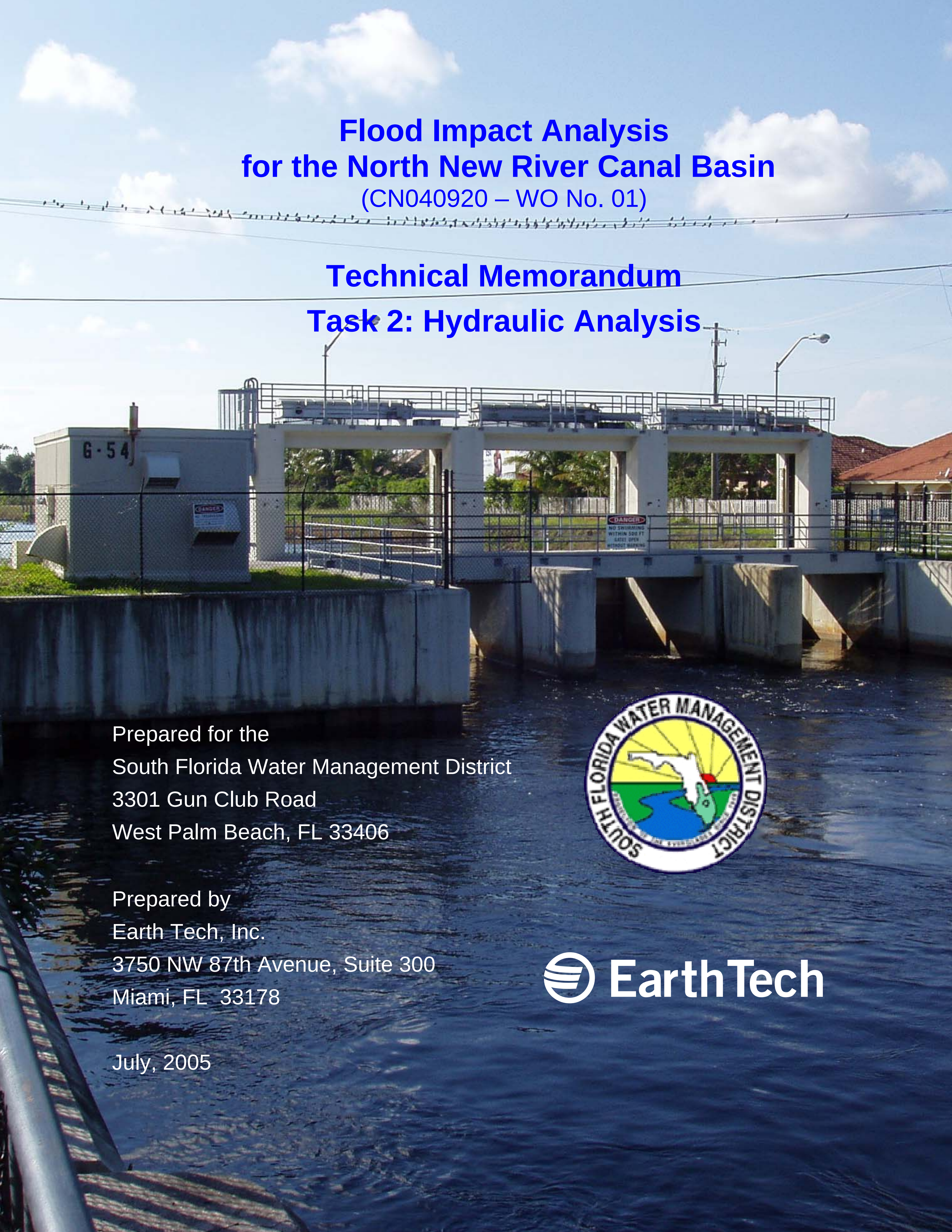
Flood Impact Analysis for the North New River Canal Basin

(CN040920 – WO No. 01)

Technical Memorandum

Task 2: Hydraulic Analysis



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1 Introduction

The 2003 Everglades Protection Area Tributary Basins Long-Term Plan for achieving Water Quality Goals recommends discontinuing the use of G-123 to pump runoff into the Water Conservation Area (WCA) 3A, other than as may be absolutely necessary for water supply emergencies. NNRC basin stakeholders have expressed concerns that discontinuing the use of the G-123 pump station may reduce flood protection in the basin.

Earth Tech has been contracted to evaluate the impact of the G-123 Pump Station operation on the flooding that occurs in the NNRC basin during storm events. For that purpose a screening-level XP-SWMM computer model is being used to simulate two recorded events and to assess flooding conditions under two scenarios for each storm: with and without the G-123 pump operation.

The NNRC Basin covers an area of about 19,000 acres (30 square miles) in eastern Broward County. The basin is located southeast of WCA 2B, west of the Florida Turnpike and north of Interstate Highway 595. The NNRC Basin is located immediately to the north of the C-11 West Basin, separated only by the NNRC. A map of the NNRC Basin is presented on Figure 1.

The project canals and control structures in the NNRC have four functions:

- To provide flood protection and drainage for the NNRC Basin
- To supply water to the basin during periods of low natural flow
- To convey excess water from Water Conservation Areas to tidewater
- To intercept and control seepage from WCA 2A

There are eight project control structures regulating flow in the NNRC Basin. The two major control structures are the Sewell Lock (G-54) and G-123. The Sewell Lock (G-54) is a spillway and lock structure along the NNRC and it regulates discharges from the NNRC to tidewater. G-123 is a pumping station located on the NNRC. During periods of regional drought, G-123 discharges water, which would be otherwise discharged to tidewater, from the NNRC into the WCA 3A.

The Long-Term Plan for Achieving EPA Water Quality Goals recommends that discontinuing the use of G-123 pump station is the most cost-effective means of diverting NNRC's stormwater runoff away from the Everglades. The plan also recognizes that a flood impact analysis must be performed to ensure the NNRC Basin's current level of flood protection will be maintained after discontinuing the use of G-123. The flood impact analysis will include the following tasks:

- Review and analysis of historical data
- Hydraulic analysis of selected storm events to evaluate the potential impacts of discontinuing the use of G-123 pump station during flood events
- Evaluations of alternatives capable of reducing or eliminating the negative impacts of discontinuing the use of G-123 pump station

The review and analysis of the available data pertaining to NNRC Basin, and the collection of the data needed to construct a hydraulic model of the NNRC Basin were reported in the memorandum summarizing the first task of this assignment. The memorandum was reviewed by the District and the stakeholders; it was then finalized and submitted to the District in February 2005.

This technical memorandum was prepared in accordance with Task 2 of Work Order CN040920-WO01 with the South Florida Water Management District (SFWMD). This memorandum documents the development of the XP-SWMM model for the North New River Canal (NNRC) basin and the simulations performed to evaluate the impact of the operation of the G-123 pump station on the flooding in the NNRC Basin.

2 Selection of Storm Events

2.1 Introduction

The pump station G-123 operation records were reviewed for a period of approximately twelve years, from January 1993 to November 2004. For the purpose of selecting the most significant storm events in recent history, these records were correlated to the hydrological records at the available gauging stations in the NNRC basin. From the results of the stage frequency analysis performed under Task 1, it is observed that over that period, the water level in the canal (tailwater level at G-123) exceeded El 5.31, the median yearly peak stage, in 19 occurrences. Data for these events are shown on Table 1.

Table 1 – Major Recent Storm Events over the NNRC Basin

Date	S-124 HW	S-124 TW	G-54 HW	S-125 TW	G-123 HW	G-123 TW	Average 3-day Rainfall		G-123 Operation
							(in)	Rank	
Nov 16, 1994	6.84	6.49	5.25	-	6.43	12.18	10.42	1	No
Dec 24, 1994	5.43	5.35	3.79	-	5.43	11.89	4.08	8	No
Nov 16, 1995	5.15	5.23	4.25	-	5.34	10.92	0.00	-	No
Nov 29, 1995	5.19	5.27	4.03	-	5.36	10.60	0.87	14	No
Mar 17, 1998	5.67	5.51	4.26	4.81	5.59	10.10	0.06	16	No
Apr 1, 1998	5.65	5.40	4.04	4.81	5.46	9.08	0.01	17	No
Apr 30, 1998	5.37	5.34	3.68	4.93	5.39	8.59	0.68	15	No
May 25, 1998	5.15	5.24	3.82	4.68	5.56	9.48	0.00	-	No
Sep 18, 1998	5.51	5.51	3.56	5.68	5.31	10.48	6.29	5	Yes
Nov 5, 1998	6.20	5.72	4.29	6.36	5.84	10.88	6.63	4	No
Jun 2, 1999	5.84	5.16	3.68	5.50	5.34	9.04	2.86	9	No
Jun 9, 1999	7.02	6.28	3.76	6.86	6.11	10.01	4.27	7	No
Jul 1, 1999	5.88	5.45	3.35	5.74	5.74	10.82	2.04	11	No
Aug 3, 1999	5.34	5.31	3.40	5.81	5.49	10.69	2.05	10	No
Aug 24, 1999	5.77	5.28	2.89	5.71	5.48	10.64	1.50	12	No
Oct 15, 1999	8.11	7.84	4.97	8.18	7.90	11.63	9.36	2	No
Jul 27, 2000	5.75	5.45	3.20	5.65	5.49	10.16	1.28	13	Yes
Oct 3, 2000	6.59	6.28	3.62	6.88	6.15	10.86	7.38	3	Yes
May 28, 2003	5.74	5.42	3.09	5.51	5.36	10.55	4.35	6	Yes

2.2 Storm selection

Based on Technical Publication EMA#390, the one-day and three-day 5-year storms are 6.0 and 8.0 inches respectively, and the one-day and three-day 10-year storms are 7.5 and 10.0 inches respectively. From Table 1, only four storms correspond to recorded rainfall events with a return period in excess of 5-year. The average 3-day rainfall is taken as the arithmetic average of the recorded rainfall at the three gauges within the NNRC basin, i.e., S-125, S-124 and G-54. A simple arithmetic average has been used, as it is a close approximation of the contributing area calculated using the Thiessen polygon methodology.

The G-123 pump station was operated during two of these four events: September 18, 1998 and October 3, 2000 (No-Name Storm). The records also show that for the two largest rainfall events, November 16, 1994 and October 15, 1999 (Hurricane Irene), the G-123 pump station was not in operation; during these events the stage in WCA 3A was above El. 11.50.

Based on the review of the major recent storm events and the availability of pumping records in the NNRC basin, and in consultation with the District, the following two events were selected for modeling:

- October 15, 1999 (Hurricane Irene); and,
- October 3, 2000 (No-Name Storm).

The 15-minute records for the above selected storm events were obtained and synthesized in MS-Excel format. The duration of records for each storm event was extended to 10 days. The SFWMD provided the 15-minute interval records for the two storm events (October 13, 1999 to October 23, 1999, and September 30, 2000 to October 9, 2000), for the following stations:

- Headwater and tailwater stages for structures G-123, S-34, S-124, S-125 and G-54
- Precipitation at S-124, S-125 and G-54
- Discharges at pump station G-123, and structures S-34, S-124, S-125 and G-54
- Gate opening at structure G-54.

These records are graphically presented on Figures 2 to 9.

3 XP-SWMM Model for the NNRC Basin

3.1 Introduction

The principal objective of the NNRC Flood Impact Analysis is to perform a hydraulic analysis of selected storm events to evaluate the potential impacts of discontinuing the use of the G-123 pump station during flood events. The hydraulic analysis requires construction of an XP-SWMM model of the NNRC and C-42 Canal using readily available information. The model extends along the NNRC from G-54 upstream to the to the G-123 pump station, and along the C-42 Canal from S-125 structure downstream to its confluence with the NNRC. Task 1 Technical Memorandum, which summarized the data collection and analysis effort, concluded that the available data is sufficient to construct a screening-level model. Figure 10 presents a schematic of the model domain and inflow points.

The following paragraphs describe the setup of the hydraulic network of the XP-SWMM model of the NNRC, hydrologic calculations and parameters, and flow routing calculations.

3.2 Software Description

The model used for this study was XP-SWMM2000 version 9.50. XP-SWMM (Stormwater Management Model) is based on the original EPA SWMM. SWMM is a very comprehensive urban hydrology model that is widely used and accepted in South Florida. It applies links and nodes computational concept to execute hydrology, hydraulics and water quality analysis of stormwater and wastewater systems.

Nodes symbolize the junction of hydraulic links and also function as a location for input of flow into the drainage system. A node can represent a storage device such as a pond or lake, a point junction representing a point of change in channel geometry, a boundary condition in the model or a watershed in runoff. Links represent hydraulic elements for flow and element transport through the system that is modeled. Examples of information stored in links include pipes, channels, pumps, orifices and weirs.

XP-SWMM has three modules. The first module, called the Runoff Module, is a stormwater module for hydrology and water quality. It creates surface runoff and subsurface flow based on rainfall hyetographs, antecedent conditions, land use, soils, hydraulic properties and topography.

The second module is the transport module, called Sanitary Module that produces wastewater flows including water quality routing and treatment. This module has not been used in the present study.

The final module is the Hydraulics Module for hydraulic simulation of open and closed conduit including canals and culverts.

3.3 Hydraulic Network

The NNRC hydraulic network extends approximately 13.6 miles along the NNRC from G-123 pump station upstream to the G-54 structure, and approximately 3.7 miles along the C-42 canal from the S-125 structure upstream to its confluence point with the NNRC. All vertical elevations are referenced to National Geodetic Vertical Datum (ft-NGVD). The model network includes twelve (12) bridges, two culverts, and three control structures. The system also features twenty inflow points representing runoff flows from several municipalities and adjacent highways. The model was constructed using information previously identified and reported in the Data Review Task. The model network consists of 96 nodes and 113 links.

River cross-sections are a key element in developing the NNRC hydraulic model. The most recent NNRC cross-section survey was conducted on June 1979 at 32 locations from the confluence of NNRC with L-35A canal to the G-54 structure. The only C-42 cross-section survey found in the data review was conducted in February 1951 at four (4) locations between S-125 structure and the confluence with the NNRC. For the purpose of this analysis, a survey of the centerline profile along NNRC between G-123 and G-54 structures, and along the C-42 canal, between S-125 structure and the confluence with the NNRC, was conducted in November 2004 by the SFWMD.

The initial setup of the canal cross-sections used the 1979 survey along the NNRC and the 1951 survey along the C-42 canal. The NNRC cross-section between G-123 and S-124 structure was initially represented by the 1979 cross-section survey around S-124 structure, and the aerial maps showing the canal width. The canal cross-sections were then refined using the 2004 centerline profiles to incorporate changes in the canal cross-sections resulting from sediment accumulation between the time the cross-section survey was conducted and 2004. In particular, it was noted in the Task 1 Technical Memorandum, that the 2004 river profile showed that the approximately 12,000-ft long reach between Pine Island Bridge and Sewell Lock (G-54) has accumulated a significant volume of sediment, or vegetation, which may result in additional head losses along that reach.

The NNRC and C-42 canals are modeled as 36 and 21 conveyance links, respectively. Tables 2 and 3 below show the lengths of theses links.

Table 2 - NNRC Conveyance Links

Link Name	Length (ft)	Link Name	Length (ft)	Link Name	Length (ft)
L001	3,378	L014	1,216	L031	1,338
L002	4,076	L015	3,209	L032	2,650
L003	2,550	L016	1,204	L034	1,850
L004	3,044	L017	575	L035	3,361
L005	2,773	L019	1,269	L037	730
L006	2,994	L020	2,090	L038	3,220
L007	1,847	L022	756	L039	1,780
L008	3,692	L023	450	L041	2,850
L009	2,220	L024	750	L042	2,115
L010	1,426	L026	2,685	L044	413
L011	874	L028	1,050	L046	2,875
L013	560	L030	1,500	L047	3,484

Table 3 - C42 Canal Conveyance Links

Link Name	Length (ft)	Link Name	Length (ft)	Link Name	Length (ft)
C42-L001	675	C42-L010	1,543	C42-L017	476
C42-L003	1,470	C42-L011	840	C42-L018	1,566
C42-L004	414	C42-L012	836	C42-L019	293
C42-L005	592	C42-L013	936	C42-L021	302
C42-L006	1,182	C42-L014	640	C42-L022	359
C42-L007	927	C42-L015	1,534	C42-L023	1,578
C42-L008	1,422	C42-L016	470	C42-L024	1,112

The NNRC model includes twelve bridges along the NNRC and C-42 canal. Bridges were simulated in the model as a cross-section with piles and spacing, and entrance and exit losses. The right of way and cross-section information found in the SFWMD permits were used to construct those bridges in the model.

Table 4 - Bridges and Culverts

Bridge/Culvert	Piers/Bents	Bridge/Culvert	Piers/Bents
North New River Canal			
Markham Park	2 bents	Hiatus Road	4 bents
Stiles Corp.	2 bents	Nob Hill Road	4 bents
SW 136th Avenue	4 bents	Pine Island Avenue	3 bents
Commodore Drive	4 bents	I-595 Ramp	1 pier
SW 125th Avenue	4 bents	University Drive	4 bents
Flamingo Road	4 bents		
C-42 Canal			
Broward Blvd	10'-2" by 15'-7" Arch	W Sunrise Blvd	10'-2" by 15'-7" Arch
NW 29th Street	2 bents		

The NNRC model includes two culverts along the C-42 canal at Sunrise Blvd and 29th Street bridges. The NNRC data review identified culvert design information for the Sunrise Blvd culvert. This culvert information was used to simulate both Sunrise Blvd and 29th Street culverts. The two culverts were simulated as arch pipe 15.7 feet wide by 10.2 feet high. The approximate locations of bridges and culverts were determined from the aerial photographs.

3.4 Hydrology Calculations

3.4.1 General

The NNRC model includes runoff contributions from drainage districts, private properties, adjacent highways and the canal right-of-way itself. The runoff enters the system by either gravity discharge or via a pump station. The NNRC basin model was simulated with 14 gravity basins, totaling approximately 5 square miles and 5 pumped basins, equipped with 12 pump stations having capacities ranging from 45,000 gpm to 200,000 gpm. Simulation techniques for each type of basin are described in the following sections.

3.4.2 Gravity Basins

The Runoff Module available in the XP-SWMM model performs runoff calculations for drainage basins. The runoff non-linear reservoir routing method was used to simulate hydrographs for the NNRC basin because it allows simulation of groundwater and infiltration which are crucial components of the hydrologic cycle in NRRC basin, and because it is a physically based method that allows flexibility in adjusting the model.

Detailed meteorological data and surface characteristics were entered as input to the Runoff Module to generate a runoff hydrograph from each of the sub-basin areas. Surface characteristics define each sub-basin and provide information necessary to estimate and route runoff flows. Characteristics such as drainage area, land slope, width of overland flow and Manning's surface roughness coefficient were used to determine the velocity and volume of overland runoff flow. In addition, soil infiltration, groundwater parameters, depression storage, and percent imperviousness were used to calculate the volume of runoff flow.

The flow in the NNRC and C-42 canal is driven by rainfall in the NNRC basin and upstream watersheds. Precipitation data were recorded at the S-124, S-125, and G-54 structures at 15-minute intervals. This network of three rain gauges provided spatial coverage of the project area. Rain gauges were assigned to sub-basins on the basis of proximity. Evaporation rates are subtracted from rainfall intensities at each time step and are also used to replenish depression storage and provide an upper bound for soil moisture and groundwater evaporation. As the evaporation rates are temperature dependant and vary seasonally, monthly rates were used in the model. Evaporation rate of 0.15 inch/day were assigned for the month of October when the two selected events occurred. The average potential evaporation was estimated using long-term pan evaporation daily records from the National Weather Service gauge in Hialeah, FL for the period 1948 to 1994.

