, reaching a maximum elevation in the vicinity of Milam Dairy Road (Palmetto Expressway) and Flagler St. Further east near Red Road, land surface elevation declines to about 2 ft above sea level. Soils consist of a 1-6 ft thickness of fine-grained organic sand containing pebbles (**USACE, 1953**)

East of Red Road, the natural land elevation dropped from to 2 ft at Red Road to below sea level at the eastern end of Blue Lagoon. Soils consisted of a thin layer of peat overlying a thin bed of marl. Blue Lagoon has been dredged to a depth of –5 ft and fill from this site and various rockpits in the watershed were used to raise the elevation of lands surrounding the canal from Red Road east to the Miami Canal, to an elevation of about 3 ft above sea level (**USACE, 1953**).

Drainage History

The original canals were constructed in the basin primarily to provide drainage of the eastern lands near the ridge and to provide fill for construction of Tamiami Trail, a road constructed in the 1920s that connects from Miami across the Everglades to Naples. As development expanded westward from the coastal ridge and into the Everglades. The Tamiami Canal and tributary secondary canals were improved to provide drainage for agricultural lands in the western basin and drainage and fill for the Miami Airport. Water control structures were installed to protect against over-drainage and saltwater intrusion.

The USACE project in the early 1950s reexamined the entire drainage network for Dade County. Emphasis of the study was placed on providing additional flood protection for developed lands along the coastal ridge in the eastern portion of the county. These lands were designated as “Area A” on design maps and include most of the land located east of 87th Avenue in the C-4 Basin. The western portion of the county was largely undeveloped or in low-intensity agricultural use. These lands were designated as “Area B.” The USACE determined that the cost of providing full flood protection for the entire basin substantially exceeded the likely benefits over the 50-year projected lifetime of the plan. Therefore, the decision was made to design the system to provide full protection for lands in Area A and less flood protection for lands in Area B, which included most the C-4 Watershed.

Designs were developed in the 1950s and facilities constructed during the 1950s, 1960s, and 1970s. Agricultural lands were later replaced by industrial, commercial and urban development with consequent need for additional drainage and flood protection. Major well-fields were developed in the watershed to help meet water supply needs of Greater Miami (SFWMD, 2014b**)**. Rock mining has been a major activity in the watershed, converting shallow wetlands into deep lakes and using the fill to create roadways, airport runways and elevated land suitable for construction. Despite these improvements, portions the C-4 canal watershed were flooded periodically during severe storms, resulting in significant damage to homes and businesses.

By the 1970s, urban development was rapidly expanding into the western portion of the C-4 Basin and it became clear that was periodic flooding of these lands was a major issue. A number of steps have been taken since that time to reduce flooding risk, including:

- Construction of extensive secondary and tertiary drainage facilities to facilitate removal of water from developed areas and directing water to French drains, lakes and remaining undeveloped wetlands and eventually to the C-4 Canal for discharge to the Biscayne Bay.
- Addition of municipal pumps to reduce local flooding impacts by discharging excess water from adjacent urban sub-basins into the C-4 Canal
- Improved surface water management permitting criteria that require additional onsite retention capacity for stormwater runoff.

- Construction of a reservoir and pumping stations to provide additional storage within the western watershed. (SFWMD, 2014d)
- Construction of the S-380 divide structure to reduce flow of water from the Everglades through the C-4 Canal (USACE, 2002)
- Construction of a 600 cfs pump station adjacent to S-25B to increase the amount of water discharged from the structure. (Wilsnack and Konyha, 2003)
- Construction of a "flood wall" along both sides of the C-4 Canal in the reach from the junction with the C-3 Canal near the Palmetto Expressway to the Junction with the Bird Drive Canal near 132nd Ave.
- Construction of a seepage barrier to reduce groundwater flow into the basin from the Everglades (Bates, 2015)

Despite implementation of these projects, there has not been a systematic reassessment of the primary drainage system in this watershed to determine the level of flood protection that currently exists, whether additional flood management improvements are needed, or whether further development can be accommodated with existing infrastructure, regulatory criteria and management practices.

Water Management System Operations

This section provides a general overview of operations within the C-4 watershed. More details concerning specifications for water management structures can be found in the Water Control Atlas: Miami River System Atlas (**SFWMD, 2015a**) and in the Appendices to this report

Structures:

There are five primary structures controlling flow in C-4. Three C&SF project structures (S25B S25A, S336 and S380) and one SFWMD structure (G-119) control flow in the C-4 watershed. Design criteria for the structures in the watershed are given in **Table 4**. Descriptions are as follows:

G-199 and S-380 are used to maintain stages to create and preserve wetlands and to reduce seepage and drainage out of Water Conservation Area 3B. They also provide supplemental water supply deliveries eastern Miami-Dade County. S-336 is part of South Dade Conveyance System. S25A is a gated culvert that is normally closed but may be opened occasionally to flush water through the C-5 canal when water in C-5 becomes saline S-25B is located just downstream of LeJeune Road, discharges to C-6 and controls water surface elevations in C-4. Water levels in C-4 are a generally maintained at a level adequate to prevent saltwater intrusion into local groundwater. The S-25B Forward Pump Station provides positive drainage during periods when gravity discharge falls below 600 cfs due to high tides.

The C-4 Storm Water Detention Reservoir is an above ground pumped storage area providing approximately 13,000 cubic feet of storage, designed to receive water during a storm event and then release once downstream capacity is available. Two remotely operated pump stations, G-420 and G-422 are available to remove water during the peak of a storm from the west end of C-4 and discharge this water into the detention reservoir. G-420 seepage pump is used to recirculate water collected in the seepage collection canal, back into the southern EDB area. The G-421 Outlet Spillway structure is utilized after a storm event has passed to release water back into to the C-4 to make storage available for the next storm. The G-423 Interbasin Transfer Structure is used on an interim basis to move water into and out of the northern portion of the reservoir.

Two additional structures located outside the C-4 Basin have significant impacts of flood protection in the watershed. G93 passes the design flood flows from the C-3 Basin, plus a small discharge from the C-4 watershed. It also prevents saline intrusion during periods of high flood tides. S-22 is a gated spillway, located in C-2 Canal that controls water surface elevations in C-2 and discharges to tidewater. A headwater stage is maintained by S-22 adequate to prevent saltwater intrusion into local groundwater.

Water Level Monitoring Gages:

T-5W remote terminal unit water level monitoring station located at the NW corner of Florida's Turnpike and C-4 Canal west of the intersection with the C-2 Canal. Data from this are used to regulate operations of the Emergency Detention Basin and in conjunction with MRMS1 for operation of the S25B forward pumps. MRMS1 remote terminal unit water level monitoring station is located south of the intersection of C-4 canal and the Miami River and relates to the operation of the S25B and S26 forward pumps. C-4 Coral is a flow and stage recording station located on the Tamiami Canal at SW 8th ST and 82 AVE. Data from this station is considered when determining whether to release water from the C-4 EDB after a flooding event.

Flood Control Operations

Today, extensive urban development has occurred within the eastern portion of the western basin, while much of the western portion of the original Area B remain as non-urban sub-watersheds with large wetland areas. Heavy industry also occurs in this area, including limestone quarries, aggregates operations, block plants, cement plants and ready-mix facilities.

Much of the need for additional flood protection in the western C-4 Basin has been met through construction and operation of new water management features, improved construction standards and improved water management permitting criteria for new development. However, significant problems still occur during high rainfall periods.

Normal Operations.

As stated in the C&SF design memorandum, the C-4 Canal and the S-25B outflow structure were designed to pass 100 percent of the Standard Project Flood from the urban area located generally east of S.W. 87th Avenue (designated as Area A). The western portions of the C4 watershed (designated as Area B) were not afforded the same level of drainage. Under normal operations, water flows east through the C-4 canal and discharges to tidewater through a spillway structure, S25B, to the C-6 Canal. Excess water can also be released from the C-4 basin by routing additional flow through the C-3 and C-2 Canals if conditions in these watersheds allow.

Localized Flood Conditions.

As urban development spread into the western areas, the need for improved flood control has increased over the years. This additional development has been allowed to occur by implementing home construction and regulatory programs that require higher elevations for construction of roads and buildings and additional retention of water additional within secondary and tertiary drainage systems. In addition, several municipalities have constructed pumping stations that pump directly into the C-4 Canal from the local drainage network of exfiltration trenches that are otherwise disconnected from the C-4 system. Excess flood waters can also be routed to the C-2 and C-3 Canals, if additional capacity is available in these watersheds. Nevertheless, in recent years, widespread flooding has occurred, notably in areas of Sweetwater east of S.W. 117th Avenue, in October of 1999 (Hurricane Irene) and October of 2000 (Tropical Storm Leslie) (**Schweigert et al., 1999**; **Miami-Dade County, 2000**). These events prompted development of regional mitigation projects.

Coastal Pumping

S-25 is the gravity flow structure that provides discharge from C-4 Canal downstream to tidewater. During periods of high rainfall, a pump (S-25B) has been added adjacent to gravity structure to provide for additional discharge from the basin during flood conditions and discharge during periods when the tailwater stage at S-25 is too high to permit adequate gravity discharge.

Western Detention Facility

A reservoir and pumping stations have been constructed in the western C-4 Basin approximately 11.5 miles to the west of the S25B spillway to provide storage of excess floodwaters. Water levels for flood control operations are measured at the gage T5W, an intermediate gage measuring water levels in the C-4 canal west of the intersection of C-2 canal. The western detention basin is located approximately 11.5 miles to the west of the S25B spillway. Water levels for flood control operations are recorded at the gage T5W, which measures water levels in the C-4 canal west of the intersection with C-2 canal.

METHODS

The current study is based on results of a number of investigations within the basin that provided baseline data and assessments. Water management features and conditions in the watershed were recently documented in the updated version of the Miami River Basin Atlas (**SFWMD, 2015a**). In addition to the atlas, a number of other studies are included as appendices and attachments to this report. Basic water management and flood control concepts and the performance measures used to evaluate model outputs are described in **Appendix A.** Detailed hydrologic data needed for the model were compiled from existing databases and in some cases synthesized from additional analyses (**Appendix B**). Future water management scenarios were developed based on projected rates of sea level rise over the next 50 years (**Appendix C**) to provide estimates of future changes in water levels and storm surge at the tidal boundaries. Details of operations at tidal water management structures in the watershed were documented and analyzed for incorporation into the models (**Appendix D**). Existing modeling tools were upgraded to provide tools for analysis of a range of future hydrologic, environmental and water management conditions (**Appendix E**). The models were then calibrated and verified (**Appendix F**) and used to analyze water conditions over a range of rainfall and events and storm surge conditions for existing conditions and three future sea level rise scenarios (**Appendix G**). Results from the analyses were analyzed based on the predetermined performance measures that were developed specifically to assess level of service for flood protection within District watersheds (**Appendix A**)

The boundaries of the C-4 watershed were established for the western edge of the basin at the interface with Everglades Water Conservation Area 3, on the south, where C-4 Canal connects to the C-2 and C-3 canals, and on the north and East at the connections to the C-6 and C-5 Canal basins and the Miami River coastal watershed A basin atlas has recently been completed that compiles recent information about facilities and operations in the Miami River watershed including the C-4 sub-watershed. A hydrologic model of the C-4 Watershed has been developed to simulate water levels and water movement.

Modeling Approach

The C-4 Canal is an integral part of the regional water management infrastructure and, as such is directly linked to adjacent canals and watersheds (C-2, C-3, C-6 canals, the Water Conservation areas, Everglades National Park and areas downstream from the coastal water control structures) and cannot be truly examined in isolation. Due to limitations of existing models, time and resources, this study has attempted to examine the C-4 watershed by itself. Connections to other features were established as boundary conditions to the model. Results of this study may need to be reconsidered after similar studies have been conducted in the remaining Miami- Dade County water management system.

The hydraulic model chosen for this investigation is a canal flow routing model (HEC-RAS) (**HEC, 2010a,b**). A simplified hydrologic approach is used, in which the flows (including surface and groundwater) between sub-basins and canals, are computed as linear functions of the head differentials between water bodies multiplied by a conductance term which is obtained by calibration. The sub-basins in the model are represented as storage units that include soil and above ground storage. The HEC-RAS model receives inflows directly from rainfall and as boundary flows then carries the collected flows along the canals through bridges and control structures to the tidal basin outlets.

The HEC-RAS model was calibrated for a significant wet period (August-September, 2012) during which flood control operations were activated to maintain flood waters out of the basins. After model calibration, the model parameters were used to validate the model with data from Hurricane Irene that occurred in October, 1999 (**Miami-Dade County, 2000**).

The evaluation of current LOS with the calibrated HEC-RAS model involves the routing of inflows and outflows through the sub-basins and canals in the C-4 Basin to an internal storage reservoir and coastal discharge structures. Several pre-determined return frequency storm events were analyzed which include the 5-, 10-, 25-, and 100-yr rainfall events in conjunction with corresponding tide surge conditions, for current and future sea level rise scenarios. The output data from the model were analyzed to determine water levels and flows in the canal, water levels in sub-basins, structure and pump station operations. The level of service for flood protection was assessed by examining a series of five performance measures that were developed specifically for these investigations.

Conceptual Model

Recent modeling efforts by the District in the C-4 basin (**SFWMD, 2009**) indicated that accurate representation of stages requires the use of integrated surface-groundwater models due to the high interconnectivity of the surficial aquifer with the canal system. In that study, the District used a 1-D hydrodynamic canal routing model of the C2, C3, C4, C5 and C6 canals (HEC-RAS) coupled with a 3-D groundwater flow model (MODFLOW) to simulate the exchange of water between the underlying aquifer surficial aquifer and the canals during flooding events (**SFWMD, 2011**). The calibrated version of the combined model was then used to evaluate the effectiveness of flood control operations in the basins. Despite the successful application of the coupled HEC-RAS/MODFLOW model for evaluating operational protocols in the basin, the model had severe limitations due to size, complexity and long run times.

In this project, a simplified approach is used to circumvent the use of coupled surface/ groundwater models in the basin. **Figure 3** shows a schematic of how the model represents the interaction between sub-basin storage and a canal segment in the modified HEC-RAS model. This simplified approach results in fast development and application of the model for evaluation of flood level of service scenarios in the C-4 Basin. The hydrologic processes in the basin are simplified to include only direct rainfall and evaporative losses. Only the primary canal system and some secondary canals built and managed by Miami-Dade County's Department of Environmental Resource Management (DERM) are included in the model. Tertiary drainage features such as infiltration trenches and sewers are neglected. Net inflow (rainfall minus evaporative losses) enter the basin (groundwater and above ground combined) storage without lag time due to surface flow and unsaturated zone routing.

The C-4 Basin area is subdivided into sub-basins that interact with the primary canal system via seepage and canal overbank flows. Sub-basin groundwater storage is a function of soil porosity, which is assumed constant over the sub-basin. Sub-basin above-ground storage includes lake and topographic depression storage, determined from sub-basin LIDAR data (**SFWMD, 2014a**). The seepage flow interactions between sub-basins and from sub-basin to canal are a function of the head differential between the two water bodies and a conductance term that is determined though model calibration. The overbank flow interaction is a function of the head differential between the sub-basin and the water elevation above the canal bank and the length over which the overbank flow occurs.

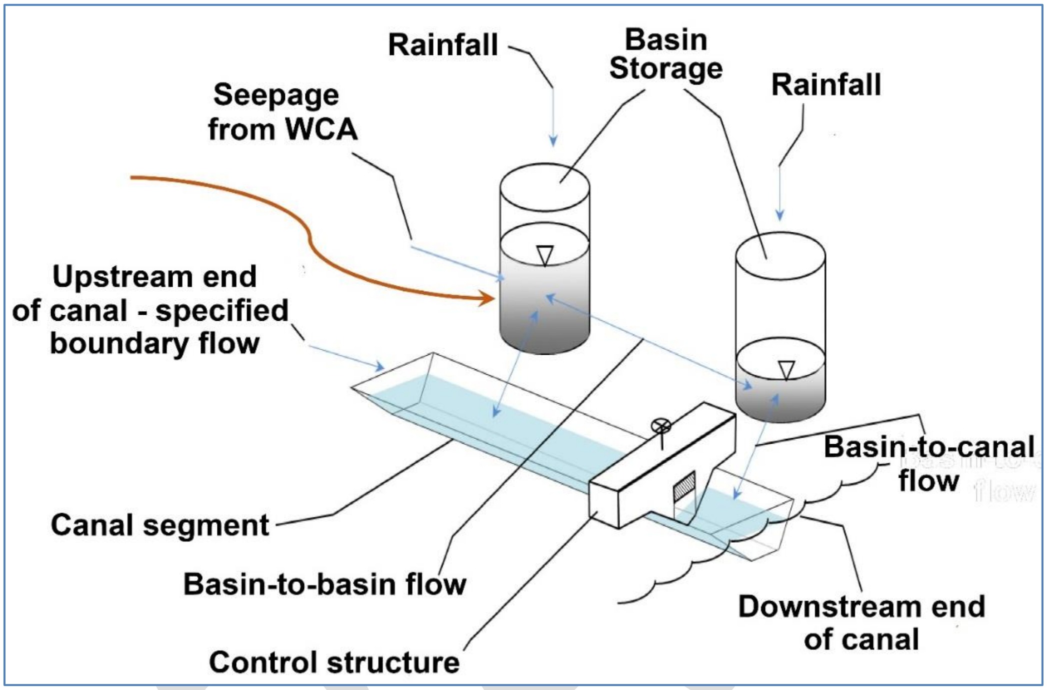


Figure 3. Schematic of Sub-basin and Canal interactions in HEC-RAS Model

Model Selection and Application

The selection of appropriate hydraulic modeling tools for such applications typically depends on local factors such as topography, geology, rainfall, water management practices, land use etc., as well as the objectives of the analysis. For flood control analysis, a number of model codes may need to be applied. These include, but are not limited to, rainfall-runoff models, open-channel flow models, 2-dimensional surface water flow models and integrated ground water / surface water flow models. The C-4 Flood Level of Service Study includes:

- Primary and secondary canal network characterization in the C-4 Canal basin.
- Setup of the hydraulic model of the primary and secondary canals, bridges and control structures.
- Hydraulic model calibration
- Model applications (sea level rise scenarios)

For the C4 Study, a groundwater/storage-driven HEC-RAS (Version 5, beta release) model approach was selected to evaluate the effectiveness of flood control structural and operational measures. The HEC-RAS model, developed by **HEC (2015)** has been extensively tested and applied in projects throughout the world and in South Florida. The modeling approach for this project requires that groundwater seepage to the canal system must be considered during the flood routing.

The combined stage-storage for each sub-watershed in the C-4 Basin was developed using a GIS tool to derive the above ground storage and a spreadsheet program to obtain the soil water storage. The groundwater seepage to the canal—driven by the head difference between the groundwater levels in the sub-watersheds and its neighboring canal water stages—is coded with rules in HEC-RAS. The groundwater exchanges among adjacent sub-watersheds are calculated with rules as well.

The impoundment pump stations are imposed as boundary conditions, using the historical record for the calibration, and incorporating rule-based operations in the application runs. The structure controls at S-380 are implemented in HEC-RAS with the District's rules. The forward pump at S-25B is set up as a boundary condition with the historical record for the calibration and rule-based operations in the application runs.

Model Calibration and Validation

HEC-RAS is a generalized modeling tool used for analysis of hydrologic conditions in a region. In order to adapt the model to a specific area, calibration and validation of the model are required as essential parts of the modeling effort. Details of model calibration and verification are provided in **Appendix F.** Calibration adjusts features of the general model to represent local conditions. Various parts of the model, including the value of model input values, are changed so that measured values (often called observations) are matched by equivalent simulated values and, hopefully, the resulting model accurately represents important aspects of the actual system. Validation compares results of the calibrated model output to actual measurements made in the field as a means to verify model performance. HEC-RAS model calibration consisted of adjusting the value hydraulic parameters, primarily those that directly affect the magnitude of canal stages and flows in the canal system, to match measured values.

Since actual field data are used in model calibration, it is important that the time period during which the calibration data are collected is different from the period used for validation. Both time periods should include hydrologic conditions (e.g. droughts, floods, wet season, dry season) that are representative of the conditions that are of primary interest for the investigation. Data to perform the calibration of the HEC-RAS model for this study, field data were collected during the period of August 7 to September 30, 2012. Validation was performed by simulating the period from October 14 – 31, 1999, during which Hurricane Irene struck South Florida on 14-15.

Historical stage and flow data in the canals were used to compute a set of model parameters that in turn yield computed values of stages and flows in the canals and stages in the sub-basins that closely follow the observed data. The search for a set of model parameters is done with the PEST++ software (**Welter et al., 2015**). The parameter set consisted of lumped flow conductance terms between sub-basins and sub-basins to canals, as well as roughness coefficients in the canals.

Large uncertainties can be expected in the calibrated conductance terms due to lack of observed stage data in the sub-basins (i.e., groundwater level data) and structural errors in the model. Once the calibration objective function has been adequately reduced, a conservative approach using Pest ++ was used that sacrifices some goodness of fit in the calibration of water levels and flows in the canals to obtain more reasonable values for water levels in the sub-basins.

The purpose of the calibrated model is to determine the system response to synthetic storm events of various return periods under existing and future levels of sea level rise. Model run times are very fast leading to relatively easy evaluation of alternative plans.

Rainfall, tidal fluctuations, storm surge and sea level rise

Analysis of flooding impacts is based on as series of assumptions regarding current and future conditions in the watershed. The model accounts for routing of water to the western storage reservoir. This occurs when runoff to the eastern portion of the watershed exceeds the discharge capacity of the coastal structure and water begins to accumulate in the western sub-basins. Water from the western sub-basins is then routed to the reservoir until the reservoir reaches its capacity. After that point, all excess flood water flows to the east.

The primary external drivers of the model that influence flooding are the amount of rainfall occurring over the watershed, and the rate at which water can be discharged from the coastal structures. This discharge rate, in turn is affected by the downstream (tailwater) elevation that varies daily based on tidal and storm surge fluctuations. The tailwater elevation is expected to increase over time as a result of sea level rise and thus the discharge capacity of the coastal structure will decrease accordingly.

Rainfall

High resolution 15-min rainfall data is the main hydrologic component of the C-4 HEC-RAS model. For the calibration period of September-October, 2012, NEXRAD rainfall data of good quality and temporal resolution are available for South Florida. The NEXRAD data are available as gridded data with a minimum temporal resolution of 15 minutes covering a spatial grid of 2 by 2 kilometers cells over the study area. The data were processed to produce 15-minute rainfall hyetographs for each sub-basin as described in **Appendix G**.

A SFWMD 72-hour rainfall hydrograph was constructed for each design storm frequency. These hydrographs are shown in **Figure 4** for the 5-yr, 10-yr, 25-yr and 100-yr design rainfall events, respectively and are based on the assumptions that design rainfall events occur uniformly over the entire C-4 basin and that rainfall occurring over a sub basin is initially retained in that basin.

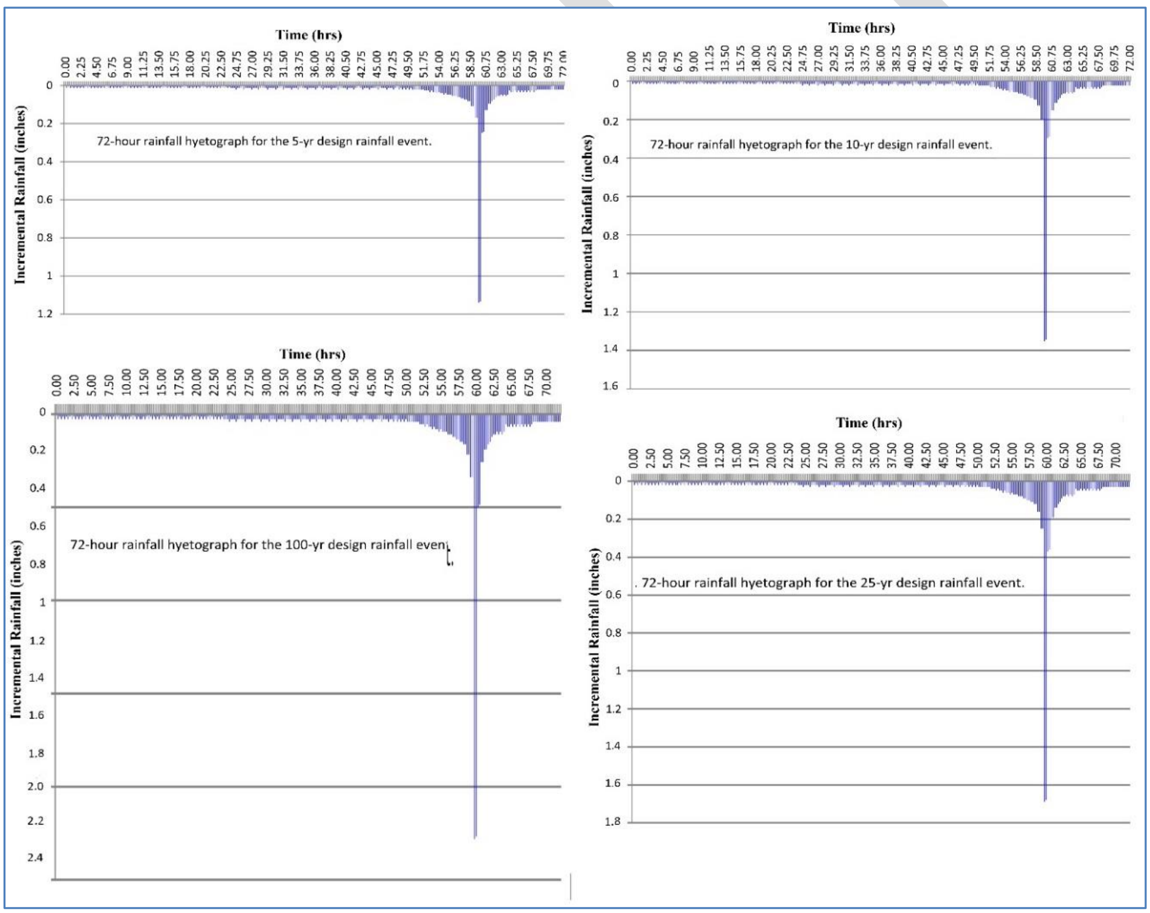


Figure 4. Examples of 72-Hour hyetographs for design rainfall events in the C-4 Watershed

Tidal Data

Details of the process used for analysis of tidal data are provided in **Appendix C.** Boundary conditions, which consisted of the water levels downstream of the coastal tidal water control structures S-25B, S-22 and G-93, were computed for different combinations of storm event return periods, sea level rise projections, and planning horizons. Historical data were compiled, and frequency analyses were performed to determine extreme stages for different return periods. A base storm hydrograph was selected from the historical data and was re-scaled so that its peak reproduces the extreme value derived from the frequency analysis. Offset values derived from the sea level rise projections were then added to the re-scaled hydrographs to produce the final boundary conditions.

The breakpoint archived tailwater stage data for the structures S-25B, S-22 and G-93, were extracted from DBHYDRO (**SFWMD, 2015b**) for the period from June 1985 to September 2014 for S-25B and S-22, and October 1991 to September 2014 for G-93. Headwater stages and gate openings for the structures, and the flow computed using the Districts FLOW program at the structures were also obtained. The flow data at S25B were corrected for negative flows (when the tailwater stages were slightly higher than the headwater stages). Data were further analyzed to create two additional data sets for each structure: a) Daily maxima recorded at the end of the day, and b) Hourly water level recorded at the end of the hour. The R package for statistical computing and graphics (**R Development Core Team, 2008**) was then used to prepare time series plots of the tailwater data for the three structures S-25B, S-22 and G-93. Data outliers were removed manually. Further analyses were performed using the Extreme Values method for weather and climate applications.

Storm Surge - Frequency Analysis for Peak Stages

Details of the process used for analysis of tidal data are provided in **Appendix C.** The Level of Service analysis for watersheds upstream of the coastal structures require the prediction of peak tailwater elevation corresponding to a series of return periods (RP) currently selected as 2, 5, 10, 25, 50, and 100 years. The design peak elevations are referred to as Return Levels (RL). Determination of the return level corresponding to given return period requires an extreme value analysis which consists of assigning appropriate probability distributions to extreme sea levels. Because tailwater elevation records at water control structures contain several components arising from a variety of causes, the frequency analysis is not straight forward and there are multiple ways to deal with the complexity of data. A total of twelve methods were used for computing return levels (see **Appendix C**). Given the observed variability in the results and the lack of criteria at this point to select one of the methodologies as the best, it was decided to use the average of all the methods as the final peak stages. **Figure 5** is a plot showing the return frequency of tailwater stages at the three coastal structures. Peak stages derived from the frequency analyses are presented in **Table 2.**

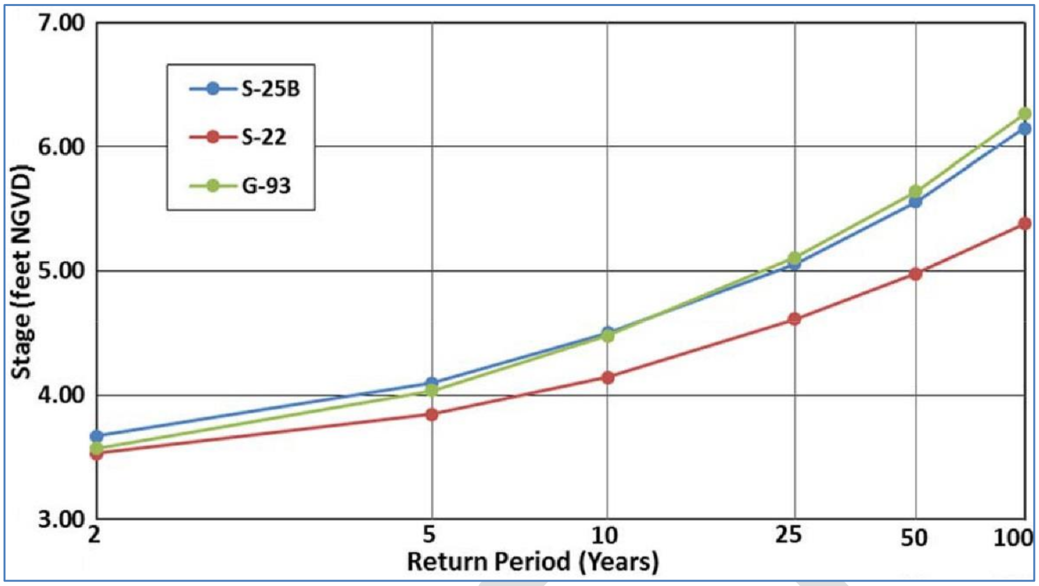


Figure 5. Tailwater Stage Return Levels (feet NGVD) for different Return Periods (RP) at coastal locations in the C-4 basin.

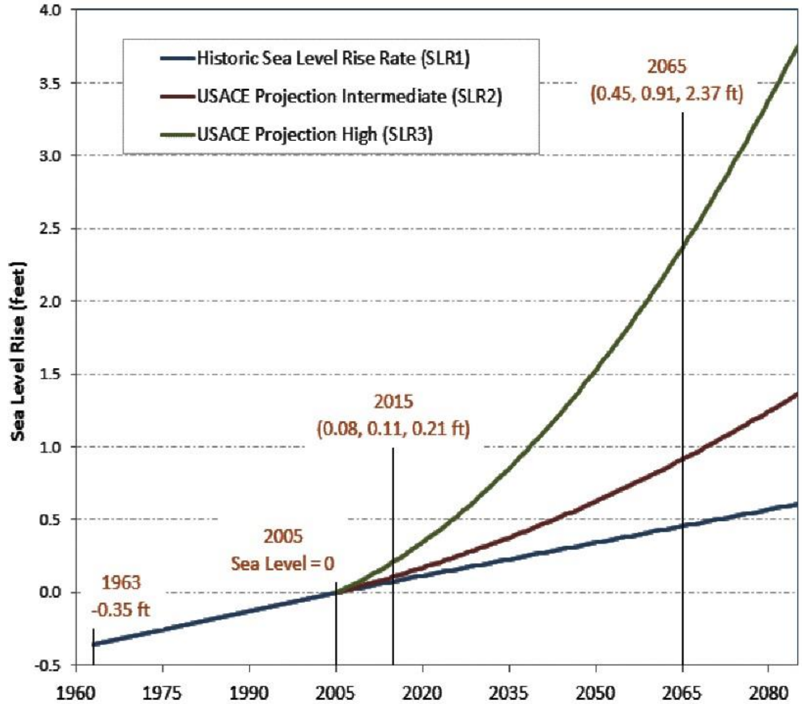
Table 2. Tailwater Stage Return Levels (feet NGVD) for different Return Periods (RP) at coastal locations in the C-4 basin.

RP (years)	2	5	10	25	50	100
S-25B	3.68	4.10	4.50	5.06	5.56	6.15
S-22	3.54	3.85	4.15	4.61	4.98	5.38
G-93	3.58	4.04	4.48	5.11	5.65	6.26

Determination of Sea Level Change

The methodology adopted to evaluate sea-level change (SLC) in the determination of stage hydrographs at coastal structures under the LOS project is taken from ER 1100-2-8162 (**USACE, 2013**). Details are provided in **Appendix C**. The global mean sea level changes by the year 2100 are projected to be 0.53, 1.55 and 4.79 ft. for low, intermediate and high scenarios, respectively. Site specific values, corrected or vertical land movement, were determined based on published or regionally corrected rates that are provided by NOAA.

The global predictions were adjusted to determine sea level change projections in the future for structures S-25B, S-22 and G-93 for 2015 (existing) and future conditions (**Figure 6**).



CSL = Current Sea Level is 0.11 feet above the 2005 base.

SLR#1 is +0.34 feet above CSL

SLR#2 is +0.80 feet above CSL

SLR#3 is +2.26 feet above CSL

1963 sea level = -0.46 feet below CSL

Peak Surge at Structure (feet NGVD29) for 100-y Event

	S25B	G93	S22
CSL	6.3	6.4	5.5
SLR1	6.6	6.7	5.8
SLR2	7.1	7.2	6.3
SLR3	8.5	8.6	7.8

Figure 6. Sea Level Rise Projections used for the C-4 Watershed Study

The Key West Tide gauge record, which covers 102 years and is an active station, was selected for this analysis. The NOAA published rate of 2.31 mm/year for the Key West gauge was used to calculate sea level changes relative to 2005, the mid-point of the project epoch (1996-2014). The tail water boundary condition that existed at the time of construction of the coastal structures (circa 1963) was also determined (see **Appendix C**). The mean sea level change between 1963 and 2005 was calculated as 0.3545 ft.

Boundary Conditions at Tidal Structures

Stage boundary conditions at tidal structures were calculated for six return periods (2, 5, 10, 25, 50, and 100 years), three projected rates of Sea Level Rise (SLR) (historic, medium and high), and two planning horizons – existing (2015) and three future conditions for a total of 36 simulations for each of the three structures. The first step in this process was to select a base storm stage hydrograph from the available period of record. For S-25B, the October 1999 storm was chosen. In the second step, the storm base hydrograph was re-scaled so that the peak stage in the boundary condition hydrograph agrees with the return level for the selected return period. A peak factor is derived as the ratio between the return level for the specified frequency and the observed peak stage in the hydrograph. The final set of tailwater hydrographs for the three structures are presented in **Appendix C**.

Model Development and Features

Sub-basins Delineation

The C-4 model area was divided into sub-basins for this study based on the delineation method used by Miami-Dade County in the development of the XPSWMM model (**Miami-Dade County, 2004**). In their previous hydrologic study, Miami-Dade County used a more detailed surface water modeling approach to examine specific flooding conditions in the watershed under design storm conditions. They divided the C-4 watershed into approximately 400 hydrologic units. For the present study a more generalized modeling approach was needed that gave consideration to both groundwater and surface water conditions, and allowed for a more detailed assessment of water conditions in the primary canal system. The model also needed to allow for consideration of changing boundary conditions for future sea level rise and storm surge scenarios. The District model aggregated the hydrologic units used in the Miami-Dade County study to create 34 sub-basins as shown in **Figure 7**.

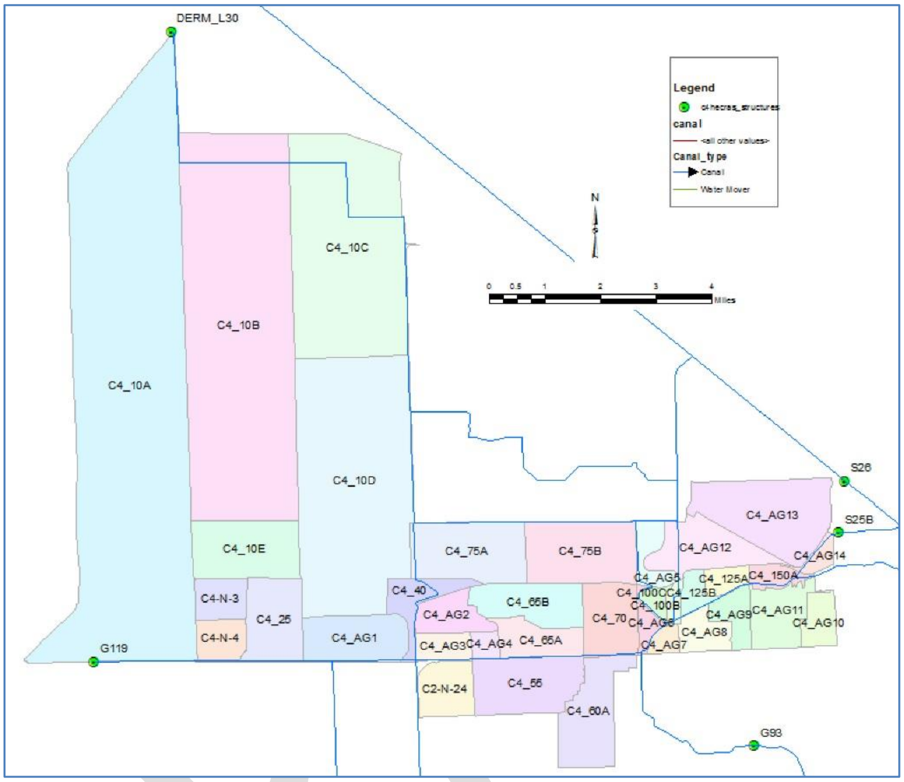


Figure 7. Sub-basin Delineation of the C-4 Basin.

The model domain area also includes the C-2, C-3 and C-5 basins which are modelled, for simplicity, as entire basins represented by a single node. There are a total of 39 sub-basins including the C-2, C-3 and C-5 basins. The total area of the C-4 Basin is 83.3 square miles (53,321 acres) and the largest sub-basin is the Pennsuco Wetland (Sub-basin C4_10A) in the western edge of the basin with an area of 21.2 square miles (13,582 acres).

Canals

The HEC-RAS model includes the District's primary canals C-2, C-2 EXT, C-3 and C-4 and some secondary canals (DERM) including the Florida East Coast (FEC) Railway canal, and the closed tunnel connection through the Miami International Airport, the Northline Canal between C-2 EXT and the FEC canal, the Bird Drive Canal between C-2 and the Krome Avenue borrow canal.

The primary canal network to be modeled in HEC-RAS includes the C-4 canal between the structures G-119 and S-25B and the confluence with the C-6 canal, the C-3 canal upstream and downstream of G-93, the C-2 canal between C-4 and the S-22 structure, the C2-EXT canal north of C-4 between the DERM L-30 structure and the T5 gage. The canal network in this study is shown in **Figure 8.**

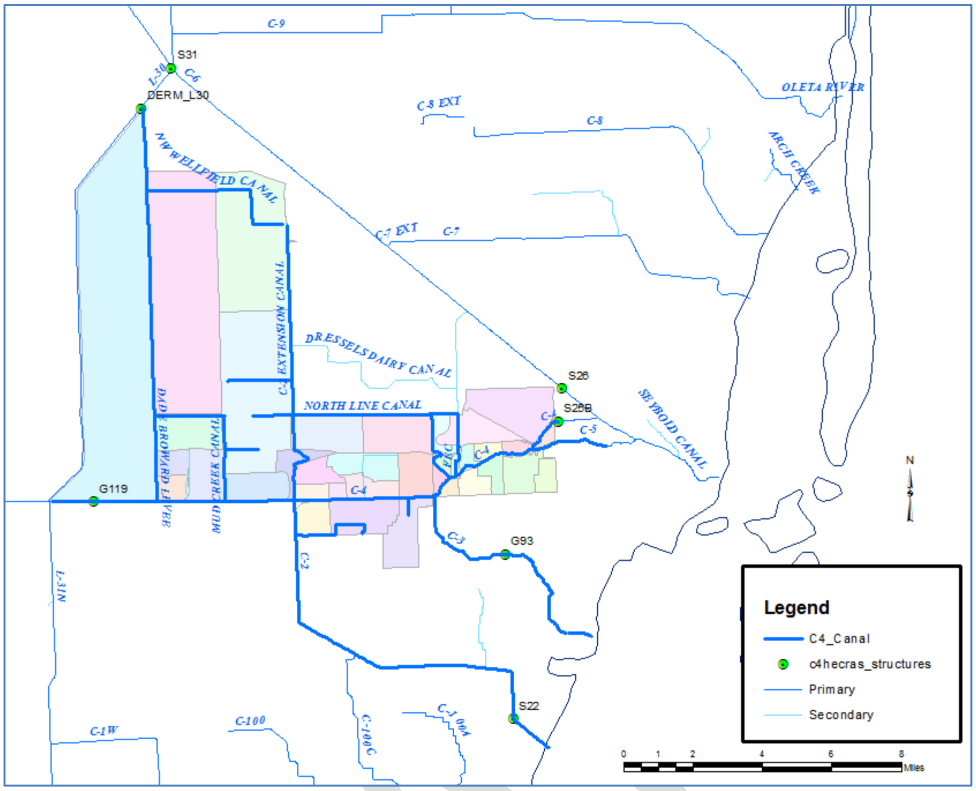


Figure 8. C-4 Basin Primary and Secondary Canal Network

Control Structures and Operations

Currently the District operates twelve control structures in the C-4, and C-5 canals for flood control purposes. General features operations of primary structures for flood control and water supply were described previously. Details of structure design and operation are provided in the appendices

Stormwater Drainage Municipal Pumps

In addition to these regional projects, several municipalities have constructed and currently operate pump stations that collect and discharge stormwater runoff from the local drainage network of exfiltration trenches to the primary drainage system. **Figure 9** below shows the location of these pump stations in relation to the basins and primary drainage system.

The four municipalities that have constructed such drainage facilities include the cities of Miami, West Miami, Sweetwater and Belen. **Table 4** shows the total pumping capacity of each municipal project. Currently the agreement between the District and the municipalities to operate the system during flooding conditions only limits the municipal pumps from operating after the C-4 Impoundment has reached full capacity, at which time all municipal pumpage stops until there is enough capacity in the system.

Groundwater Levels

Groundwater levels in the C-4 Basin are closely related to surface water levels due to the high hydraulic conductivity of the surficial aquifer system (SAS). Additionally, the SAS and canal system are extremely well connected in the C-4 portion Miami-Dade County due to the canals penetrating into the highly permeable Biscayne aquifer. Seepage from the WCAs can also occur west to east as underflow from the WCAs through the protective levees that separate the natural and urban areas. These discharges are in part intercepted by canals (L-30 and L-31) and wellfields located in the eastern, urbanized areas and the S-380 structure in C-4 Canal. The high aquifer transmissivities allow a tertiary drainage network of exfiltration trenches that penetrate the cap rock, routing surface water into the canal via the surficial

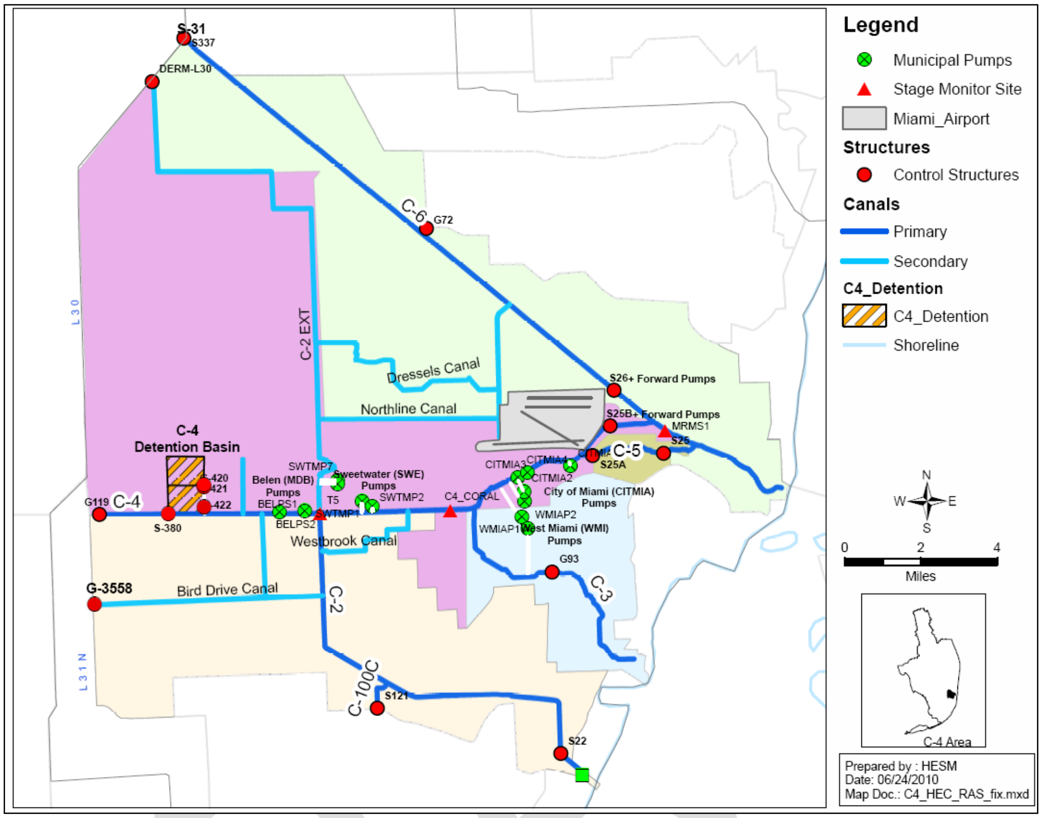


Figure 9. Location of Flood Mitigation Structures in the C4 Basin.

Table 3. Summary of Flood Mitigation Project MUNICIPAL Flood Control Structures for the C-4 Watershed

	ID	Municipality	Structure Type	Basin	Max. Discharge Capacity (cfs)
Municipal Projects	MIA	City of Miami	Pump: to C4	C-4	200
	WMI	West Miami	Pump: to C-3, C-4	C-4	200
	SWE	City of Sweetwater	Pump: to C4	C-4	150
	MDB	City of Belen	Pump: to C4	C-4	200

aquifer. In general, drainage in the secondary canal system is limited by the available capacity in the primary canal system. In addition, a recent study (**Hughes and White, 2014**) estimated increases in groundwater levels in Miami-Dade caused by sea level rise. In the C-4 area the increase ranged from 1.2 feet near the S25B structure to 0.25 feet increase in the far western reaches of the watershed. These stage increases were used to modify pre-storm stages throughout the watershed for future sea level rise scenarios.

Figure 10 shows the conceptual representation of the Basin-to-Basin and Basin-to-Canal flow exchanges in the HEC-RAS model of the C-4 Basin. In this figure, the nodes represent the sub-basins and canal water bodies, the red dashed lines the groundwater flow exchange between sub-basins and canals and the black dashed lines, the basin to basin groundwater flow connectors. Given the availability of stage, flows and gate opening data at the structures, the model calibration task was carried out by imposing gate openings at the structures and allowing HEC-RAS to compute the headwater and tailwater stages as well as the discharge through the structure.

Model Calibration

The model-independent parameter estimation software, PEST++ ver. 3, was used to facilitate the calibration process (**Welter et. al., 2015**). This calibration tool uses industry-standard parameter

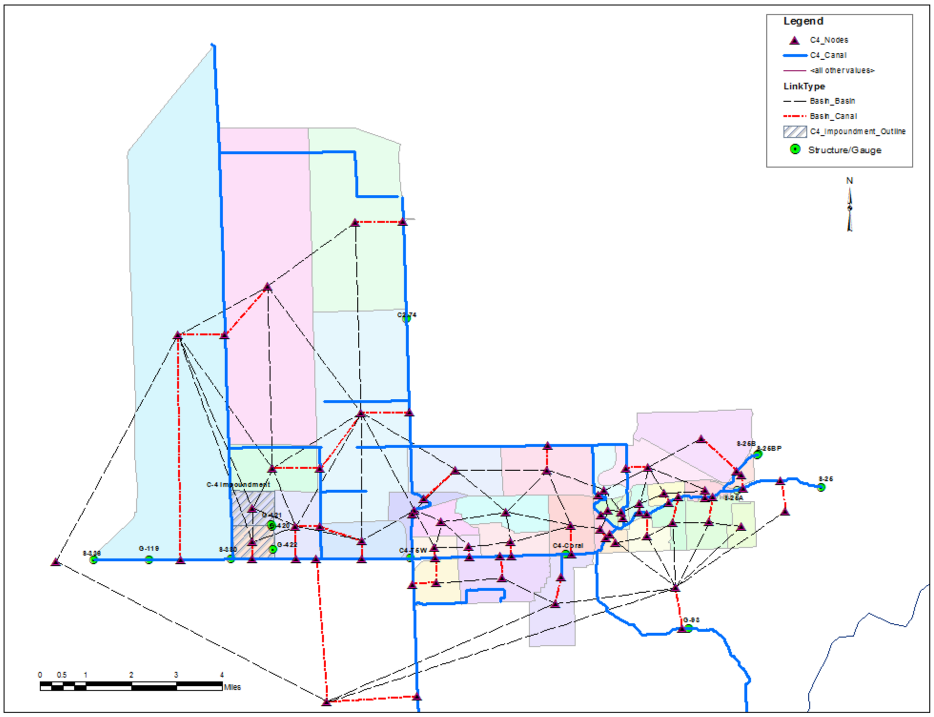


Figure 10. Basin-to-Basin and Basin-to-Canal Flow Exchanges in the C-4 Basin Model

estimation techniques to solve the inverse problem of any model with distributed parameters. The advantages of automated model calibration are numerous and have been widely documented in the literature (**Doherty, 2008**).

The period August 1st to September 30, 2012 was selected for model calibration since this period includes recent enhancements to the system and encompasses two significant storm events. Stage and flow conditions were fully monitored prior, during and after each storm event at all District canal structures providing sufficient data for calibration (**Table 4**).

Table 4. Data sources for model calibration and initial stage data for current and future model runs

Data Sources for Model Calibration

Gauge	Canal	Data Type		Source/Description
S-336	C-4	STAGE	15-min TW	SFWMD / Not used for calibration
G-119	C-4		15-min TW	SFWMD
G-93	C-3		15-min HW and TW	SFWMD
G-420	C-4 Impoundment		15-min HW and TW	SFWMD / Not used for calibration
G-421	C-4 Impoundment		15-min HW and TW	SFWMD / Not used for calibration
G-422	C-4 Impoundment		15-min HW and TW	SFWMD / Not used for calibration
S-25B	C-4		15-min HW and TW	SFWMD
S-25	C-5		15-min HW and TW	SFWMD / Not used for calibration
S-25A	C-5		15-min HW and TW	SFWMD / Not used for calibration
S-22	C-2		15-min HW and TW	SFWMD
T5W	C-4		15-min canal stage	SFWMD
C4.Coral	C-4		Hourly canal stage	USGS
C2.74	C-2 Extension		Hourly canal stage	USGS
S-380	C4		15-min canal stage	SFWMD
Gauge	**Canal**	**Data Type**		**Source/Description**
G-93	C-3	FLOW	15-min	SFWMD
G-420	C-4 Impoundment		15-min pumpage	SFWMD
G-421	C-4 Impoundment		15-min	SFWMD / Not used for calibration
G-422	C-4 Impoundment		15-min pumpage	SFWMD
S-25B	C-4		15-min	SFWMD
C4.Coral	C-4		Hourly	USGS
S-25	C-5		15-min	SFWMD / Not used for calibration
S-25A	C--5			SFWMD / Not used for calibration
S-22	C-2		15-min	SFWMD
S-380	C-4		15-min	SFWMD
Gauge	**Canal**	**Data Type**		**Source/Description**
G-119	C-4	GATE OPENINGS	15-min	SFWMD / Not used for calibration
G-93	C-3		15-min	SFWMD
S-25B	C-4		15-min	SFWMD
S-25	C-5		15-min	SFWMD / Not used for calibration
S-25A	C-5		15-min	SFWMD / Not used for calibration
S-22	C-2		15-min	SFWMD
S-380	C-4		15-min	SFWMD

CALIBRATION: Sep-Oct 2013 - C-2, C-3, C4 and C-5 Canal Stage, Flow and Gate Opening Gauges plus Rainfall and Evaporation

VALIDATION: Oct 14 – 31, 1999 (Hurricane Irene) - no impoundments, pumps, floodwalls.

Initial stages for sub-basins under current and future conditions.

Storage Area	Current Conditions	Future Conditions
C2	3.44	3.74
C2-N-24	3.36	3.81
C3	1.84	2.69
C4-N-3	5.11	5.31
C4-N-4	4.93	5.13
C4_100B	2.66	3.46
C4_100C	2.57	3.37
C4_10A	6.31	6.41
C4_10B	5.32	5.52
C4_10C	4.41	4.76
C4_10D	4.33	4.68
C4_10E	4.95	5.15
C4_125A	2.3	3.15
C4_125B	2.47	3.29
C4_150A	1.96	2.81
C4_25	4.59	4.84
C4_40	3.68	4.08
C4_55	3.04	3.64
C4_60A	2.65	3.35
C4_65A	3.12	3.72
C4_65B	3.15	3.75
C4_70	2.83	3.53
C4_75A	3.36	3.86
C4_75B	2.86	3.51
C4_AG1	3.99	4.34
C4_AG10	1.75	2.65
C4_AG11	1.93	2.78
C4_AG12	2.39	3.21
C4_AG13	2.02	2.92
C4_AG14	1.64	2.54
C4_AG2	3.51	3.96
C4_AG3	3.49	3.94
C4_AG4	3.3	3.8
C4_AG5	2.63	3.43
C4_AG6	2.7	3.5
C4_AG7	2.6	3.4
C4_AG8	2.39	3.21
C4_AG9	2.21	3.06
C5	1.5	2.4

The objective function to be minimized by PEST++ is the weighted sum of the squared residual errors between the computed and measured average hourly canal stages and flows in the model domain. However, other error statistical measures such as error bias and the root mean squared error (RMSE) are used in the presentation of the results. Bias or mean error indicates whether simulated values tend to be disproportionately overestimated or underestimated when compared to historical measurements. The closer the bias is to zero, the better the model prediction. This model calibration metric indicates the presence of systematic error in model predictions that causes all computed values to deviate from the measured values by a consistent amount and in a consistent direction (higher or lower than). The error bias and RMSE are used to compare the relative goodness of fit of each gauge against each other and against the pre-established calibration targets for canal stages.

The main difference between the calibration and application versions of the HEC-RAS model is in the operations of control structures. In calibration mode, the HEC-RAS model uses observed gate openings and observed pumpages to compute flow through control gated structures and pump stations. In application mode, the HEC-RAS model uses rule-driven operations to determine the necessary gate openings and pumpage needed to control water levels according to the flood control plan in the basin.

Generally, gated structures have pre-established control triggers or water level targets to open, hold steady or close the structures' gates or turn pumps on or off. These pre-established operations were programmed into HEC-RAS as user defined rules or as standard HEC-RAS gate operations.

Data needed to specify initial condition for model simulations consist primarily of water levels and flows in or out of the system. Typically, water levels can be estimated from observations (**Table 4**), however, initial flows have to be estimated. In a complex system such as the C-4 basin, initial total flow can be assumed to equal observed base flow at the S-25B structure during dry periods, and then apportioned according to the size of each sub-basin. Initial water levels for the sub-basins and canals were approximated by interpolation from sparse observed values from a few recorders in the basin. Due to the lack of data, the initial water levels were treated as calibration parameters in the calibration process. Upper and lower bounds for the parameters were determined using the observed data. Initial flows were assumed to equal zero. The model was allowed to generate values by running for sufficient time prior to the window of interest to allow water levels to stabilize. Starting the model calibration on August 1st, allows for quick model stabilization since no rainfall occurs until August 3rd, 2012.

Model Validation

Validation of the calibrated HEC-RAS of the C-4 Basin model consisted in simulating the basin and canal flows and stages for one of the largest storm events in recent history, Hurricane Irene which occurred on October 14-15, 1999. The current conditions version of the HEC-RAS model was developed to reflect the current infrastructure and operations currently used to maintain water levels in the basin below those that could result in structural damage, particularly in the flood prone areas mentioned above. Removing the structures that were not in operation in 1999, resulted in a simpler version of the HEC-RAS model with only structures G-119 and S-25B in the C-4 Canal, S-25A and S-25 in the C-5 Canal, G-93 in the C-3 Canal and S-22 in the C-2 Canal. Rainfall data consisted of 15-min values recorded near Miami International Airport. The selected validation period was October 14 – 31, 1999 with the storm event occurring on October 14-15.

Performance measures

Level of Service

The level of service (LOS) of flood protection for a water management network reflects the amount of protection provided for specific features within a watershed (see **Appendix A**). A series of six metrics are applied by the SFWMD to quantify this protection. The first four metrics apply to the primary water management system and measure the amount of flow released from the structures under design conditions, the peak flow capacity of the structure, and water levels upstream of the control structure. The fifth metric measures the ability of secondary and tertiary water management facilities to maintain pre-established water levels that are needed to protect features within the subwatersheds. The sixth indicates the ability to maintain suitable water levels within the primary canal system

Typically, there is no single LOS for a watershed but rather a family of features each with its own LOS. For example, local stormwater infrastructure in residential neighborhoods is generally designed for a storm event with a 20 % annual exceedance probability (a so-called "5 year storm"). The floors of residential homes, however, should be protected during a storm with 1% annual exceedance probability (a "100-Year storm"). The primary canal network may be able to accommodate a storm with a 4% annual exceedance probability (a "25-year storm"). Thus, in the case of a major flood event, the expectation is that the roads and grounds of properties may become flooded while house pads are not. An example of how such level of service criteria might be expressed for a watershed is provided later in this report

This protection is achieved through the combined effects of primary, secondary and tertiary water management infrastructure at a particular location within the watershed. Each of these infrastructure levels serves a different function and thus has different specifications for their system. The secondary and tertiary water management infrastructures are designed to maintain stages needed to protect physical features within the watershed. The primary system infrastructure is designed to provide the water levels in major canals and flow capacity needed to remove the water discharged from secondary and tertiary systems. The three levels of infrastructure working in concert provide the overall levels of service needed throughout the watershed.

The primary water management system in most south Florida watersheds consists of the canals and facilities that are operated by the SFWMD. These canals receive inflows from secondary and tertiary water management facilities operated by local governments and water control districts. The level of service for the primary water management system is referred to as the "**primary level of service,**" and is determined as the amount of flow conveyance capacity (**Q**) that is needed to discharge water to regional storage or to tide to avoid flooding while not exceeding the design maximum water levels in the canals.

Measuring Effects on Level of Service

Five performance metrics have been developed to quantify the flood protection Level of Service (LOS). Two focus on the primary canals, two focus on the impacts of sea-level rise and one examines frequency of flooding on developed lands. Details and Examples of these performance measures are provided in **Appendix A.**

The hydraulic models are used to analyze each of the scenarios for a range of design storm conditions. In order to determine how the level of service is affected under current and future conditions, data resulting from application of the model are analyzed to produce tables and graphs that represent conditions in the watershed under each scenario. Scientists study and interpret these model "outputs" as ways to assess the extent of flooding and potential damage that may occur.

Performance of the regional water management facilities under current and future conditions is assessed based on a determination of whether the flow capacities of the primary canal system and at the downstream structures meet their design specifications and whether water levels within different parts of the watershed are maintained at or below the levels specified for the design storm. This performance is quantified using four LOS performance metrics: two for the primary canal system and two for the tidal structures:

LOS Metric #1: Maximum stage in primary canals

LOS Metric #1 is the peak stage profile in the primary canal system. This profile is developed for a range of design storms. The largest design storm that stays within the canal banks (with a prescribed freeboard) establishes the LOS of the primary canal system.

During large rainfall events the runoff generated from the watershed may exceed the design discharge capacity of the primary system, raising water levels within the canals. The capacity to remove water from the watershed may be limited either by the conveyance capacity of the canal or the discharge capacity of the water control structures or both. In either case, water levels in the canals increase beyond their design stages, reducing the capacity for secondary canals to discharge to the primary system and increasing the possibility of local flooding.

This performance metric is based on interpretation of the results of model runs to determine upstream water levels in the canals during design storm events. These values are compared to bank elevations of the primary canals, adjacent land elevations, design elevations of secondary water control structures, and elevations at strategic monitoring sites within the watershed that are used in District operations to manage operations of flood control pumps, structure and reservoirs.

Future increases in sea level are expected to result in higher groundwater stages in the watershed, which will lead to even higher stages in the canals and more runoff from the watershed, increasing the likelihood that flooding will occur during storm events.

LOS Metric #2: Maximum Discharge Capacity throughout the Primary Canal Network

LOS Metric #2 shows the maximum discharge capacity throughout the primary canal network. Discharge is shown as aerially weighted flow (CSM or cubic feet per second per square mile). Tidal effects are eliminated by using a 12-hour moving average of flow.

Based on the C&SF system capacity as defined in the USACE reports, SFWMD has established discharge rates for basins throughout the District (**SFWMD ERP Manual, 2014**) that provide the basis for issuing Environmental Resource Permits. These rates are expressed as aerially weighted flows (CSM or cubic feet per second per square mile) and are associated with a design level of service. The rates are used to size discharge structures within the basin. In many cases these discharge rates were established as part of the design of the primary water management system.

LOS Metric #2 is based on analyses of the results from a hydraulic model of the watershed to quantify the discharge capacity throughout the primary water management network and for the entire watershed. These modeled discharge rates can be compared to permitted discharge rates. Discharge capacity can vary within the watershed due to changes in land use or conditions in the primary water management network, such as constrictions in the depth or width. Used together, LOS Metrics #1 and #2 can distinguish between areas with low internal water management and areas where the primary canal network has limited conveyance.

LOS Metric #3: Structure Performance – effects of sea level rise

Metric #3 shows the effective capacity of the tidal structures and is comparable to the static, design condition assumed in the original structural design. Metric #3 compares structure flow over a range of storm surge events and a range of sea level rise scenarios. Such comparisons are used to identify tidal structures in need of eventual modification and help prioritize the need for redesign relative to other structures.

As sea-level raises tail-water elevations, tidal structure flows may be suppressed. PM #3 shows the impact of sea level rise and storm surge at a structure. The metric is developed from simulations of structure flow during conditions when the upstream stage is fixed at the design headwater stage and the downstream stage oscillates with the tide. Structure performance based on this metric can be compared to the original structure design, which was based on the cross-sectional area of the structure opening, the design upstream water level and the maximum observed downstream water level.

In current modeling efforts, the tide cycle is considered along with transitions from gravity flow to pumped flow at some structures. With dynamic modeling, flows vary throughout the tide cycle even when though headwater is constant. Consequently, average flow over the tide cycle is used to determine effective capacity and to compare to design flows.

The structure capacity values without storm surge can be compared to one another to show the effects of sea-level rise alone. These impacts would be observed each year during the king-tide period

(Sep-Nov), irrespective of storm surge. Storm surge further reduces the structure capacity. The surge effect is limited to the duration of the surge (less than one tide-cycle) but nevertheless greatly reduces structure discharge rate. As sea level rises, smaller and more frequent storm surges have the same impact on structure capacity that large and infrequent storm surges had when the sea level was lower.

LOS Metric #4: Peak Storm Runoff - effects of sea level rise

Metric #4 shows the maximum conveyance capacity of a watershed at the tidal structure for a range of design storms. It shows the maximum conveyance (moving 12-hour average) for a specific design storm and a specific tidal boundary condition. This metric examines the relative sensitivity of the system to sea-level rise and storm surge.

Metric #4 examines peak storm runoff at the tidal structure. Unlike Metric #3, which focuses on the structure and tidal conditions, Metric #4 looks at the response of the entire watershed to design events. For the design event, design rainfall and design storm surge are assumed to occur simultaneously.

A full watershed model is needed for Metric #4, even though the metric only examines flow at the tidal structure. Metric #4 shows the maximum conveyance capacity of a watershed at the tidal structures for a range of design storms. Metric #4 is based on examination of a range of tidal boundary conditions but this metric looks system response to design rainfall as well as design tide, including the effects of canal conveyance and watershed hydrology.

Sea-level rise indirectly affects available capacity of the canal because the higher sea level reduces tidal seepage (i.e. groundwater flow to tide) and redirects it to canal seepage. This results in higher base flow into the canals and, during design storms, the higher base flow lowers the available capacity of the canal. Changes in design rainfall can also change the design storm. Design rainfall can change for two reasons: reanalysis of existing rainfall records and changes in rainfall intensity associated with climate change. A reanalysis of existing rainfall records is being considered and the resulting impact on LOS can be assessed when new design rainfall data become available.

LOS Metric #5: Frequency of Flooding -- stage-based LOS for sub-watersheds:

Metric #5 is a table that shows the duration of flooding in a developed sub-watershed; that is, the amount of time that stages in each sub-watershed exceed defined LOS targets. LOS targets include both a stage target and a frequency at which water levels exceed that stage. This performance measure is used to compare local expectations of flood protection with regional system performance. These performance measures are evaluated in the sections that follow.

LOS metrics 1 through 4 define the level of service in the primary canal system. However, higher stages in the primary canals can result in higher flood depths on the landscape. Stage-based metrics are needed to show these impacts. In urban areas, local governments may establish desired levels of flood protection for communities. However, due to lack of data for older developed areas, the elevations of existing structures in many parts of the county are unknown and therefore the level of protection provided for these facilities can only be determined during a severe storm and/or after damage has occurred.

For many areas within the C-4 watershed, elevations of existing structures are unknown. To address these uncertainties, an estimate of potential risk to facilities was made based on existing land surface elevations within the sub-watersheds. This criterion is referred to as the ***inundation threshold elevation.*** Land surface elevations were obtained from LIDAR data. Known lakes, wetlands and retention areas were removed from the analysis. The statistical distribution of the range in land elevations was analyzed and an arbitrary decision was made that damage may occur in the watershed if water levels were greater than the 20th percentile of these elevations. For example, let's say that examination of the LIDAR data showed that the range of elevations within a particular sub-watershed was from 6 ft to 8 ft. Statistical analysis of these data determined that 20% of this land was less than 6.5 feet in elevation and 80% was above 6.5 ft. Therefore, significant flooding was deemed to possibly occur in the sub-watershed when water levels exceeded the ***inundation threshold elevation*** of 6.5 feet.

The fifth metric determines the water levels that may occur within sub-watersheds under design storm conditions. The water levels needed within the sub-watersheds are defined by local interests that provide and operate the secondary and tertiary water management systems. These water management systems are sized and designed to control water levels and distribute excess water within their jurisdictions to protect features such as roads and buildings from flooding. Local interests define these maintenance water levels under different design storm conditions, with the assumption that during such storms, a certain amount of water will discharge to the primary system.

Hydrologic modeling is used to verify that during a design storm the primary canal has the capacity to receive the designated amount of water from secondary systems and that, under these conditions, the water level in the sub-watershed will be at or below the level specified by the secondary water management system design. For example, in Miami-Dade County, five flood protection goals have been established by county policy for each sub-watershed within the C-4 watershed.

LOS Metric #6: Duration of Flooding – effects of sea level rise:

Metric #6 shows the duration of flooding in the primary canal network for a range of design storms and a range of sea level rise scenarios. Like metrics 1 through 4, LOS metric 6 relates to the level of service in the primary canal system. This metric compares the recovery time of a range of flood events. The metric looks at water levels at a specific location in the canal system and tracks the time that water levels are above a target canal stage. The location and the target stage are unique to each watershed and are selected by District water managers. The target stage is substantially higher that the water control elevation for the system and indicates a flood state, i.e. a state where all control structures are being operated to maximize drainage.

RESULTS

Numerical results of the model runs are produced as table of numbers that represent time series of water levels ad flows at different locations throughout the C-4 watershed. These data were further analyzed to represent the six performance measures described above that characterize the level of service for flood protection.

LOS Metric #1: Maximum Stages in the Primary Canals

LOS Metric #2: Maximum Flow Capacity throughout the Primary Canal Network

LOS Metric #3: Structure Performance – Effects of Sea Level Rise

LOS Metric #4: Peak Storm Runoff - Effects of Sea Level Rise

LOS Metric #5: Frequency of Flooding - Stage-Based LOS for Sub-Watersheds

LOS Metric #6: Duration of Flooding

1. Maximum Stages in Primary Canals (updated)

Four sets of maximum water surface profiles were computed for the C-4 canal. Each set of water surface profiles contains the maximum water surface profiles resulting from the 5, 10, 25 and 100-year storm events. One set reflects current conditions while the remaining sets pertain to future conditions with the three different sea level rise projections. The low level of future sea level rise (SLR1) is 0.45 feet higher than the 2005 sea level. The intermediate level (SLR2) 0.91 feet higher and the highest level (SLR3) is 2.37 feet above 2005 sea level. Level of service comparisons between current and future conditions are given below.

Current Conditions

Figure 11 shows the set of maximum water surface profiles for current conditions, represented by purple, green, red and blue lines, representing the combined effects of rainfall and storm surge for the 1-in-5, 1-in-10, 1-in 25, and 1-in-100 year events, respectively. Shown also as thinner pink lines in **Figure 11** are inundation threshold elevations for the adjacent sub basins (situated both north and south of the canal). Local floodwalls have been constructed adjacent to the canal along the approximately five-mile long reach from the junction with the C-3 Canal, located 30,000 feet west of S-25B, to the Bird Drive Canal at 132nd Ave., located approximately 60,000 feet west of S-25B. Local floodwall top elevations are represented by the heavy yellow lines on **Figure 11**.

To facilitate interpretation of these plots, reference points were established to represent water conditions in different segments of the canal. Water levels predicted by the model to occur within the canal at the six reference points, for each design storm, are summarized in **Table 5**.

Structure and Eastern Basin. S-25 B maximum tailwater elevations range from 4.2 feet for a 1-in-5 year storm event to 6.3 feet for a 1-in-100 year event, resulting in a corresponding head water level range of 4.6 feet to 6.7 feet. Near Pump Mia1, (site ②), water levels increase from 4.9 feet for a 1-in-5 year storm to 7.2 for a 1-in-100 year storm event. Further west in the eastern basin at 72nd Ave. (site ③), water levels range from 5.2 feet for a 1-in-5 year storm event to 7.4 feet for a 1-in-100 year storm.

Western Basin. The highest water levels in the canal typically occur near the Palmetto Expressway at site ④, since areas to the east drain to tide and areas to the west drain toward the impoundment. Under current conditions, water levels range from 5.6 feet during a 1-in-5 year event to 8.0 feet during a 1-in 100 year storm event. Further west at 137th Ave, site ⑤near the reservoir, water levels range from 5.8 feet during a 1-in- 5 year storm to 7.1 feet during a 1-in 100 year storm. Water levels in the wetland and rock mining areas west of the S-380 divide structure (site ⑥) range from 6.4 feet in a 1-in-5 year storm event to 8.1 feet during a 1-in-100 year storm.

Summary: Results of this analysis indicate that the current level of service is 1-in-10 for the eastern sub-basins and western sub-basins south of the canal. The LOS is 1-in-25 for most of the sub-basins north of the canal.

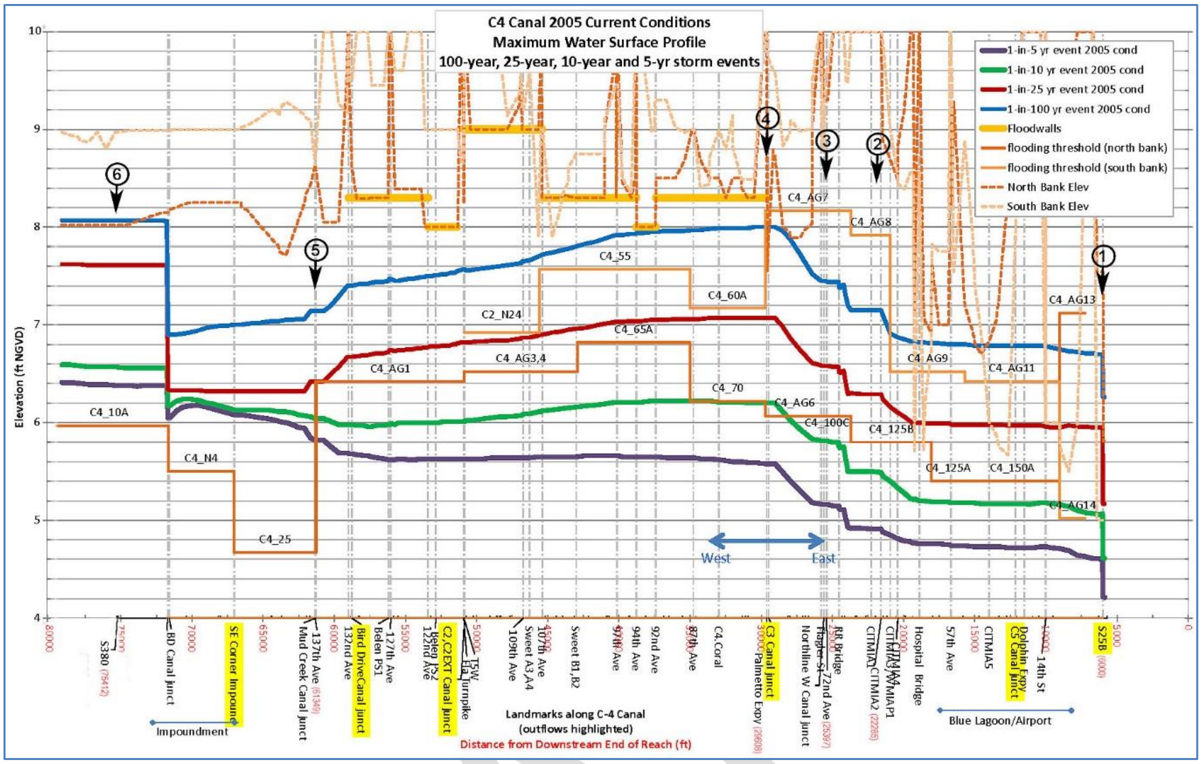


Figure 11. Maximum C-4 water surface profiles for current conditions. Numbers in circles refer to locations described in the text and Table 5.

Table 5. Water levels in feet (NGVD) predicted by the model to occur at reference site locations (numbers in circles refer to points labeled on Figure 11) under current conditions for various design storm events.

Design Storm Return Period (yrs)	①S-25B Structure (6000)*		②City of Miami Pump Mia1 (22286) (ft NGVD)	③East of C3 Near 72 Ave (25397) (ft NGVD)	④Palmetto Exwy (29608) (ft NGVD)	⑤137 Ave (61349) (ft NGVD)	⑥W of S-380 (75412) (ft NGVD)
	Tailwater (ft NGVD)	Headwater (ft NGVD)					
5	4.2	4.6	4.9	5.2	5.6	5.8	6.4
10	4.6	5.1	5.5	5.8	6.2	6.0	6.6
25	5.2	6.0	6.3	6.6	7.1	6.4	7.6
100	6.3	6.7	7.2	7.4	8.0	7.1	8.1

*numbers in parenthesis refer to distance (feet) west (upstream) of S-25B structure; reference site numbers in circles refer to sites designated on **Figure 11**).

SLR1 Conditions

Figure 12 shows maximum water surface profiles for SLR1 compared to current sea level conditions. Water levels predicted by the model to occur within the canal at the six reference points shown in **Figure 5**, for each design storm, are summarized in **Table 6.**

Structure and Eastern Basins.

At site ① (see **Figure 12**) S-25B tailwater levels increase by about 0.3 feet between current and SLR1 conditions (**Table 6**) resulting in and 0.3 to 0.4 foot increase relative to current conditions in

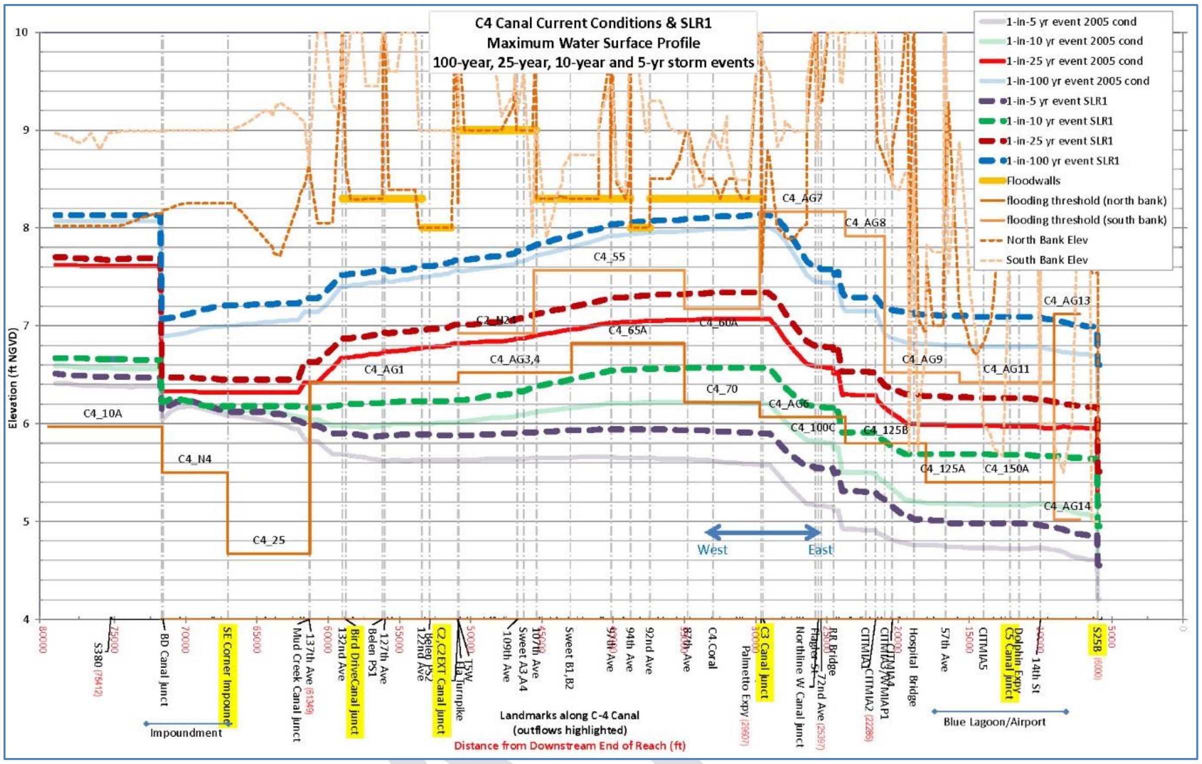


Figure 12. Maximum C-4 water surface profiles for the lowest sea level of future rise (SLR1) conditions and various design storm events.

Table 6. Water levels in feet (NGVD) predicted by the model to occur at reference site locations (numbers in circles) shown on Figure 12 under the lowest projection of future sea level rise (SLR1) conditions for various design storm events.

Design Storm Return Period (yrs)	①S-25B Structure (6000		②City of Miami Pump Mia1 (22286) (ft NGVD)	③East of C3 Near 72 Ave (25397) (ft NGVD)	④Palmetto Exwy (29608) (ft NGVD)	⑤137 Ave (61348) (ft NGVD)	⑥W of S-380 (75000) (ft NGVD)
	Tailwater (ft NGVD)	Headwater (ft NGVD)					
5	4.6	4.9	5.3	5.6	5.9	6.0	6.5
10	5.0	5.6	5.9	6.2	6.6	6.2	6.7
25	5.5	6.2	6.5	6.8	7.3	6.6	7.7
100	6.6	7.0	7.3	7.6	8.1	7.3	8.1

***** numbers in parenthesis refer to distance (feet) west (upstream) of S-25B structure; reference site numbers in circles refer to sites designated on **Figure 11**

headwater levels 1-in-5 year storm conditions. A 0.6 foot increase in headwater level occurs during a 1-in-10 year event, and a 0.3 foot increase relative to current conditions occurs during 1-in-25 and 1-in-100 year storm events. Similar changes occur at intermediate locations in the eastern basin. For the 1-in-100-year storm event, water levels west of site ② are only about 0.1 foot higher than current conditions.

Western Basins. At the Palmetto Expressway, site ④ on **Figure 12**, the effects of SLR1 result in a 0.3-0.4 foot increase in flood elevation (**Table 6**) relative to the current condition for the 1-in-5 and 1-in 10 storm events, 0.2 feet for the 1-in-25 year storm event, and 0.1 feet for the 1-in-100 year storm event. At 137th Ave (site ⑤), water levels increase by 0.2 feet under SLR1 conditions. Water levels west of S-380 are not significantly affected.

C-4 Canal. With an increase in sea level of 0.34 foot (see **Appendix C**), water levels in the C-4 Canal exceed adjacent inundation thresholds in during a 1-in-10 year storm event in several sub-watersheds located north of the canal, and throughout most of the C-4 watershed during a 1-in-25 year storm event. Since levels in the canal are generally the same as water levels in the adjacent drainage basins, localized flooding may occur in low-lying areas.

Level of Service. The level of service for most of the sub basins north of the canal diminishes to 1-in-5 years with SLR1 conditions while, south of C-4, the level of service diminishes to 1-in-10 years for sub basins C2_N24 and C4_60A. Sub basins C4_55, C4_AG7, C4_AG8, C4_AG9 and C4_AG11 still maintain a 1-in-25 year level of service. On the other hand, sub basin C4_AG13 (located north of C-4) appears to be protected against the 100-year storm event while C4_AG14 is only protected against the 1-in-5 year event.

SLR2 Conditions

Figure 13 contains the set of maximum water surface profiles for SLR2 conditions. Also shown for comparative purposes are the maximum water surface profiles associated with current sea level conditions. Water levels predicted by the model to occur within the canal, for each design storm, are summarized in **Table 7.**

Structure and Eastern Basin. S-25B (site ① on **Figure 13**) maximum tailwater elevations range from 5.0 feet for a 1-in-5 year storm event to 7.1 feet during a 1-in-100 year event, resulting in corresponding increases of upstream (headwater) levels of from 5.4 feet to 7.3 feet (**Table 7**).

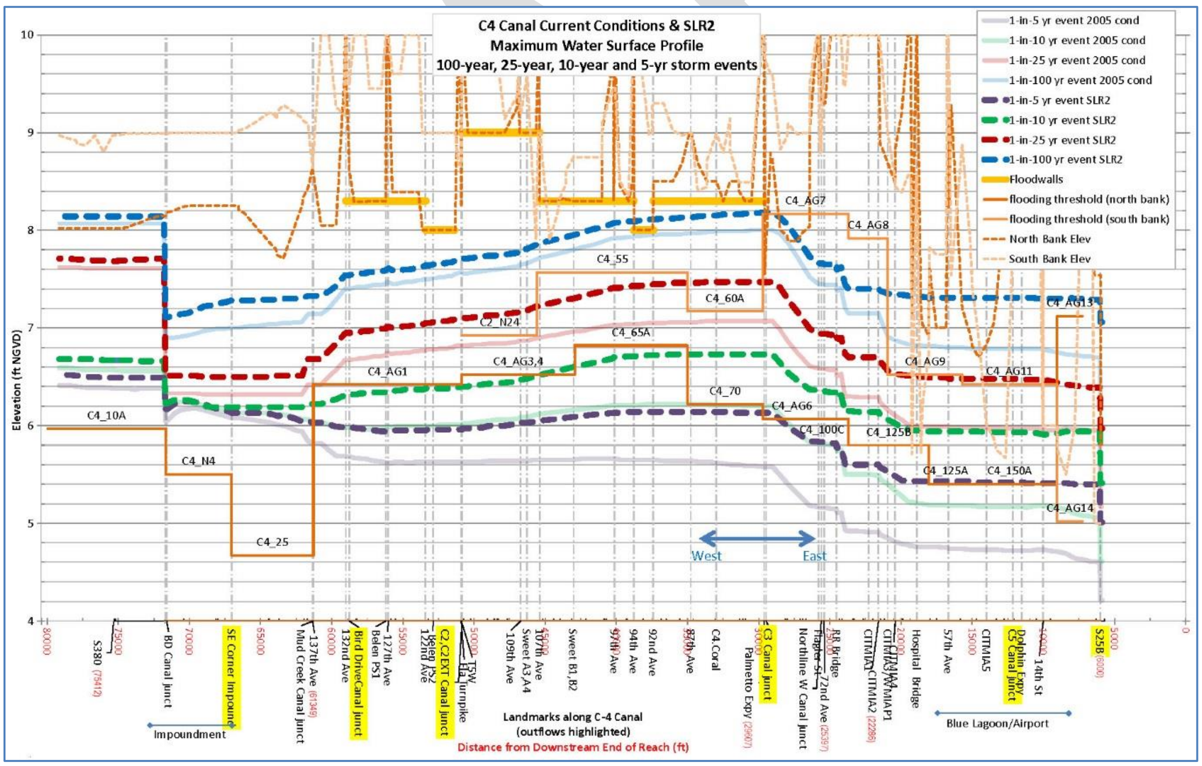


Figure 13. Maximum C-4 water surface profiles for SLR2 conditions

Table 7. Water levels in feet (NGVD) predicted by the model (Figure 13) to occur at reference sites for the intermediate projection of future sea level rise (SLR2) during design storm events.

Design Storm Return Period (yrs)	①S-25B Structure (6000)*		②City of Miami Pump Mia1 (22286) (ft NGVD)	③East of C3 Near 72 Ave (25397) (ft NGVD)	④Palmetto Exwy (29608) (ft NGVD)	⑤137 Ave (61349) (ft NGVD)	⑥W of S-380 (75412) (ft NGVD)
	Tailwater (ft NGVD)	Headwater (ft NGVD)					
5	5.0	5.4	5.6	5.8	6.1	6.0	6.5
10	5.4	6.0	6.1	6.4	6.7	6.2	6.7
25	6.0	6.4	6.7	6.9	7.5	6.7	7.7
100	7.1	7.3	7.4	7.7	8.2	7.3	8.1

numbers in parenthesis refer to distance (feet) west (upstream) of S-25B structure; reference site numbers in circles refer to sites designated on **Figure 11.

For the 1-in-100 year storm, the difference between headwater and tailwater stages is only 0.2 feet, so gravity discharge from the structure is compromised. Further west at the City of Miami pumps (site ②) and 72nd Ave (site ③), water levels range from 5.6 to 7.4 feet. The 1-in-5-year flood elevations for SLR2 are about 0.4 feet higher than the corresponding 1-in-5-year elevations for SLR1 conditions. The 100-year flood elevations for SLR2 are 0.1 feet higher than the elevations observed at these sites for the SLR1 scenario.

Western Basins. At the Palmetto Expressway (site ④), the effects of SLR2 result in higher flood elevations of 0.0 to 0.2 feet relative to the SLR1 storm events. Similarly, at 137th Ave (site ⑤), water levels for SLR2 were only 0.1 to 0.2 feet above levels that occurred during the SLR1 scenario. West of S-380 (Site ⑥), water levels were the same under the SLR2 condition relative to SLR1.

C-4 Canal. With an increase in sea level on the order of 0.81 feet above current for the SLR2 scenario, water levels in the eastern portion of the canal increased from 0.4 to 0.9 feet relative to current conditions. The estimated stages for the 100-year storm exceed the adjacent sub-basin inundation thresholds throughout almost the entire C-4 watershed, although they do not exceed the floodwall elevations in the reach between C-3 Canal and Bird Drive Canal, except between 92nd and 94th Avenues.

Level of Service for Flood Control. With the higher sea levels and storm surges associated with the SLR2 scenario, C-4 may not be able to even provide a 1-in-5 year level of service for several sub basins located north of the canal. These include C4_AG6, C4_125A and C4_150A. In contrast, C4_65A will experience a 1-in-10 year level of service and C4_AG13 will no longer be protected against the 100-year storm. South of C-4, the canal should be able to maintain at least a 10-year level of service for all of the basins, with C4_55, C4_AG7, C4_AG8, and C4_AG9 experiencing a 1-in-25 year level of service. In contrast, the level of service for C4_AG14 is less than 1-in-five years.

SLR3 Conditions

Figure 14 contains the set of maximum water surface profiles for SLR3 conditions. Also shown in **Figure 14** for comparative purposes are the maximum water surface profiles associated with current sea level conditions. Water levels predicted by the model to occur within the canal are summarized in **Table 8.**

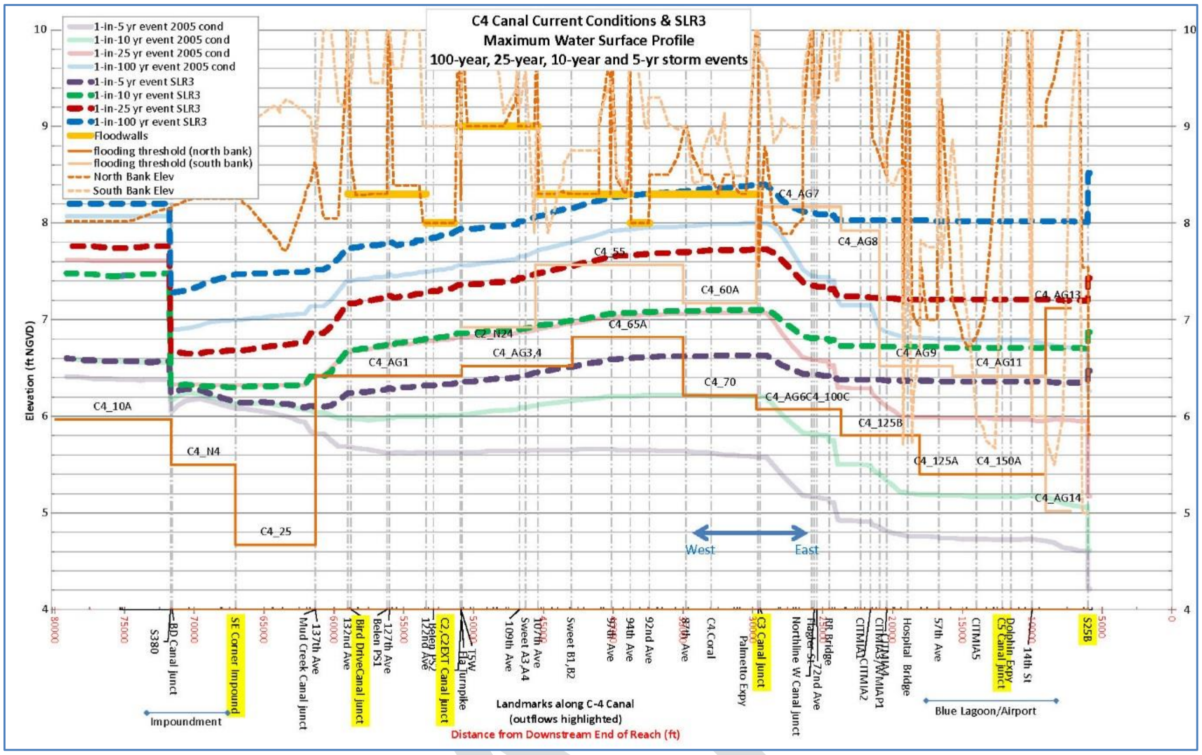


Figure 14. Maximum C-4 water surface profiles for SLR3 conditions relative to current conditions

Table 8. Water levels in feet (ngvd) predicted by the model to occur at reference site locations (numbers in circles) shown on figure 14 under the highest projection of future sea level rise (slr3) for various design storm events.

Design Storm Return Period (yrs)	①S-25B Structure (6000)*		②City of Miami Pump Mia1 (22286) (ft NGVD)	③East of C3 Near 72 Ave (25397) (ft NGVD)	④Palmetto Exwy (29608) (ft NGVD)	⑤137 Ave (61349) (ft NGVD)	⑥W of S-380 (75412) (ft NGVD)
	Tailwater (ft NGVD)	Headwater (ft NGVD)					
5	6.5	6.4	6.4	6.5	6.6	6.1	6.6
10	6.9	6.7	6.7	6.8	7.1	6.4	7.5
25	7.4	7.2	7.2	7.3	7.7	6.8	7.8
100	8.5	8.0	8.0	8.1	8.4	7.5	8.2

numbers in parenthesis refer to distance (feet) west (upstream) of S-25B structure; reference site numbers in circles refer to sites designated on **Figure 11.

Structure and Eastern Basin. Tailwater elevations at S-25B (site① on **Figure 14**) range from 6.5 feet during a 1-in-5 year storm event to 8.5 feet (**Table 8**) during a 1-in-100 year event under the high sea level scenario (SLR3). Headwater elevations range from 6.4 feet during a 1-in-5-year event to 8.0 feet during a 1-in-100 year event. Tailwater elevations exceed headwater elevations during all storm events, so there is pump discharge of 600 cfs but no gravity discharge from S-25B. Further west at the City of Miami pump (site ②), the canal stage during the 1-in-5 year storm is 6.4 feet, which is 0.1 foot higher than the water level that occurs under current conditions during a 1-in-25 year storm. Furthermore, the stage during a 1-in-25 year storm is 7.2 feet, which is equivalent to the level that occurs under current conditions during a 1-in-100 year storm event. During a 1-in-100 year event, the entire eastern portion of the basin is between 8.0 and 8.1 feet.

Western Basins. At the Palmetto Expressway (site ④), with the SLR3 scenario, water levels range from 6.6 feet for the 1-in-5-year storm to 8.4 feet for the 1-in-100 year storm. The water level for the 1-in-10 year storm is equal to the water level experienced during a 1-in-25 year storm under current conditions. The water level for the 1-in-100 year storm is 0.4 feet higher than the water level that occurs during a 1-in-100 year storm under current conditions. At 137th Ave (site ⑤), water levels range from 6.1 feet for the 1-in-5 year storm to 7.5 feet for the 1-in-100 year storm event. The 1-in-5 year storm with SLR3 is similar to a 1-in-10 year storm under current conditions while a 1-in-10 year storm is equivalent to a 1-in-25 year storm. Water levels west of Palmetto Park expressway in the 1-in-25 year and 1-in-100 year storm events are 0.4 to 0.6 feet higher than water levels that occur in these storm events under current conditions. West of S-380, water levels ranged from 6.6 for the 1-in-5 year storm event to 8.2 feet for the 1-in-100 year storm event.

C-4 Canal. Water levels in C-4 Canal exceeded the inundation thresholds of the eastern sub-watersheds south of the canal during the 1-in-5 year storm event. During the 1-in-100 year storm event, water levels in the canal exceed inundation thresholds in the entire eastern portion of the basin except in the vicinity of 72nd Ave.

Level of Service for Flood Control. With this highest prediction of sea level rise, only sub basins C4_AG1, C4_AG3, C4_AG4 and C4_65A maintain a 1-in-5 year level of service north of C-4 while C4_AG13 exceeds a 1-in-10 year level of service. South of C-4, all of the adjacent sub basins appear to maintain at least a 1-in-5 year level of service, with C2_N24, C4_55, C4_60A ,maintaining a 1-in-10-year level of service and C4_AG7 and C4_AG8 maintaining a 1-in-125 year level of service. In contrast, the expected level of service for C4_AG14 is well below 1-in-5 years.

Based on the results presented in this section, the level of service that can be designated for the C4 canal itself appears to be a 1 in 10 year storm event. Additionally, it should be noted that since the model allows only momentary reverse flows through or around the coastal structures, the sea level rises primarily serve to restrict outflows. Consequently, it is inherently assumed that at each coastal structure a dike exists and it will prevent storm surges from moving into the C4 watershed. A cursory view of the aerial imagery suggests that the construction of such a barrier is realistic.

As discussed in **Appendix G**, USGS estimates of increases in wet season ground water levels due to sea level rise suggest that increases of approximately 0.1 foot to 0.85 foot in initial water table stage are possible. These increases in initial groundwater levels lead to similar increases in maximum canal stages.

According to **Figures 12 to 14**, the effects of SLR on maximum canal stages appear to be more pronounced for the smaller storms than for the larger ones. This implies that nuisance flooding will become more substantial and more frequent. For example, near the midpoint of the C-4 canal, SLR1 causes maximum C-4 stages to increase by more than 0.3 foot, relative to current conditions during the 5-year storm while corresponding increases during the 100-year storm are close to 0.1 foot above current conditions. Similarly, increases during the 5-year storm due to SLR3 are close to 0.8 foot in the same area while corresponding increases during the 100-year storm are approximately 0.4 foot.

2. Maximum Flow Capacity throughout the Primary Canal Network (updated)

An allowable discharge has been defined for each the SFWMD Canals (**SFWMD ERP Applicant's Handbook Volume II, Appendix A, 2014**). The allowable discharge values are based on the design discharge capacity of the canal. In general, these were established as part of the original C&SF design. For the C4 canal, "the allowable discharge rate is based on the peak discharge rate after development not exceeding the rate that existed prior to development. The design storm is the 25 year event…" (**page 20, Appendix A, ERP Applicant's Handbook Volume II**).

Current Conditions

PM #2 establishes the effective discharge capacity of the canal under current and future sea level rise conditions. **Figure 15** illustrates how discharge capacity is generated from computed instantaneous flows.

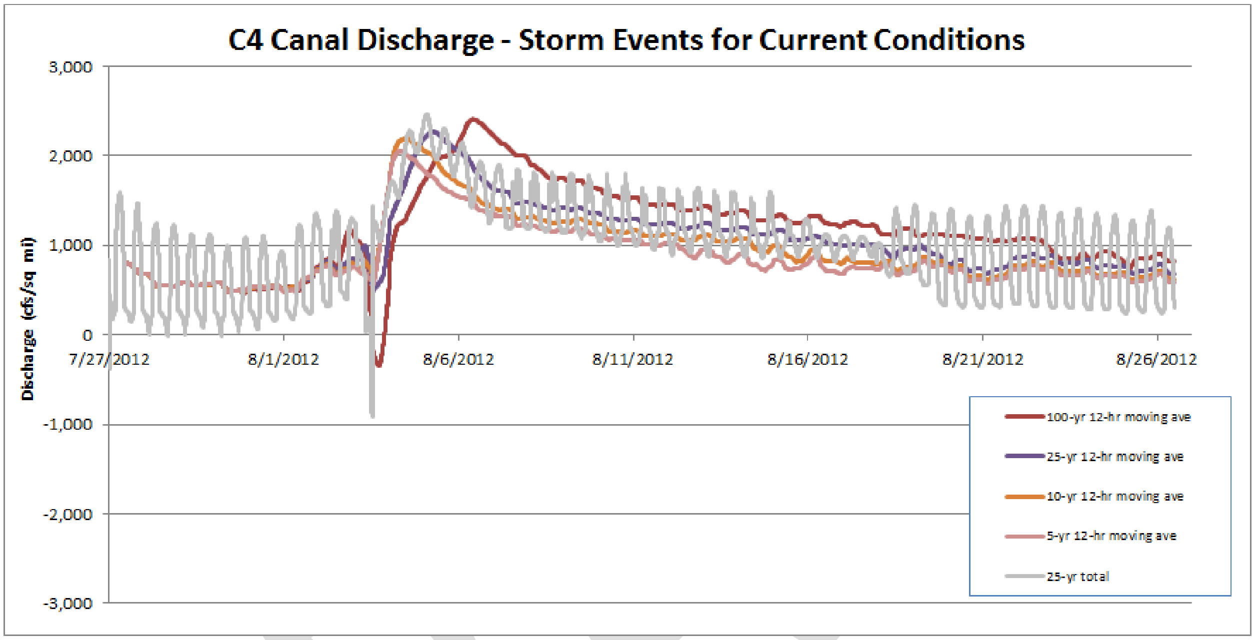


Figure 15. C-4 canal discharge for the 5, 10 25 and 100-year storm events under current sea level conditions.

The computed flows are obtained from the model simulations reflecting current conditions, where the instantaneous flow of the C4 canal is taken to be the sum of the instantaneous flow through S-25B plus the sum of the instantaneous flows entering the C2, C3, C5 and 132nd Ave canals. Since instantaneous flow oscillates with the tide, a 12-hour average is used to filter out tidal effects. The maximum value of the 12-hour average flow for the design storm (as determined by PM#1) is considered to be the discharge capacity and can be compared to the permitted discharge.

Figure 15 shows that, for all design storms, flow increases slowly during days 1 and 2 (August 1 and 2) of the 3-day design event, when rainfall intensity is low. Near the middle of day 3, peak rainfall and peak storm surge occur simultaneously. Infiltrated storm water fills the canal while the storm surge suppresses outflow. This results in some negative discharge values, since the gates cannot close immediately when a negative water level condition occurs at one or more of the coastal structures. As the storm surge recedes, the discharge increases to a maximum of 2200 cfs to 2400 cfs, depending on the design storm. It subsequently recedes, returning to near base flow conditions by the end of the simulation. Additionally, it is evident that discharge capacity increases slightly with larger design storms, from about 2200 cfs for the 5-year event to about 2400 for the 100-year event. Furthermore, base flow resulting from storm water is delayed while durations of high flows are increased for larger storms.

Future Conditions

Table 9 provides the discharge capacity for the four design storms and the four sea level scenarios. Following convention, the maximum flow is normalized by dividing the watershed outflow by the watershed area; the units are CSM, or cfs per square mile of watershed. **Table 9** indicates that, for current sea level, SLR1 and SLR2, discharge capacity remains nearly constant, ranging from 25 to 29 CSM. Under these conditions, sea level rise, rainfall intensity and storm surge appear to have a negligible effect on discharge capacity.

Table 9. C4 canal discharge capacity (CSM)*

Tidal Condition	Return Period of the Design Storm			
	5-yr	10-yr	25-yr	100-yr
Current	25	27	28	29
SLR1	26	27	27	29
SLR2	25	25	26	28
SLR3	17	19	22	23

*Discharge capacity is defined as the peak canal discharge from a design storm. Flow is averaged over the 12-hour tidal cycle to filter out tidal effects and normalized over the watershed area.

In contrast, the higher tidal conditions associated with SLR3 result in a significant decrease in discharge capacity. Under these conditions, it ranges from to 17 to 23 CMS (**Table 9**). This reduction is due to a decrease in coastal structure capacity. That is, the structure (and not the canal) limits the discharges under these conditions.

The results are not much different for the SLR1 and SLR2 future scenarios. However, for SLR3 (Figure 15B), the pattern of discharge after the storm is significantly different, reflecting the impact of the downstream increase in sea level at S-25B. **Table 9** summarizes the discharge capacity for the four design storms and the four sea level scenarios.

Effects of Sea Level Rise on Maximum Discharge capacity. The peak occurs later and lasts longer with larger storm events but the peak capacity is the same for all events for CSL and SLR#1 and SLR#2. Discharge Capacity drops by 1/3 for SLR#3

3. Structure Performance – Effects of Sea Level Rise

Details of the derivation of this performance measure are provided in **Appendix H.** This metric is developed from simulations of structure flow where the upstream stage is fixed at the design headwater stage and downstream stage oscillates with tide. Four days into the simulation, a storm surge of known intensity (as indicated by its return period) raises tail-water stage and suppresses flow. Results of each simulation are further analyzed to determine the minimum 12-hour flow through the structure. For this analysis, five sea-level scenarios were modeled and, for each scenario, a range of six design storm surges were examined. The minimum flow values from each of these 30 model runs were then compared on a single plot.

Model Simulation Results

Figure 16 shows the performance of the S25B structure as affected by sea level rise and storm surge. Similarly, **Figure 17** shows the performance of the G93 structure while **Figure 18** shows the performance of the S22 structure.

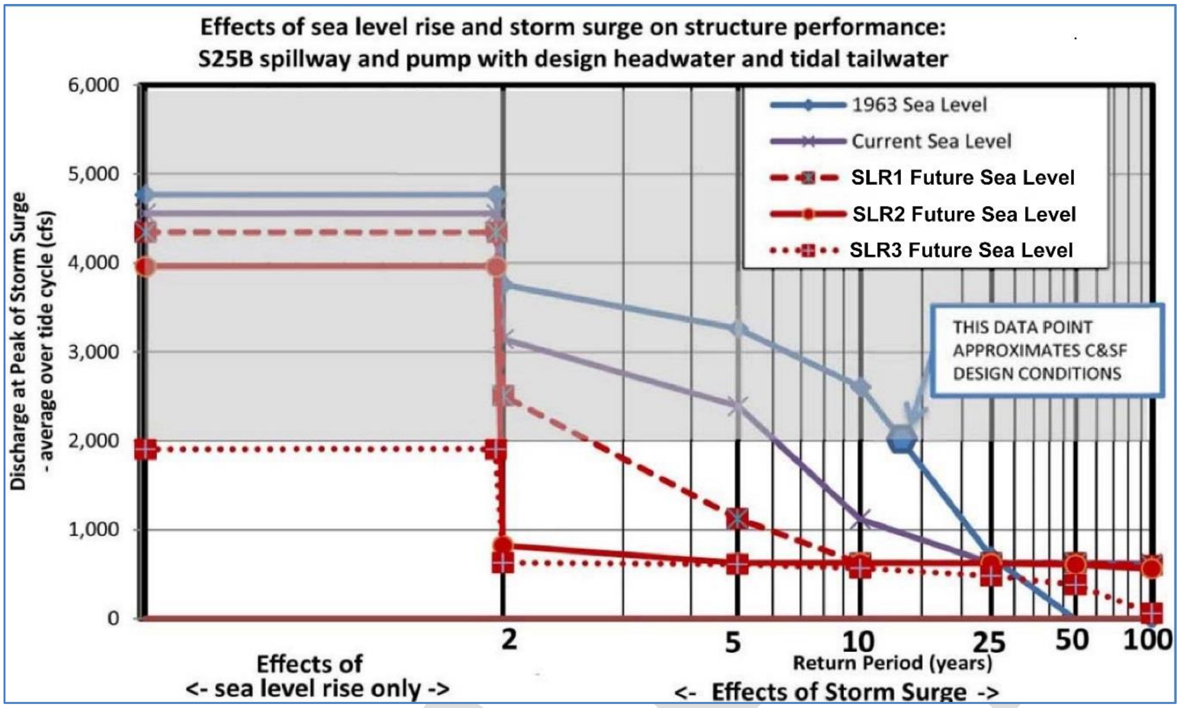


Figure 16. Effects of sea level rise and storm surge on the capacity of S25B Spillway and Pump (no pumps in original C&SF design but pumps are in current and future scenarios).

These figures are quite complex. Because flows cycle with the tide 12-hour averages are used to filter the tidal oscillations. All figures display five sea level scenarios: structure flows associated with 1963 storm surge boundary conditions are depicted with blue diamonds; flows associated with current conditions (CSL) are shown as purple x's; flows associated with SLR1 (0.34 ft above current sea level), SLR2 (0.80 ft) and SLR3 (2.26 ft) conditions are shown using red markers. Each scenario has, on the left, a horizontal line showing flow without storm surge and, on the right, a curve showing how storm surge suppresses flow, with storm surge plotted by its return period. Also shown on the plot, as part of the 1963 surge conditions, is the original design conditions is displayed using an enlarged blue hexagon. Flows are calculated using the District's FLOW Program algorithms and the return period for the design tail-water stage was determined using 12-hour average tail water stages. Finally, the top portion of each figure is shaded. Flows in the shaded area have never been observed at the structure and show the limits of canal conveyance; data points in the shaded area are theoretically possible but in reality the canal cannot deliver these flows so headwater stage would not reach design stage.

A comparison of **Figures 16 through 18** reveals that the S22 structure is the structure most strongly impacted by sea level rise. This is to be expected since it is the most low-lying. Under storm surges that are greater than or equal to the SLR2 sea level estimates, the S22 structure cannot pass any flow. This is even true for a storm surge with a modest 2-year return period. The S25B and G93 structures, on the other hand, are designed for higher tailwater elevations and can pass design flows as long as the storm surge is negligible and sea levels do not exceed the SLR2 estimates.

Likewise, under the same sea level conditions, the capacity of the G93 structure decreases to less than 400 cfs (65% of design) for a 5 year return period surge. However, because 400 cfs is the approximate capacity of the C3 canal, the G93 structure is effectively overdesigned and the 65% loss in structure capacity is unlikely to have a detrimental impact.

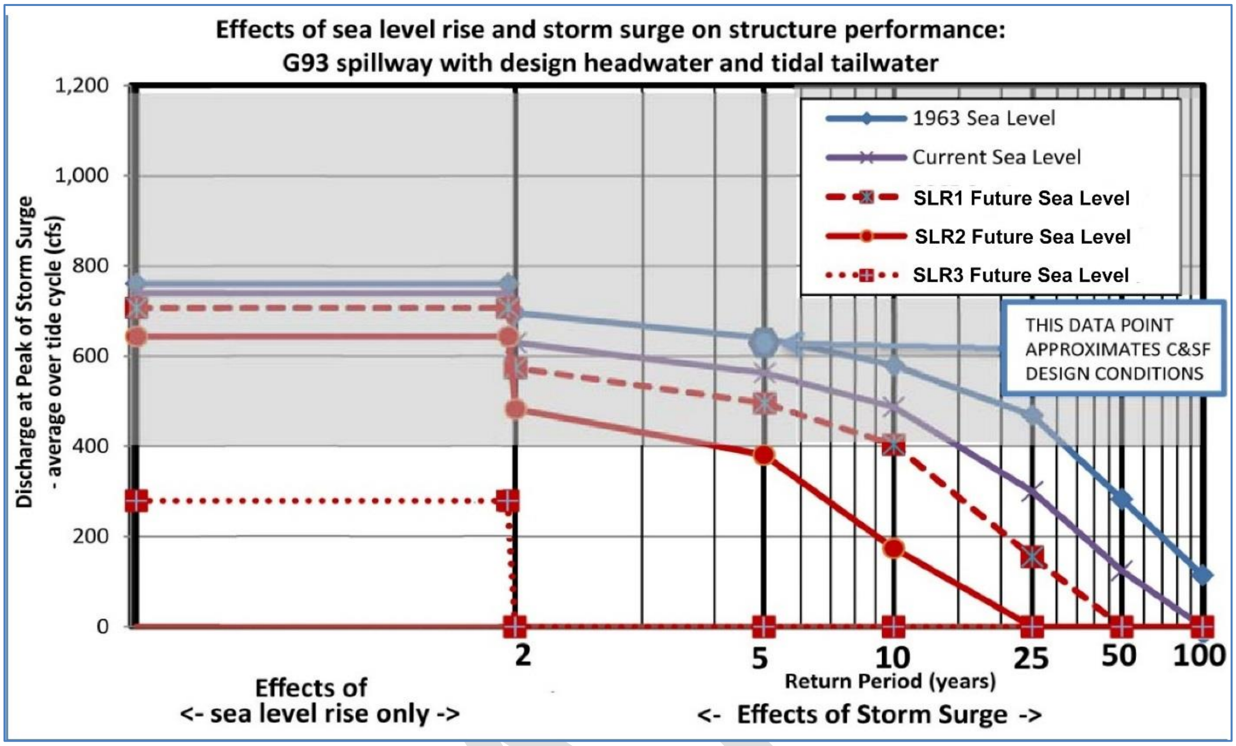


Figure 17. Effects of sea level rise and storm surge on the capacity of G93 Spillway (G93 is not a C&SF structure).

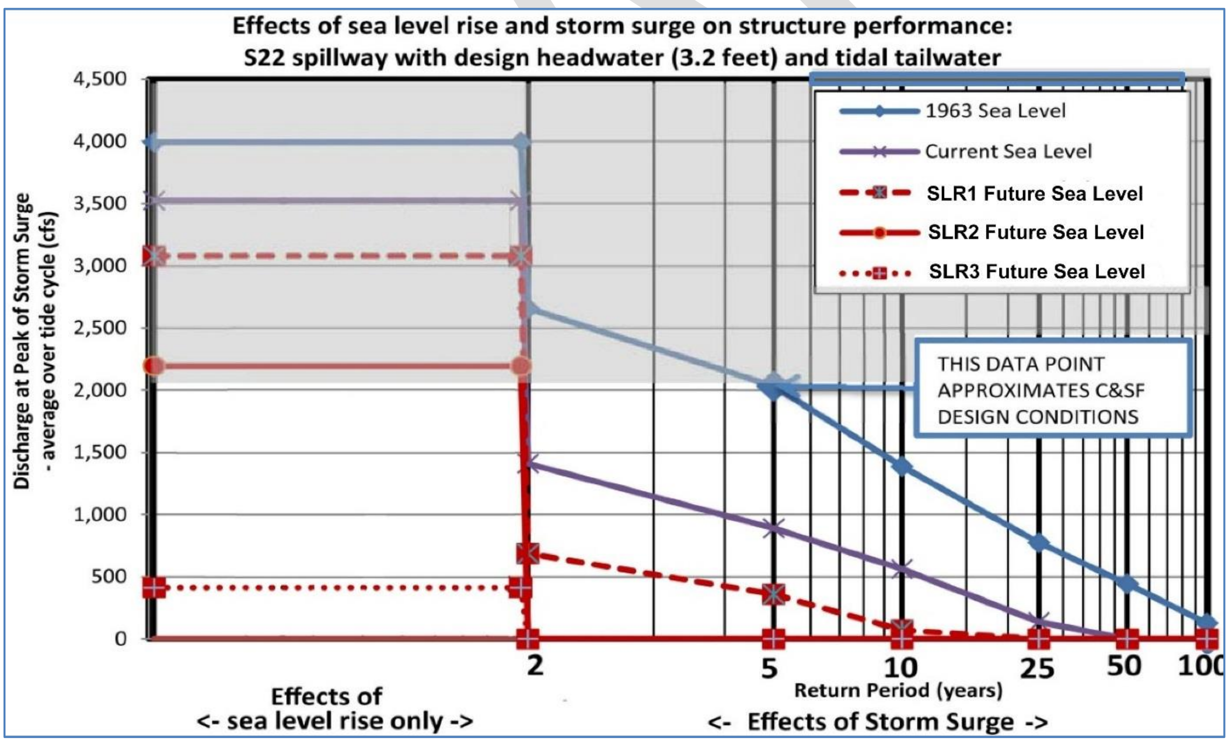


Figure 18. Effects of sea level rise and storm surge on the capacity of S22.

Observations for Specific Structures.

S25B (Figure 16): Without considering storm surge, the structure capacity exceeds the 2000 cfs design capacity for all sea level scenarios except under SLR3 conditions which is a bit under 2000 cfs; the structure can carry the design flow except during storm surge events.

Storm surge reduces structure capacity and sea level rise makes reduced capacity situations more frequent. Back in 1963, the design tail-water stage (4.1 feet NGVD29[1]) was a relatively rare event, equivalent to a storm surge with an approximate return period of 12-13 years. Current sea level is almost 6 inches higher than in 1963 and the 2000 cfs capacity is limited by storm surges with 6-year return period storm surge or greater. These are still infrequent events. But future sea level rise makes reduced capacity situations increasingly likely. For the SLR1 scenario (+0.34 ft), storm surges with a 3 year return period will limit flow. With the SLR2 (+0.80 ft) and SLR3 (+2.26 ft) scenarios, even common storm surges reduces the structure capacity to 600 cfs, the capacity of the forward pumps.

G93 (**Figure 17**): Without considering storm surge, the structure capacity exceeds the 560 cfs design capacity for all sea level scenarios except under SLR3 conditions. For SLR3 conditions the capacity drops to 270 cfs, far below the 560 cfs design capacity and considerably below the observed 400 cfs canal capacity.

Storm surge has less effect on the capacity of the G93 structure than it does on the S25B. This is because G93 is designed for a 0.5 ft head drop across the structure while S25B is designed for a 0.3 ft head drop. Back in 1963, the design tail-water stage (3 feet NGVD29) was an infrequent event, equivalent to a storm surge with an approximate return period of 5 years. For current sea levels, this equates to storm surges with a 3 year return period – still quite rare. Even SLR1 conditions require storm surges with a 2 year return period to limit flow to the design capacity. SLR2 sea level conditions make storm surges problematic if the 560 cfs design capacity is considered but the canal's carrying capacity is only 400 cfs and the structure can carry this flow when 2-year storm surges occur. It is only with SLR3 sea level conditions that storm surge limits flow.

S22 (**Figure 18**): Without considering storm surge, the structure capacity exceeds the 2000 cfs design capacity for all sea level scenarios except under SLR3 conditions. For the SLR3 condition, capacity is severely reduced to under 500 cfs.

Back in 1963, the design tail-water stage (3.0 feet NGVD29) was a relatively rare event, equivalent to a storm surge with an approximate return period of 5 years. This is no longer true. Current sea level is almost 6 inches higher than in 1963 and even 2-year storm surges reduce the capacity to 1500 cfs. For the SLR1 scenario (+0.34 ft), the 2-year storm surges limits capacity to 600 cfs and for the SLR2 scenario (+0.80 ft), the 2-year storm surge limits capacity to 0 cfs. Past sea level rise makes this structure problematic during any significant storm surge event; future sea level rise will restrict or stop drainage during common storm surge events.

4. Impact of sea level rise on structure flow

Results

PM#4 looks at the impact of sea level rise under design storm (design rainfall and design storm surge) conditions. Unlike PM#3, the headwater at the structure is not at design levels but is determined

[1] The frequency of a particular tailwater stage changes with each location. S25B and G93 are both located several miles away from the ocean while S22 is close to the ocean. Runoff from both upstream and downstream runoff affects tailwater. Appendix C has more detail regarding the development of tailwater frequency relationships at each station.

by runoff from the rainfall, surge and structure operations. Like PM#3 the peak flows are averaged over the tide cycle to eliminate the 12-hour tidal oscillation effect. Unlike PM#3, which selects the minimum flow caused by the storm surge suppression, PM#4 selects the maximum flow that occurs as the storm surge recedes. This reveals possible stability concerns if stages or stage differences exceed design values.

Figures 19, 20 and 21 show the impact of sea level rise on structure flow under design rainfall conditions for structures S25B, S22 and G93, respectively.

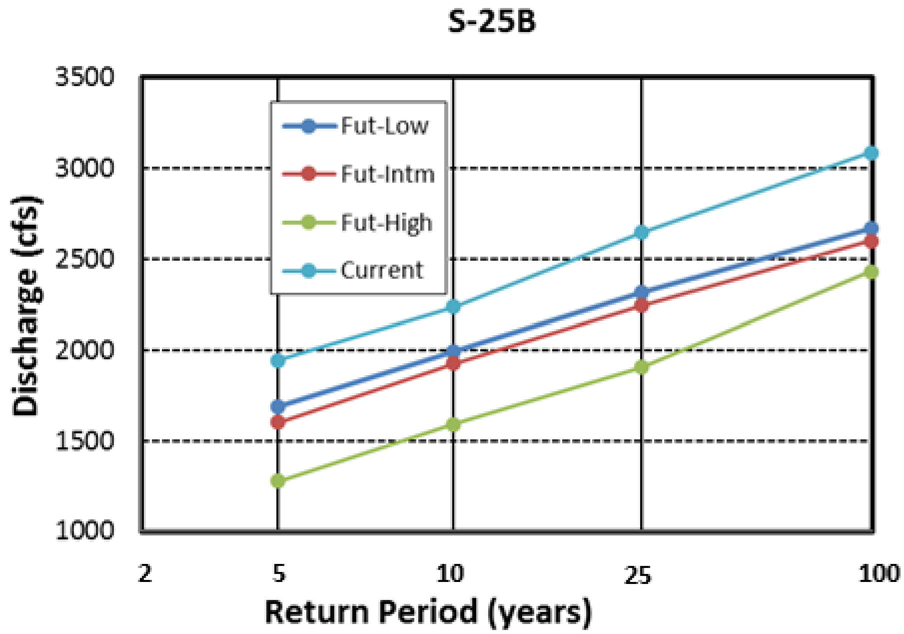


Figure 19. Impact of Sea Level Rise on Structure Flow at S-25B under Design Rainfall Conditions.

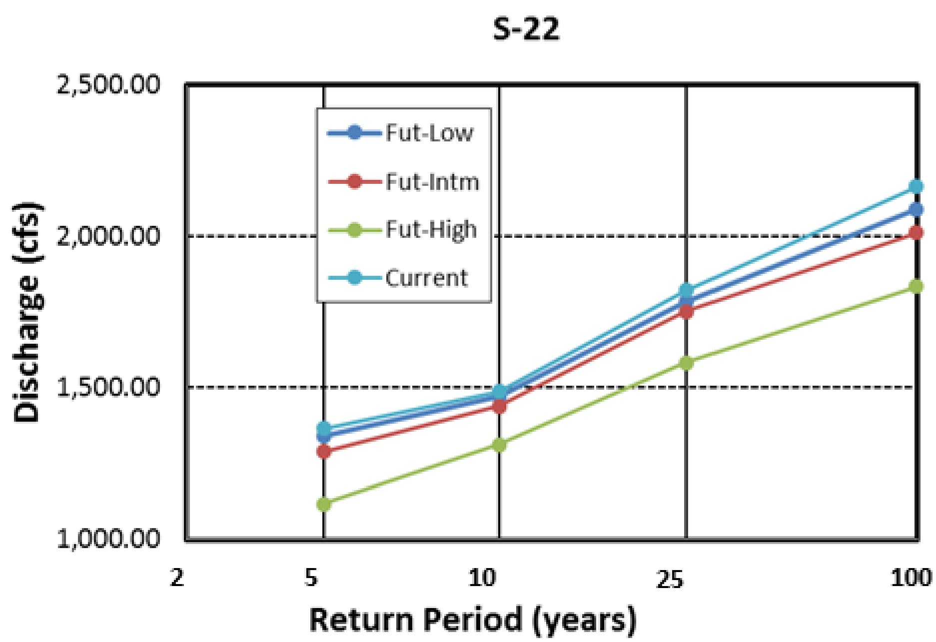


Figure 20. Impact of Sea Level Rise on Structure Flow at S-22 under Design Rainfall Conditions.

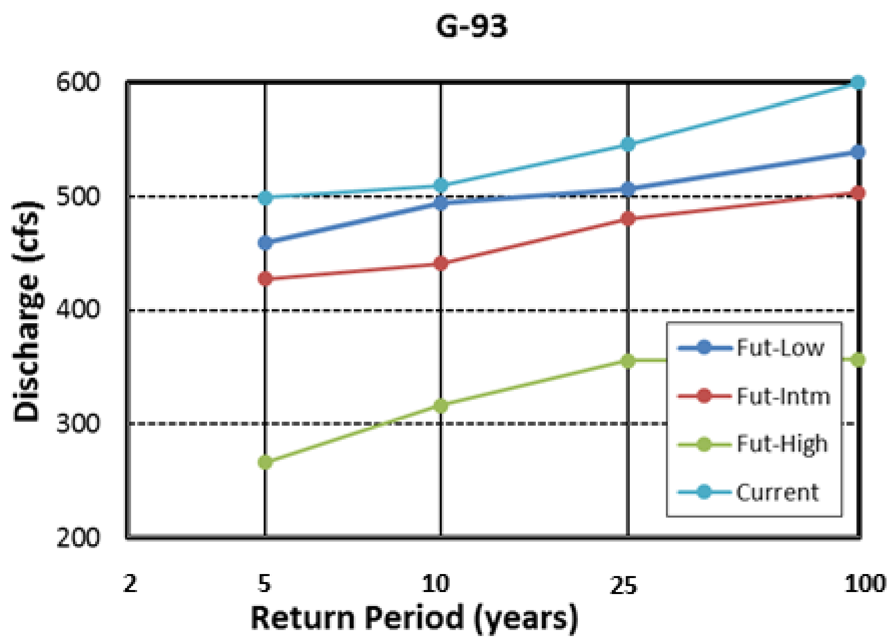


Figure 21. Impact of Sea Level Rise on Structure Flow at S-G93 under Design Rainfall Conditions.

All the model simulations assumed that the peak storm surge occurs at the same time as the peak rainfall and the results presented here follow that assumption. However, review of results for PM#4 indicated that the offset between the peak rainfall and peak storm surge at the coastal structure is an important parameter to consider during design, maybe as important as selecting the magnitude of the rainfall or the peak tidal surge stage. For instance, some sensitivity and trial-and-error simulations suggest that an offset of 13 hours (storm surge peaks 13 hours later than rainfall) created the maximum reduction on peak runoff at the S25B structure.

Observations:

- Maximum structure flow increases with design storm because both runoff and storm surge increase with the design storm. This causes more runoff to be impounded upstream of the structure resulting in higher headwater stage and higher flows when the surge retreats.
- The maximum structure flows exceed design capacity when headwater stage is above design stage and the head drop across the structure exceeds design head drop. The impact of the high flows and high stages on structural stability may be significant and should be assessed when the structure is refurbished.
- Under all storm events and all scenarios, the impact of sea level rise is minimal for all structures for SLR1 and SLR2. The 0.34 feet increase of SLR1 and the 0.80 feet increase of SLR2 do not impact peak outflows.
- SLR3 scenario significantly reduces peak runoff for all storm events at all structures. The 2.26 feet increase of SLR3 is great enough to reduce the outflows from the structures. [This is also seen in PM#2 – conveyance capacity, which is also reduced under SLR3 and only SLR3.]
- The impact of SLR3 is least at S25B and G93 for the 100 year event. The reasons for these are not clear:
- In most of the cases, the figures shown above present flow magnitudes which exceed design flows for the structures. One factor contributing to this is head differentials in the simulations exceeding design head differentials. Large head differentials appear for both positive and negative (reverse flows).

- These head differentials need to be accounted for during design and day-to-day operations. Large head differentials need to be considered during future designs so that appropriate stability checks are conducted for large positive (tilting/sliding to the ocean) and negative (tilting/sliding inland)
- Day to day operations need to consider the Maximum Permissible Head Differential (MPHD). While operating structures, water managers maintain head differentials which do not compromise the safety of the structure.

It is clear from the discussion above that simulation of realistic head differentials (positives and negatives) is extremely important for the design of water control structures, not only in the determination of the hydraulic capacity, but also to analyze and assure the stability of the structure.

Head differentials during the operation of the structure are a function of the runoff coming to the structure, the operations of the structure, the headwater elevation and tailwater elevation. Head differential translates into a certain discharge passing the structure. Head differential will also depend on the timing of the peak rainfall, transit times in the basin and the arrival of the storm surge. For instance, when peak runoff at the structure occurs close to storm surge, large suppression of flow will take place, which will translate in increased stages upstream. In the case when peak runoff and peak storm surge are far apart in time, runoff will be properly evacuated, but the closing of gates may create large undesirable negative head differentials.

Further analysis of the role of the offset between peak rainfall and storm surge is necessary. It is clear that this offset could be as important as other design parameters such as rainfall and storm surge magnitude in order to properly quantify the capabilities of the structure. Methodologies to select the values to use in the design need to be explored.

5. Frequency of Flooding -- stage-based LOS for sub-watersheds

Maximum Stage

Sea level rise can affect flooding within the subwatersheds in several ways: long-term suppression of flow, short-term suppression caused by storm surge, higher water tables caused by suppressed groundwater flow, and increased base flow caused by higher water tables. All are considered in these analyses.

Figure 22 shows the maximum water stage in each of the thirty-three sub-watersheds of the C4 watershed when exposed to a 5-year, 10-year, 25-year and 100-year design storm event under current sea level conditions. The figure includes a dashed line that shows the flood threshold of each watershed. The flood threshold is a criterion developed by District staff for use as an indicator that substantial flooding may be occurring within the subwatershed. This criterion is defined as the stage where 20% of the developed land (excluding marshes, lakes, and rock-pits) within a sub-watershed is underwater; this criterion filters out open-water area within the developed lands. Sub-watersheds dominated by wetlands and rock-pit areas are included in the graph but were not considered when establishing flood protection level of service; these seven subwatersheds are shaded gray.

Maximum stages for the 5-year event are below the flood threshold for all twenty-seven sub-watersheds. Maximum stages for the 10-year event are below the flood threshold for all but two sub-watersheds (AG4 and 100B). Maximum stages for the 25-year event are below the flood threshold for eleven of the twenty-six subwatersheds. Based on this, the C4 system provides flood protection for the 10-year storm event; protection is marginal for the 25-year storm event. Maximum stages for the 100-year event are below the flood threshold for only one of the subwatersheds and the flood depth (the difference between the maximum depth and the threshold elevation) exceeds 1.0 feet in thirteen of the twenty-six subwatersheds.

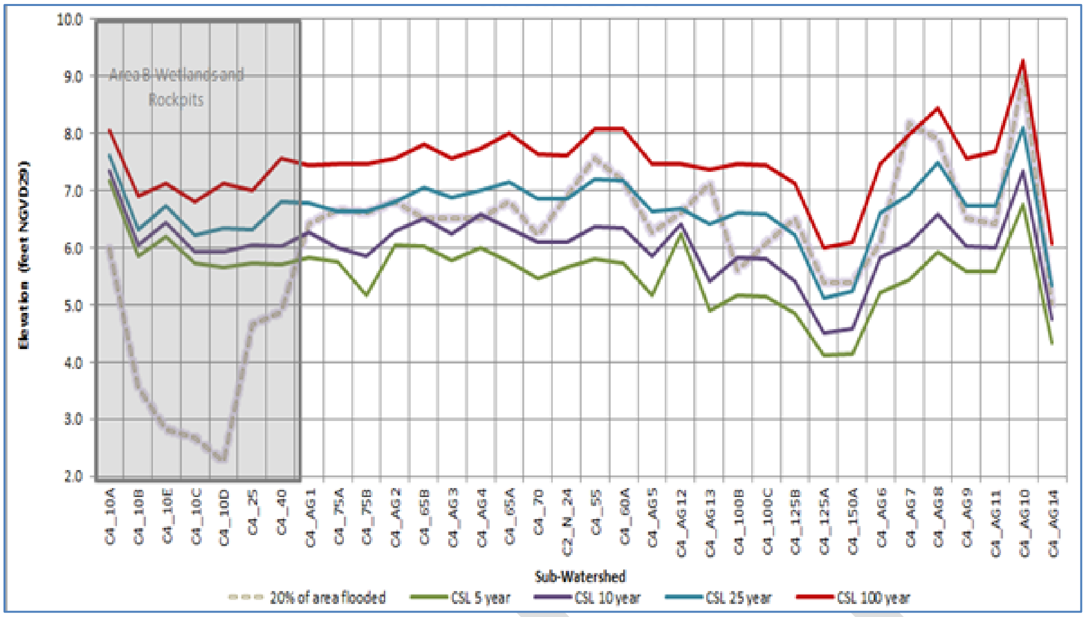


Figure 22. Maximum Stage in Sub-Watersheds for Four Design Storm Events: Current Sea Level.

It should be noted that the design flood simulated for this analysis is not the same as the design flood used in flood rate insurance (FIRM) mapping. The focus of this study is to assess the impact of sea level rise on the primary drainage system. Flood depth estimates derived in this study are approximations and should only be used for relative comparisons; they cannot be compared to FIRM flood depths. Under this study, rainfall patterns follow the SFWMD 3-day design pattern and tidal boundary conditions assume a storm surge with the same probability as the rainfall event. These rainfall patterns and tidal boundaries are more severe than those used in FEMA simulations. Graphs similar to **Figure 22** are available in **Appendix H** that show maximum watershed stages for the three future sea level rise scenarios SLR1 (0.34 feet higher than the 2005 sea level) SLR2 (0.80 feet higher), and SLR3 (2.26 feet higher).

Increase in Peak Stage in Sub-Watersheds

PM#5 compares the peak stages in sub-watersheds for SLR1, SLR2 and SLR3 against the peak stages from the 2005 base. These data are tabulated in **Appendix H** but, since there are 27 sub-watersheds and twelve simulations to compare (four design storms and three sea levels) the information is difficult to assess. Therefore **Figure 23** shows a statistical interpretation the tabular data. The vertical bars summarize the water level increase for each of the twelve simulations. The bar shows the average water level increase, the 90th percentile increase and the 10th percentile increase. Sub-watersheds dominated by marsh, open water and rock pits are excluded from the assessment.

Additional details of the flooding analysis, including maps showing the sub-watersheds that are flooded and range of flooding depths for design storm events current and future sea level rise conditions are available in **Appendix H. Figure 24** compares flooding from the 25-year storm under the four sea level rise scenarios.

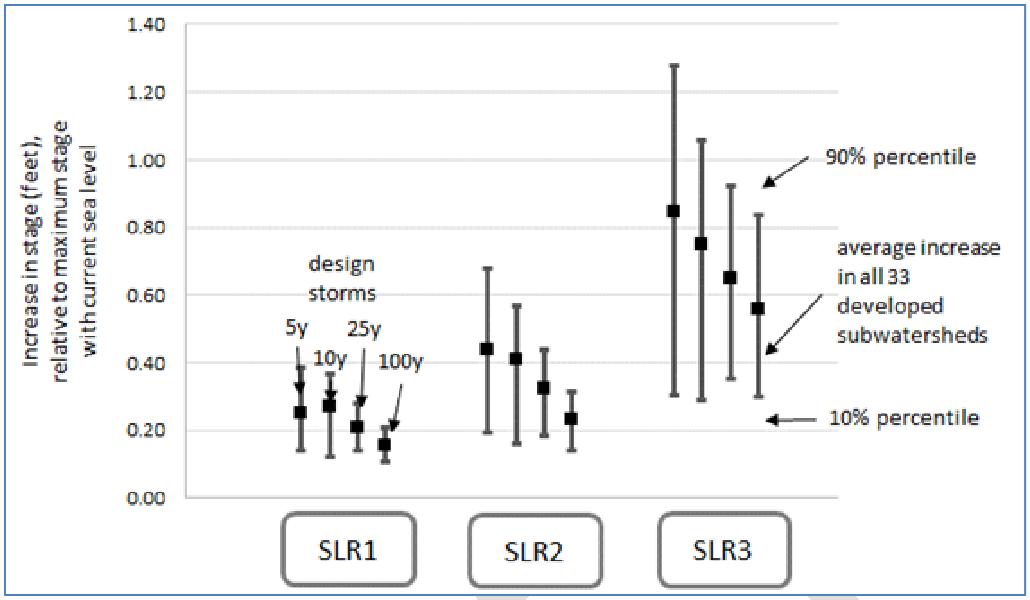


Figure 23. PM#5: Increase in Maximum Water Level in Sub-Watersheds Caused by Sea Level Rise

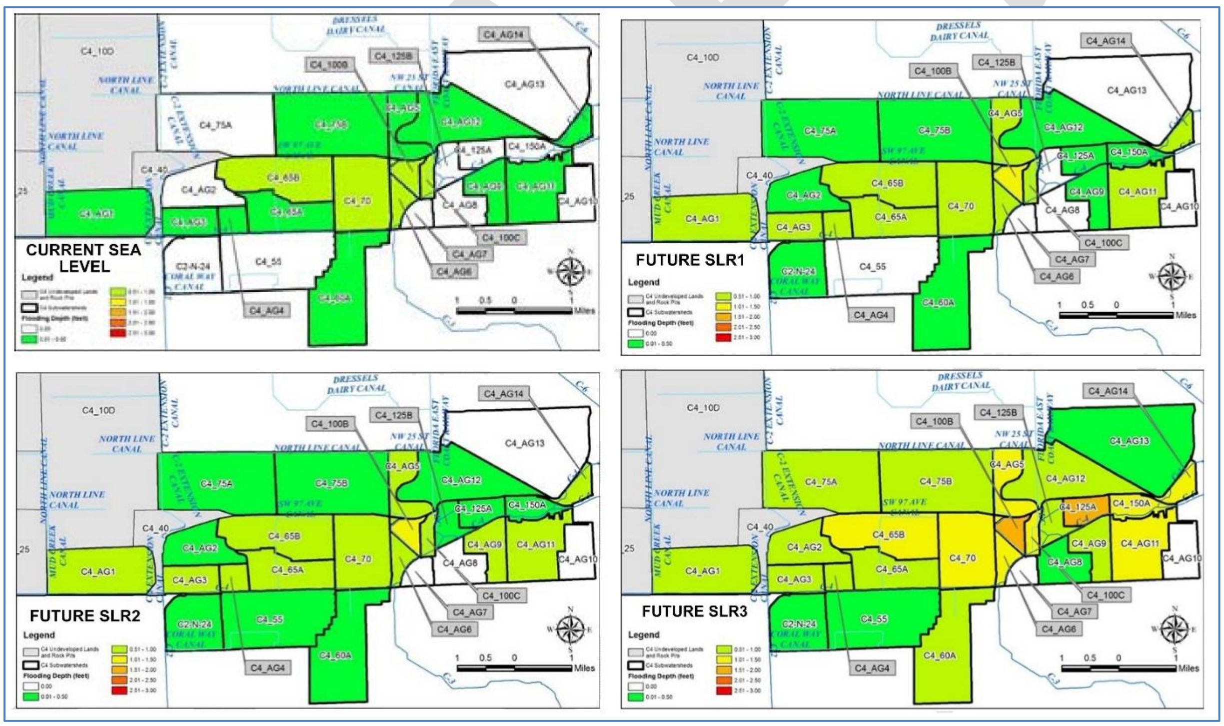


Figure 24. Flooding in the C4 watershed for the four sea level rise scenarios: CSL, SLR1, SLR2, and SLR3. All show the 25-year storm event.

6. Duration of Flooding in Primary Canal System

Metric # 6 examines the time needed to recover from a flood event. The metric looks at water levels at the T5 gage in the C4 canal and tracks the time that water levels are above a target canal stage of 4.5 feet. (This target stage is substantially higher that the water control elevation for the system and indicates a flood state, i.e. a state where all control structures are being operated to maximize drainage.)

Figure 25 shows PM6 for different Sea Level Rise scenarios. Under current conditions, PM6 ranges from 4 days for a 1-in-5 year storm to approximately 15 days for a 1-in-100 year storm. The duration of flooding increases under future sea level rise conditions showing that even minor increases in sea level impact the system. SLR1 (+0.34 ft) adds two days to the flood durations, from 6 days for a 1-in-5 year storm to 17 days during a 1-in-100 year storm. SLR2 (+0.80 ft) adds five days to the flood durations, from 10 days for a 1-in-5 year storm to 19 days during a 1-in-100 year storm. For SLR3 (+2.26 ft) the metric failed because flooding still existed at the end of the 30 day simulation for all design storms.

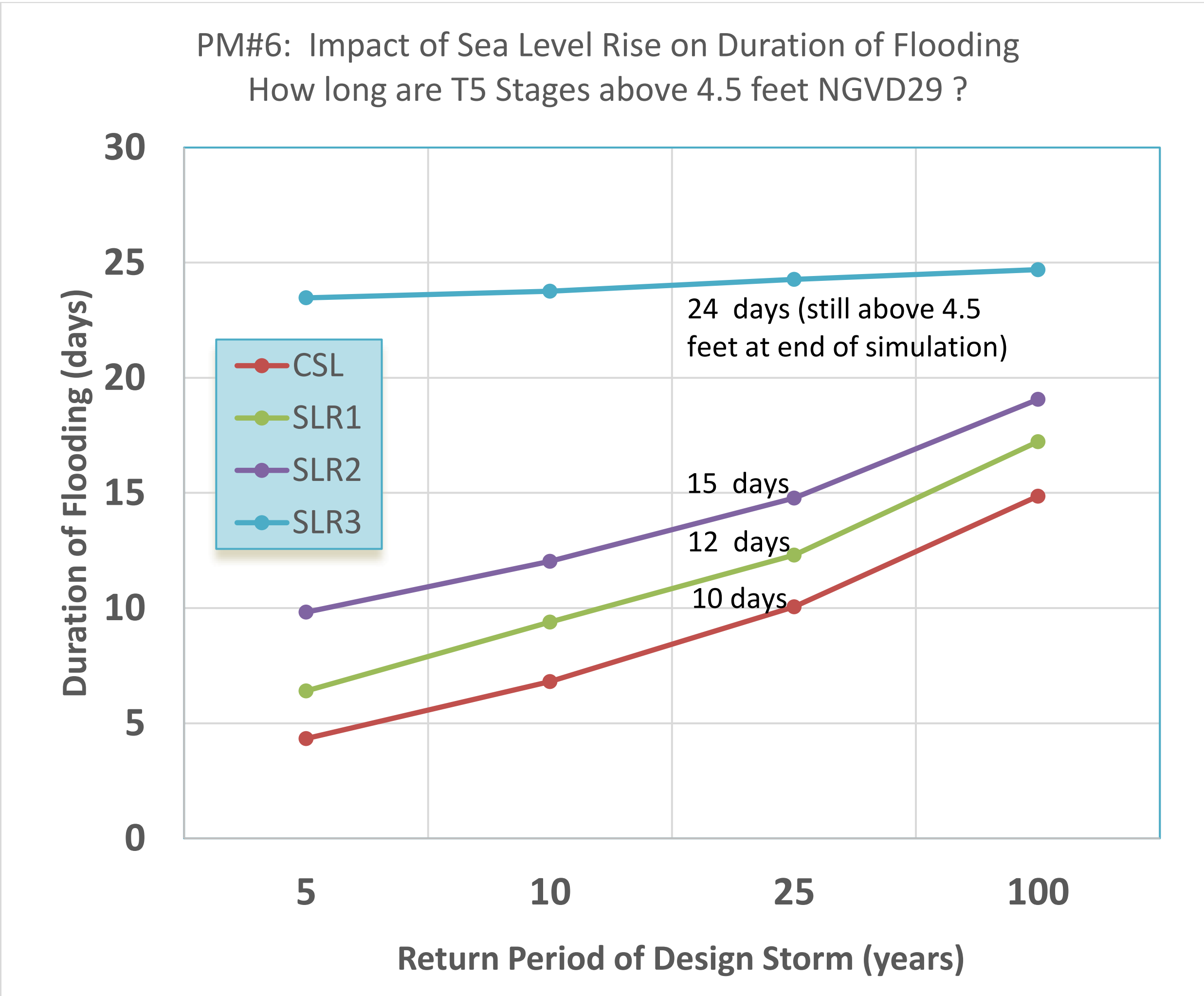


Figure 25. Comparison of Performance Measure # 6 for current conditions (CSL) and three future sea level rise scenarios (SLR1, SLR2, and SLR3). Vertical axis represents the number of days that water levels at the T5 monitoring gage in C-4 Canal are above the designated target management water elevation (4.5 ft NGVD) of the canal - see text)

CONCLUSIONS AND RECOMMENDATIONS

Primary System Performance

Basin Level of Service

The level of service (LOS) of flood protection for a water management network reflects the amount of protection provided for specific features within a watershed. Typically, there is no single LOS for a watershed. The LOS for a particular location or property is a function of feature s provided by the property owner and the performance of the primary, secondary and tertiary drainage systems that serve the property. The level of service for the primary water management system is referred to as the “primary level of service,” and is determined as the amount of flow conveyance capacity (Q) needed to remove the cumulative inflows of water that occur under design storm conditions and the resulting water levels (H) throughout the primary canal network. The primary purpose of this study is to evaluate the level of service for flood protection provided by the primary drainage network in the C-4 watershed located Miami-Dade County.

The design storm for a watershed is the most severe storm for which the canals and structures in the watershed will accommodate that storm’s runoff without flooding occurring in the watershed. A severe storm is described by the frequency with which it may occur. On a long term average a storm of given intensity may occur, for example, once in every ten years. This is written as 1-10 years, and is read as one in ten years.

With the completion of the C-4 Mitigation Project, the <u>current</u> design level of flood protection, is for a 1-in-25 year return period storm. That is, the canal stages within the C-4 will not overtop the canal bank at any time during the 1-in-25-year storm event. Although PM#1 suggests that the canal bank elevation may not be exceeded along most of the C-4 canal during a 1-in-25 year storm event, the high rates of seepage in the watershed indicate that water level in the adjacent watersheds will likely be at or near the canal water levels.

PM# 5 was used to evaluate LOS for developed land in the C-4 watershed in more detail. Of the 33 sub-watersheds in the C-4 Watershed, 26 are developed and the remaining seven consist primarily of wetlands, rockpits and retention areas. A further analysis was made to estimate the elevations of developed lands within each sub-basin. Results indicated that the land elevations in most sub-basin adjacent to the canal are significantly lower than expected water levels in the canals and that moderate (0.6 – 0.8 feet) flooding would likely occur during a 1-in-25 year flood event.

The current LOS is about 1-in-10. This may be somewhat conservative because we assumed that the watershed would experience both a design rainfall and, simultaneously a storm surge with the same probability. This could potentially occur during a hurricane, but would be unlikely for other large events. For SLR2, the LOS was reduced from 1-in-10 to 1-in-5.

Effects of Sea level Rise

Sea level has changed since the construction of primary features in the C-4 Basin and is anticipated to change further in the future. During the 52 years between 1963 and 2015, sea level changed by 0.46 ft. Three projections for future sea levels were evaluated for this study: SLR 1 is +0.34 ft, SLR2 is 0.80 ft and SLR3 is +2.26 ft. above current sea level. In addition, increased sea level will lead to increased groundwater stages, ranging from a 1.2 feet increase near the S-25B structure to 0.25 feet increase in the far western reaches of the watershed. These stage increases were used to modify pre-storm stages throughout the watershed.

In general, sea level rise will increase canal and groundwater stages more for smaller storms (5-y and 10-y) than for the larger storms (25-y and 100-y). This implies that nuisance flooding will be more frequent and more severe and sea level rise occurs but that severe flooding will be only moderately impacted as long as the outlet structures remain effective. The lowest rise (SLR1) had small impact on flooding during major storm events but its impact during small events was more pronounced and nuisance flooding was worse. Water levels throughout the canal increased by about 0.2 feet under SLR1. The impact was due in part to the higher pre-storm groundwater levels caused by sea level rise.

The second sea level rise scenario (SLR2) had a more significant impact on flooding, but the effect from large events is still only 0.2 feet. The impact was caused by the storm surge, in combination with higher sea level raising tail-water stages enough to suppress outflow during the surge. For SLR2, the impact could be seen for large storm events as well as small events. The structures could carry the watershed runoff (i.e. the structure capacity was greater than the canal capacity) except during the storm surge. The capacity of the S22 structure is significantly reduced, resulting in a reduced ability to carry C-4 peak runoff. The LOS was reduced from 1-in-10 to 1-in-5. The backwater flooding effect c for SLR2 stage profiles it increases the depth of the 100-year flooding upstream of S-25B by 0.5 feet (from 6.55 feet ngvd to 7.05 feet ngvd).

The third sea level rise scenario (SLR3) had a serious impact on flooding throughout the watershed. The sea level was so high that structure conveyance was less than canal conveyance at S25B and S22. Loss of structure conveyance was especially pronounced at S22 where the capacity was reduced to 400 cfs. Suppression of flow during the storm surge was more severe and more prolonged for SLR3. Excluding the backwater effects, the impacts of SLR under SLR3 was almost 1 foot greater for minor storms, but only 0.3-0.4 foot greater for the 100 year storms. When backwater profiles were included, the effects extended upstream to the C3 canal. The depth of 100-year flooding upstream of S25B increased by 1.45 feet (from 6.55 feet ngvd to 8.00 feet ngvd). The maximum flood depth in the canal was almost equal to the maximum depth of the storm surge. SLR#3 shows a major loss of flood protection with all watersheds showing a significant increase in flood depths. Out-of-bank flows occurred for 10-year storm events. The discharge capacity of system dropped from 28 CSM to 15 CSM. Flood depths on urban sub-basins increased by 0.2 to 0.7 ft for the 25-year storm event, from moderate flooding (0.6-0.8 feet above ground surface) to problematic flooding (1-2 ft feet above ground surface). Areas near the S25B structure showing 1.6 feet deeper flood depths and the depth of flooding increased by 0.5 to 1.0 feet throughout the remainder of the watershed.

