North Springs Improvement District

Task 5

Evaluation of Existing Conditions and Impacts of Proposed Impoundment on Hillsboro Canal Hydraulics

FINAL

July 16, 2004

Prepared by:

ecology and environment, inc.
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Prepared for:

South Florida
Water Management
District

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List of Acronyms and Abbreviations

_H	headwater (of a hydraulic structure; e.g., G56_H)
_T	tailwater (of a hydraulic structure; e.g., G56_T)
cfs	cubic feet per second
DWOPER model	dynamic wave operational model
E & E	Ecology & Environment, Inc.
ECP	Everglades Construction Project
EFA	Florida's Everglades Forever Act
EPA	Everglades Protection Area
ESP	Everglades Stormwater Program
F.S.	Florida Statutes
FDEP	Florida Department of Environmental Protection
ft	feet
ft^3	cubic feet
GIS	Geographic Information System
H&H	hydrology and hydraulic
HILLS	a station roughly corresponding to node HN887 of the present models
LTP	Long-Term Plan
LWDD	Lake Worth Drainage District
NAD	North American Datum
NGVD	National Geodetic Vertical Datum
NSID	North Springs Improvement District
RTC	Real Time Control
SFWMD	South Florida Water Management District
USBR	United States Bureau of Reclamation
UTC	Universal Time Coordinated
UTM	Universal Transverse Mercator
WCA	Water Conservation Areas

List of Acronyms and Abbreviations, continued

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Executive Summary

Ecology & Environment, Inc. (E & E) has conducted a feasibility-level hydrology and hydraulic (H&H) analysis of the potential impacts of increased North Springs Improvement District (NSID) discharge on Hillsboro Canal stages for the 10-, 25-, and 100-year storm events. Future channel improvements and storage in the study area also have been considered. Due to the preliminary nature of the modeling effort, limitations in the model (XP-SWMM), and simplifying assumptions, the model outputs are intended to represent potential impacts to the Hillsboro Canal stages and should not necessarily be viewed as absolute values.

Three model scenarios (a Base Model and two alternatives) were initially run. The Base Model represents current conditions and was used for model calibration purposes. The two alternatives reflect discharge of all NSID stormwater to the Hillsboro Canal (currently excess discharge pumped to Water Conservation Area [WCA] 2A), but at slightly different rates. Also, the two alternatives reflect some channel improvements and storage in the project area.

The Base Model was calibrated to a 25-year/72-hour storm event that is closely represented by Hurricane Irene. The model calibrated reasonably well to the stages observed during Irene.

The alternatives were initially modeled for the 25-year/72-hour storm event and both showed a significant and similar rise in water level. This rise is attributable to a variety of factors, including increased flows from the NSID, increased bank heights in the new cross sections (less overtopping), and effects of the G56 hydraulic structure. In the calibrated base case, 3.3% of the total water volume is lost over the top of the bank; only 0.08% is lost in Alternatives 1 and 2. While this is a relatively small percent, the channel is sensitive to a rise in stage if the outflow cannot keep up to the inflow. Rough calculations reveal an inflow of 100 cubic feet per second (cfs) in excess of the outflow from the G56 structure for two days will cause a 3-foot rise in water surface level. When the water level rises to an elevation of 9.8 feet relative to the National Geodetic Vertical Datum (NGVD), it hits the bottom of the

gate and essentially creates a constriction (head rises, but flow remains steady) causing more backwater effects. In reality, it is unlikely that the flow would remain steady as depicted by XP-SWMM, thus the model may overestimate stages. Significant attempts were made to further refine the discharge for the G56 structure and to simulate the structure's actual operation; however, XP-SWMM has limitations in how it handles complex structures and variable flow regimes, so additional refinement would require code modifications.

A multitude of additional model runs were conducted to assess the impacts of channel improvements and the effects of the G56 gates. Also the efforts of varying storm events were modeled. A detailed analysis of all these individual factors indicates that channel improvement in a 6-mile-long reach starting from the tailwater side of the existing S39A structure and the replacement of the existing S39A structure with S527B structure are the most influential factors in causing the rise of the water surface elevation in the alternative models.

To assess the impacts of excess NSID pumping, Alternatives 1 and 2 were also modeled for the 25-year/72-hour storm event using the existing cross sections. The results showed no significant change in water levels over the Base Model. Essentially, the lower banks in the existing cross sections allowed some minor overtopping and this prevented significant stage increases. With the new cross sections included in Alternatives 1 and 2, less water is lost and thus the stage increases.

The effect of the G56 gate was modeled with two scenarios for the 25-year/72-hour storm event:

- Lower the gate at G-56 in the Base Model to allow water to hit the bottom of the gate and compare to the original Base Case.
- Raise the gate at G56 in Alternative 1 so that water does not hit the bottom of the gate and compare to Alternative 1.

These runs confirmed that the gate does have an effect on the upstream stage. In the first scenario with the lowered gate, there is an approximate 1.5-foot increase in stage height at G56 headwater (G56_H). In the second scenario, with the gate artificially raised, there is a similar drop in stage height stage throughout the channel.

A final set of model runs was performed to assess the effects of other storm events (10-year/24-hour storm and 100-year/72-hour storm) for Alternatives 1 and 2. The proposed cross sections and storage incorporated in the initial alternatives were also incorporated into these model runs. The results show stages for the 10-year event to be roughly 1.5 to 2.5 feet lower than in the 25-year storm event. The 100-year event had stages that were typically 0.5 feet to 1.7 feet higher than the 25-year storm event. There was little difference between the two alternatives.

In conclusion, the increased NSID discharge coupled with channel improvements do have an impact on the Hillsboro Canal. However, due to the sensitivity of the system to the amount of water input and discharged into the canal, a more detailed model is needed to better quantify the impact. At a minimum, this model should include code modifications to the XP-SWMM model for the G56 structure, calibrated hydrographs for all the input nodes that include the effects of internal control structures, and confirmation of the cross-sectional data including extensions of the cross sections to cover sufficient widths of the north and south overbank areas of the Hillsboro Canal.

Alternately or in conjunction with more detailed modeling, several options could also be evaluated to provide a clearer course of action with regards to the NSID reduced discharges to WCA 2A and increased discharges to the Hillsboro Canal. The first option involves using the Bishop Property as an above-ground reservoir with berms, seepage control and pumped inflow instead of an at grade, gravity flow storage area. The above-ground reservoir would provide more storage and could offset the effect of increased NSID discharges on the Hillsboro Canal. The second option focuses on allowing discharges from NSID to WCA 2A only during large rainfall events (10 year or greater). Since these events occur relatively infrequently, the impact of these low frequency events on water quality in WCA 2A could be evaluated. If the impacts are negligible, then changes in NSID operational protocol could be adopted in lieu of expensive storage options.

1 Introduction

On behalf of the South Florida Water Management District (SFWMD), Ecology & Environment, Inc. (E & E) has been tasked to conduct a feasibility-level hydrology and hydraulic (H&H) analysis for the North Springs Improvement District (NSID). The NSID Basin has an area of approximately 11 square miles, or 7,064 acres. It is located in northern Broward County along the eastern border of Water Conservation Area (WCA) 2A. The northern boundary is the Broward-Palm Beach County line. The Sawgrass Expressway transects the area, entering from the east and turning south as it exits along the western border. NSID contains the northern portion of the City of Coral Springs (north of Wiles Road) and the western portion of the City of Parkland (west of University Drive; SFWMD no date). Figure 1.1 illustrates the location of the NSID.

The analysis is divided into multiple tasks. Initial work efforts included the refinement of an existing H&H model that had been developed for the NSID Basin. The results of this model refinement were summarized in a report submitted by E & E to the SFWMD in October 2003. The next phase of investigation involved a feasibility-level H&H evaluation of alternatives to maximize the storage of surface waters within the basin. This evaluation was limited to hydraulic feasibility only and did not include factors such as cost, implementation, permitting, and site constraints, among others, that may affect feasibility.

The present investigation (Task 5) involves an initial evaluation of the impact of various discharges from the NSID and system improvements on the hydraulics of Hillsboro Canal. The study area includes the Hillsboro Basin and extends from the intersection of the Hillsboro Canal and the L-36 Canal eastward approximately 2 miles downstream of the G56 structure near Federal Highway (see Figure 1.2). This report summarizes the work efforts for the present H&H evaluation task, the results, and conclusions.

Figure 1.1 Site Map

Figure 1.2 Project Area

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2 Background

Florida's Everglades Forever Act (EFA), Florida Statutes (F.S.) 373.4592 establishes the Everglades Protection Area (EPA) that includes WCAs 1, 2A, 2B, 3A, 3B, the Arthur R. Marshall Loxahatchee National Wildlife Refuge, and the Everglades National Park. The EFA requires that the SFWMD obtain a permit from the Florida Department of Environmental Protection (FDEP) to operate and maintain water control structures, such as pumps, gates, and culverts which discharge water into, within, or from the EPA and which are not included in the Everglades Construction Project (ECP). The purpose of this permit is to establish limitations on discharge quantities with the objective to meet long-term water quality goals designed to restore and protect the EPA. The SFWMD obtained such a permit (#06, 502590709) from FDEP; this permit is designated as the Non-ECP Permit.

Subsequent to the issuance of this permit, the SFWMD initiated the implementation of the permit conditions through the creation of the Everglades Stormwater Program (ESP). The ESP includes eight basins, one of which is the NSID Basin. The long-term plan (LTP) of the Everglades restoration effort is to combine point source controls, basin-level solutions, and regional solutions in a system-wide approach to ensure that all waters discharged into the EPA meet the numeric phosphorous criterion and other applicable state water quality standards. In order to achieve this goal, the SFWMD has developed the *Everglades Protection Area Tributary Basins Long-Term Plan for Achieving Water Quality Goals (Long-Term Plan)* to ensure that all discharges from these basins to the EPA meet the final water quality objectives.

A concept under consideration is the elimination of NSID discharge into the EPA and redirecting all NSID water into the Hillsboro Canal. Presently, two existing NSID pump stations (the north, PS2, and south, PS1, pump stations) discharge water into the L-36 Canal which flows into the Hillsboro Canal to the north of the NSID Basin. NSID PS1 can also discharge water into WCA 2A. The NSID's surface water permit currently limits the discharges to the L-36 Canal when the capacity of the Hillsboro Canal is exceeded (reaches a specific elevation), and the excess NSID flows are discharged into WCA 2A via the NSID

PS1. The LTP for the NSID Basin recognizes that the conveyance of NSID flows to the Hillsboro Canal and the Hillsboro Site 1 Impoundment or other storage is the most cost-effective means of diverting all NSID stormwater runoff away from WCA 2A. In order to ensure this plan, it is necessary that the excess flow that would have been discharged to WCA 2A to the Hillsboro Canal be minimized. A previous investigation (Task 4) was aimed to evaluate the feasibility of storage of certain portions of the excess flow within an impoundment adjacent to the NSID Basin while the remaining portions of the excess flow are pumped at certain rates to the Hillsboro Canal. The present effort (Task 5) is directed to determine the impacts of this excess flow on the hydraulics of Hillsboro Canal.

The various drainage districts or areas of primary interest from which discharge to Hillsboro Canal takes place within the present scope of study are shown on Figure 2.1. This figure also illustrates the extents of the L-36 borrow canal and the Hillsboro Canal that are included in the present investigation (modeling).

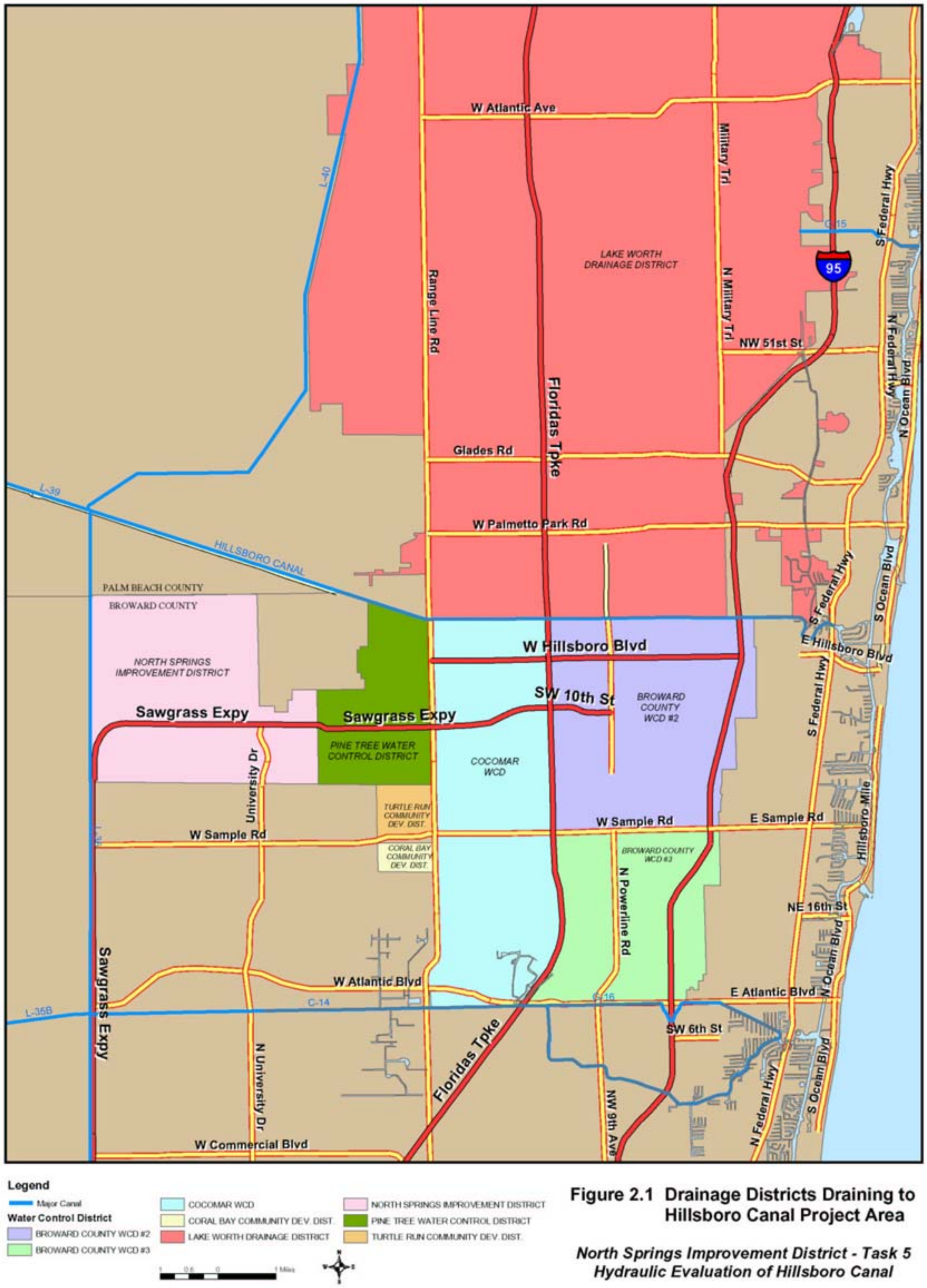

Figure 2.1 Drainage Districts Draining to Hillsboro Canal Project Area

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3 Modeling Objective

The primary objective of this feasibility-level modeling effort is to assess the potential impacts of the NSID discharge on Hillsboro Canal stages for the 10-year/24-hour, 25-year/72-hour, and 100-year/72-hour storm events. Future channel improvements and storage in the study area are also considered. Due to the preliminary nature of the modeling effort and simplifying assumptions, the model outputs are not intended to be viewed as absolute values but rather as an indication whether the NSID discharge and future features will affect the stages in the Hillsboro Canal.

Three initial model scenarios were devised to assess the impacts to the Hillsboro Canal. The first model scenario (base case) represents current conditions and includes the following:

- Discharge from NSID PS1 and PS2 is according to the pump station hydrographs developed under 'Base Model' condition in Task 4 with the exception that high flows from PS1 (444 cubic feet per second [cfs]) are not routed to the Hillsboro Canal. The high flows from PS1 are assumed to still go to WCA 2A.

- A storage area (henceforth designated as 'Bishop Area') exists for the excess NSID flow; the discharge from this area is self-contained and is not routed to the Hillsboro Canal.

- No Hillsboro Site 1 Impoundment exists; the discharge from this area (henceforth designated as 'HIS Area') is routed directly to Hillsboro Canal through an inlet to the north of the NSID (henceforth designated as 'HIS node on Hillsboro Canal').

- The cross-sectional geometry of Hillsboro Canal is the same in the model scenario as what exists under present conditions (no channel improvements).

- Hydraulic structure S39A is incorporated into the model with all geometric configurations that exist under present conditions.

- Hydraulic structure G56 (the Deerfield structure) is incorporated into the model with all geometric configurations that exist under present conditions.
- The inflow hydrographs to Hillsboro Canal have been developed to model discharge from the HIS Area, the unincorporated portions of Palm Beach County, Lake Worth Drainage District (LWDD), and Broward County for the 25-year/72-hour storm event.

In this study, the model that incorporates all of the criteria set above is designated as the Base Model and is used for calibration purposes (25-year event) and to compare the relative effects of increasing NSID discharge and system enhancements included in subsequent model runs.

In the second modeling scenario (Alternative 1), all NSID flow is discharged into the Hillsboro Canal and the Bishop Property rock pit is utilized as storage (detention) for the NSID. Further details are summarized below.

- Discharge from NSID PS1 and PS2 is according to the pump station hydrographs developed under 'Final Model' condition in Task 4.
- The Bishop Property rock pit is the storage area for excess NSID flow; there is no discharge from the Bishop Area to the Hillsboro Canal.
- Hillsboro Site 1 Impoundment exists; there is no discharge from the HIS Area to the Hillsboro Canal.
- The cross-sectional geometry of Hillsboro Canal incorporates proposed channel improvements to the east of the S39 hydraulic structure.
- Hydraulic structure S39A is incorporated into the model with all geometric configurations that have been proposed for its improvements.
- Hydraulic structure G56 is incorporated into the model with all geometric configurations that exist under present conditions.

- The inflow hydrographs to Hillsboro Canal have been developed to model discharge from the unincorporated portions of Palm Beach County, LWDD, and Broward County under three storm events. The storm events are 10-year/24-hour, 25-year/72-hour, and 100-year/72-hour events.

The third scenario (Alternative 2) is similar to the second scenario except for changes in NSID pumping:

- Discharge from NSID PS1 and PS2 is according to the maximum pumping rates as established by the SFWMD.
- There is no discharge from the Bishop Area to either the Hillsboro or L-36 Canals.
- Hillsboro Site 1 impoundment exists; there is no discharge from the HIS Area to the Hillsboro Canal.
- The cross-sectional geometry of Hillsboro Canal incorporates proposed channels improvements to the east of the S39 hydraulic structure.
- Hydraulic structure S39A is incorporated into the model with all geometric configurations that have been proposed for its improvements.
- Hydraulic structure G56 is incorporated into the model with all geometric configurations that exist under present conditions.
- The inflow hydrographs to Hillsboro Canal have been developed to model discharge from the unincorporated portions of Palm Beach County, LWDD, and Broward County under three storm events. The storm events are 10-year/24-hour, 25-year/72-hour, and 100-year/72-hour events.

Based on the results of the above model runs for the 25-year event, additional model simulations were conducted and are discussed in greater detail in Section 10 of this report.

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4 Modeling Procedures and Limitations

4.1 Modeling Software

All H&H model calculations in the present investigation were undertaken through the Storm Water Management Model – Version 9.1 (XP-SWMM v.9) software developed by XP-Software, Inc. Geographic Information System software Arc GIS (version 8.3), developed by ESRI, was used to manipulate the aerial photographs, line maps of the drainage areas and channels, point locations where cross-sectional geometry data were available, and locations of hydrologic nodes (points where inflow hydrographs to the L-36 and Hillsboro Canals were input to the models). The GIS data were used as a background to build the channel network within the XP-SWMM environment.

4.2 Basic Theory

The basic partial differential equations for solving unsteady open channel flow in XP-SWMM are derived from the gradually varied, one-dimensional unsteady-flow equations, otherwise known as the *St. Venant* equations. These equations combine the equations of continuity and conservation of momentum. Since these are non-linear, partial differential equations, various forms of this combination have been developed in hydraulic literature for the purpose of linearization and amenability to numerical solutions. The form used in the XP-SWMM software is given as Equation (1).

$$\frac{\partial Q}{\partial t} + \frac{g\,k}{R^{3/4}} QA|V| - V\frac{\partial A}{\partial t} + gA\frac{\partial H}{\partial x} + \left(K_{ec} + K_{ee}\right)\left[\frac{A}{2}\frac{\partial \left(Q/A\right)^2}{\partial x}\right] = 0 \qquad (1)$$

where:

V = Average Channel Velocity

A = Channel Cross-sectional Area

Q = Channel Flow

g = Gravitational Acceleration

$k = \left(\frac{n}{1.49}\right)^2$; n = Manning's Roughness (U.S. Customary unit has been used)

R = Hydraulic Radius

H = Hydraulic Head (= $z + h$; z = Channel Invert Elevation; h = Water Depth in Channel)

K_{ec} = Expansion or Contraction Loss Coefficient

K_{ee} = Entrance or Exit Loss Coefficient

Equation (1) is termed as the complete *dynamic* flow equation. Further simplifying assumptions result in either *kinematic* (if bed slope equals friction slope or $S_0 = S_f$) or *diffusion* wave equations. Although XP-SWMM allows the use any of these simplifications, the models presented in this study have been developed using the dynamic flow equation. In general, the standard dynamic flow equation used in XP-SWMM adopts the widely accepted procedure of ignoring the non-linear acceleration term in the *St. Venant* equation when the flow becomes super-critical but allows the user to choose the non-linear acceleration term. However, in the present calculations, this option has not been selected since there are only a few points where abrupt changes in the flow areas occur in the channel or friction slope approaches zero.

Equation (1) is solved by the method of finite difference approximation. The solution is implicit for channel (conduit) flow and explicit for junction (node) depth. The numerical solution of Equation (1) requires that upstream and downstream boundary conditions and initial conditions defined as inputs to the model.

4.3 Boundary Conditions

The upstream boundary of the model domain is the NSID pump station 1 (Node PS1) on the L-36 canal. The downstream boundary of the model domain is set at a point (Node N110) on Hillsboro Canal. This point is to the east of the Deerfield structure (G56) and at the intersection of Hillsboro Canal and U. S. Highway 1 (Figure 2.1).

Depending on the availability of data or presence of a hydraulic structure (such as a dam or tidal gate), XP-SWMM offers a wide variety of downstream (outfall) boundary conditions to be defined. In the present model, choice of the upstream boundary condition was straightforward since a pump station hydrograph [Q (t)] was input to the model at Node PS1. However, choice of the downstream boundary condition was somewhat arbitrary since it is neither a well-defined outfall structure nor a point where either a set of time series data [such as $Q(t)$ or $h(t)$] or other data (such as a rating curve $Q = ah^b$) are available. In fact, there is no such point downstream of the node where an inflow hydrograph was used as an input to simulate flow contribution to Hillsboro Canal from the surrounding drainage basins (unless the downstream boundary would be extended to a point where tidal data might be available). In light of this situation, the downstream boundary condition was chosen as the fixed backwater boundary where the minimum of either the normal depth or the critical depth for the flow in the outfall conduit would be maintained at all times.

4.4 Initial Conditions

XP-SWMM allows specification of initial conditions in *one* of the following forms:

- **Initial Head.** Initial heads (stage heights) can be specified at junctions. In this case, the model does not estimate the initial conduit flow if the initial conduit flow is entered as zero.

- **Initial Flow.** Initial flows in conduits may be input. In this case, the program computes the normal depth corresponding to the initial flow. Junction heads are approximated as the average heads of adjacent conduits for purposes of beginning the computation sequence.

- **Constant Inflow.** Constant inflows may be input to the system and the initial conditions established by letting the model run for enough time steps to establish steady-state flows and heads. XP-SWMM offers a capability (hot-start) that can then be used to store these initial conditions for use at the beginning of additional simulations.

From the discussion above, it is evident that setting up an accurate set of initial conditions requires availability of long-term historical data providing either stream flow or stage height or both of these measurements. Such historical data collected at certain stations along Hillsboro Canal—albeit limited in nature—have been obtained from the SFWMD web site (DBHYDRO Browser). These data are summarized in Tables 4.1 and 4.2.

Table 4.1 Statistical Summary of Historical Data on Stage Heights[a] at Various Gauging Stations on Hillsboro Canal

Station Name	DBKEY No.	Period of Record	Coordinates		Mean	Maximum	Minimum
			X	Y			
G 56_H	05727	1985 - 2004	940840.3	725811.67	7.179	8.776	1.781
G 56_T	05729	1985 - 2004	940840.30	725811.67	1.09	6.523	-0.812
G 56_T	12304	1985 - 2004	940840.30	725811.67	2.333	7.49	-0.59
G 56_T	12305	1985 - 2004	940840.30	725811.67	-0.157	5.86	-1.55
HILLS	00373	1975 - 2002	914175.33	726448.55	7.673	12.09	4.55

Notes: [a] Feet with respect to the National Geodetic Vertical Datum.
Underscore '*H*' (_H) implies headwater and underscore '*T*' (_T) implies tailwater of a hydraulic structure. The station 'HILLS' roughly corresponds to node HN887 of the present models.

Table 4.2 Statistical Summary of Historical Data on Stream Flow at Various Gauging Stations on Hillsboro Canal

Station Name	DBKEY No.	Period of Record	Coordinates		Flow (cubic feet per second)		
			X	Y	Mean	Maximum	Minimum
G56_S	15707	1991- 2004	940840.30	725811.67	254.367	1583.54	957.517
HILLS	00374	1975 - 2002	914175.33	726448.55	203.831	1300.00	-247.00
LWD.17E_P	12722	1988 - 2004	928191.36	725930.46	6.864	79.224	0.00
LWD.17E_P	12723	1988 - 2004	801014.15	927827.40	13.921	100.00	0.00
NSID2	LG896	1975 - 2004	876958.89	691808.34	-	315.60	0.00

Note: The station 'HILLS' roughly corresponds to node HN887 of the present models.
Key :
LWD = Lake Worth Drainage District
NSID = North Springs Improvement District.

Since limited data suitable for setting up an accurate set of initial conditions is available, the following approach has been taken. A set of trial runs was made where initial head was assigned to each of the links starting from the upstream conduit to the conduit joining the G56-Headwater (G56_H) node such that the head gradually decreases from 7.80 to 7.18 feet. From the G56-Tailwater (G56_T) node to the downstream boundary such initial heads declined gradually from 2.33 to 1.1 feet (all elevations are relative to the National Geodetic Vertical Datum [NGVD] 29). These selections were made based on the information presented in Table 4.1. However, when initial conditions were defined in this way, the stage heights at the beginning of the simulation period (storm event) fell well below the long-term average values. Consequently, the choice of these initial conditions was abandoned.