Basin drainage areas were determined from available Surface Water Management (SWM) permits. For areas that were identified during data review and were missing SWM permits, GIS coverage and aerial photographs were used to estimate the drainage areas with confirmation from the local drainage districts. The width of the drainage area is defined as the ratio of the basin area to the average length of the overland flow. Percent imperviousness values were determined from available SWM permits. For areas that were missing percent imperviousness information, aerial photographs were used to estimate the percent imperviousness initial values. Depression storage is a volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas. Initial value of 0.1 inch was assigned for both pervious and impervious areas. Runoff simulation also requires input of Manning's coefficient and impervious area percent-of-zero-detention, which is a percent of the impervious area that is assigned zero depression storage in order to promote immediate runoff. Typical Manning's coefficients of 0.014 and 0.03 were assigned for impervious and pervious areas, respectively. A typical value of 25 percent was assigned for percent-of-zero-detention for impervious areas.

Computations of the gravity runoff flows were simulated in the Runoff Module. The runoff flow routing and the surface water management were simulated in the Hydraulics Module using available depth-area-volume-discharge curves for the management facilities. The size of the storage facilities, such as lakes and canals, were determined from available SWM permits. For areas without SWM permits, the surface areas were estimated from aerial photographs. A summary of the hydrologic characteristics of the gravity basins is presented in Table 5.

Table 5 - Hydrologic Characteristics of the Gravity Basins

Basin Name	Area (acres)	Lake Area at Normal Level (acres)	Percent Impervious
Sunrise Basin No7	293	27	67%
Markham Park	665	144	38%
Mobile-home Park	286	14	60%
Shenandoah	648	55	60%

The Shenandoah basin consists of 648 acres of mostly residential land. The drainage system consists of a system of lakes interconnected by culverts that provide flood protection. The chain of lakes has two outlets: one to the north, which overflows into the NNRC, the other outlet discharges to the south into N-30 canal. The outlet structure to the NNRC consists of a concrete weir measuring 5 feet wide by 2.5 feet high with a crest at 5.0 NGVD. Following Hurricane Irene, when extreme high water levels in the NNRC were observed to backflow into the Shenandoah basin, a flap gate was installed at this structure to prevent backflow.

The XP-SWMM model simulates this condition during the Hurricane Irene, by allowing water to initially discharge from the NNRC basin into the Shenandoah lakes as the water level in the NNRC rises faster than in the lakes. As flood levels recede, runoff stored into these lakes can discharge into the NNRC. For the purpose of this modeling, a conservative approach to flooding in the NNRC was adopted by not allowing discharges through the southern outlet of the Shenandoah basin. For the No-Name storm, the NNRC basin did not contribute runoff to Shenandoah lakes as a flap gate was installed after Hurricane Irene.

The surface water management for the I-595 and I-75 highways includes grass swales, seepage ponds and culvert that are designed to capture the first inch of the storm. The grass swales were simulated as a storage node with an orifice and a weir to discharge the excess runoff flow. Direct rainfall on the canals and their right-of-way was accounted for by introducing six gravity basins assumed to have 90% impervious area and a short 50 feet of overland flow. Adjacent highways and direct runoff contribution are summarized on Table 6.

Table 6 - Hydrologic Characteristics of Highway and Canal Basins

Basin Name	Area (acres)	Percent Impervious
I-595	173	60%
I-595 – NNRC	287	56%
I-75	156	60%
I-75 – I-595	135	50%
I-75 – US27	119	50%
NNRC	344	90%
C-42 Canal	70	90%

3.4.3 Pumped Basins

Pumping records for the two selected storms were available for the pump stations in the Old Plantation Water Control District, Bonaventure Drainage District, and City of Sunrise Basin 8. The runoff flows from these areas were simulated in the Hydraulics Module and the pumping records were entered as user defined flows. The pumping capacities for these districts are indicated on Table 7 below.

Table 7 - Pumping Capacity for Water Districts with Available Records

Pump Station	Estimated Basin Area (acres)	Discharge Capacity (cfs)
Bonaventure # 1	700(1)	122
Bonaventure # 2	550(1)	100
Sunrise # 8	2,540	444
Old Plantation # 1	n.a.	400
Old Plantation # 2	n.a.	400

Note: (1) Basin area (1,250 acres) of the District estimated in proportion of the pump station discharge capacity.

The total volumes pumped for each of the selected storms indicated in the records are shown on Table 8 below. For the purpose of comparison Table 8 also shows the pumped volume in inches; for the Bonaventure and Sunrise basins the total precipitation measured at the nearest gauge, S-124 Structure, were 9.8 and 7.5 inches for Hurricane Irene and the No-Name Storm, respectively.

Table 8 – Pump Discharge Records

Pump Station	Estimated Basin Area (acres)	Hurricane Irene		No-Name Storm	
		Pumped Volume (ac-ft)	Pumped Volume (inch)	Pumped Volume (ac-ft)	Pumped Volume (inch)
Bonaventure # 1	700(1)	929(2)	15.9(2)	381	6.5
Bonaventure # 2	550(1)	540	11.8	671	14.6
Sunrise # 8	2,540	1,177	5.6	142	0.7
Old Plantation # 1	n.a.	1,587	n.a.	877	n.a.
Old Plantation # 2	n.a.	1,451	n.a.	901	n.a.

Note: (1) Basin area (1,250 acres) of the District estimated in proportion of the pump station discharge capacity.
(2) Including 375 ac-ft pumped from Oct 19 to Oct 26, 1999

The runoff from Plantation Acres Improvement District (PAID) is pumped into the C-42 canal via 6 pumps. The pump records for these PAID pumps were not available for the two selected storms. The PAID runoff flow was calculated in the Runoff Module using available surface characteristics. This district has a drainage area of 2,063 acres; for the purpose of the simulation, it was assumed that each pump was controlling one sixth of the total acreage. Percentages of impervious areas as estimated from aerial photographs are indicated on Table 9 below.

Table 9 - Hydrologic Characteristics of Plantation Acres Improvement District

Basin Name	Area (acres)	Percent Impervious
PAID 01	344	40%
PAID 02	344	45%
PAID 03	344	40%
PAID 04	344	50%
PAID 05	344	50%
PAID 06	344	35%

The pump capacities were estimated based on communications with the Plantation Acres District. The pumps have a combined capacity of approximately 150 cfs at normal water level, dropping to 50 cfs when the water surface elevation in the C-42 canal reaches 8.0 NGVD. Pump operations were simulated as per the permit conditions; an estimated storage node was added to the model to activate the pumps between stages 3.5 and 4.5 ft-NGVD.

The runoff from Lago Mar Country Club (318 acres) was simulated as pumped flow into the NNRC. The pump operation rules were found in the SWM permit. The Lago Mar runoff was computed in the Runoff Module and the flow routing and surface water management were simulated in the Hydraulics Module using the pump operation rules and available depth-area-volume curves for the management facilities. As indicated in the permit, the 100-cfs pumping facility begins operation when the water level in the lake reaches 5.75 ft-NGVD, until it is lowered to 5.5 ft-NGVD. The system is drained by gravity until the lake level reaches 4.5 ft-NGVD.

3.5 Hydraulic Calculations

The Hydraulics Module of XP-SWMM was used for flow routing calculations in the NNRC model. The Hydraulics Module solves the complete St. Venant dynamic flow equations throughout the drainage network which simulates backwater effects, flow reversal, surcharging, looped connections, pressure flow, hydraulic control structures, tidal boundaries, and real-time control. Simulation output contains time-varying water surface elevations and flow rates at selected locations.

The initial roughness coefficient used for the canals was 0.03 for the center banks and 0.06 for the left and right banks. The initial roughness coefficient used for the culverts and connection pipes was 0.014. Initial values were subsequently modified during model adjustment.

Seepage from the WCA 2B occurs along the upstream portion of the NNRC from the G-123 pump station to the S-124 structure. Seepage was simulated using a special conduit equation in Hydraulics Module and calculated as a function of a seepage factor (cfs/ft) and the head difference between the calculated NNRC elevation and the observed WCA 2B elevation. The WCA 2B was assigned as a boundary condition with daily recorded elevations.

The G-54 structure was simulated as a gate with upstream and downstream nodes. The gate was simulated as a time-variable orifice. The gate operation logs for the two storm events were used to mimic the actual operation (opening and closing) of the gates. The historical records for G-54 tailwater stage were used as the downstream boundary condition of the model.

3.6 Boundary Conditions

The boundary conditions were specified at upstream and downstream ends of the NNRC and C-42 Canal. The following is list of boundary points established in the NNRC model:

- S-124 structure on L-35A Canal (flow history)
- S-125 structure on C-42 Canal (flow history)
- G-123/S-34 structures on NNRC (flow history)
- G-54 structure on NNRC (gate opening and downstream stage histories)
- WCA 2B / NNRC Levee (stage history)

The time-discharge boundary conditions were assigned at the G-123 pump station to account for flow leaving the system to the WCA 3A. The time-discharge boundary conditions were assigned at S-34, S-124 and S-125 structures to account for flows entering the system from WCA 3A, L-35A canal and upstream C-42 canal, respectively. The time-stage boundary conditions were assigned at G-54 structure to control the flow leaving the system; this time-stage boundary is the tailwater stage record of structure G-54. The G-54 gate opening records were used to calculate discharge and upstream water stages at G-54. The time-stage boundary conditions were assigned at WCA 2B for calculations of seepage flow entering the NNRC canal from the WCA 2B. The DBHYDRO 15-minute data was assigned at the G-123, S-34, S-124, S-125, and G-54 boundary structures for the two rain events. For the purpose of estimating seepage from WCA 2B, daily stage was assigned at the WCA 2B boundary for the two rain events.

Initial conditions are the time-variable values at the start of the model simulation. The G-123 pump, at the NNRC upstream boundary, was operational during the October 2000 storm event and was withdrawing water from the NNRC into the WCA 3A but it was not in operation during Hurricane Irene. The S-125 gate, at the C-42 canal upstream boundary, was closed during the two selected storms.

The XP-SWMM has a feature that compares model results to gauged data. This feature requires input of the gauged data in a format readable by XP-SWMM, and referred to as “HIS” files. The measured flow data at the G-54 structure, measured tailwater at the S-124, S-125, and S-34/G-123 structures, and headwater at the G-54 structure were compiled for the two events and formatted into XP-SWMM format to be used for comparison with the simulation results.

4 "No-Name Storm" (October 3, 2000)

4.1 Baseline Conditions with Pump Station G-123 in Operation

Using the model of the NNRC basin presented in Section 3, the storm event of October 3, 2000 was simulated. In order to capture the flood event in full, the period of simulation extended from September 30 to October 5, or 144 hours.

Initial runs were used to estimate the seepage from the WCA 2B. There was a small amount of rainfall on the basin during the first two and one half days of the event amounting to 0.9 inch. During that time, the only known inflow entering the system occurs at Structure S-124 (approximately 55 cfs), and the G-123 pump station was discharging approximately 305 cfs continuously over that period. As the water surface level in the canal was fairly constant over that period, it was assumed that the deficit of approximately 250 cfs was compensated by a similar volume of seepage inflow through the levee of WCA 2B. The seepage factor for the WCA 2B levee was adjusted until the simulated canal water levels matched the observed. The resulting seepage factor was 5 cfs/mile/foot of head difference. Results show an excellent "fit" between observed and simulated water levels in the canal for the initial days (September 30 to October 2, 2000) of simulation (Figures 11 through 14).

During the peak of the storm, it appeared that the volume of water simulated in the system was significantly larger than observed, indicating an interaction between surface and groundwater. A groundwater component was introduced into the model. Records from monitored wells located in the basin, show the groundwater level at the time of the storm was approximately between 4.0 and 4.5 ft-NGVD. For the purpose of the simulation a groundwater level of 4.25 ft-NGVD was assumed. Using this groundwater component, a good "fit" between the observed and simulated discharge at Structure G-54 was achieved during the period of peak flooding on October 3 and 4, 2000 (Figure 15).

Finally, the model was adjusted for head losses along the canals. It is observed that a somewhat high Manning's coefficient of 0.045 was needed to fit reasonably well the various water levels measured in the canals. The coefficients of expansion and contraction used were 0.5 and 1.0, respectively.

The simulation results are graphically summarized on Figures 16 through 20, where the history of simulated water levels at structures G-123, S-124, S-125 and G-54 are compared to the observed values for the six-day period of simulation. Figure 25 presents a comparison of the simulated discharge through G-54 with the calculated values reported by the SFWMD. Table 12 below presents a comparison of the peak water levels reached at the various structures during this storm event.

Table 10 - No-Name Storm - Peak Water Stage

Structure	Observed Stage (ft-NGVD)	Simulated Stage (ft-NGVD)
G-123	6.56	6.64
S-124	6.70	6.70
S-125	6.88	6.90
G-54	4.50	4.55

4.2 Impact of the G-123 Pump Station Flooding in the NNRC Basin

Based on the excellent "fit" achieved between observed and simulated water levels in the canal system during the No-Name storm, it is expected that the model developed for the NNRC basin will be a good approximation of the basin behavior under flooding conditions. In order to assess the impact of the G-123 operation on flooding, the model was modified to eliminate the pumping for the duration of the simulation. To properly simulate the canal system under these conditions, the G-54 gates were operated in accordance with the recommended procedure to maintain the water level upstream of G-54 at 4.50 ft-NGVD.

The simulation results are graphically summarized on Figures 26 through 29, where the simulated water levels at structures G-123, S-124, S-125 and G-54 during the No-Name storm under the actual operating conditions are compared to the simulated values for the same storm without G-123 pump station operating. Figure 30 presents a comparison of the discharge through G-54. Table 13 below presents a comparison of the peak water levels reached at the various structures during this storm event.

Table 11 - No-Name Storm Model
G-123 Impact on the NNRC Basin Flooding

Structure	Peak Stage without G-123 Pumping (ft-NGVD)	Peak Stage with G-123 Pumping (ft-NGVD)	Difference (ft)
G-123	7.29	6.64	0.65
S-124	7.27	6.70	0.57
S-125	7.34	6.90	0.44
G-54	4.53	4.55	-0.02

5 Hurricane Irene (October 15, 1999)

5.1 Baseline Conditions without Pump Station G-123 in Operation

Using the model of the NNRC basin presented in Section 3, rainfall and pumping records for Hurricane Irene were incorporated into the simulation. For that event, the G-123 pump station was not in operation. Structure S-34, located adjacent to G-123, was in fact discharging approximately 140 cfs into the NNRC basin prior to the storm event (or until 8:15 a.m. on October 14). The simulation covers the period of October 13 to 18, 1999.

The results presented graphically on Figures 11 to 15 show a good "fit" between the simulated and observed data. A summary of the peak water level reached in the system is shown on Table 10 below.

Table 12 – Hurricane Irene – Peak Water Stage

Structure	Observed Stage (ft-NGVD)	Simulated Stage (ft-NGVD)
G-123	8.56	8.28
S-124	8.19	8.26
S-125	8.16	8.04
G-54	5.54	5.60

5.2 Impact of the G-123 Pump Station Operation on the NNRC Basin Flooding

In order to assess the impact of the G-123 operation on flooding during Hurricane Irene, the model was modified to introduce pumping from G-123. As indicated above, structure S-34 was discharging into the NNRC system at the beginning of the simulation. For the simulation, it was assumed that the pump station starts operation with all four pumps (400 cfs) when the S-34 gates are closed, approximately 30 hours into the simulation period. It is recognized that pumping might not have been allowable during the hurricane, as the water level in WCA 2B (11.8 ft-NGVD) was already higher than recommended by the operating procedures, which call for recharge operations to cease when WBA 2B levels exceed 11.5 ft-NGVD. This recommendation was ignored as the purpose of the simulation is to assess the impact that the G-123 pump station would have had on the water level in the NNRC basin had it been allowed to operate for flood control under emergency conditions. The operation of the gates at structure G-54 was not modified as water level in the canal was already above the control level when the pump operation started.

The simulation results are graphically illustrated on Figures 16 through 19, where the simulated water levels at structures G-123, S-124, S-125 and G-54 during Hurricane Irene under the actual operating conditions are compared to the simulated values for the same storm with the G-123 pump station in operation. Figure 20 presents a comparison of the discharge through G-54. Table 11 below presents a comparison of the peak water levels reached at the various structures during this storm event.

Table 13 – Hurricane Irene Model
G-123 Impact on the NNRC Basin Flooding

Structure	Peak Stage without G-123 Pumping (ft-NGVD)	Peak Stage with G-123 Pumping (ft-NGVD)	Difference (ft)
G-123	8.28	7.51	0.77
S-124	8.26	7.57	0.69
S-125	8.04	7.51	0.53
G-54	5.60	5.56	0.04

6 Conclusion and Recommendations

6.1 Results and Conclusion

From the results presented on Tables 11 and 13, it appears that the G-123 pump station is capable of lowering the water level in the NNRC and C-42 canals between approximately 6 and 9 inches. The simulation indicates that during the No-Name storm (less than a 5-year storm event), the water levels would have been between 0.44 and 0.65 feet higher if the pump station had not extracted approximately 300 cfs from the system. It also indicates that during Hurricane Irene (smaller than a 10-year storm event) the water levels could have been between 0.53 and 0.77 feet lower if the pump station had extracted 400 cfs from the system. It means that the decommissioning of Pump Station G-123 would have an adverse impact on the flood level in the North New River Basin during storm events of similar magnitude.

The simulation also reveals a high head loss between Structure S-124 and G-54 relative to other similar sized canal systems. In the adjustment phase of the simulation, the Manning's n coefficient was increased to a 0.045 value, which is considered higher than usual. As an illustration, head loss calculations along that 46,000-ft long reach are shown on Table 14 below.

Table 14 - Head Loss along North New River Canal

	No-Name Storm	**Hurricane Irene**	
	Oct 4, 2000 12:00 pm	Oct 13, 1999 6:00 pm	Oct 17, 1999 12:00 pm
Obs. TW Stage S-124	6.32	4.66	7.29
Obs. HW Stage G-54	3.58	3.24	4.11
Discharge at G-54 (cfs)	571	305	717
Approx. Avg. Velocity (fps)	0.8	0.7	0.9
Head Loss (ft)	2.74	1.42	3.18
Slope (ft/mile)	0.31	0.16	0.37

In Table 14 above, the column corresponding to October 13, 1999 shows a dry-weather condition. Even under these conditions the head difference between G-54 and S-124 is 1.42 ft which is considered high. For the purpose of comparison, similar computations were performed on the C-11 canal along the 53,400-ft long reach between Structure S-13A and pump station S-9. These calculations are presented on Table 15 below.

Table 15 - Head Loss along C-11 Canal

	Hurricane Irene	
	Oct 15, 1999 0:00 am	Oct 17, 1999 12:00 pm
Obs. TW Stage S-13A	3.80	6.05
Obs. HW Stage S-9	3.18	4.92
Discharge at S-9 (cfs)	1,703	2,574
Approx. Avg. Velocity (fps)	0.9	1.3
Head Loss (ft)	0.62	1.13
Slope (ft/mile)	0.06	0.11

The gradient along the NNRC is significantly larger than that of the C-11 canal, which has larger flow velocities as indicated on Tables 14 and 15 above. Losses along the NNRC may be the result of sedimentation, excessive vegetation, or some artificial restriction of the flow within the reach. It is recommended that these possibilities be investigated by survey and/or inspection of the canal. It is also important that remedial measures that would reduce head losses along the canal, including dredging or clearing the canal, be considered in the next task.