Coastal Structures

There has been a loss of capacity at the tidal structures because of sea level rise during the past 50 years but this has not affected system performance. The structures were overbuilt and system performance is limited by the canal capacity, except when storm surge creates high tail-water conditions. With sea level rise, the design tail-water elevation now occurs more frequently. The combined effects of storm surge and sea-level rise have a major impact on the LOS. The drainage network always carries water to the outlet, where ground elevations tend to be low, but when storm surge suppresses structure flow, the stormwater cannot drain through the structure but instead backs up in the canal system, causing flooding upstream of the structures.

G93 has the highest headwater control elevation as is the least impacted by sea level rise. The design tail water for the G93 structure had a return frequency of five years in 1963; currently it has a return frequency of two years. The capacity at G93 without-storm surge under SLR3 conditions is 280 cfs.

S22 is the lowest lying structure and will be affected by sea level rise before the other structures. S22 is somewhat affected by SLR2 and greatly affected by SLR3. Under SLR3 conditions, the structure would have substantially reduced capacity (down from 2000 cfs to 450 cfs) even without storm surge. Any storm surge would prevent flow at S22. At S22 the design tail-water in 1963 had a 5-y return frequency; it now has a 2-y return frequency.

S-25 has lower headwater control than G-93, but is higher than S-22. For the S25B structure, the design tail-water in 1963 had a return frequency of 12 years. Today it has a 6-y return frequency of six years. S25B discharge is impacted somewhat by SLR3 and not at all by SLR1 or SLR2. In the SLR2 scenario, storm surge suppression backed up water behind the structure and this impacted flooding in the low lying areas around the structure. The LOS was reduced from 1-in-10 to 1-in-5. Without tidal surge, the capacity at S25B, under SLR3 conditions is 2000 cfs;

Canal Capacity

The C3 canal was not improved as part of the C&SF Project (hence the G93 name) and the 400 cfs capacity of the C3 canal is much less than the 600 cfs design capacity of the structure. There is a bridge with two 12 foot diameter half-culverts that substantially restrict flows in the C3. Structural improvements at G93 will have limited impact without accompanying canal conveyance improvements

Modeling

The HEC-RAS model used for this analysis performed adequately for the purpose of this study – to determine the level of service provided by the primary drainage system. Previous SWMM modeling by Miami-Dade County provided more detailed analysis of water depths and distributions within sub-watersheds. Comparison of the two modeling efforts indicated general agreement on water depths and flows within the watershed.

Strong points of the modeling

The HEC-RAS model demonstrated a number of advantages for the District's modeling purposes relative to the SWMM modeling approach. In the HEC-RAS model, hydraulics is complete, and includes primary and secondary canals, bridges, structures and works associated with the C4 mitigation project, including flood walls, municipal pumps, impoundments, and S25B forward pumps. Watershed storage includes both above ground and below ground storage. The Model looks at the direct impact of sea level rise on flow at the downstream coastal structures and indirect impacts of sea level rise on increased groundwater stages throughout the C4 Canal. Consideration of these effects provides improved estimates of resulting impacts on base flow and reductions in available groundwater storage. In addition, use of this model provide a means to consider impacts of both storm surge and sea level rise. All structure operations are simulated including municipal pump operations, gate opening and closing, switching from spillway to pump flow at S25B, filling and draining of the impoundments, and gate operations at S380.

Known deficiencies

Results of this modeling effort also revealed a number of weaknesses and deficiencies in the model that should be addressed in future studies. Our modeling is focused on the canal and therefore sub-basin hydrology is simplified, for example, evapotranspiration is not included. The model is limited to C4 watershed with very simple representations of the C2, C3 and C5 watersheds. The C6 watershed and C6 canal are not in the model. The Miami River and the Miami River watershed are not in the model.

The model conceptualization assumes that the local drainage can be improved to the extent that the primary canal network becomes the feature that limits flood protection. This assumption will require further review in future modeling efforts. The model simulated operations of the municipal pumps, but seepage values in the model are so high that watershed stage and canal stage equalize quickly. The model therefore simulates all but one of the municipal pumps as ineffective.

The model also assumes that water levels are flat within-each sub-basin and uses equations to describe water exchange between sub-basins and from sub-basins to the canal. Seepage flow increases linearly with head gradient until sub-basin stage exceeds ground surface. Then an overland weir flow is used to simulate overland flow to the canal. There were very few data stations to calibrate groundwater model against. The auto-calibration methodology weighed canal stage very high, relative to canal flow. Calibration results indicate that the flows are insensitive to the seepage coefficients.

Some basins behaved oddly, perhaps because the seepage rates were low and/or the overland weirs are not properly modeled. The flow parameters may be appropriate for large storms such as those used in this study, but perhaps not for simulating normal or lesser rainfall conditions. The model indicated that high flows were occurring from C-4 entering into the Bird Drive Canal via the 132nd Ave canal. This flow route is not apparent based on examination of drainage system maps

The model was unable to use the District's structure flow equations. Therefore, the equations built into the model were used, which can differ by as much as 20%. Structure behavior in the model may not accurately simulate how the system would respond during an actual storm event. In addition, storm surge impacts were developed from limited (about 30 years) tidal data. Consequently, the 100-year storm surge boundary conditions have high uncertainty bands. Finally, there was no examination of uncertainty.

Modeling Assumptions and Applications

A number of basic assumptions used in this LOS modeling that should be examined in more detail.

- Further analyses of the combined effects of storm surge and sea-level rise should be conducted. This analysis assumes that the rainfall and surge have the same return period and occur simultaneously. The probability of this is not clear.
- The SFWMD rainfall pattern was used. This pattern has the most intense rainfall and probably the greatest flooding of the various design rainfall patterns available (USCOE Type II 5-d, USCOE Type II 3-d, USCOE Type II 1-d, SFWMD 1-d).
- The 1-in-2 year return period seasonal high water level is used to establish initial stages throughout the watershed.
- The initial sea level assumed a worst case scenario – the tide was at its maximum in the tidal cycle and also at its annual high when seasonal changes in water temperatures raise sea levels. While appropriate for design storms, these assumptions may be overly conservative when estimating economic impacts.
- Economic impacts assessments may require consideration of the joint-probability of initial condition, rainfall pattern, storm surge, tidal variations and sea level rise. This joint-probability approach would generate a LOS range rather than the low-value estimate of LOS that is generated by this approach.
- Evaluation of hydrologic effects of potential structural and/or operational changes that could be made to address impacts of changes in LOS.
- Evaluation of potential additional impacts of reduced seepage, increased development and changes in land use, and changes in rainfall frequency.

Flood Control Level of Service

In South Florida, ground elevations are sometimes so low that gravity drainage is difficult. Several C&SF structures are sized with only a 6-inch difference between the design headwater and design tailwater. However, sea levels in 2015 are now 6 inches higher than they were in 1950 and the design tailwater condition is no longer a rare occurrence. Operators today close tidal structures when tailwater levels are higher than headwater levels. At today's sea level, these closings only occur during the highest part of the tide cycle. Despite these occasional closures, the S25B, G93 and S22 structures can still carry design flows. even with a 1-in-10 year storm surge.

Even with the gates open, tidal pulses suppress flows, causing runoff to back-up in canals; this tidal pulsing can be seen throughout much of the C4 system. The C&SF system is a designed so that the canals and structures have matching capacities. Tidal pulses do not create problems as long as the average daily flow remains at or above the design capacity and the design stages at the structure are not exceeded.

CONCLUSION #1: Existing LOS is 1-in-10.

a. Despite today's higher sea level, the flow capacity of the S22, G93 and S25B structures can still carry the design flow capacity of the canal and the C4 canal can carry runoff from the 1-in-10 year storm without flooding, i.e. the waters stay in-bank.
b. However, during the tidal surge, structure flows are temporarily suppressed and flood waters build-up in the canals upstream of the structure. Response to larger storms brings waters out-of-bank and cause unacceptable flooding.

RECOMMENDATION 1: Examine and revise operational changes to the primary system and revised as means to reduce extent and duration of nuisance flooding.

CONCLUSION #2. For SLR1, the LOS is still at 1-in-10.

a. With SLR1 conditions, the primary system barely meets the 1-in-10 Level of Service. Nuisance flooding is worse, the duration of a flood increases by 2 days, headwaters upstream of the structure slightly exceed design, but average daily flow is still at the 2000 cfs capacity of the structure. Operational management is worse but the design would be OK without additional infrastructure changes in the C4.
b. The additional increase in sea level (0.34 ft) during the SLR1 future scenario further reduces the gravity discharge capacity of S-25 B and S-22 because it raises the normal tailwater elevation at the structure and increases the effects of tidal fluctuation and storm surge.
c. Flood depths increase during a 1-in-100 year event slightly by about 0.2-0.3 ft, and the duration of flooding increases from 12 days (current) to 14 days S22 is a problem…

RECOMMENDATION 2: In addition to operational changes to primary system, examine opportunities to improve secondary systems, infrastructure, drainage and building criteria

CONCLUSION #3: For SLR2, LOS drops to 1-in-5.

a. The current system can only marginally manage SLR2 conditions, duration of flood conditions in the canal has increased by 5 days, S22 performs poorly, S25B flows during the day of the surge have dropped well below the design 2000 cfs, and flooding is worse during a 1-in-100 year event.

b. Under the SLR2 scenario (0.80 ft) gravity discharges from the S-25B and S-22 structures are intermittently blocked during conditions of high tide and storm surge from a 1-in-5 year event. The S-25B structure continues to provide pumped discharge.

c. Flooding depths increase by 0.5 to 0.6 ft and the duration of flooding is extended to 16 days.

d. Installation of forward pumps provides a means to maintain discharge capacity despite periodic fluctuations in tailwater levels.

RECOMMENDATION 3: Consider installing a larger or additional forward pump at S25B and installation of forward pumping capacity at S22.

CONCLUSION #4: For SLR3, Everything fails.

a. The current system cannot handle SLR3 conditions.

RECOMMENDATION #4: Replace/retrofit primary infrastructure, upgrade secondary drainage systems, revise water management permitting and building criteria, and install forward pumps before sea level reaches this elevation

CONCLUSION #5: This study helps define levels of existing protection and threshold water elevations that pose potential future risks.

a. This study therefore provides an initial step to implement an Adaptive-Resilience approach for addressing impacts due to future sea level conditions (see **Figure TS-3**).

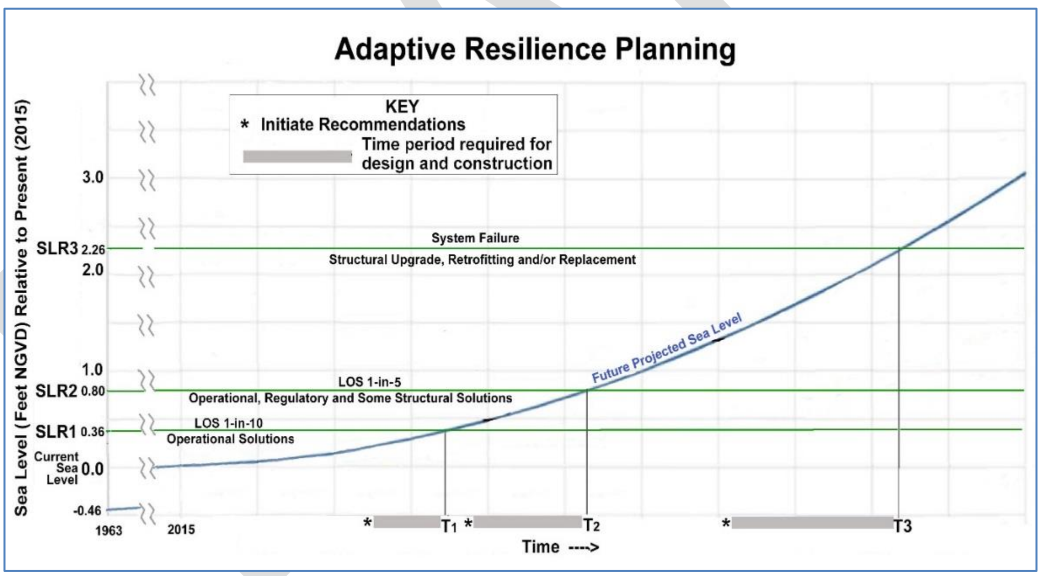


Figure 26. . Adaptive Resilience Planning in Response to Sea Level Rise. The sea level rise curve represents the projected change in water levels. Times will be established from analysis of ongoing monitoring data relative to present (2015) sea level. Sea level has already increased by 0.46 ft between 1963 and 2015. T1 through T3 indicate times in the future when sea level will reach elevations SLR1 (+0.36 ft), SLR2 (+0.80 ft) and SLR3 (+2.26 ft) described in this study, respectively. Recommended actions associated with each SLR threshold are discussed in this report and should be implemented prior to reaching each of these sea level thresholds. Shaded bars represent time periods required for design, construction and/or implementation of recommended procedures and facilities.

RECOMMENDATION 5: Efforts should be initiated in upcoming budget and planning efforts to:

a. Monitor rates of change in sea level and establish rate of change of sea level and thresholds for hydrologic changes that trigger the need for infrastructure replacement.
b. Examine and implement non-structural, operational and regulatory changes to reduce the extent and duration of flooding. Obtain better elevation data for existing buildings and infrastructure
c. Develop methods and tools needed to better asses risk, costs and benefits, lead time needed and sources of funding for constructing new infrastructure.
d. Initiate detailed planning, design and construction efforts, at appropriate time intervals to ensure that new operational protocols, permitting criteria and facilities are in place before critical thresholds for infrastructure failure are reached.

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Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed

Appendix A: LOS Basic Concepts

South Florida Water Management District
Hydrology and Hydraulics Bureau

December 29, 2015

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Flood Protection Level of Service Basic Concepts

This document was produced by the H&H Bureau as a project deliverable for project 100888 (Flood Protection Level of Service, within the Sea Level Rise project).

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Flood Protection Level of Service Basic Concepts

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EXECUTIVE SUMMARY

The South Florida Water Management District has begun the process of conducting a system-wide review of the regional water management infrastructure to determine the level of service (LOS) presently provided for flood protection. This paper provides background information on terminology and concepts used in these analyses.

Flood control within a watershed is a shared responsibility of individual landowners, homeowners associations, local water control districts and the SFWMD. Local landowners, developers and building contractors must take steps on their property to protect buildings, septic tanks and drain-fields and assure that excess surface water is directed toward adjacent wetlands, ponds, roadways, swales or ditches.

Local conveyances and storage facilities are typically managed by private entities, landowners or homeowners associations and comprise the tertiary water management system. Larger canal systems, which may include weirs, pump stations and additional storage areas are typically operated by cities, counties or local water control districts; and are considered as components of the secondary water management system. The primary water management system, operated by the South Florida Water Management District, consists of canals, often aided by pump stations and large control structures that convey water discharged from secondary facilities into storage or to natural rivers, major lakes, rivers and the ocean.

The level of service (LOS) of flood protection for a water management network reflects the amount of protection provided for specific features within a watershed. Typically there is no single LOS for a watershed. The level of service for the primary water management system is referred to as the "**primary level of service,**" and is determined as the amount of flow conveyance capacity (**Q**) needed to remove the cumulative inflows of water that occur under design storm conditions and the resulting water levels (**H)** throughout the primary canal network.

The Level of Service analysis for the primary canal system is based on a determination of flow capacity from the watershed as well as consideration of upstream water levels in the canal that are used as the basis for water management operations. The level of service has changed over the life of the C&SF project. One major factor causing change is sea level rise, but other factors, including changes in the secondary water management network, canal conveyance improvement projects, the addition of storm-water detention, and changes in water tables can all influence peak flows and peak flood depths.

Hydraulic models are used to assess these changes. The impact of sea level rise on structures is quantified using six LOS performance metrics **-** two for the primary water management system, three for sea level rise impacts and one for the secondary system. The first metric for the primary water management system shows the maximum canal stage from a design rainfall. The second metric for the primary water management system shows the peak discharge capacity of the primary canal network. The first metric for the impact of sea level rise looks at structure flow under design headwater conditions. The second metric for the impact of sea level rise looks at watershed runoff under design rainfall conditions. The third metric looks at the duration of flooding in the primary canal network. Lastly, the metric for the developed lands shows the combined ability of primary and secondary systems to maintain pre-established peak water levels needed to protect features within the sub-watersheds.

Flood Protection Level of Service Basic Concepts

Flood Protection Level of Service Basic Concepts

INTRODUCTION

The South Florida Water Management District (SFWMD) has begun the process of conducting a system-wide review of the regional water management infrastructure to determine the level of service (LOS) presently provided for flood protection. The LOS describes the amount of protection provided by water management facilities within a watershed. The purpose of this system-wide review is to determine the LOS provided by existing water management systems in watersheds throughout the District. The review considers sea level rise, future development and known water management issues in each watershed. This information can then be used by local governments, the SFWMD and other state agencies, and federal partners to determine areas where improvements to the design and construction or upgrade of water management facilities are required, the appropriate entity or entities responsible for making these improvements and funding and technical resources available to support these efforts. This paper provides background information on terminology and concepts used in these analyses, so that citizens, resource managers and stakeholders can better understand and appreciate the nature and scope of these investigations, and participate in their review and future refinements.

The existing facilities of the Central and Southern Florida Project for Flood Control and other Purposes (C&SFFC Project) were initially designed by the United States Army Corps of Engineers (USACE) and the Central and Southern Florida Flood Control District (CSFFCD, which became the SFWMD in the early 1970's) in the 1950's and constructed during the 1960's and 1970's, based on a 50-year projection of water management needs. Much of the original infrastructure today is beyond its design life and needs to be replaced or refurbished. In addition, the current distribution of population and development in South Florida differs substantially from the original projections (**USACE, 1952; 1954**). At the same time, many changes have been made to the original design and operations of the water management infrastructure during the past fifty years by private development, local governments, the SFWMD and USACE. The SFWMD and local entities have also implemented regulatory requirements to make efficient use of existing water management capacity and improve local water management facilities. Population increases and changes in land use and development have altered the needs for flood protection within the watershed. The perception is that the existing water management infrastructure still provides adequate flood protection, but this assumption needs to be verified by systematic analysis of individual watersheds.

An emerging critical issue for many parts of South Florida is sea level rise. Since the original project was designed, South Florida has experienced an increase in sea level of approximately 5 inches (**Obeysekera et al. 2011**), which was not adequately considered in the initial design. In addition, the region faces the prospect of accelerated rates of sea level rise in the foreseeable future (**SF Climate Compact Report, 2012**). Because of the uncertainties regarding current capabilities of the regional water management system and the perceived need to plan for higher sea level conditions in the future, the SFWMD has initiated two continuing projects.

The first project will update the description and operation of the existing water management system. Basin Atlases are being prepared to provide comprehensive descriptions of infrastructure and water management operations, as well as emerging issues within each basin. The initial atlases will consist of documents that update reports and studies that were written in the 1980's and 1990's to examine various watersheds within the SFWMD. The next phase of the Basin Atlas project will create online dynamic versions of the atlases in which data and descriptions will be updated as changes occur.

The second project will reexamine the level of service for flood protection that is presently being provided in the various watersheds. The initial focus of these studies will be to determine the existing capacity of the primary stormwater management system (C&SFFC Project) to remove water that is discharged into the canals from adjacent local stormwater management networks. The next step will be to determine whether the existing system meets the original design capacity and, if not, to determine the current level of service. Additional analyses will then be conducted to determine how potential increases in sea level may affect the level of service provided by the primary stormwater management system in the future.

BASIC CONCEPTS

Flood control within a watershed is a shared responsibility of individual landowners, homeowners associations, local water control districts and the SFWMD (**SFWMD, 2014**). Several entities are responsible for managing water that leaves or accumulates on individual properties. In most cases, for private landowners, the amount of flooding that occurs is determined by the limitations of the local water management network.

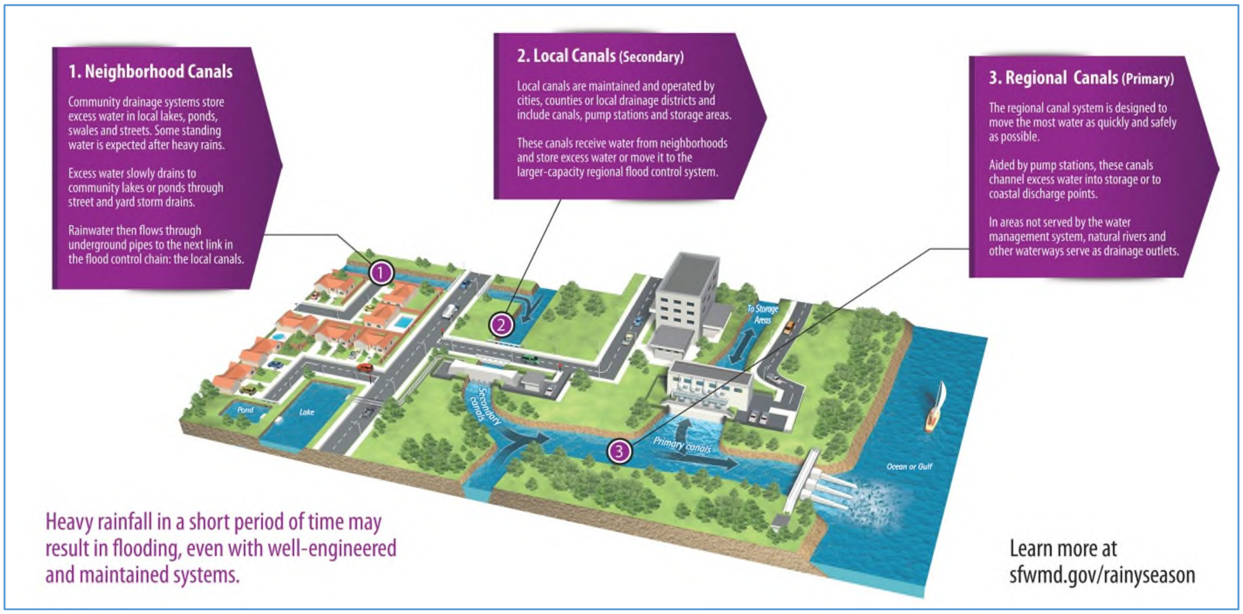

Figure A-1. How a storm-water management system works. Water from private yards and neighborhoods is discharged to secondary canal systems operated by local government entities and then to the regional canal system that is managed by the SFWMD and USACE.

Homeowner Responsibilities

Most of the water that falls on the landscape during a light rain evaporates or soaks into the ground where it provides moisture for plants or provides recharge to groundwater aquifer. During moderate rains, water cannot soak in fast enough and forms puddles or collects in depressions, ponds or swales. This water eventually soaks into the ground or evaporates. This accumulation of surface water is a normal condition that is designed to keep water away from homes, capture water in the ground for future use, and minimize the amount of water that becomes storm-water runoff. Local landowners, developers and building contractors must take steps on their property to ensure that buildings, septic tanks and drain-fields are elevated to receive appropriate protection from flooding and that excess surface water is directed toward adjacent wetlands, ponds, roadways swales or ditches. In addition, landowners need to ensure that channels on or near their property are not blocked by dirt or debris.

Tertiary Water Management Systems

During moderate rains, the puddles and swales overflow and the water begins to flow as runoff toward ditches, channels, or storm drains. This runoff water is often contaminated with bacteria, chemicals and pesticides that may potentially impact the environment. The channels and drains generally empty into ponds, lakes, wet and dry detention systems, or retention wetlands that are designed to hold first-flush runoff. Living organisms and sediments in the lakes and wetlands help remove contaminants and provide more time for water to soak into the ground. These local conveyances and storage facilities are typically managed by private entities, landowners or homeowners associations and comprise the tertiary water management system. The SFWMD's Regulation Division has established design criteria for these systems.

Secondary Water Management Systems

When local facilities fill up, the water overflows through a water control structure into larger canal systems, which may include weirs, pump stations and additional storage areas. These larger capacity facilities are typically operated by cities, counties or local water management districts; and are considered as components of the secondary water management system. Design criteria for these systems are also specified by the SFWMD's Regulation Division.

Primary Water Management System

During heavy rainfall events, or after a series of lesser rainfall events during extended wet periods, the local groundwater becomes saturated and secondary water management systems reach their capacity. Excess water from the secondary systems is then discharged into natural rivers and outlets that discharge to major lakes, rivers and the ocean, or into the regional primary canal system operated by the South Florida Water Management District. Aided by pump stations and large control structures, this primary water management system channels water to storage or into a receiving body.

Design Storm

The design storm is not a real storm but rather a synthetic event with a clearly defined volume, duration, intensity and pattern. The design storm provide a consistent method of analyzing the response of a watershed and for sizing flood control structures in the watershed. The design storm is the most severe storm that the canals and structures in a watershed can manage without flooding occurring in the watershed. Sometimes a watershed is described as having "flood protection" up to a certain design storm. In other cases, features within the watershed have specified levels of flood protection.

A design storm is described by the frequency with which it may occur. On a long term average, a storm of given intensity may occur, for example, once in every ten years. This magnitude of storm is said to have a ten percent chance of occurring in any given year and is called a one-in-ten-years (1-in-10) storm event. It must be emphasized, however, that in reality a storm of a given intensity can occur at any time, regardless of the frequency assigned to it.

The amount of runoff generated from a watershed by a severe storm depends on many factors: water table elevation, land use, soil type, the presence of lakes or canals, and the distribution of rainfall temporally and spatially. The amounts of runoff that occur for design storms at various recurrence intervals are typically estimated as part of the system design and permitting processes for local water management systems.

Level of Service

The level of service (LOS) of flood protection for a water management network reflects the amount of protection provided for specific features within a watershed. A set of six metrics are applied by the SFWMD to quantify this protection. Five metrics apply to the primary water management system and measure the amount of flow released from the structures under design conditions, the peak flow capacity of the structure, and water levels upstream of the control structure. A final metric measures the ability of secondary and tertiary water management facilities to maintain pre-established water levels that are needed to protect features within the subwatersheds.

Flood Protection Level of Service Basic Concepts

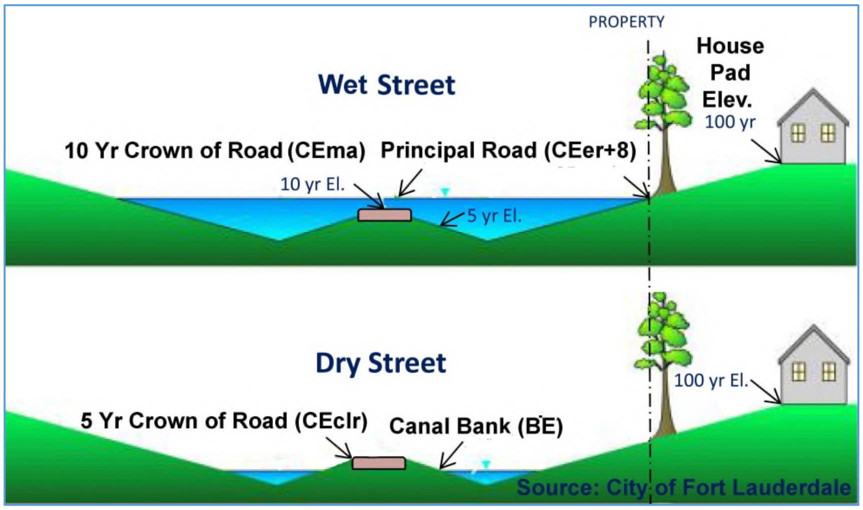

Figure A-2. An example of relative elevations of manmade features that are used to define Level of Service (LOS) within a watershed.

Typically there is no single Flood Protection LOS for a watershed but rather a family of features each with its own LOS. For example, local stormwater infrastructure in residential neighborhoods is generally designed for a storm event with a 20 % annual exceedance probability (a so-called "5 year storm"). The floors of residential homes, however, should be protected during a storm with 1% annual exceedance probability (a "100-Year storm"). The primary canal network may be able to accommodate a storm with a 4% annual exceedance probability (a "25-year storm"). Thus in the case of a major flood event, the expectation is that the roads and grounds of properties may become flooded while house pads are not. An example of how such level of service criteria might be expressed for a watershed is provided later in this report

This protection is achieved through the combined effects of primary, secondary and tertiary water management infrastructure at a particular location within the watershed. Each of these infrastructure levels serves a different function and thus has different specifications for their system. The secondary and tertiary water management infrastructures are designed to maintain stages needed to protect physical features within the watershed. The primary system infrastructure is designed to provide the water levels in major canals and flow capacity needed to remove the water discharged from secondary and tertiary systems. The three levels of infrastructure working in concert provide the overall levels of service needed throughout the watershed.

The primary water management system in most south Florida watersheds consists of the canals and facilities that are operated by the SFWMD. These canals receive inflows from secondary and tertiary water management facilities operated by local governments and water control districts. The level of service for the primary water management system is referred to as the "**primary level of service,**" and is determined as the amount of flow conveyance capacity (**Q**) that is needed to discharge water to regional storage or to tide to avoid flooding while not exceeding the design maximum water levels in the canals.

Comparing 'Design' and 'Operational' Level of Service

The Flood Protection Level of Service program focuses on the long term infrastructure needs of the District. Infrastructure needs are based on analyses of the system under design flood conditions. Design floods represent an unfortunate set of circumstances which, while rare, have a reasonable chance of occurring sometime during the life of the system. Such events are based on a combination of conditions such as excessive rain that covers the entire watershed, soils that are already wet from previous rainfall, higher than normal tides, and a tropical storm with both high winds and storm surge. The chances of these conditions happening simultaneously are not as improbable as first appears, since they are linked to one another – but quantifying the probability is difficult. The USACE used best-professional-judgment in the original C&SF design. The methods used in this study are an update of the USACE methods.

In day-to-day operations, attention shifts away from the response of the system under design flood conditions toward the response of the system under the wide range of conditions experienced in normal operations. Operators must maximize flood protection while simultaneously managing the system for the other District priorities of water supply, water quality, and ecosystem protection. To do this, operators need to understand the sensitivity of flooding to initial conditions, canal maintenance, and vegetation management. In a well maintained system, the effects of a large rainfall may be mitigated, relative to design conditions. Thus if rainfall occurs locally, or initial soil conditions are dry, or if tides are not a concern, then the impacts will be reduced. In addition, the effects of a large rainfall can, under favorable circumstances, be mitigated through operations – by pre-storm drawdowns that lower canal levels and in turn lower regional water tables, by managing flows at structures to mitigate local flooding effects, or by diverting flood waters into adjacent canal systems that have excess capacity. On the other hand, the system response may be made worse by blockage of a canal or by maintenance issues such as canal constrictions, weeds, and poorly maintained structures.

Whether examining ‘design’ LOS or ‘operational’ LOS, the modeling tools are the same: a hydrologic model that generates runoff hydrographs from the land into the canal and a hydraulic model that routes the runoff through the primary canal network. But the tools may not be configured identically. The ‘design’ tools must simulate conditions of the extreme events: out-of-bank flow; suppression of runoff when canals are full; high tail-water at the outlet structures. ‘Operational’ LOS studies require flexible tools that can examine a range of conditions. One study might require long-term simulations to show the effect of pre-storm water management; another study might require detailed information to examine the effectiveness of coordinated structure operations; a third study might evaluate the effectiveness of canal maintenance. Similarly, performance metrics, while the same for ‘design’ and ‘operational’ LOS studies will emphasize different performance goals. Generally the ‘operational’ LOS studies focus on flooding experienced on developed lands (Performance Metric #5 below) rather than metrics related to system conveyance (Performance Metrics #1 to #4 and #6 below).

EVALUATION

The Level of Service analysis for the primary canal system is based on water levels in the canal and the ability to convey water. When upstream water levels are too high, there is both a direct risk of flooding as water overtops banks in the primary and secondary canals and a secondary risk of flooding because discharges from the secondary structures are inhibited by high canal stages. Therefore, when water levels in the canal reach certain limits, a decision is made to open the downstream structure or to start the pumps and move water from the canal to the ocean or into a reservoir.

Water levels in a canal generally become higher as one moves inland away from the structure. These increases are greater during periods of heavy rainfall when additional surface water and groundwater flows into the canal. The rate of flow out of the canal is based on the downstream water control structure characteristics (structure type, number of gates, gate width, sill elevation, etc.). The difference in elevation between the water above the structure and the water downstream from the structure determines the rate of discharge. Under normal operations, water levels upstream from the structures are significantly higher than water levels downstream (see **Figure A-3**). During heavy rainfall events, downstream water levels may increase so that the structures cannot discharge enough water to prevent flooding (**Figure A-4**).

Flood Protection Level of Service Basic Concepts

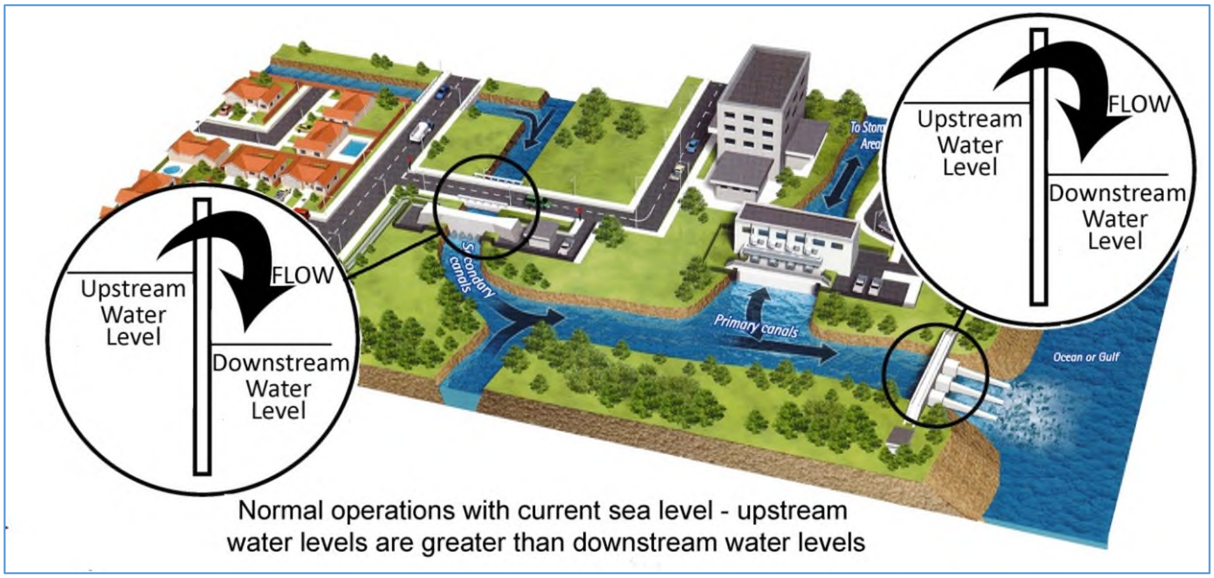

Figure A-3. Normal operation of a water management system: Water levels in primary canals are higher than sea level. Water flows out through the structure by gravity. Water levels in secondary canals are higher than levels in primary canals, so that water drains from secondary systems into the primary canal.

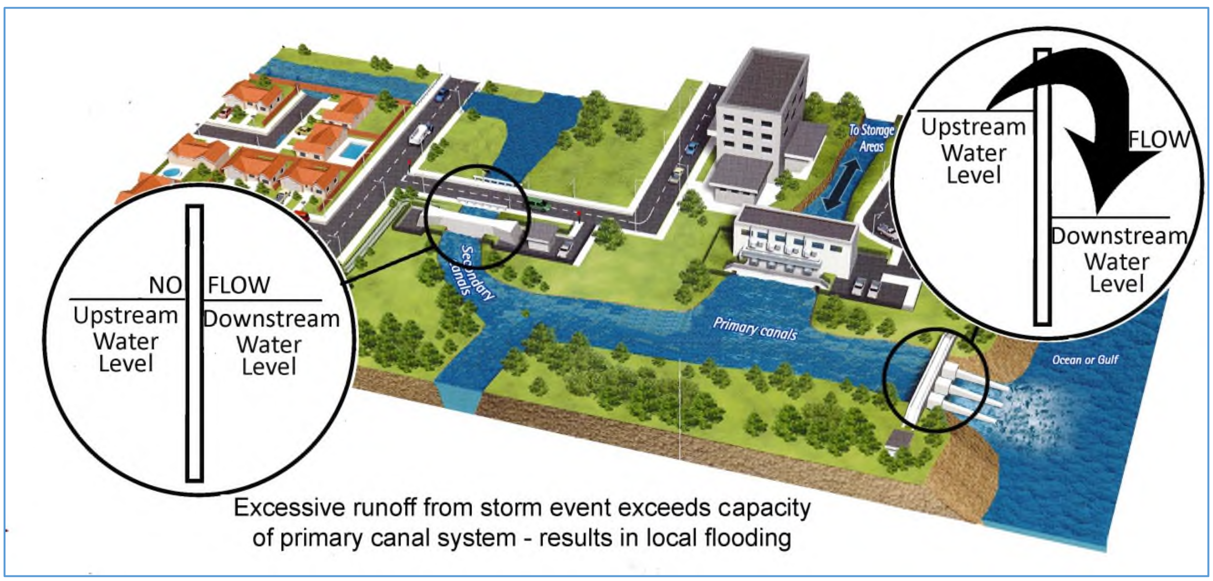

Figure A-4. Operation of a water management system during an excessive rainfall event: More water flows into the canal from upstream than can be discharged through the coastal structure. Water levels in primary canals are equal to or higher than water levels in secondary canals. Local Flooding occurs.

The LOS provided by the canal depends both on how much water the outlet structure can discharge and how much water the canal can carry. To provide as much protection as possible, water levels in the canal are carefully monitored, structures are opened, and pumps are operated when possible, to move excess water to the ocean or into a reservoir, to prevent upstream water levels from becoming too high.

The LOS has changed since the time (more than 60 years ago) when the C&SF Project facilities were first designed. In the 1950's, south Florida had more farms and fewer people, houses and businesses. More of the landscape was undeveloped and canals in agriculture areas were sized assuming that flooding could be tolerated every five to ten years. In addition, sea levels in 2014 are about 5.7 inches higher than they were in 1955 and it is anticipated that sea levels will continue to rise in the future. Sea level rise can reduce flows at tidal structures, raise groundwater stages throughout the watershed, raise water levels in the primary canals, and increase the depth and duration of flooding in the secondary systems (**Figure A-5**).

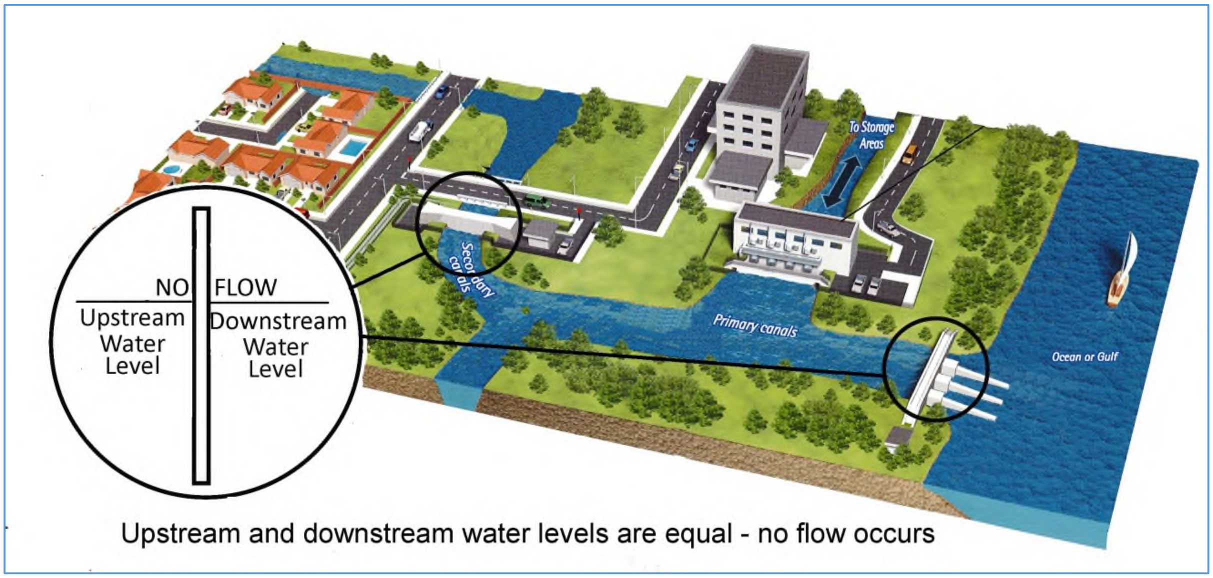

Figure A-5. Effect of Future sea level rise on the ability to discharge water from coastal watersheds. At high sea level conditions, such as storm surges and extreme high tides, coastal structures cannot release water. Water backs up in the canals to the extent that secondary structures also cannot discharge, and local flooding occurs.

Other factors, such as the following, can also impact the level of service by influencing peak flows and peak flood depths:

- Changes in original basin size due to re-plumbing or improved basin delineation,
- Changes in canal cross sections due to bank erosion, sediments deposition, bridges.
- changes in rainfall due to climatic trends or improved estimation techniques,
- changes in land use,
- changes in the secondary water management network canal conveyance as a result of improvement projects,
- the addition of storm-water detention systems, and
- changes in water tables

In order to consider all of these different factors, a computer representation of the water management system (called a hydraulic model) is created. This allows engineers to change different features of the system (water levels, structures, canal sizes, etc.) and test their performance over a range of possible present and future conditions of rainfall, water levels, land use and water management configurations within the watershed.

The model uses the information provided concerning future conditions and produces a simulation of resulting water level conditions throughout the watershed, indicating water depths at different locations, and water movement through the canals and structures before, during and after a rainfall event. The numerical results given by the model are then further analyzed to produce tables, maps and graphs that show the extent, depth and duration of flooding within the watershed.

Estimating Future Sea Level Rise

There is widespread consensus that the sea level has been rising steadily during the last century and will continue to increase at a higher rate globally in the future. The estimates in **Figure A-6** are provided using the estimated mean sea level in 2005 as the reference.

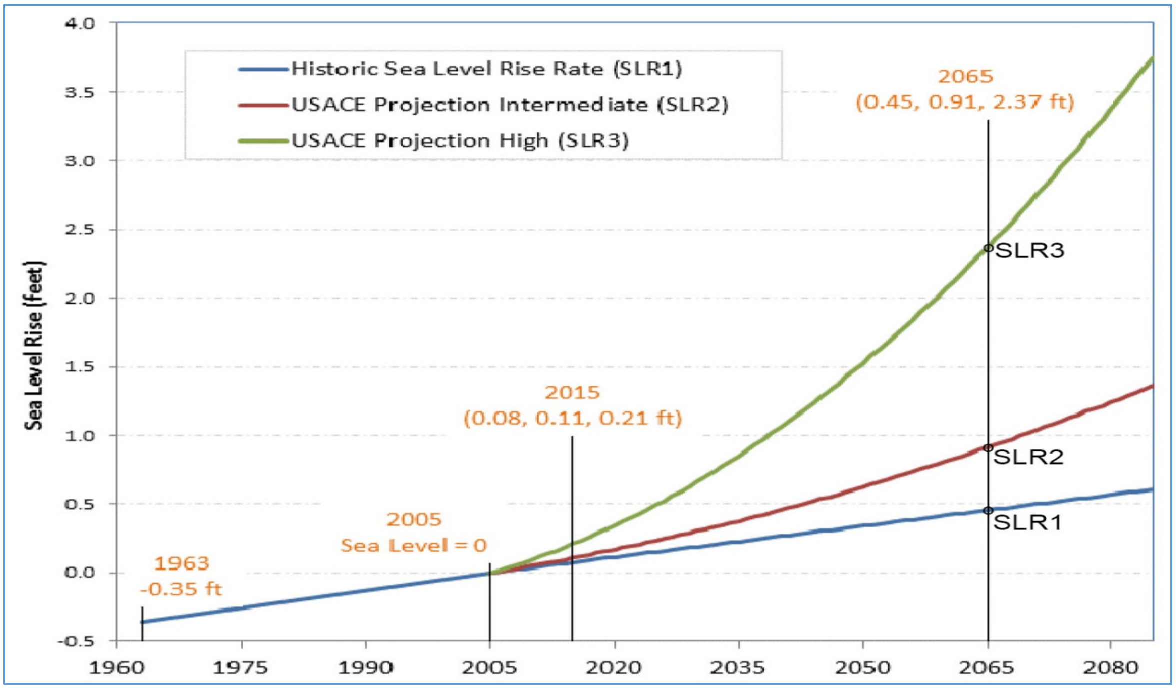

Figure A-6. Sea level rise projections developed by the USACE (2014) for use in evaluating future projects. Historic projected rate (blue line) is compared to a range of possible future rates that depend on different future climate scenarios. SLR1-3 represent projections of future conditions that are being considered for SFWMD watershed studies (see text). The above curves correspond to S-25B structure.

Because of low land elevation and flat topography, south Florida is particularly vulnerable to changes in sea level. The United States Army Corps of Engineers (**USACE 2013, 2014a, 2014b**) has developed guidance documents that are intended to provide a consistent approach to future changes in sea level. All planning efforts for USACE projects are now required to consider the effects of sea level rise on project design and performance. To facilitate this evaluation, the USACE has developed estimates of the rate of sea level rise that need to be evaluated. Based on the guidance provided by the USACE and a detailed analysis of water levels downstream of coastal structures, site specific projections of mean sea level rise curves have been developed. **Figure A-6** shows historic increases between 1960 and 2005 and projected increases between 2005 and 2065 for the S-25B structure. A projection of mean sea level back to early 1960s used the linear rate that has been observed at the Key West tide gage.

Since the magnitude of sea level rise is uncertain, the USACE requires that a range of values for the estimated amount of sea level rise is to be evaluated to bracket system behavior. The lowest projection (SLR1) is an extension of the historic rate and anticipates an increase of 0.45 feet by 2065. The other two levels represent the anticipated increase in sea level based on global climate change projections and range from 0.9 feet (SLR2) to about 2.4 feet SLR3) by 2065. These projections need to be considered in evaluation of the level of service that will exist in the C-4 watershed in the future.

Quantifying Flood Protection Level of Service

Current and Future Conditions

In order to determine the level of service provided by the water management system, five scenarios are analyzed:

1. Design Conditions: the original specifications that were developed by the USACE (1963) and used in the design and construction of project facilities during the 1950's and 1960's.
2. Current Conditions: existing condition and specifications for project facilities, existing population and land use that exist today after fifty years of system maintenance and improvements and an increase in downstream water levels of approximately 5 inches of sea level rise occurring between 1963 and 2010.
3. Future Conditions: future conditions that consider the effects of sea level rise, both the direct effect of increased tail-water levels downstream from the control structures and the less obvious impacts of sea level rise on groundwater levels in the watershed that in turn increases base flows in the canals and increase rainfall-induced runoff. Future conditions also include hydrologic changes caused by development, changes caused by restoration, as well as changes related to water deliveries and well-field protection. Since future conditions are uncertain, three levels of impact are analyzed – LOW, INTERMEDIATE and HIGH:
   a) SLR1 – LOW - Historic rate of sea level rise projected to 2065 (0.45 feet)
   b) SLR2 – INTERMEDIATE - A rate of sea level rise based on a modest change in climate conditions (0.91 feet)
   c) SLR3 - HIGH – An elevated rate of sea level rise based on a greater change in climate conditions (2.37 feet).

Measuring Effects on Level of Service

Five performance metrics have been developed to quantify the flood protection Level of Service (LOS). Two focus on the primary canals, two focus on the impacts of sea-level rise and one examines frequency of flooding on developed lands.

Flood Protection Level of Service Metrics

Performance Metrics that focus on the Primary Canal Network
- #1: Maximum Stage in Primary Canals
- #2: Maximum Daily Discharge Capacity throughout Primary Canal Network

Performance Metrics that focus on the Impact of Sea-Level Rise
- #3: Tidal Structure Flow Performance – effects of sea level rise
- #4: Peak Storm Runoff – effects of sea level rise

Performance Metrics for Developed Lands
- #5: Frequency of Flooding

Performance Metrics for Operations
- #6: Duration of flooding

The hydraulic models are used to analyze each of the scenarios for a range of design storm conditions. In order to determine how the level of service is affected under current and future conditions, data resulting from application of the model are analyzed to produce tables and graphs that represent conditions in the watershed under each scenario. Scientists study and interpret these model "outputs" as ways to assess the extent of flooding and potential damage that may occur.

Performance of the regional water management facilities under current and future conditions is assessed based on a determination of whether the flow capacities of the primary canal system and at the downstream structures meet their design specifications and whether water levels within different parts of the watershed are maintained at or below the levels specified for the design storm. This performance is quantified using four LOS performance metrics: two for the primary canal system and two for the tidal structures:

LOS Metric #1: Maximum stage in primary canals

LOS Metric #1 is the peak stage profile in the primary canal system. This profile is developed for a range of design storms. The largest design storm that stays within the canal banks (with a prescribed freeboard) establishes the LOS of the primary canal system.

During large rainfall events the runoff generated from the watershed may exceed the design discharge capacity of the primary system, resulting in higher water levels within the canals. This condition is represented in **Figure A-5**, above**.** The capacity to remove water from the watershed may be limited either by the conveyance capacity of the canal or the discharge capacity of the water control structures or both. In either case, water levels in the canals increase beyond their design stages, reducing the capacity for secondary canals to discharge to the primary system and increasing the possibility of local flooding (**Figures A-7 and A-8**).

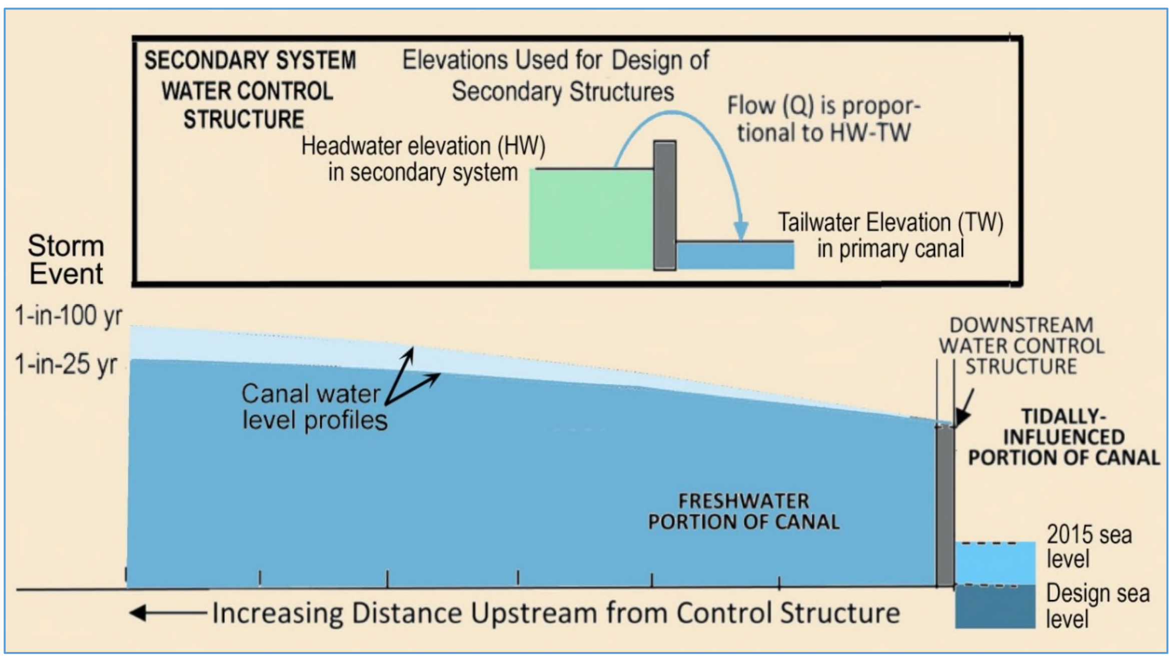

Figure A-7. Schematic water level profile upstream of a tidal structure. Water levels increase with distance upstream from the control structure. Canal profiles provide tail-water elevations that are used to design and size secondary water control structures. Under design storm conditions, water levels in canals may exceed canal bank elevations resulting in local flooding. Water levels within the primary canal may constrain discharges from secondary sub-watersheds.

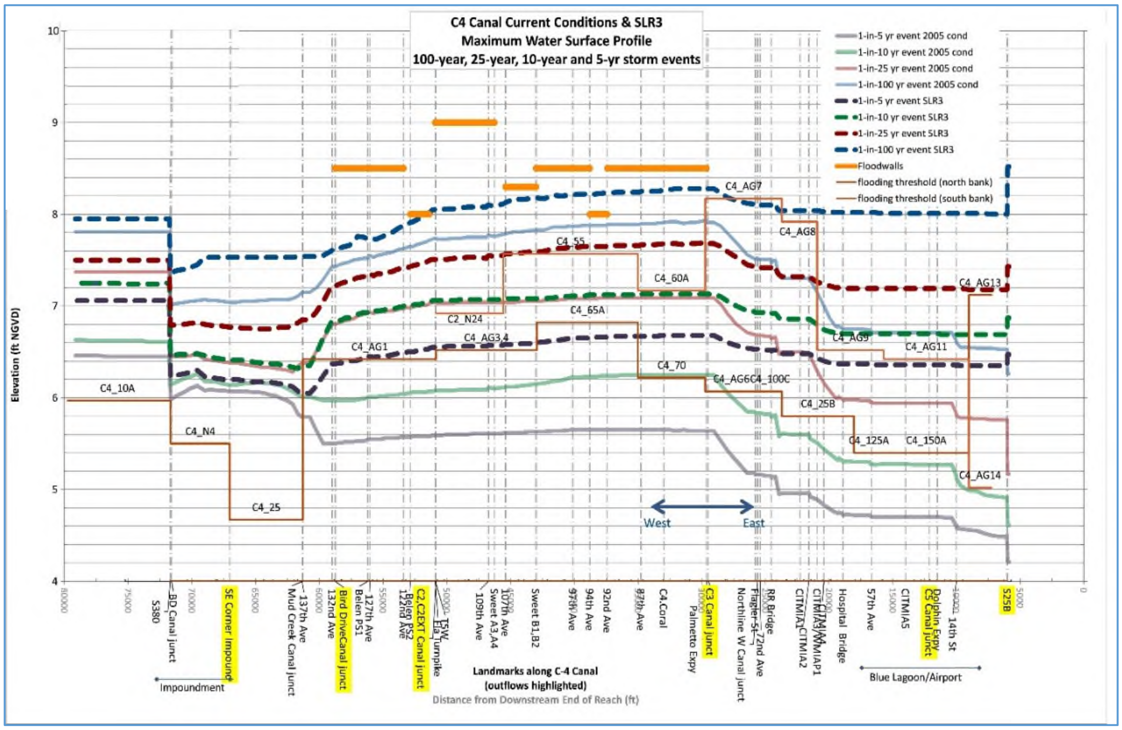

Figure A-8. Water level profile of the C-4 canal upstream of tidal structure S25-B, comparing 2050 SLR#3 to current conditions. LOS is below 1-in-5 for SLR#3. Flood depth increases all along the canal but is worst near the structure. Backwater effects extend to the City of Miami municipal pump locations for all events. The maximum flood depth is now at 8 feet, while tail water stage was at 8.5 feet. Caution: this is an example figure based on early modeling.

This performance metric (**Figure A-8**) is based on interpretation of the results of model runs to determine upstream water levels in the canals during design storm events. These values will be compared to bank elevations of the primary canals, design elevations of secondary water control structures, and elevations at strategic monitoring sites within the watershed that are used in District operations to manage operations of flood control pumps, structure and reservoirs.

Future increases in sea level are expected to result in higher groundwater stages in the watershed, which will lead to even higher stages in the canals and more runoff from the watershed, increasing the likelihood that flooding will occur during storm events.

LOS Metric #2: Maximum Discharge Capacity throughout the Primary Canal Network

LOS Metric #2 shows the maximum discharge capacity throughout the primary canal network. Discharge is shown as aerially weighted flow (CSM or cubic feet per second per square mile). [Technical note: Tidal effects are eliminated by using a 12-hour moving average of flow]

Based on the C&SF system capacity as defined in the USACE reports, SFWMD has established discharge rates for basins throughout the District (**SFWMD ERP Manual 2014**) that provide the basis for issuing Environmental Resource Permits. These rates are expressed as aerially weighted flows (CSM or cubic feet per second per square mile) and are associated with a design level of service. The rates are used to size discharge structures within the basin (**Table A-1**). In many cases these discharge rates were established as part of the design of the primary water management system (**Figure A-9**).

Table A-1. C-4 Canal Discharge Capacity in cubic feet per second per square mile (CSM) - example

Tidal Condition	Return Period of the Design Storm			
	5-yr	10-yr	25-yr	100-yr
Current	27	28	29	30
SLR1	27	28	29	29
SLR2	26	26	28	28
SLR3	15	17	20	21

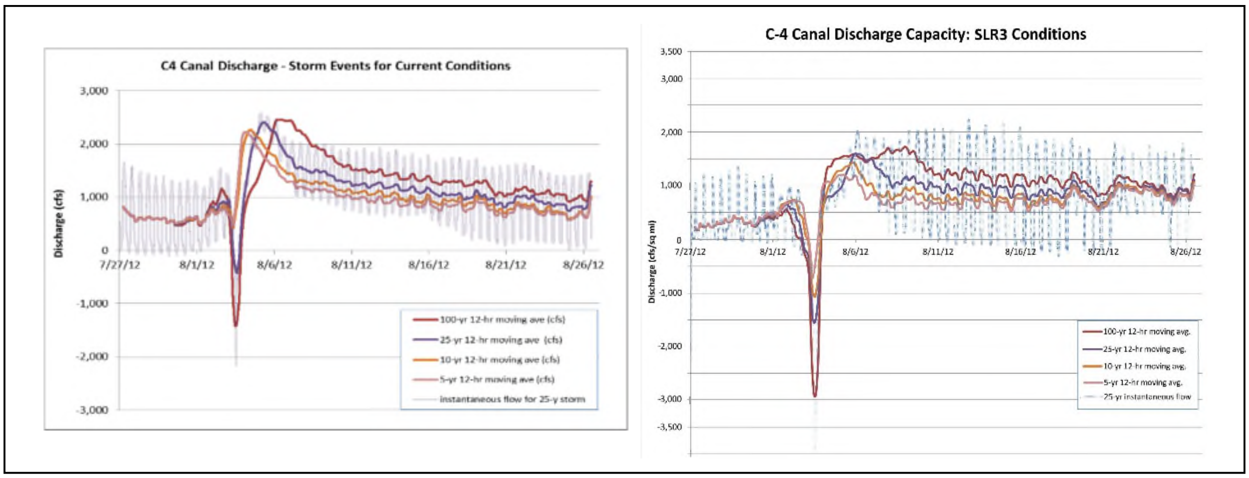

Figure A-9. Maximum discharge flow capacity in cubic feet per second for four storm events for current conditions. The peak occurs later and lasts longer with larger storm events. The peak discharge capacity is the same for CSL, SLR#1 and SLR#2 events, but declines by 1/3 for SLR#3.

LOS Metric #2 is based on analyses of the results from a hydraulic model of the watershed to quantify the discharge capacity throughout the primary water management network and for the entire watershed. These modeled discharge rates can be compared to permitted discharge rates. Discharge capacity can vary within the watershed due to changes in land use. For example, undeveloped lands with no drainage system will generally have low discharge rates, while lands with high-value agriculture have aggressive water management systems and are operated at high discharge rates. On the other hand, discharge capacity can vary due to conditions in the primary water management network. For example, an undersized water management canal will reach capacity early, cause high canal stages, and result in local flooding. Used together, LOS Metrics #1 and #2 can distinguish between areas with low internal water management and areas where the primary canal network has limited conveyance.

LOS Metric #3: Structure Performance – effects of sea level rise

Metric #3 shows the effective capacity of the tidal structures and is comparable to the static, design condition assumed in the original structural design. Metric #3 compares structure flow over a range of storm surge events and a range of sea level rise scenarios. This analysis is used to identify tidal structures in need of eventual modification and helps prioritize the need for redesign relative to other structures.

As sea-level raises tail-water elevations, tidal structure flows may be suppressed. PM #3 has been developed to show the impact of sea level rise and storm surge at a structure. The metric is developed from simulations of structure flow during conditions when the upstream stage is fixed at the design headwater stage and the downstream stage oscillates with the tide (**Figure A-10**). Structure performance based on this metric can be compared to the original structure design, which was based on the cross-sectional area of the structure opening, the design upstream water level and the maximum observed[1] downstream water level. In current modeling efforts, the tide cycle is considered along with transitions from gravity flow to pumped flow at some structures. With dynamic modeling, flows vary throughout the tide cycle even when headwater is constant. Consequently, average flow over the tide cycle is used to determine effective capacity and to compare to design flows. **Figure A-11** shows an example of PM#3.

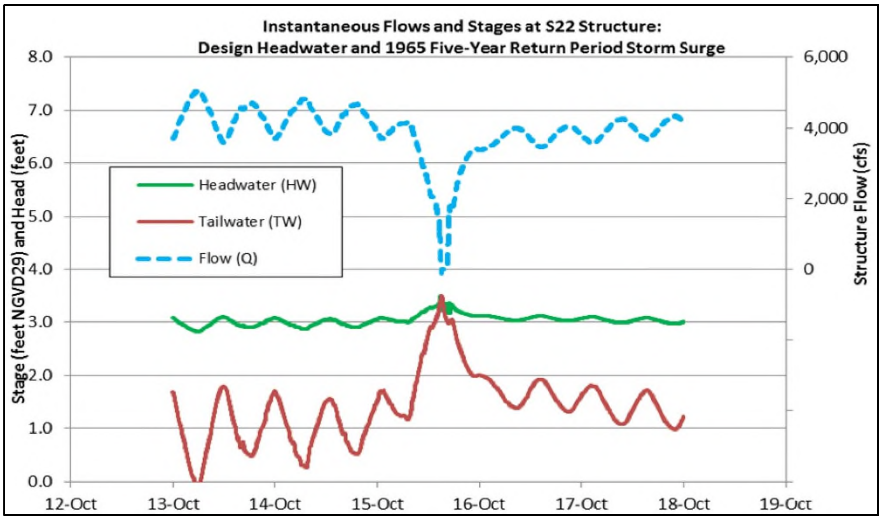

Figure A-10. Discharge capacity from a coastal structure is suppressed by tidal surge during a storm event

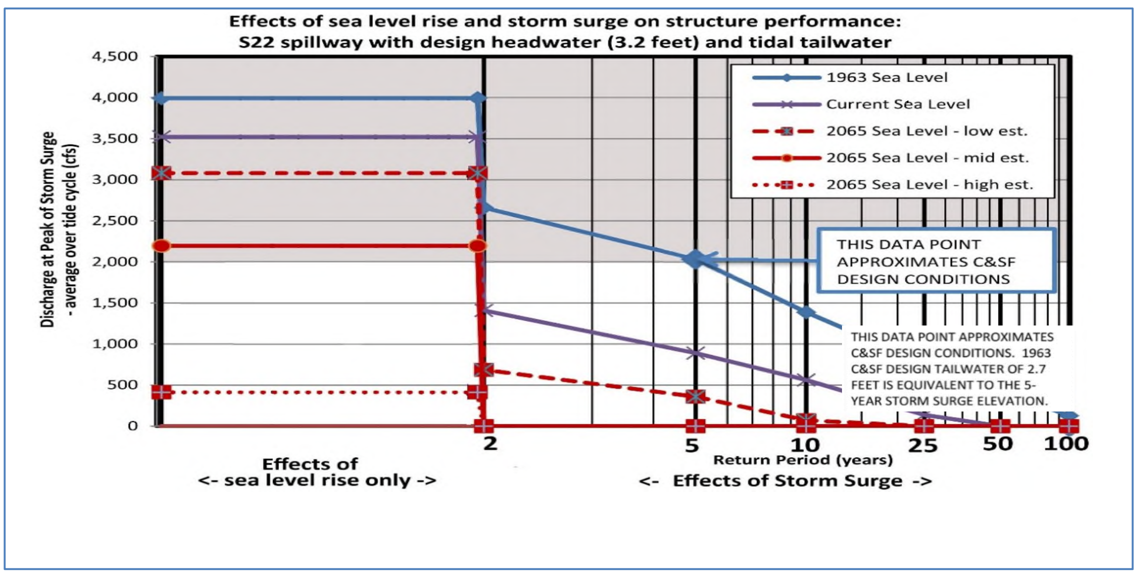

Figure A-11. Flow through a coastal structure under historic (as designed in 1963) vs current (2015) and future sea level conditions for a range of storm surge conditions. Sea level rise causes both a long-term suppression of flow, as seen for the “King Tide” with no surge, and a short-term suppression, as seen by reduced flows during storm surge (see Attachment B for additional explanation).

[1] In the C&SF design, desing tail-water stage was developed from a limited record of tide station data. The return period of the design tail-water can be estimated by examining the suite of 1963 design storm-surge time-series and comparing the 12-hour moving average of the tail-water stage to the design tail-water. Using this method for the S-22 structure, the 1963 design tailwater had a 1-in-5 year return period.

Figure A-11 shows the S-22 spillway's capacity for different sea-level rise scenarios (1963 design, 2015 existing, 2065-LOW, 2065-INTERMEDIATE, and 2065-HIGH). Each scenario has a suite of tidal boundaries (2-yr to 100-yr return period). The capacity is defined as the minimum 12-hour flow that occurs during the tidal surge. The original C&SF Project design flow was 1,915 cfs for a design headwater of 3.2 feet and tail-water of 2.7 feet. The five-year return period storm surge for 1965 sea level conditions has a 12-hour average stage of 2.7 feet, which is the same as the design tail-water. The dynamic model capacity of 2,030 cfs is very close to the design flow.

The structure capacity values for the 0-year return period storm surge can be compared to one another to show the effects of sea-level rise alone, without storm surge. These impacts would be observed each year during the king-tide period (Sep-Nov), irrespective of storm surge.

Storm surge further reduces the structure capacity. The surge effect is limited to the duration of the surge (less than one tide-cycle) but nevertheless greatly reduces structure discharge rate. As sea level rises, smaller and more frequent storm surges have the same impact on structure capacity that large and infrequent storm surges had when the sea level was lower. A detailed description of PM#3 is presented in **Attachment B.**

LOS Metric #4: Peak Storm Runoff - effects of sea level rise

Metric #4 shows the maximum conveyance capacity of a watershed at the tidal structure for a range of design storms. It shows the maximum conveyance (moving 12-hour average) for a specific design storm and a specific tidal boundary condition. This metric examines the behavior of the system under severe stress and can be used to check if conditions exceed design limits.

PM#4 examines peak storm runoff at the tidal structure. Unlike PM#3, which focuses on the structure and tidal conditions, PM#4 looks at the response of the entire watershed to design events. For the design event, design rainfall and design storm surge can be assumed to occur simultaneously or with a temporal offset that maximizes stress on the structure. The results can reveal flows and stages that exceed the design capacity of the structures, indicating a need for a more detailed structural stability analysis.

A full watershed model is needed for PM#4, even though the metric only examines flow at the tidal structure. Metric #4 shows the maximum conveyance capacity of a watershed at the tidal structures for a range of design storms (**Figure A-12)**. Like Metric #3, Metric #4 is based on examination of a range of tidal boundary conditions but this metric looks system response to design rainfall as well as design tide including the effects of canal conveyance and watershed hydrology.

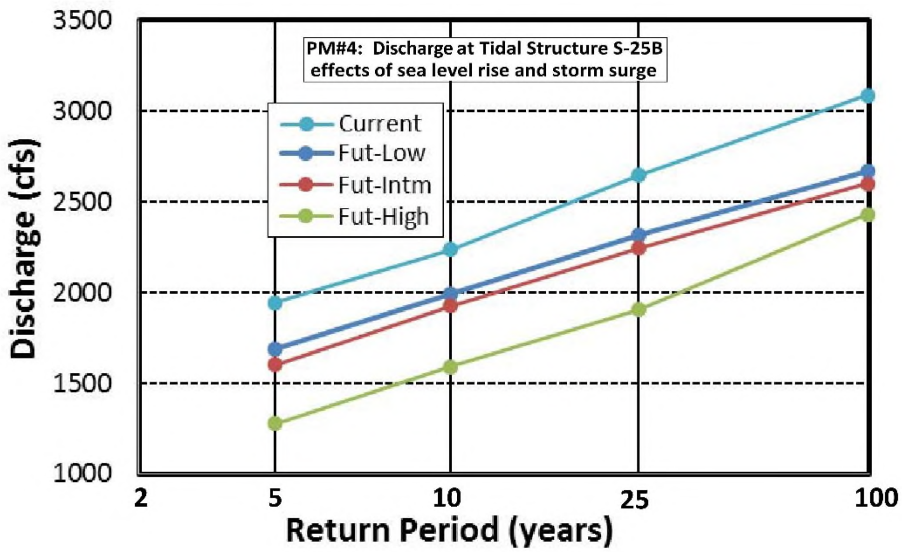

Figure A-12 Combined effects of sea level rise, storm surge and rainfall during design storm conditions on maximum flow at a coastal structure, averaged over the tidal cycle.

LOS Metric #5: Frequency of Flooding -- stage-based LOS for sub-watersheds:

Metric #5 is a table showing the duration of flooding in a developed sub-watershed, i.e. the amount of time when stages in each sub-watershed exceed locally-defined LOS targets. LOS targets include both a stage target and a frequency for which water levels exceed that stage. This PM is used to compare local expectations of flood protection with regional system performance.

LOS metrics 1 through 4 define the level of service in the primary canal system. However, higher stages in the primary canals can result in higher flood depths on the landscape. Stage-based metrics are needed to show these impacts. In urban areas, local governments often establish desired levels of flood protection for communities throughout the county. The fifth metric determines the overall ability of the water management infrastructure to maintain water levels within sub-watersheds needed to protect infrastructure features such as residential roads, major roads and house pads.

Ideally, the water levels needed within the sub-watersheds are defined by local interests that provide and operate the secondary and tertiary water management systems. These water management systems are sized and designed to control water levels and distribute excess water within their jurisdictions to protect features such as roads and buildings from flooding. Local interests define the water levels that can be managed within their systems under different design storm conditions, with the assumption that during such storms, a certain amount of water will discharge to the primary system.

In some cases, elevations needed to protect resource within a watershed are unknown or poorly known, often because development occurred before regulations and detailed records were in place to require or document survey data, or because subsequent development in a watershed has altered the natural drainage features. In such cases, methods have been developed to use existing data such as LIDAR (Light Detection And Ranging or LIght raDAR) surveys, to examine elevations of developed land and uplands within a sub-watershed as a means to estimate an elevation at which water levels may cause nuisance flooding or property damage. Such analyses provide a basis to compare results from different model runs, but are not useful for estimating damages or costs due to flooding. For this method, land areas that are under water within lakes or ponds, or that are obviously intended for use as stormwater retention areas, are excluded. Representative LIDAR elevations within the remainder of the subbasin are collected and statistically analyzed to determine the 20th percentile (elevation below which 20% of the points occur). This level is defined as the *threshold flood level* and is used as a surrogate threshold for water levels at which nuisance flooding is an issue and damages may be occurring.

Hydrologic modeling is used to verify that during a design storm the primary canal has the capacity to receive the designated amount of water from secondary systems and that, under these conditions, the water level in the sub-watershed will be at or below the level specified by the secondary water management system design. For example, Miami-Dade County, have established five flood protection goals for individual sub-watersheds.

- All structures (commercial, residential and public) should have pad elevations higher than the maximum flooding caused by the 100-year storm event.
- Principal arterials, including major evacuation routes, should be passable during the 100-year storm event.
- All canals should operate within their banks during their respective design floods.
- Minor arterials (4-lane roads) should be passable during the 10-year storm event.
- Collector and local residential streets should be passable during the 5-year storm event.

These qualitative flood protection goals can then translated into specific numeric water level values associated with each goal for each sub-watershed within the model domain.

Performance metric #5 requires basin-scale hydrologic and hydraulic modeling that simulates water levels and flows in the canal network and water levels throughout the watershed. This model is used to convert a design rainfall scenario into runoff from each sub-watershed within the basin. The ability to maintain water levels below these target stages under design storm conditions is then evaluated to identify the level of service that exists in the basin. **Figure A-13** shows a hypothetical "Watershed 50" with five sub-watersheds, A, B, C, D, and E. **Table A-2** provides an example of how such level of service criteria might be expressed for this watershed by local interests. To determine whether these metrics are achieved, water level data from the hydraulic model for each sub-watershed are compared to the LOS specifications as shown in **Table A-2**.

Figure A-13. A hypothetical watershed (50) with five sub-watersheds A-E.

Table A-2. Example of flood protection level of service target stages defined by local interests for each of the five sub-watershed in the hypothetical watershed 50.

Design Storm Return Period (years):	*100*	*25*	*10*	*5*
Subwatershed	Flood Protection Target Elevation (feet NGVD29)			
	House Pad Elev.	Secondary Canal Bank Elev.	Minor Arterial Road Crown Elev.	Residential Street Elev.
50A	4.1			2.94
50B	5.3	6.3		
50C	6.1	5.3	6.11	5.27
50D	6.1	6.4		3.99
50E	6.1	6		4.11

Data Analysis and Interpretation

For current conditions and each of the three sea level rise scenarios, water levels are determined by the model along the length of the canal and within each watershed over the duration of the storm event. An initial assessment of the performance of the primary drainage system can be made to determine whether water levels in the canal remained below the elevation of the canal banks. This is an indicator of whether the original design performance of the canal is being achieved.

A more important consideration, however, is the level of flood protection being achieved within the surrounding watershed. A representative graph of model output data for a single watershed for a 1-in-100 year storm is shown in **Figure A-14**. Data of this type, representing water levels versus time, are used to determine the peak water level that occurs within each watershed, the amount of time that water level was above the threshold elevation for flooding (based on data for streets and house elevations if available or surrogate data to determine a threshold flood elevation if detailed data are not provided) and the duration of significant flooding.

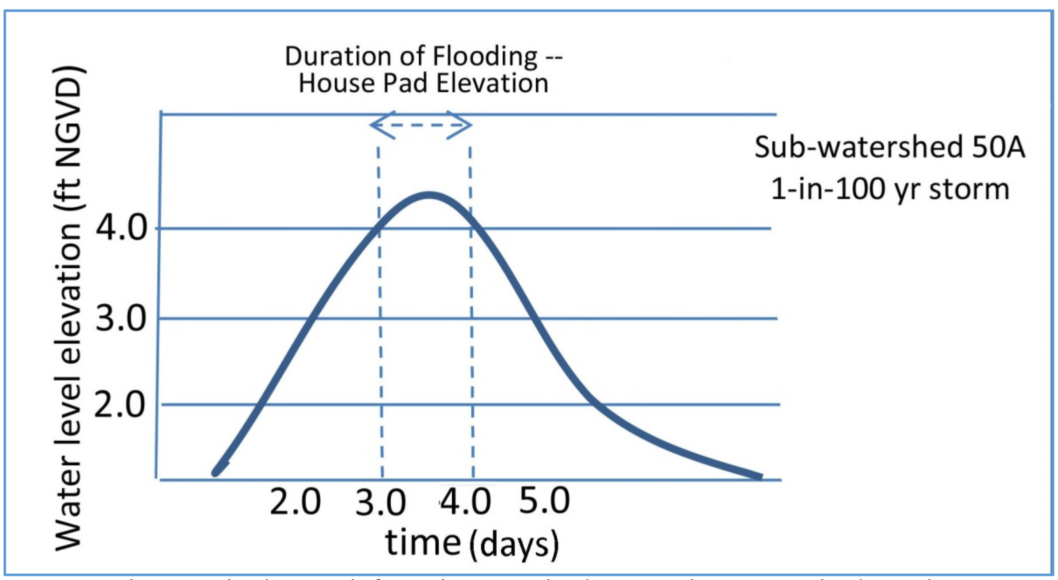

Figure A-14. Sample stage hydrograph for sub-watershed 50A within Watershed 50, showing water level criteria in the secondary water management system during a 1-in-100 Year Storm Event. House pad elevations for the watershed are defined in **Table A-1**.

The graph of water level versus time is then used to determine the peak stage that occurs and the amount of time that water levels remain above the flood protection target stages identified for each watershed. In the example above, the stage in sub-watershed 50A during a 1-in-25 year storm event reached a peak of about 4.4 feet and stayed above the target house pad elevation (4.1 feet) for 1 day.

For each watershed, the modeled stage hydrographs from the appropriate design storm are compared to the appropriate stage targets. The duration (hours) of flooding is then determined for each target water level and a table is generated that shows the flood duration (water level above the target elevation) in the sub-watershed for each run for each target elevation. This information can be summarized visually on a map, like **Figure A-15**, indicating areas where house pads within Watershed 50 would likely be flooded during a 1-in-100 year storm event.

Modeling Tools Required for Development of Performance Metrics

Two types of hydraulic models are used in the development of the performance metrics: structure-scale and watershed-scale. PM #5 is developed using a structure-scale hydraulic model with fixed upstream head and variable downstream head. The remaining performance metrics are developed using a watershed-scale hydraulic model with different design rainfall and variable downstream head conditions and post-processing the results of the simulations. A special post-processing tool has been developed for PM #5.

Figure A-15. Hypothetical example of a map showing sub-watersheds (blue shading) within a watershed where significant flooding of house pads would likely occur or water levels are likely to exceed threshold elevations (see text) by 1 to 1.1 feet during a 1-in-100-year storm event.

Interpreting Results

The model provides output in the form of water level profiles over time for each sub-basin, similar to that shown in **Figure A-14**. These data can be further analyzed to show maximum water level within the basin and the amount of time that the water level exceeds threshold levels. **Table A-3** shows how such model data can be compiled for a single sub-watershed.

Table A-3. Example of compiled model output Data for PM#5 for two sub-watersheds in the C-4 watershed for a 1-in-5-year design storm. : Maximum stages in sub-watersheds, increases in stage and duration of flooding are shown for SLR1, SLR2, and SLR3.

		current	SLR1		SLR2		SLR3	
BASIN	RP (yr)	max stg (ft NGVD)	depth incr. (ft)	duration (hr)	depth incr. (ft)	duration (hr)	depth incr. (ft)	duration (hr)
C4_25	5	5.96	0.1	301	0.12	348	0.23	550
C4_AG1	5	6.48	0.04	3	0.41	19	0.51	63

Flood Protection Level of Service Basic Concepts

Similar data from the entire sub-watershed can be further represented statistically as shown in **Figure A-16** to provide an overall summary of the relative effects of different sea level rise scenarios. Flood depth is relative to the land elevation that it is compared to. In this analysis, the flood threshold elevation is defined as the water level at which 20% of the uplands in the subwatershed are flooded.

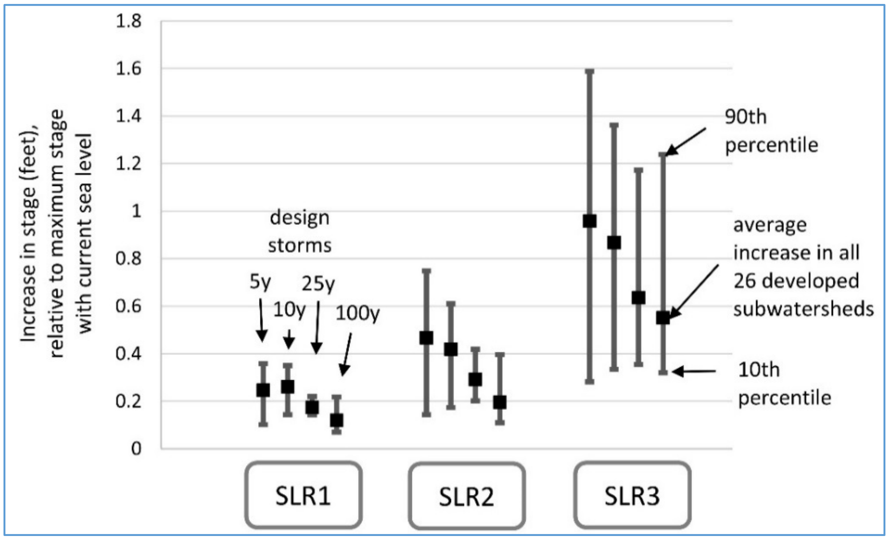

Figure A-16. Relative effects of different sea level rise scenarios on water levels within developed sub-basins of the C-4 Canal watershed resulting from 5, 10, 25 and 100-year design storms.

Figure A-17 is an example of another type of output from this analysis that shows areas within the watershed where flooding is likely to occur to various depths under different storm and sea level rise conditions.

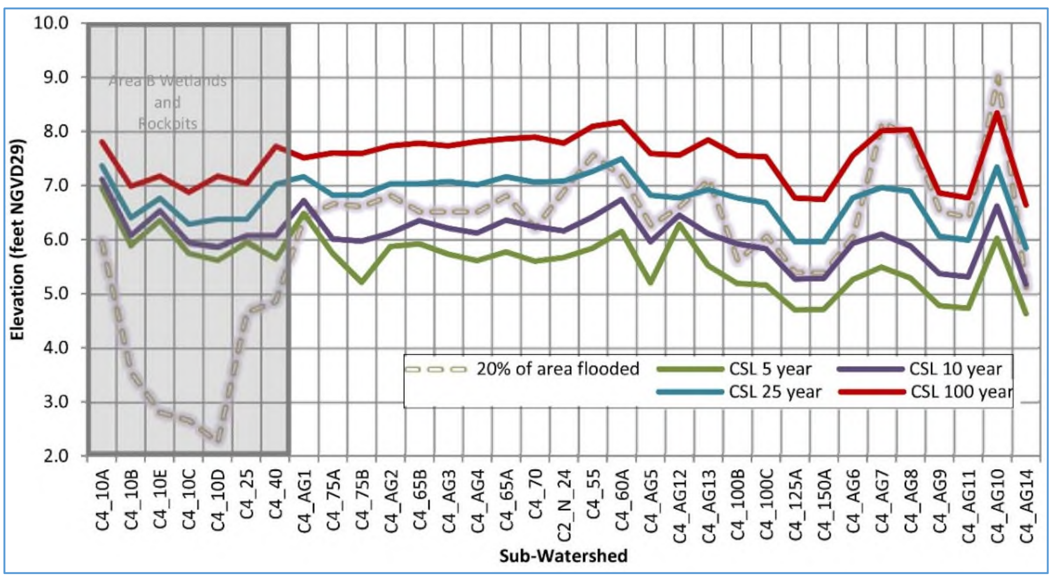

Figure A-17. Example of model output that has been post-processed to show the impact of sea level rise on sub-watersheds in the C-4 Watershed. Of the 33 sub-basins, 10 are undeveloped (shaded areas on the left-hand side of the graph) consisting of wetlands, rockpits or stormwater impoundment areas. Maximum water levels for design storms (solid color lines) are shown and can be compared to threshold flood level (see text) in the subbasins (dashed line)

Figure A-18 shows how flood impact data can be visually represented for a design storm within an entire watershed, over a range of current and future sea level rise conditions

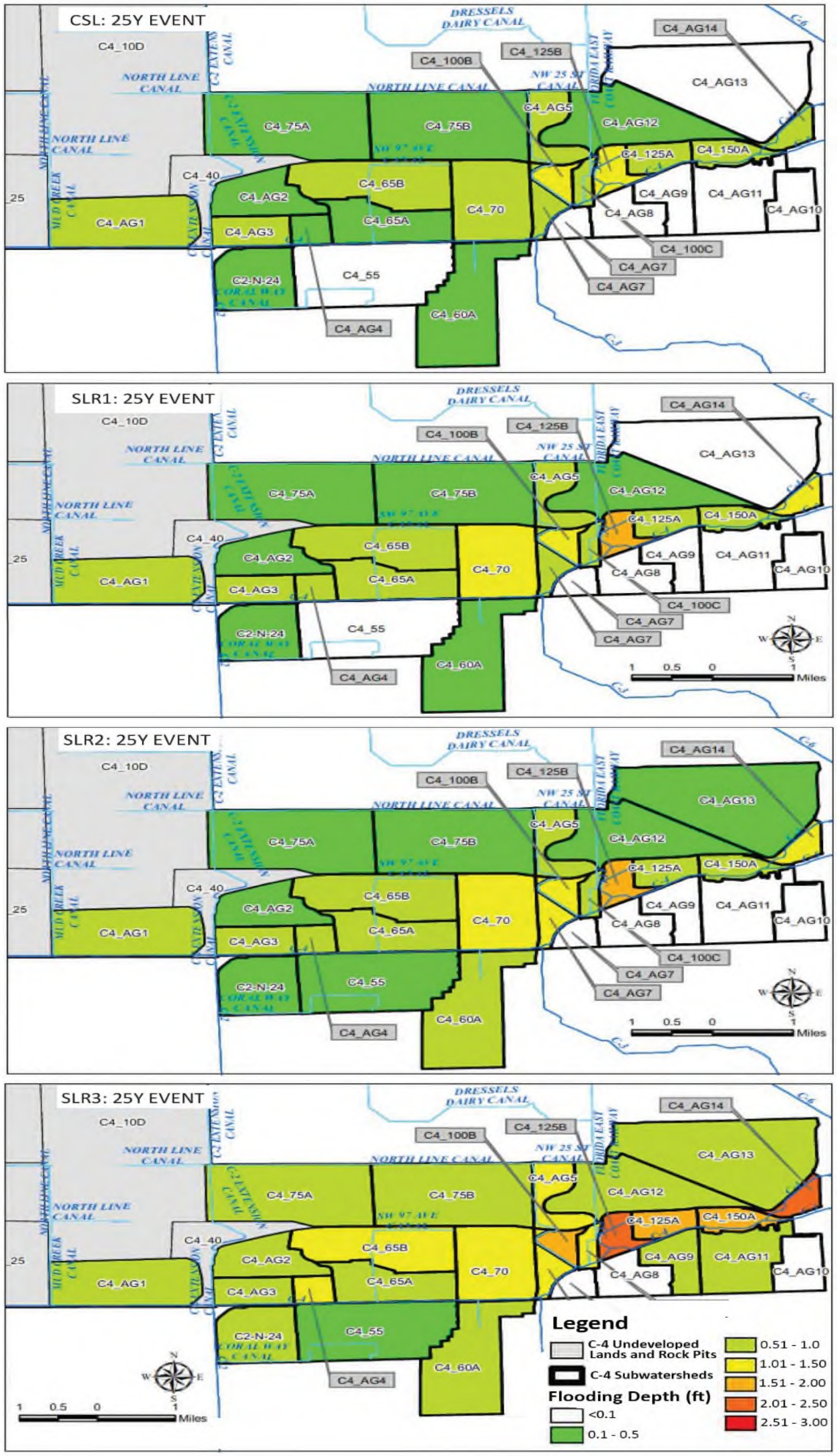

Figure A-18. Effects of a 1-in-25 year storm on flood water elevations (in feet) within sub-watersheds of the C-4 Basin for current sea level and three levels of predicted future sea level rise (SLR1 – SLR3).

LOS Metric #6: Duration of Flooding – effects of sea level rise:

Metric #6 is a figure showing the duration of flooding in the primary canal network for a range of design storms and a range of sea level rise scenarios. This PM is shows how recovery time varies with the design storm and how recovery time is affected by sea level rise.

Like metrics 1 through 4, LOS metric 6 relates to the level of service in the primary canal system. This metric compares the recovery time of a range of flood events. The metric looks water levels at a specific location in the canal system and tracks the time that water levels are above a target canal stage. The location and the target stage are unique to each watershed and are selected by District water managers. The target stage is substantially higher that the water control elevation for the system and indicates a flood state, i.e. a state where all control structures are being operated to maximize drainage.

Figure A-19 shows how duration of flooding is determined. It is the duration, in days, that the stage at the target location exceeds the target stage. For this example, the location is the T5 gage in the C4 canal and the target stage is 4.5 feet NGVD29. The duration of flooding is 12 days for the 100-y storm event under existing water level conditions. This represents one point on the PM#6 figure. The process is repeated for all design storm events for all sea level scenarios. PM#6 shows the flood duration for the 5-y, 10-y 25-y and 100-y design storms for each sea level rise scenario.

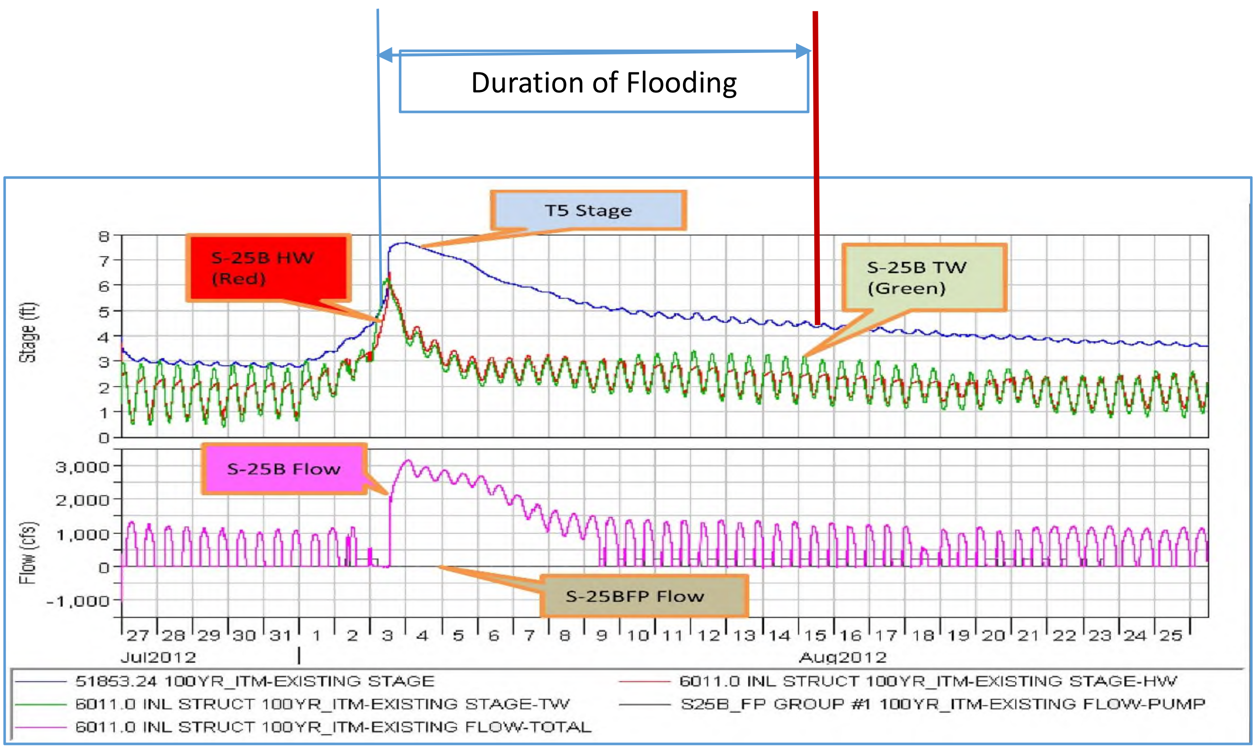

Figure A-19. How flood duration is determined.

Figure A-20 shows an example of PM#6. Four curves are shown. Each curve shows the response of the system to a range of design storms. One curve shows response to the existing sea level (CSL); the others show the response to a small sea level rise (SLR1), a moderate sea level rise (SLR2), and high sea level rise (SLR3).

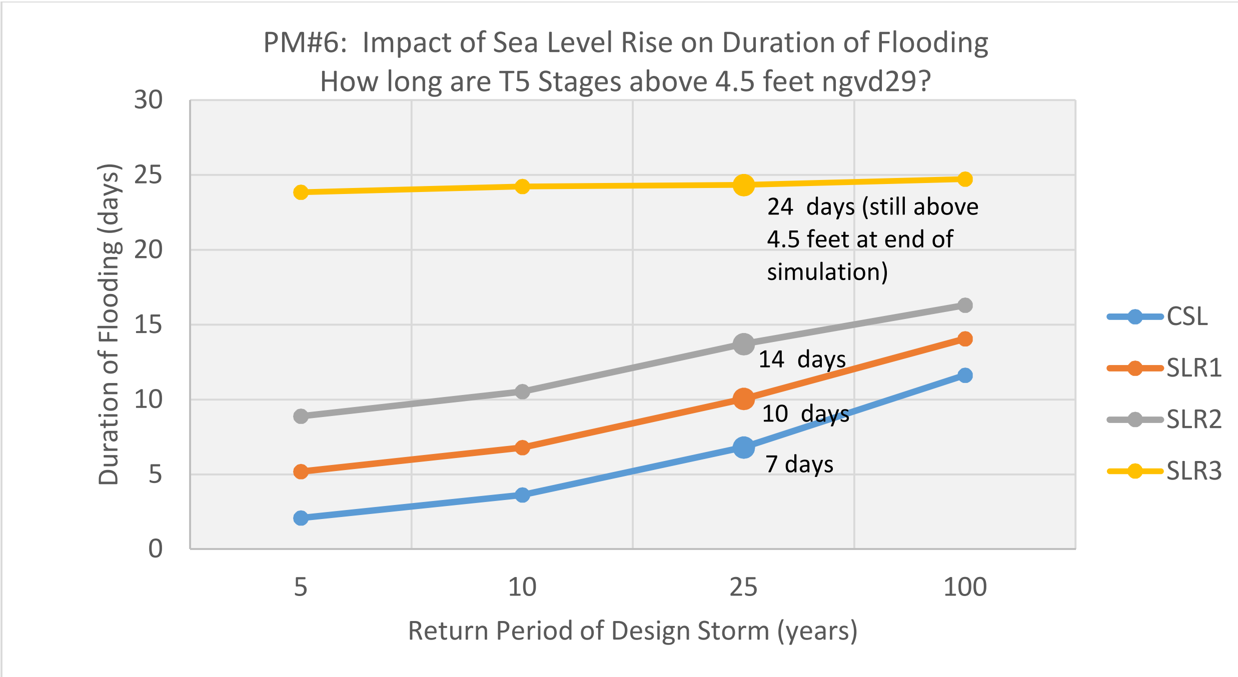

Figure A-20 PM#6 Impact of Sea Level Rise on Duration of Flooding (example)

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Flood Protection Level of Service Basic Concepts

ATTACHMENT 1: OVERVIEW OF SFWMD FLOOD PROTECTION LEVEL-OF-SERVICE (LOS) PROJECT.

Project Description.

The SFWMD has initiated a Flood Protection Level-of-Service project that assesses existing and future level-of-service to identify and prioritize long-term District infrastructure needs. The goal is to quantify the level of flood protection provided by the existing District infrastructure under today's conditions and to assess the level of flood protection provided by the current infrastructure under future conditions - considering the effects of sea-level rise, increased basin development, and completion of the Central Everglades Planning Project recommendations. System deficiencies identified in this project will be addressed through adaptation strategies.

To achieve this objective, a set of watershed-scale assessments will be conducted. Assessments include a review of the original system design, an assessment of District infrastructure and structure operating rules, sea level rise assessments and their impact on structure conveyance, and flood protection assessments for current and future conditions.

Project Components

Goals

- Establish existing and future LOS for all watersheds in the District
- Identify issues affecting LOS
- Identify problematic District structures and features

Project Elements

- LOS Fundamentals
- Assessment Procedure
- Update design storm rainfall
- Sea Level Rise
- Basin-Scale Assessment Projects and LOS modeling

Project Tasks

- Develop assessment template for Flood Protection Level-of-Service projects
- Describe the existing system and identify basin issues related to level of service and potential operational responses to sea level rise
- Develop performance metrics describing the level of flood protection provided by the primary and secondary water management system
- Assess changes in rainfall-frequency relationships to establish design storm criteria
- Assess the existing level of flood protection throughout the watershed
- Assess potential effects of sea-level rise
- Conduct Local-Scale Modeling to evaluate impacts and management options

Local Scale Modeling Efforts.

Studies will be conducted for watersheds throughout the entire District, beginning with the C-4 Miami Canal watersheds. For each watershed or group of watersheds, one or models are used to represent the hydraulics of the watershed. The model domain includes both the regional (primary) canals plus secondary canals within the watershed. In areas where groundwater transmissivity is very high, both surface water and groundwater flow to the canal must be· simulated.

Subwatersheds within the main watershed may be represented as 'basins.' A basin stage-storage relationship is then developed to include both surface and groundwater storage. Basin-to-basin and basin to canal flows are included in the model. Interflow between the basin being analyzed and adjoining watersheds may be assumed to be negligible, may be represented by predetermined boundary flows, or may be derived from other models.

For design events, the model simulates the operations of all structures according to current District operational plans for pumps, structures, storage features, etc. For tidally-influenced structures, the model incorporates a tidally adjusted flow equation, if practicable, using downstream stage time-series as a boundary condition.

Water levels within each basin are required as initial conditions, as well as water levels for surrounding basins that are used as boundary conditions. Based on these stages a predetermined baseflow is calculated in the canal.

Watershed Studies

The analysis for each watershed, or a group of watersheds, is typically conducted as a series of six efforts that occur simultaneously:

1. Model Development (may not always be necessary)

Develop model to represent watershed and infrastructure components and features, including canal network and basin connectivity, basin (subwatersheds) stage-storage relationships, canal cross-sections and structure characteristics, and historic water levels and flows.

2. Calibration (may not always be necessary)

Develop time-series of watershed inflows and outflows from control structures, boundary conditions, and equations for tidally corrected flows (if needed), as well as time-series for rainfall and basin inflows. Define calibration criteria and time-series for calibration targets and perform the model calibration

3. Application Modeling

Incorporate rule-driven operations into the model to validate the operational rules. Establish initial conditions and boundary conditions for design events. Develop design rainfall events and data time series. Five design storms (5-yr, 10-yr. 25-yr, 50-yr and 100-yr) will be analyzed under four conditions (current, future with SLR1, future with SLR2, and future with SLR3). The future case is based on population and land use projections for the 50-year design life of the tidal structures, i.e. 2065.

4. Level of Service Analysis

These stages and flows are assessed to determine Level of Service. Model data (Q and H) from each of the five design storm events are used to determine the corresponding level of service provided by the system for current condition and each of three future sea level conditions

5. Documentation

Develop documentation for each of the four prior efforts including models used, how the models were modified, calibrated and applied within the watershed, including data sources and assumptions, and how initial conditions and boundary conditions were determined, used, etc. and details of the level of service analysis and assumptions used to derive future conditions.

6. Guidance

Describe problems or issues encountered, needs for additional data or analyses and suggestions for how future studies should be conducted and improved.

ATTACHMENT 2: PROCEDURE TO GENERATE PERFORMANCE METRIC #3 – STRUCTURE PERFORMANCE

As sea-levels rise, the increasing tail-water elevations result in less flow at the tidal structures. PM #3 has been developed to show the impact of sea level rise and storm surge at a structure. The metric is developed from simulations of structure flow where the upstream stage is fixed at the design headwater stage and downstream stage oscillates with the tide. Three days into the simulation, a storm surge of known intensity (as indicated by its return period) raises tail-water stage and suppresses flow. Results of each simulation are further analyzed to determine the minimum 12-hour flow through the structure. For this analysis, five sea-level scenarios were modeled and, for each scenario, a range of six design storm surges were examined. The minimum flow values from these 30 model runs were then compared on a single plot.

A simple hydraulic model is used, consisting of the structure, a short length of canal upstream and a short length of canal downstream of the structure. The upstream boundary condition is a fixed head; the downstream boundary condition uses the same suite of time-varying tidal boundary conditions as those used for other performance measures; and all structure operations are modeled fully, as described in the Basin Atlas (SFWMD, 2015). This metric is independent of rainfall.

Tidal boundary conditions are shown on **Figure A-2-1** and **Figure A-2-2. Figure A-2-1** shows an example of the suite of tidal boundaries (2-yr to 100-yr return period) associated with storm surge for 1963 sea level conditions. **Figure A-2-2** shows tidal boundary water levels for five sea level rise scenarios, all with a 5-yr storm surge. The sea level rise scenarios are for the year of the design report (1963), (2015 (current conditions), and three possible sea level rise scenarios, with increases of +0.34 ft, (SLR1) +0.80 ft (SLR2) and +2.26 ft (SLR3) (0.10, 0.24, and 0.69 m, respectively). Such increase may be expected to occur between now and the year 2065.

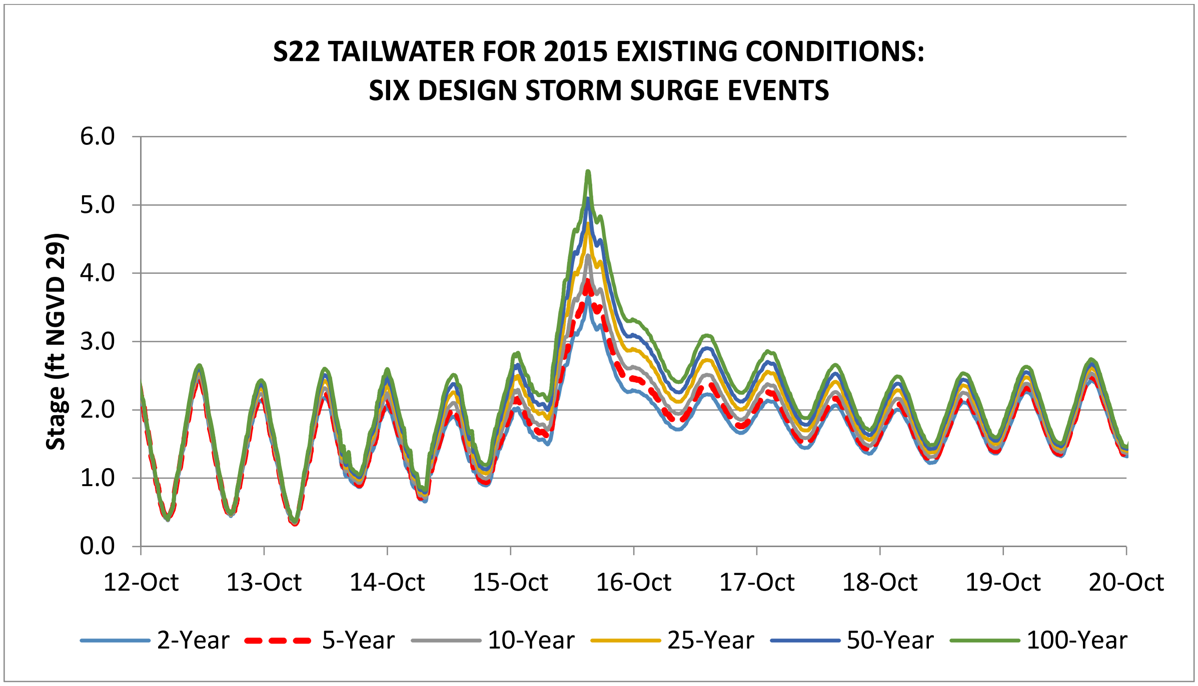

Figure A-2-1. tail-water for 2015 existing conditions: six design storm surge events

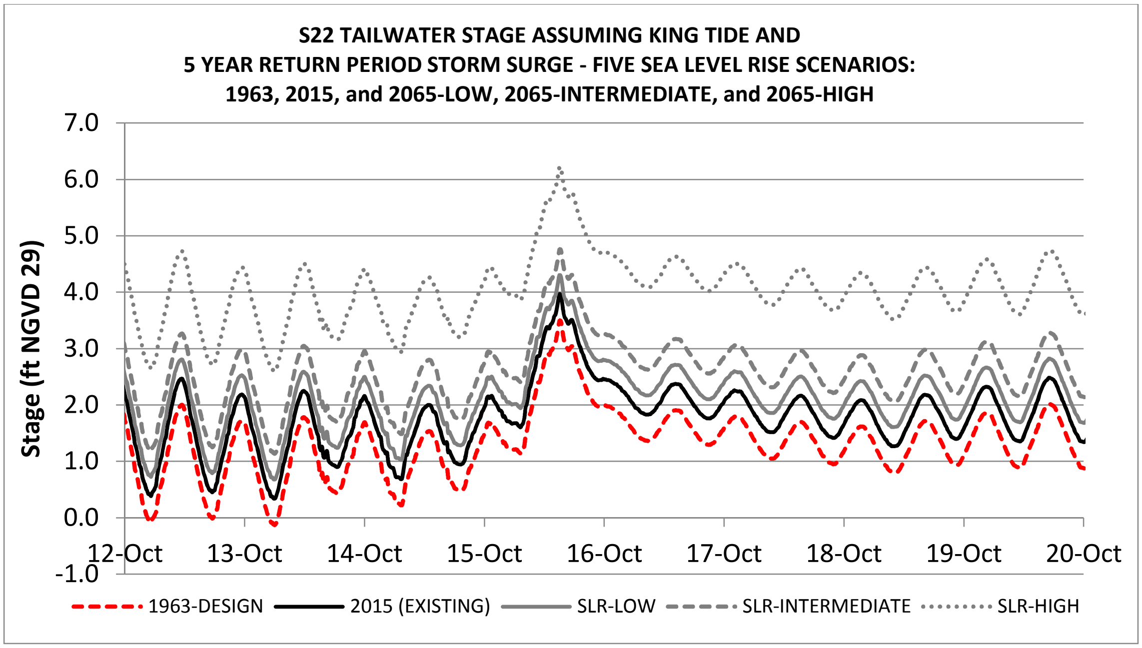

Figure A-2-2. S22 tail-water stage for five sea level rise scenarios (5-year storm surge for all scenarios).Scenario 2065-Low=SLR1; 2065-Intermediate=SLR2 and 2065-High = SLR3

Values for stage and flow during the storm surge event are shown on **Figure A-2-3**, as instantaneous values, and on **Figure A-2-4**, as 12-hour moving average values. The example shows headwater stage, tailwater stage and flow through the structure for the 1963 sea level with 5-year storm surge event.

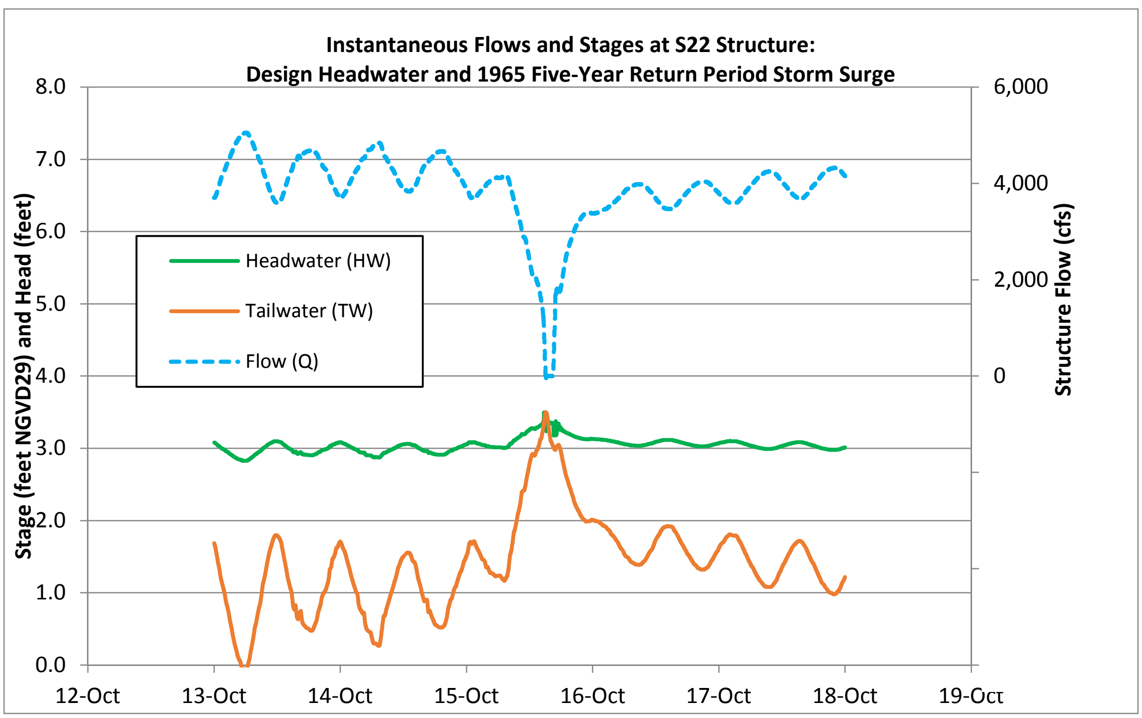

Figure A-2-3. Instantaneous Flows and Stages at S22 Structure: Design Headwater and 1965 Five-Year Return Period Storm Surge

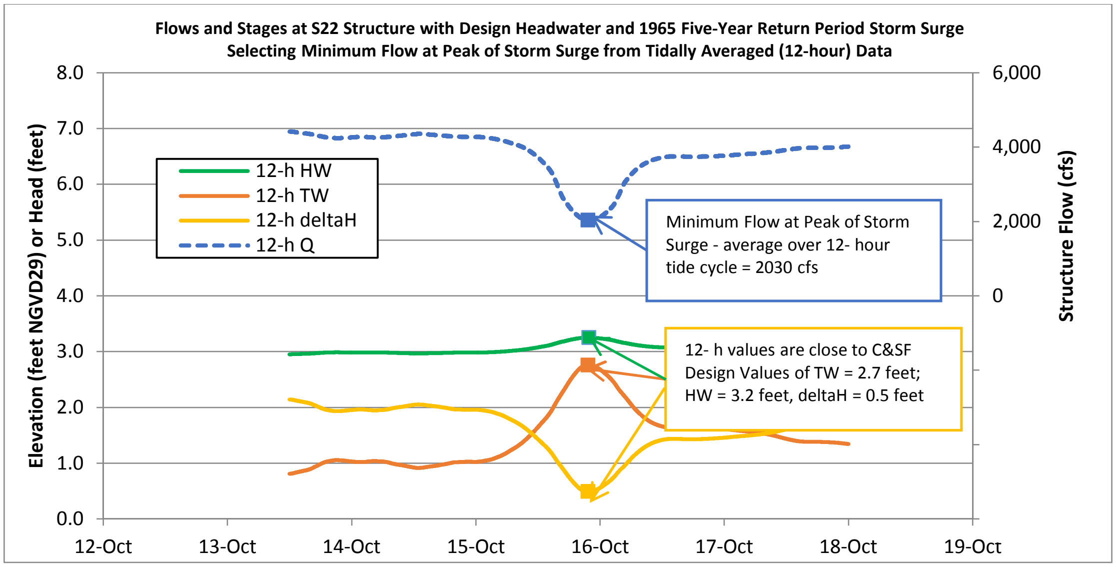

Figure A-2-4. Selecting Minimum Flow at Peak of Storm Surge using Tidally Averaged (12-hour) Data

This simulation has 12-hour average stage values that match the steady-state design conditions for headwater (3.2 feet) and tail-water (2.7 feet). The minimum flow occurs at the peak of the storm surge and has a simulated value of 2030 cfs. This value is comparable to steady-state flow value of 1930 cfs from the original C&SF design.

Figure A-2-5 is Performance Metric #3. It displays the minimum flow values for each of the thirty simulations (five sea level scenarios each with six design storm surge events). The value equivalent to the 1963 design value is highlighted using an enlarged blue square. The suite of structure flows associated with 1965 storm surge boundary conditions are shown as a blue squares; flows associated

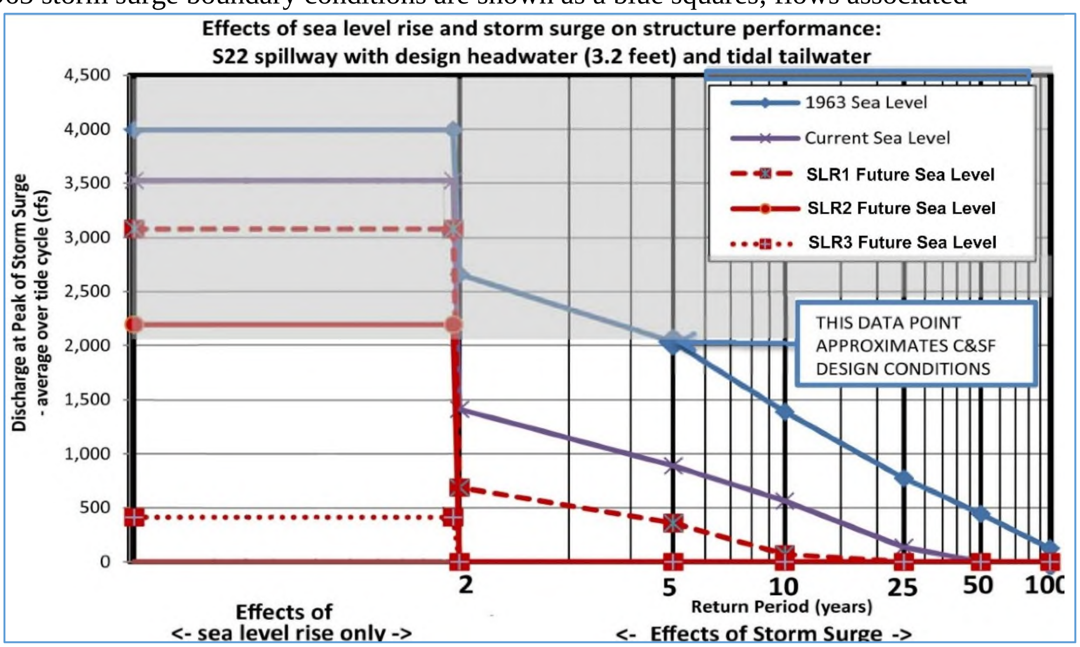

Figure A-2-5. Structure Performance - Effects of Sea Level Rise and Storm Surge

with current 2015 conditions are shown as a green triangles; flows associated with SLR1, SLR2 and SLR3 conditions are shown as red circles. Each line shows the short-term suppression of flows caused by storm surge while the King Tide No-Surge values shows the prolonged suppression of flows caused by sea level rise. This metric will vary with each structure.

Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed

Appendix B: Data Collection

South Florida Water Management District
Hydrology and Hydraulics Bureau

December 30, 2015

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Appendix B – Data Collection

This document was produced by the H&H Bureau as a project deliverable for project 100888 (Flood Protection Level of Service, within the Sea Level Rise project).

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Additional project documents can be found on a District server at
\\ad.sfwmd.gov\dfsroot\data\hesm_nas\projects\basin_studies\7_tidal_struc_modeling

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Appendix B – Data Collection

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Appendix B – Data Collection

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INTRODUCTION

This appendix describes the data used in the C-4 basin Level of Service (LOS) modeling, including model calibration and application to storm event modeling under current and future sea level rise scenarios. The purpose of this appendix is to provide a description of the data sets, the sources of the data and a location where the data can be found. The appendix is divided into static data which includes geographical data that describes the characteristics of the study area including topography, soil storage, canal cross-sectional data, etc. Time-series are used to characterize the hydrologic/hydraulic response of the C-4 watershed during flood events. These data includes rainfall, canal flow and water level, groundwater stages, etc. District data source will be referenced as DBHYDRO throughout this appendix. External data sources are referenced with a full description of the source.

The C-4 Basin encompasses an area of about 83.3 square miles of highly urbanized areas on eastern portion of the basin including the Miami International Airport, The Florida International University main campus, the cities of Belen and Sweetwater and portions of the cities of Doral, Miami and West Miami. The portion of the basin west of the Florida Turnpike is mostly undeveloped and includes large rock mining areas and the Pennsuco Wetlands just east of the L-31 Canal (**Figure B-1**). The drainage capacity of the primary canal in the C-4 Basin is highly influenced by flood control operations in the C-2 and C-3 Basins and the stormwater detention system south of the rock mining area. **Figure B-1** also shows the location of the C-4 and adjacent basins in Central Miami-Dade County. The C-2 Basin is located to the South of the C-4 Canal and its primary drainage feature is the C-2 Canal which intersects the C-4 Canal just west of the Florida Turnpike. Flows and water levels in the C-2 Canal are controlled by gate operations at the S-22 Structure located at the tidal end of the C-2 Canal just east of SW 57th Avenue. Also, south of the C-4 Canal, the C-3 Basin is drained by the C-3 Canal which affects the flows in the C-4 Canal. The C-3 Canal intersects the C-4 Canal just east of the Palmetto Expressway and is the only primary drainage feature for the City of Coral Gables. The G-93 structure located near the intersection of SW 56th Avenue and SW 35th Street is operated by the District to control flows and canal stages in the C-3 Canal. Because of their significant influence on flows and stages in the C-4 Basin, the C-2 and the C-3 Basins were included, in a simplified manner, as part of the model for the C-4 Basin.

STATIC DATA

Topography

Topographic data was needed to estimate the total water storage in the C-4 sub-basins. The topographic data set for this purpose was the 2007-08 Miami-Dade 5-ft DEM in NAVD 1988, Release Version 1, originally provided by the County. The current version of the dataset has been processed by the District into a 5-ft digital elevation model (DEM) of bare earth for eastern and southern portions of Miami-Dade County, as well as relatively small portions of Monroe County that are adjacent to Miami-Dade's southern and southwestern border. Elevation values are in feet, NAVD 1988. The vertical accuracy of the source data used to produce the DEM (bare earth mass points and break lines) has a reported vertical accuracy of 0.6 ft @ 95% confidence level. **Figure B-2** shows the portion of the DEM created from the Lidar data over the C-4 Watershed. The DEM file is called *miamiriv_5ft* and can be found at *\\ad.sfwmd.gov\dfsroot\data\hesm_nas\projects\basin_studies\2_basin_atlas\pilot_gis\spatial\fgdb\MiamiRiver_DEM.gdb*.

Appendix B – Data Collection

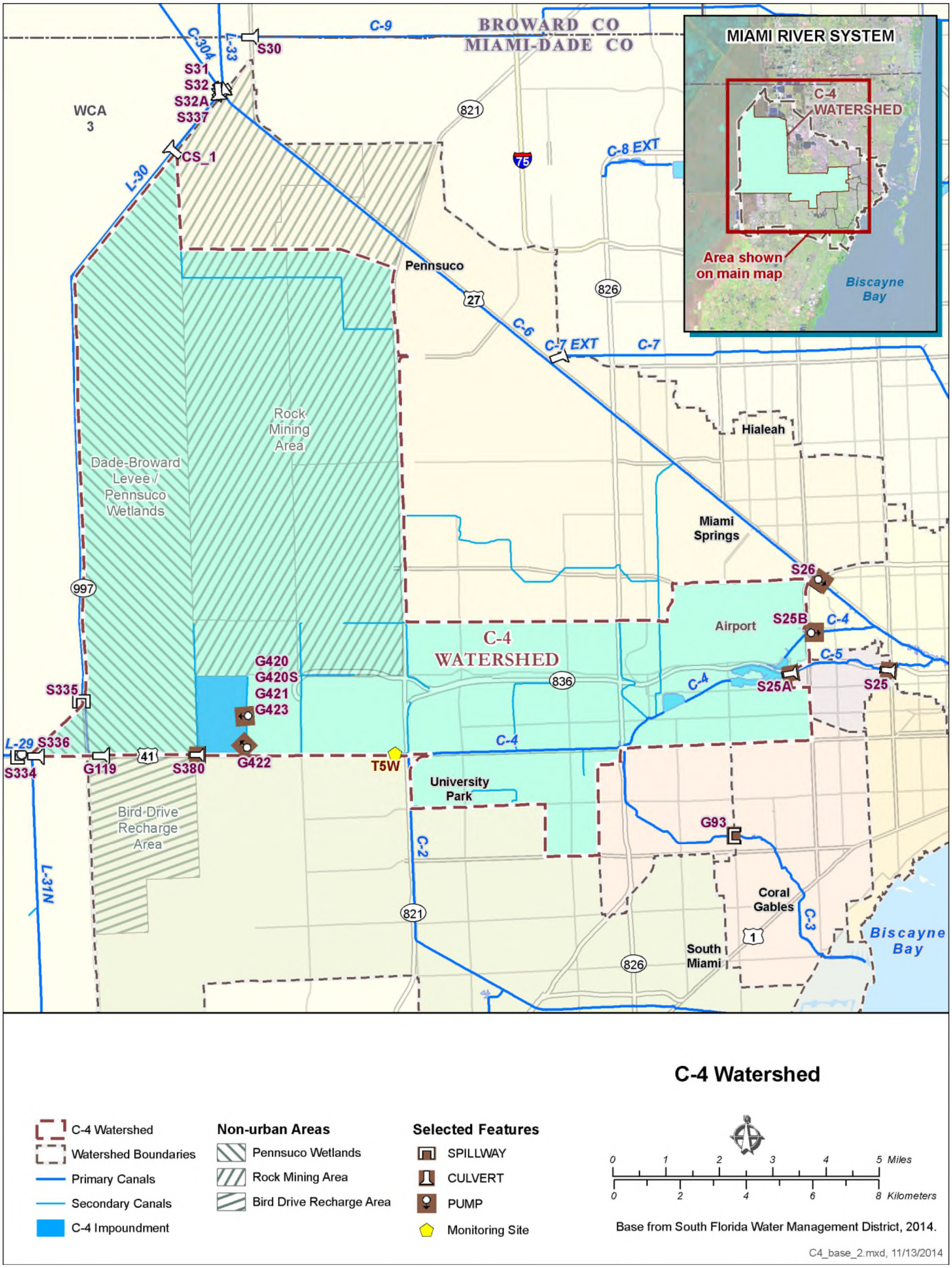

Figure B-1. Location of the C-4 Basin in Miami-Dade County, Florida

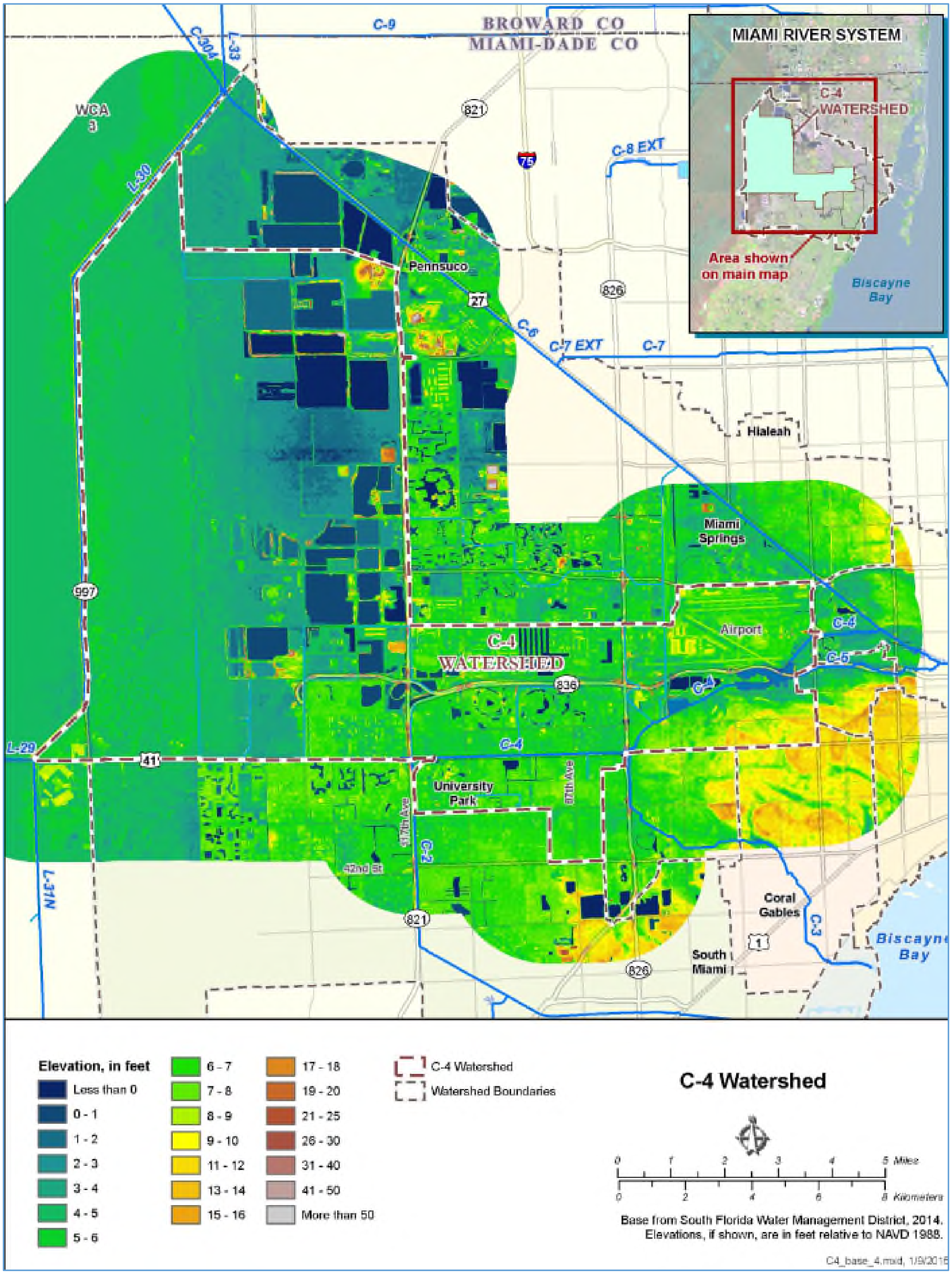

Figure B-2. Digital Elevation Map of the C-4 Watershed

Sub-basins Delineation

Discretization of the C-4 model area into sub-basins for this study follows the sub-basins delineation method used by Miami-Dade County in the development of the XPSWMM model (Miami-Dade County, 2004). Hydrologic lumped parameter approaches typically required characterization of hydrologic parameters at the sub-basin level. These lumped parameters are used to approximate hydrologic processes such as infiltration, runoff, etc. In this study, a lumped parameter approach is followed to represent groundwater exchange between sub-basins and canals. The lumped parameters for these processes represent hydraulic conductance used to compute the exchange of water between water bodies.

There are a total of 39 sub-basins including the C-2, C-3 and C-5 basins. The total area of the C-4 Basin is 83.3 square miles (53,321 acres) and the largest sub-basin is the Pennsuco Wetland (Sub-basin C4_10A) in the western edge of the basin with an area of 21.2 square miles (13,582 acres). The sub-basin delineation for this study is shown in **Figure B-3**. The model domain area also includes the C-2, C-3 and C-5 basins which are modelled, for simplicity, as entire basins represented by a single node. The shape-file of the subbasins is called *MD_Basins_May19_04.shp* and is located at *\\ad.sfwmd.gov\DFSroot\data\hesm_nas\projects\basin_studies\GIS\MD-Basins*.

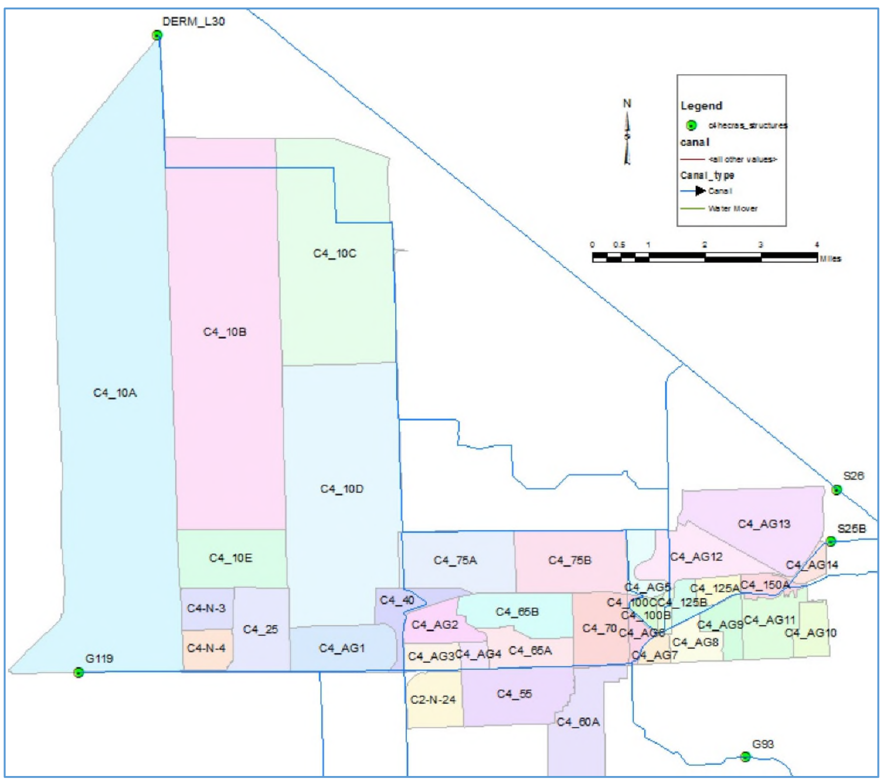

Figure B-3. Sub-basin Delineation of the C-4 Basin

Soil Storage

The C-4 version of the HEC-RAS model uses combined surface water and groundwater storage to simulate the flow exchange between sub-basins and from sub-basins to canals. Total storage in sub-basins is approximated by combining surface or above ground storage with soil storage of the surficial aquifer underneath the sub-basin area. Surface storage was estimated using a digital elevation map (DEM) of the study area derived from high resolution Lidar ground elevation data. Sub-basin total storage was conducted with a spreadsheet which requires two input data sets:

1) Soil water drained versus water table depth relationship and
2) Surface water area and volume versus Elevation table

A soil water drained versus water depth relationship was generated from the soil water characteristic curve for Krome soil type which is typical in the area of interest. The soil water characteristic curve was obtained from the USGS (2004) and represents a soil with a porosities with a range of values between 0.08 - 0.12. The FORTRAN program WTDVOLDRN (Skaggs, 1981) was used to convert the soil water characteristic curve into a soil water drained versus depth relationship by assuming hydrostatic equilibrium within the water column. This relationship was applied to all the sub-basins in the C-4 watershed. **Figure B-4** shows the computed elevation versus surface storage, elevation versus groundwater storage and elevation versus total storage for sub-basin C4_AG3. An Excel file of the stage-area-storage tables for each sub-basins is called, *total_storage_v_depth_results_V3.0.xlsx* and can be found at *\\ad.sfwmd.gov\DFSroot\data\hesm_nas\projects\basin_studies\5b_data_collection\Stage-Storage*.

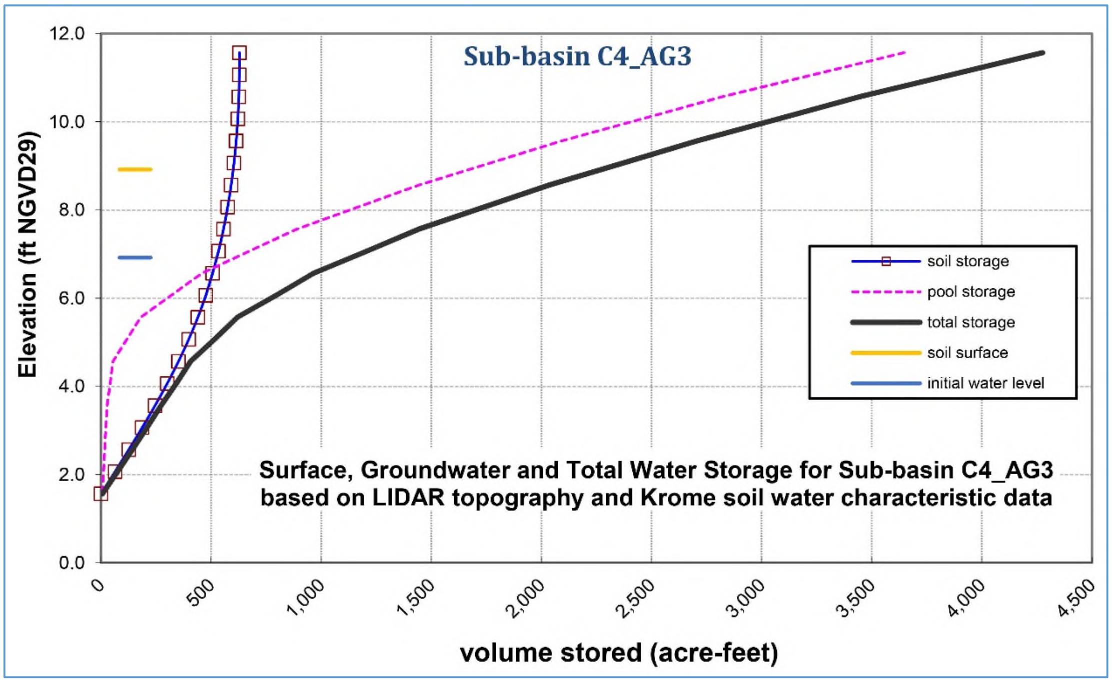

Figure B-4. Surface, groundwater and total Storage for Sub-basin C4_AG3

Canal Cross-Section Data

The existing canal data to define the geometry of the C-2, C-3 and C-4 canals in HEC-RAS were obtained from several sources which include as-built drawings and canal surveys by Miami-Dade County (DERM), US Army Corps of Engineers, SFWMD and the Florida Department of Transportation (FDOT). Most of these information was already incorporated into a XPSWMM model originally developed for the District (PBS&J, 2005). Recent improvements to the C-4 Canal banks by the District (Jackson, 2011) were also incorporated into the canal cross-sectional data for the HEC-RAS model as well as the latest operations of the C-4 EMD and S-25B Forward Pumps. Currently the model includes 1,131 canal cross-sections. **Figure B-5** shows the location of the canal cross-sections used in the HEC-RAS model many of which were interpolated from the surveyed canal cross-sections as needed by the model to reduce model instability during flow computations.

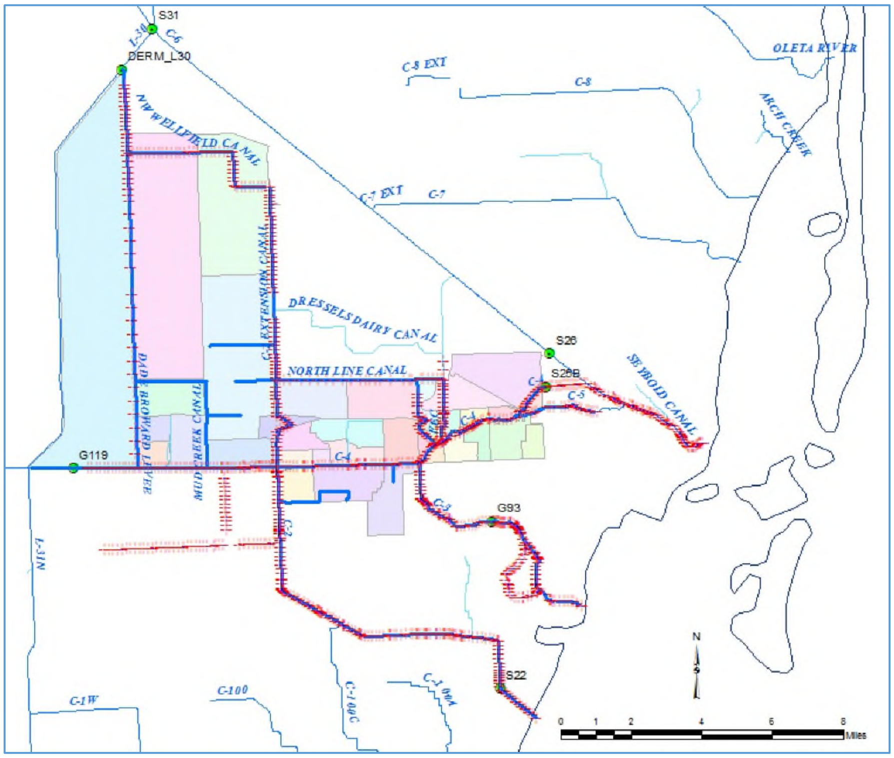

Figure B-5. Location of Canal Cross-Sections

Bridges

Bridges across primary and secondary canals are common features in urban areas such as the C-4 Basin. Bridges can have a significant effect in flows and stages in a canal, particularly when the presence of the bridge structural elements that are in contact with the flow of water, produces frictional loss of energy, that translates in higher stages on the upstream side of the structures. For significant flood events, bridge head losses can result in bridge overtopping and flow velocities that can result in canal bed erosion that may compromise their stability. Most bridges along the C-4 canal have been designed to minimize head losses, however, the cumulative bridge head loss effect for the entire C-4 canal needs to be accounted for in flood analyses. The current condition version of the model includes twenty-one (21) bridges in the C-2, canal, seven (7) in the C-3 Canal and twenty-two (22) in the C-4 canal. Most of the bridge geometric information in this version of the model was obtained from Miami-Dade County and the Florida Department of Transportation. **Figure B-6** shows the location of bridges in the current condition HEC-RAS model, while **Figure B-7** (left-hand side) shows an example of a bridge located in the C-4 Canal at 132nd Avenue. Most bridges were implemented using the cross-section geometric information about piers and bridge deck obtained from the surveys. The Yarnell method of computing head losses across bridges is used for the majority of the bridges in the model. This method requires specification of a drag coefficient and a pier shape factor (**Figure B-7** right-hand side). In cases where pressure flow occurs through the bridge opening due to submergence of the bridge chord, the energy method was chosen as the default.

Structure Operations

The latest canal structure operations for the C-4 are described in **Appendix A** (C4 Basin Water Control Operations Atlas: Miami River System Final Draft, August 31, 2015). These operations include gate operations at spillways S25B, S-22 and G-93, culverts G-119 and G-93, pumps G420, G-422 and S-25B Forward Pump and all Municipal Pumps discharging into the C-3 and C-4 Canals.

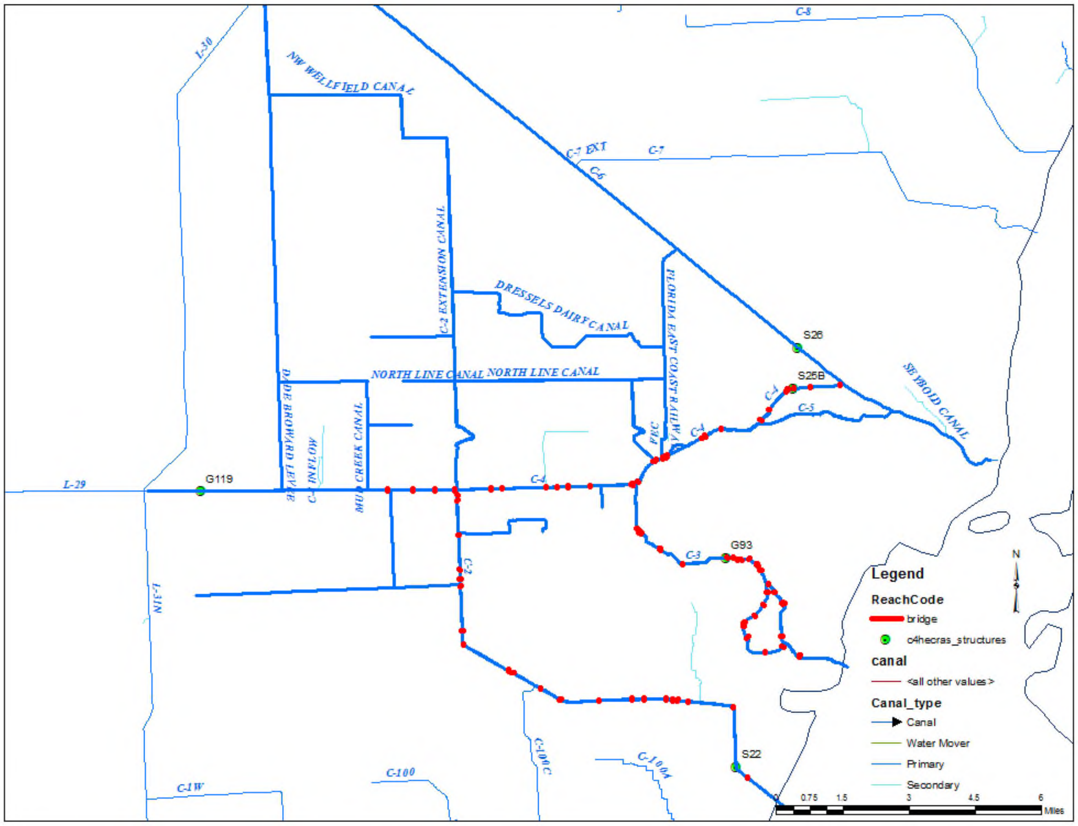

Figure B-6. Location of Bridges in the C-2, C-3 and C-4 Canals

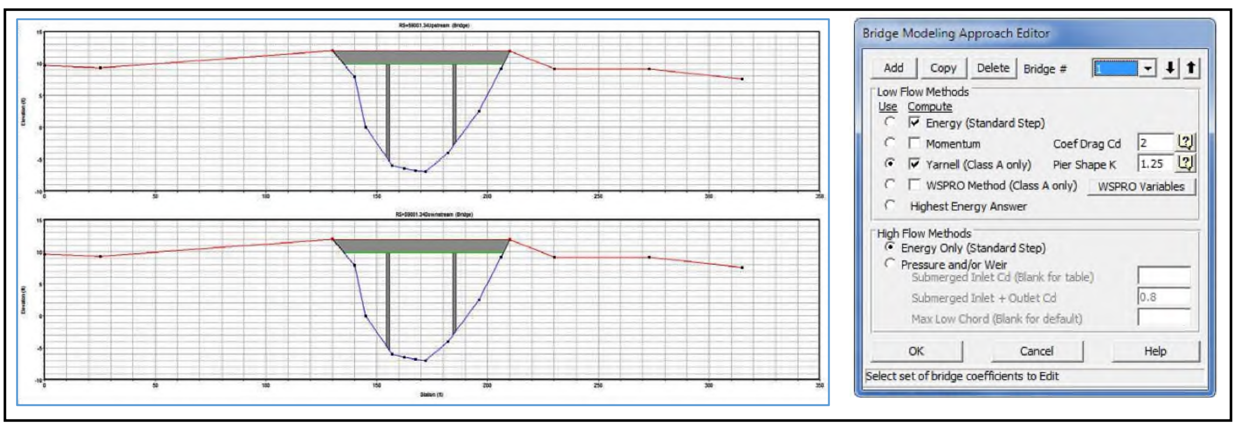

Figure B-7 Cross-section of bridge location in the C-4 Canal at 132nd Avenue (left). The Yarnell method of computing head losses across bridges requires specification of a drag coefficient and a pier shape factor in the model (right)

For model calibration while specifying operations, the HEC-RAS model does not use rule-driven operations at the structures, instead, it uses observed 15-min values of gate openings at the operable gates to compute flows at the structures and, at pump stations, with specified pumpage. The use of these specified data during model calibration, reduces uncertainty in the calibration parameters by eliminating errors in the model response due to uncertainty of the operations. The description of the gate operation data sets is presented in the subsequent section of this appendix.

TIME SERIES DATA FOR MODEL CALIBRATION

Rainfall

High-resolution rainfall data was used in the C-4 HEC-RAS model, to generate the sub-basin inflows into the sub-basin storage, in the model. The year of 2012 was screened for rainfall data amounts for model calibration. The National Weather Service (NWS) office located in the Florida International University (FIU) just south of Sweetwater in West Miami recorded 99.42 inches of rain for the year, or about 76% above the normal rainfall for the year value of 56 inches. Of the 99.42 inches of rain measured at FIU for the year, 81.04 inches were recorded between May 1st and October 15, 2012. Many of the storm events that accounted for the majority of the rainfall in Sweetwater, were localized events of high intensity and low duration and resulted in street flooding. For example, the cities of Doral and Sweetwater received 8-10 inches of rain in a period of 6 hours on May 22. This event represent a storm event close to a 10-yr return period event. At the SFWMD Field Station, north of the Miami International Airport, a total of 5.03 inches were recorded on August 4, 2012, and on September 11, the City of Sweetwater recorded 3.8 inches of rain in a 2 hour period.

The selected calibration period was September 1st to October 31, 2012. For this period, NEXRAD rainfall data were available as gridded data with a minimum temporal resolution of 15 minutes. The spatial grid resolution of the grid cells is 2 by 2 kilometers as shown in **Figure B-8** over the study area. **Figure B-9** is an example of a 15-minute rainfall hyetograph from the Nexrad database for sub-basin C4_N-4 for the year of 2012. A HEC-DSS file of the 15-minute calibration rainfall is called *NetRF.dss* at: *\\ad.sfwmd.gov\DFSroot\data\hesm_nas\projects\basin_studies\5_modeling\Calibration/hecras\C4_LOS_HEC-RAS_Recalibration_Oct_23_2015*.

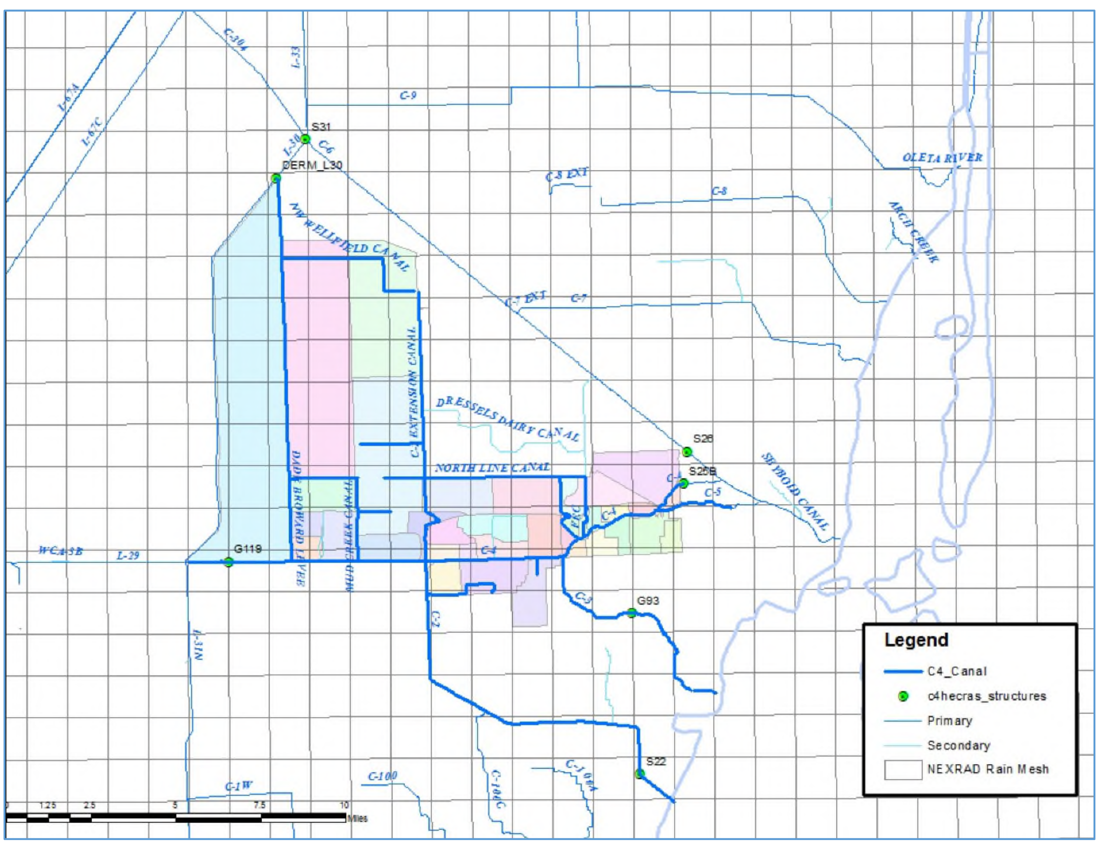

Figure B-8. Nexrad Rainfall Data Grid

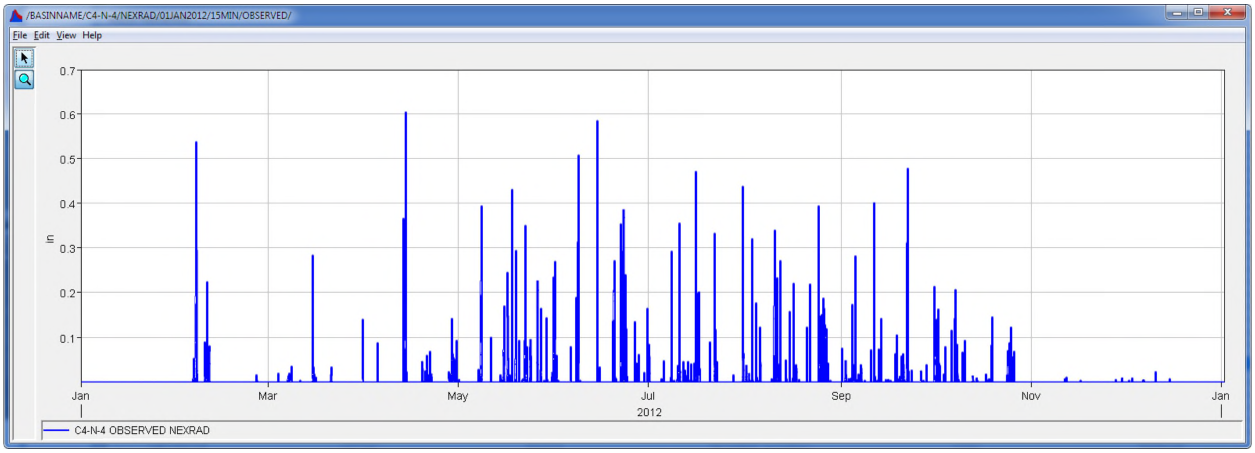

Figure B-9. Rainfall Hyetograph for Sub-basin C4-N4 for Calibration Period

Evaporation

Evaporative losses are included in the model for the calibration period of August –September of 2012. Evaporation data observed at three sites near the C-4 Basin were assessed for data availability. The three evaporation gauge sites used were 3AS3WX in the Water Conservation Area 3A, S331W at the site of the structure S331 on the C-111 Canal, and JBTS in the Everglades National Park. The location of these three sites relative to the C-4 Basin is shown in **Figure B-10**. The selected representative site was S331W due to its superior data quality and proximity to the study area.