Another set of trial runs were made where all conduits had an initial flow of 200 cfs. This value was chosen on the basis of the mean value given for the station 'HILLS' (Table 4.2) where the period of record is longest. In either case, no 'constant inflow' value was assigned to either the upstream boundary (node) or any of the internal nodes. Physically, these constraints imply that the S39 structure is always closed (no flow from WCA 2 to Hillsboro Canal) and NSID PS1 receives no flow from the upstream direction. If no constant inflow is input to the upstream boundary condition and either initial heads or initial flows are assigned to the nodes and the links respectively, then prior to the beginning of the storm flow surge, the stage heights will gradually decline, implying a gradual drying of the channel. For this reason, the dry runs to set up the initial conditions were made for a simulation period of only 24 hours. Both of these approaches (i.e., specifying either initial head or initial flow across the network) yield almost identical results. Consequently, all subsequent runs with the storm hydrographs were made after simulating flow for 24 hours with constant inflows specified at certain nodes such that the desired stage heights are attained at the nodes. These initial conditions were used as the input files (hot re-start option) for runs simulating the storm events. The initial conditions are described as follows.

Constant inflows at certain nodes upstream of the G56 structure were assigned so that at the end of a 24-hour dry-condition run, the stage heights at G56_H and G56_T are 7.5 and 2.4 feet, respectively (Figures 4.1 and 4.2). In these runs, the gate at G56 is slightly open (up to 2.9 feet) and the flow through G56 is 2,177 cfs (Figure 4.3). In this way, the stage at the node N79 (S39A tailwater [S39A_T] side) is 9.6 feet (Figure 4.4).

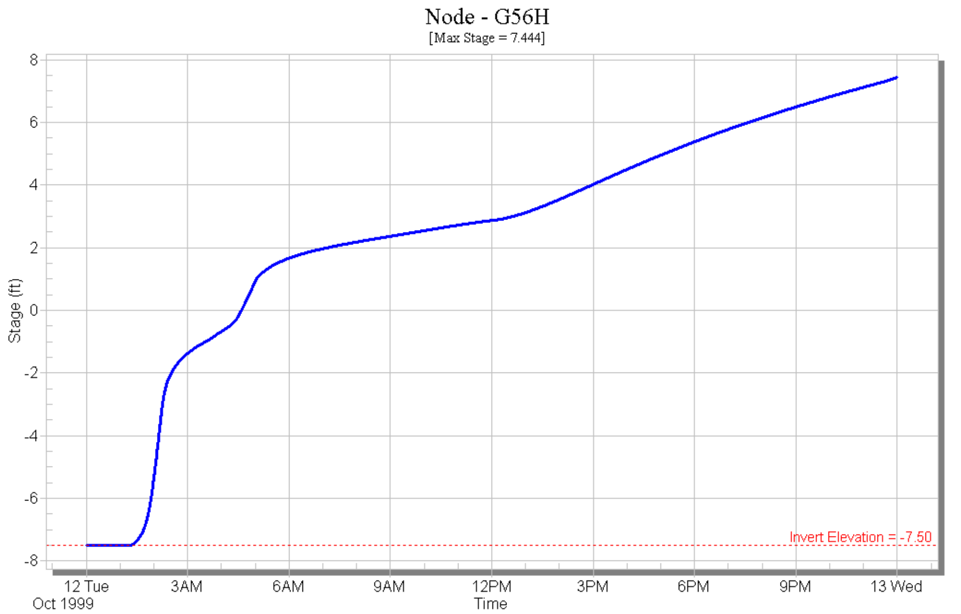

Figure 4.1 Stage Height at G56_H at the End of 24-hour Dry-condition Run to Define the Initial Condition of the Storm Event Runs (Base Model)

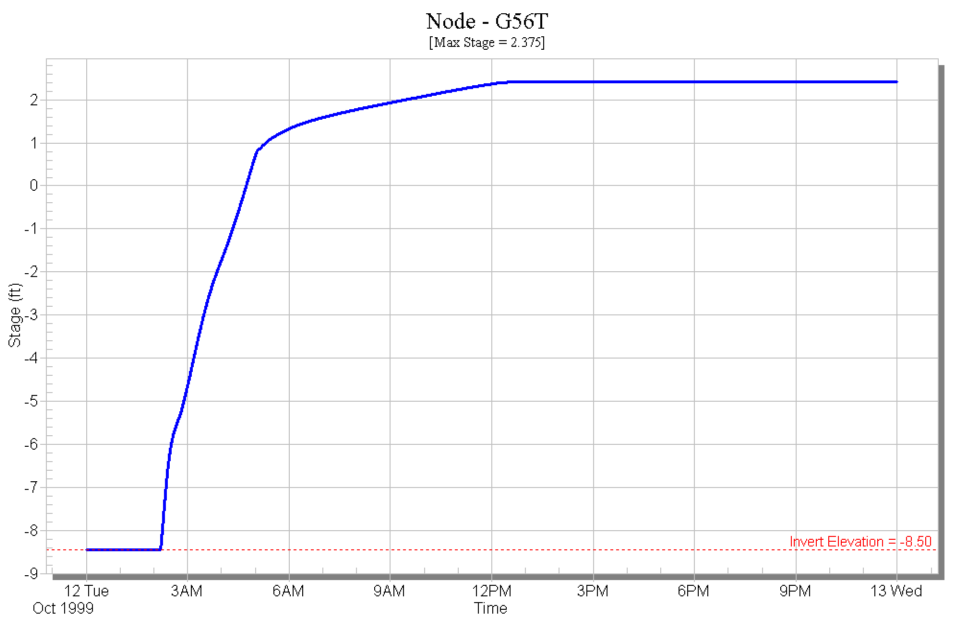

Figure 4.2 Stage Height at G56_T at the End of 24-hour Dry-condition Run to Define the Initial Condition of the Storm Event Runs (Base Model)

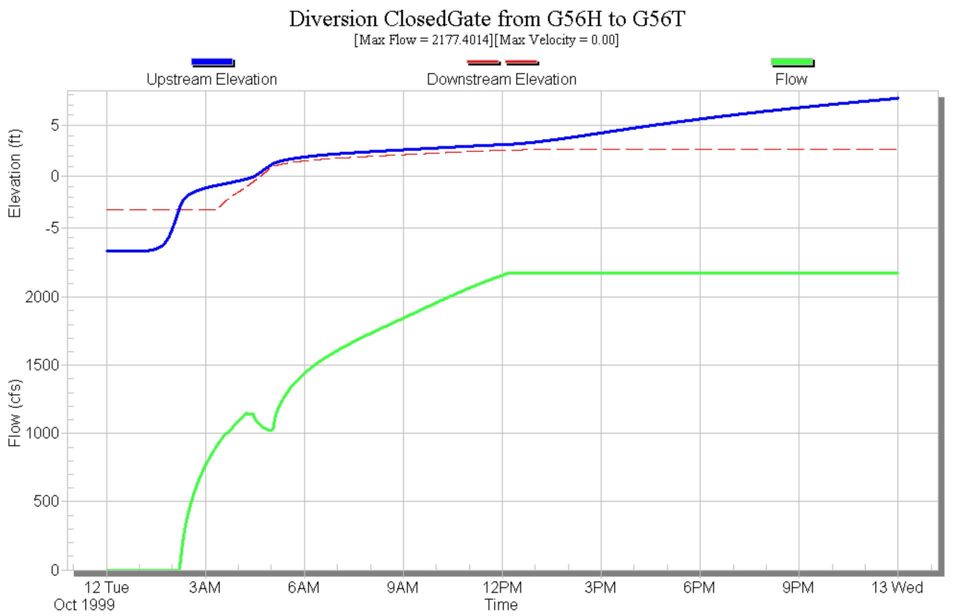

Figure 4.3 Flow Through the Deerfield (G56) Structure at the End of 24-hour Dry-condition Run to Define the Initial Condition of the Storm Event Runs (Base Model)

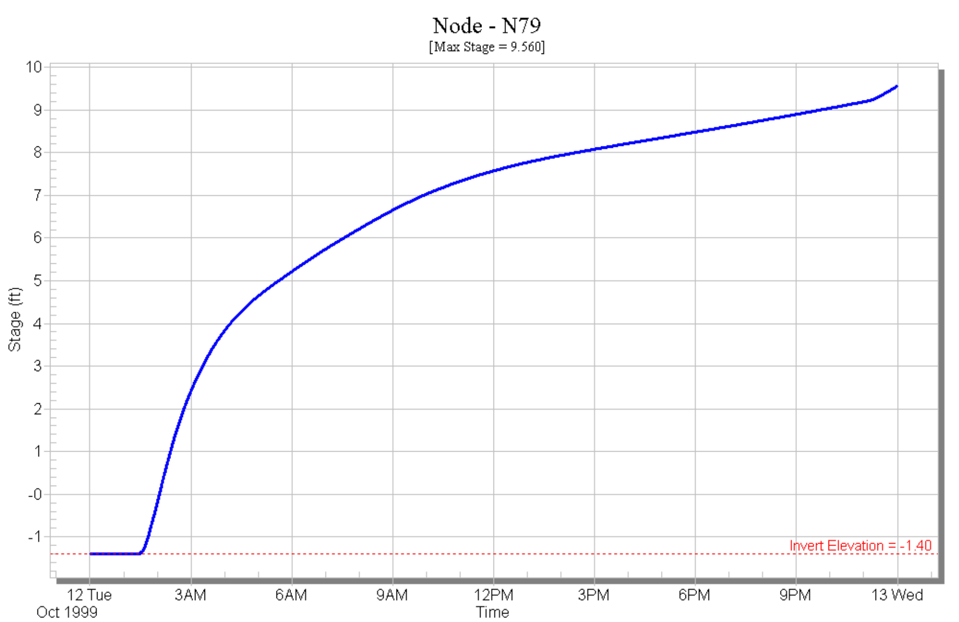

Figure 4.4 Stage Height at S39A_T at the End of 24-hour Dry-condition Run to Define the Initial Condition of the Storm Event Runs (Base Model)

In the alternative models, the initial conditions were defined in the same way except that due to channel improvements, adjustments in the constant inflow values at certain nodes were necessary. Figures 4.5 through 4.8 show the initial conditions for the storm periods at the same places noted above.

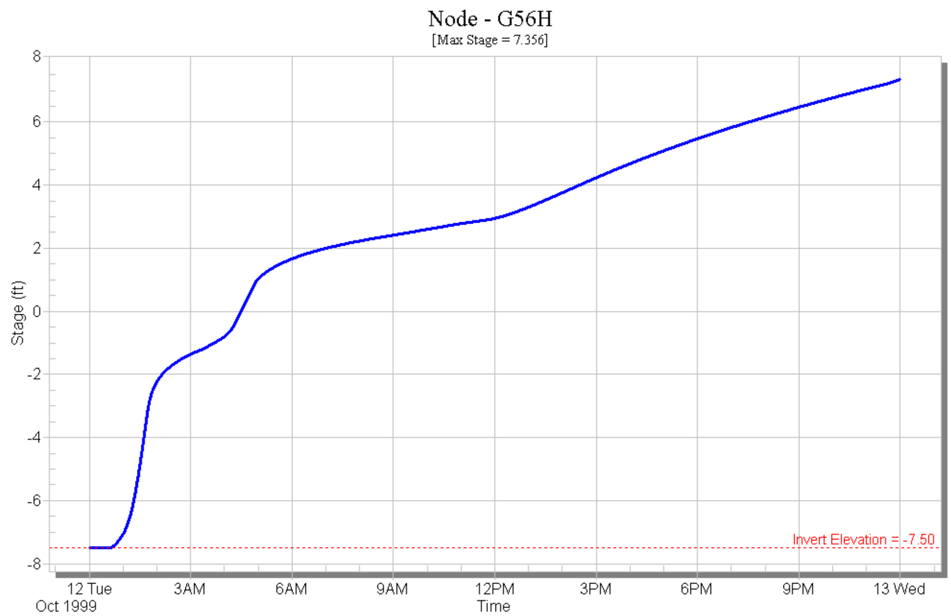

Figure 4.5 Stage Height at G56_H at the End of 24-hour Dry-condition Run to Define the Initial Condition of the Storm Event Runs (Alternative Models).

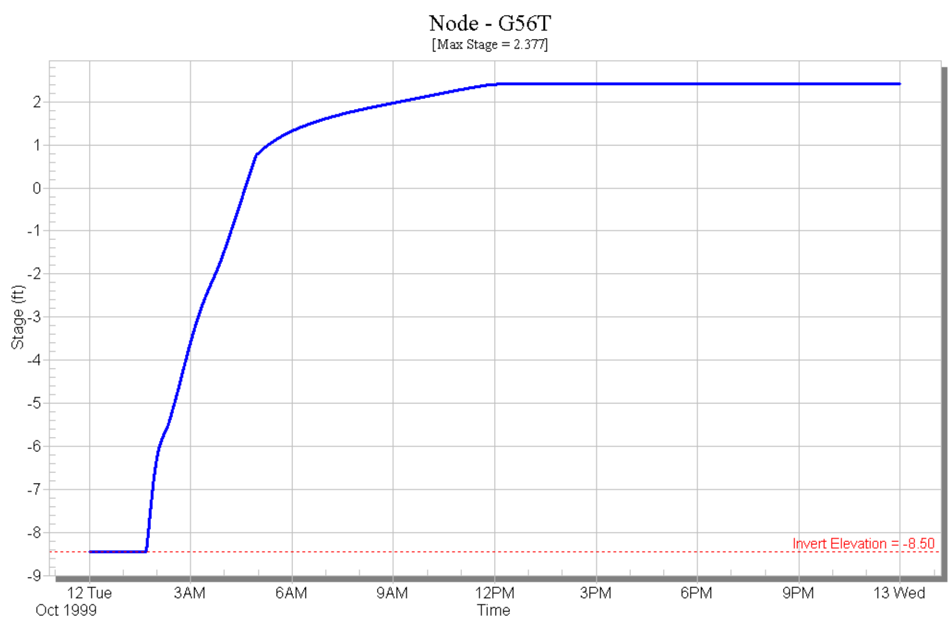

Figure 4.6 Stage Height at G56_T at the End of 24-hour Dry-condition Run to Define the Initial Condition of the Storm Event Runs (Alternative Models)

4 Modeling Procedures and Limitations

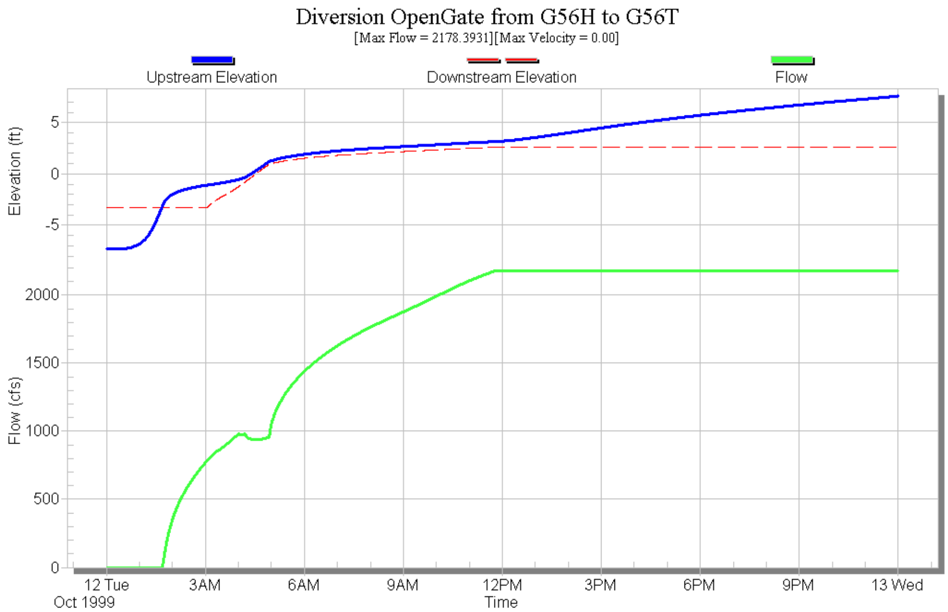

Figure 4.7 Flow Through the Deerfield (G56) Structure at the End of 24-hour Dry-condition Run to Define the Initial Condition of the Storm Event Runs (Alternative Models)

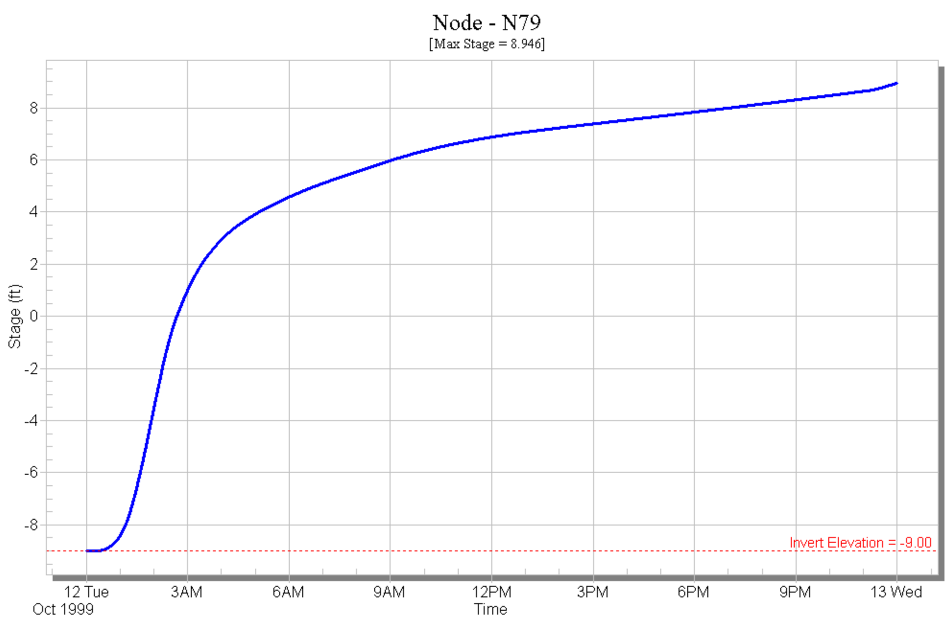

Figure 4.8 Stage Height at S39A_T at the End of 24-hour Dry-condition Run to Define the Initial Condition of the Storm Event Runs (Alternative Models)

4.5 Modeling Limitations

The modeling effort is based on a feasibility-level of evaluation. A number of simplifying conditions have been made in consultation with the SFWMD and include the following:

- **Use of synthetic inflow hydrographs.** Synthetic hydrographs were generated by Marco Water Engineering to combine all inflows from a particular drainage basin into the Hillsboro Canal by considering average hydrologic properties of the basin, area, and 10-, 25-, and 100-year storm total precipitations, and temporal rainfall distribution patterns in these areas developed by the SFWMD. In this way, a gross estimate of stormwater runoff from the adjacent basins was made by ignoring the hydrologic details of the sub-basins and rainfall patterns that may, in actuality, affect the magnitude of the volumetric contributions. In other words, the hydrographs were not calibrated against actual storm events of similar depth-duration-frequency. Since the purpose of the investigation was to determine the changes in hydraulics of the Hillsboro Canal under base and alternative conditions from identical rainfall events, the approach is reasonable. However, the results from this approach may not produce the results that are actually observed under base conditions due to the limitations noted above.

- **Use of spatially lumped nodes.** While separate synthetic hydrographs were developed for all major inflow structures, minor inflow structures or sheet flows within a discrete area were combined into one hydrograph and shown as a single discharge point. While this does not precisely match what actually occurs, it is uniformly applied to all the model runs.

- **No incorporation of storage node when overtopping occurs.** As the water stages increase, a limited amount of bank overtopping may occur and this water is not modeled to return back to the Hillsboro Canal. More detailed information is necessary to accurately model when and how this water will return to the canal, however, this may not be critical for assessing peak stages. As will be discussed in more detail in subsequent sections, model runs indicate that the total volume of water lost from the system due to overtopping is limited. More importantly, this water likely will not flow back into the canal until well after the peak stage or will flow back in gradually.

- **Available cross-sectional data.** The cross-sectional data used in the models were made available by the SFWMD from previous studies and varied sources. In many cases, the cross-sectional geometry was not completely defined far beyond the left and right overbanks and only defined the main channels. For these sections, if overtopping occurred, there was no real control to define the geometry of the overbank areas. Furthermore, due to varied sources of the data, the systematic errors in one source when combined with those from another source caused some random errors in the geometry. Since the solutions of energy or momentum equations for modeling hydraulics of a channel are sensitive to consistency of the errors in geometrical data, the random error introduced in the defined channel and overbank geometry make the models less accurate.

While XP-SWMM is a powerful tool, it does have limitations for its application to the Hillsboro Canal and more particularly the G56 structure unless customized model code modifications are made. This issue is further illustrated in Appendix A.

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5 Input Data for Base Condition: Base Model

The Base Model evaluates the base condition and optimizes all parameters through calibration against a large storm event for which stage heights at certain gauging stations along Hillsboro Canal are available. The key features of this model are iterated below.

- Bishop Property is assumed to be self-contained.
- Hillsboro Site 1 Impoundment is not included as a storage basin within the area to the north of Hillsboro Canal (Palm Beach County). Rather, this area is used as a basin from where runoff hydrograph is routed to Hillsboro canal directly.
- Existing geometry of Hillsboro canal and L-36 borrow canal is used.

The Base Model utilizes the following information.

5.1 Cross Sections

The canal network in XP-SWMM has been constructed by introducing dummy nodes between a numbers of links or conduits. The purpose of creating different links (reaches) is to accommodate different cross sections of the canal. Each link is represented by a uniform cross section. Thus, the dummy nodes in the network model are used simply to connect links with uniform cross sections.

For the Base Model, the existing geometry of the cross sections is used. In other words, no channel improvement is incorporated into the model; only the following data are incorporated into the model to capture the existing geometry.

During the initial stage of the model development, canal cross-sectional data of 140 stations were entered into the XP-SWMM global database. These cross-sectional data were obtained from Appendix C of the *Water Management Plan for the Hillsboro Canal Basin, Phase I Flooding Potential Report* (*Dynamic Wave Operational [DWOPER] Model, May 1991*). Since there were 140 cross sections, sets of cross sections with the same X and Y values were selected and each set was used to represent one link. Nevertheless, these data were later considered to be outdated and were only used in parts.

A new set of existing canal cross-sectional data was obtained from the Florida Atlantic University (June 1999). In this data set, the station locations are based on the Universal Transverse Mercator (UTM) coordinates system relative to the 1983 North American Datum (NAD 83). GIS was used to prepare digital maps of the station locations. In addition, an aerial photograph (mosaic from three individual photographs and geo-referenced to the same coordinate system) was used as a background during construction of the canal network.

Since, in general, the newer cross sections are deeper than the old cross sections, the old cross-sectional locations between S39 and G56 were completely replaced with the new cross sections. The cross sections on L-36 canal were not changed and they are based on old cross sections (1991 data, as noted above). For the canal section to the east of I-95, cross-sectional data from a post-dredge survey prepared by Sea Systems Corporation, Pompano Beach, Florida, were used. Immediately downstream of the G56 structure, one cross section was used from the study conducted by Burns & McDonnell (1989).

Table 5.1 provides the summary of the data used in the construction of the cross-sectional geometry of the various reaches (links) of Hillsboro Canal.

As discussed above, for the purpose of accommodating the various cross-sectional geometries of the channels, both L-36 and Hillsboro Canals are divided into a number of segments (conduits), which are referred to as links within XP-SWMM. The various links and their geometric parameters used in the model are summarized in Table 5.2. The hydraulic parameters of the links are summarized in Table 5.3. For the initial model runs, the value of Manning's *n* used for the main channel was 0.035 and for both the left and right overbanks, it was 0.030. However, subsequent calibration produced the values shown in Table 5.3. Figure 5.1 shows the node-link network of the Base Model.

Figure 5.1 (11 x 17)

Back of Figure 5.1 (11x17)

5 Input Data for Base Condition: Base Model

Table 5.1 Summary of the Cross-sectional Data for the L-36 and Hillsboro Canals used in Model 1 (Base Model)

Canal Section	No. of Cross Sections (Links)	Comments	Reference	Source	Future Reference
L-36 (from PS1 to S39A)	4	Between PS1 and S 39A, there are 36 cross sections out of which four cross sections were selected based on similar geometries between stations 1 to 10, 11 to 19, 20 to 31 and 32 to 36. Each link is approximately 4,000 ft long.	Water Management Plan for the Hillsboro Canal Basin Phase I – Flooding Potential Report, Appendix C DWOPER model input and output data	Greenhorne & O'Mara, Inc. May 1991	Additional cross sections can be added by subdividing the existing links into smaller links.
S39A to G56	30	These cross-sectional stations are placed based on a GIS shapefile created from station coordinates given in UTM system. Each link, represented by uniform cross-sectional geometry, is approximately 1,700 ft long.	Hillsboro Canal Cross Section and Longitudinal Profiling Report	Florida Atlantic University June 1999	Due to closely spaced nature of the cross sections, compared to the total length of the modeled section of Hillsboro Canal, placement of additional cross-sectional stations may not be necessary.
Upstream and Downstream of G56 Structure	2	Bottom width of 90 ft and side slopes 2.5 H: 1V are used, top of bank is assumed 16 ft NGVD	G56 cross-section drawings	Burns & McDonnell 1989	
G56 to I-95 a) CS16 to HN910	3		G56 cross-section drawings	Burns & McDonnell 1989	There are no data in the new data set; however, G56 cross-section data are used.
b) HN910 to N50	1	The cross section is from Post Dredge Survey at station 50.	Post Dredge Survey Hillsboro Canal Deepening	Sea Systems Corporation July 1998	There are no data in the new data set; however, Post Dredge Survey data are used.
I-95 to east end of canal	10	The lengths of the links vary from approximately 300 to 600 ft.	Post Dredge Survey Hillsboro Canal Deepening	Sea Systems Corporation July 1998	Additional cross sections at the end of the canal can be added but that may not be necessary.

Key :
DWOPER = Dynamic Wave Operational.
ft = Feet.
GIS = Geographical Information System.
NGVD = National Geodetic Vertical Datum
UTM = Universal Transverse Mercator.