Based on the simulation performed for this study, it has been determined that in its current condition, the NNRC basin depends on G-123 for storm protection. It cannot be determined from the available information, however, what specific improvements to the canal would mitigate a reduction in use or the decommissioning of the G-123 pump station. A detailed design and cost estimate for construction would require additional survey and is beyond the scope of this work order. A generalized "cleaning" of the canal, however, to restore more typical conveyance losses can be simulated by modifying the Manning's n for all canal reaches. If cleaning does not reduce "without G-123" conditions to "with G-123" conditions, further improvements can be modeled by deepening canal sections for additional conveyance. Costs associated with any canal improvements will be estimated with engineering judgment at a schematic level, as detailed canal bottom topography is not available.

6.2 Task 3 Recommended Action

The results of the modeling point to a reduction in the canal head losses as a possible mitigation for the flooding in the NNRC basin. As part of the next task, a simulation of such improvement should be performed in order to estimate the potential hydraulic benefit of reducing head losses. Improvements to the canal system shall be made to the model until it is determined to have offset the impact of not operating G-123 for the selected storms. One alternative proposed in the original Statement of Work, the addition of a pump station at the G-54 structure, has been determined unrealistic and will not be considered. The following simulations may be performed:

- Reduction of the Manning's n from 0.045 to 0.035 which would be representative of a cleared channel, free of excessive vegetation and irregularities;
- Lowering of the canal bottom, this could be achieved by a dredging operation to remove accumulated sediment. Incremental analysis shall estimate benefits of dredging the

NNRC from G-54 structure to the C-42 canal and the C-42 canal to the S-125 structure as compared to also improving the NNRC from the C-42 canal to the S-124 structure.
- Additional improvements will be considered if the above changes do not offset impact to the basin's flood stages.

The cost of implementing the required improvements based on the model will be estimated at a schematic level for planning purposes.

Other work to be performed in Task 3 is an estimate of flows from the G-123 pump station for flood protection based on historical record. Filtering of flow records based on flood control versus water supply conditions would clarify the potential impact of continued use of the G-123 pump station for non-water supply discharges. Further analysis based on operational protocols and the frequency analysis prepared as part of Task 1 can help in approximating the magnitude and return period of storms estimated to trigger pumping to the west though G-123. For planning purposes, an estimated average annual volume discharged may be calculated as well with some assumptions regarding G-123 operation though coordination with SFWMD operations staff.

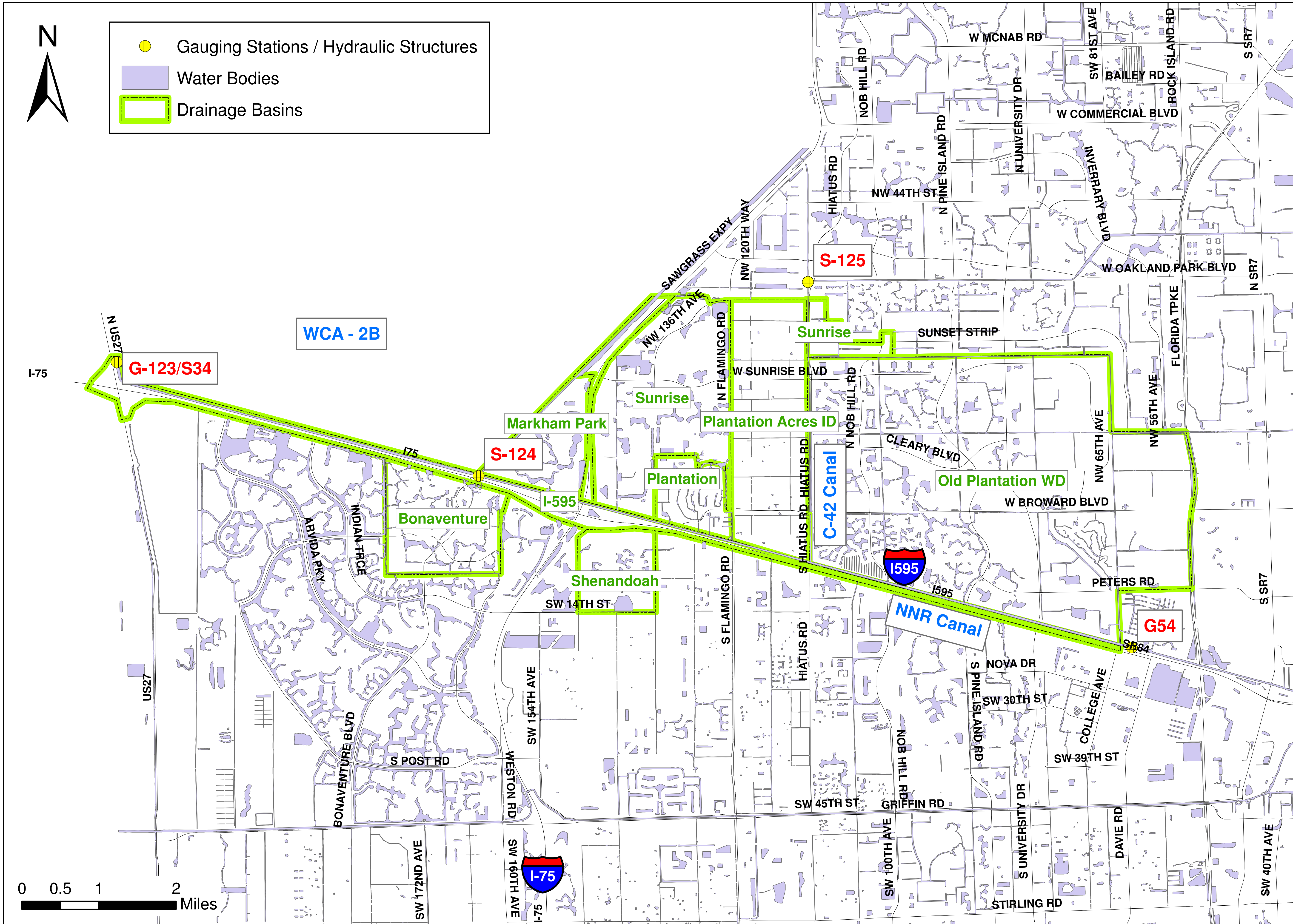
6.3 Future Recommended Action for SFWMD in the North New River Canal Basin

Specific flow restrictions or basis for the above average Manning's roughness coefficient were not observed, but established in the model's calibration to historical storm records. Therefore, outside the scope of this analysis, it is recommended that to optimize flood protection within its facilities the District initiate incorporate inspection and survey to identify the potential sources of head losses, including:

1. Research should first be performed to investigate whether improvements made after October of 2000 have improved the canal's conveyance since the modeled events.
2. Detailed inspection of the canal by boat to identify potential obstructions and restriction of the flow.
3. Bathymetric survey of the canal to better define the existing profile and cross-sections and, to develop canal improvement schemes.
4. Perform a steady state head loss measurement with a controlled release at Structure S-34. This would include stage measurements at regular intervals along the canal, and possibly upstream and downstream of each bridge.
5. Initiate the development of a detailed hydrologic\hydraulic model of the basin. This would mainly consist of incorporating more detailed canal cross sections from above mentioned survey or bathymetry and defining the hydrology of the basins currently modeled by historical pump records. The model developed as part of the present effort could be readily to model the hydrologic response of all the basins, thereby allowing the simulation of design storm events.

This Work Order is related to the Long-Tern Plan recommendation to discontinue the use of G-123 to pump stormwater runoff into the WCA 3A. Items 1-5 above relate to maintenance and conveyance capacity issues that are not within the scope of this work order, but have been included to aid SFWMD in maintaining long-term flood protection for this basin.

FIGURES



Flood Impact Analysis for the North New River Canal Basin

North New River Canal Basin Map

Figure 1

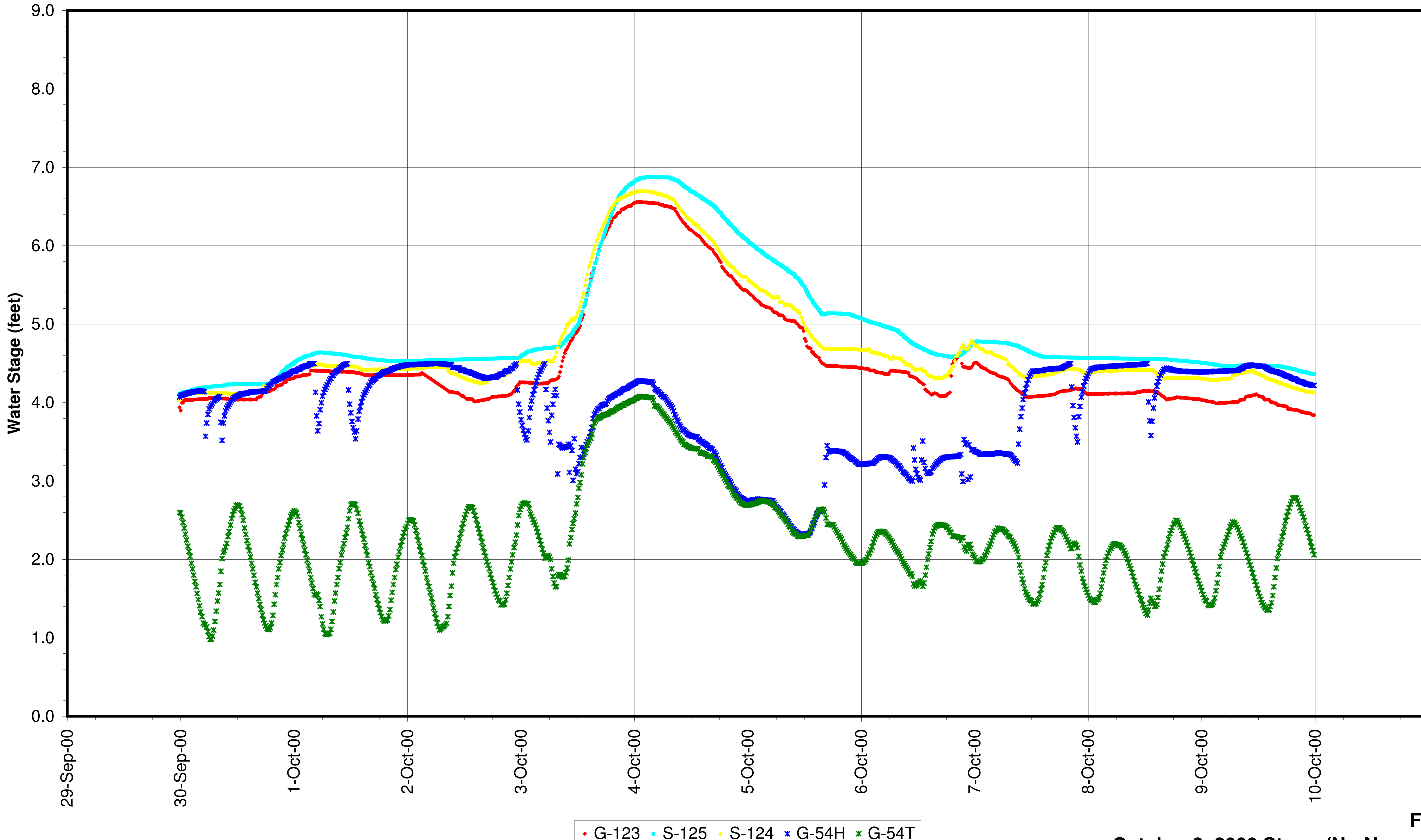


Figure 2
October 3, 2000 Storm (No-Name Storm)
NNRC Observed Water Stages
July 2005

EarthTech
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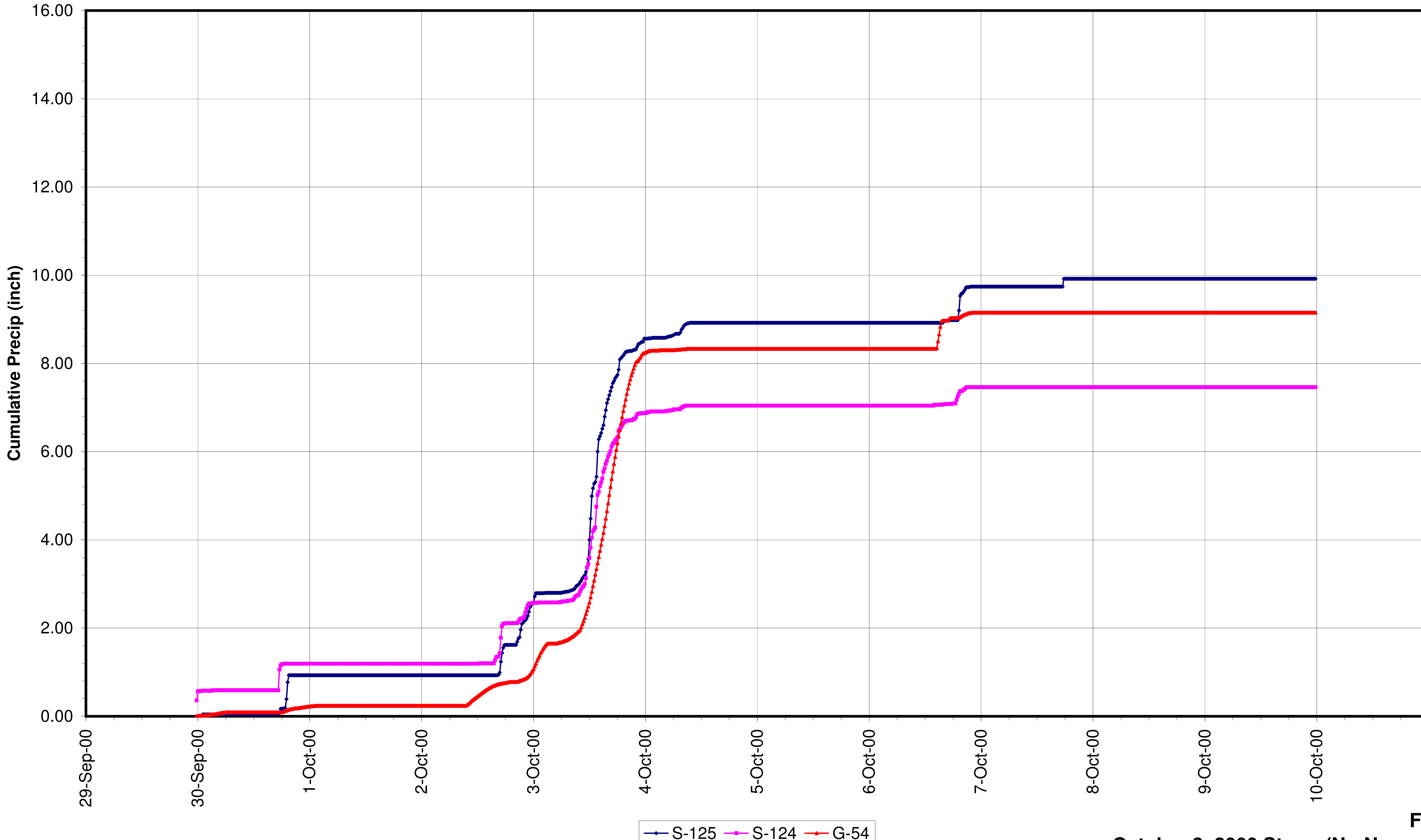

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A Tyco International Ltd. Company

Figure 3
October 3, 2000 Storm (No-Name Storm)
NNRC Basin Precipitation
July 2005

Flood Impact Analysis for the North New River Canal Basin
Task 2 - Technical Memorandum