The observed evaporation data consists of daily readings of pan evaporation corrected with a factor of 0.7. The adjusted daily values of evaporation were subtracted from the rainfall volumes, resulting in net inflows for each of the sub-basins in the HEC-RAS model. Daily evaporative losses were combined with rainfall to produce total inflows for each sub-basin in the file *NetRF.dss* described above.

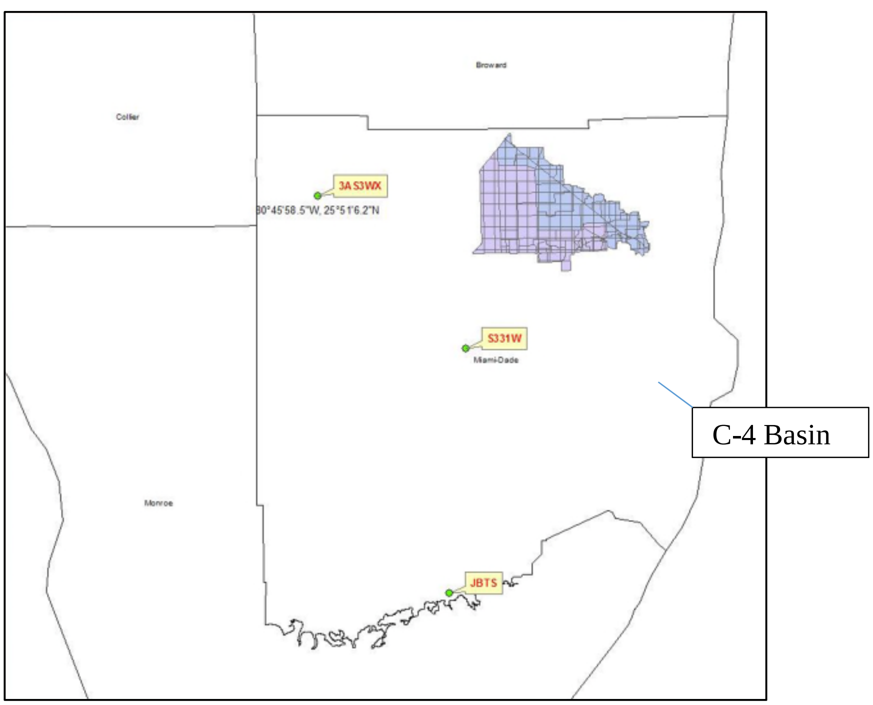

Figure B-10. Location of Evaporation Data Sites near the C-4 Basin

Canal Flow, Stage and Gate Operations

Water level data and flow discharge data are needed for the task of model calibration of the C-4 Basin HEC-RAS model. There are fourteen (14) gauge locations in the C-2, C-3 and C-4 canals were 15-min stage and/or flow data are available. The location of these gauges is shown in **Figure B-11** while **Table B-1** summarizes the data type and availability for the calibration period for each gauge.

Transient water levels and flow recordings at the water control structures and other gauges were screened for continuity of records during severe wet periods. Due to the nature of the transient flow characteristics in the C-4 canal, 15-min data was necessary to characterize the basin's response during severe storm events. The largest recent storm event with adequate data occurred in October of 1999 (Hurricane Irene) and in October 2 - 5 of 2000 (No-name storm) which caused widespread flooding in South Florida. As a result of flood damages from these events, local, state and federal agencies developed a number of structural and operational measures in the C-4 Basin for flood mitigation which include the C-4 Impoundment and pumps, Structure S-380, and the S-25B Forward Pumps which were completed in 2004. Despite being the most severe storm event in recent history, flood control operations during Hurricane Irene (with a return frequency of about 25 years) is not representative of the current infrastructure and flood management operations in the C-4 Basin, and therefore is not adequate for calibration of current conditions in the C-4 Basin. Several more recent wet periods with significant events were screened for the selection of the calibration period, and include Hurricane Katrina and Tropical Storm Rita (September 18-26, 2005), Tropical Storm Fay (August 15 – 20, 2008), Tropical Storm Nichole (September 28 - 30, 2010) and the unusually wet season of 2012 (May to September). Most of the severe storm events that occurred after 2004 reflect flood mitigation operations that required the operation of the C-4 Impoundment and the S-25B Forward Pumps.

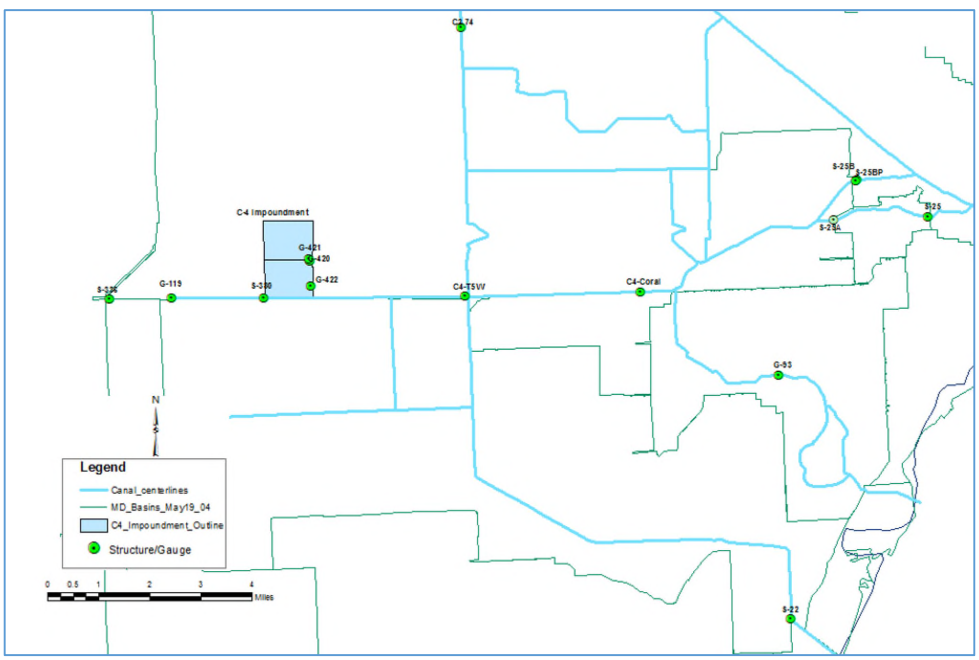

Figure B-11. Canal Stage/Flow Monitoring Sites for Model Calibration

The selected calibration period for this study was August 1st to September 30st, 2012. This period includes several storm events including Tropical Storm Isaac (August 26 – September 3, 2012) as mentioned above. Fifteen-minute and hourly canal water elevation and flow data in this period was retrieved from the District's Dbhydro database for nine (9) water control structure and canal gauges. **Table B-1** summarizes the type of data retrieved from Dbhydro from each of the gauges.

Table B-1. C-2, C-3, C4 and C-5 Canal Stage, Flow and Gate Opening Gauges

Gauge	Canal	Data Type		Source/Description
S-336	C-4	STAGE	15-min TW	SFWMD / Not used for calibration
G-119	C-4		15-min TW	SFWMD
G-93	C-3		15-min HW and TW	SFWMD
G-420	C-4 Impoundment		15-min HW and TW	SFWMD / Not used for calibration
G-421	C-4 Impoundment		15-min HW and TW	SFWMD / Not used for calibration
G-422	C-4 Impoundment		15-min HW and TW	SFWMD / Not used for calibration
S-25B	C-4		15-min HW and TW	SFWMD
S-25	C-5		15-min HW and TW	SFWMD / Not used for calibration
S-25A	C-5		15-min HW and TW	SFWMD / Not used for calibration
S-22	C-2		15-min HW and TW	SFWMD
T5W	C-4		15-min canal stage	SFWMD
C4.Coral	C-4		Hourly canal stage	USGS
C2.74	C-2 Extension		Hourly canal stage	USGS
S-380	C4		15-min canal stage	SFWMD
G-93	C-3	FLOW	15-min	SFWMD
G-420	C-4 Impoundment		15-min pumpage	SFWMD
G-421	C-4 Impoundment		15-min	SFWMD / Not used for calibration
G-422	C-4 Impoundment		15-min pumpage	SFWMD
S-25B	C-4		15-min	SFWMD
C4.Coral	C-4		Hourly	USGS
S-25	C-5		15-min	SFWMD / Not used for calibration
S-25A	C--5		15-min	SFWMD / Not used for calibration
S-22	C-2		15-min	SFWMD
S-380	C-4		15-min	SFWMD
G-119	C-4	GATE OPENINGS	15-min	SFWMD / Not used for calibration
G-93	C-3		15-min	SFWMD
S-25B	C-4		15-min	SFWMD
S-25	C-5		15-min	SFWMD / Not used for calibration
S-25A	C-5		15-min	SFWMD / Not used for calibration
S-22	C-2		15-min	SFWMD
S-380	C-4		15-min	SFWMD

Note: HW = Headwater elevation in ft NGVD; TW = Tailwater elevation in ft NGVD

Given the availability of stage, flows and gate opening data at the structures, the model calibration task was carried out by imposing gate openings at the structures and allowing HEC-RAS to compute the headwater and tailwater stages as well as the discharge through the structure. As an example, **Figure B-12** shows the observed, headwater and tailwater stages, gate openings and flows at structure S-25B for the calibration period. A HEC-DSS file of the stage, flow and gate-opening data used for calibration is called *stage_15min.dss* and can be found at *\\ad.sfwmd.gov\DFSroot\data\hesm_nas\projects\basin_studies \basin_studies\5_modeling\Calibration\hecras\C4_LOS_HEC-RAS_Recalibration_Oct_23_2015*.

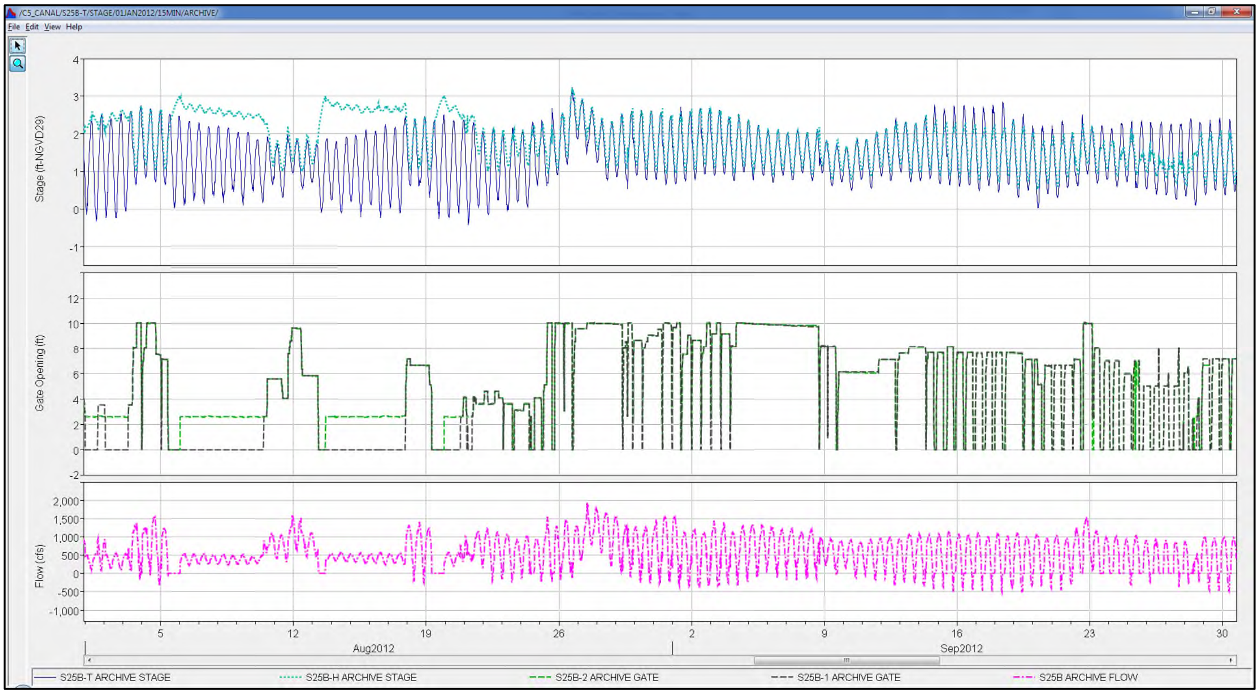

Figure B-12. 15-min Recorded Headwater and Tailwater Stages, Gate Opening and Flows at Structure S-25B for Calibration Period (Aug 1, Sep 30, 2012)

Groundwater Stage

Groundwater hourly and daily stage data from twelve (12) groundwater monitor wells in the C-4 Basin were used for calibration purposes. **Figure B-13** shows the location of the observation wells with respect to the C-4 sub-basins. As this figure shows, there are multiple gauges in single basins such as C4_10A and C2. There were a total of eight (8) gauges with data for the calibration period of which five were used. For the case of C4_10A sub-basins, the gauge Krome was selected due to the proximity of the other two gauges to the NW Wellfield, and gauge G3439 in the C2 Basin was determined to be better correlated to the model's output. **Table B-2** summarizes the data available for model calibration. A HEC-DSS file of the groundwater stage data used for model calibration is called *gw.dss* and can be found at *\\ad.sfwmd.gov\DFSroot\data\hesm_nas\projects\basin_studies|basin_studies\5_modeling\Calibration\hecras\C4_LOS_HEC-RAS_Recalibration_Oct_23_2015\…*

Table B-2. Summary of Groundwater Stage Date for Model Calibration

GW Gauge	Sub-basin	Data Type		Source/Description
G1488	C4_10A	STAGE	Daily	USGS/Not used for model calibration
G3329	C4_AG12		Daily	USGS
G3439	C2		Daily	USGS
G3572	C2		Daily	USGS/Not used for model calibration
G3676	C4_10E		Daily	USGS
G975	C4_10A		Daily	USGS/Not used for model calibration
TARMAC	C4_10B		Hourly	DERM
Krome	C4_10A		Hourly	DERM

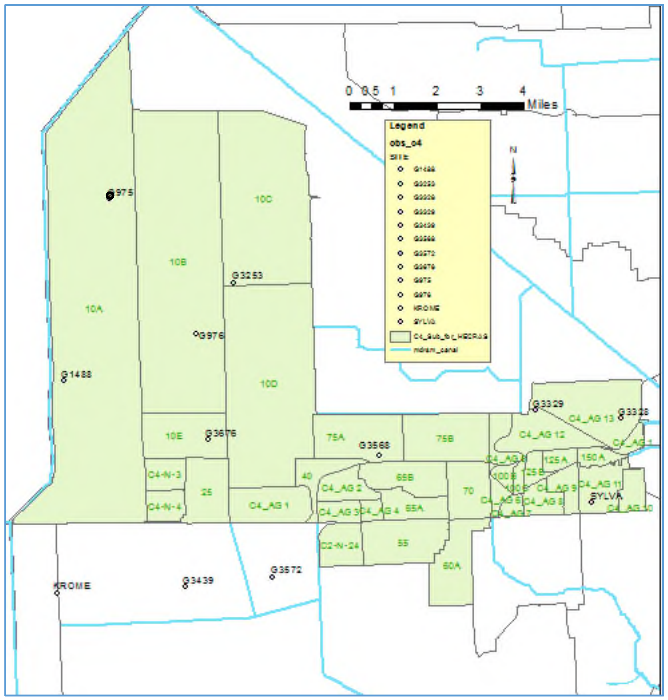

Figure B-13. C-4 Sub-basins and Groundwater Observation Wells

Water Supply Pumpage

There were no actual pumpage records available for the public water supply wellfields in the C4 basin during the calibration period. What is available is the permitted water supply withdrawals for the major well fields in the basin. Pumpage withdrawals for water supply in the C-4 Basin was assumed to be negligible during the calibration period when the seasonal water levels in the western portion of the basin are at their highest. The only wellfield in the C-4 Basin is the Northwest Wellfield located between the C-2 Extension Canal and the Dade-Broward Levee Canal This wellfield has average monthly withdrawals of 82 cfs and 68 cfs per day in the months of August and September, 2012 (**Figure B-14**). Location of the well is shown in **Figure B-15**. There were no daily or hourly wellfield withdrawal rate data during the calibration period and, since these withdrawal rates are considered small compared to the runoff contributions from the sub-basins, it was decided to not include the water supply pumpage during the calibration period.

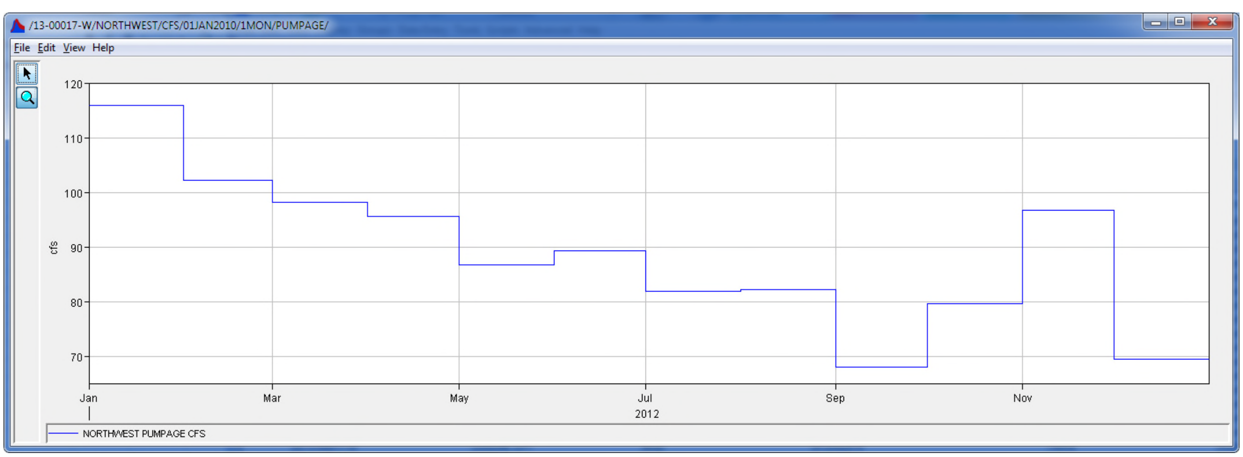

Figure B-14. Average Monthly Water Supply Pumpage for the Northwest Wellfield (2012)

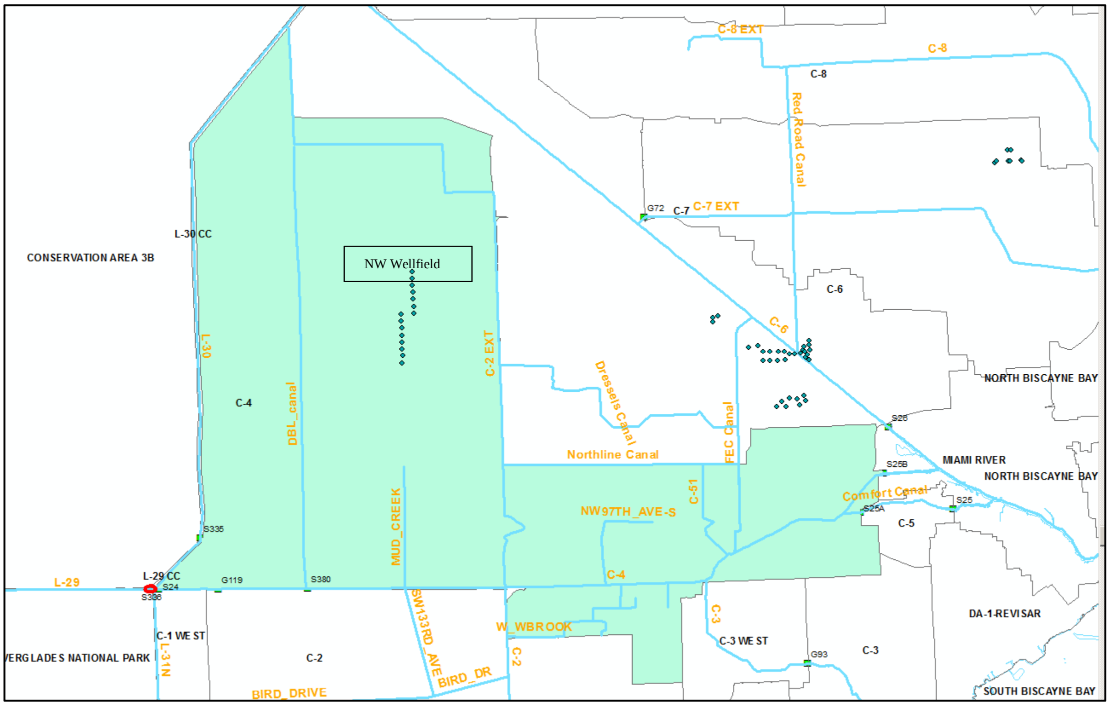

Figure B-15. Location of the Northwest Wellfield in the C-4 Basin

Seepage Boundary

Seepage estimates from the western boundary of the model, consisted of estimated seepage computed using observed water levels, in the Water Conservation Area 3B and the Everglades National Park, and seepage parameters that were obtained as part of the calibration process. The time series of daily water levels were inputted in the model to computed seepage inflows to the Pennsuco Wetland (sub-basins C4_10A) and the C2 sub-basin. **Figure B-16** shows the observed time series of water levels in the western boundary of the model. Additional details on the calibration of the seepage coefficients for boundary flows can be found in the “Seepage Boundary Flow Parameters subsection” of the “Model Calibration” section **in Appendix F** (SFWMD, 2015)

Initial conditions

Data needed to specify initial condition for model simulations consist primarily of water levels and flows in or out of the system. Typically, water levels can be estimated from observations, however, initial flows have to be estimated. In a complex system such as the C-4 basin, initial total flow can be assumed to equal the observed base flow at the S-25B structure, the main outlet for the basin, during dry periods, and then apportioned according to the size of each sub-basin. Initial water levels for the sub-basins and canals were approximated by interpolation from sparse observed values at a few observation recorders in the basin. Because of the lack of data in the water level data set to define the initial water levels in the sub-basins, the initial water levels were treated as calibration parameters in the calibration process. The upper and lower bounds for the parameters were determined using the observed data.

Initial flows were assumed to equal zero, and the model was allowed to generate its own values by running the model with sufficient time prior to the window of interest to allow for stabilization of water levels. Starting the model calibration on August 1st, allows for quick model stabilization since there is no rainfall events until August 3rd, 2012.

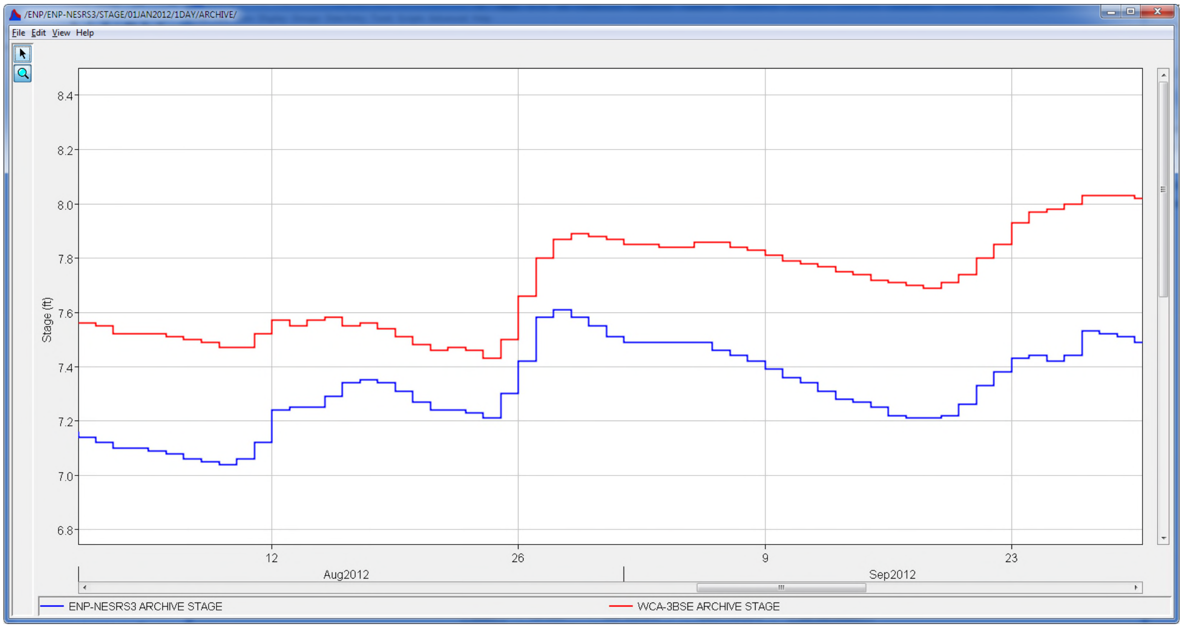

Figure B-16. Observed Water Levels in the Water Conservation Area 3B and Everglades National Park

TIME SERIES DATA FOR DESIGN STORM EVENT RUNS

The main difference between the calibration and application versions of the HEC-RAS model is in the operations of control structures and the choice of rainfall data. In calibration mode, the HEC-RAS model uses observed gate openings and observed pumpages to compute flow through control gated structures and pump stations. In application mode, the HEC-RAS model uses rule-driven operations to determine the necessary gate openings and pumpage needed to control water levels according to the flood control plan in the basin. Generally, gated structures have pre-established control triggers or water level targets to open, hold steady or close the structures' gates or turn pumps on or off. These pre-established flood control operations were programmed into HEC-RAS as user defined rules or as standard HEC-RAS gate operations with simple On and Off triggers to open or close the structures. Details on the operations of each structure in the C-4 Basin can be found in **Appendix A** (Basin Atlas) and in **Appendix E (Model Development and Quality Assurance)**.

Rainfall and Evapotranspiration

The evaluation flooding Levels of Service in the C-4 Basin requires the application of the model for several design storm events, including the 5-, 10-, 25- and 100-yr and duration of 72 hours. Rainfall distributions for these synthetic storm events have been previously developed by the District and is documented in SFWMD, 2000 (SFWMD, 2000).. **Figure B-17** displays as an example the 25-yr 72-hr rainfall distribution. Typically, lower return period events such as the 5- and 10-yr events with duration of 24 hours are used for designing crown of road elevations within the District, however, for this study more conservative events for flood control with duration of 72 hours are used for all return frequencies of interest. For further information on the application of the rainfall events to the evaluation of C-4 Basin LOS for flooding can be found in **Appendix G** of this report.

The effects of evaporation were neglected, in the application of all synthetic storm events which typically are in the order of a few days, including the post storm period. Most storm simulations described in **Appendix G** have durations of less than a month.

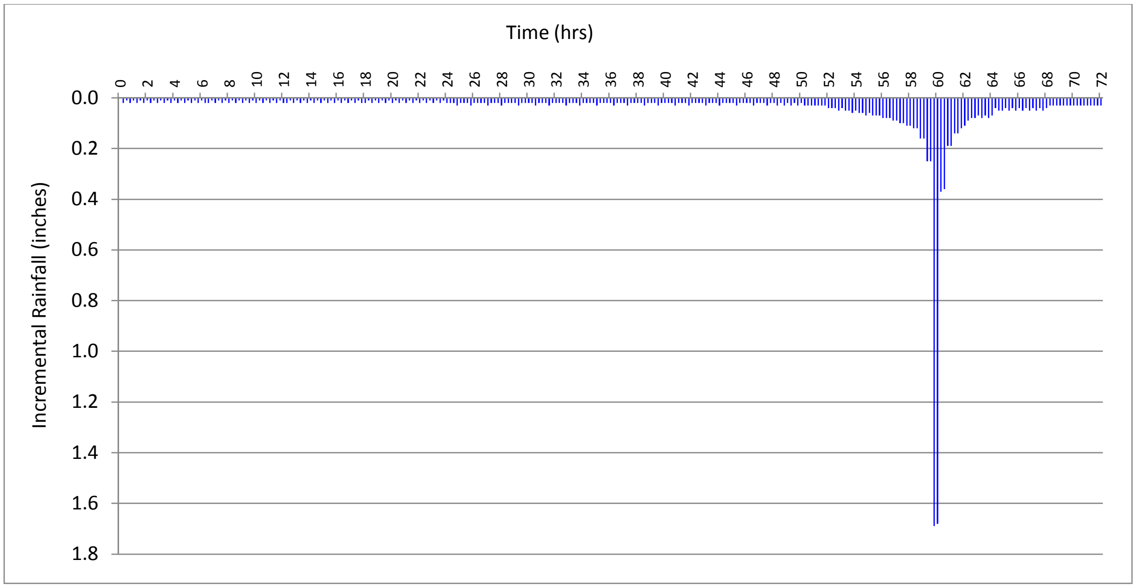

Figure B-17. Rainfall Distribution for the 25-year 72-hr Storm Event

Gated Structure and Pump Operations

The application of the HEC-RAS model for design event flood conditions in the C-4 Basin, requires the model to replicate the flood control operations at the control structures, as described in the C-4 Basin Water Control Plan described in **Appendix A** (Basin Atlas). The application of these rules for flood control using the HEC-RAS model was described in **Appendices E and G.**

Tidal Boundary Condition

Tidal stages applied at the tidal structures of the C-2, C3 and C4 canals in the HEC-RAS model are needed for each of the storm events of interest. A total of 16 events are considered in the evaluation of the flooding LOS in the C-4 Basin, with return periods of 5, 10, 25 and 100 years, and four levels of sea level rise: current and Sea Level Rise 1, 2, and 3 scenarios, as described in **Appendix C** which documents the development of the statistically derived tidal sea water elevations for all the design storm events of interest.

Seepage Boundary Condition

At the western boundary of the model domain, the HEC-RAS model requires specified flows from the Water Conservation Area 3B (WCA3B) and the Everglades National Park (ENP) as boundary conditions for all the simulated storm events. The boundary seepage flows are estimated in HEC-RAS using a stage hydrograph applied at the WCA3B and ENP boundaries of the model, adjacent to the C-4 and C-2 basins, and the use of a seepage coefficient to compute the groundwater flows. This section of the report documents the procedure followed to develop the stage boundary conditions at the boundary of the model with the WCA3B and ENP.

LOS application run needs stage Boundary Condition in WCA3B and ENP, however, because the stage boundary conditions are for synthetic storm events and not historical events, the stage hydrographs have to be associated with storm events of particular return periods. This was accomplished by associating observed stage hydrographs in WCA3A and ENP with historical storm events whose return period was assigned using a statistical data fit to a Log Pearson distribution of daily rainfall at selected rainfall gauges. In order to establish the stage hydrographs at these gages corresponding to 5-yr, 10-yr, 25-yr and 100-yr storm events, rainfall records at these gages were analyzed to isolate the specific storm events.

Rainfall Data Source

Rainfall data for the selected gauges 3B-SE and NESRS3 were extracted from the SFWMD 2x2 binary rainfall file that covers the period of 1965 to 2010. The rainfall data in this file has already been processed and data gaps filled for continuous records. **Figure B-18** shows the location of the selected gauges with respect of the SFWMD 2-mile by 2-mile grid. After the daily rainfall data were extracted, 72-hr rainfall records were computed at each gauge by adding consecutive rainfall totals for consecutive 3-day periods and fitting Log Person distributions to the two 72-hr rainfall records. **Figures B-19 and B-20** show the Log Pearson distributions for gauges 3B-SE and NESRS3. The associated 5-, 10- and 25-yr rainfall totals for each distribution are shown in **Tables B-3 and B-4**.

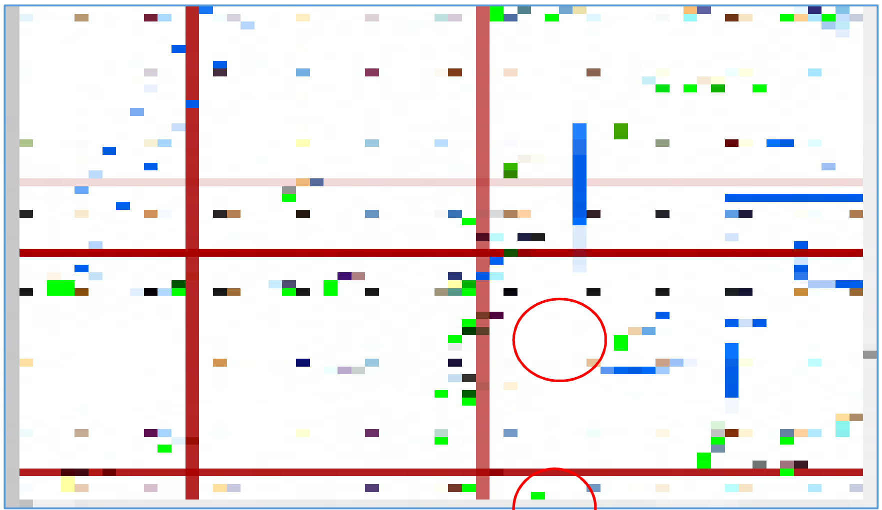

Figure B-18. 2x2 Model Cells and the Location of Gauges 3B-SE and NESRS3

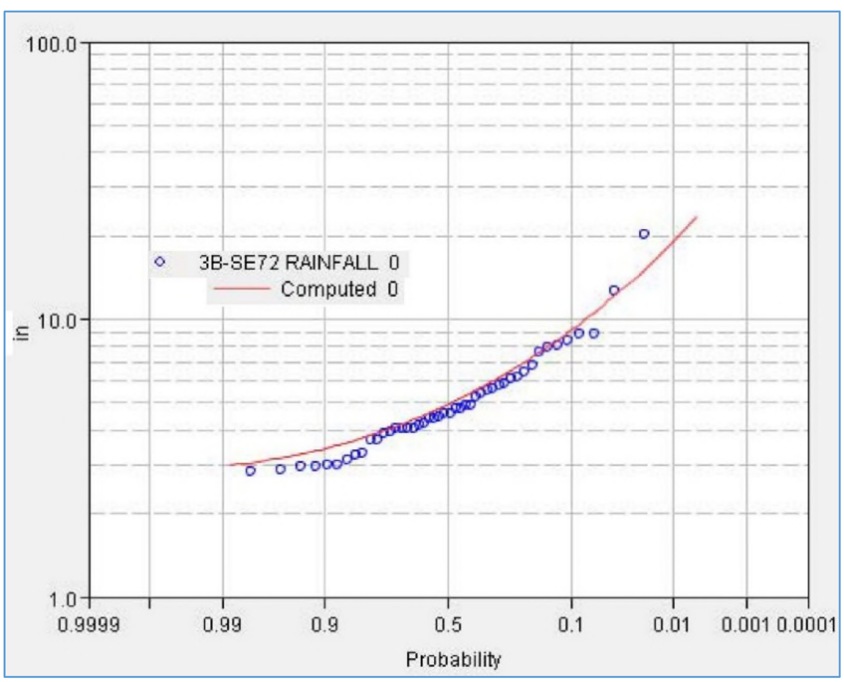

Figure B-19. The Log-Pearson Distribution of Rainfall Depth at Gage 3B-SE

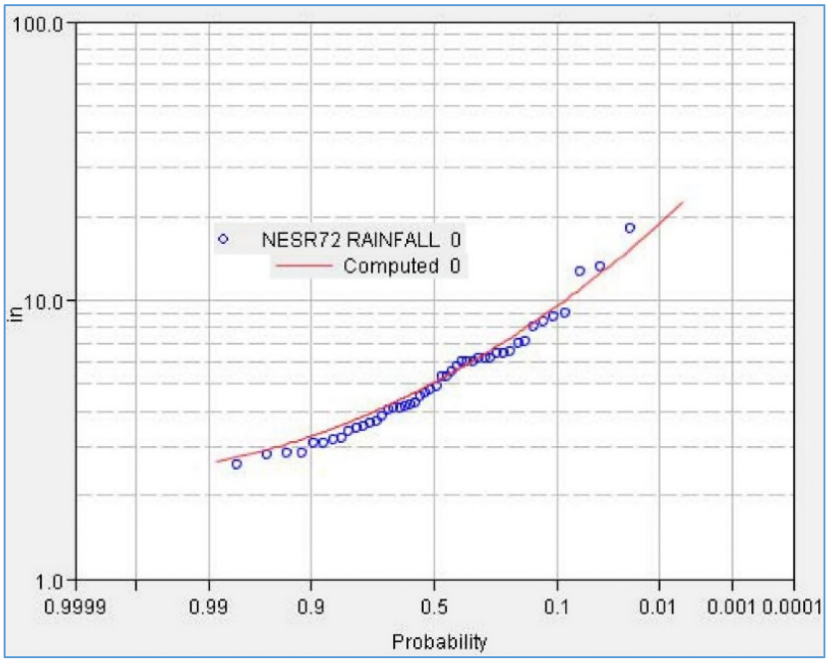

Figure B-20. The Log-Pearson Distribution of Rainfall Depth at Gage NESRS3

Table B-3. 72-hr Rainfall Depth for Design Storms at 3B-SE

Probability	Return Period (yr)	72-hr RF (in)
0.04	25	12.41
0.1	10	9.20
0.2	5	7.20

Table B-4. 72-hr Rainfall Depth for Design Storms at NESRS3

Probability	Return Period (yr)	72-hr RF (in)
0.04	25	12.73
0.1	10	9.59
0.2	5	7.54

Stage Data Source

Daily stage data at the 3B-SE and NESRS3 gauges were extracted from the District's Dbhydro database to associate rainfall volume with observed stages. For gage 3B-SE, the dataset with DBKey **5584** (05/10/1984 to 07/14/1994) and **15934** (07/14/1994 to 2010) were combined to get daily a continuous record of mean stage data for the period of 1984 to 2014. Similarly, for gage NESRS3_B, the datasets with DBKey **5516** (08/02/1984 to 07/14/1994) and **15923** (07/14/1994 to 2010) were also combined into a continuous data set of daily mean stage values. **Figure B-21** shows the rainfall and stage data at gage 3B-SE from year 1984 to year 2010. In this figure, lines corresponding to the rainfall depth for the 5-yr, 10-yr and 25-yr are plotted together with the rainfall time series to identify the design storm events. As this figure shows, there are four 5-yr storms 11/17/1994, 6/22/1995, 6/10/1997, and 9/17/1998 in the period of analysis. Also, there are two 10-yr storms events occurring on 6/3/1999 and 10/4/2000, and one 25-yr storm event on 10/16/1999 (hurricane Irene). **Figures B-22 to B-25** show the rainfall hyetographs and stage hydrographs for the four identified storm events with a return period of 5 years, while **Figures B-26 and B-27** show the stage hydrographs and rainfall hyetographs for the two storm events with return period of 10 years. **Figure B-28** shows the rainfall and stages for the only 25-yr return period event.

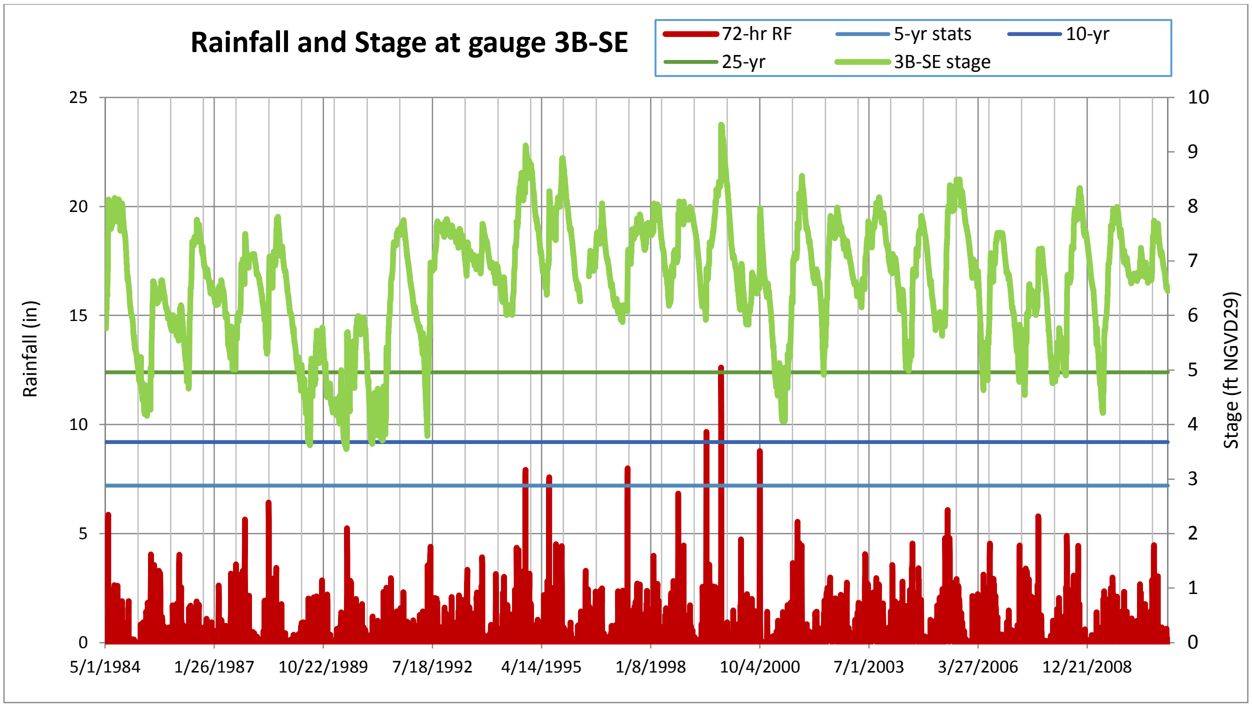

Figure B-21. Rainfall and Stage at 3B-SE (1984 to 2010)

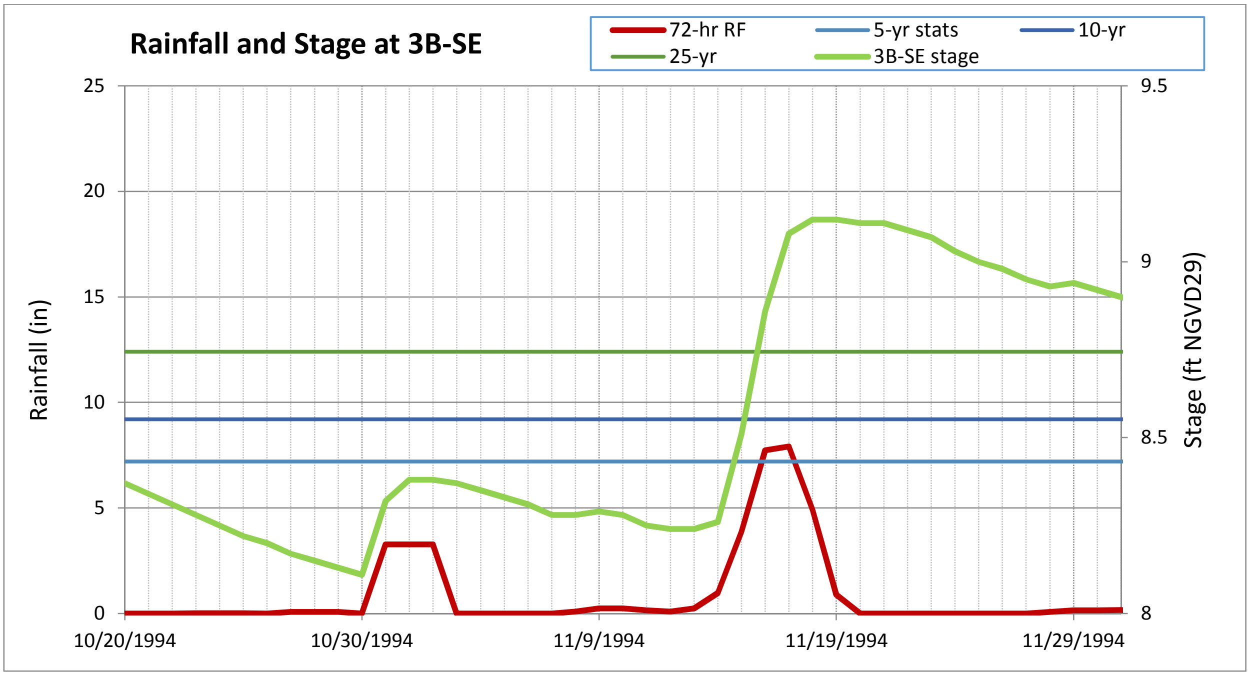

Figure B-22. 5-yr Storm at 3B-SE (11/17/1994)

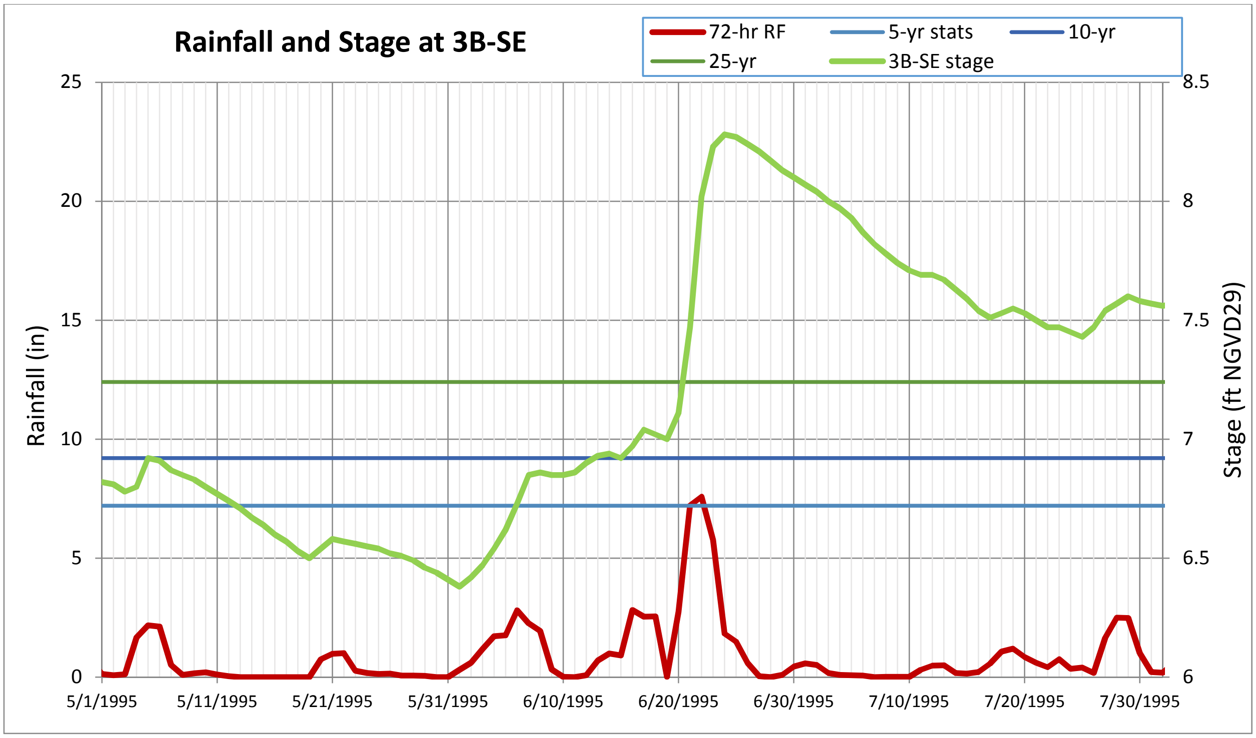

Figure B-23. 5-yr Storm at 3B-SE (6/22/1995)

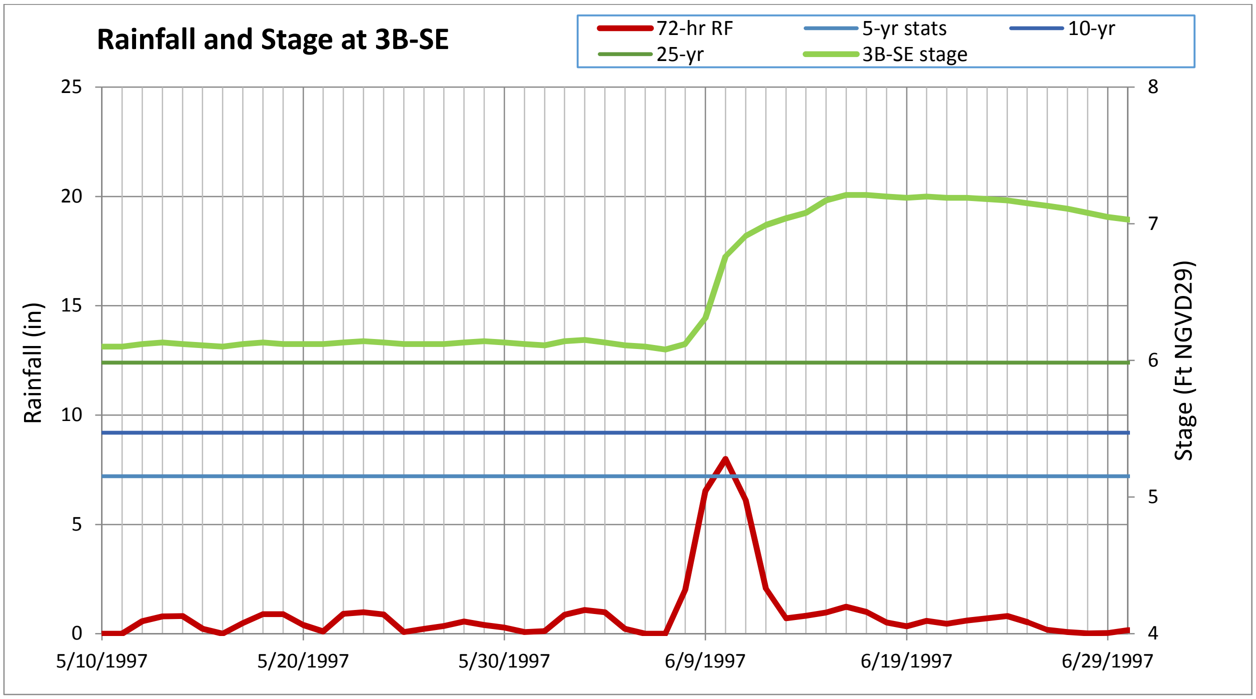

Figure B-24. 5-yr Storm at 3B-SE (6/10/1997)

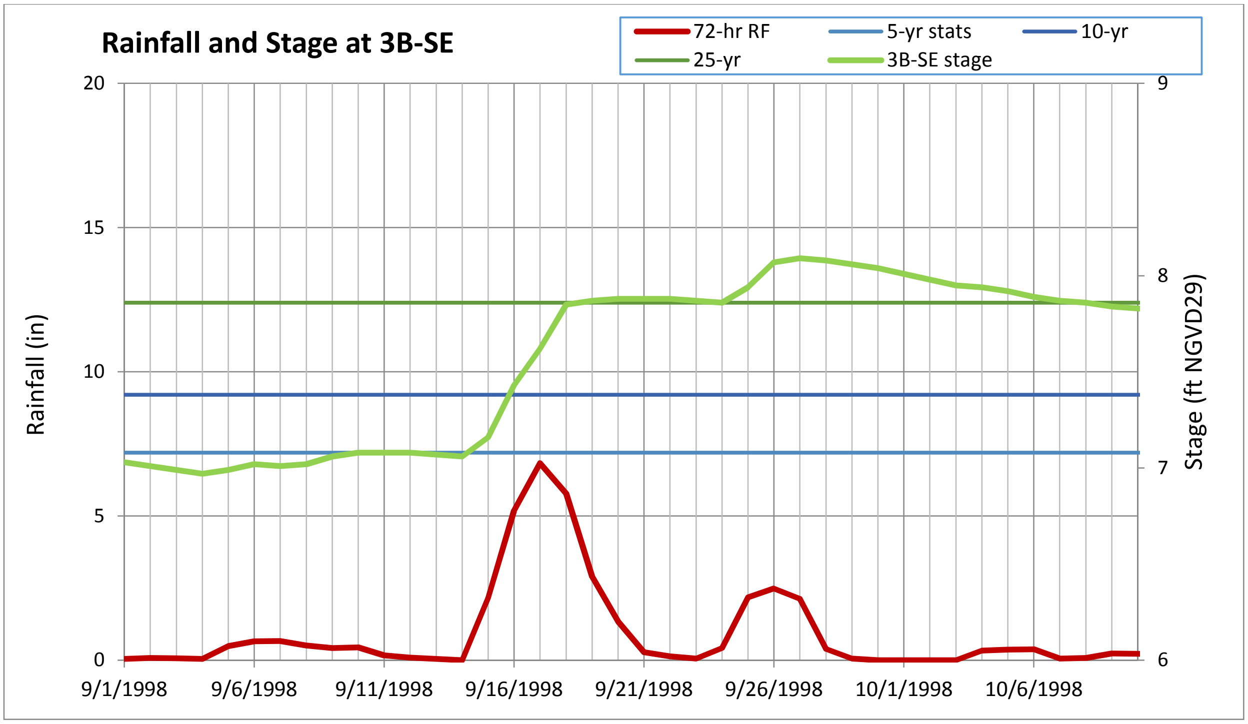

Figure B-25. 5-yr Storm at 3B-SE (9/17/1998)

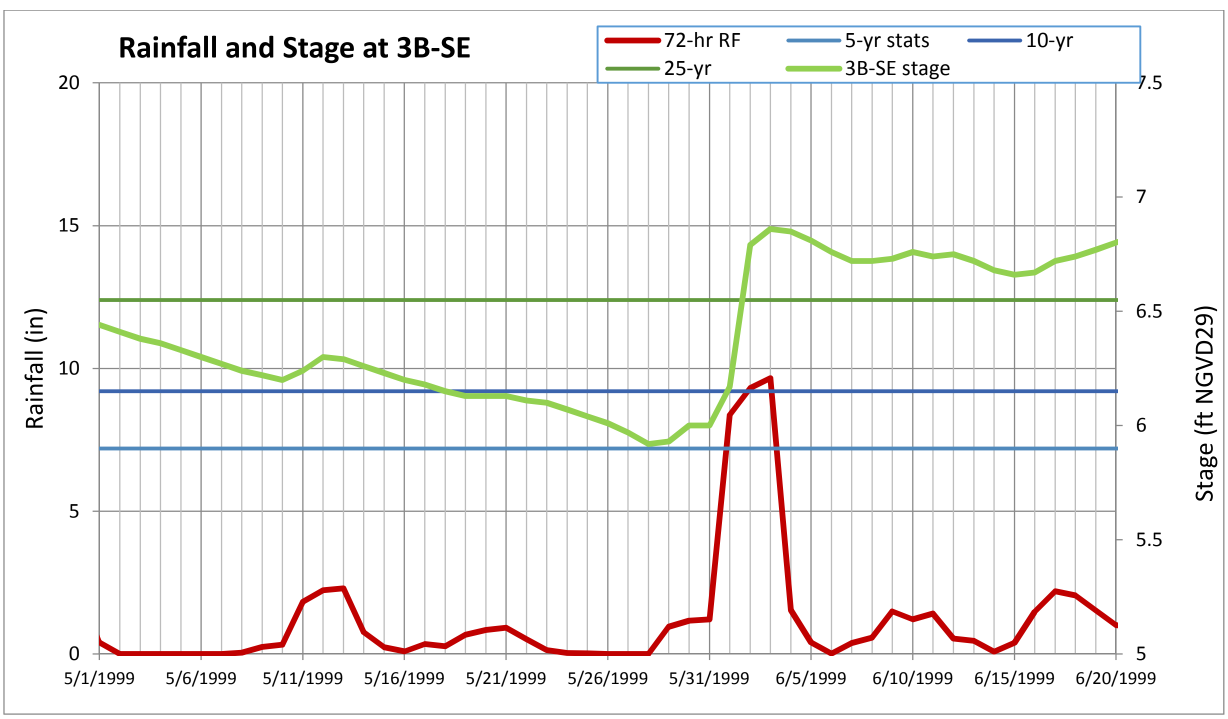

Figure B-26. 10-yr Storm at 3B-SE (6/3/1999)

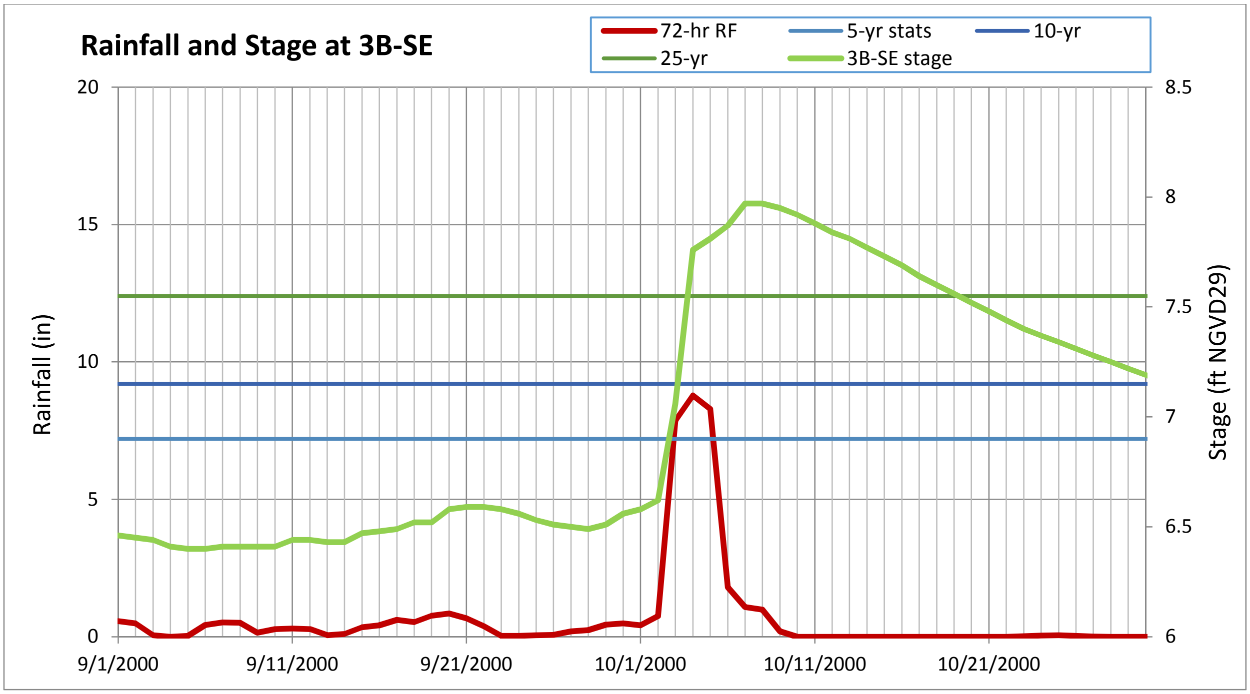

Figure B-27. 10-yr Storm at 3B-SE (10/4/2000)

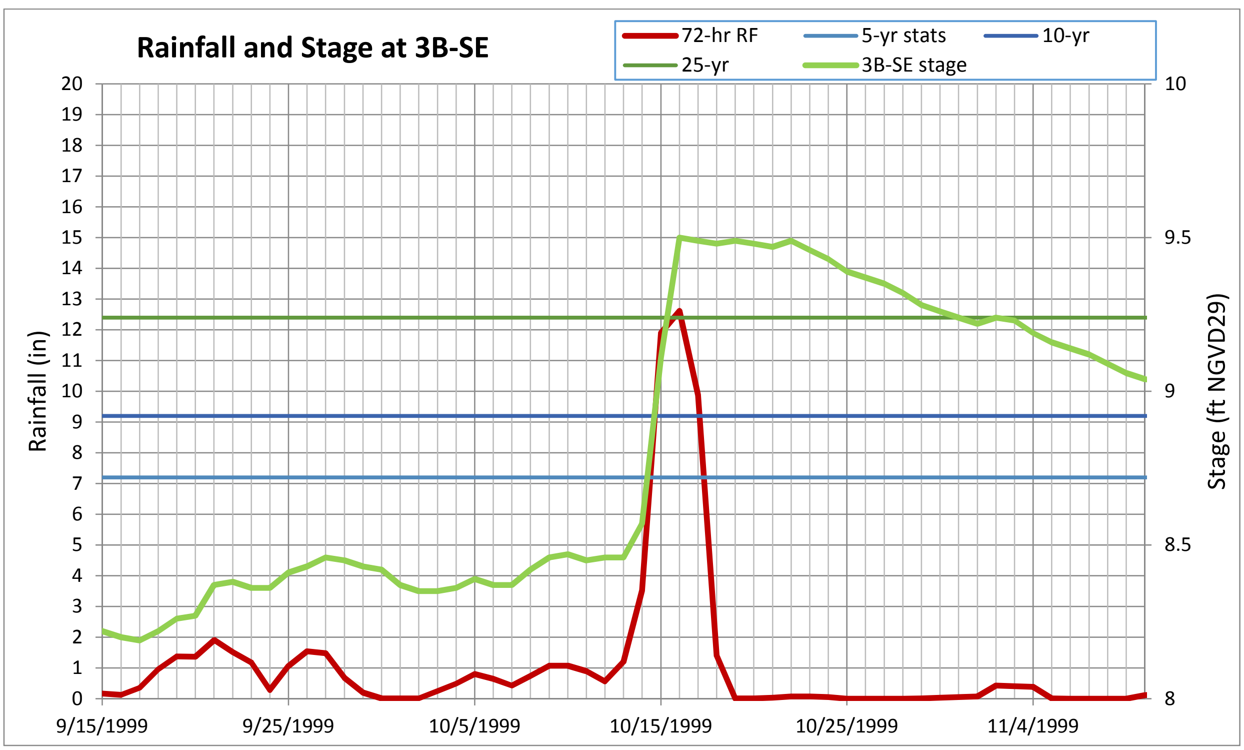

Figure B-28. 25-yr Storm at 3B-SE (10/16/1999)

For gauge NESRS3 a similar analysis was performed to identify storm events with return periods of 5, 10 and 25 years. **Figure B-29** shows the rainfall and stage data at gage NESRS3 for the period of record with available data of 1984 to 2010. The graph also includes rainfall depth lines indicating the value of the rainfall corresponding to the 5-yr, 10-yr and 25-yr storm events.

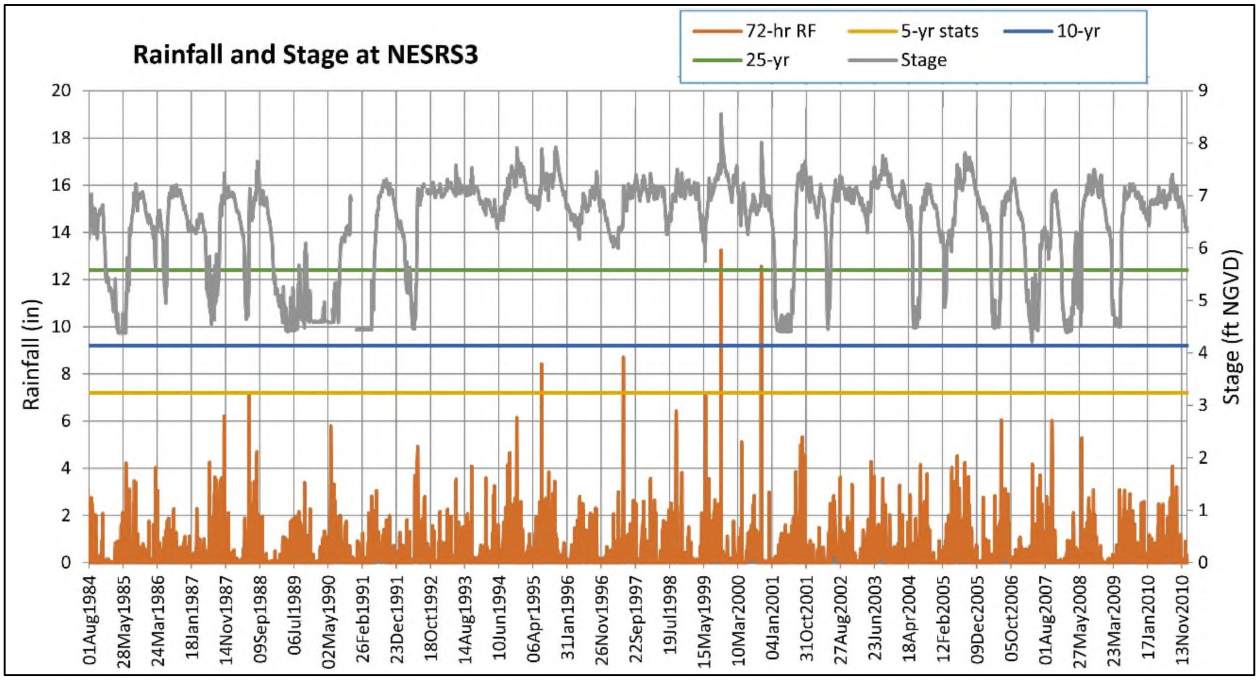

Figure B-29. Rainfall and Stage at NESRS3 (1984 to 2010)

The statistical analysis of the rainfall at gauge NESRS3 indicated that two events with return period of 5 years (6/8/1988, and 6/3/1999) shown in **Figures B-30 and B-31**; two events of 10 years return frequency (6/22/1995, and 6/10/1997) shown in **Figures B-32 and B-33** and two 25-yr events one 10/15/1999 (hurricane Irene) and 10/4/2000 (No-name storm) shown in **Figures B-34 and B-35**.

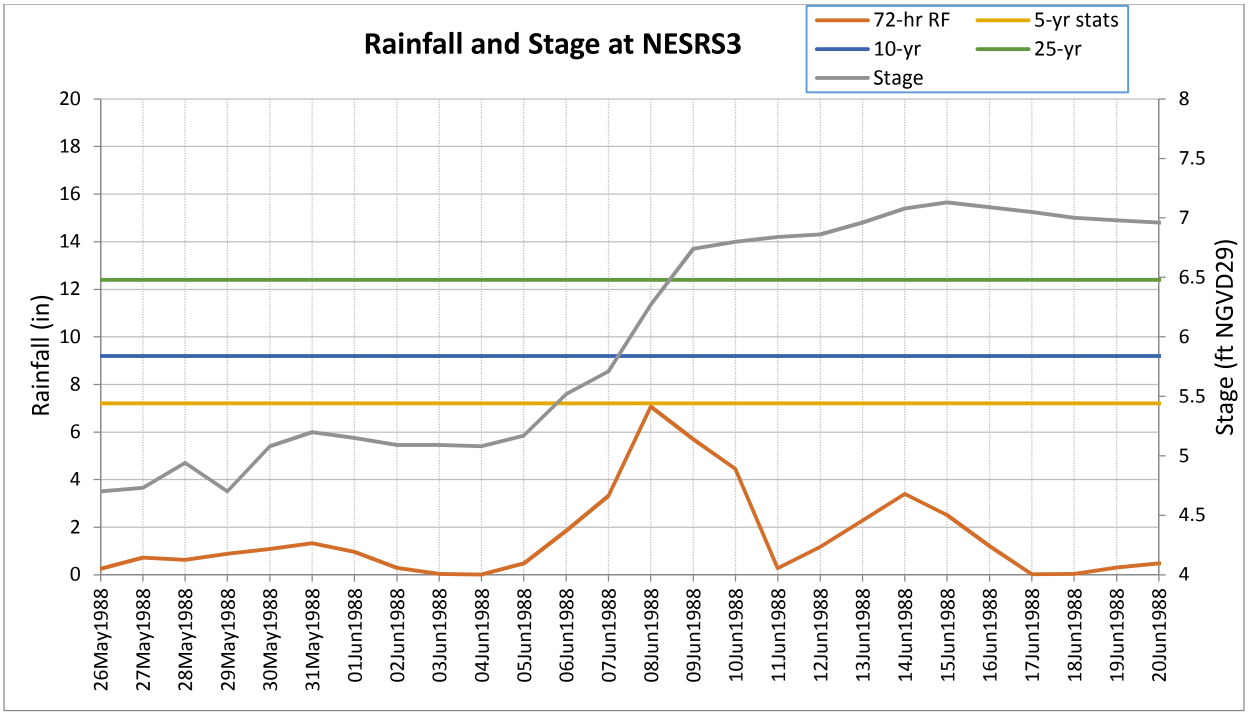

Figure B-30. 5-yr Storm at NESRS3 (6/8/1988)

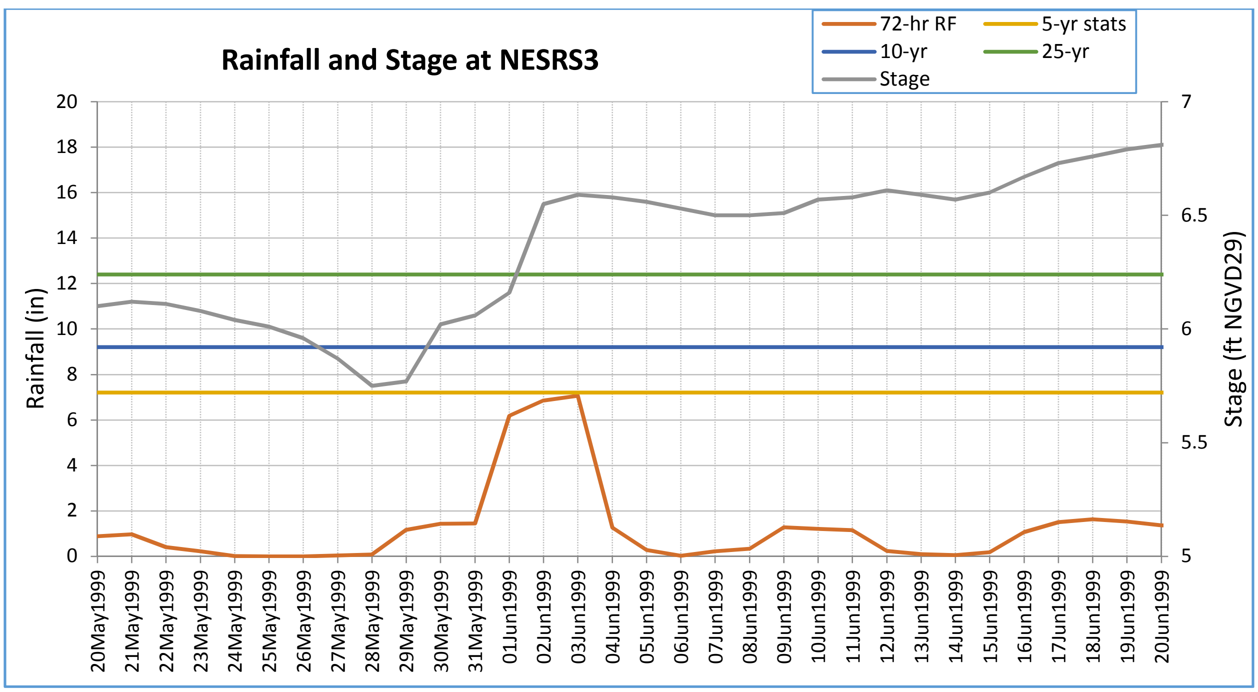

Figure B-31. 5-yr Storm at NESRS3 (6/3/1999)

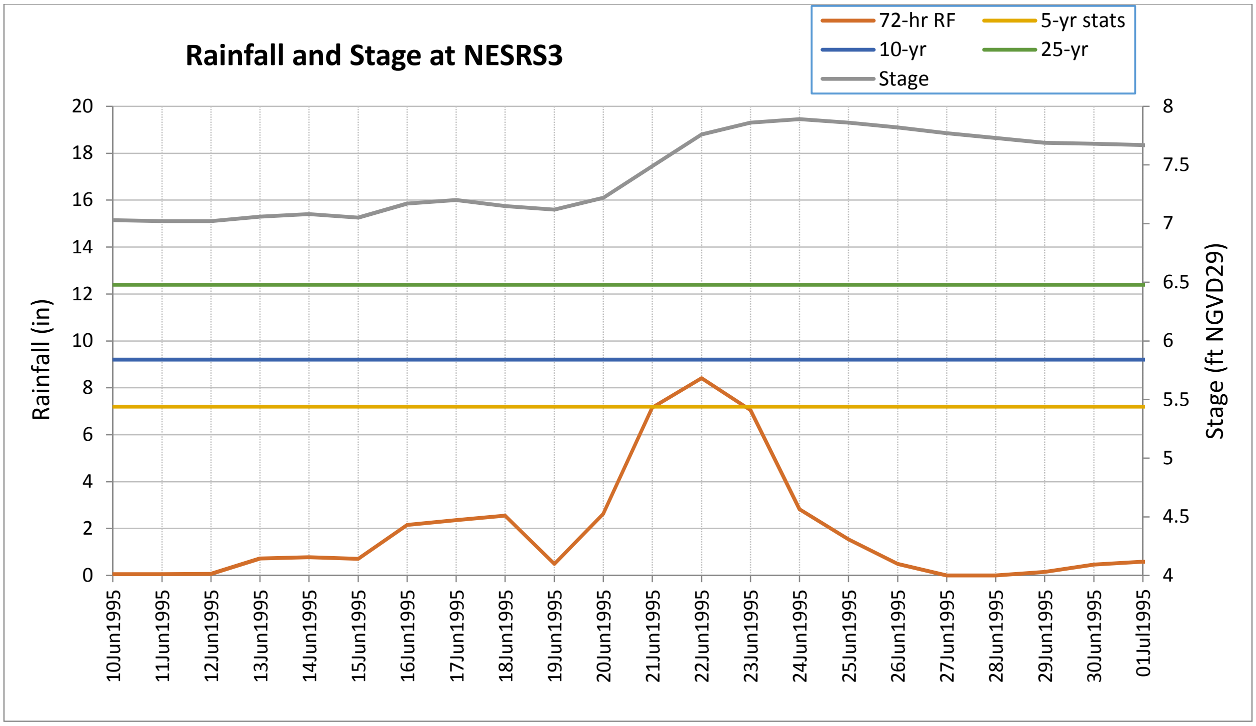

Figure B-32. 10-yr Storm at NESRS3 (6/22/1995)

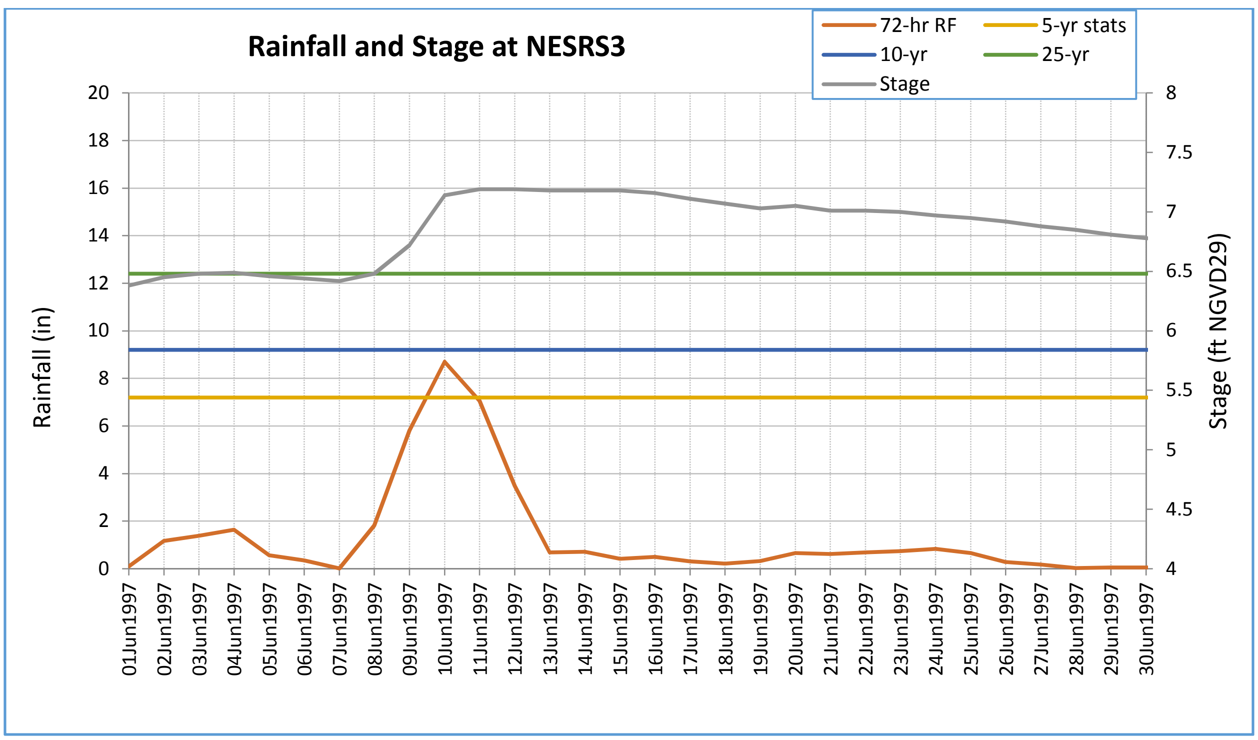

Figure B-33. 10-yr Storm at NESRS3 (6/10/1997)

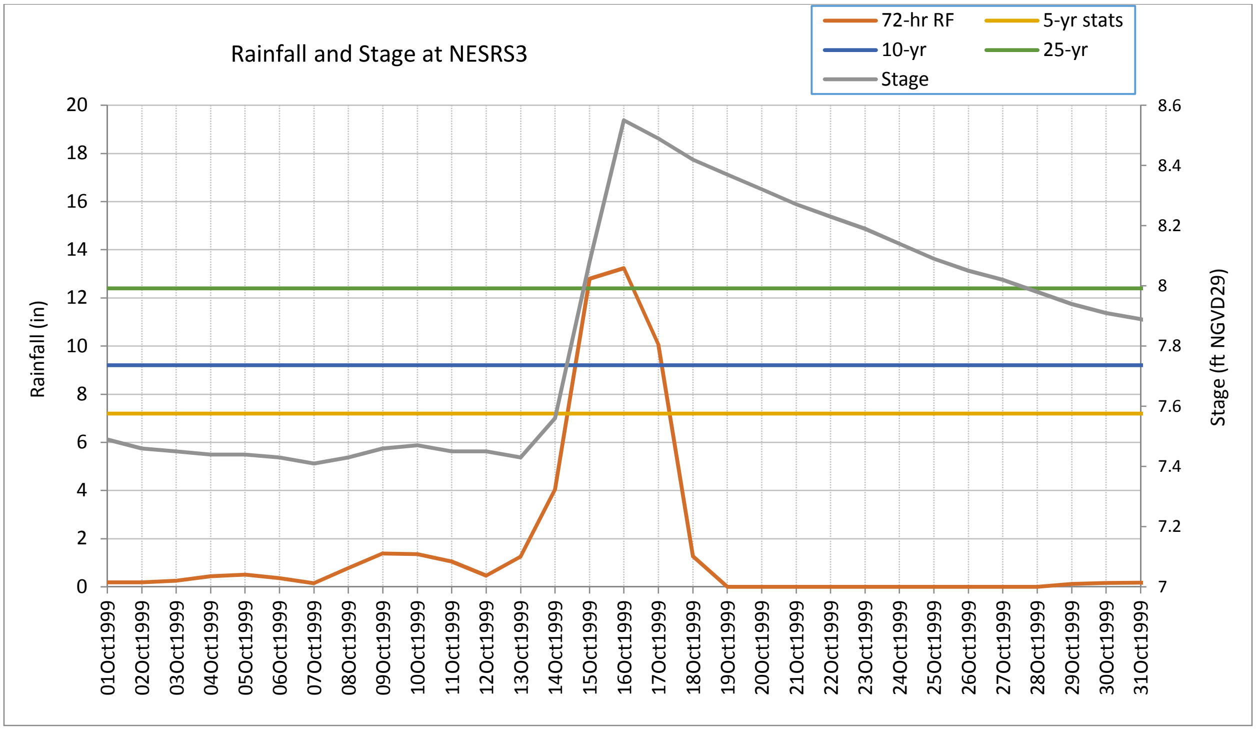

Figure B-34. 25-yr Storm at NESRS3 (10/16/1999)

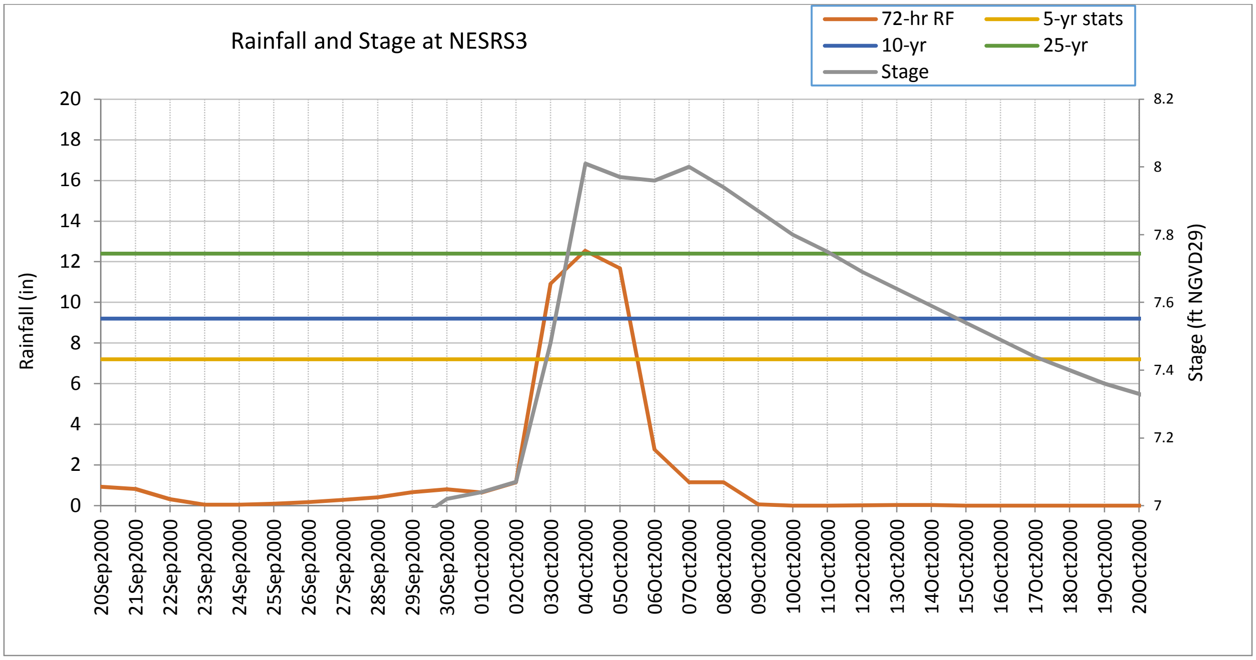

Figure B-35. 25-yr Storm at NESRS3 (10/4/2000)

After the 5-year, 10-year and 25-year storm events for the WCA3B and ENP were identified, the shape of the stage hydrographs at WCA3B and ENP was established using a time to peak value of 2 days for all storm events and, for the recession portion of the hydrographs, a declining slope obtained from the historical events. **Tables B-5** for gauge 3B-SE and **Table B-6** for gauge NESRS3 summarize the parameters needed to define the shape of the stage hydrographs for the 5-year, 10-year, 25-year and 100-year storm events. These tables assume that the all storm events start on July 26 to July 31st as pre-storm warm-up period, beginning of storm on August 1st, peak stage on August 3rd, and recession from peak starting on August 4th to August 24th. **Figure B-36** shows the stage hydrographs for the 5-year, 10-year, 25-year and 100-year events in the ENP (gauge NESR3S) while **Figure B-37** the corresponding hydrographs in the WCA3B (gauge 3B-SE).

Table B-5. Stage Boundary Condition at Gage 3B-SE for Design Storms

Gage 3B-SE	5 yr	10 yr	25 yr	100 yr
design 24 hr RF (in)	6.75	8.00	10.00	13.50
design 72-hr RF (in)	9.17	10.87	13.59	18.35
Draw down Slope	0.02	0.03	0.03	0.03
initial stage (ft)	8.50			
26-Jul-12	8.50	8.50	8.50	8.50
1-Aug-12	8.50	8.50	8.50	8.50
3-Aug-12	9.26	9.41	9.63	10.03
26-Aug-12	8.80	8.62	8.94	9.34

Table B-6. Stage at Gate NESRS3 for Design Storms

Gage NESRS3	5 yr	10yr	25yr	100yr
design 24 hr RF (in)	6.75	8.00	10.00	13.50
design 72-hr RF (in)	9.17	10.87	13.59	18.35
Draw down slope	0.02	0.03	0.04	0.04
original stage (ft)	7.20			
26-Jul-12	7.20	7.20	7.20	7.20
1-Aug-12	7.20	7.20	7.20	7.20
3-Aug-12	7.96	8.11	8.33	8.73
26-Aug-12	7.50	7.32	7.41	7.81

Initial condition

Initial water levels and flows in the HEC-RAS model are required for the storm events runs for LOS evaluation. Since the storm event runs are synthetic storm events, the initial conditions have to be generated statistically or with another model. In this case, each of the initial condition stages and flows were generated with a Steady State version of the C-4 Basin HEC-RAS model which reflects a 2-year return period base flow of 405 cfs at the S-25B structure. The procedure to estimate the base flow value at structure S-15B follows.

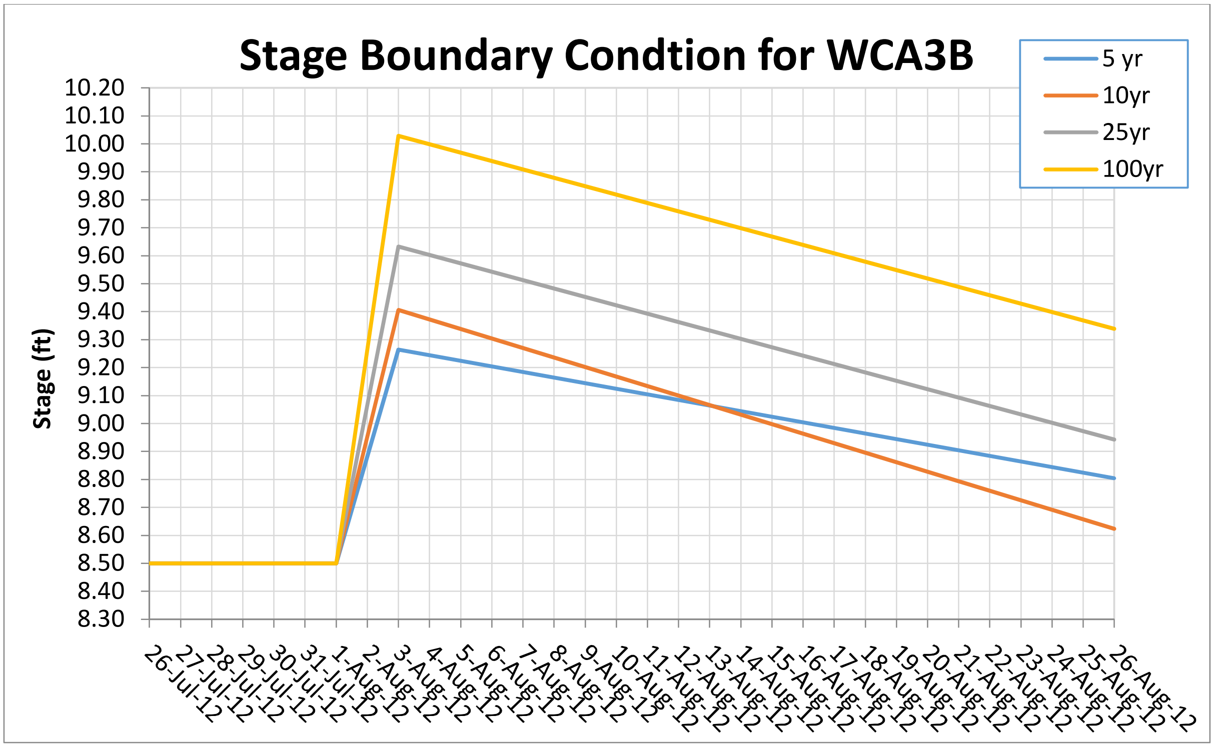

Figure B-36. Stage Boundary Condition for WCA3

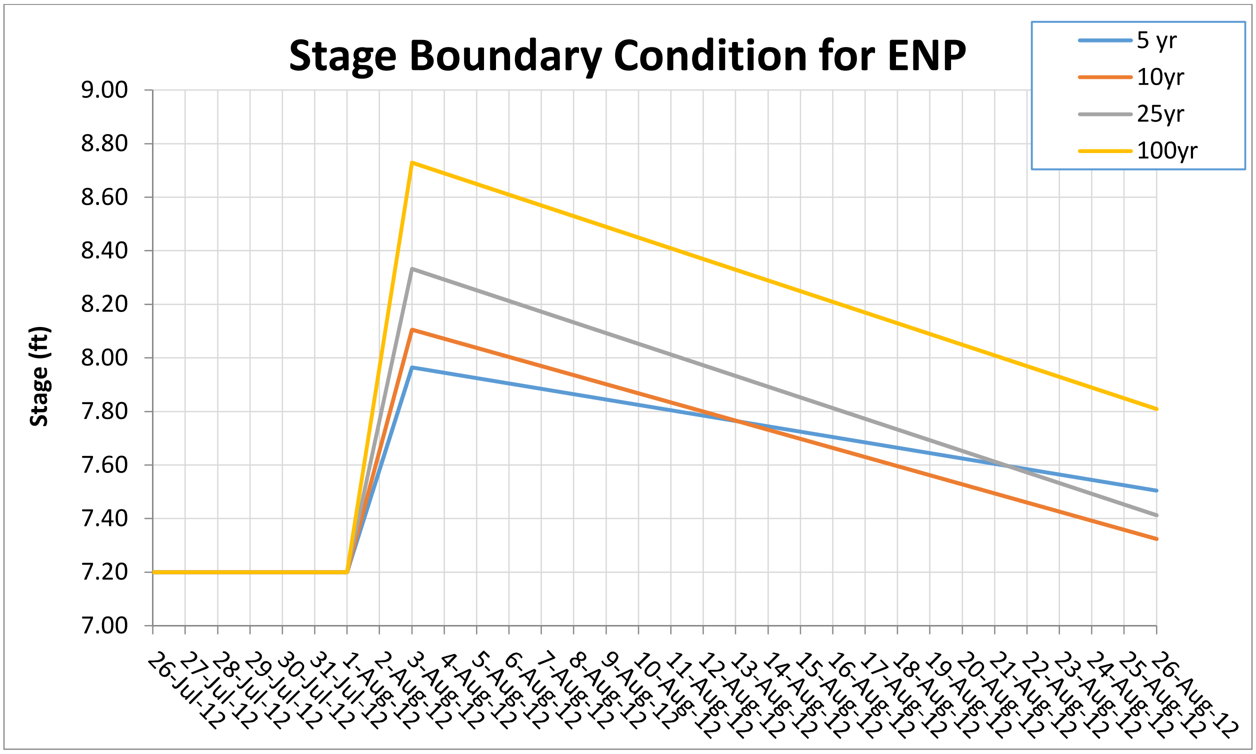

Figure B-37. Stage Boundary Condition for ENP

Base flow at S25B

Base flow provides initial condition for the HEC-RAS model for LOS project. Since the model will simulate storm events of short periods, initial condition will have an impact the modeling results. It is important to get a reasonable base flow estimation to provide reliable initial condition for the model. The steps to estimate the base flow at structure S-25B is as follows:

1. Use daily mean flow time series at S25B (Dbkey **06780**) with a period of record of 1985 - 2014.
2. Pick a threshold value for the increase of flow at a storm event. The threshold value is chosen to be 300 cfs for S25B. When the increase of flow (dQ) is over the threshold value, the storm is counted as an event.
3. Identify the base flow of a storm event by comparing flow over three day period. If the difference between the flow on the current day and the flow three days before is over 300 cfs, record the date (three days before) and flow on that day. This is the base flow at the start of an event.
4. Calculate annual maximum flows over the period of record (1985 to 2014).
5. The average of the maximum flows of 405.3 cfs for the period of record represents the 1-in-2 annual base flow initial condition.
6. Review the S25B flows during the calibration period and select a date where flows approximate the 400 cfs base flow value and use the modeled sub-watershed stage values for that date as the initial condition for all current-condition model application, i.e. use hot-start stages as initial stations.

Figure B-38 displays the time series of flows for the period of 1985 to 2012 with the estimates of the annual maximum base flows and line that represents the average annual base flow at structure S-25B.

A more detailed discussion on the procedure to generate the steady state initial condition hot start files for each of the model can be found in **Appendix G**. The resulting sub-basin initial water levels for current and future conditions with sea level rise are summarized in **Table G-1 of Appendix G.**

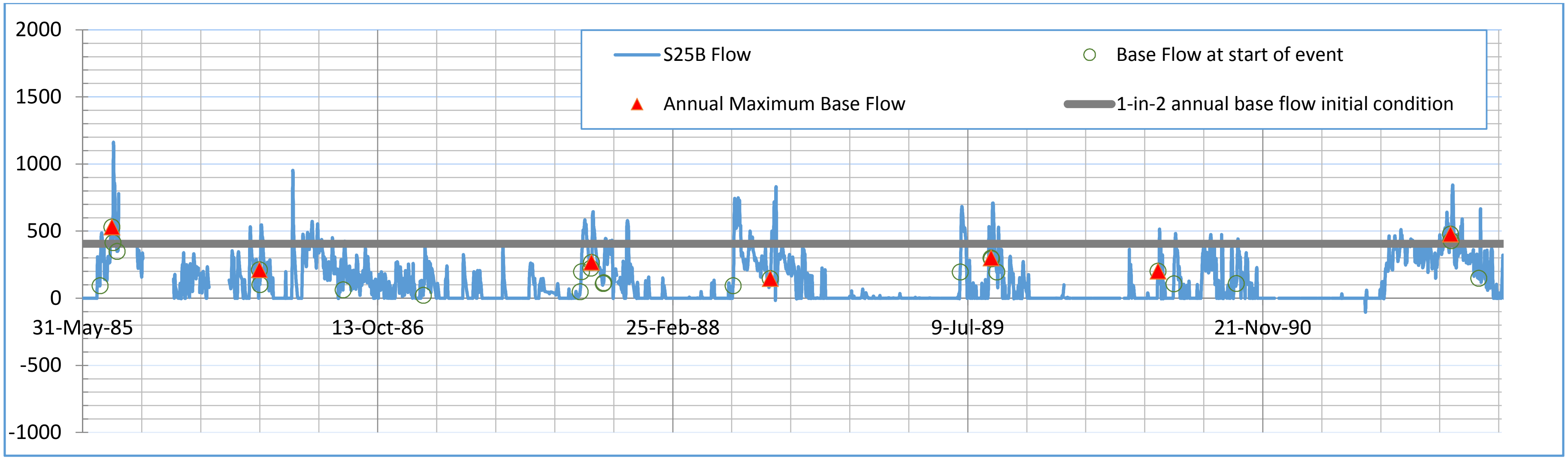

Figure B-38. Daily and Average Maximum Base Flows at Structure S-25B (1985 – 2014)

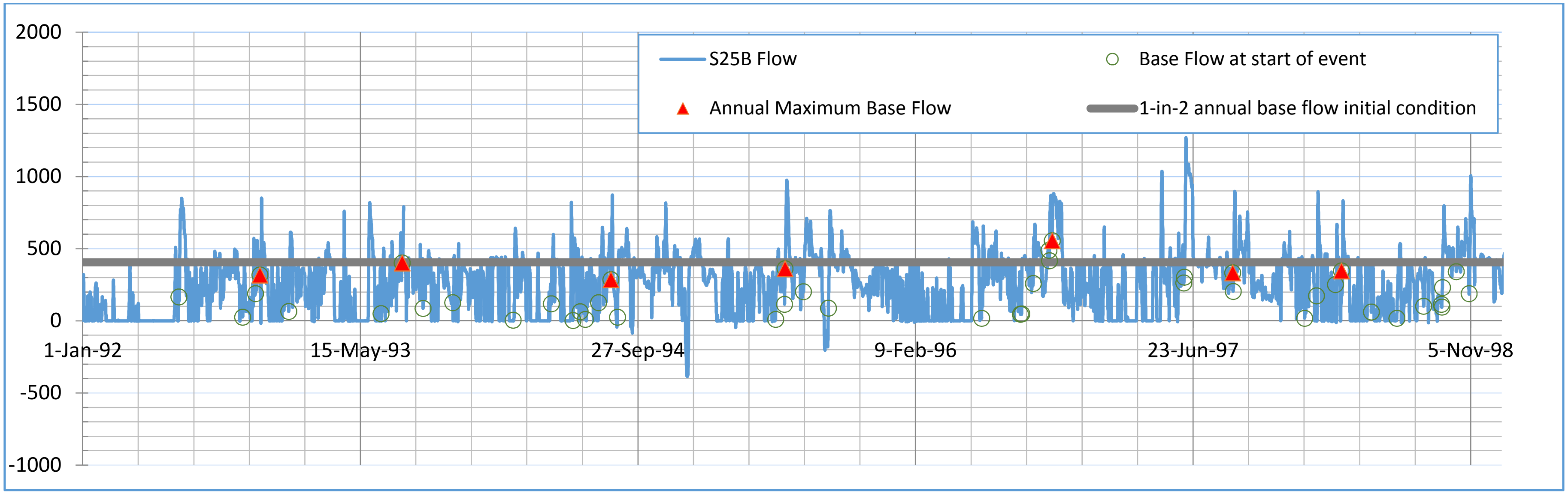

Figure B-38. Daily and Average Maximum Base Flows at Structure S-25B (1985 – 2014) (Cont.)

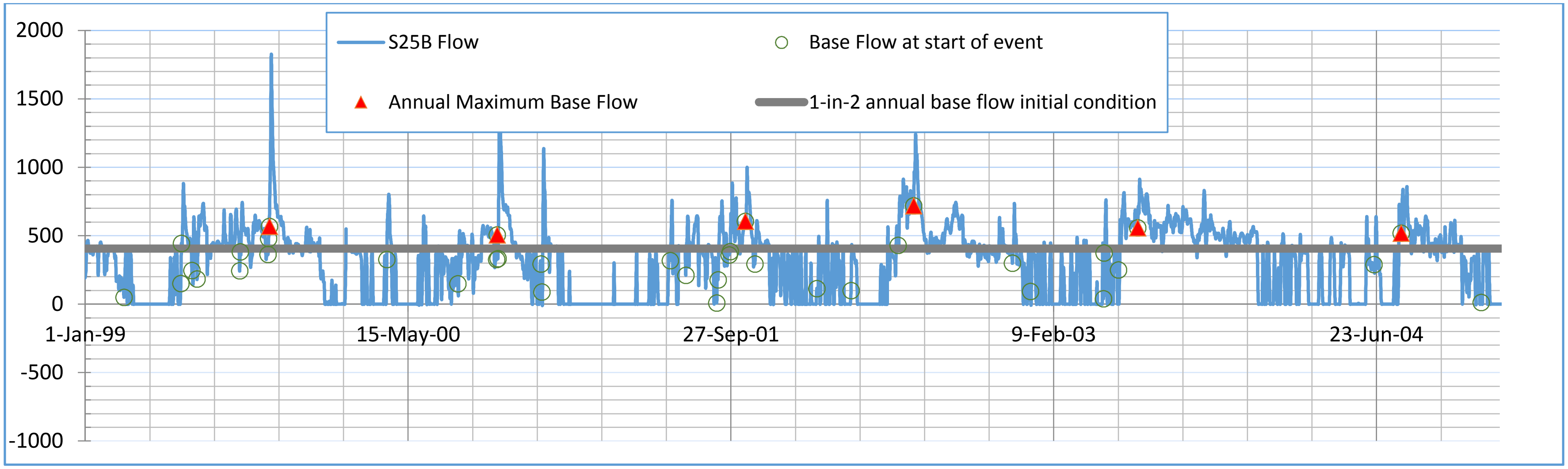

Figure B-38. Daily and Average Maximum Base Flows at Structure S-25B (1985 – 2014) (Cont.)

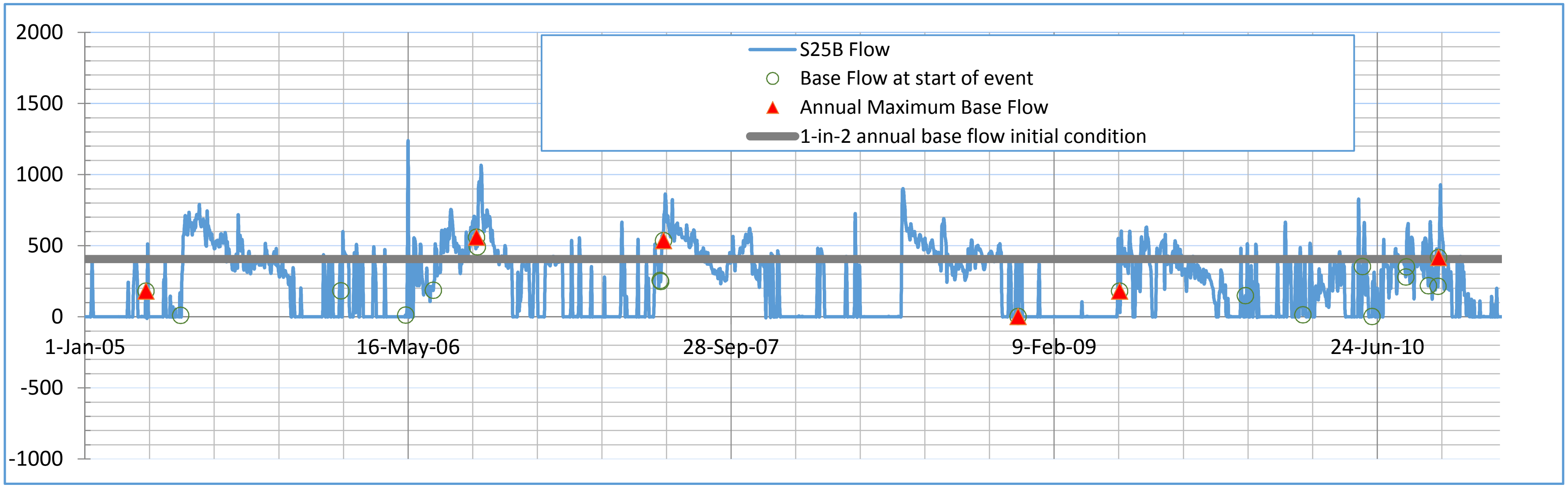

Figure B-38. Daily and Average Maximum Base Flows at Structure S-25B (1985 – 2014) (Cont.)

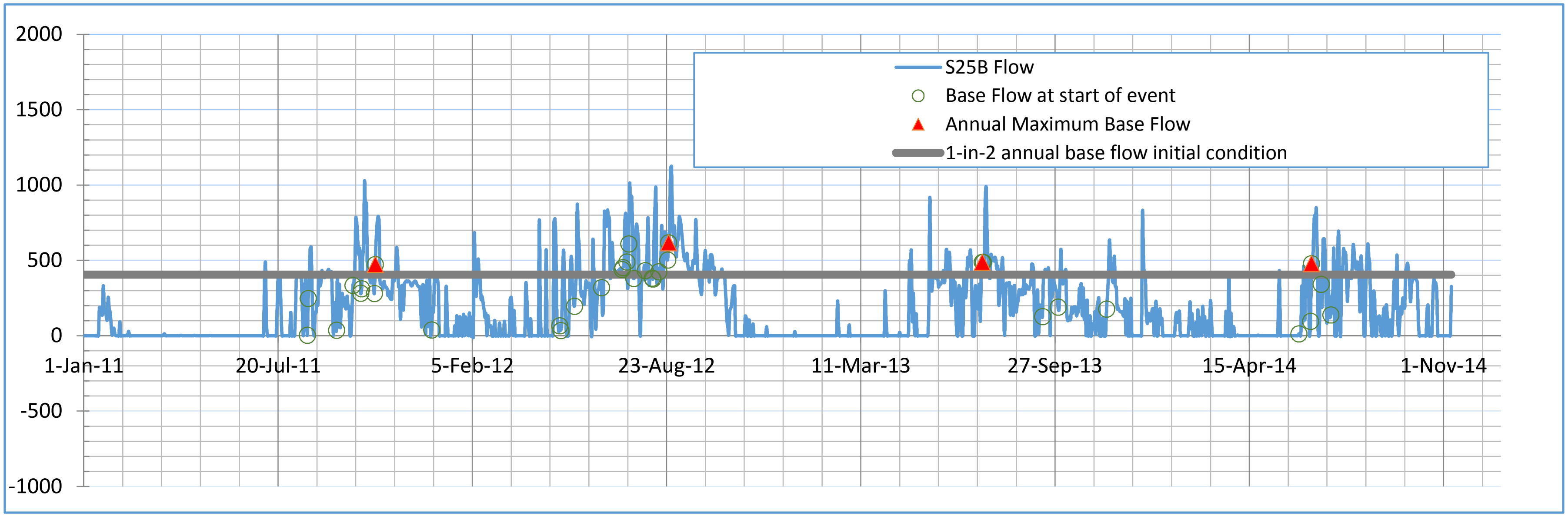

Figure B-38. Daily and Average Maximum Base Flows at Structure S-25B (1985 – 2014) (Cont.)

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Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed

Appendix C: Preparation of Boundary Conditions at the Tidal Structures

South Florida Water Management District
Hydrology and Hydraulics Bureau

December 17, 2015

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Appendix C – Boundary Conditions at Tidal Structures

This document was produced by the H&H Bureau as a project deliverable for project 100888 (Flood Protection Level of Service, within the Sea Level Rise project).

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Additional project documents can be found on a District server at:

\\ad.sfwmd.gov\dfsroot\data\hesm_nas\projects\basin_studies

Suggested citation:

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Appendix C – Boundary Conditions at Tidal Structures

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INTRODUCTION

The flood waters of the greater C-4 basin discharge to the ocean via several coastal water control structures which are also known as "salinity barriers." In the C&SF project, they were designed and constructed as gravity drainage structures with gates which could be used to prevent saltwater intrusion inland. There is very little evidence that the original designs of the structures considered future sea levels of the magnitude that are being discussed in the current literature. Under design conditions, most structures in Miami-Dade county had only about 6 inches of head loss across the structure and as a result of sea level rise during the last half century, the operation of the costal structures have become extremely challenging, particularly during higher high tides (known also as perigean spring tides or king tides). Under conditions of higher "tailwater" (elevated water level on the ocean side of the structure) which may occur due to astronomical tides, offshore winds and storms, climate effects, and ocean currents, the coastal structures of Miami-Dade typically are closed twice a day to prevent reverse flow of ocean water into the inland portion of the canal. Along the coast of southeast Florida, tides are semi-diurnal and hence there are two high tides events every day. The tidal amplitude varies spatially. It is beyond the scope of this report to describe the basic science of tides, their natural characteristics in the region and the reader is referred to the NOAA website (https://tidesandcurrents.noaa.gov/) for an excellent discussion of various topics associated with tides.

Tides experience diurnal, seasonal, interannual, and decadal scale variations due to a variety of factors. Besides the astronomically induced variation, the tides also experience seasonal variation due to variations in temperature, salinity and wind patterns. The interannual variation are due to what are known as teleconnections such as variations in gulfstream, and in some cases El Nino. The longest cycle considered in the tidal projection is due to lunar effects and this is approximately 19 years. Therefore, at least 19 years are needed to resolve multi-year variations in tides. There are three tide gages with longer term water level measurements in the region: Key West, Vaca Key, and Virginia Key (formerly located in Miami Beach prior to Hurricane Andrew). Although they are useful for understanding tides and sea level extremes, what is needed for this C-4 LOS study are the water levels immediately downstream of the coastal structures. Because some of the coastal structures (e.g. S-25B, G-93) are located several miles inland, the tidal effect at the structure could be different from those observed at the NOAA tide gages and the effects of local rainfall could be greater.

The projections of tidal boundary conditions was facilitated by high-resolution water level data downstream of the coastal structures available since about 1985 (in most cases) and therefore the records exceed the 19-year requirement. These records reflect effect of tidal variations, and other effects such as those of storms and offshore winds, seasonal variations, and teleconnections. Consequently, the team assumed that there was no reason to account for various factors separately although such a treatment may be necessary in a more rigorous analysis later.

There is no standard practice for the production of time-dependent coastal boundary conditions for flood modeling. Some have used computationally intensive hydrodynamic modeling of historical and synthetic hurricanes and tropical storms but that is beyond the scope of C-4 basin LOS project. Since this as an active area of research, the team decided used multiple methods from the literature to determine a reasonable set of extreme sea levels under storm conditions corresponding to multiple return periods. They constituted a variety of methods for treating extreme values using both annual maxima and peaks over threshold (Coles 2001). The methods may or may not directly incorporate tides but the effect of pre- and post-storm tidal variations are superimposed using the tail-water stage data at a particular structure.

Coastal boundary conditions are in the form of stage hydrographs located at the downstream (tidal) side of the structure for different return periods, sea level rise projections, and planning horizons. The process to produce the hydrographs starts with the collection of historical data, followed by the application of frequency analysis to determine extreme stages for different return periods. A base storm hydrograph is selected from the historical data and this hydrograph is re-scaled so that its peak reproduces the extreme value derived from the frequency analysis. Offset values derived from the sea level rise projections are finally added to the re-scaled hydrographs to produce the final boundary conditions.

It is beyond the scope of this appendix to detail the technical methods and intermediate results of the work associated with the C-4 pilot project. Since the same methods may be used and refined in the future, the team plans to prepare a detailed technical report covering the recommended approach (2016).

DATA COLLECTION & PREPARATION

The breakpoint archived tailwater stage data for the structures S-25B, S-22 and G-93, were extracted from DBHYDRO (**SFWMD, 2015**). Also extracted as breakpoint data were the headwater stages and gate openings for the structures, and the flow computed using the District's FLOW program at the structures. Whenever possible and in order to maximize the period of record, data sets observed at the same location but through different sensors-recorders (i.e. telemetry, graphics, data-loggers) were used. The flow data at S25B were corrected for negative flows (when the tailwater stages are slightly higher than the headwater stages) using the tidal correction methodology (**Ansar and Raymond, 2002**). The data was stored as irregular (IR) time series in HEC-DSS format, and the periods of record for the extracted historic data was June 1985 to September 2014 for S-25B and S-22, and October 1991 to September 2014 for G-93.

From the irregular time series data stored in DSS, using the HEC-DSSVUE GUI, the following datasets were created for the tailwater boundary conditions at each of the three structures.

- Daily maxima recorded at the end of the day
- Hourly water level recorded at the end of the hour using interpolation

Time series plots of the S-25B and G-93 tail water stages produced using HEC-DSSVUE (**USACE-HEC, 2009**) are shown in **Figures C-1 and C-2.**

The DSS tailwater datasets were converted to csv files, and imported into the R package for statistical computing and graphics (**R Development Core Team, 2008**). Time series plots (using R) of the tailwater data for the three structures S-25B, S-22 and G-93 are shown in **Figure C-3**. The plots were inspected visually and data outliers corresponding to extreme high values (shown as scatter points within the red ovals in **Figure C-3**) were removed manually. These data points were carefully checked to ensure that only the unrealistic values (due to errors in data collection and processing) were removed. The remaining data were stored as R data for further analysis using the Extreme Values method for weather and climate applications.

Appendix C – Boundary Conditions at Tidal Structures

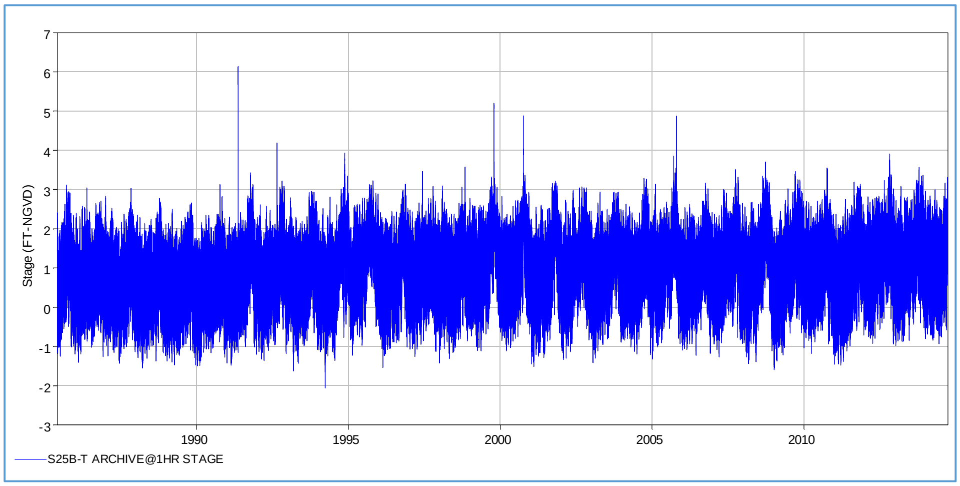

Figure C-1. Time series plot of the S-25B tailwater for the period of record.

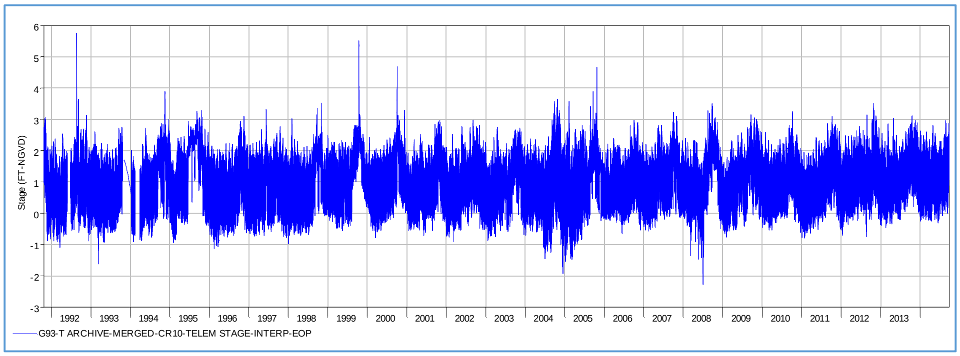

Figure C-2. Time series plot of the G-93 tailwater for the period of record.

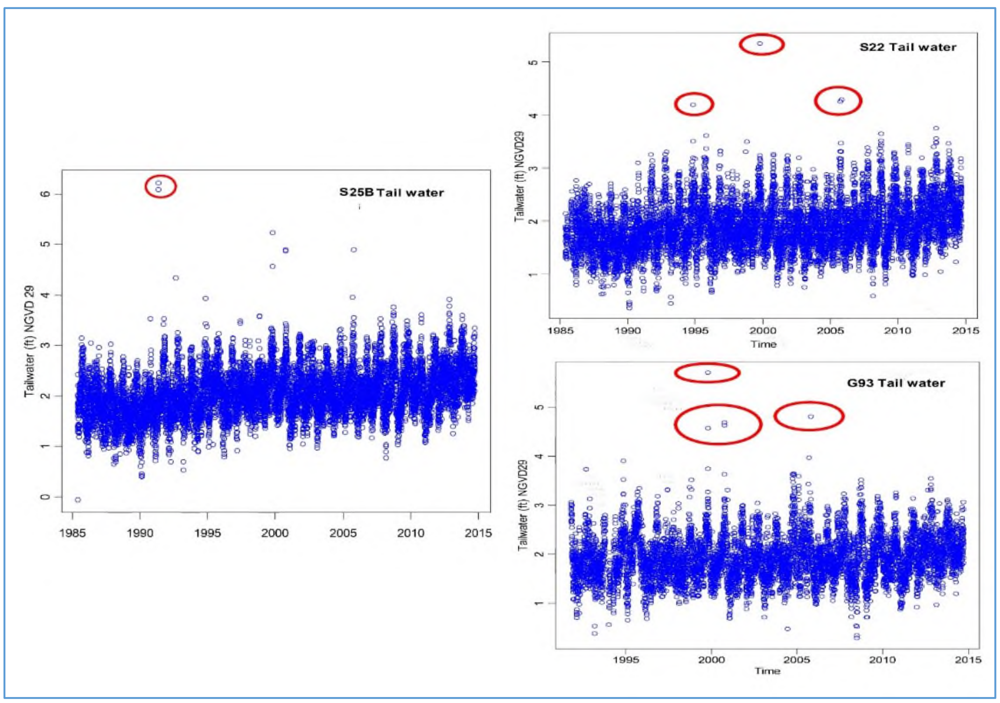

Figure C-3. Time series plot of the tailwater stages at S-25B, S-22 and G-93 Structure for their periods of record. Removed outlier points are shown within the red oval.

Frequency Analysis for Peak Stages

About 30 years of both daily maxima and hourly data are available at most of the coastal structures. The Level of Service analysis for watersheds upstream of the coastal structures require the prediction of peak tailwater elevation corresponding to a series of return periods (RP) currently selected as 2, 5, 10, 25, 50, and 100 years. For the sake of generality such design peak elevations will be referred to as Return Levels (RL). Determination of the return level corresponding to given return period requires an extreme value analysis which consists of appropriate probability distributions of extreme sea level data. Because tailwater elevation records at water control structures contain several components arising from a variety of causes, including tidal fluctuations, longer-term trends due to sea level rise, storm surge including wave heights, and the occasional occurrence of tropical storms, the frequency analysis is not straightforward. There are multiple ways to deal with the complexity of data and they include but are not limited to:

1. Extreme Value of Modeling of Block Maxima (BM) (**Coles, 2001**)
2. Extreme Value Modeling of Peaks Over Threshold (POT) (**Coles, 2001**)
3. Mixture Distributions (MD)
4. Monte-Carlo Joint Probability Methods (**Goring et al., 2011**)
5. Regional Frequency Analysis (**Hosking and Wallis, 1997**)

A typical application for a project may only use few of the above methods. In this pilot project, we examine most of the methods in order to validate and provide a broader range of projections.

The above methods may be applied to data sets in a variety of ways. They include:

1. With or without removing systematic trend (due to sea level rise)
2. With or without removing tides and seasonality
3. Non-stationary approach for including co-variates such as time and flow through the structure
4. Using Skew Surge

The skew surge uses the tail water elevation above the peak tide without considering the timing of them within a tidal cycle. This method is a popular approach in the United Kingdom.

A total of eleven methods were used for computing return levels. The following is a brief description of the methods.

Block Maxima

In case of Block Maxima (BM) modeling, the daily maxima data sets are separated into blocks (**Figure C-4a**). A common block length is one year and in that case the block maxima will be known as Annual Maxima (AM). One of the fundamental assumptions in AM method is that the sample data are independent in time. The data points are so far apart that any serial correlation at the annual level will be minimal. Although there are techniques to deal with serial correlation, they will not be used for this exercise.

There is a myriad of probability distributions that may be use for modeling block maxima. However, the theory of extremes is well documented, and as an asymptotic distribution the Generalized Extreme Value distribution (GEV) is widely used (**Coles, 2001**). Specifically, the cumulative distribution function for the GEV of the extreme (annual maximum) sea level rise random variable (T) can be expressed as:

$$F(T,t) = \exp\left\{-\left[1+\varepsilon\left(\frac{T-\mu(t)}{\sigma}\right)\right]^{-1/\varepsilon}\right\} \qquad (1)$$

where $\mu(t)$ is the location parameter representing the time-dependent sea level rise, and σ and ε are the scale and shape parameters of the GEV respectively. Extreme coastal sea levels are rising primarily due to mean sea level rise, that is, there is not an independent or emerging physical force driving the increase

(a) Block Maxima (BM)	(b) Peaks Over Threshold (POT)

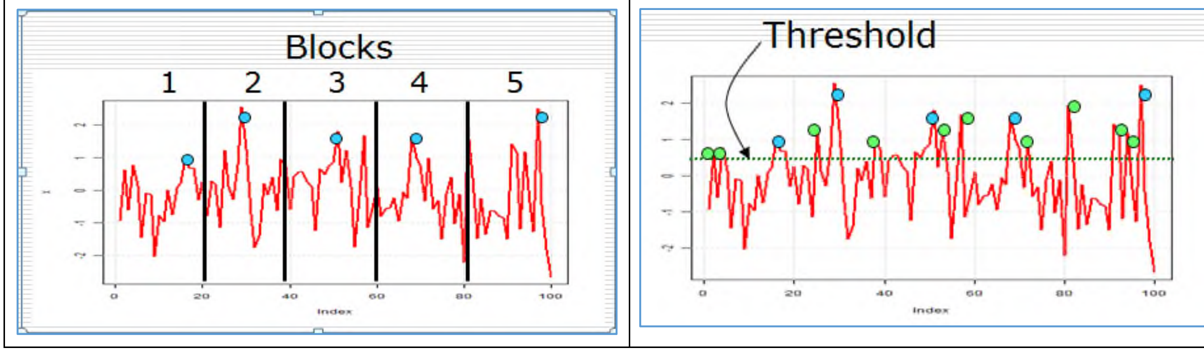

Figure C-4. Illustration of Block Maxima and Peaks Over Threshold (POT) in extreme value modeling

in extremes, therefore we assume that the GEV scale and shape parameters can be modeled as constants. Further, this suggests that there is an offset relating mean sea level rise and increases in extremes at a particular location. It should be noted that the methods presented here may be generalized to incorporate non-stationarity in the extreme value distributions by assuming that all three parameters are functions of time, t.

The fitting of the above probability distribution can be accomplished by using a variety of methods, including Method of Moments (MoM), Method of Maximum Likelihood (ML), and the Method of L-Moments. For this analysis, the ML was used. It should be noted that, in case of data which shows no non-stationarity due to trend, the three parameters in Eq. (1) are assumed to be constant. Even in the case of non-stationary data, for example a trend of increasing sea levels, the data can be pre-processed to remove trend and then the stationary methods assuming constant parameters may be used.

Peaks Over Threshold (POT)

In case of short data records, there is a desire to use multiple peaks within a given block to improve parameter estimation (**Figure C-4b**). In such an application, known commonly as the Peaks Over Threshold (POT) method, a threshold value (u) is selected to identify the peaks and that value becomes one of the parameters in the methods. It has been shown (**Coles, 2001**), POT values follow a Generalized Pareto Distribution (GP) of the form:

$$\mathrm{H(y)} = 1 - \left(1 + \frac{\varepsilon y}{\tilde{\sigma}}\right)^{\varepsilon} ;\ y = X - u \text{ conditioned on } X > u \qquad (2)$$

The parameters of the GP distributions are the shape parameter, ε and the scale parameter, $\tilde{\sigma}$. **Coles (2001)** provides methods for determining an appropriated threshold value, u. The parameters may be estimated using a variety of methods but for this exercise, the ML method was used.

Mixed Distributions

South Florida is impacted by a variety of storm types and one aspect that is rare but may influence the estimation of return levels is the occurrence of tropical storms/hurricanes. In most locations, the data comes from at least two distributions, one from tropical storms and the remainder due to occasional wind driven or rainfall events. In such cases, the use of a mixed distribution is warranted. It is of the form:

$$F(x) = \alpha F_1(x) + (1 - \alpha) F_2(x) \qquad (3)$$

Where F_1 and F_2 are the distribution functions corresponding to each storm type and α is the mixture parameter. One disadvantage of this approach is the large number of parameters required to fit the mixture and typically the accuracy depends on the number of data points affected by the peaks generated by rate events. In this study, the Weibull Distribution was used for the two types of events.

Monte Carlo Joint Probability (MCJP) Method

This is a complex method which uses a new empirical simulation technique (EST) to predict extreme sea levels and it is particularly suited for short records (**Goring et al., 2011**). In this method, the complete hourly sea-level record is decomposed into its constituent components (tides, storm surge, mean sea level etc.) and then they are recombined randomly to form a long time series of possible realization of future sea levels. As a first step, the sea level record is de-tided to separate the variation due to tides. This was accomplished by using a de-tiding tool called UTide (**Codiga, 2011**). Then the non-tidal residuals are further decomposed into various frequency bands using wavelet analysis. The details of the methodology are provided in the paper by **Goring et al. (2011**). The analysis required for applying this method was accomplished by a MATLAB code provided by Scott Stephens (second author of the paper) and the de-tiding code UTide.

A total of 12 different methodologies were derived from the frequency analysis tools described above. As explained, some of them are very similar (i.e. introducing non-stationarity in one of the parameters) and some very different (i.e. Block Maxima compared to Mixed Distributions). Joint examination of results show that the variability in the estimated RLs goes from close to 1.0 feet for a 2-year return period to close to 3.0 feet for the 100-year return period. Given the observed variability in the results and the lack of criteria at this point to select a preferred methodology, it was decided to use the average of all the methods as the final peak stages. Typical use of the extreme value modeling for projecting future sea levels selects a single model as opposed to multiple methods used here. It should be noted that the scope and schedule did not permit a more detailed analysis to conduct a formal model selection and the proposed approach was deemed appropriate for the screening level analysis of the pilot LOS project. Further development of the approach will be considered in future LOS projects and the final methodology will be documented in a Technical Report that would be prepared following such project. This is scheduled to be completed in 2016.

Final peak stages derived from the frequency analysis for the three different locations are presented in **Table C-1** and depicted in **Figure C-5** as a frequency plot (the X-axis is the return period for a standard normal deviate). Detailed peak stage values derived from the 11 methods for each of the six return periods considered are depicted in **Table C-2**, **Table C-3** and **Table C-4** for coastal structures S25B, S22 and G93, respectively.

Table C-1. Tailwater Stage Return Levels (feet NGVD) for different Return Periods (RP) at coastal locations in the C-4 basin.

Structure	RP (years)					
	2	5	10	25	50	100
S-25B	3.68	4.10	4.50	5.06	5.56	6.15
S-22	3.54	3.85	4.15	4.61	4.98	5.38
G-93	3.58	4.04	4.48	5.11	5.65	6.26

Table C-2. Intermediate Tailwater Stage Return Levels (feet NGVD) at coastal structure S25B

Appendix C – Boundary Conditions at Tidal Structures

Method	RP (years)					
	2	5	10	25	50	100
Nonstationary,Cov=Year	3.41	3.91	4.43	5.41	6.49	7.99
Nostationary, Cov = Year + Q	3.46	3.90	4.31	5.04	5.77	6.71
Block maxima, monthly, cyclic	4.31	4.61	4.80	5.06	5.24	5.43
POT daily	3.49	3.93	4.28	4.78	5.21	5.70
Adding Seasonality	4.32	4.69	4.93	5.22	5.44	5.66
POT Hourly	3.46	3.88	4.21	4.69	5.10	5.56
POT daily max dtrended	3.41	3.82	4.21	4.87	5.53	6.36
Nonstionary, Daily	3.43	3.85	4.18	4.68	5.12	5.62
Skew Surge	4.04	4.48	4.87	5.48	6.06	6.75
Mixed Weibull	3.42	3.76	4.55	4.98	5.2	5.36
MJCP	3.68	4.27	4.77	5.46	5.99	6.49
Nonstationary,Cov=Year	3.41	3.91	4.43	5.41	6.49	7.99

Table C-3. Intermediate Tailwater Stage Return Levels (feet NGVD) at coastal structure S22

Method	RP (years)					
	2	5	10	25	50	100
Nonstationary,Cov=Year	3.27	3.65	4.01	4.62	5.23	6.01
Nostationary, Cov = Year + Q	3.32	3.64	3.93	4.38	4.79	5.30
Block maxima, monthly, cyclic	4.00	4.30	4.49	4.73	4.91	5.08
POT daily	3.36	3.77	4.06	4.45	4.76	5.09
Adding Seasonality	4.35	4.65	4.83	5.04	5.17	5.30
POT Hourly	3.32	3.70	4.00	4.44	4.80	5.21
POT daily max dtrended	3.29	3.68	3.98	4.40	4.76	5.16
Nonstionary, Daily	3.36	3.76	4.05	4.44	4.75	5.08
Skew Surge	3.90	4.28	4.58	5.01	5.37	5.78
Mixed Weibull	3.35	3.39	3.80	4.89	5.55	6.12
MJCP	3.38	3.73	4.06	4.54	4.92	5.32
Nonstationary,Cov=Year	3.27	3.65	4.01	4.62	5.23	6.01

Table C-4. Intermediate Tailwater Stage Return Levels (feet NGVD) at coastal structure G93

Method	RP (years)					
	2	5	10	25	50	100
Nonstationary,Cov=Year	3.27	3.78	4.29	5.24	6.26	7.64
Nostationary, Cov = Year + Q	3.36	3.82	4.21	4.82	5.4	6.09
Block maxima, monthly, cyclic	4.11	4.48	4.7	4.98	5.17	5.36
POT daily	3.42	3.94	4.34	4.94	5.46	6.04
Adding Seasonality	4.26	4.71	5	5.38	5.66	5.94
POT Hourly	3.35	3.86	4.29	4.96	5.58	6.31
POT daily max dtrended	3.34	3.84	4.26	4.9	5.47	6.13
Nonstionary, Daily	3.35	3.85	4.29	4.98	5.63	6.41
Skew Surge	3.91	4.41	4.85	5.56	6.24	7.07
Mixed Weibull	3.28	3.61	4.53	5.41	5.77	6.04
MJCP	3.69	4.16	4.52	5.04	5.46	5.87
Nonstationary,Cov=Year	3.27	3.78	4.29	5.24	6.26	7.64

DETERMINATION OF SEA LEVEL CHANGE

The methodology adopted to evaluate sea-level change (SLC) in the determination of stage hydrographs at coastal structures under the LOS project is taken from ER 1100-2-8162 (**USACE, 2013**). The USACE Engineering Regulation incorporates three sea-level change projections; namely, Low (continuation of historic SLC), Intermediate (Modified NRC Curve I), and High (Modified NRC Curve III). The NRC curves were originally developed by the National Research Council and were modified based on the current National Tide Datum Epoch of 1983-2001 with a start year of 1992 (i.e., mid-point of the current national Tide epoch) and the 2007 Intergovernmental Panel on Climate Change (IPCC) projections (**IPCC, 2007**) on global sea level change trend. The modified NRC curves for global sea level change are described by the following equation:

$$E(t) = 0.0017t + bt^2 \qquad (4)$$

Where *t* represents years, starting in 1992, *E(t)* is the eustatic sea level change in meters, as a function of *t*, and *b* is a variable whose value is 0.0000271, 0.00007 and 0.000113 for modified NRC Curve I

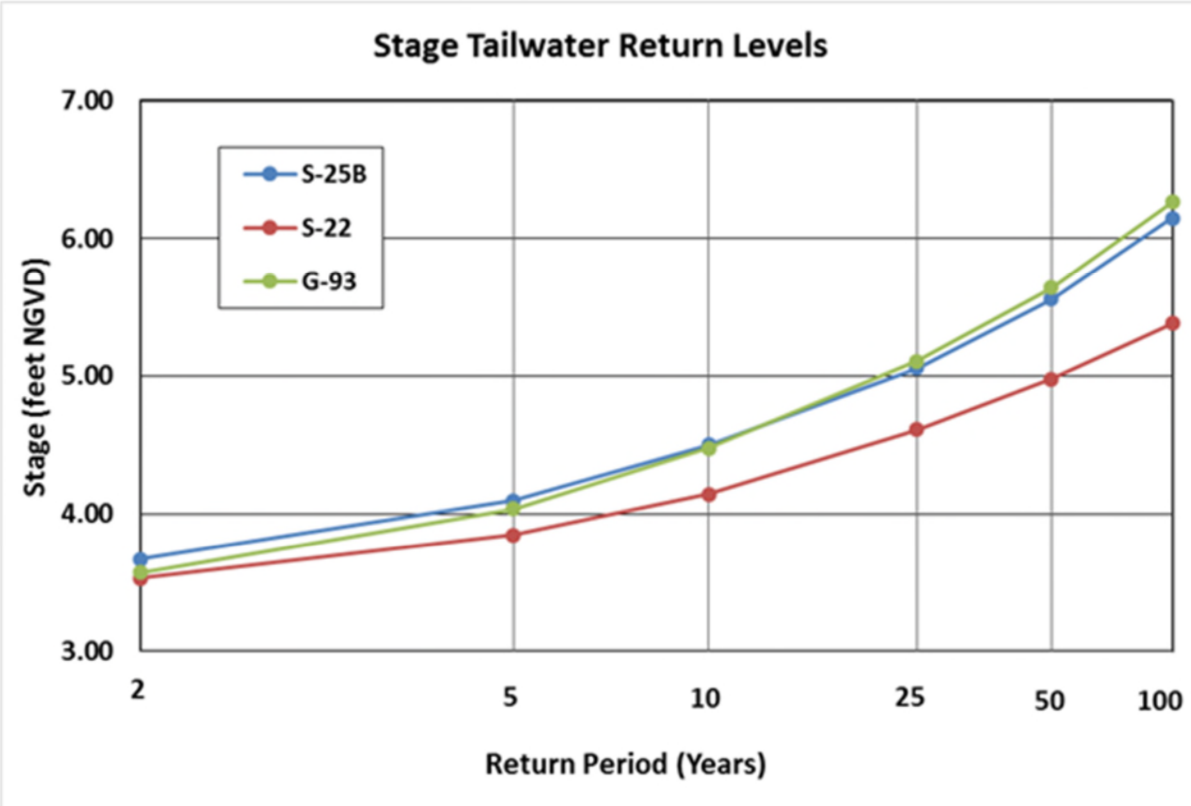

Figure C-5. Tailwater Stage Return Levels (feet NGVD) for different Return Periods (RP) at coastal locations in the C-4 basin.

(USACE Intermediate), modified NRC Curve II (not used by USACE) and modified NRC curve III (USACE High), respectively.

For low (historic) USACE sea level change curve, Eq. (4) with a *b* value of zero can be used. The resulting global mean sea level change with respect to the starting year 1992 for the three USACE sea level change scenarios are depicted in **Figure C-6**. The global mean sea level change by the year 2100 is projected to be 0.53, 1.55 and 4.79 ft. for low, intermediate and high scenarios, respectively.

According to USACE Eq. (4) does not include the local Vertical Land Movement (VLM) which is about -0.5 mm/yr for the region. The rate employed to develop the local relative sea level change is a combination of the widely accepted eustatic rate of 1.7 mm/year (linear coefficient in Eq. (4)) plus the local Vertical Land Movement. Thus, it is a common practice to substitute the eustatic rate in Eq. (4) by either the published or regionally corrected rates that are provided by NOAA. Therefore, the low, intermediate and high scenarios for NOAA tide gauges can be computed using Eq. (5) with the appropriate annual sea level rate for the site. An on-line sea level change calculator for NOAA tide gauges, provided by **USACE (2013)**, is located at http://www.corpsclimate.us/ccaceslcurves.cfm.

$$E(t) = Mt + bt^2 \qquad (5)$$

Where ***M*** is published or the regionally corrected rate.

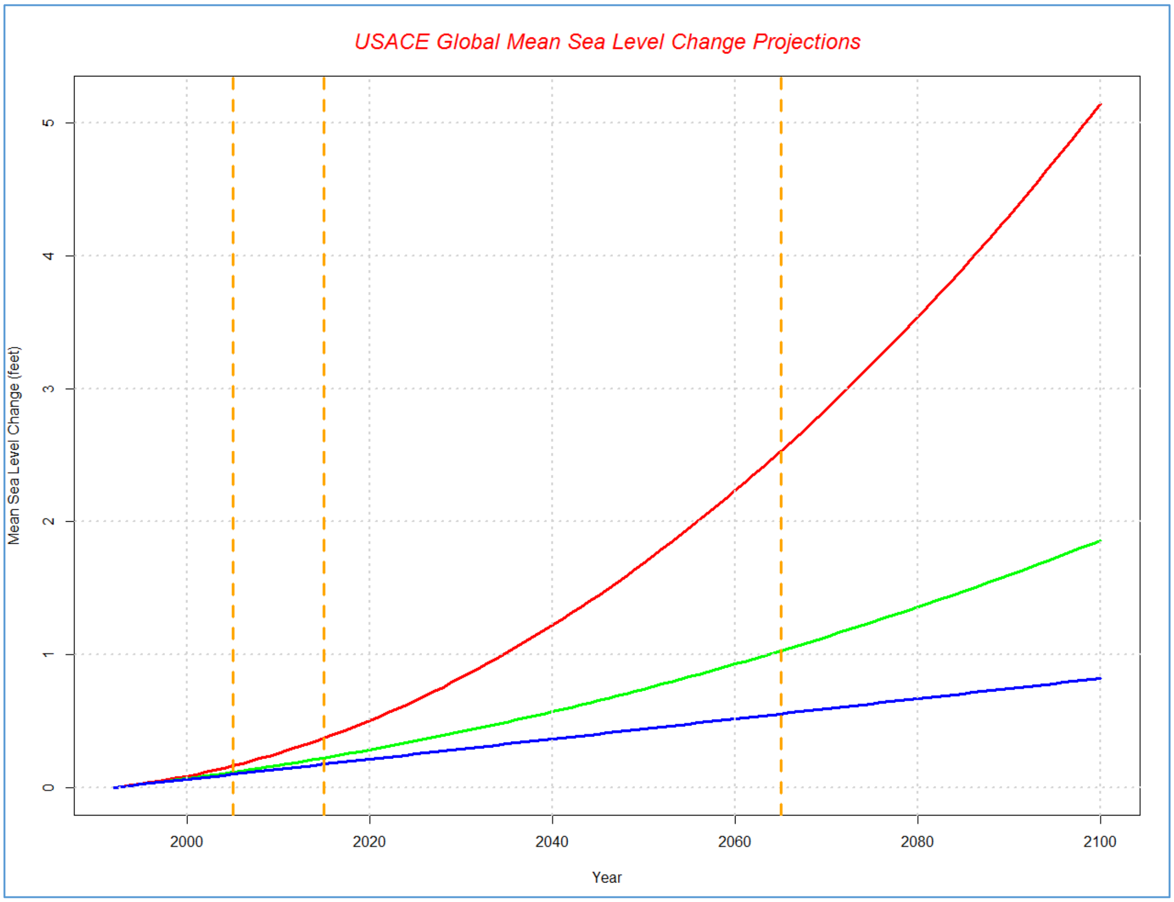

Figure C-6. USACE Global Mean Sea Level Change Projections (Historic change in blue; Intermediate in green and High in red. Orange lines represent years 2005, 2015 and 2065).

Sea Level Change Projections at SFWMD Coastal Structures

As part of tailwater boundary condition development at coastal structures under the LOS Project, it is required to determine sea level change projections in the future for the local structures S-25B, S-22 and G-93. (**Figure C-7**). The sea level change projections are sought for the years 2015 (existing condition) and 2065 (future condition).

The sea level change projections at these coastal structures require long term water level data at tide gauges. The USACE sea level change projection tool makes use of the NOAA tide gauges. Water level data and published sea level change rate at the NOAA tide gauge are supplied by the NOAA Center for Operational Oceanographic Products and Services (CO-OPS) that has an extensive QA/QC program for the collection of water level. For this reason, the sea level change at coastal structure will be determined using NOAA tide gauge data.

The NOAA Tide gauges located in Southern Florida region are depicted in **Figure C-7** and their detailed information is listed in **Table C-2**. Based on proximity to the coastal structures, the Miami Beach and Vaca Key Tide gauges are closer to the coastal structures under consideration; however, the Miami Beach Tide gauge record ends in 1981 and does not include recent sea level change trends. The Vaca Key Tide gauge record starts in 1971 and has a shorter record. On the other hand, the Key West Tide gauge record covers 102 years and is an active station. Thus, the Key West Tide gauge is selected to determine the sea level change projections at coastal structures for the LOS project.

Table C-2. Published Mean Sea Level Trends at NOAA Tide Gauges (2013)

NOAA Tidal Gauge	Gauge ID	Period	Ann mean SL Trend (mm/yr)	95% Conf. Interval (± mm)
Key West	8724580	1913-2014	2.31	0.15
Cava Key	8723930	1971-2014	3.18	0.48
Miami beach	8723170	1931-1981	2.39	0.43
Naples	8725110	1965-2014	2.4	0.48

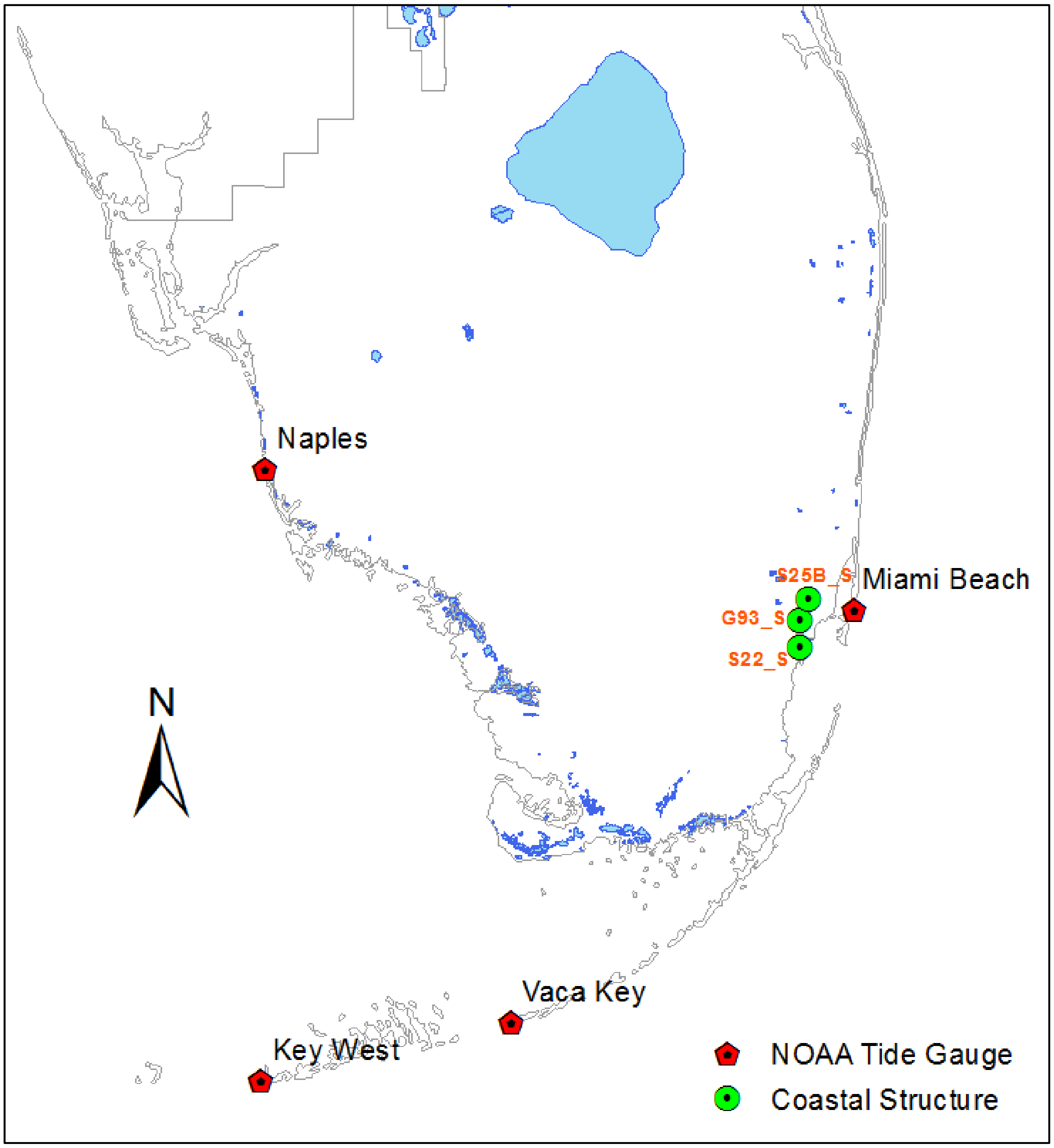

Figure C-7. Location NOAA Tide Gauges with SFWMD Coastal Structures for LOS project

Using NOAA published rate of 2.31 mm/year for Key West Tide gauge, sea level change were calculated employing Eq. (5). The computed sea level change with respect to the year 1992 (mid-point of the current National Tidal Datum Epoch) until the year 2100 is depicted in **Table C-3** while sea level change with respect to 2005, the mid-point of the project epoch (1996-2014), is depicted in **Table C-4**.

Table C-3. Sea Level Change (feet) at Key West Tide Gauge with respect to 1992

Year	Low	Intermediate	High
1992	0.00	0.00	0.00
1995	0.02	0.02	0.03
2000	0.06	0.07	0.08
2005	0.10	0.11	0.16
2010	0.14	0.17	0.26
2015	0.17	0.22	0.37
2020	0.21	0.28	0.50
2025	0.25	0.35	0.65
2030	0.29	0.42	0.82
2035	0.33	0.49	1.01
2040	0.36	0.57	1.22
2045	0.40	0.65	1.44
2050	0.44	0.74	1.69
2055	0.48	0.83	1.95
2060	0.52	0.93	2.23
2065	0.55	1.03	2.53
2070	0.59	1.13	2.85
2075	0.63	1.24	3.18
2080	0.67	1.36	3.54
2085	0.70	1.47	3.91
2090	0.74	1.60	4.30
2095	0.78	1.72	4.71
2100	0.82	1.86	5.14

Table C-4. Sea Level Change (feet) at Key West Tide Gauge with respect to 2005

Year	Low	Intermediate	High
2005	0.00	0.00	0.00
2010	0.04	0.05	0.10
2015	0.08	0.11	0.21
2020	0.11	0.17	0.34
2025	0.15	0.23	0.49
2030	0.19	0.30	0.66
2035	0.23	0.38	0.85
2040	0.27	0.46	1.06
2045	0.30	0.54	1.28
2050	0.34	0.63	1.53
2055	0.38	0.72	1.79
2060	0.42	0.81	2.07
2065	0.45	0.91	2.37
2070	0.49	1.02	2.69
2075	0.53	1.13	3.02
2080	0.57	1.24	3.38
2085	0.61	1.36	3.75
2090	0.64	1.48	4.14
2095	0.68	1.61	4.55
2100	0.72	1.74	4.98

Back Projecting Sea Level Change at Coastal Structures for 1963

As part of the LOS project, it was required to determine the tail water boundary condition at the time of construction of the coastal structures (circa 1963). Thus, the sea level change between 1963 and 2005 (i.e., the mid-point of the project tidal datum epoch) was calculated based on the historical water level data at Key West Tide gauge. First, two 19-year periods with mid-points at 1963 and 2005 were identified (i.e., 1954 – 1972 and 1996 – 2014). Then mean monthly stages for the two periods were determined. Finally, the difference between the two mean monthly stages of the two periods was calculated to arrive at the mean sea level change between 1963 and 2005, that is, 0.3545 ft. **Table C-5** shows the sea level change between 1963 and 2005 and **Figure C-10** depicts mean monthly water level at Key West Tide Gauge for the period of record with mean sea levels for the two epochs around 1963 and 2005.