5 Input Data for Base Condition: Base Model

Table 5.2 Summary of the Geometric Parameters of the Links used in Base Model

Link Name	Length (ft)	Upstream Crown Elevation (ft)	Downstream Crown Elevation (ft)	Upstream Invert Elevation (NGVD)	Downstream Invert Elevation (NGVD)	Average depth (ft)	Upstream Node Name	Downstream Node Name
XS1	6315	20	20	-1	-1	21	PS1	N74
XS11	3729	20	20	-1	-1	21	N74	PS2
XS11.1	3729	19	20	-1	0	20	PS2	N77
XS20	7895.12	20	19	0	-1	20	N77	N78
XS32	2838.3	20	20	-1	-1	21	N78	N78.1
S39A	54	5	4.6	-1	-1.4	6	N78.1	N79
NXS1	1738.28	15.3	14.7	-1.4	-2	16.7	N79	N95
NXS2	1959.27	14.7	13.6	-2	-3.1	16.7	N95	N97
NXS3	1965.54	12.2	11.9	-3.1	-3.4	15.3	N97	BISHOP
NXS4	2126.68	11.9	12.1	-3.4	-3.2	15.3	BISHOP	HIS
NXS5	2180.08	16	14.9	-3.2	-4.3	19.2	HIS	N10
NXS6	1940.7	14.9	14.4	-4.3	-4.8	19.2	N10	N129
NXS7	2000.81	22.1	21.1	-4.8	-5.8	26.9	N129	N101
NXS8	1740.31	21.3	23.1	-5.8	-4	27.1	N101	N102
NXS9	2052.91	20.2	21.1	-4	-3.1	24.2	N102	UWPBN
NXS10	2301.97	21.1	18.4	-3.1	-5.8	24.2	UWPBN	UWPBS
NXS11	2066.24	18.5	20.1	-5.8	-4.2	24.3	UWPBS	N105
NXS12	2000.13	20.1	18	-4.2	-6.3	24.3	N105	HNS39_T
NXS13	2029.65	17.9	17.1	-6.3	-7.1	24.2	HNS39_T	N158
NXS14	1926.49	17.1	18	-7.1	-6.2	24.2	N158	N159
NXS15	1942.35	23.1	22.7	-6.2	-6.6	29.3	N159	HN887
NXS16	1040.62	31.3	31.3	-6.6	-6.6	37.9	HN887	HN888
NXS16.a	1040.62	25.8	24.3	-6.6	-8.1	32.4	HN888	HN889
NXS17	845	12.6	15.5	-8.1	-5.2	20.7	HN889	CS14W
NXS18	1073	15.5	15.5	-5.2	-5.2	20.7	CS14W	HN890
NXS18.1	1074	15.5	14	-5.2	-6.7	20.7	HN890	CS14G
NXS19	2006.08	24.5	22.9	-6.7	-8.3	31.2	CS14G	HN891
NXS20	774.05	18.5	19.8	-8.3	-7	26.8	HN891	HN892
NXS21	1446.2	19.3	19.8	-7	-6.5	26.3	HN892	HN893
NXS22	2147.08	15.3	16.1	-6.5	-5.7	21.8	HN893	CS17W
NXS23	1194.6	16.1	17.2	-5.7	-4.6	21.8	CS17W	HN894
NXS24	1195.57	15.7	15.8	-4.6	-4.5	20.3	HN894	HN895
NXS25	497	15.8	15.8	-4.5	-4.5	20.3	HN895	CS17E

Table 5.2 Summary of the Geometric Parameters of the Links used in Base Model

Link Name	Length (ft)	Upstream Crown Elevation (ft)	Downstream Crown Elevation (ft)	Upstream Invert Elevation (NGVD)	Downstream Invert Elevation (NGVD)	Average depth (ft)	Upstream Node Name	Downstream Node Name
NXS25.2	497	15.8	15.8	-4.5	-4.5	20.3	CS17E	HN896
NXS25.1	994	15.8	16.7	-4.5	-3.6	20.3	HN896	HN897
NXS26	1056.04	16.8	16.8	-3.6	-3.6	20.4	HN897	HN898
NXS26.1	1056.04	16.8	16.5	-3.6	-3.9	20.4	HN898	HN899
NXS27	986.04	16.1	16.1	-3.9	-3.9	20	HN899	HN900
NXS27.1	985.03	13.3	14	-3.9	-3.2	17.2	HN900	HN901
NXS28	1003.04	14	14	-3.2	-3.2	17.2	HN901	HN902
NXS28.1	1004.04	14	12.2	-3.2	-5	17.2	HN902	HN903
NXS29	1847.33	15.3	16	-5	-4.3	20.3	HN903	HN904
NXS30	1848	16	12.8	-4.3	-7.5	20.3	HN904	HN905
G56US	412.02	16	16	-7.5	-7.5	23.5	HN905	G56_H
OpenGate							G56_H	G56_T
G56DS	225	16	15.55	-8.5	-8.95	24.5	G56_T	G56_T.1
G56DS.1	225.01	16.45	16	-8.95	-9.4	25.4	G56_T.1	CS16
G56XS	1187.2	16	15.1	-8.5	-9.4	24.5	CS16	HN911
G56XS.1	2059	14.6	16	-9.4	-8	24	HN911	HN910
NXS50.1	1037	15	15	-8	-8	23	HN910	N50
NXS50	396.08	15	14.9	-8	-8.1	23	N50	N5
NXS54	299.14	13.7	15	-8.1	-6.8	21.8	N5	N7
NXS57	299	15	13.8	-6.8	-8	21.8	N7	N6
NXS60	405	15	15.3	-7.1	-6.8	22.1	N6	N4
NXS64	598	13.8	15	-8	-6.8	21.8	N4	N8
NXS70	590	15	13.8	-6.8	-8	21.8	N8	N9
NXS76	796	14.9	15	-8	-7.9	22.9	N9	HN908
NXS84	603	15	14.9	-7.9	-8	22.9	HN908	N1
NXS90	607.56	14.7	15	-8	-7.7	22.7	N1	N2
NXS96	400.21	15	15	-7.7	-7.7	22.7	N2	N3
NXS100	400.36	15	16	-7.7	-6.7	22.7	N3	E4
NXS104	561.26	14	14.2	-7.7	-7.5	21.7	E4	N110

Key :
_H = Headwater. _T = Tailwater.
ft = Feet. NGVD = National Geodetic Vertical Datum

5 Input Data for Base Condition: Base Model

Table 5.3 Summary of the Hydraulic Parameters of the Links used in Model 1 (Base Model)

Link Name	Cross-section Identification Number	Contraction Expansion Loss Coeff.	Entrance Loss	Exit Loss	Other Loss	Entrance/Exit Loss Type	Left Overbank Manning's *n*	Main Channel Manning's *n*	Right Overbank Manning's *n*
XS1	1.1	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
XS11	1.2	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
XS11.1	1.3	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
XS20	1.4	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
XS32	1.5	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
S39A	1.5	0	0.2	0.9	0.1	Energy/Loss Coeff	0.03	0.022	0.03
NXS1	1.6	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS2	1.7	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS3	1.8	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS4	1.9	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS5	2	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS6	2.1	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS7	2.2	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS8	2.3	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS9	2.4	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS10	2.5	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS11	2.6	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS12	2.7	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS13	2.8	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS14	2.9	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS15	3	0	0	0	0.5	Energy/Loss Coeff	0.03	0.025	0.03
NXS16	3.1	0	0	0	0.5	Energy/Loss Coeff	0.03	0.025	0.03
NXS16.a	3.2	0	0	0	0.7	Energy/Loss Coeff	0.03	0.025	0.03
NXS17	3.3	0	0	0	0.2	Energy/Loss Coeff	0.03	0.025	0.03
NXS18	3.4	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS18.1	3.5	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS19	3.6	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS20	3.7	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS21	3.8	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS22	3.9	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS23	4	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS24	4.1	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS25	4.2	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS25.2	4.3	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS25.1	4.4	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS26	4.5	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03

Table 5.3 Summary of the Hydraulic Parameters of the Links used in Model 1 (Base Model)

Link Name	Cross-section Identification Number	Contraction Expansion Loss Coeff.	Entrance Loss	Exit Loss	Other Loss	Entrance/Exit Loss Type	Left Overbank Manning's *n*	Main Channel Manning's *n*	Right Overbank Manning's *n*
NXS26.1	4.6	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS27	4.7	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS27.1	4.8	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS28	4.9	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS28.1	5	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS29	5.1	0.5	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS30	5.2	0.5	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
G56US	5.3	0.3	0	0	0	Energy/Loss Coeff	0.03	0.02	0.03
OpenGate									
G56DS	5.3	0.3	0	0	0	Energy/Loss Coeff	0.03	0.02	0.03
G56DS.1	5.3	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
G56XS	5.4	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
G56XS.1	5.5	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS50.1	5.6	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS50	5.7	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS54	5.8	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS57	5.9	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS60	6	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS64	6.1	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS70	6.2	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS76	6.3	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS84	6.4	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS90	6.5	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS96	6.6	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
NXS100	6.7	0	0	0	0.1	Energy/Loss Coeff	0.03	0.025	0.03
NXS104	6.8	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03

5.2 Inflow Hydrographs

The following drainage basins with corresponding nodes have been incorporated in the Base Model.

1) **North Springs Improvement District.** Discharge from the NSID to L-36 Canal takes place according to two pump station hydrographs obtained from the Base Model of Task 4 with the exception that the high flows (444 cfs) in PS1 are not routed to the Hillsboro Canal and the inflow hydrograph to the Hillsboro Canal changed accordingly. These pump stations (PS1 and PS2) are designated as nodes PS1 and PS2.

2) The principal drainage basin to the north of Hillsboro Canal is the LWDD. However, the LWDD (Palm Beach County) does not encompass the area to its southwest that lies to the north of Hillsboro Canal. There is another area just north of Hillsboro Canal where Hillsboro Impoundment will be located. In the Base Model, inflow from this area to Hillsboro Canal is modeled through a node, designated as "HIS" (see Table 5.2 and Figure 5.1 for its location).

3) Two nodes designated as "UWPBN" and "UWPPS" route the flow from the unincorporated area in Palm Beach County (the area to the southwest of the LWDD).

4) Six nodes, designated as CS14W, CS 14G, CS17W, CS17E, and E4 are used to model the inflow from the LWDD into Hillsboro Canal.

5) There are four drainage districts or water control districts to the south of Hillsboro Canal (Broward County). The nodes (total 23) used to represent flow from these areas into Hillsboro Canal are designated as follows.

   - **Cypress Head Water Control District.** Node HNS39_T.

   - **Pine Tree Water Control District.** Nodes HN887, HN888, and HN889.

- **Cocomar Water Control District.** Nodes HN890, HN891, HN892, HN893, and HN894; note that there is no flow at nodes HN890 and HN891.
- **Water Control District 2.** Nodes HN895, HN896, HN897, HN898, HN899, HN900, HN901, HN902, HN903, HN904, HN905, HN906, HN907, HN909, and HN911; note that there is no flow at HN905.

6) In addition, two nodes designated as HN908 and HN910 represent drainage from Broward County to the east of Water Control District 2.

Marco Water Engineering provided the hydrographs for the three storm events, from each of the basins noted above. Hydrographs for the LWDD were developed based on basin characteristics and simplifying assumptions using the ICPR Model. The Broward County hydrographs were extracted from previous XP-SWMM modeling efforts conducted by Broward County. Further details are provided in Appendix B. These hydrographs have been input at hydrologic nodes along Hillsboro Canal.

5.3 Hydraulic Structures

Two hydraulic structures are incorporated into the models:

1) **G56 Structure.** G56 is a gated spillway with the following geometric data that are used to incorporate it in the model:

- Crest shape: Ogee.
- Weir length, L: 60 feet.
- Conduit length (Weir width): 38 feet.
- Crest elevation: -3.5 feet NGVD.
- Design head, H_0: 11 feet (7.5 + 3.5).

- Top elevation of gate fully closed: 8.7 feet NGVD.
- Top elevation of gate fully open: 9.8 feet NGVD.
- Under normal conditions, the structure maintains headwater elevation between 7.0 and 8.0.

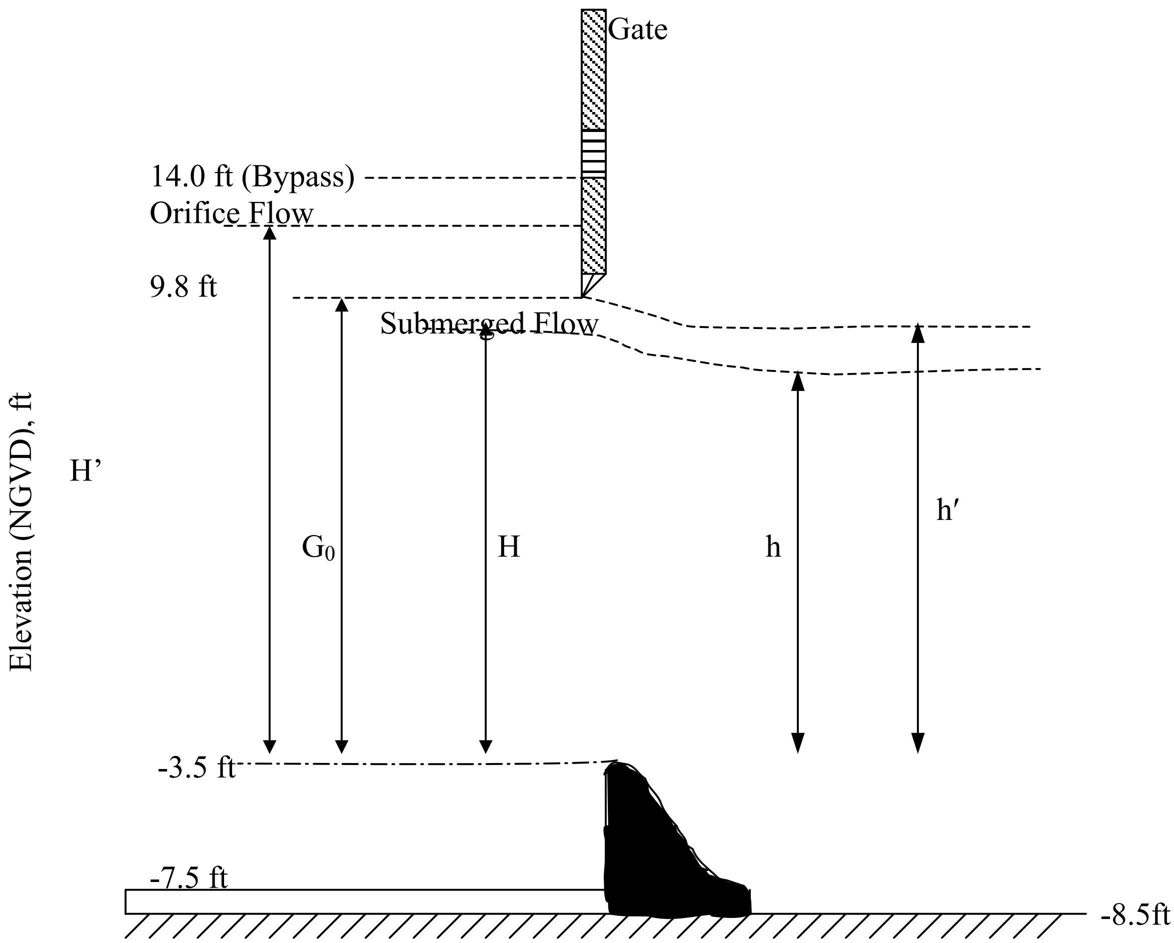

Figure 5.2 G56 Hydraulic Structure

The structure is incorporated as a gated weir with discharge equation in XP-SWMM given as:

$$Q = CLH^{1.5} \tag{2}$$

where:

Q = Flow rate in cfs

C = Weir flow coefficient (C_o= Coefficient at design head)

H = Upstream energy head above the spillway crest

The SFWMD (Ansar and Alexis 2003) recommends the use of 2.9 as the weir coefficient. However, a discharge coefficient of 4.1 is also used for free flow over an ogee-shaped spillway (United States Bureau of Reclamation [USBR] 1987) in most other parts of the United States. Model runs were initiated using a weir coefficient of 2.9, however, subsequent runs using a weir coefficient of 4.1 resulted in a closer calibration. Therefore, 4.1 was used for the model runs presented in this report.

2) **S39A Structure.** S39A is a culvert located in the L-36 borrow canal at its junction with Hillsboro Canal.

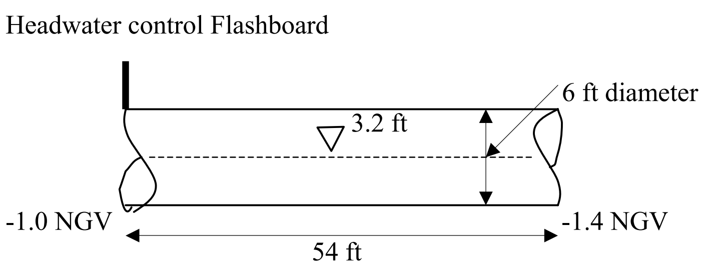

Figure 5.3 S39A Hydraulic Structure

The other parameters of this structure used in the model are:

- Number of barrels: three (3), corrugated metal pipe.
- Normal flow line elevation: 3.2 feet.
- Modeled as a closed, circular conduit with three barrels.
- Routing option: Standard dynamic wave.
- Energy loss coefficient:

a) Entrance loss, $K = 0.5$ (headwall and wing wall square edge),

b) Exit loss, $K = 0.4$ (with bend).

- Roughness coefficient: Manning's $n = 0.024$

- Contraction – Expansion loss coefficient to next conduit = 0.3

6 Input Data for the Models with Storage Alternatives

For modeling the two initial alternative scenarios, the following modifications to the Base Model (Model 1) were made.

6.1 Modifications to the Cross Sections of Hillsboro Canal

The proposed Hillsboro Canal channel improvements have been incorporated in both Alternatives 1 and 2 (also referred to as Models 2 and 3, respectively). There are 12 "improved" or "proposed" cross sections east of the S39 structure, between Nodes N79 and HN887. The proposed cross sections are adopted from Appendix B of Engineering Design, Draft WPA (October 2001) and have also been entered into the global database of the XP-SWMM models. The most significant changes to the modified cross sections are higher banks (less overtopping), wider and deeper channel (see Appendix C). Table 6.1 summarizes the geometric parameters of the modified links used in the Alternative Models (Models 2 and 3). The hydraulic parameters of the modified links are summarized in Table 6.2.

6 Input Data for the Models with Storage Alternatives

Table 6.1 Summary of the Geometric Parameters of the Modified Links used in Alternative Models

Link Name	Main Channel Length (ft)	Upstream Crown Elevation (ft)	Downstream Crown Elevation (ft)	Upstream Invert Elevation (NGVD)	Downstream Invert Elevation (NGVD)	Average Depth (ft)	Upstream Node Name	Downstream Node Name
XS1	6315	20	20	-1	-1	21	PS1	N74
XS11	3729	20	20	-1	-1	21	N74	PS2
XS11.1	3729	19	20	-1	0	20	PS2	N77
XS20	7895.12	20	19	0	-1	20	N77	N78
XS32	2892.3	20	19.5	0	-0.5	20	N78	N78.2
XS-S527B	2892.3	20	16	-1	-5	21	N78.2	N78.1
S527B	2892.3	5.5	5.5	-2.5	-2.5	8	N78.1	N79
PXS5	7833.33	15	15	-9	-9	24	N79	HIS
PXS6	2166.67	15	15	-9	-9	24	HIS	N202
PXS7	1833.33	22.1	22.1	-9	-9	31.1	N202	N203
PXS-E1W	3743.14	22.1	22.1	-9	-9	31.1	N203	N204
PXS10	2090.2	22.1	22.1	-9	-9	31.1	N204	UWPBN
PXS11	2333.33	20.2	20.2	-9	-9	29.2	UWPBN	UWPBS
PXS12	2083.33	18	18	-9	-9	27	UWPBS	N207
PXS13	2000	17	17	-9	-9	26	N207	HNS39_T
PXS14	2000	17	17	-9	-9	26	HNS39_T	N209
PXS15	1916.67	23.4	23.4	-9	-9	32.4	N209	HN887
PXS16	958.45	32	32	-9	-9	41	HN887	HN888
PXS16.1	958.45	25.7	25.7	-9	-9	34.7	HN888	HN889
PXS-E1	1847.84	11.7	15.49	-9	-5.21	20.7	HN889	CS14W

Key :
ft = Feet.
NGVD = National Geodetic Vertical Datum.

Table 6.2 Summary of the Hydraulic Parameters of the Modified Links used in Alternative Models

Name	Contraction Expansion Loss Coefficient	Entrance Loss	Exit Loss	Other Loss	Entrance/Exit Loss Type	Left Overbank Manning's *n*	Main Channel Manning's *n*	Right Overbank Manning's *n*
XS-S527B	0.3	0	0	0	Energy/Loss Coeff	0.033	0.033	0.033
S527B	0	0.5	0.9	0.1	Energy/Loss Coeff	0.03	0.025	0.03
PXS5	0.3	0	0	0	Energy/Loss Coeff	0.03	0.011	0.03
PXS6	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS7	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS-E1W	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS10	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS11	0.3	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS12	0.1	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS13	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS14	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS15	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS16	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS16.1	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03
PXS-E1	0	0	0	0	Energy/Loss Coeff	0.03	0.025	0.03

6.2 Modifications to the Inflow Hydrographs

In the alternative scenario models, there are two modifications to the inflow hydrographs:

- The inflow hydrographs from the NSID have been changed from Model 1 (Base Model) to divert all NSID water to the Hillsboro Canal, but at different rates. In Model 2 (Alternative 1), discharge from the NSID to the L-36 Canal takes place according to two pump station hydrographs obtained from the Final Model of Task 4. The inflow hydrographs at the nodes designated as PS1 and PS2 corresponding to pump stations 1 and 2, respectively, are identical to the corresponding pump station hydrographs generated with Bishop Property as the storage area of NSID excess flow. In Model 3 (Alternative 2), the pump station hydrographs are those according to the modifications given by the SFWMD. In both of Models 2 and 3, there is no flow to the Hillsboro Canal at the “Bishop” node described above.

- There is no flow to the “HIS” node described above since Hillsboro Site 1 Impoundment has been incorporated in the alternative models.

6.3 Modifications to the Hydraulic Structure S39A on L-36 Canal

For the alternative models, the S39A structure is replaced by a new gated culvert designated as S527B (Table 6.3).

Table 6.3 Design Data of S527B Gated Culvert

Revision 5 January - Original Submission	
XY Coordinates:	893330 733120
Location:	NE corner of southern Hillsboro Impoundment. On C-525S at Hillsboro Canal.
Purpose:	Control seepage and allow North Springs water to reach Hillsboro Canal.
Design Conditions	
Discharge:	600 cubic feet per second
Headwater Elevation:	7.75 feet National Geodetic Vertical Datum (NGVD)
Tailwater Elevation:	7.00 feet NGVD

Table 6.3 Design Data of S527B Gated Culvert

Maximum Expected Stages	
Headwater Elevation:	10.00 feet NGVD
Tailwater Elevation:	9.50 feet NGVD
Maximum Head Difference	
Maximum Headwater Elevation:	9.00 feet NGVD
Minimum Tailwater Elevation:	7.00 feet NGVD
Culvert Data	
Number of Barrels:	2
Single Barrel Width:	8.0 feet
Single Barrel Height:	8.0 feet
Barrel Length:	135.0 feet
Barrel Invert Elevation:	-2.50 feet NGVD
Type of Control	
Canal Data	
Side Slopes:	Cotangent 1
Upstream Bottom Width:	25.00 feet
Upstream Bottom Elevation:	-5.00 feet NGVD
Downstream Bottom Width:	Pool feet
Downstream Bottom Elevation:	-9.00 feet NGVD
Riprap Requirements	
Design Barrel Velocity:	4.69 feet per second
Riprap Design Velocity:	6.00 feet per second
Riprap Protected Area:	900 square feet
Riprap Thickness:	1.5 feet
Riprap Bedding Thickness:	1.0 feet
Control Protection Elevation:	11.00 feet NGVD
Gated Box Culverts	
Slide Gate	

Notes: Riprap requirements have not been verified with Geotech.

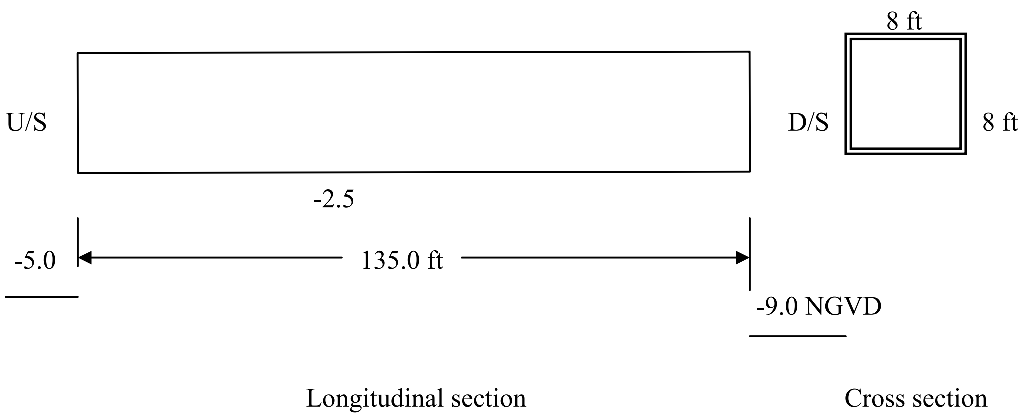

Figure 6.1 Schematic of the S527B Gated Culvert

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7 Data for Model Calibration

For the purpose of the calibration of one of the models, selection of a real storm event for which short-term time series data on either stage or flow are available was necessary. The relatively recent storm event, Hurricane Irene, was selected for this purpose.

7.1 Synoptic History of Hurricane Irene Event

A broad area of low pressure prevailed over the southwestern Caribbean from October 8 to 10, 1999, accompanied by disorganized clouds and thunderstorms. This system did not show signs of tropical cyclone development until a tropical wave reached the western Caribbean Sea on October 11. Tropical Depression Thirteen formed in the northwestern Caribbean Sea at approximately 0600 Universal Time Coordinated (UTC), October 13. It reached tropical storm status by 1200 UTC on October 13. At that time, Irene was a strengthening tropical storm. Irene moved on a general northward track and slowed down considerably before curving to the north-northeast just to the southwest of the Isle of Youth, Cuba, where it made its first landfall at 1200 UTC on October 14. The center of the tropical cyclone then crossed the Havana and Ciudad Havana provinces between 2100 and 2300 UTC on the 14th. Irene reached hurricane status over the Florida Straits and the calm of the center moved over Key West near 1300 UTC on October 15. Most of the hurricane force winds were confined to the east of Irene's center over the lower to middle Florida Keys. Irene made its fourth landfall near Cape Sable, Florida, and then moved across southeast Florida bringing tropical storm conditions (sustained 39- to 73-mile-per-hour winds) and torrential rains (10 to 20 inches).

Irene moved back over water in northern Palm Beach County near Jupiter, Florida, shortly after 0000 UTC on October 16. It retained hurricane strength and moved on a general northward track paralleling the Florida east coast heading for the Carolinas.