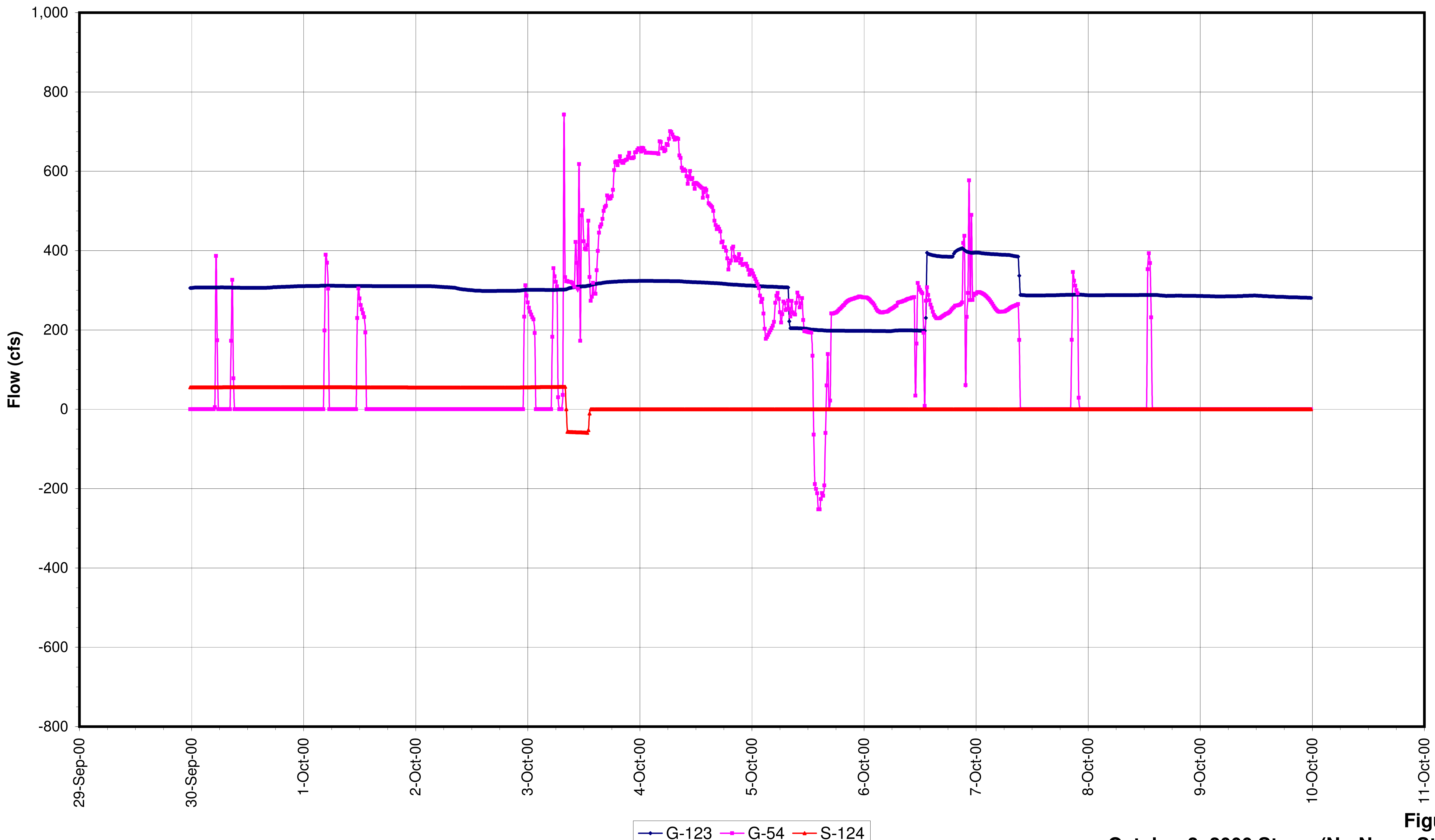

Figure 4
October 3, 2000 Storm (No-Name Storm)
Discharge Records
July 2005

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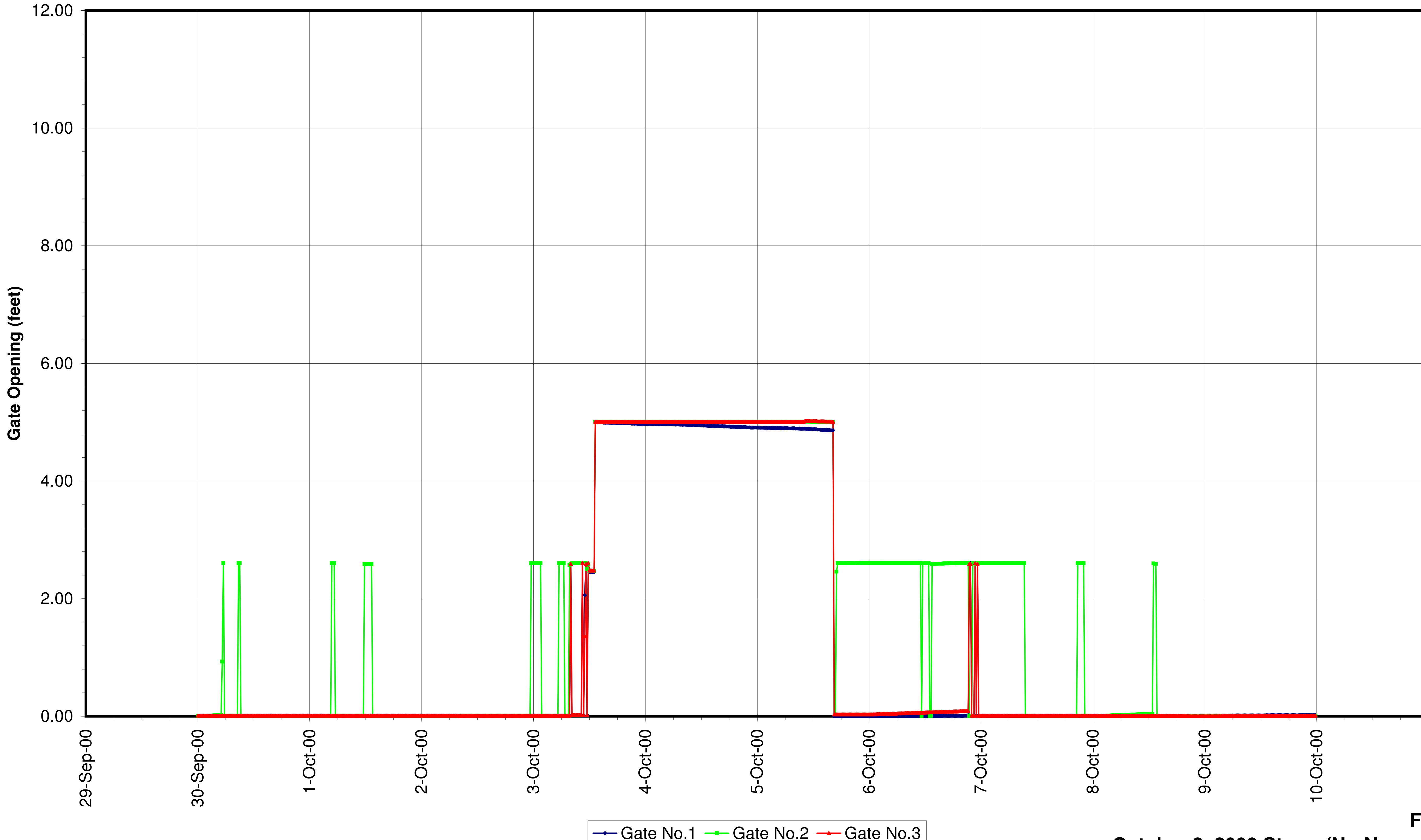

Figure 5
October 3, 2000 Storm (No-Name Storm)
Structure G-54 Gate Openings
July 2005

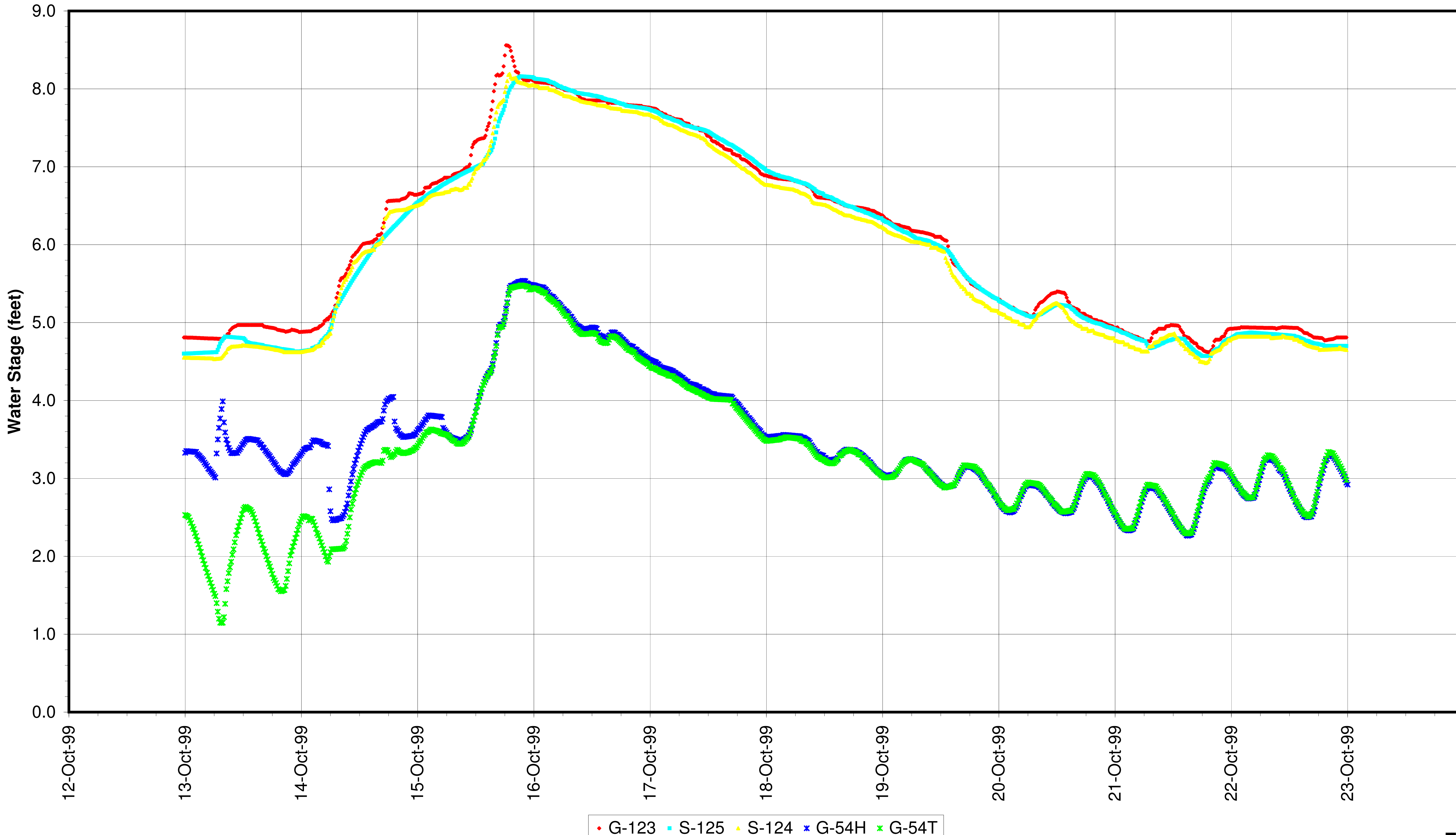

Figure 6
October 15, 1999 Storm (Hurricane Irene)
NNRC Observed Water Stages
July 2005

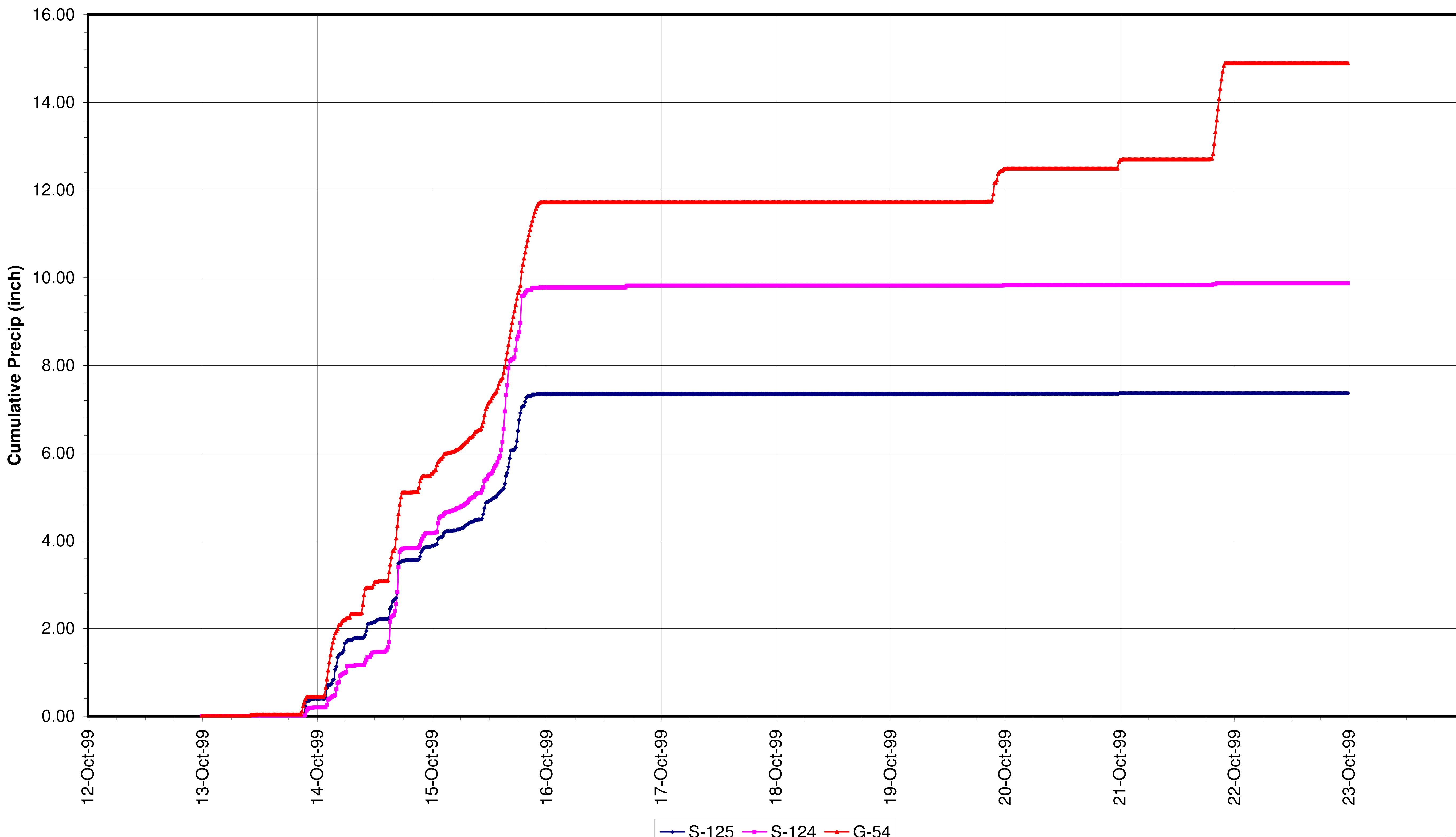

Figure 7
October 15, 1999 Storm (Hurricane Irene)
NNRC Basin Precipitation
July 2005

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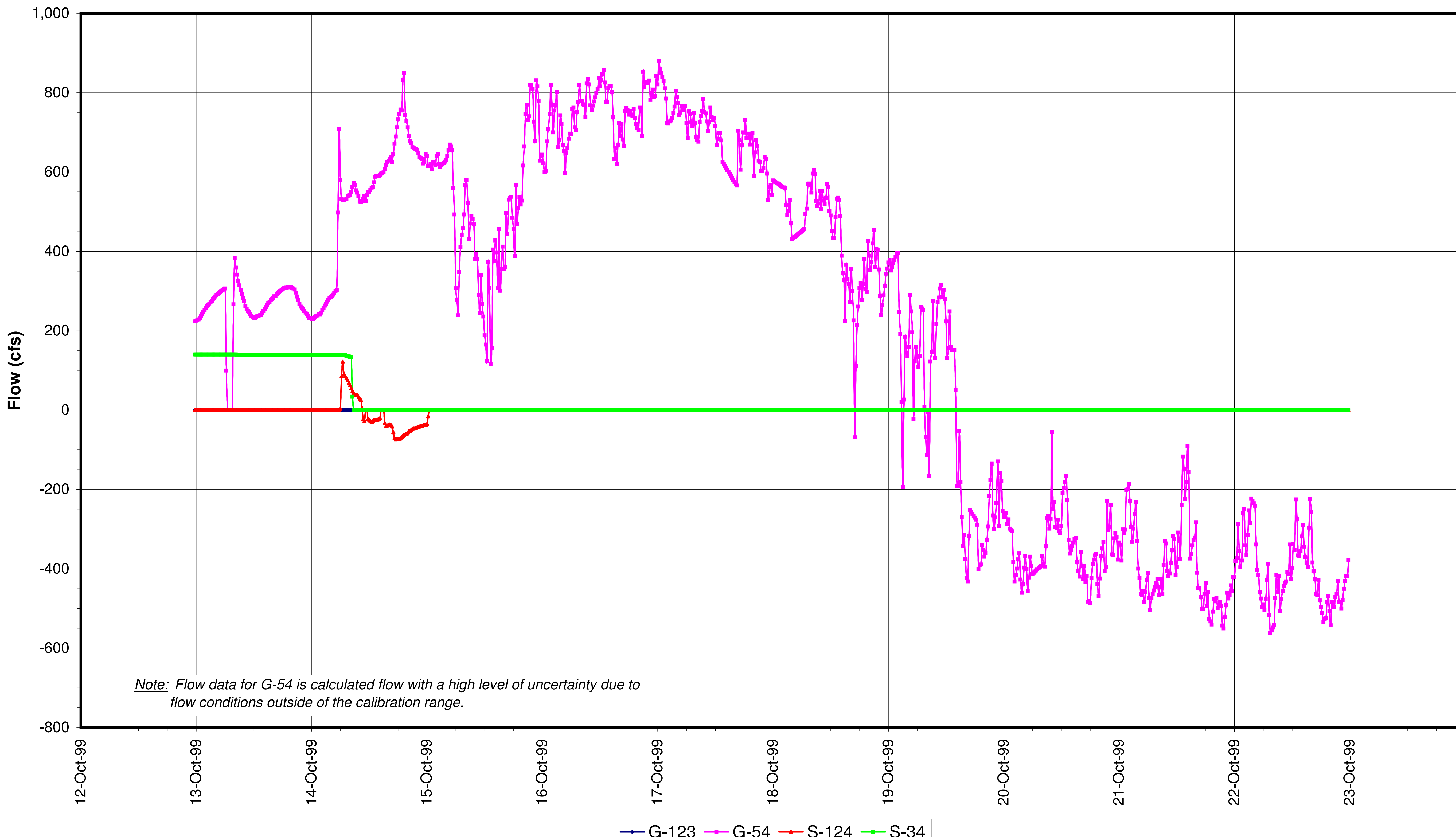

Figure 8
October 15, 1999 Storm (Hurricane Irene)
Discharge Records
July 2005

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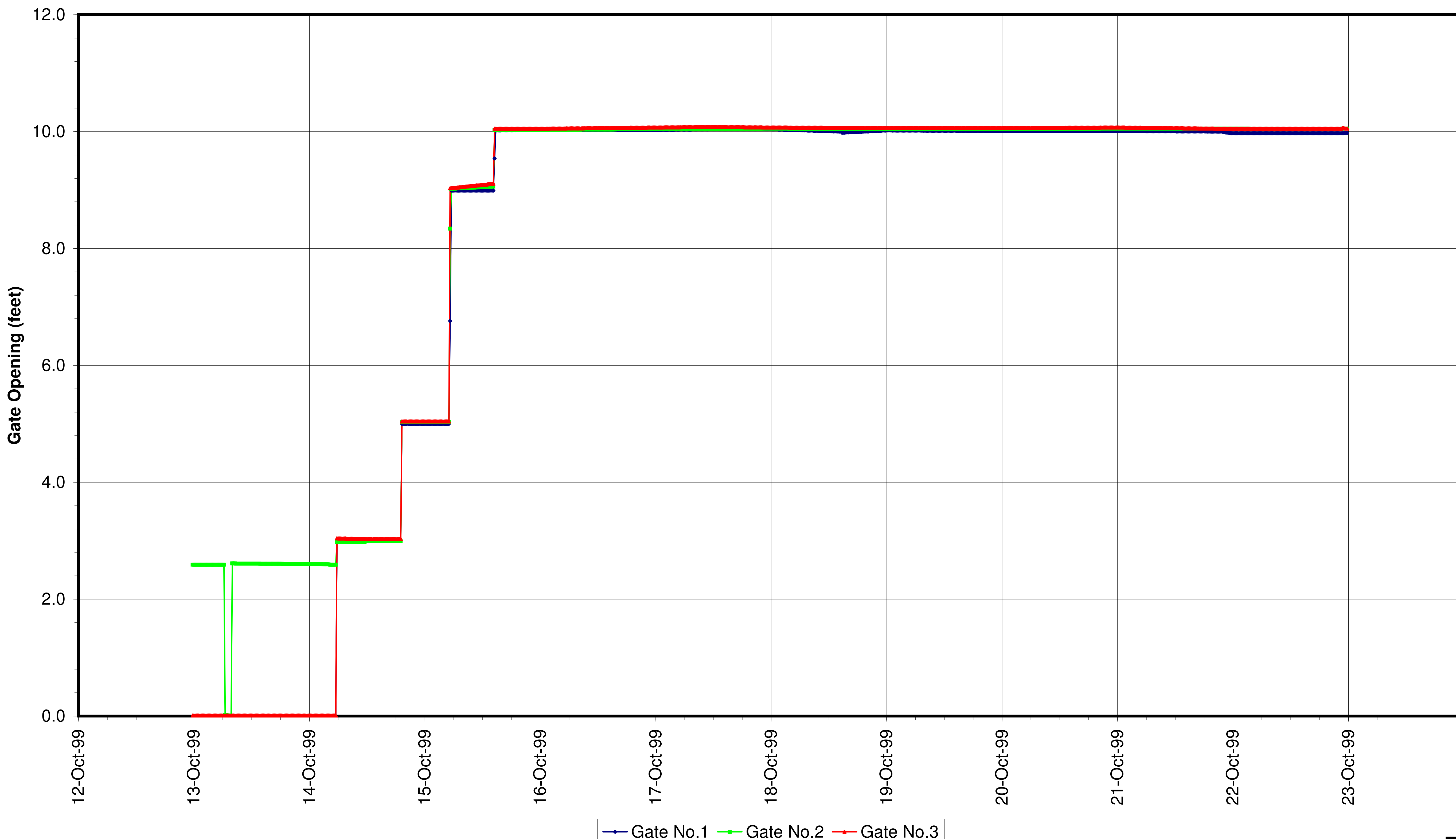

Figure 9
October 15, 1999 Storm (Hurricane Irene)
Structure G54 Gate Openings
July 2005