Table C-5. Mean Sea Level Change at Key West Tide Gauge between 1963 and 2005

Start Year	Mid Year	End Year	Epoch	MSL ft NAVD 88	Difference (feet)
1996	2005	2014	Epoch 1	-0.7701894	0.3545
1954	1963	1972	Epoch 2	-1.124671	

Sea Level Change Calculation Summary

The sea level change at Key West Tide Gauge for three scenarios with reference to the mean sea level at the year 2005 (the mid-point of the project epoch) with back projection for the year 1963 are depicted in **Figure C-9** and listed in **Table C-6**.

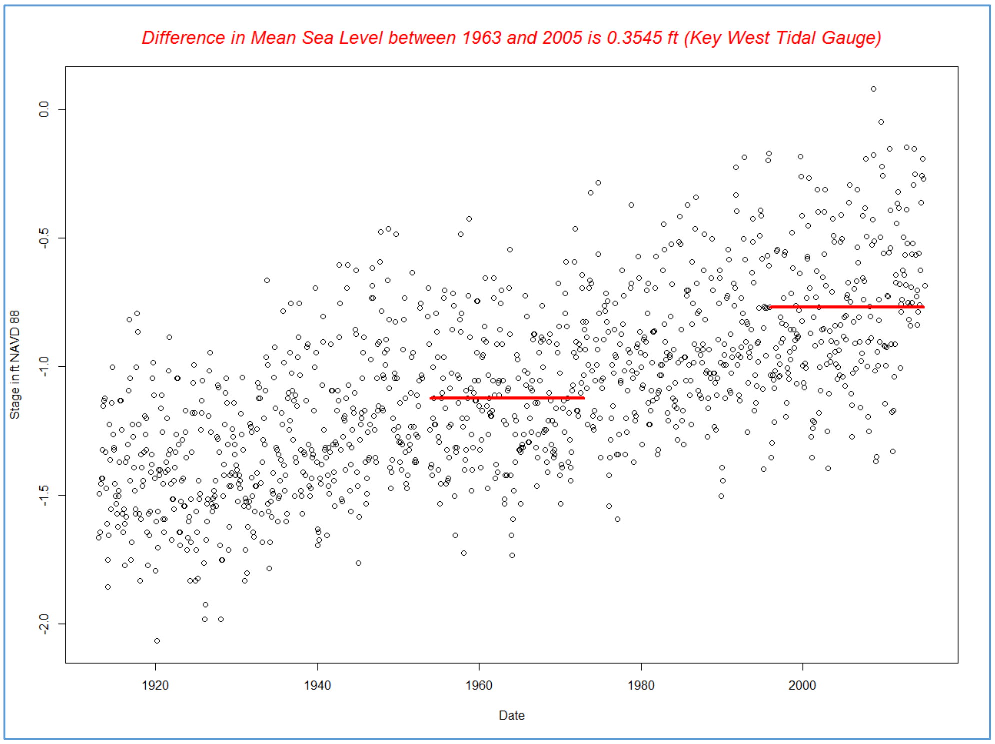

Figure C-8. Mean Sea Level Change between 1963 and 2005 at Key West Tide Gauge

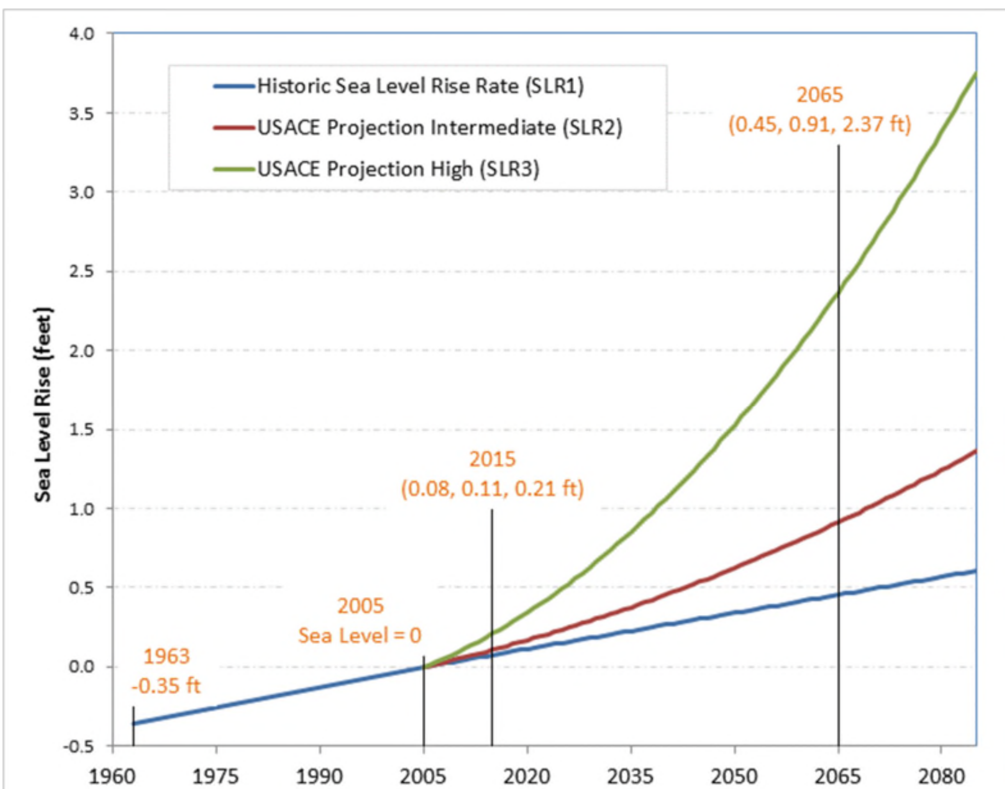

Figure C-9 Sea Level Change Projections at Key West Tide Gauge

Table C-6. Relative Sea Level Change at Key West Tide Gauge (feet) with respect to 2005

Year	Historic	Intermediate	High	Remarks
1963	-0.35	-	-	Construction of Coastal Structures (S25B_S, S22_S and G93_S)
2005	0	0	0	Mid-point of the Epoch used to calculate mean sea level (2014-1996)
2015	0.08	0.11	0.21	Existing Condition
2065	0.45	0.91	2.37	Future Condition (50 Year after 2015)

PREPARATION OF BOUNDARY CONDITIONS AT TIDAL STRUCTURES

The simulation of storm events require the definition of stage boundary conditions at tidal structures, which are the terminal water control locations in a canal through which the canal discharges into the ocean. Stage boundary conditions are required for all the tidal structures which are part of the basin being modeled. In addition to being location dependent, stage boundary conditions are indexed by return period, projected Sea Level Rise (SLR) and planning horizon.

For a particular structure, the input data and information required for this process are the time series of historical stage data at the tail-water (ocean) side of the structure, the corresponding return levels for 2, 5, 10, 25, 50 and 100 years, projected SLC and the planning horizon. It is recommended that historical stage data be provided as an instantaneous 15-minute time series for the available period of record. Return levels are derived from frequency analysis of the tail-water stage data and SLC projections are obtained from Federal or other government organizations recommendations. Two planning horizons were included in this analysis, existing and future.

As stated previously, extreme value analysis of stage values used historical data, adjusted to correspond to the 1996-2014 epoch and the derived return levels are placed at middle of the epoch, 2005. Two planning horizons are considered, existing and future. Existing is placed at 2015 and future is 2065. Results for existing conditions are obtained by adding the SLC change 2005-2015 to 2005 results. In a similar way, results for future conditions are derived by adding the 2005-2065 SLC to 2005 results.

The first step in this process is the selection of a base storm stage hydrograph from the available period of record. This selection is made by inspecting the stage time series and taking into account the following recommendations:

- The base storm stage hydrograph must come from a period when the king tide was dominating (typically in October due to seasonal highs).
- The base storm stage hydrograph needs to exhibit three well differentiated periods, regular tide, storm surge and regular tide, in that order. The initial regular tide period needs to be long enough to assure that transient start-up effects are filtered out. In a similar fashion the ending regular tide cycle needs to allow the structure to fully transition back to normal operating regimes. Each of the three periods needs to contain several within the day tidal cycles.
- The longer the storm surge period the better.

It is recommended to extract and compare several storms from the time series of historical stages. It is good practice to select the storm with the highest stage and the longest storm surge period, within reasonable limits. This comparison is very easily performed by centering all the hydrographs around the peak stage at the same point in time. **Figure C-10** provides an example for S-25B. For this particular case, the storms in October 1999 and October 2000 are very similar. The October 1999 storm was selected as the base stage storm. For basins where several coastal locations are being modeled it might be advantageous to examine and compare historical stage hydrographs at all the locations before the base storm is selected.

In the second step for the preparation of stage boundary conditions, the storm base hydrograph needs to be re-scaled such that the peak stage in the boundary condition hydrograph agrees with the return level for the selected return period. A peak factor is derived as the quotient between the return level for the

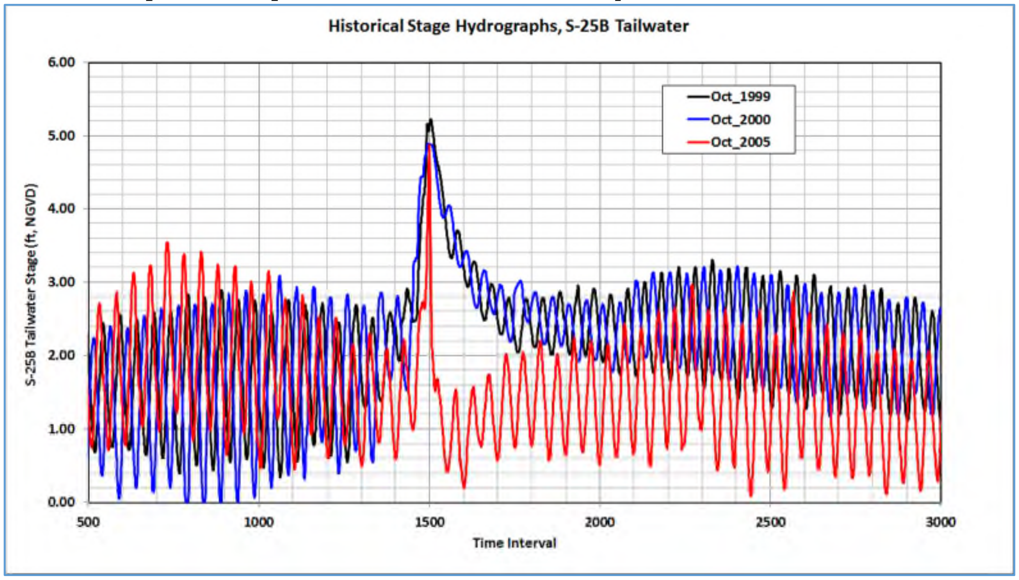

Figure C-10. Historical Stage Hydrographs for S-25B Tailwater

specified frequency and the observed peak stage in the hydrograph. The idea is to apply the peak factor to the peak stage in the observed hydrograph, but with a smooth transition starting from the regular tidal cycle and moving into the surge, and then transitioning back again to the second regular tidal cycle. Initial and ending times in the historical stage hydrograph are user selected. For each time step in between these times, a new re-scale factor is computed by linearly interpolating from 1.0 at the initial time to the peak factor at the peak time, and from the peak factor at the peak time to 1.0 at the ending time. The peak factor can take values greater than and less than 1.0. When the peak factor is less than 1.0, care must be exercised such that the storm surge phase does not become similar to the regular tidal cycles at the beginning and end. **Figure C-11** provides an example of the variation of the rescale factor to produce a 100-yr storm surge at

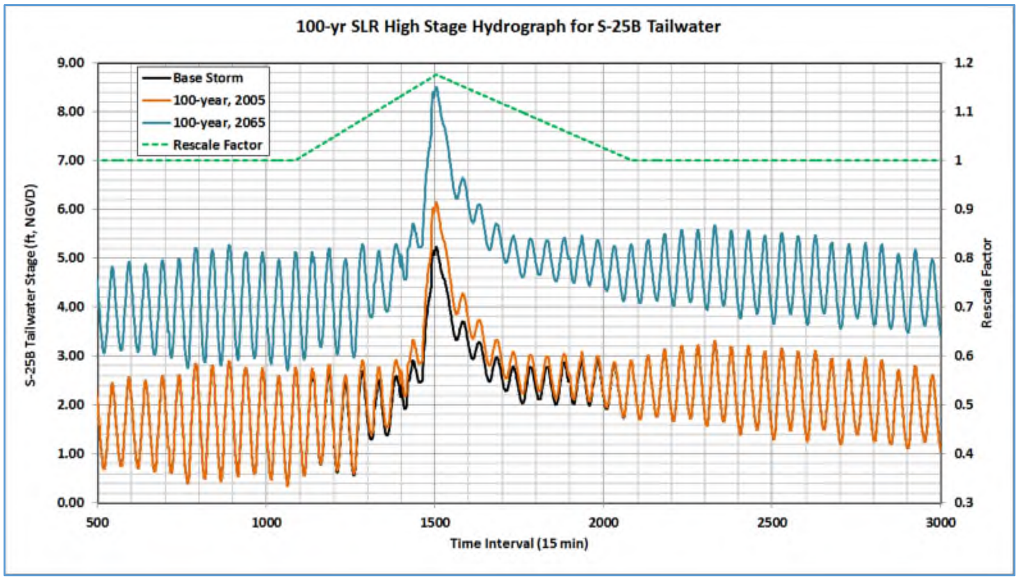

Figure C-11. Computation of the 100-yr 2065 SLR High Stage Hydrograph for S-25B Tailwater

S-25B. In the final step of the preparation of stage boundary conditions, the SLC projection in the form of a change in stage is selected. The previously obtained hydrograph for the specified frequency is shifted by this amount and the final result is the desired stage hydrograph boundary condition. **Figure C-11** shows the final 100-yr 2065 stage hydrograph for a “High” SLC projection circa 2060 at S-25B.

In the modeling of the C-4 Canal basin, tidal boundary conditions are needed for three different locations: S-25B, S-22 and G-93. Each location requires 6 return frequencies (periods), 3 sea level scenarios and two planning horizons, for a total of 36 hydrographs for each location. Return levels for the three locations for the required return periods are provided in **Table C-1**, while increase due to SLC are listed in **Table C-6.**

An additional special case requires the preparation of stage boundary conditions around 1963, at the time when the C&SF project was under design and construction. This special case will include the same return periods, but no SLR scenarios since the estimated historical change between 1963 and 2005 will be used. Following the methodology explained before, the same period October and November 1999 was used to extract the base storm at each location. **Figures C-12 to C-20** present the prepared boundary condition stage hydrographs for the tailwater (ocean side) stage at the three required locations S-25B, S-22 and G-93.

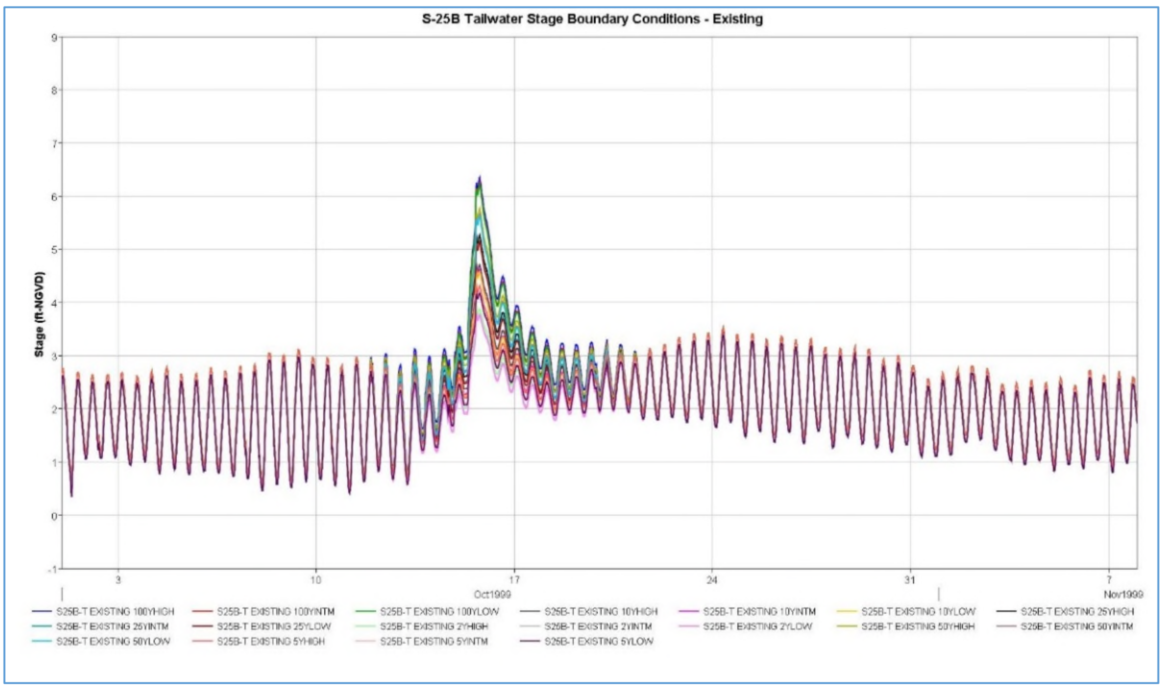

Figure C-12. Stage hydrographs at the tailwater side of S-25B for the existing planning horizon.

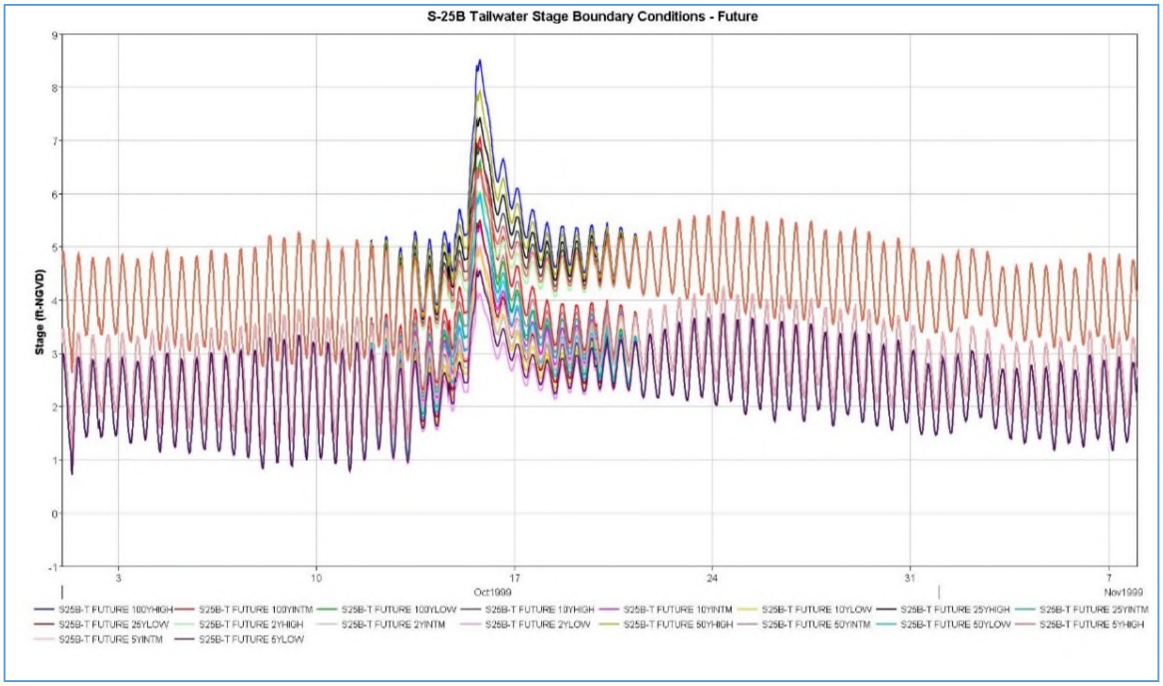

Figure C-13. Stage hydrographs at the tailwater side S-25B for the future planning horizon.

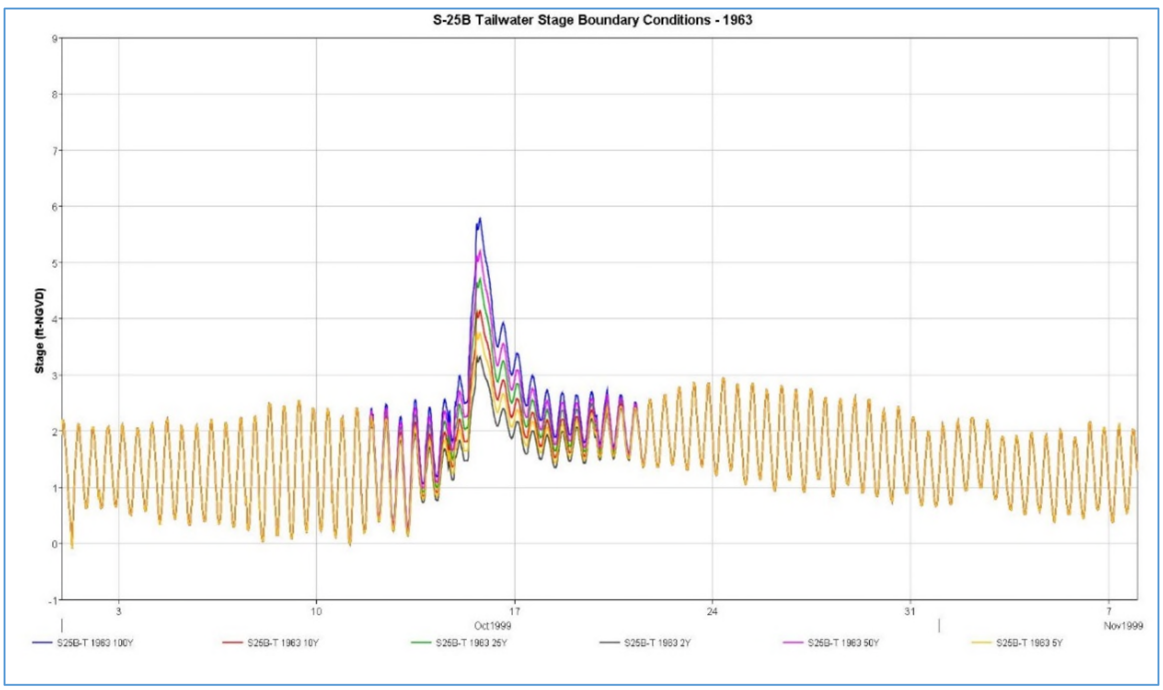

Figure C-14. Stage hydrographs at the tailwater side S-25B circa 1963.

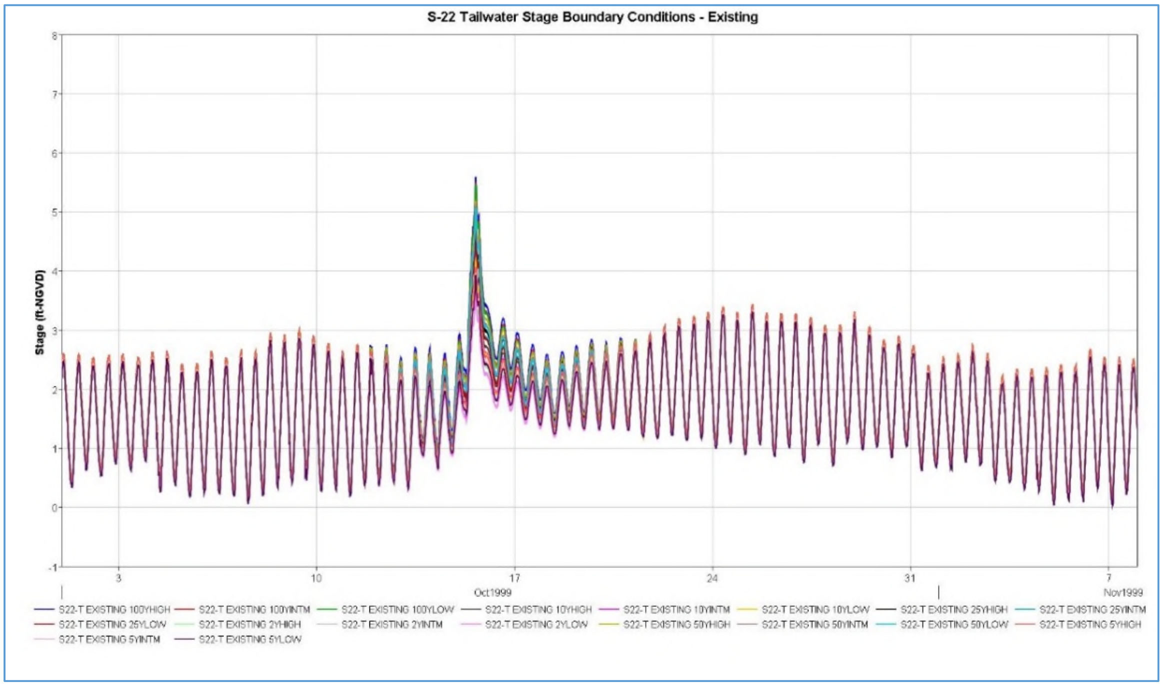

Figure C-15. Stage hydrographs at S-22 for the existing planning horizon

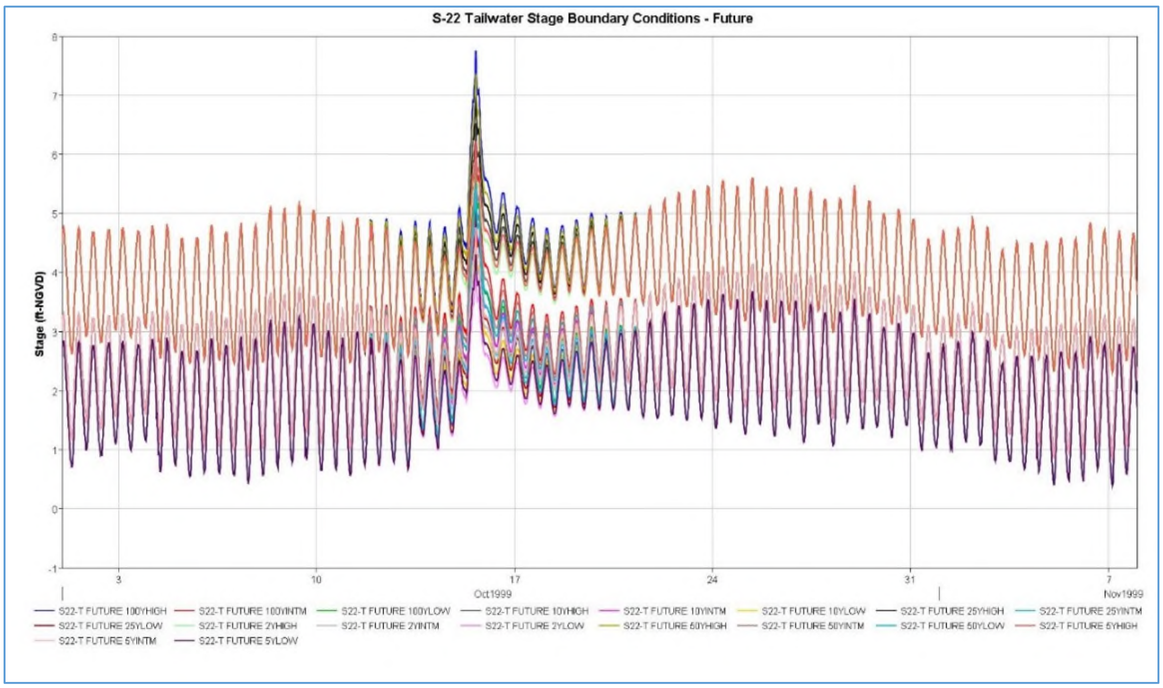

Figure C-16. Stage hydrographs at the tailwater side S-22 for the future planning horizon

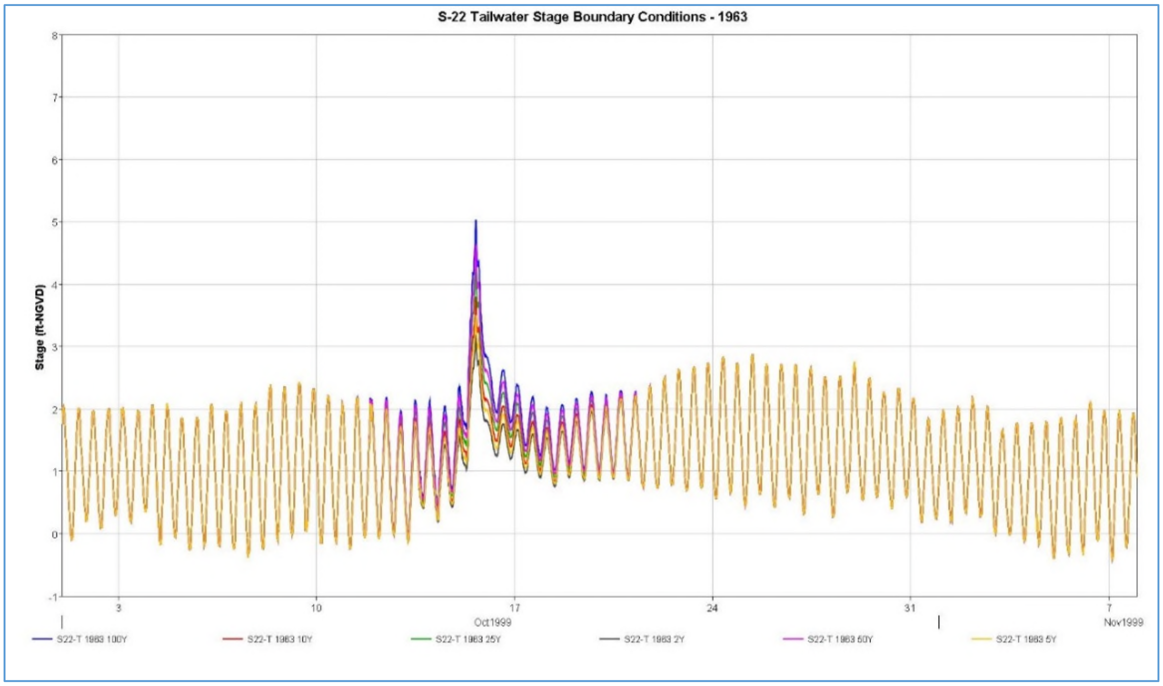

Figure C-17. Stage hydrographs at the tailwater side S-22 circa 1963.

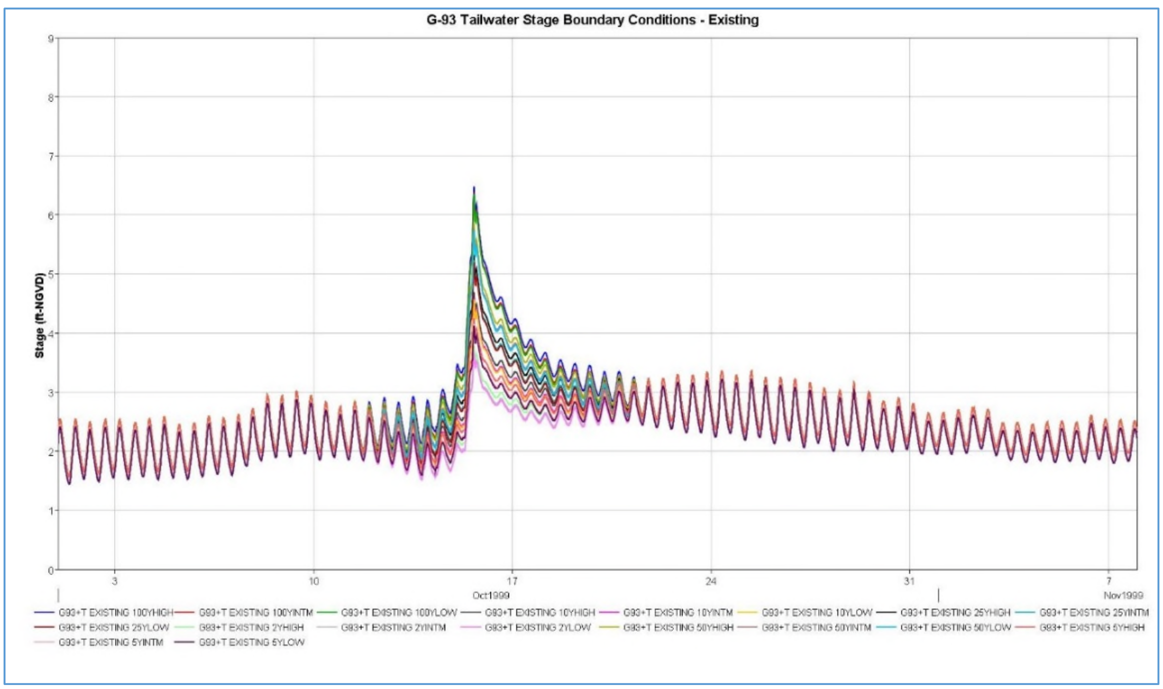

Figure C-18. Stage hydrographs at the tailwater side G-93 for the existing planning horizon

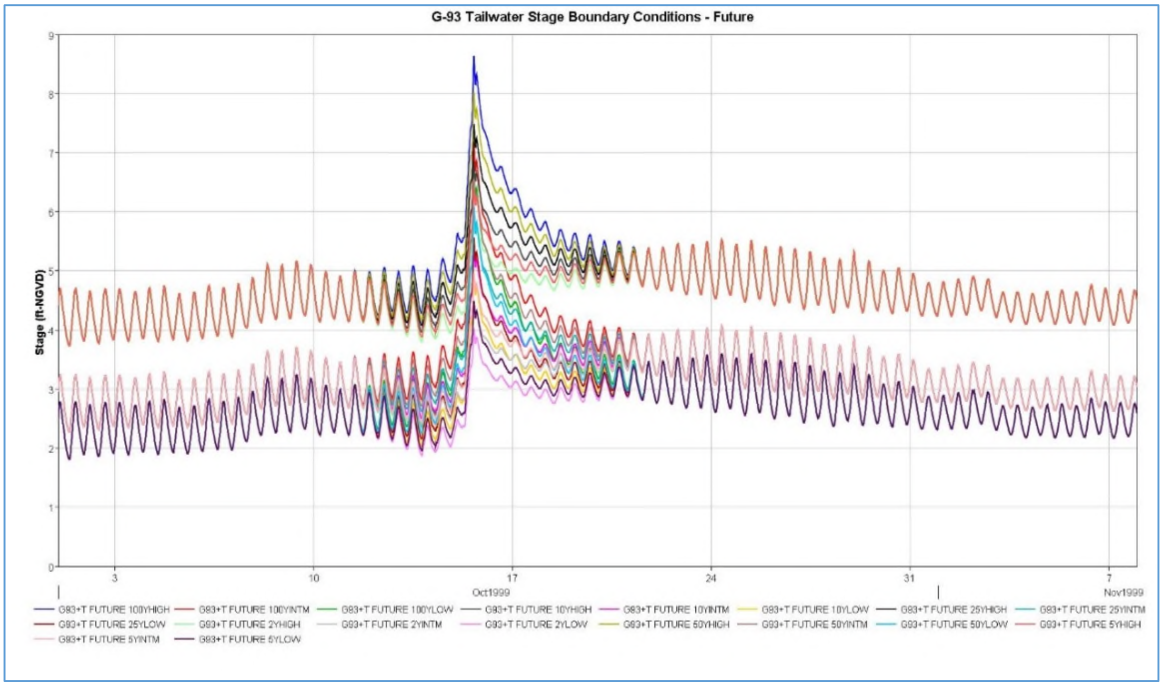

Figure C-19. Stage hydrographs at the tailwater side G-93 for the future planning horizon

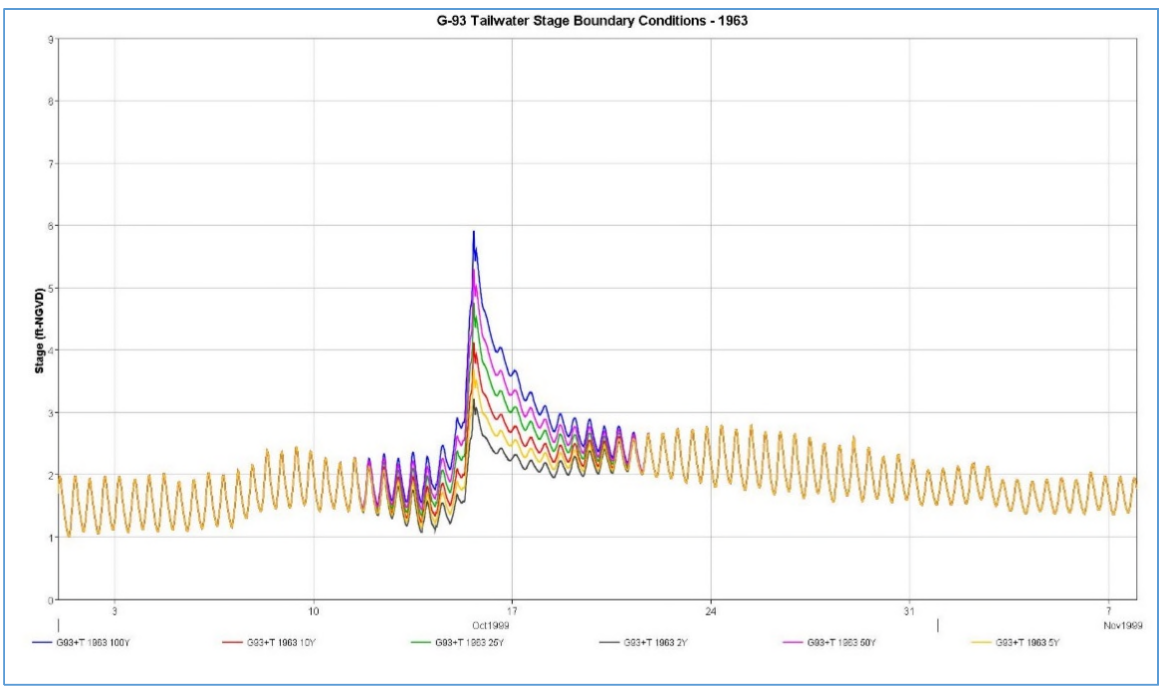

Figure C-20. Stage hydrographs at the tailwater side S-22 circa 1963.

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ATTACHMENT C-1

Data Storage System for Boundary Condition Time Series

As part of tail water boundary condition generation for the hydraulic model used for the Level of Service (LOS) project, a total of 108 stage hydrographs corresponding to three coastal structures (S-25B, S-22 and G-93), three sea level change scenarios (Low, Intermediate and High), six return levels (2-Year, 5-Year, 10-Year, 25-Year, 50-Year, 100-Year), one base storm event (IRENE), and two planning scenarios Existing (2015) and Future (2065) were completed. The Design Condition (1963) brings 6 additional hydrographs per location. For storing these stage hydrographs, the United States Army Corps of Engineers Data Storage System (DSS) (USACE, 2009) is adopted. DSS files corresponding to each of the three coastal structures were created. **Table C-1-1** depicts description of the DSS file parts A through F and

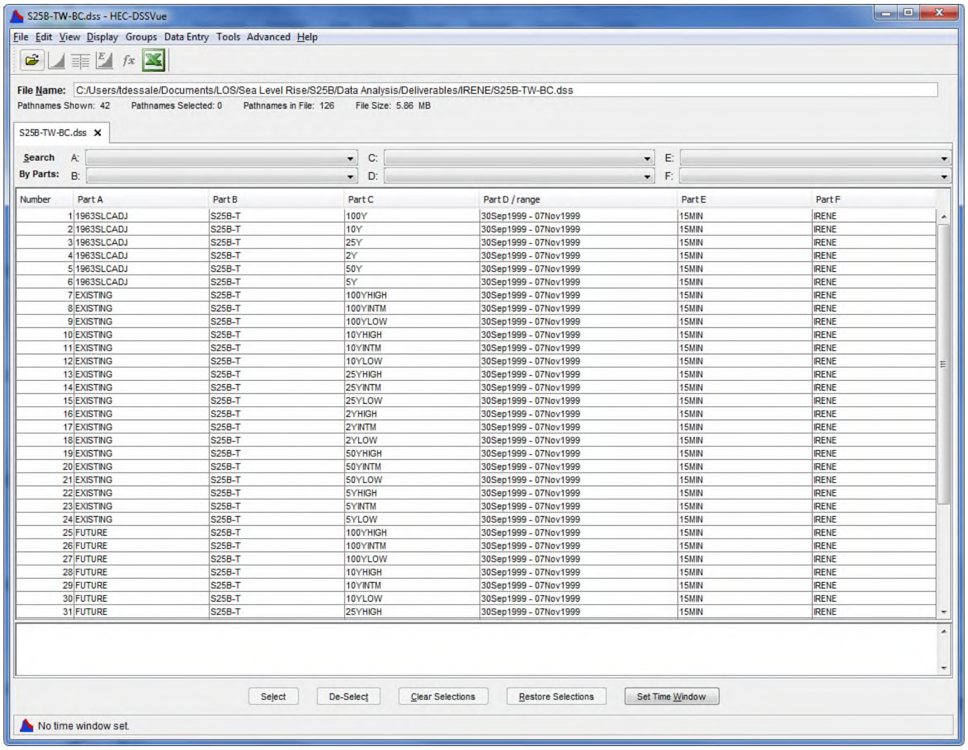
Table C-1-2 lists file names and their sizes created to store stage hydrographs for LOS project. **Figures C-1-1** through **FigureC-1-3** depict condensed DSS catalogs for S-25B, S-22 and G-93, respectively.

Table C-1-1. Description of DSS Path Parts

DSS Path Part	Content	Entries
Part A	Project Condition	Existing; Future; 1963SLADJ
Part B	DCVP Station ID	S25B-T;S22-T; G93+T
Part C	Return Level & Sea Level Change Scenario	100YHIGH; 100YINTM; 100YLOW; 50YHIGH; 50YINTM; 50YLOW; 25YHIGH; 25YINTM; 25YLOW; 10YHIGH; 10YINTM; 10YLOW; 5YHIGH; 5YINTM; 5YLOW; 2YHIGH; 2YINTM; 2YLOW;100Y; 50Y; 25Y; 10Y; 5Y; 2Y
Part D	Period of Record Range	30sep1999-07Nov1999
Part E	Interval	15MIN
Part F	Storm	IRENE

Table C-1-2. List of DSS Files for Tailwater Hydrographs at Coastal Structures

Coastal Structure	File Name	Size
S25B_S	S25B-TW-BC.dss	5,441 KB
S22_S	S22-TW-BC.dss	5,699 KB
G93_S	G93-TW-BC.dss	14,883 KB



Number	Part A	Part B	Part C	Part D / range	Part E	Part F
1	1963SLCADJ	S25B-T	100Y	30Sep1999 - 07Nov1999	15MIN	IRENE
2	1963SLCADJ	S25B-T	10Y	30Sep1999 - 07Nov1999	15MIN	IRENE
3	1963SLCADJ	S25B-T	25Y	30Sep1999 - 07Nov1999	15MIN	IRENE
4	1963SLCADJ	S25B-T	2Y	30Sep1999 - 07Nov1999	15MIN	IRENE
5	1963SLCADJ	S25B-T	50Y	30Sep1999 - 07Nov1999	15MIN	IRENE
6	1963SLCADJ	S25B-T	5Y	30Sep1999 - 07Nov1999	15MIN	IRENE
7	EXISTING	S25B-T	100YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
8	EXISTING	S25B-T	100YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
9	EXISTING	S25B-T	100YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
10	EXISTING	S25B-T	10YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
11	EXISTING	S25B-T	10YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
12	EXISTING	S25B-T	10YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
13	EXISTING	S25B-T	25YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
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15	EXISTING	S25B-T	25YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
16	EXISTING	S25B-T	2YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
17	EXISTING	S25B-T	2YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
18	EXISTING	S25B-T	2YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
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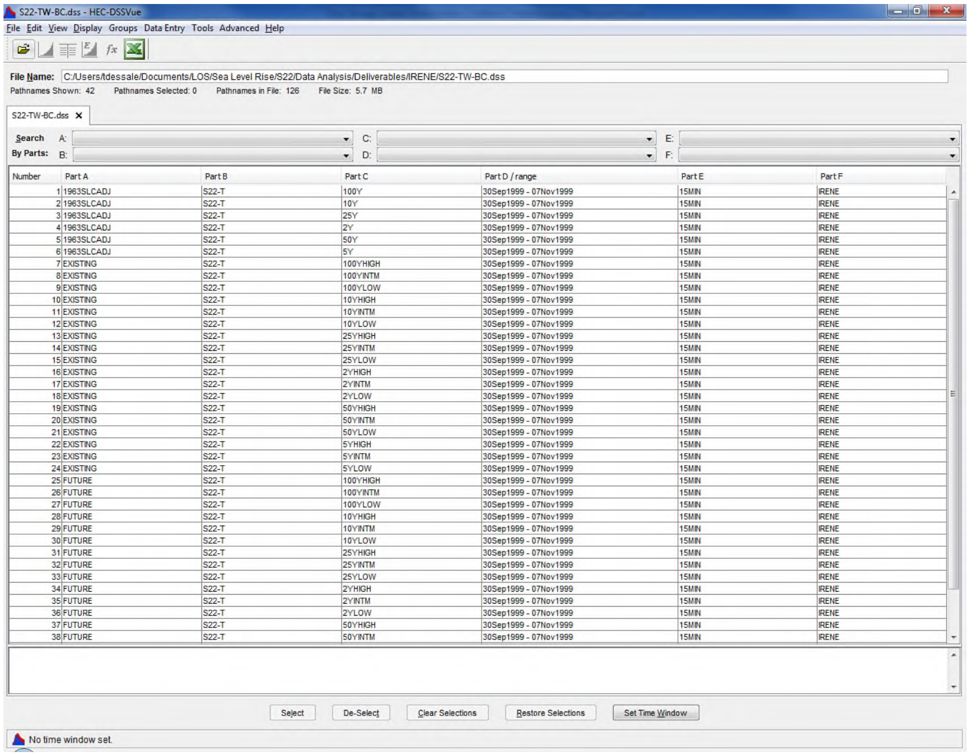
20	EXISTING	S25B-T	50YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
21	EXISTING	S25B-T	50YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
22	EXISTING	S25B-T	5YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
23	EXISTING	S25B-T	5YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
24	EXISTING	S25B-T	5YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
25	FUTURE	S25B-T	100YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
26	FUTURE	S25B-T	100YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
27	FUTURE	S25B-T	100YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
28	FUTURE	S25B-T	10YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
29	FUTURE	S25B-T	10YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
30	FUTURE	S25B-T	10YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
31	FUTURE	S25B-T	25YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE

Figure C-1-1. Condensed DSS catalog for S25B_S



Number	Part A	Part B	Part C	Part D / range	Part E	Part F
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4	1963SLCADJ	S22-T	2Y	30Sep1999 - 07Nov1999	15MIN	IRENE
5	1963SLCADJ	S22-T	50Y	30Sep1999 - 07Nov1999	15MIN	IRENE
6	1963SLCADJ	S22-T	5Y	30Sep1999 - 07Nov1999	15MIN	IRENE
7	EXISTING	S22-T	100YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
8	EXISTING	S22-T	100YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
9	EXISTING	S22-T	100YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
10	EXISTING	S22-T	10YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
11	EXISTING	S22-T	10YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
12	EXISTING	S22-T	10YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
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17	EXISTING	S22-T	2YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
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19	EXISTING	S22-T	50YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
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32	FUTURE	S22-T	25YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
33	FUTURE	S22-T	25YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
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35	FUTURE	S22-T	2YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
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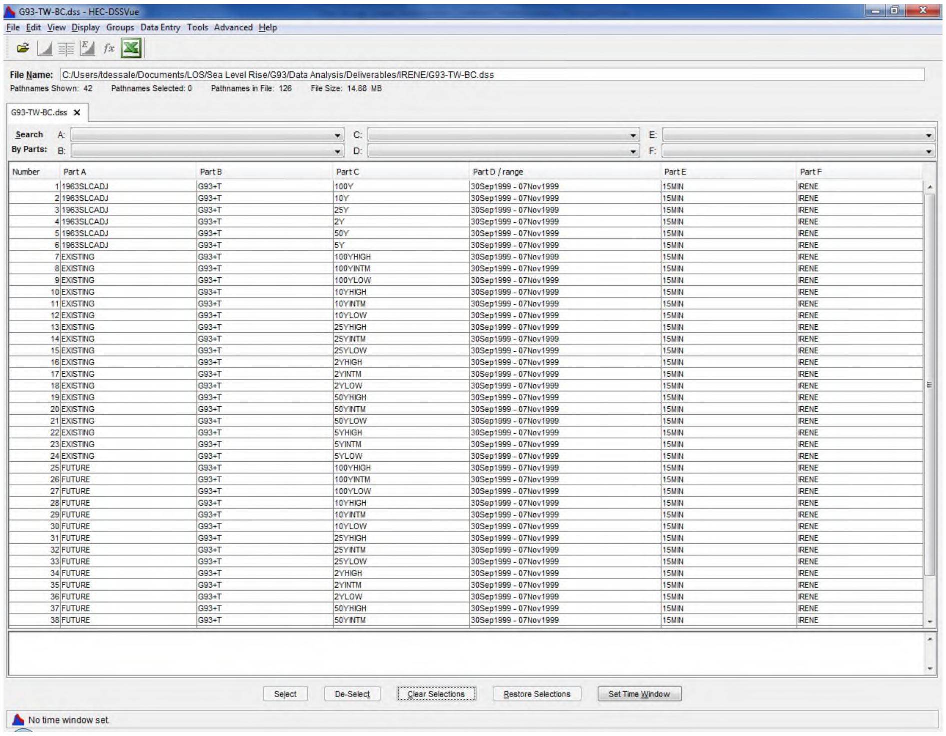
Figure C-1-2. Condensed DSS catalog for S22_S



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4	1963SLCADJ	G93+T	2Y	30Sep1999 - 07Nov1999	15MIN	IRENE
5	1963SLCADJ	G93+T	50Y	30Sep1999 - 07Nov1999	15MIN	IRENE
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25	FUTURE	G93+T	100YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
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27	FUTURE	G93+T	100YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
28	FUTURE	G93+T	10YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
29	FUTURE	G93+T	10YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE
30	FUTURE	G93+T	10YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
31	FUTURE	G93+T	25YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
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36	FUTURE	G93+T	2YLOW	30Sep1999 - 07Nov1999	15MIN	IRENE
37	FUTURE	G93+T	50YHIGH	30Sep1999 - 07Nov1999	15MIN	IRENE
38	FUTURE	G93+T	50YINTM	30Sep1999 - 07Nov1999	15MIN	IRENE

Figure C-1-3. Condensed DSS catalog for G93_S

Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed

Appendix D: C4 Basin v1.0 HEC-RAS Water Control Structure Operations

South Florida Water Management District
Hydrology and Hydraulics Bureau

December 30, 2015

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Appendix D – Water Control Structure Operations

This document was produced by the H&H Bureau as a project deliverable for project 100888 (Flood Protection Level of Service, within the Sea Level Rise project).

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\\ad.sfwmd.gov\dfsroot\data\hesm_nas\projects\basin_studies

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INTRODUCTION

Tidal water control structures are gated spillways whose objective is to effectively discharge water into the sea side of the structures while preventing salt water intrusion. Tidal structures are also the terminal water control structures in the canals, where stages downstream (sea side) from the structure are controlled by local runoff, tidal action and wind effects. Tidal structures S-25 and S-26 in the C-4 and C-6 canals were retrofitted in the past with forward pumping units that are used during periods of high tide to continue discharge from the land side to the sea side of the structure, while avoiding the risk of salt water intrusion. Operations of the tidal water control structures have negligible effects on the tidal stages at the confluences of the canals with Biscayne Bay. **Table D-1** provides information on the geometry and operational parameters for the tidal structures associated with operations of the C-4 canal.

COASTAL SPILLWAYS

Operation Protocols

The following list of actions describes the operations that typically take place at the tidal gated spillways:

1. When the headwater elevation rises above the open headwater stage, the gates will open at a rate of six inches per minute.
2. When the headwater elevation falls to below the close headwater stage, the gates will close at a rate of six inches per minute.
3. When the headwater elevation rises or falls to the mid-stage between the open and close headwater elevations, the gates will become stationary (Note: This operation was not implemented in the HEC-RAS model).

Settings for dry periods (open and close headwater stages) are higher than settings used during flood control or wet periods in order to conserve water in the canals. **Table D-1** lists sets of open and close stages that have been used for different operational scenarios in the past. For instance, values listed in red correspond to the super-low operation range that is used when the C-4 basin is wet and there is a forecast for significant rainfall with the expectation that the C-4 Emergency Detention Basins will be used. In **Table D-1**, and henceforth in this appendix, “HW” denotes “head water” and “TW” denotes “tail water”.

In order to prevent sea water from intruding into the canal at the structure location, the operations of the structure will override any opening commands and proceed to close the gates, regardless of the upstream water level, in the event that the head differential across the structure is 0.1 foot or less. In order to prevent frequent opening and closing of the gates, they are not enabled to re-open until the head differential across the structure increases to a value higher than 0.3 foot. Once this occurs, the check against 0.3 foot is no longer made and the gates will be free to open as long as the head differential is higher than 0.1 foot and the headwater stage is higher than the opening trigger stage. Checking the head differential against the 0.3 foot criterion occurs only after the head differential goes below the lower threshold of 0.1 foot and the gate is closed.

Table D-1. Geometric and operational parameters for tidal structures

Structure	Canal	Type	Design HW stage (ft NGVD)[1]	Design TW stage (ft NGVD)[1]	Optimum Stage (ft NGVD)[1]	By-Pass Stage (ft NGVD)	Design Q (cfs)	HW Gate Open (ft NGVD)[1]	HW Gate Close (ft NGVD)[1]	Maximum Gate Opening (ft)
S-25B	C-4	Spillway, 2 gates, 22 ft x 11.9 ft Crest L = 44 ft Crest Elev = -7.9 ft NGVD	4.4	4.1	2.8	5.7	2000	3.0 2.0 1.0	2.0 1.0 0.0	13.6
S-25B-P	C-4	Pump Station (Electric) 3x200 cfs pumps	Min HW Elev. 1.0	N/A	N/A	5.7	3 @ 200 (600 total) P speed 305 rpm	Pumps on for HW ≥ 1.4 ft NGVD	Pumps off for HW < 1.0 ft NGVD	N/A
G-93	C-3	Spillway, 2 gates, 5 ft x 10 ft Crest L = 20 ft Crest Elev = -1.8 ft NGVD	4.5	3.0	2.8	6.0	640	2.7 2.3	1.0 1.0	8.5
S-22	C-2	Spillway, 2 gates, 17 ft x 15 ft Crest L 34 ft Crest elev.= -11.0 ft NGVD	3.2	2.7	2.9	7.5	1915	3.5 3.0 2.5	2.5 2.0 1.5	15.1

[1] **Settings to control high stages in canals. Settings to conserve water in the basin are usually higher**

The primary operational features reproduced in HEC-RAS for the tidal structures are: 1) Gate opening headwater elevation, 2) Gate closing headwater elevation, 3) Head differential salinity threshold for closing gates, and 4) Head differential salinity threshold to re-start gate openings. The gate opening and closing rates were set to 0.5 foot/minute for S-25B and S-22 while 0.1 foot/minute was specified for G-93. Although all the structures for the C-2, C-3 and C-4 canals have two gates, only one gate is simulated in HEC-RAS and the results are duplicated for the second gate in regards to gate position and computed flow.

One of the major challenges faced in coding the operational rules in HEC-RAS was the ability to obtain a realistic representation of the gate changes that would take place under real storm conditions. For instance, in addition to the 0.3 foot head differential threshold to re-open the gates after a high tide event, minimum time delays of 10 - 30 minutes for gate re-openings were implemented in the operation rules. The operational protocols for the coastal spillways are summarized in **Figure D-1**.

Flow Computations

The gated spillways were simulated in HEC-RAS using the in-line structure option. In addition to simulating the District structure real-time operations in HEC-RAS, it is desired that the computed discharge through a tidal structure be based on its SFWMD flow computation algorithm. To achieve this, both the operational rules and the algorithm used to compute flow are coded as rules. These rules define the internal boundary conditions at these structures. The intrinsic flow computation algorithms of HEC-RAS and the SFWMD flow rating models are similar in form but differ significantly in terms of both the discharge parameters and the definition of transition zones between flow regimes.

In the initial attempt to model the tidal structures in HEC-RAS, the official District operating criteria and flow computation procedures were used. Unfortunately, the results did not adequately reflect historical operations while the simulations would often experience numerical instabilities. To overcome these difficulties, an alternative approach was adopted where the same gate opening and closing procedures were maintained while the intrinsic HEC-RAS flow computation procedures were used instead of imposing

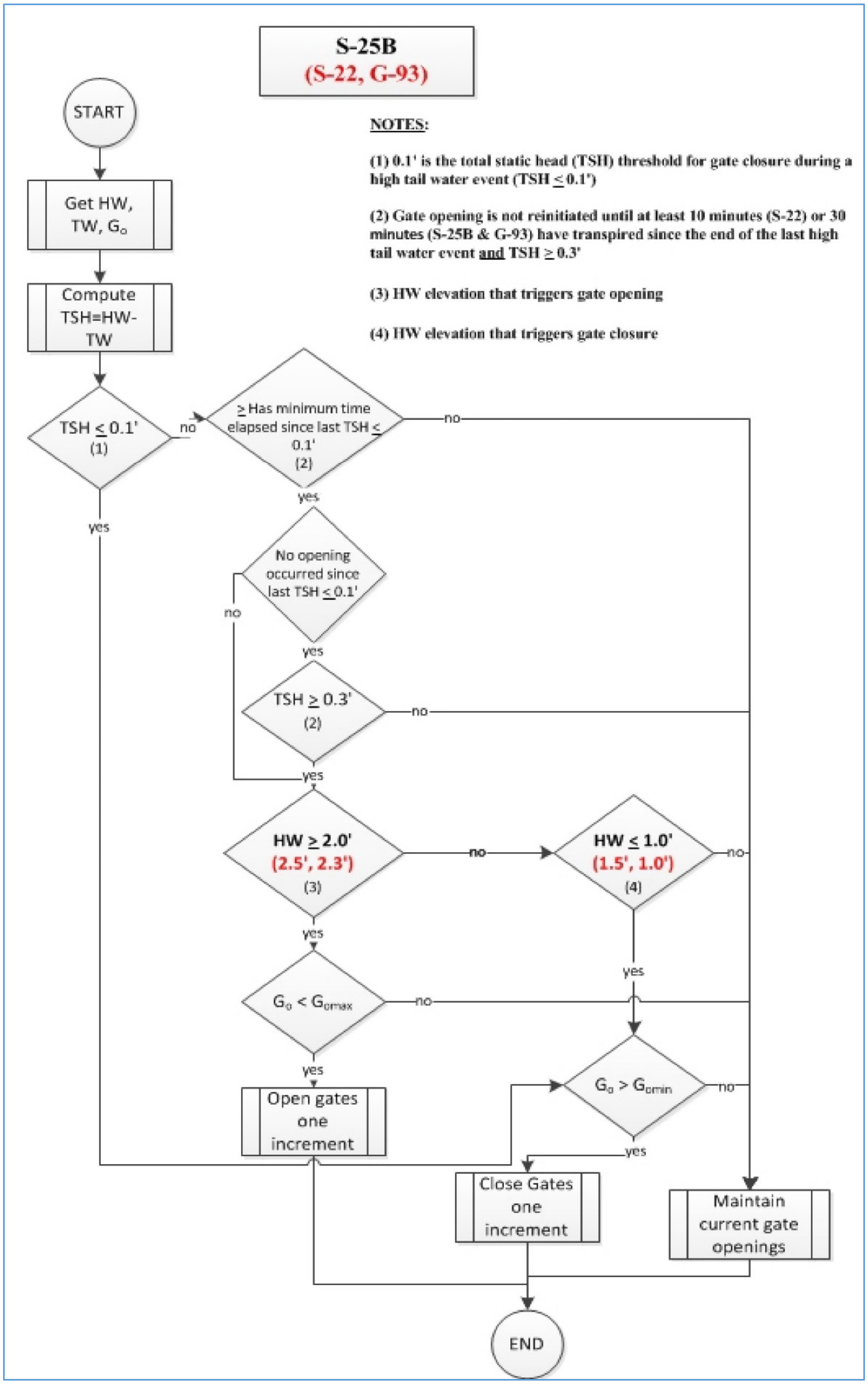


Figure D-1. Operational protocol for the coastal spillways discharges computed with the rating equations. The flow computation parameters were adjusted as needed to obtain reasonable agreement between flows computed with the intrinsic HEC-RAS equations and the SFWMD rating equations. Discharge computation equations are described in the HEC-RAS Hydraulics Reference Manual and the User Manual (**HEC, 2010a, 2010b**).

The SFWMD rating equations used to compute discharges at S-25B and G-93 spillways are described in the District Atlas of Flow Computations (**SFWMD, 2010**). This set of equations is also known as the USACE Flow Equations (**Table D-2**). The empirically derived discharge coefficient values for both S-25B and G-93 are a function of the flow regime:

- Controlled Submerged (CS), $C_d = 0.75$
- Controlled Free (CF), $C_d = 0.75$
- Uncontrolled Submerged (US), $C_d = 0.9$
- Uncontrolled Free (UF), $C_d = 2.9$

The discharge equations for computing discharge at the S-22 spillway are different from those shown in Table D-2 and are based on dimensionless analysis (**SFWMD, 2010**). These equations are provided in **Table D-3**. The corresponding parameters for S-22 are:

- Controlled Submerged (CS), $C_a = 1.0073$, $C_b = 0.2824$
- Controlled Free (CF), $C_a = 0.9$, $C_b = 0.3$
- Uncontrolled Submerged (US), $C_a = 1.02$, $C_b = 0.3$
- Uncontrolled Free (UF), $C_a = 0.71$

In **Tables D-2 and D-3**, H and h denote the head water and tail water depths, respectively above the spillway crest while Go is the gate opening.

Table D-2. Summary of the USACE Flow Equations

Flow Condition	Equation	Restriction	Remarks
Controlled-submerged (CS)	$Q = C_d L G_o \sqrt{2g(H-h)}$	$\frac{H}{G_o} > 1.7$ and $\frac{h}{G_o} \geq 0.5$	Also known as submerged orifice
Controlled-free (CF)	$Q = C_d L G_o \sqrt{2g(H-0.5G_o)}$	$\frac{H}{G_o} > 1.7$ and $\frac{h}{G_o} < 0.5$	Also known as free orifice
Uncontrolled-submerged (US)	$Q = C_d L h \sqrt{2g(H-h)}$	$\frac{H}{G_o} < 1.0$ and $\frac{h}{H} \geq 0.5$	Also known as submerged weir
Uncontrolled-free (UF)	$Q = C_d L \sqrt{H^3}$	$\frac{H}{G_o} < 1.0$ and $\frac{h}{H} < 0.5$	Also known as free weir
Transitional Flow	$Q = \min(Quncontrolled, Qcontrolled)$	$1.0 \leq \frac{H}{Go} \leq 1.7$	

Table D-3. Summary of flow equations based on dimensional analysis

Flow Condition	Equation	Restriction	Remarks
Controlled-submerged (CS)	$Q = L\sqrt{gy_c^{\ 3}}$ $y_c = aG_o\left(\frac{H-h}{G_o}\right)^b$	$\frac{h}{G_o} \geq 1.0$	Also known as submerged orifice
Controlled-free (CF)	$Q = L\sqrt{gy_c^{\ 3}}$ $y_c = aG_o\left(\frac{H}{G_o}\right)^b$	$\frac{h}{G_o} < 1.0 \ \& \ \frac{H}{G_o} \geq \frac{1}{K}$ $K = 2/3$	Also known as free orifice
Uncontrolled-submerged (US)	$Q = L\sqrt{gy_c^{\ 3}}$ $y_c = aH(1-\frac{h}{H})^b$	$\frac{h}{G_o} < 1.0, \frac{H}{G_o} < \frac{1}{K}, \& \frac{h}{H} \geq K$ $K = 2/3$	Also known as submerged weir
Uncontrolled-free (UF)	$Q = L\sqrt{gy_c^{\ 3}}$ $y_c = aH$	$\frac{h}{G_o} < 1.0, \frac{H}{G_o} < \frac{1}{K}, \& \frac{h}{H} < K$ $K = 2/3$	Also known as free weir
Transitional Flow	No transition region		

Note: y_c denotes Critical depth (ft) while other symbols are the same as defined previously.

COASTAL PUMP STATIONS

Operational Protocols

As indicated previously, S-25B was retrofitted with forward pumps that are operated to maintain discharges from the land side to the sea side of the structure when gravity capacity is limited or the gates need to be closed due to the threat of salinity intrusion. There are 3 pumping units which can be operated independently (**Table D-1**). Operation of the pump units at S-25B is an integral part of C-4 Basin Operating Plan (**SFWMD, 2011b**) and at least one unit will be operated to control upstream stages when all of the conditions listed below occur concurrently:

1. The T5W gage stage exceeds 3.8 feet NGVD (see **Table D-4**). The T5W gage is located on the C-4 canal, just upstream from its crossing under the Florida Turnpike.
2. The MRMS1 gage stage is less than 4.75 feet NGVD. This gage is located on the Miami Canal (C-6) just downstream of the NW 27th Ave. Bridge.
3. The headwater elevation at S25-B exceeds 1.4 ft NGVD.
4. The available gravity capacity at the S-25B spillway is less than the total capacity of the pumping units (600 cfs).

The pump station will be turned off when any of the conditions listed above is not met or the upstream water surface elevation measured at the S-25B headwater location recedes below 1.00 foot NVGD. Additionally, the operation of pump units at S-25B during a storm event follows the “ramp up” style of operational rules shown in **Table D-4** as a function of the stage at T5-W. The operational procedures for the S-25B pump station are summarized in **Figure D-2.**

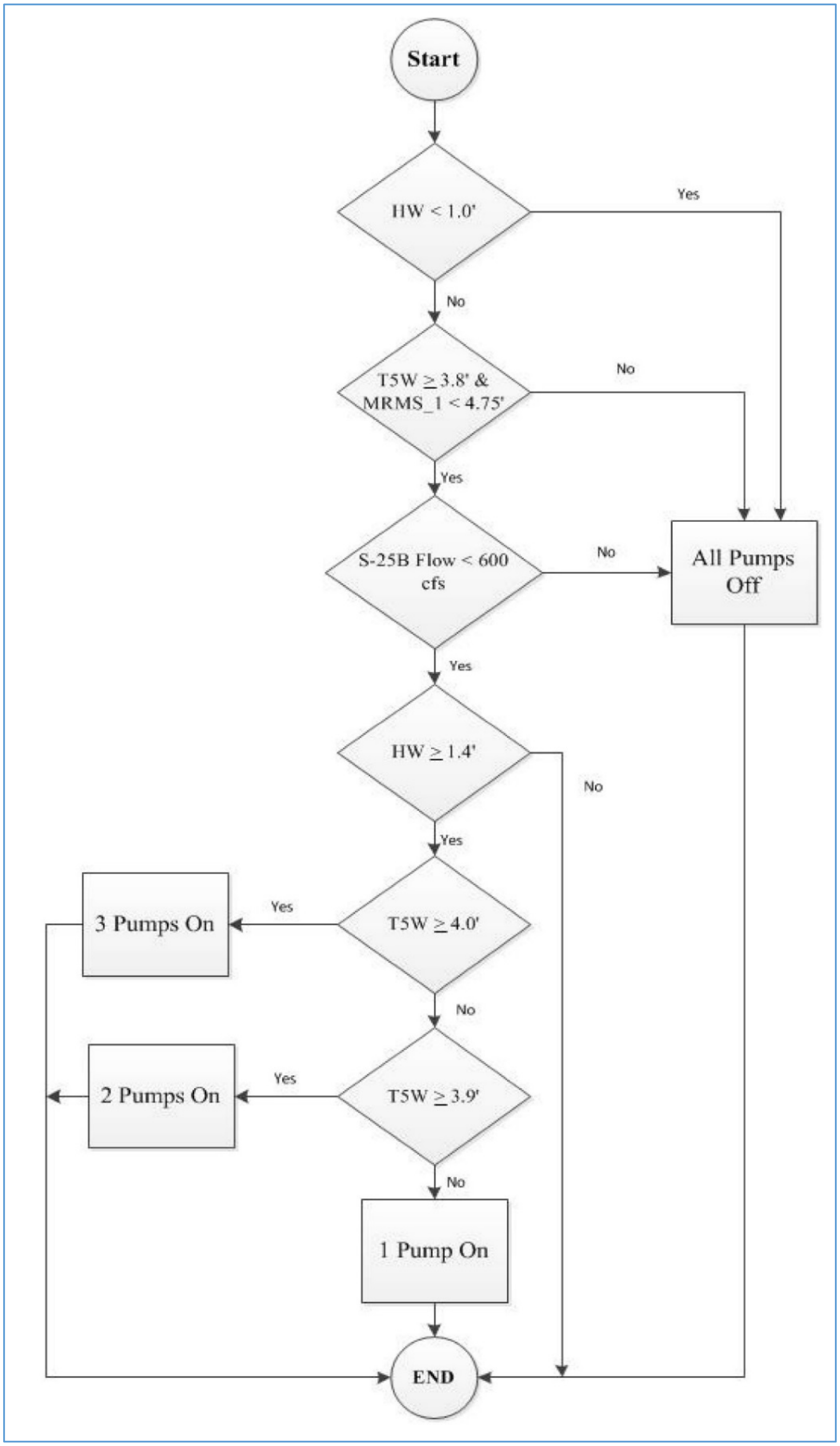


Figure D-2. Operational procedures for the S-25B pump station

Table D-4. Ramp-up of S-25B pump units during a storm event based on the stage at T5W

T5W Stage (ft NGVD)	Number of pumps on
< 3.8	0
3.8 to 3.9	1
3.9 to 4.0	2
≥ 4.0	3

Flow Computations

When operated, the discharge through each pump unit was computed within the operational script using the District's flow rating equation for this pump station. Flow computations are the same for each pump. Detailed information on the rating equation is given by Imru and Wang (**2004**).

INTERIOR SPILLWAYS

The spillways located within the interior of the C-4 basin include G-421 and S-380. The operations and flow computations for these structures are discussed below. Additional information is provided in **Table D-5**.

Table D-5. Geometric and operational parameters for interior structures

Structure	Canal	Type	Design HW stage (ft NGVD)	Design TW stage (ft NGVD)	Optimum Stage (ft NGVD)	By-Pass Stage (ft NGVD)	Design Q (cfs)	HW Gate Open (ft NGVD)	HW Gate Close (ft NGVD)	Maximum Gate Opening (ft)
S-380	C-4	Gated Culverts, 5 72-in Barrels, L = 96 ft	4.2	3.85 (approx.)	4.0	10.0	400	4.2	3.8	6.0
G-420	Detention Basin Feeder Canal	Pump Station (Electric) 3x223 cfs pumps	Min HW Elev. 3.6	Max TW Elev. 10.0	N/A	N/A	3 @ 223 (669 total) P speed 270 rpm	(1)	(1)	N/A
G-421	Detention Basin Feeder Canal	Spillway, 1 gate, 4 ft x 20 ft Crest L = 20 ft Crest Elev = 6.0 ft NGVD	N/A	N/A	N/A	10.0	N/A	(1)	(1)	4.0
G-422	Detention Basin Feeder Canal	Pump Station (Electric) 7x89 cfs pumps	Min HW Elev. 4.0	Max TW Elev. 10.0	N/A	N/A	7 @ 89 (623 total) P speed 297 rpm	(1)	(1)	N/A

[1] This structure is operated in accordance with the C-4 Basin Operating Plan (**SFWMD, 2011b**) Operation Protocols

S-380

S-380 is a gated culvert structure located in the C-4 canal near the southwestern corner of the detention facility. It is comprised of five 72-inch gated barrels and is generally operated in accordance with the C-4 Basin Operating Plan (**SFWMD, 2011b**). These operational procedures for S-380 are summarized in **Figure D-3,** where TSH denotes the total static head across the structure (TSH = HW – TW).

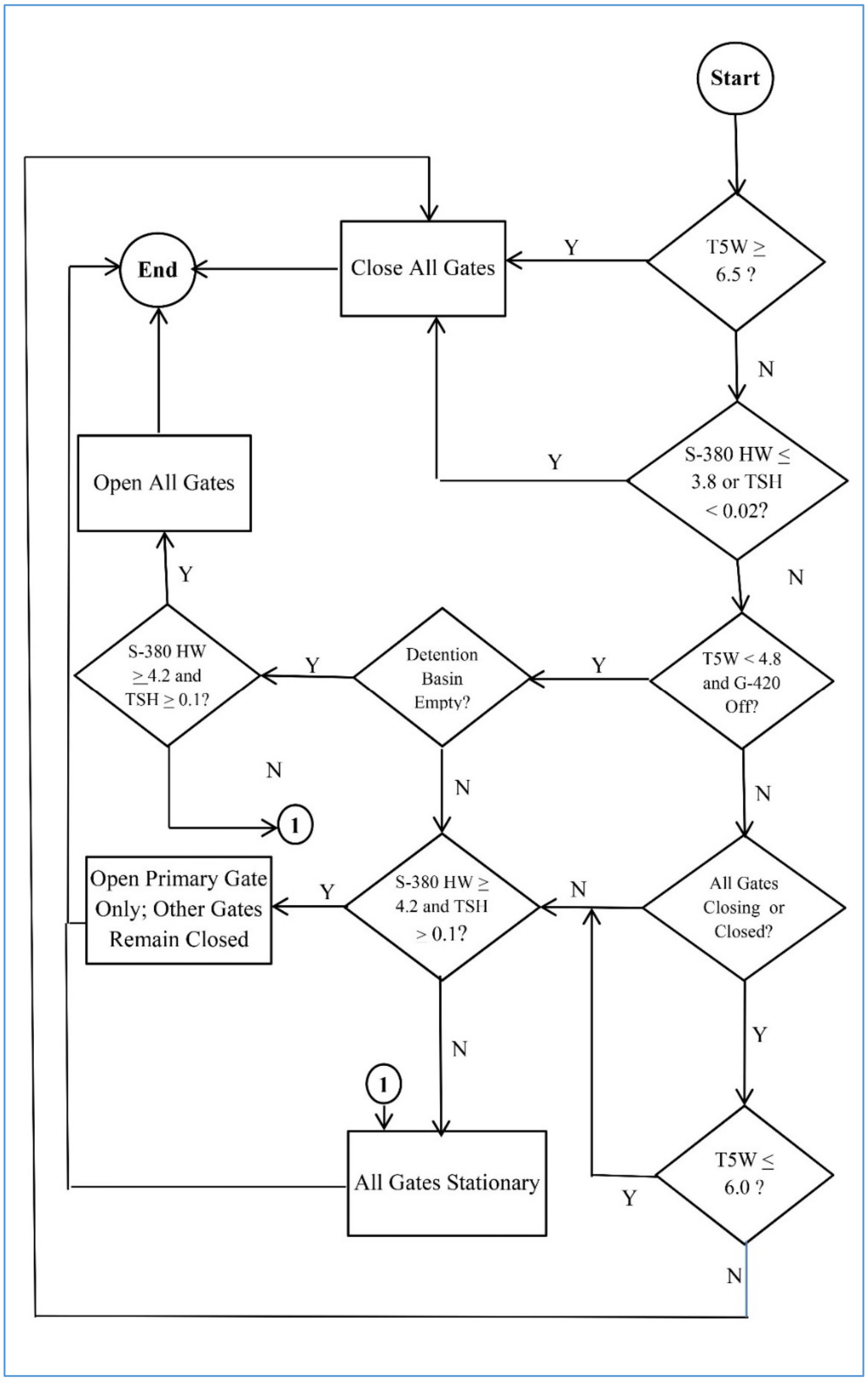


Figure D-3. S-380 operational procedure.

G-421

G-421 is a gated spillway located near the southeastern corner of the emergency detention basin. It contains one gate and serves as the outlet for the detention basin. It is operated in accordance with the C-4 Basin Operating Plan mentioned above. Its operational procedures are summarized in **Figure D-4**.

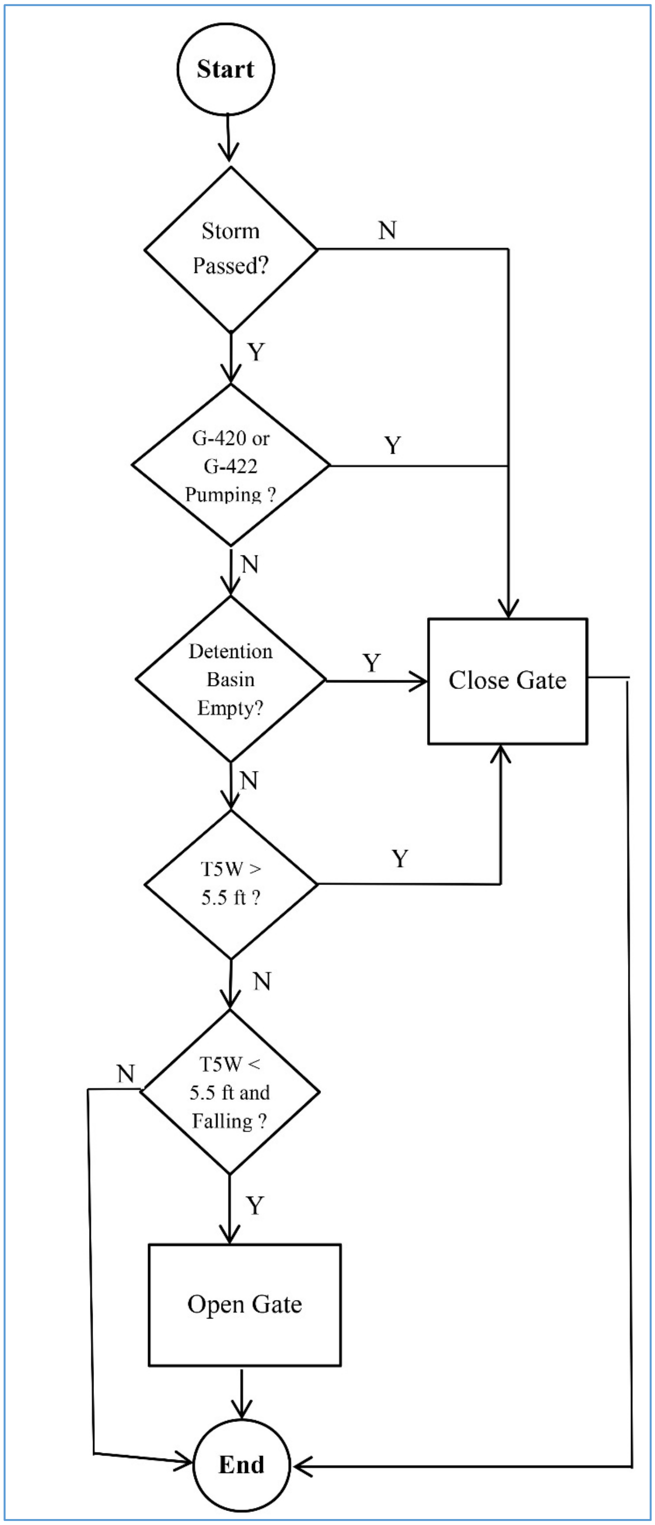


Figure D-4. G-421 operational procedure.

INTERIOR PUMP STATIONS

The pump stations located within the interior of the C-4 basin include G-420, G-422 and the various local pump stations that serve the municipalities located along C-4. The operations and flow computations for these structures are discussed below. Additional information is provided in **Table D-6.**

Operation Protocols

G-420

G-420 is the primary inflow pump station to the emergency detention area. It contains three pumping units with a combined capacity of 669 cfs. Its operational scheme (**Figure D-5**) is somewhat complex and is part of the overall C-4 Basin Operating Plan (**SFWMD, 2011b**). It essentially turns on when the C-4 stage at gage T5W exceeds 4.8 feet NGVD and continues pumping until either the detention basin is full (10.0 feet NGVD) or the stage at T5W falls below 4.8 feet. In addition, if the detention basin stage reaches 8 feet NGVD while the C-4 stage at T5W is below 5.9 feet NGVD, G-420 will stop pumping until the stage at T5W rises to 6.5 feet or higher. At this trigger stage pumping will resume until the either detention basin is full or the stage at T5W falls back below 5.9 feet.

G-422

G-422 has an operational role that is similar to that of G-420. It is comprised of seven pump units. If G-420 is pumping at capacity and the stage at T5W rises to 5.0 feet NGVD, the three primary units with a combined capacity of 267 cfs commence pumping. The four secondary units (combined capacity = 356 cfs) will turn on subsequently if the T5W stage rises to 5.2 feet. None of the units will be running if G-420 is not pumping at capacity while all units will be operational if the T5W stage rises to 6.5 feet and the detention basin is not full. **Figure D-6** summarizes this operational protocol.

Municipal Pumps

A number of municipal storm water pump stations are located along the C-4 channel. They provide local flood control benefits to the various municipalities within the basin. Information on these pump stations was obtained from SFWMD (**2011a**) and is summarized in **Table D-6**.

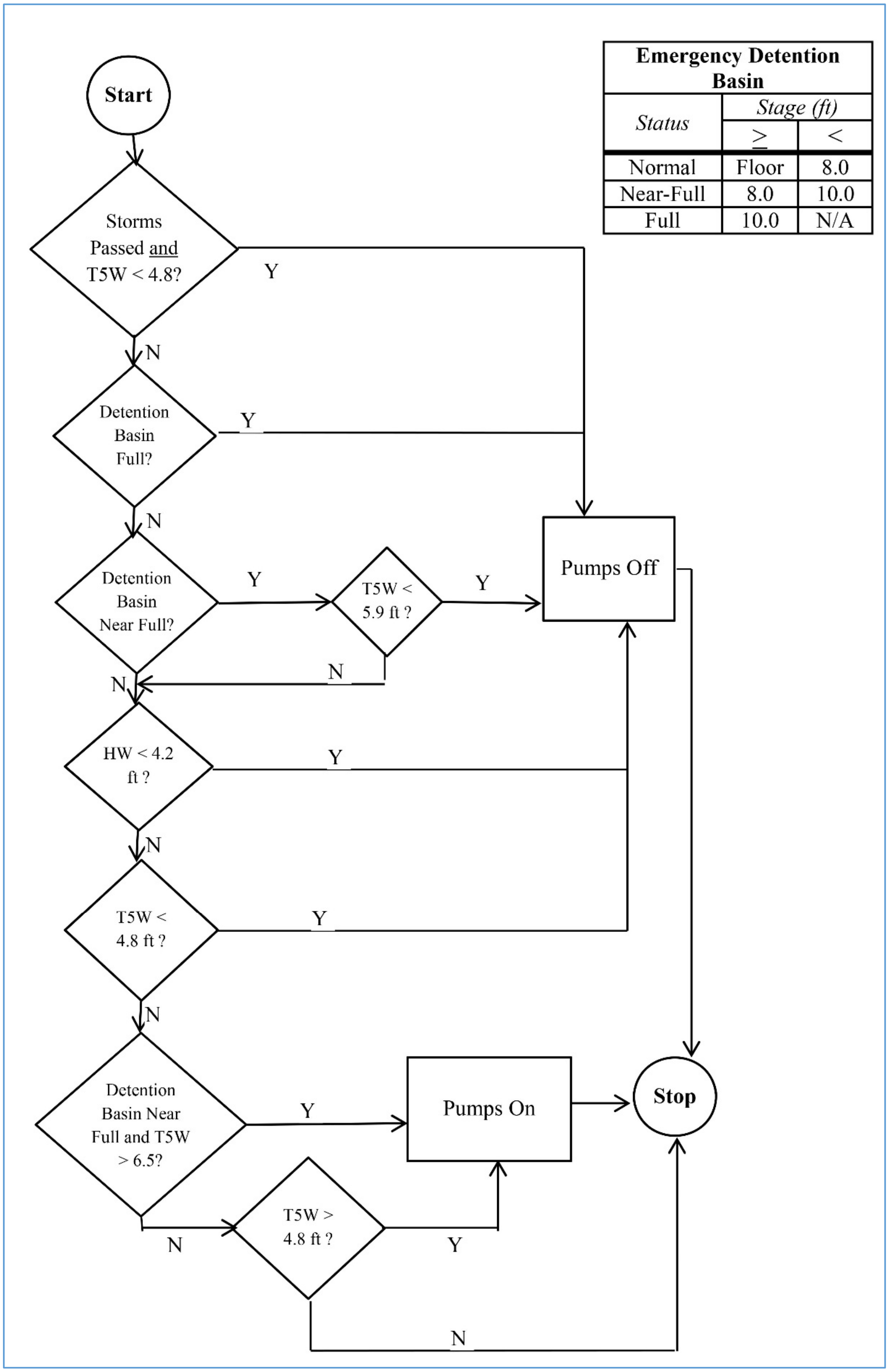
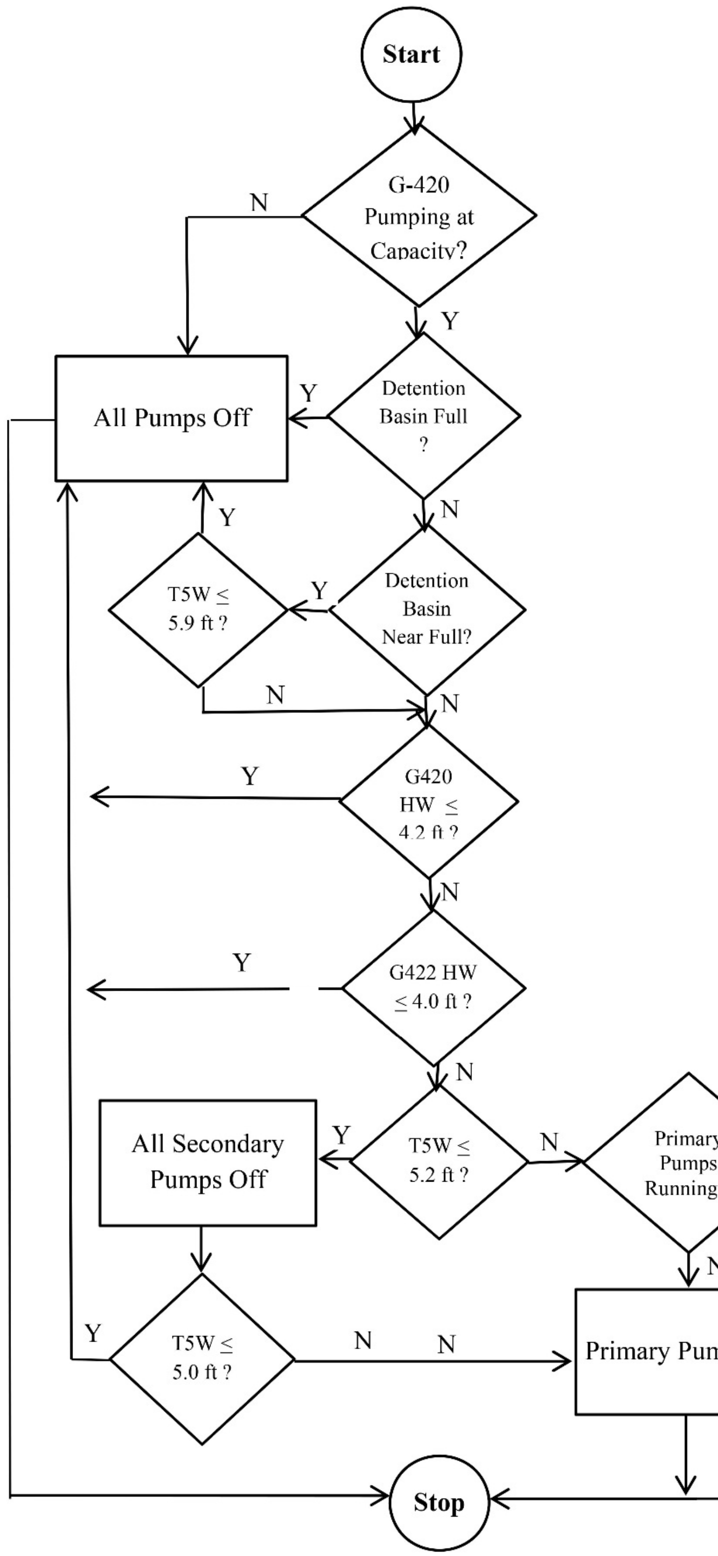


Figure D-5. G-420 operation procedure



Emergency Detention Basin		
Status	*Stage (ft)*	
	≥	<
Normal	Floor	8.0
Near-Full	8.0	10.0
Full	10.0	N/A

Figure D-6. G-422 operation procedure

Table D-6. Municipal storm water pump stations

Municipality	Pump Station	Capacity (CFS)	Head Water Stage Trigger		Outfall Location
			On	Off	
Belen	PS1	100	5	4.5	SW 127th Ave @ C-4
	PS2	100	5	4.5	SW 120th Ave @ C-4
Sweetwater	B1	20	5.5	5	SW 102ndAve @ C-4
	B2	20	6	5.5	SW 102ndAve @ C-4
	B15	25.4	6.5	6.25	NW 4th Terr @ C-2
	B16	25.4	6.5	6.25	NW 4th Terr @ C-2
	IIA#3	27	6.5	6	SW 108ndAve @ C-4
	IIA#4	27	6.5	6	SW 108ndAve @ C-4
	IIB#1	27	5.5	5	SW 110thAve @ C-4
	IIB#2	27	6	5.5	SW 110thAve @ C-4
Miami	1	30	6	5.41	SW 67ndAve @ C-4
	2	60	6	5.41	NW 65thAve @ C-4
	3	40	6	5.41	SW 64thAve @ C-4
	4	54	6	5.41	NW 63rdAve @ C-4
	5	15	6	5.41	NW 7th St near NW 52nd Ave @ C-4
West Miami	1	90	5	2.51	SW 64thAve @ C-4
	2	100	5	2.51	SW 64thAve @ C-3

The municipal storm water pump stations are operated in accordance with both the C-4 basin operations plan and the MOU between the SFWMD and the municipalities. These operations are summarized in **Figure D-7**.

Flow Computations

No flow computations were performed for the interior pump stations. When operational, the discharge from each pump was set to its nominal capacity.

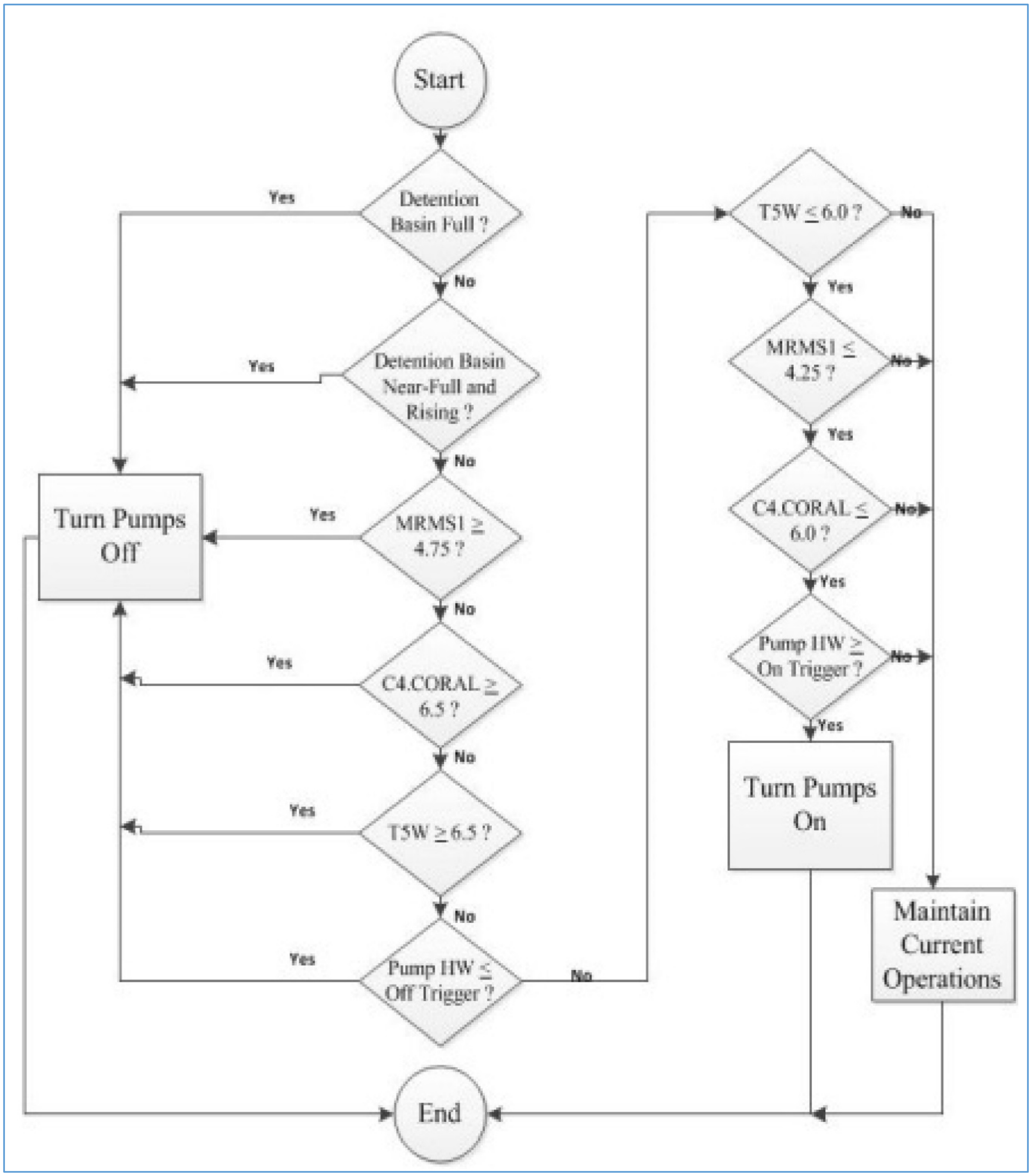


Figure D-7. Municipal storm water pump operations

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Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed

Appendix E: Flood Model Development and Quality Assurance

South Florida Water Management District
Hydrology and Hydraulics Bureau

December 22, 2015

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Appendix E: Flood Model Development and Quality Assurance

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This document was produced by the H&H Bureau as a project deliverable for project 100888 (Flood Protection Level of Service, within the Sea Level Rise project).

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MODEL SELECTION

The C-4 Basin Level of Service (LOS) assessment project required the evaluation of drainage capacity and constraints in the C-4 Basin under existing and future with sea level rise conditions. The principal tool for this task was a hydrologic/hydraulic model of the C-4 basin. In the initial stages of the project, several models were considered through an evaluation process which looked into all applicable models for this task. Among these models were Mike-SHE/Mike-11, XPSWMM, HEC-HMS/HEC-RAS and the District's Regional Simulation Model. **Table E-1** summarizes the model evaluation matrix used for selecting the code for this project. **Table E-1** is a summary of the model evaluation matrix used to compare model features applicable to this study.

Table E-1. Model Selection Matrix

	MRW decoupled : HEC-RAS MODFLOW	MRW coupled : HEC-RAS MODFLOW	XP-SWMM	RSM – STAGE BASED	HEC-RAS HEC-HMS of one watershed
PROs					
FEMA approved	✓	✓	✓		✓
Detailed primary canal hydraulics	✓	✓	✓	✓	✓
Complete St. Venant hydraulics	✓	✓	✓		✓
Calibrated for the C-4 basin	✓	✓	✓		
Short Simulation Time				✓	✓
Complex Structure Operations			✓	✓	✓
Includes SFMWD Structures	✓	✓		✓	✓
Other	Simulates aquifer seepage	Fully coupled	Locally Accepted	Entire region	
CONs					
No runoff component	×	×			
Limited aquifer component			×		×
No guaranteed mass balance	×				
Manual Coupling	×				
Recalibration required	×	×	×	×	×
Requires updating to current conditions	×	×	×	×	×
Proof-of-Concept only		×			
Simple structures only				×	
Limited experience with model			×		
Very long simulation time		×			

As **Table E-1** indicates, no one model has all the features needed for this application, thus requiring to look for an alternate modeling approach. After the evaluation process, the project team selected the HEC-RAS model for several reasons including:

- The HEC-RAS model is FEMA certified,
- It is a fully hydrodynamic model with customizable hydraulic structure operations, including District gated structures and complex operations.
- Public domain and has continuous support from the Army Corps of Engineers,
- District staff has experience with the code and application in the C-4 basin,
- A calibrated HEC-RAS model from a previous C-4 Basin study was already available.

The requirement to include canal/aquifer interaction in the model was fulfilled with a simplified approach which is uniquely justifiable in this region due to the extremely high conductivities of the surficial aquifer and interaction with the primary canals. This model conceptualization is further explained under Conceptual Model section of this appendix.

CONCEPTUAL MODEL

Recent modeling efforts by the District in the C-4 basin (**SFWMD, 2009; 2011**) indicate that accurate representation of stages in the basin and in the canal require the use of integrated surface-groundwater models due to the high interconnectivity of the surficial aquifer with the canal system. In this study, the District used a 1-D hydrodynamic canal routing model of the C2, C3, C4, C5 and C6 canals (HEC-RAS) coupled with a 3-D groundwater flow model (MODFLOW) to simulate the exchange of water between the underlying aquifer surficial aquifer and the canals during flooding events. The calibrated version of the model was used to evaluate the effectiveness of flood control operations in the basins. Despite the successful application of the coupled HEC-RAS/MODFLOW in the C-4 basin, for evaluating operational protocols in the basin, the model has severe limitations due to size, complexity and long run times.

In this project, a simplified approach is used to circumvent the use of coupled surface/groundwater models in the basin. This simplified approach results in fast development and application of the model for evaluation of flood level of service scenarios in the C-4 Basin. The hydrologic processes in the basin are simplified to include only direct rainfall and evaporative losses over the basin.

In this conceptual drainage model only the primary and some secondary canals built and managed by Miami-Dade County's Department of Environmental Resource Management (DERM) are included in the model. Tertiary drainage features such as infiltration trenches and sewers are neglected. **Figure E-1** shows a schematic of the model representation of sub-basin storages interacting with a canal segment in the

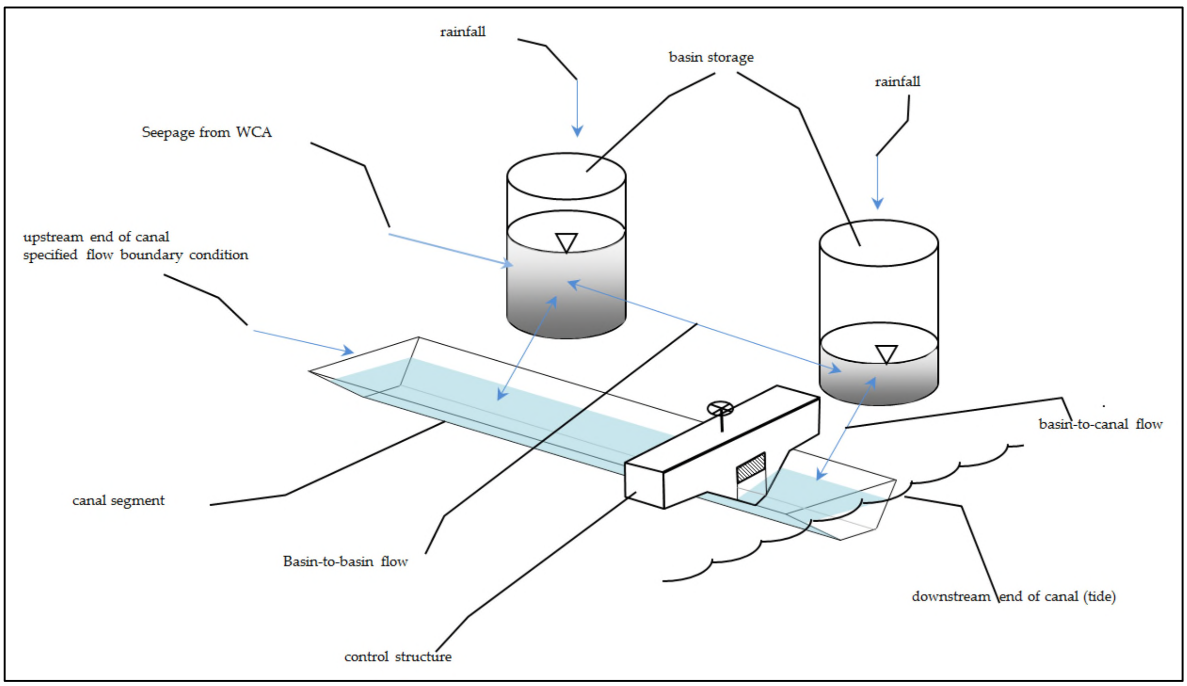

Figure E-1. Schematic of Sub-basin and Canal interactions in HEC-RAS Model

HEC-RAS model. Net inflow (rainfall minus evaporative losses) enter the basin (groundwater and above ground combined) storage without lag time due to surface flow and unsaturated zone routing. Sub-basin groundwater storage is a function of soil porosity which is assumed constant over the sub-basin. Sub-basin above-ground storage includes lake and topographic depression storage captured in the LIDAR data of the sub-basin.

The C-4 Basin area is subdivided into sub-basins that interact with the primary canal system via seepage and canal overbank flows. The seepage flow interaction from basin to basin and from basin to canal is a function of the head differential between the two water bodies and a conductance term to be determined though model calibration. The overbank flow interaction is a function of the head differential between the sub-basin and the water elevation above the canal bank and the length over which the overbank flow occurs. There are several water control structures in the C-4 canal system such as pumps that drain sub-basin runoff to the C-4 Canal and pumps that remove excess runoff from the C-4 Canal to the C-4 Emergency Detention Basin in the Western portion of the C-4 Basin.

MODEL FORMULATION

This section of the report describes the drainage system of the C-4 Basin that are relevant to the model developed for the evaluation of the Level of Service for flood protection under existing and future with sea-level rise conditions. More detailed description of the basin drainage attributes and operations for flood control can be found in the Water Control Operations Atlas for the Miami-River System (**SFWMD, 2015**).

STUDY AREA

The C-4 Basin encompasses an area of about 83.3 square miles of highly urbanized areas on eastern portion of the basin including the Miami International Airport, The Florida International University main campus, the cities of Belen and Sweetwater and portions of the cities of Doral, Miami and West Miami. The portion of the basin west of the Florida Turnpike is mostly undeveloped and includes large rock mining areas and the Pennsuco Wetlands just east of the L-31 Canal (**Figure E-2)**. The drainage capacity of the primary canal in the C-4 Basin is highly influenced by flood control operations in the C-2 and C-3 Basins. **Figure E-2** also shows the location of the C4 and adjacent basins in Central Miami-Dade County. The C-2 Basin is located to the South of the C-4 Canal and its primary drainage feature is the C-2 Canal which intersects the C-4 Canal just west of the Florida Turnpike. Flows and water levels in the C-2 Canal are controlled by gate operations at the S-22 Structure located at the tidal end of the C-2 Canal just east of SW 57th Avenue. Also, south of the C-4 Canal, the C-3 Basin is drained by the C-3 Canal which affects the flows in the C-4 Canal. The C-3 Canal intersects the C-4 Canal just east of the Palmetto Expressway and is the only primary drainage feature for the City of Coral Gables. The G-93 structure located near the intersection of SW 56th Avenue and SW 35th Street is operated by the District to control flows and canal stages in the C-3 Canal. Because of their significant influence on flows and stages in the C-4 Basin, the C-2 and the C-3 Basins were included, in a simplified manner, as part of the model for the C-4 Basin.

Sub-basins Delineation

Discretization of the C-4 model area into sub-basins for this study was based on the sub-basin delineation method used by Miami-Dade County in the development of the XPSWMM model (**DERM, 2004**). DERM's sub-basin maps include 245 sub-basin in the C-4 Basin, however, for this study a coarser sub-set of 39 sub-basins were used to characterize the drainage of the C-4 Basin. Hydrologic lumped parameter approaches typically required characterization of hydrologic parameters at the sub-basin level. These lumped parameters are used to approximate hydrologic processes such as infiltration, runoff, etc. In this study, a lumped parameter approach is followed to represent groundwater exchange between sub-basins and canals. The lumped parameters for these processes represent hydraulic conductance used to compute the exchange of water between water bodies.

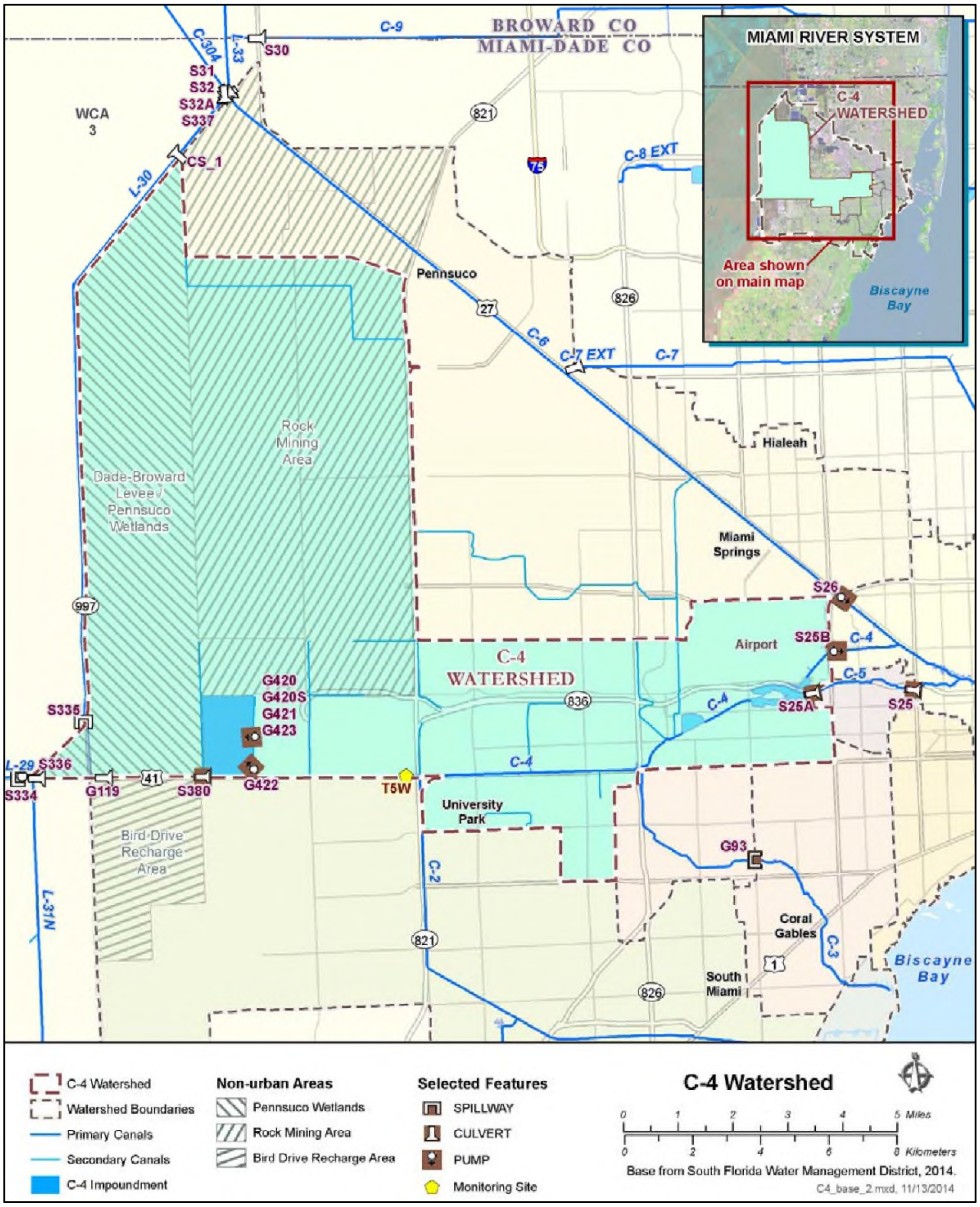

Figure E-2. Location of the C-4 Basin in Miami-Dade County, Florida

The sub-basin delineation for this study is shown in **Figure E-3**. The model domain area also includes the C-2, C-3 and C-5 basins which are modelled, for simplicity, as entire basins represented by a single node. As mentioned above, there are 39 sub-basins including the C-2, C-3 and C-5 basins. The total area of the C-4 Basin is 83.3 square miles (53,321 acres) and the largest sub-basin is the Pennsuco Wetland (Sub-basin C4_10A) in the western edge of the basin with an area of 21.2 square miles (13,582 acres).

As described in the Conceptual Model section of this Appendix, rainfall enters a sub-basin storage instantaneously. In reality, there is a time lag that delays the recharging of the surficial aquifer. This delay in the infiltration process is a function of the land cover type and soil characteristics in each sub-basin. However, in the eastern portion of the C-4 Basin, the construction of French Drains is a common drainage practice that is used to remove surface runoff and direct it into the surficial aquifer.

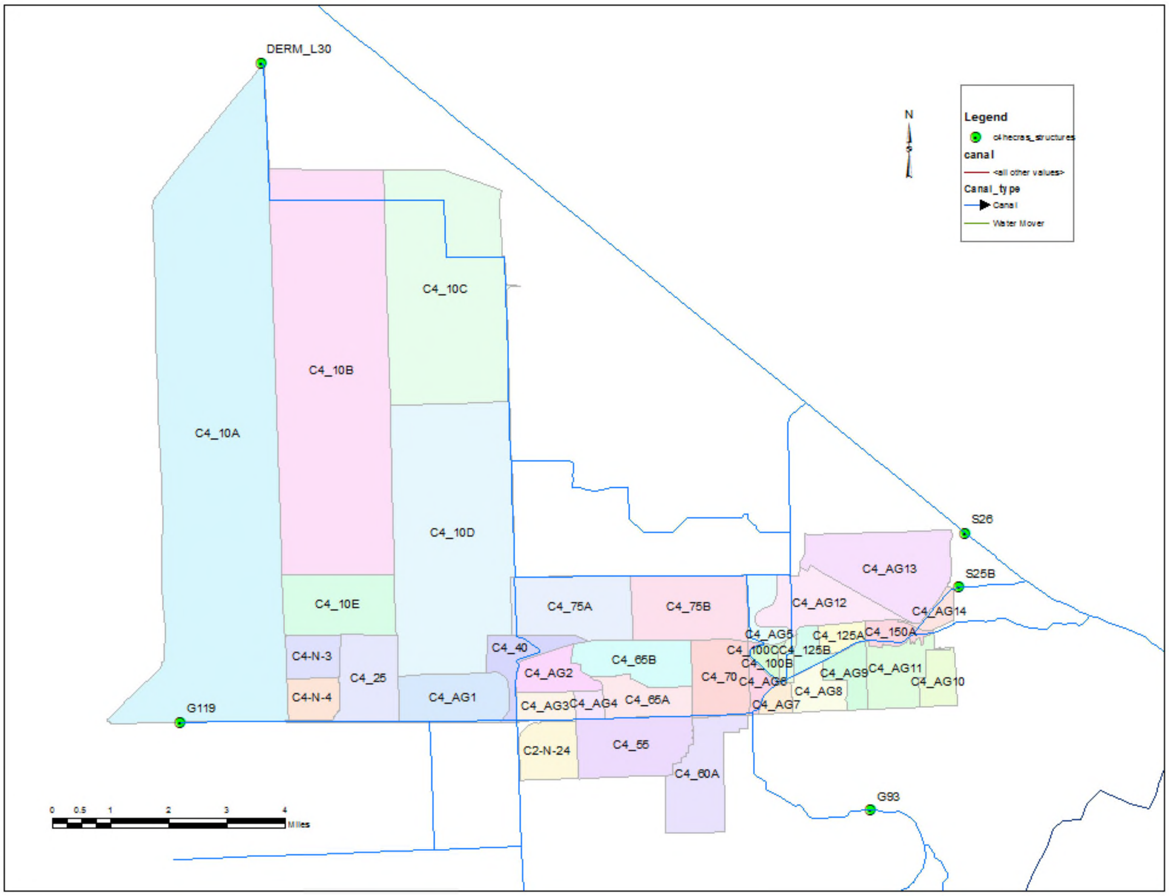

Figure E-3. Sub-basin Delineation of the C-4 Basin

Assuming that these drainage features are properly maintained and free of obstructions, the assumption of zero time delay for infiltration is reasonable for the purpose of this model. In the western portion of the C-4 basin where large tracts of natural wetlands and undeveloped land remain, this assumption may not be valid however in this area there are large portions of open pits which provide direct connection between the surface runoff and the sub-surface water. Errors in computed water levels in the sub-basins with the proposed model will probably reflect in the timing of the runoff hydrograph peak which will tend to peak earlier in the proposed model for this study.

Canal Network, Control Structures and Flood Control Operations

The purpose of the model for the C-4 basin is to compute estimates of water levels in the sub-basins and in the canals included in the model. The ability of the C-4 Basin to drain surface runoff from its sub-basin areas depends in the primary and secondary canals conveyance which, in turn is a function of drainage operations of the control structures in the basin and the highly transient water elevations at the tidal end of the C-4 Canal. There are open connections in the C-4 to the C-2, C-2 Extension and C-3 Canals which highly influence the stages and flows in the C-4 Canal making necessary the inclusion of these two canals as part of this investigation. Other secondary canals in the C-4 Basin include the Northline Canal on portions of the northern boundary of the C-4 Basin, the FEC Canal just west of the Miami International

Airport, and the Westbrook Canal on the southern boundary of the C-4 Basin. **Figure E-4** shows the primary and secondary canals in the C-2, C-3 and C-4 Basins. Currently the District operates twelve control structures in the C-2, C-3, C-4 and basins for flood control purposes. These structures are also shown in **Figure E-4**. The flood control operations in each basin are discussed next.

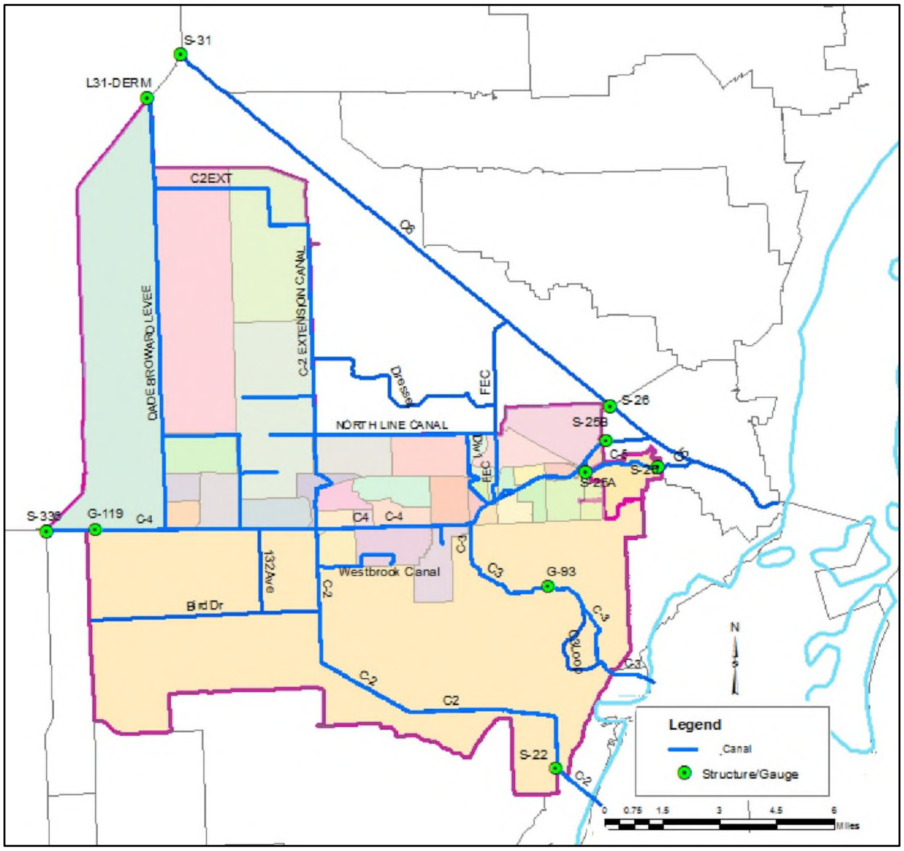

Figure E-4. -C-2, C-3 and C-4 Basin Canals and Control Structures

C-4 Basin Drainage Features

At the western end of the C-4 canal, east to the confluence with canal L-30, structure S-336 operates by telemetry in conjunction with structure G-119 to discharge excess water from Water Conservation Area 3A when capacity is available in the C-4 canal (when stage at T5W is below 3.0 ft NGVD). During severe storm events in the C-4 basin when there is not enough capacity to discharge water from WCA-3A into the C-4 Canal, both S-336 and G-119 remain closed.

If flooding conditions worsen in the C-4 Basin as established by a water elevation of 4.8 ft NGVD or higher in the C-4 Canal at the intersection with the C-2 Canal near the Florida Turnpike (T5W gauge), two pump stations are turned on to send stormwater runoff from the C-4 Canal via the Feeder Canal, into the C-4 Impoundment. To facilitate pumpage operations into the C-4 Impoundment, structure S-380 can be partially closed to allow sufficient water from the west to maintain minimal water levels in the Feeder Canal to prevent cavitation and damage to the pumps.

In addition to the C-4 Impoundment and associated pumps, the District also operates a pump station in coordination with the gated spillway structure S-25B located on the C-4 Canal, near LeJeune Rd. This structure was designed to drain the C-4 Basin under gravity flow conditions but because of high tide conditions on the east side of the structure during sever storm events associated with tropical storms, the structure's capacity to discharge by gravity is severely limited or totally curtailed during high tidal conditions. In 2004, the District completed the construction of a 600 cfs pump station on the tailwater side of the spillway that can be operated during high tides allowing continuous discharge of the C-4 Basin to tide.

The following is a more detailed description of the structures currently used for flood control operations in the C-4 basin.

Structure S-336

This structure is a triple-barreled gated culvert located on the C-4 Canal just east of the L-31 Canal. The structure is typically used for water supply deliveries from the WCA 3A to the east in dry season and, when there is capacity in the C-4 Canal, it can be used to discharge excess water from the conservation area to tide. For modeling purposes this structure is not included in the model since it replicates the operation of structure G-119 just east of S-336.

Structure G-119

This structure consists of two 72-inch diameter corrugated metal pipes with a manually operated slide gate on each pipe. The structure design capacity is 200 cfs with a design headwater stage of 4.2 ft and tailwater stage of 3.7 ft. In the HEC-RAS model this structure is used as the upstream end of the C-4 canal representing a flow boundary condition in the canal with zero flow when the structure is closed during severe storm events. This structure is not modelled explicitly in the HEC-RAS model. It represents the end of the C-4 Canal in the model and is represented as a no-flow boundary condition when the structure is closed.

Structure S-380

In 2004, a gated water control structure (S-380) was installed on the C-4 Canal at the intersection with the Dade-Broward Levee southeast of the Pennsuco wetlands. The purpose of the structure is to raise surface and groundwater levels west of it to help rehydrate the wetlands, reduce seepage from the wetlands and WCA-3B, and increase aquifer recharge to enhance water supply. The structure is operated to maintain an optimum headwater stage of 4.0 ft NGVD during the dry season by setting the opening setting at elevation 4.2 ft NGVD and closing at elevation 3.8 ft NGVD. During flood control operations in the C-4 basin, this structure may be open to allow for pre-storm operations to lower stages in the C-4 canal between structure G-119 and structure S-25B, and more importantly, during flood conditions as defined by a water elevation of 4.8 ft NGVD or higher on the downstream side of the structure (at T5W gauge) when flood water from the C-4 are being pumped into the C-4 Impoundment. To accomplish this, the structure is partially closed to prevent stormwater runoff from the western portion of the C-4 basin to flow to the east. Up to four of its five culverts can be closed only to pass enough water downstream to prevent drying up the C-4 Impoundment Feeder canal and thus damaging the G-420 and G-422 pumps. After the flood stages have receded below elevation 4.8 ft NGVD at the T5W gauge, the structure may be completely opened again to drain the western portion of the basin.

This structure is modeled explicitly in HEC-RAS using the latest flow rating equations developed by the District. For model calibration the District's flow rating equation is used with observed gate openings as internal boundary conditions at the structure to compute the flows through the culverts.

C-4 Emergency Detention Impoundment and Pumps

The C-4 Emergency Detention Basin (**Figure E-5**) was constructed as a part of FEMA's Hazard Grant Mitigation Program funding to increase the level of flood protection for the cities of Sweetwater, West Miami and Miami. The pump stations G-420 and G-422 and gated spillway G-422 are part of the operations of the C-4 detention basin. The C-4 Impoundment was built in two phases. The Phase I portion of the impoundment covers an area of about 434 acres and has a storage capacity of approximately 1734 acre-feet. Under this phase, impoundment had an average bottom elevation of 6.0 ft and was designed to a maximum water elevation of 10.0 ft. The Phase II project added 1610 acre-feet of storage to the original

Figure E-5. C4 Emergency Detention Basin and Structures.

impoundment. The total surface area and storage capacity of the impoundment, including phases I and II are 836 acres and 3344 acre feet, respectively.

In the HEC-RAS model, the two cells of the C-4 Impoundment are modelled explicitly as storage nodes. The elevation-storage relationship for each cell was developed from the Nexrad rainfall data with active storage from elevation 6.0 ft NGVD to 10.0 ft NGVD which represents the top elevation of the perimeter levee. The pump stations G-420 and G-422 were not modelled explicitly during model calibration; instead, measured pumpage at each pump station was imposed in the model as internal flow boundary condition at each pump location. In the production version of the model used to simulate water levels for design storm events, the model includes the two pump stations explicitly with the operational parameters (ON and OFF triggers) as defined in the C-4 Impoundment Operational Plan. The pump operations are summarized next.

The G-420 pump with a rated capacity of 669 cfs is operated to pump water from the C-4 canal though the supply canal into the impoundment. The G-420S seepage structure, part of the detention basin, pumps water from the seepage canal into the C-4 emergency detention basin for surrounding area flood control. The structure G-422 is a seven unit pumping station with rated capacity of 622 cfs. The total inflow capacity of G-420 and G-422 is 1,292 cfs. The G-421 gated spillway structure is the outlet for the C-4 Emergency Detention Basin. This structure is used to release water from the detention basin after the flood water has been released in the C-4 canal.

The Feeder Canal to the impoundment from the C-4 canal consists of a 3,800 ft long supply canal located on the eastern boundary of the impoundment. The pump station is turned off whenever any of the following conditions occur:

- The S-25B forward pump station is turned off, or;
- The stage measured at the pump station recedes below elevation 1.2 ft, or;
- The S-26 spillway has excess available capacity.

S-25B Structure

The S-25B structure on the C-4 canal is a 44-ft long gated spillway with invert at -7.9 ft NGVD controlled by telemetry. The purpose of the structure is to maintain an optimum head water elevation of 2.8 ft in the C-4 canal during normal conditions. This is accomplished by opening the structure when the headwater stage of S-25B reaches elevation 3.0 ft and closes when the stage falls below elevation 2.0 ft. For flood control the gate opening setting is lowered to elevation 2.0 ft and the gate closing setting is set at elevation 1.0 ft. During high tide conditions, the water level on the downstream side of the structure can rise above that on the headwater side resulting in flow moving upstream of the C-4 Canal through the structure. These negative flows can significantly reduce the ability of the system to drain by gravity the upper portions of the basin, particularly during severe storm events. The S-25B Forward Pump described next is used in conjunction with the spillway in order to boost the structure’s total capacity during high tide conditions. In the HEC-RAS model, this structure is modelled explicitly using the District’s flow rating equation developed at this site (**Imru & Wang, 2004b**). Flows through the structure are computed using measured gate openings during model calibration and rule-based gate operations in the production version of the model for design storm events.

S-25B Forward Pump Station

The S-25B Forward pump is located just downstream of the S-25B spillway and is operated when the spillway gravity capacity is exceeded during high tide conditions downstream of the structure. The pump station operates to maintain an upstream stage of 1.0 to 1.4 ft NGVD. The structure consists of three 54-inch submersible pumps with rated capacity of 200 cfs each. The pumps are operated when:

- The stage at gauge T5W is above elevation 4.0 ft NGVD, and;
- the gravity discharge through the spillway of structure S-25B is reduced below 600 cfs due to high tide conditions, and;
- The downstream tidal stage at gauge MRM1 is above elevation 4.75 ft NGVD, and;
- The stage measured at the pump station is above elevation 1.4 ft NGVD.

The pump station is turned off whenever any of the following conditions occur:

- The water level at gauge T5W has receded below stage 1.4 ft NGVD, or;
- The stage measured at the pump station recedes below elevation 1.0 ft NGVD, or;
- The S-25B spillway has excess available capacity.

Similar to the forward pumps at structure S-25B, pumps at structure S-26 in the C-6 Canal have been installed to boost the discharge capacity of structure S-26 during periods of high tide when the S-25B forward pumps are turned on. This pump station also has a total discharge capacity of 600 cfs and is operated whenever all the following conditions occur concurrently:

- The S-25-B forward pumps are discharging, and;
- The stage measured at the pump station is above elevation 1.7 ft.

Discharges from the S26 spillway and pump station in the C-6 Canal can affect the tailwater elevation of structure S-25B, therefore, it is recommended that this structure be included in the next version of the C-4 Basin model. In the current version of the C-4 Basin HEC-RAS model, this pump is modelled as a boundary flow by imposing observed pumpages during the calibration runs. The production version of the model operates the pumps according to the above rules when simulating the design storm events.

Stormwater Drainage Municipal Pumps

In addition to these regional projects, several municipalities have constructed and currently operate pump stations that collect and discharge stormwater runoff from the local drainage network of exfiltration trenches to the primary drainage system. The flour municipalities that have constructed such drainage facilities include the cities of Miami, West Miami, Sweetwater and Belen. **Figure E-6** below shows the location of these pump stations in relation to the basins and primary drainage system while **Table E-2** shows the total pumping capacity of each municipal project. **Table G-5** in **Appendix G** lists the operating threshold elevations for these pumps.

Currently the agreement between the District and the municipalities to operate the system during flooding conditions only limits the municipal pumps from operating after the C-4 Impoundment has reached full capacity, at which time all municipal pumpage stops until there is enough capacity in the system.

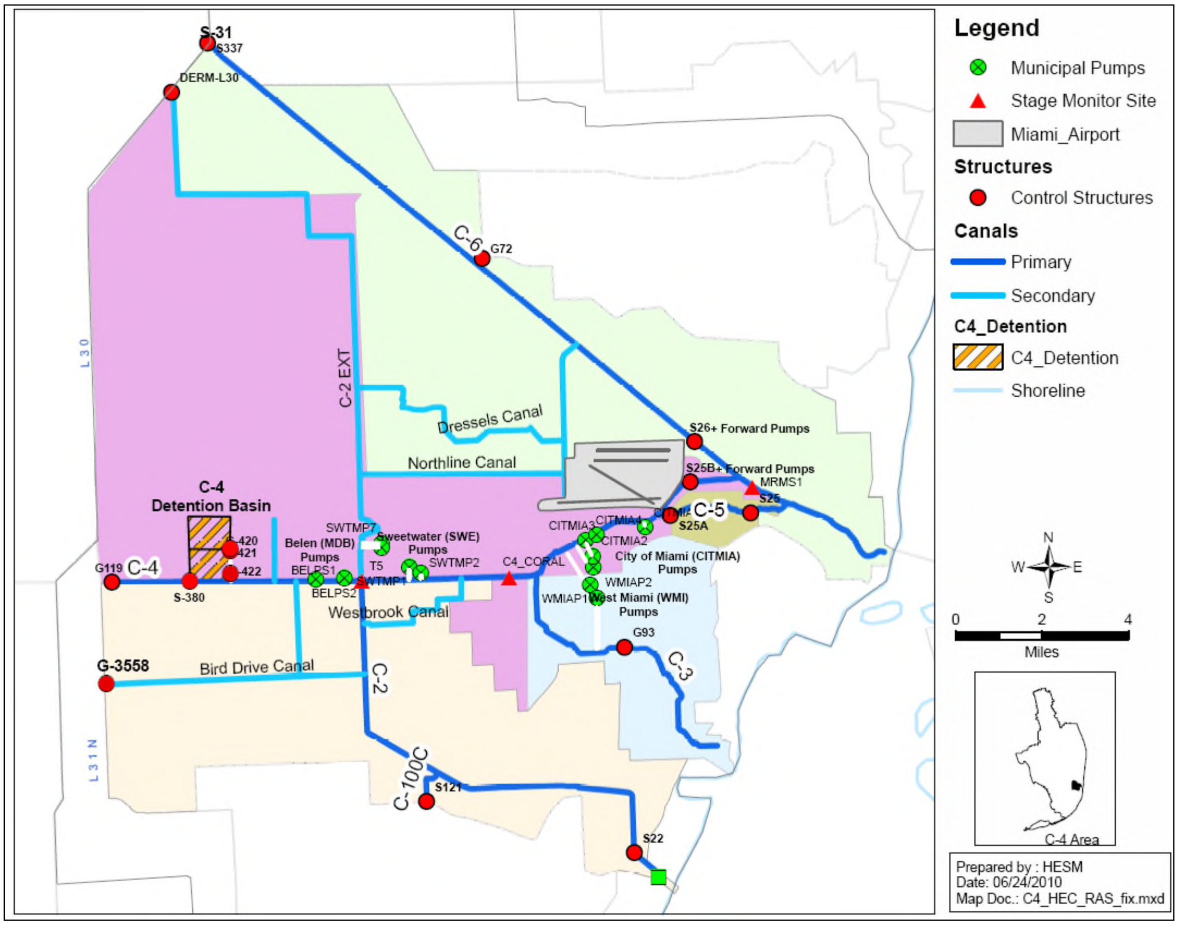

Figure E-6. Location of Flood Mitigation Structures in the C4 Basin

Table E-2. Summary of Flood Mitigation Project Flood Control Structures for the C-4

	ID	Municipality	Structure Type	Basin	Total Discharge Capacity (cfs)
Municipal Projects	MIA	City of Miami	Pump: to C4	C-4	200
	WMI	West Miami	Pump: to C-3, C-4	C-3,C-4	200
	SWE	City of Sweetwater	Pump: to C4	C-4	150
	MDB	City of Belen	Pump: to C4	C-4	200

C-2 Basin Drainage Features

The flood control operations in the C-2 Basin are carried out by operating the S-22 gated spillway located on the C-2 Canal about 1.5 mile upstream of the canal's end at Biscayne Bay. The structure is operated during flood events to maintain a stage of 2.9 ft NGVD on the headwater side of the structure. The automatic gate settings are 3.5 ft NGVD to open the gates and 2.5 ft NGVD for closing. In the calibration version of the HEC-RAS model, flows are computed using observed gate openings imposed as internal boundary conditions at the structure and the standard HEC spillway equations internal to the model. For production runs of design storm events, the control rules for the gates are used to compute the flow through the structure.

C-3 Basin Drainage Features

The G-93 spillway is located at the intersection of the C-3 Canal and Red Road in Miami-Dade County. During storm events, the G-93 structure is a manually operated in conjunction with the S-25B structure to maintain a stage of 2.8 ft NGVD on the headwater side of the structure. The structure has a maximum flow capacity of 640 cfs. In the calibration version of the HEC-RAS model, flows are computed using observed gate openings imposed as internal boundary conditions at the structure and the standard HEC spillway equations internal to the model. For production runs of design storm events, the control rules for the gates are used to compute the flow through the structure.

C-5 Basin

There are two water control structures in the C-5 basin: the S-25 and S-25A structures Drainage from the C-5 Basin only affects the C-4 canal at the downstream side of structure S-25B. During flood events, structure S-25A is closed preventing flows from the C-4 Basin to enter the basin. Structure S-25 is a single barreled metal culvert with a slide gate. The structure is operated under normal conditions to prevent flooding and saltwater intrusion by maintaining a headwater stage of 2.0 ft. This is accomplished by setting the open elevation at 2.2 ft and close at 1.8 ft. There is an automatic override that closes the structure whenever there is an extreme high tide event. The S-25A structure is a manually operated single-barreled metal pipe culvert is operated only for salinity control and is only open when the headwater elevation at structure S25 recedes below elevation 1.5 ft. In the calibration version of the HEC-RAS model, flows are computed using observed gate openings imposed as internal boundary conditions at the structure and the standard HEC pipe equations internal to the model. For production runs of design storm events, the control rules for the gates are used to compute the flow through the structure.

Table E-3 summarizes the physical dimensions and operational parameters for the canal structures in the C-2, C-3, C-4 and C-5 basins, part of this study.

Table E-3. Dimensions and Operational Parameters for Control Structures

STRUC	TYPE	DIMENSIONS	Qmax (cfs)	WET SEASON (JUNE – NOVEMBER) OPEN/ON ft-NVGD	WET SEASON (JUNE – NOVEMBER) CLOSE/OFF ft-NGVD	OPT HW STAGE ft-NGVD
S-336	Gated culvert	Triple slide gated 54" x 85'culverts with invert elevation at -1.8 ft NGVD	145			3.0 ft NGVD at T5 gauge during flood releases to tide. 2.0 ft in dry season
G-119	Gated culvert	Five 72-inch CMP culverts with slide gates	400	For water supply open when stage at S-25B ≤ 2.0 ft in dry season. For flood releases from WCA-3A open only if stage at gauge T-5 < 3.0 ft	Close when stage at S-25B > 2.0 ft in dry season. For flood releases from WCA-3A close if stage at gauge T-5 > 3.0 ft.	2.0 ft in dry season
S-380	Gated Culvert	Five 72-inch diameter CMP culverts with slide gates	400	OPEN (Dec-May) 4.2 ft (Jun-Nov)	OPEN (Dec-May) 3.8 ft (Jun-Nov)	4.0 ft
G-420	G-420P	Pump Station	669	Q = 669 cfs if stage at T-5 > 4.8 ft or;	If stage at T-5 rises to 6.0 ft pump until T-5 begins to recede or;	If stage in the detention basin =8.0 allowing stage at T-5 to rise to 6.0 ft or; If stage at detention basin ≥ 8.0 ft and stage at T-5 begins to recede or; Stages upstream of G-420 pump ≤ 4.2 ft or; Stages at T-5 show receding trend or; Stage in the detention basin ≥ 10.0 ft or; Stage at the pump or S-25B TW < 1.0 ft
G-421S	Gated Spillway			If stage at T-5 recedes to elevation 5.5 ft and the storm has passed.	Until the Detention Basin is discharged completely	
G-422P	Pump Station		623	Q = 267 cfs if stage at T5 > 5.0 ft Q = 623 cfs if stage at T5 > 5.2 ft	OFF if stage at detention basin ≥ 8.0 ft and stage at T5 begin to recede or; Stage upstream of G-420 pump ≤ 4.2 ft or; Stages at T5 show receding trend or; Stages in the detention basin ≥ 10.0 ft	
S25B-PUMP	Pump Station	3x200 cfs pumps	600	stage at T5 > 4.0 ft and; S-25B Q < 600 cfs due to high tide and; Stage at the pump or S-25B TW > 1.4 ft	S-25B Q > 600 cfs due to high tide or; stage at MRM1 ≥ 4.75 ft Stage at the pump or S-25B TW < 1.0 ft	1.0 – 1.4 at S-25B TW
S-25B	Gated spillway	Crest L = 44 ft Crest Elev.= -7.9 ft	2000	2.0 ft	1.0 ft	1.5 ft
G-93	Gated spillway	Pump: C4 to reservoir	640	2.0 ft at S-25B HW	1.0 ft at S-25B HW	1.5 ft
S-22	Gated spillway	Crest L = 34 ft Crest Elev.= -11.0 ft	1915	3.5 ft	2.5 ft	2.9 ft
S-25	Gated culvert	Single barreled CMP culvert	100	2.2 ft	1.8 ft	2.0 ft
S-25A	Gated culvert	Single barreled CMP culvert	N/A	closed	closed	1.5 ft at S-25 HW

Groundwater Seepage and Canal Flow Interaction

Groundwater levels in the C-4 Basin are closely related to surface water levels due to the high hydraulic conductivity of the surficial aquifer system (SAS). Additionally, the SAS and canal system are extremely well connected in the C-4 portion Miami-Dade County due to the canals penetrating into the highly permeable Biscayne aquifer. Seepage from the WCAs can also occur west to east as underflow from the WCAs through the protective levees that separate the natural and urban areas. These discharges are in part intercepted by canals (L-30 and L-31 Canal) and wellfields located in the eastern, urbanized areas. The high aquifer transmissivities allow a tertiary drainage network of exfiltration trenches that penetrate the cap rock, routing surface water into the canal via the surficial aquifer. In general, drainage in the secondary canal system is limited by the available capacity in the primary canal system.

Appendix E: Flood Model Development and Quality Assurance

In order to avoid dependence in this study on a groundwater flow model to estimate the groundwater levels in the C-4 basin, a simplified approach has been proposed to conceptualize groundwater exchange between sub-basins and between sub-basins and canals. In this approach, the exchange of water between water bodies is a function of the head differential times a linear coefficient that represents a flow conductance as determined by model calibration. Flow between two adjacent sub-basins is computed as:

$$Q_{bb} = K_{bb}\, H\, \Delta h \qquad \text{(E-1)}$$

Where:

- Q_{bb} = Groundwater flow rate between two storage sub-basins, $ft^3/sec/ft$
- K_{bb} = Sub-basin-to-sub-basin conductance, ft/sec
- H = $h_{us} – B$, ft
- Δh = $h_{us} - h_{ds}$, ft
- B = Aquifer bottom elevation, ft NGVD
- h_{us} = Water elevation of the upstream sub-basin, ft NGVD
- h_{ds} = Water elevation of the downstream sub-basin, ft NGVD

The computed flow rate with equation (F-1) is per unit width of the length of contact between the two water bodies. The total flow rate is then,

$$Q_{bb\ total} = Q_{bb}\ W \qquad \text{(E-2)}$$

Where:

- W = Length of interface between the two sub-basins

Similarly, the flow rate between a sub-basin and a canal can is computed as follows.

$$Q_{bc} = K_{bc}\, H\, \Delta h \qquad \text{(E-3)}$$

Where:

- Q_{bc} = Groundwater flow rate between sub-basins and canal, $ft^3/sec/ft$
- K_{bc} = basin-to-canal conductance, ft/sec
- H = $h_{us} – B$, ft
- Δh = $h_{us} - h_{ds}$, ft
- B = canal bottom elevation, ft NGVD
- Δh = $h_{us} - h_{ds}$, ft
- h_{us} = Water elevation of the upstream sub-basin, ft NGVD
- h_{ds} = Water elevation of the downstream canal, ft NGVD

The computed flow rate with equation (F-3) is per unit width of the length of contact between the sub-basins and the canal reach. The total flow rate is then,

$$Q_{bc\ total} = Q_{bc}\ W \qquad \text{(E-4)}$$

Where:

- W = Length of contact between the sub-basin and the canal

Figure E-7 shows the conceptual representation of the Basin-to-Basin and Basin-to-Canal flow exchanges in the HEC-RAS model of the C-4 Basin. In this figure, the nodes represent the sub-basins and canal water bodies, the red dashed lines the groundwater flow exchange between sub-basins and canals and the black dashed lines, the basin to basin groundwater flow connectors.

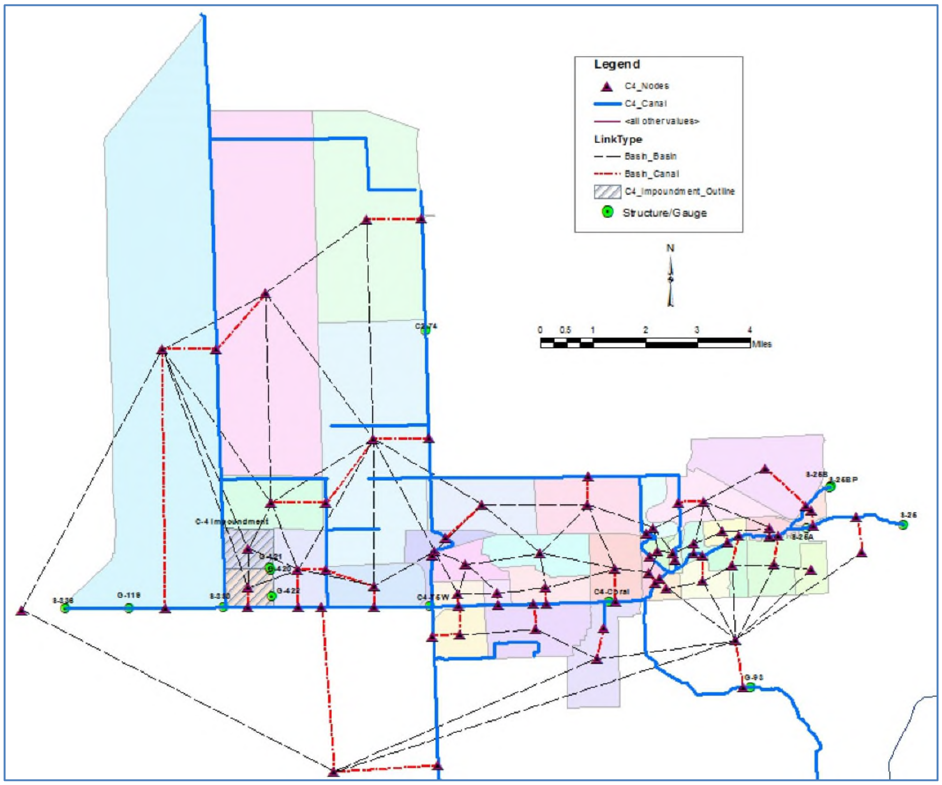

Figure E-7. Basin-to-Basin and Basin-to-Canal Flow Exchanges in the C-4 Basin Model

Summary of Model Assumptions

A summary of modeling assumptions is shown in **Table E-4.**

Table E-4. Summary of Model Assumptions

Feature	Assumption
Vertical Datum	All elevations in the models and operational parameters are referenced to NGVD29 vertical datum
Period of Simulation	Event based simulation Model calibration: August 7 to September 30, 2012 Model validation: Hurricane Irene (October 14-15, 1999) LOS evaluation runs: 5-, 10-, 25- and 100-yr storm event
Climate	15-min NEXRAD Rainfall data Daily Evaporation data SFWMD rainfall distributions used for the 5-, 10-yr 24-hr, 25-yr 72-h and 100-yr 72-hr events.
Topography	Ground elevations dataset is from LIDAR 5 ft resolution (SFWMD, 2015b) Canal cross-section from previous District and DERM surveys
Hydrology	For HEC-RAS model calibration, validation and LOS event simulations assumed rainfall uniformly applied over sub-basins. Rainfall enters into HEC-RAS sub-basin storage with no time lag. ET was subtracted from rainfall in the calibration runs, however, it is neglected in event simulation runs.
HEC-RAS hydraulics	Level-pool assumption holds for water levels within each sub-basin. Flow between storage units and between storage units and canals were routed in HEC-RAS as seepage using a seepage parameter and the head differential between two water bodies. Lateral runoff from sub-basins to canals were routed as seepage and overbank weir flow. Upstream canal boundary condition was seepage flow from the WCA-3A and ENP. Downstream canal boundary conditions were tidal stages.
System Operations	Observed gate openings at gated structures and recorded pumpage were used at pump stations during model calibration runs. For LOS event simulation runs, rule-driven operations were imposed at all control structures as defined in the C-4 Basin Water Control Plan.

C4 HEC-RAS IMPLEMENTATION

Base Case Features

The HEC-RAS model that represents existing drainage conditions and operations of the C-4 Basin was developed from the calibration version of the model modified to include the current water control structures and rule-driven operations of the system. Structures added to the model included the S-22 structure in the C-2 Canal, structure G-93 in the C-3 Canal, structures G-119, S-380, G-420, G421, G-422, S25B and S25B Forward Pump. Municipal pumps added to the model also included pump stations for the City of Belen, City of Sweetwater, City of Miami and City of West Miami. In order to accurately represent the flood control operation for current conditions established in the C-4 Basin Water Control Plan (SFWMD, 2015), the operations of these structures had to be defined using the user-defined rules instead of HEC-RAS default structures. **Appendix D** describe with detail the structure operations. This Section of **Appendix E** describes the implementation of model features in HEC-RAS to simulate the current drainage conditions features of the C-4 Basin.

Sub-basin Inflows

Inflows into the sub-basins in HEC-RAS consist of rainfall and boundary flows. As explained in the previous section of this appendix, the rainfall over a sub-basin was assumed to enter sub-basin storage without time lag. For model calibration, NEXRAD rainfall volume rates were converted to flow rate per 15-minte time steps and entered directly into each sub-basin. There were a total of thirty-nine (39) sub-basin inflows from rainfall, one for each sub-basin. **Figure E-8** shows a picture for the inflow hydrograph for Sub-basin C4_10C, as an example.

Western Flow Boundary Condition

The second type of external inflows into the C-4 sub-basins was seepage from the Water Conservation Area 3A and the Everglades National Park. Zero flow was used at the western end of the C-4 Canal for all model simulations, however seepage flow from the Water Conservation Are 3B flows are computed based on statistically derived water levels on the west side of the L30 Canal/levee and a seepage parameter obtained through model calibration. The water elevations derived for the 25-yr storm event are shown in **Figure E-9** and the seepage flow equation coefficient from the WCA-3B to Sub-basin C4_10A is shown in **Figure E-10**. Similar seepage inflows from the Everglades National Park were estimated as inflows into sub-basin C2.

Sub-basin Storage

Sub-basin total storage includes aquifer and above-ground storage. Underground storage is a function of the soil properties, specifically soil porosity. Details of the methodology used to develop the sub-basins' groundwater elevation versus storage relationships can be found in **Appendix B**. Surface or above-ground storage for each sub-basin was developed using the latest Lidar topographic data from the County. The elevation vs. storage relationships that were entered into HEC-RAS for each sub-basin reflect the combined underground and surface storage. As an example, **Figure E-11** shows the stage versus storage relationship computed for sub-basin C4_75B.