No statistical frequency analysis has been conducted to evaluate the return period of the rainfall associated with the Irene event. However, the magnitude of Hurricane Irene can be judged by the fact that on October 15, 1999, it dropped over 9 inches of rainfall, on average, across Palm Beach, Broward, and Miami-Dade Counties, and that the three-day (October 14 to 16) rainfall totals at specific measuring sites throughout Broward and Palm Beach Counties ranged between 10.88 and 17.47 inches. These rainfall amounts generally correspond to a 25-year event (Appendix D).

7.2 Stream Gauge Records

Records of stage heights during the Irene storm event at various gauging stations along the Hillsboro Canal within the present study limits have been obtained from the SFWMD (including DBHYDRO Browser). The parts of the data relevant to the present study are summarized in Table 7.1.

Table 7.1 Record of Stage Heights during Hurricane Irene at Three Gauging Stations within the Study Limits of the Hillsboro Canal

Station Name	DBKEY No.	Period of record	Coordinates		Stage, feet NGVD 29		
			X	Y	Max	Date	Time
G56_H	05728	10/10/1999 – 10/20/1999	940840.3	725811.67	7.67	Oct 16	00:00
G56_T	05730	10/10/1999 – 10/20/1999	940840.30	725811.67	7.49	Oct 16	00:00
S39_T	06663	10/10/1999 – 10/20/1999	886178.12	735692.00	11.77	Oct 15	23:00

Key :
_H = Headwater.
_T = Tailwater.
NGVD 29 = National Geodetic Vertical Datum of 1929.

Figures 7.1, 7.2, and 7.3 show the time history of the stage heights at the three stations noted in Table 7.1. In addition to these data, the maximum stage height and stream flow recorded at station 'HILLS' (Tables 4.1 and 4.2) are also used for the verification of the Base Model run using 25-year/72-hour storm hydrographs.

Insert Figure 7.1

Insert Figure 7.2

Insert Figure 7.3

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8 Results of Base Model Calibration

All models were first run for a period of 24 hours (October 12 midnight to October 13 midnight) without any flow contributions from the storm event to set up the initial conditions. Subsequently, model computations were carried out for a period of five days (October 13 midnight to October 18 midnight)

The results of the 25-year/72-hour Base Model run were compared with the field data available for the Hurricane Irene event (see Table 8.1).

Table 8.1 Comparisons of Observed and Calculated Stage Heights at Nodes Where Observational Data are Available

Station Name	Maximum Stage Height (feet)/ Date/Time (Observed)	Maximum Stage Height (Calculated)	Percentage Difference
S39_T	11.77/10-15-1999/23:00:00	11.917	+ 1.01
G56_H	7.67/10-16-1999/00:00:00	8.647	+ 1.12
G56_T	7.49/10-16-1999/00:00:00	7.659	+ 1.02
HILLS	11.40/10-16-1999/-- -- --	12.133	+ 1.06

Note: The station 'HILLS' roughly corresponds to node HN887 of the present models.
Key :
_H = Headwater.
_T = Tailwater.

Some observations on flow measurements are also compared with the calculated flow values (Table 8.2).

Table 8.2 Comparisons of Observed and Calculated Flow at Conduits Where Observational Data are Available

Station Name	Maximum Flow (cfs)/ Date/Time (Observed)	Maximum Flow (cfs) (Calculated)	Remarks
S39	485.53/10-13-1999 0/10-13-1999 to 10-18-1999		This is the control structure to allow flow from WCA 2A to Hillsboro Canal. During the storm event it is closed.
S39A_T	No observation	524	
HILLS	1,300	1,536	

Note: The station 'HILLS' roughly corresponds to node HN887 of the present models.
Key :
_T = Tailwater.
cfs = cubic feet per second.
WCA = Water Conservation Area.

Note that the comparisons of the observed and calculated stage heights (Table 8.1) show an error slightly above 1%. Thus, the calibration is considered good. However, in terms of absolute error of the model, the error is given as follows.

$$\text{Error} = \text{Inflow} + \text{Initial Volume} - \text{Outflow} - \text{Final Volume}$$

The error, noted as 'error in continuity' is –0.25% for the Base Model indicating insignificant error (if the continuity error is $\leq$ 5%, an XP-SWMM model is considered good). In spite of the overall good continuity error, there was overtopping at two nodes namely "BISHOP" and "HNS39_T." The total volume of water lost at these two nodes due to overtopping is 74.44×10^6 cubic feet (ft^3) compared to the total volume of inflow plus initial volume of 2.25×10^9 ft^3. Thus, only 3.3% of water was lost due to overtopping. This minor loss can be disregarded for two reasons:

1) The loss can be considered part of the calibration since the inflow hydrographs were not calibrated; and

2) During actual flooding, the overtopped or spilled water may not recede either fully to the canal or during the duration of the storm event (i.e., even if the entire spilled volume returns to the system that may not happen during the simulation period and hence the model simulates the actual situation).

In consideration of the facts presented above, the Base Model is a good approximation of the present hydraulics of Hillsboro Canal during extreme events.

9 Comparisons and Results of Initial Alternative 1 and Alternative 2 Scenarios

The scenarios for Alternatives 1 and 2 were initially run for the 25-year/72-hour storm event and were compared to the calibrated model. The three models (Base Model-25 year, Alternative 1-25 year, and Alternative 2-25 year) are compared by observing stage-time graphs at key nodes and flow hydrographs at key conduits (see Table 9.1 and discussion below).

Table 9.1 Comparisons of Water Surface Elevations in the Three Models for the 25-year Event

Nodes/ Conduits	Base Model		Alternative 1		Alternative 2	
	Stage (ft)	Flow (cfs)	Stage (ft)	Flow (cfs)	Stage (ft)	Flow (cfs)
PS1	11.9		14.98		14.9	
NXS1/PXS5		165		590		570
N79	11.9		14.64		14.6	
HN887	12.12		14.48		14.45	
NXS16		1290		1640		1630
G56_H	8.647		11.238		11.208	
G56_T	7.659		8.71		8.7	
N110	1.37		1.68		1.68	

Key :
H = Headwater
T = Tailwater

It should be noted that in the stage-time graphs provided in this section, the drop in water surface elevation from the initial conditions during the beginning hours of the simulation period is due to lack of inflow from the input hydrographs during this period. Furthermore, as previously discussed, without code modifications XP-SWMM does not have the ability, even with the Real Time Control (RTC) Module, to easily model the G56 structure as it is actually operated. As explained in Appendix A, even if the gate movement can be simulated with the RTC module, XP-SWMM does not possess the capability of using various flow equations under varied flow conditions that can prevail over a gated spillway. For the models simulated in the present investigation, the gate is assumed to fully open at the beginning of the rainfall event. This causes a significant drop in water level until the runoff from the storm event begins to reach the Hillsboro Canal. In reality, the gate gradually opens when water rises to 8.0 feet NGVD and continues to open as long as the water level stayed above 7.0 feet NGVD. Instead of a significant drop in water level and low flows as portrayed in the model, the water level remains between 7.0 to 8.0 feet at the beginning of the storm and a larger volume of water is discharged out the G56 structure.

9.1 Observations at Key Nodes and Conduits

Node N79

Node N79 is just downstream of the S39_T structure (Base Model) or the S527B structure (Alternative Models). In both alternatives, the stage heights increase by 2.7 feet at this node relative to that at Base Model (Figures 9.1, 9.2, and 9.3).

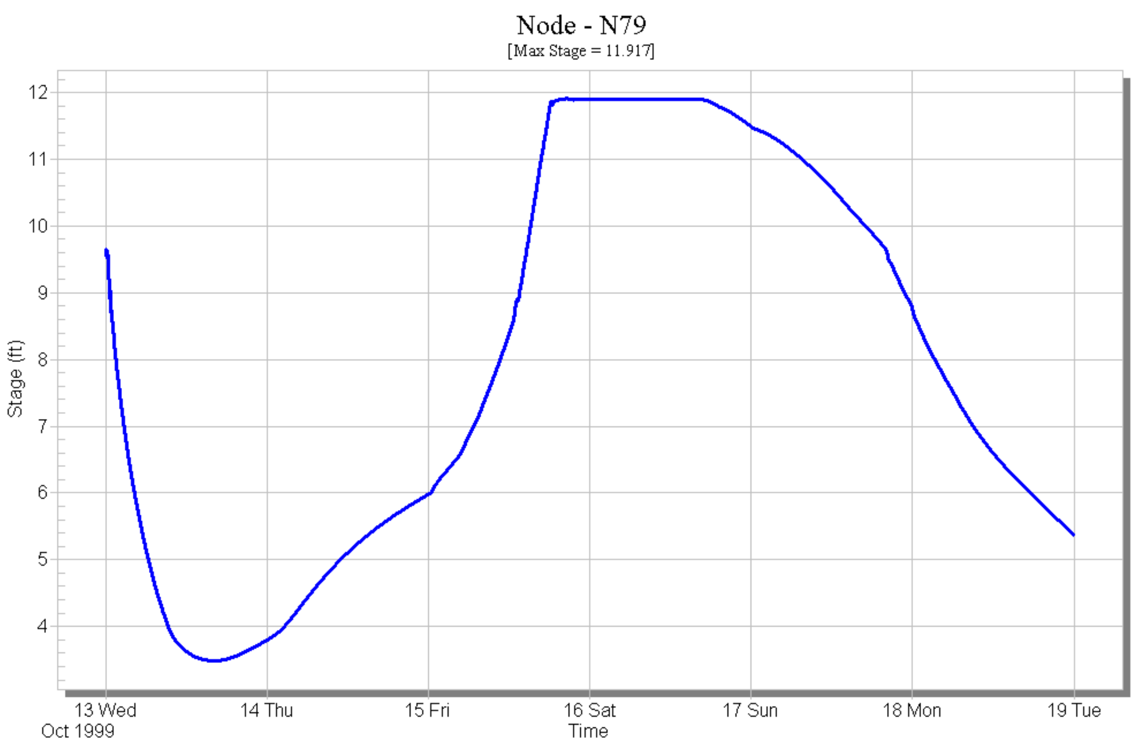

Figure 9.1 Stage-time Graph at Node N79 under Base Model

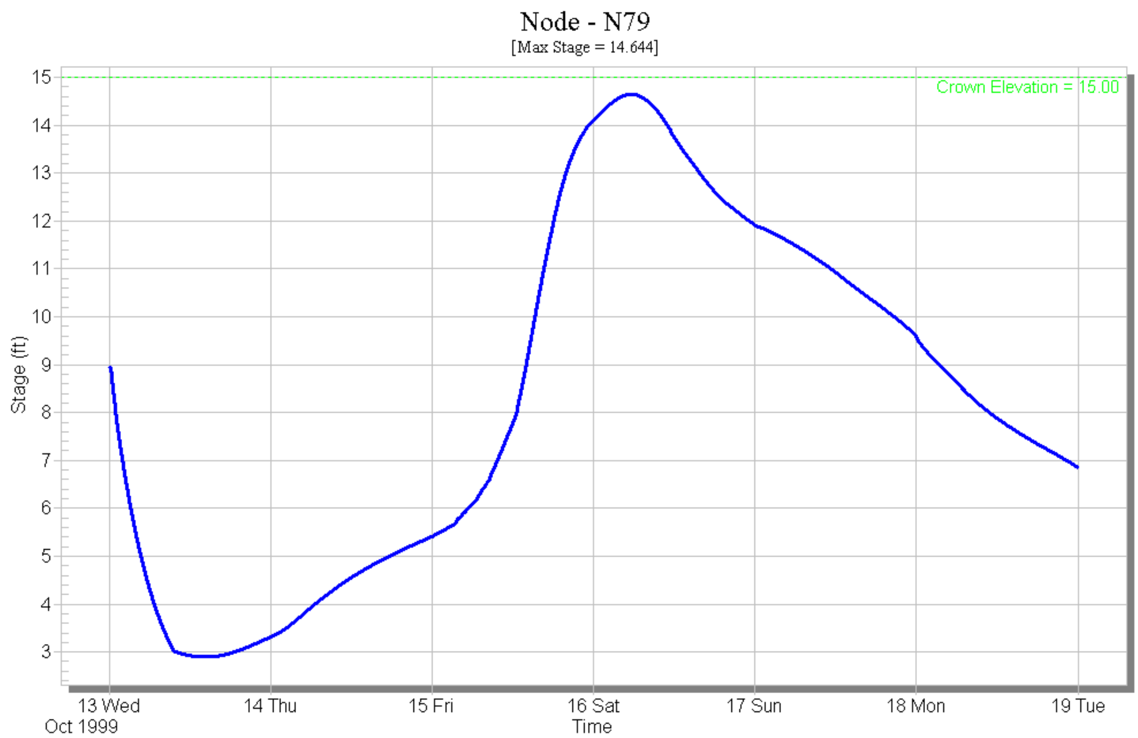

Figure 9.2 Stage-time Graph at Node N79 under Alternative 1

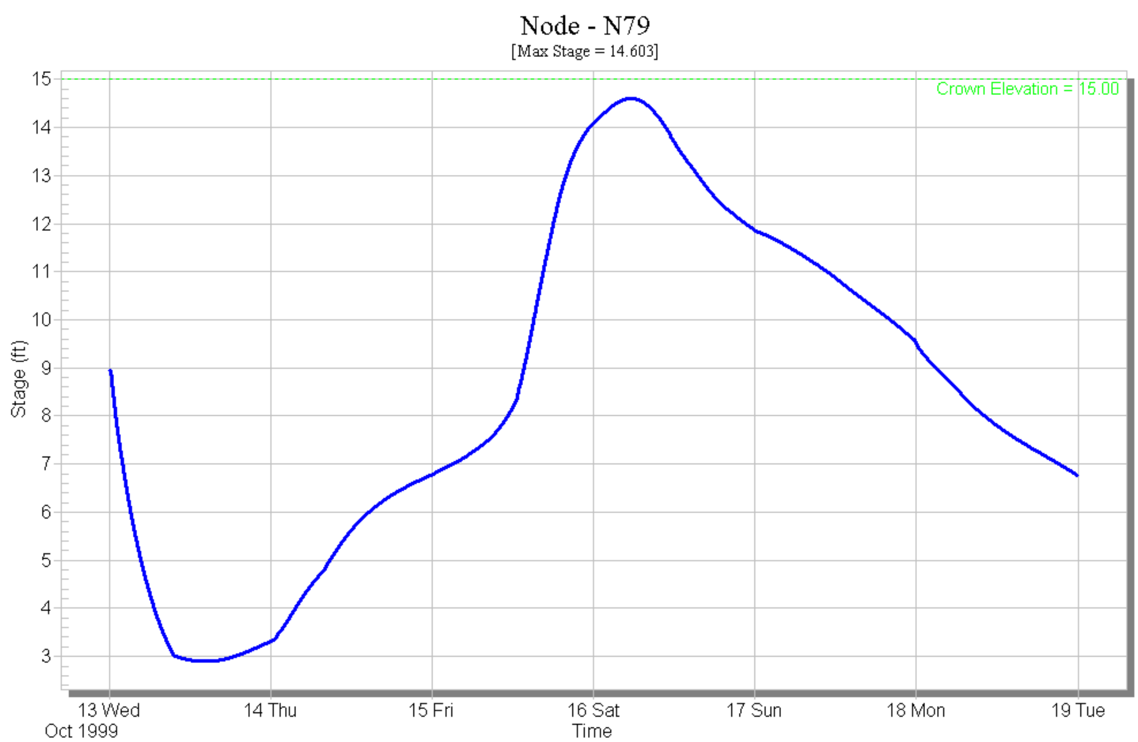

Figure 9.3 Stage-time Graph at Node N79 under Alternative 2

Node G56_H

Node G56_H is just at the upstream (headwater) of the G56 (Deerfield) structure. In both alternatives, the stage heights increase by 2.6 feet at this node relative to that at Base Model (Figures 9.4, 9.5, and 9.6).

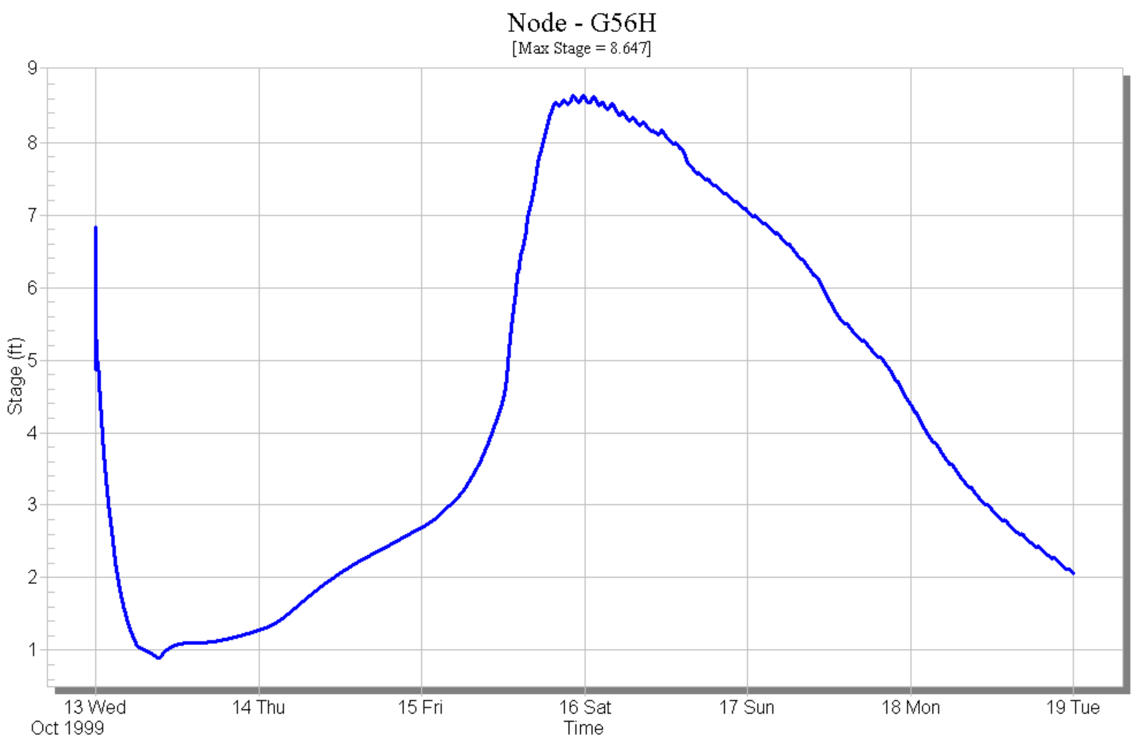

Figure 9.4 Stage-time Graph at Node G56_H under Base Model

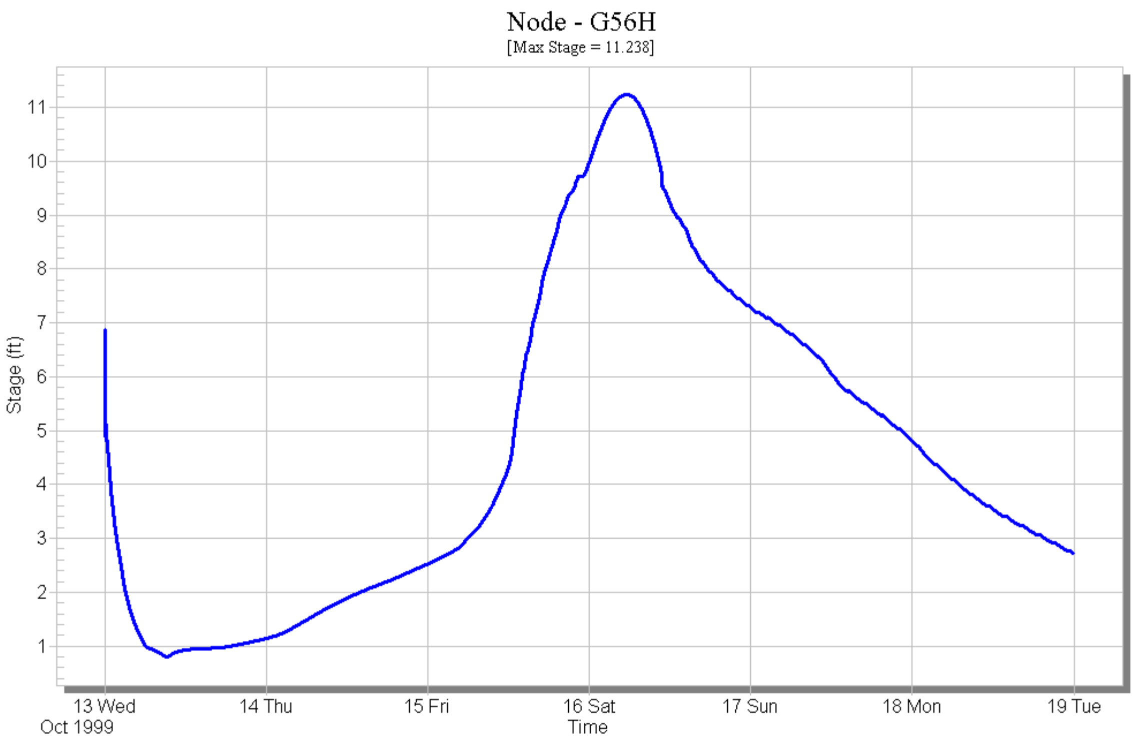

Figure 9.5 Stage-time Graph at Node G56_H under Alternative 1

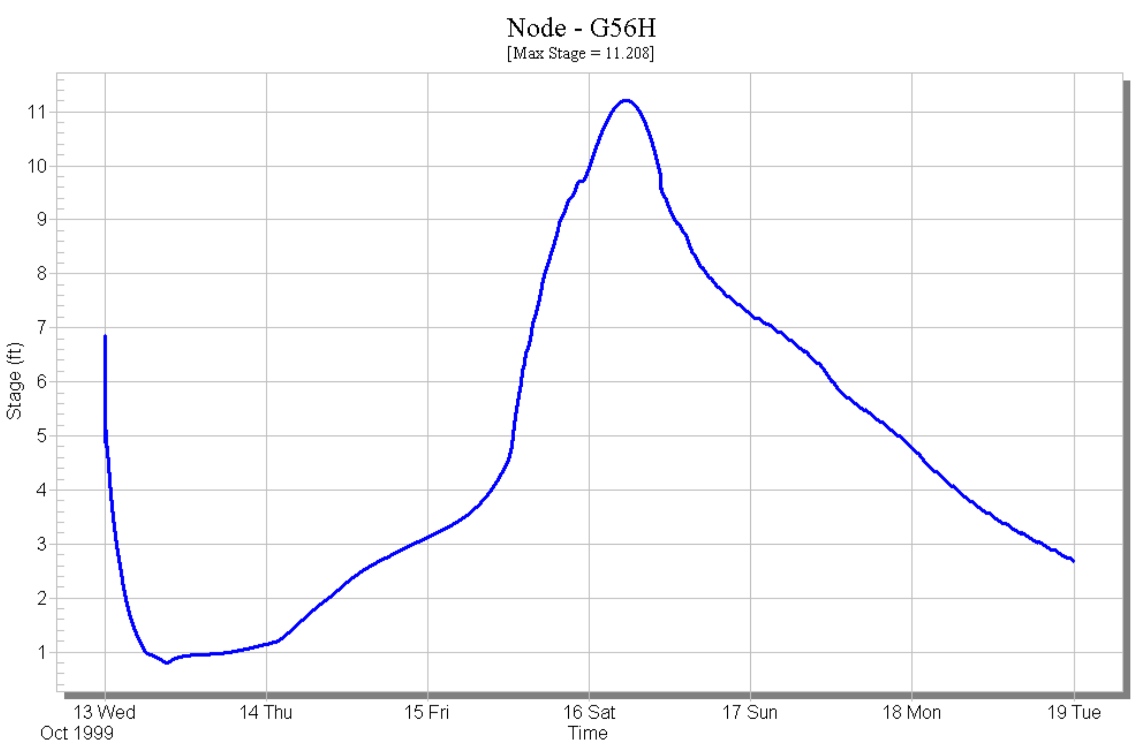

Figure 9.6 Stage-time Fraph at Node G56_H under Alternative 2

Node G56_T

Node G56_T is the node just at the downstream (tailwater) of the G56 (Deerfield) structure. In both alternatives, the stage heights increase by 1.05 feet at this node relative to that at the Base Model (Figures 9.7, 9.8, and 9.9).