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Figure 10 - XP-SWMM Model of the North New River Canal Basin

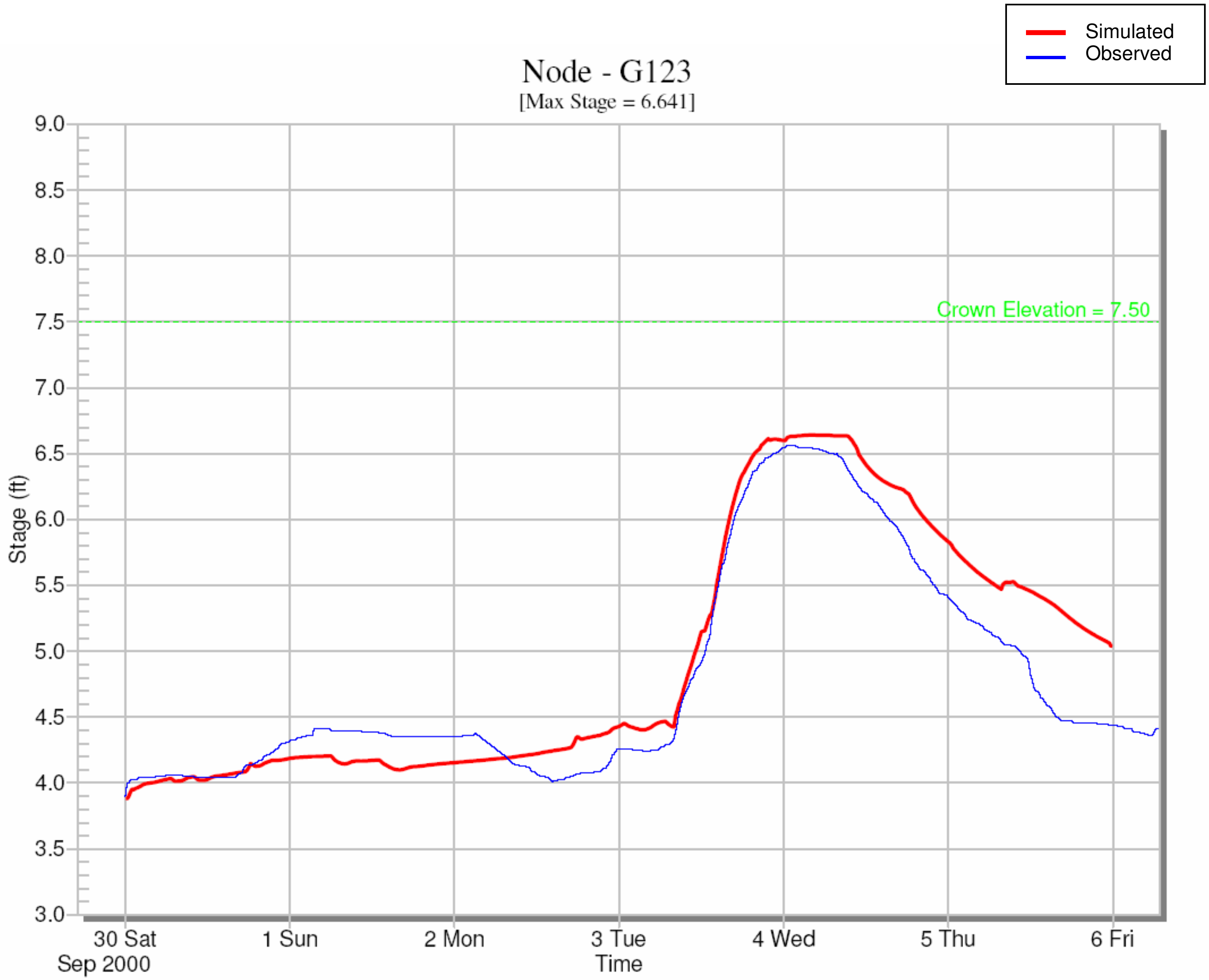

Figure 11 - No-Name Storm - Water Level at Structure G-123/S-34

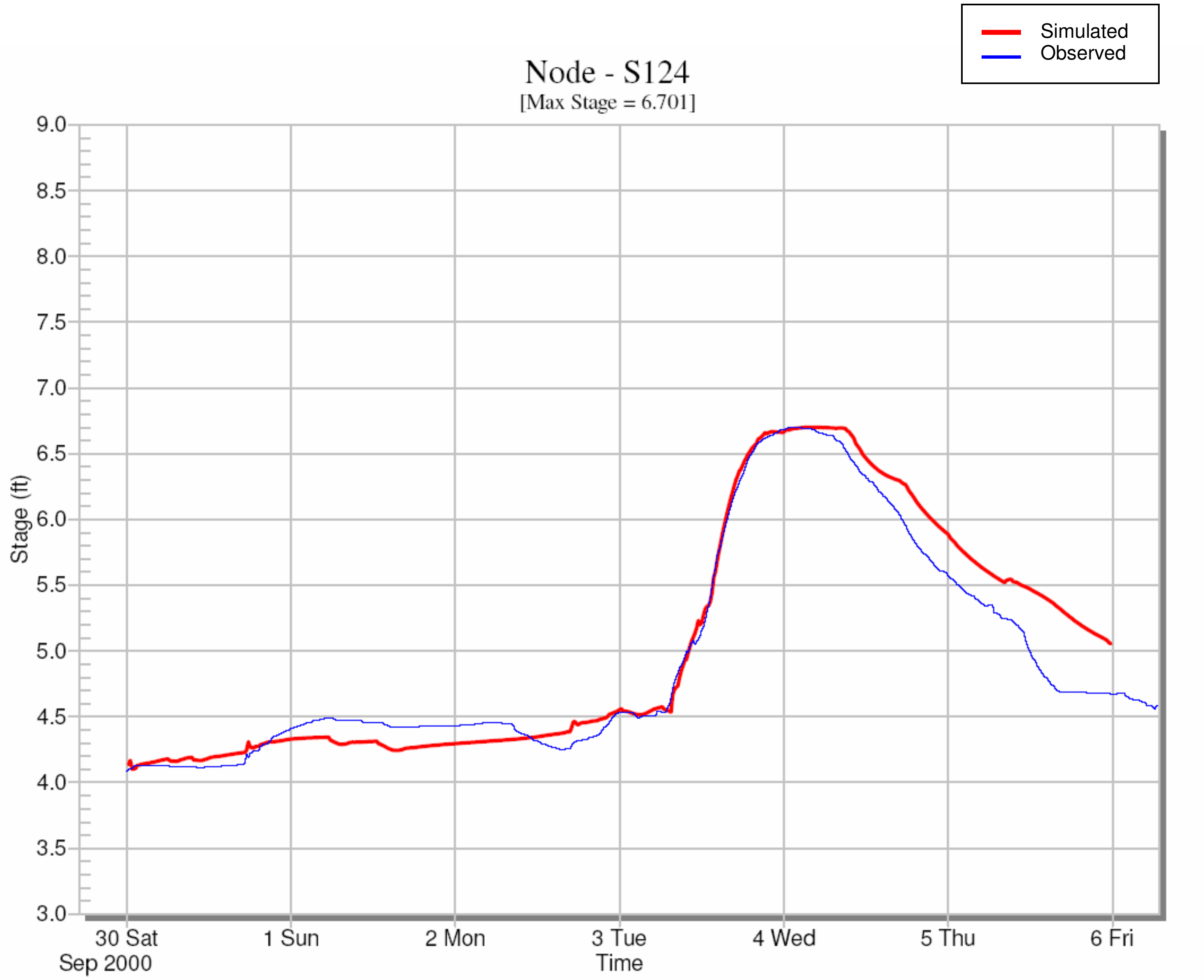

Figure 12 - No-Name Storm - Water Level at Structure S-124

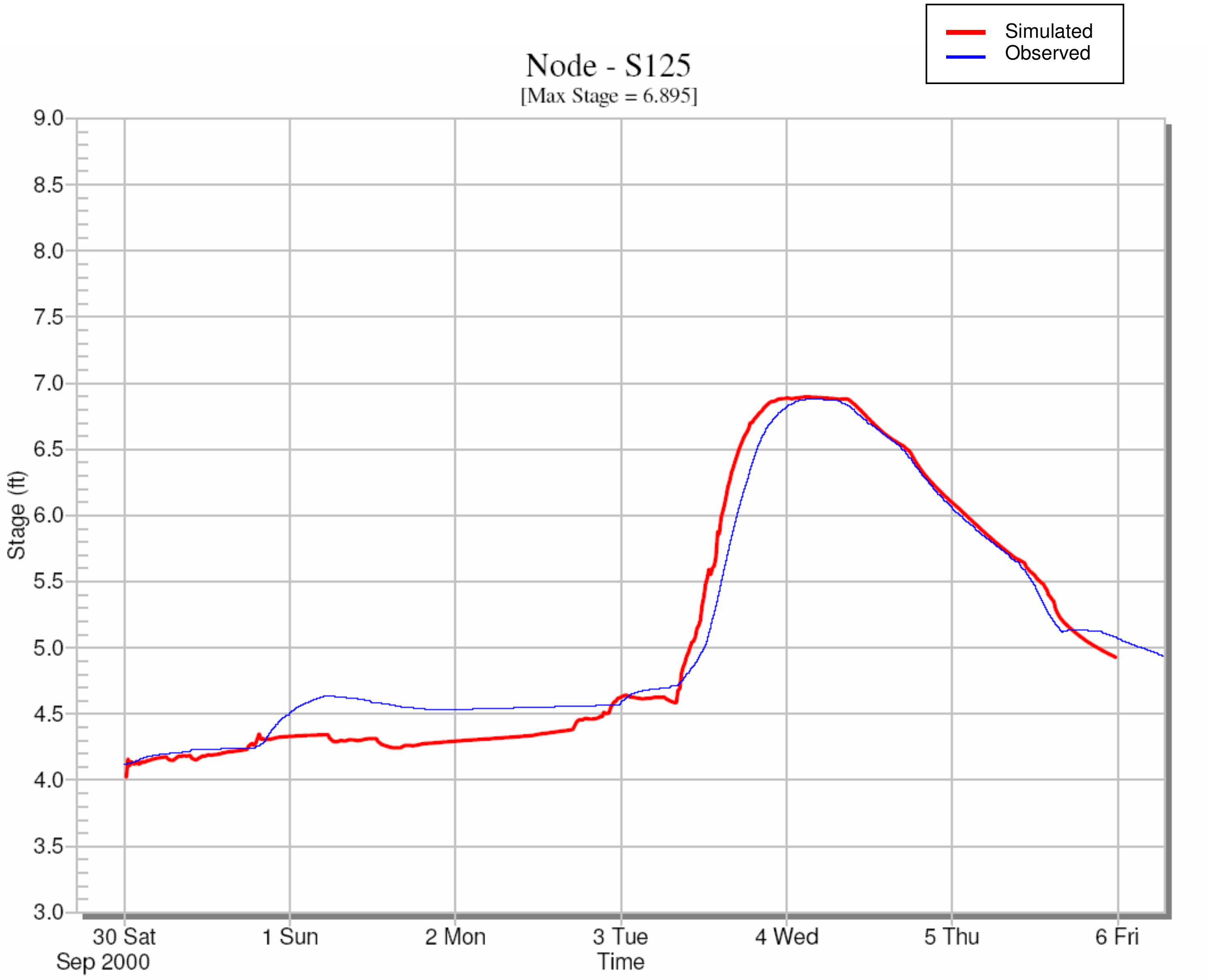

Figure 13 - No-Name Storm - Water Level at Structure S-125

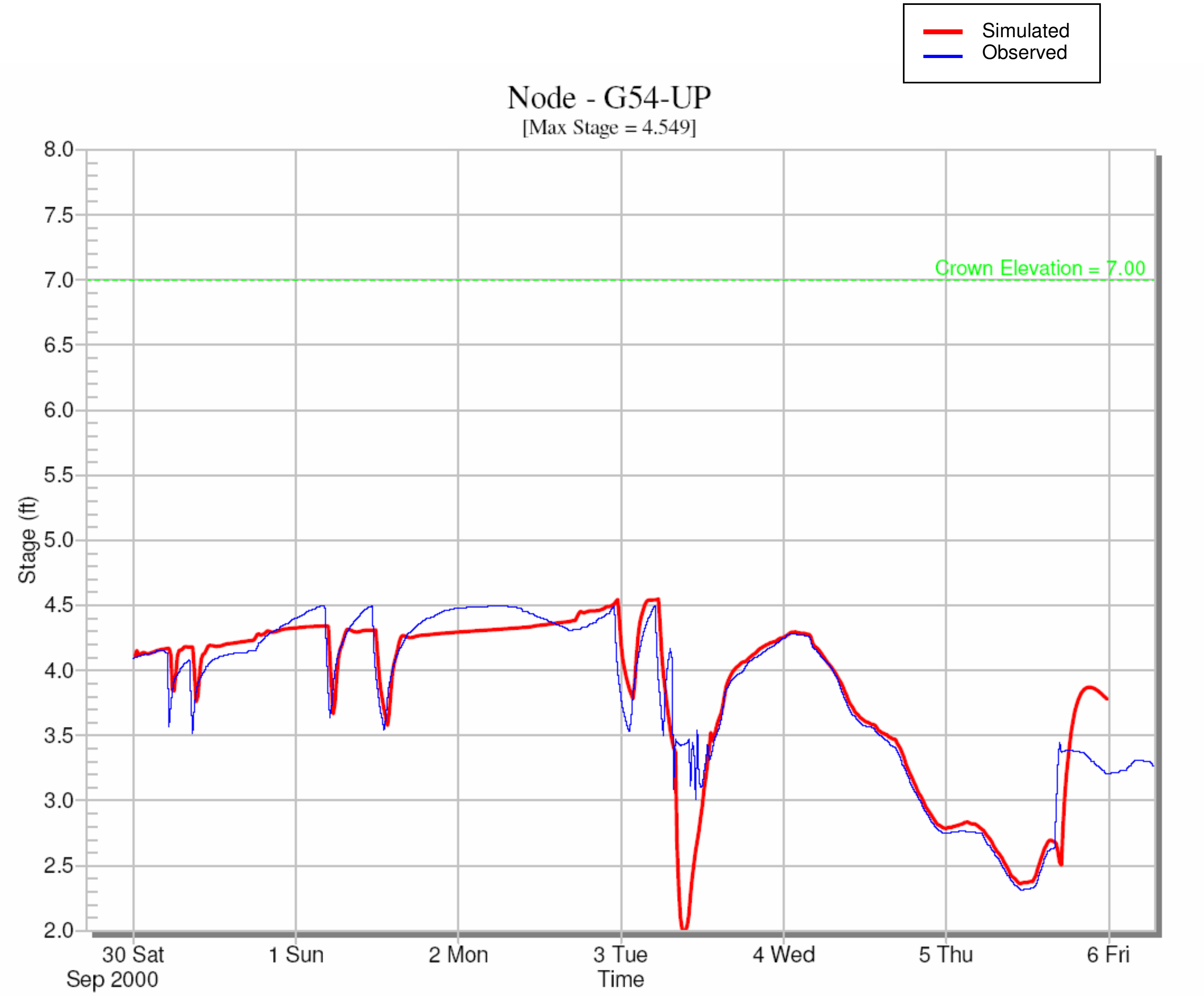

Figure 14 - No-Name Storm - Water Level Upstream of Structure G-54

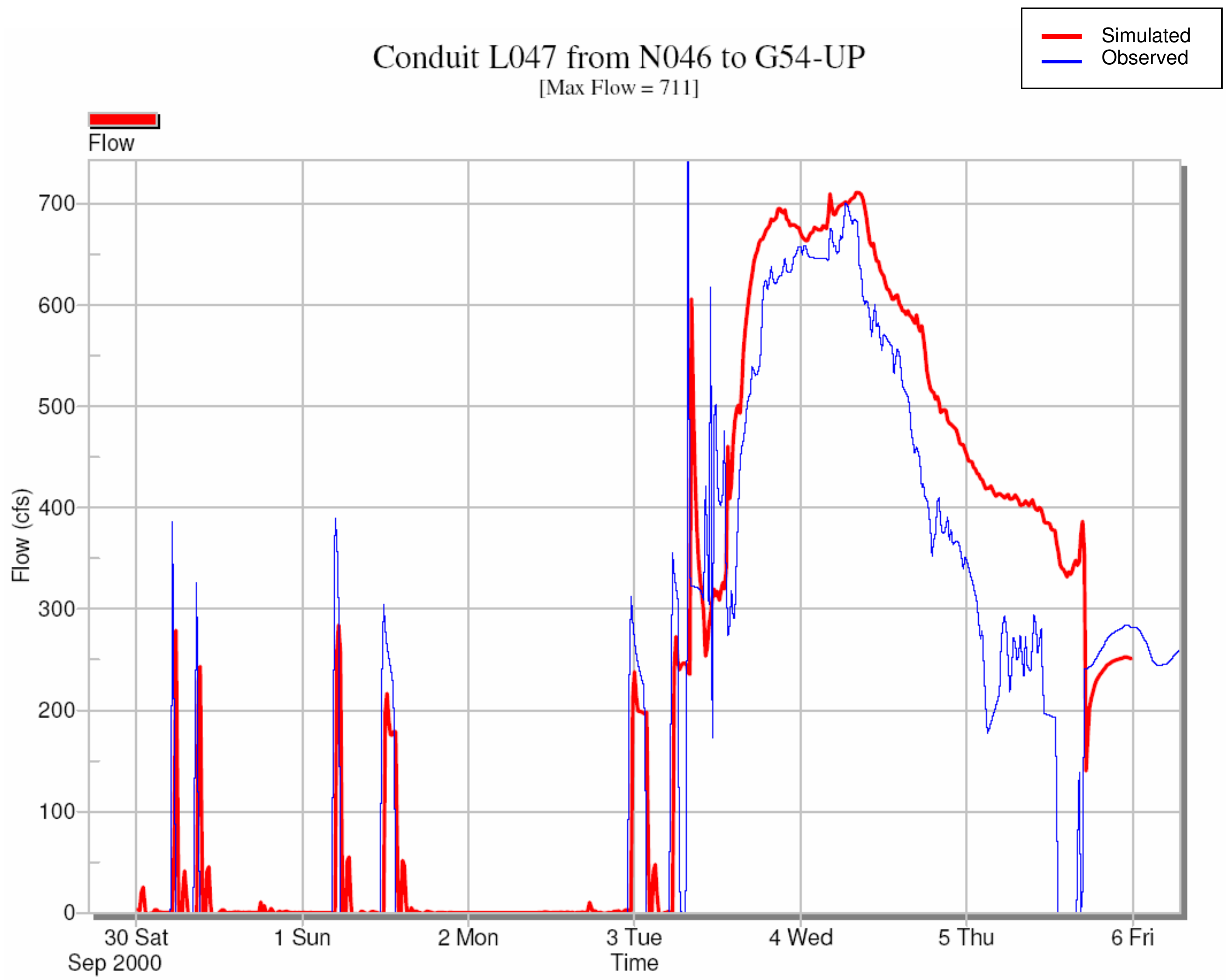

Figure 15 - No-Name Storm - Discharge through Structure G-54

Note: *Under the "Base Scenario" the G-123 pump station discharges approximately 300 cfs.*