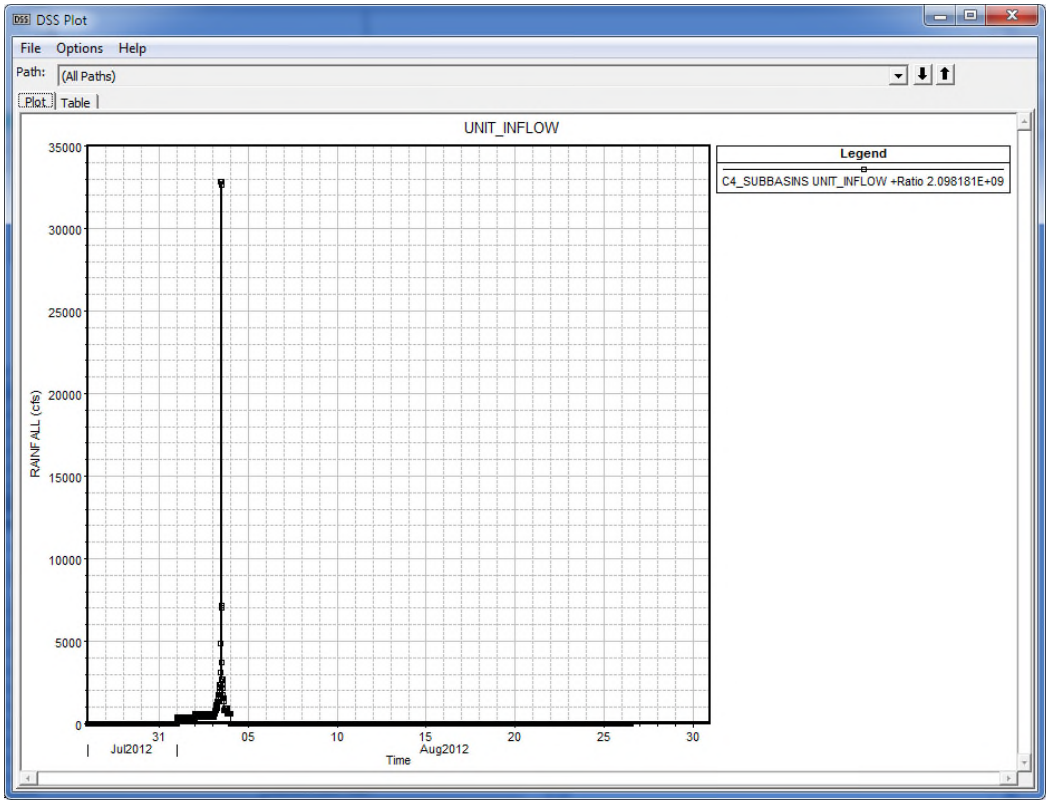

Figure E-8. . Rainfall Flow Rate for Sub-basin C4_10C for the 25-yr Storm Event

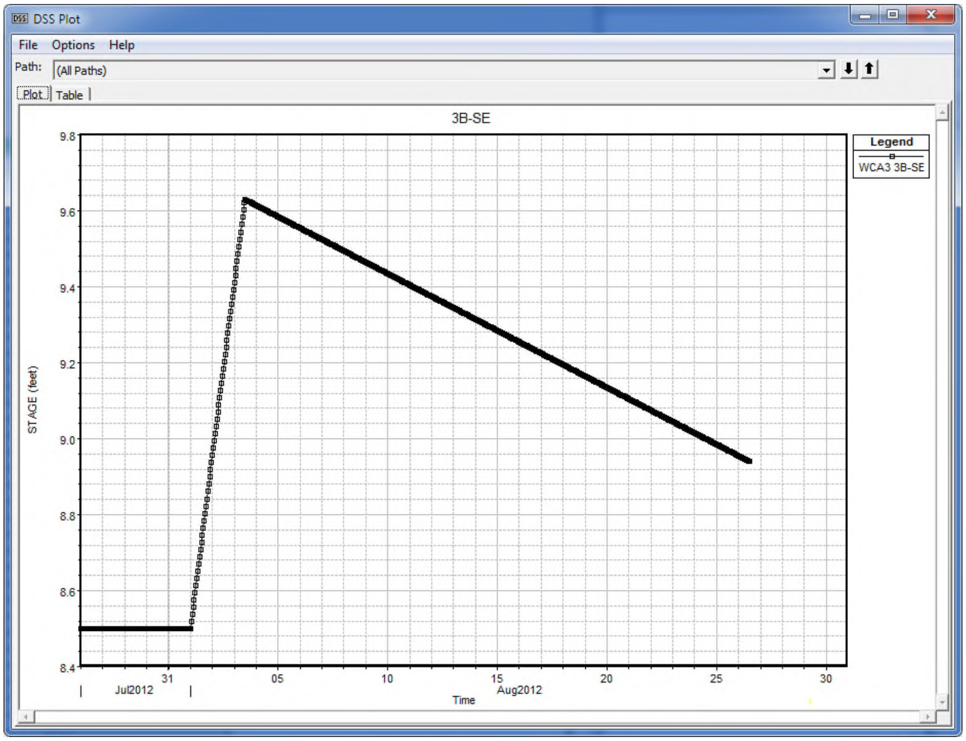

Figure E- 9. Water Levels at WCA 3B

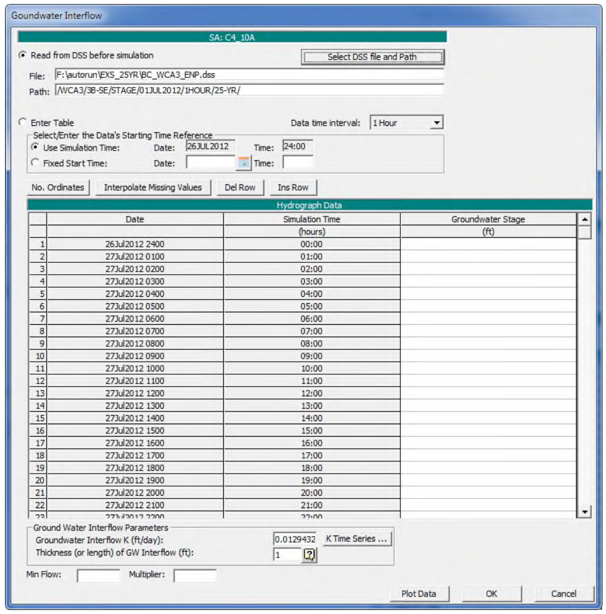

Figure E-10. Groundwater Boundary Flow Equation Coefficient between WCA 3B and Sub-basin C4_10A

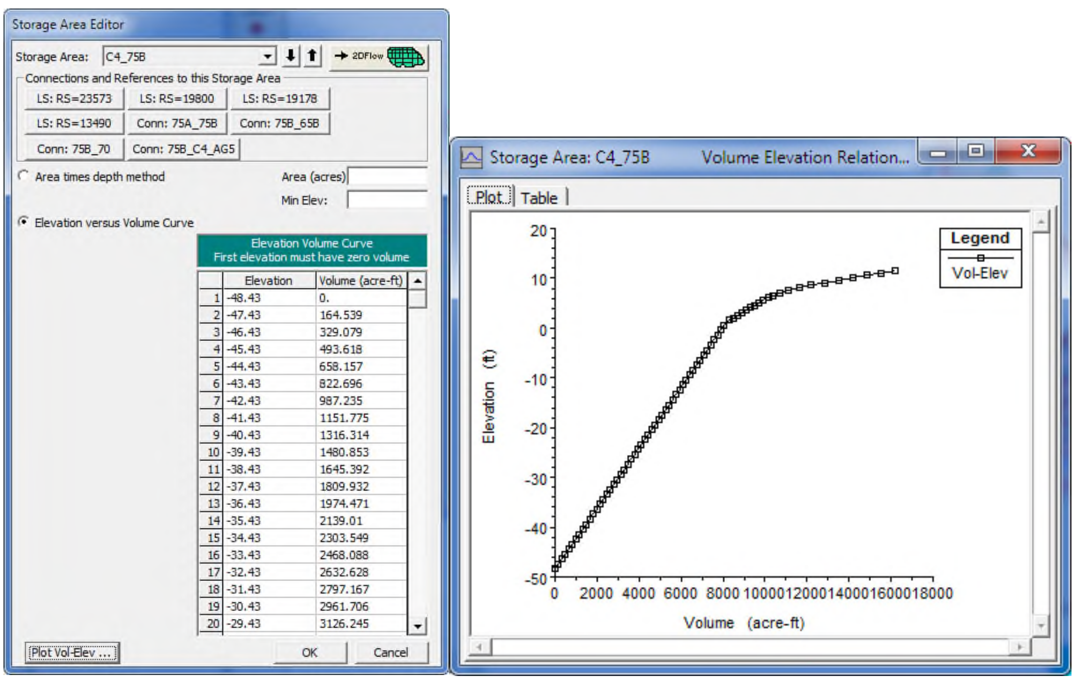

Figure E-11. Stage vs Storage Relationship for Sub-basin C4_75B

Canal Conveyance

Canal carrying capacity or conveyance is defined in HEC-RAS using the geometric properties of the canals and structures. The existing canal geometric data consists mainly of cross-sections along the primary canals, most of which were obtained from as-built drawings and canal surveys by Miami-Dade County (DERM), US Army Corps of Engineers, SFWMD and the Florida Department of Transportation (FDOT). Recent improvements to the C-4 Canal banks by the District (Jackson, 2011) were also incorporated into the canal cross-sectional data for the HEC-RAS model. **Figure E-12** shows the location of the canal cross-sections used in the HEC-RAS model many of which were interpolated from the surveyed canal cross sections as needed by the model to reduce model instability during flow computations.

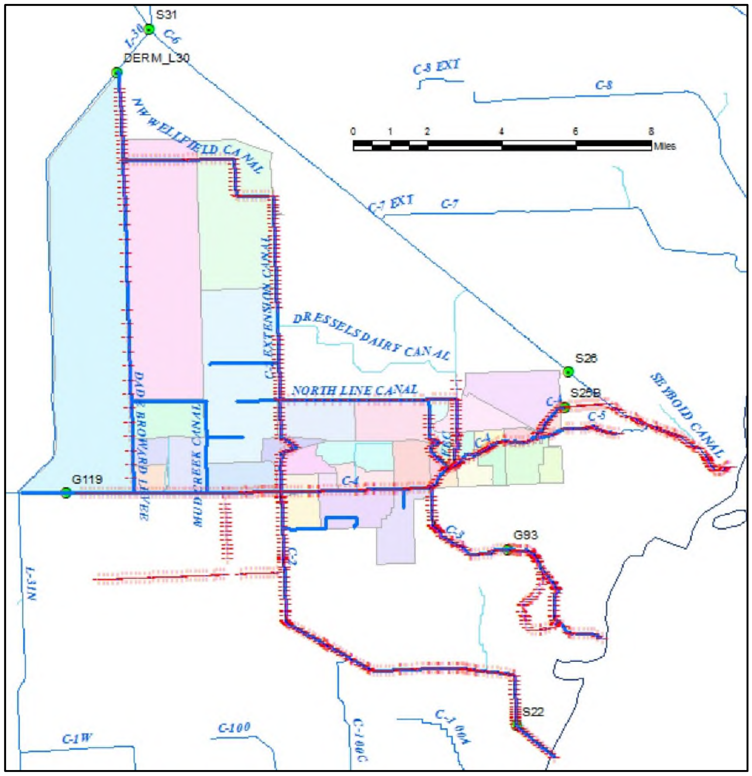

Figure E-12. Location of Canal Cross-Sections

Bridges

Bridges across primary and secondary canals are a common feature in urban areas such as the C-4 Basin. Bridges can have a significant effect in flows and stages in a canal when the bridge structural elements that are in contact with the water produce frictional losses of energy that translate in higher stages on the upstream side of the structure. For significant flood events, bridge head losses can result in bridge overtopping and flow velocities that lead to canal bed erosion. This may compromise the stability of the structure. When HEC-RAS computes the head loss through a bridge that results in higher energy on the downstream section, repeats the calculations with a different method. Currently, HEC-RAS has four methods of computing head losses through bridges: Yarnell, Energy, Momentum and WSPRO (USGS). Computing head losses through bridges in HEC-RAS often requires the application of more than one method if the chosen method does not result in a positive energy loss through the bridge. Depending on the geometry of the bridge components and the canal cross sections, a particular method may not produce realistic head loss. For example, the Momentum method is may produce excessive head losses when there is a significant change of conveyance between the two canal cross sections that bound the bridge. Additionally, the Yarnell method sometimes becomes inaccurate when the energy grade line through the bridge is very flat. Most bridges were implemented for this study using the cross-section geometric information about piers and bridge deck from the surveys. The Yarnell method of computing head losses across bridges was used for the majority of the bridges in the model as the default method. This method was chosen as the default since the head losses are primarily due to the influence of the bridge piers when flow is below the bridge deck (which is usually the case). This method required specification of a drag coefficient and a pier shape factor (**Figure E-13**). In cases where pressure flow occurs through the bridge opening due to submergence of the bridge chord, the energy method was chosen as default (**Figure E-13**).

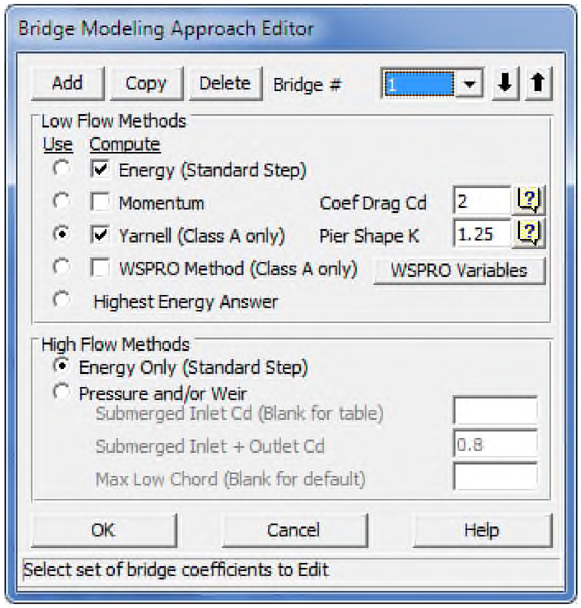

Figure E-13. Parameters for Bridge on C4 Canal at 132nd Avenue

Most bridges along the C-4 canal have been designed to minimize head losses, however, the cumulative bridge head loss effect for the entire C-4 canal needs to be accounted for in flood analyses. The current condition version of the model includes twenty-one (21) bridges in the C-2, canal, seven (7) in the C-3 Canal and twenty-two (22) in the C-4 canal. Most of the bridge geometric information in this version of the model was obtained from Miami-Dade County and the Florida Department of Transportation. **Figure E-14** shows the location of bridges in the current condition HEC-RAS model while **Figure E-15** shows an example of a bridge located in the C-4 Canal at 132nd Avenue.

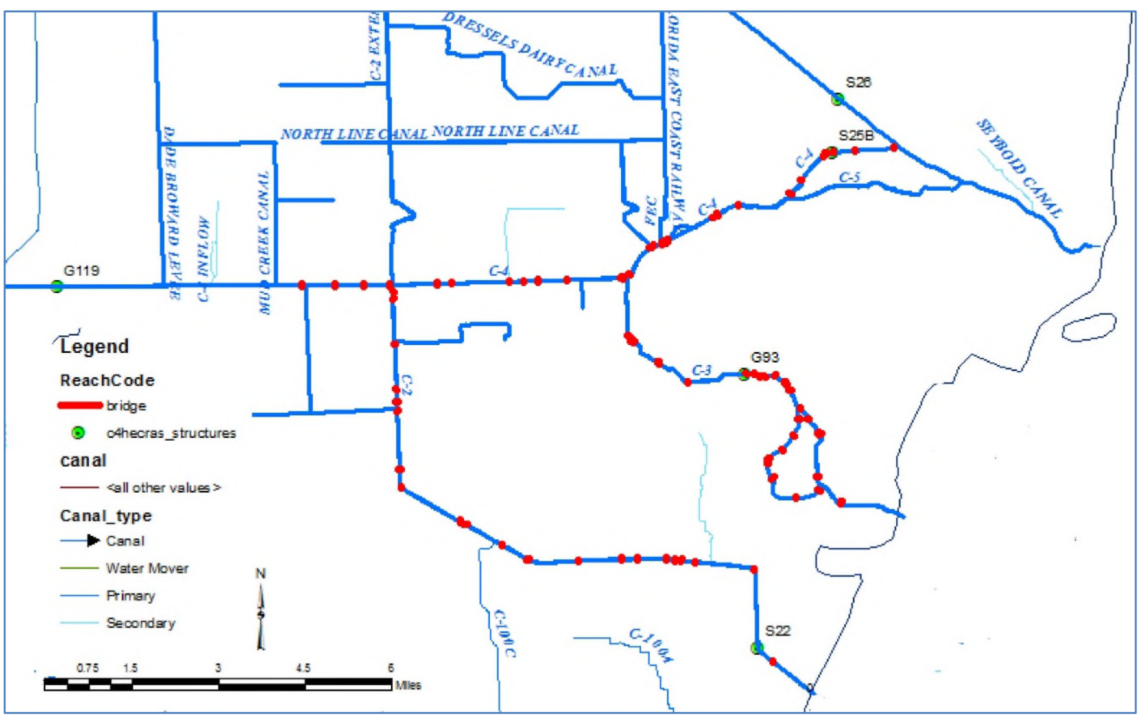

Figure E-14. Location of Bridges in the C-2, C-3 and C-4 Canals

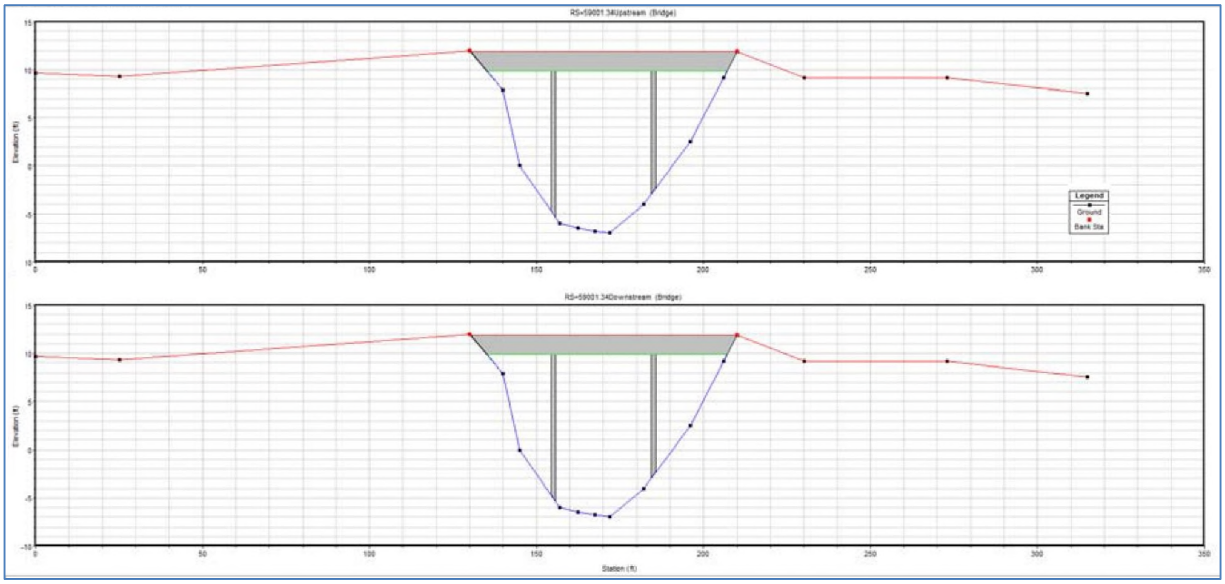

Figure E-15. Geometry of Bridge on C4 Canal at 132nd Avenue

Gated Structures

Flood control operations in the C-2, C-3 and C-4 Canals are carried out by the District by means of adjusting gate openings of several gated spillways and culverts in these canals. Currently the District operates three gated structures in the C-4 Canal two of which are gated culverts (G-119 and S-380) and the S25B tidal spillway. The physical characteristics of this and all other structures in the study area were briefly discussed and summarized previously in **Table E-3** of this report. In the C-2 Canal the only structure simulated is the S-22 tidal spillway and, in the C-3 canal, the G-93 tidal spillway.

Typically, in HEC-RAS, each structure is entered using the geometrical characteristics of the structure which includes the number and size of the gates or culverts as well as the elevation at which the structure is overtopped, if flow over the structure is to be simulated. **Figure E-16** is an example showing the characteristics of the S-25B structure and the gates' discharge and orifice flow discharge coefficients.

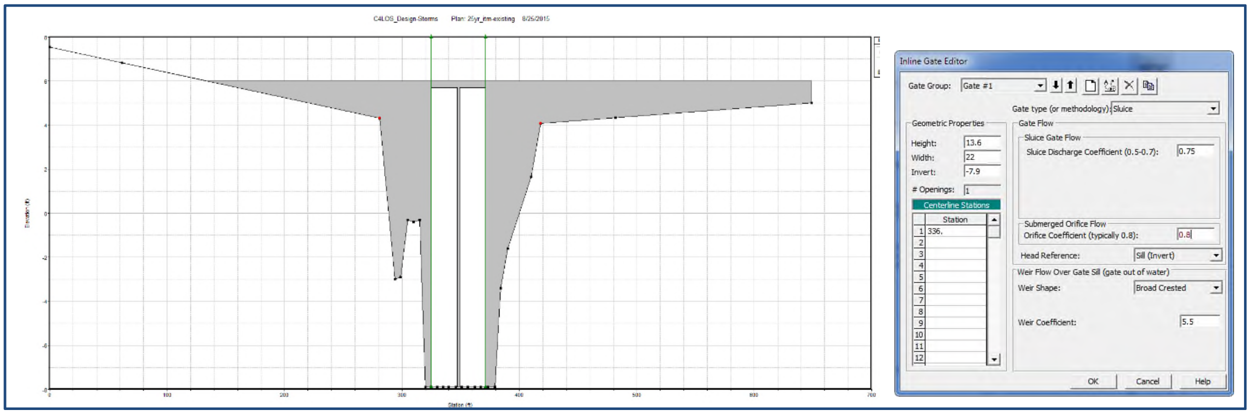

Figure E-16. Geometry of Gated S-25B Structure on C4 Canal and Flow Coefficients for Gate 1 at Structure S-25B.

In this study, however, the generic HEC-RAS flow computational methods were not followed. Flow rating equations developed by the District at structures S-25B, S-22 and G-93 were used instead.

These flow rating equations account more accurately for flow reversals at the structure particularly when the tailwater stage rises above the headwater due to high tide. A more detailed description of these flow rating equations is presented in **Appendix D**.

Pump Stations

S-25B Forward Pump

Several pump stations are used in the C-4 Basin during flood events. In the C-4 Canal, at the S-25B structure a pump station is used in conjunction with the S-25B spillway to increase the capacity of the spillway during high tide conditions. As typical of tidal structures in South Florida, the S-25B spillway is subject to high tide water levels which can reduce or completely suppress the ability of the spillway to discharge by gravity. At these times, the S-25B Forward Pump is activated to supplement the gravity discharge of the spillway with a capacity of up to 600 cfs. A schematic representation of the S-25B Forward Pump in HEC-RAS is shown in **Figure E-17**. The operation of this pump station is coordinated with operations of the C-4 Emergency Detention Basin (EMD) in the western portion of the C-4 Basin. Details of the operation of the forward pump and the emergency detention basin can be found in **Appendix D**.

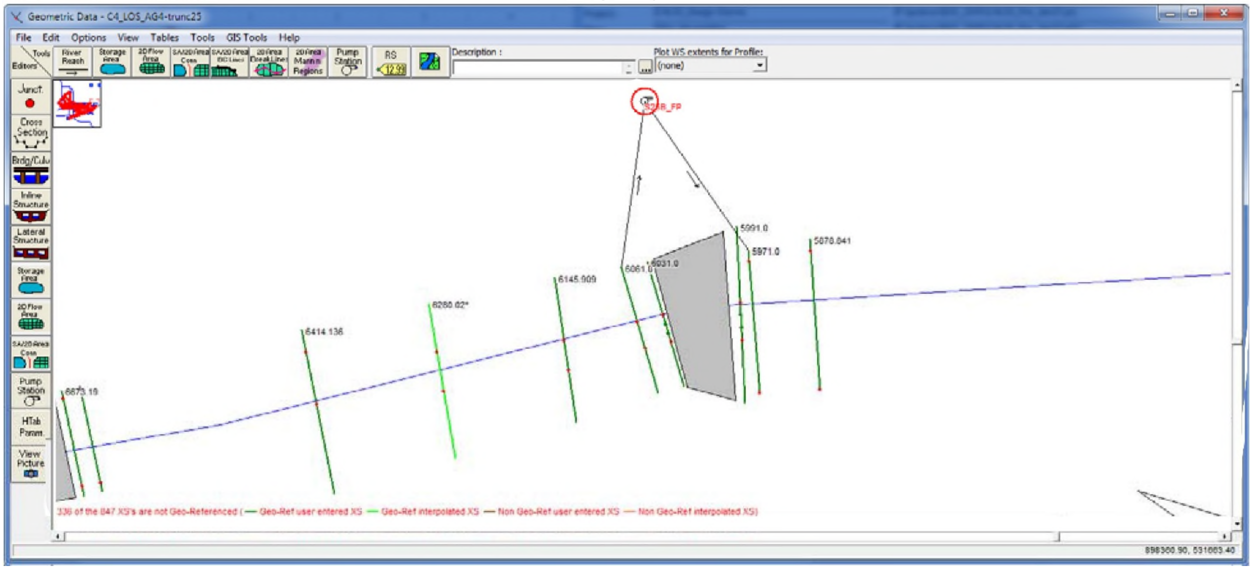

Figure E-17. Schematic of Forward Pump at Structure S-15B

G-420 and G-422 Pump Stations

In the western C-4 Canal, the C-4 Emergency Detention Basin is used during flood events to store stormwater runoff from the C-4 Canal west of the Turnpike via two pump stations (G-420 and G-422) and the operation of the S-380 gated culvert. By lowering the stage in the portion of the C-4 canal between the City of Belen and Sweetwater, stormwater runoff from these two municipalities can be discharged to the C-4 Canal via pump stations a faster rate than it was discharged prior to the constructions of the municipal pumps. Other municipalities such as West Miami and the City of Miami also benefit from additional canal storage via pump stations as part of the C-4 Emergency Detention Basin operations. All municipal pumps are allowed to discharge to the C-4 Canal as long as there is capacity in the C-4 EMD. Once the C-4 EMD reaches full capacity at elevation 10.0 ft NGVD, all municipal pumpage ceases until the EMD is drained and capacity available.

The location of the G-420 and G422 pump stations was shown previously in **Figure E-5** and their operation was described in **Table E-3**. The implementation of the G-420 and G-422 pump stations in HEC-

RAS followed the standard pump station setup through the HEC-RAS GUI. **Figure E-18** shows a schematic of the HEC-RAS model of the C-4 EMD, Feeder Canal and the two pump stations G-420 and G-422. The two pump stations were set up as typical pumps in HEC-RAS by defining the pumps' efficiency curves. **Figure E-19** shows the pump efficiency curve for pump station G-420 with a maximum capacity of 669 cfs. As required in the C-4 Emergency Operation Plan, the operation of the G-420 and G-422 pumps does not follow the standard pump ON and OFF triggers provided by HEC-RAS. Instead, a user-defined rule was entered in order to accommodate the more complex operation of the G-421 and G-422 which requires multiple trigger locations and stage conditions in the C-4 canal and within the EMD. A detailed description of the G-420 and G-422 pumps operations can be found in **Appendix D** of this report.

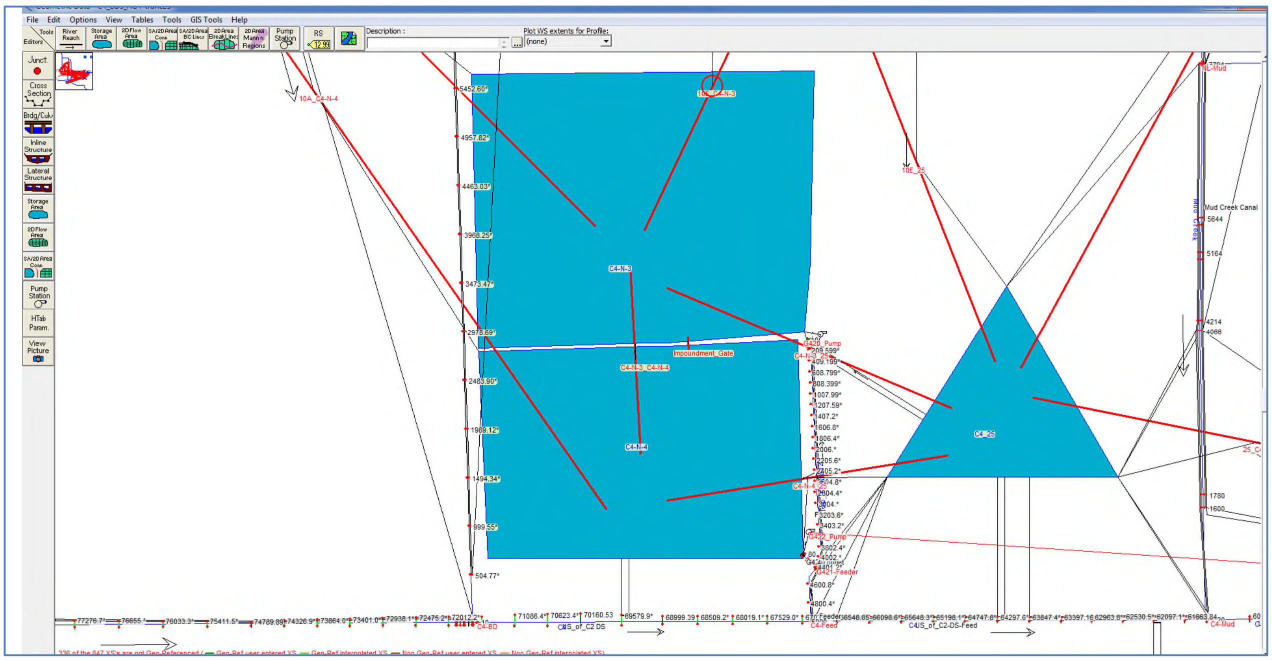

Figure E-18. C-4 EMD, Feeder Canal and Pump Stations G-420 and G-422 in the HEC-RAS model

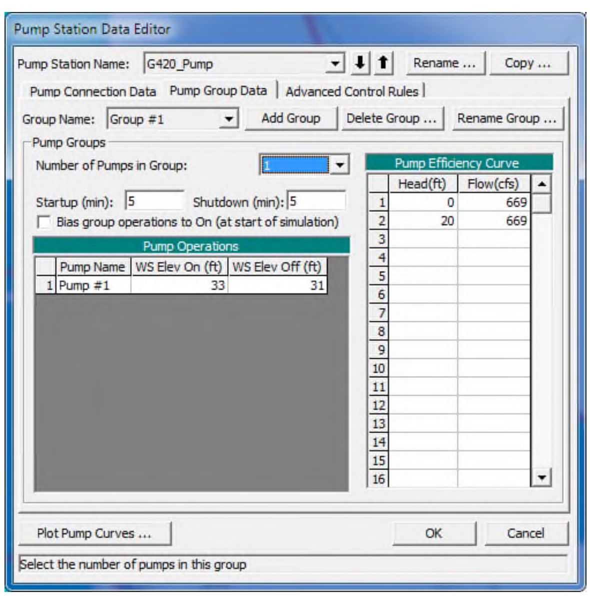

Figure E-19. G-420 Pump Efficiency Curve in HEC-RAS

Canal Inline Storage

Canal in-line storage along the C-4 Canal consists of several drainage features that can provide significant peak flow attenuation during large events, due to their relative size compared to the storage in the C-4 Canal reaches. These areas were dug as borrow pits with depths of up to 50 ft. Most of them are located between the S-25B tidal structure and the Palmetto Expressway. The three largest storage areas on the C-4 Canal are Maule Lake, Blue Lagoon and Angler Lake as shown in **Figure E-20**.

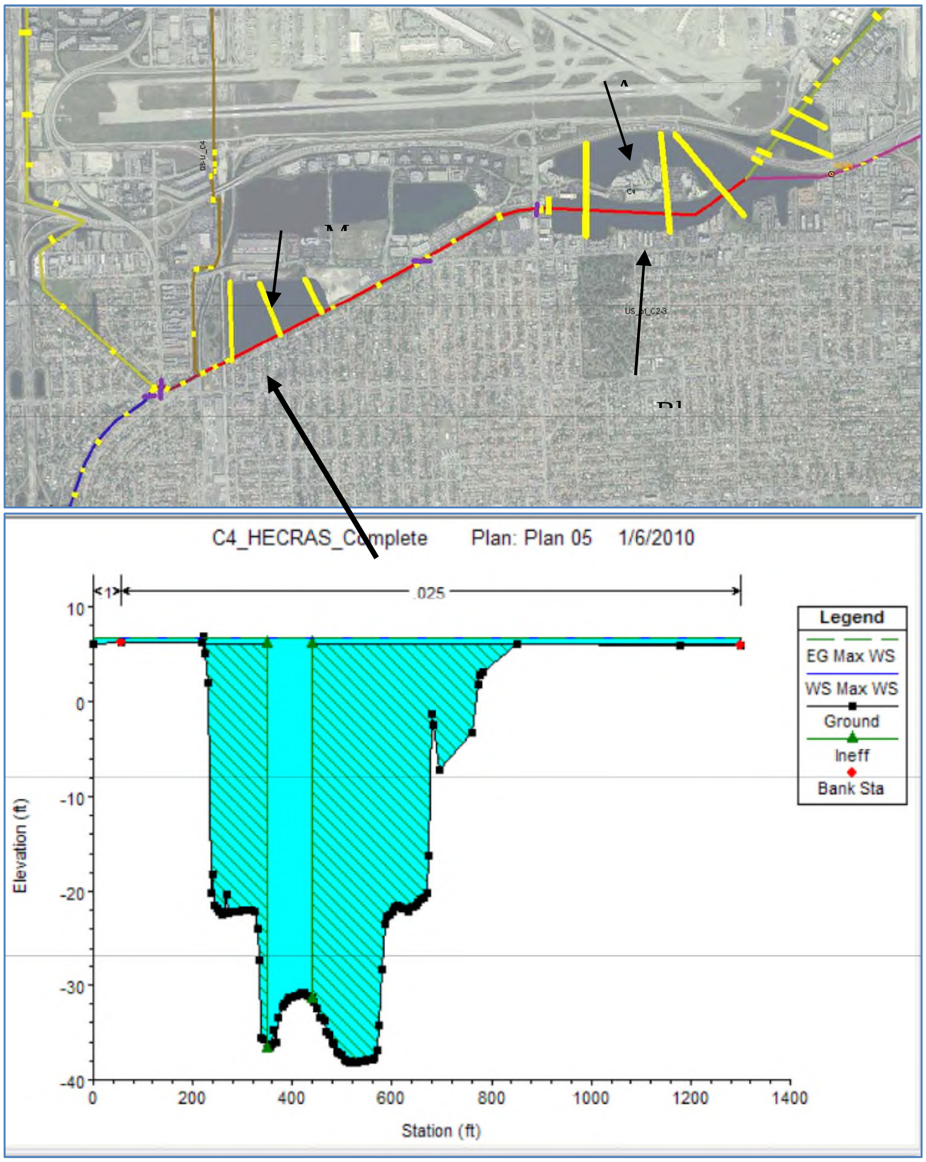

Figure E-20. C-4 Canal Cross-section in Maule Lake

In this investigation, these canal storage features were characterized by the geometry of the canal cross-sections at several locations across the lakes. An example of one of the canal cross-sections near the middle of Maule Lake. In the HEC-RAS model the conveyance is reduced to account only the cross-sectional area of the canal, however, the storage is computed using the entire cross-sectional area of the lake.

Basin to Basin Seepage Flow Interaction

Flows between sub-basins in the model were conceptualized as groundwater seepage as described previously in this report and shown in **Figure E-6**. Each of the fifty-five (55) sub-basin-to-sub-basin seepage equations were implemented in HEC-RAS using the user-defined rules which accommodate unique seepage groundwater flow coefficients for each connector. As an example, **Figure E-21** shows the seepage connector between sub-basins C4_10A and C4_10E while **Figure E-22** illustrates the user-defined seepage equation between these two sub-basins.

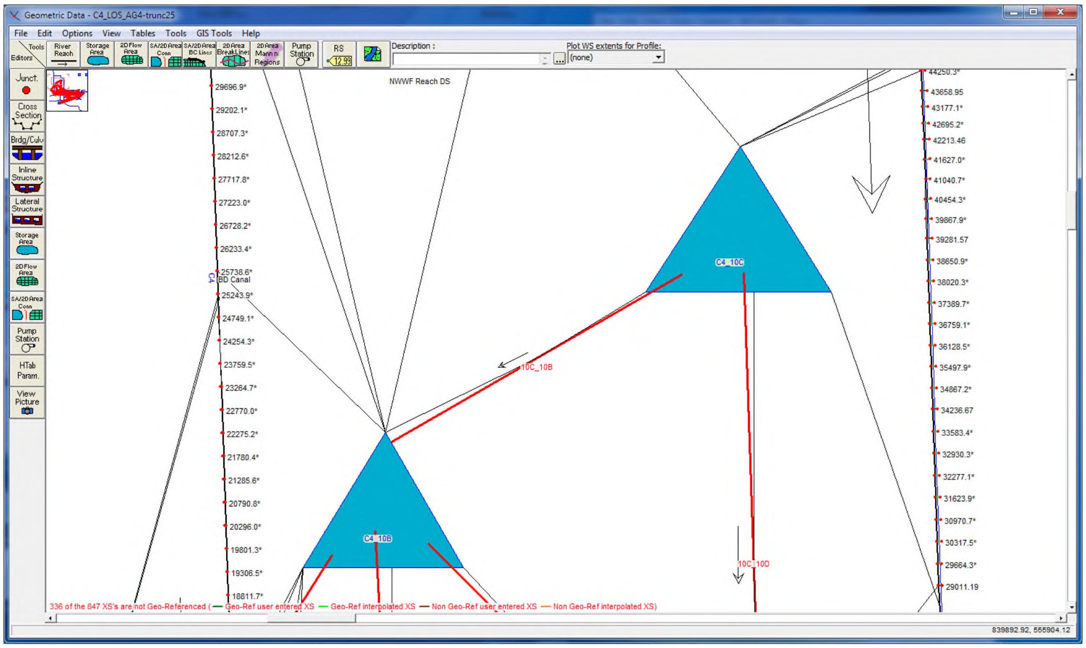

Figure E-21. Basin-to-Basin groundwater Connector between Sub-Basins C4_10B and C4_10C

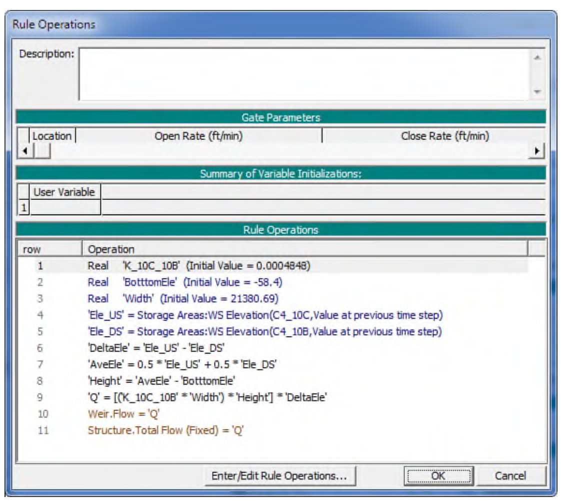

Figure E-22. Groundwater Equation for Flow between Sub-basin C4_10B and C4_10C

Sub-basin to Canal Flow Interaction

Seepage flow between sub-basins to canals in the model were implemented in a similar manner as the basin-to-basin seepage connectors with the main difference in the seepage equation which uses the bottom of the canal to calculate the area of contact between the sub-basin and the canal reach for seepage calculations. There are forty-two (42) sub-basin-to-canal connections in the model for which the seepage coefficient values were determined in the model calibration process. In the case of sub-basin-to-canal seepage connectors, the user-defined rules were implemented in HEC-RAS using the traditional lateral structure (overflow weir) whose discharge was over-ridden with the seepage flow value computed by user-defined equation. As an example, **Figure E-23** shows the seepage connector between sub-basin C4_10C and the C-2 Extension canal. **Figure E-24** shows the corresponding sub-basin-to-canal seepage user-defined equation.

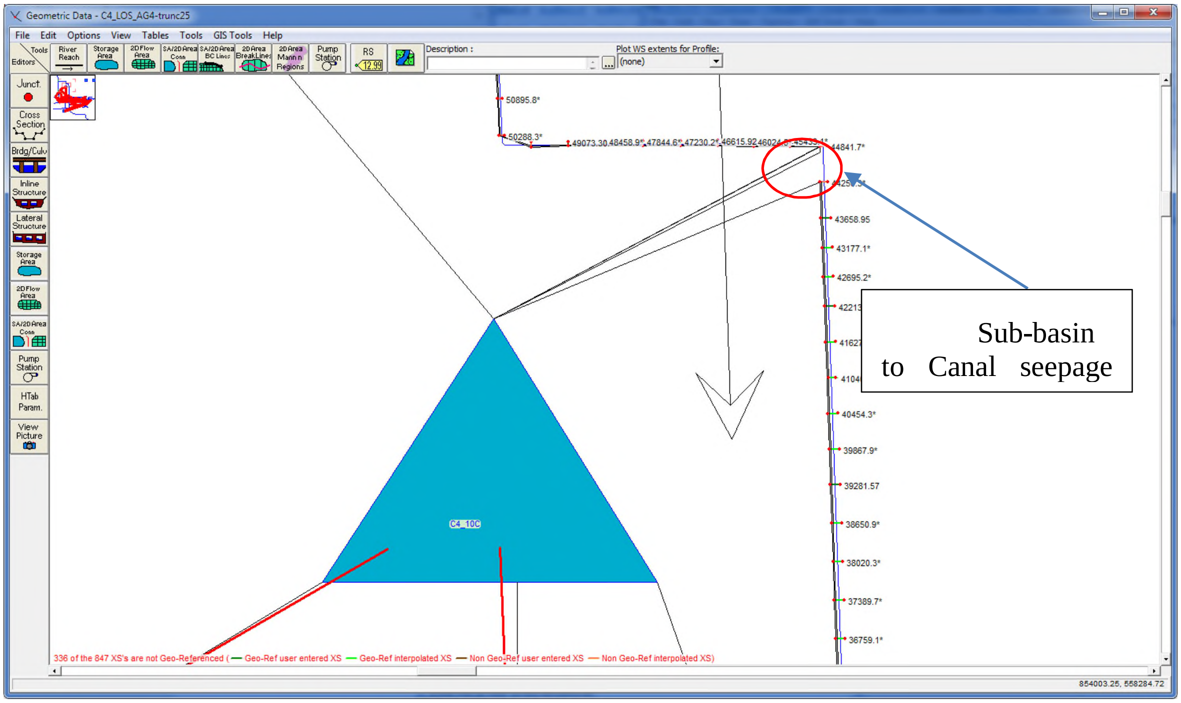

Figure E-23. Groundwater Flow Connector between Sub-Basin C4_10C and the C2 Extension Canal

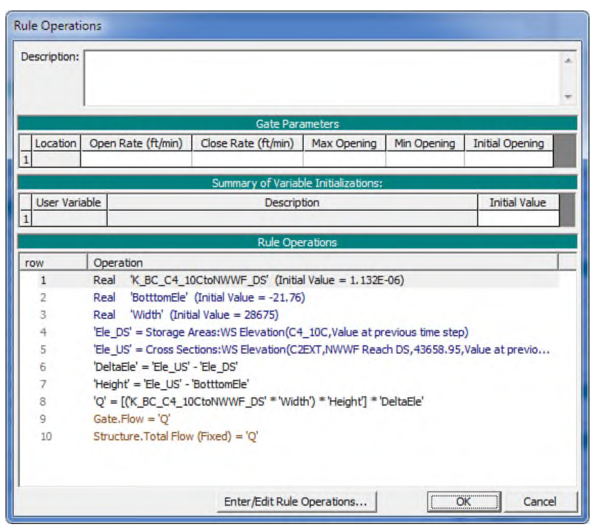

Figure E-24. Groundwater Flow Equation for Sub-Basin C4_10C and the C2 Extension Canal

Canal Boundary Conditions

Canal boundary conditions were defined for all primary canals in the model including the C-2, C-3 and C-4 canals and consisted primarily of specified flows at the upstream end of the canals and specified stage values at the tidal ends.

Tidal Boundary Condition

Tidal stages were specified at the downstream end of the primary canals. For model calibration (see **Appendix F**), 15-min measured stages were respectively applied on the tailwater side of structures S-22, G-93 and S-25B at the eastern end of the C-2, C-3 and C-4 canals. For LOS event simulation runs (See **Appendix G**), statistically derived tidal stages were imposed at the tidal end of the canals. Further details on the determination of the tidal stages for current and future conditions with sea level rise projections can be found in **Appendix C**. As an example, **Figure E-25** shows the statistically derived tidal stages for the 25-yr storm event under current conditions.

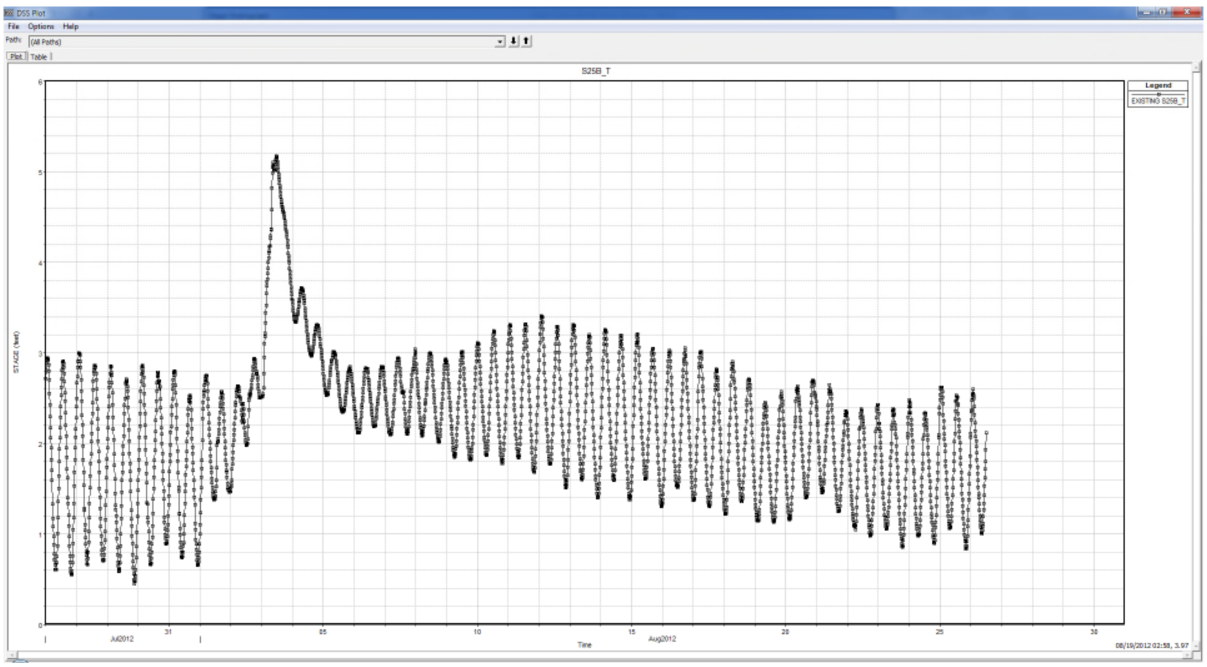

Figure E-25. C-4 Canal Tidal Boundary Stages for the 25-yr Strom Event

SUMMARY AND CONCLUSIONS

This report described the datasets and procedures followed to construct a hydrologic/hydraulic model of the C-4 Basin in Central Miami-Dade County. The chosen model for this project was the HEC-RAS developed by the Army Corps of Engineers. The model was conceptualized as a simple network of storage sub-basins and canal network that represent the exchange of surface and groundwater flows in the basin. Overland flow processes are not included in the model and sub-basin storage represents combined surface and groundwater storages in each sub-basin. Model hydraulics include canal conveyance, bridge, culvert and gated structure energy losses, pump stations and the flood control operations of the C-4 Basin Water Control Plan. Appendix B of this report describes with further detail the data collection process while Appendix F describes the model calibration process. The model application to evaluation of flooding LOS under current and future with sea level rise conditions is further explained in Appendix G of this report.

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Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed

Appendix F: C4 Basin v1.0 HEC-RAS Model Calibration

South Florida Water Management District
Hydrology and Hydraulics Bureau

December 16, 2015

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Additional project documents can be found on a District server at:

\\ad.sfwmd.gov\dfsroot\data\hesm_nas\projects\basin_studies

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South Florida Water Management District, 2015. Flood Protection Level of Service Analysis for the C-4 Watershed. Appendix F: C4 Basin v1.0 HEC-RAS Flood Model Calibration. H&H Bureau, SFWMD, West Palm Beach, FL. 68 pp. December 16 , 2015.

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INTRODUCTION

The District is in the process of implementing a new program to evaluate current and future Level of Service for flood protection (LOS). In order to develop the methodology for such program, a pilot study is being carried out in which the current and future LOS are being evaluated for the C-4 Basin in Miami-Dade County. For this C-4 Basin LOS assessment a simplified hydrologic and hydraulic model analysis has been selected to evaluate the current and future drainage capacity of the C-4 basin under flooding conditions. The hydraulic model for this effort is a canal flow routing model (HEC-RAS v5.0 beta) and uses a simplified hydrologic approach in which the flows (including surface and groundwater) between sub-basins and canals, are computed as a linear function of the head differential between water bodies times a conductance term which is obtained by calibration. The sub-basins in the model are represented as storage units in which the total storage includes soil and above ground storage. The HEC-RAS model receives the inflows directly from rainfall as boundary flows then carries the collected flows along the canals through bridges and control structures to the tidal basin outlet.

The HEC-RAS model was calibrated for a significant wet period (August-September, 2012) during which flood control operations were activated to maintain flood waters out of the basins. After model calibration, the model parameters were used to validate the model with data from Hurricane Irene (October, 1999).

The evaluation of current LOS with the calibrated HEC-RAS model involves the routing of inflows and outflows through the sub-basins and canals in the C-4 Basin from several pre-determined return frequency storm events which include the 5-, 10-, 25- and 100-yr rainfall events under corresponding current and future sea level rise tidal surge conditions. All geometric data, boundary conditions and model results in this report are referenced to NGVD29 vertical datum.

This report summarizes the tasks of model calibration and validation. The tasks of model conceptualization and development are documented in **Appendix E**. The application of the model to evaluate LOS under current and future sea level rise conditions is documented in **Appendices G and H.**

STUDY AREA

The C-4 Basin encompasses an area of about 83.3 square miles of highly urbanized areas on eastern portion of the basin including the Miami International Airport, The Florida International University main campus, the cities of Belen and Sweetwater and portions of the cities of Doral, Miami and West Miami. The portion of the basin west of the Florida Turnpike is mostly undeveloped and includes large rock mining areas and the Pennsuco Wetlands just east of the L-31 Canal (**Figure F-1**).

The drainage capacity of the primary canal in the C-4 Basin is highly influenced by flood control operations in the C-2 and C-3 Basins. **Figure F-1** also shows the location of the C-4 and adjacent basins in Central Miami-Dade County. The C-2 Basin is located to the south of the C-4 Canal and its primary drainage feature is the C-2 Canal which intersects the C-4 Canal just west of the Florida

Turnpike. Flows and water levels in the C-2 Canal are controlled by gate operations at the S-22 Structure located at the tidal end of the C-2 Canal just east of SW 57th Avenue. Also, south of the C-4 Canal, the C-3 Basin is drained by the C-3 Canal which affects the flows in the C-4 Canal. The C-3 Canal intersects the C-4 Canal just east of the Palmetto Expressway and is the only primary drainage feature for the City of Coral Gables. The G-93 structure located near the intersection of SW 57th Avenue and SW 35th Street is operated by the District to control flows and canal stages in the C-3 Canal. Because of their significant influence on flows and stages in the C-4 Basin, the C-2 and the C-3 Basins were included, in a simplified manner, as part of the model for the C-4 Basin.

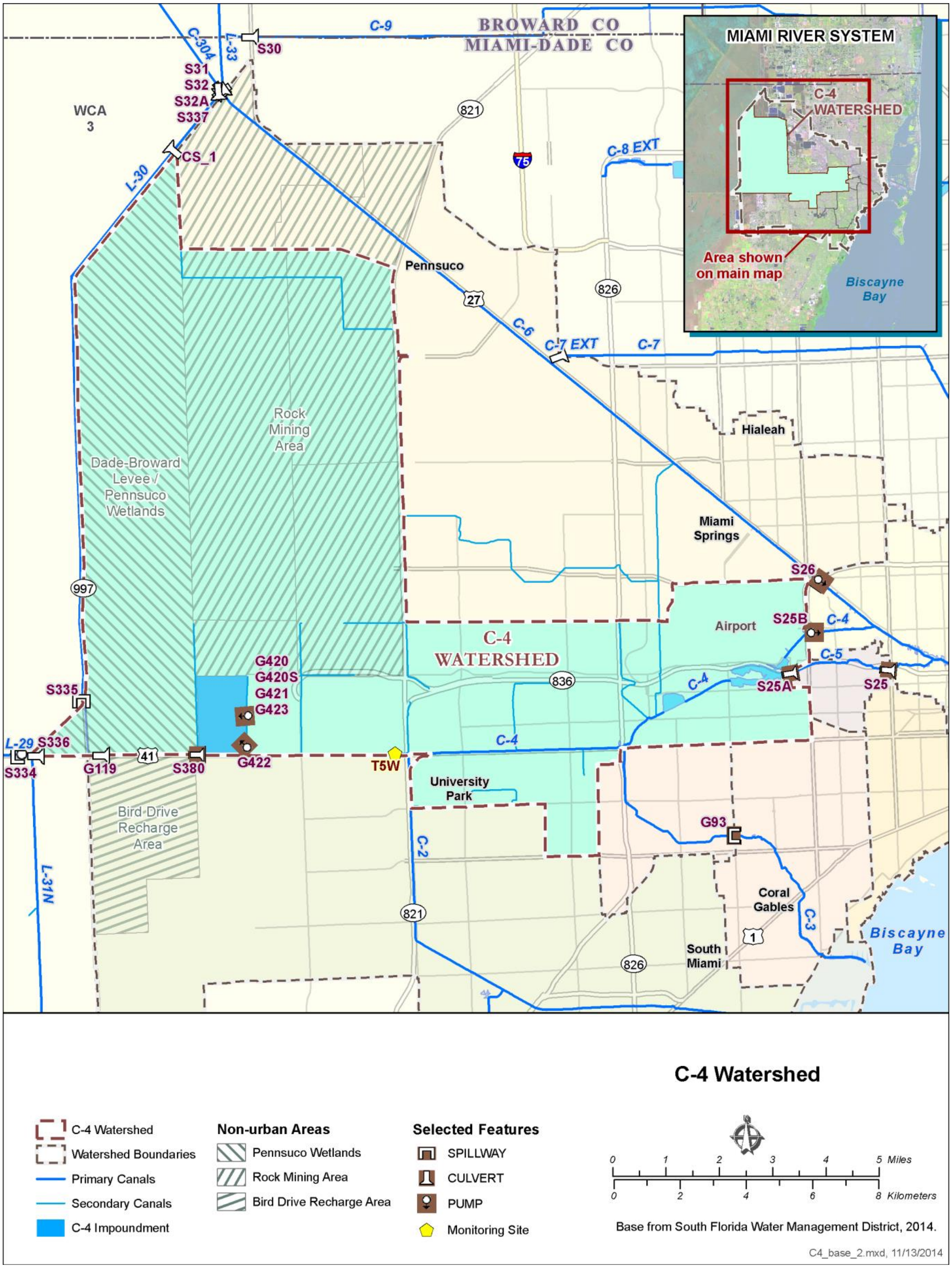

Figure F-1. Location of the C-4 Basin in Miami-Dade County, Florida

DATA COMPILATION

Channel and Bridge Geometric Data

Geometric data to describe the conveyance features in the HEC–RAS model consist primarily of canal cross-sections and representation of the openings at the control structures. In the case of canal cross-sectional data, several sources of information were used to compile an adequate number of canal cross-sections in all the canals in the model which include the C-2, C-3, C-4 and C-5 canals. The primary source of information was existing hydraulic models of these basins from DERM (**2004**) and SFWMD (**2011**). Most of the canal cross sectional data from these studies come from District's and US Army Corps of Engineers surveys of the canal and structures and more recently from DERM's XP-SWMM model. No new canal cross-sectional data was collected as part of this study.

The geometry of the primary control structures, including G-119, S-380, S-25B, S-25BP, S-22 and G-93, was verified with the original as-built drawings from District's and USACE databases. In the case of the three tidal structures S-25B, S-22 and G-93, the District's flow rating equations used in DbHydro were used in lieu of the HEC-RAS standard spillway and culvert equations (**Imru & Wang, 2004a; 2004b; Dessalegne, 2013**). As an example, **Figure F-2** shows the geometry of S-25B structure gate openings and cross-sections immediately upstream and downstream of the structure.

Flow Computations at Water Control Structure in HEC-RAS

The computation of discharge in HEC-RAS for spillways follows the methods established by the USACE and is fully documented in the HEC-RAS User's Manual (**HEC, 2010**). The following is a summary of the methods used to compute flow through spillways which is the most common type of control structure in the study area.

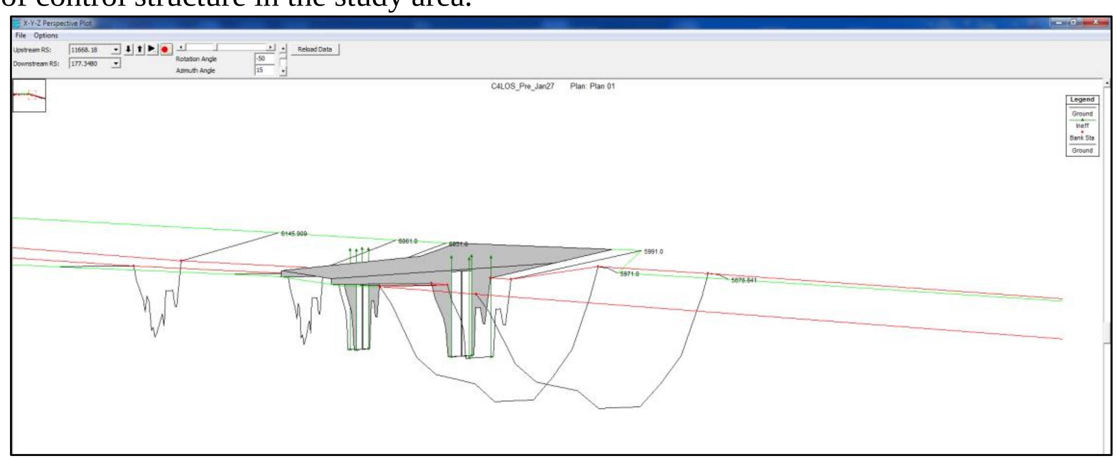

Figure F-2. Geometry of Structure S-25B and neighboring Canal Cross-sections

Free Flow

Free flow conditions in a gated spillway occur whenever the downstream tailwater elevation (Z_{TW}) does not result in an increase in upstream headwater elevation for a given flow rate, i.e. when the tailwater depth (Z_{TW}) divided by the headwater energy depth above the spillway ($Z_{HW} - Z_{TW}$) is less than 0.67. **Figure F-3** shows a schematic of this type of flow condition. In this case, the flow rate through the gate opening is represented by the following orifice flow equation:

$$Q = C\ W\ G_o \sqrt{2g\ (Z_{HW} - Z_{Spillway})} \qquad \text{(F-1)}$$

Where:

- C = Discharge coefficient (0.5 – 0.7)
- W = Width of gated spillway
- G_o = Height of gate opening
- Z_{HW} = Elevation of the upstream energy grade line in ft NGVD29
- $Z_{spillway}$ = Spillway crest elevation in ft NGVD29

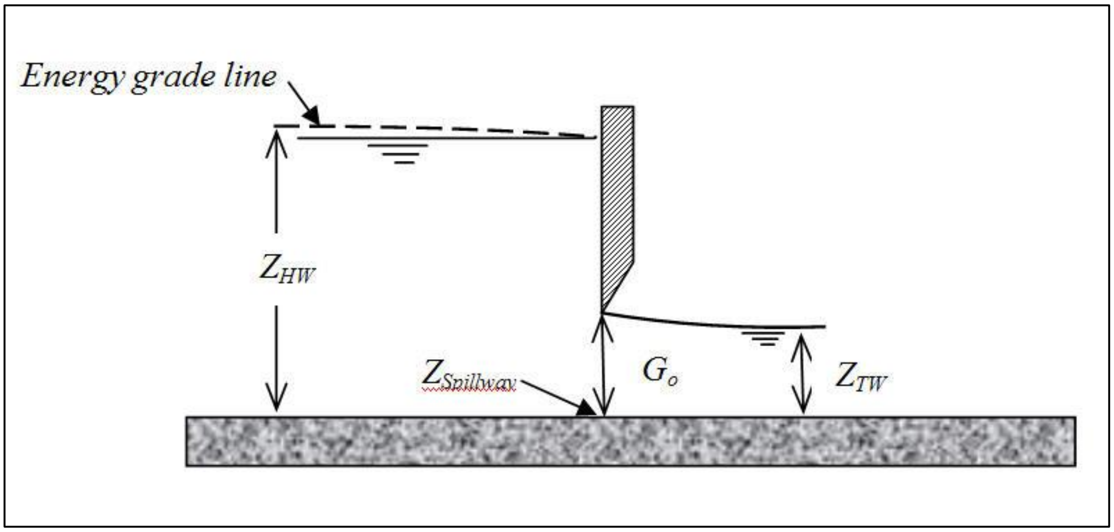

Figure F-3. Spillway under Free Orifice Flow Conditions

Free to Submerged Flow

Transition flow conditions from free to submerged begin when the tailwater elevation increases and the discharge through the gate is no longer under the influence of the tailwater stage. Computationally, HEC-RAS defines submergence at the point when the tailwater depth (Z_{TW}) divided by the headwater energy depth above the spillway ($Z_{HW} - Z_{TW}$) is between 0.67 and 0.8. **Figure F-4** shows the schematic of partially submerged flow conditions. The equation to compute the flow through the gate under this condition is:

$$Q = C\ W\ G_o \sqrt{2g\ \ 3(Z_{HW} - Z_{TW})} \qquad \text{(F-2)}$$

Where:

Q = Flow rate through partially submerged orifice if ft^3/sec

C = Discharge coefficient (0.5 – 0.7)

W = Width of gated spillway in feet

G_o = Height of gate opening in feet

Z_{HW} = Elevation of the upstream energy grade line in ft NGVD

Z_{TW} = Elevation of the downstream water surface in ft NGVD

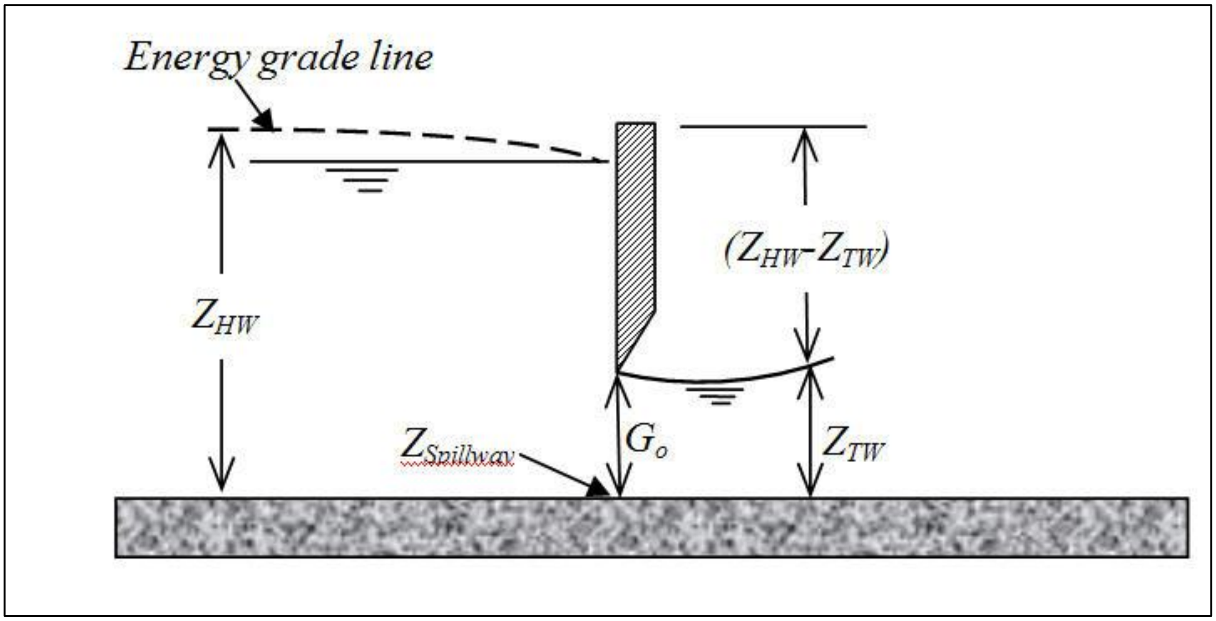

Figure F-4. Spillway under partially submerged orifice flow conditions

Submerged Flow

HEC-RAS considers fully submerged flow conditions when the tailwater depth (Z_{TW}) divided by the headwater energy depth above the spillway ($Z_{HW} – Z_{TW}$) is greater than 0.8. In this case, the discharge through the gate is computed using the following orifice flow equation. **Figure F-5** shows a schematic of flow under full submergence.

$$Q = C\,A\,\sqrt{2g\,(Z_{HW} - Z_{TW})} \qquad \text{(F-3)}$$

Where:

Q = Flow discharge in ft^3/sec

C = Discharge coefficient (0.5 – 0.7)

A = Area of gate opening in ft^2

G_o = Height of gate opening in ft

Z_{HW} = Elevation of the upstream energy grade line in ft NGVD

Z_{TW} = Elevation of the downstream water surface in ft NGVD

Submerged Weir Flow

When the gates are open above the water level over the crest of the spillway, the flow is considered as weir flow. The transition from orifice to weir flow occurs in HEC-RAS when the upstream head is between 1.0 to 1.1 times the height of the gate opening. **Figure F-6** shows a schematic of flow under these conditions.

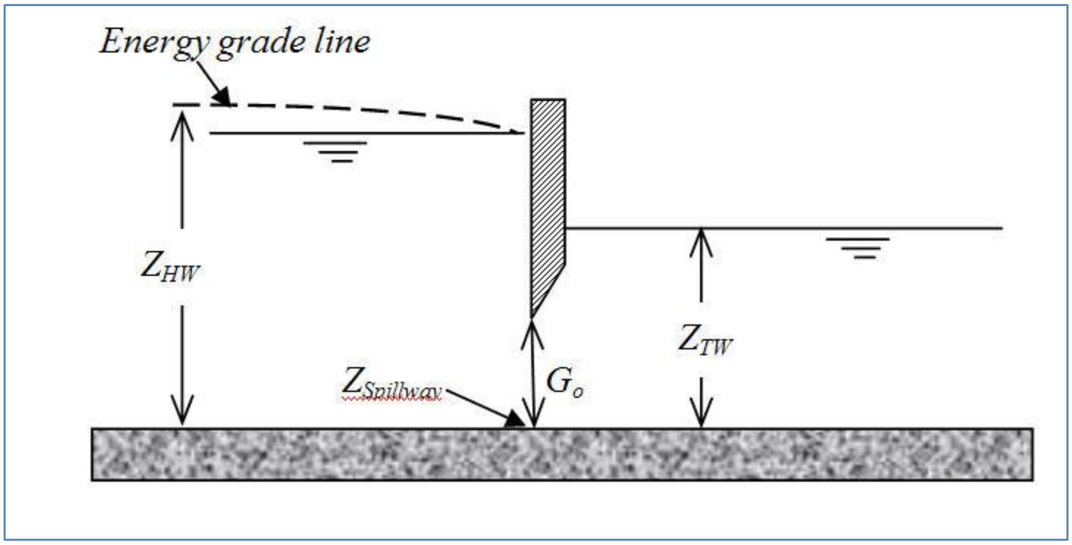

Figure F-5. Spillway under Fully Submerged Orifice Flow Conditions

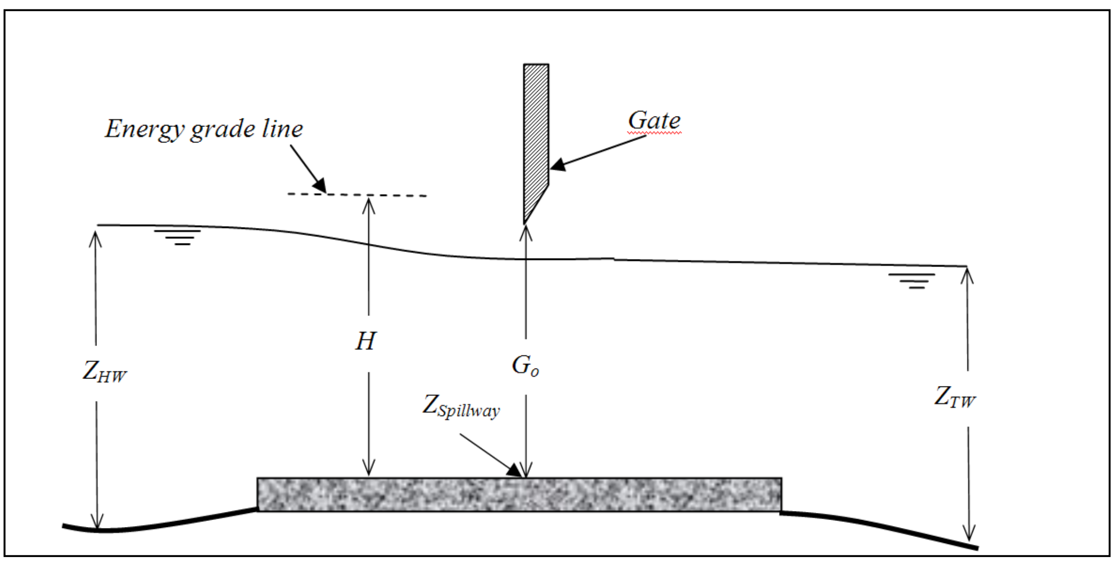

Figure F-6. Spillway under Submerged Weir Flow Conditions

Under these conditions, the flow through the structure is calculated using the following weir flow equation.

$$Q = C\ L\ H^{3/2} \qquad \text{(F-4)}$$

Where:

Q = Flow rate through broad crested weir if ft^3/sec

C = Discharge coefficient (2.6 – 4.1; default value = 3.0)

L = Length of spillway crest in ft

H = Upstream energy head above weir crest, ft

Z_{HW} = Elevation of the upstream energy grade line in ft NGVD

Z_{TW} = Elevation of the downstream water surface in ft NGVD

Submergence is accounted in the weir flow equation with a varying discharge coefficient. As the tailwater level rises, the degree of submergence increases, resulting in reduction of flow. Submergence is defined as the ratio Z_{TW}/H and the correction factor for submergence used in HEC-RAS is shown in **Figure F-7.** A maximum submergence factor of 0.95 is set as a default but can be changed by the user.

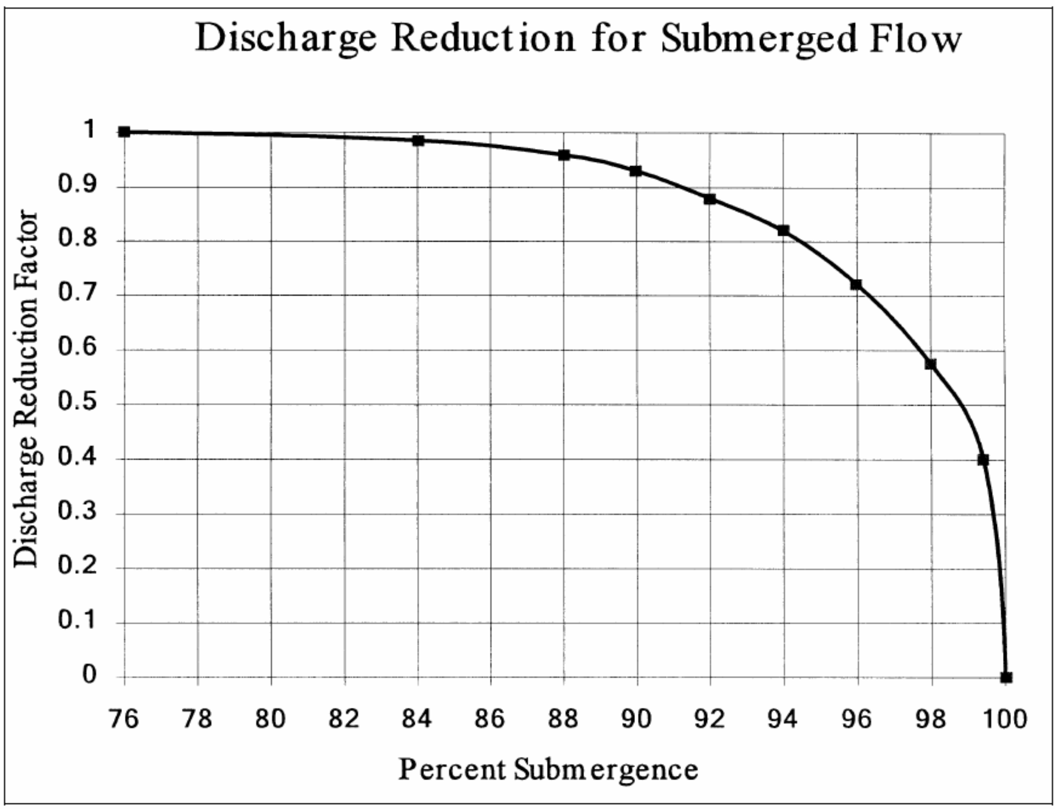

Figure F-7. Flow Reduction Factor for Submergence (**HEC, 2010**)

Orifice and weir flow coefficients for the coastal structures were set up in the model for each coastal structure using typical values found in the literature and kept fixed during model calibration. The value for sluice gate discharge coefficients were set at 0.6. For submerged flow conditions, the discharge coefficients had values in the range of 0.7 to 0.8, and for open weir flow conditions when gates are completely out of the water, the weir coefficient had values between 2.7 and 3.0.

Structure Weir and Orifice Coefficients

Water control structures located at the tidal end of the C2, C3, and C4 canals are simulated in HEC-RAS as simple weirs and orifices depending on the elevation of the water on both sides of the gates with respect to the elevation of the gate's bottom when opened. Generally speaking, water levels on both sides of tidal structures in the Miami River Watershed (MRW) are under tidal fluctuations which may lead to several types of flow conditions. The computation of flows through the three tidal structures in the model under different types of flow regimes is documented in **Appendix D** of this study.

Canal Roughness Coefficients

The canal roughness coefficient is a parameter in the open channel flow equation that is a function of the material of the channel bed. This is the only flow equation parameter needed in the calibration process for flows and stages in the canals. Typically, the value of this parameter is defined per canal reach and is assumed constant on that reach (**Figure F-8**). In urban man-made natural canals like the C-4 Canal, the value of the coefficient ranges between 0.02 and 0.04 depending on the vegetation cover at the bottom of the canal. In Miami-Dade County, the District maintains the canals free of vegetation particularly during wet season when aquatic vegetation growth can significantly reduce canal conveyance. The initial values (pre-calibration) of canal roughness coefficients in the canal reaches follows the above rang of values which then were adjusted in the calibration process, which is fully described in the next section of this report. In the model, the calibrated values of the roughness coefficients may reflect higher than normal values as a result of localized vegetation growth or lack of maintenance in the canals.

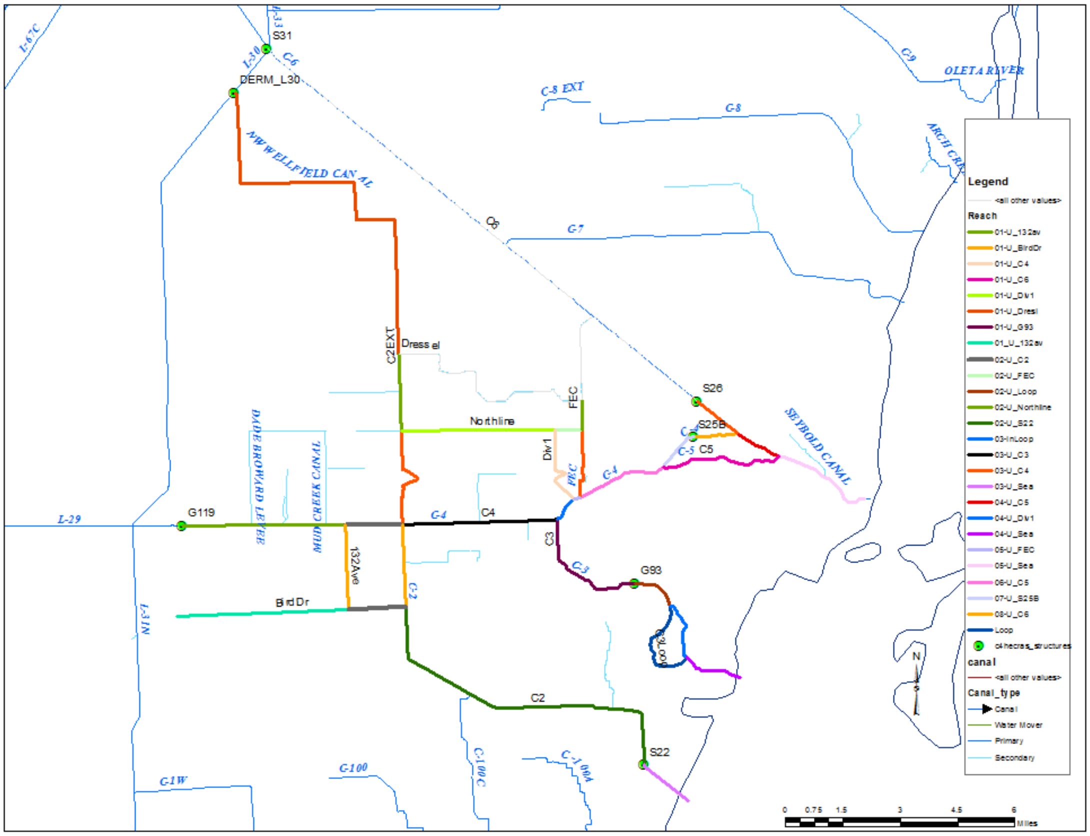

Figure F-8. Canal Reaches for Model Parameterization

Canal-Aquifer Interaction

As indicated before, the exchange of water between the surficial aquifer and the canal in Miami-Dade County is important and must be included for accurate estimation of flows in and out of the canals. The simplified approach described previously in the report was chosen to avoid dependence on a groundwater model of the aquifer whose coupling with the HEC-RAS model will result in prohibitory long run times. The lumped parameter approach assumes that exchange of water between sub-basins and sub-basins and canals in the HEC-RAS model is a function of the head differential between water bodies and a conductance or coefficient that is obtained by model calibration which is documented in the following section of this report. Since the lumped conductance terms were not derived from field measurements such as the case of aquifer hydraulic conductivities, there was no field data collection for these parameters.

MODEL CALIBRATION

HEC-RAS model calibration consisted of adjusting the value of hydraulic parameters, primarily those that directly affect the magnitude of canal stages and flows in the canal system, to match measured values during the calibration period of August 1st to September 30, 2012. This section of the report describes the optimization process by which the model parameters were adjusted to obtain a reasonably calibrated model given the quantity and quality of the observation data available.

Objective

The objective of HEC-RAS model calibration is to use historical stage and flow data in the canals to compute a set of model parameters that in turn yield computed values of stages and flows in the canals and stages in the sub-basins that closely follow the observed data. The search of the set of model parameters is done with the PEST++ software (**Welter et al., 2015**), a widely used software package for parameter optimization in the area of water resources. In this particular application, the parameter set consisted of lumped flow conductance terms between sub-basins and sub-basins to canals, as well as roughness coefficients in the canals. Large uncertainties in the calibrated conductance terms can be expected due to lack of observed stage data in the sub-basins (i.e., groundwater level data) and structural errors in the model. A formal effort to assess and reduce the predictive uncertainty of the model was not part of this study. Instead of performing a formal uncertainty analysis, this effort employed a conservative approach of constrained optimization using PEST++ to obtain more reasonable values of water levels in the sub-basins once the calibration objective function has been adequately reduced. This approach sacrifices some goodness of fit in the calibration of water levels and flows in the canals for more reasonable values of water levels in the sub-basins.

Calibration Period

The period August 1 to September 30, 2012 was selected for model calibration since this period is recent enough to include recent enhancements to the system and encompasses two major storm events. Stage and flow conditions were fully monitored prior, during and after the storm event at all District canal structures providing sufficient data for calibration.

Methodology

The model-independent parameter estimation software, PEST++ ver. 3, was used to facilitate the calibration process (**Welter et. al., 2015**). This calibration tool uses industry-standard parameter estimation techniques to solve the inverse problem of any model with distributed parameters. The advantages of automated model calibration are numerous and have been widely documented in the literature (**Doherty, 2008**). PEST uses industry-standard parameter estimation techniques to solve the inverse problem of any model with distributed parameters. For this HEC-RAS calibration effort the objective function of residuals (ϕ) to be minimized was defined as the sum of square-error (RMSE) of hourly stages and flows at locations where historical measurements are available. Mathematically, the residual function to be minimized by PEST++ is defined as follows.

$$\Phi = \sum_{i=1}^{m}\left[w_i \; (p_i - o_i)\right]^2 = \sum_{i=1}^{m} \; (w_i \; r_i)^2 \tag{F-5}$$

where:

Φ = objective function

p_i = model predicted value i

o_i = observed value i

w_i = weight observation value i

r_i = ith residual

m = total number of observations

For a general model of n parameters (b_n) represented by $Mb = p$, the above equation for the objective function is equivalent to:

$$\Phi = (o - p)^T \; Q \, (o - p) = (o - Mb)^T \; Q \, (o - Mb) \qquad \text{(F-6)}$$

where:

Q = observation weight matrix

M = linear or non-linear model

b = parameter vector

PEST++ computes the predicted model response to a small change in the parameters is given by:

$$p_1 = p_0 + J(b_1 - b_0) \qquad \text{(F-7)}$$

where:

p_0 = predicted model outcome prior to parameter perturbation

p_1 = predicted model outcomes after parameter perturbation

b_0 = parameter set prior to perturbation

b_1 = perturbed parameter set

$$\text{Jacobian matrix} = J_{mxn} = \begin{bmatrix} \frac{\partial o_1}{\partial b_1} & \frac{\partial o_1}{\partial b_2} & \cdots & \frac{\partial o_1}{\partial b_n} \\ \frac{\partial o_2}{\partial b_1} & \ddots & \cdots & \frac{\partial o_2}{\partial b_n} \\ \vdots & \vdots & \ddots & \\ \frac{\partial o_m}{\partial b_1} & \frac{\partial o_m}{\partial b_2} & \cdots & \frac{\partial o_m}{\partial b_n} \end{bmatrix} \qquad \text{(F-8)}$$

The upgraded objective function, Φ_1, becomes:

$$\Phi_1 = (o - p_0 - J(b_1 - b_0))^T \, (o - p_0 - J(b_1 - b_0)) \qquad \text{(F-9)}$$

Which is minimized if $(o - p_0) = J(b_1 - b_0)$.

PEST++ solves for the parameter correction vector, $(b_1 - b_0)$, with the following equation:

$$(b_1 - b_0) = (J^T\ Q\ J)^{-1}\ J^T\ Q\ (o - p_0) \qquad \text{(F-10)}$$

This correction to the parameter vector is applied in an iterative process until convergence to final parameter set is obtained. PEST++ also uses the Marquardt method to search for the optimal set of parameters by adding a factor to the diagonal elements of the sensitivity matrix J^TJ as follows:

$$(b_1 - b_0) = (J^T Q\ J + \lambda\ I\ )^{-1}\ J^T\ Q\ (o - p_0) \qquad \text{(F-11)}$$

where λ is the Marquardt lambda and I is the identity matrix. The value of λ is chosen by PEST++ so that the computation of the parameter correction vector results in a more robust search for Φ that meets the convergence criteria set by the user. PEST++ also provides an advanced option to include adaptive Tikhonov regularization in the calibration process. When run in this mode, PEST++ uses two objective functions; the calibration or measurement objective function, Φ_m, and the regularization objective function, Φ_r and solves the constrained optimization problem: minimize Φ_r subject to the constraint:

$$\Phi_m \leq \Phi_m^l \qquad \text{(F-12)}$$

where Φ_m^l is an upper limit on the permissible calibration objective function. By definition, Tikhonov regularization is used to enforce a preferred state of the system parameters, however in this analysis it is used to maximize the water levels in the sub-basins once a specified level of calibration has been achieved.

Calibration Targets

In this model calibration exercise, the objective function to be minimized by PEST++ is the weighted sum of the squared residual errors between the computed and measured average hourly canal stages and flows in the model domain. However, other error statistical measures such as error bias and the root mean squared error (RMSE) are used in the presentation of the results.

Bias or mean error indicates whether simulated values tend to be disproportionately overestimated or underestimated when compared to historical measurements. The closer the bias is to zero, the better the model prediction. This model calibration metric indicates the presence of systematic error in model predictions. This error causes all computed values to deviate from the measured values by a consistent amount and in a consistent direction (higher or lower than). The bias is calculated as:

$$bias = \frac{\sum_{i=1}^{m}(p_i - o_i)}{m} \quad ; \quad -\infty \leq bias \leq \infty \qquad \text{(F-13)}$$

where:

p_i = predicted head or flows value i

o_i = observed value i

M = total number of data values

The root-mean-square-error (RMSE) or standard error of the estimate gives an indication of the magnitude of a typical error. The closer the RMSE is to zero, the better the model prediction. It is related to the bias in that it is the square root of the mean of the squares of the deviations/error bias, and is given by the following expression:

$$RMSE = \sqrt{\frac{\sum_{i=1}^{m}(p_i - o_i)^2}{m}} \;\; ; \;\; RMSE \geq 0 \qquad \text{(F-14)}$$

Although not used as part of the objective function in PEST, the error bias and RMSE are used to compare the relative goodness of fit of each gauge against each other and against the pre-established calibration targets for canal stages. Previous model calibration efforts in Miami-Dade County have employed calibration targets for stage bias and RMSE of 0.5 ft (**SFWMD 2010a, 2010b**). The large variability of flows and relative magnitude of errors when estimating flows from observed canal stages on both sides of a structure make it difficult to establish an error tolerance for computed flows, particularly at tidal structures where a small difference in heads can lead to large errors in estimated flows. In addition, at tidal structures standard methods for field data collection and flow computations are impractical and inaccurate because of the low flow velocities, flow reversals, and bi-directional flow in which high-salinity water flows inland under freshwater flowing out to the ocean. For these reasons, bias and RMSE for computed flows had no pre-determined calibration target values in this application.

Calibration Parameters

HEC-RAS model calibration of canal water levels and flows was performed using PEST++ to adjust the following 149 parameters which are described below.

Canal-Aquifer Interaction Parameters

The exchange of water between sub-basins and between sub-basins and canals was conceptualized as function of lumped conductances and hydraulic head differentials between water bodies. There were a total of sixty-five (65) basins-to-basins and forty-two (42) basin-to-canal conductances. The initial values for all the conductances vary widely since the pre-calibrated version of the model showed high sensitivity to these parameters and became easily unstable for certain high values of basin-to-canal conductances, particularly along the C-4 canal.

Seepage Boundary Flow Parameters

Seepage flow from the Water Conservation Area (WCA) 3B and from the Everglades National Park (ENP) enters the model domain at the two most-western located sub-basins in the HEC-RAS model. These flow boundary conditions are estimated as a function of the head differential between the WCA and ENP water bodies and the two receiving sub-basins in the model (sub-basins C4-10A and C2) and seepage coefficients that are treated as calibration parameters. The head differential needed to compute the seepage flows is computed with observed stages in the WCA and ENP and the computed stages in the two sub-basins.

Canal Roughness Coefficients

Canal Manning's n roughness coefficients in the canals were parameterized as constant values in each canal reach of the system. Figure F-8 shows the canal segmentation used for calibration. A total of 34 canal segments were used with initial values of the coefficients varied between 0.02 and 0.1. The high value of n during model calibration to compensate head losses due to the presence of aquatic vegetation in the canals and missing drainage features such as culverts and bridges for which no data was available.

Gate and Orifice Flow Coefficients

Computation of flows in HEC-RAS at spillway structures requires flow discharge coefficients for two different types of flow regimes that may occur during a simulation which are sluice gate and submerged orifice flow conditions. These coefficients consist of gate and orifice flow coefficients for structures G-93, S-22 and S-25B. A total of six (6) gate and orifice coefficients were treated as calibration parameters in this study. Initial values for these coefficients were defined as the average of the recommended values in HEC-RAS of 0.6 and 0.8 for sluice gate and orifice flow conditions, respectively.

PEST++ Calibration of HEC-RAS

Calibration of the HEC-RAS MRW model involved PEST++ adjusting values of canal roughness coefficients and bottom hydraulic conductivities. The canals were discretized into thirty-four (34) segments for which canal roughness coefficient values were assigned as pre-calibration values. **Tables F-1 through F-5** summarize the parameters initial (pre-calibration), final (optimized), upper and lower limits. **Table F-1** describes the values of the groundwater flow conductances between sub-basins. There are sixty-five (65) values of this parameter which is used to calculate the flow of water between two adjacent sub-basins. Initial (pre-calibration) values for these parameters were established during the model setup which included manual adjustments to some conductances to keep the model stable during execution. Pre-calibration values for these parameters ranged between 5.0E-05 ft/sec and 5.0E-02 ft/sec. The final calibrated values of this parameter ranged between 3.92-06 ft/sec and 6.13-04 ft/sec with an average value of 8.56E-05 ft/sec. Similarly, for groundwater flow between sub-basins and canals, the model employed forty-two (42) basin-to-canal conductances whose pre-calibration values were also established in the model set-up phase attempts to get the model to run without instability.

Table F-2 summarizes the initial, final, maximum and minimum allowable values of the parameters during calibration. The range of values for the final calibrated parameters is between 8.698E-07 ft/sec and 4.017-03 ft/sec with an average value of 4.017E-03 ft/sec. Canal roughness coefficients consisted of thirty-four (34) values whose initial, calibrated, minimum and maximum allowable values are summarized in **Table F-3**. As this table shows, the range of calibrated values varies between 0.02 and 0.05 with an average value of 0.031.

Two (2) boundary to basin conductance coefficients were calibrated for computations of seepage flow from the WCA 3B and sub-basin C4-10A (Pennsuco Wetland) and from the Everglades National Park to the C-2 basin **(Table F-4**). The calibrated values of these two coefficients were 0.0016 ft/sec and 0.0243 ft/sec, respectively. Finally, the calibrated gate and orifice flow coefficients (**Table F-5**) for structures G-93 were 0.68 and 0.9, for structure S-22 were 0.64 and 0.79 and for structure S-25, 0.56 and 0.55.

Table F-1. Basin to Basin Conductance Parameter Names, Initial, Final, Lower and Upper Values for PEST Calibration

Parameter Name	From Basin	To Basin	Descrip-tion	Initial Value ft/sec	Final Value ft/sec	Lower Value ft/sec	Upper Value ft/sec
k_100b_100c	c4_100b	c4_100c	Basin to Basin Lumped Conductance	5.00E-06	3.95E-06	1.00E-07	5.00E-05
k_100b_c4_ag6	c4_100b	c4_ag6		5.00E-04	4.35E-04	1.00E-07	6.00E-04
k_100c_125b	c4_100c	c4_125b		5.00E-06	5.01E-06	1.00E-07	5.00E-05
k_10a_10e	c4_10a	c4_10e		1.00E-04	8.90E-05	1.00E-07	9.00E-03
k_10a_c4-n-3	c4_10a	c4-n-3		1.00E-04	1.17E-04	1.00E-07	4.00E-02
k_10a_c4-n-4	c4_10a	c4-n-4		5.00E-06	5.51E-06	1.00E-07	4.00E-02
k_10b_10a	c4_10b	c4_10a		5.00E-06	7.83E-06	1.00E-07	5.00E-04
k_10b_10d	c4_10b	c4_10d		5.00E-06	4.98E-06	1.00E-07	5.00E-05
k_10b_10e	c4_10b	c4_10e		5.00E-06	3.92E-06	1.00E-07	5.00E-05
k_10c_10b	c4_10c	c4_10b		5.00E-04	4.85E-04	1.00E-07	5.00E-03
k_10c_10d	c4_10c	c4_10d		5.00E-04	4.31E-04	1.00E-07	5.00E-03
k_10d_10e	c4_10d	c4_10e		5.00E-06	4.97E-06	1.00E-07	5.00E-05
k_10d_25	c4_10d	c4_25		5.00E-06	4.74E-06	1.00E-07	5.00E-05
k_10d_40	c4_10d	c4_40		5.00E-04	4.41E-04	1.00E-07	5.00E-03
k_10d_75a	c4_10d	c4_75a		5.00E-06	5.14E-06	1.00E-07	5.00E-05
k_10d_c4_ag1	c4_10d	c4_ag1		5.00E-06	5.50E-06	1.00E-07	5.00E-05
k_10e_25	c4_10e	c4_25		5.00E-05	5.30E-05	1.00E-07	1.00E-04
k_10e_c4-n-3	c4_10e	c4-n-3		5.00E-06	4.59E-06	1.00E-07	5.00E-05
k_125a_150a	c4_125a	c4_150a		5.00E-06	4.08E-06	1.00E-07	5.00E-05
k_125b_125a	c4_125b	c4_125a		5.00E-06	5.98E-06	1.00E-07	5.00E-05
k_25_c4_ag1	c4_25	c4_ag1		5.00E-06	4.72E-06	1.00E-07	5.00E-05
k_40_75a	c4_40	c4_75a		5.00E-06	5.02E-06	1.00E-07	5.00E-05
k_40_c4_ag1	c4_40	c4_ag1		5.00E-06	5.71E-06	1.00E-07	5.00E-05
k_40_c4_ag2	c4_40	c4_ag2		5.00E-06	4.98E-06	1.00E-07	5.00E-05
k_40_c4_ag3	c4_40	c4_ag3		5.00E-06	4.88E-06	1.00E-07	5.00E-05
k_60a_55	c4_60a	c4_55		5.00E-06	4.55E-06	1.00E-07	5.00E-05
k_60a_c2	c4_60a	c2		5.00E-06	6.11E-06	1.00E-07	5.00E-04
k_60a_c3	c4_60a	c3		5.00E-04	3.41E-04	1.00E-07	5.00E-03
k_65a_70	c4_65a	c4_70		5.00E-06	5.47E-06	1.00E-07	5.00E-05
k_65b_65a	c4_65b	c4_65a		1.00E-04	1.26E-04	1.00E-07	9.00E-03
k_65b_70	c4_65b	c4_70		5.00E-06	5.77E-06	1.00E-07	5.00E-05
k_70_c4_ag6	c4_70	c4_ag6		5.00E-06	6.04E-06	1.00E-07	5.00E-05
k_75a_65b	c4_75a	c4_65b		5.00E-06	4.70E-06	1.00E-07	5.00E-05
k_75a_75b	c4_75a	c4_75b		5.00E-06	6.82E-06	1.00E-07	5.00E-05
k_75b_65b	c4_75b	c4_65b		5.00E-06	5.51E-06	1.00E-07	5.00E-05
k_75b_70	c4_75b	c4_70		8.00E-04	6.13E-04	1.00E-07	1.00E-03
k_75b_c4_ag5	c4_75b	c4_ag5		5.00E-06	4.21E-06	1.00E-07	5.00E-05
k_c2-n-24_55	c2-n-24	c4_55		5.00E-06	6.08E-06	1.00E-07	5.00E-05
k_c2_c3	c2	c3		5.00E-06	1.83E-05	1.00E-07	5.00E-02
k_c4-n-3_25	c4-n-3	c4_25		5.00E-06	5.34E-06	1.00E-07	5.00E-05
k_c4-n-3_c4-c-4	c4-n-3	c4-n-4		5.00E-04	5.90E-04	1.00E-07	5.00E-03
k_c4-n-4_25	c4-n-4	c4_25		5.00E-06	5.26E-06	1.00E-07	5.00E-05
k_c4_ag10_c3	c4_ag10	c3		5.00E-06	5.52E-06	1.00E-07	5.00E-04
k_c4_ag10_c4_ag11	c4_ag10	c4_ag11		5.00E-06	5.10E-06	1.00E-07	5.00E-05
k_c4_ag10_c5	c4_ag10	c5		5.00E-06	4.94E-06	1.00E-07	5.00E-04
k_c4_ag11_c3	c4_ag11	c3		5.00E-06	6.38E-06	1.00E-07	5.00E-04
k_c4_ag12_125a	c4_ag12	c4_125a		5.00E-06	4.70E-06	1.00E-07	5.00E-05
k_c4_ag12_125b	c4_ag12	c4_125b		5.00E-06	5.96E-06	1.00E-07	5.00E-05
k_c4_ag12_150a	c4_ag12	c4_150a		5.00E-06	5.45E-06	1.00E-07	5.00E-05
k_c4_ag12_c4_ag5	c4_ag12	c4_ag5		5.00E-06	4.92E-06	1.00E-07	5.00E-05
k_c4_ag13_c4_ag12	c4_ag13	c4_ag12		5.00E-06	4.68E-06	1.00E-07	5.00E-05
k_c4_ag2_65b	c4_ag2	c4_65b		5.00E-04	4.93E-04	1.00E-07	5.00E-03
k_c4_ag2_c4_ag3	c4_ag2	c4_ag3		1.00E-04	1.08E-04	1.00E-07	2.00E-04
k_c4_ag2_c4_ag4	c4_ag2	c4_ag4		5.00E-04	4.09E-04	1.00E-07	5.00E-03
k_c4_ag3_c4_ag4	c4_ag3	c4_ag4		5.00E-06	4.20E-06	1.00E-07	5.00E-05
k_c4_ag4_65a	c4_ag4	c4_65a		5.00E-06	4.66E-06	1.00E-07	5.00E-05
k_c4_ag5_100b	c4_ag5	c4_100b		5.00E-04	5.61E-04	1.00E-07	5.00E-03
k_c4_ag5_100c	c4_ag5	c4_100c		5.00E-06	5.36E-06	1.00E-07	5.00E-05
k_c4_ag7_c3	c4_ag7	c3		5.00E-06	6.61E-06	1.00E-07	5.00E-04
k_c4_ag7_c4_ag8	c4_ag7	c4_ag8		5.00E-06	4.72E-06	1.00E-07	5.00E-05
k_c4_ag8_c3	c4_ag8	c3		5.00E-06	6.99E-06	1.00E-07	5.00E-04
k_c4_ag8_c4_ag9	c4_ag8	c4_ag9		5.00E-06	5.18E-06	1.00E-07	5.00E-05
k_c4_ag9_c3	c4_ag9	c3		5.00E-06	4.92E-06	1.00E-07	5.00E-04
k_c4_ag9_c4_ag11	c4_ag9	c4_ag11		5.00E-06	5.05E-06	1.00E-07	5.00E-05
k_c5_c3	c5	c3		5.00E-06	4.96E-06	1.00E-07	5.00E-04

Table F-2. Basin to Canal Conductance Parameter Names, Initial, Final, Lower and Upper Values for PEST Calibration

Parameter Name	From Basin	To Canal	Canal Reach	River Station	Descrip-tion	Initial Value	Final Value	Lower Value	Upper Value
k_bc_c2-n-24tos22_us	c2-n-24	c2	s22_us	63000	Basin to Canal Lumped Conductance	2.00E-02	1.73E-02	1.00E-07	5.00E-02
k_bc_c2tos22ud-ds_brd	c2	c2	s22_us-ds_brd	51000		1.00E-03	5.48E-04	1.00E-07	5.00E-02
k_bc_c4_10ctonwwf_ds	c4_10c	c2ext	nwwf reach ds	44500		1.00E-06	1.13E-06	1.00E-07	5.00E-03
k_bc_c4_10dtonwwf_ds	c4_10d	c2ext	nwwf reach ds	23000		8.00E-05	2.80E-04	1.00E-07	8.00E-04
k_bc_c4_75atonwwf_2	c4_75a	c2ext	nwwf reach-2	8900		1.00E-02	8.75E-03	1.00E-07	2.00E-02
k_bc_c4_40tonwwf2	c4_40	c2ext	nwwf reach-2	4900		8.00E-05	6.38E-05	1.00E-07	1.00E-03
k_bc_c3tog93us	c3	c3	g93_us	28000		1.00E-06	8.70E-07	1.00E-07	5.00E-03
k_bc_c4_10atousofc2us	c4_10a	c4	us_of_c2 us	78000		8.00E-04	8.13E-04	1.00E-07	5.00E-03
k_bc_c4_10atobdcanal	c4_10a	c4	bd canal	25000		8.00E-05	2.37E-04	1.00E-07	5.00E-04
k_bc_c4_10btobdcanal	c4_10b	c4	bd canal	10000		8.00E-05	1.00E-03	1.00E-07	5.00E-03
k_bc_c4-n-4tousofc2ds	c4-n-4	c4	us_of_c2 ds	69000		8.00E-03	8.11E-04	1.00E-07	2.00E-02
k_bc_c4_25tousofc2ds	c4_25	c4	us_of_c2 ds	64000		8.00E-05	5.00E-03	1.00E-07	1.00E-02
k_bc_c2tousofc2ds	c2	c4	us_of_c2 ds	62000		5.00E-04	2.00E-03	1.00E-07	8.00E-03
k_bc_c4_ag1tousofc2ds132	c4_ag1	c4	us_of_c2-ds132	55500		8.00E-05	1.00E-02	1.00E-07	2.00E-02
k_bc_c4_ag3toc2toc3	c4_ag3	c4	c2_to_c3	49000		8.00E-03	7.85E-03	1.00E-07	5.00E-02
k_bc_c2-n-24toc2toc3	c2-n-24	c4	c2_to_c3	47000		8.00E-03	8.24E-03	1.00E-07	5.00E-02
k_bc_c4_ag4toc2toc3	c4_ag4	c4	c2_to_c3	44000		8.00E-03	7.19E-03	1.00E-07	5.00E-02
k_bc_c4_55toc2toc3	c4_55	c4	c2_to_c3	41000		8.00E-03	7.97E-03	1.00E-07	5.00E-02
k_bc_c4_65atoc2toc3	c4_65a	c4	c2_to_c3	37000		1.00E-02	1.10E-02	1.00E-07	5.00E-01
k_bc_c4_60atoc2toc3	c4_60a	c4	c2_to_c3	33630		1.00E-02	1.85E-03	1.00E-07	5.00E-01
k_bc_c4_70toc2toc3	c4_70	c4	c2_to_c3	32000		2.00E-02	1.38E-03	1.00E-07	5.00E-01
k_bc_c4_ag7toc3s25bus	c4_ag6	c4	c3_s25bus	28000		8.00E-03	1.08E-03	1.00E-07	5.00E-02
k_bc_c4_ag6toc3s25bus	c4_ag6	c4	c3_s25bus	27000		8.00E-03	1.65E-03	1.00E-07	5.00E-02
k_bc_c4_ag8toc3s25bus3	c4_ag8	c4	c3_s25b_us-3	22900		8.00E-03	1.12E-03	1.00E-07	5.00E-02
k_bc_c4_125btoc3s25bus3	c4_125b	c4	c3_s25b_us-3	22000		8.00E-05	6.64E-03	1.00E-07	5.00E-02
k_bc_c4_125atoc3s25bus3	c4_125a	c4	c3_s25b_us-3	18500		8.00E-03	1.92E-03	1.00E-07	5.00E-02
k_bc_c4_ag9toc3s25bus3	c4_ag9	c4	c3_s25b_us-3	18000		8.00E-03	1.82E-03	1.00E-07	5.00E-02
k_bc_c4_150atoc3s25bus3	c4_150a	c4	c3_s25b_us-3	15000		8.00E-03	1.27E-03	1.00E-07	5.00E-02
k_bc_c4_ag11toc3s25bus3	c4_ag11	c4	c3_s25b_us-3	13500		8.00E-03	1.89E-03	1.00E-07	5.00E-02
k_bc_c4_ag14tos25b_us	c4_ag14	c4	s25b_us	10000		8.00E-03	1.09E-03	1.00E-07	5.00E-02
k_bc_c4_ag13tos25bus	c4_ag13	c4	s25b_us	9000		8.00E-03	2.79E-02	1.00E-07	5.00E-02
k_bc_c4_ag14toc4c6	c4_ag14	c5	c4_c6_connection	14400		8.00E-05	7.89E-05	1.00E-07	5.00E-04
k_bc_c5toc4c6	c5	c5	c4_c6_connection	11600		8.00E-04	6.02E-04	1.00E-07	5.00E-03
k_bc_c4_25tomud	c4	mud creek	mud creek canal	4000		8.00E-05	7.32E-05	1.00E-07	5.00E-04
k_bc_c4_ag1tomud	c4	mud creek	mud creek canal	1500		8.00E-05	7.50E-05	1.00E-07	5.00E-04
k_bc_c4_10dtonorthline	c4_10d	north line	north	4000		8.00E-05	6.45E-05	1.00E-07	5.00E-04
k_bc_c4_10etonorthline	c4_10e	north line	north	1600		8.00E-05	7.41E-05	1.00E-07	5.00E-04
k_bc_c4_75betonothline	c4_75b	northline	01-u_div1	19800		8.00E-05	6.10E-05	1.00E-07	5.00E-04
k_bc_c4_ag12tofec	c4_ag12	northline	02-u_fec	7000		8.00E-05	6.88E-05	1.00E-07	5.00E-04
k_bc_c4_100ctofec	c4_100c	northline	02-u_fec	1800		8.00E-05	7.37E-05	1.00E-07	5.00E-04
k_bc_c4_ag5tonorthlinens	c4_ag5	northlinens	01-u_c4	6360		8.00E-05	7.59E-05	1.00E-07	5.00E-04
k_bc_c4_100btonorthlinens	c4_100b	northlinens	01-u_c4	2500		8.00E-05	8.17E-05	1.00E-07	5.00E-04

Table F-3. Canal Roughness Coefficient Parameter Names, Initial, Final, Lower and Upper Values for PEST Calibration

Parameter Name	HEC-RAS Canal	HEC-RAS Canal Segment	Descrip-tion	Initial Value	Final Value	Lower Value	Upper Value
C2_N1	C2	S22_US	Manning's Canal Roughness Coefficients	0.03	0.050	0.02	0.06
C2_N2	C2	S22_US-DS_BRD		0.03	0.032	0.02	0.06
NLN1	NORTHLINE	01-U_DIV1		0.03	0.027	0.02	0.06
NLN2	NORTHLINE	02-U_FEC		0.03	0.027	0.02	0.06
NLNSN1	NORTHLINENS	01-U_C4		0.03	0.029	0.02	0.06
IMPFN1	IMP_FEEDER	FEEDER		0.03	0.032	0.02	0.06
G421N1	G421S_TW	G421_OUTLET		0.03	0.033	0.02	0.06
IMPFN2	IMP_FEEDER2	FEEDER2		0.03	0.028	0.02	0.06
MUDCN1	MUD CREEK	MUD CREEK CANAL		0.03	0.026	0.02	0.06
C3N1	C3	G93_US		0.03	0.035	0.02	0.10
C3N2	C3	EAST LOOP		0.03	0.020	0.02	0.06
C3N3	C3	WEST_LOOP		0.03	0.025	0.02	0.06
C3N4	C3	TIDAL_DS		0.03	0.024	0.02	0.06
BDN1	BIRDDR	01_U_132AV		0.03	0.031	0.02	0.06
BDN2	BIRDDR	02-U_C2		0.03	0.027	0.02	0.06
132N1	132AVE	01-U_BIRDDR		0.03	0.050	0.02	0.10
N_LN1	NORTH LINE	EAST		0.03	0.031	0.02	0.06
N_LN2	NORTH LINE	NORTH		0.03	0.029	0.02	0.06
C2XN1	C2EXT	NWWF REACH US		0.03	0.031	0.02	0.06
C2XN2	C2EXT	NWWF REACH DS		0.03	0.050	0.02	0.06
C2XN3	C2EXT	NWWF REACH-2		0.03	0.032	0.02	0.06
C5N1	C5	C4_C6_CONNECTION		0.03	0.031	0.02	0.06
C4N1	C4	US_OF_C2 US		0.03	0.020	0.02	0.06
C4N2	C4	BD CANAL		0.03	0.020	0.02	0.06
C4N3	C4	US_OF_C2 DS		0.03	0.020	0.02	0.06
C4N4	C4	US_OF_C2-DS-FEED		0.03	0.020	0.02	0.06
C4N5	C4	US_OF_C2-DS132D		0.03	0.027	0.02	0.06
C4N6	C4	US_OF_C2-DS132		0.03	0.029	0.02	0.06
C4N7	C4	C2_TO_C3-2		0.03	0.027	0.02	0.06
C4N8	C4	C2_TO_C3		0.03	0.038	0.02	0.06
C4N9	C4	C3_S25BUS		0.03	0.035	0.02	0.06
C4N10	C4	C3_S25B_US-2		0.03	0.033	0.02	0.06
C4N11	C4	C3_S25B_US-3		0.03	0.034	0.02	0.06
C4N12	C4	S25B_US		0.03	0.038	0.02	0.06

Table F-4. Boundary to Basin Conductance Parameter Names, Initial, Lower and Upper Bounds for PEST Calibration

Parameter Name	From Basin	To Basin	Description	Initial Value	Final Value	Lower Value	Upper Value
wca3bk	WCA3B	C4_10A	BC to Basin Lumped Conductance	0.01	0.0243	1.00E-07	0.1
c2bck	ENP	C2		0.01	0.0016	1.00E-07	0.1

Table F-5. Structure Gate and Orifice Flow Coefficient Parameter Names, Initial, Lower and Upper Bounds for PEST Calibration

Parameter Name	Structure	Type	Description	Initial Value	Final Value	Lower Value	Upper Value
G93_GC	G-93	gate	Gate and Orifice Flow Coefficients	0.6	0.678	0.3	0.8
G93_OC	G-93	orifice		0.8	0.900	0.5	0.9
S22_GC	S-22	gate		0.6	0.642	0.3	0.8
S22_OC	S-22	orifice		0.8	0.791	0.5	0.9

S25_GC	S-25B	gate		0.6	0.556	0.3	0.8
S25_OC	S-25B	orifice		0.8	0.546	0.5	0.9

Observations Data and Weights

For model calibration, PEST separates calibration observations into groups. The group selection was determined by the combination of history-matching variable type (stage, flow) and performance statistic (sum of stage and flow squared errors). Calibration weighting factors for each individual observation were determined as the product of weight for the individual observation and the weight for the group that the observation belongs to. Individual observation weights are generally function of data reliability and accuracy; whereas group weights are function of the importance of the variable and statistic, and the relative magnitude of the variable compared to other variables. For example, weights for structure flows are much smaller than weights for stages since the magnitude of structure and canal flows, typically measured in cubic feet per second (cfs) is much larger than the magnitude of canal stages (ft NGVD).

Twelve stage and five flow monitor sites (**Figure B-11, Appendix B**) were used for model calibration had 15-minute data but the history matching process was done at hourly intervals to reduce the size of the data sets. As explained before, several stage and flow data sets were not used for calibration because of proximity of gauges, noise and gaps in the data sets. The observation period was reduced to 29 days to exclude two days at the beginning of the simulation that may have additional error due to initial conditions in the canals. A total of 19,535 hourly and daily stage and flow observations were used for calibration of the HEC-RAS model. **Table F-6** summarizes the number of hourly observations for model calibration. Adjustments to the observation weights were made to add temporal relevance of the data to give more importance to stage and flow observations around the peak of the storm event which is the time when the structure gates were fully opened and the flooding peak stages occurred along the canals.

Table F-6. PEST Observations for HEC-RAS Model

Gauge	Type	PEST Observation Group	No. of Observations
C2.74	Canal stage	c2.74_h	1,221
G-93	Canal stage	c3_g93_hw	1,221
G-93	Canal stage	c3_g93_tw	1,221
T5W	Canal stage	c4_t5_hw	1,221
S-25B	Canal stage	c4_s25b_hw	1,221
S-25B	Canal stage	c4_s25b_tw	1,221
C4.CORAL	Canal stage	c4_coral_hw	1,221
S-380	Canal stage	c6_s380_tw	1,221
S-380	Canal stage	c6_s380_hw	1,221
S-22	Canal stage	c2_s22_tw	1,221
S-22	Canal stage	c2_s22_hw	1,221
S-25B	Flow	c4_s25b_q	1,221
S-25B	Flow	c3_g93_q	1,221
C4.CORAL	Flow	c4_coral_q	1,221
S22	Flow	c2_s22_q	1,221
c4-n-4	Sub-basin stage	c4-n-4	1,221
c4-n-3	Sub-basin stage	c4-n-3	1,221
c2	Sub-basin stage	c2_stg	61
ag12	Sub-basin stage	ag12_stg	61
10a	Sub-basin stage	10a_stg	1,221

10b	Sub-basin stage	10b_stg	1,221
		TOTAL:	**19,536**

Calibration Criteria

Water level tolerances of ±0.5 foot have been established in previous Miami-Dade County modeling planning studies models for wetland and groundwater levels, including North Miami-Dade County MODFLOW (**Wilsnack, et al., 2000). Restrepo, et al. (2001**) used an absolute tolerance of ±1.0 foot as a calibration target for groundwater stages in the south Miami-Dade County MODFLOW model. For planning studies where regional or sub-regional models are used to predict water levels and flows over a period of several years, the 0.5 ft stage tolerances may be adequate to establish a successful calibration, however, for flooding studies where hydraulic models are used over relatively small areas or periods of time, more restrictive criterion may be needed. A canal stage ±0.5 ft calibration criteria was used in a previous calibration study (**SFWMD, 2010b**) for bias and RMSE. For this exercise a ±0.50 ft error tolerance for bias and RMSE was chosen for stages in the canals.

Calibration Results

Given the limited amount of observation data available for model calibration, most of the efforts with PEST were focused on adjusting the observation weights for each of the residual sites included in the objective function. As explained before, observations weights are used to include proper amount of information in the objective function so that most of the residuals are included in the optimization process without having one or more sites overwhelming the process. Flow site residuals were reduced to contribute same level of information as stage residuals. Weights during the peak of the storm event were also increased over residuals prior and later at the tail end of the event. **Table F-7** summarizes the error contributions from each residual site from the initial and the final optimization iteration. As this table shows, the final percent contribution to error from each residual including both stages and flows are fairly evenly distributed with no one gauge outweighing the others.

According to **Table F-7,** for computed canal stages, the largest contribution to error as a percent of the total error was in gauge T5W (9.6 percent), followed by S-22 tailwater (8.7 percent) and G-93 headwater and S-25B tailwater both with 7.0 percent. The highest residuals for flows are at structure S-25B with 13.5 percent contribution to the total error and gauge, S-22 with 6.3 percent and C4.Coral gauge with 5.3 percent and the structure. The largest error contribution from flows at structure S25B is in line with the larger uncertainties in the flow/stage data at structures S-25B as explained previously in the Calibration Targets section of this report. The table also shows the reduction or increase in error at each individual gauge from the initial and final set of parameters during the PEST optimization process. The gauge with largest error increase was the tailwater stage at structure G-93 (c3_g93_hw) with an increase of 9.2% to the contribution to total error in the objective function. Conversely, the gauge with largest reduction of contribution to total error in the objective function was C4-S380_HW (headwater stage at structure S-380) with a reduction of 61.6% to the contribution to the total error. The average error reduction for all gauges was 13.2%, including stages and flows.

Table F-8 summarizes for each canal stage observation gauge, the computed bias and RMSE used for relative comparison of model fit. The average stage bias and RMSE for all the gauges were -0.07 ft and 0.29 ft respectively. At individual stage gauges, the largest stage biases were computed in the western portion of the C-4 canal, at gauges S380_HW, S-22_TW and S-25B_TWwith biases of -0.57 ft, 0.19 ft and 0.16 ft and the largest RMSE corresponded to S380_HW, G93_HW and S22_TW with values of 0.77 ft, 0.27 ft and 0.24 ft, respectively. The gauges with the smallest biases were G93_TW (0.01 ft), C4.Coral (-0.01 ft) and S380_TW (0.03 ft) and smallest RMSE were C2.74 (0.11 ft), G93_TW (0.16 ft) and S22_HW (0.17 ft). Also in Table F-8 are included the bias and RMSE for computed sub-basin stages included in the calibration. Of the six sub-basins, C4-10B, C4-10A and C4-AG12 had the largest bias values of 0.51 ft, -0.32 ft and -0.14 ft and sub-basins C4-10B, C4-N-3 and C4-N-4 had the largest RMSE values respectively.

Table F-7. PEST Calibration Initial and Final Contributions to Total Error from each Observation

Observation	Location	First PEST Iteration		Final PEST Iteration		Percent Change in Objective Function ϕ (%)
		Contrib. to Total Error	% of Contrib. to Total Error	Contrib. to Total Error	% of Contrib. to Total Error	
c4_s380_hw	C-4 Canal – Headwater stage at structure S-380	218.3	14.0	83.8	6.9	-61.6
c4_s380_tw	C-4 Canal – Tailwater stage at structure S-380	48.8	3.1	46.8	3.9	-4.3
c2ext_74	C-2 Extension Canal - stage at Gauge C2.74	40.2	2.6	28.1	2.3	-30.2
c3_g93_hw	C-3 Canal – Headwater stage at structure G-93	77.7	5.0	84.9	7.0	9.2
c3_g93_tw	C-3 Canal – Tailwater stage at Structure G-93	29.9	1.9	31.1	2.6	4.0
c4_t5_h	C-4 Canal stage at gauge T5	142.9	9.2	116.5	9.6	-18.5
c4_s25b_hw	C-4 Canal – Headwater stage at structure S-25B	81.0	5.2	65.2	5.4	-19.5
c4_s25b_tw	C-4 Canal – Tailwater stage at structure S-25B	88.8	5.7	84.3	7.0	-5.0
c4_coral_h	C-4 Canal stage at gauge C4.Coral	62.8	4.0	52.3	4.3	-16.7
c2_s22_hw	C-2 Canal – Headwater stage at structure S-22	52.9	3.4	52.4	4.3	-0.9
c2_s22_tw	C-2 Canal – tailwater stage at Structure S-22	106.1	6.8	105.1	8.7	-1.0
c4_s25b_q	C-4 Canal - Flow at structure S-25B	198.6	12.8	164.0	13.5	-17.4
c3_g93_q	C-3 Canal - Flow at structure G-93	4.0	0.3	4.3	0.4	7.2
c4_coral_q	C-4 Canal - Flow at gauge C4.Coral	110.1	7.1	63.8	5.3	-42.0
c2_s22_q	C-2 Canal - Flow at structure S-22	72.4	4.7	76.9	6.3	6.1
c4-n-4	Stage in sub-basin C4-N-4	96.7	6.2	65.8	5.4	-32.0
c4-n-3	Stage in sub-basin C4-N-3	89.8	5.8	51.2	4.2	-43.0
c2_stg	Stage in sub-basin C2	9.7	0.6	8.3	0.7	-13.8
ag12_stg	Stage in sub-basin C4-AG12	3.7	0.2	3.7	0.3	-0.7
10a_stg	Stage in sub-basin C4-10a	5.6	0.4	6.0	0.5	6.1
10b_stg	Stage in sub-basin C4-10B	16.8	1.1	16.2	1.3	-3.5
	SUM:	**1,556.9**	**100.0**	**1,210.6**	**100.0**	**AVG = 13.2 %**

Table F-8. Computed Canal Stage Bias and RMSE from HEC-RAS Calibration

Calibration Site	Canal/Sub-basin	Type	Stage Bias (ft)	Stage RMSE (ft)
S22_HW	C-2 canal	Canal Stage	0.05	0.17
S22_TW	C-2	Canal Stage	0.19	0.24
G93_HW	C-3	Canal Stage	0.12	0.27
G93_TW	C-3	Canal Stage	0.01	0.16
S25B_HW	C-4	Canal Stage	-0.06	0.16
S25B_TW	C-4	Canal Stage	-0.16	0.22
T5W	C-4	Canal Stage	-0.09	0.21
C4.Coral	C-4	Canal Stage	-0.01	0.18
S380_HW	C-4	Canal Stage	-0.57	0.77
S380_TW	C-4	Canal Stage	0.03	0.18
C2.74	C-2_EXT	Canal Stage	-0.07	0.11
C2	Sub-basin	Stage	0.08	0.35
C4-10A	Sub-basin	Stage	-0.32	0.35
C4-10B	Sub-basin	Stage	-0.51	0.51
C4-AG12	Sub-basin	Stage	-0.14	0.27
C4-N-3	Sub-basin	Stage	0.11	0.43
C4-N-4	Sub-basin	Stage	0.14	0.35
		AVG=	**-0.07**	**0.29**

These error statistics indicate that most of the gauges met the ±0.5 error tolerance for stage bias calibration with exception of gauges S380_HW and sub-basin C4-10B and, for RMSE, S380_HW and sub-basin C4-10B.

For computed canal flows, **Table F-9** summarizes bias and RMSE at five gauge locations. The largest bias corresponds to structure S-25B and C4.Coral with flow biases of -46.6 cfs and 46.1 cfs respectively. Gauges S-25B (133.0 cfs) and S22_Q (91.0 cfs) had the largest RMSE values. As explained before, flow bias and RMSE were not used as part of the calibration criteria however, the error statistics were computed and shown here for comparison purposes only.

Table F-9. Computed Canal Flow Bias and RMSE from HEC-RAS Calibration

Calibration Site	Canal	Type	Flow Bias (cfs)	Flow RMSE (cfs)
S22_Q	C-2	Structure Flow	-28.5	91.0
G93_Q	C-3	Structure Flow	6.3	25.1
S25B_Q	C-4	Structure Flow	-46.6	133.0
C4.Coral	C-4	Canal Flow	46.1	84.9
		AVG:	**-5.7**	**83.5**

Model computed and observed stage and flow values at individual gauges are presented in **Figures F-9 through F-31** and close-up plots during TS Isaac (August 27 through September 6, 2012) in **Figures F-32 through F-41**. **Figure F-9** shows the computed and observed stages at gauge S380_HW in the westernmost reach of the C-4 Canal. The computed stages consistently show large deviations from the observed values except at the peak of the TS Isaac event. Structural errors and boundary effects are likely the cause of the model misfit. In **Figure F-10** the canal stages at S380_TW reflect the effects of the C-4 Impoundment pump operations (G-420 and G-422) at the peak of the TS Isaac on August 27 when the pumps became active producing a sudden decline on canal stages until the pumps feeding the C-4 Impoundment were turned off (**Figure F-33**). Similarly, on September 22, another significant storm event triggered pump operations at eh C-4 Impoundment resulting in another sudden plunge in canal stages over-estimated by the model. As the figure shows, the model was able to closely reproduce the observed canal stages even though the decline in stage was over-predicted when the pumps were shut off. Downstream of Structure S-380, at the T5W gauge near the intersection of the C-4 Canal with the C-2 Extension Canal, the model stages closely follow the observed values (**Figure F-11**) from the beginning of the calibration period through the end of TS Isaac and under-predicted the stages afterwards resulting in a stage bias of -0.09 ft and RMSE of 0.21 ft. **Figure F-34** is a close-up look at the computed stages during TS Isaac at this location and shows that the difference of computed and observed peak stages on August 27 was 0.08 ft and a time lag of 1 hour and 30 minutes.

Figure F-12 shows the computed and observed stages for gauge C4.Coral, east of the City of Sweetwater. The model consistently under-predicted the computed stages with a bias of -0.18 ft and RMSE of 0.24 ft. The hourly stage data at this location are from a USGS gauge seems to have erroneous data prior around the time of the peak of TS Isaac. Corresponding flows at this location has several periods of missing data as shown in **Figure F-20** which shows the computed and observed flows for the calibration period. At the downstream end of the C-4 Canal (Structure S-25B), the observed and computed headwater stage is presented in **Figures F-13** and in more detail during TS Isaac, in **Figure F-36**. At this location, the model reproduced the stages with a bias of -0.06 ft and RMSE of 0.16 ft and the peak stage did not occur at the time when stages peaked upstream on the canal due to low tide conditions downstream of the structure, however, the stage difference between the computed and observed stages at the peak time of the storm was 0.18 ft. On the tailwater side of structure S-25B, the stages were computed with a bias of -0.16 ft and RMSE of 0.22 ft. **Figure F-14** shows the computed and observed tailwater stages for the calibration period. Computed and observed flows at Structure S-25B are shown in **Figure F-21** while **Figure F-37** shows the same in more detail at the time of peak of the storm on August 27 at 11:30 am with an observed peak flow value of 1,929 cfs and corresponding computed value of 1,687 cfs.

In the C2-Extension canal the only gauge with data for calibration was C2.74 where the model computed stages with a bias of 0.-07 ft and RMSE of 0.11 ft, however, the computed peak stage for TS Isaac was under-estimated by 0.4 ft as shown in **Figure F-15**. In the C-3 Canal, at the G-93 structure, the computed stages at the headwater side of the structure (gauge G93_HW) had a bias of 0.12 ft and RMSE of 0.27 ft and are shown in **Figure F-16** and in more detail during the peak of TS Isaac, in **Figure F-38**. The tailwater stages with a bias 0.01 of ft and RMSE of 0.16 ft are shown in **Figure F-17**. The peak stage occurred on August 26th as a result of partial gate openings. Observed and computed flows at Structure G-93 are shown in detail in **Figures F-22 and F-39**. In the C-2 Canal, on the headwater side of Structure S-22, the computed and observed canal stages are shown in **Figures F-18 and F-40** while flows are shown in **Figures F-23 and F-41**. **Figure 40** indicates the peak stage during TS Isaac occurred on August 26 at hour 17:00 while **Figure F-41** shows the peak flow occurred on August 27 at 11:00am. The stage bias and RMSE at this location in the model were, respectively, 0.05 ft and 0.17 ft. **Figure F-19** shows the computed and 15-min observed stages on the tailwater side of the structure.

In addition to canal water levels, stage data from six groundwater gauges were used in the calibration of computed stages in sub-basins. **Table F-8** also includes the bias and RMSE for each of these sub-basins included in the calibration. One problem with using a single groundwater gauge per sub-basin in the model is the approximation of the water levels in the sub-basins as level pool stages while in reality there could be significant water level gradient in some sub-basins, particularly those affected by groundwater pumpage and near the canals. A water level gradient is evident in sub-basin C4-10A where groundwater level data are available at three locations: gauge G-1488 near the L30 Canal, gauge G-975 in the northern portion of the basin near the NW Wellfield (NWWF) and gauge G-594 on the west side of the Dade-Broward Levee. All three gauges show significant differences in the water level data during the period of calibration. In this study the water level data from gauge G-975 were selected to represent the stages in sub-basin C4-10A. **Figure F-24** illustrates the computed stages in sub-basin C4-10A corresponding geographically to the Pennsuco Wetlands in the western portion of the model domain with a stage bias of -0.32 ft and RMSE of 0.35 ft. The persistent negative bias at this location is likely a result of structural error and boundary effects in representing the seepage flows from the Water Conservation Are 3B to the C-4 Basin and the effects of wellfield pumpage in the Northwest Wellfield, just east of the wetland. **Figure F-25** show the stages in sub-basins C4-10B (TARMAC gauge) with stage bias of -0.51 ft and RMSE of 0.51 ft. Data for sub-basins C4-10D (USGS G-3264A) and C4-10E (USGS G-3676) were not used in calibration due to questionable quality of the data but were used for post-calibration comparison and are shown in **Figures F-26 and F-27**. The sub-basins corresponding to the C-4 Impoundment (C4-N-3 and C4-N-4) were compared with the tailwater stages of the G-420 and G-422 pumps respectively. Although not representative of the stage data at the center of the storage areas, the data were considered good for calibration purposes. **Figures F-28 and F-29** show the computed and observed stages for sub-basins C4-N-3 and C4-N-4 with corresponding stage bias of 0.11 ft and 0.1 ft and RMSE of 0.43 ft and 0.35 ft respectively. In the eastern side of the C-4 Basin, near the Miami International Airport, the computed stages in sub-basin C4-AG12 were history-matched with the observed stages at observation well G-3329 with a resulting bias and RMSE of -0.14 ft and 0.27 ft, respectively. Finally, the stages in sub-basin C2 were calibrated against the stages in observation well G-3572 as shown in **Figure F-31**. The resulting stage bias and RMSE in this sub-basin were 0.08 ft and 0.35 ft.