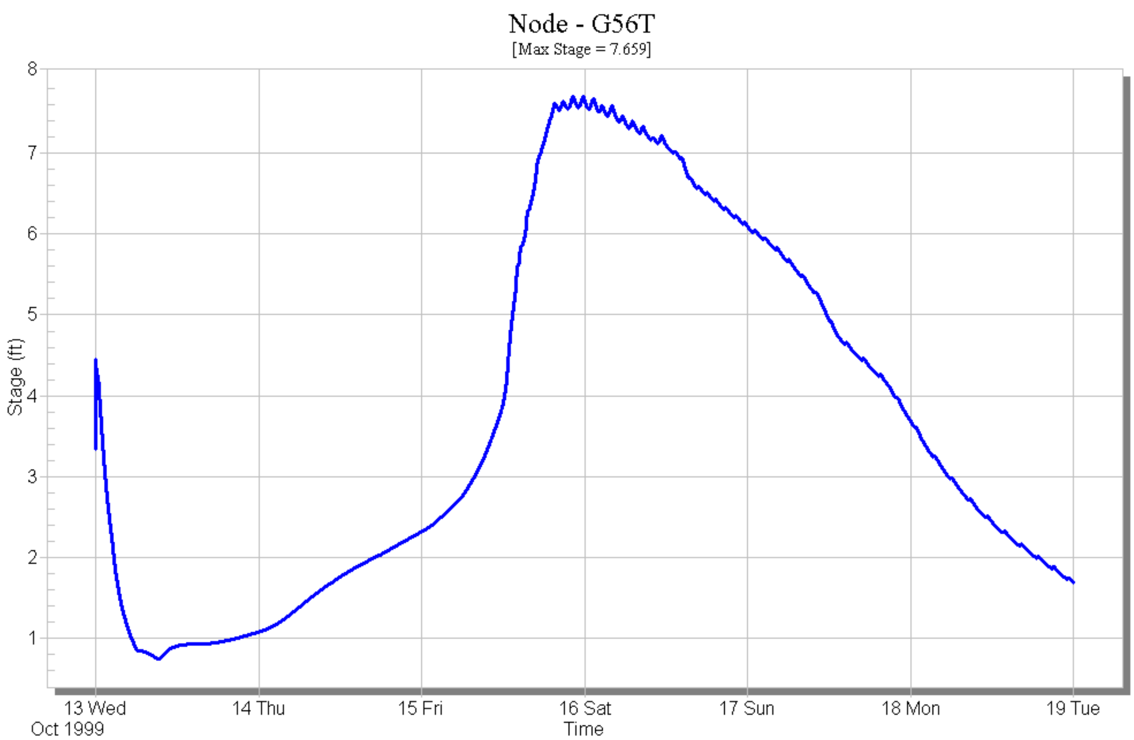

Figure 9.7 Stage-time Graph at Node G56_T under Base Model

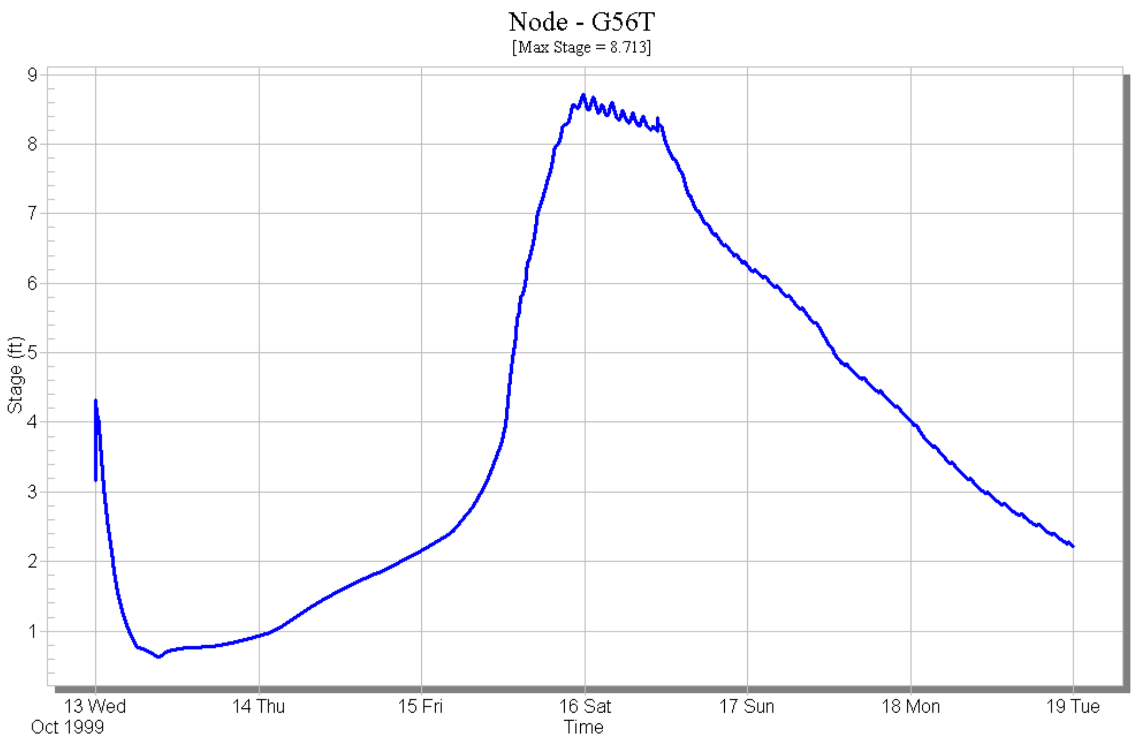

Figure 9.8 Stage-time Graph at Node G56_T under Alternative 1

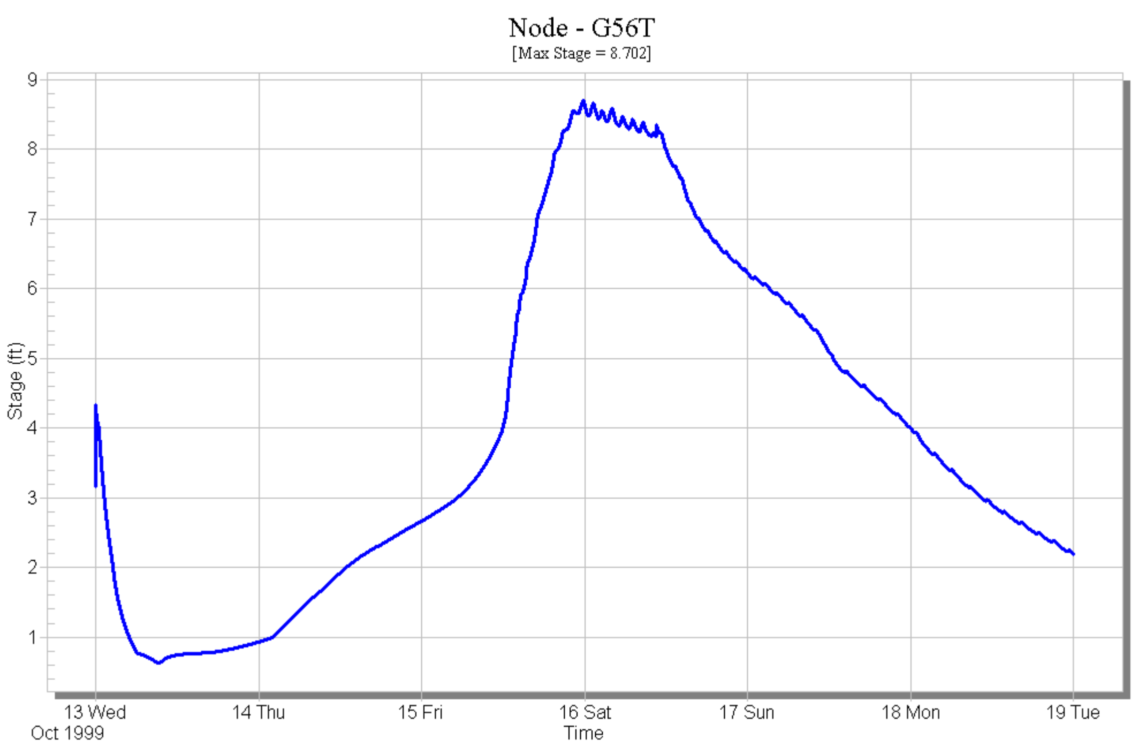

Figure 9.9 Stage-time Graph at Node G56_T under Alternative 2

Conduit NXS1 or PXS5

Conduit NXS1 or PXS5 is the channel segment just downstream of the S39A or S527B structure. Flow in this conduit is a measure of contributions from the NSID pumping. The flow hydrographs in this conduit under base and alternative conditions are shown in Figures 9.10, 9.11, and 9.12. Note that there are pronounced differences in the pattern of flow from the Base Model to the Alternative Models. Under the Base Model, after considerable reverse flow at the end of 48 hours, there is a quasi-stationary condition for more than 24 hours and then the forward flow reaches a maximum of only 200 cfs. Under alternative conditions, however, the forward flow reaches a maximum of 600 cfs (in the case of Alternative 2 it is slightly less due to less NSID pumping), and then on the average flows at a rate of 500 cfs for the remainder of the simulation period. This increase in flow through the conduit when translated further downstream causes an increase in flow through the G56 structure. The increase in flow is caused by channel improvements (i.e., widening and deepening of the channel causes an increase in hydraulic radius which, in turn, increases the total volumetric discharge).

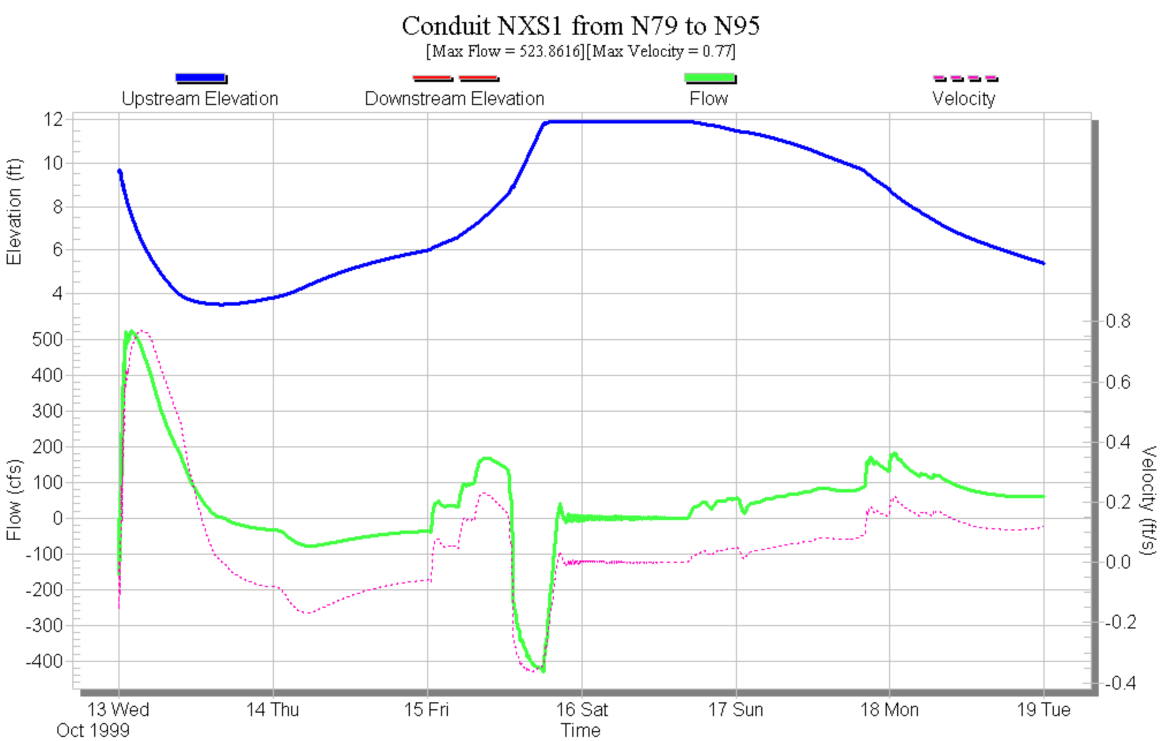

Figure 9.10 Hydrograph for Conduit NXS1 under Base Model

9 Comparisons and Results of Initial Alternative 1 and Alternative 2 Scenarios

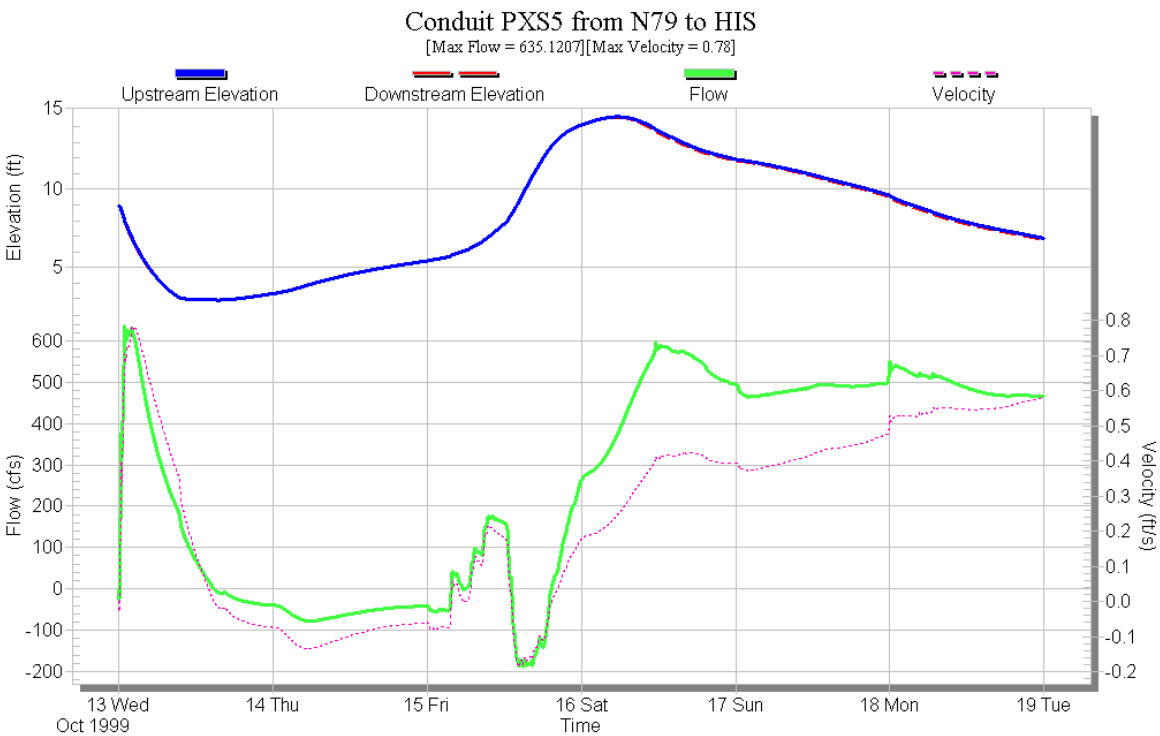

Figure 9.11 Hydrograph for Conduit PXS5 under Alternative 1

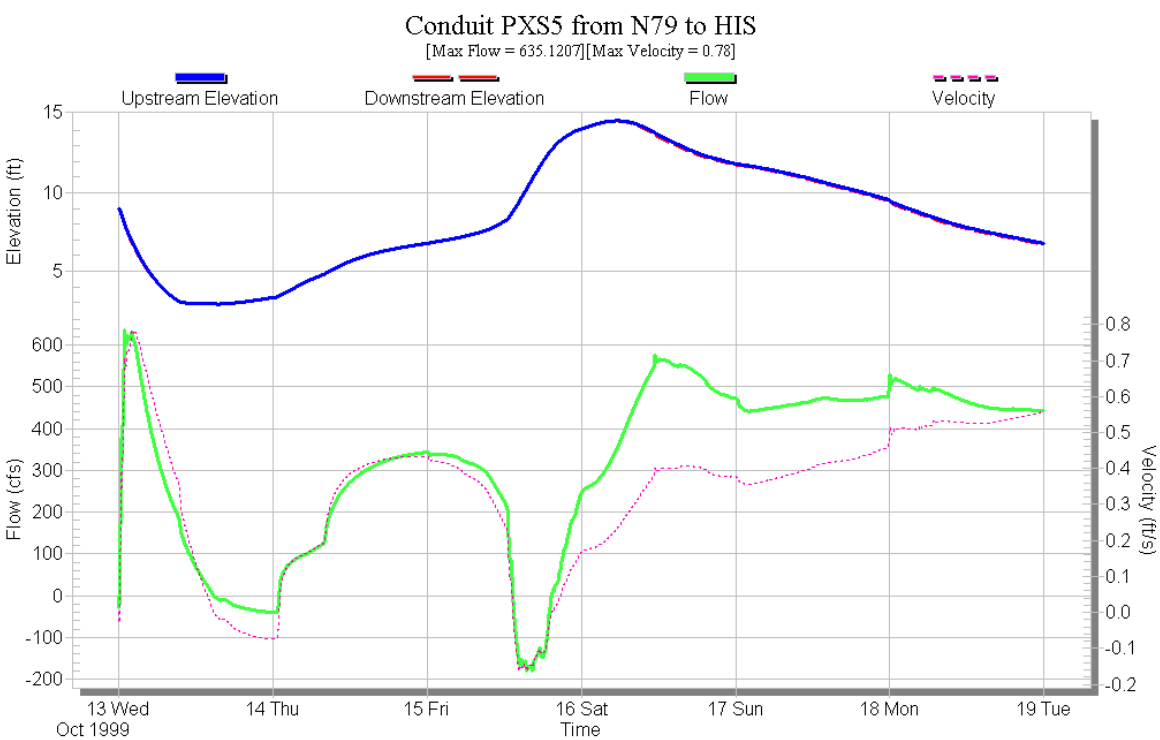

Figure 9.12 Hydrograph for Conduit PXS5 under Alternative 2

Flow Through the G56 Structure

The flows over the G56 spillway under the three model scenarios are shown in Figures 9.13, 9.14, and 9.15. Note that the initial high values of the discharge (deluge) are due to instantaneous opening of the gate from 2.9 feet (initial condition) to 9.8 feet at the beginning of the simulation.

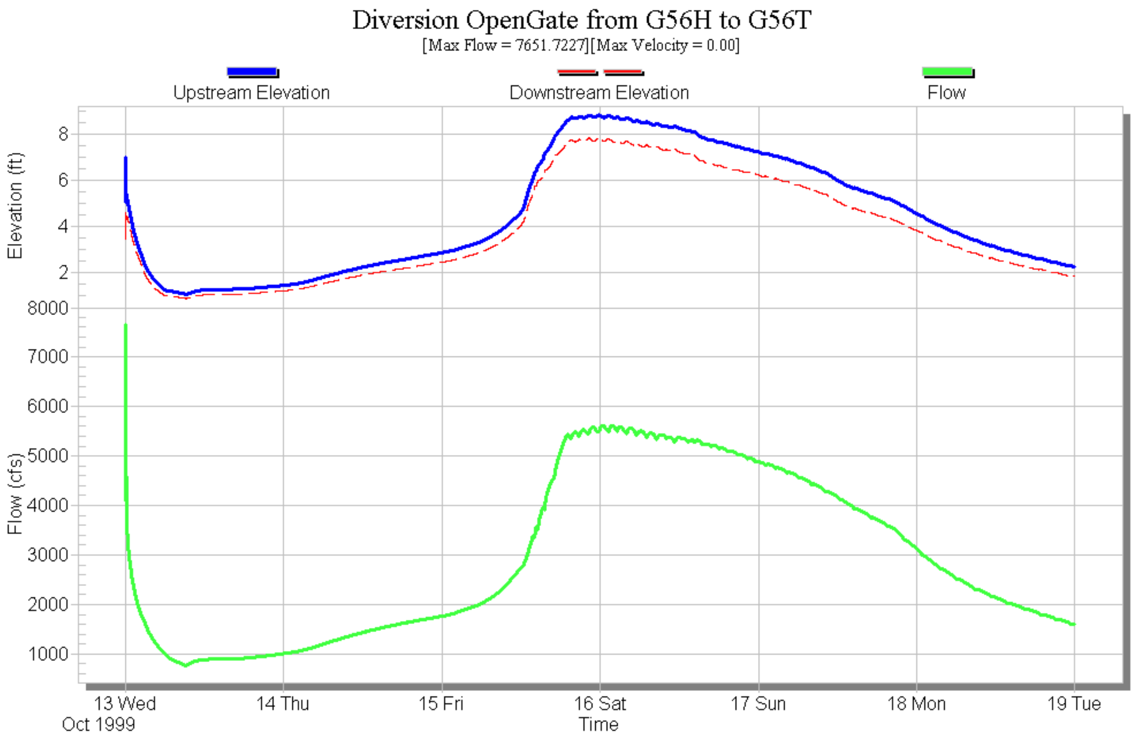

Figure 9.13 Flow Over G56 Spillway under Base Model

For the alternatives, due to a change from uncontrolled flow to controlled flow the flow remains steady for nearly 24 hours and then when the water level drops below the gate bottom there is an instantaneous peak discharge at a rate much higher than the rate at which the steady flow has been occurring. However, as noted earlier, this is due to the way XP-SWMM calculates orifice flow over a gated spillway and the limitations of the XP-SWMM program in modeling various types of flows over a gated spillway. For this reason, in subsequent sections, the average flow during the steady-state flow condition is taken as the peak flow through this conduit under alternative conditions. This is considered to be an average since, in reality, the flow peaks and drops due to the differential head between the headwater and tailwater elevations as shown in Figures 9.14 and 9.15.

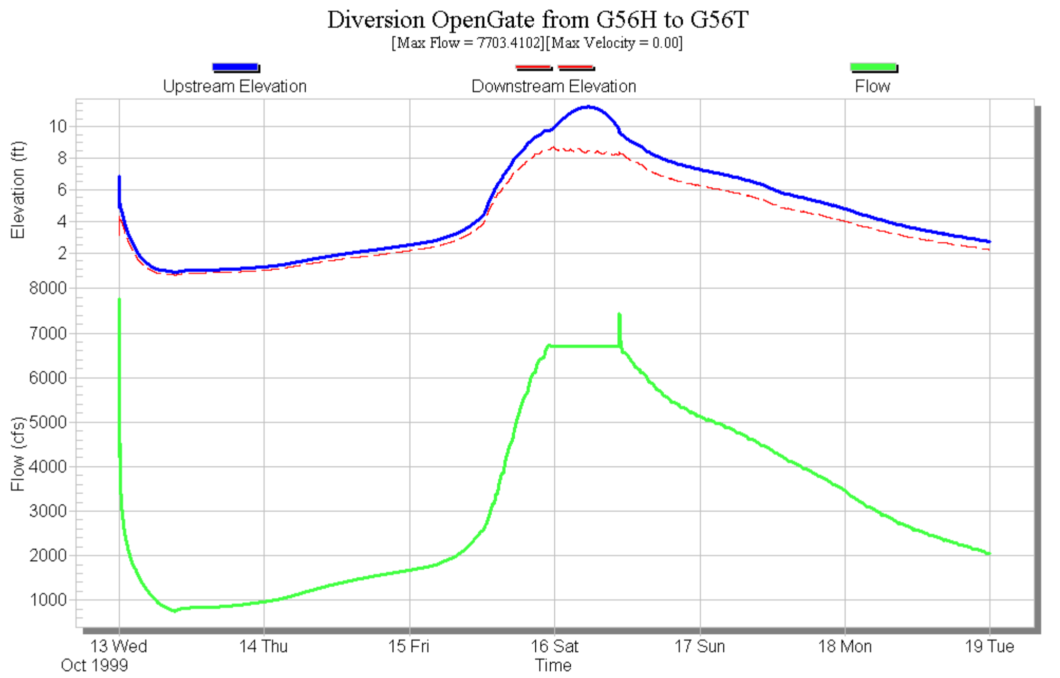

Figure 9.14 Flow Over G56 Spillway under Alternative 1

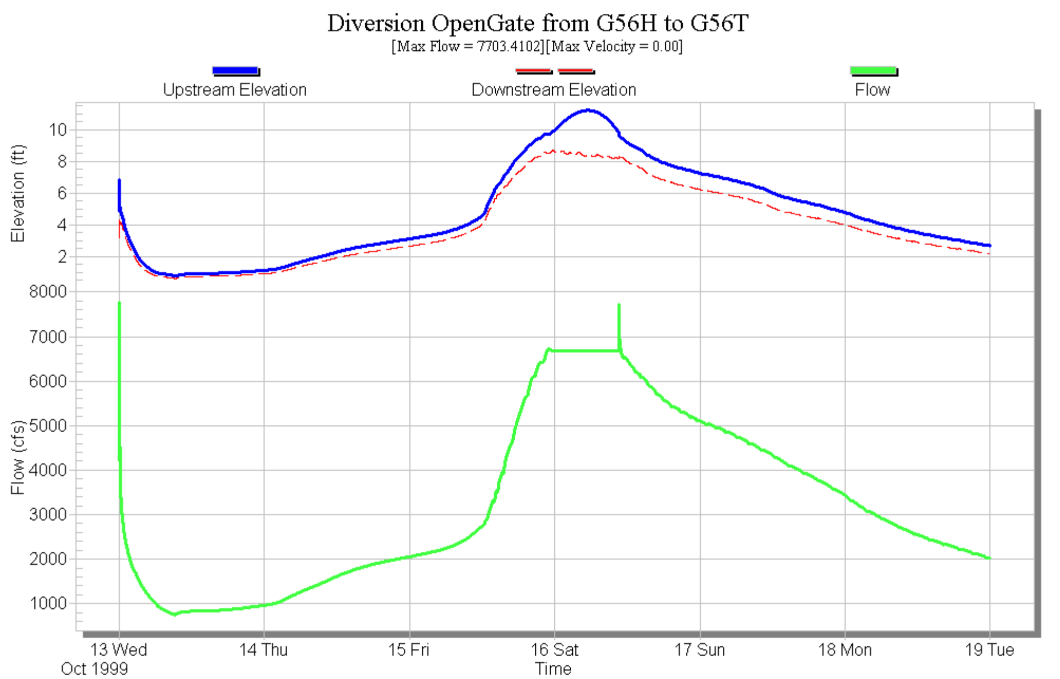

Figure 9.15 Flow Over G56 Spillway under Alternative 2

9.2 Model Comparisons

As shown in the figures and discussed above, with both alternatives, there is a significant rise in water level over the Base Model. However, the rise is slightly less with Alternative 2 than that in the case of Alternative 1. This rise appears to be a result of the increased NSID discharge coupled with higher bank elevations and channel deepening associated with the proposed channel improvements and flow control at the G56 structure.

In the base case, 74.44×10^6 ft^3 of water overtopped the north banks at two nodes (HNS39_T and Bishop). This spilled-out volume represents only 3.3% of the total volume of inflow and initial volume present in the canal. However, this overtopping keeps the water level in the channel from rising much higher. In Alternatives 1 and 2, the tops of the banks are higher and less water is lost. In these two scenarios, 1.82×10^6 ft^3 and 1.83×10^6 ft^3 of water is lost, respectively, through the north bank of the single node (HNS39_T). In both cases, the spilled-out volume represents 0.08% of total volume of inflow and initial volume present in the canal. While the difference in overtopping between the Base Case and the Alternatives is not great (approximately 3.2%), this volume does have an influence on stage heights. Rough calculations indicate that the channel rise is sensitive to flow increases if the outflow does not keep up with the inflow. For example, an inflow in excess of the outflow of approximately 100 cfs for two days will raise the Hillsboro Canal 3 feet.

If the headwater at the G56 structure rises to 9.8 feet NGVD, water will hit the bottom of the gate and a constriction/control will result. Flow characteristic changes from either uncontrolled-free or uncontrolled-submerged flow to orifice-type flow. Under this situation, XP-SWMM calculates orifice flow as a steady flow (internally computes C_d to adjust for the difference between headwater and tailwater elevations). As the upstream head rises even further, the flow remains steady, causing more backwater effects. In reality, there are some changes or unsteadiness in flow as the headwater and tailwater elevation differential fluctuates during the orifice-type flow condition, thus the flow is not steady as modeled by XP-SWMM (Figures 9.14 and 9.15).

To further assess the influence of the NSID discharge, channel improvements and the G56 structure, additional model runs were conducted and include the following:

- Alternatives 1 and 2 were also modeled using existing cross sections to assess the effect of NSID pumping only. Comparisons were made between base conditions and these model runs. Also comparisons were made between these model runs and Alternatives 1 and 2 with the new cross sections. In this way, the effects of channel improvements can also be assessed at the same time.

- The gate at G56 is kept partially open at 8.0 feet in the Base Model to allow water to hit the bottom of the gate and compare the model to the original Base Model to assess the effect of flow control at the G56 structure.

- The gate at G56 in Alternative 1 is artificially open to 12 feet so that water does not hit the bottom of the gate and this model run is compared to Alternative 1 with the gate at 9.8 feet to assess the effect of no control of flow at the G56 structure on channel improvements.