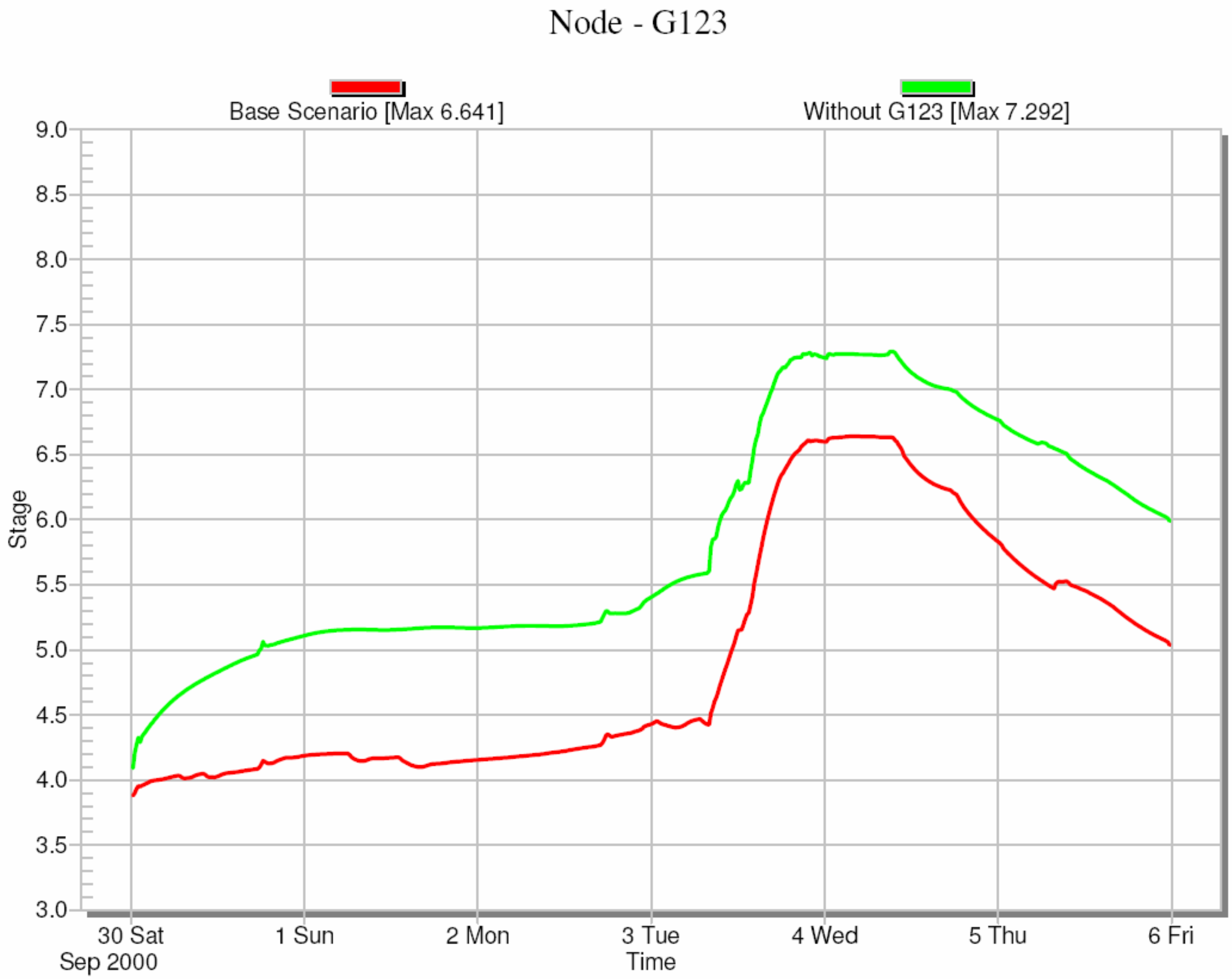

Figure 16 - No-Name Storm Model – Impact of G-123 Operation on the Water Level at Structure G-123/S-34

Note: Under the "Base Scenario" the G-123 pump station discharges approximately 300 cfs.

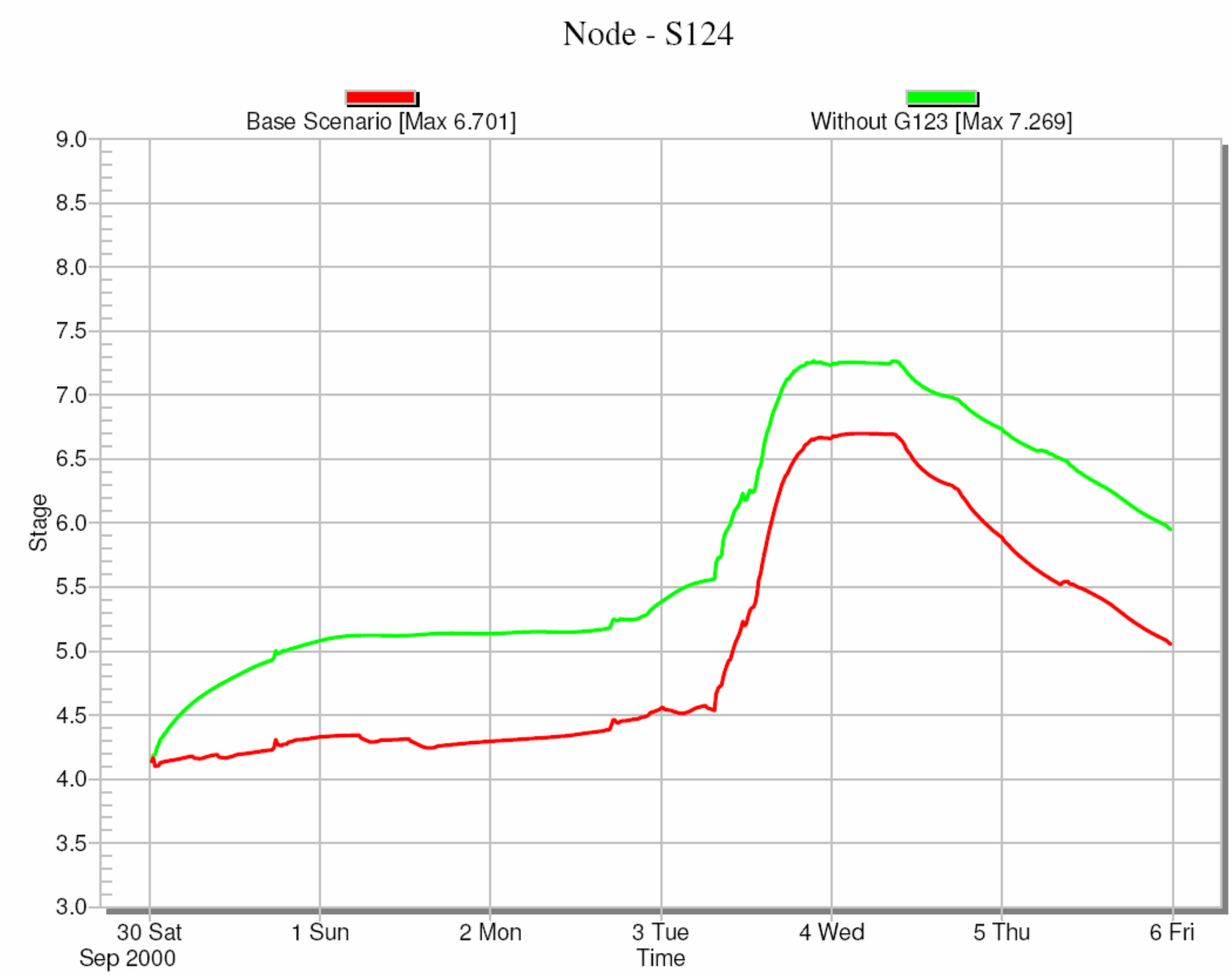

Figure 17 - No-Name Storm Model - Impact of G-123 Operation on the Water Level at Structure S-124

<u>Note:</u> *Under the "Base Scenario" the G-123 pump station discharges approximately 300 cfs.*

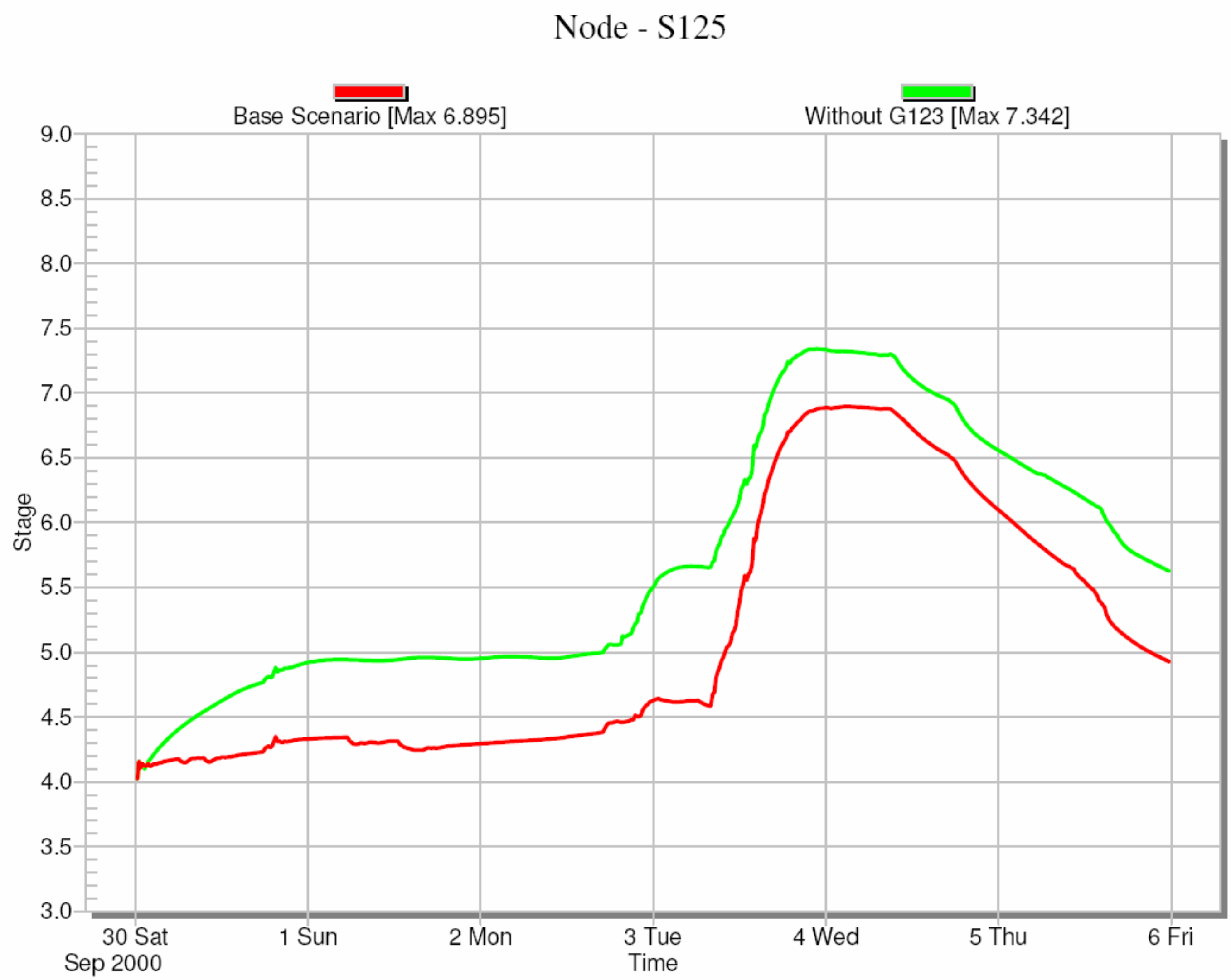

Figure 18 - No-Name Storm Model - Impact of G-123 Operation on the Water Level at Structure S-125

<u>*Note:*</u> *Under the "Base Scenario" the G-123 pump station discharges approximately 300 cfs.*

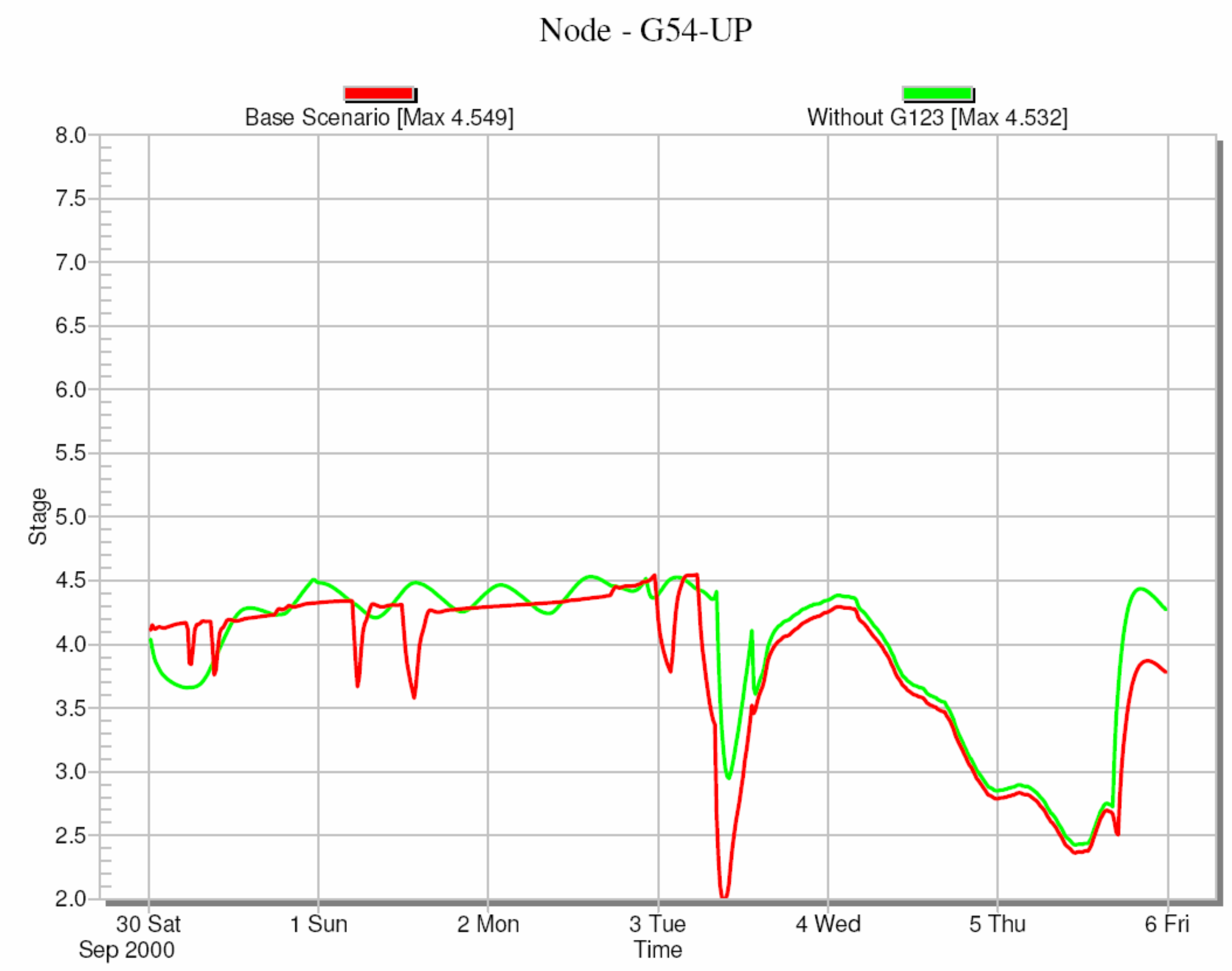

Figure 19 - No-Name Storm Model - Impact of G-123 Operation on the Water Level Upstream of Structure G-54

<u>Note:</u> *Under the "Base Scenario" the G-123 pump station discharges approximately 300 cfs.*

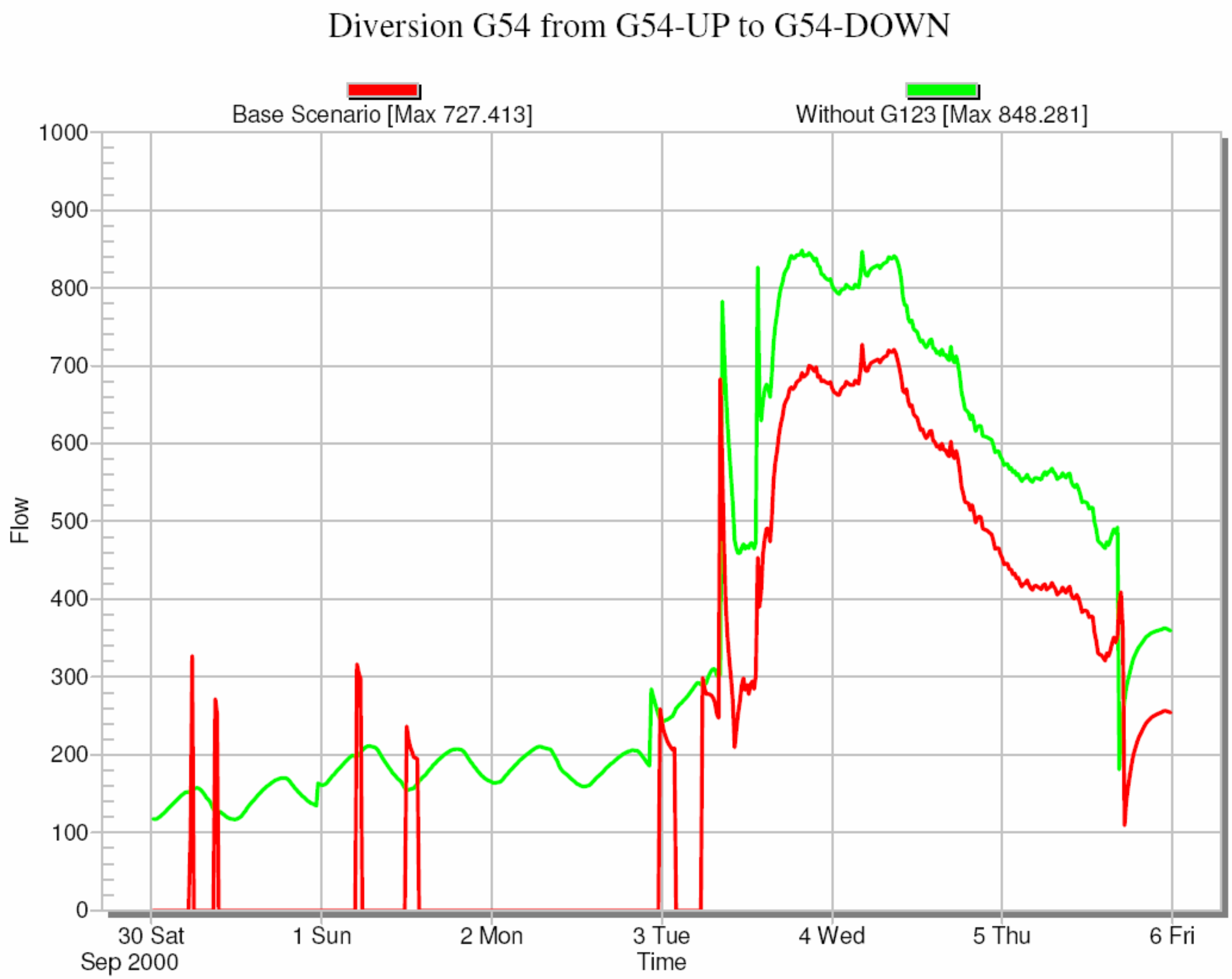

Figure 20 - No-Name Storm Model - Impact of G-123 Operation on the Flow through Structure G-54