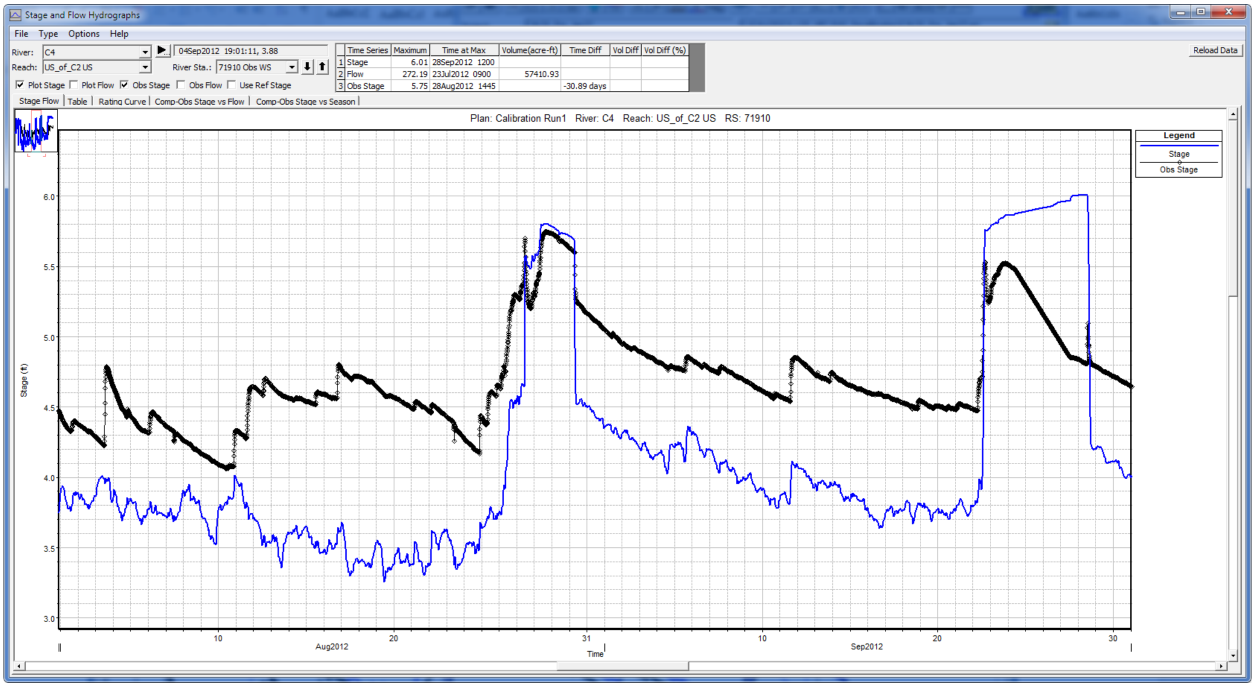

Figure F-9. Computed vs Simulated Stage in the C-4 Canal at S-380_HW Gauge

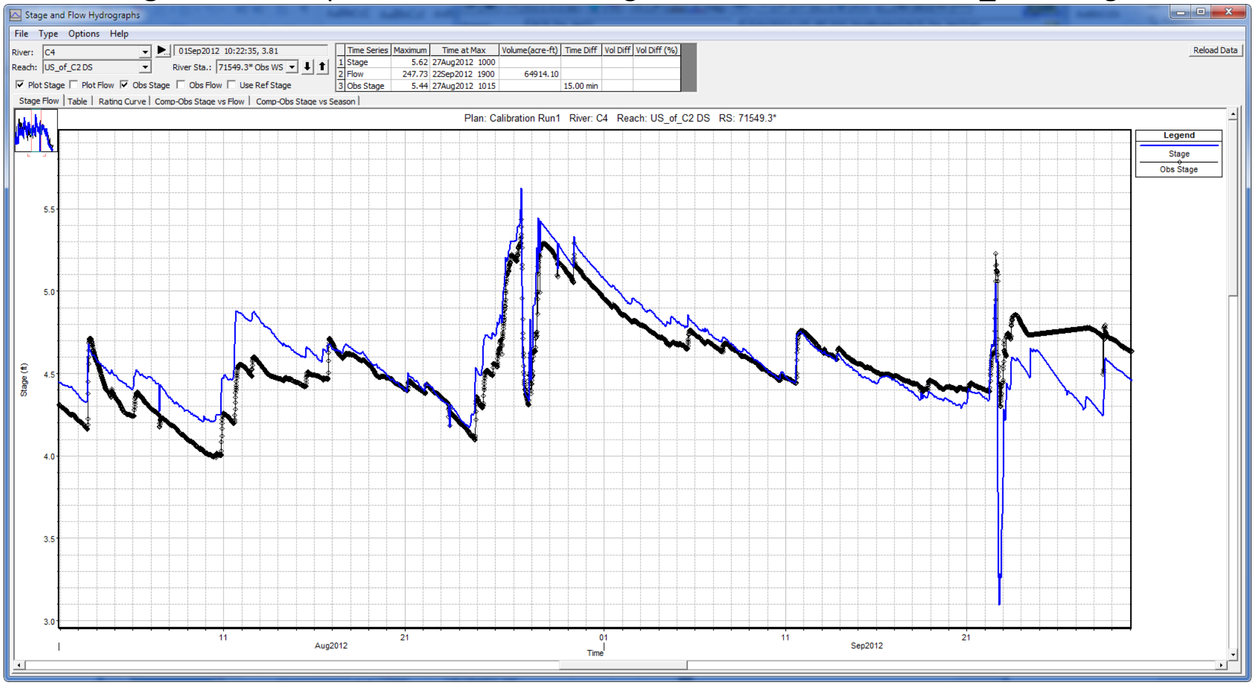

Figure F-10. Computed vs Simulated Stage in the C-4 Canal at Gauge S380_TW

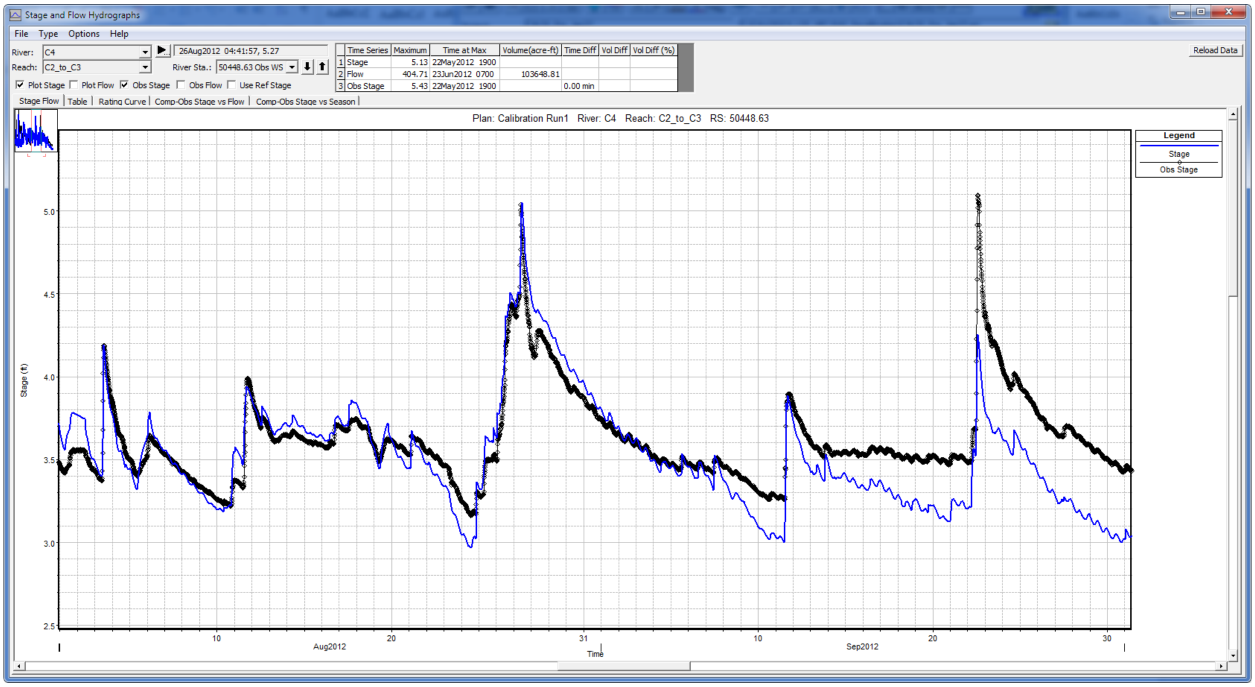

Figure F-11. Computed vs Simulated Stage in the C4 Canal at T5W gauge

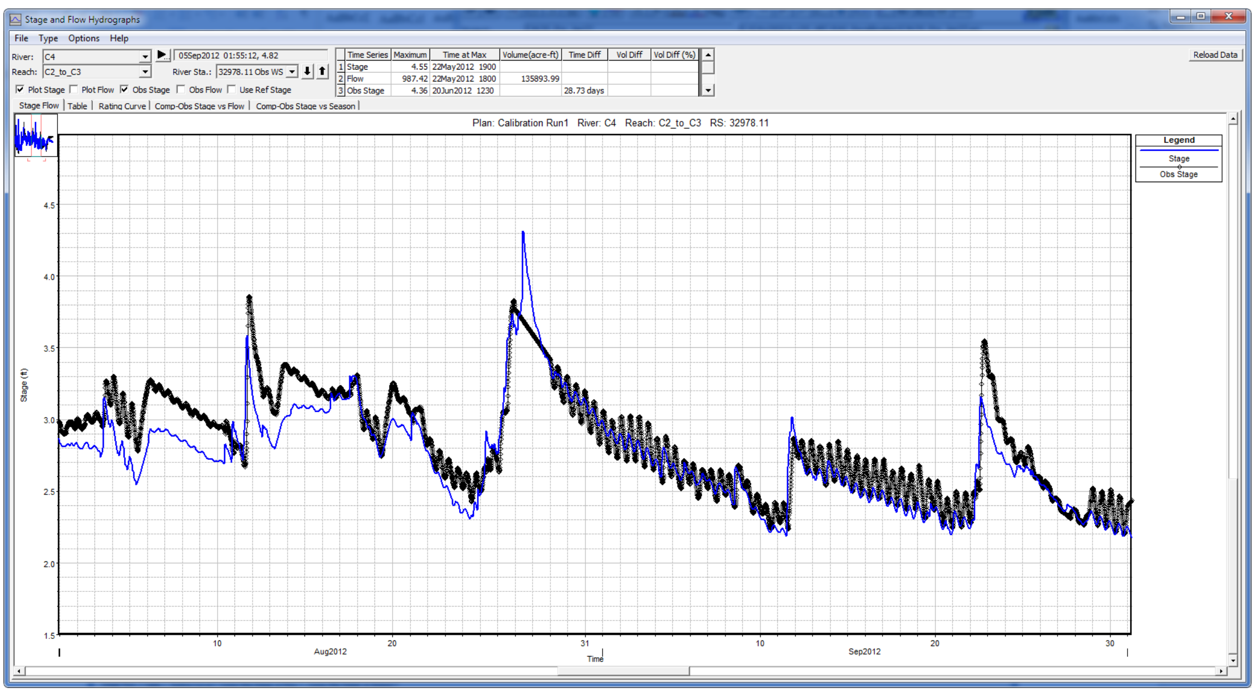

Figure F-12. Computed vs Simulated Stage in the C-4 Canal at C4.Coral Gauge

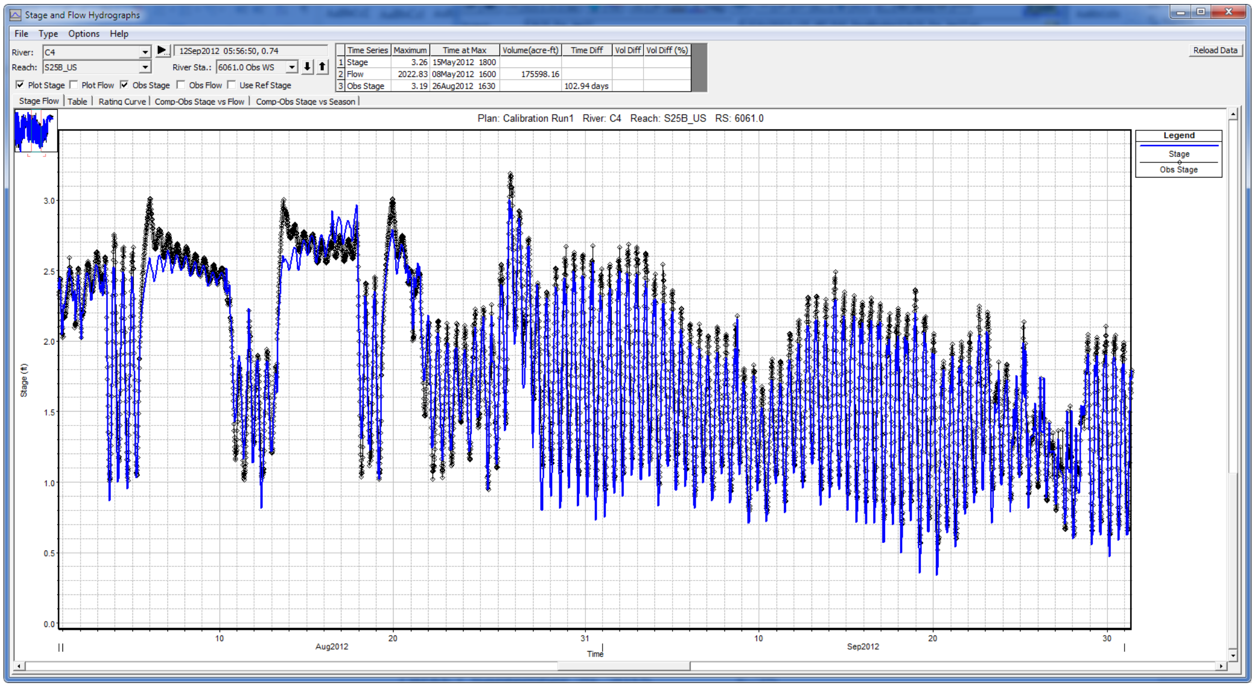

Figure F-13. Computed vs Simulated Stage in the C-4 Canal at S-25B_HW Gauge

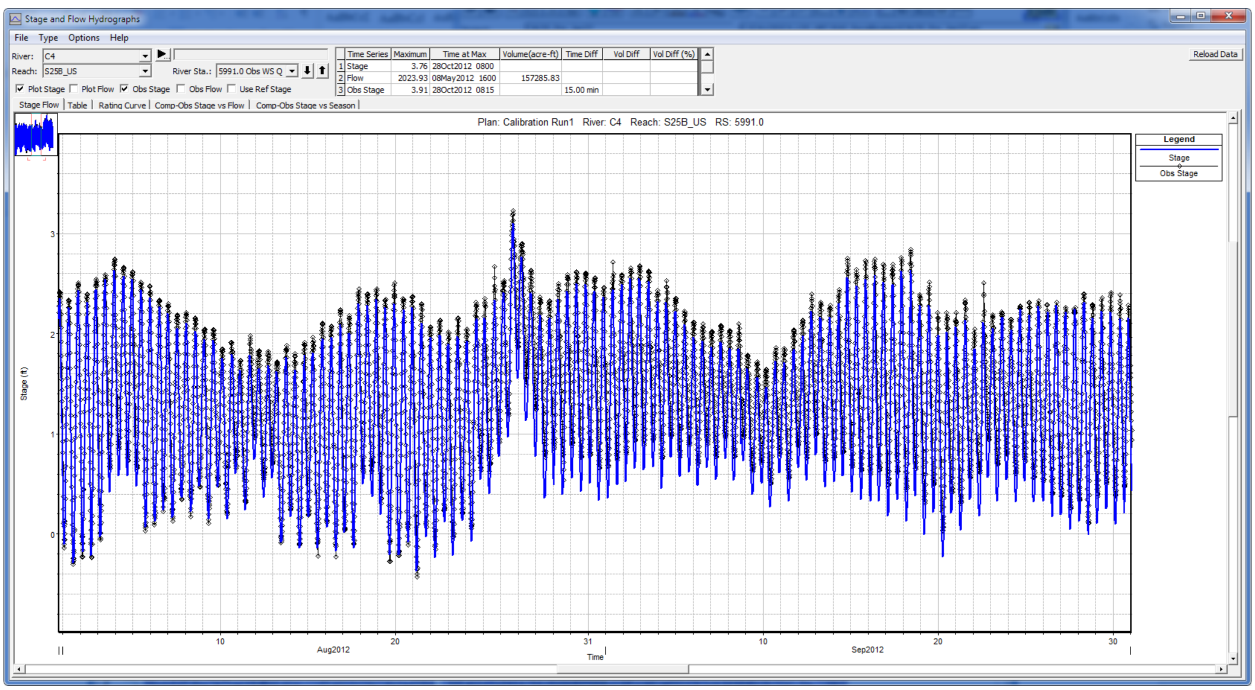

Figure F-14. Computed vs Simulated Stage in the C-4 Canal at S-25B_TW Gauge

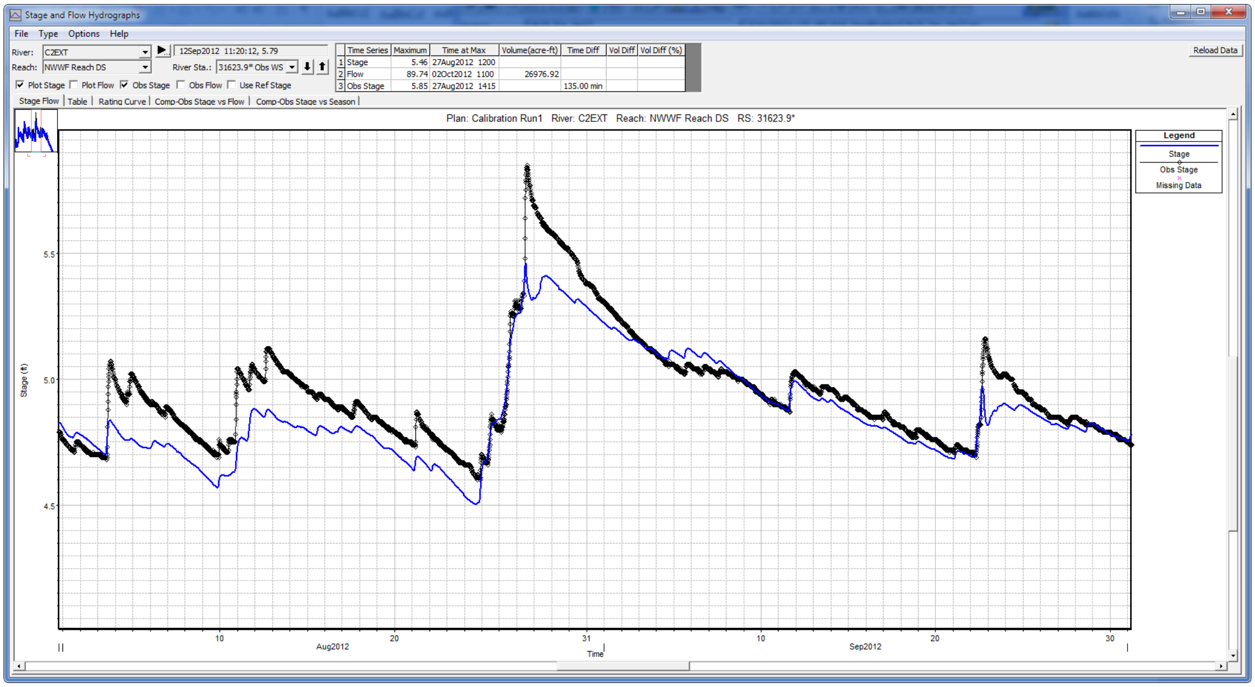

Figure F-15. Computed vs Simulated Stage in the C-2 Extension Canal at C2.74 Gauge

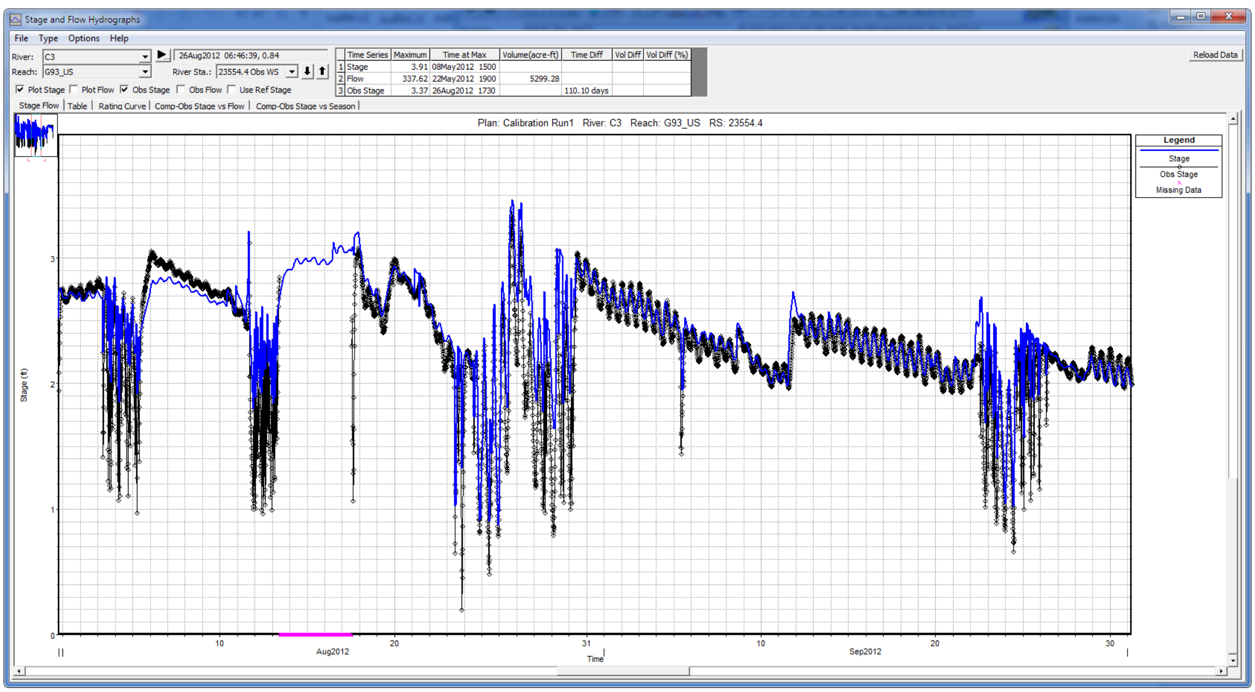

Figure F-16. Computed vs Simulated Stage in the C-3 Canal at G-93_HW Gauge

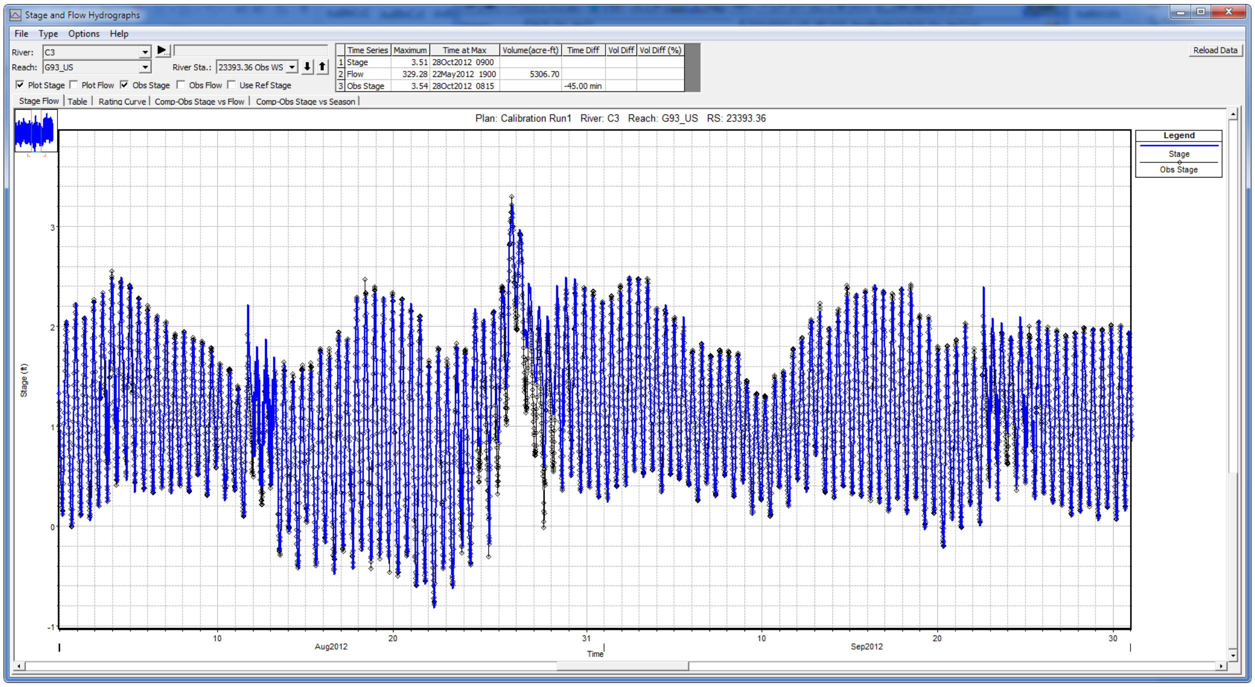

Figure F-17. Computed vs Simulated Stage in the C-3 Canal at G-93_TW Gauge

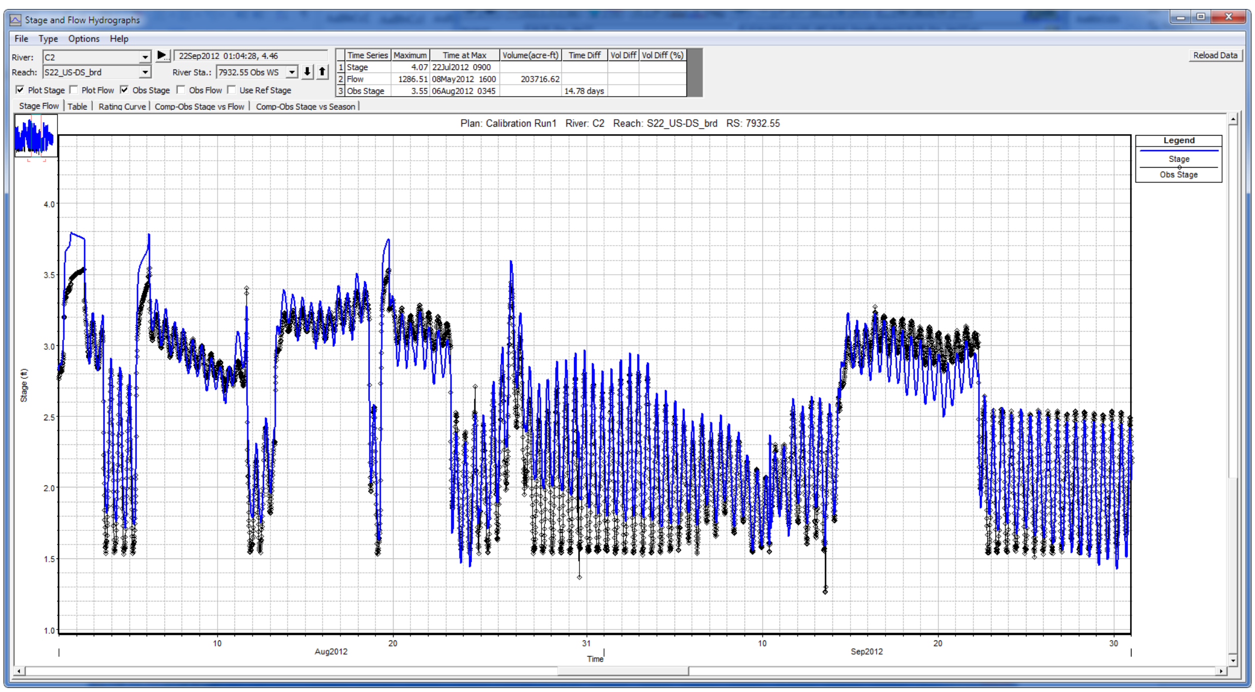

Figure F-18. Computed vs Simulated Stage in the C-2 Canal at S-22_HW gauge

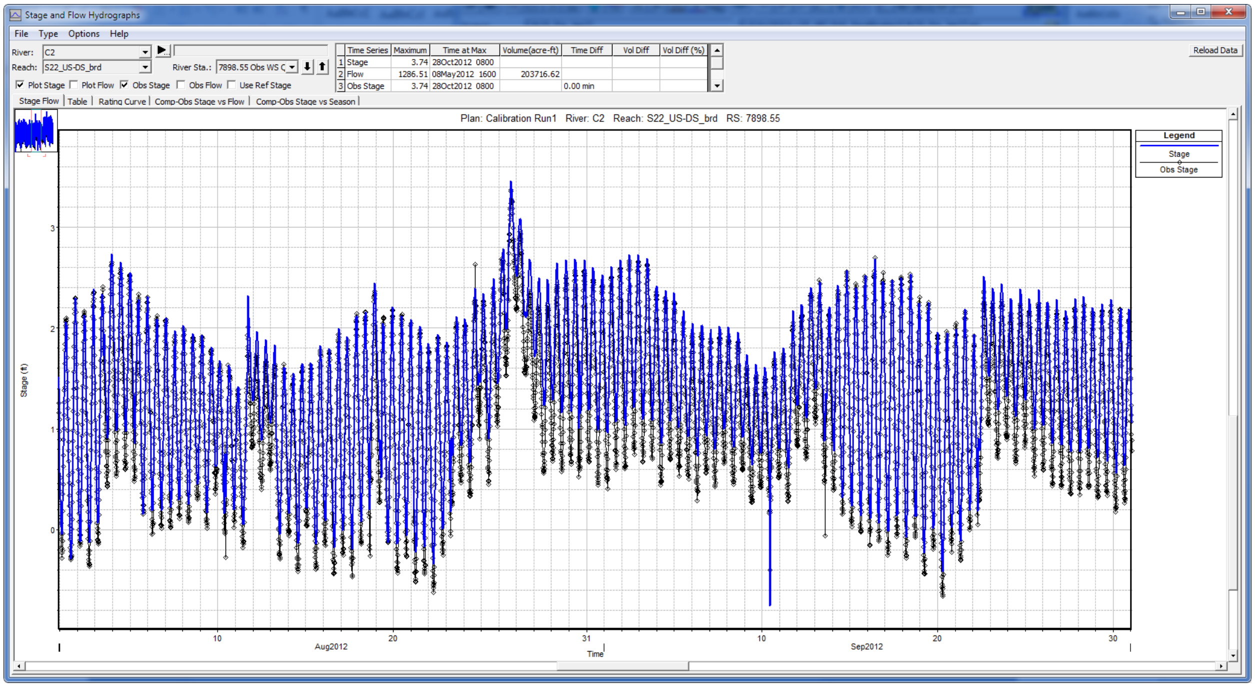

Figure F-19. Computed vs Simulated Stage in the C-2 Canal at S-22_TW gauge

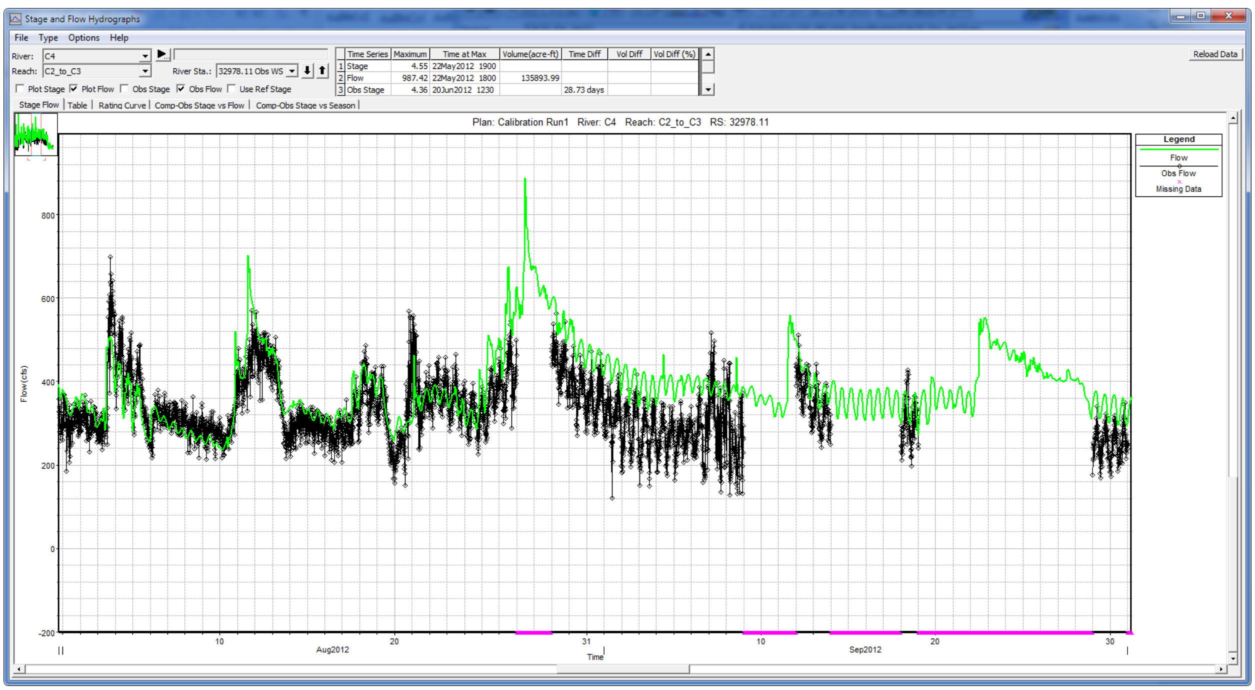

Figure F-20. Computed vs Simulated Flow in the C-4 Canal at C4.Coral Gauge

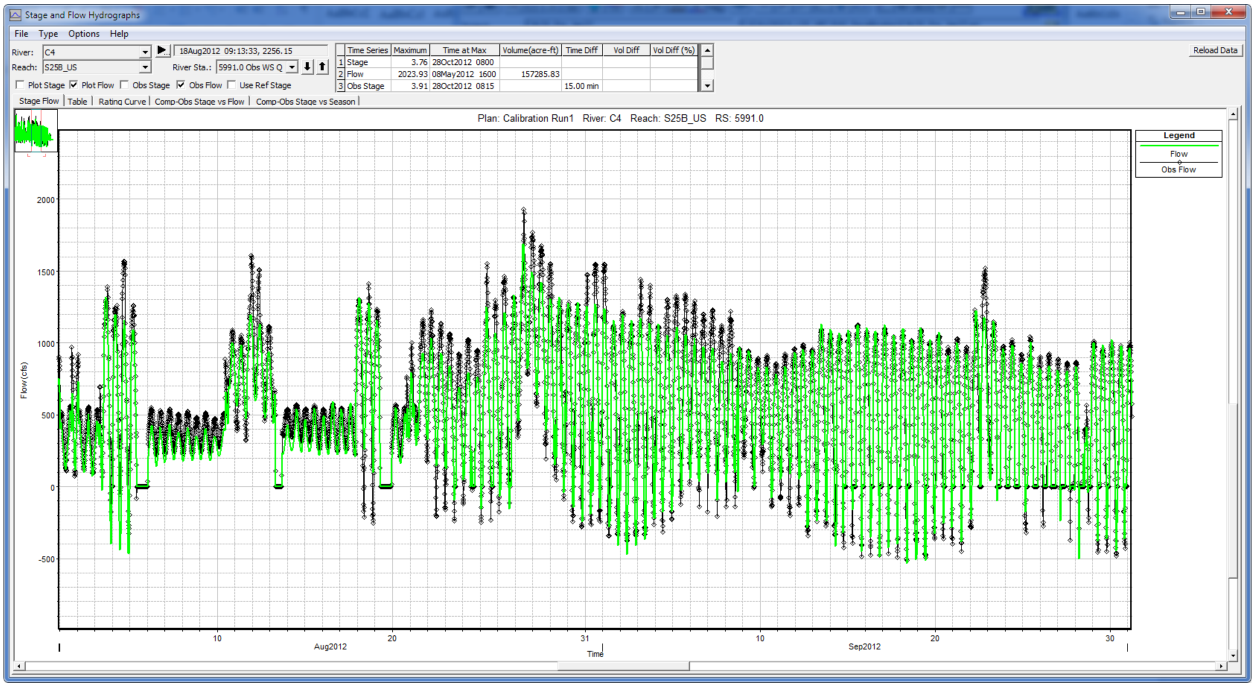

Figure F-21. Computed vs Simulated Flow in the C-4 Canal at S-25B Gauge

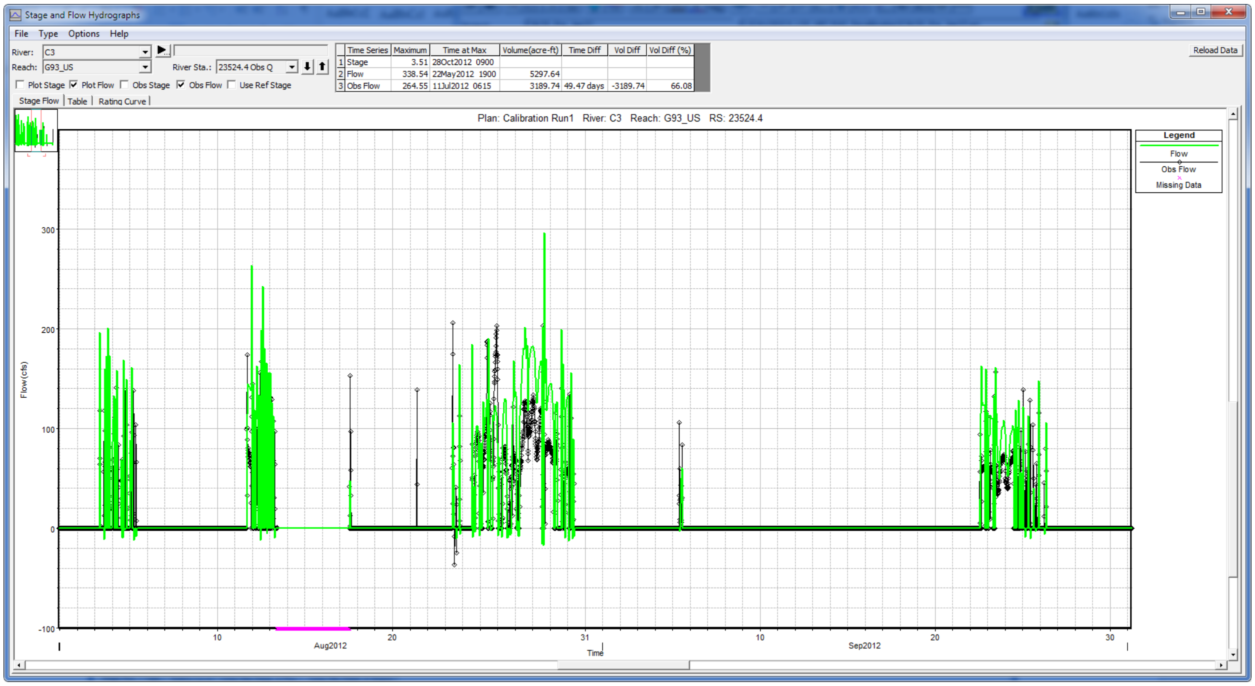

Figure F-22. Computed vs Simulated Flow in the C-3 Canal at G-93 Gauge

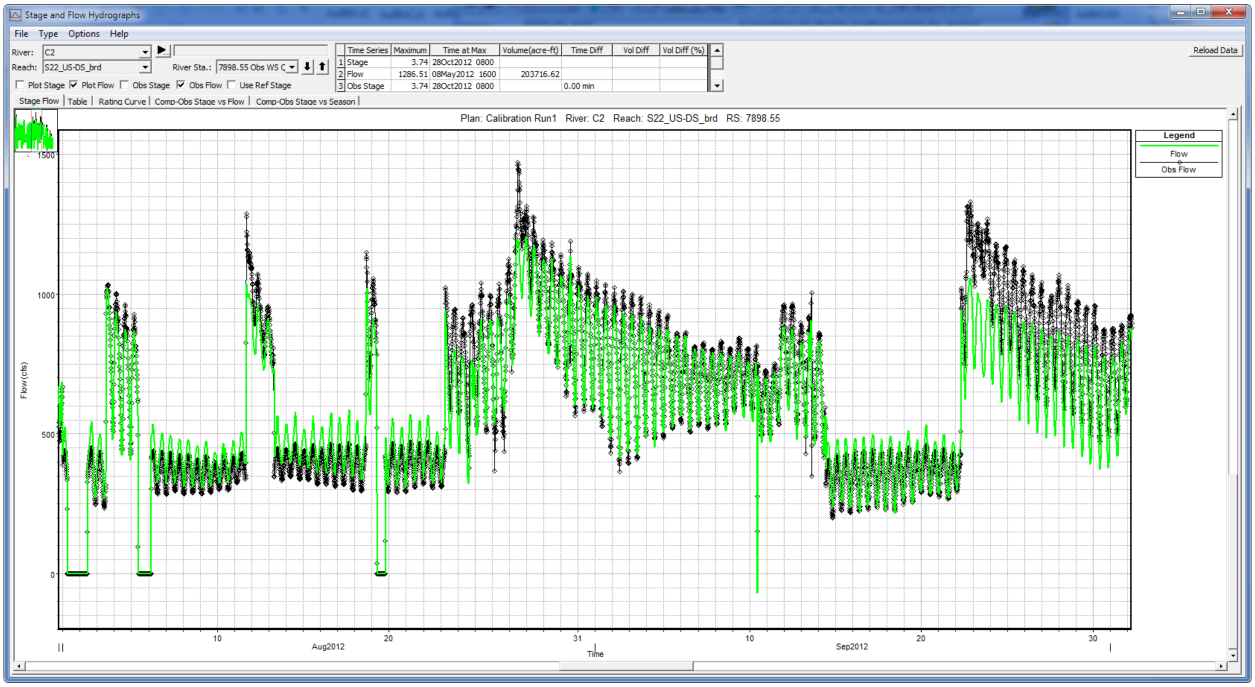

Figure F-23. Computed vs Simulated Flow in the C-2 Canal at S-22 Gauge

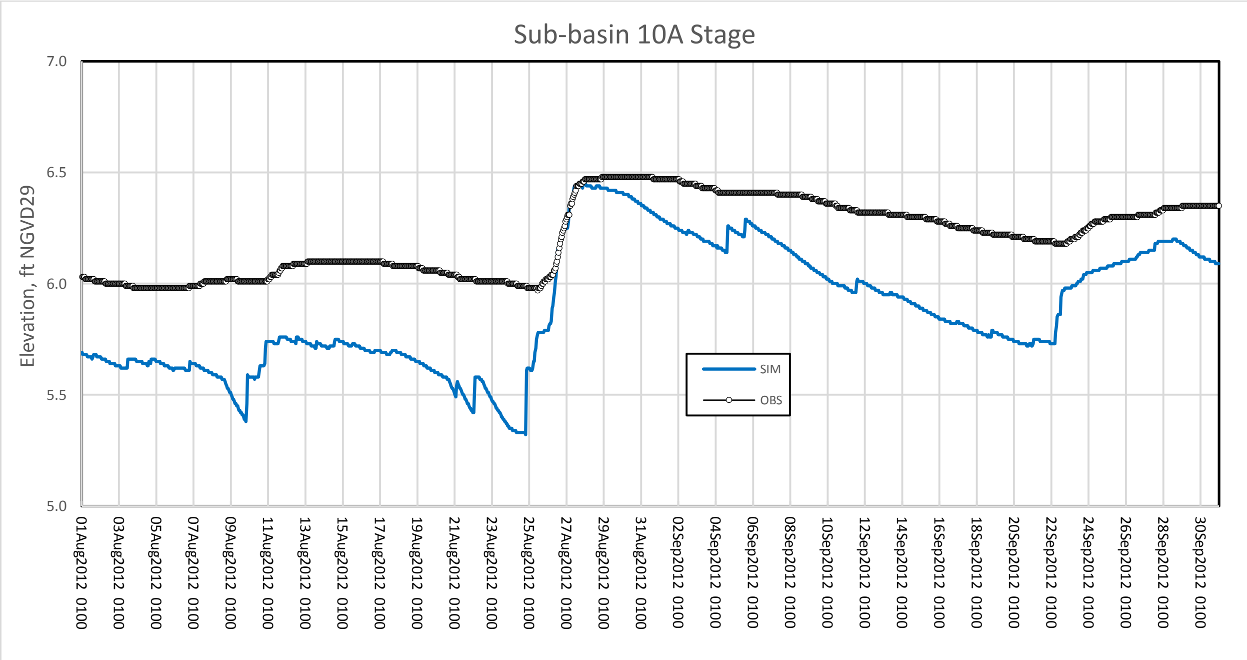

Figure F-24. Computed vs Simulated Flow in Sub-basin C4_10A (USGS gauge G-975)

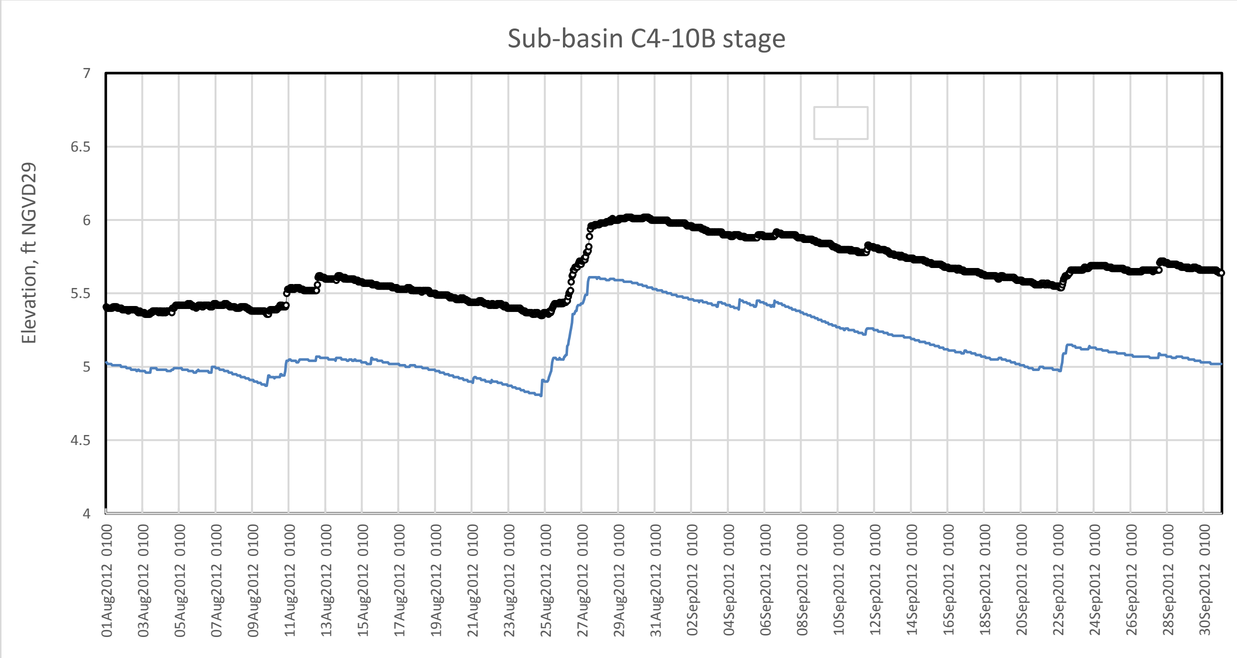

Figure F-25. Computed vs Simulated Stage in Sub-basin C4_10B (DERM gauge Tarmac 1)

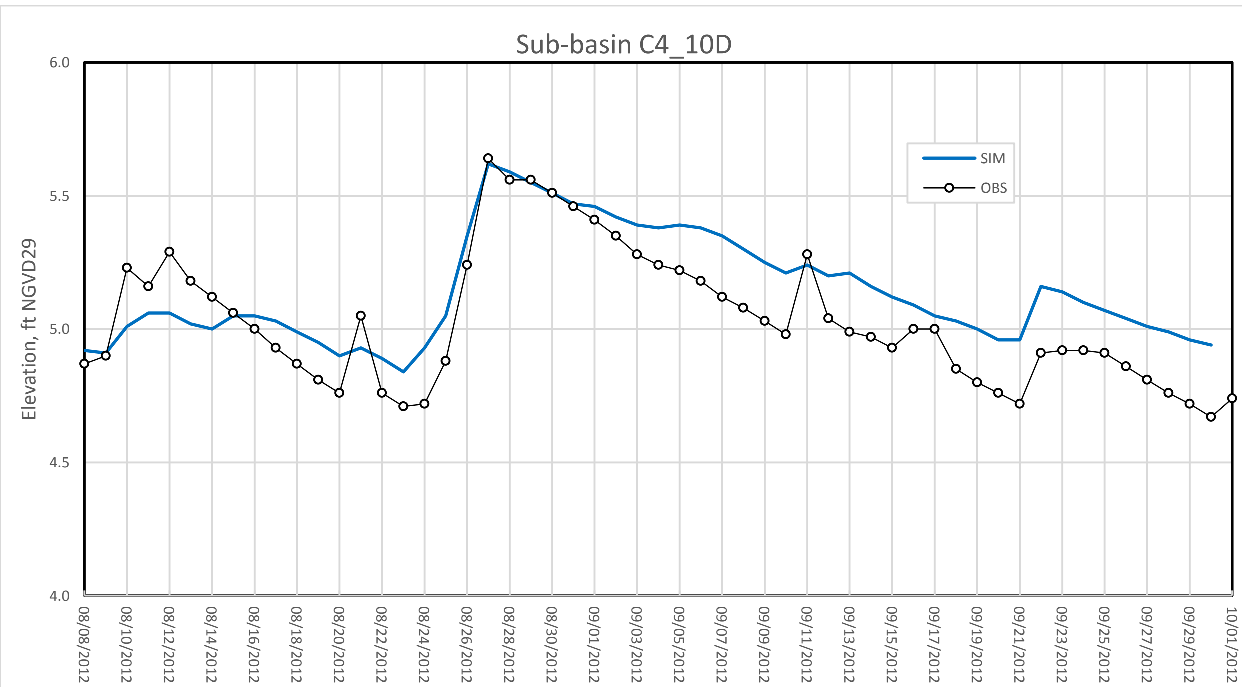

Figure F-26. Computed vs Simulated Stage in Sub-basin C4_10D (DERM gauge USGS G-3264A)

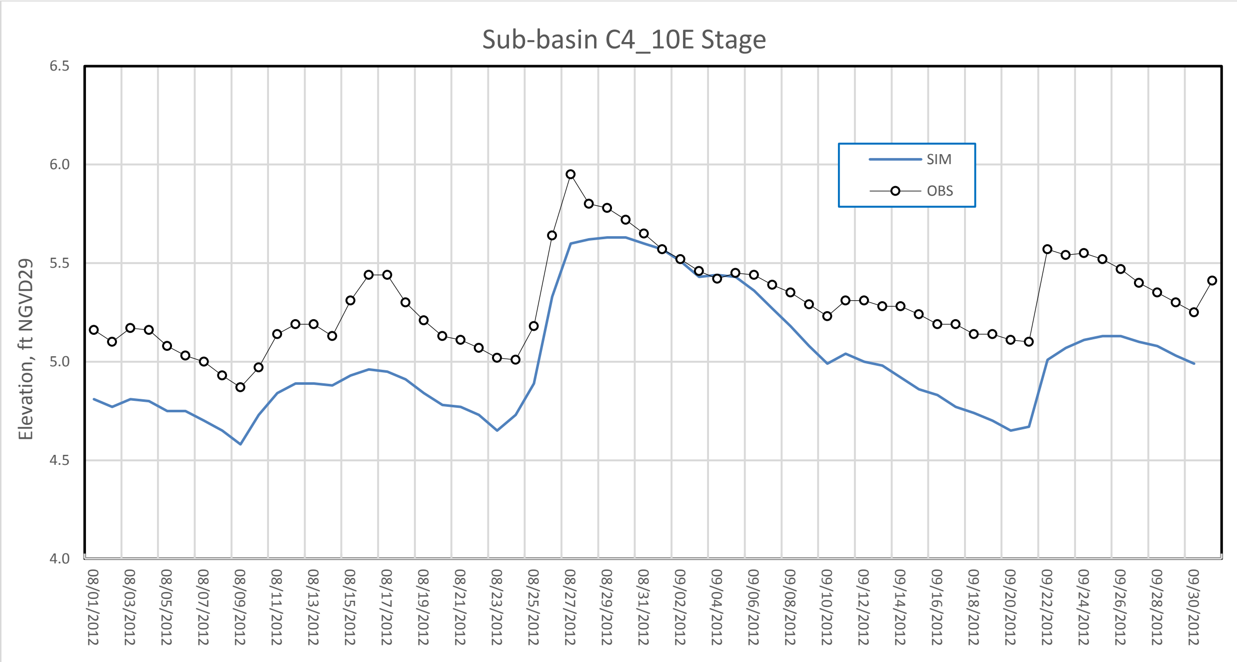

Figure F-27. Computed vs Simulated Stage in Sub-basin C4_10E (USGS gauge G-3676)

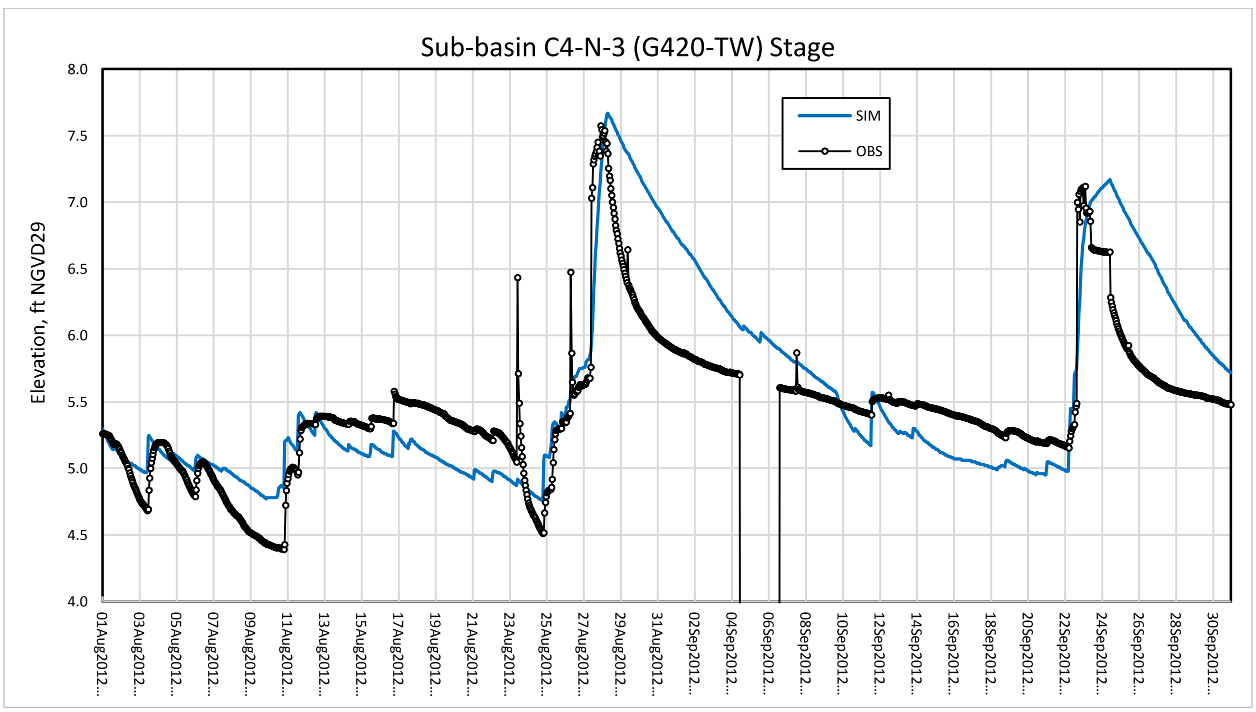

Figure F-28. Computed vs Simulated Stage in Sub-basin C4-N-3 (gauge G-420 TW)

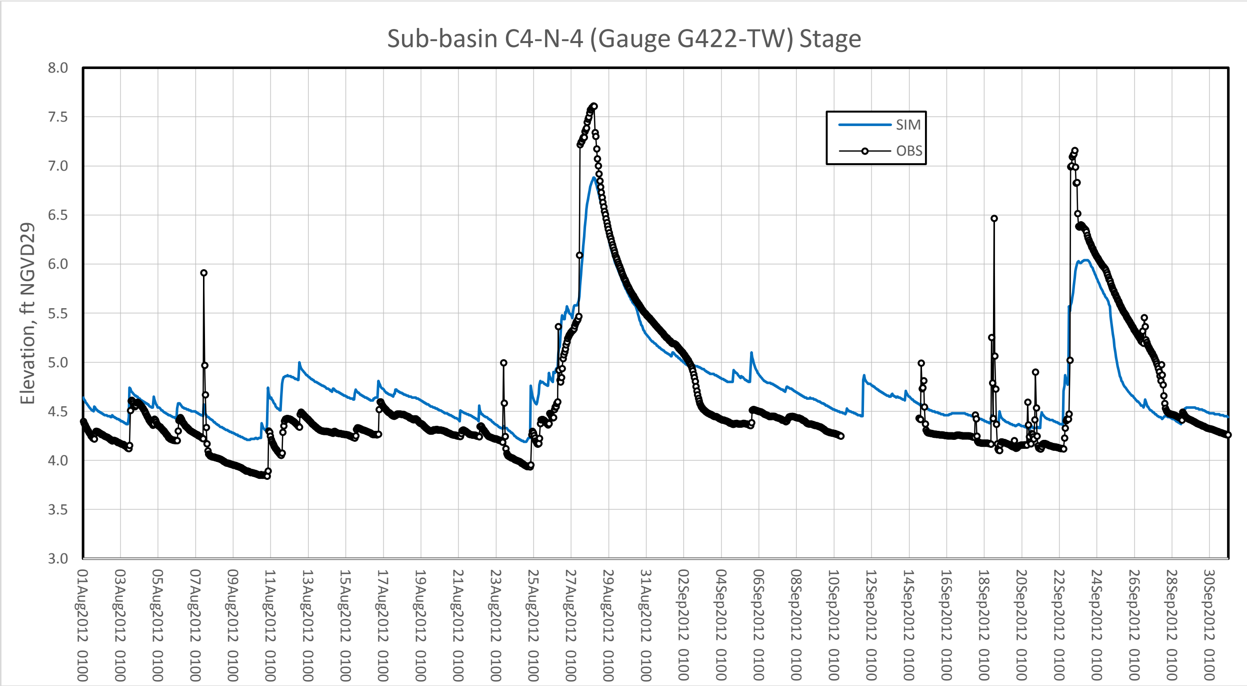

Figure F-29. Computed vs Simulated Stage in Sub-basin C4-N-4 (gauge G-422 TW)

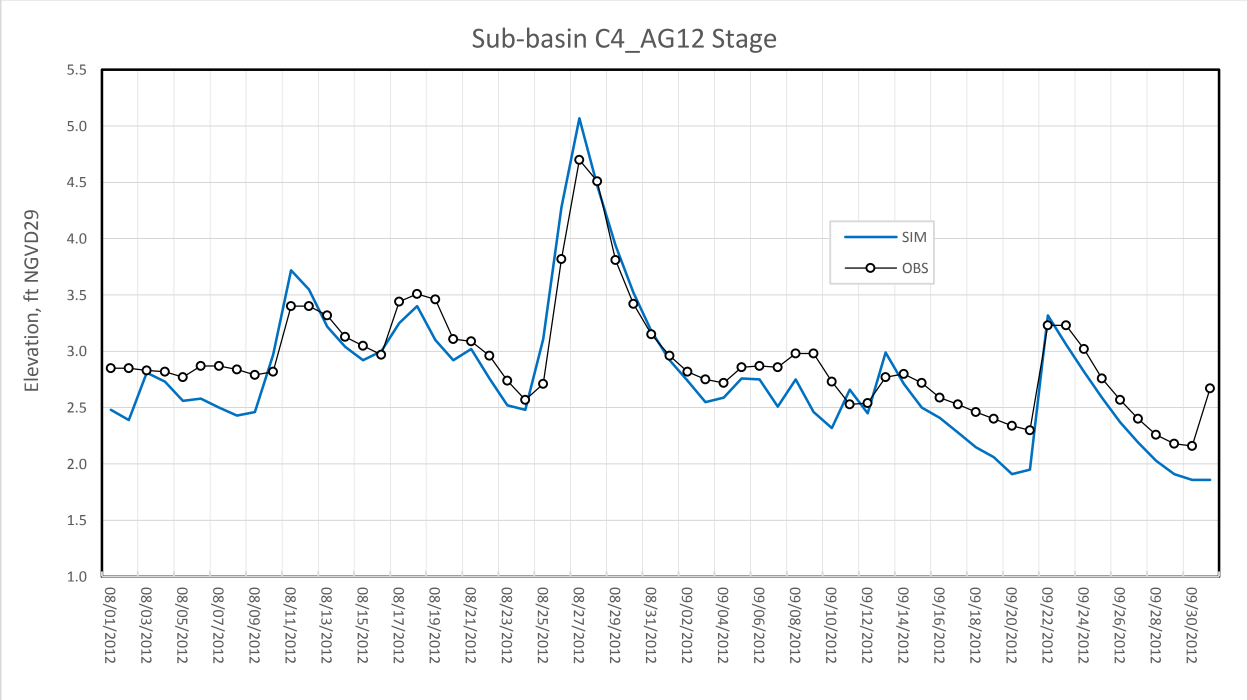

Figure F-30. Computed vs Simulated Stage in Sub-basin C4_AG12 (USGS Gauge G-3329)

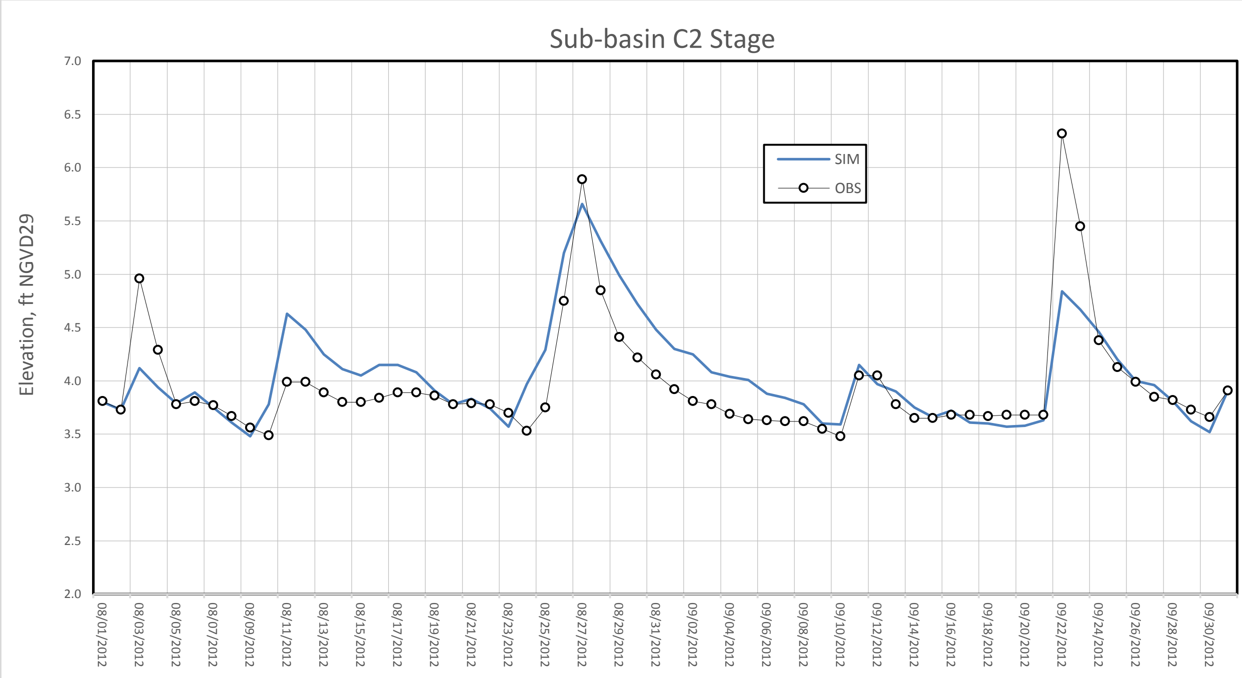

Figure F-31. Computed vs Simulated Stage in Sub-basin C2 (USGS Gauge G-3439)

The error statistics from the model calibration were for an extended period of time of August 1st to September 30, 2012, however, the statistics would have reflected a better fit if the simulation window was around the Tropical Storm Isaac (August 27 – September 6, 2012). During this event, the majority of the structures were fully open and the system was operated to discharge at its full capacity. The observed and predicted stages and flows at the peak of the storm event on August 27, 2012 are shown in **Figures F-32 through F-41**. The ability of the model to reproduce single-event peak stages and flows is important in this project because of the ultimate use of the model which will be applied to synthetic storm events of different return periods to evaluate the corresponding flood conditions in the C-4 Basin. The next section of the report (model validation) shows the performance of the calibrated HEC-RAS model for a single storm event (Hurricane Irene, October, 1999) which had a return frequency of about 25 years.

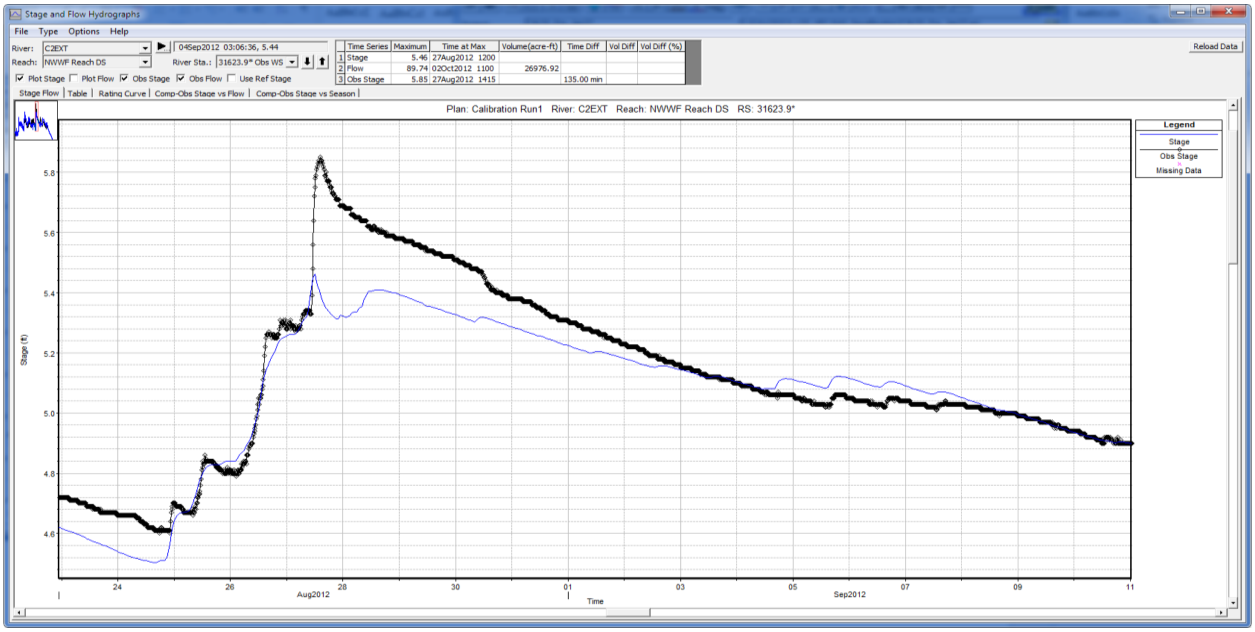

Figure F-32. Computed and Simulated Canal Stages at Gauge C2.72 during TS Isaac.

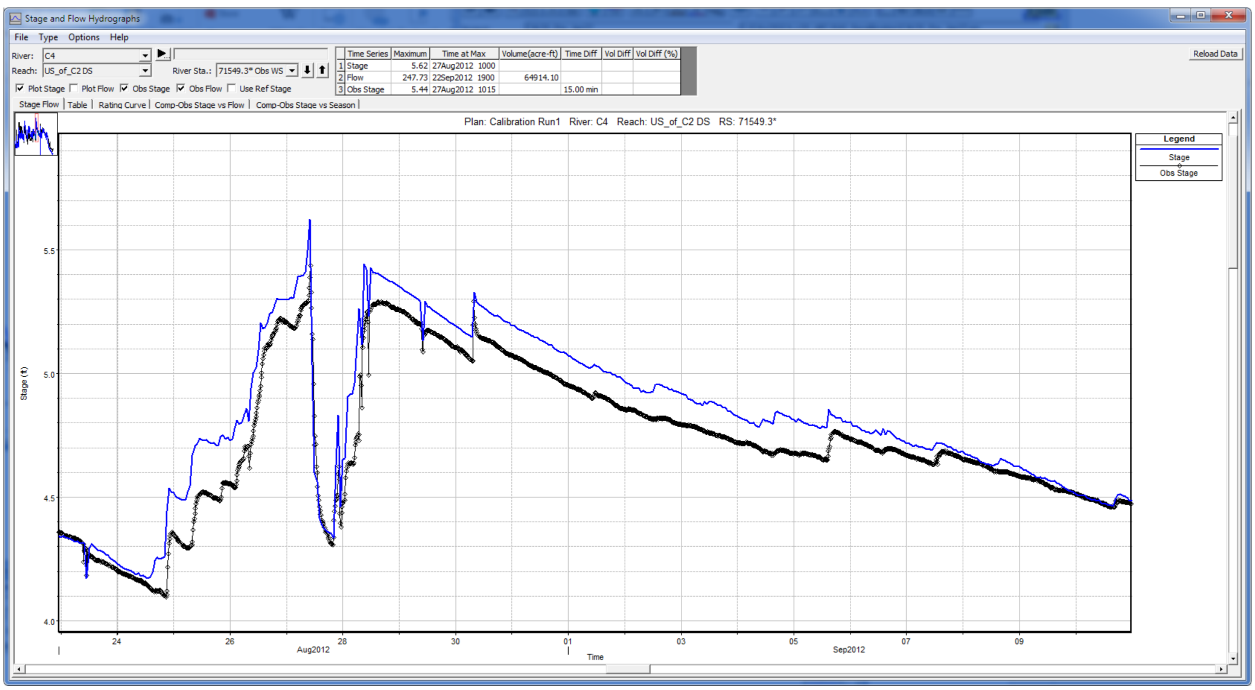

Figure F-33. Computed and Simulated Canal Stages at Gauge S-380 TW during TS Isaac.

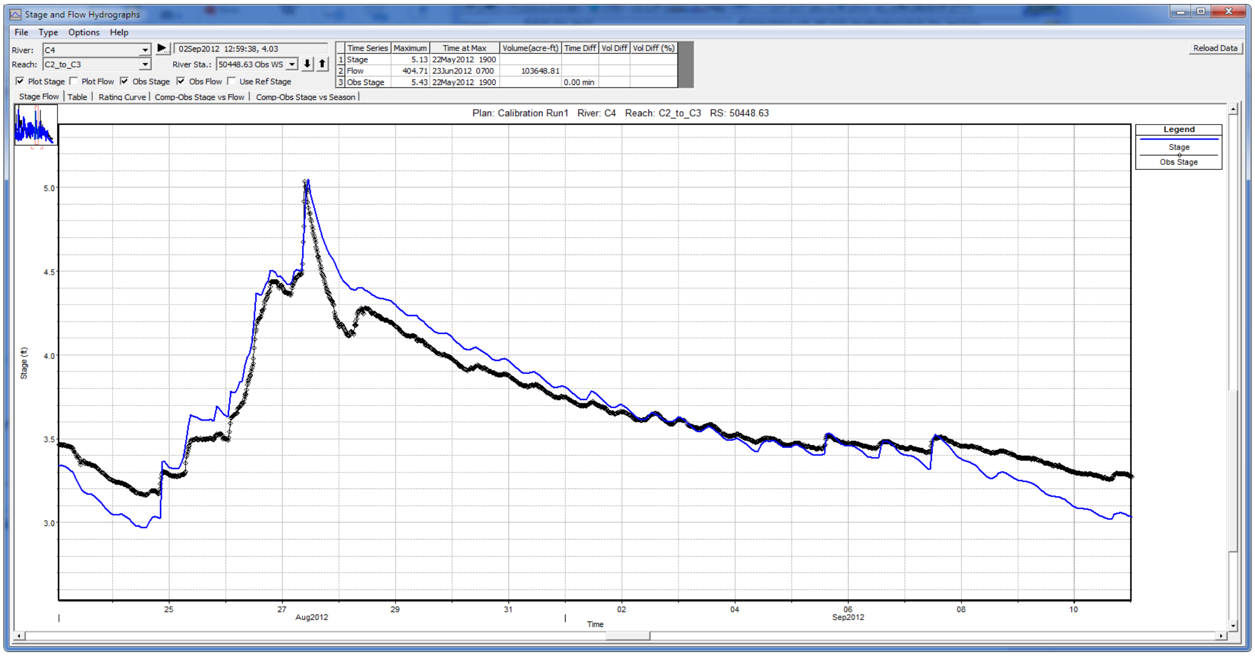

Figure F-34. Computed and Simulated Canal Stages at Gauge T5W during TS Isaac.

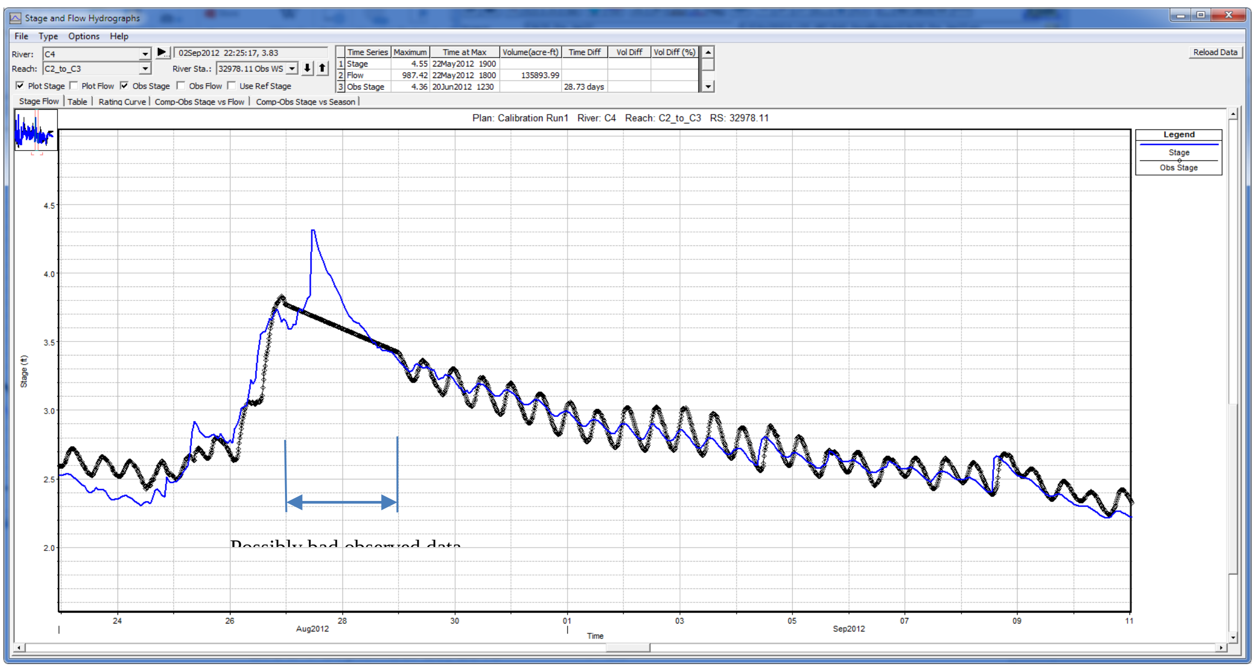

Figure F-35. Computed and Simulated Canal Stages at Gauge C4.Coral during TS Isaac.

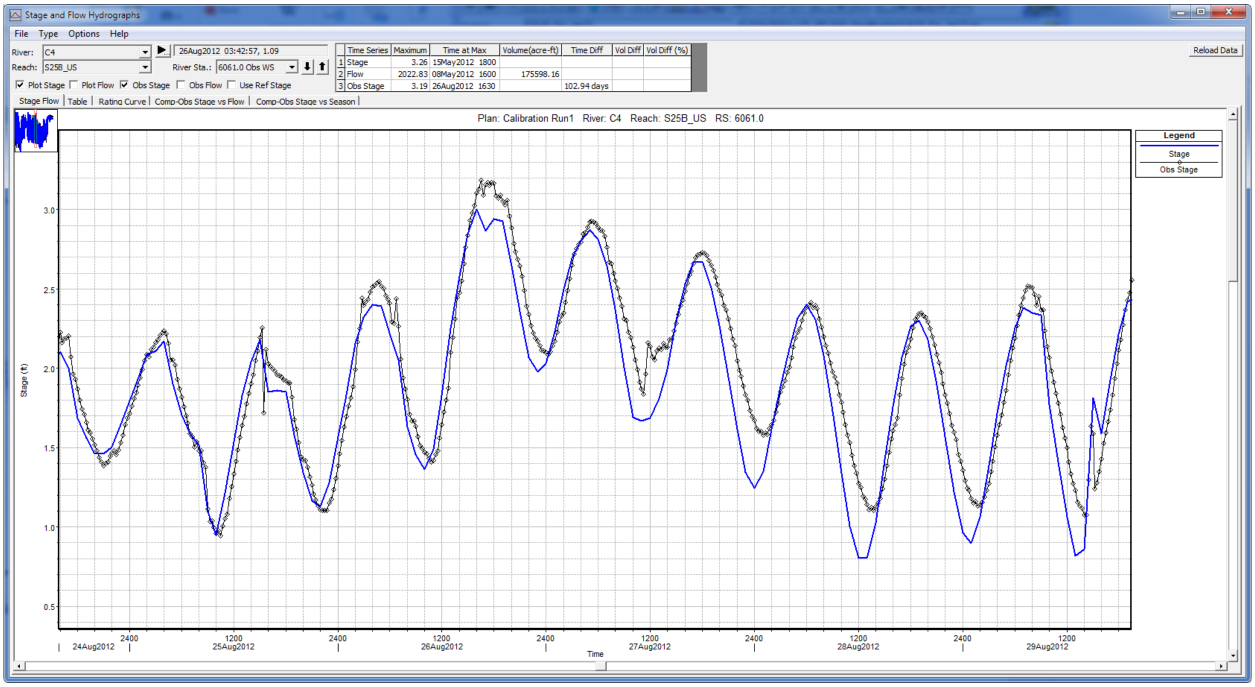

Figure F-36. Computed and Simulated Canal Stages at Gauge S-25B HW during TS Isaac.

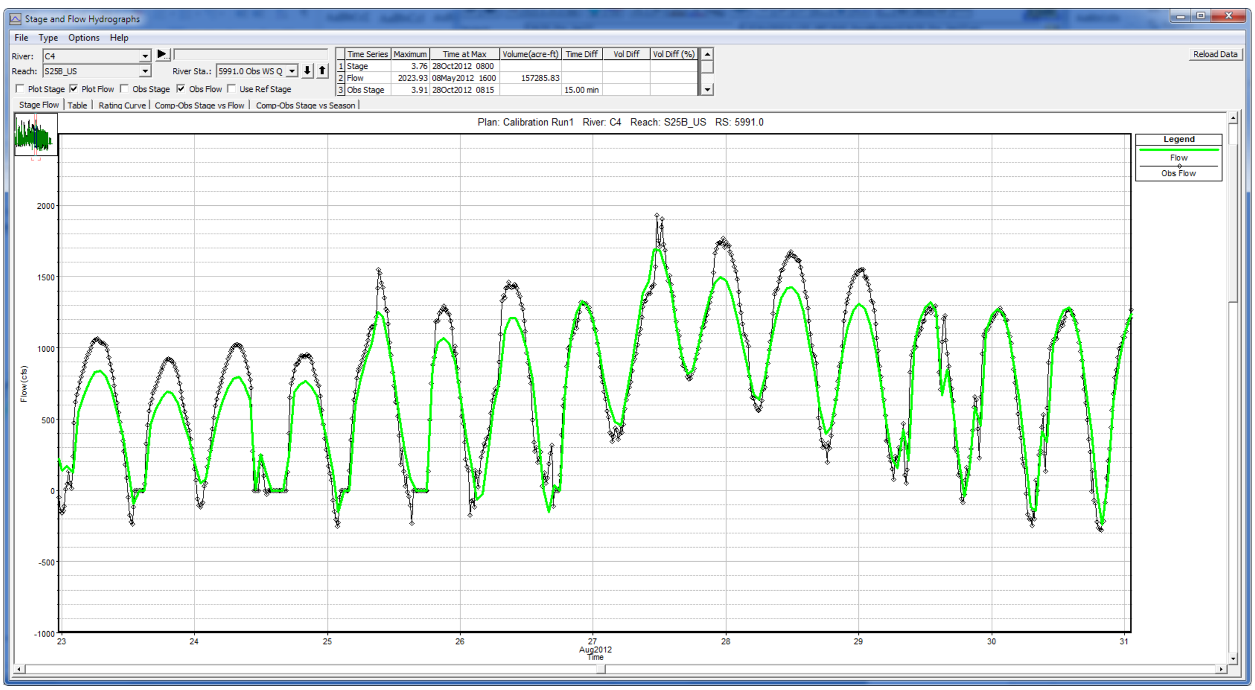

Figure F-37. Computed and Simulated Canal Flow at Gauge S-25B during TS Isaac.

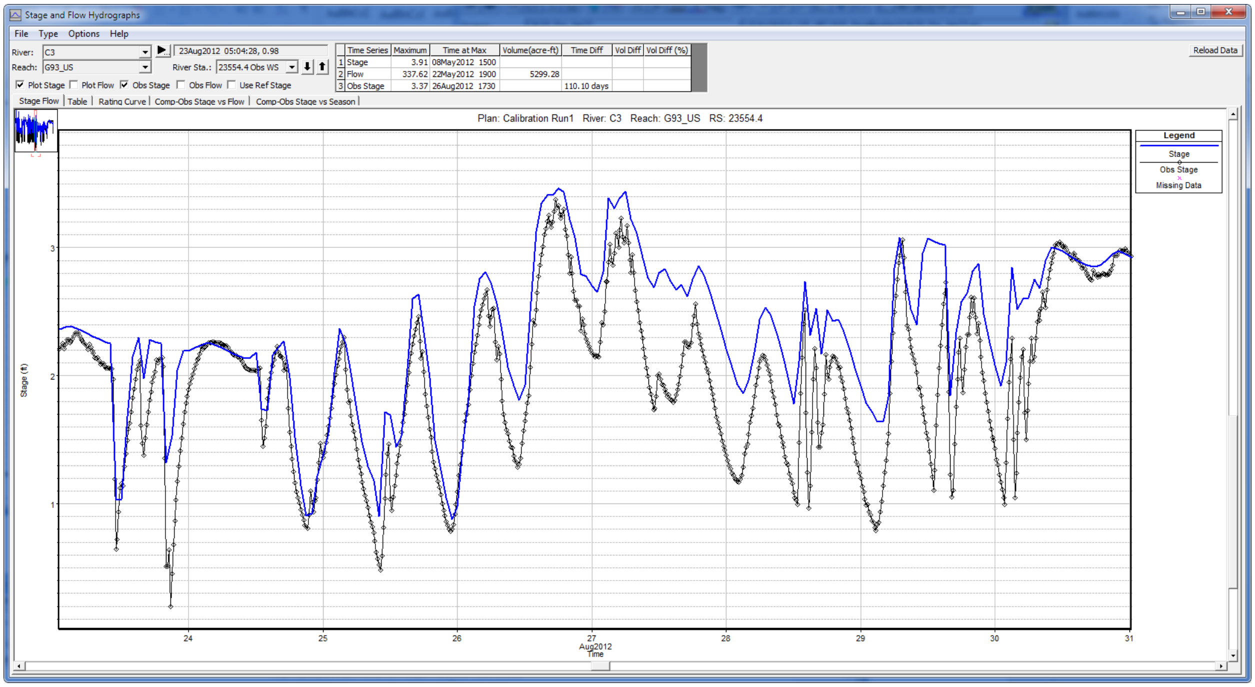

Figure F-38. Computed and Simulated Canal Stages at Gauge G-93 HW during TS Isaac.

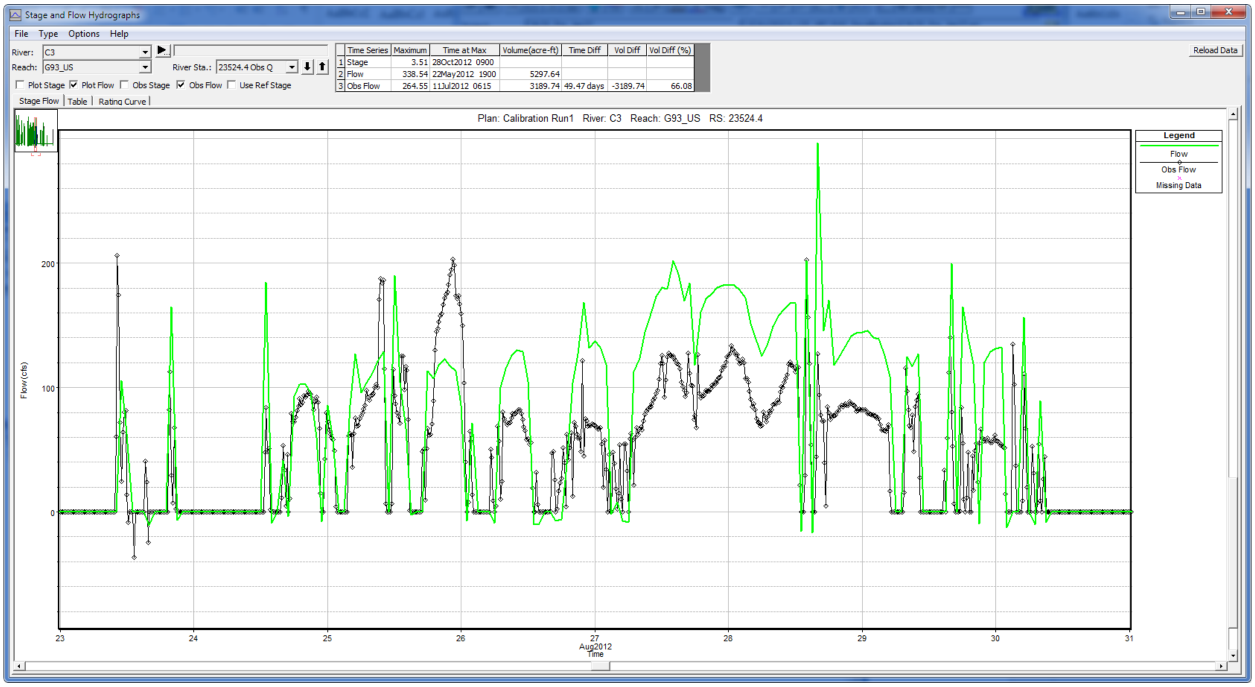

Figure F-39. Computed and Simulated Canal Flow at Gauge G-93 during TS Isaac.

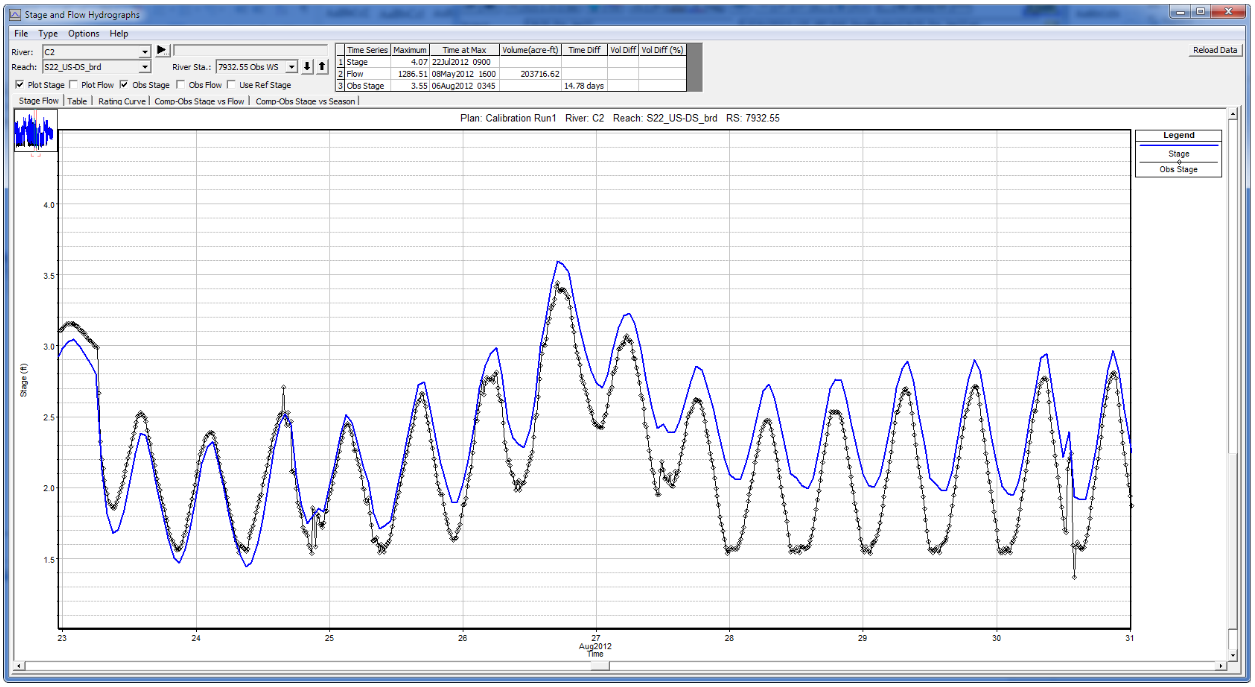

Figure F-40. Computed and Simulated Canal Stages at Gauge S-22HW during TS Isaac.

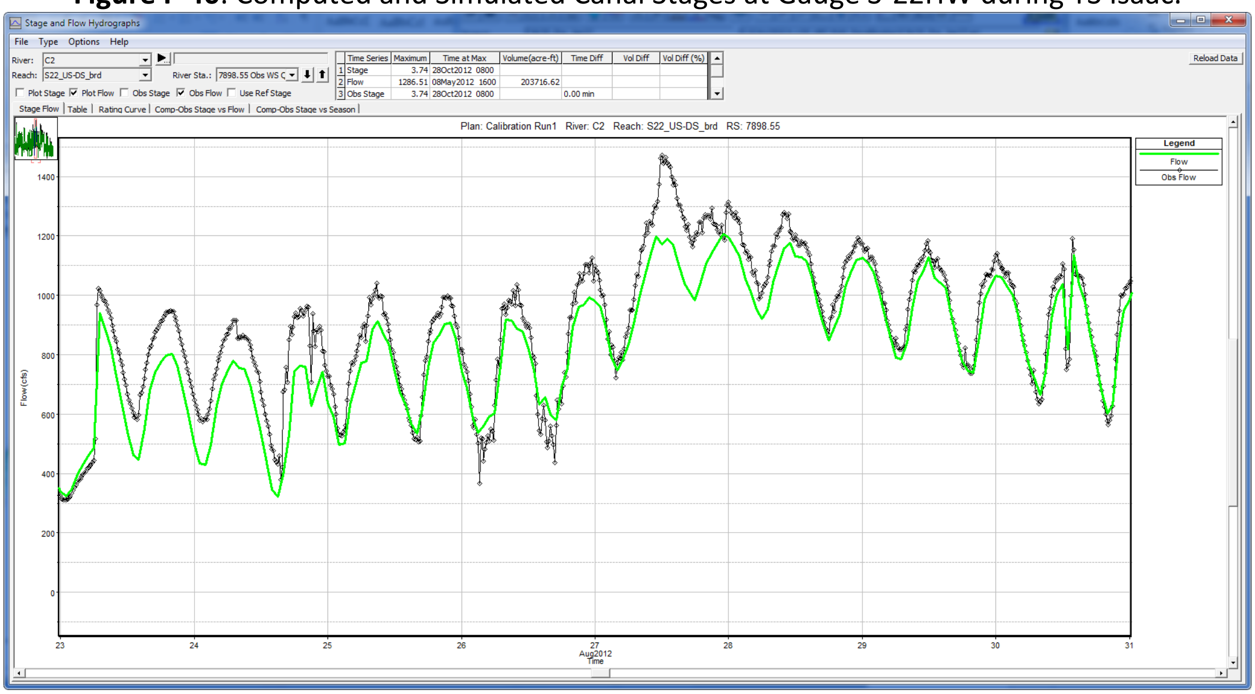

Figure F-41. Computed and Simulated Canal Flow at Gauge S-22 during TS Isaac.

MODEL VALIDATION

Validation of the calibrated HEC-RAS of the C-4 Basin model consisted in simulating the basin and canal flows and stages for one of the largest storm events in recent history, Hurricane Irene which occurred on October 14-15, 1999. Flooding damages from this event triggered a series of structural and operational drainage improvements in the C-4 basin which are currently used by the District and local municipalities to mitigate flooding in flood prone areas such as the cities of Belen, Sweetwater, West Miami and Miami. Because this particular storm occurred prior to the implementation of the current flood mitigation plan in the C-4 Basin, the model had to be modified to reflect the drainage infrastructure and operations at the time of the event. This section of the report summarizes the changes to the model and the resulting stages and flows compared to the observation data. Finally, an assessment is made of the accuracy of the model to predict stages and flows for this event.

Model Setup

The current conditions version of the HEC-RAS model was developed to reflect the current infrastructure and operations currently used to maintain water levels in the C-4 Basin beyond which could result in structural damage, particularly in the flood prone areas mentioned above. To achieve this, the District and local municipalities operate a number of water control structures in the canals and municipal pumps that enhance the local drainage during severe storm events. The structures and their operations were presented in **Table E-3 in Appendix E** of this report.

Figure F-42 shows the location of water control structures used during Hurricane Irene (October 1999). After removing the structures that did not exist in 1999, the resulting HEC-RAS model was a much simpler version with only structures G-119 and S-25B in the C-4 Canal, S-25A and S-25 in the C-5 Canal, G-93 in the C-3 Canal and S-22 in the C-2 Canal. Like the calibration model, the water control operations in the model were reflected by imposing observed 15-min gate openings at these structures. Rainfall data shown in **Figure F-43** consisted of 15-min values recorded at Structure S-336 in the west and near Miami International Airport in the east. NEXRAD rainfall data prior to year 2005 are unreliable particularly during events with high winds such as the case of Hurricane Irene. Single gauge data for the entire model domain is a limitation of this application and should be considered as part of the model uncertainty.

The selected validation period was October 14 – 31, 1999 with the storm event occurring on October 14-15. The extended period after the event allows for the slow recession of water levels and flows through the system. Seepage from the Everglades National Park and from WCA 3A to the west of the model domain, was computed using stage data in these two water bodies and the seepage coefficients obtain in the model calibration .

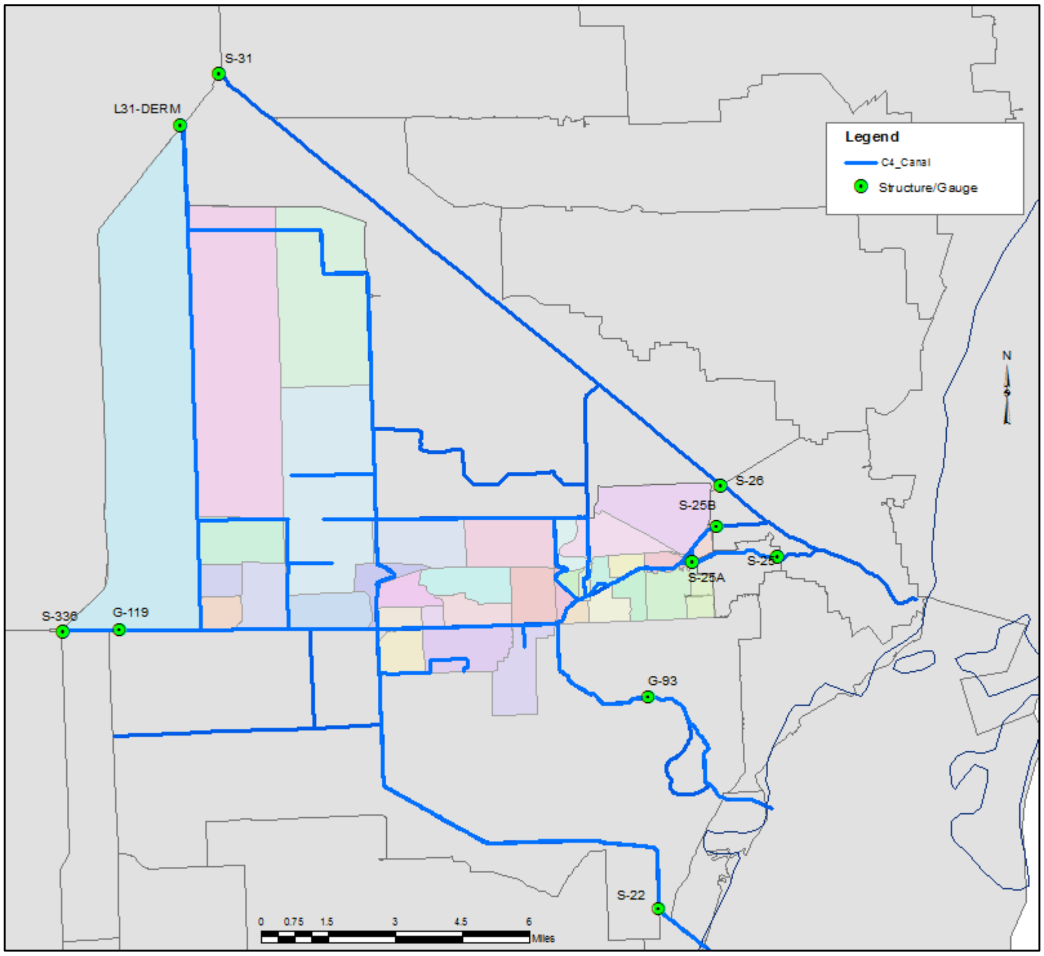

Figure F-42. Control Structures in C-2, C-3, C-4, C-5 and C-6 Basins (1999)

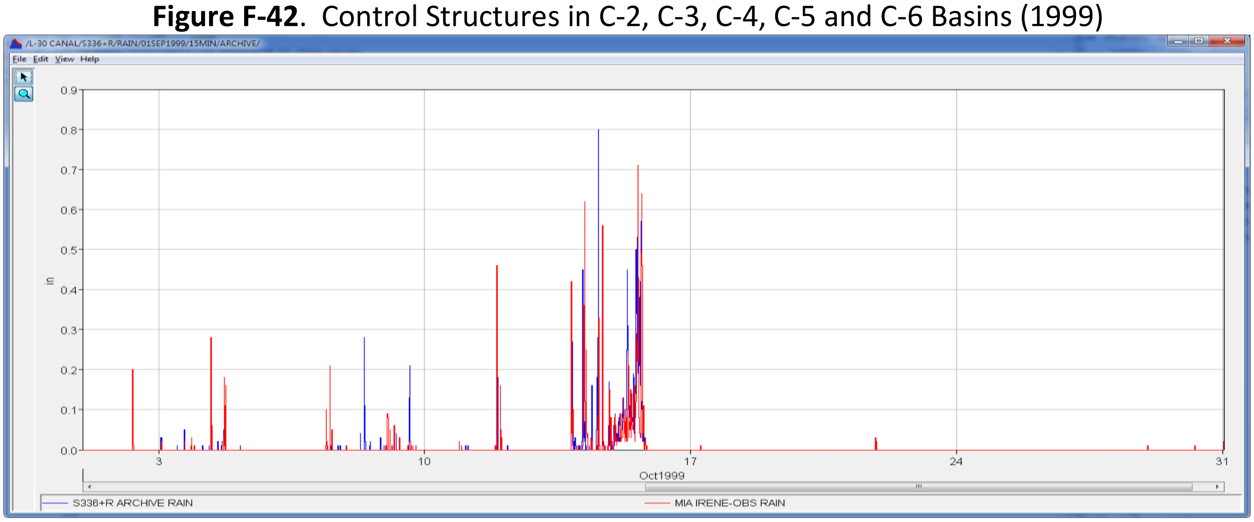

Figure F-43. 15-min Rainfall Distributions for October, 1999 at Gauges S-336 (blue line) and Miami-FS (red line)

Model Validation Performance

Model validation error bias and RMSE for canal stage and flows are summarized in **Tables F-10** and **F-11. Table F-10** indicates individual stage gauge biases under the 0.5 ft error tolerance for calibration at twelve of eighteen sites or 67 percent. The largest bias occurred at sub-basin C2 which represents the entire C-2 Basin and errors in the model can be attributable to structure deficiencies in representing the system in this basin. Consequently, stages at Structure S-22 were overestimated by 0.64 ft and 0.63 ft on the headwater and tailwater side of the structure and a had a flow bias of 395 cfs and RMSE of 411 cfs. Similarly, the C3 sub-basin representing the entire C-3 basin has the second largest stage bias of 0.68 ft and RMSE of 0.89 ft. In the C-4 Canal, the bias and RMSE of all the sites were under 0.5 ft except for T5W which had values of -0.54 ft and 0.62 ft. Overall the average stage bias and RMSE were 0.38 ft and 0.48 ft respectively.

Table F-10. Computed Canal Stage Bias and RMSE from HEC-RAS Validation

Calibration Site	Canal	Type	Stage Bias (ft)	Stage RMSE(ft)
S22_HW	C-2	Canal stage	0.64	0.68
S22_TW	C-2	Canal stage	0.63	0.68
G93_HW	C-3	Canal stage	0.30	0.32
G93_TW	C-3	Canal stage	0.34	0.36
S25B_HW	C-4	Canal stage	0.24	0.29
S25B_TW	C-4	Canal stage	0.13	0.15
T5W	C-4	Canal stage	0.54	0.62
C4.Coral	C-4	Canal stage	0.39	0.46
G119_TW	C-4	Canal stage	0.44	0.45
C2-10A	C2-10A	Sub-basin stage	-0.06	0.11
C4-10B	C4-10B	Sub-basin stage	0.35	0.36
C4-10C	C4-10C	Sub-basin stage	0.63	0.64
C4-10E	C4-10E	Sub-basin stage	0.10	0.18
C4-AG2	C4-AG2	Sub-basin stage	0.01	0.10
C4-AG12	C4-AG12	Sub-basin stage	0.28	0.42
C4-AG13	C4-AG13	Sub-basin stage	-0.20	0.43
C2	C2	Sub-basin stage	1.46	1.58
C3	C3	Sub-basin stage	0.68	0.89
		AVG:	**0.38**	**0.48**

Table F-11. Computed Canal Flow Bias and RMSE from HEC-RAS Validation

Calibration Site	Canal	Type	Flow Bias (cfs)	Flow RMSE (cfs)
S22_Q	C-2	Structure Flow	395	411
G93_Q	C-3	Structure Flow	-39	48
S25B_Q	C-4	Structure Flow	-457	533
C4.Coral	C-4	Canal Flow	-259	316
		AVG:	**288**	**327**

Figures F-44 through F-65 show the observed and computed water levels and flows at different locations in the C-2, C-3 and C-4 Basins during the model validation period. **Figures F-44 through F-48** show the water elevations at gauges in the C-4 Canal from west to east. In **Figure F-44** at gauge G119_TW indicates computed water levels with a positive bias of 0.44 ft and RMSE of 0.45 ft. Most of the error occurs after the peak of the storm event and could be attributed to initial and boundary conditions stage data and seepage estimates from the Water Conservation Area 3A to the C-4 and C-2 Basins. In general, the stage bias and RMSE gradually decreases at gauges from west to east in the C-4 Canal. In the C-3 Canal at structure G-93 the computed and observed stages on the headwater and tailwater side of the structure are shown in **Figures F-50 and F-51** whereas the flows are shown in **Figure F-52**. This structure had positive stage biases for the headwater (G93_HW) and the tailwater (G93_TW) side of the structure of 0.30 ft and 0.34 ft and stage RMSE of 0.32 ft and 0.36 ft respectively, however, at the peak stage prediction by the model was only higher than the observed value by only 0.15 ft. In the C-2 Canal, stage predictions with the model were made only at Structure S-22 at which the model predicted stages on both sides of the structure with biases of 0.64 ft and 0.63 ft, and RMSE's of 0.68 ft however, the peak stage error was only 0.01 ft. Flows at Structure S-22 were over predicted with a bias and RMSE of 395 and 411 cfs respectively and a peak flow error of 274 cfs. **Figures F-57 through F-65** show stage hydrographs in several sub-basins where groundwater level data was available during the storm event. These stages were not used as part of the model calibration but are enclosed to show the model performance for computing stages in the sub-basins.

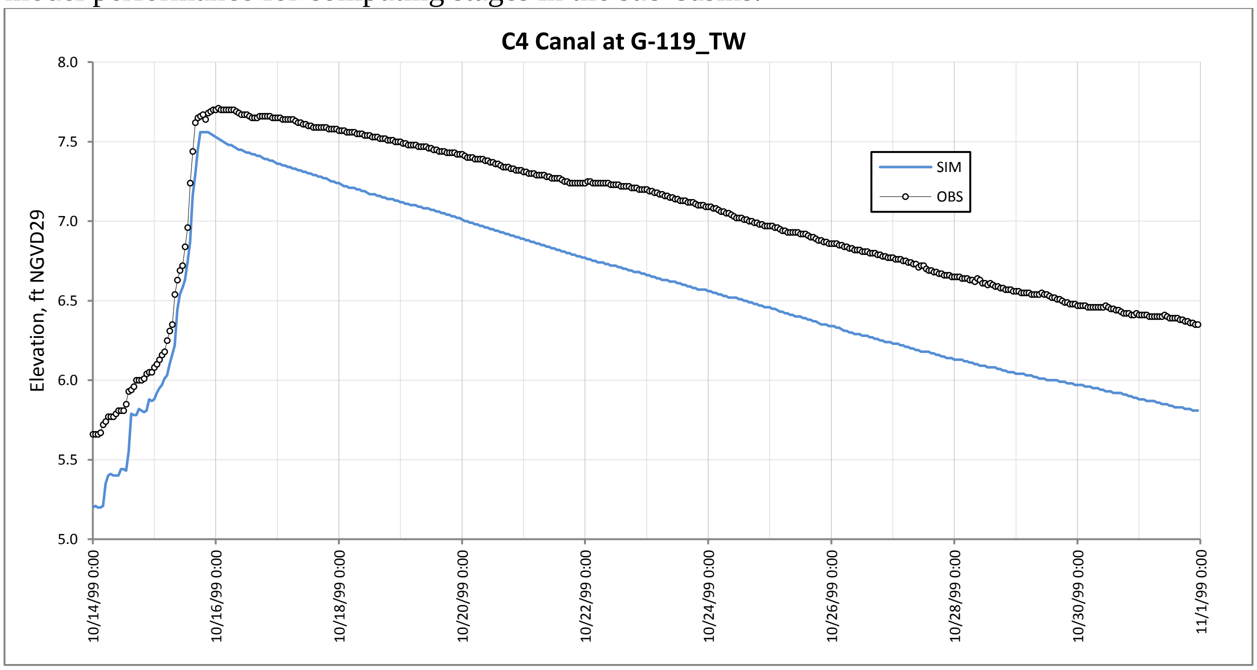

Figure F-44. Observed and Simulated C-4 Canal stages at gauge G119_TW for Model Validation (Hurricane Irene, October 14-15, 1999)

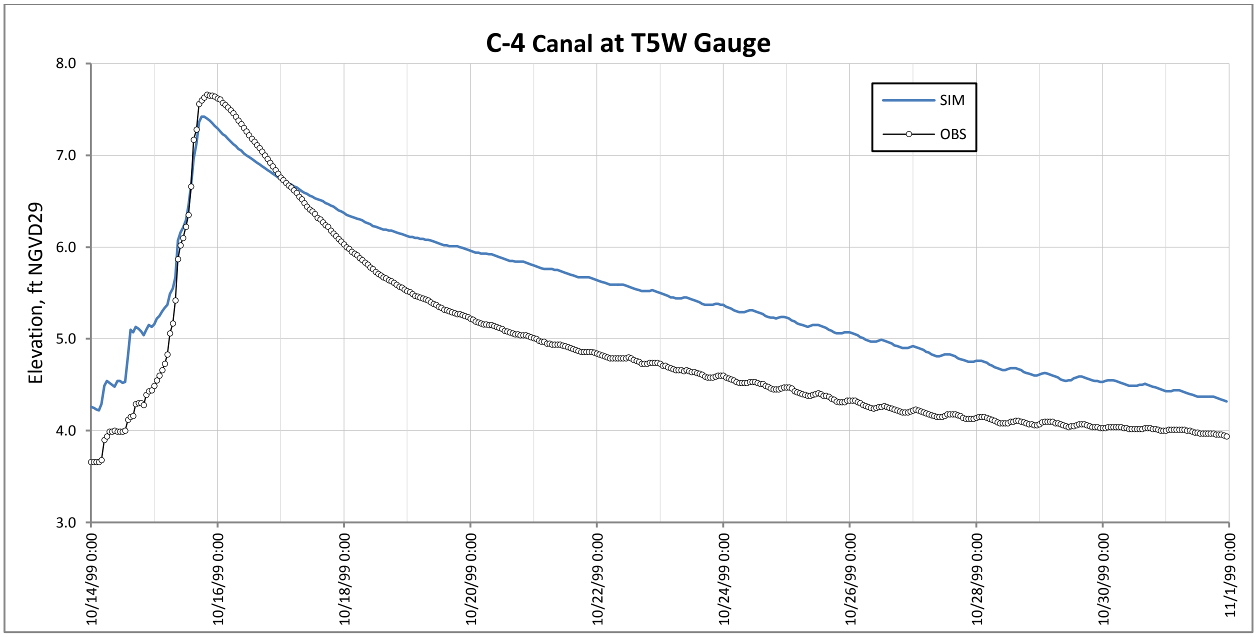

Figure F-45. Observed and Simulated C-4 Canal stages at gauge T5W for Model Validation (Hurricane Irene, October 14-15, 1999)

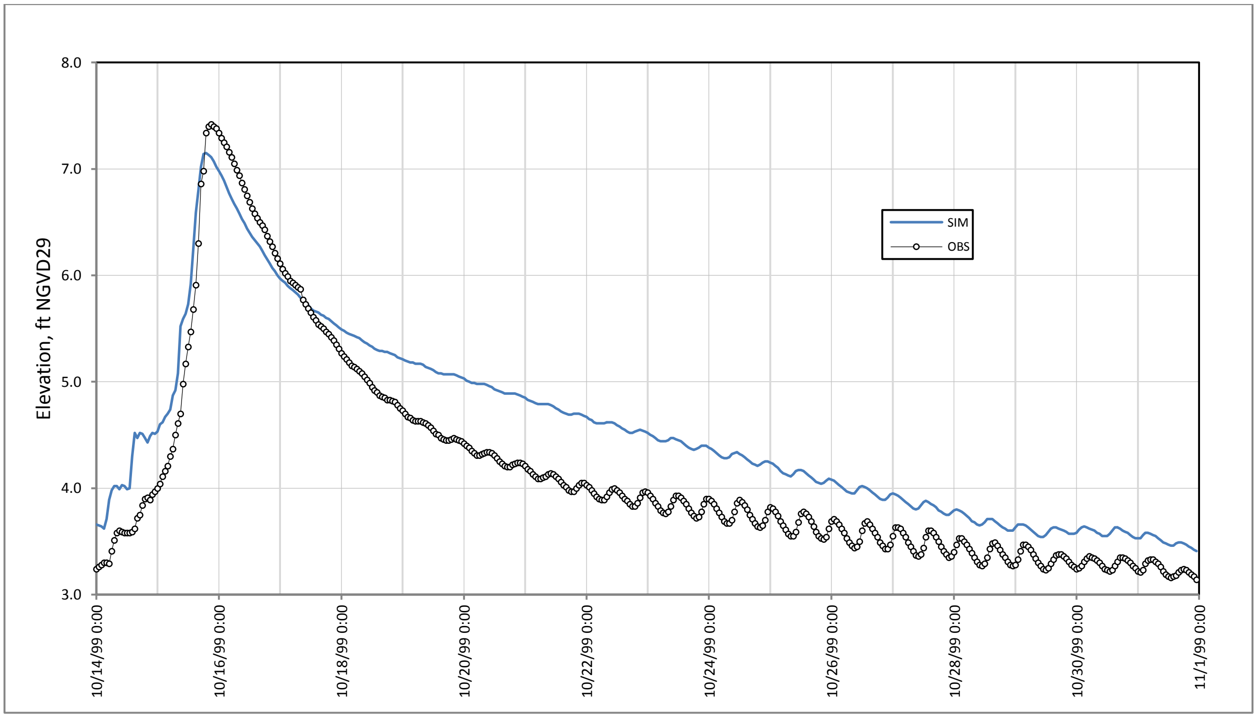

Figure F-46. Observed and Simulated C-4 Canal stages at gauge C4.Coral for Model Validation (Hurricane Irene, October 14-15, 1999)

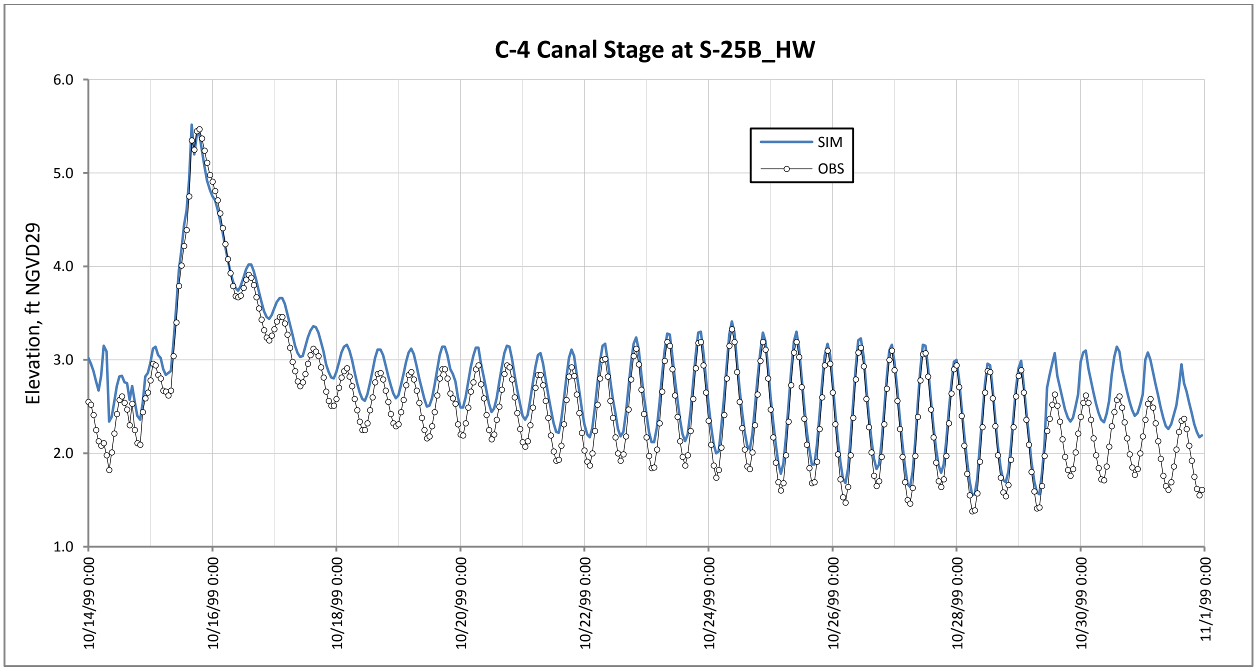

Figure F-47. Observed and Simulated C-4 Canal stages at gauge S25B_HW for Model Validation (Hurricane Irene, October 14-15, 1999)

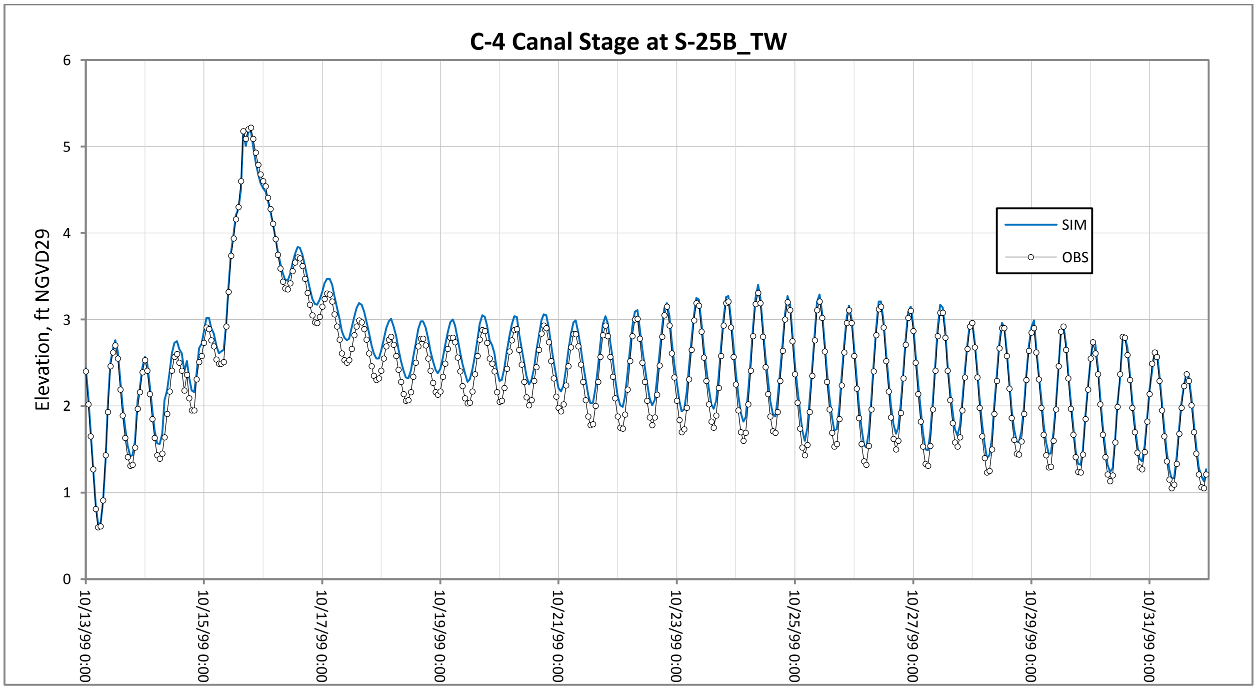

Figure F-48. Observed and Simulated C-4 Canal stages at gauge S25B_TW for Model Validation (Hurricane Irene, October 14-15, 1999)

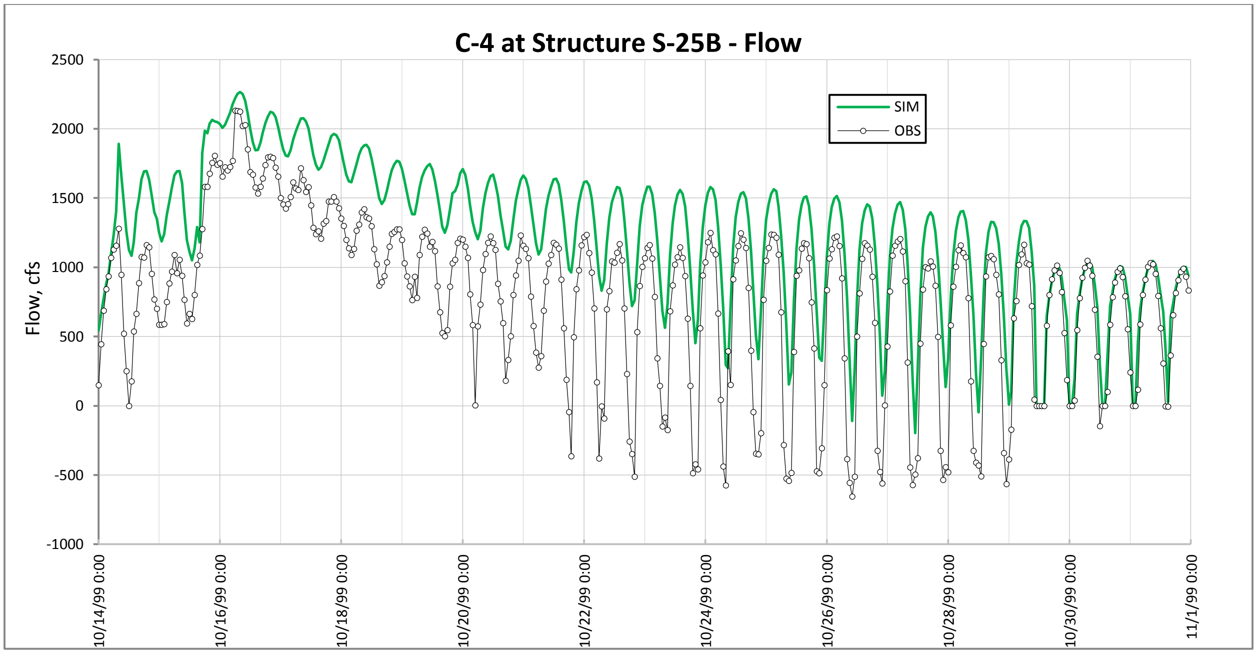

Figure F-49. Observed and Simulated C-4 Canal Flow at gauge S-25B for Model Validation (Hurricane Irene, October 14-15, 1999)

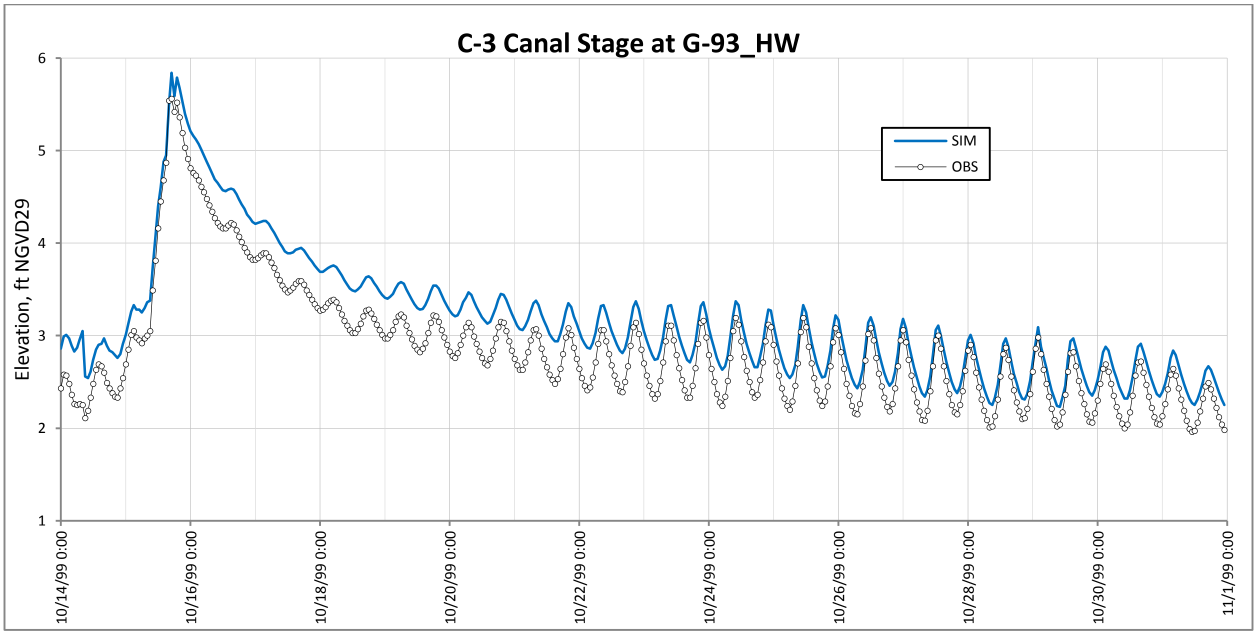

Figure F-50. Observed and Simulated C-3 Canal Stage at gauge G93B_HW for Model Validation (Hurricane Irene, October 14-15, 1999)

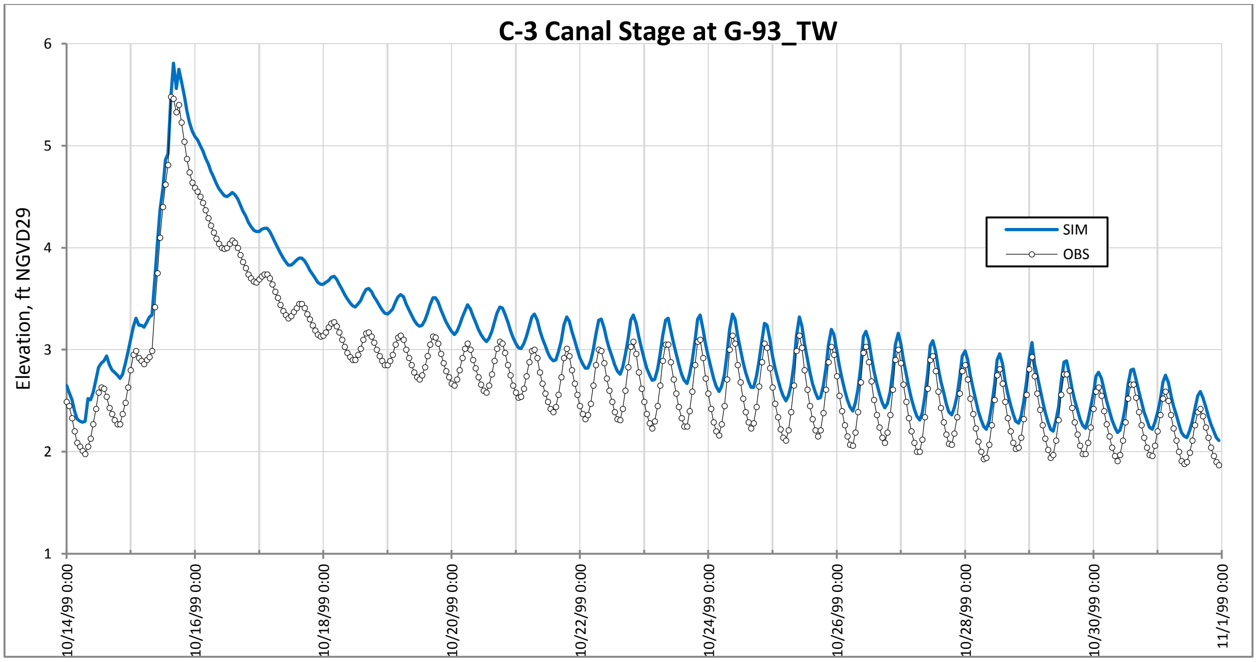

Figure F-51. Observed and Simulated C-3 Canal Stage at gauge G93B_TW for Model Validation (Hurricane Irene, October 14-15, 1999)

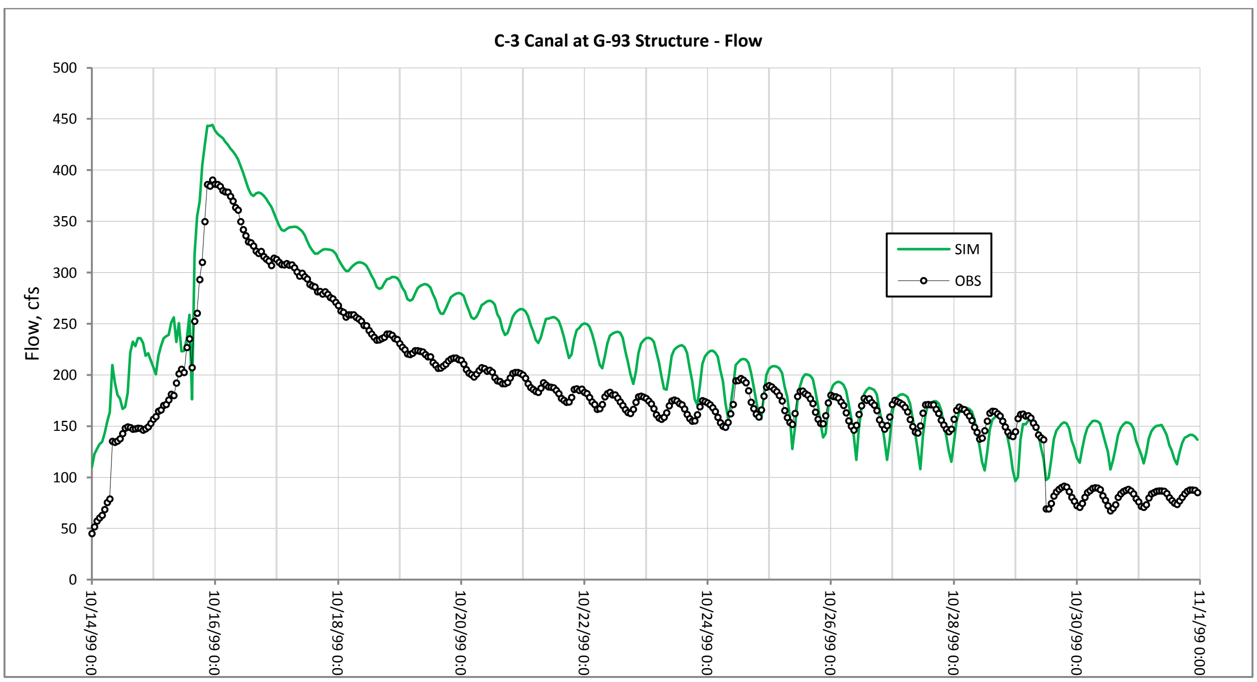

Figure F-52. Observed and Simulated C-3 Canal flow at gauge G93B_TW for Model Validation (Hurricane Irene, October 14-15, 1999)

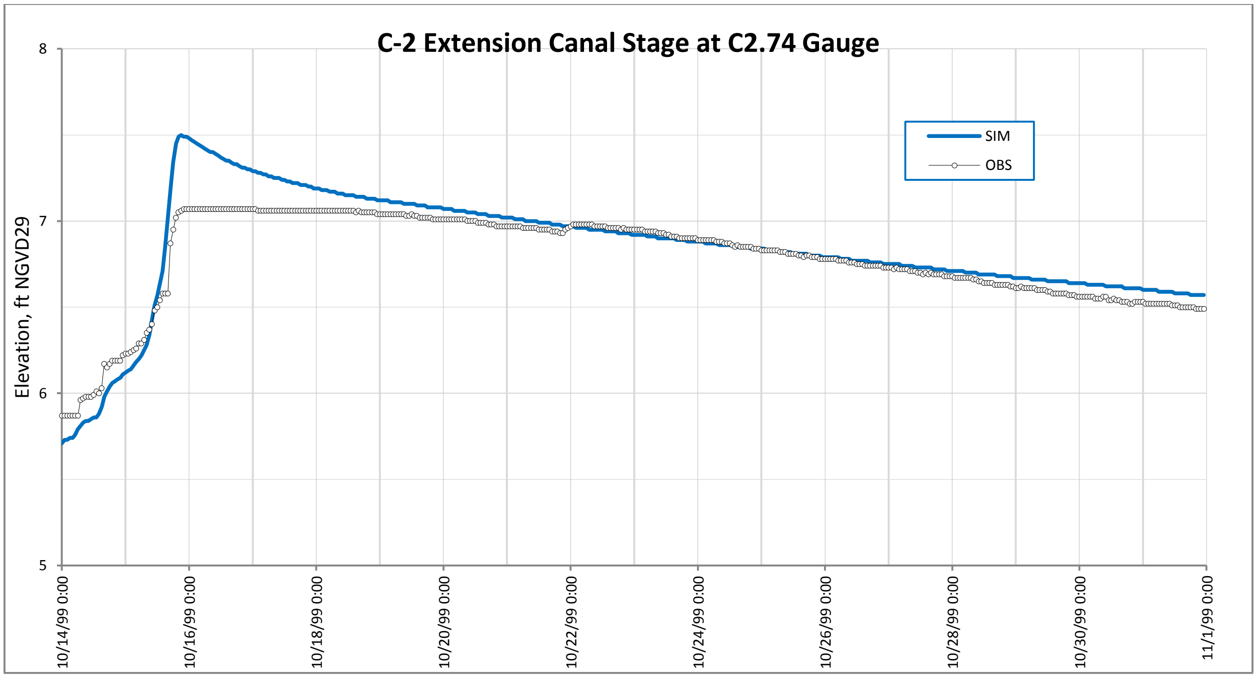

Figure F-53. Observed and Simulated C-2 Extension Canal Stage at gauge C2.74 for Model Validation (Hurricane Irene, October 14-15, 1999)

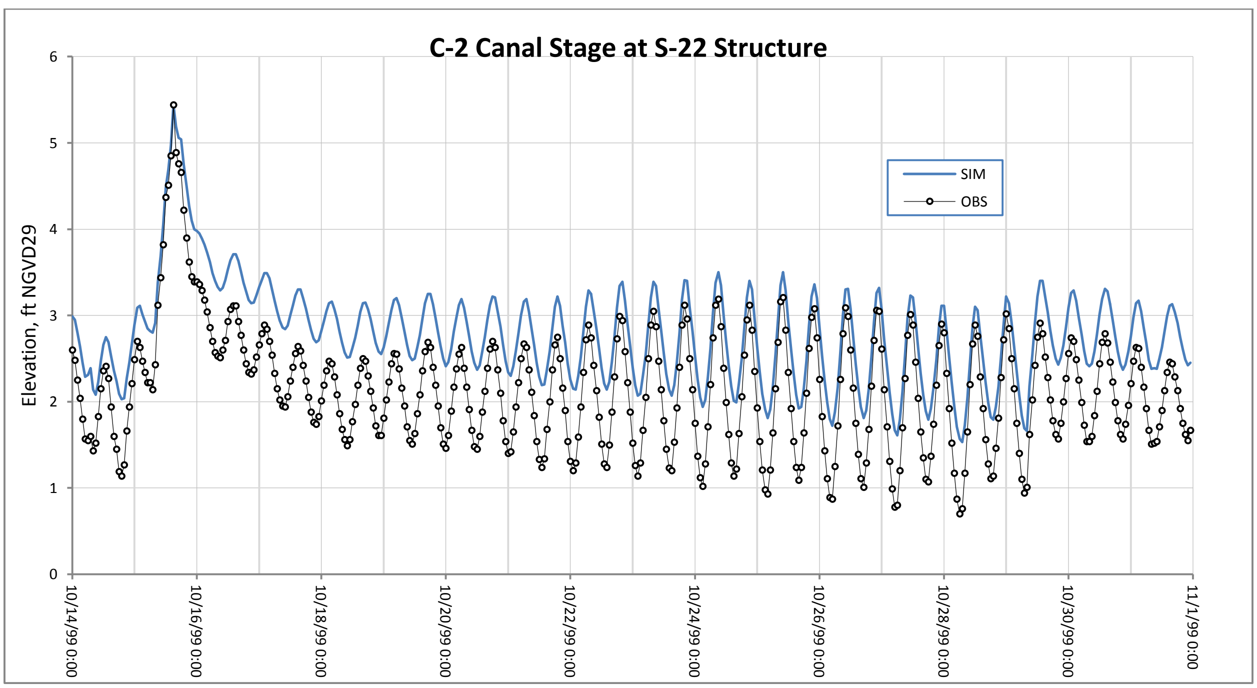

Figure F-54. Observed and Simulated C-2 Canal Stage at Gauge S22_HW for Model Validation (Hurricane Irene, October 14-15, 1999)

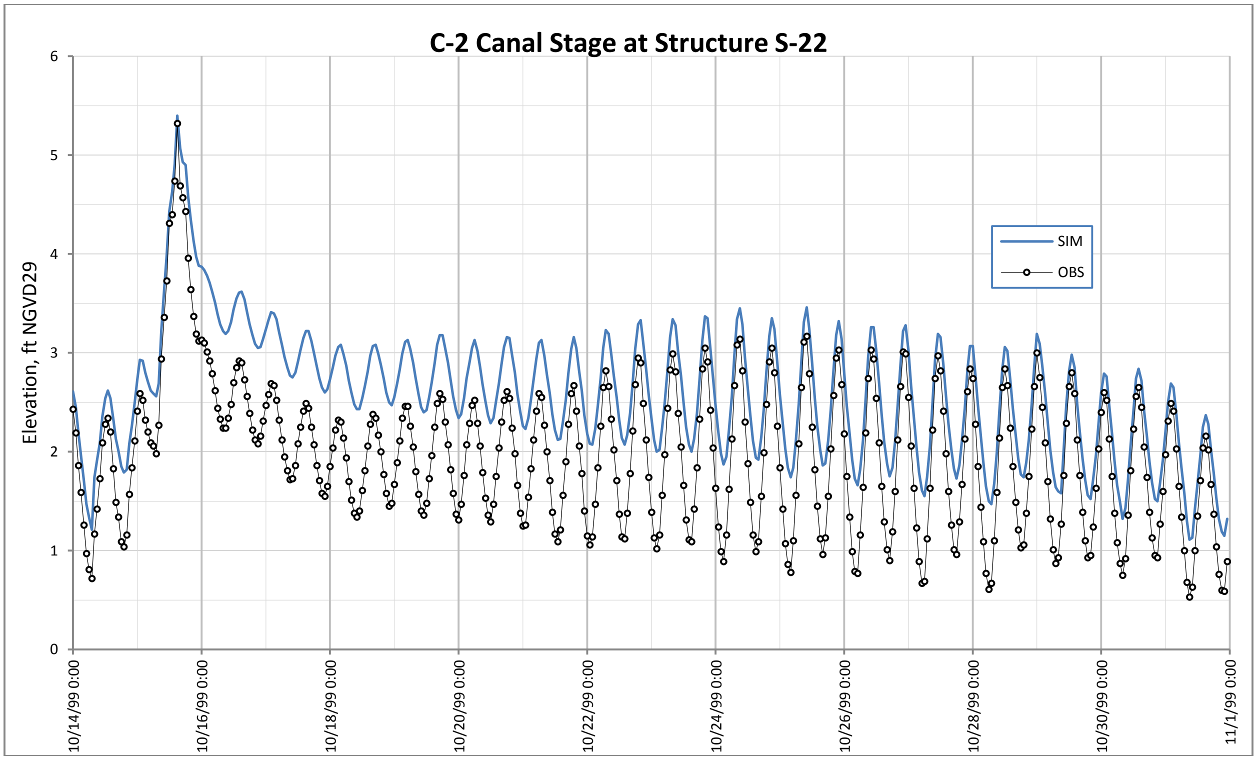

Figure F-55. Observed and Simulated C-2 Canal Stage at Gauge S22_TW for Model Validation (Hurricane Irene, October 14-15, 1999)

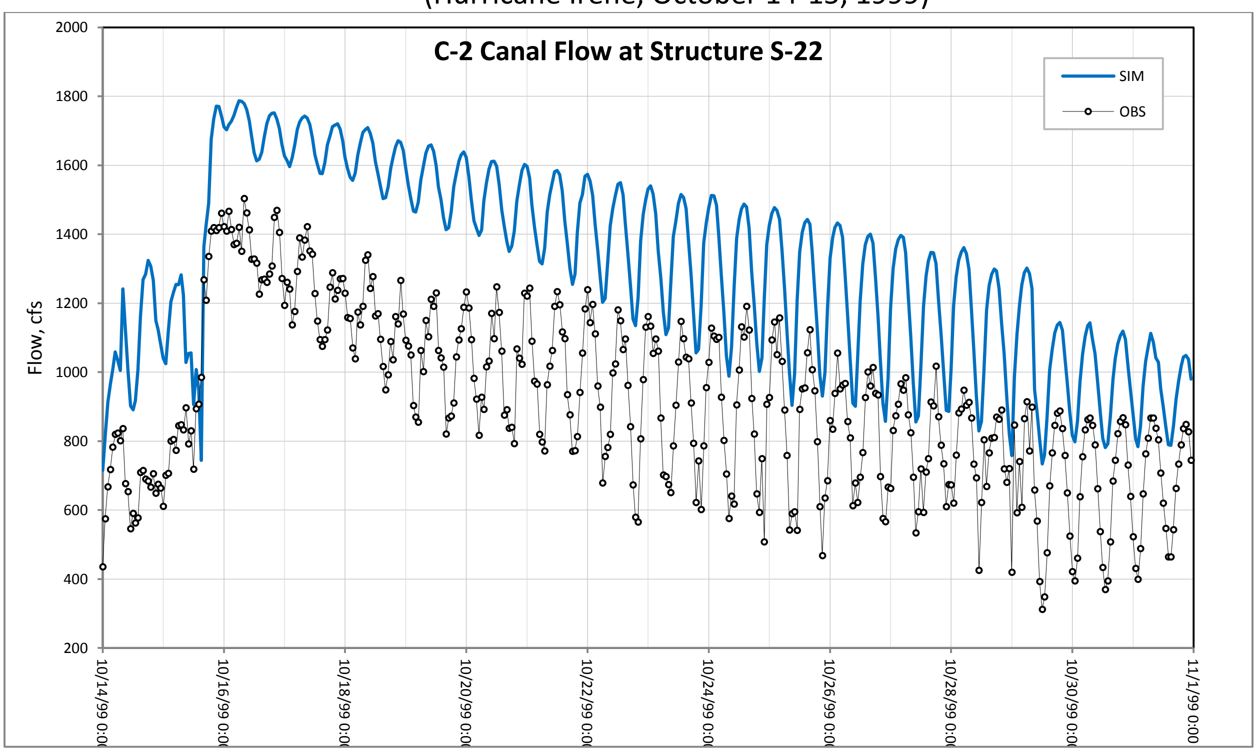

Figure F-56. Observed and Simulated C-2 Canal Flow at Structure S-22 for Model Validation (Hurricane Irene, October 14-15, 1999)

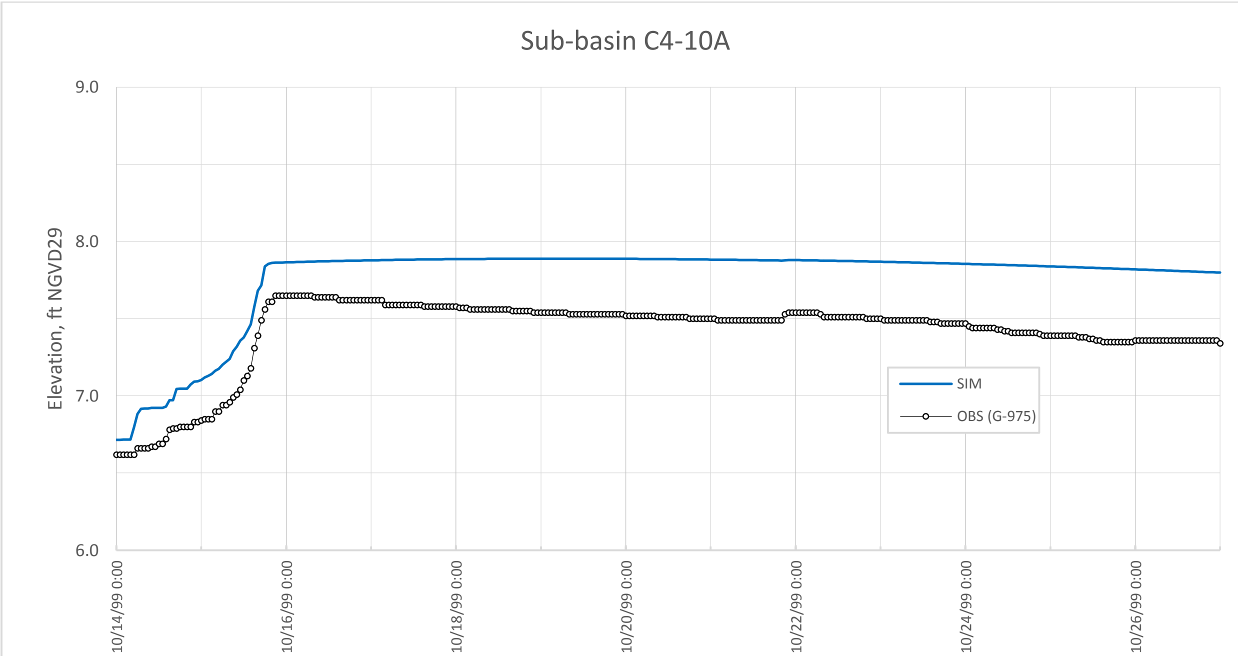

Figure F-57. Observed and Simulated Stage in Sub-basin C4-10A and Groundwater Well G-975 for Model Validation (Hurricane Irene, October 14-15, 1999)

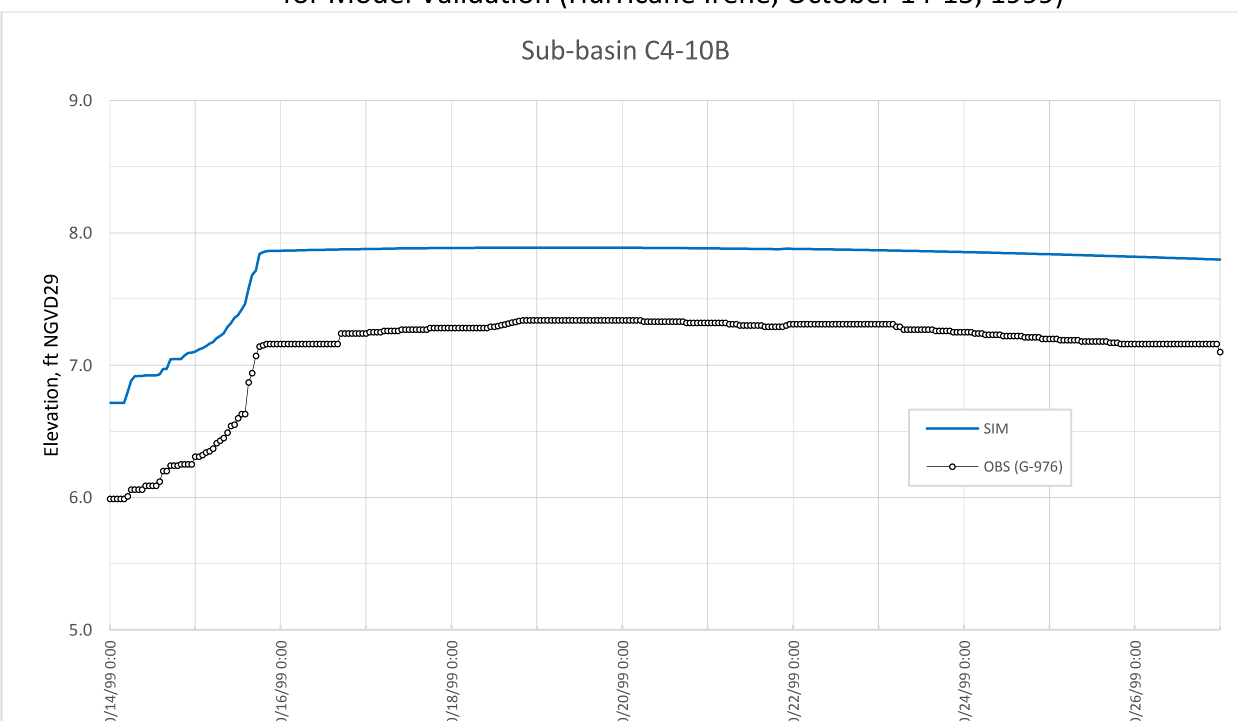

Figure F-58. Observed and Simulated Stage in Sub-basin C4-10B and Groundwater Well G-976 for Model Validation (Hurricane Irene, October 14-15, 1999)

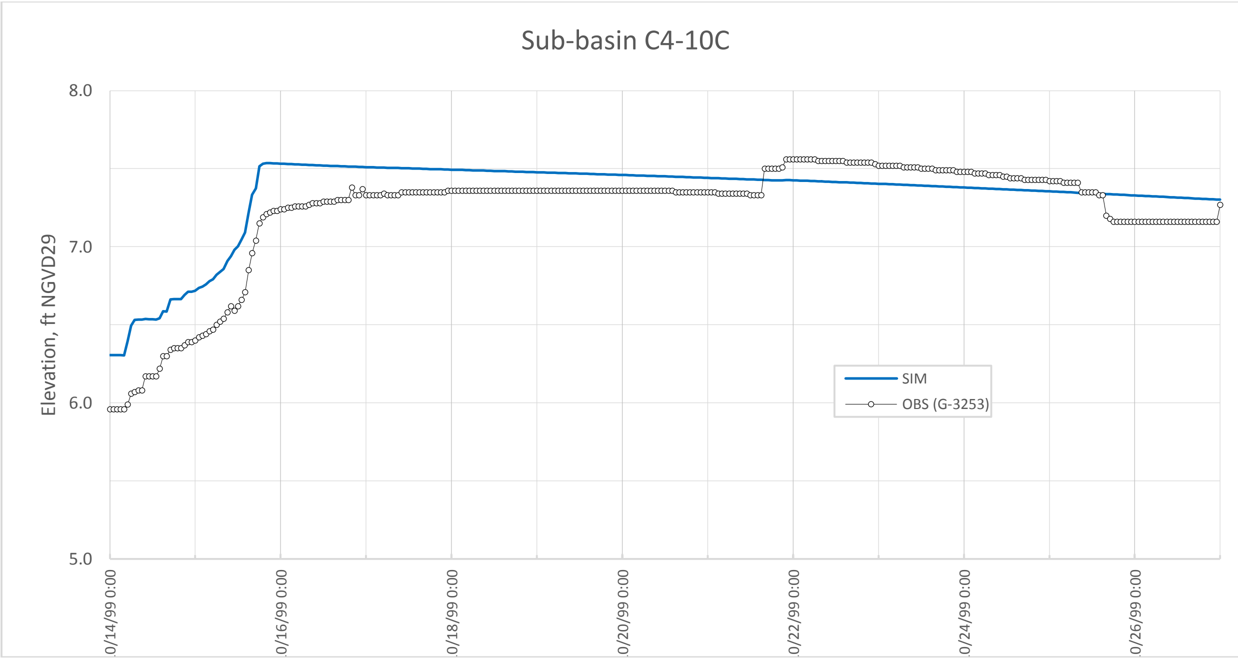

Figure F-59. Observed and Simulated Stage in Sub-basin C4-10C and Groundwater Well G-3253 for Model Validation (Hurricane Irene, October 14-15, 1999)

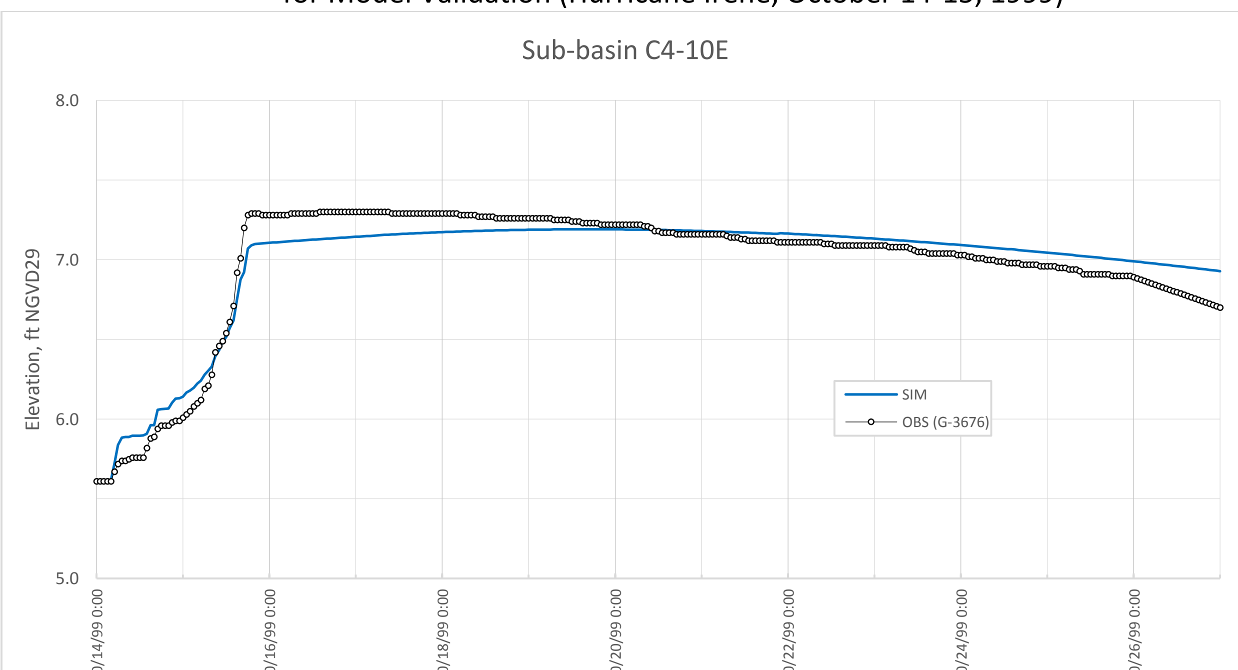

Figure F-60. Observed and Simulated Stage in Sub-basin C4-10E and Groundwater Well G-3676 for Model Validation (Hurricane Irene, October 14-15, 1999)

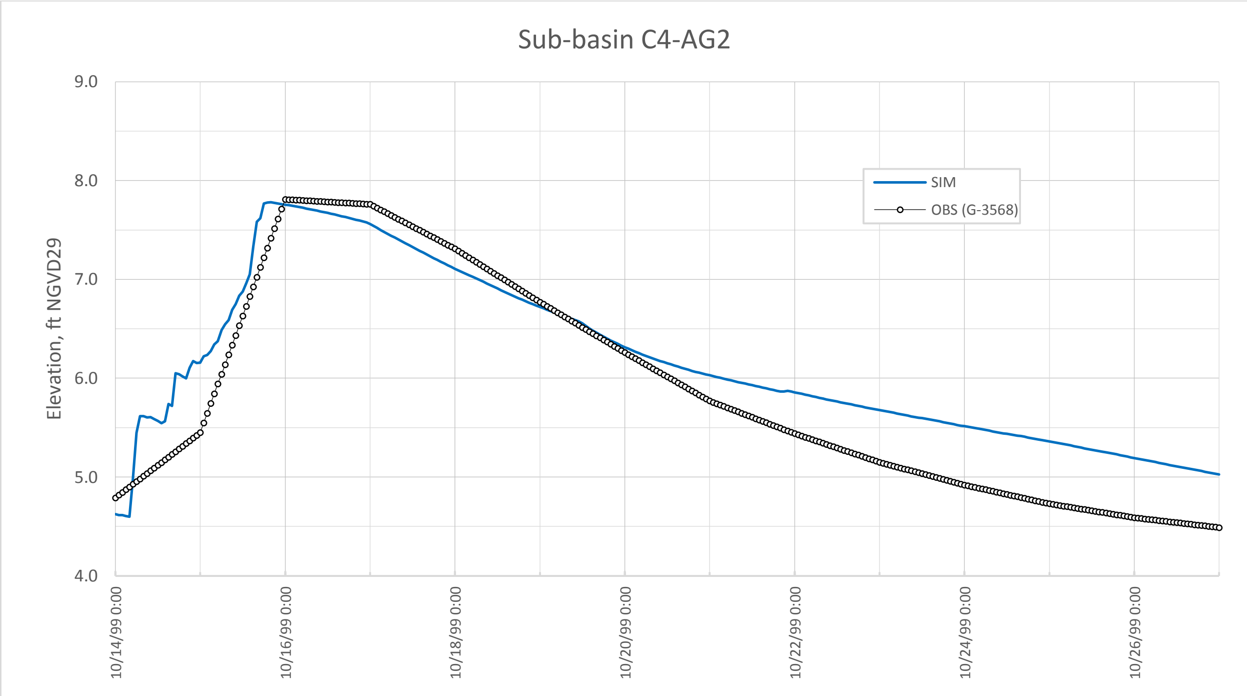

Figure F-61. Observed and Simulated Stages in Sub-basin C4-AG2 (City of Sweetwater) and Groundwater Well G-3568 for Model Validation (Hurricane Irene, October 14-15, 1999)

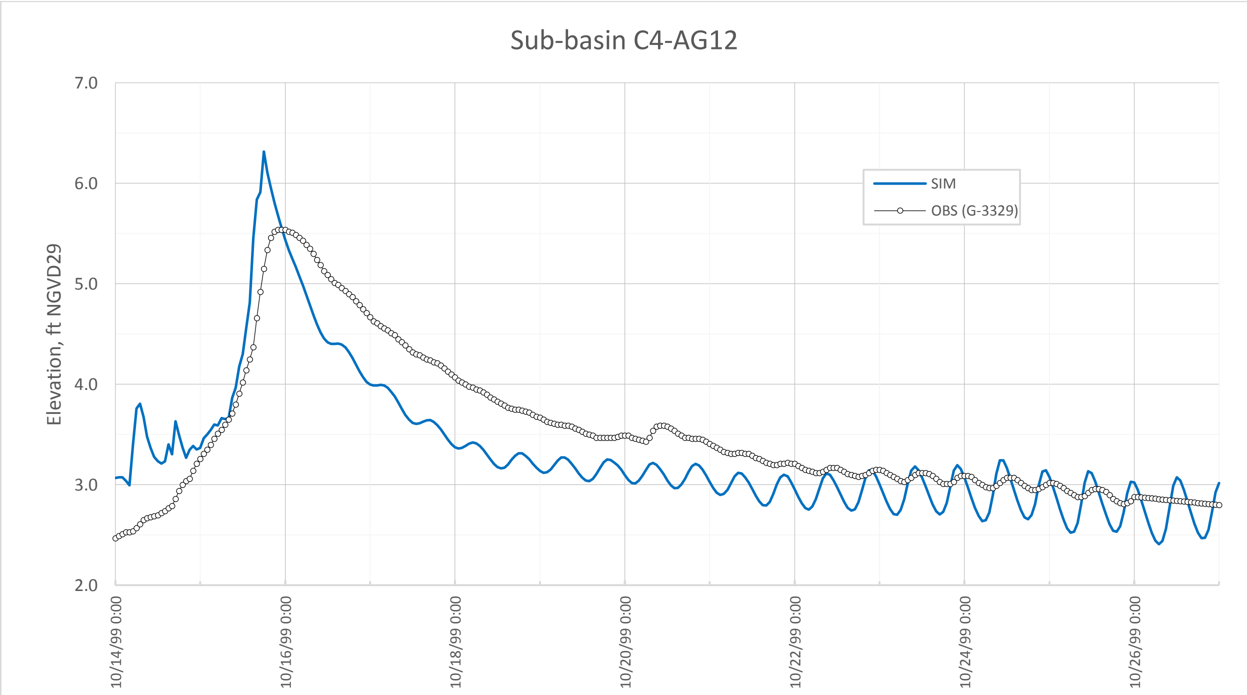

Figure F-62. Observed and Simulated Stage in Sub-basin C4_AG12 and Groundwater Well G-3329 for Model Validation (Hurricane Irene, October 14-15, 1999).

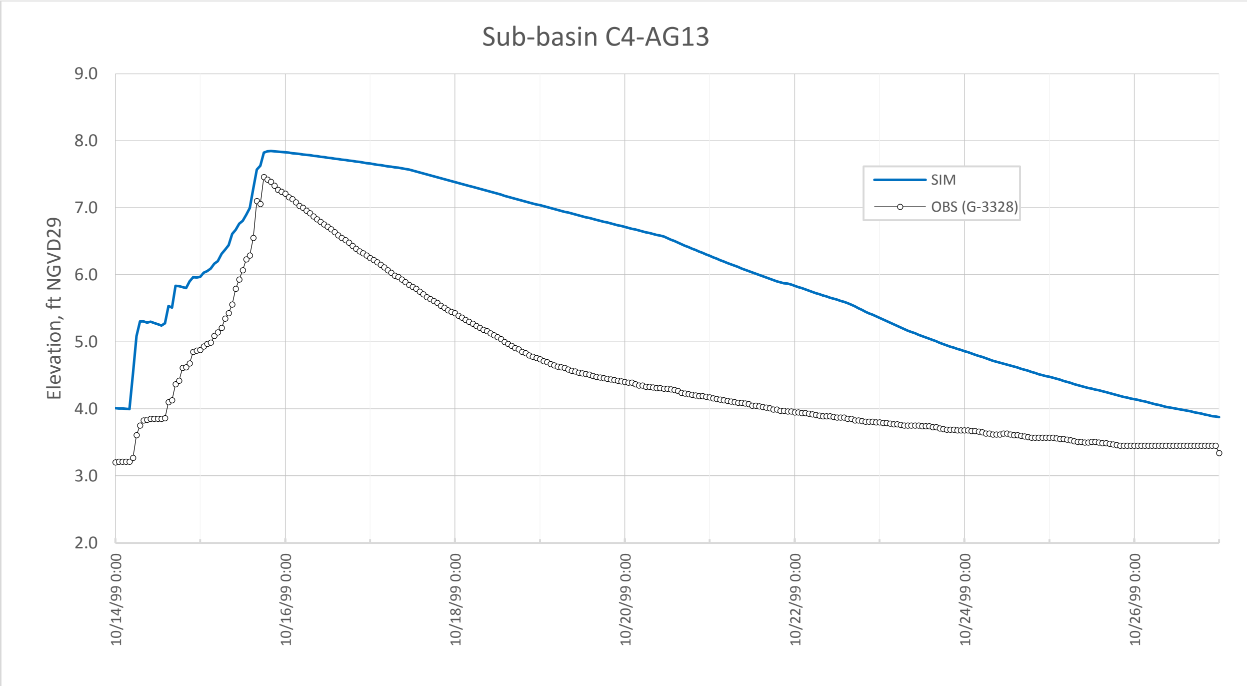

Figure F-63. Observed and Simulated Stage in Sub-basin C4_AG13 and Groundwater Well G-3328 for Model Validation (Hurricane Irene, October 14-15, 1999)

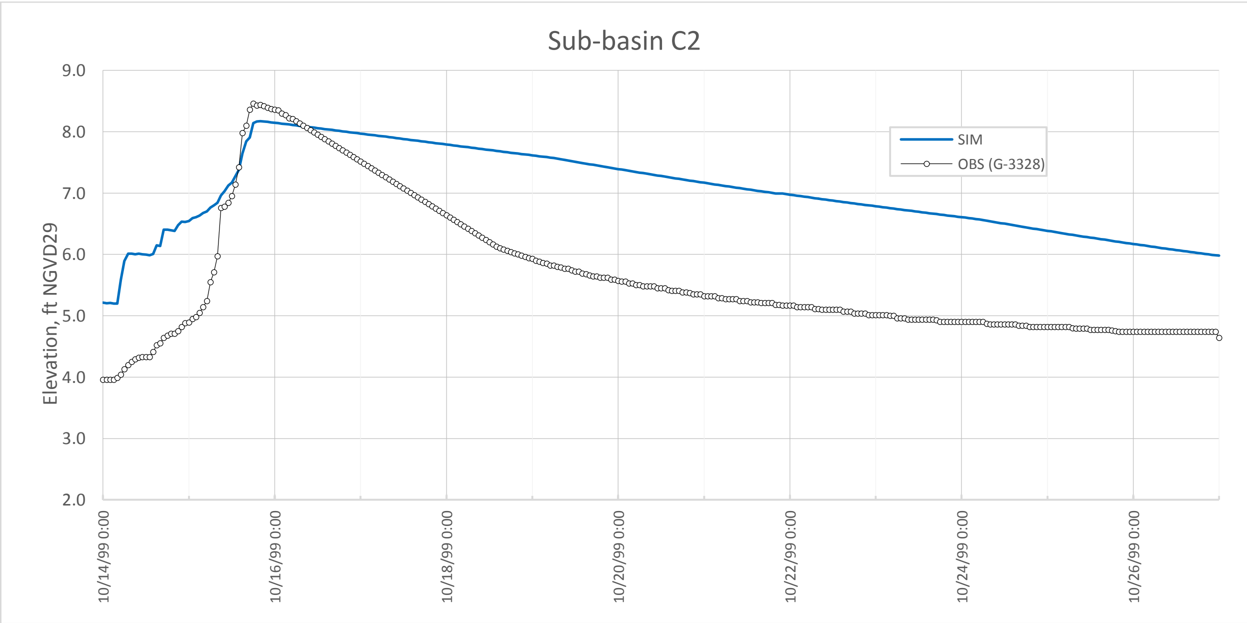

Figure F-64. Observed and Simulated Stage in Sub-basin C2 for and Groundwater Well G-3572 for Model Validation (Hurricane Irene, October 14-15, 1999)

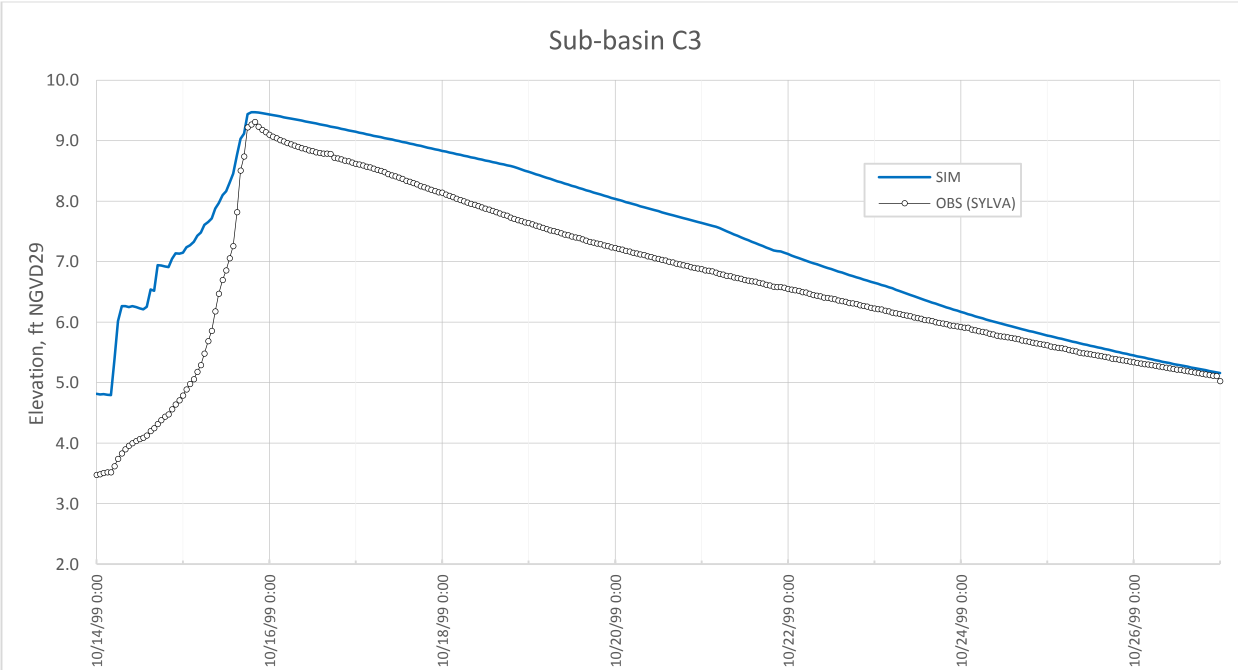

Figure F-65. Observed and Simulated Stage in Sub-basin C3 and Groundwater Well SYLVA for Model Validation (Hurricane Irene, October 14-15, 1999)

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

The hydrology and hydraulic performance of the C-4 Basins and canal drainage system were successfully represented with a HEC-RAS model of the C-4 Basin. The model was conceptualized using simple storage units that represent the storage in sub-basin areas of the basin and a full representation of the primary canal system and control structures currently in use in the basin. The conceptual model for the C-4 Basin required the inclusion of the C-2, C-3 and C-5 watersheds which have a significant effect on the flows and stages in the C-4 basin canals. These basins were simplified and included in the C-4 Basin model as single basins. Some structural error in the C-4 Basin model is expected from this simplification.