- Alternatives 1 and 2 were modeled for 10-year/24-hour and 100-year/72-hour storm events to assess the effects of rainfall volume on the magnitude of rise and fall of stages at various locations along the Hillsboro Canal.

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10 Results of Additional Model Runs

10.1 Effect of NSID Pump Stations on the Hydraulics of Hillsboro Canal

Three models—Base Condition (25-Year), Alternative 1 (25-year/Base Cross Section), and Alternative 2 (25-year/Base Cross section)—offer the opportunity to evaluate what effects the excess discharge from the NSID pump stations may have on the hydraulics of Hillsboro Canal since the cross-sectional geometry of the canal remains identical (existing cross sections) in all of these models. With Alternatives 1 and 2, however, there is no inflow into the HIS node, but the pump station hydrographs from PS1 and PS2 discharge more water into the L-36 canal when compared to those under the Base Condition. The results show that for increased NSID pumping, the stage elevation at N79 rises negligibly (+0.08 for Alternative 1 and +0.07 for Alternative 2) and peak flow increases by 350 cfs in Alternative 1 and 329 cfs in Alternative 2 through the conduit PXS5 (Table 10.1). Through the G56 structure, there is no flow increase for Alternative 1 and it is only 5 cfs for Alternative 2. Similarly, there is virtually no increase in stage heights at the upstream and downstream nodes of the G56 structure (G56_H, G56_T) in Alternatives 1 and 2. There is a slight increase in the amount of water overtopping the banks in the Alternatives 1 and 2 Base Cross Section model runs as compared to the Base Condition. Thus, the effect of increased NSID pumping on the hydraulics of the Hillsboro Canal as proposed in the alternative models is negligible. The flow is consistent with the proportion at which there is an increase in NSID pumping in Alternative models 1 and 2.

10.2 Effect of Channel Improvement on the Hydraulics of Hillsboro Canal

Four model runs—Alternative 1 (25-year), Alternative 2 (25-year), Alternative 1 (25-year/Base Cross Section), and Alternative 2 (25-year/Base Cross section)—offer the opportunity to evaluate what effects the channel improvements may have on the hydraulics of the Hillsboro Canal since the three models presented in Section 10.1 demonstrate that

Table 10.1 Effect of NSID Pump Station: Comparisons of Alternatives 1 and 2 to Base Case using Same/Existing Cross-Sectional Geometry

	Maximum Stage Heights (feet with respect to the National Geodetic Vertical Datum)						Flow (cubic feet per second) through Critical Conduits					Percent Water Loss via Overtopping
Model Scenario	**PS1**	**N79**	**HN887**	**G56_H**	**G56_T**	**N110**	**NXS1/ PXS5**	**NXS16 /PXS16**	**G56US**	**G56**	**G56DS**	
Base (25-year)	11.9	11.9	12.12	8.647	7.659	1.37	165	1290	5550	5550	5550	3.3
Alternative 1 (25-year Base Cross Section)	12.54	11.98	12.12	8.637	7.65	1.37	515	1418	5550	5550	5550	3.7
Alternative 2 (25-year Base Cross Section)	12.47	11.97	12.12	8.637	7.65	1.37	494	1388	5555	5555	5555	3.59

Key :
_H = Headwater.
_T = Tailwater.

Table 10.2 Effect of Changes in Channel Geometry: Comparison of Alternatives 1 and 2 with New and Existing Cross Sections

	Maximum Stage Heights (ft with respect to the National Geodetic Vertical Datum)						Flow (cubic feet per second) through Critical Conduits					Percent Water Loss via Overtopping
Model Scenario	**PS1**	**N79**	**HN887**	**G56_H**	**G56_T**	**N110**	**NXS1/ PXS5**	**NXS16 /PXS16**	**G56US**	**G56**	**G56DS**	
Alternative 1 (25-year)	14.98	14.64	14.48	11.238	8.71	1.68	590	1640	6642	6642	6642	0.079
Alternative 2 (25-year)	14.9	14.6	14.45	11.208	8.7	1.68	570	1630	6627	6627	6627	0.079
Alternative 1 (25 year/Base Cross Section)	12.54	11.98	12.12	8.637	7.65	1.37	515	1418	5550	5550	5550	3.7
Alternative 2 (25-year/ Base Cross Section)	12.47	11.97	12.12	8.637	7.65	1.37	494	1388	5555	5555	5555	3.59

Key :
_H = Headwater.
_T = Tailwater.

there is negligible effect of excess NSID pumping on the hydraulics of the Hillsboro Canal if the cross-sectional geometry remains same.

The results show that the changed (proposed) cross sections (including S527B structure) have dramatic effects on both the stage heights and peak discharge (Table 10.2). With Alternative 1, the stage rises by +2.74 feet compared to the Base Model at Node N79, +2.59 feet at node G56_H, and +1.05 feet at node G56_T. For Alternative 2, these rises are slightly less: +2.70, +2.56, and +1.04 feet at N79, G56_H, and G56_T, respectively. The increase in peak flow through PXS5 is 425 cfs for Alternative 1 and 405 cfs for Alternative 2. While these increases are proportional to the flow increase through the NSID pump stations, the increase in flow through the G56 structure is dramatic: 1,092 cfs for Alternative 1 and 1,077 cfs for Alternative 2.

The channel improvements involve deepening and widening of the channel for a length of 6 miles starting from the tail water end of the S39A structure, converting the three-barrel, circular culvert of the S39A structure to two, barrel, box culverts, and in places, raising the bank elevations. All of these improvements substantially increase the storage capacity of this section of the Hillsboro Canal. There is significantly less overtopping with the improved cross sections and more volume that discharges out the G56 structure and increases the stage. Furthermore, the channel, under the improved condition has an adverse slope where the Hillsboro Canal bends straight eastward. The adverse slope causes the water surface to rise quickly in the downstream direction and the increased storage in the upstream causes increased momentum that further raises the water downstream at a much faster rate. These combined effects cause water to hit the bottom of the gate at the G56 structure and consequently water backs up from G56_H all the way to N79. For these reasons, the improved cross sections have a significant effect on the hydraulics of the Hillsboro Canal.

It should be noted that L-36 channel banks were modeled with an elevation of 20 feet NGVD based on information on the 1991 Greenhorne & O'Mara Report. However, at least one location has been recently reported to be at an elevation of 14.5 feet. If this is indeed correct, the stages at PS1 and several stations downstream would be a little lower than reported since some overtopping will occur.

10.3 Effect of G56 Gate on the Hydraulics of Hillsboro Canal

Four runs—Base (25-year), Base (8-foot Gate/25-year), Alternative 1 (25-year), and Alternative 1 (12-foot Gate/25-year)—demonstrate the relative effects of channel improvement and flow control by the gate at the G56 structure on the hydraulics of the Hillsboro Canal. In the second model, everything is the same as in the first model except the gate at the G56 structure is opened up to 8 feet (instead of 9.8 feet) and stays at that level through the entire storm event. In the fourth model, everything is the same as in the third model, but the gate is artificially opened to 12 feet. While these scenarios will not occur in the field, they do provide additional insight.

Comparison of the first and second models show that due to lowering of the gate, flow through the G56 structure becomes orifice flow when the peak is reached but the peak flow rate remains the same (i.e., the flows through G56 for the first and second model are nearly same; see Table 10.3). In the case of the second model, however, the stage at N79 rises by +0.02 feet, the stage at G56_H rises by +1.58 feet, and the stage at G56_T falls by -0.35 feet. These runs clearly show that the effect of flow control at the G56 structure is simply to enhance the backwater effect upstream of G56.

Comparison of the third and fourth runs show that flow though G56 remains as uncontrolled submerged. Peak flow through G56 slightly increases in the case of fourth model compared to the third model (Table 10.4). As a result, the stage heights at N79 and G56_H fall by -0.32 feet and -1.24 feet, respectively, and that at G56_T rises by +0.09 feet in the fourth model compared to the third model. These two runs demonstrate that if water is allowed to flow freely over G56, there will be slight rise in water surface elevation downstream of G56, but the backwater effect upstream of G56 will be reduced.

Table 10.3 Effects of Maintained Lowered Gate

Model Scenario	Maximum Stage Heights (feet with respect to the National Geodetic Vertical Datum)						Flow (cubic feet per second) through Critical Conduits					Percent Water Loss via Overtopping
	PS1	N79	HN887	G56_H	G56_T	N110	NXS1/PXS5	NXS16/PXS16	G56US	G56	G56DS	
Base (25-year)	11.9	11.9	12.12	8.647	7.659	1.37	165	1290	5550	5550	5550	3.3
Base (8-foot Gate/ 25-year)	11.93	11.92	12.39	10.228	7.314	1.23	168	1330	5045	5045	5045	4.7

Key :
_H = Headwater.
_T = Tailwater.

Table 10.4. Effects of Artificially Raised Gate

Model Scenario	Maximum Stage Heights (feet with respect to the National Geodetic Vertical Datum)						Flow (cubic feet per second) through Critical Conduits					Percent Water Loss via Overtopping
	PS1	N79	HN887	G56_H	G56_T	N110	NXS1/PXS5	NXS16/PXS16	G56US	G56	G56DS	
Alternative 1 (25-year)	14.98	14.64	14.48	11.238	8.71	1.68	590	1640	6642	6642	6642	0.079
Alternative 1 (12-foot Gate/ 25-year)	14.66	14.32	14.16	10	8.798	1.68	540	1515	6950	6950	6950	0.070

Key :
_H = Headwater.
_T = Tailwater.

The four runs discussed above show that the rise in flow and stage that accompany Alternatives 1 and 2 are mostly due to the effects of channel improvements and creation of an adverse slope in the channel.

10.4 Effect of Storm Intensity/Depth on the Hydraulics of Hillsboro Canal

Three runs—Alternative 1 (25-year), Alternative 1 (10-year), and Alternative 1 (100-year)—show the effects of decreased and increased rainfall events. Compared to the first case (25-year/72-hour rainfall), stage heights in the second case (10-year/24-hour rainfall) decrease by -1.64 feet, -2.54 feet, and -1.09 feet at N79, G56_H, and G56_T, respectively (Table 10.5). But compared to the first case (25-year/72-hour rainfall), stage heights in the third case (100-year/72-hour rainfall) increase by +0.46 feet, +1.68 feet, and +0.02 feet at N79, G56_H, and G56_T, respectively. Thus, the effect of increase in stage heights in the case of a 100-year event when compared to a 25-year event is less dramatic than that from a 10-year event compared to a 25-year event.

Two additional model runs were conducted for the base case for the 10-year/24 hour rainfall and 100-year/72 hour rainfall. The results are also provided in Table 10.5 and are compared to base case for the 25-year/72 hour rainfall.

Table 10.5 Comparisons of Various Storm Events for Alternatives 1 and 2 and Base Model

Model Scenario	Maximum Stage Heights (feet with respect to the National Geodetic Vertical Datum)						Flow (cubic feet per second) through Critical Conduits					Percent Water Loss via Overtopping
	PS1	N79	HN887	G56_H	G56_T	N110	NXS1/ PXS5	NXS16/ PXS16	G56US	G56	G56DS	
Alternative 1 (25-year)	14.98	14.64	14.48	11.238	8.71	1.68	590	1640	6642	6642	6642	0.079
Alternative 1 (10-year)	13.37	13.0	12.83	8.694	7.62	1.03	550	1390	5800	5800	5800	0
Alternative 1 (100-year)	15.46	15.1	15.19	12.916	8.73	2.13	650	1960	6580	6580	6580	4.9
Alternative 2 (25-year)	14.9	14.6	14.45	11.208	8.7	1.68	570	1630	6627	6627	6627	0.079
Alternative 2 (10-year)	13.29	12.97	12.79	8.675	7.6	1.02	526	1370	5770	5770	5770	0
Alternative 2 (100-year)	15.55	15.12	15.19	12.91	8.73	2.13	680	1980	6590	6590	6590	4.9
Base (25-year)	11.9	11.9	12.12	8.647	7.659	1.37	165	1290	5550	5550	5550	3.3
Base (10-year)	12.33	11.95	11.97	8.326	7.35	1.11	500	1485	5300	5300	5300	2.39
Base (100-year)	11.94	11.92	12.77	9.23	8.13	1.85	260	1405	6130	6130	6130	8.45

Key :
_H = Headwater
_T = Tailwater

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11 Summary and Conclusions

The salient features of the models discussed in the previous sections are summarized in Table 11.1. The water surface profiles for a number of these conditions are shown on Figures 11.1 through 11.9. The associated overflows and percent continuity errors in the various model runs are summarized in Table 11.2.

The following conclusions can be made from the figures and tables presented here:

- There is little difference between water levels in Alternatives 1 and 2.

- Alternatives 1 and 2 with the improved cross sections result in significant increases in stage heights over the Base Model in the Hillsboro and L-36 Canals (Figures 11.1 and 11.2). Three factors contribute to the rise in water surface elevations in alternative models compared to the Base Models. These are: 1) increased NSID pumping; 2) increasing top of bank height, deepening and widening of the channel causing less overtopping, increased storage and creation of an adverse slope after a six mile reach of channel improvement, and 3) water control by the G56 structure. Of these, the second factor, namely, the channel modification, is most significant in causing the rise of the water surface elevation under alternative conditions.

- The discharge of NSID without the channel improvements results in a negligible stage increase in the Hillsboro Canal and a increase of approximately 0.5 feet in the L-36 Canal (Figures 11.3 and 11.4). This suggests that the canal discharge may have less impact on stage than the new canal cross sections.

- Figures 5 and 6 further illustrate the effects of the channel modifications. Alternative 1 with the modified cross sections has a considerably higher water level in the Hillsboro Canal and the L-36 Canal when compared to Alternative 1 with the existing cross sections (referred to as “Alt 1 in Base 25 Years”).

Approximately 3.2% more overtopping occurs with the existing cross sections, thus the stages will not get as high.

- During severe storm events under the present schedule of NSID pumping and existing cross sectional geometry, if water is controlled at the G56 gate (i.e., water hits the bottom of the gate and does not open any further), there is a backwater effect (Figures 11.7 and 11.8 lower two curves). This is true in spite of the fact that the model with base condition and lower gate at G56 encountered more overtopping compared to the model with G56 gate fully open (Table 11.2).

- Under the proposed channel modification, there will be significant increase in flow and stage heights, but if water is allowed not to be controlled by the G56 gate then the rise in stage height will be less in the Hillsboro Canal (Figure 11.7 and 11.8 upper two curves). Thus, all Hillsboro Channel hydraulic models must be carefully modeled and code modified according to the actual water control by the G56 structure.

- In spite of several limitations of the present models as elaborated in previous sections, the overall continuity error of the models are well below the acceptable limit of 5% (Table 11.2).

From the studies presented above the following recommendations are made:

- Develop detailed hydrologic models of all of the contributing basins including the water control structure and present calibrated sets of hydrographs as the input to the hydraulic models.

- Modify the codes of the XP-SWMM software to accurately incorporate various equations of discharge under varied flow regimes that can prevail over a gated spillway.

- Obtain a consistent set of cross-sectional data covering sufficient widths of the left and right overbank areas and input those consistent sets into the hydraulic models.

- Evaluate the use of the Bishop Property to store more water (i.e. bermed impoundment with pumped inflows) to offset the impact to the Hillsboro Canal.

- Assess the phosphorus loading impact of infrequent discharges to WCA 2A. Based on current operations, it appears that NSID may only need to discharge to WCA 2A during large storm events. These events are relatively infrequent and a reduced frequency of discharge will result in lower phosphorus loads. If the impacts are minimal to WCA 2A, then the possibility of pumping water from large rain events to WCA 2A could be further pursued. This option would be significantly less expensive than providing storage.

Table 11.1 Comparisons Stage Heights and Peak Flow at Certain Stages and Conduits of Various Scenarios

	Maximum Stage Heights (feet with respect to the National Geodetic Vertical Datum)						Flow (cubic feet per second) through Critical Conduits				
Model Scenario	**PS1**	**N79**	**HN887**	**G56_H**	**G56_T**	**N110**	**NXS1/ PXS5[a]**	**NXS16/ PXS16**	**G56US[b]**	**G56[b]**	**G56DS[b]**
Base (25 year)	11.9	11.9	12.12	8.647	7.659	1.37	165	1290	5550	5550	5550
Alternative 1 (25 year)	14.98	14.64	14.48	11.238	8.71	1.68	590	1640	6642	6642	6642
Alternative 2 (25 year)	14.9	14.6	14.45	11.208	8.7	1.68	570	1630	6627	6627	6627
Alternative 1 (25 year/ Base Cross Section)	12.54	11.98	12.12	8.637	7.65	1.37	515	1418	5550	5550	5550
Alternative 2 (25 year/ Base Cross Section)	12.47	11.97	12.12	8.637	7.65	1.37	494	1388	5555	5555	5555
Base (10 year)	12.33	11.95	11.97	8.326	7.35	1.11	500	1485	5300	5300	5300
Alternative 1 (10 year)	13.37	13.0	12.83	8.694	7.62	1.03	550	1390	5800	5800	5800
Alternative 2 (10 year)	13.29	12.97	12.79	8.675	7.6	1.02	526	1370	5770	5770	5770
Base (100 year)	11.94	11.92	12.77	9.23	8.13	1.85	260	1405	6130	6130	6130
Alternative 1 (100 year)	15.46	15.1	15.19	12.916	8.73	2.13	650	1960	6580	6580	6580
Alternative 2 (100 year)	15.55	15.12	15.19	12.91	8.73	2.13	680	1980	6590	6590	6590
Base (8 foot gate/25-year)	11.93	11.92	12.39	10.228	7.314	1.23	168	1330	5045	5045	5045
Alternative 1 (12 foot Gate/ 25-year)	14.66	14.32	14.16	10	8.798	1.68	540	1515	6950	6950	6950

Notes :
[a] The high flow values observed during the initial stage are discarded.
[b] The sharp instantaneous peaks at the end of orifice flows are discarded.

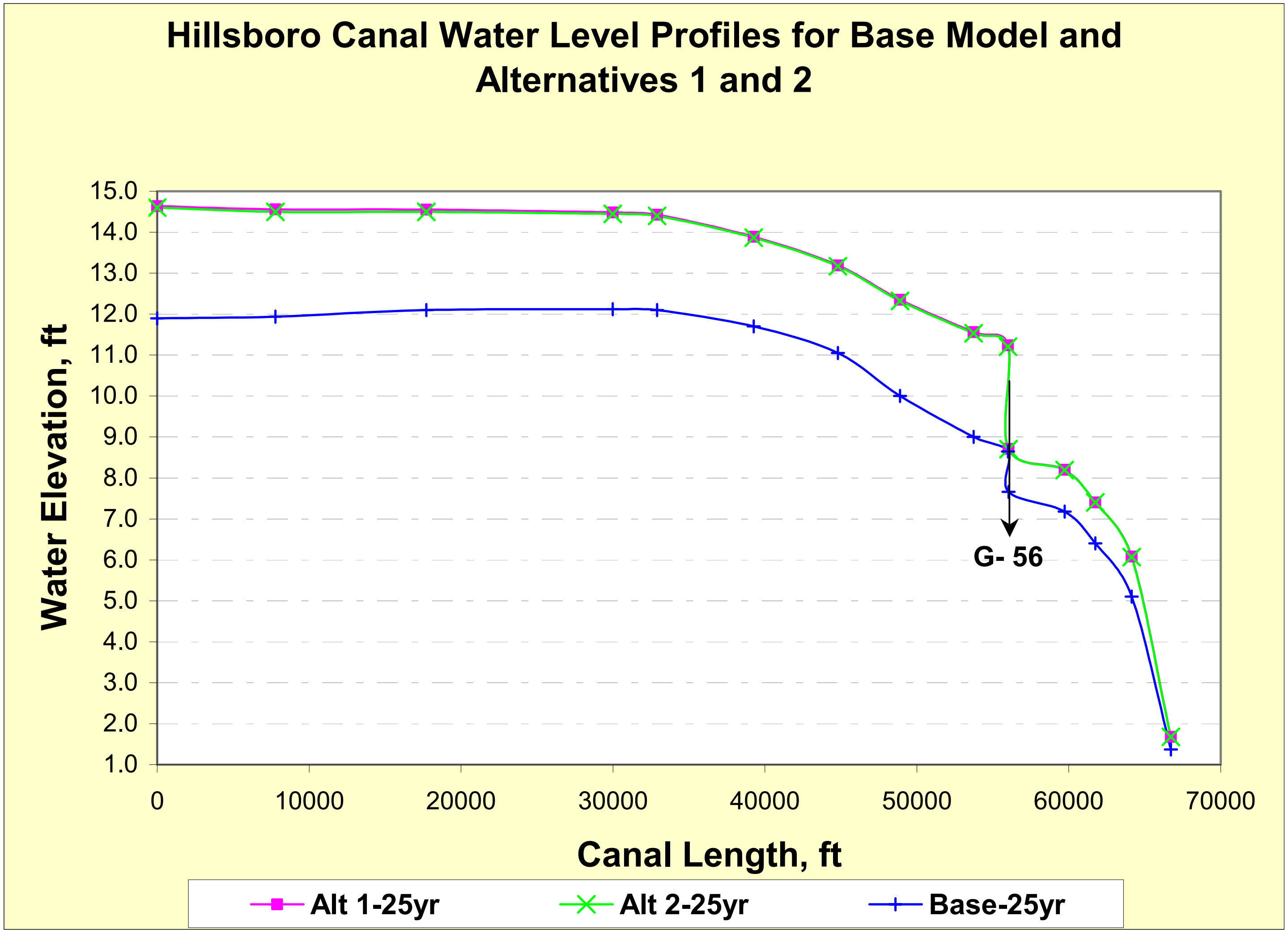

Figure 11.1 Comparison of Hillsboro Canal Water Level Profiles for Base Model and Alternatives 1 and 2

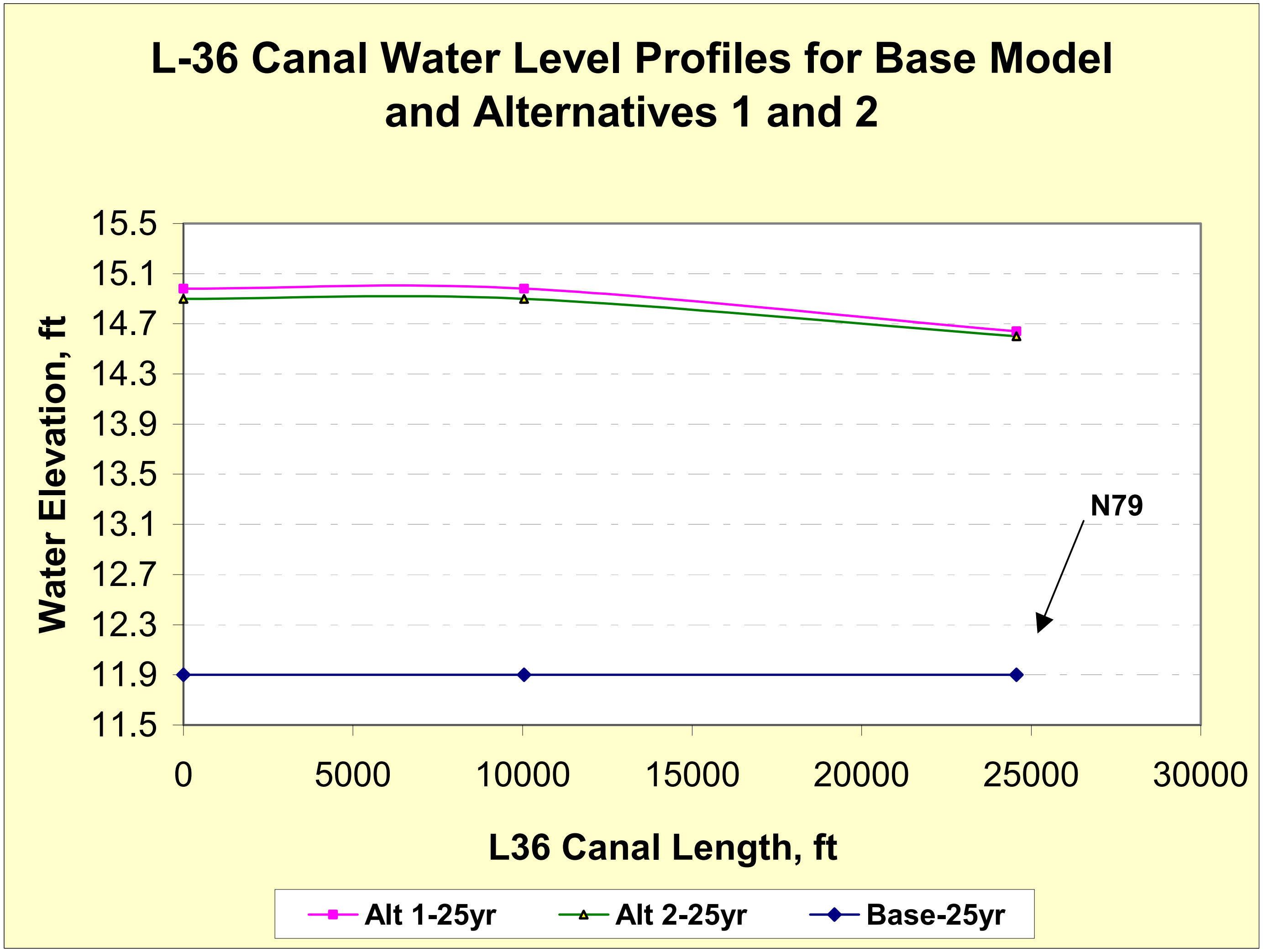

Figure 11.2 Comparison of L-36 Canal Water Level Profiles for Base Model and Alternatives 1 and 2