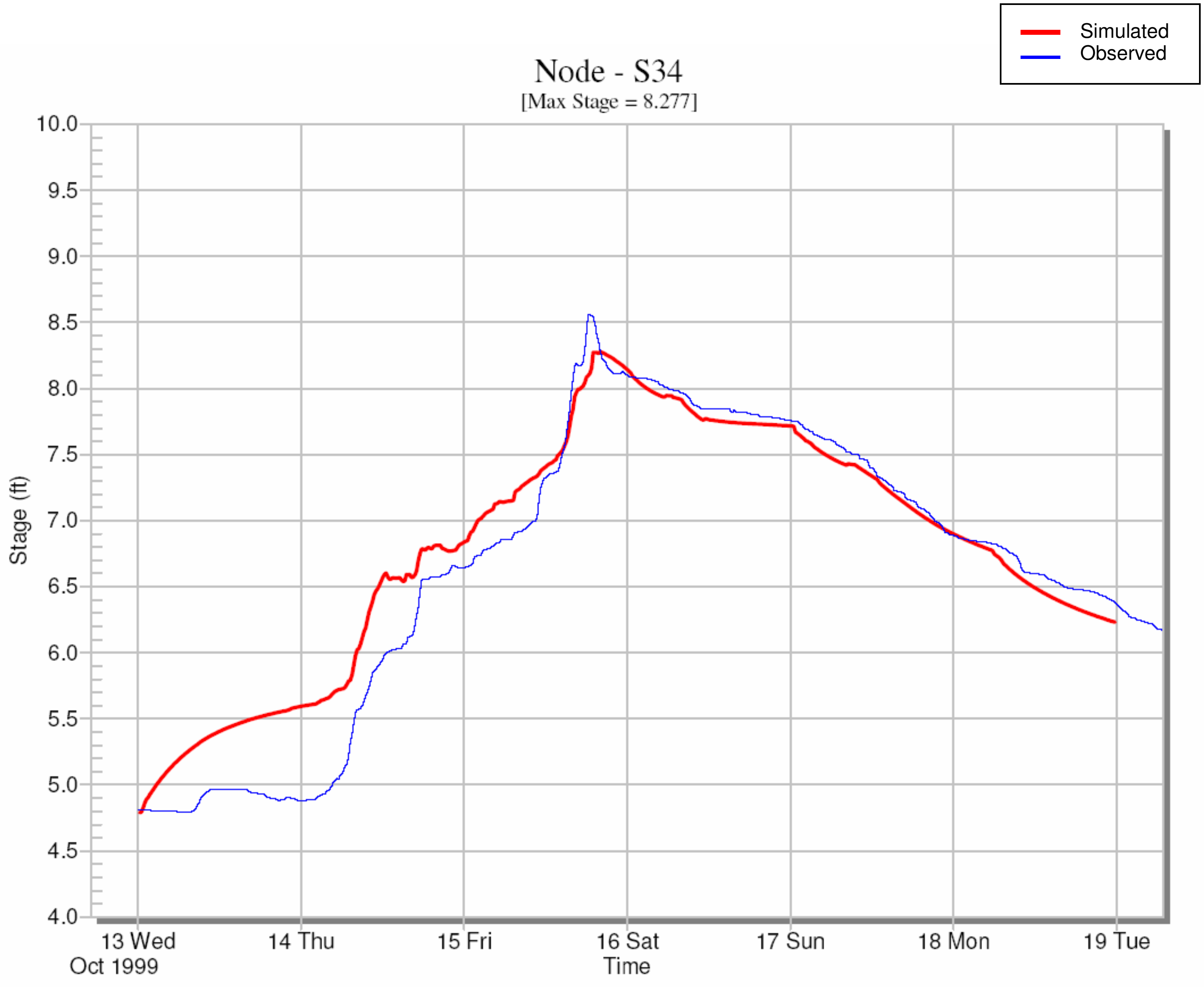

Figure 21 - Hurricane Irene - Water Level at Structures G-123/S-34

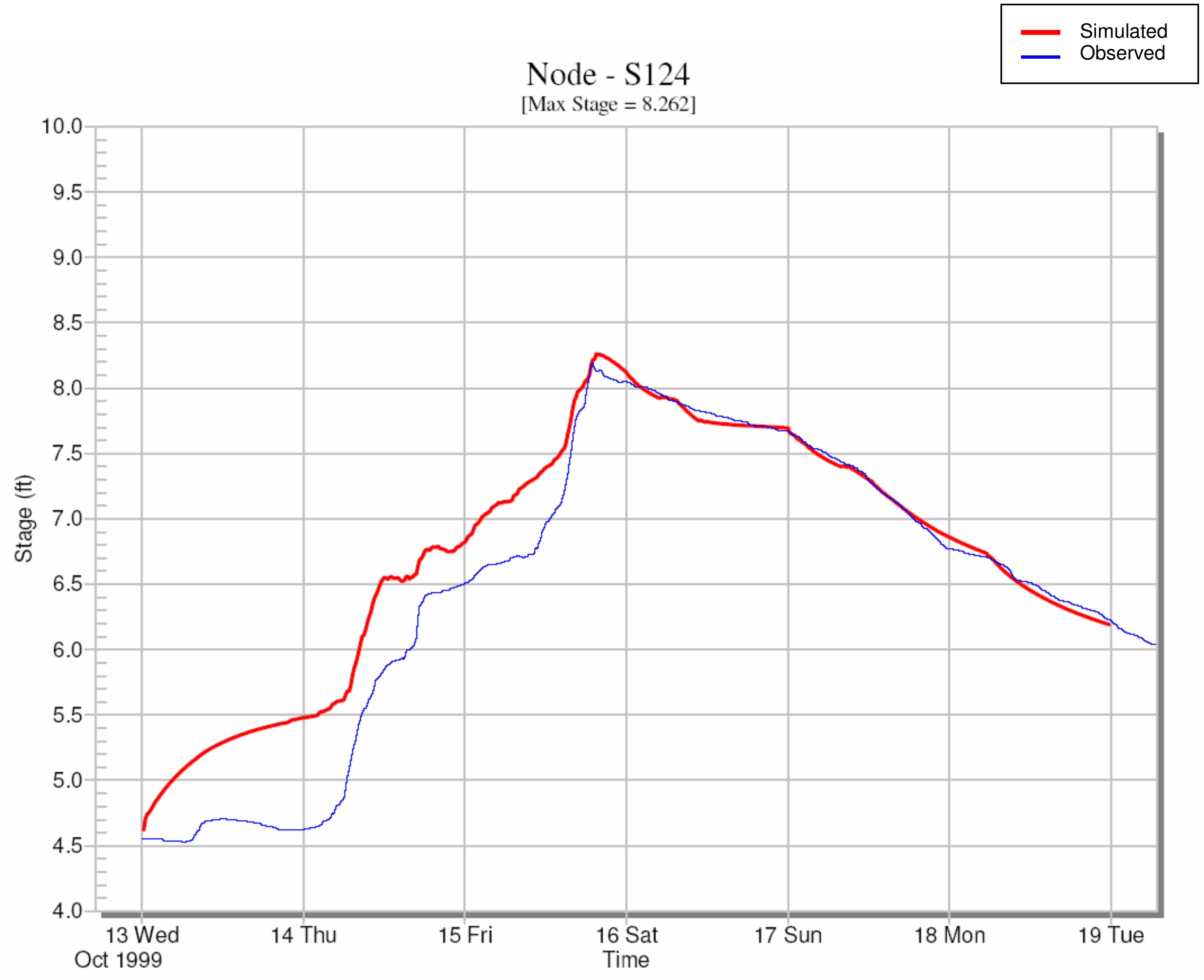

Figure 22 - Hurricane Irene - Water Level at Structure S-124

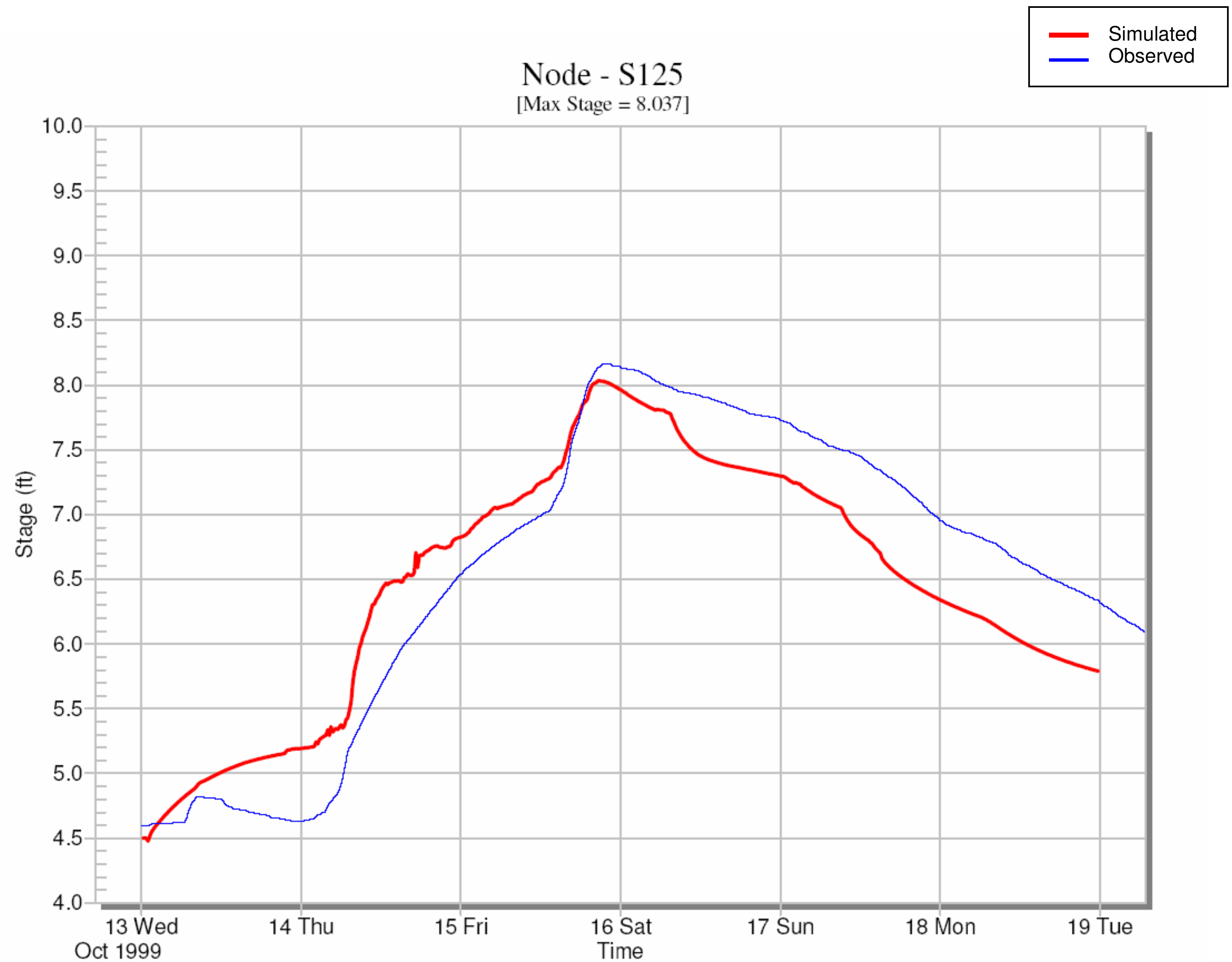

Figure 23 - Hurricane Irene - Water Level at Structure S-125

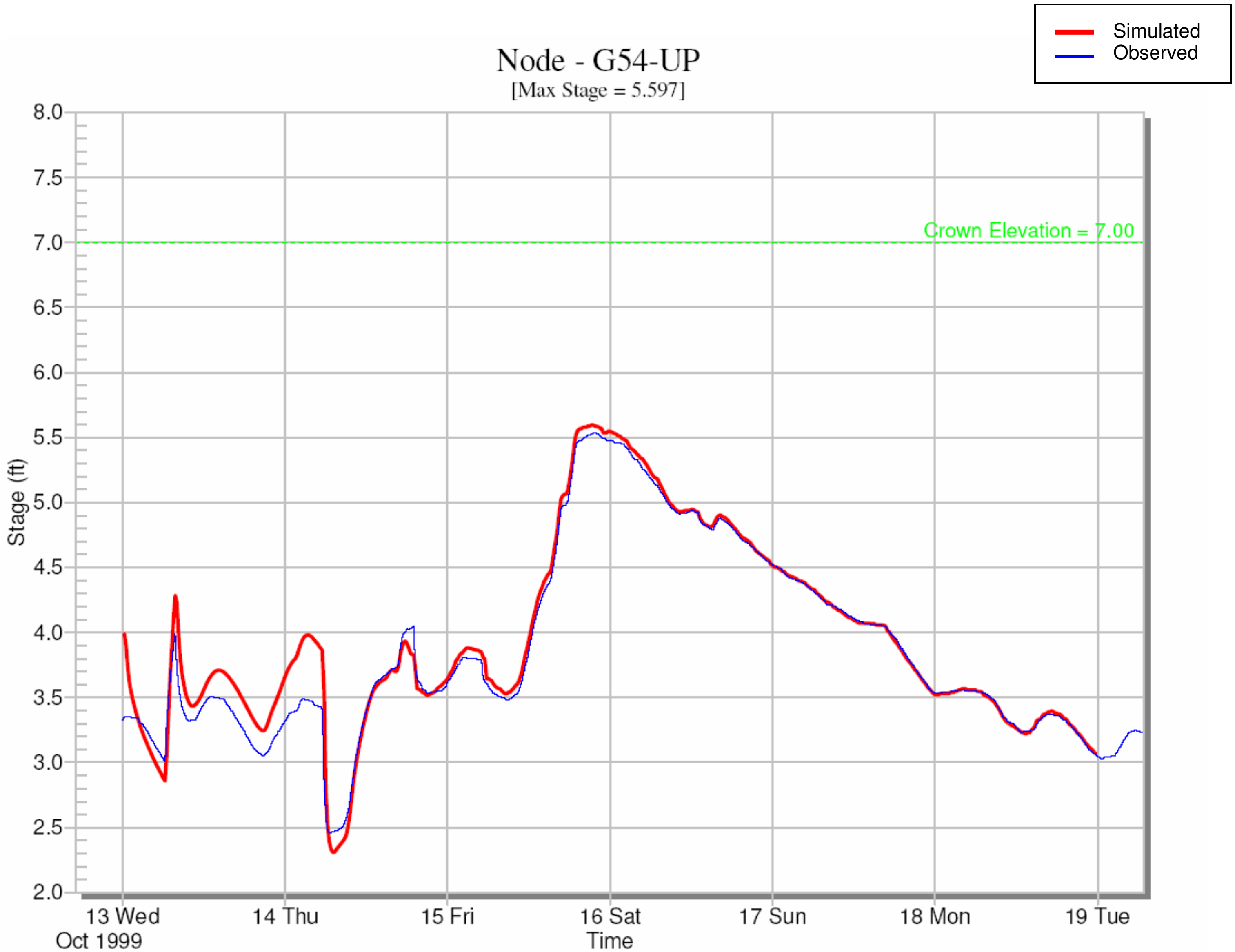

Figure 24 - Hurricane Irene - Water Level Upstream of Structure G-54

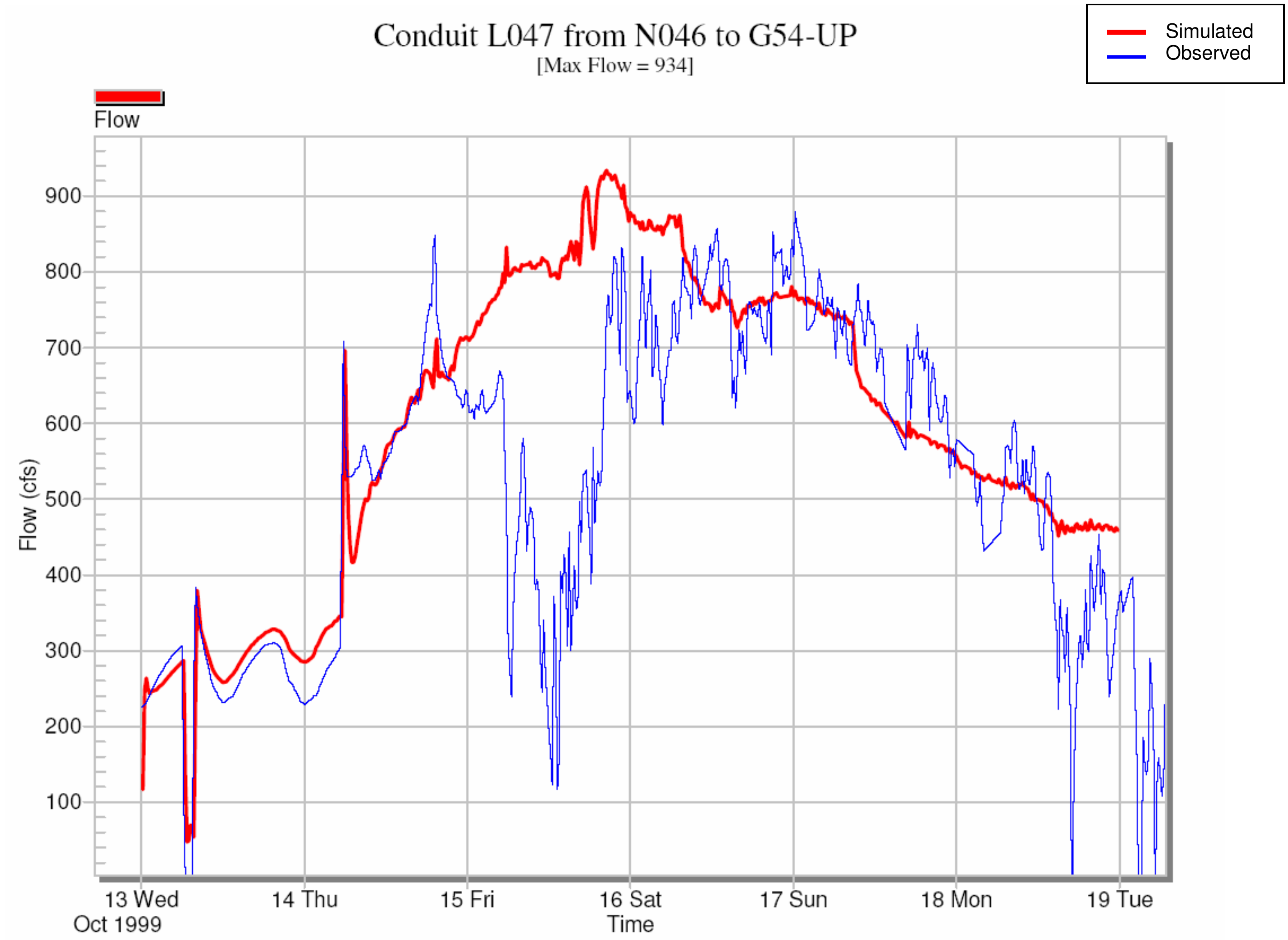

Figure 25 - Hurricane Irene - Discharge through Structure G-54

<u>Note:</u> *The simulation "With G-123" assumes that the G-123 pump station operates at a maximum capacity of 400 cfs.*