A total of 149 parameters were identified for calibration of which 65 were basin-to-basin flow conductances, 42 basin-to-canal conductances, 34 canal roughness coefficients, 6 structure gate and orifice flow coefficients and 2 boundary seepage conductances. The parameter adjustment during model calibration was performed with the PEST software using calibration error tolerances of 0.5 ft for both stage and RMSE. The model parameters were obtained through automatic calibration using the PEST software. After calibration, the model was validated using data from Hurricane Irene in October 1999.

The calibrated model performed reasonably well when reproducing canal stages and flows, and in some instances water levels in some sub-basins by satisfying the calibration error criteria for stage bias and RMSE (0.5 ft) at fifteen of seventeen (88%) calibration sites. For model validation, the error criteria was met at twelve of eighteen (67%) calibration sites. Uncertainty on the calibrated model parameters remains high due to the nature of the model conceptualization to represent groundwater flow between sub-basins and sub-basins to canal and since there are not field measurements of the parameters for validation. The model calibration could have benefited with data from additional groundwater stage data in the sub-basins since only six had observation data out of 44 sub-basins in the model domain.

A better representation of the surface storage in the system using 2-dimensional domains for the sub-basins would improve the model and allow for better estimation of flooding depths. The newest version of HEC-RAS (v 5.0 Beta) allows for linking 1-dimensional canal segments with 2-dimensional areas (sub-basins) in the same model. High resolution LiDAR data already available in the County would make this modeling improvement even more viable.

The conceptual model which includes both surface and groundwater flow exchanges between the sub-basins and the canals performs adequately for computing flow and stages in the canals given the limited number of sub-basins and canal segments in the model. The resulting HEC-RAS model run times are fast leading to relatively easy evaluation of alternative plans. The purpose of the calibrated model is to determine the system response to synthetic storm events of various return periods under existing and future levels of sea level rise. This task will be done and documented later.

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Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed

Appendix G: Design Flood Conditions

South Florida Water Management District
Hydrology and Hydraulics Bureau

December 29, 2015

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Appendix G: Design Flood Conditions

This document was produced by the H&H Bureau as a project deliverable for project 100888 (Flood Protection Level of Service, within the Sea Level Rise project).

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Additional project documents can be found on a District server at \\ad.sfwmd.gov\dfsroot\data\hesm_nas\projects\basin_studies\7_tidal_struc_modeling

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INTRODUCTION

In order to evaluate the C-4 basin level of service using the performance measures discussed previously, the state of the system was simulated for a number of flood events with varying recurrence intervals and tidal conditions. The tidal stages associated with these storm events were established for both current (i.e. 2009) sea level conditions and future projected sea levels that reflect a range of forecasted rises. In particular, three forecasts of sea level rise were incorporated into the model simulations: low, intermediate and high. The results of these simulations were used to quantify flood protection level of service (Appendix H).

MODEL SETUP AND CONCEPTUALIZATION

Simplifying Assumptions

The C-4 basin is highly complex in regards to land use, infrastructure and water management operations. The spatial distribution of rainfall over the basin can vary widely between multiple storms with the same frequency and duration, resulting in significant differences in basin stages, flooding durations and structure operations. Consequently, in order to assess level of service with the hydraulic model, a number of simplifications and assumptions had to be made. Essentially, these are:

- The design rainfall events occur uniformly over the entire C-4 basin.
- Rainfall occurring over a sub basin is initially retained in that basin.
- Inflows from the ENP and WCA-3B occur through ground water flow.
- No changes to basin infrastructure or structure operating rules occur during the time horizon associated with the sea level rise forecasts.
- The current C-4 basin operation plan remains in effect across all storm events and time horizons. Furthermore, no subjectivity in operating decisions is accounted for.
- Both current and future tidal stages were determined at the tail water monitoring locations for the coastal structures (S-25B, G-93 and S-22).
- The ENP and WCA-3B stages associated with each storm recurrence frequency are stationary.

Design Rainfall Events

A SFWMD 72-hour rainfall hyetograph was constructed for each design storm frequency. These hyetographs are shown in **Figure G-1** for the 5-yr, 10-yr, 25-yr and 100-yr design rainfall events, respectively. As indicated previously, the resultant rainfall hydrographs depicting sub basin rainfall were applied uniformly as direct inflow to each basin.

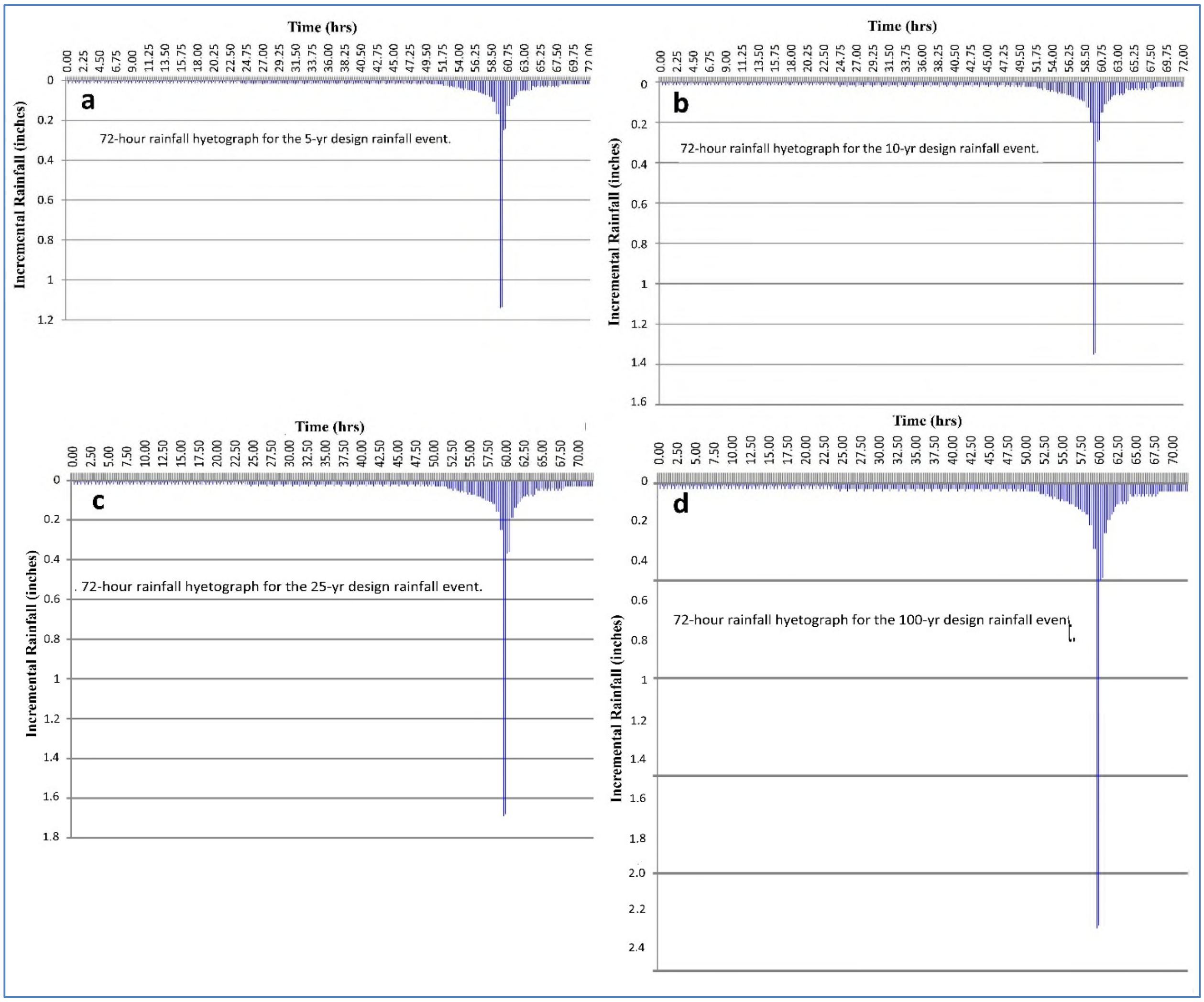

Figure G-1. Rainfall hyetographs for design rainfall events: a. 5-year, b. 10-year, c. 25-year, d. 100-year.

Tidal and Other Boundary Conditions

Downstream water levels at tidal structures for different rainfall conditions

For each model simulation, the external boundary conditions included zero flow through the structures located at the western and northern ends of the model domain along with direct ground water flow into the basins adjoining ENP and WCA-3B. At the eastern ends of C-2, C-3 and C-4, tidal stage hydrographs were denoted as the boundary conditions. In each case the tidal stage hydrograph pertained to the frequency of both the simulated storm event and the tidal conditions. For each of the storm recurrence intervals, three tidal stage hydrographs were developed in order to account for the range of probable forecasts. One hydrograph was designated as an intermediate forecasted tidal level while the other two reflected the low and high ends, respectively, of the probable tide stage range downstream of each coastal structure, so only results or the intermediate tide range are shown in the following graphs. Downstream water levels for S-25B, S-22 and G-93 structures for the 5, 10, 25 and 100-year design storm surge conditions are shown in **Figure G-2**.

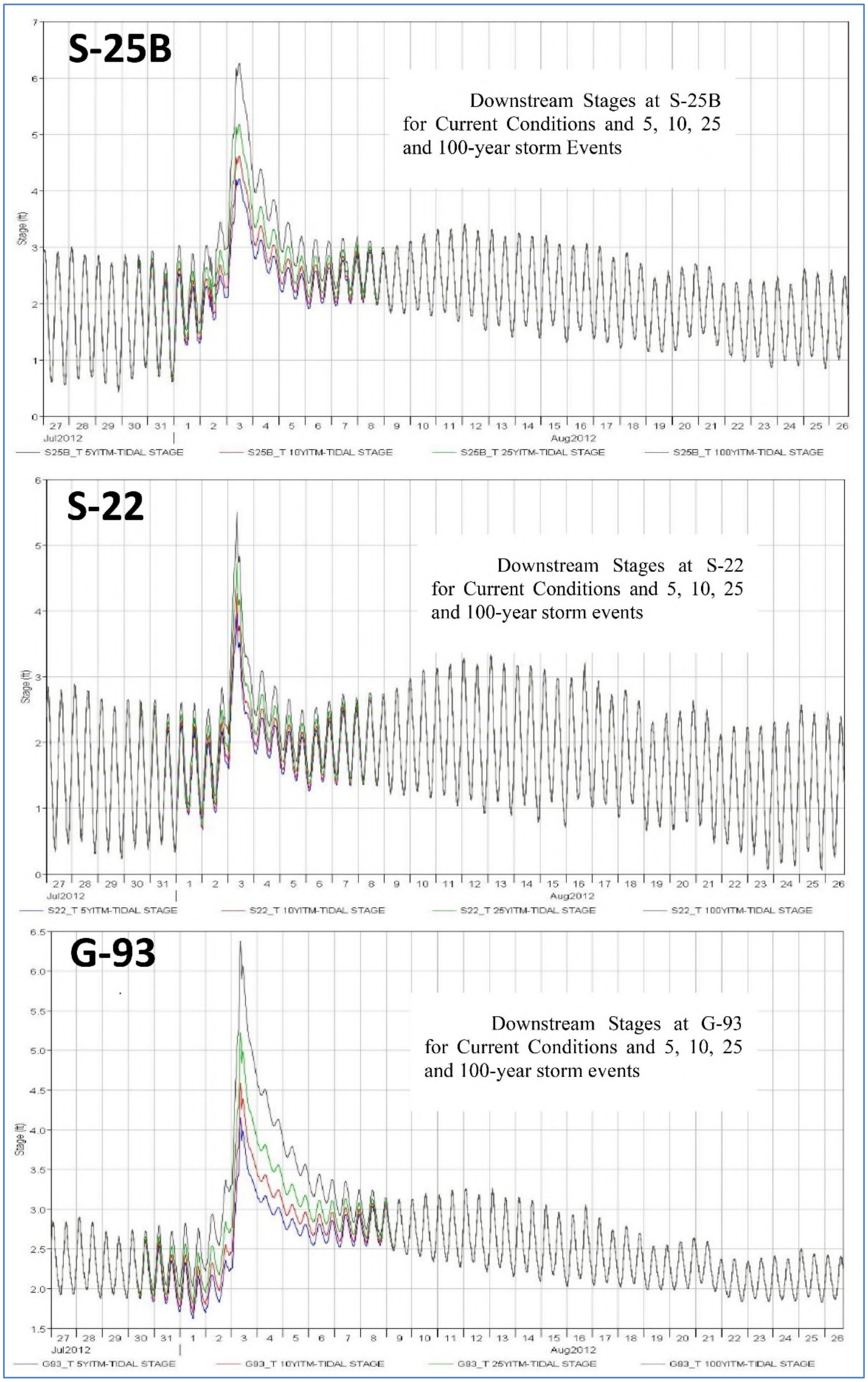

Figure G-2. Tidal stage hydrographs downstream of S-25B, S-22 and G-93 structures for current conditions and recurrence intervals of 5, 10, 25 and 100 years.

Flows at tidal structures under future sea level rise conditions

Under conditions that reflect projected sea level rise, the hydrographs differ significantly for each of the three sea level rise scenarios. These hydrographs are shown in **Figures G-3 through G-5**. Internal boundary conditions were specified at all water control structures and inter-basin connections. These boundary conditions included, where applicable, gate operating rules and flow computations.

Information describing the operational logic for the SFWMD structures within the C-4 basin are provided in **Appendix D**. Additional details on the operations of the structures during storm conditions are provided in the C-4 Basin Operating plan (**SFWMD, 2011**).

Initial Conditions

The initial basin conditions applied to the model were the same for each storm event and, as discussed in a separate appendix, were assumed to coincide with the 1 in 2 year annual base flow in the C-4 canal at structure S-25B. This flow rate is approximately 405 cfs. This flow occurred on September 29, 2012. Water levels measured on this date were used as initial stages for the model sub basins. Steady State conditions for the model were simulated using these water levels and a specified discharge rate at S-25B set equal to the base flow rate of 405 cfs. The sub basin stages and resultant canal flows were used as initial conditions for each storm simulation that reflects current conditions.

For model simulations depicting future conditions, the initial sub basin stages were increased so as to approximate future increases in wet season water levels predicted by Hughes and White, 2014. In this case, no base flow requirement for S-25B was specified. **Table G-1** lists the initial stages for the model sub-basins under both current and future conditions.

Simulation Time Window

The time window for the model simulations was designed to a) provide a start-up period where the model could transition from the assumed initial conditions to the wet-season hydrologic stresses imposed by the ambient tide cycle; b) impose the three-day storm event on the model after it has equilibrated with the wet season conditions inherent to the start-up period; and c) allow the model to continue simulating the system after the design storm and tidal events have passed and the system returns to normal transient wet season conditions. To accomplish these objectives, the model simulation time window included a five-day start-up period that was immediately followed by the three-day storm event. The simulation then continued for another 17 days after the peak of the storm event to ensure ample time for system recovery. The starting time and date were 2400 on 26 July 2012.

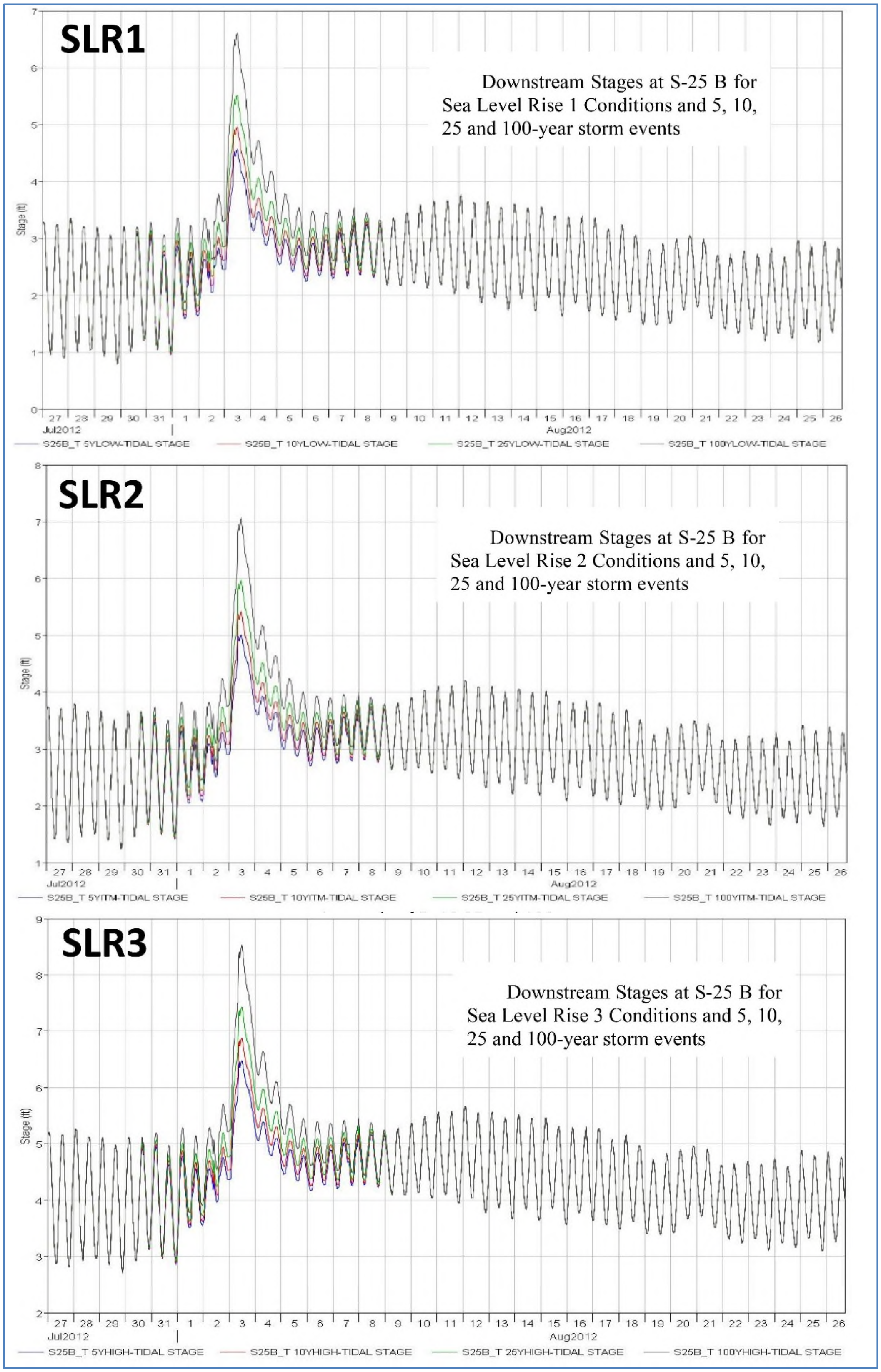

Figure G-3. Tidal stage hydrographs downstream of S-25B for future SLR1, SLR2 and SLR3 conditions and recurrence intervals of 5, 10 25 and 100 years.

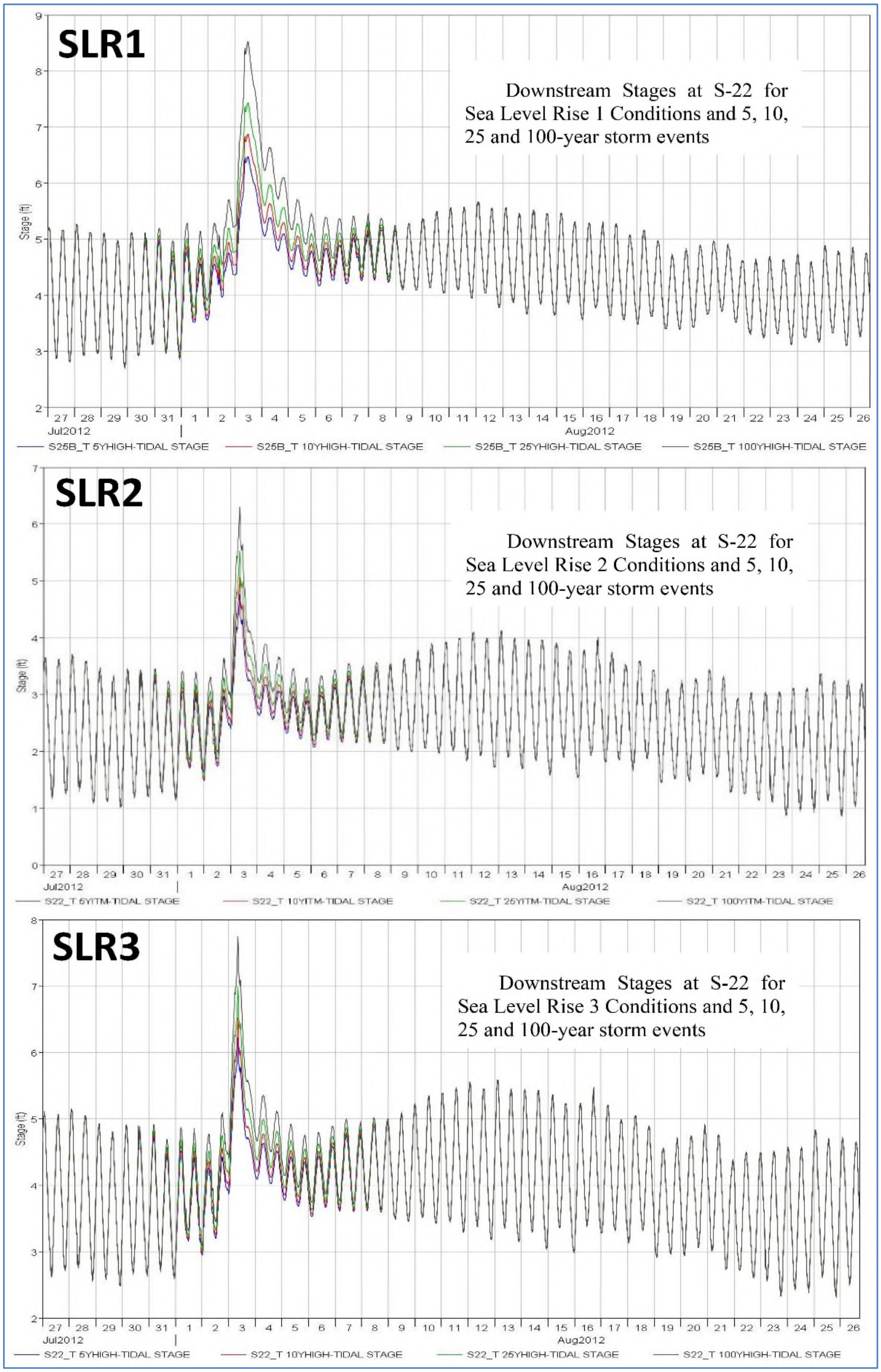

Figure G-4. Tidal stage hydrographs downstream of S-22 for future SLR1, SLR2 and SLR3 conditions and recurrence intervals of 5, 10 25 and 100 years.

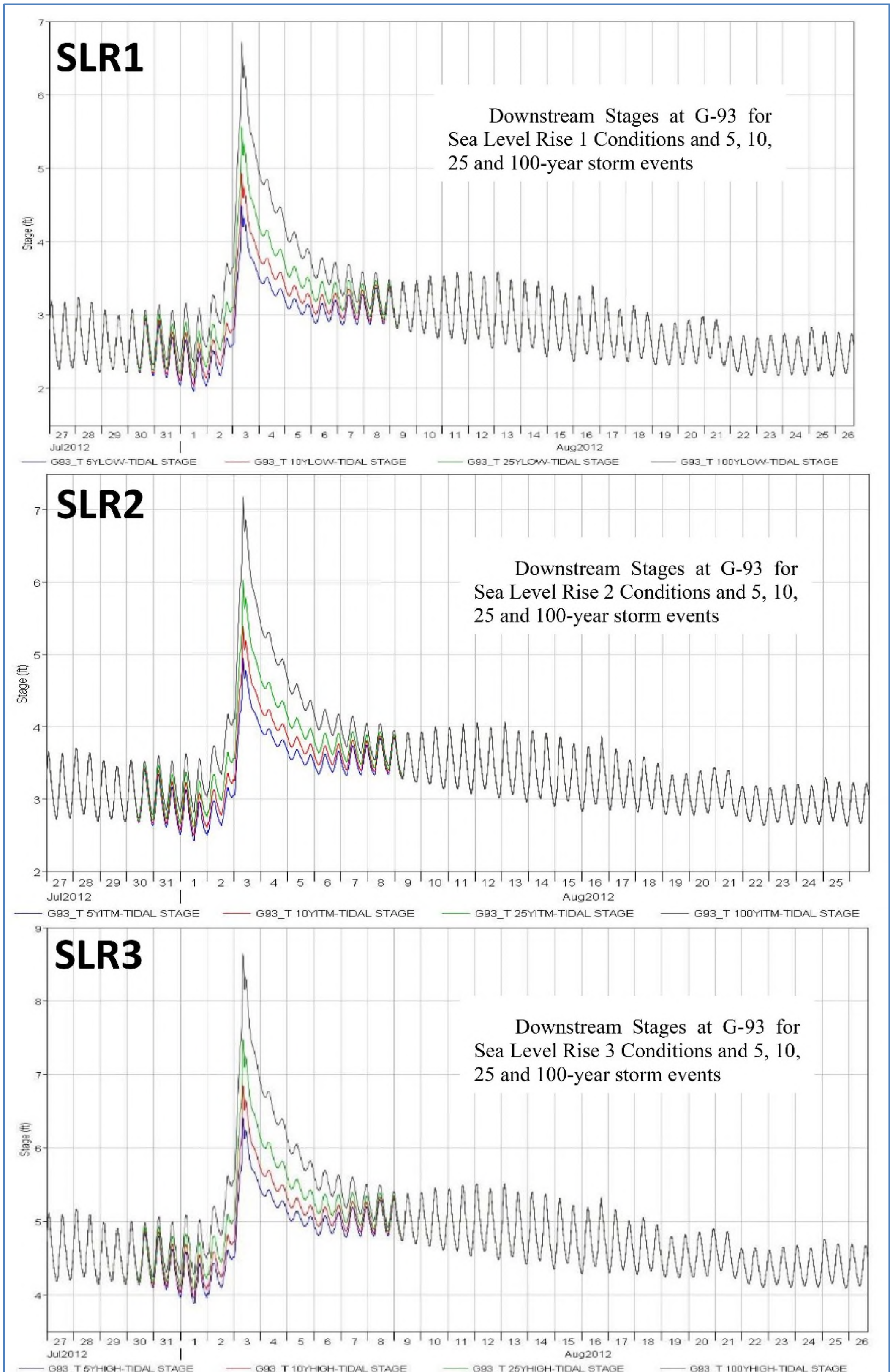

Figure G-5. Tidal stage hydrographs downstream of G-93 for future SLR1 conditions and recurrence intervals of 5, 10 25 and 100 years.

Table G-1. Initial stages for the model sub-basins under current and future conditions.

Storage Area	Current Conditions	Future Conditions
C2	3.44	3.74
C2-N-24	3.36	3.81
C3	1.84	2.69
C4-N-3	5.11	5.31
C4-N-4	4.93	5.13
C4_100B	2.66	3.46
C4_100C	2.57	3.37
C4_10A	6.31	6.41
C4_10B	5.32	5.52
C4_10C	4.41	4.76
C4_10D	4.33	4.68
C4_10E	4.95	5.15
C4_125A	2.3	3.15
C4_125B	2.47	3.29
C4_150A	1.96	2.81
C4_25	4.59	4.84
C4_40	3.68	4.08
C4_55	3.04	3.64
C4_60A	2.65	3.35
C4_65A	3.12	3.72
C4_65B	3.15	3.75
C4_70	2.83	3.53
C4_75A	3.36	3.86
C4_75B	2.86	3.51
C4_AG1	3.99	4.34
C4_AG10	1.75	2.65
C4_AG11	1.93	2.78
C4_AG12	2.39	3.21
C4_AG13	2.02	2.92
C4_AG14	1.64	2.54
C4_AG2	3.51	3.96
C4_AG3	3.49	3.94
C4_AG4	3.3	3.8
C4_AG5	2.63	3.43
C4_AG6	2.7	3.5
C4_AG7	2.6	3.4
C4_AG8	2.39	3.21
C4_AG9	2.21	3.06
C5	1.5	2.4

SIMULATION SCENARIOS

As discussed previously, each scenario reflects a recurrence frequency that pertains to both the rainfall storm event and the peak tidal stage. The recurrence intervals included 5, 10, 25 and 100 years. Since the three tidal stage hydrographs discussed earlier were nearly the same for current conditions, only four model simulations depicting current conditions were carried out: one for each storm recurrence frequency, and the intermediate tidal stage hydrograph was assigned as the boundary condition downstream of each coastal spillway. In contrast, since the hydrographs depicting the three tidal stage forecasts under future conditions differ significantly, twelve model simulations reflecting future conditions were performed. That is, the simulation associated with each of the four storm events was repeated three times using each of the three forecasted tidal stage hydrographs (SLR1, SLR2 and SLR3). This resulted in 16 model simulations that were used to assess basin level of service.

A key assumption inherent to these model simulations was that the peak rainfall rate occurred at the same time as the peak tidal stage. Due to routing effects, though, this will not result in peak flows reaching the coastal structures at the same time as the peak tidal stage. For a conservative assessment, additional simulations were carried where peak tidal stages better coincided with peak flows. This was accomplished by simply adding an appropriate time delay between peak rainfall and peak tidal stage.

SIMULATION RESULTS

Current Conditions

Results from the model simulations for current conditions are summarized in Tables G-2 to G-4. These results include peak flows and stages at the water control structures, water levels in the sub-basin storage areas, and flows through the pump stations.

Table G-2. Summary of the peak flow (cfs) through pump stations

Pump Stations	5-yr	10-yr	25-yr	100-yr
G420_Pump	669	669	669	669
G422_Pump	623	623	623	623
Belen_PS1	100	100	100	100
Belen_PS2	100	100	100	100
S25B_FP	630	630	630	630
Miami_1	0	0	0	0
Miami_2	0	0	0	0
Miami_3	0	0	0	0
Miami_4	0	0	0	0
Miami_5	0	0	0	0
Sweetwater_B1	20	20	20	20
Sweetwater_B2	20	20	20	0
Sweetwater_B15	0	0	0	0
Sweetwater_B16	0	0	0	0
Sweetwater_IIA3	0	0	0	0
Sweetwater_IIA4	0	0	0	0
Sweetwater_IIB1	27	27	27	27
Sweetwater_IIB2	0	0	27	0
West_Miami_1	90	90	90	90

Table G-3. Summary of the peak flow and maximum stages at structures

Structure	5-yr			10-yr			25-yr			100-yr		
	Max. HW (ft)	Max. TW (ft)	Peak Flow (cfs)	Max. HW (ft)	Max. TW (ft)	Peak Flow (cfs)	Max. HW (ft)	Max. TW (ft)	Peak Flow (cfs)	Max. HW (ft)	Max. TW (ft)	Peak Flow (cfs)
S22	4.06	3.96	1454	4.30	4.26	1561	4.67	4.72	1913	5.28	5.49	2242
G93	5.65	4.15	641	6.01	4.59	586	6.54	5.22	660	7.42	6.37	706
S380	6.38	6.06	315	6.56	6.17	329	7.61	6.33	359	8.06	6.90	191
S25B	4.62	4.22	2164	5.06	4.62	2423	5.95	5.17	2857	6.70	6.26	3236
S25A	4.73	3.71	0	5.17	4.03	0	5.98	4.43	0	6.79	6.34	0
S25 Gate	3.69	5.46	269	4.01	5.27	243	4.42	5.34	249	6.26	6.26	252
G421	8.04	6.08	256	8.41	6.12	260	10.03	6.32	278	10.03	7.00	341

Table G-4. Maximum sub-basin water levels (ft NGVD) under existing conditions for four simulated rainfall events

Storage Area	5-yr (ft. NGVD)	10-yr (ft. NGVD)	25-yr (ft. NGVD)	100-yr (ft. NGVD)
C2	6.28	6.69	7.14	7.88
C2-N-24	5.66	6.11	6.85	7.61
C3	5.64	6.24	7.09	8.03
C4-N-3	8.04	8.42	10.04	10.05
C4-N-4	8.04	8.41	10.03	10.03
C4_100B	5.15	5.83	6.62	7.47
C4_100C	5.13	5.80	6.58	7.44
C4_10A	7.18	7.34	7.62	8.06
C4_10B	5.86	6.06	6.33	6.90
C4_10C	5.73	5.93	6.23	6.80
C4_10D	5.67	5.92	6.35	7.13
C4_10E	6.20	6.43	6.73	7.13
C4_125A	4.10	4.52	5.11	5.99
C4_125B	4.81	5.42	6.23	7.12
C4_150A	4.11	4.59	5.25	6.10
C4_25	5.74	6.06	6.32	7.00
C4_40	5.70	6.02	6.82	7.56
C4_55	5.80	6.37	7.19	8.08
C4_60A	5.75	6.34	7.18	8.07
C4_65A	5.76	6.35	7.14	8.00
C4_65B	6.02	6.52	7.06	7.82
C4_70	5.45	6.09	6.85	7.65
C4_75A	5.76	6.00	6.64	7.48
C4_75B	5.16	5.85	6.63	7.48
C4_AG1	5.82	6.28	6.79	7.45
C4_AG10	6.78	7.34	8.11	9.27
C4_AG11	5.60	6.01	6.73	7.68
C4_AG12	6.24	6.42	6.68	7.46
C4_AG13	4.90	5.42	6.41	7.38
C4_AG14	4.34	4.75	5.35	6.07
C4_AG2	6.04	6.29	6.82	7.56
C4_AG3	5.78	6.25	6.87	7.56
C4_AG4	6.00	6.60	7.01	7.73
C4_AG5	5.17	5.84	6.63	7.47
C4_AG6	5.20	5.83	6.62	7.47
C4_AG7	5.44	6.06	6.91	7.97
C4_AG8	5.93	6.59	7.49	8.44
C4_AG9	5.59	6.03	6.73	7.58
C5	5.57	5.90	6.44	7.16

Base Case Results for 1-in-100 Year Storm Event

Maximum Stage Profiles

The 100-yr flood water level profiles in **Figure G-6** depict the maximum computed water elevations along the C-4 canal under current conditions. The computed maximum stage in C-4 canal is 8.2 feet NGVD and occurs approximately 40,000 feet upstream of structure S-25B.

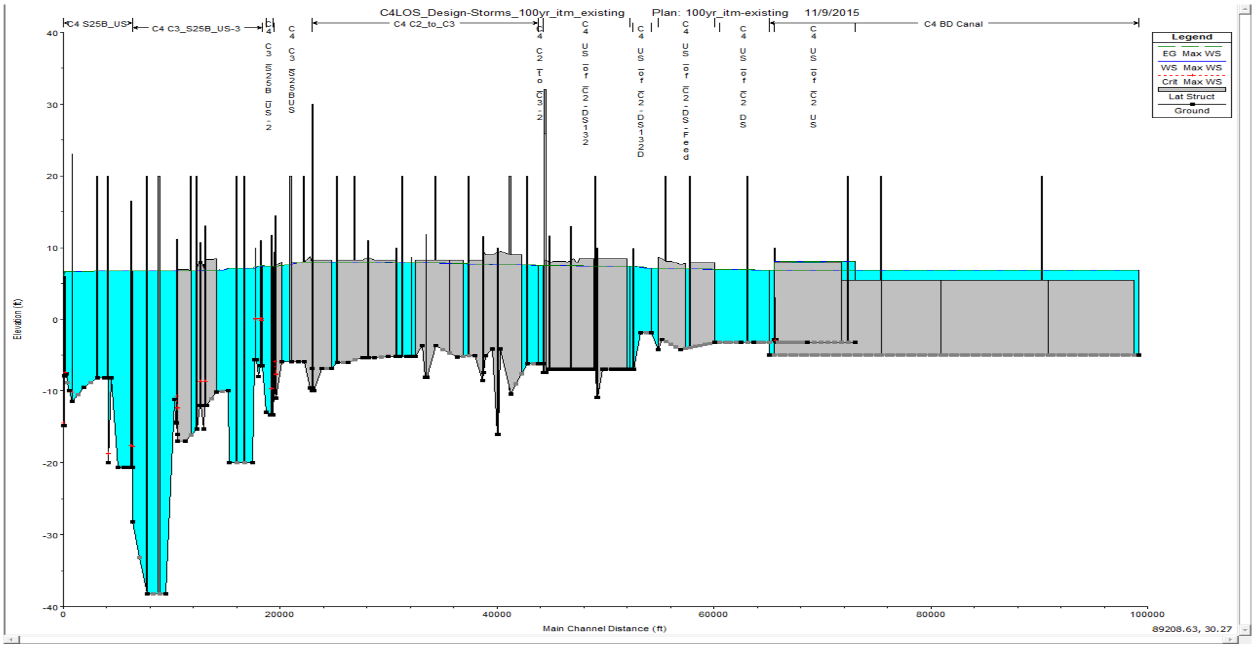

Figure G-6. 100-yr flood water level profiles for current conditions

Major Structures

S-25B

The headwater and tail water stages of the structure S-25B are shown in **Figure G-7**. The peak of the event occurred at about 1:00 pm on Aug 3, when the headwater stage is at 6.7 ft. This is 1.0 foot higher than the posted overflow elevation of 5.7 feet. The forward pumps turned off during the peak time because the gravity discharge limit of 600 cfs was exceeded at the structure. From the peak of the event on, the pumps remained inactive until the gravity flow through the gates fell below 600 cfs on Aug. 8. The gates were subsequently closed intermittently due to the proximity of upstream and downstream stages (≤ 0.1 feet).

S380

Figure G-8 presents the computed headwater and tail water stages at S-380 along with its computed flows. The headwater and tail water stages peaked at elevations 8.1 and 6.9 ft. NGVD, respectively on Aug. 7. Since the water levels at T5 were higher than the Zone 5 trigger stage (6.5 feet), all five culverts at the structure were closed during the peak of the event. After the peak of the storm, one culvert started to operate (i.e. the primary gate) when the T5 water level fell below the trigger stage of 6 feet NGVD established in the C-4 basin operations plan (SFWMD, 2011).

G-420 and G-422

Downstream of structure S-380 are the C-4 Emergency Detention Basin and its inflow pump stations G-420 and G-422. **Figure G-9** and **Figure G-10** show the stages and flows associated with these structures during the 100-year storm event simulation. Boxes with arrows are no longer used in the remaining figures, the color coding should be self-explanatory. The C-4 Detention Basin inflow pumps G-420 and G-422 were activated at the specified C-4 canal trigger stages (SFWMD, 2011) until the water level in the Detention Basin reached an elevation of 10.0 ft. After that, the pumps operated intermittently at lower detention basin stages until the storm passed and further pumping into the C-4 Detention Basin was no longer needed. The G-420 and G-422 pumps were turned off at their designated trigger stages (SFWMD, 2011).

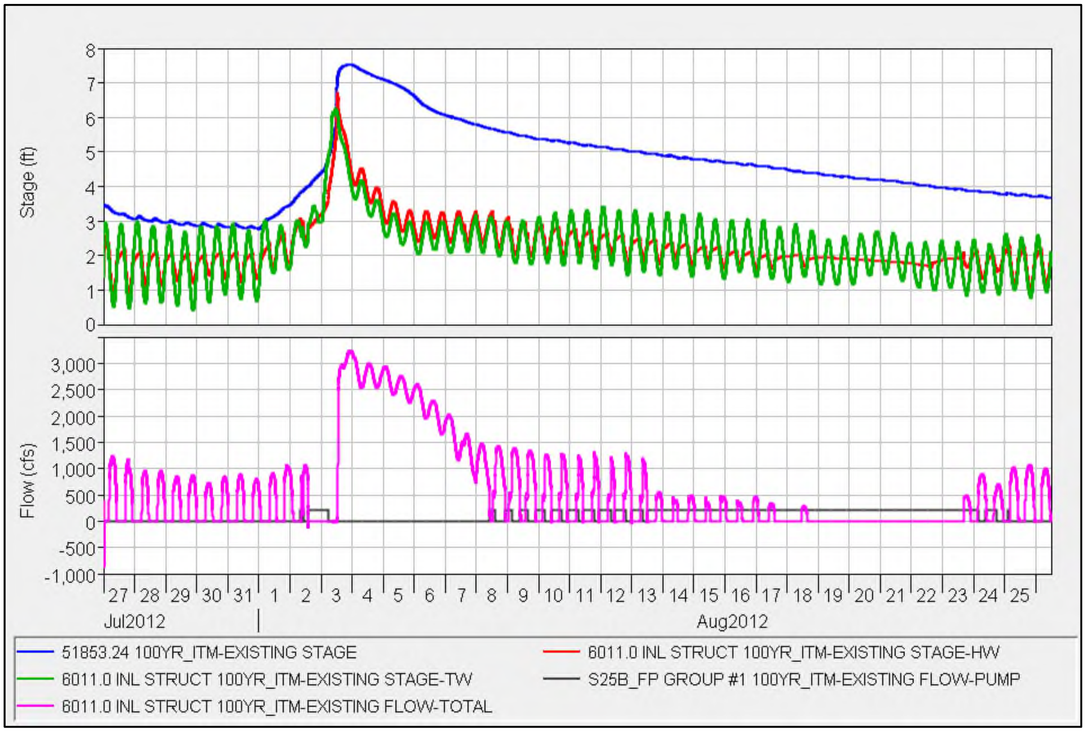

Figure G-7. 100-yr event at S-25B for current conditions: the top half shows HW&TW stages, as well as T5 stages; bottom half shows flows through S-25B and S-25B forward pumps.

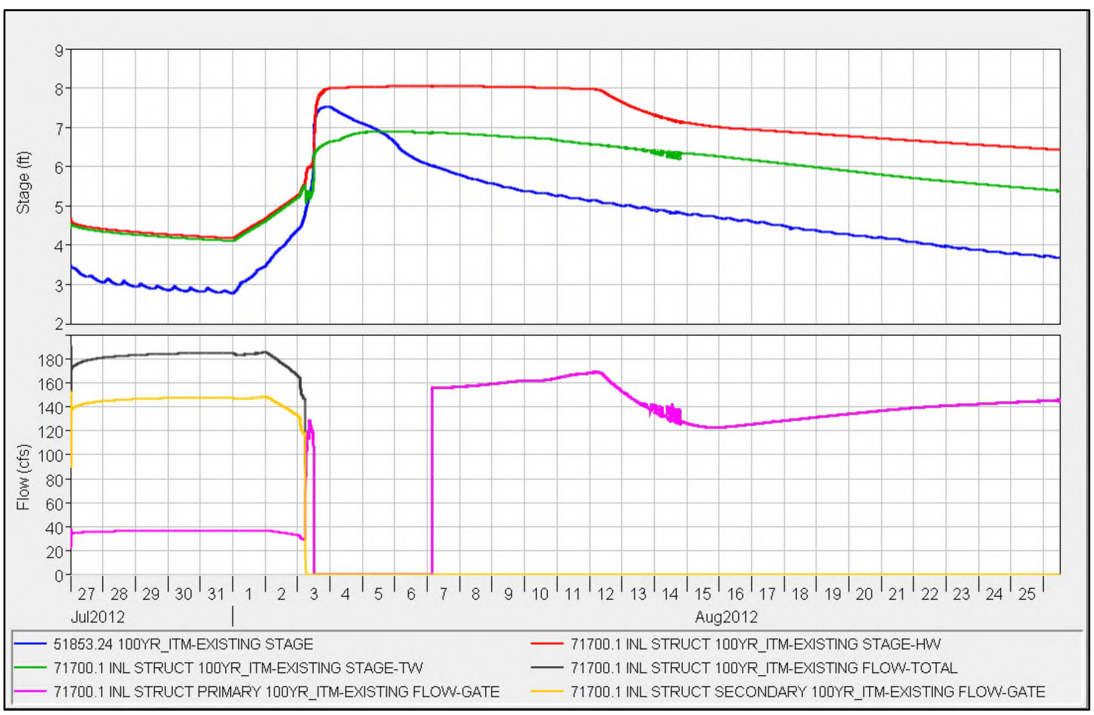

Figure G-8. 100-yr S-380: top half shows HW&TW stages, as well as T5 stages; bottom half shows flows through S-380.

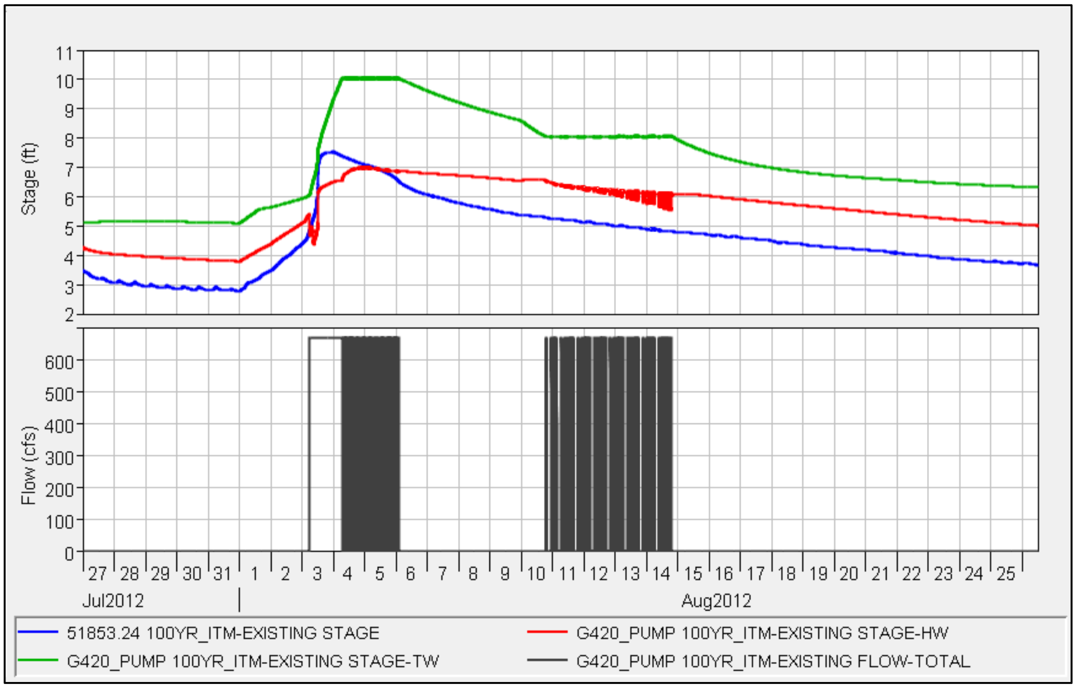

Figure G-9. 100-yr Pump G420: top half shows HW&TW stages, as well as T5 stages; bottom half shows G-420P total flow.

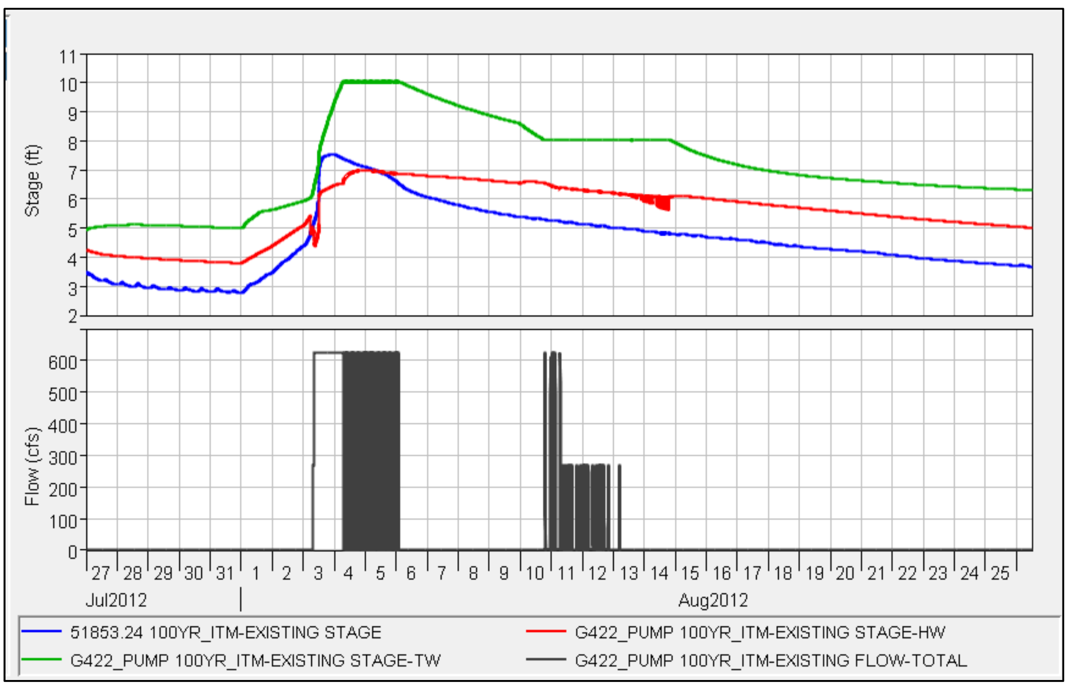

Figure G-10. 100-yr Pump G422: top half shows HW&TW stages, as well as T5 stages; bottom half shows G-422P total flow.

G-421

Figure G-11 illustrates the G-421 head water, tail water and total flow. It remained closed during the peak of the event. After the storm passed it released storm water from the detention area to the C-4 Canal as specified in the C-4 basin operations plan (SFWMD, 2011).

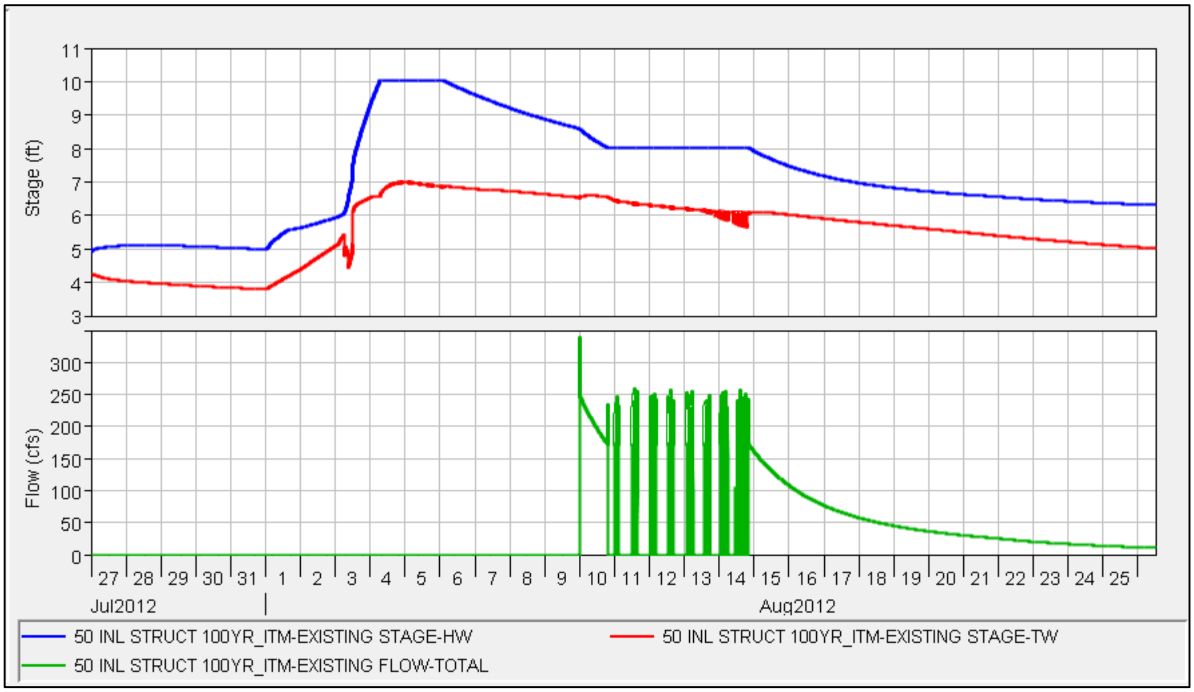

Figure G-11. 100-yr G-421: top half shows HW&TW stage, bottom half shows flows from G-421.

G-93

Figure G-12 shows headwater, tail water and total flow through G-93. The peak headwater stage is 7.43 ft.

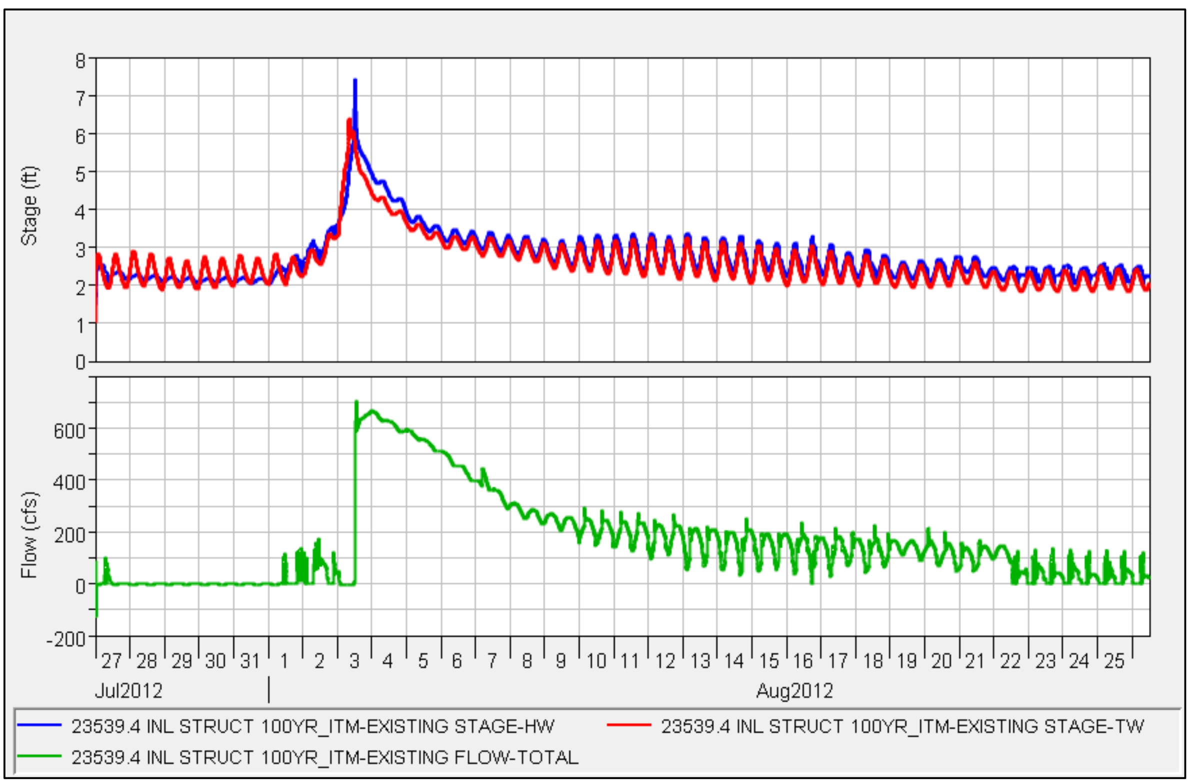

Figure G-12. 100-yr G-93: top half shows HW&TW stage, bottom half shows flows from G-93.

S-22

Figure G-13 shows head water, tail water and total flow through structure S-22. During the peak period, the headwater is at 5.28 ft.

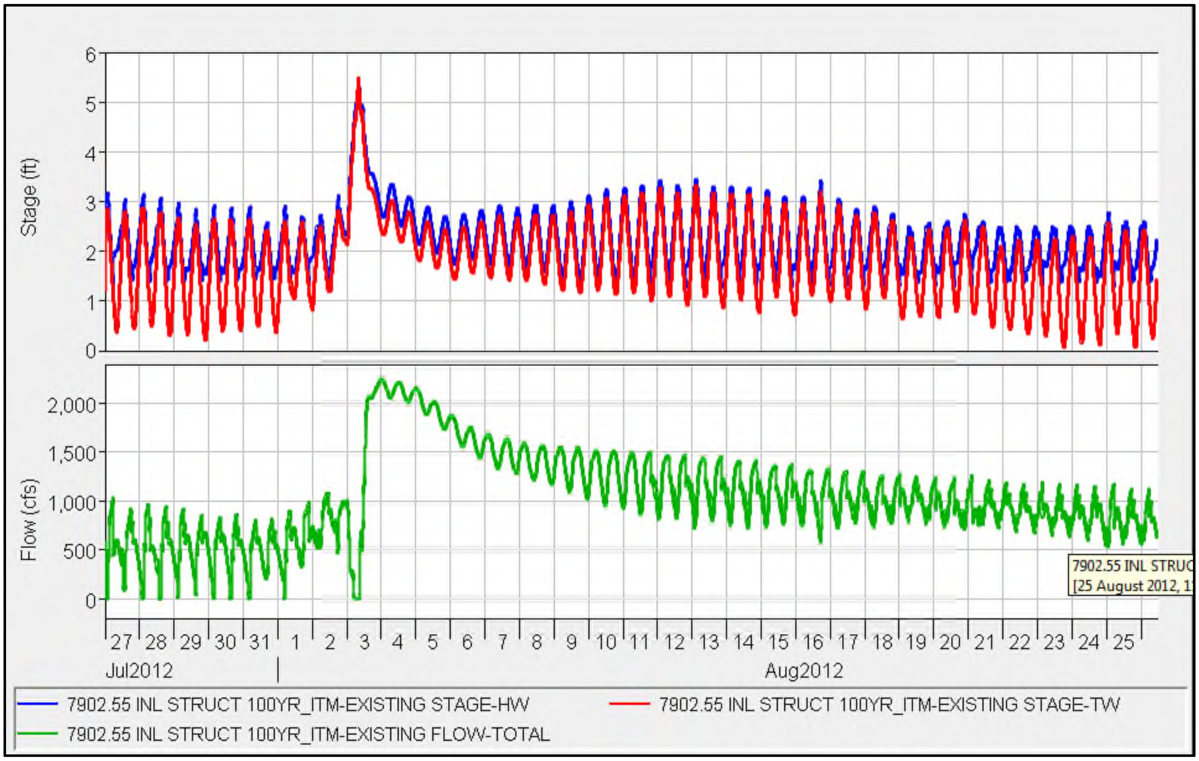

Figure G-13. 100-yr S-22: top half shows HW&TW stage, bottom half shows flows from S-22.

Municipal Pumps

Operations of the municipal pumps were based on the water levels within the sub basins and in the C-4 canal. Pumped discharges from the sub basins stop when (i) water levels exceed 6.5 feet NGVD in the C-4 canal at either the T5W or C4-Coral gauge; or (ii) The C-4 stage at gauge MRMS1 exceeds 4.75 feet NGVD; or (iii) The detention basin is full; or (iv) the detention basin is near full and its stage is rising; or (v) the sub basin water levels fall to the designated elevations for pump shutoff . **Table G-5** summarizes the pump trigger stages for these municipal pumps.

Figure G-14 and G-15 show the head water and tail water stages along with total flow for pump stations Belen PS1 and PS2. These two stations pump water from sub basin C4_AG1 to the C4 Canal. Belen PS1 and PS2 were activated when the water level in C4_AG1 reached 5 feet during the storm and stopped when the water level in C4_AG1 fell below 4.5 ft. The pumps were off during the peak period of the storm when the water levels at T5 exceeded the control level of 6.5 ft.

Figure G-16 through G-20 show the head water and tail water stages along with total flow for pump station Miami #1 to #5, respectively. All the pumps are primarily turned on or off based on established trigger stages within the respective sub-basins served by the pumps (Table G-5). However, their operations are also limited by the maximum allowable stages at gages T-5W, C4.Coral and MRMS1. The pumps were off during the peak period of the storm when the water levels at T5 exceeded the control level of 6.5 ft. Furthermore, the pumps remained off after the peak period because the head water levels were lower than the pump trigger stages.

Figure G-21 and **G-22** show the head water and tail water stages along with total flow for pump stations West Miami #1 and #2, respectively. West Miami #1 pumps water from sub basin C3 to the C4 canal and West Miami #2 pumps water from sub basin C3 to the C3 canal. All the pumps were turned off during the peak period of the storm when the water levels at C4-Coral and T5 were above their trigger values. The pumps were operated after the peak storm period when the water levels at C4-Coral and T5 fell below specified values. Because of limited pump capacity, operation of the pumps had no apparent significant effect on the water level in the sub basin.

Figure G-23 through G-30 show head water, tail water and total flow through Sweetwater pump stations B1, B2, B15, B16, IIB1, IIB2, IIA3, and IIA4. All the pumps were turned off during the peak period of the storm when the water levels at C4-Coral and T5 were above their trigger values. Pump station B1 and IIB1 were operated for a short periods of time when the water levels at C4-Coral and T5 were low enough and their head water levels were high enough. Because of the low capacity of the pumps, they had no apparent significant effect on the basin water level.

Table G-5. Summary of the pump trigger stages for pump stations

Pump Station	From	To	Headwater Triggers (ft-NGVD)	
			On	*Off*
Belen_PS1	C4_AG1	C4 Canal	5	4.5
Belen_PS2	C4_AG1	C4 Canal	5	4.5
Miami_1	C4_AG8	C4 Canal	6	5.41
Miami_2	C4_AG8	C4 Canal	6	5.41
Miami_3	C4_AG9	C4 Canal	6	5.41
Miami_4	C4_AG9	C4 Canal	6	5.41
Miami_5	C4_AG11	C4 Canal	6	5.41
Sweetwater_B1	C4_AG4	C4 Canal	5.5	5
Sweetwater_B2	C4_AG4	C4 Canal	6	5.5
Sweetwater_B15	C4_AG2	C2EXT Canal	6.5	6.25
Sweetwater_B16	C4_AG2	C2EXT Canal	6.5	6.25
Sweetwater_IIA3	C4_AG3	C4 Canal	6.5	6.25
Sweetwater_IIA4	C4_AG3	C4 Canal	6.5	6.25
Sweetwater_IIB1	C4_AG3	C4 Canal	5.5	5
Sweetwater_IIB2	C4_AG3	C4 Canal	6	5.5
West_Miami_1	C3	C4 Canal	5	2.51
West_Miami_2	C3	C4 Canal	5	2.51

* Pump station operations were also limited by the stages at gages T-5W (off @ 6.5’, on @ 6’), C4.Coral (off @ 6.5’, on @ 6’), MRMS1(off @ 4.75’, on @ 4.25’) and C4-N-4 (off @ 9.8’)

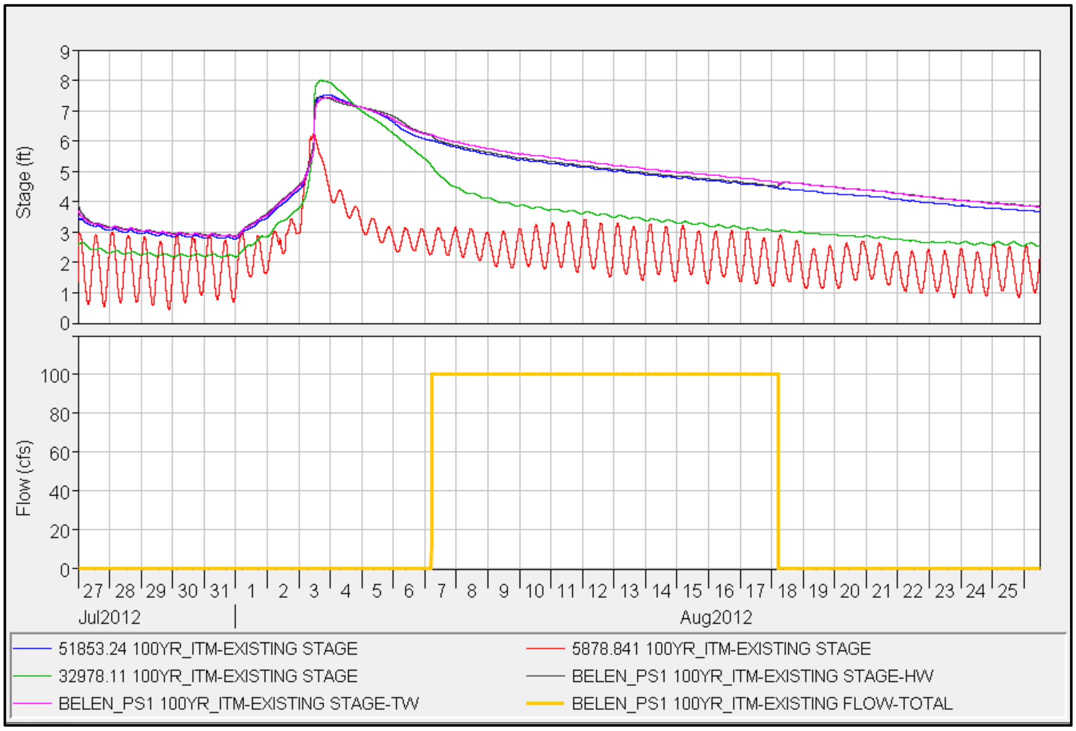

Figure G-14. 100-yr Pump Belen_PS1: top half shows HW&TW stage, as well as T5 Stage, bottom half shows Belen_PS1 total flow.

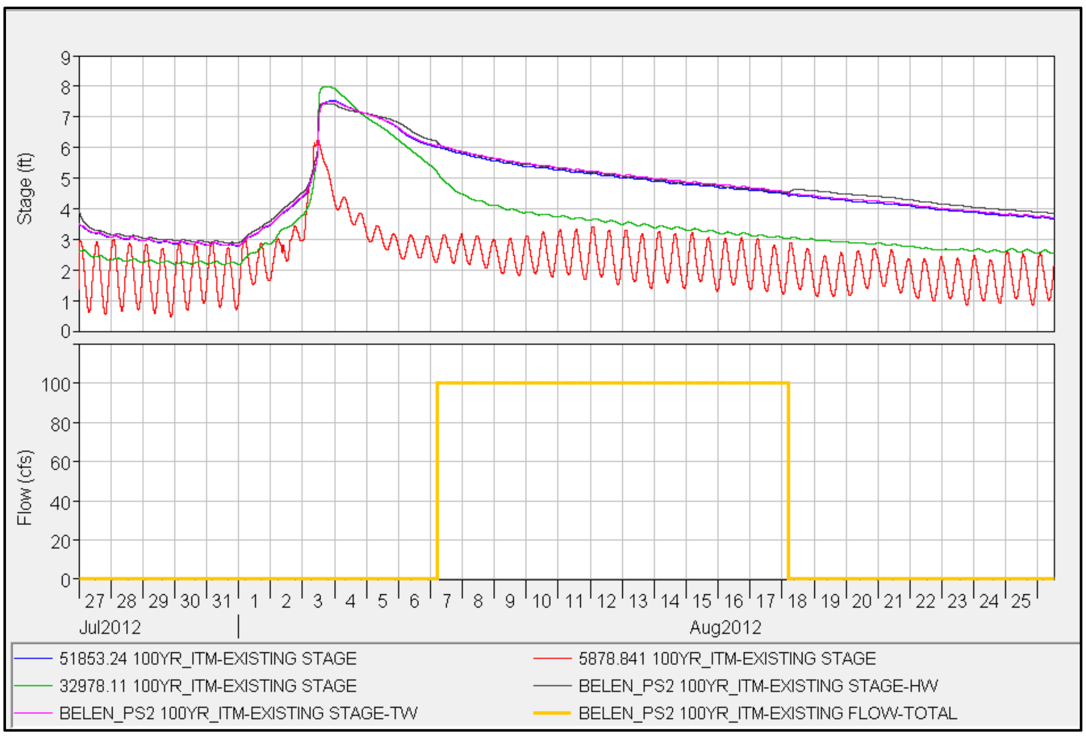

Figure G-15. 100-yr Pump Belen_PS2: top half shows HW&TW stage, as well as T5 stage, bottom half shows Belen_PS2 total flow.

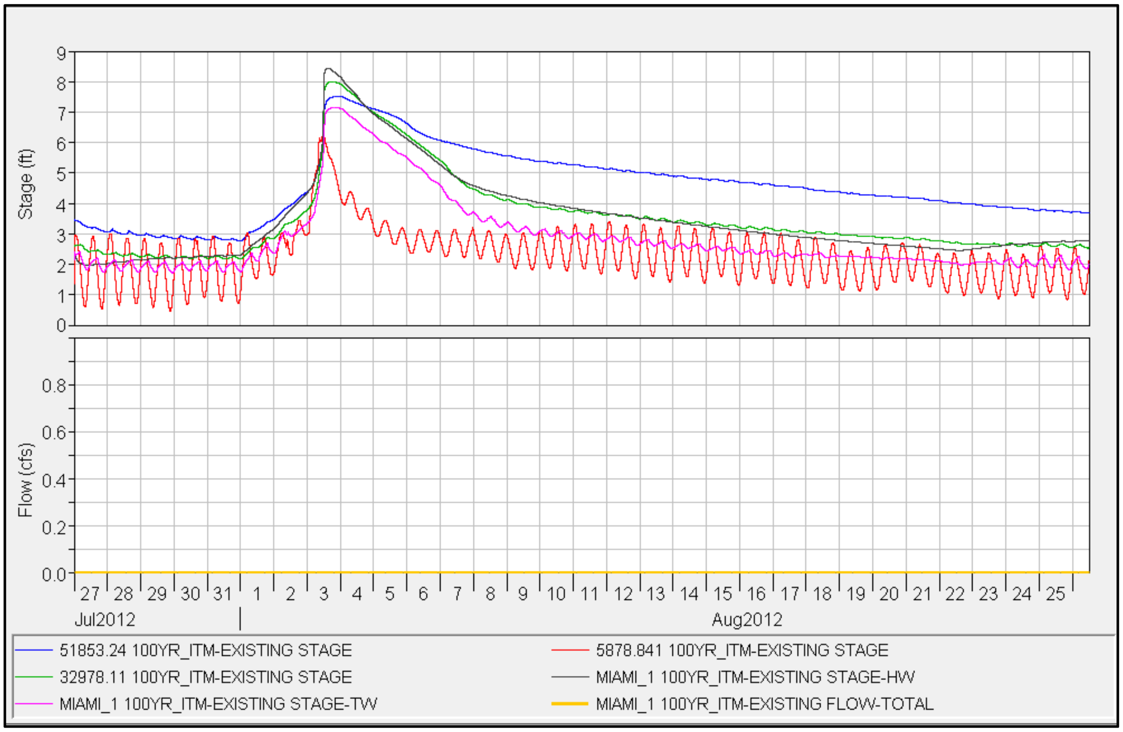

Figure G-16. 100-yr Pump Miami #1: top half shows HW&TW stage, C4CORAL and T5 stage, bottom half shows Miami #1 total flow.

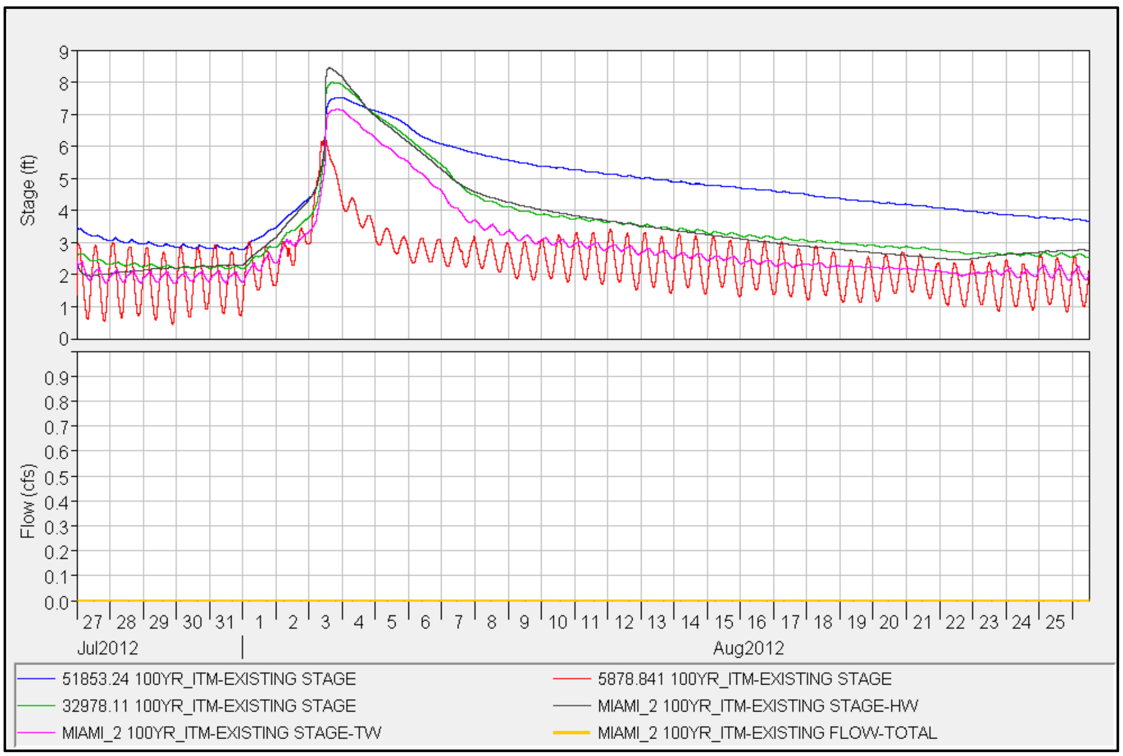

Figure G-17. 100-yr Pump Miami #2: top half shows HW&TW stage, C4CORAL and T5 stage, bottom half shows Miami #2 total flow.

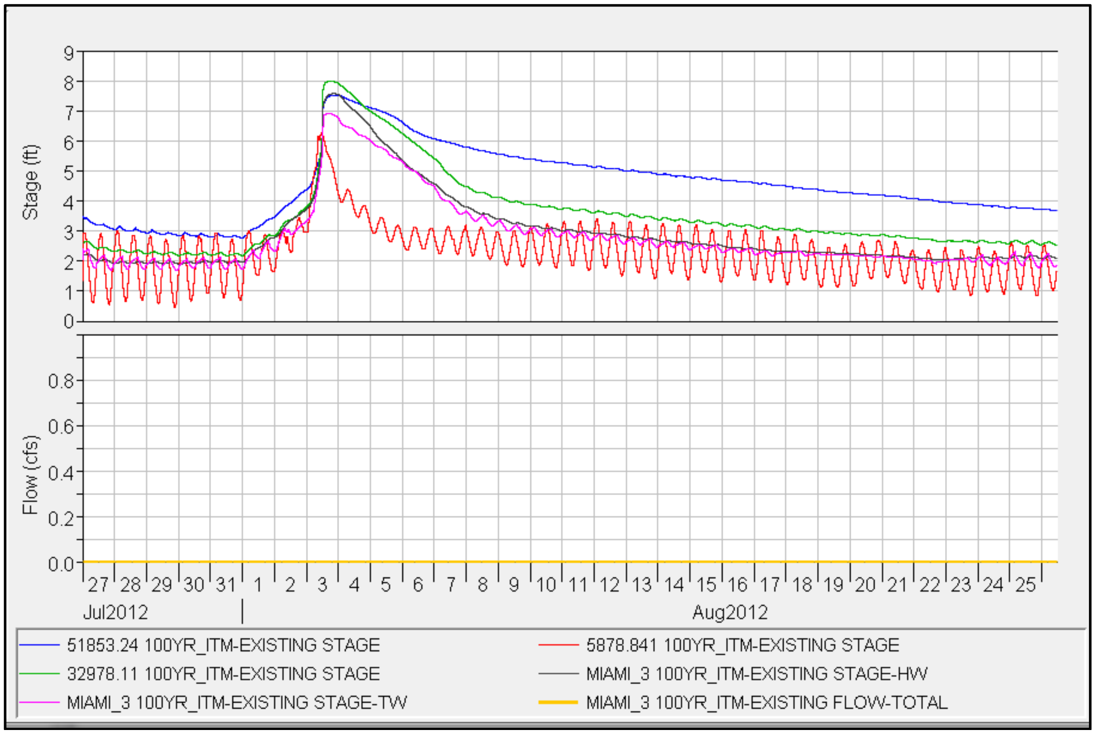

Figure G-18. 100-yr Pump Miami #3: top half shows HW&TW stage, C4CORAL and T5 stage, bottom half shows Miami #3 total flow.

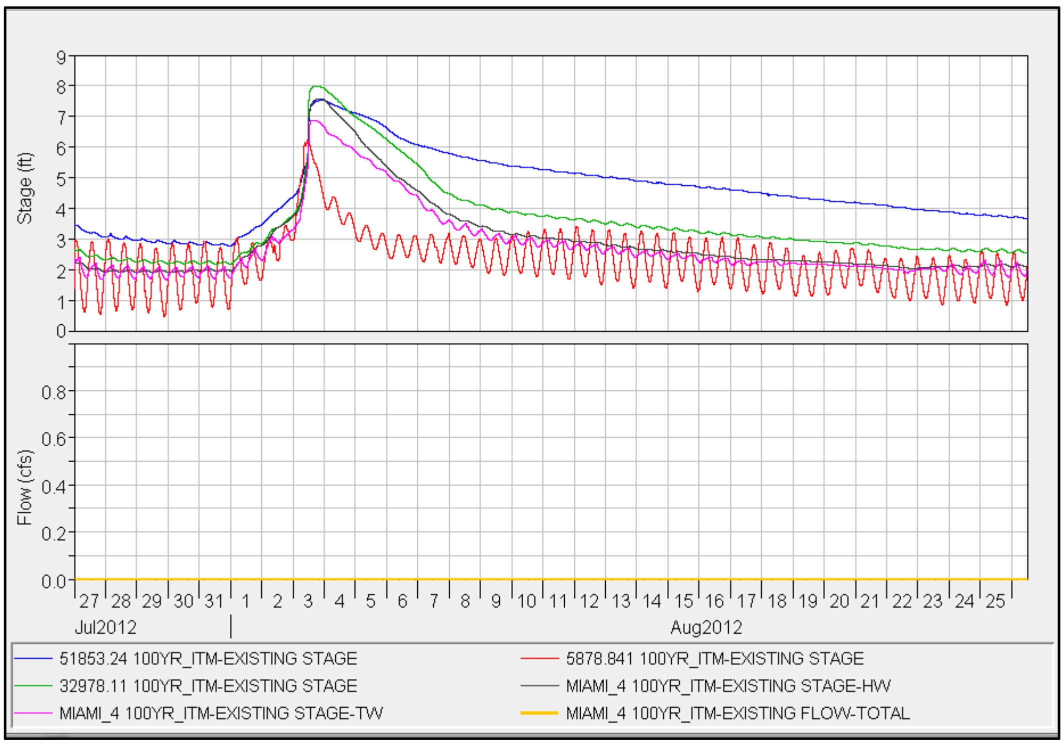

Figure G-19. 100-yr Pump Miami #4: top half shows HW&TW stage, C4CORAL and T5 stage, bottom half shows Miami #4 total flow.

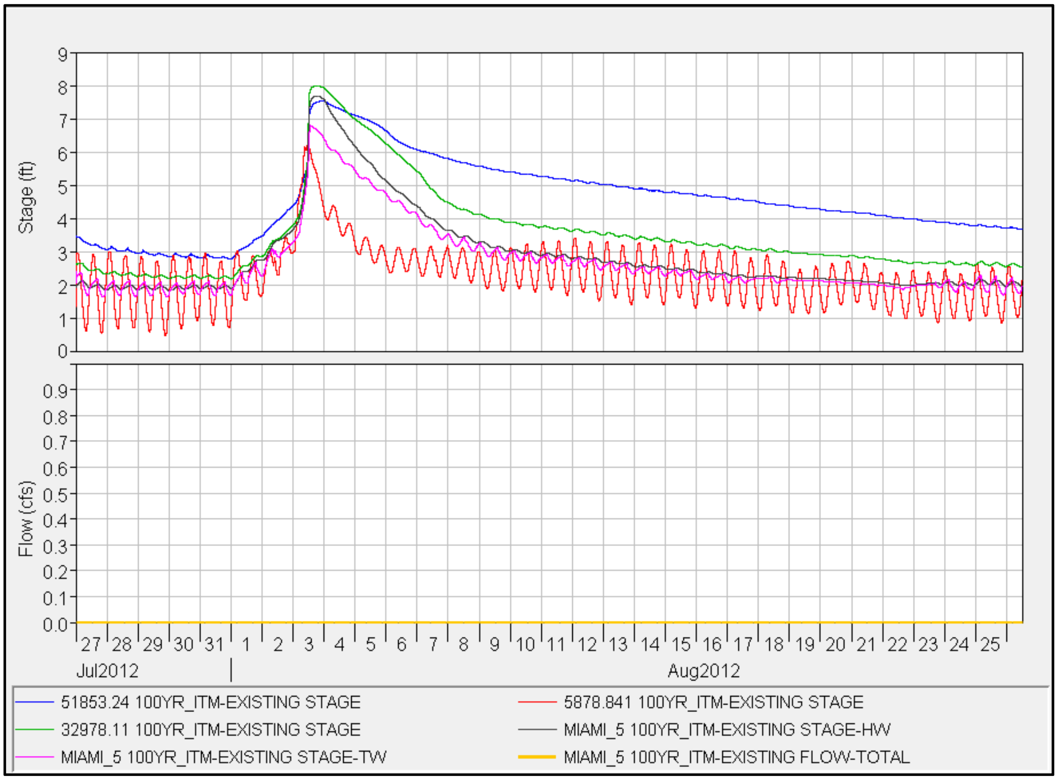

Figure G-20. 100-yr Pump Miami #5: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows Miami #5 total flow.

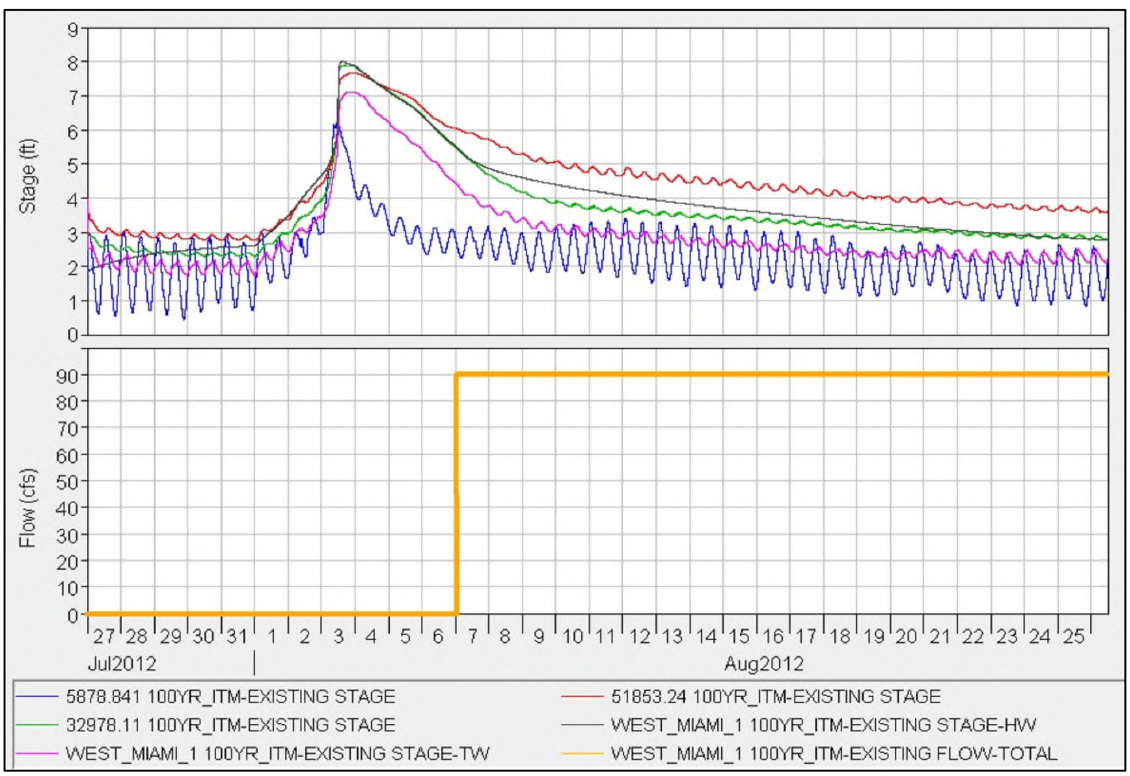

Figure G-21. 100-yr Pump West Miami #1: top half shows HW&TW stage, C4CORAL and T5 stage, bottom half shows West Miami #1 total flow.

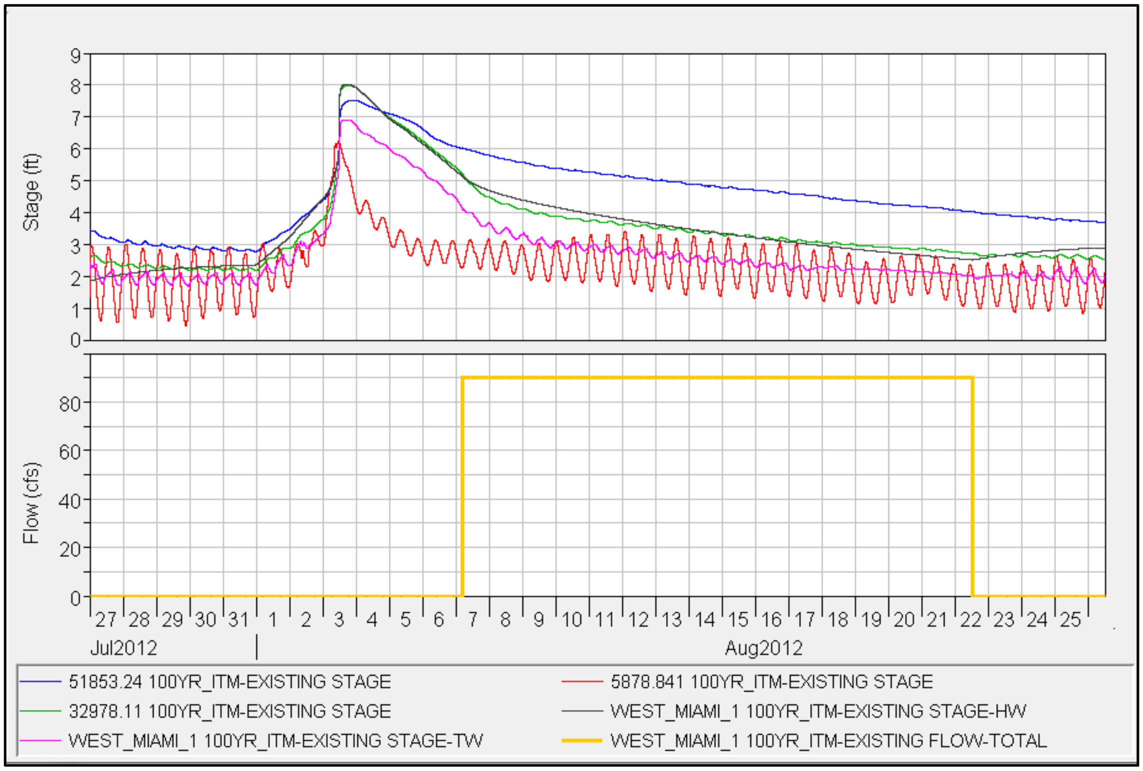

Figure G-22. 100-yr Pump West Miami #2: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows West Miami #2 total flow.

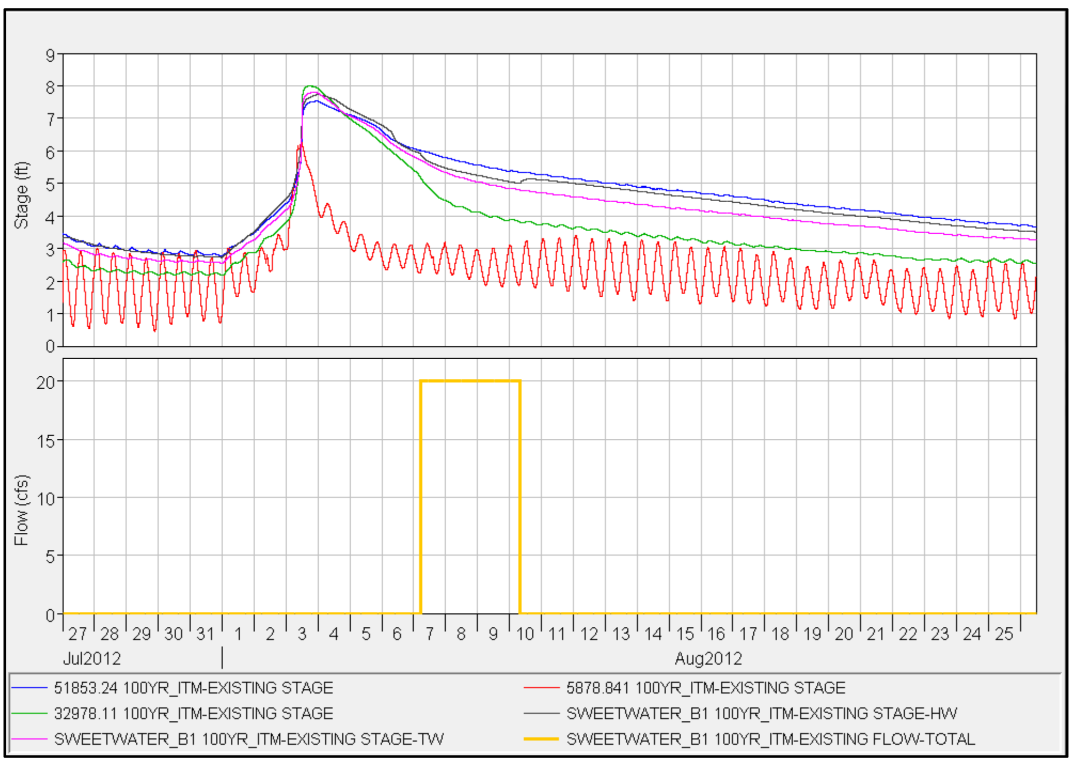

Figure G-23. 100-yr Pump Sweetwater B1: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows Sweetwater B1 total flow.

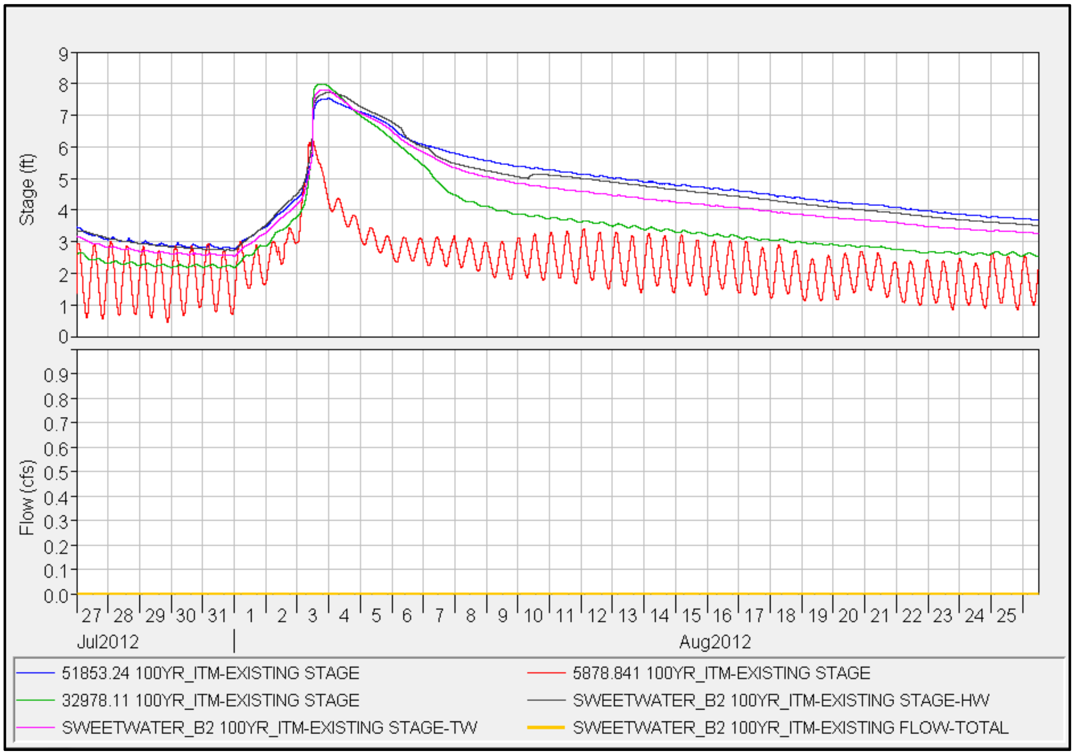

Figure G-24. 100-yr Pump Sweetwater B2: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows Sweetwater B2 total flow.

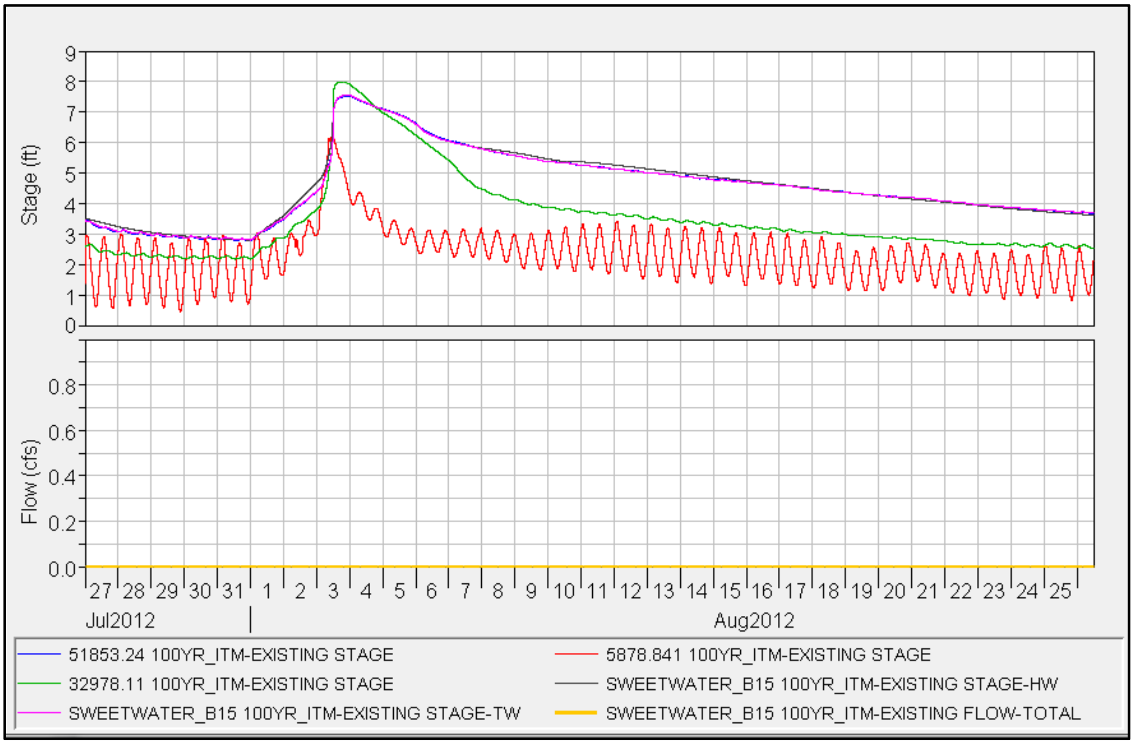

Figure G-25. 100-yr Pump Sweetwater B15: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows Sweetwater B15 total flow.

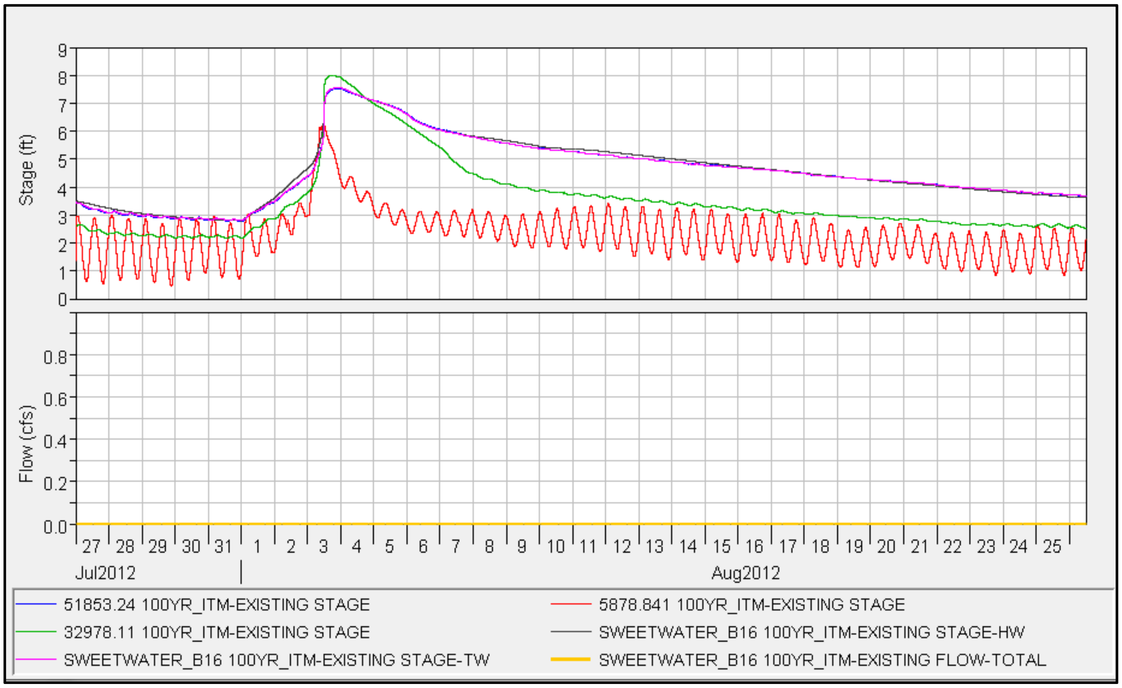

Figure G-26. 100-yr Pump Sweetwater B16: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows Sweetwater B16 total flow.

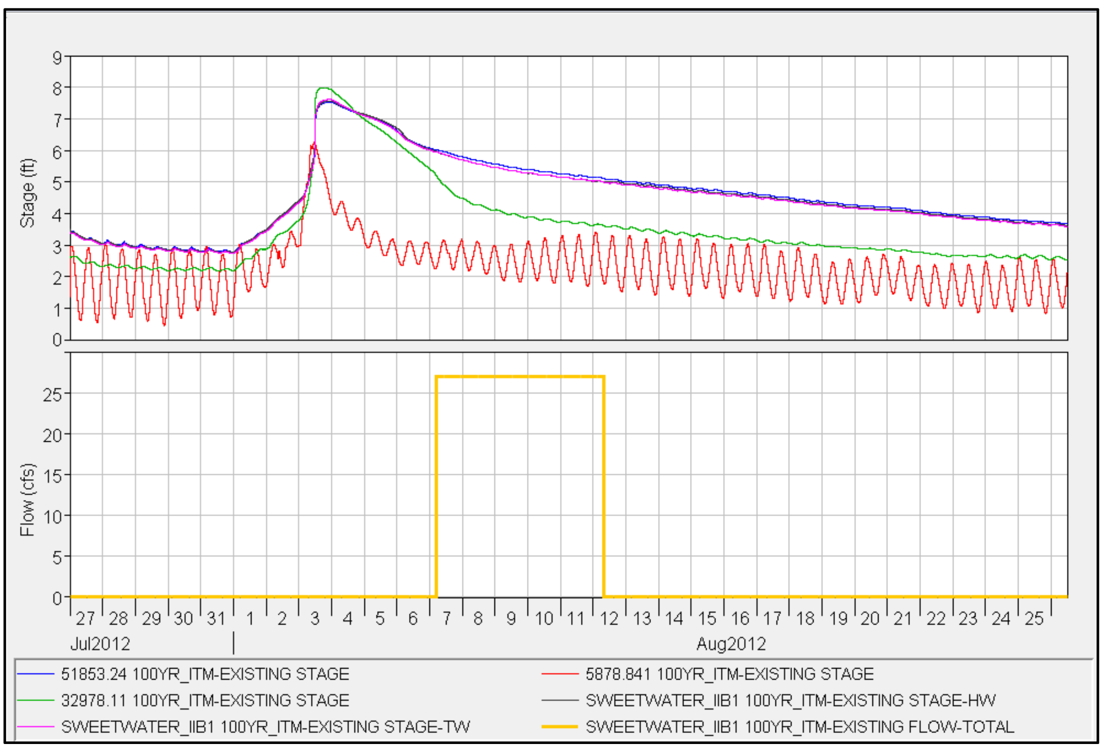

Figure G-27. 100-yr Pump Sweetwater IIB1: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows Sweetwater IIB1 total flow.

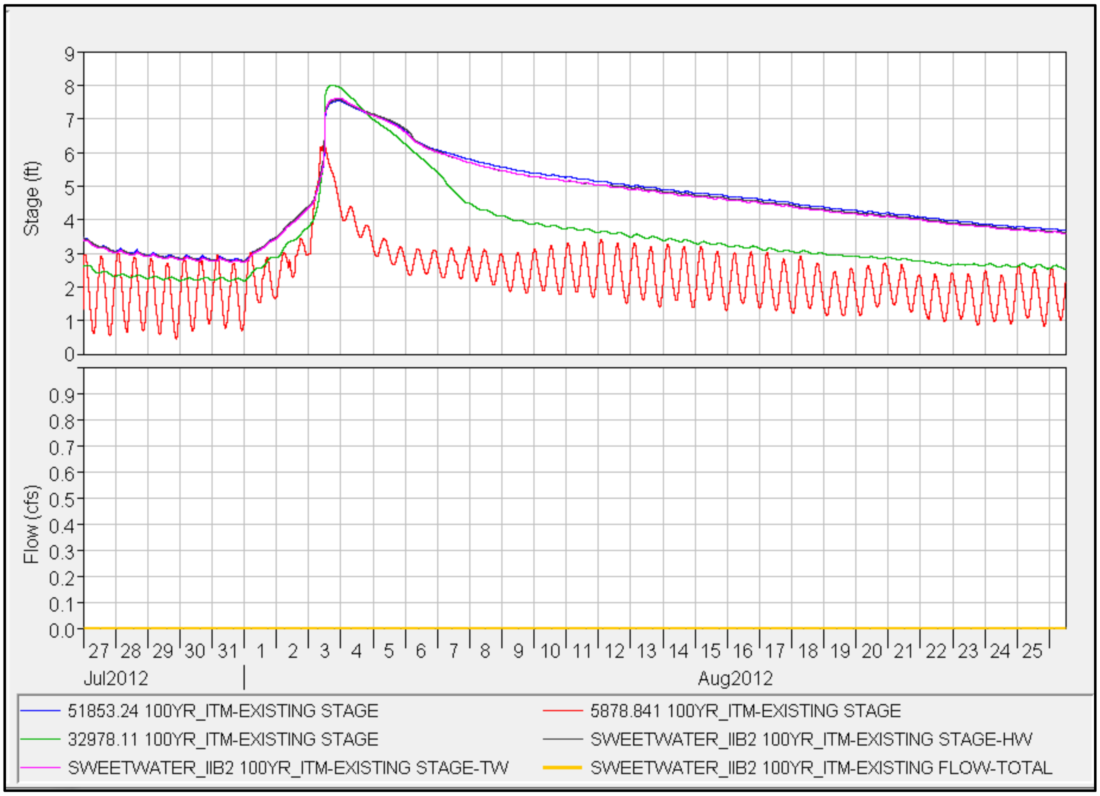

Figure G-28. 100-yr Pump Sweetwater IIB2: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows Sweetwater IIB2 total flow.

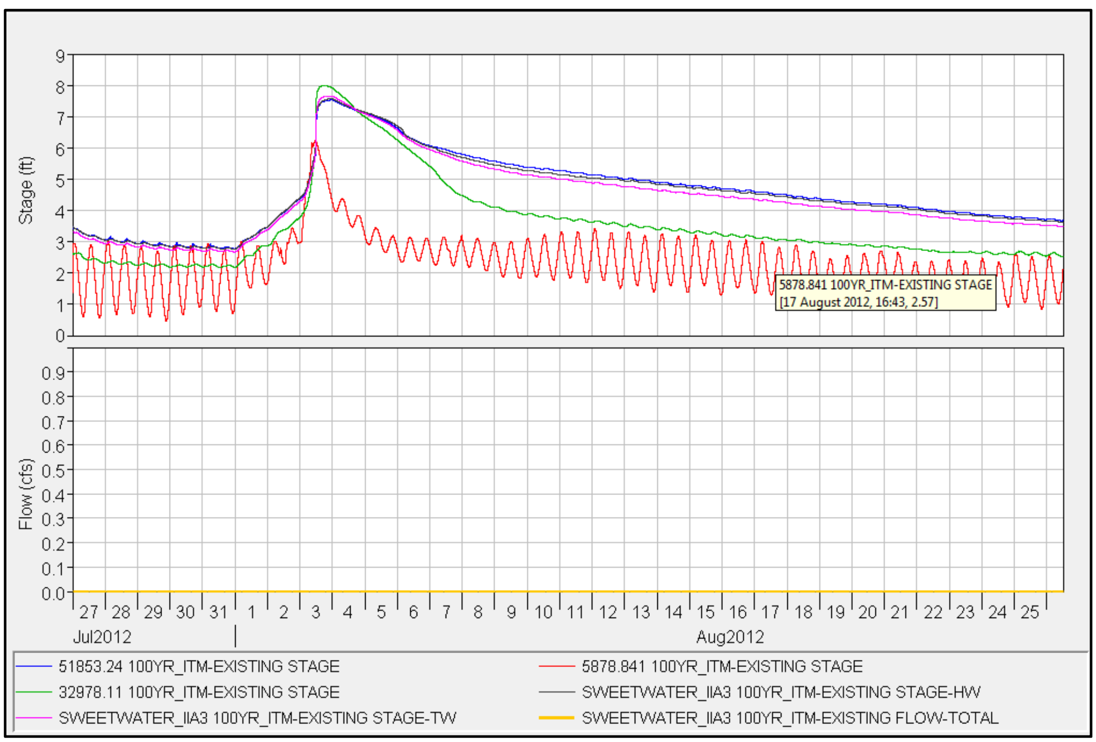

Figure G-29. 100-yr Pump Sweetwater IIA3: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows Sweetwater IIA3 total flow.

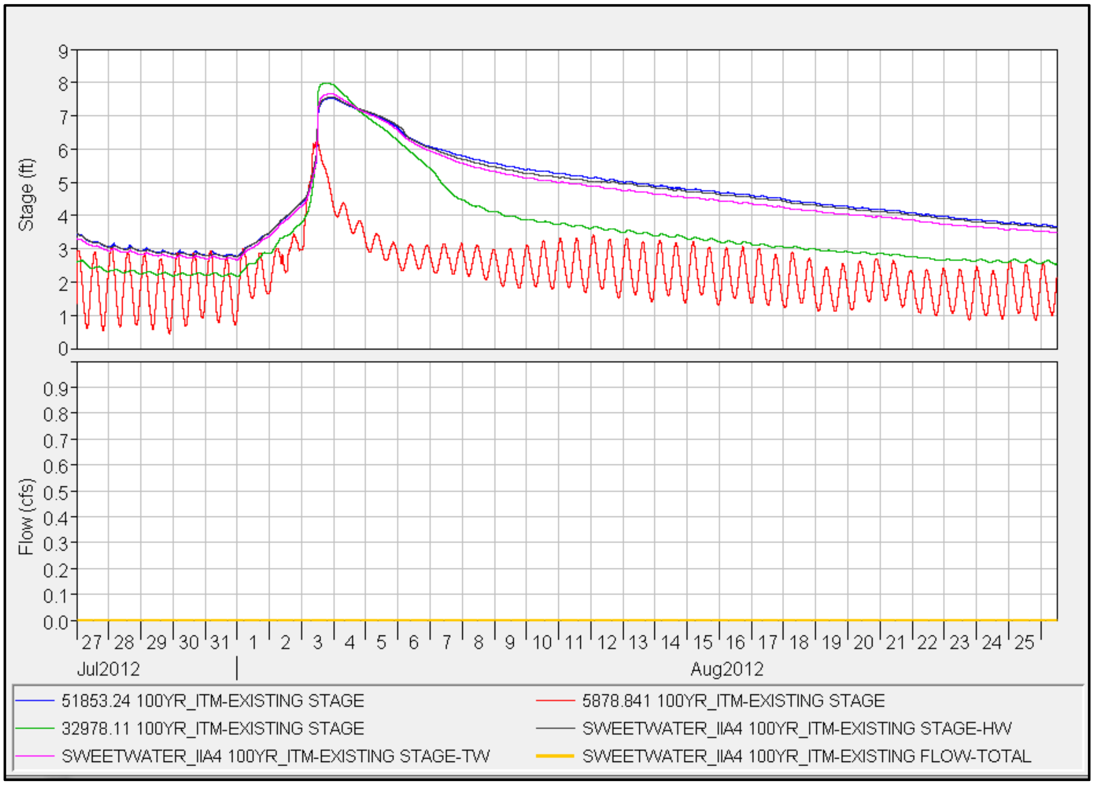

Figure G-30. 100-yr Pump Sweetwater IIA4: top half shows HW&TW stage, C4CORAL and T5 Stage, bottom half shows Sweetwater IIA4 total flow.

Future Conditions with Three Sea Level Rise Scenarios

Results from the model simulations under existing and future conditions are summarized in Tables G-6 to G-8 for the 100-year recurrence interval.

Table G-6. Summary of the peak flow and maximum stages at structures

Structure	Existing - 100yr			Future - 100yr								
				Low			Intermediate			High		
	Max. HW (ft)	Max. TW (ft)	Peak Flow (cfs)	Max. HW ft	Max. TW ft	Peak Flow cfs	Max. HW ft	Max. TW ft	Peak Flow cfs	Max. HW ft	Max. TW ft	Peak Flow cfs
S22	5.28	5.49	2242	5.56	5.83	2208	5.89	6.29	2137	6.77	7.75	1920
G93	7.43	6.37	707	7.57	6.71	581	7.66	7.17	547	8.46	8.63	649
S380	8.06	6.90	191	8.13	7.07	190	8.14	7.11	190	8.20	7.28	190
S25B	6.70	6.26	3236	6.98	6.6	3435	7.29	7.06	3425	8.01	8.49	3727
S25A	6.79	6.34	0	7.09	6.65	0	7.31	7.14	0	8.02	8.52	0
S25	6.26	6.26	252	6.61	6.61	233	7.04	7.04	211	8.52	8.76	217
G421	10.03	7.00	341	10.03	7.21	335	10.03	7.28	298	10.04	7.48	261

Table G-7. Maximum sub-basin water levels under future conditions during the 100-year storm.

Storage Area	Existing – 100yr (ft. NGVD)	Future – 100yr Low (ft. NGVD)	Future – 100yr Intermediate (ft. NGVD)	Future – 100yr High (ft. NGVD)
C2	7.88	7.97	8.01	8.15
C2-N-24	7.61	7.72	7.76	7.97
C3	8.03	8.17	8.22	8.48
C4-N-3	10.05	10.05	10.06	10.05
C4-N-4	10.03	10.03	10.03	10.04
C4_100B	7.47	7.61	7.67	8.11
C4_100C	7.44	7.59	7.66	8.1
C4_10A	8.06	8.13	8.14	8.2
C4_10B	6.9	7.07	7.11	7.27
C4_10C	6.8	6.97	7	7.17
C4_10D	7.13	7.27	7.31	7.49
C4_10E	7.13	7.27	7.32	7.49
C4_125A	5.99	6.43	7.14	8.03
C4_125B	7.12	7.28	7.38	7.95
C4_150A	6.1	6.42	6.61	7.45
C4_25	7	7.21	7.28	7.48
C4_40	7.56	7.67	7.7	7.94
C4_55	8.08	8.21	8.25	8.45
C4_60A	8.07	8.19	8.23	8.43
C4_65A	8	8.12	8.15	8.33
C4_65B	7.82	7.94	7.98	8.11
C4_70	7.65	7.76	7.81	8.14
C4_75A	7.48	7.62	7.68	8.1
C4_75B	7.48	7.61	7.68	8.11
C4_AG1	7.45	7.55	7.58	7.68
C4_AG10	9.27	9.45	9.52	9.84
C4_AG11	7.68	7.89	8	8.58
C4_AG12	7.46	7.6	7.67	8.1
C4_AG13	7.38	7.6	7.67	8.12
C4_AG14	6.07	6.28	6.41	6.93
C4_AG2	7.56	7.67	7.71	7.94
C4_AG3	7.56	7.67	7.71	7.94
C4_AG4	7.73	7.82	7.85	8
C4_AG5	7.47	7.61	7.68	8.11
C4_AG6	7.47	7.61	7.67	8.11
C4_AG7	7.97	8.18	8.25	8.58
C4_AG8	8.44	8.58	8.61	8.82
C4_AG9	7.58	7.73	7.8	8.25
C5	7.16	7.33	7.37	7.56

Table G-8. Summary of the peak flow at pump stations

Storage Area	Existing – 100yr (cfs)	Future – 100yr		
		Low (cfs)	Intermediate (cfs)	High (cfs)
G420_Pump	669	669	669	669
G422_Pump	623	623	623	623
Belen_PS1	100	100	100	100
Belen_PS2	100	100	100	100
S25B_FP	630	630	630	630
Miami_1	0	0	0	0
Miami_2	0	0	0	0
Miami_3	0	0	0	0
Miami_4	0	0	0	0
Miami_5	0	0	0	0
Sweetwater_B1	20	20	20	20
Sweetwater_B2	0	0	0	0
Sweetwater_B15	0	0	0	0
Sweetwater_B16	0	0	0	0
Sweetwater_IIA3	0	0	0	0
Sweetwater_IIA4	0	0	0	0
Sweetwater_IIB1	27	27	27	27
Sweetwater_IIB2	0	0	0	27
West_Miami_1	90	90	90	90
West_Miami_2	100	100	100	100

MODEL LIMITATIONS

Due to the simplifying assumptions inherent to the model setup, the shortcomings associated with the model calibration, and the numerical difficulties encountered in obtaining accurate model solutions, the stages and flows generated by the model will have limitations that should be taken into account when interpreting and comparing model results. First, it should be recalled that the model conceptualization discussed earlier (and in Appendix E) represents a substantial simplification of a highly complex system. This alone can introduce structural errors into the model results (Doherty and Welter, 2010). Second, it should be recalled that there was a substantial lack of data available for calibrating the model to basin water levels. This can exasperate the uncertainties of the predicted stages and associated levels of service. Third, computed discharges through the coastal structures often deviated from the corresponding discharges computed with the structure flow rating equations. The coastal structure flows were not computed in the model directly with the SFWMD flow rating equations since this caused the model simulations to become less stable. The discharges were computed using the spillway flow equations that are intrinsic to HECRAS. These flow equations differ from the SFWMD flow rating equations, especially under submerged flow conditions. Figure G-31 illustrates these differences at S-25B over a selected 72-hour time window (1300 August 3 – 1300 August 6) containing the peak stages and flow during the 25-year storm event with a SLR2 tidal boundary condition.

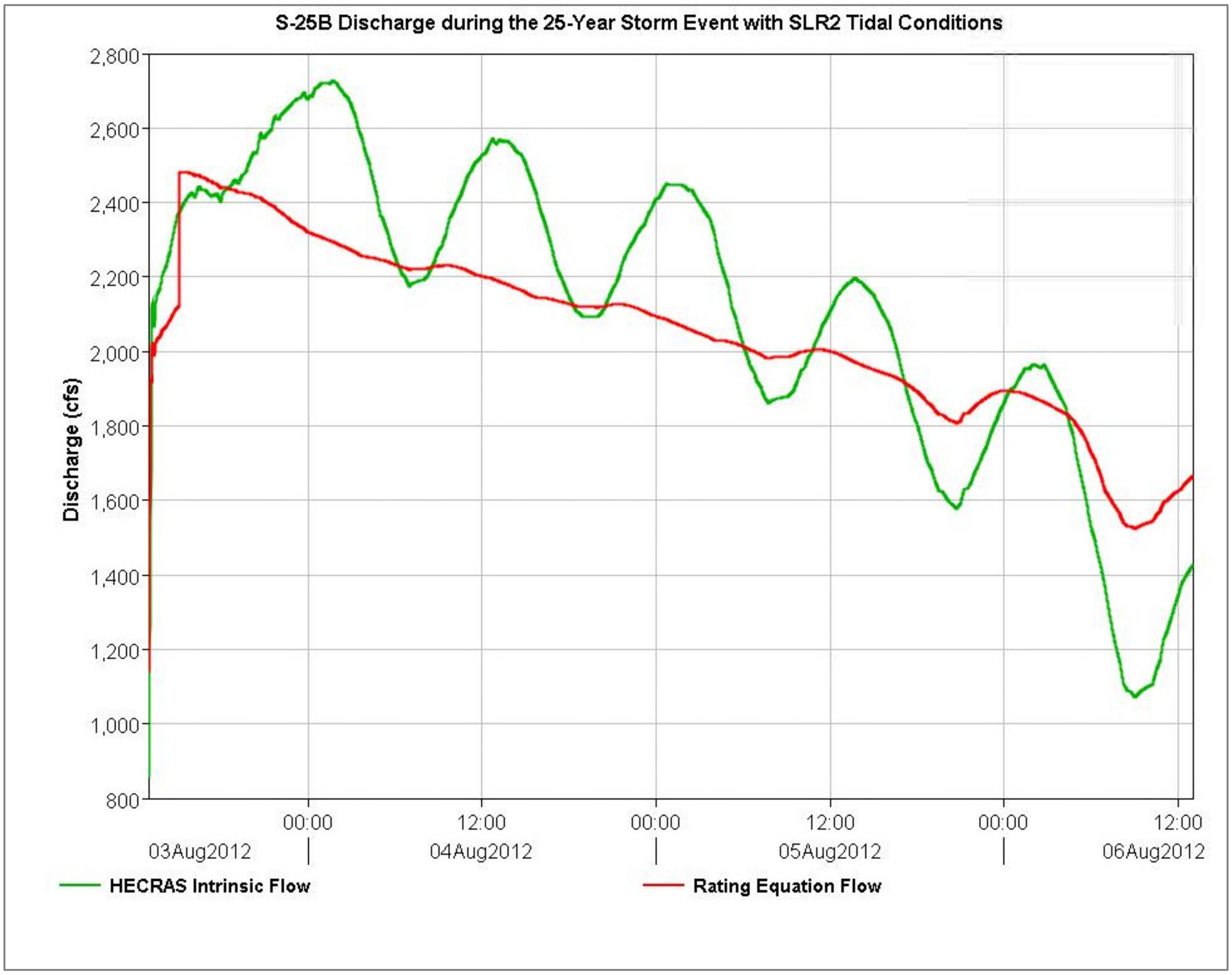

Figure G-31. Simulated and rated S-25B flows during the 25-year storm event with SLR2 tidal conditions

Deviations between simulated and computed flows vary with tidal stage and discharge rate. This is illustrated further in Figures 32 through 34, where the error is defined as the difference between computed and rated flows, averaged over the time window mentioned previously. It is expressed as a percentage of the average rated discharge rate for the same time window.

Figures 32 through 34 indicate that, for each of the coastal structures, errors in computed discharges appear to be correlated with both discharge rate and tidal stage. At S-25B, the model under predicts discharge rates for the 5 and 10-year events and over predicts flow for the 25 and 100-year events. These errors range from about -15% to about 25% for all tidal boundary conditions except SLR3, where the errors ranged from approximately -28% to approximately 5%. Furthermore, the biases generally decrease with increasing tidal stages.

Similar error trends are evident in Figure 33 for structure S-22, although the biases are less than the ones noted for S-25B. These errors range from about -20% to about 10% for all tidal boundary conditions except SLR3, where the errors ranged from approximately -33% to approximately -10%.

In contrast, Figure 34 shows that errors at G-93 tend to decrease with increasing flow rate for all tidal boundary conditions except SLR3, where errors remained nearly constant. Furthermore, the marked decrease in error that occurred at S-25B and S-22 when going from SLR2 to SLR3 is not evident at G-93.

Instead, such a decrease occurs between current conditions and SLR1. On the other hand, the general trend of decreasing error with increasing tidal stage that exists for S-25B and S-22 appears to exist for G-93 as well. Overall, the errors in computed flows through G-93 ranged from about -13% to almost 34%.

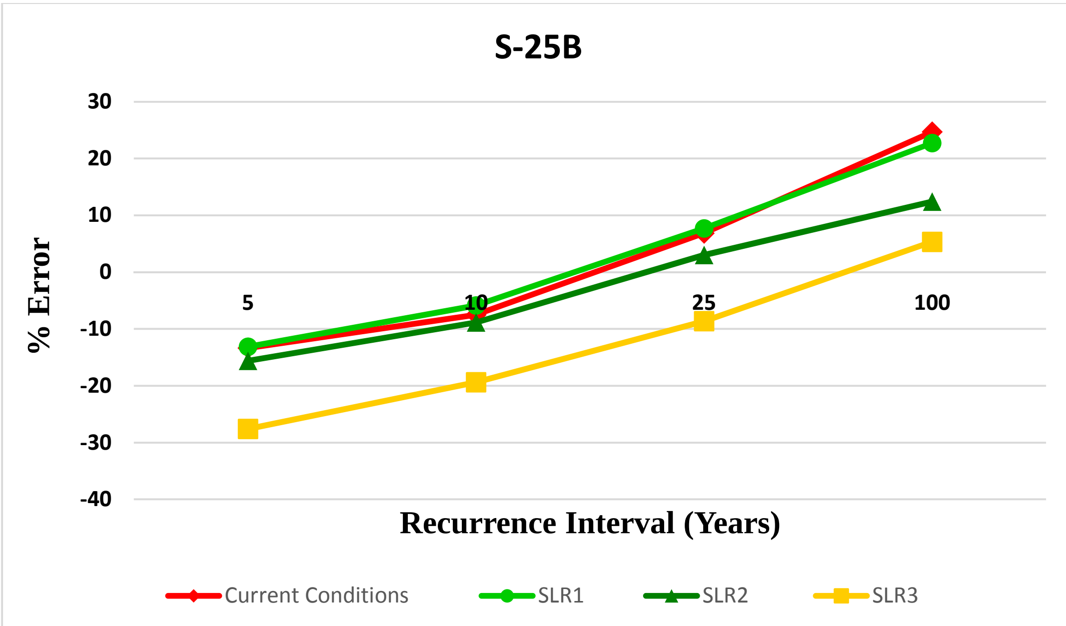

Figure G-32. Deviations between computed and rated flows at S-25B

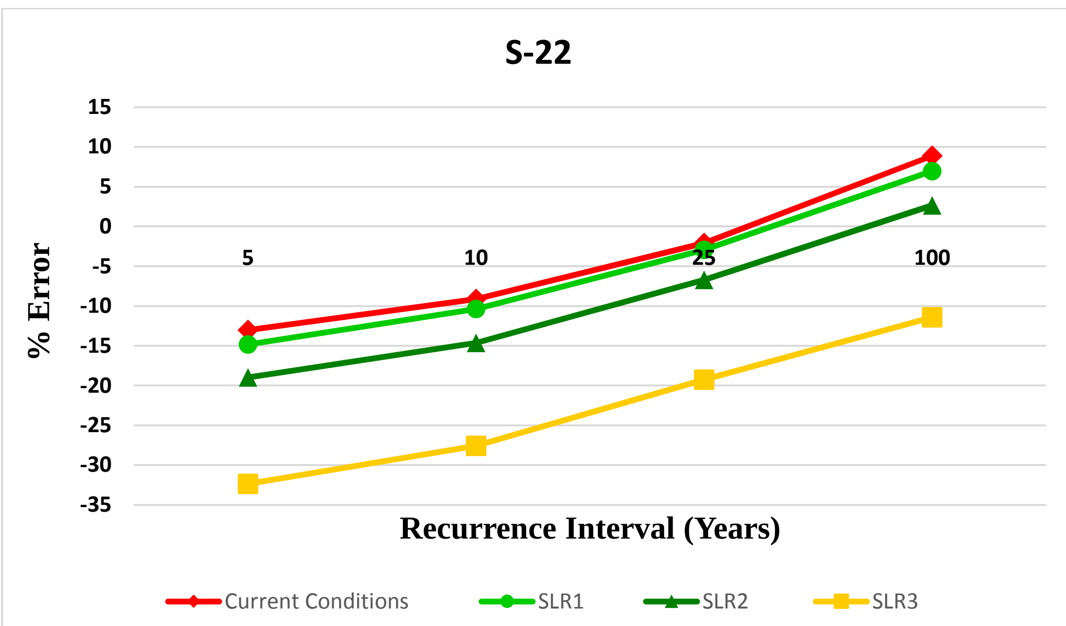

Figure G-33. Deviations between computed and rated flows at S-22

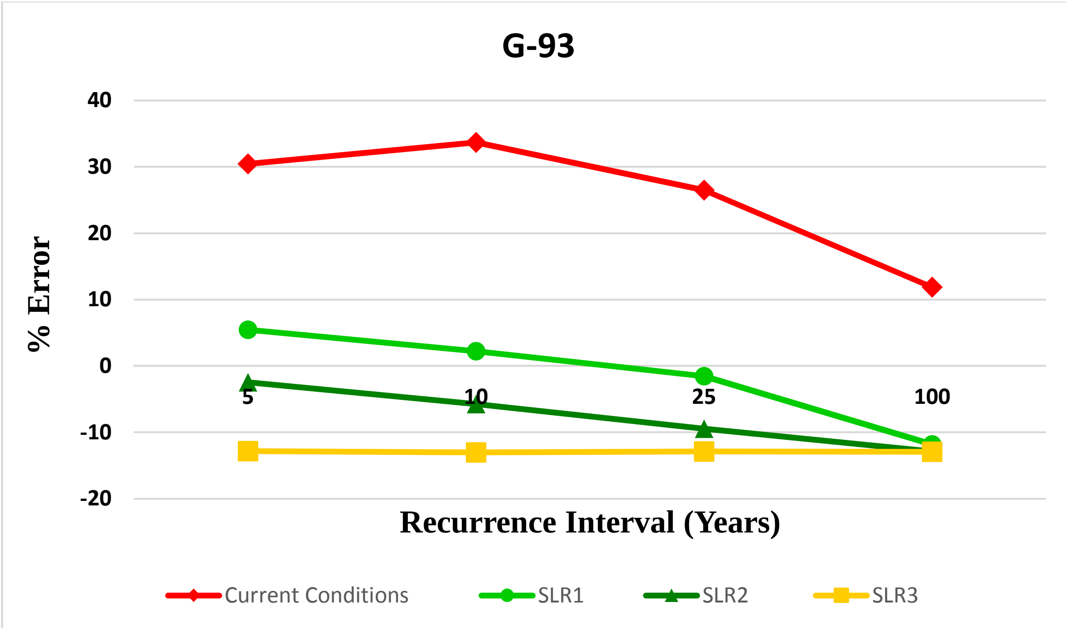

Figure G-34. Deviations between computed and rated flows at G-93

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Flood Protection Level of Service (LOS) Analysis for the C-4 Watershed

Appendix H: Details of Level of Service Analysis for C-4 Watershed

South Florida Water Management District
Hydrology and Hydraulics Bureau
May 19, 2016

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Appendix H: Details of Level of Service Analysis for C-4 Watershed

This document was produced by the H&H Bureau as a project deliverable for project 100888 (Flood Protection Level of Service, within the Sea Level Rise project).

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\\ad.sfwmd.gov\dfsroot\data\hesm_nas\projects\basin_studies\5_modeling

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INTRODUCTION

This Appendix presents the model-based LOS analysis of the C-4 basin. LOS is evaluated with respect to five performance measures. These performance measures are defined in Appendix A. For convenience, they are summarized below. Additional background information on the performance measures is given in **Appendix A**.

LOS Metric #1: Maximum Stages in the Primary Canals

LOS Metric #1 is the peak stage profile in the primary canal system. This profile is developed for a range of design storms. The largest design storm that stays within the canal banks (with a prescribed freeboard) establishes the LOS of the primary canal system.

LOS Metric #2: Maximum Flow Capacity in the Primary Canal Network

LOS Metric #2 shows the maximum discharge capacity throughout the primary canal network. Discharge is shown as aerially weighted flow (CSM or cubic feet per second per square mile). Tidal effects are eliminated by using a 12-hour moving average of flow.

LOS Metric #3: Structure Performance – Effects of Sea Level Rise

Metric #3 shows the effective capacity of the tidal structures and is comparable to the static, design condition assumed in the original structural design. Metric #3 compares structure flow over a range of storm surge events and a range of sea level rise scenarios. Such comparisons are used to identify tidal structures in need of eventual modification and help prioritize the need for redesign relative to other structures.

LOS Metric #4: Peak Storm Runoff - Effects of Sea Level Rise

Metric #4 shows the maximum conveyance capacity of a watershed at the tidal structure for a range of design storms. It shows the maximum conveyance (moving 12-hour average) for a specific design storm and a specific tidal boundary condition. This metric examines the relative sensitivity of the system to sea-level rise and storm surge.

LOS Metric #5: Frequency of Flooding - Stage-Based LOS for Sub-Watersheds

Metric #5 is a table that shows the duration of flooding in a developed sub-watershed; that is, the amount of time that stages in each sub-watershed exceed defined LOS targets. LOS targets include both a stage target and a frequency at which water levels exceed that stage. This performance measure is used to compare local expectations of flood protection with regional system performance. These performance measures are evaluated in the sections that follow.

LOS Metric # 6. Duration of Flooding in Primary Canal System

Metric # 6 represents the duration of flooding that occurs in the primary canal system, based on the stages above the normal operating levels of the canal as measured at one or more key gages.

METRIC #1: MAXIMUM STAGES IN THE PRIMARY CANALS

Four sets of maximum water surface profiles were computed for the C-4 canal. Each set of water surface profiles contains the maximum water surface profiles resulting from the 5, 10, 25 and 100-year storm events. One set reflects current conditions while the remaining sets pertain to future conditions with the three different seal level rise projections. The low level of future sea level rise (SLR1) is 0.45 feet higher than the 2005 sea level. The intermediate level (SLR2) 0.91 feet higher and the highest level (SLR3) is 2.37 feet above 2005 sea level (see T**able C-6** in **Appendix C**). Level of service comparisons between current and future conditions are given below.

LOS Under Current Sea Level Conditions

Figure H-1 shows the set of maximum water surface profiles for current conditions, represented by purple, green, red and blue lines, representing the combined effects of rainfall and storm surge for the 1-in-5, 1-in-10, 1-in 25, and 1-in-100 year events, respectively. Shown also as thinner orange lines in **Figure H-1** are inundation threshold elevations for the adjacent sub basins (situated both north and south of the canal). Local floodwalls have been constructed adjacent to C-4 along the approximately seven-mile long reach from its junction with the C-3 Canalto 132nd Ave. Local floodwall top elevations are represented by the heavy orange lines on **Figure H-1**, while the light and dark orange dashed lines are actual top of south and north bank elevations, respectively.

To facilitate interpretation of these plots, reference points were established to represent water conditions in different segments of the canal. Water levels predicted by the model to occur within the canal at the six reference points, for each design storm, are summarized in **Table H-1**.

Table H-0-1. Water levels in feet (NGVD) predicted by the model to occur at reference site locations under current conditions for various design storm events.

Design Storm Return Period (yrs)	①S-25B Structure (6000)*		②City of Miami Pump Mia1 (22286) (ft NGVD)	③East of C3 Near 72 Ave (25397) (ft NGVD)	④Palmetto Exwy (29608) (ft NGVD)	⑤137 Ave (61349) (ft NGVD)	⑥W of S-380 (75412) (ft NGVD)
	Tailwater (ft NGVD)	Headwater (ft NGVD)					
5	4.2	4.6	4.9	5.2	5.6	5.8	6.4
10	4.6	5.1	5.5	5.8	6.2	6.0	6.6
25	5.2	6.0	6.3	6.6	7.1	6.4	7.6
100	6.3	6.7	7.2	7.4	8.0	7.1	8.1

*numbers in parenthesis refer to distance (feet) west (upstream) of S-25B structure; reference site numbers in circles refer to sites designated on **Figure H-1**).

Structure and Eastern Basin. S-25 B maximum tailwater elevations range from 4.2 feet for a 1-in-5 year storm event to 6.3 feet for a 1-in-100 year event, resulting in a corresponding head water level range of 4.6 feet to 6.7 feet. Near Pump Mia1, (site ②), water levels increase from 4.9 feet for a 1-in-5 year storm to 7.2 for a 1-in-100 year storm event. Further west in the eastern basin at 72nd Ave. (site ③), water levels range from 5.2 feet for a 1-in-5 year storm event to 7.4 feet for a 1-in-100 year storm.

Western Basin. The highest water levels in the canal typically occur near the Palmetto Expressway at site ④, since areas to the east drain to tide and areas to the west drain toward the impoundment. Under current conditions, water levels range from 5.6 feet during a 1-in-5 year event to 8.0 feet during a 1-in 100 year storm event. Further west at 137th Ave, site ⑤near the reservoir, water levels range from 5.8 feet during a 1-in- 5 year storm to 7.1 feet during a 1-in 100 year storm. Water levels in the wetland and rock mining areas west of the S-380 divide structure (site ⑥) range from 6.4 feet in a 1-in-5 year storm event to 8.1 feet during a 1-in-100 year storm.

Land Elevations Adjacent to the Canals. Modeling results indicate that water levels in the canals are essentially the same as water levels in watersheds adjacent to the canals. The thin orange lines on **Figure H-1** represent inundation thresholds within the watersheds rather than actual land elevations Water levels in the canal are generally at or below inundation thresholds in the adjacent basins for the 1-in-5 and 1-in-10 year storm events. In contrast, canal stages are generally above the inundation thresholds during the 1-in-25 and 1-in-100 year storm events, which may lead to localized flooding in low-lying areas. Flooding within a particular sub-watershed depends on both the canal bank elevation and the inundation threshold elevation of the adjacent lands.

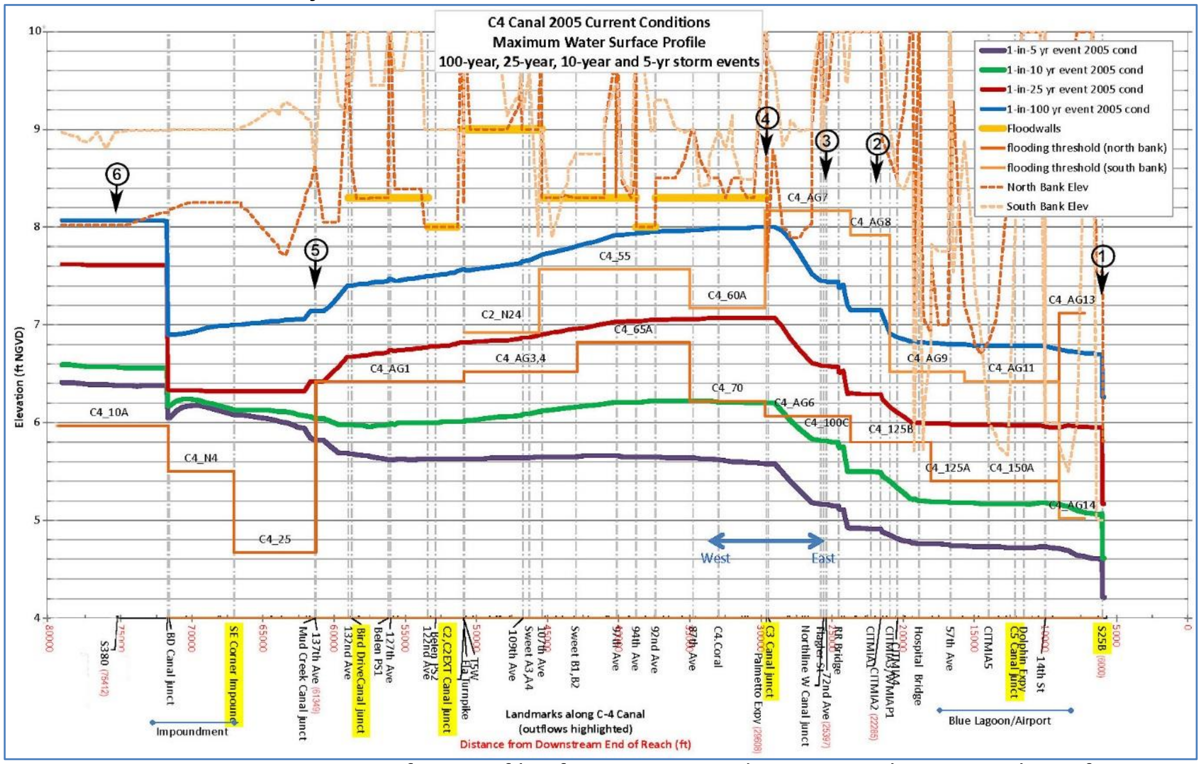

Figure H-1. Maximum C-4 water surface profiles for current conditions. Numbers in circles refer to locations described in the text and **Table H-1**.

Level of Service for Flood Protection. Under current conditions, the C-4 channel provides a 1-in-10 year level of service for most of the sub basins located north of the channel. Exceptions to this include C4_10A, C4_N4, C4_25 and part of C4_AG6 (see **Figure H1-1** in **Attachment 1** to this appendix for locations). C4_10A includes the Pennsuco wetlands, an area that is expected to periodically flood. Similarly, C4_N4 is the lower detention basin that is designed to impound water. C4_25 contains a number of quarries; so ponded surface water usually exists. C4_AG13 appears to be well protected against the 100-yr storm under current conditions. North of the canal, C-4 provides a 1-in 25 year level of service for all adjacent sub basins under current sea level rise conditions, except C4_AG14, which has a level of service that is slightly below 1-in-10 years.

LOS Under Future Sea Level Rise Conditions

Sea Level Rise 1

Figure H-2 contains the set of maximum water surface profiles for SLR1 conditions, representing an increase of 0.34 ft above current sea level. Also shown in **Figure H-2** for comparative purposes are the maximum water surface profiles associated with current sea level conditions.

Structure and Eastern Basins.

At site ① (see **Figure H-1**) S-25B tailwater levels increase by about 0.3 feet between current and SLR1 conditions (Table H-2) resulting in and 0.3 to 0.4 foot increase relative to current conditions in headwater levels 1-in-5 year storm conditions. A 0.6 foot increase in headwater level occurs during a 1-in-10 year event, and an 0.3 foot increase relative to current conditions occurs during 1-in-25 and 1-in-100 year storm events. Similar changes occur at intermediate locations in the eastern basin. For the 1-in-100-year storm event, water levels west of site ② are only about 0.1 foot higher than current conditions.

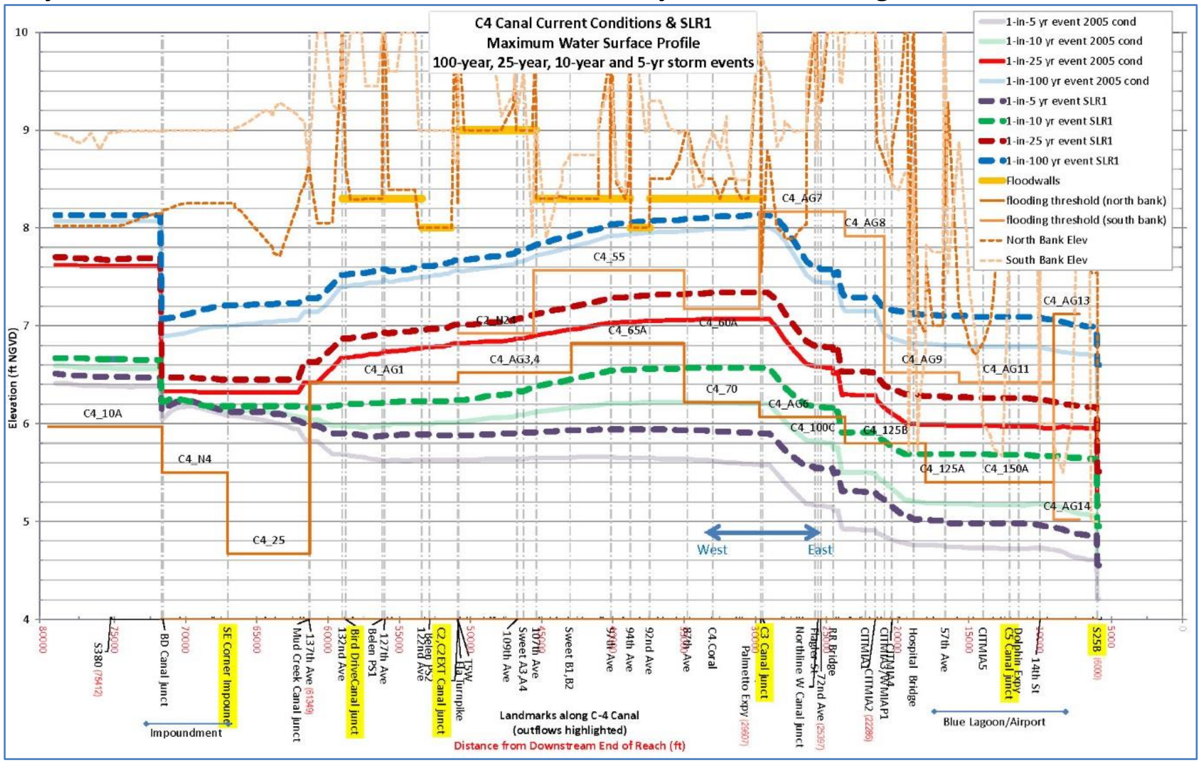

Figure H-2. Maximum C-4 water surface profiles for the lowest sea level of future rise (SLR1) conditions and various design storm events.

Western Basins. At the Palmetto Expressway, site ④ on **Figure H-1**, the effects of SLR1 result in a 0.3-0.4 foot increase in flood elevation (**Table H-2**) relative to the current condition for the 1-in-5 and 1-in 10 storm events, 0.2 feet for the 1-in-25 year storm event, and 0.1 feet for the 1-in-100 year storm event. At 137th Ave (site ⑤), water levels increase by 0.2 feet under SLR1 conditions. Water levels west of S-380 are not significantly affected.

Table H-0-2. Water levels in feet (NGVD) predicted by the model to occur at reference site locations (numbers in circles) shown on **Figure H-2** under the lowest projection of future sea level rise (SLR1) conditions for various design storm events.

Design Storm Return Period (yrs)	①S-25B Structure (6000		②City of Miami Pump Mia1 (22286) (ft NGVD)	③East of C3 Near 72 Ave (25397) (ft NGVD)	④Palmetto Exwy (29608) (ft NGVD)	⑤137 Ave (61348) (ft NGVD)	⑥W of S-380 (75000) (ft NGVD)
	Tailwater (ft NGVD)	Headwater (ft NGVD)					
5	4.6	4.9	5.3	5.6	5.9	6.0	6.5
10	5.0	5.6	5.9	6.2	6.6	6.2	6.7
25	5.5	6.2	6.5	6.8	7.3	6.6	7.7
100	6.6	7.0	7.3	7.6	8.1	7.3	8.1

***numbers in parenthesis refer to distance (feet) west (upstream) of S-25B structure**

C-4 Canal. With an increase in sea level of 0.34 foot (see Appendix C), water levels in the C-4 Canal exceed adjacent inundation thresholds in during a 1-in-10 year storm event in several sub-watersheds located north of the canal, and throughout most of the C-4 watershed during a 1-in-25 year storm event. Since levels in the canal are generally the same as water levels in the adjacent drainage basins, localized flooding may occur in low-lying areas.

Level of Service. The level of service for most of the sub basins north of the canal diminishes to 1-in-5 years with SLR1 conditions while, south of C-4, the level of service diminishes to 1-in-10 years for sub basins C2_N24 and C4_60A. Sub basins C4_55, C4_AG7, C4_AG8, C4_AG9 and C4_AG11 still maintain a 1-in-25 year level of service. On the other hand, sub basin C4_AG13 (located north of C-4) appears to be protected against the 100-year storm event while C4_AG14 is only protected against the 1-in-5 year event.

Sea Level Rise 2

Figure H-3 contains the set of maximum water surface profiles for SLR2 conditions. Also shown for comparative purposes are the maximum water surface profiles associated with current sea level conditions.

Structure and Eastern Basin. S-25B (site ① on **Figure H-1**) maximum tailwater elevations range from 5.0 feet for a 1-in-5 year storm event to 7.1 feet during a 1-in-100 year event, resulting in corresponding increases of upstream (headwater) levels of from 5.4 feet to 7.3 feet (**Table H-3**).

Table H-0-3. Water levels in feet (NGVD) predicted by the model (**Figure H-3**) to occur at reference sites for the intermediate projection of future sea level rise (SLR2) during design storm events.

Design Storm Return Period (yrs)	①S-25B Structure (6000)*		②City of Miami Pump Mia1 (22286) (ft NGVD)	③East of C3 Near 72 Ave (25397) (ft NGVD)	④Palmetto Exwy (29608) (ft NGVD)	⑤137 Ave (61349) (ft NGVD)	⑥W of S-380 (75412) (ft NGVD)
	Tailwater (ft NGVD)	Headwater (ft NGVD)					
5	5.0	5.4	5.6	5.8	6.1	6.0	6.5
10	5.4	6.0	6.1	6.4	6.7	6.2	6.7
25	6.0	6.4	6.7	6.9	7.5	6.7	7.7
100	7.1	7.3	7.4	7.7	8.2	7.3	8.1

******numbers in parenthesis refer to distance (feet) west (upstream) of S-25B structure; reference site numbers in circles refer to sites designated on **Figure H-1**.

For the 1-in-100 year storm, the difference between headwater and tailwater stages is only 0.2 feet, so gravity discharge from the structure is compromised. Further west at the City of Miami pumps (site ②) and 72nd Ave (site ③), water levels range from 5.6 to 7.4 feet. The 1-in-5-year flood elevations for SLR2 are about 0.4 feet higher than the corresponding 1-in-5-year elevations for SLR1 conditions. The 100-year flood elevations for SLR2 are 0.1 feet higher than the elevations observed at these sites for the SLR1 scenario.

Western Basins. At the Palmetto Expressway (site ④), the effects of SLR2 result in higher flood elevations of 0.0 to 0.2 feet relative to the SLR1 storm events. Similarly at 137th Ave (site ⑤), water levels for SLR2 were only 0.1 to 0.2 feet above levels that occurred during the SLR1 scenario. West of S-380 (Site ⑥), water levels were the same under the SLR2 condition relative to SLR1.

C-4 Canal. With an increase in sea level on the order of 0.81 feet above current for the SLR2 scenario, water levels in the eastern portion of the canal increased from 0.4 to 0.9 feet relative to current conditions. The estimated stages for the 100-year storm exceed the adjacent sub-basin inundation thresholds throughout almost the entire C-4 watershed, although they do not exceed the floodwall elevations in the reach between C-3 Canal and Bird Drive Canal, except between 92nd and 94th Avenues.