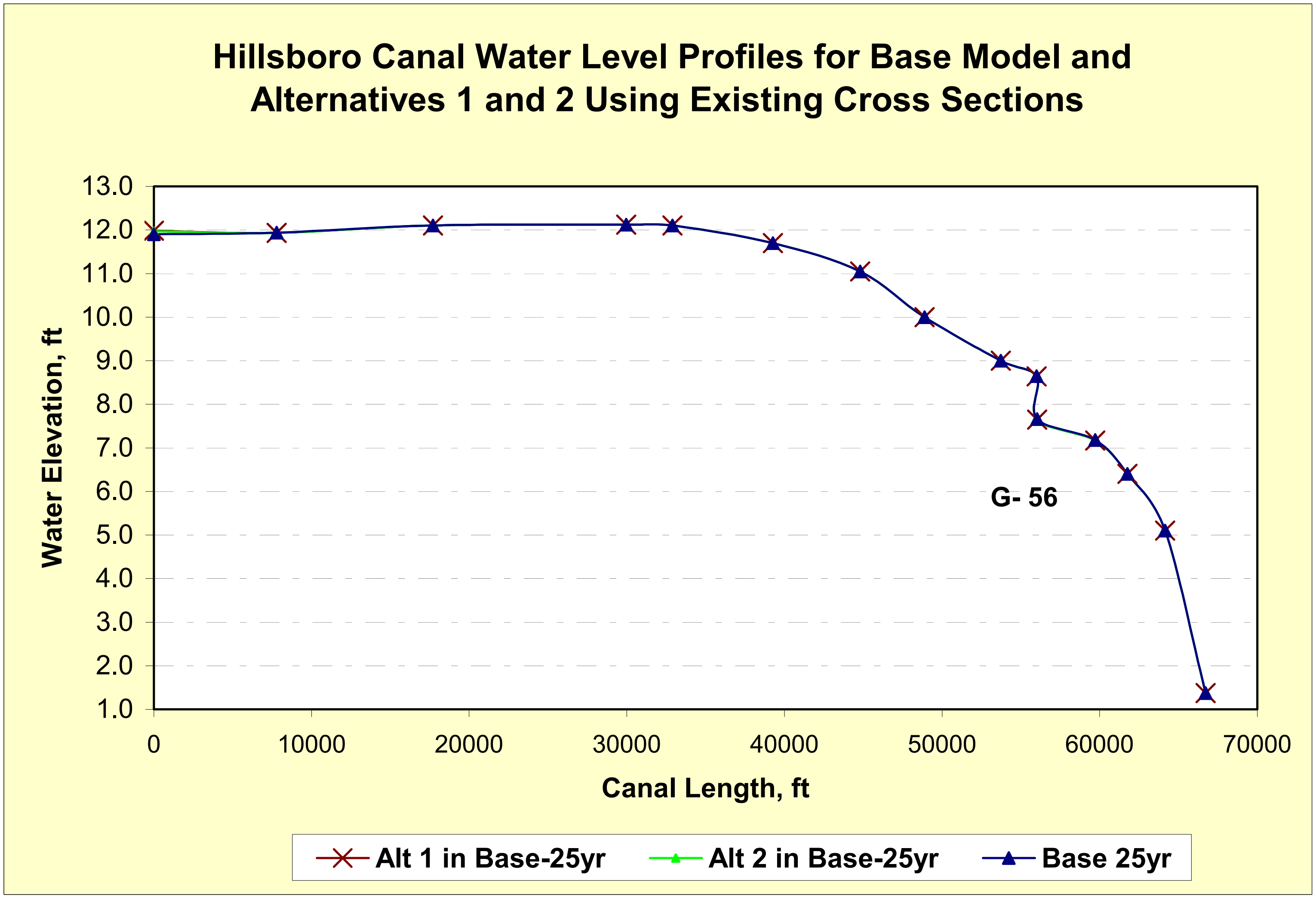

Figure 11.3 Comparison of Hillsboro Canal Water Level Profiles for Base Model and Alternatives 1 and 2 Using Existing Cross Sections

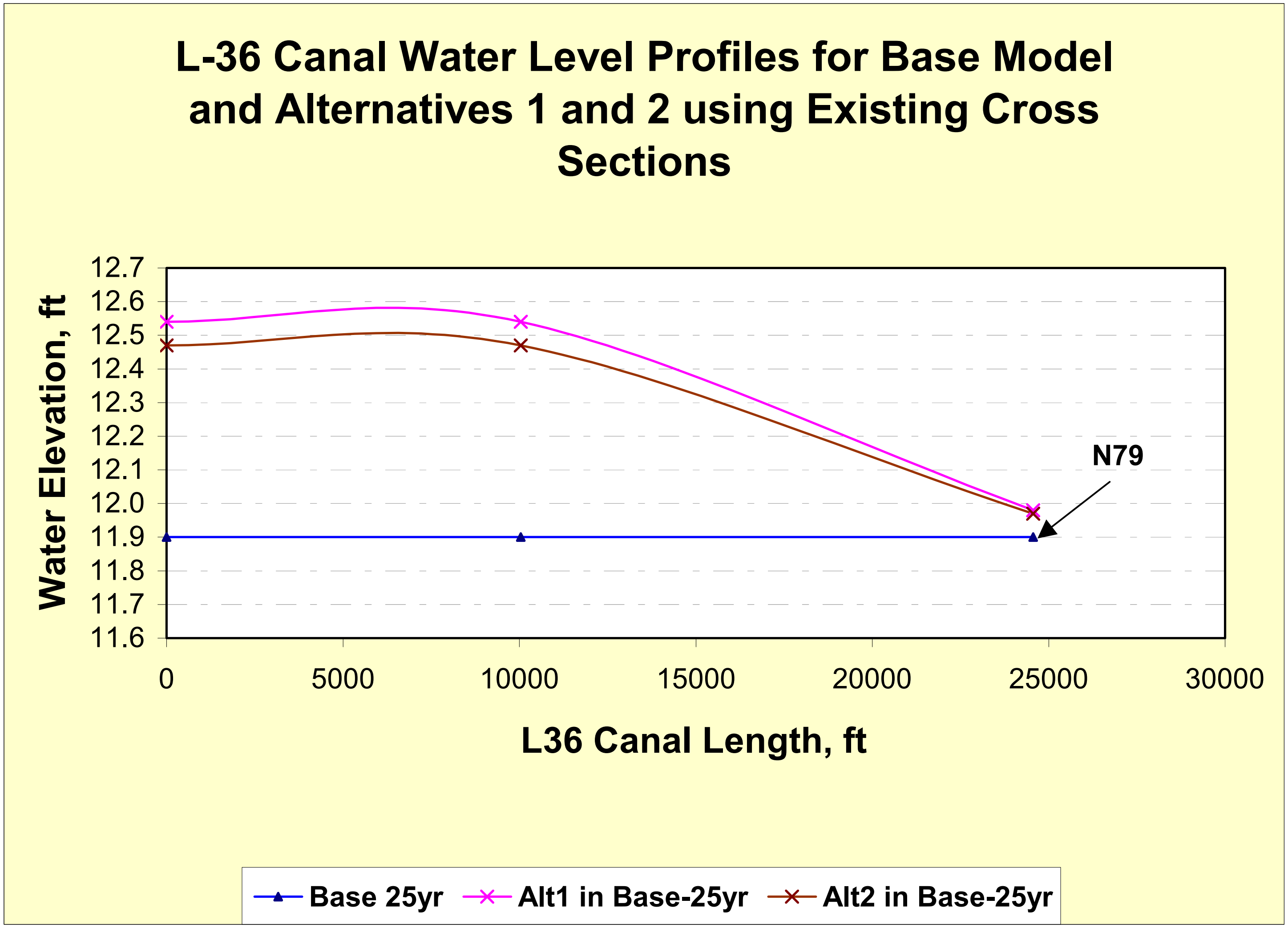

Figure 11.4 **Comparison of L-36 Canal Water Level Profiles for Base Model and Alternatives 1 and 2 Using Existing Cross Sections**

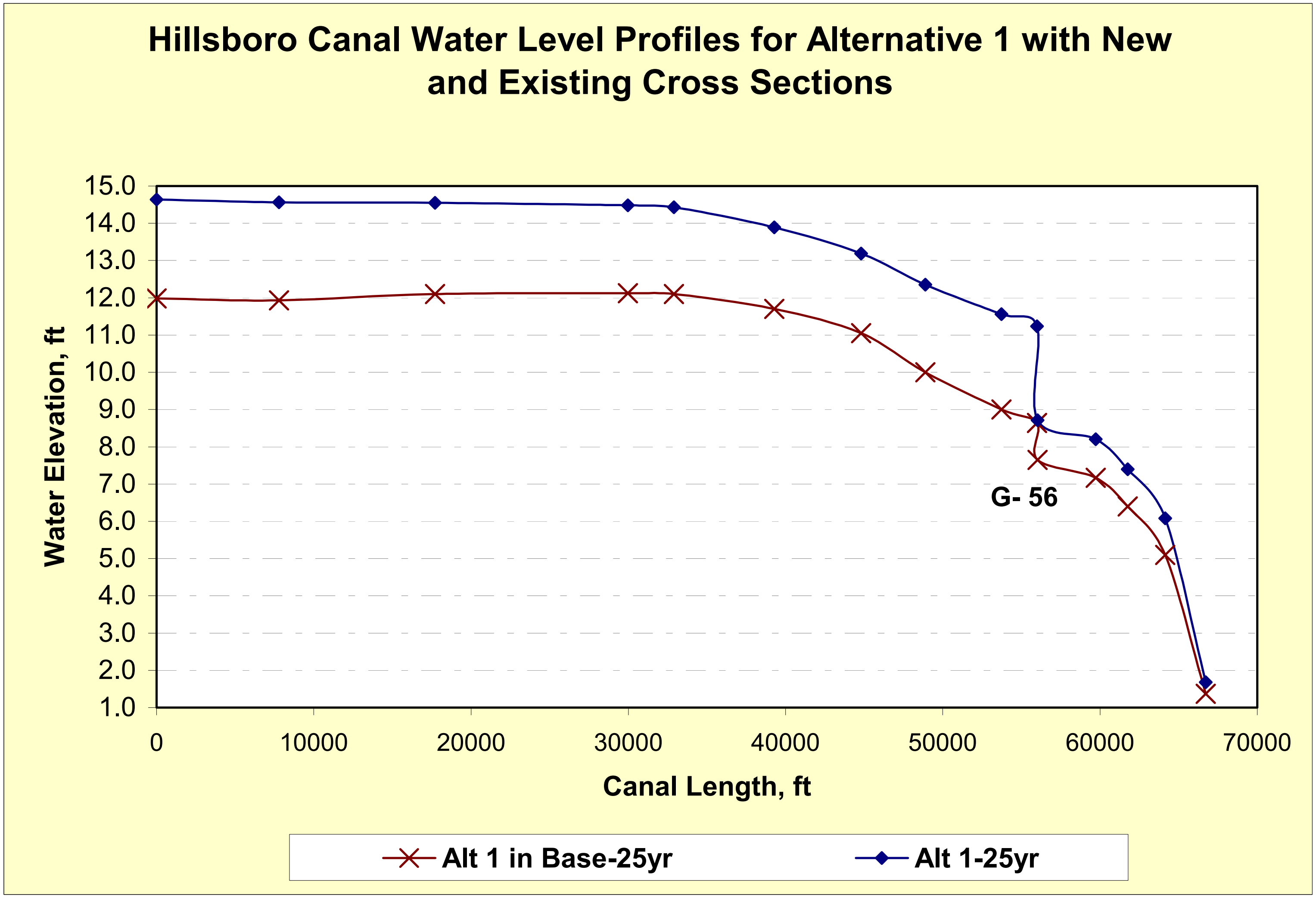

Figure 11.5 Comparison of Hillsboro Canal Water Level Profiles for Alternative 1 with New and Existing Cross Sections

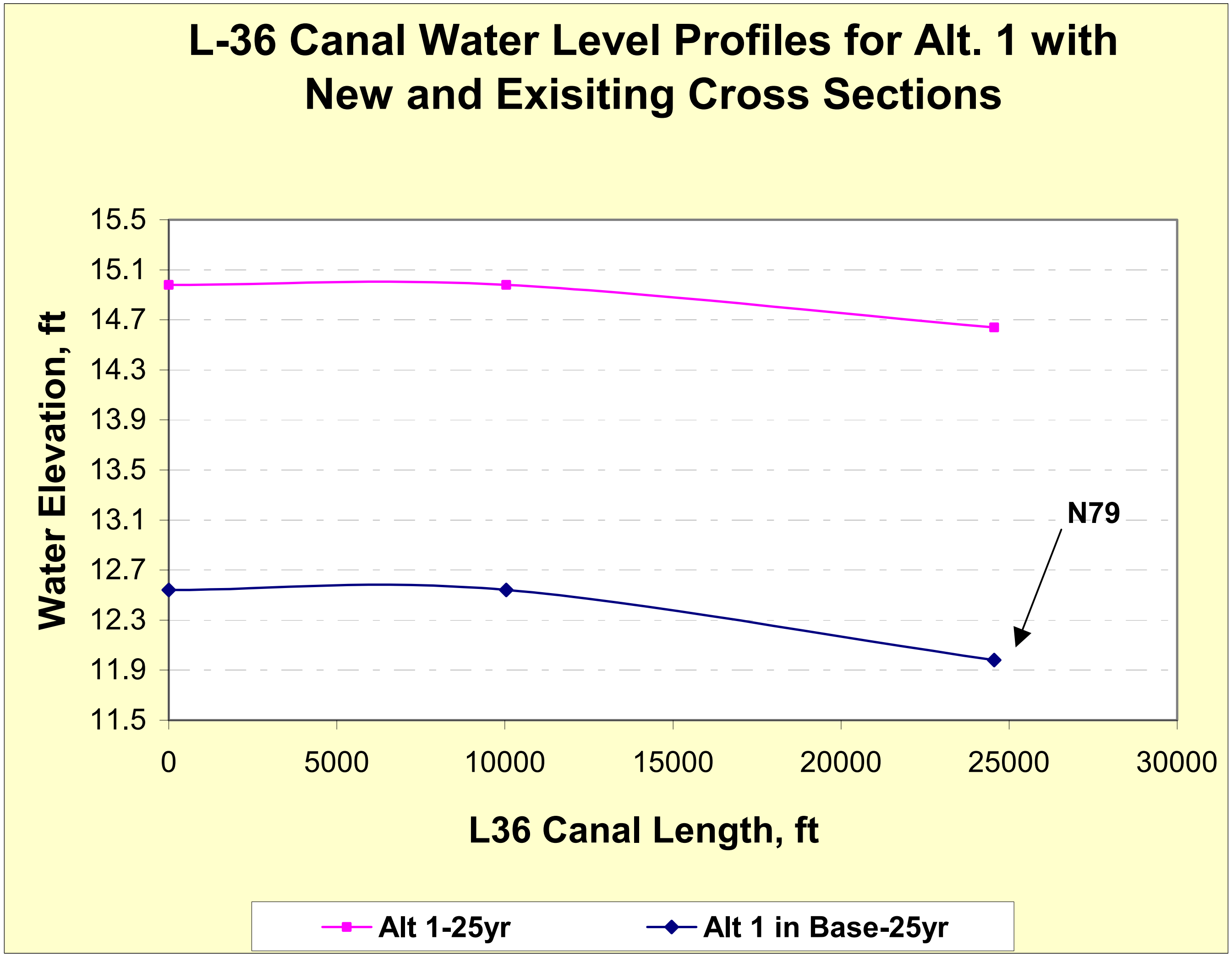

Figure 11.6 Comparison of L-36 Canal Water Level Profiles for Alternative 1 with New and Existing Cross Sections

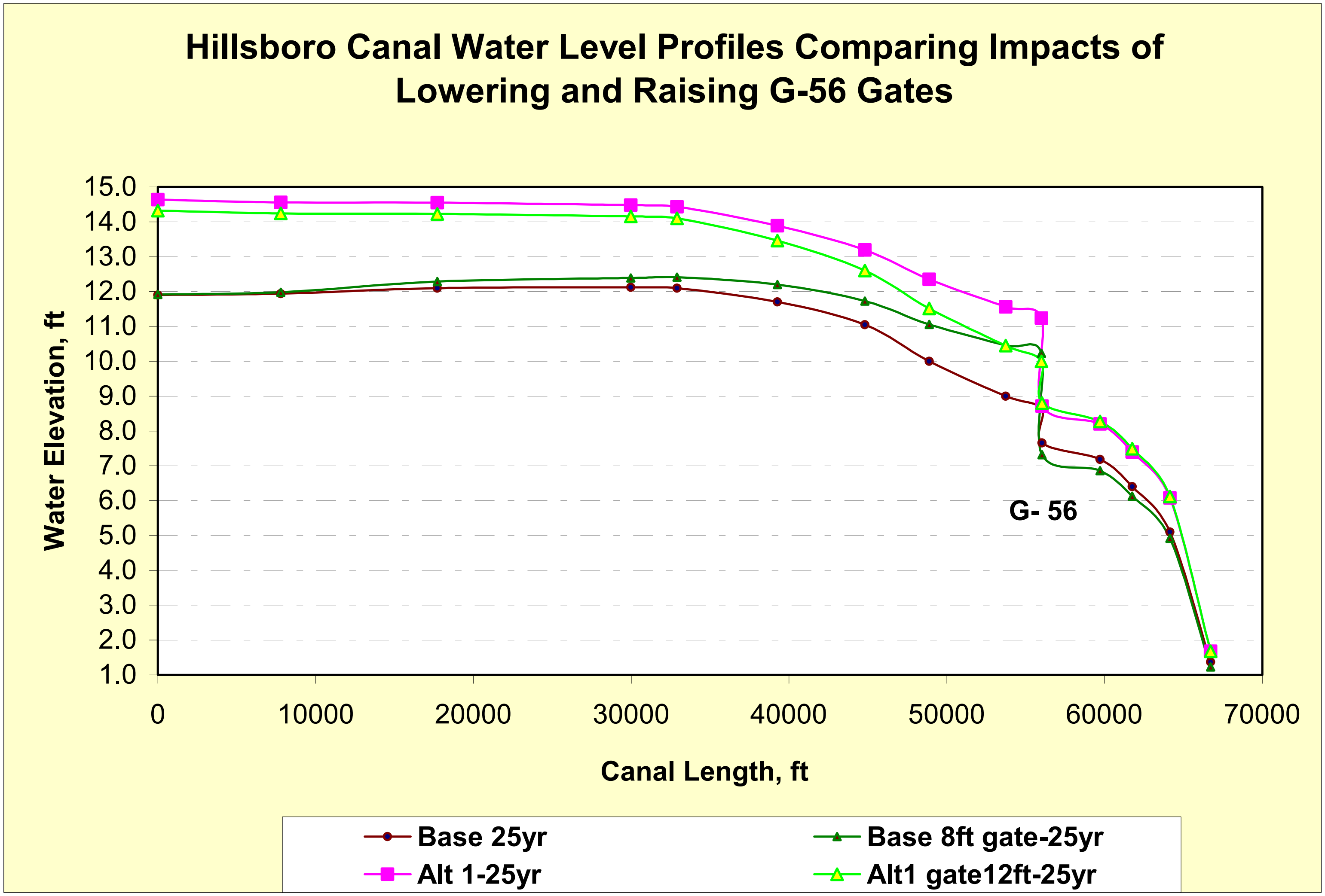

Figure 11.7 **Hillsboro Canal Water Level Profiles Comparing the Base Model to a Condition with a Base Model Lowered Maintained Gate and Comparing Alternative 1 to a Condition with an Alternative 1 Artificially Raised Gate**

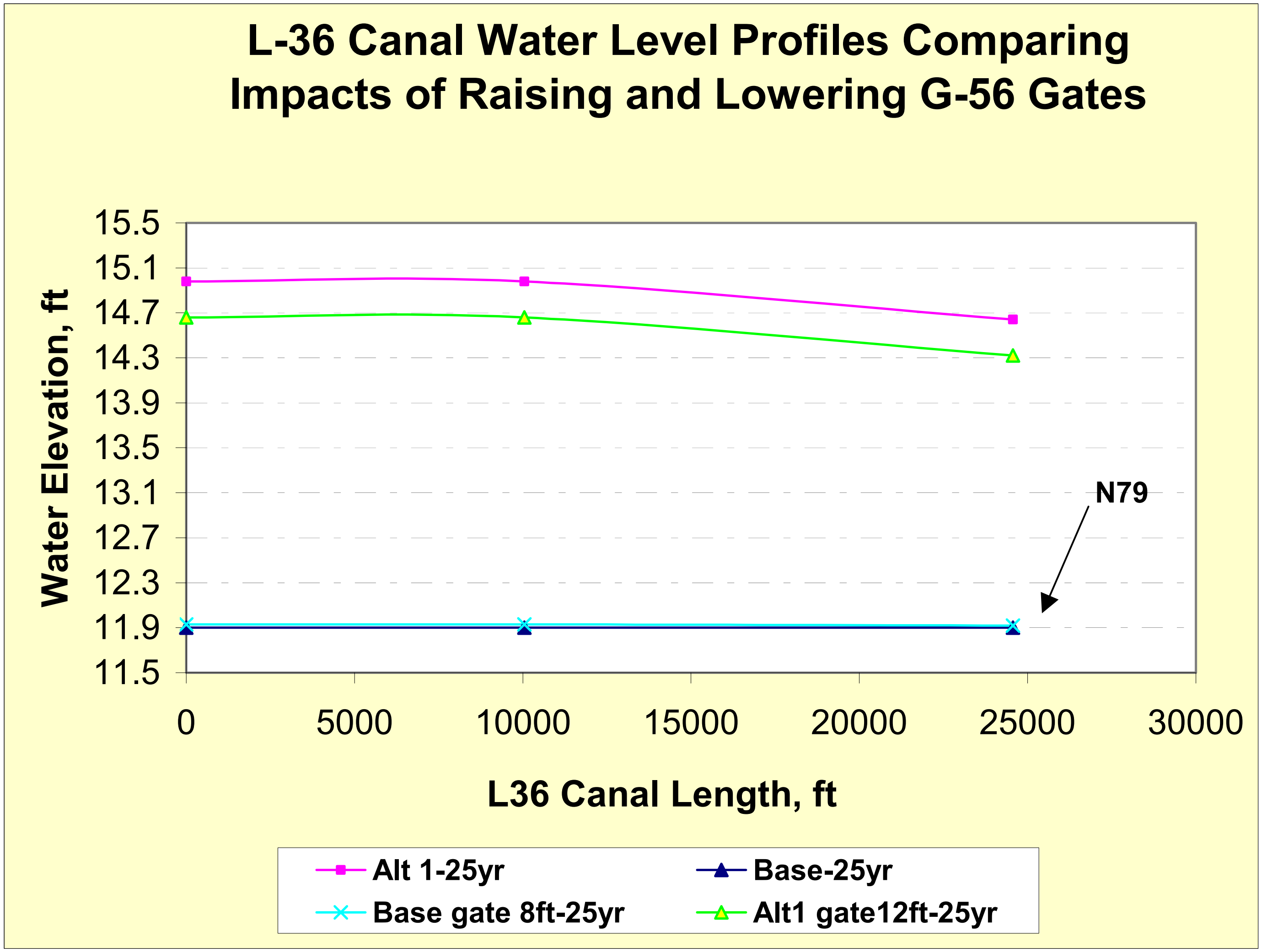

Figure 11.8 **L-36 Canal Water Level Profiles Comparing the Base Model to a Condition with a Base Model Lowered Maintained Gate and Comparing Alternative 1 to a Condition with an Alternative 1 Artificially Raised Gate**

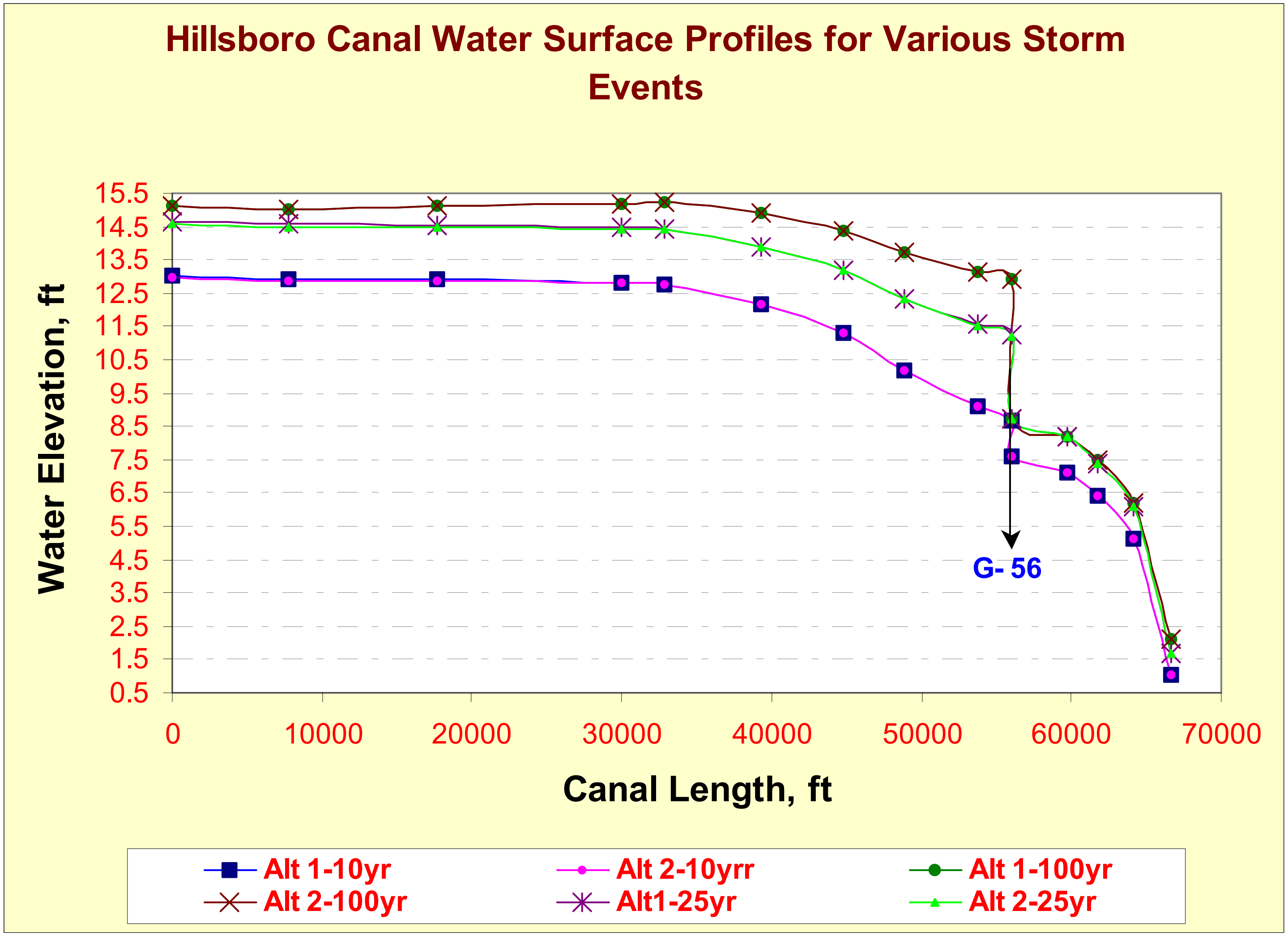

Figure 11.9 Water Surface Profiles for 10-year/24-hour Event, 25-year/72-hour Event, and 100-year/72-hour Event

Table 11.2. Summary of Overtopping and Continuity Calculations in the Model Runs

Model Scenario	Nodes overtopped	Volume Lost (cubic feet)	Total Inflow + Initial Volume (cubic feet)	Percent loss	Percent Error in Overall Continuity
Base (25 year)	BISHOP HNS39_T	72.8E+06 2.13E+06	2.245219E+09	3.3	-0.2452
Alternative 1 (25 year)	HNS39_T	1.82E+06	2.281651E+09	0.079	-0.1396
Alternative 2 (25 year)	HNS39_T	1.83E+06	2.311605E+09	0.079	-0.1333
Alternative 1 (25 year/ Base Cross section)	BISHOP HNS39_T	82.84E+06 1.68E+06	2.282334E+09	3.7	-0.1561
Alternative 2 (25 year/ Base Cross section)	BISHOP HNS39_T	81.41E+06 1.68E+06	2.312287E+09	3.59	-0.1539
Alternative 1 (10 year)	None	None	1.729421E+09	0	-0.3909
Alternative 2 (10 year)	None	None	1.718894E+09	0	-0.4045
Alternative 1 (100 year)	HIS HNS39_T	149.48E+06 1.64E+06	3.062590E+09	4.9	-0.035
Alternative 2 (100 year)	HIS HNS39_T	153.65E+06 1.64E+06	3.116575E+09	4.9	-0.0239
Base (8-foot Gate/ 25-year)	BISHOP HNS39_T	103.85E+06 2.09E+06	2.245219E+09	4.7	-0.2445
Alternative 1 (12-foot Gate/ 25-year)	HNS39_T	1.8E+06	2.281651E+09	0.07	-0.1396

12 References

Ansar, M., and A. Alexis, A., 2003, *Atlas of Flow Computations at District Hydraulic Structures*, Technical Publication EMA Report, Hydrology and Hydraulics Division, South Florida Water Management District, West Palm Beach, Florida.