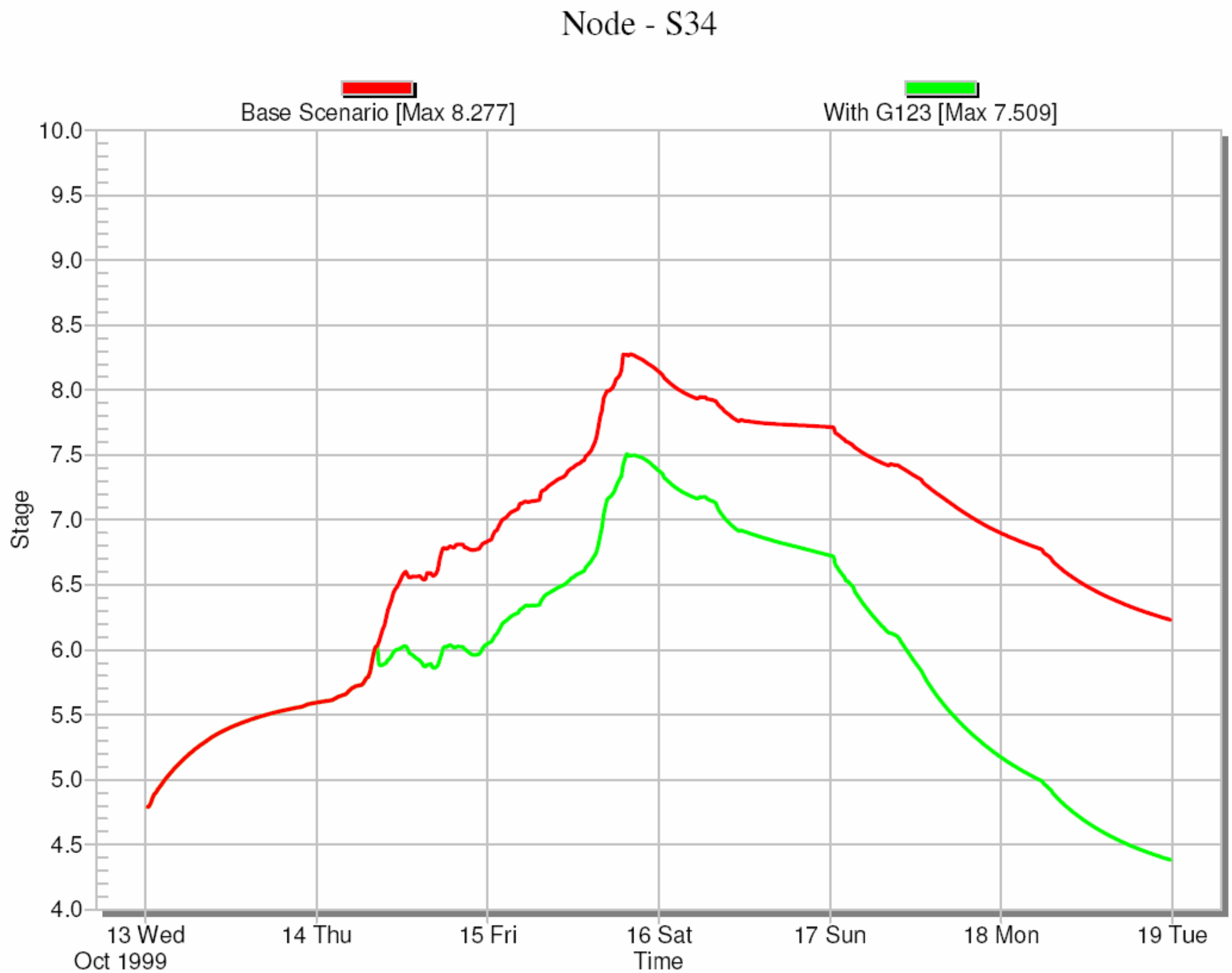

Figure 26 - Hurricane Irene Model - Impact of G-123 Operation on the Water Level at Structures G-123/S-34

<u>*Note:*</u> *The simulation "With G-123" assumes that the G-123 pump station operates at a maximum capacity of 400 cfs.*

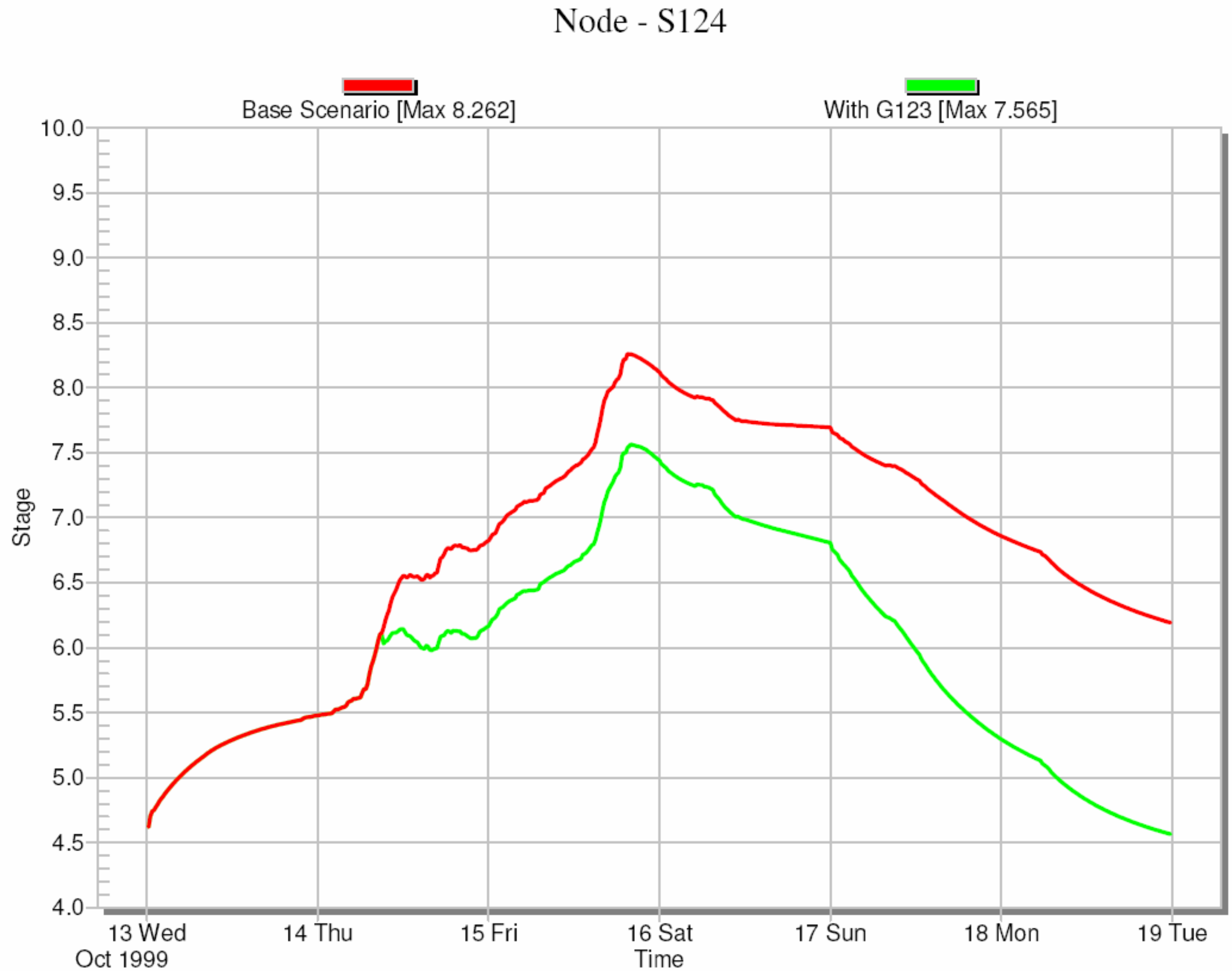

Figure 27 - Hurricane Irene Model - Impact of G-123 Operation on the Water Level at Structure S-124

<u>*Note:*</u> *The simulation "With G-123" assumes that the G-123 pump station operates at a maximum capacity of 400 cfs.*

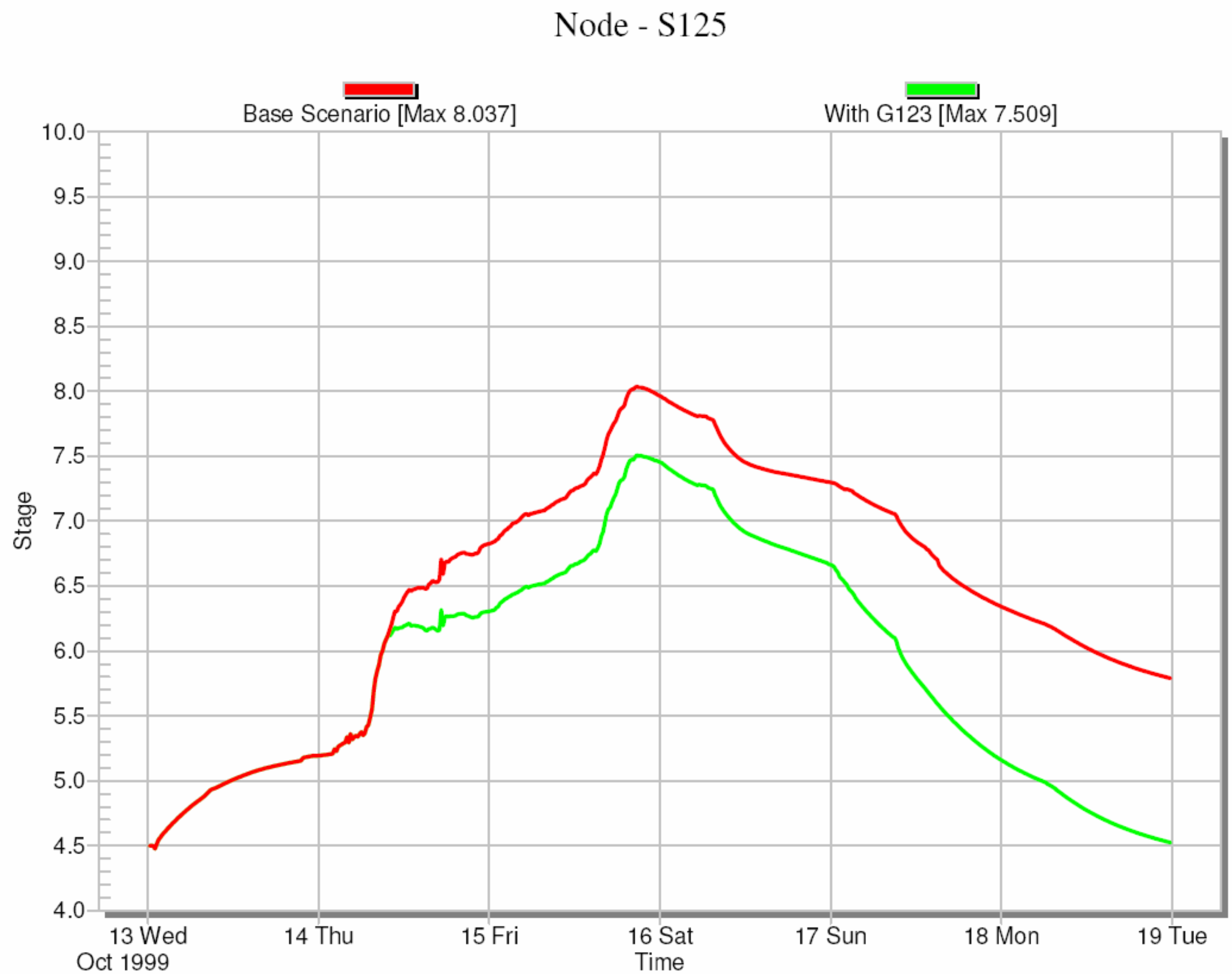

Figure 28 - Hurricane Irene Model - Impact of G-123 Operation on the Water Level at Structure S-125

<u>*Note:*</u> *The simulation "With G-123" assumes that the G-123 pump station operates at a maximum capacity of 400 cfs.*

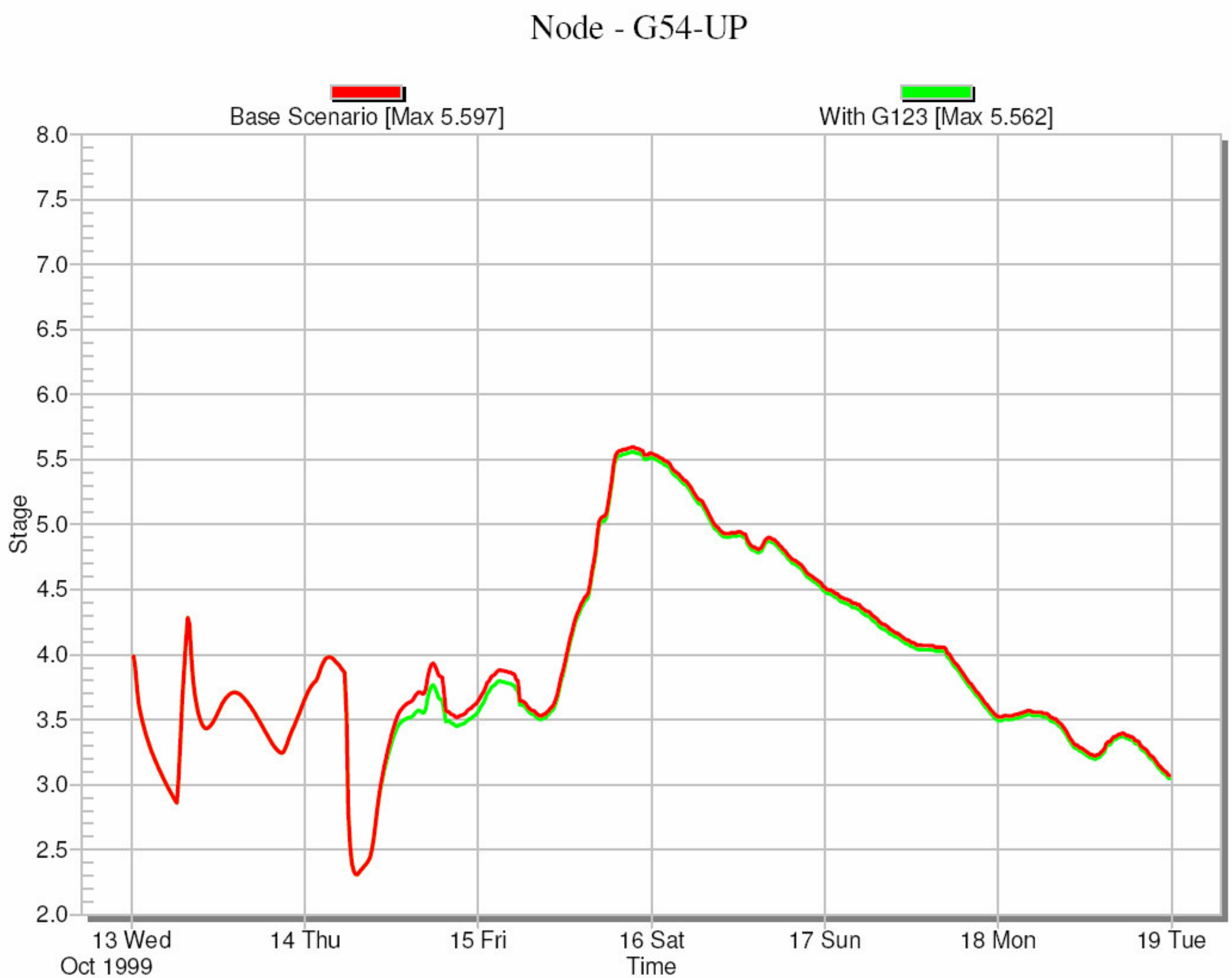

Figure 29 - Hurricane Irene Model - Impact of G-123 Operation on the Water Level Upstream of Structure G-54

Note: The simulation "With G-123" assumes that the G-123 pump station operates at a maximum capacity of 400 cfs.

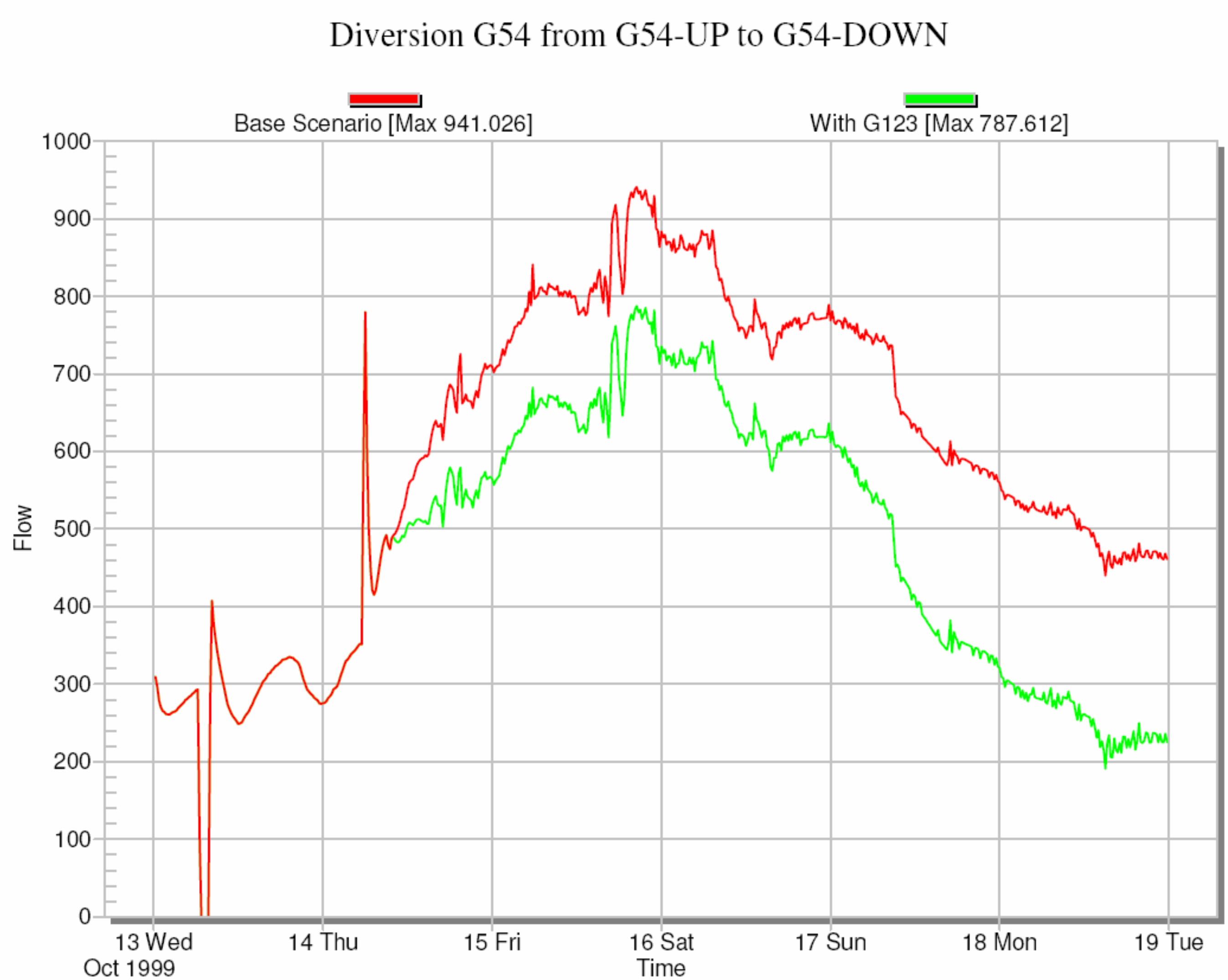

Figure 30 - Hurricane Irene Model - Impact of G-123 Operation on the Flow Through Structure G-54