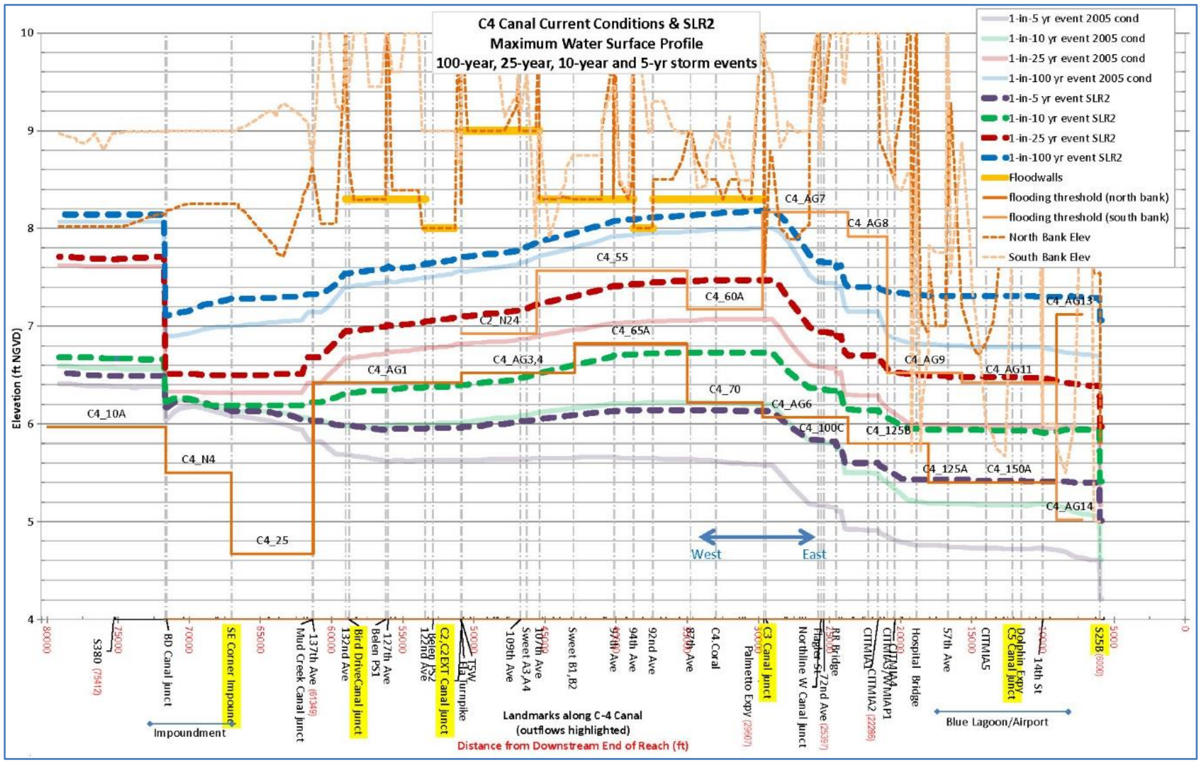

Figure H-3. Maximum C-4 water surface profiles for SLR2 conditions

Level of Service for Flood Control. With the higher sea levels and storm surges associated with the SLR2 scenario, C-4 may not be able to even provide a 1-in-5 year level of service for several sub basins located north of the canal. These include C4_AG6, C4_125A and C4_150A. In contrast, C4_65A will experience a 1-in-10 year level of service and C4_AG13 will no longer be protected against the 100-year storm. South of C-4, the canal should be able to maintain at least a 10-year level of service for all of the basins, with C4_55, C4_AG7, C4_AG8, and C4_AG9 experiencing a 1-in-25 year level of service. In contrast, the level of service for C4_AG14 is less than 1-in-five years.

Sea Level Rise 3

Figure H-4 contains the set of maximum water surface profiles for SLR3 conditions. Also shown in Figure H-4 for comparative purposes are the maximum water surface profiles associated with current sea level conditions.

Structure and Eastern Basin. Tailwater elevations at S-25B (site① on **Figure H-1**) range from 6.5 feet during a 1-in-5 year storm event to 8.5 feet (**Table H-4**) during a 1-in-100 year event under the high sea level scenario (SLR3). Headwater elevations range from 6.4 feet during a 1-in-5-year event to 8.0 feet during a 1-in-100 year event. Tailwater elevations exceed headwater elevations during all storm events, so there is pump discharge of 600 cfs but no gravity discharge from S-25B. Further west at the City of Miami pump (site ②), the canal stage during the 1-in-5 year storm is 6.4 feet, which is 0.1 foot higher than the water level that occurs under current conditions during a 1-in-25 year storm. Furthermore, the stage during a 1-in-25 year storm is 7.2 feet, which is equivalent to the level that occurs under current conditions during a 1-in-100 year storm event. During a 1-in-100 year event, the entire eastern portion of the basin is between 8.0 and 8.1 feet.

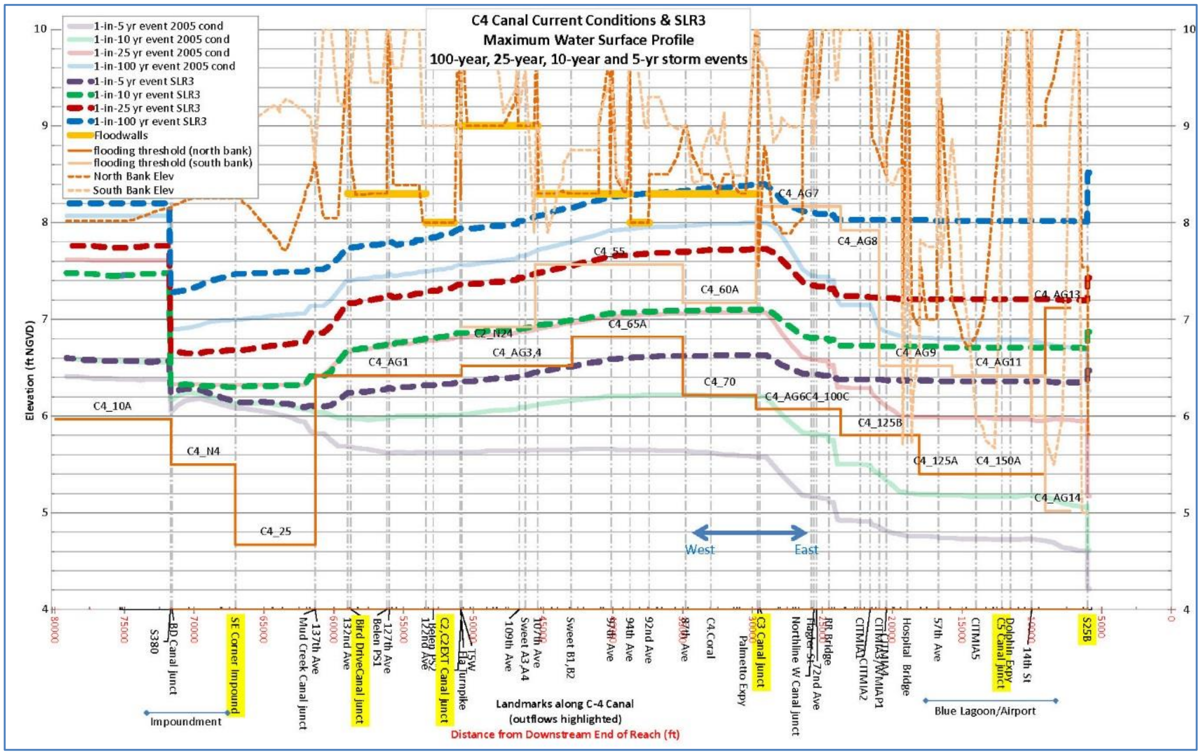

Figure H-4. Maximum C-4 water surface profiles for SLR3 conditions relative to current conditions

Table H-0-4. Water levels in feet (NGVD) predicted by the model to occur at reference site locations (numbers in circles) shown on **Figure H-4** under the highest projection of future sea level rise (SLR3) for various design storm events.

Design Storm Return Period (yrs)	①S-25B Structure (6000)*		②City of Miami Pump Mia1 (22286) (ft NGVD)	③East of C3 Near 72 Ave (25397) (ft NGVD)	④Palmetto Exwy (29608) (ft NGVD)	⑤137 Ave (61349) (ft NGVD)	⑥W of S-380 (75412) (ft NGVD)
	Tailwater (ft NGVD)	Headwater (ft NGVD)					
5	6.5	6.4	6.4	6.5	6.6	6.1	6.6
10	6.9	6.7	6.7	6.8	7.1	6.4	7.5
25	7.4	7.2	7.2	7.3	7.7	6.8	7.8
100	8.5	8.0	8.0	8.1	8.4	7.5	8.2

numbers in parenthesis refer to distance (feet) west (upstream) of S-25B structure; reference site numbers in circles refer to sites designated on **Figure H-1.***Red text** **indicates tailwater > headwater elevation, no gravity discharge is occurring, only 600 cfs pumping**

Western Basins. At the Palmetto Expressway (site ④), with the SLR3 scenario, water levels range from 6.6 feet for the 1-in-5-year storm to 8.4 feet for the 1-in-100 year storm. The water level for the 1-in-10 year storm is equal to the water level experienced during a 1-in-25 year storm under current conditions. The water level for the 1-in-100 year storm is 0.4 feet higher than the water level that occurs during a 1-in-100 year storm under current conditions. At 137th Ave (site ⑤), water levels range from 6.1 feet for the 1-in-5 year storm to 7.5 feet for the 1-in-100 year storm event. The 1-in-5 year storm with SLR3 is similar to a 1-in-10 year storm under current conditions while a 1-in-10 year storm is equivalent to a 1-in-25 year storm. Water levels west of Palmetto Park expressway in the 1-in-25 year and 1-in-100 year storm events are 0.4 to 0.6 feet higher than water levels that occur in these storm events under current conditions. West of S-380, water levels ranged from 6.6 for the 1-in-5 year storm event to 8.2 feet for the 1-in-100 year storm event.

C-4 Canal. Water levels in C-4 Canal exceeded the inundation thresholds of the eastern sub-watersheds south of the canal during the 1-in-5 year storm event. During the 1-in-100 year storm event, water levels in the canal exceed inundation thresholds in the entire eastern portion of the basin except in the vicinity of 72nd Ave.

Level of Service for Flood Control. With this highest prediction of sea level rise, only sub basins C4_AG1, C4_AG3, C4_AG4 and C4_65A maintain a 1-in-5 year level of service north of C-4 while C4_AG13 exceeds a 1-in-10 year level of service. South of C-4, all of the adjacent sub basins appear to maintain at least a 1-in-5 year level of service, with C2_N24, C4_55, C4_60A ,maintaining a 1-in-10-year level of service and C4_AG7 and C4_AG8 maintaining a 1-in-125 year level of service. In contrast, the expected level of service for C4_AG14 is well below 1-in-5 years.

Discussion of Performance Metric 1

Based on the results presented in this section, the level of service that can be designated for the C4 canal itself appears to be a 1 in 10 year storm event. Additionally, it should be noted that since the model allows only momentary reverse flows through or around the coastal structures, the sea level rises primarily serve to restrict outflows. Consequently, it is inherently assumed that at each coastal structure a dike exists and it will prevent storm surges from moving into the C4 watershed. A cursory view of the aerial imagery suggests that the construction of such a barrier is realistic.

As discussed in Appendix G, USGS estimates of increases in wet season ground water levels due to sea level rise suggest that increases of approximately 0.1 foot to 0.85 foot in initial water table stage are possible. These increases in initial groundwater levels lead to similar increases in maximum canal stages.

According to **Figures H1 to H3**, the effects of SLR on maximum canal stages appear to be more pronounced for the smaller storms than for the larger ones. This implies that nuisance flooding will become more substantial and more frequent. For example, near the midpoint of the C-4 canal, SLR1 causes maximum C-4 stages to increase by more than 0.3 foot during the 5-year storm while corresponding increases during the 100-year storm are close to 0.1 foot. Moreover, increases during the 5-year storm due to SLR3 are close to 0.8 foot in the same area while corresponding increases during the 100-year storm are approximately 0.4 foot.

METRIC #2: MAXIMUM FLOW CAPACITY THROUGHOUT THE PRIMARY CANAL NETWORK

An allowable discharge has been defined for each the SFWMD Canals (SFWMD ERP Applicant's Handbook Volume II, Appendix A, 2014). The allowable discharge values are based on the design discharge capacity of the canal. In general, these were established as part of the original C&SF design. For the C4 canal, "the allowable discharge rate is based on the peak discharge rate after development not exceeding the rate that existed prior to development. The design storm is the 25 year event…" (page 20, Appendix A, ERP Applicant's Handbook Volume II).

PM #2 establishes the effective discharge capacity of the canal under current and future sea level rise conditions. **Figure H-5** illustrates how discharge capacity is generated from computed instantaneous flows. The computed flows are obtained from the model simulations reflecting current conditions, where the instantaneous flow of the C4 canal is taken to be the sum of the instantaneous flow through S-25B plus the sum of the instantaneous flows entering the C2, C3, C5 and 132nd Ave canals. Since instantaneous flow oscillates with the tide, a 12-hour average is used to filter out tidal effects. The maximum value of the 12-hour average flow for the design storm (as determined by PM#1) is considered to be the discharge capacity and can be compared to the permitted discharge.

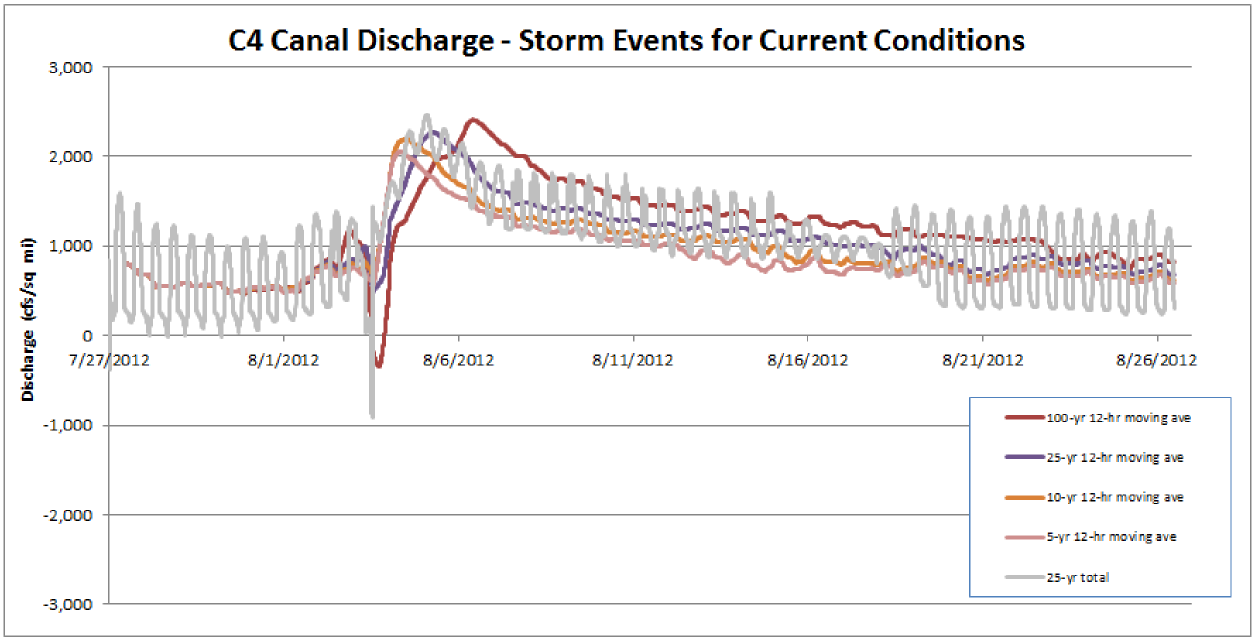

Figure H-5. C-4 canal discharge for the 5, 10 25 and 100-year storm events under current sea level conditions.

Figure H-5 shows that, prior to the storm, the C-4 flow as defined above is about 600 cfs. This represents base flow, and is reflective of the initial conditions used for the model simulations. The derivation of these initial conditions is discussed in Appendix G.

Figure H-5 also reveals that, for all design storms, flow increases slowly during days 1 and 2 (August 1 and 2) of the 3-day design event, when rainfall intensity is low. Near the middle of day 3, peak rainfall and peak storm surge occur simultaneously. Infiltrated storm water fills the canal while the storm surge suppresses outflow. This results in some negative discharge values, since the gates cannot close immediately when a negative water level condition occurs at one or more of the coastal structures. As the storm surge recedes, the discharge increases to a maximum of 2200 cfs to 2400 cfs, depending on the design storm. It subsequently recedes, returning to near base flow conditions by the end of the simulation. Additionally, it is evident that discharge capacity increases slightly with larger design storms, from about 2200 cfs for the 5-year event to about 2400 for the 100-year event. Furthermore, base flow resulting from storm water is delayed while durations of high flows are increased for larger storms.

Table H-5 provides the discharge capacity for the four design storms and the four sea level scenarios.

Following convention, the maximum flow is normalized by dividing the watershed outflow by the watershed area; the units are CSM, or cfs per square mile of watershed. Table H-1 indicates that, for current sea level, SLR1 and SLR2, discharge capacity remains nearly constant, ranging from 26 to 30 CSM. Under these conditions, sea level rise, rainfall intensity and storm surge appear to have a negligible effect on discharge capacity.

In contrast, the higher tidal conditions associated with SLR3 result in a significant decrease in discharge capacity. Under these conditions, it ranges from to 15 to 21 CMS (**Table H-5**). This reduction

Table H-0-1. C4 canal discharge capacity (CSM)*

Tidal Condition	Return Period of the Design Storm			
	5-yr	10-yr	25-yr	100-yr
Current	27	28	29	30
SLR1	27	28	29	29
SLR2	26	26	28	28
SLR3	15	17	20	21

*Discharge capacity is defined as the peak canal discharge from a design storm. Flow is averaged over the 12-hour tidal cycle to filter out tidal effects and normalized over the watershed area.

is due to a decrease in coastal structure capacity. That is, the structure (and not the canal) limits the discharges under these conditions.

METRIC #3: EFFECTS OF SEA LEVEL RISE ON STRUCTURE PERFORMANCE

Characteristics of Model Simulations

The metric is developed from simulations of structure flow where the upstream stage is fixed at the design headwater stage and downstream stage oscillates with the tide. Four days into the simulation, a storm surge of known intensity (as indicated by its return period) raises tail-water stage and suppresses flow. Results of each simulation are further analyzed to determine the minimum 12-hour flow through the structure. For this analysis, five sea-level scenarios were modeled and, for each scenario, a range of six design storm surges were examined. The minimum flow values from these 30 model runs were then compared on a single plot.

A simple hydraulic model is used to carry out the simulations. It is comprised of the structure, a short length of canal upstream and a short length of canal downstream of the structure. The upstream boundary condition is a fixed head while the downstream boundary condition uses the same suite of time-varying tidal boundary conditions as those used for other performance measures. Furthermore, all structure operations are modeled in accordance with the C-4 Basin Operating Plan (SFWMD, 2011). This metric is independent of rainfall and basin runoff.

The tidal boundary conditions are depicted in both **Figure H-6** and **Figure H-7. Figure H-6** shows an example of the suite of tidal boundaries (2-yr to 100-yr return period) associated with the storm surges for current (i.e. 2005) sea level conditions. **Figure H-7** shows the tidal boundary water levels for five different sea level rise scenarios, all reflecting a 5-yr storm surge. These sea level rise scenarios pertain to 1963 (the year of the USACE design report), 2005 (current conditions), and three estimates of future sea level conditions: SLR1, SLR2, and SLR3.

Example stage and flow values simulated during a storm surge event are shown in **Figure H-8** as instantaneous values and **in Figure H-9** as 12-hour moving average values.

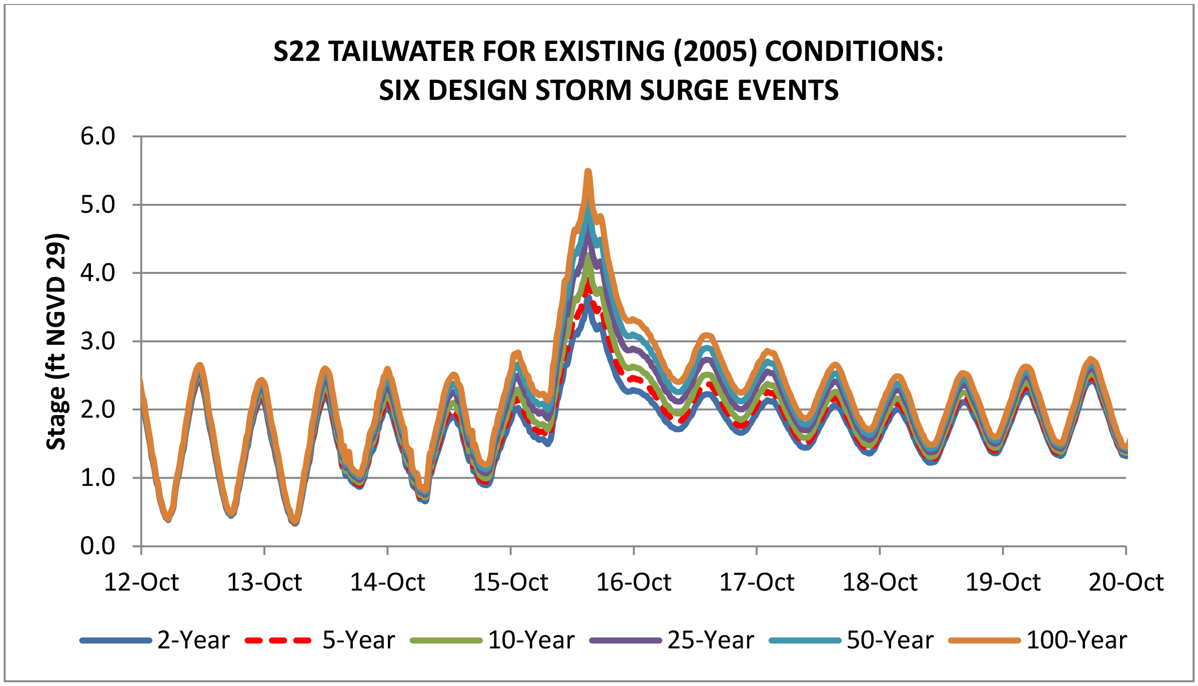

Figure H-6. S-22 tail-water for existing (2005) conditions: six design storm surge events

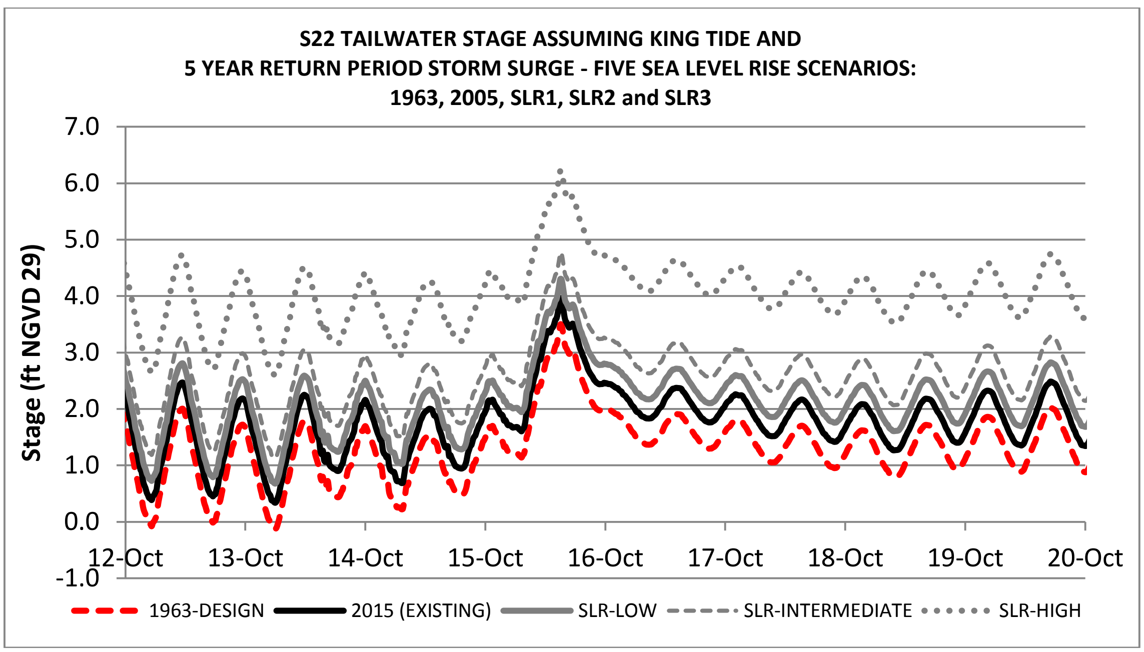

Figure H-7. S22 tail-water stage for five sea level rise scenarios (5-year storm surge for all scenarios)

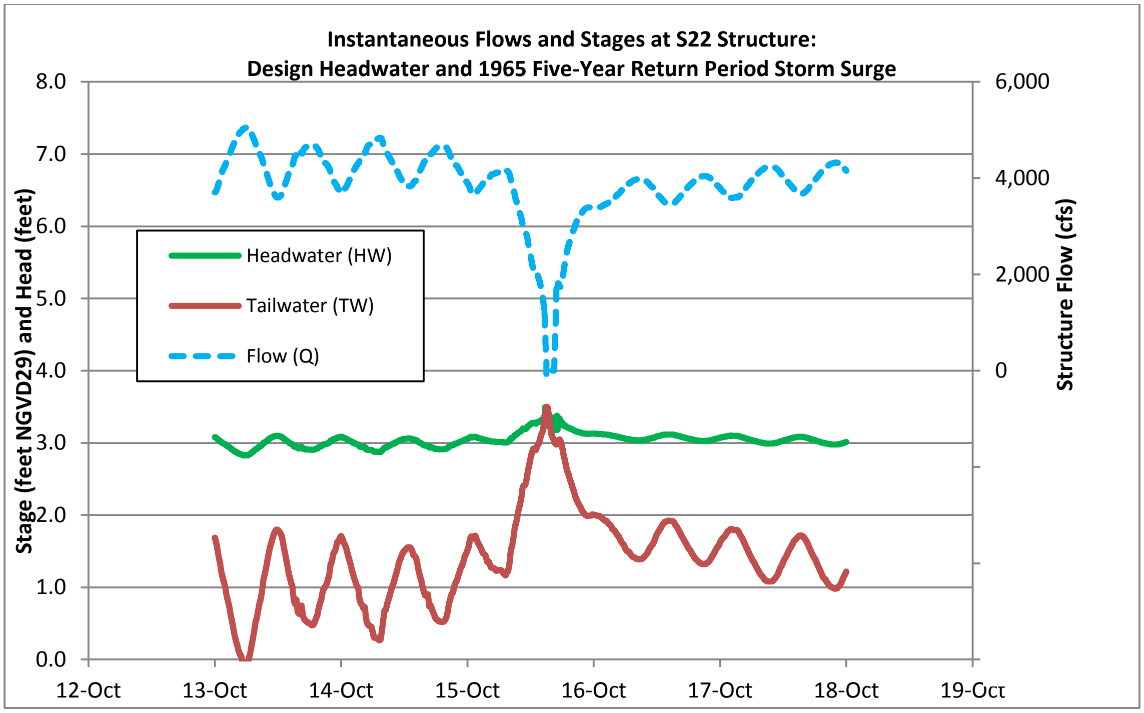

Figure H-8. Instantaneous Flows and Stages at S22 Structure: Design Headwater and 1965 Five-Year Return Period Storm Surge.

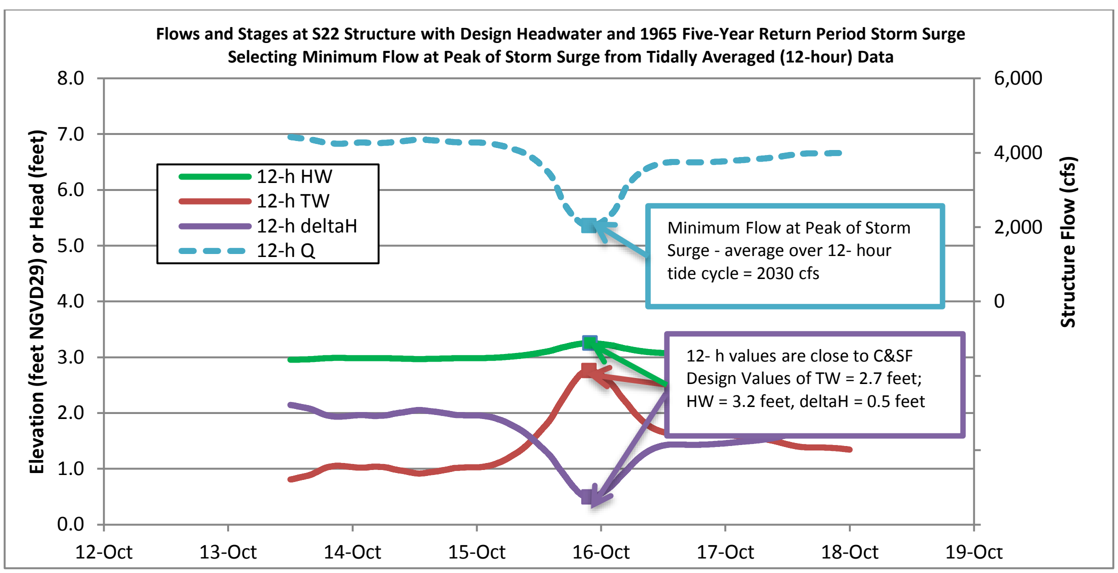

Figure H-9. Selecting Minimum Flow at Peak of Storm Surge using Tidally Averaged (12-hour) Data

This example depicts headwater stage, tail-water stage and flow through the structure for the 1963 sea level with a 5-year storm surge event. Furthermore, the model simulation produced 12-hour average stage values that simultaneously match the steady-state design conditions for headwater (3.2 feet) and tail-water (2.7 feet). Note that the minimum 12-hour flow occurs at the peak of the storm surge and has a simulated value of 2030 cfs. This value is comparable to the steady-state design discharge value of 1930 cfs obtained from the original C&SF design.

Model Simulation Results

Figure H-10 shows the performance of the S25B structure as affected by sea level rise and storm surge. Similarly, **Figure H-11** shows the performance of the G93 structure while **Figure H-13** shows the performance of the S22 structure. All figures display the minimum flow values (derived in the manner discussed in the previous section) for each of the thirty simulations (five sea level scenarios each with six design storm surge events). The structure flows associated with 1963 storm surge boundary conditions

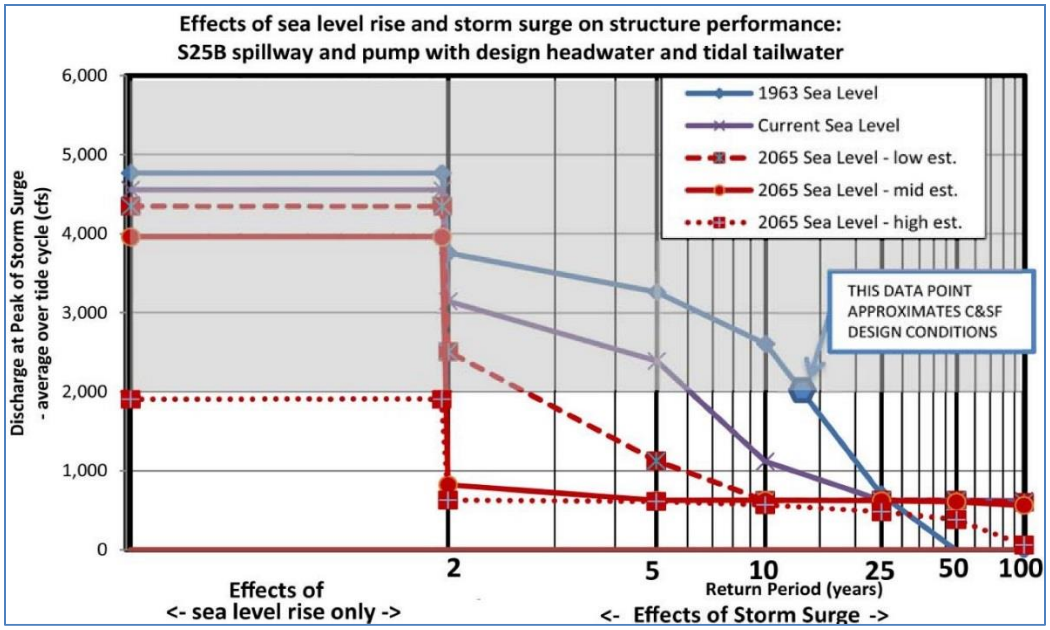

Figure H-10. Effects of sea level rise and storm surge on the capacity of S25B Spillway and Pump (no pumps in original C&SF design but pumps are in current and future scenarios)

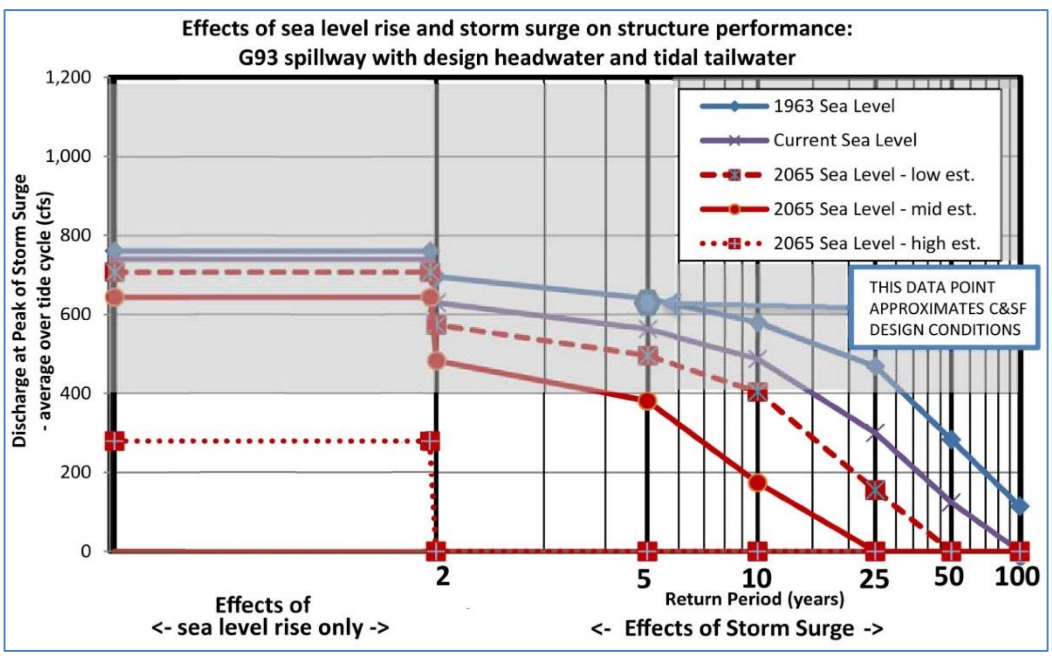

Figure H-11. Effects of sea level rise and storm surge on the capacity OF G93 Spillway (G93 is not a C&SF structure).

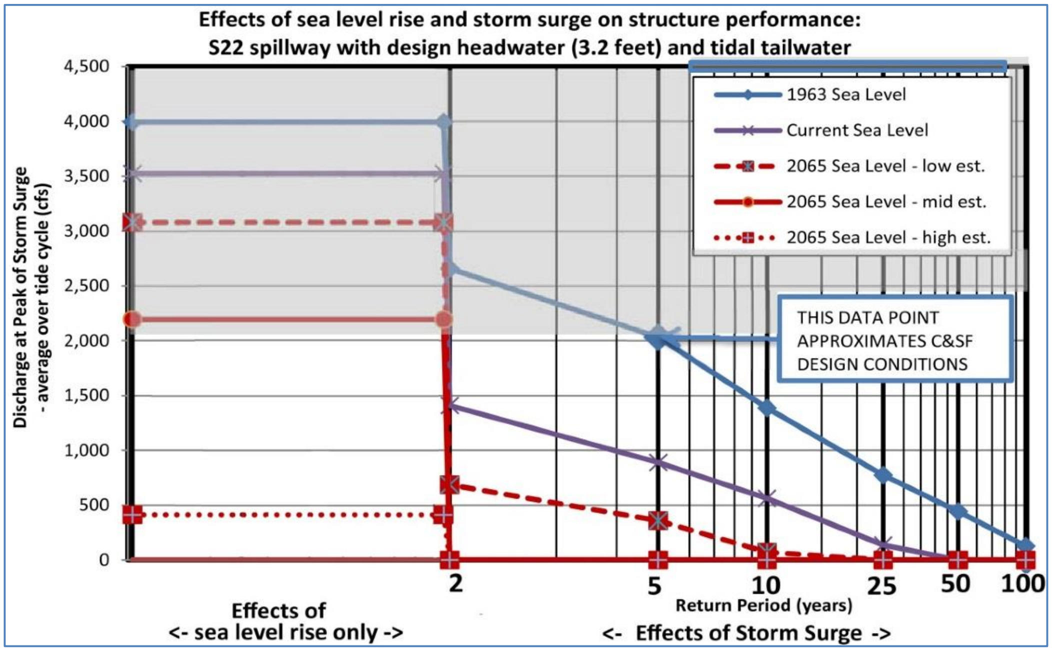

Figure H-12 Effects of sea level rise and storm surge on the capacity of S22

are depicted with blue diamonds; flows associated with current conditions are shown as a purple x's; flows associated with SLR1, SLR2, and SLR3 conditions are shown using the red markers indicated. Furthermore, each line shows the short-term suppression of flows caused by storm surge. Comparison of the N-Surge values shows the prolonged suppression of flows caused by sea level rise.

The original design performance is also displayed on each figure using an enlarged blue hexagon. The discharge associated with the enlarged blue hexagon is based on the design head-water and tail-water values along with the flow computation algorithms implemented for the structure in the FLOW Program. The return period for the design tail-water stage was determined using 12-hour average tail water stages.

A comparison of **Figures H-10 through H-12** reveals that the S22 structure is the structure most strongly impacted by sea level rise. This is to be expected since it is the most low-lying. Under storm surges that are greater than or equal to the SLR2 sea level estimates, the S22 structure cannot pass any flow. This is even true for a storm surge with a modest 2-year return period. The S25B and G93 structures, on the other hand, have higher headwater elevations and can pass design flows as long as the storm surge is negligible and sea levels do not exceed the SLR2 estimates.

Also apparent from these results is that storm surge significantly decreases structure capacity. For SLR2 sea levels, the capacity of the S25B structure drops to the 600 cfs pump capacity for any storm surge with a return period equal to or greater than 5 years. Likewise, under the same sea level conditions, the capacity of the G93 structure decreases to less than 400 cfs (65% of design) for a 5 year return period surge. However, because 400 cfs is the approximate capacity of the C3 canal, the G93 structure is effectively overdesigned. Therefore, the 65% loss in capacity is unlikely to have a detrimental impact.

Observations for Specific Structures.

Observations for Specific Structures.

S25B (Figure 16): Without considering storm surge, the structure capacity exceeds the 2000 cfs design capacity for all sea level scenarios except under SLR3 conditions which is a bit under 2000 cfs; the structure can carry the design flow except during storm surge events. Storm surge reduces structure capacity and sea level rise makes reduced capacity situations more frequent. It should be noted that the frequency of a particular tailwater stage changes with each location. S25B and G93 are both located several miles away from the ocean while S22 is close to the ocean. Runoff from both upstream and downstream runoff affects tailwater. Appendix C has more detail regarding the development of tailwater frequency relationships at each station. Back in 1963, the design tail-water stage (4.1 feet NGVD29) was a relatively rare event, equivalent to a storm surge with an approximate return period of 12-13 years. Current sea level is almost 6 inches higher than in 1963 and the 2000 cfs capacity is limited by storm surges with 6-year return period storm surge or greater. These are still infrequent events. But future sea level rise makes reduced capacity situations increasingly likely. For the SLR1 scenario (+0.34 ft), storm surges with a 3 year return period will limit flow. With the SLR2 (+0.80 ft) and SLR3 (+2.26 ft) scenarios, even common storm surges reduces the structure capacity to 600 cfs, the capacity of the forward pumps.

G93 (**Figure 17**): Without considering storm surge, the structure capacity exceeds the 560 cfs design capacity for all sea level scenarios except under SLR3 conditions. For SLR3 conditions the capacity drops to 270 cfs, far below the 560 cfs design capacity and considerably below the observed 400 cfs canal capacity.

Storm surge has less effect on the capacity of the G93 structure than it does on the S25B. This is because G93 is designed for a 0.5 ft head drop across the structure while S25B is designed for a 0.3 ft head drop. Back in 1963, the design tail-water stage (3 feet NGVD29) was an infrequent event, equivalent to a storm surge with an approximate return period of 5 years. For current sea levels, this equates to storm surges with a 3 year return period – still quite rare. Even SLR1 conditions require storm surges with a 2 year return period to limit flow to the design capacity. The SLR2 sea level conditions make storm surges problematic if the 560 cfs design capacity is considered but the canal's carrying capacity is only 400 cfs and the structure can carry this flow when 2-year storm surges occur. It is only with SLR3 sea level conditions that storm surge limits flow.

S22 (**Figure 18**): Without considering storm surge, the structure capacity exceeds the 2000 cfs design capacity for all sea level scenarios except under SLR3 conditions. For the SLR3 condition, capacity is severely reduced to under 500 cfs.

Back in 1963, the design tail-water stage (3.0 feet NGVD29) was a relatively rare event, equivalent to a storm surge with an approximate return period of 5 years. This is no longer true. Current sea level is almost 6 inches higher than in 1963 and even 2-year storm surges reduce the capacity to 1500 cfs. For the SLR1 scenario (+0.34 ft), the 2-year storm surges limits capacity to 600 cfs and for the SLR2 scenario (+0.80 ft), the 2-year storm surge limits capacity to 0 cfs. Past sea level rise makes this structure problematic during any significant storm surge event; future sea level rise will restrict or stop drainage during common storm surge events.

METRIC #4: IMPACT OF SEA LEVEL RISE ON STRUCTURE FLOW UNDER DESIGN STORM CONDITIONS

Criteria and Results

PM#4 looks at the impact of sea level rise under design storm (design rainfall and design storm surge) conditions. Unlike PM#3, the headwater at the structure is not at design levels but is determined by runoff from the rainfall, surge and structure operations. Like PM#3 the peak flows are averaged over the tide cycle to eliminate the 12-hour tidal oscillation effect. Unlike PM#3, which selects the minimum flow caused by the storm surge suppression, PM#4 selects the maximum flow that occurs as the storm surge recedes.

All the model simulations assumed that the peak storm surge occurs at the same time as the peak rainfall and the results presented here follow that assumption. However, review of results for PM#4 indicated that the offset between the peak rainfall and peak storm surge at the costal structure is an important parameter to consider during design, may be as important as selecting the magnitude of the rainfall or the peak tidal surge stage. For instance some sensitivity and trial-and-error simulations suggest that an offset of 13 hours (storm surge peaks later than rainfall) created the maximum reduction on peak runoff at the S25B structure.

Figures H-13, H-14 and H-15 show the impact of sea level rise on structure flow under design rainfall conditions for structures S25B, S22 and G93, respectively.

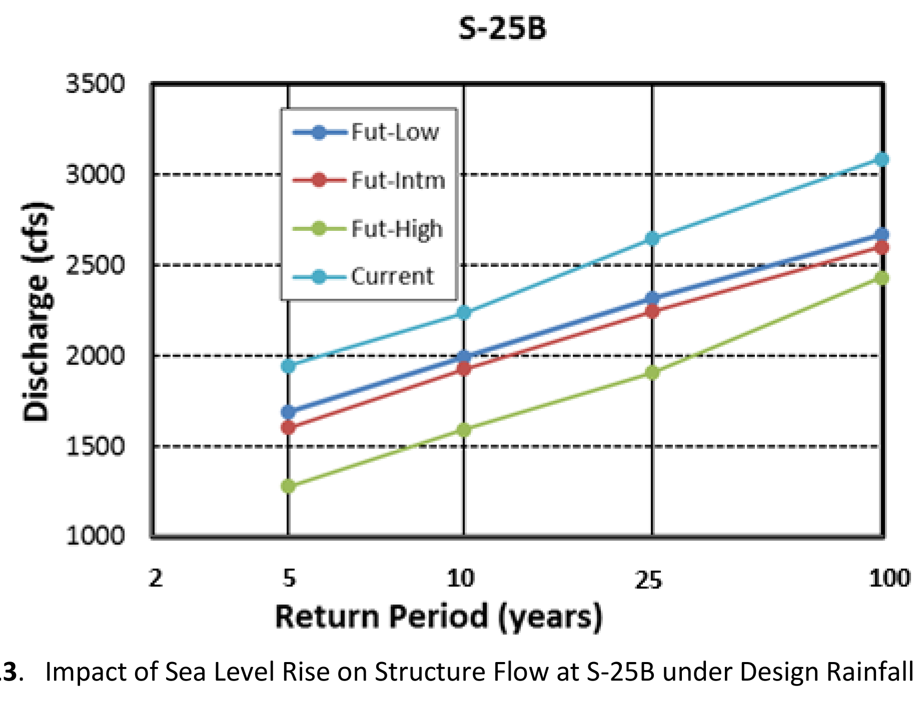

Figure H-13. Impact of Sea Level Rise on Structure Flow at S-25B under Design Rainfall Conditions.

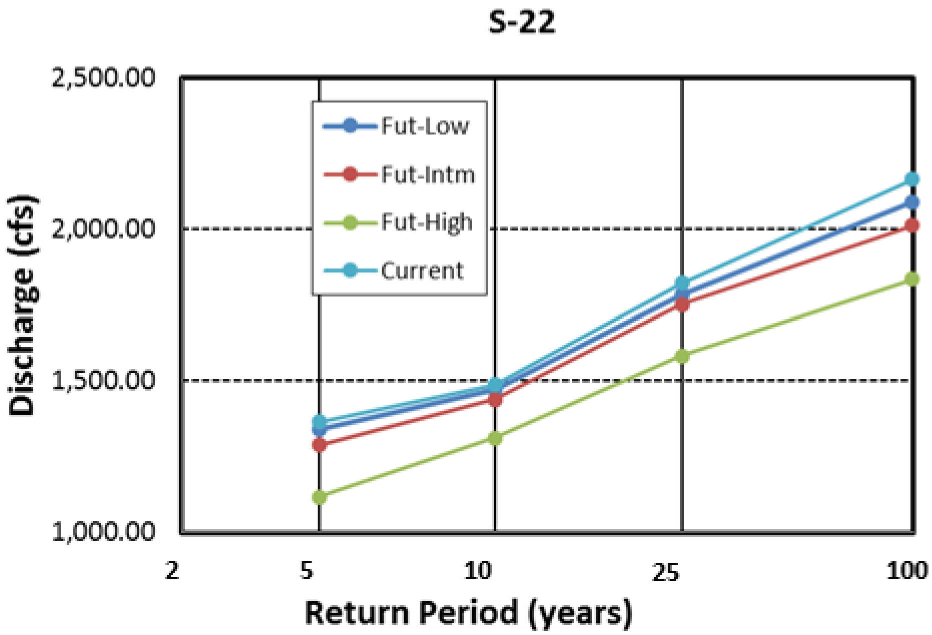

Figure H-14. Impact of Sea Level Rise on Structure Flow at S-22 under Design Rainfall Conditions.

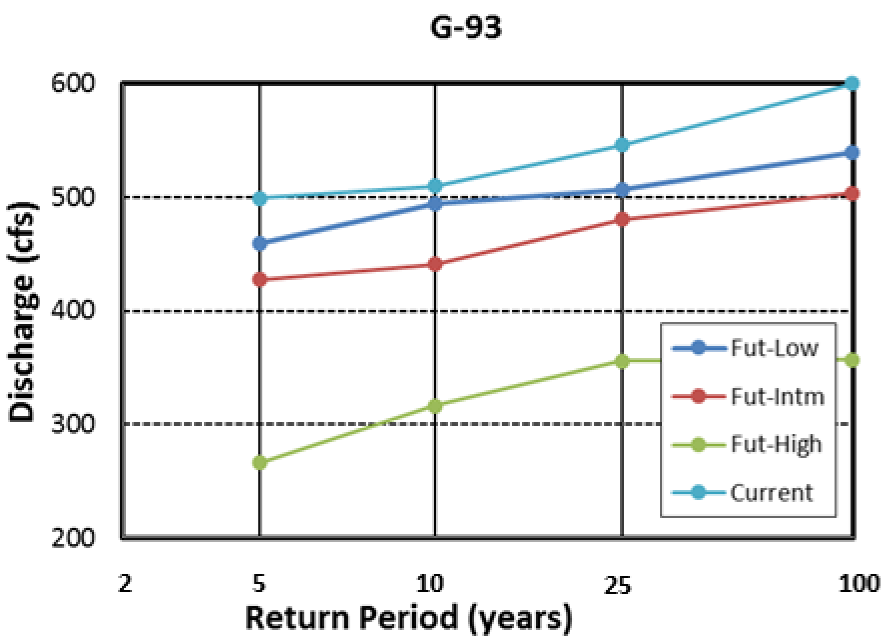

Figure H-15. Impact of Sea Level Rise on Structure Flow at S-G93 under Design Rainfall Conditions.

Observations and Interpretation

- Maximum structure flow increases with design storm because both runoff and storm surge increase with the design storm. This causes more runoff to be impounded upstream of the structure resulting in higher headwater stage and higher flows when the surge retreats.

- The maximum structure flows exceed design capacity when headwater stage is above design stage and the head drop across the structure exceeds design head drop. The impact of the high flows and high stages on structural stability may be significant but were not analyzed here.
- Under all storm events and all scenarios, the impact of sea level rise is minimal for all structures for SLR1 and SLR2. The 0.34 feet increase of SLR1 and the 0.80 feet increase of SLR2 do not impact peak outflows.
- SLR3 scenario significantly reduces peak runoff for all storm events at all structures. The 2.26 feet increase of SLR3 is great enough to reduce the outflows from the structures. [This is also seen in PM#2 – conveyance capacity, which is also reduced under SLR3 and only SLR3.]
- The impact of SLR3 is least at S25B and G93 for the 100 year event. The reasons for these are not clear:
    - This may be complicated by significant reverse flow at S25B and G93 during the peak of the surge.
    - There is no reverse flow at S22.
    - The reverse flow at G93 is significant but the volume is minor because of the small upstream storage.
- In most of the cases, the figures shown below present flow magnitudes which exceed design flows for the structures. One factor contributing to this is head differentials in the simulations exceeding design head differentials. Large head differentials appear for both positive and negative (reverse flows).
- These head differentials need to be accounted for during design and day-to-day operations. Large head differentials need to be considered during the design so that appropriate stability checks are conducted for large positive (tilting/sliding to the ocean) and negative (tilting/sliding inland)
- Day to day operations need to consider the Maximum Permissible Head Differential (MPHD). While operating structures, water managers maintain head differentials which do not compromise the safety of the structure.

It is clear from the discussion above that simulation of realistic head differentials (positives and negatives) is extremely important for the design of water control structures, not only in the determination of the hydraulic capacity, but also to analyze and assure the stability of the structure.

Head differentials during the operation of the structure are a function of the runoff coming to the structure, the operations of the structure, the headwater elevation and tailwater elevation. Head differential translates into a certain discharge passing the structure. Head differential will also depend on the timing of the peak rainfall, transit times in the basin and the arrival of the storm surge. For instance, when peak runoff at the structure occurs close to storm surge, large suppression of flow will take place, which will translate in increased stages upstream. In the case when peak runoff and peak storm surge are far apart in time, runoff will be properly evacuated, but the closing of gates may create large undesirable negative head differentials.

Further analysis of the role of the offset between peak rainfall and storm surge is necessary. It is clear that this offset could be as important as other design parameters such as rainfall and storm surge magnitude in order to properly quantify the capabilities of the structure. Methodologies to select the values to use in the design need to be explored.

METRIC #5: IMPACT OF SEA LEVEL RISE ON SUB-WATERSHEDS

Maximum Stage

Sea level rise can affect flooding within the subwatersheds in several ways: long-term suppression of flow, short-term suppression caused by storm surge, higher water tables caused by suppressed groundwater flow, and increased base flow caused by higher water tables. All are considered in these analyses.

Figure H-16 shows the maximum water stage in each of the thirty-three sub-watersheds of the C4 watershed when exposed to a 5-year, 10-year, 25-year and 100-year design storm event under current sea level conditions. The figure includes a dashed line that shows the flood threshold of each watershed. The flood threshold is a criterion developed by District staff for use as an indicator that substantial flooding may be occurring within the subwatershed. This criterion is defined as the stage where 20% of the developed land within a sub-watershed is underwater. The analysis to determine this water level excludes marshes, lakes, and rock-pits. Sub-watersheds dominated by wetlands and rock-pit areas are included in the graph but were not considered when establishing flood protection level of service; therefore, these seven subwatersheds are shaded gray.

Maximum stages for the 5-year event are below the flood threshold for all twenty-seven sub-watersheds. Maximum stages for the 10-year event are below the flood threshold for all but two sub-watersheds (AG4 and 100B). Maximum stages for the 25-year event are below the flood threshold for eleven of the twenty-six subwatersheds. Based on this, the C4 system provides flood protection for the 10-year storm event; protection is marginal for the 25-year storm event. Maximum stages for the 100-year event are below the flood threshold for only one of the subwatersheds and the flood depth (the difference between the maximum depth and the threshold elevation) exceeds 1.0 feet in thirteen of the twenty-six subwatersheds.

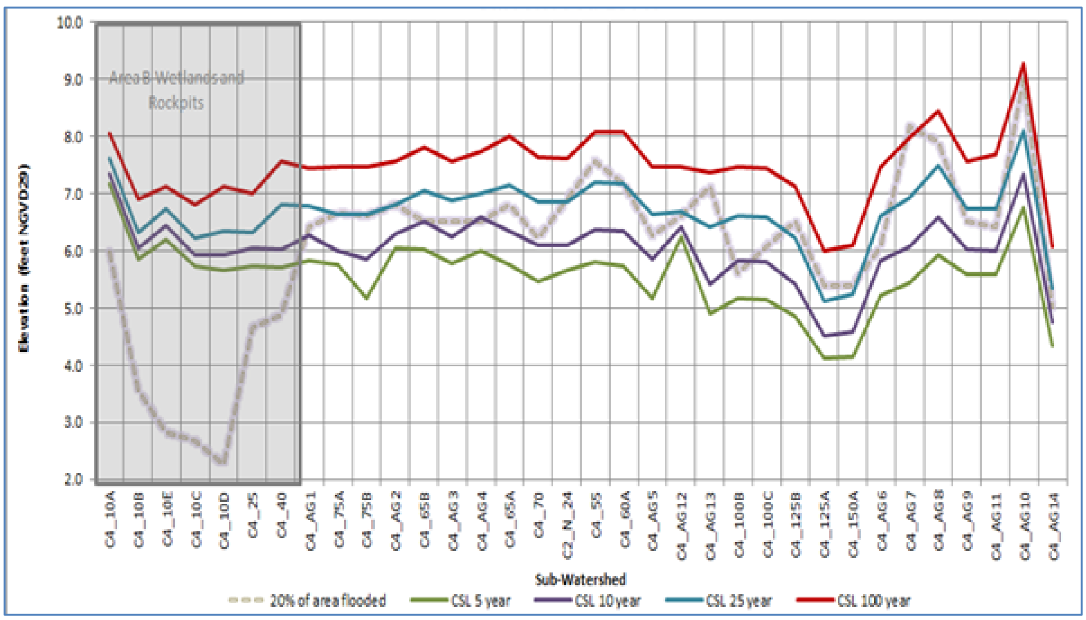

Figure H-16. Maximum Stage in Sub-Watersheds for Four Design Storm Events: Current Sea Level.

It should be noted that the design flood simulated for this analysis is not the same as the design flood used in flood rate insurance (FIR) mapping. The focus of this study is to assess the impact of sea level rise on the primary drainage system. Flood depth estimates derived in this study are approximations and should only be used for relative comparisons; they cannot be compared to FIRM flood depths. Under this study, rainfall patterns follow the SFWMD 3-day design pattern and tidal boundary conditions assume a storm surge with the same probability as the rainfall event. These rainfall patterns and tidal boundaries are more severe than those used in FEMA simulations.

Figures H-17, H-18 and H-19 show maximum watershed stages for the three future sea level rise scenarios SLR1 (0.34 feet higher than the 2005 sea level) SLR2 (0.80 feet higher), and SLR3 (2.326 feet higher), respectively.

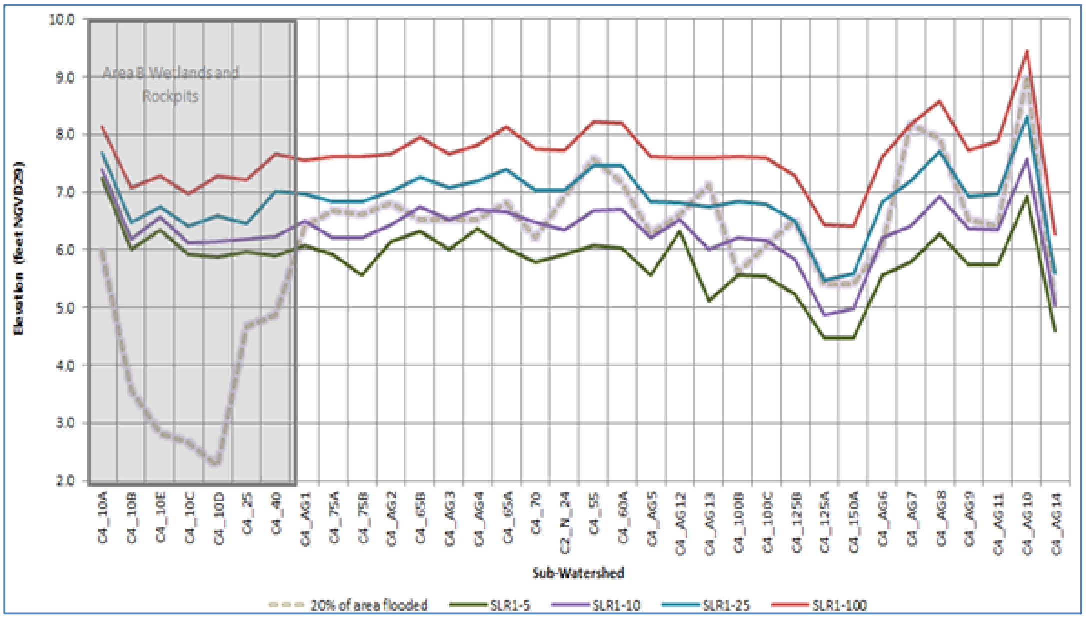

Figure H-17. Maximum Stage in Sub-Watersheds for Four Design Storm Events: Sea Level Rise #1

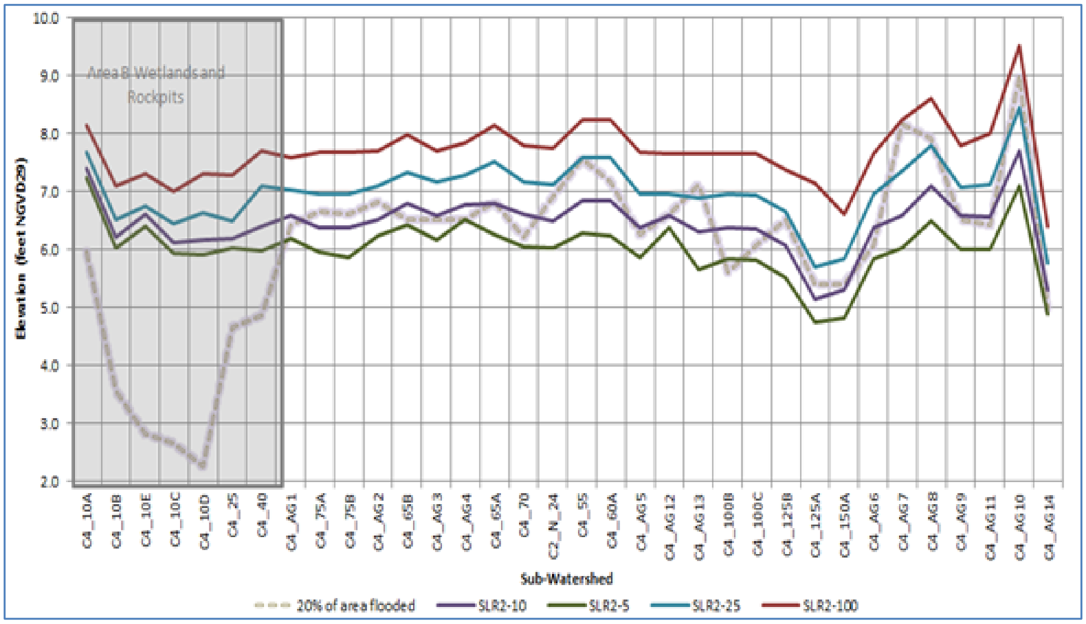

Figure H-18. Maximum Stage in Sub-Watersheds for Four Design Storm Events: Sea Level Rise #2

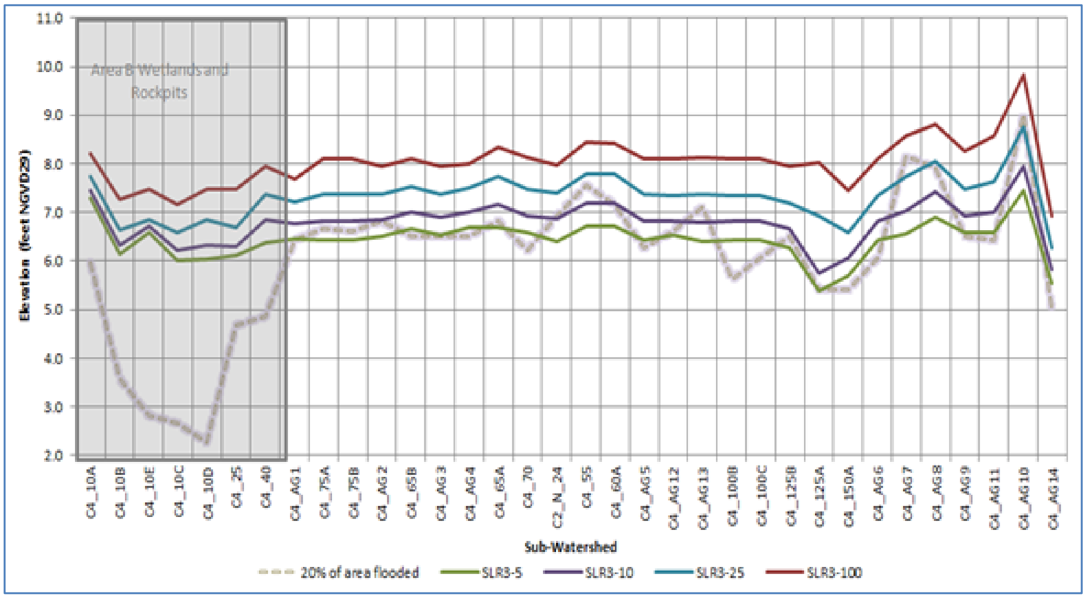

Figure H-19. Maximum Stage in Sub-Watersheds for Four Design Storm Events: Sea Level Rise #3

Figure H-17 shows that flood levels under SLR1 are similar to current conditions and the level of flood protection (LOS) is still 1-in-10. **Figure H-18** shows an increase in flood levels under SLR2 and the 10-year storm results in peak stages in several sub-watersheds that are above the flood threshold. The LOS for SLR2 has dropped to 1-in-5. **Figure H-19** shows a substantial increase in flood levels for SLR3. Even the five year storm results in flooding in several sub-watersheds. The LOS for SLR3 is less than 1-in-5.

Increase in Peak Stage in Sub-Watersheds

PM#5 compares the peak stages in sub-watersheds for SLR1, SLR2 and SLR3 against the peak stages from the 2005 base. These data are tabulated in **Table H2-1** (see **Attachment 2** to this appendix) but, since there are 27 sub-watersheds and twelve simulations to compare (four design storms and three sea levels) the information is difficult to assess. Therefore **Figure H-20** shows a statistical interpretation the **Table H2-1** data. The vertical bars summarize the water level increase for each of the twelve simulations. The bar shows the average water level increase, the 90th percentile increase and the 10th percentile increase. Sub-watersheds dominated by marsh, open water and rock pits are excluded from the assessment.

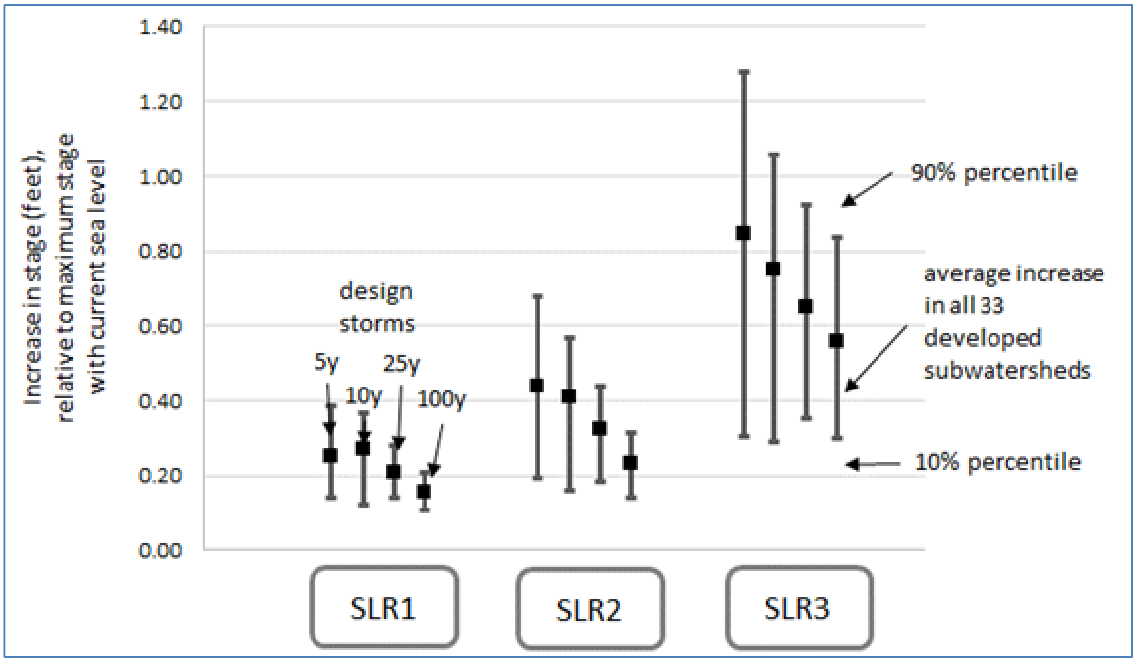

Figure H-20. PM#5: Increase in Maximum Water Level in Sub-Watersheds Caused by Sea Level Rise

Additional details of the flooding analysis, including maps showing the sub-watersheds that are flooded and range of flooding depths for design storm events current and future sea level rise conditions are presented in **Attachment H2.**

The increase in peak stage for SLR1 are minor (<0.4 feet) and are caused by higher initial water levels rather than by reduced flows at the structures.

The increase in peak stage for SLR2 ranges from 0.14 feet to 0.68 feet. The impact is greater for small events than for the larger events but increase is because the water levels are below the ground surface flow the smaller events and soil porosity causes a greater water table rise for the same rainfall than if the water levels are above ground surface. Spatially the impact is greatest for sub-watersheds near the S25B structure where storm surge blocks outflows and creates a backwater effect upstream of the structure.

The increase is greatest for SLR3 and ranges from 0.3 feet to 1.27 feet. The impact of SLR3 is seen on all sub-watersheds but the impact is greatest near the S25B outlet structure. The combined effect of sea level rise and storm surge affect these numbers.

[NOTE: A recent USGS study (2014) used the groundwater model MODFLOW to estimate increases in groundwater levels in Miami-Dade caused by sea level rise. In the C4 area the increase ranged from 1.2 feet near the S25B structure to 0.25 feet increase in the far western reaches of the watershed. These stage increases were used to modify pre-storm stages throughout the watershed. And are incorporated into the results presented in this report.]

METRIC # 6. DURATION OF FLOODING IN PRIMARY CANAL SYSTEM

Metric # 6 examines the time needed to recover from a flood event. The metric looks at water levels at the T5 gage in the C4 canal and tracks the time that water levels are above a target canal stage of 4.5 feet. This target stage is substantially higher that the water control elevation for the system and indicates a flood state, i.e. a state where all control structures are being operated to maximize drainage. (**Figure H-21**).

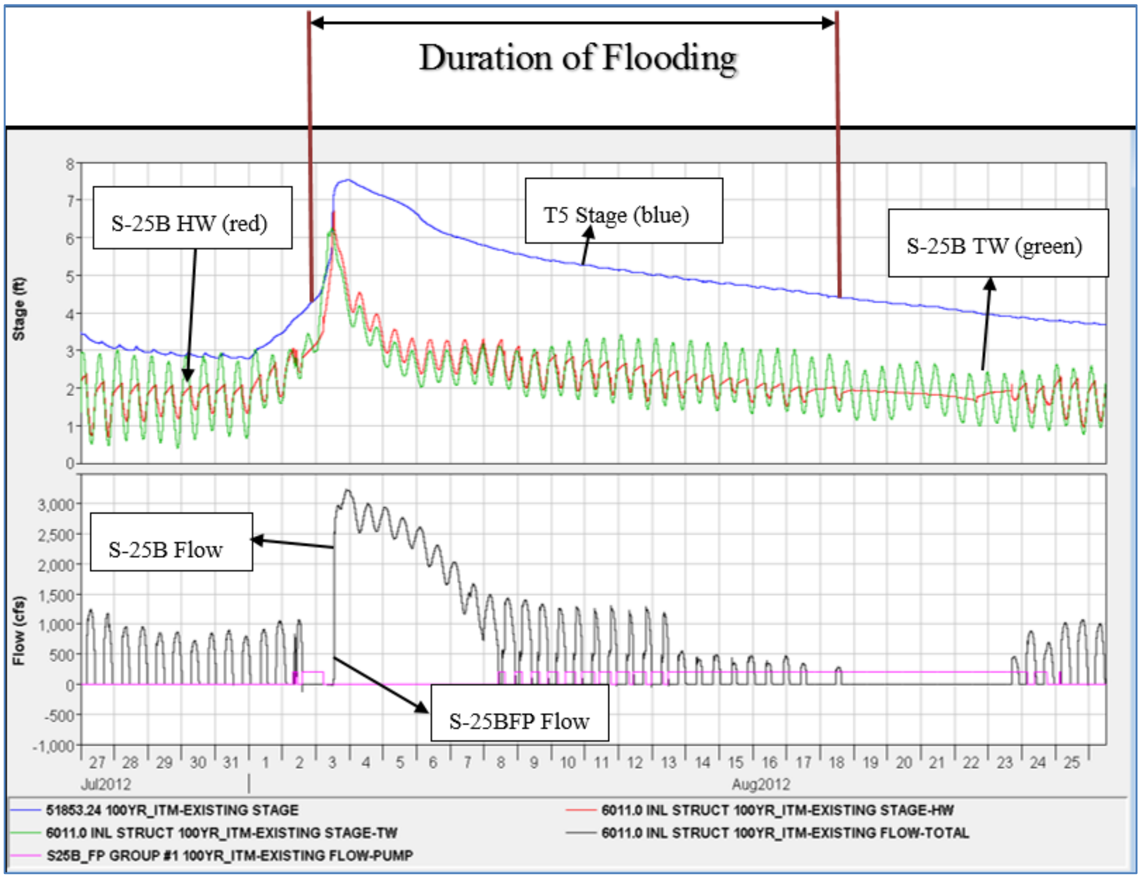

Figure H-21. **.** Procedure for determination of Performance Measure # 6, based the duration of flooding determined from output data from the HEC-RAS Model. PM #6 measures the length of time that water levels are above the normal operating range at the T-5 Gage (4.5 ft).

Figure H-21 shows this performance measure for different Sea Level Rise scenarios. Under current conditions, PM6 ranges from 4 days for a 1-in-5 year storm to approximately 15 days during a 1-in-100 year storm. The duration of flooding increases under future sea level rise conditions: SLR1 adds two days to the flood durations, from 6 days for a 1-in-5 year storm to 17 days during a 1-in-100 year storm. SLR1 adds five days to the flood durations, from 10 days for a 1-in-5 year storm to 19 days during a 1-in-100 year storm. For SLR3 the metric failed because flooding still existed at the end of the 30 day simulation for all design storms.

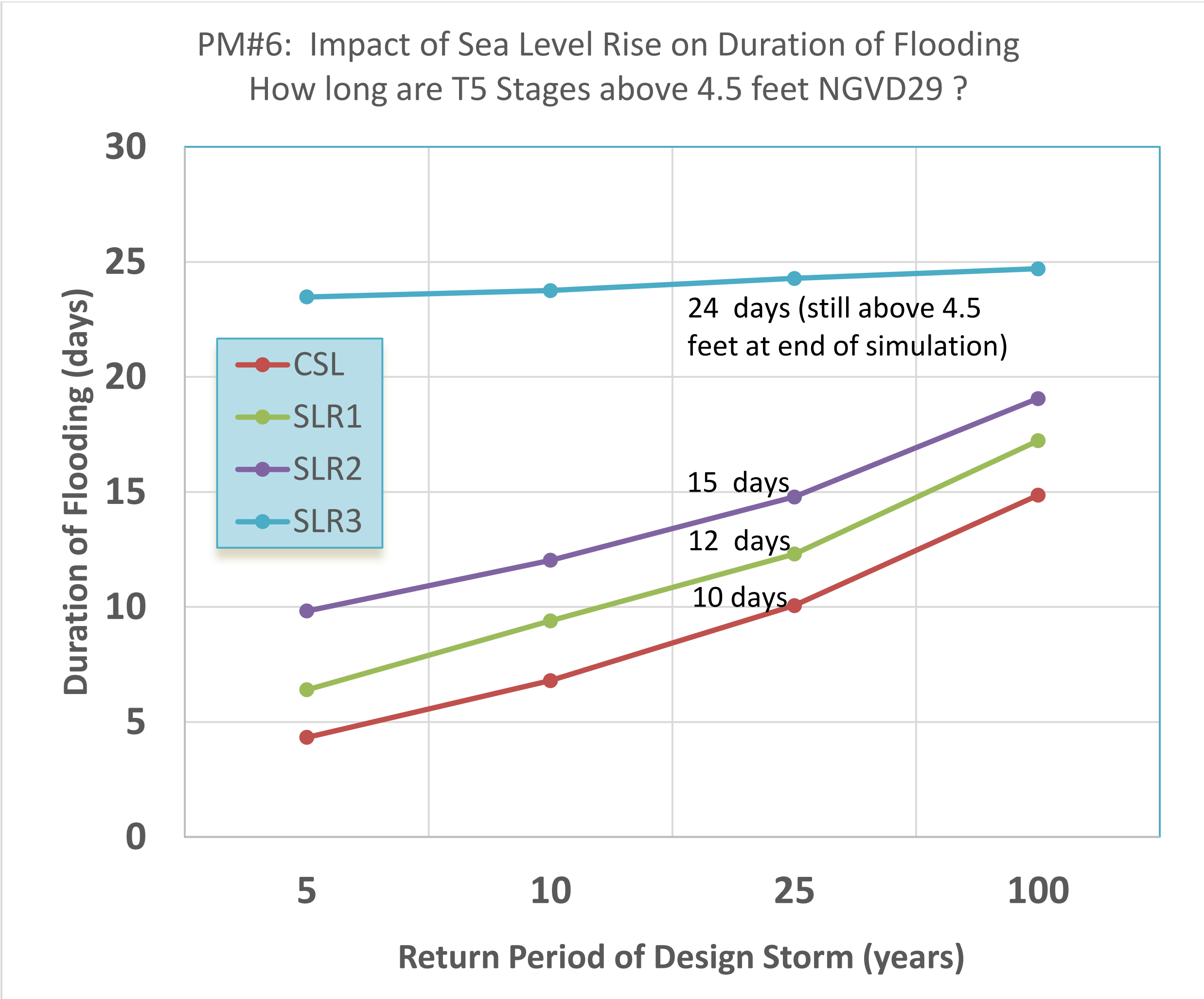

Figure H-22. Impact of Sea Level Rise on Structure Flow at S-25B under Design Rainfall Conditions. Comparison of Performance Measure # 6 for current conditions (CSL) and three future sea level rise scenarios (SLR1, SLR2, and SLR3 - see text).

SUMMARY & RECOMMENDATIONS

The section begins with a summary of the major findings, using select performance metrics. This is followed by a more complete description of the individual performance metrics.

----- [this section is being drafted and will be included in final document] -----

ATTACHMENT H1: FLOOD DEPTHS IN SUB-WATERSHEDS

The flood depths have been divided into color-coded categories according to **Table H1-1**. The categories are in half-foot increments. **Table H1-2** summarizes the figures but counting the sub-watersheds in each flood depth category.

Table H1-1. Flood Depth Category for Flood Depth Maps.

COLOR	The lowest 20% of the sub-watershed have water depths that are …	Interpretation: Portions of the watershed have …
white	below ground surface	Minimal flooding
green	less than ½ feet deep	Local nuisance flooding
Yellow-green	Between ½ feet and 1 foot deep	Nuisance flooding
Yellow	At least 1 foot deep	Problematic flooding (some roads, low structures, …)
Orange-yellow	1 ½ to 2 feet deep	Significant flooding with low house pads under water
Dark Orange	2 to 2 1/2 feet deep	Major flooding
Red	2 ½ to 3 feet deep	Major flooding

Table H1-2. Count of sub-watersheds in each flood depth category

depth range (feet):		< 0	0.0 to 0.5	0.51 to 1.0	1.01 to 1.5	1.51 to 2.0	2.01 to 2.5	2.51 to 3.0
color code		White	Green	Yellow-Green	Yellow	Orange-Yellow	Dark Orange	Red
CSL (2005 sea level)	5-year	27	0					
	10-year	25	2	0				
	25-year	14	11	2	0			
	100-year	1	2	11	12	1		
SLR1 (+0.45 feet)	5-year	27						
	10-year	19	7	1	0			
	25-year	6	9	11	1			
	100-year	0	3	8	12	4		
SLR1 (+0.91 feet)	5-year	25	2					
	10-year	15	11	1				
	25-year	4	10	12	1			
	100-year	0	2	6	13	5	1	
SLR1 (+2.37 feet)	5-year	14	11	2				
	10-year	6	13	7	1			
	25-year	2	4	11	8	2		
	100-year	0	1	4	10	6	5	1

Figures H1-1 through H1-16-show flood depth maps in the eastern twenty-six sub-watersheds of the C4 watershed. There are four sea level scenarios and, for each, four design storm scenarios. The flood depth is the maximum stage minus the flood threshold elevation.

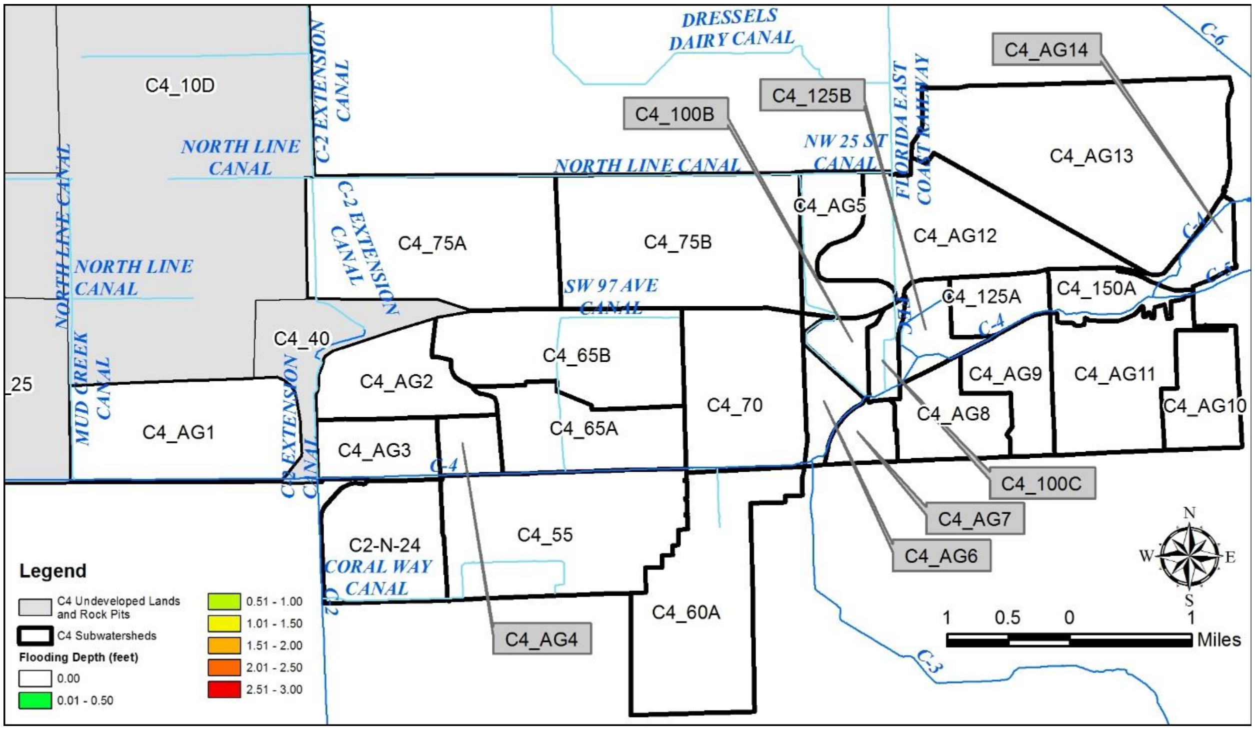

Figure H1-1. Flooding in the C4 Watershed with 2005 Sea Levels: 5-year design storm and 5-year storm surge

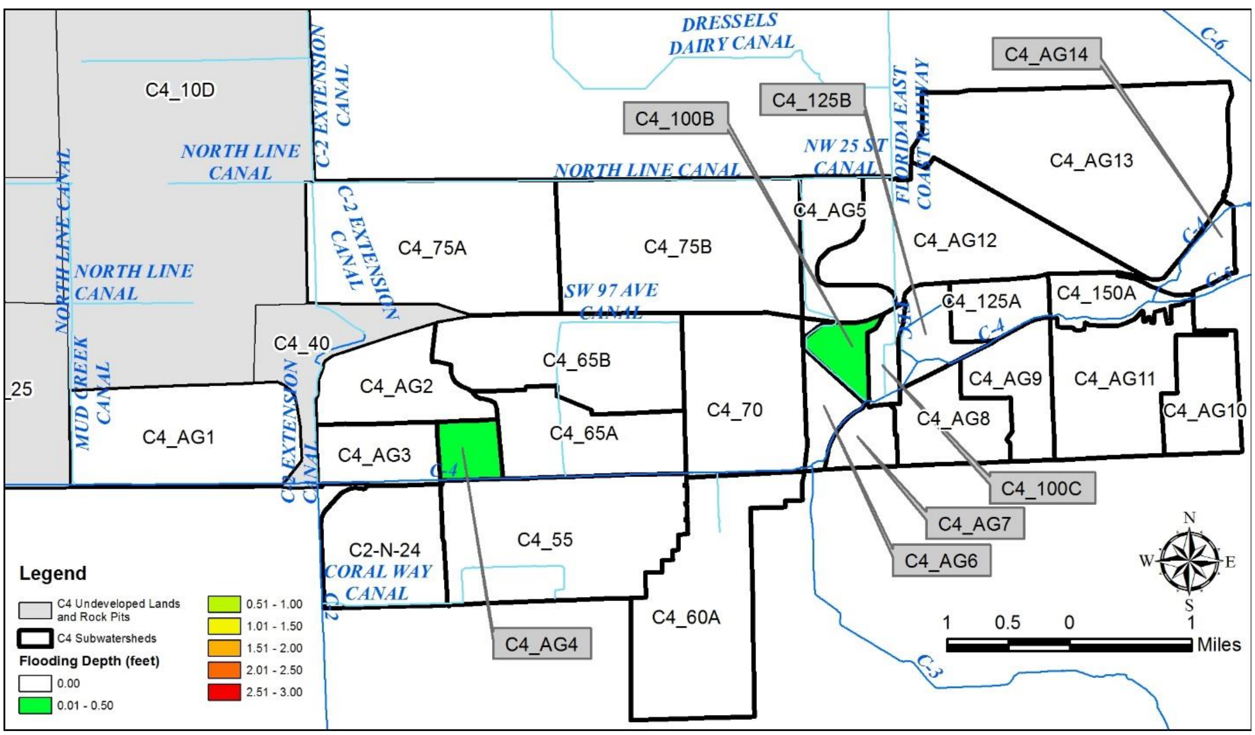

Figure H1-2. Flooding in the C4 Watershed with 2005 Sea Levels: 10-year design storm and 10-year storm surge.

Appendix H: Details of Level of Service Analysis for C-4 Watershed--Attachment H1

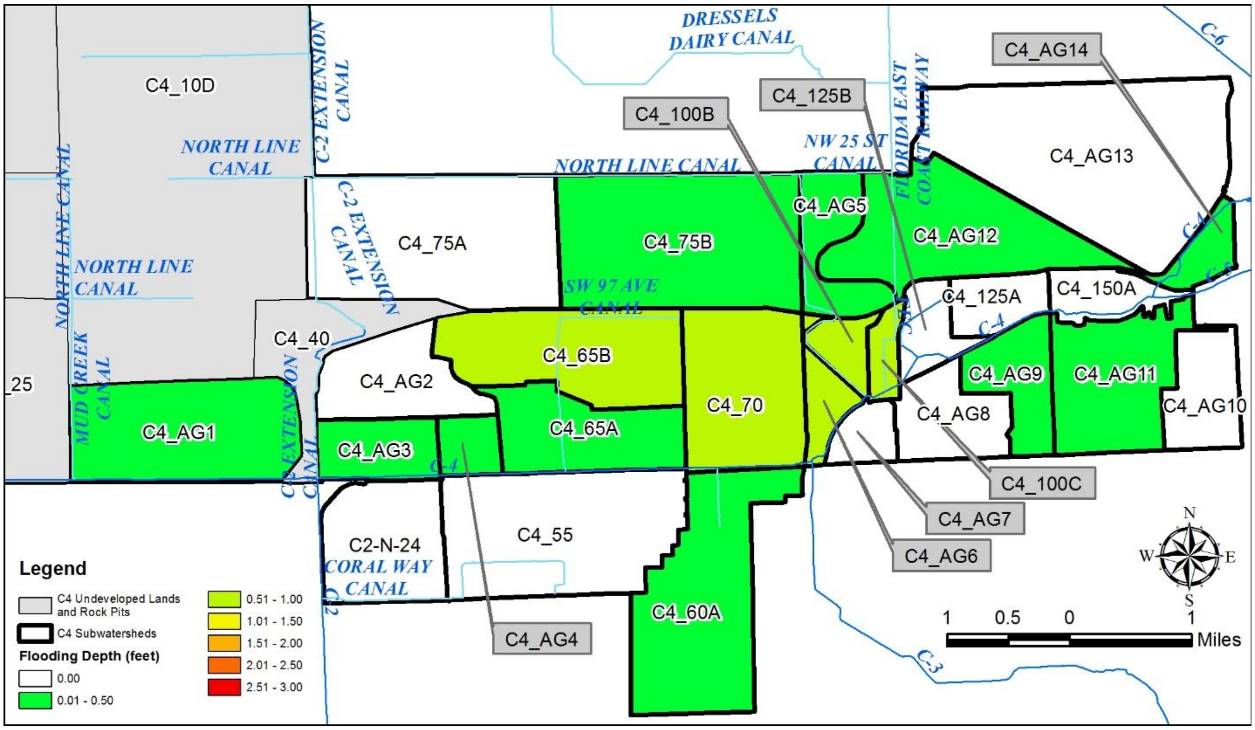

Figure H1-3. Flooding in the C4 Watershed with 2005 Sea Levels: 25-year design storm and 25-year storm surge.

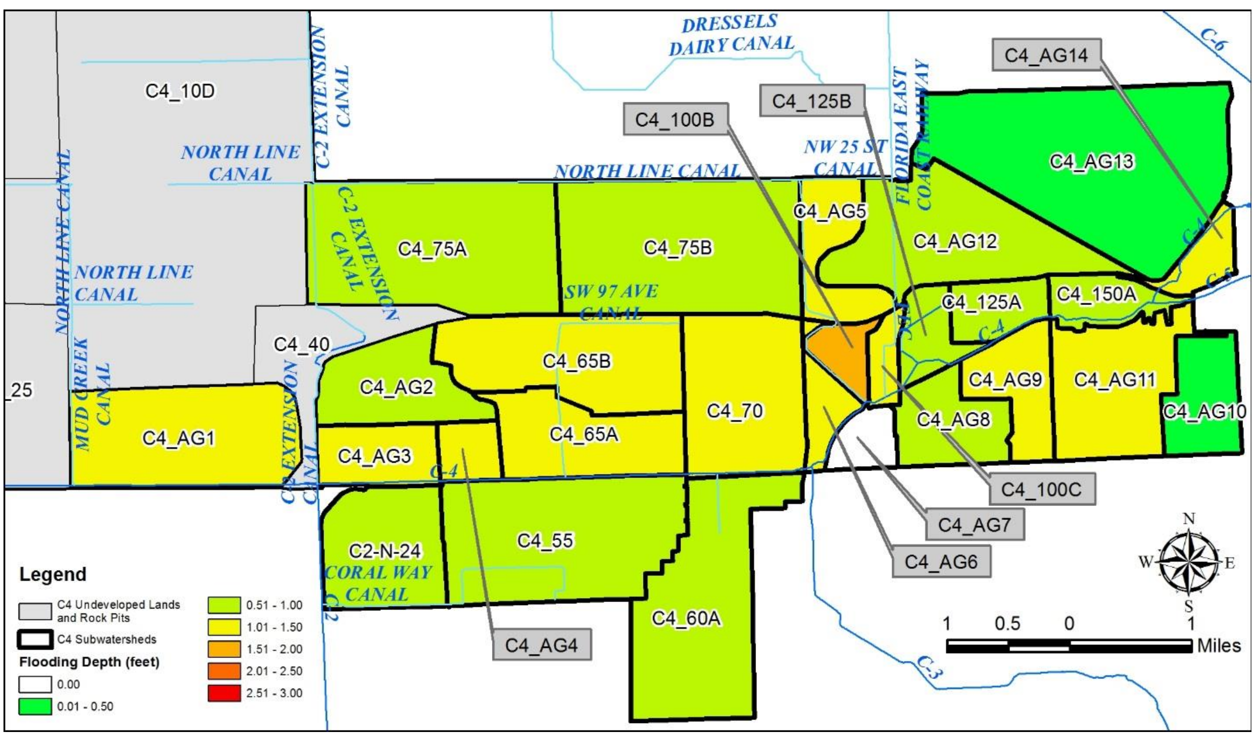

Figure H1-4. Flooding in the C4 Watershed with 2005 Sea Levels: 100-year design storm and 100-year storm surge.

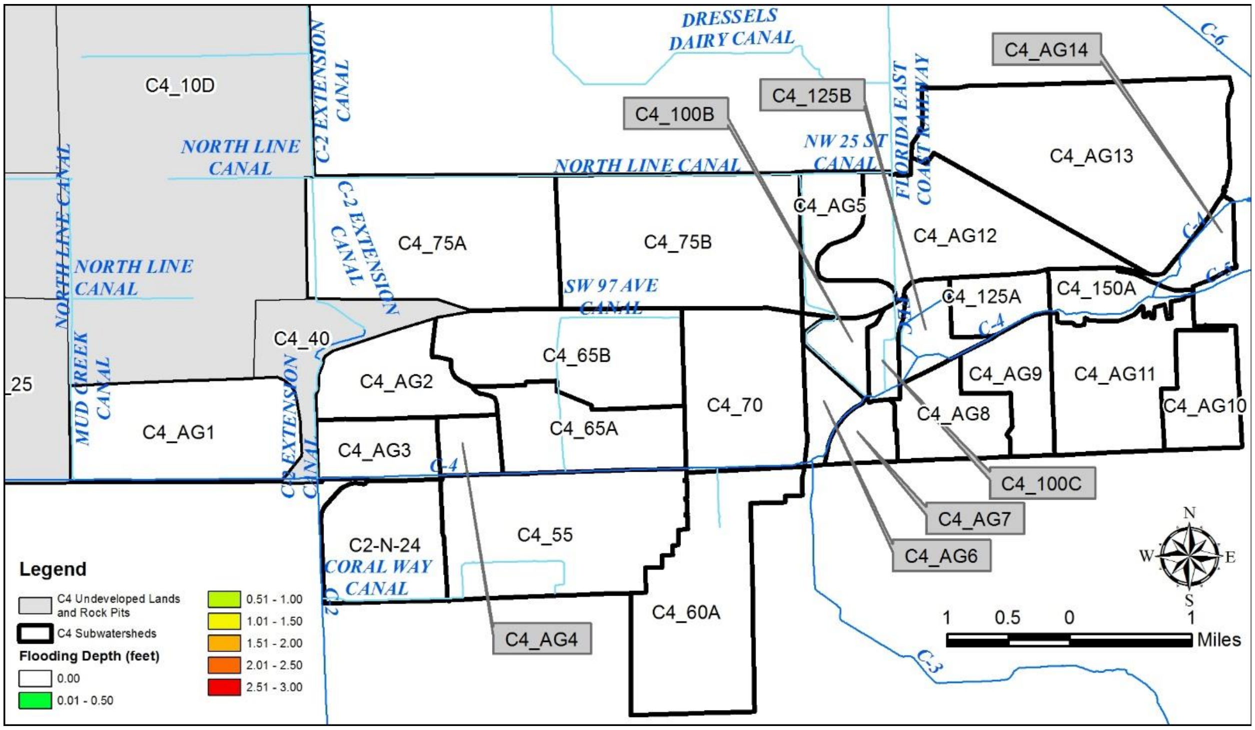

Figure H1-5. Flooding in the C4 Watershed with SLR1 Sea Levels: 5-year design storm and 5-year storm surge.

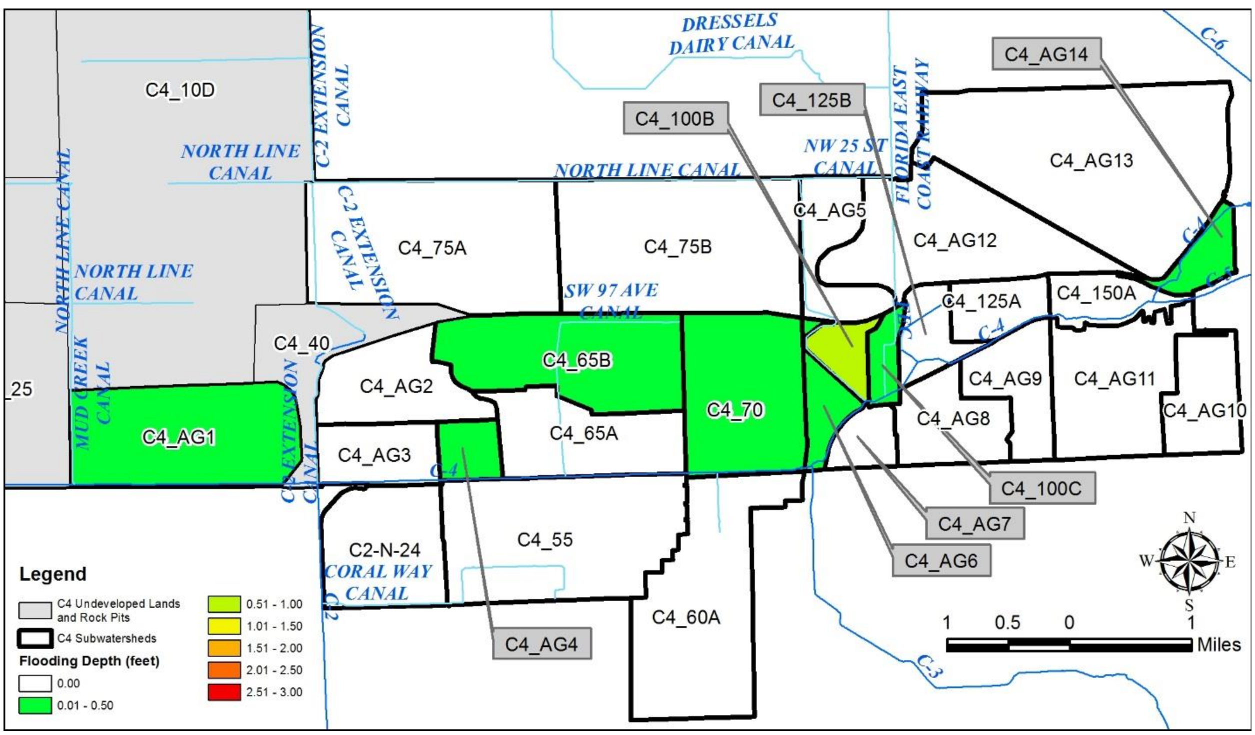

Figure H1-6. Flooding in the C4 Watershed with SLR1 Sea Levels: 10-year design storm and 10-year storm surge.

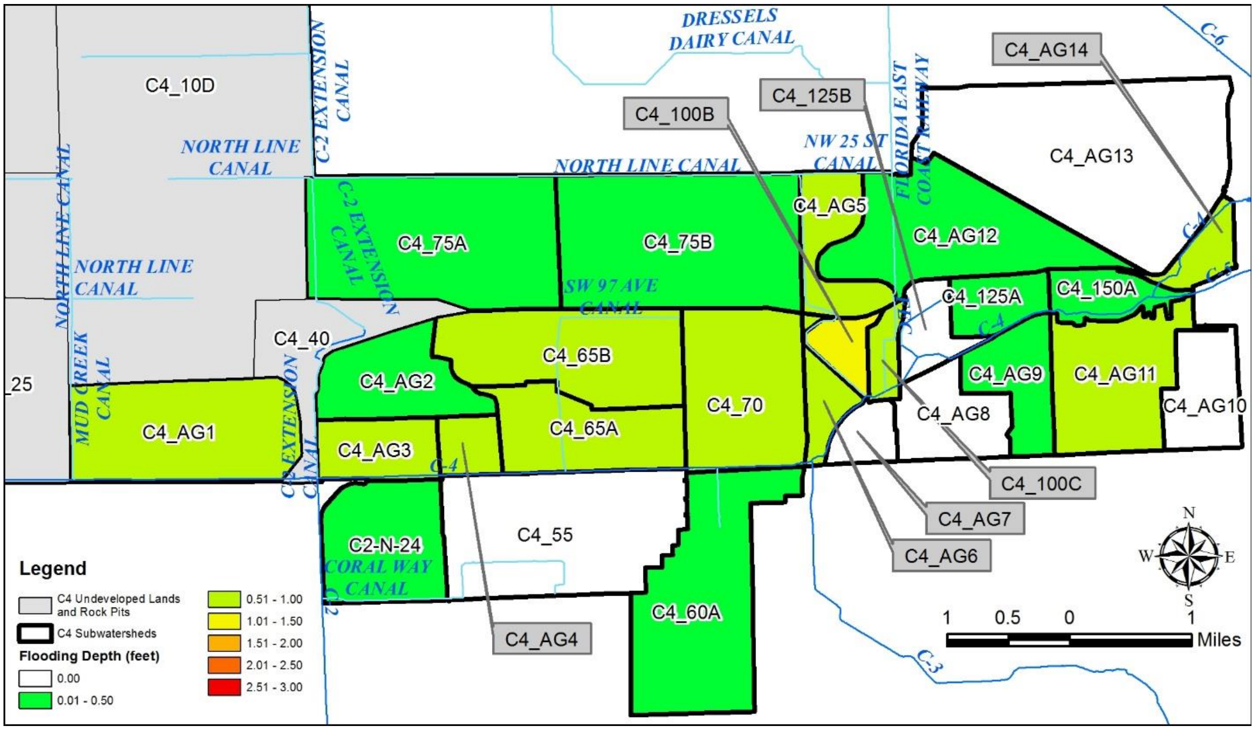

Figure H1-7. Flooding in the C4 Watershed with SLR1 Sea Levels: 25-year design storm and 25-year storm surge.

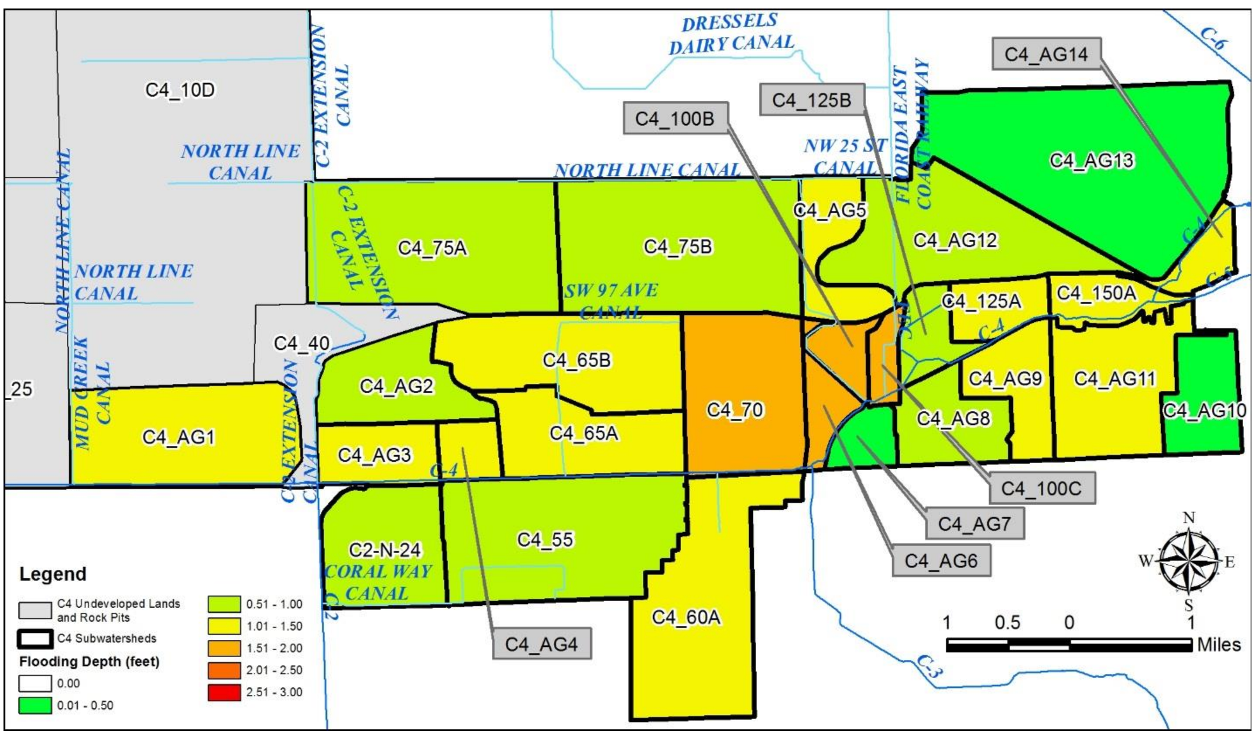

Figure H1-8. Flooding in the C4 Watershed with SLR1 Sea Levels: 100-year design storm and 100-year storm surge.

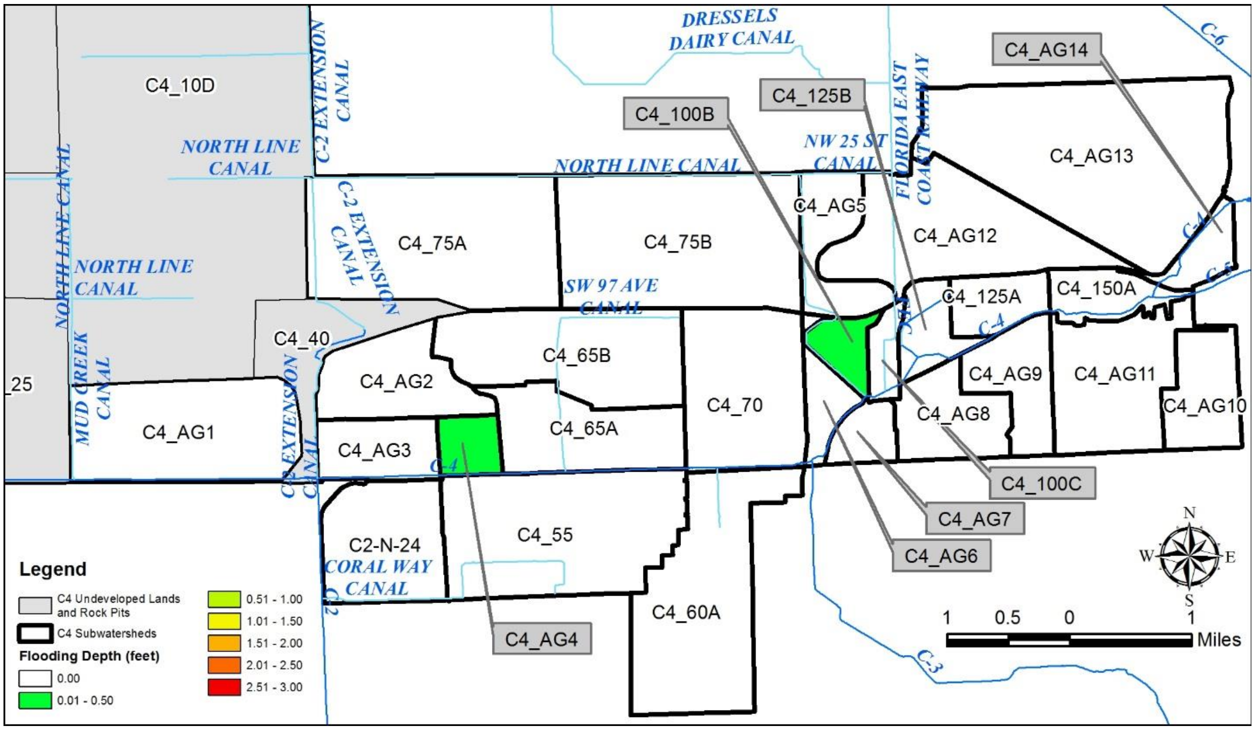

Figure H1-9. Flooding in the C4 Watershed with SLR2 Sea Levels: 5-year design storm and 5-year storm surge.

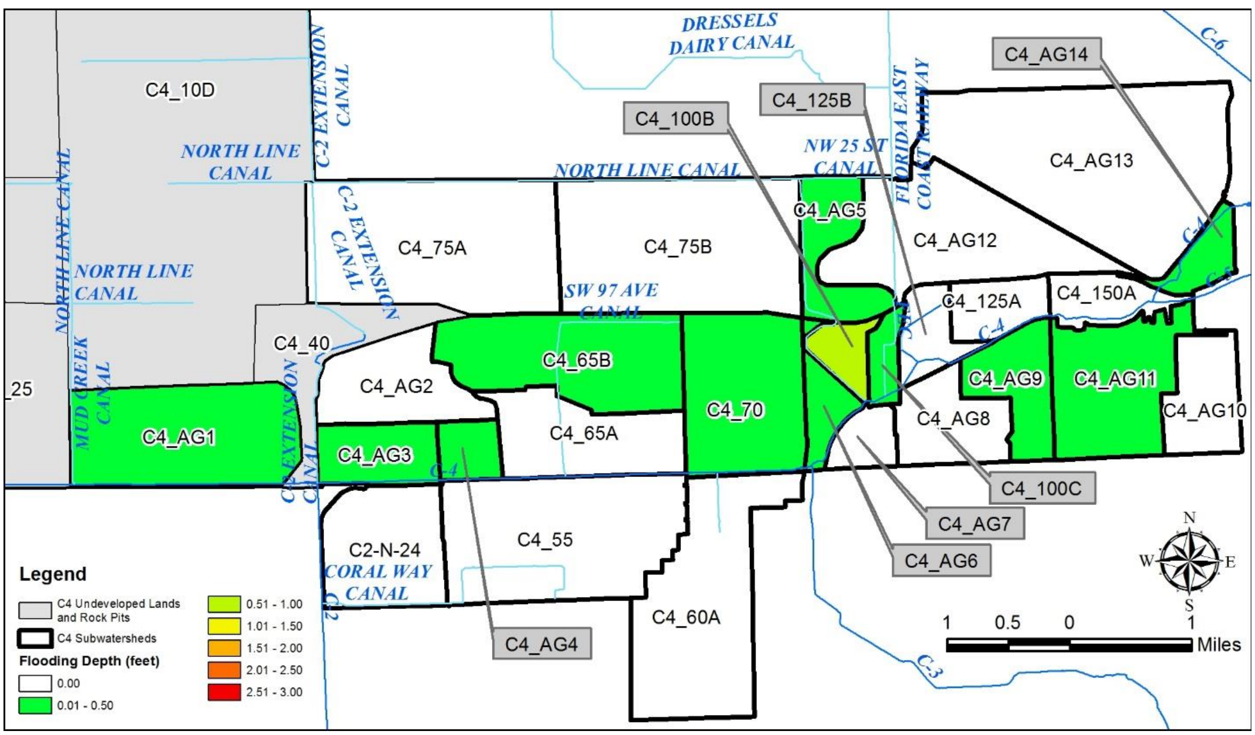

Figure H1-10. Flooding in the C4 Watershed with SLR2 Sea Levels: 10-year design storm and 10-year storm surge.

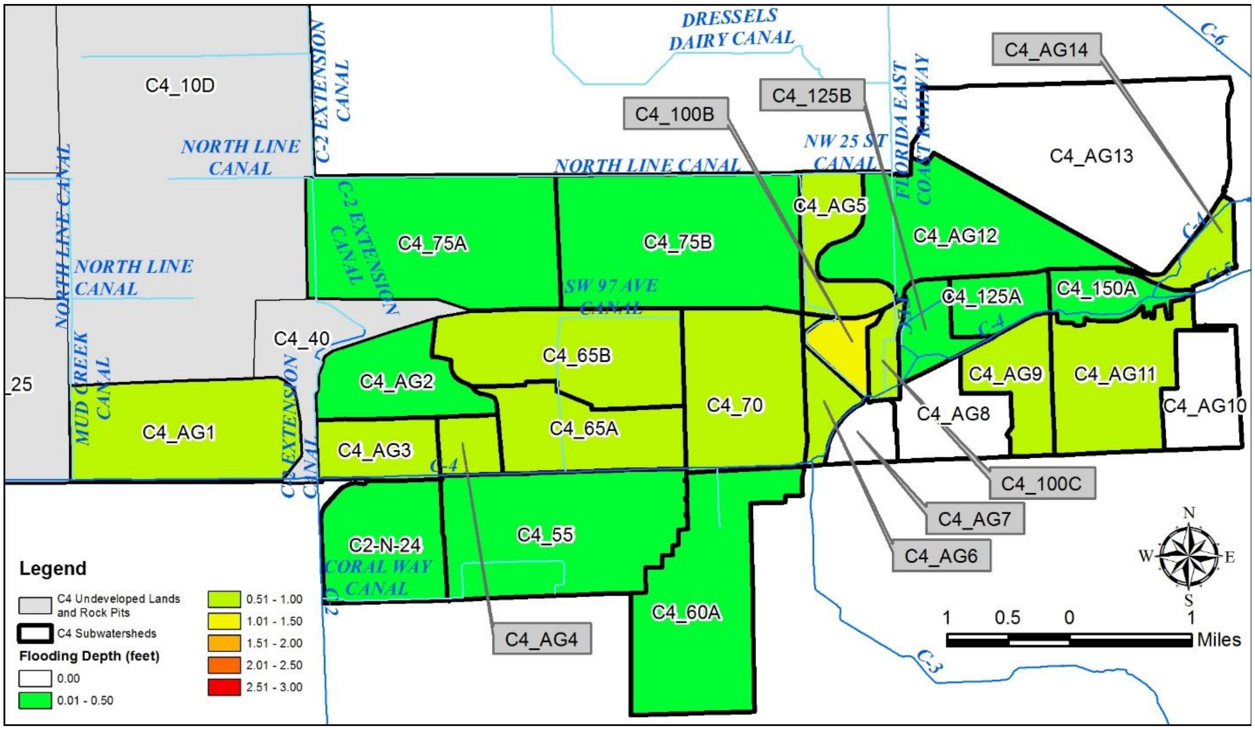

Figure H1-11. Flooding in the C4 Watershed with SLR2 Sea Levels: 25-year design storm and 25-year storm surge.

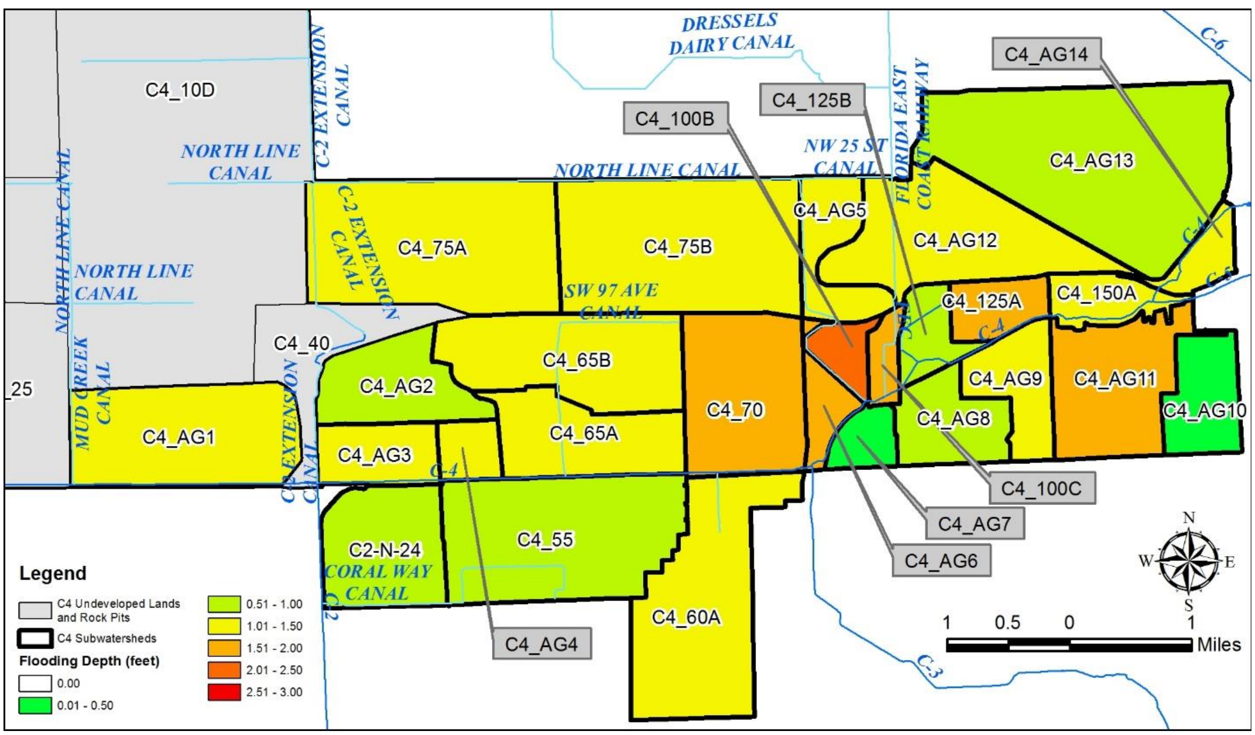

Figure H1-12. Flooding in the C4 Watershed with SLR2 Sea Levels: 100-year design storm and 100-year storm surge.

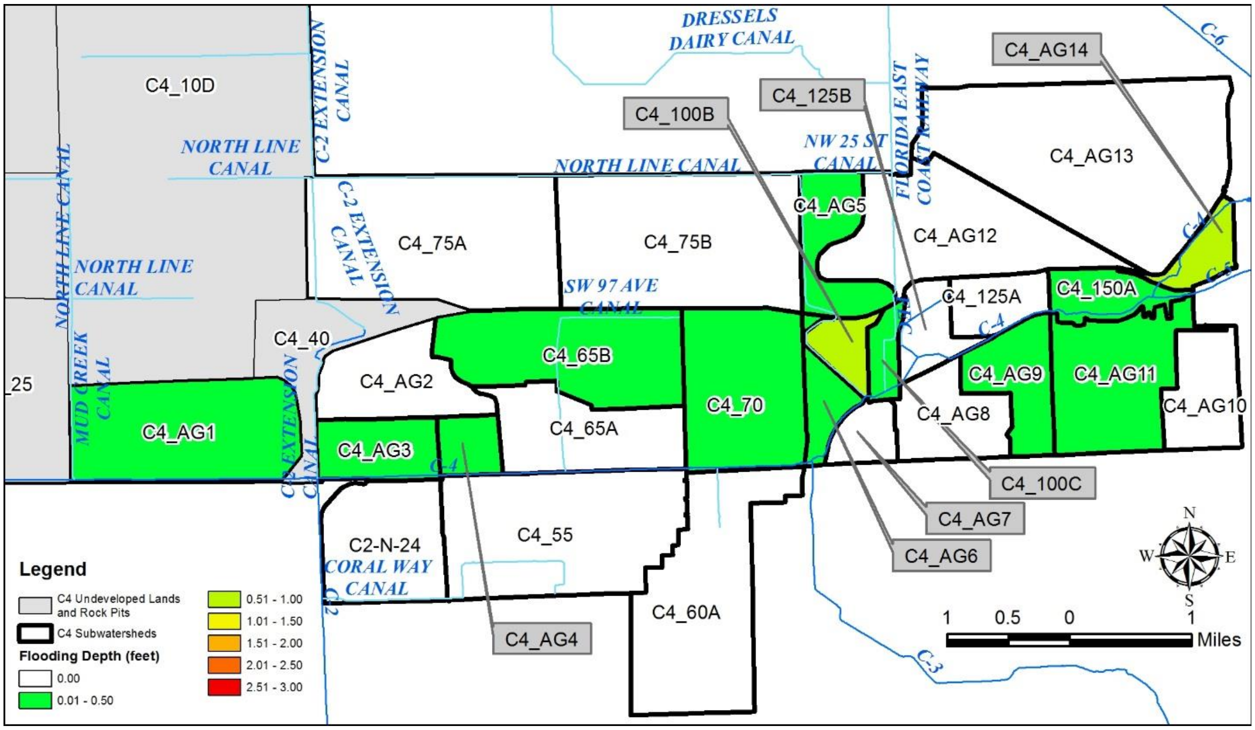

Figure H1-13. Flooding in the C4 Watershed with SLR3 Sea Levels: 5-year design storm and 5-year storm surge.

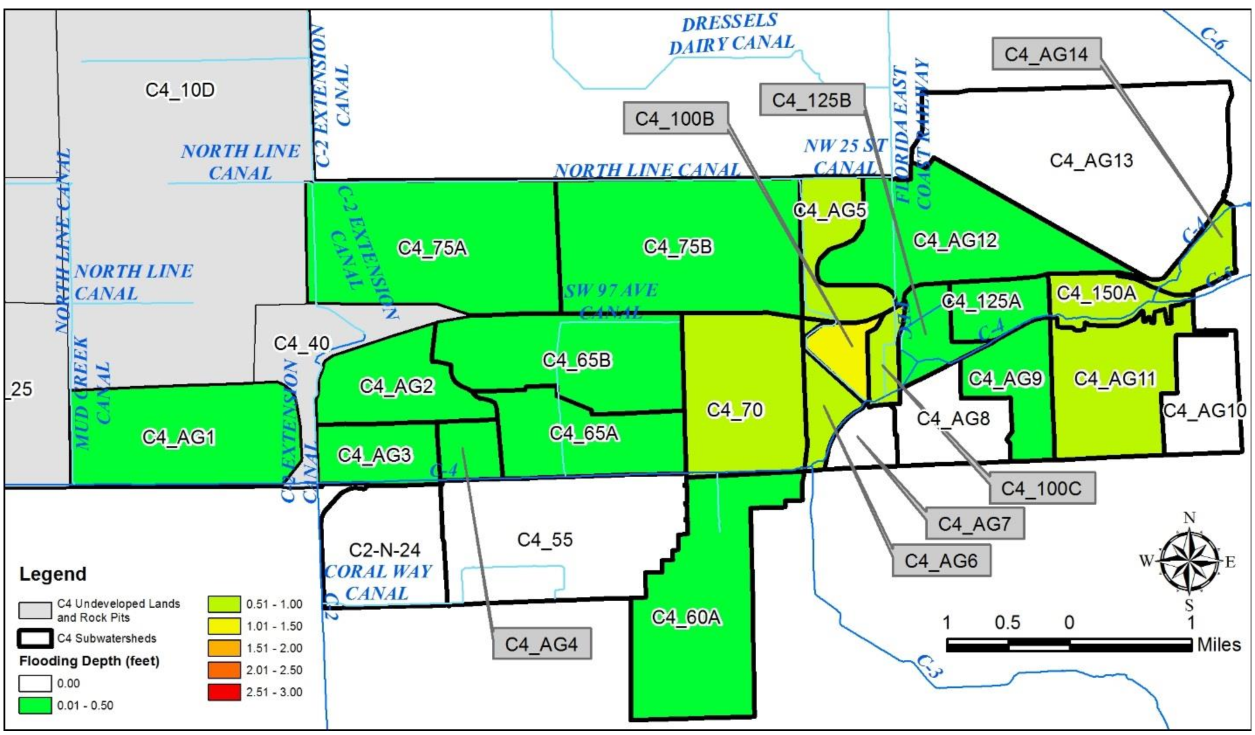

Figure H1-14. Flooding in the C4 Watershed with SLR3 Sea Levels: 10-year design storm and 10-year storm surge.

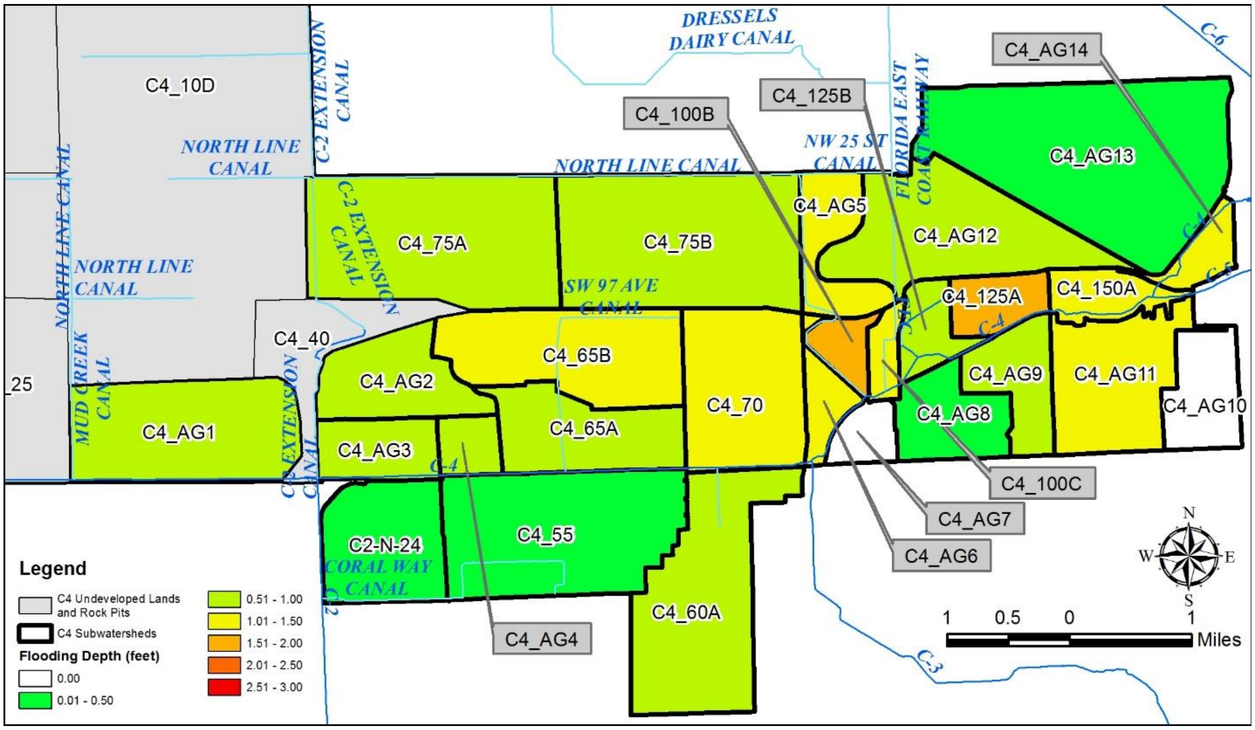

Figure H1-15. Flooding in the C4 Watershed with SLR3 Sea Levels: 25-year design storm and 25-year storm surge.

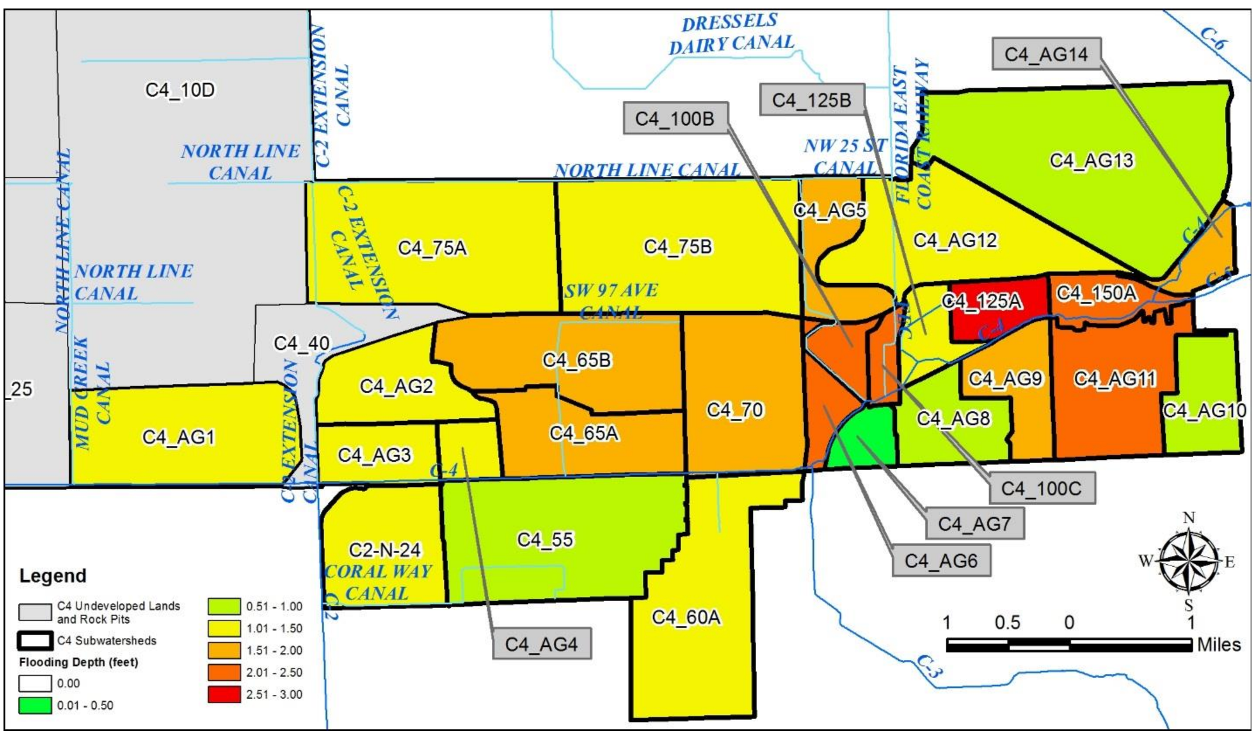

Figure H1-16. Flooding in the C4 Watershed with SLR3 Sea Levels: 100-year design storm and 100-year storm surge.

Observations:

CSL (2005 sea levels): minor flooding with 10-year storm and 25-year storm is problematic (i.e. sub-watersheds with >1 foot flooding)

SLR1 (CSL+0.45 feet): Similar to CSL. Minor flooding with 10-year storm but 25-year storm is problematic.

SLR2 (CSL+0.91 feet): minor flooding with 5-year storm and 10-year storm is problematic. Flood depths are greater everywhere but the rise is not enough to change flood categories in most sub-watersheds. However, two basins are now in the major flood category; these are near the structure and are result of backwater from storm surge.

SLR3 (CSL+2.37 feet): Flood depths are greater everywhere and the rise is enough for many sub-watersheds to jump into a deeper flood category. Even 5-year storm is problematic. Eight of sub-watersheds are in the major flood category. These are all upstream of the structure and, as with SLR2 are the result of backwater from storm surge.

ATTACHMENT H2: DATA FOR PM#5

Table H2-1 shows the raw data used in creating performance measure #5 for all sub-watersheds, comparing current sea level and the three alternative scenarios, SLR1, SLR2, and SLR3.

Table H2-1. Data for PM#5: Maximum stage in C-4 canal sub-watersheds for current sea level and increase in stage and duration for SLR1, SLR2, and SLR3 and 5-yr, 10-yr, 25-yr and 100-yr events

1-in-5-Year Storm

Grp	SUB-BASIN	Current max stg(ft NGVD)	SLR1		SLR2		SLR3	
			depth incr. (ft)	dura-tion (hr)	depth incr. (ft)	dura-tion (hr)	depth incr. (ft)	dura-tion (hr)
1	C4_10A	7.18	0.06	234	0.06	263	0.11	457
1	C4_10B	5.86	0.14	157	0.17	178	0.27	321
2	C4_10E	6.20	0.15	198	0.20	241	0.38	504
2	C4_10C	5.73	0.19	197	0.21	219	0.29	371
2	C4_10D	5.67	0.20	79	0.24	94	0.36	217
2	C4_40	5.70	0.20	16	0.28	22	0.67	48
4	C4_25	5.74	0.23	123	0.29	143	0.38	288
4	C4_AG1	5.83	0.24	6	0.37	13	0.63	40
5	C4_75A	5.76	0.16	16	0.21	21	0.68	42
5	C4_75B	5.18	0.39	15	0.68	22	1.26	51
5	C4_AG2	6.04	0.11	5	0.19	8	0.46	21
5	C4_65B	6.02	0.30	52	0.40	66	0.65	122
6	C4_AG3	5.78	0.23	12	0.40	16	0.75	39
6	C4_AG4	6.00	0.36	5	0.53	12	0.70	26
6	C4_65A	5.76	0.28	13	0.50	18	0.94	36
6	C4_70	5.46	0.33	17	0.60	23	1.13	44
7	C2-N-24	5.66	0.26	16	0.38	22	0.75	47
7	C4_55	5.80	0.27	14	0.48	19	0.93	36
7	C4_60A	5.74	0.30	13	0.51	17	0.98	35
8	C4_AG5	5.18	0.39	17	0.68	23	1.26	51
8	C4_AG12	6.24	0.09	11	0.14	12	0.29	16
8	C4_AG13	4.89	0.22	6	0.76	12	1.52	63
8	C4_100B	5.17	0.39	15	0.68	23	1.26	51
8	C4_100C	5.15	0.39	15	0.67	23	1.28	51
8	C4_125A	4.11	0.36	70	0.64	111	1.28	438
8	C4_150A	4.15	0.31	27	0.68	46	1.54	355
8	C4_AG6	5.21	0.35	16	0.64	23	1.22	50
9	C4_AG7	5.45	0.33	14	0.59	20	1.12	42
9	C4_AG8	5.93	0.34	4	0.56	12	0.97	28
9	C4_AG9	5.59	0.16	11	0.42	17	0.99	40
9	C4_AG11	5.59	0.15	10	0.41	16	1.01	38
9	C4_AG10	6.77	0.16	9	0.34	13	0.67	23
9	C4_AG14	4.35	0.26	32	0.54	57	1.18	336

1-in-25-Year Storm

Grp	SUB-BASIN	Current max stg(ft NGVD)	SLR1		SLR2		SLR3	
			depth incr. (ft)	dura-tion (hr)	depth incr. (ft)	dura-tion (hr)	depth incr. (ft)	dura-tion (hr)
1	C4_10A	7.62	0.06	233	0.07	256	0.12	365
1	C4_10B	6.33	0.14	160	0.18	177	0.32	271
2	C4_10E	6.73	0.02	92	0.02	104	0.12	185
2	C4_10C	6.23	0.18	195	0.21	214	0.35	319
2	C4_10D	6.35	0.24	62	0.29	77	0.50	142
2	C4_40	6.82	0.19	17	0.28	22	0.54	39
4	C4_25	6.32	0.13	82	0.18	103	0.36	184
4	C4_AG1	6.79	0.17	20	0.24	25	0.42	45
5	C4_75A	6.64	0.20	18	0.33	23	0.72	41
5	C4_75B	6.63	0.21	17	0.34	23	0.73	41
5	C4_AG2	6.82	0.19	17	0.28	22	0.54	40
5	C4_65B	7.06	0.20	51	0.27	64	0.47	108
6	C4_AG3	6.87	0.20	19	0.29	24	0.49	40
6	C4_AG4	7.01	0.18	19	0.27	24	0.50	38
6	C4_65A	7.15	0.25	14	0.38	17	0.58	28
6	C4_70	6.85	0.19	19	0.31	25	0.63	43
7	C2-N-24	6.85	0.19	17	0.28	22	0.54	39
7	C4_55	7.19	0.26	14	0.40	17	0.60	27
7	C4_60A	7.18	0.27	14	0.40	16	0.61	26
8	C4_AG5	6.63	0.21	17	0.34	23	0.73	41
8	C4_AG12	6.68	0.14	14	0.27	21	0.67	38
8	C4_AG13	6.41	0.34	4	0.49	6	0.95	21
8	C4_100B	6.62	0.21	17	0.34	23	0.73	40
8	C4_100C	6.58	0.21	17	0.36	22	0.77	40
8	C4_125A	5.11	0.36	67	0.59	92	1.82	253
8	C4_150A	5.25	0.33	30	0.60	44	1.34	106
8	C4_AG6	6.62	0.21	17	0.34	23	0.73	40
9	C4_AG7	6.92	0.28	15	0.44	18	0.81	31
9	C4_AG8	7.49	0.21	8	0.31	12	0.57	19
9	C4_AG9	6.73	0.20	14	0.34	19	0.76	37
9	C4_AG11	6.73	0.23	13	0.40	16	0.90	32
9	C4_AG10	8.11	0.19	11	0.33	14	0.65	22
9	C4_AG14	5.35	0.26	39	0.43	60	0.93	152

1-in-10 Year Storm

Grp	SUB-BASIN	Current max stg(ft NGVD)	SLR1		SLR2		SLR3	
			depth incr. (ft)	dura-tion (hr)	depth incr. (ft)	dura-tion (hr)	depth incr. (ft)	dura-tion (hr)
1	C4_10A	7.34	0.06	231	0.07	261	0.12	397
1	C4_10B	6.06	0.13	151	0.15	165	0.26	283
2	C4_10E	6.43	0.14	202	0.18	247	0.29	466
2	C4_10C	5.93	0.18	195	0.19	214	0.29	341
2	C4_10D	5.92	0.22	78	0.26	91	0.41	186
2	C4_40	6.03	0.21	16	0.37	22	0.83	43
4	C4_25	6.06	0.12	65	0.13	90	0.24	186
4	C4_AG1	6.28	0.21	12	0.30	15	0.50	35
5	C4_75A	6.00	0.22	15	0.39	20	0.82	39
5	C4_75B	5.85	0.36	16	0.53	22	0.97	43
5	C4_AG2	6.29	0.14	1	0.24	14	0.57	31
5	C4_65B	6.52	0.22	49	0.28	62	0.48	113
6	C4_AG3	6.25	0.27	12	0.35	16	0.65	34
6	C4_AG4	6.60	0.10	9	0.17	14	0.40	30
6	C4_65A	6.35	0.31	13	0.46	17	0.81	31
6	C4_70	6.09	0.38	17	0.53	22	0.85	41
7	C2-N-24	6.11	0.24	15	0.38	20	0.77	39
7	C4_55	6.37	0.31	14	0.48	17	0.83	32
7	C4_60A	6.34	0.35	13	0.51	17	0.85	31
8	C4_AG5	5.85	0.36	16	0.53	21	0.97	43
8	C4_AG12	6.42	0.10	2	0.16	3	0.39	25
8	C4_AG13	5.42	0.60	8	0.89	12	1.37	39
8	C4_100B	5.84	0.36	16	0.53	21	0.98	43
8	C4_100C	5.80	0.37	16	0.55	21	1.01	44
8	C4_125A	4.52	0.36	67	0.62	100	1.23	356
8	C4_150A	4.59	0.39	32	0.72	46	1.47	251
8	C4_AG6	5.84	0.36	16	0.53	21	0.98	43
9	C4_AG7	6.07	0.35	15	0.52	19	0.96	37
9	C4_AG8	6.59	0.33	11	0.50	14	0.84	26
9	C4_AG9	6.03	0.33	14	0.55	19	0.90	39
9	C4_AG11	6.01	0.33	13	0.55	17	0.99	37
9	C4_AG10	7.34	0.23	11	0.37	14	0.61	22
9	C4_AG14	4.75	0.29	39	0.57	59	1.07	238

1-in 100 year Storm

Grp	SUB-BASIN	Current max stg(ft NGVD)	SLR1		SLR2		SLR3	
			depth incr. (ft)	dura-tion (hr)	depth incr. (ft)	dura-tion (hr)	depth incr. (ft)	dura-tion (hr)
1	C4_10A	8.06	0.07	188	0.08	213	0.14	299
1	C4_10B	6.90	0.17	123	0.21	137	0.37	208
2	C4_10E	7.13	0.14	37	0.19	43	0.36	76
2	C4_10C	6.81	0.16	173	0.19	187	0.36	256
2	C4_10D	7.13	0.14	37	0.18	43	0.36	76
2	C4_40	7.56	0.11	14	0.14	17	0.38	34
4	C4_25	7.00	0.21	39	0.28	55	0.48	94
4	C4_AG1	7.45	0.10	12	0.13	13	0.23	43
5	C4_75A	7.48	0.14	13	0.20	17	0.62	33
5	C4_75B	7.48	0.13	13	0.20	17	0.63	33
5	C4_AG2	7.56	0.11	14	0.15	17	0.38	34
5	C4_65B	7.82	0.12	60	0.16	71	0.29	116
6	C4_AG3	7.56	0.11	14	0.15	17	0.38	34
6	C4_AG4	7.73	0.09	14	0.12	18	0.27	36
6	C4_65A	8.00	0.12	12	0.15	14	0.33	23
6	C4_70	7.65	0.11	16	0.16	21	0.49	41
7	C2-N-24	7.61	0.11	11	0.15	14	0.36	30
7	C4_55	8.08	0.13	12	0.17	14	0.37	21
7	C4_60A	8.07	0.12	12	0.16	13	0.36	21
8	C4_AG5	7.48	0.13	13	0.20	17	0.63	33
8	C4_AG12	7.46	0.14	13	0.21	17	0.64	33
8	C4_AG13	7.38	0.22	3	0.29	5	0.74	16
8	C4_100B	7.47	0.14	13	0.20	17	0.64	33
8	C4_100C	7.45	0.14	13	0.21	17	0.65	32
8	C4_125A	5.99	0.44	79	1.15	112	2.04	175
8	C4_150A	6.10	0.32	28	0.51	37	1.35	81
8	C4_AG6	7.47	0.14	13	0.20	17	0.64	33
9	C4_AG7	7.97	0.21	12	0.28	13	0.61	20
9	C4_AG8	8.44	0.14	7	0.17	9	0.38	15
9	C4_AG9	7.57	0.16	14	0.23	17	0.68	37
9	C4_AG11	7.68	0.21	14	0.32	17	0.90	31
9	C4_AG10	9.27	0.18	11	0.25	13	0.57	22
9	C4_AG14	6.07	0.21	42	0.34	59	0.86	138

ATTACHMENT H3: THRESHOLD ELEVATION AND AREA FOR SUB-WATERSHEDS

Table H3-1. Threshold Elevation and Area for each Sub-watershed in C4 Watershed

SUBWATERSHED	TOTAL AREA (ACRES)	DEVELOPED AREA (ACRES)	THRESHOLD ELEVATION (FEET NGVD29): 20% OF AREA FLOODED
C4_10A	13,576	4,328	5.97
C4_10B	8,657	1,711	3.57
C4_10E	1,233	383	2.82
C4_10C	4,928	1,408	2.67
C4_10D	5,738	3,031	2.27
C4_25	994	412	4.67
c4_n_3	432	0	*2.50*
c4_n_4	406	0	*2.50*
C4_40	440	287	4.87
C4_AG1	993	894	6.42
C4_75A	1,379	1,114	6.67
C4_75B	1,371	928	6.62
C4_AG2	503	465	6.82
C4_65B	880	625	6.52
C4_AG3	319	310	6.52
C4_AG4	156	152	6.52
C4_65A	590	257	6.82
C4_70	824	576	6.22
C2_N_24	610	496	6.92
C4_55	1,219	1,147	7.57
C4_60A	1,109	1,018	7.17
C4_AG5	326	303	6.27
C4_AG12	967	963	6.62
C4_AG13	1,786	1,791	7.12
C4_100B	140	63	5.62
C4_100C	120	107	6.07
C4_125B	208	112	3.57
C4_125A	225	167	*5.40*
C4_150A	236	136	*5.40*
C4_AG6	167	117	6.07
C4_AG7	155	154	8.17
C4_AG8	384	383	7.92
C4_AG9	349	357	6.52
C4_AG11	697	712	6.42
C4_AG10	314	354	9.02
C4_AG14	165	132	5.02

ATTACHMENT H4: COMPARISON OF LOS 100-YEAR CURRENT CONDITION FLOODING TO FEMA FIRM FLOODING

The C4 LOS model results for 100-year design storm and 100-yr surge under current sea level condition were qualitatively compared with the FEMA floodplain mapping results from the most recent Flood Insurance Study in the Miami-Dade County (Revised September 11, 2009). The latest FEMA floodplain was downloaded from http://espgis.com/FRIS (beta version). According to the disclaimer of the site, this product is under development and is still draft. Therefore, the comparison made in this attachment is generally qualitative, for information purposes, and does not imply that the District endorses the third party data.

The results from the LOS flood model are not identical to results from the FEMA FIS model, due to several factors that contribute to the differences. First is the tidal boundary. In the Miami-Dade County FIS study, the FEMA storm surge model was utilized to simulate the hydrodynamic behavior of the surge generated by various synthetic storms; whereas in the C4 LOS model, the 100-year storm surge was developed from a statistical analysis of tidal data and tailwater stage data at S25B, resulting in the development of a storm surge hydrograph for the S25B structure (see **Appendix C** for details). The LOS method generates higher tailwater stages at S25B than the FEMA FIS method. The difference in magnitude and timing of the surge boundary condition can impact the maximum flood depths throughout the watershed, especially near the tidal structure.

A second factor is the complexity of the models. The C4 LOS study focuses on the primary and the secondary canal systems. The model simulates the complex flood operational protocols, including municipal pumping, pumping into stormwater detention basins, forward pumping at S25B, and gate closures to prevent backwater flows. The conceptualization of the secondary and tertiary drainage netwark is simplistic, but includes groundwater-canal interactions and groundwater flows between the subwatersheds. In contrast, the FIS study simulates a substantially more detailed drainage system but has simplistic groundwater-canal interactions and only partial implementation of structural flood operations. The current FEMA FIRM model for the C4 does not include elements of the C4 Basin flood mitigation efforts.

A third factor is the interpretation of the simulation results. In contrast to the FEMA FIRM map, which bases flooding on variable groundwater elevations, the LOS model results show flooding as shaded areas (see **Figure H4-1**) whose flood depths are relative to a single 'threshold' ground elevation for each sub-watershed. At the threshold elevation, 20% of the developed land in the sub-watershed is at a lower elevation and 80% is at a higher elevation. This assumption is adequate for the LOS model, which is making relative comparisons to show the impact of sea level rise. It does not imply flooding over an entire area. For example, in the airport area, the 20% threshold elevation is below the elevations of buildings, facilities, runways or taxiways.

Given the above differences, the FEMA FIRM map largely agrees with the C4 LOS 100-year design storm and 100-yr surge under current sea level scenario (**Figure H4-1**). Both the FEMA floodplain and C4 LOS study results indicate somewhat widespread distribution of hazardous flooding in the developed areas. Both the FIRM study and the C4 LOS study results match in the two subwatersheds without major flooding issues, i.e. in C4_AG7 and C4_AG10, due partially to their higher elevations than the surrounding areas. One area where the two studies seem to differ is C4_AG13, which includes the Miami International Airport. In this area, the apparent difference is caused by choice of a threshold elevation that is not representative of the important features withn the area.

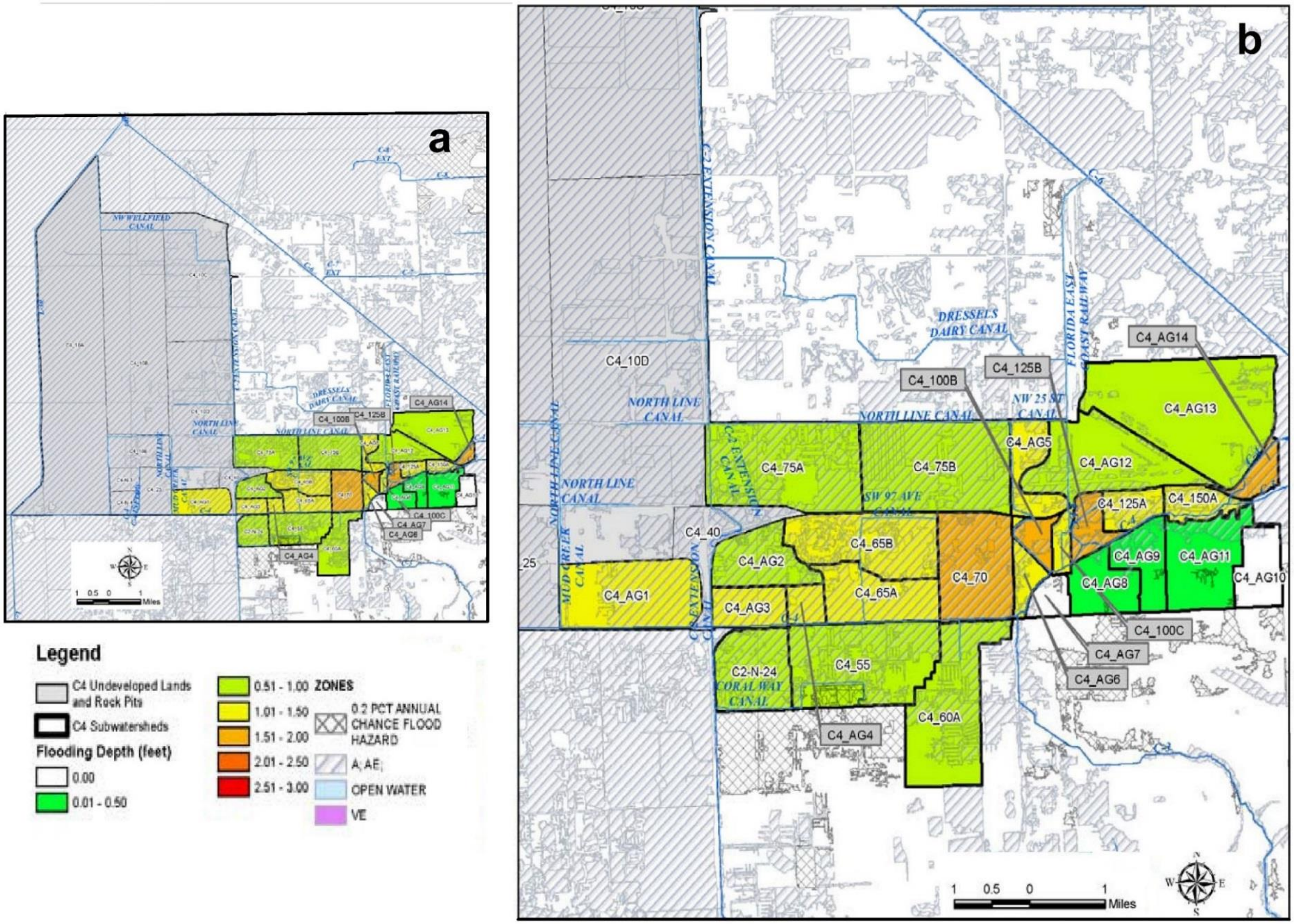

Figure H4-1 Comparison of FEMA Floodplain and C4 LOS 100-year design storm & 100-yr surge under current sea level