Burns & McDonnell, March 17, 2003, *Everglades Protection Area Tributary Basins Long Term Plan for Achieving Long-Term Water Quality Goals (Long-Term Plan),* prepared for the South Florida Water Management District, Project No. 32233, Burns & McDonnell, Kansas City, Missouri.

Burns & McDonnell, 1989, G-56 Deerfield Structure on the Hillsboro Canal, Design Memorandum, Volume 1 January 1989.

Florida Atlantic University, June 1999, Hillsboro Canal Cross Section and Longitudinal Profiling Report, Hydrological Modeling Center.

Greenhorne & O'Mara, Inc., May 1991, "Water Management Plan for the Hillsboro Canal Basin, Phase I."

South Florida Water Management District (SFWMD), no date, North Springs Improvement District home page available at http://www.sfwmd.gov/org/reg/esp/norths.html, SFWMD, West Palm Beach, Florida.

United States Bureau of Reclamation (USBR), 1987, *Design of Small Dams*, USBR, Denver, Colorado.

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Appendix A

Analysis of Flow over a Gated Spillway with Special Reference to G56 Structure

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Appendix B

Hillsboro Canal Inflow Hydrology

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Appendix C

Illustrative Cross Sections and Longitudinal Sections for Base and Alternative Cases

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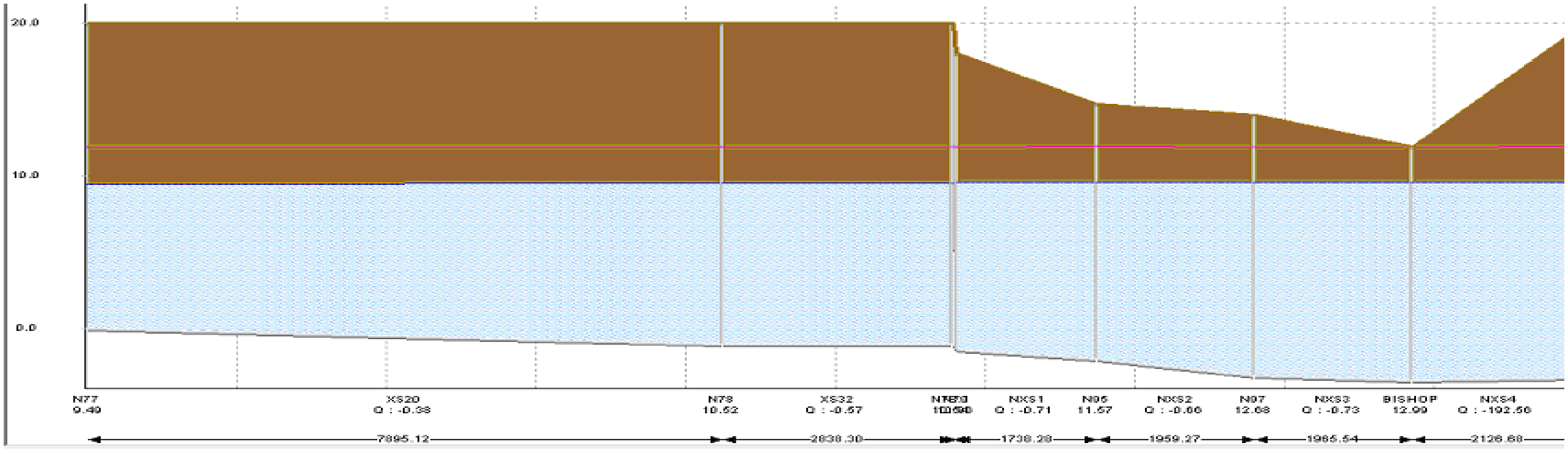

Figure C.1 Base Case (Existing) Longitudinal Section near S39A

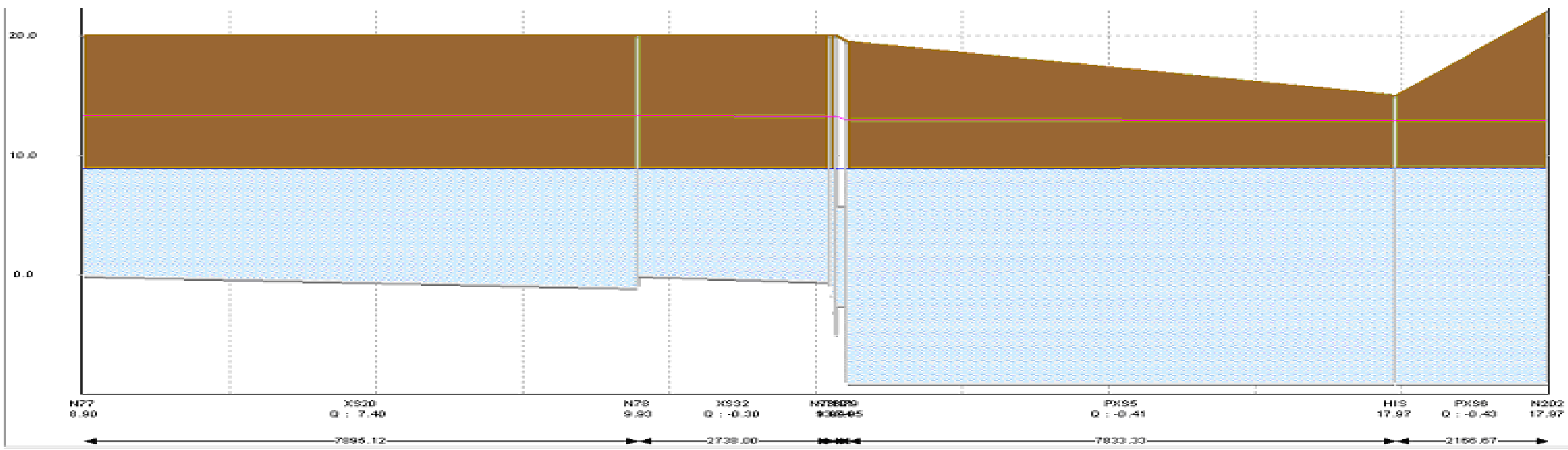

Figure C.2 Proposed Longitudinal Section near S39A

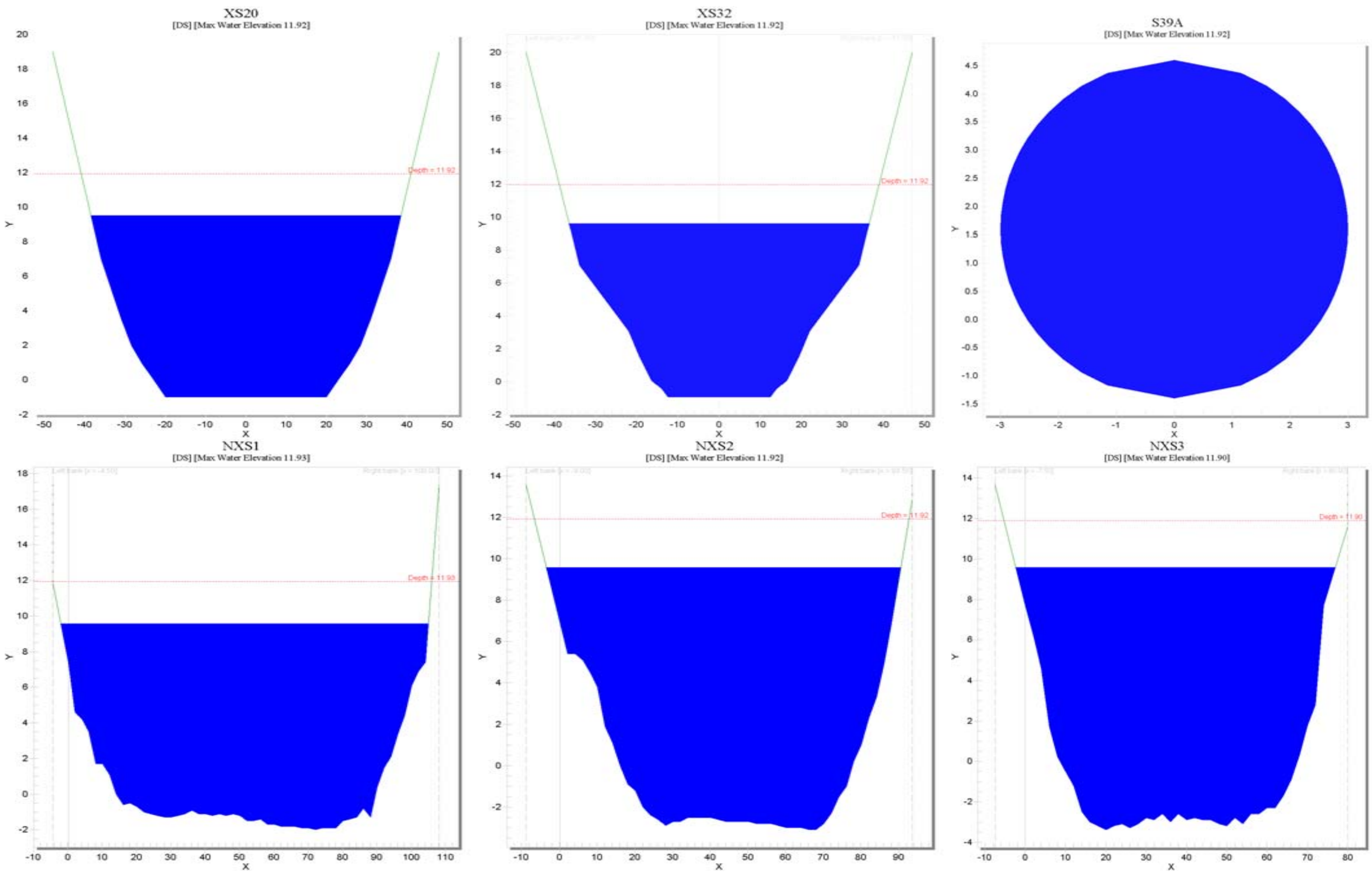

Figure C.3 Base Case Cross Sections

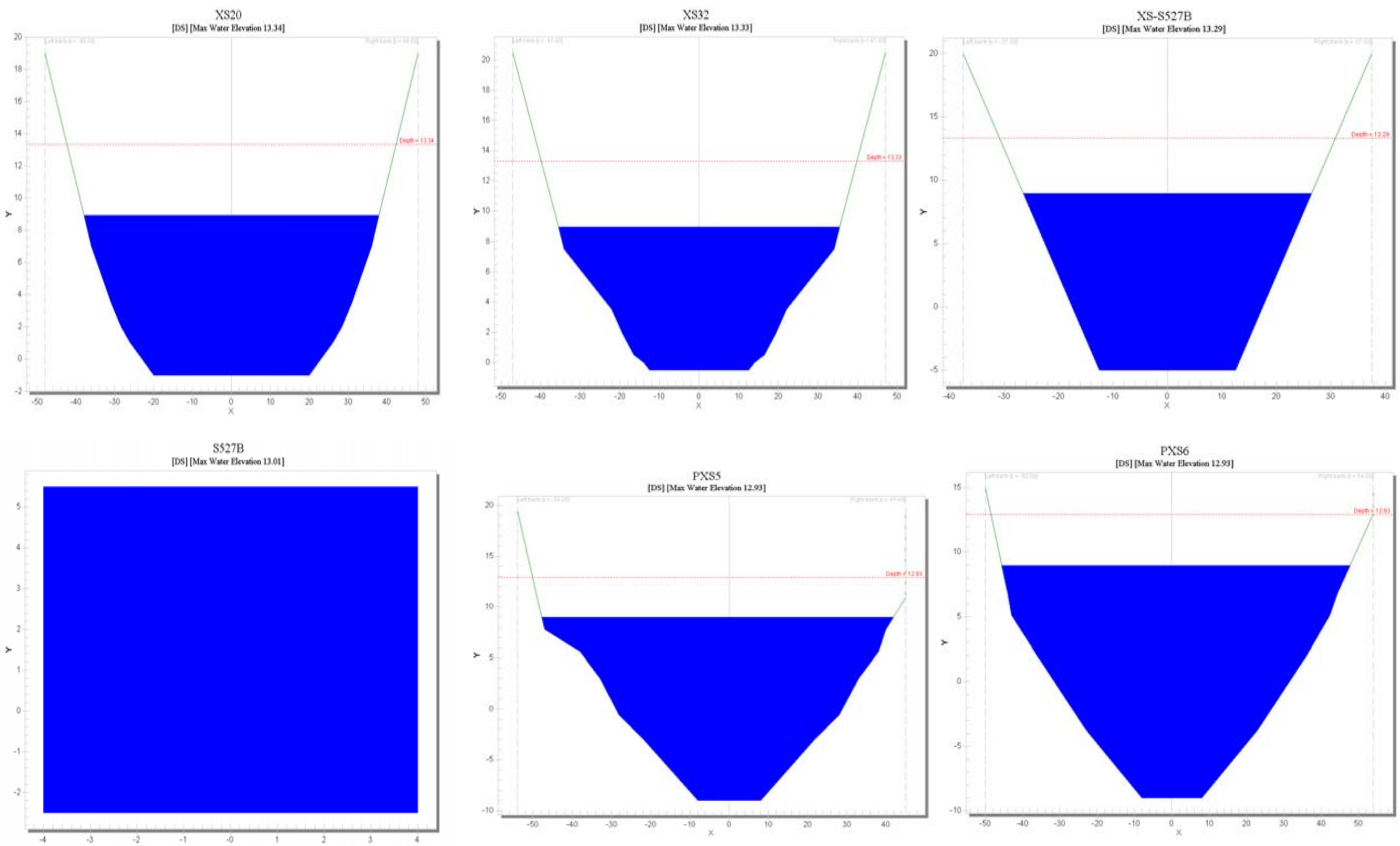

Figure C.4 Channel Modifications: Cross Section Changes (After XS32)

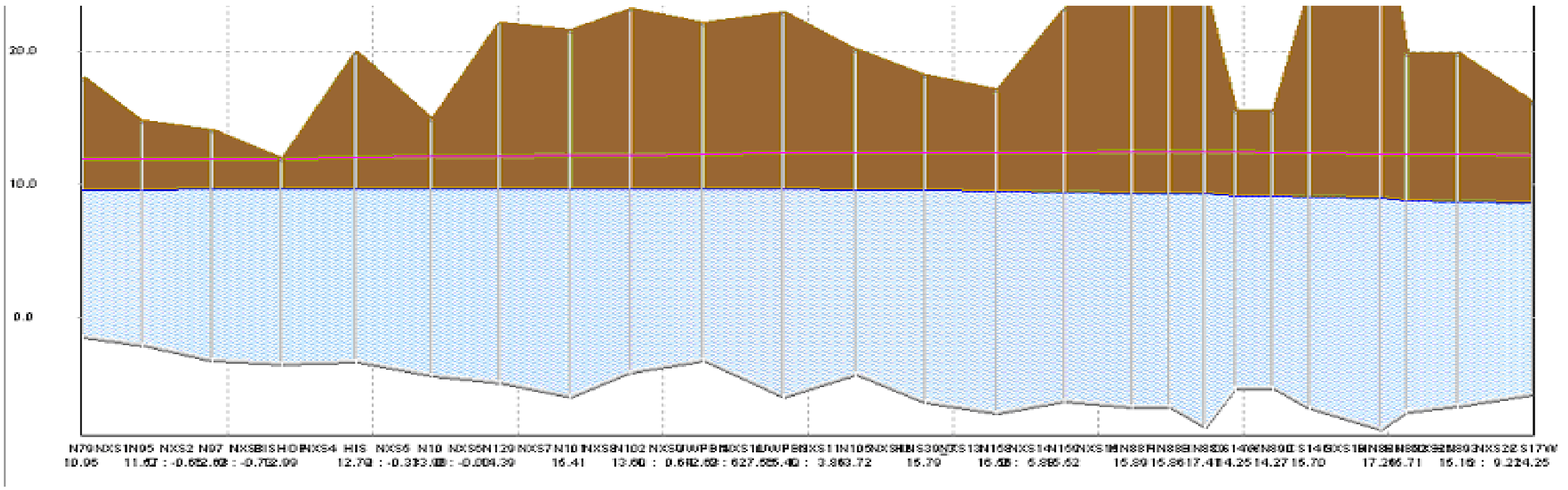

Figure C.5 Existing Longitudinal Section downstream of S39A structure for approximately 6.0 miles length

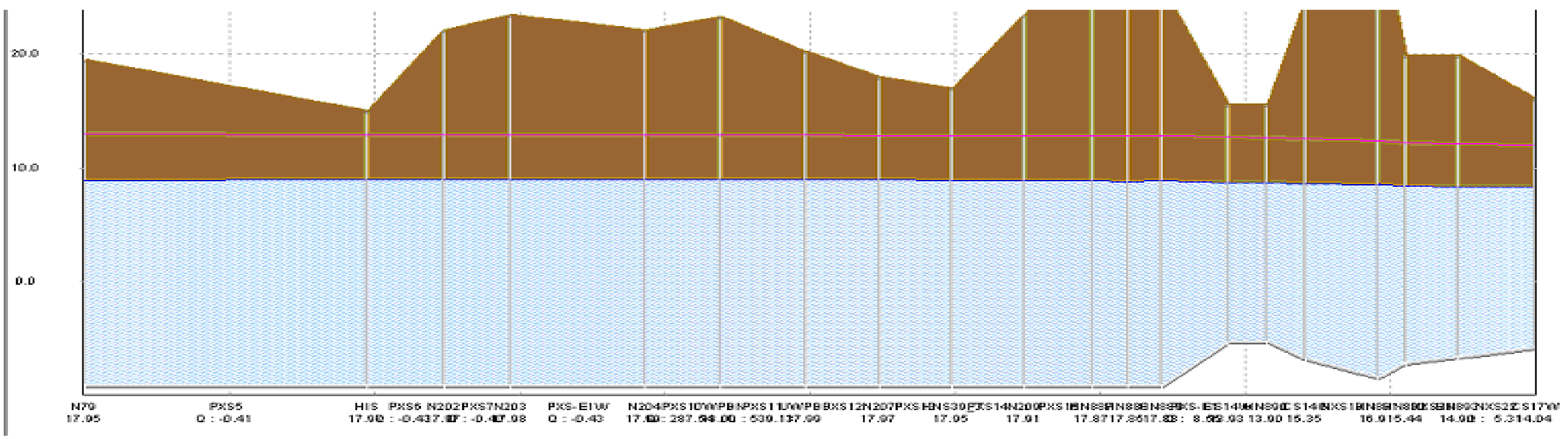

Figure C.6 Proposed Longitudinal Section (–9.0ft NGV) downstream of S39A structure for approximately 6.0 miles length

Appendix D

Rainfall Data for Hurricane Irene

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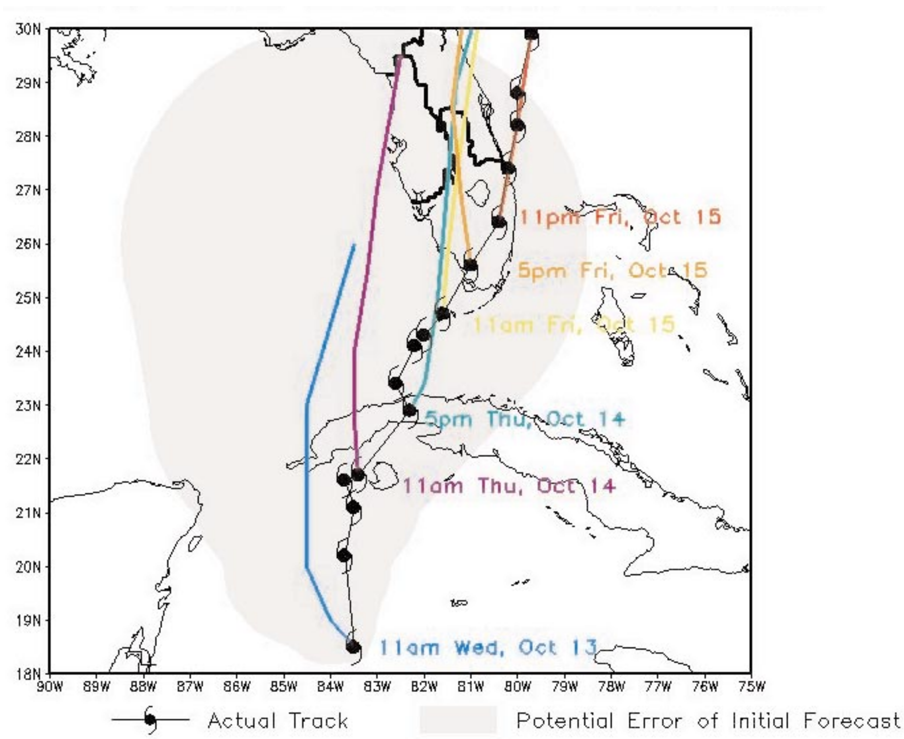
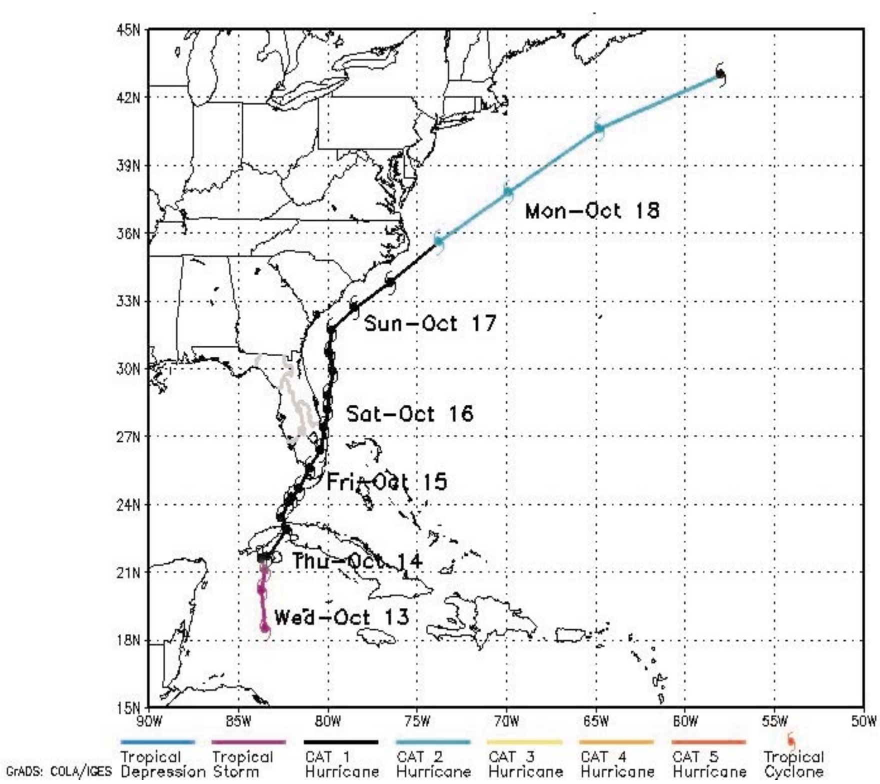
Model Calibration with Hurricane Irene





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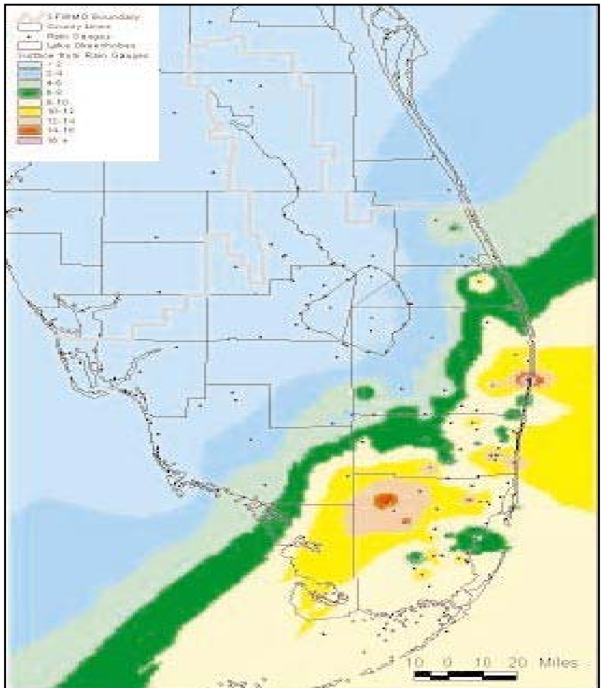
Hurricane Irene:
An Event with 25-Year Return Frequency



Storm Event	Broward Total Rain (in)	LWDD Other Sub- Total Rain (in)
10-Year 1-Day	10.5	9.5
25-Year 3-Day	17.7	13.0
100-Year 3-Day	21.8	18.2

PRELIMINARY RAINFALL TOTALS FROM OCTOBER 14-16, 1999

	INCHES
BROWARD COUNTY	
Pompano (S-37A)	13.20
Fort Lauderdale Field Station	14.08
Coral Springs	11.23
Coopertown	15.17
Hollywood	13.13
West Miramar	13.45
PALM BEACH COUNTY	
Boynton Beach(S-41)	17.47
Delray Beach	2.20
Palm Beach International	10.88
West Palm Beach Field Station	14.50

Rainfall Distribution – October 14-16, 1999

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Appendix E

Model Files and Output Files of Model Runs (CD)

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